US Army Corps
of Engineers
Los Angeles District

Gila River Basin

Phoenix, Arizona, and Vicinity
(Including New River)

New River Dam

Embankment Criteria And Performance Report

February 1987

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

U.S.ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
FEBRUARY 1987

DEPARTMENT OF THE ARMY
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
P. O. BOX 2711
LOS ANGELES, CALIFORNIA 90053

REPLY TO
ATTENTION OF:

January 7, 1988

Office of The Chief
Engineering Division

Mr. D.E. Sagramoso
Chief Engineer and General Manager
Flood Control District of Maricopa County
3335 West Durango Street
Phoenix, Arizona 85009

Dear Mr. Sagramoso:

Enclosed for your use and information are one copy of the Embankment Criteria and Performance Report and one copy of the Foundation Report for New River Dam, dated February 1987 and October 1985.

Sincerely,

Robert C. Koplin

Carl F. Enson
Chief, Engineering Division

Enclosures

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY
(INCLUDING NEW RIVER)

NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT
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ERRATA

1. Page 31, paragraph 10.05 replace the last two sentences with "The reanalysis indicates that the as-constructed embankment requires a 1.6-foot thick gravel drain to control seepage. A 2-foot thick gravel drain was constructed; therefore the as-constructed gravel drain is capable of handling the as-constructed seepage quantities."

2. Page 36, paragraph 14.04 replace the first sentence with "The number of passes required to compact the core material with a tamping roller and the transition and pervious shell materials with a vibratory roller to each material design density was reduced from 8 to 4 passes based on the results of the verification and demonstration fills."

3. Page 36, paragraph 14.04, line 5 change "90" to "95" and "1557" to "698".

4. Page 36 add the following paragraph:

14.06 The requirements for the downstream slope protection are independent of the requirements for the upstream slope protection because they are being placed for different purposes. The downstream slope protection is placed to prevent erosion of that slope due to the runoff from rainfall while the upstream slope protection is placed to prevent erosion of the upstream slope due to waves. Incorporating erosion and landscape requirements into one layer of stone that satisfies both requirements is recommended; therefore, reducing the quantity of stone and the cost of the project.

Encl

NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

CONTENTS

CONTENTS (Continued)

CONTENTS (Continued)

TABLES

FIGURES

PLATES

PLATES (Continued)

LIST OF PHOTOS

PHOTOS

ATTACHMENTS

APPENDIXES

NEW RIVER DAM
PERTINENT DATA

Contact number: DACW09-83-C-0050
Awarded on: 31 August 1983
Completed on: 8 February 1985
Contractor: M.M. Sundt Construction Company. Inc

Feature Description	Unit	Data
Drainage Area	sq. mi	164
Type of dam	--	compacted earthfill
Main embankment:		
Crest elevation	ft, NGVD	1486.7
Maximum height above streambed	ft	104
Crest length	ft	2320
Freeboard	ft	5.6
Dike No. 1:		
Crest elevation	ft, NGVD	1486.3
Crest length	ft	7464
Freeboard	ft	5.2
Maximum height	ft	36
Dike No. 2:		
Crest elevation	ft, NGVD	1484.0
Crest length	ft	256
Freeboard	ft	2.9
Maximum height	ft	9
Spillway (Detached):		
Crest elevation	ft, NGVD	1456.2
Crest width	ft	75
Max. water surface elevation	ft, NGVD	1481.1
Max. spillway outflow	CFS	29,850
Outlet conduit (Ungated):		
Interior dimension	ft	9.5 H x 6.25 W
Length	ft	433
Inlet elevation	ft, NGVD	1389.25
Outlet elevation	ft, NGVD	1386.31
Max. outlet outflow	CFS	3150
Energy dissipator:		
Length	ft	60.98
Width	ft	31.0
Floor elevation	ft, NGVD	1372.0
Wall height	ft	22.0

NEW RIVER DAM
PERTINENT DATA (Continued)

Feature Description	Unit	Data
Outlet channel:		
Base width	ft	16.0
Sideslope	--	2.5 H to 1V
Levee height	ft	1.0 - 8.0
Length	ft	730.32
Reservoir area:		
Spillway crest	acres	1780
Max. water surface	acres	2900
Capacity (gross):		
Spillway crest	acre ft	43,520
Max. water surface	acre ft	102,520
Storage allocation below spillway crest:		
Flood Control (net)	acre-ft	38,600
Sedimentation	acre ft	4920
Standard project flood:		
Total volume	acre ft	49,300
Peak inflow	CFS	45,000
Drawdown time (to empty)	days	10.1
Probable maximum flood:		
Total volume	acre ft	105,000
Peak inflow	CFS	144,000
Peak outflow	CFS	33,000
Drawdown time (to spillway crest)	days	3.3

NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

I. PURPOSE AND SCOPE

1.01 The purpose of this report is to provide in one volume the significant information on the design, specification requirements, and construction of the embankment and appurtenances. The report can be used to provide information about the design and construction of the embankment for engineers unfamiliar with the project, for re-evaluation of the embankment in the future, if required, for periodic inspection reports, and background data for design and construction of similar projects.

1.02 The report summarizes embankment features, design data, construction control data, and record test results. Construction equipment, construction procedures, significant construction modifications and changes, and notes are presented. Also, the embankments are re-evaluated using the design parameters developed from laboratory test results of record samples obtained during construction.

II. REFERENCES

2.01 "Gila River Basin, Phoenix, Arizona and vicinity (including New River), New River Dam (including New River to Skunk Creek), Design Memorandum No. 3, General Design Memorandum Phase II, Project Design Part 3," dated November 1982.

2.02 Contract drawings "Gila river Basin, Phoenix, Arizona and vicinity (including New River), New River Dam, Maricopa County, Arizona," dated June 1983.

2.03 Specifications No. DACW09-83-B-0016, Gila River Basin, Phoenix, Arizona and vicinity (including New River), "New River Dam, Maricopa County, Arizona," dated August 1983, including amendments 1 through 4.

2.04 "Gila River Basin, Phoenix, Arizona and vicinity, (including New River), New River Dam, Foundation Report", dated October 1985.

III. GENERAL

AUTHORITY

3.01 New River Dam was authorized by the Flood Control Act of 1965, Public Law 89-298, 89th Congress. Authority for preparation of Embankment Criteria and Performance Report for New River Dam is contained in ER 1110-2-1901, dated 31 December 1981.

PROJECT PURPOSE

3.02 New River Dam is a part of the Phoenix, Arizona and vicinity (including New River), Flood Control Project. The dam functions as a detention basin to provide flood control along New River. The detention basin reduces the standard project flood peak from an inflow of 45,000 CFS to an outflow of 2665 CFS.

PROJECT LOCATION

3.03 New River Dam is located on New River in Maricopa County, Arizona, approximately 22 miles northwest of downtown Phoenix and 6 miles west of Interstate Highway I-17, see plate 1. The dam is located on New River approximately 4.5 miles south of Carefree Highway and spans a narrow valley between East and West Wing Mountains.

PROJECT DESCRIPTION

3.04 The project consists of the following features:

a. A zoned earthfill dam approximately 104 feet high (maximum) above streambed and approximately 2320 feet long at the crest. The crest elevation is 1486.7 feet (without settlement allowances) above National Geodetic Vertical Datum (NGVD). (Photos 1, 2, and 3).

b. A zoned earthfill dike (dike no. 1) approximately 36 feet high (maximum) and approximately 7475 feet long at the crest, about 1.7 miles northwest of the right abutment of the dam. The crest elevation is 1486.0 feet above NGVD (without settlement allowances). (Photos 4 and 5).

c. Another earthfill dike (dike no. 2) approximately 9 feet high and approximately 256 feet long at the crest, about 1/2 mile northeast of the left abutment of the dam. The crest elevation is 1484.0 feet above NGVD (without settlement allowances). (Photo 6).

d. An ungated outlet, 6.25 feet wide by 9.5 feet high, near the base of the left abutment. (Photos 7, 8, 9, and 10).

e. A detached unlined spillway, 75 feet wide at the base, about 700 feet northwest of the right abutment of the dam. (Photo 11). A general plan of the project is shown on plate 1.

PROJECT HISTORY AND AUTHORIZATION

3.05 The Phoenix, Arizona and Vicinity (including New River) Flood Control Project was authorized by the Flood Control Act of 1965 (Public Law No. 89-298, 89th Congress) to help alleviate the flood hazard that exists in the Phoenix metropolitan area. Post-authorization studies were initiated in the spring of 1969 and during Phase I studies, a combination structural-nonstructural plan was determined to be the best solution to the flood problem in the area. The approved plan, which differs from the authorized plan, involves the construction of four earthfill dams (Dreamy Draw Dam, completed in 1973; Cave Buttes Dam, completed in 1980; Adobe Dam, completed in 1982; and New River Dam, completed in 1985) and the Arizona Canal Diversion Channel presently under construction.

3.06 Two alternative damsites were geotechnically investigated and evaluated for the New River project. The first site considered was explored between March 1970 and January 1972 and is discussed in the Phase I GDM, dated March 1976. This site, originally designated the interim report site was renamed the Phase I site in the Phase II GDM. The second and eventual construction site, originally designated alternative site no. 1, but subsequently referred to as the Phase II site, was located about 1500 feet downstream from the Phase I site. This site was explored in detail for the Phase I GDM between November 1979 and May 1981. Pre-construction explorations done to augment the design studies were made in June 1982. This additional work, although not included in the GDM, was included in the contract plans. Geotechnical considerations had no significant influence on site selection. The rationale for selecting the Phase II site was made on the basis of economics. All explorations pertinent to the foundation and excavation studies, including those made in 1984 during construction, are presented in this report.

3.07 The New River Dam construction contract was advertised under bid reference No. DACW09-83-B-0016. Bids were opened on 2 August 1983 and M. M. Sundt Construction Company of Tucson, Arizona was awarded Contract No. DACW09-83-C-0050 on 31 August 1983 with a total bid of $10,250,000.00. The remaining bids ranged from $10,340,000.00 to $18,497,400.00. The Government estimate was $12,331,105.00. Construction began in October 1983 and the dam was officially dedicated on 8 February 1985, approximately 6 months in advance of the required completion date. The final contract cost was $11,749,344.60; the increase due primarily to additional processing required to obtain adequate and acceptable type I stone protection and to obtain suitable landscape stone, additional quantities for stage 1 excavation, west abutment excavation, foundation preparation, outlet works excavation and gutter, and the addition of three observation wells.

Subcontractors used by M. M. Sundt to perform work relative to the construction of the embankment were as follows:

a. W. G. Jaques Co., Des Moines, Iowa--drilling and grouting subcontractor.

b. Western Technology, Inc., Phoenix, Arizona--material testing.

c. Brooks Hersey, Tempe, Arizona--project surveyors.

d. United Metro Division, Phoenix, Arizona--concrete subcontractor.

CONSTRUCTION AND DESIGN STAFF

3.08 Key Corps of Engineers personnel involved in the design and construction of New River Dam are listed below:

a. Engineering Division

Project Manager	Stan Lutz
Civil Design	Yan Bahaudin
Technical Specialist	Albert Honda
Soils Design	Abbas Roodsari
Soils Design	Ted Ingersoll
Geology	Vernon Minor
Geology	Robert Thurman
Materials and Investigations	William Halczak
Hydraulics	Edward Chew
Environmental Planning	Lynn Almer
Cultural Resources	Helen Wells
Landscape Architecture	Mike Evasovic
Survey	Harlan Anderson

b. Construction Division

Resident Engineer	Neil Erwin
Project Engineer	Cpt. Bob Dunne
Office Engineer	Michael Ternak
Office Technician	Satsuki Carrington
Assistant Project Engineer/ Field Superintendent	Joe Salinez
Laboratory Chief	Rick Flott

The project office staff, in addition to the above mentioned Construction-Operations Division personnel, consisted of two inspectors and a field laboratory staff of seven civilian and military personnel.

IV. TOPOGRAPHY AND GEOLOGY

REGIONAL TOPOGRAPHY

4.01 The New River Dam is located in that portion of Arizona referred to as the Gila Lowland Section of the Sonoran Desert Subprovince, Southern Basin and Range Physiographic Province. The Province is characterized by broad, gently sloping connected valleys or plains bounded by moderately high, rugged mountain ranges rising abruptly to maximum heights of several thousand feet above the fairly flat valley floors. The project is at the southern edge of a topographic and structural basin and is bounded to the southeast and southwest by the low-lying East and West Wing Mountains, respectively, and to the east by an unnamed group of peaks. The broad alluvial plain to the north and west of the project extends up to the New River and Hieroglyphic Mountains. The dam embankment spans a relatively narrow valley between the East and West Wing Mountains at the northern edge of Deer Valley, a small undissected tributary valley within the larger alluvial plain of the Salt River Valley.

REGIONAL GEOLOGY

4.02 The rock types found in the mountainous areas that border the project consist of (1) an igneous and metamorphic basement complex composed predominantly of Precambrian granite and related crystalline rocks with lesser amounts of schist and gneiss, (2) Cretaceous to Tertiary intrusive igneous rocks, consisting mainly of granite and monzonite, and (3) Tertiary volcanic rocks in the form of basalt and andesite with local accumulations of tuff, flow breccia and agglomerate. The basement complex is extensively exposed along the eastern and southeastern margins of the project. Elsewhere, particulary along the southwestern margin of the project, Tertiary age lava flows rest unconformably upon the basement complex. Exposures of intrusive igneous rocks are limited, occurring mainly in the mountains to the east.

4.03 Older sediments that constitute the valley fill are Quaternary in age and are composed mainly of poorly-to well-consolidated gravel, sand, silt and clay, representing several environments of deposition. The constituent materials were eroded from the adjacent mountain masses by stream and sheet runoff. Calcium carbonate cementation is common and considerable caliche is present near the mountain fronts. Recent (Quaternary) alluvium, consisting mainly of unconsolidated sand and gravel, fills the channels of the main stream courses and the tributaries associated with flood plain washes. The total thickness of the alluvial materials varies from zero along the mountain fronts to depths exceeding 1200 feet under the valley interior.

GEOLOGIC HISTORY

4.04 The Cenozoic history of southwestern Arizona was ushered in during the Laramide orogeny, which began in the late Cretaceous, some 90 million years ago, and continued for about 40 million years, into the early Tertiary. This portion of the State was severely affected by this tectonic episode which was characterized by regional uplift, eruption of rhyolitic to andesitic volcanic rocks, and by intrusion of several large bodies or plutons of igneous rock of a granitic composition. Following the cessation of the Laramide orogeny, southwestern Arizona was an area of general magmatic quiescence. During this time, a broad erosional surface dipping northeastward was developed and deposition of sediments in localized interior drainage basins occurred.

4.05 Another period of widespread tectonism began approximately 30 million years ago during the late Oligocene and lasted about 10 million years, into the middle Miocene. This episode, referred to as the "mid-Tertiary orogeny" was accompanied by extrusion of great quantities of rhyolitic to andesitic tuffs, breccias and flows, and deposition of thick sequences of clastic sediments in newly formed interior drainage basins. Rocks deposited during and preceding this event were subject to low angle normal faulting, steep tilting and local folding. Many of the older extrusive rocks in the Salt River Valley area were products of this orogeny. With the waning of the mid-Teritary orogeny, a profound unconformable surface was developed. Topographic lows became sites for fanglomerate and lacustrine deposition. Tuff beds and extrusive flows intercalated in these sedimentary deposits indicate continued though minor volcanic activity.

4.06 During the late Miocene, approximately 15 million years ago, subsidence, high angle normal block-faulting and erosion occurred in southwestern Arizona, which disrupted all earlier landforms. This resulted in the development of a typical basin and range structure of mountain-forming horsts separated by valleys underlain by grabens or half-grabens. Deposition of sediments began in the basins as the basins were formed. In the Salt River Valley area, these sediments were deposited under oxidizing conditions in fluviatile and lacustrine environments and consisted of clastics and evaporite sequences. Included in the sedimentary sequence are occasional interbeds of extrusive volcanic rocks of basaltic composition. Approximately 10 million years ago, faulting began to wane and sedimentation in previously separate interior basins began to coalesce.

SITE TOPOGRAPHY

4.07 The New River, an ephemeral stream, flows generally south from its headwaters in the New River Mountains across a broad gently sloping valley to the dam, a distance of approximately 24 miles. The stream gradient in the vicinity of the project is about 10 feet per mile. At the dam, the valley narrows considerably, to a width of about 2000 feet. The dam embankment spans the New River between the West Wing Mountains, which form the right abutment, and Keefer Hill, a westward projection of the East Wing Mountains, which form the left abutment. The mountains are characteristically steep and rugged although they attain only moderate heights. Elevations in the project area range from 1390 feet in the streambed up to 2000 feet at the crests of the surrounding mountains. South of the dam, the New River flows through Deer Creek, then flows about 8 miles further downstream before merging with the Agua Fria River.

SITE GEOLOGY

4.08 The geological formations present within the project area consist generally of (1) Precambrian granitic rocks, (2) Tertiary volcanic rocks, and (3) Quaternary alluvial deposits. This section presents a general discussion of the site geology. The rock names used in this report are based primarily on petrographic analyses. However, color and textural characteristics were occasionally considered in the classification process to more clearly distinguish between chemically similar rock types. Rock unit names and/or designations used in the original contract plans were changed to conform to the lithologic classifications used in this report.

4.09 Granitic Rock. The Precambrian age granitic rocks, composed primarily of granite, diorite, and related crystalline rocks, are extensively exposed in the East Wing Mountains, including Keefer Hill, and underlie the outlet works and a portion of the dam foundation on the east side of the valley. These rocks are collectively referred to as the Precambrian basement complex in this report. Granite and diorite are the dominant rock types present and appear to be of plutonic origin. The granite is characterized by its medium- to coarse-grained texture, small percentage of mafic minerals and light gray to reddish-brown color. The diorite, found in close association with the granite, is characterized by its medium- to coarse-grained texture, mottled appearance due to a high percentage of mafic minerals, and medium to whitish-gray color. Scattered occurrences of a fine- to medium-grained, mottled, medium to dark gray quartz diorite may represent a possible postmagmatic alteration of material near the margins of the diorite pluton. The granite and diorite have been intruded in numerous locations by dikes of a medium gray to black, aphanitic granitic rock which also frequently appears as inclusions within the surrounding rock mass.

4.10 Volcanic Rocks. Teritary-age volcanic rocks, composed of andesite, several varieties of tuff, flow breccia and agglomerate are extensively exposed in the West Wing Mountains. The dominant rock type, a light to medium gray aphanitic andesite, is present on the right abutment of the dam and in the spillway excavation and also underlies a portion of the dam foundation on the west side of the valley. A reddish-brown to pinkish-gray porphyritic andesite outcrops in the northwestern part of the West Wing Mountains and underlies the south abutment of dike no. 1. Associated with the aphanitic andesite is a reddish-brown volcanic cinder flow breccia, which is present as a fairly continuous layer below the andesite on the southeast flank of the mountains near the downstream end of the spillway excavation. The breccia also locally caps the andesite and infills joints in the bedrock on the northeast flank of the mountains immediately upstream of the right abutment and near the upstream end of the spillway excavation. The tuff sequence, composed of a series of pyroclastic rocks reflecting different modes of origin which have undergone varying degrees of consolidation, is exposed in the spillway excavation below the flow breccia unit and forms a prominent ridge with columnar-type jointing along the southeastern flank of the West Wing Mountains. Rock unit lithologies range from well-stratified welded ash-fall tuffs to non-stratified slightly welded ash-flows tuffs. A tuffaceous agglomerate, exposed along the northeast flank of the mountains upstream of the right abutment, was found to locally cap the andesite bedrock underlying the dam foundation of the west side of the valley.

4.11 Alluvium. The Quaternary-age alluvium can generally be designated as either older poorly- to well-consolidated valley fill, alluvial fan and flood plain deposits; or younger consolidated stream channel and tributary wash deposits. The older Quaternary alluvium also includes the usually thin spotty veneer of residual soil and slope wash found on the slopes of the East and West Wing Mountains; colluvium, consisting of desert varnished andesite blocks and rubble, which caps the hills in the vicinity of the right abutment and the spillway excavation; and rounded granite boulders which mantle the slopes and crests of the hills north of the left abutment and cover the lower slopes downstream of the left abutment. The valley floor is covered principally by finer-grained flood plain deposits consisting mostly of silts and sands which attain a maximum thickness of approximately 9 feet. The underlying coarser-grained valley fill deposits, consisting mainly of sands, gravels, and clayey gravels with numerous layers and lenses of older stream channel cobbles and boulders present to a depth of about 25 feet, extend down to bedrock, which is at known maximum depths ranging from 136 to 144 feet beneath the dam foundation near the center of the valley. Erratic, near-surface zones of caliche cementation are common on the east side of the valley above the shallow granitic bedrock pediment. The alluvium covering the slopes of the mountains is generally less than 2 feet thick.

FAULTING AND SEISMICITY

4.12 The greatest concentration of faults, particularly Quaternary faults, in the State of Arizona occur in a poorly-defined band stretching diagonally from northwest to southeast across the state, generally coinciding with areas of historical seismicity. Most faults generally exhibit steep dips and normal separation. Quaternary faults are rare in southwestern Arizona and none have been identified in the vicinity of the New River Dam project.

4.13 The closest fault system to the project is the 45-mile-long Verde fault system, located approximately 45 miles to the northeast at its southerly extent. This system consists of several splays and segments, the longest and most continuous of which is the 17-mile-long central segment. A maximum credible earthquake of Richter magnitude 7.0 could be produced by movement over the total length of the Verde fault system, resulting in a maximum bedrock acceleration of approximately 0.08 g at the dam. However, the largest earthquake ever recorded to date near the Verde system was the 1976 Chino Valley event with a Richter mangnitude of 5.1 and an epicenter location about 65 miles north of the project. This would have produced acceleration of less than 0.01 g at the dam. This fault system has shown evidence of Quaternary movement, but no historic or Holocene surface ruptures have been recorded.

4.14 The Basin and Range province in southwestern Arizona has been considered to be tectonically inactive due in part to the low levels of historical seismicity and the extensive pedimentation of mountain blocks. Evidence of average regional recurrence intervals between surface-rupturing earthquakes over the last 15,000 years range from 3500 years to possibly 15,000 years or more, indicating a lesser degree of seismic hazard potential for this portion of the state. New River Dam is located in Zone 1 of the Seismic Zone Map of the Contiguous States, an area of low seismicity. Only five earthquakes with maximum epicentral intensities between V and VI on the Modified Mercalli intensity scale have been reported within a 50-mile radius of the project since 1871. Research published by the United States Geological Survey indicates that the project has a 90-percent probability of experiencing bedrock accelerations no greater than 0.04 g in 50 years. A bedrock acceleration of 0.08 g produced by a magnitude 7.0 earthquake on the Verde fault system would require simultaneous movement on all segments and is not likely to occur during the design life of the project.

GROUND WATER AND SUBSIDENCE

4.15 Ground water information for the New River Dam project was obtained from three sources (1) water well records obtained from the State of Arizona Department of Water Resources, (2) subsurface investigations conducted prior to and during construction, and (3) observation wells installed after construction of the dam embankment had been completed.

4.16 In the basin area upstream of the dam, ground water withdrawal has been minimal and the water table gradient is flatter than the corresponding topographic slope, based on water level data provided by the following 3 wells. In 1985, ground water was measured at a depth of 100 feet (elevation 1320 feet) in well (A-5-1) 25bca, approximately 1 mile upstream, and at a depth of 121 feet (elevation, 1342 feet) in observation well no. 3, about 2-1/4 miles upstream. In 1984, ground water was measured at a depth of 212 feet (elevation, 1362 feet) in well (A-5-1) 10aab, approximately 4-1/2 miles upstream of the dam and about 1-1/2 miles north of dike no. 1.

4.17 At the dam, the ground water table appears to rise significantly to a known maximum elevation of 1357 feet, based on information from observation wells 1 and 2. In 1985, ground water was measured at a depth of 36 feet (elevation, 1351 feet) in observation well 1,350 feet downstream of dam at station 24+50, and a depth of 43 feet (elevation, 1357 feet) in observation well 2,305 feet upstream of dam station 24+50. However, since both wells are fully perforated below a depth of 25 feet, these water levels may be influenced to some extent by any localized zones of perched or semiperched ground water. In fact, underflow from nearby ephemeral borrow ponds which "cascaded" briefly into well 2, indicates the presence of at least one separate zone of saturated alluvium. Measurements of water levels and hole depths in observation wells 1 and 2 in September 1985 indicate that these wells may be plugged and that the "static" water levels may not reflect true ground water conditions. Beneath the dam, preconstruction subsurface borings encountered ground water between elevations of approximately 1300 and 1340 feet. These variations may be due to seasonal fluctuations, perched conditions or a possible ground water mound condition beneath the active stream channel. In any event, the overall higher ground water table at the dam is probably caused by shallow bedrock constrictions in the relatively narrow confines of the valley.

4.18 South of the project, in Deer Valley, the watertable declines rapidly to depths exceeding 300 feet due to intense ground water development for agricultural as well as residential and industrial purposes. In 1984, ground water was measured at a depth of 351 feet (approximate elevation, 1029 feet) in well (A-4-1) 1baa, approximately one mile south of the dam.

Subsidence

4.19 Surface subsidence and associated earth fissure development have occurred in the Phoenix metropolitan area as a result of major ground water declines. Long-term survey data are not available to determine if subsidence has occurred at the project. However, subsidence has probably been negligible due to the relatively shallow depth to bedrock and the lack of any extensive ground water development and should not pose any future problems for the dam embankment and appurtenances. The closest occurrences of measured subsidence has been in Deer Valley along portions of Beardsley Road west of Interstate Highway I-17. The maximum amount of subsidence detected between 1967 and 1981 has been 0.45 foot at 83rd Avenue, approximately 4-1/2 miles south of the dam. Subsidence would be expected in this area where the alluvial materials are much thicker and where ground water declines of up to 300 feet have occurred in the past 30 to 40 years.

4.20 Earth fissures have not been observed in the project area or in Deer Valley. The closest occurrences are about 15 miles to the southwest in the vicinity of Luke Air Force Base, where 1 to 3 feet of subsidence has been detected or estimated.

V. FOUNDATIONS

INVESTIGATIONS

5.01 Foundation investigations of the right and left abutment, outlet works, and streambed consisted of geologic mapping and reconaissance, shallow seismic refraction surveys, diamond core drilling, bucket-type power auger drilling, trenching with a dozer and backhoe, and in-situ density and permeability testing. Detailed discussions of the foundation investigations are presented in the reference listed in paragraph 2.01.

Dam Foundation

5.02 The investigation of the streambed portion of the dam foundation consisted of drilling 8 core holes to depths from 49 to 161 feet, 7 borings with a bucket-type power auger to depths from 10 to 97 feet and excavating 30 trenches with a backhoe or dozer to depths from 10 to 33 feet. The location of the core holes are shown on plate 2 and the location of the soils borings and trenches are shown on plate 3. The core hole logs and the soil logs of the borings and trenches are summarized on plates 4 through 7.

5.03 Twenty-five in-situ density tests were performed in the near surface embankment foundation materials. Five of the in-situ density tests were performed using the sand displacement method at depths ranging between 1 and 5 feet while 20 of the in-situ density tests were performed using the large scale water displacement method at depths ranging between 5 and 18 feet. The results of the density tests in the foundation are shown in table 1.

5.04 Permeability tests were conducted in test holes to obtain large scale field data to determine a representative coefficient of permeability of the foundation material. The results of the permeability tests in the foundation are shown in table 2.

West Abutment

5.05 Investigations of the west abutment consisted of drilling 2 diamond core holes to depth of 74 to 80 feet. The locations of the core holes are shown on plate 4 and the logs of the core holes are shown on plate 8.

East Abutment

5.06 Investigation of the east abutment consisted of drilling 2 diamond core holes to depths of 48 to 77 feet and excavating 2 trenches to depths of 4 to 5 feet with a dozer. The locations of the core holes are shown on plate 4 and logs of the core holes are shown on plate 8.

Outlet Works

5.07 Investigations of the outlet works consisted of drilling 4 diamond core holes to depths of 24 to 35 feet and excavating 23 trenches with a backhoe or dozer to depths of 3.5 to 12 feet and conducting one seismic refraction survey. The locations of the core holes, test trenches, and seismic refraction survey are shown on plate 9 and the logs of the core holes and test trenches are shown on plates 10 through 14.

Dike No. 1 Foundation

5.08 The investigation of the foundation of dike no. 1 consisted of drilling 5 borings with a bucket-type power auger to depths of 9.5 to 25 feet and excavating 7 trenches with a backhoe or dozer to depths of 1 to 9.5 feet and conducting eight seismic refraction surveys. The location of the borings, trenches and seismic refraction surveys are shown on plate 15. The soil logs of the borings and trenches are summarized on plate 16.

Dike No. 2 Foundation

5.09 The investigation of the foundation of dike no. 2, consisted of drilling 3 diamond core holes and conducting 3 seismic refraction surveys. These investigations were initially performed to investigate an alternative spillway site. The location and logs of the diamond core holes are shown on plate 17.

FOUNDATION TREATMENT

General

5.10 Discussion of the foundation treatment is divided into 3 segments. The dam foundation from station 13+00 to 31+90, the west abutment from station 31+90 to 33+27, and the east abutment from station 10+00 to 13+00 see plate 18. The construction sequence, geology, excavation, grouting, and clearing of the left and right abutments and the portions of the dam foundation founded in rock (station 13+00 to 21+06 and station 29+64 to 31+90) are discussed in more detail in the foundation report referenced in paragraph 2.04.

Dam Foundation

5.11 <u>Streambed Materials</u>. Foundation materials encountered in construction during foundation and core trench excavation are the same as anticipated during design.

5.12 The foundation materials consist of layered homogeneous alluvial soils extending to a depth of maximum 136 feet. Interpretation of the data contained on the soil logs and visual examination of the sides of the dozer trenches excavated in the dam foundation indicate that there are three distinct soil layers in the dam foundation above bedrock.

5.13 The top soil layer, designated stratum A, varies in thickness from 2 to 9 feet. It consists mostly of silty sand.

5.14. The middle soil layer, designated stratum B, varies in thickness from 6 to 10 feet. It consists mostly of sandy gravels. The gradational range of stratum B is shown in figure 1.

5.15 The bottom soil layer, designated stratum C, extends from stratum B down to bedrock. It consists mostly of cemented sandy gravels. The gradational range and plasticity chart of stratum C is shown in figure 2.

5.16 The foundation treatment consisted of excavating the near surface silty sands, caliche, and sandy gravels down to bedrock or elevation 1380 NGVD from station 13+00 to 31+90. A core trench was excavated beneath the core and transition zones. The core trench was excavated down to bedrock between stations 13+00 and 21+06 and stations 29+64 to 31+90 and to elevation 1365 NGVD between stations 21+23 and 29+00. The actual depth of foundation and core trench excavation is shown in red on plate 18.

5.17 Initially only the central portion of the foundation and the core trench down to bedrock or elevation 1365 NGVD were excavated as shown on plate 18. The excavation was accomplished with two push dozers and scrapers (photos 12, 13, 14, and 15). A tracked backhoe and end dumps were used to excavate the stage I core trench from station 29+10 to 31+72 to preclude degradation of the highly fracturer andesite bedrock and the core trench from station 13+00 to 16+90 to preclude degradation of the highly fractured and weathered granitic bedrock (photos 16 and 17).

5.18 After inspecting the core trench bottom, benches, and sidewall for unsuitable materials and to ensure that the embankment materials and the foundation materials satisfied filter criteria, Corps personnel approved sections of the core trench for fill placement. When the approved section was in alluvium the surface was scarified to a depth of 6 inches and compacted with 8 passes of a 50-ton articulating rubber tired roller (photo 18). Density tests using large-scale water displacement method were performed in the core trench bottom and on the benches to determine the in-place dry density. The results of the density tests are shown in table 3 and the gradation of the materials in figure 3.

5.19 After a section of core trench was approved and accepted, suitable materials from the foundation excavation satisfying core requirements were placed in the core zone of the core trench. Suitable materials from either borrow area No. 3 or the foundation excavation were processed for placement in the transition zones. The locations of the borrow areas are shown on plate 19.

5.20 After a section of the foundation was excavated, Corps personnel inspected that section of the foundation for unsuitable materials and to ensure that embankment materials and the foundation materials satisfied filter criteria. Upon approval that section of the foundation was ready for fill placement. When the approved section was not in bedrock it was scarified to a depth of 6 inches and compacted with 8 passes of a 50-ton articulating rubber tired roller. Density tests using a large-scale water displacement method were performed in the foundation and to determine the in-place dry density. The results of the density tests are shown in table 4 and the gradation of the materials in figure 4.

5.21 West Abutment. The west abutment of the dam is located on the east slope of the West Wing Mountains. The abutment consists of volcanics composed mainly of andesite.

5.22 Typically the west abutment was prepared in three phases. The first phase consisted of excavating the materials within the core-transition contact zone to depths of 3 feet using a dozer with rippers (photo 19). The second phase, the surface prepartation, was the most sensitive phase. It consisted of cleaning the core-transition contact zone to a suitable foundation (photos 20 and 21). The third phase consisted of stripping one foot of the surface materials within the pervious shell contact zones.

5.23 East Abutment. The east abutment of the dam is founded on the west slope of Keefer Hill. The abutment consists of granitic rock.

5.24 The east abutment was, also, prepared in three phases. The first phase consisted of stripping the surface materials to a maximum depth of 1 to 4 feet using a dozer with rippers. The second phase consisted of excavating materials within the core-transition contact zone to a maximum of 14 feet using tracked backhoe (photos 22, 23, and 24). The third phase, the surface preparation, was the most sensitive phase. It consisted of cleaning the core-transition contact zone to a suitable foundation (photos 25 and 26).

5.25 Treatment of the abutments and rock exposed in the core trench, after cleaning, consisted of subsurface grouting and final surface preparation. Subsurface grouting, consisting of a single line grout with a 10-foot primary hole spacing curtain along the core contact centerline, was placed by subcontractor W.G. Jaques Company of Des Moines, Iowa from January to March 1984. The grouting plan and profile are shown on plates 20 through 22.

5.26 Final surface preparation of the abutment foundaton and rock exposed in the core trench, beneath the transition zones, consisted of removing loose materials by hand and minimal air blasting. Surface preparation, beneath the core zone, consisted of intensive air blasting and placement of dental concrete (photo 27).

5.27 The rock surfaces to receive core materials was treated with dental concrete which was placed in larger depressions and low areas to provide a uniform surface for embankment material placement and compaction, and also to protect area of highly fractured bedrock. The dental concrete consisted of low slump, 3/4-inch aggregate, 1000 pounds per square inch concrete. Cleaned surfaces were wetted with water, prior to placement of dental concrete. The dental concrete was usually placed with a crane hoisted bucket. After the concrete had set, the edges were trimmed to avoid porous feathered edges. To consolidate and ensure bonding with the foundation, the concrete was carefully vibrated in place.

Dike No. 1

5.28 The dike no. 1 foundation materials consist of non-homogeneous alluvial soils and andesite bedrock. There are two distinct layers in the upper 25 feet. The top layer is sandy clay that varies between 0.5 and 2 feet in thickness. The bottom layer, which extends down to at least 25 feet below original ground surface, is a caliche cemented sandy gravel.

5.29 The foundation excavation consisted of removing the sandy clay down to the caliche and ripping 2 feet of caliche cemented sand from beneath the upstream transition and pervious shell. The sandy clays were excavated with an elevating scraper while the caliche was ripped and then excavated with two push dozers and scrapers. The depth of the foundation excavation is shown on plates 23 through 25.

5.30 After completion of the foundation excavation, an exploration trench was excavated beneath the core zone to a depth of approximately 5 feet as shown in red on plates 23 through 25. The exploration trench was ripped and then excavated with two push dozers and scrapers (photos 28, 29, 30, and 31). Typical materials encountered consisted of caliche cemented sandy gravel and andesite bedrock (photos 32 and 33).

5.31 After inspecting the foundation exploration trench bottom and side-walls to ensure compatability of the embankment material with the foundation materials, Corps personnel approved sections of the dike foundation for fill placement. When the section approved was not in bedrock it was scarified, wetted, and proof-rolled before embankment materials were placed.

5.32 South Abutment. The south abutment of dike no. 1 is founded on the north slope of the West Wing Mountains. The abutment consists of volcanics composed of andesite rock.

5.33 In general the excavation of the south abutment was accomplished in two phases. The first phase consisted of excavating the surface materials to depths of 4 feet using a dozer with rippers. The second phase consisted of cleaning the core contact zone to a suitable foundation. The construction sequence, geology, and excavation are discussed in more detail in the foundation report referenced in paragraph 2.04.

5.34 Dental concrete was placed within the core contact zone between stations 78+65 and 84+15.

Dike No. 2.

5.35 The dike no. 2 foundation materials consist of a 0- to 6-foot layer of alluvial soils composed of silty sandy gravel overlying hard, highly fractured, soft weathered granitic bedrock.

5.36 The foundation treatment consisted of excavating the upper 1-foot of material. The silty sandy gravels and soft weathered bedrock were excavated with a dozer. The extent of the foundation excavation is shown on plate 26 (photo 34).

5.37 After inspection and approval of the foundation by Corps personnel, the area was scarified to a depth of 6 inches, wetted, and proof rolled with 8 passes of a 50-ton articulating rubber tired roller (photo 18).

VI. EMBANKMENTS

FEATURES

Main Embankment

6.01 The embankment is a compacted, zoned earthfill structure composed of pervious shell zones, transition zones, a central core, and a horizontal toe drain from station 19+00 to 31+00. The upstream slope is 1.0V on 2.5H and is protected by a 12 to 24-inch thick layer of type I stone. The downstream slope is 1.0V on 2.0H and is covered by a 12-inch thick layer of type III stone. The embankment plan, profile, and cross sections are presented on plates 18 and 27 through 31.

6.02 The embankment was constructed in three stages. Stage 1 consisted of foundation and core trench excavation from station 26+50 to 31+90, west abutment excavation from station 31+90 to 33+27, foundation preparation and treatment within the core-transition contact zone of the west abutment and rock exposed in the core trench from station 29+64 to 31+90, and constructing the embankment to elevation 1380 from station 26+90 to 31+90. Stage 2 consisted of east abutment excavation from station 10+00 to 13+00, foundation, and core trench excavation from station 13+00 to 26+50, treatment within the core-transition contact zone of the east abutment and rock exposed in the core trench from station 13+00 to 21+00, and constructing the embankment to elevation 1380 at station 28+90 to crest elevation at station 23+30 and to crest elevation from station 10+00 to 23+30. Stage 3 consisted of constructing the embankment to crest elevation from station 23+30 to 33+27 and treatment within the core-transition contact zone of the west abutment.

Dike No. 1

6.03 Dike no. 1 is a compacted, zoned earthfill structure composed of pervious shell zones, transition zones, and a central core. The upstream slope is 1.0V on 2.5H and is protected by a 12 to 24-inch thick layer of type I stone. The downstream slope is 1.0V on 2.0H and is covered by a 12-inch thick layer of type III stone. The embankment plan, profile, and cross sections are presented on plates 23 through 25 and 32.

Dike No. 2

6.04 Dike no. 2 is a compacted earthfill structure composed of pervious shell material. The upstream slope is 1.0V on 6.0H while the downstream slope is 1.0V on 2.0H and is covered by 12 inches of type III stone. The embankment plan, profile, and cross sections are presented on plate 26.

MATERIALS

6.05 Core materials meeting specification requirements were obtained by blending the near surface materials of borrow area 1 to a depth of approximately 4 feet, borrow area 2 to a depth of approximately 6 feet, borrow area 3 to a depth of approximately 3 feet, and from material obtained from foundation excavation. The location of the borrow areas is shown on plate 19 and the gradation of the as-placed core materials is shown on plate 33.

6.06 Transition materials meeting specification requirements were obtained by grading pervious shell materials obtained from beneath the core materials of borrow area 3. The location of borrow area 3 is shown on plate 19 and the gradation of the as-placed transition materials is shown on plate 37.

6.07 Pervious shell materials meeting specification requirements were obtained from beneath the core materials of borrow area 3. The location of borrow are 3 is shown on plate 19 and the gradation of the as-placed pervious shell material is shown on plate 40.

6.08 Gravel drain materials were obtained by crushing and grading pervious shell materials (photo 24). Gradations of the as-placed gravel drain materials are shown on plate 43.

6.09 Type I stone was obtained from the oversize materials developed during grading of the pervious shell materials to produce transition, gravel drain, and type III stone.

6.10 Type III stone was obtained by crushing and grading pervious shell materials (photo 35).

6.11 Topsoil fill was selected from the near surface soil during the foundation stripping of dike no. 1.

6.12 Landscape stone was obtained from spillway excavation.

6.13 Desert gravel was obtained from offsite sources.

VII. EMBANKMENT QUALITY CONTROL, QUALITY ASSURANCE, AND RECORD TESTING

GENERAL

7.01 Contrator quality control and Government quality assurance testing of the embankment fill was performed to ensure quality work and to check conformance of the placed materials with contract specifications. These activities involved the combined efforts of the Contractor's Quality Control personnel, and the Corps of Engineers inspectors and laboratory personnel. The results of these activities assured that materials were placed within specified gradations and moisture contents, and that design densities were being obtained by the specified procedural compaction methods. Corps of Engineers personnel periodically obtained both disturbed and undisturbed record samples to establish the consolidation, permeability and shear strength parameters of the as-built embankment materials in order to verify adopted design parameters.

CONTRACTOR QUALITY CONTROL (CQC)

7.02 Contract provisions required the contractor to ensure embankment quality. Accordingly, a Quality Control program was established by the contractor. The following items, pertaining to the embankment, were performed by the contractor:

a. Reviewed contract requirements, checked worksite for readiness and checked that lines and grades had been established.

b. Checked for compliance with contract specifications and that required testing procedures were being followed.

(1) Continuously monitored embankment fill operation.

(2) Established necessary moisture-density relationships for contractor information and use.

(3) Performed field density tests (ASTM D 1556) to determine degree of compaction per ASTM D 698 for backfills or ASTM D 1557 for roadfills where end-product was specified.

(4) Performed gradation testing on embankment materials per ASTM D 422.

(5) Performed quality tests on stone protection materials per: ASTM C 88, C 127, C 136, C 131, and C 535.

(6) Supervised the installation of specified instrumentation.

(7) Prepared daily quality control reports which listed activities, described quality control surveillance activities and instruction, summarized material quantities and listed all test results.

CORPS OF ENGINEERS QUALITY ASSURANCE (QA)

GENERAL

7.03 Several inspectors provided continuous monitoring of embankment fill operations. In addition, Corps of Engineers on site Soils Laboratory personnel performed QA tests which consisted of field density, placement moisture contents, gradations, moisture-density relationships, and vibratory maximum-minimum density tests. The Geotechnical Branch provided an embankment engineer and a project geologist responsible for technical supervision of construction.

Field Density Tests

7.04 In-place density tests on core and transition material were performed in accordance with ASTM Standard D 1556, "Density of Soil in Place by the Sandcone Method". In place density tests on pervious shell, gravel drain, and main embankment foundation materials were performed using a large-scale water displacement method. See Appendix I.

Moisture Content Tests

7.05 A laboratory moisture determination was made in accordance with ASTM D 2216 for each field density test. Visual assessment and microwave oven results were used for rapid determination of moisture content and checked with standard oven drying test results.

Gradation Tests

7.06 Gradation tests were performed in accordance with ASTM D 422 on material collected from each density test. In additon, numerous gradation tests were performed on representative samples of the gravel drain, and slope protection materials to verify compliance with specifications.

Moisture Density Tests

7.07 Moisture-density relationships for the core materials were determined by ASTM D 698. Moisture-density relationships for the pervious shell and transition materials were determined using a compaction test method comparable to ASTM D 698 in that the compactive effort applied is 12,300 foot pounds per cubic foot and the equipment has been devised to maintain the ratios between the mold diameter, rammer diameter, and maximum particle size. A family of compaction curves for the pervious shell, transition, and core materials were developed prior to the start of fill placement.

7.08 During construction a one-point compaction test was performed on the sample obtained with each in-place density test. The percent maximum dry density was then interpolated from the family of compaction curves for the material. For approximately every five in-place density tests on a material type, a five-point compaction test was performed to augment the family of curves for that material.

Relative Density Tests

7.09 A small number of relative density tests were performed on the pervious shell, transition, and gravel drain material in accordance with ASTM Standard D 2049, "Relative Density of Cohesionless Soil". These tests were performed near the beginning of the placement to ensure that the specified procedural placement of these materials was yielding acceptable densities.

Record Sampling and Testing

7.10 Record samples (photo 36) of the as-built embankment were periodically obtained by Corps of Engineer personnel. These samples, both disturbed and undisturbed, were obtained at strategic locations predetermined by design personnel. The samples were shipped to the SPD Laboratory for record testing in order to determine the material properties of the as-built embankment. The testing program included classification, compaction, triaxial shear, permeability, and consolidation. Two field density determinations were made above and below each core record sample location.

VIII. CONSTRUCTION PROCEDURES

CORE MATERIALS

8.01 Moisture was introduced into the core materials prior to excavation by prewetting the borrow area with a sprinkler system (photo 37). Core materials were excavated and hauled with two push dozers and scrapers or a front end loader and end dump trucks. (Photos 38 and 39.) The surface of the preceeding lift was scarified with the rippers attached to a motor grader to a depth of 6 inches prior to placement of the next lift. The materials were spread on grade in 12-inch lifts with a motor grader or a dozer. When the core materials had dried back, a 10,000-gallon water pull truck was used to add water either prior to the compaction of the lift or prior to placement of the next lift. Compaction was accomplished with 8 passes of a towed, double drum tamping roller. The drums were 5-foot in diameter and 5-foot wide and were ballasted to 20,000 pounds (photo 40).

8.02 Because the contractor did not use the same towed, double drum tamping roller to construct the embankments as he had used in the demonstration and verification fills, the contractor was required to compact the core materials with 8 passes of a towed, double drum tamping roller as specified.

8.03 Core materials were placed wet of optimum at the rock-core contacts (photo 41). The purpose of placing the wetter core materials was to insure bonding between the rock and the core materials and to maximize the filling of voids and cracks in the rock with core materials. Loose material was removed from the treated rock surface 5 to 8 feet ahead of core placement by air hoses (photo 42). The cleaned and treated rock surface was throughly wetted prior to the placement of core materials. The initial lifts were placed in 6 to 12-inch thickness with a front end loader (photos 43 and 44). Compaction was accomplished by 8-wheel passes of the front end loader with a loaded bucket. Rubber tired wheel rolling was used to prevent damage to the treated abutment surface by the tamping roller. The compacted surface was scarified by back dragging the bucket teeth prior to placing a new lift. The core materials were placed against each abutment at a slope of approximately 1V on 4H (photo 45). Establishment of these ramps allowed the tamping roller to compact closer to the abutment since the rock surface was protected from the tamping roller by a layer of core material. Compaction with a tamping roller was initiated when a sufficient thickness of material covered the abutment.

TRANSITION MATERIALS

8.04 Transition materials were either excavated with a front end loader, processed through a screening plant, and loaded directly into bottom dump trucks or excavated with a front end loader, loaded directly into bottom dump trucks (photo 46), and processed on grade by a motor grader with a blade modified to remove the oversize material (photo 47). The compacted surface of the preceding lift was scarified to a depth of 6 inches prior to placement of the next lift with the rippers on a motor grader. The materials were spread in 12-inch lifts by a motor grader or a dozer. Each lift was compacted by four passes of an steel drum vibratory roller which weighed 30,640 pounds and produced 69,160 pounds of dynamic force and drum width of 84 inches wide or four passes of an steel drum vibratory roller which weighed 37,700 pounds and produced 83,500 pounds of dynamic force and drum width of 100 inches (photo 46).

8.05 No special procedures were used in placing and compacting transition materials at the rock contacts. Nested cobbles at the rock contacts were removed prior to compaction of the lift.

PERVIOUS SHELL MATERIALS

8.06 Pervious shell materials were excavated and hauled with two push dozers and scrapers or with a front end loader and dump trucks. The compacted surface of the preceding lift was scarified to a depth of 6 inches prior to placement of the next lift with the rippers on a motor grader (photo 46). The materials were spread in 24-inch lifts by a motor grader or dozer (photo 48). Each lift was compacted by four passes of a steel drum vibratory roller.

8.07 No special procedures were used in placing and compacting pervious shell materials at the rock contacts. Nested cobbles at the rock contacts were removed prior to compaction of the lift.

GRAVEL DRAIN

8.08 Gravel drain material for the downstream horizontal toe drain was obtained by crushing and grading pervious shell materials (photo 35). The materials produced were stockpiled prior to placement. The gravel drain materials were placed with end dump trucks, spread with a rubber tired dozer and compacted by 8 controlled passes of the rubber tired dozer in order to minimize particle crushing.

TYPE I STONE

8.09 Type I stone was obtained using a screening plant to process the oversize, from the pervious shell material, developed during the production of transition, gravel drain and type III stone. Type I stone was placed on grade with a tracked backhoe (photos 55 and 56) and 150-ton crane with a BG blade.

TYPE III STONE

8.10 Type III stone was obtained by crushing and grading pervious shell material (photo 35). The type III stone was stockpiled prior to placement. The type III stone was placed on grade with a tracked backhoe and a 150-ton crane with a BG blade.

SPILLWAY

8.11 Excavation of the spillway is discussed in detail in the foundation report referenced in paragraph 2.04. The spillway excavation, in general, consisted of drilling explosive charge holes, blasting, and excavating. Excavation and hauling of the loosened rock was accomplished with push dozers and scrapers or with front end loader and end dump trucks. The excavated materials were stockpiled upstream of the right abutment and placed in the downstream pervious shell. The spillway walls were trimmed with a slope board attached to a dozer. The slope trimming was conducted to remove overhangs, loose material, and dress up the slopes.

OUTLET

8.12 Excavation and cleaning of the outlet is discussed in detail in reference cited in paragraph 2.04. The following is a brief description of the construction procedures of the outlet. The outlet trench was ripped with a dozer (photo 49) and excavated with push dozers and scrapers while the energy dissapator was excavated by drilling explosive charge holes, blasting, and excavating. Excavation of the loosened rock was accomplished with a tracked backhoe. The methods and procedures used to clean the outlet trench were similar to those used for the abutment.

8.13 A concrete plug or 1V on 1H concrete fillet was constructed within the core zone on both sides of the outlet conduit from the surface of rock to the top of the conduit to preclude seepage paths along the outlet trench and to minimize differential settlements (photos 50, 51, 52, and 53). The concrete plug was not constructed as designed due to the greater depth of excavation in the adjacent core trench. A low slump, 3/4-inch aggregate mix was placed with a concrete bucket and crane. The low slump allowed the concrete to be placed on the 1V on 1H slope without forms. The outlet conduit and rock contact zones were prewetted before the concrete was placed and the concrete was vibrated with emphasis where the concrete contacted the outlet conduit and rock.

8.14 The outlet conduit and concrete plug (fillet) were treated as abutments within the core zone. Loose material was removed from the surface of the concrete by air hoses. The cleaned surface of the concrete was thoroughly wetted prior to the placement of core materials. The initial lifts were placed with a front end loader and were placed wet of optimum in 6 to 12 inches thick lifts. Compaction was accomplished by 8 wheel passes of the front end loader with a loaded bucket. Wheel rolling was used to prevent damage to the surface of the concrete. The compacted surface was scarified by back dragging the bucket teeth prior to placing a new lift. Compaction with a tamping roller was initiated when a sufficient thickness of material covered the concrete plug or conduit.

8.15 Backfill and fill adjacent to the conduit placed outside of the core zone consisted of transition materials. The materials within 2 feet of the conduit were placed in 4-inch lifts and compacted with hand held power tampers and hand operated vibrating rollers to at least 95 percent maximum density as determined by ASTM D 698. Before each lift was placed the preceeding lift was scarified by a small lawn tractor equipped with a rake. Also before each lift was compacted the lift was processed to remove the plus 3-inch material which was in contact with the conduit. Nested gravels in point contact along the conduit were also removed.

FOUNDATION DEPRESSION

8.16 A 12 to 14-foot depression was excavated by a tracked backhoe in the soft weathered diorite and intrusive rock during excavation of the core trench. The depression was located between station 14+85 and 15+25 and between 5 and 35 feet upstream of the centerline of the dam.

8.17 Material for backfill of the depression consisted of core materials. Loose material was removed from the sides and bottom of the depression by air hoses, brooms, shovels, and buckets. The cleaned surfaces of the depression were throughly wetted prior to the placement of core materials. The core materials were wet of optimum, placed in 4-inch lifts, and compacted with hand held power tampers to at least 95 percent maximum density as determined by ASTM D 698. Each lift was scarified before the next lift was placed (photo 54).

ESTHETIC TREATMENT

8.18 Topsoil fill, desert varnished landscape stone, and desert gravel were placed over the type III stone on the downstream slopes of the dam and dike no. 1. The purpose of the esthetic treatment was to visually integrate the embankment into the surrounding terrain. The 150-ton crane with a BG blade and a tracked backhoe were used to place the topsoil, landscape stone, and desert gravel (photos 55 and 56).

IX. MATERIAL PROPERTIES

GENERAL

9.01 As required by ER 1110-2-1925, "Field Control Data for Earth and Rockfill Dams," field control results were summarized by the Resident Engineer staff and periodically transmitted to Engineering Division during active construction periods. These reports, along with the Report of Soil Tests on the New River Dam record samples, yielded the following results:

CORE MATERIALS

Field Control Results

9.02 Final statistical summaries of field control test results on the core material are presented graphically on plate 33. The monthly field control and placement data obtained as part of the QA program, are shown on plate 34. A plan and profile of the field control test locations are shown on plate 35.

a. Moisture-Compaction Trends. Project specifications required the placement moisture content of the core material to be within 3 percent below to 3 percent above the optimum moisture content, and that the material be compacted by 8 passes of a roller. Design required the material be compacted to not less than 95 percent of maximum dry density as determined by ASTM D 698. The field control test results indicate that core fill was generally placed slightly dry of optimum with a mean of 0.2 below optimum moisture content. The plot of placement moisture content for the core material indicates slightly drier placement during the late spring and summer months. An upward trend in placement moisture is observed during the autumn months through the end of the project. This is attributed to the cooler temperatures. Results of field density tests indicate that the core materials were compacted to an average of 101.1 percent of maximum dry density (ASTM D-698) with an average dry density at 122.2 pcf.

b. Gradation. Specifications required the core material to have a minimum of 20 percent by weight passing the No. 200 sieve. Results of field control tests indicate that none of the tests had less than 22 percent passing the No. 200 sieve, while 10 percent of the tests had more than 52 percent passing the No. 200 sieve. The fines content anticipated during design had a median of 35 percent by weight passing the No. 200 sieve, while the field control results had an median of 39 percent by weight passing the No. 200 sieve.

Record Test Results

9.04 Test results performed by the SPD Laboratory on record samples of the core material are summarized on plate 36.

a. Permeability. Permeability tests were performed in both the horizontal and vertical directions on samples from three undisturbed core material record samples. The results are shown on plate 36. The horizontal permeabilities ranged from 0.011 to 0.058 feet per day (fpd) and the vertical permeabilities ranged from 0.012 to 0.06 fpd. Both the maximum horizontal and vertical permeabilities were lower than the design permeabilities of 1.5 and 0.3 fpd, respectively.

b. Shear Strength. Core material shear strengths were determined for undisturbed record samples using triaxial compression tests in accordance with the procedures described in EM 1110-2-1906,"Laboratory Soil Testing," 30 November 1970. The total strength was not determined under the unconsolidated undrained condition (Q-type) because the tests performed for the General Design Memorandum, which is referenced in paragraph 2.01, indicated that the design strength determined from unconsolidated-undrained condition (Q-type) was higher than the design strength determined from the consolidated-undrained condition (R-type). This occurs because samples tested in the unconsolidated-undrained condition are not back pressure saturated and therefore developed lower pore-pressures than similar samples tested in the consolidated-undrained condition which are back pressure saturated. Both the total and effective strengths were determined under consolidated-undrained conditions with pore-pressures measured and recorded. The as-built strengths were higher than the selected design strengths. The as-built "R" strength had an angle of internal friction of 16 degrees and a cohesion of 980 pounds per square foot (psf). This strength was higher than the design angle of internal friction of 12 degrees and a cohesion of 600 psf. The as-built effective "S" strength had an angle of internal friction of 34 degrees. This strength was higher than the design angle of internal friction of 32 degrees. The selection of the strength parameters is based on the guidelines outlined in Section 9 of EM 1110-2-1902, Stability of Earth and Rockfill Dams, 1 April 1970.

c. Consolidation. Consolidation tests were performed on undisturbed record samples obtained from the core zone of the embankment. The results of these tests are shown graphically on plate 36, in terms of void ratio (e) versus pressure (log P) curves. The record samples had consolidation curves similar to those used in design. The initial void ratio of the undisturbed record samples varied from 0.38 to 0.45.

TRANSITION MATERIALS

Field Control Results

9.05 Final statistical summaries of field control test results on the transition material are presented graphically on plate 37. The monthly field control and placement data obtained as part of the QA program, are shown on plate 38. A plan and profile of the field control test locations are shown on plate 35.

a. Moisture-Compaction Trends. Specifications had no requirement on the placement moisture content of the transition material. Design required the material be compacted to not less than 80 percent relative density and the average compaction be at least 85 percent relative density as determined by ASTM D 2049. Studies performed during placement of the verification and demonstration fill indicated that in order to achieve 85 percent relative density the materials had to be compacted to 98 percent of the maximum density as determined by a compaction test equivalent to ASTM D 698.

The mean placement moisture content was 3.4 percent. The materials were significantly drier during the dry, hot summer months. Field density tests show the transition material is compacted to an average of 99.4 percent of maximum dry density with an average dry density of 136.5 pcf. Appendix II discusses the method used to correct for oversize.

b. Gradation. Specifications required the transition material be processed pervious shell material.

Record Test Results

9.06 Test results performed by the SPD Laboratory on remolded record samples of the transition material are shown on plate 39. At least 75 percent of the field dry densities were denser than 132 pcf; therefore, the record samples were remolded to 132 pcf for permeability and shear tests.

a. Permeability. Results of the record permeability tests on the transition material ranged from 0.56 to 1.2 fpd with an average value of 1 fpd. This is lower than the 20 fpd assumed in design.

b. Shear Strength. Transition material shear strengths were determined for remolded record samples using triaxial compression tests. Strengths were determined under consolidated undrained conditions with pore pressures measured and recorded (R-type). The as-built strengths were higher than the assumed design strengths. The as-built "R" strength had an angle of internal friction of 27 degrees and a cohesion of 1000 psf. The selected design "R" strength had an angle of internal friction of 23 degrees and a cohesion of 930 psf. The as-built effective "S" strength had an angle of internal friction of 40 degrees. This was slightly higher than the design angle of internal friction of 39 degrees. The selection of the strength parameters is based on the guidelines outlined in section 9 of EM 1110-2-1902, Stability of Earth and Rockfill Dams, 1 April 1970.

PERVIOUS SHELL MATERIALS

Field Control Results

9.07 Final statistical summaries of field control test results on the pervious shell material are presented graphically on plate 40. The monthly field control and placement data obtained as part of the QA program, are shown on plate 41. A plan and profile of field test locations are shown on plate 35.

a. Moisture-Compaction Trend. Specifications had no requirement on the placement moisture content of the pervious shell material. Design required the material be compacted to not less than 80 percent relative density and the average compaction be at least 85 percent relative density as determined by ASTM D 2049. Studies performed during the verification and demonstration fill, indicated that in order to achieve 85 percent relative density the materials had to be compacted to 98 percent of the maximum density as determined by a compaction test equivalent to ASTM D 698.

The mean placement moisture content was 2.8 percent. The materials were significantly drier during the dry, hot summer months. Field density tests show the pervious shell material was compacted to an average of 103.1 percent of maximum dry density with an average dry density of 144.6 pcf. Appendix II discusses the method used to correct for oversize.

b. Gradation. Specifications required the pervious shell material to have no more than 10 percent by weight passing the No. 200 sieve. Field control test results indicate that less than 12 percent of the tests had more than 10 percent passing No. 200 sieve. The average fines content for the pervious shell zones was 6 percent.

Record Test Results

9.08 Test results performed by the SPD Laboratory on remolded record samples of the pervious shell material are shown on plate 42. When corrected for oversize, see Appendix II, the matrix of at least 75 percent of the field dry densities were denser than 132 pcf; therefore the record samples were remolded to 132 pcf for permeability and shear tests.

a. Permeability. Results of record permeability test on the pervious shell materials ranged from 7 to 46 fpd with an average value of 35 fpd. This is higher than the 20 fpd selected in design.

b. Shear Strength. Pervious shell shear strengths were determined for remolded record samples using triaxial compression tests. Strengths were determined under consolidated undrained conditions with pore-pressures measured and recorded (R-type). The as-built strengths are higher than the selected design strengths. The as-built "R" strength has an angle of internal friction of 27 degrees and a cohesion of 1000 psf. The selected design "R" strength has an angle of internal friction of 23 degrees and a cohension of 930 psf. The as-built effective "S" strength has an angle of internal friction of 40 degrees. This is slightly higher than the design angle of internal friction of 39 degrees. The selection of the strength parameters is based on the guidelines outlined in Section 9 of EM 1110-2-1902, Stability of Earth and Rockfill Dams, 1 April 1970.

GRAVEL DRAIN MATERIAL

Field Control Results

9.09 a. Density. Specifications required the gravel drain material be compacted to not less than 85 percent of relative density as determined by ASTM D 2049. Field dry density tests show the gravel drain material is compacted to an average of 102 percent of relative density with an average dry density of 109 pcf.

b. Gradation. A final statistical analysis of field control gradation test results on gravel grain material is summarized on plate 43.

Record Test Results

9.10 Test results on remolded record test samples of the gravel drain material are shown on plate 43. The samples were remolded to 109 pcf.

a. Permeability. Results of the record permeability tests on the gravel drain material ranged from 1000 to 1800 fpd with an average value of 1500 fpd. This is lower than the 7000 fpd selected in design.

b. Shear Strength. Gravel drain material shear strengths were determined for remolded record samples using triaxial compression tests. Strengths were determined under consolidated drain conditions (S-type). The as-built "S" strength has an angle of internal friction of 40 degrees. This is higher than the selected design angle of internal friction of 35 degrees. The selection of the strength parameters is based on the guidelines outlined in Section 9 EM 1110-2-1902, Stability of Earth and Rockfill Dams, 1 April 1970.

c. Rock Quality Tests. The results of L.A. Rattler, specific gravity, and sulfate soundness tests are summarized in table 5. The test results indicate that the materials meet specification requirements.

INFILLING MATERIAL

9.11 An inspection of the core trench surface, following the completion of stage I excavation, revealed the presence of a pervasive green clay infilling material along predominantly high angle joint planes in the andesite bedrock. The infilling material was restricted to the core trench between station 29+64 at the edge of bedrock excavation and approximately station 31+70 near the base of the right abutment slope. Prior to foundation preparation, samples of the green clay infilling material were obtained and sent to the SPD and ERTEC laboratories for testing. Due to the small size of the samples, only dispersion and classification tests were performed. The results are enclosed as attachments 1 and 2.

Classification

9.12 The abutment infill material classifies as a plastic clay (CH).

Dispersion

9.13 Dispersion tests indicate the infill materials is nondispersive.

X. EMBANKMENT ANALYSIS

SLOPE STABILITY

10.01 The results of tests performed on record samples indicate that the permeability of the as-constructed pervious shell material is higher than the selected design permeability and that the shear strength of the as-constructed embankment materials are higher than the design shear strengths. Therefore, the upstream shell will drain faster during drawdown and the slope stability safety factors of the as-constructed embankment slopes will exceed the original design safety factors. The stability of the embankment slopes was not reanalyzed. The design values are summarized in table 6.

SETTLEMENT

10.02 The results of the consolidation tests on record samples from the core materials of the as-constructed embankment indicate no significant variation in the e versus log p curves when compared to the design consolidation tests. The expected settlements, therefore, should not exceed the estimated settlements calculated during design.

SEEPAGE

10.03 The results of tests performed on record samples indicate that the permeabilities of the core, transition, and gravel drain materials of the as-constructed embankment are lower than the design permeabilities for these materials and that the permeability of the pervious shell material of the as-constructed embankment is higher than the design permeability for the pervious shell material.

10.04 The through seepage analysis of the as-constructed dam embankment will differ from the analysis performed during design. The flow net developed to determine the through seepage quantities for the dam embankment during design was based on the following assumptions (1) the core was cracked and filled with transition material and (2) the permeabilities of the pervious shell and transition materials were equal. As shown in figure 5, the through seepage quantities were estimated to be on the order of 245 cubic feet per day per foot of embankment length with the pool at spillway crest. The flow net developed to determine the through seepage quantities for the as-constructed dam embankment, see figure 6, was based on the assumption that the core was cracked and filled with transistion material. From the figure, the through seepage quantities were estimated to be on the order of 50 cubic feet per day per foot of embankment length with the pool at spillway crest.

10.05 The analysis of the as-constructed gravel drain will differ from the analysis performed during design because the as-constructed permeability of the gravel drain material is lower than the design permeability. The reanalysis indicates that the as-constructed gravel drain would be capable of handling the as-constructed through seepage quantities. The as-constructed embankment through seepage is only 1/5 of the design value requiring a thinner gravel drain blanket than required by design.

10.06 The through seepage analysis of the as-constructed dike no. 1 embankment will not differ from the analysis performed during design. The flow net developed to determine the through seepage quantities for the dike no. 1 embankment during design, see figure 7, was based on an intact core. The lower permeability of the as-constructed core material would reduce the through seepage quantities.

XI. DIVERSION AND CONTROL OF WATER

11.01 The diversion and control of water consisted of the construction of two temporary diversion levees to pass the 25-year flood flow of 28,000 cfs. The first diversion levee (stage I diversion levee) was constructed to protect the west (right) abutment and the stage I foundation and core trench excavation and embankment, see plate 44. After grouting the rock exposed by the stage I core trench excavation and grouting the west abutment to elevation 1406, the stage I embankment was constructed to elevation 1380 and capped with a 1-foot layer of erosion resistant material and a 2-foot layer of type I stone. The materials from the stage I diversion levee were used to construct the second temporary diversion levee (stage II diversion levee) which protected the east abutment, outlet works, and the stage II foundation and core trench excavation and embankment, see plate 44. A 300-foot wide breach was left in the embankment between the stage II diversion levee and the west abutment. Materials from the stage II diversion levee were placed in the appropriate zones of the embankment during the closure.

11.02 Closure of the breach commenced on October 1984. The embankment was constructed to elevation 1456 by November 27, 1984, and to elevation 1485 by December 31, 1984. The embankment was topped out in January 1985.

