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SEISMIC STABILITY EVALUATION OF FOLSOM DAM AND RESERVOIR PROJECT

Report 2
INTERFACE ZONES

by

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The man-made water retaining structures at the Folsom Dam and Reservoir Project, located on the American River about 20 miles upstream of the city of Sacramento, California, have been evaluated for their seismic safety in the event of a Magnitude 6.5 earthquake occurring on the East Branch of the Bear Mountains Fault Zone at a distance of about 15 km. This report documents the study of the interface zones where the Left and Right Wing Dams envelope the Concrete Gravity Dam. The evaluation process involved extensive review of construction records, field and laboratory investigations, analytical studies and an assessment of subsurface conditions. It has been determined that the interface zones are stable in response to design earthquake loads.

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PREFACE

The US Army Engineer Waterways Experiment Station (WES) was authorized to conduct this study by the US Army Engineer District, Sacramento (SPK), by Intra-Army Order for Reimbursable Services Nos. SPKED-F-82-2, SPKED-F-82-11, SPKED-F-82-34, SPKED-F-83-15, SPKED-F-83-17, SPKED-F-84-14, and SPKED-D-85-12. This report is one in a series documenting the seismic stability evaluations of the man-made water retaining structures of the Folsom Dam and Reservoir Project, located on the American River in California. The reports in this series are as follows:

Report 1: Summary
Report 2: Interface Zones
Report 3: Concrete Gravity Dam
Report 4: Mormon Island Auxiliary Dam - Phase I
Report 5: Dike 5
Report 6: Right and Left Wing Dams
Report 7: Upstream Retaining Wall
Report 8: Mormon Island Auxiliary Dam - Phase II

The work on these reports is a joint endeavor between SPK and WES. Messrs. John W. White and John S. Nickell, of Civil Design Section 'A', Civil Design Branch, Engineering Division at SPK were the overall SPK project coordinators. Messrs. Gil Avila and Matthew Allen, of the Soil Design Section, Geotechnical Branch, Engineering Division at SPK, made critical geotechnical contributions to field and laboratory investigations. Support was also provided by the South Pacific Division Laboratory. Personnel of the US Bureau of Reclamation, especially Mr. Steven Herbst, provided flow rate data and onsite assistance during visits by WES personnel. The WES Principal Investigator and Research Team Leader was Dr. Mary Ellen Hynes of the Earthquake Engineering and Geophysics Division (EEGD), Geotechnical Laboratory (GL), WES. The Primary Engineer on the WES team for the portion of the study documented in this report was Mr. David W. Sykora, EEGD, GL. Additional engineering contributions to the study were made by Messrs. Richard H. Ledbetter and Michael K. Sharp (EEGD) and Professor N. Y. Chang, University of Colorado, Boulder. Messrs. William Hanks and Charles Schneider (SMD), Mr. Bennie Washington (EGRMD), and personnel of Information Products Division, Information Technology Laboratory, WES, provided drafting services.

Professors H. Bolton Seed, Anil K. Chopra, and Bruce A. Bolt of the University of California, Berkeley; Professor Clarence R. Allen of the California Institute of Technology; and Professor Ralph B. Peck, Professor Emeritus of the University of Illinois, Urbana, served as Technical Specialists and provided valuable guidance during the course of the investigation.

Overall direction at WES was provided by Dr. A. G. Franklin, Chief, EEGD, and Dr. W. F. Marcuson III, Chief, GL.

COL Dwayne G. Lee, EN, was Commander and Director of WES. Dr. Robert W. Whalin was Technical Director.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	Abbreviation	By	To Obtain
feet	ft	0.3048	metres
inches	in.	2.54	centimeters
inches	in.	25.4	millimeters
miles (US statute)	mi	1.609	kilometers
pounds	lb	4.448	newtons
pounds per square foot	psf	47.880	pascals
tons per square foot	tsf	95.761	kilopascals
gallons per minute	gpm	0.0038	cubic metres per minute

SEISMIC STABILITY EVALUATION OF FOLSOM DAM AND RESERVOIR PROJECT

Report 2: Interface Zone

PART I: INTRODUCTION

General

1. This report is one in a series documenting the investigations and results of a seismic stability evaluation of the man-made water retaining structures at the Folsom Dam and Reservoir Project, located on American River in Sacramento, Placer and El Dorado Counties, California, about 20 air-miles* northeast of the city of Sacramento. A location map and plan of the project area are shown in Figures 1 and 2, respectively. This seismic safety evaluation was performed as a cooperative effort between the US Army Engineer Waterways Experiment Station (WES) and the US Army Engineer District, Sacramento (SPK). Professors H. Bolton Seed, Anil K. Chopra, and Bruce A. Bolt of the University of California, Berkeley, Professor Clarence R. Allen of the California Institute of Technology, and Professor Ralph B. Peck, Professor Emeritus of the University of Illinois, Urbana, served as Technical Specialists for the study.

2. This report summarizes a safety assessment for the seismic performance of interface zones. Interface zones are defined as the fill materials in the general vicinity of contact between the Concrete Gravity Dam (CGD), upstream and downstream retaining walls, and flanking zoned embankment Right and Left Wing Dams (RWD and LWD, respectively). Plan and profile views of the CGD, retaining walls, and embankment fill are shown in Figure 3.

3. Evaluation of seismic safety for interface zones was treated separately from the portions of the RWD and LWD that could reasonably be analyzed with plane-strain approximations described in Report 6 of this series. A feasibility study to assess the usefulness of numerical techniques applied to the problem at the interface zones was conducted. Professor N. Y. Chang on sabbatical from the University of Coloradc was a member of the WES

* A table of factors for converting US customary units of measurement to metric (SI) units is presented on page 4.

feasibility study team. The findings of the feasibility study are described below.

4. The interface zones present a dynamic soil-structure interaction problem involving complex three-dimensional geometries and three different primary media: soil, concrete and rock. To apply the principles of continuum mechanics, equations of motion, constitutive relations, and strain-displacement continuity relations must exist and be satisfied as the mass is subjected to inertial forces with surface tractions or displacements specified along all segments of the boundary.

5. An approximate solution to this problem is difficult to determine. Exact analytical solutions are unavailable. Numerical methods such as the Finite Element Method (FEM) or Boundary Element (Integral) Method (BEM) are available to provide approximate solutions in three-dimensional space but considerable judgement is required in the application of these results because of problem idealization, numerical model limitations and the lack of extensive field verification. Use of these methods is restricted also by the high costs involved in their application. The boundary conditions that exist between soil and jagged rock in the field and the complex non-linear soil behavior of the embankment fill cannot be modeled accurately with existing constitutive models. Most importantly, the results of these methods, applied to a problem such as this one, cannot be verified by field measurements or analytical solutions. Steps toward verification through model studies using a device such as a centrifuge would be very expensive, and would not eliminate the need for judgement in the application of numerical results. Following the feasibility study, WES, SPK, and the Technical Specialists concluded that the results of a three-dimensional numerical analysis would not be fruitful from the standpoint of practical safety decision making.

6. Because of the lack of observations of field performance of interface zones in other projects subjected to earthquakes and the technical difficulties listed above, the course of assessing safety in the RWD and LWD interface zones evolved into a more qualitative approach that consisted of the following steps:

a. Examine interface geometry and fill materials.

b. Anticipate potential modes of failure.

c. Identify key elements that control stability.

d. Study these elements to assess adequacy of performance.

7. The potential modes of failure that required study were narrowed to the three modes listed below:

- a. Cracking and separation of the embankment core and the CGD.
- b. Slope instability resulting in sufficient deformations to allow overtopping, either by sliding of the embankment shell to expose the core to erosion by the reservoir pool, or sliding involving liquefaction of the core materials.
- c. Liquefaction of the core, piping of liquefied core material into downstream shell and progressive development of internal erosion.

8. The key elements that control stability for these potential failure modes were identified as follows:

- a. The presence of defensive design measures built into the project, in particular the large volume of cohesionless fill in shell and filter zones available to fill cracks if they occur.
- b. The liquefaction resistance and post-earthquake strength of shell and transition gravels.
- c. Retaining wall stability and stability of shell backfill if retaining walls slide or overturn.
- d. The liquefaction resistance of core materials and the presence of downstream filter zones to prevent piping and progressive internal erosion.

9. Stability studies indicated that, during and after the earthquake, adequate performance of the interface zones is controlled primarily by adequate performance of the gravel shell, filter and transition zones because of the role these zones play in slope stability, defensive design, and prevention of internal erosion. Detailed studies of the performance of these zones during and after the design earthquake are documented in Report 6 of this series, which examines the seismic stability of the Wing Dams. It was concluded in Report 6 that:

- a. The Wing Dams would perform satisfactorily during the design seismic event.
- b. The embankment shell, filter and transition gravels had more than adequate cyclic strength and would not develop significant residual excess pore water pressures.
- c. The embankment shell, filter and transition materials would have more than adequate post-earthquake strength.
- d. No significant deformations would occur in the Wing Dams as a result of the design seismic event.

10. Another key element in the adequate seismic performance of the interface zones is the sensitivity of slope stability in this zone to movement

or failure of the two downstream retaining walls and Upstream Retaining Wall B which protects the intake ports for the power plant in the RWD wrap-around area. Report 7 of this series documents the study of the retaining walls and the sensitivity of slope stability to retaining wall movement or overturning. It was concluded in Report 7 that:

a. None of the retaining walls will undergo sufficient seismically-induced movement to result in slope instability.

b. Even complete failure and removal of the walls would not result in slope instability of the interface zones sufficient to allow loss of the pool.

11. The remaining key element that controls seismic stability of the interface zones is the potential for liquefaction of the core materials. This report examines in more detail the geometry and materials in the immediate vicinity of the contact between the Wing Dams and the CGD, the procedures used during construction in this area, and the results of field and laboratory investigations performed during construction and more recently as part of this study, with a view toward closer examination of liquefaction potential of the core materials in this zone. On the basis of these studies and those documented in Reports 6 and 7 of this series, it was concluded that the interface zones will perform satisfactorily during and immediately after the design earthquake and no remedial action of any kind is indicated for this project feature from a seismic stability viewpoint.

Project History

12. The Folsom project was designed and built by the Corps of Engineers in the period 1948 to 1956, as authorized by the Flood Control Act of 1944 and the American River Basin Development Act of 1949. Upon completion of the project in May 1956, ownership of the Folsom Dam and Reservoir was transferred to the US Bureau of Reclamation for operation and maintenance. As an integral part of the Central Valley Project, the Folsom Project supplies water for irrigation, domestic, municipal, industrial and power production purposes. It also provides flood protection for Sacramento and the surrounding area and extensive water-related recreational facilities. Releases from the Folsom Reservoir are also used to provide water quality control for project diversions from the Sacramento-San Joaquin Delta, to maintain fish runs in the

American River below the dam, and to help maintain navigation along the lower reaches of the Sacramento River.

Hydrology and Pool Levels

13. Folsom Lake impounds the runoff from 1,875 square miles of mountainous terrain. The reservoir has a storage capacity of one million acre-ft at gross pool and is contained by approximately 4.8 miles of man-made, water-retaining structures that have a crest elevation of 480.5 ft above mean sea level. These structures are the RWD and LWD, the CGD, Mormon Island Auxiliary Dam, and 8 Saddle Dikes. At gross pool, elevation 466 ft, there are 14.5 ft of freeboard. This pool level was selected for the safety evaluation on the basis of a review of current operational procedures and hydrologic records (obtained for a 29-year period, from 1956 to 1984) for the reservoir which shows that the pool typically reaches elevation 466 ft about 10 percent of the time during the month of June, and considerably less than 10 percent of the time during the other months of the year. Under normal operating conditions, the pool is not allowed to exceed elevation 466 ft. Hydrologic records show that emergency situations which would cause the pool to exceed elevation 466 ft are rare events.

Site Geology

14. At the time of construction, the geology and engineering geology concerns at the site were carefully detailed in the foundation reports by US Army Engineer District, Sacramento (1953). These foundation reports from construction records and a later paper by Kiersch and Treasher (1955) are the sources for the summary of site geology provided in this section. Figure 4 shows a geologic map of the project area.

15. The Folsom Dam and Reservoir Project is located in the low, westernmost foothills of the Sierra Nevada in central California, at the confluence of the North and South Forks of the American River. Relief ranges from a maximum elevation of 1,242 ft near Flagstaff Hill located between the upper arms of the reservoir, to 150 ft near the town of Folsom just downstream of the Concrete Gravity Dam. The North and South Forks once entered the confluence in mature valleys up to 3 miles wide, but further downcutting resulted in a V-shaped inner valley 30 to 185 ft deep. Below the confluence, the inner

canyon was flanked by a gently sloping mature valley approximately 1.5 miles wide bounded on the west and southeast by a series of low hills. The upper arms of the reservoir, the North and South Forks, are bounded on the north and east by low foothills.

16. A late Pliocene-Pleistocene course of the American River flowed through the Blue Ravine and joined the present American River channel downstream of the town of Folsom. The Blue Ravine was filled with late Pliocene-Pleistocene gravels, but with subsequent downcutting and headward erosion, the Blue Ravine was eventually isolated and drainage was diverted to the present American River Channel.

17. The important formations at the dam site are: a quartz diorite granite which underlies the CGD, RWD, and LWD and Saddle Dikes 1 through 7; metamorphic rocks of the Amador group which underlie Saddle Dike 8 and the foundation at Mormon Island Auxiliary Dam; the Mehrten formation, a deposit of cobbles and gravels in a somewhat cemented clay matrix which caps the low hills that separate the saddle dikes and is part of the foundation at Dike 5; and the alluvium that fills the Blue Ravine at Mormon Island Auxiliary Dam. Bedrock geology is indicated on the plan in Figures 2 and 4.

18. Material for the impervious core of the RWD and LWD was obtained from the residual soil stratum derived from quartz diorite. This saprolitic material typically classifies as a silty to clayey sand according to the Unified Soils Classification System (USCS). In general, this material is considered to be resistant to liquefaction because of its origin, high percentage of fines (material passing the No. 200 sieve), presence of clay fines, angularity of its grains, and well-graded particle distribution. Sowers (1979) indicates that residuum of granite generally is a silty sand to sandy silt containing varying amounts of kaolinite and typically is good construction material.

Seismic Hazard Assessment

Seismological and geological investigations

19. Detailed geological and seismological investigations in the immediate vicinity of Folsom Reservoir were performed by Tierra Engineering, Incorporated to assess the potential for earthquakes in the vicinity, to estimate the magnitudes these earthquakes might have, and to assess the potential for

ground rupture at any of the water-retaining structures (see Tierra Engineering, Inc. 1983 for a comprehensive report). A 12-mile wide by 35-mile long study area centered on the Folsom Reservoir was investigated extensively using techniques such as aerial imagery analysis, ground reconnaissance, geologic mapping, and detailed fault capability assessment. In addition, studies by others relevant to the geology and seismicity of the area around Folsom were also compiled. These additional literature sources include numerous geological and seismological studies published through the years, beginning with the "Gold Folios" published by the US Geological Survey in the 1890's, the engineering geology investigations for New Melones and the proposed Marysville and Auburn Dams, studies performed for the Rancho Seco Nuclear Power Plant as well as unpublished graduate student theses and county planning studies.

20. It was determined that no capable faults underlie any of the water-retaining structures of the main body of the reservoir at the Folsom Project. The tectonic and seismicity studies also indicate it is unlikely that Folsom Lake can induce major seismicity. Since the faults that underlie the water retaining structures at the Folsom Project were found to be noncapable, seismic fault displacement in the foundations of the water retaining structures is judged to be highly unlikely.

21. The closest capable fault is the East Branch of the Bear Mountains fault Zone which has been found to be capable of generating a maximum magnitude $M = 6.5$ earthquake. The return period for this maximum earthquake is estimated to exceed 400 years (Tierra Engineering, Inc. 1983). Determination that the East Branch of the Bear Mountains fault zone is a capable fault came from earthquake evaluation studies conducted for nearby Auburn Dam. The minimum distance between the East Branch of the Bear Mountains fault zone and Mormon Island Auxiliary Dam is 8 miles, and the minimum distance between this fault zone and the CGD is 9.5 miles. The focal depth of the earthquake is estimated to be 6 miles. This hypothetical maximum magnitude earthquake would cause more severe shaking at the project than earthquakes originating from other known potential sources.

Selection of design ground motions

22. Professors Bruce A. Bolt and H. B. Seed used the results of the seismological and geological study to determine appropriate ground motions for the seismic safety evaluation of the Folsom Dam Project. This fault zone has an extensional tectonic setting and a seismic source mechanism that is normal

dip-slip. The slip rate from historic geomorphic and geological evidence is very small, less than 10^{-3} centimeters per year with the most recent known displacement occurring between 10,000 and 500,000 years ago in the Pleistocene Epoch.

23. Bolt and Seed (1983) recommend the following design ground motions on the basis of their studies of the horizontal ground accelerations recorded on an array of accelerometers normal to the Imperial Valley fault during the Imperial Valley earthquake of 1979, as well as recent studies of a large body of additional strong ground motion recordings:

Peak horizontal ground acceleration	= 0.35 g
Peak horizontal ground velocity	= 20 cm/sec
Bracketed Duration (≥ 0.05 g)	≅ 16 sec

It is expected that the earthquake accelerations might be relatively rich in high frequencies due to the presence of granitic plutons at the site.

24. Bolt and Seed (1983) provided 2 accelerograms that are representative of the design ground motions expected at the site as a result of a maximum magnitude M = 6.5 earthquake occurring on the East Branch of the Bear Mountains fault zone. The accelerograms are designated as follows:

M6.5 - 15K - 83A. This accelerogram is representative of the 84-percentile level of ground motions that could be expected to occur at a rock outcrop as a result of a Magnitude 6-1/2 earthquake occurring 15 km from the site. It has the following characteristics:

Peak acceleration	= 0.35 g
Peak velocity	≅ 25 cm/sec
Duration	≅ 16 sec

M6.5 - 15K - 83B. This accelerogram is also representative of the 84-percentile level of ground motions that could be expected to occur at a rock outcrop as a result of a Magnitude 6-1/2 earthquake occurring 15 km from the site. It has the following characteristics:

Peak acceleration = 0.35 g

Peak velocity ≅ 19.5 cm/sec

Duration ≅ 15 sec

Plots of acceleration as a function of time and response spectra for two design accelerograms are shown in Figures 5 and 6, respectively.

Report Presentation

25. This report begins with a review of construction records to achieve an understanding of the complex geometry of the interface zone and the particular construction practices used in this area. The review follows a chronological progression from initial foundation excavation to compaction of embankment fill in the interface zones near completion of this project. The section following the review of construction records presents and examines the results of field and laboratory tests performed during construction and, more recently, as part of this seismic stability study. The information in these two sections and from Reports 6 and 7 of this series is then used to evaluate the seismic stability of interface zones in the next section. Conclusions drawn from the results of this study are provided in the last section. The terms interface zone, wraparound areas and envelopment areas are used interchangably throughout this report to refer to the materials in the general vicinity of the contact between the ends of the CGD and the embankment Wing Dams that envelop them.

PART II: REVIEW OF CONSTRUCTION RECORDS

General

26. Seed, Makdisi and De Alba (1978) reviewed observed performance of earth dams during earthquakes and concluded that:

- a. "Virtually any well built dam can withstand moderate earthquake shaking, say with peak accelerations of about 0.2 g and more, with no detrimental effects."
- b. "Dams constructed of clay soils on clay or rock foundations have withstood extremely strong shaking ranging from 0.35 g to 0.8 g from a magnitude 8.25 earthquake with no apparent damage."
- c. "The fact that a number of dams have failed in periods up to 24 hr after an earthquake suggests that piping through cracks resulting from earthquake shaking may well have been responsible for the failure. This fact reemphasizes the need to provide an adequate system of filter materials in constructing dams in seismic regions to ensure that progressive erosion through continuous cracks cannot occur."

These conclusions were used in the review of construction records to focus attention on aspects of design and construction that control seismic stability.

27. A comprehensive review of construction records for the Folsom Project was undertaken to achieve an understanding of the complex geometry of the interface zone, the fill materials located there, the methods of placement and compaction of these materials, and particular construction practices used in this area. The purposes of this study were:

- a. To examine the care with which core and other embankment materials were placed and compacted since quality construction is an indicator of good seismic performance, as stated above.
- b. To examine the fine-grained fraction of the core materials in the interface zone since clayey materials perform well during extremely severe seismic events (addressed in more detail in Part III).
- c. To observe the embankment zone design to demonstrate that large, cohesionless filter and transition zones exist upstream and downstream of the core to prevent progressive piping of core materials if cracks were to develop as a result of earthquake shaking.
- d. To examine the geometry of rock and concrete boundaries at the bases of the concrete monoliths, since the presence of such rigid boundaries inhibits the development of cyclic shear strains necessary to cause the development of high, seismically-induced residual excess pore water pressures.

e. To assist in the planning of field investigations described in Part III of this report.

28. Records of design, site conditions, and various aspects of construction of the Folsom Dam and Reservoir Project were available for the review: foundation reports, specification documents, engineering daily logs, design memoranda, design drawings, and approximately 2,500 photographs were examined. In addition, Mr. John Ott, who was Materials Engineer in charge of the Folsom Dam Project Laboratory during construction, was interviewed as part of this study. A summary of his comments is contained in Appendix A.

29. Pertinent information for the above sources is summarized in this chapter. First, the embankment design, materials and foundation geology are described. Then, the construction sequence is stepped through with the assistance of construction photographs and applicable excerpts from construction specifications. Finally, conclusions are drawn from this information.

Description of Wing Dams, Interface Zone, Retaining Walls and Concrete Gravity Dam

Right and Left Wing Dams

30. The Wing Dams are zoned embankment dams founded on weathered quartz diorite granite. Plans of the Wing Dams are shown in Figures 2 and 7. The Right Wing Dam has a crest length of approximately 6,700 ft, and has a maximum height of approximately 195 ft. The core consists of well-compacted decomposed granite and suitable fine-grained materials from the American River channel. Gravels excavated from the American River channel were used as upstream and downstream transition zones. An uncompacted rock-fill shell was constructed on the upstream and downstream slopes over most of the length of the dam. The upstream slopes are 2.25 horizontal to 1 vertical, and the downstream slopes are 2 horizontal to 1 vertical. Typical sections are shown in Figure 8.

31. The Left Wing Dam is approximately 2,100 ft long and 167 ft high. The core consists of well compacted decomposed granite and is flanked upstream and downstream by 12-ft wide filters. The upstream and downstream shells are constructed of gravels, which come from dredged tailings in the Blue Ravine. The filters are the -2-in. fraction of the Blue Ravine gravels. The slopes are the same as the Right Wing Dam. A plan of the Left Wing Dam is shown in Figure 2 and typical sections are shown in Figure 9.

Concrete Gravity Dam

32. The Right and Left Wing Dams flank the Concrete Gravity Dam. The Concrete Gravity Dam consists of twenty eight 50-ft-wide monoliths founded on hard granodiorite rock. The overall length of the concrete structure is 1,400 ft, the maximum height is 340 ft measured from the foundation to the crown of the roadway, elevation 480.5 ft (3.5 ft below the top of parapet, elevation 484.0 ft), and the crest width is about 32 ft. Monoliths are numbered consecutively (1 through 28) beginning at the right abutment. Plans and elevations are shown in Figures 10 and 11.

33. A gated central overflow spillway section with a crest elevation of 418.0 ft was constructed in the Concrete Gravity Dam. This section consists of eight gated sluice outlets, 5 ft-by-9 ft. Three 15 ft-6 in. diameter penstocks are located through the right nonoverflow section of the Concrete Gravity Dam. An 84-in. intake conduit was constructed through the right abutment nonoverflow section to furnish water to the Folsom Power Plant, located immediately downstream of the Right Wing Dam envelopment area on the north side of the river.

Interface zone and retaining walls

34. Concrete Dam Monoliths 1 through 6 interface with the Right Wing Dam and are fully to partially embedded in the Right Wing envelopment fill. Monoliths 22 through 28 interface with the Left Wing Dam and are partially to fully embedded in the Left Wing envelopment fill. Typical envelopment sections are shown in Figures 12 and 13. Three retaining walls were constructed in the vicinity of the Concrete Gravity Dam in the wrap-around area parallel to the river. Downstream retaining walls were constructed on both the Right and Left wrap-around areas. Upstream, only the Right wrap-around area required a retaining wall, denoted Retaining Wall B in Figure 3.

35. Retaining Wall B prevents the earth fill of the Right Wing Dam envelopment section from blocking the penstock and powerhouse inlets. During construction, Retaining Wall B also protected the diversion tunnel inlet channel. Plans and sections of the wall from US Army Engineer (1955) are shown in Figure 14. The wall is 406 ft long and consists of 12 monoliths. The crest elevation varies between elevation 310 and 350 ft and is controlled by the intersection of the wall with the designed slope of the earth-fill envelopment. The elevation of the base of the wall varies between elevation 270 and 290 ft. The elevation of the base of individual monoliths was adjusted

according to the existing topography and the quality of the foundation rock. The maximum height of the wall is 82 ft, near wall axis Station 0+29, at the juncture of the wall with Monoliths 6 and 7 of the Concrete Gravity Dam. The minimum height of the wall is 27 ft at wall axis Station 4+35. A two-lane construction road exists at the base of the riverward face of the wall.

36. Retaining Walls A and E provide support to the Right Wing and Left Wing envelopment areas, respectively. Retaining Wall A is 173 ft long and about 54 ft tall at maximum section. This is a combination gravity and cantilever structure connected to the Concrete Gravity Dam at Monolith 7. Retaining Wall E is 239 ft long and 60 ft tall at maximum section. This is a gravity type wall and adjoins the left edge of the flip bucket in Monolith 20. The surface of the backfill behind both walls, the downstream Right and Left envelopment shells, is sloped at 1 vertical to 2 horizontal at the contact with the Concrete Gravity Dam. Foundation conditions and preparation for both walls were similar to those for Retaining Wall B.

Foundation Conditions at Wing Dams, Interface Zone and Retaining Wall B

Wing Dams and interface zone

37. The foundation rock beneath the Right and Left Wing Dams is a weathered granite. The degree of weathering decreases with depth and with distance away from the joint planes. The primary joint set strikes generally N 45° E and dips NW 40° - 45°. Stripping removed organic material and loose, wet soils to expose firm decomposed granite. At the Right Wing Dam, the depth of stripping ranged from 0.5 ft where hard rock was close to the original ground surface, to as much as 18 ft in soft, mucky areas. The average depth of core trench excavation at the Right Wing Dam ranged from about 2 to 3 ft near the right abutment to about 10 ft near the envelopment area. No major faults were encountered in the foundation rock during stripping and excavation of the core trench of the Right Wing Dam.

38. At the Left Wing Dam, stripping depths ranged from 1 to 5 ft, and the depth of excavation for the core trench reached a maximum of 20 ft. A fault striking N 88 E and dipping steeply SE was encountered in the core trench near Station 303+00. No special treatment of this zone was considered necessary. The foundation rock in the core trench was slush grouted as necessary, and outside the core trench the decomposed granite was scarified (where

possible) to a depth of about 6 in. and compacted with either sheepsfoot or pneumatic rollers. Areas immediately adjacent to hard, bouldery masses were hand-tamped.

39. The foundation rock was grouted with a single line of grout holes along the entire length of both Wing Dams. Staged-grouting methods were used. The grout curtain at the Right Wing Dam extended to a depth of about 60 ft, and at the Left Wing Dam the grout curtain extended to a depth of about 75 ft. The grout curtain beneath the Wing Dams was tied in with the grout curtain beneath the Concrete Gravity Dam at the envelopment areas. Identification and treatment of faults encountered in the envelopment areas are described in more detail by Sharp (1988) in Appendix A of Report 3 of this series.

Retaining Wall B

40. The foundation rock is quartz diorite with varying degrees of weathering. Several faults and shears were encountered in the foundation. The most significant were two parallel faults that strike northeast (about N 45° E) and dip northwest (roughly N 45° W), near wall axis Stations 1+65 and 2+17. The fault near wall axis Station 1+65 contained a 0.3- to 8.0-ft wide zone of weathered, brecciated rock, and was exposed in the foundation for Wall Monoliths 1 through 4. The second fault, near wall axis Station 2+17, was exposed in the foundation for Wall Monoliths 5 and 6. No brecciated zone was present where the fault near wall axis Station 2+17 passed beneath the retaining wall. After excavation and cleanup were completed, the foundation rock exposure consisted of sharp, irregularly blasted surfaces, terminating at joint planes. Where the two northwest dipping faults crossed the foundation, V-shaped excavations were used to remove the soft, brecciated, and weathered rock. Between the heel of the Concrete Gravity Dam and wall axis Station 1+20, the brecciated fault zone is at maximum width. Loose material was hand-excavated and the breccia zone was cut vertically to minimize its adverse effect on the foundation. No springs or seeps were present in the mapped area. Eight-inch diameter vitrified-clay pipe drains were installed at the heel (rear face) of Retaining Wall B. The foundation was leveled with 1,811 cu yd of grout and concrete to facilitate forming and placement of subsequent lifts.

Embankment Materials

41. The Right Wing Dam consists of 3 zones, as shown in Figure 8. Zone A is constructed of a fairly dirty rockfill and forms the upstream and downstream shells over most of the length of the dam. Zone B is a transition zone constructed of gravel from the American River. Zone C is the impervious core constructed of compacted decomposed granite from Borrow Area No. 2. (Borrow area locations are indicated in Figure 2.) The Left Wing Dam also consists of 3 zones, as shown in Figure 9. Zone E consists of compacted gravel dredged tailings from the Blue Ravine and forms the upstream and downstream shells. Zone F is the -2-in. fraction of the Zone E gravel and was used as a filter zone between the impervious core and the gravel shells. Zone G, the impervious core, is constructed of compacted decomposed granite from Borrow Area No. 1. The embankment zones, their use in the dams, and the borrow sources are listed in Table 1. The specifications for placement of these materials are listed in Table 2. Gradations for the embankment materials are shown in Figures 15 and 16. Gradations for recent samples in Figure 15 were obtained from US Army Engineer (1986); gradations for record samples were obtained from US Army Engineer (1957). Material properties used in initial design, based on laboratory tests performed prior to construction, are listed in Table 3.

42. The Zone A rockfill was originally planned to contain less than 10 percent sand sizes or smaller (passing No. 4 sieve) and to be placed in the same manner as the Zone B gravels. The source materials for Zone A were found to typically contain about 30 percent passing the No. 4 sieve. The construction records indicate that an effort was made to place the cleaner materials in the upstream shell. The decision was made to place the Zone A material in 12-ft dumped lifts. No additional compaction was applied to these materials. In the design of the Right Wing Dam, the Zone A rockfill was assumed to have the same properties as the Zone B gravel.

Construction Sequence for Foundation Preparation, Concrete Placement, and Embankment Fill Placement and Compaction

43. This section contains a chronological summary of the construction sequence used in the interface zones. The summary begins with foundation

excavation, preparation, and initial placement of concrete, steps through backfilling of core materials near the bases of the concrete monoliths, and ends with the placement and compaction of embankment fill. Applicable excerpts from construction specifications and information from engineering daily logs that help elucidate construction procedures, problems that occurred and their solution, and other changes made during construction of the envelopment areas, are incorporated in the text. The construction photographs were a particularly useful tool for assessing subsurface conditions at the interface and for observing construction sequence and procedures. The construction photographs made it possible to do the following:

- a. Examine the geometry of areas between placed concrete and excavated rock slopes from various vantage points.
- b. Observe the type, placement and compaction of backfill in contact with monoliths.
- c. Observe the extent of concrete placed beyond monoliths at the foundation level.

However, although the photographs were most descriptive, they represent only a brief moment in time. Remaining questions regarding construction procedures, compliance with specifications, and the implementation of solutions to problems that arose during construction were addressed in the interview with Mr. Ott.

Foundation excavation

44. The interface zones include the areas downstream of Monolith Nos. 1 through 7 and 20 through 28, upstream of Monolith Nos. 1 through 6, and material retained by Retaining Walls, A, B, and E as shown in Figure 3. The foundation conditions for these structures are discussed in this section.

45. Foundation conditions in interface zones were well documented in Foundation Reports (US Army Engineer 1952, 1953, 1954a-e) and construction photographs. Foundation reports included detailed plan and sectional drawings of faults, joints, and top-of-rock profiles observed following excavations. A detailed summary of the foundation conditions encountered for each CGD monolith was reported by Sharp (1988), and can be found in Appendix A of Report 3 of this series. Pertinent excerpts from this summary are presented in this section.

46. The excavation for foundations of the CGD monoliths and interface zones was conducted in two phases. The first phase involved an initial excavation for the CGD. This excavation allowed access to and exposure of

foundation bedrock for more detailed geological studies. This excavation occurred prior to any diversion of the American River. The extent of the initial excavation is depicted in Figures 17 and 18 which show photographs taken on 2 January 1951.

47. The second phase of excavation encompassed the foundation areas for the RWD, LWD, and retaining walls. The extent of this phase is apparent in Figure 19 taken on 1 April 1952. The jagged protrusions and slope of remnant bedrock is also noticeable. However, these conditions were changed somewhat prior to placement of fill at those locations as noted in later photographs.

48. Much of the foundation preparation for monoliths of the CGD was conducted in the summer of 1952. During this time, detailed geological studies and further explorations were performed. A photograph documenting the progress in the area of the CGD as of 26 September 1952 is shown in Figure 20.

49. Some important information is contained in the photograph shown in Figure 20. Visible in this photograph is the outline of Monolith No. 28, the end monolith of the CGD at the left abutment. One point of interest in this photograph is the height of rock slope at the right abutment adjacent to the end of the CGD. The height varies from about 35 ft at the core trench to an estimated maximum of 100 ft located downstream of the centerline of the dam. Also, a railway is under construction on the downstream side of the area of Monolith 28 construction. The presence of this railway is of interest because it impeded placement of fill until construction of the CGD was completed. There were requirements placed upon the contractor to maintain the elevation of fill around the CGD at a minimum distance above the lowest concrete monolith of the CGD. Specifically, the embankment was to be kept "50 to 70 ft above the lowest concrete block" (Engineering Daily Log (EDL) 26 October 1953). This clause provided for protection against unexpected flooding which could overtop the concrete sections. It was assumed that this provision expired at the time the concrete spillways were completed.

50. A close-up view of the excavation at the end of the CGD at the right abutment is provided by the photograph shown in Figure 21. Grouting operations were underway at the time of the photograph. Grout pipes were located along the centerline of the dam and perpendicular to the centerline about 10 ft from the end of the monolith (US Army Engineer 1953). This photograph provides a good view of the excavated rock surfaces in the interface zone.

51. Four faults were encountered during the first phase of excavation. Three of these were found in the right abutment; a fourth, the largest, was found in the left abutment. Of the faults in the right abutment, one trends through Monolith No. 1 (US Army Engineer 1954a), dipping northwestward (essentially away from the river channel). Two other faults that trend to the northwest exist in the foundations of Monolith Nos. 4, 5, and 6 (US Army Engineer 1954b). It was concluded from these investigations that the three faults did not pose a stability problem for the CGD or RWD.

52. The fault located in the left abutment was more extensive than originally expected at the design stage. This fault is located between Monolith Nos. 22 and 23, has a trend of north-northeast, and dips 20 to 30° below the horizontal towards the northwest, essentially towards the river channel (US Army Engineer 1954c). As a consequence of the extent of the fault at foundation level, an extensive exploration program was conducted using 16 standard NX core drill holes and five drifts accessed by timber-reinforced shafts. About 45 percent of the estimated total amount of gouge material that existed beneath Monolith 23 was removed in the process of exploration. Following detailed examination by SPK geologists, the exploration holes were backfilled with 2,264 cu yd of concrete. The rock above the fault and fault gouge material in the foundation area of Monolith No. 22 were removed to avoid potential problems with stability that could result from lower strength and seepage pressure in the fault zone.

53. The impact of the fault located in the left abutment was assessed by Tierra Engineering Consultants, Inc. (1983) as part of the overall Folsom study. They concluded:

> ...no faults or lineaments striking toward or extending through the main dam (CGD)...were found... Faulting and shearing observed in the main dam foundation excavations (unpub. Corps construction records, Kiersch and Treasher, 1955) may perhaps be related to the intrusion of the pluton. In the absence of strong lineaments of pre-reservoir or recent imagery, mapped fault zones or geomorphic indicators of faulting near Folsom Reservoir impoundment structures, it is concluded that the possibility of fault displacements within the foundations of these structures is extremely remote.

Foundation preparation

54. The preparation of foundation rock for placement of concrete

monoliths and embankment fill was very thorough to provide a good contact between rock, concrete and fill material, as indicated in the construction documents and photographs. An example of a foundation area prior to placement of concrete is shown in Figure 22. This photograph of the foundation for Monolith No. 28 shows several workers hand cleaning and preparing the rock. Extensive use was also made of dental techniques.

Initial placement of
concrete in monoliths

55. Construction of concrete monoliths progressed more rapidly at the left abutment than at the right abutment. In Figure 23, a photograph taken on 22 July 1953 indicates the progress made as of that day. The monoliths are numbered for convenience. This photograph provides additional insight into the condition of excavated rock slopes downstream at the right abutment and upstream at the left abutment. For the right abutment, the rock slopes are near-vertical and as yet very jagged. For the left abutment, the slopes appear to be much less steep. This photograph also shows concrete extensions constructed at the bases of some monoliths. Figures 24 indicates the locations of these extensions, particularly in interface zones, as compiled from drawings in US Army Engineer (1954a-e).

56. At the right abutment, concrete extensions were utilized on the downstream side of all monoliths in the interface zone. The concrete extensions provided a more positive contact between soil, rock, and concrete in these locations. It can be observed in Figure 24 that the concrete monolith extensions on the downstream side at the right abutment were sufficiently long that core material, Zone C, was compacted against a large, flat surface. The concrete extensions were long enough that a wide portion of the adjacent Zone B gravel filter was also compacted against concrete extensions as shown for the transverse section of Monolith No. 3 in Figure 24. The concrete extensions were stopped short of the downstream rock face (see transverse section of Monolith 3 in Figure 24) only where Zone B gravel filter and Zone A rockfill materials would fill the trench formed by the end of the concrete extension and the foundation rock.

57. Concrete extensions at two different elevations were constructed parallel to the dam axis at the right end of the CGD, Monolith No. 1, and are indicated in Figures 25 and 26. The photograph in Figure 25 shows the locations of these lateral extensions, as well as the downstream extension. These

lateral concrete extensions provided relatively flat surfaces, about 20 ft wide or more, upon which core material could be compacted and tied in well with the steep, blocky foundation rock slope to the right of Monolith No. 1. One of the two lateral concrete extensions had a surface elevation of 298 ft (formed by lift 298 indicated in Figure 26), and was constructed from approximately the centerline to the upstream edge of Monolith No. 1, as shown in Figure 25.

58. The other lateral extension had a surface elevation of 288 ft (formed by lift 288); it extended from the centerline to the downstream foundation rock face, and tied into the downstream concrete extension which had a surface elevation of 293 ft (formed by lift 293). The downstream extension of Monolith No. 1 can be seen in Figure 25. The lateral extension at elevation 288 ft cannot be seen clearly in Figure 25 because of construction debris present in this area on the day the picture was taken. As discussed in the next section, this debris was cleared prior to placement of fill in this area. The design of this area did not call for the construction of concrete extensions on the upstream side of the envelopment monoliths, except at Monolith No. 6, as shown in Figure 24.

59. At the left end of the CGD, Monolith No. 28, lateral concrete extensions were not constructed. It can be seen in the photograph of the foundation preparation for Monolith No. 28, Figure 22, and the longitudinal sections in Figure 26, that a fairly wide, relatively flat rock foundation surface existed in this area upon which the embankment fill could be compacted for a positive contact between soil, concrete and foundation rock. Downstream concrete extensions were constructed at the bases of Monolith Nos. 24 through 27, as shown in Figure 24. (These extensions can be seen in the background of the photograph in Figure 30.) The foundation rock in this area sloped steeply towards the river channel, and also formed a fairly steep, blocky slope face downstream (see photographs in Figures 19 through 23). These downstream concrete extensions (shown in Figure 22) were sufficiently long to provide a large, flat surface upon which the Zone G core, Zone F gravel filter, and Zone E gravel shell materials could be compacted. Figure 24 shows that concrete extensions were constructed upstream for Monolith Nos. 24 through 26.

60. As described in a previous section, the excavation for Retaining Wall B, located in the upstream right envelopment area, exposed sharp, blocky

rock surfaces and a number of faults. Figure 27 shows a photograph of the exposed foundation for the upstream retaining wall prior to clean-up. The fault zones received careful treatment to minimize their adverse effect on the foundation. A drainage system was installed along the base of the wall, and eight drainage ports were installed normal to the dam axis to drain retained water into the river channel. This drainage system would relieve high pore pressures that could otherwise develop behind the wall in the event of rapid drawdown. As shown in the sections of the wall in Figure 14, only Zone A rockfill and Zone B gravels are in contact with the wall, not core material. After excavation and clean-up were completed, the foundation was leveled with grout and concrete to facilitate forming and placement of subsequent lifts. The photograph in Figure 28 shows progress of construction of the wall and the shape of the bedrock behind the wall.

61. The photograph in Figure 29 shows the progress of construction in the vicinity of the CGD as of 29 July 1953. This photograph shows the rise in the rock foundation behind Retaining Wall B and the rock foundation boundary conditions at the bases of most of the CGD monoliths. The support structure for the railway used to transport concrete for monoliths is shown under construction downstream of the right envelopment monoliths. Envelopment embankment fill was not placed and compacted in this area until the support structure was removed. The excavated rock face downstream of Monolith Nos. 1 through 6 was steep and blocky as of the date of this photograph. The vertical relief from the base of Monolith No. 1 to the foundation level for the Right Wing Dam (where the cars are parked in Figure 29) is approximately 100 ft. In the background in Figure 29, the foundation conditions upstream of the bases of the left envelopment monoliths can be seen.

62. The photograph in Figure 30 was taken on 15 October 1953 and shows the downstream area of the right envelopment monoliths in the foreground. The exposed rock face downstream is less steep and smoother in this photograph than in previous shots. This smoothing and flattening of the rock face made it possible to achieve a more positive contact between the foundation rock and the embankment fill in this area.

63. In the background of Figure 30, a crib wall can be seen on the upstream side of the CGD at the left abutment. Engineering Daily Log dated 3 August 1953 documents the contractor's request to construct this crib wall to retain fill to facilitate construction of monoliths of the CGD. Permission

was granted to use the crib wall provided that it and the fill were removed completely prior to placement of specified backfill. Later construction photographs show that the crib wall and temporary fill were indeed removed.

Backfilling near the bases of monoliths

64. As stated in the beginning of this chapter, Seed et al. (1978) concluded, on the basis of observations of performance of many dams during earthquakes that virtually any well-built dam will perform safely under moderate earthquake shaking. One aspect of well-built earth dams is careful placement and adequate compaction of the embankment fill materials. It was concluded in Report 6 of this series that the embankment materials in all zones of the Wing Dam were well placed and adequately compacted and would not develop significant residual excess pore pressures if subjected to the design earthquake ground motions. In the interface zones, placement and compaction of earth materials were complicated by: (a) the mixed-face conditions, concrete and rock, against which positive contact needed to be achieved, particularly with core materials, and (b) the geometry of this mixed-face surface, namely the blocky and sometimes steep nature of the exposed foundation rock, the sharp angles of the ends of the CGD and the extensions at the bases of the monoliths, and the somewhat narrow spaces formed by the rock foundation and the concrete faces near the bases of the monoliths. In some cases, these conditions made it necessary to work by hand in the placement and compaction of fill and prevented the use of heavy compaction equipment.

65. The presence of a continuous zone of loose core material would not necessarily result in inadequate seismic performance of the interface zones, because of the defensive design of the section: wide, cohesionless filter and transition zones were constructed upstream and downstream and would prevent piping if high pore pressures developed or cracking occurred in the core or at the contact of the core with the concrete or rock. Also, the rigid boundary conditions formed by the concrete structure and the sloped foundation rock face at the bases of the monoliths would inhibit the development of seismically-induced shear strains necessary to result in significant residual excess pore pressures. Nevertheless, consistently well compacted core and cohesionless materials in the interface zones are the preferred condition.

66. In the interface zones at the Folsom Project, particular care was taken in the design of the section and during construction to carefully place and compact earth materials to achieve a positive contact between the

embankment fill, especially core materials, the concrete monoliths, the concrete extensions at the bases of the monoliths, and the rock foundation. Particular care was also taken in the placement and compaction of the different embankment zones to prevent fouling of filter and transition zones with core materials. This section presents pertinent excerpts from specification documents; these documents, engineering daily logs, and the construction monitoring observations provided by Mr. Ott, demonstrated the care with which fill was placed and compacted in the interface zones. A number of construction photographs are included to help illustrate the procedures used in the envelopment areas.

67. Two specification documents apply to the interface zones: Specification No. 1532 (US Army Engineer 1950a) which prescribed construction procedures covering embankment, borrow and backfill for the Wing Dams in general, and Specification No. 1359 (US Army Engineer 1950b) which specifically addressed procedures to be used in the right envelopment area. Four excerpts from these documents that address backfilling procedures are printed below.

68. Specification document No. 1359 (US Army Engineer 1950b), Section 4-03d, states:

> After the foundation for embankment and cut-off trench has been stripped and excavated in accordance with the provisions of Section 2, the Contractor shall fill all holes and depressions by placing, moistening and compacting approved materials with tamping rollers or pneumatic tampers, all as directed. The entire foundation area shall then be graded, moistened and compacted as directed. When ordered, prior to placing material for the embankment proper, the foundation shall be roughened by discing or scarifying and moistened, to provide a satisfactory bond. The entire foundation area shall be approved prior to placing of any fill thereon. Fill placed below the tolerance limits, specified for required excavation, will be at the expense of the Contractor.

69. Specification document No. 1532 (US Army Engineer 1950a), Section 4-03q (6), states:

> Loose thickness of layers shall be 4 in. Layers shall be compacted to a density equivalent to that obtained by the specified rollers. Hand operated pneumatic tampers shall be similar and equal to the Ingersoll Rand backfill tamper, size 34, having a butt or tamping surface area of approximately 25 sq in., operated

at an air pressure ranging from 80 to 100 lb/sq in. at all times.

70. Sections 4-08 and 4-10 of Specification document No. 1532 state:

> (4-08) EMBANKMENT CONSTRUCTION OF ENVELOPMENT OF ENDS OF MAIN CONCRETE DAM: Where the Right and Left Wing Dams envelope the ends of the Main Concrete Dam*, the Zone C and Zone G materials immediately adjacent to the concrete shall be sloped up against the concrete, if necessary, to permit the contact surfaces of the compaction equipment to work within 2 in. of the surface of the concrete. Extreme care shall be exercised in controlling all placing operations in Zone C and Zone G immediately adjacent to the concrete dam to insure uniform required moisture content and uniform required compaction in a manner which will obtain the highest quality embankment construction.

> (4-10) BACKFILLING STRUCTURES: Concrete structures shall be backfilled with approved materials of the types indicated on the drawings. No backfill shall be placed against concrete work until concrete is at least 7 days old and all forms and bracing have been removed. Backfilling on opposite sides of concrete structures shall be kept at approximately the same elevation to equalize the loading. Care shall be exercised at all times during backfilling operations to prevent any damage to the concrete. Where backfill is to be placed on concrete, the backfill shall be completed to a height of 4 ft over the concrete before heavy construction equipment will be allowed thereon. Placing, watering, and compacting backfill for the Embankment Retaining Walls A, B, C, and D shall be as specified in paragraph 4-03.

71. The language and detail in these excerpts from construction specifications indicate the high degree of attention paid by the designers to assure well-built interface zones. It is usual US Army Corps procedure to inspect carefully the construction of such key design features and to enforce stringently specifications and modifications thereto regarding construction. Attention to detail mentioned in the engineering daily logs and the observations of Mr. Ott (see Appendix A) confirmed that Corps personnel closely monitored these areas to assure the construction of well-built envelopment zones. Although photographs of the placement and compaction of the first few

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* Concrete Gravity Dam.

lifts of backfill at the bases of the envelopment monoliths were not available among the construction records, Mr. Ott confirmed that:

- a. Cleanup before placement of embankment material was thorough.
- b. Areas not accessible to heavy compaction equipment were compacted using hand tampers on material placed in 4-in. lifts.
- c. Tests were made to ensure that materials were properly compacted.
- d. Temporary fill materials previously placed by the contractor for his benefit were removed and replaced in approximately horizontal lifts. This included materials with substantial height differentials that may have existed at the ends of the CGD, upstream and downstream of the envelopment monoliths, and behind retaining walls.

72. Figures 31 through 33 show photographs of fairly early stages of fill placement in the right envelopment area. The lower left corner of the photograph in Figure 31 shows core material placed at the landward end of Monolith No. 1. This photograph was taken on 28 April 1953. The elevation of the top of this fill was estimated to be 308 ft, which corresponds to 10 and 20 ft above the surface elevations of the two landward end extensions at the base of Monolith No. 1. These extensions are shown in Figures 25 and 26. In view of the steepness of the upstream side of this fill indicated in Figure 31, it was estimated that equipment carrying core material for this fill accessed this area, at least after the first few lifts, by the core trench of the RWD. Two photographs dated 15 October 1953, Figures 32 and 33, show that the elevation of fill upstream at the right end of the CGD (Monolith No. 1) is high enough (estimated elevation 310 ft) that the gap between the CGD and foundation rock is wide enough to allow passage of heavy vehicles for placement and compaction of backfill. At this stage of construction, it was possible to use in the interface zones many of the same procedures and equipment used to haul, prepare, place, and compact fill used in the RWD and LWD. Also evident in Figure 33 is the placement of Zone B material in the background.

Placement of embankment fill

73. Specification documents provided details regarding excavation of borrow materials, moistening, material placement and compactive effort in addition to many other aspects of construction. A portion of specification document 1532, Section 4-03a, pertinent to placement of embankment fill is printed below:

a. General: The embankments shall be constructed to establish gross dimensions with approved materials obtained, as previously specified, from required excavation, from stockpile areas, and from borrow areas. All materials placed in the embankments shall be free from roots, brush, rubbish, and other objectionable materials. In all cases, the Contracting Officer will determine the moisture content, processing of material, and additional compaction rolling. The Contracting Officer will at all times direct the location where the material is to be placed in the embankment. During construction, the Contractor shall suitably identify the limits of the various embankment zones and equipment transporting materials thereto to insure placement of the various materials in their proper zones and facilitate the necessary inspection therefor. The construction of the various zones constituting the embankments shall be carried on concurrently as follows:

(1) Right Wing Dam: Except for Zone A, the various zones comprising the Right Wing Dam shall be constructed in such a manner that at no time will the difference in elevation of adjacent zones exceed 5 ft. Zone A may be constructed following completion of Zone B. Zone B materials will be permitted to encroach upon or into Zone A, but in no case will they be permitted to encroach upon or into Zone C. Zone C materials will be permitted to encroach upon or into Zone B.

(2) Left Wing Dam: The various zones comprising the Left Wing Dam shall be constructed in such a manner that at no time will the difference in elevation of adjacent zones exceed 5 ft. However, no encroachment of Zone E materials upon or into Zone F, and of Zone F materials upon or into Zone G will be permitted. Zone G materials will not be permitted to encroach upon or into Zone F.

The compactive effort required for each type of zone material is summarized in Table 1.

74. Specification document 1532 made special provisions for the placement of fill in contact with concrete and existing fill slopes. Adjacent to concrete, the core backfill materials were to be sloped up against the concrete to permit compaction equipment to work within 2 in. of the concrete (Section 4-08). Specification document 1532 also required that core materials be sloped up on existing fill slopes so that compaction equipment could simultaneously compact lifts of material on the slope and on the new fill surface.

The document further stated that a maximum lift thickness of 12 in. could be used for core materials along with a minimum of 12 passes of a sheepsfoot roller or six passes of a pneumatic-tired roller (Section 4-03a(b)).

75. A review of construction documents pertaining to placement of fill in interface areas indicated that proper care was exercised in preparing, placing, and compacting fill materials. A few important points are highlighted next.

76. The contractor experienced a problem in placing fill around the CGD because of the presence of the support structure of the railway used to place concrete in the CGD monoliths. This problem was well documented in the Engineering Daily Logs. Pertinent EDLs are reprinted in Appendix B.

77. Another observation was the differential in heights of adjacent fill zones shown in photographs of construction of the RWD. Figure 34 shows a photograph, dated 16 April 1954, of fill placement in the right upstream envelopment area. The differential in heights of Zone B transition and Zone C core materials is shown in this photograph and also in the photograph in Figure 35 taken 10 weeks later. The maximum height differential between these zones as observed in construction photographs was estimated to be about 50 ft. The specifications required that height differentials between adjacent fill zones not exceed 5 ft. This specification requirement was not enforced rigidly in the field in the vicinity of the envelopment areas.

78. It was concluded that the detrimental effects of these height differentials were considered small enough to be acceptable to the designers and the Corps personnel in the field. It was also estimated that rigid enforcement of this specification in the envelopment areas would have caused unwarranted construction delays. For example, some means had to be employed to tie in the embankment fill of the RWD which was nearly completed with the envelopment fill, which lagged behind. This situation is shown in Figure 34. It was concluded in this study that the height differentials that occurred during construction were not significant to the seismic performance of the shell zones. It was also noted that the cohesionless zones were built up faster than the core zones. This procedure minimized the possibility of fouling the filter materials with fines, and is the preferred construction sequence.

79. Once the support structure for the railway was removed, fill operations on the downstream side of the CGD caught up with the right envelopment fill placement. In the photograph dated 11 June 1954 shown in Figure 36, this

has occurred. This photograph also shows that a fully-loaded dump truck was used to compact core material, particularly at the contact with the CGD. Although this equipment was not mentioned in specification documents, (the estimated wheel loads of this type of fully-loaded dump truck with 85-kip capacity are only about 65 percent of the wheel load of a specified pneumatic-tired roller with 100-kip capacity), a fully-loaded dump truck was considered by the designers and is considered to be an acceptable means to compact the core materials.

80. A fully-loaded dump truck has been used successfully in the construction of a number of dams. For example, Low and Lyell (1967) document the use of this equipment as a means to compact a silty to clayey sand in the Portage Mountain Dam, Canada. Sherard and Dunnigan (1985) document the use of this equipment to compact soil in contact with concrete to minimize seepage at the Guri Hydroelectric Project in Venezuela. Numerous construction photographs of the process indicate that great care was exercised in using this procedure at the Folsom Project.

81. A photograph taken on 1 July 1954 and shown in Figure 37 shows that the crib wall on the upstream side of the CGD at the left abutment had been removed. The area had not yet been cleaned and consequently prepared for backfill. Fill operations had been underway in areas just outside of this photograph as evidenced by the photograph taken the previous Fall dated 29 October 1953, shown in Figure 38.

82. Figure 39 is a photograph dated 8 September 1954 that shows that the railway located on the downstream side of the left abutment still inhibited placement of downstream gravel shell material at the LWD. The impact is more pronounced in the photograph shown in Figure 40. A view of the completed dam following initial reservoir filling is provided in the photograph shown in Figure 41.

Conclusion

83. A comprehensive review of construction records for the Folsom Project was undertaken to achieve an understanding of the complex geometry of the interface zone, the fill materials located there, the methods of placement and compaction of these materials, and particular construction practices used in this area. This review was assisted by Mr. John Ott who was the Materials

Engineer for the Folsom Project during construction. The observations drawn from this study were:

a. That the core and other embankment materials were placed and compacted with more than adequate care to ensure that the envelopment fill met or exceeded design requirements.

b. That large, cohesionless filter and transition zones exist upstream and downstream of the core to prevent progressive piping of core materials if cracks were to develop as a result of earthquake shaking.

c. That the geometry of rock and concrete boundaries at the bases of the concrete monoliths form rigid boundaries that inhibit the development of cyclic shear strains necessary to the development of high, seismically-induced residual excess pore pressures.

PART III: FIELD AND LABORATORY INVESTIGATIONS

General

84. The results of recent field and laboratory tests were combined with data from record samples obtained and tested during construction to determine index properties and penetration resistance of compacted core material in the interface zones. These properties included Standard Penetration Test (SPT) resistance, in-place density, and maximum dry density as determined by compaction tests. Also, various parameters relating to grain size distribution, soil classification, and Atterberg Limits were determined to allow comparisons of SPT N-value, in-place density and maximum dry density at interface zones with the same parameters at other locations in the Wing Dams. These laboratory and field test results are presented in this chapter, and later used in the stability evaluation presented in Part IV.

Record Samples

85. Personnel from SPK observed fill placement and performed quality control testing during construction. Quality control testing included in-place density and moisture measurements, laboratory compaction tests, Atterberg Limit tests, grain size distribution tests, and specific gravity tests. These data are summarized in the report of soil tests on record samples (US Army Engineer 1957). Data pertaining to compacted core material in the interface areas relevant to this study were extracted and are presented in this section.

86. A summary of tests performed during construction on record samples of core material from the interface zones is presented in Table 4. Average values are provided to facilitate comparisons with recent test results. The ranges in grain size distribution curves of record samples in the RWD and LWD interface areas are shown in Figures 42 and 43, respectively. The results of modified-effort compaction tests (AASHTO T99-49) of record samples from the RWD and LWD interface areas are shown in Figures 44 and 45, respectively.

87. The results of physical property tests listed in Table 2 and shown in Figures 42 through 45 generally indicate that the gradations, Atterberg limits and compaction characteristics measured for core materials at interface zones during construction are consistent with each other. The bands

representing the ranges in grain size distribution are narrow and similar for Zone "C" and "G" materials. The variation in Atterberg Limits is also small. Maximum dry densities and optimum moisture contents from modified compaction tests are more variable with densities ranging from 131.4 to 136.8 pcf and moisture contents ranging from 6.8 to 8.7 percent. The ranges in in-place density and moisture content are 120 to 131 pcf and 5.1 to 10.4 percent, respectively. The average in-place density (128 pcf) corresponds to about 96 percent of the average maximum dry density (134.0 pcf). Results for only nine record samples from the interface areas were found in construction records. The elevations of these record samples ranged from 353 to 427 ft. In general, record sample test results indicate that the materials were placed in a satisfactory manner and good fill material control was maintained.

Drilling at Interface Zones

88. The full height of compacted core fill adjacent to the concrete monoliths was of interest in this study. A drilling exploration program was designed in order to sample the full height of fill in very close proximity to the CGD at each end. The ends of the CGD are battered at 0.05 horizontal to 1.0 vertical, so a series of drill holes was used, with holes offset specific distances from the soil-concrete contact at the surface, to sample core material placed close to the CGD. As-built cross sections of the Wing Dams at the ends of the CGD (Station 285+35 and 299+35) are shown in Figure 46 along with the extent of drill holes at interface zones. The locations of drill holes at interface zones are shown in Figure 47. Profiles at the ends of the dam that show the locations of drill holes and record samples are provided in Figures 48 and 49.

89. Five holes were advanced through embankment fill at each interface zone. Two holes at each end were used to obtain "undisturbed" samples. Undisturbed samples were extracted to depths of 120 and 102 ft for the RWD and LWD, respectively, which correspond to about 62 and 63 ft above the base of the concrete monoliths or extensions, respectively. Three holes at each end were used to perform the SPT and obtain samples. The SPT measurements were performed through the full height of fill. Three different drill rigs were used: a CME 75, a CME 550, and a Mobile B-53. Energy was delivered to the SPT spoon using one of three systems: a rope-and-cathead-operated safety

hammer, a wireline-operated safety hammer, and an automatic trip hammer. Hole stability was maintained using either hollow-stem augers with no drilling fluid or mud rotary methods. Two different diameters of hollow-stem augers were used. A summary of drill information is contained in Tables 5 and 6.

Recent Laboratory Tests

90. Laboratory tests were performed on numerous disturbed and undisturbed samples extracted from drill holes at the ends of the CGD. These include physical property tests (sieve analysis, Atterberg Limits, specific gravity), determination of field unit weight and moisture content, engineering property tests (compaction, triaxial compression, and unconfined compression), and dynamic property tests. All testing was performed by the SPD laboratory and data reported in US Army Engineer (1986) and US Army Engineer (1989). Results of tests performed on samples from interface areas, as reported by SPD, are reproduced in Appendices C and D, respectively.

