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US Army Corps
of Engineers
Los Angeles District

Gila River Basin
Arizona

ADOBE DAM FOUNDATION REPORT

October 1982

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Los Angeles District

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ATTACHMENTS

1. *Introduction*

Project Location

1.01 The Adobe project is on Skunk Creek in Maricopa County, Arizona, approximately 16 miles north of downtown Phoenix. (See Project Location Map, fig. 1.) It crosses a projection of Deer Valley Road about 1.8 miles west of the Black Canyon Highway, I-17, and is crossed by 35th Avenue. Physio-graphically, the site lies in the northern portion of Deer Valley, a part of the Salt River Valley. The dam is between the southern end of the Hedgepeth Hills on the west and Adobe Mountain on the east (photo 1). Ephemeral Skunk Creek, the prominent stream in the 89.6-square-mile drainage area above the dam, flows southwesterly from its headwaters to the dam, a distance of approximately 20 miles.

Geologic Setting

1.02 Deer Valley and the larger Salt River Valley are in the south central Arizona portion of the Basin and Range Geomorphic Province. The Province is characterized by low but rugged mountains separated by broad alluvial filled intermontane basins. The rocks which make up the individual mountains are varied lithologically and in some areas are almost entirely concealed beneath the vast coalescing alluvial fans which surround them. The basins between the mountains are filled with thick accumulations of alluvium ranging from lacustrine silt and clay deposits to river laid gravel and cobbles. The block fault basins are generally very deep. Some contain over 3000 feet of accumulated sediments. Adobe Mountain, the Hedgepeth Hills, and the valley between are typical of the area except that no concealed pediment is present, at least along the dam alinement. The mountains rise abruptly above flat valley floor to a moderate height. Both are covered by talus blocks derived from later Tertiary to Quaternary volcanic flows. The underlying bedrock is also of volcanic origin and includes a complex association of rock types including latite, andesite, flow breccia, and tuffaceous agglomerate. Alluvium under the dam is in excess of 1250 feet and the groundwater level is approximately 500 feet below the ground surface. Although the landforms in the area are the result of widespread faulting and tectonism, these forces reached a maximum during the Miocene period, and the present day seismicity in the area of the dam is considered insignificant.

Project Description

1.03 Adobe Dam is one of four dams (Dreamy Draw, Cave Buttes, New River, and Adobe) which, along with a diversion channel downstream, will provide flood protection for the Phoenix Metropolitan area. Collectively, these elements comprise the New River and Phoenix City Streams Flood Control Project authorized by the Flood Control Act of 1965. The dam is a compacted earthfill structure containing 2.3 million cubic yards with a maximum height of 63 feet above streambed near the right abutment (photo 2). At the left abutment, 2.1 miles to the northeast, the dam is about 8 feet high. The embankment has a sloped core and random shells. It contains an inclined and horizontal grave drains (photo 3). The upstream random shell is underlain by a 5-foot-thick core blanket.

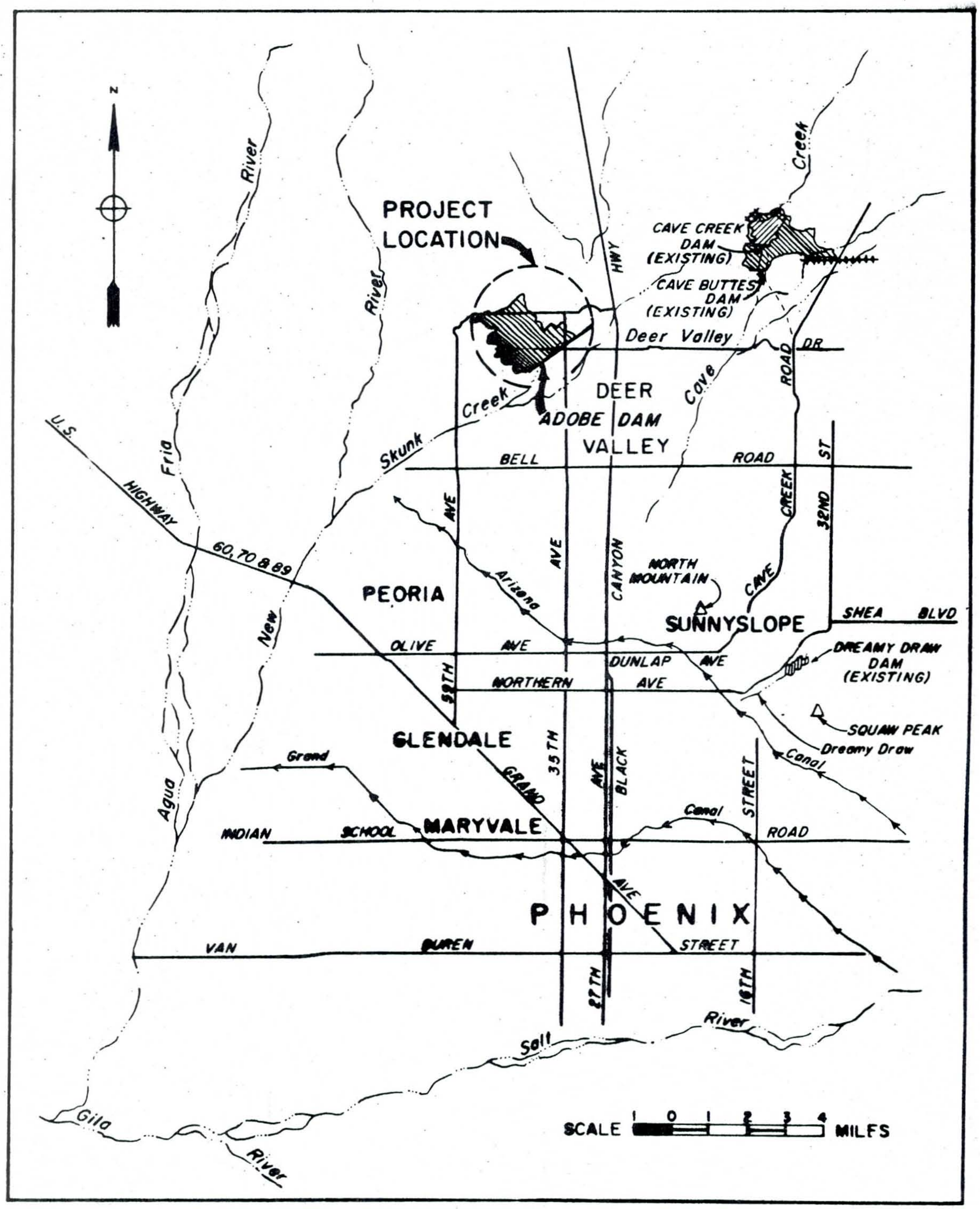


Figure 1. Project Location Map.

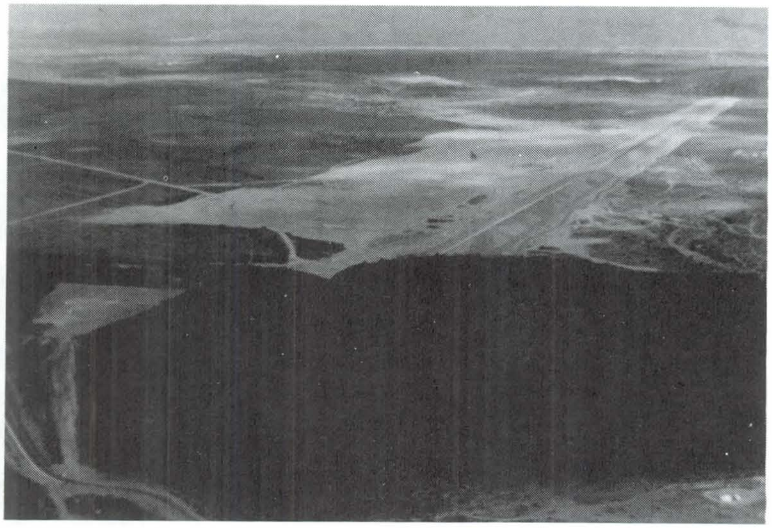


Photo 1
Aerial view of completed dam looking north-northeast. Spillway cut at lower left, Hedgepeth Hills in foreground, Adobe Mountain at upper right. (8 Apr 1982)

Photo 2
Aerial view of completed dam looking west (upstream); Hedgepeth Hills at upper left, outlet channel at center left. Soil patches on embankment are part of the esthetic treatment. (8 Apr 1982)

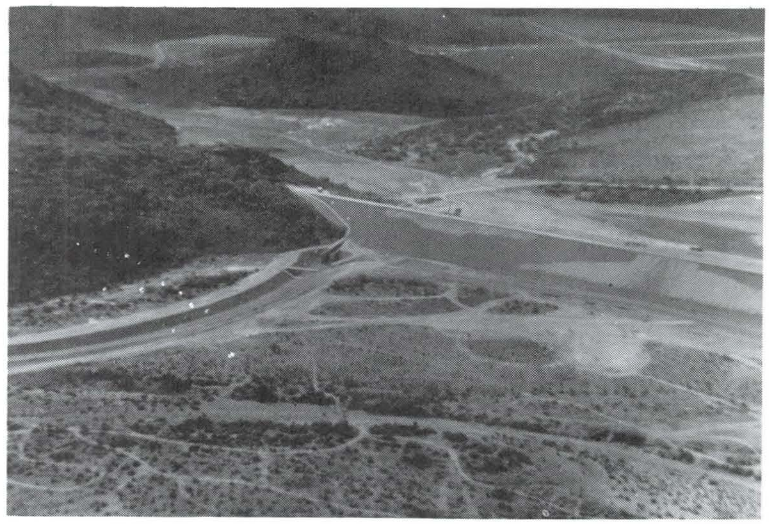


Photo 3
Right abutment of Adobe Dam after dental excavation. Zoned embankment in foreground; horizontal gravel blanket drain at left, inclined drain at lower left center, core blanket at right. (21 Oct 1981)

(See amended pls. 13 and 14. Selected amended contract drawings are included as plates in this report and are sequentially numbered in the order in which they appeared in the plans and specifications.) The foundation for the dam is valley fill alluvium except at the right abutment where the dam is constructed on the volcanic bedrock of the Hedgepeth Hills. The detention basin has a capacity of 18,350 acre-feet and reduces the 66,000 ft^3/s peak inflow of the standard project flood to an outflow of 1890 ft^3/s through an ungated rectangular outlet conduit. Drawdown time for the standard project flood (SPF) is 9-1/2 days. The unlined spillway, which is designed to pass the probable maximum flood (PMF), is excavated in rock 2000 feet west of the west abutment (photo 1). It has a total profile length of 1325 feet and is 36 feet wide at the bottom with 1/2:1 side slopes.

Project History

1.04 During project formulation, several alternative sites for Adobe Dam were evaluated including a project-document site 4.5 miles upstream from the eventual construction site. Exploration at the document site was initiated in 1963 and exploration of the alternate sites began in 1973. By 1976, the as-constructed site, which had previously been referred to as alternate site 4, had been selected for Adobe Dam. Geotechnical considerations had no significant influence on site selection. The construction contract was advertised under bid solicitation DACW09-80-B-0035 on 7 August 1980. M.M. Sundt Construction Company of Tucson, Arizona, was awarded contract DACW09-80-C-0121 with a total bid of $8,388,025.00. The remaining bids ranged between $9,906,497 and $18,432,287.50. The Government estimate was $13,372,303. Construction began with the ground breaking on 30 October 1980. The dam was officially dedicated on 8 April 1982, approximately 6 months in advance of the required completion date.

Key Resident and Design Staff

1.05 The following is a partial list of Corps of Engineers personnel responsible for the design and construction of Adobe Dam.

Engineering Division:

Project Manager	Nick Romanzov
Project Design Leader	Vance Carson
Project Geologist	David Lukesh
Embankment Design	Tak Yamashita
Hydraulic Design	Ken Warner

Construction Division:

Project Engineer	Terry Buckley
Project Officer	Captain Paul Dunn
Embankment Engineer	Paul Ching
Office Engineer	Dan Moore
Field Superintendent	Joe Salinaz
Laboratory Chief	Dewayne Godsell

Contract Supervision and Quality Control

1.06 In addition to the Construction Division personnel listed above, the Adobe Dam Project Officer maintained a staff of six inspectors and a laboratory crew of seven civilian and military personnel. Quality control over embankment placement was the responsibility of the Corps of Engineers. The Contractor had quality control over the other aspects of construction, which were nonetheless under constant inspection by the Project Office staff. Supervision and inspection of geotechnically related items such as spillway and outlet works excavation, foundation drilling and grouting, and abutment preparation and initial material placement were the responsibility of the Engineering Division Geotechnical Branch, primarily the Project Geologist.

Subcontractors

1.07 The prime contractor, M.M. Sundt, subcontracted several aspects related to the construction of Adobe Dam. The major subcontractors performing geotechnically related tasks were as follows:

- WBC Consultants, Inc., Phoenix, Arizona, the project surveyors.
- W.G. Jaques Co., Des Moines, Iowa, the drilling and grouting subcontractor.
- D.C. Speer Construction Co., Phoenix, Arizona, the rock crushing contractor.
- Engineers Testing Laboratories Inc. (ETL), Phoenix, Arizona, performing materials testing.

Purpose of Foundation Report

1.08 The purpose of this report is to satisfy the requirements set forth in ER 1110-1-1801, dated 15 December 1981. To that end the following topics will be addressed:

- Foundation conditions expected and those actually encountered.
- Construction methods utilized.
- Testing and explorations performed during construction.
- Solutions to construction problems or unexpected foundation conditions.
- Foundation design and changes to that design.
- Conditions which may require post-construction observation or treatment.

This information is intended for use (1) in planning additional foundation treatment should the need arise after project completion, (2) in evaluating the cause of failure or partial failure, (3) for guidance in planning foundation explorations and in anticipating foundation problems for future comparable construction projects, (4) as an information base in determining the validity of claims made by the contractor in connection with foundation conditions, and (5) as a part of the permanent collection of project engineering data.

2. Pre-Construction Phase: Geotechnical Aspects

Field Investigations

2.01 Following geologic reconnaissance and preliminary mapping, subsurface geotechnical investigations were initiated at the damsite in 1973. The foundation of the dam, right abutment, outlet works, three alternative spillways, and potential borrow areas were explored in order to established the design criteria and feasibility of constructing an earthfill dam and appurtenant works at the site. Subsurface investigations consisted of deep and shallow seismic refraction surveys, uphole seismic surveys, downhole electrical and gamma ray surveys, diamond core drilling, bucket auger drilling, dozer and backhoe trenching, and in-place density and percolation testing. Amended plates 2 and 9 show the location of the geological and geophysical investigations (exclusive of spillway alternative 2). The investigations performed are summarized briefly in the following paragraphs. Additional information is available in the Phase II, General Design Memorandum, appendix 1A, dated April 1979.

SPILLWAY

2.02 In addition to exploration at the as-constructed spillway site (alternative no. 1), subsurface investigations were conducted at two additional sites. Spillway alternative no. 3 was sited on the right abutment near the contact with the embankment. The drilling of core hole D-23 was the only exploration performed for this alinement. Spillway alternatives 1 and 2 were in natural topographic saddles west of the dam. A combination of seismic refraction surveys and diamond core holes was used to assess the rock types to be encountered, the rippability of materials, and the suitability of the rock for use as stone protection for the dam. Because of the apparent chaotic nature of the volcanic rocks encountered in the core borings and the possibility that lower velocity rock might exist at depth, a Meissner uphole wavefront survey was conducted at the as-constructed spillway site. The purpose of this survey was to (1) establish the depth, thickness, and lateral extent of geologic materials; and (2) estimate rock-breaking characteristics during excavation. In addition to these investigations, two test trenches were excavated at site no. 1 using a D9-H dozer and single-tooth ripper in order to further assess the excavation characteristics of the varied rock types. (See amended pls. 4 and 5 for the plan and profiles of the geologic investigations at the as-constructed spillway.)

RIGHT ABUTMENT

2.03 The investigation of the right abutment consisted of four diamond core holes (D-9, D-10, D-11, and D-23) ranging in depth from 50.1 to 81.2 feet, one test trench at the downstream toe, and several seismic refraction survey lines. (See amended pls. 2 and 3 for the location of the core holes and geophysical lines. See fig. 2 for the location at the test excavation.) The core holes were drilled in order to determine (1) the thickness of overburden, (2) the subsurface geologic structure, and (3) the rock-mass permeability. The purpose of the trench was to assess the excavation characteristics of the overburden and to inspect the abutment foundation surface. A D8-H dozer, however, was unsuccessful in its attempt to excavate to bedrock. After digging only about 3 feet into the abutment toe, the trench was aborted because of the large size of the talus blocks, the caliche cementation between the blocks, and the constant breakdown of the equipment. The geophysical surveys were intended to yield information on bedrock velocity and the overburden/bedrock contact. The results of the surveys on the abutment (survey lines 11, 12, and 13) were poor, indicating only qualitative data which suggested that the foundation rock surface dipped steeply toward the northeast under the alluvium. The near-surface velocity profile yielded inconclusive results because of the extensive talus pile at the surface.

OUTLET WORKS

2.04 Exploration for the outlet works is depicted on amended plate 3. Six diamond core holes were drilled along the as-built outlet works alinement, in addition to three holes drilled along a preliminary alinement away from the toe of the abutment. One test trench, 78-1, was also excavated near the inlet portal. The purpose of the core holes was to determine the nature and thickness of the overburden and the competance of the bedrock foundation under the outlet. Trench 78-1, like the attempted trench at the downstream toe of the abutment, was excavated in order to assess the insitu nature of the bedrock and the excavation characteristics of the overburden. The logs of the core holes and test trench are shown on amended plate 6.

LEFT ABUTMENT

2.05 Foundation explorations were not conducted at the left abutment.

MAIN EMBANKMENT

2.06 Between 1973 and 1979, 10 auger holes, 6 rotary holes, and 7 backhoe and 6 dozer trenches were excavated along or near the embankment axis (amended pl. 9). The auger holes were used to obtain representative samples for mechanical analysis of each change in soil type or at 3-foot intervals and to conduct standard penetration tests. Percolation tests were conducted in two holes. Electrical and gamma ray logging was performed in 5 of the rotary holes in conjunction with the auger-hole testing in order to better correlate the properties of the alluvium from hole to hole across the valley. The trenches were excavated to determine the near-surface foundation conditions by visual examination, mechanical analysis, sand-cone density testing, and undisturbed tube and carved sampling. In addition to the above investigations, seismic refraction surveys were conducted at various locations along the alinement in order to determine the subsurface velocity profile.

Anticipated Geologic Conditions

2.07 Information from the geotechnical investigations was analyzed to determine the foundation characteristics at the damsite. Design criteria, requirements for excavation, and foundation treatment were formulated based upon these anticipated conditions. Altough the analysis of the data is detailed in the Phase II GDM and implied in the contract plans, the anticipated foundation conditions associated with the major features of the dam will be briefly summarized. The actual conditions encountered during construction are discussed later in this report.

SPILLWAY

2.08 Data from the 13 core holes indicated that the material to be excavated from the spillway site was a complex association of Quarternary/Tertiary volcanics. Surface expression of the rock was masked by up to 4-1/2 feet of residual soil and slope wash. The rock types represented included andesite, basalt, and flow breccia, which were overlain by tuffaceous agglomerate and caliche-cemented volcanic colluvium at the upstream end. The distinction between andesite and basalt was based upon visual examination (primarily the relative degree of vesicularity) and not chemical composition. Pre-construction petrographic analysis was not performed on rock samples from the spillway. Although correlation between the different units was very difficult from the logs of the core holes, surface geologic mapping indicated that the flows dipped upstream at approximately 20 degrees. Compressional wave velocities recorded in the geophysical surveys ranged from 4500 to 11,000 ft^3/s. Based upon this information and the condition of the recovered core, it was believed that the more highly fractured and jointed rock and tuffaceous agglomerate could be ripped, that the flow breccia would require selective excavation because of the variability in hardness, and that the harder and more massive andesite/basalt would require blasting. As shown on amended plate 4, it was believed that a concealed normal fault trended sub-parallel to the spillway axis on the west side (amended pl. 1). The existence of such a fault, as mapped by the U.S. Geological Survey, was implied by the topography in the saddle and the relative position of rock units as encountered in the core holes. It was felt that such a fault, if it did actually exist, would have been associated with ancient Basin and Range tectonism and would have no significant effect on the slopes or the stability of the sill.

RIGHT ABUTMENT

2.09 The right abutment of Adobe Dam had the appearance of a rubble pile of desert varnished volcanic blocks up to 6 feet in maximum dimension. (See photo 4 for a view of the unaltered portion of the abutment as it exists today.) Under this surficial cover, the core borings indicated that the talus blocks were infilled with loose silty sand slope wash and locally cemented with caliche. This overburden varied in thickness from 5 to 12 feet, and the approximate bedrock surface depicted on plate 29 of 72 of the contract plans (amended pl. 12) was based upon extrapolation between the corings along the embankment and the outlet works centerlines. The cores indicated that the bedrock was an irregularly fractured basalt (latite) with fine grained ash infilling between the blocks. The contact between the overburden and bedrock was difficult to distinguish because the talus blocks were of the same composition as the rock below. The determination was based primarily upon the degree of induration and composition of the infilling.

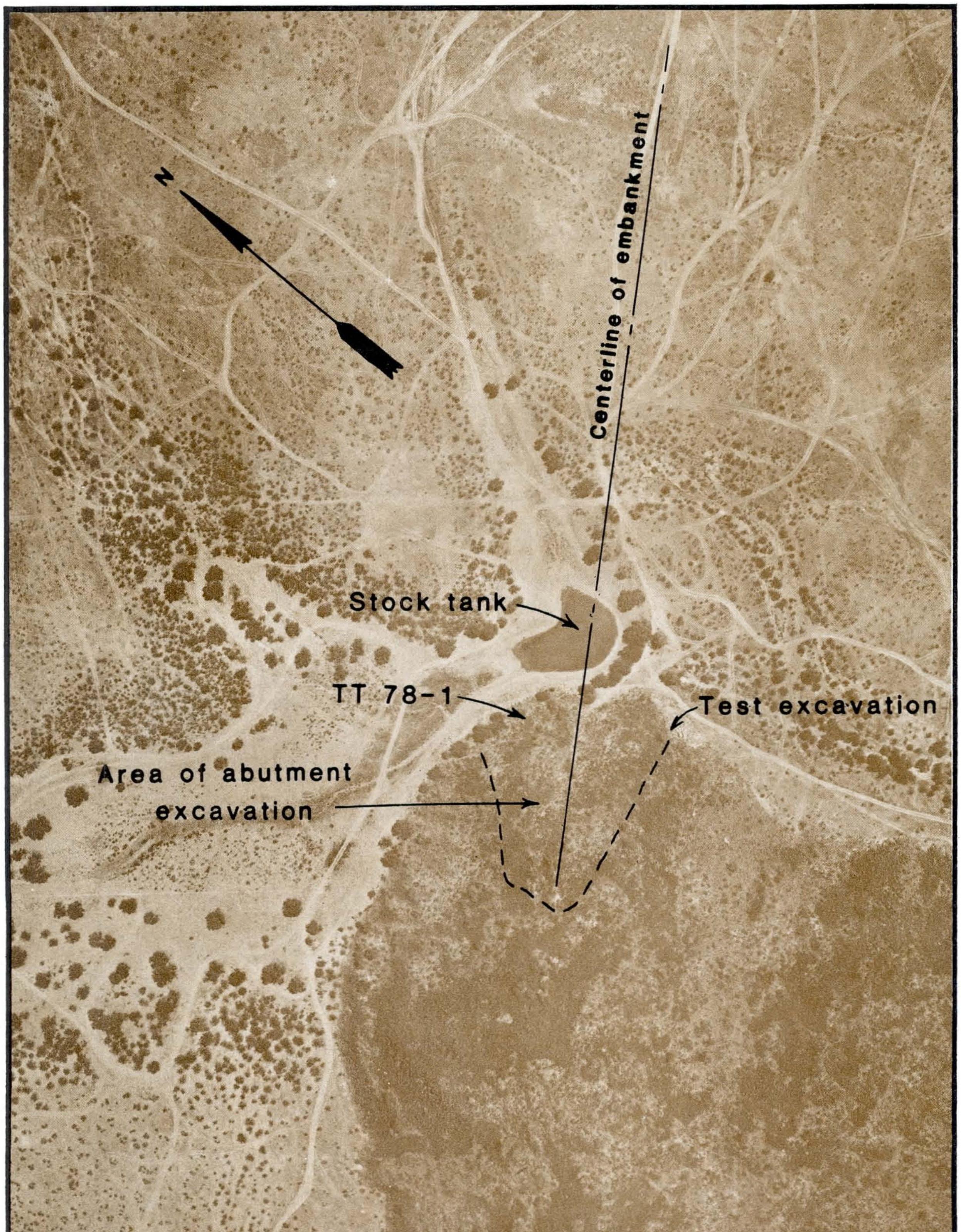


Figure 2. Right Abutment Pre-construction.

The basalt, which was 20 to 35 feet thick, graded to a medium to highly fractured andesite. Pressure tests indicated that the basalt had a low average permeability (0.4 ft/day) and the underlying fractured andesite had computed permeabilities of up to 5 ft/day. An exploratory grouting program was designed (amended pl. 11), which assumed an essentially tight foundation in the upper 30 feet.

OUTLET WORKS

2.10 The foundation for the outlet works was anticipated to be fractured basalt with thin pockets of tuffaceous material, overlain at the downstream margin by a well-indurated tuffaceous agglomerate. It was believed that the excavation of the outlet trench would require blasting, and a 5-foot buffer zone was specified in order to control the excavation limits.

LEFT ABUTMENT

2.11 The left abutment is at the toe of Adobe Mountain, which has the same surficial appearance (photo 5) as the Hedgepeth Hills at the right abutment. It was believed that materials encountered in the minor excavation necessary would consist of variably cemented volcanic talus and would be suitable as a foundation for the random fill embankment section.

MAIN EMBANKMENT FOUNDATION

2.12 A review of the logs of borings and trenches and results of detailed tests indicated a change of materials and properties in the alluvial foundation at a depth of about 5 feet (amended pl. 10). The materials within the upper 5 feet included 1 to 5 feet of fine grained collapsible soils classifying predominantly as sandy silts and clays to silty and clayey sands. (These materials were used as core material in the dam embankment). With the exception of several isolated areas of coarse grained sands and gravels, the fine soils were believed to be widespread along the dam alinement. Because of the variable thickness and the potential for collapse of the fine grained soils, the entire embankment foundation was excavated to a depth of 5 feet and replaced upstream with a blanket of core material 5 feet thick. The alluvium below a depth of 5 feet classified predominantly as noncollapsible silty and clayey gravelly sands with permeabilities estimated to range between 0.1 and 100 ft/day. Under the core zone, an exploration trench with a 20-foot bottom width and side slopes on 1V on 1.5H was included in the embankment design (amended pl. 14) so that deleterious materials such as nested cobbles and boulders or pockets of gravel could be identified and removed.

Photo 4
View of completed embankment looking toward Adobe Mountain (left abutment). Desert varnished volcanic talus blocks typical of surficial cover on the Hedgepeth Hills at lower right. (9 Apr 1982)

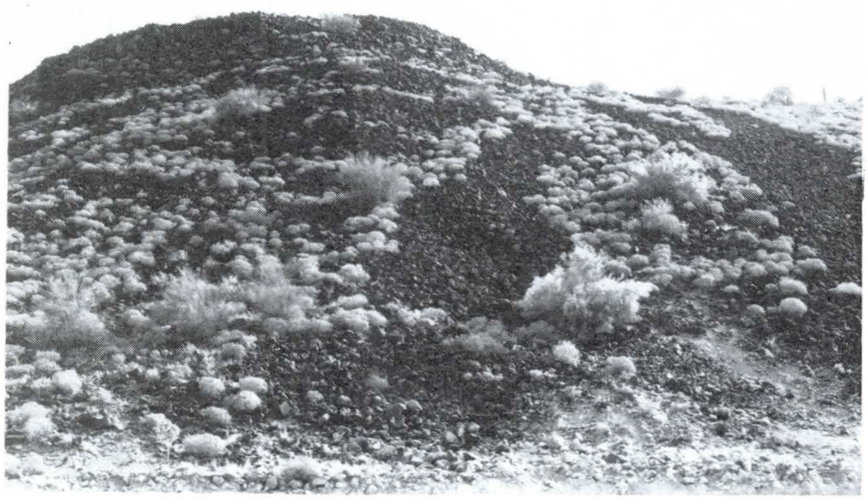


Photo 5
Surficial cover of desert varnished volcanic talus blocks on Adobe Mountain (left abutment). Excavation scar for "end of dam" at very bottom of photo. (26 Mar 1981)

3. *Construction Phase: Geotechnical Aspects*

Spillway

3.01 The spillway for Adobe Dam is excavated in rock through a natural topographic saddle about 2000 feet west of the left abutment (photo 1). It was designed to retain the SPF while providing a controlled discharge of the PMF. Sill invert is at elevation 1377.8. A 95-foot maximum cut and removal of 233,000 cubic yards of material was required to excavate to this elevation. Although the amount of excavation would have been less at alternative spillway no. 2, the increased distance from the Skunk Creek channel would have required costly flowage easements. (See amended pl. 1 for the location of the spillway sites.) The as-constructed spillway is 1325 feet in total length with a minimum base width of 36 feet. Irregularities in the natural rock slopes resulted in a varied bottom width. The invert section is stabilized by a concrete sill across the bottom and up the side slopes; otherwise the spillway is unlined. Based upon hydrologic, hydraulic, and least-cost requirements the spillway is trapezoidal in cross-section with side slopes of 2V on 1H and benches 12 feet wide at 30-foot height intervals (photo 6). Determination of the side-slope angle was based upon experience in similar rock. The benches were designed to reduce the velocity of surface runoff down the slopes and to catch slope talus before reaching the relatively narrow bottom. Since the contractor planned on utilizing at least some of the excavated spillway material in the construction of the dam, he began his excavation soon after mobilizing to the site. The initial test blast was conducted on 18 November 1980. Excavation was continuous through March and was then sporadic through October 1981. The concrete sill was completed in January 1982.

GEOLOGY

3.02 The geology at the spillway site is complex both lithologically and structurally. The various rock units are described separately followed by a discussion of the geologic structure.

Lithologic Units

3.03 Both walls of the excavation were mapped at a scale of 1:120; however, only the geology of the east wall, plate 1 (which is representative of the west wall), is included in this report. The floor of the excavation was never fully cleaned and was not mapped. For mapping purposes, the rocks exposed in the spillway excavation were divided into six major units: (1) agglomerate, (2) upper andesite, (3) transitional andesite/flow breccia, (4) flow breccia, (5) lower andesite, and (6) amygdaloidal andesite. The rock names were based at least partially on a petrographic analysis of five samples collected in March 1981 in which both the upper andesite and transitional rock classified as porphyritic andesite. In an independent analysis of the same rock units as performed by Dr. Michael Sheridan of Arizona State University, they were classified as trachyte to trachyte-andesite. The contract plans referred to the rocks as either andesite or basalt with the distinction based primarily upon color and texture instead of chemical composition. Since the actual rock name is based upon slight differences in chemical composition and both the engineering properties and the interpretation of the geologic structure are not affected by the classification, the six unit names used during mapping, which are based both upon chemical analysis and rock-mass characteristics, will be used in this report. Correlation with rock units, as presented in the contract plans, is tenuous and not significant. Generally, rock identified in the contract plans as basalt is actually one of the two andesite units. The flow breccia was usually identified correctly. Rock unit names were changed on amended plates 4, 5, 7, and 8 to conform to the lithologic classification used in this report. Contacts between the different lithologic units, however, were difficult to identify even while mapping a fully excavated face; and therefore, the contacts as shown on the drill logs should be considered only as approximations.

3.04 **Agglomerate.** The youngest geologic unit in the spillway, other than the pervasive thin blanket of residual soil and colluvium (Qoal), is an agglomerate which is extensively exposed at the northeast end of the excavation upstream from station 23 (photo 7). It has variously been classified as (1) calichified breccia (Sheridan, attachment 1), (2) volcanic lithic limestone (SPD laboratory petrographic analysis), and (3) tuffaceous agglomerate (contract plans). Although no one term is entirely appropriate, in this report the unit will be referred to simply as an agglomerate instead of a breccia in order to avoid confusion with a volcanic flow breccia exposed elsewhere in the spillway. It is a crudely stratified assemblage of sub-angular to sub-rounded heterolithic volcanic clasts in a variably cemented, calcium carbonate-rich matrix. Its formation was probably the result of caliche-type cementation of colluvium over a long period of time during a colder and wetter paleoclimatic cycle. Although the agglomerate unit is approximately 40 feet thick, calichification to this degree is not uncommon in central Arizona (Dr. Sheridan, personal communication). The clasts are predominantly red and grey andesites which vary in size from gravel to 3-foot diameter boulders. Apparent stratification of the clasts is subparallel to the contact with the volcanic rocks below. This contact, which dips approximately 30 degrees upstream, is relatively planar and probably represents a modified fault scarp. The base of the unit is very poorly cemented, contains 70- to 80-percent cobble to boulder-size clasts and is prone to sluffing (photo 8). It is only 5 feet thick on the east side, but is up to 15 feet thick on the west side. With the degree of cementation in the overlying section ranging from moderate to extreme, it could not be easily ripped, even with a D9-L dozer (photo 9), and blasting was required for excavation. Evidence of the pre-split blasting is well-preserved in the cemented agglomerate (photo 10) and the slopes are stable.

3.05 **Upper Andesite.** The upper andesite flow is topographically the highest rock unit exposed in the spillway excavation. It occurs on the top and flanks of a flow breccia unit as well as sill-like intrusions within the breccia (pl. 1). Its absolute age is undetermined, but it was probably deposited, along with the other volcanic rocks exposed in the spillway, during the late Tertiary before or concurrent with Basin and Range fault uplift. The flow is more or less coherent and exhibits extreme platy jointing (photo 11), which is locally contorted into swirling patterns (photo 12). The joints are smooth, generally uncoated, and planar to arcuate. Their orientation is varied and unpredictable (pl. 1). In some areas the broad, arcuate joint planes are adversely oriented toward the excavation, which has allowed portions of the rock mass to slide. In general, the upper andesite forms stable slopes. The closely spaced joints made excavation in this unit possible with mechanical equipment. Much of the unit along with the remaining rock in the spillway, however, was blasted to facilitate removal. Because of the uniform nature of the upper andesite, its relatively high bulk specific gravity (2.62 to 2.67 SSD), and the natural size distribution related to the joint spacing, it was selected for use as Type III stone protection on the dam. Petrographic analysis showed the rock to contain from 1- to 5-percent montmorillonite clay, and some breakdown occurred during the specified wetting and drying tests. Nonetheless, the rock was approved for use as Type III stone because it was believed that the amount of breakdown would not affect the gradation enough to reduce the effectiveness of the downstream slope erosion protection. In April 1982, just before the official dam dedication, the rock was observed to be breaking down on the slopes (photo 13). A 10-cycle wetting-and-drying test was conducted on a randomly selected sample of the downstream slope protection (photo 14). The rock was divided during sampling into obviously fractured (breaking down) and apparently unfractured classes. During the test, the unfractured rocks underwent almost no degradation. The fractured rock gradation did change somewhat; however, most of the breakdown occurred during the first cycle, and very little subsequent breakdown occurred. The overall combined gradation change was not significant. Since the wetting and drying test did not seem to produce the expected results (based upon field observation), the expansion and shrinkage of the clay may not be the operative mechanism responsible for the breakdown. Thermal expansion and contraction may play a significant role. Regardless of the cause, the andesite used as Type III stone protection should be monitored for excessive degradation during the life of the project.

3.06 **Flow Breccia.** The bulk of the spillway excavation was in the breccia. It is a strongly fragmented flow that underwent extreme brecciation at the time of emplacement. Pods of coherent lava, which may be old lava tubes, channels, or sills, are seen, but generally the lava is composed of isolated blocks. This is a common chacteristic of some intermediate composition lava flows resulting from flow-induced shear and incipient rupturing that is termed "autobrecciation" (Sheridan, attachment 1). Variability, which is the chief characteristic of this unit, is the reason flow breccia was not used to Type III stone protection, which had specific physical requirements. Locally, the individual rock blocks, which are generally sub-angular and as large as 5 feet in maximum dimension, can comprise from 40 to 80 percent of the rock mass (photo 15). The matrix varies from loose and crumbly to almost crystalline. The colors of the cinders include reddish grey, brown, and red. The maroon and grey rock blocks add more variability to the appearance of the unit. In general, the blocks downstream from station 21 are reddish andesite, while upstream they are dark grey and very similar in appearance to a blocky latite flow exposed at the abutment. Some portions of the flow are characterized by small cavities which are incompletely filled with calcite, zeolites, and hydrous silicates. The secondary mineralization helps to stabilize the otherwise crumbly cinder matrix. Excavation of the flow breccia posed the greatest problems of any of the rock units because of its extreme variability. Most of the unit was comprised of loose rock blocks in a cindery matrix which could be ripped. However, the localized pods of coherent lava, the zones of crystalline matrix, and the larger andesite rock blocks required blasting (photo 16). The slopes excavated in the flow breccia, although quite irregular, appear to be generally stable. Isolated rockfalls are likely and unavoidable over the life of the project and could pose a safety hazard to persons below.

3.07 **TRANSITIONAL ANDESITE/FLOW BRECCIA.** The transitional rock occurs as a zone up to 10 feet wide at the contact between the andesite flows and the flow breccia (photo 17). The rock, which is typically grey to purplish grey with altered phenocrysts of feldspar and blebs of volcanic glass, has a glassy and slightly vesicular texture and classifies as a porphyritic andesite (SPD laboratory analysis) like the upper andesite unit. Exposures are generally discontinuous, massive, and hard. The rock is gradational between the andesite and the flow breccia on either side and is especially difficult to distinguish from more coherent portions of the flow breccia and the more blocky lower andesite unit. Because it is hard and massive, the transitional rock could not be ripped, and it formed "hard rock humps" at the invert and at the toe of slopes which were otherwise mechanically excavated. The pre-split blasting in the transitional rock worked very well (photo 18), although blow-outs in the softer flow breccia below were a problem (photo 19). Crushed transitional rock was used for bedding material and in the gravel drain.

Photo 6
Completed spillway excavation; view looking downstream from station 24 + 50. Note relatively steep side-slopes and varied rock types. Sill shows as a thin light band in center of photo. (5 Apr 1981)

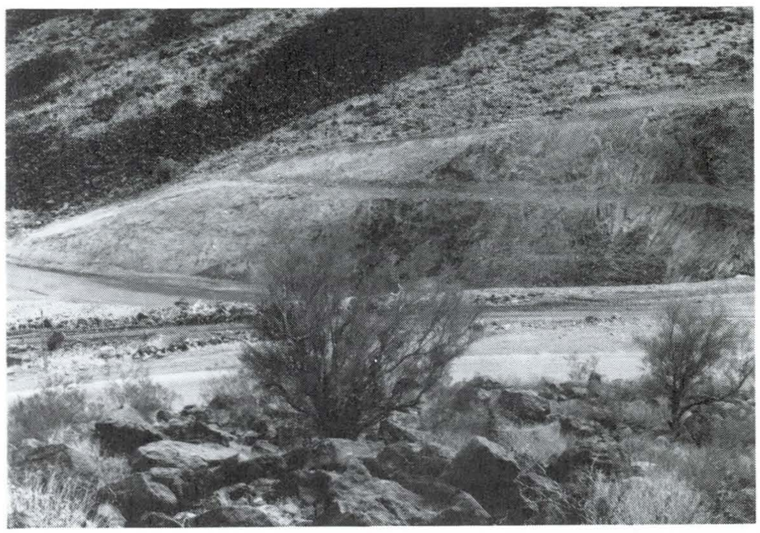


Photo 7
East wall of spillway before excavation below lowermost bench. Note planar contact between agglomerate and flow breccia unit at left (partially obscured by tree). (6 Feb 1981)

Photo 8
Base of agglomerate unit as exposed on the west wall of the spillway excavation. Lowermost bench in center of photo; contact with cindery flow breccia at left. (29 Jul 1981)

Photo 9
D9-L dozer unsuccessfully attempting to rip to invert grade in area of highly cemented agglomerate at station 25 + 00. Note poorly cemented agglomerate slope at left. (24 Aug 1981)

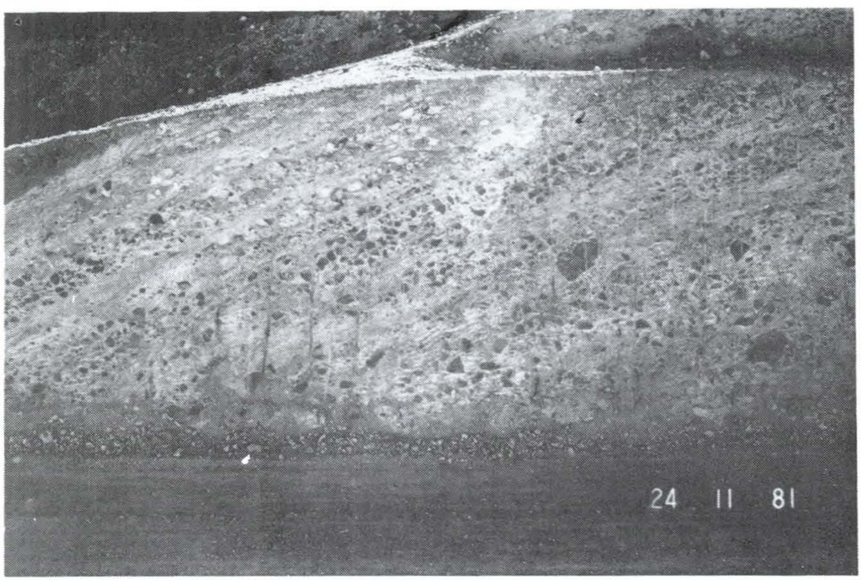


Photo 10
Well-cemented agglomerate as exposed on east side of spillway, elevation 1403-1433, station 24 + 60 – 25 + 40. Crude stratification dips approximately 30 degrees upstream. Note pre-split blast holes. (24 Nov 1981)

Photo 11
Platy jointing developed in upper andesite unit as exposed on west wall, station 23 + 25, upper bench level. (23 Dec 1980)

Photo 12
Swirling joint pattern with adverse dip towards spillway cut as developed in upper andesite unit; east side near station 22 + 50. Fifty percent of 12-foot-wide bench is already missing due to rock falls along joint planes. (6 Feb 1981)

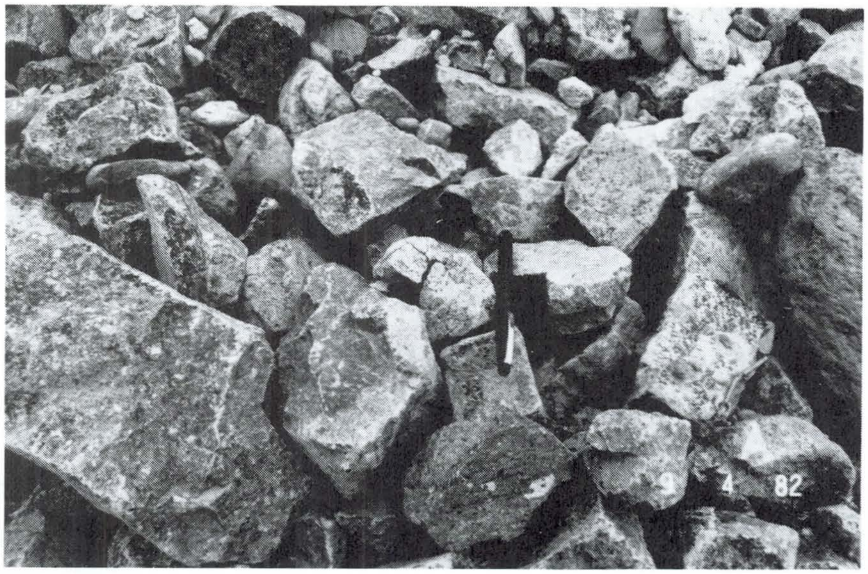


Photo 13
Typical example of Type III stone breakdown on downstream face of dam.
(19 Apr 1982)

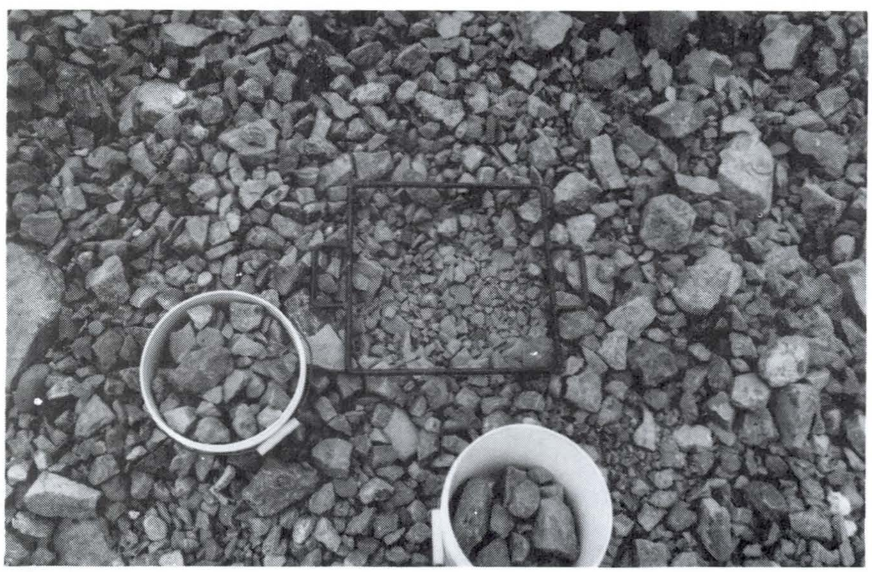


Photo 14
Area, 18 x 18 inch, mid-slope on downstream face of dam near station 60, after sorting into cracked and non-cracked Type III stone. Cracked rock in bucket at right. Note rock at lower left corner of metal template. (6 Apr 1982)

Photo 15
Detail of abundant individual andesite clasts in typical flow breccia. Spillway station 19 + 50, east side, middle slope. (30 Mar 1982)

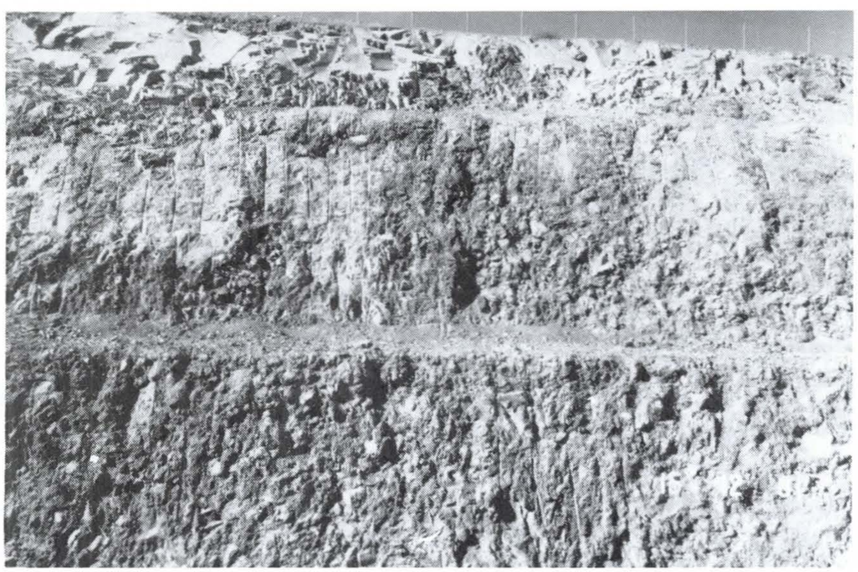


Photo 16
Typical example of variability within the flow breccia. Note irregular contacts between pre-split, coherent masses and more cindery material. Highly jointed upper andesite at top of photo. Spillway station 22 to 23, west side, middle slope.
(15 Dec 1981)

Photo 17
Typical gradational sequence of flow breccia (lower left), transitional, and andesite (upper right). Spillway station 17 + 50, east side, lower slope. (16 Dec 1981)

Photo 18
Re-drilling hard transitional rock protrusion after initial pre-split test blast. Note excellent results of pre-splits on 3-foot centers. (10 Dec 1980)

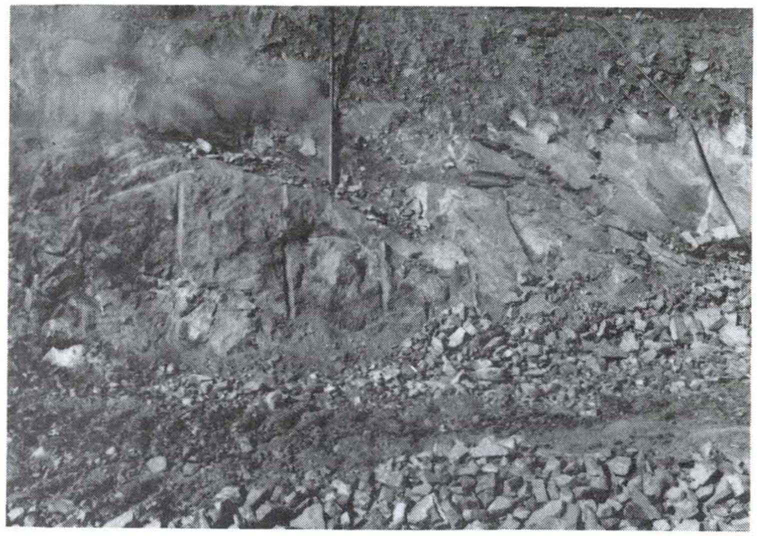


Photo 19
Blow-out in soft, cindery flow breccia below pre-split transitional rock. Spillway station 20 + 45, west side, upper slope. (10 Dec 1980)

3.08 **LOWER ANDESITE.** Between stations 18 and 22, in the center of the excavation, another andesite flow is exposed. It occurs below the flow breccia. In hand specimen, many samples are indistinguishable from the upper andesite while others are very similar to the transitional rock. It is mapped as a separate unit because of its stratigraphic position in the volcanic sequence and because of its characteristically more blocky jointing (photo 20). It can, however, be massive to highly jointed (photo 21). Like the upper andesite, the joint orientations are quite varied and the smooth, semi-arcuate joint planes are sometimes adversely oriented toward the excavation, which causes unstable conditions (photo 22). Excavation of the lower andesite was generally possible by mechanical means; however, it was difficult to maintain slope lines within the tolerances specified because of the erratic joint spacing and orientations.

3.09 **AMYGDALOIDAL ANDESITE.** This rock unit, which is the oldest flow exposed in the excavation, is exposed only at the southwest end of the spillway on the upthrown side of a normal fault at station 16+50 (photo 23 and pl. 1). The rock is maroon/grey, moderate in hardness, with a crystaline groundmass containing weathered iron-rich minerals. Petrographic analysis was not performed on a sample of this rock and it is classified as an andesite by its similarity to the transitional andesite/flow breccia. It is distinguished from the transitional rock by the lack of abundant phenocrysts and by its vuggy texture. The vesicularity ranges from slight to moderate. The size of the vesicles ranges from millimeter to centimeter size and almost all are partially filled with secondary zeolite mineralization. The rock mass is irregularly fractured. Most breaks are at least partially re-healed. Some of the larger spaces between the fractures have been completely filled with a fine grained, cemented, clastic tuff (photo 24). This sandy tuff also overlies the surface of the rock unit. This relationship is particularly well-exposed on the east side of the spillway excavation (photo 23). Excavation of the amygdaloidal andesite was accomplished by mechanical means primarily because of the more uniform nature of the unit which was devoid of localized hard, massive rock. Side slopes were shaped using a slope board. Because of its limited occurrence and its removal late in the construction sequence, the rock was not used in the construction of the dam.

Geologic Structure

3.10 The geology of the spillway consists of a complex lava flow pile which was tilted during the period of Basin and Range Tectonism (Sheridan, attachment 1). Although the physical appearance of the volcanic rocks are quite dissimilar (photo 25) owing to color and textural variation, the basic chemical composition is very similar, and all the rocks exposed represent subaerially erupted flows, probably from the same intermediate composition source (Sheridan, attachment 1). The agmygdaloidal rock exposed by faulting at the south end of the spillway represents the scoriaceous surface of the oldest flow exposed by the excavation. This surface was covered with a thin layer of tuffaceous, probably wind-deposited sands which worked their way down into the fractured rock mass (photos 23 and 24). Subsequent flows of intermediate andesitic composition produced the upper and lower andesite and flow breccia units which comprise the bulk of the spillway excavation. The flow breccia is in gradational contact with the irregular surface of the lower andesite and, in turn, is mostly overlain by the upper andesite flow, which shows strong evidence of flow jointing. The intimate association of both andesite flows with the breccia, the transitional zones between the units, and the similarity in chemical composition suggest that the flows may have been contemporaneous with the more coherent andesite representing relatively fast-moving lava channels in an otherwise more visous brecciated flow margin. Locally, the andesite flowed in sill-like channels within the breccia and in other places was deposited on its surface. The more oxidized nature of the contact between the flow breccia and lower andesite might indicate a weathered surface on an older flow.

3.11 The thick accumulation of cemented colluvium at the north end of the spillway may represent a modified fault scarp along a major upthrust fault block against which the colluvium was deposited and cemented. The exposed faulting between stations 16 and 16+50 (photo 26), although of a much smaller magnitude, is probably related to the major fault block movements. Because of the varied topography which exists in the Hedgepeth Hills today, the actual number, orientation, and amount of separation of faults in the area remain unknown. The steeply dipping normal fault on the west side of the spillway, as shown on the contract plans (amended pl. 4), was not encountered during excavation. It is interesting to note that the texture and appearance of the rock in the grey-black flow breccia exposed at the extreme upstream end of the spillway (photo 27) very closely resembles the shattered "latite" block flow exposed in the upper portion of the abutment excavation. Even though they occur at approximately the same elevation, direct correlation is tenuous and the infilled blocky flow, characteristic of the right abutment foundation, is not encountered anywhere in the spillway.

EXCAVATION

3.12 The excavation of the spillway at Adobe Dam required the removal of almost 1/4-million cubic yards of material. Of primary concern to the contractor was accomplishing the excavation at a minimum cost while producing materials useful in the construction of the dam at the same time. Of primary concern to the Government was the creation of an end-product excavation as close as possible to the lines and grades shown on the contract plans with the resultant rock slopes in the best possible condition. Since these two viewpoints might not be compatible, the specifications for spillway excavation were written to yield the best end-product. As a part of those specifications, buffer zones were created adjacent to final slope lines and along a portion of the spillway invert. In these buffer zones, controlled blasting or mechanical excavation methods were required. Smooth blasting was specified in Section 2C, para. 17.3 of the contract specifications (attachment 2); however, alternate <u>approved</u> methods were also acceptable. The purpose of the buffer zones was to protect and optimize the end-product excavation on: (1) the lower slope on both sides and for the entire length of the spillway because this portion will channelize the PMF flows, (2) the spillway invert 150 feet upstream and 200 feet downstream from the sill because this section is critical in maintaining an effective spillway crest elevation of 1377.8, and (3) all slopes on the west side in order to protect to the maximum extent possible the slopes nearest to a 42-inch water pipeline from a City of Phoenix reservoir just west of the spillway (amended pls. 18, 19, and 20). The upper slopes on the east side and the remaining invert, although not designated with buffer zones, were still covered under portions of the specifications which required the contractor to use methods which would prevent damage to rock outside the prescribed limits of excavation.

Test Blast

3.13 In order to conform to the contract requirements while satisfying his own requirements for cost effective production, the contractor used several methods of excavation at different times depending on (1) the nature of the rock to be removed, (2) the end-product use of the excavated material, (3) the availability of equipment, and (4) the logistics. The initial excavation in the spillway consisted of a test blast on 18 November 1980. Blast no. 1 was conducted on the east side of centerline between stations 23 and 24. Pre-split holes were drilled on 3-foot centers along a line 8 feet from the final excavation line. A 6x8-foot spacing of 3-inch ANFO loaded holes yielded a powder factor of 1.1, #/yd^3. Hercomix no. 4 was detonated with 2x8-inch Unigel primers using Millidet electric caps. Hercosplit (7/8x24-inch sticks) was used in the pre-split holes with 4 feet of stemming. Except for a change to Herco-mix no. 1 in January 1981, the same combination of explosives was used for all blasting in the spillway and at the abutment. The results of the test blast were not conclusive, especially for the pre-split demonstration, primarily because the rock blasted was mostly agglomerate, which is not representative for the bulk of the spillway excavation. Subsequent test or demonstration blasts were likewise inconclusive because of the lateral and vertical variability of the different rock units which had different excavation characteristics. In the final analysis, the contractor's blasting methodology could only be judged in relation to the end-product slopes over a large area which included different rock types.

Pre-Split Demonstration

3.14 After the test blast, the contractor elected to excavate by ripping and scraping. He found after only a few days that, although much of the material could be ripped, there were resistant outcrops which could not be mechanically removed. Once exposed, they were difficult to treat individually. Two such "hard rock humps" were encountered at the west slope along the final grade line. The contractor proposed using these two localities (sta. 20+00 to 20+50 and 21+50 to 22+00) to demonstrate his pre-split (controlled) blasting technique. In lieu of the smooth blasting method as described in Section 2C, para. 17.3 (attachment 2), he proposed drilling pre-split holes on 36-inch centers, loading each with pre-split stick dynamite as in the original test blast. Since the Government felt that a 3-foot spacing might be excessive, the contractor was directed to drill a portion of one area (sta. 20+00 to 20+35) on 2-1/2-foot centers and load only alternate holes. The pre-split demonstration blast occurred on 9 December 1980. The hard rock exposed at the surface did split nicely (photo 18). This was, however, in the stemmed portion of the holes; and it was agreed that, onlv after the entire shot was exposed, could the results be evaluated. Therefore, the contractor, using a 9x9-foot pattern, was allowed to blast the burden in front of the test section from station 21+25 to 22+00. ANFO-loaded toe breaker holes were allowed for this demonstration, even though contrary to the specifications (para. 17.3.4, attachment 2). Mid-slope breakers were not allowed. The results of the entirely exposed pre-split demonstration indicated that, where the rock was hard and massive (rock needing to be pre-split), the controlled blasting on 3-foot centers worked very well. The highly fractured ande-site and crumbly flow breccia, however, did not require pre-splitting, and the blasting effects of pre-splitting and ANFO breaker holes near the slope might have unnecessarily harmed the rock face.

Approved Blasting Plan

3.15 On 17 December, a compomise plasting plan was agreed upon by the contractor and the Government. (See fig. 3 for the details of the plan.) Basically, the plan called for exploratory holes to be drilled on 9-foot centers along the final slope lines with pre-split holes drilled on 3-foot centers where competent rock was encountered. In zones of loose, crumbly flow breccia, holes would be deck-loaded based on driller's reports. Vertical ANFO-loaded breaker holes would be allowed mid-slope only in areas previously split. Toe breakers would be drilled with a 6-inch sub-drill and would be shot regardless of the nature of the slope rock. Although this blasting plan was contrary to several items in the specifications, it was agreed to as a reasonable and constructible compromise considering the variability of the excavation characteristics of the spillway rocks.

Photo 20
Sinuous to blocky jointing developed in the lower andesite. Note gradational contact with flow breccia above. Spillway station 21 + 30, east side, lower slope. (30 Mar 1982)

Photo 21
Irregular character and spacing of joints in lower andesite; blocky to massive at left, highly jointed at right. Spillway station 21 + 25, east side, lower slope. (30 Mar 1982)

Photo 22
West sill slope after scaling off loose rock. Note unstable dip-slope along jointing in lower andesite at right. Flow breccia exposed above bench. (15 Dec 1981)

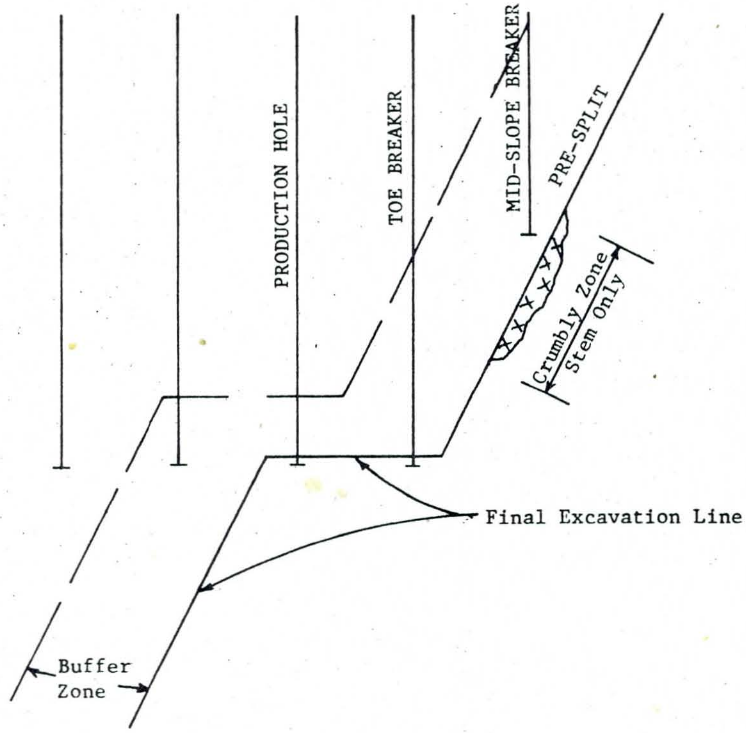
Photo 23
Normal fault at spillway station 16 + 50, east side, trends N50-60W, dips 60 degrees NE. Upper andesite at left; upper andesite/clastic tuff/amygdaloidal andesite at right. (6 Apr 1982)

Photo 24
Typical example of amygdaloidal andesite rockmass. Note irregular fracturing and apparent complete infilling by tuffaceous sand from above. (6 Apr 1982)

Photo 25
View of east wall of spillway excavation from station 20 + 00 looking upstream. Note varied texture and color of different rock units. (6 Apr 1982)

Photo 26
Fault wedge at spillway station 16 + 00, east side. Amygdaloidal andesite at right; upper andesite/tuffaceous sand/amygd. andesite sequence at left. (6 Apr 1982)

Photo 27
Scoriaceous to cindery "latite" flow breccia as exposed at upstream end of spillway, station 24 + 25, east wall, elev. 1383. (6 Apr 1982)



PRODUCTION DRILL AND SHOOT

1. Drill 8x8 to 10x10 pattern; depth will vary from 12 to 30 feet
2. Shots will be delayed in the following order with millisecond delays;
   a. pre-split holes
   b. production holes from centerline outward
   c. toe breaker holes
   d. mid-slope breaker holes
3. Complete shot information to be indicated on pre-drill reports
4. Post blast reports to be submitted after each blast

PRE-SPLIT HOLES

1. Exploratory holes drilled along the final excavation line on 9-foot centers
2. Pre-split holes drilled on 3-foot centers in competent material
3. Pre-split holes will be loaded/stemmed according to driller's reports

MID-SLOPE BREAKER HOLES

1. Drilled on 8x6 pattern to within 2 feet above final excavation line
2. Loaded with ANFO and shot only in areas previously pre-split

TOE BREAKER HOLES

1. Drilled to 2 feet beyond toe of the slope, 6 inches below bench
2. Loaded with ANFO and shot before the mid-slope breakers

Figure 3. Approved Spillway Blasting Plan

Production Blasting

3.16 From November 1980 to October 1981, almost 50 separate blasts were conducted in the spillway (photo 28). The contractor was required to submit a pre-blast report showing the area to be drilled and the pattern of the shop; and after each blast, both the contractor and the Government inspector filed post-blast reports. The pertinent data from these reports are tabulated in the Spillway Blasting Summary, table 1. According to this summary, 164,000 pounds of either Hercomix #1 or Hercomix #4 were used to blast 185,000 cubic yards of rock. This is an average powder factor (PF) of 0.9. Originally, the contractor used a 10x10-foot pattern with 6 feet of stemming in 3-1/2-inch holes and a powder factor did not give adequate fragmentation. An increased powder factor was required to break-up the rock because the loose, crumbly ash in the flow breccia and the well developed joints in the andesite provided access for premature dispersion of the gaseous energy. In order to increase the powder factor, the spacing was reduced to 8x9 and subsequently to 7x9 (photo 29) for a PF of 1.2 pounds per cubic yard. Approximately half of the blasting was done using the 10x10-foot pattern and half using the 7x9-foot pattern. The general configuration of a typical blast is shown in the approved blasting plan, figure 3. During the progress of the work, certain deviations from the approved blasting plan occurred: (1) pattern depth generally varied between 12 and 24 feet with occasional shallow shots to remove isolated hard rock humps; (2) production blast holes in excess of 24 feet were not drilled; (3) mid-slope breaker holes were drilled and shot only when the pattern depth exceeded 12 feet; (4) breaker holes were drilled on the same pattern spacing as the remaining production holes, and (5) toe breakers were drilled 1 foot from the slope toe and mid-slope breakers 1 foot above the final excavation line. Up to 15-millisecond delays were used for each blast in the order indicated on the Approved Blasting Plan. Since the center production holes were shot first (after pre-splits), the muck was heaved to the middle of the excavation. This allowed access for drilling behind (downstream from) previously blasted areas (photo 30). By using the delay blasting method, the maximum amount of explosive detonated at one time was also reduced. Some of the earliest blasts, however, used as much as 589 pounds per delay.

Vibration Monitoring

3.17 On the west side of the spillway excavation, the City of Phoenix had recently constructed a 42-inch water pipeline (amended pls. 17, 18, 19, and 20). To insure that the blasting would not damage the pipe, the contractor arranged for John McCormick of Evans, Goffman and McCormick to monitor the ground motion particle velocity of a typical production blast (#16A on 17 Jan 1981). The powder factor was 1.0, $\#/yd^3$. Based upon an assumed safe velocity of 6 inches/sec at the pipe, a maximum of 300 pounds per delay was recommended for all subsequent blasts.

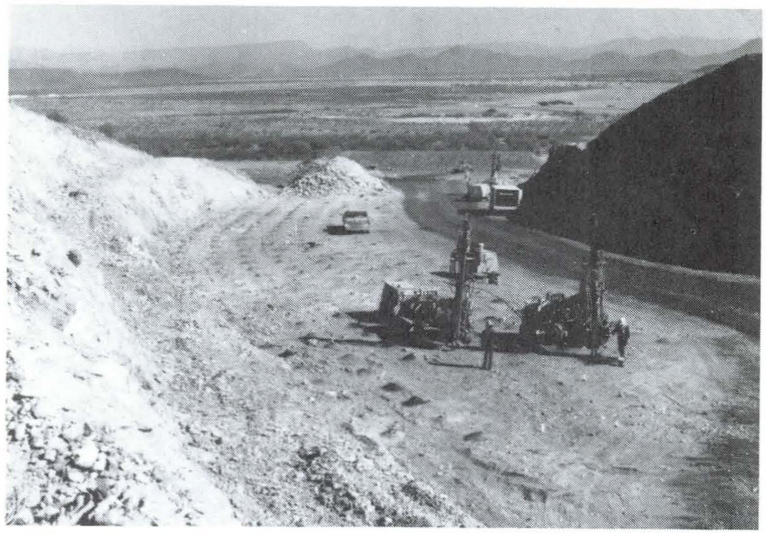
Pre-Split Blasting

3.18 Excavation to a neat line on the spillway slopes was difficult because of the variability of the rock units and the differences in rock competence within the flow breccia unit itself. It was obvious that some blasting was necessary to break hard rock masses. The location of these areas was difficult to anticipate because of their unpredictable distribution and any unnecessary blasting along the slopes might adversely affect the more crumbly or highly jointed rock. The approved blasting plan, therefore, called for drilling exploratory holes on 9-foot centers along the final slope line and 3-inch pre-split holes on 3-foot centers in areas of hard drilling. Each hole was logged in 1-foot intervals as hard, soft, or spotted (alternately hard and soft). The holes were then loaded, deck-loaded, or not loaded at all based on these reports. This procedure was initiated with blast 10A on 19 December 1980 and continued until 16 January 1981. The procedure was modified, beginning with blast 16A, because it was discovered that some areas which had been deck-loaded had to be re-drilled and shot. After 16 January, the following procedure for pre-split shooting was utilized:

- Pre-split holes were drilled every 3 feet along the final excavation line without any exploratory-type holes.
- All holes were logged as before (for record purposes) but the logs were not used in loading (variations in depths measured, the subjective nature of the logging, and the great number of holes drilled made the logs essentially useless in loading).
- All holes were loaded except those which caved in to such a shallow depth (about 8 feet) that loading and stemming was not practical.

Photo 28
Typical production blast; no 10B, spillway station 22 + 50-23 + 40. Pre-splits detonated on both sides with first production delay in center. (22 Dec 1980)

Photo 29
Drilling production holes on 7x9-foot pattern for blast no. 18; spillway station 21 + 00. (22 Jan 1981)



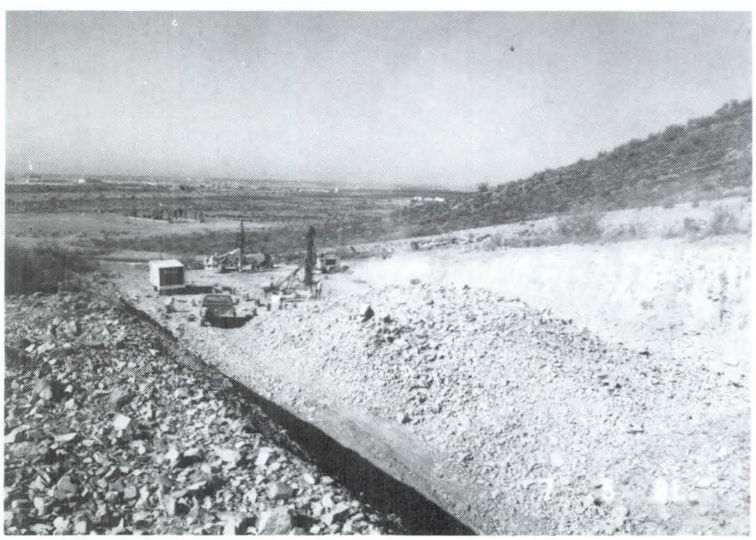


Photo 30
Drilling and loading production holes for blast no. 23P at downstream end of spillway. Note how muck from blast no. 230 was heaved toward center of the excavation. (7 May 1981)

Table 1. Spillway blasting summary

Blast No.	Date Blasted	Stationing	Top Elevation	Depth Drilled	Bottom Elevation	Production Hole Dia.	Pattern	Stemming	Pounds ANFO	Number of Delays	Maximum Pounds/delay	Cubic Yards Rock in Place	Powder Factor	Production Holes	Pre-splits east	Pre-splits west	Remarks
1	18Nov80	23+00 - 24+00	1454 1439	24 6	1430 1433	3"	8x6	7 4	2570	7	430	2340	1.1	99	32	0	Test blast; east side of centerline only
2	26Nov80	21+24 - 22+20	1465	24	1441	3½"	9x9	8	4200	?	?	6000?	0.7	?	0	0	
3	3Dec80	21+00 - 21+27	1458	12	1446	3½"	9x9	6	750	11	70	830	0.9	43	0	0	
4	5Dec80	20+00 - 21+00	1450 1448	12 9	1438 1439	3½"	9x9	6 5	3625	15	242	5310	0.7	177	0	0	
5	8Dec80	22+70 - 23+50	?	12	?	3½"	10x10	6	725	5	145	1678	0.4	42	0	0	
6	9Dec80	21+50 - 22+00 20+00 - 20+50	1450 1454	12 12	1439 1443									0 0	0 0	17 17	Pre-split demonstration; holes on 2½ and 3' centers
7	10Dec80	21+67 - 21+76	1450	15	1436									0	0	4	Continuation of pre-split demonstration
8	11Dec80	20+50 - 22+50	1447	11	1436	3½"	10x10	6	6000	15	400	9972	0.6	244			19 pre-splits, location unknown; misfire on shot
9	12Dec80	18+70 - 21+10	1447	10	1437	3½"	10x10	5	1700	6	284	4144	0.4	112	0	19	
10A	19Dec80	23+50 - 24+50	1427 1434	19 24	1408 1410	3½"	10x10	8	5175	14	510	8700	0.6	119	12	27	First blast after estab. authorized blast plan and reporting procedure
10B	22Dec80	22+50 - 23+40	1428	24	1404	3½"	10x10	6	6607	15	589	9768	0.6	104	21	34	
12A	2Jan81	19+25 - 21+00	1451 1445	10 8	1441 1437	3½"	8c.c.	6	600	6	100	1005	0.6	40	25	20	Breaker and pre-split holes to remove high spots on first bench
12B	6Jan80	21+50 - 22+50	1441	24	1417	3½"	10x10	6	5750	15	502	9605	0.6	118	7	30	2-3/4" pre-split holes; poor breakage, blew out bottom

Table 1. Spillway blasting summary

Blast No.	Date Blasted	Stationing	Top Elevation	Depth Drilled	Bottom Elevation	Production Hole Dia.	Pattern	Stemming	Pounds ANFO	Number of Delays	Maximum Pounds/delay	Cubic Yards Rock in Place	Powder Factor	Production Holes	Pre-splits east	Pre-splits west	Remarks
12C	7Jan81	21+00 - 21+50	1439	24	1415	3½"	10x10	6	3450	15	605	5683	0.6	64	11	14	
12D	8Jan81	22+10 - 22+60	1432	15	1417	3½"	10x10	5	835	4	210	1166	0.6	21	0	0	Re-shoot of 12B
12E	9Jan81	20+10 - 20+80	1439	24	1415	3½"	10x10	6	4215	15	305	5960	0.6	67	11	15	
15A	13Jan81	19+50 - 20+20	1438	24	1414	3½"	10x10	6	5765	15	440	8710	0.6	107	24	17	Tried ripping this area with D9G double tooth ripper
15B	15Jan81	19+00 - 19+45	1440	24	1416	3½"	8x9	6	3280	15	550	3642	0.9	62	11	13	Spacing tightened to increase fragmentation
16A	16Jan81	18+40 - 18+90	1438 1433	24 20	1414 1413	3½"	8x9	5	4700	15	314	4800	1.0	80	14	18	Blast monitored by John McCormick; "new" blasting procedure initiated
16B	19Jan81	17+37 - 18+28	1432 1422	16 7	1416 1415	3½"	8x9	6 1½	2906	15	210	3601	0.8	136	24	27	No mid-slope breaker holes
17A	21Jan81	24+29 - 25+10	1415	12	1403	3½"	7x9	5	2654	10	372	2905	0.9	85	0	20	
17B	23Jan81	23+30 - 24+10	1415	13	1402	3½"	7x9	5	2464	9	320	2360	1.1	77	0	30	Minimal fly-rock; uniform fragmentation
17C	26Jan81	24+00 - 25+00	1416 1413	10 10	1406 1403	3½"	7x9	5	1892	9	256	1803	1.0	56	31	0	Pre-splits sub-drilled 1 to 3' and stemmed at bottom
17D	27Jan81	22+73 - 23+91	1417	14	1403	3½"	7x9	5	3252	15	288	3252	1.0	102	26	10	Pre-splits sub-drilled 2 to 6 feet and stemmed at bottom
18A	29Jan81	22+00 - 22+81	1418	15	1403	3½"	7x9	5	4500	15	330	4520	1.0	131	14	26	ANFO changed from Hercomix No.4 to Hercomix No.1

Sheet 2 of 4

Table 1. Spillway blasting summary

Blast No.	Date Blasted	Stationing	Top Elevation	Depth Drilled	Bottom Elevation	Production Hole Dia.	Pattern	Stemming	Pounds ANFO	Number of Delays	Maximum Pounds/delay	Cubic Yards Rock in Place	Powder Factor	Production Holes	Pre-splits east	Pre-splits west	Remarks
18B	29Jan81	21+15 - 21+90	1420	16	1404	3½"	7x9	5	4938	15	342	4637	1.1	126	23	22	
18C	30Jan81	20+25 - 21+00	1420 1418	18 18	1402 1400	3½"	7x9	5	5412	15	396	5092	1.1	123	24	27	
18D	5Feb81	19+55 - 20+15	1425	18	1407	3½"	7x9	5	5435	15	315	4181	1.3	101	21	20	
18E	11Feb81	19+05 - 19+60	1425 1423	20 20	1405 1403	3½"	7x9	5	5350	15	416	4600	1.2	100	30	16	Eschelon delay pattern
18F	17Feb81	18+45 - 19+00	1424 1420	16 16	1408 1404	3½"	7x9	5	3658	15	266	3469	1.1	91	14	20	
18G	18Feb81	17+80 - 18+30	1421 1418	17 14	1404 1404	3½"	7x9	5	3030	15	224	2898	1.1	90	17	20	
21A	20Feb81	25+00 - 25+50	1402	24	1378	3½"	7x9	6	3410	15	275	3138	1.1	62	18	18	Pre-split holes drilled to elevation 1370
21B	25Feb81	24+50 - 24+95	1405	22	1383	3½"	7x9	5	3186	15	?	2655	1.2	54	17	16	Pre-split holes drilled to elevation 1370
21C	26Feb81	24+00 - 24+50	1409	24	1385	3½"	7x9	5	2772	15	198	2318	1.2	42	13	15	
21D	27Feb81	23+50 - 24+00	1409	24	1385	3½"	7x9	5	3050	15	264	2650	1.2	48	16	16	
21E	4Mar81	22+70 - 23+40	1404	24	1380	3½"	7x9	5	4488	15	330	3740	1.2	68	24	27	
21F	5Mar81	22+39 - 22+70	1404	24	1382									0	15	15	Pre-splits only
21G	23Mar81	22+00 - 22+70	1406 1403	24 21	1302 1302	3½"	7x9	5	4500	15	354	4131	1.1	81	7	7	

Sheet 3 of 4

Table 1. Spillway blasting summary

Blast No.	Date Blasted	Stationing	Top Elevation	Depth Drilled	Bottom Elevation	Production Hole Dia.	Pattern	Stemming	Pounds ANFO	Number of Delays	Maximum Pounds/delay	Cubic Yards Rock in Place	Powder Factor	Production Holes	Pre-splits east	Pre-splits west	Remarks
21H	25Mar81	21+50 - 22+00	1406 1404	24 24	1382 1380	3½"	7x9	5	3186	15	236	2754	1.2	54	15	17	
21I	27Mar81	20+90 - 21+40	1404	24	1380	3½"	7x9	5	3186	15	236	2754	1.2	54	13	15	
21J	31Mar81	20+50 - 20+85	1404	24	1380	3½"	7x9	5	2655	15	?	2213	1.2	45			23 total pre-split holes
21K	?	20+00 - 20+50	1405	24	1381	3½"	7x9	5	3186	15	236	2754	1.2	54	15	14	
21L	27Apr81	19+25 - 20+00	1408	22	1386	3½"	7x9	5	3835	?	?	3195	1.2	54	20	20	Deck loaded pre-split holes as necessary
23M	30Apr81	18+60 - 19+15	1406	22	1384	3½"	7x9	5	3658	?	?	3048	1.2	62	14	19	Deck loaded pre-split holes as necessary
23N	5May81	17+85 - 18+50	1408	22	1386	3½"	7x9	5	3717	?	?	3098	1.2	63	14	19	
23O	6May81	17+25 - 17+80	1406	22	1384	3½"	7x9	6	3207	?	?	2673	1.2	63	24	20	
23P	7May81	16+45 - 17+10	1406	22	1384	3½"	7x9	6	3776	?	?	3147	1.2	64	19	20	
30	3Sep81	24+50 - 25+90	1382 1376	9 3	1373 1373	3"	5x7	4	1275	15	85	1397	0.9	150	0	0	Irregular surface left by rippers/scrapers; 9" sub-drill
31	18Sep81	20+25 - 22+20	1384 1376	11 3	1373 1373	3"	5X7	4	1850	?	?	2313	0.8	211	56	56	Irregular surface left by rippers/scrapers; 9" sub-drill
32	19Sep81	18+25 - 20+15	?	12 4	?	3"	5x7	4	2800	?	?	3415	0.8	211	52	52	34 holes drilled in sill, 4'c.c. burden, 3'c.c. pre-splits
33	16Oct81	16+00 - 17+70	?	?	?	3"	5x5	?	1050	6	150	1272	0.8	210	?	?	Depth of holes varied, drilled to invert +9"sub-drill

3.19 In the final analysis, the modified pre-split technique produced acceptable results considering the geologic conditions. The 3-foot spacing with 7/8-inch stick powder in 3-inch-diameter holes worked well to pre-split the hard rock and the blasting effects did not noticeably degrade the less competent rock (photo 31). Over 1500 line (pre-split) holes were required in the spillway excavation, mostly on the middle and lower slopes.