XII. INSTRUMENTATION

12.01 Instrumentation consisted of installing 50 settlement monuments and 3 observation wells. Twenty-two monuments were installed at the upstream edge of the crest of the dam embankment and twenty-one monuments were installed at the upstream edge of the crest of dike no. 1 to monitor crest settlements. Seven monuments were installed on the upstream slope of the dam embankment to monitor slope movements. See plate 45 for the location of the settlement monuments. The three observation wells were installed to monitor the ground water levels. See plate 1 for the location of the observation wells.

XIII. CONSTRUCTION NOTES

MODIFICATIONS AND CHANGES

13.01 Modifications and changes were made to the plans and specifications during contruction to utilize available construction materials and due to conditions not anticipated during design. The geotechnical related contract modifications and field changes are listed in tables 7 and 8. The final contract bid items with estimated and actual quantities are in table 9.

13.02 The actual quantities for the geotechnical related bid items 7b, 9, and 10b were significantly higher than the estimated amounts. Items 7b and 10b were higher because more foundation preparation was done during construction than was anticipated during design. Item 9 was higher because the top of rock was lower than was anticipated during design, see plate 18.

13.03 The actual quantities for the geotechnical related bid items 4, 6a, 6b, 22, 39f, and 39g were significantly lower than the estimated amount. Items 6a, 6b, and 22 were not used at all. Item 4 was lower because the amount of material suitable for topsoil was thinner than anticipated during design. Items 39f and 39g are discussed in detail in the foundation report referenced in paragraph 2.04

XIV. RECOMMENDATIONS AND CONSIDERATIONS

14.01 The following items noted during various construction phases may be helpful for the design, specification preparation, and construction of other projects.

14.02 A well defined verification fill should be required by specifications and included in the project plans to demonstrate, verify, and evaluate the contractors embankment construction procedures consisting of placement, spreading, compacting, and scarifying. This would aid the contractor and inspection personnel in embankment construction control.

14.03 The processing requirements necessary to produce a selected gradation of stone from alluvial borrow materials and to estimate the quantity of the selected stone available should be based on actual data such as mass gradations performed on representative samples of the material rather than on visual estimates of the percentages of cobbles and boulders and on an estimate of the maximum particle size while logging the test trenches and test holes during field exploration. The size of the representative samples, should be significantly large and based on the relative abundance of oversize and the maximum particle size. The mass gradations should be used to adjust the visual estimates of the percentages of cobbles and boulders for the logs of test trenches and test holes for which mass gradations are not performed.

14.04 The number of passes required to compact the core, transition, and pervious shell materials to each materials design density was reduced from 8 to 4 passes based on the results of the verification and demonstration fills. In future Design Memorandums and Plans and Specifications the number of passes required to compact similar materials to 90 percent ASTM D 1557 should be reduced from 8 passess to 6 passes.

14.05 The material for pervious shell was identified in the specifications as clean, free draining gravelly sand and sandy gravel, with not more than 10 percent of the material, by weight, shall pass the No. 200 sieve. Because terms such as clean and free draining are interpreted differently by different occupations, the identification and suitability of materials should be based on gradation and Atterberg limits tests only.

XV. SUMMARY

15.01 The embankment was constructed in accordance with plans and specifications with few field modifications. Based upon record test results the as-built embankments meet or exceed design requirements. The well constructed project is the direct result of the good design and excellent cooperation between design and construction personnel during construction.

TABLES

Table 1. DAM FOUNDATION-FIELD DENSITIES, RELATIVE DENSITIES, DESIGN DATA

			Dry Density					
Test hole or trench[a]	Depth (ft)[b]	Field (pcf)	ASTM D 2049[d] Max. (pcf)	Min. (pcf)	ASTM D 698-70[e] Max. (pcf)	RD (%)	Compaction (%)	Foundation zones
TT 80-197	0	98.5	(*)	(*)	(*)			Stratum A
TT 80-198	0	98.9	(*)	(*)		(*)		Stratum A
TT 80-199	0	105	(*)	(*)		(*)		Stratum A
TT 80-201	0	99.1	(*)	(*)		(*)		Stratum A
TT 80-202	0	105.8	(*)	(*)		(*)		Stratum A
TH 80-34	10	146.5	147.4	124.6	(*)	97	(*)	Stratum B
TH 80-35	8	139.7	139	124.3	(*)	104	(*)	Stratum B
TT 80-197	8	[f]85.1	140.7	119.3	(*)	264	(*)	Stratum B
TT 80-198	8	143.6	146.9	131	146.2	81	91	Stratum B
TT 80-199	9	140.2	143.2	130.6	(*)	78	(*)	Stratum B
TT 80-199	15	135.2	144.4	126	139.8	53	97	Stratum B
TT 80 200	7	125.2	139.8	112.7	134.4	52	93	Stratum B
TT 80 201	6	135.1	140.1	121.6	138.6	76	97	Stratum B
TT 80-201	13	143.1	146.6	134	144.6	74	99	Stratum B

Note: See footnotes at end of table.

Table 1. (Continued)

Test hole or trench[a]	Depth (ft)[b]	Field (pcf)	Dry Density: ASTM D 2049[d] Max. (pcf)	Min. (pcf)	ASTM D 698-70[e] Max. (pcf)	RD (%)	Compaction (%)	Foundation zones
TT 80-202	9	141.9	140.9	128.3	137.6	107	103	Stratum B
TH 79-32	13	149.9	148.6	115.9	(*)	103	(*)	Stratum C
TH 79-33	9	137.5	133.9	110.2	(*)	112	(*)	Stratum C
TH 79-34	18	141.1	138	125.2	(*)	121	(*)	Stratum C
TH 80-35	18	(*)	142.3	128.7	(*)	(*)	(*)	Stratum C
TT 80-197	12	148.3	147	123.5	(*)	105	(*)	Stratum C
TT 80-198	15	139.4	144.8	125.1	(*)	75	(*)	Stratum C
TT 80-199	18	(*)	150.4	129.5	146.3	(*)	(*)	Stratum C
TT 80-200	13	[f]123.5	142.2	114.3	(*)	38	(*)	Stratum C
TT 80-201	18	138.2	138.3	115.8	141	98	98	Stratum C
TT 80-202	15	142.1	136.9	120.1	139.9	126	102	Stratum C

Note: Astrisk (*) indicates not tested or determined.

a. See plate 3 for location of test holes and test trenches.

b. Depth indicates distance from ground surface to the top of the sample.

c. Field densities were performed on minus 12-inch material.

d. ASTM D 2049 tests performed on minus 3-inch material and the densities corrected for percent oversize.

e. ASTM D 698-70 tests performed on -3/4-inch material, and the densities corrected for percent oversize.

f. Error in large-scale density.

Table 2. DAM FOUNDATION - FIELD PERMEABILITY, DESIGN DATA

Test hole	Depth (ft)	Field permeability (fpd)	Field Dry Density (pcf)	Materials Source
79-34	10	22.7	146.5	Stratum B
80-35	8	19.4	139.7	Stratum B
79-32	13	18.5	149.9	Stratum C
79-33	9	8.5	137.5	Stratum C
79-34	18	6.2	141.0	Stratum C
80-35	18	8.0	--	Stratum C

*From large-scale density tests.
- No value obtained.

Table 3. CORE TRENCH FIELD DENSITIES AND COMPACTION TESTS RESULTS CONSTRUCTION DATA

Station	Offset (ft)	Elevation (ft)	Field Dry Density (pcf)	ASTM D 698-70 Max. (pcf)	Compaction (%)
28 + 15	35US	1371.2	147.4	125.7	115
29 + 68	30DS	1371.8	153.7	124.9	123
23 + 05	1DS	1364.8	130.2	118.6	104
26 + 00	1DS	1365.0	128.2	113.8	112
23 + 40	30DS	1370.0	133.8	125.7	101
23 + 40	30US	1370.0	143.1	125.7	111
23 + 45	1US	1365.0	138.7	130.3	105
26 + 25	1US	1366.0	119.8	118.8	100

a. US denotes upstream of centerline and DS denotes downstream of centerline.
b. Elevation indicates top of sample.
c. ASTM D 698-70 tests performed on 2-inch material and densities corrected for percent oversize.

Table 4. DAM FOUNDATION-FIELD DENSITIES AND COMPACTION TEST RESULTS, CONSTRUCTION DATA

Station	Offset	Elevation (ft)	Field Dry Density (pcf)	ASTM D 698-70 Max (pcf)	Compaction (%)
27 + 05	185US	1380.1	146.5	136	
24 + 25	185US	1380	132.6	128.1	100
22 + 05	110US	1380	141.6	131.5	108
20 + 50	150US	1380	150.8	136.6	109
27 + 25	135DS	1380	136.1	133.7	102
18 + 80	210US	1380	146.9	137.5	102
17 + 50	230DS	1380.1	153.8	141.0	109
22 + 70	185DS	1381.4	156.7	140.	112
31 + 70	66US	1381.8	127.8	131.4	91
31 + 65	120US	1381.8	140.5	134.4	100
31 + 06	114DS	1380.3	130.8	133.9	91
30 + 75	130DS	1379.8	141.4	136.2	99

a. US denotes upstream of centerline and DS denotes downstream of centerline.
b. Elevation indicates top of sample.
c. ASTM D 698-70 tests performed on 2-inch material and the densities corrected for percent oversize.

Table 5. EMBANKMENT GRAVEL DRAIN MATERIAL

L. A. Rattler loss %	Specific Gravity	Sulphate Soundness %
18.2	2.65	5.3

Table 6. EMBANKMENT AND FOUNDATION, SUMMARY OF GDM AND AS-CONSTRUCTED DESIGN VALUES

	Unit Weight				Shear Strength								Permeability	
					S		R				Q			
	Wet (pcf)		Sat (pcf)		Ø (Degree)		Ø (Degree)		C (psf)		Ø (Degree)		(fpd)	
Zone	GDM	Const	GDM	Const	GDM	Const	GDM	Const	GDM	Const	GDM	Const	GDM	Const
Core	[a]128	(136)	[a]135	(139)	32	(34)	12	(16)	600	(980)	22	—	0.3	(0.06)
Shell	[b]136	(149)	[b]146	(154)	39	(40)	23	(21)	930	(3000)	20		20	(35)
Transition	[b]130	(141)	144	(149)	39	(40)	23	(27)	930	(3000)	20		20	(1.0)
Drain	[c]124	(112)	[c]134	(131)	35	(40)							[c]7000	(1500)
Found Stratum B	134	---	151	---	40	---	29	---	1000	---			20	----
Found Stratum C	144	---	150	---	36	---	14	---	2000	---			8	----
Found Dike #1	136	---	142	---	36	---	14	---	2000	---			8	----
Found Dike #2	150	---			45	---							1	----

a. Compacted to 95-percent maximum density ASTM D 698.
b. Compacted to 85-percent relative density ASTM D 2049.
c. Values developed from data available in the Los Angeles District files.
d. Values in parenthesis were developed from field density test performed during construction or from laboratory tests performed on record test samples.

Table 7. GEOTECHNICAL RELATED CONTRACT MODIFICATIONS

Mod. No.	Item	Description of Change	Cost
P00005	Stage I Excavation of Unsuitable Foundation	Excavation of core trench from elevation 1365 as shown on the contract drawings to elevation 1355, from station 29+64 to 31+90. (unsuitable foundation)	$ 68,227.00
P00009	Grouted Stone Upstream Intake Structure	Place and grout stone upstream of Intake Structure, station 20+99.27 to 21+30.	$ 24,779.00
P00010	Stone Protection	Contractor was required to significantly alter borrow procesing methods and procedures in order to obtain adequate and acceptable type I stone.	$480,000.00
P00013	Observation Wells	Install three observations wells at the New River dam site.	$ 53,877.00

Table 8. FIELD CHANGES

Item	Date	Description	Cost
Core*		Increase lift thickness from 8" to 12" and reduced number of passes from 8 to 4 based on demonstration and verification fill construction results.	No Cost
Transition		Reduce number of passes from 8 to 4 based on demonstration and verification fill construction results.	No Cost
Pervious Shell		Reduce number of passes from 8 to 4 based on demonstration and verification fill construction results	No Cost

*Because the contractor did not use the same towed, double drum tamping roller to construct the embankments as he used in the demonstration and verification fill, the contractor was required to compact the core materials with 8 passes of a towed, double drum tamping roller as specified.

Table 9. ESTIMATED AND ACTUAL QUANTITIES AND UNIT PRICES

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
1.	MOBILIZATION AND PREPARATION WORK	1	Job	L.S	1
2.	DIVERSION AND CONTROL OF WATER	1	Job	L.S	1
3.	CLEARING AND GRUBBING	1	Job	L.S	1
4.	STRIPPING, DIKES 1 & 2	85,000	Cu. Yd	$ 1.25	68,540
5.	EXCAVATION, DIKE NO. 1 (EXPLORATION TRENCH)	33,000	Cu. Yd.	$ 1.75	41,443
6.	SCALING				
	(a) FIRST 1000 Cu. Yd.	1,000	Cu. Yd.	$17.00	0
	(b) OVER 1000 Cu. Yd.	500	Cu. Yd.	$16.00	0
7.	EXCAVATION				
(a)	DAM EAST ABUTMENT	11,700	Cu. Yd.	$ 7.00	8,313
(b)	DAM WEST ABUTMENT	2,000	Cu. Yd.	$16.00	8,653
8.	EXCAVATION, FOUNDATION (DAM)	325,000	Cu. Yd.	$ 1.25	288,106
9.	EXCAVATION, CORE-TRENCH (DAM)	60,000	Cu. Yd.	$ 1.60	77,360
10.	FOUNDATION PREPARATION				
	(a) FIRST 1,500 Man Hr.	1,500	Man Hr.	$30.00	1,500
	(b) OVER 1,500 Man Hr.	1,500	Man Hr.	$28.00	5,813
11.	EXCAVATION, OUTLET WORKS	29,000	Cu. Yd.	$ 7.00	41,652
12.	EXCAVATION, SPILLWAY	96,000	Cu. Yd.	$ 5.00	101,181
13.	EXCAVATION, ACCESS ROAD	5,500	Cu. Yd.	$ 4.00	6,052
14.	EXCAVATION, TOE	25,000	Cu. Yd.	$ 1.25	10,000
15.	FILL, OUTLET WORKS	15,000	Cu. Yd.	$ 2.50	17,094
16.	FILL, ACCESS ROAD	71,000	Cu. Yd.	$ 1.50	73,676
17.	FILL, CORE	524,000	Cu. Yd.	$ 1.50	539,214
18.	FILL, TRANSITION	410,000	Cu. Yd.	$ 1.75	422,265
19.	FILL, PERVIOUS SHELL	1,640,000	Cu. Yd.	$ 1.50	1,627,807

TABLE 9. (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
20.	FILL, TOE	21,000	Cu. Yd.	$ 1.25	15,884
21.	FILL, MISCELLANEOUS	245,000	Cu. Yd.	$ .40	154,684
22.	ADDITIONAL ROLLING	500	Hours	$ 90.00	0
23.	GRAVEL DRAIN	9,000	Cu. Yd.	$ 7.00	9,000
24.	STONE, TYPE I	35,000	Ton	$ 5.00	35,000
25.	STONE, TYPE II	40,000	Ton	$ 5.00	40,000
26.	STONE, TYPE III	50,000	Ton	$ 6.00	50,000
27.	GROUTING, STONEWORK	1,600	Cu. Yd.	$ 50.00	1,600
28.	CONCRETE, CONDUIT	1,400	Cu. Yd.	$200.00	1,400
29.	CONCRETE, OUTLET CHANNEL SILL	13	Cu. Yd.	$100.00	13
30.	CONCRETE, SPILLWAY SILL	36	Cu. Yd.	$125.00	36
31.	INTAKE STRUCTURE	1	Job	L.S.	1
32.	ENERGY DISSIPATOR	1	Job	L.S.	1
33.	CONCRETE PLUG (LEAN MIX)				
	(a) FIRST 300 Cu. Yd.	300	Cu. Yd.	$ 50.00	300
	(b) OVER 300 Cu. Yd.	100	Cu. Yd.	$ 45.00	100
34.	CONCRETE, DENTAL				
	(a) FIRST 1000 Cu. Ft.	1,000	Cu. Yd.	$ 90.00	1,000
	(b) OVER 1000 Cu. Ft.	750	Cu. Yd.	$ 85.00	750
35.	GROUT, SLURRY				
	(a) FIRST 500 Cu. Ft.	500	Cu. Ft.	$ 22.00	500
	(b) OVER 500 Cu. Ft.	500	Cu. Ft.	$ 20.00	500
	OPTION No. 1				
36.	PORTLAND CEMENT	8,400	Cwt.	$ 7.00	8,400
37.	STEEL REINFORCEMENT	240	Ton	$700.00	240
38.	WATERSTOP	400	Lin. Ft.	$ 9.00	400

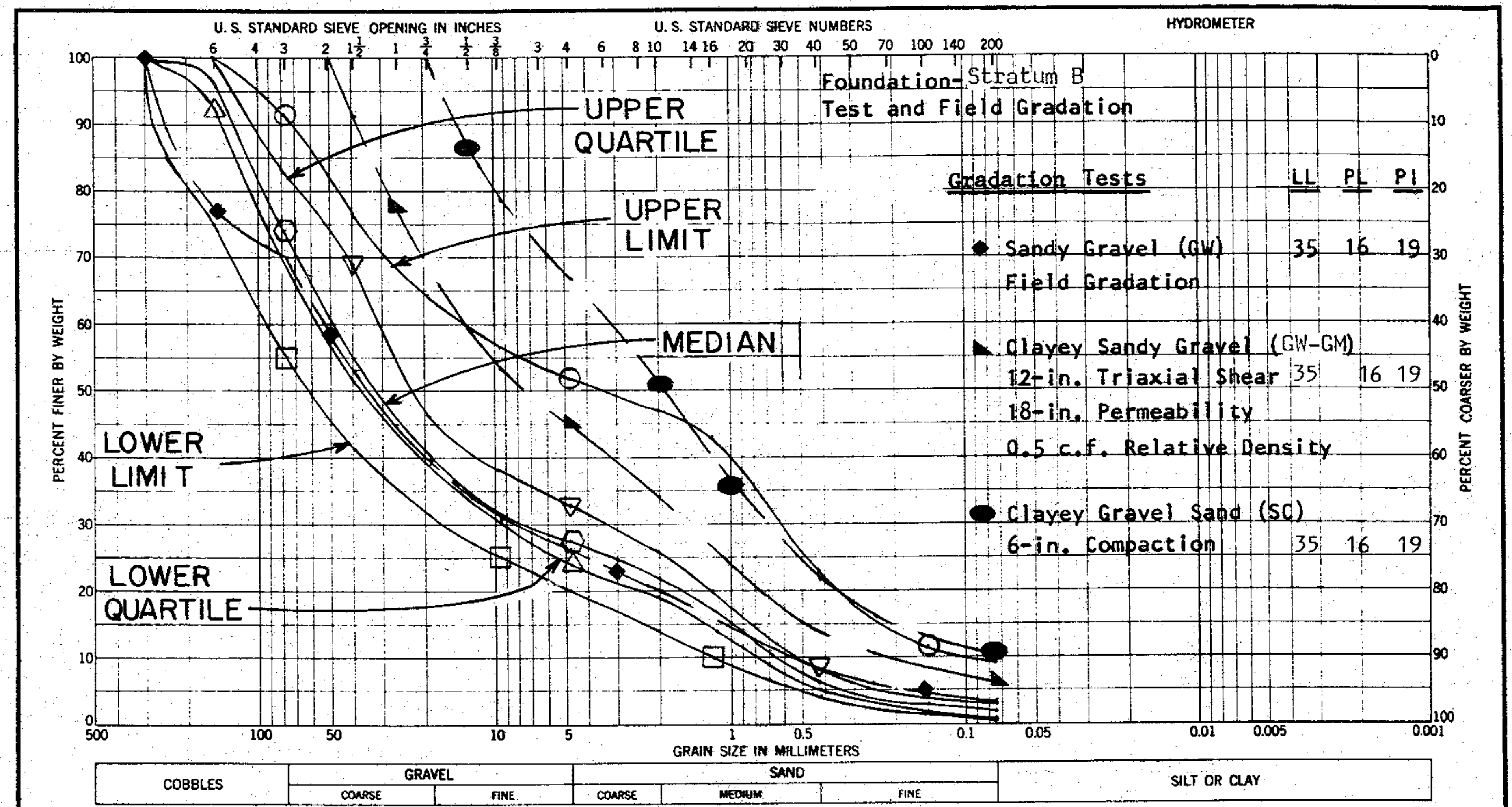
TABLE 9. (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
39.	FOUNDATION DRILLING AND GROUTING				
	(a) MOBILIZATION AND DEMOBILIZATION	1	Job	L.S.	1
	(b) DRILLING EXPLORATORY GROUT HOLES	350	Lin. Ft.	$ 25.00	350
	(c) DRILLING GROUT HOLES	9,100	Lin. Ft.	$ 8.00	9,100
	(d) PIPE FOR GROUT HOLES	500	Lin. Ft.	$ 5.00	500
	(e) DRILL SET-UPS				
	(1) GROUT HOLES	370	Each	$ 25.00	370
	(2) EXPLORATORY GROUT HOLES	4	Each	$ 50.00	5
	(f) PRESSURE TESTING	175	Hour	$ 60.00	47.5
	(g) GROUT PUMP CONNECTIONS	375	Each	$ 50.00	225
	(h) PLACING GROUT	2,500	Sack	$ 20.00	2,097.5
40.	PIPE GATE, OUTLET WORKS	1	Each	$1,500.00	6
41.	DRIVE GATE	1	Each	$1,000.00	1
42.	DOUBLE DRIVE GATE	2	Each	$1,500.00	5
43.	REINFORCED CONCRETE CULVERT, IKE NO. 1	1	Job	L.S.	1
44.	CORRUGATED METAL PIPE, 24 INCH	60	Lin. Ft.	$ 35.00	60
45.	CORRUGATED METAL PIPE, 36 INCH	60	Lin. Ft.	$ 55.00	60
46.	CORRUGATED METAL PIPE, 48 INCH	660	Lin. Ft.	$ 65.00	660
47.	METAL END SECTIONS FOR 36" CMP	2	Each	$ 300.00	2
48.	METAL END SECTIONS FOR 24" CMP	2	Each	$ 200.00	2
49.	AGGREGATE BASE, ROAD	2,400	Cu. Yd.	$ 9.00	2,353
50.	ASPHALT CONCRETE PAVEMENT	2,800	Ton	$ 35.00	1,039
51.	5' CHAIN LINK FENCE	16,000	Lin. Ft.	$ 8.00	16,512
52.	GUTTER	4,800	Lin. Ft.	$ 15.00	7,992
53.	GUARDRAIL	450	Lin. Ft.	$ 15.00	410
54.	LOG BARRIER	38	Lin. Ft.	$ 25.00	40

TABLE 9. (Continued)

Item No.	Description	Estimated Quantity	Unit	Unit Price	Actual Quantity
55.	PROJECT SIGN	1	Job	L.S.	1
56.	HYDROLOGIC FACILITIES	1	Job	L.S.	1
57.	STAFF GAGES AND MONUMENT	1	Job	L.S.	1
58.	GAGING STATION BRIDGE	1	Job	L.S.	1
59.	TOPSOILING	23,000	Cu. Yd.	$ 300.00	22,369
60.	LANDSCAPE STONE	11,300	Cu. Yd.	$ 6.00	16,283
61.	DESERT GRAVEL	4,100	Cu. Yd.	$ 15.00	4,367
62.	DESERT VARNISH FINISH	4,500	Gal	$ 20.00	4,500
63.	CONCRETE STAIN	330	Gal	$ 25.00	300
64.	SEEDING	50	Acre	$2,500.00	50
65.	PLANTING	1	Job	L.S.	1

FIGURES



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
○	Fine Limit					NP
▽	Upper Quartile					NP
⬡	Median					NP
△	Lower Quartile					NP
□	Coarse Limit					NP

GRADATION CURVES

Project NEW RIVER DAM

Gradation Limits (minus 12")

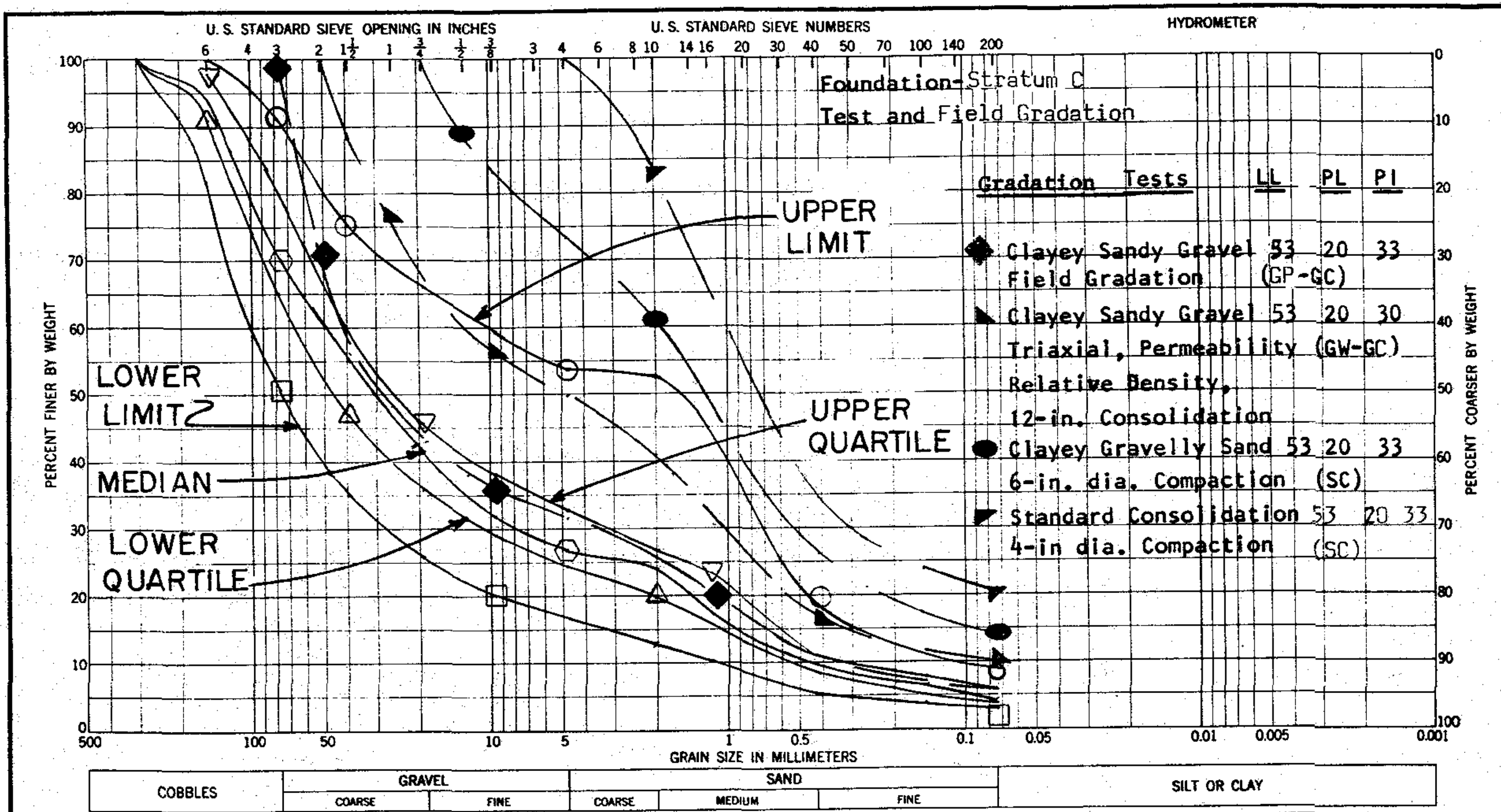
Area Foundation - Stratum B

Boring No. (Sandy Gravel)

Date

ENG FORM 1 MAY 63 2087

Figure 1. Stratum B, field and laboratory gradations - GDM



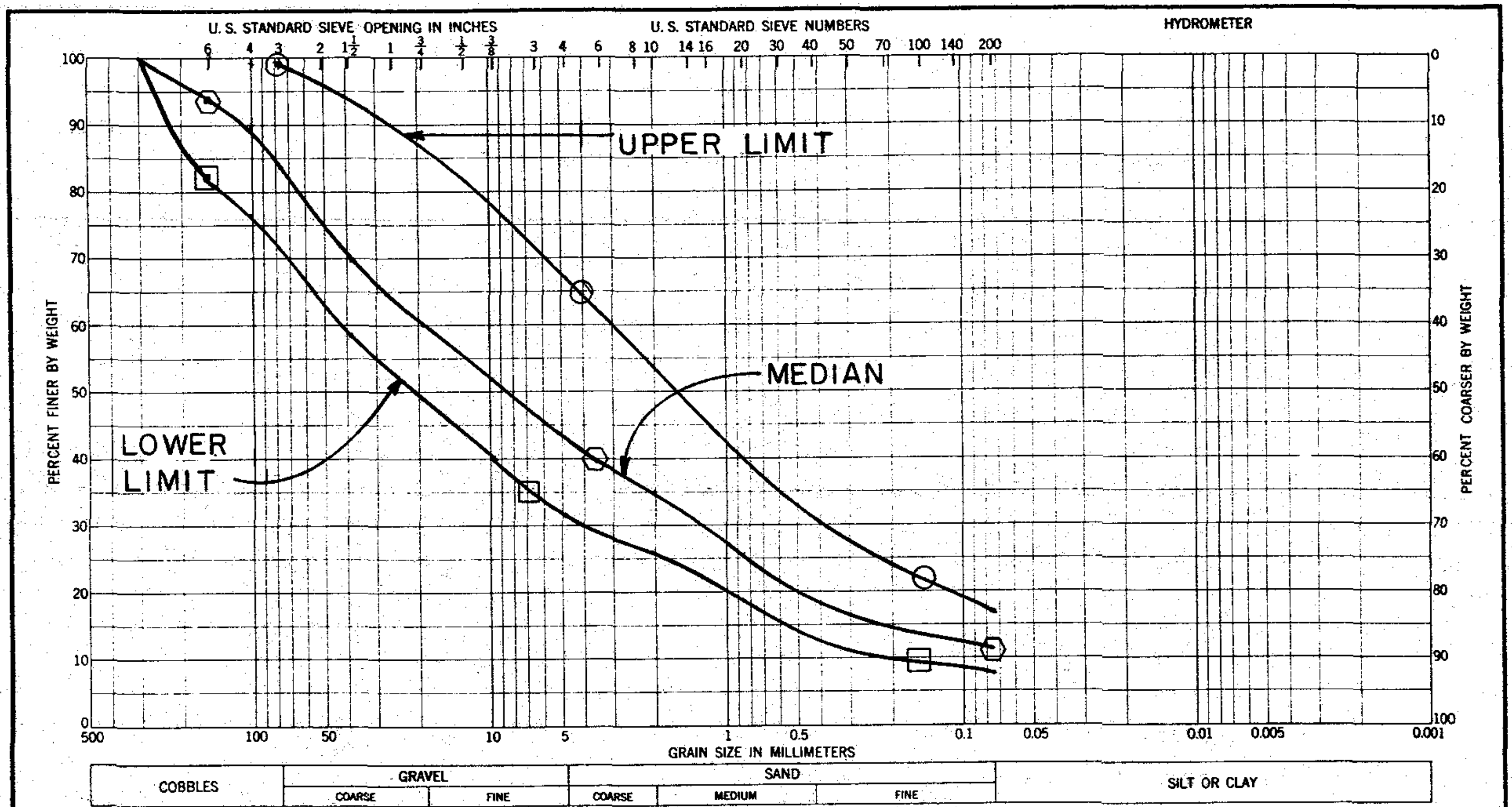
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
○	Fine Limit			75	29	46
▽	Upper Quartile			58	27	31
⬡	Median			48	25	23
△	Lower Quartile			33	23	10
□	Coarse Limit			27	23	4

GRADATION CURVES

Project: NEW RIVER DAM
Gradation Limits (Minus 12")
Area: Foundation- Stratum C
Boring No.: (Cemented Sandy Gravel)
Date:

ENG FORM 1 MAY 63 2087

Figure 2. Stratum C, field and laboratory gradations —GDM



Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
○	Fine Limit					
⬡	Median					
□	Coarse Limit					

GRADATION CURVES

Project NEW RIVER DAM

Gradation Limits (Minus 12")

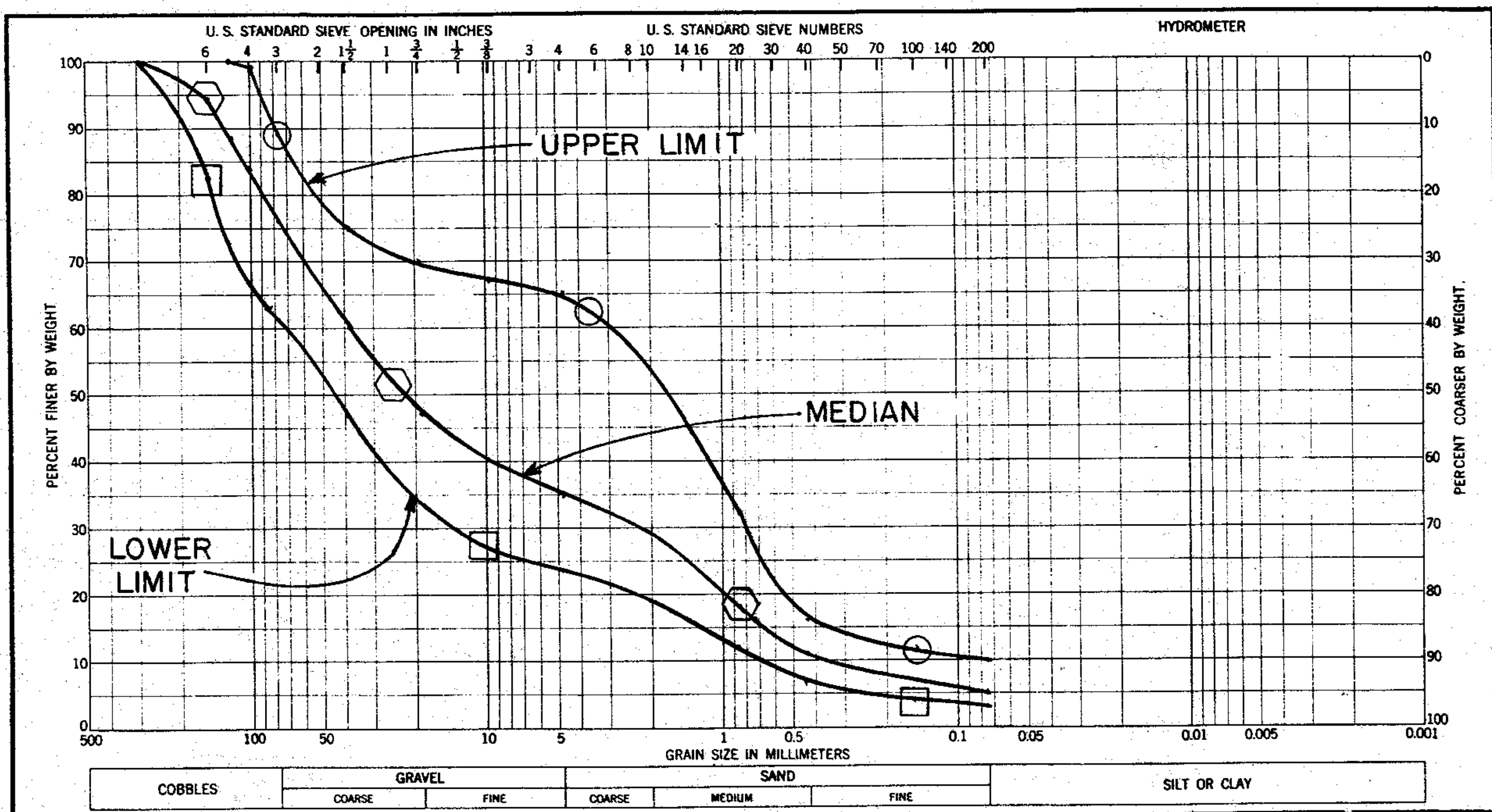
Area Core Trench—Stratum C

Boring No. As—Constructed

Date

ENG FORM 1 MAY 63 2087

Figure 3. Stratum C, field gradations —Core Foundation



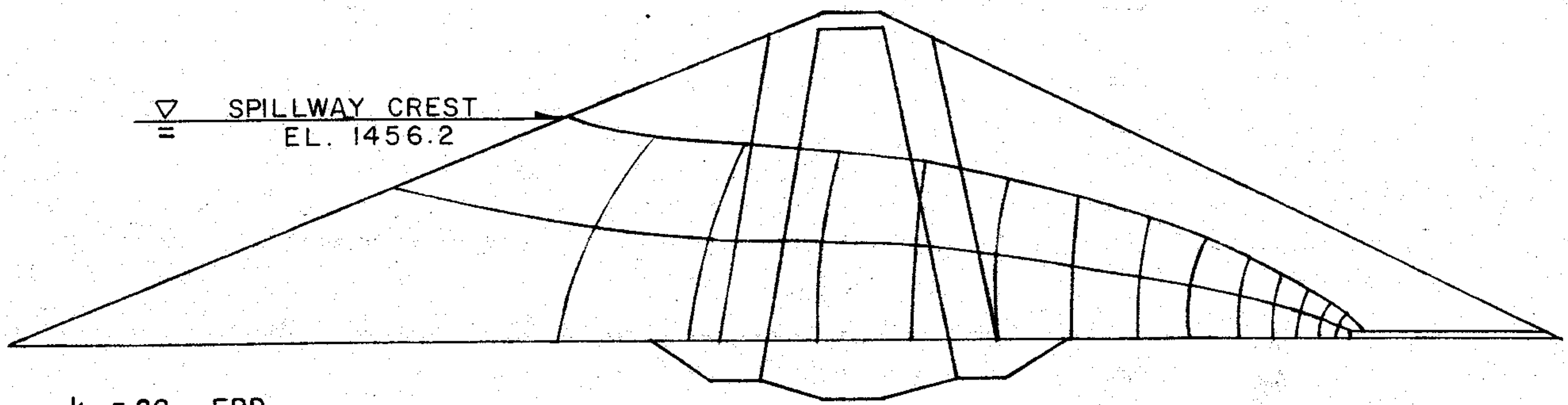
Sample No.	Elev or Depth	Classification	Nat w %	LL	PL	PI
○	Fine Limit					
⬡	Median					
□	Coarse Limit					

Project NEW RIVER DAM
Gradation Limits (Minus 12")
Area Foundation Stratum B
Boring No. As- Constructed
Date

GRADATION CURVES

ENG FORM 1 MAY 63 2087

Figure 4. Stratum B, field gradations—Shell Foundation.

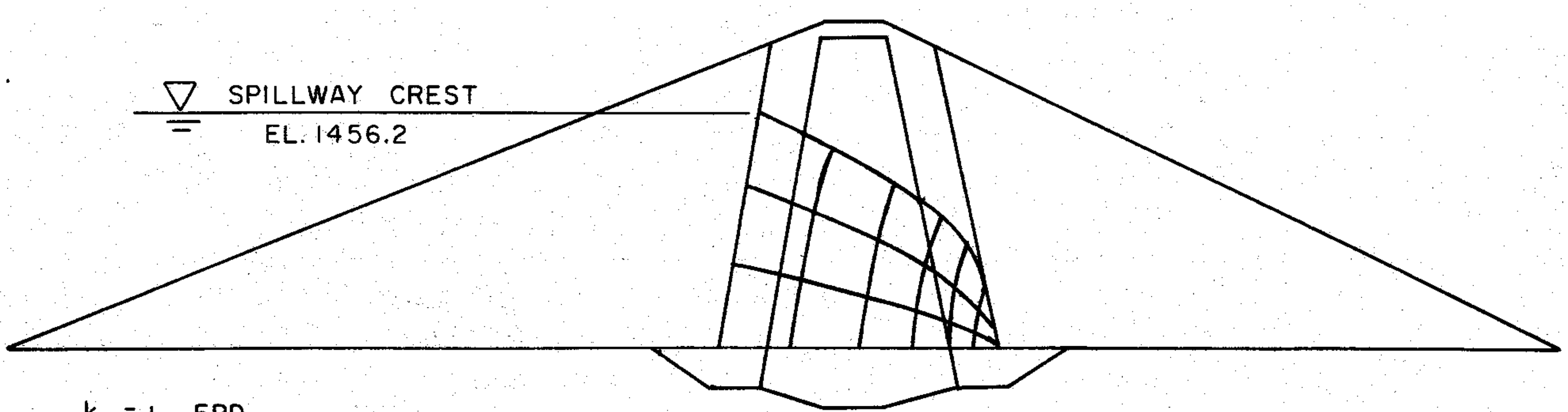


k_h = 20 FPD
k_v = 20 FPD
k_e = 20 FPD
n_f = 2
n_e = 14

$q = kH\frac{n_f}{n_e}$ = 248 CFS/FT OF EMBANKMENT LENGTH

DAM EMBANKMENT
THROUGH SEEPAGE

Figure 5 **Dam embankment through seepage, Design**



$k_h = 1$ FPD

$k_v = 1$ FPD

$k_e = 1$ FPD

$n_f = 3$

$n_e = 6$

$q = kH \frac{n_f}{n_e}$ = 48 CFS/FT OF EMBANKMENT LENGTH

Figure 6. Embankment Through Seepage, As-Constructed.

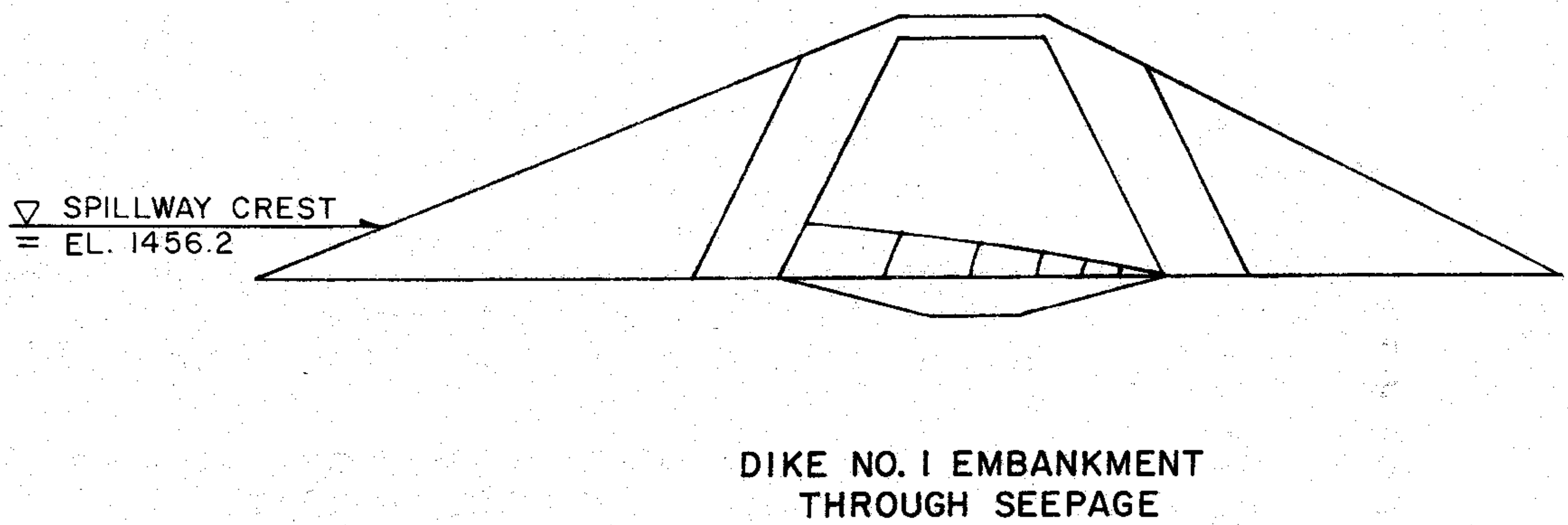


Figure 7 Dike No. 1 embankment through seepage.

PLATES

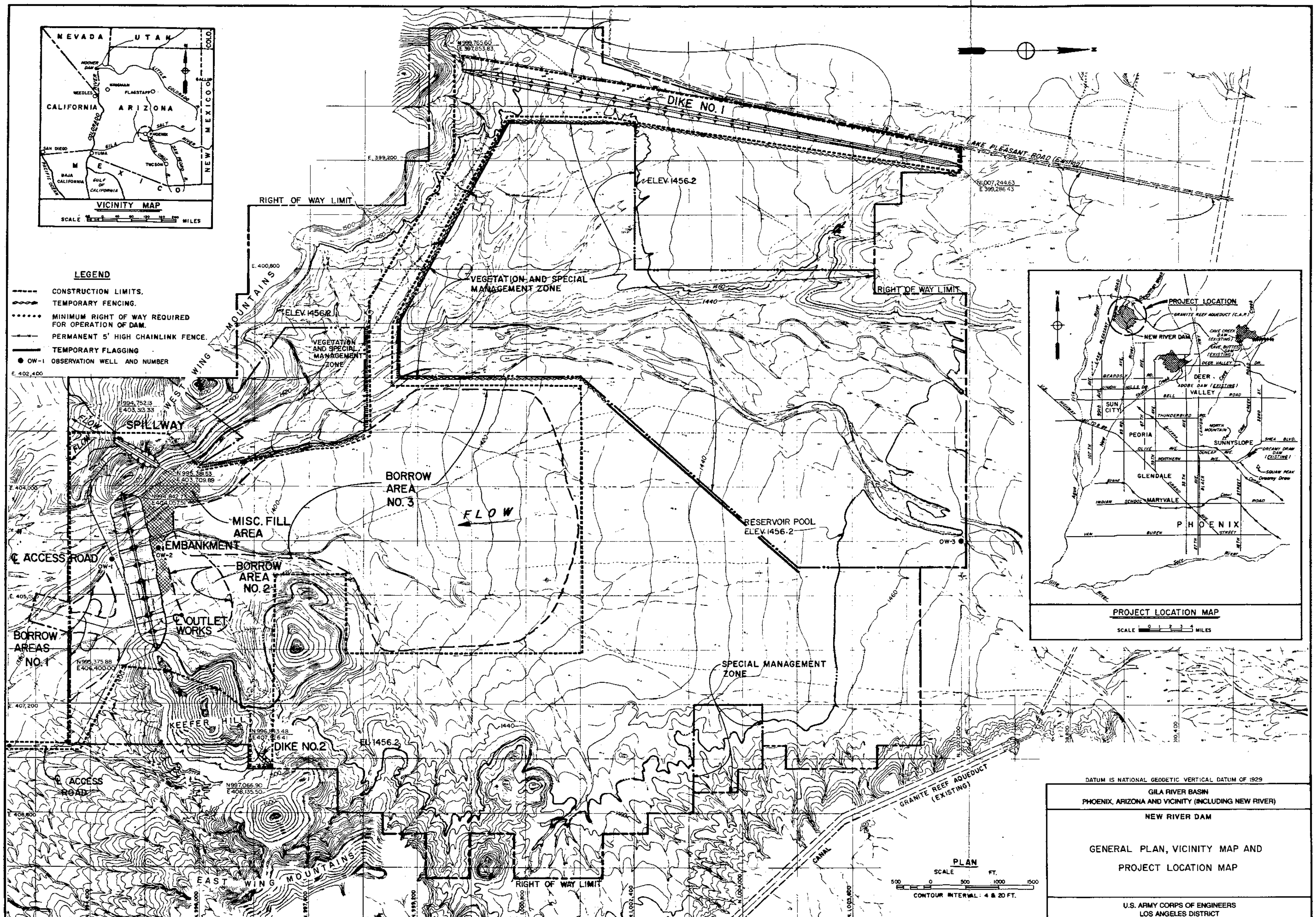
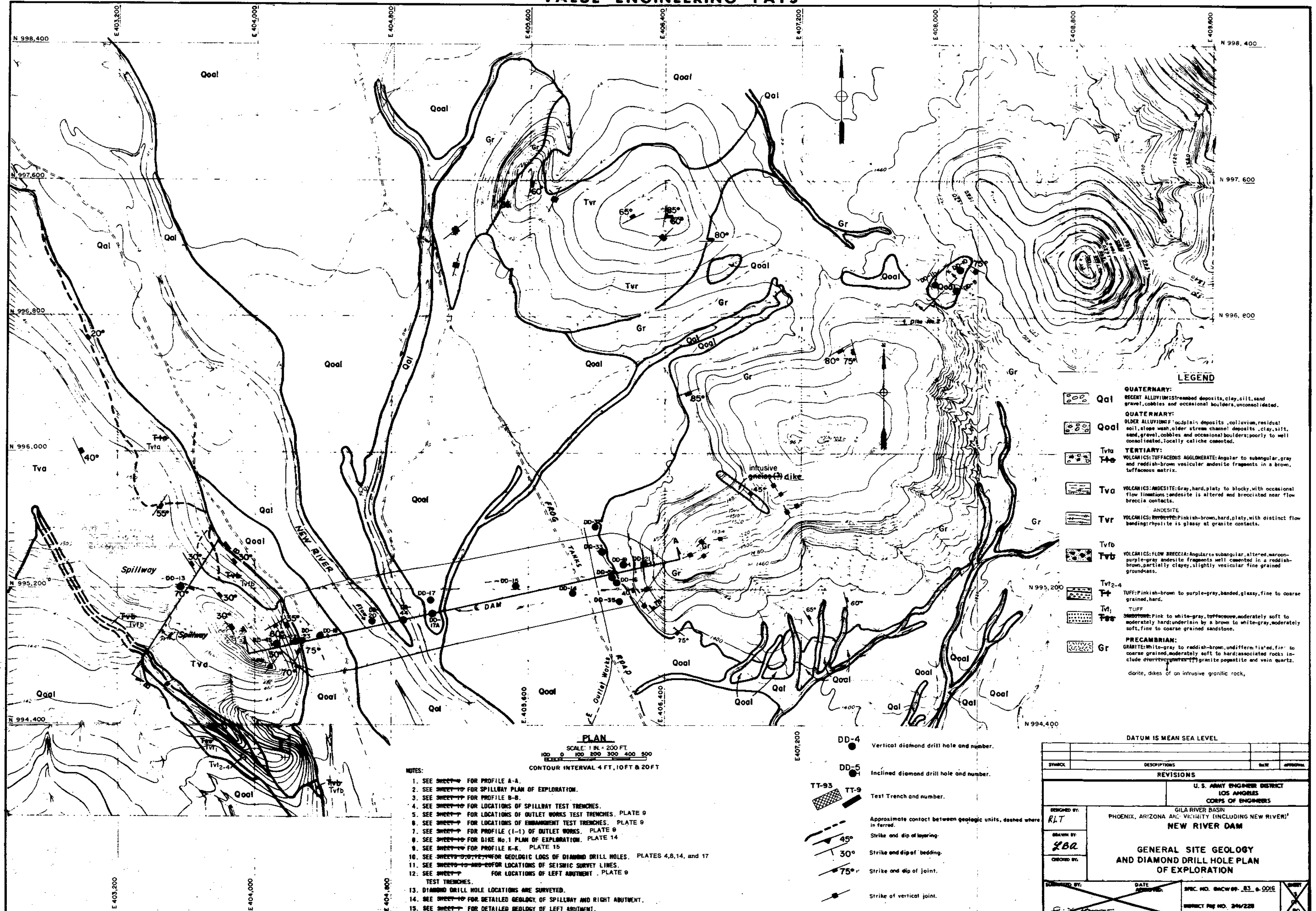

VICINITY MAP
NEVADA
UTAH
COLO.
CALIFORNIA
ARIZONA
NEW MEXICO
MEXICO
HOOVER DAM
NEEDLES
KINGMAN
FLAGSTAFF
GALLUP
PHOENIX
SAN DIEGO
YUMA
TUCSON
BAJA CALIFORNIA
GULF OF CALIFORNIA
PACIFIC OCEAN
SCALE
MILES
LEGEND
CONSTRUCTION LIMITS.
TEMPORARY FENCING.
MINIMUM RIGHT OF WAY REQUIRED FOR OPERATION OF DAM.
PERMANENT 5' HIGH CHAINLINK FENCE.
TEMPORARY FLAGGING
OW-1 OBSERVATION WELL AND NUMBER
DIKE NO. 1
LAKE PLEASANT ROAD (Existing)
N.999,765.60
E.397,853.83
N.1,007,244.63
E.399,286.43
ELEV. 1456.2
RIGHT OF WAY LIMIT
VEGETATION AND SPECIAL MANAGEMENT ZONE
WEST WING MOUNTAINS
SPILLWAY
FLOW
N.994,752.13
E.403,313.33
N.995,381.53
E.403,709.89
N.994,842.73
E.404,057.51
MISC. FILL AREA
EMBANKMENT
BORROW AREA NO. 3
BORROW AREA NO. 2
BORROW AREAS NO. 1
℄ ACCESS ROAD
℄ OUTLET WORKS
OW-1
OW-2
OW-3
RESERVOIR POOL ELEV. 1456.2
SPECIAL MANAGEMENT ZONE
N.995,375.88
E.406,400.00
KEEFER HILL
N.996,863.48
E.407,826.41
DIKE NO. 2
EL. 1456.2
N.997,066.90
E.408,135.50
EAST WING MOUNTAINS
GRANITE REEF AQUEDUCT (EXISTING)
CANAL
PROJECT LOCATION
GRANITE REEF AQUEDUCT (C.A.P.)
NEW RIVER DAM
CAVE CREEK DAM (EXISTING)
CAVE BUTTES DAM (EXISTING)
DEER VALLEY
ADOBE DAM (EXISTING)
SUN CITY
PEORIA
NORTH MOUNTAIN
SUNNYSLOPE
DREAMY DRAW DAM (EXISTING)
GLENDALE
MARYVALE
PHOENIX
PROJECT LOCATION MAP
SCALE MILES
PLAN
SCALE FT.
CONTOUR INTERVAL: 4 & 20 FT.
DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
GENERAL PLAN, VICINITY MAP AND PROJECT LOCATION MAP
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

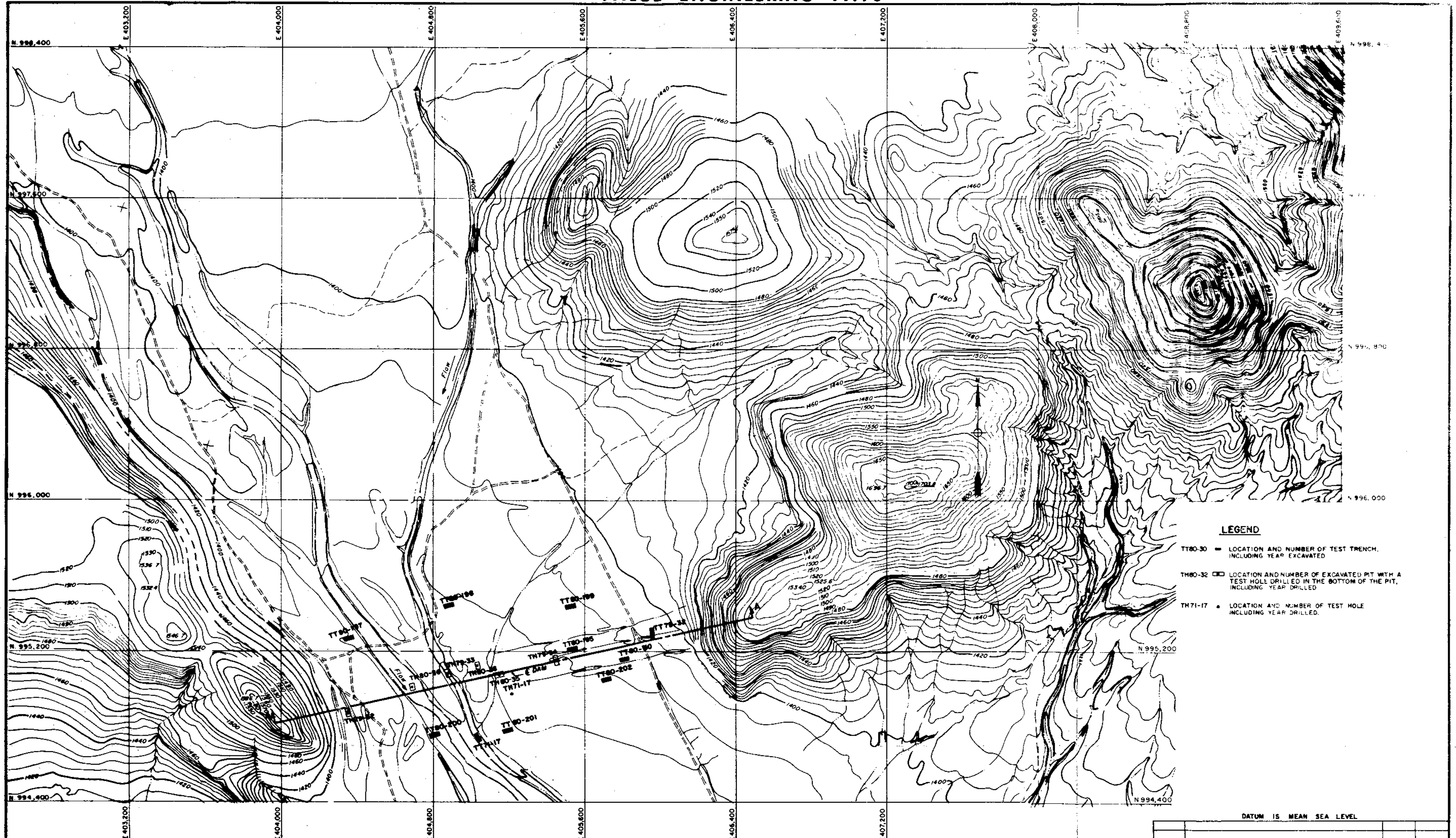
PLATE 1

VALUE ENGINEERING PAYS



SAFETY PAYS

PLATE 2



LEGEND

TT80-30 ▬ LOCATION AND NUMBER OF TEST TRENCH, INCLUDING YEAR EXCAVATED

TH80-32 ▭ LOCATION AND NUMBER OF EXCAVATED PIT WITH A TEST HOLE DRILLED IN THE BOTTOM OF THE PIT, INCLUDING YEAR DRILLED

TH71-17 • LOCATION AND NUMBER OF TEST HOLE INCLUDING YEAR DRILLED.

TEST TRENCHES

NO.	DEPTH BELOW GROUND SURFACE	DATE OF EXCAVATION	REMARKS
71-17	10'	3-71	TRENCHED TO 10 FEET BY DOZER
80-30	15'	1-80	TRENCHED TO 4 FEET BY DOZER
79-32	10'	12-79	TRENCHED TO 5 FEET BY DOZER
80-195	23'	9-80	TRENCHED TO 23 FEET BY DOZER
80-197	12'	9-80	TRENCHED TO 12 FEET BY DOZER
80-198	23'	9-80	TRENCHED TO 18 FEET BY DOZER
80-199	16'	9-80	TRENCHED TO 18 FEET BY DOZER
80-200	15'	9-80	TRENCHED TO 13 FEET BY DOZER
80-201	33'	9-80	TRENCHED TO 18 FEET BY DOZER
80-202	22'	9-80	TRENCHED TO 15 FEET BY DOZER

TEST HOLES

NO.	DEPTH BELOW GROUND SURFACE	DATE OF EXCAVATION AND DRILLING	REMARKS
71-17	10'	2-71	
79-32	26'	12-79	TRENCHED TO 13 FEET, THEN DRILLED
79-33	34'	12-79	TRENCHED TO 9 FEET, THEN DRILLED
79-34	24'	12-79	TRENCHED TO 18 FEET, THEN DRILLED
80-35	97'	1-80	TRENCHED TO 18 FEET, THEN DRILLED
80-36	62'	8-80	TRENCHED TO 13 FEET, THEN DRILLED
80-38	77'	8-80	TRENCHED TO 20 FEET, THEN DRILLED

PLAN OF EXPLORATION

100 0 100 200 300 400 500

CONTOUR INTERVAL: 4 FEET

SCALE: 1 IN. = 200 FT.

NOTES:

1. TEST HOLES WERE DRILLED BY A BUCKET-TYPE POWER AUGER. THE BUCKET DIAMETER WAS 24 INCHES.
2. TEST TRENCHES WERE EXCAVATED WITH A BACKHOE AND/OR D-8 DOZER.
3. FOR LOGS OF TEST TRENCHES AND HOLES. SEE ~~SHEETS 26 THROUGH 28~~ PLATES 5,6, and 7

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: TRI

DRAWN BY: RY ZBA

CHECKED BY: ATR

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

DAM EMBANKMENT FOUNDATION
SOILS EXPLORATION PLAN

SUBMITTED BY: DATE APPROVED:

G. L. [signature]
GEOTECHNICAL BRANCH

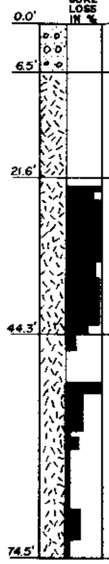
SPEC. NO. DACW09- 83 B-0016

DISTRICT FILE NO. 241/249

SHEET 2 OF 80

EMBANKMENT DD-14

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
24.8	NO TEST		
36.0	0	.04	.02
46.1	0	.04	.01
	10	0.6	0.1
	20	0.8	0.1
	30	1.5	0.2
57.2	50	10.4	0.8
57.2 - 74.5	10	6.6	0.7
66.0	20	19.7	1.5
	10	10.6	1.7
74.5	20	19.9	2.5



ELEV. 1394.9'

ALLUVIUM: silty sand to gravelly silty sand.

GRANITE: white to brown, coarse-grained, soft, highly weathered.
Top of Rock from TT-48

GRANITE: white to gray, coarse-grained moderately soft, highly weathered with moderately hard to hard lenses and nodules of fresher material; shattered

GRANITE: gray, hard, coarse-grained slightly weathered, moderately to highly fractured.

moderately soft; 44.6-45.7'.

shattered: 44.6-45.2', 57.2-57.6', 57.9-58.9'.

Joints dip primarily 40-50° and near vertical and are slightly weathered with minor rust staining.

46% Recovery January 1980

Drilled with a tri-cone rock bit from 0.0 to 19.6'; from 19.6 to 74.5' for N X sized core; N X casing set from 0.0 to 37.5'. 100% drill water return from 0.0 to 57.2' and 72.6 to 74.5'. 20% drill water loss from 57.2 to 72.6'.

EMBANKMENT DD-15

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
36.1	NO TEST		
	10	11.2	0.8
	20	17.0	0.9
	40	26.0	0.9
	70	44.3	1.1
70.1			

ELEV. 1390.7'

ALLUVIUM: silty sand to sandy gravel with 20% cobbles and occasional boulders.

GRANITE: white to gray, hard, coarse-grained, slightly to moderately weathered, highly fractured; moderately hard: 45.1-45.9', 56.7-58.0', 66.5-67.6', 68.0-70.1'.

brecciated: 24.0-24.8', 26.5-27.6', 42.6-44.5'.

brecciated and rehealed: 26.2-26.5', 56.7-58.0', 66.5-67.6', 68.0-70.1'.

void or very soft zone: 30.1-34.7'.

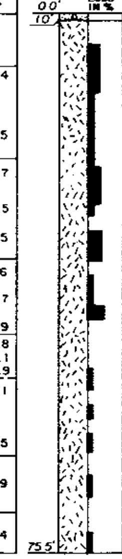
Joints dip 20° to N.V. with minor rust staining and red clay, are slightly to moderately weathered.

48% Recovery September 1980

Alluvium excavated by D-8 dozer and backhoe to 23.0'. Drilled from 23.0 to 70.1' for N W sized core. 25% drill water loss for the entire hole.

EMBANKMENT DD-16

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
6.4	NO TEST		
	10	4.8	1.4
20.5	20	8.5	1.5
	10	13.1	2.7
	20	17.7	2.5
34.2	30	23.3	2.5
	10	7.7	1.6
	20	11.3	1.7
44.4	40	19.1	1.9
40.2	10	7.1	2.8
TO	20	10.7	3.1
44.4	40	15.2	2.9
42.2	10	10.6	1.1
61.6	25	20.3	1.5
61.6 - 75.5	0	19.4	2.9
70.5			
75.5	6	24.6	6.4



ELEV. 1399.1'

ALLUVIUM: Residual soil-silty sandy gravel.

GRANITE: gray, hard, coarse-grained slightly weathered to unweathered, moderately to highly fractured to 70.0'; moderately fractured: 70.0-75.5', moderately soft to moderately hard: 1.0-4.3'.

moderately soft; highly weathered with clay gouge: 25.1-25.4'.

shattered: 8.8-9.2', 12.0-12.3', 25.1-25.4', 27.0-27.3', 27.6-28.0', 28.2-28.6', 30.0-30.4', 40.1-41.1', 42.7-43.6', 60.0-60.3', 60.7-61.2'.

brown to gray, altered, rust-stained: 6.5-6.8', 16.3-17.2'.

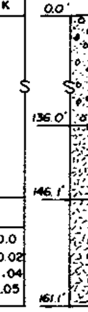
Joints dip primarily N.V. and 30°-60° and are slightly weathered to unweathered: Iron staining prevalent from 8.8 to 10.0', 16.4-20.5' and 49.8-51.0'.

89% Recovery February 1980

Drilled with a tri-cone rock bit from 0.0 to 4.3'; from 4.3 to 75.5' for N X sized core. N X casing set from 0.0 to 2.3'. 100% loss of drill water 38.3 to 38.8', 69.5 to 75.5'. 50-100% return all other intervals.

EMBANKMENT DD-17a (17)

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
146.1	NO TEST		
150.1	NO TEST		
	10	0.1	0.0
	20	0.4	0.02
	40	0.8	.04
161.1	70	1.3	.05



ELEV. 1386.3'

ALLUVIUM:

0.0-25': silty sand, sandy gravel with cobbles and boulders.

25-136.0': clayey sandy gravel to gravelly sandy clay.

encountered water at 70' depth in DD-17

GRANITE: white to gray, moderately soft to moderately hard, moderately to highly weathered, highly fractured.

shattered: 146.1-148.0'.

Joints dip generally 30-50°, mostly parallel with some intersecting 70° to near vertical, are moderately to highly weathered with red and green clay and some slickensides.

estimate top of rock at 136' from drill action and cuttings.

91% Recovery January 1981

DD-17A: Drilled with a tri-cone rock bit from 0.0-146.1'; from 146.1-161.1' for N W sized core. 100% drill water return 0.0 to 161.1'.

DD-17: Trenched to 20' by dozer, drilled from 20 to 77.0' with a power auger and from 77.0' to 104.0' with a tri-cone rockbit. No bedrock encountered.

EMBANKMENT DD-44

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
	NO TEST		
106.0			

ELEV. 1387.6'

ALLUVIUM:

0.0-16.0': silty sand, gravelly silty sand with layers of cobbles and boulders.

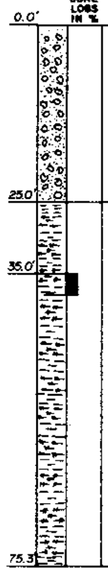
16.0-106.0': gravelly sandy clay to clayey sandy gravel.

