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Little Rock District

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RED RIVER WATERSHED

GILLHAM LAKE

COSSATOT RIVER, ARKANSAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

OCTOBER 1987

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This report provides a summary record of significant design, construction and operational data on the Gillham Dam and Reservoir. It was prepared in accordance with ER 1110-2-1901, "Embankment Criteria and Performance Report," and is for use by engineers to familiarize themselves with the project, re-evaluate the embankment when needed, and for guidance in design of comparable future projects. The report presents a general description of the foundation conditions, type of material and placement methods of the various sections of the embankment, the design considerations on stability and seepage control, significant operational events, and an evaluation of the conditions of the embankment. Pertinent drawings, design and construction data, and photos are also included. The Gillham Dam and Reservoir are located in Howard and Polk Counties, Arkansas. The dam consists of a rockfill embankment with compacted earth core, concrete gravity spillway controlled by tainter gates, and an intake structure. The project purposes are flood control, water supply, water quality, recreation, and fish and wildlife conservation.

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GILLHAM DAM AND RESERVOIR
COSSATOT RIVER, ARKANSAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

October 1987

U. S. Army Engineer District, Little Rock
Corps of Engineers

Prepared by
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Little Rock, Arkansas

GILLHAM DAM AND RESERVOIR
COSSATOT RIVER, ARKANSAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

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GILLHAM DAM AND RESERVOIR
SALINE RIVER, ARKANSAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PERTINENT DATA

LOCATION

Cossatot River, Mile 49, Howard and Polk County, Arkansas.

PROJECT STRUCTURES

Rockfill dam with compacted earth core, concrete gravity spillway controlled by tainter gates, and intake structure with facilities for a flood control conduit, low flow outlet, water supply outlet, an office building, and recreational facilities.

PROJECT PURPOSES

Flood control, water supply, water quality control, and fish and wildlife conservation.

DRAINAGE AREA

Two hundred seventy one (271) square miles upstream from the dam.

ELEVATIONS, AREAS AND STORAGES

Feature	Elevation (feet, NGVD)	Areas (Acres)	Storage Acre-feet)	Storage (Inches)(1)
Top of Dam	586.0	6,000	-	-
Maximum Pool	581.0	5,590	283.310	19.60
Top Flood Control Pool	569.0	4,680	221,780	15.34
Spillway Crest	527.0	2,330	78,580	5.44
Top Conservation Pool	502.0	1,370	33,030	2.28
Top Inactive Pool	464.5	310	3,720	0.26
Flood Control Storage	502.0-569.0	-	188,750	13.06
Conservation Storage	464.5-502.0	-	29,310(2)	2.03

(1) From total drainage area of 271 square miles.

(2) Includes 20,600 acre-feet for water supply and 8,700 acre-feet for water quality control.

EMBANKMENTS

Location	Cossatot River at mile 49.0
Type	Nonoverflow embankment
Type of Fill	Random rock with impervious core
Slope Protection	Riprap upstream and downstream
Height	160 feet above streambed
Length	1,750 feet
Crest Width	32 feet
Top Elevation	586.0 feet NGVD
Design Flood	Maximum probable flood
Freeboard	Five feet above maximum pool
Used for Roadway	Yes
Elevation of Streambed	426.0 feet NGVD

SPILLWAY

Location	Right abutment about 1,500 feet west of main embankment
Type	Gated concrete gravity ogee weir
Crest Elevation	527.0 NGVD
Net overflow Width	200 feet
Number and Size of Gates	Four - 50' x 41'
Type of Gates	Tainter
Top of Gate Elevation	569.0 NGVD
Induced Surcharge	None
Design Head	54.0 feet
Maximum Discharge Capacity	272,700 c.f.s.
Bridge Deck Elevation	586.0 NGVD
Type Energy Dissipator	Flip bucket
Time Required to Open/ Close all Gates	42 minutes

OUTLET FACILITIES

Intake Structure	One intake structure with facilities for flood control conduit, water supply, and low flow intakes

FLOOD CONTROL CONDUIT

Location	Near west end main embankment - Right Abutment
Purpose	Flood Control
Type of Outlet	Concrete-lined tunnel
Size of Outlet	10' diameter
Type of Service Gate	Hydraulically-operated slide gates
Number and size of Gates	Two 4.5' x 10'
Entrance Invert Elevation	437.0 NGVD
Minimum Time Required to Open/Close Service Gate	10 minutes
Type Emergency Closure and Time Required	Hydraulically-operated slide gate requiring 10 minutes to close gate.
Type Energy Dissipator	Concrete stilling basin with baffles

LOW FLOW OUTLET

30" Pipe

Location	Near west end main embankment
Purpose	Low flow releases
Type of Outlet	Circular pipe
Size of Outlet	30-" diameter at vent pipe
Type of Service Gate	Hydraulically-operated butterfly valve. Original insert gates located in main service gates were deleted in 1986.

Multilevel Intake

Elevation	487.0 and 472.0 NGVD

WATER SUPPLY OUTLET

Location	Near west end main embankment
Purpose	Water Supply
Type of Outlet	Circular pipe
Size of Outlet	24" diameter
Type of Service Gate	Gear-operated gate valve
Number and Size of Gate	One - 24 inches
Multilevel Intake Elevations	477.0 and 458.5 NGVD

1. INTRODUCTION

1.1 Purpose and Scope of Report. This report provides a summary record of significant design, construction and operational data on the Gillham Dam and Reservoir. It was prepared in accordance with ER 1110-2-1901, "Embankment Criteria and Performance Report," dated 31 December 1981 and is for use by engineers to familiarize themselves with the project, re-evaluate the embankment when needed, and for guidance in design of comparable future projects. This report was prepared jointly by Grimes & Johnson, Inc. and Grubbs, Garner & Hoskyn, Inc., for the Soils and Materials Section, Little Rock District, under the provisions of Contract No. DACW38-86-D-0070, Delivery Order No. 0007.

The report presents a general description of the foundation conditions, type of material and placement methods of the various sections of the embankment, the design considerations on stability and seepage control, significant operational events, and an evaluation of the conditions of the embankment. Pertinent drawings, design and construction data, and photos are also included. More detailed descriptions of the foundations conditions are contained in the report "Gillham Dam Foundation, Part III, Embankment and Foundation Grouting."

1.2 Brief Description and Purpose of Project. The Gillham Dam and Reservoir are located in Howard and Polk Counties, Arkansas. The dam is at mile 49.0 on the Cossatot River, a tributary of the Little River in southwest Arkansas. It is approximately 6 miles northeast of the town of Gillham, Arkansas and 75 miles north of Texarkana, Arkansas.

The dam consists of an embankment, a gated spillway and outlet works. The main dam embankment is a 1,750 foot long rockfill dam with compacted earth core rising to 160 feet above the streambed. Located on the right abutment in a natural saddle, is a gated concrete gravity ogee weir spillway 200 feet wide, with 4 - 50-foot tainter gates. The outlet works has a 10-foot diameter conduit (concrete lined tunnel) controlled by hydraulically operated slide gates in the gate tower with releases made through the stilling basin at the downstream portal.

There is also a 30" diameter low flow outlet and a 24" diameter water supply outlet. The outlet works is located at the right abutment of the main dam. The service bridge to the gate tower parallels the main dam from the right abutment overlook area. Spur dikes, saddle dikes, and a training dike are located as shown on Plate 2. Other structures include an office/maintenance building and recreational facilities.

The project purposes are flood control, water supply, water quality, and fish and wildlife conservation. Recreation was not a designated project purpose; however, it was later added as incidental to them.

1.3 Project Authorization. The Gillham Dam and Reservoir project was authorized by the Flood Control Act approved 3 July 1958 (Public Law 85-500, 85th Congress, 2nd Session). Related legislation included House Document No. 170, the report "Millwood Reservoir and Alternate Reservoirs, Little River, Oklahoma and Arkansas."

1.4 Project Construction History. Construction of the right abutment access road, Howard County road relocations, and the project buildings began in June 1963 and was completed in September 1966. Construction of the outlet works began in January 1966 and was completed in June 1968. Construction commenced in April 1968 on the spillway and was halted in February 1971. Work resumed in 1972 and was completed in August of that year. The contract for construction of the main embankment was awarded in July 1972 and completed in October 1975. Photographs taken during construction are contained in Appendix B. Impoundment of the lake began in May 1975 and reached conservation pool elevation of 502.0 NGVD in April 1976. The pool of record occured on 5 December, 1982 at elevation 561.64 NGVD.

The public use areas at Gillham Lake were constructed between June 1975 and July 1976. They include Cossatot Reefs, Cossatot Point, Coon Creek, and Little Coon Creek.

A summary of construction contracts for the entire project is contained on the following page.

LIST OF PROJECT CONTRACTS

CONTRACT NO.	CONSTRUCTION FEATURE	CONTRACT AMOUNT	START DATE	COMPLETION DATE
DA-34-066 CIVENG-65-1726	Right Abutment Access Road	$ 394,223	20 Jun 63	22 Sept 66
Contractor:	G. F. Thomison Const., P. O. Box 15, Eufaula, OK			
DA-34-066 CIVENG-65-1728	Construction of Project Buildings	$ 134,730	11 Mar 65	02 Jun 66
Contractor:	LaForge & Budd Co., P. O. Box 393, Parsons, KS			
DA-34-066 CIVENG-66-223	Construction of Outlet Works	$1,901.000	14 Jan 66	07 Jun 68
Contractor:	Martin K. Eby Const., 610 N. Main St., Wichita, KS 67201			
DACW56-66-C-0384	Relocation of Howard Co Roads	$ 558,055	11 May 66	17 Nov 67
Contractor:	E. W. Blair Inc., P. O. Box 450, Broken Bow, OK			
DACW56-68-C-0183	Construction of Dikes & Spillway	$2,925,859	26 Apr 68	01 Jul 72
Contractor:	Martin K. Eby Const., 610 N. Main St., Wichita, KS 67201			
DACW56-73-C-0008	Construction of Main Embankment	$4,131,725	31 Jul 72	10 Oct 75
Contractor:	Hixson & Lehenbauer, P. O. Box 1574, Topeka, KS 66601			
DACW56-73-C-0217	Addition to Intake Structure	$ 129,422	11 Jun 73	17 Jul 74
Contractor:	Amis Const. Co., P. O. Box 1871, Ok City, OK			
DACW56-73-C-0242	Overlook Structure No.1	$ 98,300	14 Jun 73	10 Apr 74
Contractor:	LaForge & Budd Co., P. O. Box 393, Parsons, KS			
DACW56-75-C-0209	Construction of Public Use Areas Stage I (Cossaatot Point, Cossatot reefs, Coon Creek & Little Coon Creek)	$ 970,873	30 Jun 75	15 Jul 76
Contractor:	Triark Inc. & Souter & Assoc., P. O. Box 1052, LR, AR 72203			
DACW56-76-C-0104	Repair of Structural Cracks & Weir Surface of Spillway	$ 29,680	N/A	N/A
Contractor:	Hunt Process Corp.			

1.5 Periodic Inspections. In accordance with the requirements for periodic inspection and continuing evaluation of completed civil works structures, the following have been published for periodic inspections that have been performed on the embankment, spillway and outlet works at Gillham Dam and Reservoir. They contain the problems developed after construction and instrumentation data.

Periodic Inspection Report No. 1	April 1975
Periodic Inspection Report No. 2	April 1976
Periodic Inspection Report No. 3	November 1979
Periodic Inspection Report No. 4	April 1983
Periodic Inspection Report No. 5	March 1985

2. GEOLOGY

2.1 General Geology and Topography. The area of the project is a part of the rugged Ouachita Mountain section of the Ouachita physiographic province. Bedrock consists of intensely folded Paleozoic rocks. As a result of the erosion of these folded strata, the general area is characterized by parallel hogback ridges separated by narrow, steep-sided valleys. Trellis drainage patterns are common. Streams are swift and flood plains are poorly developed or absent.

2.2 Geology of Site. At the Gillham site, the Cossatot River, flowing southward, has cut through an east-west trending ridge to form a narrow, steepwalled valley. The configuration of the dam site and that of the spillway site, located in a saddle about 1,700 feet northwest of the right abutment, is shown on Plate 2. Bedrock is the Stanley shale of Mississippian-Pennsylvanian age. This formation consists of hard quartzitic sandstones and hard silty to sandy shales. The overburden mantle, which usually masks the bedrock even on steep slopes, has an average thickness of approximately 6 feet along the dam axis and about 4 feet in the spillway area. The greatest thickness of overburden was in Hole No. 31 near the base of the right abutment where talus more than 31 feet thick was found. Talus forms a large part of the overburden on the steep slopes, especially on the right abutment. The overburden is mostly residual silt and sand. There is almost no flood plain, and very little alluvium at the site.

The water table was near river level in the right abutment (Hole No. 37), whereas in the left abutment the water table was observed to rise to about 20 feet above the level of the river (Hole No. 34). Several seeps and small springs were observed about 10 feet above the river at the base of the left abutment. In the spillway area, the water table was found at elevation 515, about 25 feet below the surface, at the low point of the saddle (Hole No. 28).

The principal structural feature in the site area is an eastward plunging syncline whose axis is near the axis of the dam and roughly parallel to it. The spillway site is on the north flank of the syncline and, since the syncline is plunging, the rocks strike at an angle of about 30° to the spillway axis. Therefore, both downstream and right-to-left dip components are formed. The thick shale which underlies the left side of the spillway site has been eroded from higher elevation to form the notch or saddle in which the spillway is located. The shale was found to exist at each end of the outlet tunnel (Plate 13). Local contortions which affect both dip and strike occur often and are particularly noticeable in this shale.

2.3 Foundation Description.

2.3.1 Spillway. Bedrock for the spillway consists of both shale and sandstone. A line drawn diagonally from the downstream right corner through the upstream left corner of the apron would be close to the contact of the sandstone and shale at design grade. Upstream and to the right of this line, the foundations for the weir, right nonoverflow and half of the apron will be predominantly sandstone. Downstream and to the left of the diagonal line most of the left nonoverflow and the other half of the apron is underlain by shale. The sandstone is hard, fine-grained quartzitic and contains many shaly zones, shale beds and shale partings. Joints are abundant and are usually open and water stained. Partial or complete drill-water loss was experienced in most of the holes drilled in the sandstone. A few of the joints contain calcite or quartz fillings. The silty to sandy shale which forms part of the spillway foundation has been metamorphosed to a hard brittle slickensided rock. All bedding planes are smooth or slickensided and sometimes have a polished appearance. Where unweathered, the shale is not readily susceptible to air slaking. The rocks in the spillway area strike in a direction of N 67° E, or parallel to the diagonal line across the apron, mentioned above, and dip to the southeast (downstream and from right to left) at an angle of 22° (40 feet per hundred). The top of firm rock, as shown on the log sections on Plate 6, was found to vary from 13 to 33 feet in depth.

2.3.2 Outlet Works. The shale and sandstone in the outlet works area are similar to the shale and sandstone described for the spillway. The shale which was excavated in the approach and outlet channels of the outlet works is the same member which forms the foundation for the left nonoverflow section of the spillway. The sandstone overlying the shale, which is apparently continuous almost from portal to portal of the tunnel, is more massive with fewer and less prominent shale beds than the stratigraphically lower spillway sandstone. Lenticular beds of both shale and sandstone occur near the contact of the two members, as illustrated by the logs of Holes Nos. 38 and 39. As shown on Plate 13, dips were into the tunnel from both ends. Based on nearest outcrops, dip components parallel to the axis of the tunnel amount to 19° at the downstream end and 11 at the upstream end.

2.3.3 Embankment. The flood plain is very narrow, the valley is steep-sided and the main stream flows on a rock bottom. The overburden along the axis is quite thin, attaining a maximum thickness of only 8.8 feet as determined in Core Hole 33 located near the base of the right abutment. The overburden consists primarily of low plasticity silts (ML) with some basal deposits of gravelly clays. A geological profile along the axis of dam showing the sparse overburden cover is shown on Plate 3. Bedrock along the cutoff trench for the dam is sandstone containing minor amounts of shale lenses. Most of the shale is found in the left abutment. Both shale and sandstone, like the rock in general at the Gillham site, are slightly metamorphosed and are both hard competent rocks. Some of the sandstone is intensely jointed with open joints sometimes only inches apart. The upper portion of Hole No. 37, located on the right abutment, penetrated several feet of such rock. Firm rock varies from the top of rock under the river to depths of nearly 20 feet below the top of rock in the abutments. The presence of many closely spaced open joints in the upper rock usually accounts for the deep top of firm rock in the abutments.

2.4 Investigations. Twenty core holes were drilled and nine test pits were excavated by backhoe at the embankment. Twenty-three core holes were drilled along the outlet works abutment and forty-nine core holes were drilled in the spillway area. Results of foundation borings are contained on Plates 3 through 5.

3. FOUNDATION TREATMENT

3.1 Design Considerations.

3.1.1 Embankment. Bedrock along the cutoff trench for the dam consists predominantly of sandstone and lesser amounts of shale. The rock is hard and slightly metamorphosed but the sandstone contains many open and tight water stained joints. Top of the firm rock was near top of rock under the river, as shown on Plate 3. Consequently, a cutoff trench to firm rock or to a depth of no greater than 10 feet below top of rock was selected from about station 23+20A on the right abutment to about station 38+80A on the left abutment. The trench design had a 25 foot bottom width below elevation 550 and a 15 foot bottom width between elevations 550 and 569 at the ends of the cutoff trench. Where the foundation rock contained many open joints resulting in depths to firm rock in excess of 10 feet below top of rock, a layer of grout or concrete was used to cover the bottom of the trench to protect the core from piping. The same treatment was to be used for open joints in the bottom of the trench where the trench is excavated to firm rock. Because of the extensive jointing which occurs above the firm rock line, the downstream slope of the trench was covered with gunite. A grout curtain was designed in the bottom of the cutoff trench from end to end of the dam below the flood control pool level. The grout curtain for the embankment would overlap and center between two rings of curtain grout for the outlet works tunnel.

(a) Underseepage Probabilities. Underseepage was not considered to be a major problem due to the limited thickness of soil cover at the site; however, to insure a strong foundation which is essential for a dam embankment over 150 feet in height, the foundation under the entire embankment was stripped to rock. Excavation of rock was confined to a 25 foot wide cutoff trench along the centerline. The cutoff trench was backfilled with impervious clay soils.

(b) Settlement Conditions. Appreciable magnitudes of settlement were not anticipated, particularly in view of the absence of any thick compressible clay horizons. Most of the anticipated minor settlement would occur during the construction period, at least to the extent that overbuilding of the embankment was not considered warranted.

3.1.2 Spillway. Vertical or near vertical faces against which concrete sections were placed, were either line drilled or presplit. The shale faces were protected with either a bituminous sealing compound or pneumatically applied concrete. The sandstone required no protective coating. Rock bolts were installed to prevent undercutting of the rock face at the right end of the weir section with wire mesh to prevent rock from falling to the lower level. Rock slopes in sandstone were as steep as 4:V on 1:H or steeper except on the approach channel, where slopes of 1:V on 1:H were selected to conform to the dip. Slopes in shale were also cut to 1:V on 1:H. The design included rock anchors installed to a depth of 15 feet in rock beneath the weir section and to a depth of 10 feet beneath the stilling basin slab.

A grout curtain was provided from the gallery beneath the spillway and a series of drain holes were drilled from and vented into the gallery, (Plate 31). Drain holes extended 10 ft. into rock beneath the spillway slab. A cutoff trench about 10 feet into rock and 15 feet wide extended from each end of the nonoverflow monoliths to the ends of the embankment. A grout curtain was provided over the length of the cutoff trench.

3.1.3 Outlet Works. In the outlet works area, the sandstones were cut to steep slopes, 4:V on 1:H or steeper, with shales cut to 1:V on 1:H slopes. Vertical or near vertical faces, against which concrete was placed, were formed by line drilling or presplitting. Shale faces exposed longer than a few days were covered by a bituminous sealing compound on the steep slopes and with protective concrete on the horizontal or near horizontal faces. Rock bolts and wire mesh were used both in the tunnel and on the steep slopes of the open cuts. Steel ribs were installed at the tunnel portals and wherever necessary for tunnel support. Seepage, although expected in the tunnel, was not considered to create any major problem.

To insure a solid contact between the concrete liner and the rock, contact grouting in the crown of the tunnel was required, and consolidation grouting was performed to fill the fissures and voids in the rock, (Plate 48). Two rings of curtain grouting were placed 10 feet apart near the center of the tunnel at the axis of the dam. The ring curtain grouting was to overlap the embankment grout curtain centered between the grout rings (Plate 48).

3.2 Embankment Design.

3.2.1 Main Embankment.

(a) General. A revised typical section for the embankment is shown on Plates 14 and 16. The zoning revisions were made primarily to provide for greater use of quarry-run rock as well as more efficient use of the other available borrow materials.

(b) Impervious Material. The core zone was not changed from initial design. It was constructed of clays and plastic silts (CL, CH, and ML) having a liquid limit exceeding 20 and a percent of fines of 60 or more. The fill was allowed to contain scattered stones with a maximum size of 3 inches.

(c) Random Fill. Zones of random materials were added both upstream and downstream of the core zone through the revisions. The width of the downstream zone was increased approximately 11 feet to use more of the available random material. These zones, which are essentially the same as the impervious zones of the initial design, consist of materials having a percent of fines of 40 or more. Maximum size stones within the random fill material did not exceed 6 inches.

(d) Filter A Material. A design change was made during construction which consisted of the 3 foot zone of fine filter (filter A) being moved downstream 11 feet as a result of increasing the width of the random fill. Filter A consisted of concrete sand.

(e) Transition Material. Design changes during construction in the downstream transition zone consisted of eliminating the horizontal portion and including a small zone of filter C material to transition the filter A zone to the rockfill A zone. The gradation of filter C material is as follows:

Sieve size	Percent by Weight passing
6 inch	100
4 inch	90-100
2 inch	65-85
1 inch	40-70
3/8 inch	15-35
No. 4	0-10

The upstream transition material remained essentially the same except that the material was placed between the random fill and random rockfill zones. Widths of the transition zones varied between 8 and 12 feet, as shown on Plates 14 and 16, to use as much as possible of the by-product obtained from the production of rockfill A.

(f) Rockfill. The upstream and downstream rockfill shells were rezoned, as shown on Plate 14, to use more rockfill B (random) and less rockfill A (processed). This was due primarily to a better quality of stone for the rockfill B zones. Specifications for these materials are as follows:

(1) Rockfill B. Fresh or weathered quarry-run sandstone with a maximum size of 200 pounds and not more than 15 percent by weight of friable (highly weathered) sandstone and shale. The material was placed in 18 inch lifts and compacted with four passes of a 10 ton vibratory or 50 ton pneumatic roller.

(2) Rockfill A. Fresh, processed sandstone with a maximum size of 250 pounds, except for the outer 5 feet of the upstream shell which may contain 350 pound maximum size stone. All rockfill A materials were processed to remove shale and other materials smaller than 3 inches. After processing the finished product did not contain more than 5 percent by weight of shale or material smaller than 3 inches.

(g) Stability. Revised stability analyses for the embankment design are presented on Plates 21 through 26. Minimum safety factors obtained for these analyses of the revised embankment are shown in Table 1.

TABLE 1

EMBANKMENT SAFETY FACTORS

Condition	Shear strength used	Type of analysis	Safety factors Computed	Safety factors Required
End of construction				
Upstream	Q	Arc	1.55	1.3
	Q	Wedge	2.17	1.3
Downstream	Q	Arc	1.77	1.3
	Q	Wedge	2.32	1.3
Sudden drawdown	R	Arc	1.17	1.0
Steady seepage	R	Arc	1.49	1.5
	S	Arc	1.50	1.5
	R+S/2	Wedge	2.05	1.5

The results of the stability analyses indicate the revised embankment section is satisfactory.

3.2.2 Cofferdams.

(a) General. The cofferdams for construction of the outlet works were inconsequential (see Design Memorandum No. 10, Outlet Works). However, the closure cofferdam (Cofferdam No. 2) (Plate 14) constituted a significant portion of the embankment. To use the cofferdam to the maximum possible extent as a part of the permanent embankment, the position and configuration presented in General Design Memorandum No. 4 was revised by Supplement No. 1 to Design Memorandum No. 9. The cofferdam (see typical section on Plate 14) was over 100 feet in height, and except for temporary layers on the upstream slope, consists of portions of all zones of the embankment below El. 525. The upstream cofferdam slope was set equal to that of the permanent embankment (1:V on 2:H). The downstream slope was also 1:V on 2:H although not extending to the ultimate width of the downstream rockfill section. In deference to the potential for overtopping and related loss of embankment materials, a cable anchorage system was provided to protect the rockfill sections. The downstream cofferdam (No. 3) was minor in height (8 feet maximum) and posed no unique problem.

(b) Wire Mesh System. It was anticipated that the central cofferdam No. 2 would be overtopped during construction. If overtopping and subsequent breaching occurred, the Government would not only be faced with the cost and time delays in making repairs, but with significant increases in quantity and cost of fill materials. The price increases would result as follows:

(1) The upstream borrow areas contain only minor quantities of reserve fill materials, not enough to replace a major washout.

(2) An alternate borrow source about 7 miles downstream on private lands would involve truck haul over county and access roads, making the cost of these materials about 5 times more than those obtained from the upstream borrow areas.

(3) The second alternative of replacing a portion of the soil materials with rockfill would increase the cost by a factor of 4.

To engage large rock masses in resistance to the erosion or drag forces that would develop during overtopping a wire mesh reinforcement system was selected (Plate 9). The mesh would function similar to a Gabion system where small rock is tied together to act as a single unit. The specifications required that the crest profile be maintained level to prevent concentration of flow and that the contractor be prepared to cover the downstream lip with reinforcement whenever overtopping was considered imminent.

The District investigated the possibility of leaving a portion of the cofferdam low to serve as a spillway but found that the increased depth of flow and the interference with construction made it more desirable to allow overflow over the full length. The control elevation was located at the lip of the downstream slope. The reinforced cofferdam slope was contained within the finished embankment, and therefore it was not considered necessary to remove or overlay the reinforcement.

With the above alternatives in mind, the addition of an anchored wire mesh system was felt warranted in the revision of 1970 (Plate 18). This system offered greater resistance to the potential overflow forces than individual stones. Subsequently, in 1973 the final modification to the slope protection for Cofferdam No. 2 was adopted after loss of embankment material during overtopping (Plate 17).

3.2.3 Impervious Materials. Borrow was limited to those areas located in the flood plain upstream from the dam axis, all within the fee purchase limits. Estimates based on explorations indicated sufficient materials were available to meet the basic requirements for the impervious portions of the embankment. The soils for the select impervious zone were in short supply. As shown in Table 1, Appendix A, these soils were confined to borrow area C, although alternate sources were available at increased costs.

3.2.4 Rockfill. Seven individual ledge outcrops of quartzitic sandstone were explored in the search for an adequate and suitable quarry site for the source of rockfill material. An estimated 5,500,000 cubic yards of usable rockfill material could be excavated from the portions of the ledge where the overburden was not excessive. A typical sample of the quartzitic sandstone in the quarry was tested and the results of the physical tests are listed in Table 2, Appendix A.

3.2.5 Transition and Filter Materials. The transition material was produced from the waste at the rockfill quarry. The sand filter layer was obtained from off-site sources.

3.2.6 Spillway.

(a) Spillway Wraparounds. A section of the spillway wraparound is shown on Plate 2. The section was designed as a rockfill shell with an impervious earth core. The impervious zone was placed against the concrete with a minimum horizontal dimension of 5 feet near the crest and a 1:V on 1:H slope to the base. A layer of coarse material was placed between the impervious core and the rock shells to form a transition. The maximum exterior slope was 1:V on 2.5:H. All soil was removed from the wraparound foundation areas. Spur dikes, placed on either side of the spillway entrance, were constructed of spillway rock excavation with exterior slopes of 1:V on 2.5:H.

(b) Embankment. The embankment design was the same as the main dam embankment as presented.

(c) Saddle Dikes. A typical section of the saddle dikes is shown on Plate 29. The dikes attained a maximum height of only 12 feet and were an all earth section rather than rockfill as used in the main dam. This was a more economical design because of the near proximity of earth materials in comparison to the distant location of rock either from the required excavation or the quarry site. The slopes were flattened to 1 on 6 to eliminate the need for riprap and backing. An inspection trench was excavated to rock along the centerline of Dike No. 2.

3.2.7 Outlet Works.

(a) Tunnel. The 10-foot-diameter concrete lined tunnel began 48 feet downstream from the downstream face of the gate tower. The tunnel was designed in accordance with EM 1110-2-2901, using case IV. Supporting steel ribs were used during excavation where needed for roof support and remain as reinforcing for the tunnel lining. Sections of the tunnel are shown on Plates 46, 47 and 48.

(b) Stilling Basin. The stilling basin extends downstream from the outlet portal and provides for a reduction in velocity of the tunnel discharge. Immediately downstream from the outlet portal, the structure consists of a U-shaped channel with vertical walls which flare out to the full width of the stilling basin slab. The walls and slab were anchored to the adjoining rock by grouting anchor bars into drilled holes. Drains were provided behind the walls and beneath the slab to reduce the hydrostatic pressure. The unbalanced hydrostatic uplift for the stilling basin slab was reduced by 50 percent as recommended by EM 1110-2-2400. Details of the stilling basin are shown on Plate 49.

3.2.8 Compaction Control. Compaction control criteria given in the specifications for the embankment zones are presented in Tables 3.2.8(a) and 3.2.8(b). The Test Embankment Report prepared by Tulsa District is contained in Appendix C.

TABLE 3.2.8(a)

COMPACTION CONTROL CRITERIA FOR EMBANKMENT MATERIALS

Item	Type of Material				
	Select Impervious	Impervious	Filters	Rockfill: Weathered	Rockfill: Fresh
Roller, types and number of passes					
Tamping	8	8	Prohibited	Prohibited	Prohibited
Pneumatic, 50-ton,	Prohibited	6	3	4	Prohibited
Vibratory, 10-ton,	Prohibited	Prohibited	(1)3	Prohibited	4
Tractor, crawler	Prohibited	Prohibited	3	Prohibited	Prohibited
Lift thickness, loose inches	8	(2)8	12	12	24
Target densities (3)	95	95	70	(4)	(4)

(1) A lightweight steel wheel roller was allowed for the sand filter.
(2) Twelve inches, if pneumatic roller was used.
(3) In terms of standard AASHO method except for filter criteria which refers to relative density.
(4) Methods of control were identical to those used for test fill. Periodic check tests (by one cubic yard test pits) were also used to determine density and gradations of in-place rockfill.

TABLE 3.2.8(B)

MOISTURE CONTROL CRITERIA FOR EMBANKMENT MATERIALS

Zone	Range in Moisture, Percent from wo
Select Impervious	0 to +4
Impervious	-3 to +3
Filter	None (i)

(1) Filter materials were moistened in accordance with the latest revisions to the guide specification. Sluicing was prohibited.

4. FOUNDATION CONSTRUCTION PROCEDURES

4.1 Embankment Foundation.

4.1.1 Overburden Excavation. Hixson and Lehenbaur, Inc., of Topeka, Kansas, the prime contractor, began clearing the embankment area on 19 September 1972. Stripping of overburden material began on 19 October 1972. Suitable materail was placed in cofferdam No. 1. Unsuitable material was used to construct haul roads and a river crossing or wasted in designated areas. This excavation was accomplished with the use of three D-8 Caterpillar dozers, two 621 Caterpillar scrapers, one 988 Caterpillar loader, and two 769 Caterpillar tail dumps.

4.1.2 Embankment. A cutoff trench was presplit and excavated to a depth of 10 feet from station 23+15 to Station 27+80 and station 31+50 because top of rock was firm. Total planned width of the cutoff trench was 25 feet. However, the trench was widened and deepened in the loose fractured rock encountered between station 26+00 and station 27+50. Where the foundation rock had many open joints in the bottom of the trench, the joints were filled with concrete or grout to protect the core from piping. The downstream slope of the trench from station 25+90 to station 27+30 and station 32+20 to station 39+75 was covered with a 2" coat of gunite because of extensive jointing. A grout curtain was formed along the dam axis between station 23+20 and station 39+40. A single line of holes, varying in depth from 20 feet to 100 feet, was drilled and grouted. The holes were angled 25 into the left and right abutments with a transition between station 32+40 and station 33+30. The grout curtain for the embankment overlaps and centers between two rings of curtain grout provided for the outlet works tunnel.

4.1.3 Treatment. Surfaces to receive materials other than impervious or random fill did not require special foundation preparation. Blade cleaning of these surfaces as part of the common excavation was adequate. Treatment of the foundation rock within the limits of the random fill and impervious zones consisted of cleaning all rock surfaces, open joints, and fractures by washing, air jet, or both. Open joints in the rock surface were sealed and capped with grout or mortar. Small fractures and open joints in the rock were cleaned to the depths directed and covered with cement mortar or thick grout. Large fractures and large open joints in the bottom of the cutoff trench were cleaned out to a depth directed by the Contracting Officer and filled with concrete. Exposed final rock surfaces which were subject to deterioration were immediately covered and protected by a layer of the overlying embankment material.

4.1.4 Slide Area. During the night of 16 January 1973 a slide of approximately 15,000 cubic yards of insitu material occurred in a portion of the right abutment. The slide encompassed an area some 200 feet square from station 23+90 to station 25+90 immediately downstream of the cutoff trench. This block of material moved down dip into the excavated cutoff trench with an apparent pivot point approximately 200 feet downstream of centerline at approximately station 25+80. (See photographs 1 through 5). A thin clay shale seam, immediately above the floor of the cutoff trench and lubricated by percolating groundwater, provided the slippage plane. Removal of the slide material began on 6 February 1974 and was completed 13 February 1974. Approximately 12,000 cubic yards were removed and incorporated in the embankment rockfill section.

4.1.5 Overexcavation. Considerable widening of the cutoff trench was required on the lower portion of the right abutment. Because of the extremely fractured, loose condition of the rock encountered from station 25+00 to station 27+50 the cutoff trench was widened as well as deepened. See paragraph 4.2.2. After consultation with District Office personnel, it was decided to remove the upstream and downstream faces until suitable rock was reached. In any case, the widening was not carried beyond the limits of the toe of the random fill. This widening varied from 20 feet to 70 feet upstream and 40 feet to 60 feet downstream of the planned centerline. Total plan width of the cutoff trench was 25 feet. Some minor overexcavation also occurred on the left abutment along the upstream face where the dip slope was intercepted by the cutoff trench causing some sloughing along the dip slope.

4.2 Grout Curtain.

4.2.1 Drilling and Grouting. The grout curtain was formed by drilling and grouting a single line of holes, varying in depth from 20 to 100 feet, along the dam axis from station 23+20 to station 39+40. Generally the holes were angled 25 into the left and right abutments with the transition occurring from station 32+40 to station 33+30. The primary holes (20-foot centers) were drilled to full depth, washed, pressure tested, and grouted in accordance with the "stop grouting, split spacing" method. The secondary holes were located midway between the primary holes. This split spacing continued until the holes were determined to be tight. The drilling and grouting was done by the Judy Company, a subcontractor. The Judy Company began operations on 15 January 1973 and completed their work on 28 April 1974; however, work was not continuous within these dates (See paragraph 4.2.2). Equipment used is listed in Table 4.2.1. Photograph 18 shows grouting operations.

TABLE 4.2.1

GROUTING EQUIPMENT

Item	Quantity
Chicago - Pneumatic 600 Air Compressor	1
Gardner - Denver 900 Air Compressor	1
Chicago - Pneumatic AirTrac Drill	1
Gardner - Denver AirTrac Drill	1
Gardner - Denver Grout Pump	1
Wilden Air Pump, 2 inch	1
Deming Air Pump, 3 inch	1
Truck, 2-ton	1
Pickup, 1/2-ton	2
Parts Trailer	1
Assorted Valves, Gages, Hoses	

4.2.2 Difficulties Encountered. The slide mentioned previously occurred during the night of 16 January 1973 forcing Judy Company to delay operations approximately one month. (See photograph 2.) Operations resumed 19 February with drilling grout holes at station 30+40. The valley bottom and left abutment was grouted with no difficulty. On 28 May 1973 grouting operation began on the right abutment and problems were experienced. It was originally planned to form the grout curtain from station 26+50 to station 27+00 by drilling a fanshaped pattern of holes from station 26+20 to station 26+40. Since pressure testing resulted in considerable water losses above 15 feet, grout was introduced by gravity flow. After approximately 30 minutes grout began to leak from a spring 40 feet upstream at approximately station 27+80. After pumping 874 sacks of cement with no apparent tightening of the foundation, the grouting operation was suspended temporarily on 12 June 1973.

The cutoff trench was deepened approximately 8 feet by drilling and blasting. This exposed a large fracture partially filled with grout at station 26+45 to 26+65. (See photographs 28, 29 and 30.) The decision was made to fill this area with concrete. Since routine grouting methods were not successful in this area, a Field Order was issued directing the Contractor to drill and sand grout a line of holes on 1-foot centers 16 feet deep, 10 feet upstream of centerline from station 26+23 to 26+68, to form a plug between the grout curtain along centerline and the open joints and fractures upstream. Work on the Field Order began 9 July 1973 and was completed 18 July 1973. On 24 July 1973 the grout curtain was completed to the base of the slide area.

Judy Company resumed operations on 18 February 1974. Another problem area was encountered from station 25+20 to station 25+70. See photographs 19, 20, 23, 24, 25, 39, 40 and 42.) Two large open fractures extended across the cutoff trench at station 25+30 and station 25+50. The larger of these, a V-shaped opening 7 - 10 feet deep and 2 - 3 feet wide at the top at station 25+30 also extended upstream under the abutment in excess of 65 feet. The accessible portions were filled with high slump concrete and an attempt was made to grout the centerline. Since grout was bypassing the concrete plug, 13 holes 18 feet deep were drilled upstream of the cutoff trench to intersect the fracture. A thick 3:1 sand grout was mixed in ready-mix trucks and a total of 26 cubic yards were poured into these holes (see photographs 21 and 22), with an additional 3 cubic yards poured into the grout holes on centerline at station 25+30 (1/2 cubic yard) and station 25+50 (2-1/2 cubic yards). When the centerline had been grouted to refusal, additional rows of holes 10 feet and 5 feet upstream were drilled and grouted to insure that the area was tight (See photograph 41). As a final check, two holes were drilled 15 and 35 feet upstream of the face to intersect the fracture. These holes were tight. Grouting was completed 28 April 1974.

4.2.3 Grouting Summary. Table 4.2.3 summarizes the embankment foundation grouting totals for the job. A breakdown of data included in Table 4.2.3 shown in Table 4.2.4 by stations and zones.

4.2.4 Conclusions. The sandstone and shale comprising the foundation of the embankment were firm and fresh and considered adequate to support the imposed load. The entire length of the grout curtain for the full depth was pressure grouted to refusal. Subsequent high water retained by the cofferdam produced no unusual increase in seepage downstream. These facts indicate that the grout curtain is performing the design function.

TABLE 4.2.3

GROUTING SUMMARY

Drilling (ft)	Pres. Test	Sacks of Cement	Pumptime	Connections
19,085	71 hr. 57 min.	7,347	386 hr. 33 min.	523

TABLE 4.2.4

GROUT TAKE SUMMARY

Stations	Zone 1 (below 40')	Zone 2 (15'-40')	Zone 3 (2'-15')	Total	Total Drilled	Sacks Cement Per Foot
			Primary Hole (20' Centers)			
23+20-27+90	74.2	17.9	280.7	372.8	1905	0.20
28+00-31+95	301.2	142.8	29.8	473.8	2000	0.24
32+00-39+40	10.5	541.6	494.5	1046.6	2950	0.35
			Secondary Holes (10' Centers)			
23+20-27+90	47.6	25.6	890.1	963.3	2100	0.46
28+00-31+95	122.3	30.2	28.4	180.9	2000	0.09
32+00-39+40	9.4	93.4	227.0	329.8	2900	0.11
			Tertiary Holes (5' Centers)			
23+20-27+90	3.7	66.8	806.5	877.0	880	1.00
28+00-31+95	0.0	6.6	27.9	34.5	848	0.04
32+00-39+40	-	325.1	70.4	395.5	600	0.66
			Quaternary Holes (2.5' Centers)			
23+20-27+90	-	47.5	676.4	723.9	665	1.09
28+00-31+95	-	-	-	-	-	-
32+00-39+40	-	0.0	86.0	86.0	208	0.41
			Quinterary Holes (1.25' Centers)			
23+20-27+90	-	7.2	168.8	176.0	320	0.55
28+00-31+95	-	-	-	-	-	-
32+00-39+40	-	0.0	0.0	0.0	80	0.00

		Holes (0.63' Centers)				
23+20-27+90	-	-	0.0	0.0	80	0.00
28+00-31+95	-	-	-	-	-	-
32+00-39+40	-	-	-	-	-	-

Grout Take By Zones

23+20-27+90	125.5	165.0	2822.5	3113.0
28+00-31+95	423.5	179.6	86.1	689.2
32+00-39+40	19.9	960.1	877.9	1857.9
	568.9	1304.7	3786.5	5660.1

Grout pumped before deepening

C.O.T. - Station 26+20 to 26+40	874.0 sacks
Grout and sand pumped 6' US from Station 26+33 to 26+68	365.4 sacks
Backfill and allowable waste	447.5 sacks
TOTAL	7347.0 sacks

4.3 Spillway. Vertical or near vertical faces against which concrete was placed were either line drilled or presplit. All shale faces were covered with double spray applications of bituminous protection. Brecciated shale was removed along the faulted sandstone-shale contact. The foundation beneath the spillway structure and the cut-off trench for the wrap-around embankments were grouted. Grout holes in the cut-off trench were drilled vertically on 10 feet centers. Grout holes in the spillway gallery were inclined upstream at 28 measured from vertical as shown on Plate 12. Grout holes were formed through the concrete by black steel pipe installed on 5 feet centers and extending from the foundation to the gallery gutter. The grouting profile is also shown on Plate 12. Foundation drains were installed on the downstream side of the gallery to depths of 35 feet and 50 feet. Drain holes were spaced 10 feet on centers and were inclined downstream at an angle of 20° measured from vertical.

4.4 Outlet Works. Excavation faces were formed by presplitting and close line drilling. Shale faces were covered with a bituminous sealirg compound on the steep slopes and with protective concrete on horizontal or near horizontal faces. Rock bolts and wire fabric were placed throughout the tunnel except in the transition section and areas containing steel rib supports. Steel ribs were also installed from station 5+60T to station 6+23T and station 10+78T to station 10+96T on 3 feet centers for a total of 29 supports. A concrete tunnel liner was installed and contact grouting was performed to provide a solid contact between the concrete liner and the rock. After the contact grouting was completed the voids in the rock surrounding the bore were grouted. Two rings of curtain grout were placed 10 feet apart near the center of the tunnel at the axis of the dam. The ring curtain grouting overlaps the embankment grout curtain which was centered between the grout rings.

5. CONSTRUCTION CONTROL DATA.

5.1 Construction Sequence. The general construction sequence for the main dam embankment was as follows:

(a) Construct cofferdams No. 1 and No. 3.

(b) Construct log boom.

(c) Grout and excavate foundation.

(d) Construct cofferdam No. 2.

(e) Complete embankment and associated work.

5.2 Embankment Fill.

5.2.1 Typical Section. A typical section of the main embankment is shown on Plate 16. The maximum height of the main embankment is 162 feet. The section is essentially symmetrical with a central impervious core, random fill zones flanked by transition material, and outer shell zones of rockfill. A three foot thick inclined filter A is connected to the downstream rockfill zone by a transition zone and coarse filter C material.

5.2.2 Borrow. Earth materials for the impervious and random zones were obtained from borrow areas A1, A2, B and C. Additional borrow materials were obtained from borrow area E for construction of embankment and dikes in the spillway area. Additional borrow for construction of the main embankment was obtained from Hunter's Creek downstream of the dam site and from Striklin Creek located upstream along the left abutment. The soils ranged from fine-grained to granular types. The predominant soil was a very lean sandy clay, low in plasticity, essentially borderline between a clayey silt and a clayey sand (CL, sandy but also includes ML-CL, sandy and some SC.) Gravel and shale were present in the lower elevations of most areas. A summary of borrow soil test results is contained in Appendix A.

5.2.3 Filter. Materials for the inclined filter (filter A), and filter C were obtained from the southern portion of borrow area B. These materials, primarily gravels, were processed to the extent necessary to meet the specifications.

5.2.4 Rockfill and Transition Materials. The quarry site is located upstream from the dam site and about 5,000 feet east of the river. Prior to construction of the embankments, a test fill was built to develop engineering data for materials (Rockfill A and Rockfill B) from the quarry site. Details of the test embankment are shown in Appendix C, in the Test Embankment Report, Gillham Dam, prepared by Tulsa District. Material for the Rockfill "B" zones consisted of Class I or II rock with a maximum size of 200 pounds with less than 15 percent by weight either passing the No. 4 screen, or consisting of friable (highly weathered) stone, or shale material, in any load. Material for Rockfill "A" consisted of Class I rock with a maximum size of 250 pounds with the exception that stone placed in the outer five feet of the upstream Rockfill "A" zone had a maximum size of 350 pounds and an intermediate size that 40% of the stone weighed more than 50 pounds. Processing was required to remove shale and materials smaller than three-inch size and this by-product was used for the transition material.

During construction, three in-place density determinations were made for the Rockfill "A" and Rockfill "B" materials. The results of the tests are shown below.

ROCKFILL A

Test No.	Station	Offset	Elevation	Dry Density #/ft3	Minus 3" %
1	31+07	147'D.S.	439.1	113.2	28
2	29+85	138'D.S.	463.1	120.8	25
3	31+00	80'U.S.	540.0	116.3	16.9

ROCKFILL B

Test No.	Station	Offset	Elevation	Dry Density #/ft3	Minus 3" %
1	31+08	220'U.S.	443.9	138.6	15.8
2	28+00	130'U.S.	474.0	136.6	13.0
3	33+00	140'D.S.	500.0	136.4	11.0

The results of the Rockfill density tests indicated that the in-place densities were some 5% to 15% greater than the design unit weights for Rockfill B and A respectively.

5.3 Embankment Stability.

5.3.1 Tests and Design Values. Shear tests were conducted on representative soils in the remolded state for impervious and random soils. Test results from test fills at Gillham Dam and Dierks Dam were used in determining the unit weights. Results of tests of borrow materials are contained in Appendix D, results of tests of borrow soil are contained in Appendix E, and results of tests of rock, rockfill, riprap and concrete aggregate are contained in Appendix F. The Adopted Design Strengths are shown in the following tabulation:

			Adopted Design Strengths					
	Saturated	Submerged	Q		R		S	
	Unit Weight	Unit Weight	Ø	C	Ø	C	Ø	C
Materials	(pcf)	(pcf)	(deg)	(tsf)	(deg)	(tsf)	(deg)	(tsf)
Rockfill A (1)	135	72.5	42	0.	42	0.	42	0.
Rockfill B (2)	135	72.5	36	0.	36	0.	36	0.
Transition (3)	125	62.5	33	0.	33	0.	33	0.
Filter (3)	125	62.5	33	0.	33	0.	33	0.
Randomfill	125	62.5	5	0.5	22	0.2	28	0.
Impervious	125	62.5	5	0.5	22	0.2	28	0.