Mechanical Excavation

3.20 The blasting methods previously discussed were used until mid-May 1981. At that time only the final 15 to 20 feet remained to be excavated (photo 32) and work was discontinued. In August, work was resumed and, after removing 10 feet of already shot rock with scrapers (photo 33), the contractor tried ripping to the final invert grade using a D9G with single-tooth ripper even though this method had previously proved unsuccessful. The results were much the same when ripping was first attempted in November 1980. About 3 feet of rock was loosened; however, localized hard rock humps prevented further excavation (photo 34). On 20 August 1981, a D9L dozer with double-tooth ripper began working (photo 35), and significantly more rock was loosened. Even after a maximum effort by the bigger and more powerful dozer, however, there still remained high spots which required shallow blasting (photo 36) using tight drilling patterns (blasts number 30 through 33). Mechanical equipment was also used to excavate some slopes in lieu of pre-splitting. Most of the east side upper slope was excavated to grade using a slope board on a D9G dozer (photo 37) as were the slopes on both sides downstream from station 17. Where the rock condition allowed this method of excavation, the results were excellent.

Rock Hauling

3.21 The muck generated by either ripping or blasting was removed mainly from the spillway cut using caterpillar 651B scrapers (photo 33) or rock trucks and a caterpillar 988B front-end loader (photo 38). The type of method used was dependent on (1) availability of equipment, (2) steepness of the exposed face, (3) size of the rock to be removed, and (4) width of the cut. The scrapers were more efficient in removing the material; but required free access in both directions, a relatively level bottom, and smaller maximum rock size. The rock trucks fed by a front-end loader allowed removal of muck at a steep face directly adjacent to on-going drilling operations (photo 38). The excavated spillway rock was stockpiled rock at the upstream end of the excavation and later processed by D.C. Speer Construction Company (photo 39), making either Type III stone, bedding material, or gravel drain. On 19 October 1981, the last load of rock was removed, and the spillway was down to grade. Only the sill excavation and construction remained.

SPILLWAY SILL

3.22 The spillway for Adobe Dam is unlined except for a narrow concrete sill section at station 20+00. The purpose of the sill is to protect the integrity of the spillway crest invert elevation, 1377.8, and to provide a controlled channel section for the PMF. Under normal conditions, the spillway at Adobe Dam will only pass flows in excess of the standard project flood (SPF). As originally designed, the sill was to be trapezoidal in cross-section, extending across the full width of the spillway floor and up both side slopes to elevation 1393.5 (PMF flow limit).

Photo 31
Typical example of pre-split blasting results. Note casts of holes in hard and massive zones within both upper andesite and flow breccia. Remainder of slope apparently not adversely affected. Spillway station 20 to 21, east side, middle slope. (15 Apr 1981)

Photo 32
Spillway excavation as of 20 May 1981. Bottom elev. 1395, lower slope not yet dressed with slope board. View of east wall looking upstream from approx. sta. 17 + 00. (20 May 1981)

Photo 33
D9-H push cats and 651B scraper mucking-out spillway in area of flow breccia. (21 Aug 1981)

Photo 34
Hard rock hump left in floor of spillway after ripping. Area eventually shot to grade during blast no. 32. Spillway station 19 + 50, east side, 5 feet above final invert. (20 Aug 1981)

Photo 35
D9-L outfitted with double-tooth ripper excavating to near final invert grade near spillway station 22. (21 Aug 1981)

Photo 36
Drilling shallow production holes for blast no. 31 to remove isolated high spots. (2 Sept 1981)

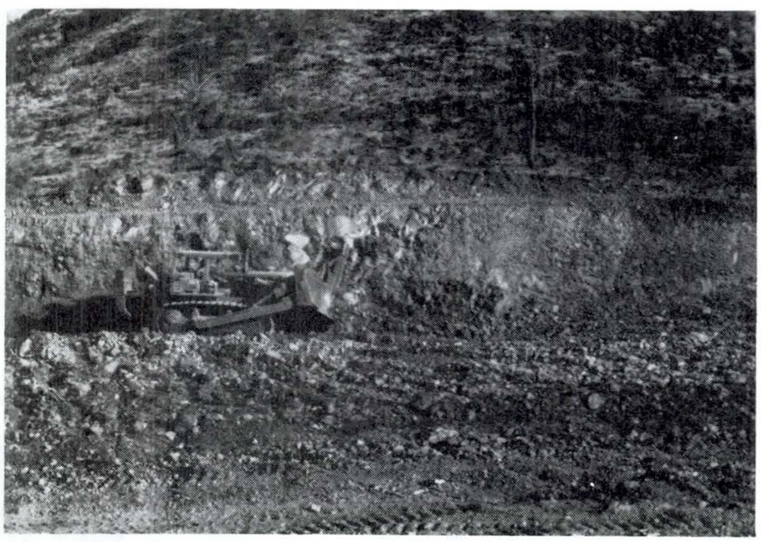


Photo 37
Dozer with slope board excavating to final slope grade. Spillway station 21 + 50, east side, upper slope. (8 Dec 1980)

Photo 38
D9-H dozer pushing muck to Caterpillar 988C loader and rock truck during spillway excavation at station 20 + 50. Flow breccia being loaded was hauled to waste upstream from the spillway. Note close proximity of drilling operation at upper right. (6 Feb 1981)

Photo 39
D.C. Speer Construction Company processing rock from spillway excavation for use as Type III and bedding material. Plant setup is in the reservoir area midway between the right abutment and the spillway (19 Mar 1981)

The maximum width was to be 4-1/4 feet, tapering to 2 feet at the base of the excavations. The original design showed B-line tolerance of 6 to 9 inches. Because of the nature of the rock at station 20+00 and the contractor's excavation methods, the as-constructed sill deviated significantly from this design (amended pl. 19).

Side Wall Excavation

3.23 The rock at station 20+00 in the area of the sill was the lower andesite unit along with flow breccia on the east side (photo 40). The irregular, blocky jointing in the andesite and the large rock blocks in the flow breccia made excavation to the neat lines shown on the contract plans very difficult, especially on the side slopes. The sill across the bottom was blasted on 19 September 1981 using pre-splits on 3-foot centers and a single line of burden holes on 4-foot centers. Blasting on the side slopes, however, was not allowed because of the condition of the rock mass. The contractor took it upon himself to attempt the side-wall excavation using a CAT 235 backhoe even though he had been previously advised to consider other methods (ie., jackhammer, hand-barring, and wedging, etc.) to control the shape of the slot. The results of the attempted backhoe excavation were disappointing. The irregular depth and width of the cuts was controlled by adversely oriented joints in the andesite and the large rock blocks in the flow breccia (photos 41 and 42). Open, partially mineralized voids several inches wide and several feet deep were common on the east slope. The excavation on the west side was between 6 and 8 feet wide (instead of 4-1/2 feet as shown on the contract plans), and it exposed a smooth, arcuate, dip-slope joint surface and unstable rock blocks above (photo 42).

3.24 After this first attempt at excavation, the Resident Engineer proposed to delete or change the design of the side-wall sills. Because the proposal to delete the sills was unacceptable to the hydraulic designers, it was recommended that the slopes first be scaled to remove unsafe rock blocks, then use hand or very controlled mechanical methods to excavate back to distinct joint surfaces or rock masses and construct a notch 2-feet wide and 1-foot deep, or as deep as necessary, to extend 4 feet into the theoretical slope plane. These notches would be used as a key to secure a modified rebar cage into the slopes. On 15 and 16 December 1981, two laborers on a man-lift scaled the unsafe slopes above the rough sill excavations (photo 43) and used pry bars and shovels to remove the loose rock from the face of the cuts. On the west side, additional prominent joint planes were exposed, many of which had adverse orientations toward the spillway cut (photo 44). Rock blocks could be removed with difficulty by hand or easily using pry bars. The blocks, however, were wedged together (photo 45) so that removal of one made many others unstable. To keep the overall size of the excavation to a minimum, many "loose" blocks were left in place. On the east side, the apparently loose blocks in the flow breccia above elevation 1393 were actually quite stable and very few were removed using the pry bar. The lower portion of the sill excavation was in the blocky andesite unit. Smooth, arcuate, dip-slope joint planes were exposed (photo 46), and the excavation near the toe of the slope ended up to be approximately 10 feet wide. A jackhammer was used in an attempt to trim rock blocks in the flow breccia. The hard, massive, reddish andesite, boulder-size blocks, however, were too tough to excavate in this manner (photo 47).

Photo 40
Condition of east slope at station 20 + 00 before side-wall sill excavation. Note heterogeneous and blocky nature of the rock. (20 Aug 1981)

Photo 41
Condition of east slope at station 20 + 00 after initial excavation attempt with CAT 235 backhoe. (21 Aug 1981)

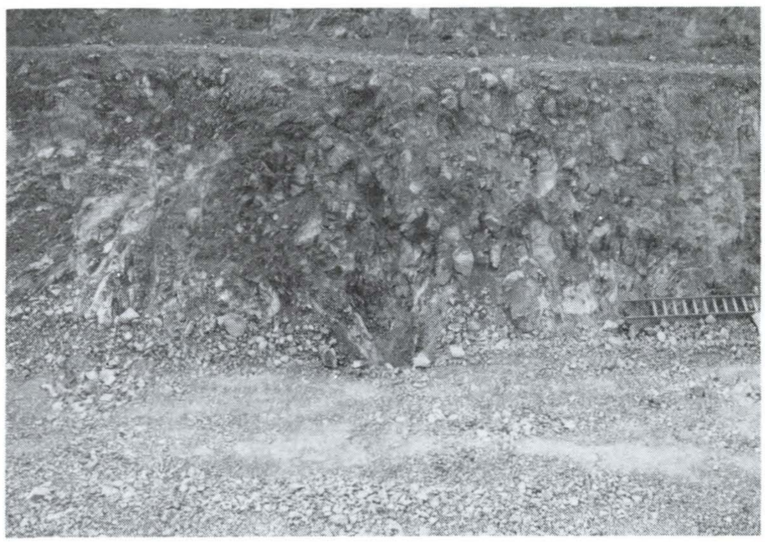


Photo 42
Condition of west slope at station 20 + 00 after initial excavation attempt with CAT 235 backhoe. Note dip-slope joint surfaces and unstable rock above cut. (21 Aug 1981)

Photo 43
Laborers using shovels and pry bars to dislodge loose and unsafe rocks from above the initial excavation for the side-wall sill on the east spillway slope. (16 Dec 1981)

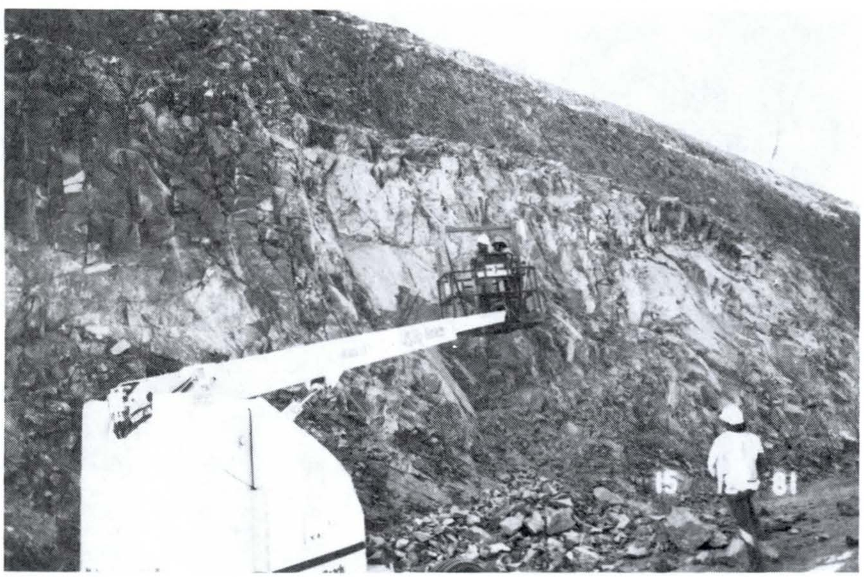


Photo 44
West slope at sill (station 20 + 00) after scaling of unstable rock from above initial cut into slope. Note distinct, smooth, joint planes dipping toward excavation. (16 Dec 1981)

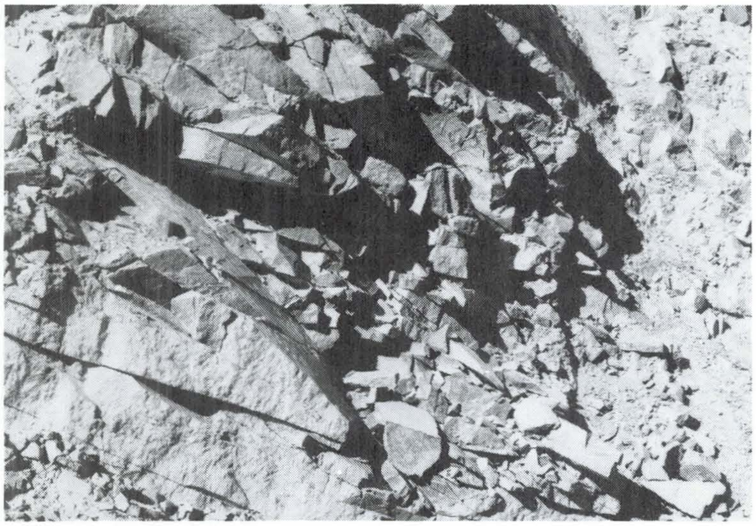


Photo 45
Wedge-shaped intersecting joint blocks developed in lower andesite unit as exposed on west wall, adjacent to excavation for side-wall sill. (25 Nov 1981)

Photo 46
Dip-slope joint planes developed in lower andesite unit as exposed in excavation for side-wall sill in east slope. Attitudes: N70E 45N; N55E 70N; N60E 45N; N60E 65N. (13 Dec 1981)

Photo 47
Evidence of attempts to jack-hammer protruding rock on east side-wall sill slope. Note cavity and open fracture plane. (16 Dec 1981)

The use of the jackhammer was discontinued because of the poor results and the marginally safe working conditions. It was decided to abandon the attempt to create a narrow notch 4 feet deep into slope. Instead, the contractor was instructed to remove, by hand, the relatively loose rocks along the sill centerline and place the pre-formed rebar cage (as shown on the original contract plans, amended pl. 19) as far as possible into the slope while maintaining close to the original 1/2 on 1 slope.

Sill Trench Excavation

3.25 As soon as the slopes had been excavated, a backhoe was used to dig out the previously blasted sill trench across the bottom of the spillway. The backhoe easily excavated the blocky andesite to the depth indicated on the contract plans. The shape of the excavation, however, deviated significantly from the plans (photo 48). The trench averaged 4 feet in width across the bottom (instead of 2 feet) and was 7 feet wide at the top (instead of 4-1/2 feet). More significantly, the existing spillway grade was between 0.8 and 1.6 feet below the sill crest elevation of 1377.8. Also, it was apparent that, during the spillway excavation, the contractor had overexcavated an additional 1 to 1-1/2 feet and subsequently backfilled the area with muck from the spillway excavation. A 2-inch-thick layer of crushed gravel was uncovered on both sides of the trench for the full width of the spillway near elevation 1375, almost 3 feet below the spillway crest elevation (photo 49). The contractor was instructed to excavate the backfill material on both sides of the trench 15 feet upstream and downstream from the sill centerline (photo 50). The configuration of the resulting excavation is depicted in figure 4. No attempt was made to excavate to "sound, intact" rock because of the loose, blocky nature of the andesite rock mass. The resulting excavation probably would have been excessive for the intended foundation purpose. Hand labor was used to remove dislodged rock from the bottom of the sill trench and at the toe at the side slopes.

Sill Construction

3.26 A new spillway sill tailored to the resulting excavations was designed. The redesigned, as-constructed sill across the floor of the spillway is 6 feet wide at the top. It is level at elevation 1377.8. From the edge of this level surface, a triangular shaped concrete section slopes away at 1V on 2H down to the top of intact rock near elevation 1375. (See amended pl. 19 for the configuration of the sill and photos 51 through 59 for the sequence of construction.) The sidewall sill excavations are backfilled with shotcrete. A welded wire fabric reinforcement extends under the surface of the shotcreted slopes and across the concrete sill (photos 51, 52, and 58). The original steel reinforcement was unchanged except that the cages on the slopes were set as far back as the excavations would allow while maintaining close to the 1/2:1 slope. This resulted in several contact points between the steel and the rock, but the majority of the steel had a clearance of 1 to 6 inches because of the irregular rock surface. In general, the bases of the cages were inset further than the top, creating a somewhat steeper inclination. At elevation 1377.8, the inside edge of the cage was 20.0 feet from the spillway centerline on the west side and 21.4 feet on the east side. As originally designed, the total distance between the two would have been approximately 36 feet.

3.27 Seventy-four cubic yards of 3000 lb/in^2, 3/4-inch concrete was used to pour the sill. The concrete was placed on a wetted foundation surface of loose rock (photo 55) except at the toe of the slopes where it was placed on clean, intact rock (photo 53). Rebar dowels were left sticking out of the concrete at the ends (photos 54 and 57). The rebar cages under the shotcrete were later spliced into these dowels. Particular attention was paid to adequately vibrating the concrete along the rock contacts at the slopes (photo 54). After the concrete had cured for a couple of days, the rebar cages and wire mesh reinforcement were laid against the slopes and tied to the dowels at the base (photo 57). In some cases, the rebar stirrups had to be moved to allow for protruding rocks. The wire mesh was cut and bent to conform to the shape of the excavation and tightly tied to the rebar cage (photo 58). Pneumatically placed concrete was applied on 8 January 1982 (photo 59). The area where the shotcrete was to be placed was washed down to (1) remove any objectionable material, (2) pre-wet the surface and (3) insure a good bond with the shotcrete. The materials and application met the specifications for wet mix pneumatically placed concrete. The shotcrete was applied by spraying it back and forth, starting at the bottom and working toward the top until a minimum thickness of 3 inches covered the wire mesh. The edges were rounded into the irregular rock surface and thin feather edges were avoided. After the concrete and shotcrete were placed in the sill, the vicinity of the sill was brought to grade with backfill material, and the spillway construction was complete (photo 6).

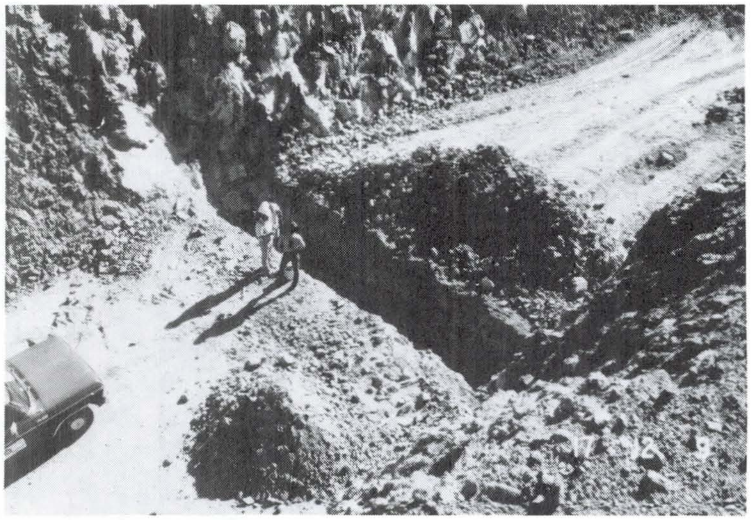
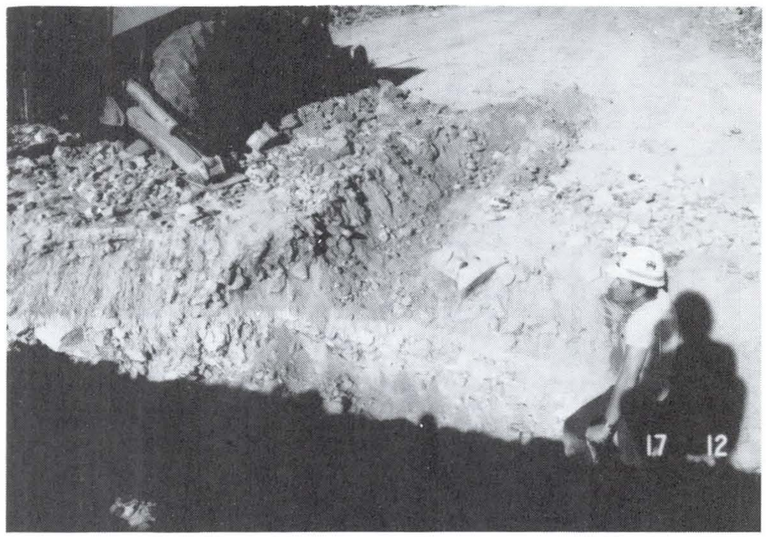


Photo 48
View of initial sill trench excavation looking from west side lower bench. Note irregular side-wall excavation on east slope. (17 Dec 1981)

Photo 49
Trench excavated normal to sill on upstream side exposing gravel layer at base of backfill. Contractor had overexcavated invert by almost 3 feet. (17 Dec 1981)



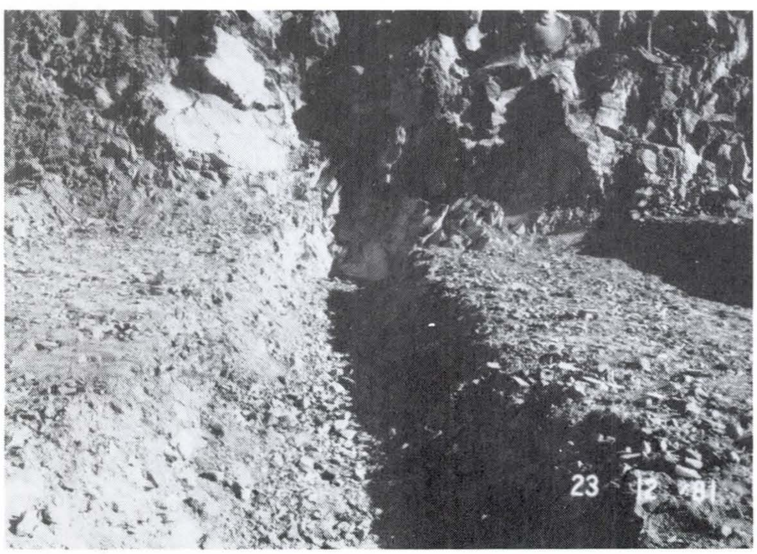
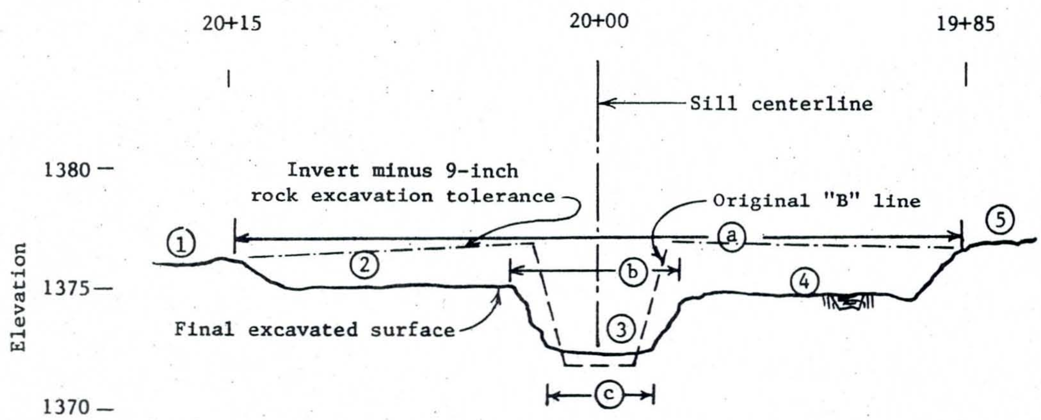


Photo 50
Sill trench after excavation of backfill 15 feet upstream and downstream from sill centerline. Excavation at toe of slope almost complete. View looking east. (23 Dec 1981)



Typical Section

		9 Feet east of centerline	Spillway centerline	9 Feet west of centerline
Dimensions	a	30.0'	30.0'	29.0'
	b	6.5	7.5	7.0
	c	3.5	4.0	2.5
Elevations	1	1376.3	1376.0	1375.8
	2	1375.3	1375.1	1375.0
	3	1372.4	1372.6	1372.8
	4	1374.8	1374.5	1374.9
	5	1376.9	1376.9	1377.0

Figure 4. Sill Trench Excavation Cross Section.

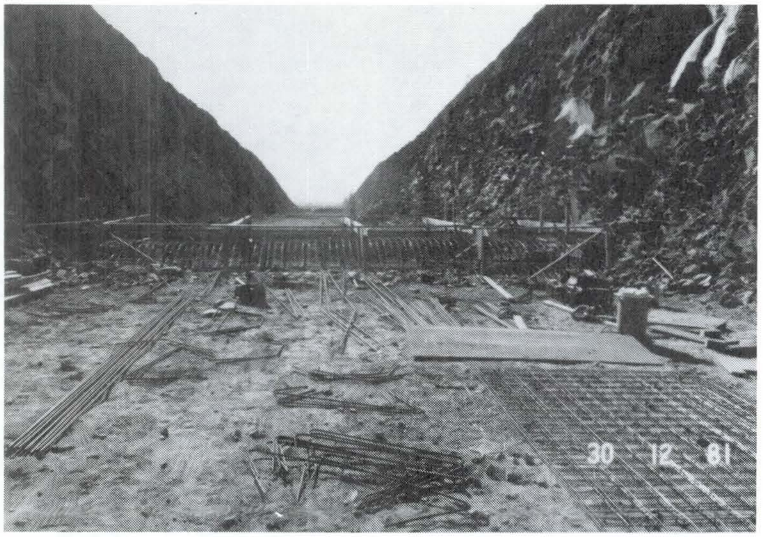


Photo 51
Sill trench with steel reinforcement and wire mesh in place. Note amount of overexcavation. (top of steel at spillway invert). (30 Dec 1981)

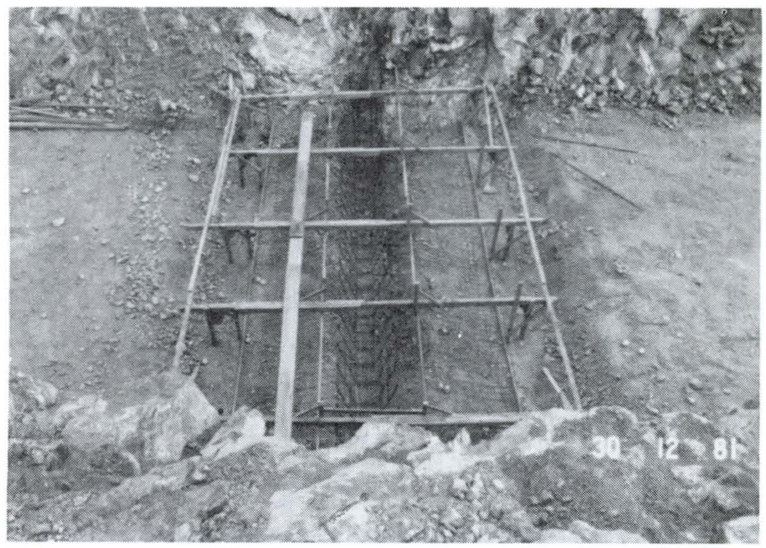


Photo 52
Overhead view of sill-trench steel reinforcement and wire mesh. Side-wall reinforcement to be installed later. Wood-screed forms completed. (30 Dec 1981)

Photo 53
Wetting down rock surface at east end of sill trench prior to initial placement of concrete. (31 Dec 1981)

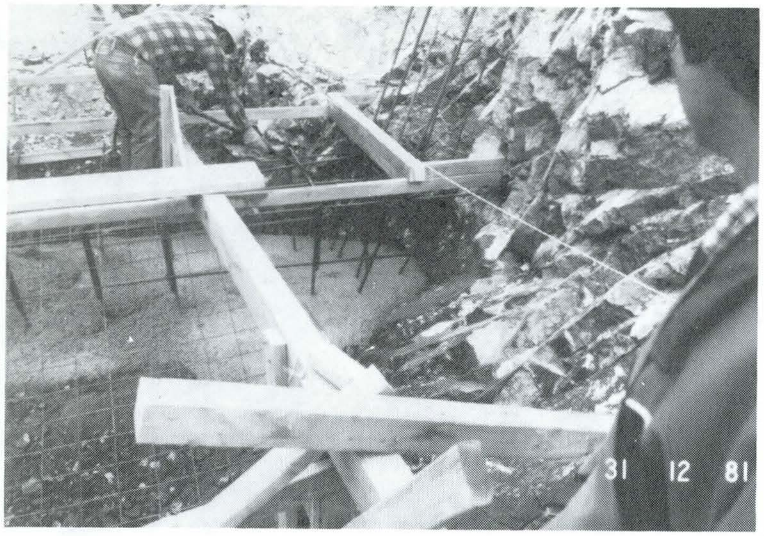


Photo 54
Vibrating concrete along rock contact at east end of sill trench. Note rebar dowels protruding out of concrete and up the side wall. (31 Dec 1981)

Photo 55
Vibrating concrete under wire mesh on downstream side of sill trench. Concrete placed on wetted but not thoroughly cleaned "rock". (31 Dec 1981)

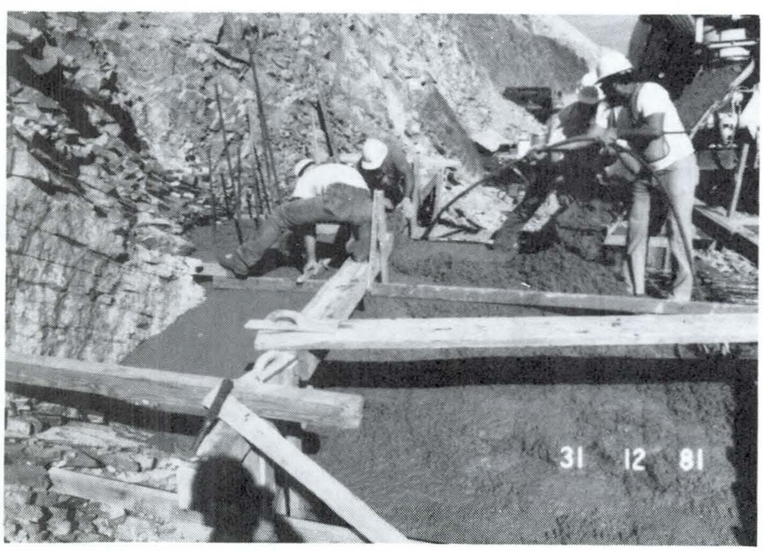


Photo 56
Laborers using screed to level 6-foot wide sill invert. Note 1V on 2H contact with rock face at left (west side of trench). (31 Dec 1981)

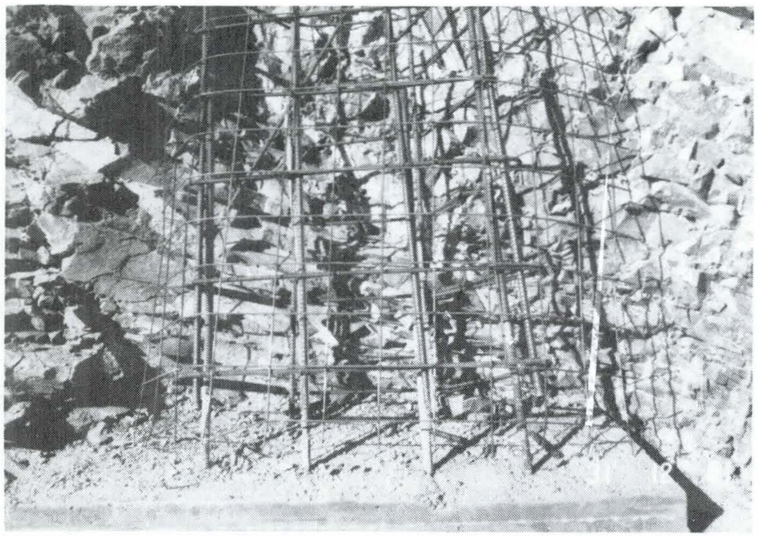


Photo 57
Rebar cage and wire mesh tied to dowels at west end of sill trench concrete. (5 Jan 1982)

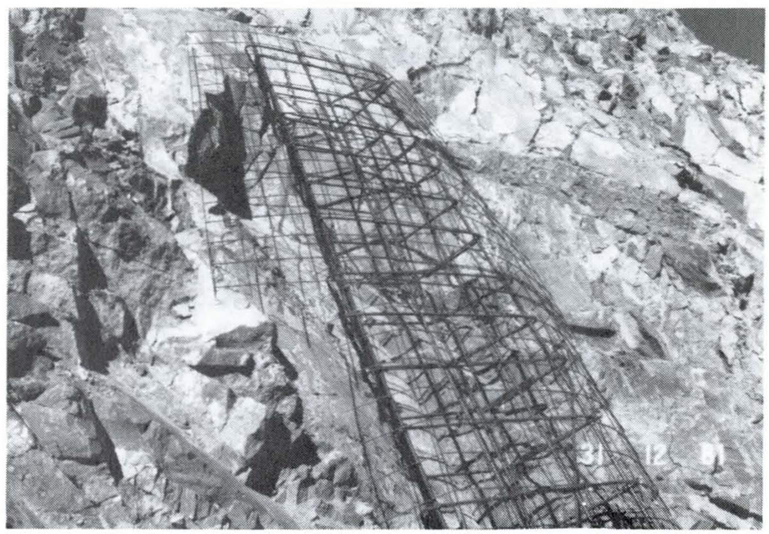


Photo 58
West side-wall sill steel reinforcement in place. Note how wire mesh conforms to the irregular shape of the excavation. (5 Jan 1982)

Photo 59
Shotcrete being placed on west side-wall sill. (8 Jan 1982)

Right Abutment

3.28 The excavation and foundation preparation of the right abutment at Adobe Dam was the most critical aspect of construction. It was important in the geotechnical sense because it represented the interface between foundation rock and compacted embankment at the thickest (highest) portion of the dam. It was also the single most important link in the sequence of construction. Excavation and construction of the outlet works, which passes through the abutment, could not begin until at least the upper abutment surface had been entirely excavated. More important, completion of the abutment surface treatment was the last step before dam closure.

3.29 The general sequence of abutment construction was as follows:

Dec 80 through Apr 81, excavated and cleaned the abutment above the outlet works.

Mar 81 through May 81, excavated the outlet works trench.

May 81 through July 81, constructed the outlet conduit and backfilled the trench.

Aug 81 through Sep 81, grouted the upper abutment (above the outlet works).

Aug 81, excavated and cleaned the lower abutment (below the outlet works).

Sep 81 through Nov 81, grouted the lower abutment.

Sep 81 through Dec 81, placed dental concrete.

Nov 81 through Dec 81, placed embankment materials.

3.30 Construction costs significantly in excess of the contract estimated amounts and modifications to the contract were related to the abutment excavation and foundation preparation. These included costs related to:

Abutment excavation (bid item no. 7)...................	$ 13,728
Dental excavation over 100 Cy (bid items no. 10B).......	340,020
Dental concrete over 60 CY (bid item no. 38B)..........	76,050
Foundation drilling and grouting (bid item no. 43)......	149,060
Investigation of abutment (MOD P00005).................	13,141
Revised abutment excavation (MOD P00008)...............	43,125
Abutment filter (MOD P00009)...........................	97,560
Irregular abutment surface (MOD P00010)................	43,495
Delays due to irregular abutment surface (MOD P00014)...	49,625
Extra costs for foundation drilling and grouting (MOD P00015)..	5,851
Abutment fill costs (MOD P00018).......................	10,000
Total	$841,655

The added costs were basically due to:

- The unexpected nature of the foundation rock formation (volcanic block flow).
- The unexpected difficulty in excavating the overburden (blasting required).
- The unexpected extreme relief of the bedrock surface.

- The unexpected high grout takes (4694 placed sacks versus 200 sacks estimated).

EXCAVATION

3.31 The contract plans called for the removal of between 5 and 12 feet of overburden from the abutment surface. The overburden was classified on the drill logs as slope wash and talus debris containing boulders to 6 feet maximum dimension, partially cemented with caliche. Beginning on 1 December 1980, the contractor attempted to strip the overburden from the abutment bedrock surface using conventional excavation methods; (D-9H dozers with double-tooth rippers (photo 60). By 10 December, it was obvious that this method was very inefficient in loosening the large, caliche-cemented blocks. Only the very surficial, loose, uncemented talus blocks had been removed (photo 61). The contractor requested permission to drill and blast the overburden in order to loosen the large volcanic blocks and facilitate their removal.

Test Blast

3.32 It was agreed to evaluate the contractor's proposed abutment blasting technique in a small test area which would eventually underlie the random shell of the embankment. The area was 20x40 feet in dimension between stations 11+80 and 12+20, 50 feet downstream from centerline. At the surface, it was typical of the entire abutment after the initial D-9H excavation. Boulders (talus blocks) to several feet in diameter protruded through powdery, sandy silt and caliche (photo 62). A rotary percussion airtrack drilled holes 3-inches in diameter, 8 to 11 feet deep, on a 7x7-foot pattern. The hole bottoms were 1 foot above the projected top of bedrock. Drilling action indicated hard boulders to 5 feet thick separated by soft, brown infilling up to 2 feet thick. One exploratory hole was drilled 24 feet deep at station 12+32, 60 feet right of centerline. Drill action indicated no very soft zones below approximately 8 feet; however, a softer brown zone about 1-1/2 feet thick was encountered at about 18 feet.

3.33 The area was shot on 16 December (blast no. 11). The powder factor was 1.0. The blast effectively dislodged the caliche-cemented volcanic blocks from the surface (photo 63). The blast area was mucked out and further excavated as deep as possible with a D-9 dozer. The muck included numerous blocks which were at least 5 feet in dimension. Even though blocks up to 7 feet in longest dimension were partially exposed in the bottom of the roughly cleaned area (photo 64), it appeared that they could have been dislodged if properly attacked with CAT D-9H dozer. The working area, however, was too small and steep-sided for the dozer to operate efficiently. The general feeling, after the excavation had been inspected, was that the top of bedrock had not yet been exposed. The volcanic blocks had no apparent systematic orientation and were surrounded by a matrix of sandy silt (photo 64). In order to better evaluate the abutment bedrock surface, it was decided to loosen the remaining overburden downstream from the core zone by similar blasting techniques (blasts 14A and 14B). (See the abutment blasting summary, Table 2, for pertinent blasting data). After blasting on 24 and 30 December 1980, the downstream portion of the abutment above the existing ground surface was mucked out using a CAT D-9H (photo 65). By

12 January, the area had been roughly cleaned. Not only had blasted material been removed, but also whatever could be dislodged by the dozer. In some areas the excavation was 4 to 5 feet below the blast depth and the projected "top of rock." The average surface elevation was 8 feet below the original ground. It still was not possible, however, to evaluate the nature of the abutment surface because of the covering of loose rock and powdery, sandy silt and caliche.

Exploratory Drilling

3.34 The excavation had been deep enough to encounter the top of rock as projected from the abutment and outlet alinement core holes. Only in isolated areas toward the top of the abutment, however, was there any indication of a typical bedrock surface. In these areas, the angular basalt (latite) clasts were tightly bound in a cemented, clastic matrix (photo 66). A detailed cleaning was required to properly evaluate the abutment surface and determine if a suitable foundation had been uncovered. It was decided, however, to postpone this cleaning until after exploratory drilling in the hope that, by closely monitoring the drilling action of a rotary percussion airtrack drill, it would be possible to determine the nature of the underlying foundation and if a change would be encountered with additional excavation.

3.35 Twenty holes were located on a grid 50x25 feet between station 10+50 and 13+00 in the excavated downstream area. On 16 January, 17 holes were drilled to a one-"steel" depth, approximately 11 feet, and 3 holes to two-steel depth, approximately 23-1/2 feet. The rate of drilling resistance was qualitatively classified as (1) hard, (2) moderately hard, (3) moderate, (4) moderately soft or (5) soft. The differences between hard and soft were obvious. Gradations in-between were less distinct. The time required to drill 10 feet varied from less than 1 minute to more than 2 minutes. The exploratory drilling had the following results:

a. Uniform hard drilling was not encountered for the full depth in any holes.

b. Sixty-five percent of the drilling was hard and 31 percent moderately soft to soft.

c. Soft zones of 5-1/2 and 6 feet in thickness were encountered with the average thickness of 2 feet.

d. Hard zones up to 12 feet thick were encountered with the average thickness of 2.7 feet.

e. Uniform hard drilling was not encountered below any particular depth or elevation.

f. Very generally, harder drilling was encountered higher up on the abutment.

The conclusions reached from this exploratory exercise were twofold.

a. The nature of the "bedrock" would not change appreciably with increased depth (up to at least 10 feet and possibly deeper).

b. The bedrock foundation would consist of both hard volcanic rock blocks and softer clastic infilling.

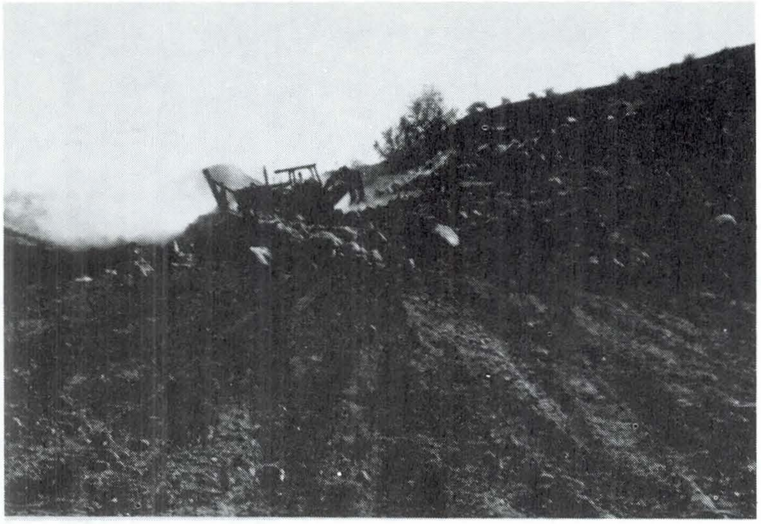
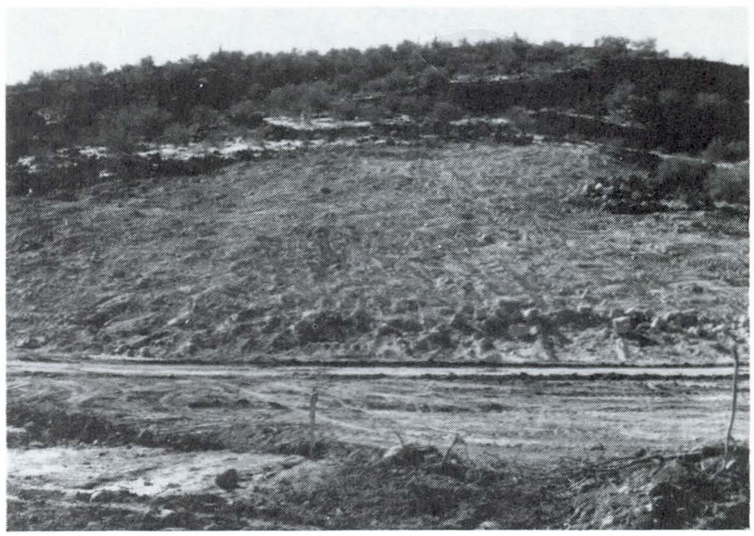


Photo 60
Surface stripping of overburden from right abutment using CAT D-9G. Boulders to 4-foot maximum dimension in muck pile. (9 Dec 1980)

Photo 61
Right abutment after stripping with dozers; only 1 to 2 feet removed. Note talus blocks on untouched portion of the abutment. Area of subsequent test blast at extreme left center of photo. (15 Dec 1980)



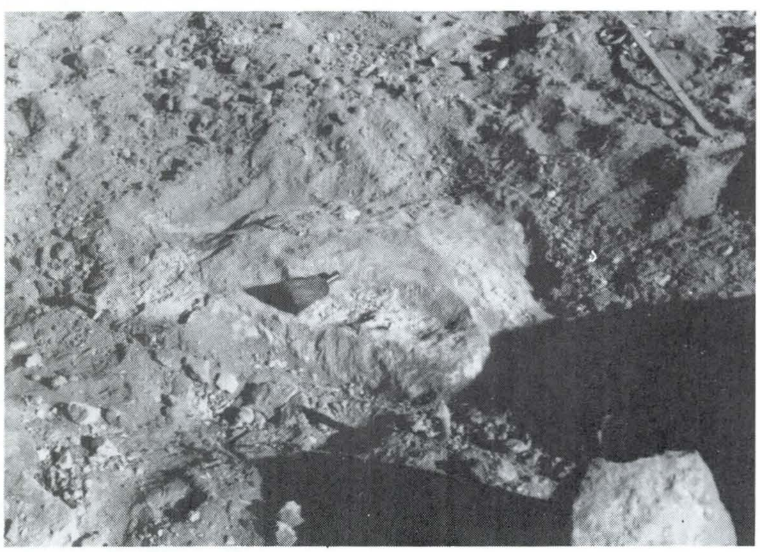


Photo 62
Five-foot-diameter basalt block swept clean of dust and rubble as exposed in test blast area before drilling. Approx. station 12 + 00, 50 feet right of centerline. (15 Dec 1980)

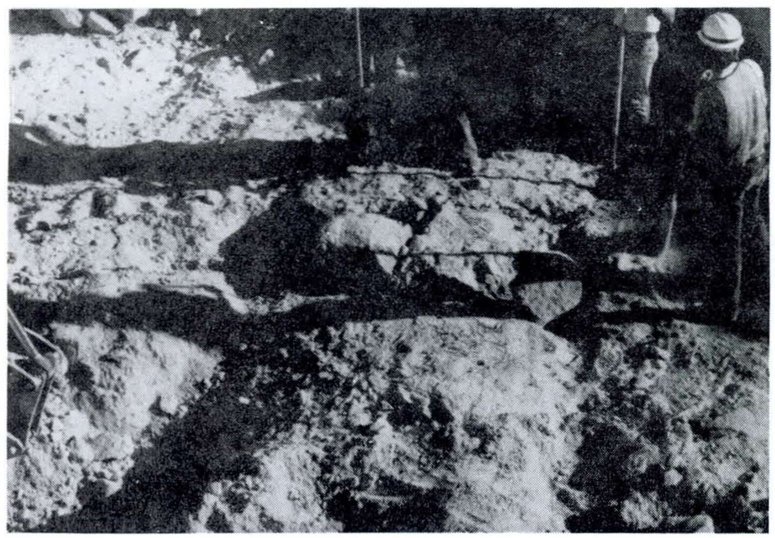
Table 2. Abutment blasting summary

Blast No.	Date Blasted	Stationing	Depth Drilled	Hole Diameter	Blast Pattern	Stemming	Pounds ANFO	No. Delays	Maximum Pounds/Delay	Cubic Yards Material in Place	Powder Factor	Remarks
11	16Dec80	11+80-12+20	8'-10'	3"	7'x7'	3'	450#	8	65	454	1.0	Test blast; 20'x40' area, 50 feet right of centerline.
14A	24Dec80	12+00-13+00	4'-10'	3"	7'x7'	2'	2062#	13	182	2079	1.0	Lower half of downstream blasting.
14B	30Dec80	10+50-12+00	8'-10'	3"	7'x7'	3'	3434#	15	222	4171	0.7	Upper half of downstream blasting.
20A	4Feb81	11+99-12+97	6'-12'	3"	7'x7'	4'	5093#	15	325	5681	0.9	Holes above station 12+48 drilled 3 feet deeper than calculated depth to bedrock.
20B	12Feb81	11+22-11+92	10'-12'	3"	7'x7'	4'	4000#	15	272	4525	0.9	All holes drilled 3 feet deeper than calculated depth to bedrock.
19A	12Feb81	10+66-11+15	10'-12'	3"	7'x7'	4'	2200#	13	176	2488	0.9	All holes drilled 3 feet deeper than calculated depth to bedrock.
19B	12Feb81	9+68-10+59	10'	3"	7'x7'	5'	3038#	15	210	3928	0.8	All holes drilled 3 feet deeper than calculated depth to bedrock.

Sheet 1 of 1

Photo 63
Right-abutment test-blast area immediately after shot (blast no. 11). Note large size of rock blocks dislodged; several are 5 feet in maximum dimension. (15 Dec 1980)

Photo 64
Bottom of test-blast excavation after mucking with dozer and rough cleaning with shovel and broom. Note 6x7½-foot rock mass in center foreground apparently surrounded by loose infilling. (17 Dec 1980)



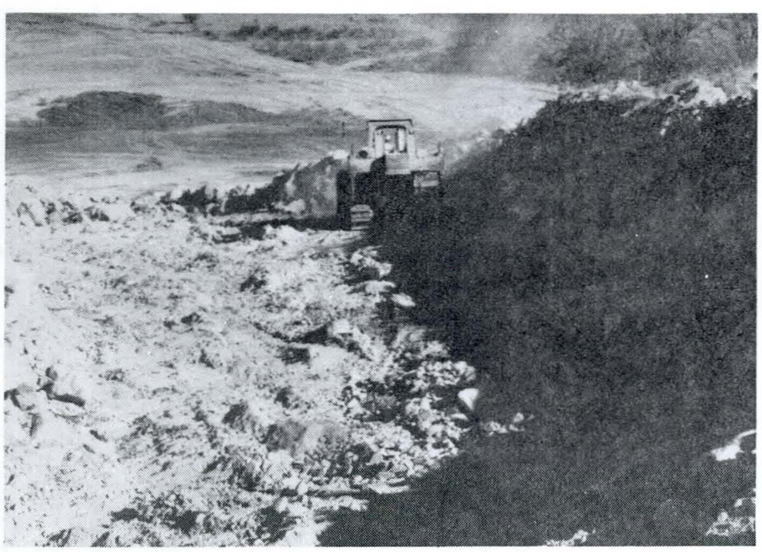


Photo 65
CAT D-9H mucking-out downstream portion of right abutment excavation after blasts nos. 14A and 14B. Dozer working at station 11 + 00. (8 Jan 1981)

Detailed Cleaning

3.36 In order to inspect the foundation conditions near the stripping elevation (approximate existing bedrock surface as shown on amended pl. 12) and to assess the degree of difficulty in preparing the surface, a section 40x40 feet was cleaned in detail. The area was between stations 11+50 and 11+90, 35 to 75 feet right of centerline. A backhoe and hand labor were used to perform the work (photo 67), which was completed on 20 January 1981.

Upstream Excavation

3.37 After the exploratory drilling and inspection of the cleaned area of 40x40 feet, the contractor was granted approval to loosen the overburden from the remainder of the abutment using the previously demonstrated drilling and blasting techniques (photo 68). Originally, it was planned to drill and shoot to the projected top of rock as shown on the contract plans. Since in certain areas this would have involved excavation of only a couple of feet, however, the contractor was directed to drill all holes three (3) feet deeper than the surveyed and computed depth to bedrock up to a maximum total depth of twelve (12) feet. The reasons for the directed change were as follows:

a. Deeper holes would allow for more effective loading and stemming.

b. A more uniform drilling depth (12-foot maximum) would facilitate drilling operations.

c. Increased depth of excavation would only serve to improve the quality of the ultimate foundation since the competence of the infilling material seemed to improve slightly with increased depth.

d. Relogging of several holes, especially toward the upstream end of the outlet works, indicated that the top of rock might be lower than originally believed.

3.38 Four separate blasts (photo 69) were conducted between 4 and 13 February 1981 on the upstream portion of the abutment. They were designated 199A, 19B, 20A, and 20B. A 7x7-foot pattern with powder factors between 0.77 and 0.95 was used. (See the Abutment Blasting Summary, table 2, for details.) The results of these blasts were the same as for the downstream blasts. The surficial "overburden", which consisted of large, angular to sub-angular, volcanic blocks in a sandy silty matrix, were effectively dislodged (photos 70 and 71). As before, the loosened material was moved down the abutment by D-9 dozers and stockpiled for later use as backfilling material along the upstream toe of the dam (photo 72 and amended pl. 14). (See photo 73 for a panoramic view of the abutment after mucking with dozers.)

Headwall Slope Blast

3.39 After the rough dozer excavation was completed, the contractor began further cleaning of the abutment using a backhoe and hand labor. The contractor was instructed to remove all loose or broken rock from the surface in the area of the grout curtain as specified in Section 2D

5.2.1. of the contract. This work was begun at the top of the abutment near station 9+68 in an area of shattered and cindery rock (photo 74). The condition of the rock was due in part to the nature of the rock mass in this area and in part to the effects of blasting. A review of post-blast report no. 19B indicates that holes near the centerline at the top of the abutment were drilled and shot as much as 7 feet below the calculated depth to bedrock instead of 3 feet as directed. (See paragraph 3.37.) Regardless of the cause, the subsequent removal of loose and broken rock resulted in an oversteepened slope at the head of the abutment excavation (photo 75). Therefore, the contractor was instructed to slope back the headwall on a 1:1 slope to facilitate later foundation treatment and material compaction in the area of the core contact. The contractor drilled pre-split holes on 3-foot centers and shot the headwall with pre-split stick powder (photo 76). The blasting succeeded in laying back the slope at the specified angle. However, subsequent backhoe and hand cleaning of the loose, cindery rock again caused a somewhat oversteepened final slope. The angle was not as extreme as before the blast and was judged to be suitable for material placement after appropriate foundation surface preparation.

Abutment/Dental Excavation

3.40 After all the blasting had been completed, the surface was still covered with rock debris and dust. Individual volcanic blocks up to 3 feet in dimension were loose enough to be removed by a small backhoe. The removal of these loose blocks and the surficial debris could not be accomplished by the heavy equipment (dozers or front-end loaders) because of the irregularity of the abutment foundation surface.

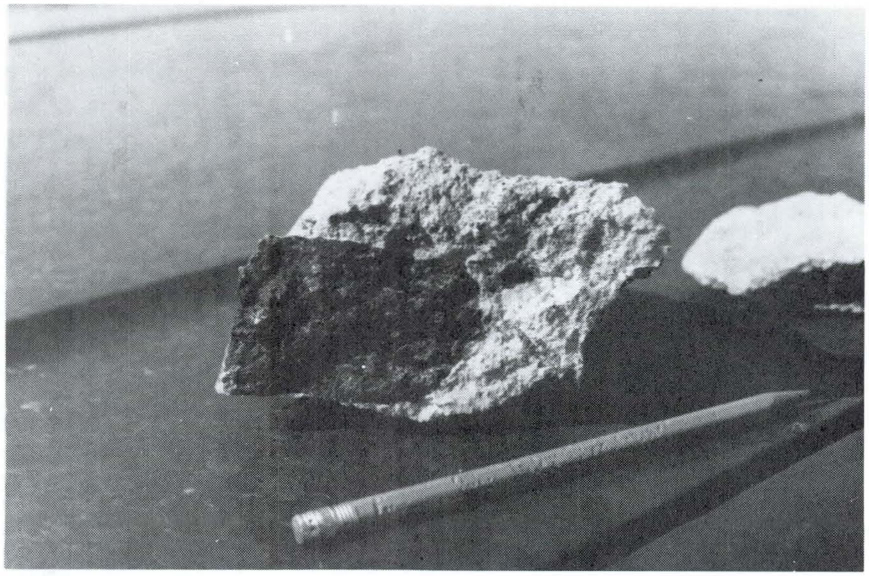


Photo 66
Rock sample from surface of abutment excavation at station 10 + 50 (approximately 9 feet below original ground surface). Note the intimate bond between the clastic tuffaceous infilling and the angular latite clast. (10 Jan 1981)

Photo 67
Start of backhoe and hand cleaning of 40x40-foot inspection area on right abutment. Stake at extreme left center of photo is at station 11 + 50, 75 feet right of centerline. Note relief already developing. (15 Jan 1981)

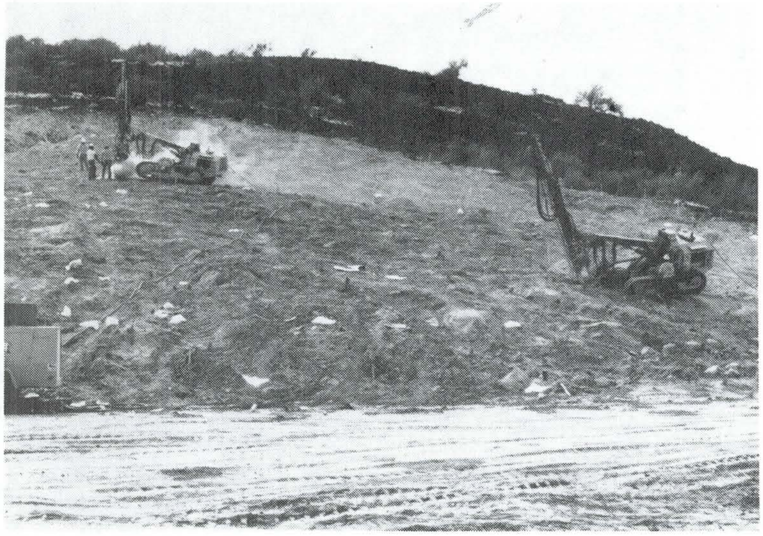


Photo 68
Drilling production blast holes on upstream portion of right abutment at stations 12 + 35 and 11 + 85; 3-inch holes on a 7x7-foot pattern. (3 Feb 1981)

Photo 69
Blast no. 20A at right abutment; stations 11 + 99 to 12 + 97, upstream portion of abutment. First of four separate blasts. (4 Feb 1981)

Photo 70
Results of production blast no. 20A at right abutment. Note previously excavated downstream portion of abutment at left. (4 Feb 1981)

Photo 71
Close-up view of back slope (station 11 + 99) resulting from blast no. 20A at the right abutment. Note varied size of generally sub-angular basalt blocks; several are larger than 4 feet in maximum dimension. (4 Feb 1981)

Photo 72
Oversize volcanic blocks excavated from right abutment (at right) were used as backfill in the trench along the upstream toe of the dam. Grader seen leveling bedding stone in trench at station 117 + 00 prior to placement of backfill stone. (26 Mar 1981)

Photo 73
Panorama of right abutment after blasting and stripping with dozers. Drill at left in position for outlet works test drilling and blasting. Area of detailed cleaning, 40x40 feet, just to right of drill. Note grey tone at upper right portion of excavation and brown tone at lower left (basalt vs. infilling). "Dental" excavation already underway at top of stripped area. (6 Mar 1981)

Photo 74
Broken and cindery rock mass exposed in original slope excavation at top of right abutment. (12 Mar 1981)

Photo 75
Steep headwall excavation at top of abutment prior to sloping back at 1:1. (12 Mar 1981)

Photo 76
Muck pile from headwall slope blast at top of right abutment. Note variety of sizes of the sub-angular basalt blocks. (25 Mar 1981)

The contractor, therefore, used backhoes and hand labor to perform the final phase of the abutment excavation (photo 77). Since this was not considered by the contractor or the Contracting Officer to represent "normal" abutment excavation, the quantity of material removed in this manner (as determined by surveys) was considered to be dental excavation (amended pl. 13, cross section at station 10+00). As a result, the amount of dental excavation for the entire project exceeded 5700 cubic yards; 38 times the pre-construction estimate.

3.41 The contractor excavated the entire abutment surface above the outlet works trench between 9 March and mid-April 1981. Similar "dental" excavation techniques were used on the abutment below the outlet works (photo 78) after the conduit had been constructed and the trench backfilled. Excavation began on 13 August, and the lower abutment was approved for grouting on 29 August. (See fig. 5 for an aerial view of the status of abutment construction on 5 September 1981, showing the abutment excavation, completed outlet conduit, and exploration.)

3.42 At the upstream edge of the lower abutment, the contract plans called for excavation to rock down to elevation 1332.7 (amended pl. 13). Due to the blocky nature of the rock surface, some excavation below this elevation was required to remove loose rocks and pockets of debris generated from cleaning above (photo 79). This over-excavated area was replaced with compacted core material to elevation 1332.7 after the unsuitable material had been removed. The 5-foot thick upstream core blanket was ultimately placed on top of this backfilled and compacted area.

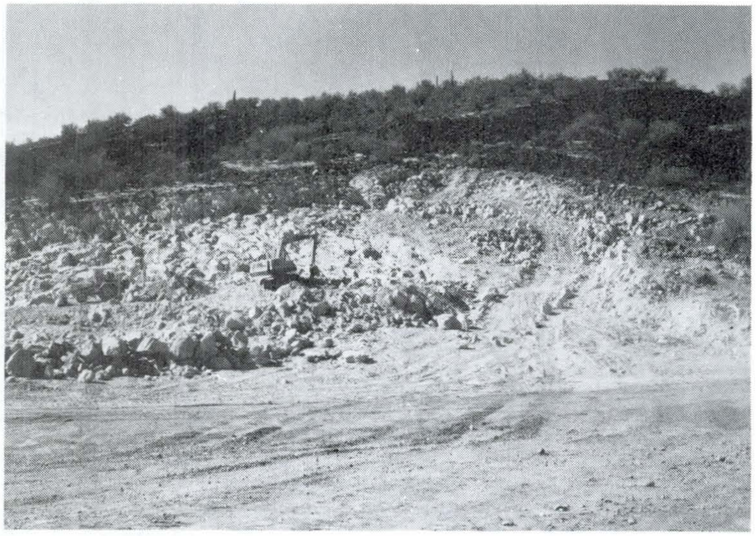


Photo 77
Backhoes being used to perform "dental" excavation on right abutment surface. Note irregularity of cleaned surface at left, and muck from headwall slope blast at top of excavation. (26 Mar 1981)

Photo 78
Three small and one large backhoe and laborers being used for lower abutment/dental and exploration trench excavation. Note grout plant set-up on completed outlet works conduit. (19 Aug 1981)

Photo 79
Upstream edge of abutment/alluvium contact below outlet works (backfill at lower right). Rock debris and loose soil was removed below elevation 1332 and replaced with compacted core material. (28 Sept 1981)

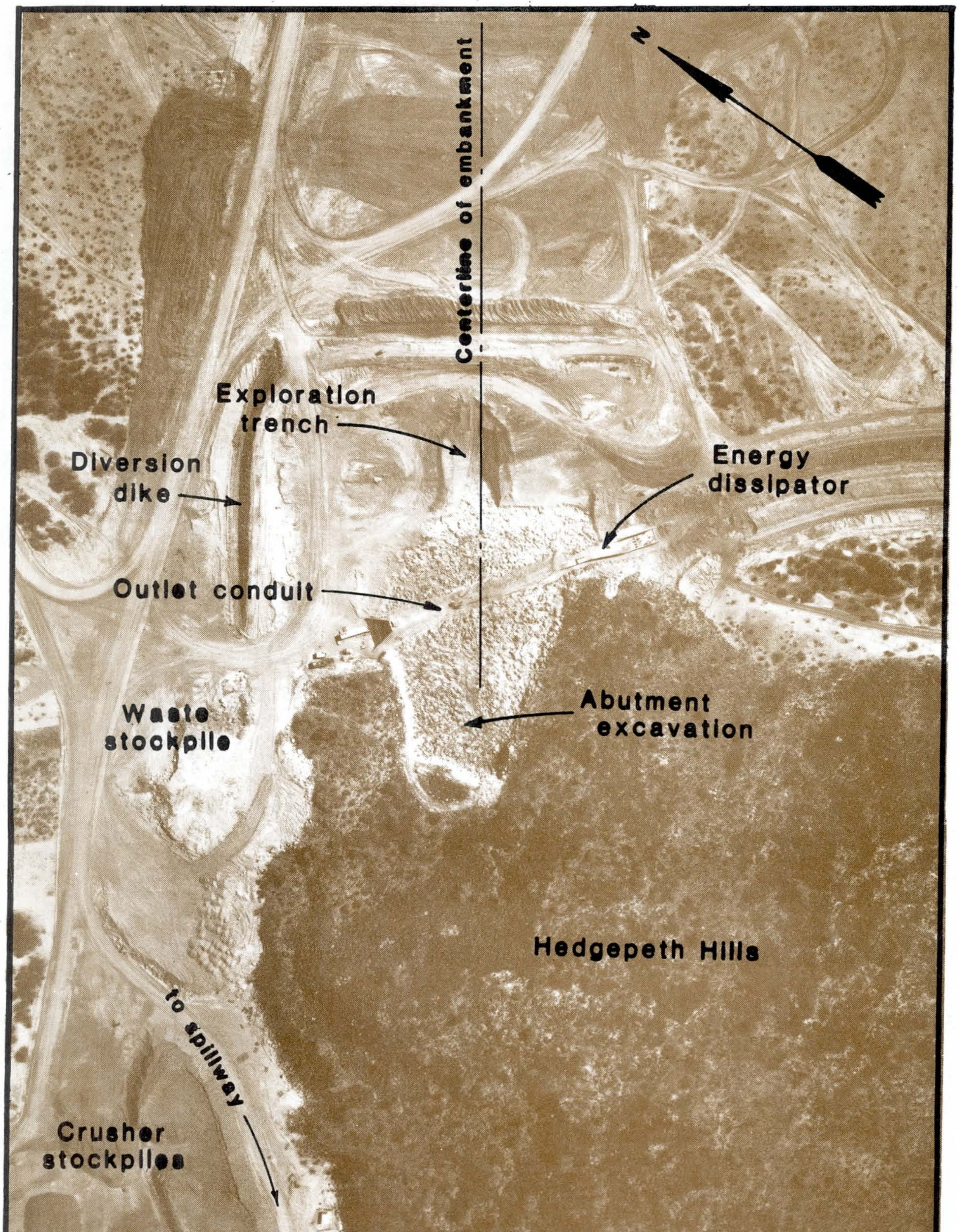


Figure 5. Status of Abutment Construction on 5 September 1981.

3.43 As stated previously, the foundation surface at the right abutment was expected to be a fractured basalt (latite) with infilling of fine grained ash. The surface was anticipated to be somewhat irregular and uneven. A more regularly and more highly jointed andesite was believed to underlie the basalt; and because of the cleaner and more numerous fractures, the andesite was expected to be more permeable than the infilled basalt. Along portions of the flanks of this volcanic sequence, a tuffaceous agglomerate (QTt) was the expected foundation rock. In retrospect, the actual geologic conditions encountered were substantially as expected. Initially, however, identification of the bedrock "surface" was difficult because the magnitude of the physical characteristics of the basalt was underestimated. There was a greater percentage of infilling to basalt than expected (photo 80) and the individual basalt blocks were larger than invisioned, creating a very irregular and uneven surface (photo 81 and amended pl. 13, cross section at Sta. 13+20). The permeability of the underlying andesite was also underestimated. This caused much higher grout takes than anticipated.

Lithologic Units

3.44 In the original contract plans, the basalt and andesite were collectively referred to as QTv, Quaternary/Tertiary volcanics. The age determination was based upon the Geologic Map of Maricopa County, Arizona, published in 1957. More recent information, however, suggests that the volcanics were probably deposited during the mid-Tertiary Orogeny and are approximately 20 million years old (Eberly and Stanley).[1]

1. Eberly, D., and B. Stanley. "Cenozoic Stratigraphy and Geologic History of Southwestern Arizona," GSA Bull., 89, pp. 921-40, June 1978.

They will, therefore, be collectively referred to as Tv. The basalt and andesite will be designated Tvb and Tva, when appropriate, to emphasize their different geotechnical properties. The "tuffaceous agglomerate" (QTt) will retain its Quaternary/Tertiary age but will be symbolized QTa for reasons discussed in the description of the unit. Amended plates 1 and 3 have been changed accordingly.

3.45 **AGGLOMERATE.** The agglomerate unit at the right abutment (QTa) is very similar to the agglomerate at the northeast end of the spillway excavation. At the abutment, only the surface of the unit was uncovered. Its thickness and lateral extent can only be postulated from the limited exposures and from drill holes D-1, D-14, and D-39 in which between 4 and 7 feet were cored. As with the agglomerate at the spillway, the unit could and has been classified as: (1) calichified breccia, (2) cemented colluvium, or (3) tuffaceous agglomerate. The term agglomerate will be used to provide continuity with the spillway nomenclature although the unit at the abutment is probably best described as a calichified colluvium or breccia. The unit symbol was changed from QTt (in the original contract plans) to QTa because it is more an agglomerate (as opposed to conglomerate) than it is a tuffaceous deposit. It was exposed during foundation preparation in the core trench at the base of the abutment between stations 13+60 and 13+86 (amended pl. 3 and photo 82). It was also exposed and prepared as a foundation for the outlet works' slab between stations 71+75 and 71+50. The unit is characterized by sub-angular to sub-round, somewhat heterolothic, volcanic clasts in a cemented calcium carbonate-rich matrix. Where exposed at the surface, it is very well cemented. Core recovered from D-39 and the grouting at the base of the abutment further suggest that it is a competent foundation material for both the dam embankment and the outlet works' energy dissipator. There are no indications of uncemented or poorly cemented layers at the base of the unit which are indicative of the agglomerate in the spillway.

3.46 **BASALT.** The volcanic unit exposed at the surface of the excavated abutment is Tvb, Tertiary basalt. The basalt classification is actually a misnomer and was originally based on a field description of the dark grey, vesicular, extrusive rock. A core sample of the rock (D-24, 15.0 to 15.6 feet) was classifed by the SPD laboratory as a vesicular latite. The rock changes very gradually, apparently with depth, to what is mineralogically classified as andesite. The critiera for igneous rock nomenclature is based primarily upon the relative percentage of the various feldspar minerals. Chemical distinctions between latite-andesite-basalt can be very slight and generally are not significant. Therefore, the mineralogically incorrect but more widely recognized term "basalt" will be used to identify the upper volcanic rock unit at the abutment. The rock is dark grey and weathers to a distinctive dark brown/black characteristic of the talus blocks covering the surface of the Hedgepeth Hills. It is both vesicular and porphyritic with a fine grained groundmass rich in andesine (photo 83). Feldspar phenocrysts and megacrysts are common as are discrete xenoliths of aphyric lava. The intermediate chemical composition, high crystal content, and vesicular nature suggests a high viscosity lava (Sheridan, attachment 1). The high viscosity of the lava created a locally brecciated and generally blocky flow.

3.47 **Block Flow Structure.** The basalt block flow is exposed for the full width of the abutment excavation from the very top (station 9+37) to station 13+60 in the core trench. The extent of the flow away from the abutment is unknown. Similar block lava, however, is present locally as a cap on the Hedgepeth Hills and as widespread talus on the slopes. The flow, therefore, was evidently quite extensive. At the abutment it can be divided into two distinctive sections based upon the regularity of the flow structure. On the upper half of the abutment (generally above station 11+20), the flow is typified by near vertical cooling joints. Two joint sets predominate; one steeply dipping in the direction 135° and the other in the direction 210°. Some joints, up to 3 feet wide, contain broken and brecciated rock. Most, however, are relatively smooth and regular and show separation between the blocks of 1 to 3 inches.

3.48 On the surface of the flow, below an irregular, north-south trending "contact" near station 11+20, the distinctive jointed structure, for the most part does not exist. Instead, the abutment surface has the appearance of large, randomly oriented volcanic blocks. In one area located in the vicinity of station 11+30, 30 to 70 feet right of centerline, the jointed "structure" was identified. The area, however, remains an "island" in an otherwise structureless area. Indistinct lineations can be traced for short distances only to be terminated by large intact blocks. Even in the floor and walls of the outlet trench excavation, lineations and apparent joint structures are discontinuous and very poorly developed. The reason for the structural dissimilarity is unknown, but is possible that the jumbled lower abutment represents a disrupted fault block or slide. It is more likely that the lower abutment is either the leading edge or margin of the highly viscous flow. (See pl. 2 for an aerial photograph of the excavated abutment and the superimposed structure at the basalt unit.)

3.49 **Infilling Material.** The exploratory drilling performed during excavation of the abutment indicated that approximately 30 percent of the near surface abutment foundation consisted of material which was softer than the hard volcanic blocks. For most of the area, from station 10+00 to 13+20, this relatively soft material is a light brown, slightly indurated, lithic ashy sand. It occurs as infilling between the basalt blocks and in the subvertical joints. In the "jumbled" area on the lower portion of the abutment, the relative amount of the infilling to basalt blocks is the greatest. In certain areas the exposed apparent thickness was as much as 3 feet (photo 84). The infilling is tightly bonded to the basalt blocks (photos 66 and 80) and completely permeates the block flow mass. The nature of the infilling was well exposed during excavation of the outlet works trench. No voids or continuous loose areas were observed during repeated detailed inspections. On the abutment surface, the material appears to be unstratified and rather chaotic (photo 80). In the outlet works trench, however, thin stratification and occasional contorted or wavy bedding was noted (photos 85 and 86). Differing textures (photo 87) and occasional offsets within the infilling (photo 88) suggest several periods of infilling along with possible mass movements within the rock mass.

3.50 Because of the magnitude of infilling and its importance as a foundation material, an expert volcanologist, Dr. Michael Sheridan of Arizona State University, was contracted to determine the genesis and probable continuity of the infilling (attachment 1). Using techniques such as mechanical grain size analysis and electron microscopy as well as field observation, he believes the sands, which were transported by wind, to be volcanic in origin, probably from a nearby source. They were deposited into the blocky surface of the flow, which acted as an excellent trap for the material. The sands were later or simultaneously reworked into the flow mass by water, which also deposited calcium carbonate and minor zeolite cement. Dr. Sheridan believes the sandy infilling probably continues throughout the blocky part of the flow. This interpretation is supported by visual evidence in the outlet trench, pre-construction core borings, and permeability tests (pre-construction and during grouting).