January 1981

Drilled with a tri-cone rock bit and heavy revert mud mix from 0.0 to 106.0'. No bedrock encountered.

EMBANKMENT DD-18

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
27.0	NO TEST		
35.0	10	0.0	0.0
	20	1.0	0.1
47.9	30	1.5	0.1
	10	1.1	0.05
	20	2.7	0.1
	40	7.0	0.3
75.3	60	9.8	0.3



ELEV. 1388.8'

ALLUVIUM: silty sand, gravelly silty sand with layers of cobbles and boulders.

ANDESITE: gray, hard, moderately fractured.

moderately hard to hard: 25.0-35.0'.

shattered: 38.1-39.1'.

Joints dip primarily 40°-70° and are usually clay lined, slightly weathered with occasional slickensides and minor calcite filling.

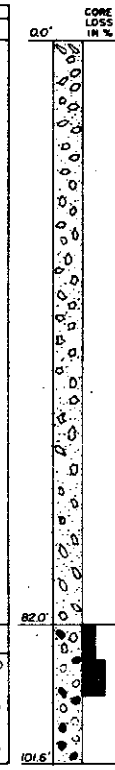
Possible shear zone: 40.7-40.9'.
(brown clay gouge with breccia dipping 60°.)

99% Recovery June 1980

Drilled with a tri-cone rock bit from 0.0 to 35.0'; from 35.0 to 75.3' for NW sized core. 100% drill water return 0.0 to 75.3'.

EMBANKMENT DD-20

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
82.0	NO TEST		
85.6	NO TEST		
	20	0.0	0.0
	40	0.5	.03
101.6	60	0.7	.03



ELEV. 1377.8'

ALLUVIUM: sandy gravel, clayey sandy gravel. 20-30% cobbles and boulders to 16' depth.

Encountered water at 39' depth.

PLATE 5
NOTE: SEE ~~SHEET 26~~ FOR SOIL LOG, TH 80-36 OF THIS HOLE, 0.0 TO 82.0'.

TUFFACEOUS AGGLOMERATE: hard, with angular to subangular gray andesite fragments up to 1" in diameter and reddish brown clasts of vesicular andesite up to 2" in diameter in a brown, tuffaceous matrix, slightly fractured by two 25° clean jts at 91.6 and 91.8', unweathered to slightly weathered.

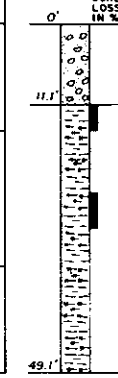
79% Recovery September 1980

Trenched to 13' by dozer, drilled from 13' to 82.0' with a power auger and from 82.0' to 101.6' for NW sized core.

100% drill water return for entire hole.

EMBANKMENT DD-23

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
15.1	NO TEST		
	10	5.9	1.0
	20	8.8	1.0
34.1	30	9.8	0.8
	10	1.2	0.2
	30	6.4	0.5
49.1	50	13.3	0.8



ELEV 1389.3'

ALLUVIUM: silty sandy gravel and slopewash.

ANDESITE: gray, hard, moderately to highly fractured.

shattered: 11.3-12.3', 29.1-31.0', 33.7-34.1', 32.3-32.7'.

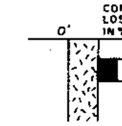
Joints dip 20°-70° and are slightly weathered with minor clay filling, no rust-staining.

98% Recovery September 1980

Drilled from 0.0 to 49.1' for N W sized core. 100% drill water return. 0.0 to 14.8' and 24.1 to 34.1'; 25% loss, 14.8 to 24.1' and 34.1 to 49.1'.

LEGEND

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
6.0	NO TEST		
11.4	5	4.0	4.3



WATER PRESSURE TEST:
Pressure testing was accomplished with a single packer.
DEPTH: The depth (in feet) at which the packer was set.
PRESSURE: Water pressure in pounds per square inch (PSI).
Q: Average water loss in gallons per minute for the interval tested.
K: Permeability in feet/day for the interval tested.
NO TEST: Interval not tested.
GRAVITY: Packer not used during testing.

CORE LOSS: The % of core loss for each interval drilled in bedrock is shown by the bar graph. A completely shaded area represents 100% core loss and an unshaded area represents 0% core loss.
Stratigraphic symbol to left of core loss column represents the geologic unit described on the log.

CLASSIFICATION OF FRACTURES:
Shattered rock: 1 inch fracture spacing or less.
Highly fractured rock: 1 to 4 inch fracture spacing.
Moderately fractured rock: 4 to 12 inch fracture spacing.
Slightly fractured rock: over 12 inch fracture spacing.

HARDNESS DEFINITIONS:
Soft = little, if any, of core remains intact; very friable.
Moderately soft = most of core remains intact, can be manually broken.
Moderately hard = core can be broken with light hammer blow.
Hard = core can be broken with moderate to heavy hammer blow.

NOTES:
1. SEE ~~SHEET 3~~ PLATE 2 FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION.
2. SEE ~~SHEET 4~~ FOR DAM EMBANKMENT FOUNDATION GEOLOGIC PROFILE.
3. SEE ~~SHEET 20~~ PLATE 5 FOR SOILS LOG OF TH80-36.
4. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED.
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.

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

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: RLT
DRAWN BY: M.P.
CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM.
DAM EMBANKMENT
GEOLOGIC LOGS OF DIAMOND DRILL HOLES

SUBMITTED BY: DATE APPROVED:
CHIEF, GEOTECHNICAL BRANCH

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/230
SHEET 6 OF 80

TH80-35

EL. 1388±

Depth	LOG	MC	LL	PI	-4	-200	N	DESCRIPTION
7.0′	SM		20	3	99	41		SILTY SAND; light brown, dry, loose
			20	2	98	39		
18.0′	GW				+			SANDY GRAVEL, brown, dry, dense, 10-20% cobbles, 10-20% boulders to 18".
					25	2		
	GP				+ 17	5		GRAVEL/CLAYEY GRAVEL: reddish brown, moist, dense, 15% cobbles to 10"
24.0′	GC		29	9	30	5		SANDY GRAVEL/CLAYEY SANDY GRAVEL
27.0′	GW GC		73	49	56	12		
30.0′	GW GM		77	39	48	10		SANDY GRAVEL/SILTY SANDY GRAVEL.
33.0′	GC		76	50	53	13		CLAYEY SANDY GRAVEL
36.0′	GP GC		61	30	46	10		SANDY GRAVEL/CLAYEY SANDY GRAVEL
39.0′	GW GC		62	47	37	12		
42.0′	GC		70	47	45	17		CLAYEY SANDY GRAVEL.
44.0′	GP GC		71	42	27	9		SANDY GRAVEL/CLAYEY SANDY GRAVEL.
48.0′	GC		58	30	46	15		CLAYEY SANDY GRAVEL; dark brown, cohesive.
51.0′	GP GC		55	39	41	11		SANDY GRAVEL/CLAY SANDY GRAVEL, reddish brown, cemented, 5-10% cobbles to 10".
54.0′	GC		72	50	51	18		CLAYEY SANDY GRAVEL; dark brown, cemented, 5-10% cobbles.
57.0′	GW GC		46	24	37	11		SANDY GRAVEL/CLAYEY SANDY GRAVEL; dark brown.
60.0′	GM		44	17	52	20		SILTY SANDY GRAVEL; dark brown, 5-10% cobbles.
63.0′	GW GM				43	12		SANDY GRAVEL/SILTY SANDY GRAVEL; reddish brown, few cobbles to 8".
66.0′	SM				66	19		SILTY GRAVELLY SAND; reddish brown
70.0′			NS					CORE BARREL, FLIGHT AUGER, BUCKET AUGER and GAD USED- due to a nest of boulders.
72.0′	SC		34	12	62	16		CLAYEY GRAVELLY SAND; reddish brown, 5-8% cobbles to 7".
75.0′	GP GM				50	8		SANDY GRAVEL/SILTY SANDY GRAVEL; reddish brown
78.0′	GW GC		43	21	42	12		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, cemented.
W 84.0′	GP GC		43	22	46	12		
88.0′	GM				49	13		SILTY SANDY GRAVEL; reddish brown, 5% cobbles.
90.0′	SM				80	38		GRAVELLY SILTY SAND; dark brown, wet, 5-6% cobbles.
	GM				48	14		SILTY SANDY GRAVEL, dark brown.
96.0′			37	10	58	28		
97.0′			NS					DRILLING stopped due to nest of boulders.

TH80-36

EL. 1384±

Depth	LOG	MC	LL	PI	-4	-200	N	DESCRIPTION
6.0′	GW GM			NP	30	5		SANDY GRAVEL/SILTY SANDY GRAVEL, brown grey, dry, dense, 20-30% subrounded cobbles and boulders to 14"
13.0′	GP			NP	+ 34	4		SANDY GRAVEL, reddish brown, moist, dense, 20-25% subrounded cobbles to 10"
16.0′	GP GC		58	37	+ 43	12		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, moist, dense, 20-25% subrounded cobbles to 10"
19.0′	SC		44	18	84	26		GRAVELLY CLAYEY SAND; grey, subrounded gravel
22.0′	SM			NP	73	16		SILTY GRAVELLY SAND
25.0′	GW GM		38	6	47	9		SANDY GRAVEL/SILTY SANDY GRAVEL.
28.0′	SW SM			NP	57	11		GRAVELLY SAND/SILTY GRAVELLY SAND
38.0′			NS					
W 39.0′	SW SM			NP	60	10		GRAVELLY SAND/SILTY GRAVELLY SAND, brown, perched water.
43.0′				NP	61	10		wet
47.0′	GP			NP	19	3		SANDY GRAVEL; brown, auger speed alternates between fast and slow as if drilling through alternating soft and hard soil layers.
	SW SM			NP	59	6		GRAVELLY SAND/SILTY GRAVELLY SAND
53.0′				NP	72	12		
56.0′	GW GM			NP	51	7		SANDY GRAVEL/SILTY SANDY GRAVEL.
59.0′	SW SM			NP	69	6		GRAVELLY SAND/SILTY GRAVELLY SAND
60.0′	SC		30	83	87	50		CLAYEY SAND; white
63.0′	SP SM		40	14	57	10		GRAVELLY SAND/SILTY GRAVELLY SAND.
66.0′	SM			NP	63	15		SILTY GRAVELLY SAND.
67.0′	GP GM			NP	52	11		SANDY GRAVEL/SILTY SANDY GRAVEL.
70.0′	SM			NP	79	22		GRAVELLY SILTY SAND.
73.0′	GW GM			NP	51	6		SANDY GRAVEL/SILTY SANDY GRAVEL.
76.0′	GP GM			NP	50	11		
80.0′	SM			NP	59	14		GRAVELLY SILTY SAND.

REMARKS. 60-80' depth, change in auger speed implies drilling through alternating soft and hard soil layers.

TH80-38

EL. 1389±

Depth	LOG	MC	LL	PI	-4	-200	N	DESCRIPTION
	SM			NP	95	25		SILTY SAND, light brown, dry, loose
6.0′				NP	94	34		
14.0′	* GP		NS					SANDY GRAVEL, light brown, dry, dense, 20% subrounded cobbles to 8"
20.0′	SP SC		33		99	6		SAND /CLAYEY SAND: reddish brown, moist, medium dense.
23.0′	GM		71	36	52	13		SILTY SANDY GRAVEL, reddish brown, moist, dense, cemented, 20% subrounded cobbles and boulders to 15"
26.0′	GW GC		48	21	39	9		SANDY GRAVEL/CLAYEY SANDY GRAVEL.
	SM		42	10	60	17		SILTY GRAVELLY SAND
32.0′			53	22	63	17		few boulders to 24"
35.0′	GW GC		61	30	31	8		SANDY GRAVEL/CLAYEY SANDY GRAVEL. 5% cobbles to 4"
38.0	GC		60	31	54	21		CLAYEY SANDY GRAVEL
41.0′	GP GM		69	26	40	12		SANDY GRAVEL/SILTY SANDY GRAVEL.
	GM		62	30	51	19		SILTY SANDY GRAVEL.
47.0′			65	29	45	16		
50.0′	GC		NS					CLAYEY SANDY GRAVEL, dark brown, moist, dense, subrounded gravel and sand.
	GM		68	35	39	16		SILTY SANDY GRAVEL, 5% cobbles to 8".
56.0′			58	27	37	15		
57.0′	GC		66	35	40	17		CLAYEY SANDY GRAVEL
58.0′	GM		69	34	38	17		SANDY GRAVEL/SILTY SANDY GRAVEL, reddish brown, moist, dense.
59.0′	GC		66	35	39	15		CLAYEY SANDY GRAVEL, dark brown, moist, dense.
62.0′	SC		63	33	67	23		CLAYEY GRAVELLY SAND, dark brown, moist, subrounded gravel and sand.
65.0′	GM		53	21	41	13		SILTY SANDY GRAVEL
68.0′	GP GC		51	26	34	12		SANDY GRAVEL/CLAYEY SANDY GRAVEL.
W 70.0′ / 71.0′	GP GM		58	25	35	12		SANDY GRAVEL/SILTY SANDY GRAVEL, water needed at 71' to prevent caving
74.0′	GM		59	27	37	16		SILTY SANDY GRAVEL
77.0′	GP GC		40	18	29	8		SANDY GRAVEL/CLAYEY SANDY GRAVEL

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than no. 200 sieve size.	GRAVELS More than half of coarse fraction is larger than no. 4 sieve size.	Clean gravels	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS More than half of coarse fraction is smaller than no. 4 sieve size.	Clean sands	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly-graded sands, gravelly sands, little or no fines.
		Sands with fines	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS More than half of material is smaller than no. 200 sieve size.	SILTS AND CLAYS	Low liquid limit. (<50)	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
		High liquid limit. (≥50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
Highly organic soils			Pt	Peat and other highly organic soils.

NOTES:

1. Boundary Classification: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example, GW-GC, well-graded gravel-sand mixture with clay binder.
2. All sieve sizes on this chart are U. S. Standard.
3. The terms "silt" and "clay" are used respectively to distinguish materials exhibiting lower plasticity from those with higher plasticity. The minus no. 200 sieve material is silt if the liquid limit and plasticity index plot below the "A" line on the plasticity chart (Table VI, Military Standard 619B), and is clay if the liquid limit and plasticity index plot above the "A" line on the chart.
4. For a complete description of the Unified Soil Classification System, see "Military Standard 619B" dated 20 March 1976

LEGEND

TT 80-30	LOCATION AND NUMBER OF TEST TRENCH.
TH 80-35	LOCATION AND NUMBER OF TEST HOLE.
MC	FIELD MOISTURE CONTENT IN PERCENT OF DRY WEIGHT.
LL	LIQUID LIMIT.
PI	PLASTICITY INDEX (LIQUID LIMIT MINUS PLASTIC LIMIT).
NP	NONPLASTIC
-4	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 4 SIEVE.
-200	PERCENT OF MATERIAL BY WEIGHT PASSING NO. 200 SIEVE.
N	NUMBER OF BLOWS OF A 140-POUND DROPHAMMER FALLING 30 INCHES REQUIRED TO DRIVE A SAMPLING SPOON ONE FOOT. OUTSIDE DIAMETER OF SPOON IS 2 INCHES; INSIDE DIAMETER IS 1-3/8 INCHES. PROCEDURE IS CALLED STANDARD PENETRATION TEST.
W	DEPTH TO WATER
Exf	MATERIALS TO BE EXCAVATED FROM THE FOUNDATION.
C	BLENDED MATERIAL SUITABLE FOR CORE.
PS	BLENDED MATERIAL SUITABLE FOR PERVIOUS SHELL.
NS	NO SAMPLE.
*	VISUAL CLASSIFICATION.
+	MINUS 12-INCH GRADATION SAMPLE.
++	MINUS 6-INCH GRADATION SAMPLE.

NOTES:

1. SEE ~~SHEET 24~~ PLATE 3 FOR LOCATION OF TEST HOLES AND TRENCHES.
2. PERCENTAGE OF COBBLES, BOULDERS AND MAXIMUM SIZES WHERE INDICATED WERE VISUALLY ESTIMATED.
3. ALL GRADATIONS EXCEPT THOSE INDICATED WERE PERFORMED ON MINUS 3-INCH SAMPLES.
4. DATESOF DRILLING: JANUARY 1971-APRIL 1971, DECEMBER 1979-FEBRUARY 1980, AND AUGUST 1980-SEPTEMBER 1980.

VERT. SCALE: 1 IN. = 5 FT.

5 0 5 10 15

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: TRJ.
DRAWN BY: F.W.
CHECKED BY: ATR

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DAM EMBANKMENT FOUNDATION SOIL LOGS

SUBMITTED BY: ... GEOTECHNICAL BRANCH
DATE APPROVED:

SPEC. NO. DACW 09-83-B-0016
DISTRICT FILE NO. 241/251
SHEET 26 OF 80

TT 80-30

EL. 1395'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
5.0'	SM			NP	92	22		SILTY SAND; light brown, dry, loose.
9.5'	ML		22	2	100	53		SANDY SILT; cemented by caliche.
15.0'	GP		NS					SANDY GRAVEL; light brown, 10-15% cobbles, 10-15% boulders.

TT 80-195

EL. 1393'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
6.0'	SM SC		23	6	80	41		GRAVELLY SILTY SAND/GRAVELLY CLAYEY SAND; light brown, dry, loose, caliche lenses at 4-5' depth.
12.0'	GP			NP	+14	1		GRAVEL; whitish, dry, very dense, caliche, 20% subrounded cobbles to 10".
20.0'	GW		41	18	+24	2		SANDY GRAVEL; reddish brown, dry, very dense 5% boulders to 14".
23.0'	GP GC		69	42	+20	6		GRAVEL/CLAYEY GRAVEL; black, wet, medium dense, 15% subrounded cobbles to 6".

TT 80-197

EL. 1389'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
2.0'	* SP		NS					SAND; grey, dry, very loose, subrounded fine sand.
	* GP		NS					SANDY GRAVEL; grey, dry, loose, 40% subrounded gravel, 10% subrounded cobbles to 8".
10.0'			29	8	+35	3		reddish brown, moist, medium dense, 20% subrounded cobbles to 8".
12.0'	GW		32	14	+20	3		damp, 20% subrounded cobbles to 10", perched water at 11'.

TT 80-198

EL. 1388'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
3.0'	* SM		NS					SILTY SAND; light brown, dry, loose.
	GW			NP	+28	3		SANDY GRAVEL; grey, dry, dense, 15% subrounded cobbles to 5".
15.0'				NP	+27	3		brown, damp, 40% gravel, 15% subrounded cobbles to 6".
18.0'	GP		32	10	+27	3		reddish brown, dense.
21.0'	SP SC		75	46	91	11		SAND/CLAYEY SAND; reddish brown, damp, dense.
23.0'	GW GC		45	26	36	8		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, damp, dense, 5% subrounded cobbles to 4", perched water at 22'.

TT 80-199

EL. 1395'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
	SM		22	2	94	40		SILTY SAND; light brown, dry, loose.
8.0'			21	1	80	34		GRAVELLY SILTY SAND; caliche.
	GW			NP	+28	1		SANDY GRAVEL; light brown, dry, dense, caliche, 10% subrounded cobbles to 12".
			NS					
				NP	+24	2		
20.0'			27	4	+24	3		reddish brown, damp, 15% subrounded cobbles to 8".

TT 80-200

EL. 1388'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
7.0'	* SM		NS					SILTY SAND; light brown, dry, loose.
13.0'	SP			NP	+53	3		GRAVELLY SAND; grey, dry, loose, 15% subrounded cobbles to 12".
15.0'	GP GC		49	28	+31	5		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, moist, dense, 20% subrounded cobbles to 10".

TT 80-201

EL. 1387'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
6.0'	SM			NP	98	33		SILTY SAND; tan, dry, loose.
13.0'	GP GM			NP	+52	7		SANDY GRAVEL/SILTY SANDY GRAVEL; light brown, dry, dense, 10% cobbles to 8".
16.0'	GP			NP	+23	1		SANDY GRAVEL.
	* GP GC		NS					SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, moist, dense, 30% subrounded cobbles to 6".
22.0'			48	23	+33	5		
25.0'	GW GC		69	37	34	7		
28.0'	SC		59	32	99	18		CLAYEY SAND; reddish brown, damp, dense.
31.0'	GP GC		45	25	47	7		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, damp, dense, 7% subrounded cobbles to 8".
33.0'	GP GM		52	21	48	7		SANDY GRAVEL/SILTY SANDY GRAVEL.

TT 80-202

EL. 1391'±

Depth	LOG	MC	LL	PI	-4	-200	N	Description
6.0'	SM			NP	95	37		SILTY SAND; light brown, dry loose.
10.0'	GP GC		30	8	+38	8		SANDY GRAVEL/CLAYEY SANDY GRAVEL; light brown, dry, dense, caliche, matrix 5% subrounded cobbles to 5".
	GW			NP	+28	2		SANDY GRAVEL; light brown, dry, dense, 5% subrounded cobbles to 5".
15.0'				NP	+27	1		
18.0'	GP GC		54	29	+25	7		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, moist, dense, 5% subrounded cobbles to 4".
22.0'	GP		53	31	47	4		SANDY GRAVEL; 5% subrounded cobbles to 4".

Bedrock; granite.

VERT. SCALE: 1 IN. = 5 FT.
5 0 5 10 15

NOTES:
1. SEE ~~SHEET 24~~ PLATE 3 FOR LOCATION OF TEST HOLES AND TEST TRENCHES.
2. SEE ~~SHEET 26~~ PLATE 5 FOR LEGEND, NOTES, AND BASIS OF CLASSIFICATION.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: TRI
DRAWN BY: J. F. B.
CHECKED BY: ATR

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DAM EMBANKMENT FOUNDATION
SOIL LOGS

SUBMITTED BY: ~~O. X.~~ CHIEF GEOTECHNICAL BRANCH
DATE APPROVED:

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/252
SHEET 27 OF 80

PLATE 6

TH 79-32

EL 1388'±	LOG	MC	LL	PI	-4	-200	N	Description
3.0'	SM				100	50		SILTY SAND; light brown, dry, loose.
13.0'	GP				+28	1		SANDY GRAVEL; light brown, dry, loose, 10% cobbles to 12".
24.0'	GP GC				+37	9		SANDY GRAVEL/CLAYEY SANDY GRAVEL; reddish brown, moist, 15% cobbles to 10".
26.0'			NS					BEDROCK; andesite, grey, fractures of ± 2" intervals.

TH 79-33

EL 1384'±	LOG	MC	LL	PI	-4	-200	N	Description
2.0'	SP				63	1		GRAVELLY SAND; grey brown, dry, loose, 5% cobbles to 6".
	GP				+20	1		SANDY GRAVEL; light brown, dry, dense, 10-15% cobbles to 12".
24.0'	GP GC		66	39	+42	11		reddish brown, moist, dense, 5-10% boulders to 24", cemented.
34.0'								decomposed granite boulders.

bucket auger refusal at 34'.

TH 79-34

EL 1391'±	LOG	MC	LL	PI	-4	-200	N	Description
	SM SC		24	4	94	35		SILTY SAND/CLAY SAND; light brown, dry loose.
6.0'	SM SC		24	6	100	46		brown, cemented.
9.0'	SM		NS					SILTY SAND; whitish brown, caliche.
	GP				+24	2		SANDY GRAVEL; 10-15% boulders to 24", caliche coating on boulders
18.0'	GP				+32	4		light brown, dry, dense, 10-15% boulders to 15".
24.0'	GP GC		76	49	+34	8		SANDY GRAVEL/CLAYEY SANDY GRAVEL, reddish brown, moist, dense, 1-5% boulders to 15"
25.0'			NS					BEDROCK; granite, bucket auger refusal at 25'. Pit dug in bottom of trench 6'x 5', granite bedrock exposed at 24'.

TH 71-17

EL 1390'±	LOG	MC	LL	PI	-4	-200	N	Description
3.0'	ML	5.8		NP	99	51		SANDY SILT; tan brown, medium dense.
5.0'	SM	5.2		NP	91	41		SILTY SAND.
8.0'	GC	3.8	27	12	48	16		CLAYEY SANDY GRAVEL; medium dense, 1% boulders to 16".

TT 79-32

EL 1399±	LOG	MC	LL	PI	-4	-200	N	Description
3.5'	SM SC		20	4	91	29		SILTY SAND/CLAYEY SAND; light brown, loose.
6.0'	SC		26	9	92	35		CLAYEY SAND; dark brown, cemented.
10.0'			NS					BEDROCK; granite, decomposed, fractured and jointed.

Backhoe refusal at 10'.

TT 71-17

EL 1382'±	LOG	MC	LL	PI	-4	-200	N	Description
	GP	0.6		NP	44	3		SANDY GRAVEL; tan brown, loose, some cobbles to 7".
6.0'	GP	7.1		NP	46	4		medium dense, cemented.
10.0'	SP SM	6.5		NP	64	6		GRAVELLY SAND/SILTY GRAVELLY SAND; tan brown, medium dense, cemented.

NOTES:

1. SEE ~~SHEET 24~~ PLATE 3 FOR LOCATION OF TEST HOLES AND TEST TRENCHES.
2. SEE ~~SHEET 26~~ PLATE 6 FOR LEGEND, NOTES, AND BASIS OF CLASSIFICATION.

VERT. SCALE: 1 IN. = 5 FT.

5 0 5 10 15

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: TRI
DRAWN BY: J. F. B. M.P.
CHECKED BY: ATR

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DAM EMBANKMENT FOUNDATION
SOIL LOGS

SUBMITTED BY: DATE APPROVED:
GEOTECHNICAL BRANCH

SPEC. NO. DACW09- 83 -B-0016
DISTRICT FILE NO. 241/253
SHEET [illegible] OF 80

LEFT ABUTMENT

DD-19 (inclined 45°)

PRESSURE TEST			
DEPTH	PRESSURE PSI	Q	K
12.8	NO TEST		
	10	11.2	1.8
	20	16.2	1.8
33.8	40	25.6	1.7
	10	17.0	5.7
38.8	20	32.5	7.9
	20	23.5	5.4
43.2	40	35.5	5.6
	10	6.2	0.6
	20	7.7	0.6
	40	10.6	0.6
62.2	60	14.9	0.6
	20	1.1	0.1
	40	0.5	.03
76.9	60	0.8	.04

CORE LOSS IN % ELEV 1408.5'

0' – 5.0': ALLUVIUM: Residual soil and slopewash.

GRANITE: gray, hard, coarse grained, highly fractured with zones of slightly to moderately weathered, iron stained, moderately hard to hard, gray brown granite: 10.0-11.0', 12.2-19.4', 25.4-26.3', 47.5-52.1', 55.0-58.0', 59.4-61.0' and 62.2-63.0'.

brecciated and rehealed, moderately soft to moderately hard, usually rust stained: 28.9-29.7, 48.0-48.7, 49.8-50.4, 52.1-53.5, 55.0-56.6, 57.6-58.0, 59.4-61.0.

shattered: 22.7-23.0', 29.7-30.3', 32.9-33.2', 44.8-45.5', 47.5-48.0', 58.8-59.2'.

joints dip primarily 20°-70° and are iron stained, slightly to moderately weathered with occasional slickensides.

soft, highly weathered: 5.0-10.0'.

63.0' – 69.3': FAULT ZONE: brecciated granite / fault breccia with brown clay gouge layers, slickensides, minor rust staining; zone inclined about 70°

69.3' – 76.9': GRANITE: as above.

93% Recovery September 1980

Drilled from 0.0 to 76.9' for N W sized core. 100% drill water return 0.0 to 36.0'; 0% return 36.0 to 43.2'; 50% return 43.2 to 76.9'.

LEFT ABUTMENT

DD-21 (inclined 45°)

PRESSURE TEST			
DEPTH	PRESSURE PSI	Q	K
16.0	NO TEST		
38.9	10	0.8	0.1
14.0 to 48.0	5 10 20	0.0 0.1 2.0	0.0 0.0 0.2

CORE LOSS IN % ELEV 1444.6'

0' – 5.0': ALLUVIUM: Residual soil and slopewash.

5.0' – 16.0': GRANITE: gray, hard, coarse grained, slightly weathered highly fractured; zones of slightly to moderately weathered, moderately hard to hard, gray-brown (rust stained) granite: 9.8-11.3', 12.4-16.0' soft: 5.0-9.8', moderately soft: 16.0-18.8'.

16.0' – 35.1': FAULT ZONE: inclined 20°, brecciated (occasionally rehealed) to highly fractured, rust stained, moderately soft to hard, moderately to highly weathered GRANITE with thin layers of clay gouge, 26.3-27.0' solid clay gouge.

35.1' – 48.0': GRANITE: as above.

shattered. 9.8-10.6, 11.3-11.6', 14.5-14.8'.

brecciated and rehealed, rust stained, moderately hard to hard: 12.4-13.4', 16.0-18.8', 46.8-47.3'.

Joint dip primarily 40°-70° and are slightly weathered and rust stained.

67% Recovery September 1980

Drilled from 0.0 to 48.0' for N W sized core. 100% drill water return 0.0 to 48.0'.

RIGHT ABUTMENT

DD-22 (inclined 30°)

PRESSURE TEST			
DEPTH	PRESSURE PSI	Q	K
12.0	NO TEST		
23.0	10	13.8	4.3
	10	15.4	2.1
	20	18.3	1.8
43.5	30	21.6	1.6
	10	18.3	2.8
	20	23.5	2.7
56.0	40	33.3	2.6
	10	1.9	0.2
73.6	30	2.6	0.2

CORE LOSS IN % ELEV. 1391.6'

0' – 4.0': ALLUVIUM: silty sandy gravel and slopewash.

ANDESITE: gray, hard, moderately to highly fractured.

shattered: 18.7-19.1, 29.4-30.0', 43.5-44.0', 45.7-46.7', 47.6-48.2', 67.4-67.7'.

brecciated and rehealed: 35.6-38.3, 48.8-49.4'.

Joints dip 20°-70° and are slightly weathered, with minor rust staining to 34' and minor clay filling to 56'.

73.6'

97% Recovery September 1980

Drilled from 0.0 to 73.6' for N W sized core. 100% drill water return from 0.0 to 18.5', 23.4 to 38.5' and 56.0 to 73.6'; 50-100% drill water loss 18.5-23.4 and 38.5 to 56.0'.

RIGHT ABUTMENT

DD-45 (inclined 45°)

PRESSURE TEST			
DEPTH	PRESSURE PSI	Q	K
4.6	NO TEST		
25.6	10	16	3.1
	10	14.4	1.9
45.5	20	25	2.3
	10	1.3	0.1
	20	1.5	0.1
65.4	30	5.0	0.3
	10	4.7	0.5
	20	7.6	0.6
80.4	40	16.7	0.9

CORE LOSS IN % ELEV. 1485.3'

ANDESITE gray, hard, highly fractured, occasional vugs and vesicles.

shattered: 0.0-1.9', 16.1-16.8', 22.5-24.0', 26.3-27.0', 29.3-29.6', 32.5-33.9', 35.9-36.8', 40.6-41.3', 67.7-68.2', 70.4-72.1', 73.0-73.5', 75.4-76.1'.

moderately to highly fractured: 1.9-3.3, 17.2-20.6', 50.2-55.2', 60.4-67.7'.

moderately fractured: 6.0-6.7', 8.0-10.2', 24.2-24.9'.

slightly fractured: 10.6-11.8', 13.4-14.8'.

brecciated and rehealed: 16.8-17.2', 15.6-16.1', 30.8-31.0', 31.6-31.9', 38.2-39.4'.

slightly to moderately fractured: 55.2-59.7'.

joints dip 70° near vertical, mostly parallel with intersecting 20°-60°, slightly weathered to 8.0' with brown residual soil filling. Most joints contain varying amounts of calcite with minor hard clay.

80.4'

93% Recovery January 1981

Drilled from 0.0 to 3.6' for N X sized core; from 3.6 to 80.4' for N W sized core 100% drill water return 0.0-15.5', 16.0-19.0', 45.8-51.0'; 50-100% drill water loss 15.5-16.0', 19.0-45.8', 51.0-80.4'

NOTES: PLATE 2

1. SEE ~~SHEET 3~~ FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION.
2. SEE ~~SHEET 4~~ FOR DAM EMBANKMENT FOUNDATION GEOLOGIC PROFILE.
3. PLATE 4 SEE ~~SHEET 5~~ FOR LEGEND.
4. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED.
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.
6. DIAMOND DRILL HOLE INCLINATIONS ARE MEASURED FROM HORIZONTAL.

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

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

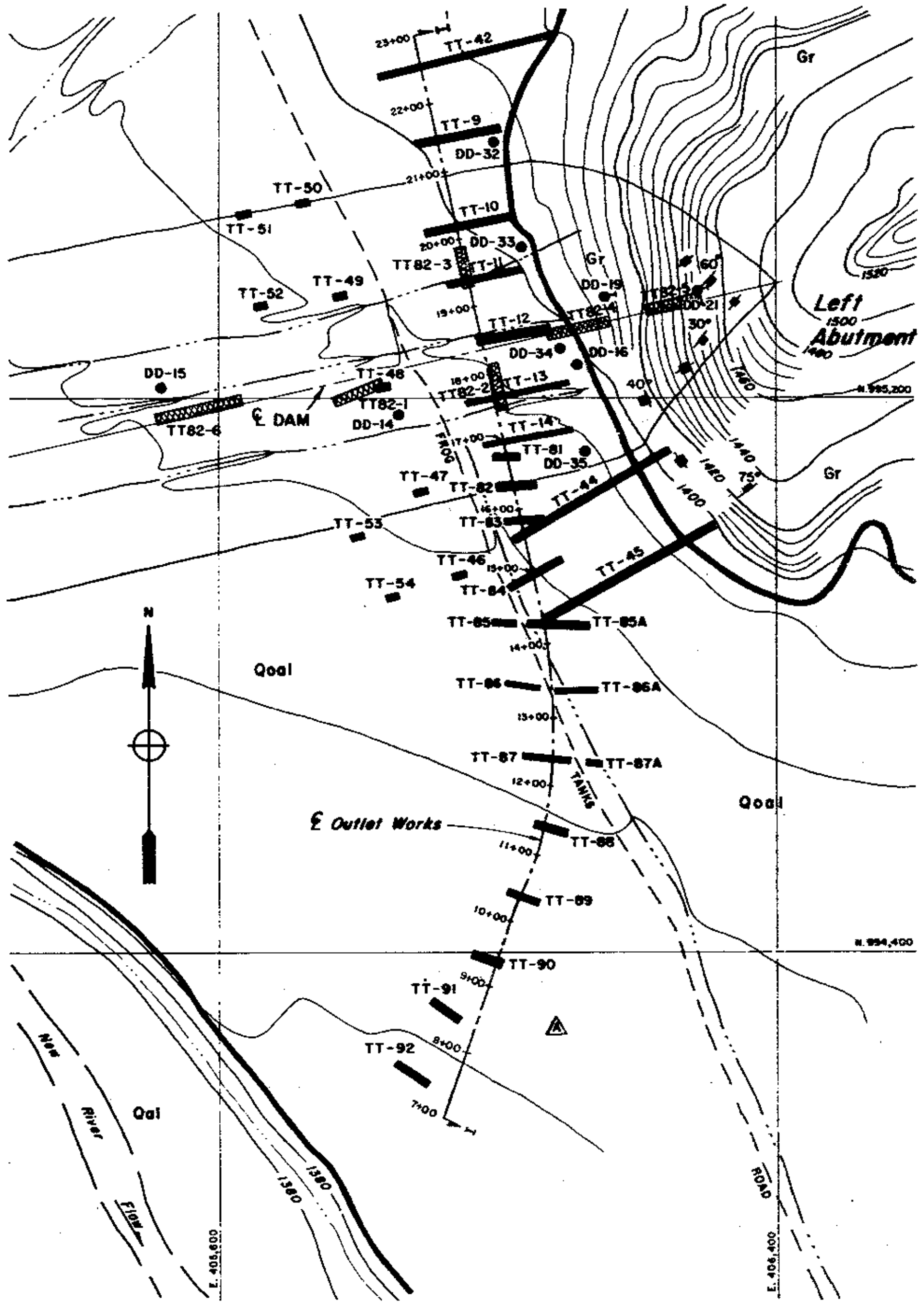
REVISIONS

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

DESIGNED BY: RLT
DRAWN BY: H. [signature]
CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DAM EMBANKMENT
(RIGHT AND LEFT ABUTMENTS)
GEOLOGIC LOGS OF DIAMOND
DRILL HOLES

SUBMITTED BY: [signature] GEOTECHNICAL
DATE APPROVED:
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/231
SHEET OF 80



LEFT ABUTMENT, EMBANKMENT-TEST TRENCH PLAN

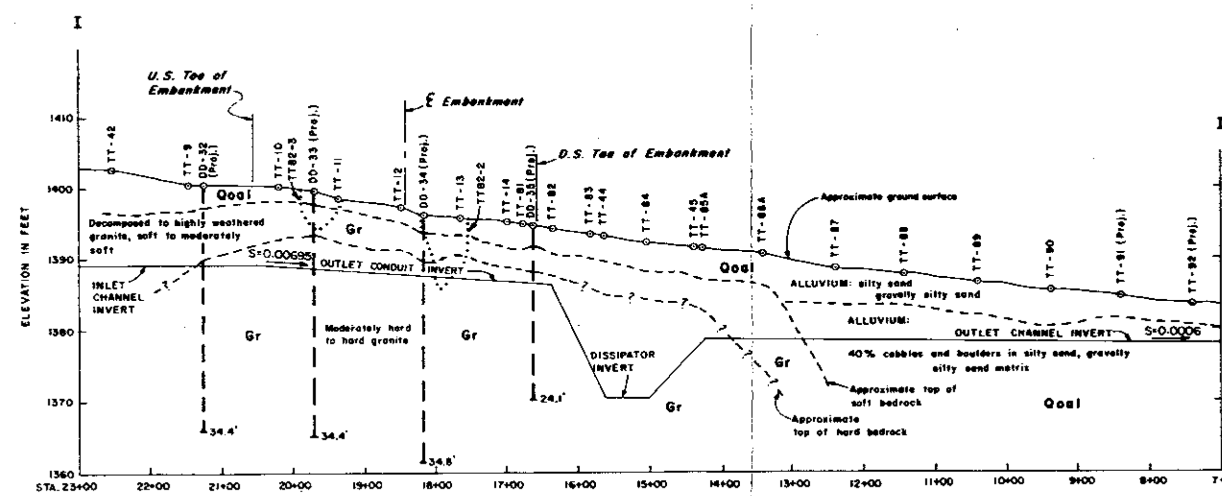
OUTLET WORKS-PLAN OF EXPLORATION

SCALE 1 IN. = 100FT.

CONTOUR INTERVAL 4 FT. AND 20 FT.

LOGS OF EMBANKMENT TEST TRENCHES

TEST TRENCH NO.	DEPTH TO BEDROCK	TOTAL DEPTH	DIMENSIONS	BEARING	REMARKS
46	—	8'	18' LONG 2' WIDE	N 67° E	0.0-6.0': SILTY SAND, GRAVELLY SILTY SAND. 6.0-8.0': CALICHE CEMENTED COBBLES AND BOULDERS, 1.5 TO 2' IN DIAMETER. REFUSAL IN COBBLES AND BOULDERS.
47	6	6.5'	12' LONG 2' WIDE	N 75° E	0.0-6.0': SILTY SAND, GRAVELLY SILTY SAND. 6.0-6.5': RED-BROWN, HIGHLY WEATHERED GRANITE. REFUSAL IN GRANITE AT 6.5'.
48	6.5	7'	16' LONG 2' WIDE	N 74° E	0.0-6.5': SILTY SAND, GRAVELLY SILTY SAND. 6.5-7.0': RED-BROWN DECOMPOSED GRANITE. REFUSAL IN GRANITE AT 7.0'.
49	5.5	6'	22' LONG 2' WIDE	N 86° E	0.0-5.5': SANDY SILT TO SILTY SAND, FEW GRAVELS. 5.5-6.0': RED-BROWN HIGHLY WEATHERED GRANITE. REFUSAL IN GRANITE AT 6'.
50	7.0	7'	22' LONG 2' WIDE	N 72° E	0.0-7.0': SANDY SILT TO SILTY SAND WITH SOME GRAVELS; REFUSAL IN RED-BROWN DECOMPOSED GRANITE AT 7.0'.
51	—	8.5'	25' LONG 2' WIDE	N 84° E	0.0-7.0': SILTY SAND TO SANDY SILT WITH SOME GRAVELS: 7.0-8.5': WELL CEMENTED COBBLES UP TO 6" IN DIAMETER. REFUSAL IN COBBLES AT 8.5'.
52	—	10.0'	25' LONG 2' WIDE	N 78° E	0.0-8.0': SANDY SILT TO SILTY SAND, POORLY CONSOLIDATED. 8.4-10.0': SANDY SILT, POORLY COMPACTED. REFUSAL IN CALICHE CEMENTED COBBLES AT 10'.
53	—	8'	25' LONG 2' WIDE	N 75° E	0.0-7.0': SANDY SILT TO SILTY SAND. 7.0-8.0': CALICHE-CEMENTED SANDY SILT. REFUSAL IN CALICHE CEMENTED COBBLES AT 8'.
54	—	9'	25' LONG 2' WIDE	N 67° E	0.0-5.0': SANDY SILT TO SILTY SAND WITH SOME GRAVELS. 5.0-9.0': POORLY CEMENTED COBBLES TO 1' DIAMETER WITH MINOR CALICHE. REFUSAL IN COBBLES AT 9'.



PROFILE ALONG ℄ OUTLET WORKS

HORIZ. SCALE: 1 IN. = 100FT.

VERT. SCALE: 1 IN. = 10 FT.

NOTES:

PLATE 2 1. SEE ~~SHEET 2~~ FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION.

PLATE 11 2. SEE ~~SHEET 9~~ FOR LOGS OF TEST TRENCHES 9 THROUGH 14.

PLATES 12 and 13 3. SEE ~~SHEETS 10&11~~ FOR LOGS OF TEST TRENCHES 81 THROUGH 92 AND 42, 44 AND 45.

4. SEE ~~SHEETS 3 AND 4~~ FOR LEGENDS.

5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.

6. TEST TRENCHES 9 THROUGH 14, 42, 44 THROUGH 54, AND 81 THROUGH 92 WERE EXCAVATED WITH A CASE 580-C BACKHOE.

7. FINAL DEPTHS OF TEST TRENCHES 46 THROUGH 54 WERE DETERMINED BY REFUSAL IN BEDROCK OR COBBLES AND BOULDERS.

8. THE SOFT BEDROCK PROFILE WAS DEVELOPED FROM DIAMOND DRILL HOLES AND TEST TRENCHES.

9. THE HARD BEDROCK PROFILE WAS DEVELOPED FROM DIAMOND DRILL HOLES AND TT82-2.

PLATE 10 10. SEE ~~SHEET 8~~ FOR LOGS OF TEST TRENCHES 82-1 THROUGH 82-6.

11. TEST TRENCHES 82-1 THROUGH 82-6 WERE EXCAVATED WITH A D9G DOZER.

12. THE NORTH WALL OF EACH TEST TRENCH WAS MAPPED WITH THE EXCEPTION OF TEST TRENCHES 82-2 AND 82-3 WHERE THE WEST WALLS WERE MAPPED.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
A	Revised Downstream alignment of outlet works	7 July 83	M.C.

REVISIONS

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

DESIGNED BY: RLT

DRAWN BY: J.B.B.

CHECKED BY:

GILA RIVER BASIN

PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DAM EMBANKMENT (LEFT ABUTMENT)
AND OUTLET WORKS
PLAN AND PROFILE OF
TEST TRENCH EXPLORATION

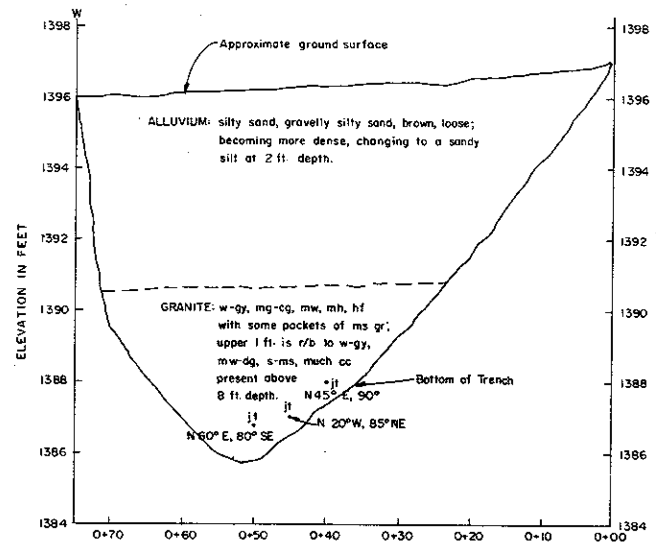
SUBMITTED BY: DATE APPROVED: GEOTECHNICAL BRANCH

SPEC. NO. DACW09-83-B-0016

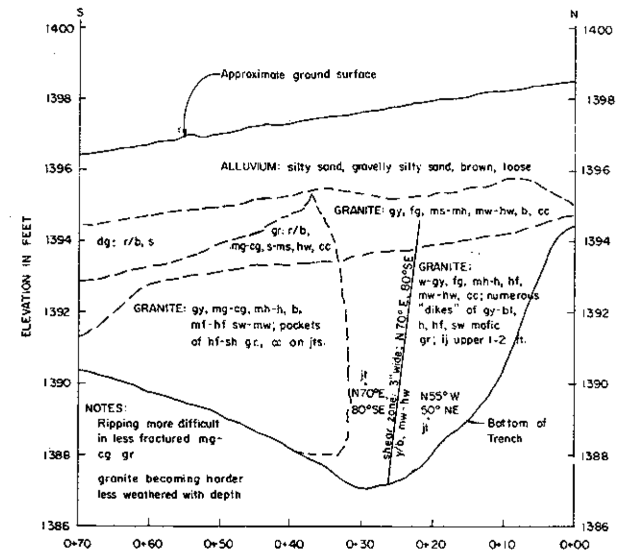
DISTRICT FILE NO. 241/232

REV "A"

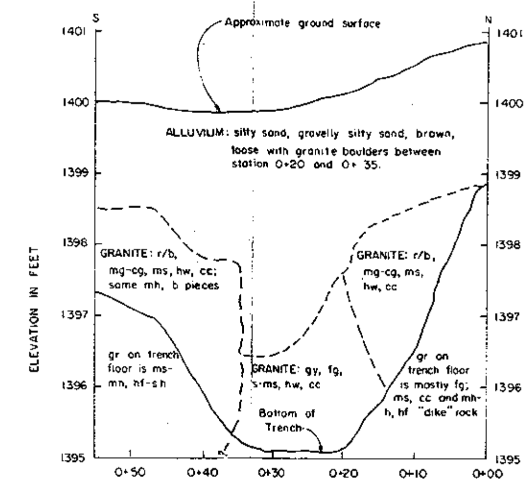
SHEET 7 OF 80



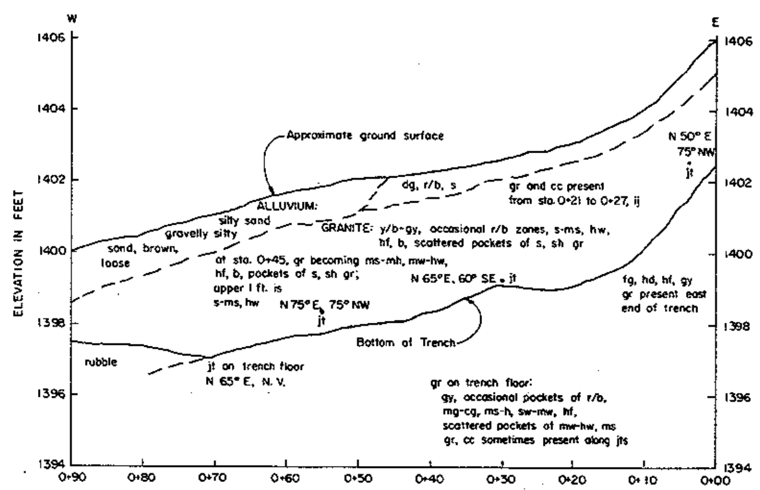
TEST TRENCH 82-1
HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 2 FT.



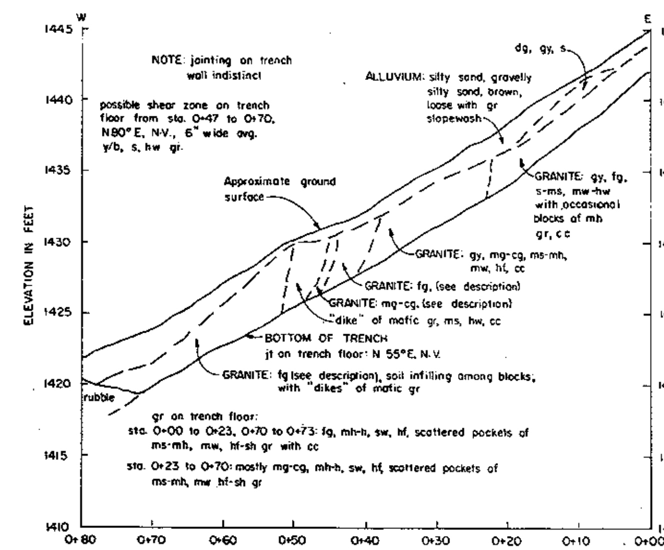
TEST TRENCH 82-2
HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 2 FT.



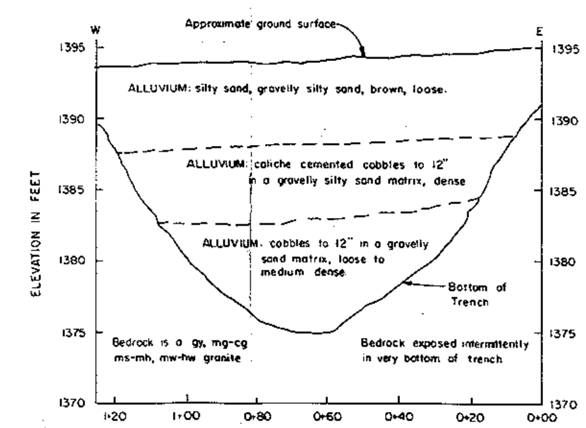
TEST TRENCH 82-3
HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 1 FT.



TEST TRENCH 82-4
HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 2 FT.



TEST TRENCH 82-5
HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 5 FT.



TEST TRENCH 82-6
HORIZ. SCALE: 1 IN. = 20 FT.
VERT. SCALE: 1 IN. = 5 FT.

LEGEND

gr	granite	sw	slightly weathered
dg	decomposed granite	mw	moderately weathered
r/b	reddish-brown	hw	highly weathered
gy	gray	mf	moderately fractured
w	white	hf	highly fractured
bl	black	sh	shattered
y/b	yellowish-brown	b	blocky
fg	fine grained	jts	joints
mg	medium grained	ij	indistinct jointing
cg	coarse grained	cc	calcium carbonate (generally occurs as infilling between rock blocks or along jt surfaces)
s	soft		
ms	moderately soft		
mh	moderately hard		
h	hard		

NOTES:

1. SEE ~~SHEET 7~~ PLATE 9 FOR LOCATION OF TEST TRENCHES.
2. TRENCHING ACCOMPLISHED USING D-9G DOZER WITH HYDRAULIC BLADE AND SINGLE TOOTH RIPPER.
3. REFUSAL WAS NOT ENCOUNTERED IN ANY TRENCH.
4. EXCAVATION ACCOMPLISHED USING ONLY BLADE IN TT82-3, TT82-4 AND TT82-5. RIPPING OF MODERATELY HARD TO HARD GRANITE REQUIRED IN TT82-1 AND TT82-2. SOME RIPPING OF CALICHE CEMENTED ALLUVIUM REQUIRED IN TT82-6.

SCALE: 1 IN. = 1 FT.
SCALE: 1 IN. = 2 FT.
SCALE: 1 IN. = 5 FT.
SCALE: 1 IN. = 10 FT.
SCALE: 1 IN. = 20 FT.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

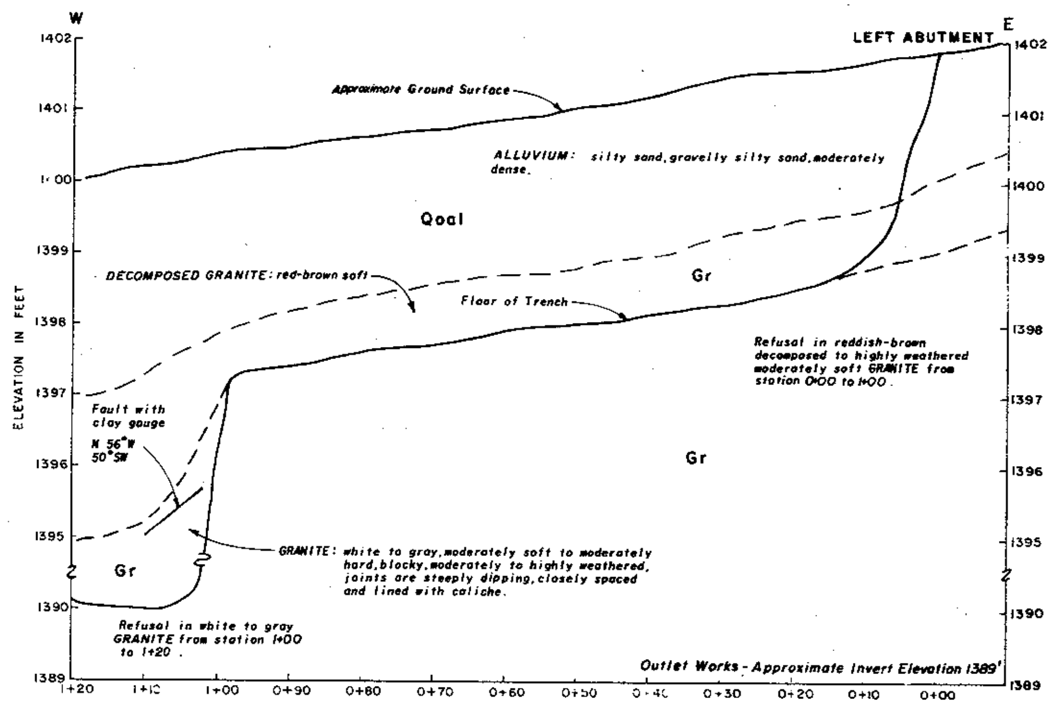
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: RLT
DRAWN BY: J.W.
CHECKED BY:

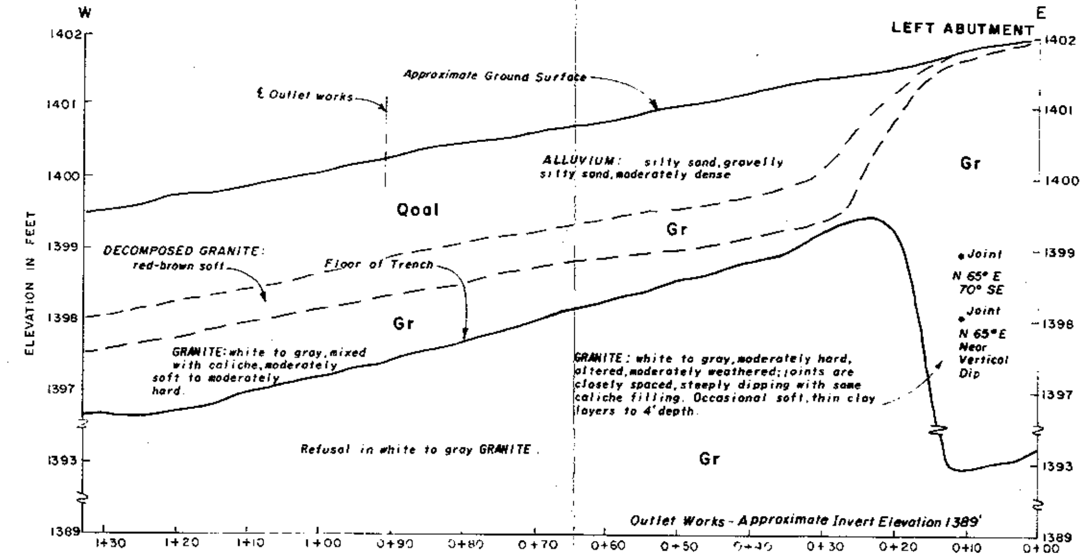
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DAM EMBANKMENT (LEFT ABUTMENT) AND OUTLET WORKS
LOGS OF TEST TRENCHES
82-1 THROUGH 82-6

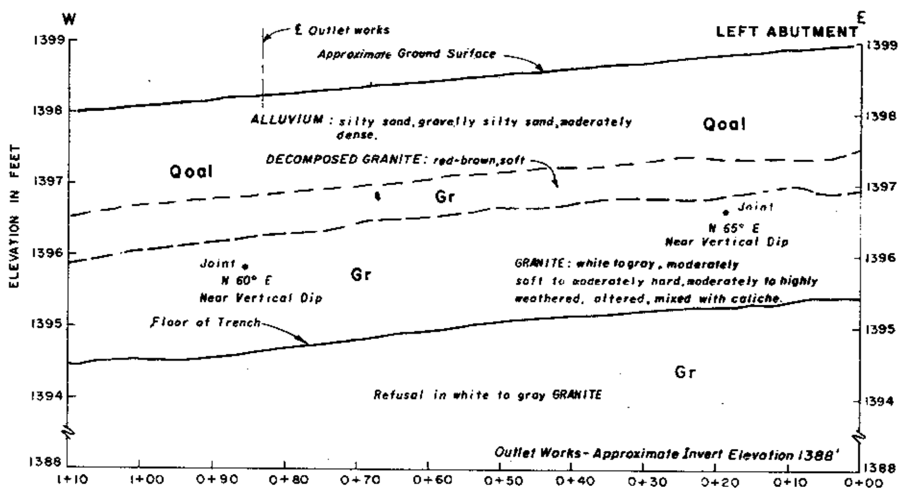
SUBMITTED BY: CHIEF, GEOTECHNICAL BRANCH
DATE APPROVED:
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/233
SHEET OF 80



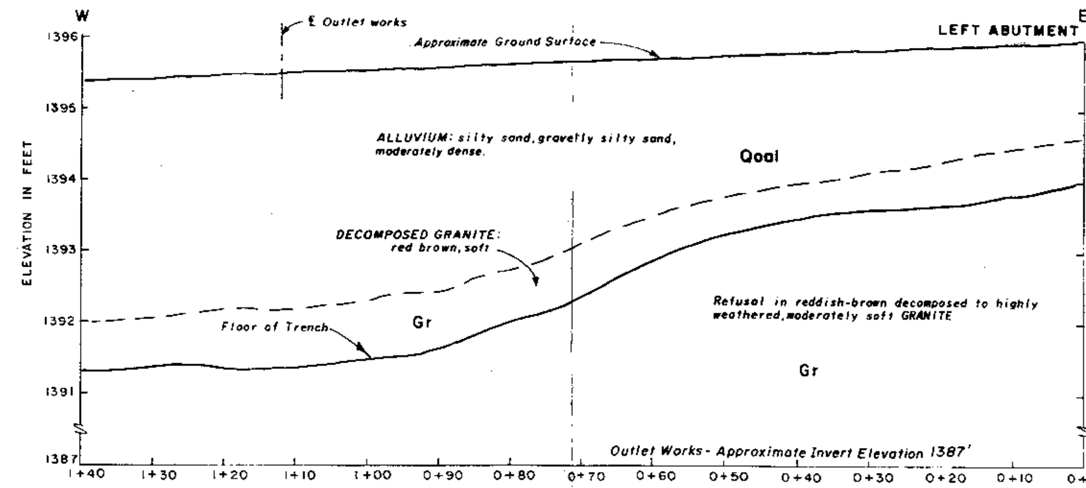
TEST TRENCH 9



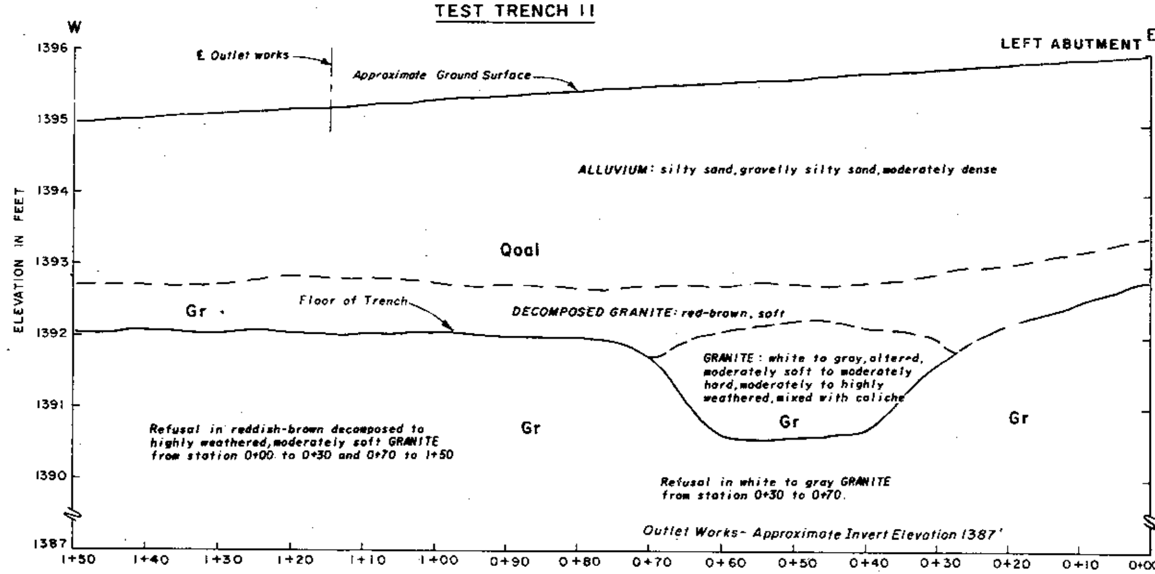
TEST TRENCH 10



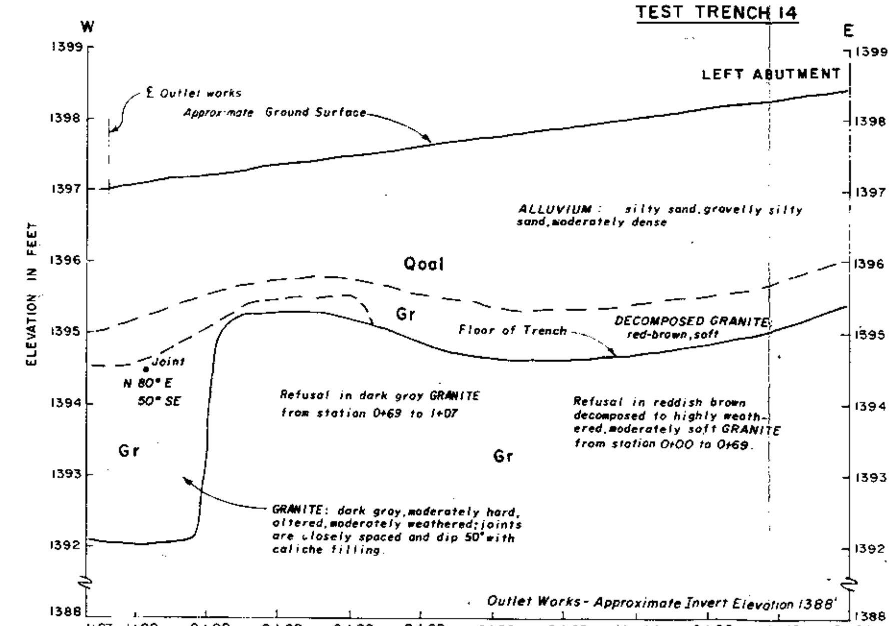
TEST TRENCH 11



TEST TRENCH 14



TEST TRENCH 13



TEST TRENCH 12

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

NOTES:

1. SEE ~~SHEET 3~~ PLATE 2 FOR LEGEND.
2. SEE ~~SHEET 7~~ PLATE 9 FOR THE LOCATIONS OF TEST TRENCHES 9 THRU 14.
3. ALL TEST TRENCHES WERE EXCAVATED WITH A CASE 580-C BACKHOE.
4. FINAL DEPTHS OF THE TEST TRENCHES WERE DETERMINED BY BEDROCK REFUSAL.
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

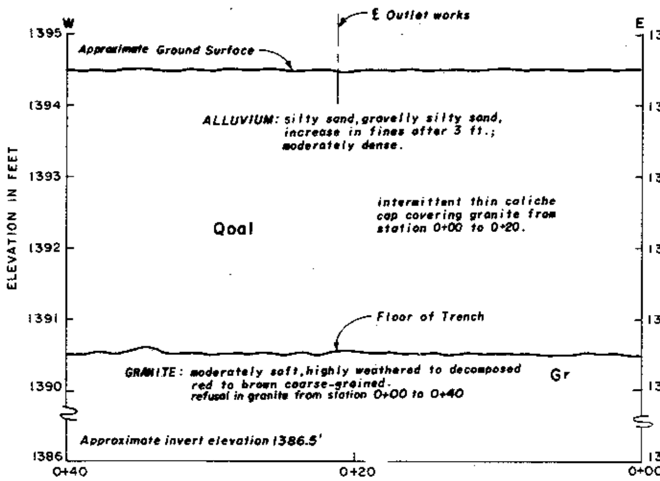
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: [signature] /ALT
DRAWN BY: VEH / YBA
CHECKED BY:

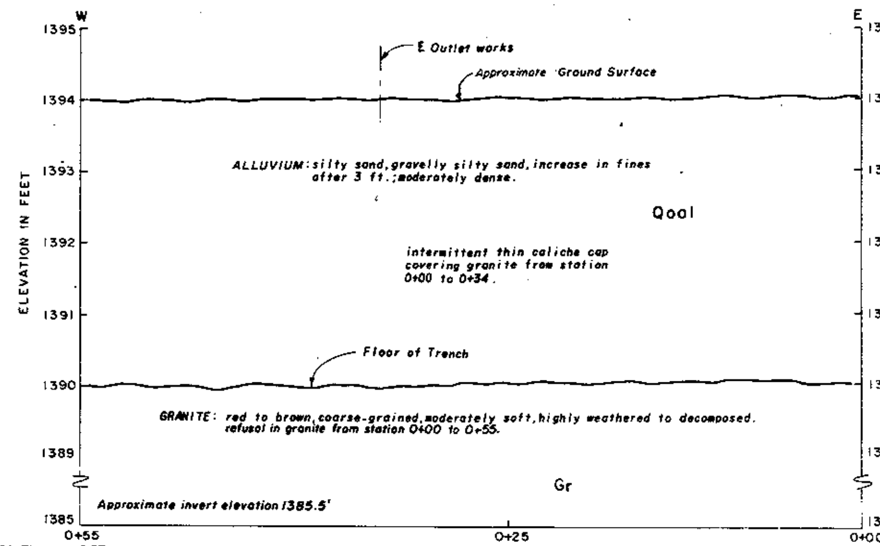
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCUDING NEW RIVER)

NEW RIVER DAM
OUTLET WORKS
LOGS OF TEST TRENCHES
9 THROUGH 14

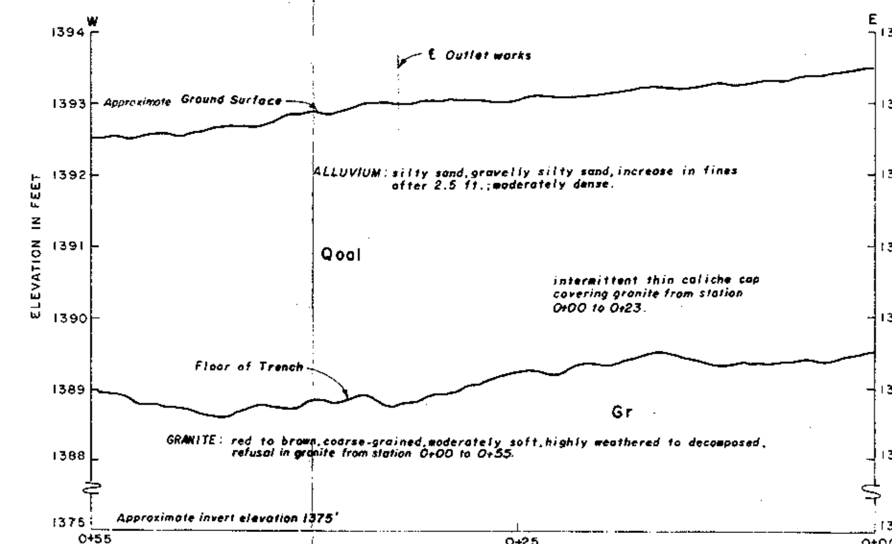
SUBMITTED BY: [signature] GEOTECHNICAL BRANCH
DATE APPROVED:
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/234
SHEET 9 OF 80



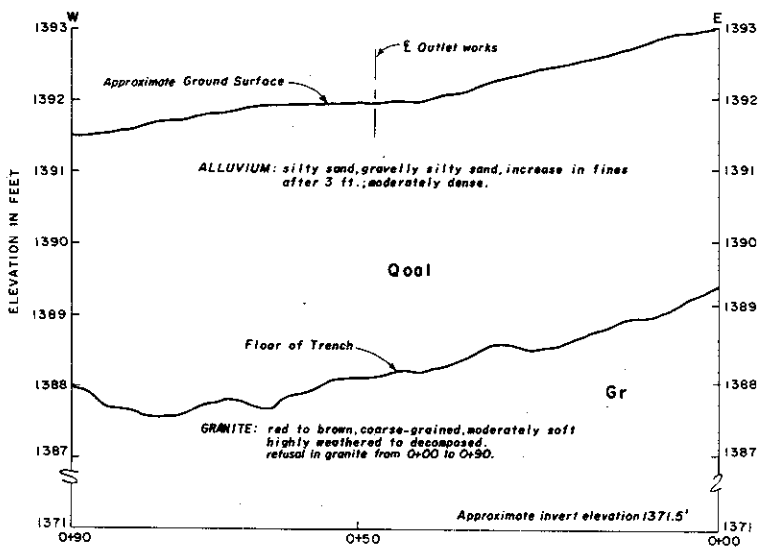
TEST TRENCH 81



TEST TRENCH 82

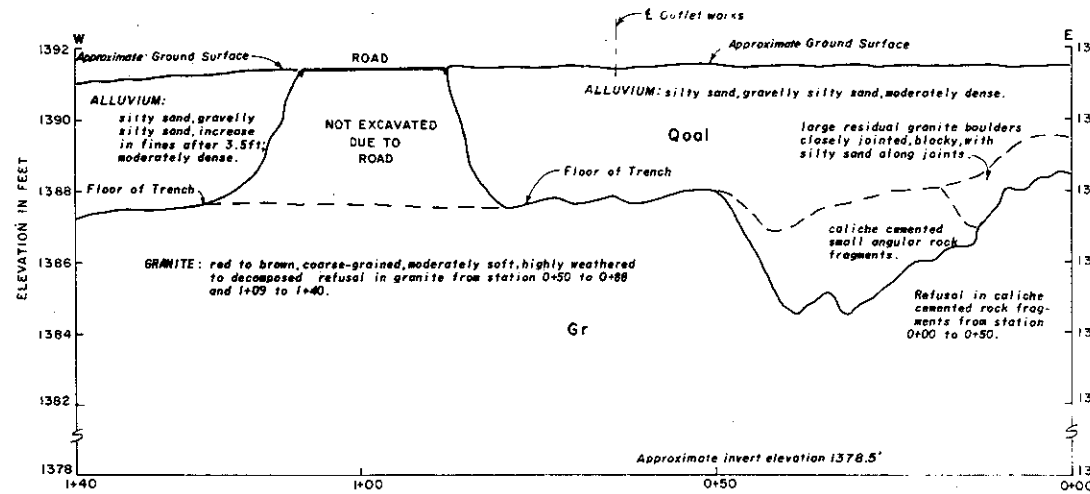


TEST TRENCH 83



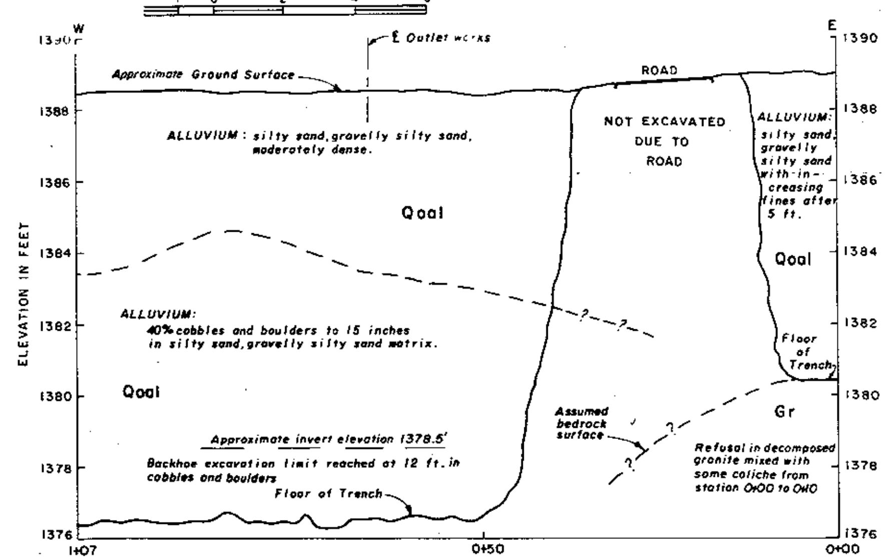
TEST TRENCH 84

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

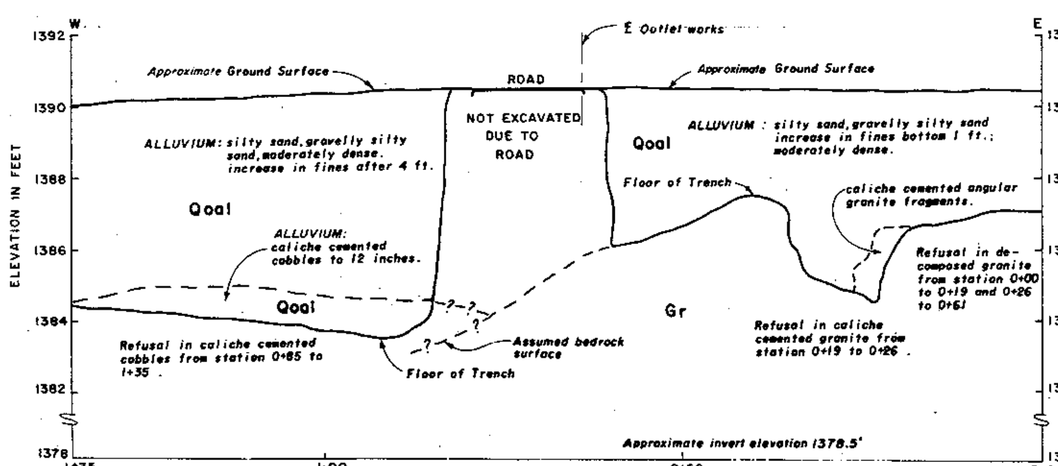


TEST TRENCH 85 and 85A

HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 2 FT.