91. Test results of most interest for this study include sieve analysis, Atterberg Limits, specific gravity, unit weight, moisture content, and maximum dry density and optimum moisture content determined in compaction tests. Data from these tests for samples in interface zones are presented in Tables 7 and 8 for the RWD and LWD, respectively. Soils were classified according to USCS.

92. The ranges in grain size distribution and Atterberg Limits for recently obtained samples from the RWD and LWD interface areas are presented in Figures 50 and 51, respectively. Ranges for record samples for the respective areas are provided for comparison. In general, the ranges in grain size for new samples are much broader than the ranges for record samples for both interface areas. The expanded ranges extend in both directions (i.e. higher gravel contents and higher fines contents). The number of recent samples from interface zones, however, is much larger than the number of record samples that could be found in construction records. The range in Atterberg Limits is similar to that from record test samples.

93. The results of modified compaction tests performed recently on samples obtained from each end of the RWD and LWD are shown in Figures 44 and 45, respectively, along with record sample data. Maximum dry densities of samples recently obtained at the interface areas are at the upper bound of the range

of dry densities for record samples, which have an average dry density of 134.0 pcf (Table 4). The overall average maximum dry density of all samples determined in modified-effort compaction tests (AASHTO T-99) in the interface areas is 134.4 pcf.

94. Measured dry densities for undisturbed samples taken from drill holes used for undisturbed sampling (drill holes R-1, R-2, L-1, and L-2) are summarized in Table 9. The degree of saturation corresponding to a specific gravity of 2.72 was also calculated for each sample. The value of 2.72 is an approximate average from record samples and recent test results performed on core materials. There is a wide range in values of dry density, due primarily to two anomalous values of density (110.1 at elevation 465 and 112.4 pcf at elevation 389) which are well below the average of 124.1 pcf. The average value corresponds to 92 percent of the average maximum compacted density of 134.4 pcf. If the two anomalous test results are excluded, the average in situ dry density is 126.1 pcf (which is 94 percent of 134.4 pcf), with a standard deviation of 3.8 pcf. As stated earlier, the average in situ dry density of record samples was 128 pcf, which is 95 percent of 134.4 pcf. The shearing that a volume of soil undergoes as a sampling tube passes through it in general causes dense soils to dilate. Consequently, it is expected that the dry densities measured for the undisturbed samples underestimate the actual in situ dry densities.

95. For the purposes of this study, a uniform density profile was assumed for effective overburden computations. The moist density above the water table was assumed to be 136 pcf, and below the water table a value of 142 pcf was selected. These values are considered to correspond to representative measured values of dry density and moisture contents. The location of the water table was determined from soundings made in observation wells or inferred from soundings made during drilling operations.

Standard Penetration Resistance

96. Many variables affect the energy applied to the split spoon sampler which has a direct effect on the N-value measured. These include the type of drill rig, hammer shape, hammer release system, hole advancement technique, and length of drill rod. Studies have been conducted and are underway to quantify these effects (e.g., Kovacs et al. 1977 and McLean et al. 1975).

Seed et al. (1985) reviewed numerous studies and recommend that N-values be adjusted to a value corresponding to 60 percent of the free fall energy, N_{60} , thereby allowing appropriate comparison of N-values measured using different equipment and procedures.

97. The N-values used for analysis in this study were adjusted to values equivalent to 60 percent energy efficiency. N-values measured using the trip hammer were assumed (based on WES experience) to correspond to an energy ratio of 80 percent and were converted to correspond to an equivalent energy of 60 percent (N_{60}) by multiplying the measured N-value (N) by a factor of 1.3 (Seed et al. 1985). N-values measured using the rope-and-cathead technique with a safety hammer were assumed to correspond to an energy efficiency of 60 percent, N_{60} . The energy efficiency for N-values measured using the safety hammer attached to a wireline is not readily apparent. Rather, some evaluation was required as described later. There were no corrections made to account for the type of drill rig or means of maintaining the stability of the drill hole. The corrections are discussed in more detail later in this section. All N-values measured at depths of 10 ft or less were multiplied by a factor of 0.75 to account for energy loss in the drive rods (Seed et al. 1983).

98. The standard penetration resistances, N-values, measured with a particular set of equipment and procedures within a soil deposit are affected by the effective state of stress and relative density (Gibbs and Holtz 1957, Marcuson and Bieganousky 1976). Vertical effective stress along a drill hole can be estimated from the profile of unit weight and depth to the phreatic surface (if hydrostatic conditions can be assumed and if the effect of the presence of the CGG is ignored). The N-value corresponding to an effective vertical stress of 2,000 psf, N_1 , is a convenient reference value to which N-values for other confining stresses can be empirically related. This correction allows an approximate evaluation of variation in relative density for homogeneous granular soils. Correlations developed by Marcuson and Bieganousky (1976) have been used by Seed et al. (1985) to derive correction factors, C_N , to adjust from N (or N_{60}) to N_1 (or $(N_1)_{60}$) as shown in Figure 52. These corrections are applied as follows:

$$N_1 = C_N \times N \quad \text{(blows/ft)} \tag{1}$$

or

$$(N_1)_{60} = C_N \times N_{60} \quad \text{(blows/ft)} \tag{2}$$

These equations are based on the assumption that level-ground and hydrostatic conditions exist.

99. The value of $(N_1)_{60}$ corresponding to clean sand (≤ 5 percent fines), $(N_1)_{60c}$, is a convenient reference value to which N-values for less clean sand can be empirically related. This is obtained using relationships such as those shown in Figure 53. Once the $(N_1)_{60}$ and percentage of fines are known, a vertical segment is drawn from the value of $(N_1)_{60}$ on the abscissa until it intersects the curve (or interpreted curve) corresponding to the percentage of fines. Then a horizontal segment is drawn from that point and intersecting the curve representing ≤ 5 percent fines. The $(N_1)_{60c}$ is determined by drawing a vertical segment intersecting the abscissa.

100. It was necessary to derive an energy efficiency ratio that would be appropriate for SPT N-values measured with the safety hammer regulated by wireline (borings 7F-88-1 and 7F-88-2 shown in Figure 47). This means of releasing the hammer is not discussed by Seed et al. (1985) who suggest energy efficiencies for different types of systems. The energy efficiency was estimated from a comparison of N_1 values derived from the wireline and safety hammer system with $(N_1)_{60}$ derived from the rope-and-cathead and safety hammer system at equal depths. At each interface zone, there is some overlap in the depths of SPT sampling using the two systems. At the RWD, SPT measurements were made in borings R-4 and 7F-88-1 in the 30-ft interval between elevations 380.5 and 350.5. At the LWD, SPT measurements were made in borings L-4 and 7F-88-2 in the 11-ft interval between elevations 380.5 and 369.0. The holes at each end are separated by about 7 ft which minimizes the potential effect of material variability.

101. Two potential sources of variability that could affect this comparison are the two methods of drill hole stabilization (hollow-stem augers with ambient water and drilling mud) and the pronounced difference in pool levels (about 80 ft lower during use of the wireline system), and consequently in pore water pressures. A recent study reported by Seed et al. (1988) indicated that the method of drill hole stabilization probably has an insignificant effect on measured blowcount. The difference in pool levels has little, if any, effect on the calculation of N_1 in this case because the effective stress is so large at the measurement depths that C_N is nearly constant.

102. The comparison of wireline blowcounts with $(N_1)_{60}$ from the other two systems is shown in Figure 54. Lines of equal energy efficiency (ER) are

also indicated in this figure for reference. The energy efficiency for the rope-and-cathead system was estimated to be 60 percent, as described previously. If the energy efficiency of the wireline system were 60 percent, the data should plot nearly evenly on both sides of the line representing 60 percent. It is apparent from the data in Figure 54 that this does not occur and that an energy efficiency of approximately 35 percent is more representative. To account for the potential sources of error cited above, an energy efficiency of 35 percent was selected to be representative for SPT N-values measured using a safety hammer delivered with a wireline system.

103. Values of $(N_1)_{60}$ determined from drill holes through embankment fill at the ends of the CGD are presented in Figures 55 and 56 for the RWD and LWD, respectively. One adaptation that was made for the presentation of this data was the use of blowcounts from penetrations of less than 18 in. Most (uncorrected) N-values measured with the wireline system were very large; many SPT tests were halted prior to attaining full penetration when 50 blows were obtained over a penetration of less than 6 in. Because of the larger correction (0.26) applied to N-values from the wireline system to obtain $(N_1)_{60}$, blowcounts have much lower values of $(N_1)_{60}$. Blowcounts with 50 blows in less than 6 in. over any segment of the 18-in. penetration can have values of $(N_1)_{60}$ of as low as 17 blows/ft (e.g., 6-in. blowcounts of 5,10,50/.45). Therefore, blowcounts were extrapolated for cases where full penetration was achieved for the first 6 in. and partial penetration existed for the last 12 in. This extrapolation consisted of taking the number of blows for the distance penetrated beyond the initial 6 in. and dividing by the distance penetrated beyond the initial 6 in. Estimated values of $(N_1)_{60}$ obtained using this procedure are differentiated ("partial drive") from standard values of $(N_1)_{60}$ ("full drive") in Figures 55 and 56.

104. The data plotted in Figures 55 and 56 indicate that there is a relatively consistent band of $(N_1)_{60}$ values throughout the height of embankment fill at both ends of the CGD. Excluding the upper reaches of fill, the values of $(N_1)_{60}$ generally are between 16 and 34. The values of $(N_1)_{60}$ at greater depths, corresponding to N-values measured using the wireline system are slightly higher but consistent with the values measured throughout the profile.

PART IV: EVALUATION OF STABILITY

General

105. The evaluation of stability and adequate seismic performance of the interface zones focussed on the following key elements:

- a. The presence of defensive design measures built into the project, in particular the large volume of cohesionless fill in shell and filter zones available to fill cracks if they occur.
- b. The liquefaction resistance and post-earthquake strength of shell and transition gravels.
- c. Retaining wall stability and stability of shell backfill if retaining walls slide or overturn.
- d. The liquefaction resistance of core materials and the presence of downstream filter zones to prevent piping and progressive internal erosion.

106. The stability studies documented in Reports 6 and 7 of this series indicated that during and after the earthquake, adequate performance of the interface zones is controlled primarily by adequate performance of the gravel shell, filter and transition zones, because of the role these zones play in slope stability, defensive design, and prevention of internal erosion. It was concluded in Report 6 that: (a) the Wing Dams would perform satisfactorily during the design seismic event, (b) the embankment shell, filter and transition gravels had more than adequate cyclic strength and would not develop significant residual excess pore water pressures, (c) the embankment shell, filter and transition materials would have more than adequate post-earthquake strength, and (d) no significant deformations would occur in the Wing Dams as a result of the design seismic event. It was concluded in Report 7 that: (a) none of the retaining walls will undergo sufficient seismically induced movement to result in slope instability, and (b) even complete failure and removal of the walls would not result in slope instability of the interface zones sufficient to allow loss of the pool.

107. The remaining key element that controls seismic stability of the interface zones is the potential for liquefaction of the core materials adjacent to the CGD monoliths. The review of constrution records presented in Part II demonstrated that the core materials were carefully placed and well-compacted in the interface zones and that large volumes of cohesionless

materials exist upstream and downstream to prevent piping. Based on performance of other dams during moderate earthquakes, Seed et al. (1978) concluded that good design, construction and compaction of embankment dam materials are important indicators of satisfactory seismic performance. A high degree of compaction results in high cyclic strength of a soil. The liquefaction potential of the core materials in the interface zone is addressed in this chapter by a comparison of the cyclic strength of these materials estimated from SPT blowcounts and the earthquake-induced cyclic stresses.

Liquefaction Assessment of Interface Core Materials

Performance-based procedure to estimate cyclic strength

108. The cyclic strength of the interface core materials was estimated using the performance-based procedure developed by Professor H. B. Seed and his colleagues at the University of California, Berkeley (Seed et al. 1985). The chart used for determining cyclic strength is shown in Figure 53. This chart relates measured $(N_1)_{60}$ values to estimated cyclic stress ratios at several sites which have been subjected to earthquake shaking from a M = 7.5 seismic event. The lines on the chart distinguish safe combinations of $(N_1)_{60}$ and cyclic stress ratios from unsafe combinations based on whether or not surface evidence of liquefaction was observed in the field. This chart is interpreted to relate $(N_1)_{60}$ to the cyclic stress ratio required to generate 100 percent residual excess pore pressure. Data for clean and silty sands with different fines contents are presented in Figure 53 along with interpreted relations between cyclic stress ratio causing liquefaction (for a confining pressure of about 1 tsf, level ground conditions, and earthquakes with M = 7.5) and $(N)_{60}$. The cyclic loading resistance is 20 percent higher, with M = 6.5 events for any value of $(N_1)_{60}$, than for M = 7.5 earthquakes.

109. The data and inferred relationships shown in Figure 53 provide a means to establish threshold values of $(N_1)_{60}$ or $(N_1)_{60c}$ above which liquefaction will not occur, regardless of the magnitude of cyclic stress ratio. These threshold values are a function of the percentage of fines. The average fines content of the SPT samples from the RWD was 26 percent and the average for the LWD was 42 percent. For a fines content of 26 percent, the threshold is approximately $(N_1)_{60}$ = 22. For a fines content of 42 percent, the

threshold is about $(N_1)_{60} = 18$. These thresholds are indicated in Figures 55 and 56. The threshold for clean sands is $(N_1)_{60c} = 30$.

110. Less than 15 percent of the blowcounts (including partial drives) fell below the threshold value determined for average fines contents. However, since the empirical charts were derived for undisturbed alluvium, some inherent conservatism was considered to exist because these relations are applied to compacted saprolite.

Comparisons with Wing Dams

111. The $(N_1)_{60}$ and $(N_1)_{60c}$ values measured in the interface core materials were compared with corresponding values measured in the core materials at locations along the Wing Dams. This comparison is shown in Figures 57 and 58. The values of $(N_1)_{60c}$ shown in Figure 58 include partial drives (greater than 6 in.) and were computed using fines contents measured for individual samples.

112. It is apparent from Figures 57 and 58 that SPT values from interface zones generally are lower than SPT values from Wing Dam locations. The average value of $(N_1)_{60}$ for Wing Dams is 51 blows/ft whereas the average value of $(N_1)_{60c}$ for interface zones is 30 blows/ft. About 18 percent of the values of $(N_1)_{60c}$ for the interface zones fell below the threshold of 30 blows/ft. Less than 5 percent of the $(N_1)_{60c}$ values from other Wing Dam locations fell below the threshold value.

Conclusion

113. The core materials in the interface zone are composed of carefully placed and compacted saprolite. Results of field and laboratory investigations showed the core materials have the following characteristics:

a. The fines content averaged 26 percent in the RWD and 42 percent in the LWD.

b. The fines classified as clayey in 33 percent of the RWD interface core samples and in 71 percent of the LWD interface core samples.

c. The $(N_1)_{60}$ values range from 11 to about 66 and average about 30; the upper bound is not well defined due to partial penetrations.

114. The Seed performance-based approach to assess liquefaction potential indicated threshold $(N_1)_{60}$ values of 18 to 22 based on average fines

contents of core materials at interface zones. Regardless of the severity of earthquake shaking, values of $(N_1)_{60}$ that exceed the threshold levels indicate materials with very high cyclic strength that will not develop residual excess pore pressures. The field measurements in the interface core materials generally showed $(N_1)_{60}$ values well above the threshold levels. Values that fell below threshold levels represent materials that are likely to be of limited extent and do not pose a stability problem. The clayey materials are expected to perform particularly well under earthquake loading on the basis of excellent observed field performance of these materials under severe earthquake loads (Seed et al. 1978). In view of these studies, it is concluded that the core materials in the interface zones will perform satisfactorily with respect to liquefaction during the design earthquake event, and that the interface zones as a whole will perform satisfactorily during the design earthquake event.

PART V: CONCLUSIONS

115. This report documents the study of the seismic stability evaluation of the interface zones at the Folsom Dam and Reservoir Project, located on the American River, about 20 miles northeast of the city of Sacramento, California. The most severe earthquake shaking was determined to be likely to come from the East Branch of the Bear Mountains fault zone, which is considered capable of producing a maximum magnitude earthquake of M = 6.5. The minimum distance between the fault zone and the CGD at the Folsom Project is 15 km. The design ground motions for the site are $a_{max} = 0.35$ g , $V_{max} = 20$ cm/sec and duration (≥ 0.05 g) = 16 sec.

116. A feasibility study was performed to assess the practicality of using numerical methods to investigate the response of the interface zones. It was concluded that numerical methods now available were not useful for practical seismic stability decision making in this case because of the extremely complex geometry of the interface areas and the limitations of existing numerical methods. On account of the lack of observations of field performance of interface zones in other projects subjected to earthquakes and the technical difficulties listed above, the course of assessing safety in the RWD and LWD interface zones evolved into a more qualitative approach that consisted of the following steps:

- a. Examine interface geometry and fill materials.
- b. Anticipate potential modes of failure.
- c. Identify key elements that control stability.
- d. Study these elements to assess adequate performance.

117. The potential modes of failure that required study were narrowed to the three modes listed below:

- a. Cracking and separation of the embankment core and the CGD.
- b. Slope instability resulting in sufficient deformations to allow overtopping, either by sliding of the embankment shell to expose the core to erosion by the reservoir pool, or sliding involving liquefaction of the core materials.
- c. Liquefaction of the core, piping of liquefied core material into downstream shell and progressive development of internal erosion.

118. The key elements that control stability for these potential failure modes were identified as follows:

a. The presence of defensive design measures built into the project, in particular the large volume of cohesionless fill in shell and filter zones available to fill cracks if they occur.

b. The liquefaction resistance and post-earthquake strength of shell and transition gravels.

c. Retaining wall stability and stability of shell backfill if retaining walls slide or overturn.

d. The liquefaction resistance of core materials and the presence of downstream filter zones to prevent piping and progressive internal erosion.

119. Stability studies performed as part of the work documented in Reports 6 and 7 of this series indicated that, during and after the earthquake, adequate performance of the interface zones is controlled primarily by adequate performance of the gravel shell, filter and transition zones. This is due to the role these zones play in slope stability, defensive design, and prevention of internal erosion. Report 6 of this series documents the seismic stability evaluation of the Wing Dams. It was concluded in Report 6 that: (a) the Wing Dams would perform satisfactorily during the design seismic event, (b) the embankment shell, filter and transition gravels had more than adequate cyclic strength and would not develop significant residual excess pore water pressures, (c) the embankment shell, filter and transition materials would have more than adequate post-earthquake strength, and (d) no significant deformations would occur in the Wing Dams as a result of the design seismic event.

120. Another key element in the adequate seismic performance of the interface zones is the sensitivity of slope stability in this zone to movement or failure of the two downstream retaining walls and Upstream Retaining Wall B which protects the intake ports for the power plant in the RWD wrap-around area. Report 7 of this series documents the study of the retaining walls and the sensitivity of slope stability to retaining wall movement or overturning. It was concluded in Report 7 that: (a) none of the retaining walls will undergo sufficient seismically induced movement to result in slope instability, and (b) even complete failure and removal of the walls would not result in slope instability of the interface zones sufficient to allow loss of the pool.

121. The remaining key element that controls seismic stability of the interface zones is the potential for liquefaction of the core materials. This report examines in more detail the geometry and materials in the immediate

vicinity of the contact between the Wing Dams and the CGD, the procedures used during construction in this area, the results of field and laboratory investigations performed during construction and more recently as part of this study, with a view toward closer examination of liquefaction potential of the core materials in this zone. On the basis of review of construction information, the results of field and laboratory tests, comparison of measurements and materials from the interface zones with data from other locations at the Folsom Project, and observations drawn from dams of known seismic performance during moderate to severe earthquake shaking, it was concluded that the core materials in the interface zones are carefully placed and well compacted, and will not develop significant excess pore pressures if subjected to the design earthquake motions.

122. On the basis of these studies and those documented in Reports 6 and 7 of this series, it was concluded that the interface zones will perform satisfactorily during and immediately after the design earthquake, and that no remedial action of any kind is indicated for this project feature from a seismic stability viewpoint.

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Table 1

Materials and Borrow Areas for Wing Dams

Zone	Material	Use in Dams	Borrow Areas
A	Rockfill (10-30 percent minus No. 4)	RWD shell	Stockpiles 1, 2, 3, and 4 (American River channel excavation)
B	Alluvial gravel dredge tailings	RWD transition	Stockpile 7 Borrow Area No. 7 Borrow Area No. 8
C	Decomposed granite (SM)	RWD core	Stockpile 6 Borrow Area No. 2
E	Dredge tailings	LWD shell	Borrow Area No. 5 (Blue Ravine)
F	Dredge tailings (processed minus 2 in.)	LWD filter	Borrow Area No. 5
G	Decomposed granite (SM)	LWD core	Borrow Area No. 1

Note: RWD = Right Wing Dam.
LWD = Left Wing Dam.

Table 2

Compaction Equipment and Effort Required by Specification Document 1532 for Zoned Fill Materials

Zone	Sources(s)	Equipment	No. of Passes	Maximum Lift Thickness (inches)
A (shell)	Stockpiles 1, 2, 3, and 4	D-8 Cat. tractor	1*	48**
E (shell)	Borrow No. 5	D-8 Cat. tractor	1*	24
B (transition)	Borrow 37 and Stockpile No. 7	D-8 Cat. tractor	1*	24
F (transition)	Borrow No. 5 (processed minus 2 in.)	D-8 Cat. tractor	1*	12
C (core)	Borrow No. 2 and Stockpile No. 6	Sheepsfoot roller	12	12
		Pneumatic-tired roller	6	18
		Pneumatic tamper	--	4
G (core)	Borrow No. 1	Sheepsfoot roller	12	12
		Pneumatic-tired roller	6	18
		Pneumatic tamper	--	4

* "Coverage" rather than "Passes." Specification documents required one complete coverage with the D-8 Cat. tractor. It was estimated that one complete coverage with this equipment corresponds to approximately 4 passes.

** Later changed to 144 in. as per Engineering Daily Log dated 18 October 1952.

Table 3

Material Properties Used in Initial Design of Wing Dams

a. Soil Characteristics

Material	Impervious Core	Dredge Tailings	Foundation Zone "A"	Foundation Zone "B"
Dry wt lb/cu ft	123.4*	125.0	108.0	141.0
Moist wt lb/cu ft	134.0	133.0	117.1	149.7
Saturated wt lb/cu ft	140.0	143.8	130.4	151.2
Buoyed wt	77.6	81.4	68.0	88.8
Tangent ϕ	0.70	0.84	0.60	1.00
Cohesion lb/sq ft	0.0	0.0	0.0	0.0
Permeability ft/day	0.5		10.0	7.0

* At 95 percent modified A.A.S.H.O. density.

b. Summary of Stability Analyses

Slope	Condition	Force	Method	Min F.S.
Upstream slope	Reservoir empty Slope 1 on 2.25	Gravity	Infinite slope	1.89
	Submerged slope Slope 1 on 2.25	Gravity	Infinite slope	1.89
	Water surface elev. 427.0	Gravity	Circular arc	1.54
	Water surface elev. 466.0	Gravity	Circular arc	1.59
	Reservoir empty Slope 1 on 2.25	Gravity and 0.05 earthquake	Infinite slope	1.66
	Submerged slope Slope 1 on 2.25	Gravity and 0.05 earthquake	Infinite slope	1.51
	Water surface elev. 427.0	Gravity and 0.05 earthquake	Circular arc	1.26
	Water surface elev. 466.0	Gravity and 0.05 earhtquake	Circular arc	1.26
Downstream slope	Reservoir empty Slope 1 on 2	Gravity	Infinite slope	1.68
	Water surface elev. 466.0	Gravity	Circular arc	1.51
	Reservoir empty Slope 1 on 2	Gravity and 0.05 earthquake	Infinite slope	1.49
	Water surface elev. 466.0	Gravity and 0.05 earthquake	Circular arc	1.33

(Continued)

Note: The typical section shown on the next page defines the material zones referred to in Table 3a and the geometry used in Table 3b.

Table 3 (Continued)

TYPICAL SECTION

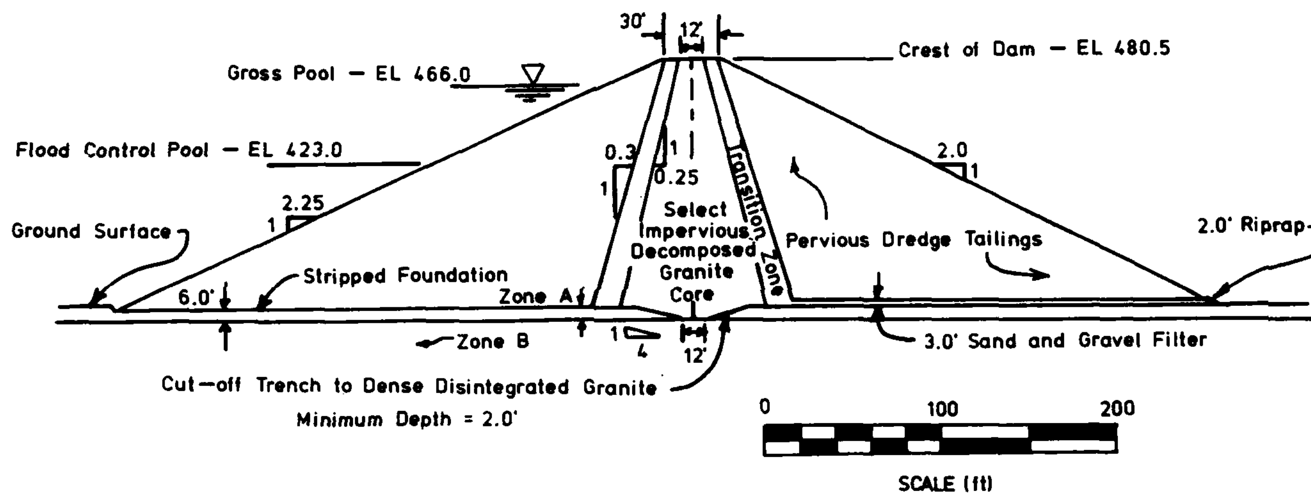


Table 4

Record Sample Testing During Construction

Sample	Elevation (ft)	Soil Type	Specific Gravity	Liquid Limit	Plastic Limit	Percentage of Gravel (%)	Percentage of Fines (%)	In Situ Dry Density (pcf)	In Situ Moisture Content (%)	Maximum Dry Density* (pcf)	Optimum Moisture Content* (%)
RWD (UE-25)	420	SM	2.69	23	21	3	18	131	8.0	134.2	8.4
RWD (UE-24)	384	SM-SC**	2.67	27	22	2	22	121	5.3	134.0	7.7
RWD (UE-23)	362	SM-SC†	2.75	23	17	2	30	131	8.0	134.8	6.8
RWD (UE-22)	††	SM	2.68	24	21	5	31	126	5.1	133.3	8.0
LWD (UE-12)	360	SM-SC**	2.70	27	22	3	27	129	10.4	135.8	7.3
LWD (TP-2)	353	SM-SC†	2.70	26	22	0	23	120	5.4	131.6	7.6
LWD (UE-14)	427	SM	2.79		NP	2	15	129	8.3	134.5	8.3
LWD (UE-13)	††	SC	2.73	24	18	0	24	130	9.4	136.8	7.6
LWD (UE-6)	††	SC	2.68	29	19	3	30	131	8.2	131.4	8.7
		Average	2.71	25	20	2	24	128	7.6	134.0	7.8

* Modified compaction test (AASHTO T99-49).
** Originally reported as SM.
† Originally reported as SC.
†† Not reported.
Does not include non-plastic (NP) soil.

Table 5

Drilling Operations at Interface with RWD

Centerline Station	Transverse Station	Hole No.	Date	Type of Sampling	Sampled Elevations	Equipment	Pool Elevations
285+32	10+05	R-1	Feb '84	Undisturbed	480.5 - 408.7	CME 75; 12 in. HSA*	435.1 - 435.7
285+29	10+05	R-2	Mar '84	Undisturbed	420.8 - 360.5	CME 550; 8 in. HSA*	442.0 - 443.2
285+32	9+95	R-3	Mar '84	SPT	480.5 - 431.5	CME 550; trip hammer; 8 in. HSA*	436.6 - 436.9
285+29	9+95	R-4	Mar '84	SPT	530.5 - 350.5	CME 550; trip hammer; 6 in. HSA*	434.9 - 436.4
285+23	9+95	7F-88-1	Sep '88	SPT	380.5 - 298.3	Mobile B-53; safety hammer; with winch mud-rotary	359.7 - 361.0

* HSA = hollow-stem augers with no drilling mud.

Table 6

Drilling Operations at Interface with LWD

Centerline Station	Transverse Station	Hole No.	Date	Type of Sampling	Sampled Elevations	Equipment	Pool Elevations
299+38	10+05	L-1	Feb '84	Undisturbed	480.5 - 431.5	CME 75; 12 in. HSA	434.9
299+43	10+05	L-2	Mar '84	Undisturbed	436.0 - 378.1	CME 550; 8 in. HSA	440.6 - 442.0
299+38	9+95	L-3	Feb '84	SPT	480.5 - 421.0	CME 550; safety hammer, rope-and-cathead; 8 in. HSA	434.9 - 435.1
299+43	9+95	L-4	Mar '84	SPT	430.5 - 369.0	CME 550; trip hammer; 8 in. HSA	436.9 - 439.1
299+47	9+95	7F-88-2	Sep '88	SPT	380.5 - 316.5	Mobile B-53; safety hammer with no winch; mud rotary	357.2 - 358.9

* HSA = hollow-stem augers with no drilling mud.

Table 7

Results of Recent Laboratory Tests of Samples in Interface Zones of RWD

Drill Hole	Depth (ft)	Soil Classification	Specific Gravity	Liquid Limit	Plastic Limit	Particle Percentage Gravel	Particle Percentage Fines
R-1	3.8-6.3	SM	2.70		NP	1	19
R-1	11.3-13.8	SC	2.70	25	16	1	27
R-3	12.8-17.7		2.74			11	13
R-1	13.8-16.5	SM-SW		23	21	2	11
R-3	17.7-19.5	SM	2.74	23	21	4	15
R-3	19.5-20.0		2.74			5	19
R-3	20.0-21.2	SC	2.74	29	21	2	23
R-1	20.8-23.3	SM	2.72	19	17	4	26
R-3	21.2-23.7	SM-SC	2.74	26	19	5	25
R-3	27.0-27.6	SM	2.74	28	23	2	21
R-3	28.0-29.3	SM-SC	2.74	28	21	0	24
R-3	29.3-32.6	ML	2.74	25	23	21	73
R-3	32.6-34.2	SM	2.74	21	18	2	29
R-3	34.2-39.5	SM	2.74	25	22	4	16
R-1	36.3-38.7	SM-SC	2.70	26	20	9	15
R-3	39.5-42.6	SM	2.74	25	23	15	15
R-1	46.2-49.1	SM-SC		22	16	3	21
R-3	46.5-47.0		2.73			0	24
R-3	47.0-49.0	SM-SC	2.73	25	20	2	26
R-4	54.0-54.5	SC	2.73	27	18	6	28
R-4	54.5-56.7	SC	2.71	28	19	11	20
R-4	64.0-67.5	SM-SC	2.71	23	19	9	22
R-4	67.5-68.6	SC	2.71	25	17	0	38
R-4	74.6-76.0	SM	2.71	23	20	6	16
R-4	76.5-78.4	SM	2.72	24	19	3	27
R-4	84.0-88.0	SM	2.72	24	22	5	20
R-2	89.6-92.3		2.73			1	21
R-4	94.0-94.8	SM	2.72	20	17	4	25

(Continued)

Table 7 (Concluded)

Drill Hole	Depth (ft)	Soil Classification	Specific Gravity	Liquid Limit	Plastic Limit	Particle Percentage Gravel	Particle Percentage Fines
R-4	94.8-98.0		2.73			3	21
7F-88-1	102.0-102.9	SC		26	19	5	22
R-4	104.0-105.5	SM	2.70	24	19	3	28
R-4	105.5-107.5	SM	2.70	22	19	2	29
R-2	107.2-109.7	SC		28	19	23	17
R-2	107.7-109.7					3	23
R-4	114.4-114.9	SM	2.70	22	20	5	25
R-4	114.9-115.5		2.70			14	21
7F-88-1	116.0-117.5	SC		26	19	9	26
R-4	116.6-117.5	SM	2.70	22	19	4	32
R-4	124.3-125.8	ML	2.70	22	19	2	90
R-4	126.7-127.2	SM	2.70	21	18	1	29
R-4	127.2-128.8	SM-SC	2.70	25	19	1	28
7F-88-1	132.0-132.5	SC		30	20	6	5
7F-88-1	162.0-162.5	SP			NP	0	4
7f-88-1	180.0-181.5	SC		29	20	0	25
Quantity of tests:			35.0	37	37	44	44
Sample average:			2.72	25	19	5	25
Sample standard deviation:			0.017	2.7	1.8	5.2	14.2

Table 8

Results of Recent Laboratory Tests of Samples in Interface Zones of LWD

Drill Hole	Depth (ft)	Soil Classification	Specific Gravity	Liquid Limit	Plastic Limit	Particle Percentage Gravel	Particle Percentage Fines
L-3	4.0-7.5	ML-CL	2.79	25	20	3	92
L-3	8.0-9.1	ML-CL	2.77	24	19	1	88
L-1	11.8-14.1	SM-SC		26	20	1	18
L-3	12.0-15.3	ML-CL	2.78	26	20	5	90
L-1	14.1-16.5	SC	2.75	30	19	3	33
L-3	16.0-17.3	CL	2.78	27	19	1	74
L-1	19.0-21.5	SM-SC		25	18	2	23
L-3	24.0-27.5	CL	2.78	27	18	1	85
L-1	24.0-26.9	SC	2.74	27	19	0	26
L-3	28.0-29.2	CL	2.78	29	20	1	89
L-1	29.1-36.6	SC		25	16	2	19
L-1	34.0-36.6	SM	2.80		NP	3	15
L-3	38.5-39.5	CL	2.78	25	17	2	91
L-3	40.0-43.5	CL-ML	2.78	23	17	2	94
L-3	48.9-51.5	CL-ML	2.78	24	20	2	91
L-4	49.8-54.0		2.75			5	15
L-3	52.1-52.6	ML	2.78	25	22	3	90
L-4	54.0-58.0	SM	2.75	25	22	5	17
L-2	54.6-57.9	SM		24	22	6	16
L-4	64.7-67.5	SC	2.75	24	19	6	16
L-4	67.0-68.0	SC	2.75	30	19	1	29
L-4	68.0-72.2	SC	2.75	28	20	2	22
L-4	72.2-73.0	SM-SC	2.75	26	19	3	21
L-4	73.0-73.5	SC	2.75	27	18	1	33
L-4	84.0-89.0	SM	2.78	25	22	3	19
L-2	87.3-89.6	SC	2.76	28	19	0	19
L-2	89.6-92.3	SC	2.76	28	19	2	22
L-4	92.5-94.0	SM	2.78	26	22	0	17

(Continued)

Table 8 (Concluded)

Drill Hole	Depth (ft)	Soil Classification	Specific Gravity	Liquid Limit	Plastic Limit	Particle Percentage Gravel	Particle Percentage Fines
L-4	94.0-99.5	SM-SC	2.78	25	21	2	20
L-4	99.0-104.0	SM	2.78	28	23	3	22
L-4	108.5-110.0	SM-SC	2.74	25	19	0	22
L-4	110.5-111.5					1	23
7F-88-2	120.0-124.0	SC		28	19	0	28
7F-88-2	138.0-140.0	SC		29	21	0	25
7F-88-2	154.0-156.0	SC		29	21	0	23
7F-88-2	162.0-164.0	SC		29	22	4	24
Quantity of tests:			28.0	34	34	36	36
Sample average:			2.77	26	20	2	40
Sample standard deviation:			0.017	1.9	1.7	2.0	31

Table 9

Results of Recent Density Testing of Undisturbed Samples of Core Material from RWD and LWD Interface Areas

Drill Hole	Depth (ft)	Soil Classification	Dry Density (pcf)	Moisture Content (%)	Moist Density (pcf)	Degree of Saturation, (%)
R-1	13.8- 16.5	SM-SW	110.1	9.4	120.7	47
R-1	16.5- 18.3	--	129.4	6.8	138.2	59
R-1	31.2- 33.7	--	122.3	11.8	136.7	83
R-1	49.1- 51.6	--	126.0	10.0	138.6	78
R-1	54.6- 57.9	SM	123.3	10.7	136.5	77
R-2	64.7- 67.2	--	127.2	10.3	140.3	84
R-2	84.7- 87.2	--	127.3	9.5	134.9	78
R-2	89.6- 92.3	SM	112.4	9.6	123.2	51
R-2	99.8-102.4	--	124.1	10.4	137.0	77
L-1	34.0- 36.6	SM	130.1	8.5	141.2	76
L-2	54.6- 57.9	SM	119.7	10.4	132.1	68
L-2	57.1- 59.6	SM	129.9	9.7	142.5	86
L-2	72.1- 74.6	--	129.4	9.3	141.4	81
L-2	89.6- 92.3	SC	120.3	10.9	133.4	72
L-2	92.3- 94.8	--	129.8	9.5	142.1	84
Sample range =			110.1 to 130.1	6.8 to 11.8	120.7 to 142.5	47 to 86
Sample average =			124.1	9.8	135.9	73
Sample standard deviation =			6.3	1.1	6.5	12.1

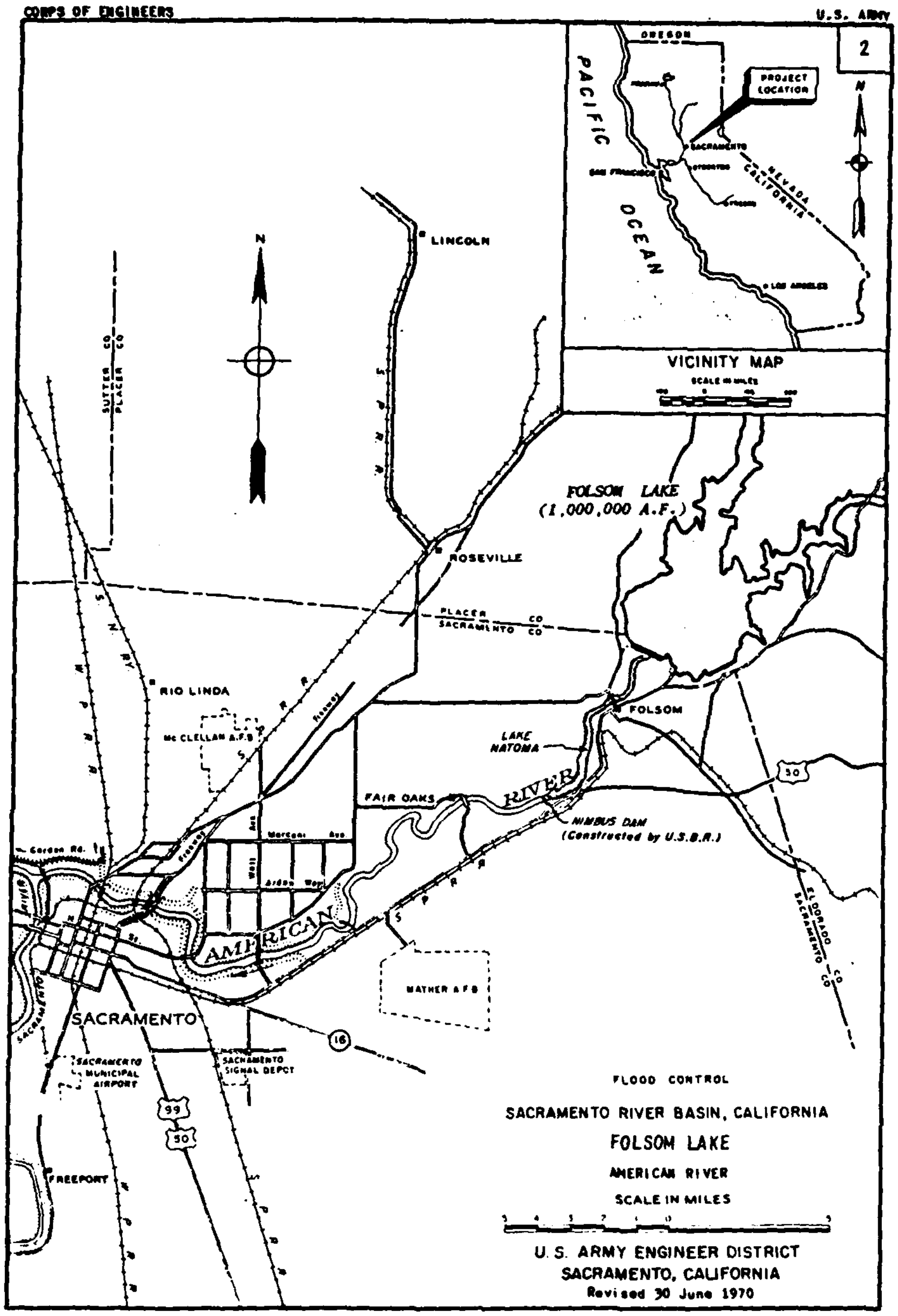
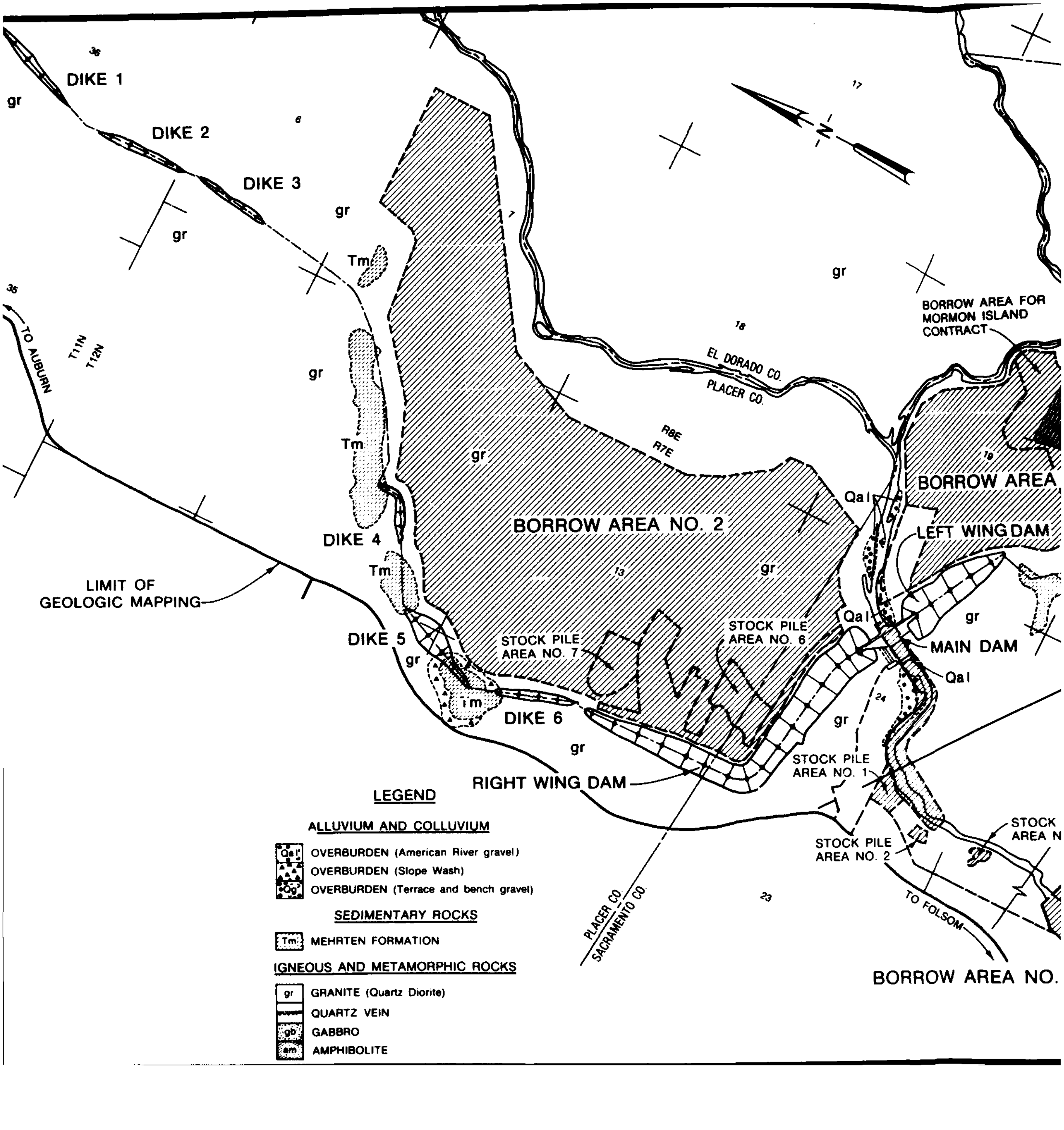


Figure 1. Location of Folsom Dam and Reservoir Project


DIKE 1
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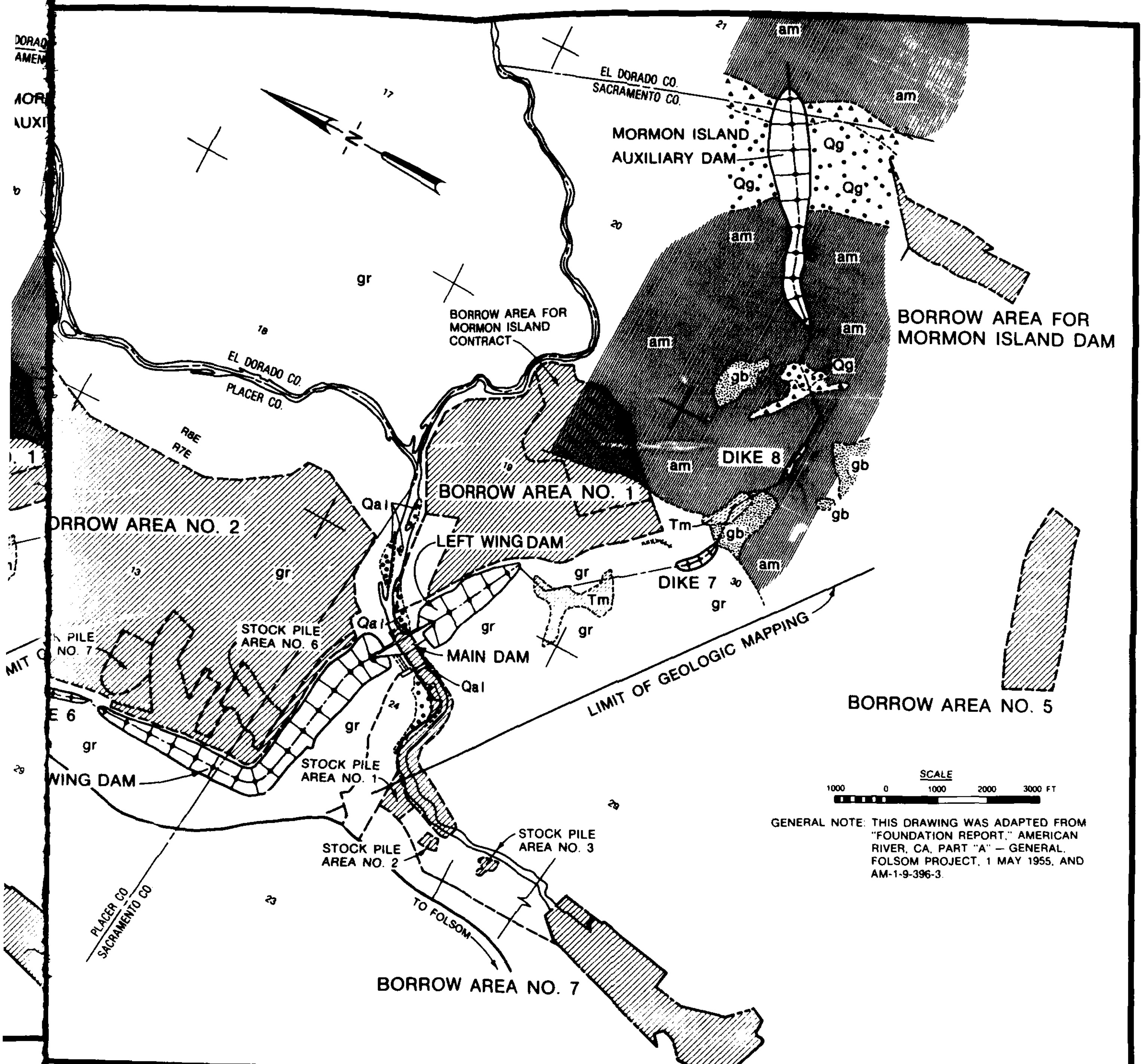
GEOLOGIC MAPPING
EL DORADO CO.
PLACER CO.
R8E
R7E
BORROW AREA NO. 2
BORROW AREA FOR
MORMON ISLAND
CONTRACT
BORROW AREA
LEFT WING DAM
MAIN DAM
Qal
STOCK PILE
AREA NO. 7
STOCK PILE
AREA NO. 6
STOCK PILE
AREA NO. 1
STOCK PILE
AREA NO. 2
RIGHT WING DAM
PLACER CO.
SACRAMENTO CO.
TO FOLSOM
BORROW AREA NO.
LEGEND
ALLUVIUM AND COLLUVIUM
Qal
OVERBURDEN (American River gravel)
OVERBURDEN (Slope Wash)
Qg
OVERBURDEN (Terrace and bench gravel)
SEDIMENTARY ROCKS
Tm
MEHRTEN FORMATION
IGNEOUS AND METAMORPHIC ROCKS
gr
GRANITE (Quartz Diorite)
QUARTZ VEIN
gb
GABBRO
am
AMPHIBOLITE



Layout of Folsom Dam and Reservoir amid surface geology and locations of borrow sources

2 of 3

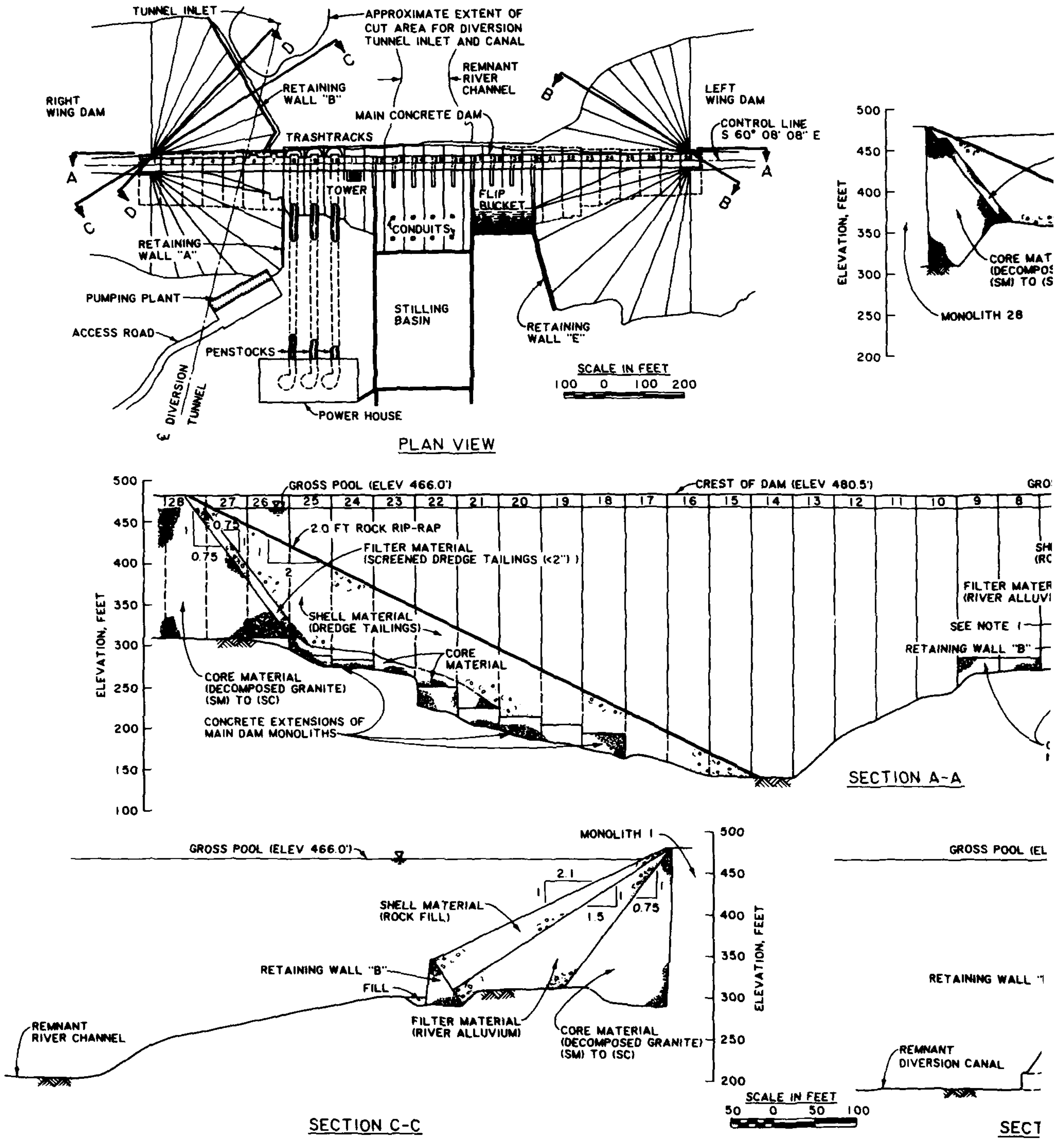
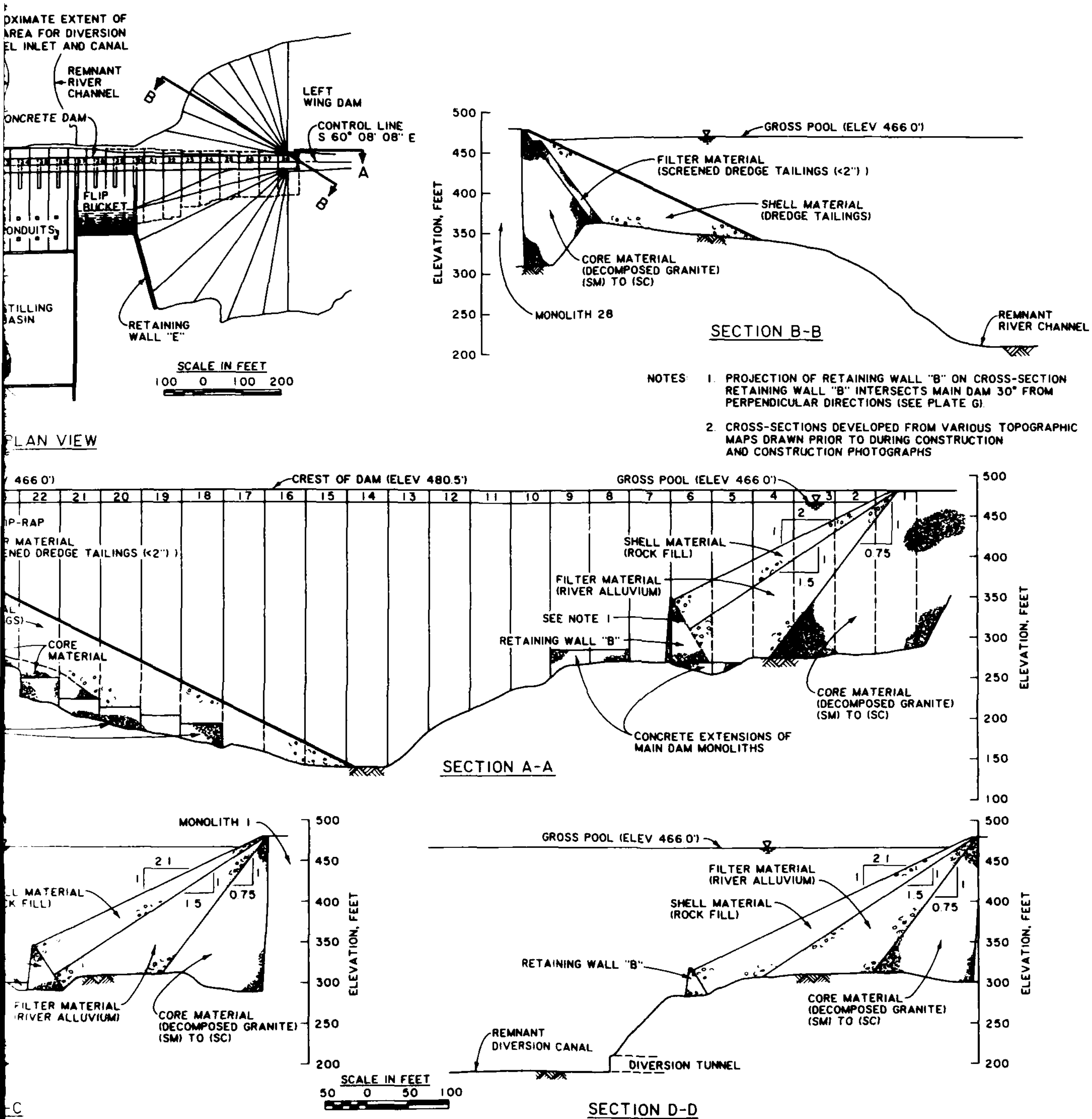