3.51 Since the infilling material forms a significant percentage of the abutment surface, various samples were tested to assess its suitability as a foundation material. Undisturbed, representative samples were excavated from the abutment foundation (photo 89) and sent to SPD laboratory as well as to a local laboratory (Engineers Testing Laboratories, Inc.). Tests included: gradation, in-place density, specific gravity, consolidation, dispersive soils, soluble salts, and permeability. The results are inclosed as attachment 3 to this report. Generally, they indicate the infilling to be a dense, relatively incompressible, relativey impervious, nonsoluble silty sand. Additional samples were allowed to soak in water for periods varying from 24 hours to several months and showed no breakdown even though the material in the dry state is somewhat to extremely friable. Based upon the test results and visual observations, the tuffaceous sandy infilling exposed on the abutment surface from approximately station 10+00 to 13+20 was determined to be a suitable foundation material (attachments 3 and 4).

3.52 At the uppermost part of the abutment, above approximately station 10+00, the infilling was not evident at the surface (see para. 3.39, headwall slope blast). Significant amounts of the tuffaceous sand were likewise not encountered in pre-construction core hole D-23 until elevation 1367 and not until elevation 1365 in exploratory grout hole 9+69.5E. In addition to the lack of infilling material, the rock in this area was more fractured and locally crumbly; therefore, open fractures and joints several inches wide were present in the abutment foundation. The area required extensive pressure grouting to a 20-foot depth below the abutment surface and was eventually covered with dental concrete.

3.53 At the lower portion of the abutment, between the agglomerate and station 13+20, the tuffaceous infilling was also not exposed at the surface. Instead, the spaces between the blocks are filled with alluvial sediments consisting of cobbles, gravel, and sand in a clay matrix (photo 90). Isolated patches rich in calcium carbonate are present. This area was not excavated to the same relative depth as the remainder of the abutment because no blasting was performed east of approximately station 13+00. (The limit of blasting shows on pl. 2 as a lineation near station 13+10). It is believed that the tuffaceous sand infilling is present between the blocks a couple of feet below the surface. The alluvial infilling was and is considered to a suitable foundation and, therefore, was not removed. The clay matrix was stiff and tightly bonded to the volcanic blocks. The alluvial infilling was not continuous in a direction transverse to the dam axis and existed in isolated pockets adjacent to the massive basalt blocks.

3.54 **ANDESITE.** The andesite unit, Tva, was not exposed anywhere on the abutment. It was encountered in pre-construction core holes D-1, D-9, D-10, D-11, and possibly D-12 and D-13 (photo 91). It was also cored during the drilling of exploratory grout hole 13+01E. As mentioned previously, it is gradational with the basalt (latite), which it underlies, and may be a portion of the same flow event. Mineralogically, the two rocks are very similar. The andesite was classified as such by petrographic analysis of a core sample from D-11 at a depth of 34.8 feet. The rock is typically light to medium grey and fine grained. It is non-vesicular and very slightly porphyritic with occasional small zenoliths. As a rock mass, it is more regularly jointed than the overlying basalt and moderately to highly fractured. Some cracks are rehealed with calcium carbonate and others are filled with minor amounts of tuffaceous sand. As a rule, however, the fractures are open and can transmit large volumes of water. During grouting and pre-construction exploratory drilling, circulation losses were common. The large volume of interconnected open fractures was evidenced by the fact that holes drilled into the andesite would "breathe"; they would blow out air during the hot afternoon hours and suck in air during the cooler morning hours. It is believed that the pervious, fractured andesite does not have continuity with surface air and that the "breathing" is due to thermal expansion and contraction of air entrapped in the fractured rock mass. Reanalysis of cores from holes D-12 and D-13, near the upstream and downstream toes of the dam, respectively, indicate that the basalt/andesite contact may be present near elevation 1290 or 40 feet below the base of the dam. (See fig. 6 for a geologic cross-section at station 13+20.)

3.55 The magnitude of the andesite rock mass porosity was best exemplified during the inundation of pre-construction drill hole D-11, which is at station 13+20 on the right side of the dam centerline. (See amended pl. 11 for the location and profile of the hole.) The hole was drilled to a total depth of 65.7 feet (elevation 1275.6), with the lower 35.7 feet drilled through the andesite unit. The hole should have been backfilled with thick grout after drilling. The drilling contractor, however, only backfilled the upper portion of the hole; and when the overburden was removed during construction, it exposed an open 3-inch hole. Initially, the hole went unnoticed until 9 September 1981 when a portion of the lower abutment was flooded during a summer storm (photo 92). The location of the hole was then indicated by a small whirlpool (photo 93). It was estimated that the maximum elevation of the flood pool was a couple of feet above the orifice and that 5000 cubic feet of water was drawn into the hole before the lake was pumped dry. Subsequently, strong air movement was noticed in and out of the hole even though some debris had accumulated in the bottom.

Photo 80
Example of pervasive tuffaceous, silty gravelly sand infilling between and around the sub-angular latite blocks as exposed in the initial detailed cleaning area in the downstream portion of the abutment excavation. (20 Jan 1981)

Photo 81
Area of 40x40 feet in downstream portion of abutment excavation after detailed hand cleaning and air blowing. Note irregularity of cleaned surface. Photo taken from station 12 + 00 at centerline. (20 Jan 1981)

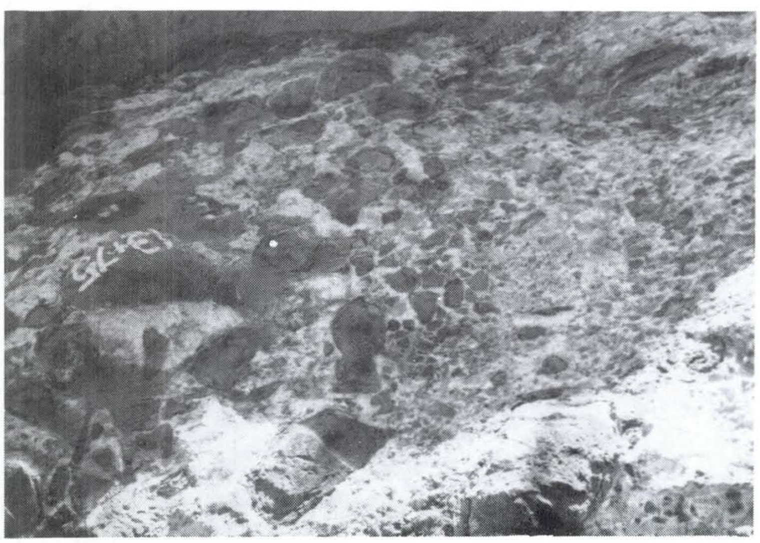


Photo 82
Well-indurated agglomerate exposed at base of abutment excavation (in exploration trench) between stations 13 + 60 and 13 + 86. View looking downstream (station 13 + 60 at right). (9 Nov 1981)

Photo 83
Detail of vesicular basalt (latite) and tuffaceous infilling as represented in core hole D-38 (downstream end of outlet). Note phenocrysts and zenoliths in the fine grained, vesicular groundmass and the intimate association of the infilling material. (29 Mar 1982)

Photo 84
Typical example of "structureless" block flow with abundant and thick accumulations of well-indurated clastic tuffaceous infilling, as exposed at the top of the west slope in the outlet trench at station 75 + 00. Overhang at lower left was treated with dental concrete during trench backfilling. (20 May 1981)

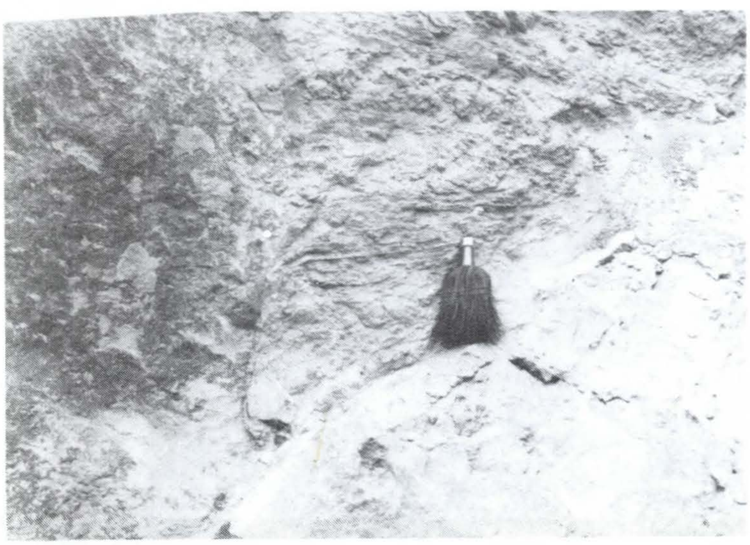


Photo 85
Example of intimate association of thinly layered and massively bedded infilling with the volcanic blocks. East slope, outlet station 73 + 90. (20 May 1981)

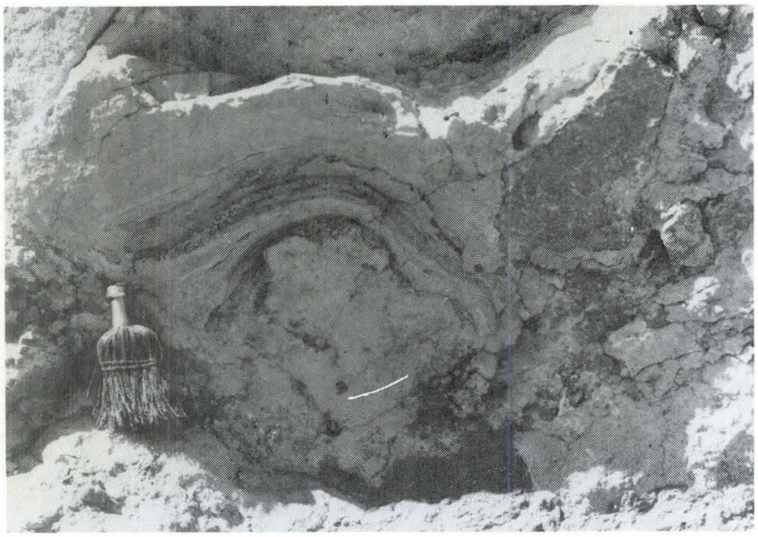


Photo 86
Contorted bedding structure developed in sandy, tuffaceous infilling. West-side outlet works' trench at station 73 + 40. (21 May 1981)

Photo 87
Differing textures within the infilling. Material at right is finer grained with apparent shears while material at left contains angular rock chips and is more structureless. Both are well-indurated. Photo taken in bottom of outlet trench excavation. (20 May 1981)

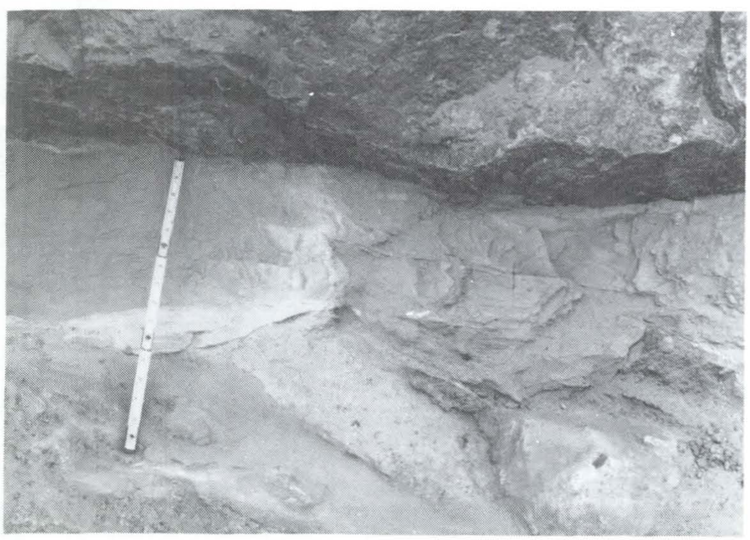
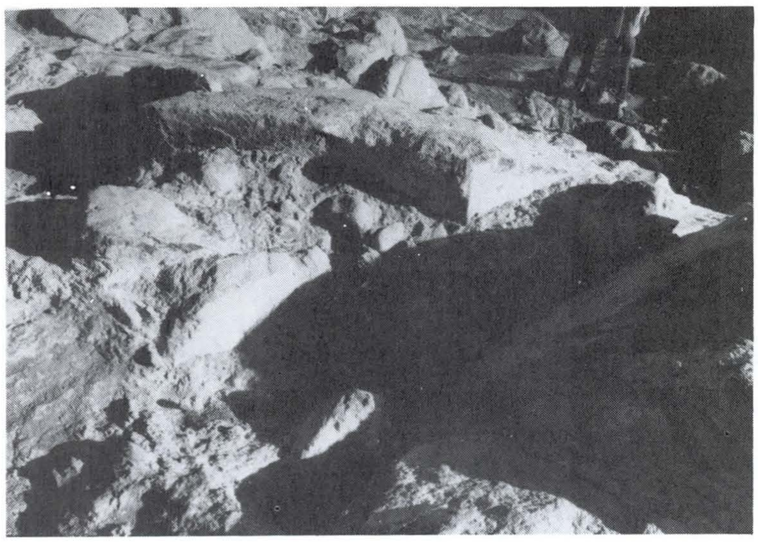


Photo 88
Very thinly stratified tuffaceous sand with offsets along distinct shear planes. White layer in center of photo is very fine, soft, uniformly graded sand. Floor of outlet trench at station 74 + 62. (20 May 1981)

Photo 89
Sample of representative infilling sent to ETL for testing. Station 11 + 56, 42-feet right of centerline, elevation 1362.7. Note sub-angular, cobble-size rock fragment imbedded in sample. (23 Jan 1981)

Photo 90
Typical example of alluvial infilling exposed between stations 13 + 20 and 13 + 35 on lower abutment surface. Note rounded gravel and cobbles in fine grained (clayey) matrix. (9 Nov 1981)



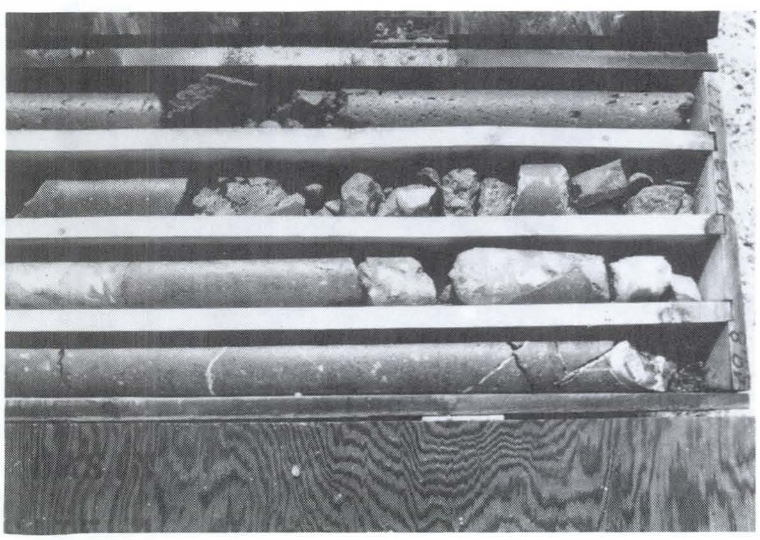
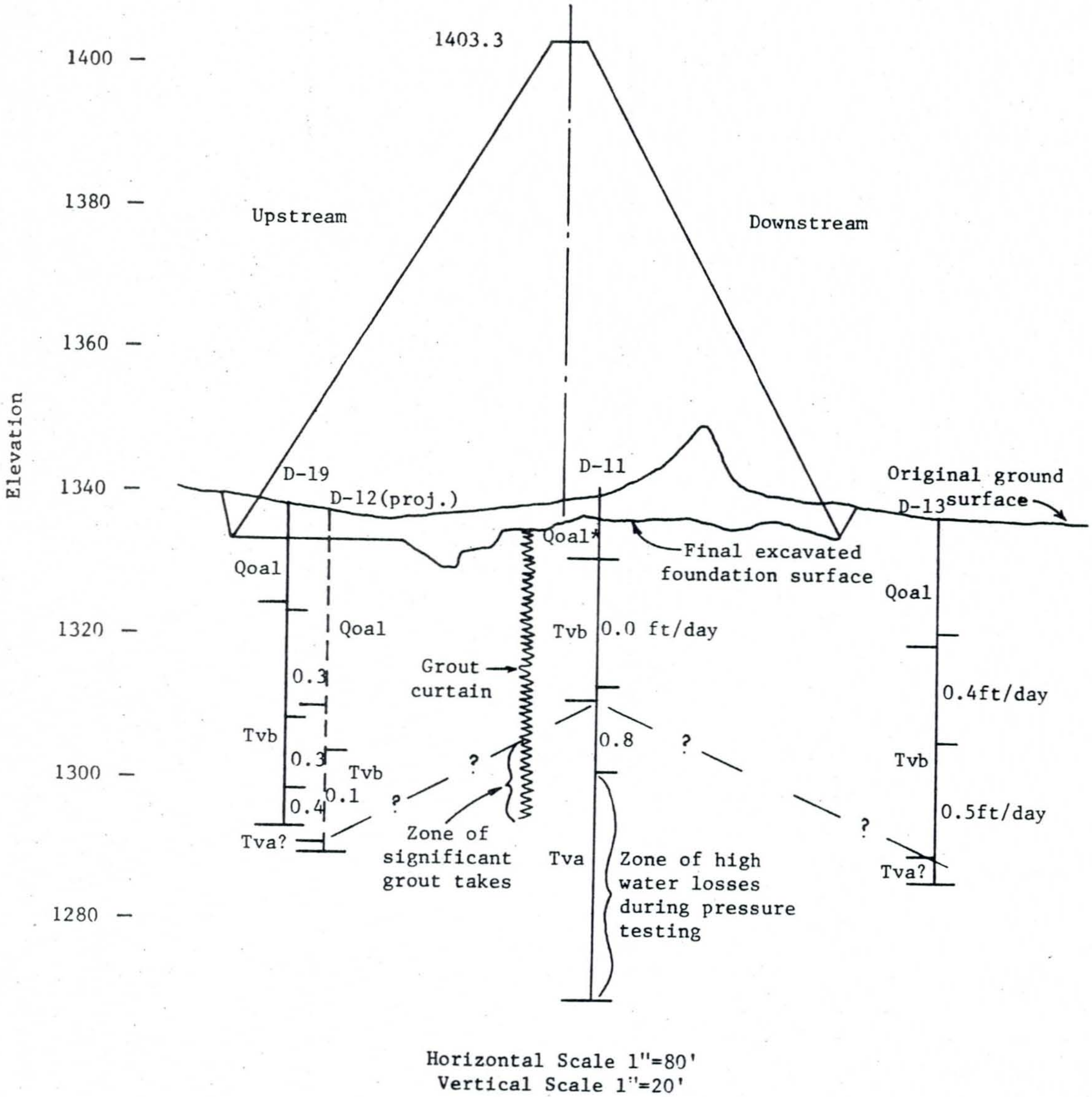


Photo 91
Detail of core from hole D-13 (downstream side of embankment). Note vesicular basalt above 37-feet, and fine grained, re-healed andesite? below 47.5-feet. (9 June 1982)



*Note: Qoal in D-11 is distinguished from Tvb on the basis of infilling between large volcanic rock blocks and may be considered Tvb for foundation purposes.

Figure 6. Foundation Cross Section, Station 13+20.

3.56 The log of the hole and the core from the hole did not suggest anything but highly fractured andesite. It was feared, however, that the very bottom of the hole might have encountered a large "cavity." In order to explore for such a void, a rotary percussion air-track drill was used to deepen the hole 6 feet to elevation 1269.5. During drilling there was no indication of a large void (rod drops). The drilling was indicative of highly fractured rock. During subsequent pressure testing, the hole accepted the maximum capacity of the pump (30 GPM) without building any back pressure. While drilling and pressure testing of nearby grout holes, no communication was established with D-11. Likewise, attempts to establish communication using smoke bombs were also unsuccessful (photo 94). Regardless, it is still believed that the high water losses and high grout takes in the andesite are the result of a porous rock mass with at least some interconnection between the fractures. The rock mass probably does not contain voids or cavities of large dimension and is covered at the abutment by the basalt block flow.

Geologic Structure

3.57 Other than the previously discussed structure within the basalt block flow, the geologic structure at the right abutment can only be inferred from isolated surface exposures and core hole data. As depicted in plan and profile on amended plate 3, it appears that the basalt block flow, Tvb, is in gradational contact with the andesite, Tva, and the contact at depth follows the surface topography. The agglomerate, QTa, has limited occurrence along the margin of the block flow.

Photo 92
Lower portion of right abutment inundated by summer rain storm when diversion dike was overtopped. Maximum water elevation approximately 1339. (9 Sept 1981)

Photo 93
Whirlpool developed over D-11 when lower abutment was flooded. An estimated 35,000 gallons of water flowed into the hole. (9 Sept 1981)

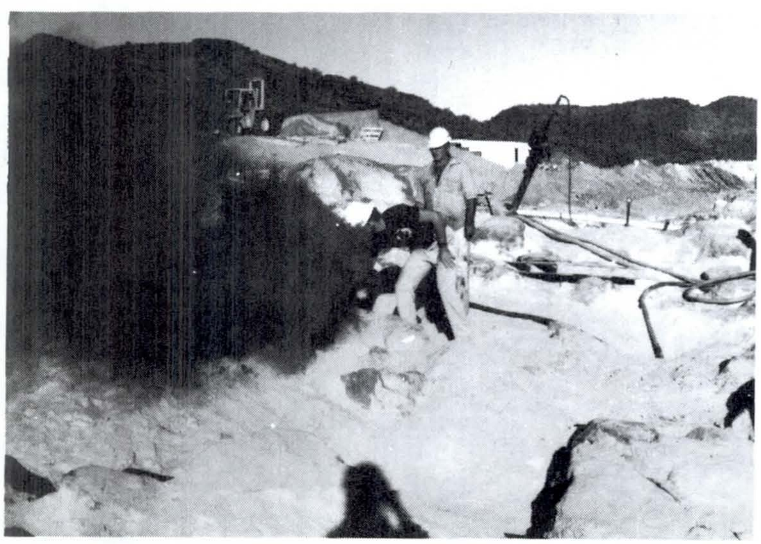


Photo 94
Attempting to establish communication between D-11 and second-stage grout holes using red smoke bomb. Attempt was unsuccessful. (29 Sept 1981)

3.58 No faulting was identified at the abutment other than the large-scale movement suggested by the steep contact between the older alluvium and the abutment bedrock. Such large-scale faulting would be associated with long-quiescent Basin and Range tectonism. The significance of the irregular and indistinct "contact" between the more regularly jointed block flow and the jumbled block flow exposed lower on the abutment remains unknown. If the lower portion of the abutment has moved, either by faulting, uplift, or sliding, in relation to the upper area, such movement is believed to be ancient and possibly contemporaneous with the late Tertiary volcanism.

FOUNDATION TREATMENT

3.59 The foundation treatment of the right abutment consisted of surface preparation prior to placement and compaction of embankment materials and subsurface pressure grouting. Surface preparation was required in order to (1) remove materials which might provide a seepage path along the foundation/embankment interface and (2) create a relatively uniform surface to facilitate compaction of embankment (core) material. The grouting was required in order to explore the subsurface geologic conditions as well as to create a relatively impermeable curtain under the core of the dam. The curtain will reduce near surface seepage through the abutment and increase the travel path for whatever seepage might occur. Both the surface preparation and the grouting, as designed and implemented, were more than adequate to satisfy these requirements. The short duration of maximum flood pool and the conservative embankment design, which includes an upstream core blanket and downstream gravel drain, also add to the protection against abutment related seepage problems.

Surface Preparation

3.60 The preparation of the abutment surface as a suitable foundation involved (1) cleaning, (2) dental concrete, and (3) slurry grout. Also, specialized material placement techniques and procedures were followed to insure the best possible foundation/embankment bond.

3.61 **SURFACE CLEANING.** Since the tuffaceous infilling between the basalt blocks had been determined to be suitable foundation material, the surface cleaning consisted primarily of excavating only soft infilling from the fractures and removing dirt and loose rock fragments from the remainder of the surface. Even the most competent infilling material was friable; and it would have been possible, but unnecessary, to clean each fracture to a depth at least three times the width as required under section 2D, paragraph 5.2.1. of the contract specifications. Instead, each fracture or pocket of infilling was "raked" with a rock pick as necessary to loosen the surface and expose competent material (photo 95). Pockets of loose rock were removed with shovel and broom (photo 96). Each area was then blown clean using a high pressure air hose (photo 97). Before any further surface treatment or placement of embankment materials, the entire abutment surface was inspected and approved by Engineering Division, Geotechnical Branch personnel as the cleaning, treatment, and placement proceeded up the abutment.

3.62 **SURFACE TREATMENT.** Originally, under the core and core blanket, the plan was to treat deep cracks and small overhangs (photo 98) with slurry grout. Dental concrete was to be used in isolated deep depressions (photo 99) in lieu of hand placement and compaction of core material. The foundation under the downstream random shell and gravel blanket had no requirement for cleaning other than removal of debris and thick accumulations of dust and dirt from the bedrock surface. (As recommended by technical experts O'Neill and Sciandrone, attachment 4, a filter blanket of sand was added beneath the gravel drain on the abutment (photo 100) to preclude migration of the fine infilling material into the drain). (See amended pl. 13.) After portions of the lower abutment had been treated in this manner, the remainder of the foundation under the core and core blanket was essentially covered with dental concrete (photos 100 and 101). This was done instead of hand compaction within the deep depressions and to facilitate equipment placement and compaction over the originally irregular surface. The final abutment surface, especially above the outlet works' conduit, resembled islands of basalt in a sea of concrete (photo 102).

3.63 **Dental Concrete Placement.** The areas in which dental concrete was placed were cleaned as previously described and wetted immediately before the concrete was poured (photo 103). A crane bucket was used to place the low-slump, 3/4-inch, 1000-lb/in^2 minimum, concrete, which was then adequately vibrated, especially along the rock contacts (photo 104). Feather edges were avoided and a 6-inch minimum thickness criteria was followed. The surface was screen-tamped to provide a rough texture which facilitated bonding with the core material. Surface irregularities were maintained as much as possible while still allowing efficient equipment compaction. A total of 910 cubic yards of dental concrete was used at the right abutment. (See photos 105 and 106 for a comparison of abutment surface before and after dental concrete placement.)

3.64 **Slurry Grout.** Because of the extensive amount of dental concrete utilized, only 34 cubic feet of slurry grout was placed. A thick mix consisting of one part sand to one part cement was mixed by hand (photo 107) and placed in cleaned and wetted cracks (photos 108 and 109). Most cracks above the level of dental concrete were filled with wet-of-optimum core material instead of slurry grout. The material was squeezed into the spaces during compaction with a front-end loader. Cracks treated in this way were located within individual rock blocks and did not have any lateral continuity.

3.65 **Embankment Placement.** Although the placement and compaction of the core material on the abutment is not a part of the surface treatment per se, the two must be compatible in order to insure a suitable abutment/embankment bond. Therefore, the procedures used, approved, and supervised by Geotechnical Branch personnel will be briefly discussed. Core material was placed and compacted as follows.

a. A Caterpillar 980C front-end loader placed 6-inch to 1-foot lifts of select wet-of-optimum core material on the cleaned and wetted abutment surface or on previously compacted and scarified material (photos 110 and 111).

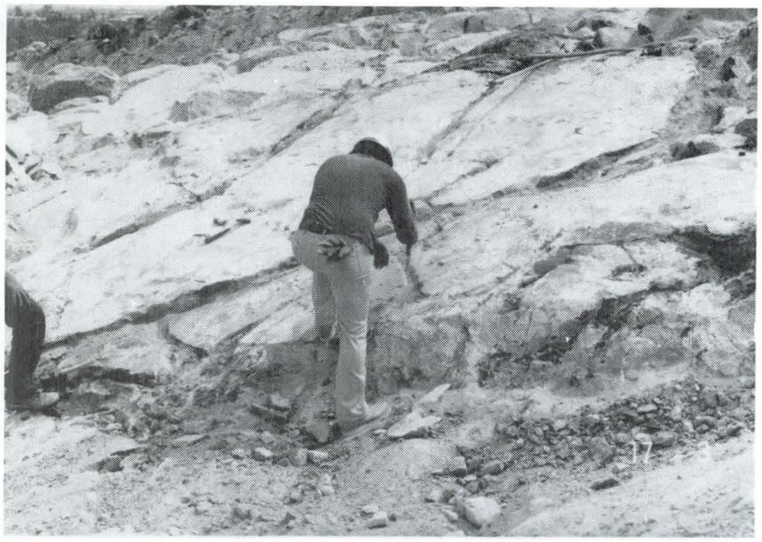


Photo 95
Laborer using rock pick to remove loose infilling material from cracks prior to air cleaning. Station 10 + 85, 25 feet left of centerline. (17 Mar 1981)

Photo 96
Laborers using shovels and brooms to clean areas delineated for dental concrete on lower portion of right abutment, station 13 + 50. Bottom of exploration trench at upper left. (3 Nov 1981)

Photo 97
Typical air cleaning operation using 2-inch air hose and nozzle and 850CFM compressor. Note how small cobble-size rock fragments are easily blown away. Photo taken during cleaning of floor of outlet trench prior to placement of leveling slab. (20 May 1981)

Photo 98
Typical slurry grout placement in small crack/overhang on lower abutment surface, station 12 + 60. Note abundant dental concrete in surrounding areas. (9 Nov 1981)

Photo 99
Portion of freshly placed 16 cubic yards of dental concrete below station 13 + 30 on lower abutment. Note grout nipples diagonally across photo from lower left. (4 Nov 1981)

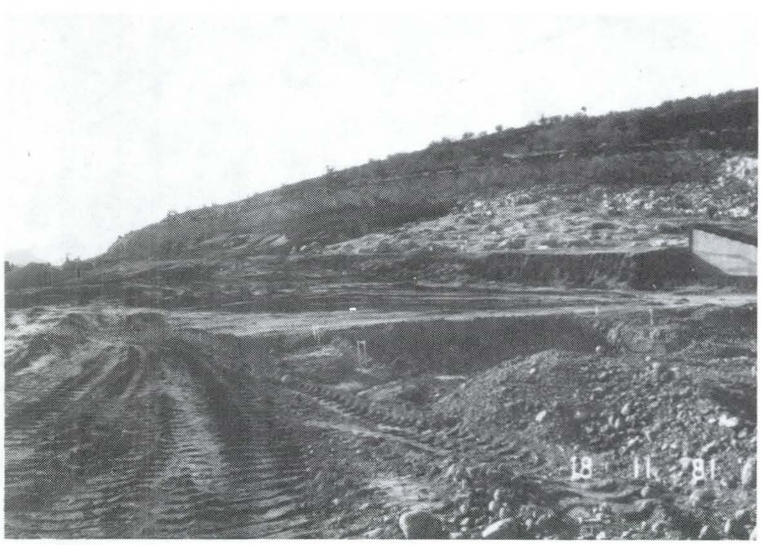


Photo 100
Status of embankment construction at right abutment on 18 Nov 1981. Core blanket has been placed up to outlet conduit; mass dental concrete covers lower half of exposed abutment; sand filter material placed downstream from dental concrete. (18 Nov 1981)

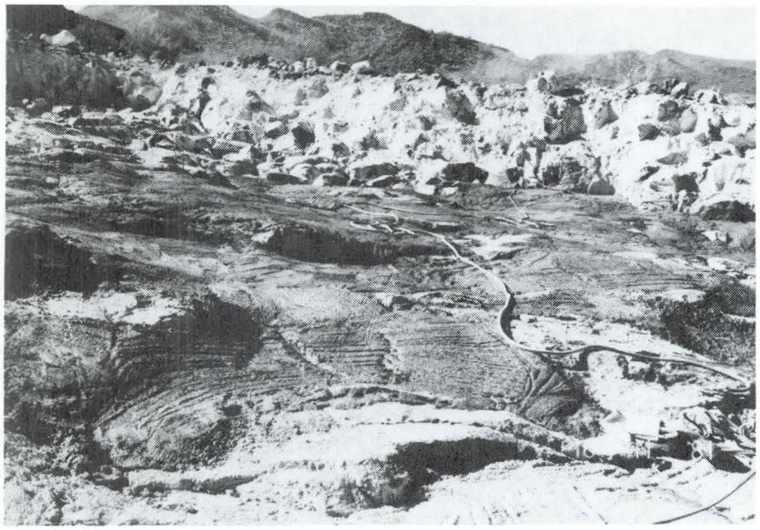


Photo 101
Portion of 100 cubic yards of concrete freshly placed between stations 11 + 00 and 10 + 50 under core blanket portion of abutment. (16 Nov 1981)

Photo 102
Dental concrete placed between stations 10 + 20 and 9 + 70. Sand filter blanket at top of photo, core material at upper left. (3 Dec 1981)

Photo 103
Typical area cleaned, wetted, and ready to accept dental concrete. Area at top of photo has not yet been air-cleaned. (29 Sept 1981)

Photo 104
Initial placement of dental concrete on the lower abutment surface. Note cleaned and wetted surface in foreground and vibrator being used along the rock contact. Note drilling and pressure testing for grouting at upper right.
(29 Sept 1981)

Photo 105
Area on abutment between stations10 + 50 and 10 + 75 along grout curtain (note grout nipples), cleaned, wetted, and ready for dental concrete.
(24 Nov 1981)

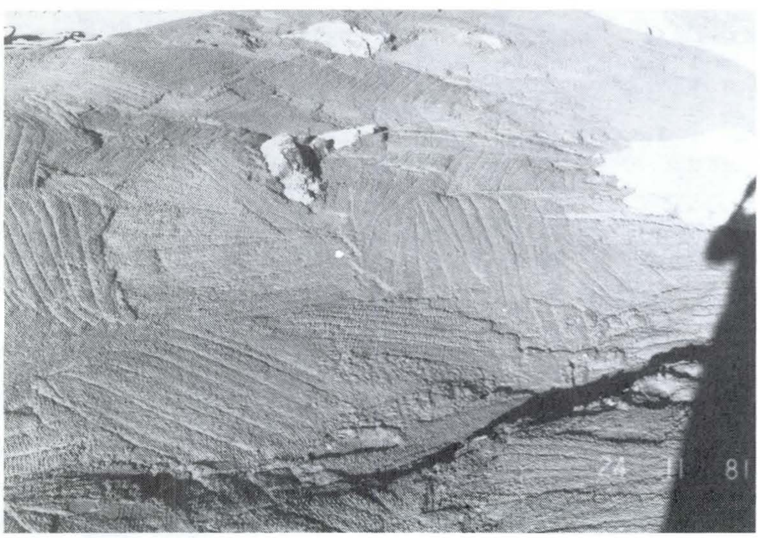


Photo 106
Same area as shown in photo 105 after placement of several loads of dental concrete. Note screen tamped surface texture. (24 Nov 1981)

Photo 107
Mixing grout slurry at a ratio of one part sand: one part dry cement (one sack of cement yielded 1½ cubic feet of slurry.
(6 Nov 1981)

Photo 108
Applying slurry grout into cleaned and wetted crack. Grout was worked into crack using a piece of wood lath.
(6 Nov 1981)

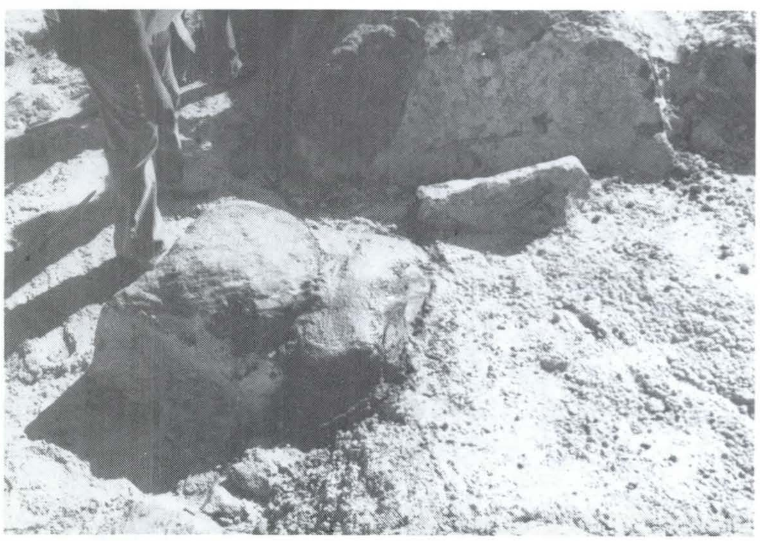


Photo 109
The finished product after slurry grout application. Note abundant dental concrete in surrounding area.
(6 Nov 1981)

b. All the surface was rolled at least eight times using the front tires of the fully loaded 980C (photo 112). Special effort was made to compact adjacent to the protruding rocks.

c. A hand-held "Whacker" was used to compact in areas inaccessible to the loader (photo 113).

d. Each compacted lift was scarified by raking with the teeth on the loader bucket (photo 114).

e. When a sufficient thickness had been accumulated on the abutment, it was further compacted with a sheepsfoot roller (photo 115).

The same procedure was followed from the bottom of the exploration trench (photo 111) to the top of the abutment (photo 116). Over 40 sand-cone density tests were conducted on the abutment (photo 117) and a test pit was hand-excavated in a randomly selected area to check the core/foundation bonding. The density testing showed an average percent compaction of 100.7 percent with a moisture content of 2.4 percent above optimum. The bonding of the core material with the foundation in the small test pit was observed to be excellent (photo 118).

Subsurface Pressure Grouting

3.66 **SPECIFICATIONS/CONTRACT (GENERAL).** Specifications for the drilling and grouting at Adobe Dam were written with close adherence to the guide specification, CE 1305.1, dated October 1959. (See attachment 5, section 20 of the Contract Specifications.) Payment items, estimated amounts and actual contract prices were as follows:

Mobilization/demobilization	1 ea.	$8,000
Drill exploratory grout holes	100 lf.	$45/lf.
Drill grout holes	975 lf.	$15/lf.
Pipe for grout holes	100 lf.	$3 ea.
Drill set-ups	50 ea.	$50 ea.
Wash and pressure test	50 hr.	$120/hr.
Grout pump connections	50 ea.	$35 ea.
Place grout	200 sacks	$30/sack

These estimates comprise approximately 110 percent of the propsed grouting scheme shown on amended plate 11. Therefore, it was implied that:

a. Twenty-foot primary hole spacing would be adequate above station 11+75.

b. Split-spacing of grout holes would generally not be required.

c. Grout takes would average approximately 0.2 sack per linear foot of hole.

3.67 In other words, it was assumed that the bedrock foundation was relatively impermeable and that large grout takes would not be the rule. These assumptions were based upon the core hole data presented on amended plate 6. The information from core holes D-9, D-10, D-11, and D-23 indicated that the abutment bedrock foundation consisted of 20 to 50 feet of relatively impermeable, fractured but infilled basalt underlain by highly fractured and relatively more permeable andesite.

Most of the grout curtain was above the basalt/andesite contact. Even though the underlying andesite transmitted large volumes of water during pressure tests, the high recovery percentage and the appearance of the core suggested that the fractures in the rock mass were smooth, regular, and without significant separation between the joint blocks.

3.68 **PRE-MOBILIZATION.** W.G. Jaques Company of Des Moines, Iowa, was selected by the contractor to perform the foundation grouting. Due in part to the delays in the excavation on the west abutment and in part to the grouting subcontractor's schedule, the grouting program was delayed until August 1981. The upper abutment area had been cleaned and ready to grout since May 1981. The completion of the grouting was on the critical path of the contractor's scheduled completion of the dam, and therefore, correspondence between the Government and the Jaques Company was initiated several months in advance of the expected mobilization in order to facilitate a timely start and efficient operation. Problems posed by the irregular abutment surface and the extreme Arizona summer temperatures were of primary concern. In the correspondence, Jaques requested and was granted permission to use a Longyear skid rig in lieu of the CP-8 specified to do the exploratory drilling (in actuality for sake of expediency, a CP-65 drill was used to drill the core holes). Jaques also proposed locating the grout plant on top of the completed outlet conduit to do all the abutment grouting, exceeding the 200-foot maximum distance to some grout holes as specified in section 20.3.4 of the contract (attachment 5). Logistically it would have been very difficult, expensive, and time-consuming to locate the plant and deliver cement within the specified distance. Permission to exceed the 200-foot limit was granted, therefore, even though it was feared that the excess line length combined with the high expected air temperature could cause grout to set up in the supply lines. The Geology Section requested that the subcontractor propose a methodology for collecting and diverting the drill and wash water in order to avoid undue degradation of the cleaned abutment. The subcontractor proposed using a flexible hose and PVC pipe system to divert water away from the abutment. On 24 June 1981, at a meeting between the prime and sub-contractor and Government representatives, these and other issues related to the grouting program were discussed.

3.69 **MOBILIZATION.** Jaques personnel and equipment arrived at the site on 10-11 August 1981. Personnel consisted of a project manager, a grouting superintendent, a drilling superintendent, and four local laborers/drillers. Several weeks later the grouting superintendent was transferred and not replaced. Equipment consisted of two CP-65 drill rigs, one grout plant, and all required appurtenances. Before any work was actually begun, a meeting was held between representatives of the resident engineer, the Geology Section, Sundt, and the Jaques project manager, Fred Scharmota, and vice president, Dave Butler. The primary item discussed was the reporting procedure to be followed. Each morning, the Jaques project manager and the Corps project geologist would agree upon the payment items from the previous day. This information was to be summarized by the grouting project manager on a Daily Shift Report for Drilling and Grouting. In addition, copies of drill reports for all holes drilled the previous day were to be transmitted to the Corps. These drill reports included hole number, stage depth, and interval drilled, and driller remarks concerning the drill action, color, and amount of return water and any unusual conditions encountered. In addition to these normally prepared reports. Jaques would submit daily Summary of Change Work Reports detailing the amount of extra man-hours required over "normal" operations because of hook-ups to the waste-water collection system and transportation of the drills over the rough abutment surface.

3.70 **DRILLING AND GROUTING (GENERAL).** The equipment used, procedures followed, and terms defined were, in some cases, not in total agreement or completely explained in the contract specifications. (See attachment 5.) These deviations can be summarized as follows:

3.71 **Equipment.** The following equipment was used.

a. Grout plant (section 20.3.3). A Jaques Over/Under Portable Grout Plant Model GP-16 was used during all grouting operations. It was equipped with a 16-cubic-foot mixing tub, an 18-cubic-foot agitator, and Moyno pump. This plant totally satisfied the specification requirements. A coarse mesh screen between the mixer and agitator was used to clean the grout in lieu of the 100-mesh vibrating screen, as specified in section 20.3.3.6.

b. Gages (section 20.3.3.9). Pressure gages graduated from 0 to 50 lb/in^2 were used to perform all pressure testing and grouting. Gages with higher pressure capacity were not needed.

c. Grout header (section 20.3.4.1). Jaques used a grout header substantially the same as that specified except that the pressure gage was on a vertical extension rather than on an elbow. (See fig. 7.)

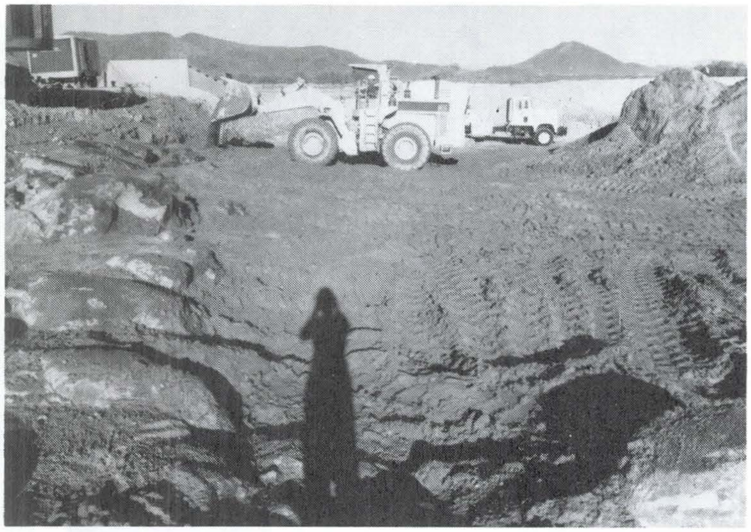


Photo 110
CAT 980C loader "sprinkling" a thin lift of select and wet-of-optimum core material on previously compacted and scarified core material just below the outlet conduit backfill. Stockpile of core material at right; crane used to place dental concrete at left. (16 Nov 1981)

Photo 111
Front-end loader leveling out first lift of core material against abutment in exploration trench. (9 Nov 1981)

Photo 112
Fully loaded CAT 980C compacting core material against abutment in exploration trench. Note how close tires work against protruding rocks. (10 Nov 1981)

Photo 113
Laborer using Wacker to compact very thin lifts of select core material adjacent to large protruding rock; dental concrete at lower left. (16 Nov 1981)

Photo 114
Loader scarifying compacted lift of core material in exploration trench. (10 Nov 1981)

Photo 115
Sheepsfoot tamper compacting material in exploration trench which had been previously placed and compacted with a front-end loader. Representatives of LADO and SPD geotechnical branches inspecting compaction of material at lower right. (9 Nov 1981)

Photo 116
CAT 980C loader compacting core material against extensively dental concreted area near top of right abutment. Laborer at left blowing off loose rock fragments prior to material placement. (3 Dec 1981)

Photo 117
COE laboratory crew taking sandcone density test in loader - compacted core material at station 12 + 65, 32-feet left of centerline. Compaction at abutment averaged 100.7 percent @ + 2.4 percent moisture content. (16 Nov 1981)

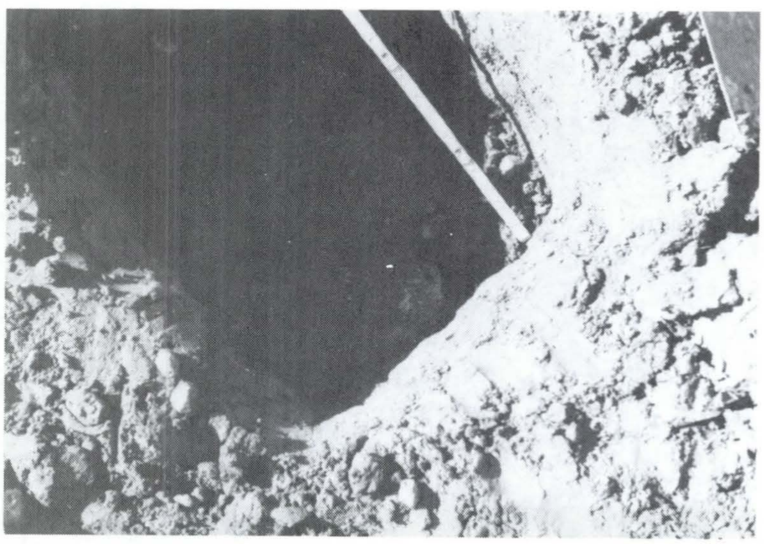


Photo 118
Bottom of hand-excavated test pit approximately 1-foot below the surface at station 12 + 50, 5-feet right of centerline. Note intimate bond between the foundation rock and the core material which was placed and compacted with the CAT 980C. (17 Nov 1981)

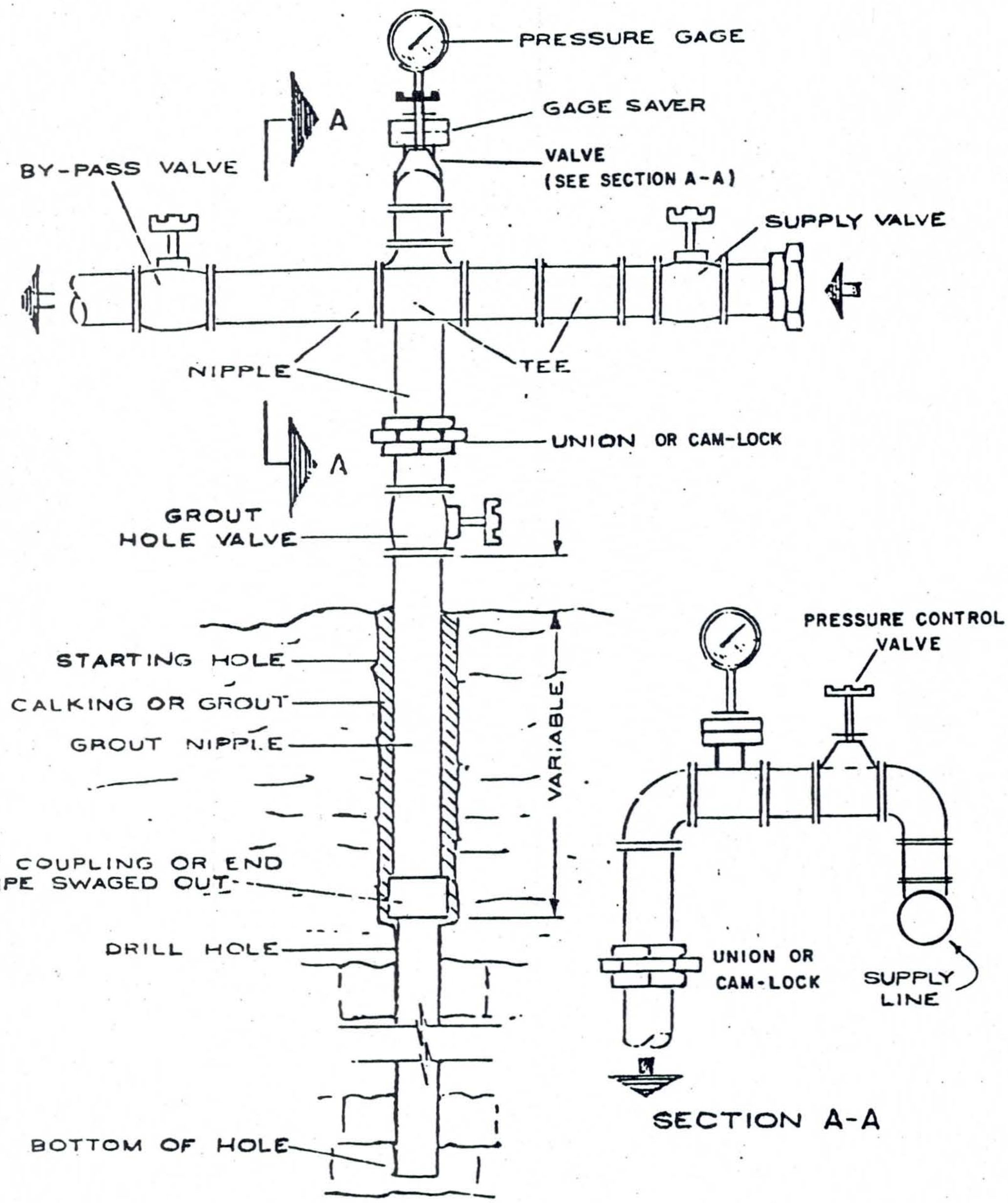


Figure 7. Grout Header.

d. Drills for grout holes (section 5.3). The contractor elected to use pneumatically actuated, rotary CP-65 drills equipped with EX-size diamond plug bits. This equipment was in accordance with the specifications. In addition, the drill return water was collected at the collar of the hole and directed away from the abutment surface through a flexible hose connection between a 4-inch PVC trunk line and a tee-fitting on top of the threaded grout pipe.

e. Drill for exploratory core holes (section 20.3.2). The contractor was allowed to use one of the CP-65 drills to perform the exploratory coring. Although a skid rig would have undoubtedly yielded better core, the logistic problems of mobilizing a larger coring rig to drilling sites on the rough surface were judged to be more critical.

3.72 **Procedures.** The grouting procedures used are described in the following paragraphs.

a. Grouting sequence (sections 20.5.1, 5.4.1, and 5.5.6). Grouting in the section above the outlet conduit (upper abutment) was performed first. Within each section, grouting proceeded from the bottom up within a given series of holes. In the upper section, the primary holes on 20-foot centers were drilled to their full depth before any secondary holes were drilled (as specified in section 20.5.5.6). In the lower section, however, the first zone (0 to 20 feet) in both primary and secondary holes on 10-foot centers was grouted before any holes were deepened into the second zone (20 to 40 feet) as specified in section 20.5.4.1. The responsibility of redrilling of grout necessary to split-space the lower zone was assumed by Jaques.

b. Split-spacing (section 20.5.4.4). Based upon the recommendation of Alan O'Neill, a technical expert under contract to the Corps, and the concurrence of SPD and OCE, the upper-zone primary holes on 20-foot centers were routinely split-spaced with secondary holes regardless of the grout takes in the primary holes. (See attachment 4.) The criteria for further split-spacing was when grout takes exceeded 0.3 sack per linear foot of hole.

c. Grout set-up time (section 20.5.7.2). The contractor was allowed to deepen grouted holes and to drill within 50 feet of a grouted hole after a period of 16 hours instead of the 24 hours specified. This allowed drilling on the morning following grouting performed on the previous afternoon.

d. Washing-out grout (section 20.5.3 and 5.5.2). In lieu of washing out grout before drilling deeper stages, the contractor elected to redrill at his own expense.

e. Washing (section 20.5.6). Immediately upon the completion of drilling to each full zone depth, the drill bit was lifted slightly off the bottom of the hole and the drill pump used to wash the hole for approximately 10 minutes or until the return water was clear. The capacity of the waste-water collection system limited the volume of wash water. When drill return water was lost, the full capacity of the drill pump was used to wash the hole for up to 15 minutes in order to attempt to re-establish circulation.

f. Pressure testing (section 20.5.6). Each hole was pressure-tested before grouting. Holes which could not be grouted on the same day that they were pressure tested were either retested or washed for several minutes before grouting. Only one pressure was used for each test interval. For holes in the upper zone, one-half of the bottom depth in feet was established as the gage pressure in pounds per square inch at the collar of the hole. Below 20 feet, the tests were usually conducted at a surface pressure equal to the depth in feet at the top of the stage. Gage pressures, however, never exceeded 30 lb/in^2. Each test was conducted for an average of 5 to 10 minutes with the duration dependant upon the rate of take and the time required to establish a uniform flow rate.

g. Grout plant location (sections 20.3.8.5 and 3.4). As previously noted, the grout plant was situated on the completed outlet conduit along the grout curtain axis. The maximum distance between the plant and a grout hole was 260 feet.

h. Deferred grouting (section 20.5.8.3). Grouting in secondary holes in the upper section and in all holes in the lower section was deferred if the pressure test data indicated a computed permeability of less than 0.1 foot/day. Grouting of primary holes in the upper section showed that 6:1 grout could not be injected into holes meeting this criteria.

i. Mix ratios (sections 20.5.8.1 and 5.8.3). Generally, all grouting was begun at a 6:1 water:cement ratio regardless of the suspected eventual take. Mixes were thickened incrementally to preclude premature stoppage.

j. Grouting pressures (section 20.5.8.2). The maximum grout pressure used was usually the same as the pressure used during water testing. This pressure was based on the criteria of 1 lb/in^2/ft of depth in feet to the middle of the first stage and lb/in^2/ft to the top of successive stages. All references to grout pressure refer to the gage pressure at the header and do not include the weight of the grout column.

k. Interruption in grouting (section 20.5.8.3). Generally, once grouting had been started on a hole, it was continued to completion without any interruption. However, while grouting at zero gage pressure with no return flow to the agitator tank, thick grout was periodically allowed to circulate through the lines for periods of approximately 1 minute in order to keep the grout from setting up in the lines.

3.72 **Definitions.** Deviations from the terms used in the contract specifications are explained as follows.

a. Zone (section 20.5.4.1). The first zone extended 20 feet downward from the ground surface. Grout pipes were generally embedded one foot below ground surface into sound rock. The second zone was also 20 feet in length. Primary holes, therefore, were either 20 or 40 feet deep (40 feet between stations 11+40 and 13+77) as opposed to the intermediate 30-foot depth shown for some holes on the contract plans (amended plate 11).

b. Section (section 20.5.4.2). Grout sections as defined in the contract specifications were never established. Instead, procedures outlined in other portions of the specifications were used to establish when and where drilling and grouting could be performed. That is, no drilling was permitted within 50 feet of a grouted hole regardless of the zone grouted or to be drilled until at least 16 hours had transpired since grouting. The abutment was divided into two parts, or "sections," by the outlet conduit. All grouting in the upper "section" (station 11+89 to 9+51) was completed prior to drilling and grouting in the lower "section."

c. Waste grout (sections 1B 22.2.7 and 20 5.8.3). Under the Measurement and Payment section of the contract specifications, it was stated that the payment for placing grout ($30/sack) included compensation for "furnishing cement, proportioning and mixing," as well as injecting the grout. Therefore, grout wasted requires three out of four procedures covered under this payment item. Under section 20, however, it states that waste will be paid for at the applicable material cost ($4.50/cwt). Since both sections seem to apply, grout mixed but not injected was paid for as waste grout at a negotiated price which was approximately 1/4 of the unit cost for placing grout.

d. Grout refusal (section 20 5.8.3). Refusal was defined as no measurable take (0.1 cubic foot) in a 5-minute period at the maximum pressure required for the stage.

3.74 **UPPER ABUTMENT GROUTING.** Most of the abutment surface above the outlet works had been cleaned as required in section 20.2.2.1 of the contract specifications since late April 1981 (photo 119). Therefore, the upper abutment was grouted first while the lower abutment surface was being excavated and cleaned (photo 120). The rock above station

9+85 was very fractured and locally crumbly, and removal of all loose or unsound rock fragments would have not been practical. This area, however, was blown as clean as possible before grouting. The grout holes were located as close as possible to the calculated middle of the trapezoidal core zone on 10-foot centers. The contract plans originally called for primary holes on 20-foot centers above the station 11+60 with split spacing only as required by high grout takes. Because of the unexpected blocky and seamy appearance of the abutment, and based upon the recommendation of an independent technical expert, Alan O'Neill, this hole spacing was reduced to 10 feet for the entire abutment. Exact locations were based upon local rock conditions and all primary and secondary holes exclusive of 9+51S and 9+62P were angled approximately 60 degrees out-of-slope in order to intercept the steeply dipping in-slope fractures above station 11+20. (See pl. 3, for the plan and profile of the entire grouted abutment.)

3.75 The grout pipes were embedded approximately 1 foot into large basalt rock blocks where possible and sealed with molten sulfur. Most were within 3 degrees of the specified angle. Drilling with the CP-65s (photo 121) was uneventful even though the machines were old and somewhat difficult to transport over the rough abutment surface. The waste-water collection and diversion system was relatively easy to hook up and worked well diverting the drill water (photo 122). Although the return water was difficult for an inspector to observe, the driller could at all times monitor the flow. Upon completion of drilling, each hole was washed for several minutes or until the return was clear. On the day of grouting, holes were pressure-tested and permeability values computed according to the relationship $K = \frac{QC_p}{H_t}$ where K is the permeability in feet per day, Q is the flow in gallons per minute, H_t is the total head in feet acting at the middle of the test section, and C_p is a conversion factor which takes into account the length of test section and size of hole. The permeability value obtained assumes a homogeneous test section and is, therefore, only a relative measure of holes in the same rock mass. In stages where drill circulation was lost, permeabilities were not computed because the effective length of the zone of water loss could not be accurately measured. All primary holes were pressure-grouted according to the specifications or procedures as previously discussed. Secondary holes and second-zone primary holes were deferred from grouting if the computed permeability was less than 0.1 foot/day.

3.76 The foundation grouting of the upper abutment was begun on 11 August and completed on 21 September 1981. The plan and profile views of all grouting are depicted on plate 3, and the pressure test and grouting data is tabulated in table 3. Grouting between station 12+00 and station 10+00 on the abutment was very much as anticipated based upon the pre-construction exploration and the appearance of the foundation surface. In this reach, 20 primary and secondary holes on 10-foot centers were drilled and grouted. A total of 12.1 sacks of thin grout were injected with an average take of 0.025 sack per linear foot of hole. Calculated permeabilities ranged from 0.1 to 0.9 ft/day with an average of 0.3 ft/day. This average value coincides exactly with the average permeabilities calculated for the infilled basalt block flow during pre-construction exploratory coring.

3.77 At the uppermost portion of the abutment, above station 10+00, the foundation conditions and, therefore, the results of the grouting were quite different. Complete drill circulation losses were common, and significant amounts of water were injected during pressure tests. Additionally, many holes "breathed"; that is, air would blow out during the afternon and suck in during the cooler morning hours. No measurement was made of the volume of air movement. Holes which exhibited this "breathing" phenomena also transmitted large volumes of water during pressure tests and generally were the only holes which accepted significant quantities of grout. Between stations 9+51 and 9+91, 16 grout holes, primary through quinary series, were required to establish an effective grout curtain to the 20-foot first-zone depth. A total of 757.9 sacks were placed; an average of 2.5 sacks/foot of hole. This value is 100 times greater than the average grout take between station 10+00 and the outlet conduit.

3.78 As can be seen on Detail "A" of plate 3, the orientations of the holes were quite varied and the theoretical split-space method was not used to establish a grout curtain (photo 123). There were several reasons for the unorthodox placement of the grout holes. The rock at the surface was severely fractured, locally crumbly, and the rock blocks were not stabilized by tuffaceous infilling in the fractures. The grout pipes could only be securely seated in the larger, intact blocks which, in some cases, were offset from the grout curtain axis. Also, several distinct, steeply dipping fracture zones were apparent at the surface which seemed to coincide with the zones of high grout takes. The tertiary, quaternary, and quinary holes, therefore, were oriented specifically to intercept these fracture zones. Even though the surface was very fractured and open cracks were numerous, breakouts of grout in the upper stage were not common. When they did occur, the surface leaks were treated by caulking and puddling grout. Communication between the holes was also uncommon. Only holes $9+65Q_2$ and $9+74Q_2$ showed evidence of interconnection during pressure testing.

3.79 In order to check the effectiveness of the grouting methodology used and to explore the nature of the rock mass below the surface, a vertical NW-core hole (9+69.5E) was drilled through the area of highest grout take. The hole was drilled using a CP-65 drill to a depth of 19 feet without any circulation losses even though the core was highly fractured, essentially devoid of infilling, and grouted only below 12 feet. (See fig. 8 for the log of exploratory core hole 9+69.5E.) Core between 13.6 and 16.4 feet was approximately 75 percent grout with 25-percent fragments of cindery, reddish brown to grey basalt (photo 124). Explanations for the high grout takes and the "breathing" holes above station 10+00 are not certain. Based upon the surficial expression of the rock and the core recovered from hole 9+69.5E, however, it is reasonable to assume that the rock mass in this area represents a boundary of the latite block flow in which the tuffaceous infilling sands have not penetrated and, therefore, open fracture zones exist below the surface. These zones evidently contain a large volume of interconnected open area, which results in the holes breathing as the entrapped air mass expands and contracts because of temperature differentials and may or may not be connected with the exposed ground surface. They apparently trend sub-normal to the dam axis owing to the surface expression of the fractures and the limited grout travel along the grout curtain. Elsewhere on the abutment, if this condition were present, it could pose serious problems for seepage directly under the dam embankment. This area, however, is at the uppermost end of the abutment where reservoir heads are minimal. In fact, the surface elevation between stations 9+85 and 9+60 is several feet above spillway invert. Regardless of the noncriticality of this area, it is fully believed that an effective barrier to underseepage to a depth of 20 feet has been constructed. Furthermore, the badly fractured surface was eventually capped by several feet of dental concrete (photo 102) before placement of embankment core material.

3.80 **CONDUIT GROUTING.** Three grout holes on each side of the outlet conduit were drilled between outlet stations 74+25 and 74+53. Each hole was angled 70 degrees away from the conduit and intercepted the contact between the concrete plug and the rock foundation at a depth of approximately 10 feet. The purpose of these holes was to grout and seal any voids which may have existed at the concrete/rock interface. All holes were pressure-tested at 10 lb/in^2. Three were absolutely tight; the other three took between 0.4 and 0.9 gal/min. Permeabilities were not computed because most of each hole was in concrete. Grouting in the three tight holes was deferred, and each was backfilled with 1:1 grout. The remaining holes were pressure-grouted with a 6:1 mix and accepted 0.1 to 0.2 sacks (0.5 to 1.1 cubic feet of grout). The foundation under the conduit was grouted by secondary holes 11+89 and 12+30. Each hole was extended to 40 feet and angled to intersect approximately 8 feet below the conduit leveling slab concrete. It is believed that these eight holes verify a tight foundation alongside and under the conduit, at least in the central portion under the core zone.

3.81 **LOWER ABUTMENT GROUTING.** The layout of the grout holes on the abutment surface below the outlet conduit began on 31 August 1981, as soon as the area had been cleaned and approved (photo 125). Holes were located on 10-foot centers and along the core centerline as permitted by the rock relief. Every other hole was designated as a primary hole. All holes were angled approximately 60 degrees into the slope. (See the profile of the grouted abutment on pl. 3.) The orientation was changed to the more conventional direction because the steeply dipping fractures characteristic of the upper abutment were not apparent below station 11+20. Before any drilling had begun on the lower abutment, the area below elevation 1339 was flooded by heavy rains over Labor Day weekend (photo 92) and had to be re-cleaned. As previously discussed in paragraph 3.55, the open, pre-construction exploratory core hole D-11 was discovered. Since the problem area in the hole was approximately 60 feet below the surface, it was decided to proceed with the normal curtain grouting while determining how to treat the problem of D-11.

3.82 All holes in the upper zone were grouted prior to deepening any primary holes. Results of the grouting in the upper 20 feet were almost identical to that in the upper zone between stations 12+30 and 13+86. The average computed permeability was 0.4 ft/day. Grouting was deferred in 11 holes, and 6.5 sacks at 6:1 were placed in the remaining 12 holes. The average take in the grouted holes was 0.027 sack/ft. Four holes developed minor-to-severe surface leaks as the infilling was washed away from around the blocks (photo 126). The leaks were treated by caulking, puddling grout, and backfilling with thick grout.

3.83 Drilling into the second zone began on 28 September and all grouting was completed on 2 November 1981. Because of the condition encountered in D-11 and a re-evaluation of data obtained from D-10 and D-11, the second zone was deepened to 45 feet in order to intercept the suspected permeable rock mass below 40 feet. Also, it was intended that several holes extend as deep as 65 feet, down to the elevation of the suspected void in D-11. Since the core hole was only 30 feet downstream from the grout curtain, it was hoped that communication could be established between it and the deep grout holes. In this way, the extent of the problem zone might be delineated. After this exploratory exercise, the plan was to clean out the core hole, backfill with a sand:cement slurry, and then pressure-grout the deep grout holes. It soon became apparent, however, that significant grout takes would be commonplace well above the second zone depth of 45 feet. Between stations 13+00 and 13+85, all primary holes experienced complete loss of drill return water between 25 and 35 feet in depth. All holes exhibited the breathing characteristic of D-11 and took significant quantities of grout. The first stage in the second zone took 575 sacks of thick grout.

3.84 As drilling and grouting progressed, a definite pattern developed. The area of high grout take was restricted to between stations 12+80 and 13+80 and below elevation 1310. No direct communication existed between holes along the curtain, and stages of high grout take were as little as 1 foot apart in some holes. Cavities or large voids were not indicated during the grout hole drilling. In stages where large quantities of grout were subsequently injected, the holes transmitted large volumes of water during pressure testing and experienced the "breathing" phenomena. These characteristics were similar to the record of drilling and pressure testing in D-11. On 22 October, the second zone depth was officially changed from 45 feet back to the original 40 feet. With the concurrence of the SPD Geotechnical Branch, this action was taken for the following reasons:

a. The original intent of deepening the curtain was to be sure to intercept and explore the suspected permeable rockmass detected in D-11. At 40 feet the grout curtain penetrated approximately 10 feet into this permeable zone.

b. Regardless of the final depth of the curtain (up to at least 60 feet), a permeable zone would still exist below the curtain. Forty feet was considered to provide adequate underseepage protection consistent with the conservative dam design.

c. The permeable zone was "capped" by at least 25 feet of grouted and infilled rock with a low natural permeability.

d. The conservative design of the embankment and planned foundation surface treatment combined with the short duration flood pool made a deeper curtain unnecessary.

Photo 119
Final air cleaning along grout curtain and under core blanket prior to aerial photography and grouting.
(22 Apr 1981)

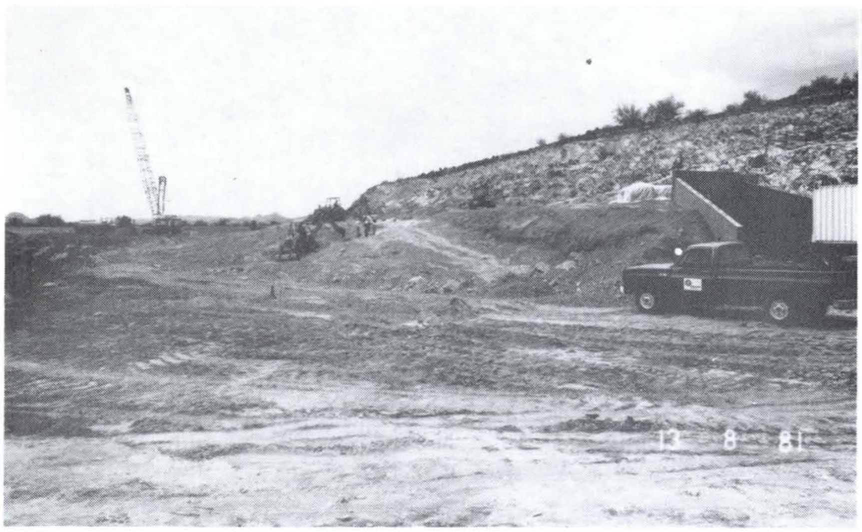


Photo 120
Dental excavation below outlet works (lower abutment) underway with backhoes and hand labor while drilling (for grouting) on upper abutment. Note grout plant set up on conduit and waste-water collection system on abutment surface.
(13 Aug 1981)

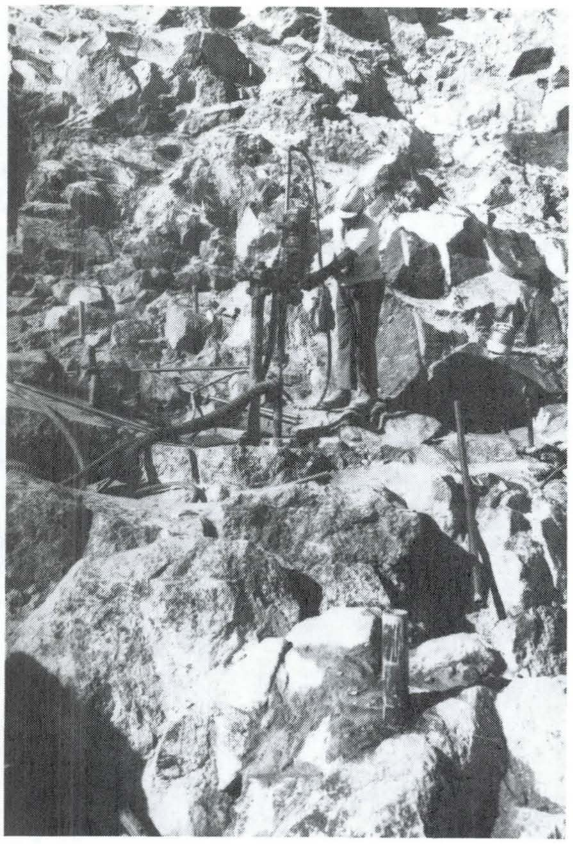


Photo 121
CP-65 drilling vertical grout hole (9 + 69Q) near top of right abutment. Note flexible hose connection for waste-water collection system. (10 Sep 1981)

Photo 122
Three-inch PVC waste-water collection truck line laid on upper abutment surface. "T" connection at bottom of photo leads to discharge pipe near outlet works intake portal. (24 Aug 1981)

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
DH-23		9+52	13R	1395	90		0-39				9/3	4.5	0	1:1	6.7		gravity backfill of open NX core hole
9+51	S	9+53.3	5.8	1393.7	90		0-18.5	14.9	10	4.2	8/27	31.1	5	5/1:1	109.4	x	
9+61	P	9+62.9	5.4	1385.1	90		0-11	5.5	3		8/18	0.3	7	3½:1	3.1		premature stoppage?
9+62	P	9+63.2	1.1	1384.4	90		0-20	1.6	10	0.4	8/21	0.1	10	6:1	0.7		re-drill of 9+61P
9+65	Q2	9+64.9	5.1	1383.9	74	N50E	0-10	4.5	5		9/15	0.2	5	6:1	1.0		grouted by 9+74
							10-20	7.0	10	2.9	9/16	1.4	10	6:1	7.9		
9+68	T	9+67.3	5.4	1383.7	69	N35E	0-16.5	18.0	10		9/3	113.0	10	6/1:1	252.5	x	lost circ. @14'
9+69	Q1	9+69.9	4.7	1384.4	90		0-11	10	6		9/9	3.3	8	4:1	15.2	x	
							11-12	17.5	4		9/11	81.1	5	6/2:1	266.2	x	
							12-20	0.4	10	0.2	9/16	0.4	10	6:1	2.3		
9+69.5	E	9+70.3	7.1	1383.0	90		0-8.5	3.9	5	3.8							hole cemented 0-8.5'
							8.5-19.0	4.1	10	1.7	9/18	1.2	10	6:1	6.7		
9+70	S	9+70.7	5.2	1384.0	75	N54E	0-10	14.3	5		8/27	24.6	5	6/2:1	94.1	?	
							10-20	18.8	10		8/28	327.7	10	5/1:1	718.4	x	lost circ. @11.0'
9+71	Q1	9+71.4	10.0	1383.2	77	N56E	0-5	5.8	2		9/9	14.4	1	4/2:1			severe surface leaks
							5-20	3.0	10	1.0	9/11	0.2	10	6:1	1.1		
9+74	Q2	9+74.0	10.1	1383.1	64	S45W	0-12	15.7	5		9/15	11.8	8	6/5:1	77.2	x	
							12-20	0.5	10	0.3	9/16	0.2	10	6:1	0.4		
9+75	T	9+75.1	2.4	1383.8	86	N57E	0-20	7.8	10	2.1	9/3	1.2	10	6:1	7.5		

See sheet 8 for explanatory notes.

Sheet 1 of 8

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
9+77	P	9+78.5	5.2	1383.8	61	N45E	0-5	surface leaks			8/18	3.4	2	5/2:1			surface leaks
							5-13	14.3	5		8/20	19.7	5	6/3:1	79.0	?	
							13-20	11.0	3		8/21	37.4	10	5/2:1	115.4		lost circ. @13.1'
9+81	Q2	9+81.4	12.5	1384.8	72	N46E	0-20	7.2	10	1.9	9/15	0.6	10	6:1	4.2		
9+83	Q1	9+83.3	13.0	1385.1	70	N42E	0-12	15.0	5		9/9	82.6	5	6/1:1	217.3	x	
							12-20	0.5	10	0.2		grouting deferred					
9+85	Q2	9+85.2	14.6	1385.4	72	N44E	0-20	4.0	10	1.1	9/15	0.2	10	6:1	1.1		
9+86	Q1	9+85.8	7.4	1382.6	47	S20W	0-20	10.0	10	2.9	9/9	0.9	10	6:1	6.0		
9+91	S	9+91.3	7.4	1382.5	63	N60E	0-20	13.3	10	3.3	8/27	2.1	10	6/4:1	14.3		
10+01	P	10+01.7	7.0	1382.8	59	N57E	0-20	0.7	10	0.2	8/18	0.2	10	5:1	1.2		
10+11	S	10+10.2	7.0	1381.1	64	N49E	0-20	2.6	10	0.7	8/28	0.4	10	6:1	2.1		
10+20	P	10+19.7	7.9	1379.5	61	N59E	0-20	0.5	10	0.1	8/18	0.1	10	5:1	0.8		
10+28	S	10+29.1	8.3	1378.5	54	N56E	0-20	1.7	10	0.5	8/28	0.3	10	6:1	1.4		
10+38	P	10+38.5	7.7	1378.9	63	N60E	0-20	0.2	10	0.1	8/18	0.1	10	5:1	0.5		
10+49	S	10+49	8.1	1377.0	60	N45E	0-20	0.8	10	0.2	8/28	0.2	10	6:1	1.5		
10+61	P	10+61.2	6.8	1375.4	61	N54E	0-20	0.4	10	0.1	8/17	0.1	10	6:1	0.5		
10+72	S	10+72.5	7.5	1373.4	65	N53E	0-20	3.2	10	0.9	8/28	0.8	10	6:1	5.0		
10+82	P	10+83.1	7.5	1371.6	59	N50E	0-20	2.9	10	0.8	8/17	8.0	10	6/4:1	45.2		

See sheet 8 for explanatory notes.