TEST TRENCH 87 and 87A



TEST TRENCH 86 and 86A

HORIZ. SCALE: 1 IN. = 10 FT.
VERT. SCALE: 1 IN. = 2 FT.

NOTES:

1. PLATE 9 SEE ~~SHEET 7~~ FOR LOCATION OF TEST TRENCHES 81 THROUGH 87/87A.
2. PLATE 13 SEE ~~SHEET 11~~ FOR LOGS OF TEST TRENCHES 88 THROUGH 92 AND 42.44 AND 45.
3. PLATE 2 SEE ~~SHEET 3~~ FOR LEGEND.
4. TEST TRENCHES WERE EXCAVATED WITH A CASE 580-C BACKHOE.
5. ALL SOIL CLASSIFICATION SHOWN ON THIS SHEET ARE VISUAL.
6. FINAL DEPTHS OF ALL TEST TRENCHES EXCEPT 86 AND 87 WERE DETERMINED BY BEDROCK REFUSAL.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

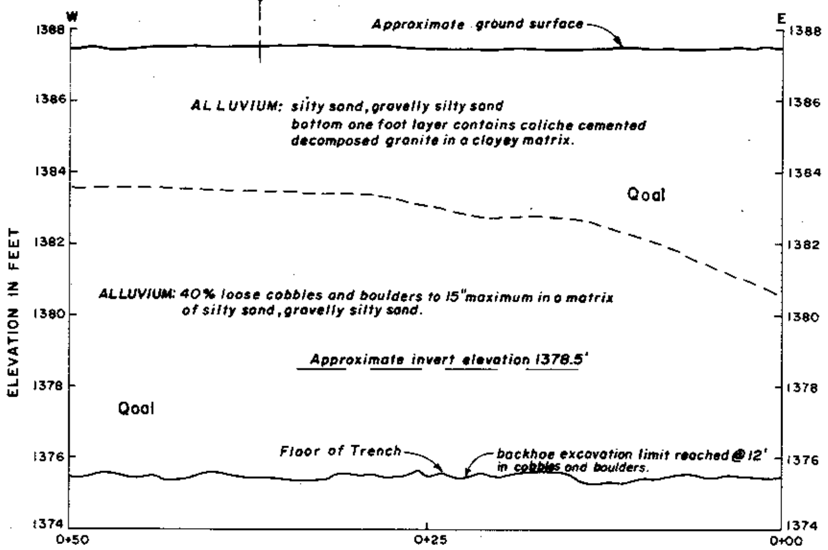
DESIGNED BY: RLT
DRAWN BY: LBA
CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

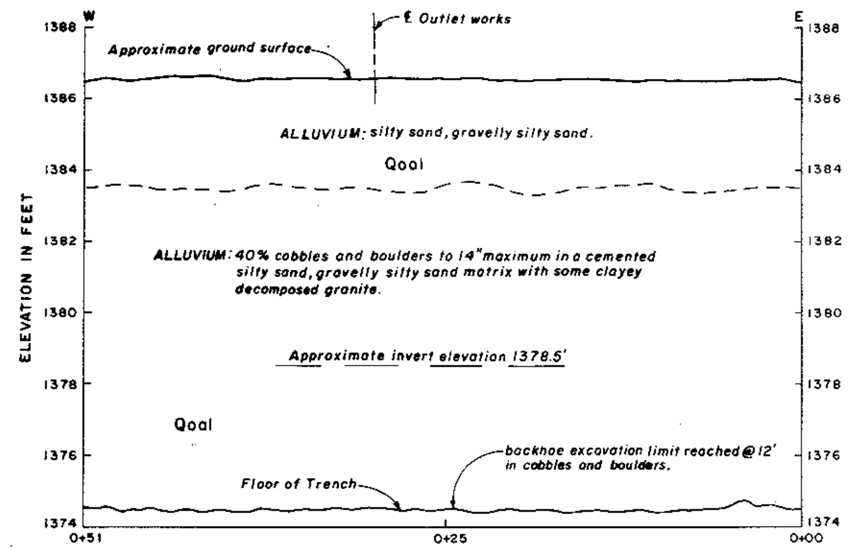
NEW RIVER DAM
OUTLET WORKS
LOGS OF TEST TRENCHES
81 THROUGH 87A

SUBMITTED BY: DATE APPROVED:
CHIEF, GEOTECHNICAL BRANCH

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/235

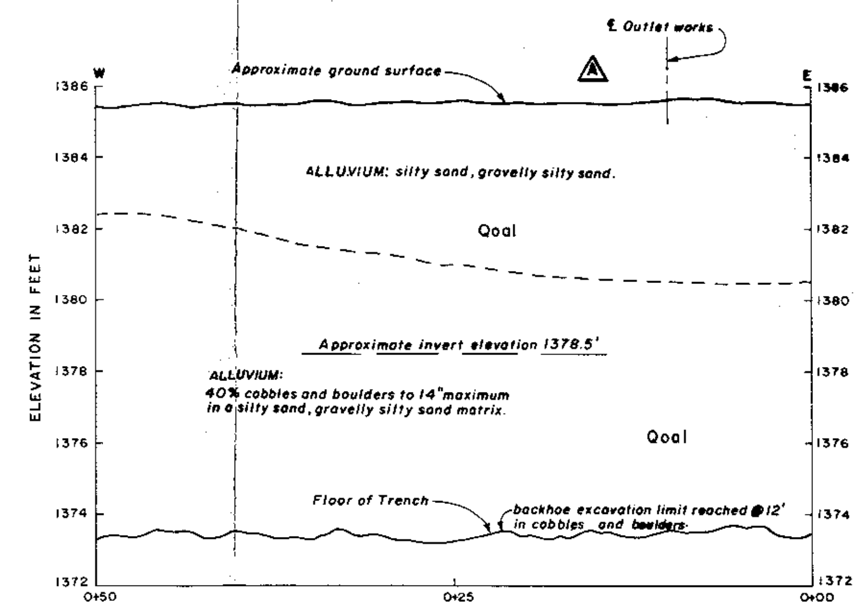


TEST TRENCH 88



TEST TRENCH 89

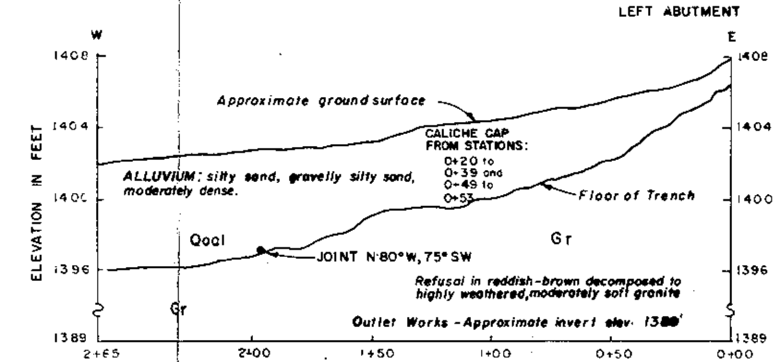
HORIZ. SCALE: 1 IN. = 5 FT.
5 4 3 2 1 0 5 10 15
VERT. SCALE: 1 IN. = 2 FT.
2 1 0 2 4 6



TEST TRENCH 90

HORIZ. SCALE: 1 IN. = 5 FT.
5 4 3 2 1 0 5 10 15
VERT. SCALE: 1 IN. = 2 FT.
2 1 0 2 4 6

LEFT ABUTMENT



TEST TRENCH 42

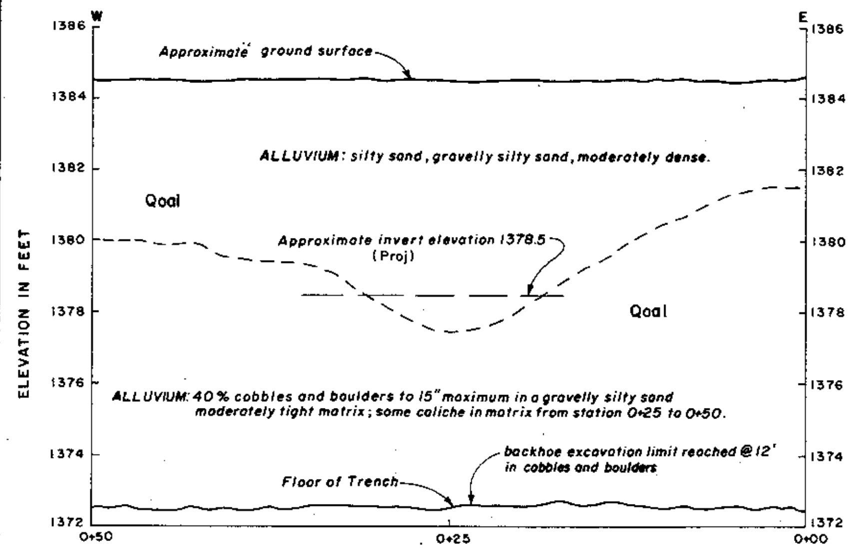
VERT SCALE 1 IN = 4 FT
4 3 2 1 0 4 8 12
HORIZ. SCALE 1 IN = 30 FT.
30 20 10 0 30 60 90

NOTES:

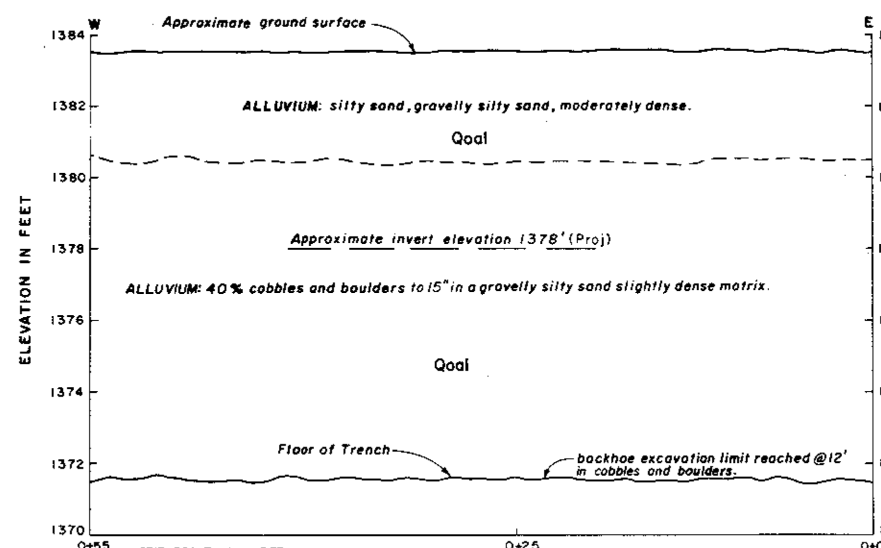
PLATE 9 1. SEE ~~SHEET 7~~ FOR LOCATIONS OF TEST TRENCHES 88 THRU 92, AND 42, 44 AND 45.

PLATE 2 2. SEE ~~SHEET 3~~ FOR LEGEND.

3. ALL TEST TRENCHES WERE EXCAVATED WITH A CASE 580-C BACKHOE.
4. FINAL DEPTHS OF TEST TRENCHES 42, 44, & 45, WERE DETERMINED BY BEDROCK REFUSAL.
5. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.



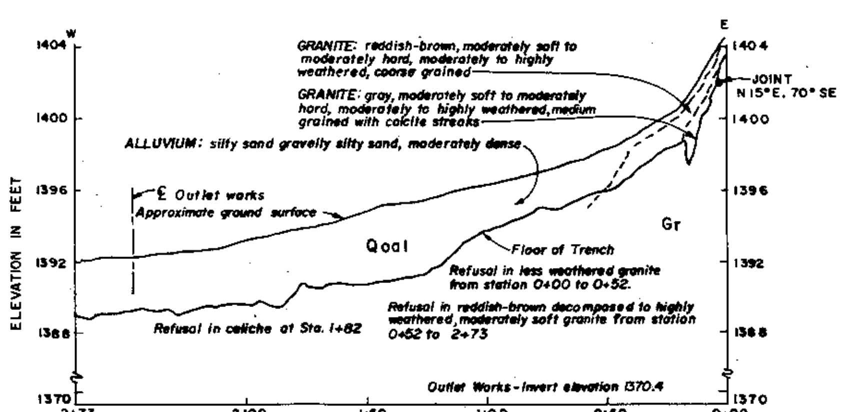
TEST TRENCH 91



TEST TRENCH 92

HORIZ. SCALE: 1 IN. = 5 FT.
5 4 3 2 1 0 5 10 15
VERT. SCALE: 1 IN. = 2 FT.
2 1 0 2 4 6

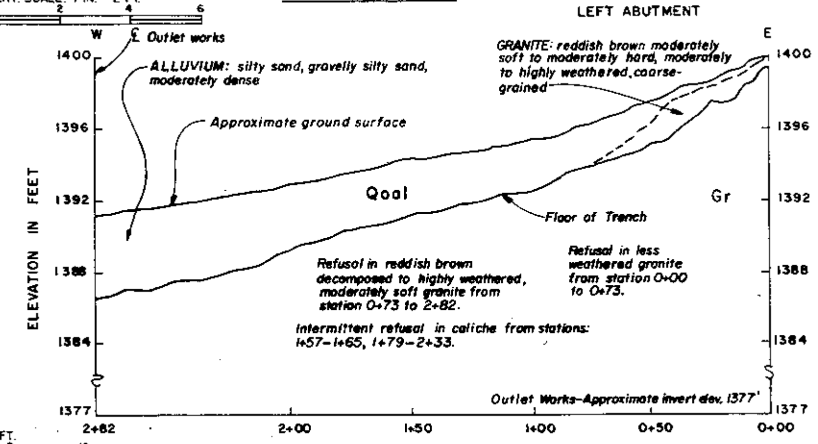
LEFT ABUTMENT



TEST TRENCH 44

VERT. SCALE: 1 IN. = 4 FT.
4 3 2 1 0 4 8 12
HORIZ. SCALE: 1 IN. = 30 FT.
30 20 10 0 30 60 90

LEFT ABUTMENT



TEST TRENCH 45

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
A	Revised ℄ of outlet works	7 July 83	M.PC

REVISIONS

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

DESIGNED BY: RLT
DRAWN BY: LBa
CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER).

NEW RIVER DAM

OUTLET WORKS

LOGS OF TEST TRENCHES
42, 44, 45, 88 THROUGH 92

SUBMITTED BY: DATE APPROVED:
GEOTECHNICAL

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/236
REV "A"
SHEET OF 80

OUTLET WORKS
DD-32

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K
10.0	NO TEST		
	10	0.0	0.0
	20	0.0	0.0
34.4	30	0.0	0.0

0.0′ — CORE LOSS IN % — ELEV. 1402.7′

5.0′ — ALLUVIUM: silty sandy gravel.

17.5′ — GRANITE: gray to yellow-brown (rust-stained), coarse-grained, moderately soft, heavily weathered, highly fractured; moderately hard to hard: 14.3-17.5′. Joints dip 40°-60° and are moderately to heavily weathered, rust-stained, with brown clay and occusional slickensides; soft, highly weathered: 5.0-11.1′

20.2′ — FAULT ZONE: inclined 60°, brecciated, rust-stained, highly weathered granite with clay gouge and slickensides.

34.4′ — GRANITE: gray, hard, coarse grained, highly fractured, slightly to moderately weathered; moderately soft: 27.2-27.7′. Joints dip 40°-60° and are slightly weathered, rust stained, clay coated, occasional slickensides.

78% Recovery August 1980

Drilled from 0.0 to 34.4′ for N W sized core. 100% drill water for the entire hole.

OUTLET WORKS/EMBANKMENT
DD-34

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K
10.0	NO TEST		
25.9	10	12	2.7
	10	3.9	1.1
	20	7.1	1.4
34.8	40	11.3	1.4

0.0′ — CORE LOSS IN % — ELEV. 1398.4′

2.5′ — ALLUVIUM: silty sandy gravel.

GRANITE: gray, hard, coarse-grained, slightly weathered, highly fractured:

moderately soft to moderately hard (moderately weathered): 4.6-4.9′, 6.4-8.9′, 27.6-30.0′; brecciated and rehealed: 9.6-10.5′, 18.1-18.9′; shattered: 27.6-30.0′; Joints dip 30°-near vertical and are slightly weathered with minor rust-staining and some clay filling.

Soft, highly weathered 2.5-4.6′, 4.9-6.4′.

34.8′

84% Recovery August 1980

Drilled from 0.0 to 34.8′ for N W sized core. 100% drill water return, 0.0 to 23.9′. No return, 23.9 to 25.9′. 25-50% return, 25.9 to 34.8′.

OUTLET WORKS
DD-33

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K
10.0	NO TEST		
	GRAVITY	0.4	0.1
34.4	10	1.0	0.2

0.0′ — CORE LOSS IN % — ELEV. 1401.3′

2.0′ — ALLUVIUM: silty sandy gravel.

19.0′ — GRANITE: dark gray, moderately hard to hard, fine-grained, altered, moderately weathered, highly fractured; moderately soft: 11.8-12.5′; brecciated: 7.8-8.3′; moderately soft, altered gouge zone, highly weathered: 16.0-17.5′. Joints dip primarily near vertical and are usually parallel, moderately weathered, with numerous thin clay gouge layers, soft, highly weathered: 2.0-6.7′.

28.1′ — FAULT ZONE: brecciated, highly weathered, moderately soft granite with clay gouge layers up to 4″ thick.

34.4′ — GRANITE: white to yellow-brown, moderately soft to moderately hard, coarse-grained, moderately to highly weathered, highly fractured; joints dip near vertical with intersecting 40°-60° and are rust-stained and moderately to highly weathered.

61% Recovery August 1980

Drilled from 0.0 to 34.4′ for N W sized core. 100% drill water return for entire hole.

OUTLET WORKS
DD-35

PRESSURE TESTS

DEPTH	PRESSURE PSI	Q	K
12.0	NO TEST		
24.1	10	17.9	5.5

0.0′ — CORE LOSS IN % — ELEV. 1396.4′

3.0′ — ALLUVIUM: silty sandy gravel.

GRANITE: gray, hard, coarse-grained, slightly weathered, highly fractured to 18.0′; moderately to highly fractured: 18.0-24.1′; brecciated and rehealed, moderately hard, brown (rust-stained), moderately weathered: 6.3-8.0′, 10.7-12.0′, 15.0-18.0′. Joints dip 20°-60° with occasional near vertical and are slightly weathered; rust stained to 23.0′.

Soft, highly weathered. 3.0-6.3′.

24.1′

64% Recovery August 1980

Drilled from 0.0 to 24.1′ for N W size core. 100% drill water return 0.0 to 17.9′ and 23.0 to 24.1′. 100% loss, 17.9 to 23.0′.

NOTES:

PLATE 2 — 1. SEE ~~SHEET 3~~ FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION.

PLATE 9 — 2. SEE ~~SHEET 7~~ FOR OUTLET WORKS PROFILE.

3. SEE ~~SHEET 4~~ FOR DAM EMBANKMENT FOUNDATION GEOLOGIC PROFILE

PLATE 4 — 4. SEE ~~SHEET 5~~ FOR LEGEND.

5. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED.

6. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.

VERT. SCALE: 1 IN. = 10 FT.

10 0 10 20 30

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: RLT

DRAWN BY: J.F.B.

CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
OUTLET WORKS
GEOLOGIC LOGS OF DIAMOND DRILL HOLES

SUBMITTED BY: — CHIEF, GEOTECHNICAL BRANCH

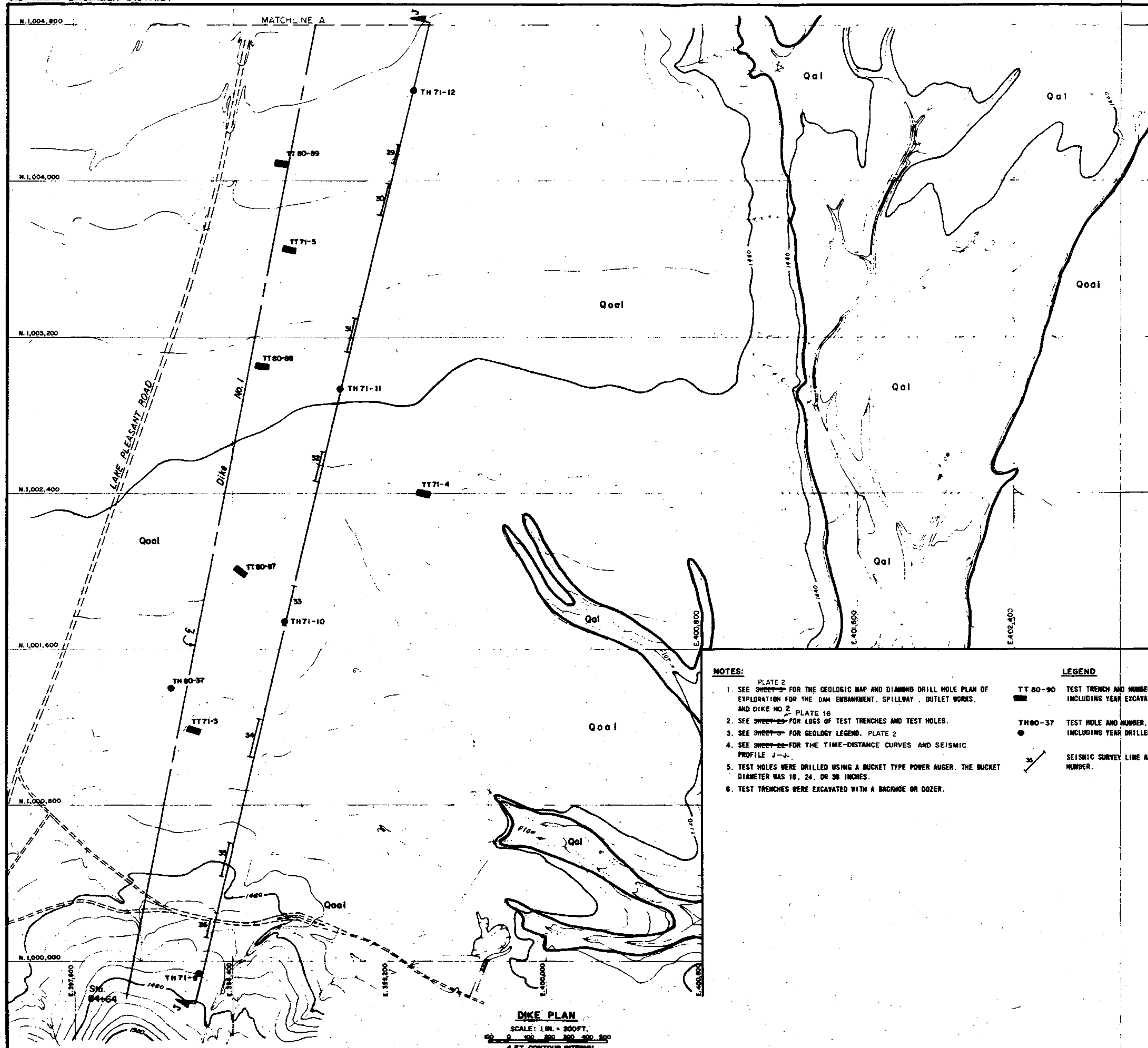
DATE APPROVED:

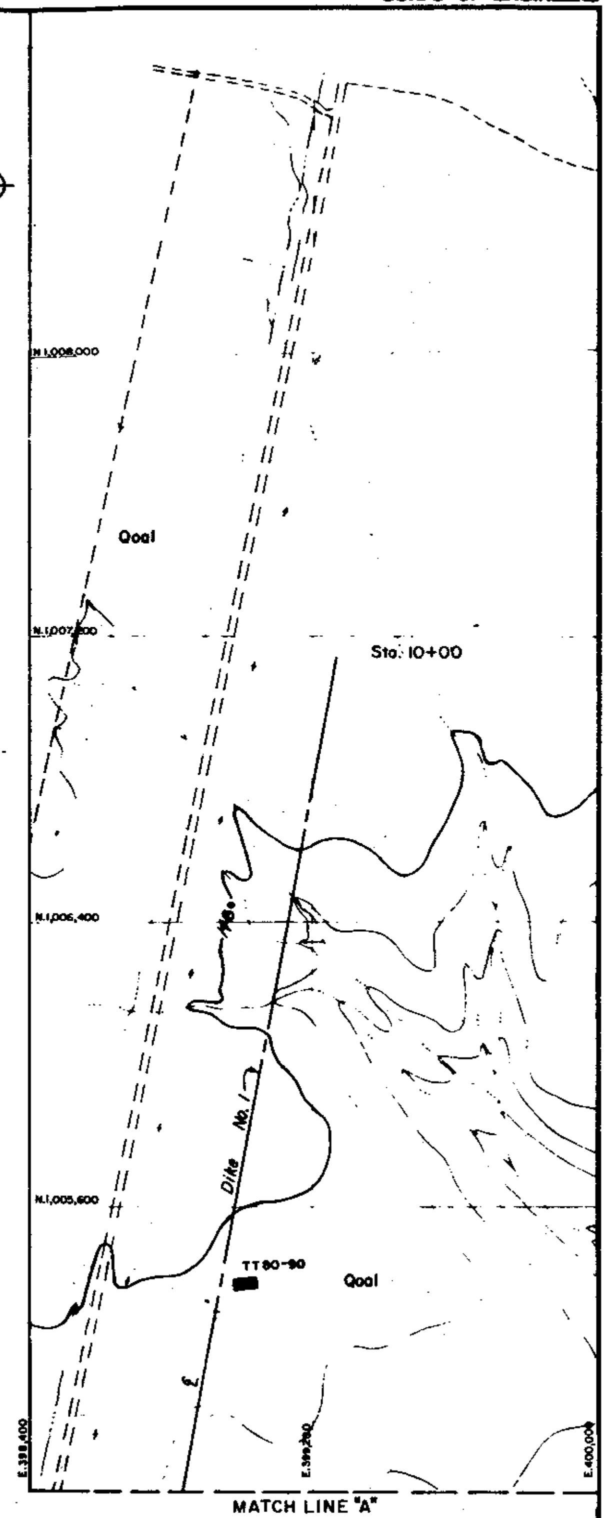
SPEC. NO. DACW09-83-B-0016

DISTRICT FILE NO. 241/237

SHEET 12 OF 80

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS





NOTES:

1. SEE ~~SHEET 3~~ PLATE 2 FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION FOR THE DAM EMBANKMENT, SPILLWAY, OUTLET WORKS, AND DIKE NO. 2
2. SEE ~~SHEET 29~~ PLATE 16 FOR LOGS OF TEST TRENCHES AND TEST HOLES.
3. SEE ~~SHEET 3~~ FOR GEOLOGY LEGEND. PLATE 2
4. SEE ~~SHEET 22~~ FOR THE TIME-DISTANCE CURVES AND SEISMIC PROFILE J—J.
5. TEST HOLES WERE DRILLED USING A BUCKET TYPE POWER AUGER. THE BUCKET DIAMETER WAS 18, 24, OR 36 INCHES.
6. TEST TRENCHES WERE EXCAVATED WITH A BACKHOE OR DOZER.

LEGEND

TT 80-90	TEST TRENCH AND NUMBER, INCLUDING YEAR EXCAVATED.
TH80-37	TEST HOLE AND NUMBER, INCLUDING YEAR DRILLED.
36	SEISMIC SURVEY LINE AND NUMBER.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: RLT
DRAWN BY: R.J.A. D.E.V.
CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DIKE NO. 1
GEOLOGY AND PLAN
OF EXPLORATION

SUBMITTED BY: DATE APPROVED:
CHIEF GEOTECHNICAL BRANCH

SPEC. NO. DACW 09-83-B-0016
DISTRICT FILE NO. 241/238
SHEET 13 OF 80

PLATE 15

TH 80-37

ELEV. 1453±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
3.0'	CL		34	15	97	63		SANDY CLAY; brown, caliche.
	SM		37	12	65	19		SILTY GRAVELLY SAND; light brown, cemented, 9" lens of caliche, 15% cobbles.
9.0'	SM		NS					CLAYEY SILTY SAND; cemented, 3" lens of caliche, 20% cobbles.
10.0'	GP GC		37	13	21	5		SANDY GRAVEL/CLAYEY SANDY GRAVEL; brown, cemented, 35% cobbles, 10% boulders to 12".
13.0'	GW GM		31	5	32	7		SANDY GRAVEL/SILTY SANDY GRAVEL
16.0'	GW			NP	32	3		SANDY GRAVEL.
18.0'	GP			NP	34	4		gray, cemented, caliche, 40% cobbles, 10% boulders to 12".
	SP SM			NP	58	7		SANDY GRAVEL/SILTY SANDY GRAVEL.
25.0'	SP SM		NS					DRILLING stopped by nest of boulders at 25'.

TT 80-87

ELEV. 1456±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
2.0'	CL		NS					SANDY CLAY; light brown, moist, loose, 9" lens of caliche, 15% cobbles.

TT 80-88

ELEV. 1462±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
2.0'	CL		NS					SANDY CLAY; light brown, moist, loose, some caliche at 1 to 1.5', very hard drilling at 2'.

TT 80-89

ELEV. 1470±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
2.0'	CL		32	14	99	63		SANDY CLAY; light brown, moist, loose, cemented, difficult digging at 2', caliche.
			37	18	100	83		

TT 80-90

EL. 1479±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
1.0	SM			NP	100	46		SILTY SAND; loose, moist, light brown, caliche nodules.

TH 71-9

EL. 1472±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
1.0	GP GM	7	22	6	62	30		SILTY SANDY GRAVEL/ CLAYEY SANDY GRAVEL; tan, difficult to drill due to caliche cementation, occasional cobbles.
	SM	3		NP	74	17		SILTY GRAVELLY SAND; tan, extremely difficult to drill due to caliche cementation, occasional cobbles.
	SM	2		NP	77	21	60+	
	SM	3		NP	76	19	60+	
9.5	SM	2		NP	77	20	60+	

TH 71-10

EL. 1454±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
1.3	ML CL	7	24	6	80	52		GRAVELLY SANDY SILT/GRAVELLY SANDY CLAY; tan, caliche cementation, difficult to drill.
	SM	3	35	6	66	20	60+	SILTY GRAVELLY SAND; tan, cemented, caliche, difficult to drill.
7.3	SM	4		NP	60	16	60+	
10.5	GP GM	4		NP	51	12	60+	SANDY GRAVEL/SILTY SANDY GRAVEL; tan, occasional cobbles, extremely difficult to drill due to caliche cementation.
13.5	SM	4		NP	59	14	60+	SILTY GRAVELLY SAND; tan, occasional cobbles, extremely difficult to drill due to caliche cementation.
14.5	GW GM	2		NP	63	11	60+	SANDY GRAVEL/SILTY SANDY GRAVEL; tan, occasional cobbles, extremely difficult to drill due to caliche cementation.

TH 71-11

EL. 1462±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
1.0	SC	8	30	12	71	41	60+	GRAVELLY CLAYEY SAND; tan, caliche cementation.
	SM	5		NP	61	17	60+	SILTY GRAVELLY SAND; tan, caliche cementation, occasional cobbles below 4', difficult drilling below 7'.
	SM	6		NP	56	15	60+	
10.0	SM	4		NP	64	13	60+	
11.5	GP GM	4		NP	52	10	60+	SANDY GRAVEL/SILTY SANDY GRAVEL; tan, occasional cobbles, difficult drilling due to caliche cementation.

TH 71-12

EL. 1470±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
0.5	CL	8	30	13	93	73	60+	SANDY CLAY; tan, caliche cementation.
	SC	4	30	12	72	34	60+	GRAVELLY CLAYEY SAND; tan, caliche cementation, difficult to drill.
6.0	SC	3	33	10	67	18	60+ / *	CLAYEY GRAVELLY SAND.
	SM	4		NP	68	17	*	SILTY GRAVELLY SAND; tan, caliche cementation, occasional cobbles and boulders to 13", extremely difficult to drill.
9.5	SM	4		NP	63	14	*	

* Penetrometer would not seat.

TT 71-3

EL. 1453±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
1.5	ML CL	6	24	6	96	67		SANDY SILT/SANDY CLAY; tan, some caliche cementation.
4.5	CL	4	35	12	77	52		GRAVELLY SANDY CLAY; light brown, some cobbles and boulders to 15", caliche cementation.
7.0	GP GM	3	44	11	47	12		SANDY GRAVEL/SILTY SANDY GRAVEL; light brown, numerous cobbles and boulders to 18", caliche cementation.

TT 71-4

EL. 1456±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
0.5	ML	4		NP	80	52		GRAVELLY SANDY SILT; light brown, some caliche.
3.5	GM	5		NP	53	13		SILTY SANDY GRAVEL; light brown, 25 to 35% cobbles to 8", cementation.
6.0	GW GM	5		NP	50	8		SANDY GRAVEL/SILTY SANDY GRAVEL; light brown, 35 to 45% cobbles and boulders to 15", caliche cementation.

TT 71-5

EL. 1469±, Ext

Depth	Class.	MC	LL	PI	-4	-200	N	Description
1.5	ML		20	3	75	54		GRAVELLY SANDY SILT; light brown.
2.0	GP GM			NP	22	8		SANDY GRAVEL/SILTY SANDY GRAVEL; light brown, material is well cemented by caliche.

VERT. SCALE: 1 IN. = 5 FT.

5' 0 5' 10' 15'

NOTES: PLATE 15

1. SEE ~~SHEET 13~~ FOR LOCATION OF TEST TRENCHES.
2. SEE ~~SHEET 26~~ PLATE 5 FOR NOTES, LEGEND, AND BASIS FOR CLASSIFICATIONS.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: T.R.I
DRAWN BY: J.F.B.
CHECKED BY: ATR

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

DIKE NO.1 FOUNDATION
SOIL LOGS

SUBMITTED BY: [signature] GEOTECHNICAL BRANCH
DATE APPROVED:

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/254

SHEET OF 80

DIKE NO. 2

DD-9

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
19.0	NO TEST		
32.0	10	1.0	0.2
26.0 TO 41.3	10	4.5	0.8
	20	3.5	0.4

CORE LOSS IN %

0' ELEV. 1477.2'

2.5 – 5.0: ALLUVIUM: silty sandy gravel. Top of soft bedrock is at 2.5'

GRANITE: gray, hard, fine grained highly fractured, slightly weathered.

moderately soft to moderately hard to 12.7'.

gray-brown (rust-stained) to 12.7'.

Joints dip primarily 40° to 60° and NV., are slightly weathered with iron staining to 34.9'

numerous clay coated / clay filled joints: 29.9 - 41.3'

Shear zones with clay gauge at 28.8 and 37.7'

41.3

70% Recovery Nov. 1979

Drilled with a tri-cone rock bit from 0.0 to 5.0', from 5.0 to 41.3' for NX sized core. NX casing set from ground surface to 10.0'; 100% drill water return from 0.0 to 25.1' and from 31.2 to 41.3'; 20% drill water loss 25.1 to 31.2'.

VERT. SCALE: 1 IN. = 10 FT.

DIKE NO. 2

DD-10

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
17.5	NO TEST		

CORE LOSS IN %

0' ELEV. 1484.7'

2.0: ALLUVIUM: silty sandy gravel.

4.5: GRANITE: gray, fine-grained, highly fractured; moderately soft and highly weathered to 4.5'; hard, slightly weathered to 17.5'; highly fractured, rust-stained: 11.8 to 14.2'. Joints dip 30° to 50° and near vertical, slightly to moderately weathered and accesionally clay filled.

17.5

88.5% Recovery December 1979

Drilled with a tri-cone rock bit from 0.0 to 4.5'; from 9.5 to 17.5' for N X sized core.

VERT. SCALE: 1 IN. = 10 FT.

DIKE NO. 2

DD-11

PRESSURE TESTS			
DEPTH	PRESSURE PSI	Q	K
8.0	NO TEST		
29.3	10	0.0	0.0
	10	0.1	0.0
48.4	20	1.6	0.2

CORE LOSS IN %

0' ELEV. 1475.1'

ALLUVIUM: silty sandy gravel.

5.4: GRANITE: gray, hard, coarse-grained, slightly weathered, highly fractured; streaked with orange and red stains to 15.3'.

shattered: 18.4-18.7', 22.3-22.7', 31.1-31.4', 36.7-37.0'.

Joints dip primarily N.V. and 30° to 90° and are slightly weathered and iron stained with some alteration along joint planes to 44.8', clay lined joints from 9.4 to 10.7', 36.7 to 37.0' and 39.2 to 41.1'.

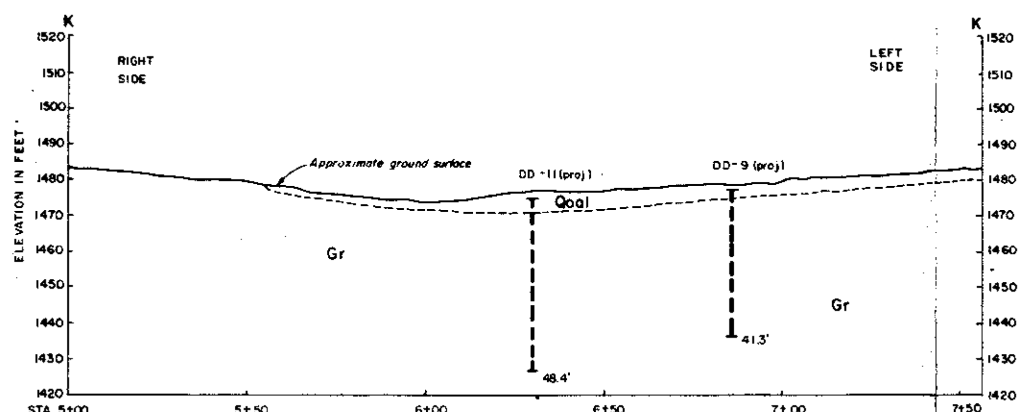
FAULT ZONES: with gouge and brecciated granite: 13.5-15.1' inclined 40° to 60° and 35.6-36.1' inclined 70° to 80°.

48.4

66% Recovery December 1979

Drilled with a tri-cone rock bit from 0.0 to 5.4'; from 5.4 to 48.4' for N X sized core. 100% drill water return from 0.0 to 48.4'.

VERT. SCALE: 1 IN. = 10 FT.



PROFILE ALONG ℄ DIKE NO. 2

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

NOTES:

PLATE 2 1. SEE ~~SHEET 3~~ FOR THE GEOLOGIC MAP AND DIAMOND DRILL HOLE PLAN OF EXPLORATION.
PLATE 2 2. SEE ~~SHEET 3~~ FOR LOCATION OF DIKE NO. 2 PROFILE.
PLATE 4 3. SEE ~~SHEET 5~~ FOR LEGEND.
4. SEE ~~SHEETS 3 AND 4~~ FOR ADDITIONAL LEGENDS.
5. DIAMOND DRILL HOLE ELEVATIONS ARE SURVEYED.
6. ALL SOIL CLASSIFICATIONS SHOWN ON THIS SHEET ARE VISUAL.

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

SCALE: 1 IN. = 20 FT.
20 0 20 40 80

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: RLT
DRAWN BY: V.F.B.
CHECKED BY:

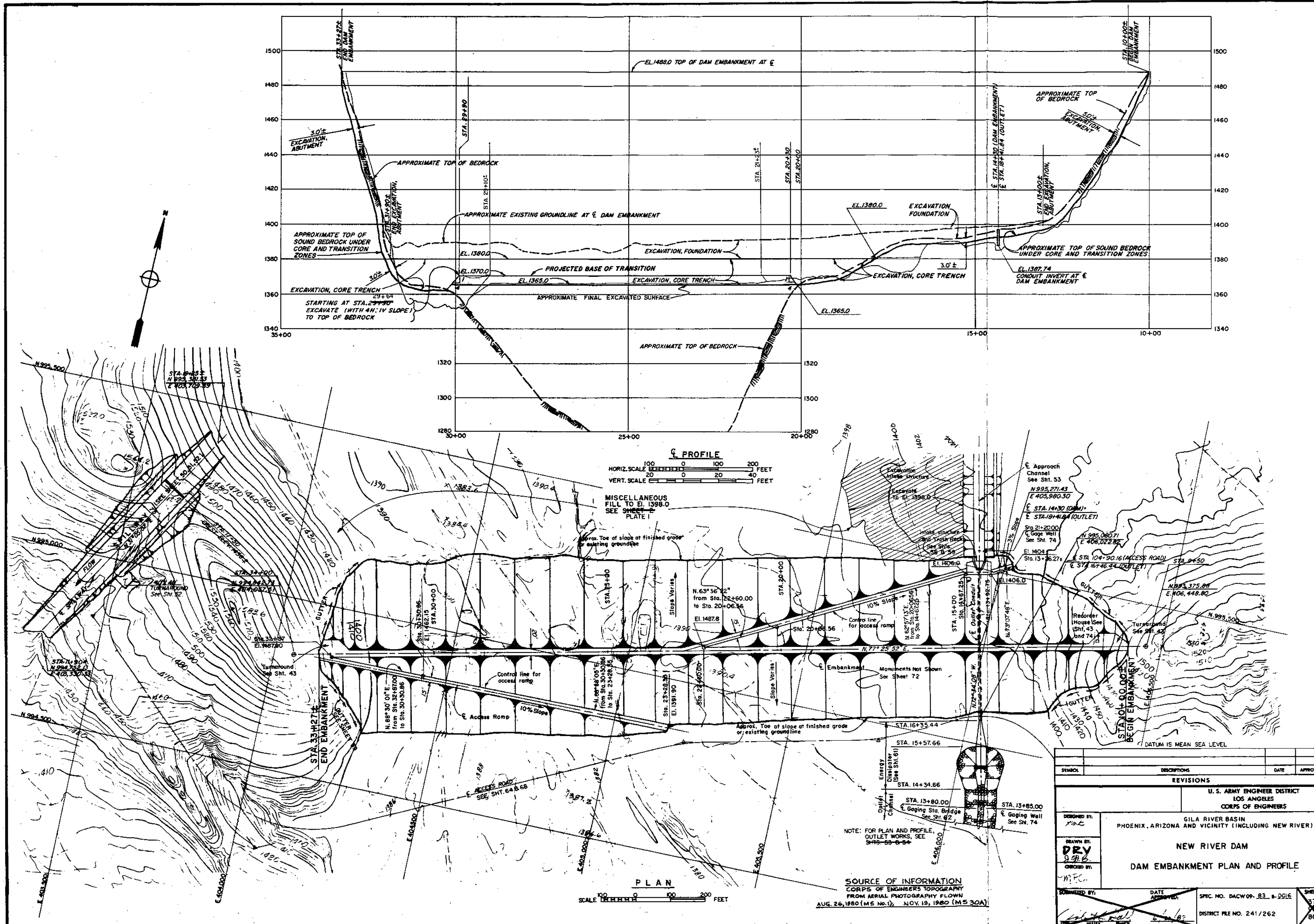
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
DIKE NO. 2
PROFILE AND GEOLOGIC LOGS OF DIAMOND DRILL HOLES

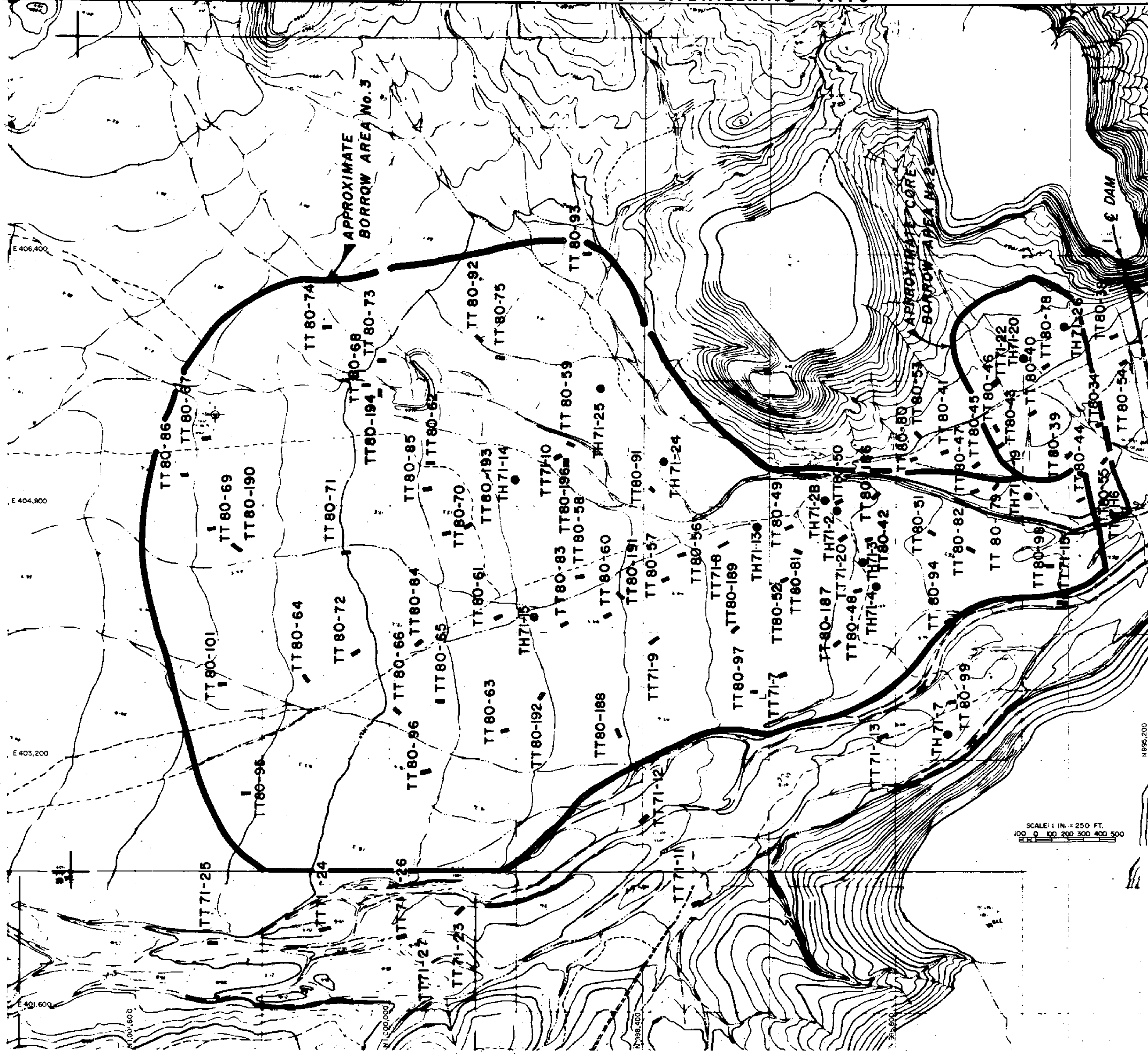
SUBMITTED BY: DATE APPROVED:
CHIEF, GEOTECHNICAL BRANCH

SPEC. NO. DACW09-_83_-B-0016
DISTRICT FILE NO. 241/239

SHEET 14 OF 80



℄ PROFILE
HORIZ. SCALE 100 0 100 200 FEET
VERT. SCALE 20 0 20 40 FEET
EL.1488.0 TOP OF DAM EMBANKMENT AT ℄
APPROXIMATE TOP OF BEDROCK
APPROXIMATE EXISTING GROUNDLINE AT ℄ DAM EMBANKMENT
APPROXIMATE TOP OF SOUND BEDROCK UNDER CORE AND TRANSITION ZONES
EXCAVATION, CORE TRENCH
PROJECTED BASE OF TRANSITION
APPROXIMATE FINAL EXCAVATED SURFACE
EXCAVATION, FOUNDATION
EXCAVATION FOUNDATION
STARTING AT STA. 29+64 EXCAVATE (WITH 4H:1V SLOPE) TO TOP OF BEDROCK
EL.1387.74 CONDUIT INVERT AT ℄ DAM EMBANKMENT
STA. 33+27± END DAM EMBANKMENT
STA. 10+00± BEGIN DAM EMBANKMENT
MISCELLANEOUS FILL TO EL. 1398.0 SEE SHEET 6 PLATE 1
Approx. Toe of slope at finished grade or existing groundline
Control line for access ramp
℄ Access Ramp
℄ Embankment
Monuments Not Shown See Sheet 72
N.77°25'52" E.
STA. 33+27± END EMBANKMENT
STA. 10+00.00± BEGIN EMBANKMENT
NOTE: FOR PLAN AND PROFILE, OUTLET WORKS, SEE SHTS. 53 & 54
PLAN
SCALE 100 0 100 200 FEET
SOURCE OF INFORMATION
CORPS OF ENGINEERS TOPOGRAPHY FROM AERIAL PHOTOGRAPHY FLOWN
AUG. 26, 1980 (MS No.1); NOV. 19, 1980 (MS 30A)
DATUM IS MEAN SEA LEVEL
REVISIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DAM EMBANKMENT PLAN AND PROFILE
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/262



NOTES:

1. TEST HOLES WERE DRILLED BY A BUCKET-TYPE POWER AUGER (JAN. 1971 THRU APR. 1971).
2. TEST TRENCHES WERE EXCAVATED WITH A INTERNATIONAL HTD 25-250 SERIES BULLDOZER OR A BACKHOE (FEB. 1971 THRU APR. 1971, JAN. 1980, AND AUG. 1980 THRU SEPT. 1980).
3. GROUND WATER WAS NOT ENCOUNTERED.
4. LOGS OF TEST TRENCHES AND TEST HOLES ARE SHOWN ON SHEETS ~~30 THROUGH 35~~

LEGEND:

TT 80-39 ▬ LOCATION AND NUMBER OF TEST TRENCH

TH 71-7 ● LOCATION AND NUMBER OF TEST HOLE

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

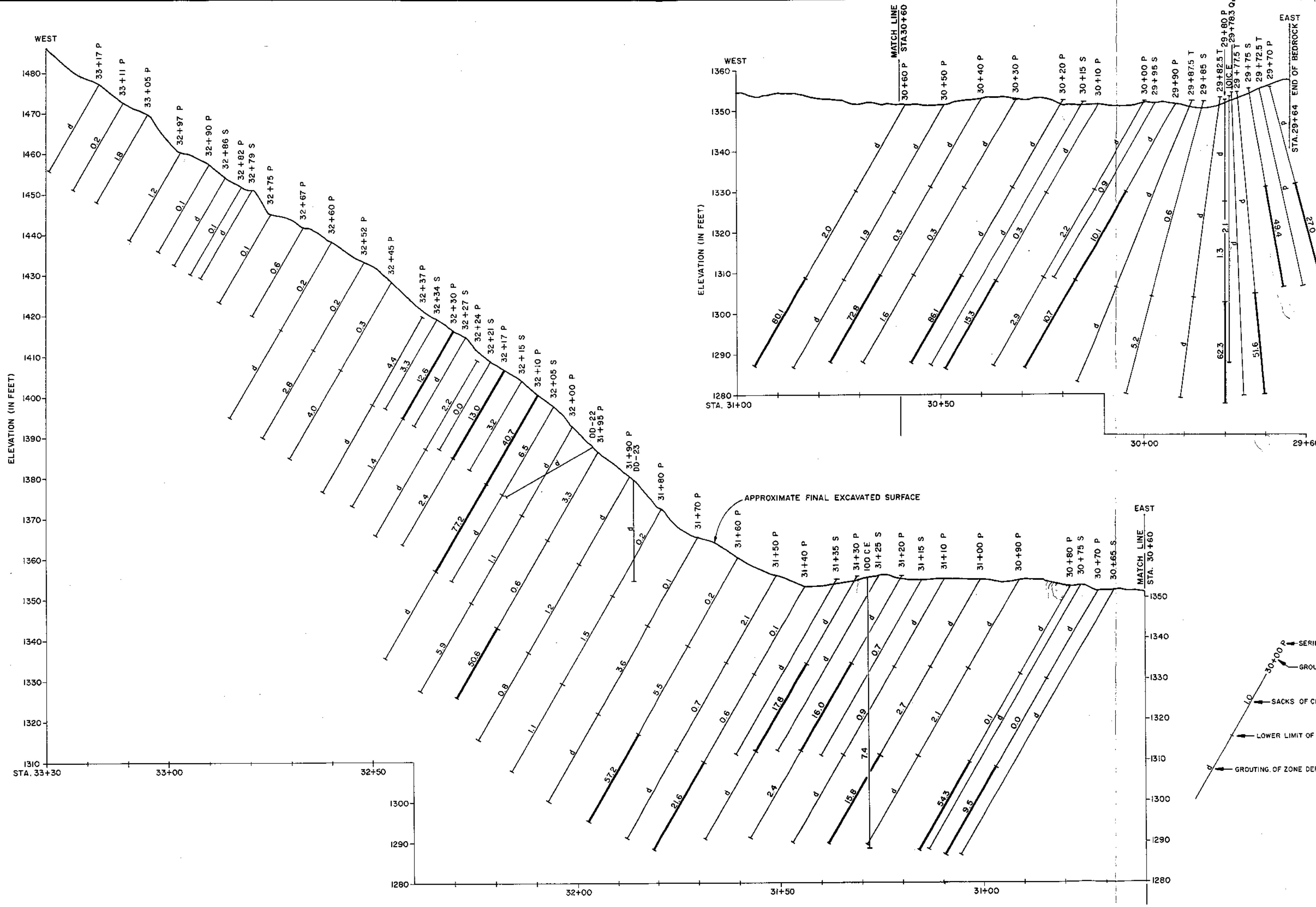
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM
BORROW AREAS
EXPLORATION PLAN

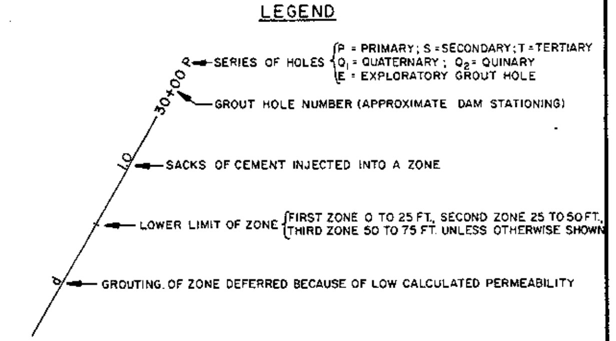
DESIGNED BY:
DRAWN BY: RSZ DRV
CHECKED BY: ATR
SUBMITTED BY:
DATE APPROVED:
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/250
SHEET 29 OF 80

PLATE 19



GROUT CURTAIN PROFILE CORE (ZONE)

SCALE 10 0 10 20 30 FEET



NOTES

1. ALL GROUT HOLES WERE PERCUSSION DRILLED USING AN INGERSOLL-RAND AIR TRACK DRILL WITH A 2 1/2 INCH PLUG BIT.
2. ALL EXPLORATORY GROUT HOLES WERE ROTARY DRILLED USING A LONGYEAR 24 SKID RIG WITH AN NW-SIZE SPLIT-TUBE CORE BARREL.
3. CORE HOLES DD-22 AND DD-23 WERE DRILLED DURING THE DESIGN PHASE PRIOR TO DAM CONSTRUCTION.
4. DETAILED PROFILES DEPICT AREAS OF SIGNIFICANT GROUT TAKE.

6. HEAVY LINES REFLECT ZONES WHERE GROUT TAKES EXCEEDED 7.5 SACKS PER ZONE AND SPLIT SPACING REQUIRED.
6. STAGE GROUTING FOR EXPLORATORY GROUT HOLES WAS PERFORMED WHENEVER 100 PERCENT LOSS OF DRILL WATER CIRCULATION OCCURRED. IT WAS ASSUMED THAT THE MAJORITY OF GROUT INJECTED IN A STAGE OCCURRED AT OR NEAR THIS POINT. THEREFORE, THE GROUT TAKE IS SHOWN IN A SMALL LOCALIZED INTERVAL.
7. FOR PERCUSSION DRILLED GROUT HOLES, GROUT TAKES WERE ASSUMED TO BE EVENLY DISTRIBUTED WITHIN EACH ZONE.