Ø=Angle of internal friction C=Cohesion

(1) Material assumed to have 40 percent voids and a saturated surface dry unit weight of 105 pcf. Angle of internal friction was assumed to be 42°.

(2) Angle of internal friction was assumed to be 36°.

(3) Angle of internal friction was assumed to be 33°.

5.3.2 Construction Control Tests. The following summary contains the results of the construction Record Samples from the impervious and random fill zones as compared to the design strengths of these materials. A graphical presentation of the shear test data is presented on Plate 20.

CONSTRUCTION CONTROL SHEAR TESTS

		Test Strength		Design Strength	
No.	Type	Ø	C	Ø	C
Tests	Test	Degrees	t.s.f.	Degrees	t.s.f.
8	Q	10	0.9	5	0.5
8	R	24	0.6	22	0.2
8	S	34	0.0	24	0.0

The results of the construction control in place density, water contend, and classification tests for the random and impervious materials are summarized below:

CONSTRUCTION CONTROL FIELD TESTS

Material			Compaction %	Wf-Wo %	Percent Fines %	LL %
		Average	99.3	-0.10	66.4	27.3
Impervious	(1)	High	103.8	+2.2	95.1	44.3
		Low	95.2	-2.5	52.0	18.0
		Average	99.4	+ .04	60.2	25.1
Random	(2)	High	106.6	+2.9	92.2	35.0
		Low	94.5	-2.6	39.5	17.0

(1) Results of 82 tests.
(2) Results of 163 tests.

The results of the field tests indicate the field moisture content of the random and impervious materials was essentially within the limits specified with only a small number of the tests having a moisture content slightly drier than specified.

5.3.3 Stability. The embankment was analyzed for stability under conditions of end of construction, sudden drawdown, partial pool and steady seepage. The trial slip surfaces were analyzed using the procedures set forth in Engineering Manual, EM 1110-2-1902, 1 April 1970. The following tabulations indicate the safety factors obtained for the most critical slip surface based on the design shear strengths and the "as built" shear strength of the random and impervious materials. The lowest safety factors obtained for each condition analyzed are shown in the following tabulation:

Condition	Shear Strength Used	Type of Analysis	Safety Factors As Built	Design	Required
End of Construction (US)	Q	Arc	1.66	1.55	1.3
End of Construction (US)	Q	Wedge	1.66	2.17	1.3
End of Construction (DS)	Q	Arc	1.86	1.77	1.3
End of Construction (DS)	Q	Wedge	1.86	2.32	1.3
Sudden Drawdown (2)	R	Arc	----	1.17	1.0
Sudden Drawdown (1)	S,R	Arc	1.28	----	1.0
Partial Pool	S,(R+S)/2	Arc	1.52	----	1.5
Steady Seepage (4)	R	Arc	----	1.49	1.5
Steady Seepage (4)	S	Arc	----	1.50	1.5
Steady Seepage (3)	S,(R+S)/2	Arc	1.70	----	1.5
Steady Seepage (4)	(R+S)/2	Wedge	----	2.05	1.5
Steady Seepage (3)	S,(R+S)/2	Wedge	1.56	----	1.5

(See Notes, Next Page)

(1) Analysis for sudden drawdown from maximum pool elevation 581.0 to conservation pool elevation 502.0. The rockfill "A" zone was assumed to be free draining.

(2) Analysis for sudden drawdown from flood control pool elevation 569.0 to conservation pool 502.0. The rockfill "A" zone was assumed to be free draining.

(3) Analysis based on differential head between maximum water surface elevation 581.0 and maximum tailwater elevation 480.0.

(4) Analysis based on differential head between flood control pool elevation 569.0 and maximum tailwater elevation 480.0.

5.3.4 Conclusion. Based on a comparison of test results of the in-place materials with those used in design, the stability of the structure is considered adequate.

5.4 Design Modification.

5.4.1 Right Abutment Slide. A slide containing approximately 15,000 cubic yards of insitu material occurred on the right abutment between sta. 23+90 and sta. 26+25 on 16 January, 1973. The slide resulted from excavation of the cutoff trench and moved along a bedding plane at the base of the cutoff trench. Removal of the slide material eliminated the downstream face of the cutoff trench through this reach and deletion of the 2-inch thick coating of pneumatically placed concrete. Approximately 12000 cubic yards of the slide mass was incorporated into the embankment sections.

5.4.2 Revised Design-Cofferdam No. 2. Cofferdam No. 2 was overtopped five times during the construction period. The first three of these overtoppings resulted in substantial losses of fill materials. The initial cofferdam design is shown on Plate 18 and the revised design on Plate 17. The original design had the timber anchorages on the same horizontal plane as the wire mesh it was holding and therefore when the overlying material was washed away there was no ballast to protect the timber anchorages. Basically, the revised design consisted of two inverted skillet weir sections which provided an anchorage system to hold the wire mesh and cable system in place when the slopes were subjected to overtoppings. An additional feature was the placement of an airfield landing mat, and a wire wrapped rockfill zone at the toe of the weirs to provide a stilling effect to prevent erosion at the toe of the slope and to absorb the shock of the abraded material.

5.4.3 Earth Fill. As identified as a possible consequence during design, the cofferdam overtoppings did in fact result in a loss of earth fill materials of approximately 200,000 cubic yards. As a result, earth borrow materials became a critical supply item and particularly those materials meeting the impervious specification requirements of liquid limit exceeding 20 and percent fines exceeding 60. To relieve this shortage, the random and impervious zones were combined into an earth fill material with a minimum liquid limit of 22 and percent fines of at least 35. The moisture and compaction controls were not changed. This modification resulted in the use of the sandy silty clays in the central core that were previously excluded because of the fines restriction. The material loss also required the use of borrow material from borrow areas A-2, C, Stricklin Creek and Hunters Creek that had not been anticipated.

6. INSTRUMENTATION.

6.1 Surface Control Monuments. To monitor the embankment movements a total of 25 embankment surface control monuments and 8 abutment control monuments have been established. The location of the monuments are shown on Plate 19.

During construction, monitoring procedures consisted generally of surveying all the completed monuments at the time a new line of monuments were installed, hence readings were taken as follows:

Date	Embankment C.L. Elev.	Remarks
17 July 1974	540	Initial Line 4 readings
28 August 1974	560	Initial Line 3 readings
10 September 1974	561	Initial Line 1 reading
27 September 1974	564	
10 December 1974	586	Initial Line 2 readings

6.2 Embankment Movements. The data through 27 September 1974 revealed minimal movements; however, measurements made on 10 December 1974 indicated that horizontal movements of 6 to over 12 inches upstream had occurred between these dates. During the period of 27 September - 10 December 1974, the embankment had been raised about 22 feet in height, requiring 116,000 approximately cubic yards of embankment materials. During this same period a total of 15.21 inches of rain fell on the watershed and the reservoir rose above elevation 475+/- four times with the maximum rise to about elevation 490 occurring on 25 November 1974. The Resident Office advised the District Office of the 10 December 1974 readings and an immediate program of obtaining weekly horizontal and monthly vertical readings was initiated. Since monitoring data showed that subsequent measured changes in monument location were within the error of observation of the survey methods (generally 0.1 to 0.2 feet), significant additional fill movements had apparently not occurred by March 1975. Four temporary monuments were also located upstream of surface monument line 1 on 18 December 1974. These temporary monuments have shown virtually no movement. Some bulging of the upstream slope is apparent in the vicinity of the line of temporary monuments; however, field personnel reported that this is due to a construction slope correction and had been evident prior to July 1974. Temporary iron pins were driven on 18 December 1974 along the dam axis to monitor vertical movements of the maximum dam section. The maximum settlement of 0.15 feet (station 28+04 and 30+01), in the same area where the large horizontal movements occurred, was measured 5 February 1975. Detailed information is contained in the report "Gillham Dam, Report of Embankment Slope Movements," by Tulsa District, dated March 1975.

6.3 Hypothesis of Movements. Several hypotheses as to the reasons for the movements have been drawn, among these are: point crushing and quarry dust removal due to wetting of the embankment and lateral spreading.

6.4 Conclusion. It appears that the construction phase movements at Gillham Dam were not caused by a design or construction deficiency, but are a result of several phenomena that occur naturally in all earth and earth-rock dams to some degree. Lateral spreading and slope material rearrangement were both very likely contributors to the movements that occurred at the Gillham Dam. The lack of sufficient monitoring data to determine the chronology of the movements was significant in that there is no definitive way of directly correlating or excluding physical events such as rainfall and pool rises as potential catalysts.

6.5 Post-Construction Embankment Movements. Periodic inspections through No. 5 in March 1985 reported the embankment to be in excellent condition with no significant changes between inspections in 1979, 1983 and 1985. Maximum measured embankment settlement from initial survey December 1974 through August 1984 has been approximately 1.2 feet (near Sta. 27+00 and Sta. 33+00). This value is less than the overbuild of 2 feet.

Minor seepage only has occurred, and was attributed to discharge from the outlet works (Periodic Inspection Report No. 3).

APPENDIX A

TABLES

TABLE 1

SUMMARY OF BORROW SOILS - CHARACTERISTICS AND ESTIMATED QUANTITIES

Borrow Area	Approximate Location	Estimated Total Quantities (cu.yd.)	Estimated Amounts of Soil (cu. yd.)			Soil Characteristics					
			Percent F (0-40)	Percent F (Impervious) (40-60)	Percent F (Select Impervious (60+)	LL			Percent F		
						Max	Min	Avg	Max	Min	Avg
A North	Right bank, 1 mile upstream	217,000	5,000	60,000	152,000	33	15	24	75	42	59
A South	Right bank, 1 mile upstream	218,000	30,000	129,000	59,000	27	14	22	66	38	51
B	Left bank, 1 mile upstream	420,000	83,000	114,000	223,000	28	17	24	68	36	57
C West	Right bank, 2 miles upstream	70,000	30,000	21,000	19,000	31	8	22	78	33	53
C East	Right bank, 2 miles upstream	120,000	---	5,000	115,000	40	20	29	87	60	73
D(abandoned)											
E West (1)	Right abutment, 1.5 miles downstream	350,000	24,000	116,000	210,000	33	16	23	80	50	63
E East	Right abutment, 1.5 miles downstream	100,000	47,000	0	53,000	33	20	28	86	37	65
F	Right bank, 4 miles downstream	500,000	20,000	185,000	295,000	29	19	24	78	53	67
G	Right bank, 6 miles downstream	2,550,000	535,000	918,000	1,097,000	27	14	19	74	50	56
	Total	4,545,000	774,000	1,548,000	2,223,000						

(1) Portions of borrow area E have been made available for constructing the right abutment access road now under contract.

TABLE 2

PHYSICAL TESTS ON QUARRY SANDSTONE

Test	Test result
Bulk specific gravity, SSD	2.63
Unit weight, p.c.f.	164.1
Absorption, percent	0.8
Soundness, percent loss	
Mg S04, 5 cycles	1.1
Freeze-thaw, 25 cycles	1.1
Abrasion, L.A., percent loss	
A grading	22.1
B grading	15.4

APPENDIX B

PHOTOGRAPHS

GILLHAM DAM AND RESERVOIR
SALINE RIVER, ARKANSAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

APPENDIX B - PHOTOGRAPHS

B-9	Photo No. 17. Foundation under the impervious zone from Sta. 34+40 to 38+65. Note the evidence of presplitting on the U/S (left Hand) face.
B-9	Photo No. 18. Grout pump used on this contract. Mixing tank is being charged. The mixed grout is emptied into the holding tank to be pumped into the grout holes.
B-10	Photo No. 19. Open fracture at Sta. 25+30 as seen looking D/S from centerline.
B-10	Photo No. 20. Open fracture at Sta. 25+30 as seen from centerline loock U/S.
B-11	Photo No. 21. Placing ready-mixed sand grout in holes drilled U/S of C.O.T. to intersect open fracture at Sta. 25+30.
B-11	Photo No. 22. View of grout holes U/S of C.O.T. at Sta. 25+30 as seen looking U/S along strike of the fracture.
B-12	Photo No. 23. D/S portion of the open fracture at Sta. 25+30 prior to backfill with concrete.
B-12	Photo No. 24. Extreme D/S end of the fracture at the D/S toe of the random zone.
B-12	Photo No. 25. Open fracture at STa. 25+30 being backfilled with concrete.
B-13	Photo No. 26. D/S half of the step face at Sta. 25+80(+/-) prior to placing concrete to eliminate the overhang.
B-13	Photo No. 27. U/S half of the step face at Sta. 25+80(+/-) priot to placing concrete to eliminate the overhang.
B-14	Photo No. 28. Large open fracture at Sta. 26+40 to 26+60 partially filled with grout. Openings below and to the left and right of the hard hat extend into the U/S face for an unknown distance.
B-14	Photo No. 29. Large open fracture at Sta. 26+40 to 26+60 partially filled with grout. An opening to the right of the hard hat extends into the D/S face for an unknown distance. In the lower right-hand corner of the photograph a coke can plugs core Hole #37 at elevation 484. Note grout pipes in foreground to be installed in openings prior to backfilling with concrete.
B-15	Photo No. 30. Placing foundation protection concrete at Sta. 26+40 to 26+60 the temporary form was used to aid in running concrete into the opening in the face.
B-15	Photo No. 31. A partially grout-filled fracture at Sta. 26+40. This is the D/S extension of the large fracture at Sta. 26+40 to 26+60. Clean up prior to backfilling with concrete.
B-16	Photo No. 32. This photograph shows the upstream extension of the fracture at Sta. 26+40 to 26+60. Note the concrete (placed 7-2-73) plug in the background.
B-16	Photo No. 33. This view (looking U/S from centerline) of the fracture at Sta. 26+40 to 26+60 shows partial filling with grout at the lower right. This area was backfilled with concrete.

B-17 Photo No. 34. Clean-up prior to application of "Gunite" to D/S face of C.O.T. in left abutment. Note gunite machine mounted on the flatbed trailer.

B-17 Photo No. 35. "Gunite" being applied to D/S face of C.O.T. in left abutment.

B-18 Photo No. 36. "Gunited" D/S face and step face at Sta. 26+90 to 27+10(+/-).

B-18 Photo No. 37. A small protable mixer was used to produce concrete to eliminate small overhangs. This photo was taken at Sta. 26+40(+/-) D/S of centerline.

B-19 Photo No. 38. Sand grout is being mixed by hand to backfill a small crack extending D/S from the previously concreted area at 26+50(+/-). The concrete plug lies under the broom at the right edge of the photo.

B-19 Photo No. 39. A view within the right abutment of the fracture 35' U/S of centerline.

B-19 Photo No. 40. A view within the right abutment of the fracture 55' U/S of centerline.

B-20 Photo No. 41. Foundation along centerline at Sta. 25+50 to 25+70. Note three lines of grout holes in this area.

B-20 Photo No. 42. Foundation from Sta. 25+50 to 25+70 as seen from a point 40 feet D/S. The open fracture at left is to be backfilled with concrete.

Photo. No. 1

Slide area from Sta 23+90 to 24+90. D S of centerline, as viewed from Sta 23+00.

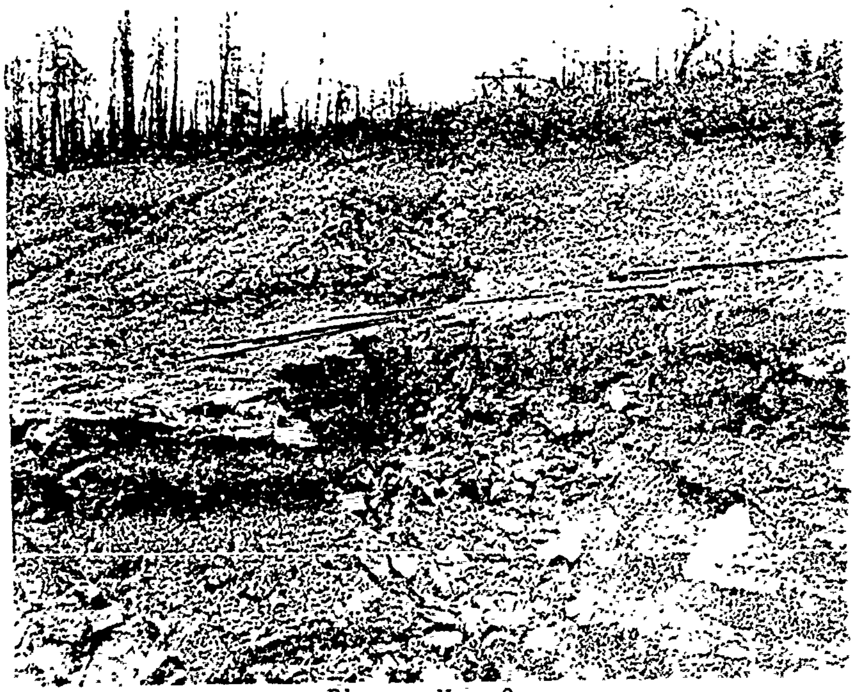


Photo. No 2

View of the upper end of the slide looking D/S from Sta 24+00. Judy Company (grouting subcontractor) is removing water and air lines.

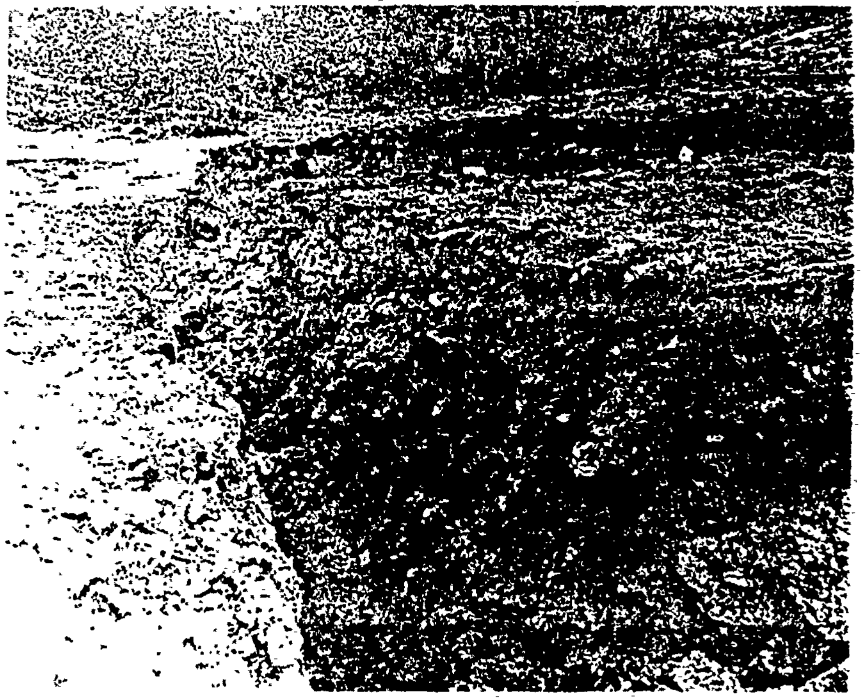


Photo. No. 3

A view of the slide looking down station from Sta 24+00.

Photo. No. 4

The slide area as viewed from Sta 27+00 looking uphill.

Photo. No. 5

The slide area after removal of loose material as viewed from Sta 23+50.

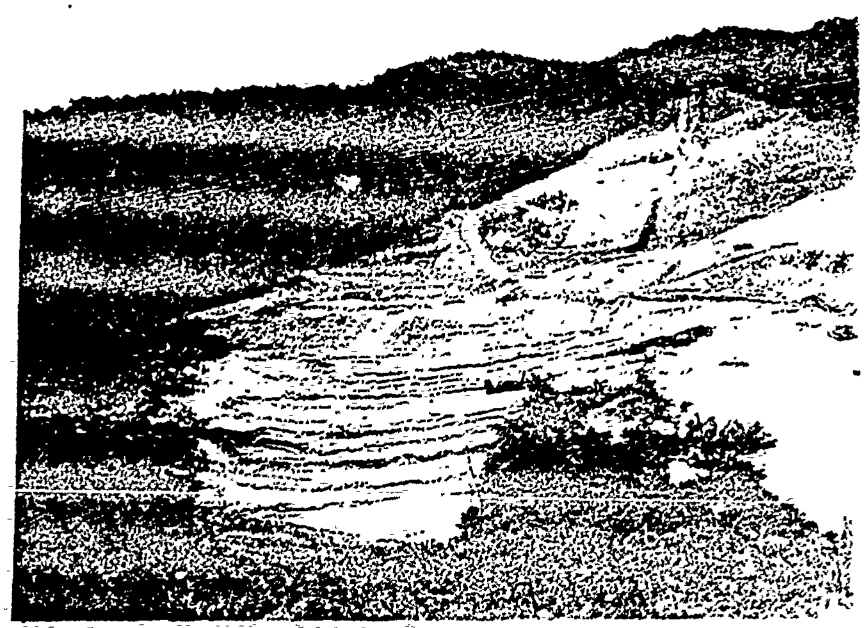


Photo. No. 6

Overall view of the foundation across the valley floor as seen from the outlet tower.

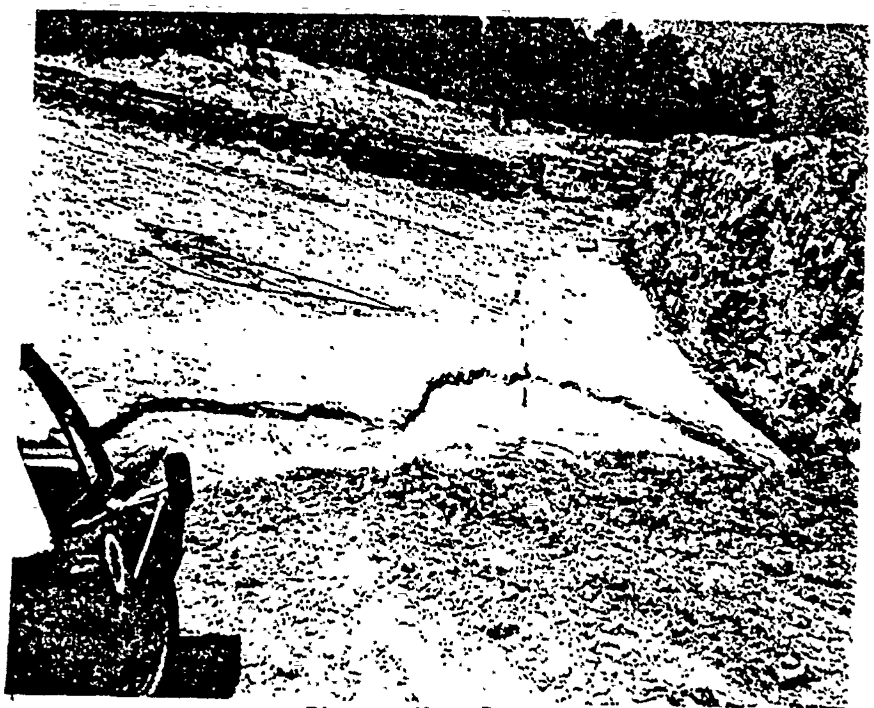


Photo. No. 7

A view of the C.O.T. above Sta 25+20. Note grout pipes along Centerline.

Photo. No. 8

An overall view of the right abutment along centerline. The fill crosses centerline at Sta 26 + 50±.

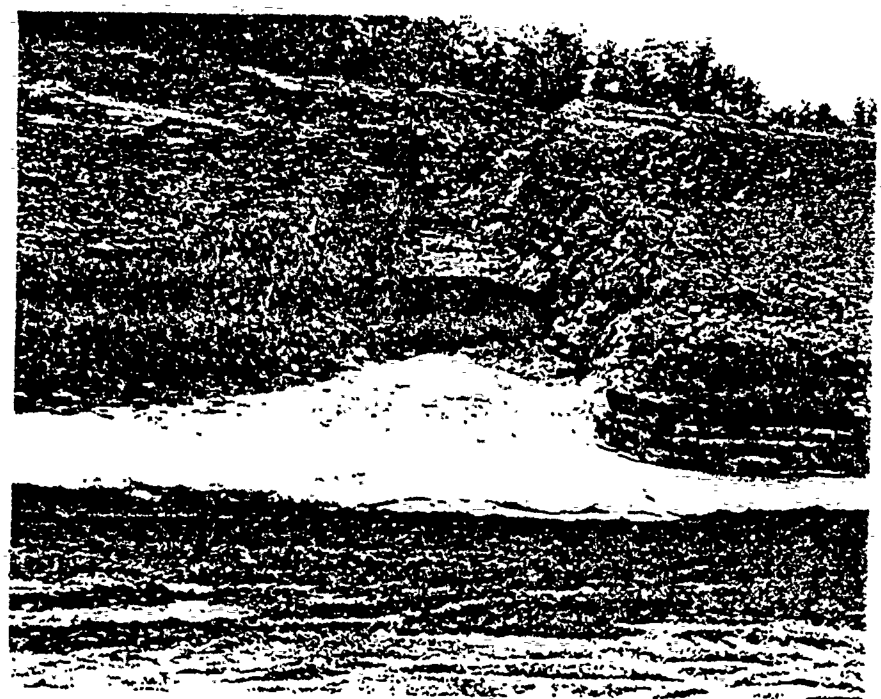


Photo. No. 9

Overall view of the right abutment. The strip of clean foundation occurs at Sta 27+50 to 27+70

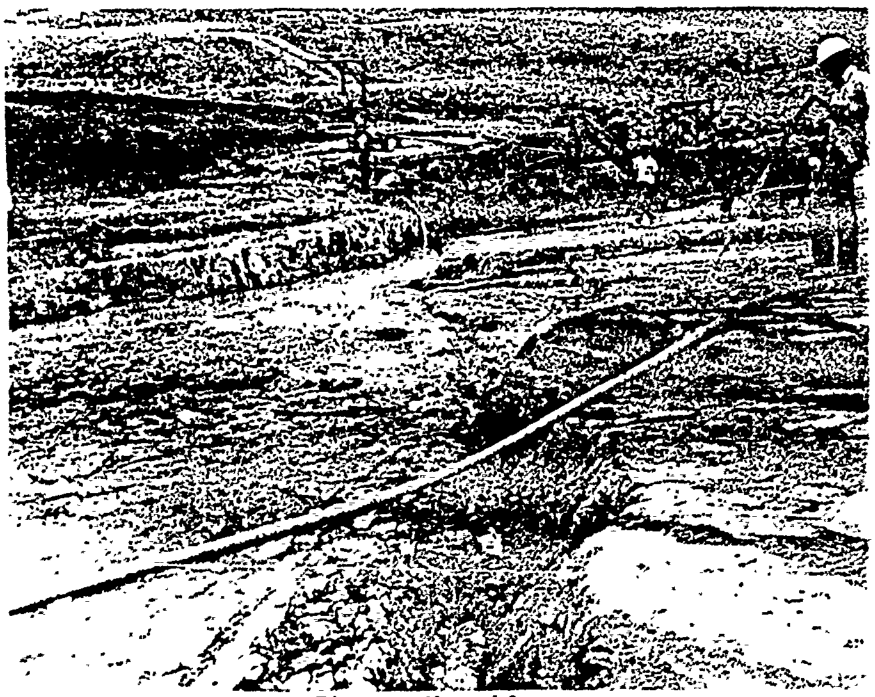


Photo. No. 10

View SW along a minor fault which crosses centerline at approx. Sta 28+40.

Photo. No. 11

Founation under random and impervious zones from Sta 30+50 to 31+80 as seen from upstream toe of the random zone at Sta 31+80.

Photo. No. 12

View of the foundation from Sta 28+60 to 30+50 along centerline as seen from Sta [illegible].

Photo. No. 13

Foundation under impervious zone from Sta 33+30 to 33+75 on left abutment.

Photo. No. 14

Foundation under impervious and U/S random zones at Sta 34+40. Note shale-sandstone contact.

Photo. No. 15

Foundation under impervious and U/S random zones at Sta 35+60 to 36+00. Note extent to which the U/S face has been removed.

Photo. No. 16

Foundation under D/S random zone from Sta 35+50 to 35+70 .

Photo. No. 17

Foundation under the impervious zone from Sta [illegible] to [illegible]-5. Note the evidence of presplitting on the U.S (left hand [illegible]

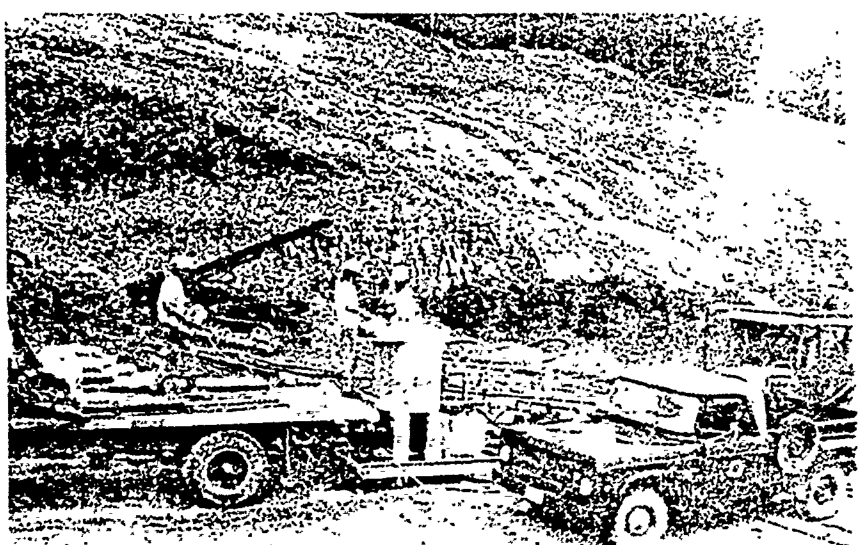


Photo. No. 18

Grout pump used on this contract. Mixing tank is being charged. The mixed grout is emptied into the holding tank [illegible] is pumped into the grout holes.

Photo. No. 19

[illegible] Sta. 25-30 as seen looking D/S from centerline.

Photo. No. 20

[illegible] 25+30 as seen from centerline looking U/S.

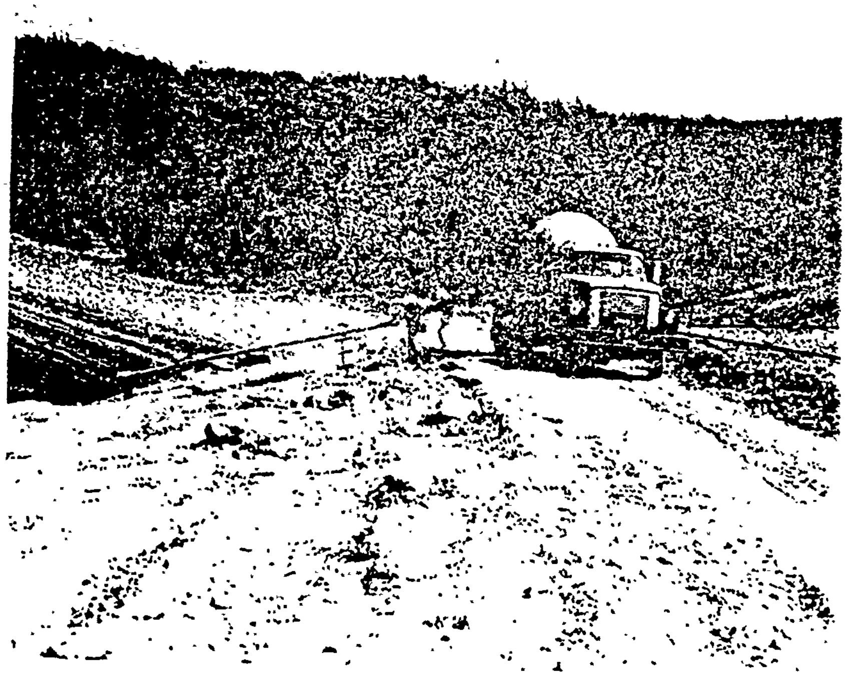


Photo. No. 21

Placing ready-mixed sand grout in holes drilled U/S of C.O.T. to intersect open fracture at Sta [illegible].

Photo. No. 22

View of grout holes U/S of C.O.T. at Sta 2?+30 as seen looking U/S along strike of the fracture.

D/S portion of the open fracture at Sta 25+30 prior to backfill with concrete.

Photo. No. 23

Extreme D/S end of the fracture at the D/S toe of the random zone.

Photo. No. 24

Open fracture at Sta 25+30 being backfilled with concrete.

Photo. No. 25

Photo. No. 26

D/S half of the step face at Sta 25+80± prior to placing concrete to eliminate the overhang.

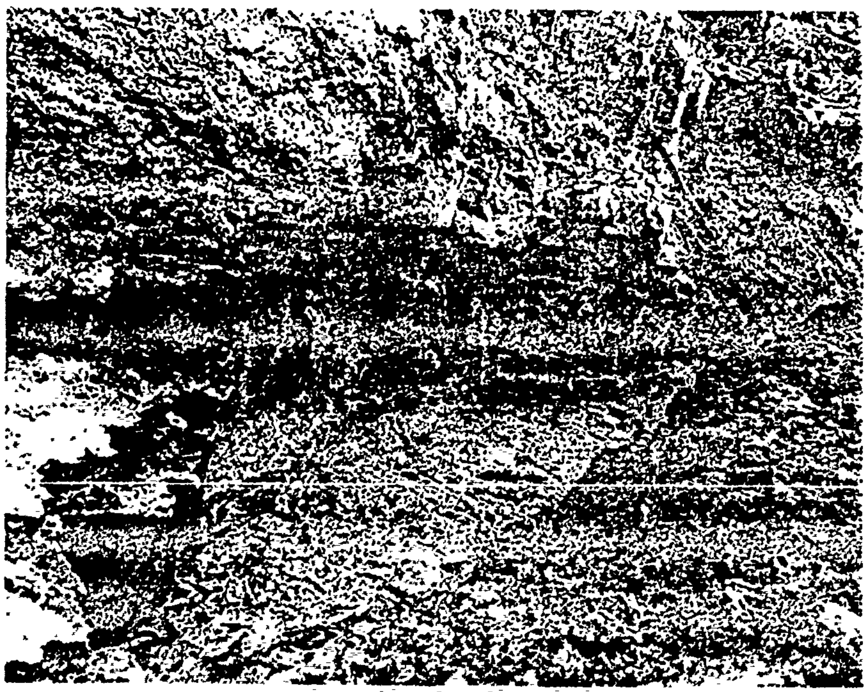


Photo. No. 27

U/S half of the step face at Sta 25+80± prior to placing concrete to eliminate the overhang.

Photo. No. 28

Large open fracture at Sta 26+40 to 26+60 partially filled with grout. Openings below and to the left and right of the hard hat extend into the U/S face for an unknown distance.

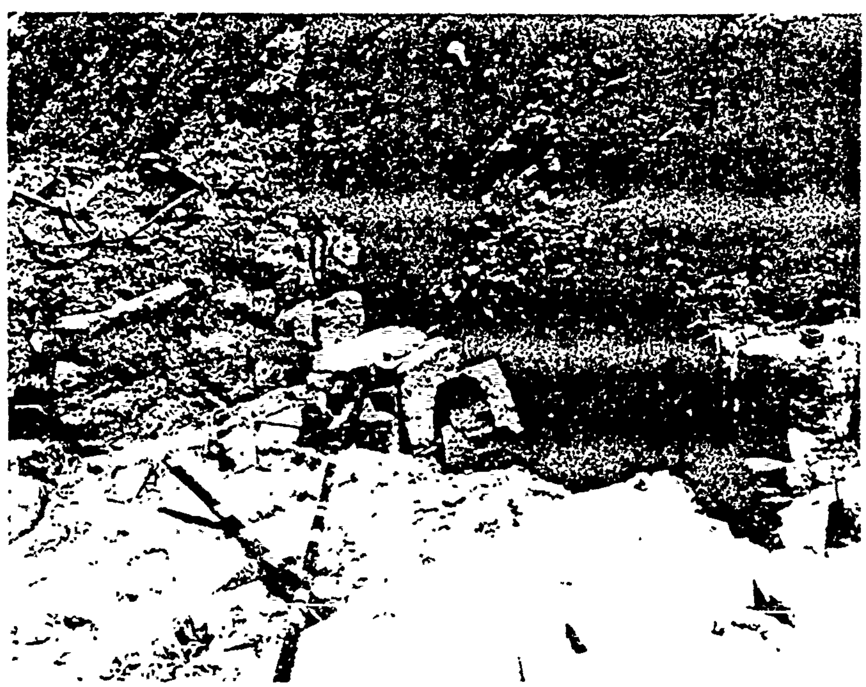


Photo. No. 29

Large open fracture at Sta 26+40 to 26+60 partially filled with grout. An opening to the right of the hard hat extends into the D/S face far an unknown distance. In the lower right-hand corner of the photograph a coke can plugs core Hole #37 at elev. 484. Note grout pipes in foreground to be installed in openings prior to backfilling with concrete.

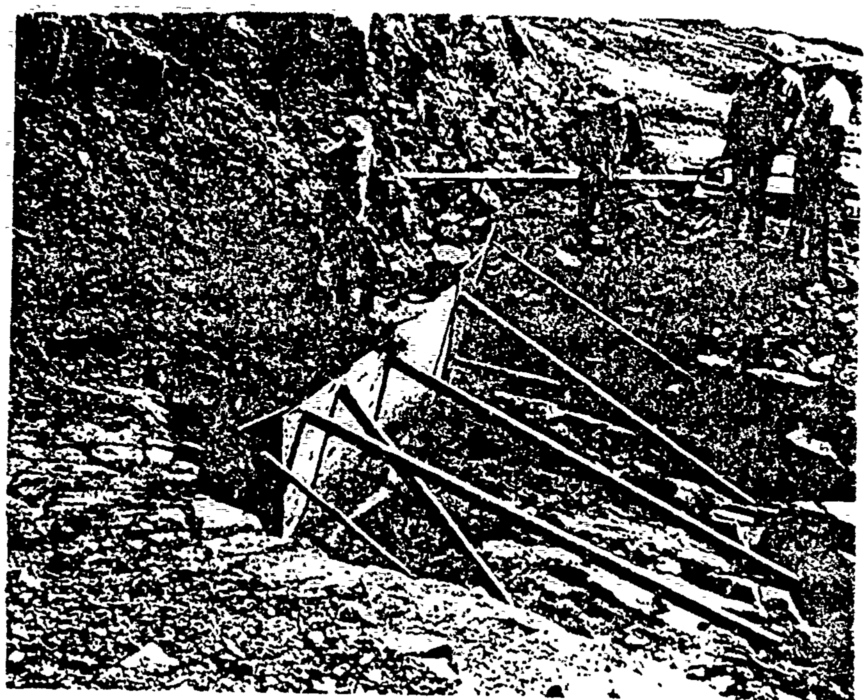


Photo. No. 30

Placing foundation protection concrete at Sta 26+40 to 26+60 the temporary form was used to aid in running concrete into the opening in the face.

Photo. No. 31

A partially grout-filled fracture at Sta 26+40. This is the D/S extension of the large fracture at Sta 26+40 to 26+60. Clean up prior to backfilling with concrete.

Photo. No 32

This photograph shows the upstream extension of the fracture at Sta 26+40 to 26+60. Note the concrete (placed 7-2-73) plug in the background.

Photo. No. 33

This view (looking U/S from centerline) of the fracture at Sta 26+40 to 26+60 shows partial filling with grout at the lower right. This area was backfilled with concrete.

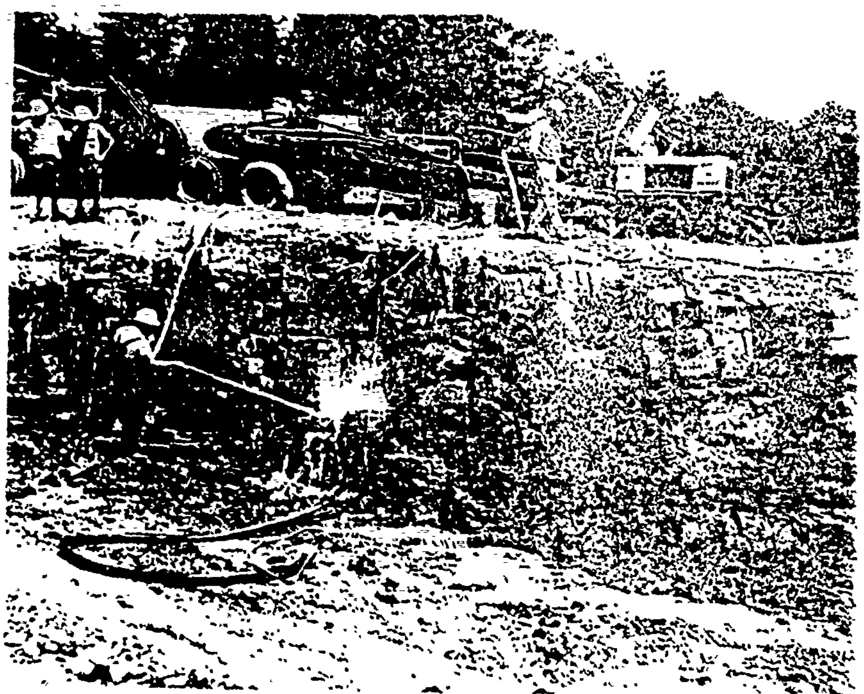


Photo. No 34

Clean-up prior to application of "Gunite [illegible] C.C.T. in left abutment. Note gunite machine mounted [illegible] trailer.

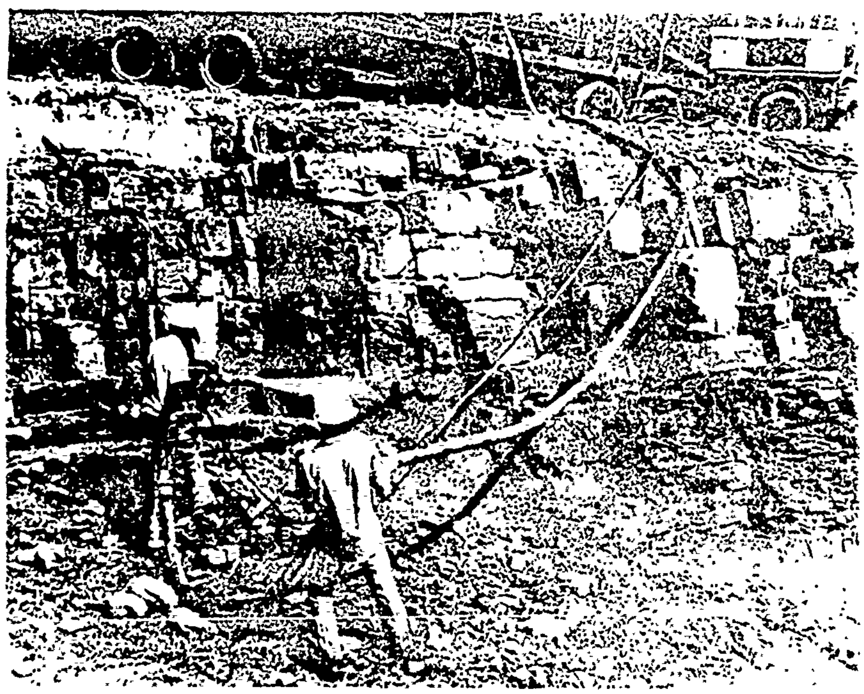


Photo No. 35

"Gunite" being applied to D/S face of C.C.T [illegible] left abutment.

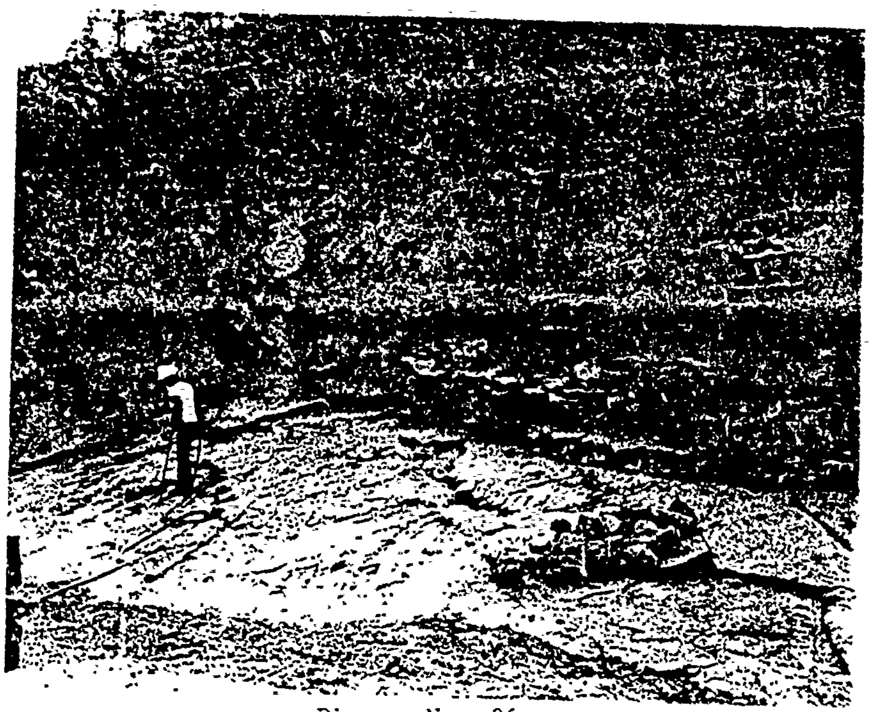


Photo. No. 36

"Gunited" D/S face and step face at Sta 26+90 to 27+10±.

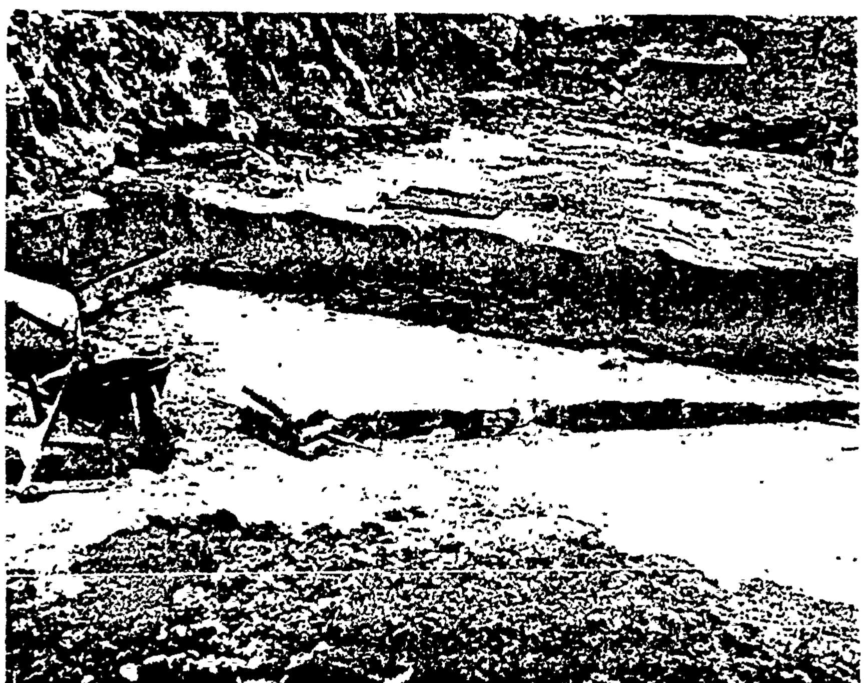


Photo. No. 37

A small portable mixer was used to produce concrete to eliminate small overhangs. This photo was taken at Sta 26 +40± D/S of centerline.