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
10+88	S	10+88.8	6.9	1368.6	71	N41E	0-20	0.3	10	0.1		grouting deferred					
10+95	P	10+95.9	10.6	1367.2	61	N57E	0-20	0.3	10	0.1	8/17	0.1	10	6:1	0.5		
11+08	S	11+08.9	12.5	1365.5	60	N70E	0-20	0.5	10	0.1		grouting deferred					
11+21	P	11+20.9	12.1	1363.3	63	N58E	0-20	0.1	10	0.1	8/17	0.1	10	6:1	0.4		
11+31	S	11+30.3	11.4	1363.8	64	N50E	0-20	0.9	10	0.2	8/26	0.1	10	6:1	0.5		
11+40	P	11+40.5	9.9	1359.9	60	N58E	0-20	0.5	10	0.1	8/17	0.1	10	6:1	0.5		
							20-40	3.2	20	0.4	8/21	0.4	20	6:1	2.4		
11+49	S	11+50.0	9.4	1358.7	63	N58E	0-20	1.7	10	0.5	8/26	0.2	10	6:1	1.1		
11+59	P	11+60.2	14.0	1358.2	58	N48E	0-20	0.4	10	0.1	8/17	0.1	10	6:1	0.8		
							20-40	1.6	20	0.2	8/21	0.3	20	6:1	1.7		
11+68	S	11+67.5	14.9	1357.2	62	N48E	0-20	1.1	10	0.3	8/28	0.1	10	6:1	0.8		
11+78	P	11+76.7	15.1	1353.2	62	N40E	0-20	0.9	10	0.2	8/17	0.1	10	6:1	0.7		
							20-40	0.8	20	0.1		grouting deferred					
11+89	S	11+89.3	16.0	1351.8	52	N61E	0-20	1.8	10	0.5	8/21	0.3	10	6:1	2.1		
							20-40	0.3	20	0.1		grouting deferred					
74+53L	C	11+96.0	25.8	1348.2	72	S40W	0-10	0.0	10			grouting deferred					conduit contact grout hole
74+39L	C	12+00.0	13.3	1348.4	70	S40W	0-13	0.7	10		9/2	0.1	10	6:1	0.6		"
74+25L	C	12+05.7	1.5R	1348.0	72	S40W	0-14	0.4	10		9/2	0.1	10	6:1	0.6		"

See sheet 8 for explanatory notes.

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
74+53R	C	12+12.3	30.6	1347.8	71	N55E	0–10	0.0	10			grouting deferred					conduit contact grout hole
74+39R	C	12+17.2	18.3	1347.5	70	N53E	0–13	0.9	10		9/2	0.2	10	6:1	1.1		"
74+25R	C	12+21.4	4.0	1347.2	71	N50E	0–13	0.0	10			grouting deferred					"
12+30	S	12+30.3	22.0	1345.4	51	S47W	0–20	2.8	10	0.8	9/23	0.3	10	6:1	2.1		
							20–40	5.6	20	0.6	9/25	0.4	20	6:1	2.4		intersects 11+89S under conduit
12+40	P	12+40.3	11.0	1344.7	60	S45W	0–20	2.4	11	0.6	9/23	0.4	10	6:1	2.2		
							20–40	5.3	20	0.6	10/3	0.7	20	6:1	4.7		
12+48	S	12+47.9	11.2	1343.2	63	S51W	0–20	0.4	10	0.1		grouting deferred					
12+58	P	12+58.6	16.9	1343.2	64	S44W	0–20	surface leaks			9/23	0.2	2	6/3:1			severe surface leaks
							20–40	0.8	20	0.1		grouting deferred					
12+68	S	12+68.6	15.2	1341.1	61	S58W	0–20	1.4	10	0.4	9/25	0.1	10	6:1	0.8		
12+81	P	12+79.6	16.2	1338.7	62	S50W	0–20	1.1	11	0.3	9/23	0.3	10	6:1	1.9		
							20–45	16.0	20	1.5	10/3	0.9	20	6:1	5.5		
12+92	S	12+90.2	14.7	1337.1	63	S56W	0–20	surface leaks				grouting deferred					hole tight below leaks; backfilled with 1:1 grout
							20–36	6.8	30	0.8	10/27	0.2	30	6:1	1.5		
13+00	P	12+98.9	19.6	1337.6	63	S52W	0–20	surface leaks			9/23	0.3	10	6:1	2.0		minor surface leaks
							20–33	13.0	20								
								4.5	20		10/3	0.3	20	6:1	1.8		after grt 13+60 @28'
							33–34	22.0	30								
								14.0	30		10/8	32.2	30	6/4:1	188.8	x	after grt 13+40 & 60

See sheet 8 for explanatory notes.

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
13+00	P						34-35	27.0	30								tested 10/10
								17.0	30								tested 10/12
								23.0	30		10/13	100.0	30	6/1:1	366.8	x	after grt 13+60@33'
							35-41	15.0	20								tested 10/16
								20.0	12		10/18	570.3	20	6/1:1	1183.0	x	after grt 13+20C@35'
							41-45	0.5	30	0.1		grouting deferred					
13+01	E	13+01.1	20.9	1336.7	90		20-34.9	6.8	20		11/4	0.7	30	6:1	4.7		lost circ. @34.9; packer set @20'
							34.9-37.8	21.0	20		11/5	12.3	10	4/1½:1	38.2		lost circ. @37.6'
13+03	T	13+03.4	19.6	1337.0	65½	S46W	0-20	0.8	10	0.2		grouting deferred					
							20-38	3.1	20	0.4	11/2	0.5	20	6:1	3.2		
13+10	S	13+09.3	20.3	1336.8	64	S45W	0-20	4.0	10	1.1	9/25	1.6	10	6:1	7.2		
							20-37	21	30		10/27	63.8	20	6/1:1	204.0	x	
							37-40	1.1	30	0.4	10/28	0.5	30	6:1	2.7		
13+15	T	13+15.3	19.8	1336.2	63	S51W	0-20	4.5	10	1.2	10/29	1.0	10	6:1	6.5		
							20-40	16	20								tested 10/30
								6.3	20		11/3	0.9	20	6:1	5.3		
DH-11		13+20	16R	1336.7	90		57-67	29.9	30		10/14	940.7	15	6/1:1	2022.5	X	zone of grout take probably @62'; see text
13+20A	P	13+21	18	1335.1	61	S50W	0-20	0.3	10	0.1		grouting deferred					lost hole; bit stuck
13+20B	P	13+21.7	16.5	1335.1	90		0-20	3.1	10	0.8	9/16	0.3	10	6:1	2.2		
							20-25	15.0	3		9/24	28.1	10	6/4:1	146.7		
							25-31	11.3	20								tested 10/29
								6.5	20		10/3	0.2	20	6:1	1.4		after grt 13+60@28'

See sheet 8 for explanatory notes.

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
13+20B	P						31-34	27.5	12								tested 10/6
								0.0	30		10/8	0.1	30	6:1	0.2		after grt 13+60@30'
							34-35	5.7	30		10/10	0.1	30	6:1	0.4		
							35-36	22.5	14		10/21	5.1	30	6:1	33.1	x	
							36-40	30.0	23		10/24	201.3	30	6/1:1	541.0	x	
13+20C	P	13+21.2	16.9	1335.3	63	S35W	0-20	1.3	11	0.3	10/10	0.1	10	6:1	0.6		
							20-34	11.0	20								tested 10/12
								7.5	20		10/13	5.5	20	6:1	35.5		after grt 13+60@33'
							34-35	4.5	20		10/16	5.7	30	6:1	36.8	x	
							35-36	15.5	22		10/21	104.8	30	6/1½:1	399.2	x	
							36-40	33.4	30		10/23	315.0	30	6/1:1	816.0	x	
13+28	T	13+27.7	16.7	1334.5	63	S46W	0-20	surface leaks				grouting deferred					hole tight; sealed leaks with 1:1 grt
							20-40	12	20		11/3	1.6	20	6:1	10.5		
13+33	S	13+33.2	16.7	1333.5	64	S40W	0-20	1.2	10	0.3	9/25	0.1	10	6:1	1.0		
							20-39	6.1	30	0.6	10/27	0.3	20	6:1	2.2		
13+40	P	13+39.1	9.9	1329.9	53	S50W	0-20	0.2	10	0.1		grouting deferred					
							20-29	7.0	20		9/29	0.8	20	6:1	5.2		
							29-30	21.5	20		10/7	169.7	30	6/1:1	547.0	x	
							30-44½	22.4	30		10/10	85.7	30	6/2:1	325.9	?	
13+51	S	13+49.3	13.1	1325.8	65	S40W	0-20	0.4	10	0.1		grouting deferred					
							20-41	23	30		10/27	211.3	20	6/1:1	375.0	x	
13+60	P	13+58.2	9.4	1321.2	67	S50W	0-20	1.6	10	0.4	9/24	1.8	10	6:1	12.0		
							20-28	16.5	11		9/30	183.2	20	6/1:1	455.8	x	
							28-30	20.5	20		10/6	147.8	30	5/1½:1	446.1	x	
							30-33	28.0	15		10/12	344.2	30	6/1:1	762.3		
							33-44	29.0	22		10/16	109.3	30	6/1½:1	315.2	x	

See sheet 8 for explanatory notes.

Table 3. Foundation grouting summary.

Hole No.	Class	Station	Offset	Elevation	Incl.	Bearing	Depth	Flow	Pressure	K value	Date Grouted	Take	Pressure	Mix Ratio	Volume Grout	Breathing	Remarks
13+67	S	13+66.8	12.6	1319.2	64	S50W	0-20	0.2	15	0.1		grouting deferred					redrill of 13+68S
							20-28	15	20		10/28	3.8	20	6:1	24.9		
							28-40	6.0	30	0.8	10/29	4.2	30	6:1	27.3		
13+68	S	13+68	13	1319.2	63	S45W	0-20	0.4	10	0.1		grouting deferred					hole abandoned; bit stuck in hole
13+77	P	13+74.7	12.8	1318.1	69	S50W	0-20	0.3	10	0.1		grouting deferred					
							20-25	24.5	11		9/30	184.4	20	6/2:1	540.9	x	
							25-45	15.0	30		10/6	10.4	30	6/5:1	62.8		lost circ. @44.7'
13+79	T	13+78.8	12.8	1316.0	69	S51W	0-20	0.0	10	0.0		grouting deferred					
							20-40	0.3	20	0.1		grouting deferred					
13+85	S	13+83.2	11.9	1314.2	69	S42W	0-20	0.0	10	0.0		grouting deferred					
							20-36	24	20		10/22	48.9	20	6/2:1	223.2	x	
13+86	T	13+84.0	11.9	1313.9	80	S50W	0-20	hole tight				grouting deferred					leaks around surface packer
							20-38	1.0	20	0.1		grouting deferred					

See sheet 8 for explanatory notes.

Sheet 7 of 8

Table 3. Foundation grouting summary.
(notes)

1. Hole numbers correspond to approximate dam stationing and were determined before final surveying.

2. D-23 and D-23 are pre-construction exploratory core holes which were incompletely backfilled after drilling.

3. Hole class designations are as follows: P = primary grout hole, S = secondary grout hole, T = tertiary grout hole, Q1 = quaternary grout hole, Q2 = quinary grout hole, E = exploratory NW-size core/grout hole, C = contact grout hole along outlet works concrete plug.

4. Grout hole locations are shown in dam stationing and feet offset left of dam centerline as determined by tape measure. Offsets right of dam centerline are designated by an R.

5. Elevations were surveyed by transit on 2 November 1981.

6. Depth increments are in feet. Stage depths less than 20 feet usually indicate loss of drill return water. (See note 14.)

7. Pressure test data is tabulated as follows: Flow (Q) is measured in gallons per minute. Pressure (Hp) is gage pressure in pounds per square inch measured at or near the collar of the hole. Permeabilities (K values) are measured in feet per day as determined by the formula $K = \frac{Q\ (30.65\ \ln 16\ L)}{Ht\ (L)}$ where Ht = Hp + average depth of interval tested in feet and L = length of test interval.

8. Permeabilities (K values) are not calculated where drill water return was lost because of the probable non-uniformity of the interval permeability.

9. Grout takes are measured in cubic foot bags of cement placed.

10. Grout pressures are gage pressure in pounds per square inch measured at or near the collar of the hole.

11. Mix ratios are the volume proportion of dry cement:water. Where more than one ratio is indicated the grout was thickened incrementally within the range shown.

12. Volume of grout placed is measured in cubic feet and includes the volume needed to fill the grout hole itself which requires 0.012 cubic foot per foot of EX-size hole and 0.049 cubic foot per foot of NW-size hole.

13. An (x) in the column marked "breathing" indicates that air was noticed either blowing out or sucking in the hole. A (?) indicates that the hole was not specifically checked for air movement, however, high water or grout takes indicate the hole was probably breathing.

14. Depths of drill water loss (lost circ.) are designated only if different from the bottom stage depth. Normally, drilling was stopped as soon as return was lost.

Sheet 8 of 8

Photo 123
Varied orientations of grout holes at upper abutment between stations 9 + 77 and 9 + 62. Spray paint can at location of exploratory core hole 9 + 69.5E. View looking downstream. This area eventually covered with several inches to several feet of dental concrete. (28 Sep 1981)

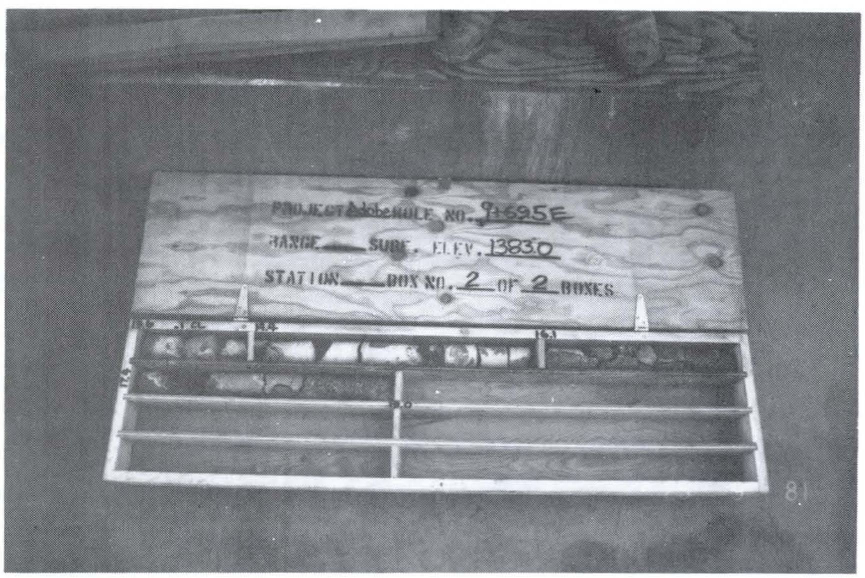
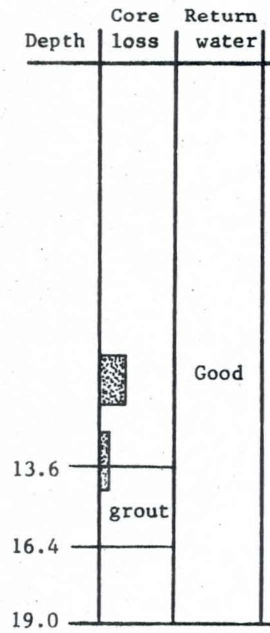


Photo 124
Core from exploratory grout core hole 9 + 69.5E. Note the significant amount of grout between 14.4 and 16.1 feet. Also note the well indurated infilling below 17.4 feet. (25 Sep 1981)

9+69.5E



Description

LATITE (Tvb): dark grey, locally reddish brown; hard, unweathered; vesicular; porphyritic; fine grained reddish zenoliths; highly fractured with irregular, mostly clean, unweathered cracks; first grout appears along ½-inch wide, semi-open, irregular crack/contact between grey and reddish brown latite at 10.5 feet; significant grout appears at 12.2 feet along irregular steeply dipping crack; core between 13.6 and 16.4 feet is approximately 75 percent grout with fragments of cindery, reddish brown latite; below 16.4 feet the rock is dark grey with thin irregular, partially healed, partially grouted, and partially open near vertical fractures; very well indurated brown, tuffaceous sandy infilling along irregular cracks between 17.9 and 18.5 feet; probable broken/ cindery rock washed from cavity in core between 17.6 and 17.9 feet.

Notes:

1. Total core recovery 97 percent.
2. See table 3, Foundation grouting summary, for pressure test and grouting data.
3. Cored using CP-65 drill and NW-size double-tube core barrel.
4. Collar elevation 1383.0.
5. See photo 124 for picture of core between 13.6 and 19.0 feet.

Figure 8. Log of exploratory Core Hole 9+69.5E.

3.85 The reach from grout hole 13+00P to 13+86T was grouted to completion using the split-space method previously described. Secondary and tertiary holes were deepened as necessary to bracket stages of high grout-take down to a maximum of 40 feet. For example, the stage from 35 to 41 feet in hole 13+00P, which took 570.3 sacks, was not split-space bracketed because the grout take was at 41 feet. In all, 2940 sacks (300 cubic yards of grout) were placed in the second zone between stations 12+80 and 13+80. This represents an average of 8.9 sacks per foot. These totals are exclusive of the 75 cubic yards and 940.7 sacks placed during the backfilling of D-11.

3.86 The original plan of backfilling the "cavity" in D-11 using a sand/cement slurry was abandoned because a cavity probably did not exist. It is believed that the high rock-mass porosity was caused by an interconnection of joints and fractures with spaces between the fracture blocks on the order of inches and not feet as originally suspected. Since a sand/cement slurry might have caused premature blockage under such conditions, the hole was backfilled to refusal with thick grout under gravity flow. After grouting for 10 hours at a 1:1 mix and placing 890 sacks of cement, approximately 1/3 cubic foot of filter sand was added to the grout. Small quantities of filter sand were added to the grout periodically. Within 2 hours, and after an additional 50 sacks, the hole was filled.

3.87 After all grouting had been completed, a vertical exploratory hole was drilled at station 13+01 (photo 127). A CP-65 drill was used for the coring. (See fig. 9 for the log of the hole.) Only a hint of grout was encountered at 34 feet even though the projection of the core hole intersected a stage in hole 13+20C in which 315 sacks were placed. Even though direct evidence of grout was not seen in the core, the hole was drilled without circulation loss down to 34.9 feet. In this stage, the hole would only accept 0.7 sack at 6:1 mix. All circulation again was lost at 37.6; and 12.3 sacks of thick grout were placed; however, this depth is slightly below the effective grout curtain, and a water loss might have been expected. Additional exploratory holes were not attempted for the following reasons:

a. The rock conditions encountered in hole 13+01E were as expected even though no grout was directly observed.

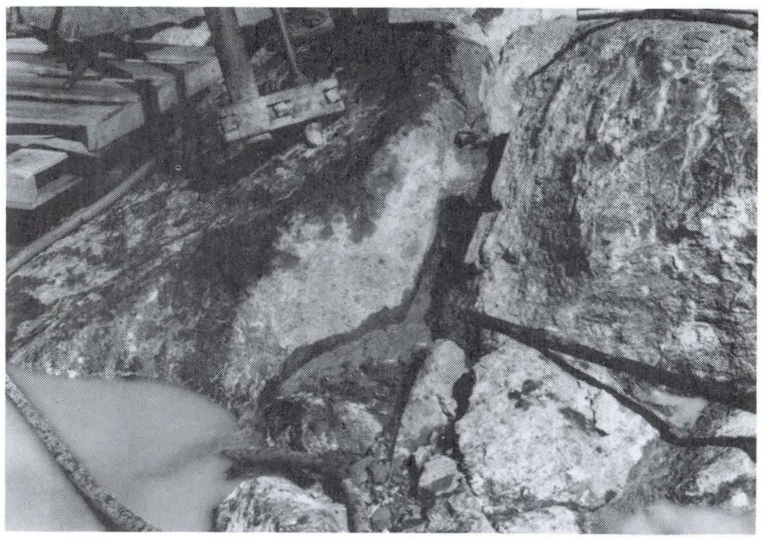
b. Because of the isolated nature of the grout takes in the second zone, it was felt that the chances of encountering a grouted seam (open joint or fracture) would have not been good.

3.88 **FINAL PAYMENT ITEM QUANTITIES.** Payment items were agreed to on a daily basis throughout the job and the final totals were easy to tabulate. Estimated and actual amounts were as follows:

		Estimated	Actual
a.	Mob/demob	1	1
b.	Drill exploratory holes	100 lf	57 lf
c.	Drill grout holes	975 lf	1761 lf
d.	Pipe for grout holes	100 lf	150 lf
e.	Drill set-ups	50	124
f.	Wash and pressure test	50 hr	26-1/2 hr
g.	Grout pump connections	50	103
h.	Place grout	200 sacks	4694.6 sacks
	Waste grout	-	194.4 sacks
	Total contract costs:	$57,164	$192,726

Photo 125
Lower abutment surface cleaned and approved for grouting. Air cleaning under core blanket area at right.
(28 Aug 1981)

Photo 126
Drill return water issuing from infilling between basalt blocks while drilling upper zone of 12 + 58P. Drilling continued to 20-foot depth. Caulked cracks and sealed with 3:1 grout at low pressure.
(10 Sept 1981)



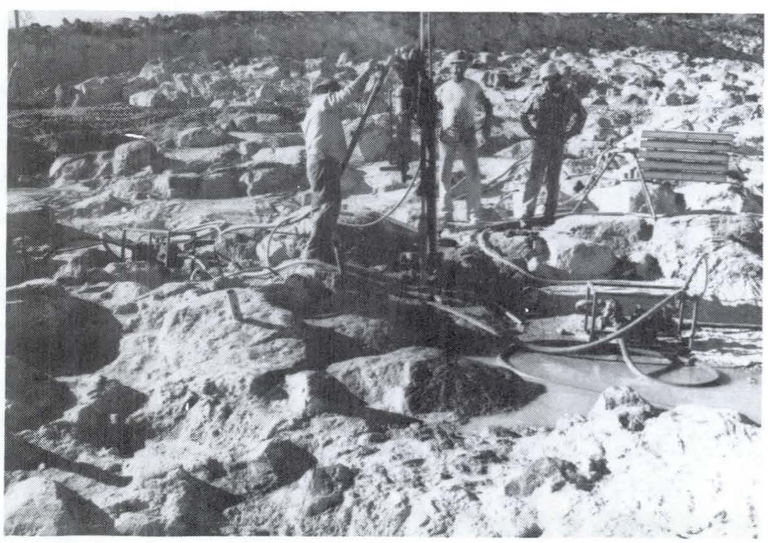


Photo 127
Using CP-65 to drill exploratory core hole 13 + 01E. Note portion of sand filter blanket at upper left. (3 Nov 1981)

13+01E

Depth	Core loss	Return water	Description
		Good	LATITE (Tvb): dark grey; hard; vesicular; porphyritic; widely spaced irregular fractures infilled with light brown, poorly to well indurated, tuffaceous, fine sand; core loss in softer infilled zones. At approximately 25 feet transitional color change to reddish grey associated with decrease in vesicularity and increased core loss. Core loss probably still associated with infilling between irregular fracture surfaces.
30.0			
30.9		Partial loss	ANDESITE (Tva): medium grey; hard; non-vesicular; slightly porphyritic; closely spaced regular joints, partially coated with calcium carbonate, sandy infilling, and possible grout at 34 feet. Joints vary from vertical to less than 20 degrees. Core loss due primarily to caving, raveling hole and resultant grinding of core. Lost return water at 34.9 feet; tried to grout with 6:1 mix but only 0.7 sacks accepted. Lost return water again at 37.6 feet; grouted with 12.3 sacks of 4/1½:1 grout.
34.9		Total loss	
37.6			
37.8			

Notes:

1. Total core recovery 83 percent.
2. See table 3, Foundation grouting summary, for pressure test and grouting data.
3. Cored using CP-65 drill and NW-size double-tube core barrel.
4. Collar elevation 1336.7.

Figure 9. Log of Exploratory Core Hole 13+01E.

3.89 At the conclusion of the job, it was mutually agreed between the Contracting Officer, Sundt and Jaques, that the Corps pay for extra man hours required because of the transportation of drills over the unusually rough abutment surface and installation and connection to the waste-water collection system. The prime contractor agreed to pay for the PVC pipe utilized. Added costs to the Government amounted to approximately 3 percent of the total contract costs associated with the foundation grouting and were paid for under MOD P00015 (table 4).

3.90 **CONCLUSIONS.** Inasmuch as the grouting represents an exploratory tool as well as improving the foundation, the grouting program at Adobe Dam was successful. Assumptions regarding the relative impermeability of the near surface rock were verified while unexpected conditions below were uncovered. In the final analysis, grouting in the upper zone probably did little to improve upon the condition which naturally existed. The split-space stage grouting methods utilized at the top of the abutment and in the lower zone at the toe undoubtedly created a much more impermeable barrier to underseepage than existed before grouting. It is reasonable, however, to expect that permeable fractures with significant lateral extent may remain ungrouted 30 feet below the foundation surface. The degree of confidence in curtain construction could have been improved by multiple rows and tighter spacing. These alternatives were judged to be unnecessary and not cost-beneficial from an engineering standpoint given the short maximum flood pool duration and the conservative design of the embankment and surface foundation treatment.

OUTLET WORKS

3.91 The outlet for Adobe Dam is a cut and cover, rectangular concrete conduit 289.5 feet in length. The outlet trench was excavated through the right abutment and the conduit is founded on the basalt block flow (photo 128). Because of the rock contour at the abutment, the outlet works is slightly askew (18-1/2 degrees) to the centerline of the dam embankment (amended pl. 15). That portion of the excavation under the core of the dam was backfilled with lean-mix concrete and contact grouted to preclude seepage along the conduit. Downstream from the conduit is an uncovered energy dissipator section which flares out from 5.9 to 20 feet in width and drops over 16 feet in invert elevation. The energy dissipator is also founded on rock (basalt and agglomerate). Excavation of the outlet works trench began on 9 March 1981. Backfill around the conduit was essentially complete by August 1981 although backfill at the downstream end and next to the energy dissipator walls continued until October.

GEOLOGY

3.92 The trench for the outlet conduit cuts diagonally across the abutment, intersecting the dam centerline at station 12+14. The minimum excavated elevation along the conduit section was approximately 1330 feet, which is about 7 feet above the projected contact between Tvb and Tva. The entire outlet conduit, therefore, is founded on the basalt block flow, Tvb, previously described under the geology of the right abutment. The intake structure is also founded on the basalt. Only the end of the energy dissipator downstream from station 71+75 is founded on well-indurated agglomerate, QTa (photo 129), instead of basalt.

3.93 After the floor and walls of the trench had been blown clean, the floor of the conduit section was mapped (pl. 4) and the walls inspected and photographed. The mapping and repeated inspection of the excavation yielded the following observations.

a. There was no sifgnificant difference in the nature of the block flow foundation except that the relative percentage of infilling increased (photo 84) and the apparent structural continuity decreased upstream from station 74+70.

b. As a whole, about 10 percent of the exposed surface was composed of moderately indurated, fine grained, ashy infilling.

c. All cracks, joints, and spaces between the basalt blocks were completely filled; no voids were observed.

d. The general appearance was that of disoriented basalt blocks to 10 feet maximum dimension; most were irregularly shaped, but some had one or more planar surfaces (photo 130).

e. There were planar lineations and occasional brecciated zones on the floor and walls (photo 130); however, most were indistinct, had very limited lateral extent, and did not extend up to the top of the trench walls.

f. The discontinuities (joints and fractures) generally trended obliquely to the axis of the outlet and dipped steeply upstream.

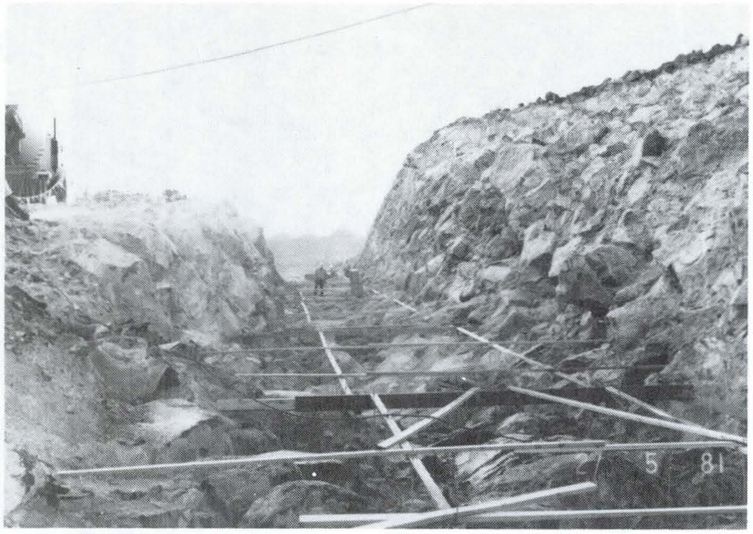
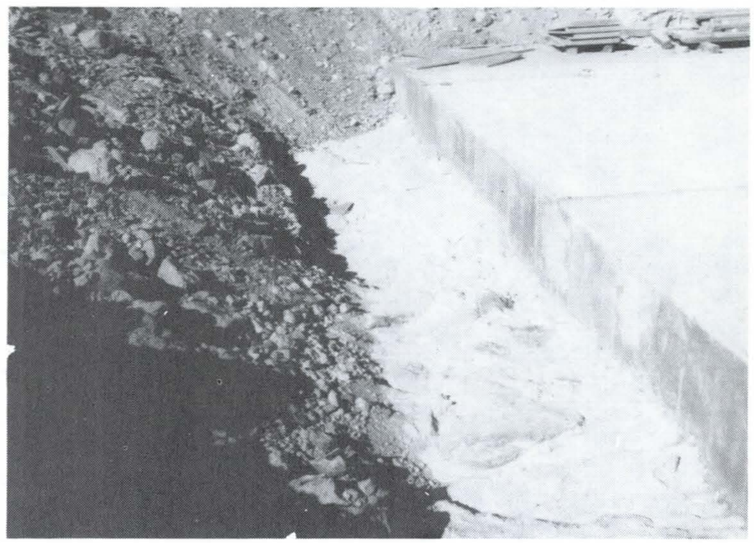


Photo 128
General configuration of outlet works' trench; view looking downstream from near station 76 + 50. Worker at left is wetting down basalt block flow foundation prior to placement of lean-mix leveling slab. (27 May 1981)

Photo 129
Exposure of hard, indurated agglomerate which serves as the foundation for the end of the energy dissipator slab. (11 June 1981)



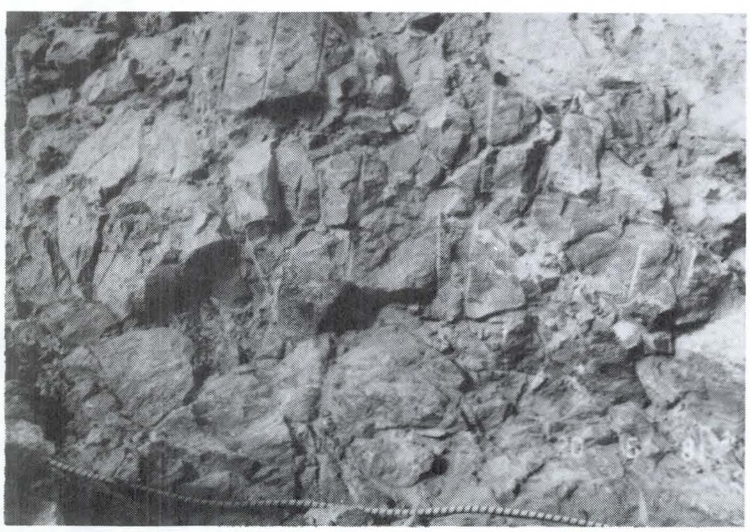


Photo 130
West slope of outlet trench at stations 74 + 00 to 74 + 40. Note relatively planar, steeply dipping discontinuities in center and lower right of photo which trend N5E 70N and N10E 75N. (20 May 1981)

EXCAVATION

Blasting

3.95 Since the outlet conduit is founded on rock and the trench excavated in the basalt flow rock mass, blasting was required to shape the 1/2-on-1 side slopes and loosen the volcanic blocks. In order to better control the final shape of the trench and protect the foundation, a 5-foot buffer zone was specified (amended pl. 16). Instead of the smooth blasting procedure described in section 2C 17.3 of the contract specifications (attachment 5), the contractor proposed a drilling and blasting scheme similar to that used with success in the spillway. His proposal consisted of shooting 3-inch diameter pre-split holes on 30-inch centers followed by delayed production holes on a 5x6-foot pattern subdrilled 2 feet below the A-line. The pre-split holes were to be logged according to drilling resistance then loaded and stemmed accordingly. The contractor was allowed to demonstrate his proposed technique downstream from station 73+00. Between stations 73+00 and 73+40, the contractor was instructed to drill a modified demonstration section with hole spacings and subdrilling as recommended by the LADO Geology Section. In the COE area, subdrilling was only 1-1/2 feet below A-line; a 4x4-deck-loaded delay pattern was used; and pre-split holes were spaced 2 feet apart (photo 131). The demonstration blast was shot on 16 March 1981.

3.96 By 27 March the test section had been cleaned in enough detail to inspect and evaluate the results of the blast (photo 132). Both test sections were excavated well below the A-line except in isolated areas along the toe of the slopes where the bottom of the pre-split and breaker holes "blew out" in softer infilling. Nonetheless, most of the trench floor was below even the B-line. No difference in the pre-split slopes was noticeable between the two test sections. The pre-split hole spacing was less critical than deviations from angle and plumbness. The floor of the excavation was hand-level surveyed on a 3x5-foot grid. This revealed that the tighter hole spacing and shallower subdrilling produced a more regular surface and reduced the amount of over-excavation. The contractor, therefore, was directed to use a 4x4-foot pattern, 1-1/2-foot subdrill, and 30-inch pre-split spacing following the same blasting procedures for the remainder of the trench excavation. This directive was covered under Modification P00011 to the contract, which also included the detailed cleaning of the test area.

Mucking

3.97 The large rock loosened by the blasting was moved out of the trench using a D-9H dozer. The volcanic blocks removed were up to 6 feet in maximum dimension (photo 133) and were used along with the rock excavated from the abutment surface as backfill along the upstream toe of the dam. Further excavation using a backhoe and extensive hand labor was required to complete the removal of loose and broken rock. The "dental" excavation began on 5 May and was completed along with the foundation preparation along the floor of the conduit section by 26 May.

Cleaning

3.98 The contractor was instructed to use the same criteria for cleaning the floor of the trench as had been used on the abutment surface. Rock picks were used to loosen the infilling material from between the basalt blocks followed by high-pressure air cleaning (photo 97). The side slopes were prepared in the same manner except that extra care was taken to avoid removing too much infilling and thereby dislodging the massive basalt blocks. (See photo 134 for a typical prepared slope surface.)

Resultant Excavation

3.99 When the final removal of all loosened rock had been completed and the area cleaned, the shape and size of the excavation deviated from the lines and grades shown on amended plate 16 (photo 128). The width at the bottom of the trench (at "B" line elevation) exceeded the dimension shown on the plans by an average of 10 percent in the conduit section of the trench. (See pl. 4.) The deviation was greatest upstream from station 74+80 in an area of increased relative percentage of infilling to basalt (photo 135). The floor of the trench was also outside the lines and grades as designed. The average elevation of the floor in the conduit section (station 75+69 to 72+80) was 2.0 feet below the "B" line. The reason for the over-excavation was partially due to the blasting techniques used, but was primarily the result of the nature of the basalt rock mass. While the blasting broke the large basalt blocks near the "B" line grade, adjacent smaller blocks and infill-ing were removed during the required excavation and cleaning (photo 136).

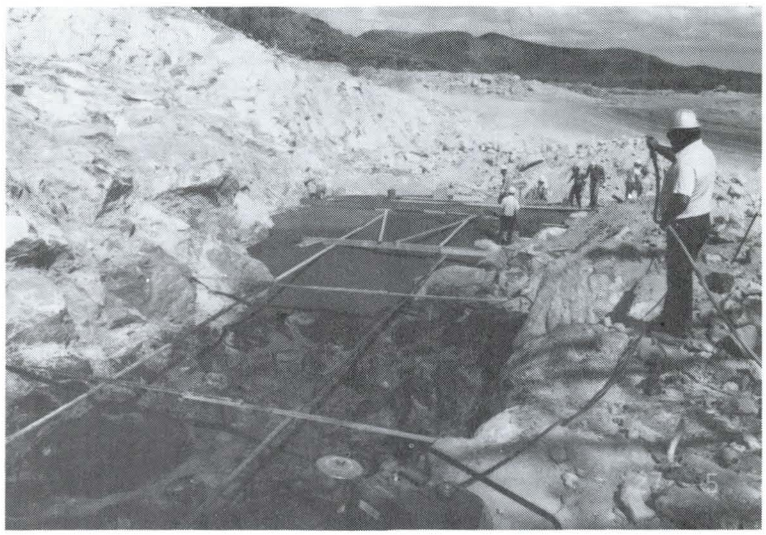
Photo 131
Partially drilled COE outlet works' test section; stations 74 + 00 to 74 + 40. 2½-inch ANFO loaded holes on 4x4-foot pattern, pre-splits on 24-inch centers. (12 Mar 1981)

Photo 132
Partially cleaned outlet works' test blast section. Bottom has been cleaned from stations 73 + 50 to 72 + 85. Note large rock mass projecting from toe of the west slope (right side) at station 73 + 40 because toe breaker hole was not shot. (28 Mar 1981)

Photo 133
CAT D-9H pushing out blasted rock from outlet trench excavation. Note the 6-foot basalt blocks already removed. Track backhoe and hand laborers cleaning the abutment surface in background. (19 Mar 1981)

Photo 134
Close-up view of typically prepared slope surface in outlet trench; east side of conduit, station 74 + 35. Infilling between the volcanic blocks is pervasive.
(9 July 1981)

Photo 135
Wetting down cleaned and prepared trench floor prior to placement of lean-mix leveling slab. Screed forms constructed of 2x4s elevated to "B" line on metal rods anchored in rock. Concrete already poured to station 75 + 30. Note the abundant infilling at the left of photo (west side of trench) and how the width and slope of the "walls" are excessive.
(27 May 1981)



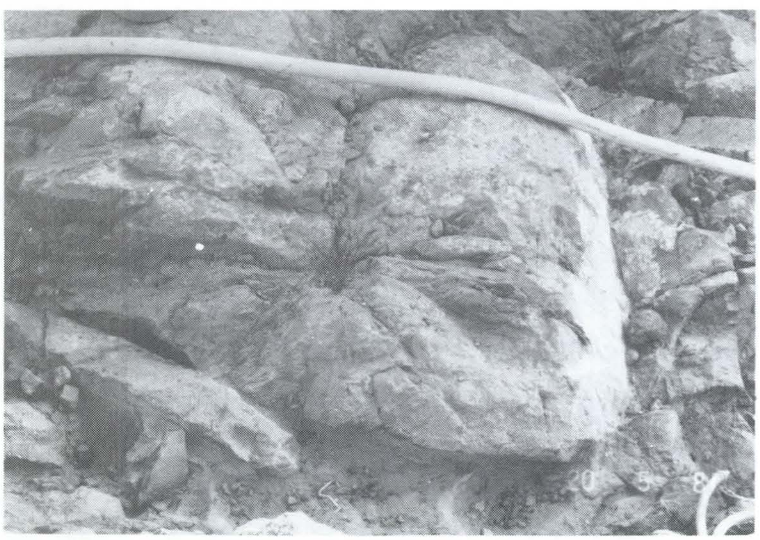


Photo 136
Effects of production blast in bottom of outlet trench. Note how smaller blocks and infilling have been removed below the level of the bottom of the blast holes in the center and upper left of photo. Final surface preparation not yet completed.
(20 May 1981)

It is likely that the amount of over-excavation could have been reduced by modifications in the blasting techniques. However, the 9-inch "B" line tolerance was unreasonable for this type of foundation rock, and protrusions above the "A" line would have been more numerous and difficult to trim. Based upon the results of the test blast between stations 73+00 and 73+40, a 2-foot relief was reasonable to expect.

LEAN-MIX LEVELING SLAB

3.100 Since most of the resultant trench excavation was below the "B" line, it was necessary to replace the over-excavated area with concrete as required under section 2C-1.4 of the contract specifications. The Government assumed responsibility for payment because of the unanticipated irregularity of the foundation surface. The work was covered under MOD P00006 at a total negotiated cost of $68,250. The change covered the placement of 410 cubic yards of concrete to "B" line elevation in the conduit section and 500 cubic yards of concrete to "A" line in the intake and energy dissipator sections. After the floor of the trench had been adequately cleaned, the contractor constructed screed forms at "B" line elevation. The forms consisted of 2x4 boards supported on 1/2-inch threaded metal rods. The rods were on 8- to 10-foot centers, anchored into the foundation rock (photo 135). Bulkheads at station 75+69 and 72+80 were constructed to contain the leveling slab (photos 137 and 138). Prior to concrete placement, small pockets of loose rock were hand-picked from the floor (photo 139) and the foundation was wetted (photos 128 and 135). The concrete surface was screed to a uniform "B' line elevation for the entire width of the trench except toward the upstream end where the excavation was the widest. In this area the edges outside the limits of the conduit foundation were somewhat sloped to save on the amount of concrete placed. The slab was poured slope-to-slope for the following reasons:

a. The lean-mix plug area from stations 73+90 to 74+90 would eventually require concrete backfill.

b. Outside the lean-mix plug area, dental concrete (at a higher unit cost) would have been necessary in many areas at the toe of the slopes.

c. Forming costs were significantly reduced.

d. The slab provided a level surface for workmen forming the conduit.

e. The slab allowed for easy removal of loose rock and infilling, which was produced by the final cleaning of the side slopes prior to concrete and core material backfill.

LEAN MIX PLUG

3.101 From 28 May to 9 July 1981, the outlet works' conduit was under construction (photo 140), and on 10 July the trench was ready to be backfilled with concrete in the area of the lean-mix plug (photo 141). The purpose of the plug was to preclude differential settlement and cracking of the core over the conduit, as well as to inhibit seepage along the outside of the box. The side walls of the trench were prepared by removing all loose rock. Infilling between the blocks was cleaned using the high pressure air hose. (See photo 134 for a typical prepared slope.) A concrete bucket and crane were used to place the concrete.

Photo 137
Bulkhead to contain "B" line leveling slab at upstream end of trench (station 75 + 69). (27 May 1981)

Photo 138
Bulkhead at downstream end of trench at station 72 + 80. Leveling slab placement already underway at upstream end of trench. Note evidence of pre-split blasting on slope at left. (27 May 1981)

Photo 139
Laborers removing pockets of loose rock by hand from floor of already air cleaned trench bottom. Screed forms under construction. (26 May 1981)

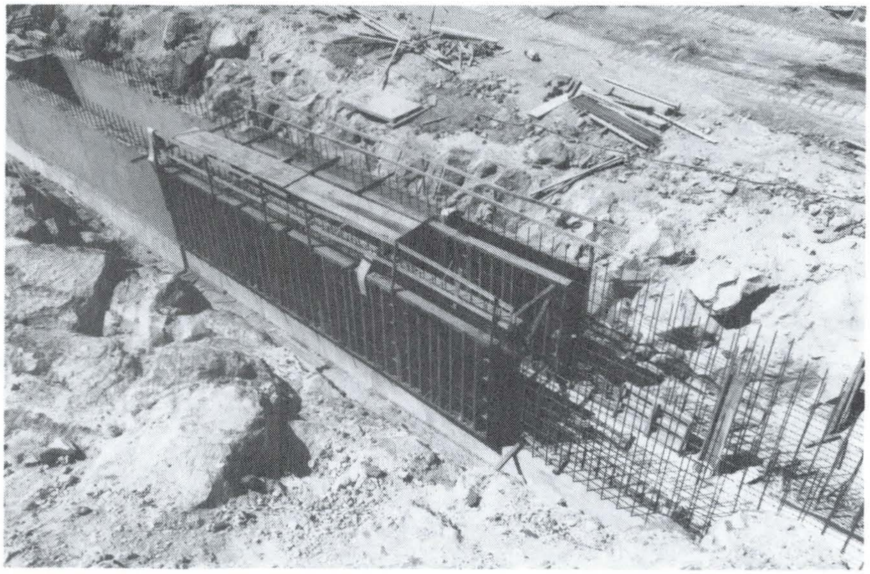


Photo 140
Forming walls for outlet conduit between stations 73 + 40 and 73 + 80.
(24 June 1981)

Photo 141
View looking downstream from station 75 + 40 along the west side of the completed outlet conduit prior to lean-mix plug placement. Plug was placed in the area where the conduit is thicker and the slope closer to the sides. Note B-line leveling slab on floor in center of photo.
(9 July 1981)

The 3/4-inch aggregate and relatively low water to cement ratio allowed the unformed concrete to be placed on the 1:1 slope specified (photo 142). Special emphasis was placed on adequately vibrating the concrete along the conduit and at the contact with the foundation. The foundation contact was also pressure grouted to seal any voids as previously discussed under conduit grouting. Since more overburden and rock had been removed than originally anticipated during the abutment excavation (amended pl. 12) the concrete plug did not extend over the top of the conduit but just to the top of edge of the box (photo 143). (See amended pl. 16 for the cross section of the plug.)

CONDUIT BACKFILL

3.102 The remainder of the outlet trench was backfilled with compacted embankment materials. The slopes were cleaned as previously described and thin lifts of backfill were hand-tamped along the rock contact (photo 144). Sand cone density testing and probing along the rock contact indicated that adequate compaction was obtained. At station 75+02 on the west side, 1-1/2 cubic yards of formed dental concrete was used to fill under a protruding rock block (photo 84). Several months after the trench was backfilled, when the embankment fill was coming up the abutment, the backfill material was sloped back, scarified, and re-wetted prior to blending with the embankent core material.

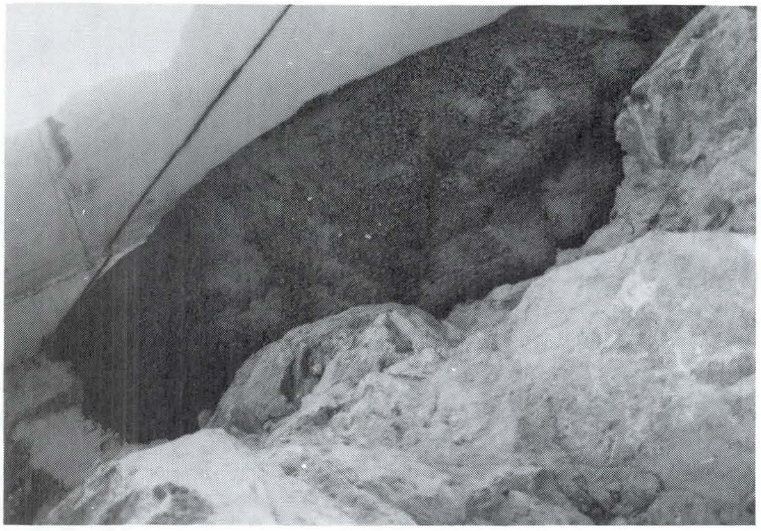


Photo 142
Lean-mix plug on downstream end, east side at station 73 + 90. Plug placed without forming on 1:1 slope (line on side of conduit).
(10 July 1981)

Photo 143
Top of completed concrete plug after much of conduit backfilling has been completed. Note transition from contact with abutment at left and the top edge of the conduit. (28 July 1981)

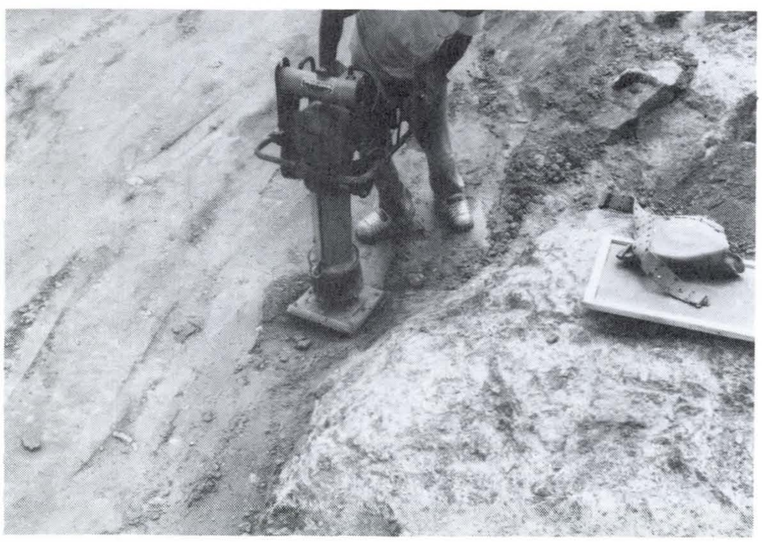


Photo 144
Laborer using Whacker to compact core material along rock contact during outlet trench backfilling upstream from lean-mix plug. A small vibrating steel wheeled roller was used for the bulk of the material compaction. (30 July 1981)

Left Abutment

3.103 Adobe Dam does not have a left abutment in the true sense, but would be more correctly designated the east end of the dam. The abutment as such is on the west side of Adobe Mountain, which is adjacent to the Black Canyon Highway, I-17. The break in slope of the moderately steep west flank is at elevation 1400 feet above sea level, the valley floor is at elevation 1395 feet, and the top of the dam is at 1403 feet. The total excavation up the hillside was approximately 5 feet and only 1 or 2 feet deep. The impervious core of the zoned embankment daylighted at station 121+10 and, therefore, the east end of the dam was composed entirely of random fill and stone slope protection.

GEOLOGY

3.104 Adobe Mountain is superficially very similar to the Hedgepeth Hills at the right abutment. Approximately 95 percent of the surface is covered by residual soil or volcanic talus blocks (photo 5), which makes geologic mapping and identification of bedrock units very difficult. Because of the non-criticality of the left abutment, detailed geologic mapping was not performed on Adobe Mountain and sub-surface explorations were deferred. The mountain was mapped using 1:3000 aerial photographs as QTv (subsequently changed to Tv, see amended plate 1) skirted by residual soil and talus. Excavation for the dam was very limited and exposed vesicular volcanic talus blocks to 1-1/2-foot maximum diameter infilled with tan tuffaceous sands and locally cemented by caliche (photo 145).

EXCAVATION

3.105 A D-9 dozer, a backhoe, and a front-end loader were required to remove the caliche-cemented talus blocks down to the grade lines as shown on the contract drawings. The final excavated surface is represented by photo 146 except that the windrow of loose material at the toe was removed prior to placement of random fill.

Main Embankment Foundation

3.106 The entire length of Adobe Dam, except for the right abutment, is founded on alluvium. The pre-construction foundation exploration had indicated that a variable thickness of between 1 and 5 feet of fine grained, compressible surface soil was present across the valley between the Hedgepeth Hills and Adobe Mountain (amended pl. 10). The alluvium below was expected to be coarser grained, incompressible, and suitable as a foundation. An exploration trench of variable depth (9 to 20 feet below the foundation stripping line) was included in the embankment design from the right abutment to station 79+60 in order to explore for and remove areas of deleterious (incompatible) foundation materials.

GEOLOGY

3.107 The geologic conditions expected in the foundation for the main embankment were the conditions actually encountered.

EXCAVATION

3.108 In only three isolated areas (one under the upstream core blanket, one under the core blanket and in the exploration trench, and one under the downstream gravel blanket) were there encountered concentrations of materials incompatible with the overlying embankment. Permeable, relatively coarse grained materials under the core between stations 19+50 and 21+00 and between 31+50 and 32+50 and silty sands under the gravel drain between stations 36+00 and 39+00 were removed and backfilled with compacted fill according to part 2C-14 of the specifications (attachment 2). In addition, under the gravel blanket between stations 13+50 and 15+00, fine grained sediments associated with a pre-existing stock tank (fig. 2) were removed as specified in the contract plans. This created a "swale" in the foundation surface (photo 147) on which the "normal" embankment section was constructed; i.e., the area was not backfilled to "grade."

Photo 145
Close-up view of caliche-cemented basalt blocks (slope wash and talus) at left abutment excavation. This represents the final surface prior to placement of random material. (26 Mar 1981)

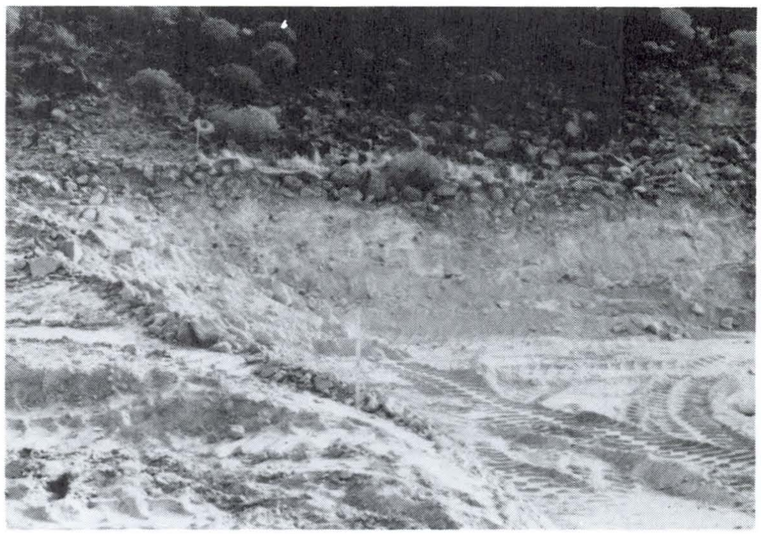


Photo 146
Final left abutment (east end-of-dam) excavation after removal of caliche-cemented basalt blocks. Windrow of loose material at toe was removed prior to placement of embankment material. (26 Mar 1981)

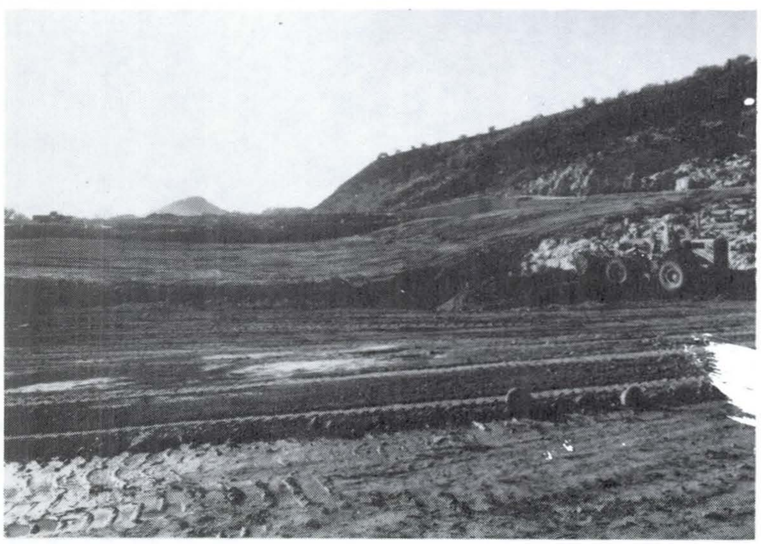


Photo 147
"Swale in foundation under horizontal gravel drain, station 13 + 50 to 15 + 00, after inspection and proofrolling. Sand filter-blanket on abutment rock at right. (3 Nov 1981)

4. Contract Totals

Final Payment Item Quantities

4.01 M. M. Sundt Construction Company was awarded contract DACW09-80-C-0121 with a total estimated bid amount of $8,388,025. The final contract amount was $9,535,322.65, exclusive of 20 modifications, some of which are still being negotiated. (See attachment 6 for the final payment item quantities which were different from estimated (obligation) amounts.) In the following cases, the estimates (obligations) were modified after the contract had been awarded; all other obligation quantities listed in attachment 6 are original estimated amounts.

Bid Item	Description	Original Estimated Quantity	Modified Estimated Quantity	Actual Quantity
7	Excavation, abutment	29,800 CY	36,000 CY	33,232 CY
10B	Excavation, dental over 100 CY	50 CY	5,400 CY	5,717 CY
11	Excavation, outlet works	9,530 CY	12,102 CY	12,110 CY
27	Filter material	3,900 CY	4,000 CY	3,659 CY
29	Stone, type II	44,500 TN	67,000 TN	35,494 TN
38B	Dental concrete over 60 CY	65 CY	940 CY	910 CY
43D	Pipe for grout holes	100 LF	130 LF	150 LF
43E	Drill set-ups	50 EA	70 EA	124 EA
43G	Grout pump connections	50 EA	70 EA	103 EA
43H	Placing grout	200 SA	864 SA	4,744 SA

Contract Modifications

4.02 In addition to the final contract amount, the modifications to the contract will constitute additional costs. Nine of the modifications were directly related to geotechnical aspects of the job; specifically, the unexpected foundation conditions at the right abutment. The total of these changes was $363,060.00 and they are summarized in table 4. No other geotechnically related modifications are pending.

Table 4. Contract modifications.
(geotechnical related)

Item	Subject	Description of Change	Negotiated Cost
MOD P00005	Investigate west abutment	Drill 20 probe holes and clean 40x40-foot area.	$13,141.00
MOD P00006	Lean mix leveling slab	Place lean mix leveling slab in outlet conduit, intake structure and energy dissipator from station 76+12.2 to 71+54; 910 cubic yards total.	68,250.00
MOD P00008	Revised abutment excavation	Drill and blast entire abutment surface; pioneer trail; blast oversteepened back slope.	43,125.00
MOD P00009	Abutment filter	Place 4420.45 cubic yards of filter sand between abutment surface and gravel blanket to prevent piping of infilling material.	97,560.00
MOD P00010	Irregular abutment surface	Additional work required of men and equipment necessary to properly excavate the irregular abutment surface.	43,495.00
MOD P00011	Revised outlet costs	Additional cost to use 4x4-foot drill pattern; air cleaning and dental excavation of demonstration blast area.	32,013.00
MOD P00014	Delays due to irregular abutment surface	Extra equipment rental and labor costs caused by delays relating to irregular abutment surface.	49,625.00
MOD P00015	Extra costs for foundation drilling and grouting	Ream out and deepen D-11; establish waste water control system; move and set-up drilling equipment over irregular abutment surface.	5,851.00
MOD P00018	Abutment fill costs	Extra costs to use COE specified equipment (front-end loader and hand tampers) to compact along irregular abutment contact.	10,000.00
		Total	$363,060.00

Sheet 1 of 1

5. *Possible Future Problems*

Type III Stone Breakdown

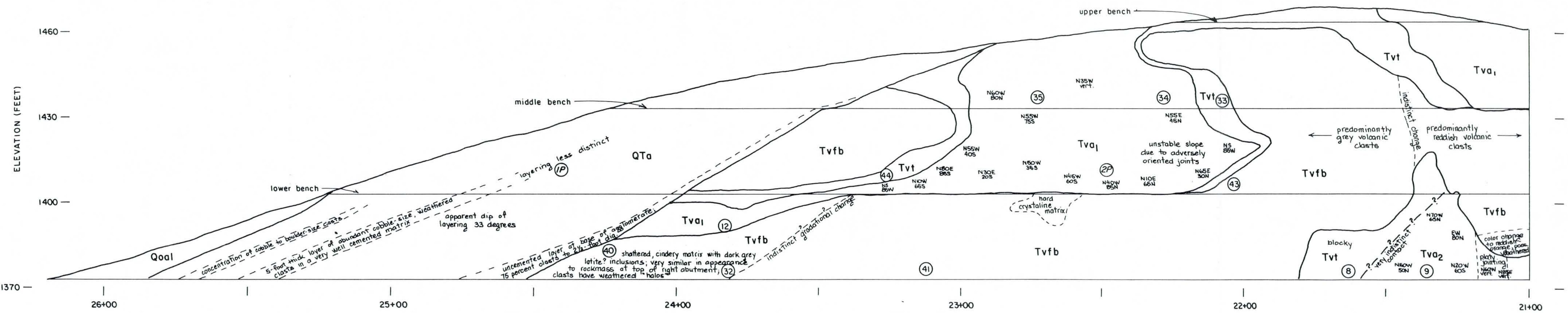
5.01 As discussed previously, the andesite from the spillway excavation used as the downstream slope protection on the dam was observed to be breaking down several months after placement. Similar andesite exposed during foundation explorations at New River damsite, several miles to the northwest, has experienced even more severe degradation. The causative mechanism for the breakdown is still in question; and it is not possible to estimate the rate of subsequent deterioration or what the ultimate effects on the gradation may be. It is unlikely, however, that the stone's effectiveness against erosion on the downstream slope will be severely affected in the near future. Nonetheless the deterioration of the stone should be monitored and recorded during the regularly scheduled periodic inspections.

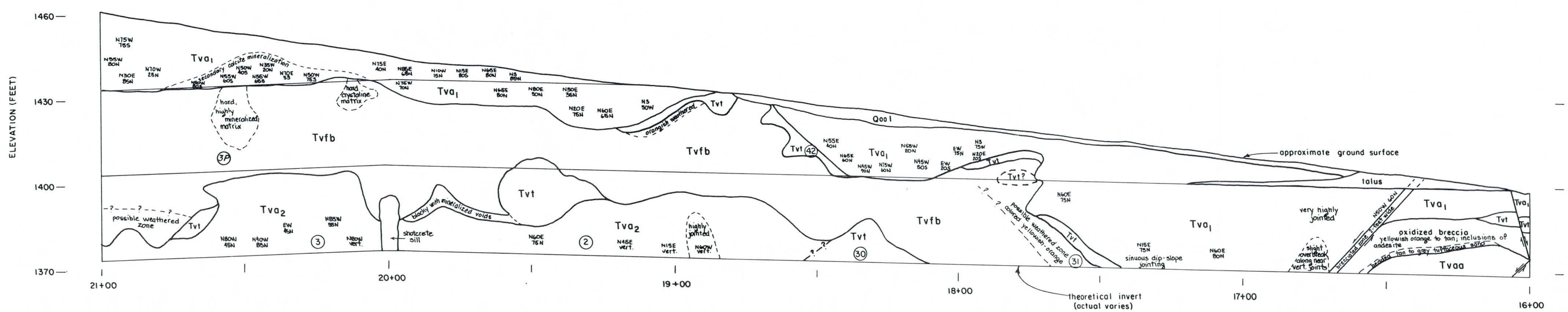
Spillway Slope Degradation

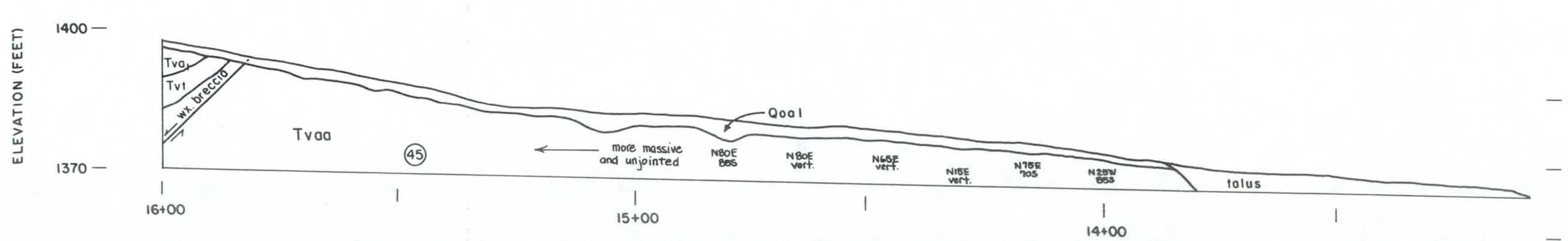
5.02 Minor rock falls and sluffing from the spillway slopes should be expected. The 12-foot-wide benches will help in reducing the velocity of the falling rocks and will keep some material off the floor of the spillway. Periodic maintenance cleaning, however, may be necessary. The most potentially serious problem areas will be in the andesite units where the joint planes have adverse orientations toward the excavation. Rockfalls from the highly jointed upper andesite on the east side at station 22+50 (photo 12) occurred during construction. Another possible problem area is at station 20+40, west side, just below the lower bench, where a prominant joint dip-slope is exposed (photo 22). Failure of this portion of the spillway will not significantly reduce its effectivenss, but could present a safety hazard for people below or on the bench above. No area in the spillway has been identified as a likely candidate for catastrophic slope failure.

Abutment Seepage

5.03 The likelihood of uncontrolled reservoir seepage through the abutment is extremely remote. If leakage were to occur, however, the abutment surface between station 13+20 and the bottom of the exploration trench would be the most likely problem area. This is because of the more heterogeneous alluvial infilling between the volcanic blocks as opposed to the tuffaceous infilling higher on the abutment. Visual inspection and surface grouting data (pl. 3) indicated, however, that the material is competent and impermeable.







SCALE 15 10 5 0 15 30 45 FEET

LEGEND:

Qoal	RESIDUAL SOIL, COLLUVIUM, AND WEATHERED ROCK: SUB-ANGULAR TO SUB-ROUNDED, HETEROLITHIC VOLCANIC CLASTS, GRAVEL TO COBBLE-SIZE; BROWN SILTY SAND MATRIX; UNCONSOLIDATED; LOCALLY CALICHE CEMENTED
QTa	AGGLOMERATE: SUB-ANGULAR TO SUB-ROUNDED, HETEROLITHIC VOLCANIC CLASTS TO 3-FOOT MAXIMUM DIMENSION; POORLY TO VERY WELL CEMENTED $CaCO_3$ MATRIX; STRATIFIED
Tva_1	UPPER ANDESITE: LIGHT GREY TO GREY; MODERATE HARDNESS; SLIGHTLY PORPHYRITIC, MICROCRYSTALINE; TYPICALLY HIGHLY JOINTED
Tvfb	FLOW BRECCIA: REDDISH BROWN AND GREY; CRUMBLY TO LOCALLY VERY HARD; 40 TO 80 PERCENT HARD ANDESITE ROCK BLOCKS TO 5-FOOT MAXIMUM DIMENSION; CINDERY MATRIX; SOME SECONDARY MINERALIZATION
Tva_2	LOWER ANDESITE: LIGHT GREY TO GREY; MODERATELY HARD; SLIGHTLY PORPHYRITIC, MICROCRYSTALINE; TYPICALLY BLOCKY JOINTING
Tvt	TRANSITIONAL ANDESITE/FLOW BRECCIA: MAROON TO GREY; HARD; PORPHYRITIC, GLASSY TO MICROCRYSTALINE; TYPICALLY MASSIVE
Tvaa	AMYGDALOIDAL ANDESITE: MAROON TO GREY; MODERATE HARDNESS; MODERATELY TO HIGHLY VESICULAR; ZEOLITE AMYGDULES; IRREGULARLY FRACTURED
	CONTACT BETWEEN ROCK UNITS; DASHED WHERE VAGUE
	LIMITS OF GRADATIONAL CHANGE WITHIN A ROCK UNIT
	FAULT
N30E 70S	ATTITUDE OF JOINT
32	ROCK SAMPLE LOCATION
2P	LOCATION OF ROCK SAMPLE FOR PETROGRAPHIC ANALYSIS

NOTES:

1. RESIDUAL SOIL AND WEATHERED ROCK IS GENERALLY NOT DESIGNATED WHERE THE THICKNESS IS LESS THAN ONE FOOT.
2. ALTHOUGH CONTACTS BETWEEN ROCK UNITS ARE SHOWN AS DISTINCT SOLID LINES, THEY ARE PREDOMINANTLY INDISTINCT AND GRADATIONAL, ESPECIALLY FOR THE TRANSITIONAL UNIT.
3. THE GEOLOGY EXPOSED ON THE WEST SIDE OF THE SPILLWAY EXCAVATION IS ESSENTIALLY THE SAME AS THAT ON THE EAST SIDE.
4. ROCK SAMPLES NOT COLLECTED FOR PETROGRAPHIC ANALYSIS WILL BE RETAINED AT THE LOS ANGELES DISTRICT OFFICE.
5. FAULTING SUB-PARALLEL TO THE SPILLWAY AXIS AS INDICATED ON THE CONTRACT DRAWINGS (SHEET 6 OF 72) WAS NOT ENCOUNTERED DURING THE SPILLWAY EXCAVATION.

ADOBE DAM FOUNDATION REPORT

GEOLOGY OF EAST WALL OF SPILLWAY EXCAVATION

PLATE I


Headwall of inlet structure
Covered outlet conduit
Grout plant
Tvb
tuffaceous sand infilling between "structureless" rock blocks
alluvial infilling between rock blocks
QTa well cemented
indistinct structure in otherwise structureless area
℄ Dam
9+50
10+50
11+50
12+50
13+50
downstream limit of core
areas of "structured" block flow
℄ Outlet works
area not mapped (under gravel drain)
D-23
D-9
D-10
D-11
N
Approx. Scale
10 5 0 10 20 30 40
Feet

PLATE 2

LEGEND:

N50W 70S	relatively planar fracture/joint
	low angle joint surface
N50W 70S	brecciated fracture
× × × × ×	area of broken or cindery rock
● D-9	pre-construction core hole
	contact between Tvb and QTa
	approximate "contact" between different types and character of infilling within Tvb unit
Tvb	Tertiary basalt block flow
QTa	Quaternary/Tertiary agglomerate

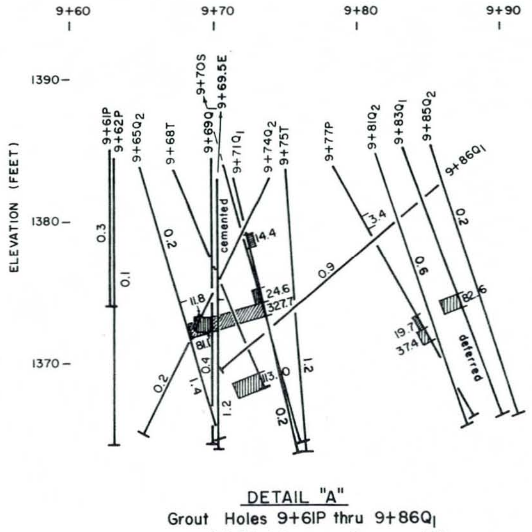
NOTES:

1. The aerial base map was shot at an original scale of 1:2400 on 5 September 1981.
2. See attachment 1 for an interpretation of the genesis of the block flow rock unit.
3. In general, all planar and brecciated fractures are infilled with cemented, tuffaceous sand, except in areas designated as broken which are open.
4. The area under the gravel drain above the outlet works was cleaned and mapped as a part of the early foundation evaluation. The same area below the outlet was not cleaned in detail and was not mapped; it is, however, the same randomly oriented, infilled basalt block flow.

ADOBE DAM FOUNDATION REPORT

RIGHT ABUTMENT

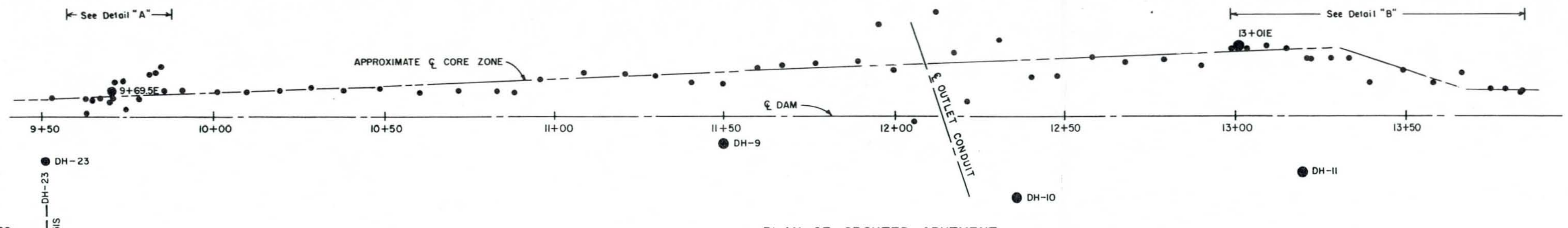
FOUNDATION GEOLOGY



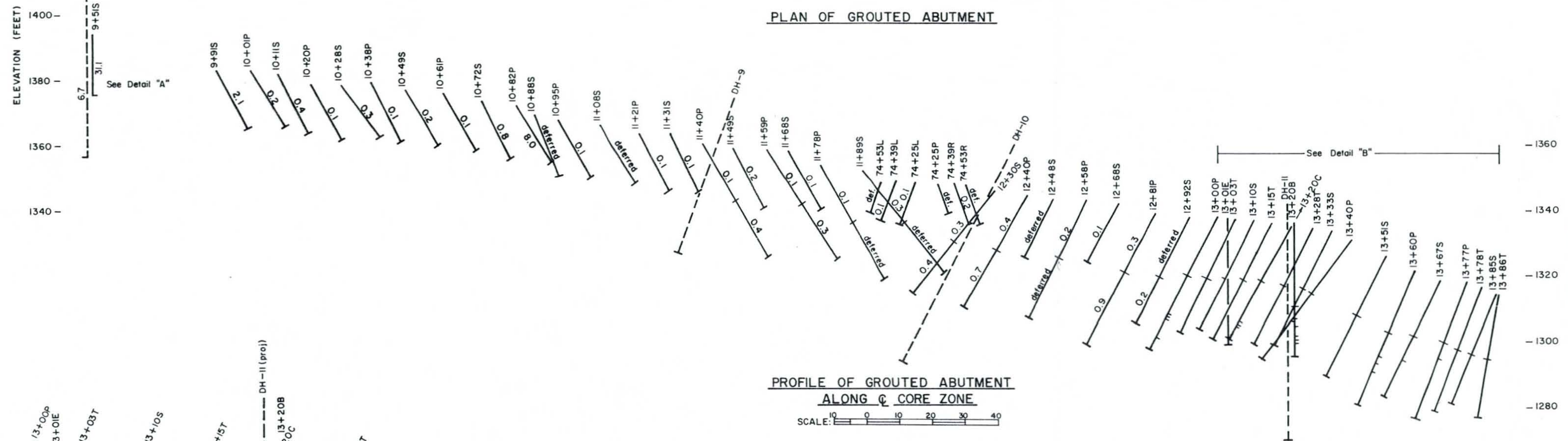
DETAIL "A"

Grout Holes 9+61P thru 9+86Q₁

SCALE: 5 4 3 2 1 0 5 10 FT.

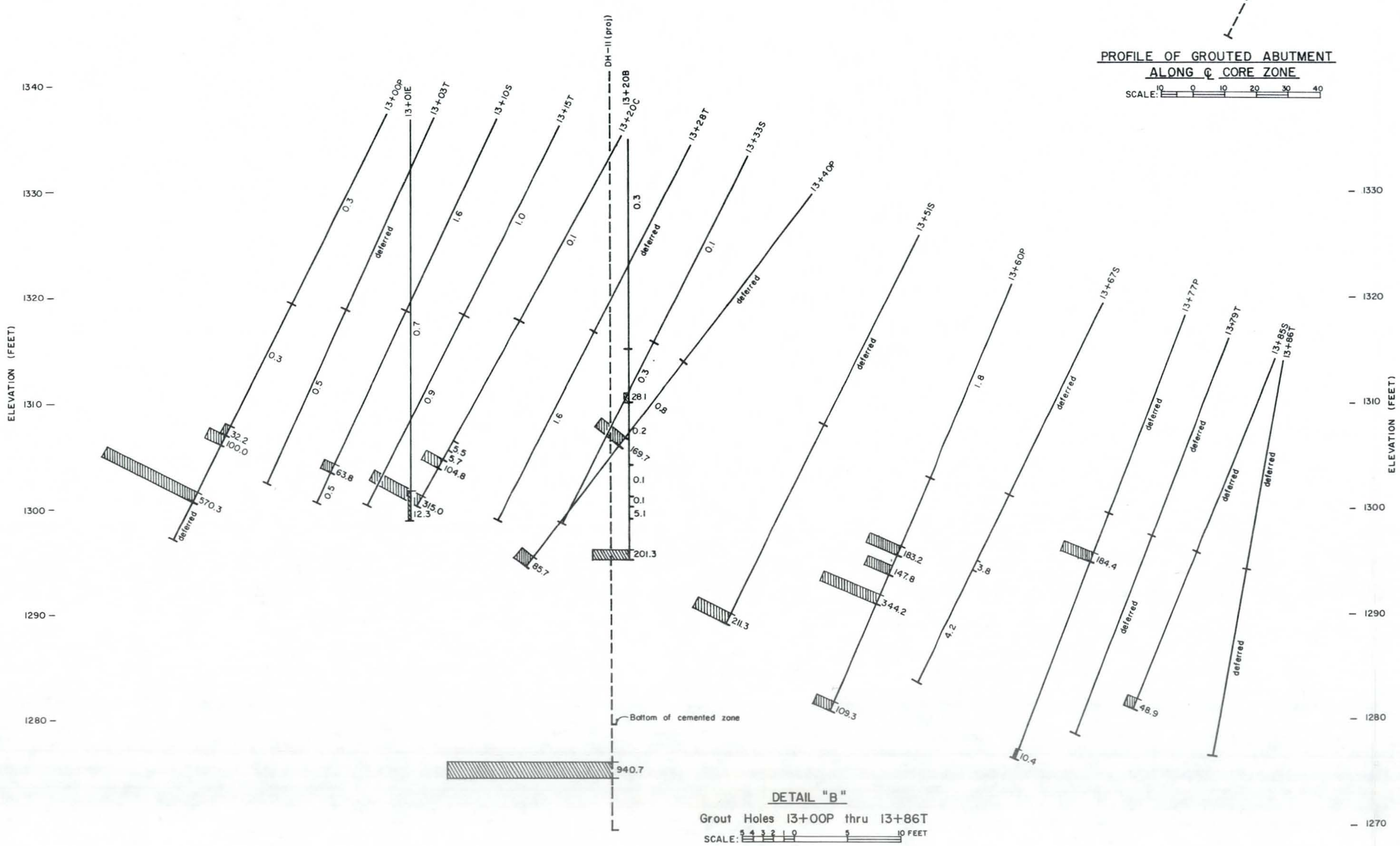


PLAN OF GROUTED ABUTMENT



PROFILE OF GROUTED ABUTMENT ALONG ℄ CORE ZONE

SCALE: 10 0 10 20 30 40



DETAIL "B"

Grout Holes 13+00P thru 13+86T

SCALE: 5 4 3 2 1 0 5 10 FEET

LEGEND:

- o — EX-size grout hole
- DH-9 O — NW-size core hole showing hole designation.
- Class of hole (P = primary; S = secondary; T = tertiary; Q = quaternary; Q_2 = quinary; E = exploratory core hole; R and L = conduit contact.
- Grout hole number (approximate dam stationing)
- Bags of cement injected in a zone.
- Lower limit of zone (first zone 0 to 20', second zone usually 20 to 40')
- Limits of stage within a zone showing bags of cement injected. (see note 4 below)
- Grouting deferred due to low calculated permeability.