Figure 3. Plan and sections of upstream envelopment areas and Re



lan and sections of upstream envelopment areas and Retaining Wall B

2 of 2

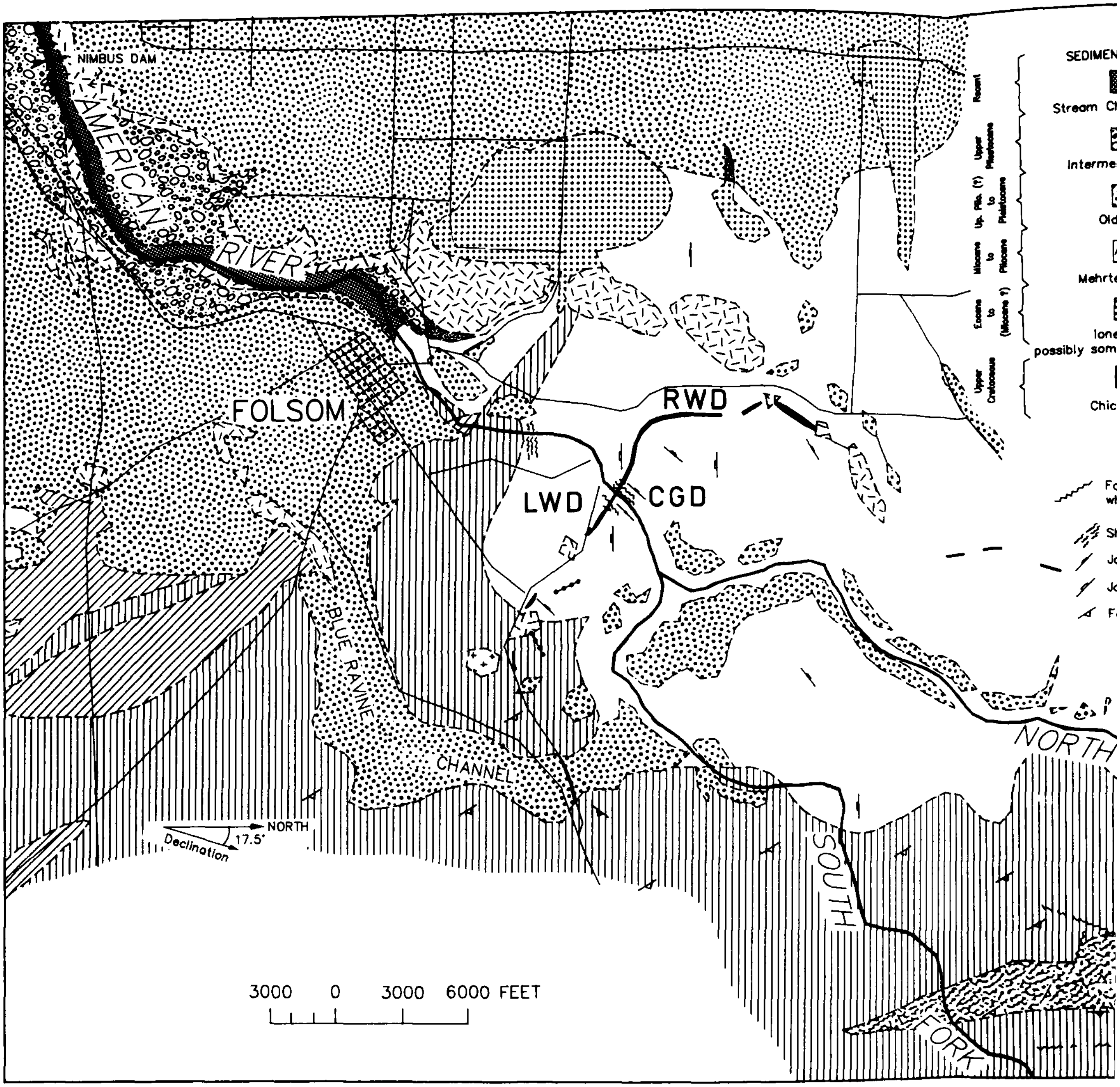
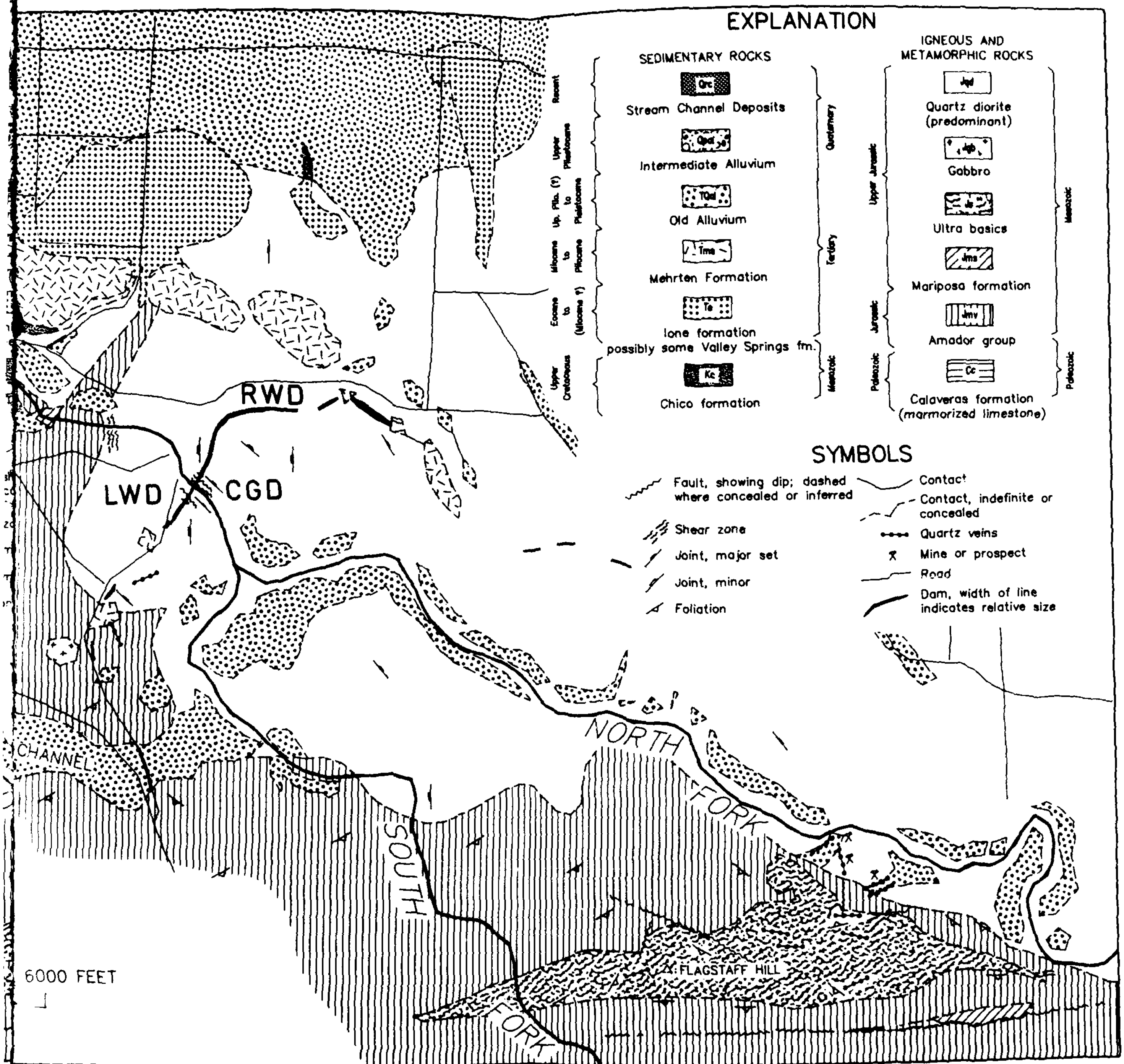


Figure 4. Geologic map, parts of the Folsom and Auburn quadran



re 4. Geologic map, parts of the Folsom and Auburn quadrangles

2 of 2

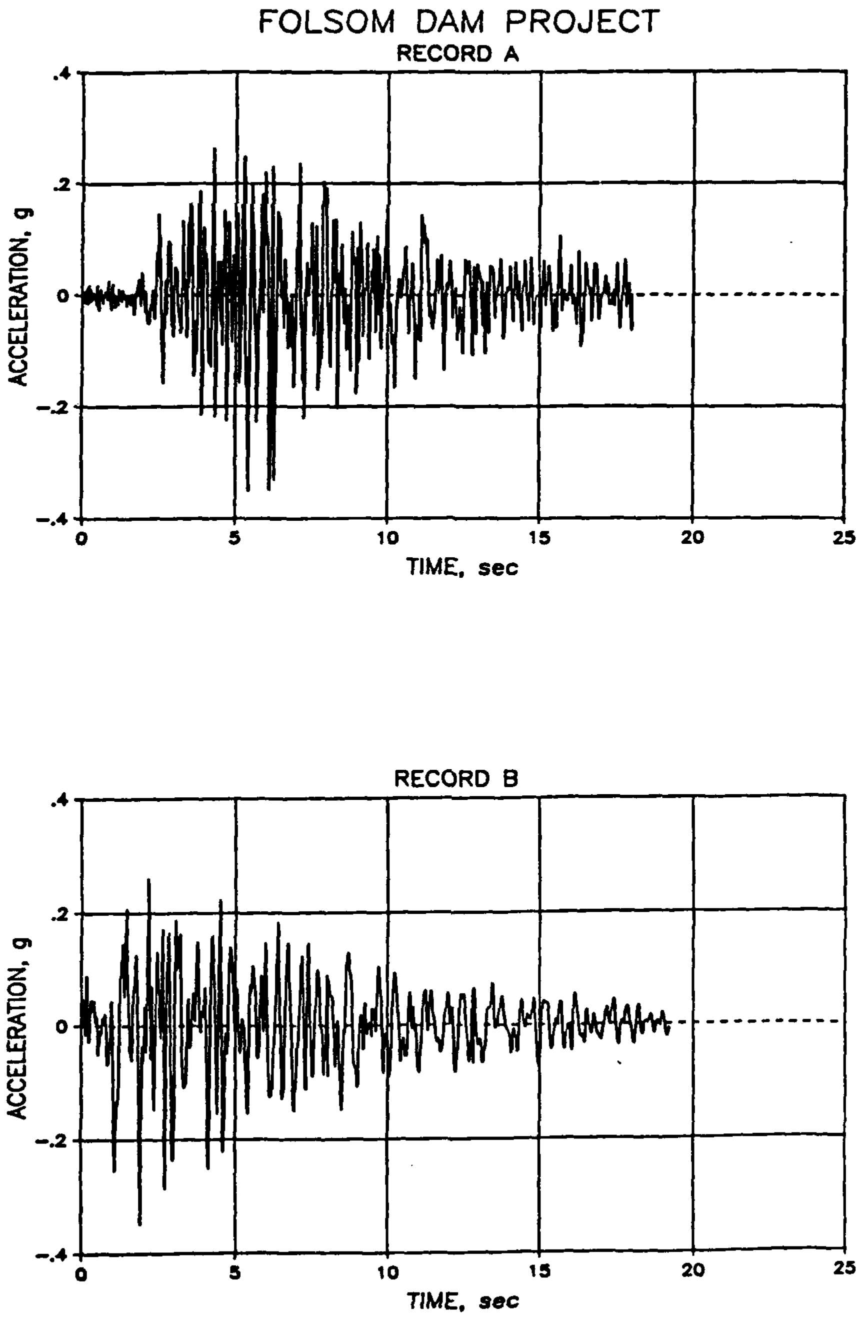


Figure 5. Acceleration time histories used in the analysis

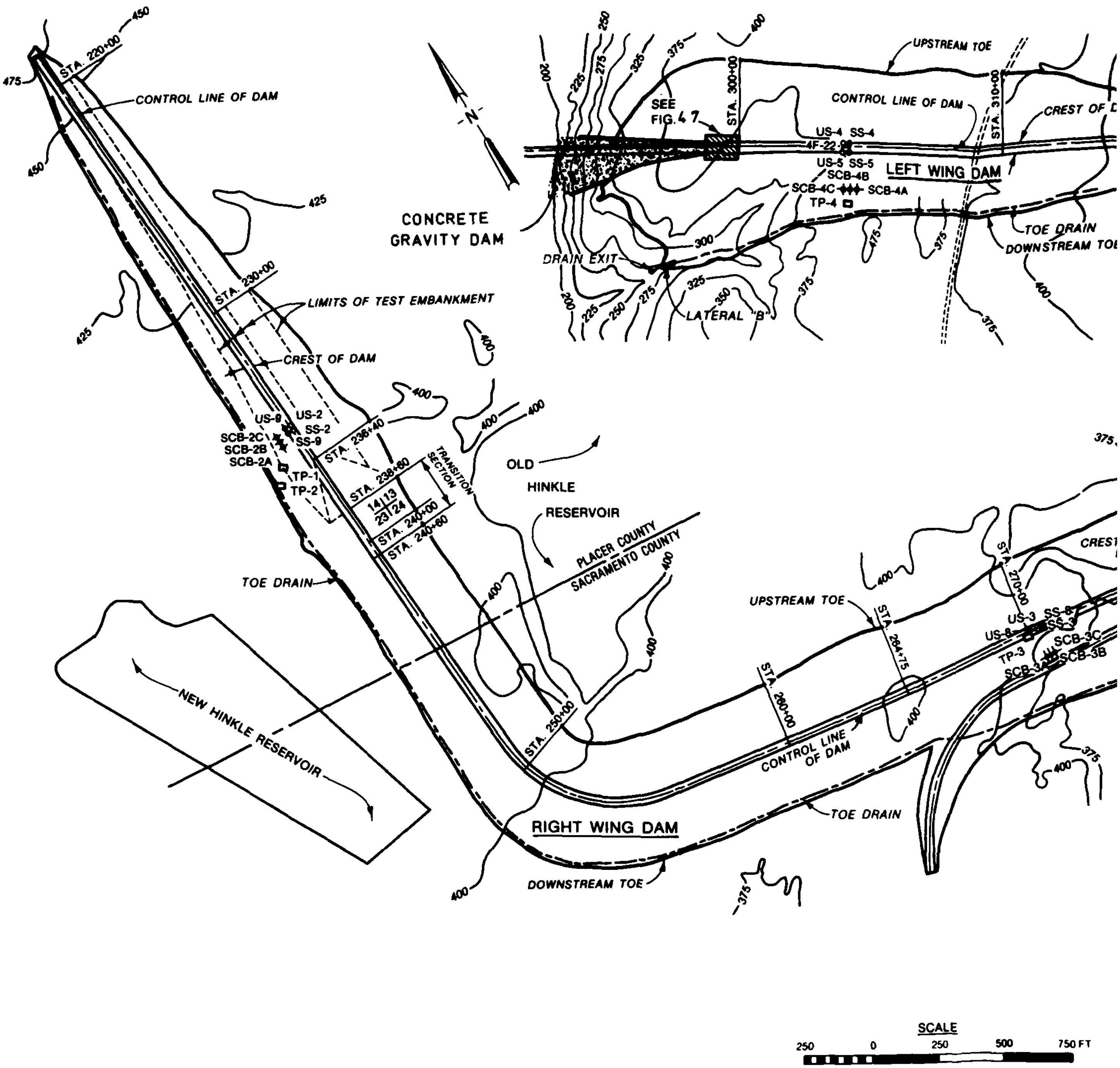
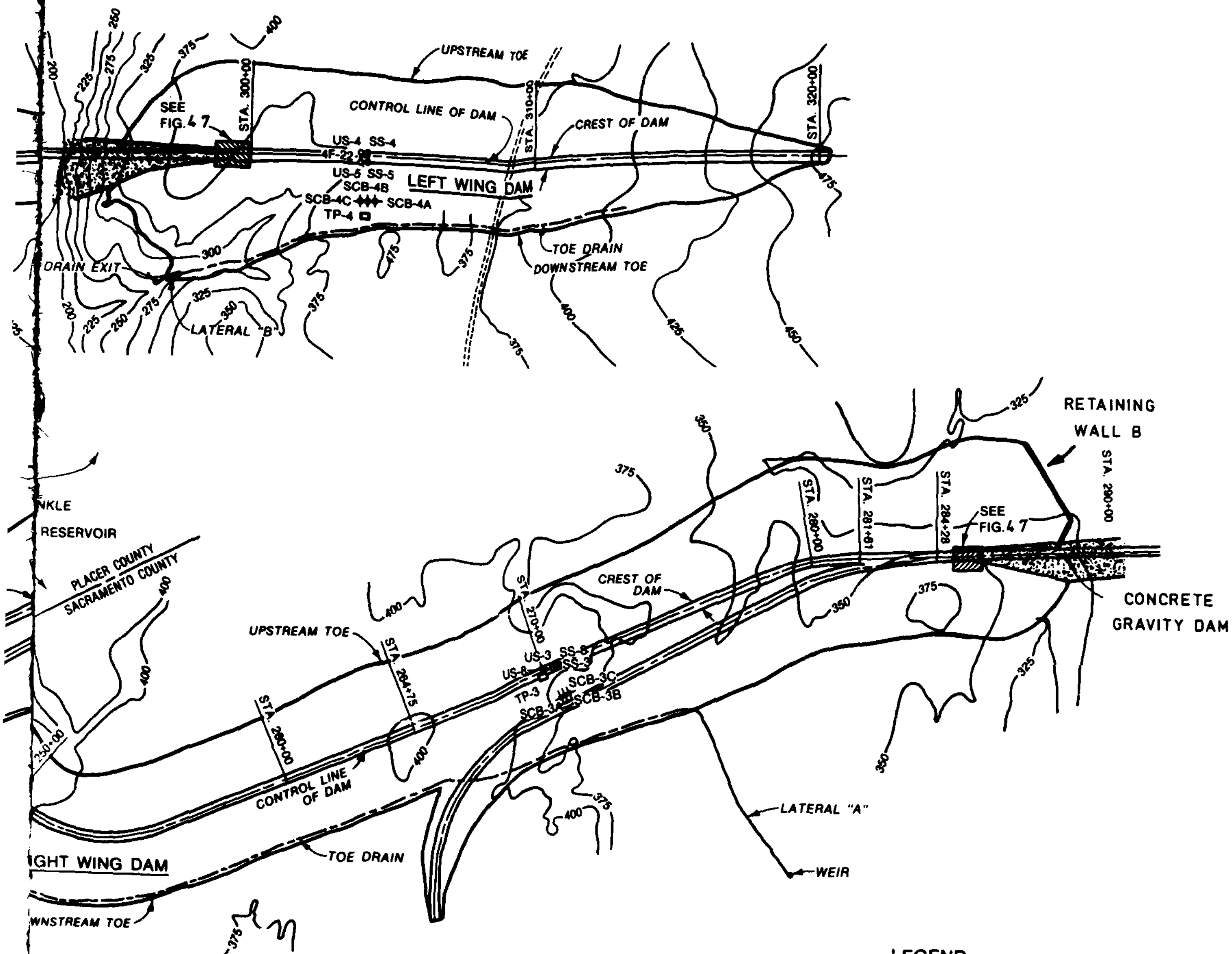


Figure 7. Plan view of Right and Left Wing Dams and detailed plan



LEGEND

- TESTS PITS MADE IN EMBANKMENT
- HOLES MADE THROUGH EMBANKMENT-UNDISTURBED SAMPLES OBTAINED
- HOLES MADE THROUGH EMBANKMENT-SPT AND DISTURBED SAMPLES OBTAINED
- HOLES MADE THROUGH EMBANKMENT-CROSSHOLE MEASUREMENTS

ew of Right and Left Wing Dams and detailed plan view of interface areas

2 of 2

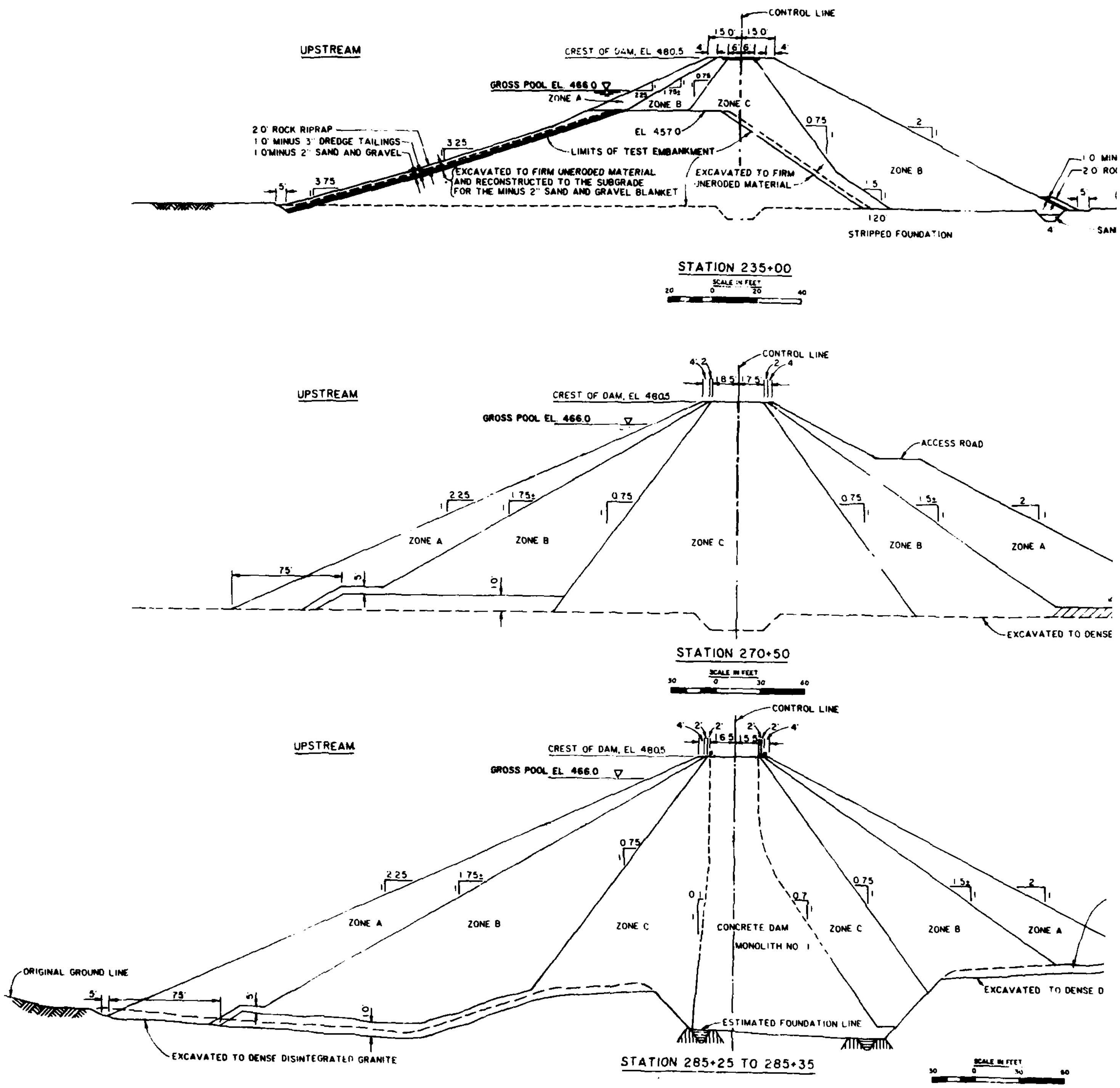


Figure 8. Typical cross-sections of RWD

1 of 2

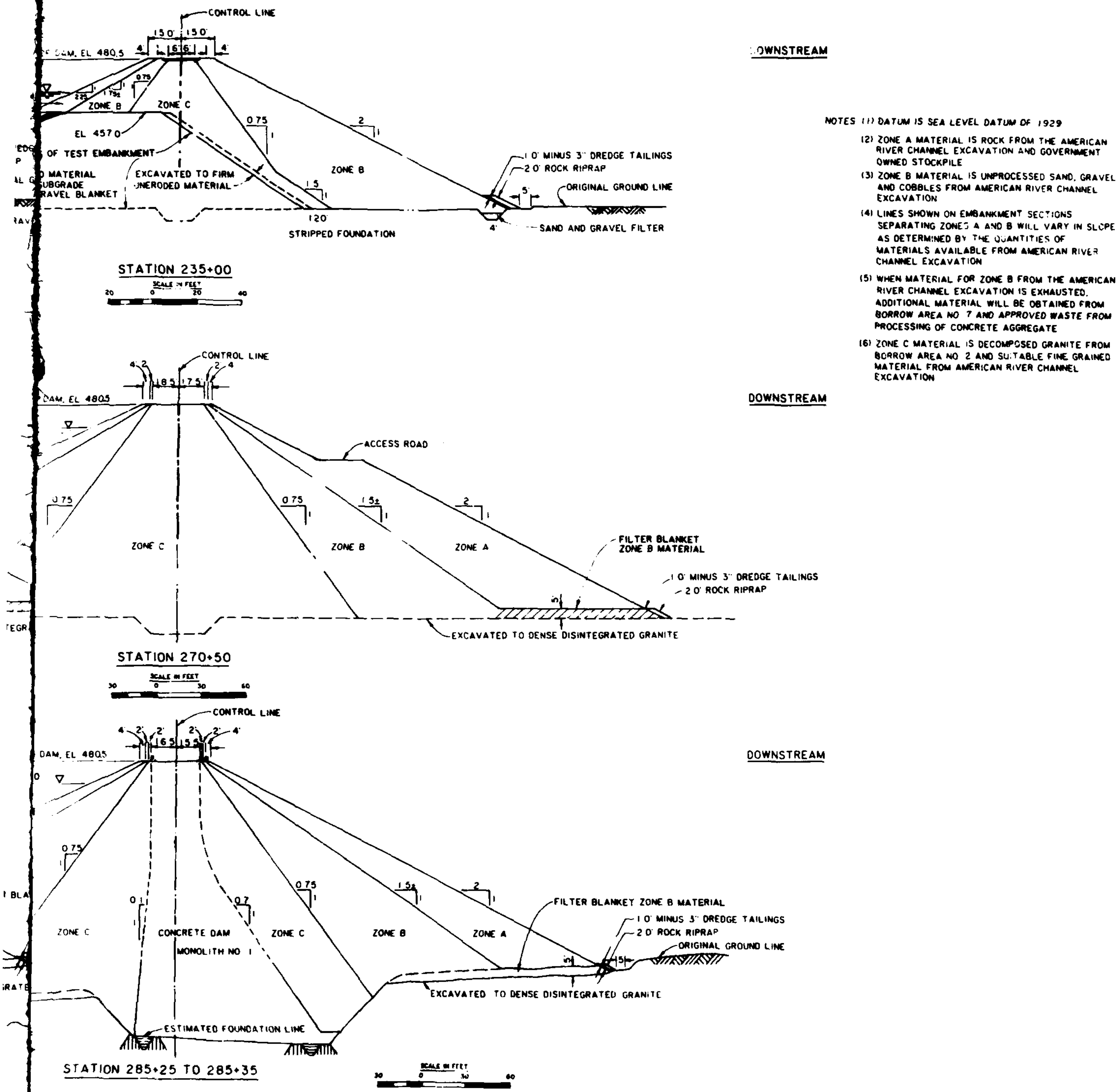


Figure 8. Typical cross-sections of RWD

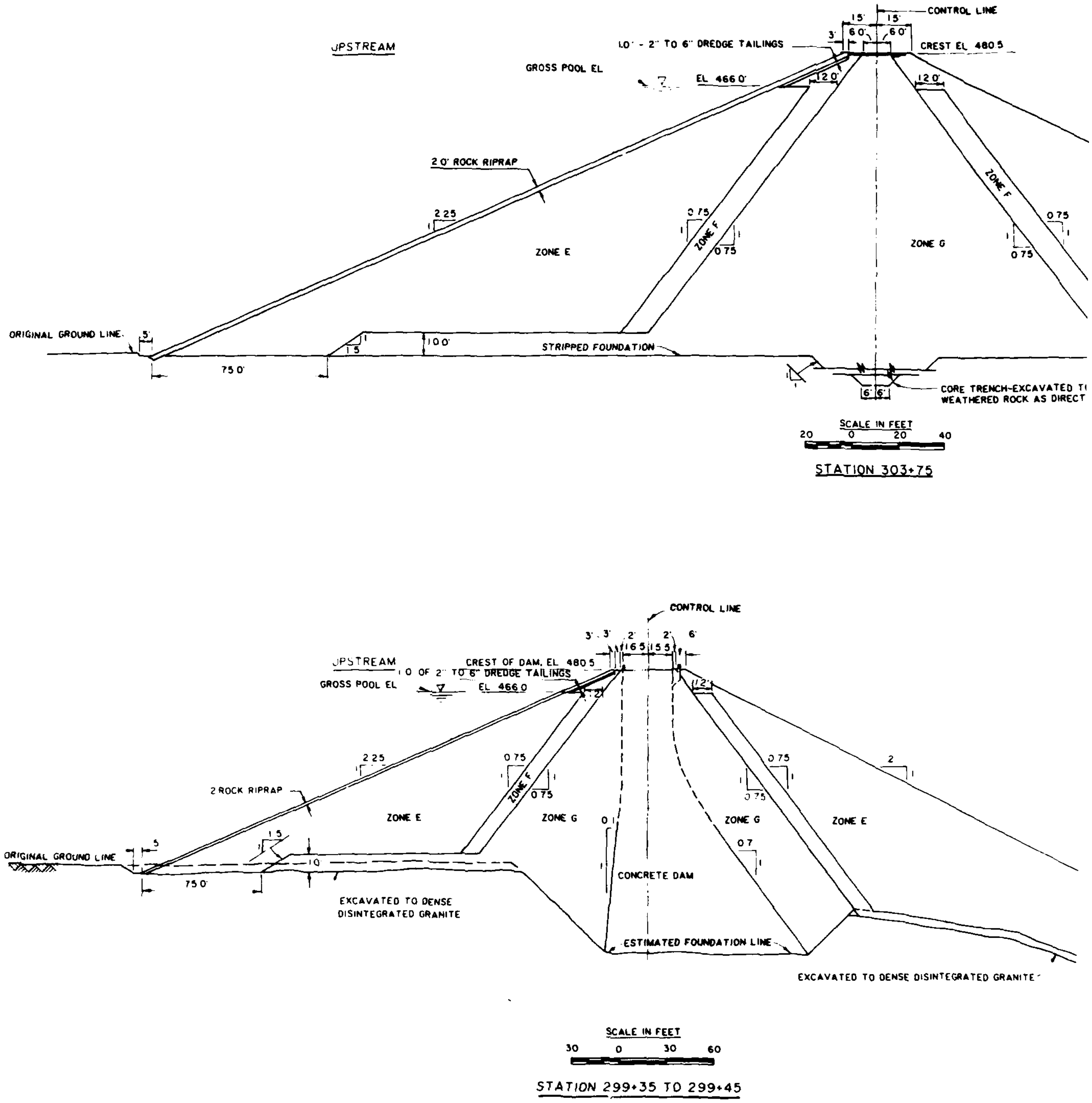
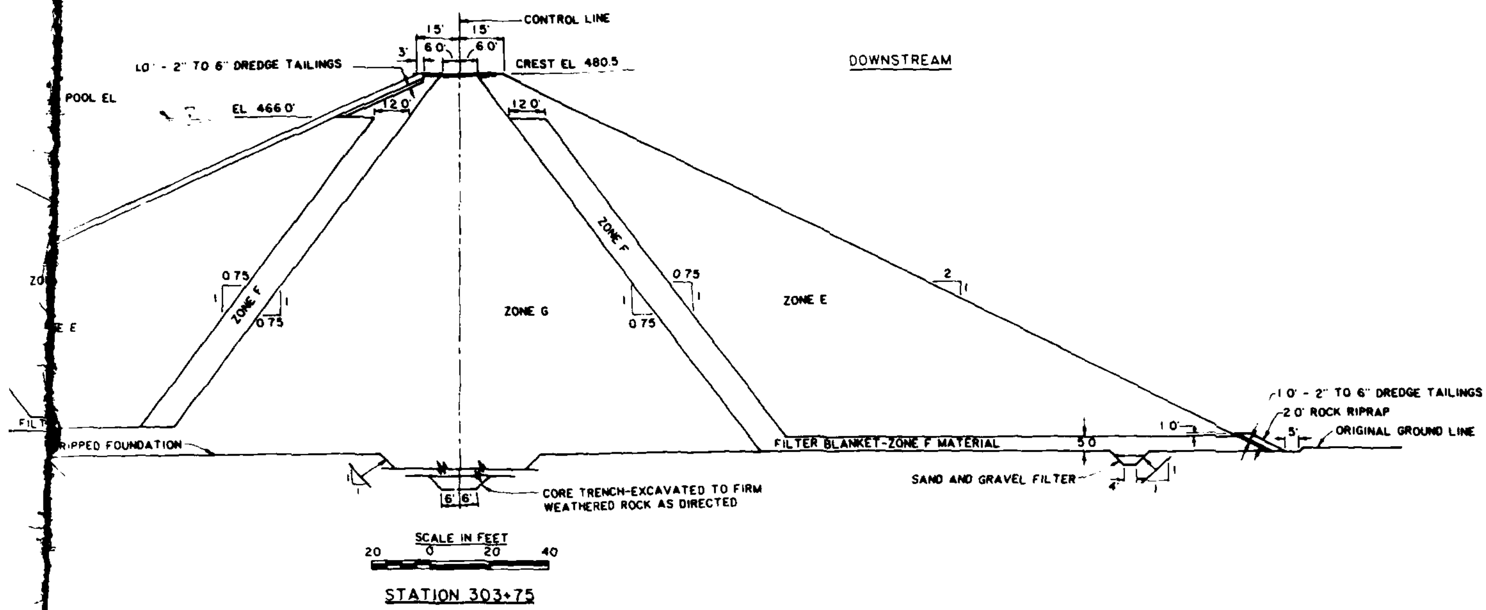


Figure 9. Typical cross-sections of LW

1 of 2



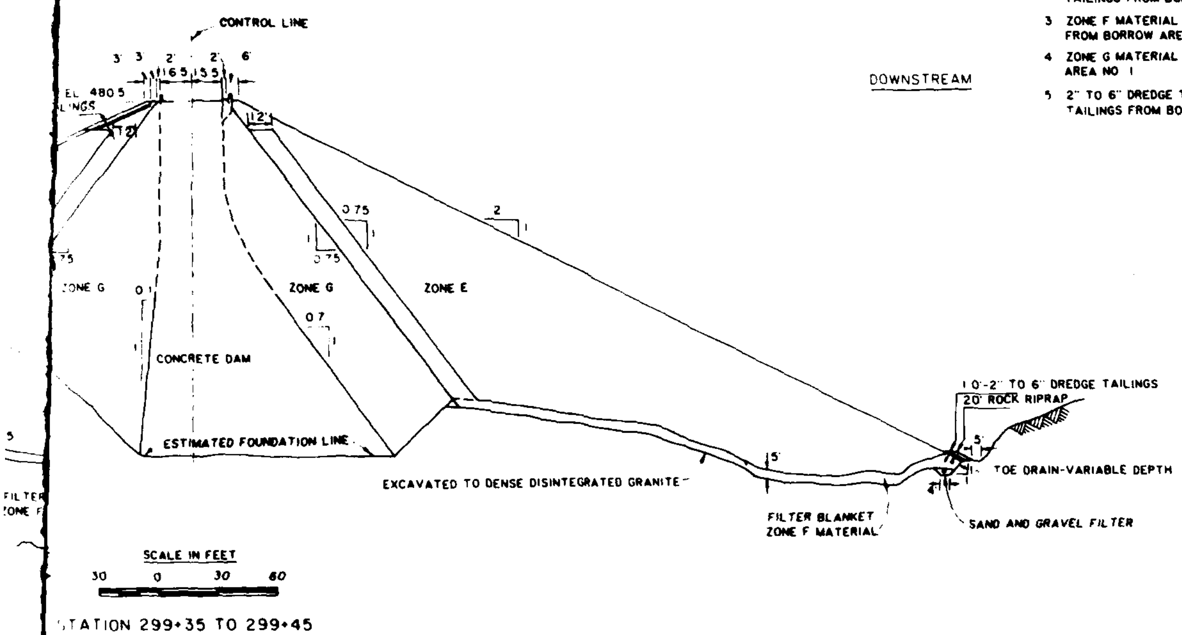


Figure 9. Typical cross-sections of LWD

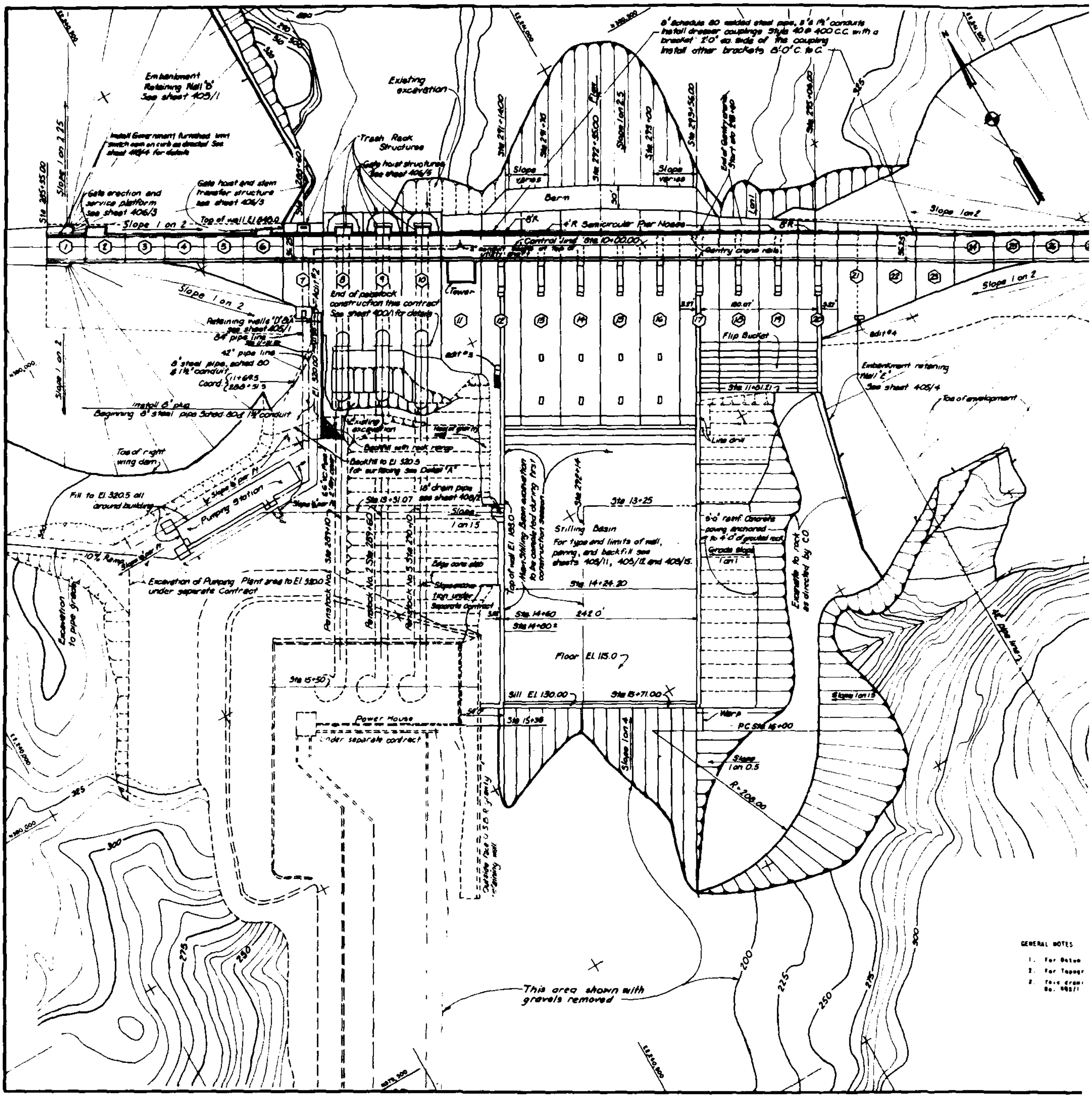


Figure 10. Plan of Concrete Gravity Dam

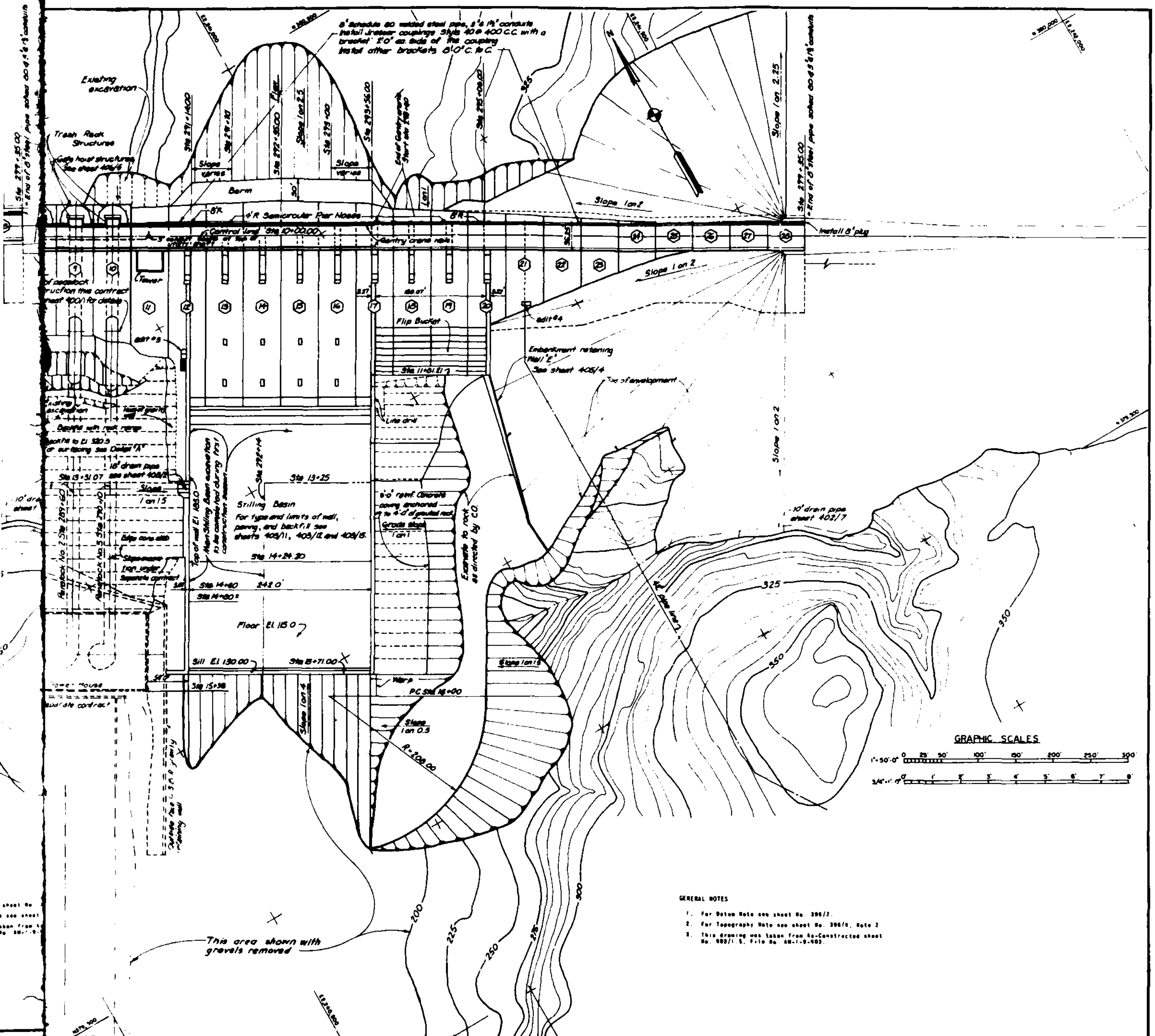


Figure 10. Plan of Concrete Gravity Dam

3 of 3

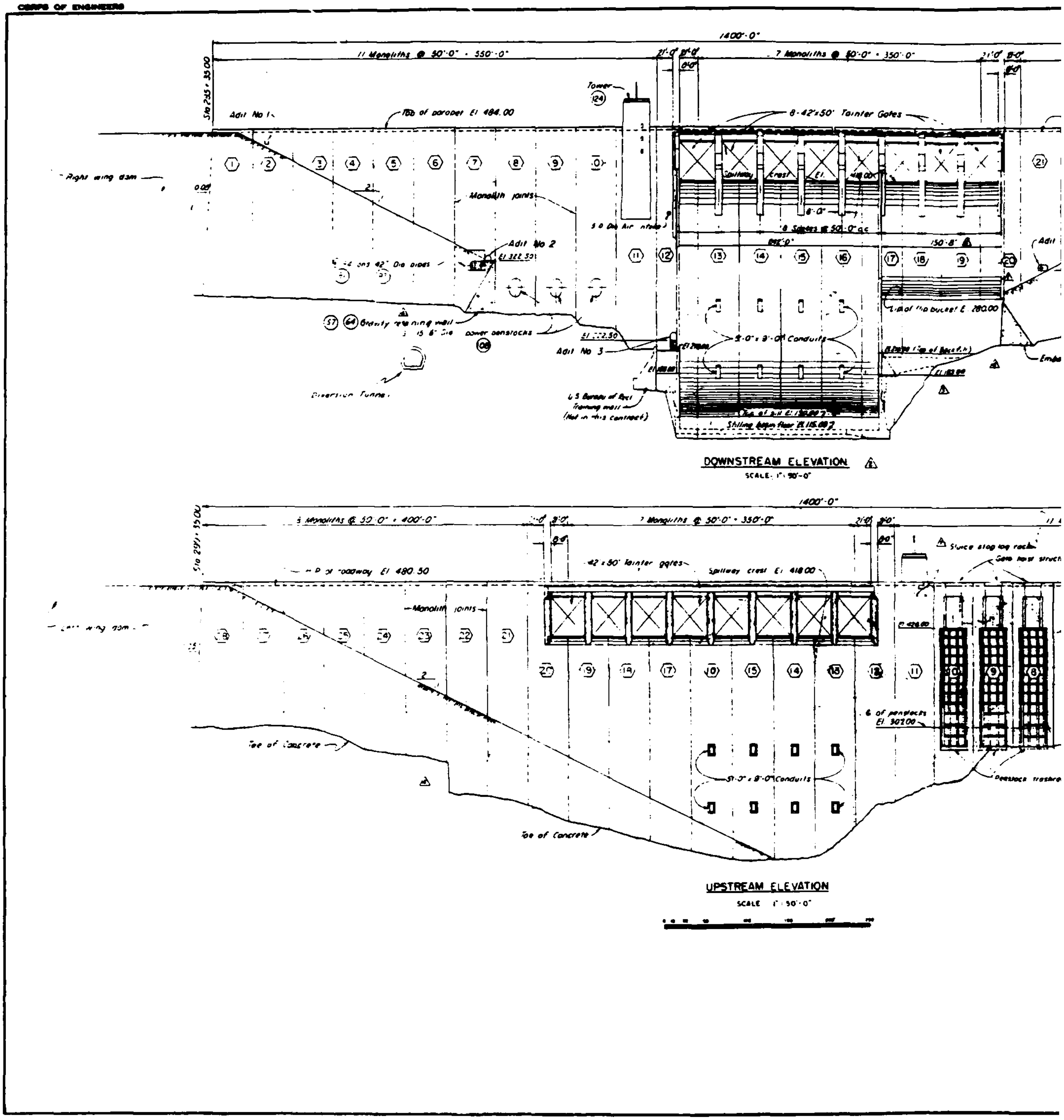


Figure 11. Concrete gravity dam elevations

1of 2

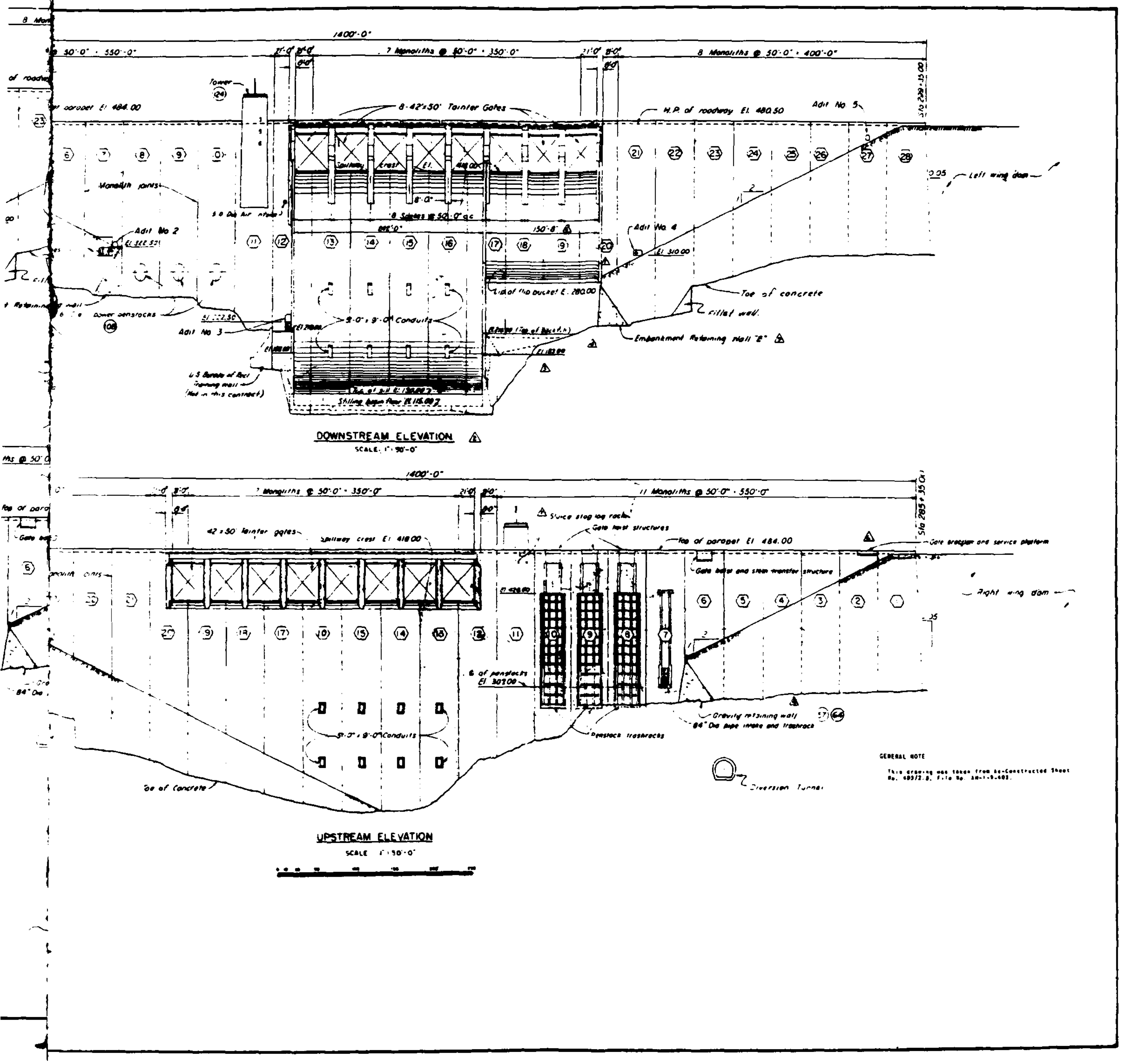


Figure 11. Concrete gravity dam elevations

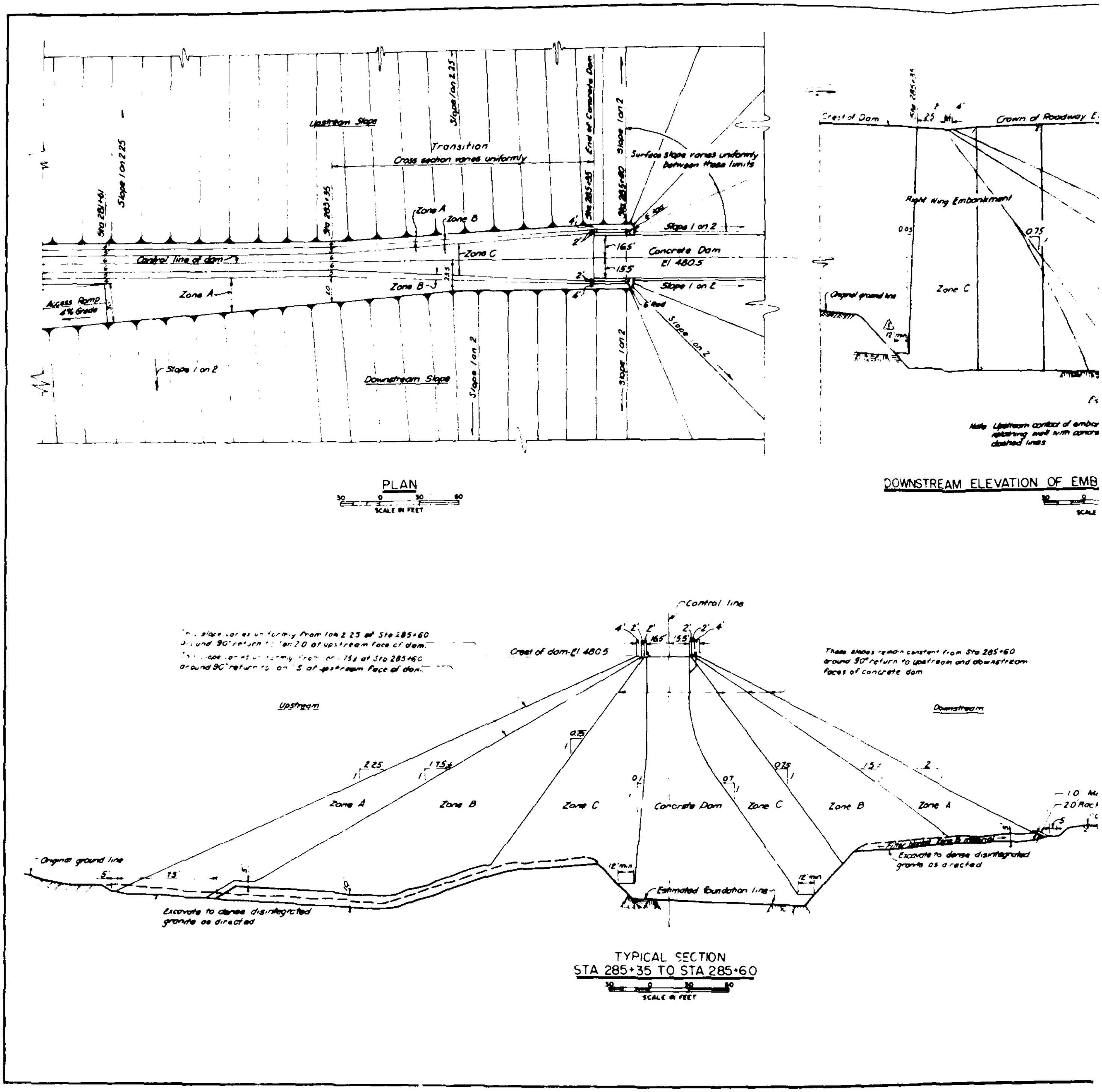
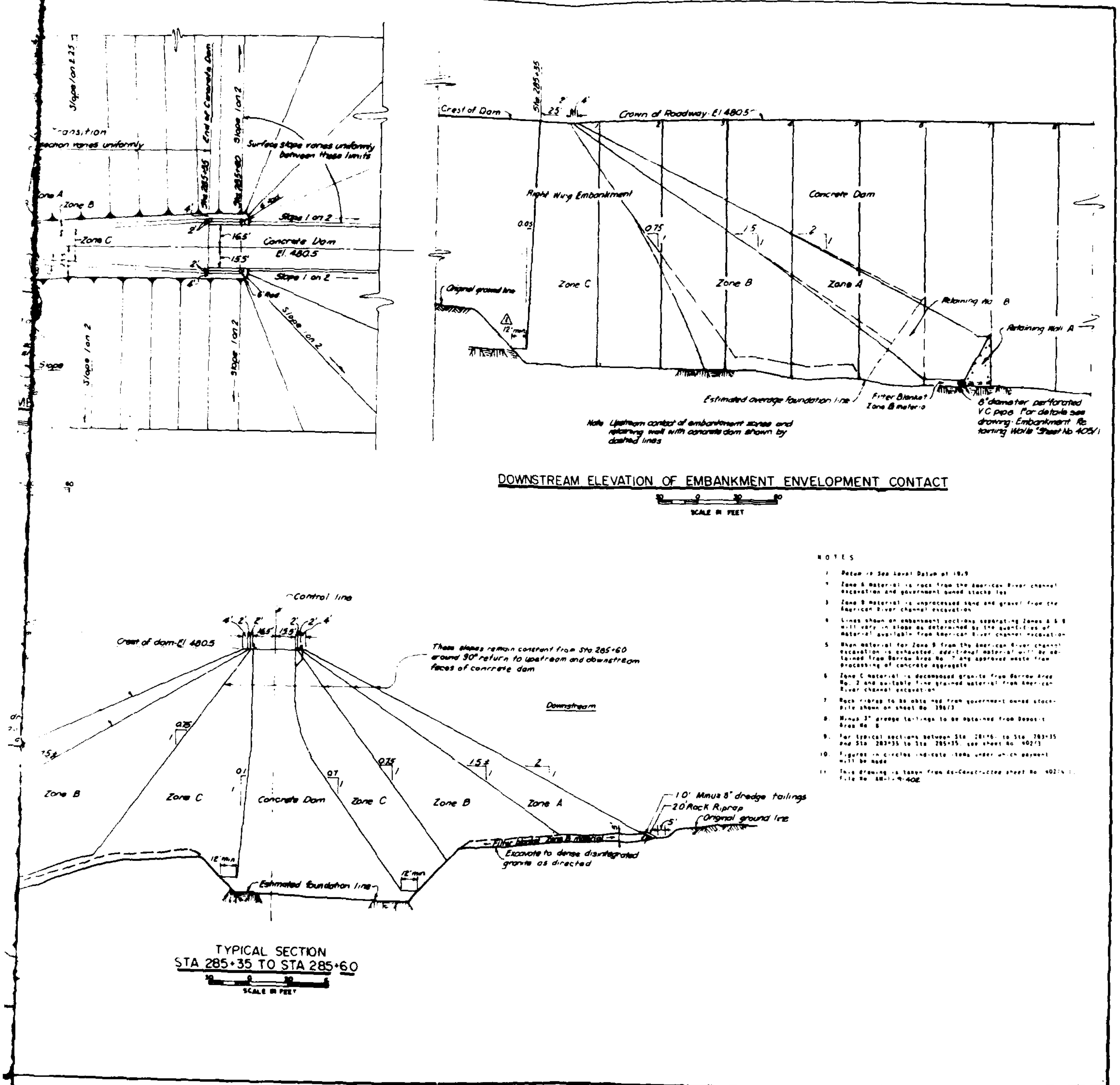


Figure 12. Plan and sections of Right Wing D[illegible] interface area



igure 12. Plan and sections of Right Wing Dam interface area

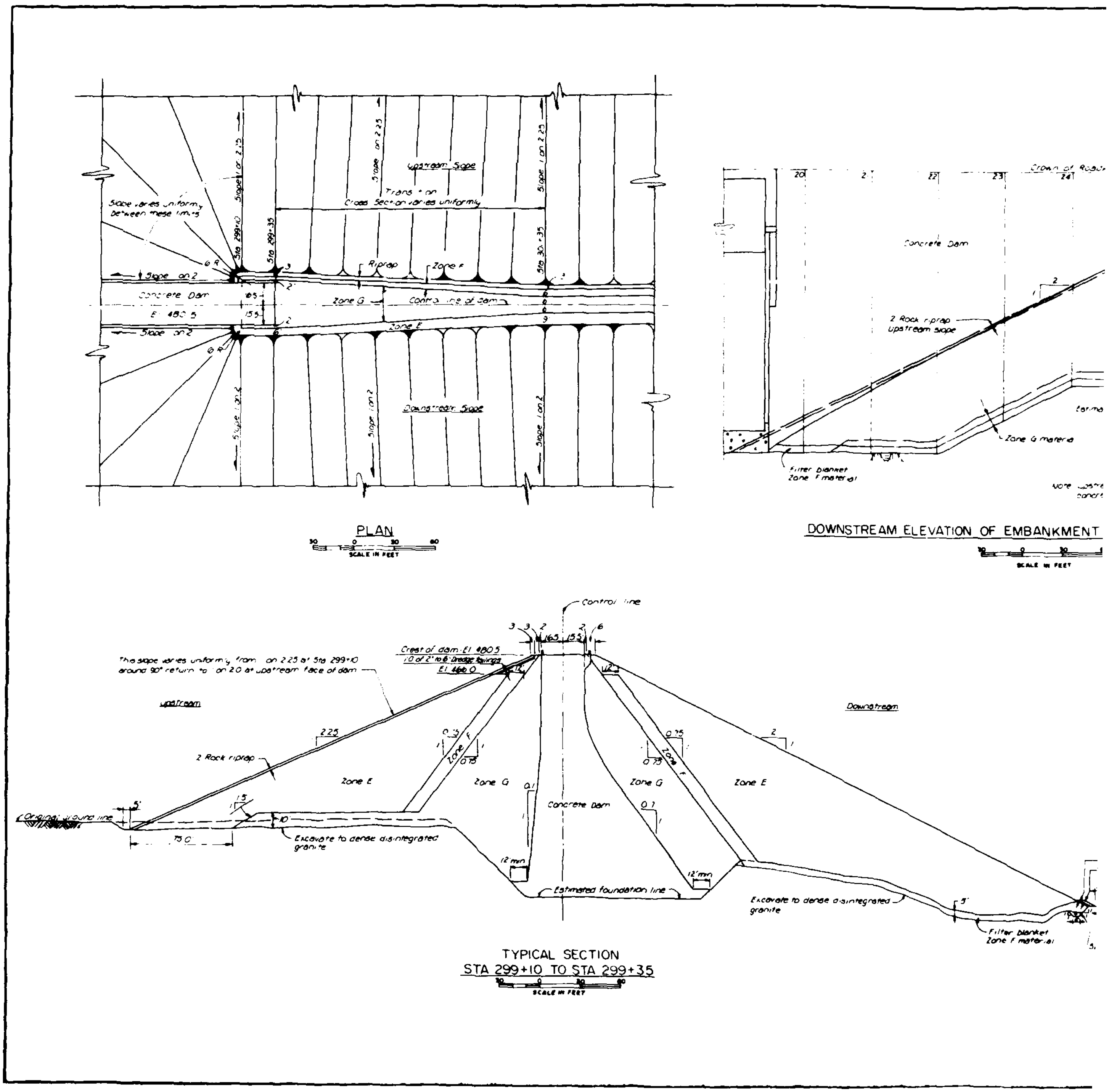
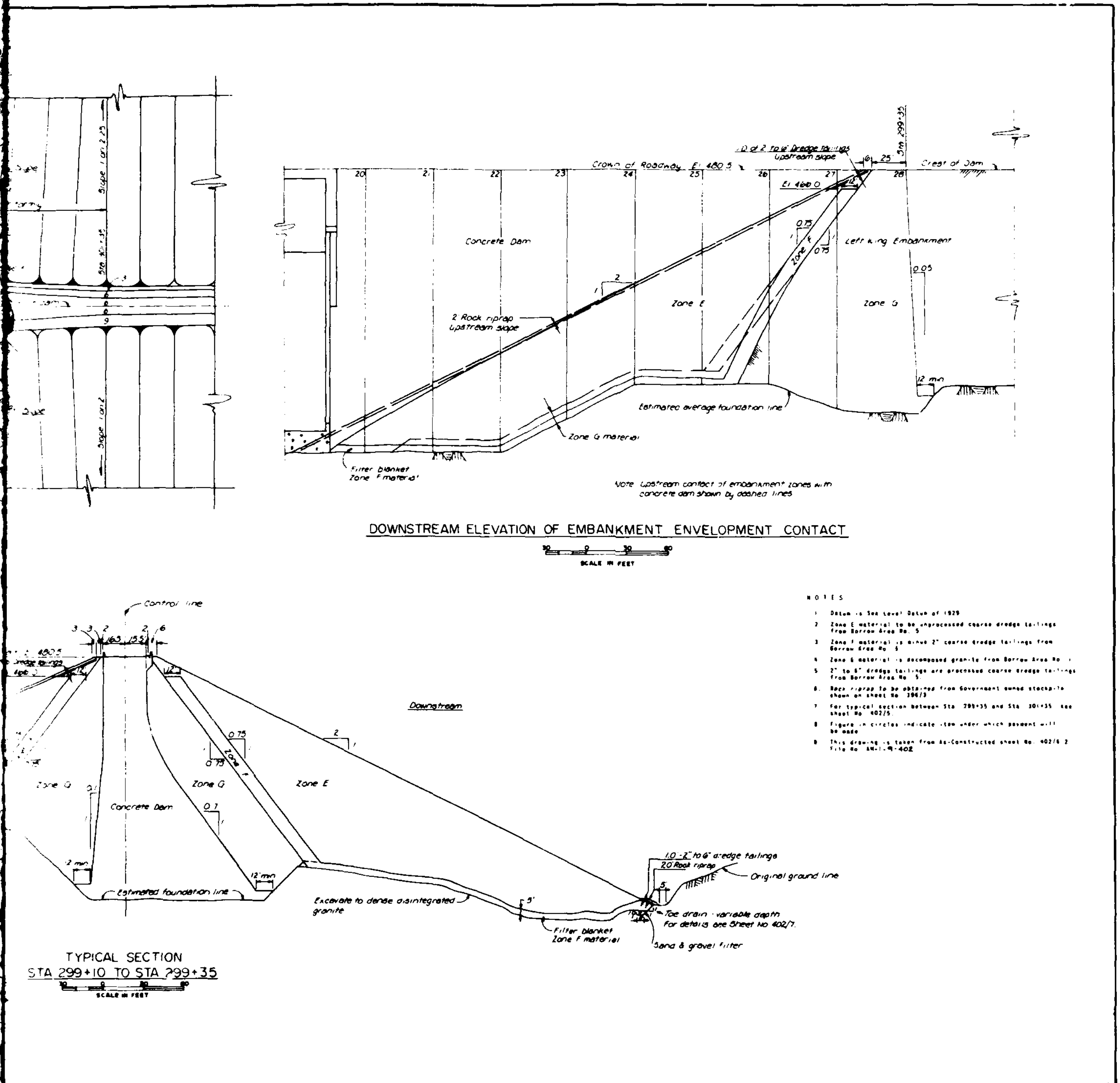