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

FOUNDATION GROUTING PROFILE
STA. 33+27 TO STA. 29+64

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

VALUE ENGINEERING PAYS

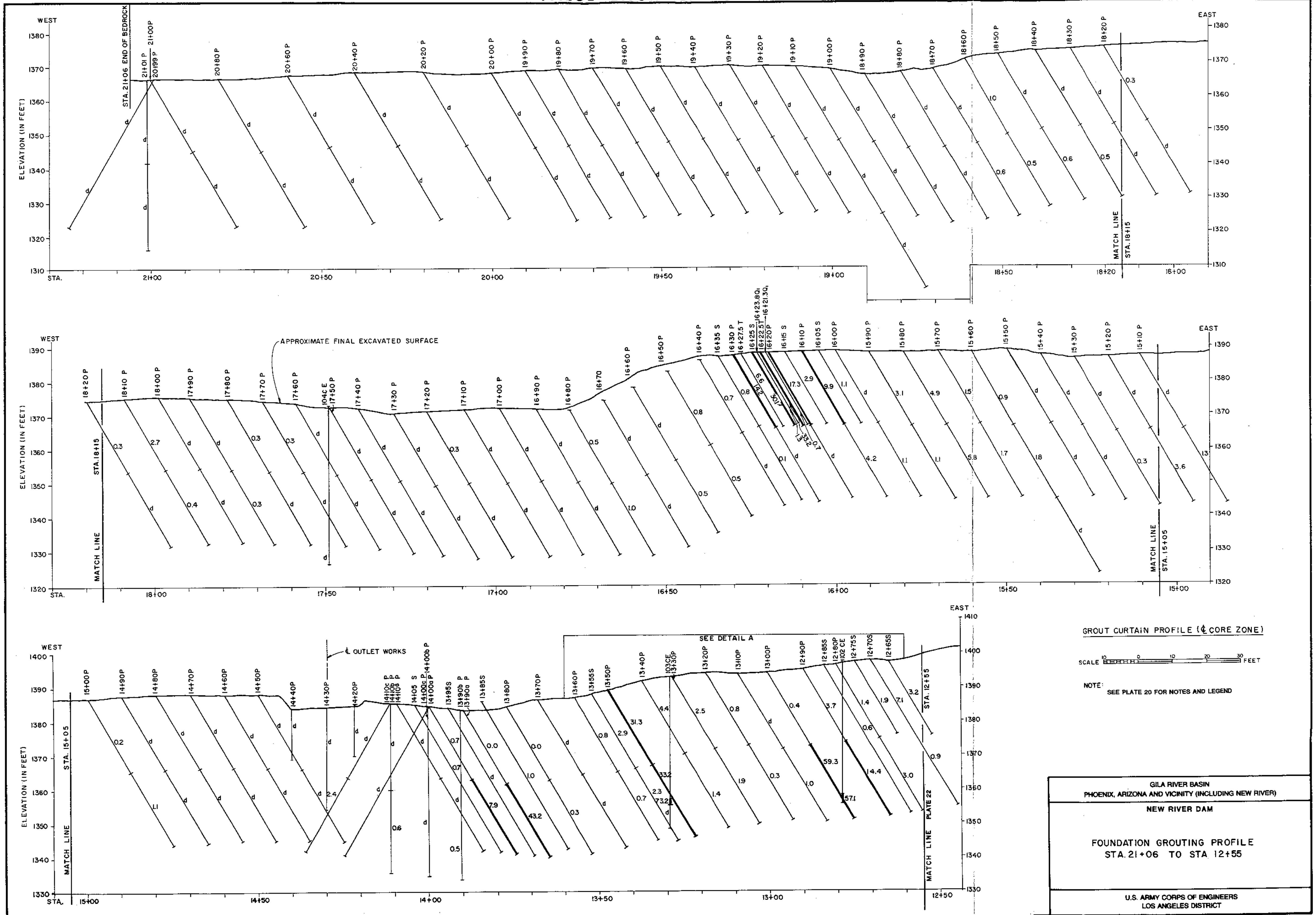
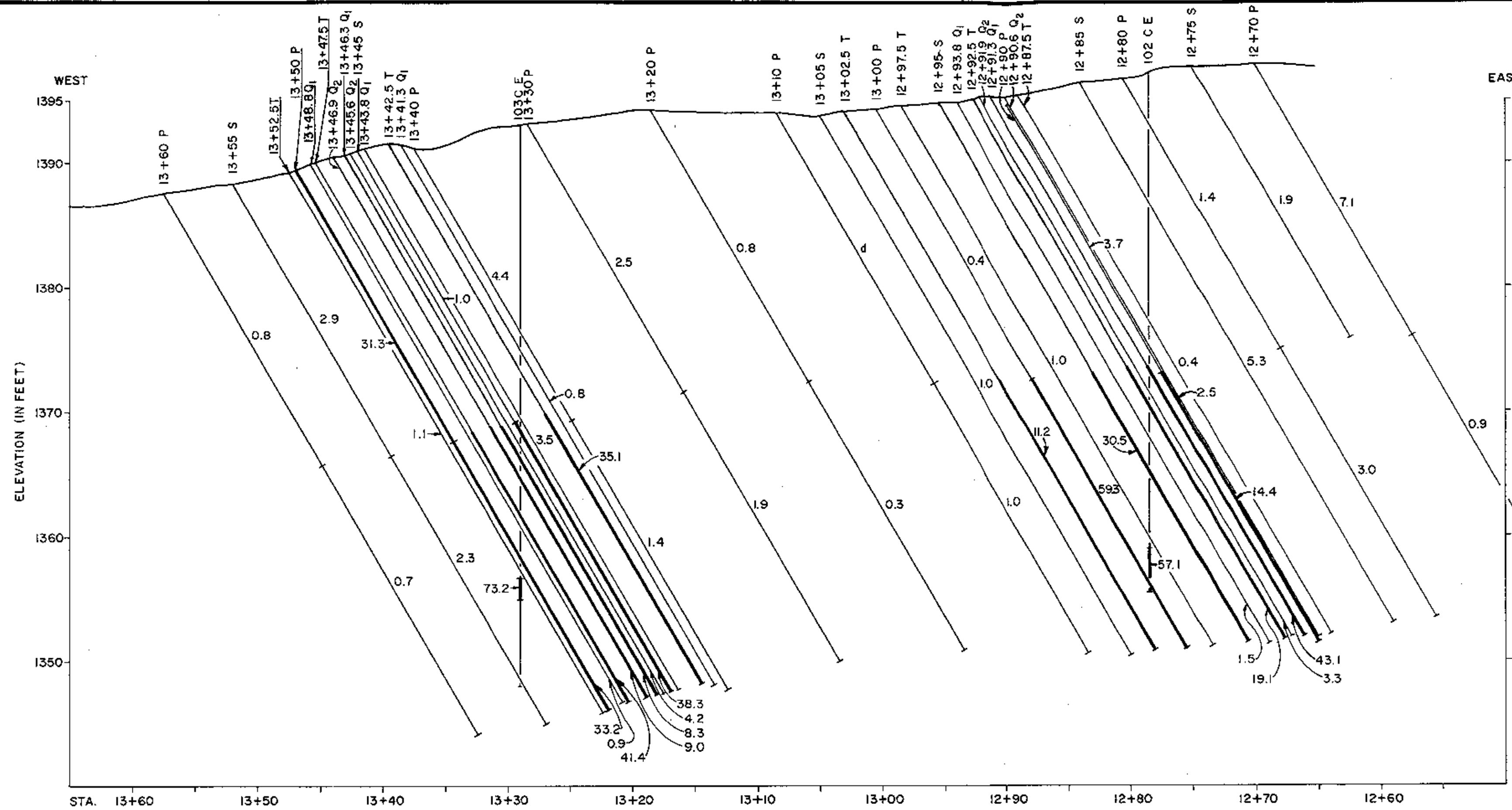


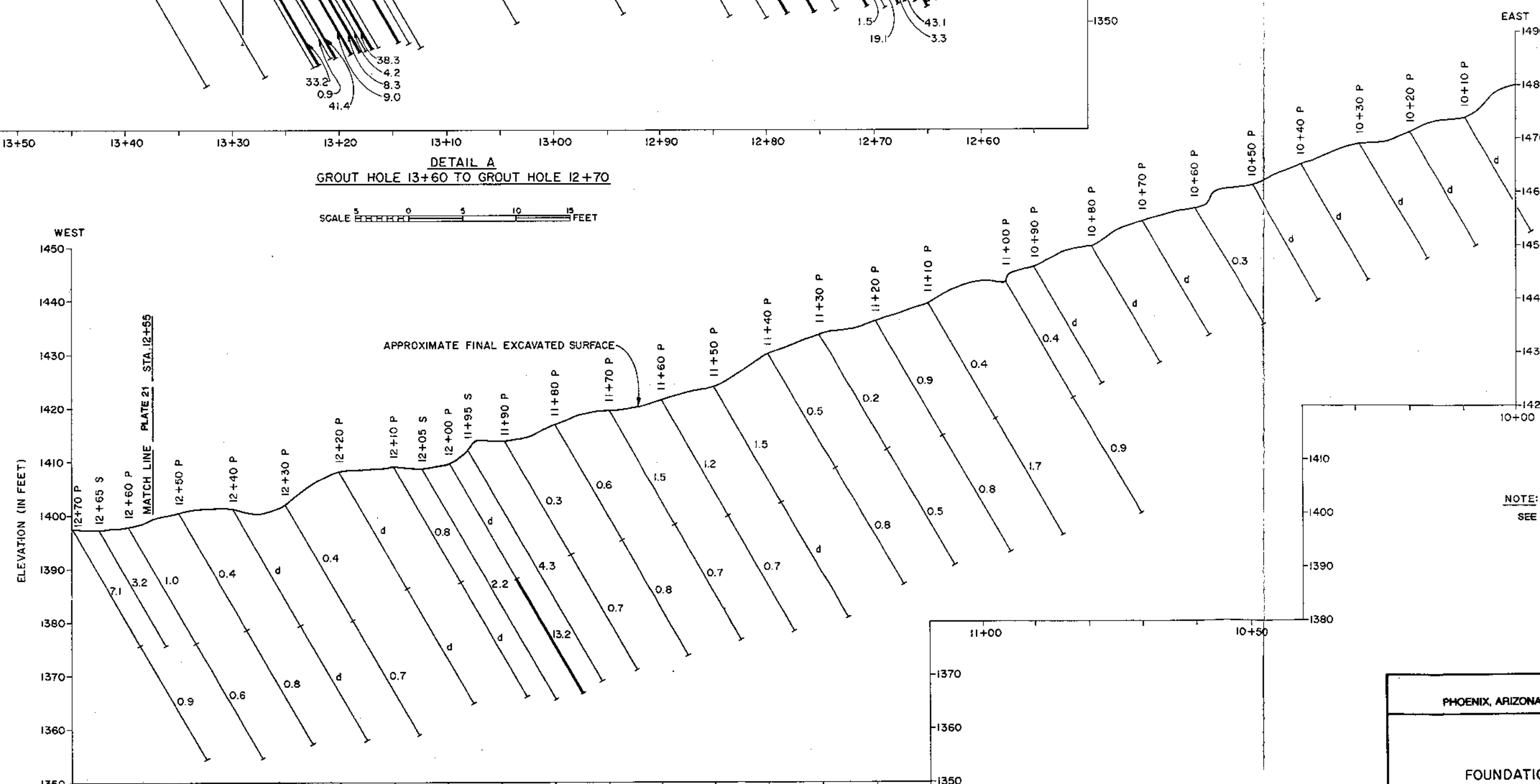
PLATE 21

SAFETY PAYS



DETAIL A
GROUT HOLE 13+60 TO GROUT HOLE 12+70

SCALE 5 0 5 10 15 FEET



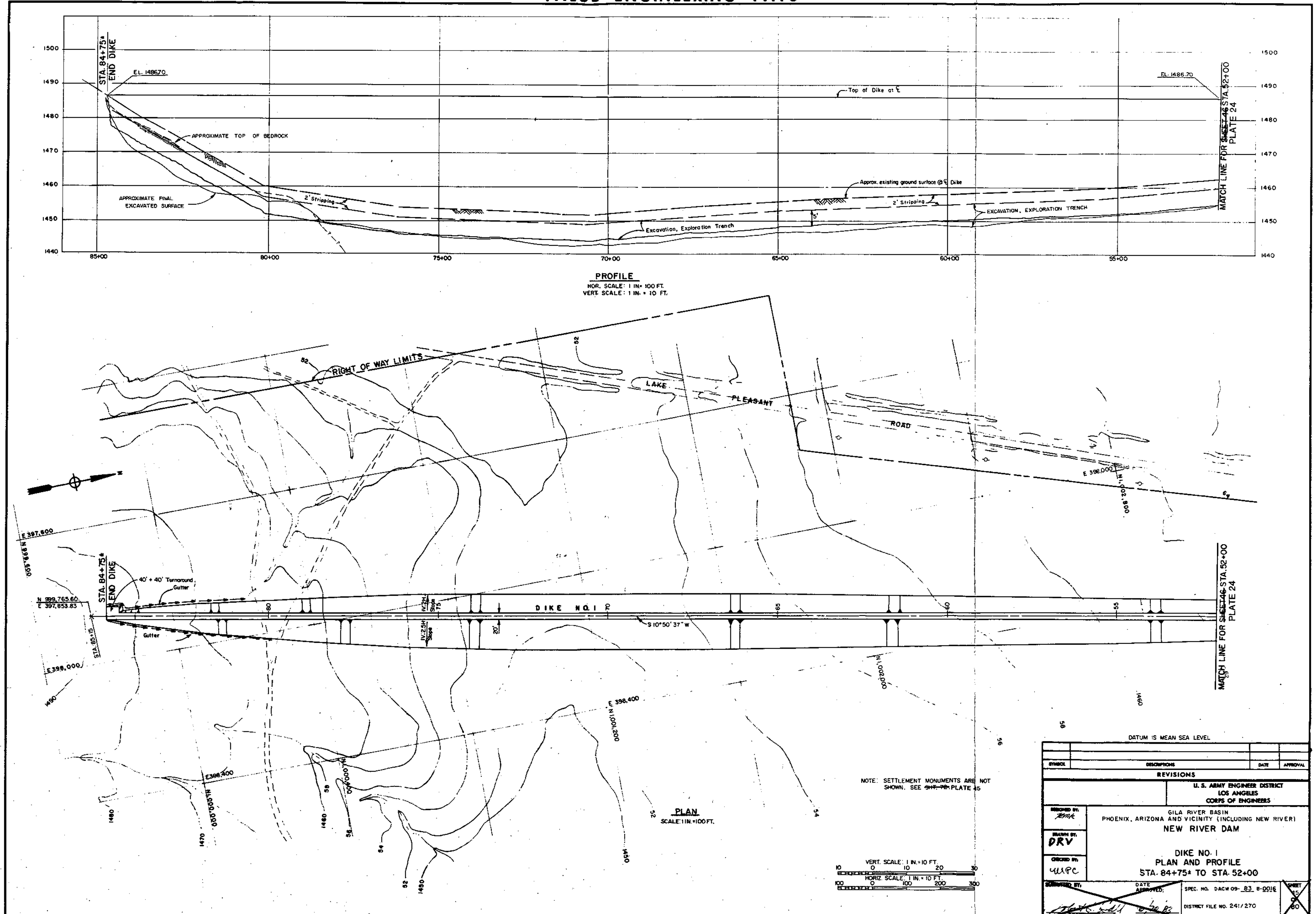
GROUT CURTAIN PROFILE (℄ CORE ZONE)

SCALE 10 0 10 20 30 FEET

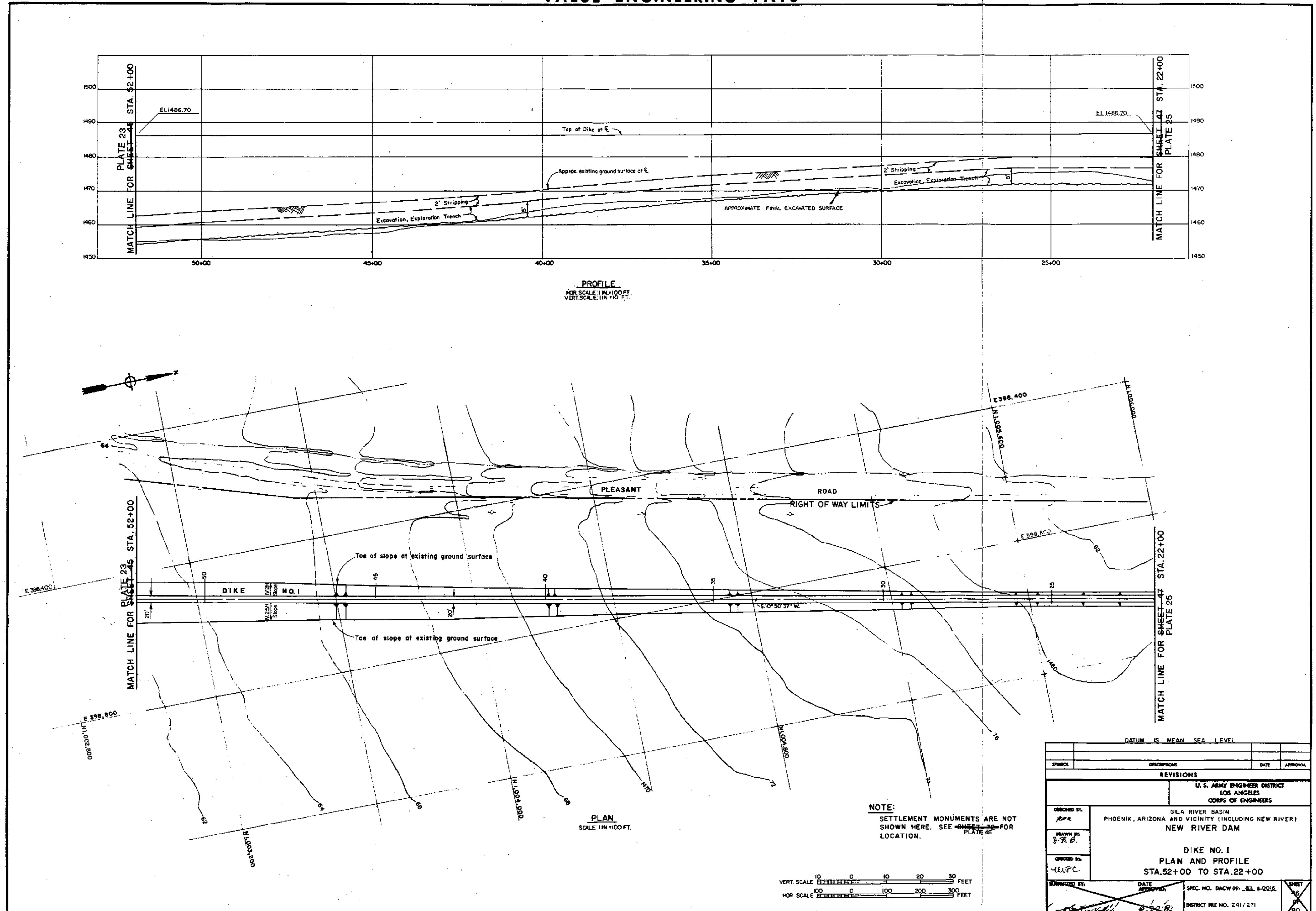
NOTE:
SEE PLATE 20 FOR NOTES AND LEGEND

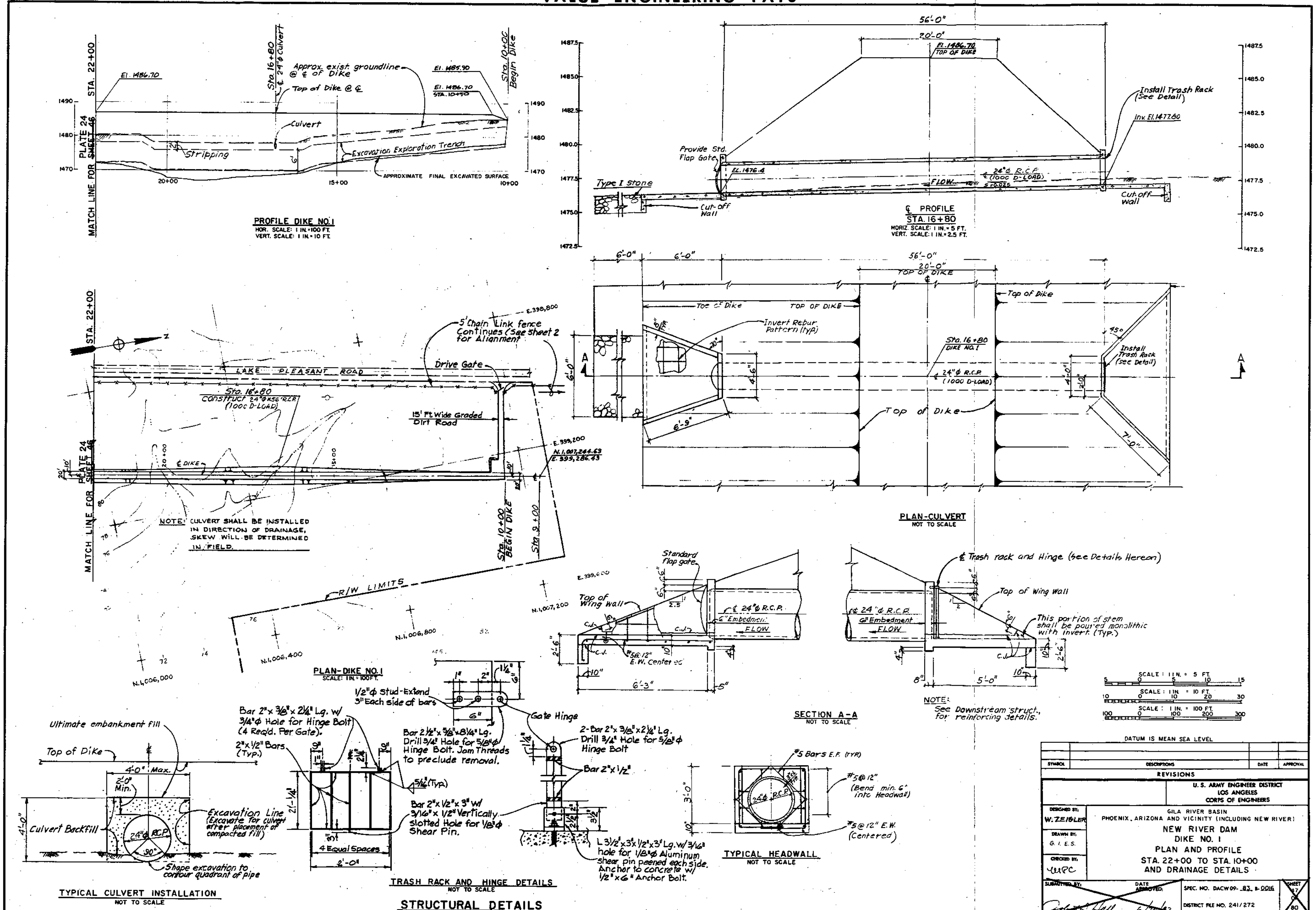
GILA RIVER BASIN PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
FOUNDATION GROUTING PROFILE STA. 12+55 TO STA. 10+00
U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

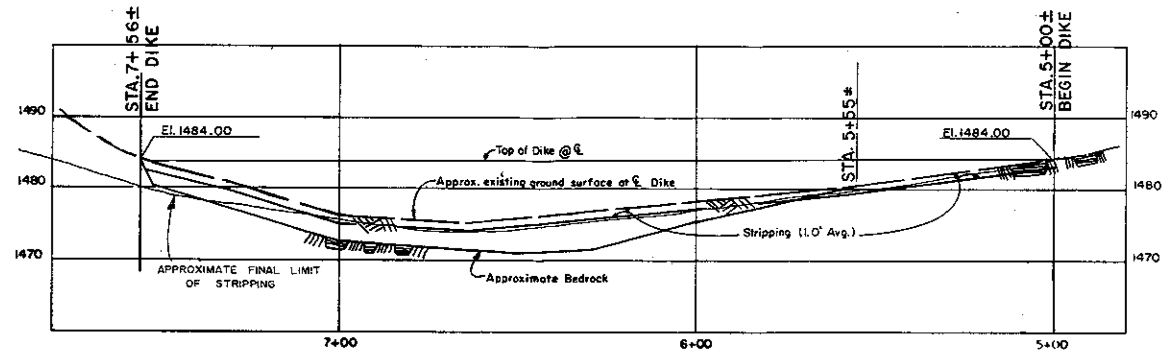
VALUE ENGINEERING PAYS



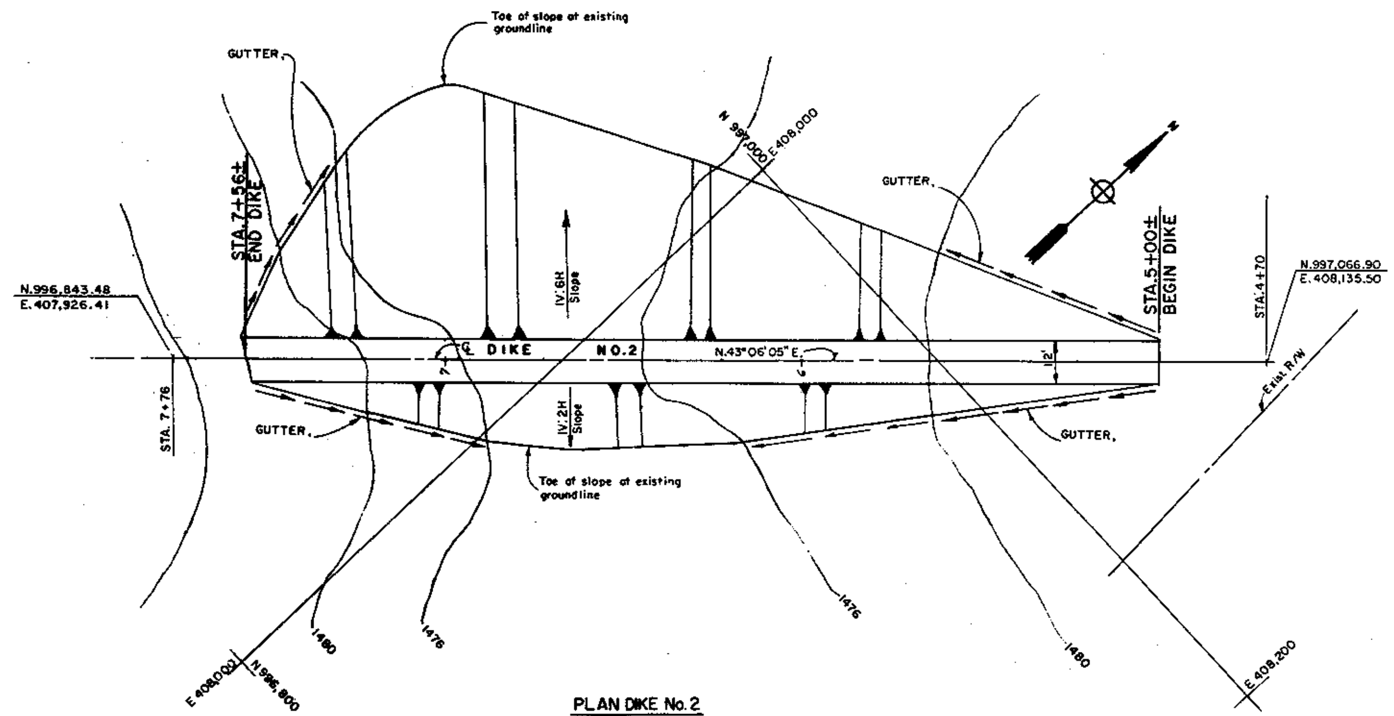
SAFETY PAYS


MATCH LINE FOR SHEET 45 STA. 52+00
PLATE 23
MATCH LINE FOR SHEET 47 STA. 22+00
PLATE 25
El. 1486.70
Top of Dike at ℄
Approx. existing ground surface at ℄
2' Stripping
Excavation, Exploration Trench
APPROXIMATE FINAL EXCAVATED SURFACE
50+00
45+00
40+00
35+00
30+00
25+00
1500
1490
1480
1470
1460
1450
PROFILE
HOR. SCALE: 1 IN. = 100 FT.
VERT. SCALE: 1 IN. = 10 FT.
E 398,400
N 1,005,600
N 1,006,000
PLEASANT
ROAD
RIGHT OF WAY LIMITS
E 398,800
Toe of slope at existing ground surface
DIKE
NO. 1
S.10°50'37"W.
1V:2H Slope
1V:2.5H Slope
20'
N 1,002,800
N 1,003,200
N 1,004,000
N 1,004,800
PLAN
SCALE: 1 IN. = 100 FT.
NOTE:
SETTLEMENT MONUMENTS ARE NOT SHOWN HERE. SEE ~~SHEET 76~~ PLATE 45 FOR LOCATION.
VERT. SCALE 10 0 10 20 30 FEET
HOR. SCALE 100 0 100 200 300 FEET
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:
SUBMITTED BY:
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DIKE NO. I
PLAN AND PROFILE
STA.52+00 TO STA.22+00
DATE APPROVED:
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/271
SHEET 46 OF 80

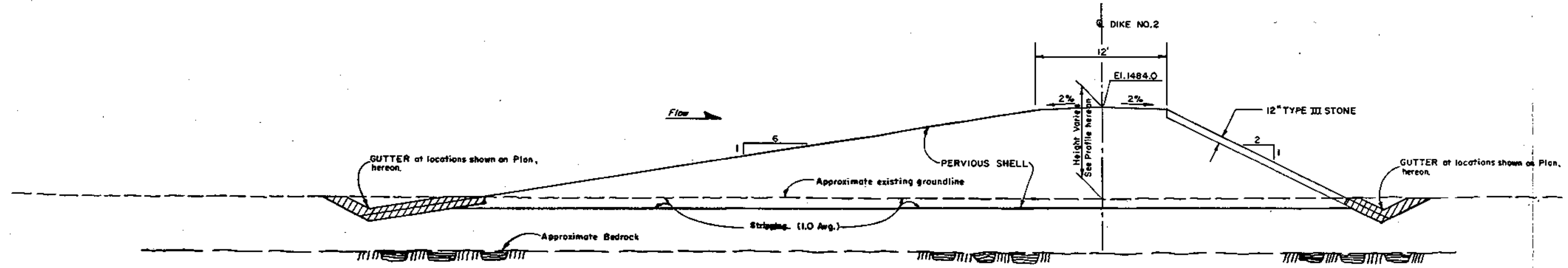

PROFILE DIKE NO. I
HOR. SCALE: 1 IN. = 100 FT.
VERT. SCALE: 1 IN. = 10 FT.
STA. 22+00
MATCH LINE FOR PLATE 24 SHEET 46
El. 1486.70
Approx. exist. groundline @ ℄ of Dike
Sta. 16+80 ℄ 24"φ Culvert
Top of Dike @ ℄
El. 1485.90
El. 1486.70 STA. 10+50
Sta. 10+00 Begin Dike
Culvert
Stripping
Excavation Exploration Trench
APPROXIMATE FINAL EXCAVATED SURFACE
20+00
15+00
10+00
℄ PROFILE
STA. 16+80
HORIZ. SCALE: 1 IN. = 5 FT.
VERT. SCALE: 1 IN. = 2.5 FT.
56'-0"
20'-0"
El. 1486.70 TOP OF DIKE
Install Trash Rack (See Detail)
Inv. El. 1477.60
Provide Std. Flap Gate
El. 1476.4
24"φ R.C.P. (1000 D-LOAD)
FLOW
Type I Stone
Cut-off Wall
PLAN-CULVERT
NOT TO SCALE
TOP OF DIKE
Top of Dike
Invert Rebar Pattern (typ)
Sta. 16+80 DIKE NO. I
24"φ R.C.P. (1000 D-LOAD)
Install Trash Rack (See Detail)
PLAN-DIKE NO. I
SCALE: 1 IN. = 100 FT.
5' Chain Link fence Continues (See Sheet 2 for Alignment)
LAKE PLEASANT ROAD
Drive Gate
Sta. 16+80 Construct 24"φ x 56' R.C.P. (1000 D-LOAD)
15' Ft. Wide Graded Dirt Road
℄ DIKE
E. 399,200
N. 1,007,244.63
E. 399,286.43
NOTE: CULVERT SHALL BE INSTALLED IN DIRECTION OF DRAINAGE, SKEW WILL BE DETERMINED IN FIELD.
Sta. 10+00 BEGIN DIKE
Sta. 9+00
R/W LIMITS
N. 1,007,200
N. 1,006,800
N. 1,006,400
N. 1,006,000
SECTION A-A
NOT TO SCALE
Standard Flap gate
Top of Wing Wall
℄ 24"φ R.C.P.
6" Embedment
FLOW
#5 @ 12" E.W. Centered
℄ Trash rack and Hinge (see Details Hereon)
Top of Wing Wall
This portion of stem shall be poured monolithic with invert. (Typ.)
NOTE: See Downstream struct. for reinforcing details.
SCALE: 1 IN. = 5 FT.
SCALE: 1 IN. = 10 FT.
SCALE: 1 IN. = 100 FT.
DATUM IS MEAN SEA LEVEL
TYPICAL CULVERT INSTALLATION
NOT TO SCALE
Ultimate embankment Fill
Top of Dike
4'-0" Max.
2'-0" Min.
Culvert Backfill
Excavation Line (Excavate for culvert after placement of compacted fill)
Shape excavation to contour quadrant of pipe
TRASH RACK AND HINGE DETAILS
NOT TO SCALE
STRUCTURAL DETAILS
1/2"φ Stud-Extend 3" Each side of bars
Bar 2" x 3/8" x 2 1/4" Lg. w/ 3/4"φ Hole for Hinge Bolt (4 Req'd. Per Gate).
2" x 1/2" Bars (Typ.)
Bar 2 1/2" x 3/8" x 8 1/4" Lg. Drill 3/4" Hole for 5/8"φ Hinge Bolt. Jam Threads to preclude removal.
Gate Hinge
2-Bar 2" x 3/8" x 2 1/4" Lg. Drill 3/4" Hole for 5/8"φ Hinge Bolt
Bar 2" x 1/2"
Bar 2" x 1/2" x 3" w/ 3/16" x 1/2" Vertically slotted Hole for 1/8"φ Shear Pin.
4 Equal Spaces
2'-0"
L 3 1/2" x 3" x 1/2" x 3" Lg. w/ 3/16" hole for 1/8"φ Aluminum shear pin peened each side. Anchor to concrete w/ 1/2" x 6" Anchor Bolt.
TYPICAL HEADWALL
NOT TO SCALE
#5 Bars E.F. (Typ)
#5 @ 12" (Bend min. 6" into Headwall)
#5 @ 12" E.W. (Centered)
REVISIONS
U. S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: W. ZEIGLER
DRAWN BY: G. L. E. S.
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DIKE NO. I
PLAN AND PROFILE
STA. 22+00 TO STA. 10+00
AND DRAINAGE DETAILS
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/272
SHEET 47 OF 80



PROFILE DIKE No. 2
HOR. SCALE: 1 IN. = 20 FT.
VERT. SCALE: 1 IN. = 10 FT.



PLAN DIKE No. 2
SCALE: 1 IN. = 20 FT.



TYPICAL SECTION: DIKE NO. 2
SCALE: 1 IN. = 5 FT.

SCALE: 1 IN. = 5 FT.
5 0 5 10 15

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

SCALE: 1 IN. = 20 FT.
10 0 10 20 30 40 50 60 70

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

DESIGNED BY: J.B.
DRAWN BY: DRV
CHECKED BY: MPC.

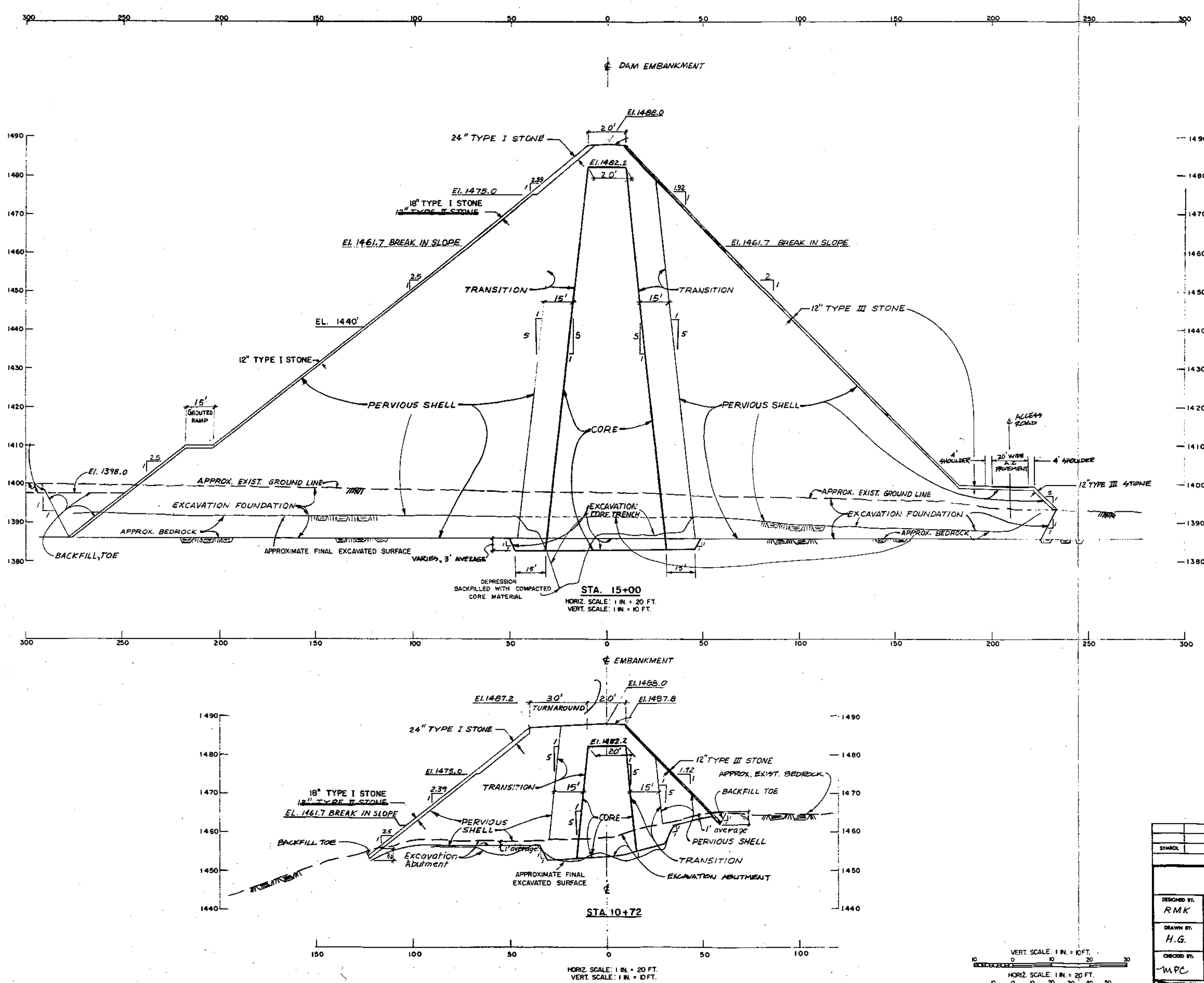
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

DIKE NO. 2
PLAN AND PROFILE AND TYPICAL SECTION

SUBMITTED BY: DATE APPROVED:
DESIGN BRANCH

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/274
SHEET 80

PLATE 26



NOTE: VIEW OF CROSS-SECTIONS ARE LOOKING TOWARD DECREASING STATIONS.

VERT. SCALE: 1 IN. = 10 FT.

HORIZ. SCALE: 1 IN. = 20 FT.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

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

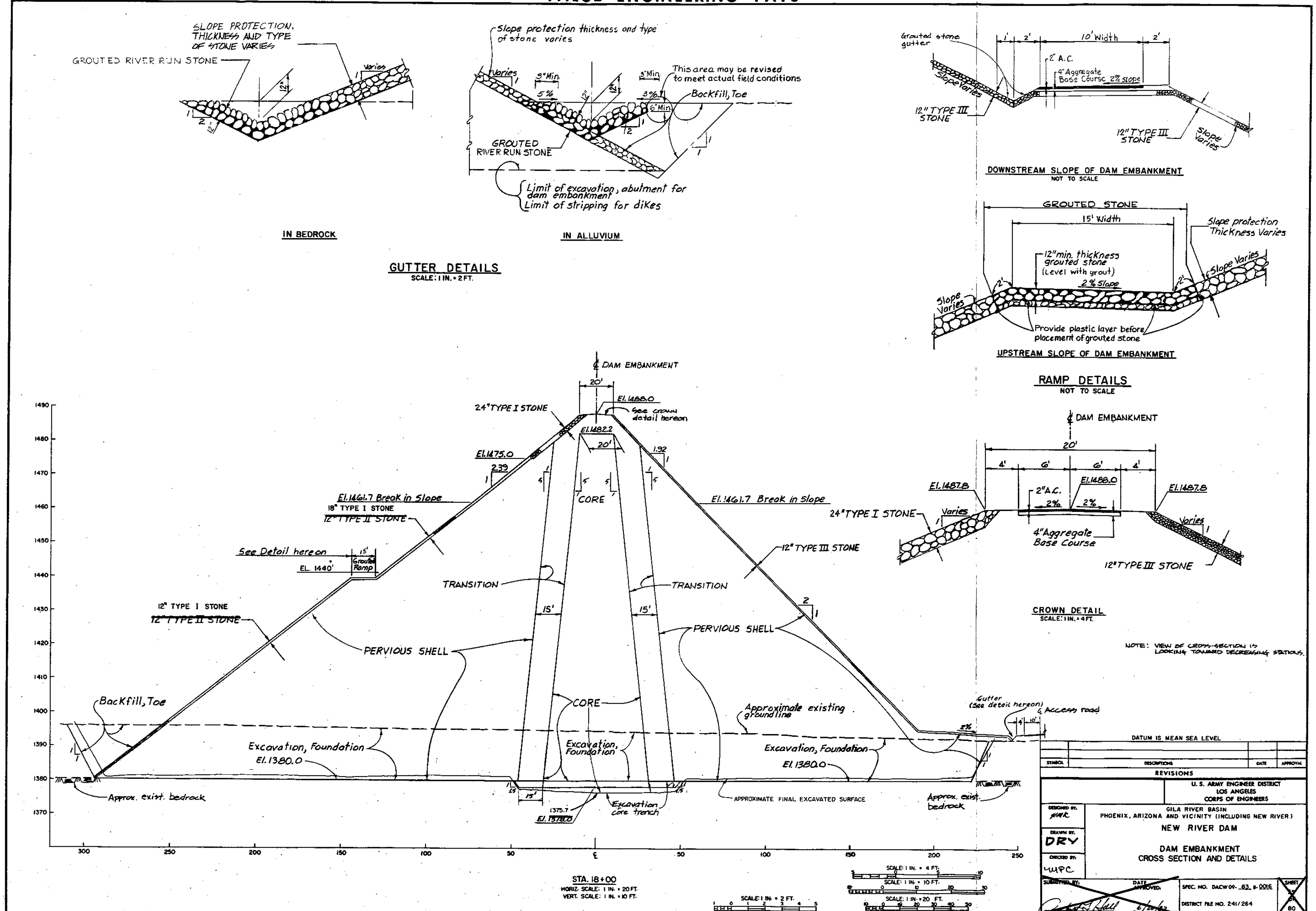
DESIGNED BY: RMK
DRAWN BY: H.G.
CHECKED BY: MPC

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

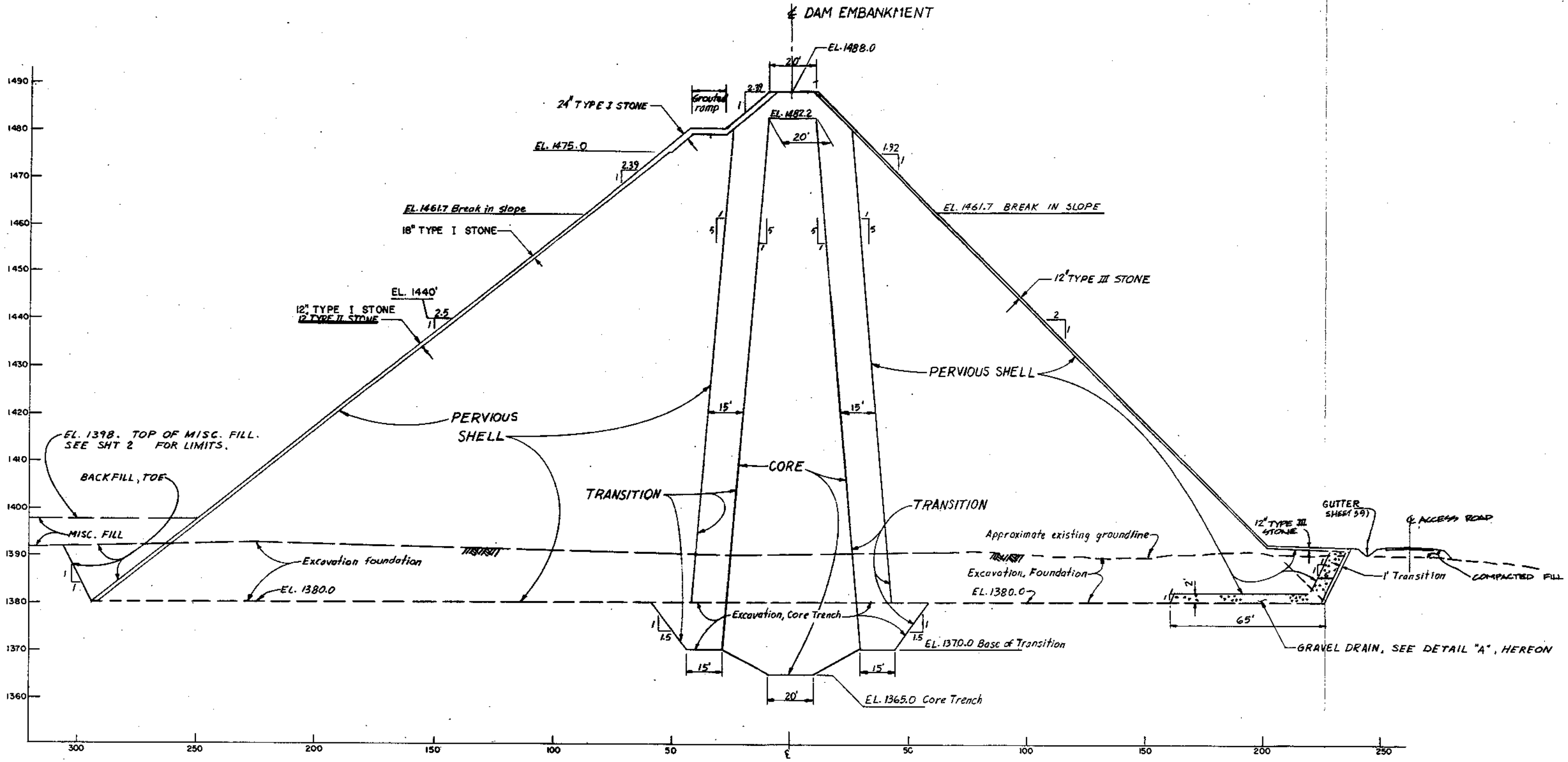
DAM EMBANKMENT
CROSS SECTIONS

SUBMITTED BY: CHIEF, DESIGN BRANCH
DATE APPROVED: 6/30/83

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/263
SHEET 38 OF 80

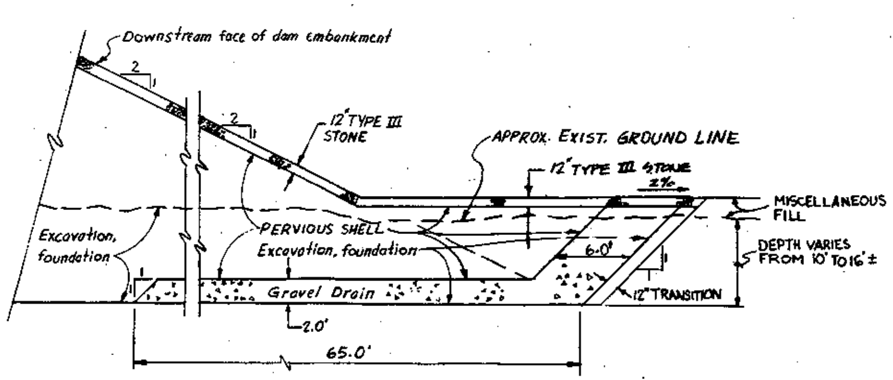


SLOPE PROTECTION, THICKNESS AND TYPE OF STONE VARIES
GROUTED RIVER RUN STONE
IN BEDROCK
Slope protection thickness and type of stone varies
This area may be revised to meet actual field conditions
Backfill, Toe
GROUTED RIVER RUN STONE
Limit of excavation, abutment for dam embankment
Limit of stripping for dikes
IN ALLUVIUM
GUTTER DETAILS
SCALE: 1 IN. = 2 FT.
Grouted stone gutter
10' Width
2" A.C.
4" Aggregate Base Course 2% slope
12" TYPE III STONE
Slope Varies
DOWNSTREAM SLOPE OF DAM EMBANKMENT
NOT TO SCALE
GROUTED STONE
15' Width
Slope protection Thickness Varies
12" min. thickness grouted stone (Level with grout)
2% Slope
Slope Varies
Provide plastic layer before placement of grouted stone
UPSTREAM SLOPE OF DAM EMBANKMENT
RAMP DETAILS
NOT TO SCALE
℄ DAM EMBANKMENT
El. 1488.0
See crown detail hereon
24" TYPE I STONE
El. 1482.2
El. 1475.0
El. 1461.7 Break in Slope
18" TYPE I STONE
12" TYPE II STONE
See Detail hereon
EL. 1440'
Grouted Ramp
CORE
TRANSITION
12" TYPE III STONE
12" TYPE I STONE
PERVIOUS SHELL
Backfill, Toe
Approximate existing groundline
Excavation, Foundation
El. 1380.0
Excavation, Foundation
Excavation core trench
El. 1378.0
Approx. exist. bedrock
APPROXIMATE FINAL EXCAVATED SURFACE
Gutter (See detail hereon)
℄ Access road
4' Aggregate Base Course
CROWN DETAIL
SCALE: 1 IN. = 4 FT.
NOTE: VIEW OF CROSS-SECTION IS LOOKING TOWARD DECREASING STATIONS.
STA. 18+00
HORIZ. SCALE: 1 IN. = 20 FT.
VERT. SCALE: 1 IN. = 10 FT.
DATUM IS MEAN SEA LEVEL
REVISIONS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DAM EMBANKMENT
CROSS SECTION AND DETAILS
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/264

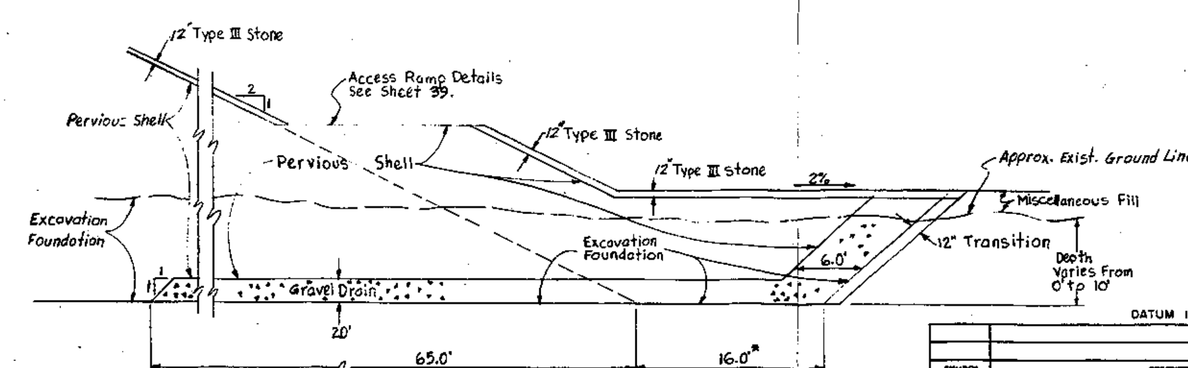


STA. 21+50
HORIZ. SCALE: 1 IN. = 20 FT.
VERT. SCALE: 1 IN. = 10 FT.

NOTE: VIEW OF CROSS-SECTION IS LOOKING TOWARD DECREASING STATIONS.



DETAIL "A": GRAVEL DRAIN
STA. 19+00 TO STA. 23+30
SCALE: 1 IN. = 6 FT.



*Varies from 0' at Sta. 23+30 to 16' at Sta. 23+90.

DETAIL "B" GRAVEL DRAIN
STA. 23+30 TO STA. 31+00
SCALE: 1 IN. = 6 FT.

SCALE: 1 IN. = 6 FT.
SCALE: 1 IN. = 10 FT.
SCALE: 1 IN. = 20 FT.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

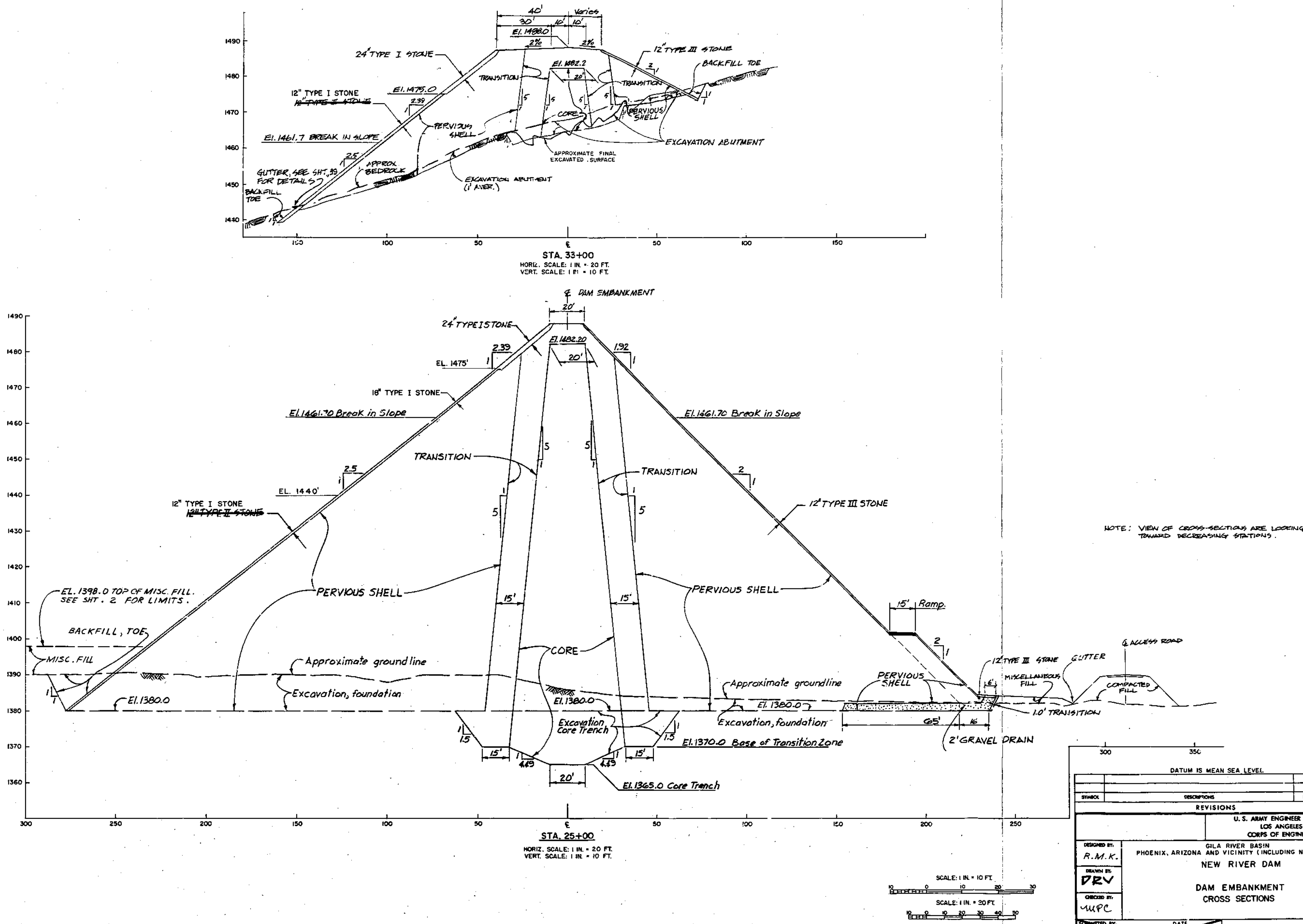
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: R.M.K.
DRAWN BY: J.W.
CHECKED BY: WPC

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

DAM EMBANKMENT
CROSS SECTION AND DETAIL

SUBMITTED BY: [signature] DESIGN BRANCH
DATE APPROVED: 2/20/83
SPEC. NO. DACW09- 83 - B- 0016
DISTRICT FILE NO. 241/265
SHEET 37 OF 80



DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

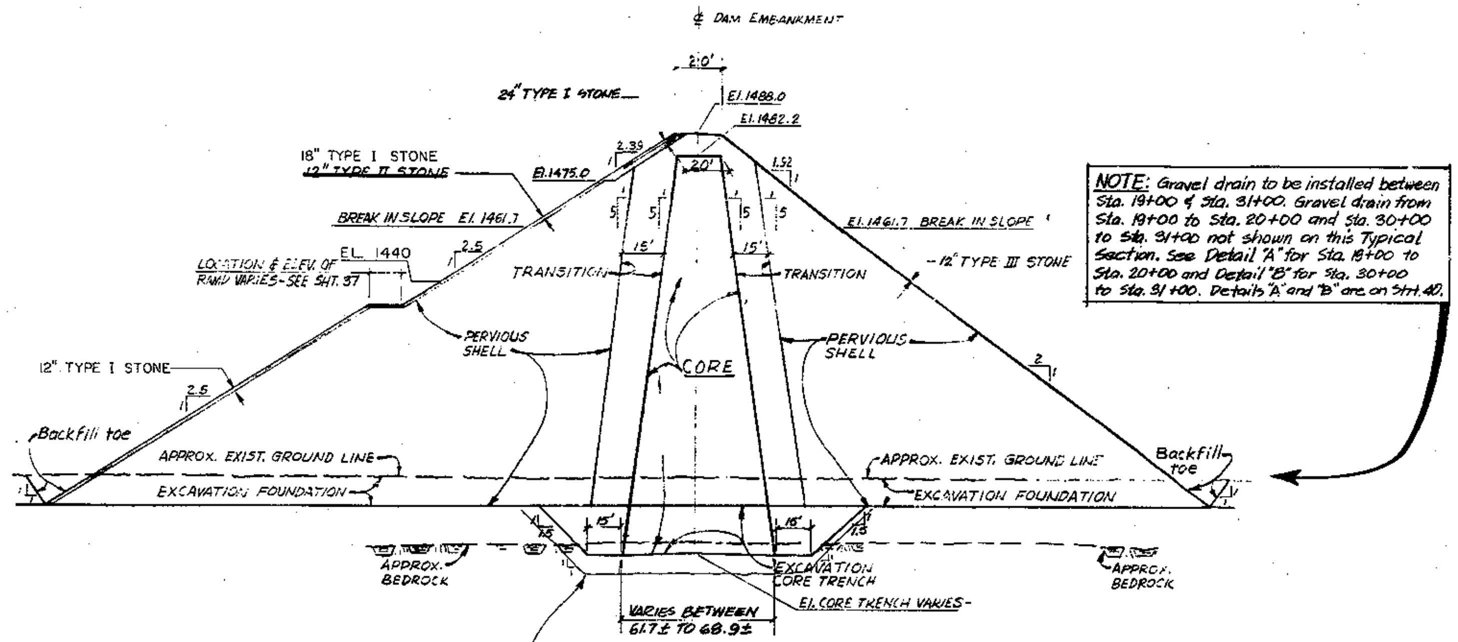
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: R.M.K.
DRAWN BY: DRV
CHECKED BY: MPC

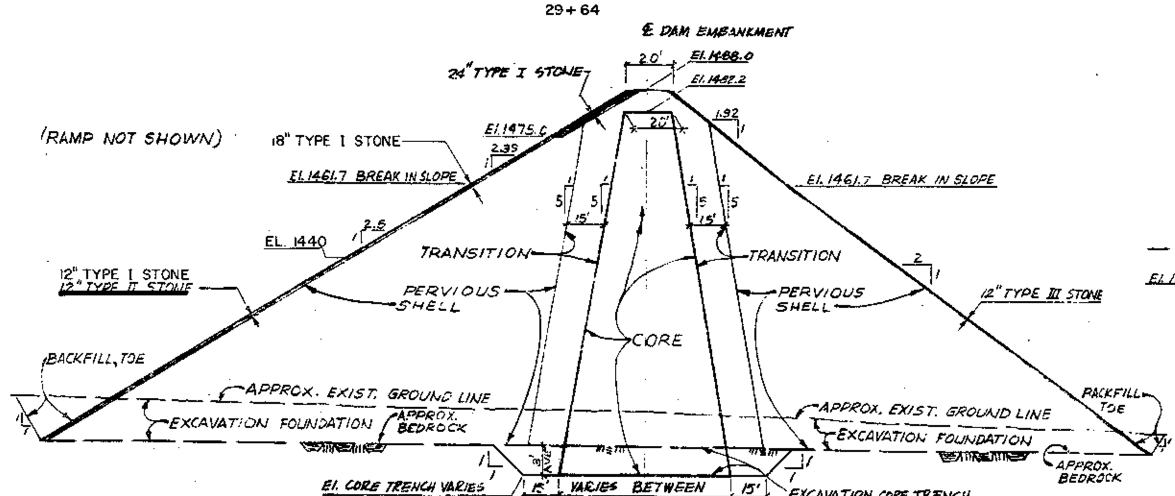
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

DAM EMBANKMENT
CROSS SECTIONS

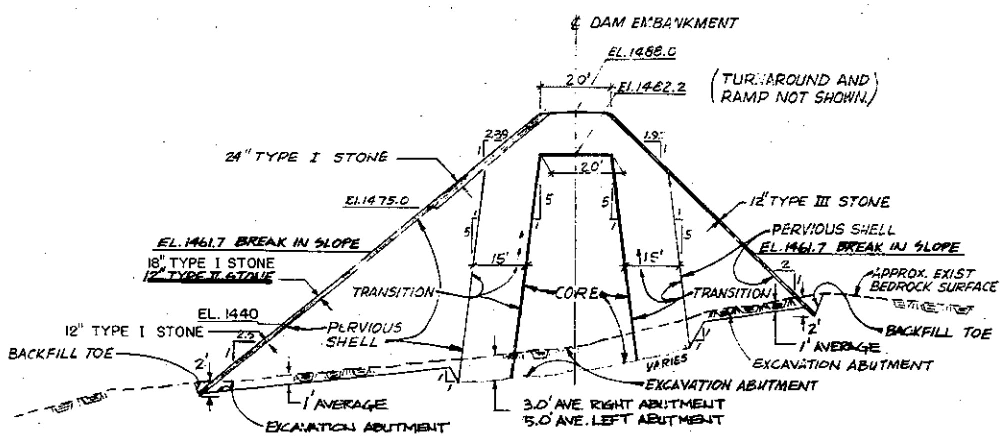
SUBMITTED BY: [signature] DATE APPROVED: 6/20/83
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/266
SHEET 41 OF 80



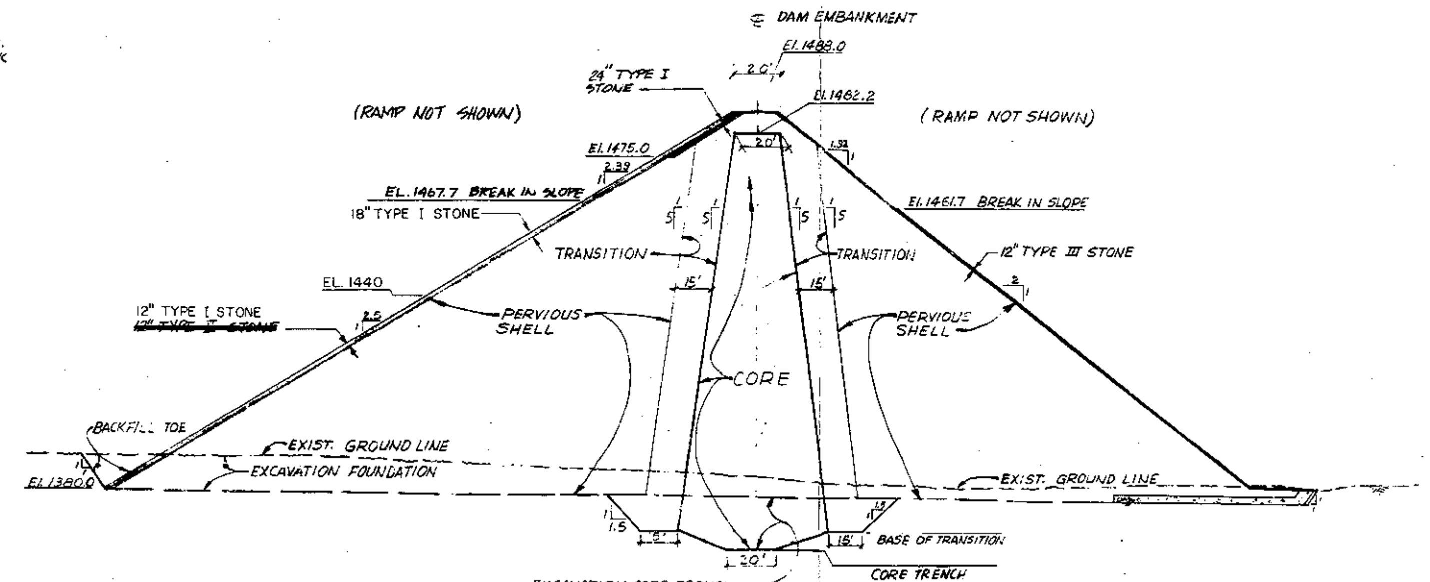
TYPICAL SECTION: STA. ~~18+00~~ 16+50± TO STA. ~~20+00~~ 21+06
STA. ~~30+00~~ 29+64 TO STA. 31+90
NOT TO SCALE



TYPICAL SECTION: STA. 13+00 TO STA. ~~18+00~~ 16+50±
NOT TO SCALE



TYPICAL SECTON: LEFT AND RIGHT ABUTMENTS
STA. 10+00± TO STA. 13+00
STA. 31+90± TO STA. 33+27±
NOT TO SCALE



TYPICAL SECTION: STA. ~~20+00~~ 21+06 TO STA. ~~30+00~~ 29+64±
NOT TO SCALE

NOTE: VIEW OF CROSS-SECTIONS ARE LOOKING TOWARD DECREASING STATIONS.

GENERAL NOTES

1. For gutter location and details at East and West abutments, see ~~shts. 37 and 38.~~ PLATES 18 AND 28
2. Landscape plan and planting schemes of downstream slope of dam embankment not shown. For details see shts. 78 and 79.
3. For gravel drain details, see ~~sht. 40.~~ PLATE 29
4. For embankment crown details, see ~~sht. 39~~ PLATE 28
5. For ramp details, see ~~sht. 89~~ PLATE 28
6. Misc. fill area and excavation upstream of dam embankment not shown on typical sections. See ~~sheet 2~~ PLATE 1 for location and limits.