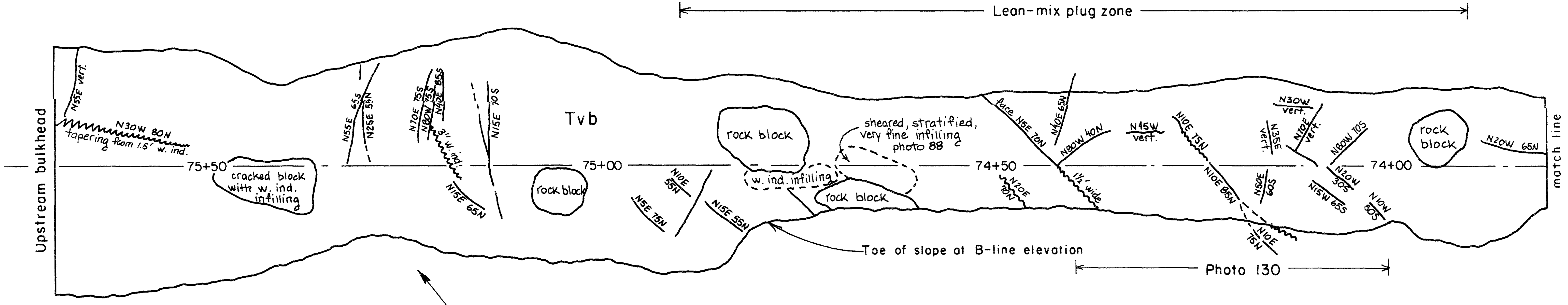
NOTES:

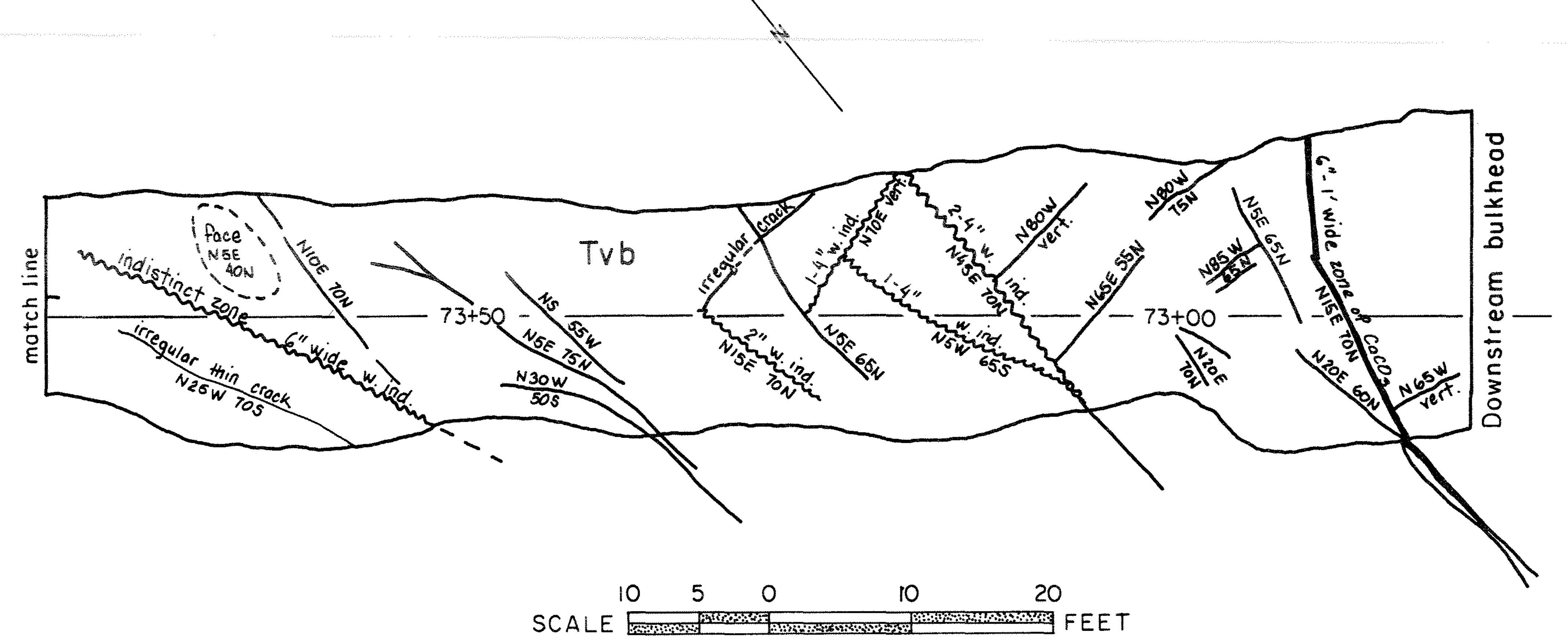
1. All grout holes rotary wash drilled using EX-size plug bit on CP-65 pneumatic drill.
2. Grout holes 9+65Q_2 thru 11+89S inclined out-of-slope due to slight in-slope dip of steeply dipping fractures, especially above station 11.
3. Projected core holes DH-9, DH-10, DH-11 and DH-23 were drilled during the design phase prior to dam construction.
4. Stage grouting was performed whenever a permanent loss of return drill water occurred. It is assumed that the majority of grout injected in a stage was at or near the point where drilling was stopped, therefore the grout take is shown in an isolated interval arbitrarily shown as one-foot wide.
5. See table 3 for exact hole locations, water pressure test data, grouting pressures and mix ratios.
6. Detailed profiles depict areas of significant grout take.

ADOBE DAM FOUNDATION REPORT

RIGHT ABUTMENT FOUNDATION GROUTING

PLAN AND PROFILES





NOTES:

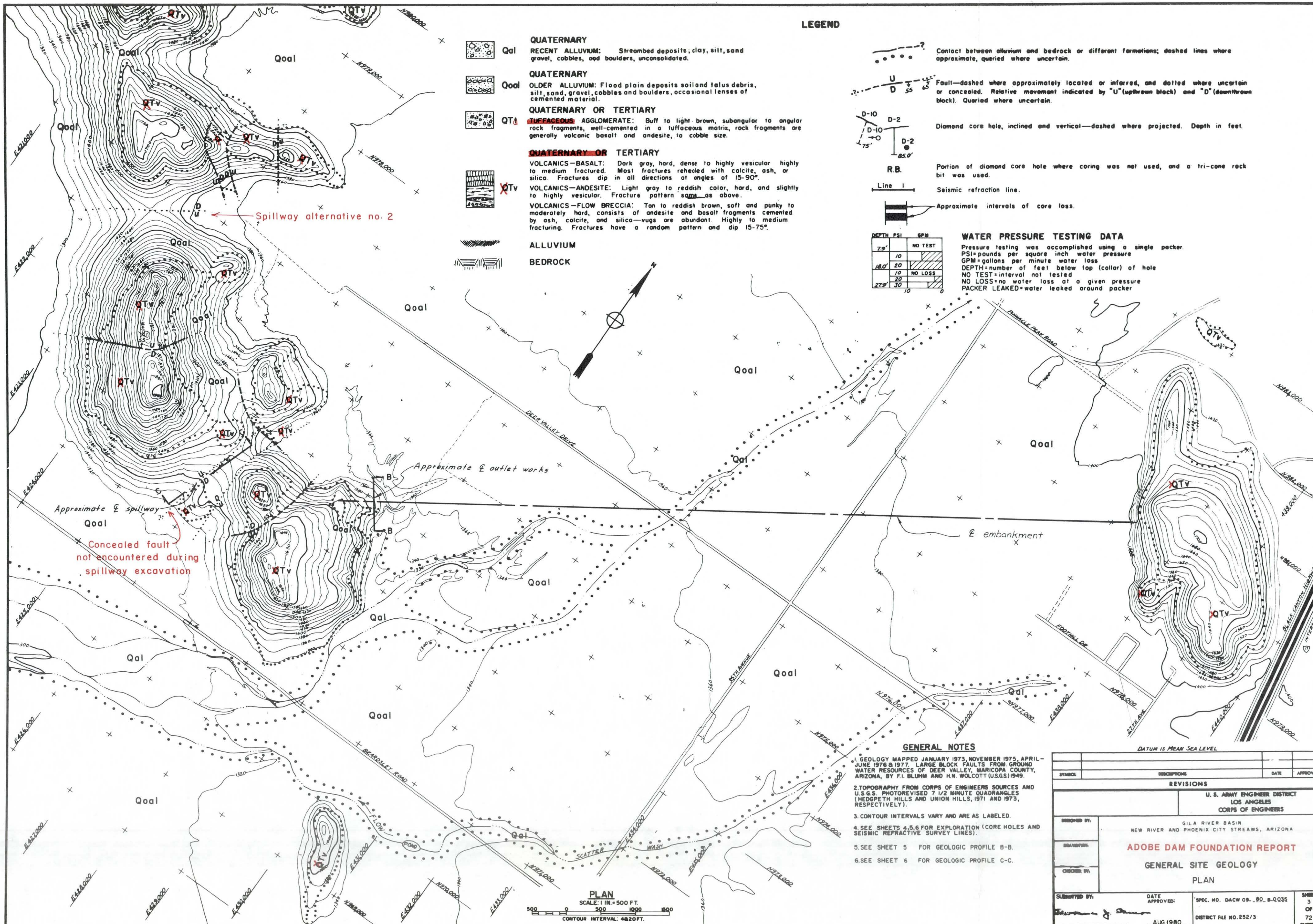
1. The foundation rock unit is Tvb, Tertiary basalt block flow. The floor surface is composed of irregularly shaped basalt (latite) blocks to 8-foot maximum unbroken dimension with semi-smooth to rough surfaces separated and infilled by fine grained, generally unstratified, tuffaceous sand 1 to 8-inches wide.
2. The structures mapped represent either: 1) planar surfaces on larger rock blocks, 2) alined lineations between rock blocks or 3) linear zones of broken rock or infilling.
3. Where shown beyond the limits of the slope toe, the lineations extend from the floor up the side-wall of the excavation.
4. Most lineations are relatively indistinct and can not be projected more than 10 or 15 feet.
5. Brecciated zones of measurable thickness are indicated by wavy lines.
6. The notation w. ind. indicates well indurated tuffaceous sand infilling.
7. The foundation under the inlet structure and energy dissipator was not mapped because of cleaning/concrete placement scheduling.
8. Bulkheads were constructed to contain the B-line leveling slab.

ADOBE DAM FOUNDATION REPORT

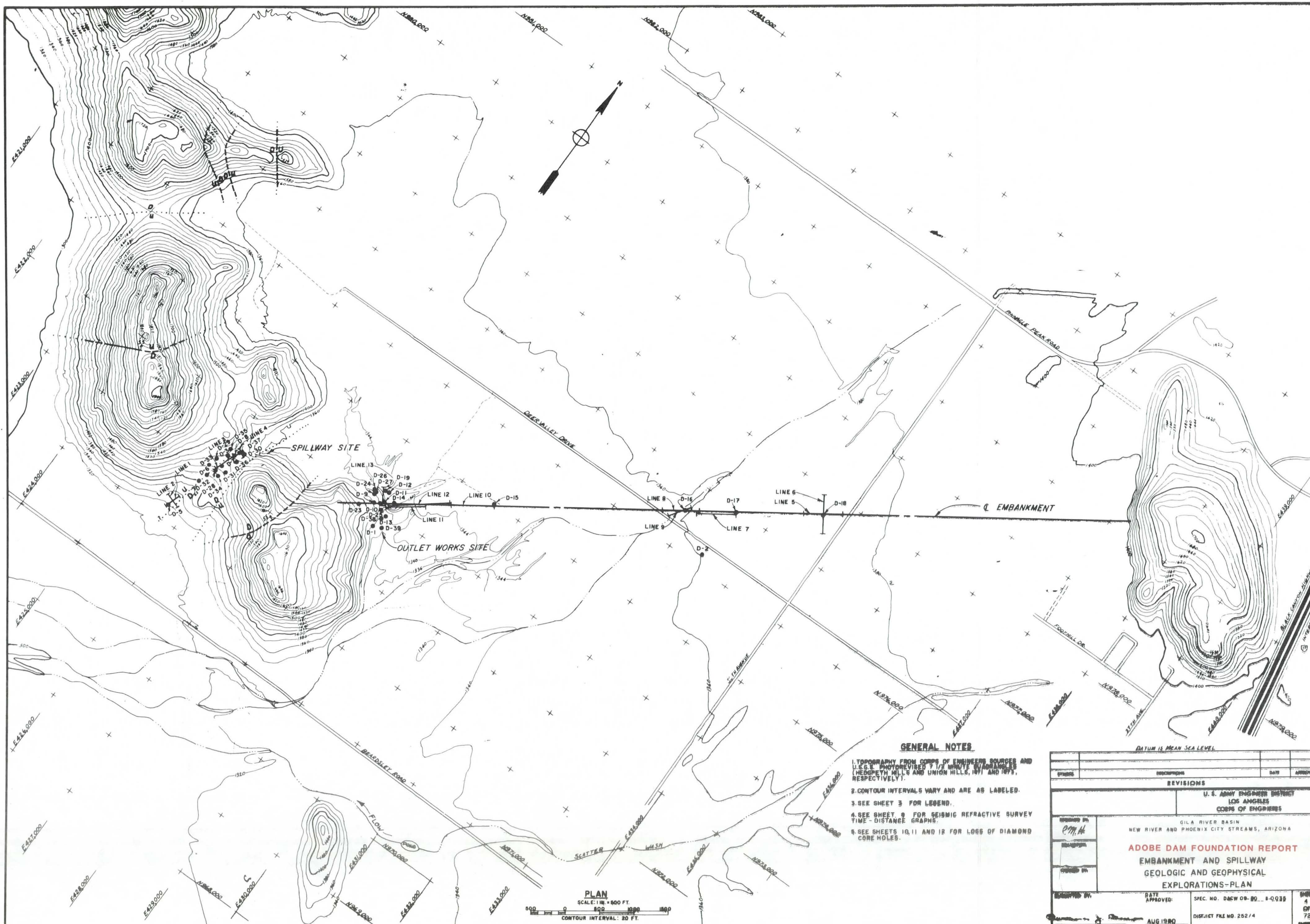
OUTLET WORKS TRENCH FLOOR FOUNDATION GEOLOGY

Amended Plates

Twenty of the 72 original contract drawings that relate to the geotechnical aspects of Adobe Dam were selected for inclosure in the foundation report. At publication time, the official as-built drawings were not available. Wherever possible, as-built changes are shown in red on the amended plates along with geotechnically related modifications to the original drawings that will not be shown on the final as-built plates. Both the as-built and the 20 amended plates should be referred to for all modifications to the contract drawings.



AMENDED PLATE 1



GENERAL NOTES

1. TOPOGRAPHY FROM CORPS OF ENGINEERS SOURCES AND U.S.G.S. PHOTOREVISED 7 1/2 MINUTE QUADRANGLES (HEDGPETH HILLS AND UNION HILLS, 1971 AND 1973, RESPECTIVELY).
2. CONTOUR INTERVALS VARY AND ARE AS LABELED.
3. SEE SHEET 3 FOR LEGEND.
4. SEE SHEET 9 FOR SEISMIC REFRACTIVE SURVEY TIME - DISTANCE GRAPHS.
5. SEE SHEETS 10, 11 AND 12 FOR LOGS OF DIAMOND CORE HOLES.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

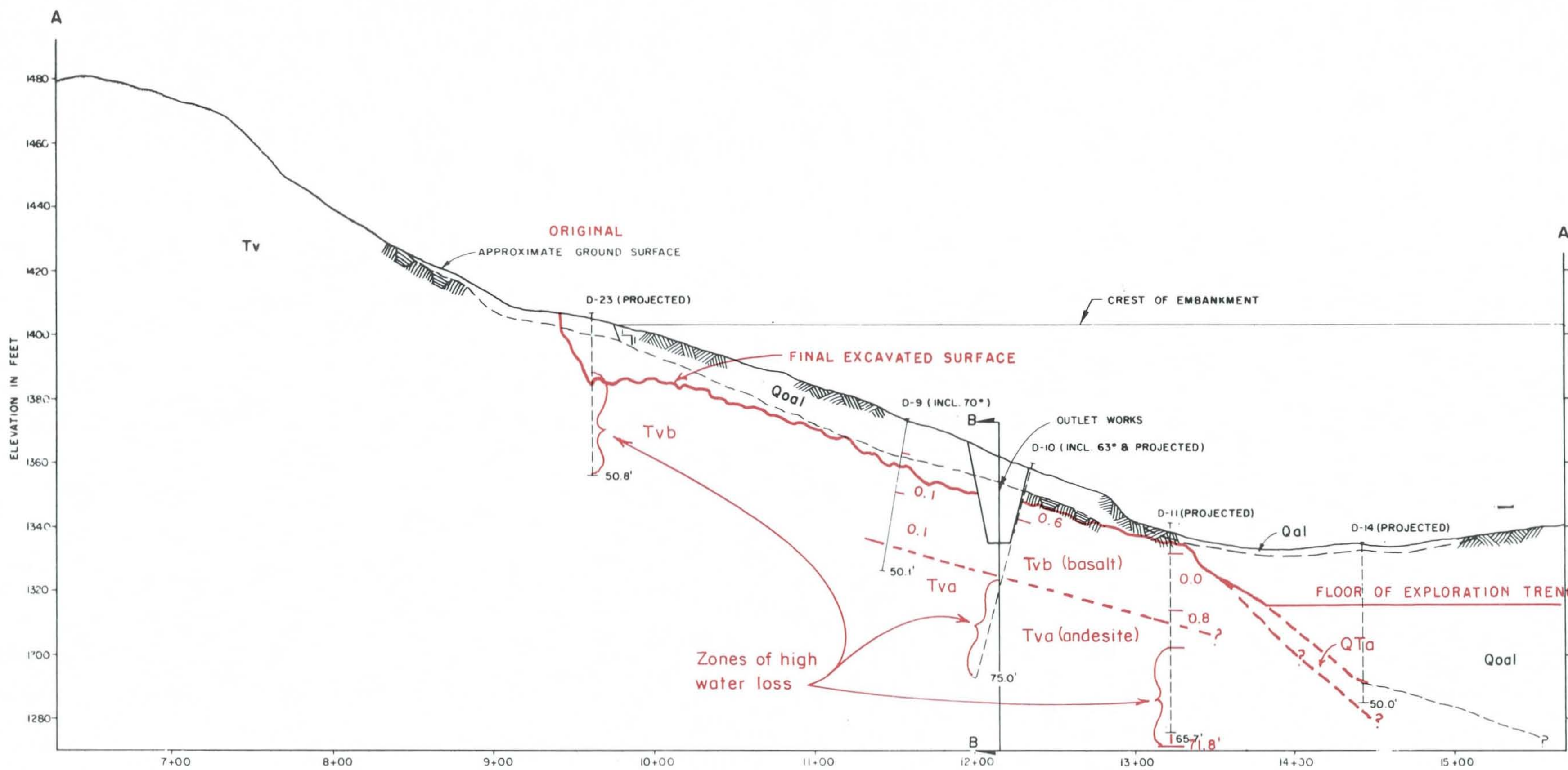
DESIGNED BY: C.M.H.
DRAWN BY:
CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT
EMBANKMENT AND SPILLWAY
GEOLOGIC AND GEOPHYSICAL
EXPLORATIONS-PLAN

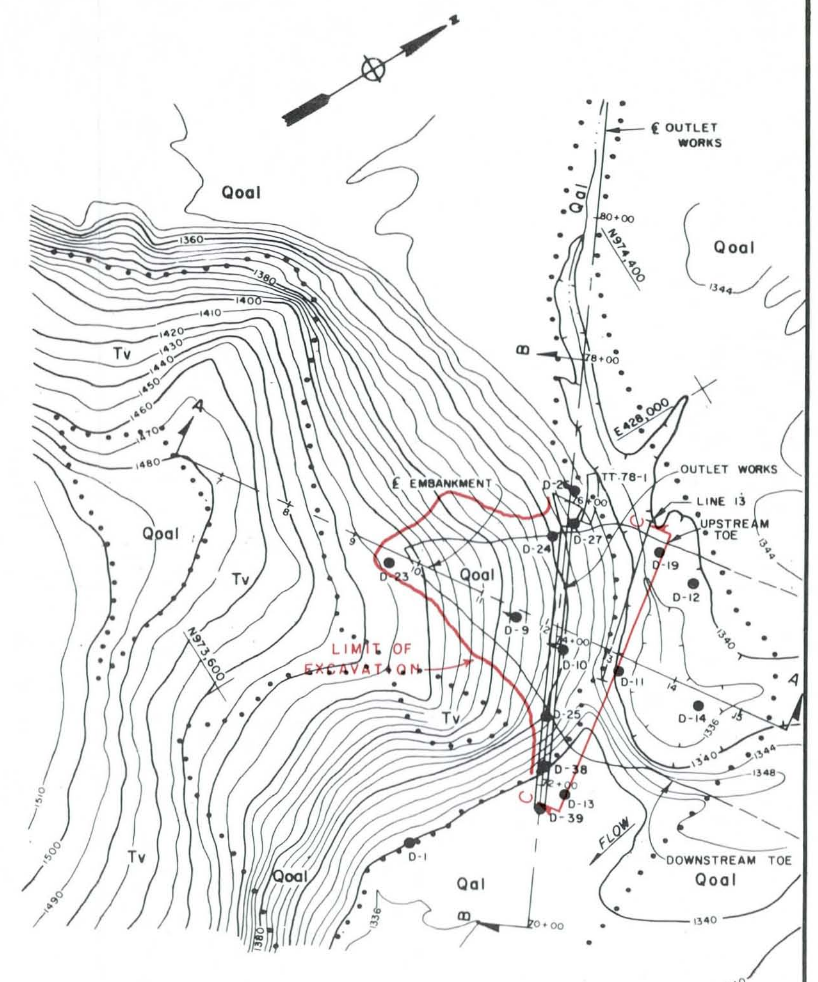
SUBMITTED BY:
DATE APPROVED: AUG 1980
CHIEF E & D. DIVISION
SPEC. NO. DACW 09-80-B-0035
DISTRICT FILE NO. 252/4
SHEET 4 OF 72 SHEETS

AMENDED PLATE 2



PROFILE A-A (WEST ABUTMENT)

HORIZ. SCALE: 1 IN. = 40 FT.
VERT. SCALE: 1 IN. = 20 FT.

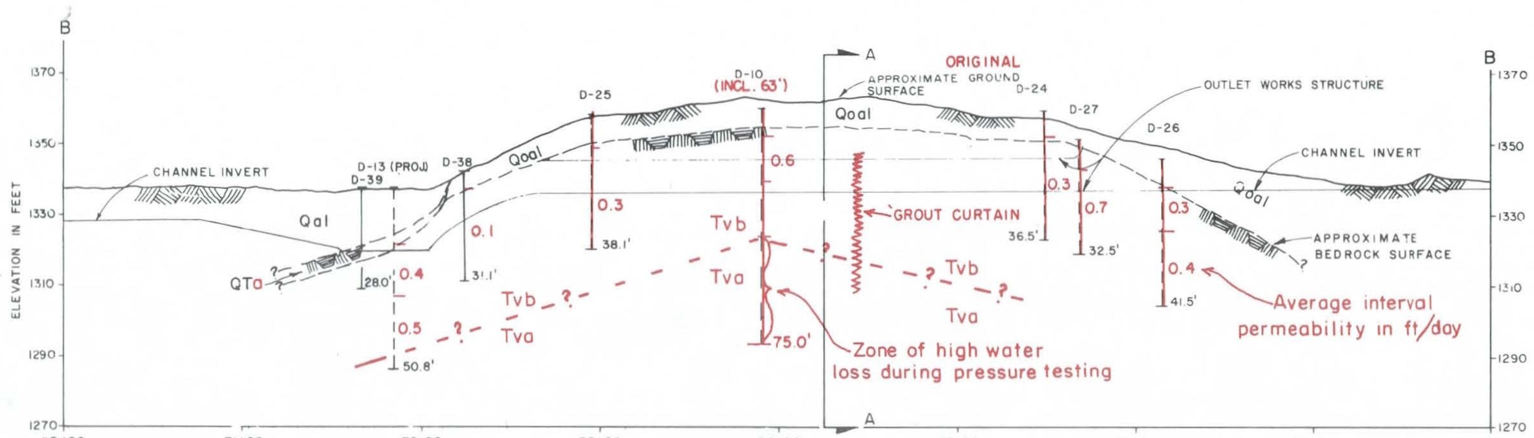


PLAN (WEST ABUTMENT AND OUTLET WORKS)

SCALE: 1 IN. = 100 FT.
CONTOURS ARE IN 4 FT AND 10 FT INTERVALS

GENERAL NOTES

1. SEE SHEET 3 FOR GENERAL GEOLOGY OF AREA AND LEGEND.
2. SEE SHEETS 4 & 6 FOR LOCATION OF OTHER SEISMIC-REFRACTIVE SURVEY LINES.
3. SEE SHEET 10, 11 AND 12 FOR LOGS OF DIAMOND CORE HOLES AND TEST TRENCH.
4. SEE FIGURE 6 FOR GEOLOGIC CROSS SECTION C-C.
5. WHERE NECESSARY FOR CLARITY, THE ORIGINAL DRAWING WAS MODIFIED PRIOR TO ADDING NEW (RED) INFORMATION.



PROFILE B-B ALONG ℄ OUTLET WORKS

HORIZ. SCALE: 1 IN. = 40 FT.
VERT. SCALE: 1 IN. = 20 FT.

DATUM IS MEAN SEA LEVEL

SCALE: 1 IN. = 100 FT.
SCALE 100 0 100 200 300 FEET

SCALE: 1 IN. = 40 FT.
HORIZ. SCALE 50 0 50 100 FEET

SCALE: 1 IN. = 20 FT.
VERT. SCALE 10 0 10 20 30 40 50 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT
WEST ABUTMENT AND OUTLET WORKS
GEOLOGY AND FOUNDATION EXPLORATION
PLAN AND PROFILES

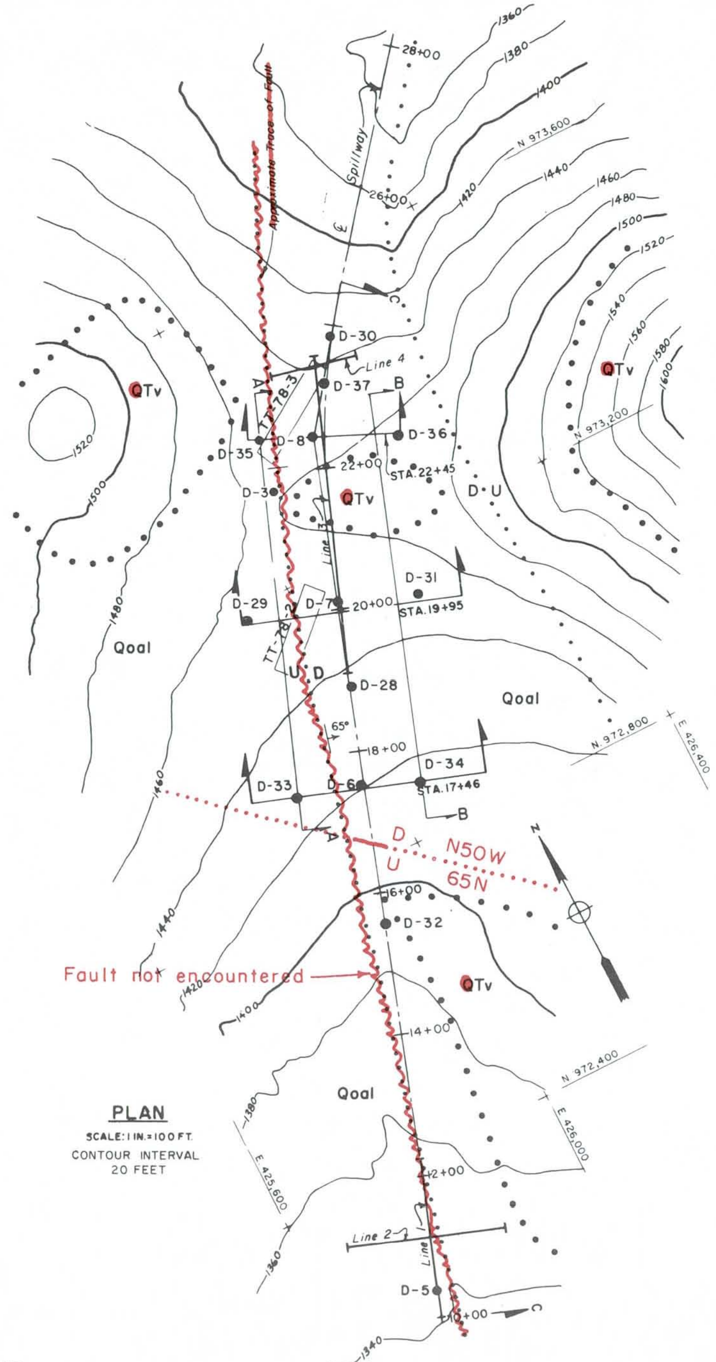
DESIGNED BY: | DRAWN BY: | CHECKED BY: | SUBMITTED BY:

DATE APPROVED: AUG 1980

SPEC. NO. DACW 09-80-B-0035
DISTRICT FILE NO. 252/5

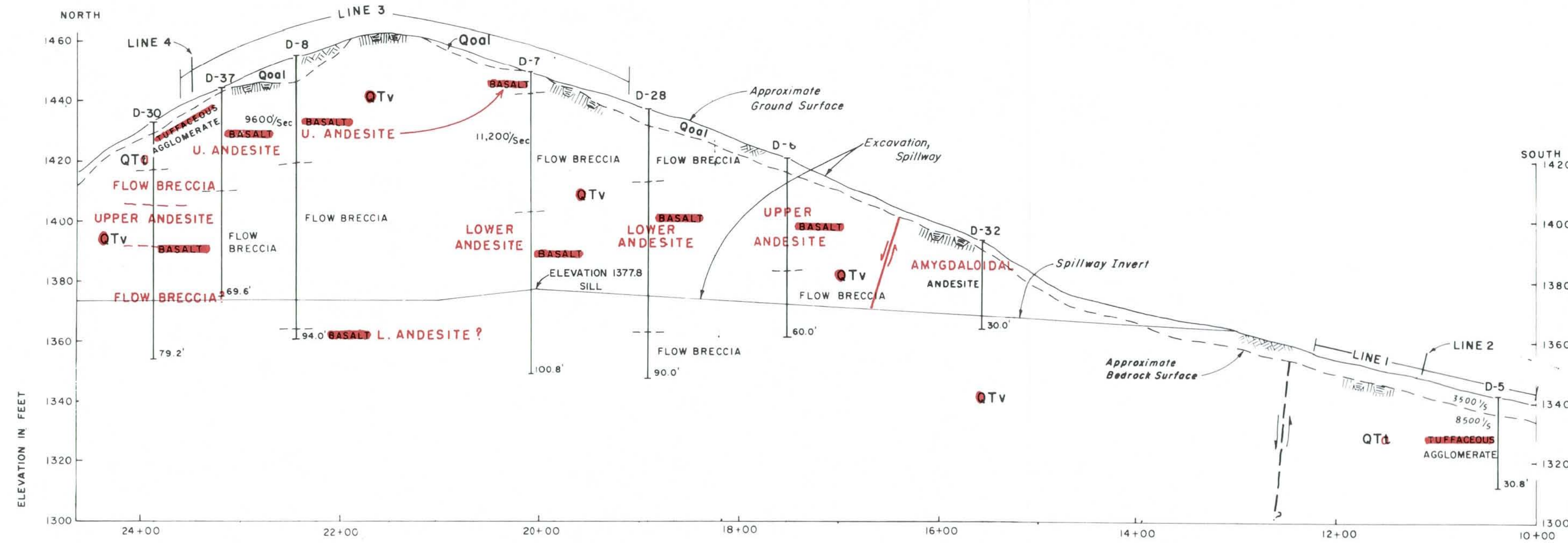
SHEET 5 OF 72 SHEETS

AMENDED PLATE 3



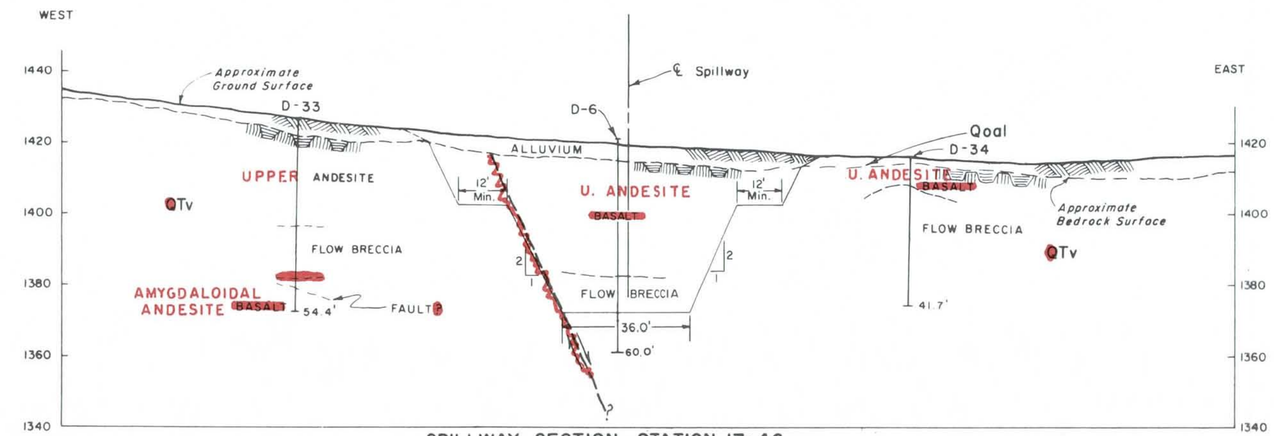
PLAN

SCALE: 1 IN. = 100 FT.
CONTOUR INTERVAL 20 FEET



PROFILE C-C ALONG SPILLWAY ℄

VERTICAL SCALE: 1 IN. = 20 FT.
HORIZONTAL SCALE: 1 IN. = 60 FT.



SPILLWAY SECTION—STATION 17+46

SCALE: 1 IN. = 20 FT.

GENERAL NOTES

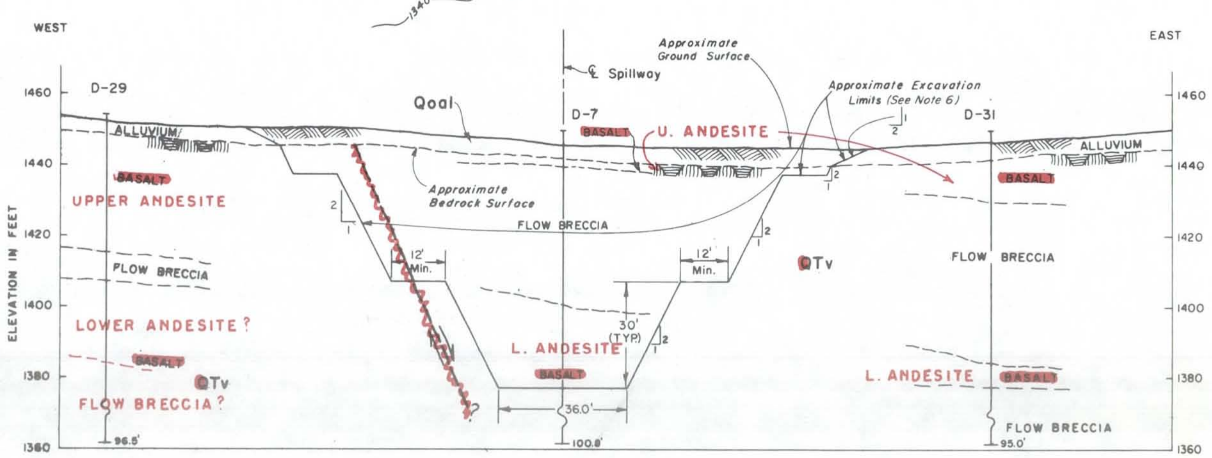
1. SEE SHEET 3 FOR LEGEND AND GEOLOGY OF ENTIRE SITE.
2. SEE SHEETS 10, 11 AND 12 FOR LOGS OF DIAMOND CORE HOLES.
3. GEOLOGY FROM CORE HOLE DATA AND GEOLOGIC MAPPING DONE IN 1973 AND 1975–77.
4. FAULT DATA FROM BLUHM & WALCOTT, 1949 (SEE NOTE 1 ABOVE). THIS IS SUPPLEMENTED BY DRILL HOLE DATA, REFRACTIVE SEISMIC DATA, ADDITIONAL GEOLOGIC MAPPING, AND TOPOGRAPHIC FEATURES.
5. CROSS SECTIONS AND PROFILES ARE FURTHER SEPARATED INTO INTERVALS OF FLOW BRECCIA, BASALT AND ANDESITE WITHIN THE "QTv" DESIGNATION. THIS IS A GUIDE FOR INTERPRETATION OF GEOLOGY FOR EXCAVATION PURPOSES.
6. CUT SLOPE AND BENCH WIDTH ON WEST SIDE OF SPILLWAY MAY BE MODIFIED BY CONTRACTING OFFICER BASED ON THE CONDITION AND GEOMETRY OF FAULT, IF EXPOSED BY THE EXCAVATION.
7. SEE SHEET 9 FOR SEISMIC TIME-DISTANCE GRAPHS OF LINE 1, 2, 3, AND 4.
8. **ROCK UNIT NAMES IN CORE HOLES MODIFIED TO AGREE WITH MAPPING NOMENCLATURE.**

SCALE: 1 IN. = 20 FT.
VERT. 20 10 0 20 40 60 80 100

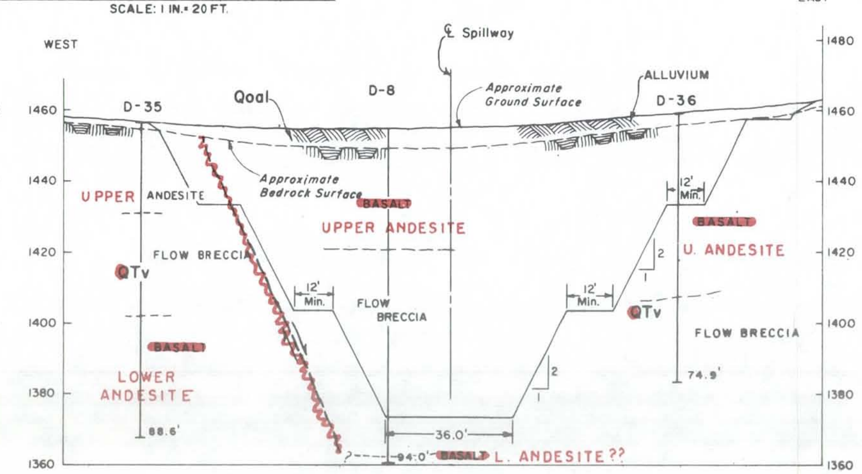
SCALE: 1 IN. = 60 FT.
HORIZ. 60 30 0 60 120 180 240 300

SCALE: 1 IN. = 100 FT.
100 50 0 100 200

DATUM IS MEAN SEA LEVEL



SPILLWAY SECTION—STATION 19+95



SPILLWAY SECTION—STATION 22+45

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

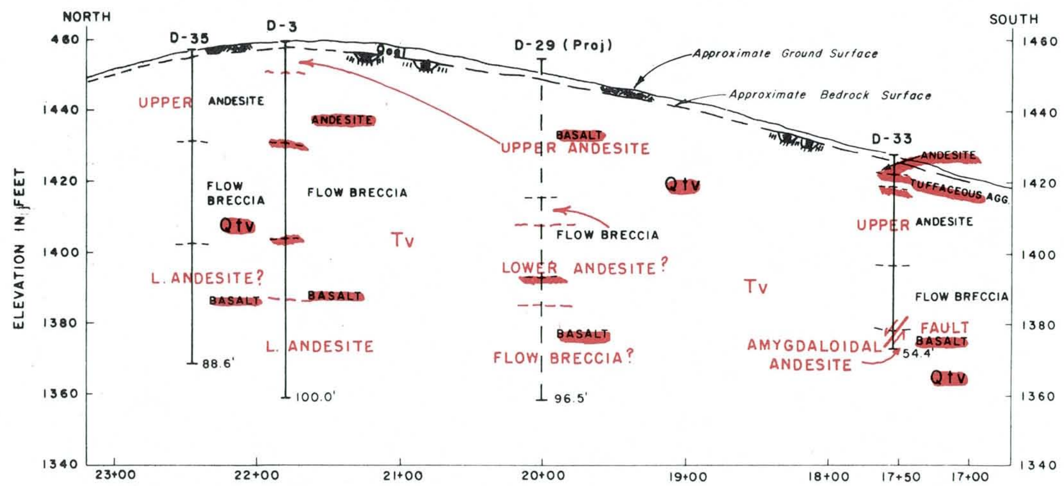
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA
ADOBE DAM FOUNDATION REPORT
SPILLWAY
GEOLOGY AND SEISMIC REFRACTIVE SURVEY
PLAN, PROFILE AND GEOLOGIC CROSS SECTIONS

SUBMITTED BY: DATE APPROVED: AUG 1980
CHIEF F. & S. BRANCH
SPEC. NO. DACW09-80-B-0035
DISTRICT FILE NO. 252/6
SHEET 6 OF 72 SHEETS

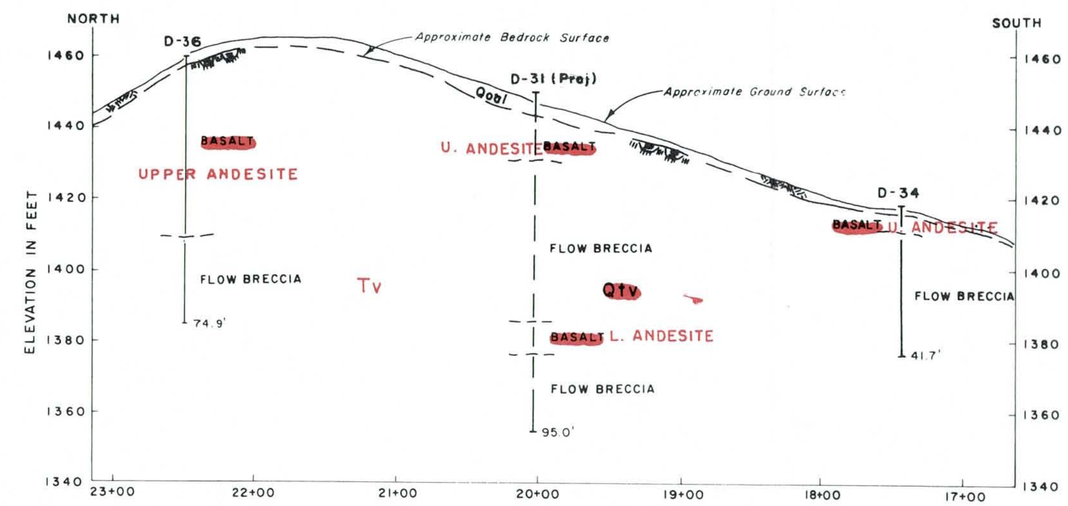
AMENDED PLATE 4



PROFILE A-A-85 FEET WEST OF SPILLWAY ℄

VERTICAL SCALE 1 IN=20 FT

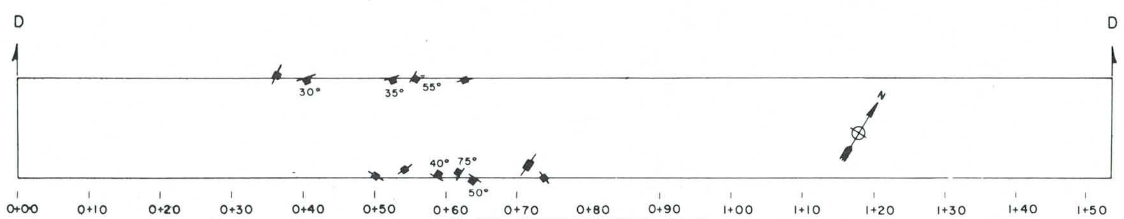
HORIZONTAL SCALE 1 IN=50 FT



PROFILE B-B-80 FEET EAST OF SPILLWAY ℄

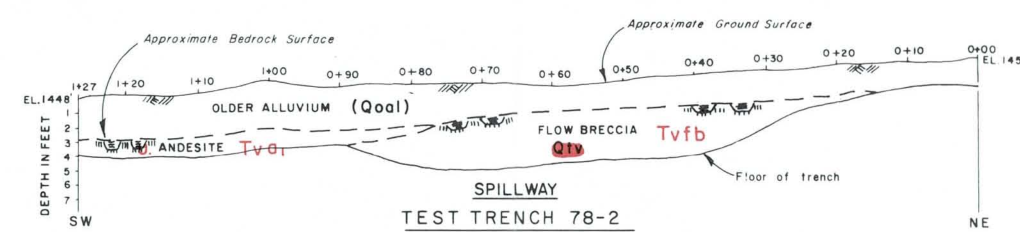
VERTICAL SCALE 1 IN=20 FT

HORIZONTAL SCALE 1 IN=50 FT



PLAN-TEST TRENCH 78-3

SCALE 1 IN=10 FT FEET



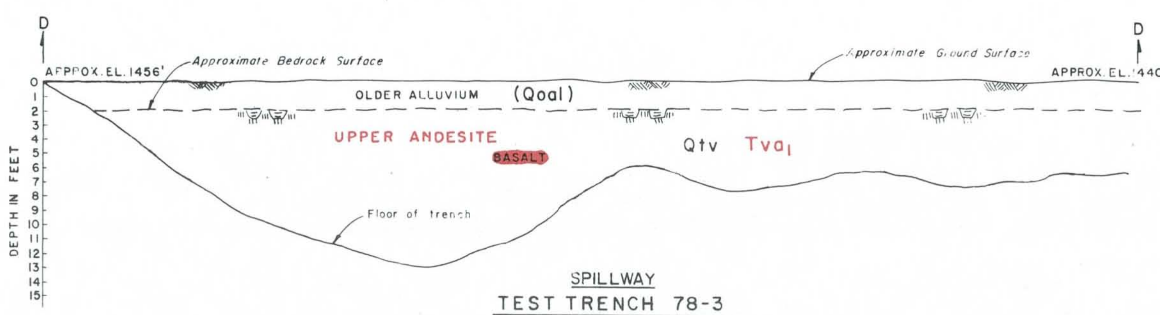
SPILLWAY

TEST TRENCH 78-2

PROFILE-WEST SIDE OF TRENCH

HORIZONTAL SCALE 1 IN=10 FT

VERTICAL SCALE 1 IN=5 FT



SPILLWAY

TEST TRENCH 78-3

PROFILE D-D-NORTH SIDE OF TRENCH

HORIZONTAL SCALE 1 IN=10 FT.

VERTICAL SCALE 1 IN=5 FT

NOTES

1. SEE SHEET 5 FOR LOCATIONS OF PROFILES A-A AND B-B.
2. SEE SHEET 6 FOR LOCATIONS OF TEST TRENCHES 78-2 AND 78-3.
3. TEST TRENCHES 78-2 AND 78-3 WERE EXCAVATED BY A D-9 DOZER WITH HYDRAULIC BLADE AND RIPPER TOOTH IN AUGUST 1978. EACH TRENCH WAS EXCAVATED TO REFUSAL.
4. SEE SHEET 3 FOR LEGEND.
5. SEE SHEETS 11 AND 12 FOR GEOLOGIC LOGS OF DIAMOND CORE HOLES SHOWN ON THIS SHEET.
6. ROCK UNIT NAMES IN CORE HOLES AND TRENCHES WERE MODIFIED TO AGREE WITH MAPPING NOMENCLATURE BASED UPON RE-EXAMINATION OF FIELD LOGS AND CORE PHOTOGRAPHS.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY:

CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT
SPILLWAY

GEOLOGY AND FOUNDATION EXPLORATION
PLAN AND PROFILES

SUBMITTED BY:

CHIEF, F.&M. BRANCH

DATE APPROVED: AUG 1980

SPEC. NO. DACW09-80-B-0035

DISTRICT FILE NO. 252/7

SHEET 7 OF 72 SHEETS

AMENDED PLATE 5

VALUE ENGINEERING PAYS

OUTLET WORKS

D-10 (INCL. 63°)

DEPTH PSI GPM: 8.0' NO TEST; 5, 10, 20.0' 20, 10, 20, 30, 40.7' 40, 45.3' 10, 20, 56.9' 30, 20, 40, 68.2' 60, 75.0'; 30 0

0.0' ELEV. 1360.0'

7.6' ALLUVIUM; slope wash and talus debris, boulders 4' max. dia., some caliche cementation.

VOLCANICS-BASALT; dark gray, hard, vesicular, most fractures are irregular and dip 60-90°. The fractures and cavities are filled with fine grained ash at 12.5-15.0', 16.0-17.4', 21.0-24.0', 25.5-26.4', 27.0-28.4', and 39.6-40.5', medium fractured 7.6-40.5' with one piece 16" long.

40.5' VOLCANICS-ANDESITE; gray, hard, mostly highly fractured rock with increments of med. fractured rock at 46-47.7', 49.9-51.0', 60.8-72.4', and slightly fractured from 72.4-75.0'. Some fractures are filled and coated with calcium cemented ash. Fractures dip 5-10° below 50.0'.

75.0' 89% Core recovery. April 1976

D-13

DEPTH PSI GPM: NO TEST; 16.0' 10, 20, 31.4' 30, 5, 15, 35 50.8'; 10 0

0.0' ELEV. 1337.2'

ALLUVIUM; sand to clayey sand, some gravelly sand, poorly cemented to 12.0', some caliche cementation 12.0-17.8', boulder 5' max. dia.

17.8' VOLCANICS-BASALT; dark gray, hard, slightly vesicular, slightly fractured 17.8-34.0', med. to highly fractured from 34.0-39.8'. BELOW 39.8' ABUNDANT TAN, CLASTIC TUFF AND ANGULAR BASALT CLASTS; WELL CEMENTED

~~39.8'~~ ~~TUFFACEOUS AGGLOMERATE; light buff to gray, med. hard to hard, well cemented large rock fragments slight to med. fractured. Fractures calcite filled.~~

47.0 VOLCANICS – ANDESITE - GREY; HARD; DENSE, CALCITE HEALED FRACTURES

50.8' 93% Core recovery. April 1976

D-24

DEPTH PSI GPM: 8.5' NO TEST; 5, 15, 25, 36.5' 35; 4 0

0.0' ELEV. 1359.8'

6.0' ALLUVIUM; slope wash and talus debris, boulders to 5' max. dia., caliche cemented.

VOLCANICS-BASALT; dark gray to black, hard, vesicular, occasional tuff intervals, medium to highly fractured. Core loss due to possible tuff filled cavities.

~~24.2'~~ ~~TUFFACEOUS AGGLOMERATE; tan, rock fragments to 4" long, highly to medium fractured, fractures calcite filled.~~ BELOW 24.2' GREATER PERCENTAGE OF TAN, CLASTIC TUFF INFILLING

36.5' 81% core recovery, June 1976

D-25

DEPTH PSI GPM: 9.2' NO TEST; 10, 20, 30, 38.1'; 5 0

0.0' ELEV. 1358.5'

8.4' ALLUVIUM; slope wash and talus debris, boulders to 5' dia. caliche cemented.

VOLCANICS-BASALT; dark gray to black, hard, vesicular, nearly all fractures are coated with tuff, fractures dip mostly 30° and 60°. vesicules dip 30°, medium to highly fractured INTERVAL FROM 30.1 TO 34.0' TAN, CLASTIC TUFF

~~30.1'~~ ~~34.0'~~ ~~TUFFACEOUS AGGLOMERATE; tan, rock fragments 4" long.~~

38.1' ~~VOLCANICS-BASALT; dark gray, vesicular, medium to highly fractured.~~

91% core recovery. June 1976

D-26

DEPTH PSI GPM: 8.0' NO TEST; 10, 22.0' 20, 10, 20, 30, 41.5' 40; 7 0

0.0' ELEV. 1346.3'

5.0' ALLUVIUM; talus, slope wash.

VOLCANICS-BASALT; dark gray to black, hard, vesicular, occasional tuff intervals, medium to highly fractured. Core loss due to possible tuff filled cavities.

41.5' 72% core recovery. May 1977

D-27

DEPTH PSI GPM: 8.0' NO TEST; 5, 15, 25, 32.5' 35; 12 0

0.0' ELEV. 1351.7'

5.0' ALLUVIUM; talus debris and slope wash, boulders to 3' dia.

VOLCANICS-BASALT; dark gray to black, hard, vesicular, occasional tuff intervals, medium to highly fractured

32.5' 85% core recovery. May 1977

D-38

DEPTH PSI GPM: 7.5' NO TEST; 8 NO LOSS; 31.1'

0.0' ELEV. 1342.0'

5.1' ALLUVIUM; talus debris, boulders to 3' max. dia.

BASALT

VOLCANICS-ANDESITE; gray, hard to very hard, slightly vesicular, slightly fractured, some core pieces over 2' long. Occasional fractures and cavities up to 7" wide filled with poorly to well cemented fine grained clastic ash

31.1' 89% core recovery. JULY 1978

D-39

0.0' ELEV. 1337.2'

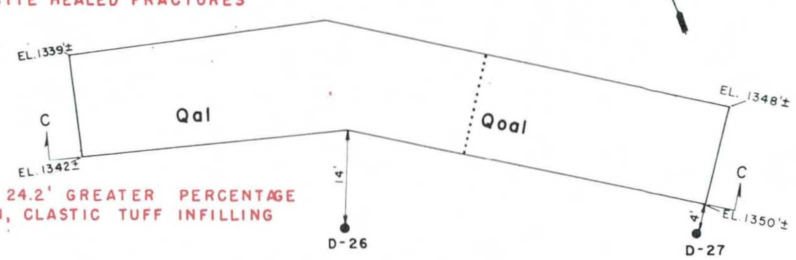
ALLUVIUM; gravelly sand to clayey gravelly sand with cobbles and boulders to 3' maximum diameter.

15.9' 20.3' TUFFACEOUS AGGLOMERATE; volcanic rock fragments to 6' diameter in a tan buff, medium to well indurated calcitic tuff matrix

28.0' BASALT

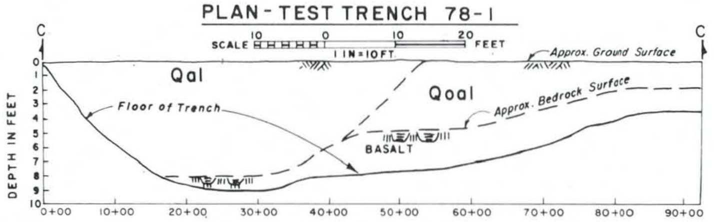
VOLCANICS-ANDESITE; gray, hard, slightly vesicular, slightly fractured some core pieces 3' long. Occasional fractures and cavities up to 6" wide filled with poorly to well cemented fine grained clastic ash

100% core recovery. JULY 1978. No pressure tests. Good water return.



PLAN - TEST TRENCH 78-1

SCALE 10 0 10 20 FEET, 1 IN = 10 FT



OUTLET WORKS
PROFILE C-C - SOUTH WALL
TEST TRENCH 78-1

HORIZONTAL SCALE 10 0 10 20 FEET, 1 IN = 10 FT
VERTICAL SCALE 5 0 5 10 FEET, 1 IN = 5 FT

TT 78-1

STA. 26 + 00

0' ALLUVIUM; unconsolidated silty sand gravel, cobbles and boulders 3-5' diameters, numerous thin layers of caliche between 2-6' depth.

8' 9' BASALT; blue and gray, hard. Bottom of Trench. August 1978

STA. 60 + 00

0' ALLUVIUM; talus debris, slope wash boulders 2-5' diameter cemented with caliche.

5' 7' BASALT; blue gray, hard. Bottom of Trench. August 1978

TT 78-1 (CONT'D.)

STA. 82 + 00

2' ALLUVIUM; talus debris, slope wash, boulders coated with caliche.

4' BASALT; gray, hard fine grained. Bottom of Trench. August 1978

SCALE: 1 IN = 20 FT.
10 0 10 20 30 40 50

WEST ABUTMENT

D-1

DEPTH PSI GPM: 8.0' NO TEST; 10.2' 10 NO LOSS; 27.2' 20; NO TEST; 36.1' 5, 43.1' 10, 8, 51.3' 15, 10 NO LOSS, 60.6' 30, 10, 72.2' 20, 75.8' 20, 81.2'; 25 0

0.0' ELEV. 1340.0'

8.0' ALLUVIUM; residual material and talus, moderately to well cemented sub-rounded rock fragments with caliche.

14.2' TUFFACEOUS AGGLOMERATE; buff to tan, hard, well cemented, a variety of rock fragments in a calcareous matrix, med. fractured.

VOLCANICS-ANDESITE; gray, hard. Fractures: 14.2-15.2' highly fractured, 15.2-32.0' slightly fractured, 32.0-81.2' med. fractured. Most fractures are rehealed, cemented with calcite or clay and steeply dipping. Gouge zone 56-57' dipping 80°. Fractures dip at all angles, but mostly at 45°, 60°, and 90°.

81.2' 93% Core recovery. January 1973

D-9 (INCL. 70°)

DEPTH PSI GPM: 10.0' NO TEST; 10, 20, 25.2' 30, 10, 30, 50 50.1'; 1 0

0.0' ELEV. 1373.8'

11.8' ALLUVIUM; slope wash and talus debris, boulders 4' max. dia., some caliche cementation.

VOLCANICS-BASALT; dark gray, hard, vesicular and slightly vuggy. Irregular fracturing and cavities filled with fine grained ash material at 27.2-28.2', 35.2-35.6', 36.7-38.0', 40.1-42.0' (less than 2" wide), most rock is med. to slightly fractured. Core loss due primarily to cavities

41.6' VOLCANICS-ANDESITE; gray, hard, med. to highly fractured 41.6-50.1', fractures dip 10-30°.

50.1' 87% Core recovery. May 1976

D-11

DEPTH PSI GPM: 10.1' NO TEST; 5 NO LOSS; 27.8' 10 NO LOSS; 10, 39.5' 20, 44.8' 30, 5, 51.8' 10, 52.7' 15, 62.3' 5, 65.7'; 20 0

0.0' ELEV. 1341.3'

10.1' ALLUVIUM; sandy clay, talus boulders to 6' max. dia. and slope wash, caliche cemented.

VOLCANICS-BASALT; dark gray, slightly vesicular, occasional fractures and cavities up to 3" wide filled with fine grained ash material. Fractures are irregular, med. fractured with one piece 18" long.

30.0' VOLCANICS-ANDESITE; gray, hard, mostly med. fractured to 41.0' with highly fractured increments from 30-32.0', 36.9-37.6', and 38.0-39.8' and highly fractured 41.0-65.7' with 2 pieces 10" and 18" long between 54.5-57.0'. Fractures dip 30-60° and occasionally 70-90°.

65.7' 94% Core recovery. April 1976

D-14

0.0' ELEV. 1334.9'

14.0' ALLUVIUM; sand to clayey sand, some gravelly sand, some cementation by caliche, occasional cobbles and boulders, more indurated below 14'.

43.1' TUFFACEOUS AGGLOMERATE; buff to tan, well cemented in calcareous rich matrix.

50.0' 87% Core recovery. No pressure tests were accomplished. June 1976

D-23

DEPTH PSI GPM: 8.0' NO TEST; 5, 10, 18.8' 15, 10, 27.9' 20, 30, 37.1' 40, 20, 30, 40, 50.8' 50; 2.5 0

0.0' ELEV. 1407.2'

5.4' ALLUVIUM; slope wash and talus debris, boulders to 5' dia. caliche cemented.

VOLCANICS-BASALT; dark gray to black, hard, vesicular, occasional tuff interval, cavities filled with ash at 9.5-10.0', 39.5-40.5', 41.2-41.7' and 44.0-45.5', medium to highly fractured 9.0-10.3', 15.0-16.5', 26.5-37.8' and 44.0-48.5', fractures dip 40-70°, vesicules dip 45-50°, slickensides and brecciation 36.5-37.0'

50.8' 96% Core recovery. May 1976

SCALE: 1 IN = 20 FT
10 0 10 20 30 40 50

GENERAL NOTES

1. SEE SHEET 3 FOR LEGEND.
2. SEE SHEETS 4,5&6 FOR PLAN VIEW, SHOWING LOCATION OF CORE HOLES.
3. FRACTURE CLASSIFICATION:
   HIGHLY FRACTURED ROCK - 0" TO 4" FRACTURE SPACING
   MEDIUM FRACTURED ROCK - 4" TO 12" FRACTURE SPACING
   SLIGHTLY FRACTURED ROCK - OVER 12" FRACTURE SPACING
4. ALLUVIUM WAS DRILLED WITH EITHER A ROCK BIT OR DIAMOND CORE BIT. NO CORE RECOVERY WAS RECORDED.
5. SEE SHEET 5 FOR LOCATION OF TEST TRENCH.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: | GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

DRAWN BY: | ADOBE DAM FOUNDATION REPORT
WEST ABUTMENT AND OUTLET WORKS
GEOLOGIC LOGS
AND
TEST TRENCH - PLAN AND PROFILE

CHECKED BY:

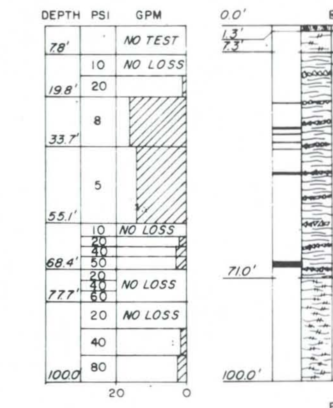
SUBMITTED BY: | DATE APPROVED: AUG 1980 | SPEC. NO. DACW09-80-B-0035 | DISTRICT FILE NO. 252/10 | SHEET 10 OF 72 SHEETS

SAFETY PAYS

AMENDED PLATE 6

SPILLWAY

D-3



0.0' ELEV 1459.0'

ALLUVIUM; residual material, silt size to gravel.

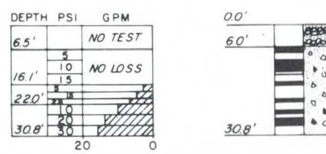
VOLCANICS-ANDESITE; gray, hard, dense, moderately fractured. Fractures are clay and calcite filled. Fractures dip 45-60°.

VOLCANICS-FLOW BRECCIA; reddish brown to gray, mostly soft and punky to moderately hard at 7.3-11.0', 15.0-20.0', 23.5-26.5' and 61.0-68.0'. Readily takes water.

VOLCANICS-ANDESITE; gray, hard, highly fractured. Occasional calcite filling on steep fractures between 93.0-100.0'. Fractures dip 50° at 77.0', 87.5', 88.2' and 90.0', 70° at 71.2', 72.8' and 93-94° and near-vertical from 84-86' and 95-100'.

95% Core recovery. February 1973

D-5



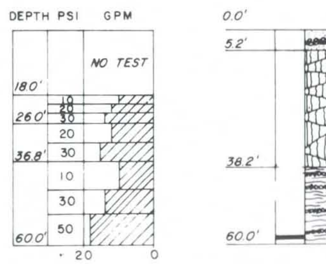
0.0' ELEV 1342.3'

ALLUVIUM; slope wash and talus debris; boulders 4' max. dia., poorly cemented with caliche.

TUFFACEOUS AGGLOMERATE; white to buff colored, rock fragments to 1" max. dia., poorly to well cemented in carbonate-rich matrix, core loss in poorly cemented intervals forms solution caverns, med. to highly fractured.

64% Core recovery. Core loss due to cavities. May 1976

D-6



0.0' ELEV 1421.0'

ALLUVIUM; slope wash and talus debris, boulders 3' max. dia., poorly cemented with caliche.

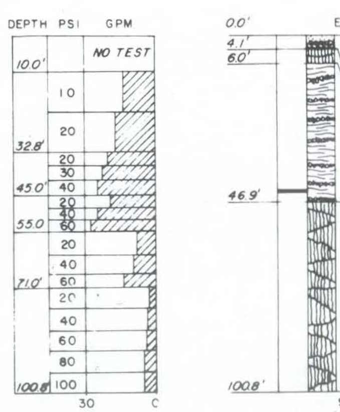
ANDESITE

VOLCANICS-BASALT; dark gray, hard and dense, slightly vesicular, numerous calcite veins throughout. Fractures dip all angles, but mostly 30°, 60°, and 90°. Highly fractured 5.2-35.0', med. fractured 35.0-38.2'.

VOLCANICS-FLOW BRECCIA; reddish, vesicular andesite fragments cemented by ash and caliche, med. to highly fractured.

99% Core recovery. June 1976

D-7



0.0' ELEV 1449.4'

ANDESITE

ALLUVIUM; slope wash and talus debris, boulders 3' maximum diameter.

VOLCANICS-BASALT; dark gray and dense.

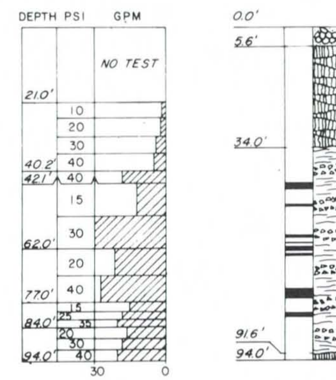
VOLCANICS-FLOW BRECCIA; dark gray and reddish vesicular andesite fragments cemented by ash and caliche. Medium fractured 6.0-15.0', slightly fractured 15.0-19.0', med. fractured 19.0-39.0' and highly fractured 39.0-46.9'.

ANDESITE

VOLCANICS-BASALT; dark gray, hard and dense, occasional thin andesite intervals at top and bottom of strata. Fractures at all angles, but mostly 35-40°, only slight alteration. Medium to slightly fractured 46.9-66.8', highly fractured 66.8-79.0', med. fractured 79.0-87.0', highly fractured 87.0-97.2', and med. to slightly fractured 97.2-100.8'.

99% Core recovery. May 1976

D-8



0.0' ELEV 1454.8'

ANDESITE

ALLUVIUM; slope wash and talus debris, boulders 4' max. dia., poorly cemented with caliche.

VOLCANICS-BASALT; dark gray, hard and dense, vesicular, med. fractured 5.6-17.0', med. to slightly fractured 17.0-23.0', highly fractured 23.0-31.0' and med. to slightly fractured 31.0-34.0'.

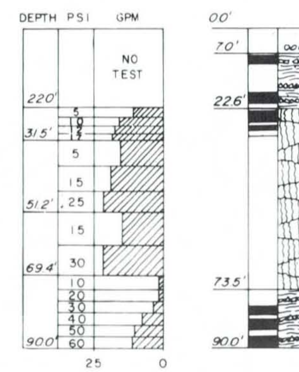
VOLCANICS-FLOW BRECCIA; reddish vesicular andesite fragments cemented by ash and calcite, med. to highly fractured 34.0-91.6'. Core loss is probably the results of poorly indurated clay matrix.

ANDESITE ?

VOLCANICS-BASALT; dark gray, hard, highly vesicular, med. to highly fractured.

90% Core recovery. May 1976

D-28



0.0' ELEV 1437.5'

ALLUVIUM; talus, slope wash.

VOLCANICS-FLOW BRECCIA; reddish brown, andesite fragments cemented by ash and calcite, high to medium fractured 7.0-22.6'. Core loss due to ground up highly fractured rock.

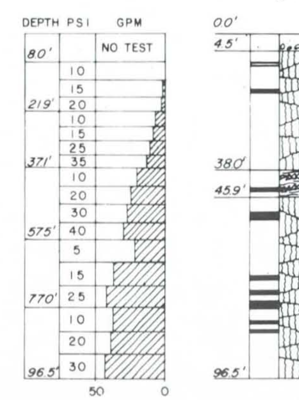
ANDESITE

VOLCANICS-BASALT; dark gray, hard, slightly vesicular, with fractures coated with calcite or iron oxide, highly fractured 22.0-38.5', 40.0-48.9', 50.2-66.9', and 68.1-73.5', medium fractured 38.5-40.0', 48.9-50.2', and 66.9-68.1'.

VOLCANICS-FLOW BRECCIA; reddish brown, same as between 7.0-22.6', high to medium fractured, core loss due to cavities.

86 % core recovery. April 1977

D-29



0.0' ELEV 1454.3'

ALLUVIUM; talus debris, slope wash.

ANDESITE

VOLCANICS-BASALT; dark gray to black, hard, vesicular, with fractures coated with calcite or iron oxide, high to medium fractured 4.5-38.0', joints dip 10-90°.

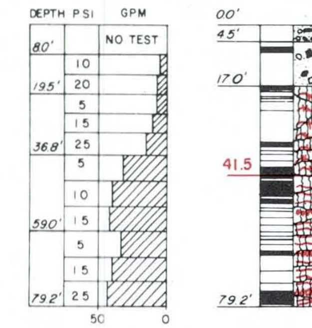
VOLCANICS-FLOW BRECCIA; reddish brown, andesite and basalt fragments cemented with ash and calcite, slightly fractured 38.0-41.8', high to medium fractured 41.8-45.9'.

ANDESITE & FLOW BRECCIA?

VOLCANICS-BASALT; same as between 4.5-38.0', high to medium fractured rock 45.9-96.5', small zone of slightly fractured rock 67.0-68.9', joints dip 30-90°, core loss due to possible tuff filled cavities.

92% core recovery. April 1977

D-30



0.0' ELEV 1432.6'

ALLUVIUM; residual soil, slope wash, boulders 3' max. diameter.

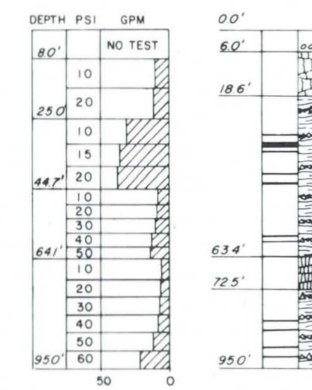
TUFFACEOUS AGGLOMERATE, white to tan, moderately hard, rock fragments to 4" long, well cemented, medium fractured.

ANDESITE & FLOW BRECCIA

VOLCANICS-BASALT; dark gray, hard, vesicular, with fractures coated with calcite or iron oxide, high to medium fractured 17.0-51.5', medium fractured 51.5-54.7', 55.9-66.4', 67.4-69.8', 71.6-77.8', 78.8-79.2', and slightly fractured 54.7-55.9', 66.4-67.4', 69.8-71.6', and 77.8-78.8', joints dip 10-60°, core loss due to possible tuff filled cavities.

79% core recovery. May 1977

D-31



0.0' ELEV 1449.4'

ALLUVIUM; residual soil and slope wash.

ANDESITE

VOLCANICS-BASALT; gray, hard, vesicular, fractures dip 20-70°, fractures coated with calcite, highly fractured 6.0-9.3' and medium fractured 9.3-18.6'.

VOLCANICS-FLOW BRECCIA; reddish vesicular andesite and black vesicular basalt, fragments cemented by ash, highly fractured.

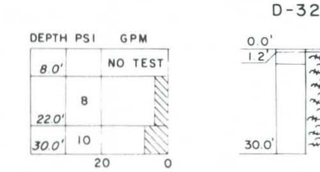
ANDESITE

VOLCANICS-BASALT; same as 6.0-18.6', medium fractured, fractures dip 45-90°.

VOLCANICS-FLOW BRECCIA; same as 18.6-63.4', highly fractured.

94% core recovery. May 1977

D-32



0.0' ELEV 1394.0'

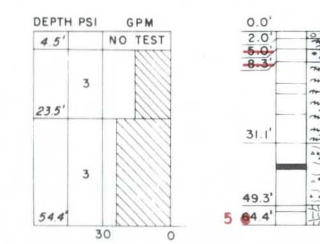
ALLUVIUM, slope wash.

AMYGDALOIDAL ANDESITE

VOCANICS-ANDESITE; gray to 28' then reddish gray, moderately hard to hard, slightly to highly vesicular, secondary calcite throughout, highly fractured to 24.0' then moderately fractured, silica/calcite cemented breccia 26.6' to 27.1'.

100 % core recovery. JUNE 1978

D-33



0.0' ELEV 1427.9'

ALLUVIUM; talus debris, slope wash.

VOLCANICS-ANDESITE; gray, moderately hard, dense, highly fractured.

~~TUFFACEOUS AGGLOMERATE; tan, moderately to poorly indurated, clasts to 4".~~

VOLCANICS-ANDESITE, gray, moderately hard, dense, highly fractured

VOLCANICS-FLOW BRECCIA; reddish brown, volcanic clasts up to 1.0', alternately dense and well indurated to vesicular and crumbly, slightly fractured, fault gauge and breccia 48.4' to 48.8'.

AMYGDALOIDAL ANDESITE

VOLCANICS-BASALT; gray, hard, moderately fractured

97 % core recovery. JUNE 1978

DATUM IS MEAN SEA LEVEL

GENERAL NOTES

1. SEE SHEET 3 FOR LEGEND.
2. SEE SHEET 12 FOR ADDITIONAL GEOLOGIC LOGS OF SPILLWAY.
3. SEE SHEETS 4,5&6 FOR LOCATION OF DIAMOND CORE HOLES.
4. ROCK UNIT NAMES CHANGED TO AGREE WITH MAPPING NOMENCLATURE. CORES WERE NOT RE-LOGGED.
5. ALLUVIUM WAS DRILLED WITH EITHER A ROCK BIT OR DIAMOND CORE BIT. NO CORE RECOVERY WAS RECORDED.
6. SEE SHEET 10 FOR FRACTURE CLASSIFICATION.
7. SEE SHEET 6 FOR SPILLWAY PROFILE.

SCALE: 1 IN = 20 FT.

10 0 10 20 30 40 50

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY:

CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT

SPILLWAY
GEOLOGIC LOGS

SUBMITTED BY: CHIEF, F. & M., BRANCH

DATE APPROVED: AUG 1980

SPEC. NO. DACW09-80-B-0035

DISTRICT FILE NO. 252/11

SHEET 11 OF 72 SHEETS

AMENDED PLATE 7

SPILLWAY

D-34

DEPTH	PSI	GPM
5.0'		NO TEST
18.2'	5	
	10	
37.9'	20	
41.7'		
	30	0

Depth	ELEV. 1418.4'
0.0'	
1.5'	ALLUVIUM; talus debris, slope wash.
6.3'	VOLCANICS-~~BASALT~~ ANDESITE; gray, hard, dense, medium fractured
41.7	VOLCANICS-FLOW BRECCIA; reddish, volcanic clasts to 1.5' long, alternately dense and well indurated to vesicular and crumbly, more competent core is medium to slightly fractured.

90% core recovery. JUNE 1978

D-35

DEPTH	PSI	GPM
5.0'		NO TEST
	10	
32.0'		
		NO TEST
54.5'		
	30	
73.4'		
		NO TEST
88.6'		
	20	0

Depth	ELEV. 1456.7'
0.0'	
1.5'	ALLUVIUM; talus debris, slope wash.
25.5'	VOLCANICS-ANDESITE; gray, moderately hard to hard, dense, highly fractured and jointed to 20.1' then medium fractured, very gradational contact with FLOW BRECCCA below.
54.5'	VOLCANICS-FLOW BRECCIA; reddish, mostly dense and well indurated with occasional crumbly zones, secondary calcite cementation above 40.1' intact core only slightly fractured, gradational contact with BASALT below.
88.6'	VOLCANICS-~~BASALT~~ ANDESITE, reddish gray to 70' then gray, hard to very hard, dense, highly fractured to 70.6' then medium fractured to 88.6' with occasional fracture spacing greater than 1.5'.