Figure 13. Plan and sections of Left Wing Dam interface ar



gure 13. Plan and sections of Left Wing Dam interface area

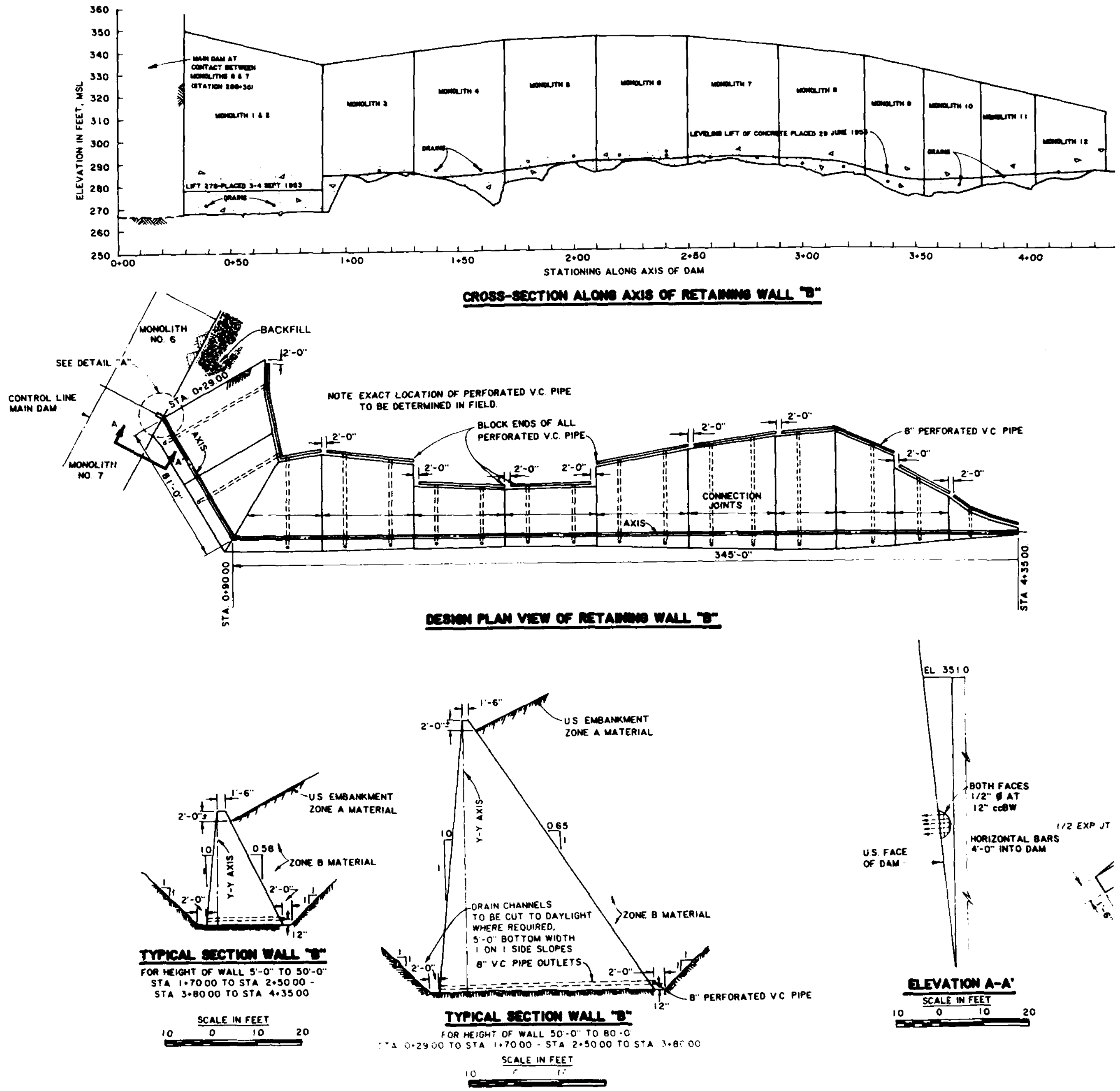
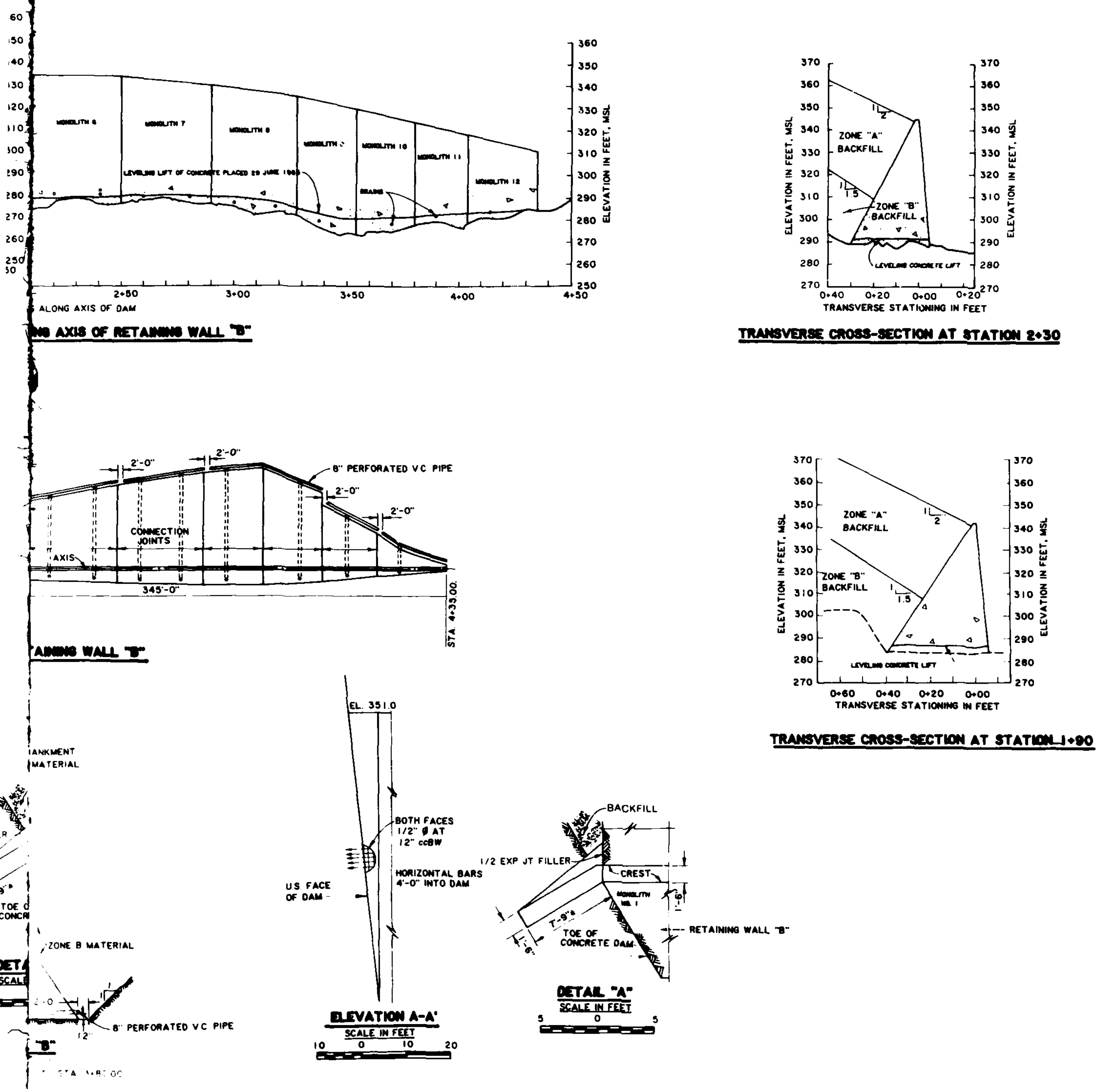


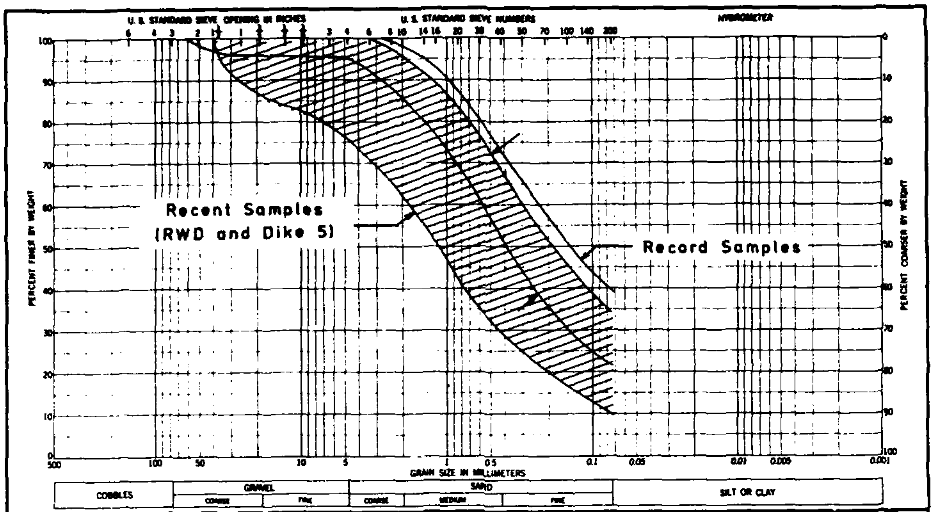
Figure 14. Plan and sections of Retaining Wall

1 of 2



re 14. Plan and sections of Retaining Wall B

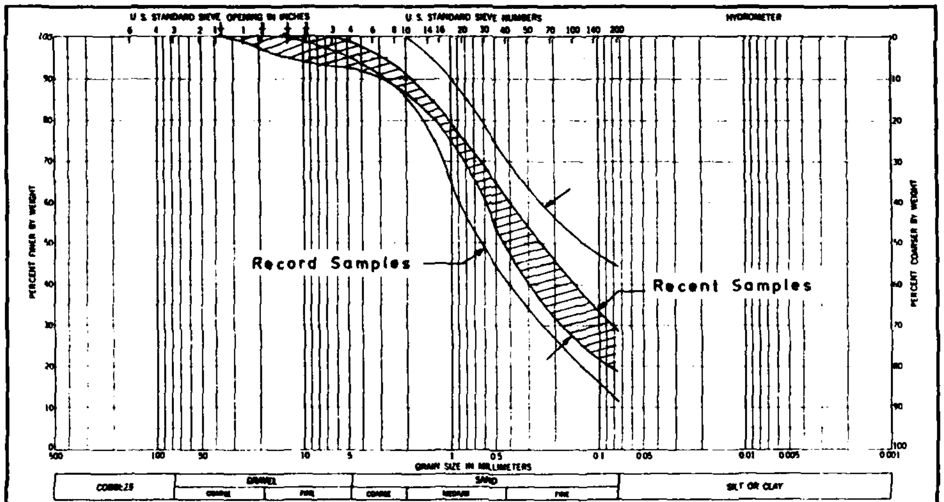
2 0 2



Sample No.	Elev or Depth	Classification		Nat w %	LL	PL	PI
		SC	(3)	9	27-31	19-21	8-10
Zone "C"	Material	SM	(1)				NP
(Recent Samples)*		SM-SC	(5)	11	24-28	18-22	6-9
	*Excludes Samples from Interface Zone						

GRADATION CURVES

Project: Folsom Dam
American River, Folsom, California
Area: Right Wing Dam
Boring No.: Record vs. Recent Samples
Date: 3 April 1989



Sample No.	Elev or Depth	Classification		Nat w %	LL	PL	PI
Zone "G"	Material	SM-SC	(1)		24	20	4
(Recent)*		SC	(5)	9-13	26-31	18-21	8-12
	*Excludes Samples from Interface Zone						

GRADATION CURVES

Project: Folsom Dam
American River, Folsom, California
Area: Left Wing Dam
Boring No.: Record vs. Recent Samples
Date: 3 April 1989

Figure 15. Grain size distribution for materials of Zone C in Right Wing Dam and Zone G in Left Wing Dam

Figure 17. Photo FOL 390 (2 Jan 1951) - Right abutment looking from left abutment following initial phase of excavation

Figure 19. Photo FOL 720 (1 Apr 1952) - Right and left abutments (foreground and background, respectively) following secondary phase of excavation

Figure 21. Grouting operations near the end of Monolith No. 1 of the CGD (right abutment) with core trench of RWD in background

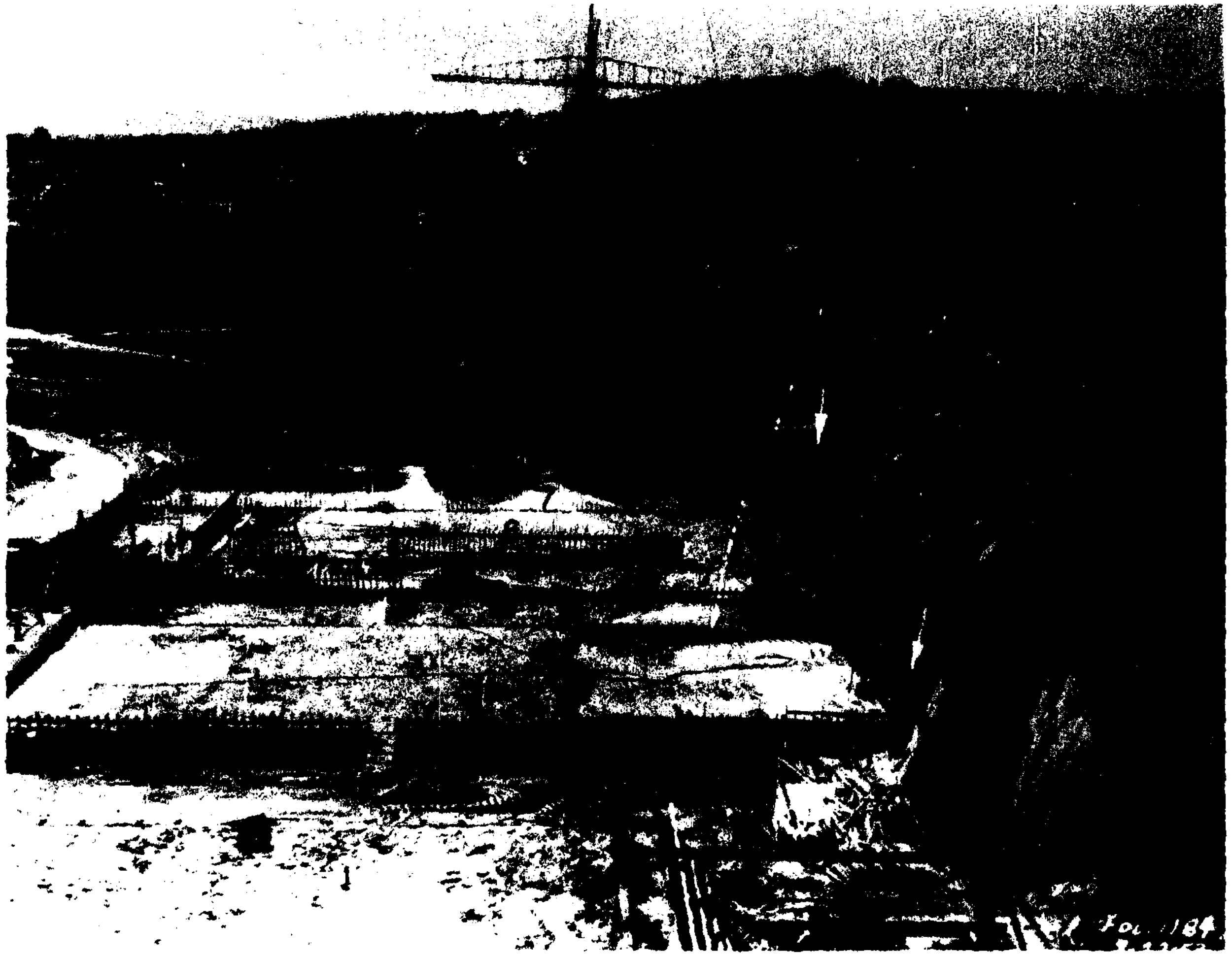


Figure 23. Photo FOL 1184 (22 July 1953) - Progress of construction of CGD looking toward the left abutment

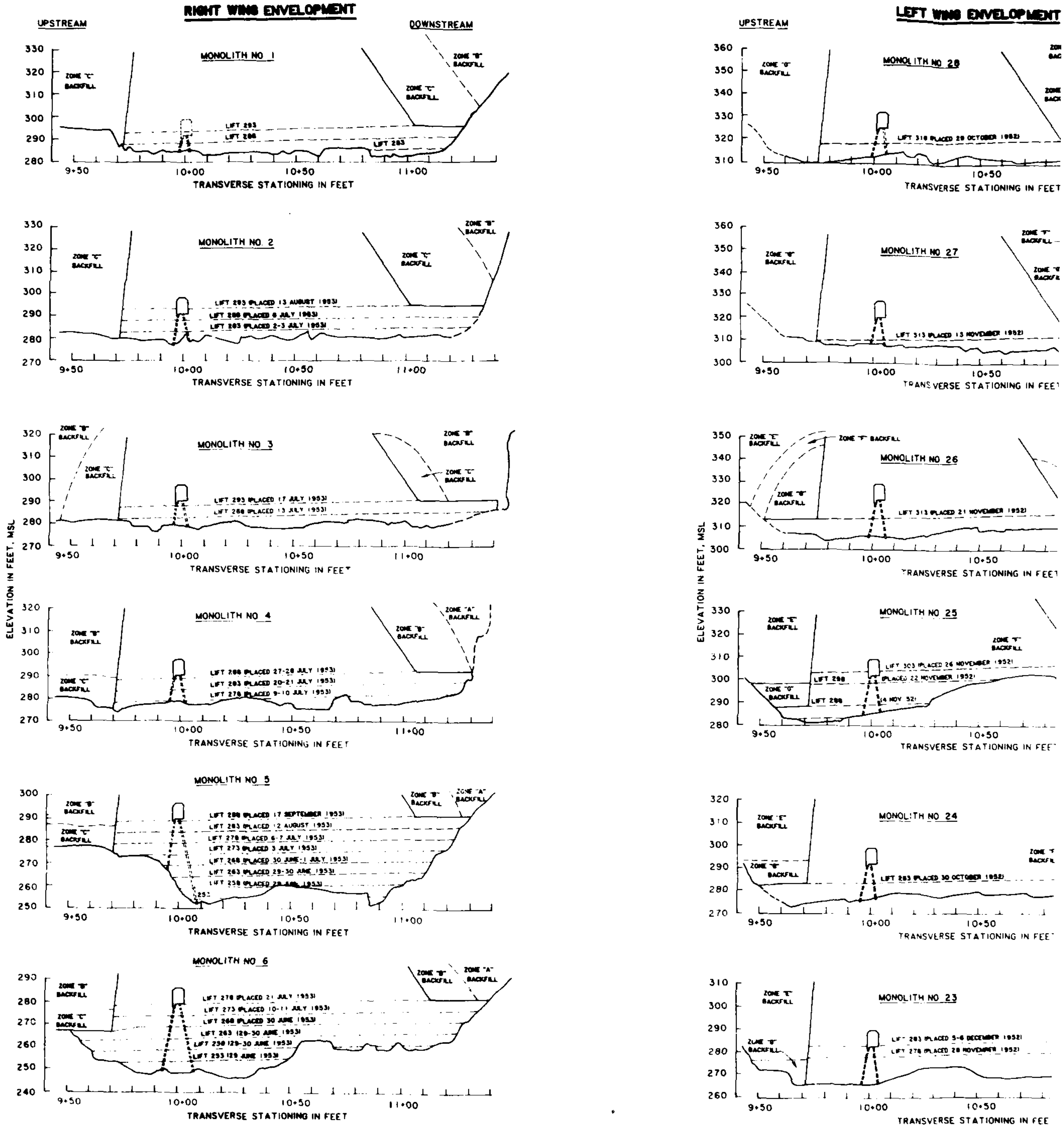
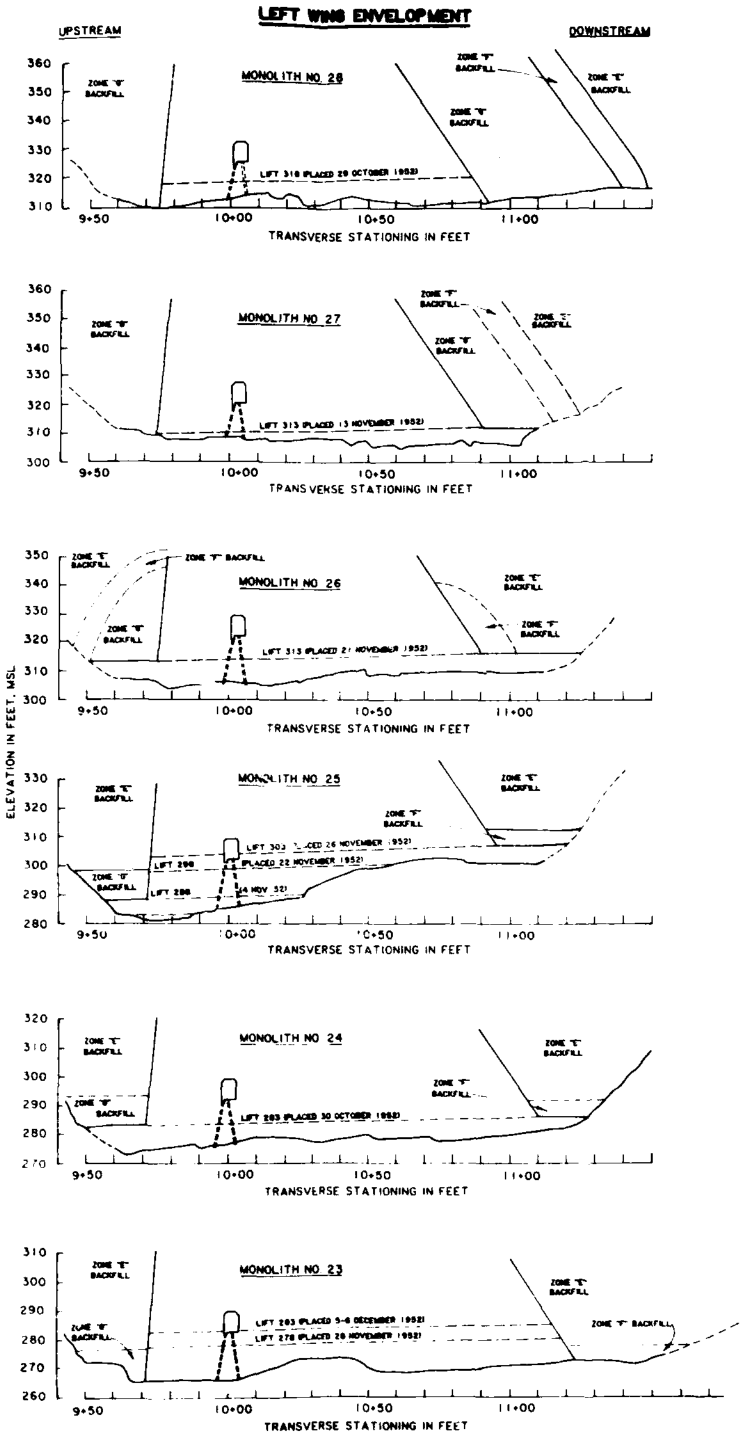
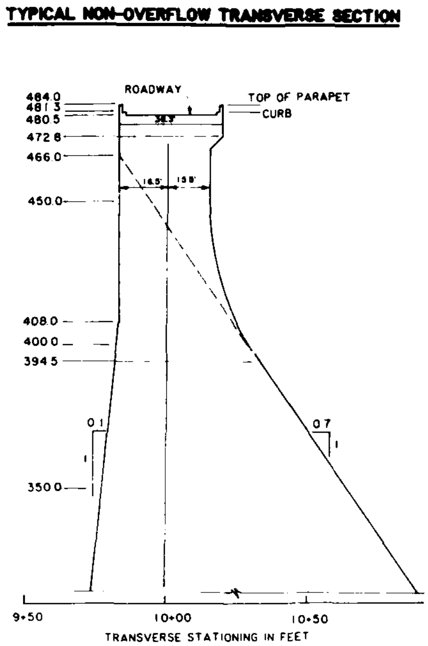


Figure 24. Transverse cross-sections of monoliths in int





NOTES

1. CROSS-SECTIONS TAKEN FROM "FOUNDATION REPORTS" FOR MONOLITHS 1-2-3, 4-5-6, 22-23, 24-25, AND 26-27-28
2. EXTENSIONS OF CROSS-SECTIONS DEVELOPED USING AM-1-9-396/5.2 AND SITE PHOTOGRAPHS. DASHED LINES INDICATE SOME QUESTION STILL EXISTS

face

...nsverse cross-sections of monoliths in interface zones

202

Figure 25. Photo FOL 1193 (22 July 1953) - Progress of construction at Monolith No. 1 of CGD (right abutment) showing concrete extension looking upstream

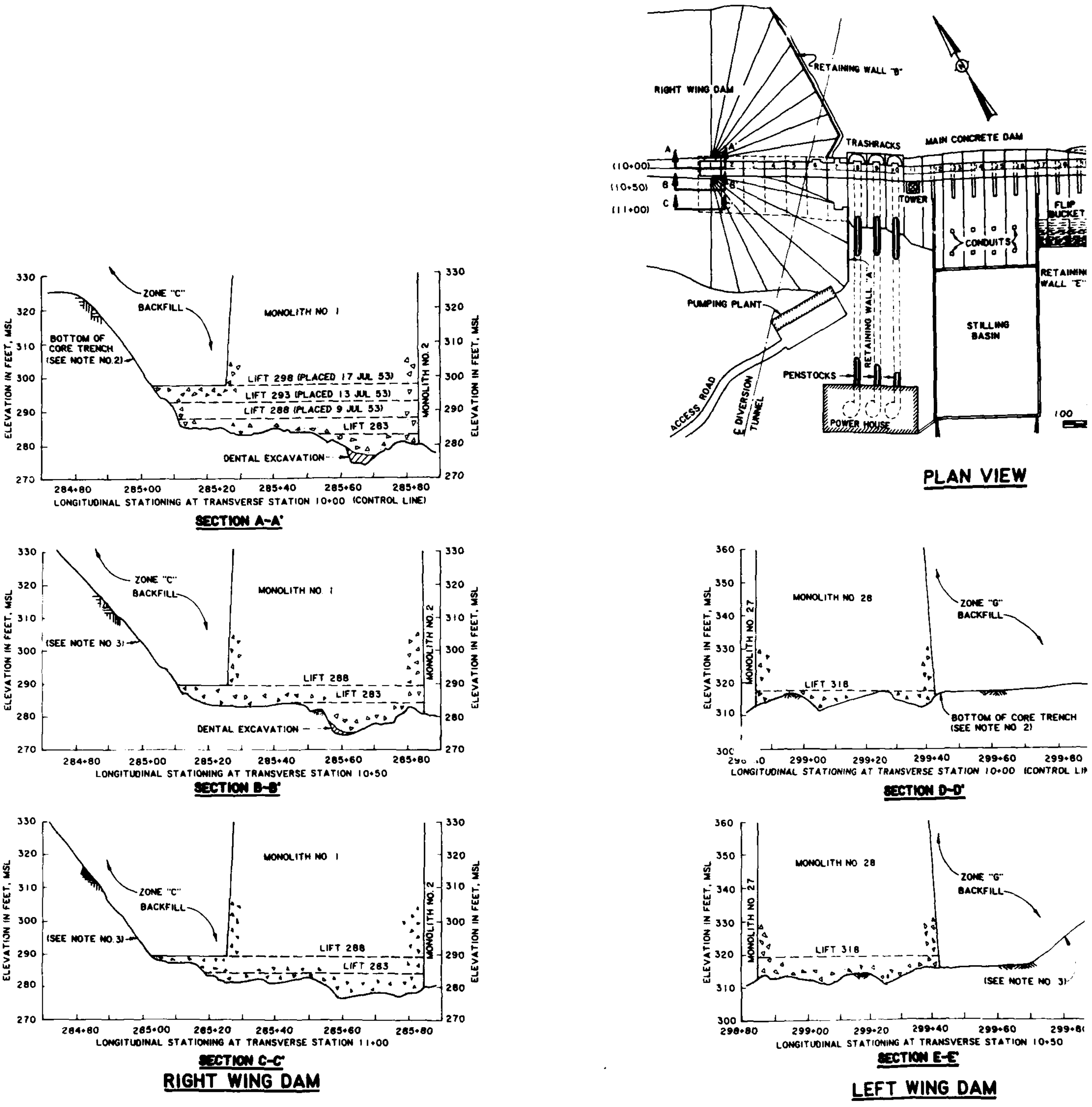
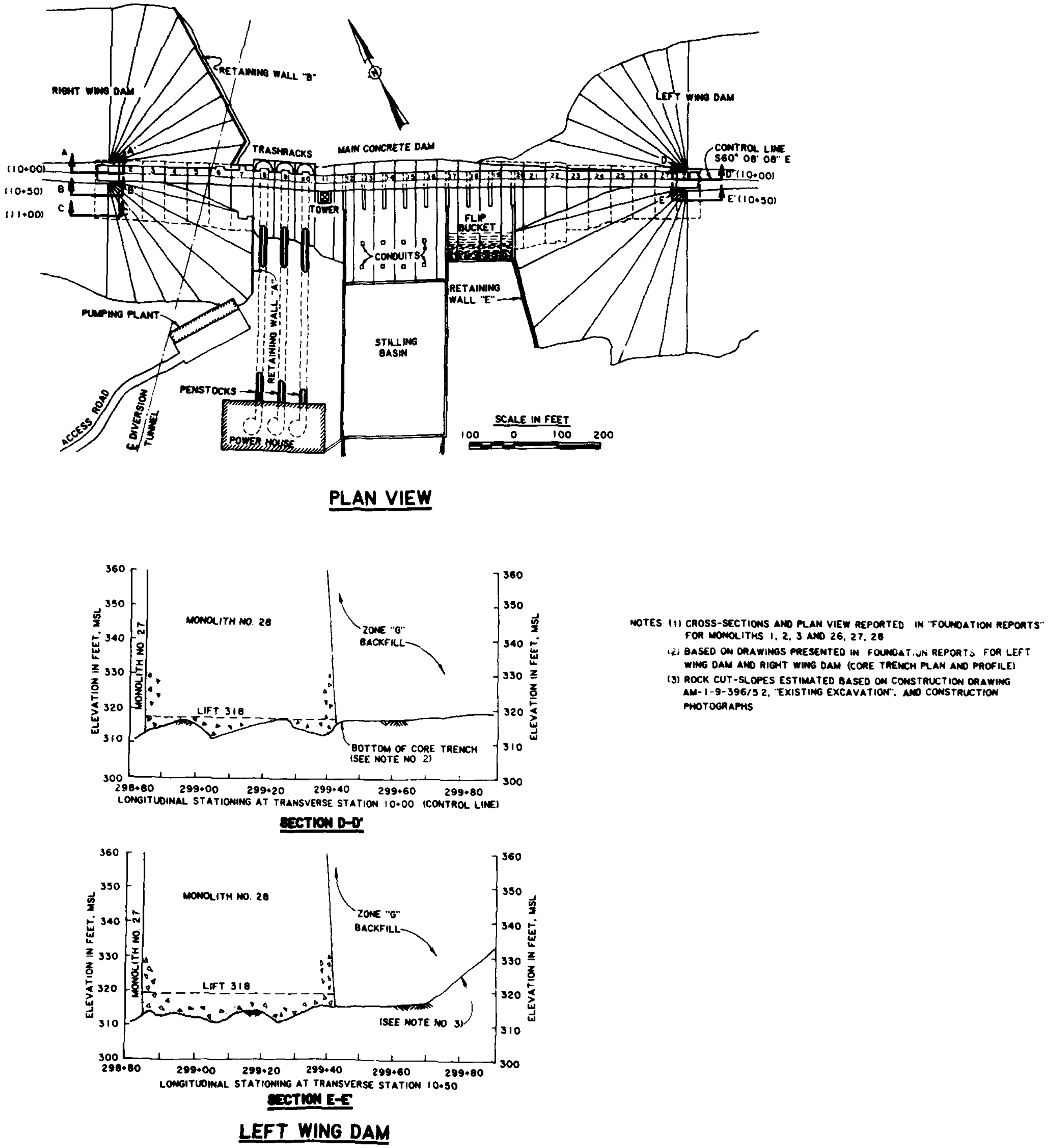


Figure 26. Longitudinal cross-sections of interface zon
from end monoliths (Nos. 1 and 28)



26. Longitudinal cross-sections of interface zones away from end monoliths (Nos. 1 and 28)

F2

Figure 27. Photo FOL 1141 (25 June 1953) - Excavation for Retaining Wall "B" looking downstream

Figure 28. Photo FOL 1185 (22 July 1953) - Construction of Retaining Wall "B" looking downstream

Figure 29. Photo FOL 1217 (29 July 1953) - Progress of construction of CGD looking toward the left abutment

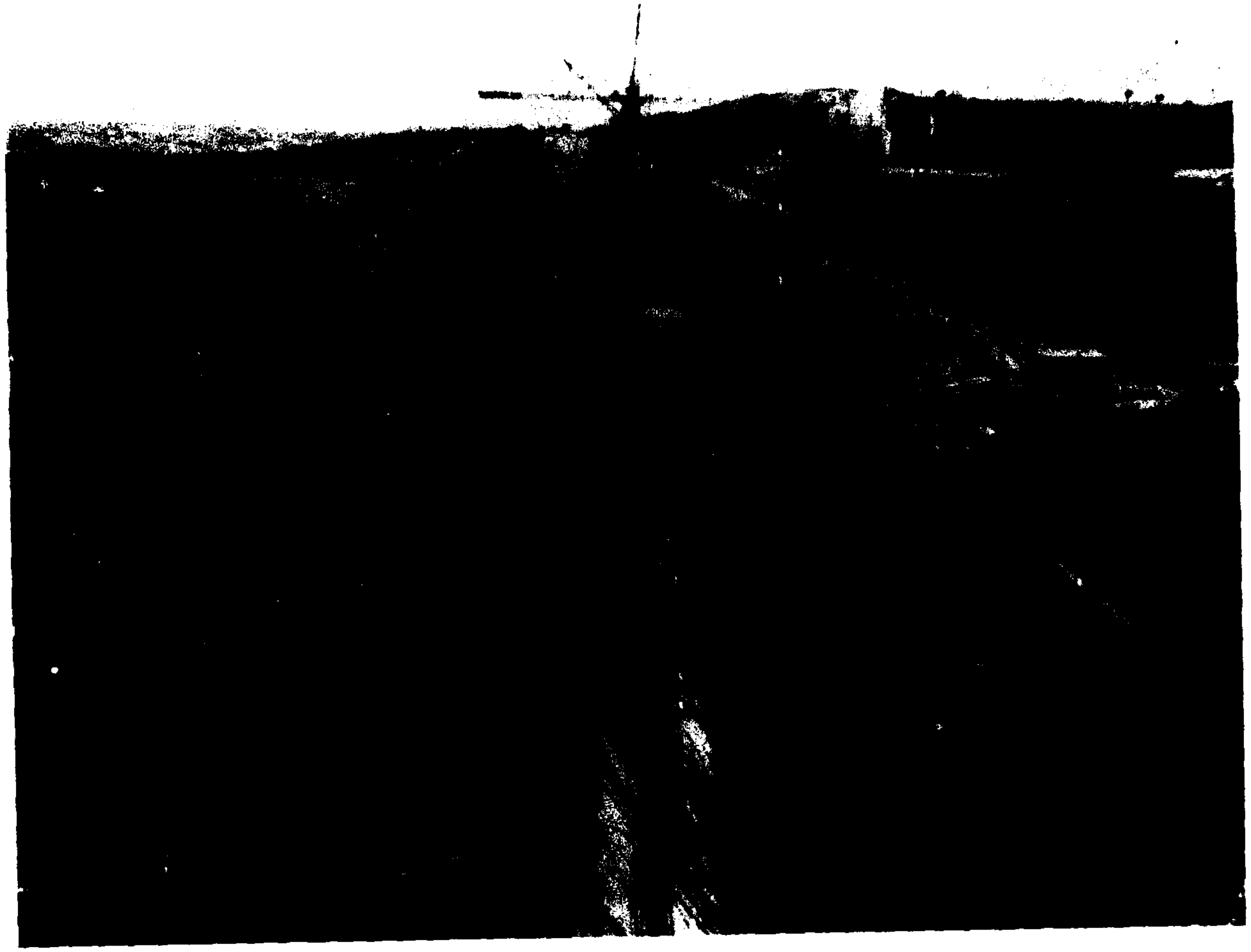


Figure 30. Photo FOL 1388 (15 Oct 1953) - Construction of support structure for railway on downstream side of CGD at right abutment and completion of crib walls on upstream side of CGD at left abutment

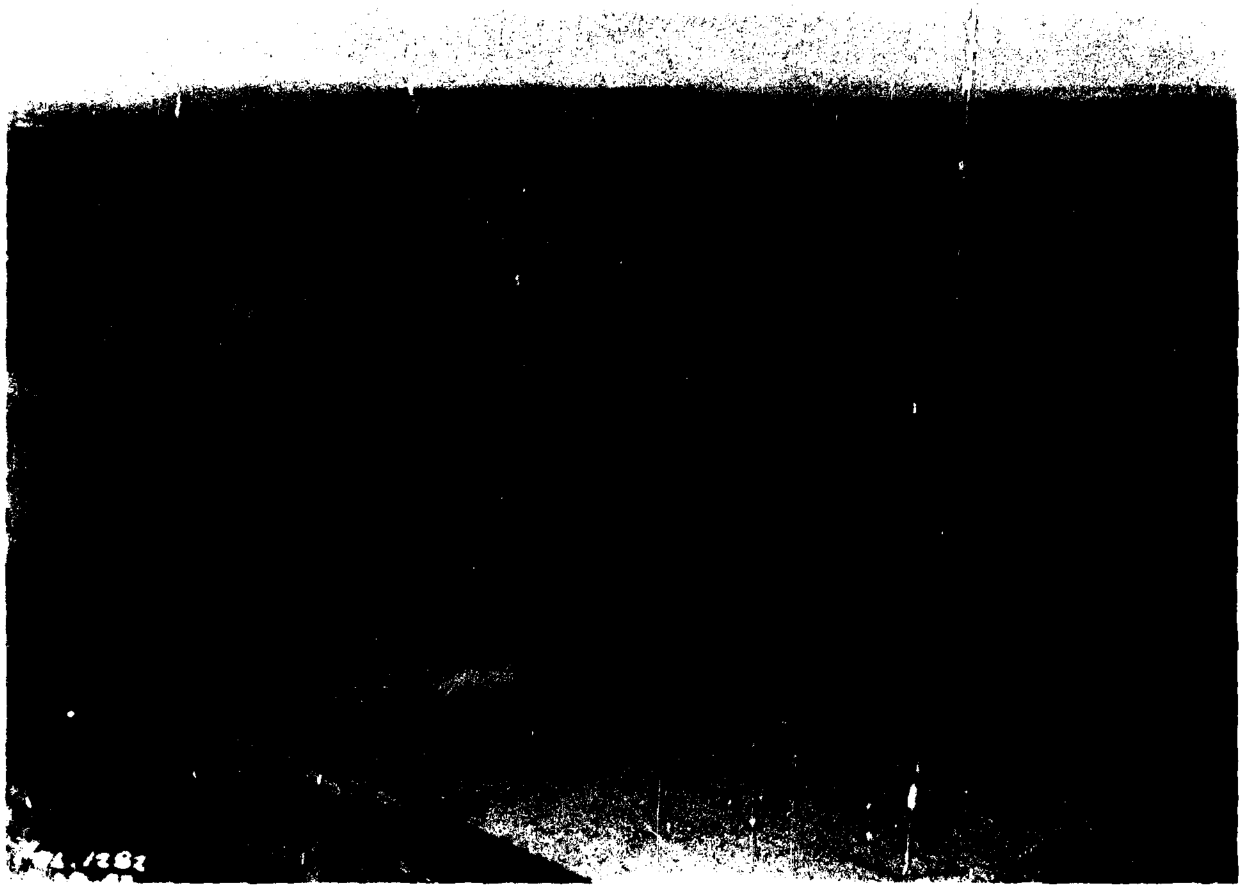


Figure 31. Photo FOL 1282 (28 Aug 1953) - Progress of construction at right abutment between CGD and Retaining Wall "B"

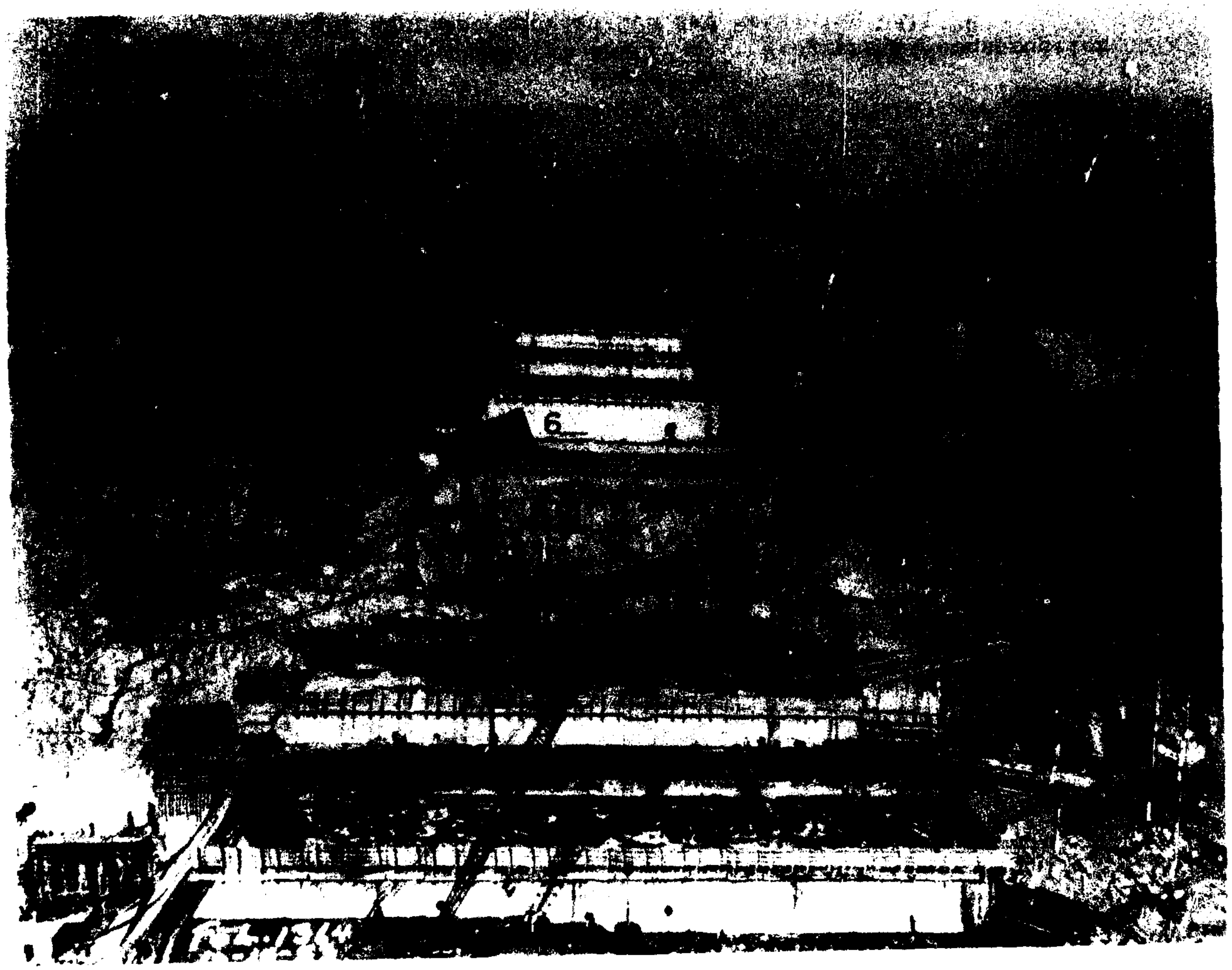


Figure 32. Photo FOL 1364 (15 Oct 1953) - Progress of construction at right abutment (background)

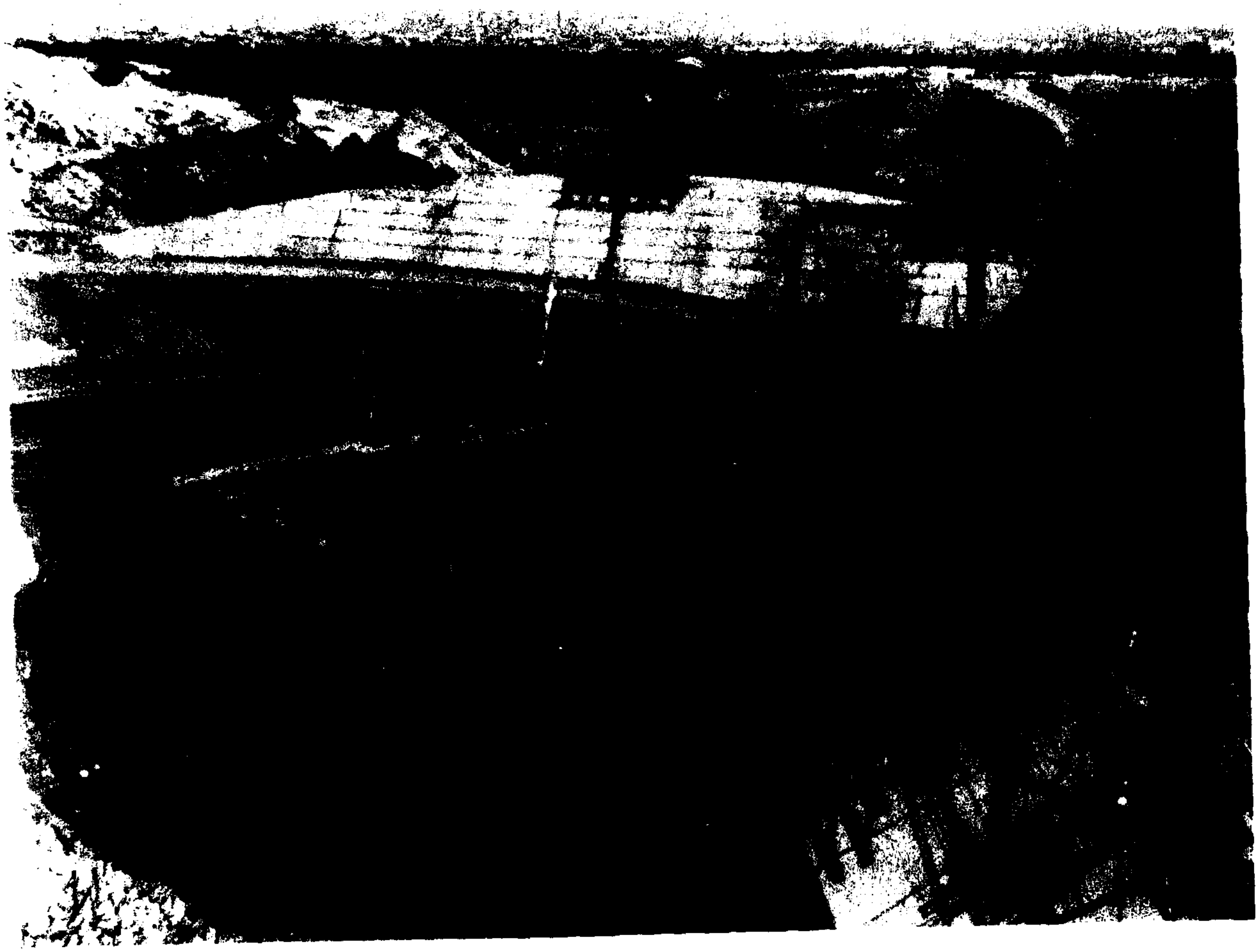


Figure 33. Photo FOL 1387 (15 Oct 1953) - Extent of fill placement at interface zones at the end and upstream of the CGD (looking upstream)

Figure 34. Photo FOL 1818 (16 Apr 1954) - Extent of fill placement in RWD and interface zones at right abutment

Figure 35. Photo FOL 1862 (29 June 1954) - Progress of construction looking toward the left abutment

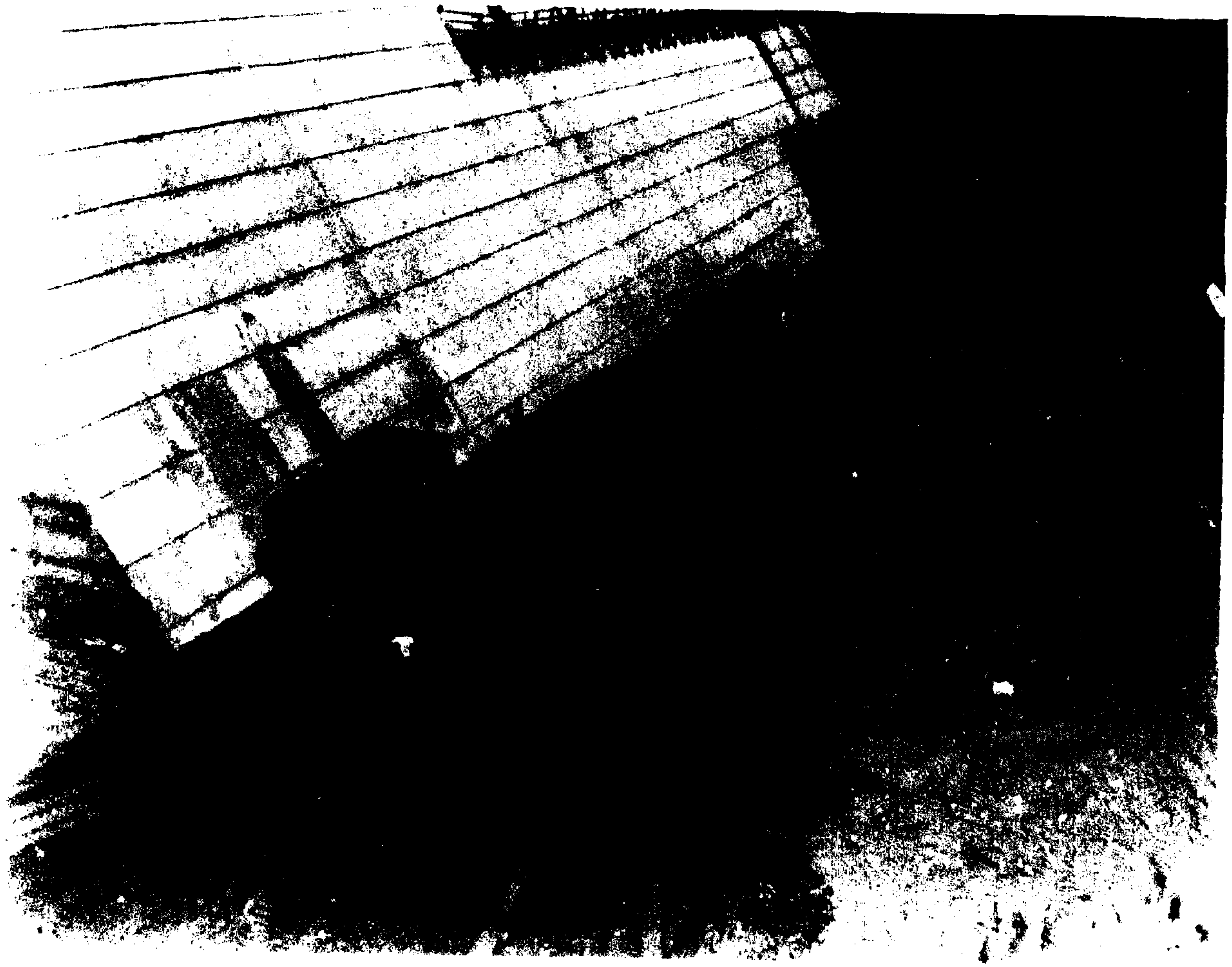


Figure 36. Photo FOL 1846 (11 June 1954) - Placement of core fill adjacent to CGD on the downstream side at the right abutment

Figure 37. Photo FOL 1919 (1 July 1954) - Progress of construction looking down and toward the left abutment

Figure 38. Photo FOL 1428 (29 Oct 1953) - Placement of core fill adjacent to CGD at the left abutment

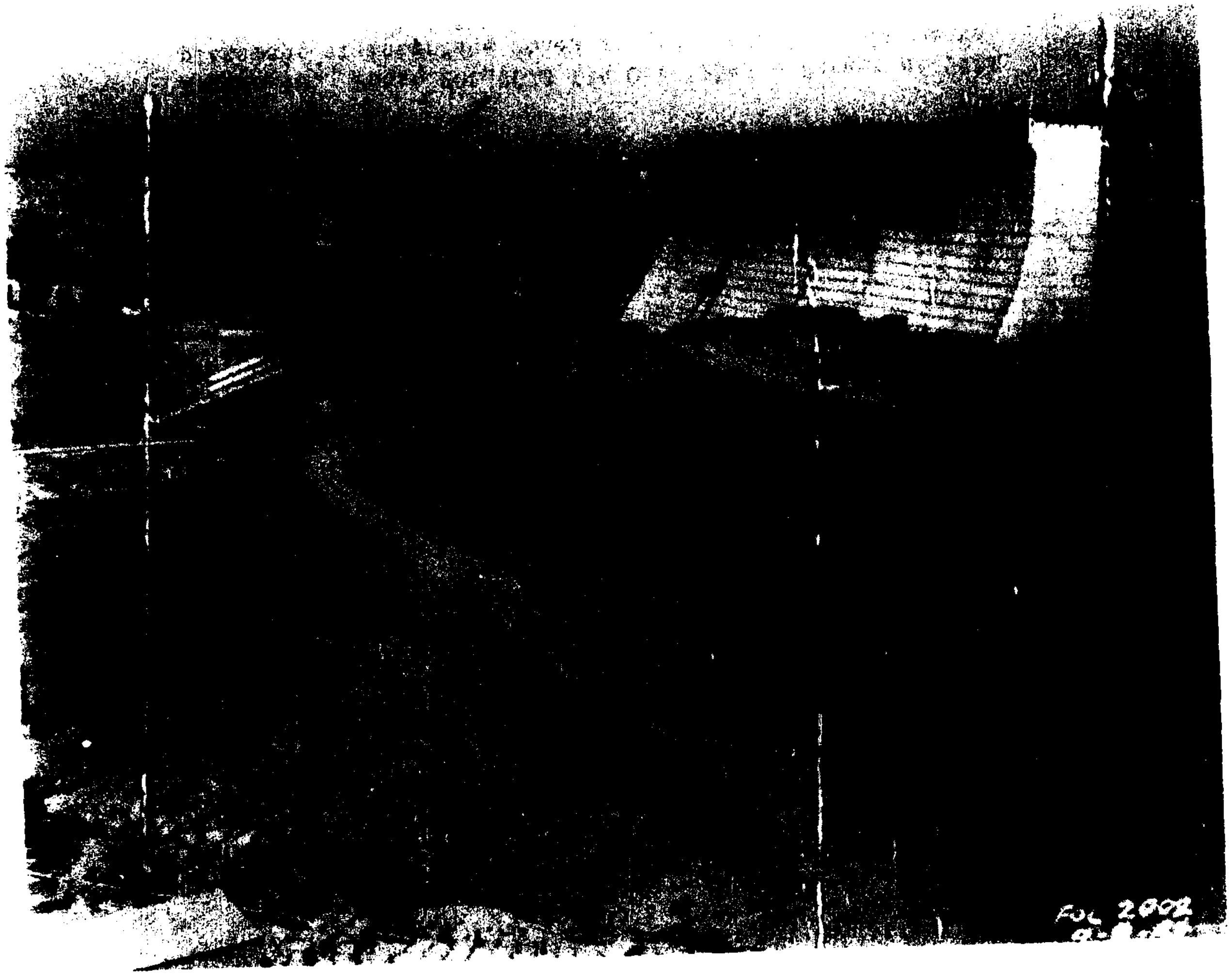
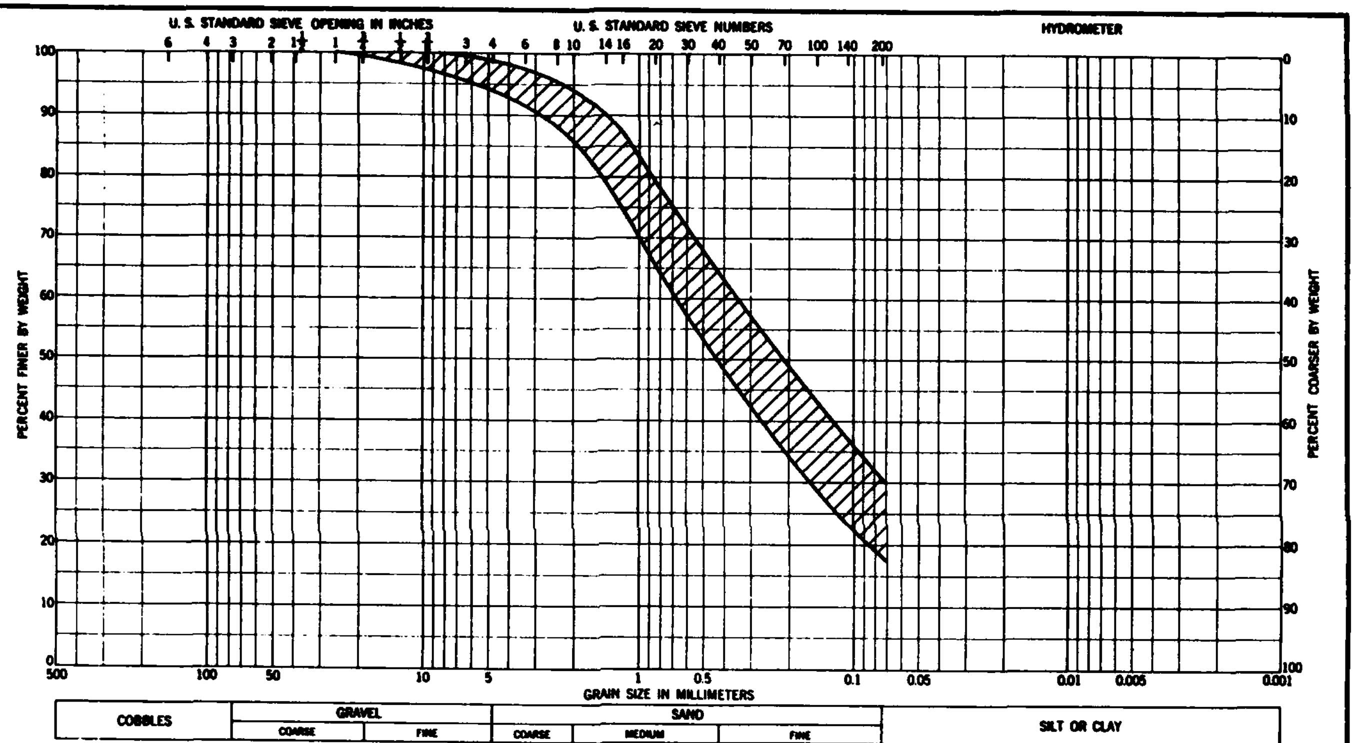