Photo. No. 38

Sand grout is being mixed by hand to back [illegible] tending D/S from the previously concreted area at [illegible] plug lies under the broom at the right edge of [illegible].

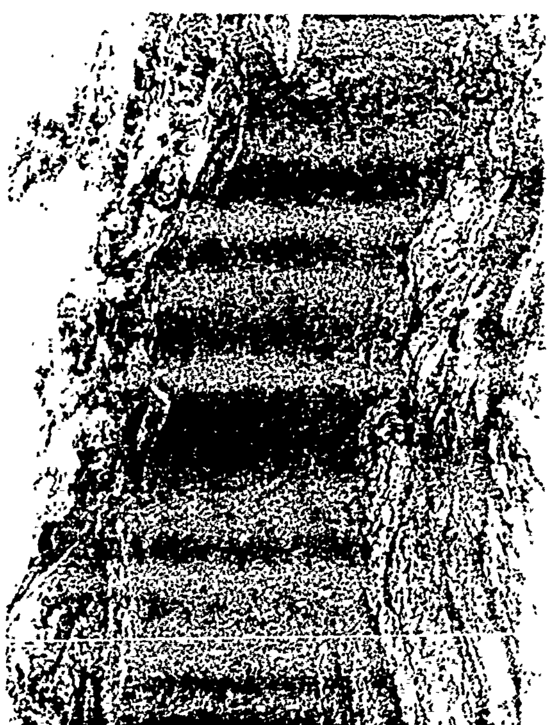


Photo. No. 39

A view within the right abut. of the fracture 35' U/S of centerline.

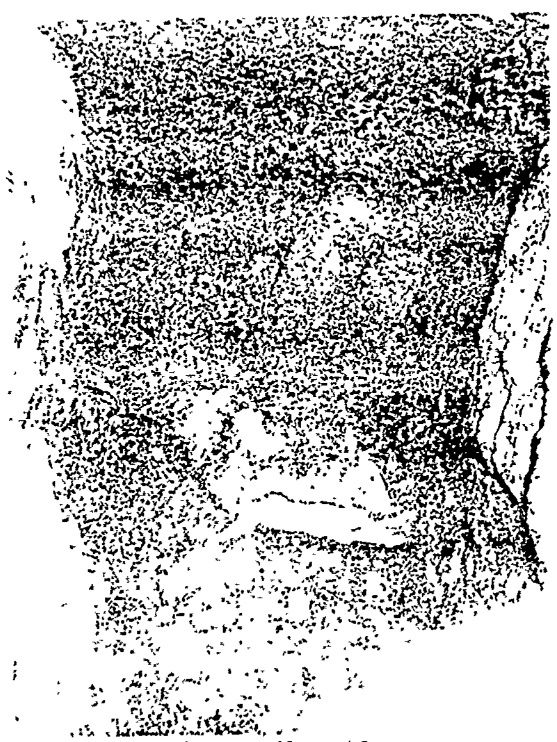


Photo. No. 40

[illegible] the right abut. [illegible] 5[illegible]' U/S of ₵.

Photo. No. 41

Foundation along centerline at Sta. 25+50 to 25+70. Note three lines of grout holes in this area.

Photo. No. 42

Foundation from Sta 25+50 to 25+70 as seen from a point 40 feet D/S. The open fracture at left is to be backfilled with concrete.

APPENDIX C

TEST EMBANKMENT REPORT

TULSA DISTRICT

GILLHAM LAKE
COSSATOT RIVER, ARKANSAS
PERIODIC INSPECTION AND CONTINUING EVALUATION
OF
COMPLETED CIVIL WORKS STRUCTURES

TEST EMBANKMENT REPORT
GILLHAM DAM
TULSA DISTRICT

EXHIBIT A

(APPENDIX II)

TEST EMBANKMENT REPORT
Gillham Dam, Tulsa District

1. General. The purpose of the test fill program was to develop engineering data to be used in preparing plans and specifications for construction of Gillham Dam, Cossatot River, Arkansas. The primary objectives were as follows:

a. Size and gradation of rock-fill material obtainable from the test quarry by varying blast-hole patterns and dimensions and by varying quantity and types of explosives.

b. Compaction of weathered and unweathered rock-fill materials by two different types of rollers (vibratory and pneumatic) using lifts of varying thickness.

c. Breakage and/or degradation of the rock-fill material due to rolling.

The test fill program consisted of one large rock-fill section having seven test panels or lanes (Nos. 1, 2, 3, 3A, 4, 5, and 6). Construction began on 19 June 1964 and was completed on 7 August 1964. Supervision for the project was furnished by the Construction Division, with technical assistance provided by the Engineering Division, Tulsa District. The information contained herein was largely taken from the Test Embankment Report, Gillham Dam, August 1964, and supplementary memoranda furnished by the Tulsa District.

2. Rock Type. The Gillham test fill consisted of two types of rock: weathered sandstone and fresh sandstone. The weathered sandstone, distinguishable by its iron-stain color was hard, dense, quartzitic, fine-grained, and slightly fractured. The weathering consisted of leaching and iron staining concentrated along bedding planes and joints. Blast fractures across the bedding and joint patterns revealed fresh blue-gray

sandstone that was very hard and dense in the center of individual blocks. After blasting, the weathered material had angular particle shapes with about 10 percent of the rock degraded to sand sizes. The fresh or unweathered sandstone was very hard, brittle, and blue-gray in color. It appeared to be mildly metamorphosed with characteristics of quartzite. Scattered hairline fractures up to 1/8 in. wide were noted throughout this material. All of these fractures were tight to well healed with secondary quartz mineralization. Maximum rock sizes used in the test fill are given in table 1.

3. Description of test fill.

a. The test fill was constructed on a leveled area approximately 1600 ft northwest of the test quarry site. Preceding fill placement, about 4 ft of badly weathered shale was removed to expose a firm shale suitable as a foundation. Settlement plates were not installed at the foundation level.

b. The test fill consisted of seven sections or panels, as shown in fig. 1. Each panel was 18 by 50 ft, with 24- to 27-ft-wide (at completion) transition zones between panels. A 1V on 10H slope, serving as a ramp for the construction and compaction equipment, was provided on either side of the embankment. Panel 3A was added on the end of the fill (see fig. 1) after construction was underway to supplement erratic data obtained from panel 3. Rock type, maximum rock size, loose lift thickness, number of lifts, compaction equipment, and number of passes for the seven test panels are given in table 1.

4. Construction.

a. Descriptions of excavation, hauling, and compaction equipment are given in table 2. Rear-dump hauling units, loaded at the test quarry by power shovel, transported the rock directly to the test fill (no stockpiling was allowed) and dumped it at the leading edge of the advancing loose lift. Spreading to the desired loose lift thickness was then accomplished by a D-8 bulldozer. Oversize rock was controlled in the quarry by selective loading. Any oversize material reaching the test embankment was wasted during spreading. The shale content of the material was kept to a minimum by selective removal where possible. However, much interbedding of shale in the sandstone was encountered, and selective removal was not always feesible. Material for the last three lifts of panel 5 and all of panel 6 was passed through a screen to remove the minus 3-in. fraction. All other material was quarry-run.

b. After spreading, each lift was smoothed by one pass of the Bros Model VP-20D 10-ton vibratory roller with the vibratory unit off or by one pass of the Ferguson 50-ton pneumatic roller, depending upon the specified method of compaction for that lift. A 6-ft grid pattern was then laid out on the smoothed lift surface from reference points beyond the test zone limits and subsequently marked with spray paint for easy identification. Initial level readings were taken on these points to establish the loose lift thickness (by comparing the new readings with the final readings on the underlying foundation or compacted lift). Compaction was then begun using either the vibratory or pneumatic roller, both towed by a D-7 bulldozer at speeds of 1 to 1-1/2 mph. The roller was towed over the test panels in alternate directions. Each lift was subjected to a total of eight passes by the roller, with level readings to measure settlement taken after every two passes.

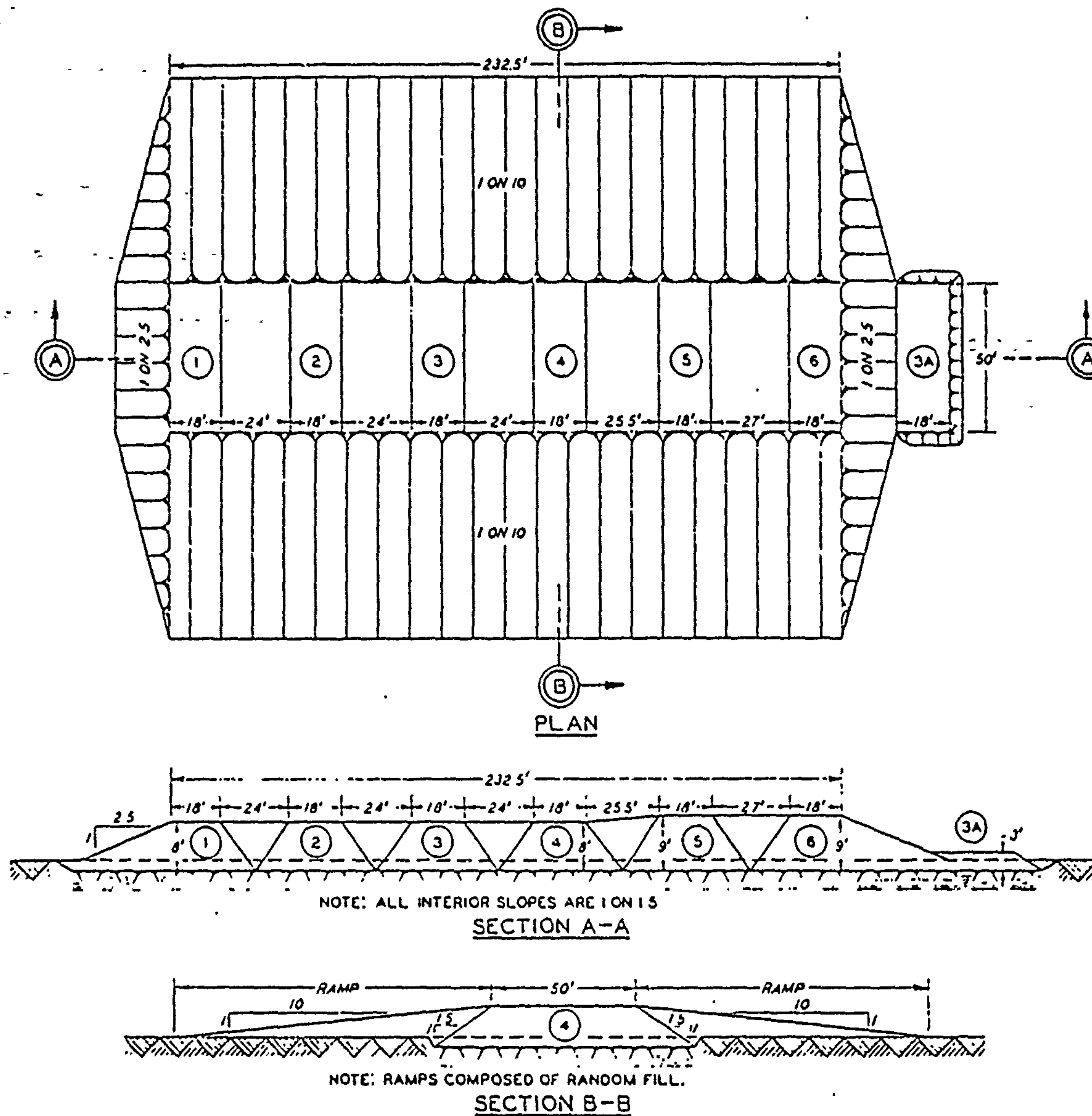


Fig. 1. Gillham test fill layout

Table 1

Construction Details, Gillham Test Fill

Panel	Material	Maximum Rock Size: Weight lb	Maximum Rock Size: Approximate Spherical Diameter* in.	Lift Thickness in.	No. of Lifts	Roller	No. of Passes per Lift
1	Weathered sandstone	80	11-1/2	12	8	Vibratory	8
2	Weathered sandstone	250	17	24	4	Vibratory	8
3	Weathered sandstone	80	11-1/2	12	8	Pneumatic	8
3A	Weathered sandstone	80	11-1/2	12	3	Pneumatic	8
4	Weathered sandstone	250	17	24	4	Pneumatic	8
5	Fresh sandstone**	200	16	18	6	Vibratory	8
6	Fresh sandstone (plus 3-in. only)	300	18	36	3	Vibratory	8

* Based on a specific gravity of 2.61
** Last three lifts had all minus 3-in. material removed

Table 2

Construction Equipment, Gillham Test Fill

Item	Function	Description
Power shovel	Loading	Lima Model 604, 1-1/2-cu yd bucket capacity
Trucks (3)	Hauling	Euclid Model R-15, rear dump (2) Euclid Model 91-FD, rear dump (1)
Tractor	Spreading	Caterpillar D-8 with dozer blade
Tractor	Towing	Caterpillar D-7 with dozer blade
Compactors (2)	Compacting	Bros Model VP-20D, 10-ton, vibratory Ferguson 50-ton pneumatic
Separator	Rock separation	3-in. screen with a wobble-type feeder adjusted to 3-1/4-in. openings and equipped with a belt-loader

5. Tests and measurements.

a. Procedures.

(1) Settlement measurements. Measurements of settlement were the primary means of assessing compactive effort. The grid marked on the surface of each lift to delineate the 24 points of settlement measurement is shown in fig. 2. Initial level readings were taken as discussed above. An average elevation at each point of the grid was obtained by placing the level rod in the center of a 12- by 12- by 1/2 in. steel plate positioned over the grid point.

(2) Density tests. Field density tests were made at the conclusion of construction in the top lifts of panels 3, 3A, 4, 5, and 6. The tests were performed by excavating through the top lift of the panel from within a 6- by 6-ft wooden guide frame placed on the surface. Excavation of the hole was by hand labor, with each of the larger rocks being weighed individually and the smaller particles in groups. An extensive number of tape measurements were made of the finished pit and averaged in an effort to obtain accurate dimensions from which to compute the volume of the hole.

(3) Mechanical analyses. Gradation tests were run on the material excavated from the density test pits in panels 3, 3A, 4, 5, and 6. These analyses established the after-compaction gradations for the top lifts of those five panels. These tests were run by first weighing the total sample, then grouping the rocks into weight ranges, and finally computing the percent of the total sample represented by each weight range. The percent smaller than 3 in. was determined by actually sieving the finer fraction over a 3-in. sieve.

(4) Inspection trench. After completion of all panels of the test fill, an inspection trench was excavated with the D-8 bulldozer. The trench, which had a base width of 14 ft, exposed all lifts of all panels in the embankment. The inspection trench permitted visual assessment of the compaction characteristics of the fill.

b. Results

(1) Settlement measurements. Figures 3 through 7 show the percent settlement of each lift of panels 1, 2, 3, 3A, and 4 which were constructed of quarry-run weathered rock. The average settlement of each of these panels is shown in the respective figures and in the plot of fig. 8. Figures 9 and 10 present the lift settlement data for panels 5 and 6, which were composed of plus 3-in. grizzled fresh rock except for

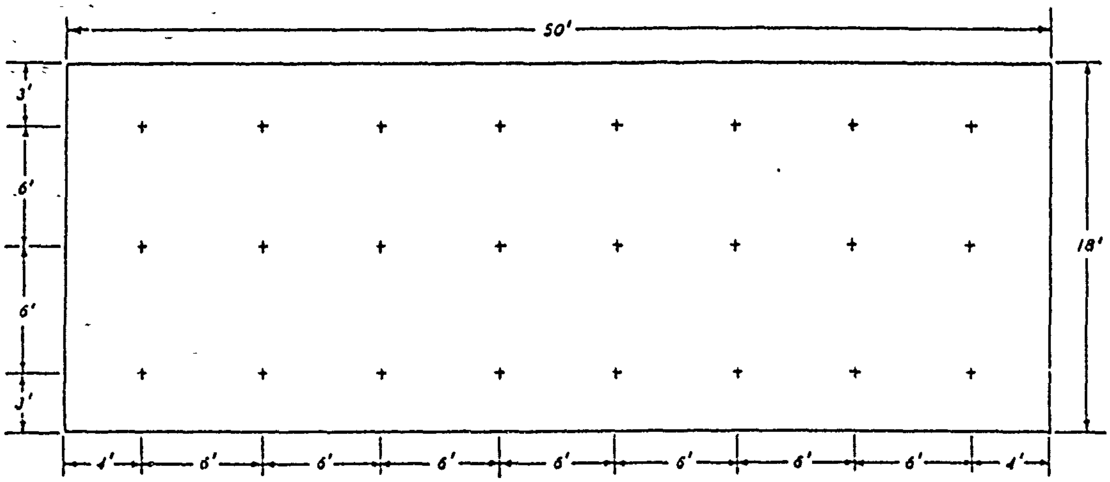


Fig. 2 . Grid layout for level readings, Gillham test fill

lifts 1, 2, and 3 of panel 5, which contained quarry-run fresh rock. The average settlement curves for panels 5 and 6 are also shown in figs. 9 and 10 and plotted separately in fig. 11. The immediately noticeable aspect of the settlement data for the panels constructed of weathered rock (panels 1, 2, 3, 3A, and 4) is the erratic data for several of the lifts. The measurement data for panels compacted with the 50-ton pneumatic roller (3, 3A, and 4) were less consistent than those for panels rolled with the vibratory unit, which might be expected because of the much smoother surface left by the vibratory roller. Further examination reveals that the most variable results were for the 12-in. lifts, regardless of the compaction equipment used. The erratic nature of the settlement data from the 12-in. lifts of panels 1, 3, and 3A probably resulted from erroneous settlement readings or the presence of oversize rock. The data (see fig. 8) indicate, however, that slightly better compaction was obtained with 12-in. lifts of weathered sandstone, and that four passes of either type roller produced most of the settlement achieved by eight passes for this lift thickness. The settlement plots for panels 5 and 6 (fresh sandstone), shown in figs. 9 and 10, are much less variable than those for the other panels. The data in figs. 9 and 10 indicate that the plus 3-in. material has superior compaction characteristics than did the quarry-run material. Comparison of figs. 9 and 10 indicates that the use of 18-in. lifts resulted in more efficient compaction than did the use of 36-in. lifts. Figure 10 shows that the rate of settlement for the 36-in. material had not decreased even after eight passes of the roller, and only 9 percent settlement was attained at that point. By comparison, fig. 9 shows that the same material in 18-in. lifts reached 14 percent settlement after six passes with a marked decrease in the rate of settlement beyond six passes.

(2) Density tests. The results of density tests made in panels 3, 3A, 4, 5, and 6 are summarized in table 3. It was observed during the density tests taken in panels 5 and 6 that the rock-to-rock contact produced by the vibratory roller resulted in an unusually high degree of stability. Considerable pick work was required to loosen the rock for excavation. The vertical sidewalls of the pits had no tendency to slide or slough, but were tight and stable.

(3) Mechanical analyses. After-compaction gradation tests were performed on the material taken from the density tests in panels 3, 3A, 4, 5, and 6. The after-compaction curves for material in panels 3, 3A, and 4 are shown in fig. 12. Since there were no before-compaction data for these panels, no assessment of particle breakage can be made. The gradation curves resulting from tests in panels 5 and 6 are given in figs. 13 and 14, respectively. Also shown in figs. 13 and

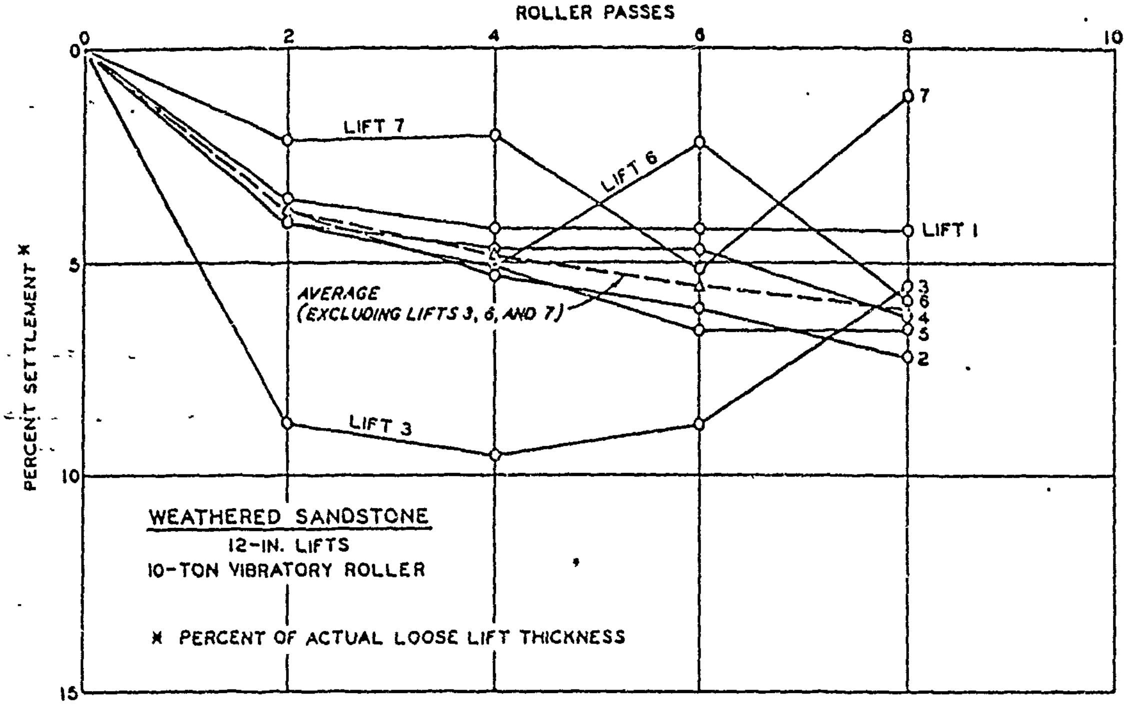


Fig. 3 . Percent settlement vs roller passes, panel 1, Gillham test fill

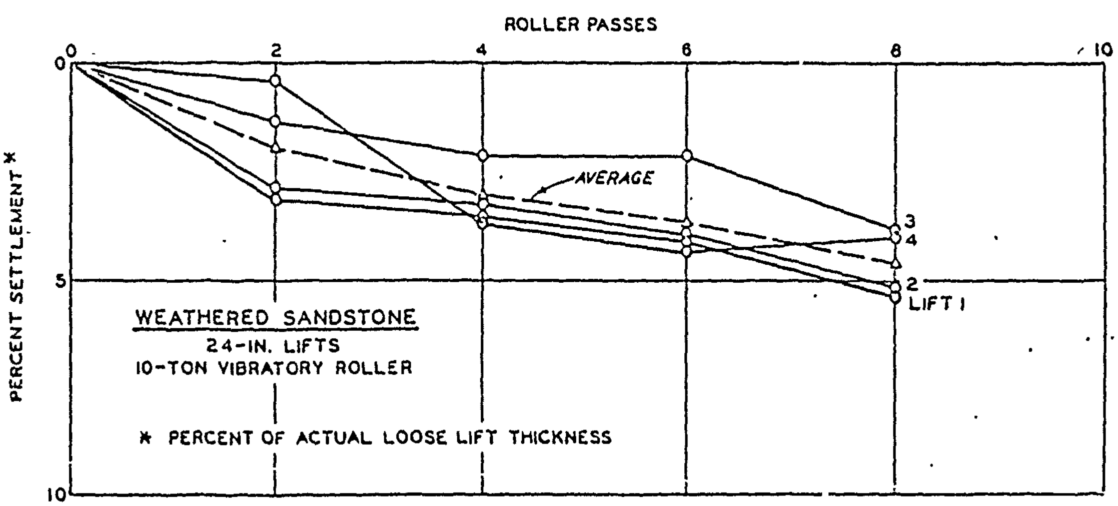


Fig.4 . Percent settlement vs roller passes, panel 2, Gillham test fill

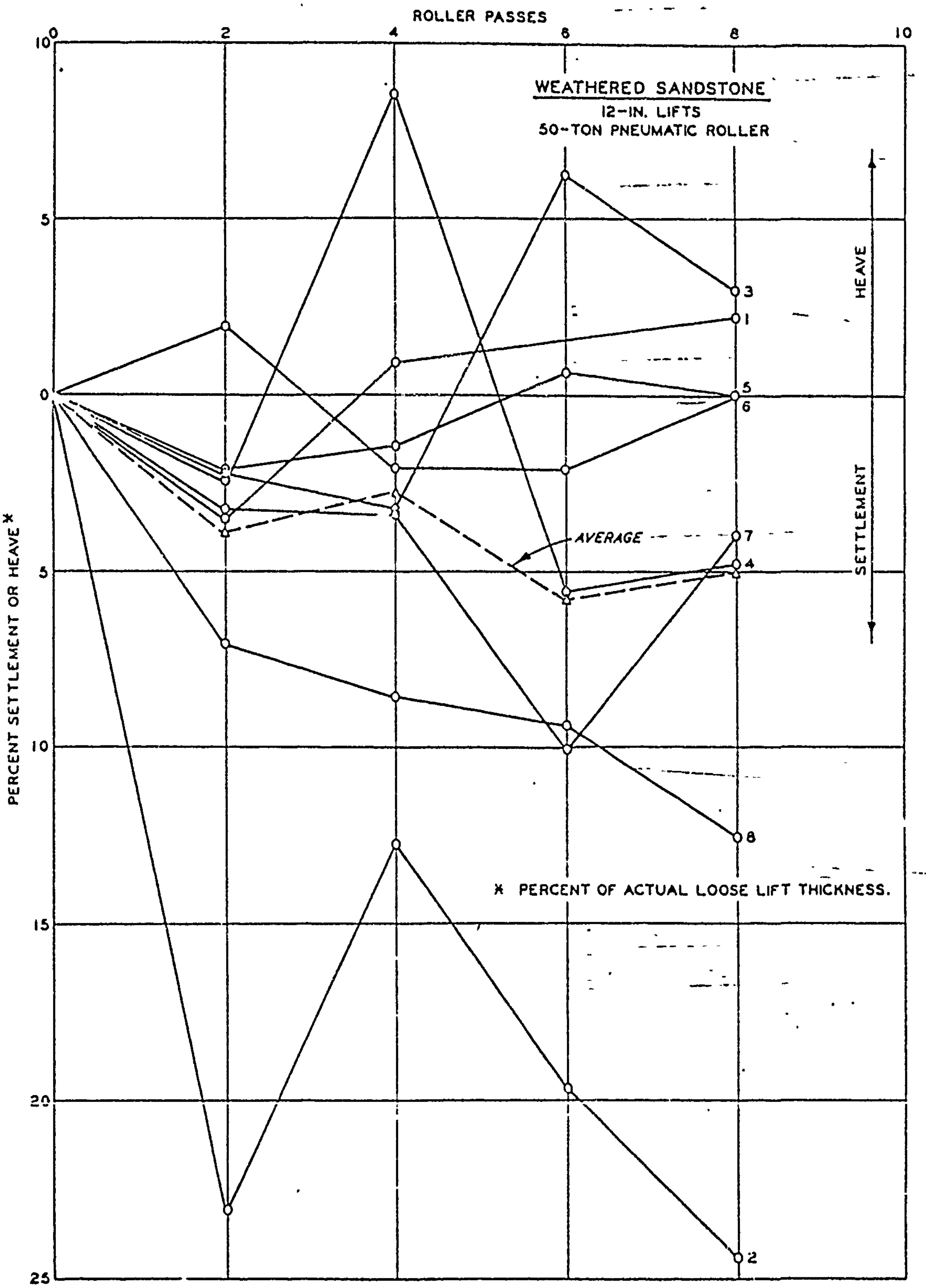


Fig. 5 . Percent settlement vs roller passes, panel 3, Gillham test fill

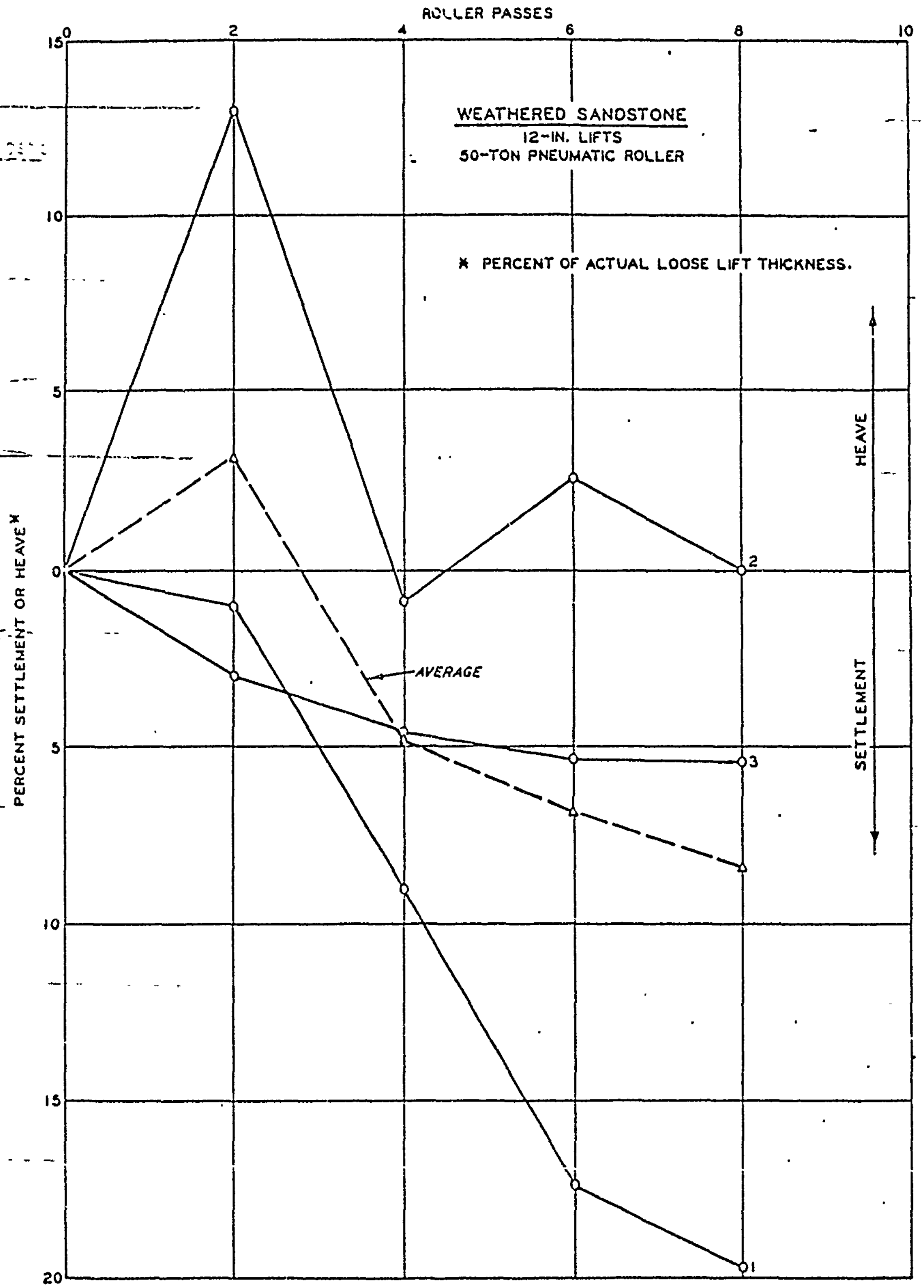


Fig. 6 . Percent settlement vs roller passes, panel 3A, Gillham test fill

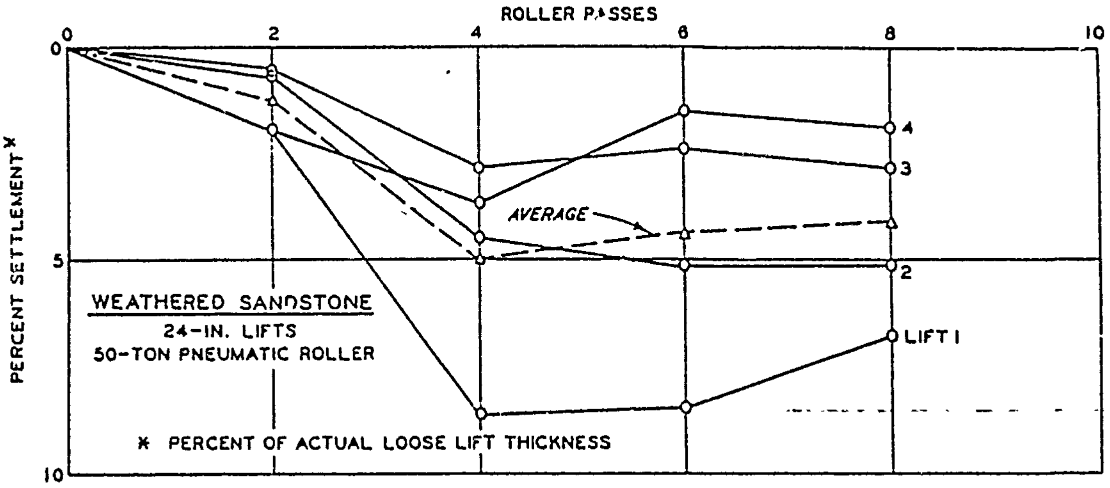


Fig. 7. Percent settlement vs roller passes, panel 4, Gillham test fill

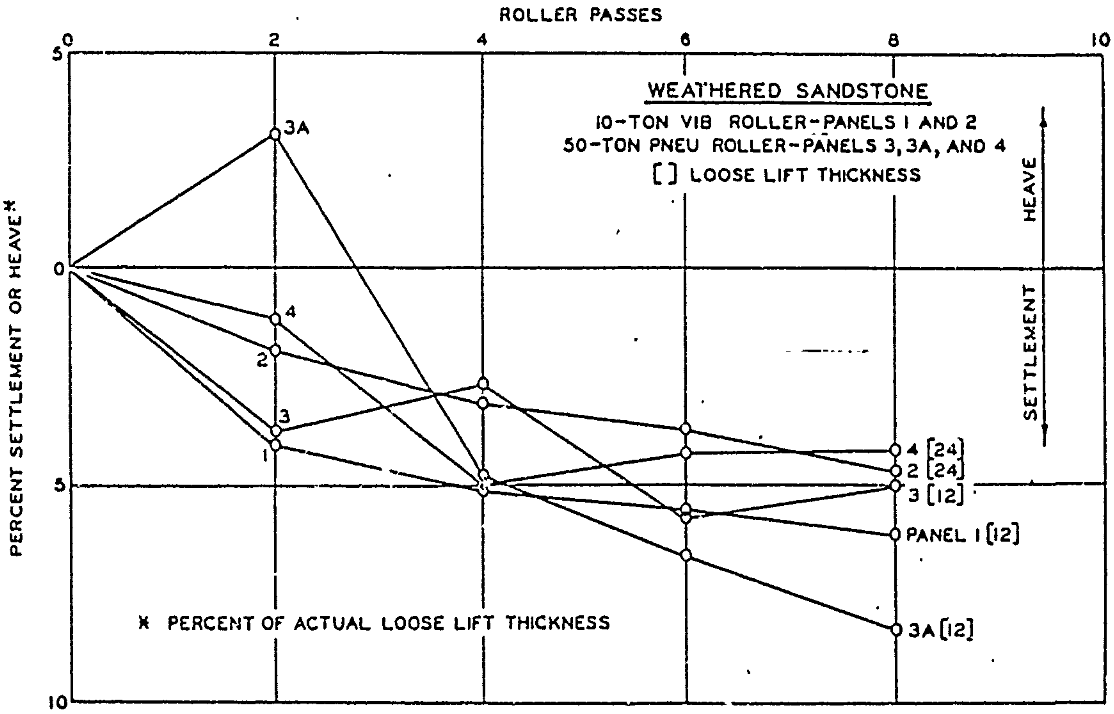
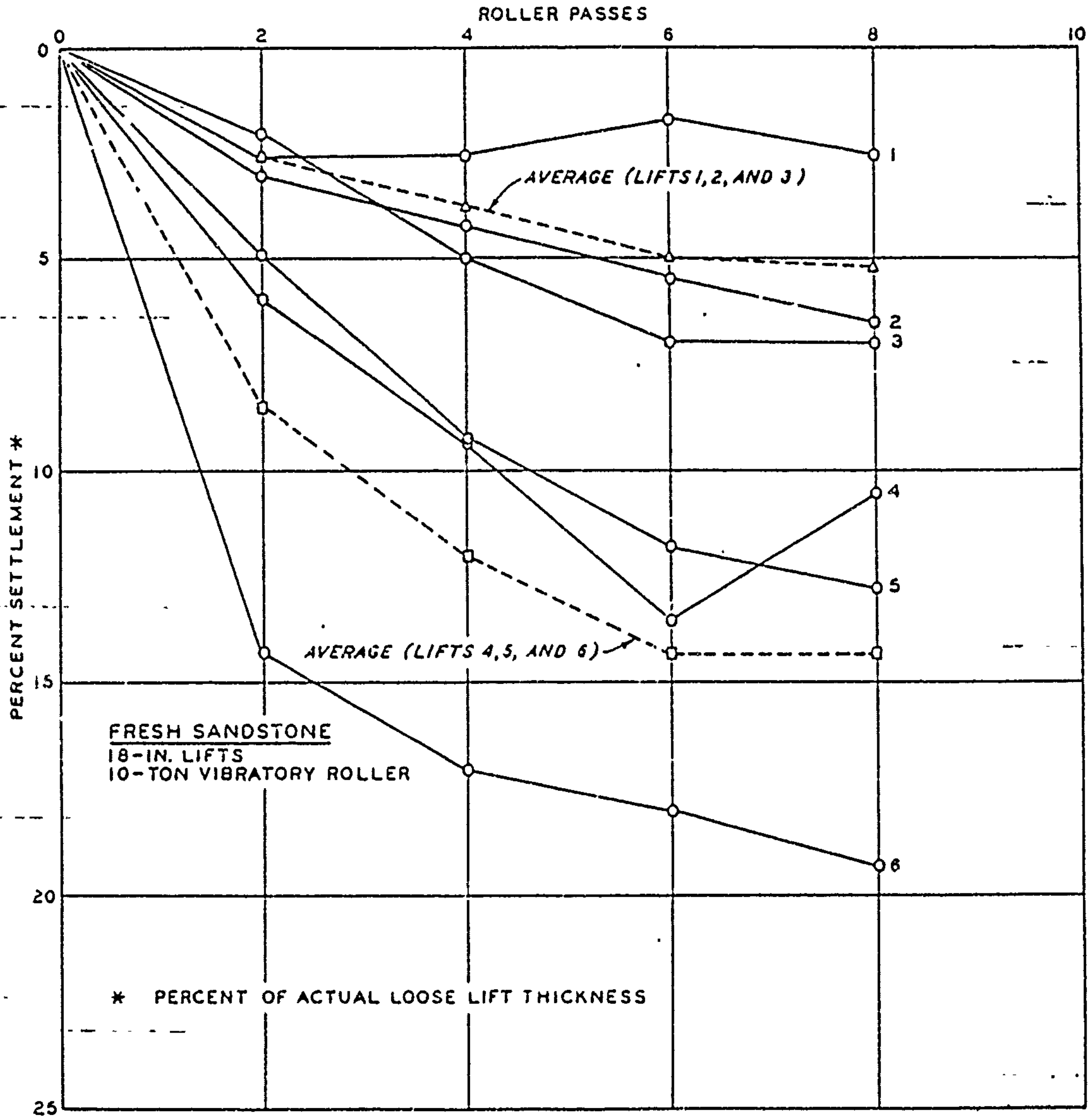


Fig. 8. Average percent settlement vs roller passes, panels 1, 2, 3, 3A, and 4, Gillham test fill



NOTE: LIFTS 1, 2, AND 3 CONSTRUCTED OF QUARRY-RUN FRESH ROCK.
LIFTS 4, 5, AND 6 CONSTRUCTED OF PLUS 3-IN. FRESH ROCK.

Fig. 9 . Percent settlement vs roller passes, panel 5, Gillham test fill

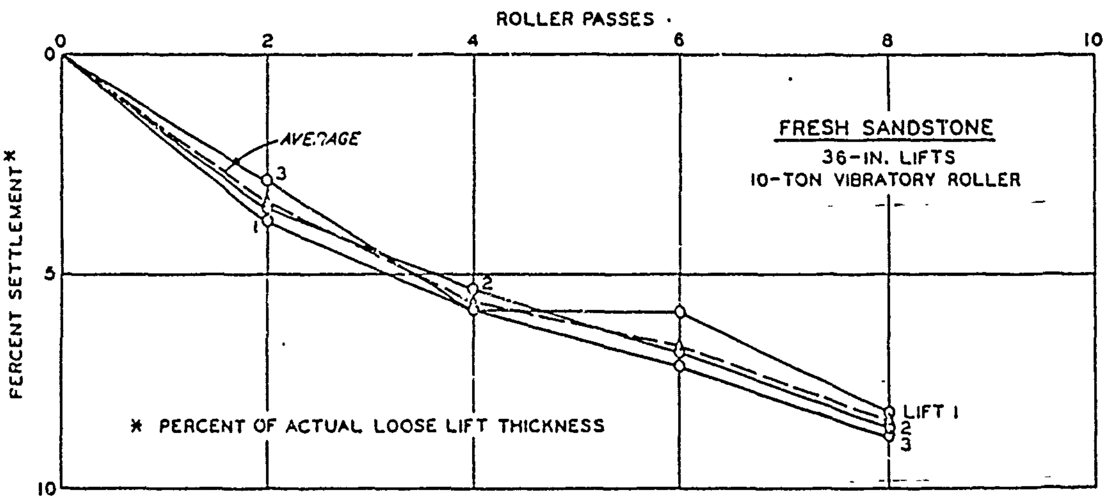


Fig. 10. Percent settlement vs roller passes, panel 6, Gillham test fill

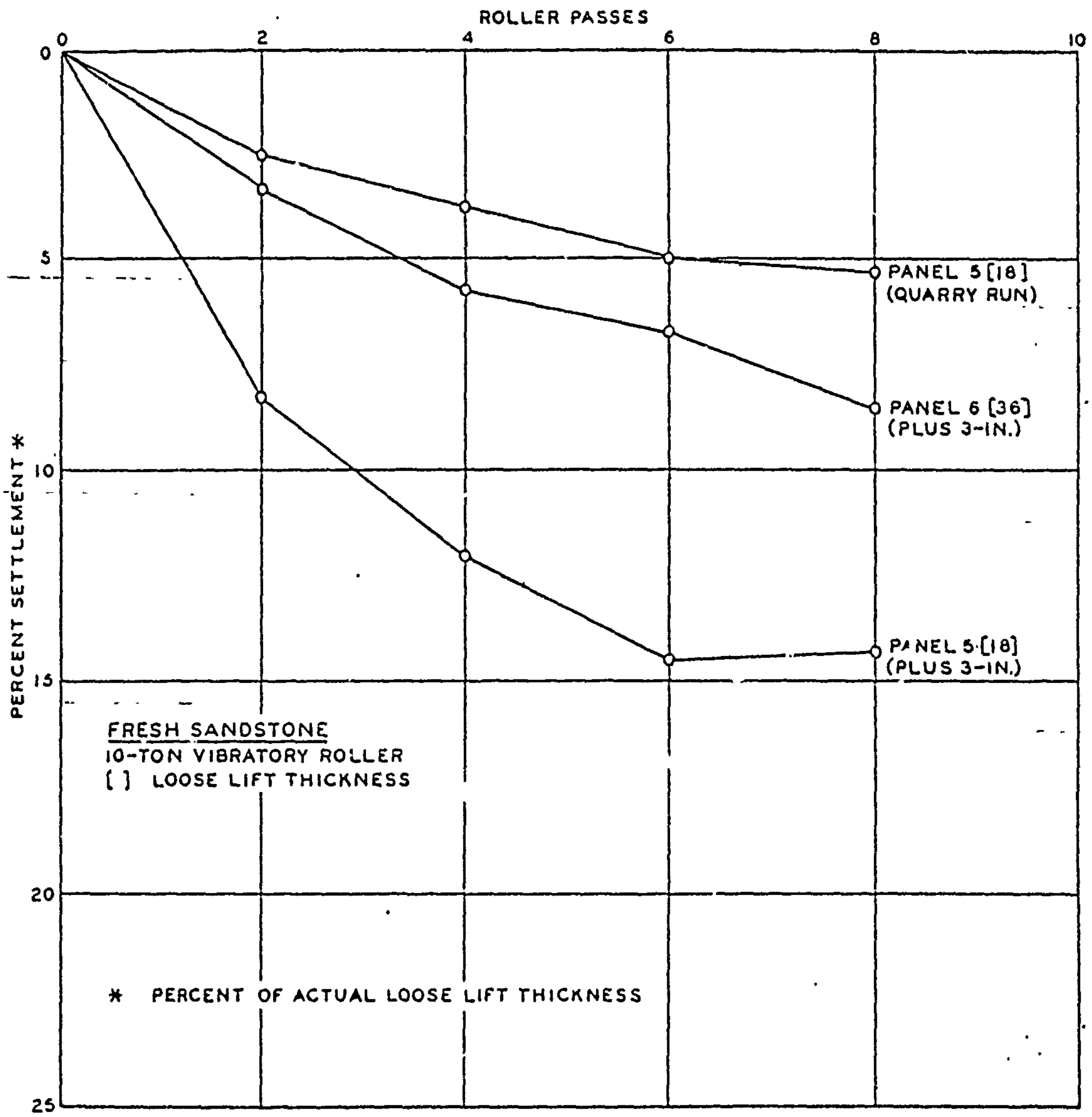


Fig. 11. Average percent settlement vs roller passes panels 5 and 6, Gillham test fill

Table3

Results of Density Tests After Compaction

Panel No.	Lift Thickness in.	Material*	Roller	In Situ Density pcf	Porosity %
3	12	Weathered sandstone	Pneumatic	135	17
3A	12	Weathered sandstone	Pneumatic	128	21
4	24	Weathered sandstone	Pneumatic	137	16
5	18	Fresh sandstone	Vibratory	91	43
5	18	Fresh sandstone	Vibratory	83	48
6	36	Fresh sandstone	Vibratory	104	36

* All weathered sandstone was quarry-run; all fresh sandstone was plus 3-in. material.