DATUM IS MEAN SEA LEVEL

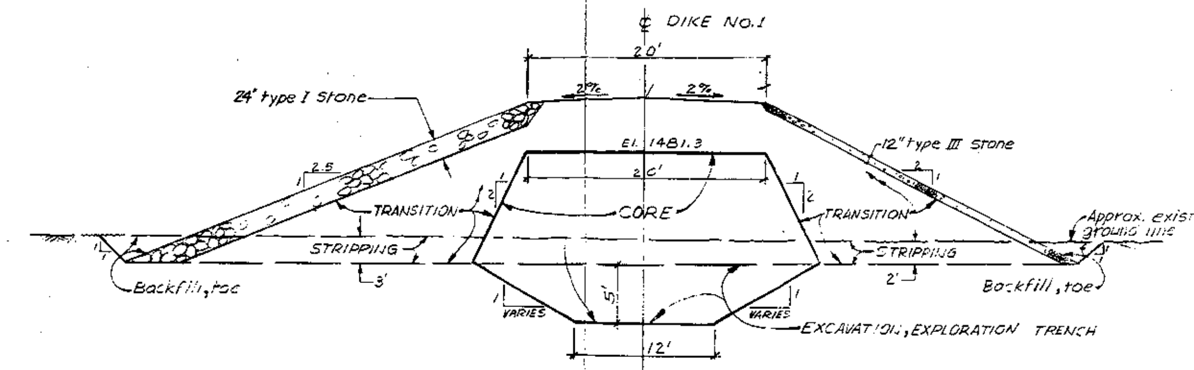
SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

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

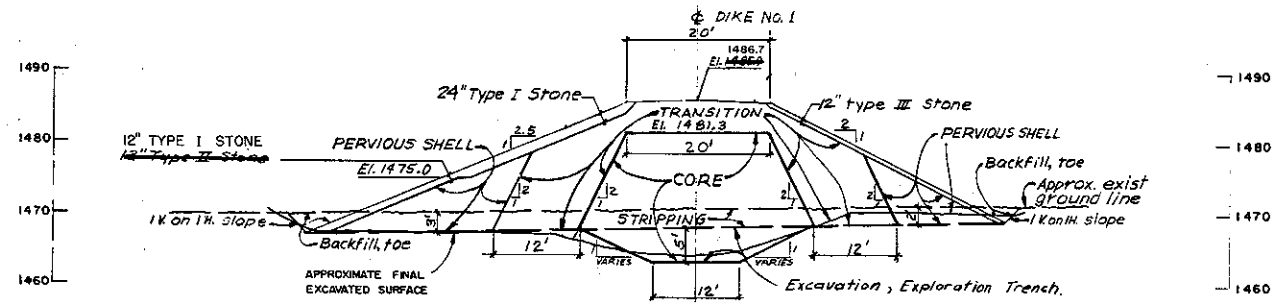
DESIGNED BY: R.M.K.
DRAWN BY: H.G.
CHECKED BY: MPC

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DAM EMBANKMENT
TYPICAL SECTIONS

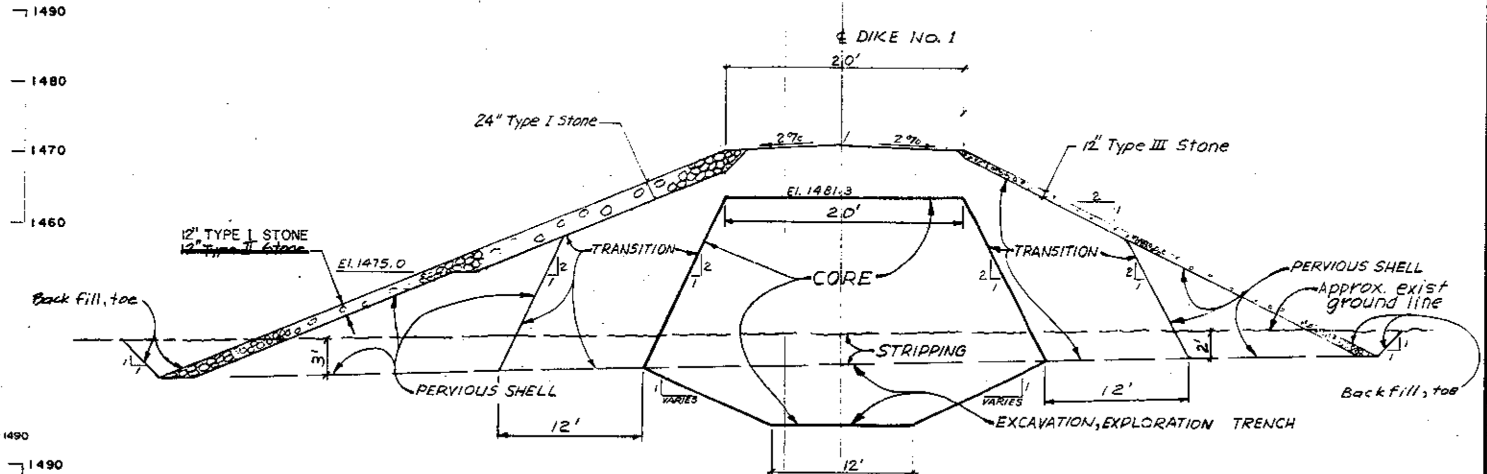
SUBMITTED BY: / DATE APPROVED:
SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/267
SHEET 42 OF 80



STA. 84+75± TO STA. 82+75
STA. 35+00 TO STA. 10+00±



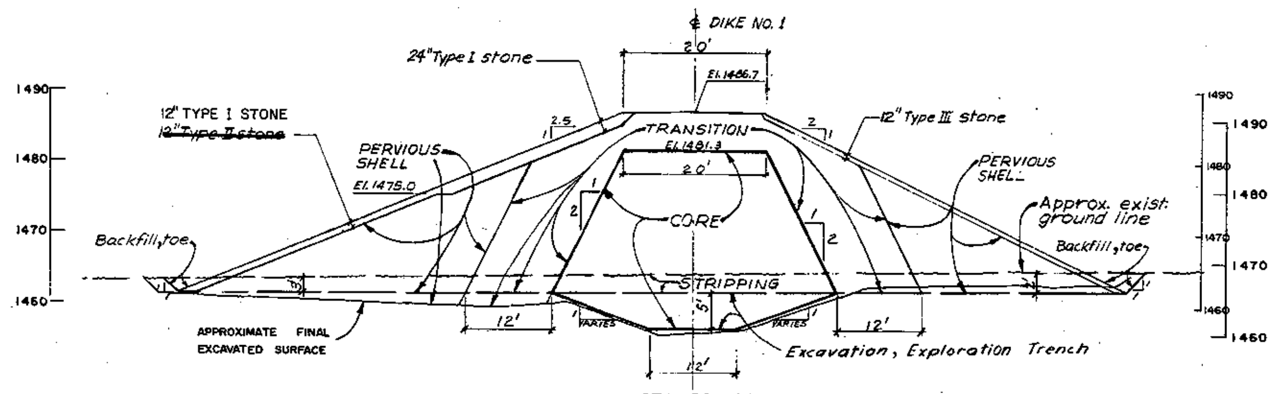
STA. 40+00
1 IN. = 10FT.



STA. 82+75 TO STA. 35+00

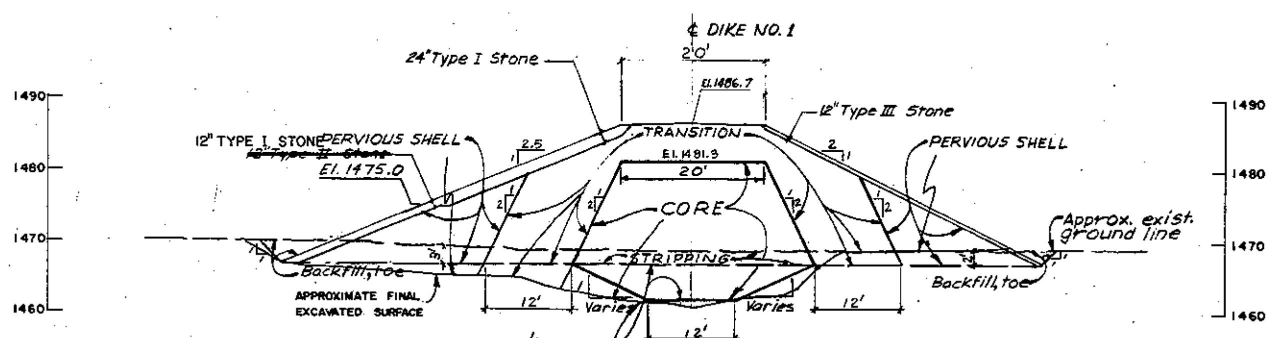
TYPICAL SECTIONS - DIKE NO. 1

NOT TO SCALE

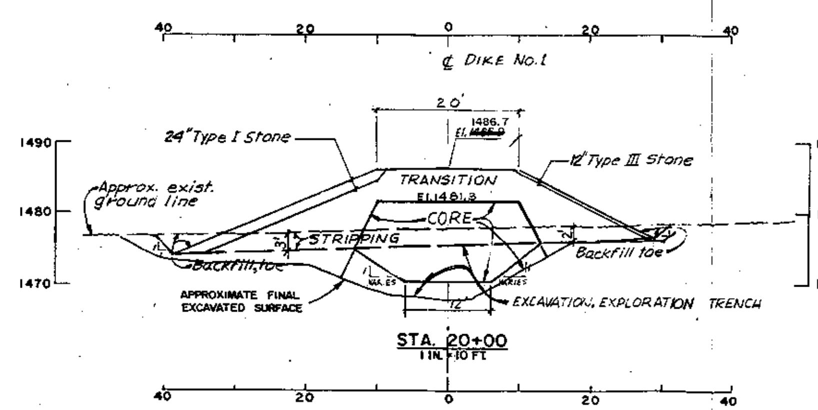


STA. 50+00
1 IN. = 10 FT.

NOTE: STRIPPING FOR DIKE NO. 1 TYPICALLY VARIES FROM 2' AT THE DOWNSTREAM TOE TRENCH TO 3' AT THE UPSTREAM TOE TRENCH.



STA. 82+00
1 IN. = 10 FT.



STA. 20+00
1 IN. = 10 FT.

SCALE: 1 IN. = 10 FT.

DATUM IS MEAN SEA LEVEL.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
REVISIONS			

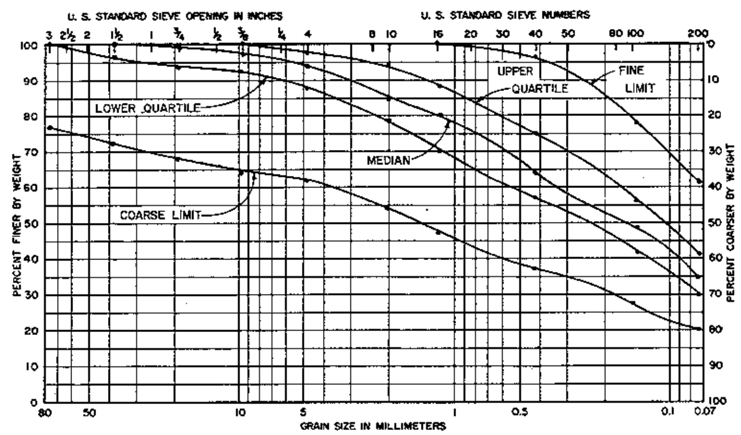
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: R.M.K.
DRAWN BY: G.I.E.S.
CHECKED BY:

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM
DIKE No. 1
CROSS SECTIONS AND TYPICAL SECTIONS

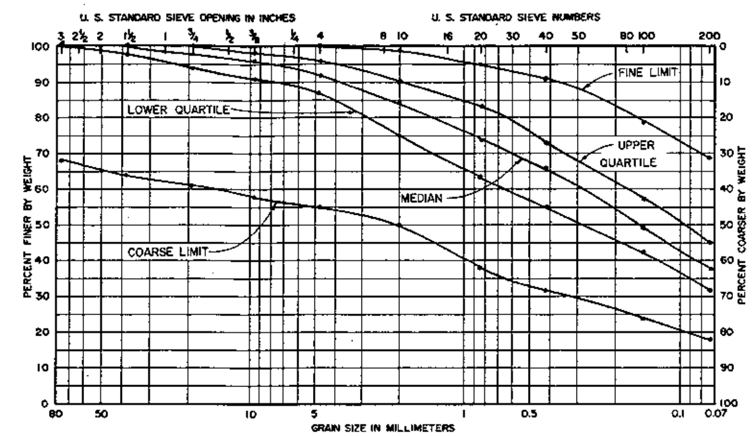
SUBMITTED BY: ... DESIGN BRANCH
DATE APPROVED: .../83
SPEC. NO. DACW 09-83-B-0016
DISTRICT FILE NO. 241/273
SHEET 49 OF 80

PLATE 32



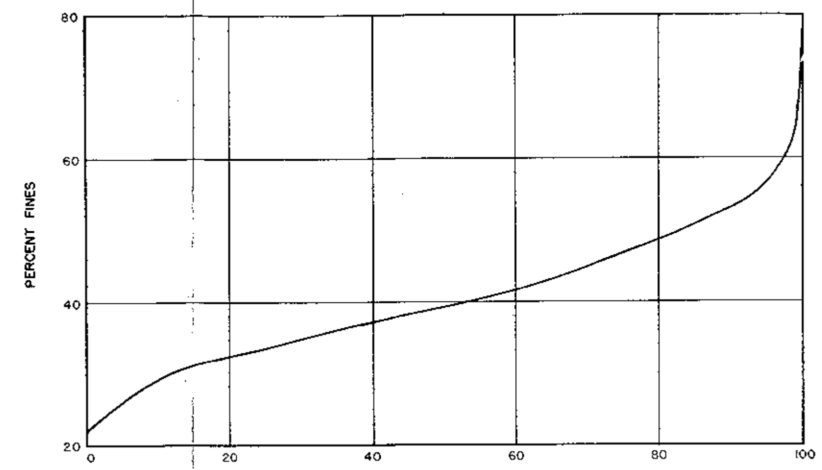
GRADATION CURVES

GDM

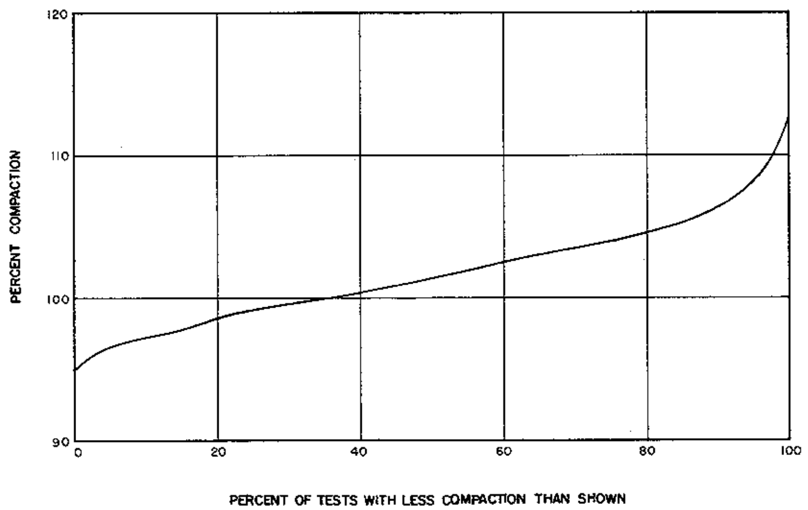


GRADATION CURVES

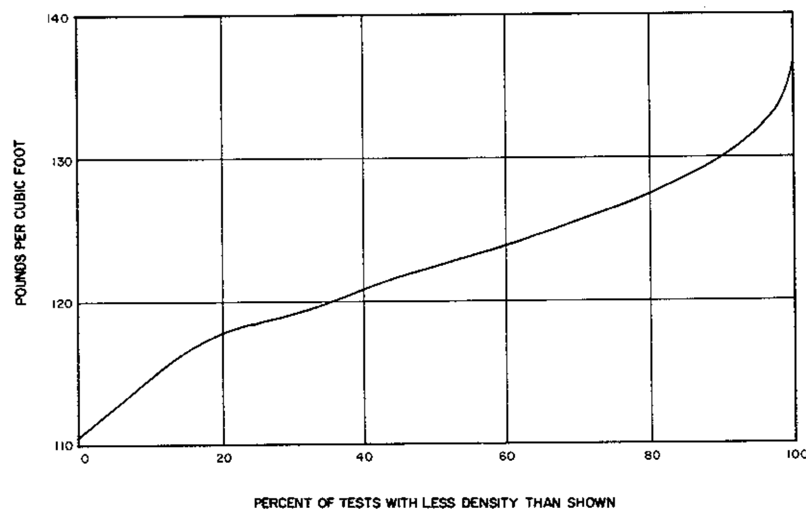
AS-PLACED



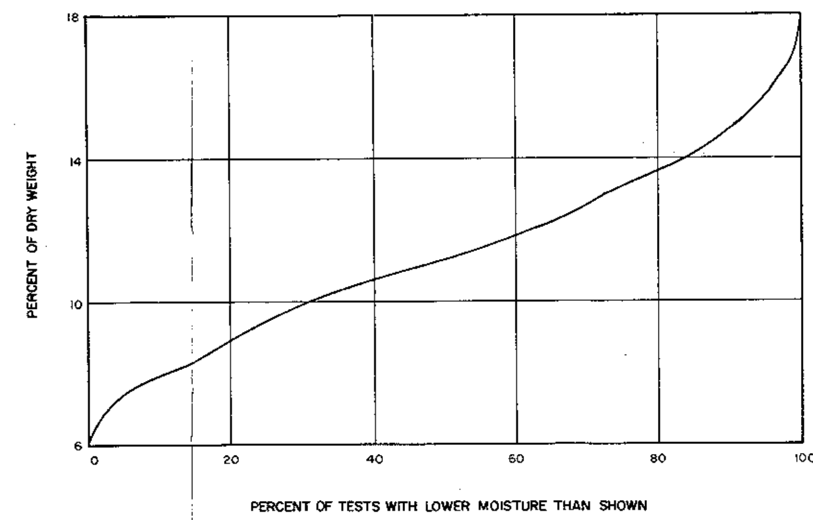
FINES



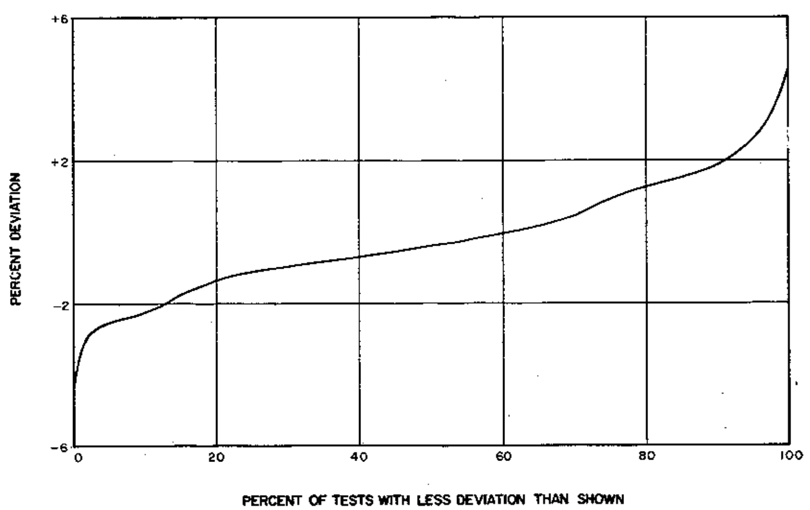
COMPACTION



DRY DENSITY



MOISTURE CONTENT



MOISTURE CONTENT DEVIATION FROM LABORATORY OPTIMUM

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

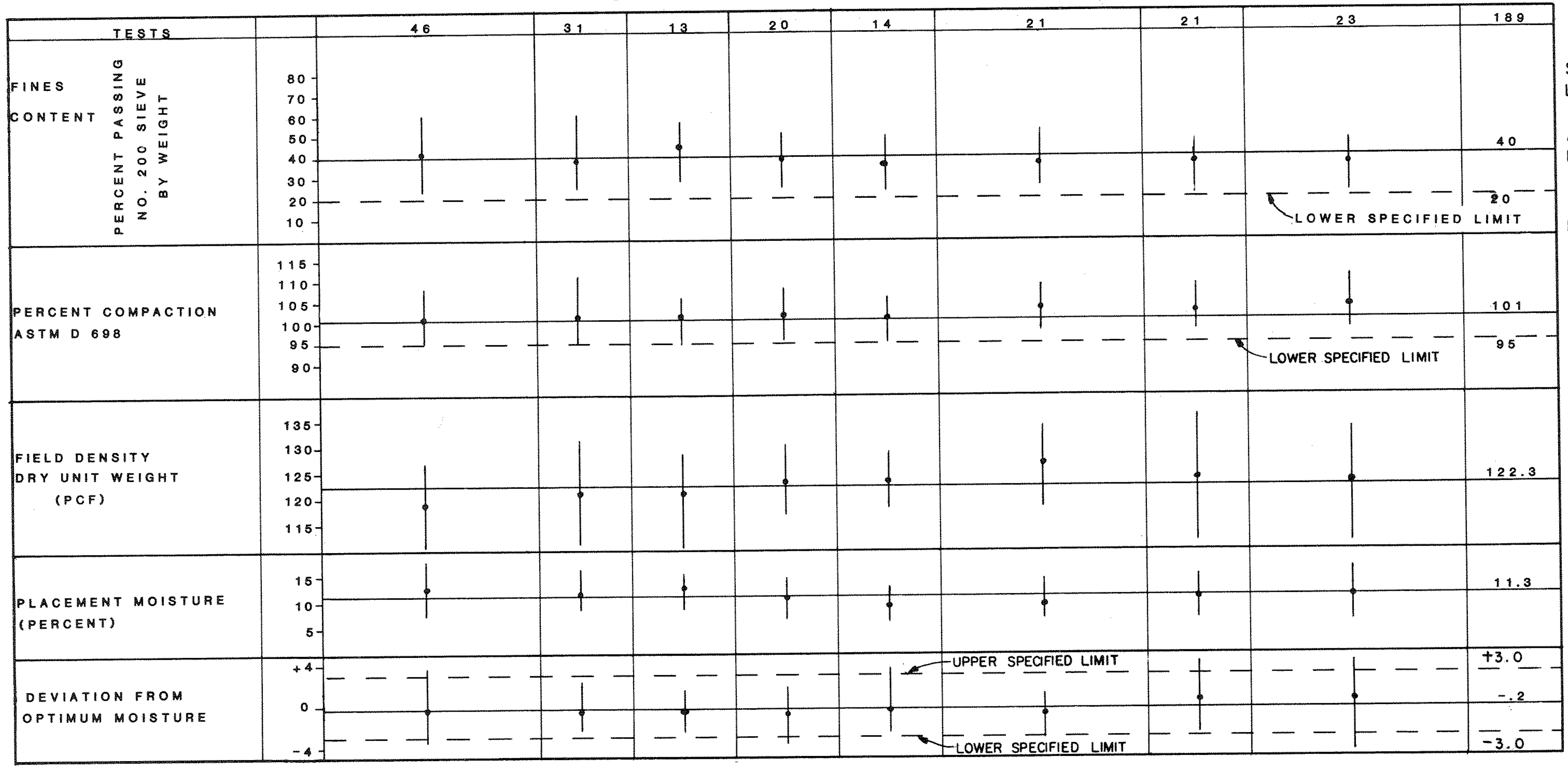
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

CORE MATERIAL
FIELD CONTROL DATA

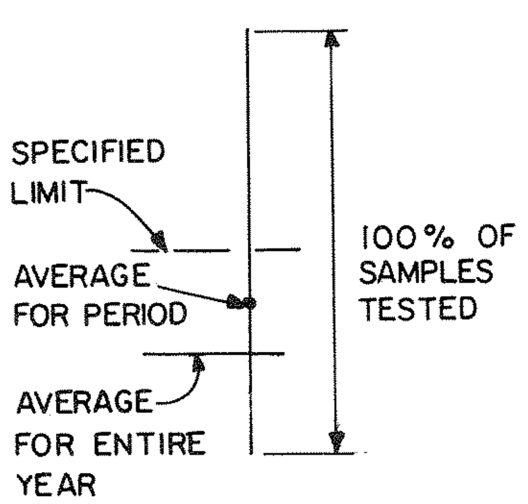
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

YEAR 1984	MONTH	JANUARY-FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY-AUGUST	OCTOBER	NOVEMBER-DECEMBER	SUMMARY

CONSTRUCTION CONTROL DATA



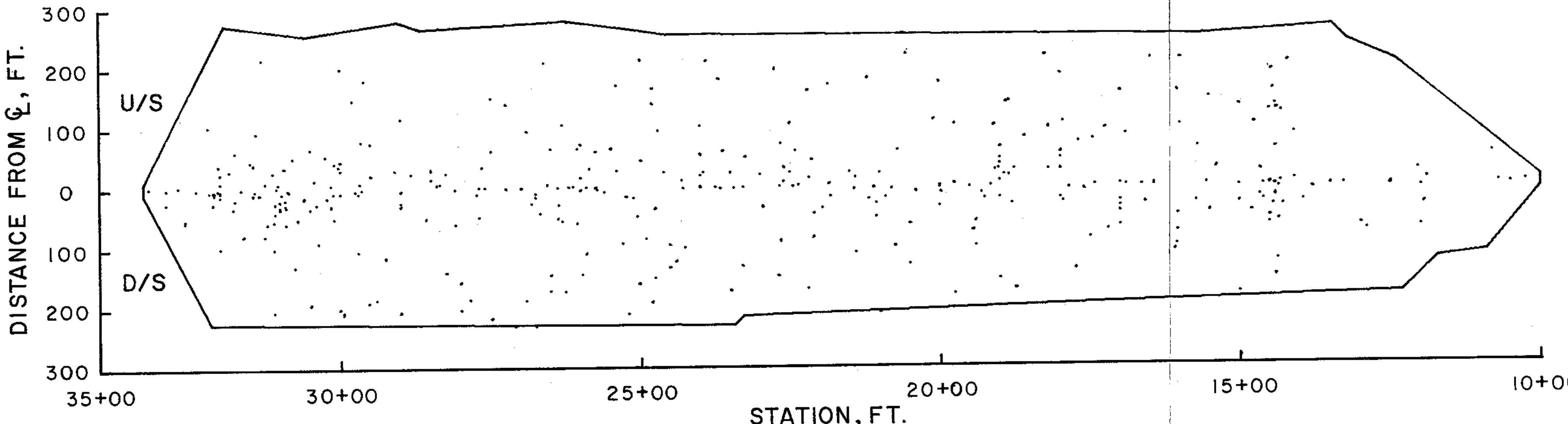
LEGEND



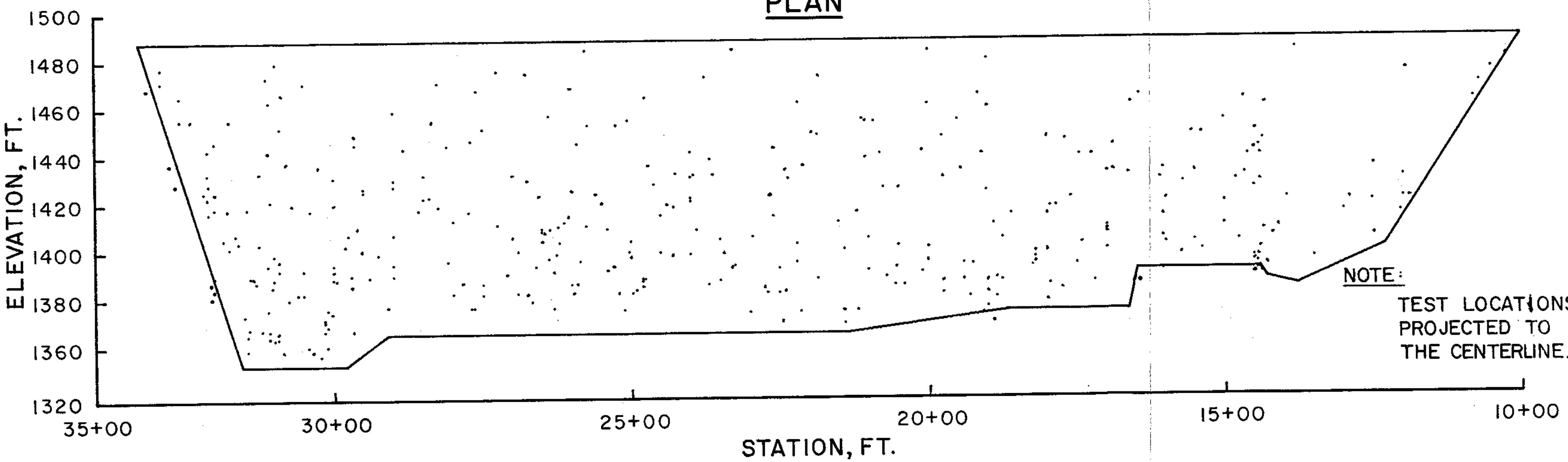
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY
(INCLUDING NEW RIVER)

NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT
CORE MATERIAL
MONTHLY QUALITY ASSURANCE TESTING

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



PLAN



PROFILE AT ℄

LEGEND


FIELD CONTROL TEST LOCATIONS

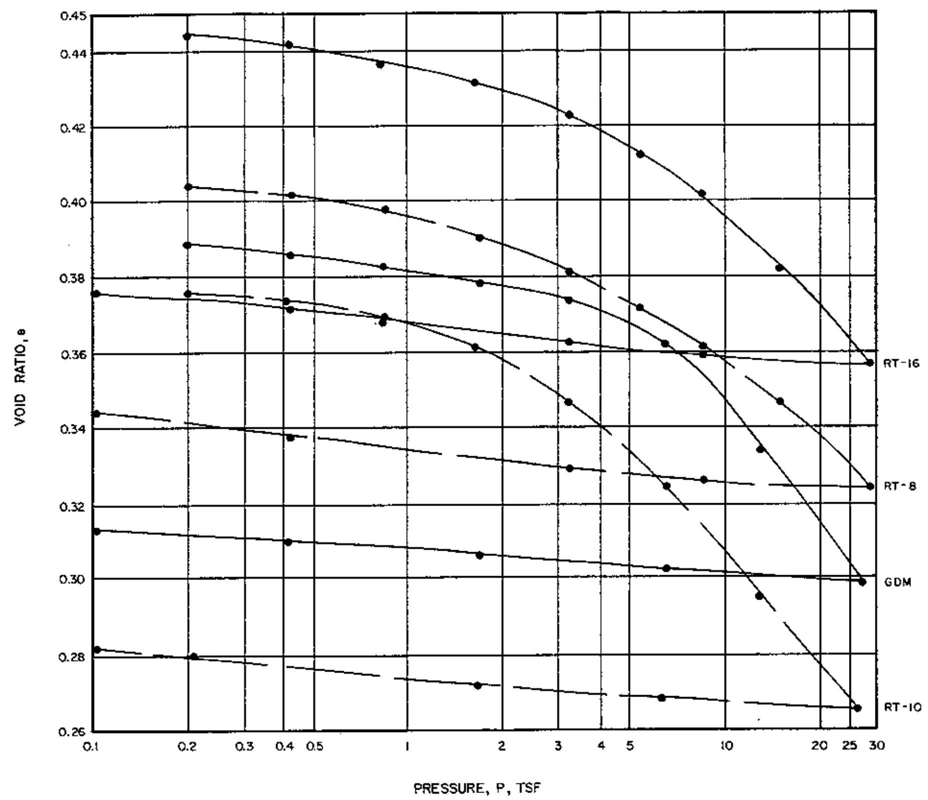
DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

GILA RIVER BASIN
PHOENIX ARIZ. & VIC. (INCL. NEW RIV.)

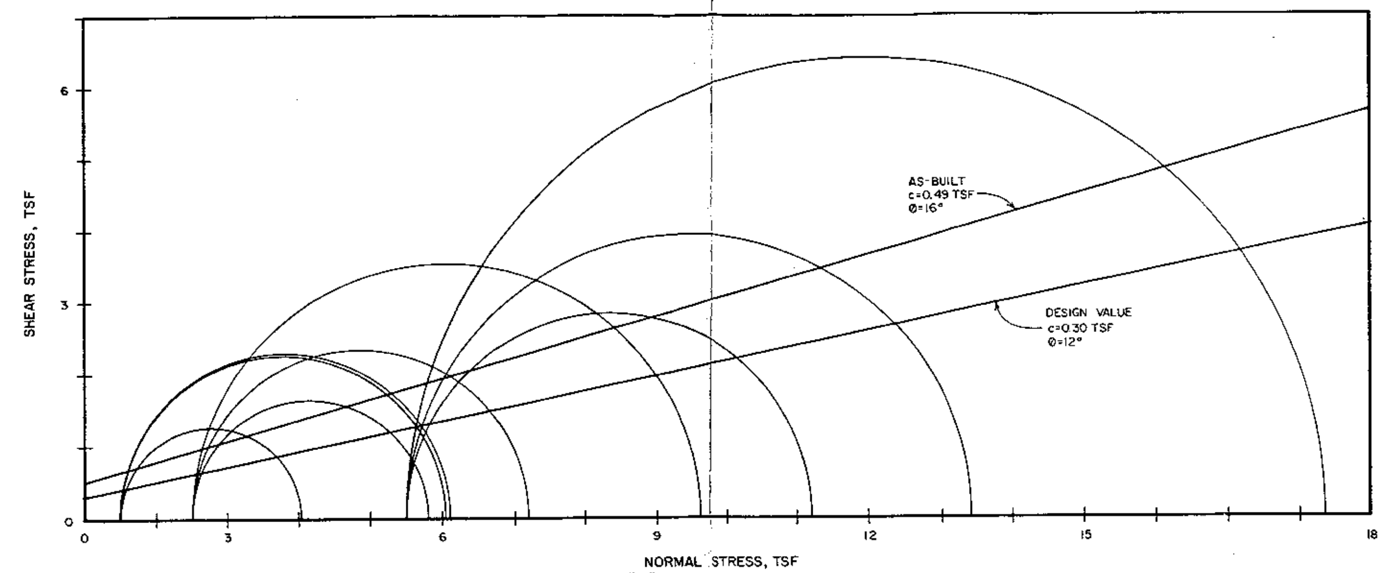
NEW RIVER DAM

PLAN AND PROFILE OF
FIELD CONTROL TEST LOCATIONS

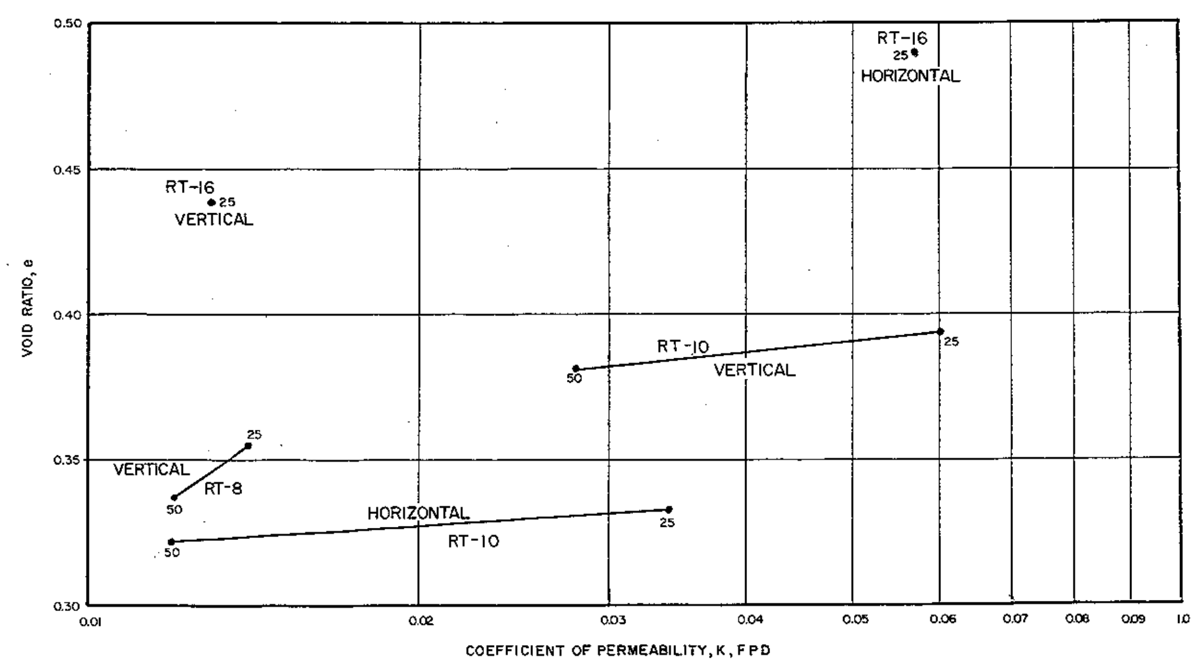
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



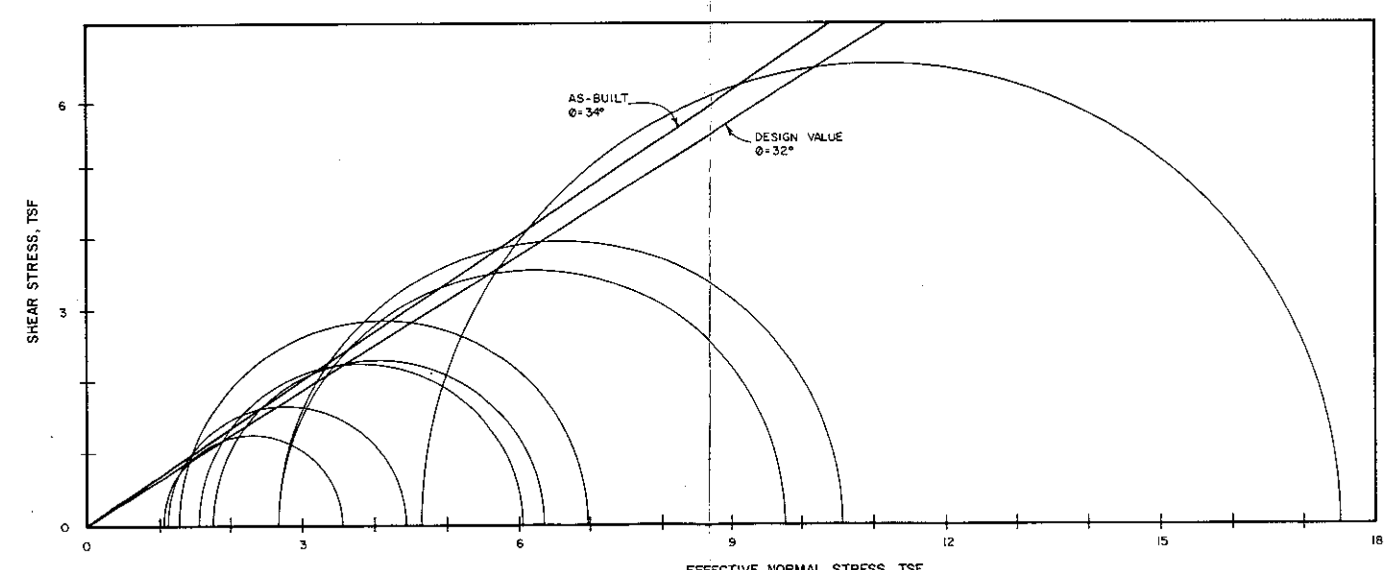
CONSOLIDATION



"R" SHEAR STRENGTH



PERMEABILITY



"S" SHEAR STRENGTH

LEGEND

RT-16	RECORD TEST AND NUMBER.
GDM	PHASE II GENERAL DESIGN MEMORANDUM.
25	CONFINING PRESSURE FOR PERMEABILTY TEST.

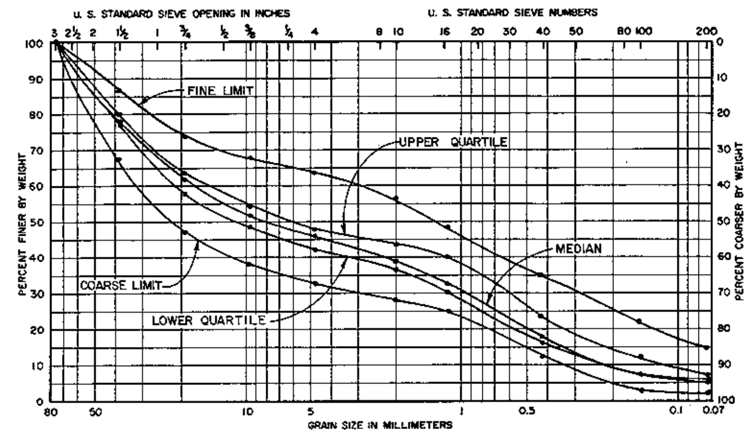
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

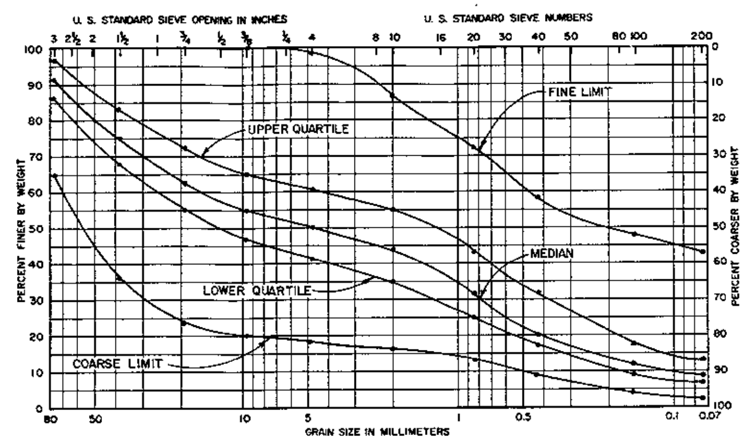
EMBANKMENT CRITERIA AND
PERFORMANCE REPORT

CORE MATERIAL
RECORD TEST RESULTS

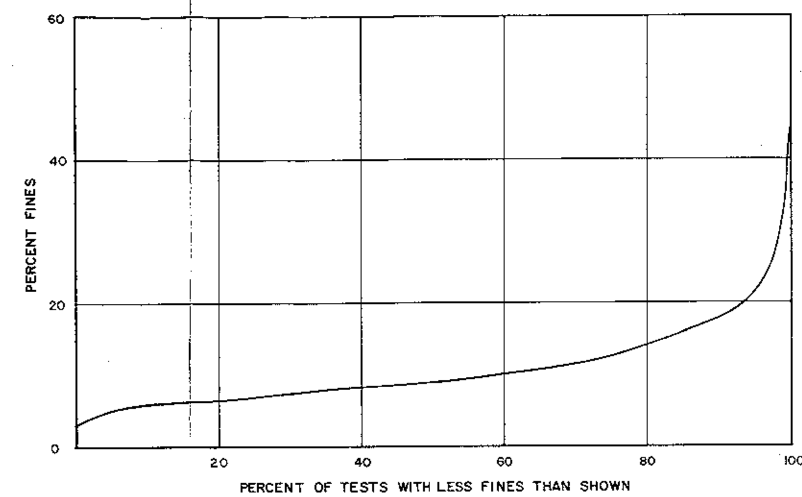
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



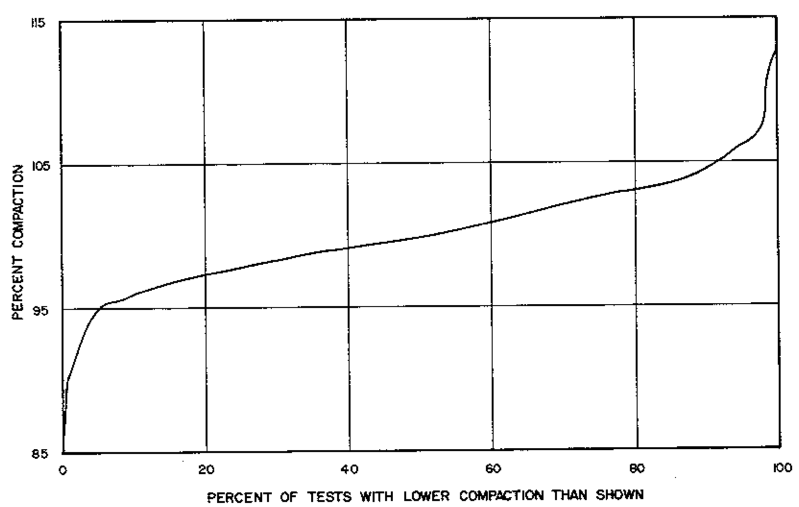
GRADATION CURVES
GDM



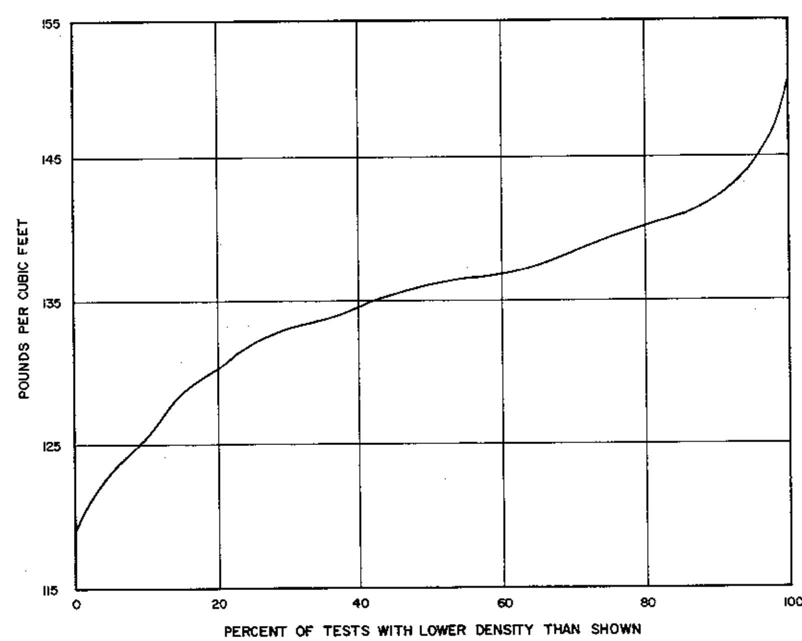
GRADATION CURVES
AS-PLACED



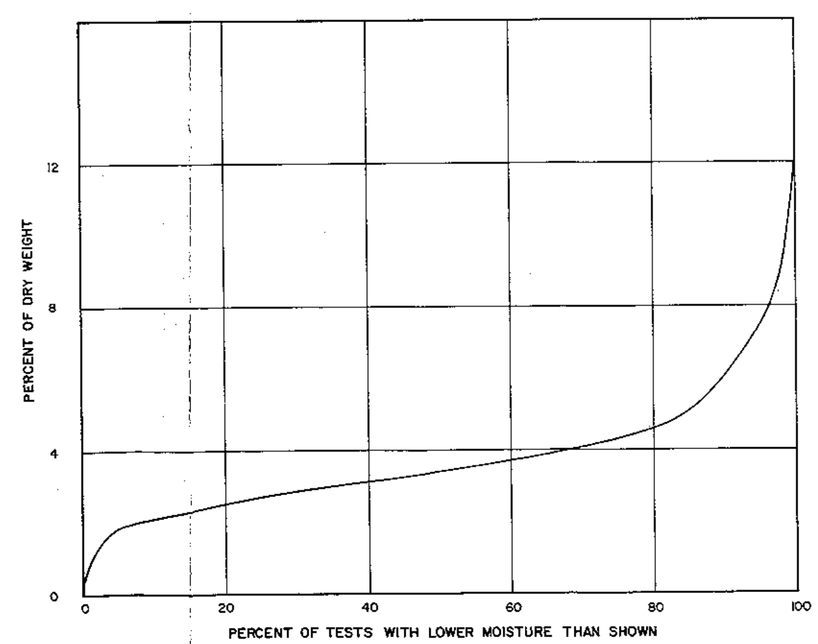
FINES



COMPACTION



DRY DENSITY



MOISTURE CONTENT

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

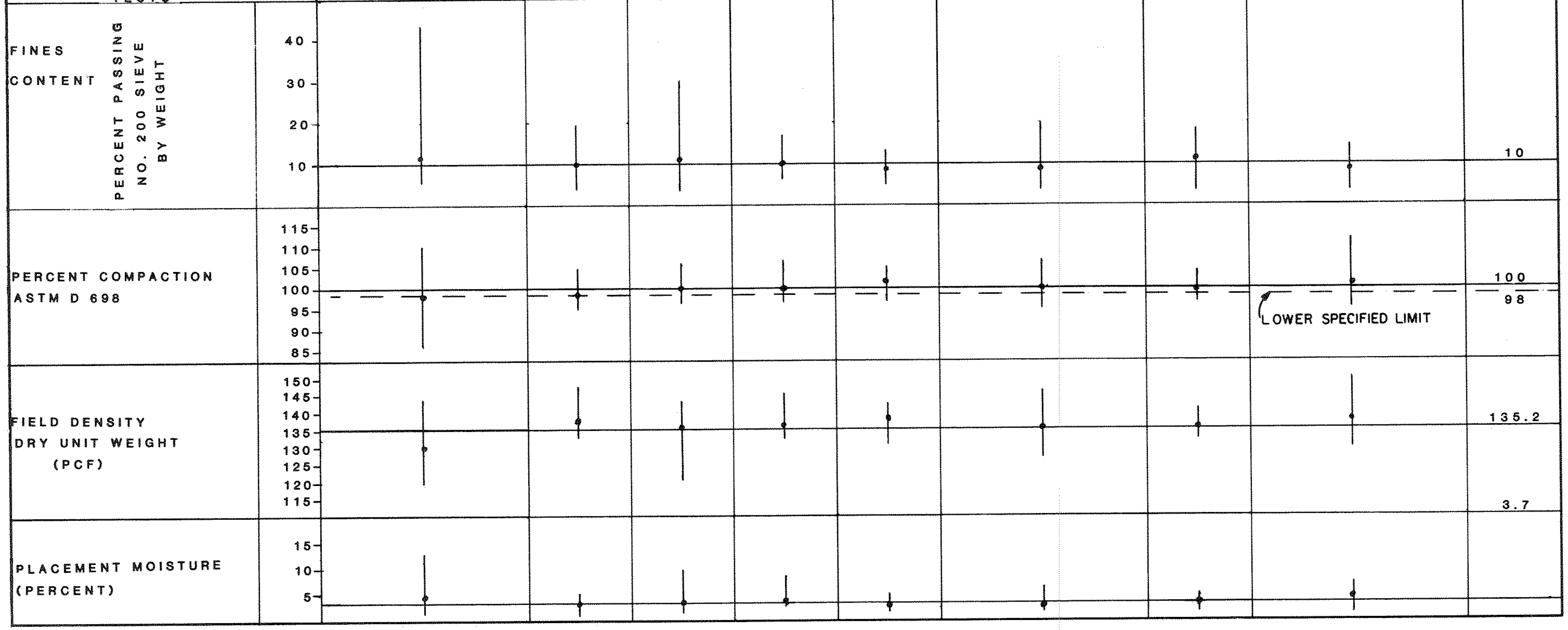
TRANSITION MATERIAL FIELD CONTROL DATA

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

YEAR 1984	MONTH	JANUARY-FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY-AUGUST	OCTOBER	NOVEMBER-DECEMBER	SUMMARY

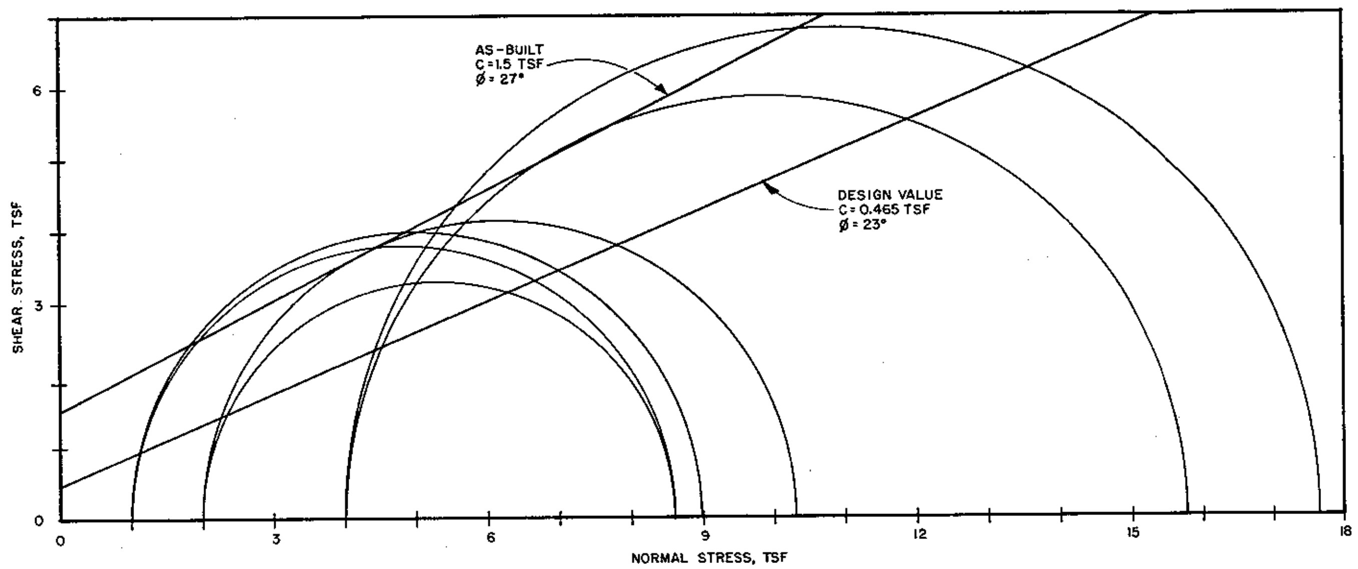
CONSTRUCTION CONTROL DATA

TESTS		32	24	16	16	7	10	8	20	133

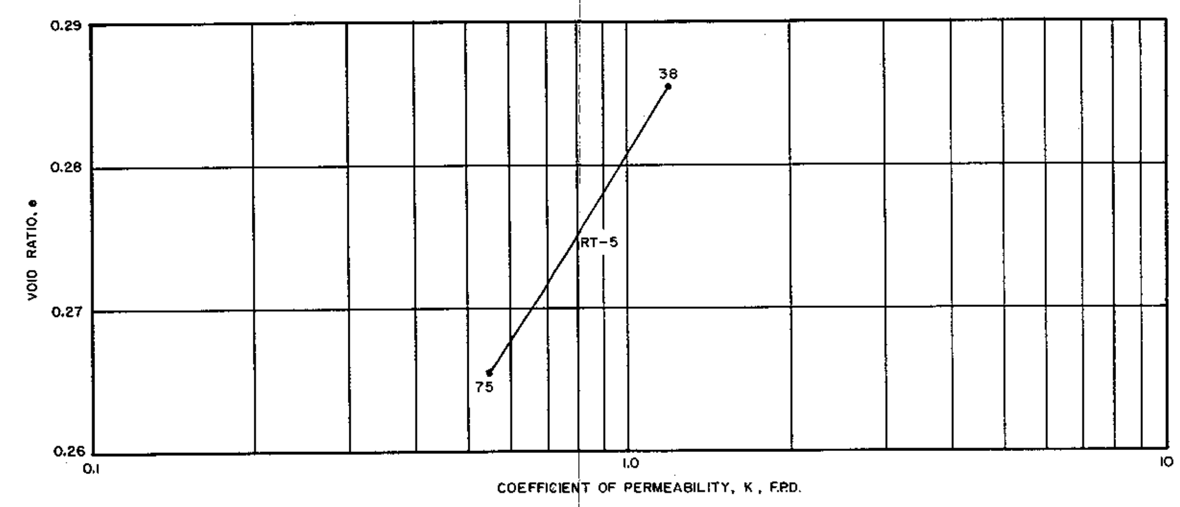


NOTE:
SEE PLATE 34 FOR LEGEND.

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY
(INCLUDING NEW RIVER)
NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT
TRANSITION MATERIAL
MONTHLY QUALITY ASSURANCE TESTING
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



"R̄" SHEAR STRENGTH

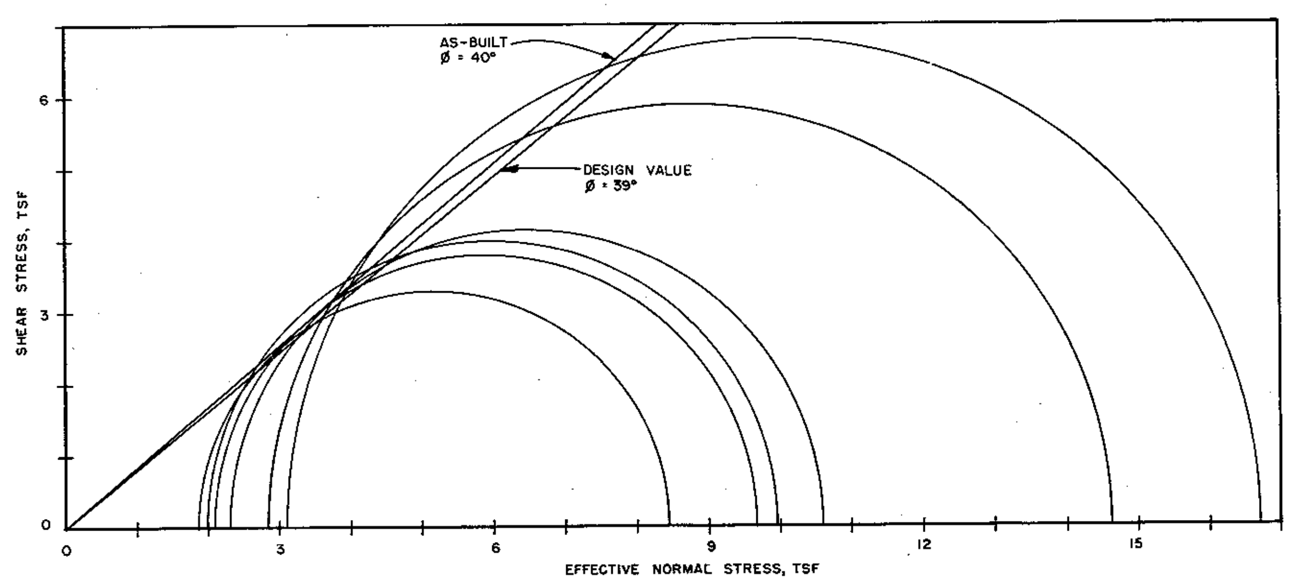


PERMEABILITY

LEGEND

RT-5 RECORD TEST AND NUMBER

38 CONFINING PRESSURE, PSI, FOR PERMEABILITY TEST



"S" SHEAR STRENGHT

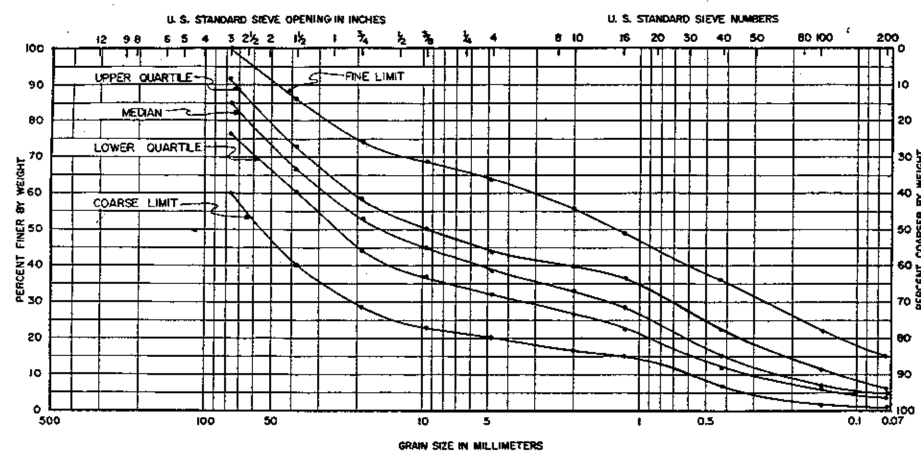
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

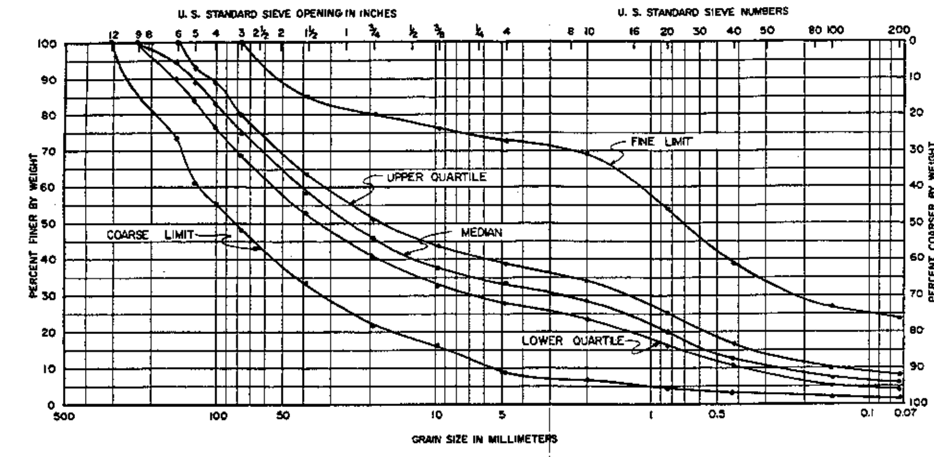
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

TRANSITION MATERIAL RECORD TEST RESULTS

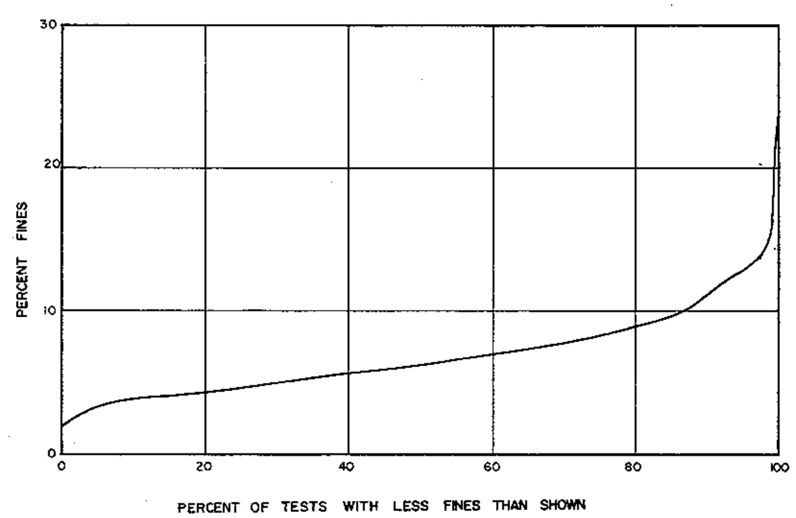
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



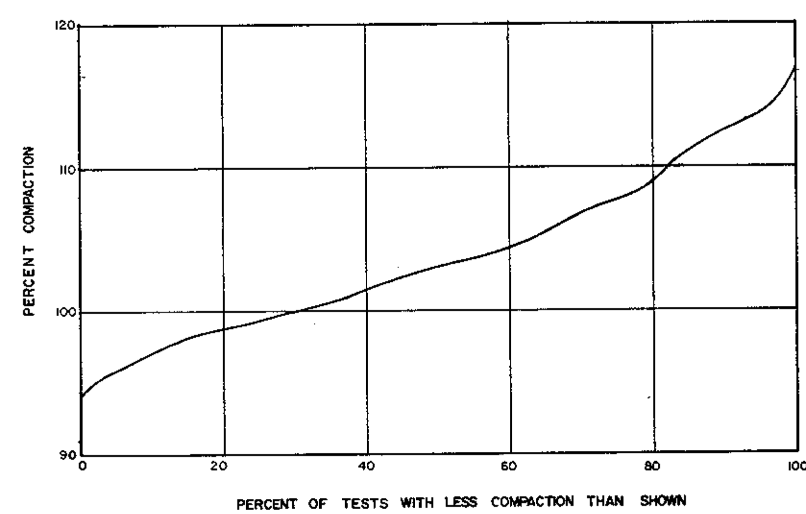
GRADATION CURVES
GDM



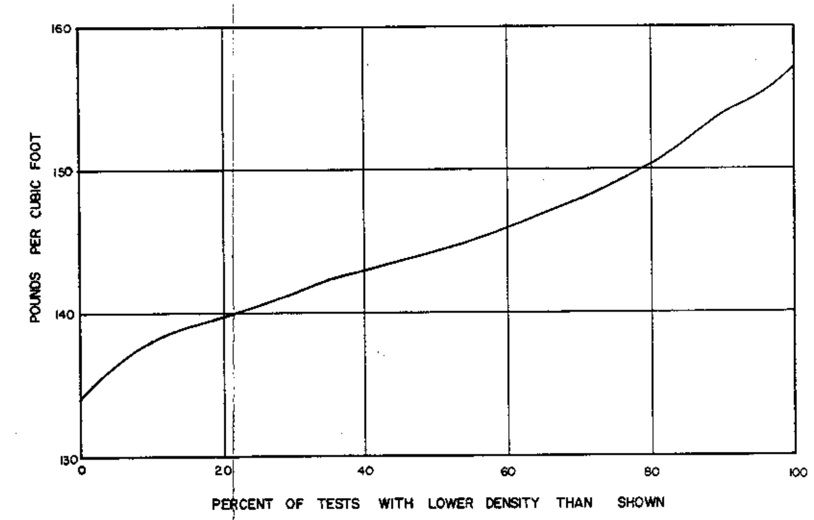
GRADATION CURVES
AS-PLACED



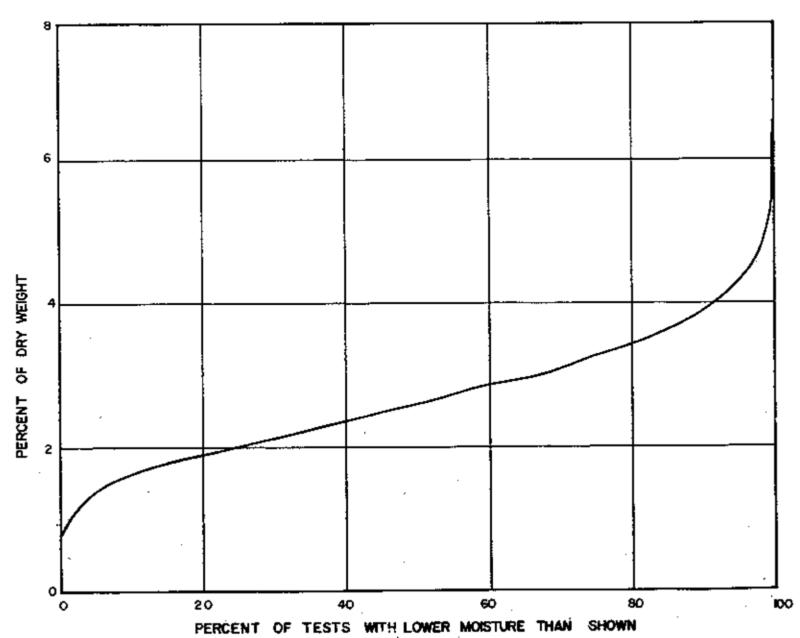
FINES



COMPACTION



DRY DENSITY



MOISTURE CONTENT

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

EMBARKMENT CRITERIA AND
PERFORMANCE REPORT

PERVIOUS SHELL MATERIAL
FIELD CONTROL DATA

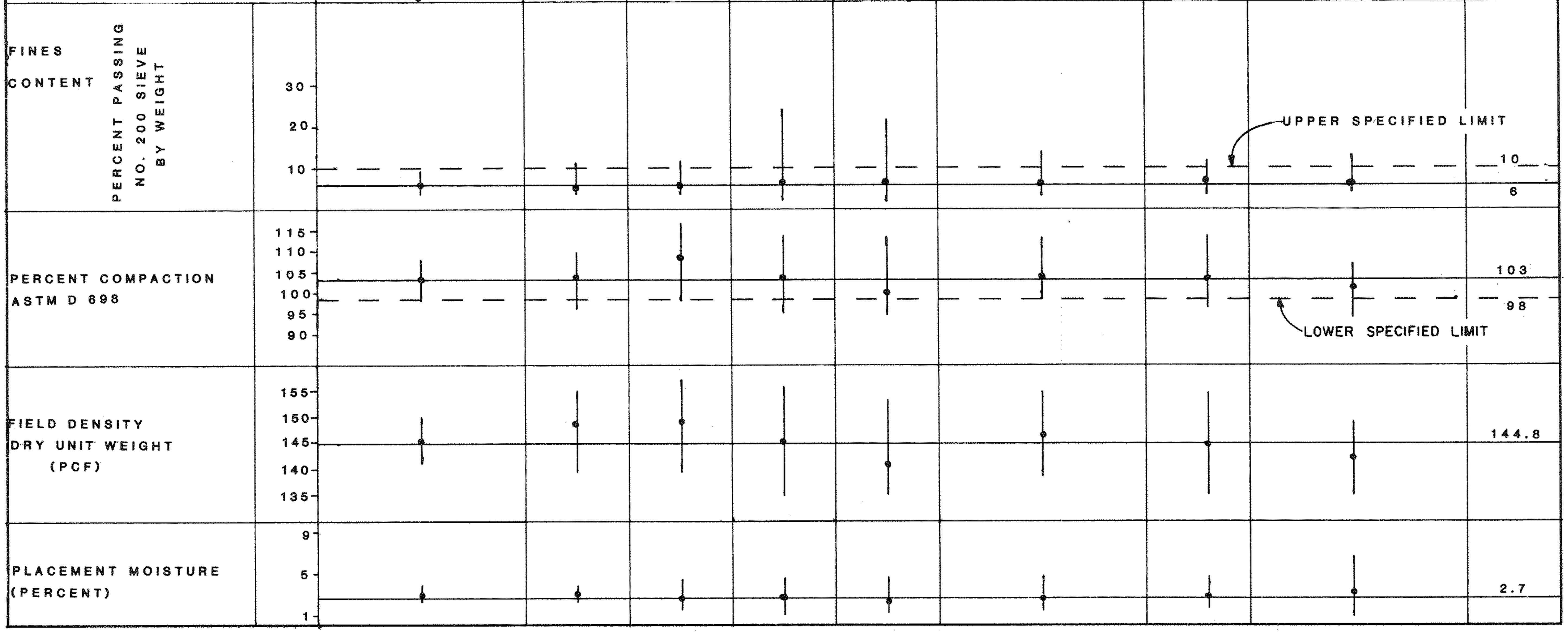
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

YEAR 1984	MONTH	JANUARY-FEBRUARY	MARCH	APRIL	MAY	JUNE	JULY-AUGUST	OCTOBER	NOVEMBER-DECEMBER	SUMMARY

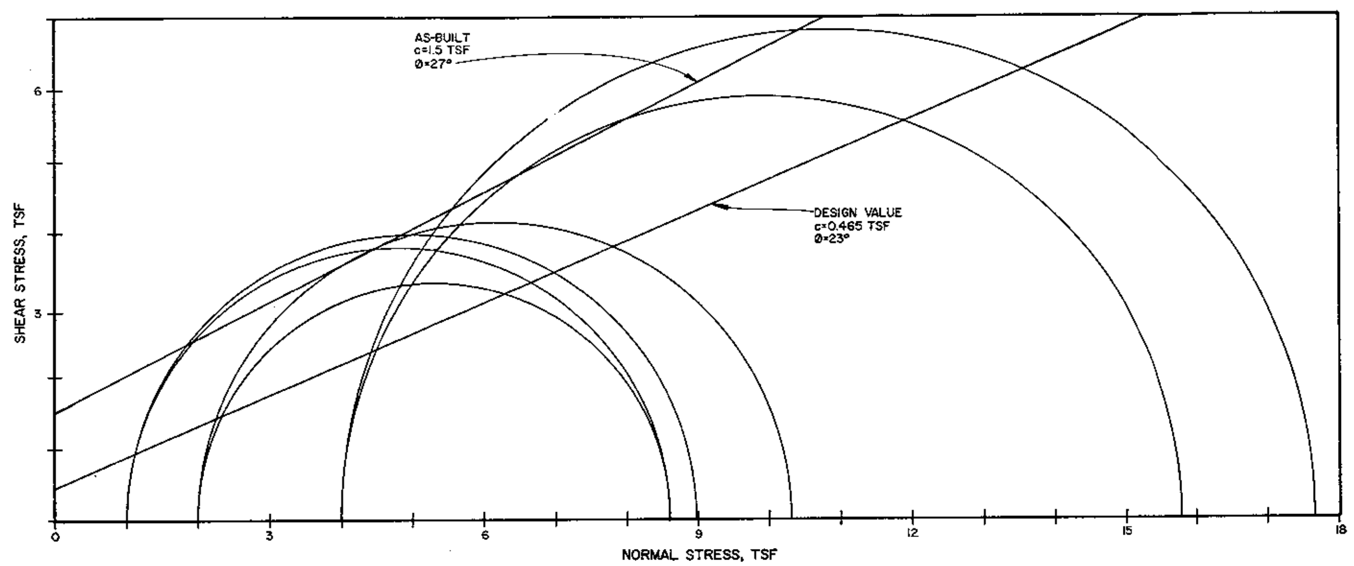
NOTE:
SEE PLATE 34 FOR LEGEND.