Figure 39. Photo FOL 2002 (8 Sept 1954) - Placement of fill on downstream side of CGD at left abutment

Figure 40. Photo FOL 2078 (14 Oct 1954) - Extent of fill placement at interface zones at right and left abutments (to left and right, respectively)

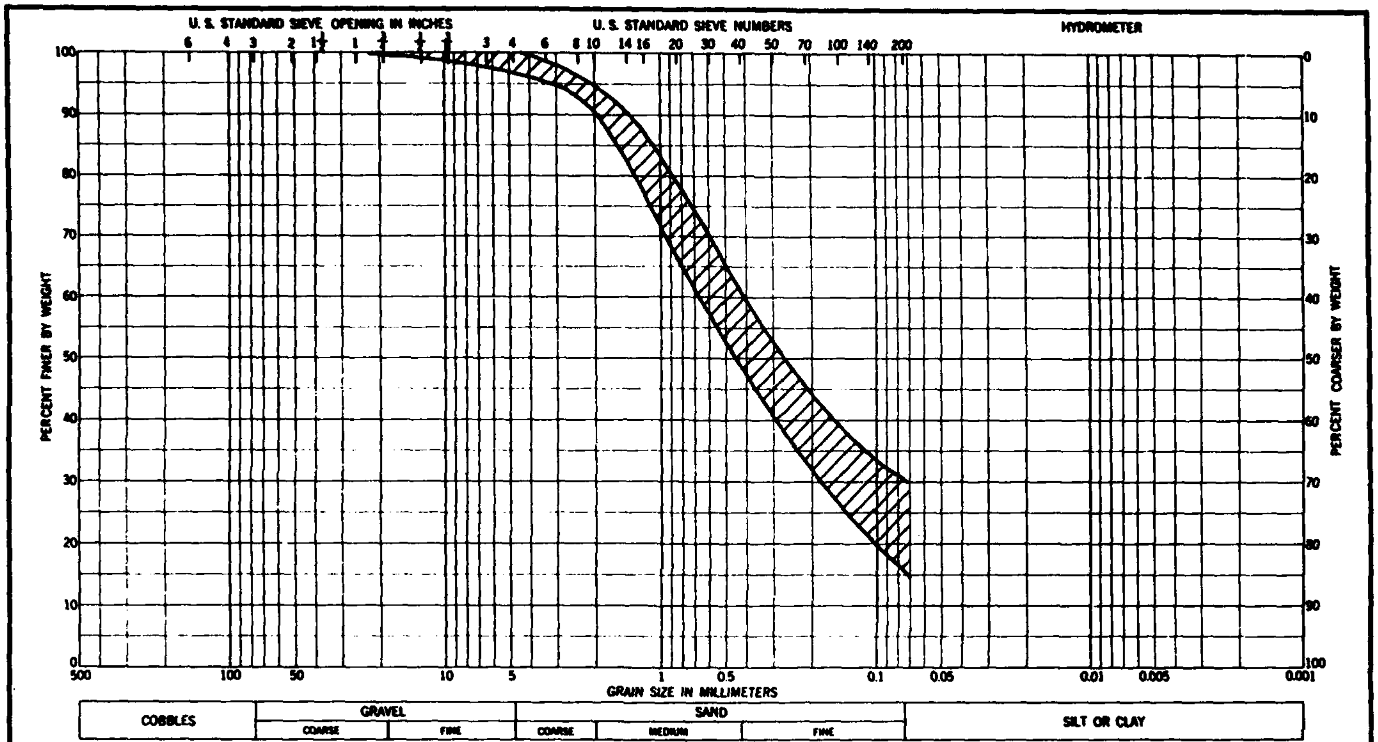
Figure 41. Downstream portion of Folsom Dam and Reservoir following initial filling of reservoir (looking east)



Sample No.	Elev or Depth	Classification		Nat w %	LL	PL	PI	
		SM	(2)		23-24	21	2-3	Project Folsom Dam
Zone "C" Material		SM-SC	(2)		23-27	17-22	5-6	American River, Folsom, California
								Area Right Wing Dam Envelopment
								Boring No. Record Samples
GRADATION CURVES								Date 1 April 1987

ENG FORM 1 MAY 63 2087

Figure 42. Ranges in grain size distribution of record samples at RWD interface zones



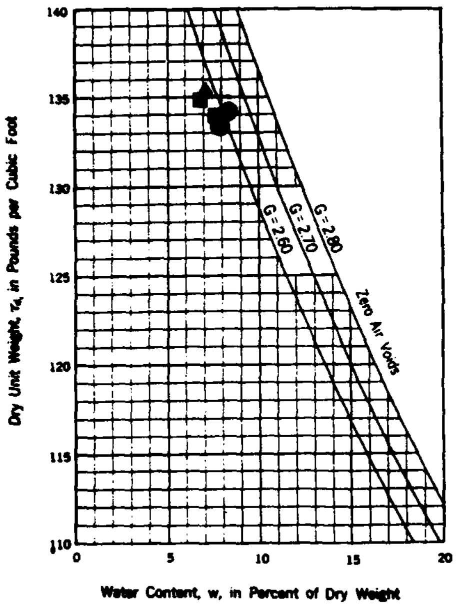
Sample No.	Elev or Depth	Classification		Nat w %	LL	PL	PI
		SM	(1)			NP	
Zone "G" Material		SM-SC	(2)		26-27	22	4-5
		SC	(2)		24-29	18-19	6-9

Project Folsom Dam
American River, Folsom, California
Area Left Wing Dam Envelopment
Boring No. Record Samples
Date 1 April 1987

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Figure 43. Ranges in grain size distribution of record samples at LWD interface zones



Modified compaction test (AASHTO T99-49)

25 blows per each of 5 layers, with 10 lb rammer and

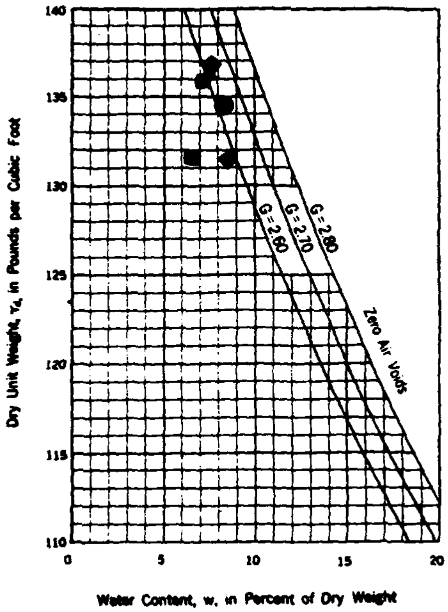
18 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification		G	LL	PL	% > No. 4	% > 3/4 in.
●		SM	(2)	2.68-2.69	23-24	21	3-5	0
■		SM-SC	(2)	2.67-2.75	23-27	17-22	2	0
▲		SM (recent)	(1)		19	17	4	1

Sample No.	SM	SM-SC		SM (recent)
Natural water content, percent				
Optimum water content, percent	8.0-8.4	6.8-7.7		7.1
Max dry density, lb/cu ft	133.3-134.2	134.0-134.8		135.3

Remarks	Project Folsom Dam	
Averages: 134.3 pcf	American River, Folsom, California	
7.6%	Area Right Wing Dam Envelopment	
	Boring No. Record Samples	Date 1 April 1987
	COMPACTION TEST REPORT	

Figure 44. Ranges in results of moisture-density relationships at RWD interface zones



Modified compaction test (AASHTO T99-49)
25 blows per each of 5 layers, with 10 lb rammer and
18 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification		G	LL	PL	% > No. 4	% > 3/4 in.
●		SM	(1)	2.79		NP	2	0
■		SM-SC	(2)	2.70	26-27	22	0-3	0
◆		SC	(2)	2.68-2.73	24-29	18-19	0-3	0
▲		SM-SC (recent)	(1)	2.74	25	18	2	1

Sample No.	SM	SM-SC	SC	SM-SC (recent)
Natural water content, percent				
Optimum water content, percent	8.3	7.3-7.6	7.6-8.7	7.5
Max dry density, lb/cu ft	134.5	131.6-135.8	131.4-136.8	136.3

Remarks	Project Folsom Dam	
Averages: 134.4 pcf	American River, Folsom, California	
7.8%	Area Left Wing Dam Envelopment	
	Boring No. Record Samples	Date 1 April 1987
	COMPACTION TEST REPORT	

Figure 45. Ranges in results of moisture-density relationships at LWD interface zones

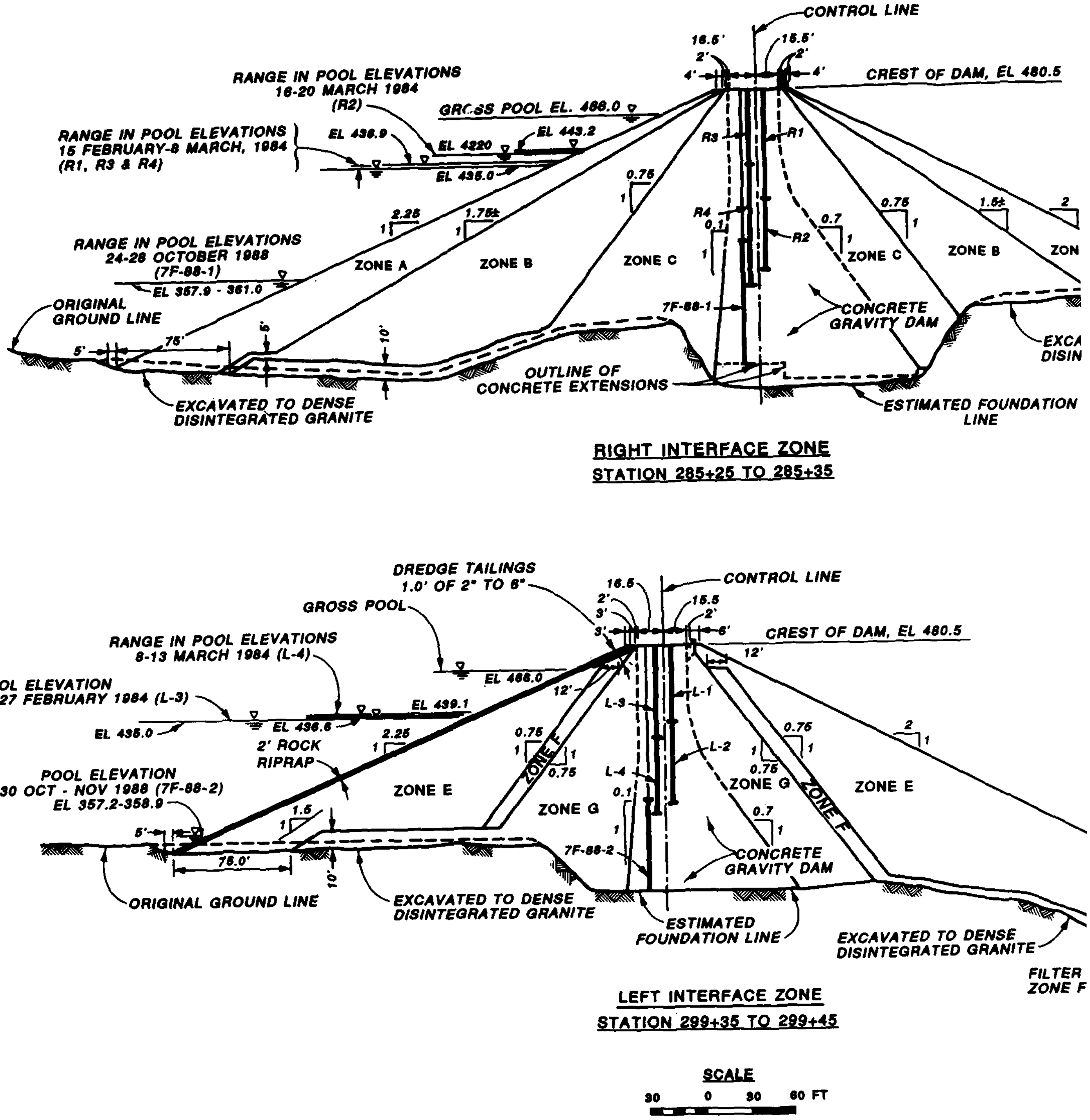
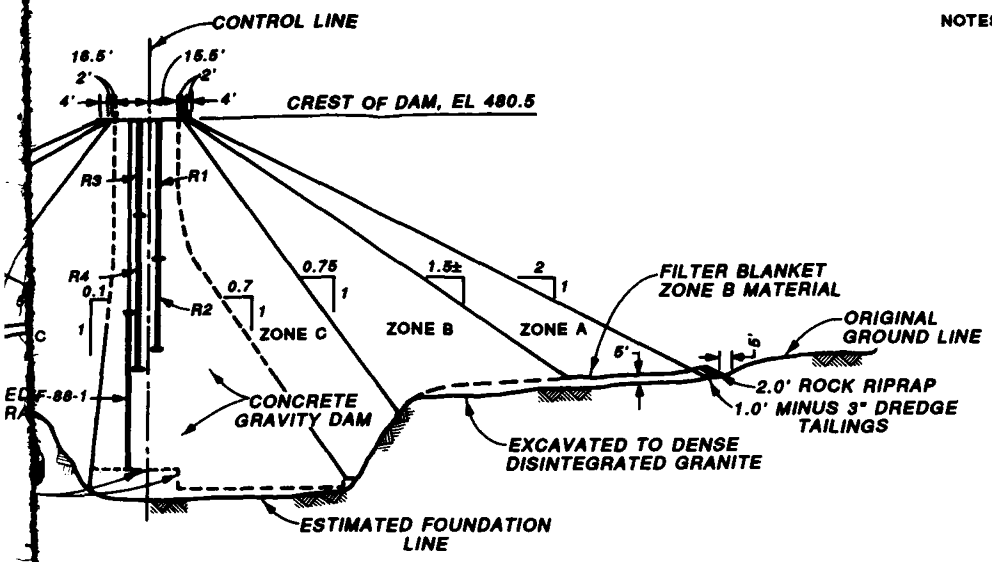


Figure 46. Cross sections of RWD and LWD a

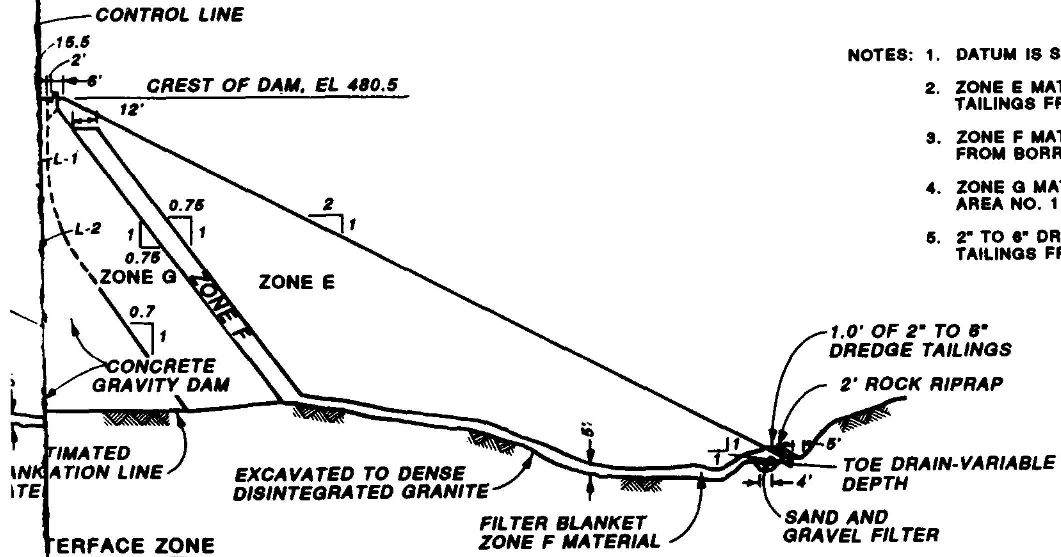
1 of 2



NOTES:
1. DATUM IS SEA LEVEL DATUM OF 1929.
2. ZONE A MATERIAL IS ROCK FROM THE AMERICAN RIVER CHANNEL EXCAVATION AND GOVERNMENT OWNED STOCKPILE.
3. ZONE B MATERIAL IS UNPROCESSED SAND, GRAVEL AND COBBLES FROM AMERICAN RIVER CHANNEL EXCAVATION.
4. LINES SHOWN ON EMBANKMENT SECTIONS SEPARATING ZONES A AND B WILL VARY IN SLOPE AS DETERMINED BY THE QUANTITIES OF MATERIALS AVAILABLE FROM AMERICAN RIVER CHANNEL EXCAVATION.
5. WHEN MATERIAL FOR ZONE B FROM THE AMERICAN RIVER CHANNEL EXCAVATION IS EXHAUSTED, ADDITIONAL MATERIALS WILL BE OBTAINED FROM BORROW AREA NO. 7 AND APPROVED WASTE FROM PROCESSING OF CONCRETE AGGREGATE.
6. ZONE C MATERIAL IS DECOMPOSED GRANITE FROM BORROW AREA NO. 2 AND SUITABLE FINE GRAINED MATERIAL FROM AMERICAN RIVER CHANNEL EXCAVATION.
7. ESTIMATED FOUNDATION LINE ALTERED FROM ORIGINAL CROSS SECTION TO ACCOUNT FOR AS BUILT INFORMATION.

TERFACE ZONE

285+25 TO 285+35



NOTES:
1. DATUM IS SEA LEVEL DATUM OF 1929.
2. ZONE E MATERIAL TO BE UNPROCESSED COARSE DREDGE TAILINGS FROM BORROW AREA NO. 5.
3. ZONE F MATERIAL IS MINUS 2" COARSE DREDGE TAILINGS FROM BORROW AREA NO. 5.
4. ZONE G MATERIAL IS DECOMPOSED GRANITE FROM BORROW AREA NO. 1.
5. 2" TO 6" DREDGE TAILINGS ARE PROCESSED COARSE DREDGE TAILINGS FROM BORROW AREA NO. 5.

TERFACE ZONE

99+35 TO 299+45

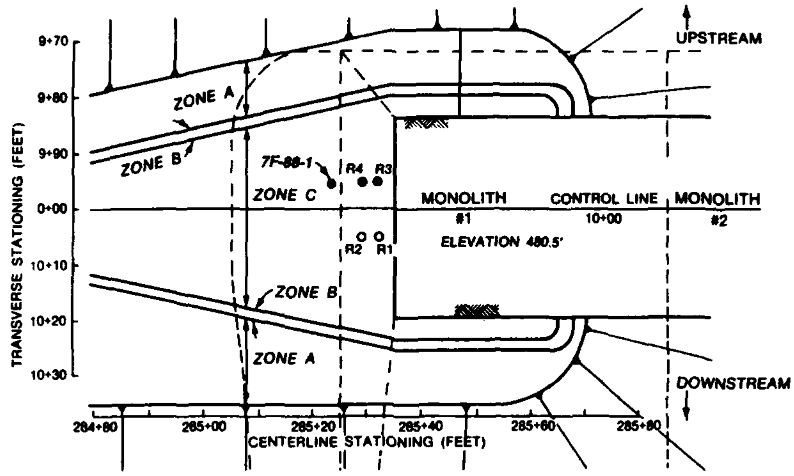
SCALE

0 30 60 FT

en

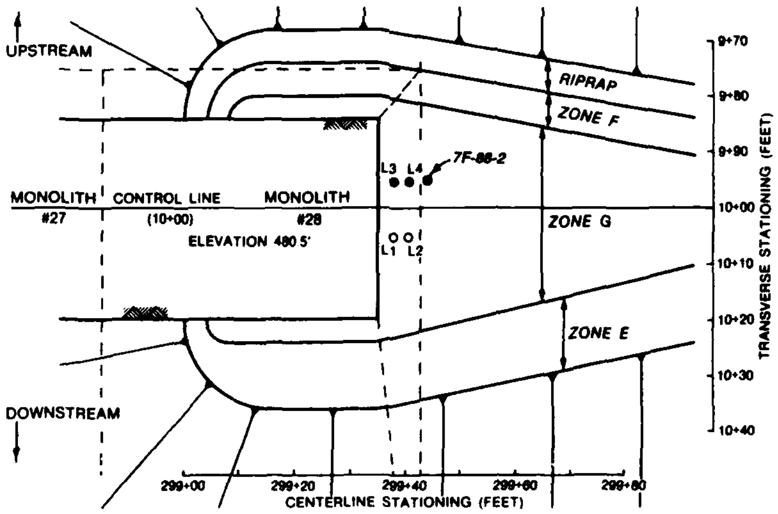
. Cross sections of RWD and LWD at ends of CGD

RIGHT WING DAM INTERFACE

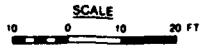


PLAN VIEW

LEFT WING DAM INTERFACE



PLAN VIEW



LEGEND

○ HOLES MADE THROUGH EMBANKMENT - UNDISTURBED SAMPLES OBTAINED

● HOLES MADE THROUGH EMBANKMENT - SPT AND DISTURBED SAMPLES OBTAINED

Figure 47. Plan view of interface zones at ends of the CGD showing locations of drill holes

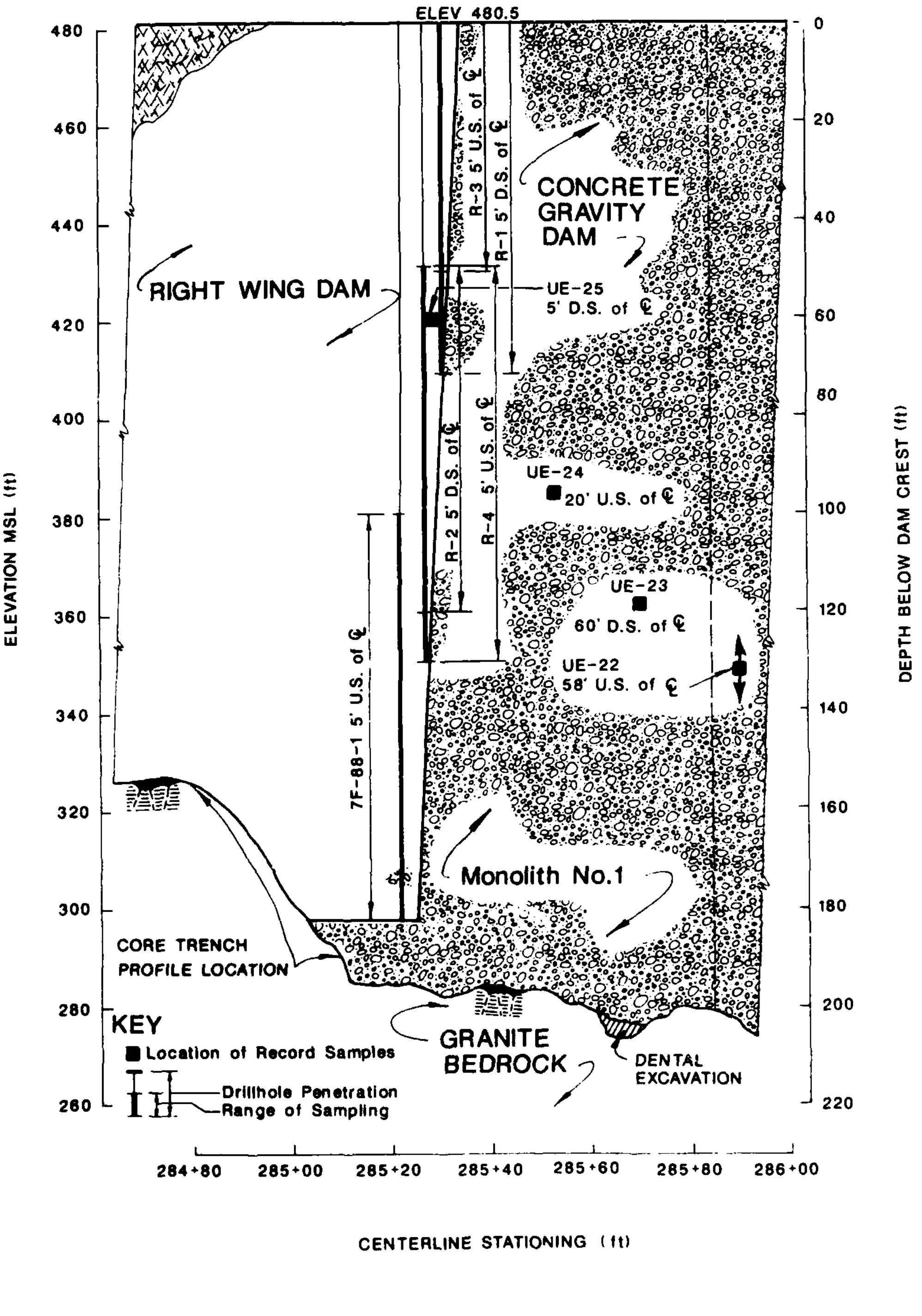


Figure 48. Profile along centerline at interface between RWD and CGD showing depths penetrated by drill holes and sample locations

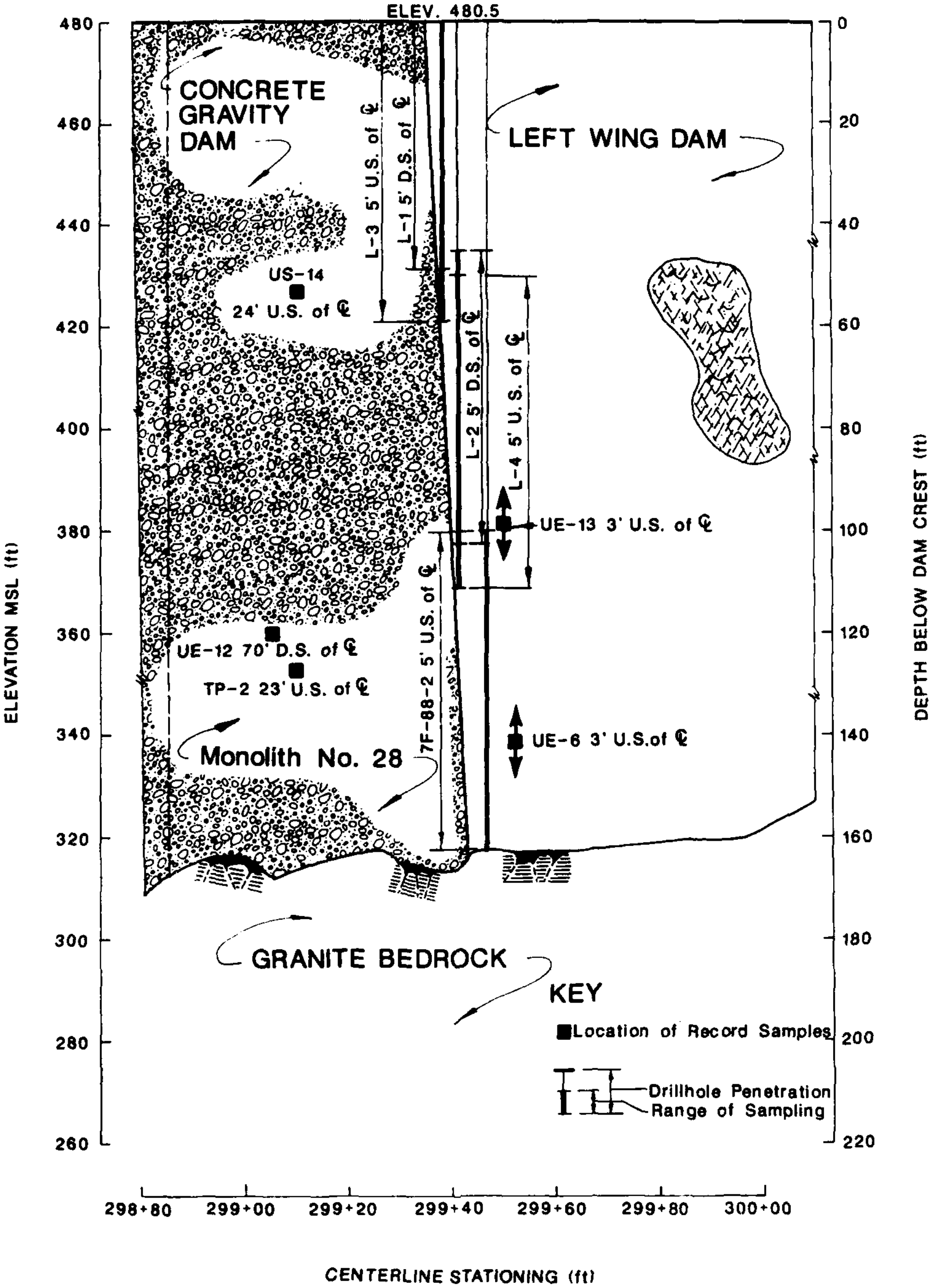
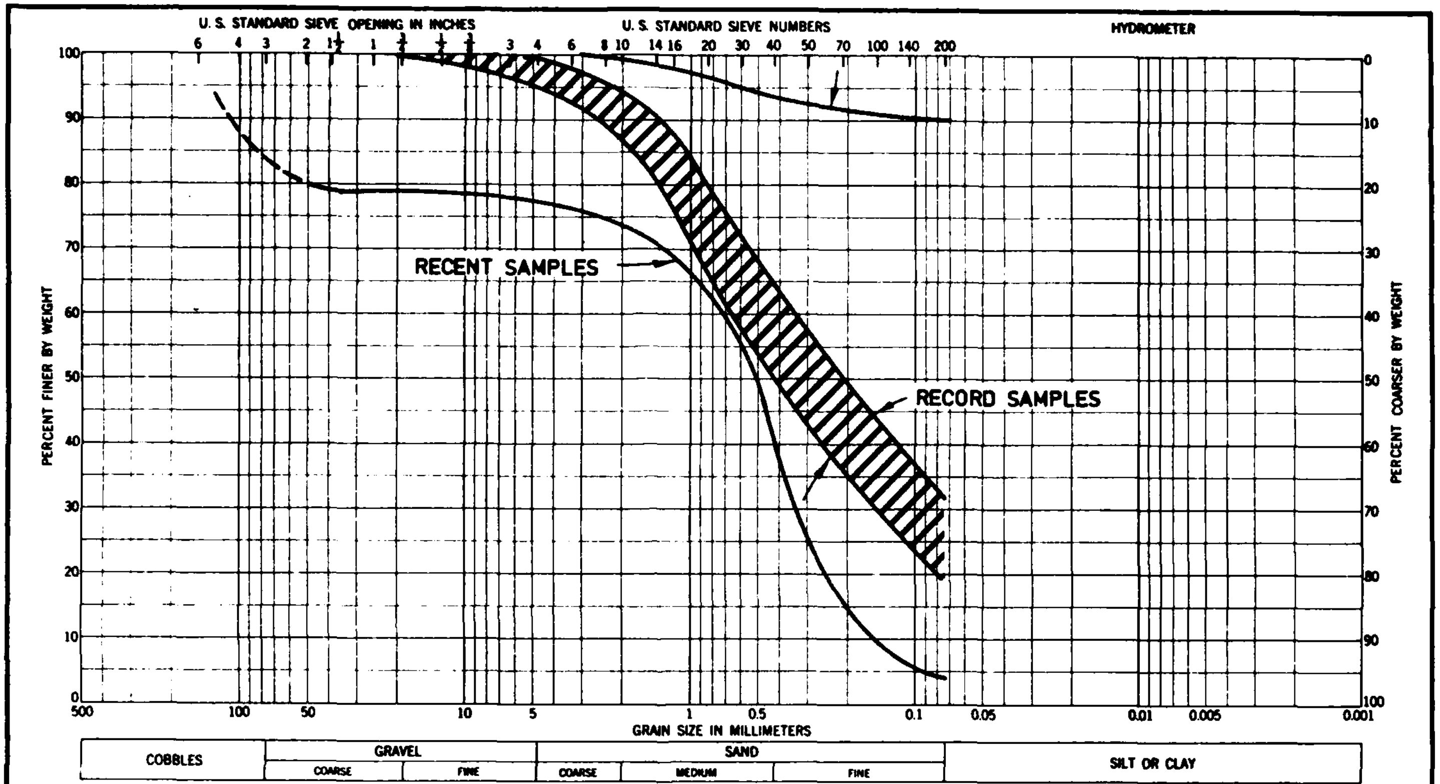


Figure 49. Profile along centerline at interface between LWD and CGD showing depths penetrated by drill holes and sample locations



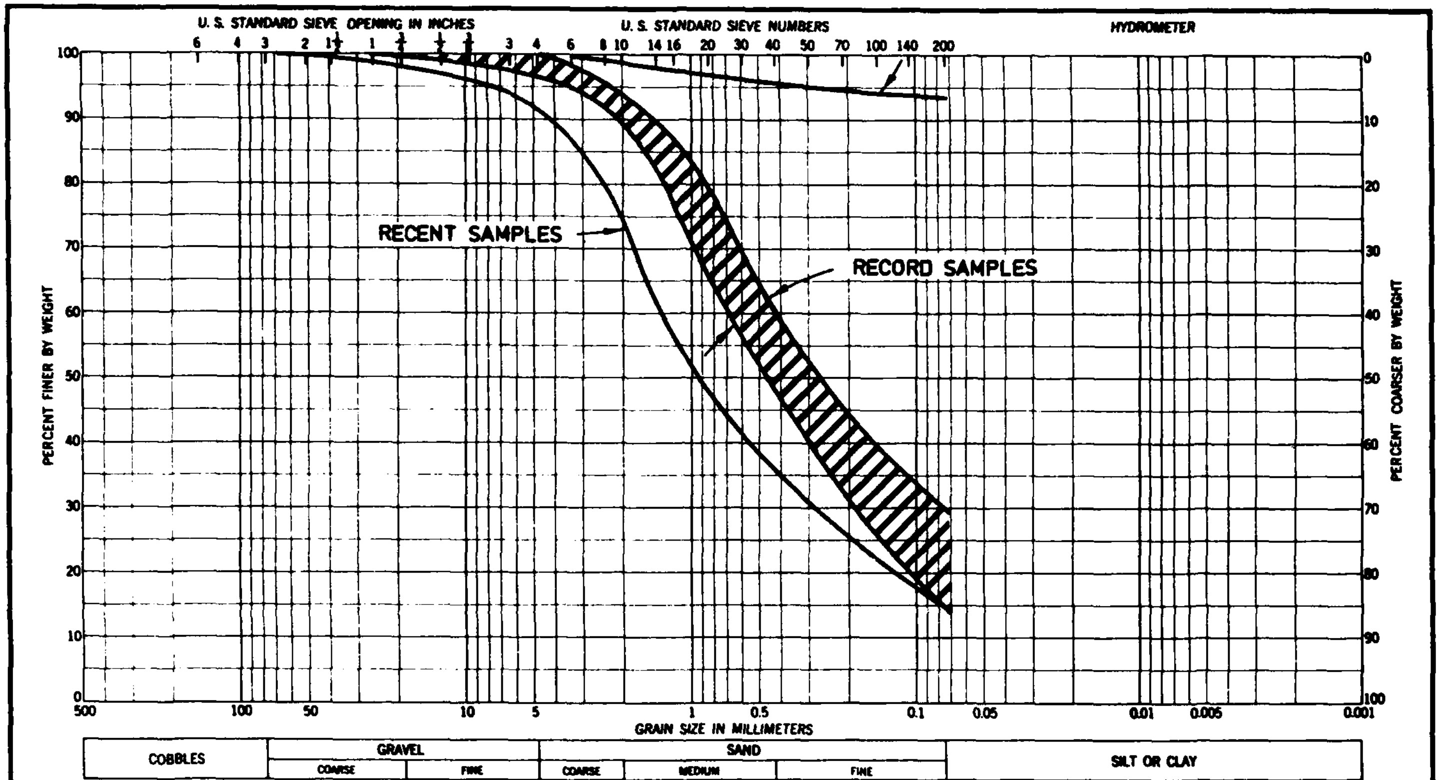
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
		SM (18)		19-25	17-23*	2-5*
Zone "C" Material		SM-SC (8)		22-28	16-23	4-7
(Recent Samples)		SM-SW & SP (2)	19	23	21*	2*
		SC (10)	11-15	25-30	16-21	7-10
		ML (2)		22-25	19-23	2-3

*And non-plastic

GRADATION CURVES

Project: Folsom Dam
American River, Folsom, California
Area: Right Wing Dam Envelopment
Boring No. R-1, R-2, R-3, R-4 and 7F-88-1
Date: 1 March 1989

Figure 50. Comparison of ranges in grain size distribution of core material at RWD interface zones



Sample No.	Elev or Depth	Classification		Nat w %	LL	PL	PI
		SM	(6)		24-28	22-23*	2-5*
Zone "G" Material		SM-SC	(6)		25-26	18-21	4-7
(Recent Samples)		SC	(13)	13-19	24-30	16-22	5-11
		ML & ML-CL	(6)		23-26	17-22	3-6
		CL	(4)		25-29	17-20	8-9

*And non-plastic

GRADATION CURVES

Project Folsom Dam

American River, Folsom, California

Area Left Wing Dam Envelopment

Boring No. L-1, L-2, L-3, L-4 and 7F-88-2

Date 1 March 1989

Figure 51. Comparison of ranges in grain size distribution of core material at LWD interface zones

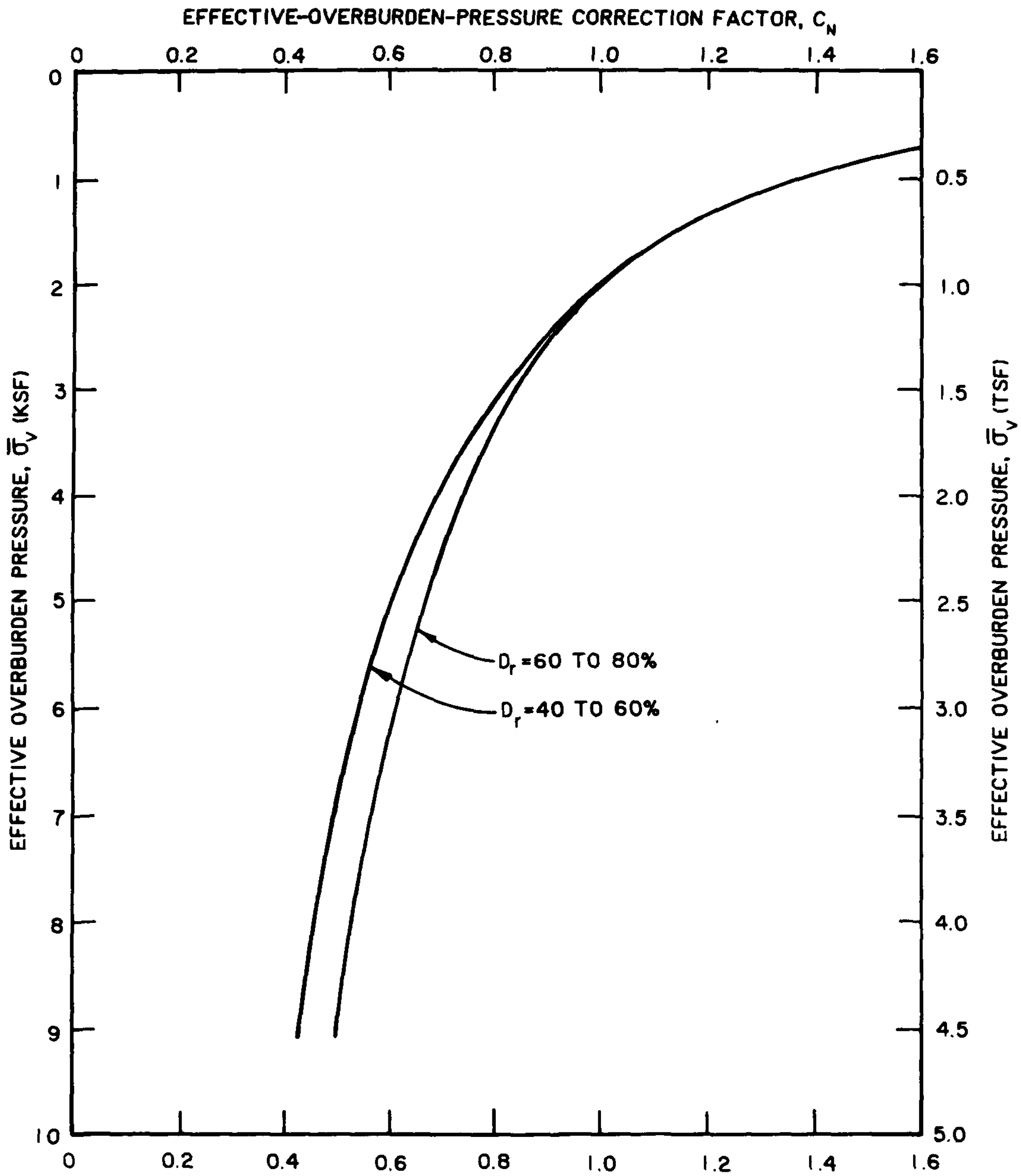


Figure 52. Empirical relations used to determine effective-overburden-pressure correction factors for SPT N or N_{60}-values

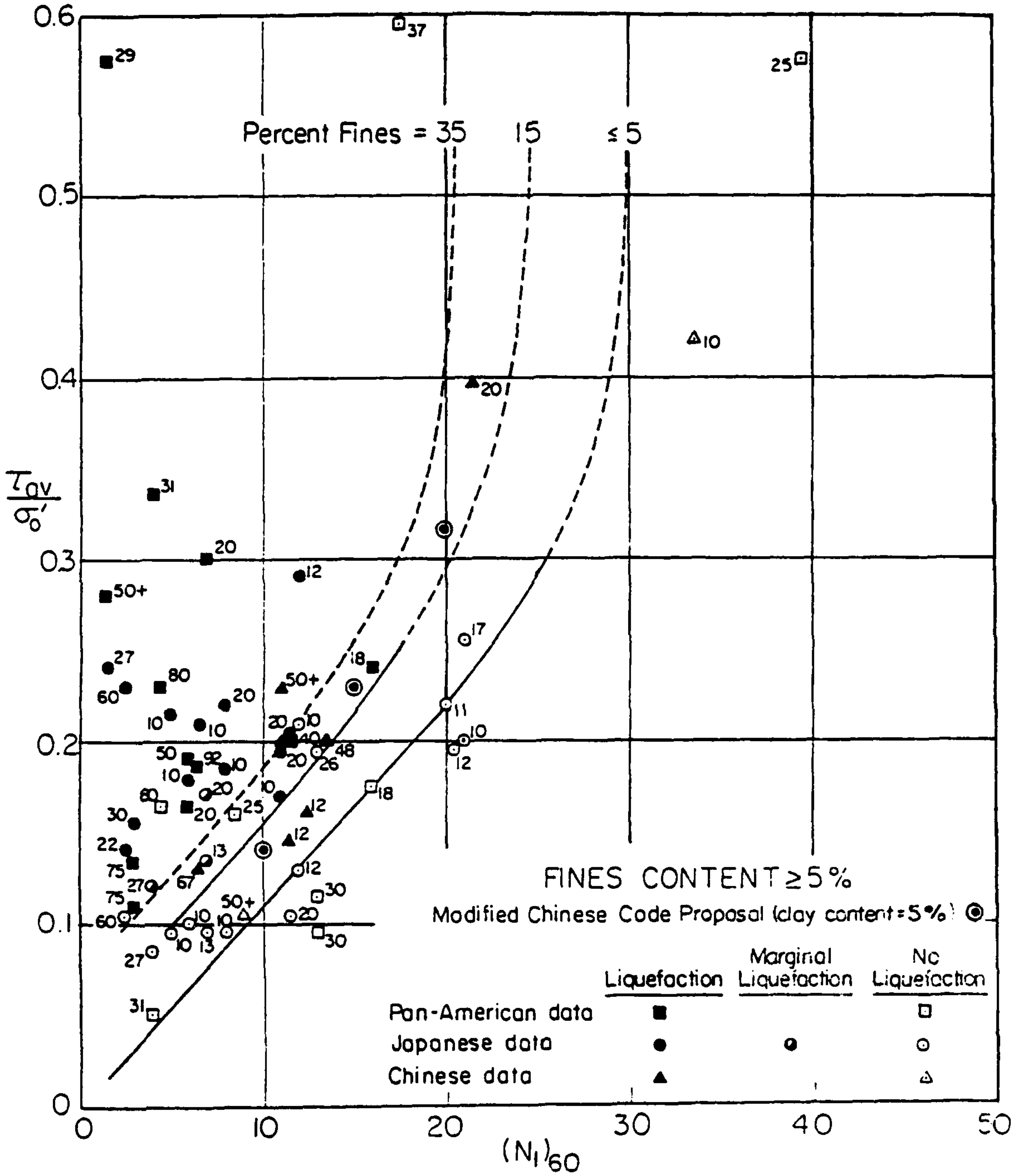


Figure 53. Relationships between stress ratio causing liquefaction and $(N_1)_{60}$ -values for silty sands for M = 7-1/2 earthquakes (from Seed et al. 1985)

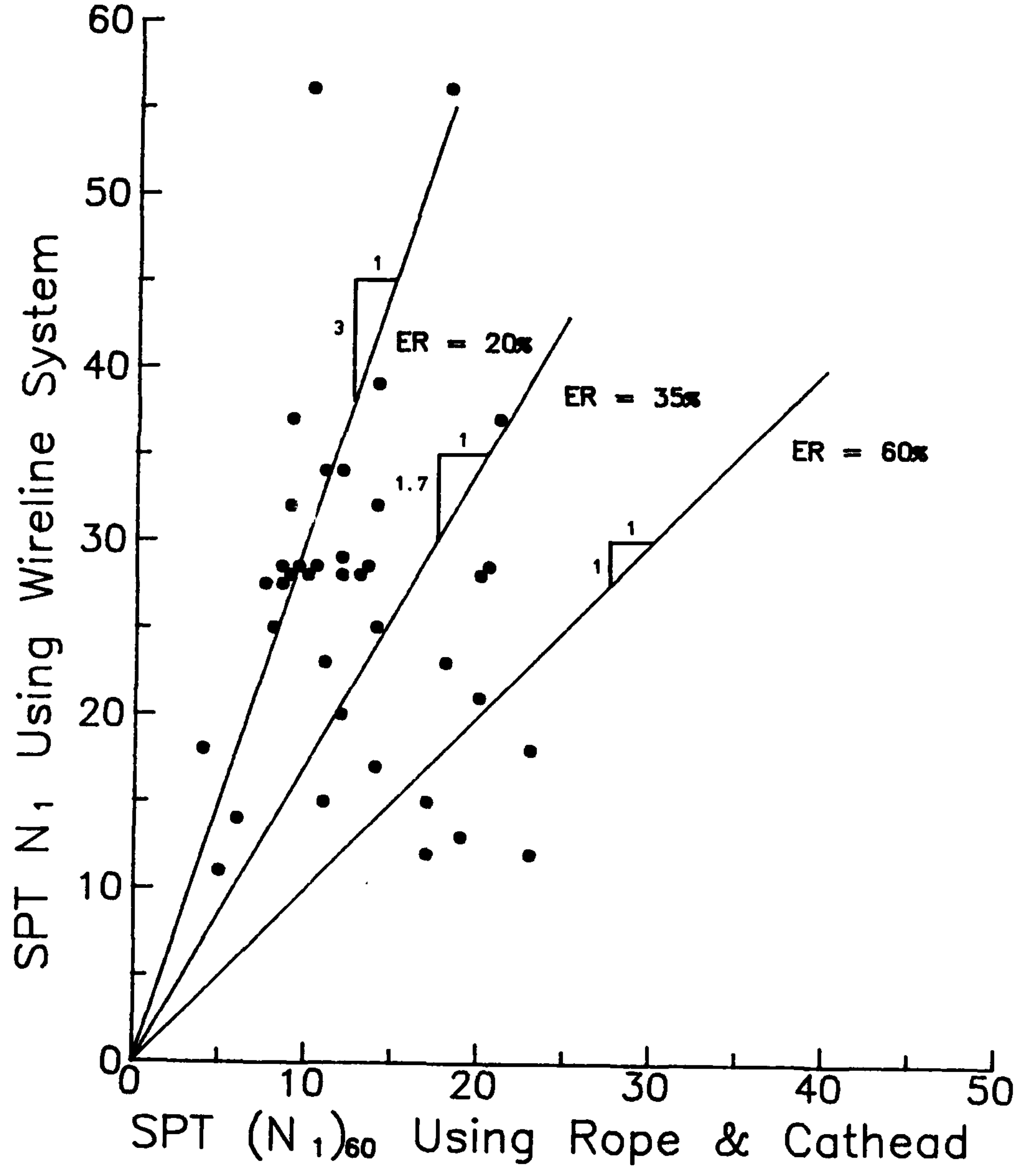


Figure 54. Comparison of blowcounts measured using rope-and-cathead and wireline systems at equal depths

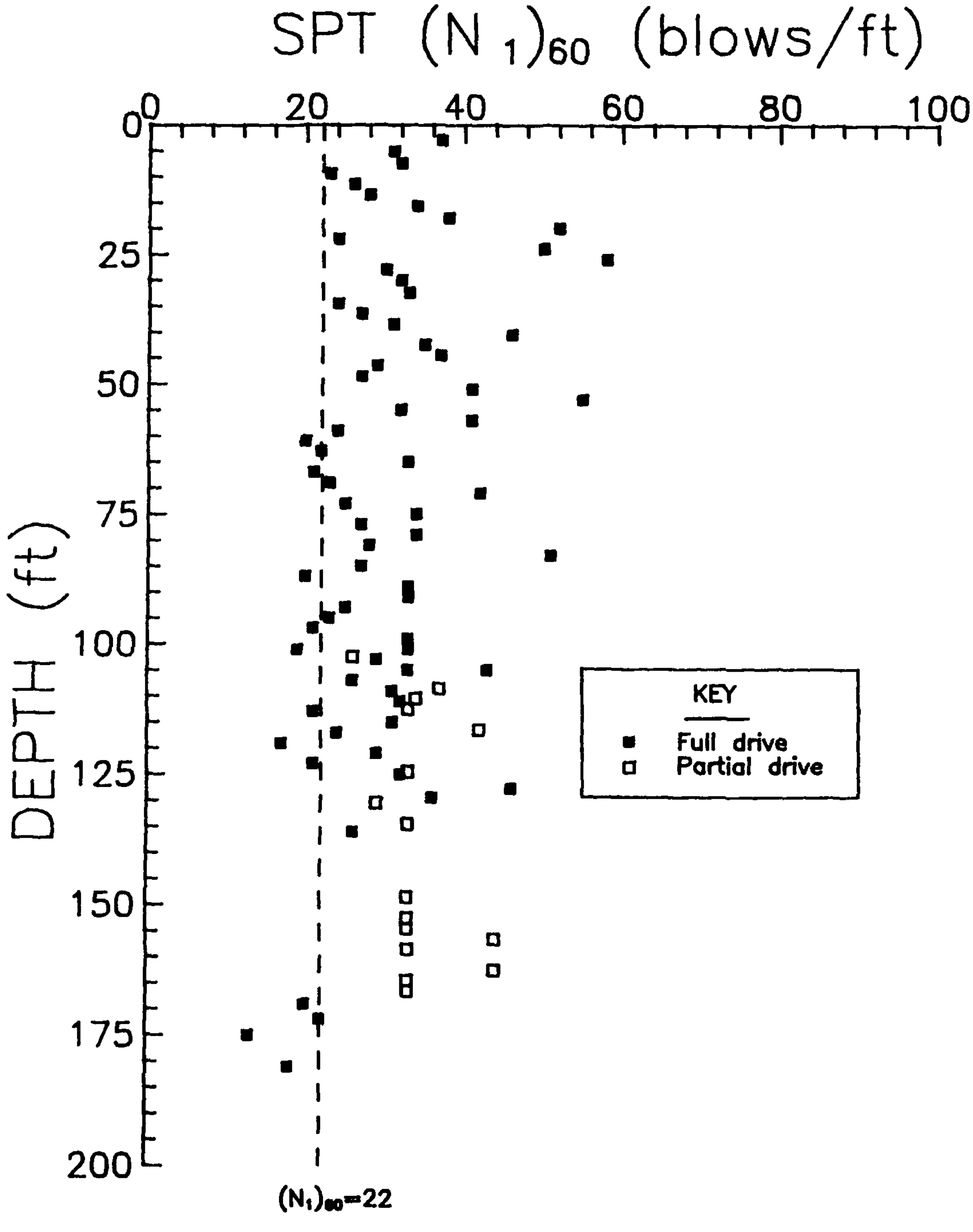


Figure 55. Results of Standard Penetration tests at RWD interface zone modified to represent an equivalent energy of 60 percent and an effective overburden stress of 1 tsf

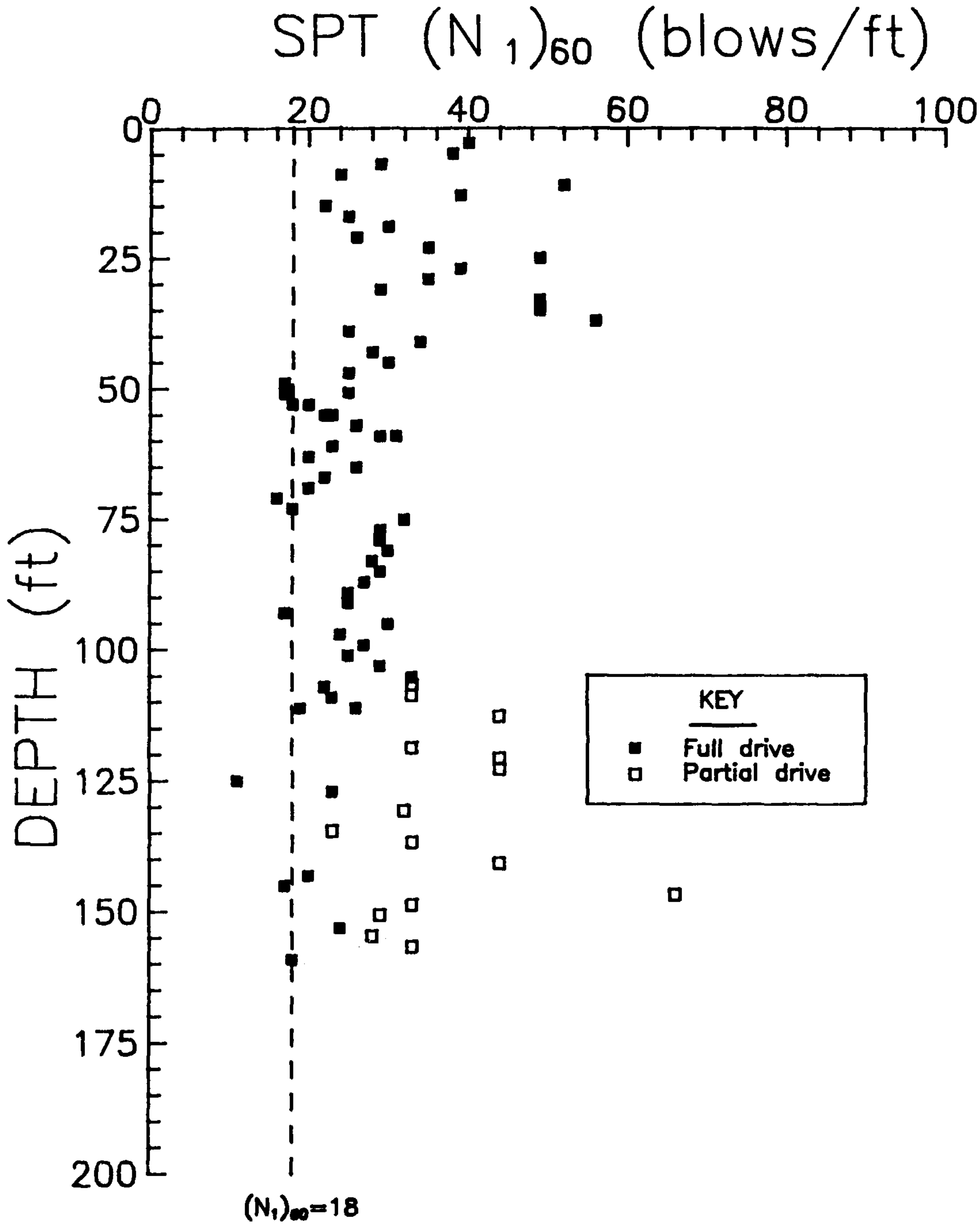


Figure 56. Results of Standard Penetration tests at LWD interface zone modified to represent an equivalent energy of 60 percent and an effective overburden stress of 1 tsf

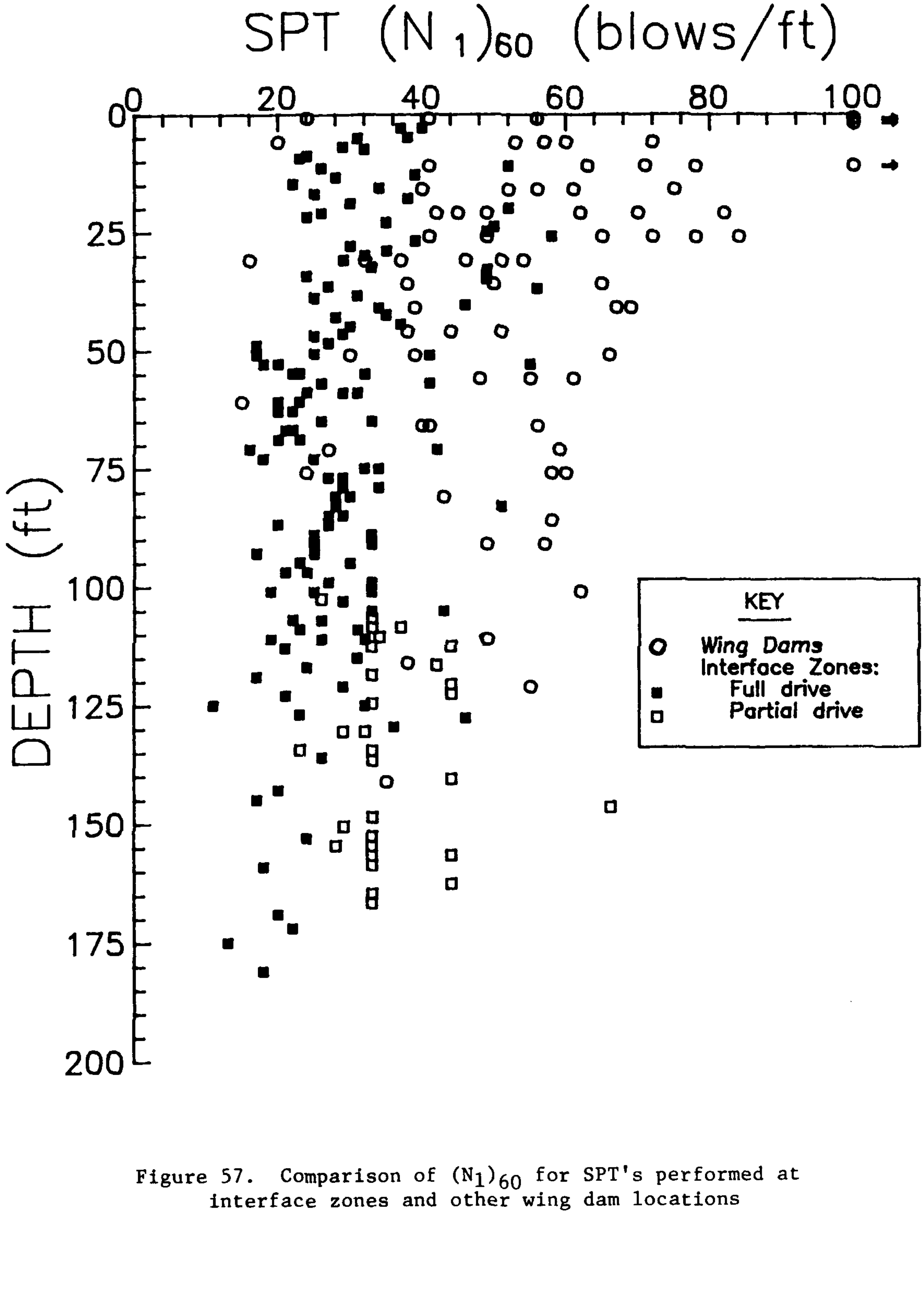


Figure 57. Comparison of $(N_1)_{60}$ for SPT's performed at interface zones and other wing dam locations

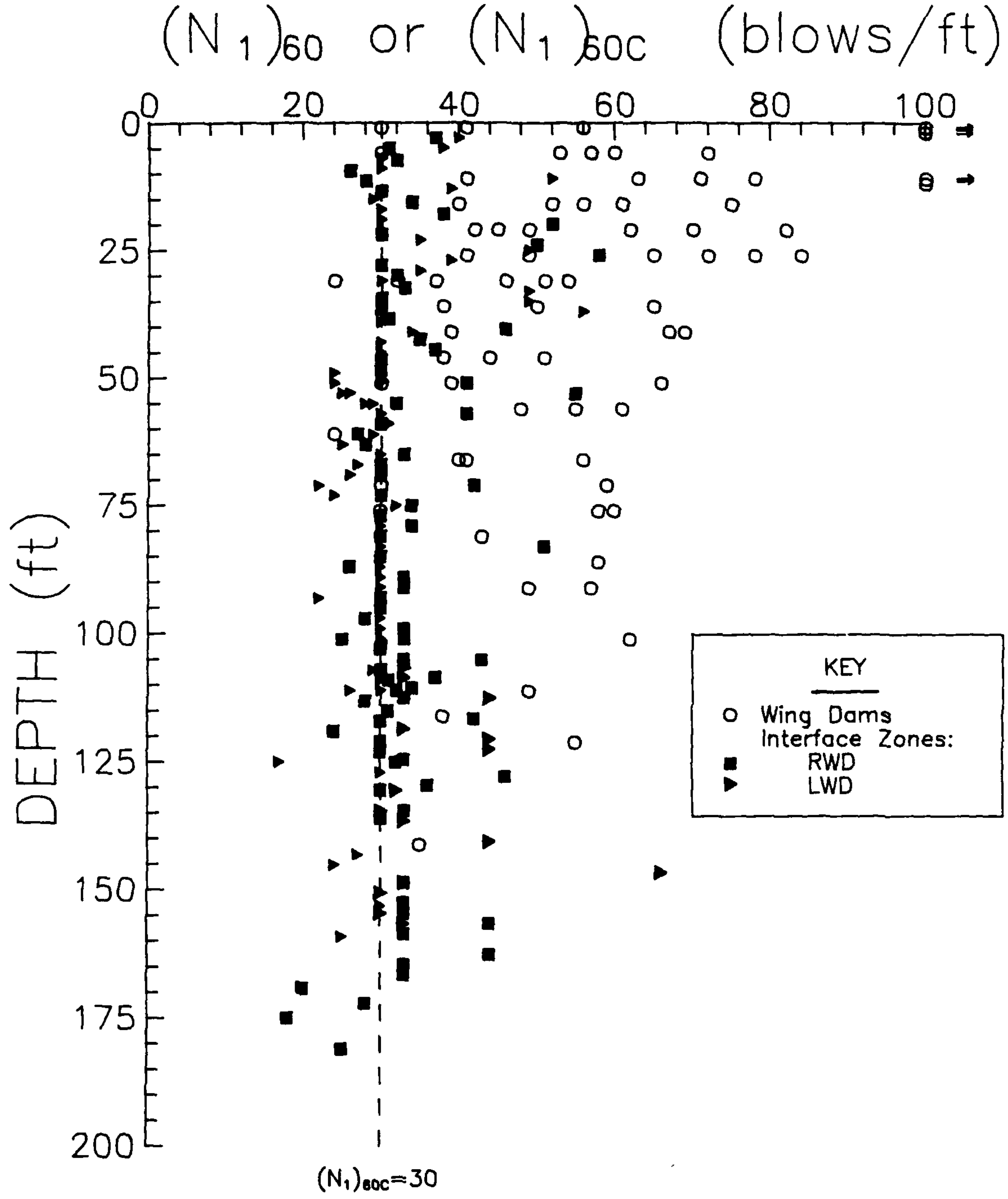


Figure 58. Comparison of $(N_1)_{60c}$ for SPT's performed at interface zones and other Wing Dam locations

APPENDIX A

DOCUMENTATION OF COMMUNICATION WITH MR. OTT BY USACE-SPK PERSONNEL ON 10 SEPTEMBER 1985

SPKED-F 10 Sep 85

MEMORANDUM FOR RECORD

SUBJECT: DSAP - Folsom Dam Wraparounds - Meeting with Mr. John Ott

1. On 10 Sep 85, the undersigned met with Mr. John Ott at his home in Berkeley, CA. This meeting was to discuss the placement of materials in the Folsom Dam wraparounds. At a meeting in Sacramento on 19 Jun 85 with Waterways Experiment Station (WES) personnel and technical specialists Dr. H. Bolton Seed and Dr. Ralph B. Peck, concerns about wraparound placement methods were expressed. It was agreed that interviews of people present during construction would be sought. Mr. Ott was in charge of the Folsom Dam Project Lab during construction. He later became Director of SPD Laboratory.