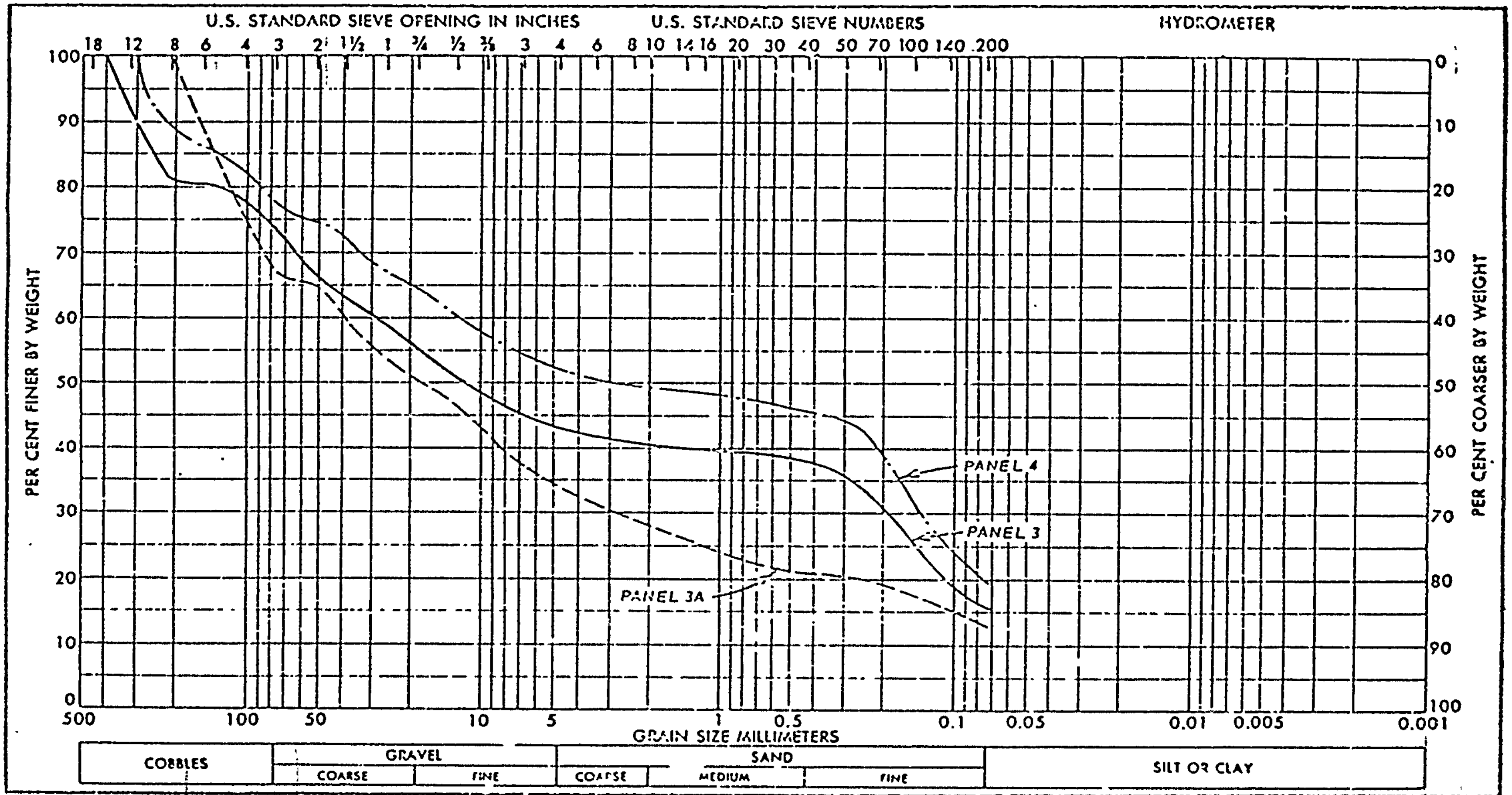


Fig. 12 . After-compaction gradation curves, panels 3, 3A, and 4, Gillham test fill

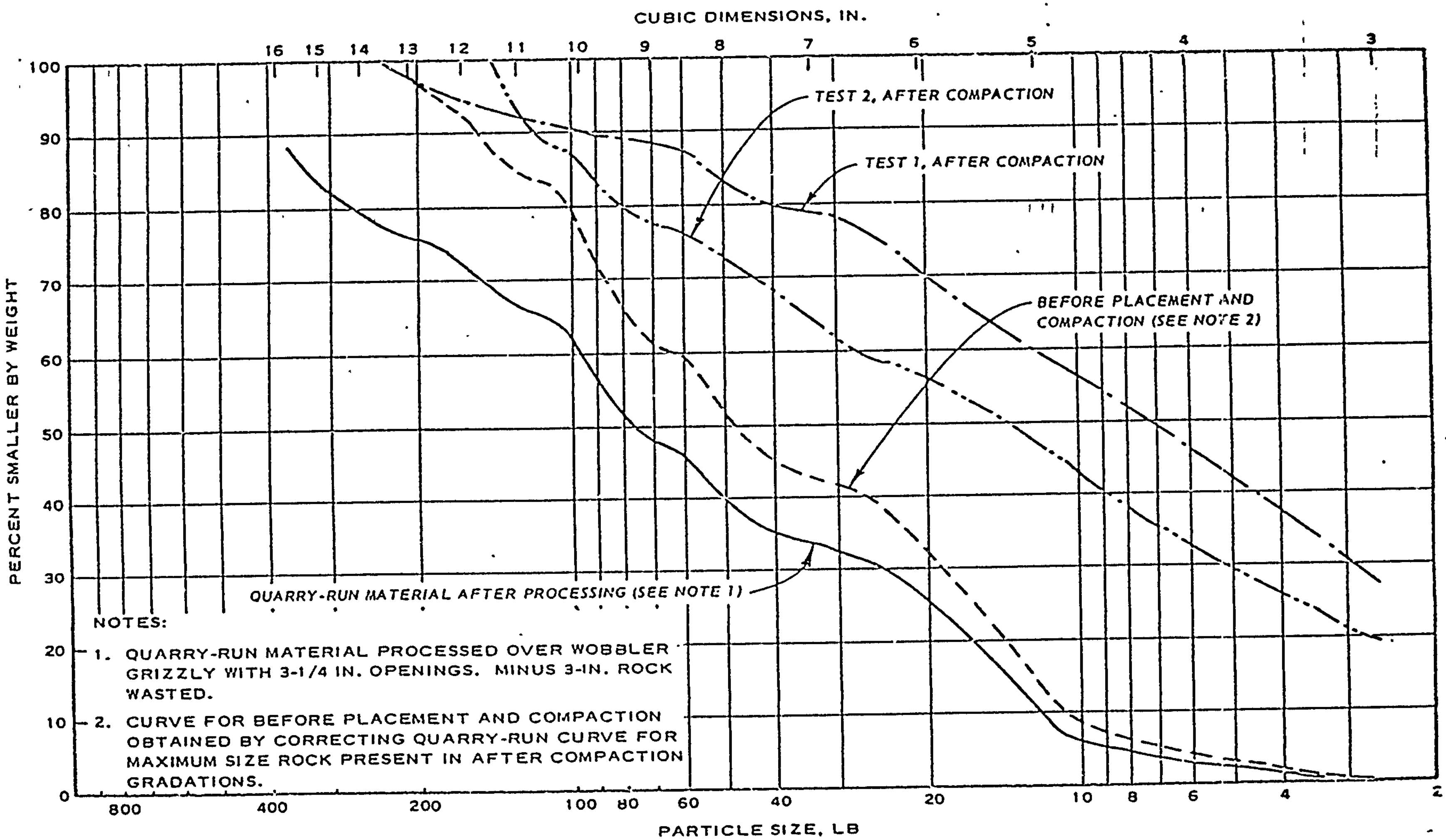


Fig. 13. Before- and after-compaction gradation curves, panel 5, Gillham test fill

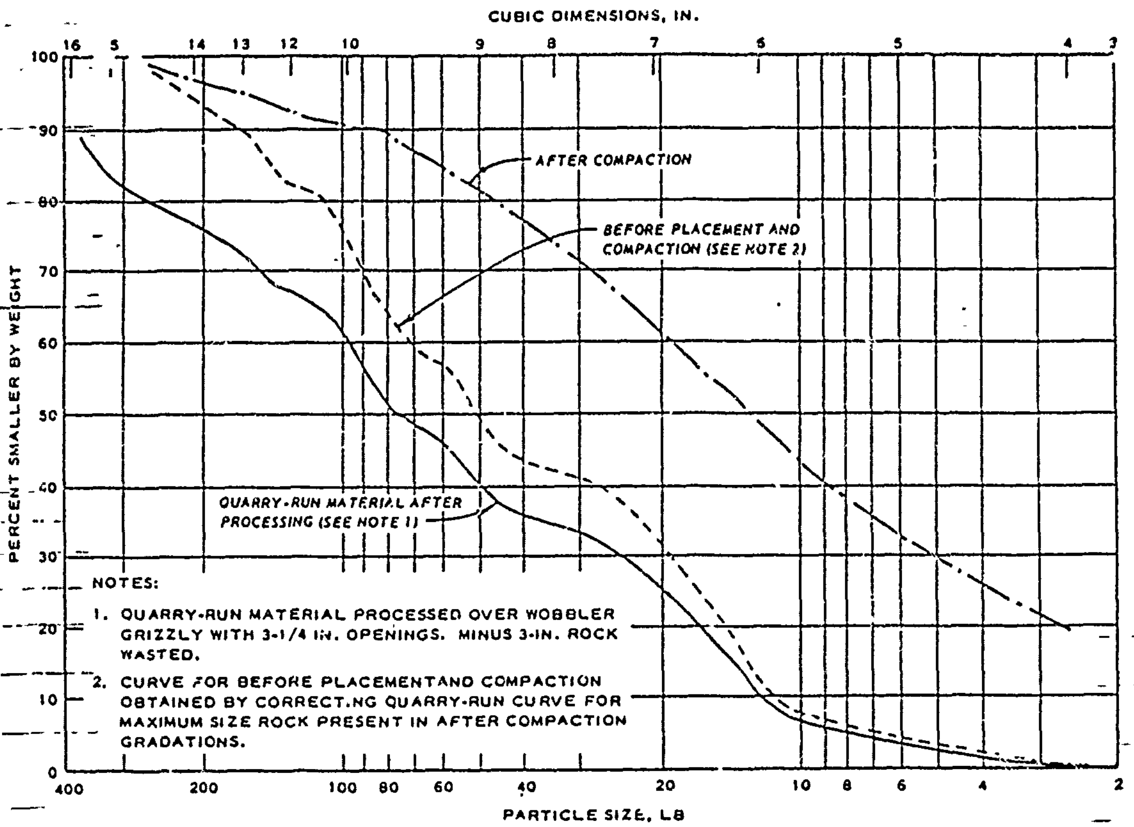


Fig. 14 . Before- and after-compaction gradation curves, panel 6, Gillham test fill

14 is the curve for the quarry-run fresh rock after processing over a wobbler grizzly with 3-1/4-in. screen openings. The curves representing the material before placement and compaction were derived from the processed quarry-run data by correcting to the maximum size rock present in the after-compaction gradations. It is seen from figs. 13 and 14 that significant degradation occurred under the action of the vibratory roller. Field personnel noted that 85 percent or more of the larger size rock chunks were broken at least once by the vibratory compactor.

(4) Inspection trench. Visual examination of the inspection trench confirmed the tightness and unusually high degree of stability of all compacted material. No mention was made of any segregated or stratified zones or that any evidence of poor bonding between lifts was found. Two views of the inspection trench are shown in the photographs in fig. 15.

6. Discussion.

a. As mentioned earlier, the settlement data indicated more settlement was obtained using the plus 3-in. fraction rather than the entire (quarry run) portion of the fresh sandstone. This also occurred at the Laurel Dam test fill, and, although no mention was made by the district about stratification and excess surface breakage, it is probable that the cause was not only that there were more open voids in the plus 3-in. material to begin with, but that a dense layer of surface fines formed in the quarry-run material, which reduced the compaction obtained in the lower part of the lift.

b. The results of the density tests on the fresh sandstone fill material (91, 83, and 104 pcf) seem low, especially in view of both the description given of the density pit sidewalls (see para 5b(2) and the settlement data. All things considered, one would tend to suspect the accuracy of the density tests. Large-volume density tests are themselves crude, even when the best possible procedures are used. In these tests,

a. View north, inspection trench excavation

b. View south, inspection trench excavation

Fig. 15 Inspection trench, Gillham test fill pit volume was determined by geometric measurements which is not desirable for a rock-fill material. Computations of volume made from rule or tape measurements usually fail to account for protruding rocks, cavities, and unsymmetrical pit shapes. It should be recognized, however, that it is possible to obtain a very stable, free-draining, and otherwise desirable rock fill without necessarily achieving a correspondingly high density.

c. The erratic nature of some of the settlement data indicates the problems involved in taking level readings on a rock-fill surface. Often a premarked point for a reading will consist of rock that has rotated under the roller action rather than having been pushed down, and thus does not reflect the average elevation of the surrounding surface. This is often true in the case of compaction by rubber-tired rollers because they do not tend to crush the surface rock as does a vibratory roller. This problem might also be aggravated by the use of thinner lifts. Some inaccuracies could also be caused if significant settlement is still occurring in underlying lifts but is being measured as occurring in the lift under compaction. An alternative to this problem would be to plot the data as actual settlement for an entire panel (as long as all lifts in the panel were of the same thickness and material). However, if settlement by lift in terms of percent of initial lift thickness is desired, the preceding lift can be rolled until no significant additional settlement is observed.

APPENDIX D

SWDGL REPORT NO. 7491

RESULTS OF TESTS OF BORROW MATERIALS

SWDGL REPORT NO. 7491

RESULTS OF TESTS OF BORROW MATERIALS

GILLHAM DAM - TULSA DISTRICT

CORPS OF ENGINEERS

U. S. ARMY

SOUTHWESTERN DIVISION LABORATORY

DALLAS, TEXAS

REQUEST FOR AND RESULTS OF TESTS

PAGE NO. 1 OF PAGES 10

REQUEST FOR TEST TU-FM-62-1 dated 23 Oct 61

1. TO:
Southwestern Division Laboratory
US Army Engineer Division, Southwestern
Corps of Engineers
4815 Cass Street
Dallas 35, Texas

2. FROM:
Chief,
Foundation and Materials Branch
Tulsa Distri t

3. PRIME CONTRACTOR AND ADDRESS

CONTRACT NUMBER

4. MANUFACTURING PLANT NAME AND ADDRESS

P. O. NUMBER

5. END ITEM AND/OR PROJECT: Gillham Dam

6. SAMPLE NUMBER

7. LOT NO

8. REASON FOR SUBMITTAL

9. DATE SUBMITTED

10. MATERIAL TO BE TESTED: Borrow

10a. QUANTITY SUBMITTED: 1 Bag Sample

11. QUANTITY REPRESENTED

12. SPEC. & AMEND AND/OR DRAWING NO. & REV. FOR SAMPLE & DATE

13. PURCHASED FROM OR SOURCE: TPA-1 Composite - 1

14. SHIPMENT METHOD

15. DATE SAMPLED AND SUBMITTED BY

16. REMARKS AND OR SPECIAL INSTRUCTIONS AND/OR WAIVERS

17. RECEIPT

THE ABOVE MATERIAL HAS BEEN RECEIVED

DATE	TYPED NAME AND TITLE	SIGNATURE
19 Oct 61	A. H. FEESE, Engineer	

RESULTS OF TEST (Continue on plain white paper if more space is required)

1. RESULTS

☐ ACCEPTABLE ☐ FAILURE INDICATED

☐ OTHER (Specify)

2. DATE REC'D	3. DATE REPORTED	4. LAB REPORT NUMBER
19 Oct 61	12 Jan 62	SWDGL-7491

5. TEST PERFORMED, RESULTS OF TEST, SAMPLE RESULT, REQUIREMENTS

See Test Data Summary and Plates 1 - 8.

6. SEND REPORT OF TEST TO

Dist Engr, Tulsa District Attn: F&M Br

7. THE PRESCRIBED TESTS WERE MADE ON 2 Nov - 26 Dec 61 (DATE) AND THE RESULTS THEREOF WERE AS SET FORTH ~~ABOVE~~ herein.

DATE	TYPED NAME AND TITLE OF PERSON CONDUCTING TEST	SIGNATURE
12 Jan 62	K. M. COEN Engineer-in-Charge Southwestern Division Laboratory	

DD FORM 1 JUL 56 1222

REPLACES DA FORMS 696, 1651 AND 10-208 WHICH ARE OBSOLETE

Sheet 1 of 1

TEST DATA SUMMARY

S W D O L REPORT NO. 2491

FEATURE: BORROW MATERIAL

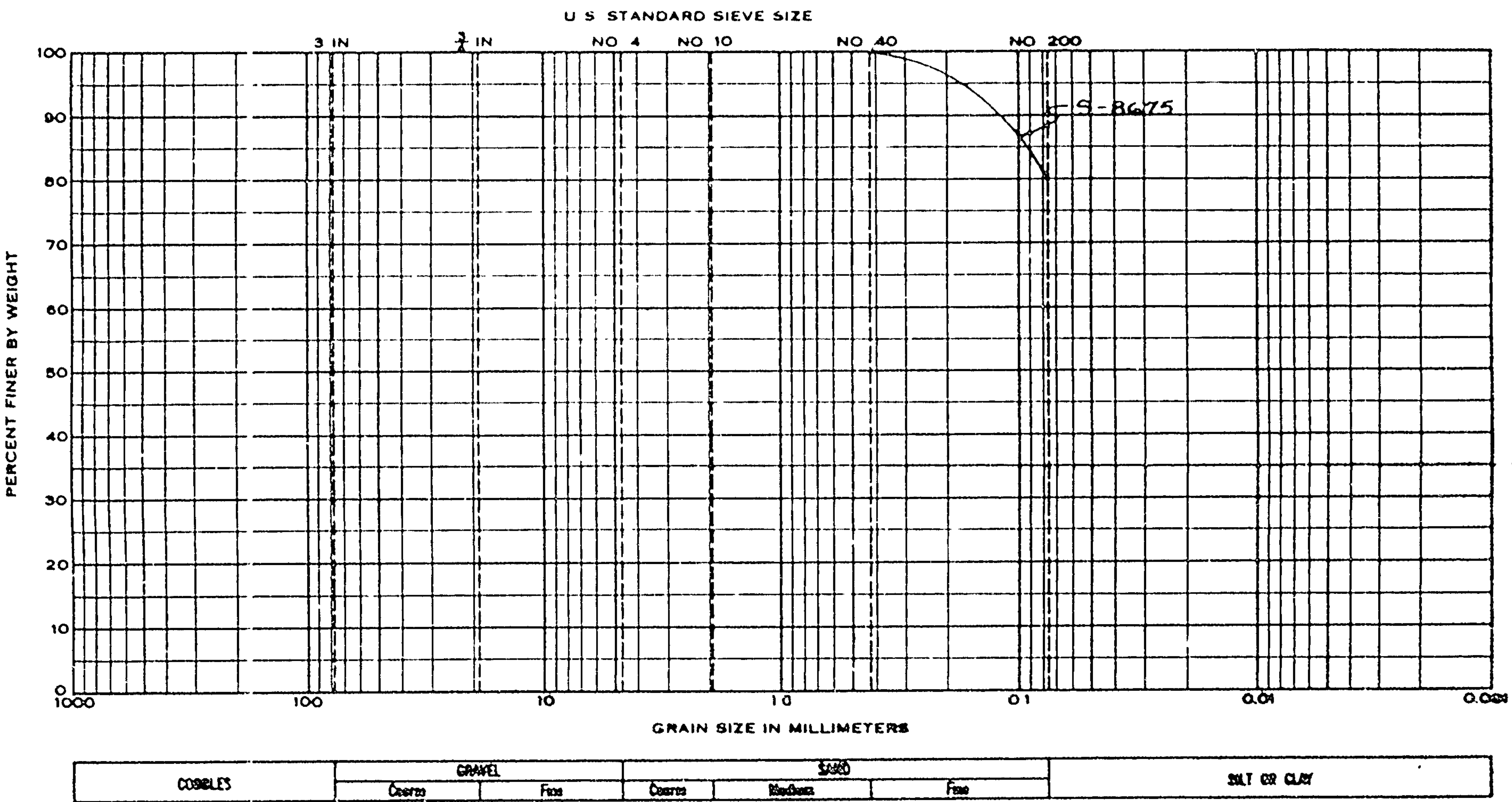
Gillham Dam

Boring No.	Sample No.	Depth or Elev. of Sample	Laboratory Classification	Mechanical Analysis: Gravel %	Sand %	Fines %	D_{10}	Atterberg Limits: LL	PL	Specific Gravity	Nat. Water Cont. %	Natural Dry Density lb/cu ft	Compaction Data: Opt. Water %	Maximum Dry Density lb/cu ft	Remarks
TPA-1															15-7
5	6475	0.0-4.0	CL	0	20	80		31	17	2.66			17.3	107.6	
Composite	1			Dark brown Sandy CLAY											
												Remolded to approx. 95.9 std AASHO max density at optimum water content			
												Remolded to approx. 95% std AASHO max density at optimum + 3%			

Initial e	Dry Density lb/cu ft	w_0 %	w_f %	S %	Type Test	Specimen Size Inches	Test	σ_3 T/sq ft	σ_1 T/sq ft	c T/sq ft	φ degrees	Permeability k		Consolidation Data p_0	p_c	C_c	
.624	102	17.3		74	T-Q	1.4 x 3.0	1	0.5	2.51	2.6	16.6						
.615	103	17.2		74	"	"	2	1.5	5.12								
.608	103	17.2		75	"	"	3	3.0	7.90								
.628	102	17.4		74	"	"	4	6.0	12.59								
.667	100	17.3	20.9	69	T-R	1.4 x 2.9	1	1.5	4.78	0.2	27.6						
.651	101	17.4	19.5	72	"	1.4 x 3.0	2	3.0	8.33								
.641	101	17.4	18.5	72	"	"	3	6.0	16.87								
.626	102	17.5	11.7	74	DS	3.0 x 3.0 x 0.5	1			2.1	28.5						
.620	102	17.5	16.9	74		"	2										
.620	102	17.5	15.7	74		"	3										
.623	102	20.3		87	T-Q	1.4 x 3.0	1	0.5	1.84	0.5	4.8						
.623	102	20.4		87	"	"	2	1.5	3.35								
.628	102	20.4		87	"	"	3	3.0	5.20								
.625	102	20.2		86	"	"	4	6.0	8.45								
.615	103	19.6	18.9	85	T-R	1.4 x 3.0	1	1.5	5.83	0.5	25.1						
.615	103	19.9	18.3	86	"	"	2	3.0	8.92								
.615	103	19.9	17.7	86	"	"	3	6.0	16.49								
.622	103	20.1	17.9	86	DS	3.0 x 3.0 x 0.5	1			0.2	27.0						
.625	102	20.3	18.0	86	"	"	2										
.617	103	20.1	17.0	87	"	"	3										

T – Triaxial Compression; UC – Unconfined Compression; DS – Direct Shear; Q UU – Unconsolidated Undrained; R CU – Consolidated Undrained; Consolidated Drained; *Values of Pressure T/SQ FT

ENG FORM 2086 — PREVIOUS EDITIONS MAY BE USED



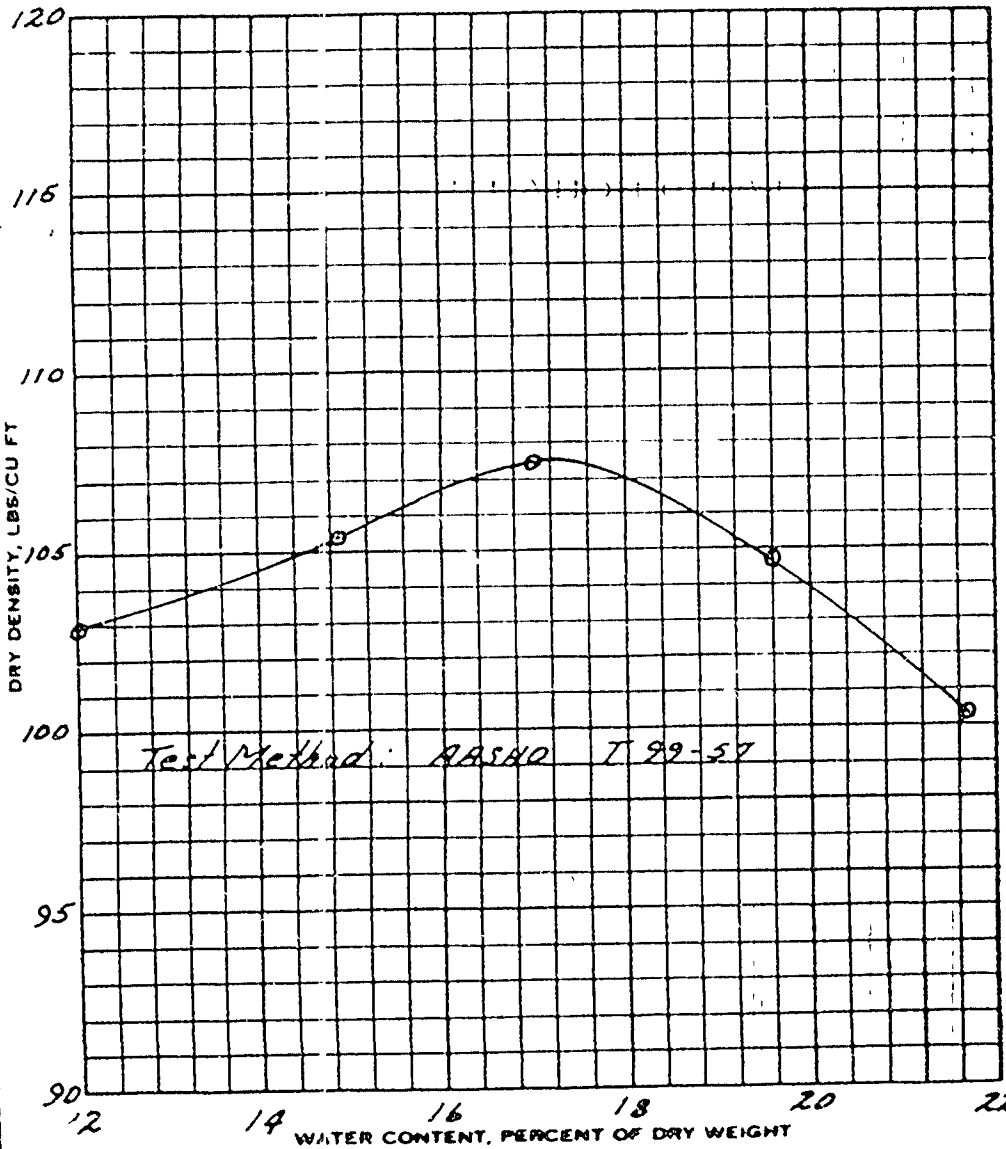
Sample No.	Elev or Depth	Classification	Nat WC	LL	PL	PI	LS	
S-8675	0.0-4.0	Sandy CLAY (CL)		31	17	14	7	Composite-1

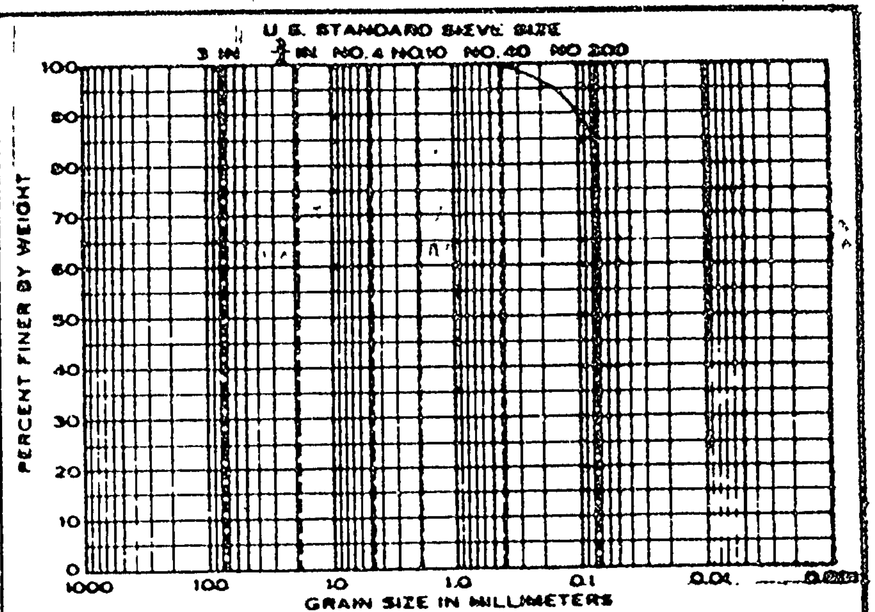
Project Gillham Dam
BORROW MATERIAL
Area TULSA DISTRICT
Boring No. TPA-1
Date JAN 62

GRADATION CURVES

ENG FORM 2087

SWDAL Project No. 7891





COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	Elev or Depth	Classification	G	LL	PL
		Sandy CLAY (CL)		31	17

Sample No.					
Optimum Water Content	%	17.9			
Max Dry Density	Lbs/Cu Ft	107.6			
Optimum Water Content Corr for +	%				
Max Density Corr for +	Lbs/Cu Ft				

Project: Gillham Dam

BORROW MATERIAL

Area: TULSA DISTRICT

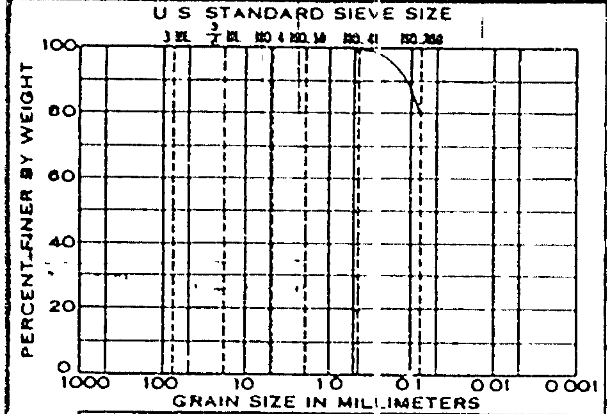
Boring No. TPA-1 | Sample No. Composite-1 S-8675

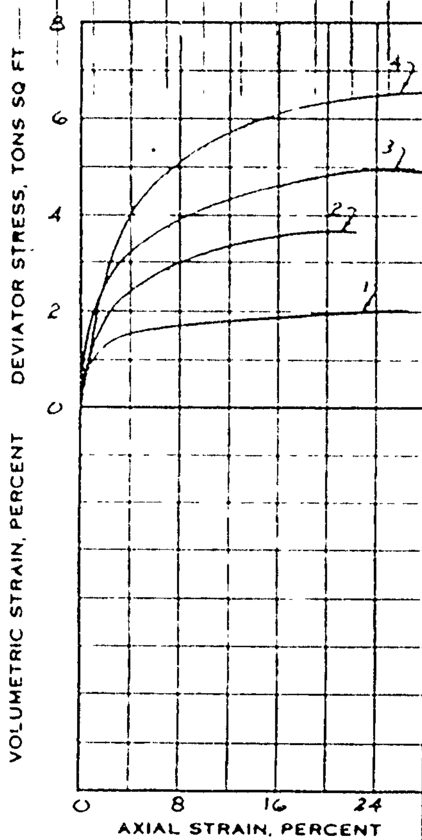
Elev or Depth 0.0-4.0 | Date JAN 62

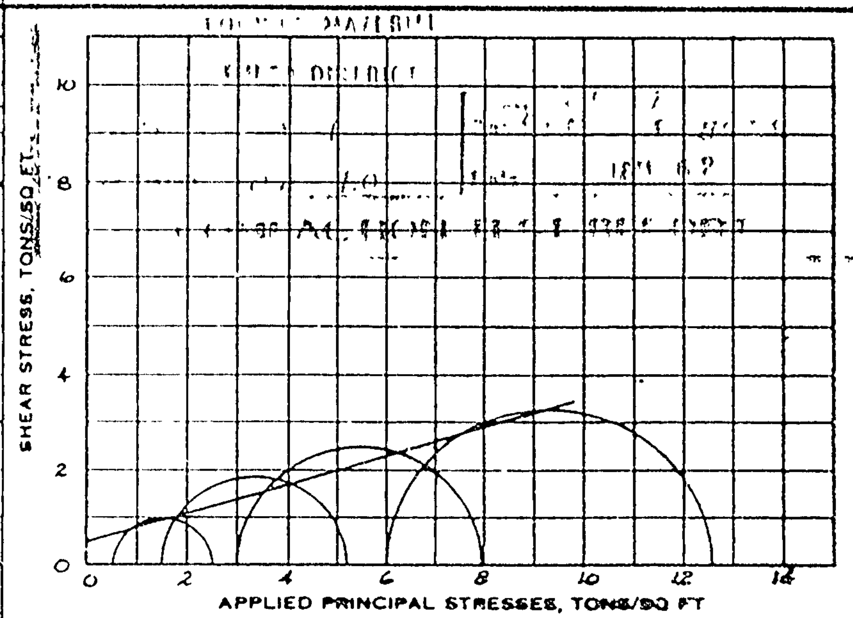
COMPACTION TEST REPORT

ENG FORM 2091 (Translucent)

PLATE NO. 2







Test No		1	2	3	4
INITIAL	Water Content, W_0 %	17.3	17.2	17.2	17.4
	Dry Density Lbs/Cu Ft	102	103	103	102
	Void Ratio, e_0	.624	.615	.608	.628
	Saturation, S_0 %	74	74	75	74
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/Sq Ft				
	W.C after Consol, W_c %				
	Void Ratio after Consol, e_c				
AT FAILURE	Max Prin Stress, σ_1 T/Sq Ft	2.51	5.19	7.95	12.59
	Min Prin Stress, σ_m T/Sq Ft	0.5	1.5	3.0	6.0
	Water Content, W_f %				
	Void Ratio, e_f				
Specimen Diameter	Inches	1.4	1.4	1.4	1.4
Initial Height	In	3.0	3.0	3.0	3.0
Test Time to Failure	Min	24	20	25	27

Type of Test - Q
Constant Strain, 0.03 in/min Control
Un Consolidated, Un Drained

Type of Specimen REMOLDED

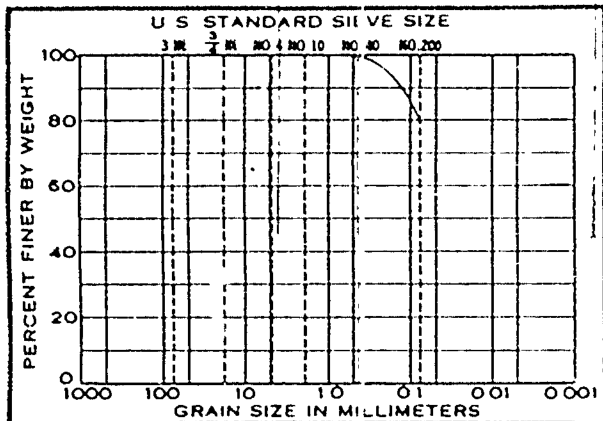
ϕ - 16.6°	Tan ϕ - .298	c - 0.6 T/Sq Ft
Classification Sandy CLAY (CL)		
LL 31	G 2.66	
PL 17	D_x	

Remarks.
Q test

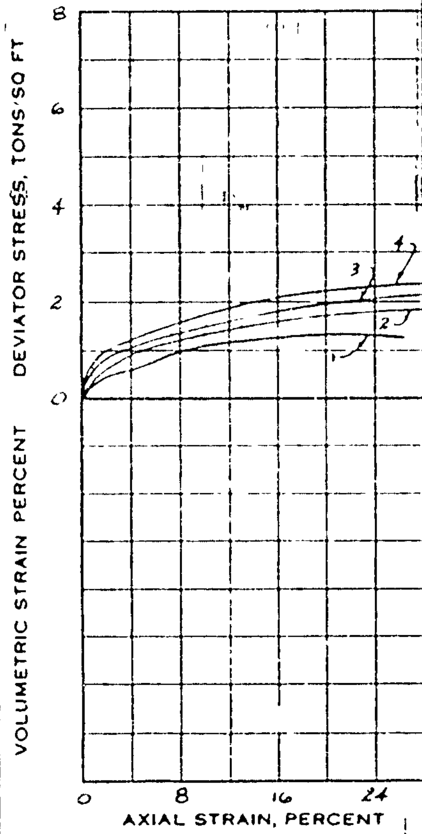
Project Gillham Dam	
BORROW MATERIAL	
Area TULSA DISTRICT	
Boring No. TPA-1	Composite -1 (95 Y, W) Sample No. S-8675
Elev or Depth 0.0 - 4.0	Date JAN 62

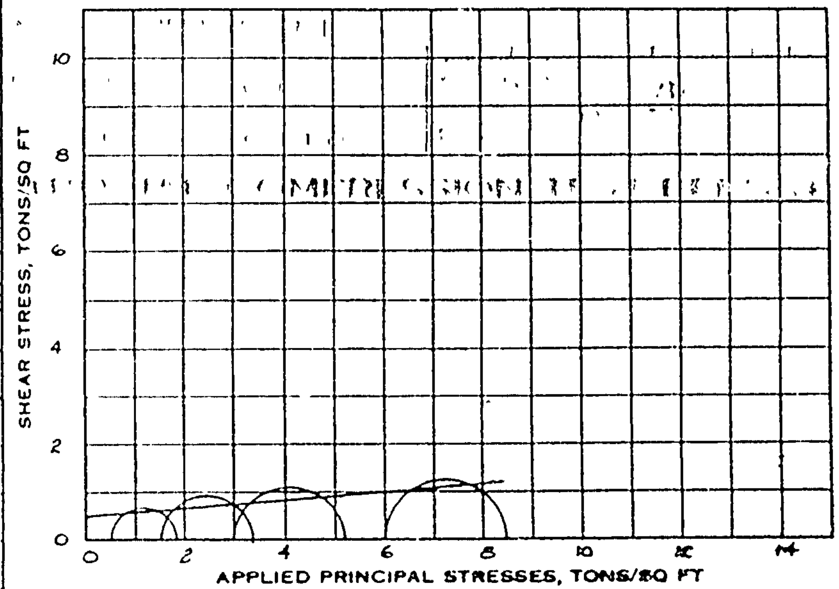
TRIAXIAL COMPRESSION TEST REPORT

PLATE NO. 3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	C	F	C	M	F	





	Test No	1	2	3	4
INITIAL	Water Content, W_o %	20.3	20.4	20.4	20.2
	Dry Density Lbs/Cu Ft	102	102	102	102
	Void Ratio, e_o	.623	.623	.628	.625
	Saturation, S_o %	87	87	87	86
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/Sq Ft				
	W.C. after Consol, W_c %				
	Void Ratio after Consol e_c				
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	1.84	3.35	5.20	8.45
	Min Prin Stress, σ_m T/Sq Ft	0.5	1.5	3.0	6.0
	Water Content, W_f %				
	Void Ratio, e_f				
	Specimen Diameter Inches	1.4	1.4	1.4	1.4
	Initial Height In	3.0	3.0	3.0	3.0
	Test Time to Failure Min	21	27	33	30

Type of Test – Q
Constant Strain, .003 in/min Control
Un Consolidated, Un Drained

Type of Specimen REMOLDED

ϕ = 4.8°	Tan ϕ = .084	c = 0.5 T/Sq Ft

Classification Sandy CLAY (CL)

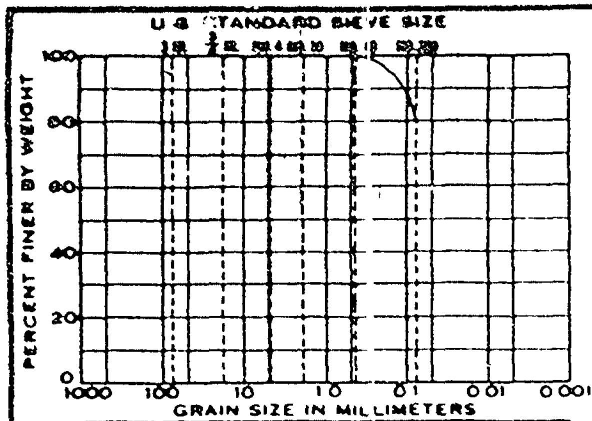
LL 31	G 2.66
PL 17	D_{10}

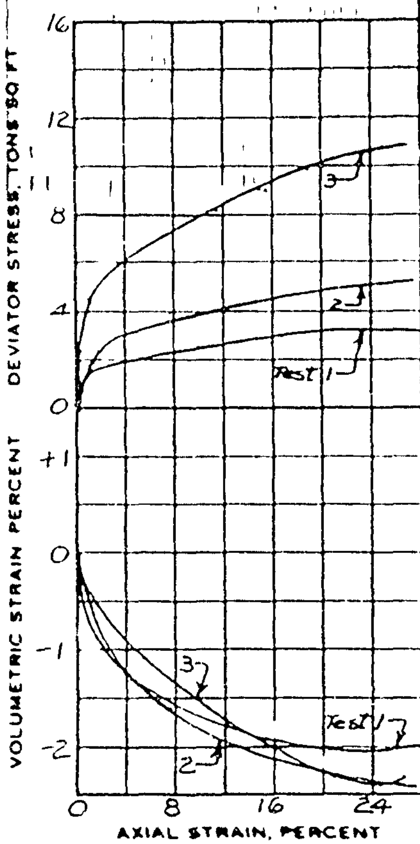
Remarks: Q test

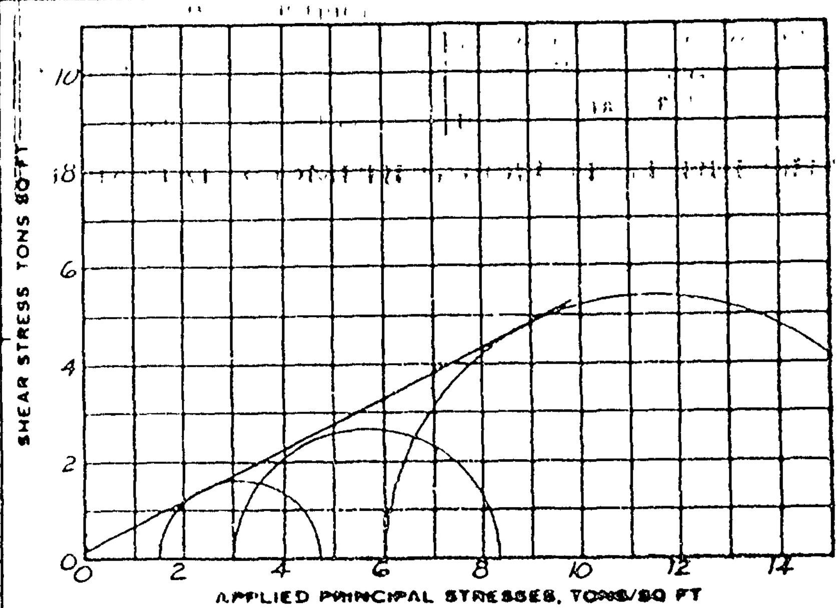
Project Gillham Dam
BORROW MATERIAL
Area TULSA DISTRICT
Boring No. TPA-1 — Composite 1 (95', wt 3) Sample No. S-8675
~~Elev or~~ Depth 0.0 - 4.0 — Date JAN 62

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 MAY 51 2089







	Test No	1	2	3
INITIAL	Water Content, W_0 %	17.3	17.4	17.4
	Dry Density Lbs/Cu Ft	100	101	101
	Void Ratio, e_0	.667	.644	.641
	Saturation, S_0 %	69	72	72
BEFORE TEST	W C after Saturation, W_s %	22.0	21.6	21.9
	Saturation, S %	88	89	90
	Consol Pressure T/Sq Ft	1.5	3.0	6.0
	W.C after Consol, W_c %	20.9	19.5	18.5
	Void Ratio after Consol e_c	.632	.581	.546
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	4.78	8.33	16.87
	Min Prin Stress, σ_3 T/Sq Ft	1.5	3.0	6.0
	Water Content, W_f %	20.9	9.5	18.5
	Void Ratio, e_f	.599	.544	.511
	Specimen Diameter Inches	1.4	1.4	1.4
	Initial Height In	2.9	3.0	3.0
	Test Time to Failure Min	23	27	26

Type of Test - R
Constant Strain, 0.03 in/min Control
X Consolidated, UnDrained

Type of Specimen REMOLDED

ϕ - 27.6° | Tan ϕ - .522 | c - 0.2 T/Sq Ft

Classification Sandy CLAY (CL)

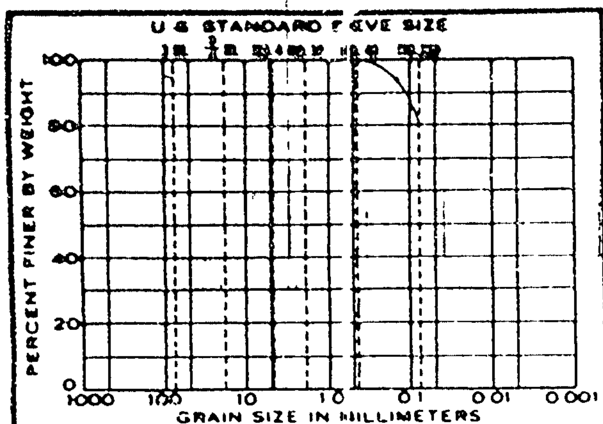
LL 31 | G 2.66
PL 17 | D_{10}

Remarks R Test

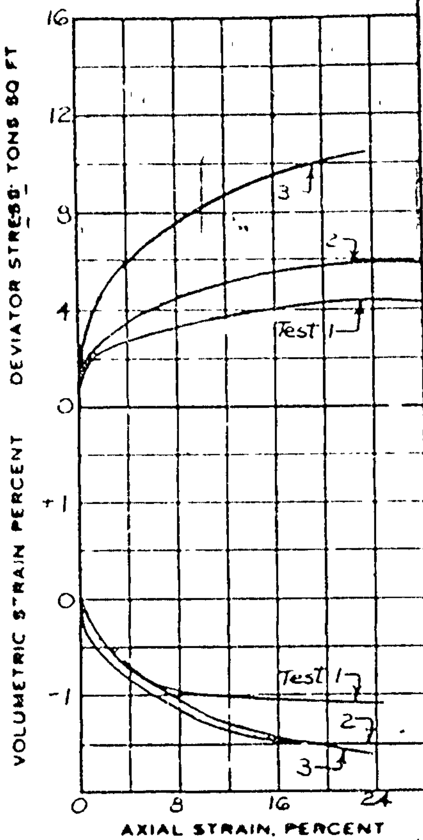
Project Gilham Dam
BORROW MATERIAL
Area TULSA DISTRICT
Boring No TPA-1
Composite - 1
Sample No. 5-8675 (95% γ, ω)
Elev or Depth 0.0-4.0
Date JAN 62

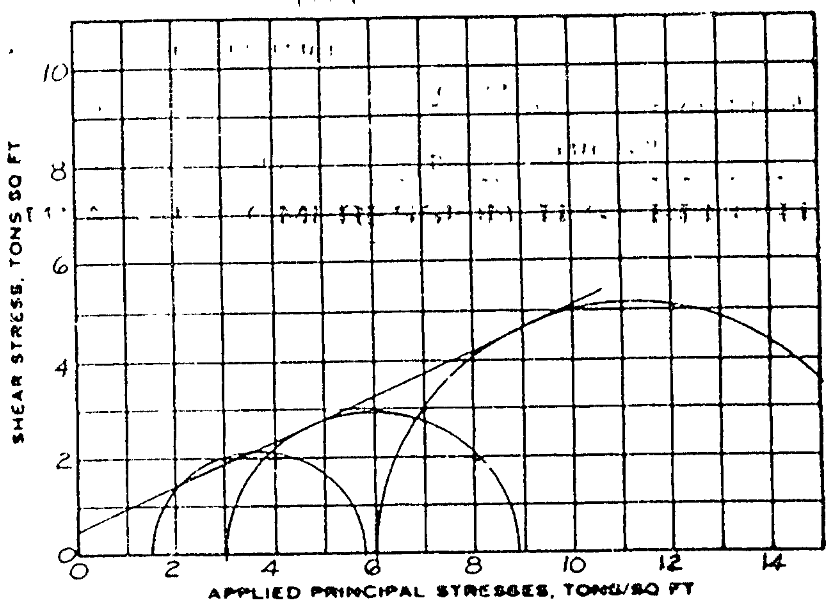
TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089 (EM 1110-2-1803) (Translucent)