CONSTRUCTION CONTROL DATA

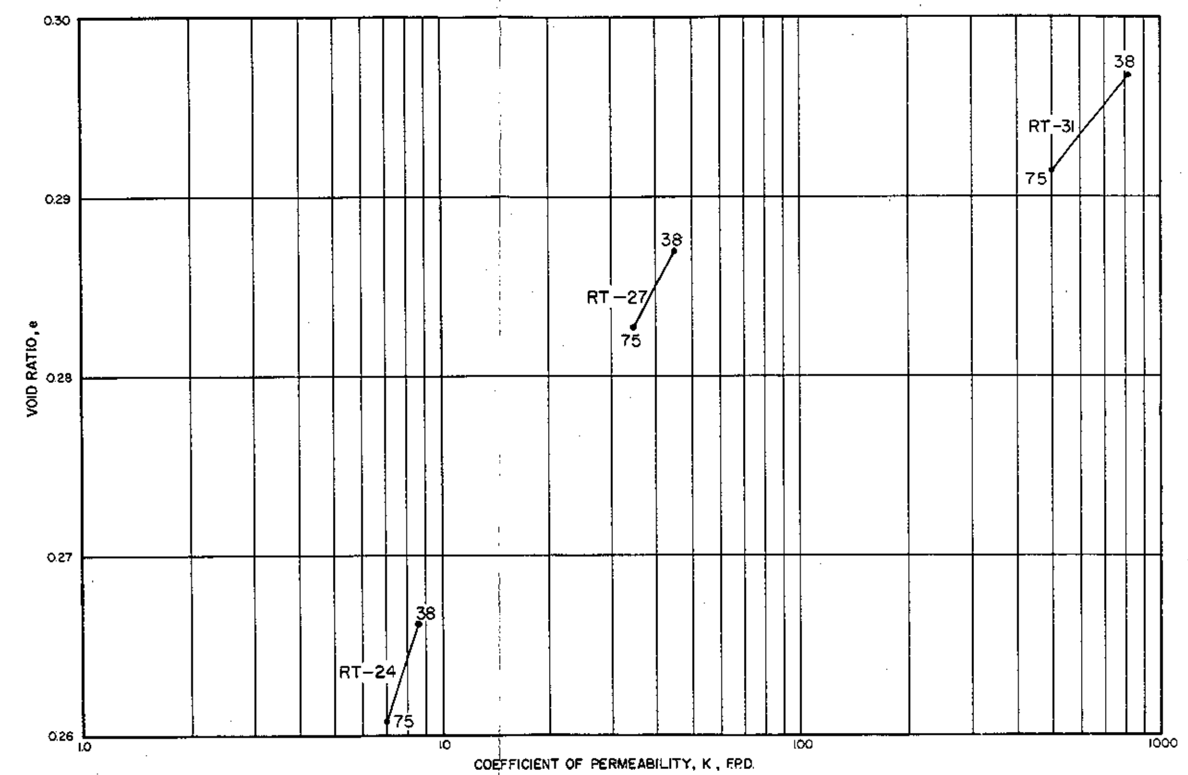
TESTS	9	5	31	54	32	15	22	18	186



GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY
(INCLUDING NEW RIVER)
NEW RIVER DAM
EMBANKMENT CRITERIA AND PERFORMANCE REPORT
PERVIOUS SHELL MATERIAL
MONTHLY QUALITY ASSURANCE TESTING
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



"R" SHEAR STRENGTH

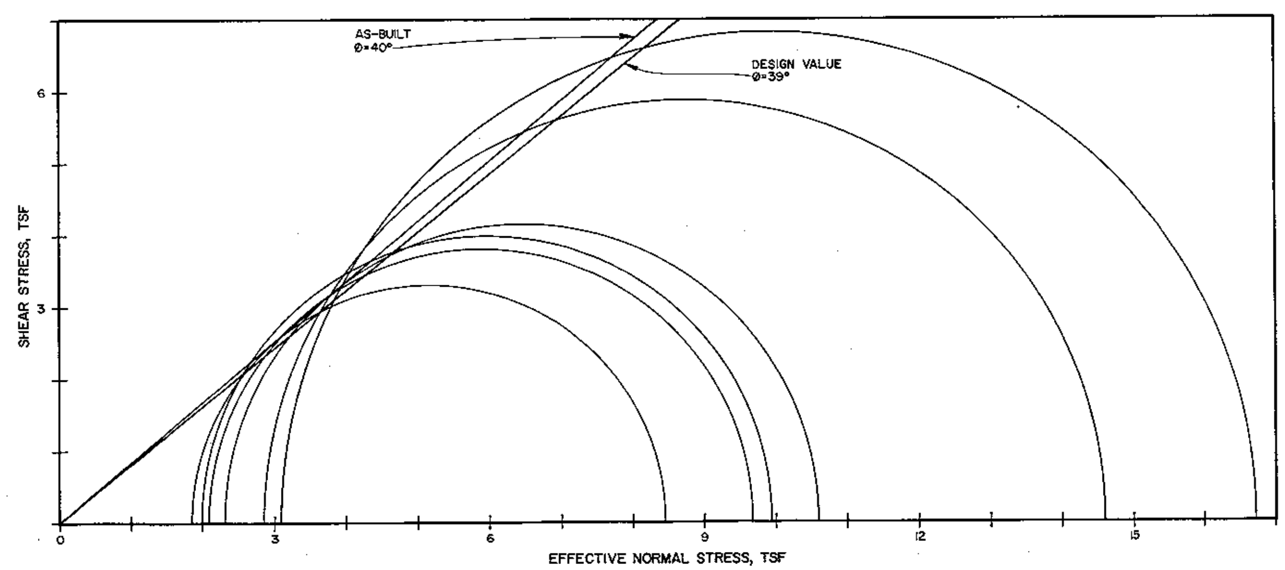


PERMEABILITY

LEGEND

RT-31 RECORD TEST AND NUMBER

38 CONFINING PRESSURE, PSI, FOR PERMEABILITY TEST



"S" SHEAR STRENGTH

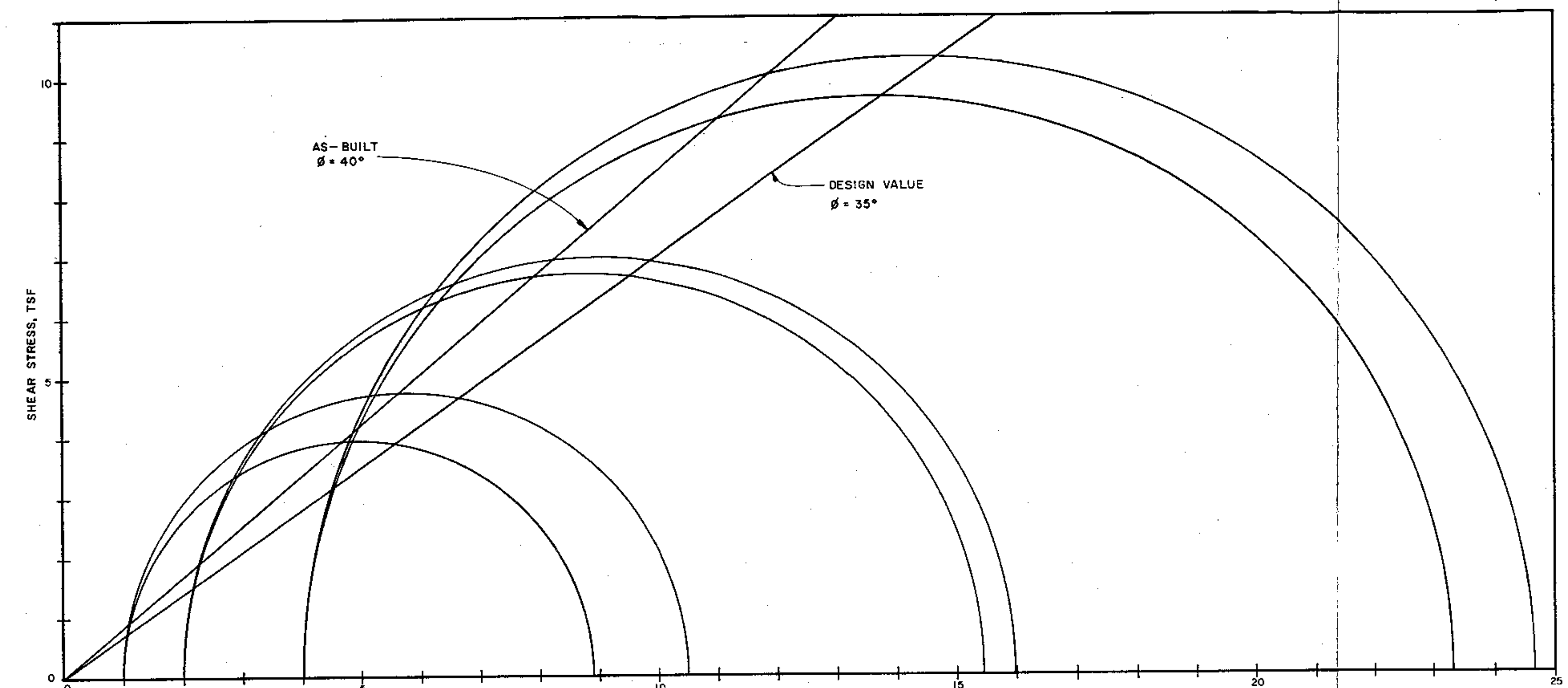
GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

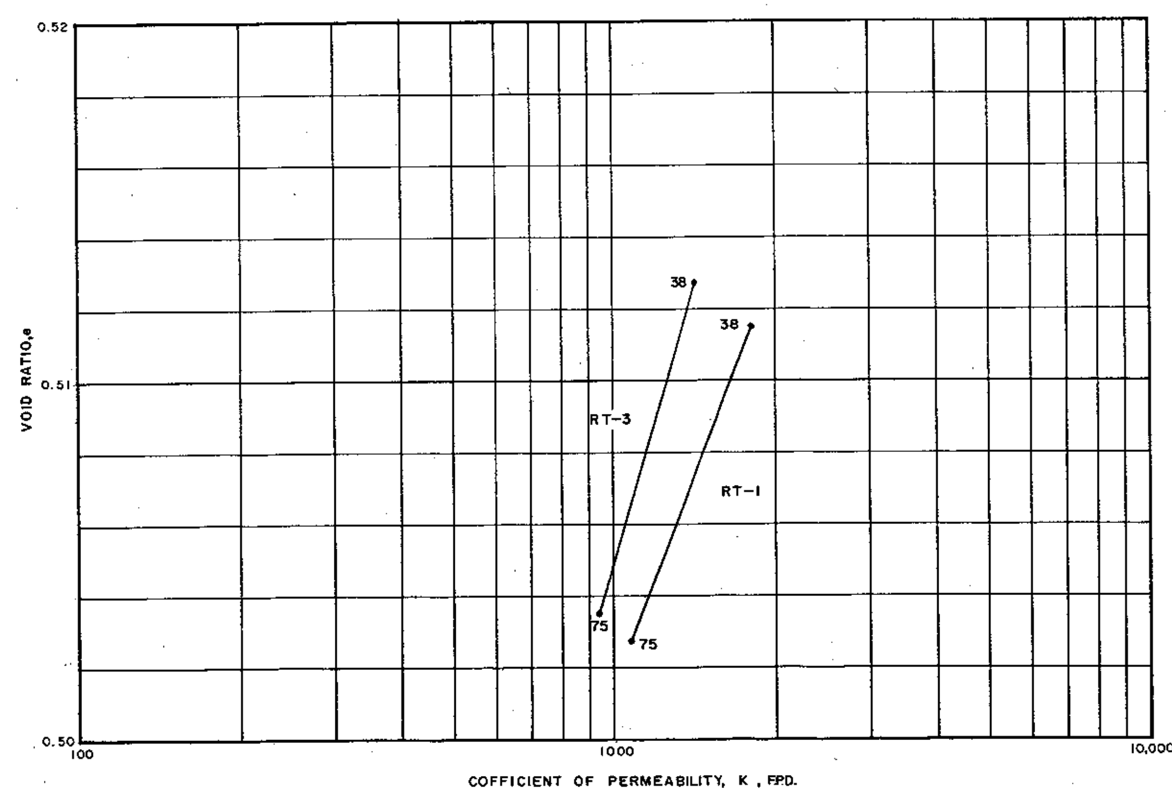
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PERVIOUS SHELL MATERIAL RECORD TEST RESULTS

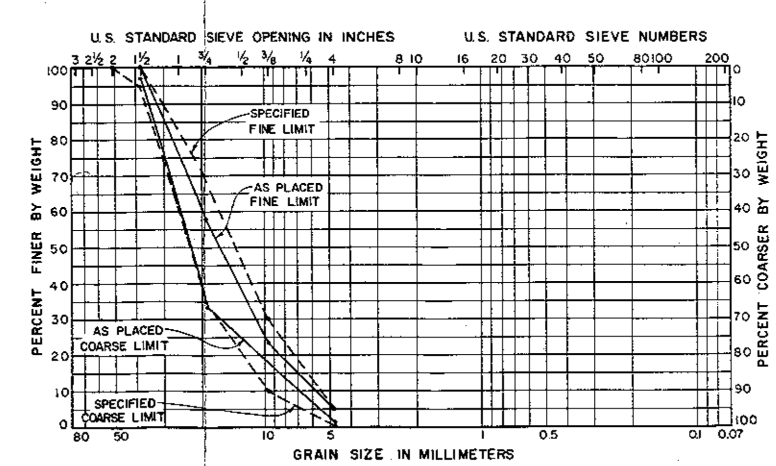
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



"S" SHEAR STRENGTH



PERMEABILITY



GRADATION CURVES

LEGEND

RT-3 RECORD TEST AND NUMBER
38 CONFINING PRESSURE, PSI, FOR PERMEABILITY TEST.

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

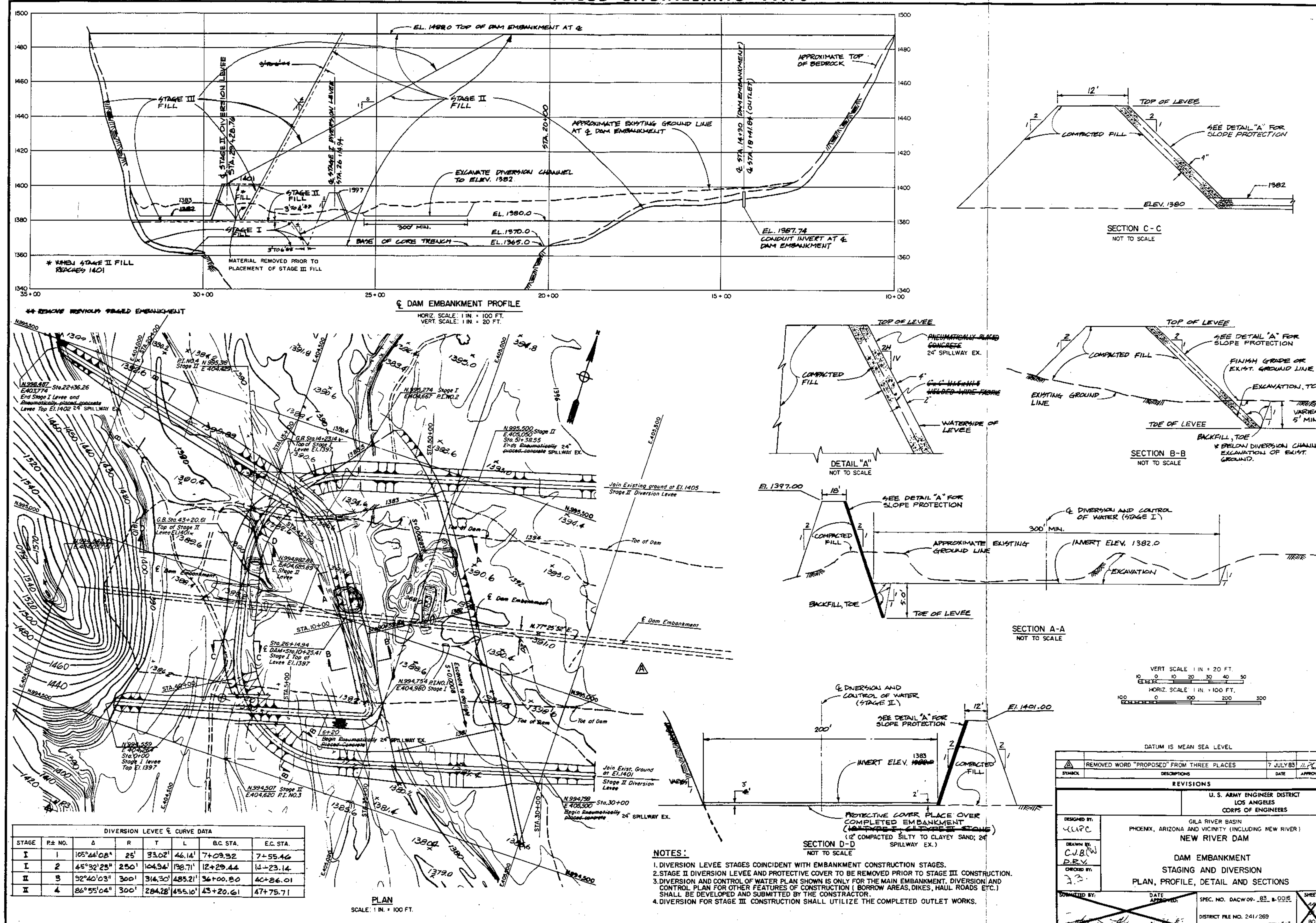
NEW RIVER DAM

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

GRAVEL DRAIN MATERIAL
RECORD TEST RESULTS

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 43



DIVERSION LEVEE ℄ CURVE DATA							
STAGE	P.I. NO.	Δ	R	T	L	B.C. STA.	E.C. STA.
I	1	105°44'08"	25'	33.02'	46.14'	7+09.32	7+55.46
I	2	45°32'25"	250'	104.94'	198.71'	12+29.44	14+23.14
II	3	92°40'03"	300'	314.30'	485.21'	36+00.80	40+86.01
II	4	86°55'04"	300'	284.28'	455.10'	43+20.61	47+75.71

NOTES:

1. DIVERSION LEVEE STAGES COINCIDENT WITH EMBANKMENT CONSTRUCTION STAGES.
2. STAGE II DIVERSION LEVEE AND PROTECTIVE COVER TO BE REMOVED PRIOR TO STAGE III CONSTRUCTION.
3. DIVERSION AND CONTROL OF WATER PLAN SHOWN IS ONLY FOR THE MAIN EMBANKMENT. DIVERSION AND CONTROL PLAN FOR OTHER FEATURES OF CONSTRUCTION (BORROW AREAS, DIKES, HAUL ROADS ETC.) SHALL BE DEVELOPED AND SUBMITTED BY THE CONSTRUCTOR.
4. DIVERSION FOR STAGE III CONSTRUCTION SHALL UTILIZE THE COMPLETED OUTLET WORKS.

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
1	REMOVED WORD "PROPOSED" FROM THREE PLACES	7 JULY 83	

REVISIONS

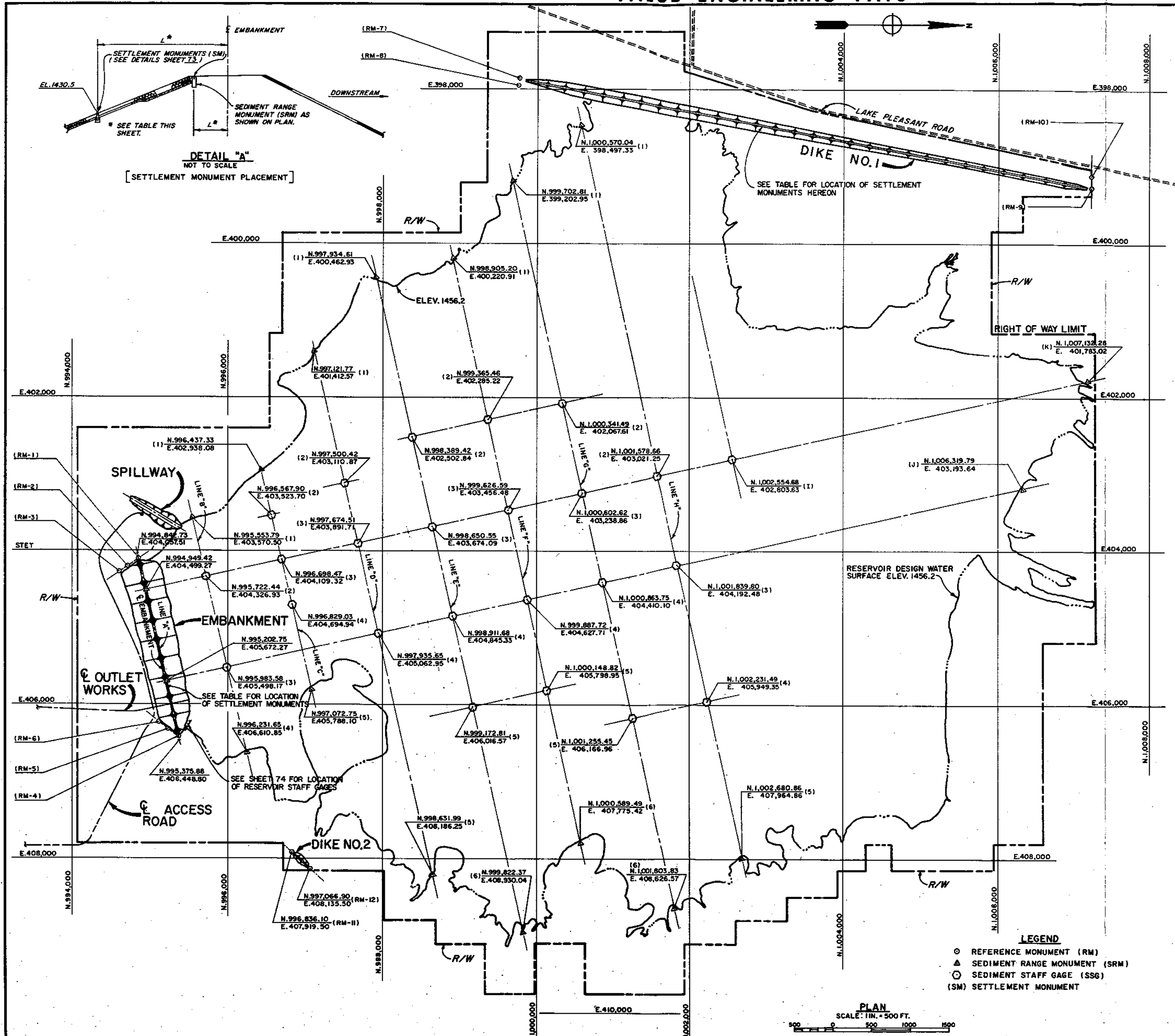
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)
NEW RIVER DAM

DAM EMBANKMENT
STAGING AND DIVERSION
PLAN, PROFILE, DETAIL AND SECTIONS

DESIGNED BY: | DRAWN BY: C.J.B. | CHECKED BY: | SUBMITTED BY: | DATE APPROVED:

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/269
REV "A"
SHEET 80



LOCATION OF SETTLEMENT MONUMENTS

	MONUMENT IDENTIFICATION NO.	EMBANKMENT STATION	*L (ft.)
DAM EMBANKMENT	SM-1	10+50	8.0
	SM-2	11+00	8.0
	SM-3	11+50	8.0
	SM-4	12+00	8.0
	SM-5	12+50	8.0
	SM-6	13+00	8.0
	SM-7	16+00	8.0
	SM-8	19+50	150.0
	SM-9	20+00	8.0
	SM-10	21+00	8.0
	SM-11	21+50	150.0
	SM-12	22+50	8.0
	SM-13	23+00	8.0
	SM-14	23+50	150.0
	SM-15	24+00	8.0
	SM-16	25+00	8.0
	SM-17	25+50	150.0
	SM-18	26+00	8.0
	SM-19	27+00	8.0
	SM-20	27+50	150.0
	SM-21	28+00	8.0
	SM-22	29+00	8.0
	SM-23	29+50	150.0
	SM-24	30+00	8.0
	SM-25	30+50	150.0
	SM-26	31+00	8.0
	SM-27	32+00	8.0
	SM-28	32+50	8.0
	SM-29	33+00	8.0
DIKE NO. 1	SM-30	83+00	8.0
	SM-31	81+00	8.0
	SM-32	79+00	8.0
	SM-33	77+00	8.0
	SM-34	75+00	8.0
	SM-35	73+00	8.0
	SM-36	71+00	8.0
	SM-37	69+00	8.0
	SM-38	65+00	8.0
	SM-39	61+00	8.0
	SM-40	57+00	8.0
	SM-41	53+00	8.0
	SM-42	49+00	8.0
	SM-43	45+00	8.0
	SM-44	41+00	8.0
	SM-45	37+00	8.0
	SM-46	33+00	8.0
	SM-47	29+00	8.0
	SM-48	25+00	8.0
	SM-49	20+00	8.0
	SM-50	15+00	8.0

(*SEE DETAIL "A" HEREON)

DATUM IS MEAN SEA LEVEL

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

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

DESIGNED BY: RK
DRAWN BY: DRV
CHECKED BY: 4UPC
SUBMITTED BY:
DATE APPROVED: 6/10/83

GILA RIVER BASIN
PHOENIX, ARIZONA AND VICINITY (INCLUDING NEW RIVER)

NEW RIVER DAM

MONUMENTS AND SEDIMENT STAFF GAGE LOCATIONS

SPEC. NO. DACW09-83-B-0016
DISTRICT FILE NO. 241/297
SHEET 77 OF 80

PHOTOGRAPHS

PHOTOS

1. Upstream face of dam embankment as viewed from the spillway. March 1985.
2. View of dam embankment crest, upstream face, and spillway from the east abutment. March 1985.
3. Landscaped downstream face of dam embankment as viewed from spillway access road. March 1985.
4. Crest and upstream face of dike no. 1 embankment as viewed from south abutment. Note grouted stone gutter along embankment toe. March 1985.
5. View of landscaped downstream face of dike no. 1 embankment looking toward south abutment. March 1985.
6. Downstream face of dike no. 2 embankment. March 1985.
7. Looking downstream at main access road, energy dissipator, gaging station bridge, and outlet channel from crest of dam. April 1985.
8. Looking upstream at energy dissipator and downstream face of dam embankment from gaging station bridge. March 1985.
9. Approach channel to intake structure as viewed from crest of dam. Note ditch to intercept slope runoff at right. April 1985.
10. Intake structure and trash rack. Note grouted stone section at approach channel. April 1985.
11. View of completed spillway excavation looking upstream. March 1985.
12. Core trench excavation using push cat and scraper. Note dozer using rippers and blade to excavate base of right abutment. December 1983.
13. Foundation excavation using two push cats and scraper. April 1984.
14. Foundation excavation using push cats, scrapers, and dozer. Note lighter colored areas of claiche or bedrock in upstream portion of dam foundation. January 1984.
15. Core trench excavation using push cats and scrapers. Note front end loader pushing overburden away from the bedrock near station 16+50 for scraper removal. March 1984.
16. Excavating andesite bedrock in stage I core trench using excavator, vicinity station 31+10. December 1983.
17. Excavator widening stage I core trench. January 1984.
18. 50-ton roller used to proof roll the foundation. April 1984.
19. West abutment "stripping using a small dozer. November 1983.
20. Surface preparation of west abutment using both air blasting and hand labor. December 1983.
21. View of completed stage I core trench excavation and west abutment. Note backhoe trench in foreground. February 1984.
22. East abutment excavation using excavator. January 1984.
23. Initial foundation preparation of east abutment surface using low pressure air blasting. Note excavator on left. January 1984.
24. Small backhoe assisting excavator during east abutment excavation. Note rock dust generated from air blasting of excavated abutment surface. January 1984.
25. Foundation preparation in core trench using backhoes and shovels; vicinity station 17+00. March 1984.
26. View of stage II diversion levee protecting completed stage II excavation to station 26+50. Note stage I embankment protective cover in foreground. March 1984.
27. Placing dental concrete on west abutment slope using concrete bucket. October 1984.

28. Dike no. 1 exploration trench excavation using two push cats and scraper. Note dozer being used to rip caliche. February 1984.
29. View of dike no. 1 exploration trench excavation looking north from south abutment. January 1984.
30. Dike no. 1 exploration trench ecavation using front end loader to cleanup the trench. Note the caliche which is visible throughout the cross section of the trench. January 1984.
31. Dike no. 1 exploration trench excavation attempting to rip caliche between station 31+00 and 30+00. February 1984.
32. Dike no. 1 exploration trench, station 81+25. Note coherent andesite blocks on both sides of photo with calichified andesite breccia toward center. January 1984.
33. View of dike no. 1 completed exploration trench excavation looking south from station 13+00. Note widespread caliche. February 1984.
34. Dike no. 2 foundation after stripping. February 1984.
35. Vibrating crusher used to crush and screen pervious shell materials to produce gravel drain and type III stone. February 1984.
36. Taking core record sample dike no. 1 station 70+10 elevation 1461.0, and 7 feet upstream of centerline. February 1984.
37. Prewetting core material in borrow area III. December 1983.
38. Front end loader loading end dump trucks with core material. April 1984.
39. End dump truck placing core material on dike No. 1. March 1984.
40. Placing core material in core trench. Note: the tamping roller being pulled by the dozer, the motorgrader scarifying the surface, the push cats and scrapers excavating the embankment foundation and placing the suitable materials in the core zone, and the water truck adding water to the core materials. June 1984.
41. Typical example of rock-core contact on east abutment (approximate elevation 1447). Pneumatic wheel rolling core material prevented damage to highly fractured granitic bedrock surface by the tamping roller. July 1984
42. Beginning west abutment surface preparation. Laborers permitted to walk only short reaches without safety lines. Note backhoe removing protective cover down to elevation 1380 near base of slope. September 1984.
43. Front end loader with loaded bucket wheel rolling core material at rock-core contact. February 1984.
44. Front end loader compacting core material against bedrock slope in core trench, vicinity station 16+50. Note dozer using blade to spread core material over foundation surface. April 1984.
45. Five-foot high ramp of core material against right abutment slope. October 1984.
46. Bottam dump truck placing transition material, vibratory roller compacting transition naterial and motor grader scarifying pervious shell material during embankment construction. November 1984.
47. Processing transition material on grade with motor grader during construction of dike no. 1. March 1984.
48. Spreading previous shell material over the gravel drain during embankment construction. April 1984.
49. Dozer ripping high spot at station 19+70 in outlet conduit trench excavation. January 1984.
50. Outline of concrete plug (fillet) area on east side of outlet conduit. Note most of conduit projects above adjacent foundation surface due to greater depth of core trench excavation. April 1984.

51. Concrete plug (fillet) placement on east side of conduit using concrete bucket. Note laborer on top box vibrating concrete near contact with conduit wall. April 1984.

52. Concrete plug (fillet) on west side of outlet conduit. Note upper portion of plug is slightly steeper than 1:1 to eleminate "feather" edges against the top of the conduit. April 1984.

53. Completed concrete plug (fillet) on east side of conduit. Concrete spills outside plug limits were subsequently removed. April 1984.

54. Backfilling foundation depression upstream at dam centerline between stations 14+85 and 15+25. February 1984.

55. Stage III dam embankment construction at approximately 1420. Completed stage II embankment with upstream stone protection at left. October 1984.

56. Dressing upstream slope at dike no. 1 with tracked excavator. June 1984.

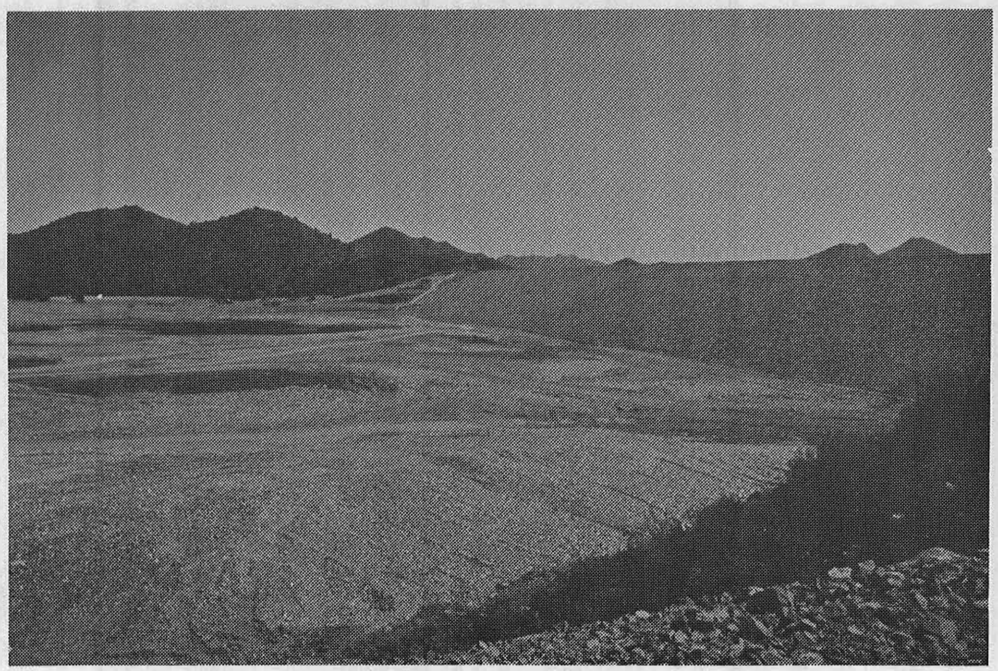


Photo 1. Upstream face of dam embankment as viewed from the spillway. March 1985

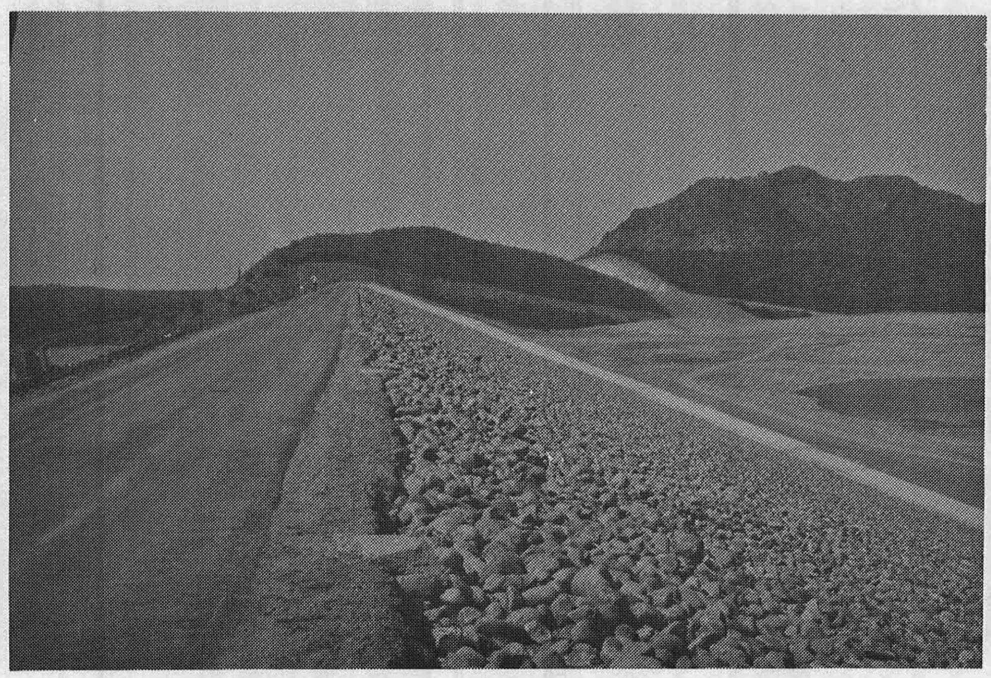


Photo 2. View of dam embankment crest, upstream face, and spillway from the east abutment. March 1985

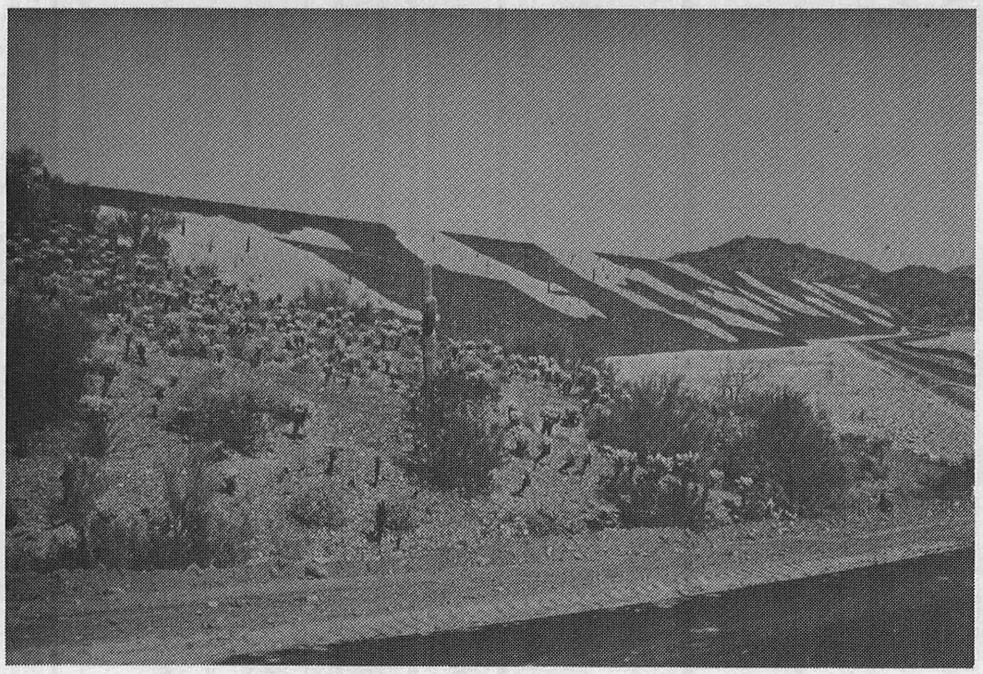


Photo 3. Landscaped downstream face of dam embankment as viewed from spillway access road. March 1985

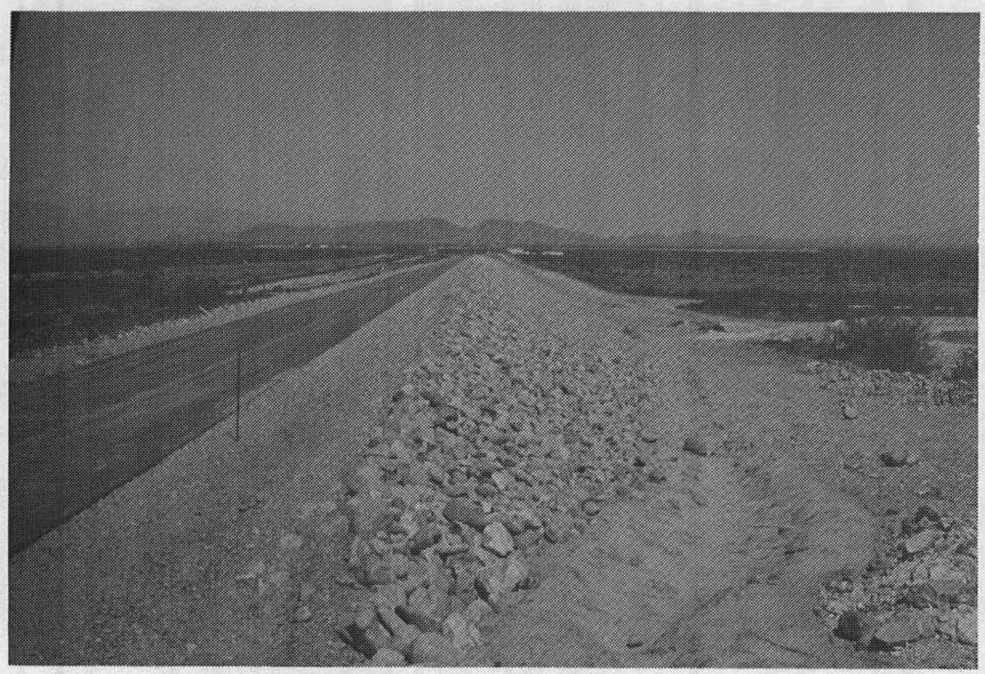


Photo 4. Crest and upstream face of dike no. 1 embankment as viewed from south abutment. Note grouted stone gutter along embankment toe. March 1985

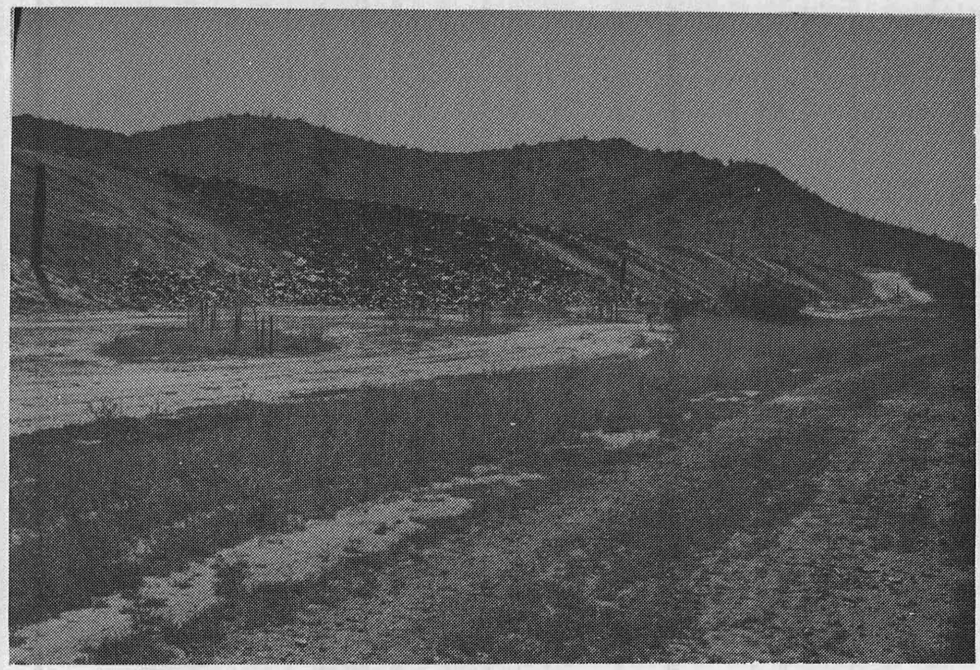


Photo 5. View of landscaped downstream face of dike no. 1 embankment looking toward south abutment. March 1985

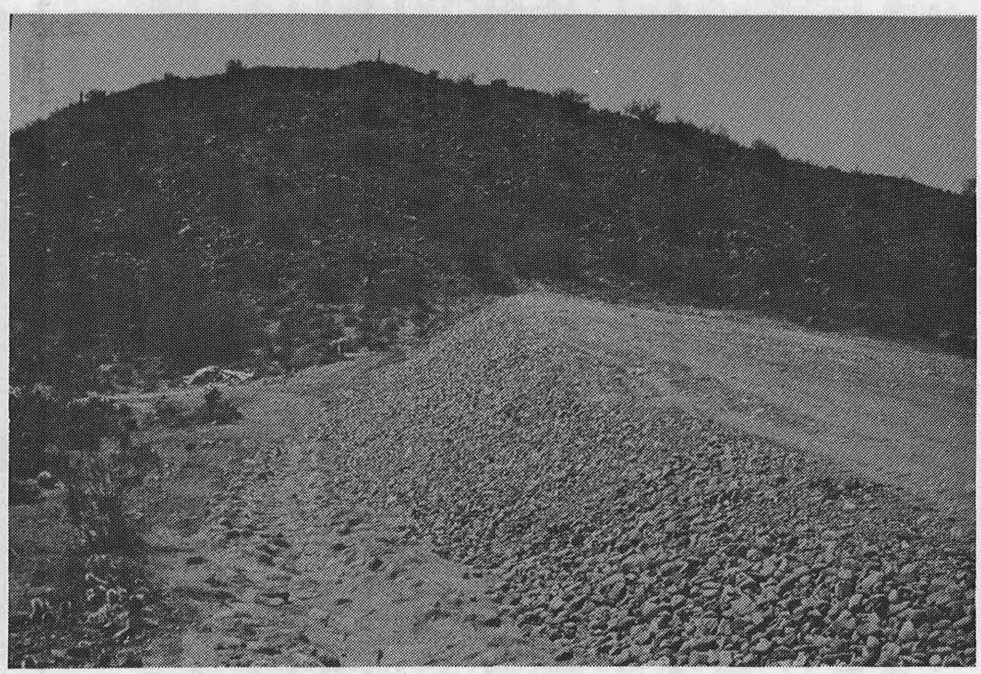


Photo 6. Downstream face of dike no. 2 embankment. March 1985

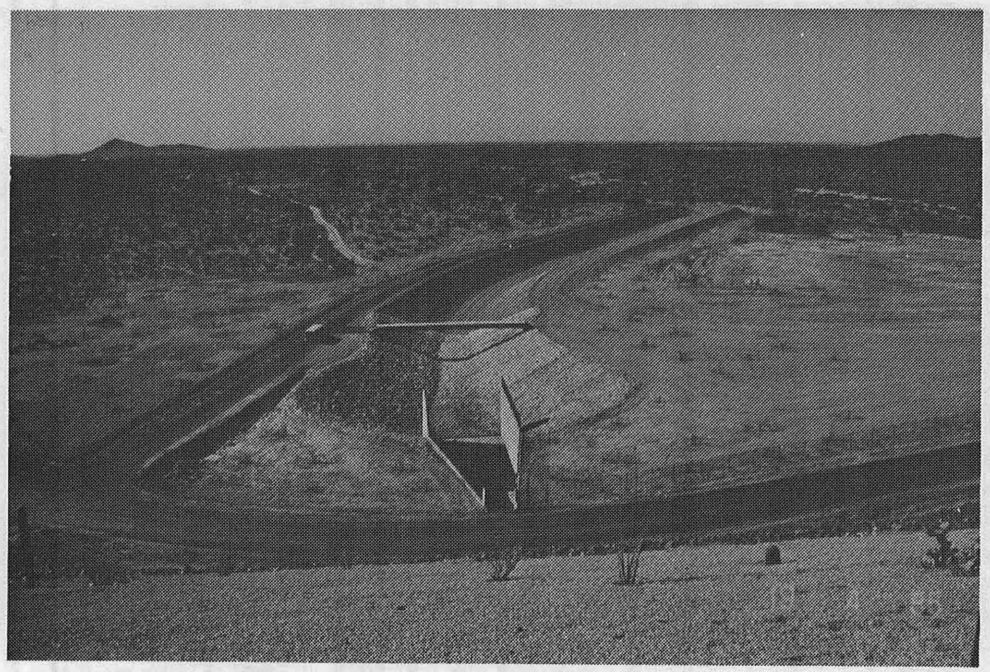


Photo 7. Looking downstream at main access road, energy dissipator, gaging station bridge, and outlet channel from crest of dam. April 1985

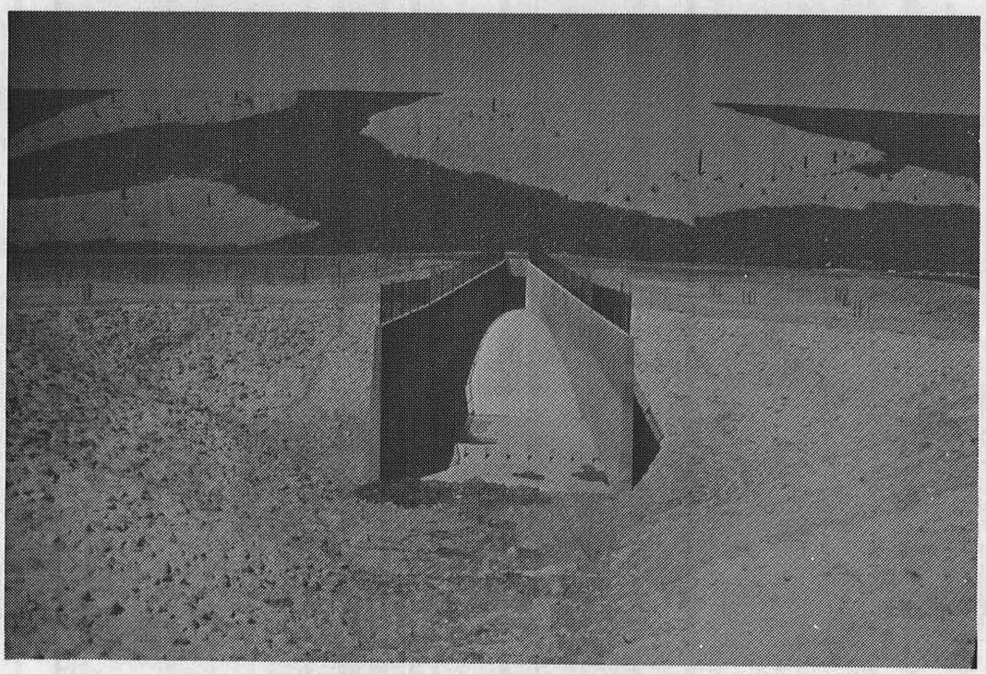


Photo 8. Looking upstream at energy dissipator and downstream face of dam embankment from gaging station bridge. March 1985

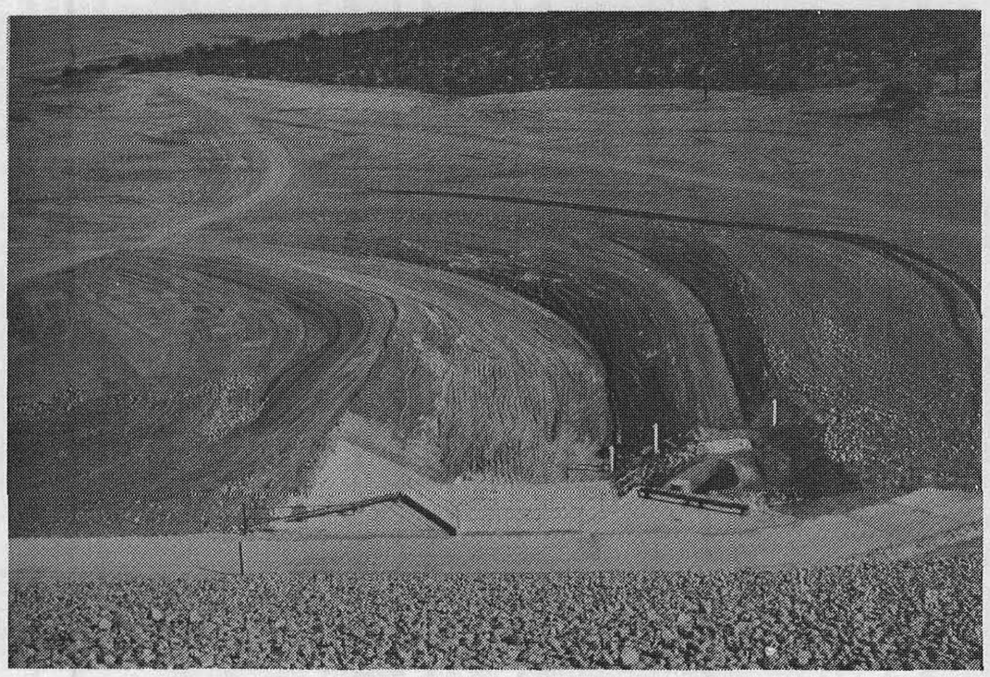


Photo 9. Approach channel to intake structure as viewed from crest of dam. Note ditch to intercept slope runoff at right. April 1985

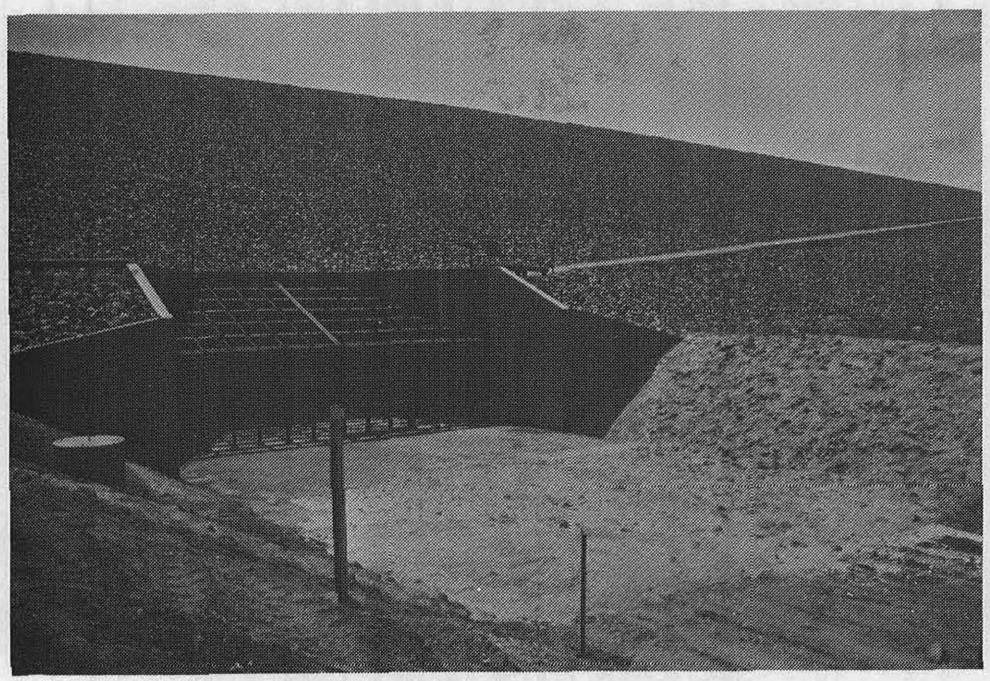


Photo 10. Intake structure and trash rack. Note grouted stone section of approach channel. April 1985

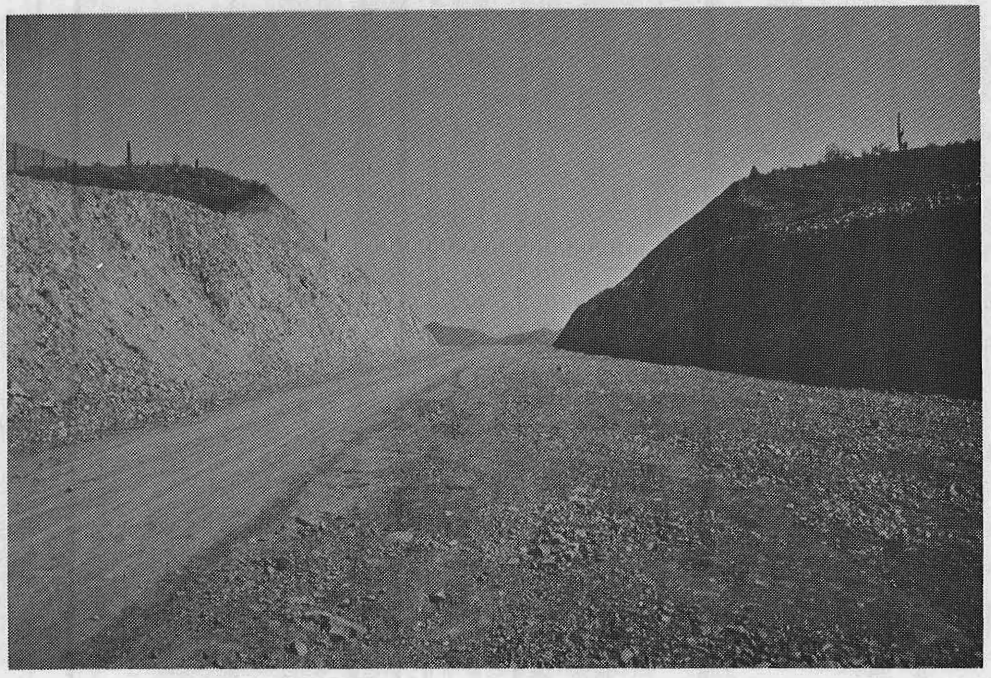


Photo 11. View of completed spillway excavation looking upstream. March 1985

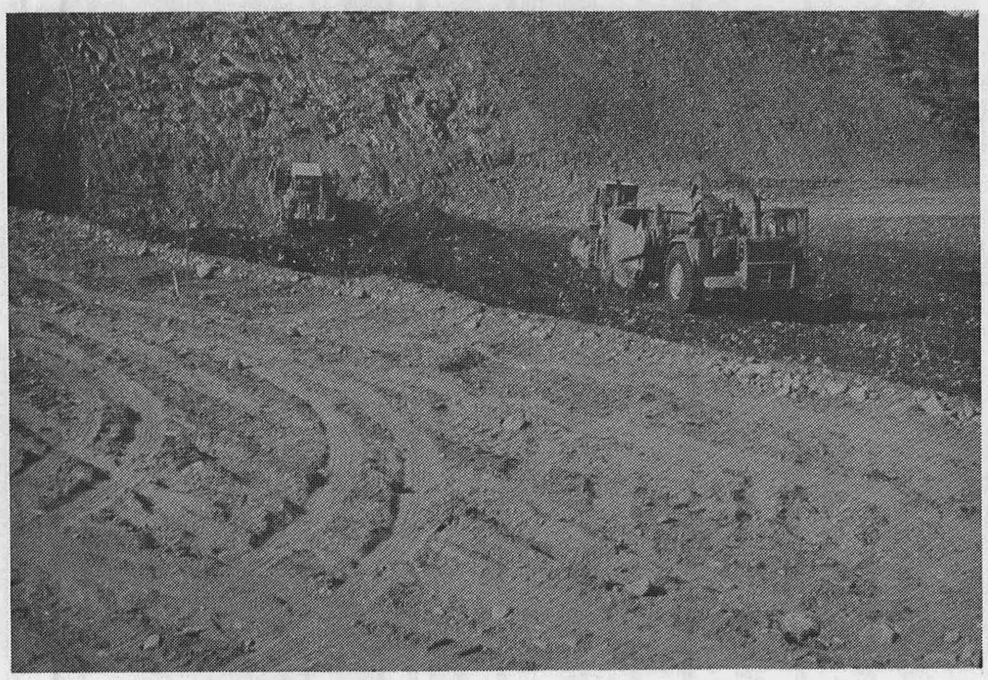


Photo 12. Core trench excavation using push cat and scaper. Note dozer using rippers and blade to excavate base of right abutment. December 1983

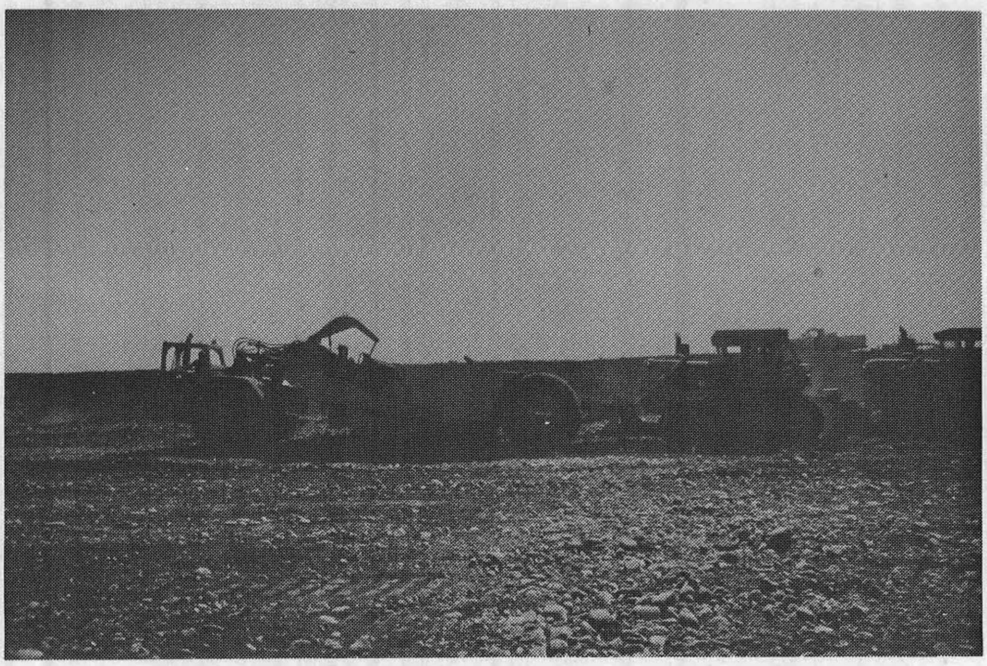


Photo 13. Foundation excavation using two push cats and scraper. April 1984

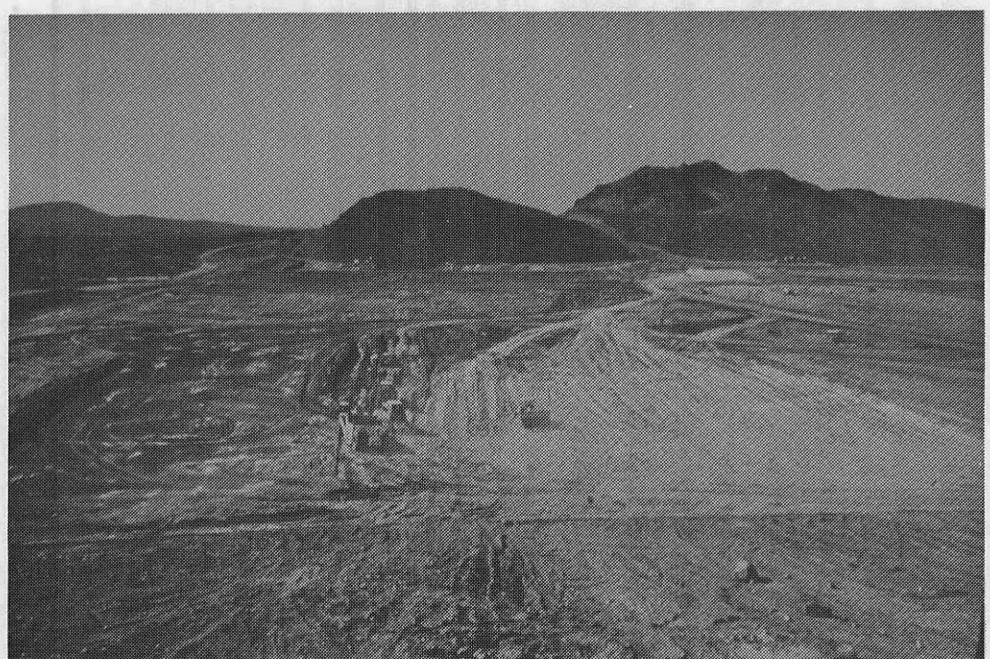


Photo 14. Foundation excavation using push cats, scrapers, and dozer. Note lighter colored areas of caliche, or bedrock in upstream portion of dam foundation. January 1984