2. At the meeting with Mr. Ott construction photos as presented by WES in the meeting of 19 Jun 85 were shown. After reviewing the photos Mr. Ott made the following statements about the wraparound; his recall of events was remarkably good.

a. The earthwork techniques at the embankment to concrete wraparound were of concern to the Corps of Engineers. Special care was taken to insure a well constructed wraparound and Corps personnel paid particular attention to this area.

b. Cleanup before placement of embankment materials was thorough.

c. Areas not accessible to heavy compaction equipment were compacted using hand tampers on material placed in 4 inch lifts.

d. Tests were made to insure materials were properly compacted.

e. Materials previously placed by and for the contractors benefit were removed and placed in approximately horizontal lifts - this included materials with a substantial height differential between the end of the concrete wall and upstream or downstream of the end monoliths and materials behind retaining walls.

f. Mr. Ott expressed confidence that the wraparound was well constructed and closely monitored by Corps personnel.

Kenneth L. Neal

KENNETH L. NEAL
Asst. Chief, Geotechnical Branch

Gilbert A. Avila

GILBERT A. AVILA
Soils Des Section

CF:
Dr. Ralph B. Peck
Dr. H. Bolton Seed
WES, ATTN: Maryellen Hynes-Griffin
John E. Ott
cc: Eng Div (2)
Civ Des Br
Geotech Br

APPENDIX B

SELECT ENGINEERING DIVISION DAILY LOGS

26 October 1953

DAILY LOGS

SUBJECT: Folsom Project - Right and Left Wing Envelopment Construction

1. On 26 October 1953, Mr. H. A. Johnson and the undersigned attended a discussion of the proposed construction of the Right and Left Wing Dam Envelopment Sections at the Folsom Project. Messrs. Beatie and W. Clark of the Folsom Project and Colonel Beckham and Mr. O. H. Hart of the Construction-Operations Division, were also present.

2. The contractor's plan, as shown on his drawing No. D-71, dated 18 September 1953, entitled "Proposed Construction Left Wing Dam Envelopment Section," requires first-state construction of the upstream side of the embankment to elevation 440 with the decomposed granite (Zone G) section partially completed and constructed on the downstream side to the same slope, and in the same plane, as the downstream face of the concrete dam (1 on 0.70). Second-state construction consists of raising the embankment to full height (elevation 480.5) and section after the trestle has been removed. Mr. Johnson pointed out the deficiencies in the contractor's plan as follows:

a. The proposed plan does not permit keeping the embankment 50 to 70 feet above the lowest concrete block as required by the specifications as the embankment will not be extended above elevation 440 at the Left Wing Envelopment until after the concrete has been completed and the trestle removed.

b. The slopes of approximately 1 on 0.70 for the decomposed granite core are too steep to be safe and in addition will not permit adequate compaction to the end of the fill as it is brought up. It was stated that the uncompleted downstream decomposed granite core could be temporarily constructed during first stage to a 1 on 1-1/2 slope as long as water was not to be ponded behind the embankment. It was agreed by all present that the problem of keeping the embankment 50 to 70 feet above the lowest concrete block did not exist for the coming wet season but the following year would be critical. It was further agreed that the decomposed granite could be constructed to partial section with a downstream slope no steeper than 1 on 1-1/2 for the coming wet season; however, the following year when there is a change that water will be behind the embankment it would have to be constructed to full cross-section, to a height of 50 to 70 feet above the low block.

3. The portions of the right and left envelopment sections now under construction were inspected. Beyond the right end of the concrete dam within the abutment excavation area decomposed granite fill has been placed to a depth of 10 to 15 feet above the bottom of the rock excavation. No material has been placed along the downstream side of the concrete. In order to permit the contractor to place as much fill as possible in the first-stage construction it was agreed that he would be allowed, if he so desired and at no extra expense to the Government, to place the decomposed granite, starting as far riverward along the downstream concrete face as the trestle bents permit, on a 1 on 1-1/2 slope extending around the end of the concrete dam with the height to be limited by the intersection of the 1 on 1-1/2 slope with the required

upstream decomposed granite core slope of 1 on 0.75. In the second-stage construction, the embankment would be brought to full cross-section to the height required by the specification.

4. On the left abutment, the decomposed granite has been brought around the end and upstream face of the concrete to a height of approximately 40 feet. The fill has been brought up flush with the downstream face of the concrete dam on a slope of 1 on 0.70. Along the upstream face of the concrete dam, the decomposed granite fill and 12-foot wide transition (Zone F) have been constructed on a slope of 1 on 0.75; however, the dredge tailing shell (Zone E) has not been placed because of the existing timber crib in Monolith No. 23 and also because fill cannot yet be placed below the crib. The slopes both on the downstream and along the upstream face are being constructed too steep to be safe and get compaction at the edge of the fill, and it was agreed that fill operation would be discontinued in this area to approximate Station 303+00. Beyond Station 303+00, the Left Wing Dam would be constructed leaving a steep slope at Station 303+00. Then when the trestle is removed allowing more room to operate, the fill in the envelopment area will be compacted by rolling perpendicular to the axis of the dam. It is not believed that the decomposed granite fill placed on the above-mentioned steep slopes can be compacted to the required density close to the edge of the fill due to lack of restraint and it is further recommended that the densities of the fill be thoroughly checked and the material removed and recompacted both upstream and along the upstream face if it does not meet the compaction requirements.

5. Mr. Stinson, the contractor's superintendent, was informed of the decisions with regard to the first and second stages of the envelopment construction. He agreed to first-stage construction to 1 on 1-1/2 slopes on the right abutment and to suspend operations between the concrete dam and approximate Station 303+00 on the left abutment. He further stated that when he received the plans on the treatment of the slide area in the left wing downstream envelopment area, he would prepare a plan of operation to construct the embankment to full cross-section above the existing trestle, in accordance with the specification requirements. This would entail removing the left end of the construction trestle after completion of the end dam monoliths but before completion of all dam concrete, by the construction of a railway for concrete cars on the downstream slope of the completed embankment for access to the remaining construction trestle. It will entail removing the right end of the construction trestle after completion of the right end monolith but before completion of all dam concrete.

JAMES A. RICHARDSON, JR.
Chief, Soils Section

11 November 1953

DAILY LOG

SUBJECT: Folsom - Right and Left Wing Envelopment Construction

1. Reference is made to the Daily Log written by the undersigned, dated 26 October 1953, subject as above. On 11 November 1953, the undersigned accompanied Mr. Dana Leslie of the Division Office on an inspection of the envelopment construction. The problems in regard to the partially constructed steep decomposed granite slopes, as discussed in the above-referenced daily log, were discussed with Messrs. W. Clark and J. Ott of the Folsom Project who were present.

2. Mr. Clark said that the contractor's plan (as approved in a District meeting on 4 November 1953) is to place the decomposed granite core material (Zone C) within the area of the right abutment concrete dam excavation landward of the right end of the concrete dam to approximate elevation 380. Elevation 380 is the approximate original ground surface in this area. No fill will be placed along the downstream face of the concrete dam resulting in practically a vertical slope in the plane of the end of the concrete dam from the vertical slope in the plane of the end of the concrete dam from the downstream, landward corner of the dam to the downstream abutment excavation slope opposite this corner. No more fill will be placed in the left wing envelopment area. Mr. Clark stated that project personnel and the contractor realized that it would be necessary to remove loose material from the outer portions of the slope not meeting compaction requirements.

3. It was pointed out to Messrs. Clark and Ott by Mr. Leslie and the undersigned, that because of the probable low densities obtained at the outer portions of the steep slopes, it would be necessary to excavate material already placed in both the right and left envelopments to obtain material of the required density. In addition, it was pointed out that the embankments were designed on the basis of the decomposed granite fill materials having a maximum shear strength of tan $\phi = 0.70$ and that any movement which produced small strains in this material could result in a reduction to the ultimate shear strength which is in the order of tan $\phi = 0.58$, i.e., a small movement of the fill (which might not be detected in the fill) due to the present steep slopes could result in weakened planes within the embankment. In order to insure that weakened planes and also low density material will not

(The continuation of this Daily Log is unavailable and therefore, could not be printed.)

30 November 1953

DAILY LOG

1. A conference was held on the subject of curtain grouting of the Main Dam at Folsom. Present at the conference were the following: Messrs. Brooks, H. Johnson, Newman, B. Clark, Dettmer, Beatie, Burke, Roddy, Heffington, Hart, and Holdredge. The discussion was largely concerned with the interrelationship between the grouting operation and the Main Dam contractor's schedule.

2 through 6. Not printed.

7. Placement of the wrap-around section of the right abutment should start not later than 1 October 1954, and it will, therefore, be in progress during at least part of the curtain grouting operation.

8. It was pointed out that it probably will not be possible to place the wrap-around fill in the downstream side of the left abutment prior to about April 1955, unless it is placed by hand-tamping methods, because the dam will not be topped out so that the trestle may be removed until January 1955. In this connection, it was pointed out that one of the controlling factors in the placement of concrete on the left abutment is the additional consolidation grouting which is planned under Monolith Nos. 22 and 23. As now scheduled, this will probably be done between 15 February and 1 April 1954, and concrete placement in those monoliths cannot start until its completion.

9 through 12. Not printed.

CLAIRE P. HOLDREDGE
Chief, Geology Section

APPENDIX C

EXCERPTS OF "REPORT OF SOIL TESTS" BY SOUTH PACIFIC DIVISION LABORATORY (1986)

CONTENTS

REPORT
OF
SOIL TESTS

FOLSOM DAM
LABORATORY PROGRAM

JULY 1986

AUTHORIZATION

1. Results of tests reported herein were requested by the Sacramento District in laboratory requests Nos. SPKED-F-83-13 dated 7 October 1982, and SPKED-F-84-68 dated 29 March 1984.

SAMPLES

2. Fifty-five ring density samples in sacks were received during the period 31 January 1983 to 8 December 1984. Six hundred sixty-two tube, jar, and bag samples were received during the period 17 April 1982 to 28 March 1984. Identification of the samples which were tested are shown on the Soil Test Result Summary plates.

TESTING PROGRAM

3. The program was in general accordance with the test request. Tests included sieve analysis, Atterberg limits, specific gravity, and stress controlled cyclic triaxial compression, monotonic R triaxial compression, unconfined compression, relative density, and compaction.

TEST METHODS

4. a. Sieve analysis, Atterberg Limits, Field Unit Weight, Triaxial Compression, Compaction, Specific Gravity, Relative Density, and Cyclic Triaxial Compression. Testing conformed to the procedure described in Engineer Manual, EM 1110-2-1906, Laboratory Soil Testing," 30 November 1970.

b. Classification. The soil was classified in accordance with "The Unified Soil Classification System," TM No. 3-357, Appendix A, April 1980.

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT Folsom Dam- Right Wing Dam **DATE**

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer											Liquid Limit	Plasticity Index	Field Moist. %
						Gravel					Sand					Fines			
			From	To		1 1/2	1	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200			
87216	R-1	2	3.8	6.3	Silty Sand(SM)	Sp.	Gr.=	2.70		100	99	91	52	40	29	19		NP	
87219	R-1	5	11.3	13.8	Clayey Sand(SC)	Sp.	Gr.=	2.70		100	99	93	60	48	37	27	25	9	
87220	R-1	6	13.8	16.5	Silty Sand (SM-SW)					100	98	86	41	28	19	11	23	2	
87223	R-1	9	20.8	23.3	Silty Sand(SM)	Sp. 100	Gr.= 99	2.7 99	99	98	96	88	51	45	36	26	19	2	
87229	R-1	15	36.3	38.7	Clayey Sand (SM-SC)	Sp.	Gr.=	2.70	100	99	91	76	40	30	21	15	26	6	
87232	R-1	18	46.2	49.1	Clayey Sand (SM-SC)			100	99	99	97	85	53	40	31	21	22	6	3.5
87234	R-1	20	54.6	57.9	Silty Sand(SM)	Sp.	Gr.=	2.71 100	99	98	97	91	60	46	43	24	24	2	
87279	R-2	18	89.6	92.3	Silty Sand(SM)	Sp.	Gr.=	2.73	100	99	99	92	59	45	33	21		NP	
87280	R-2	19	107.2	109.7	Clayey Gravelly Sand (SC)	79	79	79	79	79	77	69	41	31	24	17	28	9	
87280	R-2	19	107.7	109.7	Silty Sand(SM)					100	97	91	54	41	31	23		NP	
87615	R-3	5	12.8	17.7		Sp.	Gr.= 100	2.74 98	98	96	89	85	46	31	21	13			
87616	R-3	6	17.7	19.5	Silty Sand(SM)	Sp.	Gr.=	2.74	100	98	96	92	52	35	24	15	23	2	
87617	R-3	7	19.5	20.0		Sp.	Gr.=	2.74		100	95	87	51	38	28	19			

SPD Form 66A
1 May 83

PLATE 25

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT Folsom Dam - Right Wing Dam **DATE**

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Gravel 1 1/2	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
87618	R-3	8	20.0	21.2	Clayey Sand(SC)	Sp.	Gr.=	2.74		100	98	93	58	44	32	23	29	8	
87619	R-3	9	21.2	23.7	Clayey Sand (SM-SC)	Sp.	Gr.=	2.74	100	99	95	88	57	45	35	25	26	7	
87623	R-3	13	27.0	27.6	Clayey Sand (SM-SC)	Sp.	Gr.=	2.74		100	98	92	56	42	31	21	28	6	
87624	R-3	14	28.0	29.3	Clayey Sand (SM-SC)	Sp.	Gr.=	2.74			100	95	60	45	34	24	28	7	
87625	R-3	15	29.3	32.6	Gravelly Silt (ML)	Sp.	Gr.= 100	2.74 97	95	92	79	78	75	74	73	73	25	2	
87626	R-3	16	32.6	34.2	Silty Sand(SM)	Sp.	Gr.=	2.74		100	98	93	64	51	40	29	21	3	
87627	R-3	17	34.2	39.5	Silty Sand(SM)	Sp.	Gr.=	2.74		100	96	88	47	34	24	16	25	3	
87628	R-3	18	39.5	42.6	Silty Sand(SM)	Sp.	Gr.= 100	2.74 97	97	96	85	74	39	29	22	15	25	2	
87631	R-3	21	46.5	47.0		Sp.	Gr.=	2.73			100	94	62	47	35	24			
87632	R-3	22	47.0	49.0	Clayey Sand (SC-SM)	Sp.	Gr.=	2.73		100	98	98	93	91	~~89~~	~~28~~ 26	25	5	

SPD Form 66A

PLATE 26

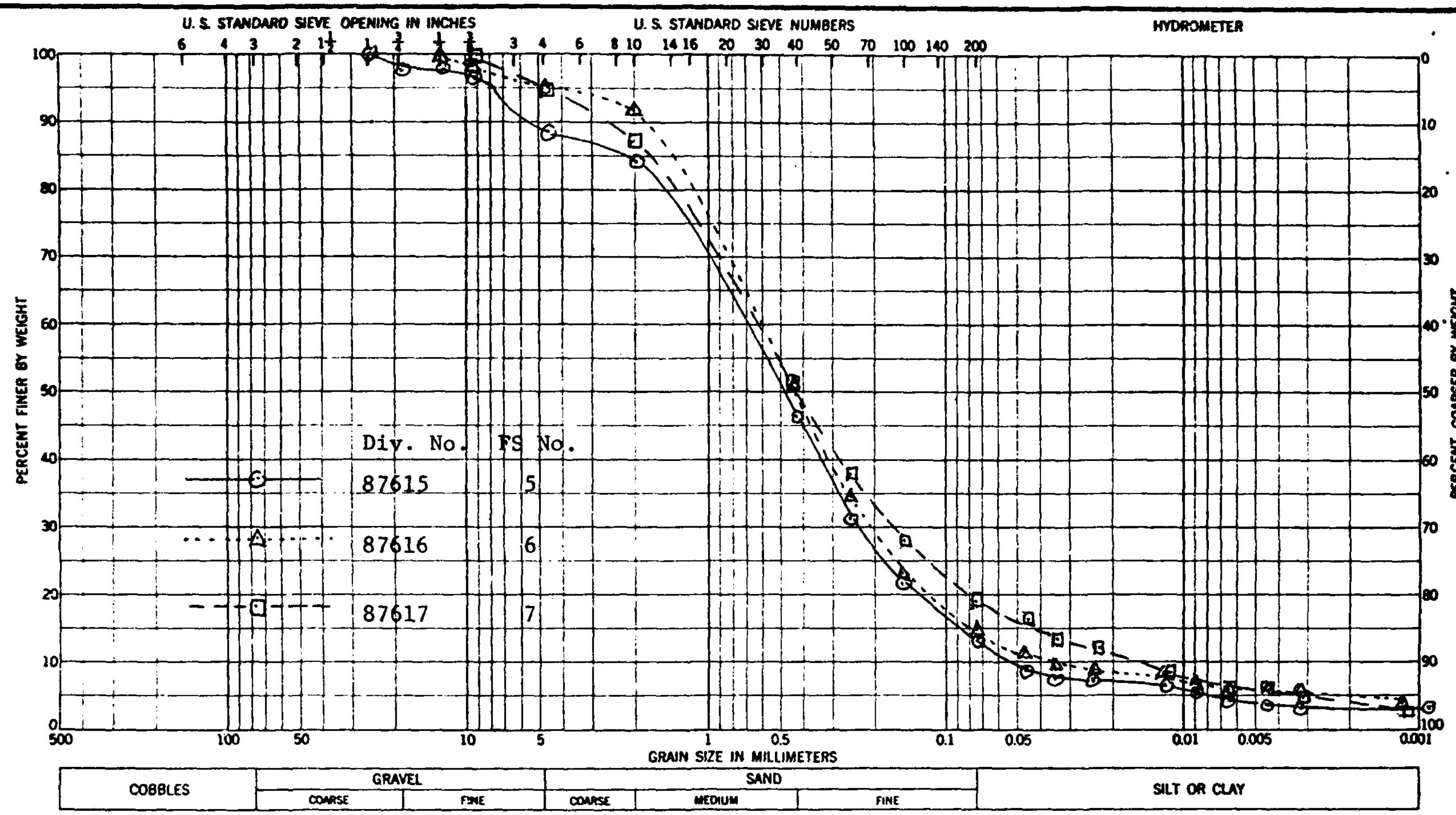
U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT Folsom Dam - Right Wing Dam DATE

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel					Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
87452	R-4	4	54.0	54.5	Clayey Sand (SC)	Sp.	Gr.=	2.73 100	98	95	94	90	63	50	38	28	27	9	
87453	R-4	5	54.5	56.7	Clayey Sand (SC)	Sp.	Gr.= 100	2.71 94	93	92	89	80	50	38	28	20	28	9	
87458	R-4	10	64.0	67.5	Clayey Sand (SM-SC)	Sp.	Gr.= 100	2.71 96	94	94	91	86	54	41	32	22	23	4	
87459	R-4	11	67.5	68.6	Clayey Sand (SC)	Sp.	Gr.=	2.71			100	98	73	61	50	38	25	8	
87462	R-4	14	74.6	76.0	Silty Sand (SM)	Sp.	Gr.=	2.71	100	99	94	80	43	33	24	16	23	3	
87464	R-4	16	76.5	78.4	Silty Sand (SM)	Sp.	Gr.=	2.72		100	97	87	59	47	37	27	24	3	
87469	R-4	21	84.0	88.0	Silty Sand (SM)	Sp.	Gr.= 100	2.72 97	97	97	95	87	54	42	31	20	24	2	
87472	R-4	24	94.0	94.8	Silty Sand (SM)	Sp.	Gr.=	2.72 100	99	99	96	90	56	43	34	25	20	3	
87473	R-4	25	94.8	98.0		Sp.	Gr.=	2.73		100	97	91	55	41	30	21			
87478	R-4	30	104.0	105.5	Silty Sand (SM)	Sp.	Gr.=	2.70 100	99	99	97	89	61	49	39	28	24	3	
87479	R-4	31	105.5	107.5	Silty Sand (SM)	Sp.	Gr.=	2.70		100	98	94	60	48	38	29	22	3	
87484	R-4	36	114.4	114.9	Silty Sand (SM)	Sp.	Gr.=	2.70 100	97	97	95	90	60	47	36	25	22	2	
87485	R-4	37	114.9	115.5		Sp.	Gr.= 100	2.70 88	88	88	86	77	50	39	30	21			
87486	R-4	38	116.6	117.5	Silty Sand (SM)	Sp.	Gr.=	2.70		100	96	90	62	50	41	32	22	3	
87493	R-4	45	124.3	125.8	Silt (ML)	Sp.	Gr.=	2.70	100	99	98	97	94	92	91	90	22	3	
87494	R-4	46	126.7 ~~127.2~~	127.2 ~~128.8~~	Silty Sand (SM)	Sp.	Gr.=	2.70		100	99	95	64	51	40	29	21	3	
87495	R-4	47	127.2	128.8	Clayey Sand (SM-SC)	Sp.	Gr.=	2.70		100	99	96	66	52	41	28	25	6	

PLATE 27



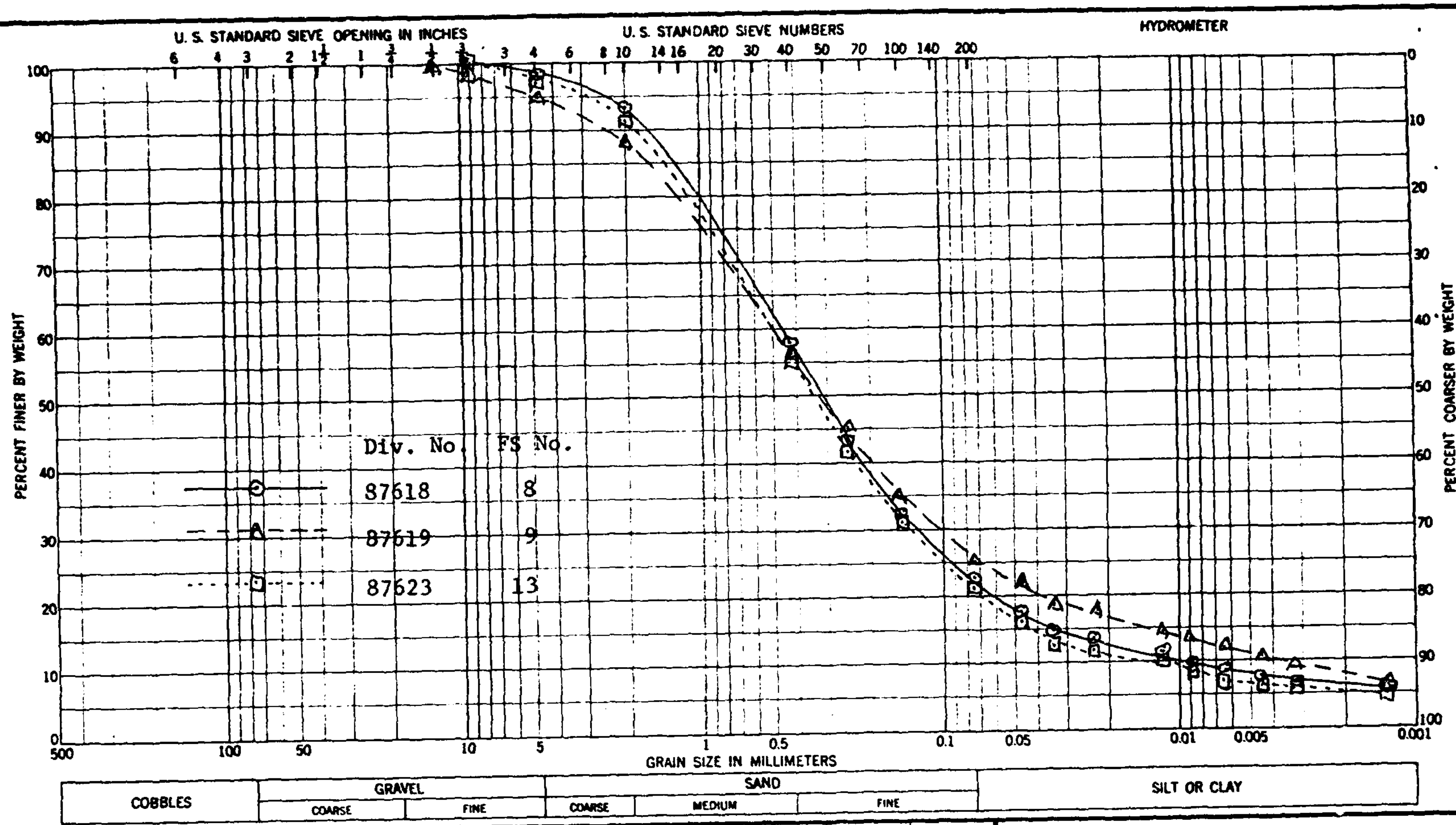
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87615	12.8-17.7	Silty Sand (SM)			NP	
87616	17.7-19.5	Silty Sand (SM)		23	21	2
87617	19.5-20.0	-		-	-	-

Project	FOLSOM DAM
Area	Right Wing Dam
Boring No.	R-3
Date	July 1984

GRADATION CURVES

ENG FORM 1 MAY 63 2087

PLATE 29



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87618	20-21.2	Clayey Sand (SC)		29	21	8
87619	21.2-23.7	Clayey Sand (SC-SM)		26	19	7
87623	27-27.6	Clayey Sand (SC-SM)		28	22	6

Project: FOLSOM DAM

Area: Right Wing Dam

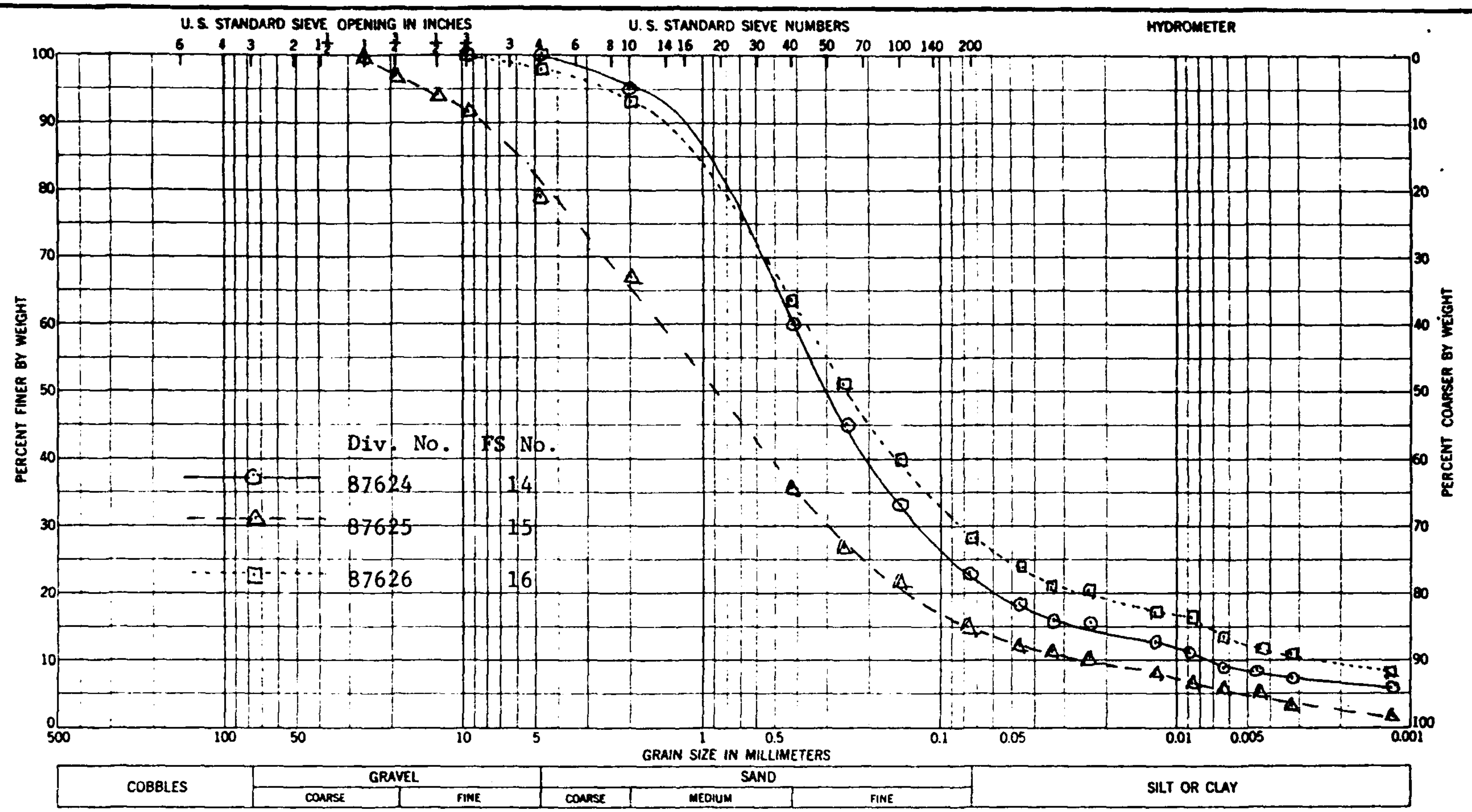
Boring No.: R-3

Date: July 1984

GRADATION CURVES

ENG FORM 1 MAY 63 2087

PLATE 30



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87624	28-29.3	Clayey Sand (SC-SM)		28	21	7
87625	29.3-32.6	Gravelly Silty Sand (SM)		25	23	2
87626	32,6-34.2	Silty Sand (SM)		21	19	2

Project FOLSOM DAM

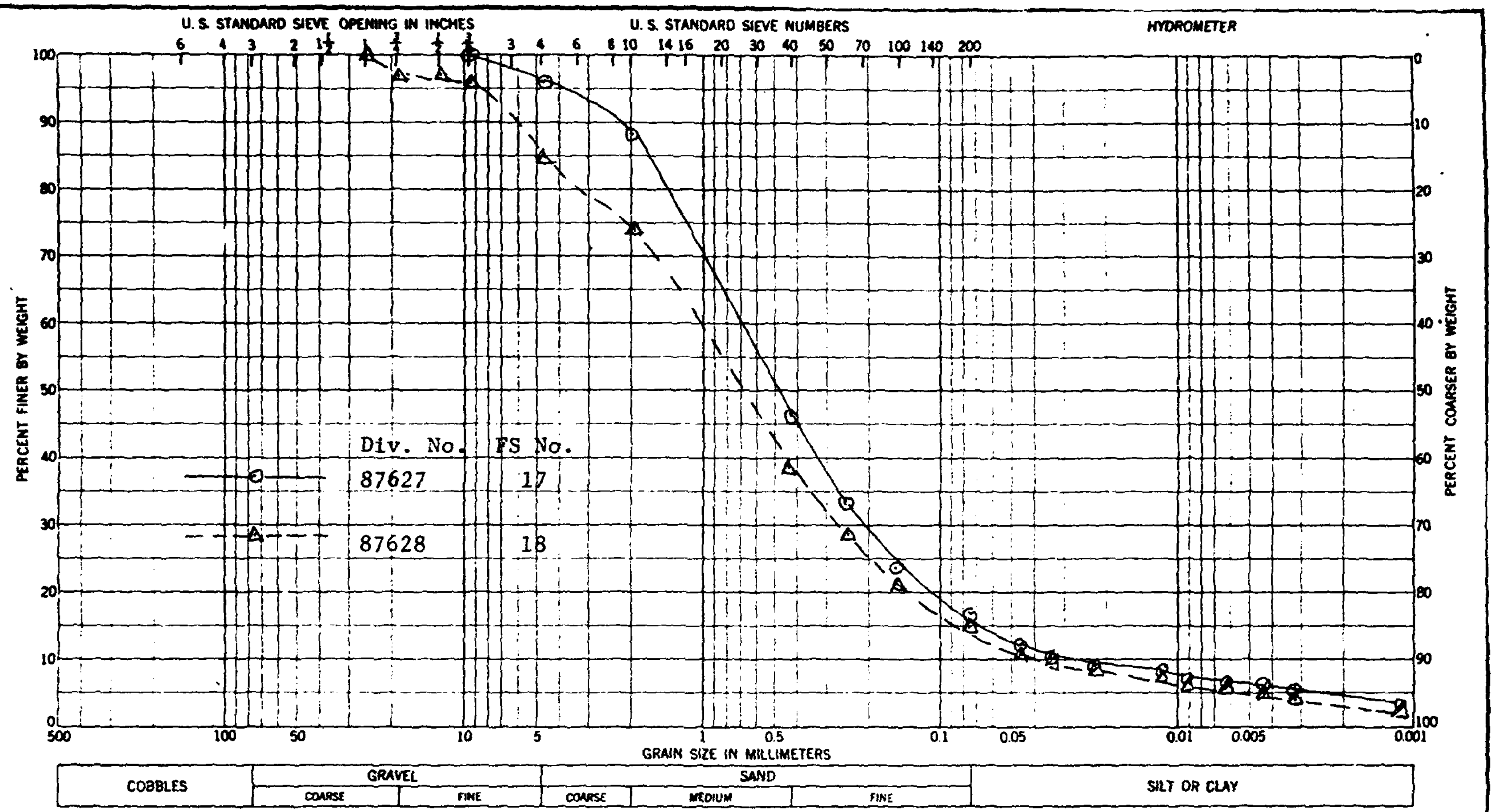
Area Right Wing Dam

Boring No. R-3

Date July 1984

GRADATION CURVES

PLATE 31



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87627	34.2-39.5	Silty Sand (SM)		25	22	3
87628	39.5-42.6	Silty Sand (SM)		25	23	2

Project FOLSOM DAM

Area Right Wing Dam

Boring No. R-3

Date July 1984

GRADATION CURVES

PLATE 32

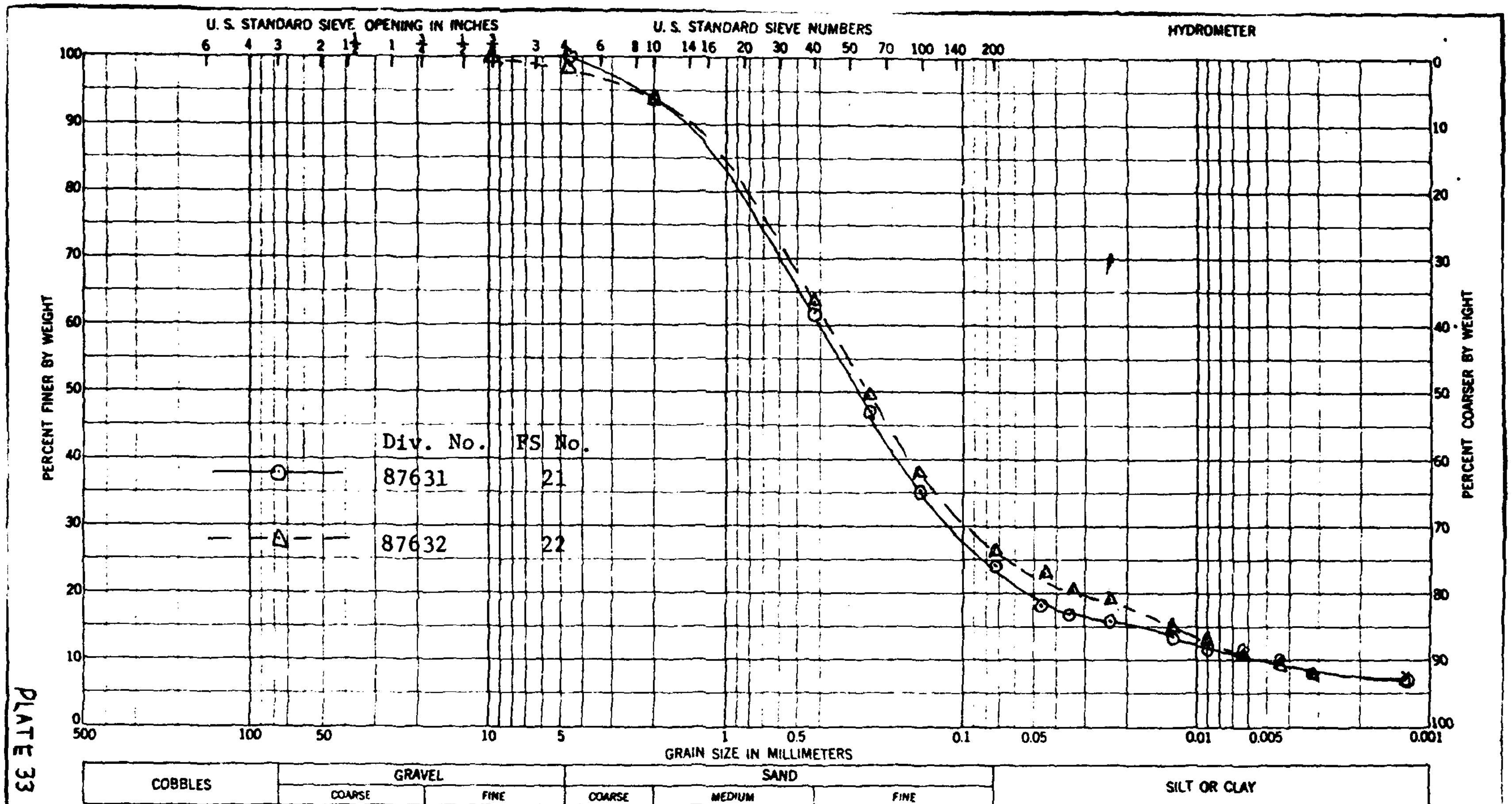
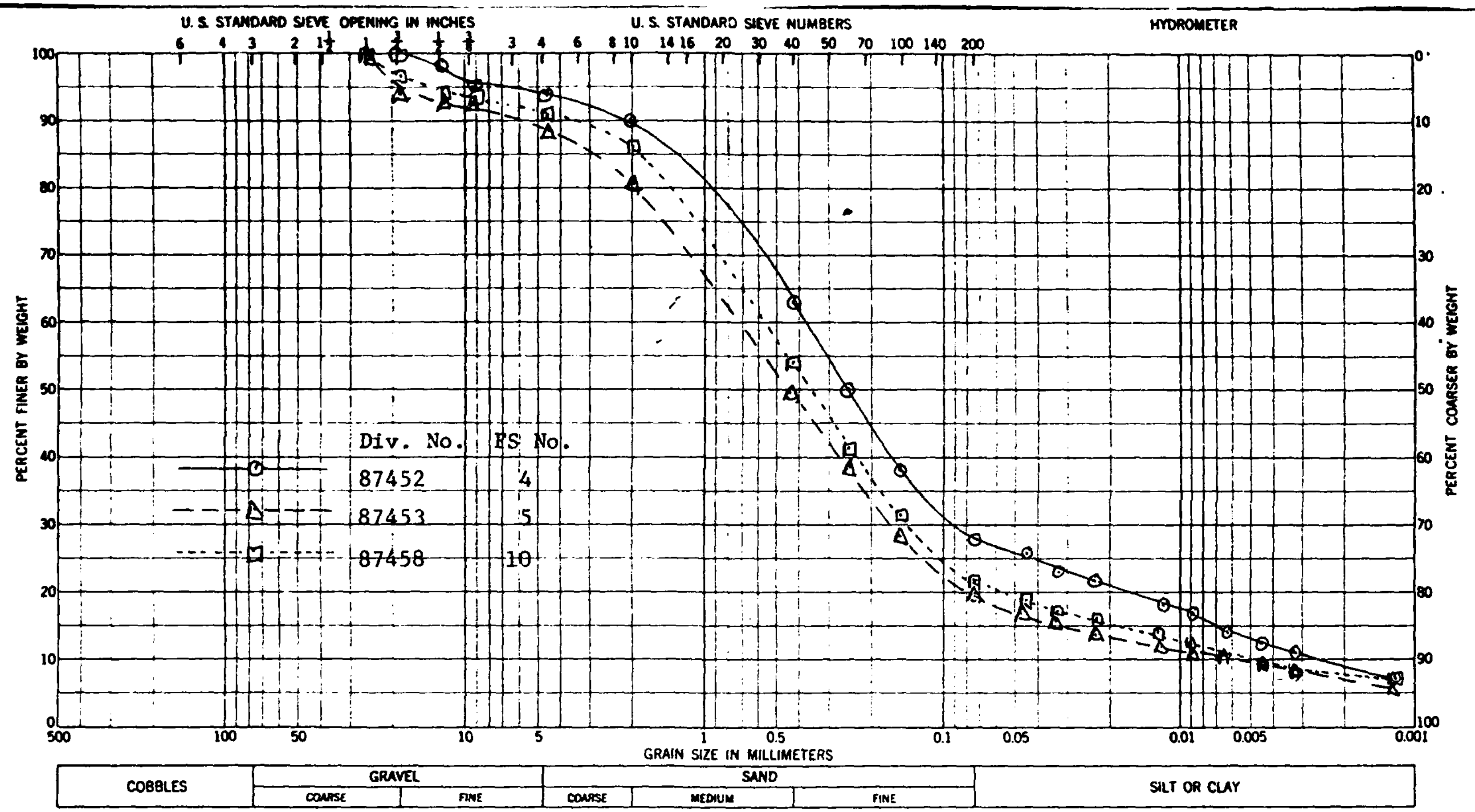


PLATE 33

Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87631	46.5-47	Silty Sand (SM)			NP	
87632	47-49	Clayey Sand (SC-SM)		25	20	5

Project	FORSOM DAM
Area	Right Wing Dam
Boring No.	R-3
Date	June 1984

GRADATION CURVES



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87452	54-54.5	Clayey Sand (SC)		26	17	9
87453	54.5-56.7	Clayey Sand (SC)		27	18	9
87458	64-67.5	Clayey Sand (SC-SM)		23	19	4

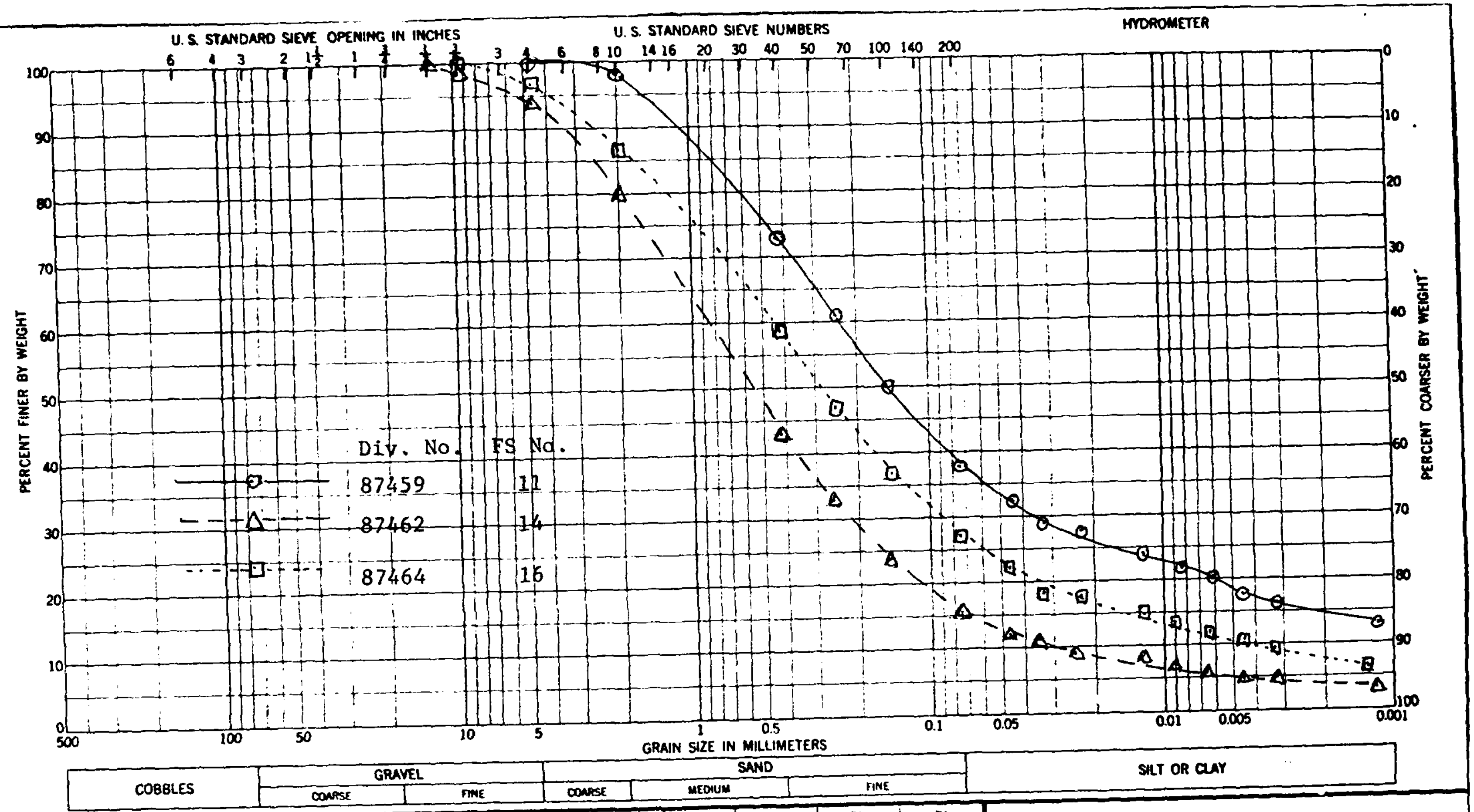
Project FOLSOM DAM

Area Right Wing Dam

Boring No. R-4

Date July 1984

GRADATION CURVES



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87459	67.5-68.6	Clayey Sand (SC)		25	17	8
87462	74.6-76	Silty Sand (SM)		23	21	2
87464	76.5-78.4	Silty Sand (SM)		24	21	3

Project FOLSOM DAM

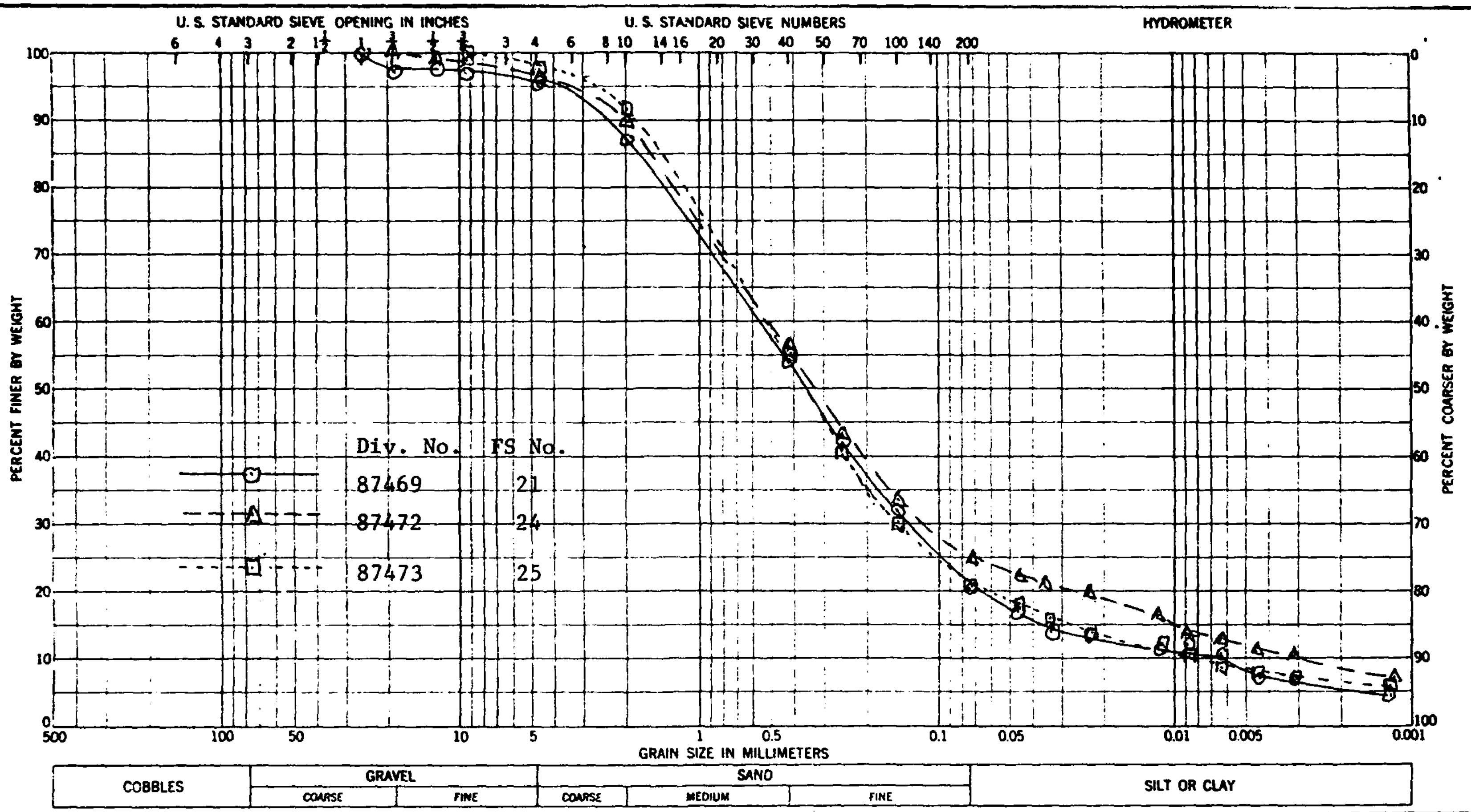
Area Right Wing Dam

Boring No. R-4

Date July 1984

GRADATION CURVES

PLATE 35



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87469	84-88	Silty Sand (SM)		24	22	2
87472	94-94.8	Silty Sand (SM)		20	17	3
87473	94.8-98	Silty Sand (SM)			NP	