COBBLES	GRAVEL C	GRAVEL F	SAND C	SAND M	SAND F	SILT OR CLAY





	Test No	1	2	3
INITIAL	Water Content, W_o %	19.6	19.9	19.9
	Dry Density Lbs/Cu Ft	103	103	103
	Void Ratio, e_o	.615	.615	.615
	Saturation, S_o %	85	85	86
BEFORE TEST	W C after Saturation, W_s %	20.	20.7	20.6
	Saturation, S %	89	91	93
	Consol Pressure T/Sq Ft	1.5	3.0	6.0
	W.C after Consol, W_c %	18.9	18.3	17.7
	Void Ratio after Consol e_c	.576	.558	.530
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	5.83	8.92	16.49
	Min Prin Stress, σ_3 T/Sq Ft	1.5	3.0	6.0
	Water Content, W_f %	18.9	18.3	17.7
	Void Ratio, e_f	.560	.535	.505
	Specimen Diameter Inches	1.4	1.4	1.4
	Initial Height In	3.0	3.0	3.0
	Test Time to Failure Min	24	25	22

Type of Test - R
Constant Strain, 0.03 in/min Control
X Consolidated, UnDrained

Type of Specimen REMOLDED

ϕ - 25.1° | Tan ϕ - .468 | c - 0.5 T/Sq Ft

Classification Sandy CLAY (CL)

LL 31 | G 2.66
PL 17 | D_{10}

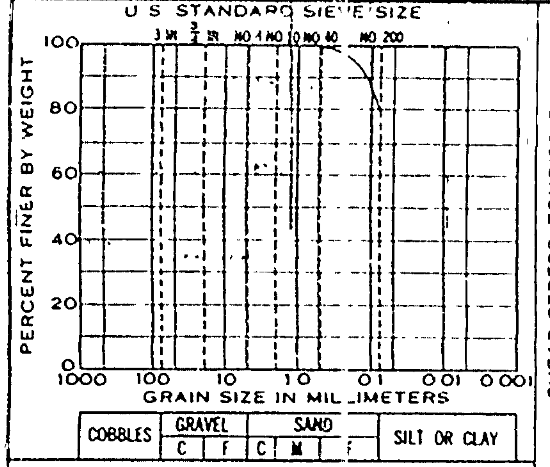
Remarks: R Lent

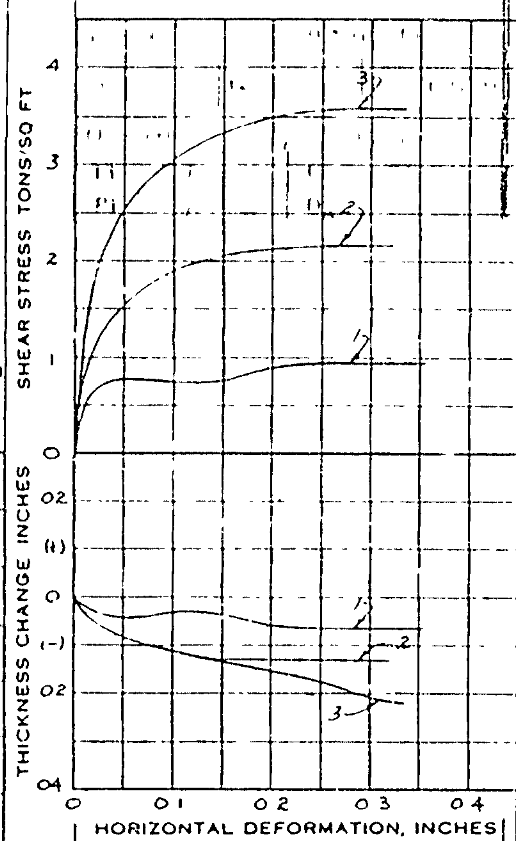
Project Gilham Dam
BORROW MATERIAL
Area TULSA DISTRICT
Boring No TPA-1
Composite - 1
Sample No. S-8675 (952 T, w+3)
Elev or Depth 00-4.0
Date JAN 62

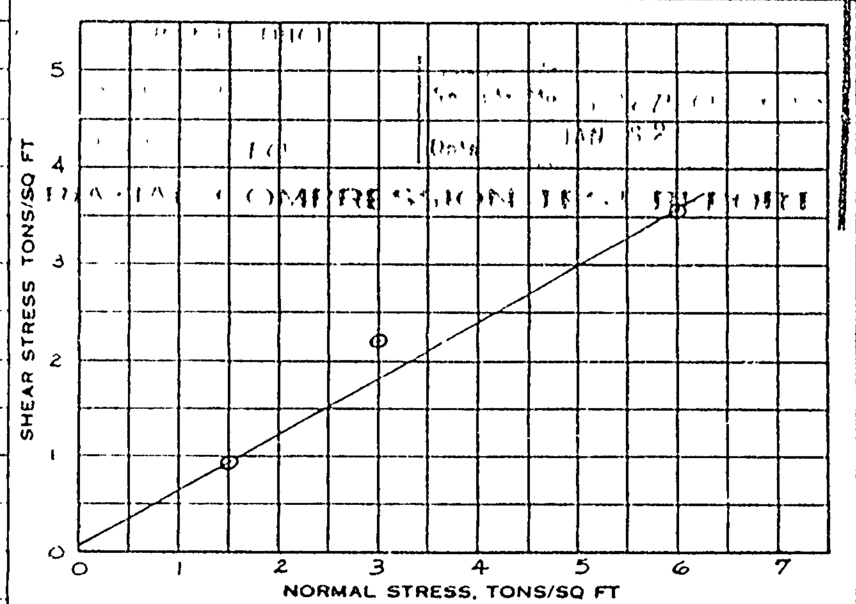
TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089 (EM 1110-2-1809) (Translucent)

SWDGL REPORT NO. 7491







Classification: *Sandy CLAY (CL)*

LL 31	G 2.66
PL 17	D_{10}

Test No	1	2	3
Initial Water Content, W_o %	17.5	7.5	17.5
Dry Density Lbs/Cu Ft	102	102	102
Void Ratio, e_o	.626	.628	.628
Saturation, S_o %	74	74	74
Duration of Test (70°F)	300'	100'	390'
WC after Test, W_f %	17.7	16.9	15.7
Normal Stress, σ T/SqFt	1.5	3.0	6.0
Max Shear Stress, τ T/SqFt	0.93	2.18	3.58

Type of Test: Strain - Control, Rate, 0.001 in/min; x Consolidated; Drained

Type of Specimen: REMOLDED; 3.0 in. square; 0.5 in. thickness

SHEAR VALUES	Ø	TAN Ø	C, T/SQ FT
Maximum	30.5	.589	0.1
Ultimate			

Remarks:

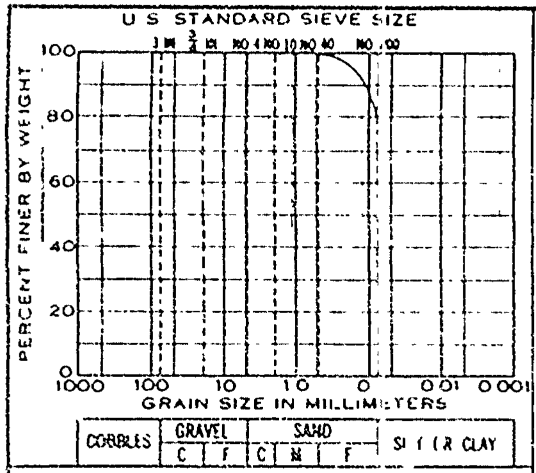
Project	Gillham Dam
	BORROW MATERIAL
Area	TULSA DISTRICT
Boring No. TPA-1	Composite - 1 (95 %, w) Sample No. S-8675
Elev or Depth 0.0 - 4.0	Date JAN 62

DIRECT SHEAR TEST REPORT

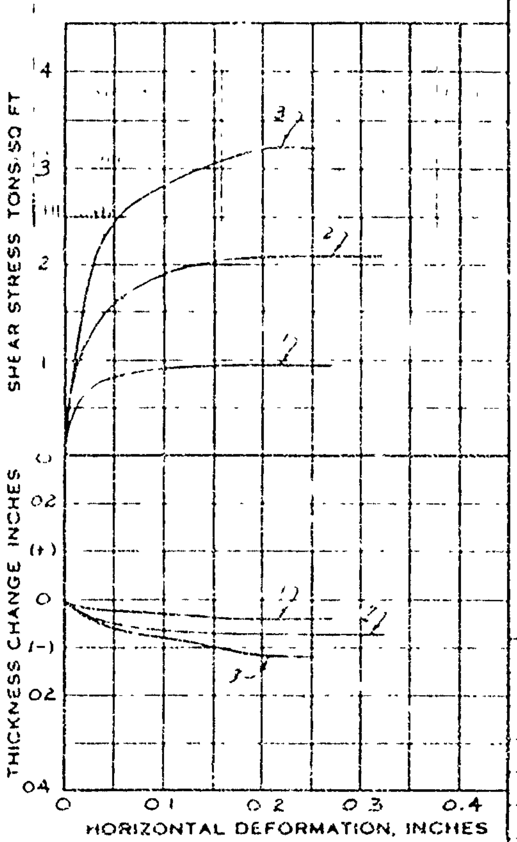
ENG FORM 2092

PLATE NO. 7

DIRECT SHEAR TEST REPORT

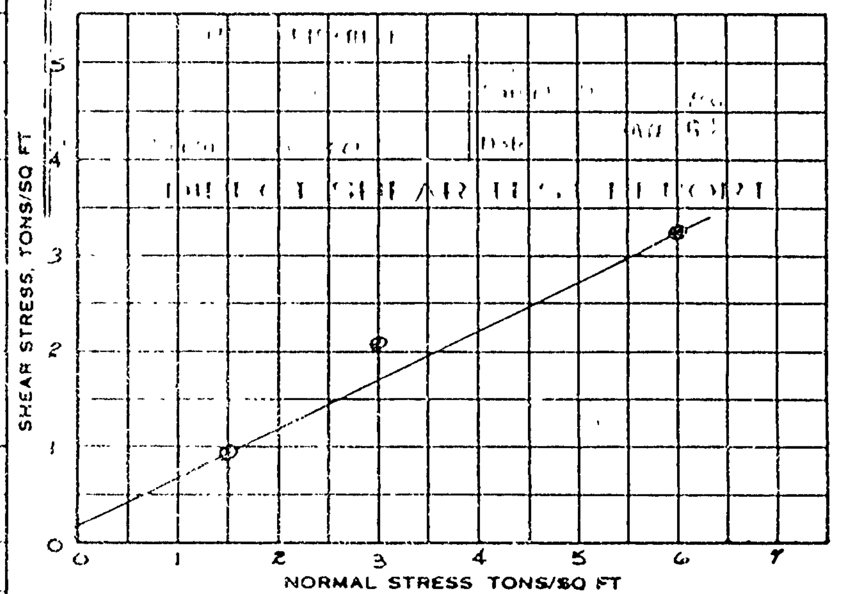


Classification	Sandy CLAY (CL)			
LL 31	G 2.66			
PI 17	D_{10}			
Test No	1	2	3	
Initial Water Content, W_o %	20.1	20.3	20.1	
Dry Density Lbs/Cu Ft	102	102	103	
Void Ratio, e_o	.622	.625	.617	
Saturation, S_o %	96	86	87	
Duration of Test (to τ)	180'	300'	315'	
WC after Test, W_f %	17.9	18.6	17.0	
Normal Stress, σ T/SqFt	1.5	3.0	6.0	
Max. Shear Stress, τ T/SqFt	0.95	2.07	3.24	



Type of Test	Type of Specimen
Strain - Control	REMOLDED
Rate, 0.001 in/min	3.0 in. square
X Consolidated	
Drained	0.5 in thickness

SHEAR VALUES	φ	TAN φ	C, T/SQ FT
Maximum	27.0	.509	0.2
Ultimate			



Remarks

Project	Gillham Dam		
	BORROW MATERIAL		
Area	TULSA DISTRICT		
Boring No.	TPA-1	Composite 1 Sample No.	(95%, W13) S-8675
Elev or Depth	0.0 - 4.0	Date	JAN 62

ENG FORM 2092

APPENDIX E

SWDGL REPORT NO. 8458

RESULTS OF TESTS OF BORROW SOIL

SWDGL REPORT NO. 8458

RESULTS OF TESTS OF BORROW SOIL
GILLHAM DAM - TULSA DISTRICT

CORPS OF ENGINEERS

U. S. ARMY

SOUTHWESTERN DIVISION LABORATORY

DALLAS, TEXAS

REQUEST FOR AND RESULTS OF TESTS	PAGE NO. 1	NO. OF PAGES 19

REQUEST FOR TEST TU-FM-63-23 dated 13 May 1963

1. TO:	2. FROM:
Southwestern Division Laboratory US Army Engineer Division, Southwestern Corps of Engineers 4815 Cass Street Dallas, Texas 75235	Chief, F&M Branch US Army Engineer District, Tulsa Tulsa, Oklahoma
3. PRIME CONTRACTOR AND ADDRESS CONTRACT NUMBER	4. MANUFACTURING PLANT NAME AND ADDRESS P. O. NUMBER

5. END ITEM AND/OR PROJECT	6. SAMPLE NUMBER	7. LOT NO.	8. REASON FOR SUBMITTAL	9. DATE SUBMITTED
Gillham Dam				

10. MATERIAL TO BE TESTED	10a. QUANTITY SUBMITTED	11. QUANTITY REPRESENTED	12. SPEC. & AMEND AND/OR DRAWING NO. & REV. FOR SAMPLE & DATE
Borrow	Ten Bags		

13. PURCHASED FROM OR SOURCE	14. SHIPMENT METHOD	15. DATE SAMPLED AND SUBMITTED BY
Holes TPB-8 and TBC-14	Gov't Vehicle	

16. REMARKS AND/OR SPECIAL INSTRUCTIONS AND/OR WAIVERS

17. RECEIPT

THE ABOVE MATERIAL HAS BEEN RECEIVED

DATE	TYPED NAME AND TITLE	SIGNATURE
9 May 63	A. H. FEESE, Engineer	

3. RESULTS OF TEST (Continue on plain white paper if more space is required)

1. RESULTS	2. DATE REC'D	3. DATE REPORTED	4. LAB REPORT NUMBER
☐ ACCEPTABLE ☐ FAILURE INDICATED ☐ OTHER (Specify)	9 May 63	29 July 63	SWDGL-8458

5. TEST PERFORMED, RESULTS OF TEST SAMPLE RESULT REQUIREMENTS

See Test Data Summary (2 sheets) and Plates 1 - 16

This report completes requested testing.

6. SEND REPORT OF TEST TO

Tulsa District O.

7. THE PRESCRIBED TESTS WERE MADE ON 18 June - 18 July 1963 (DATE) AND THE RESULTS THEREOF WERE AS SET FORTH ~~XXXX~~ herein.

DATE	TYPED NAME AND TITLE	SIGNATURE
29 July 63	K. M. COEN Engineer-in-Charge SWD Laboratory	K. M. Coen

 REPLACES DA FORMS 690, 1651 AND 10-208, WHICH ARE OBSOLETE.

TEST DATA SUMMARY

SWDGL Report No. 246-3 | FEATURE BORROW MATERIAL | Gillham Dam | Sheet 1 of 2

Boring No.	Sample No.	Depth or Elev. of Sample	Laboratory Classification	Gravel %	Sand %	Fines %	D_{10}	LL	PL	Specific Gravity G_s	Nat. Water Cont. %	Natural Dry Density lb/cu ft	Opt. Water %	Maximum Dry Density lb/cu ft	Initial e	Dry Density lb/cu ft	w_o %	w_f %	S_o %	Type Test	Specimen Size Inches	Test	σ_3 T/sq ft	σ_1 T/sq ft	c T/sq ft	φ Degrees	k	k ft/min	p_o T/sq ft	p_c T/sq ft	C_c	C_α	Remarks
														Location		Borrow Area B (TYPE-A)																	
5-17-71		1.5-4.0	CL	0	30	70		22	13	2.62			13.7	114.2																			15-A
					Sandy CLAY										.555	100	13.8		67	T-Q	1.4 x 3.0	1	0.5	2.90	0.6	20.4							
							Remolded to approx. 95% Std. AASHO max.								.545	109	13.8		68	"	1.4 x 3.0	2	1.5	9.09									
							density at optimum water content								.541	109	13.9		69	"	1.4 x 3.0	3	3.0	8.84									
															.541	109	13.7		60	"	1.4 x 3.0	4	6.0	14.36									
															.544	109	13.8		60	"	1.4 x 3.0	5	1.5	6.61									
															.550	108	17.0		83	T-Q	1.4 x 3.0	1	0.5	2.30	0.7	3.0							
							Remolded to approx. 95% Std. AASHO max.								.544	109	16.9		84	"	1.4 x 2.9	2	1.5	3.54									
							density at optimum + 3%								.541	109	16.0		81	"	1.4 x 3.0	3	3.0	5.45									
															.540	108	16.9		81	"	1.4 x 3.0	4	6.0	8.45									
															.540	109	13.7	18.0	68	T-R	1.4 x 3.0	1	1.5	4.30	0.4	20.8							
							Remolded to approx. 95% Std. AASHO max.								.559	108	14.1	17.4	68	"	1.4 x 3.0	2	3.0	2.50									
							density at optimum water content								.543	109	13.4	16.1	66	"	1.4 x 3.0	3	6.0	13.81									
															.547	109	17.0	17.5	84	T-R	1.4 x 3.0	1	1.5	4.48	0.4	19.7							
							Remolded to approx. 95% Std. AASHO max.								.550	108	16.9	16.7	83	"	1.4 x 3.0	2	3.0	7.12									
							density at optimum + 3%								.552	108	16.9	16.1	83	"	1.4 x 2.9	3	6.0	13.37									
															.552	108	14.3	16.5	70	DS-S	3 x 3 x 0.5	1			0.2	33.6							
							Remolded to approx. 95% Std. AASHO max.								.552	108	14.2	16.0	69	"	"	2											
							density at optimum water content								.553	108	14.3	14.6	70	"	"	3											
															.555	108	17.0	16.6	83	DS-S	3 x 3 x 0.5	1			0.2	31.5							
							Remolded to approx. 95% Std. AASHO max.								.555	108	17.1	16.0	83	"	"	2											
							density at optimum + 3%								.555	108	17.0	15.0	82	"	"	3											

T - Triaxial Compression; UC - Unconfined Compression; DS - Direct Shear; Q UU - Unconsolidated Undrained; R CU - Consolidated Undrained; Consolidated Drained; * Void ratio at Pressure; T/SQ FT

TEST DATA SUMMARY

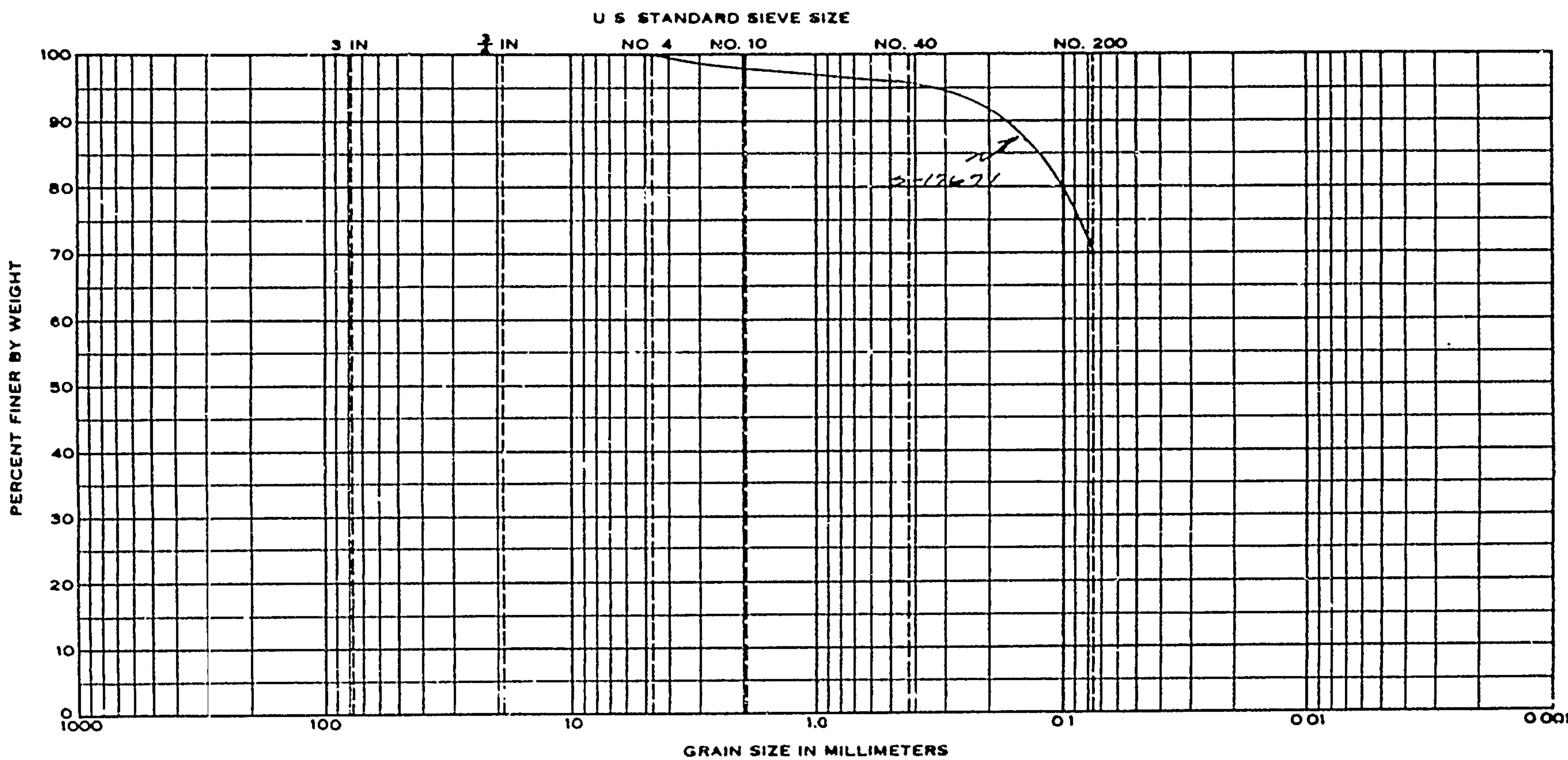
SWDGL Report No. 8558 — FEATURE: BORROW MATERIAL — Gillham Dam — Sheet 2 of 2

Boring No.	Samp. No.	Depth or Elev of Sample	Laboratory Classification	Gravel %	Sand %	Fines %	D_{10}	LL	PL	Specific Gravity G	Nat Water Cont %	Natural Dry Density lb/cu ft	Opt Water %	Maximum Dry Density lb/cu ft	Initial e	Dry Density lb/cu ft	w_o %	w_f %	S_i %	Type Test	Specimen Size Inches	Test	σ_3 T/sq ft	σ_1 T/sq ft	c T/sq ft	φ Degrees	e	k ft/min	P_o T/sq ft	P_c T/sq ft	C_c	t_{50}	Remarks
														Location		Borrow	Area C			(TPC-14)													
5-17-72		2.0-6.0	CL	0	14	86		34	17	2.69			16.6	108.4																			LS-7
				CLAY											.623	103	16.1		69	T-Q	1.4 x 3.0	1	0.5	3.30	0.8	17.0							
							Remolded to approx 95% Std AASHO max.								.618	104	16.4		71	"	1.4 x 3.0	2	1.5	6.32									
							density at optimum water content								.615	104	16.2		71	"	1.4 x 3.0	3	3.0	8.91									
															.624	104	16.3		70	"	1.4 x 3.0	4	6.0	13.48									
															.622	103	12.8		86	T-Q	1.4 x 3.0	1	0.5	2.08	0.7	19							
							Remolded to approx 95% Std AASHO max.								.616	104	13.0		86	"	1.4 x 3.0	2	1.5	5.18									
							density at optimum + 3%								.623	104	12.9		86	"	1.4 x 3.0	3	3.0	4.65									
															.629	103	12.7		85	"	1.4 x 2.9	4	6.0	7.94									
															.632	103	16.6	20.5	71	T-R	1.4 x 3.0	1	1.5	4.14	0.2	22.2							
							Remolded to approx 95% Std AASHO max.								.621	104	16.5	19.6	71	"	1.4 x 3.0	2	3.0	7.63									
							density at optimum water content								.623	104	16.4	18.4	71	"	1.4 x 3.0	3	6.0	13.34									
															.637	103	20.0	20.0	84	T-R	1.4 x 3.0	1	1.5	4.27	0.2	22.5							
							Remolded to approx 95% Std AASHO max.								.632	103	20.0	19.4	85	"	1.4 x 3.0	2	3.0	8.09									
							density at optimum + 3%								.625	103	20.0	18.4	86	"	1.4 x 2.9	3	6.0	14.25									
															.624	103	16.8	19.6	73	DS-S	3 x 3 x 0.5	1			0.2	30.4							
							Remolded to approx 95% Std AASHO max.								.631	103	16.8	19.2	72	"	"	2											
							density at optimum water content								.639	103	16.9	18.1	71	"	"	3											
															.630	103	19.7	19.2	84	DS-S	3 x 3 x 0.5	1			0.1	30.8							
							Remolded to approx 95% Std AASHO max.								.630	103	19.9	18.0	85	"	"	2											
							density at optimum + 3%								.628	103	19.6	16.5	85	"	"	3											

ENG FORM 2086 (PREVIOUS EDITIONS MAY BE USED)

T - Triaxial Compression; UC - Unconfined Compression; DS - Direct Shear; Q -UU- Unconsolidated Undrained; R -CU- Consolidated Undrained; Consolidated Drained; *Values at Pressure T/SQ FT

SWDGL Report No. 8458



COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No.	~~Elev or~~ Depth	Classification	NatWC	LL	PL	PI	LS
S-12671	1.5-4.0	Sandy CLAY (CL)		29	13	16	8

Project Gillham Dam

BORROW MATERIAL

Area Borrow Area B

Boring No. TPB-8

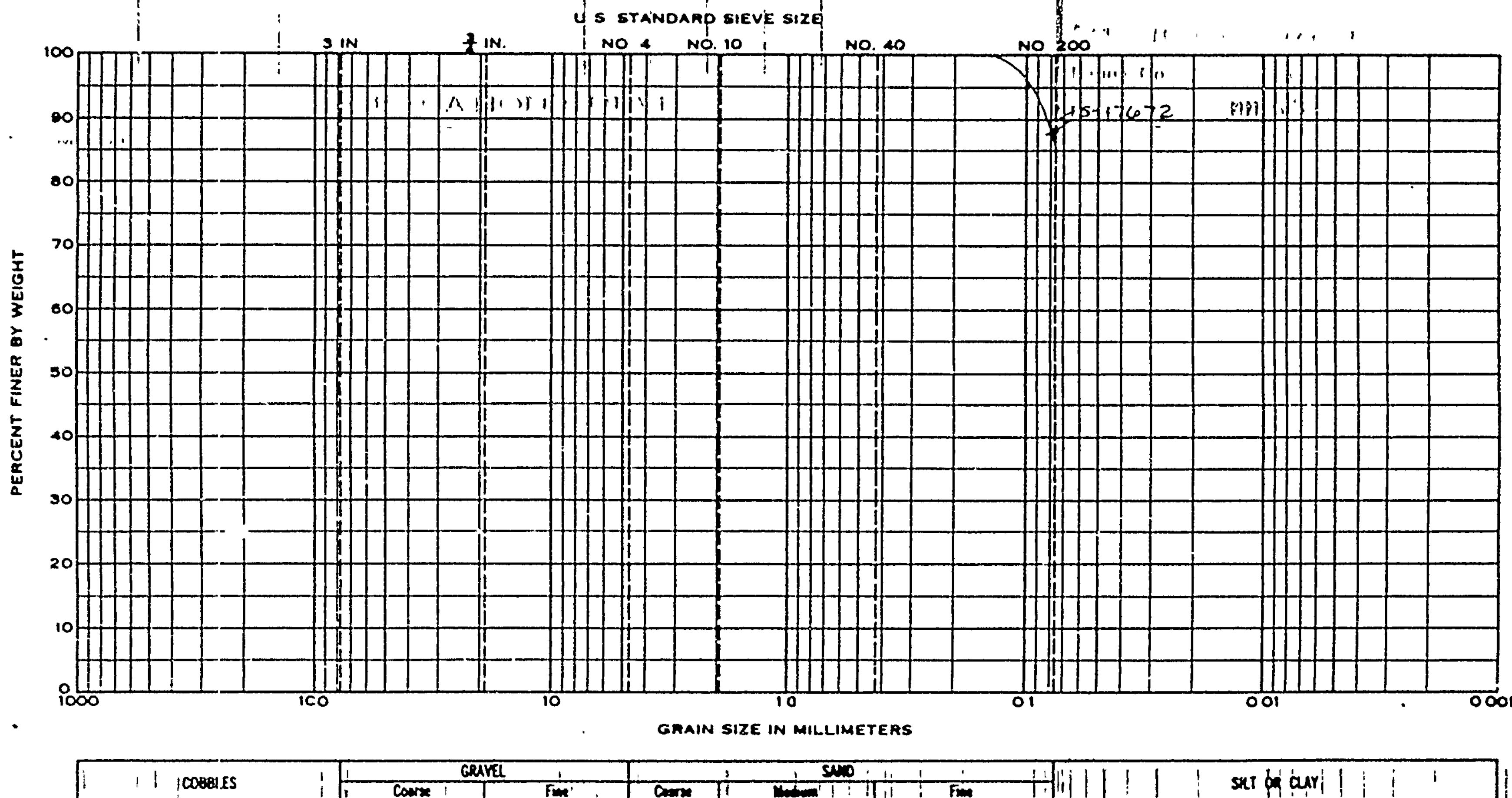
Date JUL 63

GRADATION CURVES

ENG FORM 1 OCT 51 2067 (Transluncent) (EM 1110-2-1803)

Plate No. 1

SWDGL Report No. 8858



Sample No.	~~Elev or~~ Depth	Classification	NatWC	LL	PL	PI	LS
S-17672	2.0-6.0	CLAY (CL)		34	17	17	7

Project Gillham Dam

BORROW MATERIAL

Area Borrow Area C

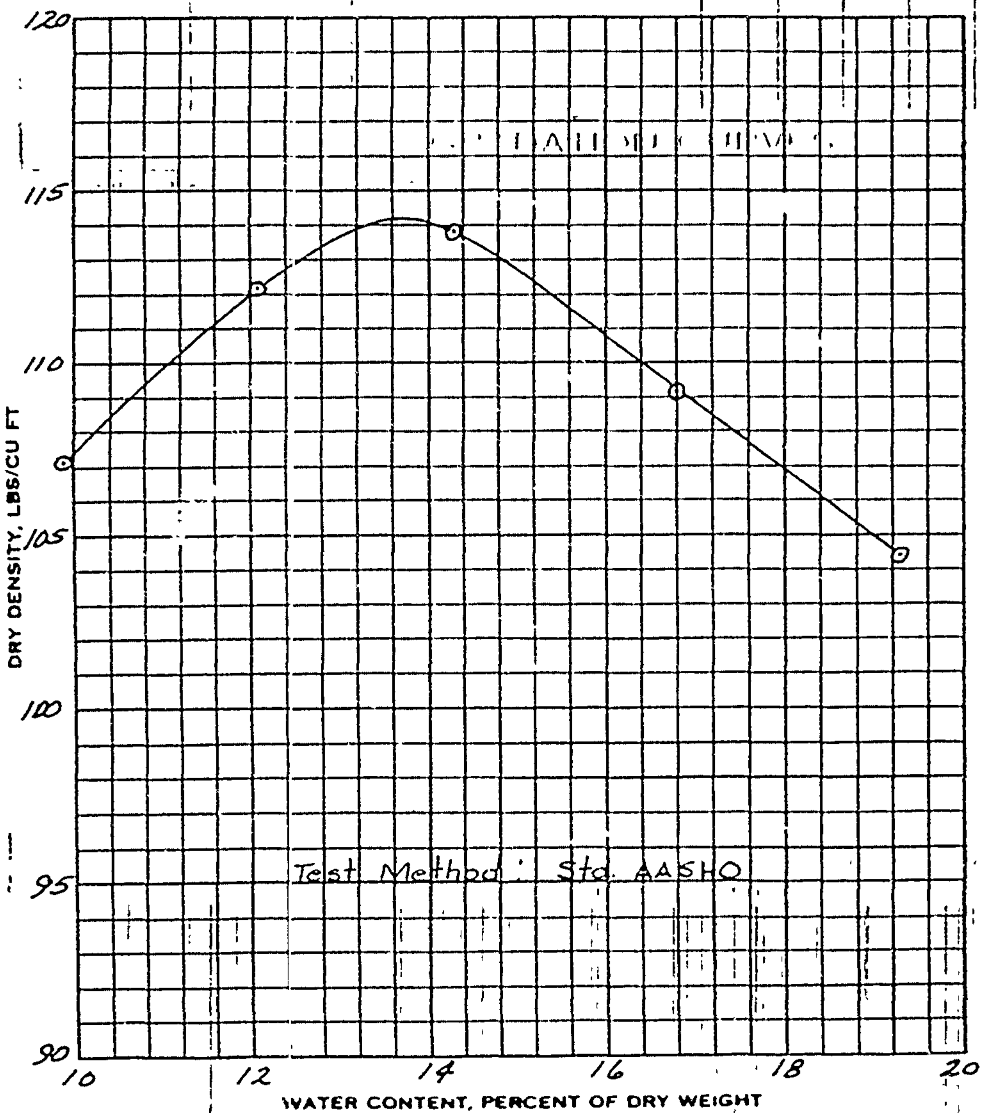
Boring No. THC 14

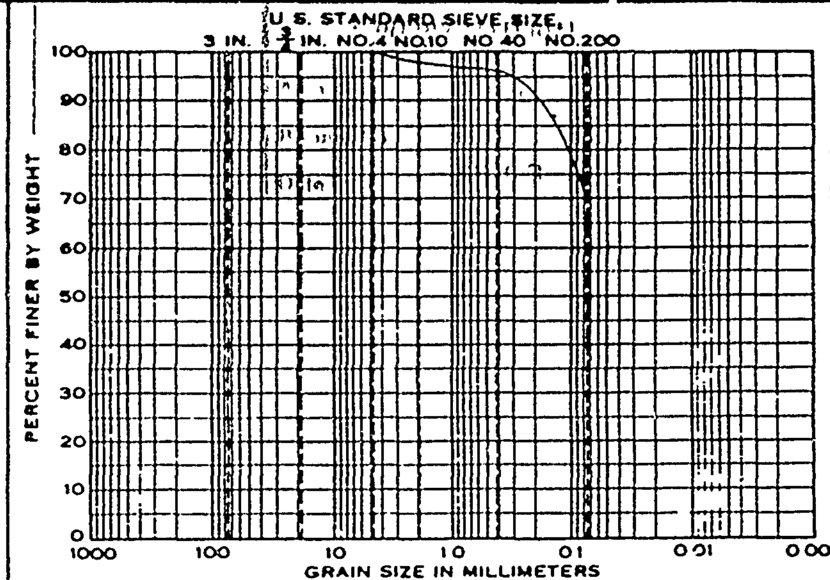
Date JUL 63

GRADATION CURVES

ENG FORM 2087 (Translucent) (EM 1110-2-1803)

Plate No. 2





COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

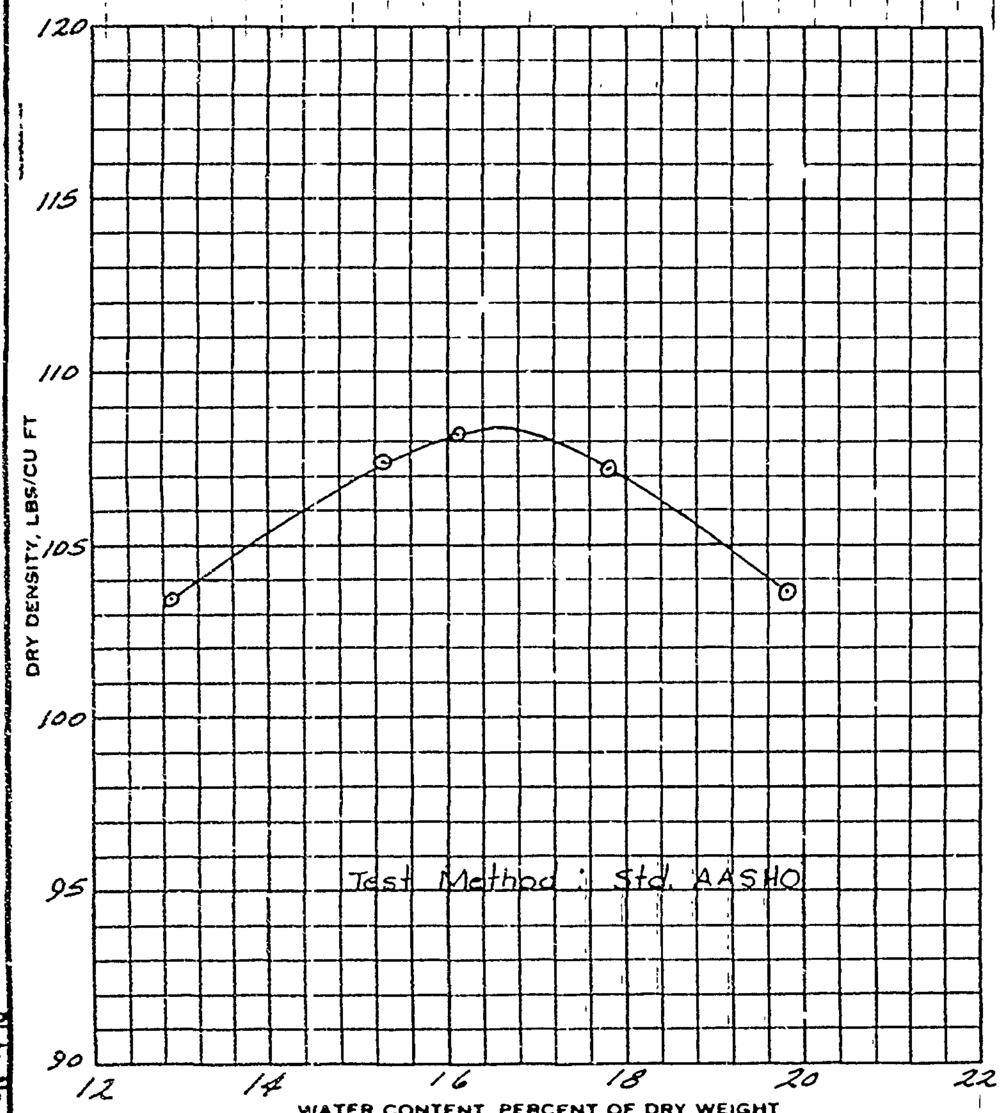
Sample No	Elev or Depth	Classification	G	LL	PL
		Sandy CLAY (CL)	2.69	29	13

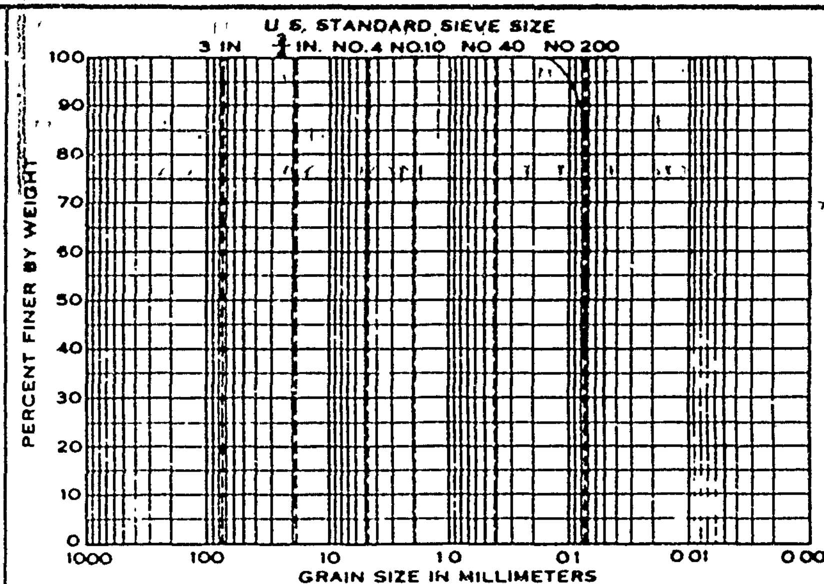
Sample No.				
Optimum Water Content %	13.7			
Max Dry Density Lbs/Cu Ft	114.2			
Optimum Water Content Corr for + %				
Max Density Corr for + Lbs/Cu Ft				

Project	Gillham Dam
	BORROW MATERIAL
Area	Borrow Area B
Boring No. TPB-8	Sample No. S-17671
Elev or Depth 1.5 - 4.0	Date JUL 63

COMPACTION TEST REPORT

SWDGL Report No. 8458





COBBLES	GRAVEL		SAND			SILT OR CLAY
	Coarse	Fine	Coarse	Medium	Fine	

Sample No	Elev or Depth	Classification	G	LL	PL
		CLAY (CL)	2.69	34	17

Sample No.				
Optimum Water Content %	16.6			
Max Dry Density Lbs/Cu Ft	108.4			
Optimum Water Content Corr for + %				
Max Density Corr for + Lbs/Cu Ft				

Project: Gillham Dam

BORROW MATERIAL

Area: Borrow Area C

Boring No. TPC - 14 | Sample No. S-17672

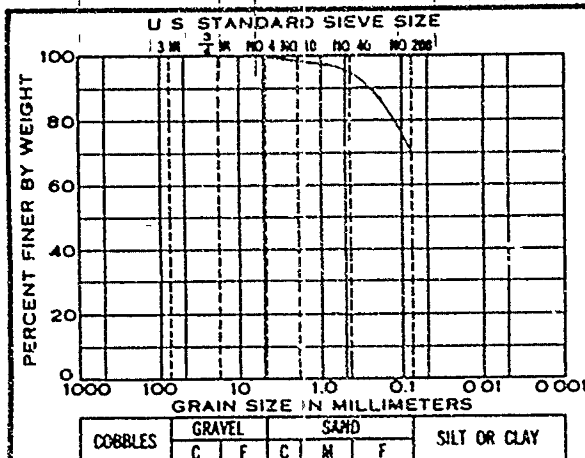
Elev or Depth 2.0 - 6.0 | Date JUL 69

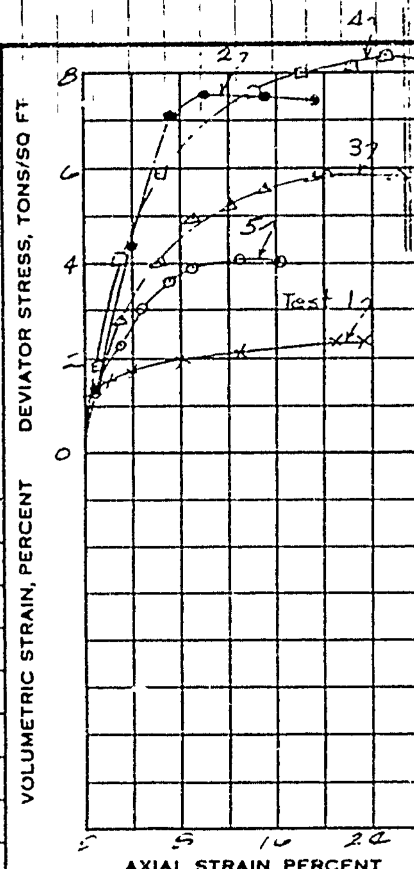
COMPACTION TEST REPORT

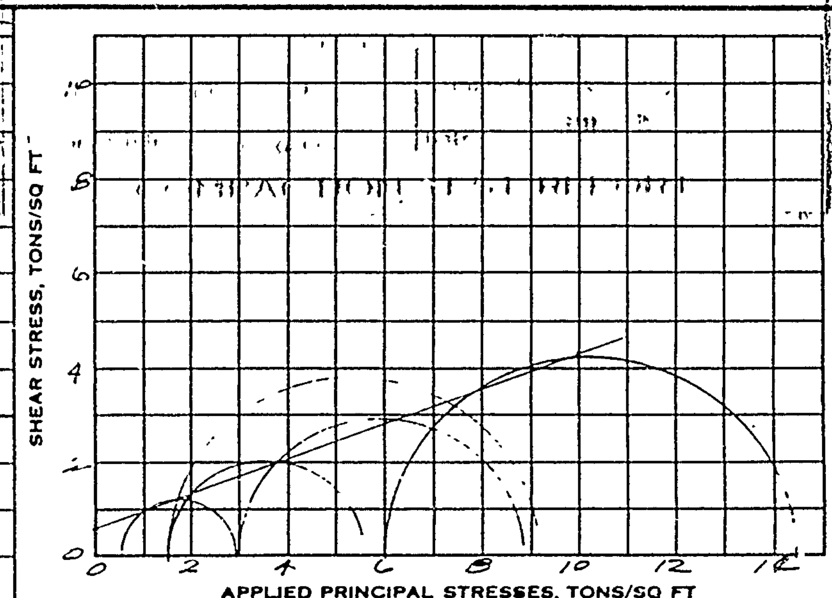
Plate No. 4

ED FORM 2081

SWDGL Report No. 8458







	Test No.	1	2	3	4
INITIAL	Water Content, W_o %	13.6	12.6	13.9	13.7
	Dry Density Lbs/Cu Ft	108	109	109	109
	Void Ratio, e_o	.555	.545	.541	.541
	Saturation, S_o %	67	68	69	68
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/Sq Ft				
	W.C. after Consol, W_c %				
	Void Ratio after Consol e_c				
AT FAILURE	Maj Prin Stress, σ_I T/Sq Ft	2.90	9.09	8.84	14.36
	Min Prin Stress, σ_{III} T/Sq Ft	0.5	1.5	3.0	6.0
	Water Content, W_f %				
	Void Ratio, e_f				
	Specimen Diameter Inches	1.4	1.4	1.4	1.4
	Initial Height In	3.0	3.0	3.0	3.0
	Test Time to Failure Min	21	11	20	25

Type of Test -Q
Constant Strain 0.03 in/min Control
Un Consolidated, Un Drained

Type of Specimen REMOLDED

ϕ = 20.4°	Tan ϕ = .373	c = 0.6 T/Sq Ft

Classification Sandy CLAY (CL)

LL 30	G 2.69
PL 13	D_{10}

Remarks:

Test No. 5, W_o = 13.6, γ = 109, e_o = .544, S_o = 68, T = 5.61, σ_{III} = 1.5, Dia. inch = 1.4, height 3.0, Test time to failure 13'.