97 % core recovery JULY 1978

D-36

DEPTH	PSI	GPM
7.5'		NO TEST
	4	
30.0'		
		NO TEST
61.7'		
	0	
74.9'		
	30	0

Depth	ELEV. 1458.6'
0.0'	
2.0'	ALLUVIUM; Talus debris, slope wash
50.8'	VOLCANICS-~~BASALT~~ ANDESITE, gray, moderately hard, dense, highly fractured with occasional core pieces up to 0.9' long, prominent near, vertical fracture 14.9' to 24.9'
74.9'	VOLCANICS FLOW BRECCIA / ~~BASALT~~; reddish gray to dark gray, alternating intervals of crumbly flow material and hard, fine grained highly fractured, slightly vesicular BASALT. Core loss due to possible tuff filled cavities.

91% core recovery, JULY 1978

D-37

DEPTH	PSI	GPM
11.5'		NO TEST
	12	
30.9'		
	12	
51.0'		
	17	
69.6'		
	20	0

Depth	ELEV. 1444.1'
0.0'	
2.0'	ALLUVIUM; Talus debris, slope wash.
34.1'	VOLCANICS-~~BASALT~~ ANDESITE, gray with reddish tint below 31', moderately hard, dense, highly fractured and jointed.
42.0'	VOLCANICS-FLOW BRECCIA; reddish brown, well cemented with calcite, slightly fractured.
69.6'	VOLCANICS-BASALT/FLOW BRECCIA; dark gray and light brown, basalt is hard. slightly vesicular, medium to slightly fractured. It is separated into intervals less than 2' thick by crumbly brown flow material several inches thick.

99% core recovery. JULY 1978

TT-78-2

Depth	STA. 0+50
0	
2	ALLUVIUM; talus debris, slope wash boulders 1-2' dia. some caliche.
6	FLOW BRECCIA; Reddish brown, hard, dense, occasional clasts of vesicular basalt. Caliche filled joints.

Bottom of Trench. August 1978

TT 78-2

Depth	STA. 1+00
0.	
3	ALLUVIUM talus debris, slope wash boulders 1-2' dia.
4.	ANDESITE; blue gray, closely jointed.

Bottom of Trench. August 1978

TT 78-3

Depth	STA. 0+50
0'	
2'	ALLUVIUM; talus debris and slope wash boulders 2" Max. dia. w/cobbles and gravel.
13'	~~BASALT~~ ANDESITE, brownish gray, to bluish gray, fine hard, weathered and stained along joints. The joints are near horizontal (sheet jointing) about 1'-3" apart and parallel.

Bottom of Trench, August 1978

EMBANKMENT

D-2

Depth	ELEV. 1360.5'
0.0'	
100.0'	ALLUVIUM; valley fill, sandy clay with occasional gravel and occasional cemented intervals. Cobbles at 35', and between 44-47.5', stiff clay from 94-99.5'. Drilling mud was used. Good water return. No pressure tests.

No core recovery.
January 1973

D-12

DEPTH	PSI	GPM
		NO TEST
33.0'	10	
	25	
	40	
47.5'	55	
	1.5	0

Depth	ELEV. 1337.1'
0.0'	
26.9'	ALLUVIUM; clayey sand, clayey tuff 13-18', gravelly clayey sand and occasional caliche and cobbles with boulders to 14" max. dia., poorly cemented.
47.5'	VOLCANICS-BASALT; dark gray, hard, slightly vesicular with fractures and cavities filled with fine grained ash material at 30.4-33.5', 39.4-39.6', and 42.5-44.5', slight to med. fractured. POSSIBLE GRADATIONAL CHANGE TO ANDESITE AT 46'

96 % Core recovery.
May 1976

D-19

DEPTH	PSI	GPM
		NO TEST
15.0'	10	
	20	
29.8'	30	
	20	
	30	
39.8'	40	
	30	
44.8'	45	
	3	0

Depth	ELEV. 1338.4'
0.0'	
13.8'	ALLUVIUM; streambed deposits, fine sand to boulders 4' in dia., unconsolidated.
44.8'	VOLCANICS-BASALT; dark gray, hard, slightly vesicular, occasional small vugs and solution cavities. Fractures and cavities (1-7" wide) filled with fine grained material. Slightly fractured to 42.0' with occasional highly fractured zones less than 1' thick, med. fractured 42.0-44.8'

98 % Core recovery.
May 1976

SCALE: 1 IN. = 20 FT.

10 0 10 20 30 40 50

GENERAL NOTES

1. SEE SHEET 3 FOR LEGEND.
2. SEE SHEET 7 FOR PLAN AND PROFILE OF TEST TRENCHES 78-2 AND 78-3.
3. SEE SHEETS 4,5&6 FOR LOCATION OF DIAMOND CORE HOLES
4. SEE SHEET 11 FOR ADDITONAL SPILLWAY GEOLOGIC LOGS.
5. ALLUVIUM WAS DRILLED WITH EITHER A ROCK BIT OR A DIAMOND CORE BIT. NO CORE RECOVERY WAS RECORDED.
6. SEE SHEET 10 FOR FRACTURE CLASSIFICATION.
7. SEE SHEET 6 FOR SPILLWAY PROFILE.
8. HOLES D-15 THRU 18 WERE DRILLED WITH A PLUG BIT 0' TO 50', TO RUN AN ELECTRIC LOGGING SURVEY. FOR DATA SEE FOUNDATION MATERIALS BRANCH FILES, US ARMY ENGINEER DISTRICT, LOS ANGELES. FOR LOCATION, SEE SHEET 4

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY:

CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT

SPILLWAY AND EMBANKMENT
GEOLOGIC LOGS

SUBMITTED BY: [signature]
CHIEF, F. & M. BRANCH

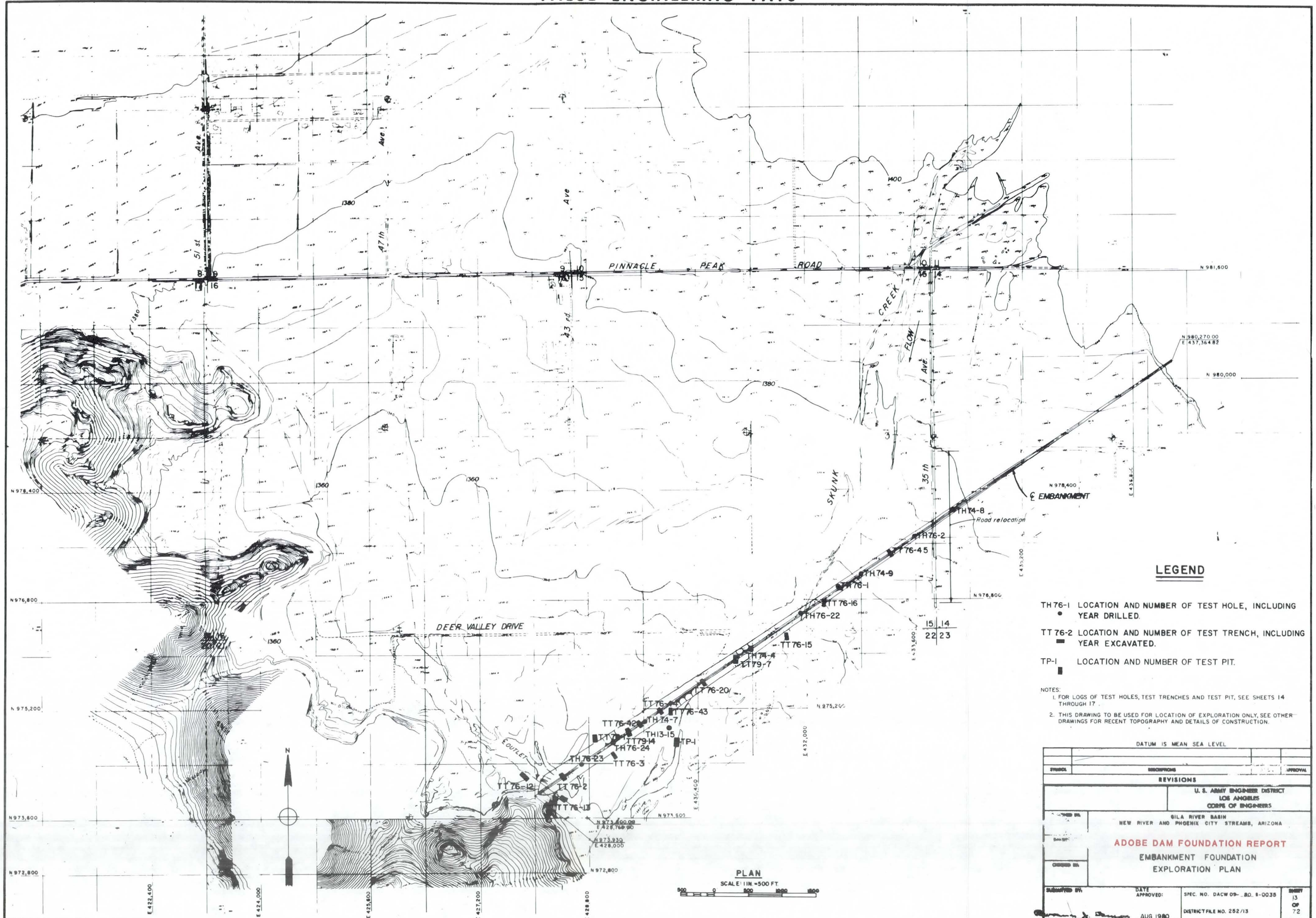
DATE APPROVED: AUG 1980

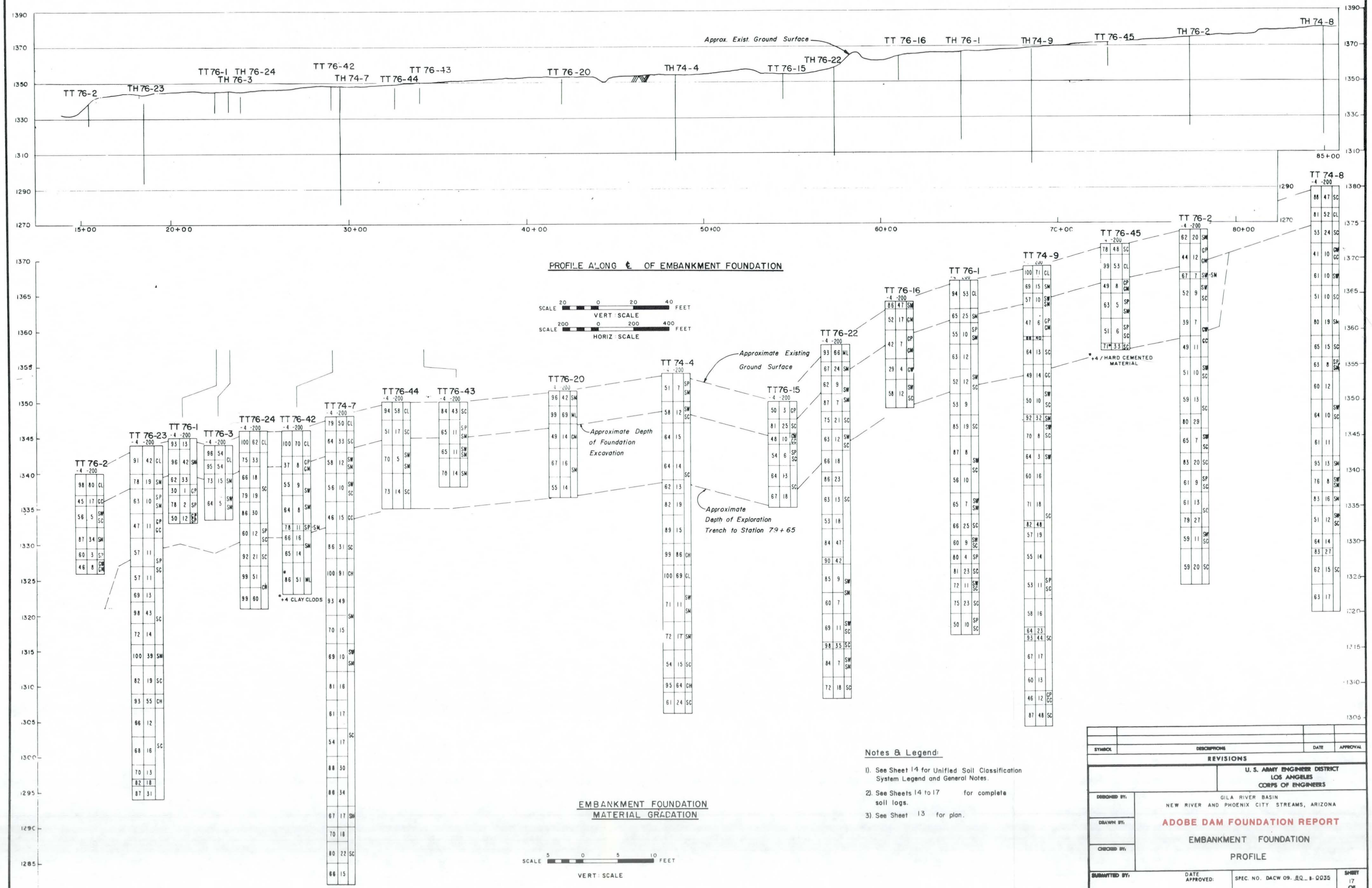
SPEC. NO. DACW 09-80-B-0035

DISTRICT FILE NO. 252/12

SHEET 12 OF 72 SHEETS

AMENDED PLATE 8


PINNACLE PEAK ROAD
DEER VALLEY DRIVE
SKUNK CREEK FLOW
51 st
47 th
43 rd
35 th
Ave
1380
1360
1400
OUTLET
℄ EMBANKMENT
Road relocation
N 981,600
N 980,270.00
E 437,364.82
N 980,000
N 978,400
N 976,800
N 975,200
N 973,600
N 972,800
E 422,400
E 424,000
E 425,600
E 427,200
E 428,800
E 432,000
E 433,600
E 435,200
E 436,400
TH74-8
TH76-2
TT76-45
TH74-9
TH76-1
TT76-16
TH76-22
TT76-15
TH74-4
TT79-7
TT76-20
TT76-43
TH74-7
TT76-42
TH13-15
TT79-14
TH76-24
TP-1
TH76-23
TT76-3
TT76-12
TT76-2
TT76-13
N
PLAN
SCALE: 1 IN = 500 FT
LEGEND
TH76-1 LOCATION AND NUMBER OF TEST HOLE, INCLUDING YEAR DRILLED.
TT 76-2 LOCATION AND NUMBER OF TEST TRENCH, INCLUDING YEAR EXCAVATED.
TP-1 LOCATION AND NUMBER OF TEST PIT.
NOTES:
1. FOR LOGS OF TEST HOLES, TEST TRENCHES AND TEST PIT, SEE SHEETS 14 THROUGH 17.
2. THIS DRAWING TO BE USED FOR LOCATION OF EXPLORATION ONLY, SEE OTHER DRAWINGS FOR RECENT TOPOGRAPHY AND DETAILS OF CONSTRUCTION.
DATUM IS MEAN SEA LEVEL
REVISIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA
ADOBE DAM FOUNDATION REPORT
EMBANKMENT FOUNDATION
EXPLORATION PLAN
DATE APPROVED:
AUG 1980
SPEC. NO. DACW 09-80-B-0035
DISTRICT FILE NO. 252/13
13 OF 72



Notes & Legend:

1). See Sheet 14 for Unified Soil Classification System Legend and General Notes.

2). See Sheets 14 to 17 for complete soil logs.

3). See Sheet 13 for plan.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT

EMBANKMENT FOUNDATION

PROFILE

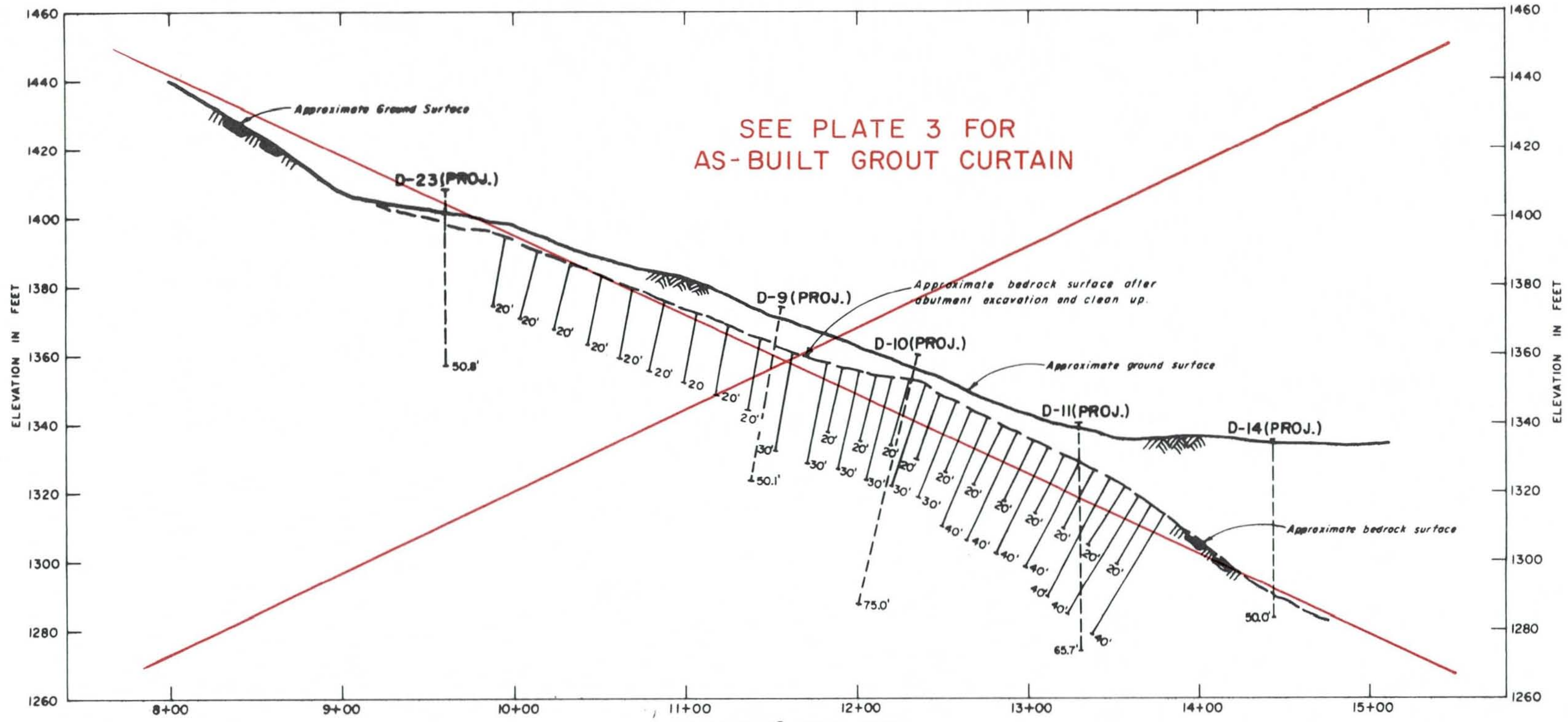
DATE APPROVED: AUG 1980

SPEC. NO. DACW 09-80-B-0035

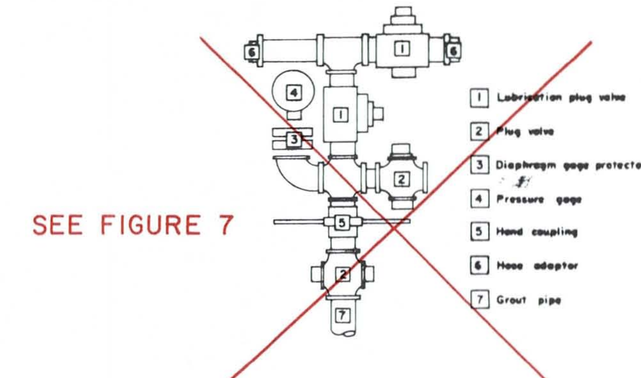
DISTRICT FILE NO. 252/17

SHEET 17 OF 72 SHEETS

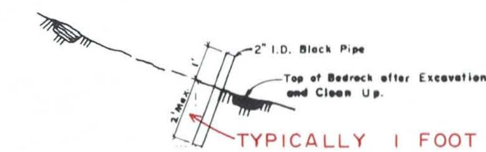
AMENDED PLATE 10



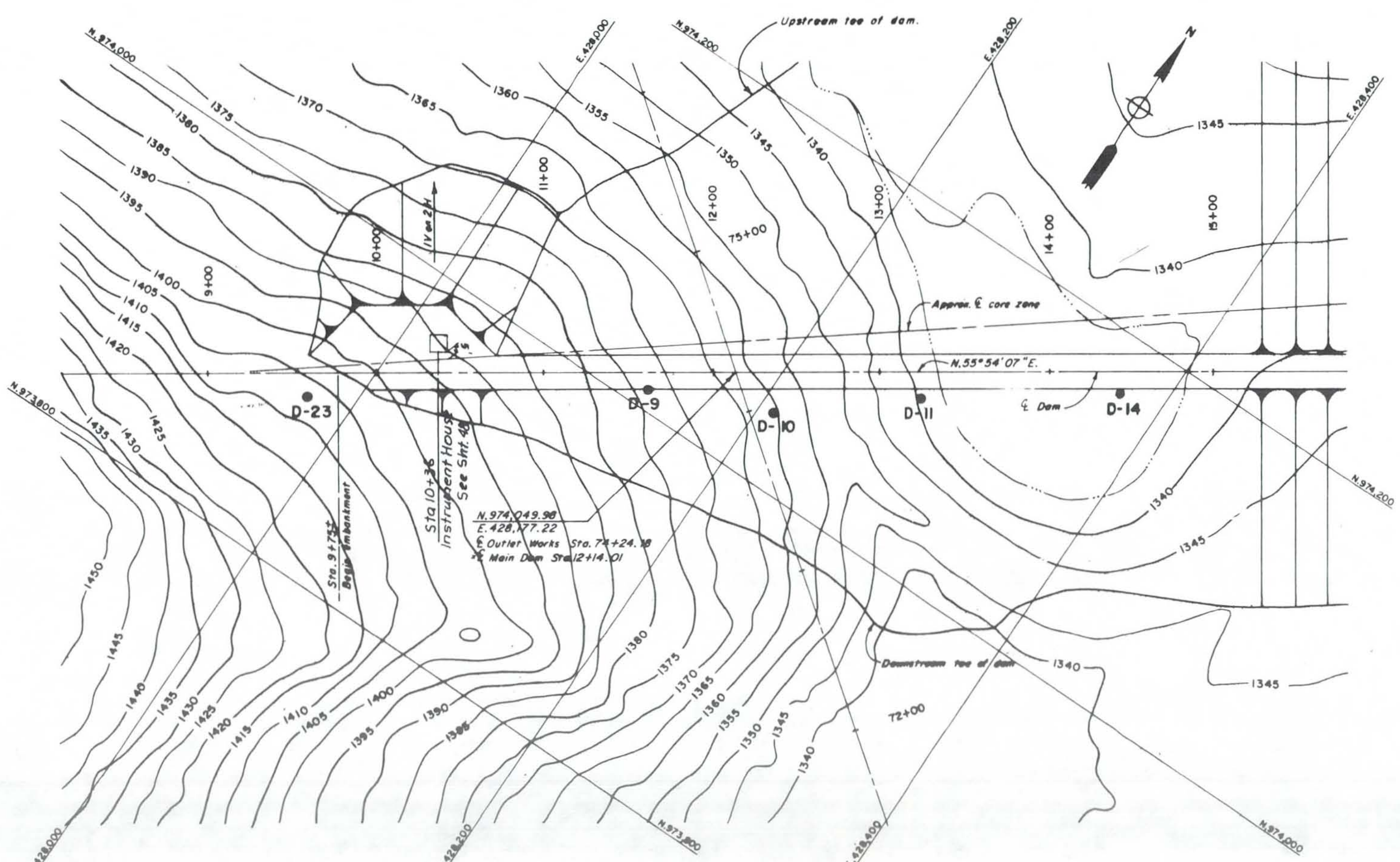
PROFILE (℄ CORE ZONE)
SCALE: VERTICAL 1 IN. = 20 FT. HORIZONTAL 1 IN. = 40 FT.



DIRECT GROUTING HEADER



TYPICAL GROUT PIPE SECTION IN ROCK

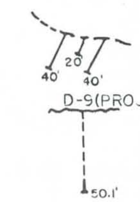


PLAN RIGHT ABUTMENT
SCALE: 1 IN. = 40 FT.

SCALE: 1 IN. = 20 FT.
SCALE: 1 IN. = 40 FT.

GENERAL NOTES

1. All grout holes shall be EX size.
2. Grout header shall be connected directly to grout pipe.
3. Split spaced grout holes will be drilled and grouted as determined in the field.
4. See Sheet 10 and 12 for logs of diamond drill holes from the previous exploration.
5. See Sheet 3 for additional legend.
6. Location of exploratory grout holes to be determind during grouting operation
7. See sheet 38 for limits of abutment excavation and cleaning.
8. See sheet 55 & 56 for excavation cross section & details of outlet works.
9. Maximum angle of hole: 45°
10. Grouting will be done along outlet works conduit, between sta 74+10 and 74+50



Grout holes.

Diamond core hole from previous exploration.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

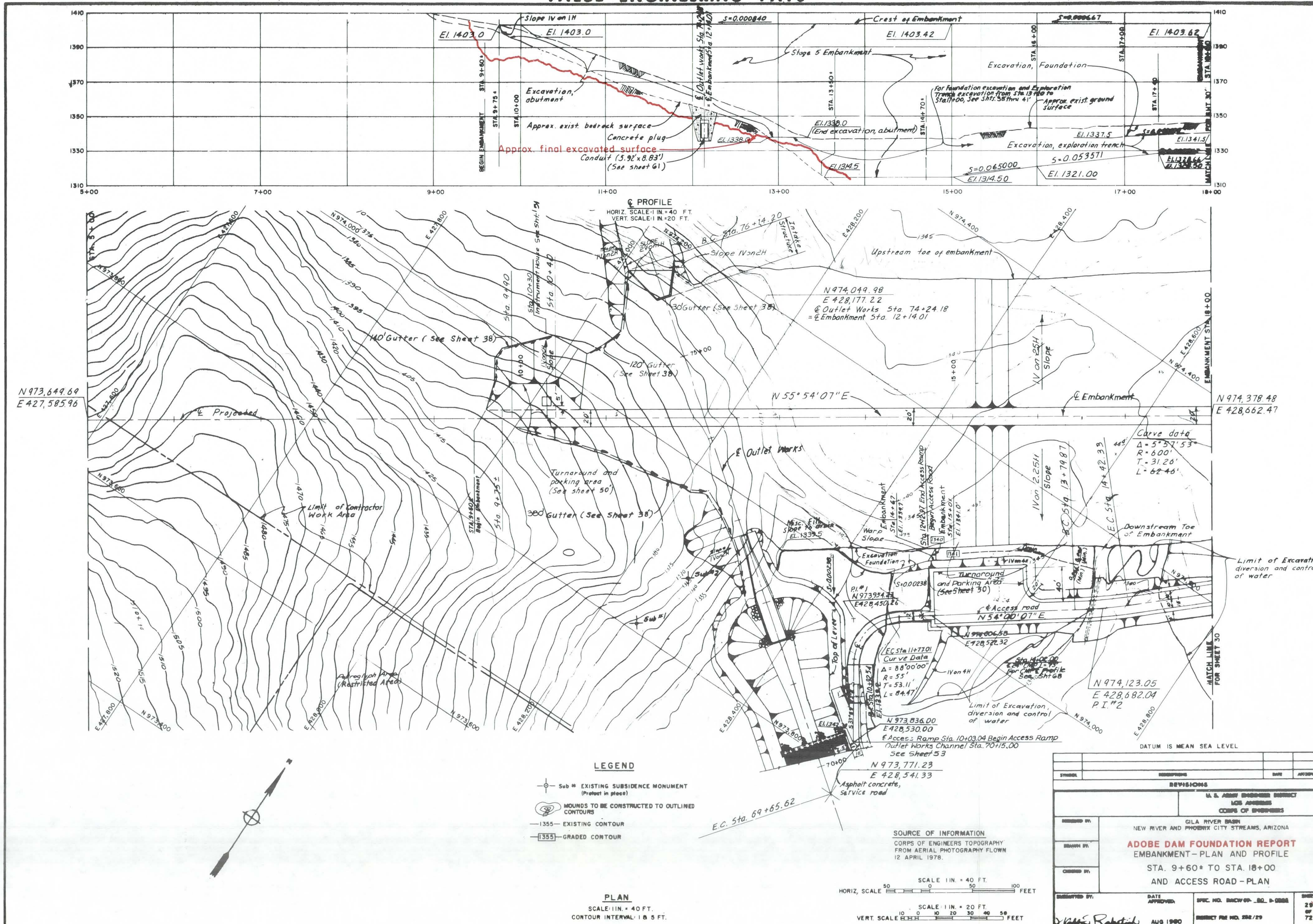
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT
WEST ABUTMENT
GROUTING
PLAN AND PROFILE

SUBMITTED BY:
CHIEF, F.&S. BRANCH
DATE APPROVED: AUG 1980
SPEC. NO. DACW09-80-B-0035
DISTRICT FILE NO. 252/26
SHEET 26 OF 72 SHEETS

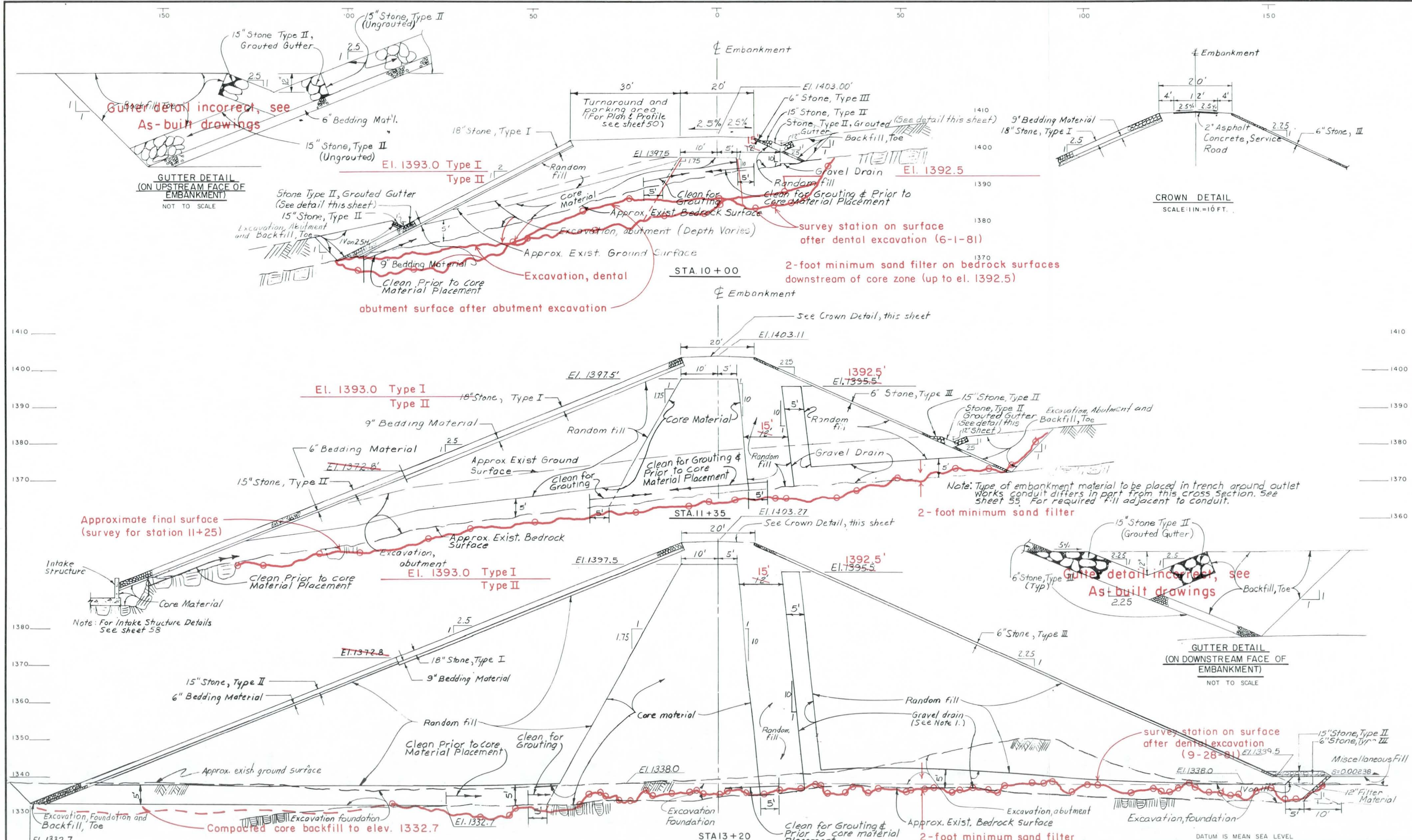
AMENDED PLATE 11

VALUE ENGINEERING PAYS



SAFETY PAYS

AMENDED PLATE 12



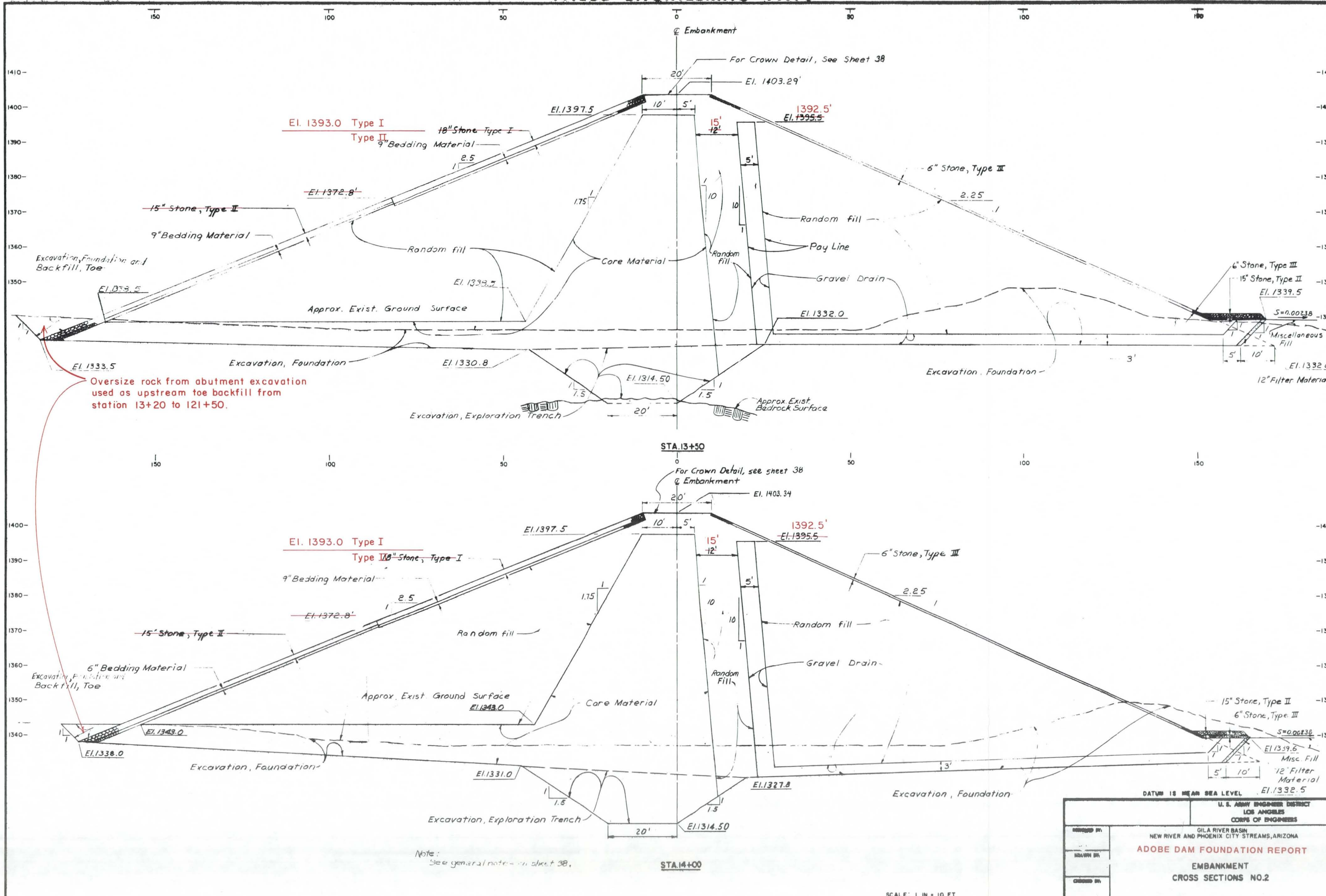
GENERAL NOTES:

1. Place 5.0 ft thick gravel drain on Foundation from Sta. 9+60± to Sta. 13+20. From Sta. 13+20 to Sta. 85+90 place 3.0 ft. thick gravel drain on foundation.
2. For Plan and Profile of Embankment, see Sheets 29 thru. Sheet 37.
3. Esthetic Treatment Fill on slopes not shown; see sheets 47 thru. 49.
4. For Plan of Grouted Gutters, see sheet 29.
5. 12" Filter material & 15" Stone, Type II, used for downstream toe protection is to run from Sta. 12+96 to Sta. 85+90.
6. Views of cross sections are looking eastward toward increasing stations.
7. Minimum widths of 5 ft. between Sta. 10+00± and Sta. 85+90 and 3 ft. between Sta. 85+90 and Sta. 121+10 are required for the inclined (1 Vertical on 10 horizontal) portion of the gravel drain. The widths may be increased for the Contractor's convenience, but at no additional costs to the Federal government.

VERT. SCALE: 1 IN. = 10 FT.
HORIZ. SCALE: 1 IN. = 10 FT.

U. S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS	
DESIGNED BY:	GILA RIVER BASIN NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA
DRAWN BY:	ADOBE DAM FOUNDATION REPORT
	EMBANKMENT
CHECKED BY:	CROSS SECTIONS NO. 1
SUBMITTED BY: Walter Rakitich, CHIEF, DESIGN BRANCH	DATE APPROVED: AUG 1980
SPEC. NO. DACW09-80-B-0030	SHEET 38 OF 72 SHEETS
DISTRICT FILE NO. 252/38	

VALUE ENGINEERING PAYS



DATUM IS MEAN SEA LEVEL

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT

EMBANKMENT
CROSS SECTIONS NO.2

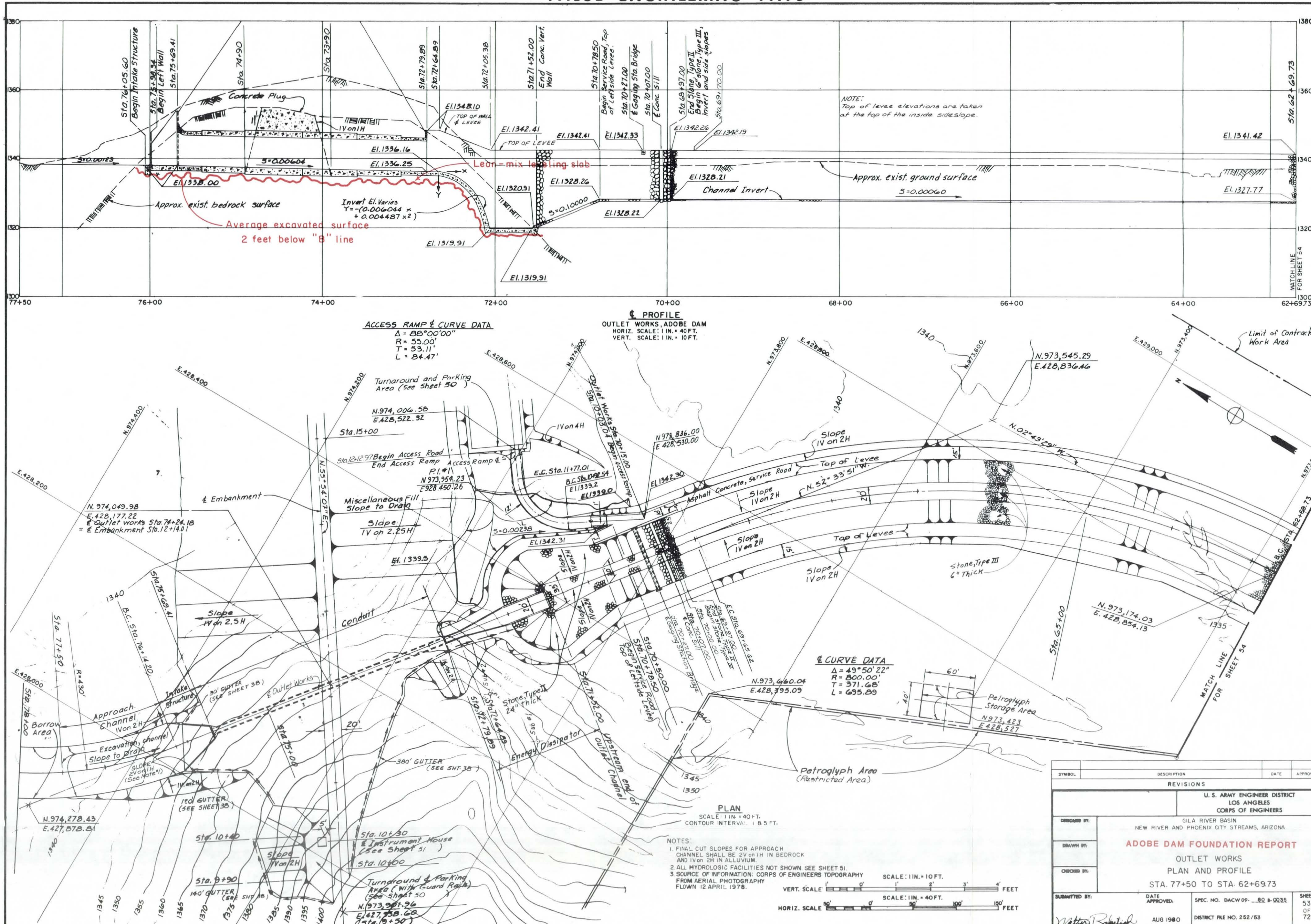
AUG 1980

SPEC. NO. DACW09-80-B-0028
DISTRICT FILE NO. 282/39

SHEET 39 OF 72 SHEETS

SAFETY PAYS

AMENDED PLATE 14



PLAN
SCALE: 1 IN. = 40 FT.
CONTOUR INTERVAL 5 FT.

NOTES:
1. FINAL CUT SLOPES FOR APPROACH CHANNEL SHALL BE 2V on 1H IN BEDROCK AND 1V on 2H IN ALLUVIUM.
2. ALL HYDROLOGIC FACILITIES NOT SHOWN. SEE SHEET 51.
3. SOURCE OF INFORMATION: CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN 12 APRIL 1978.

SCALE: 1 IN. = 10 FT.
VERT. SCALE 1' 0 1' 2' 3' 4' FEET
SCALE: 1 IN. = 40 FT.
HORIZ. SCALE 50' 0 50' 100' 150' FEET

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

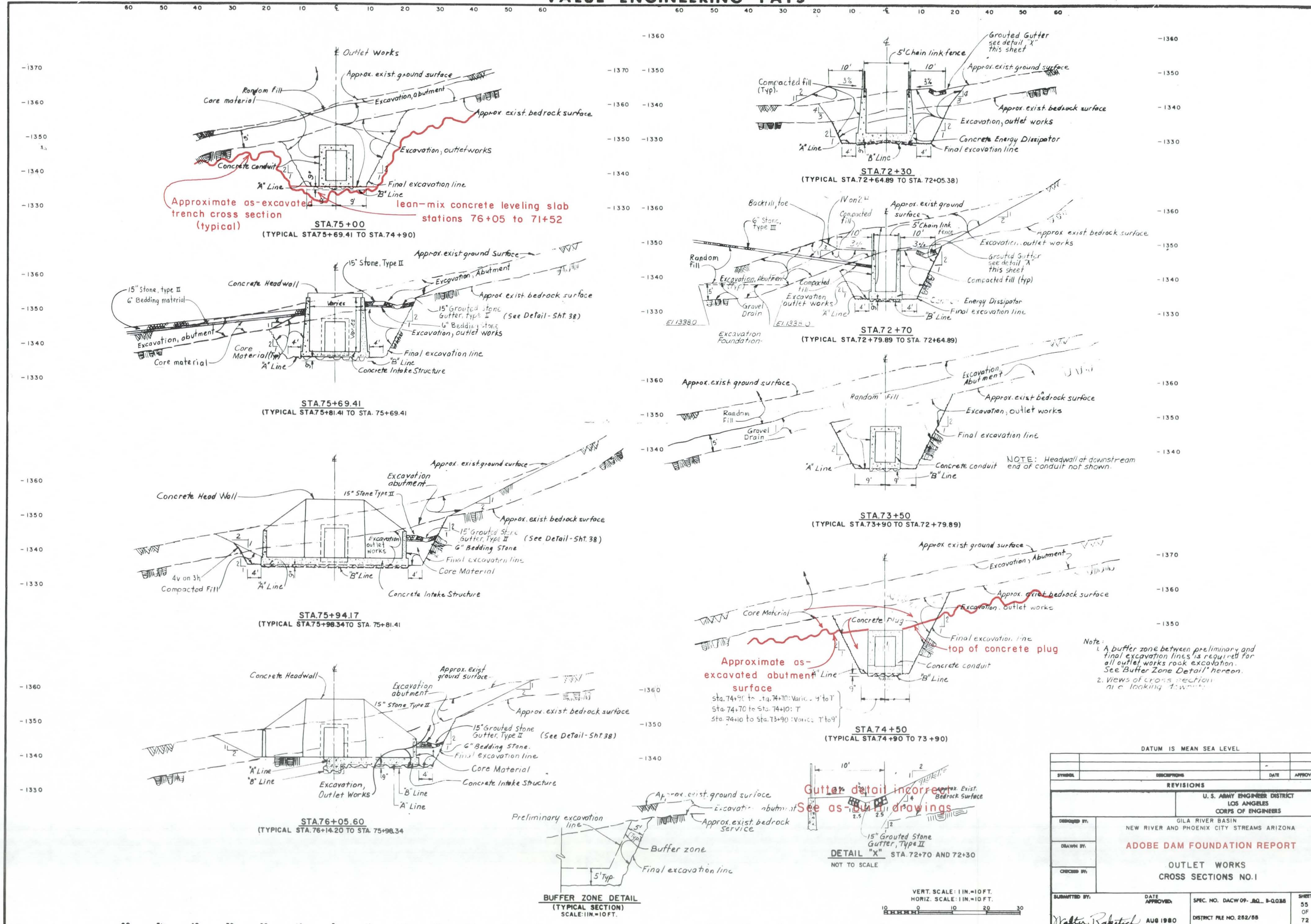
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

ADOBE DAM FOUNDATION REPORT
OUTLET WORKS
PLAN AND PROFILE
STA. 77+50 TO STA. 62+69.73

SUBMITTED BY: DATE APPROVED: AUG 1980
SPEC. NO. DACW09-80-B-0035
DISTRICT FILE NO. 252/53
SHEET 53 OF 72 SHEETS

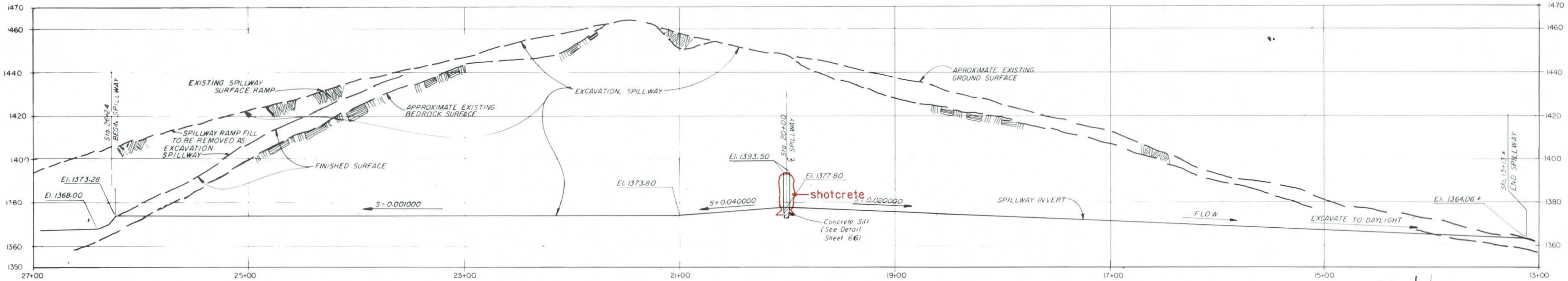
AMENDED PLATE 15

VALUE ENGINEERING PAYS

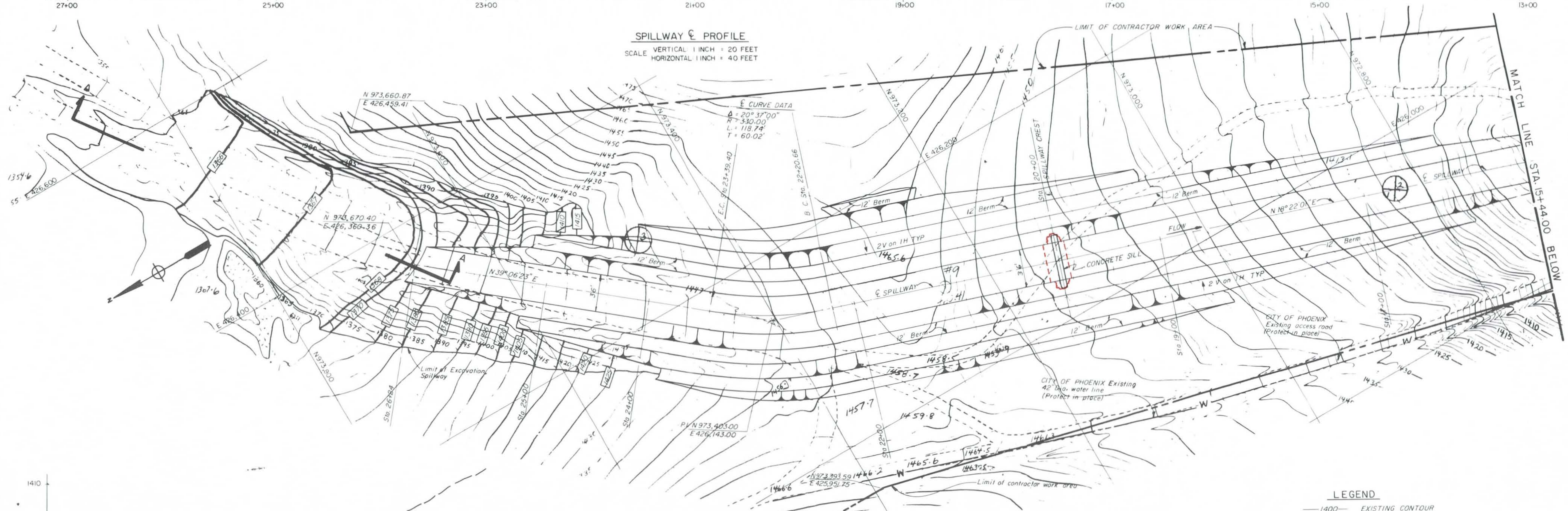


SAFETY PAYS

AMENDED PLATE 16

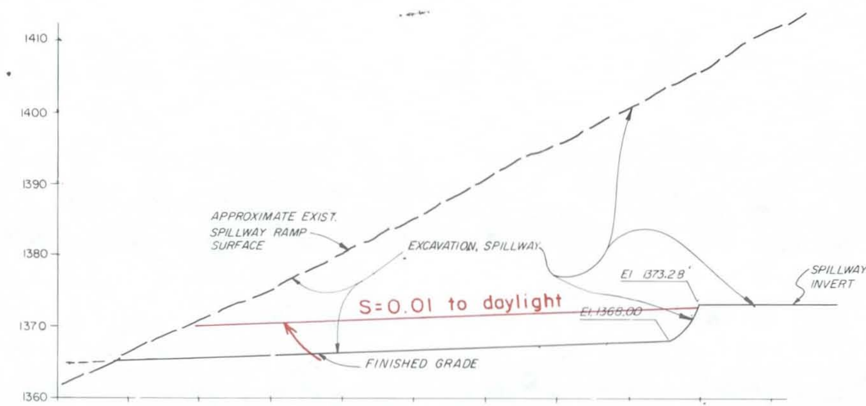


SPILLWAY ℄ PROFILE
SCALE VERTICAL 1 INCH = 20 FEET
HORIZONTAL 1 INCH = 40 FEET

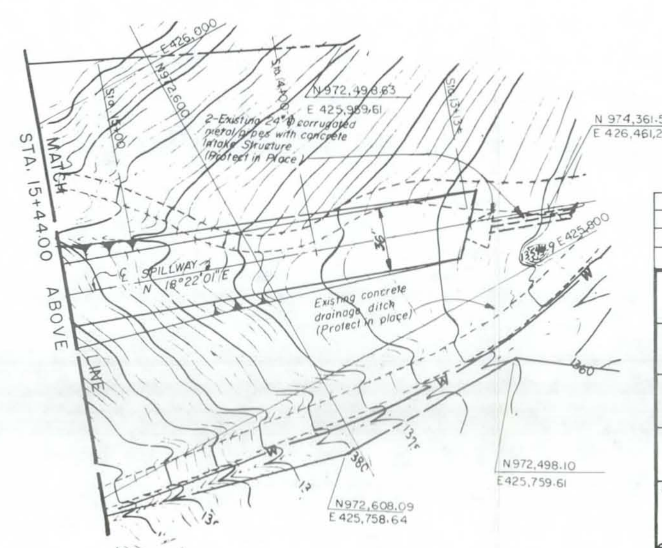


PLAN
SCALE: 1 INCH = 40 FEET
CONTOUR INTERVAL 1.8.5 FT.

LEGEND
1400 EXISTING CONTOUR
1400 FINISHED CONTOUR



SECTION A-A
(PROFILE OF EXISTING SPILLWAY RAMP)



SCALE
1 INCH = 20 FEET
20 10 0 20 40
1 INCH = 40 FEET
20 0 50 100

NOTES:
1. LOCATION OF 42" WATER LINE TO BE VERIFIED IN FIELD.
2. SOURCE OF INFORMATION; CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN JULY 11, 1980.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

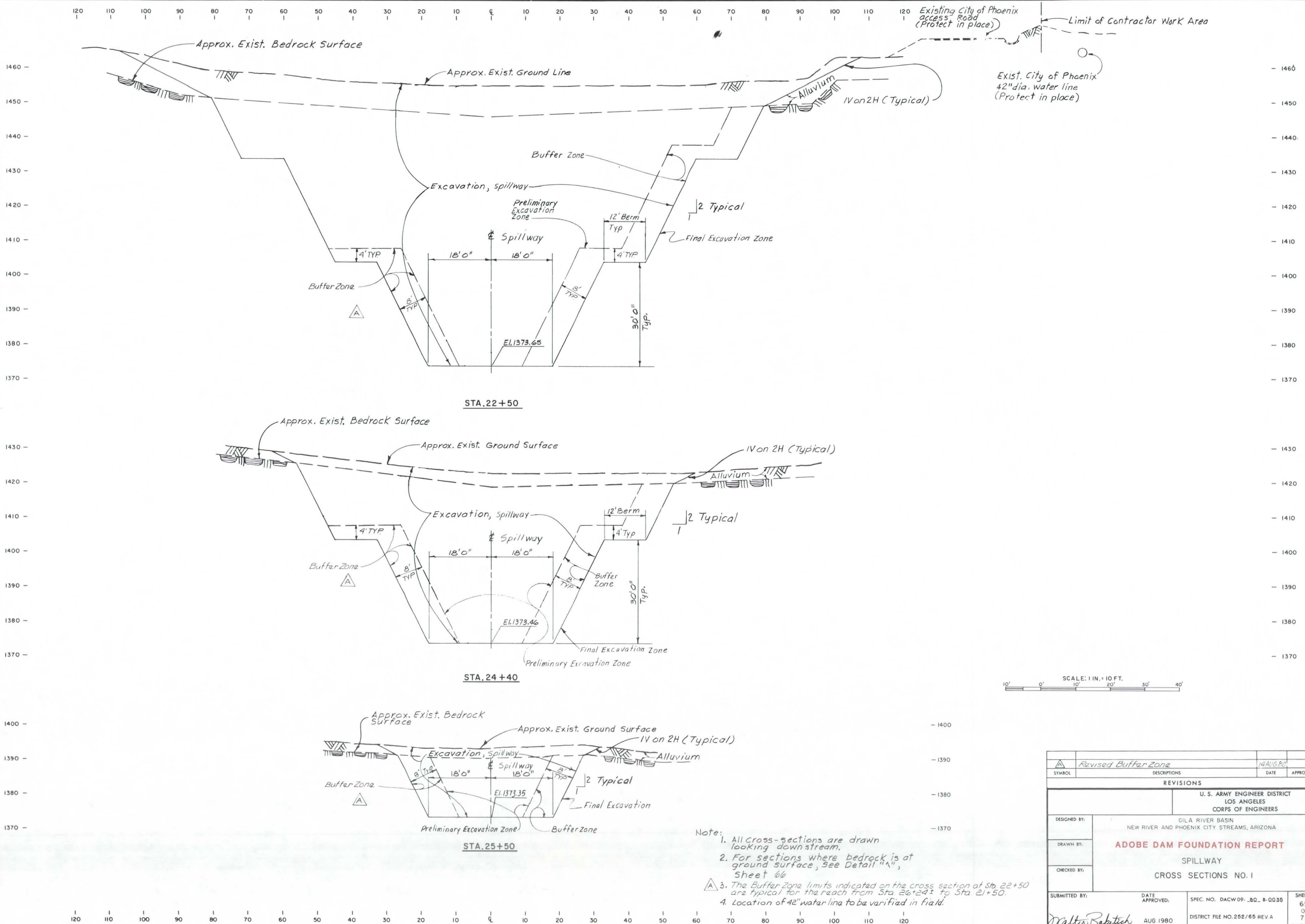
ADOBE DAM FOUNDATION REPORT
SPILLWAY
PLAN AND PROFILE
STA. 26+24 TO STA. 13+13 ±

SUBMITTED BY: CHIEF, BRANCH
DATE APPROVED: AUG 1980
SPEC. NO. DACW09-80-B-0030
DISTRICT FILE NO. 252/64.1
SHEET 64 OF 72 SHEETS

THIS DRAWING SUPERCEDES DRAWING 252 / 64

AMENDED PLATE 17

VALUE ENGINEERING PAYS



Note:
1. All cross-sections are drawn looking downstream.
2. For sections where bedrock is at ground surface, See Detail "A", Sheet 66
3. The Buffer Zone limits indicated on the cross section of Sta. 22+50 are typical for the reach from Sta 26+24± to Sta 21+50.
4. Location of 42" water line to be verified in field.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
A	Revised Buffer Zone	14 AUG 80	

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

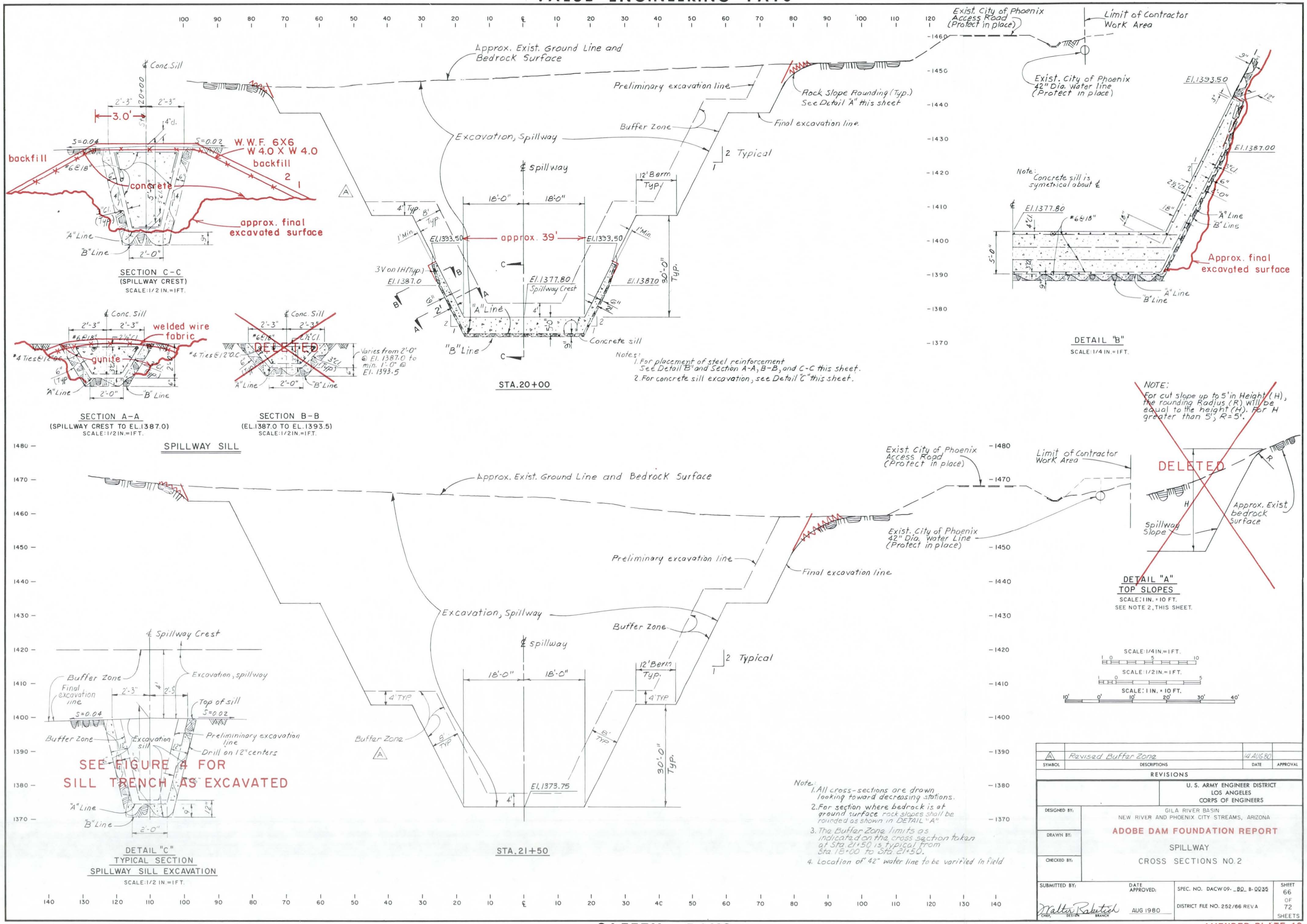
ADOBE DAM FOUNDATION REPORT
SPILLWAY
CROSS SECTIONS NO. 1

SUBMITTED BY: Walter Rabitich, CHIEF, DESIGN BRANCH
DATE APPROVED: AUG 1980
SPEC. NO. DACW09-80-B-0035
DISTRICT FILE NO. 252/65 REV A
SHEET 65 OF 72 SHEETS

SAFETY PAYS

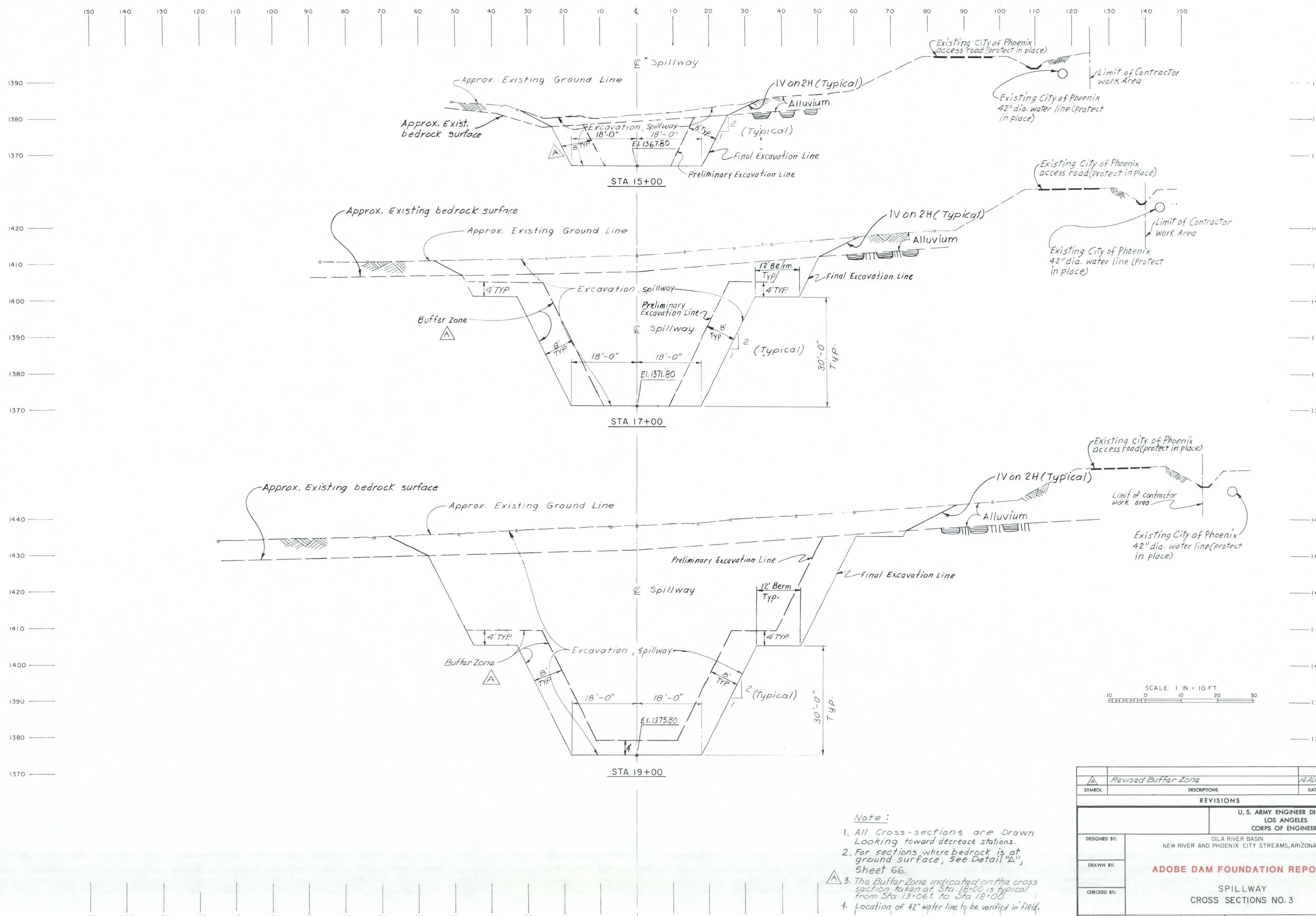
AMENDED PLATE 18

VALUE ENGINEERING PAYS



SAFETY PAYS

AMENDED PLATE 19



Note:

1. All Cross-sections are Drawn Looking toward decrease stations.
2. For sections where bedrock is at ground surface, See Detail "A", Sheet 66.
3. The Buffer Zone indicated on the cross section taken at Sta. 18+00 is typical from Sta. 13+06± to Sta. 18+00.
4. Location of 42" water line to be verified in field.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
A	Revised Buffer Zone	14 AUG 80	

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
GILA RIVER BASIN
NEW RIVER AND PHOENIX CITY STREAMS, ARIZONA

DRAWN BY:
ADOBE DAM FOUNDATION REPORT

CHECKED BY:
SPILLWAY
CROSS SECTIONS NO. 3

SUBMITTED BY: Walter Rabetich, CHIEF, DESIGN BRANCH
DATE APPROVED: AUG 1980
SPEC. NO. DACW09-80-B-0035
DISTRICT FILE NO. 252/67 REVA
SHEET 67 OF 72 SHEETS

AMENDED PLATE 20

GEOLOGIC REPORT ON THE ADOBE DAMSITE

ATTACHMENT 1

GEOLOGIC REPORT ON THE ADOBE DAM SITE

An interpretation of the volcanic rocks

July 1981

Michael F. Sheridan

Stephen Self

REGISTERED GEOLOGIST
CERTIFICATE NO.
7232
MICHAEL F. SHERIDAN
Date Signed 5 VII 81
ARIZONA, U.S.A.

Michael F. Sheridan

INTRODUCTION

The purpose of this investigation is to report on the genesis of the volcanic rocks exposed in the abutment and spillway cuts for the Adobe Dam site. One day of field work was made on May 16, 1981 by M.F. Sheridan and S. Self, and an additional day was spent on analysis of samples and photographs taken on the site. Additional data was obtained from thin sections provided by D. Lukesh and from Scanning Electron Micrographs made in the A.S.U. sedimentology laboratory.

ABUTMENT

The abutment excavation exposes a thick grey lava flow of intermediate composition. The rock varies from a dense lava to a porous vesicular material. In places the outcrop is massive and strongly jointed, but locally the lava is composed of large blocks separated by a sandy/silty matrix.

Rock type. In hand specimens the lava is vesicular and could be termed a porphyritic trachyandesite or latite. The exact rock name would depend on the chemical composition. Feldspar phenocrysts and megacrysts are common. The intermediate composition, high crystal content, and vesicular nature suggests a high-viscosity lava. The lava is well-jointed, typical of high-viscosity lavas, and the conjugate joint set appears to pass through the entire flow. At this location the lava is greater than 12-15 m thick, not unusual for typical stubby flows of this composition. Another feature of this lava are discrete xenoliths (ca. 10%) of aphyric lava. These clots would also tend to increase the viscosity of the lava leading to brecciation.

An examination of thin section #3 from the spillway breccia supports the field observations. Plagioclase occurs as large phenocrysts and as groundmass laths. Most of the larger crystals have a wide reaction band of melted and reacted material surrounded by a thin outer zone of fresh overgrowth. Both orthopyroxene and clinopyroxene occur as crystals in large clots together with plagioclase. A few large quartz crystals have strong reaction haloes of clinopyroxene. These textures suggest strong disequilibrium that is typical of the mixing of magmas of different compositions and temperatures. Thus,this lava represents a viscous hybred magma that should be distinctive in its physical characteristics and chemical composition.

Joints. The main abutment fractures appear to be typical cooling joints. At least three major sets occur: subvertica with 40° and 120° strikes and subhorizontal with a dip of 10° to 32° in the direction of 230°. The subhorizontal set has curved joint planes and may be 'ramp' joints that developed in the still-flowing, highly-viscous lava. The subhorizontal rubbly zones of blocks near the top of the exposure may represent flow unit contacts, but the entire exposure probably represents a single massive flow. Some brecciation occurs along joint planes, but this is probably due to stress in the cooling lava, rather than due to tectonic movement.

Joint filling. Subvertical joints and spaces between blocks are filled with an unconsolidated but slightly indurated lithic ashy sand. This material is laminar-bedded to wavy-bedded in places (see Fig. 1). The two principal coarse components are angular clasts (up to 5 cm) of the host lava derived from the joint walls, and weathered pumice fragments. Silicic ash forms a large component of the sand. The median-sized material consists of quartz, feldspar, and volcanic glass.

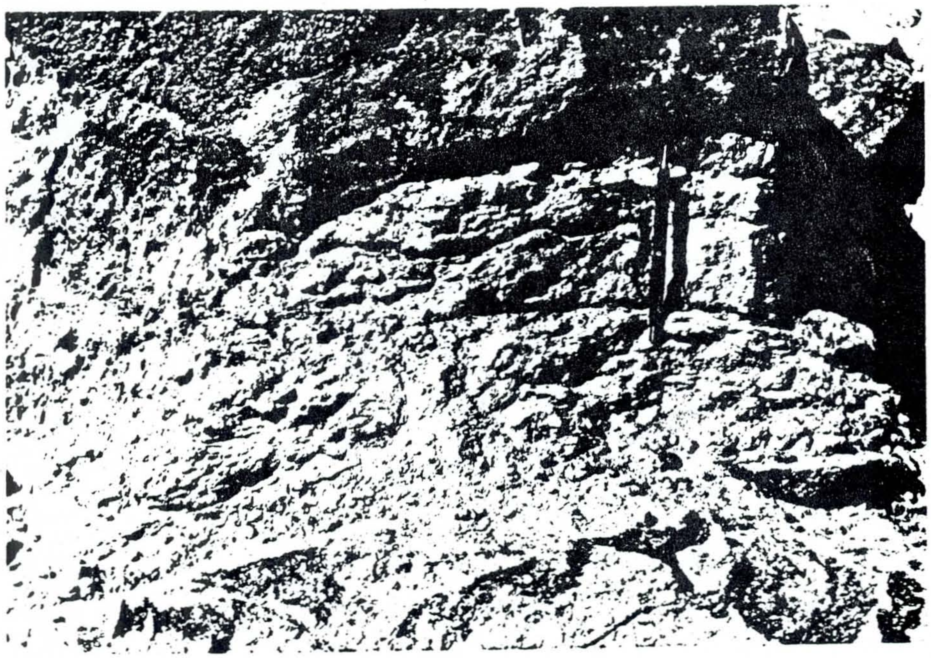
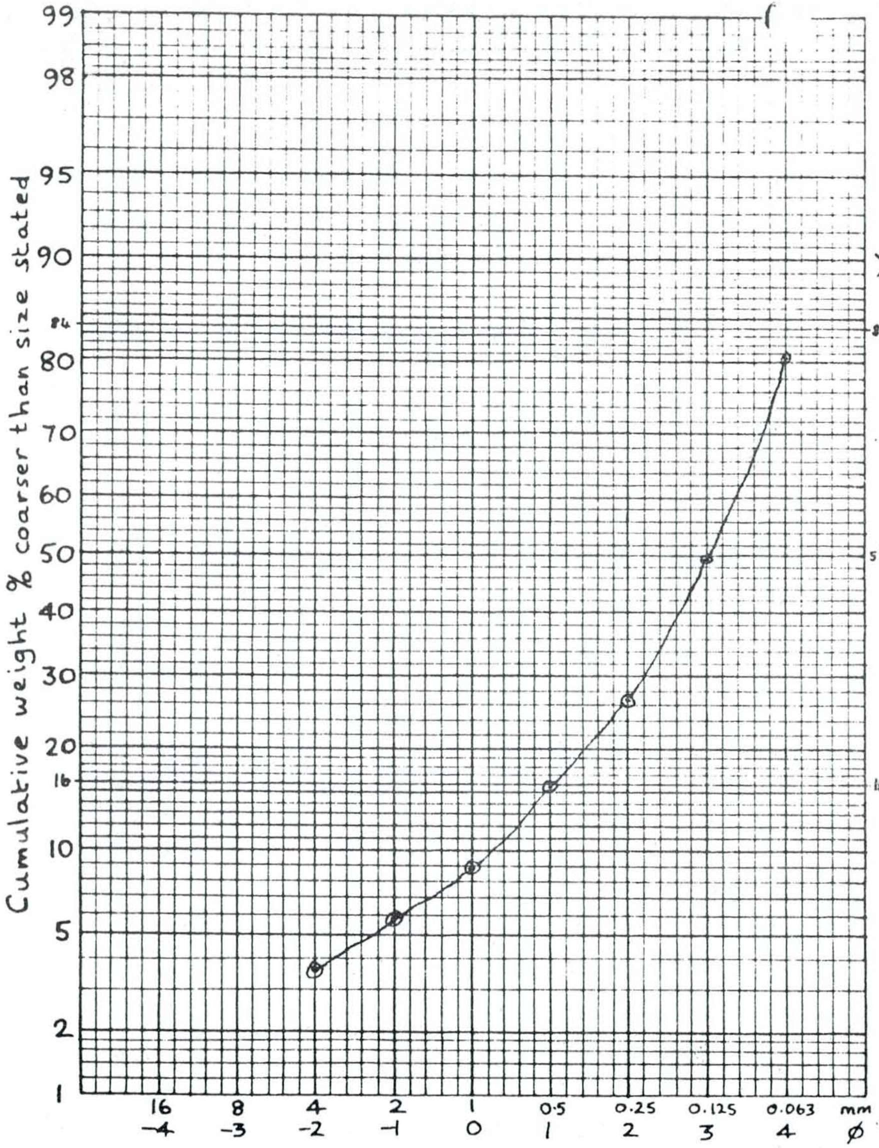


Figure 1. Bedded sands, joint filling in abutment site lava flow. pencil shows scale. Note the wavy laminar-bedding to the left of the pencil. The white material is caliche ($CaCO_3$), also present on joint faces.

TABLE 1. Grain-size analyses.

Adobe 3	Adobe 5
$Md\ \phi$ = 3.1 (ca. 0.125 mm) very fine sand $\sigma\phi$ = 1.5 moderately poorly sorted $\alpha\phi$ = -0.4 coarse tail graded	contains coarse clasts and is more indurated than Adobe 3. Too indurated to disaggregate without altering grain size. Large clasts are mainly host basalt.