Project FOLSOM DAM

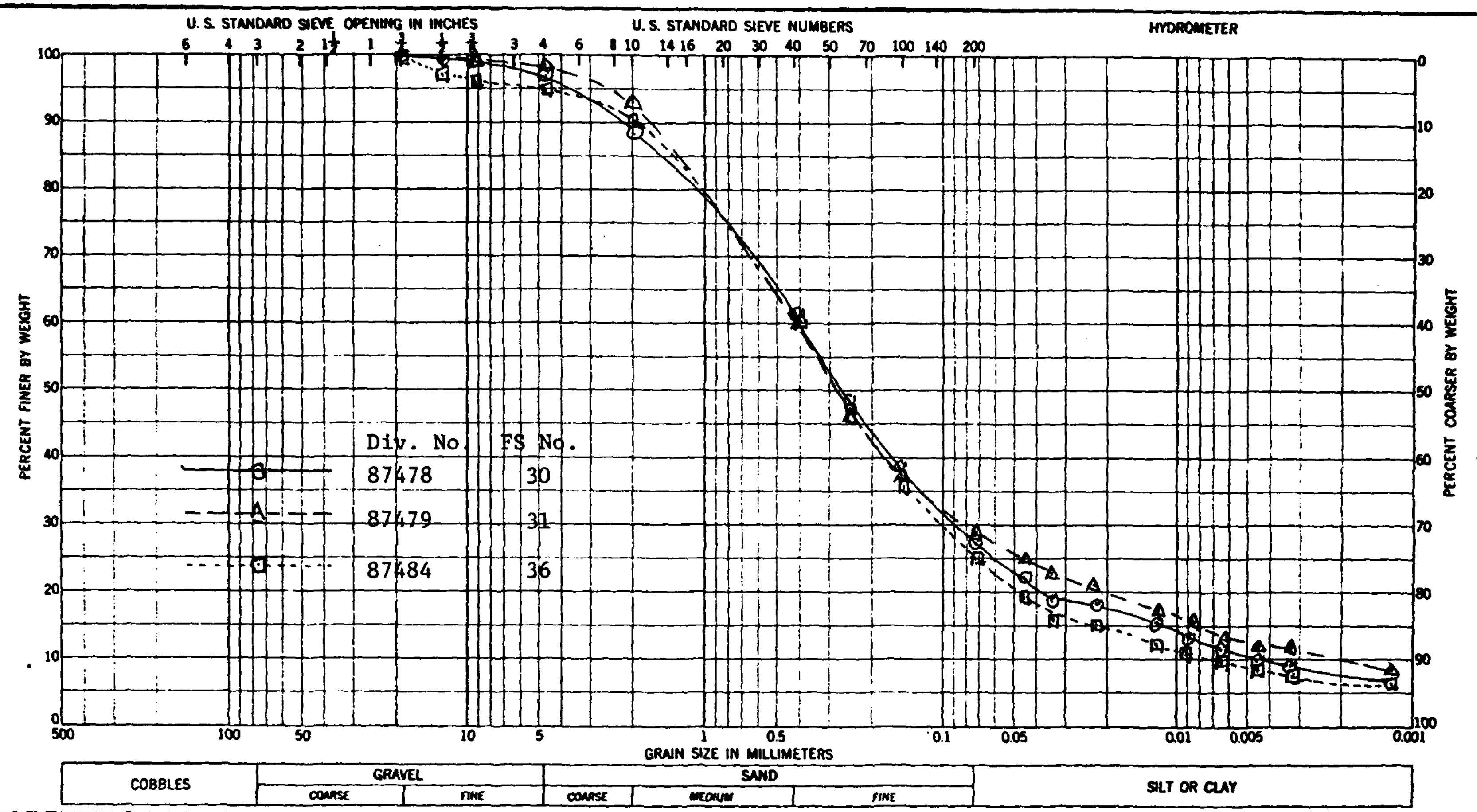
Area Right Wing Dam

Boring No. R-4

Date July 1984

GRADATION CURVES

PLATE 36

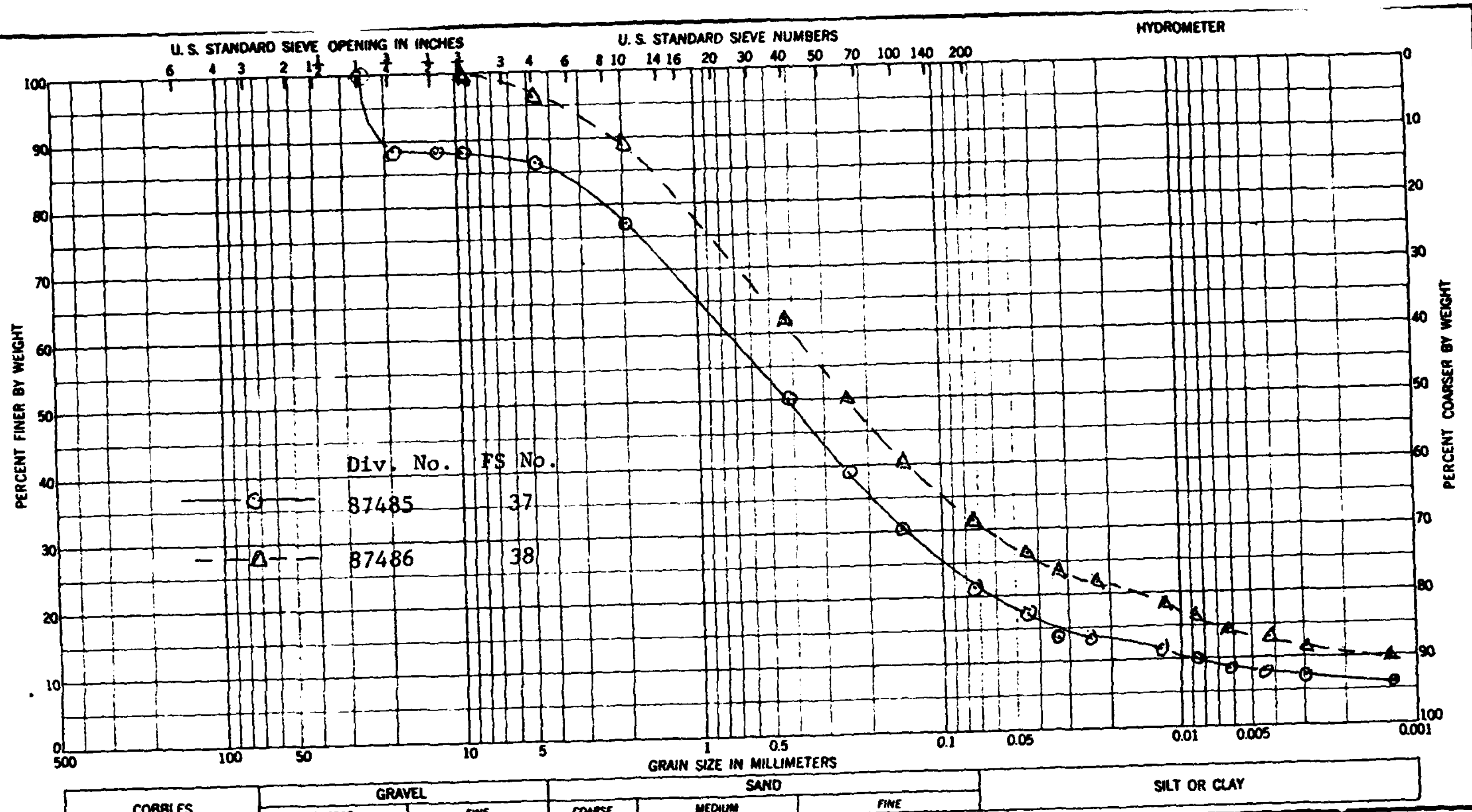


Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87478	104-105.5	Silty Sand (SM)		24	21	3
87479	105.5-107.5	Silty Sand (SM)		21	18	3
87484	114.4-114.9	Silty Sand (SM)		22	20	2

Project	FOLSOM DAM
Area	Right Wing Dam
Boring No.	R-4
Date	July 1984

GRADATION CURVES

PLATE 37



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87485	114.9-115.5	Silty Sand (SM)		22	20	2
87486	116.6-117.5	Silty Sand (SM)		22	19	3

Project FOLSOM DAM

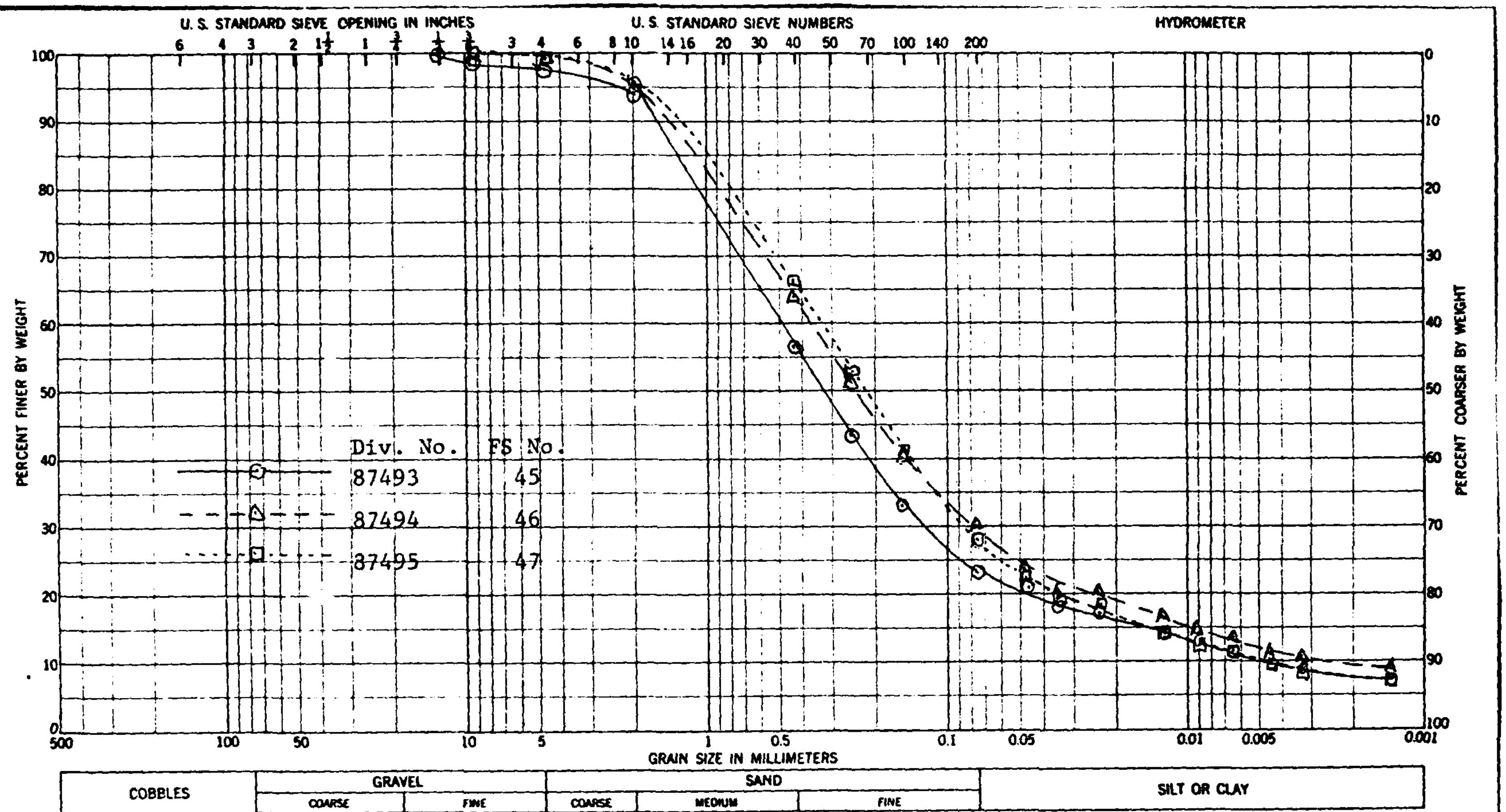
Area Right Wing Dam

Boring No. R-4

Date July 1984

GRADATION CURVES

PLATE 38



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87493	124.3-125.8	Silty Sand (SM)		22	19	3
87494	126.7-127.2	Silty Sand (SM)		21	18	3
87495	127.2-128.8	Clayey Sand (SC-SM)		25	20	5

Project FOLSOM DAM

Area Right Wing Dam

Boring No. R-4

Date July 1984

GRADATION CURVES

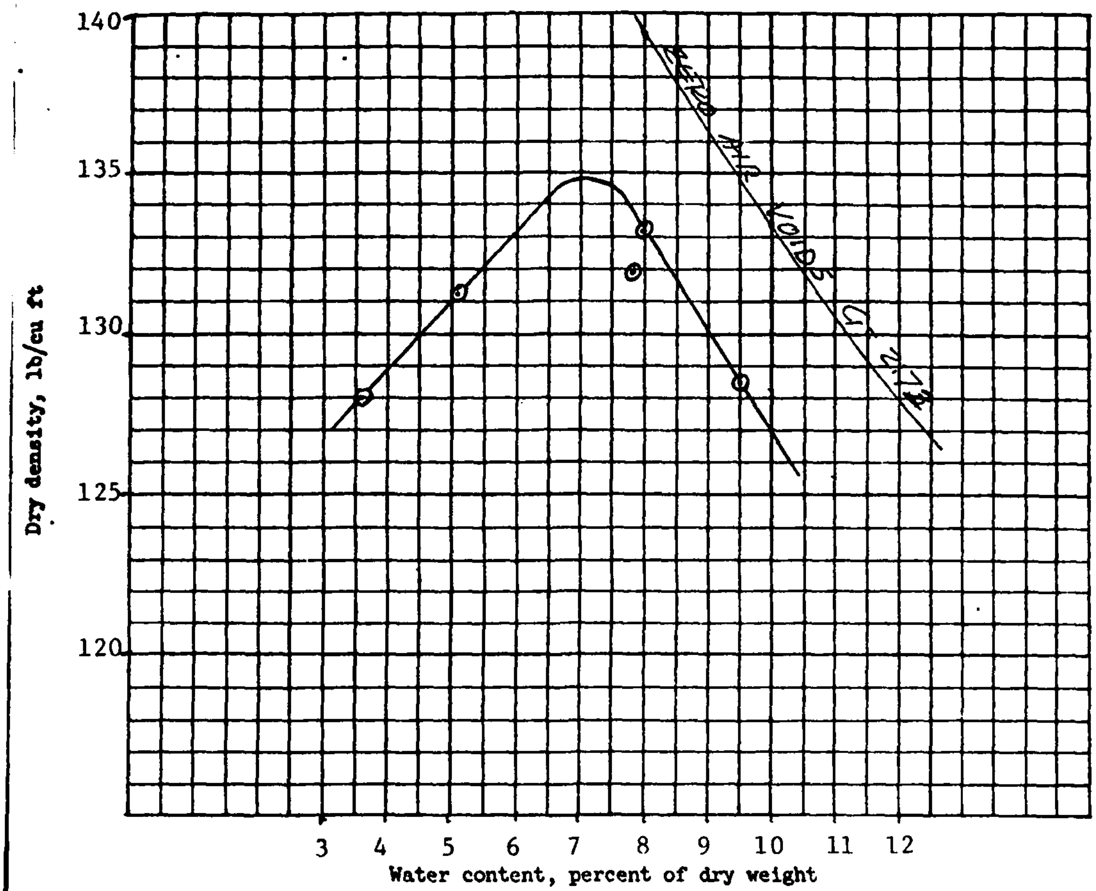
PLATE 39

FIELD UNIT WEIGHT AND RELATIVE DENSITY SUMMARY

FOLSOM DAM
RIGHT WING DAM

Division No.	Hole No.	F.S. No.	Depth, Ft.	Field Unit Weight lb/cu.ft.	Field Unit Weight % W.C.	Relative Density, pcf. Min.	Relative Density, pcf. Max.
87221	R-1	7	16.5-18.3	129.4	6.8		
87227	R-1	13	31.2-33.7	122.3	11.8		
87233	R-1	19	49.1-51.6	126.0	10.0		
Composite:							
87232	R-1	18	46.2-49.1			73.4	
87236	R-1	20	56.8-59.1				
87264	R-2	3	67.2-69.6	127.2	10.3		
87272	R-2	11	87.2-89.5	127.3	9.5		
87217	R-2	16	99.8-102.4	124.1	10.4		

PLATE 40



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
9&26	21-70	Silty Sand (SM)	2.72	19	17	4	1

Sample No.				
Natural water content, percent				
Optimum water content, percent	7.1			
Max dry density, lb/cu ft	134.9			

Remarks Composite:	Project Folsom Dam - Right Wing Dam
87223 & 87240	
	Area
	Boring No. R=1 — Date June 1984
	COMPACTION TEST REPORT

PLATE 41

CYCLIC TRIAXIAL TEST REPORT

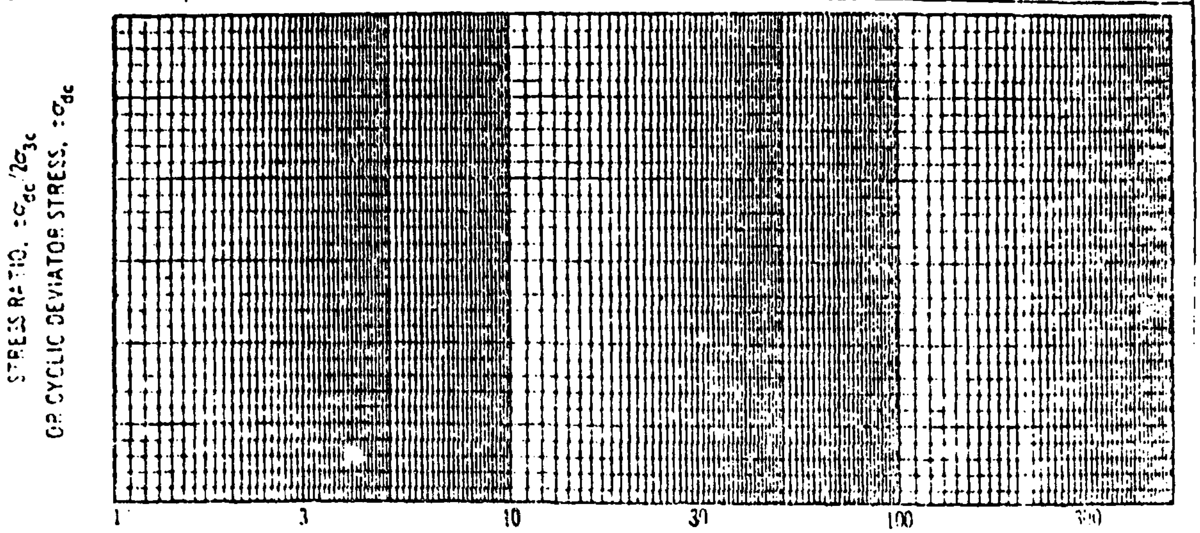
Date: May 1986

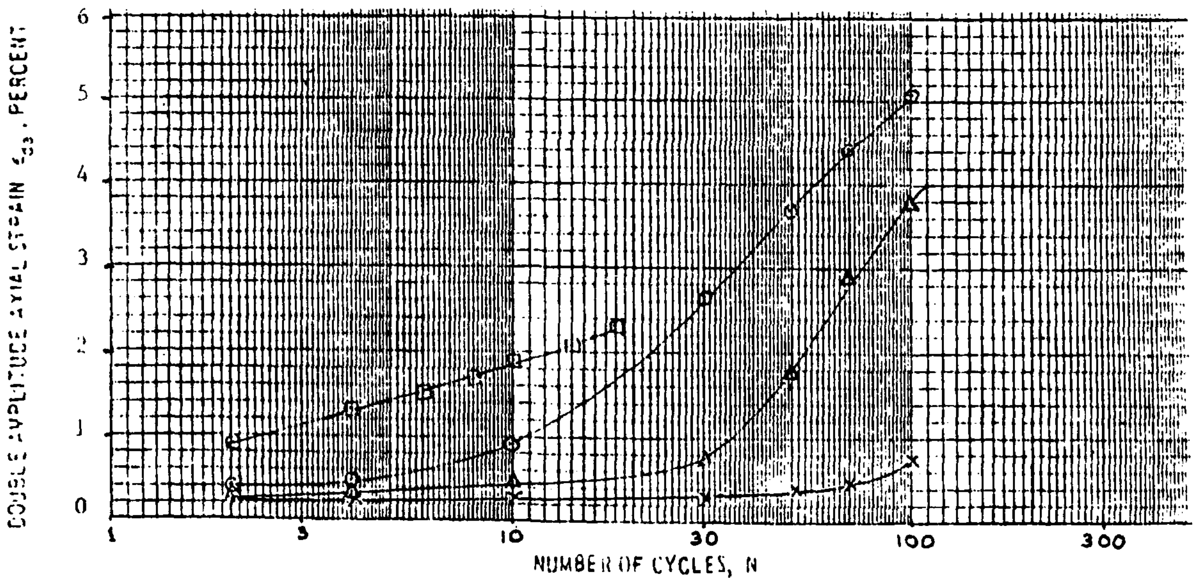
Project: Folsom Dam - Right Wing Dam

Boring No.: R-1 & R-2

Sample No.: 2,5,15,19

Method of Specimen Preparation: Undisturbed





SPECIMEN NO.	DENSITY	RELATIVE DENSITY	B	$\bar{\sigma}_{3c}$	CONSOLIDATION RATIO K_c	$\pm\sigma_{dc}$	$\pm\sigma_{dc}/2\bar{\sigma}_{3c}$	NUMBER OF CYCLES, N — PERCENT PORE PRESSURE INCREASE $N_{25\%}$	$N_{50\%}$	$N_{75\%}$	N_{i}	PERCENT STRAIN, ϵ_{da} N_1	N_2	N_4
87216	113.8	⊡ -	.97	1.0	1.0	0.75	0.37	1/2	1	14	-	2	12	
87219	118.7	× -	.98	1.0	1.0	0.44	0.22	1	67	-	-	-	-	-
87229	121.1	△ -	.97	2.0	1.0	0.97	0.24	1/2-	5	18	-	35	54	110
87280	125.7	⊙ -	.97	4.0	1.0	1.80	0.22	2	8	17	-	11	22	57

PLATE 4.3

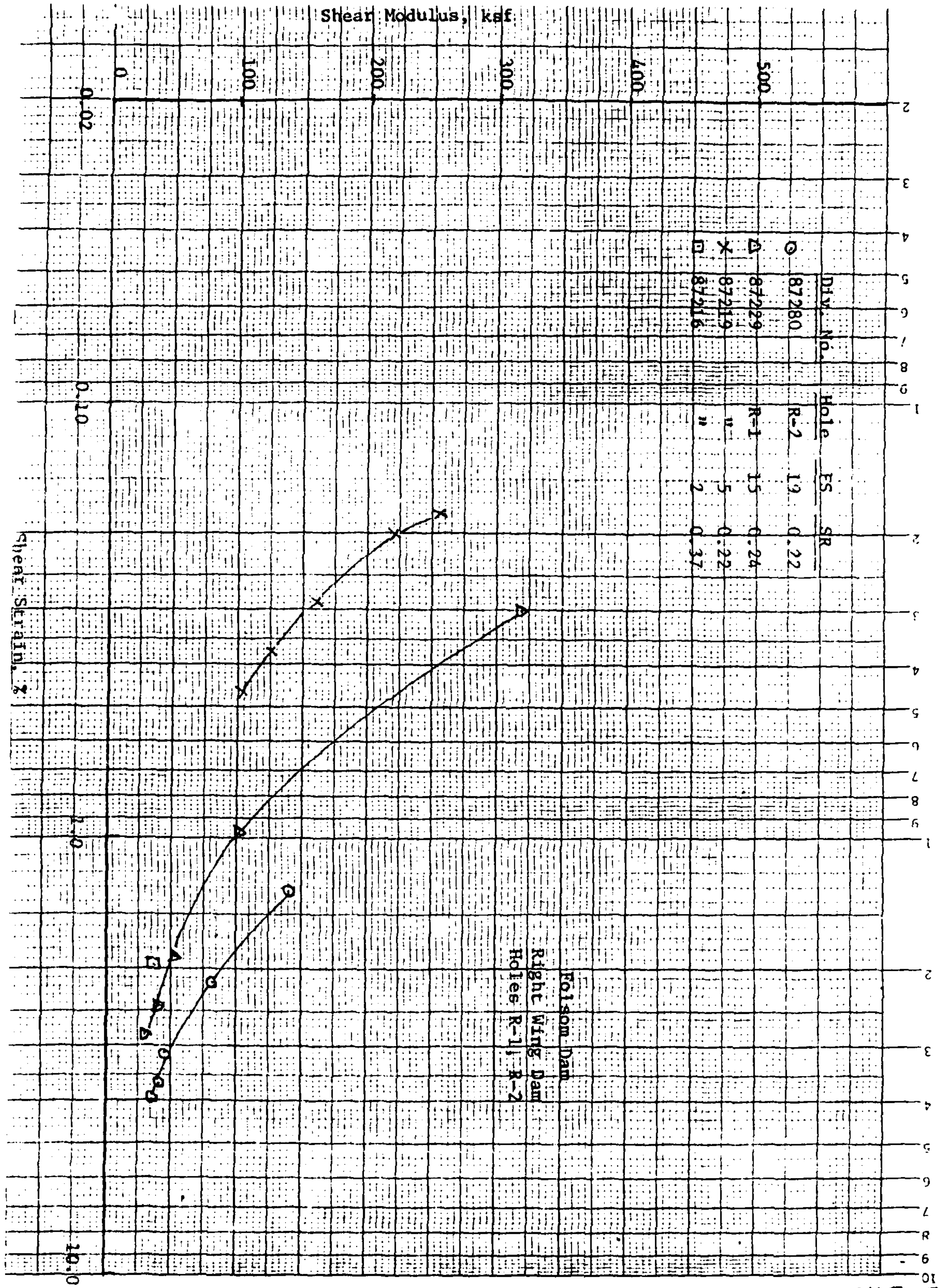

Shear Modulus, ksf
0
100
200
300
400
500
Shear Strain, %
0.02
0.10
1.0
10.0
Div. No. Hole ES SR
87280 R-2 19 0.22
87229 R-1 15 0.24
87219 " 5 0.22
87216 " 2 0.37
Folsom Dam
Right Wing Dam
Holes R-1, R-2

PLATE 45

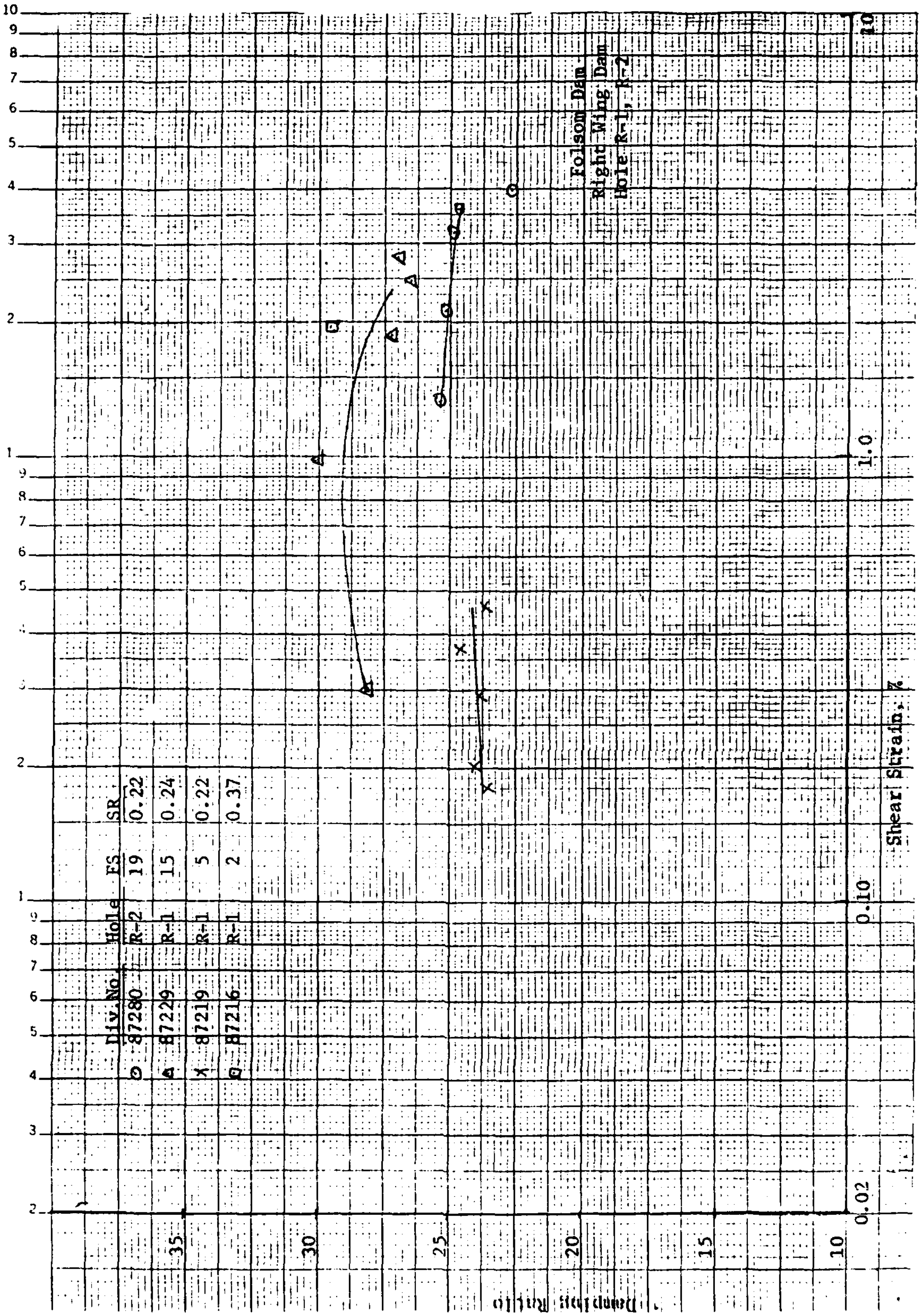
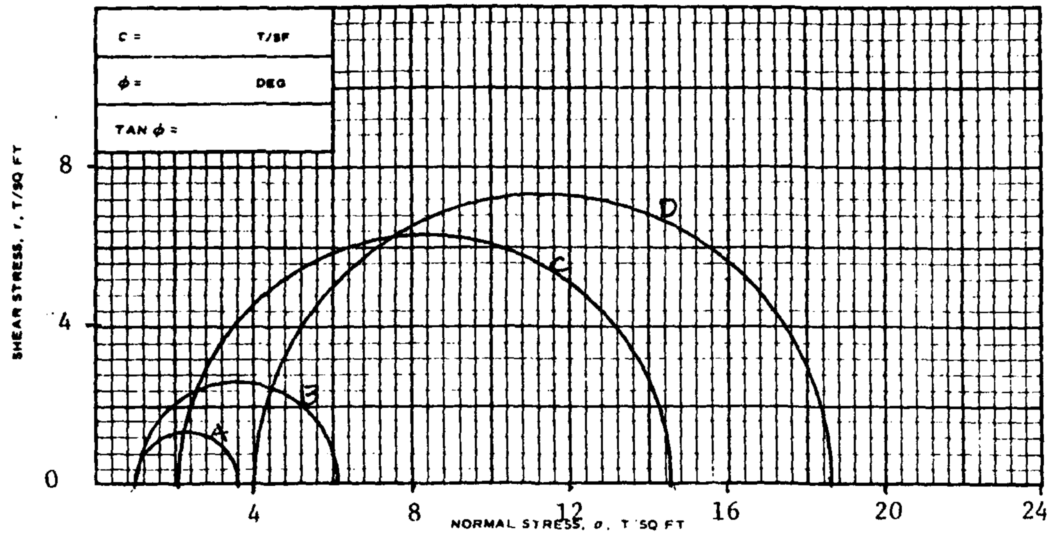
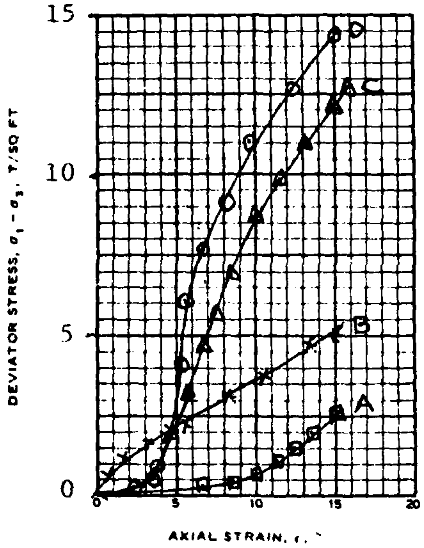

Folsom Dam
Right Wing Dam
Hole R-1, R-2
Div. No. Hole FS SR
87280 R-2 19 0.22
87229 R-1 15 0.24
87219 R-1 5 0.22
87216 R-1 2 0.37
Shear Strain, %
Damping Ratio
0.02
0.10
1.0
10
35
30
25
20
15
10

PLATE 47





SPECIMEN NO.			A	B	C	D
INITIAL	WATER CONTENT, %	w_o	12.6	12.2	11.2	4.5
	DRY DENSITY LB/CU FT	γd_o	113.8	118.7	121.1	125.7
	SATURATION, %	S_o	71	78	77	35
	VOID RATIO	e_o	0.480	0.420	0.391	0.355
BEFORE SHEAR	WATER CONTENT, %	w_c	17.0	14.8	13.8	12.1
	DRY DENSITY LB/CU FT	γd_c	115.5	120.4	1227	128.0
	SATURATION, %	S_c	100	100	100	100
	VOID RATIO	e_c	0.458	0.399	0.373	0.331
	FINAL BACK PRESSURE, T/SQ FT	u_o	6.48	4.32	7.92	5.76
MINOR PRINCIPAL STRESS, T/SQ FT		σ_3	1.00	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{MAX}$	2.60	5.15	12.67	14.51
TIME TO $(\sigma_1 - \sigma_3)_{MAX}$, MIN		t_f	14	25	24	23
ULTIMATE DEVIATOR STRESS, T/SQ FT		$(\sigma_1 - \sigma_3)_{ULT}$	–	–	–	–
INITIAL DIAMETER, IN.		D_o	2.82	2.82	2.80	2.88
INITIAL HEIGHT, IN.		H_o	6.24	6.42	6.21	6.52

CONTROLLED- Stress TEST

DESCRIPTION OF SPECIMENS A) Silty Sand (SM); B) Clayey Sand (SC); C) Clayey Sand (SC-SM); D) Silty Sand (SM)

LL See Summary	PI	Gs	TYPE OF SPECIMEN undisturbed — TYPE OF TEST $\bar{R}$

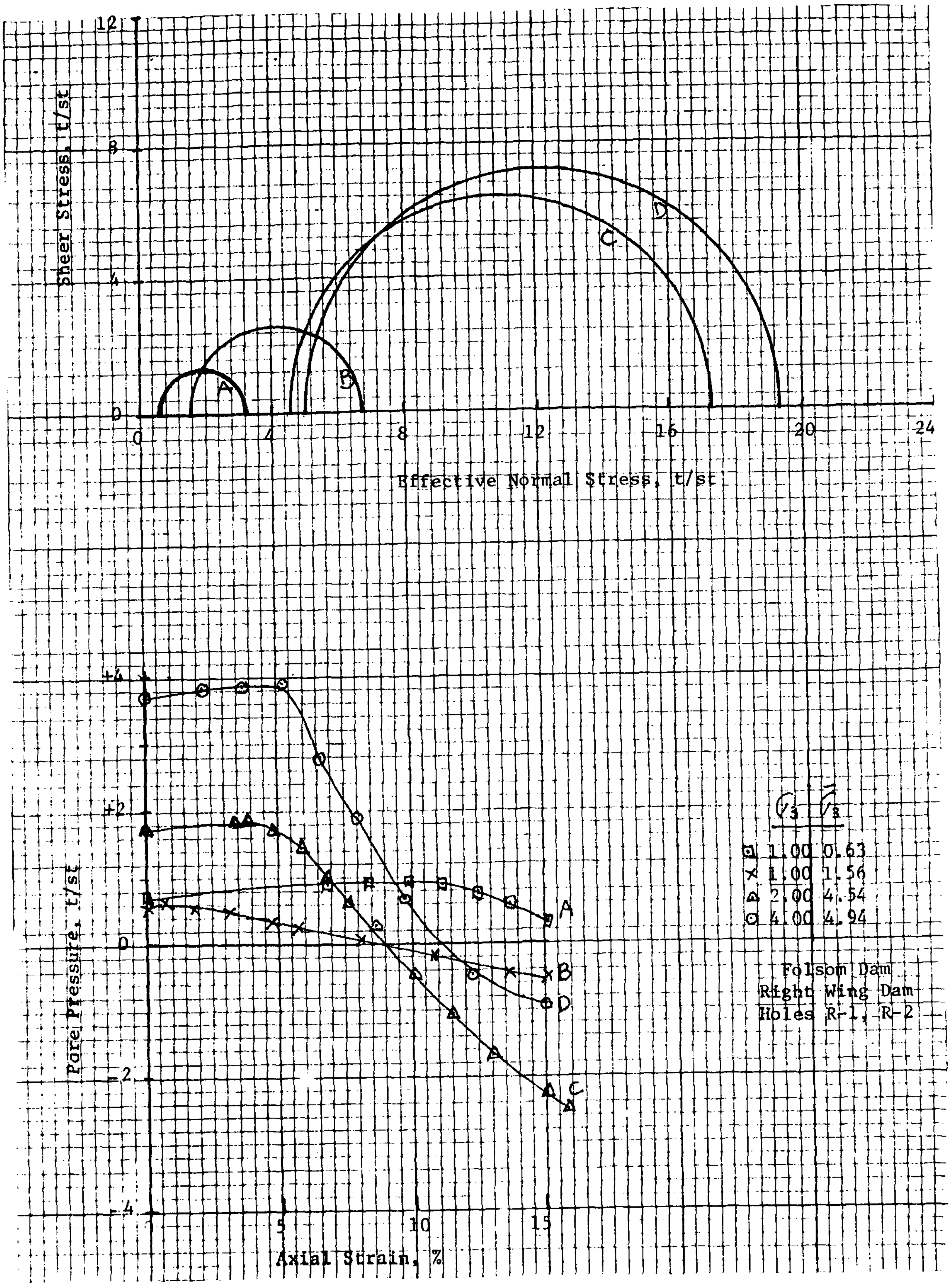
REMARKS: Post-cyclic.

Div. No.	Boring	FS	Depth
A) ⊡ 87216	R-1	2	3.8-6.3
B) x 87219	"	5	11.3-13.8
C) ∆ 87229	"	15	36.2-38.7
D) ⊙ 87280	R-2	19	107.2-109.7

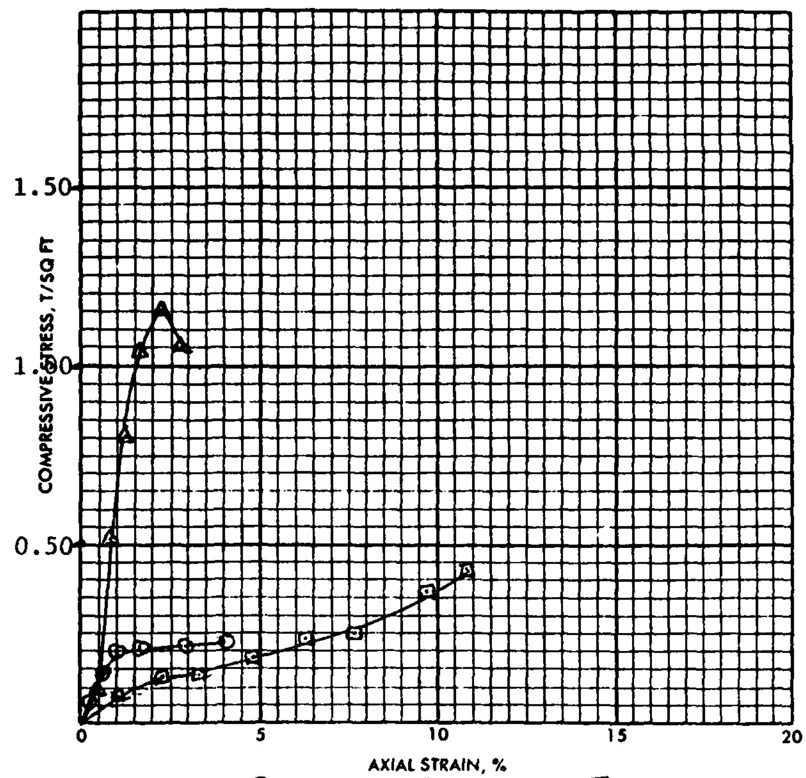
PROJECT Folsom Dam-Right Wing Dam Samples	
BORING NO. R-1 & R-2	SAMPLE NO.
DEPTH/ELEV 3.8 - 109.7	
LABORATORY SPDL	DATE March 1986

TRIAXIAL COMPRESSION TEST REPORT

PLATE 50


Sheer Stress, t/st
Effective Normal Stress, t/st
A
B
C
D
Pore Pressure, t/st
Axial Strain, %
σ_3 $\bar{\sigma}_3$
1.00 0.63
1.00 1.56
2.00 4.54
4.00 4.94
Folsom Dam
Right Wing Dam
Holes R-1, R-2

FAILURE SKETCHES



CONTROLLED STRESS

☒ CONTROLLED STRAIN

			⊙	△	⊡	
TEST NO.			R-1, FS 6	R-1, FS 20	R-2, FS 18	
TYPE OF SPECIMEN			Undist.	Undist.	Undist.	
INITIAL	WATER CONTENT	w_o	9.4 %	10.7 %	9.6 %	%
	VOID RATIO	e_o	0.547	0.371	0.516	
	SATURATION	S_o	47 %	78 %	51 %	%
	DRY DENSITY, LB/CU FT	γ_d	110.1	123.3	112.4	
TIME TO FAILURE, MIN		t_f	3.7	4.0	14	
UNCONFINED COMPRESSIVE STRENGTH, T/SQ FT		q_u	0.22	1.15	0.44	
UNDRAINED SHEAR STRENGTH, T/SQ FT		s_u	0.11	0.57	0.22	
SENSITIVITY RATIO		S_t	-	-	-	
INITIAL SPECIMEN DIAMETER, IN		D_o	2.80	2.83	2.81	
INITIAL SPECIMEN HEIGHT, IN.		H_o	6.40	6.17	6.13	

CLASSIFICATION ⊙ Silty Sand(SP-SM) △ Silty Sand(SM) ⊡ Silty Sand(SM)

LL 23, 24, - | PL 21, 22, - | PI 2, 2, NP | G_s 2.73,2.71,2.73

REMARKS

Div. No.	Hole	Depth
⊙ 87220	R-1	13.8-16.5
△ 87234	R-1	54.6-57.9
⊡ 87279	R-2	89.6-92.3

PROJECT FOLSOM DAM	
AREA Right Wing Dam	
BORING NO. R-1 & 2	SAMPLE NO.
DEPTH EL	DATE June 1984

UNCONFINED COMPRESSION TEST REPORT

PLATE XI-2

PLATE 56

U.S. ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT FOLSOM DAM - LEFT WING DAM **DATE**

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation		Laboratory Descriptive Classification	Mechanical Analysis-% Finer											Liquid Limit	Plasticity Index	Field Moist %
						Gravel					Sand					Fines			
			From	To		1 1/2	1	3/4	1/2	3/8	#4	#10	#40	#60	#100	#200			
87200	L-1	5	11.8	14.1	Clayey Sand (SC-SM)					100	99	83	43	34	26	18	26	6	
87201	L-1	6	14.1	16.5	Clayey Sand (SC)	SP.GR=2.75		100	97	97	97	92	65	54	44	33	30	11	
87203	L-1	8	19.0	21.5	Clayey Sand (SC-SM)	100	99	99	99	99	98	90	53	41	32	23	25	7	
87205	L-1	10	24.0	26.9	Clayey Sand (SC)	SP.	Gr. =	2.74			100	94	58	47	36	26	27	8	
87207	L-1	12	29.1	36.6	Clayey Sand (SC)			100	99	99	98	85	46	37	28	19	25	9	3.3
87209	L-1	14	34.0	36.6	Silty Sand (SM)	SP.	Gr =	2.8	100	99	97	79	37	29	22	15		NP	
87210	L-1	12& 15			Clayey Sand			100	99	99	99	85	46	37	28	19	25	9	
87203 87213	L-1	8& 18	19.0 44.1	21.5 46.6	Clayey Sand (SC-SM)	99	99	99	99	99	98	90	53	41	32	23	25	7	
87245	L-2	5	54.6	57.9	Silty Sand (SM)				100	99	94	74	39	31	24	16	24	2	
87257	L-2	17	87.3	89.6	Clayey Sand (SC)	SP.	Gr =	2.76			100	86	47	37	29	19	28	9	
87258	L-2	18	89.6	92.3	Clayey Sand (SC)	SP.	Gr.=	2.76	100	99	98	91	54	42	32	22	28	9	

PLATE 62

U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT FOLSOM DAM - LEFT WING DAM **DATE**

Division Serial No.	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel 1 1/2	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist %
87537	L-3	2	4.0	7.5	Clay (ML-CL)	SP.	Gr=2.	79		100	97	96	94	93	92	92	25	5	
87538	L-3	2	8.0	9.1	Clay (ML-CL)	SP.	Gr=2.	77		100	99	98	93	91	89	88	24	5	
87540	L-3	5	12.0	15.3	Clay (ML-CL)	SP.	Gr=2. 100	78 98	98	97	95	94	92	91	91	90	26	6	
87541	L-3	6	16.0	17.3	Clay (CL)	SP.	Gr=2.	78		100	99	99	96	96	95	94	27	8	
87548	L-3	12	24.0	27.5	Sandy Clay(CL)	SP.	Gr=2.	78		100	99	98	88	87	86	85	27	9	
87549	L-3	13	28.0	29.2	Clay (CL)	SP.	Gr=2.	78		100	99	98	93	92	90	89	29	9	
87553	L-3	17	38.5	39.5	Clay (CL)	SP.	Gr=2.	78	100	99	98	98	97	93	92	91	25	8	
87554	L-3	18	40.0	43.5	Clay (CL-ML)	SP.	Gr=2	78 100	99	99	98	97	96	95	95	94	23	6	
87557	L-3	21	48.9	51.5	Clay (CL_ML)	SP.	Gr=2.	78		100	98	97	94	93	92	91	24	4	
87558	L-3	22	52.1	52.6	Silt (ML)	SP.	Gr=2.	78		100	97	96	93	92	91	90	25	3	
87516	L-4	1	49.8	54.0		SP.	Gr=2	75	100	99	95	88	45	33	24	15			
87517	L-4	2	54.0	58.0	Silty Sand (SM)	SP.	Gr=2	75 100	99	99	95	87	45	34	25	17	25	3	
87520	L-4	5	64.7	67.5	Clayey Sand (SC)	SP.	Gr=2.	75 100	99	98	94	82	44	33	25	16	24	3	

PLATE 63

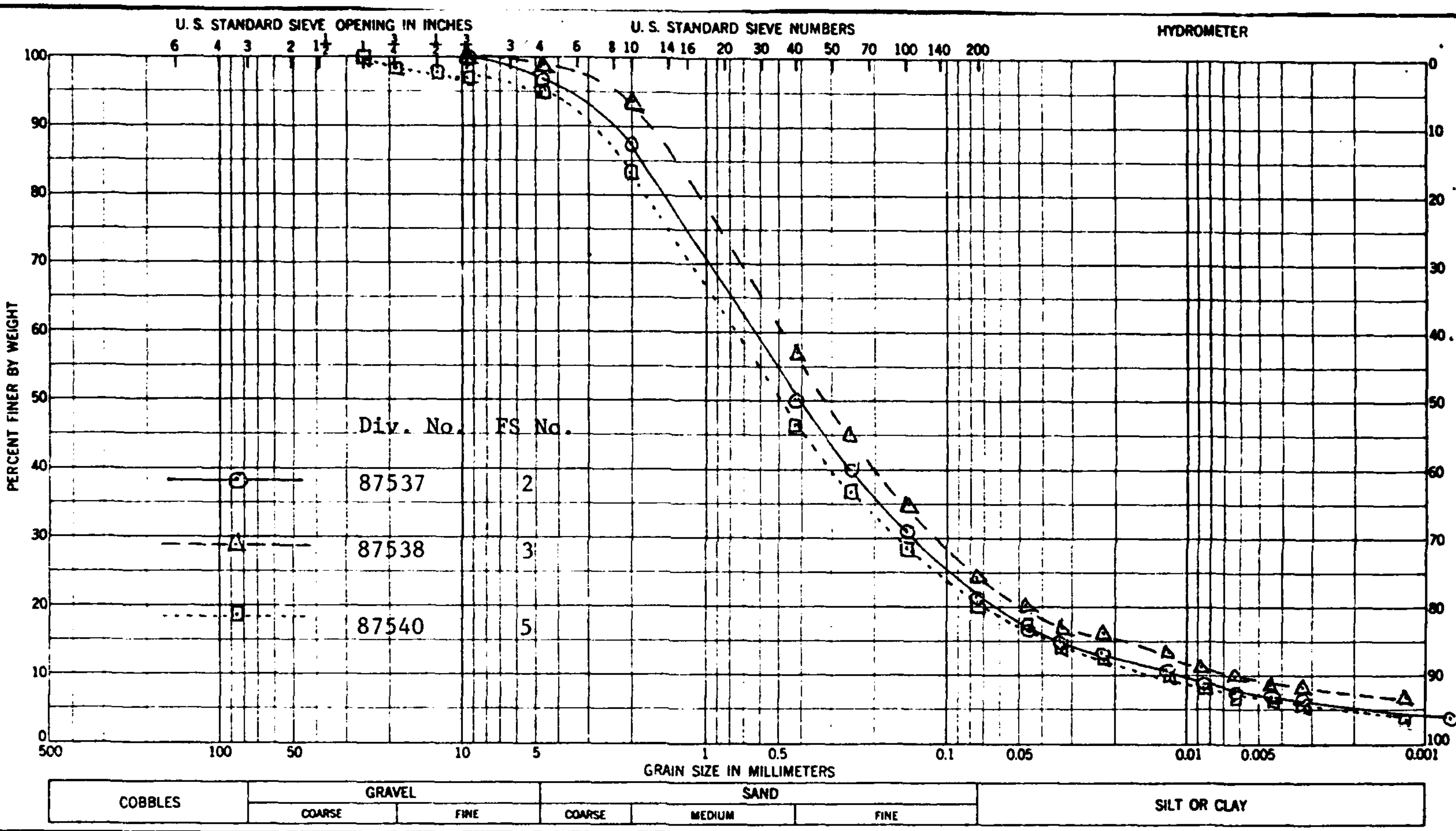
U.S. ARMY ENGINEER DIVISION LABORATORY -- SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT FOLSOM DAM- LEFT WING DAM **DATE**

Division Serial No:	Hole No.	Field Sample No.	Depth Or Elevation From	Depth Or Elevation To	Laboratory Descriptive Classification	Mechanical Analysis-% Finer: Gravel 1 1/2	Gravel 1	Gravel 3/4	Gravel 1/2	Gravel 3/8	Sand #4	Sand #10	Sand #40	Sand #60	Sand #100	Fines #200	Liquid Limit	Plasticity Index	Field Moist %
87521	L-4	6	67.0	68.0	Clayey Sand (SC)	SP.	Gr=2	75		100	99	94	61	50	40	29	30	11	
87522	L-4	7	68.0	72.2	Clayey Sand (SC)	SP.	Gr=2	75		100	98	91	51	40	31	22	28	8	
87523	L-4	8	72.2	73.0	Clayey Sand (SM-SC)	SP.	Gr=2	75		100	97	90	52	40	30	21	26	5	
87524	L-4	9	73.0	73.5	Clayey Sand (SC)	SP.	Gr=2	75		100	99	94	67	55	44	33	27	9	
87531	L-4	16	84.0	89.0	Silty Sand (SM)	SP.	Gr=2	78		100	97	86	44	35	27	19	25	3	
87534	L-4	19	92.5	94.0	Clayey Silty Sand (SM)	SP.	GR=2	78 100	98	98	91	78	44	34	26	17	26	4	
87535	L-4	20	94.0	99.5	Clayey Sand (SM-SC)	SP.	Gr=2	78		100	98	88	51	40	30	20	25	4	
87442	L-4	21	99.0	104.0	Clayey Silty Sand (SM)	SP.	Gr=2	78		100	97	87	50	39	31	22	28	5	
87446	L-4	25	108.5	110.0	Silty Clayey Sand (SM-SC)	SP.	Gr=2	74			100	95	57	44	33	22	25	6	
87448	L-4	27	110.5	111.5		SP.	Gr=2	74		100	99	96	64	48	36	23			

PLATE 64



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87537	4.0-7.5	Clayey Sand (SC-SM)		25	20	5
87538	8.0-7.1	Clayey Sand (SC-SM)		24	20	4
87540	12.0-15.3	Clayey Sand (SC-SM)		26	20	6

Project FOLSOM DAM

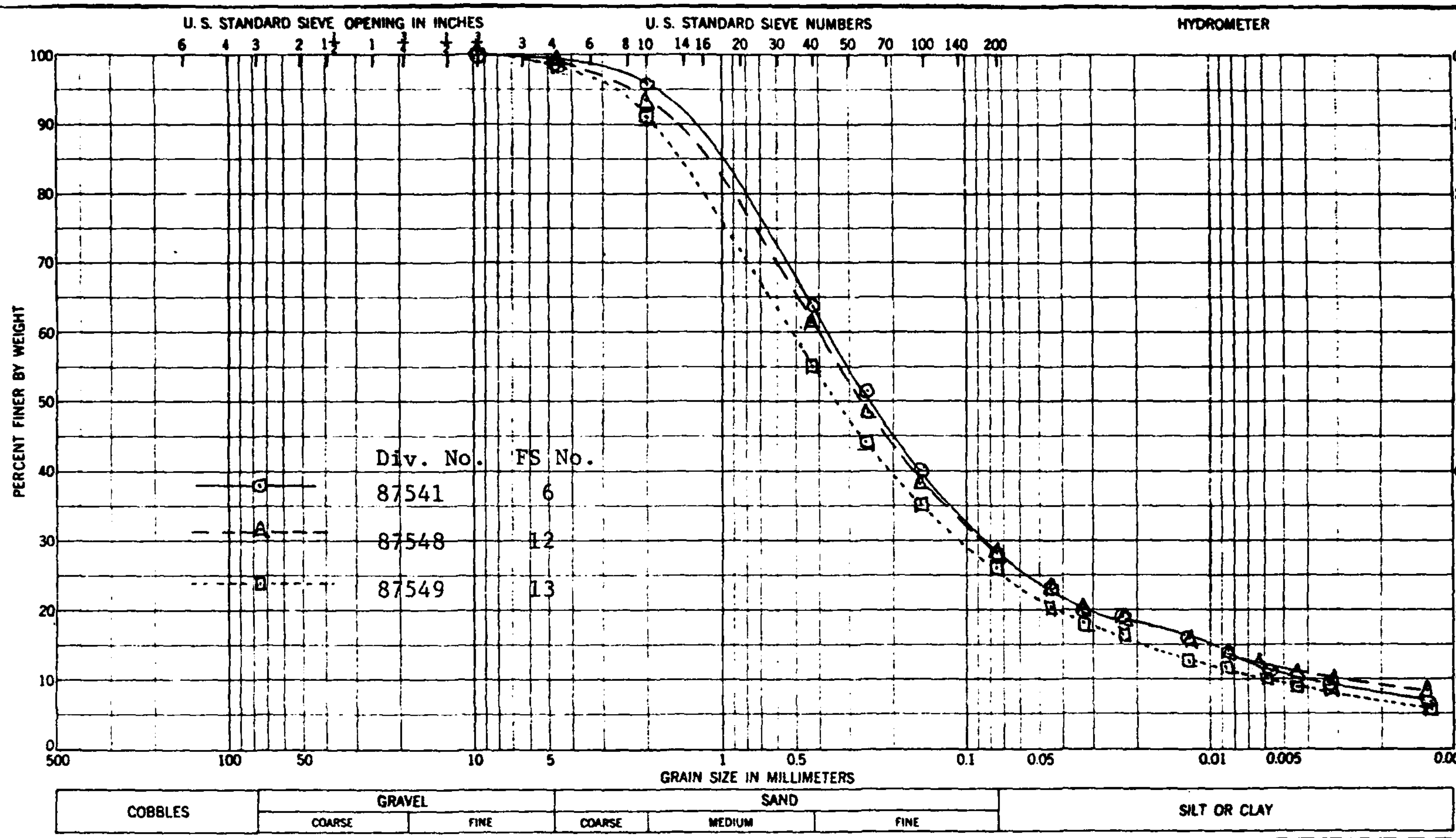
Area Left Wing Dam

Boring No. L-3

Date May 1984

GRADATION CURVES

PLATE 65



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87541	16.0-17.3	Clayey Sand (SC)		27	18	9
87548	24.0-27.5	Clayey Sand (SC)		27	18	9
87549	28.0-29.2	Clayey Sand (SC)		29	19	10

Project FOLSOM DAM

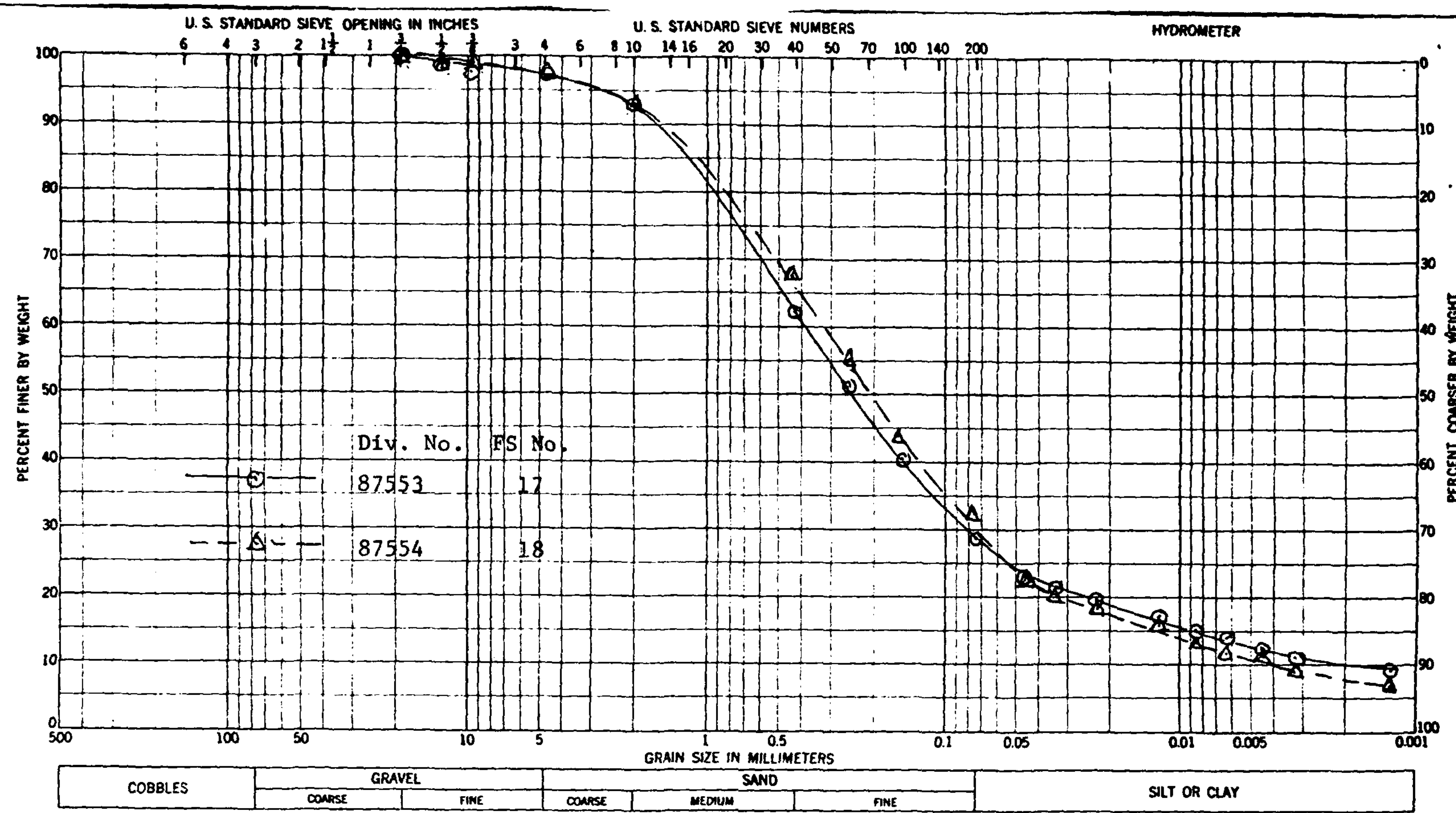
Area Left Wing Dam

Boring No. L-3

Date May 1984

GRADATION CURVES

PLATE 66



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87553	38.5-39.5	Clayey Sand (SC)		25	17	8
87554	40.0-43.5	Clayey Sand (SC-SM)		23	17	6

GRADATION CURVES

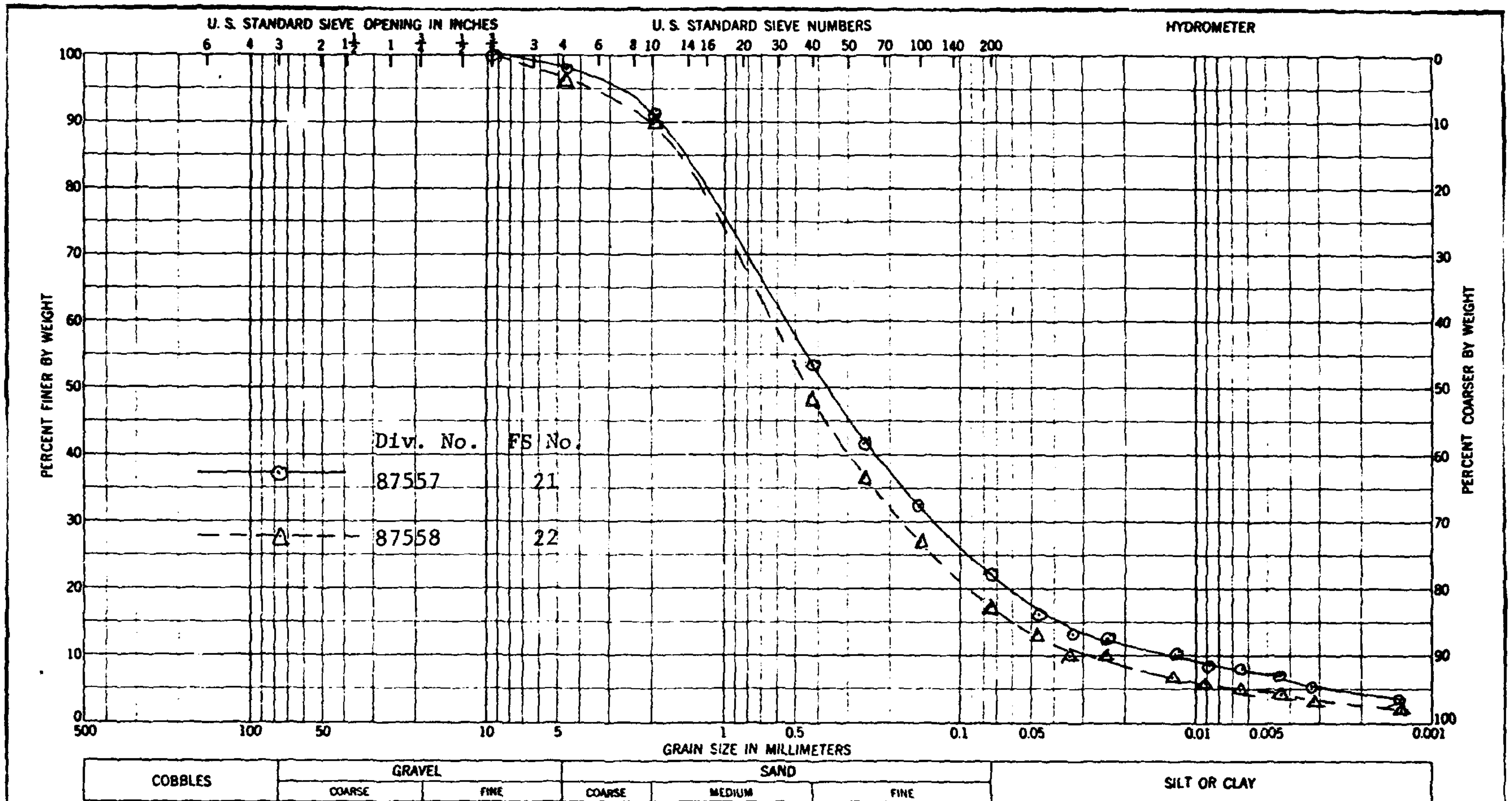
Project FOLSOM DAM

Area Left Wing Dam

Boring No. L-3

Date May 1984

PLATE 67



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87557	48.9-51.5	Clayey Sand (SC-SM)		24	20	4
87558	52.0-56.2	Silty Sand (SM)		25	22	3