Project Gillham Dam

BORROW MATERIAL

Area

Boring No. TPE-8

Sample No. S-17671 (95-8, w)

~~Elev or~~ Depth 1.5-4.0

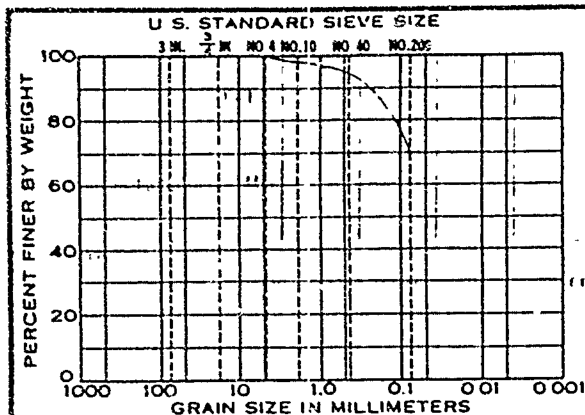
Date JUL 68

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 NOV 51 2080 (EM 1110-2-1903) (Translucent)

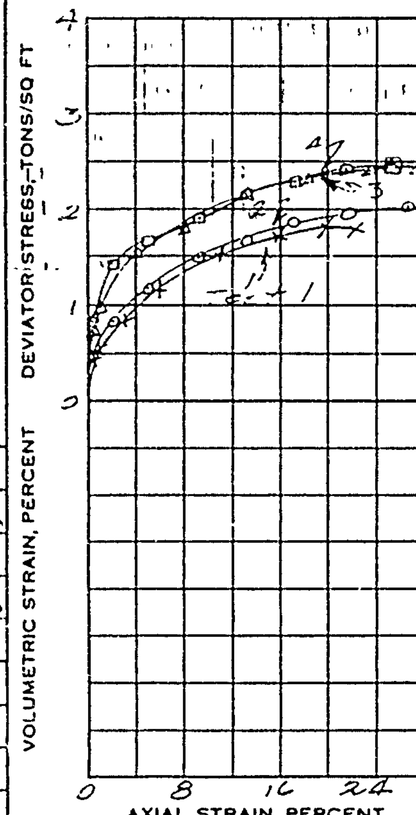
Plate No. 5

TRIAXIAL COMPRESSION TEST REPORT



COBBLES	GRAVEL		SAND			SILT OR CLAY
	C	F	C	M	F	

		1	2	3	4
	Test No.	1	2	3	4
INITIAL	Water Content, W_0 %	17.0	14.9	14.8	14.9
	Dry Density Lbs/Cu Ft	06	109	108	108
	Void Ratio, e_0	.580	.544	.561	.562
	Saturation, S_0 %	83	84	81	81
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/SqFt				
	W.C. after Consol, W_c %				
	Void Ratio after Consol e_c				
AT FAILURE	Maj Prin Stress, σ_1 T/SqFt	2.30	3.54	5.44	8.45
	Min Prin Stress, σ_m T/SqFt	0.5	1.1	3.0	6.0
	Water Content, W_f %				
	Void Ratio, e_f				
	Specimen Diameter Inches	1.4	1.4	1.4	1.4
	Initial Height In	3.0	2.9	3.0	3.0
	Test Time to Failure Min	20	26	25	25



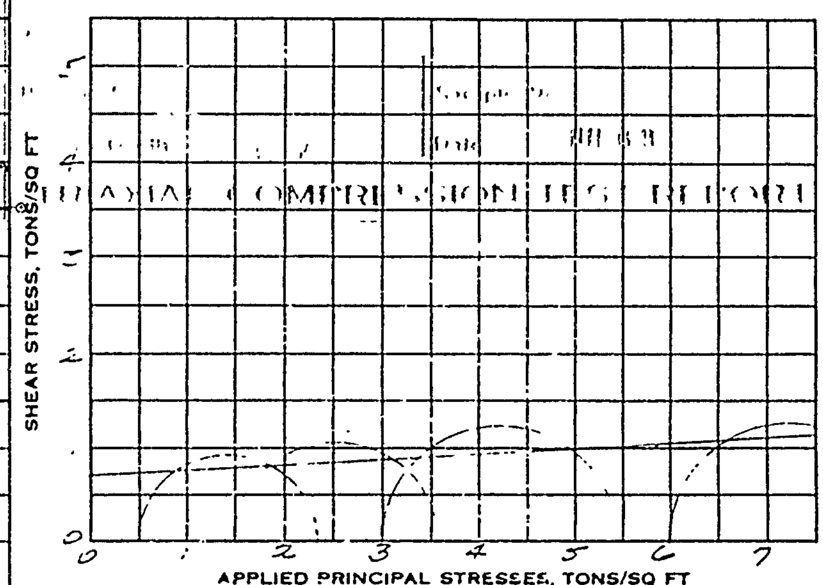
Type of Test -Q
Constant Strain 0.03 in/min Control
Un Consolidated, Un Drained

Type of Specimen REMOLDED

ϕ = 3.0°	Tan ϕ = .052	c = 0.7 T/Sq Ft

Classification Sandy CLAY (CL)

LL	29	G	2.69
PL	13	D_{10}	



Remarks:

Project Gillham Dam
BORROW MATERIAL

Area

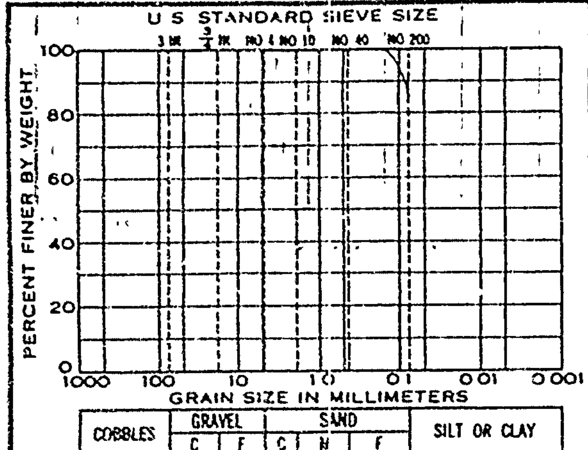
Boring No. T2B-8
Sample No. S-17621 (95 Y, W+3)

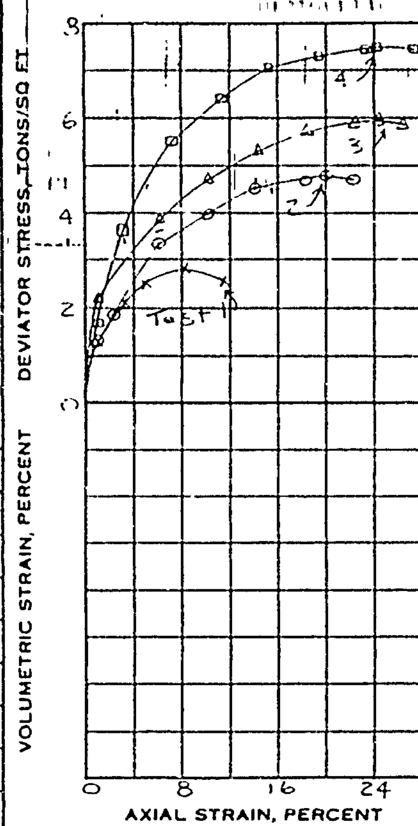
~~Elev or~~ Depth 1.5-4.0
Date JUL 68

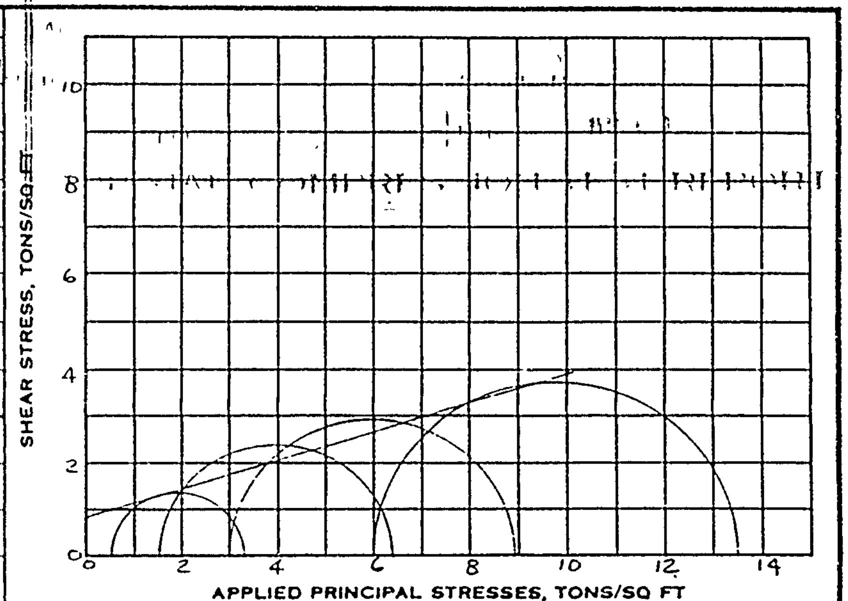
Plate No. 6

ENG FORM 1 MAY 63 2089 (EM 1110-2-1803) (Translucent)

SWDGL Report No. 8458







Remarks:

Test No.		1	2	3	4
INITIAL	Water Content, W_o %	16.1	16.4	16.2	16.3
	Dry Density Lbs/Cu Ft	103	104	104	104
	Void Ratio, e_o	.623	.618	.615	.624
	Saturation, S_o %	69	71	71	70
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/SqFt				
	W.C. after Consol, W_c %				
	Void Ratio after Consol, e_c				
AT FAILURE	Max Prin Stress, σ_1 T/Sq Ft	3.30	6.39	8.91	13.48
	Min Prin Stress, σ_m T/Sq Ft	0.5	1.5	3.0	6.0
	Water Content, W_f %				
	Void Ratio, e_f				
Specimen Diameter	Inches	1.4	1.4	1.4	1.4
Initial Height	In	3.0	3.0	3.0	3.0
Test Time to Failure	Min	8	20	24	24

Type of Test – Q
Constant Strain, 0.03 in/min
Control
UnConsolidated, UnDrained

Type of Specimen REMOLDED

ϕ = 17.0° | Tan ϕ = .306 | c = 0.8 T/Sq Ft

Classification CLAY (CL)

LL 34 | G 2.69
PL 17 | D_{10}

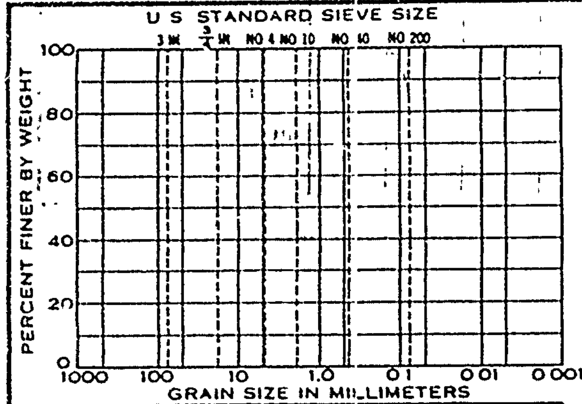
Project Gillham Dam
BORROW MATERIAL
Area
Boring No. TPC-14 | Sample No. (958, H) S-17672
~~Elev or~~ Depth 2.0-6.0 | Date JUL 63

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089 (EM 1110-2-1803) (Translucent)

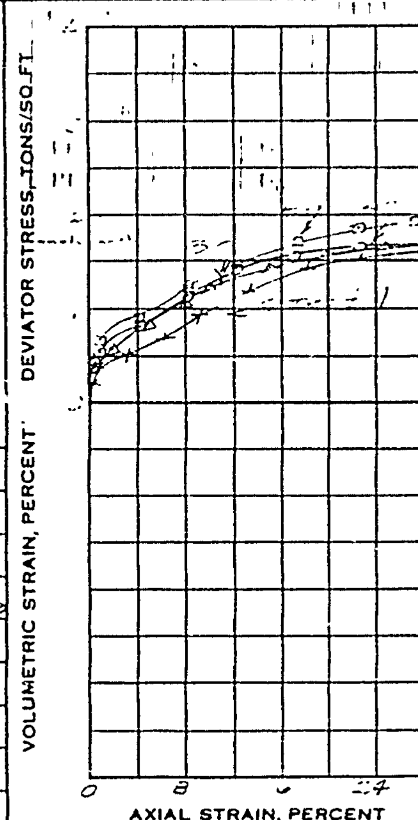
Plate No. 7

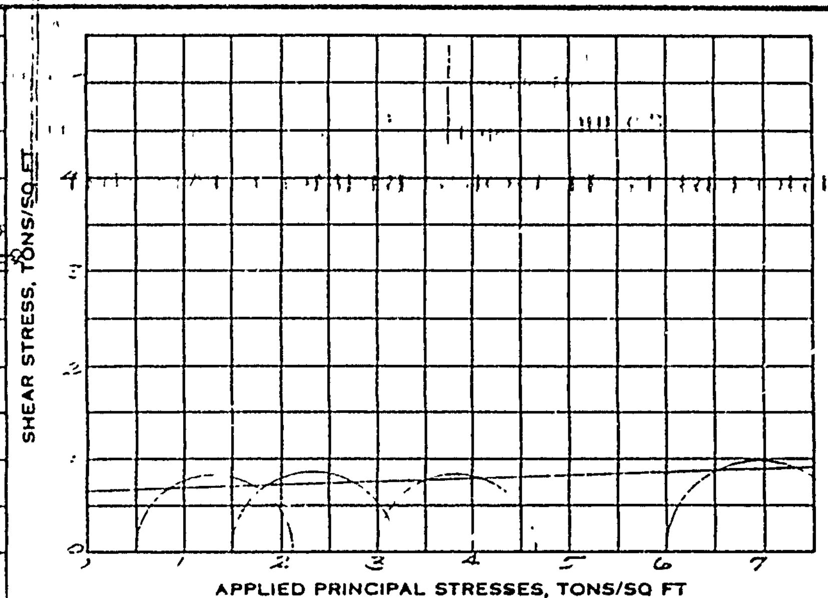
SWDQL Report No. 84-8



COBBLES	GRAVEL		SAND			SILT OR CLAY
	C	F	C	M	F	

	Test No.	1	2	3	4
INITIAL	Water Content, W_o %	[illegible]	[illegible]	[illegible]	[illegible]
	Dry Density Lbs/Cu Ft	[illegible]	104	107	[illegible]
	Void Ratio, e_o	[illegible]	[illegible]	[illegible]	[illegible]
	Saturation, S_o %	[illegible]	[illegible]	[illegible]	[illegible]
BEFORE TEST	W.C. after Saturation, W_s %				
	Saturation, S %				
	Consol Pressure T/Sq Ft				
	W.C. after Consol, W_c %				
	Void Ratio after Consol e_c				
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	[illegible]	[illegible]	[illegible]	[illegible]
	Min Prin Stress, σ_{III} T/Sq Ft	[illegible]	[illegible]	[illegible]	[illegible]
	Water Content, W_f %				
	Void Ratio, e_f				
	Specimen Diameter Inches	[illegible]	[illegible]	[illegible]	[illegible]
	Initial Height Inc	[illegible]	[illegible]	[illegible]	[illegible]
	Test Time to Failure Min	[illegible]	[illegible]	[illegible]	[illegible]





Remarks:

Type of Test - [illegible]
Constant [illegible] Control
[illegible] Consolidated, un Drained

Type of Specimen REMOLDED

ϕ = 19°	Tan ϕ = [illegible]	c = [illegible] T/Sq Ft

Classification CLAY (CL)

LL [illegible]	G [illegible]
PL [illegible]	D_{10}

Project [illegible] Dam
BORROW MATERIAL
Area
Boring No. [illegible]
Sample No. S-17672 (9: 8, w+3)
Elev or Depth [illegible]
Date JUL 68

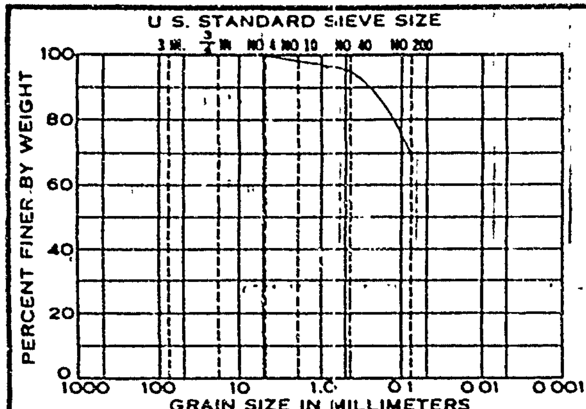
TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 2089

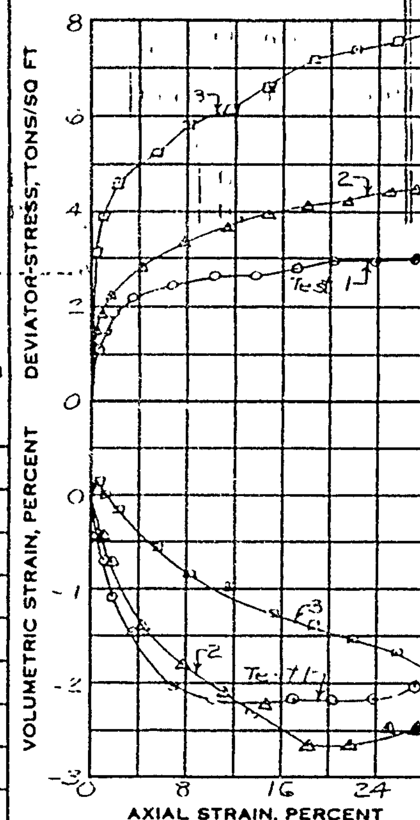
(EM 1110-2-1805)

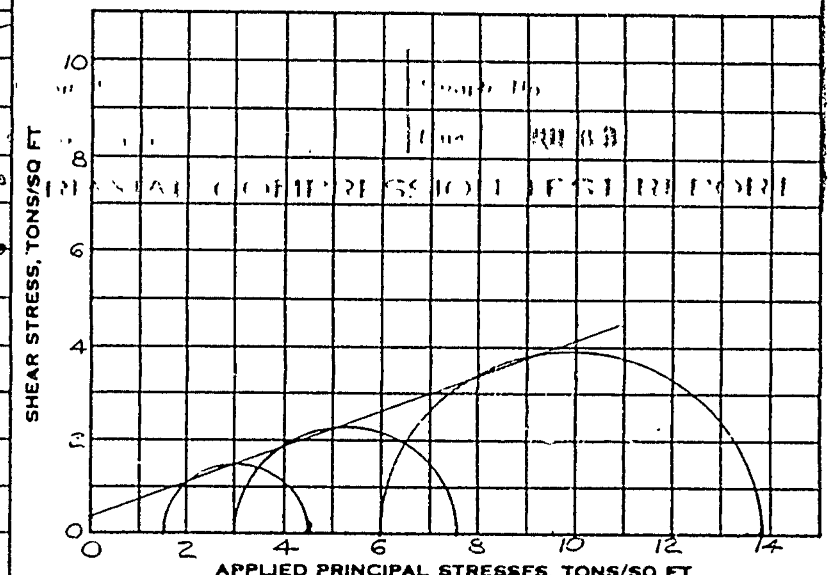
(Transparent)

SWDGL Report No. 8458



COBBLES	GRAVEL C	GRAVEL F	SAND C	SAND M	SAND F	SILT OR CLAY





	Test No.	1	2	3	
INITIAL	Water Content, W_0 %	13.7	4 [illegible]	13.4	
	Dry Density Lbs/Cu Ft	[illegible]	1[illegible]8	[illegible]9	
	Void Ratio, e_0	.540	.559	.543	
	Saturation, S_0 %	68	69	[illegible]	
BEFORE TEST	W.C. after Saturation, W_s %	[illegible]	18.5	[illegible]	
	Saturation, S %	[illegible]	[illegible]	94	
	Consol Pressure T/SqFt	1.5	[illegible]	6.0	
	W.C. after Consol, W_c %	[illegible]	[illegible]	16.1	
	Void Ratio after Consol e_c	[illegible]	.519	.492	
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	4.5[illegible]	7.[illegible]	13.81	
	Min Prin Stress, σ_{III} T/Sq Ft	1.5	3.0	6.0	
	Water Content, W_f %	18.[illegible]	17.4	16.1	
	Void Ratio, e_f	.435	.45[illegible]	.463	
	Specimen Diameter Inches	1.4	1.4	1.4	
	Initial Height Inc	3.0	3.0	3.0	
	Test Time to Failure Min	27	26	28	

Type of Test – R
Constant Strain, 0.03 in/min
Control
X Consolidated, Un Drained

Type of Specimen **REMOLDED**

ϕ = 20.8°	Tan ϕ = .381	c = 0.4 T/Sq Ft

Classification Sandy CLAY (CL)

LL 29	G 2.65
PL 13	D_{10}

Remarks:

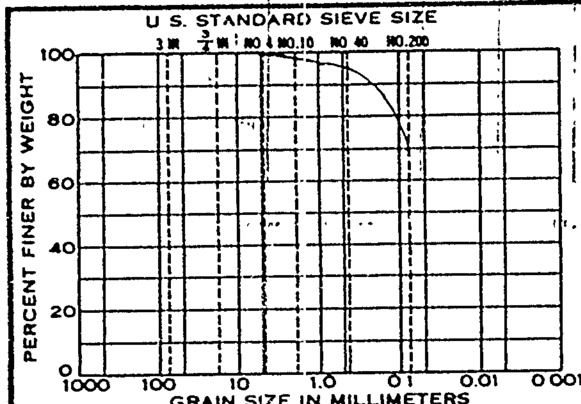
Project Gillham Dam
BORROW MATERIAL
Area
Boring No. TPB-8
Sample No. S-17671 (95 % W)
~~Elev or~~ Depth 1.5-4.0
Date JUL 63

TRIAXIAL COMPRESSION TEST REPORT

ENG FORM 1 JUL 51 2089 (EM 1110-2-1803) (Translucent)

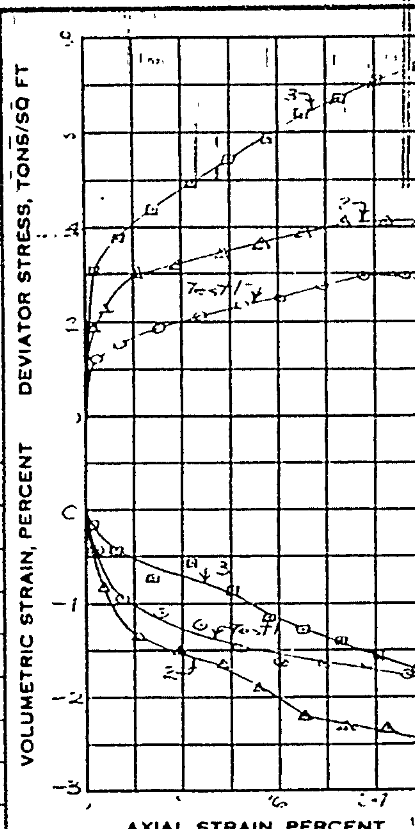
Plate No. 9

TRIAXIAL COMPRESSION TEST REPORT



COBBLES	GRAVEL C	GRAVEL F	SAND C	SAND M	SAND F	SILT OR CLAY

	Test No.	1	2	3
INITIAL	Water Content, W_o %	17.0	16	16.5
	Dry Density Lbs/Cu Ft	109	[illegible]	101
	Void Ratio, e_o	5-1	.550	.552
	Saturation, S_o %	[illegible]	[illegible]	82
BEFORE TEST	W.C. after Saturation, W_s %	[illegible]	[illegible]	[illegible]
	Saturation, S %	[illegible]	[illegible]	97
	Consol Pressure T/Sq Ft	[illegible]		6.0
	W.C. after Consol, W_c %	[illegible]	[illegible]	[illegible]
	Void Ratio after Consol e_c	.116	[illegible]	[illegible]
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	4.4	[illegible]	13.37
	Min Prin Stress, σ_{III} T/Sq Ft	1.5	[illegible]	[illegible]
	Water Content, W_f %	[illegible]	[illegible]	[illegible]
	Void Ratio, e_f	4.30	[illegible]	[illegible]
	Specimen Diameter Inches	1.4	1.4	1.4
	Initial Height In	3.0	[illegible]	[illegible]
	Test Time to Failure Min	[illegible]	[illegible]	[illegible]



Type of Test - R

Constant Strain

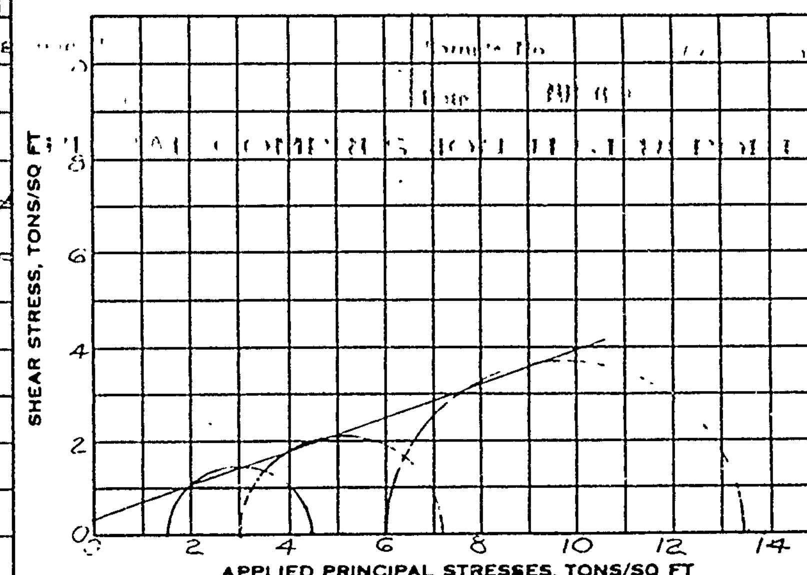
Control

X Consolidated, UnDrained

Type of Specimen **REMOLDED**

ϕ = 19.7 °	Tan ϕ = .359	c = 0.4 T/Sq Ft

Classification	CLAY (CL)
LL 29	G 2.70
PL 13	D_{fil}



Remarks:

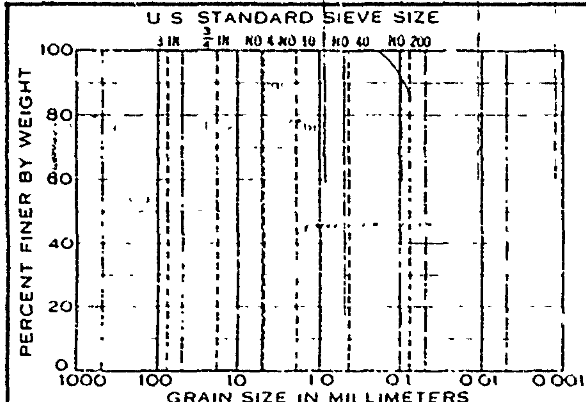
Project	Gillham Dam
	BORROW MATERIAL
Area	
Boring No. TPB-8	Sample No. S-17,671 (958W43)
~~Elev or~~ Depth 1.5-4.0	Date JUL 63

ENG FORM 1 MC 51 2089

(EM 1110-2-1803)

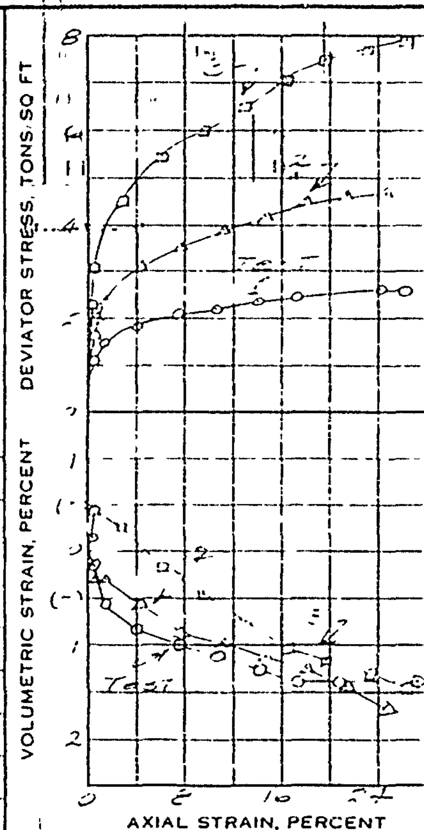
(Transclucent)

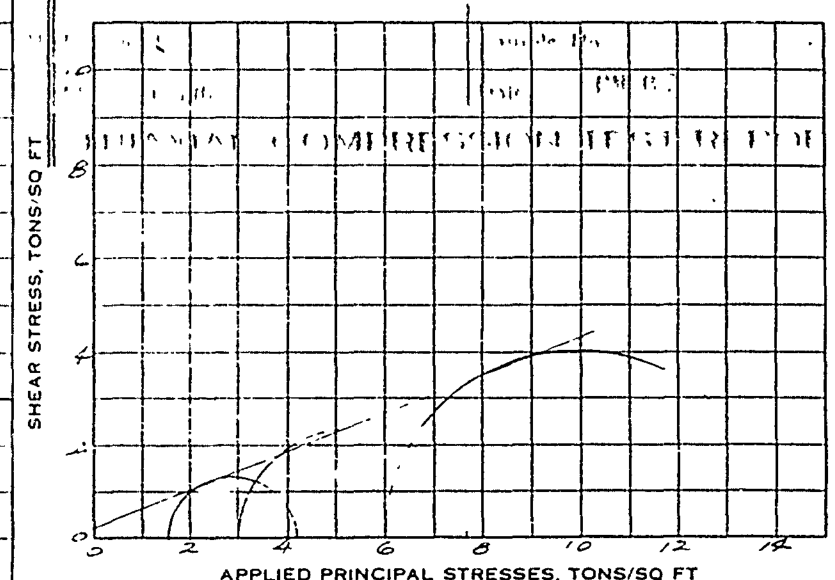
TRIAXIAL COMPRESSION TEST REPORT



COBBLES	GRAVEL		SAND			SILT OR CLAY
	C	F	C	M	F	

	Test No	1	2	3
INITIAL	Water Content, W_o %	16.6	16.5	16.4
	Dry Density Lbs/Cu Ft	103	104	104
	Void Ratio, e_o	.632	.621	.623
	Saturation, S_o %	71	71	71
BEFORE TEST	W.C. after Saturation W_s %	22.[illegible]	22.5	22.8
	Saturation, S %	[illegible]	96	98
	Consol Pressure T/Sq Ft	1.5	3.0	6.2
	W.C after Consol. W_c %	20.[illegible]	19.6	[illegible]
	Void Ratio after Consol e_c	[illegible]	.551?	.504
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	[illegible]	[illegible]	[illegible]
	Min Prin Stress, σ_{3} T/Sq Ft	[illegible]	[illegible]	[illegible]
	Water Content, W_f %	[illegible]	[illegible]	[illegible]
	Void Ratio, e_f	[illegible]	[illegible]	.482?
	Specimen Diameter Inches	[illegible]	1.4	[illegible]
	Initial Height Inc	[illegible]	[illegible]	[illegible]
	Test Time to Failure Min	[illegible]	[illegible]	[illegible]





Remarks:

Type of Test: R
Constant strain 0.03 in/min Control
✓ Consolidated, Un Drained

Type of Specimen: **REMOLDED**

Ø = 22 ? °	Tan Ø = .40?	c = 0.2 T/Sq Ft

Classification: CLAY (CL)

LL [illegible]	G 2.[illegible]
PL [illegible]	D_{10}

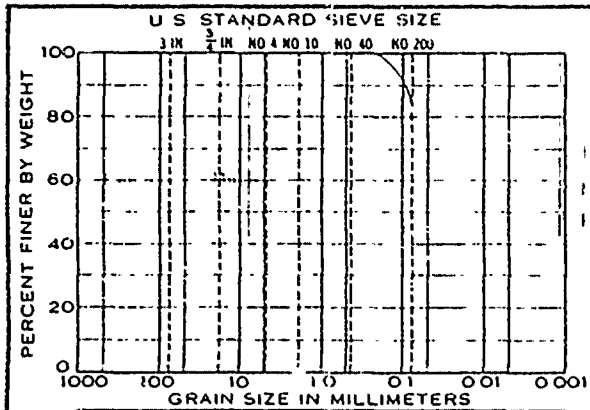
Project: Gilliam? Dam
BORROW MATERIAL

Area

Boring No. [illegible] 14	Sample No. S-12672 (95%, w)
Elev or Depth [illegible]	Date **JUL 63**

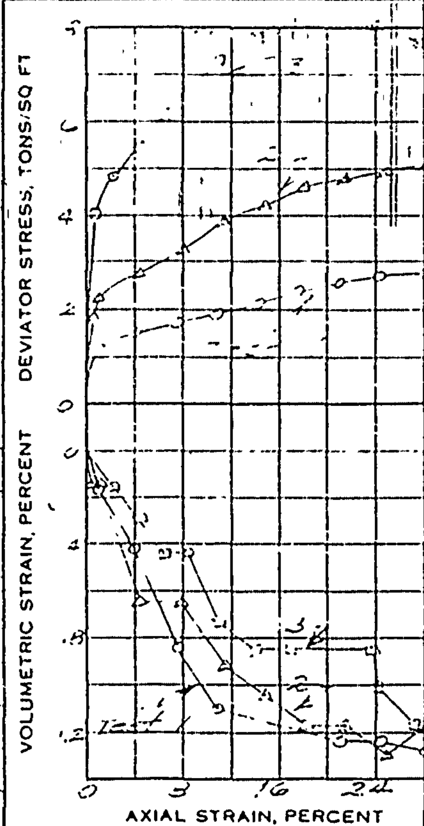
TRIAXIAL COMPRESSION TEST REPORT

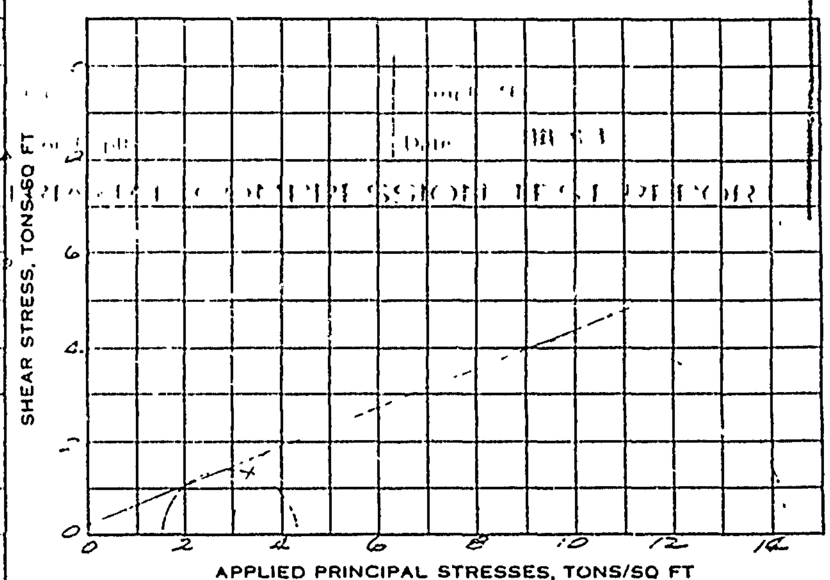
2089



COBBLES	GRAVEL		SAND			SILT OR CLAY
	C	F	C	M	F	

	Test No	[illegible]		[illegible]	
INITIAL	Water Content, W_0 %	[illegible]	[illegible]	[illegible]	
	Dry Density Lbs/Cu Ft	[illegible]	[illegible]	103	
	Void Ratio, e_0	[illegible]	[illegible]	[illegible]	
	Saturation, S_0 %	94	[illegible]	86	
BEFORE TEST	W.C. after Saturation, W_s %	[illegible]	[illegible]	[illegible]	
	Saturation, S %	[illegible]	95	94	
	Consol Pressure T/Sq Ft	[illegible]	3.0	[illegible]	
	W C after Consol, W_c %	[illegible]	19.4	[illegible]	
	Void Ratio after Consol e_c	[illegible]	[illegible]	[illegible]	
AT FAILURE	Maj Prin Stress, σ_1 T/Sq Ft	[illegible]	[illegible]	4.25	
	Min Prin Stress, σ_m T/Sq Ft	[illegible]	3.2	[illegible]	
	Water Content, W_f %	[illegible]	[illegible]	[illegible]	
	Void Ratio, e_f	[illegible]	[illegible]	.515	
Specimen Diameter	Inches	1.4	[illegible]	1.4	
Initial Height	Inc	[illegible]	[illegible]	2.9	
Test Time to Failure	Min	[illegible]	[illegible]	[illegible]	





Remarks:

Type of Test - R
Constant Strain [illegible] Control
x Consolidated, Un Drained

Type of Specimen **REMOLDED**

ϕ = [illegible]° | Tan ϕ = .415 | c = [illegible] T/Sq Ft

Classification CLAY (CL)

LL 34 | G 2.[illegible]

PL [illegible] | D_{10}

Project [illegible] Lake
BORROW MATERIAL

Area

Boring No. TPC 14 | Sample No. (95 8, w+3) S-12672

~~Elev or~~ Depth [illegible] | Date JUL 63

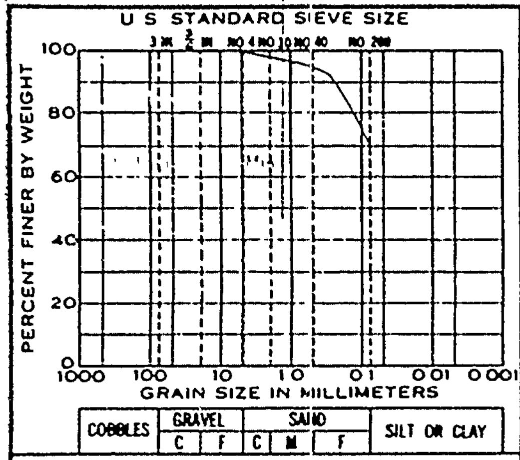
TRIAXIAL COMPRESSION TEST REPORT

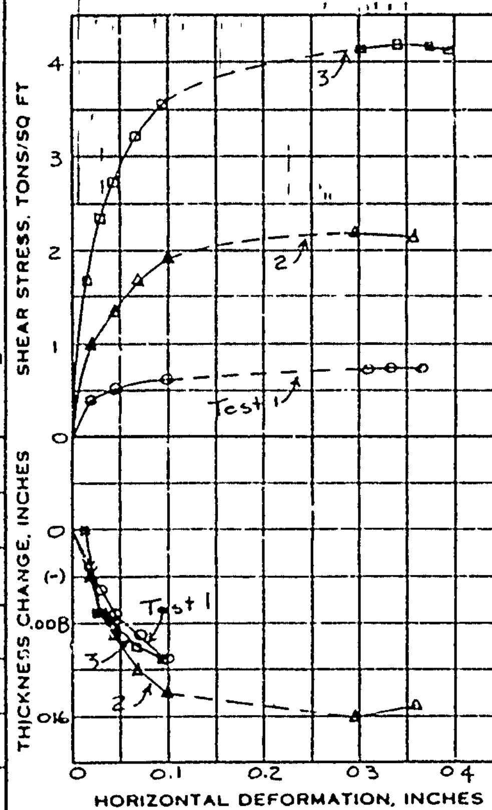
SWDGL Report No. 8456

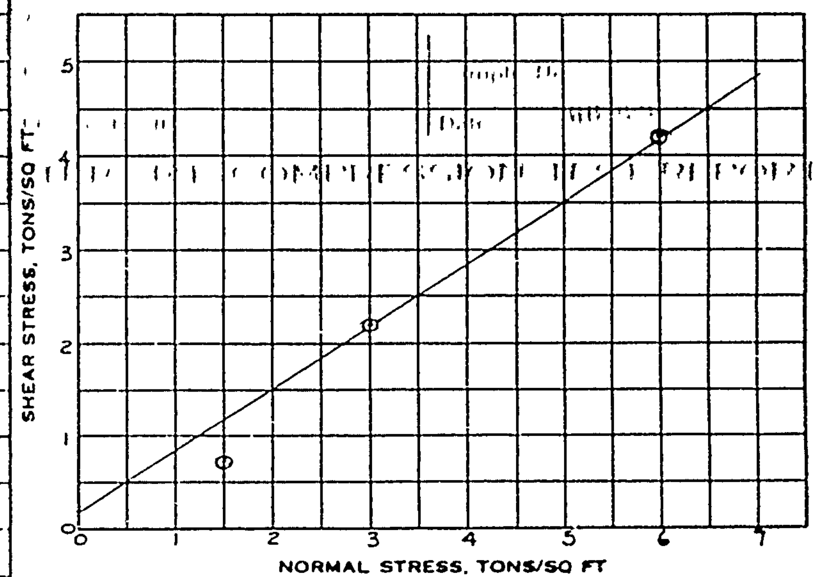
Plate No. 12

ENG 2089

SWDGL Report No. 8438







Classification	Sandy CLAY (CL)		
LL 29	G 2.69		
PL 13	D_{10}		
Test No.	1	2	3
Initial Water Content, W_o %	14.3	14.2	14.3
Dry Density Lbs/Cu Ft	108	108	108
Void Ratio, e_o	.552	.552	.553
Saturation, S_o %	70	69	70
Duration of Test (To T')	1500'	1500'	1600'
WC after Test, W_f %	16.5	15.0	14.6
Normal Stress, σ T/SqFt	1.5	3.0	6.0
Max. Shear Stress, τ T/SqFt	0.74	2.19	4.18
t_{50}, min	–	–	<1

Type of Test - S
Strain - Control
Rate: 0.0002 in/min.
X Consolidated
X Drained

Type of Specimen: REMOLDED
3.0 in. square
0.5 in. thickness

SHEAR VALUES	Ø	TAN Ø	C, T/SQ FT
Maximum	33.6	.664	0.2
Ultimate			

Remarks:

Project: Gillham Dam
BORROW MATERIAL
Area

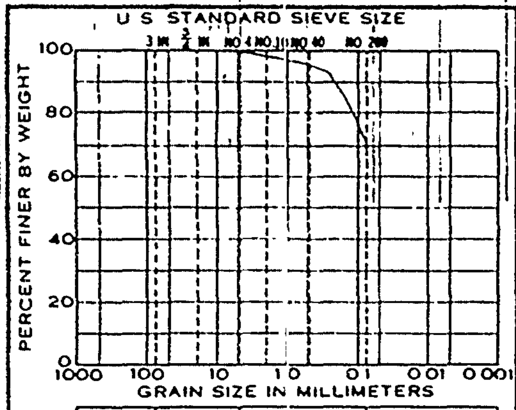
Boring No. TPB-8 | Sample No. S-17671 (958,ω)
~~Elev or~~ Depth 1.5-4.0 | Date JUL 63

DIRECT SHEAR TEST REPORT

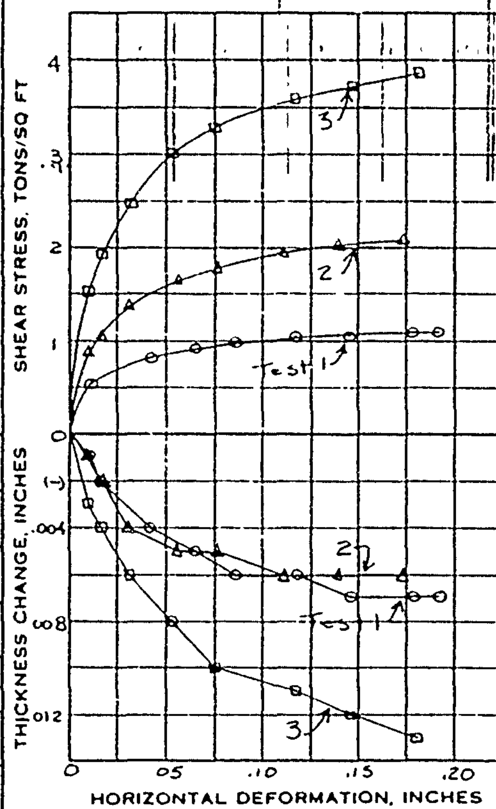
2092

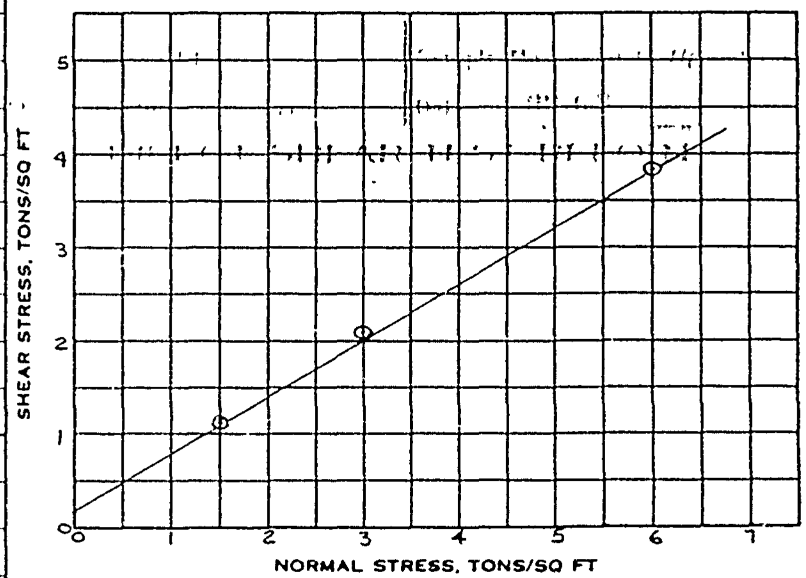
DIRECT SHEAR TEST REPORT

SWDGL Report No. 8458



COBBLES	GRAVEL		SAND			SILT OR CLAY
	C	F	C	M	F	





Remarks:

Classification	Sandy CLAY (CL)			
LL 29	G 2.69			
PL 13	D_s			
Test No.	1	2	3	
Initial Water Content, W_o %	17.0	17.1	17.0	
Dry Density Lbs/Cu Ft	108	108	108	
Void Ratio, e_o	.555	.555	.555	
Saturation, S_o %	83	83	82	
Duration of Test (T_o T)	445'	480'	480'	
WC after Test, W_f %	16.6	16.0	15.0	
Normal Stress, σ T/Sq Ft	1.5	3.0	6.0	
Max. Shear Stress, τ T/Sq Ft	1.09	2.08	3.85	
t_{50}, min	–	–	<1	

Type of Test - S	Type of Specimen
Strain - Control Rate = 0.0004 in/min.	REMOLDED
X Consolidated	3.0 in. square
X Drained	0.5 in. thickness

SHEAR VALUES	φ	TAN φ	C, T/SQ FT
Maximum	31.5	.613	0.2
Ultimate			

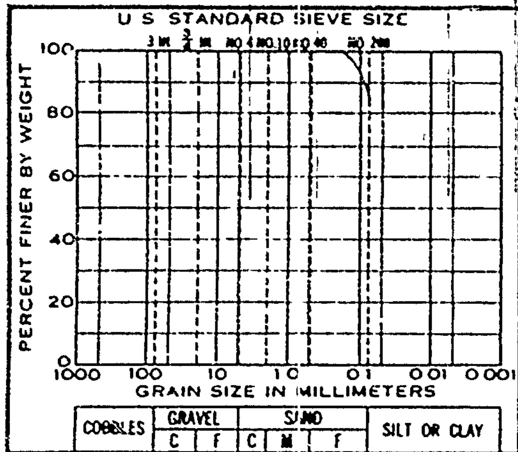
Project Gillham Dam BORROW MATERIAL	
Area	
Boring No. TPB-8	Sample No. S-17671(95%,w+3)
~~Elev or~~ Depth 1.5-4.0	Date JUL 68

ENG FORM 2002

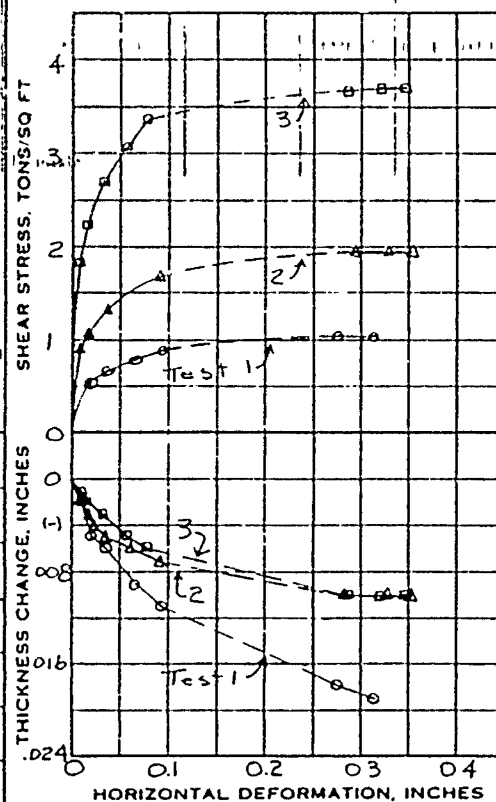
Plate No. 14

SWDGL Report No. 8458

DIRECT SHEAR TEST REPORT



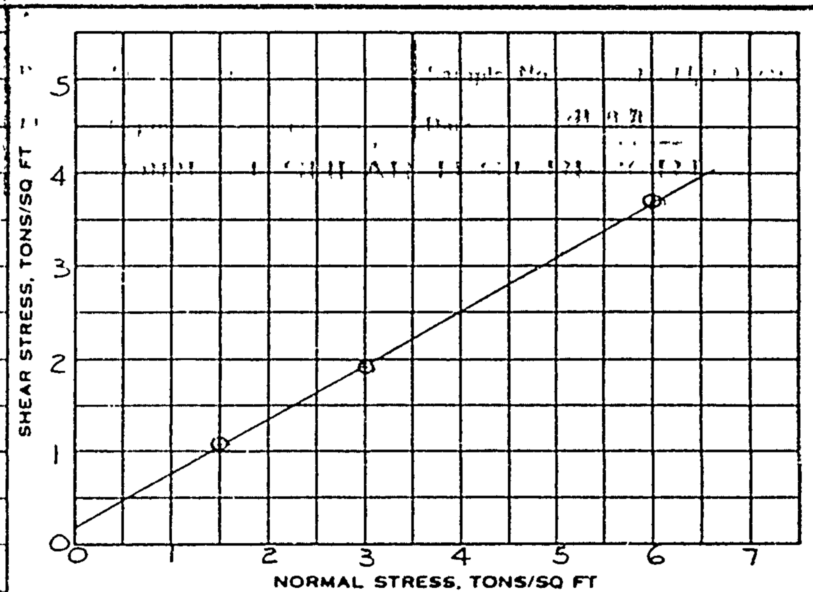
Classification	CLAY (CL)		
LL 34	G 2.69		
PL 17	D_H		
Test No.	1	2	3
Initial Water Content, W_o %	16.8	16.8	16.9
Dry Density Lbs/CuFt	103	103	103
Void Ratio, e_o	.624	.631	.639
Saturation, S_o %	73	72	71
Duration of Test (T_o τ)	1400'	1500'	1600'
WC after Test, W_f %	19.6	19.2	18.1
Normal Stress, σ T/SqFt	1.5	3.0	6.0
Max. Shear Stress, τ T/SqFt	1.05	1.92	3.69
t_{50}, min	—	—	<1



Type of Test - S
Strain-Control
Rate = 0.0002 in/min
X Consolidated
X Drained

Type of Specimen
REMOLDED
3.0 in. square
0.5 in. thickness

SHEAR VALUES	Ø	TAN Ø	C, T/SQFT
Maximum	30.4	.587	0.2
Ultimate			

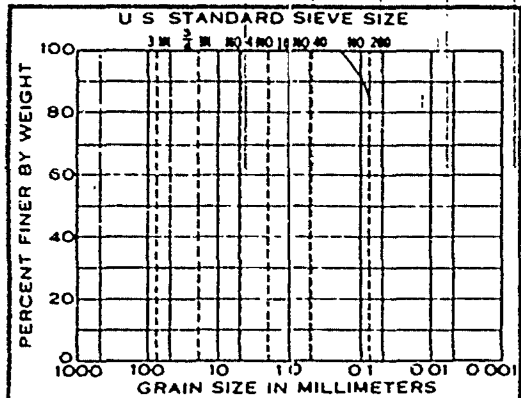


Remarks.