Photo 15. Core trench excavation using push cats and scrapers. Note front end loader pushing overburden away from bedrock slope near station 16+50 for scraper removal. March 1984

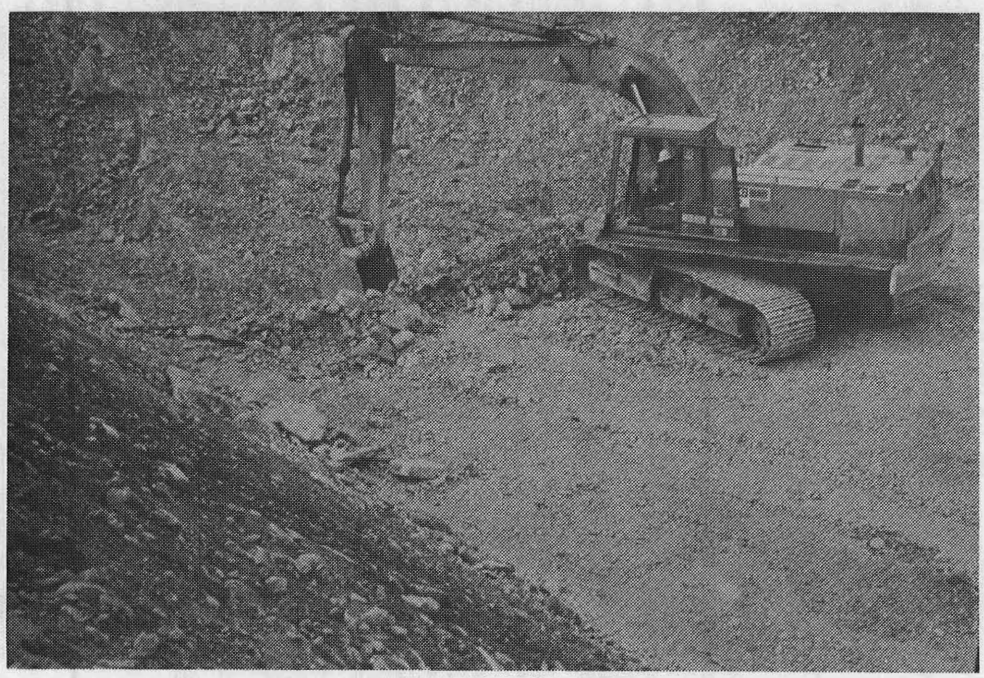


Photo 16. Excavating andesite bedrock in stage I core trench using excavator, vicinity station 31+10. December 1983

Photo 17. Excavator widening stage I core trench. January 1984

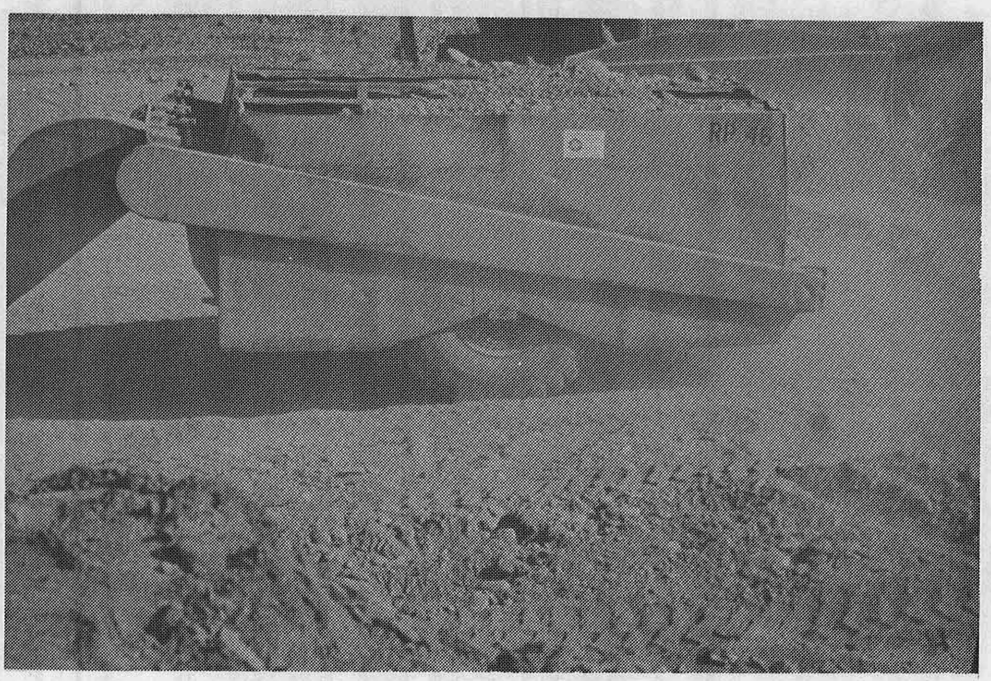


Photo 18. 50-ton roller used to proof roll the foundation. April 1984

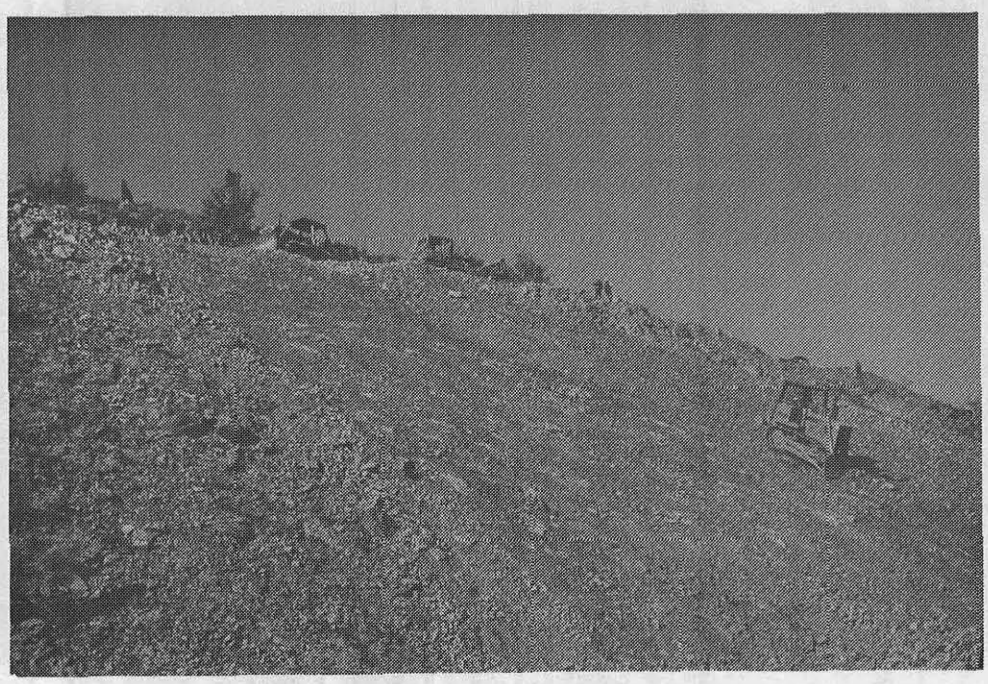


Photo 19. West abutment excavation using a small dozer. November 1983

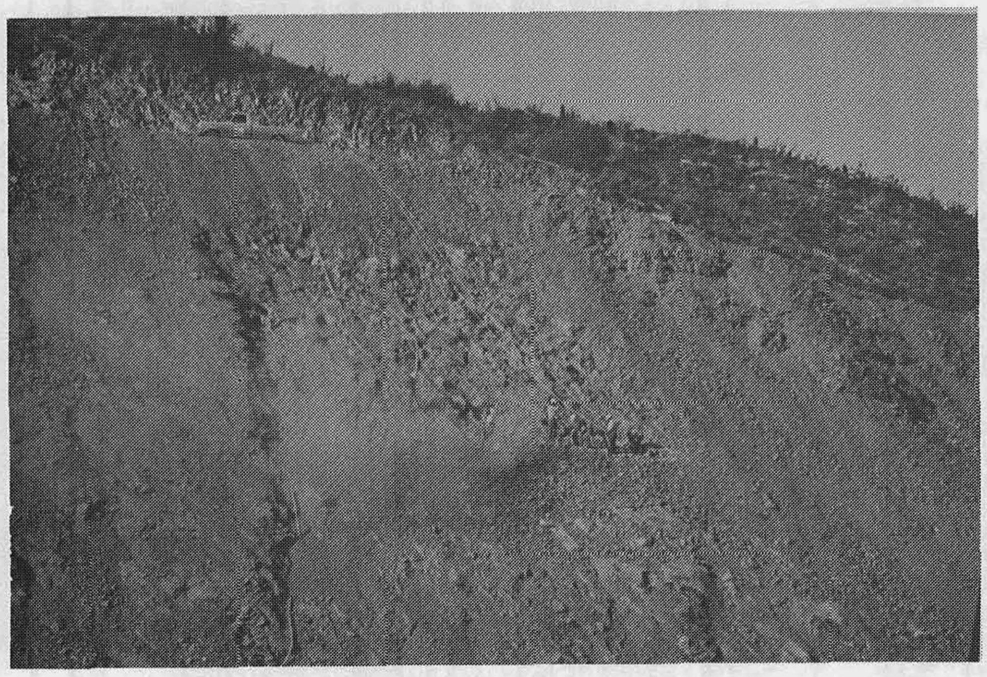


Photo 20. Surface preparation of west abutment using both air blasting and hand labor. December 1983

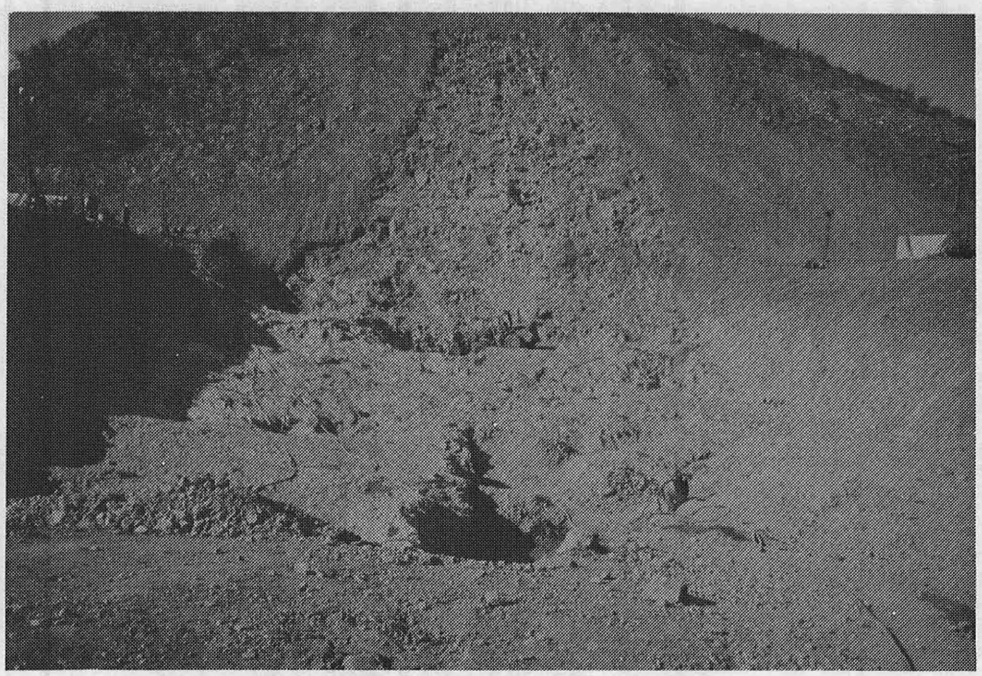


Photo 21. View of completed stage I core trench and west abutment. Note backhoe trench in foreground. February 1984

Photo 22. East abutment excavation using excavator. January 1984

Photo 23. Initial foundation preparation of east abutment surface using low pressure air blasting. Note excavator on left. January 1984

Photo 24. Small backhoe assisting excavator during east abutment excavation. Note rock dust generated from air blasting of excavated abutment surface. January 1984

Photo 25. Foundation preparation in core trench using backhoes and shovels, vicinity station 17+00. March 1984

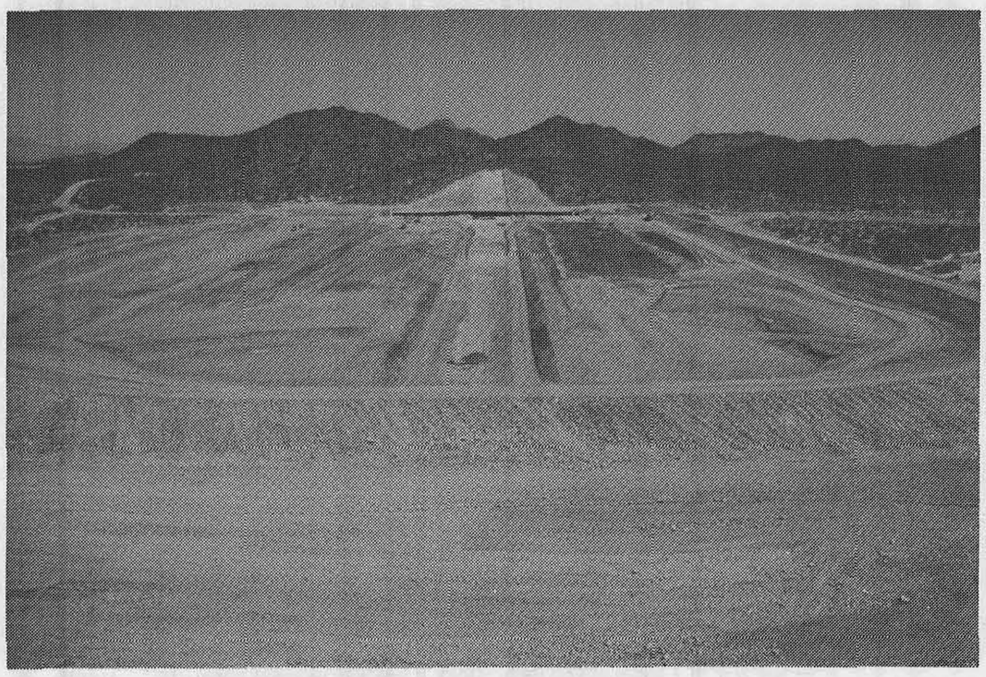


Photo 26. View of stage II diverseion levee protecting completed stage II excavation to station 26+50. Note Stage I embankment protective cover in foreground. March 1984

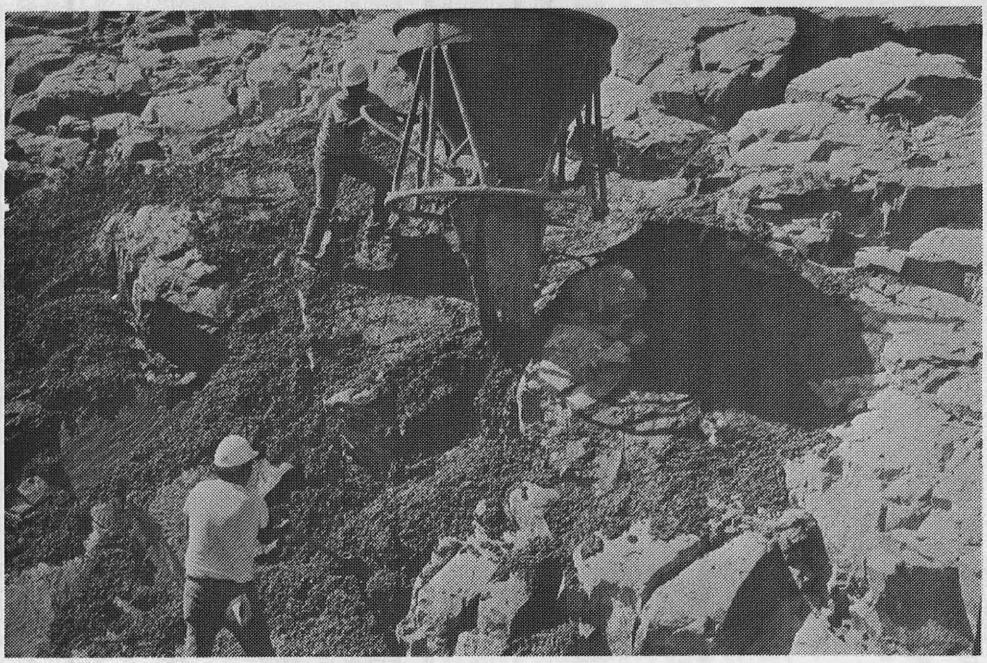


Photo 27. Placing dental concrete on west abutment slope using concrete bucket. October 1984

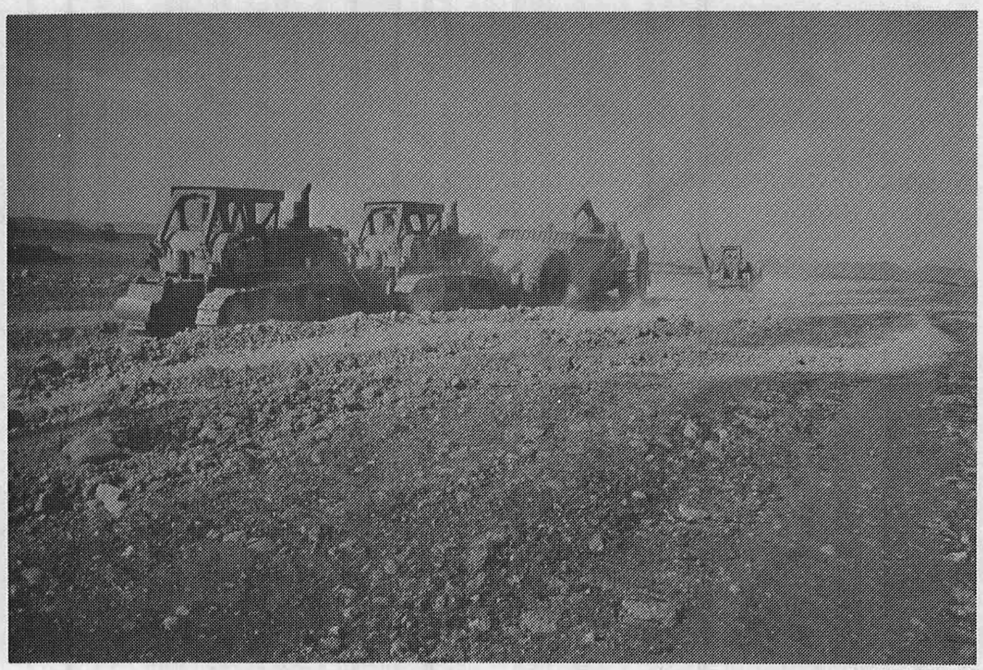


Photo 28. Dike no.1 exploration trench excavation using two push cats and scraper. Note dozer being used to rip caliche. February 1984

Photo 29. View of dike no.1 exploration trench excavation looking north from south abutment. January 1984

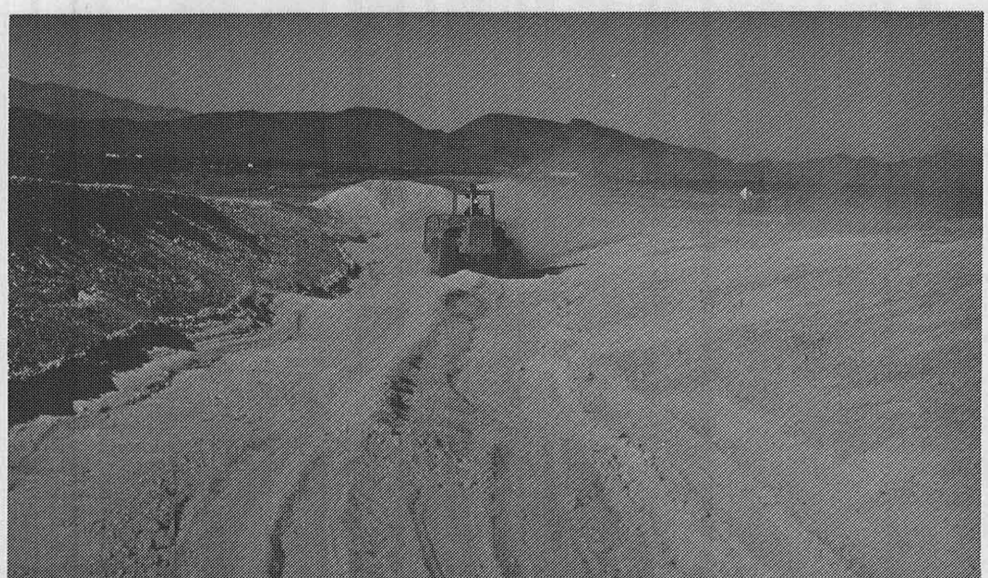


Photo 30. Dike no.1 exploration trench excavation using front end loader to clean up the trench. Note the caliche which is visible throughout the cross section of the trench. January 1984

Photo 31. Dike no.1 exploration trench excavation attempting to rip caliche between station 31+00 and 30+00. February 1984

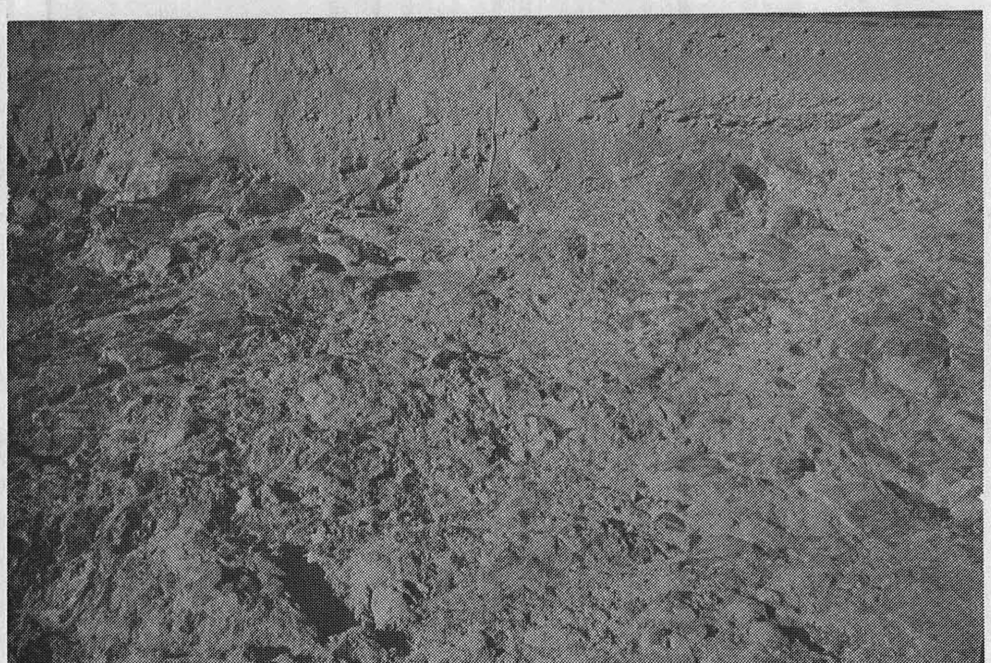


Photo 32. Dike no.1 exploration trench, station 81+25. Note coherent andesite blocks on both sides of photo with calichified andesite breccia toward center. January 1984

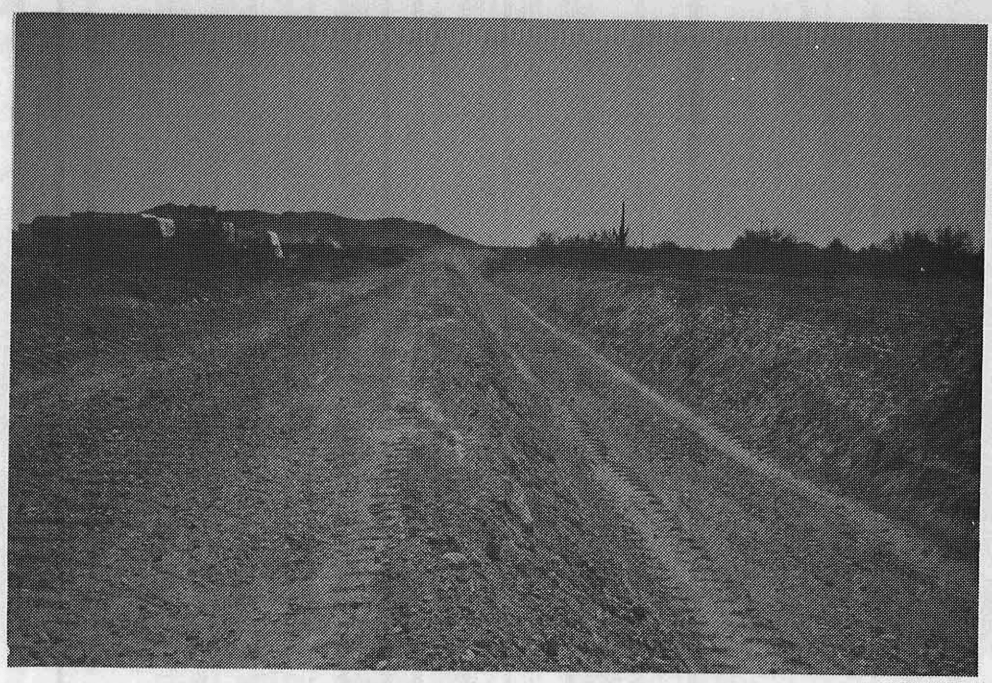


Photo 33. View of dike no.1 completed exploration trench excavation, looking south from station 13+00. Note widespread caliche. February 1984

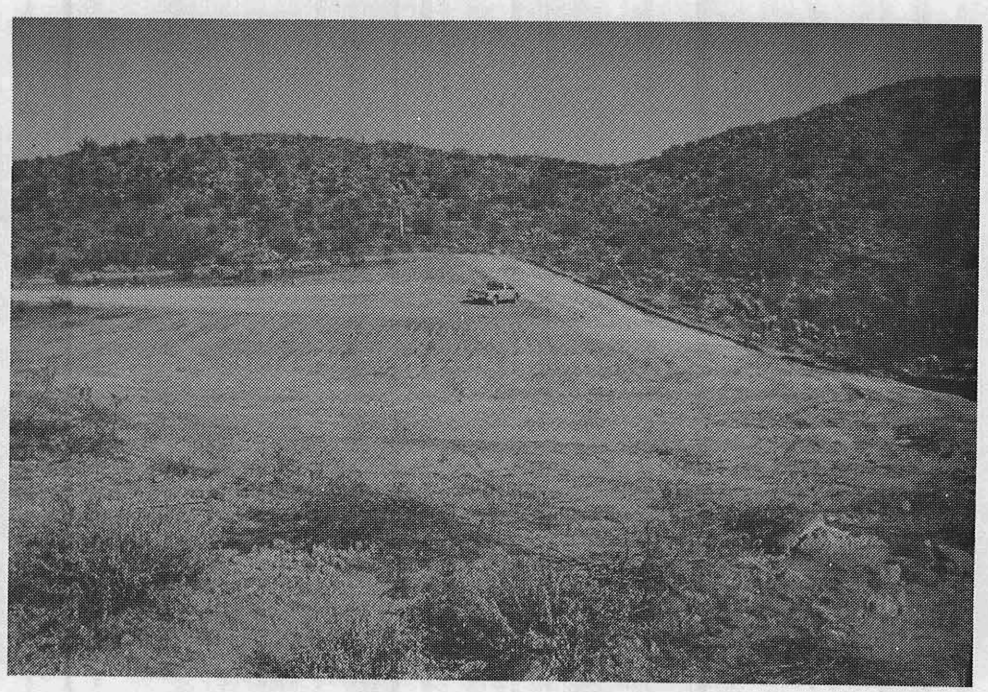


Photo 34. Dike no.2 foundation after stripping. February 1984

Photo 35. Vibrating crusher used to crush and screen pervious shell material to produce gravel drain and type III stone. February 1984

Photo 36. Taking core record sample dike No.1 station 70+10, elevation 1461.0, and 7 feet upstream of centerline. February 1984

Photo 39. End dump truck placing core material on dike no.1. March 1984

Photo 40. Placing core material in core trench. Note the tamping roller being pulled by the dozer, the motorgrader scarifying the surface, the push cats, and scrapers excavating the embankment foundation and placing the suitable materials in the core zone, and the water truck adding water to the core materials. June 1984

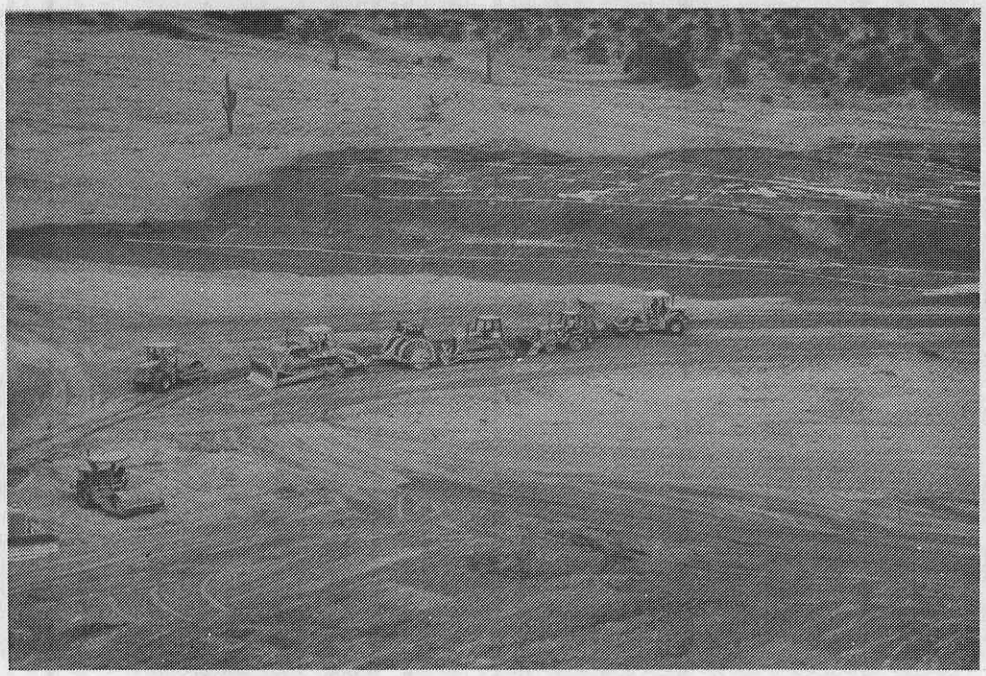


Photo 37. Prewetting core material in borrow area 3. December 1983

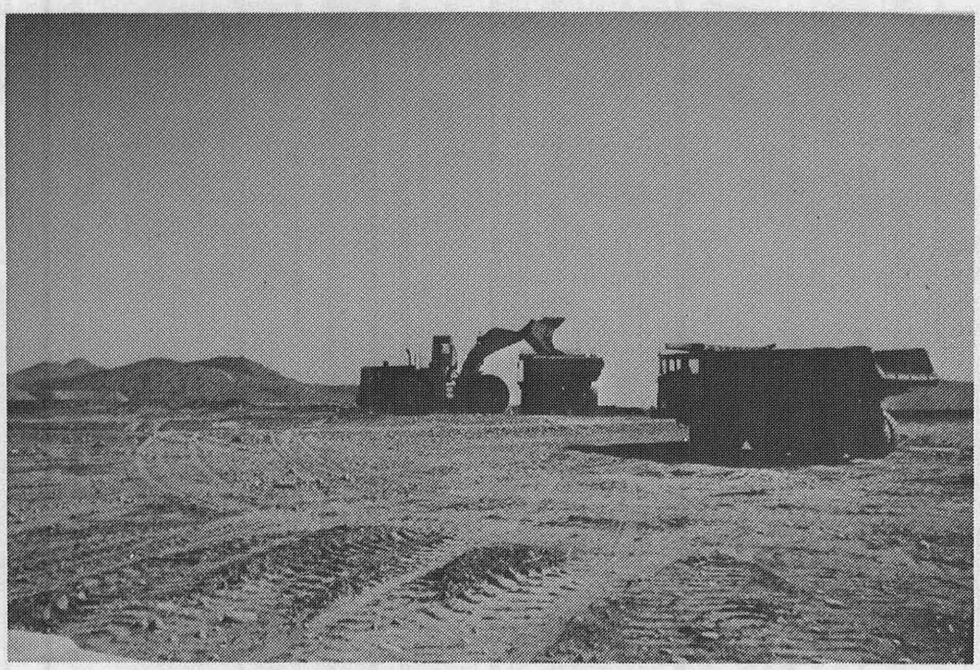


Photo 38. Front end loader loading end dump trucks with core material. April 1984

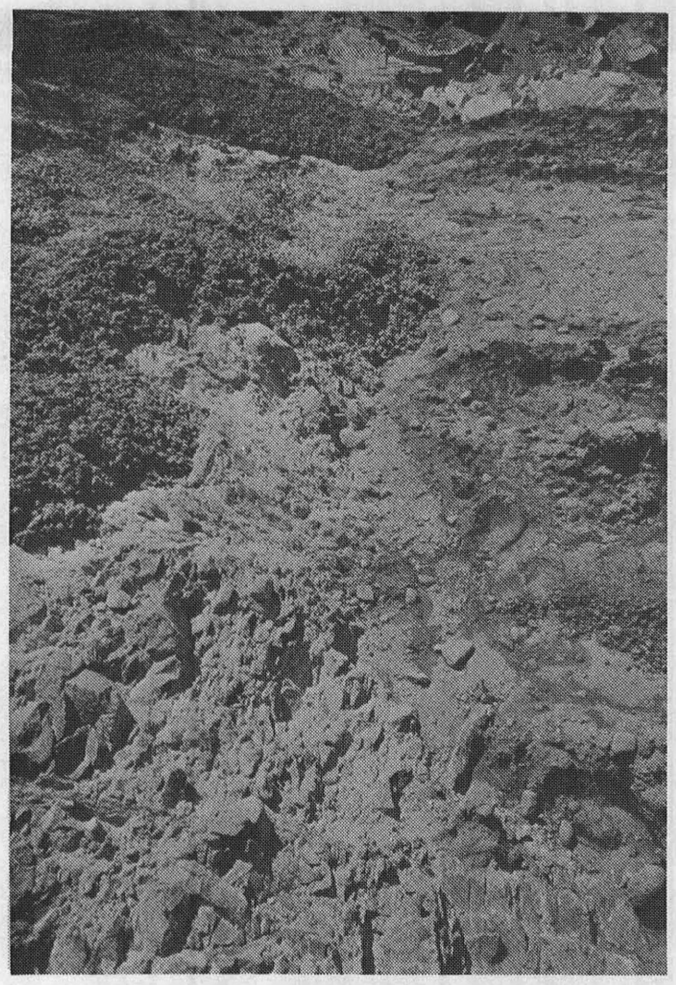


Photo 41. Typical example of rock-core contact on east abutment approximate elevation 1447). Pneumatic wheel rolling core material prevented damage to highly fractured granitic bedrock surface by tamping roller. July 1984

Photo 42. Beginning west abutment surface preparation. Laborers permitted to work only short reaches without safety lines. Note backhoe removing stage I protective cover down to elevation 1380 near base of slope. September 1984

Photo 43. Front end loader with loaded bucket wheel rolling core material at rock-core contact. February 1984

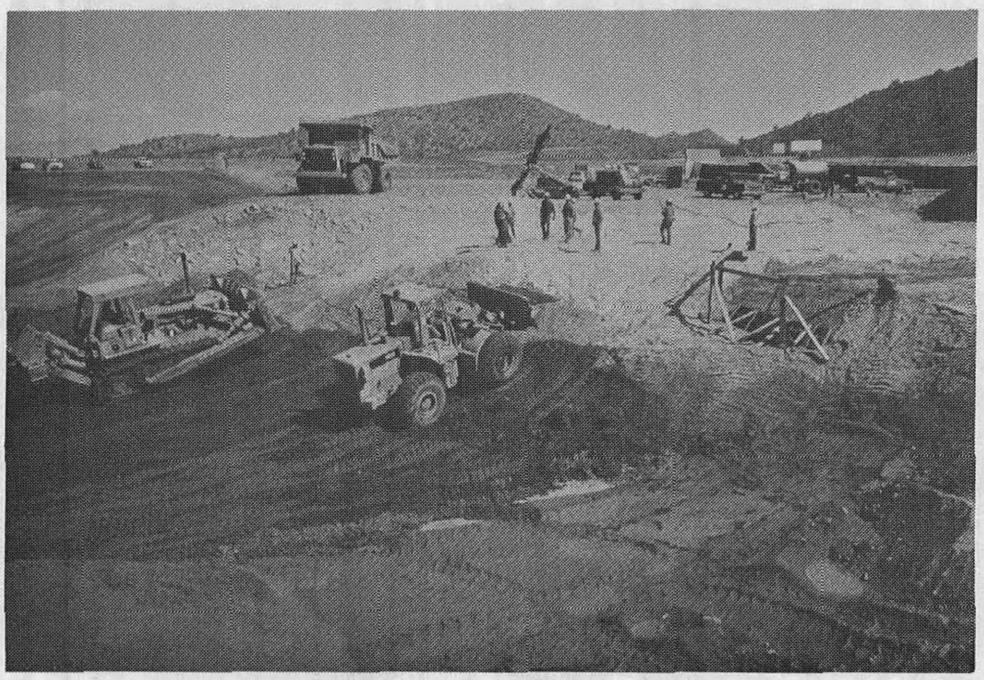


Photo 44. Front end loader compacting core material against bedrock slope in core trench, vicinity station 16+50. Note dozer using blade to spread core material over foundation surface. April 1984

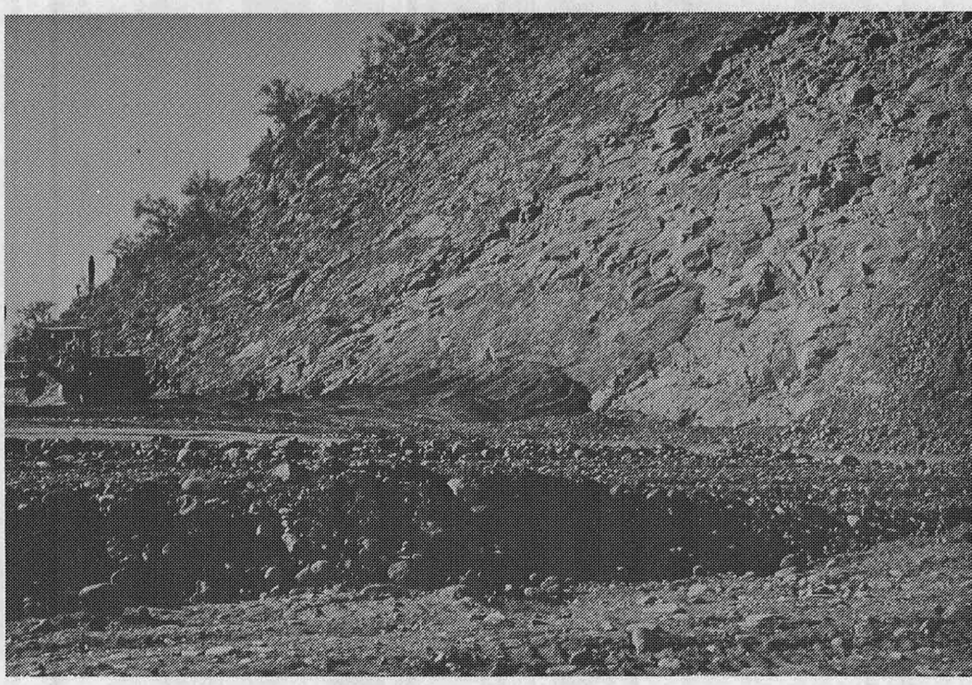


Photo 45. Five foot high ramp of core material against right abutment slope. October 1984

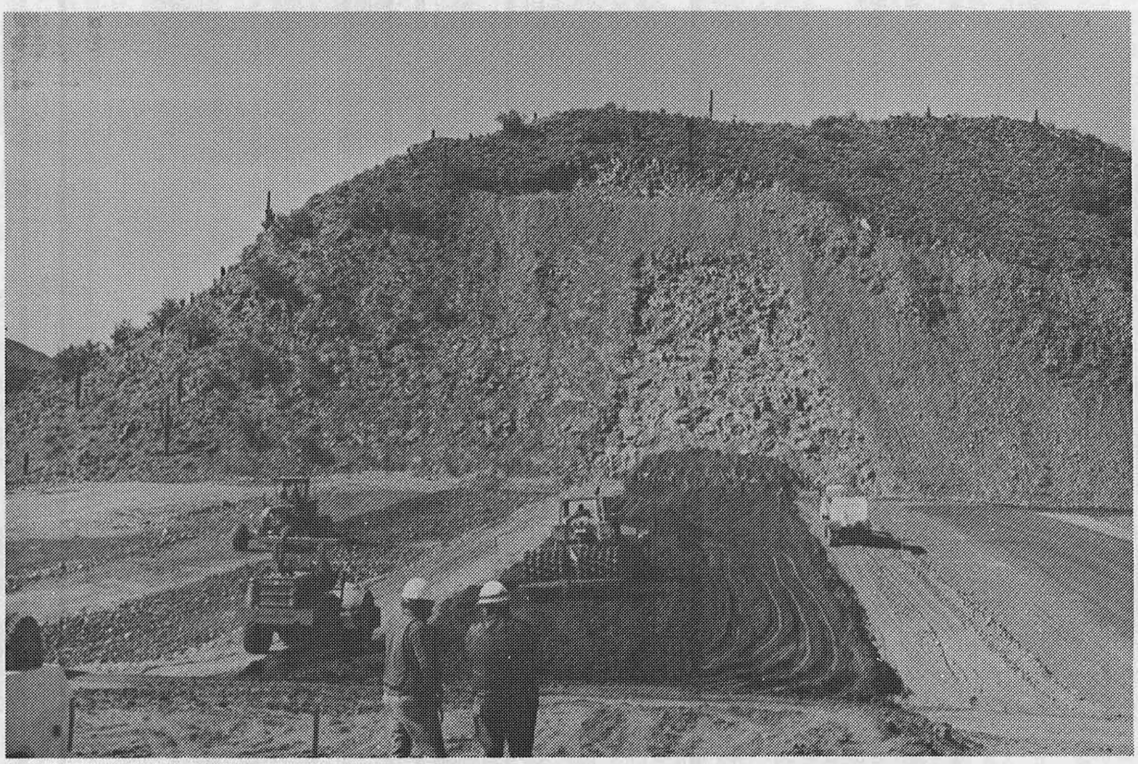


Photo 46. Bottom dump truck placing transition material, vibratory roller compacting transition material, and motor grader scarifying pervious shell material during embankment construction. November 1984

Photo 47. Processing transition material on grade with motor grader during construction of dike no.1. March 1984

Photo 48. Spreading pervious shell material over the gravel drain during embankment construction. April 1984

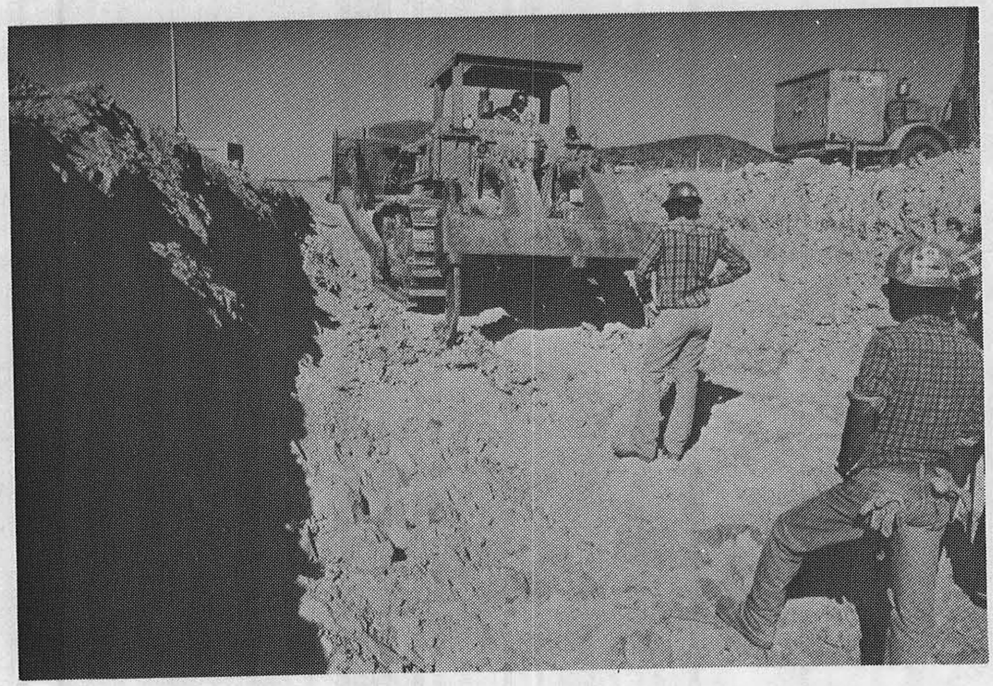


Photo 49. Dozer ripping high spot at station 19+70 in outlet conduit trench. January 1984

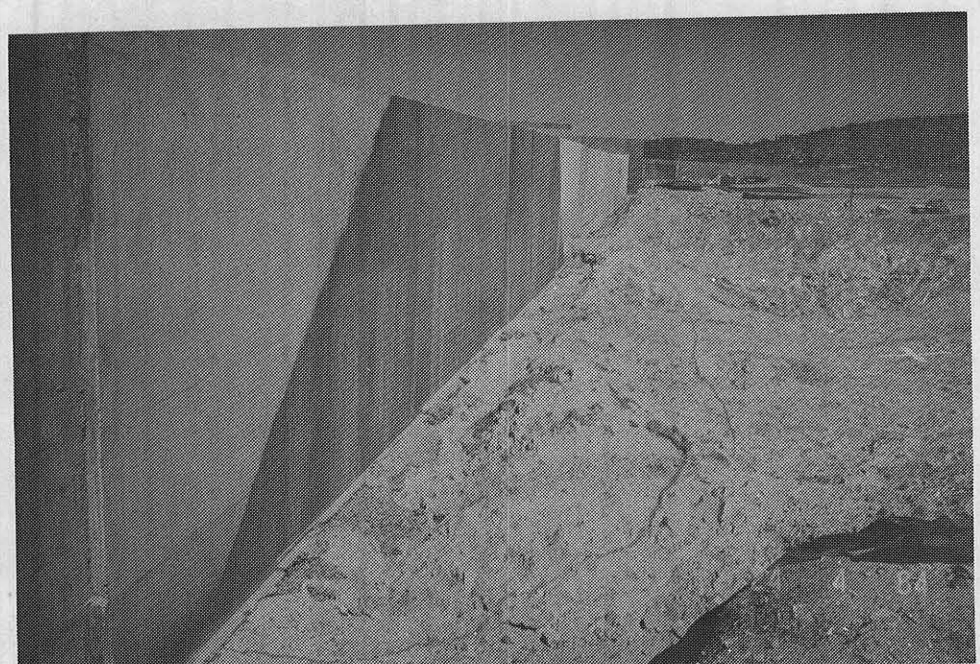


Photo 50. Outline of concrete plug (fillet) area marked on the east side of outlet. Note most of conduit projects above adjacent foundation surface due to greater depth of core trench excavation. April 1984

Photo 51. Concrete plug (fillet) placement on east side of conduit using concrete bucket. Note laborer on top of box vibrating concrete near contact with conduit wall. April 1984

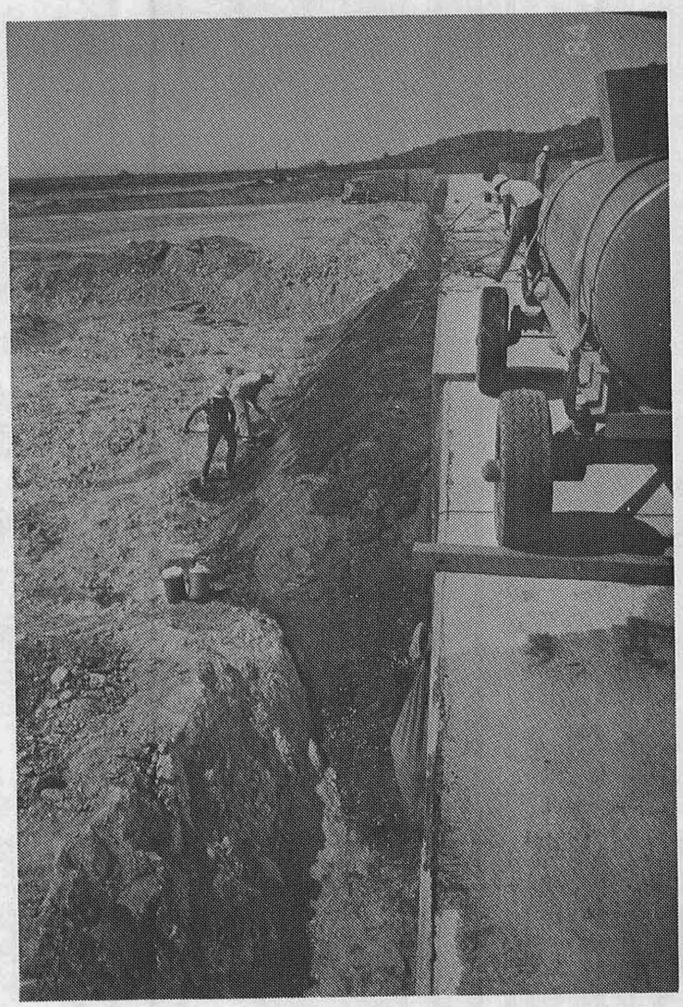


Photo 52. Concrete plug (fillet) on west side of outlet conduit. Note upper portion of plug is slightly steeper than 1:1 to eliminate "Feather" edges against the top of the conduit April 1984

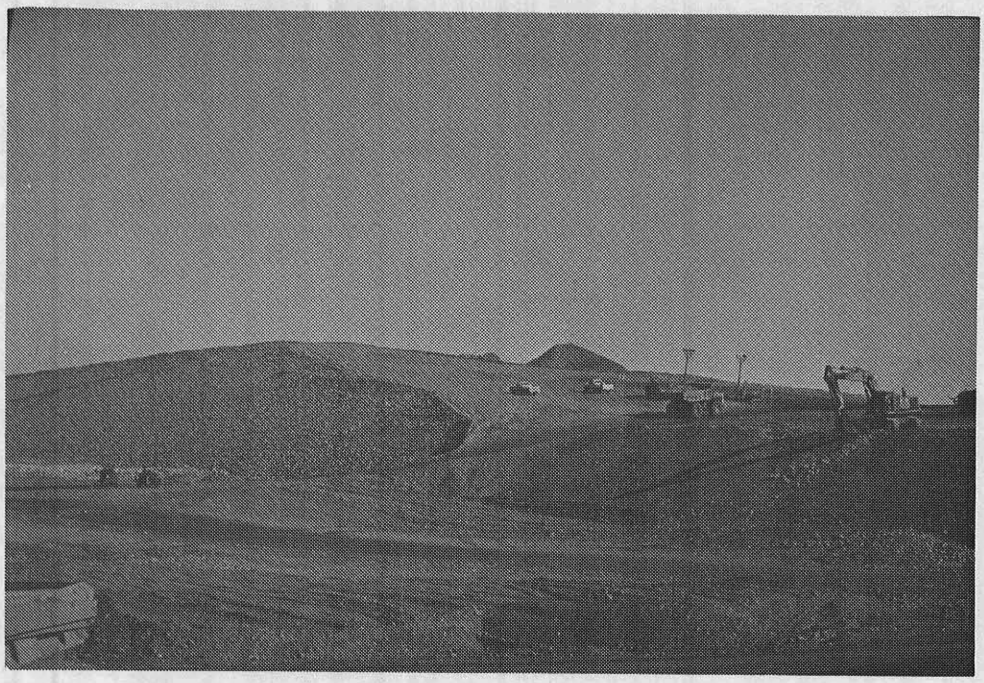


Photo 55. Stage III dam embankment construction at approximately elevation 1420. Completed stage II embankment with upstream stone protection at left. October 1984

Photo 56. Dressing upstream slope of dike no.1 with tracked excavator. June 1984

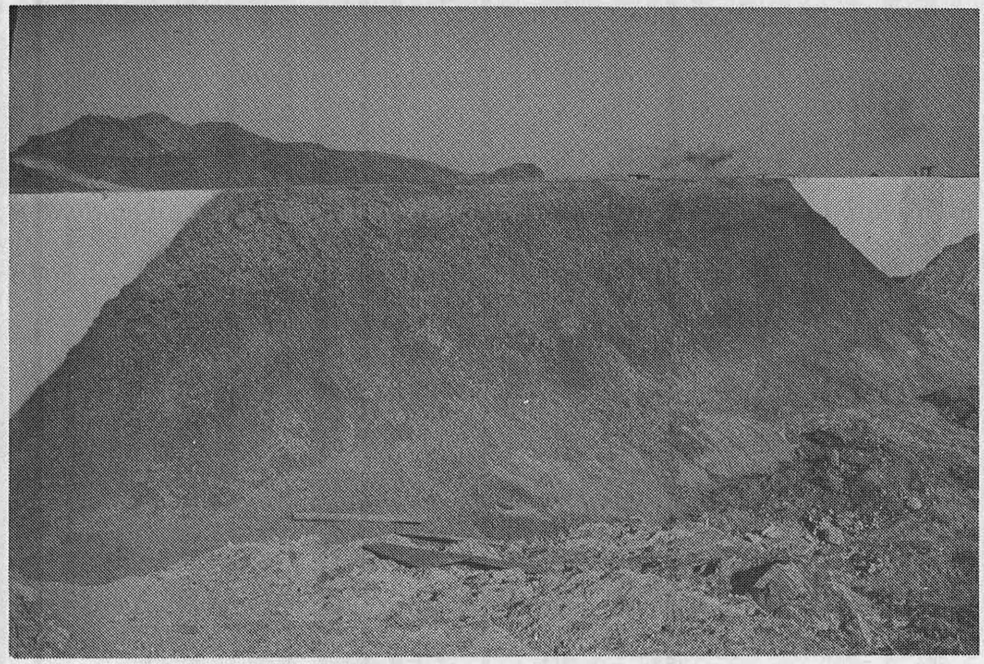


Photo 53. Completed concrete (fillet) plug on east side of conduit. Concrete spills outside plug limits were subsequently removed. April 1984

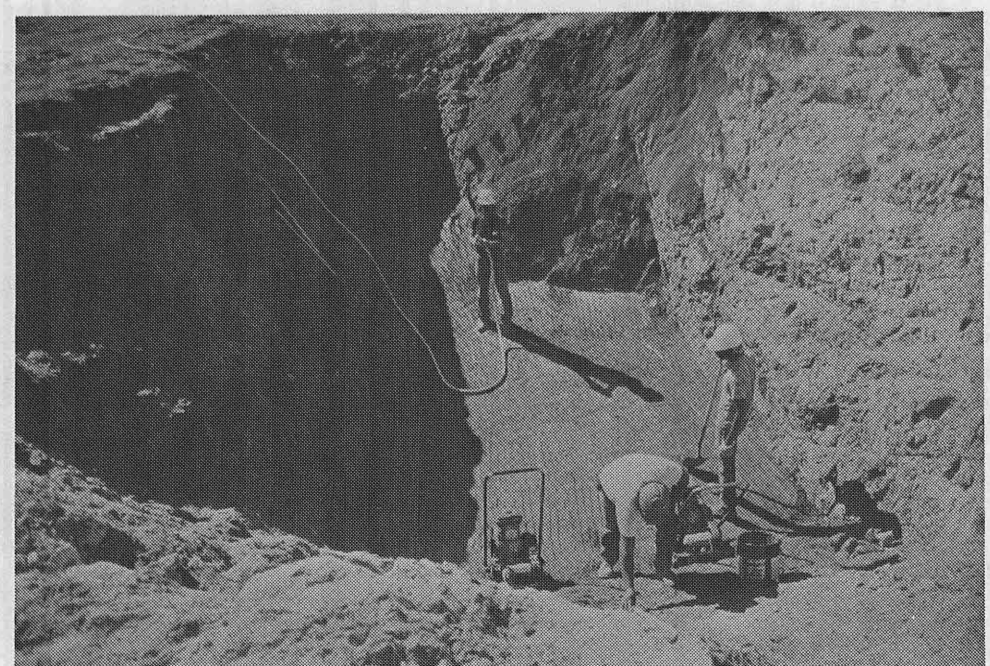


Photo 54. Backfilling foundation depression upstream of dam centerline between stations 14+85 and 15+25. February 1984

ATTACHMENTS

ATTACHMENT 1

The Earth Technology Corporation

3116 West Thomas Road, Suite 601 • P.O. Box 14570 • Phoenix, Arizona 85063
Telephone: (602) 269-7501 • Telex: 656338

January 23, 1984

U. S. Army Corps of Engineers
New River Dam
P. O. Box 2019
Sun City, Arizona 85372

Attention: Captain Dunn

Subject: Soil Sampling and Laboratory Testing
New River Dam, Arizona
Earth Technology Project No. 84-164-01

Gentlemen:

At your request we have sampled and tested four soil samples obtained from the fracture filling material in andesite bedrock fractures at the base of the core excavation at New River Dam. Samples were obtained December 30, 1983 by our staff geologist, Ron Whitler, under the direct supervision of Corps geologist Bob Thurman. Samples were obtained from fractures ranging from about 1/4 inch to 2 1/2 inches wide. At your request, the following tests were performed on each sample:

- o Atterberg limits
- o Grain size distribution
- o S.C.S. double hydrometer dispersion test
- o Salinity, soluble sulfates, and soluble chloride content

Results of the Atterberg limits, dispersion, salinity, sulfate and chloride content tests are presented in the attached tabulation. Test results were telephoned to the dam site project offices on January 1, 1984. Results of the gradation tests are presented on the attached grain size plot, Figure 1. As requested by the Corps, tests were performed on minus #40 size material to screen out rock fragment contamination picked up during sampling.

Soil descriptions and sample locations are presented below:

Sample No. 1:

CLAY (CH), light greenish gray 5GY 7/1 (Munsell), high plasticity, hard, waxy, noncalcareous, no appreciable coarse sediments except from contamination, trace of hematitic and manganese stains. From bottom of core turn, Station 29+90, 12-20 feet upstream of centerline.

Sample No. 2:

CLAY (CH), mostly light greenish gray 5GY 7/1, but some light gray 5Y 7/1 coloration with 20% of material 7.5 YR 6/8 reddish yellow hematitic coloration, high plasticity, very stiff to hard, noncalcareous, hematite and manganese stains, slightly more porous and less plastic where hematitic colored. From Station 30+20, bottom of core turn, 22 feet downstream of centerline.

Sample No. 3:

CLAY (CH), light greenish gray 5GY 7/1 and reddish yellow 7.5 YR 6/8 with manganese stains, high plasticity, hard, waxy, noncalcareous, slightly more porous than Sample No. 1. From Station 30+70, bottom of core turn, 10-25 feet upstream from centerline.

Sample No. 4:

CLAY (CH), mostly light greenish gray 5 GY 7/1 with 15% reddish yellow 7.5 YR 6/8 and a trace light gray 5Y 7/1, high plasticity, very stiff, noncalcareous, not as stiff as Sample No. 1. From Station 30+90, 12-29 feet downstream of centerline, bottom of core turn.

Please call us if we can be of any further assistance at the New River Dam site.

Sincerely,

Steven A. Haire, P.E.
Project Engineer

SH: jm
cc: R. Roodsari

LABORATORY TEST RESULTS

ANDESITE FRACTURE FILLING AT NEW RIVER DAM

Sample No.	Location	USCS Soil Type	Atterberg Limits LL	Atterberg Limits PI	Salinity (mmhos/cm)	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)	S.C.S. Double Hydrometer Dispersion (%)
1	Sta. 29+90 R 12 to 20'	Clay (CH)	96	64	0.75	127	74	13.7
2	Sta. 30+20 L 22'	Clay (CH)	102	65	0.90	136	82	16.2
3	Sta. 30+70 R 10 to 25'	Clay (CH)	110	74	1.50	165	94	20.5
4	Sta. 30+90 L 12 to 29'	Clay (CH)	115	73	0.65	148	84	18.2

ATTACHMENT 2

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
LABORATORY
P O BOX 37, SAUSALITO, CALIFORNIA 94966

REPLY TO
ATTENTION OF:

SPDED-DL

7 MAR 1984

SUBJECT: New River Dam Arizona

Commander
US Army Engineer District, Los Angeles
ATTN SPLED-GD, A. Roodsari
Post Ofice Box 2711
Los Angeles, CA 90053

1. References:

a. DA Form 2544, CIV 84-53 dated 7 February 1984, requesting testng of soil samples.

b. Samples relative to reference a received on 13 February 1984. Identification of samples is on inclosed plate.

2. Pinhole Erosion Test and Atterberg Limits tests were performed on the above samples in accordance with Engineer Manuarl, EM 1110-2-1906, "Laboratory Soil Testing", 30 November 1970.

3. Soluble salt tests will follow when completed.

4. Total cost of testing is $510.00. Billing will be made by the Sacramento District, Finance and Accounting Branch.

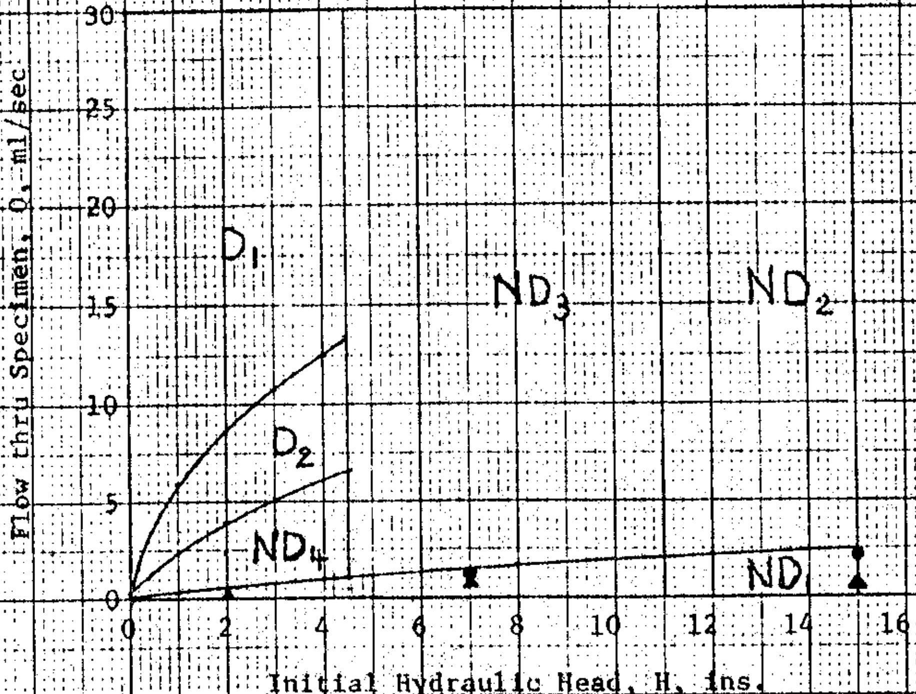
FOR THE COMMNADER:

1 Incl (trip)
as

Melvin W Cohen
MELVIN W. COHEN
Director, SPD Laboratory

SOUTH PACIFIC DIVISION
LABORATORY

PINHOLE EROSION TEST



D_1 & D_2-Dispersive and highly erodable

ND_4 & ND_3-Nondispersive intermediate erosion resistant

ND_2 & ND_1-Nondispersive erosion resistant

Div. No.	Hole No.	Depth ft.	LL	PI	Classification	Dry Density pcf	W.C. %	Disp. Class.
86641	30+50	South Trench	112	75	▲ Clay (CH)	83.7	35.8	ND_1
Composite								
86640	30+00	North Trench	109	71	● Clay (CH)	78.0	39.5	ND_1
86642	30+60	"						

Project: NEW RIVER DAM

Date: March 1984

SEE 20x20 TO INCH

APPENDIXES

APPENDIX I

APPENDIX I

LARGE SCALE IN-SITU DENSITY TEST

CONTENTS

EQUIPMENT

1.01 The large scale in-situ density test equipment consists of a 4-foot inside diameter ring and truck mounted water tank, water meter, hoist and a large platform scale in weight the excavated material.

PROCEDURES

2.01 Select a location where the materials are representative and disturbance is minimal. Level an area for the ring to a depth sufficient to remove all disturbed materials and high spots which may cause the ring to shift. Place the ring on the leveled area and fill low areas under the ring to stabilize the ring and prevent the plastic sheet from getting under the ring.

2.02 Initial volume measurements of the ring and irregular ground surface is obtained. Place a 4 mill plastic sheet in the ring and smooth to conform with the ground surface. Discharge water into the ring until the water surface reaches a fixed level. Smooth and fit the plastic sheet around the edges as the ring fills. Record the initial volume of the water.

2.03 Pump the water out and remove the plastic sheet. Care must be taken during water removal to prevent water from altering the material moisture content. Excavate the area inside the ring and place the material in a barrel. Excavate the hole carefully to prevent sharp edges and overhangs that the plastic sheet could not conform to. Weigh excavated materials and record the weight. Approximately 1000 to 1100 pounds of material must be excavated for the density sample.

2.04 Upon completion of the excavation place the plastic sheet into the hole and ring. Weigh and record the +6-inch material before placing into the hole on the plastic sheet. Discharge water into the hole. As the water fills the hole smooth the plastic sheet to conform to the shape of the hole. Bring the water to the same level as the initial filling and record the volume of the water in the hole from the meter.

2.05 The volume of the excavated hole is the difference between the final and initial volume measurements.

2.06 The in-situ density is the weight of the excavated materials divided by the volume of the hole.

APPENDIX II

APPENDIX II

ADJUSTMENT OF LABORATORY DENSITY FOR OVERSIZE

Due to the maximum particle size limitations for the laboratory tests, oversize materials were removed from the field gradation of the transition and previous shell materials. The maximum size materials allowed in the 12-inch diameter compaction mold is minus 3-inch material. The oversize mateial can be replaced mathematically to adjust the laboratory density to the in-situ density with the laboratory density assumed as the soil matrix. Adjustments for the oversize material can be made by using the following equations.

$$T = \frac{1}{\frac{P}{62.4\ G} + \frac{1-P}{m}}$$

T = Adjusted dry density
P = Decimal percent of oversize added
G = Bulk specific gravity of oversize
m = dry density of minus fraction used in density tests

$$C = \frac{100-D}{100} - T$$

C = corrected dry density
T = adjusted dry density
D = antilog (aP+b)
a = 0.017
b = -0.43
P = Percent Oversize

The adjusted densities will be corrected for the disturbances (D) effect caused by adding the oversize material.

Reference

Frost, R. J., "Some Testing Experiences and Characteristics of Boulder-Gravel Fill Earth Dams," Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM STP 523, American Society for Testing and Materials, 1973, pp 207-233.