Project FOLSOM DAM

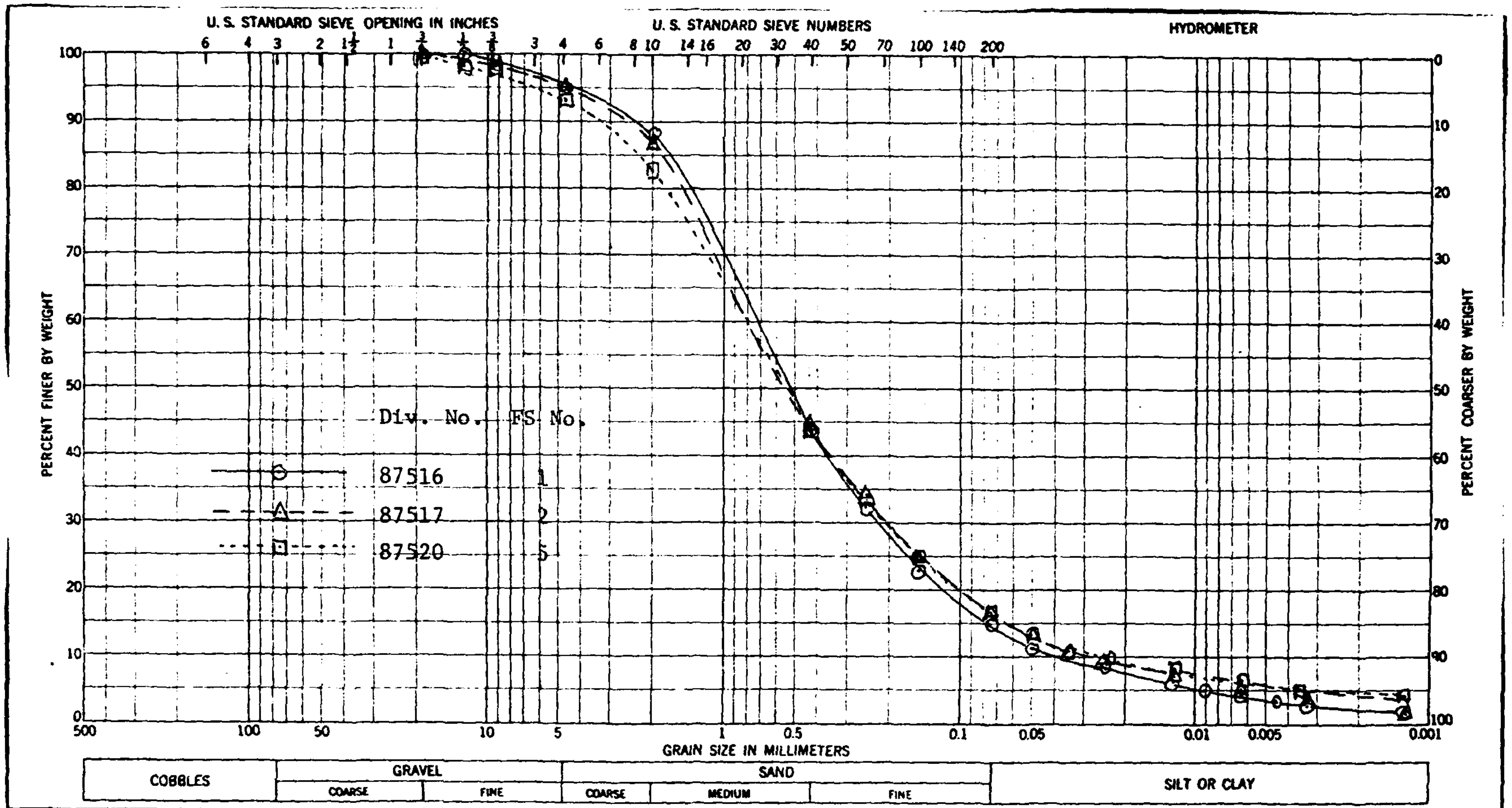
Area Left Wing Dam

Boring No. L-3

Date June 1984

GRADATION CURVES

PLATE 68

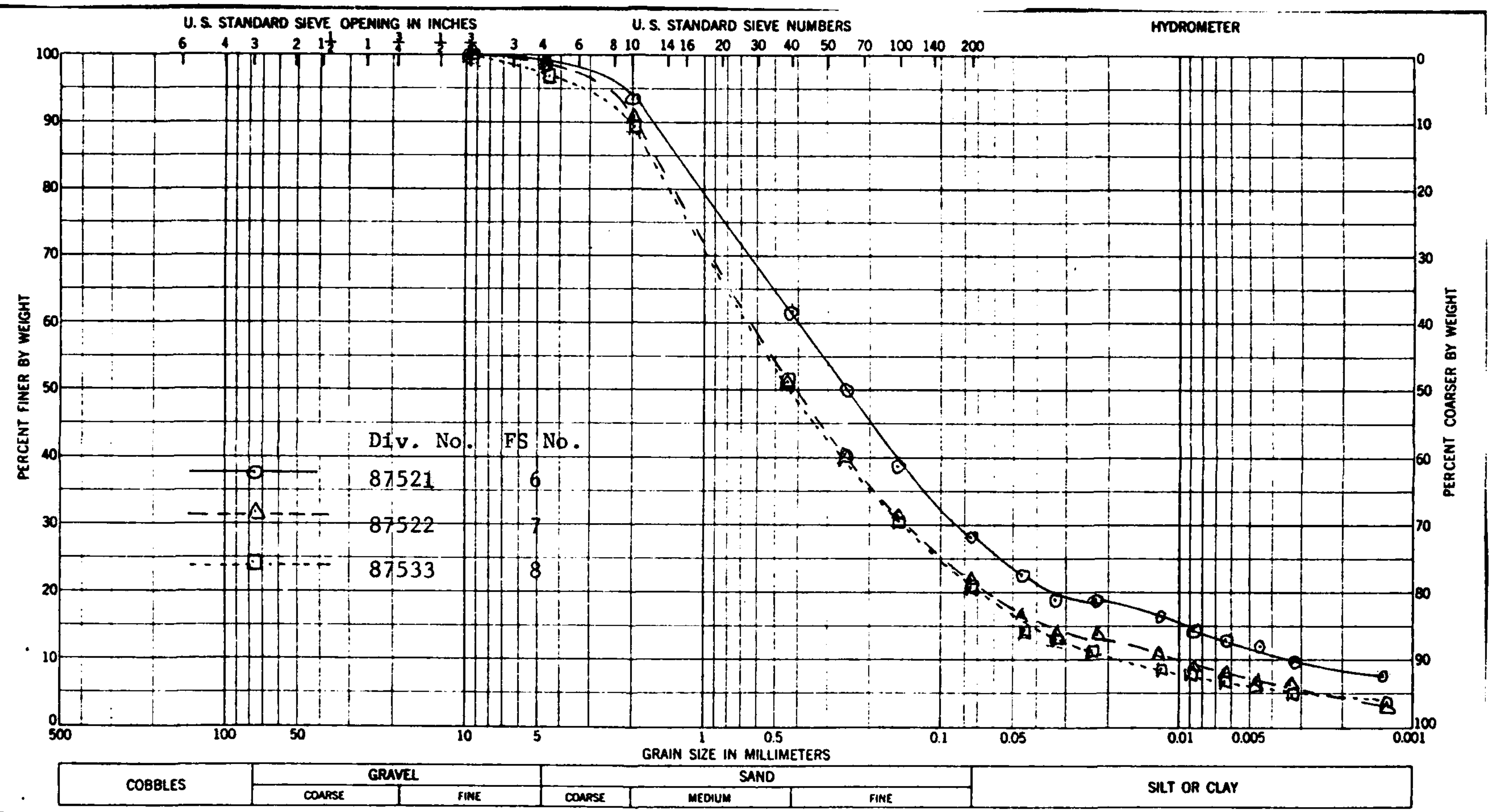


Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87516	49.8-54.0	Silty Sand (SM)			NP	
87517	54.0-58.0	Silty Sand (SM)		25	22	3
87520	64.7-67.5	Silty Sand (SM)		24	21	3

GRADATION CURVES

Project	FOLSOM DAM
Area	Left Wing Dam
Boring No.	L-4
Date	May 1984

PLATE 69



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87521	67.0-68.0	Clayey Sand (SC)		30	19	11
87522	68.0-72.2	Clayey Sand (SC)		28	20	8
87523	72.2-73.0	Clayey Sand (SC-SM)		26	21	5

Project: FOLSOM DAM

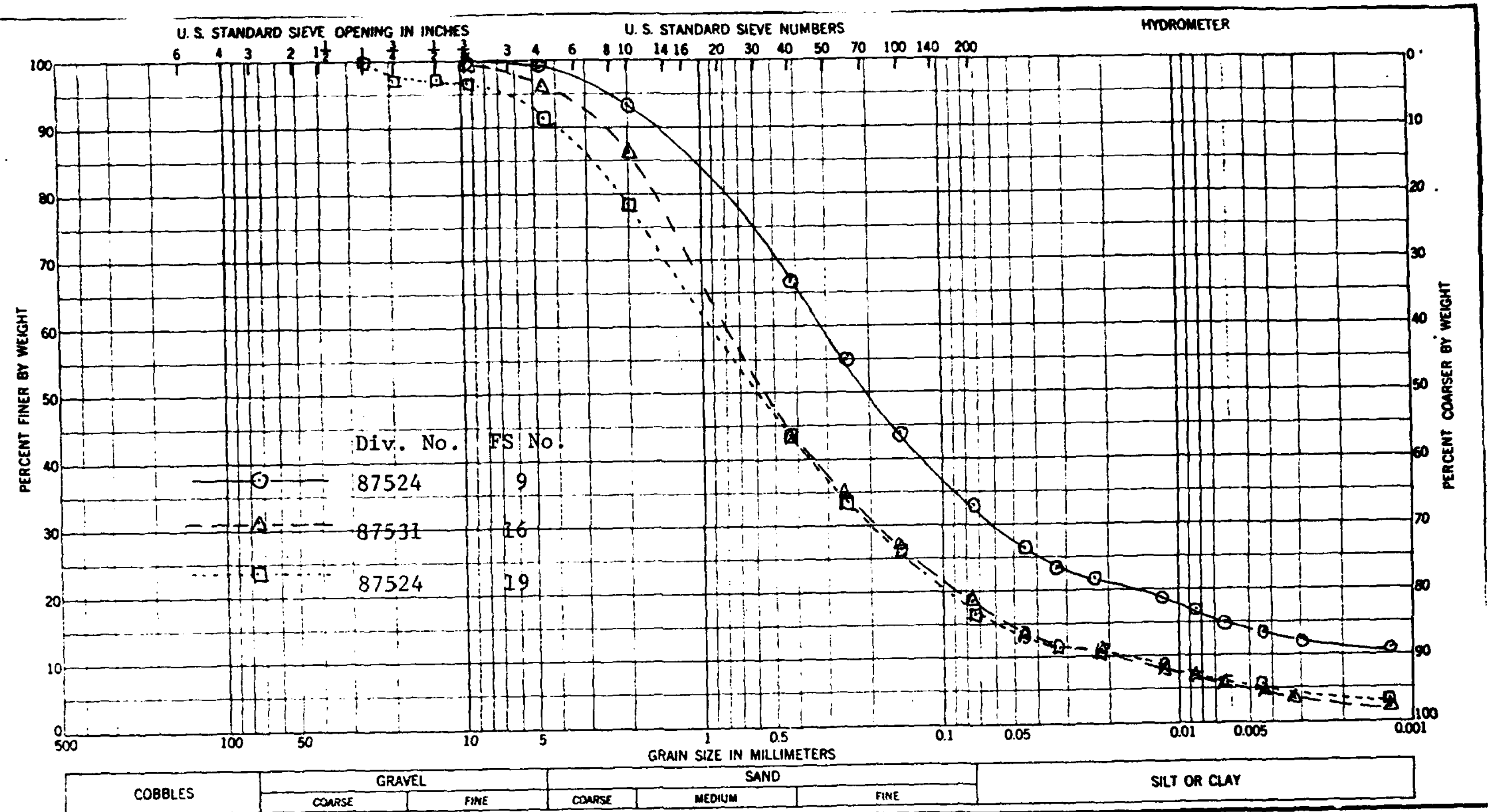
Area: Left Wing Dam

Boring No.: L-4

Date: July 1984

GRADATION CURVES

PLATE 70



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87524	73.0-73.5	Clayey Sand (SC)		26	12	9
87531	84.0-89.0	Silty Sand (SM)		25	22	3
87534	92.5-94.0	Clayey Sand (SC-SM)		26	22	4

Project FOLSOM DAM

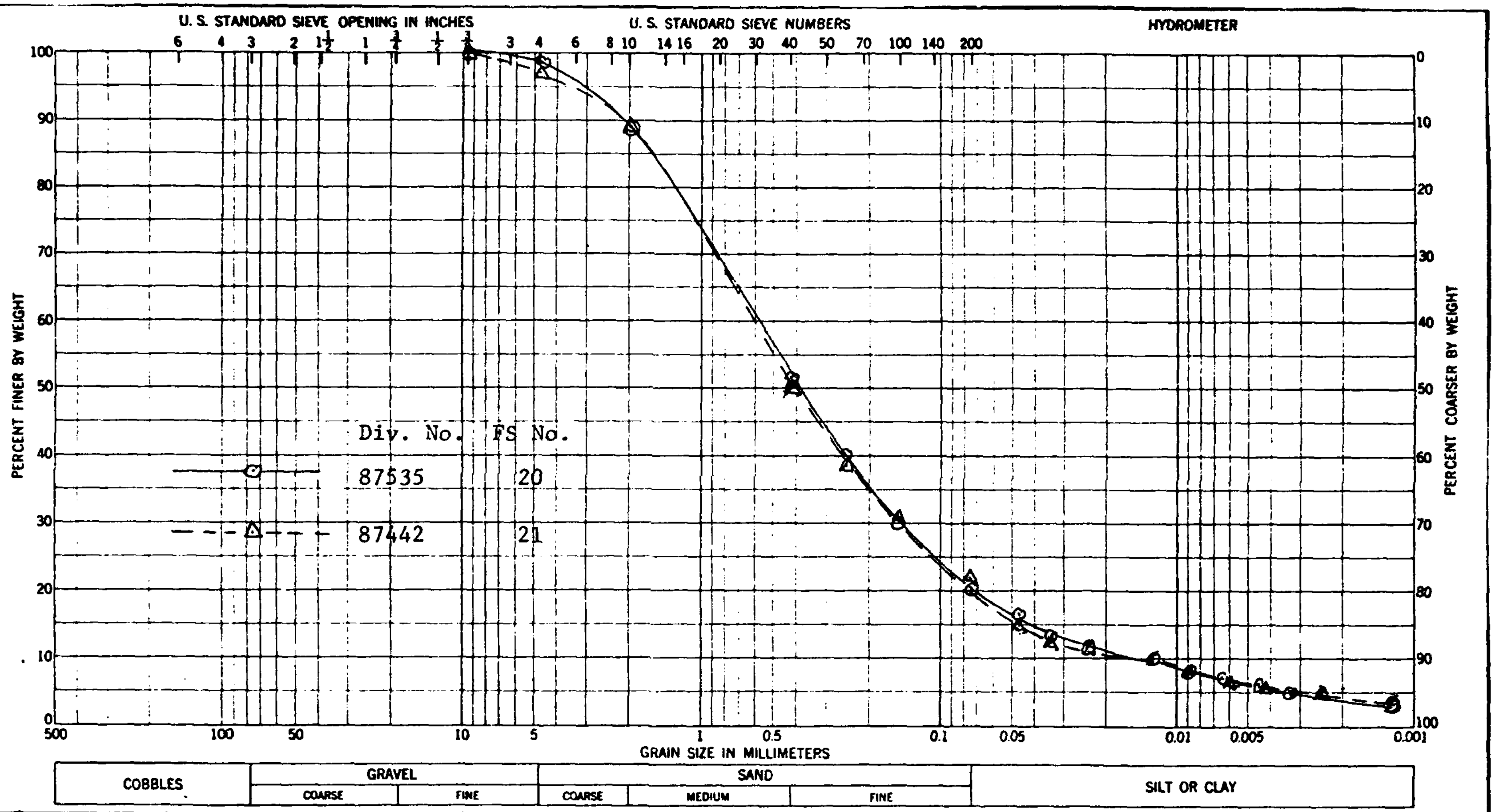
Area Left Wing Dam

Boring No. L-4

Date July 1984

GRADATION CURVES

PLATE 71



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87535	94-99.5	Clayey Sand (SC-SM)		25	21	4
87442	99-104	Clayey Sand (SC-SM)		28	22	5

Project FOLSOM DAM

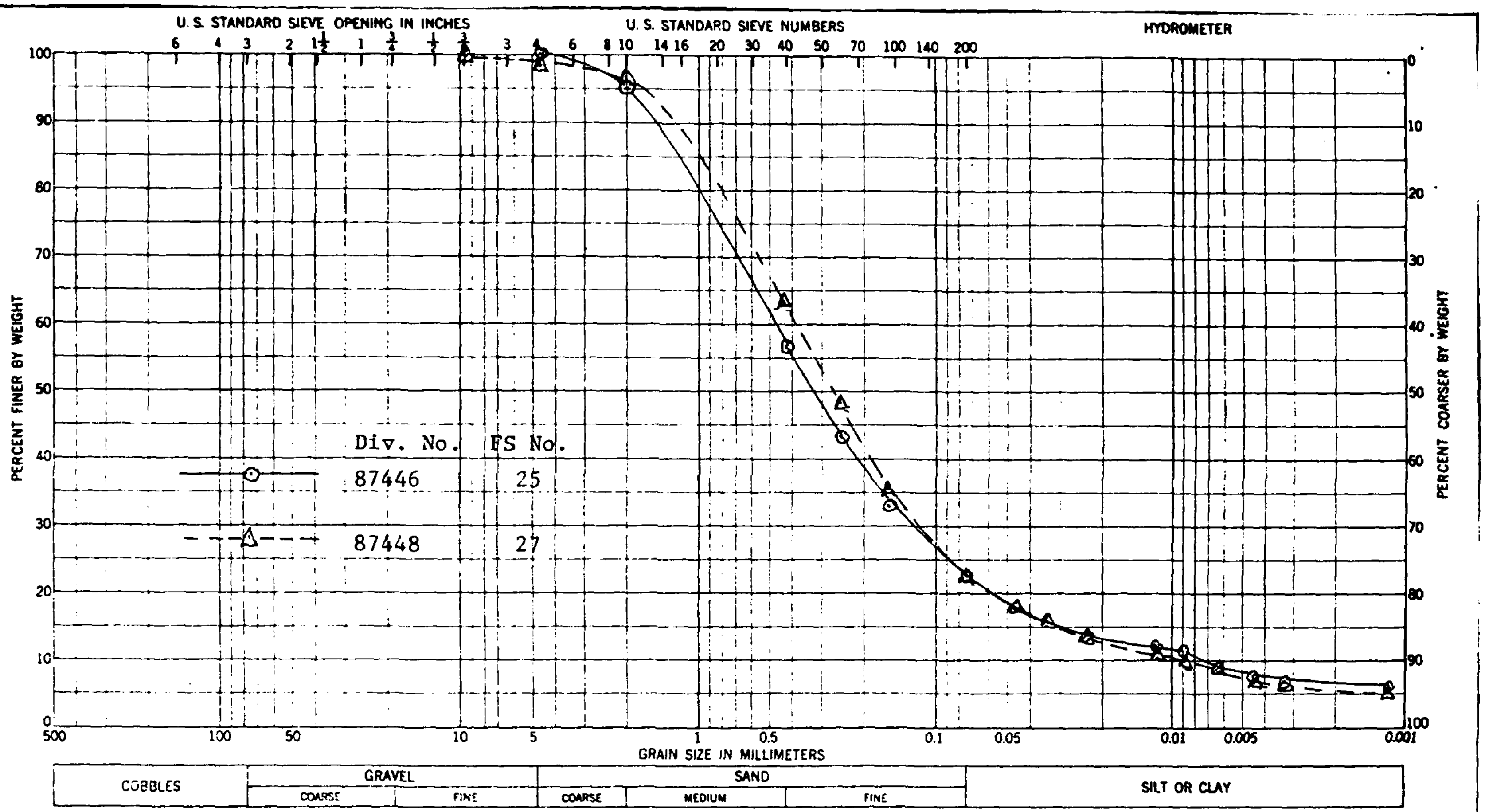
Area Left Wing Dam

Boring No. L-4

Date July 1984

GRADATION CURVES

PLATE 72



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
87446	108.5-110	Clayey Sand (SC-SM)		25	19	6
87448	110.5-111.5	Silty Sand (SM)			NP	

GRADATION CURVES

Project	FOLSOM DAM
Area	Left Wing Dam
Boring No.	L-4
Date	July 1984

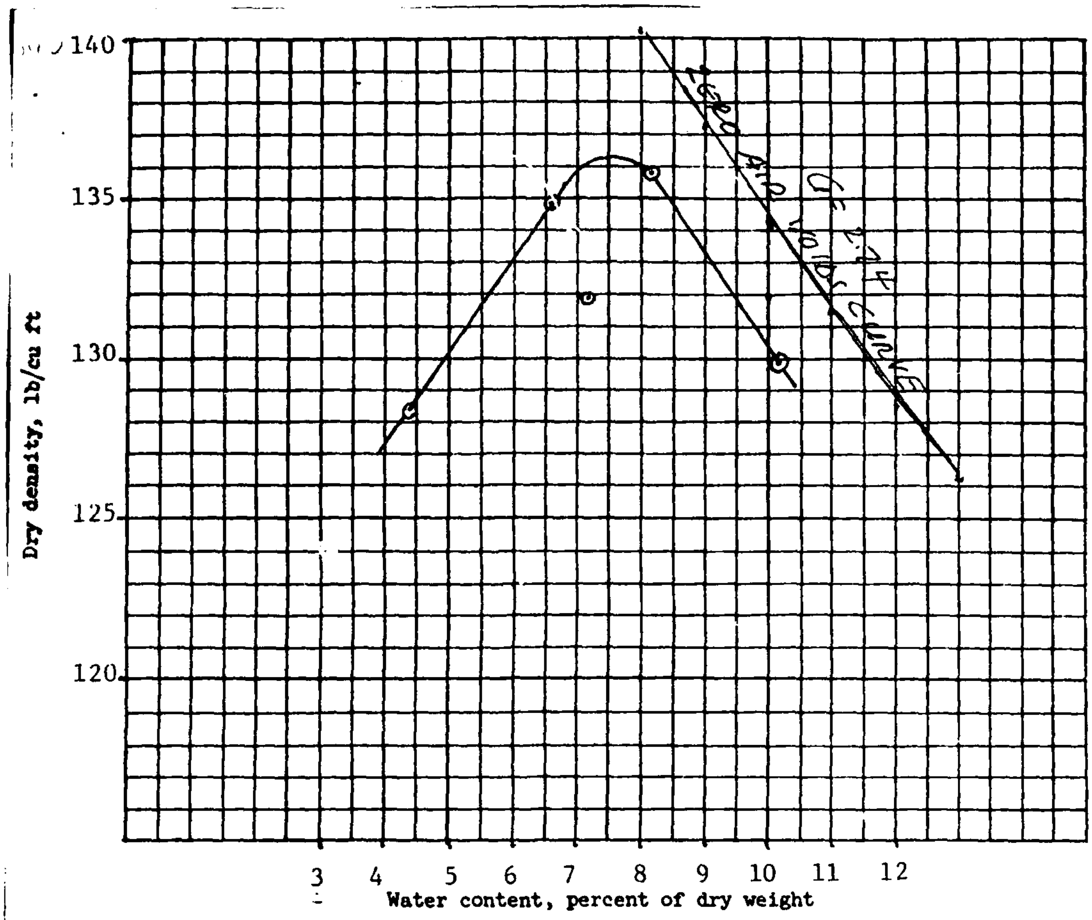
PLATE 73

FIELD UNIT WEIGHT AND RELATIVE DENSITY SUMMARY

FOLSOM DAM
LEFT WING DAM

Division No.	Hole No.	F.S. No.	Depth, Ft.	Field Unit Weight lb/cu.ft.	Field Unit Weight % W.C.	Relative Density, pcf. Min.	Relative Density, pcf. Max.
Composite:							
87207	L-1	12	29.1-31.6			80.4	
87210	L-1	15	36.6-36.0				
87246	L-2	6	57.1-59.6	129.9	9.7		
87251	L-2	11	72.1-74.6	129.4	9.3		
87259	L-2	19	92.3-94.8	128.8	9.5		

PLATE 74



Modified compaction test

25 blows per each of 5 layers, with 10 lb rammer and

18 inch drop. 4 inch diameter mold

Sample No.	Elev or Depth	Classification	G	LL	PL	% > No. 4	% > 3/4 in.
8&18	19-47	Clayey Sand (SC-SM)	2.74	25	18	2	1

Sample No.				
Natural water content, percent				
Optimum water content, percent	7.5			
Max dry density, lb/cu ft	136.3			

Remarks	Project Folsom Dam - Left Wing Dam	
Composite: 87203 and 87213		
	Area	
	Boring No. L-1	Date June 1984
	COMPACTION TEST REPORT	

PLATE 75

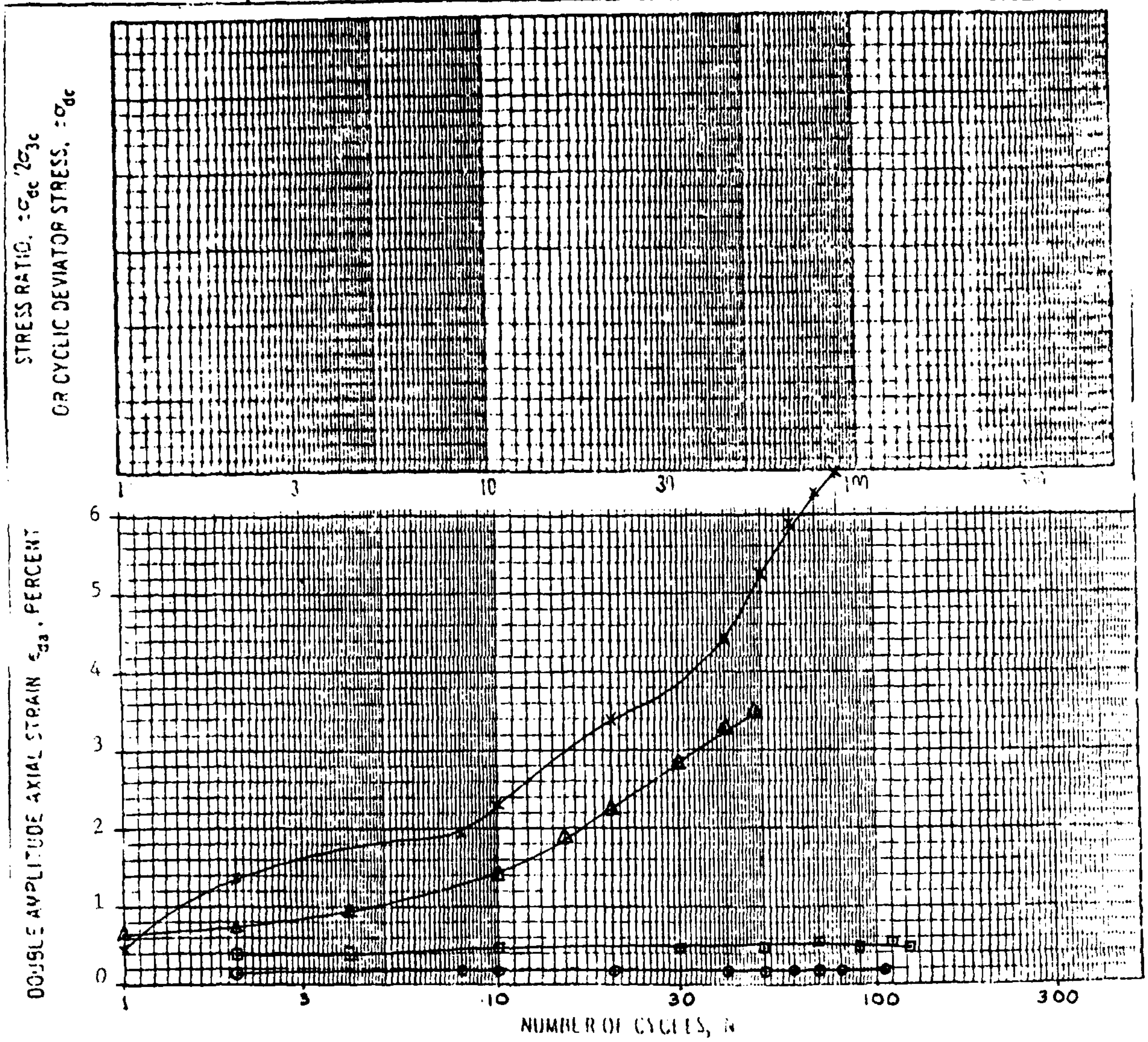
CYCLIC TRIAXIAL TEST REPORT

Date January 1986

Project Folsom Dam - Left Wing Dam

Boring No. L-1 & L-2

Sample No. 5,6,10, 17 | Method of Specimen Preparation undisturbed



SPECIMEN NO.	DENSITY	RELATIVE DENSITY	B	$\bar{\sigma}_{3c}$	CONSOLIDATION RATIO K_c	$\pm\sigma_{dc}$	$\pm\sigma_{dc}/2\bar{\sigma}_{3c}$	NUMBER OF CYCLES, N — PERCENT PORE PRESSURE INCREASE $N_{25\%}$	$N_{50\%}$	$N_{75\%}$	N_i	PERCENT STRAIN, ϵ_{da} N	N	N
87200	123.2	⊙ -	.95	1.0	1.0	0.44	0.22	70	-	-	-	-	-	-
87201	119.5	△ -	.95	1.0	1.0	1.01	0.51	1/2-	5	-	-	4.5	17	78
87205	126.4	⊡ -	.96	2.0	1.0	0.92	0.23	1	49	-	-	-	-	-
87257	125.5	X -	.97	4.0	1.0	1.71	0.21	1	5	11	-	1.5	8	33

PLATE 77

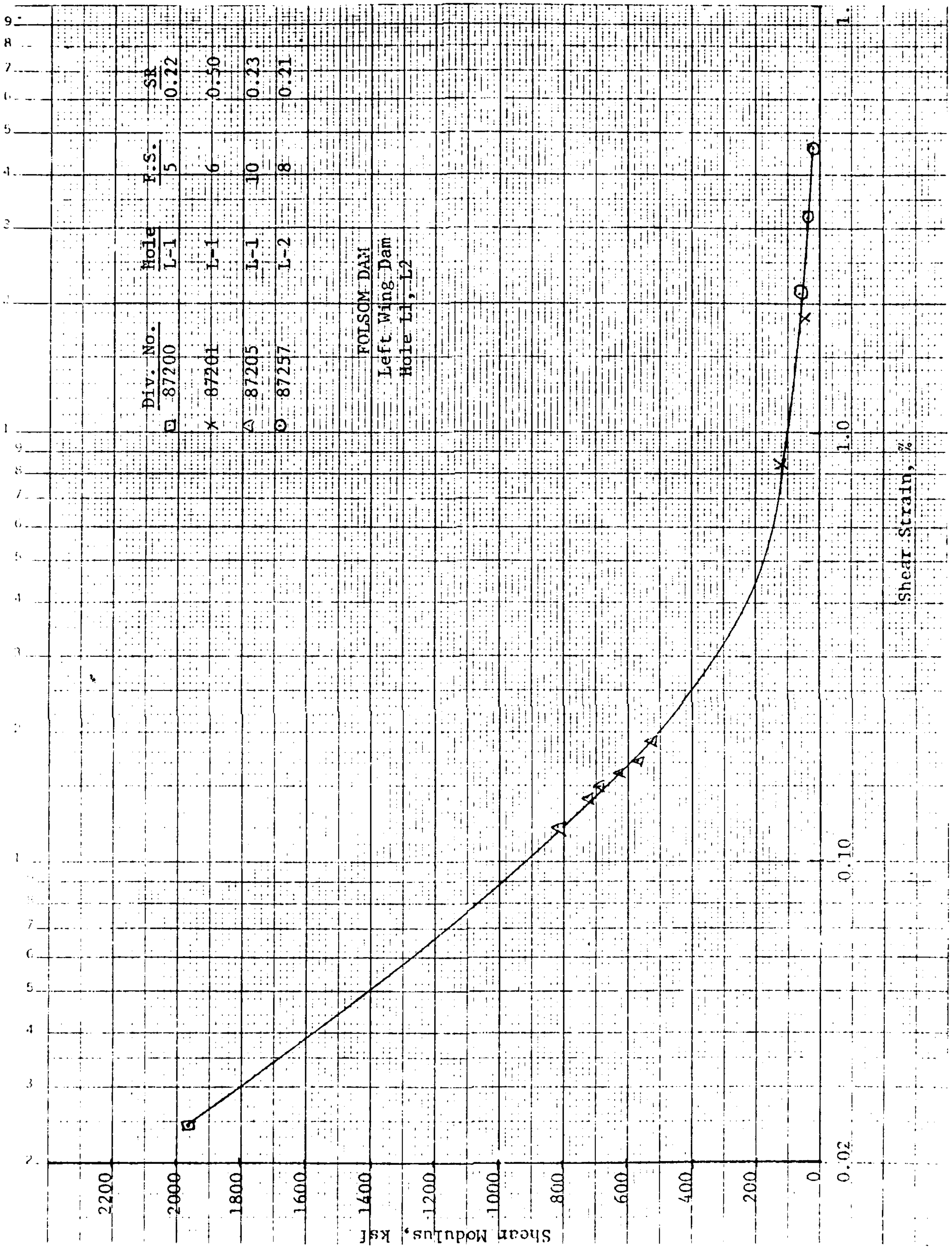
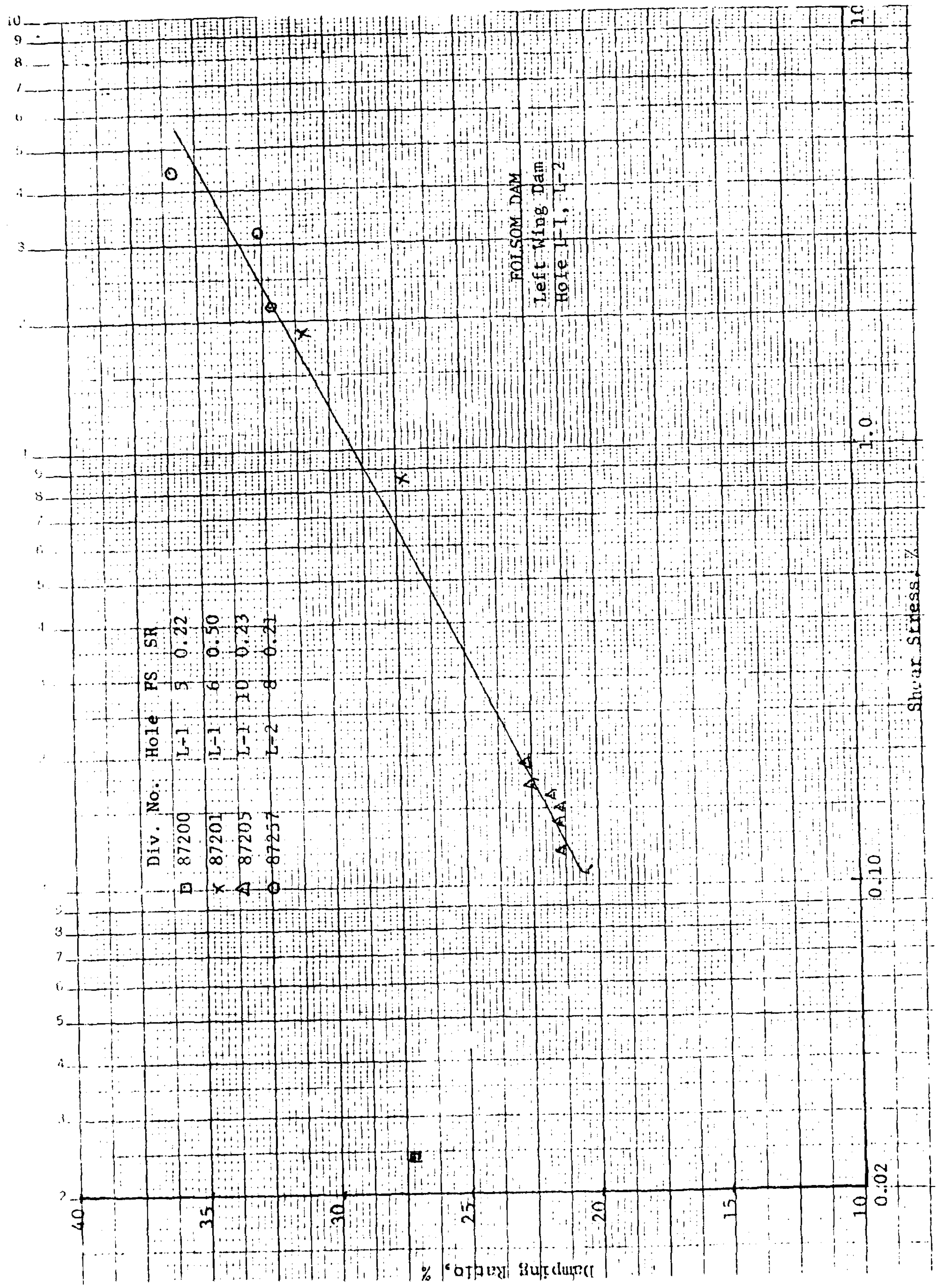
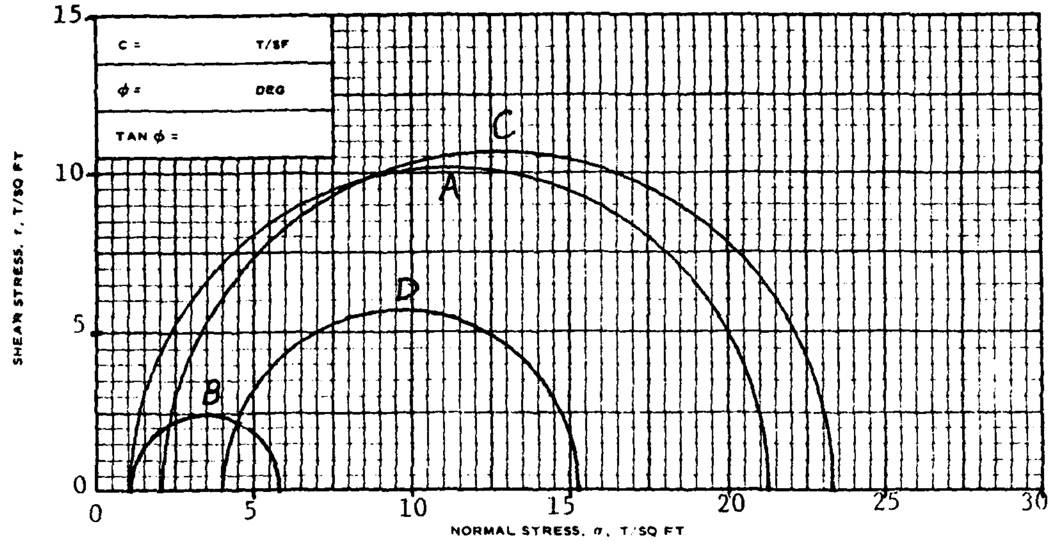
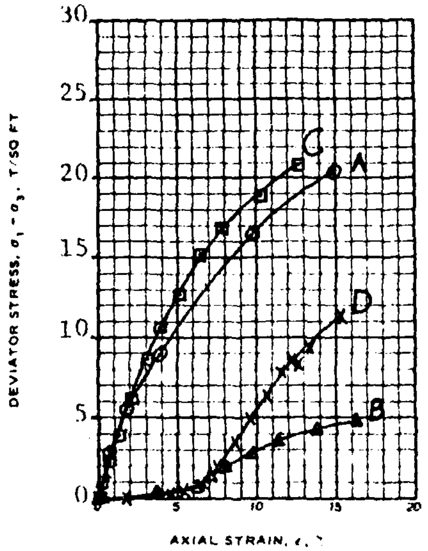

Div. No. Hole F.S. SR
87200 L-1 5 0.22
87201 L-1 6 0.50
87205 L-1 10 0.23
87257 L-2 8 0.21
FOLSOM DAM
Left Wing Dam
Hole L1, L2
Shear Strain, %
Shear Modulus, ksf
0.02
0.10
1.0
0
200
400
600
800
1000
1200
1400
1600
1800
2000
2200

PLATE 79


FOLSOM DAM
Left Wing Dam
Hole L-1, L-2
Div. No. Hole FS SR
87200 L-1 5 0.22
87201 L-1 6 0.50
87205 L-1 10 0.23
87257 L-2 8 0.21
Shear Stress, %
Damping Ratio, %
40
35
30
25
20
15
10
0.02
0.10
1.0
10





SPECIMEN NO.		A	B	C	D
INITIAL: WATER CONTENT, %	w_o	10.5	13.5	9.6	10.6
INITIAL: DRY DENSITY LB/CU FT	γ_{d_o}	123.1	119.5	126.4	124.7
INITIAL: SATURATION, %	S_o	75	85	75	77
INITIAL: VOID RATIO	e_o	0.384	0.436	0.353	0.381
BEFORE SHEAR: WATER CONTENT, %	w_c	13.9	15.7	12.1	12.8
BEFORE SHEAR: DRY DENSITY LB/CU FT	γ_{d_c}	123.6	120.0	128.3	127.3
BEFORE SHEAR: SATURATION, %	S_c	100	100	100	100
BEFORE SHEAR: VOID RATIO	e_c	0.378	0.431	0.333	0.353
BEFORE SHEAR: FINAL BACK PRESSURE, T/SQ FT	u_o	5.76	5.76	7.20	5.76
MINOR PRINCIPAL STRESS, T/SQ FT	σ_3	1.00	1.00	2.00	4.00
MAXIMUM DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{max}$	20.32	4.89	20.94	11.30
TIME TO $(\sigma_1 - \sigma_3)_{max}$, MIN	t_f	15.00	18.00	22.00	26.00
ULTIMATE DEVIATOR STRESS, T/SQ FT	$(\sigma_1 - \sigma_3)_{ult}$	–	–	–	–
INITIAL DIAMETER, IN.	D_o	2.83	2.82	2.81	2.90
INITIAL HEIGHT, IN.	H_o	6.26	6.39	6.40	6.40

CONTROLLED- Stress TEST

DESCRIPTION OF SPECIMENS A) Clayey Sand(SC-SM); B,C, & D)Clayey Sand(SC)

LL See Summary	PI	Gs 2.75	TYPE OF SPECIMEN undisturbed	TYPE OF TEST R

REMARKS: Post Cyclic

Div. No.	Boring	FS	Depth
A)87200	L-1	5	11.8-14.1
B)87201	L-1	6	16.0-16.5
C)87205	L-1	10	24-26.9
D)87257	L-2	17	87.3-89.6

PROJECT Folsom Dam
Left Wing Dam

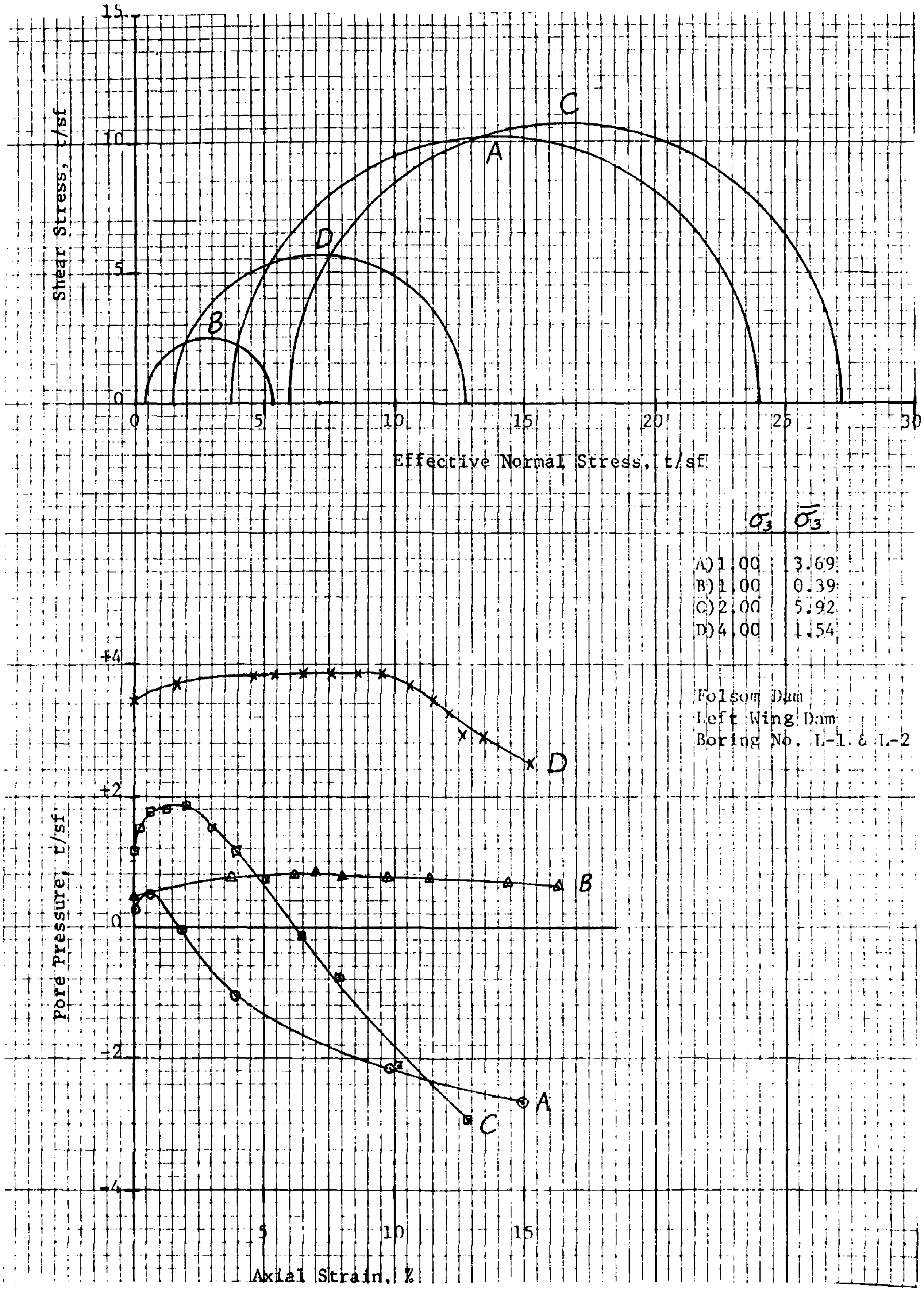
BORING NO. 11.8-89.6 SAMPLE NO.

DEPTH/ELEV

LABORATORY SPDL DATE June 1986

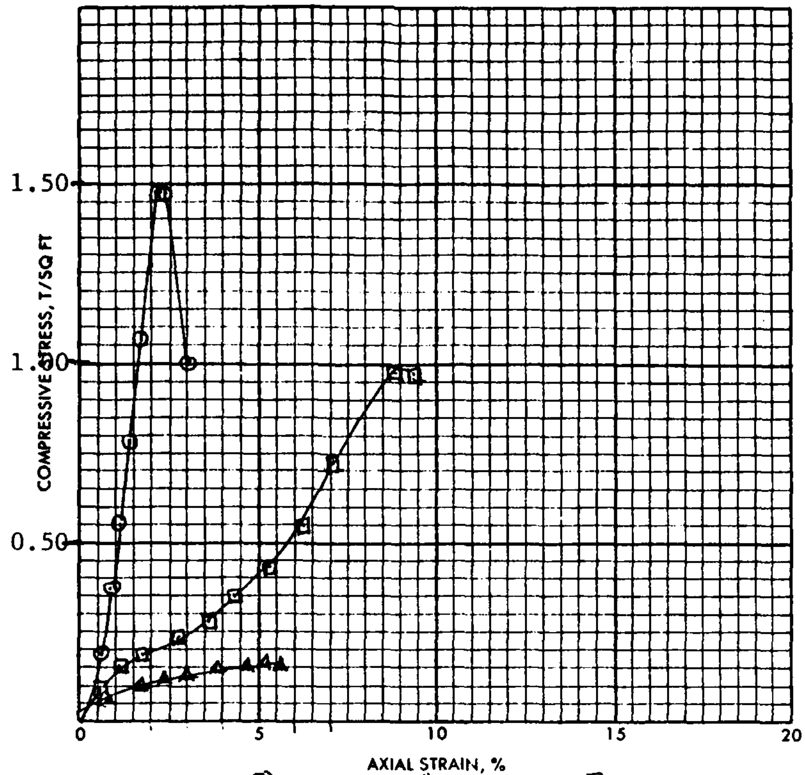
TRIAXIAL COMPRESSION TEST REPORT

PLATE 84



Shear Stress, t/sf
Effective Normal Stress, t/sf
A
B
C
D
σ3 σ̄3
A) 1.00 3.69
B) 1.00 0.39
C) 2.00 5.92
D) 4.00 1.54
Folsom Dam
Left Wing Dam
Boring No. L-1 & L-2
Pore Pressure, t/sf
Axial Strain, %

FAILURE SKETCHES



☐ CONTROLLED STRESS

☒ CONTROLLED STRAIN

		⊙	△	☐	
TEST NO.		L-1,FS 14	L-2, FS 5	L-2,FS 18	
TYPE OF SPECIMEN		Undist.	Undist.	Undist.	
INITIAL: WATER CONTENT	w_o	8.5 %	10.4 %	10.9 %	%
INITIAL: VOID RATIO	e_o	0.343	0.423	0.432	
INITIAL: SATURATION	S_o	69 %	67 %	69 %	%
INITIAL: DRY DENSITY, LB/CU FT	γ_d	130.1	119.7	120.3	
TIME TO FAILURE, MIN	t_f	5.25	7.5	10	
UNCONFINED COMPRESSIVE STRENGTH, T/SQ FT	q_u	1.49	0.16	1.00	
UNDRAINED SHEAR STRENGTH, T/SQ FT	s_u	0.74	0.08	0.50	
SENSITIVITY RATIO	S_t	-	-	-	
INITIAL SPECIMEN DIAMETER, IN	D_o	2.85	2.85	2.83	
INITIAL SPECIMEN HEIGHT, IN.	H_o	6.35	6.38	6.20	

CLASSIFICATION ⊙ Silty Sand(SM) △ Silty Sand(SM) ☐ Clayey Sand(SC)

LL -, 24, 28 | PL -, 22, 19 | PI NP, 2, 9 | G_s 2.80, 2.73, 2.76

REMARKS

	Div. No.	Hole	Depth, ft.
⊙	87209	L-1	34.0-36.6
△	87215	L-2	54.6-57.9
☐	82258	L-2	89.6-92.3

PROJECT FOLSOM DAM

AREA Left Wing Dam

BORING NO. | SAMPLE NO.

DEPTH EL | DATE June 1984

UNCONFINED COMPRESSION TEST REPORT

PLATE XI-2

PLATE 88

APPENDIX D

RESULTS OF LABORATORY TESTS BY SOUTH PACIFIC DIVISION LABORATORY (1989)

DRAFT

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: FOLSOM DAM

DATE: JANUARY 1989

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification	Mechanical Analysis - % Finer Gravel 1.5	3/4	3/8	#4	Sand #8	#16	#30	#50	#100	Fines #200	Liquid Limit	Plasticity Index	Field Moist. %
108172	7F-88-1	2	102.0	102.9	Clayey Sand (SC)		100	97	95	86	72	58	45	31	22	26	7	12.7
108177	7F-88-1	7	116.0	117.5	Clayey Sand (SC)	100	97	92	91	86	76	64	51	37	26	26	7	11.1
108182	7F-88-1	12	132.0	132.5	Clayey Sand (SC)		100	98	94	85	72	60	46	34	25	38	10	13.9
108187	7F-88-1	17	162.0	162.5	Poorly Graded Sand (SP)					100	96	62	26	8	4		NP	19.0
108192	7F-88-1	22	180.0	181.5	Clayey Sand (SC)				100	93	80	67	52	36	25	29	9	15.4
108197	7F-88-2	5	120.0	124.0	Clayey Sand (SC)				100	98	88	74	57	40	28	28	9	13.8
108202	7F-88-2	10	138.0	140.0	Clayey Sand (SC)				100	97	84	68	51	36	25	29	8	15.2
108207	7F-88-2	15	154.0	156.0	Clayey Sand (SC)				100	95	79	62	47	33	23	29	8	12.9
108212	7F-88-2	20	162.0	164.0	Clayey Sand (SC)		100	98	96	93	82	69	51	34	24	29	7	19.1

SPD FORM 66A

DRAFT

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION

SOIL TEST RESULT SUMMARY

PROJECT: Folsom Dam

DATE: January 1989

Division Number	Hole Number	Field Sample No.	Depth or Elevation From	Depth or Elevation To	Laboratory Descriptive Classification (VISUAL CLASSIFICATION)
108171	7F-88-1	1	100.0	101.5	Clayey Sand (SC): Reddish brown, moist 75% well-graded sand, 25% medium plastic fines, trace gravel to 3/8 in.
108172	7F-88-1	2	102.0	102.9	Clayey Sand (SC): Reddish brown, wet, medium subangular gravel.
108173	7F-88-1	3	105.0	105.5	Clayey Sand (SC): Brown, moist, 80% graded sand, 20% medium plastic fines
108174	7F-88-1	4	106.0	107.5	Clayey Sand (SC): Yellowish brown, moist 80% graded sand, 20% medium plastic fines, trace No. 4 gravel
108175	7F-88-1	5	110.0	111.0	Clayey Sand (SC): Yellowish brown, moist, 75% graded sand, 20% medium plastic fines, 5% medium subangular gravel to 3/4 in.
108176	7F-88-1	6	114.0	114.2	Clayey Sand (SC): Yellowish brown, moist, 80% graded sand, 20% medium plastic fines, trace No. 4 gravel.
108177	7F-88-1	7	116.0	117.5	Clayey Sand (SC): Reddish brown, wet, medium subangular gravel.
108178	7F-88-1	8	118.0	118.4	Clayey Sand (SC): Brown, moist, 80% well-graded sand, 20% medium plastic fines, trace No. 4 gravel.
108179	7F-88-1	9	122.0	122.9	Clayey Sand (SC): Yellowish brown, moist, 80% well-graded sand, 20% medium plastic fines, trace No. 4 gravel.
108180	7F-88-1	10	126.0	126.5	Clayey Sand (SC): Grayish brown, moist, 80% well-graded sand, 20% medium plastic fines, trace No. 4 gravel.
108181	7F-88-1	11	128.0	128.3	Clayey Sand (SC): Yellowish brown, moist 85% well-graded sand, 15% medium plastic fines, trace No. 4 gravel.
108182	7F-88-1	12	132.0	132.5	Clayey Sand (SC): Reddish brown, wet, medium subangular gravel.
108183	7F-88-1	13	136.0	137.5	Well-Graded Sand with Clay (SW-SC): Yellowish brown, moist, 90% well-graded sand, 10% medium plastic fines, trace No. 4 gravel.
108184	7F-88-1	14	144.0	144.3	Well-Graded Sand with Clay (SW-SC): Yellowish brown, moist, 85% well-graded sand, 10% medium plastic fines, 5% medium subangular gravel.
108185	7F-88-1	15	150.0	150.5	Clayey Sand (SC): Yellowish brown, wet, 80% well-graded sand, 15% medium plastic fines, 5% medium subangular gravel to 3/8 in.

SPD FORM 66A (MODIFIED)

DRAFT

U.S ARMY ENGINEER DIVISION LABORATORY - - SOUTH PACIFIC DIVISION					
SOIL TEST RESULT SUMMARY					
PROJECT: Folsom Dam					DATE: January 1989
Division Number	Hole Number	Field Sample No.	Depth or Elevation From	To	Laboratory Descriptive Classification (VISUAL CLASSIFICATION)
108186	7F-88-1	16	160.0	160.3	Clayey Sand (SC): Yellowish brown, moist, 75% well-graded sand, 25% medium plastic fines, trace No. 4 gravel.
108187	7F-88-1	17	162.0	162.5	Poorly Graded Sand (SP): Pale brown, wet, non-plastic.
108188	7F-88-1	18	162.5	162.8	Well-Graded Sand with Clay and Gravel (SW-SC): Yellowish brown, wet, 55% well-graded sand, 35% medium subangular gravel to 3/8 in., 10% medium plastic fines.
108189	7F-88-1	19	166.0	166.9	Clayey Sand with Gravel (SC): Yellowish brown, moist, 75% well-graded sand, 15% medium plastic fines, 10% medium subangular gravel to 3/8 in.
108190	7F-88-1	20	172.0	173.5	Well-Graded Sand with Clay and Gravel (SW-SC): Yellowish brown, moist, 80% well-graded sand, 10% medium plastic fines, 10% medium subangular No. 4 gravel.
108191	7F-88-1	21	174.0	175.3	Clayey Sand with Gravel (SC): Yellowish brown, moist, 75% well-graded sand, 15% medium plastic fines, 10% medium subangular gravel to 3/8 in.
108192	7F-88-1	22	180.0	181.5	Clayey Sand (SC): Reddish brown, wet, trace gravel.
108193	7F-88-2	1	100.0	104.5	Clayey Sand (SC): Brown, moist, 80% well-graded sand, 15% medium plastic fines, 5% medium subangular gravel to 3/8 in.
108194	7F-88-2	2	106.0	110.0	Clayey Sand (SC): Brown, wet, 80% well-graded sand, 20% medium to high plastic fines.
108195	7F-88-2	3	110.0	118.0	Clayey Sand (SC): Brown, wet, 75% well-graded sand, 25% medium plastic fines, trace gravel to 3/8 in.
108196	7F-88-2	4	118.0	120.0	Clayey Sand (SC): Reddish brown, wet, 60% well-graded sand, 40% medium to high plastic fines, trace No. 4 gravel.
108197	7F-88-2	5	120.0	124.0	Clayey Sand (SC): Reddish brown, wet, trace gravel.
108198	7F-88-2	6	124.0	124.5	Clayey Sand (SC): Yellowish brown, moist, 75% well-graded sand, 25% medium plastic fines.
108199	7F-88-2	7	124.5	126.0	Clayey Sand (SC): Brown, moist, 75% well-graded sand, 25% medium plastic fines, trace No. 4 gravel.
108200	7F-88-2	8	126.0	128.0	Clayey Sand (SC): Yellowish brown, moist, 80% well-graded sand, 20% medium plastic fines, trace No. 4 gravel.

SPD FORM 66A (MODIFIED)