Project Gillham Dam
BORROW MATERIAL
Area
Boring No. TPC-14
Sample No. S-17672 (95% w)
~~Elev or~~ Depth 2.0 - 6.0
Date JUL 68

ENG 2002

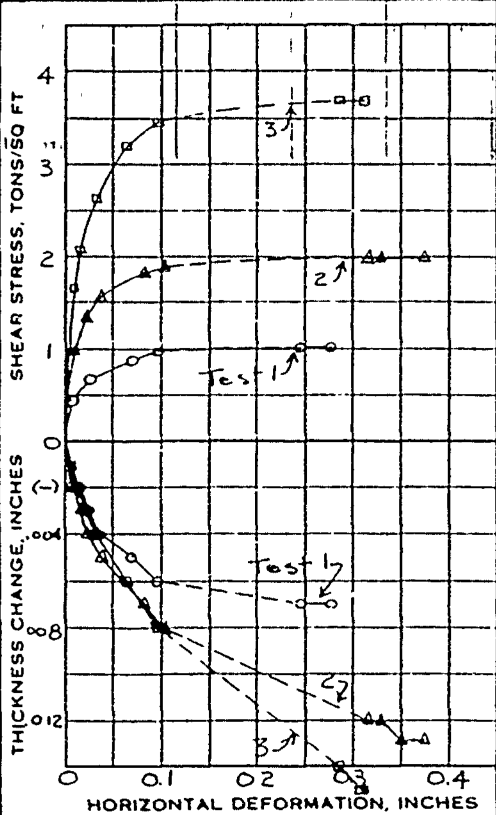
Plate No. 15



COBBLES	GRAVEL C	GRAVEL F	SAND C	SAND M	SAND F	SILT OR CLAY

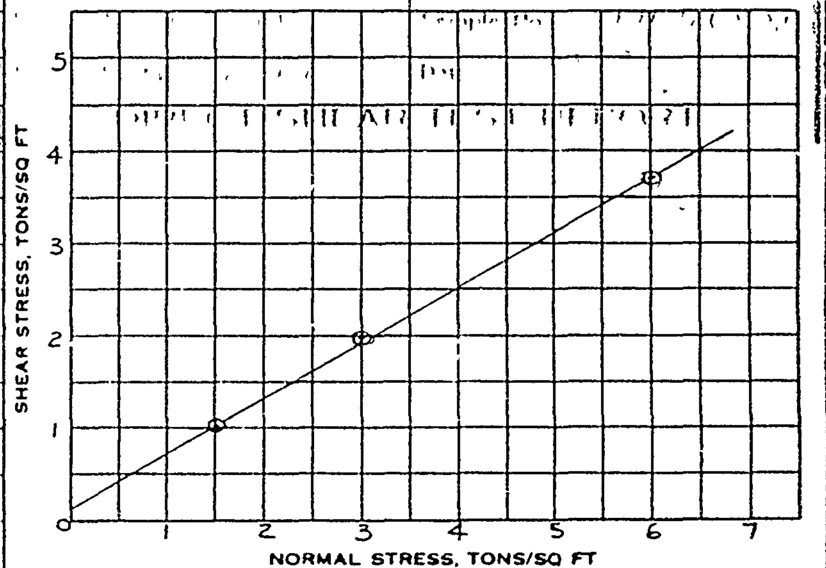
Classification	CLAY (CL)		
LL 34		G 2.69	
PL 17		D_{10}	

Test No.	1	2	3
Initial Water Content, W_0 %	19.7	19.9	19.8
Dry Density Lbs/Cu Ft	103	103	103
Void Ratio, e_0	.630	.630	.628
Saturation, S_0 %	84	85	85
Duration of Test (To τ)	1400	1500	1500
WC after Test, W_f %	19.2	18.0	16.5
Normal Stress, σ T/Sq Ft	1.5	3.0	6.0
Max. Shear Stress, τ T/Sq Ft	1.01	1.99	3.70
t_{50}, min	–	–	<1



Type of Test - S	Type of Specimen
Strain – Control	REMOLDED
Rate = 0.0002 in/min.	3.0 in. square
X Consolidated	0.5 in. thickness
X Drained	

SHEAR VALUES	ϕ	TAN ϕ	C, T/SQ FT
Maximum	30.8	.597	0.1
Ultimate			



Remarks:

Project Gillham Dam	
BORROW MATERIAL	
Area	
Boring No. TPC-14	Sample No. S-17672 (95, wt3)
~~Elev or~~ Depth 2.0-6.0	Date JUL 63

DIRECT SHEAR TEST REPORT

ENG 2092

APPENDIX F

SWDGL REPORT NO. 8473

RESULTS OF TESTS OF ROCK,

ROCK FILL, RIPRAP AND CONCRETE AGGREGATE

SWDGL REPORT NO. 8473

RESULTS OF TESTS OF ROCK

ROCK FILL, RIPRAP AND CONCRETE AGGREGATE

GILLHAM DAM - TULSA DISTRICT

CORPS OF ENGINEERS

U. S. ARMY

SOUTHWESTERN DIVISION LABORATORY

DALLAS, TEXAS

REQUEST FOR AND RESULTS OF TESTS

PAGE NO. 1 | NO. OF PAGES 5

A SWFGL REQUEST FOR TEST TU-FM-63-22 dated 13 May 1963

1. TO:
Southwestern Division Laboratory
US Army Engineer Division, Southwestern
Corps of Engineers
4815 Cass Street
Dallas, Texas 75235

2. FROM:
Chief, F&M Branch
US Army Engineer District, Tulsa
Tulsa, Oklahoma

3. PRIME CONTRACTOR AND ADDRESS

CONTRACT NUMBER

4. MANUFACTURING PLANT NAME AND ADDRESS

P O NUMBER

5. END ITEM AND/OR PROJECT: Gillham Dam

6. SAMPLE NUMBER

7. LOT NO

8. REASON FOR SUBMITTAL: Tests for use as rock fill, riprap and conc. agg.

9. DATE SUBMITTED

10. MATERIAL TO BE TESTED: Rock Core and Quarry Rock

10a. QUANTITY SUBMITTED

11. QUANTITY REPRESENTED

12. SPEC. & AMEND AND/OR DRAWING NO. & REV. FOR SAMPLE & DATE

13. PURCHASED FROM OR SOURCE

14. SHIPMENT METHOD: Gov't Vehicle

15. DATE SAMPLED AND SUBMITTED BY

16. REMARKS AND/OR SPECIAL INSTRUCTIONS AND/OR WAIVERS

17 RECEIPT

THE ABOVE MATERIAL HAS BEEN RECEIVED

DATE	TYPED NAME AND TITLE	SIGNATURE
9 May 63	L. A. PARK, Engineer	

3 RESULTS OF TEST (Continue on plain white paper if more space is required)

1 RESULTS ☐ ACCEPTABLE ☐ FAILURE INDICATED ☐ OTHER (Specify)

2. DATE REC'D: 9 May 63

3 DATE REPORTED: 22 July 63

4. LAB REPORT NUMBER: SWDGL-8473

5 TEST PERFORMED, RESULTS OF TEST SAMPLE RESULT REQUIREMENTS

See following pages.

6 SEND REPORT OF TEST TO

~~Tulsa District O.~~

7 THE PRESCRIBED TESTS WERE MADE ON 9 May - 15 July 1963 AND THE RESULTS THEREOF WERE AS SET FORTH ~~ABOVE~~ herein.

DATE	TYPED NAME AND TITLE ~~OF PERSON CONDUCTING TEST~~	SIGNATURE
22 July 63	K. M. COKE Engineer-in-Charge SWD Laboratory	

DD FORM 1 JUL 58 1222 REPLACES DA FORMS 896, 1651 AND 10-208 WHICH ARE OBSOLETE

SWDGL Report No. 8473 22 July 1963

Results of Tests of Quarry Rock, Investigation for Fill Material
Gillham Dam - Tulsa District

1. Reference Test Request No. TU-FM-63-22 dated 13 May 1963 from the Chief, Foundations and Materials Branch, Tulsa District.

2. The following materials were received 9 May 1963:

SWD Sample No.	Material	Field Hole No.	Depth, feet	Quantity
C-19765 A	2-1/8" Core, Sandstone	Q - 6	88.5 to 89.5	-
B	"	"	98.0 to 98.7	-
C	"	"	102.1 to 103.1	-
D	"	"	152.0 to 153.0	-
E	"	Q - 7	94.6 to 95.6	-
F	"	"	131.4 to 132.8	-
G	"	"	135.3 to 136.1	-
H	"	"	150.0 to 150.8	-
I	"	"	178.8 to 179.8	-
J	"	Q - 9	95.9 to 96.6	-
K	"	"	105.2 to 106.6	-
L	"	"	111.0 to 112.5	-
M	2-1/8" Core, Shale	Q - 7	104.9 to 105.4	-
N	"	"	109.4 to 109.9*	-
C-19766	2-1/8" Core, Shale	Q - 10	10.0 to 20.0	75 lbs
C-19767	Quarry Sandstone	-	-	4000 lbs

*Test request listed 108.4 - 109.4. The sample received was marked as shown.

3. The requested tests have been completed and results summarized as follows:

Results of Physical Tests of 2-1/8" Core	Table 1
Results of Tests of Riprap Material	Table 2 and Plate 1

4. The sample of quarry sandstone is being retained for additional tests as might be requested by Tulsa District.

SWDGL Report No. 8473 TABLE NO. 1 (Revised) Gillham Dam

Results of Tests of 2-1/8 Inch Rock Core Specimens

SWD Sample No.	Field Hole No.	Depth to Depth Feet	Compressive Strength, psi	Other Tests	Description
C-19765 A	Q - 6	88.5 - 89.5	32300		QUARTZITIC SANDSTONE, gray, hard, fine grained. Contains occasional hair line shale seams and fractures well healed with calcite and quartz.
B	"	98.0 - 98.7	28200		
C	"	102.1 - 103.1	45100		
D	"	152.0 - 153.0	35500	Specific gravity SSD, 2.65 Absorption, 0.3%	
C-19765 E	Q - 7	94.0 - 95.6	34000		QUARTZITIC SANDSTONE, as above.
F	"	131.4 - 132.8	17400		
G	"	135.3 - 136.1	25900		
H	"	150.0 - 150.8	26000		
I	"	178.8 - 179.8	20300		
C-19765 J	Q - 9	95.9 - 96.0	9400	Specific gravity SSD, 2.64 Absorption, 0.4%	QUARTZITIC SANDSTONE, dark gray, fine grained, hard.
K	"	105.2 - 106.6	15200		QUARTZITIC SANDSTONE, gray, fine grained, hard.
L	"	111.0 - 112.5	23800		
C-19765 M	Q - 7	104.9 - 105.4	7000	-	SHALE, black, with sandstone bands 1/2" to 1" thick at intervals of 1/2" - 3".
N	"	109.4 - 109.9	4800		
C-19766	Q - 10	10.0 - 20.0	-	Abrasion, L.A., % Loss(1) Representative sample as received - 41.3; All Residue from slaking tests - 39.2; Slaking Test, see note (2)	SHALE, SANDY SHALE and SANDSTONE mixed core..

NOTES: (1) Test conducted on 10,000 gram sample of approximately 3" to 2" lengths of core subjected to 1000 revolutions with 12 balls. Residue predominantly sandy shale.

(2) Representative pieces of the core were soaked in water 16 hours and exposed to laboratory air 8 hours for each of 10 cycles. No measurements were made but photographs before and after were taken and are shown on Plate 1.

SWDGL Report No. 8473 TABLE NO. 2 Gillham Dam

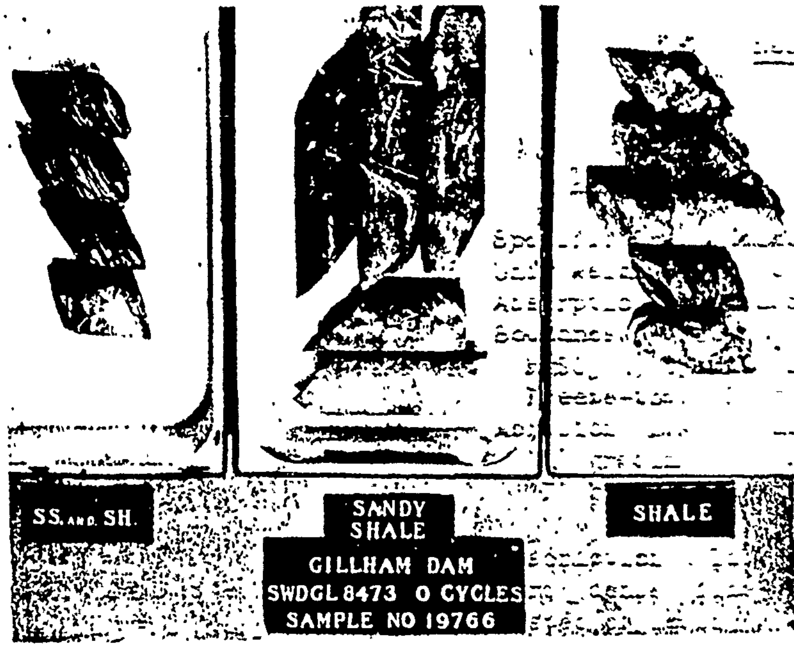
Results of Tests of Sandstone Chunk Sample

SWD Sample No. C-19707

Tests	Results
Specific Gravity, Bulk SSD (crushed to 1" - 3/8")	2.63
Unit Weight, lbs/cu.ft. (calculated from SSD sp. gr.)	164.1
Absorption, % (crushed to 1" - 3/8")	0.8
Soundness, % loss:	
$MgSO_4$, 5 cycles, 2-1/2" - 1-1/2" fraction	1.1
Freeze-thaw, 25 cycles, 2-1/2" - 1-1/2" fraction	1.1
Abrasion, L.A., % loss	
"A" grading	22.1
"E" grading	15.4

Description: This sample of ledge rock consisted of quartzitic SANDSTONE, gray, hard, dense, fine-grained, weathered and slightly fractured. The weathering consisted of leaching and ferruginous staining to a depth of 1/16" to 5", located on bedding planes and fractures. The staining appears to be in two distinct bands with the innermost being darker and harder due to it being a zone of iron concentration from the lighter leached band above it; this tends to have a slight case hardening effect on the rock. These bands appear on almost all chunks of any size and are of a uniform thickness for each chunk, although some irregularity was noted in the thicker bands. The fractures are hairline to 1/8", tight to well-healed with quartz. A small amount of talc-like material was noted on several broken fracture planes. Approximately 9 percent of the sample had chunks with lengths greater than three times their thickness. The maximum size chunk received was 2.0 x 1.2 x 1.2, and the average size was 1.1 x 0.7 x 0.6.

Photograph of Sample as Received



Before Test

After Test

SLAKING TEST

Plate 1

APPENDIX G

PLATES

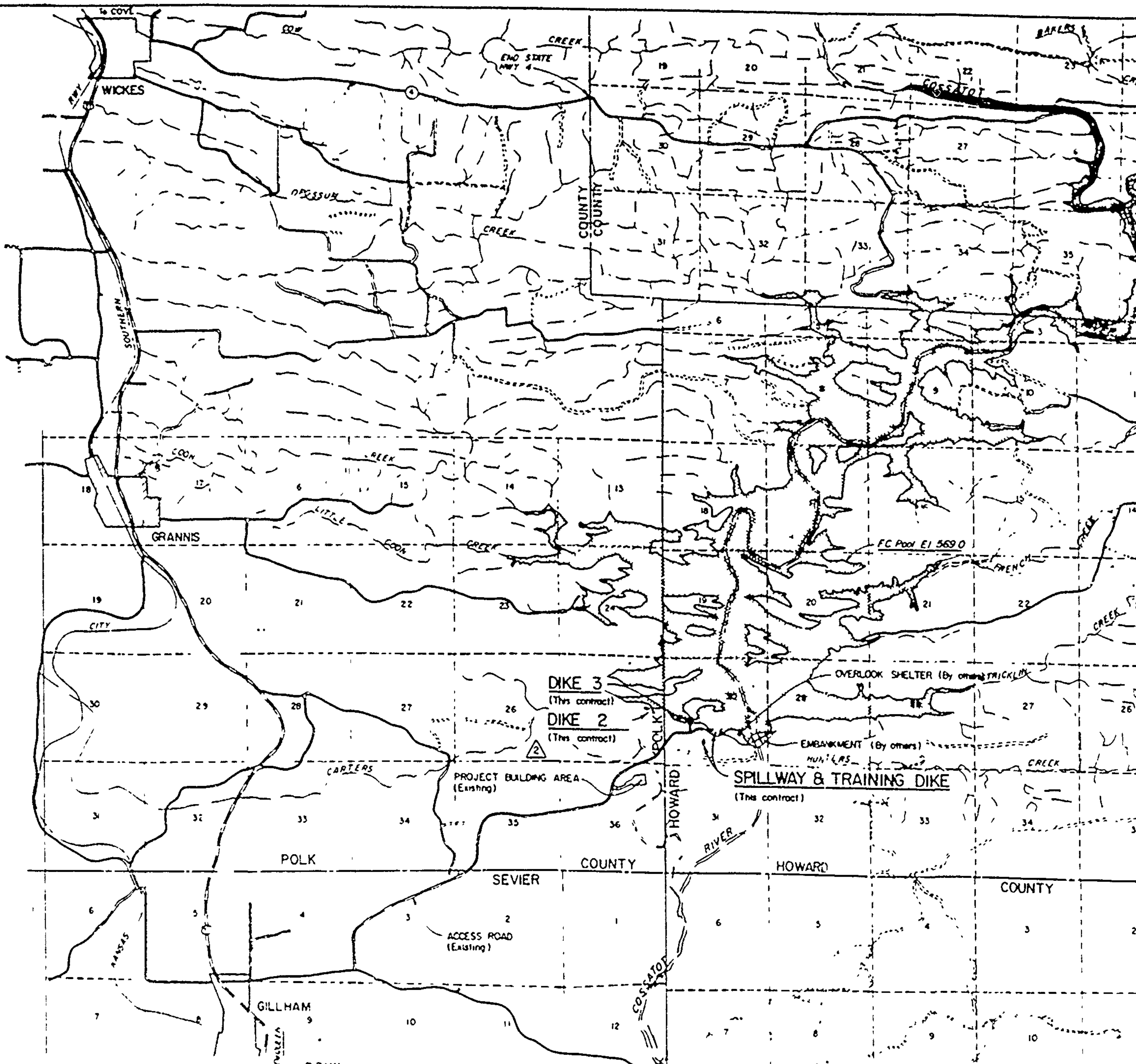
GILLHAM DAM AND RESERVOIR
SALINE RIVER, ARKANSAS

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

APPENDIX G - PLATES

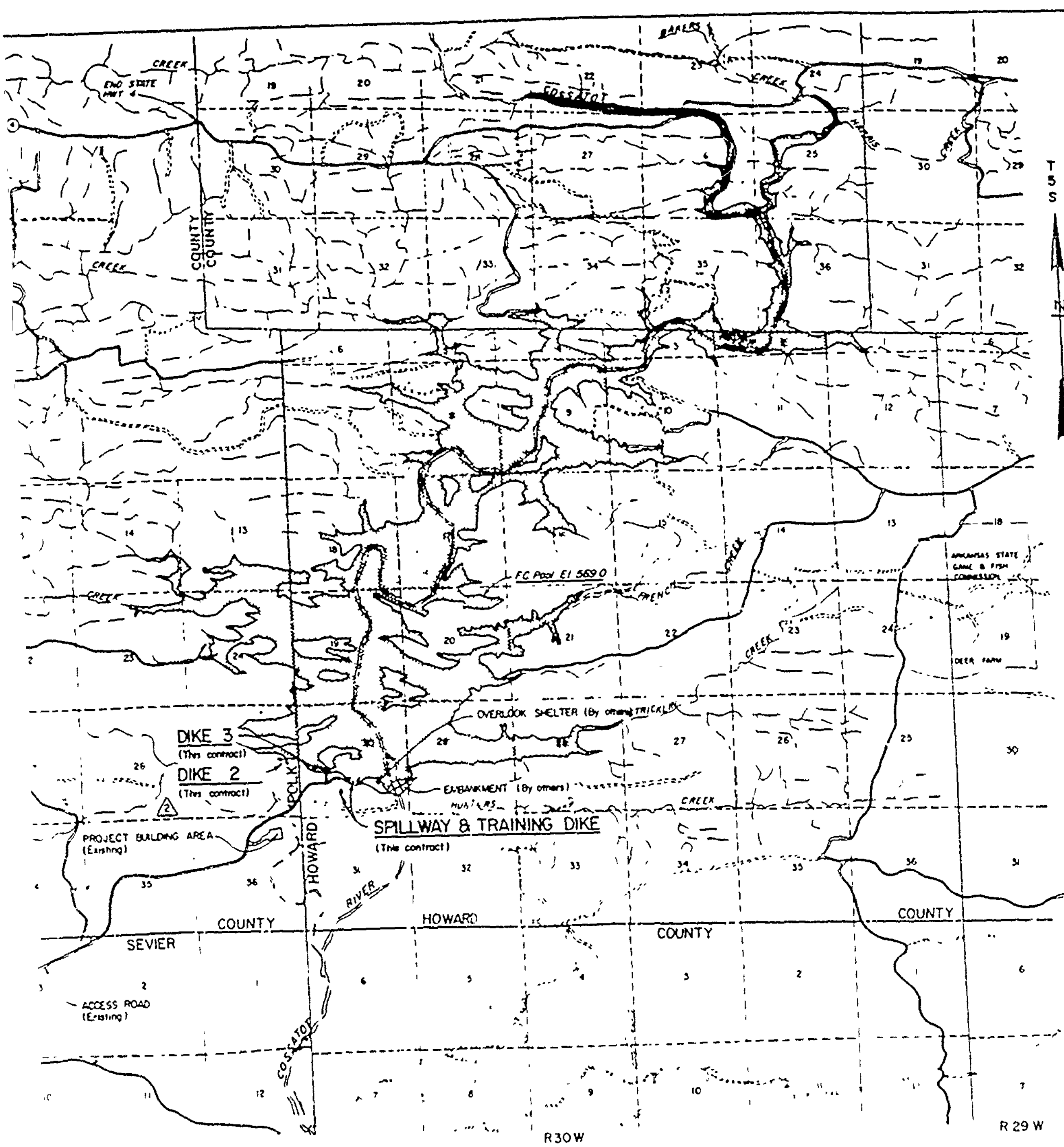
Plate No.	Drawing No.	Title
1	1770-C7-1/1.2	Location Plan
2	1770-C7-2/1.1	General Plan and Sections
3	1770-C13-98/1.1	Embankment - Plan of Explorations and Log Section
4	1770-C7-98/1.1	Spillway and Dikes - Plan of Exploration and Sections
5	1770-C7-10/1.1	Spillway Excavation Plan
6		Foundation Report - Key Plan
7		Foundation Report - Spillway
8		Foundation Report - Left Chute Wall
9		Foundation Report - Right Chute Wall
10		Foundation Report - Chute Slab
11		Foundation Report - Section Spillway and Chute Slab
12		Foundation Report - Grouting Profile
13	1770-C5-98/1	Outlet Works - Plan of Borings and Log Sections
14	1770-DM9-8/2	Embankment - Diversion - Plan and Sections
15	1770-C13-12/1	Embankment - Plan, Profile and Sections
16	1770-C13-12/2.1	Embankment - Sections and Details I
17	1770-C13-8/2A	Embankment - Wire Fabric and Cable Installation
18	1770-C13-8/2	Embankment - Cable Anchorages
19	1770-C13-13/1	Embankment - Engineering Measurement Devices
20		Embankment Record Samples - "As Built" Shear Strengths
21	1770-DM9-98/7	Stability Analysis - End of Construction Upstream
22	1770-DM9-98/8	Stability Analysis - End of Construction Downstream
23	1770-DM9-98/9	Stability Analysis - Steady Seepage
24	1770-DM9-98/10	Stability Analysis - Sudden Drawdown
25	1770-DM9-98/11	Stability Analysis - End of Construction
26	1770-DM9-98/12	Stability Analysis - Steady Seepage
27	1770-C7-12/1.2	Spillway - Plan
28	1770-C7-12/2.2	Spillway - Embankment and Wraparound Sections
29	1770-C7-12/4.2	Dikes - Plan and Sections
30	1770-C7-21/2	Spillway - Typical Structural Arrangement
31	1770-C7-21/3.3	Spillway - Typical Sections and Details
32	1770-C7-21/7.1	Spillway - Monolith 4 thru 8 - Weir Reinforcing
33	1770-C7-21/9	Spillway - Right Non-Overflow Monoliths 1 thru 4
34	1770-C7 21/20	Spillway - Left Non-Overflow Monoliths 8 thru 11
35	1770-C7-21/30.2	Spillway - Chute Slab
36	1770-DM12-21/4	Spillway - Overflow Stability Analysis
37		Spillway - Design Study
38	1770-DM12-21/5	Spillway - Non-Overflow Stability Analysis
39	1770-DM12-21/6	Spillway - Chute Wall Stability Analysis
40	1770-DM12-36/1.1	Spillway - Tainter Gates

APPENDIX G - PLATES
(continued)



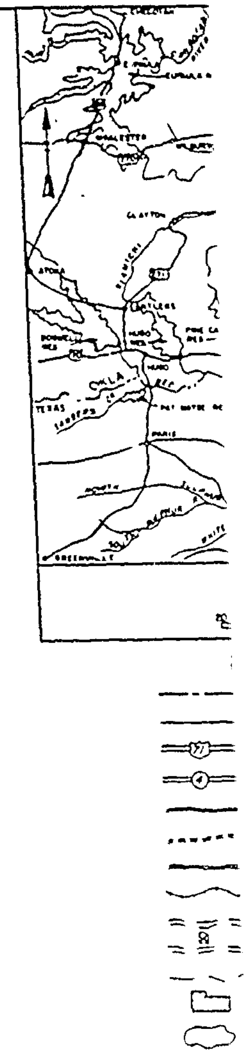
LOCATION PLAN

SCALE OF MILES



LOCATION PLAN

SCALE OF MILES



GENERAL NOTES

1 Elevations are in fe

2 Section taken and s

3 When section is tak

it is indicated thus

if is shown

4 Where the section

indicates the draw

5 Drawings shall not t

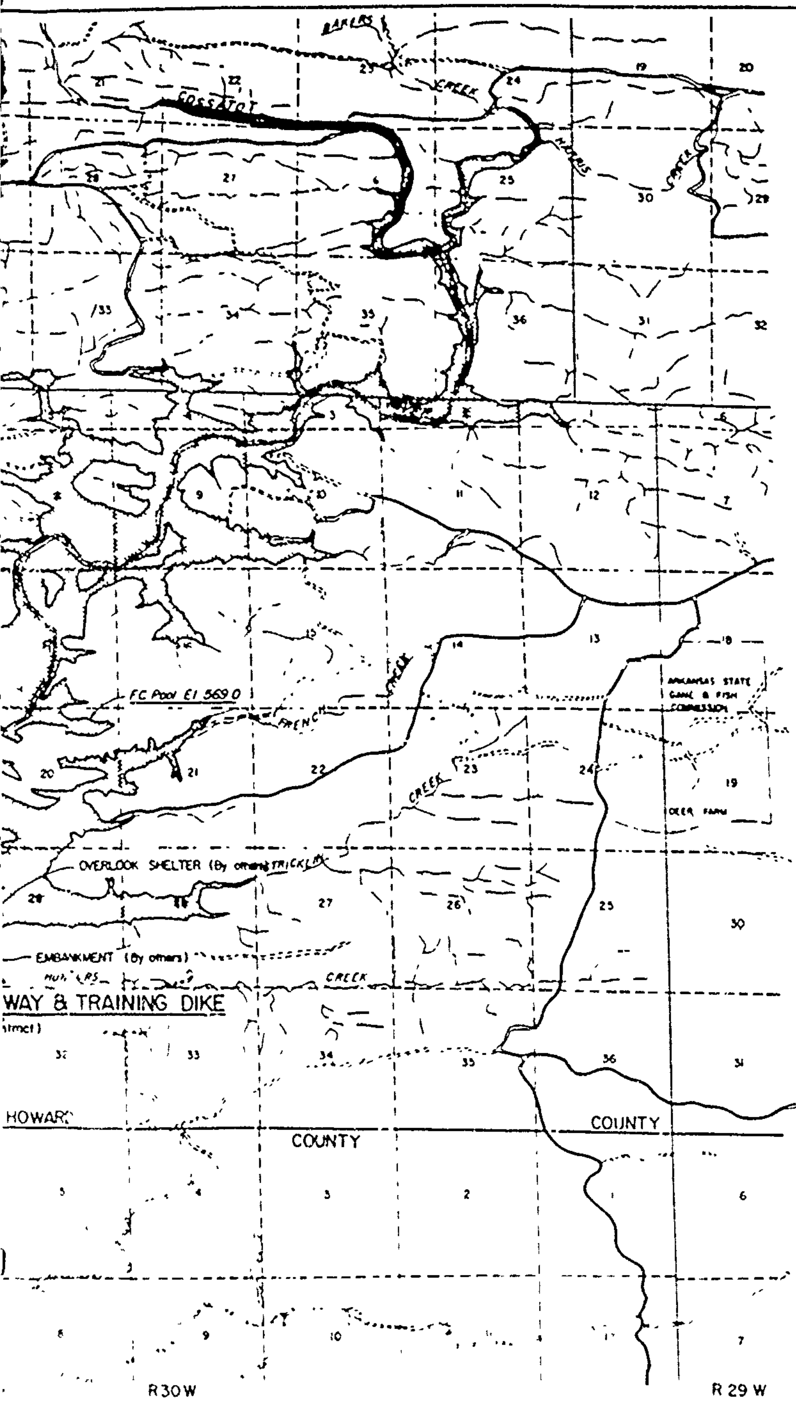
6 Bid item numbers n

RECORD (A:

2		Revised
1	4 18 68	Revised n
REV	DATE	

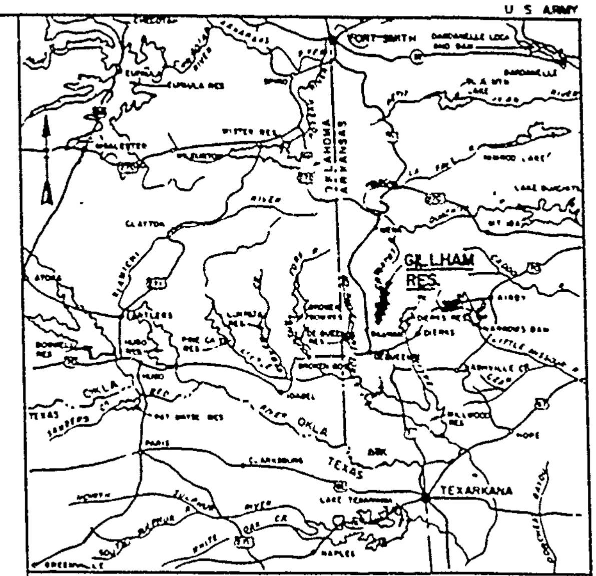
	BY	CHKD
DESIGNED		
DRAWN		
TRACED		
SUBMITTED		
RECOMMENDED		

EMBANKMENT CRITERIA AND PERFOI



LOCATION PLAN

SCALE OF MILES



VICINITY MAP

SCALE OF MILES

LEGEND

- County line
- Range and township line
- U S Highway
- State Highway
- County road
- Timber access road (not publically maintained)
- Paved road
- Railroad
- River
- River mile
- Creek
- Town
- F C pool El 569.0

GENERAL NOTES

1. Elevations are in feet and refer to mean sea level
2. Section taken and shown on the same drawing are indicated thus (A)———(A) and SECTION A-A
3. When section is taken on a drawing other than where the detail is shown it is indicated thus ——(D 21/6). the number indicates the drawing where it is shown
4. Where the section is titled thus SECTION (A 21/1), the number indicates the drawing where the section was taken
5. Drawings shall not be scaled. Use dimensions and elevations shown
6. Bid item numbers noted thus (10)

RECORD (AS BUILT) DRAWING

REV	DATE	REVISION INDICATED BY △
2	4-15-71	Revised as constructed
1	4-18-68	Revised in accordance with Amend No0001, Dated 14 MAR 1968

U. S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

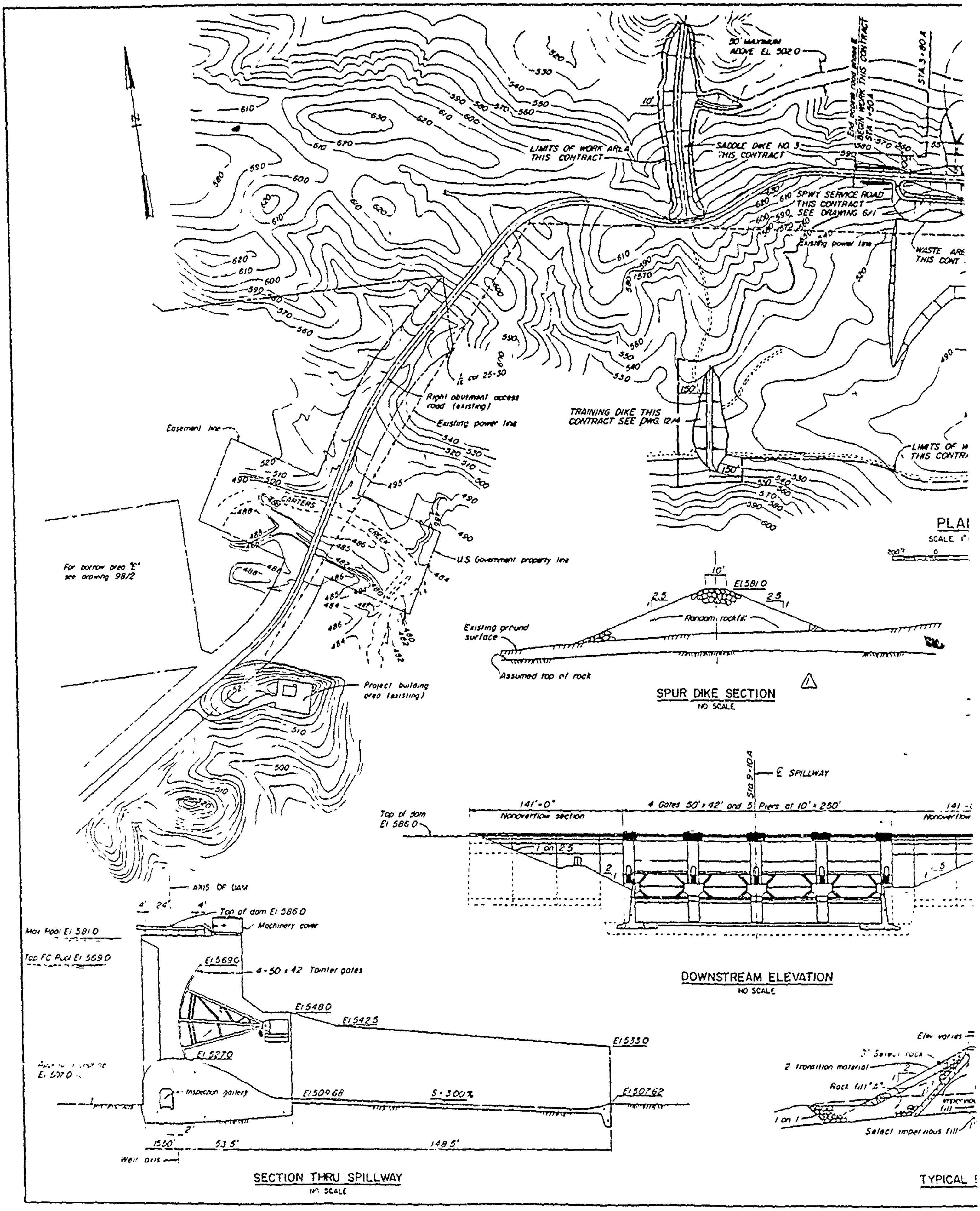
RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

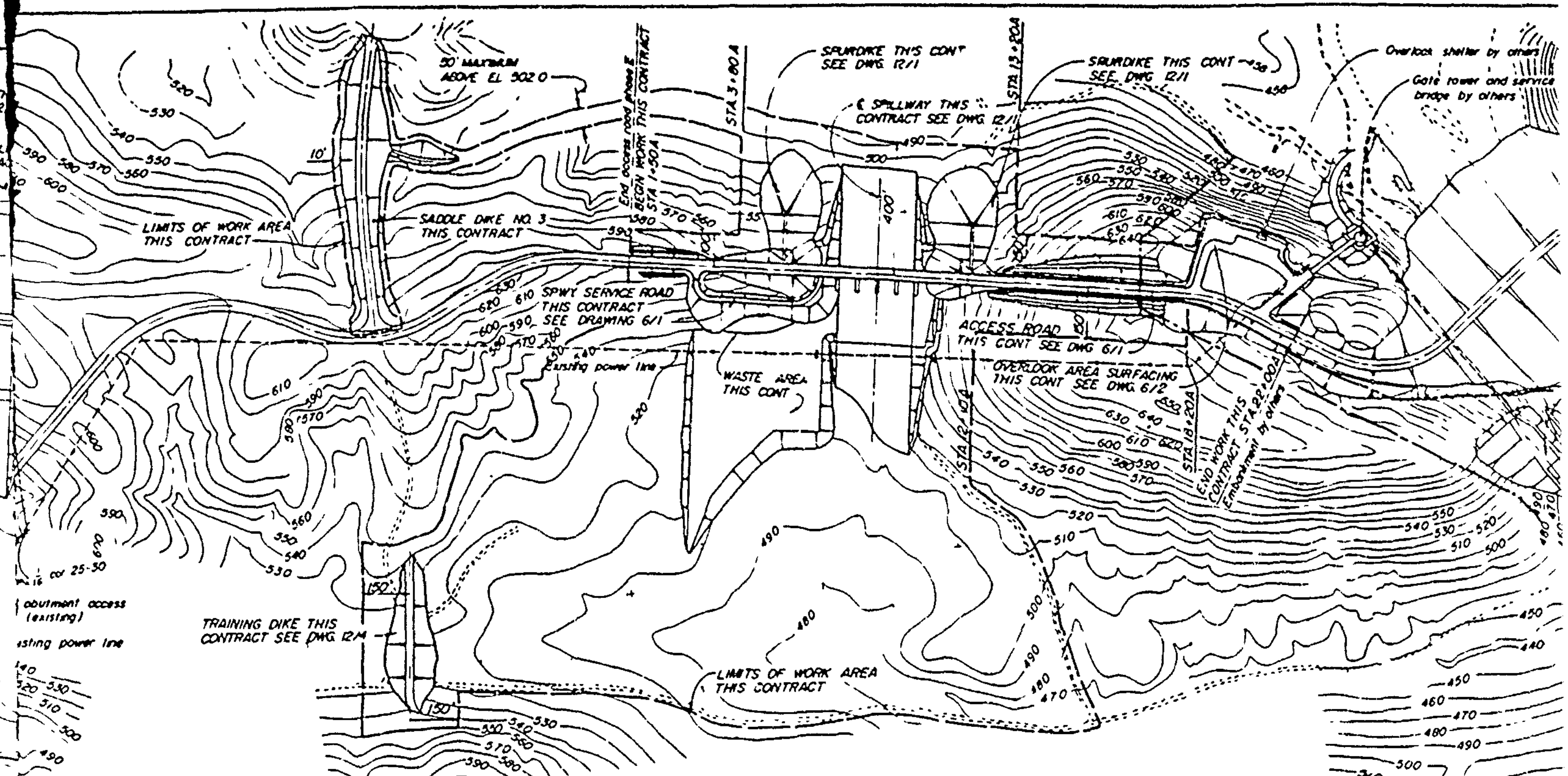
GILLHAM DAM
SPILLWAY AND DIKES
LOCATION PLAN

DESIGNED: JVN / WMW
DRAWN: PCM / JVN
TRACED
SUBMITTED: CHIEF, GEN. ENGR. SEC.
RECOMMENDED: CHIEF, DESIGN BRANCH; CHIEF, ENGR. DIV.
APPROVED: LT. COL. DEPUTY DISTRICT ENGINEER

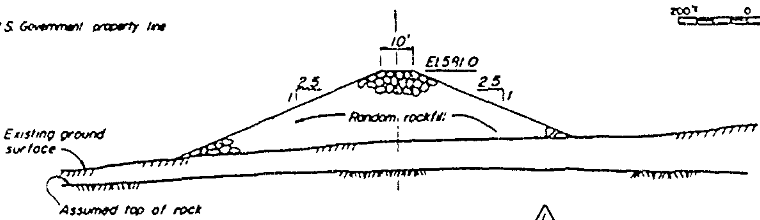
DATE: AUG 1966
INVITATION NO. DACW56-68-B-0067
SCALE
DRAWING No. ~~1770-C7-1/12~~

CORPS OF ENGINEERS

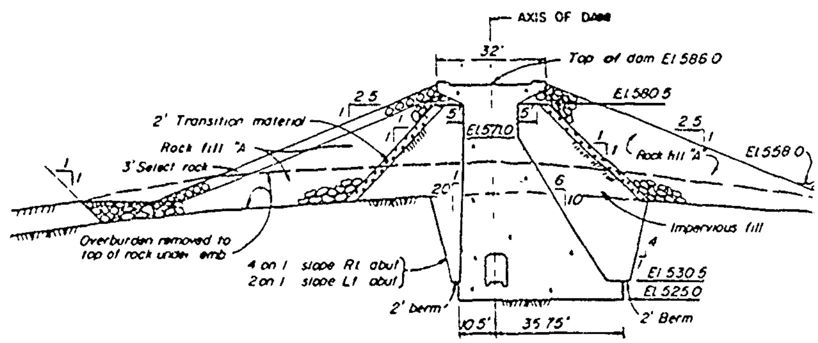




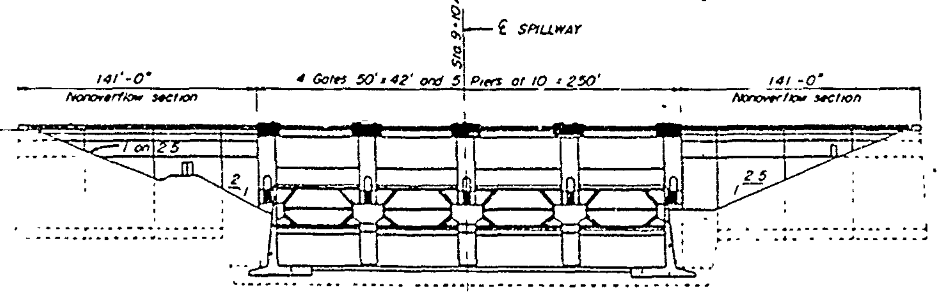
PLAN
SCALE 1" = 200'