Adobe 3

Sample of joint-filling sand

Disaggregatable : Total wt. 327.3 g

φ	grain size (mm)	wt. (g)	wt %	cuml wt %
>-2	>4	12.15	3.71	3.71
-1	2	7.20	2.20	5.91
0	1	10.0	3.06	8.97
1	0.5	22.5	6.87	15.84
2	0.25	34.9	10.66	26.50
3	0.125	72.60	22.18	48.68
4	0.063	104.35	31.88	80.56
<4	< "	63.60	19.43	100.00
		327.3g	100.00	

$\phi_{16} = 1$, $\phi_{84} = 3.9$; $\phi_{50} = 3.1$

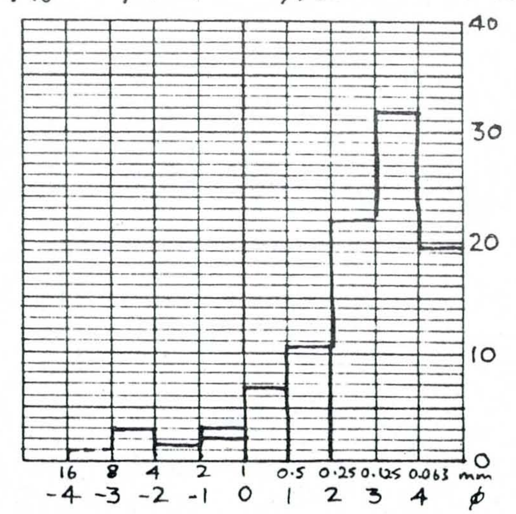


Figure 2. Particle-size data for joint-filling sand at the abutment site.

In order to interpret the origin of the joint-filling sand at the abutment site a mechanical size analysis and scanning electron microscope investigation was made. A summary of the data from the size analysis is given in Table 1, and the details of the analysis are presented in Fig. 2. The SEM investigation revealed the essential primary features of the sand grains, the nature of the cementing material, and the effects of solution and reprecipitation on grain surfaces.

The particles are chiefly volcanic glass (Figs. 3 & 4), quartz, and feldspar (Figs. 5 & 6). In many cases aggregates of grains are cemented by secondary minerals (Figs. 7 & 8) which are mainly calcite with minor zeolites. In some grains a coating of secondary material has subsequently been etched to reveal the primary grain structure by solution of cementing materials (Figs. 9 & 10). Other grains show secondary growth of small oriented projections (Figs. 11 & 12). Although only a few of the textures are illustrated in this report, approximately 60 grains were studied to reach our conclusions.

Origin of the joint-filling sand. The blocky surface of the abutment lava flow provided an excellent trap for wind-blown sands moving across the desert surface. The grain-size of the sands is similar to that of typical desert dunes. The surface textures of the grains, as revealed by SEM, show little evidence of abrasiion, indicating a local source. The majority of the grains are of volcanic origin, as is obvious for the pumice and glass shards. It seems likely that this volcanic sand is related to the prominent silicic ash deposits exposed at the northern end of Lake Pleasant and extending northward to New River and Black Canyon. These ashes are interlayered with basalts and trachytes dated at about 10 million years ago.

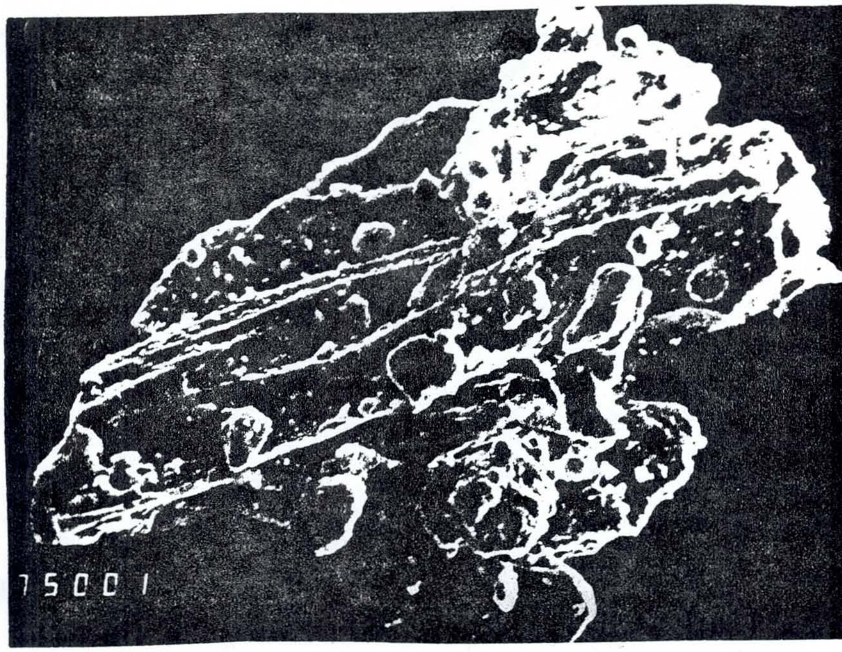


Figure 3. SEM photomicrograph 75001. Pumice fragment with attached secondary minerals. (315X)

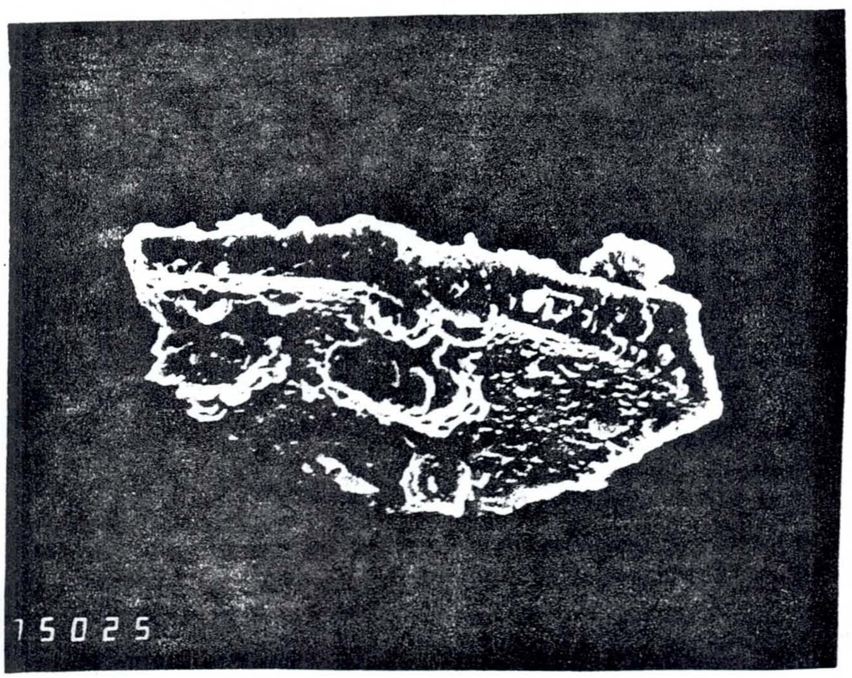


Figure 4. SEM photomicrograph 75025. Glassy plate with adhering secondary minerals. (500X)

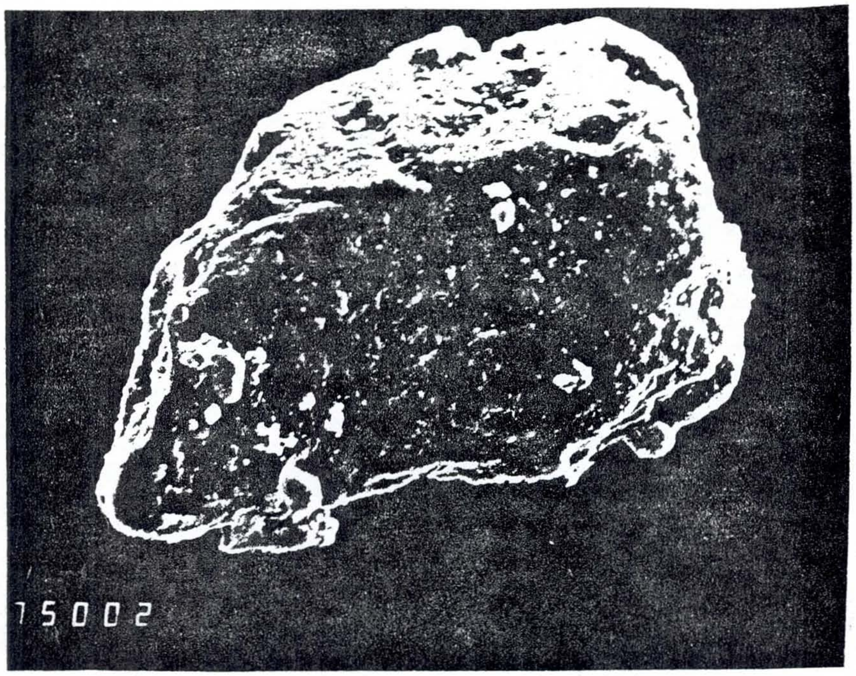


Figure 5. Feldspar crystal showing minor surface impact features indicating only slight abrasion during transport. (315X) SEM photomicrograph 75002.

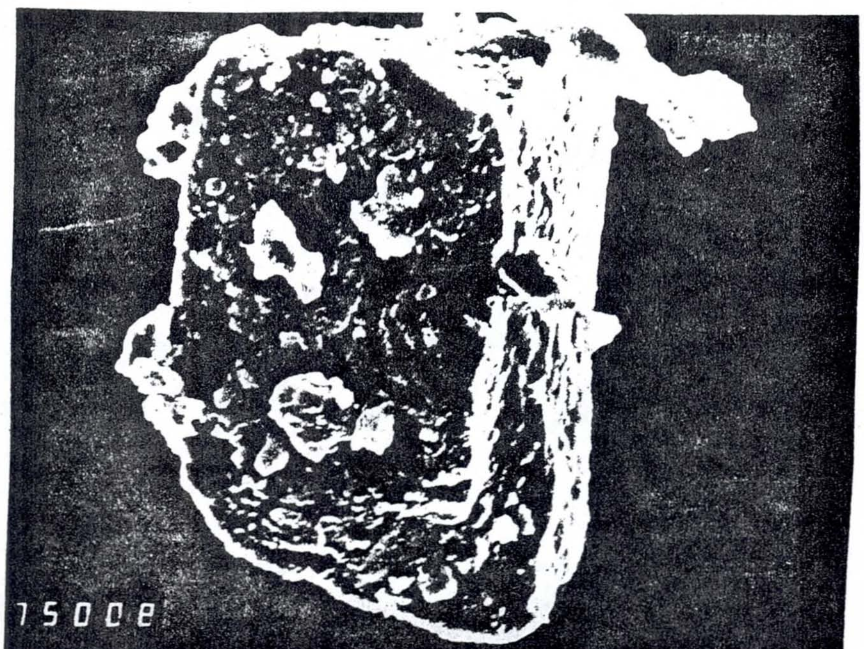


Figure 6. Feldspar crystal with good primary morphology. Edges and corners are well-preserved indicating only minor abrasion during transport. (500X) SEM photomicrograph 75010.

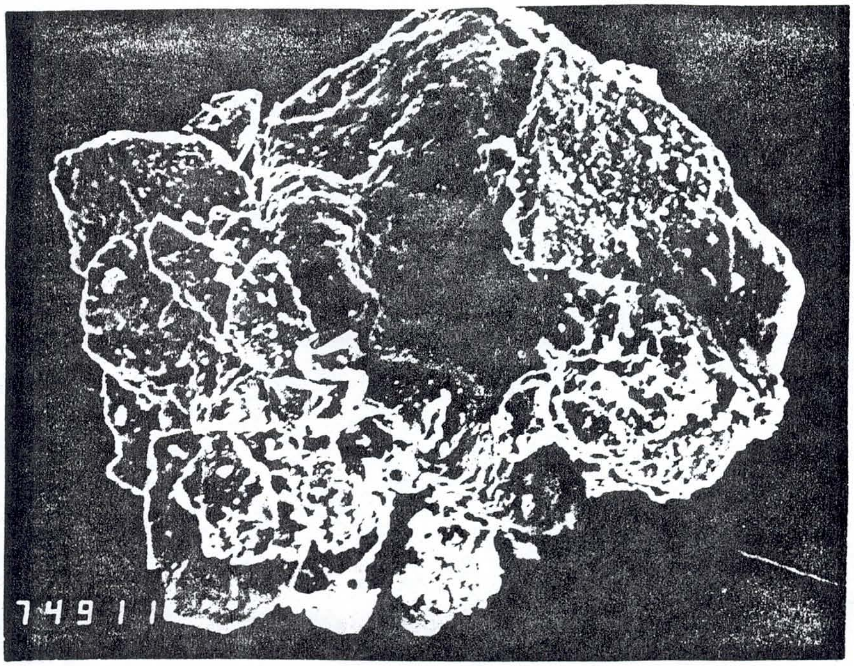


Figure 7. SEM photomicrograph 74911. Aggregate of angular fragments cemented by secondary material. (184X)

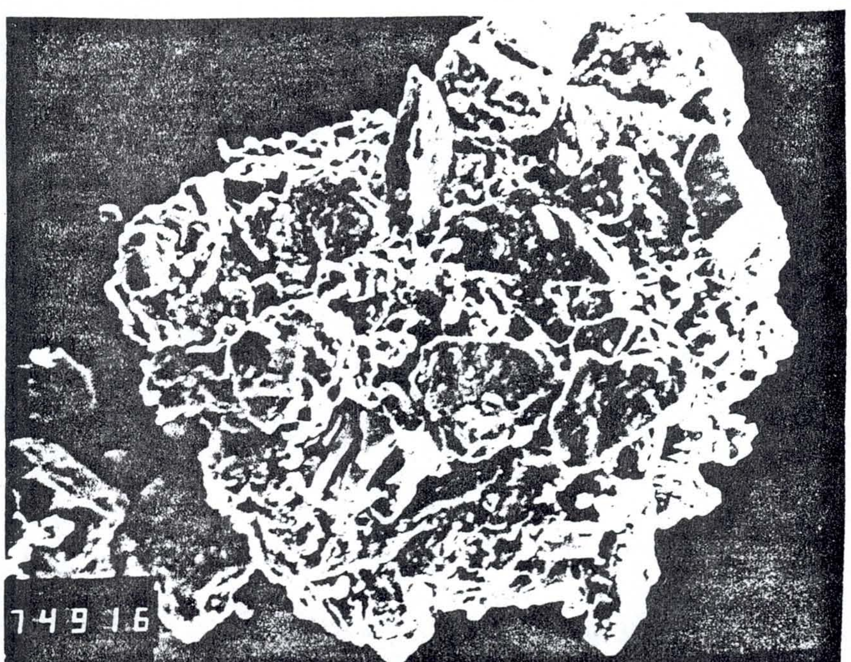


Figure 8. SEM photomicrograph 74916. Aggregate of platy and angular fragments cemented by secondary minerals. (200X)

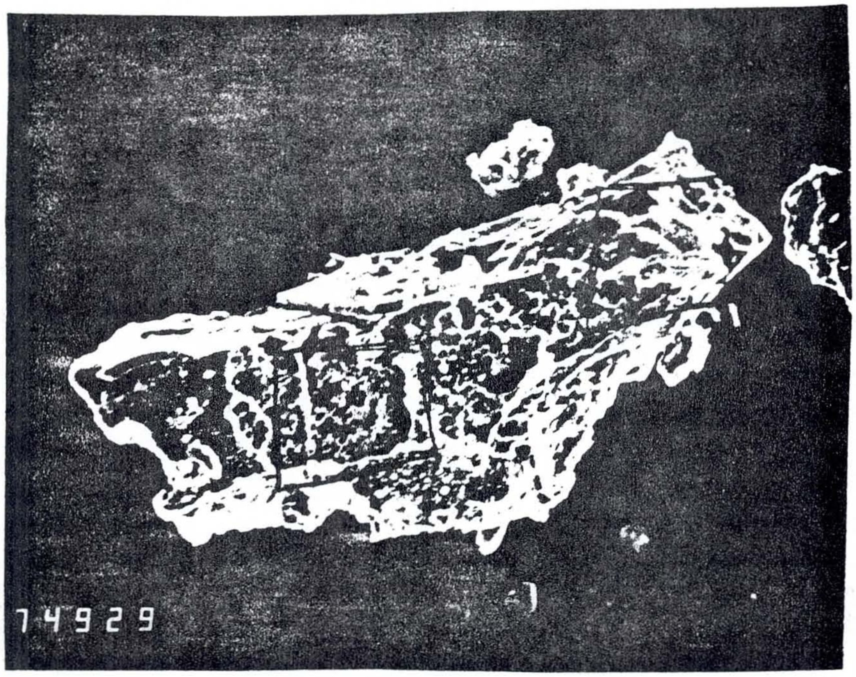


Figure 9. SEM photomicrograph 74929. Pumice fragment with a smooth coating of secondary minerals. Subsequent solution etched the coating to expose the hidden vesicular texture. (180X)

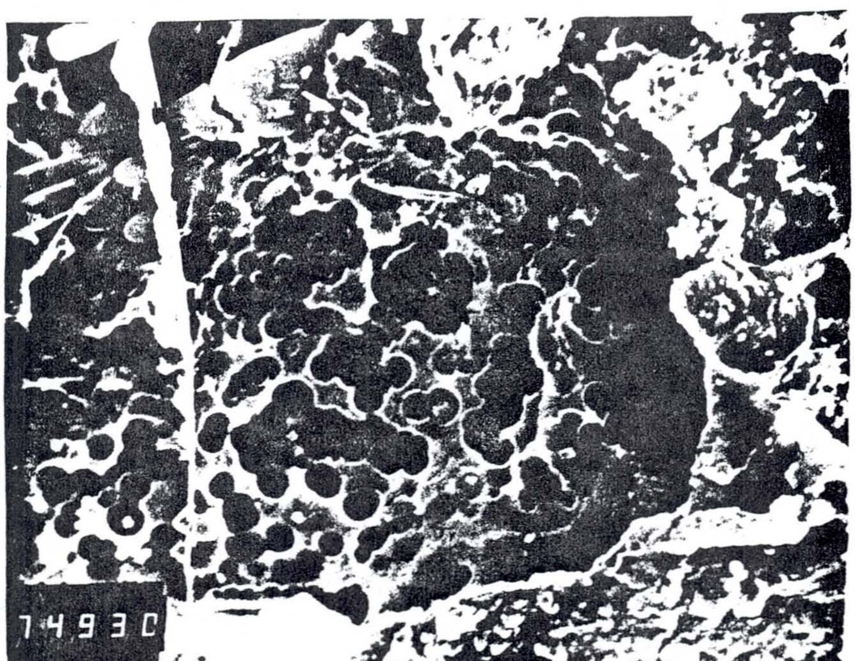


Figure 10. SEM photomicrograph 74930. Detail of pumice structure in the grain shown above. (900X)

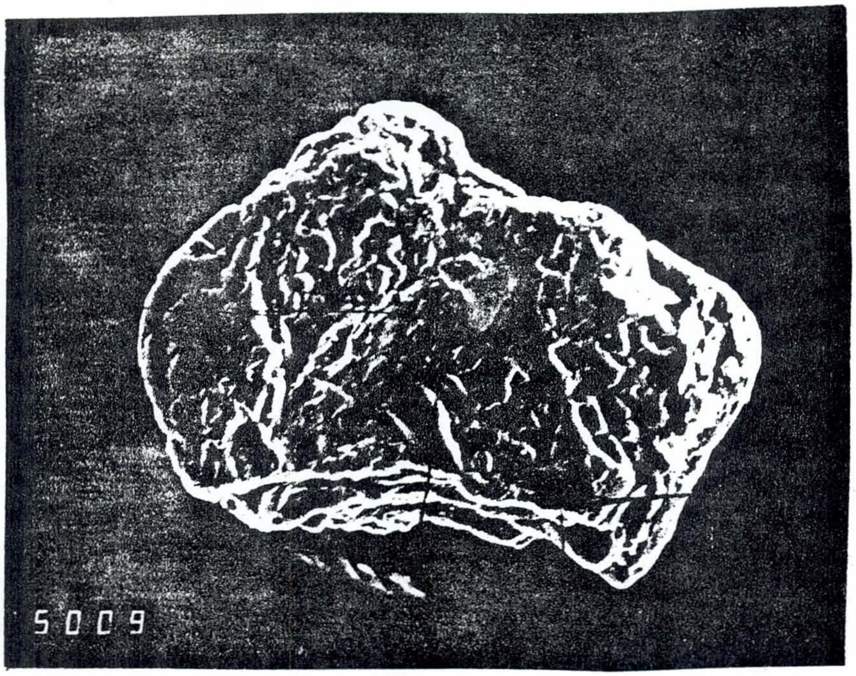


Figure 11. SEM photomicrograph 75009. Feldspar crystal showing secondary growth on crystal faces due to groundwater percolation. (350X)

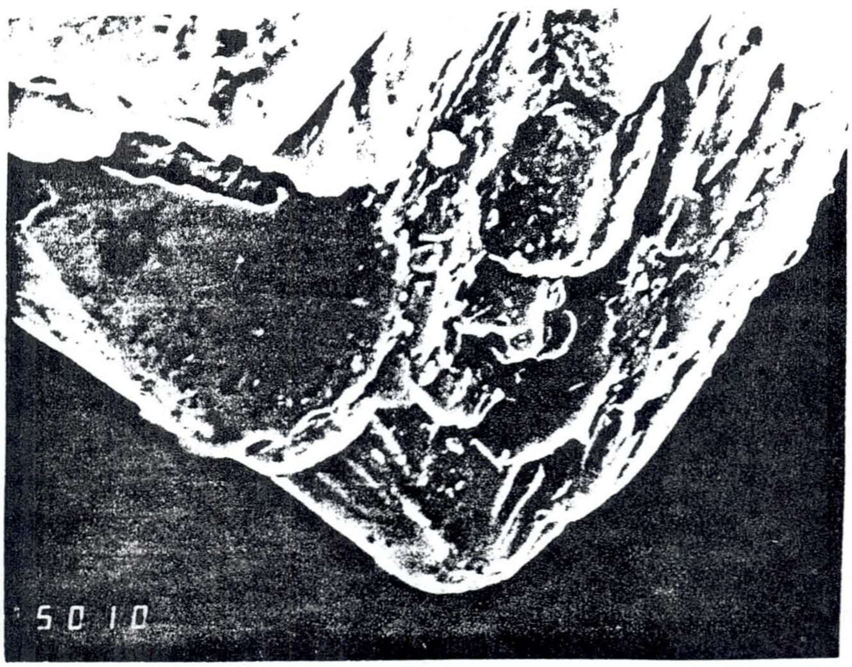


Figure 12. SEM photomicrograph 75010. Detail of overgrowth on feldspar crystal face. (2500X)

It seems likely that shortly after the eruption of the blocky hybred lava at the abutment site, ashes were blown from the northern silicic center covering the surface and filling the cracks. The laminar bedding of some of the sandy zones suggests some water reworking of the material into the cracks. It should be noted that a similar bolcky lava surface at the spillway site does not have sandy infilling material. This suggests either a short time of surface exposure for these lavas, of a restricted time during which this ash was transported southward. This would be the case of the silicic ash were transported during a period of active volcanism near New River.

SPILLWAY

The spillway excavation exposes a 20-25 meter thick section through a complex pile of at least three intermediate lava flows, all of which were erupted subaerially. The flows are very rubbly, well-jointed, and in places show intence flow foliation and jointing. This section was studied in the field and a composite photograph was made for the outcrops on both faces of the excavation.

Rock types. The lowest lava flow exposed in the spillway excavation is very similar to the one at the abutment. Above this are two less vesicular flows in complex superposition. Highly-fragmented breccia occurs between flow units. The irregular topography of the contacts may indicate sill-like intrusions into breccia levees at flow margins. Thin section observations indcate that the mineralogy and textures of these lavas are also similar to the material at the abutment. The plagiocalse phenocrysts have a wide reaction band of melted material that is surrounded by a thin zone of fresh overgrowth. Clots of plagioclase intergrown with clinopyroxene and independent crystals of clinopyroxene with hourglass zonation are common. Scattered quartz grains have strong reaction haloes of clinopyroxene. Mafic blebs in the lava contain plagioclase and clinopyroxene, but no olivine was observed. The textures vary from glassy to holocrystalline. All of the above features suggest that the lavas have a common oirgin and are part of the same complex that evolved over a short period of time.

Stratigraphy. The various features were identified on the composite working photograph and in the field. The oldest lava is exposed at the NE end of the cut. It has a very rubbly surface and appears to be the same lava as that exposed at the abutment. This lava may become more massive downwards as excavation proceeds, but at the time of the field work only the rubbly top was exposed. The rock is vesicular, scoriaceous, and has abundant aphyric inclusions of basalt.

The second lava is a strongly fragmented flow that underwent extreme brecciation at the time of emplacement. pods of coherent lava, which may be old lava tubes, channels, or sills, are seen, but in general the lava is composed of loose blocks. This is a common characteristic of some intermediate composition lava flows resulting from flow-induced shear and incipient rupturing that is termed 'autobrecciation.'

The third lava caps the undulating surface of the other flows and fills deeper depresions, forming vertical sides to some parts of the flow. Later regional tilting has produced apparent over-steepened flow margins. This flow is more or less coherent, and has extreme platy jointing that appears contorted into a swirling pattern. Calcite and opal occur as coatings on joint planes.

This last lava is glassy to microcrystalline, aphyric, in the groundmass and lacks abundant megacrysts common to the other lavas. All of these rocks are hybred and could be called trachyte to trachyandesite, although they can not really be distinguished without good chemical analyses.

The capping material at the NE end of the excavation is a calichified breccia. This material was probably empalced by mass flowage (colluvium?) during the time of a wetter and colder climate. This unit has now hardened into a firm calcrete-like material.

Structures. All of the flows are strongly jointed and brecciated. Most of this fragmentation appears to have been primary (related to lava emplacement), and there is little effect of tectonic activity that is recorded in the rocks. Small faults of up to 3-4 m were noted in the excavations. These could have been the result of displacements related to volcanism or to later tectonism. The principal fault seen in the spillway strikes at 175° and dips 75/85°. This fault may be of lical origin or related to Basin and Range faulting. Obviously, the entire abutment block has been faulted up by Basin and Range tectonism after the emplacement of the lavas. However, the main Basin and Range faults are now probably covered by the valley fill material to the east.

CONCLUSIONS

The sequence of volcanic materials exposed at the spillway and abutment sites represent a complex lava flow pile. These lavas are of intermediate hybred composition and had a high viscoisty at emplacement. The high viscosity led to strong autobrecciation and blocky, jointed flows. The joint-filling sands at the abutment site are of aeolian origin from a silicic volcanic source, probably in the vicinity of New River. This sandy infilling probably continues throughout the blocky part of the flow.

SECTION 2C
OF THE CONTRACT
(EXCAVATION)

ATTACHMENT 2

SECTION 2C

EXCAVATION

Index

1. GENERAL. Excavation shall consist of the removal of every type of material encountered except materials covered by the provisions of the section: CLEARING SITE AND REMOVING OBSTRUCTIONS in the designated areas, or from other areas as directed, to the lines, grades, and elevations indicated. The material to be removed may include but is not limited to earth, hardpan, caliche, silt, clay, sand, gravel, cemented sand and gravel, rock, adobe, detached pieces of stone and concrete, rock fills, existing fills of miscellaneous debris and rubbish, and other unsuitable materials. Slope lines indicated on the drawings for temporary cuts do not necessarily represent the actual slope to which the excavation must be made to safely perform the work. Excavation for permanent cuts shall be made to the slope lines indicated. Excavation shall be performed in a manner which will not impair the subgrade. Except as otherwise specified, the finished surface of subgrades shall be smooth and shall not vary more than one 1/2-inch from indicated grade. The exploration trench and foundation excavation for the embankment shall be conducted in a uniform, continuous manner beginning at the west abutment and proceeding easterly in complete section increments in reaches of sufficient length in which construction equipment can operate efficiently.

1.2 Equipment. All plant, equipment, tools, and machines used in the performance of the work covered by this section shall be subject to the approval of the Contracting Officer, and shall be maintained in satisfactory working condition at all times. Excavating equipment used in the areas from which fill and borrow materials are obtained shall be capable of excavating to varying depths and of producing the necessary blending required to consistently meet the specified gradation requirements.

1.3 Excavation Limits. Limits of excavation for the various structures and parts of the work are as indicated on the drawings but the right is reserved to increase or decrease the depth or areal extent of excavation if, in the opinion of the Contracting Officer, the conditions encountered warrant such modification. Except as otherwise directed, the Contractor shall make all excavations to the profiles and sections shown.

1.4 Tolerances for Rock Excavation. Deviations from the lines and grades shown on the drawings or established by the Contracting Officer for excavation in rock shall be within tolerance limits of 9-inches, except for those excavations against which concrete structures are to be constructed.

Excavations for concrete structures have certain reference lines designated as "A" line and "B" line. The "A" line is located 9 inches above the "B" line in all inverts and 6 inches inside the "B" line in all walls. The "A" line represents the inner tolerance limit inside which no rock will be permitted to project. Any projections inside of "A" line represents the inner tolerance limit inside which no rock will permitted to project. Any projections inside of "A" line shall be removed. No timbering will be permitted to remain inside of the "A" line. "B" line is the line to which measurement for payment of excavation will be made, and is considered to be the final excavation line indicated on the drawings. Measurement for payment will be made to this line regardless of whether the limit of the actual excavation fall inside or outside of it, but, sufficient excavation inside of this line shall be performed to provide for the proper installation of steel reinforcement and placement of concrete. Any excavation beyond the "B" line shall be replaced with concrete complying with applicable portions of these specifications without additional cost to the Government.

1.5 Disposal of Excavated Material. Excavated material determined suitable for embankment or stonework shall be placed in the embankment and stone protection areas as directed or approved by the Contracting Officer, or shall be temporarily stockpiled for later use, except as otherwise specified hereinafter. The gradation of the excavated material will determine the areas of placement. Excavated material determined unsuitable for embankment shall be disposed of in the disposal or borrow areas (after excavation therefrom) as specified herein. Upon completion of the work all temporary stockpile areas shall be so graded that adequate drainage will be provided and shall be smoothly and evenly dressed and left in a neat and satisfactory condition in accordance with the sections: SEEDING and ENVIRONMENT PROTECTION. Disposal of material in borrow areas shall be performed in such a manner that the amount of material in each borrow area selected for disposal purposes is equally divided between the areas. The material shall be placed in uniformly, even layers and shall be uncompacted except for that obtained from the movement of the equipment.

2. PRESERVATION OF PROPERTY. All excavation operations shall be conducted in such a manner that street pavements, utilities, or other facilities and improvements which are to remain in place permanently will not be subjected to settlement or horizontal movement. The Contractor shall furnish and install sheet piling, cribbing, bulkheads, shores, or whatever means may be necessary to adequately support material carrying such improvements or to support the improvements themselves and shall maintain such means in position until they are no longer needed. Temporary sheet piling, cribbing, bulkheads, shores or other protective means shall remain the property of the Contractor and when no longer needed shall be removed from the site. The Contractor shall submit for approval shop drawings showing proposed method of bracing which he intends to use. All shoring and bracing shall be designed so that it is effective to the bottom of the excavation, and shall be based upon calculation of pressures exerted by and the condition and nature of the materials to be retained, including surcharge imparted to the side of the trench by equipment and stored materials. Removal of shoring shall be performed in such manner as not to disturb or damage the finished concrete.

3. EXCAVATION, EXPLORATION TRENCH. Exploration trench excavation consists of the removal and disposal of all material between the lines and grades indicated. All loose boulders and debris shall be removed from the sides and bottom of the trench.

4. EXCAVATION, FOUNDATION. Foundation excavation consists of the removal and disposal of all material between the projected bottom of the embankment and the existing ground line, as indicated.

5. EXCAVATION, ABUTMENT. Abutment excavation shall consist of the removal and disposal of all materials between the existing ground surface and the top of bedrock or as indicated on the drawings. All objectionable materials, including overburden, weathered and intensely fractured rock, and detached blocks and slabs, shall be removed to a suitable foundation rock.

6. EXCAVATION, CHANNEL. Channel excavation consists of the removal and disposal of all materials within the lines and grades indicated. Suitable materials may be used in the compacted outlet works fill portion of the required fill.

7. EXCAVATION, ACCESS ROAD. The excavation for the access roads shall consist of the removal and disposal of all materials to the lines, grades, and elevations indicated on the drawings. Suitable material excavated from the access road shall be placed within the fill portions of the roads or used in the random fill portion of the embankment.

8. EXCAVATION, SPILLWAY AND OUTLET WORKS. Excavation consists of removal and disposal of all materials, from the spillway, outlet works and related structures to the lines and grades indicated on the drawings. Excavation methods (whether by ripping or blasting) shall be controlled to prevent damage to rock outside prescribed limits of excavation. If drainage occurs due to excavation methods utilized, the Contractor shall discontinue his operation and complete excavation work by smooth blasting method specified in paragraph: BLASTING. Material suitable for stone protection shall be used as bedding or type III stone.

9. EXCAVATION FOR STRUCTURES. Excavation for all structures shall be made accurately to the lines, grades, and elevations shown or as directed. Dimensions and elevations of footings and foundation excavations shown are only approximate and may be changed if necessary to assure adequate foundation support. Trenches and foundation pits shall be of sufficient size to permit the placement and removal of forms for the full length and width of structure footings and foundations as shown. Rock or other hard foundation material shall be cleaned of loose debris and cut to a firm surface, either level, stepped, or serrated, as shown or as directed. Loose disintegrated rock and thin strata shall be removed. When concrete is to be placed in an excavated area, special care shall be taken not to disturb the bottom of the excavation. Excavation to the final grade level shall not be made until just before the concrete is placed.

9.1 The finished surface of the outlet works subgrade shall not vary from the indicated grade by more than 1-1/2 inches when tested with a 10-foot straight edge. Excavated material suitable for the required fill may be used in the random fill portion of the embankments.

10. EXCAVATION, BORROW AREAS. Borrow shall be taken from the borrow areas indicated and at locations as directed. An excavation plan and methods and equipment to be used in excavating the borrow area shall be submitted to the Contracting Officer for approval prior to excavating in the borrow area. No borrow excavation shall be permitted within 300 feet of the embankment. The depth of cut in all borrow areas will vary and shall at all times be controlled to produce the specified gradations. Material shall be excavated with an inclined or vertical cut face for the full depth designated or required for each borrow area. Excavation shall be performed in a manner which will produce maximum blending of materials from top to bottom of the excavation. The excavation shall be conducted in such a manner that the borrow area will not pond water. Moisture shall be added before the excavation and during the blending if mixing plants are used. All borrow material shall be blended and moistened for the full depth of excavation prior to placing the materials on the embankment to meet the requirements specified in section: EMBANKMENT. The methods proposed for transporting material from the borrow area to the embankment shall be submitted for approval prior to use. If haul roads are required, the Contractor shall construct and maintain such roads. Upon completion of excavation in the borrow area all slopes shall be not steeper than 1V:4H and the areas shall be left in a neat condition, graded to drain and in accordance with the requirements specified in section: ENVIRONMENT PROTECTION. Unsuitable materials shall be wasted as directed. Areas of highly cemented materials will probably be encountered within the borrow areas. Whenever, in the opinion of the Contracting Officer, it is necessary to change the location of the excavating equipment working in borrow areas in order to obtain specified material, the Constractor shall move his equipment to a new location at no additional cost to the Government. The Contractor shall provide sufficient personnel in the borrow areas to monitor the excavation and direct the disposition of the excavated materials.

11. EXCAVATION, DENTAL. Unsuitable materials on the west abutment occurring in pot holes, scour channels or other small depressions underlying the core material zone of the embankment shall be removed and disposed. When conditions are such that unsuitable materials must be removed using hand methods for excavation, all materials removed in this work will be classified as "Excavation, Dental," regardless of composition. The Contracting Officer will determine, in the field, the extent to which such material shall be removed, including the depth, direction and dimensions of the excavations.

12. SCALING consists of the removal and disposal of bedrock material from vertical and over-hanging rock ledges on the west abutment, within the cut off trench below streambed, and elsewhere as directed. The required scaling will be indicated on the ground surface with stakes, paint or like means by the Contracting Officer. Large rock overhangs and protrusions shall be removed to a relatively uniform slope by means of line drilling and blasting. Material excavated from the areas scaled shall be disposed of in the designated waste areas. Scaling within 100 feet of areas to be grouted shall be completed prior to commencement of grouting operation. The abutment above streambed shall be scaled before any work is commenced in the streambed near the abutment toe.

13. STONE GUTTER. Excavation of stone gutters shall be accomplished by cutting accurately to the cross sections, grades, and elevations shown. Care shall be taken not to excavate gutters below grades shown. Excessive open gutter excavation shall be backfilled with satisfactory, thoroughly compacted

material or with suitable stone or cobble to grades shown at no cost to the Government. Materials excavated shall be disposed of as shown or as directed, except that in no case shall material be deposited less than 4 feet from the edge of a ditch. The Contractor shall maintain all excavations free from detrimental quantities of leaves, brush, sticks, trash, and other debris until final acceptance of the work.

14. REMOVAL OF UNSATISFACTORY SOILS. The removal of soils which are unsatisfactory for foundations of the embankment, structures, roads, and drains, may be required in certain areas. The Contractor will be required to excavate any such areas to the depth directed and backfill the areas with compacted fill conforming to the requirements of the sections: FILLS AND SUBGRADE PREPARATION; and EMBANKMENTS.

15. DISPOSAL OF EXCAVATED MATERIALS. Excavated materials suitable for required fills shall be placed in temporary stockpiles or used directly in the work. The miscellaneous fill areas are required fills and shall be filled in accordance with the requirements of the sections: FILLS AND SUBGRADE PREPARATION; and EMBANKMENTS. All excess excavated materials, unsuitable material from the borrow area, excavated materials not suitable for fills, and unsatisfactory materials shall be disposed of in the disposal areas indicated on the drawings. No excavated material or waste of any kind shall be disposed of at any place beyond the limits of the work under this contract without express authority. The stockpiles and disposal fills shall be placed in a manner to preclude ponding of water.

16. LANDSCAPE STONE.

16.1 General. Landscape Stone is defined as basalt with a natural desert varnish (dark colored) finish on no less than 40 percent of the exposed surfaces. The sizes and quantity of stones salvaged shall be within the limits indicated on the drawings. The Contractor shall choose stones that are as free as possible from fractures, cracks, and chipped surfaces.

16.2 Source. All stones required for landscaping work shall be salvaged from the west abutment and spillway area. The Contractor shall use only those stones lying within the areas designated for excavation.

16.3 Handling and Storage. Stones shall be selected and removed before excavation work begins. Stones shall be handled with care so that chipping, cracking or breaking of the desert varnish finish will not occur. Rocks shall be stored at a safe location, away from haul roads and other areas of heavy construction activity. Upon completion of the work, any excess stones shall be disposed of as directed by the Contracting Officer.

16.4 Placement. Landscape stones shall be transported from the stockpile and installed in a manner that will avoid chipping, cracking, and breaking of the desert varnish finish. Stones shall be placed in natural reclining position with the desert varnish side exposed. Stones positioned on the ground shall be placed so that one-third of the volume of the rock shall be below grade. Stones shall be located as shown on the contract drawings. The Contractor shall notify the Contracting Officer at least 72 hours prior to placement. Any in-place stone with over 20 percent of its desert varnish finish chipped off shall be replaced with landscape stone of comparable size.

17. BLASTING.

17.1 Safety Requirements. For these requirements reference is made to the paragraph: ACCIDENT PREVENTION in the GENERAL PROVISIONS. Blasting will be permitted only when precautions are taken for the protection of persons, the work, and property. The Contractor shall inspect the 42-inch waterline both prior to and after blasting in the spillway. The results of the inspections shall be documented and a copy presented to the Contracting Officer. Any damage to the work or property shall be repaired by the Contractor at no expense to the Government.

17.2 General.

17.2.1 The Contractor shall submit 3 copies of drawings of blasting proposals and blasting schedules for approval before starting any blasting in any area, showing the position and spacing of drill holes, types of explosives proposed for use, position and size of each charge, and the timing schedule for detonating the charges. Use of non-electric blasting caps is prohibited.

17.2.2 The contractor shall control the blasting procedures so as not to overshoot and will be required to remove, at his own expense, any material outside the authorized lines and grades indicated on the drawings which may be shattered or loosened by such blasting.

17.2.3 During blasting operations the Contractor shall have on the site, and in immediate charge of the blasting and rock excavation, a blasting expert acceptable to the Contracting Officer, who has had no less than 3 years of continuous experience in blasting and rock excavation operations. Powder handlers shall have had no less than one year of continuous experience in preparation and loading of powder charges.

17.2.4 Variations of smooth blasting procedures as described herein will be permitted providing the Contractor demonstrates that the substitute procedure will achieve an equivalent sound rock face at the final excavation lines.

17.3. Smooth Blasting

17.3.1 Smooth blasting will be required within the areas indicated for the spillway and outlet excavation and such other areas as directed. The explosives shall be of such strength and quantity and shall be used in such a manner as will neither open seams nor otherwise damage the rock outside of prescribed limits of excavation. In the spillwy areas, where smooth blasting is required, buffer zones shall be provided as indicated. Blasting in the buffer zone shall be performed in a manner which will produce relatively smooth and sound rock faces at the preliminary and final excavation lines. As the excavation approaches the preliminary and final excavation lines the depth and spacing of blast holes and the amount of explosive shall be varied with the field conditions to prevent damage to the rock faces. Whenever, in the opinion of the Contracting Officer, blasting may injure the cut slopes or rock upon which or against which concrete is to be placed, the use of explosives shall be discontinued and the excavation shall be completed by wedging, barring, channeling and broaching, or other suitable methods. Any damage to or displacement of supports and any damage to any other part of the work caused by blasting shall be repaired by and at the expense of the Contractor and in a manner satisfactory to the Contracting Officer. No blasting will be permitted within 100 feet of concrete which has been in place less than 7 days, not within 100 feet of a grouted area.

17.3.2 Excavation outside of the buffer zone shall be performed prior to any work on the buffer zones. Blasting of the buffer zones shall be done in comparatively small rounds. The rock forming the buffer zones on the side slopes shall be removed in succesive thin slices commencing with the slice adjacent to the free face of the buffer zone, indicated as the preliminary excavation line on the drawings, and advancing progressively to the final excavation line. The slices shall be parallel to the final excavation line. Multiple-row rounds shall be used with regular or long period delays between rows rather than short period or millisecond delays. The delays shall be selected to provide sufficient time between detonation of rows to allow each outer slice to clear the face prior to detonation of the next row. The spacing between the rows of blast holes and the spacing between the blast holes in the individual rows in the buffer zones shall be varied to produce a sound wall at the final excavation line except that the spacing between blast holes on the final excavation lines shall not exceed those indicated on the drawings, or as directed.

17.3.3 Blasting of the horizontal buffer zone above the spillway invert shall be done by use of successive rows of blast holes drilled perpendicular to the invert. The spacing of the vertical rows of blast holes shall not exceed 36 inches and the spacing between holes in a row shall not exceed 30 inches or as directed.

17.3.4 Ammonium nitrate type explosives shall not be used in any of the buffer zones and shall not be used within 10 feet in advance of the preliminary excavation lines.

17.4 Demonstration of Capability. The capability of the Contractor to perform any blasting technique shall be demonstrated at a point no closer than 10 feet from a preliminary excavation line. If a demonstration does not produce results satisfactory to the Contracting Officer, the Contractor shall modify his technique and perform such demonstrations as are necessary to produce the desired results.

17.5 Records. The Contractor shall keep and furnish to the Contracting Officer accurate logs and records of all operations pertaining to the preparation, drilling, blasting and excavation procedures. The records shall be submitted daily with the Quality Control Report and shall include the following.

17.5.1 The number, size, type, and make of all equipment used in the excavation process.

17.5.2 The number, size, depth, direction, and spacing of drill holes utilized for each round of blasting, to include a plan sketch drawing.

17.5.3 The quantity, type strength, and stemming of explosives used in blasting each drill hole.

17.5.4 The type, make, and system of detonation used for each round of blasting.

17.5.5 The volume of rock excavated from each round of blasting.

17.5.6 Any unusual drilling or blasting occurrences.

* * * * *

RIGHT ABUTMENT
EVALUATION

ATTACHMENT 3

Attachment 3, Right Abutment Evaluation, was originally prepared by the Los Angeles District Geotechnical Branch as a handout for the combined SPD and OCE site inspection of Adobe Dam on 21 and 22 April 1981.

ADOBE DAM

Maricopa County, Arizona
Gila River Basin
New River and Phoenix City Streams, Arizona

Contract No. DACW09-80-C-0121

Right Abutment Evaluation

ADOBE DAM

The Right Abutment

1. Prior to construction the materials on the surface of the right abutment had the appearance of a large rubble pile and consisted of talus debris, slope wash and large quantities of basalt boulders to 6-feet in size. Cemented caliche material was present in small quantities. Based upon core holes, drilled during design, the underlying bedrock surface was expected to consist of medium to highly fractured basalt and andesite with fractures and small cavities filled with a fine tuff ash material.

2. During the early stages of construction, because of the difficulty of making the abutment excavation using heavy duty dozers with rippers, the Contractor proposed to drill and blast the material to loosen it sufficiently so that it could be more easily removed with heavy construction equipment. With the concern for providing a suitable foundation for placing embankment material, the Contractor was permitted to demonstrate his excavation in the downstream portion of the abutment. The Contractor was asked to excavate and air clean a test section approximately 40x40-feet downstream of Station 11+50 to 11+90 to evaluate if his excavation procedure would provide a suitable abutment foundation. After such excavation, inspection of the exposed surface indicated satisfactory result. Hence, the excavation method was approved and the downstream abutment excavation was completed, followed by the abutment excavation for the upstream portion of the abutment.

3. The abutment foundation as exposed in the test section would be suitable after adequate surface treatment. The surface consists of an irregular surface of large basaltic blocks with tuffaceous materials filling the fractures and small cavities between the blocks. The filled fractures are interconnected, larger, and more numerous than that anticipated during the design. Overall the tuffaceous fill material is dense, relatively incompressible, relatively impervious and non-soluble. To determine the engineering characteristics of the tuffaceous materials two undisturbed chunk samples were obtained for detailed laboratory testing. The samples were tested by Engineering Testing Laboratories, Inc., Phoenix, Arizona and by the SPD Laboratory at Sausalito, California. The tests conducted by each laboratory are listed below and the results of the tests are attached.

Engineering Testing Laboratories, Inc.

a. Gradation

b. Inplace Density

c. Specific Gravity

d. Consolidation - Sample saturated at 4 TSF

e. Dispersive Soils Test - Crumb Test

f. Soluble Salts Test

SPD Laboratory

a. Gradation

b. Inplace Density

c. Specific Gravity

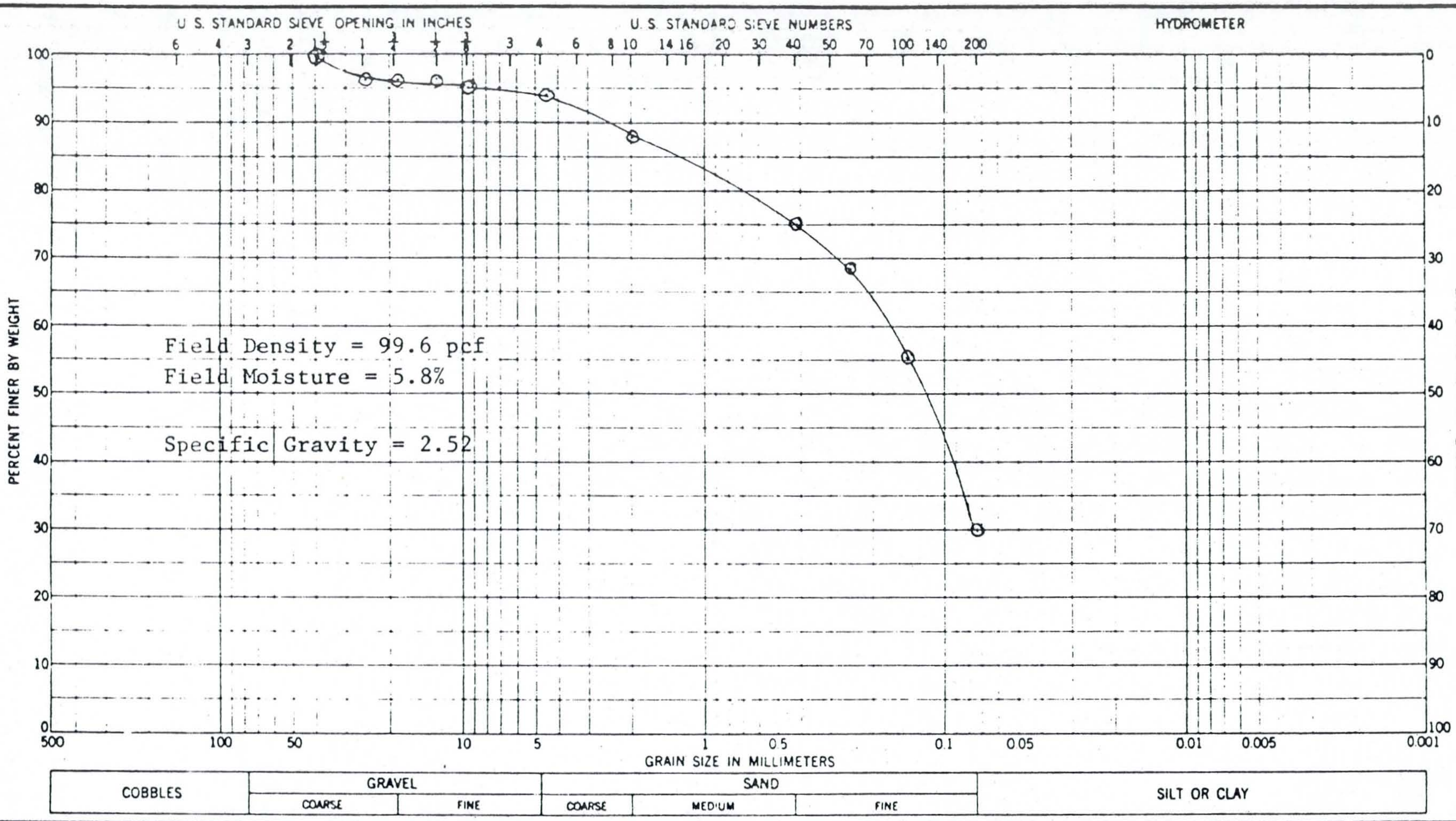
d. Consolidation

e. Soluble Salts Test - Not completed

f. Permeability Test

4. After cleaning the abutment foundation of all loose materials, the area to receive core material would be cleaned by air blasting and hand picking. Prior to placement of embankment materials, abutment areas underlying the core material would be treated with grout slurry and dental concrete, as required. Grout slurry would be used to fill fractures and joints. Dental concrete would be used in areas where core materials cannot be placed and compacted. After abutment treatment is completed, core materials would be placed and hand compacted in areas where equipment compaction cannot be used. The placement and hand compaction operation would proceed immediately ahead of embankment construction.

5. The abutment area to receive gravel drain and random material would be cleaned of loose material. Air blasting would be used to aid the removal of loose materials. After cleaning is completed, dental concrete would be placed in areas where embankment materials cannot be placed. Random material would be placed, and hand compacted in areas where compaction equipment cannot be used. The placement and hand compaction would proceed immediately ahead of embankment construction.



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
74766		Silty Sand (SM)	5.8		NP	

Project ADOBE DAM

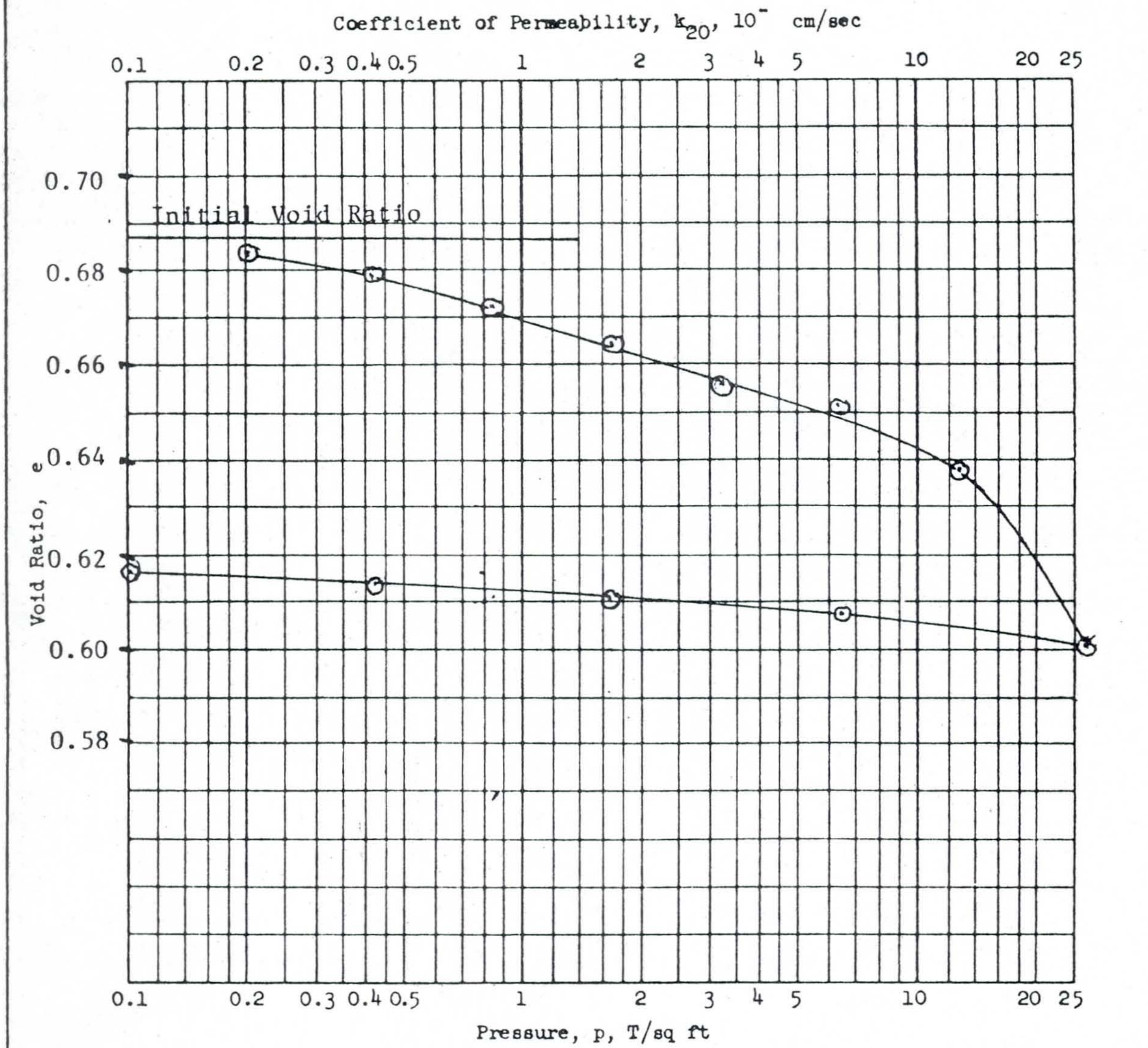
Area Abutment

Boring No.

Date February 1981

GRADATION CURVES

ENG FORM 1 MAY 63 2087

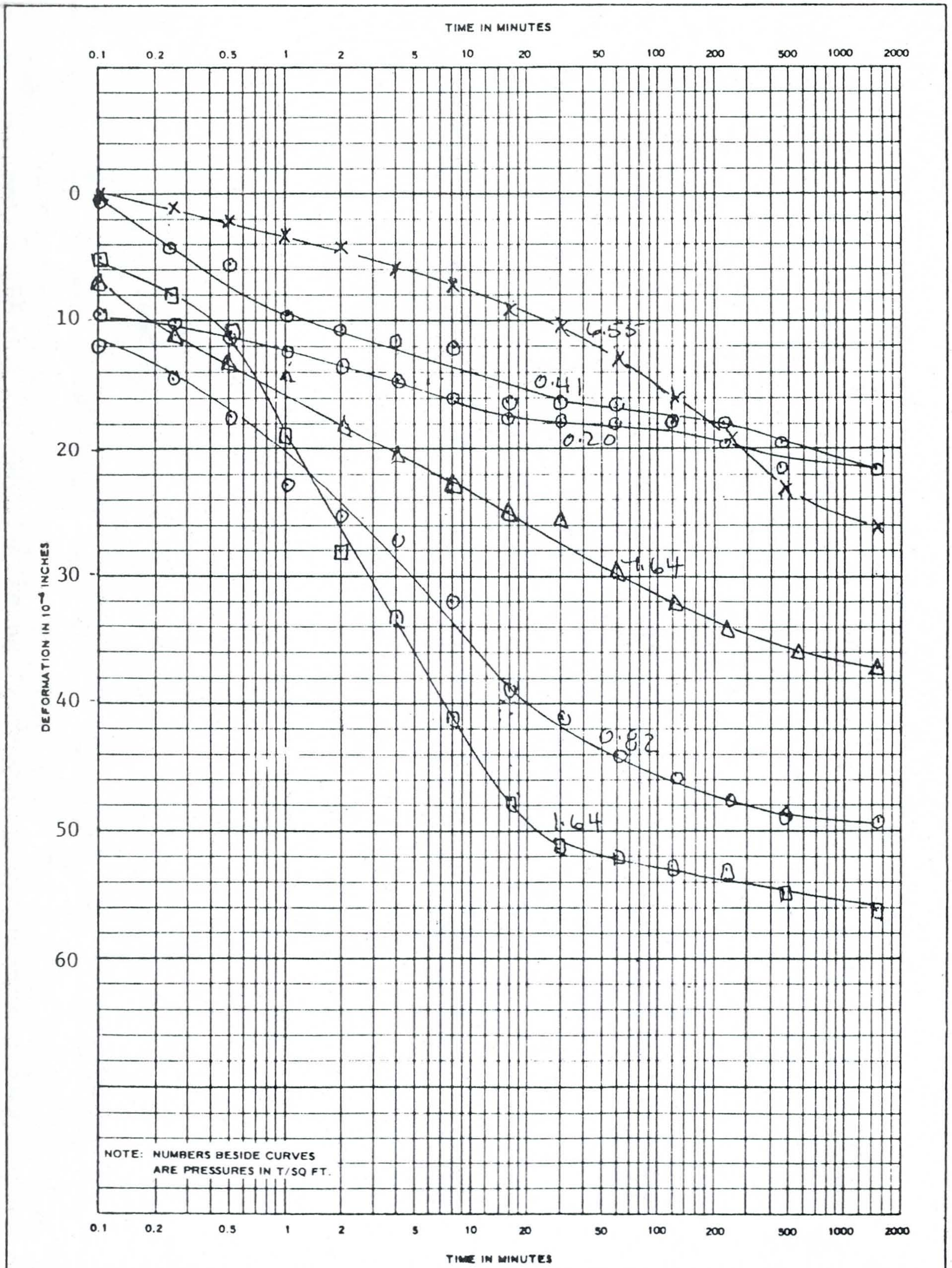


Type of Specimen Undisturbed		Before Test		After Test	
Diam 44 in.	Ht 1.00 in.	Water Content, w_o	6.4 %	w_f	25.0 %
Overburden Pressure, p_o T/sq ft		Void Ratio, e_o	0.687	e_f	0.629
Preconsol. Pressure, p_c T/sq ft		Saturation, S_o	24 %	S_f	100 %
Compression Index, C_c		Dry Density, γ_d	93.2 lb/ft^3		
Classification Silty Sand (SM)		k_{20} at e_o =	× 10⁻ cm/sec		
LL	G_s 2.52	Project	ADOBE DAM		
PL NP	D_{10}				
Remarks		Area	Abutment		
		Boring No.		Sample No. 74776	
		Depth El		Date March 1981	

CONSOLIDATION TEST REPORT

ENG FORM 1 MAY 63 2090 PREVIOUS EDITIONS ARE OBSOLETE. *(TRANSLUCENT)*

0 3424



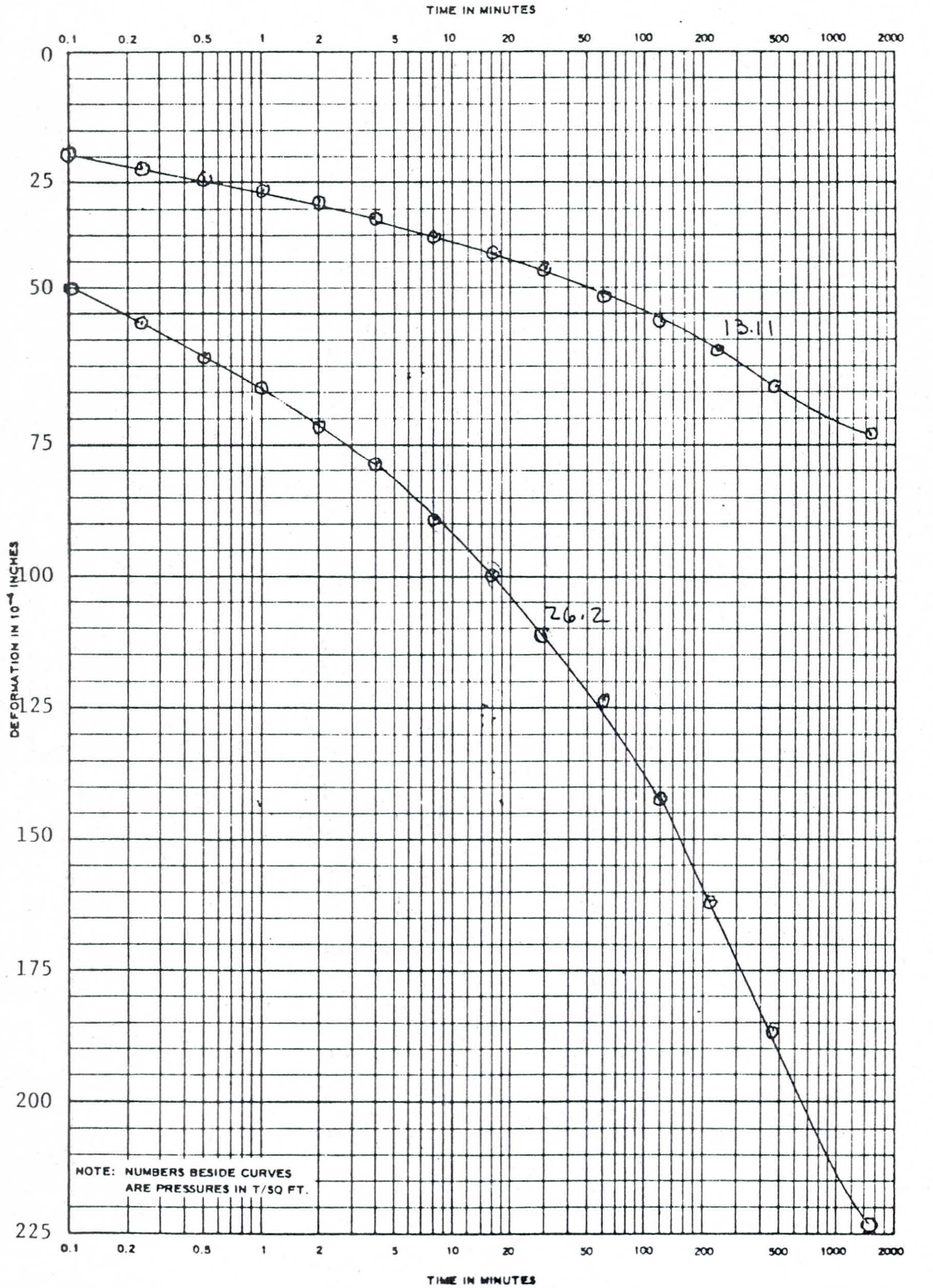
Project	ADOBE DAM		
Area	Abutment		
Boring No.	Sample No. 74776	Depth El	Date March 1981

ENG FORM 1 MAY 63 2088
PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

C 3427

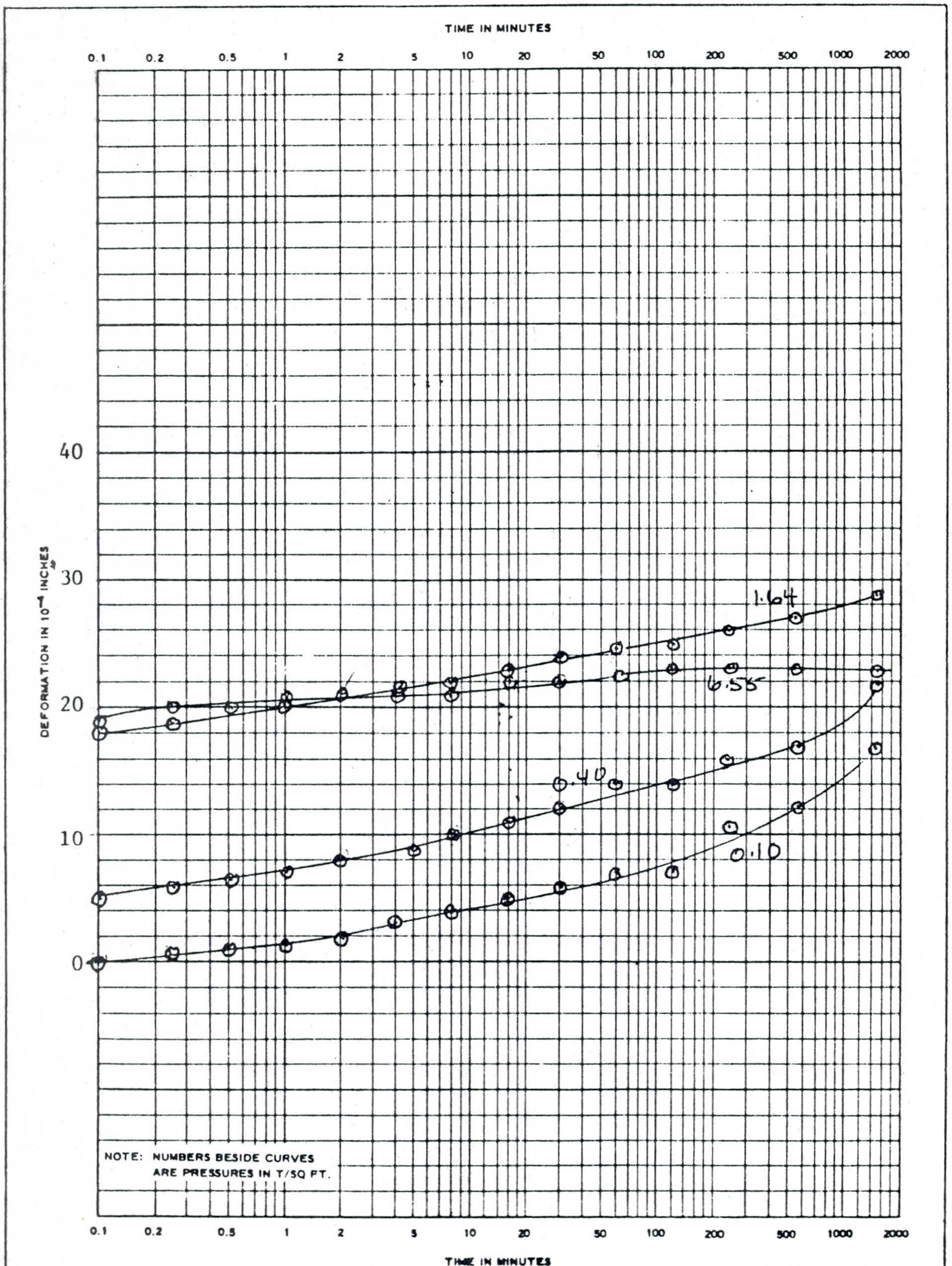


Project ADOBE DAM			
Area Abutment			
Boring No.	Sample No. 74776	Depth El	Date March 1981

ENG FORM 2088, 1 MAY 63 — PREVIOUS EDITIONS ARE OBSOLETE.

CONSOLIDATION TEST--TIME CURVES (TRANSLUCENT)

C 3427



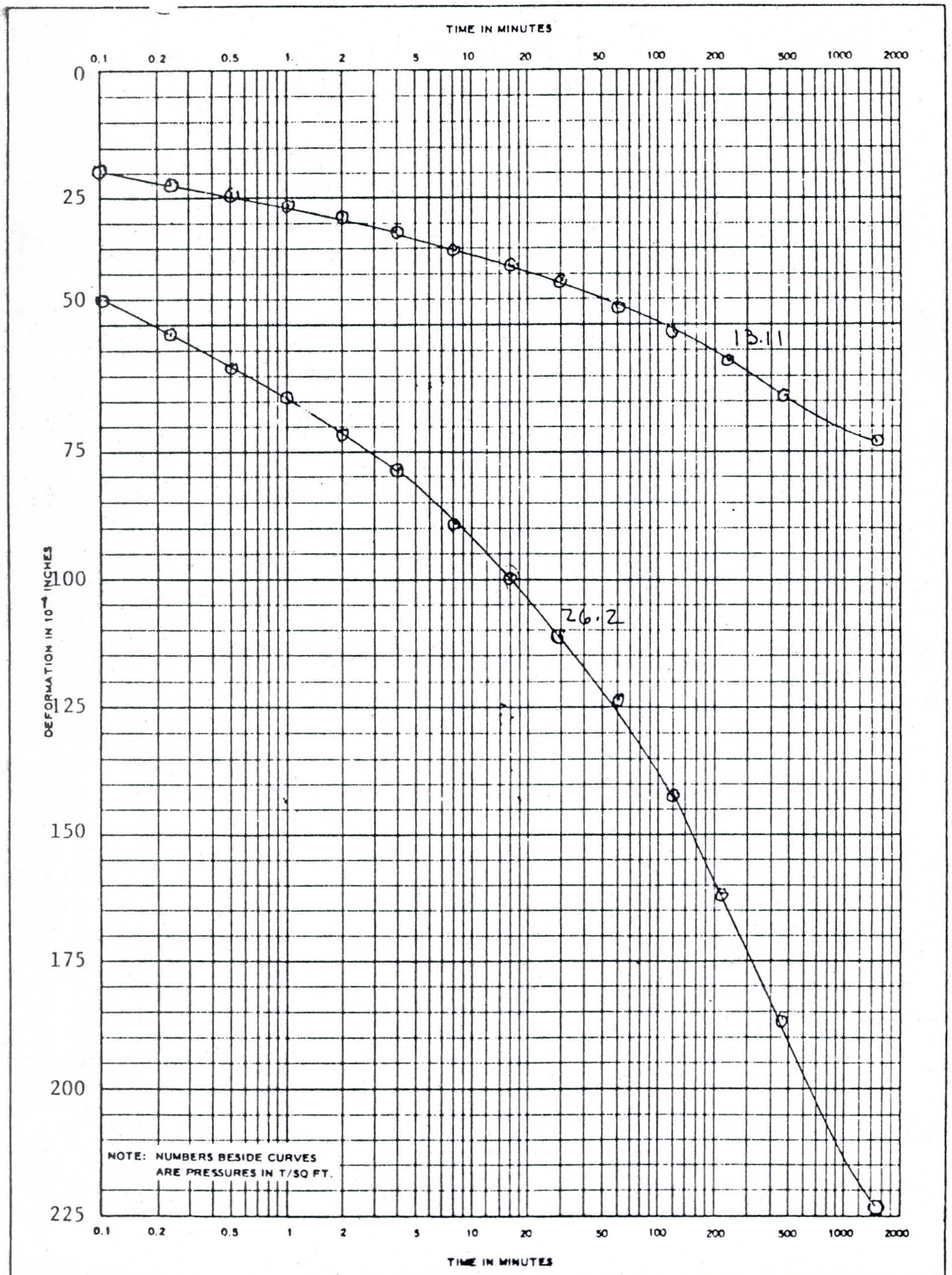
Project	ADOBE DAM		
Area	Abutment		
Boring No.	Sample No. 74776	Depth El	Date March 1981

ENG FORM 1 MAY 63 2088
PREVIOUS EDITIONS ARE OBSOLETE

CONSOLIDATION TEST--TIME CURVES

(TRANSLUCENT)

C 3427



Project	ADOBE DAM		
Area	Abutment		
Boring No.	Sample No. 74776	Depth El	Date March 1981

ENG FORM 1 MAY 63 2088 PREVIOUS EDITIONS ARE OBSOLETE.

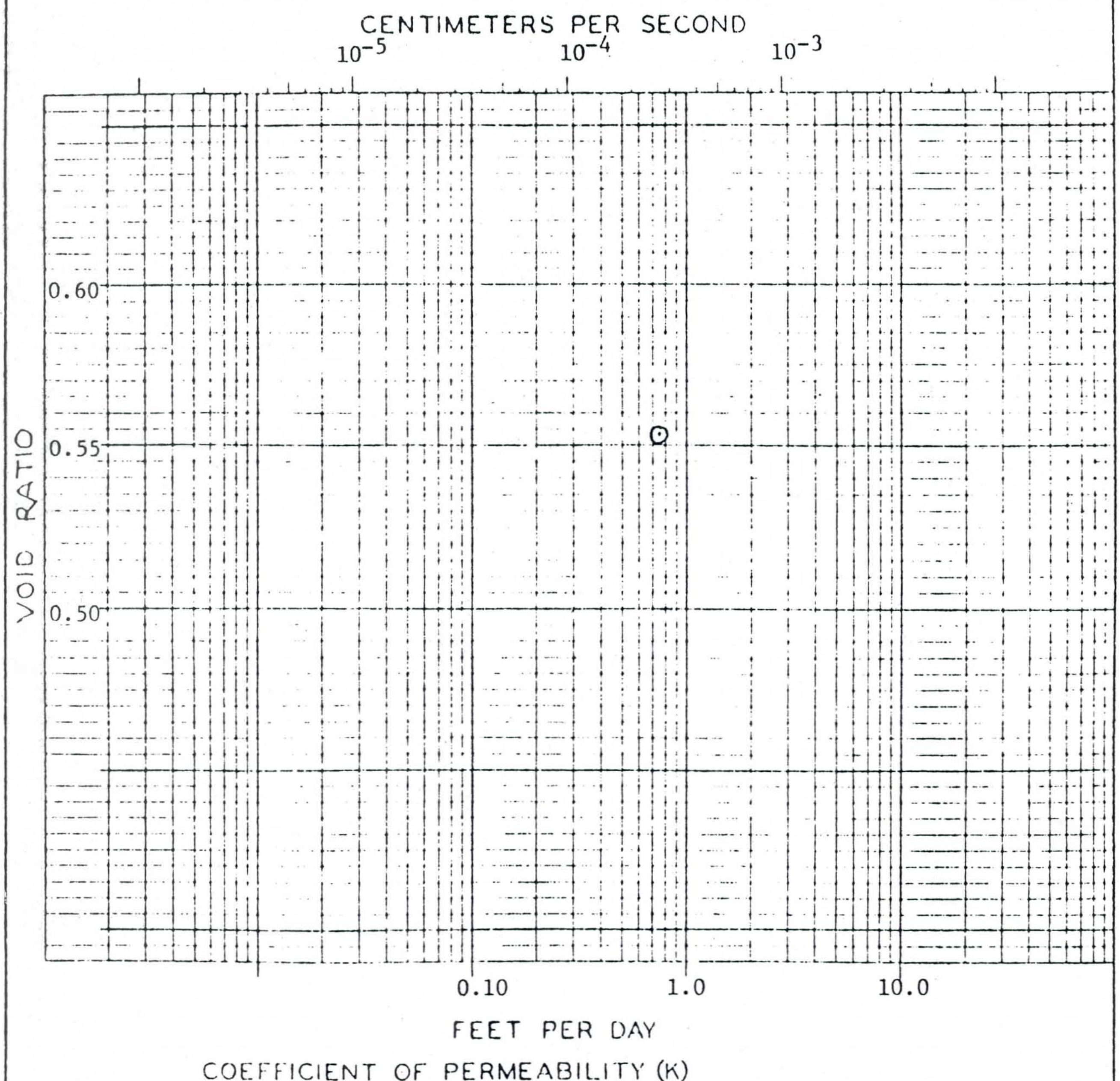
CONSOLIDATION TEST--TIME CURVES (TRANSLUCENT)

C 3427

CORPS OF ENGINEERS | U. S. ARMY

SOUTH PACIFIC DIVISION LABORATORY

PERMEABILITY



REMARKS:

CURVE	SPECIMEN			
	DIAM. IN.	HT. IN.	MAX. PARTICLE	CONDITION
	4.0	2.45	No. 4	Undisturbed Horizontal

DISTRICT: Los Angeles

PROJECT: ADOBE DAM

CURVE	DIV. NO	HOLE NO.	F.S. NO.	DEPTH FROM	DEPTH TO
	74776	Abutment			

TESTED	COMPUTED	DRAWN	CHECKED
DW			

ENGINEERS TESTING LABORATORIES, INC.
☐ 3737 East Broadway Rd, PO Box 21387, Phoenix, Arizona 85036, Tel 602-268-1381
☐ 423 South Olsen Ave, Tucson, Arizona 85719, Tel 602-624-8894
☐ 2400 East Huntington Dr, Flagstaff, Arizona 86001, Tel 602-774-4881

LETTER OF TRANSMITTAL

To: Army Corps of Engineers
c/o M. M. Sundt
R. R. 3, Box 519
Glendale, AZ 85308

Date: 2-5-81
Job No. 21200567
P. O. AD165
Lab./Invoice No. 70-060
Ref. No. Adobe Dam

Attn. David W. Lukesh

Project/Subject: Test on Dam Abutment Material

Please be informed that we are:

X	Enclosing		Engineering Reports
	Forwarding Separately	X	Laboratory Reports
	Per Your Request		Field Reports
	No. of Copies		Proposals
	Other		

More fully described as follows: Consolidation, Dispensive Soil, Permeability, Sieve Analysis, Atterberg Limits, Specific Gravity and Soluble Salts Tests.

For your:

X	Use		Information
	Approval		Action
	Files		Other

Material forwarded by:

	Our Messenger		Express Mail
X	Your Messenger		Air Priority
	First Class Mail		United Parcel Service
	Priority Mail		Motor Freight
	Certified Mail		Air Freight
	Special Delivery		City Delivery
	Other		

Copies to: addressee (5)

By [signature]
Kenneth L. Ricker, P.E.

Type of Material Bulk

Source of Material Adobe Dam

Job No. 21200567

Test Procedure ASTM: D422; D423; D424; D854

Tested/Calc. By TB Date 1-28-81

Reviewed By KLR Date 2-5-81

Hole No.	Location	Soluble Salts	Classification		LL	PI	Sieve Analysis - Accum % Passing													Specific Gravity
			AASHTO	Unified			200	100	40	16	10	4	¼	⅜	¾	1	1½	2	3	
1	-	0.12%	--	SM	-	NP	18	35	51	59	64	77	81	88	95	97	100			2.53

TABULATION OF TEST RESULTS

Job No. 21200567

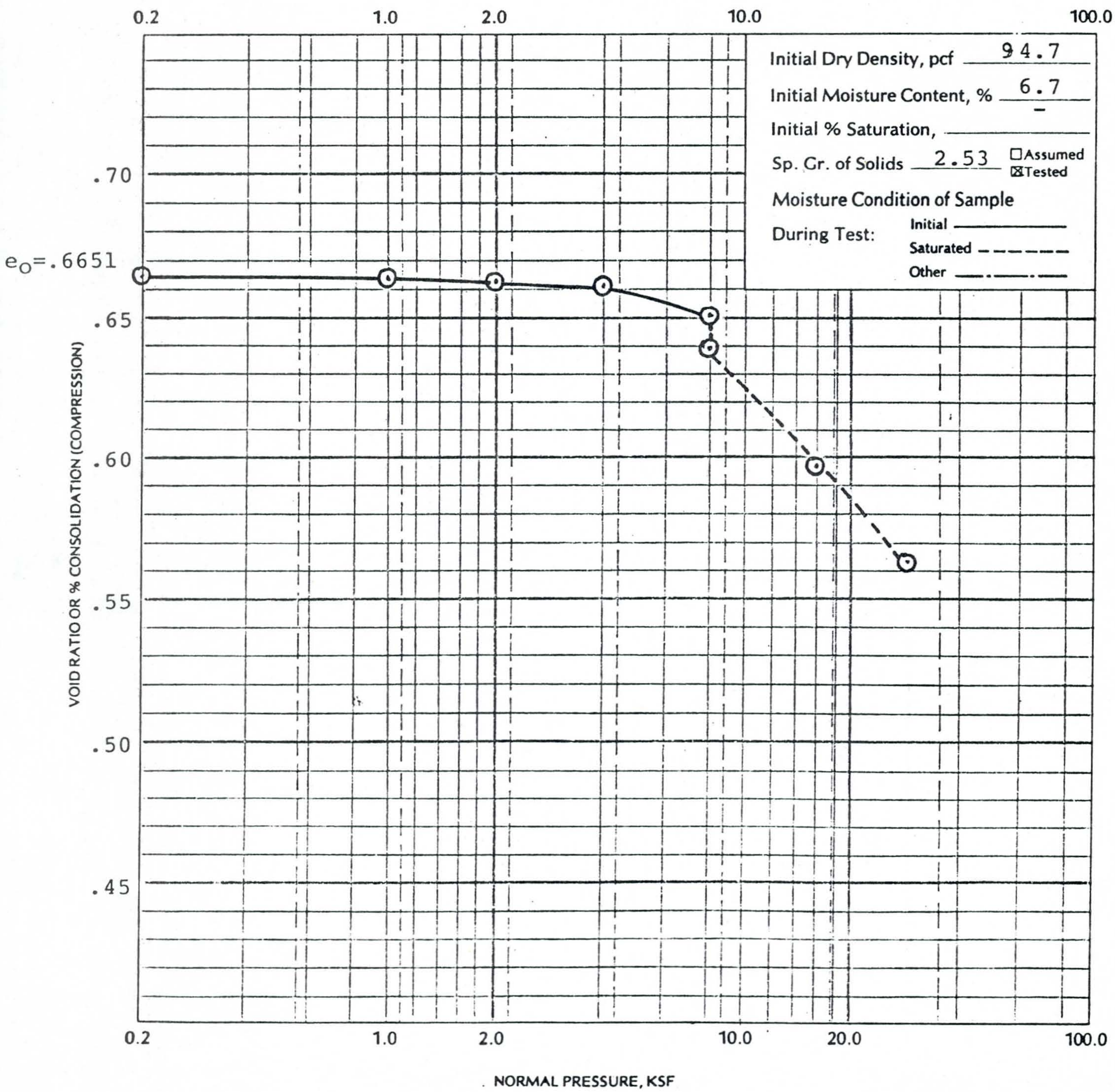
Lab No. A

CONSOLIDATION PROPERTIES OF SOIL

Type of Material: Silty Sand (SM) — ☒ Undisturbed ☐ Remolded ☐ Compacted

Source of Material: Adobe Dam — Boring: - Depth: -

Test Procedure: ASTM D2435- — Reviewed By: KLR — Date: 2-5-81



Job No. 21200567

LABORATORY REPORT

Type of Material/Specimens Bulk

Source of Material/Specimens Adobe Dam

Test Procedure Dispensive Soil & Permeability Tests Sampled By - Date

Reviewed By KLR Date 2-5-81 Submitted By BS Date

RESULTS

DISPENSIVE SOILS TEST

Crumb Test - Grade 1. No Reaction: Crumb may slake and run out on bottom of the beaker in flat pile, but no sign of cloudy water caused by colloids in suspension.

PERMEABILITY TEST

Four attempts were made at performing a constant head permeability test on a sample carved from the bulk sample delivered to the site. Each attempt was unsuccessful due to the inability to seal the samples to the permeameter.

REVIEW OF FOUNDATION CONDITIONS BY TECHNICAL EXPERTS

ATTACHMENT 4

ConverseWardDavisDixon

Geotechnical Consultants

April 2, 1981
81-4113-01

Los Angeles District
Corp of Engineers
300 North Los Angeles Street
Los Angeles, CA 90012

Attention: Mr. Lawrence J. Lauro, Chief,
Foundations and Materials Branch

Subject: Adobe Dam

Gentlemen:

On March 18, 1981, the undersigned attended a meeting and made a field inspection of Adobe Dam in accordance with your purchase order No. DACWO 9-81-M-2113. Attendees at the meeting included your contractor-engineer, Mr. Joseph Sciandrone, and representatives from Los Angeles District's Construction-Operations and Engineering Divisions, Phoenix Area Office, and Resident Engineer's staff, and from South Pacific Division's Construction-Operations and Engineering Divisions. A list of attendees is attached.

The purpose of the inspection was to review the foundation conditions of the right abutment as exposed by excavation and to discuss a potential drain material gradation modification as well as a contractor's proposal to use a vibratory roller on core material for the dam. This report will deal chiefly with the abutment foundation conditions and proposals for treatment. The subjects of drain materials and compaction will be covered in a separate report to be furnished by your contractor-engineer.

Prior to our inspection of the work we were given an excellent briefing on the status of work by Capt. Paul Dunn and on the history and chronology of the right abutment excavation by David Lukesh. At the time of inspection a good portion of the core contact area had been excavated and cleaned for the inspection. Foundation excavation in the valley section has been started and a small amount of embankment material had been placed in January between station 21+00 and 25+00. The contractor's activity on March 18 was concentrated on the east side of 35th Avenue.

Converse Ward Davis Dixon, Inc.
The Folger Building, Suite A
101 Howard Street
San Francisco, California 94105
Telephone 415 543-7273

Interpretation of design exploration core borings from the right abutment resulted in the expectation of having a foundation of medium to highly fractured andesite with fractures choked with tuffaceous ash materials. Excavation has revealed a foundation very much as expected except that fractures are larger than visualized and the cleaned surface is very irregular in relief with individual andesite blocks as large as 7 feet or more across. The relief and semi-angular shapes cause the foundation to appear as large detached blocks embedded in a matrix of dense lithic tuff. Close inspection, however, reveals that the rock mass is, indeed, in-place bedrock which appears to be a remnant andesite flow with through going joint surfaces.

To develop the proper foundation treatment for the dam an understanding of the origin of the rock mass and the reason for the extremely fractured condition is required. In my opinion, the exposed rock represents an andesite flow which has been subjected to extreme tensile stresses causing extensive fracturing of the rock. Rock in the vicinity of stations 10+00 to 12+00, has possibly slumped into position. Movement has been minimal, however, it caused blocks of rock to separate 10 inches or more. Pyroclastic volcanic activity supplied the material for backfilling of the fractures and later it appears that the tuffaceous sediments were at least partially deposited in the fracture system by water action. This scenario suggests that the fractures are completely filled with tuffaceous sediments, but with depth, there could be, and likely are, fractures which did not get backfilled and which are open to passage of water. Water testing of exploration borings supports this hypothesis. Work to be accomplished, such as excavation for the outlet conduit at the base of the abutment and the grouting program will provide further data allowing a better understanding of the foundation genisis.

Testing of the tuffaceous fracture filling material has demonstrated that it is dense, incompressible and less permeable than core materials for the dam. It has a field classification of a gravelly, silty, very fine sand. In my opinion the foundation, as exposed at the time of inspection, is satisfactory and further excavation is unnecessary. Because of the gross irregularity of the surface, partially due to dental excavation of tuffaceous materials from the fractures, it will be prudent to use backfill concrete to reduce gross foundation irregularity in the core contact area. Dental excavation of the tuffaceous infilling, to the extent as has been done, is not judged to be necessary in other areas to be prepared for foundation.

At the upper part of the right abutment foundation near station 10+00 the excavation has been cut near vertically for about 20 feet. This cut through the andesite flow reveals some open fractures. These may be representative of the type of fracture openings below that upper zone of rock having impervious tuffaceous fracture filling lower on the abutment. These open fractures will require careful treatment by grouting. The slope, as excavated, is too steep to compact core material against and makes treatment of the open fractures difficult. I support the project geologist's proposal to have the slope re-excavated to 1H:1V. This excavation will require blasting which should be carefully controlled to avoid further disturbance to the rock.

Foundation grouting on the right abutment was designed to extend the exploration program. Now that foundation conditions are better understood, the grouting takes on a more important aspect of foundation treatment. Holes which penetrate the infilled fractures will require water testing to detect voids or open conduits. Previous exploration indicates that, below the infilled fractures, there is a system of smaller open fractures that are potential seepage paths. These fractures should be grouted to the depths planned, paying particular attention to areas of unusually high grout takes. Split spaced holes should be installed in any areas where it is difficult to get grout refusal in the planned holes. The criteria used to determine need for splitting is arbitrary, but as a starting guide it is recommended that additional holes be installed if grout takes average as much as 0.3 sacks per lineal foot of hole. Where it has been necessary to split spacing closer than 5 feet, or where open fractured rock can be observed at the surface, consideration should be given to installation of a shallow line of holes, on the order of 10 feet deep, up and downstream of the main grout curtain. These auxillary lines should be no more than 8 feet each side of the main curtain. The planned depth of 40 feet for the grout holes appears to be adequate. Considering the nature of the foundation ,it is my opinion that the planned 20 foot spacing for initial holes on the upper part of the abutment should be split to 10 feet throughout. Subsequent splitting should be done if significant grout takes are encountered.

Design of the dam appropriately includes a drain blanket to rest on the rock downstream of the core and beneath the random shell. It is suggested that gradation of the drain material be checked against the finer gradation of the tuffaceous sediment filling the large fractures to determine if filter requirements are met. Although the surficial foundation treatment and grouting are expected to provide a satisfactory foundation, a redundant line of defense against piping of the fine-grained tuffaceous material can be provided with a properly graded material. If filter requirements are not met by the drain it is recommended that a finer grained, but pervious, layer of filter material be placed between the drain blanket and rock.

We made an abreviated inspection of the spillway excavation. The work appears to be progressing satisfactorily with no major problems. The large cut slopes expose an interesting sequence of the volcanic rocks. Geologic mapping of the cuts and interpretation of the geology will undoubtedly help in the final interpretation of the dam abutment foundation.

CONCLUSIONS AND RECOMMENDATIONS

Right Abutment Foundation

1. The right abutment foundation is essentially as interpreted from design exploration.

2. The rock is andesite which appears to have slumped in the geologic past creating large fracture openings subsequently filled with tuffaceous sediments.

3. The rock, as excavated at the time of inspection, is judged to provide a satisfactory foundation for the dam.

4. Because of the gross irregularity of the excavated foundation surface and difficulty of hand compacting core material in the narrow fractures, use of dental concrete or shotcrete to reduce the overall relief is recommended.

5. The grouting program will be an essential part of the foundation treatment; the planned initial hole spacing of 20 feet in the upper portions of the abtument should be split to 10 feet and spacings should be further split if significant grout takes are encountered.

6. In areas of high grout take or areas where an open fractured foundation is observed, supplementary lines of grout holes up and downstream of the curtain should be considered.

7. The steep slope at the upper right abutment should be re-excavated to 1:1 to facilitate foundation

   grouting and compaction of embankment material against the rock.

8. If the downstream drain blanket gradation is not satisfactory as a filter against the tuffaceous sediment crack filling material, a properly graded, pervious, filter should be provided between the drain and the rock.

Very truly yours,

CONVERSE WARD DAVIS DIXON

Alan L. O'Neill

Alan L. O'Neill
Vice President

ALO:ja

Enclosure

ADOBE DAM MEETING
March 18, 1981

List of Attendees

Larry Lauro

Dan Parrello

Tak Yamashita

David Lukesh

Ray Coker

R.P. Young

Terry Buckley

Capt. Paul Dunn

Paul Ching

M.P. Carlassare

Al Verza

North Smith

Al O'Neill

Joe Sciandrone

Gary L. Foote

Encl. 1

Report to

Chief, Foundation and Materials Branch

on Adobe Dam

Prepared for:

Department of the Army

Los Angeles District, Corps of Engineers

P.O. Box 2711

Los Angeles, CA. 90053

Under Blanket Purchase Agreement

Purchase Order No. DACW09-81-M-1993

By

Joseph C. Sciandrone

2 April 1981

Encl 1

Report to

Chief, Foundation and Materials Branch

on Adobe Dam

Introduction. At the request of Chief, Foundation and Materials Branch, the writer visited the Adobe Dam project in accordance with blanket purchase agreement DACW 09-81-M-1993. The request was made on 10 March 1981 and the site visit was on 18 March 1981. The purpose of the trip was to evaluate right abutment treatment, drain material requirements and compaction procedures. Other participants in the site visit were district and division engineering and construction personnel. Mr. Al O'Neill, contractor-geologist, participated and will cover the geologic aspects in a separate report. A list of attendees is on file in the project office.

The group was briefed at the project office by Captain Paul Dunn, Project Officer. The briefing included a presentation of pertinent project data, significant construction milestone dates, project organization chart, modifications, and payments. Captain Dunn's briefing was followed by a slide presentation by Mr. Dave Lukesh, project geologist. Mr. Lukesh described the site geology with emphasis on the right abutment and spillway excavation. Following the office presentations the group inspected the excavated right abutment, the spillway excavation and core material test fill.

Data Review. Prior to the field trip, the data reviewed were plans, specifications and the GDM appendix on geology, soils and materials. In addition, a preliminary report titled "Right Abutment Evaluation" was provided by the Foundation and Materials Branch.

Right Abutment. The group inspected the right abutment. Mr. Dave Lukesh discussed the geology and described the proposed treatment. The treatment will not be described in detail because it is in general agreement with the approved design memorandum, plans and specifications and the Engineering Considerations and Instructions for Field Personnel. In the Engineering Considerations paragraph titled Bedrock Preparation, the cleaning of bedrock surfaces, grouting and surface treatment are discussed. The geologic aspects of the abutment treatment will be covered in a report by the contractor-geologist.

The basic objective of the treatment will be to "remove materials and treat areas which would provide potential seepage paths along the abutment contacts." Proposed slope flattening in the upper portion of the abutment will provide a relatively uniform surface against which embankment material can be compacted. The test results included in the report "Right Abutment Evaluation" indicate that the material filling fractures and openings between blocks has engineering properties similar to the core material. The GDM describes the single line grout curtain as essentially an investigative program. The contractual ability to increase the grouting program, as required, is covered in the specifications. The expansion of the grouting program was discussed and will be covered in the contractor-geologist report.

The embankment, as designed, provides defensive measures that mitigate for minor unpredictable conditions. To decrease the potential for seepage along the abutment, the upstream portion of the abutment contact will be blanketed with a layer of impervious material with a minimum thickness of 5 feet. Seepage through the abutment will be controlled by a high capacity drainage blanket covering the downstream portion of the foundation and abutment.

Drain Material. The specified drain material is a coarse to fine gravel with less than 5 percent coarse sand. The contractor proposed to substitute a fine gravel to coarse sand with a small percentage of medium sand. To support his proposal the contractor had Engineering Testing Laboratories (ETL) perform numerous permeability tests. The test procedures briefly described, for two tests (A&B) on prepared samples, in the ETL letter to MM Sundt follow. Additional tests were performed using the described procedures, but the majority of the tests were constant head.

The permeability tests were performed by placing approximately a one foot high sample in a 4-inch diameter, 6-foot high vertical standpipe. A No. 10 screen and a cap was placed on the bottom of the standpipe. The standpipe was then filled with water. The bottom was then removed and the time required for a measured height of water to pass through the sample was measured. The test was repeated at least 5 times for each sample.

The results of the permeability tests are summarized in Table 1. The permeabilities are the average of the trials which varied from 5 to 9. The trial values were very consistent, test A-9 showing the least consistency. The falling head tests, even though they were conducted at lower densities, resulted in the lowest permeabilities. This might suggest that minus No. 10 material was removed during the initial test trials. The test results compare well with other tests on plus No. 10 material.

Obviously, the permeability of interest is the in-place permeability. The range of gradations for 5 samples taken from the grade are plotted against the modified gravel drain gradation on Figure 1. It is noted that some of the samples are borderline. The ideal location for drain material on the fine side of the gradation band is in contact with the foundation. This results in a low piping ratio and a higher safety factor against piping.

To provide a safe design against piping, the D_{85} of the foundation or soil was arbitrarily taken as the minus No. 4 gradation D_{85}, see Figure 2. As this is a conservative approach, a limiting ratio of 5 was used in lieu of 4 which would be applicable to crushed stone. The results obtained from Figure 2 are shown on Figure 3 where they are compared with the specified gravel drain gradation and the gradation shown on Figure 1. Examination of the gradation bands shown on Figure 3 shows that either band would substantially satisfy piping and permeability criteria for the random fill and foundation. The gradation shown on Figure 1 provides a higher safety factor against piping, whereas the specified gradation provides a higher permeability. The uniformity coefficient of materials within either band are low enough to minimize segregation. The fine gravel and coarse sand mixture permeability would be affected more by particle breakdown during placement and compaction.

It is important that the permeability be determined for samples representative of the end product on the grade. Consideration should be given to using a No. 100 mesh screen in permeability tests in lieu of the No. 10 screen used in the reported testing. Unless the permeability is significantly decreased from the reported values, the modified drain should provide adequate capacity to function defensively. Providing a high capacity drain is a defensive measure because the computed through seepage and underseepage quantities are small. The high capacity drain is being provided as a defense against unpredictable seepage. Should the permeability of the modified gradation become a matter of concern, the alternative for the horizontal drainage blanket would be to sandwich a one-foot layer of specified gravel between one-foot layers of the modified gradation.

Drain material would be placed against abutment bedrock as shown on Cross Sections No. 1, sheet 38 of contract drawings. Where practicable, depressions and irregularities in the bedrock surface should be filled with compacted drain material to form a leveling course. Consideration should be given to use, in these areas, of drain material that borders the fine side of the modified gradation band. Materials similar to the early drain material production should be satisfactory in these areas. Should additional data indicate that the infill material is finer than presently indicated, a finer gradation of pervious material should be considered for the leveling course.

Embankment Material Compaction. Materials usage and design are based on placing material with 20 percent or more fines in the core and material with not more than 20 percent fines in the random zone.

The procedural specifications limit the lift thickness to 12 inches and require 8 passes of a 50-ton pneumatic roller. For the core, the option is an 8-inch layer and 8 passes of a tamping roller. The equipment requirements for the pneumatic and tamping rollers are as specified in the OCE guide.

The contractor proposed to substitute a 10-ton vibratory roller for the 50-ton pneumatic with the objective of reducing the number of passes required to achieve a density of at least 95 percent of maximum density determined by ASTM D 698. Recognizing that bond is an overriding consideration for the core, procedures were established for various methods of scrarifying. Telltale marking provided by spreading lime facilitated identifying the layers in a transverse trench through the test fill. Observers agreed that the first layer did not appear to be bonded. The top layer appears to have rebounded and there was evidence of compaction overstress. Observers' opinions varied as to the adequacy of bond with depth.

The decision whether to use a vibratory roller on core material with due attention to scarifying to assure bond is a policy matter as well as a technical issue. The Embankment (For Earth Dams) Guide Specification CW-02212 Feb 76 includes tamping rollers, vibratory rollers and rubber-tired rollers. The implied usage of vibratory rollers is "for compacting rockfills, pervious sand and gravel fills, or filter and transition drainage layers." There are known exceptions to that restricted usage, the best known being the compaction of random fill zones at Warm Springs Dam, see Figure 4. Therefore, there is a precedent for use of a vibratory roller on the random fill, but no known Corps precedent for its full scale use on core material. As the objective of using the vibratory roller is to increase productivity, use of the roller on random material would accomplish that end. The estimated quantity of random fill is 1,480,000 cy whereas the estimated core material quantity is 674,000 cy. Consideration of these factors suggests limiting the use of the smooth drum vibratory roller to the random fill compaction.

<u>Conclusions and Recommendations</u>. Based on the data review and site inspection the following are concluded and recommended.

1. It has often been stated that the safety of dams involves consideration of more than technical factors. The project organization is one of the most important factors, especially the collaborative relation between engineering and construction and the interaction between the engineer and the

geologist. The attitude and competence of the Adobe Dam project organization are commendable. The site inspection demonstrates that appropriate attention is being given to addressing field conditions as they develop.

2. The design provides defense against unpredictable seepage. On the right abutment the grout curtain, with the modifications proposed by the contractor-geologist, is complemented by the upstream impervious blanket and the pervious downstream drain which can accommodate calculated seepage and seepage flow not amenable to analysis.

3. The bedrock preparation discussed in the field is in accordance with the plans, specifications and Engineering Considerations and is considered satisfactory.

4. The modified drain material gradation provides a higher safety factor against piping than the specified drain material. The specified drain material has a higher permeability and therefore a higher flow capacity under a low hydraulic gradient. If the modified gradation after compaction results in permeabilities which are not significantly lower than the reported test values, the permeability should be adequate. Should the permeability reduce significantly because of added minus No. 10 and minus No. 200 material, the alternative would be to sandwich the specified higher permeability drain material between layers of the finer drain material. Present indications are that this consideration will not be necessary.

5. Where practicable, the drain material should be used on the downstream right abutment as a leveling course to fill

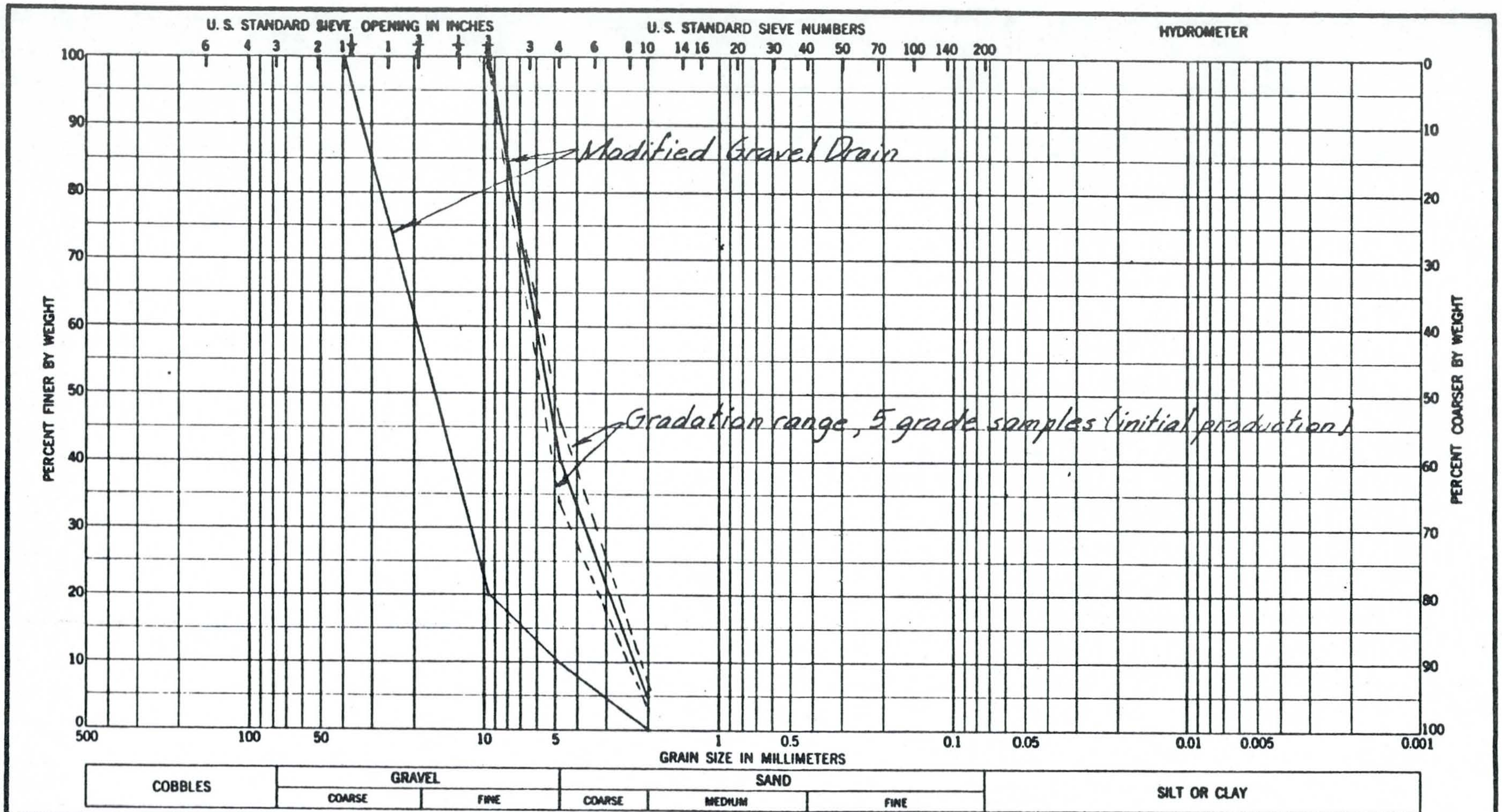
irregularities in the bedrock surface. This would provide direct contact between potential foundation seepage and the drain material. Drain material on the fine side of the gradation band or finer should be used in order to increase the safety factor against piping.

6. Use of vibratory rollers on random material, similar to the subject project random material, has produced satisfactory results. It is recommended that use of the vibratory roller to compact random material be approved.

TABLE 1
ADOBE DAM
GRAVEL DRAIN
PERMEABILITY TESTS

Sample No.	Density pcf	Relative Density	Permeability cm/sec (Ave.)	Remarks
1	105	–	3.91	100% passing 1"
2	"	–	3.24	28-42 minus #4
3	"	–	4.14	4-5 minus #10
4	"	–	4.41	2% minus #200
5	"	–	3.77	
6	"	–	4.12	
A-1 *	91	0	2.98	
A-2 *	98	59	3.01	
A-3 *	102	87	2.74	
A-4	103	96	4.91	plus #10
A-5	"	"	4.46	plus #8
A-6	"	"	4.83	plus 5/32"
A-7	"	"	4.45	plus #10 after test
A-8	105	–	4.40	
A-9	"	–	3.78	
A-10	"	–	4.36	
A *	102	92	2.52	1% passing No. 10
B *	92	–	5.29	0% passing No. 4

All tests are constant head, except where noted (*) tests are falling head



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project *Adobe Dam*
Gravel Drain

Area *Horizontal drain*

~~Boring No.~~ *5 samples from grade*

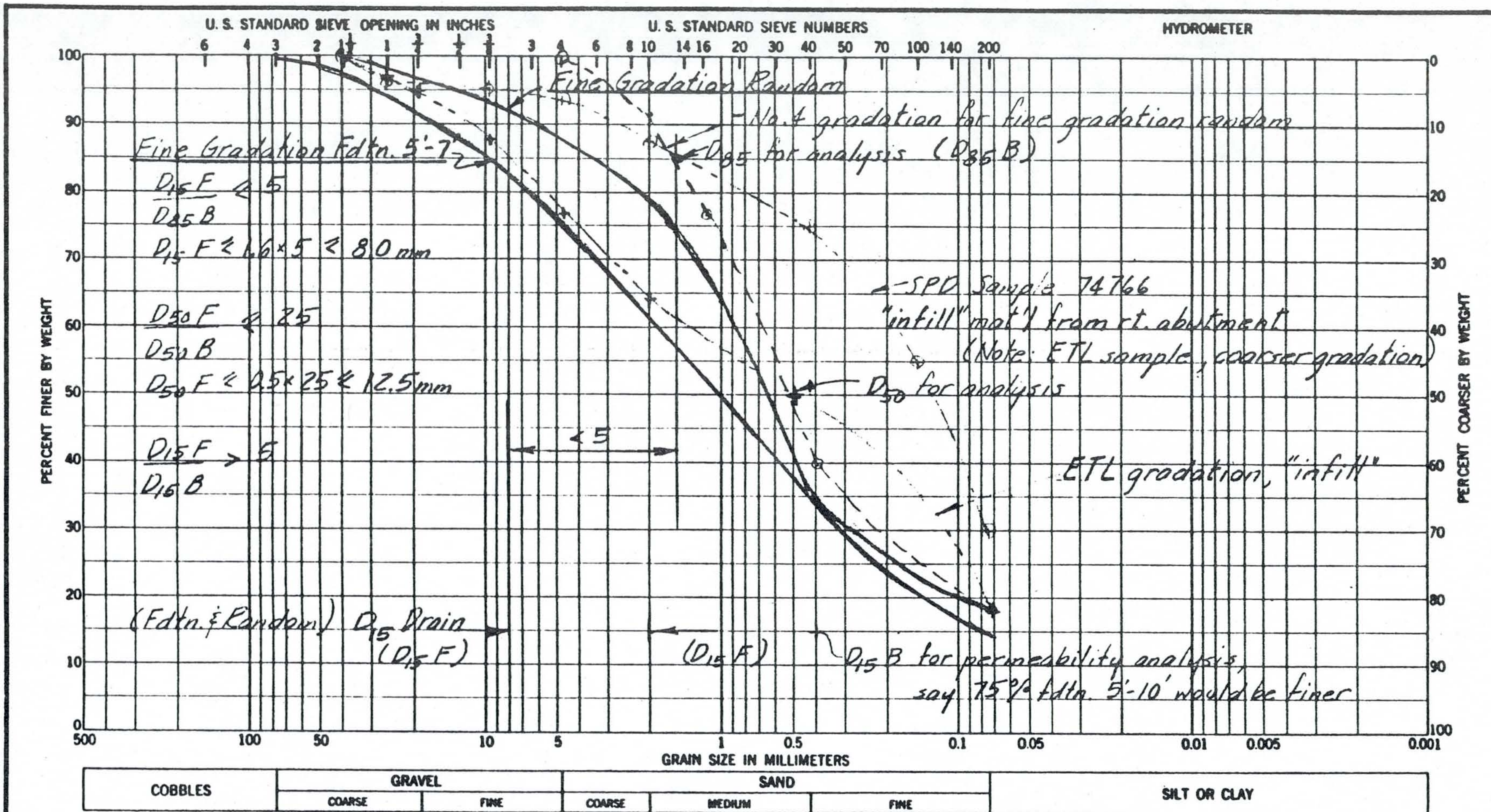
Date *12-16 Mar 81*

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE 1

FIGURE 1



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project: Adobe Dam
Gravel Drain
Filter Criteria

Boring No.

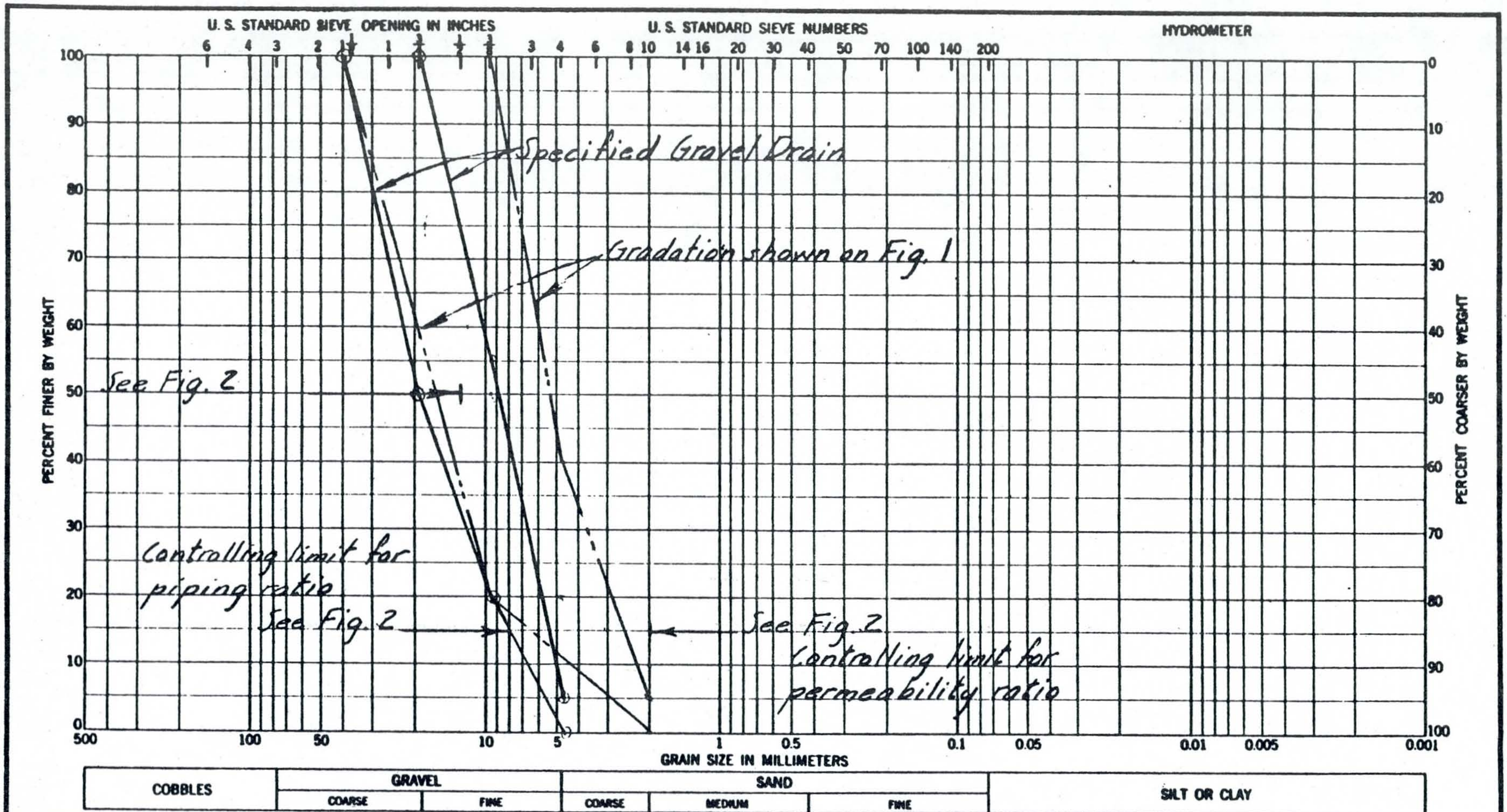
Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE 2

FIGURE 2



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

Project: Adobe Dam Gravel Drain

Area

Boring No.

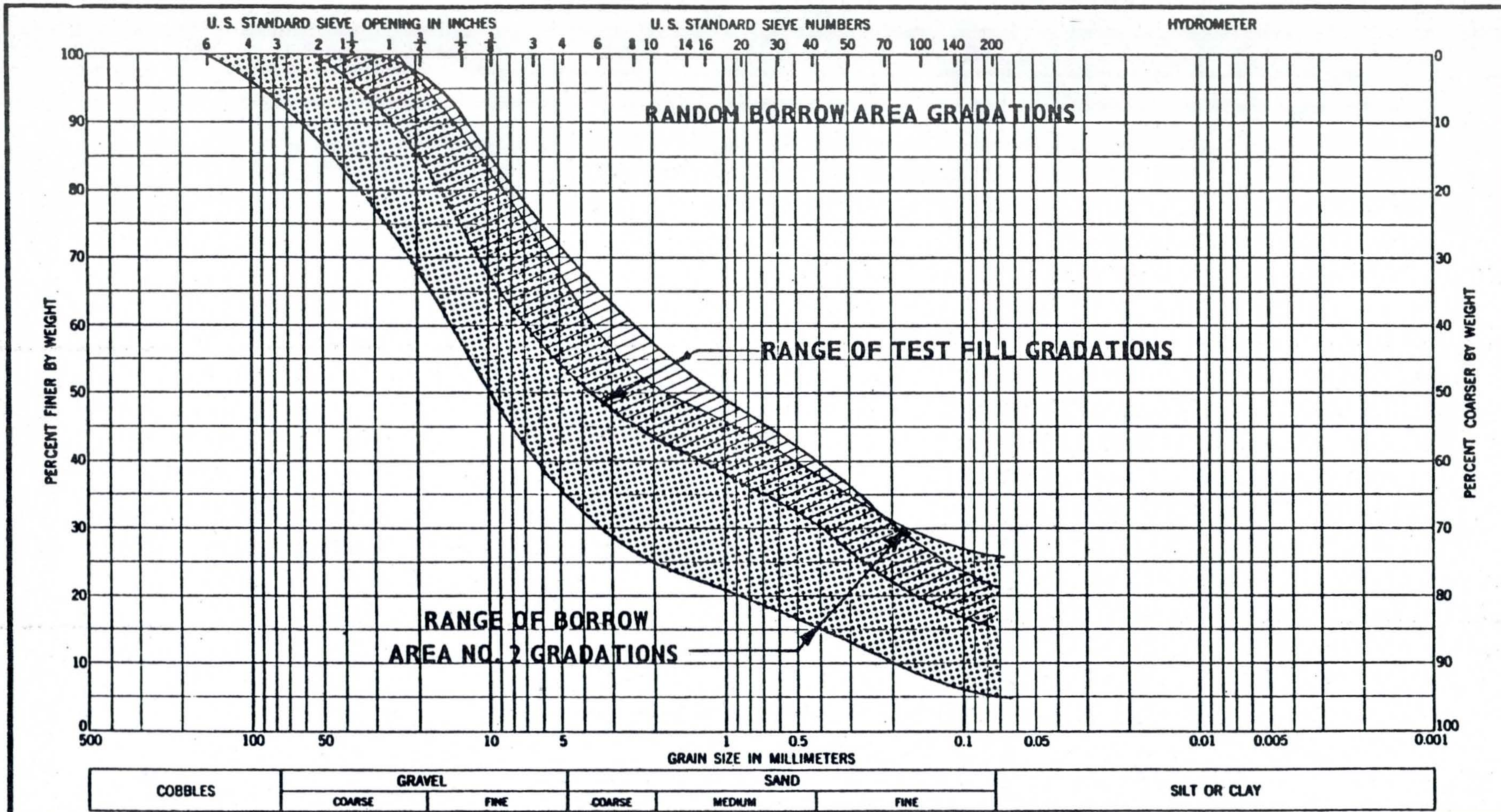
Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

FIGURE 3

FIGURE 3



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI

GRADATION CURVES

Project WARM SPRINGS DAM

RANDOM MATERIAL GRADATION

Area

Boring No.

Date DECEMBER 1977

ENG FORM 1 MAY 63 2087

FIGURE 4

SECTION 20
OF THE CONTRACT
(DRILLING & GROUTING)

ATTACHMENT 5

SECTION 20

DRILLING AND GROUTING

Index

1. APPLICABLE PUBLICATIONS. The publications listed below form a part of this specification to the extent referenced. The pulications are referred to in the text by the basic designation only.

1.1 U.S. Army Corps of Engineers, Handbook for Concrete and Cement.

CRD-C 103-60	Sieve Analysis of Fine and Coarse Aggreate for Use in Portland Cement Concrete
CRD-C 113-68	Moisture Content of Aggregate by Loss in Weight by Drying

1.2 American Society for Testing and Materials (ASTM) Standard.

A 120-73	Black and Hot Dipped Zinc-Coated (Galvanized) Welded and Seamless Steel Pipe for Ordinary Use

1.3 American National Standards Institute (ANSI) Standard.

B 163-1971	Malleable Iron Threaded Fittings, 150 and 300 lb.

2. GENERAL.

2.1 Program. The work contemplated consists of foundation exploratory drilling and grouting the bedrock beneath the abutments of the dam and the outlet works. The approximate locations, limits, and details are indicated on the drawings. The program shown on the drawings and described herein is tentative and is presented for the purpose of canvassing bids. The drilling and grouting shall be done after stripping and cleaning of the abutments and after excavation of the outlet works. The amount of drilling and grouting which actually will be required is unknown, and will be governed by conditions encountered as the work progresses. The Government reserves the right to increase or to eliminate any part of the drilling and grouting program should conditions indicate this as being desirable. Survey controls will be established, prior to drilling for grout holes, to provide reference points to determine the location and elevation of grout holes and to provide control for geologic mapping of the west abutment. The controls shall be established along the centerline of the embankment, from station 9+75 to station 13+70 on 50 foot stations, with elevations at each station.

2.1.1 Area Preparation Prior to Grouting. Prior to grouting operation on the west abutment, including setting grout pipe, all faults and seams within the core zone which might interfere with the grouting shall be cleaned as directed, free from oil, standing or running water, mud, drummy rock, coatings, debris and loose or unsound rock fragments. The area for cleaning will be as indicated below or on the drawings. Starting at elevation 1400± feet down to elevation 1,338 feet the width will be 5 feet downstream of the core zone to 5 feet upstream of the projection of the 1:1.75 upstream slope of the core. Below elevation 1,338, the abutment rock, below streambed, will be similarly cleaned the full width of the foundation trench to elevation 1314.5 feet. All surfaces shall be cleaned thoroughly by air blasting or by other approved methods. Water shall not be used in the cleaning. This area preparation does not relieve the contractor from the responsibility of additional cleaning of the area upon completion of the grouting operation.

2.2 Procedures. The sequence in which the holes are drilled and grouted will be determined in the field and shall be as directed. All grouting mixes, pressures, pumping rates and other specific grouting operations shall be controlled by and in the presence of the Contracting Officer. Survey controls will be necessary in the field for alinement and elevations of grout pipes.

3. EQUIPMENT.

3.1 General. All drilling and grouting equipment used shall be of a type, capacity, and mechanical condition suitable for doing the work as determined by the Contracting Officer. The power, equipment and layout thereof shall meet all applicable reuirements of local, state, and Federal regulations and codes, both safety and otherwise, and the applicable requirements of the SPECIAL PROVISIONS. Sufficient lighting facilities shall be provided and maintained so that all drilling and grouting operations, including calking of grout leaks, can be satisfactorily performed.

3.2 Drilling Equipment. Standard core and grout hole drilling equipment shall be used. The drill rigs used for exploratory core drilling shall be similar or equal to the No. 8 Chicago Pneumatic with hydraulic feed. The drill rights used for grout hole drilling shall be of size equal or greater than a No. 55 Chicago Pneumatic as conditions require. At the option of the Contractor, either coring or non-coring bits may be used for drilling grout holes. "N" size coring bits shall be used for exploratory drilling. A Sprague and Henwood Series "M" core barrel, or equal, shall be used for drilling exploratory holes. Water shall be furnished in sufficient quantity to insure drilling operations will not be delayed by shortages.

3.3 Grouting Equipment. Grout plants shall be capable of supplying, mixing, stirring and pumping grout to the satisfaction of the Contracting Officer. Each plant shall have a minimum capacity of 30 g.p.m. of grout injected at a pressure not greater than 150 psi. Plants shall be maintained in first-class operating condition at all times and any grout hole that is lost or damaged due to mechanical failure of equipment or inadequacy of grout supply shall be replaced by another hole, drilled by the Contractor at his expense. Each grout plant shall include the following minimum equipment.

3.3.1 Two specially equipped, air-driven, single screw type pumps or other approved type.

3.3.2 A mechanically driven grout mixer capable of effectively mixing and stirring the grout and which can be emptied into a sump without interrupting grout pump operations. It shall have a capacity of not less than 3/4 the capacity of the sump. The mixer shall be equipped with a suitable water measuring device calibrated to read in cubic feet and tenths and so designed that after each delivery the hands can be conveniently set back to zero. (Similar and equal to Neptune Disc Meter, Model 106 with 6-inch verical dial).

3.3.3 A mechanically agitated sump, so designed as to be capable of effectively stirring and holding in suspension all solid matter contained in the grout. It shall have a minimum capacity of 16 cubic feet and a means of measuring the quantity contained to 0.1 cubic feet.

3.3.4 A tank for auxiliary water supply to be used in pressure testing, flushing, and pressure washing operations.

3.3.5 Such valves, pressure hose, small tools, and accessories as may be necessary to provide a continuous supply of grout and accurate pressure control. The inside diameter of the delivery line shall be not less than 1-1/2 inches and have a length not more than 200 feet from the sump to the header and be connected and operated such that objectionable blocking of the line will not take place when pumping at the minimum discharge capacity of the pump.

3.3.6 A 100-mesh vibrating screen with a minimum of 576 square inches of screening area for screening grout from the grout hole return line to the sump.

3.3.7 Ground plugs and packers and attachment devices of an approved type for pressure testing and grouting of grout and exploratory holes.

3.3.8 A water meter graduated in gallons per minute for use in pressure testing operations.

3.3.9 Pressure gages graduated to read 0 to 100 psi for use in pressure testing and grouting operations. An accurately calibrated, high precision pressure gage shall be furnished for periodically checking the accuracy of all gages.

3.3.10 A pressure gage graduated to read 0 to 50 psi with no greater than 2 psi divisions on the dial for use in grouting operation at shallow depths.

3.3.11 Sufficient stop cocks and screw caps for capping holes within 50 feet of grouting operations.

3.3.12 A variety of caulking tools with a supply of quick setting mortar, sacked sand, oakum, lead wool, and such other materials that would be useful in sealing grout leaks through open fractures and joints.

3.4 The arrangement of the grouting equipment shall be such as to provide a continuous circulation of grout throughout the system and to permit accurate pressure control by operation of a valve on the grout return line, regardless of how small the grout take may be. Any hole being grouted shall be no farther than 200 feet from the grout pump. The equipment and lines shall be prevented from becoming fouled by the constant cirulation of grout and by the periodic flushing out of the system with water. Flushing shall be done with the grout intake valve closed, the water supply valve open, and the pump running at full speed. Either of the following two distinct arrangements of connecting and operating grout equipment may be required.

3.4.1 Direct grouting shall be done by connecting the grout supply hose to the grout pipe with a "direct grouting header" as shown on the drawings. This method shall be used on clean holes which pressure tests indicate may take a moderate amount of grout.

3.4.2 Packer grouting shall be done by setting a packer in the hole at the depth directed and using a direct grouting header. Several packer settings may be used during one grouting operation.

3.5 Records. The Contracting Officer will keep records of all drilling and grouting operations, such as a log of the exploratory and grout holes, results of washing and pressure testing operations, time of each change of grouting operation, pressure, rate of pumping, amount of cement for each change in water-cement ratio, and other data as deemed by him to be necessary. The Contractor shall furnish all necessary assistance and cooperation to this end.

3.6 Communications. When, for his own convenience, the contractor has the individual elements of his plant so located that communication by normal voice between these elements is not satisfactory, the Contractor shall install a satisfactory means of communication, such as a telephone or other suitable device.

3.7 Lighting. The Contractor shall furnish weatherproof, minimum 300 watts each, light clusters on portable stands, in such quantity and with cables, bulbs, etc., as to provide adequate lighting for all elements of grout operations in the event grouting must be extended into hours of darkness. A power source, furnished by the Contractor, shall be available at all grout operation locations.

3.8 Protection to Work and Cleanup. During grouting operations the Contractor shall take such precautions as may be necessary to prevent drill cuttings, equipment exhaust oil, wash water, and grout, from defacing or damaging any part of the permanent structure. The Contractor will be required to furnish such pumps as may be necessary to care for waste water and grout from his operation. The Contractor shall upon completion of his operations remove all grout pipe above the foundation surface, clean up all waste resulting from his operations that is unsightly or would interfere with the efficient operation of the project as anticipated by the original design. All holes or depressions thus formed shall be patched with a mortar composed of one part Portland cement, two parts sand, and an admixture containing unpolished aluminum powder. The admixture shall consist of one pound unpolished aluminum powder dry mixed with 50 pounds of Portland cement. Five ounces of this mixture shall be added to each sack of Portland cement. The patching shall be done in a neat and workmanlike manner so as to provide a surface smoothness at least equal to undisturbed areas of the concrete surface. Prior to final acceptance of the work, all concrete surfaces shall be cleaned and restored to their original condition, as nearly as practicable as determined by the Contracting Officer.

4. GROUTING MATERIALS.

4.1 Water. Water to be used for drilling, washing, pressure testing and grouting operations shall be fresh, clean, and free from any sediment. Water shall be furnished by the Contractor and shall be supplied in quantities to prevent interruption of any of these operations.

4.2 Cement. Cement used in grout shall by Type II and shall conform to the requirements of section: CONCRETE. Only cement furnished in cloth or paper bags will be accepted for use in the work. A bag (sack) shall contain 94 pounds net. A sufficient quantity of cement shall be stored at or near the site of the work to insure that grouting operations will not be delayed by shortage of cement. In the event the cement is found to contain lumps or foreign matters of a nature and in amounts which, in the opinion of the Contracting Officer, may be deleterious to the grouting operations, such cement shall be removed from the site at no cost to the Government.

4.3 Pipe. All metal pipe and fittings required for constructing grout and exploratory holes, grout hole connections and air vents shall conform to ASTM A 120, standard weight, galvanized. All fittings shall conform to ANSI B 16.3, 150 lb. class.

5. DRILLING AND GROUTING.

5.1 General. All exploratory grout and grout holes shall be drilled at the locations shown on the drawings or as directed. In general, grouting will be started at the lower elevation of the abutment and at the lower part of a section being grouted. Exploratory grout and grout holes may be drilled to depths ranging to 40 feet or as directed. All exploratory grout or grout holes shall be washed, pressure tested and grouted. The location of the first series of holes (primary) shall be determined by conditions in the field. The location of intermediate holes (secondary series) shall be determined by the split-spacing method. The number of grout holes may be increased, progressively, by the split-spacing method, as desired.

5.2 Installing Pipe for Grouting. The Contractor may be required to furnish and install pipe for drilling and grouting as directed. The grout pipes shall be securely anchored into the firm bedrock. The pipes are to be inserted by flaring the bottom of the pipe, or other approved methods, then grouting or caulking it in place with some suitable material to preserve alinement, to prevent entry of foreign materials into the hole, and to prevent leakage. The pipe and fittings shall be cleaned thoroughly of all dirt, grease, oil, grout and mortar immediately before installing. All joints shall be made up snug and the assembly held securely in position and protected from damage or displacement. The Contractor shall take all necessary precautions to prevent any pipe from becoming clogged or obstructed from any cause and any pipe which becomes clogged shall be cleaned out in a manner satisfactory to the Contracing Officer at the Contractor's expense. The presence of tramp metal such as nails, wire, bolts, nuts, and other foreign material in the pipes through which grout holes are to be drilled shall be considered as obstructions.

5.3 Grout Hole Drilling. Grout holes shall be drilled into firm bedrock through pipes embedded in the bedrock. Grout holes shall be drilled with standard rotary non-percussion drilling equipment as hereinbefore designated. No core recovery will be required and the type of bit used shall be optional with the Contractor. The minimum diameter of hole shall be 1-1/2 inches at the point of maximum penetration. No grout hole will be drilled at an angle greater than 45 degrees measured from the vertical. The angle and bearing of the hole shall be measured by a Brunton compass or similar device with a maximum tolerance of 3 degrees. Drilling shall be done in accordance with the procedures hereinafter described. The use of grease, "rod dope" or other lubricant on rotary drill rods will not be permitted except that an approved neutral liquid soap may be added to the drill water. Each hole shall be protected from becoming clogged or obstructed by means of a cap or other suitable device on the collar and any hole that becomes clogged or obstructed before completion of operations shall be cleaned out in a manner satisfactory to the Contracting Officer or another hole provided by and at the expense of the Contractor. Whenever the drill water is lost, or artesian flow is encountered, the drilling operations shall be stopped and the hole grouted before drilling operations are resumed in such hole. The grout within a partially completed hole shall be removed therefrom by washing or other methods before it has set sufficiently to require redrilling. Redrilling required because of the Contractor's failure to clean out a hole before the grout has set shall be performed at the Contractor's expense.

5.4 Definition and Procedure for Drilling and Grouting. Drilling and the grouting shall be done in sections using split spacing, stage grouting method as described herein.

5.4.1 Zone. A zone is a predetermined partial depth of grouting. The first zone extends 20 feet downward from bottom of the grout pipe. The second zone extends 20 feet downward from the bottom of the first zone. In general, all grouting in a given zone and section will be finished before work is started in the next underlying zone. Also, grouting of second zone in any one section shall not begin until the first zone of the adjoining section is completed.

5.4.2 Section. A section is a reach in the grout area, not more than 200 feet in length in which grouting operations will not be permitted at the same time that drilling is in progress. In so far as practicable, the grout area will be subdivided into sections in manner which will facilitate the Contractor's operations. Hole spacing for any one arrangement of holes will be varied in accordance with conditions encountered and as directed.

5.4.3 Stage. A stage is a partial or complete depth of hole within any given zone. The actual depth of a stage depends upon geologic conditions encountered in drilling. It may vary from a fraction to the full depth of the zone, and is marked by the loss or gain of drill water in appreciable amounts or by some other conditions which prevent the hole from being drilled to the full depth of a particular zone.

5.4.4 Split Spacing. Split spacing is the procedure of locating an additional grout hole between two previously drilled and grouted holes.

5.5 Stage Grouting. Stage grouting is a complete cycle of drilling and grouting of any poriton of a hole or all of the hole within a given zone. It involves the placement of a grout curtain by drilling and grouting in successive operations in accordance with the following general procedure.

5.5.1 Primary holes for grouting shall be drilled within the first zone. The depths will be governed by the foundation conditions. Normal spacing of primary holes is 20 feet.

5.5.2 After the grouting of any hole, the grout within the hole shall be removed by washing or by other methods before it has set sufficiently to require redrilling.

5.5.3 After the interval of time as specified in paragraph: STAGE GROUTING PROCEDURES, has elapsed the primary holes not already drilled to the limit of the first zone shall be drilled as directed by the Contracting Officer to additional depths not exceeding the zone limit.

5.5.4 The primary holes thus deepened shall again be washed and pressure tested and then grouted at higher pressures as directed by the Contracting Officer.

5.5.5 The process of successfully drilling primary holes to addtional depths and grouting at higher and higher pressures in stages in all the zones, shall be repeated until all of the primary holes in a section have been completely drilled and grouted as directed by the Contracting Officer.

5.5.6 After the primary holes in all the zones have been completed as specified above, the secondary holes shall be drilled in a manner similar to the primary holes. These and succeeding series of holes are determined by the "split spacing method," and shall be drilled and grouted to the depth of the first zone in like manner until the first zone of that section is completely grouted as directed.

5.5.7 The process of successively drilling to specified depths and grouting at higher pressure in stages for the first series of holes and then for succeeding series of holes shall be repeated for the second zones of that section. Other sections along the grout curtain shall be grouted in like manner until grouting is completed, to the satisfaction of the Contracting Officer. As the drilling and grouting work progresses, it may develop that conditions are such that all or parts of the foundation already grouted require additional grouting. In such event, the equipment shall be returned and additional holes for grouting shall be drilled and grouted as directed by the Contracting Officer and no additional allowance above the contract prices will be made for drilling and grouting such holes or for the expense of any movement of equipment to the performance of such work.

5.6. Washing and Pressure Testing. Immediately upon completion of drilling each grout hole the hole shall be thoroughly washed with clean water with water pumps capable of pumping 50 gallons per minute. All drill cuttings, intersecting rock seams and crevices containing clay or other washable materials shall be washed by circulating water from the bottom of the hole to the top with the maximum pump capacity. The circulation shall continue until the water is clear or for a maximum of 10 minutes. No separate payment will be made for washing. Immediately before pressure grouting of any hole, the hole shall be pressure tested starting with low pressures and increasing the pressure to successively higher rates until the maximum pressure, for the increment being tested, has been reached. Maximum pressure for testing shall be the same as the maximum grouting pressure indicated in 5.8.2. Where no pressure can be built up, the zone shall be tested at the maximum pumping capacity for 5 minutes.

5.7. Stage Grouting Procedures.

5.7.1. First Stage. The Contractor shall perform the first stage or low-pressure grouting, by washing and grouting holes at locations indicated on the drawings or as directed. Before grouting is begun in any hole of a given series in any section, at least the two adjacent holes in advance of each such hole shall be completely drilled for the same stage and the adjacent hole completely washed and pressure tested to facilitate washing and flushing out of any intervening clay-filled seams, fractures, or solution openings.

5.7.2. Second Stage. After all first stage grouting in any section has been completed, as specified above, the Contractor shall proceed, when so directed, with second stage drilling and grouting in accordance with the procedure outlined herein but in no case shall the deepening of any hole preparatory to grouting be commenced before a minimum period of 24 hours has elapsed since completion of the previous stage-grouting at the hole, nor shall second stage grouting be conducted within a distance of approximately 50 feet of any hole in which a previous stage of grouting has been completed until the grout in such previous stage holes has set for a period of 24 hours. Grouting at subsequent stages shall conform to the same requirements as to minimum time and distance.

5.8. Grouting. All pressure grouting operations shall be performed in the presence of the Contracting Officer, and shall be in accordance with the following general procedure.

5.8.1. Grout Mixes. Mixes shall be in the proportion directed by the Contracting Officer who will, from time to time, direct changes to suit the conditions in the particular grout hole as revealed by the drilling and grouting operations. The water cement ratio by volume will be varied to meet the characteristics of each hole and will range between 0.6 to 6.0, the greater part of grout probably being placed at a ratio of about 2.0 to 4.0. The grout shall be neat cement consisting of cement and water.

5.8.2. Grouting Pressures. Grouting pressures to be used in the work will vary with conditions encountered in the respective holes and the pressures used shall be as directed by the Contracting Officer. It is anticipated that pump pressures will range from 0 to 40 psi but in no event will pressure in excess of 50 psi be required, based on grouting pressure of 1.0 psi per foot of depth.

5.8.3. Grout Injection. In general, if pressure tests indicate a tight hole, grouting shall be started with a thin mix of approximately 6:1. If an open hole condition exists, as determined by loss of drill water or inability to build up pressure during washing operations, then grouting shall be started with a thicker mix and with the grout pump operating as nearly as practicable at constant speed at all times; the ratio will be decreased, if necessary, until the required pressure has been reached. When the pressure tends to rise too high, the water-cement ratio shall be increased as may be required to produce the desired results. If necessary to relieve premature stoppage, periodic applications of water under pressure shall be made. Under no conditions shall the pressure or rate of pumping be increased suddenly, as either may produce a water-hammer effect which may promote stoppage. The grouting of any hole shall not be considered completed until that hole refuses to take any grout whatsoever at 3/4 the maximum pressure required for the stage. Should grout leaks develop and grout escape through open fractures or grout pipes, plugging, caulking, or capping will be required as directed. In general, surface grout leaks through open fractures shall be sealed with a thick mortar containing a quick setting agent. At times, however, oakum, lead wool or other such materials may be used. Grout escaping through adjacent grout holes shall be capped and before the grout has set, the grout pump shall be connected to each of these holes and the grouting completed at the pressures specified as directed. Once grouting has been started it shall be continued to completion without any interruption whatsoever. If, due to size and continuity of fracture, it is found impossible to reach the required pressure after pumping a reasonable volume of grout at the minimum workable water-cement ratio, the speed of the pumping shall be reduced. Following such reduction in pumping speed, if the desired result is not obtained, grouting in the hole shall be discontinued when directed by the Contracting Officer. In such event, the hole shall be cleaned, the grout allowed to set, and additional drilling and grouting shall then be done in this hole or in the adjacent area as directed, until the desired resistance is built up. After grouting of any hole, the pressure shall be maintained by means of a stopcock or other suitable device until the grout has set to the extent that it will be retained in the hole. Grout that cannot be placed, for any reason within 2 hours after mixing shall be wasted. If such grout is mixed at the direction of the Contracting Officer or with his knowledge or consent, such wasted grout (except grout or the material constituents thereof, wasted due to the improper anchorage of the grout pipe or connections, negligence on the part of the Contractor, or improper mixing) will be paid for at the contract prices for the materials contained therein. After grout has set in completed holes, the hole shall be kept filled with grout to ground surface by hand pouring. Any holes in which the grouting is omitted shall be filled with grout to ground surface by hand pouring. Any grout holes lost due to insufficient cement or other materials shall be replaced by the Contractor at no additional cost to the Government.

5.9. Exploratory Grout Hole Drilling.

5.9.1. General. Explorary holes shall be drilled in order to explore the foundation material and to check the penetration of the grout. Exploratory holes shall be located by the Contracting Officer. The core holes shall be drilled to the depths and angles directed and the core diameter shall be minimum 2-1/8-inches diameter. The drilling shall be done in a workmanlike manner and, in order to insure maximum core recovery, the coring equipment used shall be similar in construction and equal in performance to Sprague and Henwood "M" series equipment. Exploratory holes shall be washed to their full depth, pressure tested, and pressure grouted as specified.

5.9.2. Core Boxes shall be well constructed of new, seasoned fir and plywood and shall be of such length, width, and height that four 4-foot cores may be placed in each box, separated by firmly fastened wooden strips and equipped with at least six screws each. Standard hardware hooks, 2 inches in length, and eyes will be attached to the covers opposite the hinges so the covers can be firmly hooked to the box. Each box shall contain sufficient loose wooden blocks, 2-1/8 x 2-1/8 x 5/8 inches, to be used as marked for the bottom end of each core run. Details of core boxes to be used in the work shall be submitted for approval prior to their construction. Core boxes shall be ready for use well in advance of their actual need so that at no time will it be considered necessary to employ makeshift methods in storing the core samples. The Contractor shall furnish sufficient core boxes to accommodate all cores recovered. A separate core box or group of boxes shall be provided for each core hole drilled. All core boxes shall be constructed for NX size cores.

5.9.3. Procedure. The drilling shall be done in a workmanlike manner and all necessary precautions shall be taken to secure the best possible cores. The core barrel shall be pulled as soon as the advance is equal to core capacity of the core barrel or as often as is necessary to prevent injury to the cores whichever is shorter. As soon as any core block occurs, the core barrel shall be pulled. The core shall then be removed, placed in a wooden V-trough, washed and placed in proper used for separating the core runs. As each box is filled with the core samples, it shall be suitably and legibly marked with project name, hole number, depth, and other pertinent information as directed. No box shall contain cores from more than one hole. The boxes shall be delivered in the vicinity of the work, as directed. Exploratory holes may be grouted under pressure, if conditions so indicate, but in all cases the holes will be grouted to full depth.

6. QUALITY CONTROL. In addition to records kept by the Contracting Officer the Contractor shall establish and maintain quality control for all drilling and grouting to assure compliance with contract requirements and maintain records of his quality control for all operations including but not limited to the following:

(a) Drilling of exploratory grout and grout holes. Records of drilling should provide plan showing location of all holes and information on type of drilling equipment used, depths of drilling, drilling rates, washing and pressure testing and depths of water loss.

(b) Grouting. Records should be kept of type of grouting plant used, including all grouting equipemnt, grouting depths, grouting mixes, and amount of cement for each change in water cement ratio, pressures and pumping rates, ambient temperature, and grout takes and wastage, and time of each change of grouting operation.

6.1 Records and Tests. A copy of these records and tests as well as the records of corrective action taken, will be furnished the Government as required in paragraph: CONTRACTOR QUALITY CONTROL of section: GENERAL REQUIREMENTS.

* * * * *

FINAL CONTRACT QUANTITIES

ATTACHMENT 6

MISCELLANEOUS OBLIGATION DOCUMENT

For use of this form, see AR 37-108; the proponent agency is Office of the Comptroller of the Army.

Use reverse side for continuation of partial payment record.	MISCELLANEOUS OBLIGATION DOCU NR	DATE PREPARED 22 APR 1982	
INSTALLATION Adobe Dam Phoenix,Arizona	ACCOUNTING CLASSIFICATION 96X3122 Const Gen. C of E Civil BE0640410A		
PREPARED BY (Signature and title) Paul N Dunn, CPT, CE Project Engineer	APPROVED BY Paul W. Taylor COL, CE Contracting Officer		

DESCRIPTION	AMOUNT	
Final Quantity Decrease:	-$98,258	21

PARTIAL PAYMENT RECORD	NAME OF VENDOR OR CONTRACTOR	CONTRACT OR PURCHASE ORDER NUMBER
	M.M. Sundt Construction Co.	DACW09-80-C-0121

DATE 1	DESCRIPTION 2	OBLIGATION 3	ACCRUED EXPENDITURE 4	ACCOUNTS PAYABLE BALANCE 5	DISBURSEMENT 6	(Check appropriate box) ☐ UNLIQUIDATED ☐ UNDELIVERED BALANCE 7
B.I.# 6(A)	Scaling First 300 CY	300 CY	120 CY	- 180 CY	.977X$ 17.00	-$ 2,989.62
B.I.# 6(B)	Scaling over 300 CY	200 CY	0 CY	- 200 CY	.977X$ 15.00	-$ 2,931.00
B.I.# 7	Excavation, Abutment	36,000 CY	33,232 CY	- 2,768 CY	.977X$ 4.00	-$10,817.34
B.I.# 8	Excavation, Foundation	512,000 CY	485,723 CY	-26,277 CY	.977X$ 1.00	-$25,672.63
B.I.# 9	Excavation, Exploration Trench	100,000 CY	98,969 CY	- 1,031 CY	.977X$ 1.00	-$ 1,007.28
B.I.# 10(B)	Excavation, Dental over 100 CY	5,400 CY	5,717 CY	317 CY	.977X$ 60.00	+$18,582.54
B.I.# 11	Excavation, Outlet Works	12,102 CY	12,110 CY	8 CY	.977X$ 6.00	+$ 46.90
B.I.# 12	Excavation, Spillway	233,000 CY	236,937 CY	3,937 CY	.977X$ 2.50	+$ 9,616.12
B.I.# 13	Excavation, Channel	23,000 CY	24,521 CY	1,521 CY	.977X$ 1.50	+$ 2,229.03
B.I.# 15	Excavation, Toe	1,920 CY	1,928 CY	8 CY	.977X$ 2.50	+$ 19.54
B.I.# 16	Fill, Outlet Works	16,000 CY	17,900 CY	1,900 CY	.977X$ 3.00	+$ 5,568.90
B.I.# 19	Fill, Toe	37,400 CY	46,260 CY	8,860 CY	.977X$ 1.25	+$10,820.28

INSTALLATION	ACCOUNTING CLASSIFICATION
Adobe Dam Phoenix, Arizona	96X3122 Const Gen C of E , Civil BE0640410A

PARTIAL PAYMENT RECORD (Continued)	NAME OF VENDOR OR CONTRACTOR	CONTRACT OR PURCHASE ORDER NUMBER
	M.M. Sundt Construction Co	DACW09-80-C0121

DATE 1	DESCRIPTION 2	OBLIGATION 3	ACCRUED EXPENDITURE 4	ACCOUNTS PAYABLE BALANCE 5	DISBURSEMENTS 6	(Check appropriate box) ☐ UNLIQUIDATED ☐ UNDELIVERED BALANCE 7
B.I.#22	Embankment, Core	674,000 CY	687,853 CY	13,853 CY	.977X$ 1.00	+$ 13,534.38
B.I.#23	Embankment, Random	1,480,000 CY	1,465,000 CY	-15,000 CY	.977X$ .90	-$ 13,189.50
B.I.#24	Gravel Drain	142,000 CY	149,589 CY	7,589 CY	.977X$ 8.00	+$ 59,315.62
B.I.#25	Additional Rolling	500 HR	0 HR	-500 HR	.977X$100.00	-$ 48,850.00
B.I.#26	Bedding Material	33,450 CY	31,900 CY	-1,550 CY	.977X$ 8.00	-$ 12,114.80
B.I.#27	Filter Material	4,000 CY	3,659 CY	-341 CY	.977X$ 8.00	-$ 2,665.26
B.I.#28	Stone, Type I	97,400 TN	88,830 TN	-8,570 TN	.977X$ 6.00	-$ 50,237.34
B.I.#29	Stone, Type II	67,000 TN	35,494 TN	-31,506 TN	.977X$ 6.00	-$184,688.52
B.I.#31	Grouting, Stonework	350 CY	274 CY	-76 CY	.977X$ 75.00	-$ 5,568.90
B.I.#33	Concrete, Outlet Channel Sill	22 CY	25 CY	3 CY	.977X$100.00	+$ 293.10
B.I.#34	Concrete, Spillway Sill	43 CY	96 CY	53 CY	.977X$125.00	+$ 6,472.62
B.I.#37(B)	Concrete, Plug (Lean Mix)	325 CY	300 CY	-25 CY	.977X$ 75.00	-$ 1,831.88
B.I.#38(B)	Dental Concrete	940 CY	910 CY	-30 CY	.977X$ 90.00	-$ 2,637.90
B.I.#39(A)	Grout, Slurry	100 CF	34 CF	-66 CF	.977X$ 6.00	-$ 386.89
B.I.#39(B)	Grout, Slurry	100 CF	0 CF	-100 CF	.977X$ 5.00	-$ 488.50
B.I.#40(A)	Portland Cement	5,850 CWT	6,640 CWT	790 CWT	.977X$ 4.50	+$ 3,473.24
B.I.# 43(B)	Drilling Exploratory Grout Holes	100 LF	57 LF	-43 LF	.977X$ 45.00	-$ 1,890.50

☆ U.S. G.P.O. 1972-769272/227

FOR USE OF THIS FORM, SEE AR 37-108; THE PROPONENT AGENCY IS OFFICE OF THE COMPTROLLER OF THE ARMY.

of partial payment record. | 22 APR 1982

INSTALLATION
Adobe Dam
Phoenix,Arizona

ACCOUNTING CLASSIFICATION
96X3122 Const Gen C of E, Civil BE0641410A

PREPARED BY (Signature and title)
Paul N Dunn, CPT, CE
Project Engineer

APPROVED BY
Paul W. Taylor
COL, CE
Contracting Officer

DESCRIPTION	AMOUNT
Final Quantity Decrease:	

PARTIAL PAYMENT RECORD	NAME OF VENDOR OR CONTRACTOR	CONTRACT OR PURCHASE ORDER NUMBER
	M.M. Sundt Construction Co.	DACW09-80-C-0121

DATE 1	DESCRIPTION 2	OBLIGATION 3	ACCRUED EXPENDITURE 4	ACCOUNTS PAYABLE BALANCE 5	DISBURSEMENT 6	(Check appropriate box) ☐ UNLIQUIDATED ☐ UNDELIVERED BALANCE 7
B.I.# 43(C)	Drilling Grout Holes	975 LF	1,761 LF	786 LF	.977X $ 15.00	+$ 11,518.83
B.I.# 43(D)	Pipe for Grout Holes	130 LF	150 LF	20 LF	.977X $ 3.00	+$ 58.62
B.I.# 43(E)	Drill Set-Ups	70 EA	124 EA	54 EA	.977X $ 50.00	+$ 2,637.90
B.I.# 43(F)	Washing & Pressure Testing	50 HR	26.5 HR	– 23.5 HR	.977X $120.00	-$ 2,755.13
B.I.# 43(G)	Grout Pump Connections	70 EA	103 EA	33 EA	.977X $ 35.00	+$ 1,128.44
B.I.# 43(H)	Placing Grout	864 SA	4,744 SA	3,880 SA	.977X # 30.00	+$113,722.80
B.I.# 48	Aggregate Base, Access Rd	1,650 CY	1,806 CY	156 CY	.977X $ 12.00	+$ 1,828.94
B.I.# 50	Asphalt Concrete Pavement	1,860 TN	1,954 TN	94 TN	.977X $ 30.00	+$ 2,755.14
B.I.# 52	Asphalt Used in BST	25 TN	35.1 TN	10.1 TN	.977X $250.00	+$ 2,466.92
B.I.# 53	Aggregate Used in BST	260 TN	305 TN	45 TN	.977X $ 20.00	+$ 879.30
B.I.# 55	Guardrail	297 LF	336.5 LF	39.5 LF	.977X $ 15.00	+$ 578.87
B.I.# 56	Log Barrier	174 LF	147 LF	-27 LF	.977X $ 20.00	-$ 527.58

DA FORM 1 MAY 71 3717 REPLACES DA FORM 14-84, 1 OCT 56, WHICH IS OBSOLETE.

INSTALLATION	ACCOUNTING CLASSIFICATION
Adobe Dam Phoenix, Arizona	96X3122 Const Gen C of E, Civil BE0640410A

PARTIAL PAYMENT RECORD *(Continued)*	NAME OF VENDOR OR CONTRACTOR	CONTRACT OR PURCHASE ORDER NUMBER
	M. M. Sundt Construction Co.	DACW09-80-C-0121

DATE 1	DESCRIPTION 2	OBLIGATION 3	ACCRUED EXPENDITURE 4	ACCOUNTS PAYABLE BALANCE 5	DISBURSEMENTS 6	(Check appropriate box) ☐ UNLIQUIDATED ☐ UNDELIVERED BALANCE 7
B.I. #63	Desert Varnish Finish	9.120 SF	27,695 SF	18.575 SF	.977 X $ .30	+$ 5,444.33

☆ GPO: 1971 431-070/111-

Sheet 4

MISCELLANEOUS OBLIGATION DOCUMENT

FOR USE OF THIS FORM, SEE AR 37-108; THE PROPONENT AGENCY IS OFFICE OF THE COMPTROLLER OF THE ARMY.

Use reverse side for continuation of partial payment record.

INSTALLATION Adobe Dam Phoenix, Arizona	ACCOUNTING CLASSIFICATION 96X8862 Contributed Funds, Other FW 532 0410A	MISCELLANEOUS OBLIGATION DOCU NR	DATE PREPARED 15 March 1982
PREPARED BY (Signature and title) Paul N. Dunn PAUL N. DUNN CPT, CE Project Engineer	APPROVED BY PAUL W. TAYLOR, COL, CE Contracting Officer		

DESCRIPTION	AMOUNT	
Final Quantity Increase	$2,931	15

PARTIAL PAYMENT RECORD	NAME OF VENDOR OR CONTRACTOR	CONTRACT OR PURCHASE ORDER NUMBER
	M. M. Sundt Construction Co.	DACW09-80-C-0121

DATE 1	DESCRIPTION 2	OBLIGATION 3	ACCRUED EXPENDITURE 4	ACCOUNTS PAYABLE BALANCE 5	DISBURSEMENT 6	(Check appropriate box) ☐ UNLIQUIDATED ☐ UNDELIVERED BALANCE 7
BI # 18	Fill, 35th Avenue	91,000 CY	93,043 CY	+2,043 CY	$ 1.20	+$2,553.75
BI # 30A	Stone, Type III, 35th Ave	4,400 CY	4,234 TN	- 166 TN	$ 6.00	-$ 996.00
BI # 49	Aggregate Base, 35th Ave	1,870 CY	1,993 CY	+ 123 CY	$12.00	+$1,476.00
BI # 51	Asphalt Concrete Pavement	786 TN	781 TN	- 5 TN	$30.00	-$ 150.00
BI # 54	Preservative Seal	6,980 SY	7,138 SY	+ 158 SY	$ 0.30	+$ 47.40
					TOTAL	+$2,931.15

DA FORM 1 MAY 71 3717 REPLACES DA FORM 14-84, 1 OCT 58, WHICH IS OBSOLETE

Sheet 5