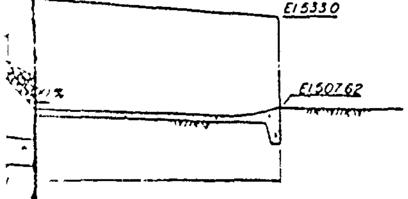
SPUR DIKE SECTION
NO SCALE

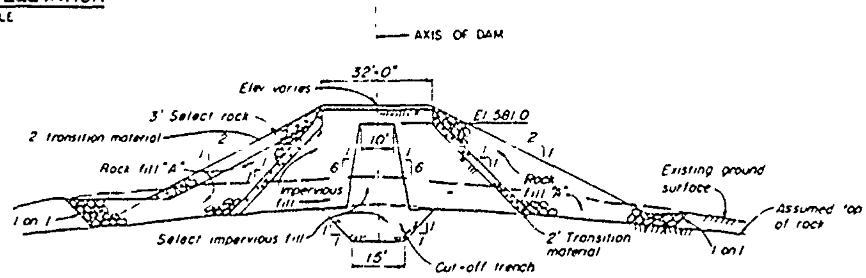


TYPICAL WRAP-AROUND SECTION
NO SCALE



DOWNSTREAM ELEVATION
NO SCALE





TYPICAL EMBANKMENT SECTION
NO SCALE

NOTES
1 ——— Indicates limits of
2 —·—·— Indicates work are
3 —··—·· Indicates work b
4 For general notes see dwg

THIS DRAWING WAS
FOR USE AS A CO
WAS REPRODUCED F

RECORD (AS BU

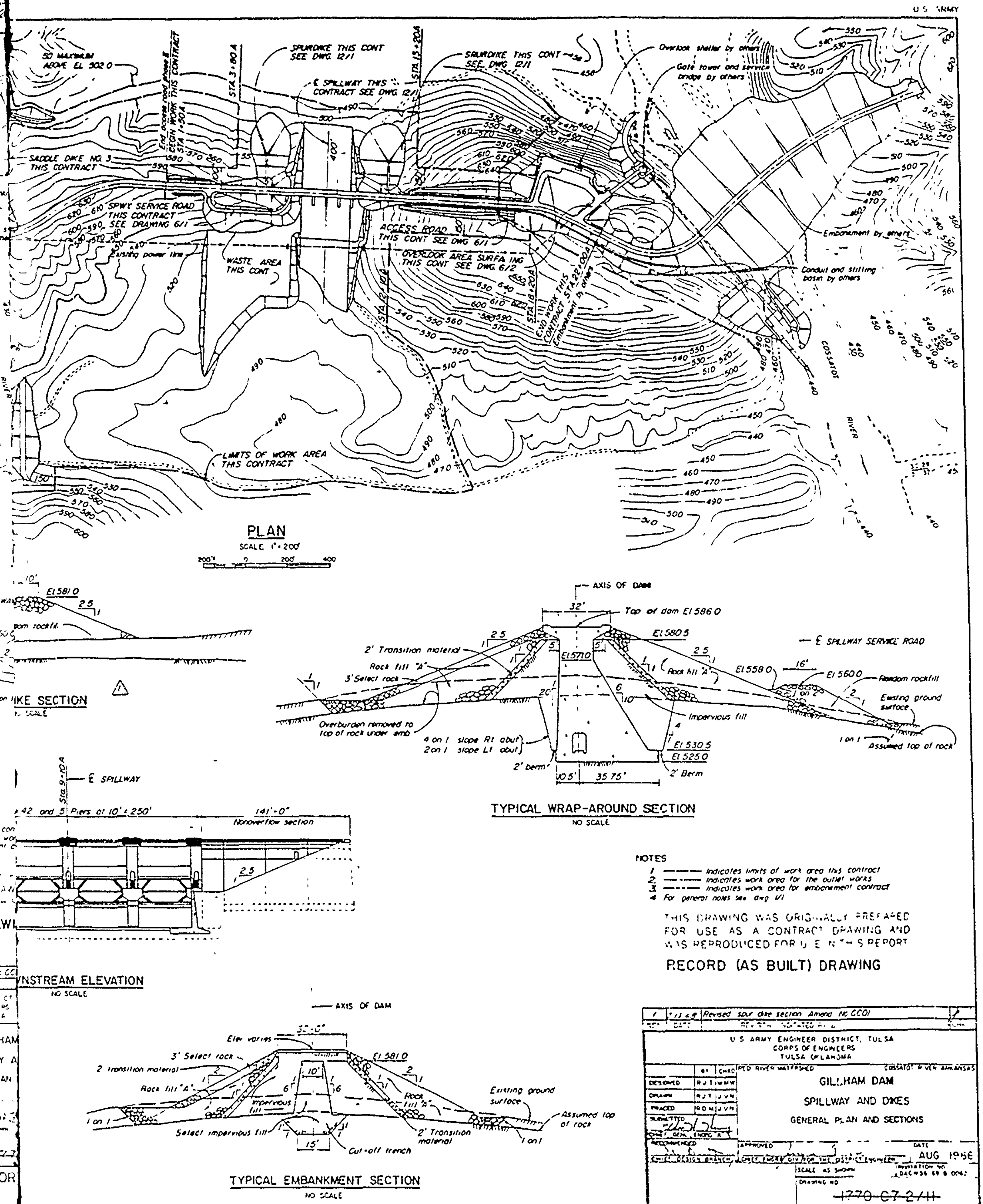
REV	DATE	REVISION
1	1-13-69	Revised spur dike s

U.S. ARMY

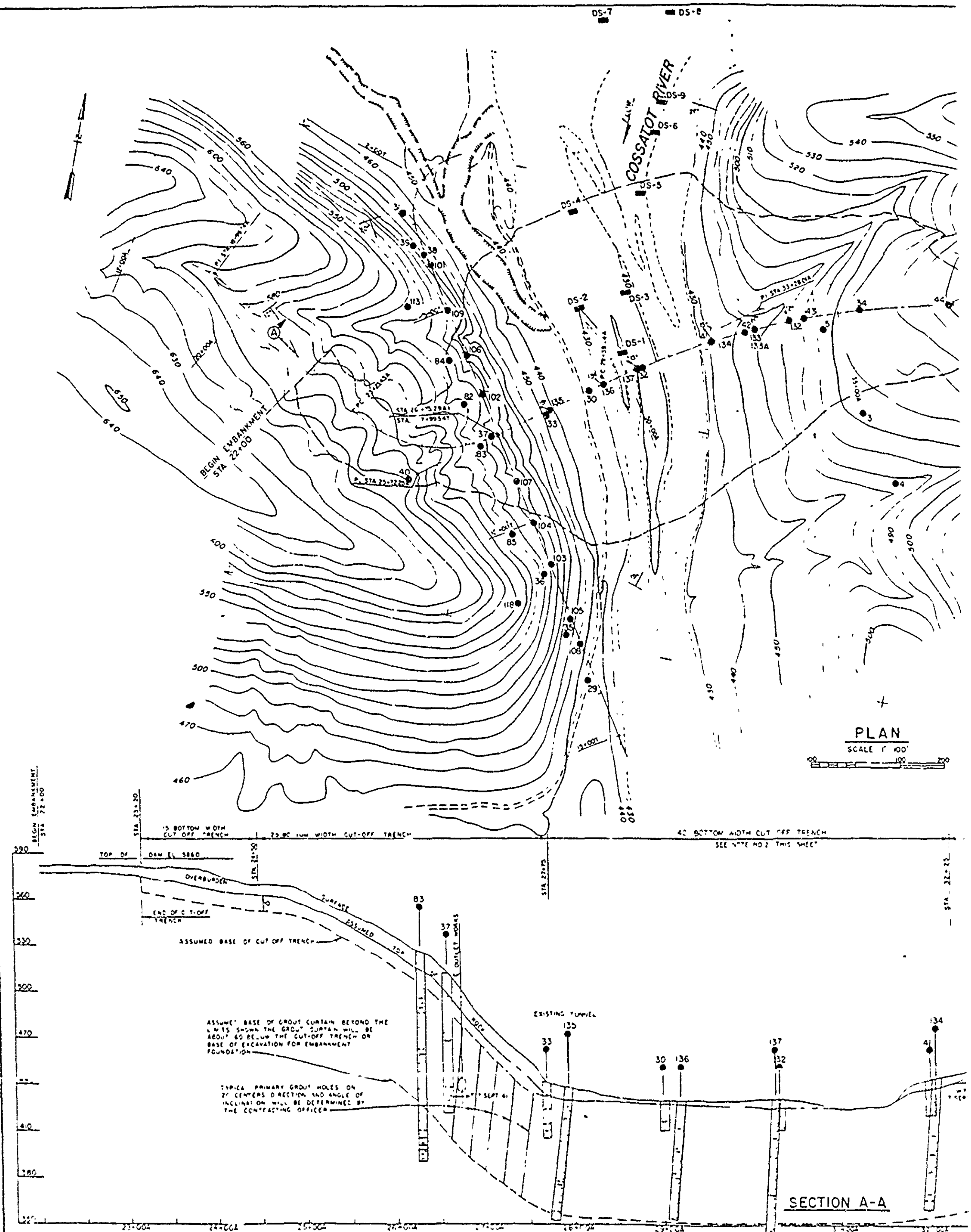
RED RIVER

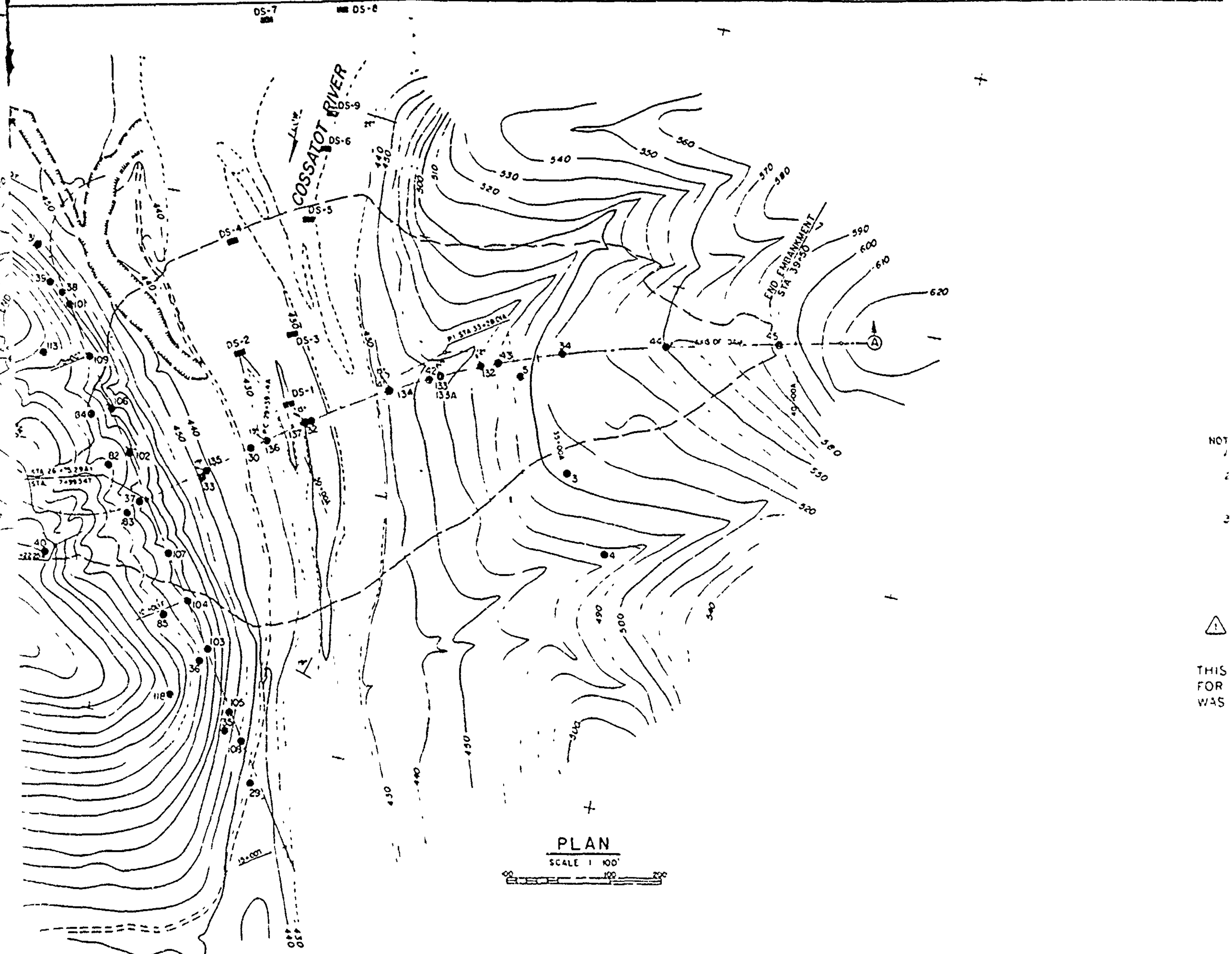
DESIGNED	RJT WMW
DRAWN	RJT JVN
TRACED	ROM JVN
SUBMITTED	
RECOMMENDED	APPROVED
CHIEF, DESIGN BRANCH	CHIEF, ENGR

EMBANKMENT CRITERIA AND PERFORMA



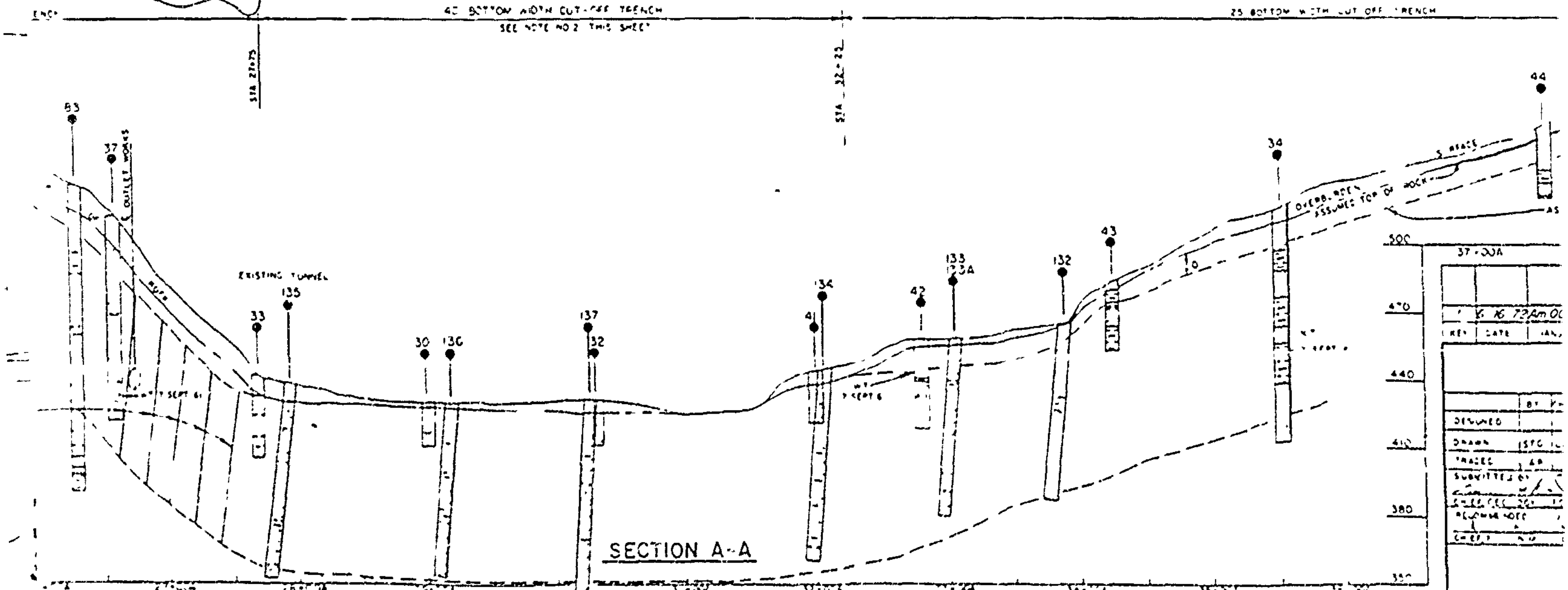
EMBANKMENT CRITERIA AND PERFORMANCE REPORT - PLATE NO. 2


COSSATOT RIVER
BEGIN EMBANKMENT STA 22+00
PLAN
SCALE 1" 100'
EXISTING TUNNEL
ASSUMED BASE OF CUT OFF TRENCH
OVERBURDEN
SURFACE
ROCK
40' BOTTOM WIDTH CUT OFF TRENCH
SECTION A-A



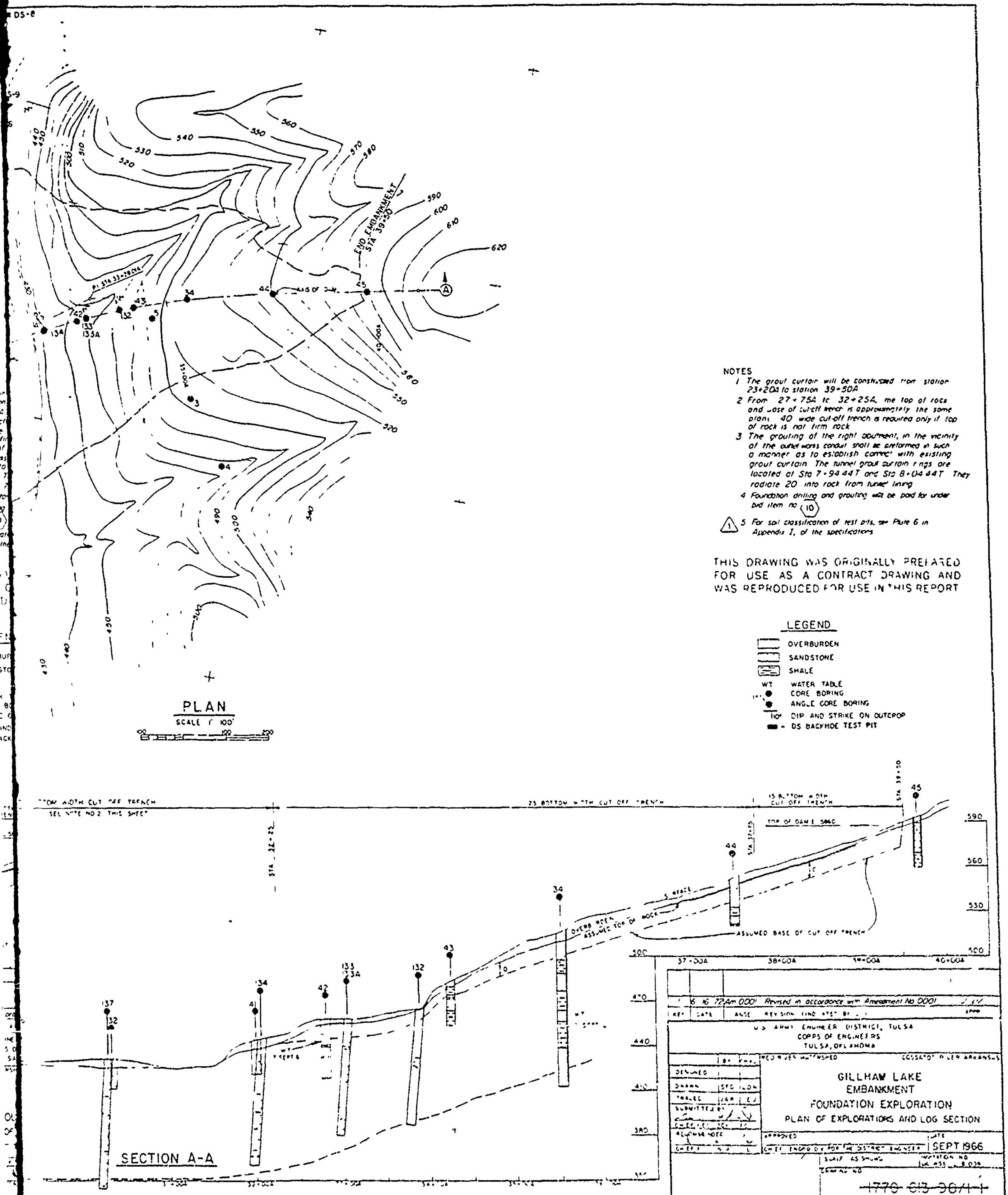
PLAN

SCALE 1" 100'

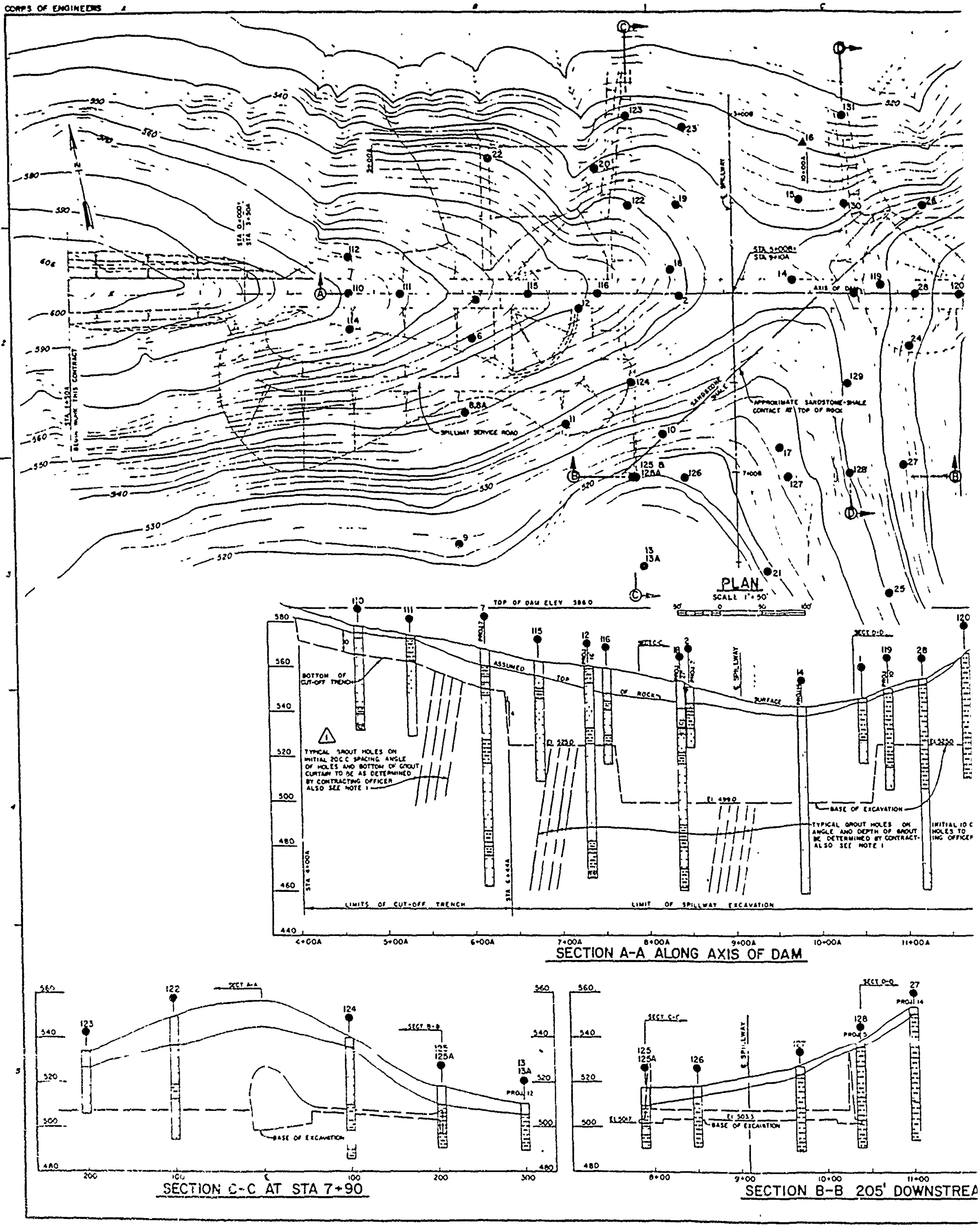


SECTION A-A

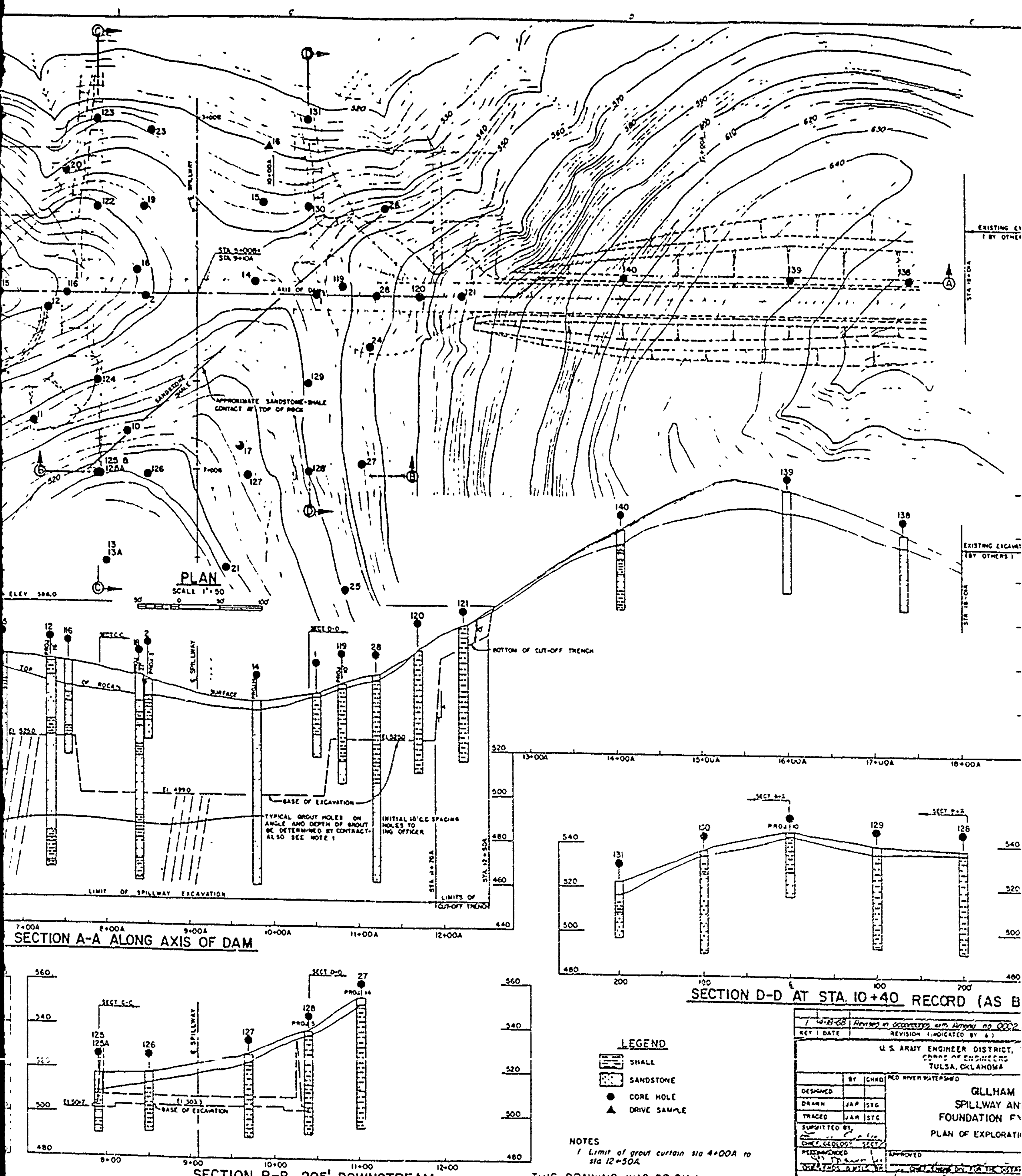
EMBANKMENT CRITERIA AND PERF

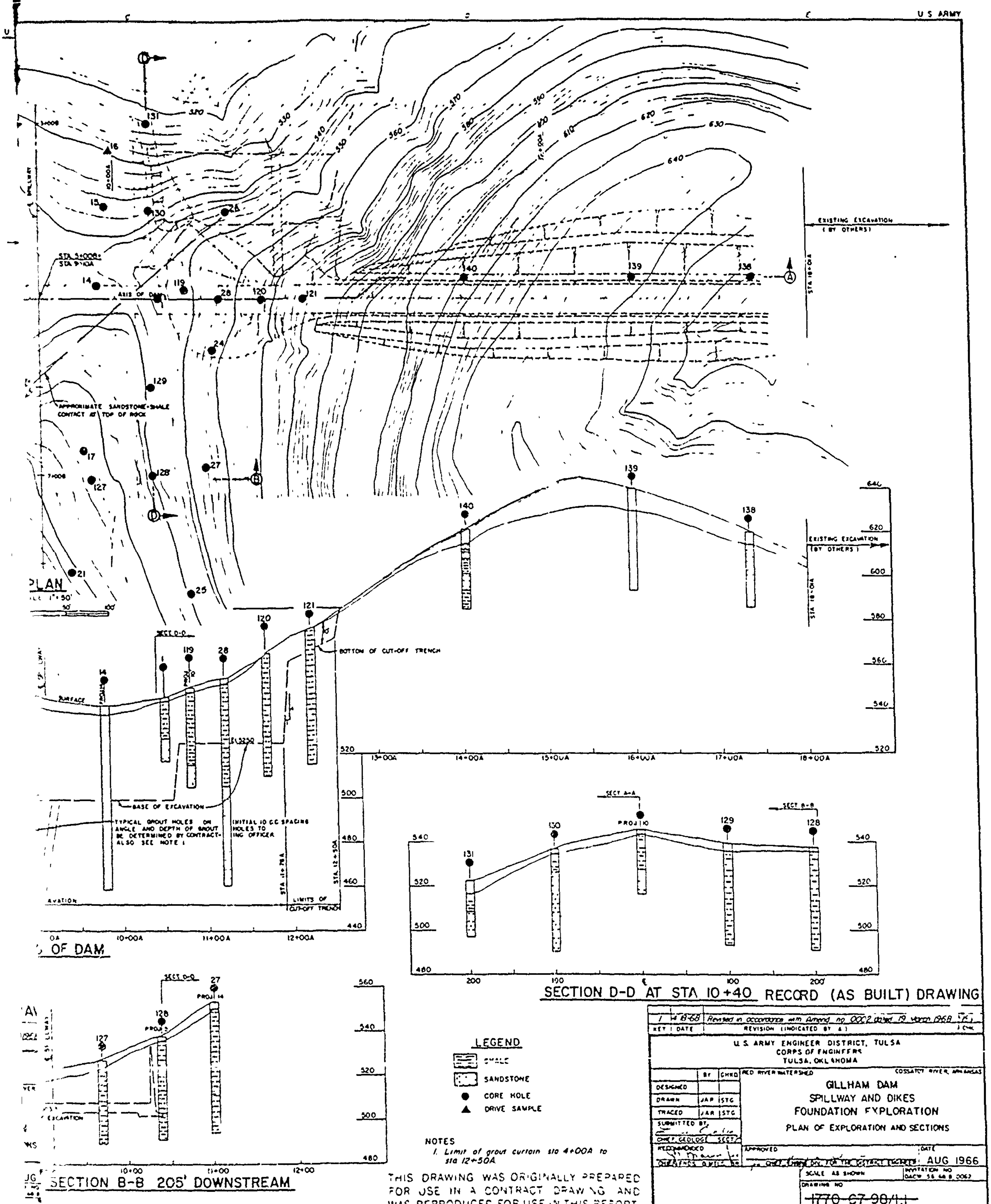


EMBANKMENT CRITERIA AND PERFORMANCE REPORT - PLATE NO 3

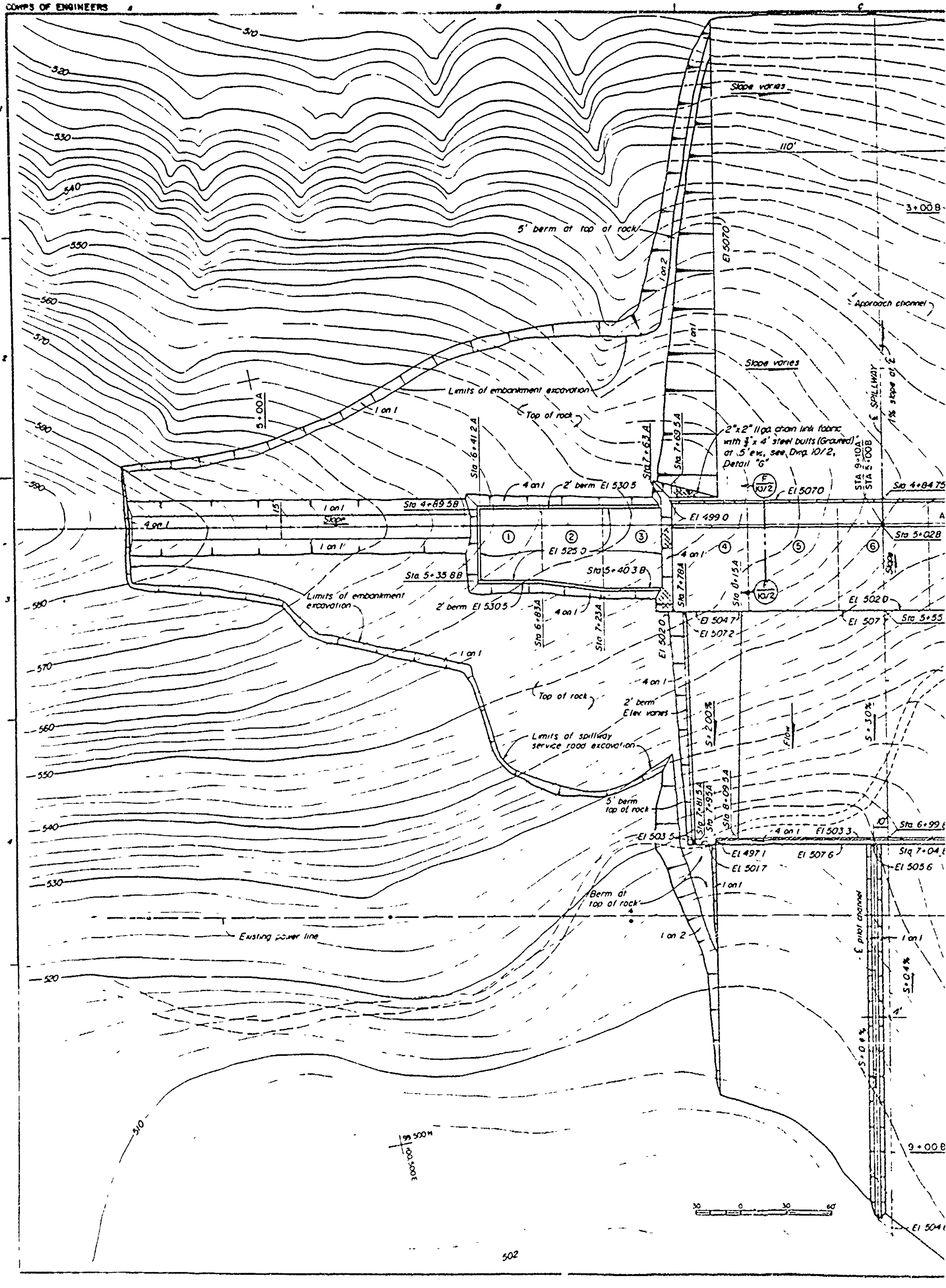


CORPS OF ENGINEERS
PLAN
SCALE 1" = 50'
AXIS OF DAM
TOP OF DAM ELEV 586.0
SPILLWAY SERVICE ROAD
APPROXIMATE SANDSTONE-SHALE CONTACT AT TOP OF ROCK
SANDSTONE
SHALE
BOTTOM OF CUT-OFF TRENCH
ASSUMED TOP OF ROCK SURFACE
TYPICAL GROUT HOLES ON INITIAL 20 C.C. SPACING. ANGLE OF HOLES AND BOTTOM OF GROUT CURTAIN TO BE AS DETERMINED BY CONTRACTING OFFICER. ALSO SEE NOTE 1
BASE OF EXCAVATION
TYPICAL GROUT HOLES ON ANGLE AND DEPTH OF GROUT HOLES TO BE DETERMINED BY CONTRACTING OFFICER. ALSO SEE NOTE 1
LIMITS OF CUT-OFF TRENCH
LIMIT OF SPILLWAY EXCAVATION
SECTION A-A ALONG AXIS OF DAM
SECTION C-C AT STA 7+90
SECTION B-B 205' DOWNSTREA

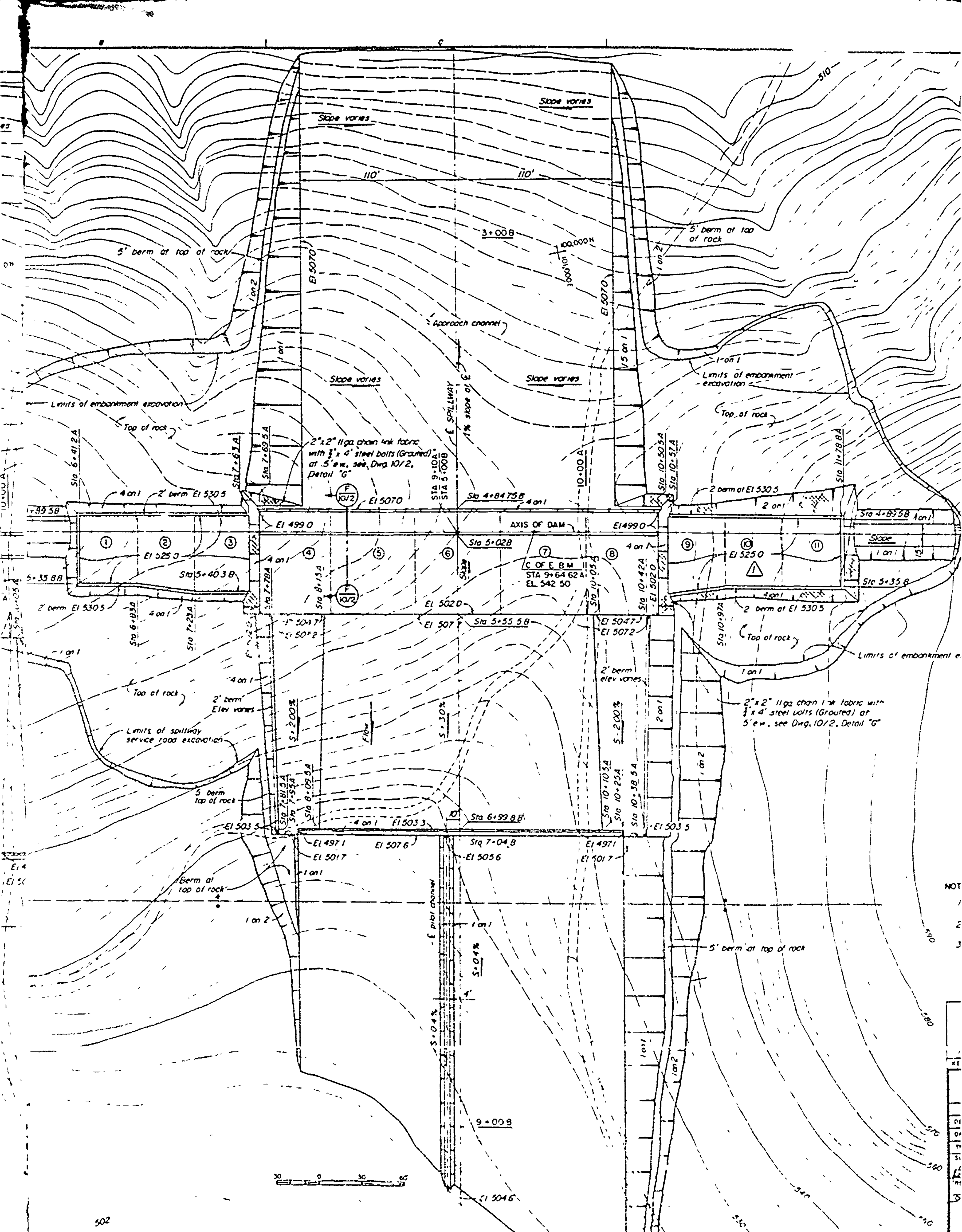

PLAN
SCALE 1" = 50'
AXIS OF DAM
APPROXIMATE SANDSTONE-SHALE CONTACT AT TOP OF ROCK
EXISTING EXCAVATION (BY OTHERS)
SECTION A-A ALONG AXIS OF DAM
TOP OF ROCK SURFACE
BOTTOM OF CUT-OFF TRENCH
BASE OF EXCAVATION
TYPICAL GROUT HOLES ON ANGLE AND DEPTH OF GROUT TO BE DETERMINED BY CONTRACTING OFFICER ALSO SEE NOTE 1
INITIAL 10' C.C. SPACING HOLES TO ING OFFICER
LIMIT OF SPILLWAY EXCAVATION
LIMITS OF CUT-OFF TRENCH
SECTION D-D AT STA. 10+40 RECORD (AS B
SECTION B-B 205' DOWNSTREAM
LEGEND
SHALE
SANDSTONE
CORE HOLE
DRIVE SAMPLE
NOTES
1 Limit of grout curtain sta 4+00A to sta 12+50A
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
U.S. ARMY ENGINEER DISTRICT,
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
GILLHAM
SPILLWAY AN
FOUNDATION EX
PLAN OF EXPLORATI
DESIGNED
DRAWN
TRACED
SUBMITTED BY
APPROVED
SCALE AS SHOWN
DRAWING NO
1770-67-9


U S ARMY
EXISTING EXCAVATION (BY OTHERS)
AXIS OF DAM
APPROXIMATE SANDSTONE-SHALE CONTACT AT TOP OF ROCK
PLAN
BOTTOM OF CUT-OFF TRENCH
BASE OF EXCAVATION
TYPICAL GROUT HOLES ON ANGLE AND DEPTH OF GROUT BE DETERMINED BY CONTRACTING OFFICER ALSO SEE NOTE 1
INITIAL 10 CC SPACING HOLES TO
LIMITS OF CUT-OFF TRENCH
OF DAM
SECTION D-D AT STA 10+40
RECORD (AS BUILT) DRAWING
SECTION B-B 205' DOWNSTREAM
LEGEND
SHALE
SANDSTONE
CORE HOLE
DRIVE SAMPLE
NOTES
1. Limit of grout curtain sta 4+00A to sta 12+50A
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
FOUNDATION EXPLORATION
PLAN OF EXPLORATION AND SECTIONS
AUG 1966
SCALE AS SHOWN
1770-67-98/1.1

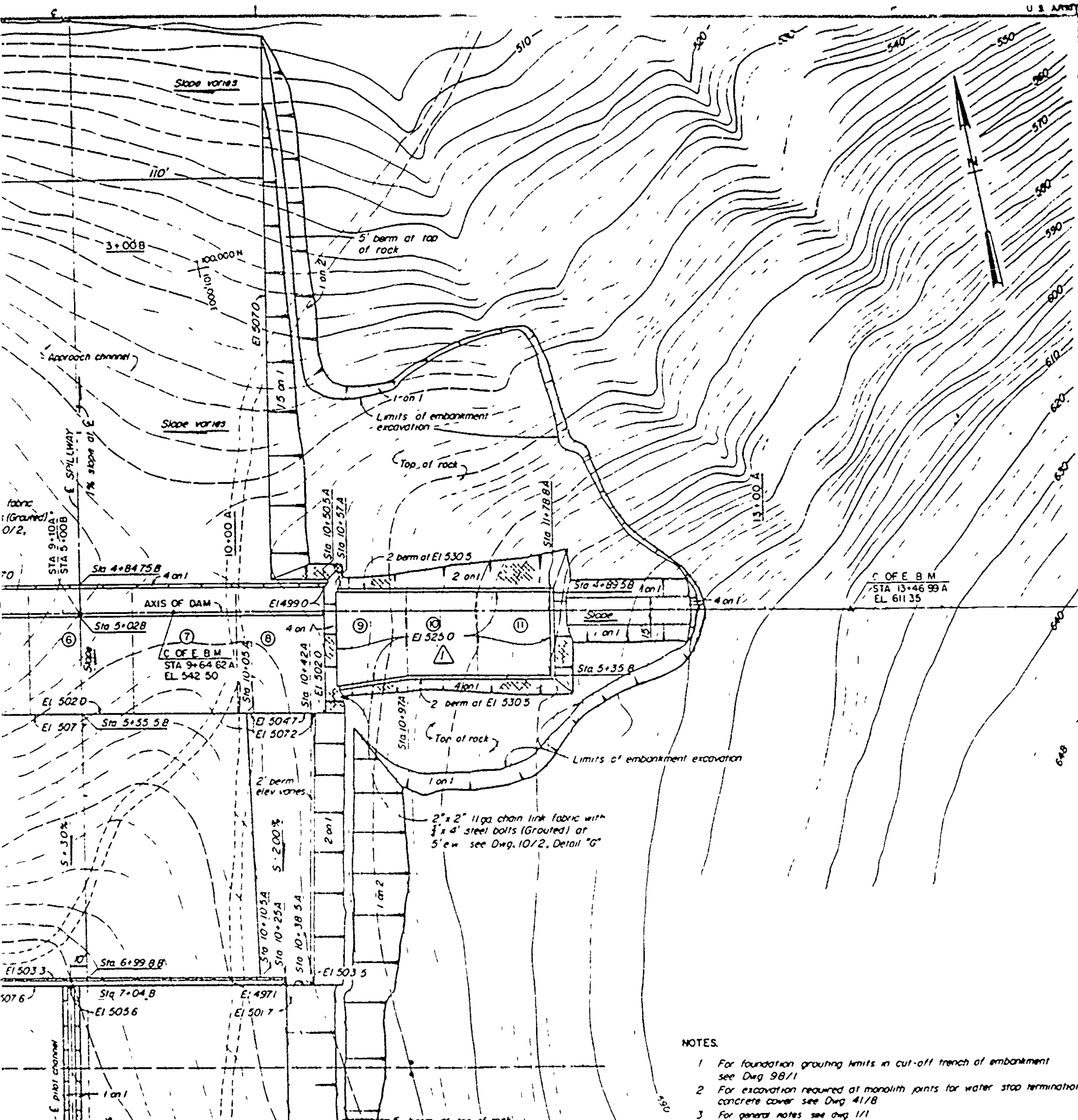
CORPS OF ENGINEERS



502



EMBANKMENT CRITERI



NOTES.

1. For foundation grouting limits in cut-off trench of embankment see Dwg 98/1
2. For excavation required at monolith joints for water stop termination concrete cover see Dwg 41/8
3. For general notes see dwg 1/1

KEY	DATE	REVISION (INDICATED BY △)	CHKD
1	4-18-68	Revised in accordance with Amend No 0002 Dated 13 MAR 1968	

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER ARKANSAS

GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY
EXCAVATION PLAN

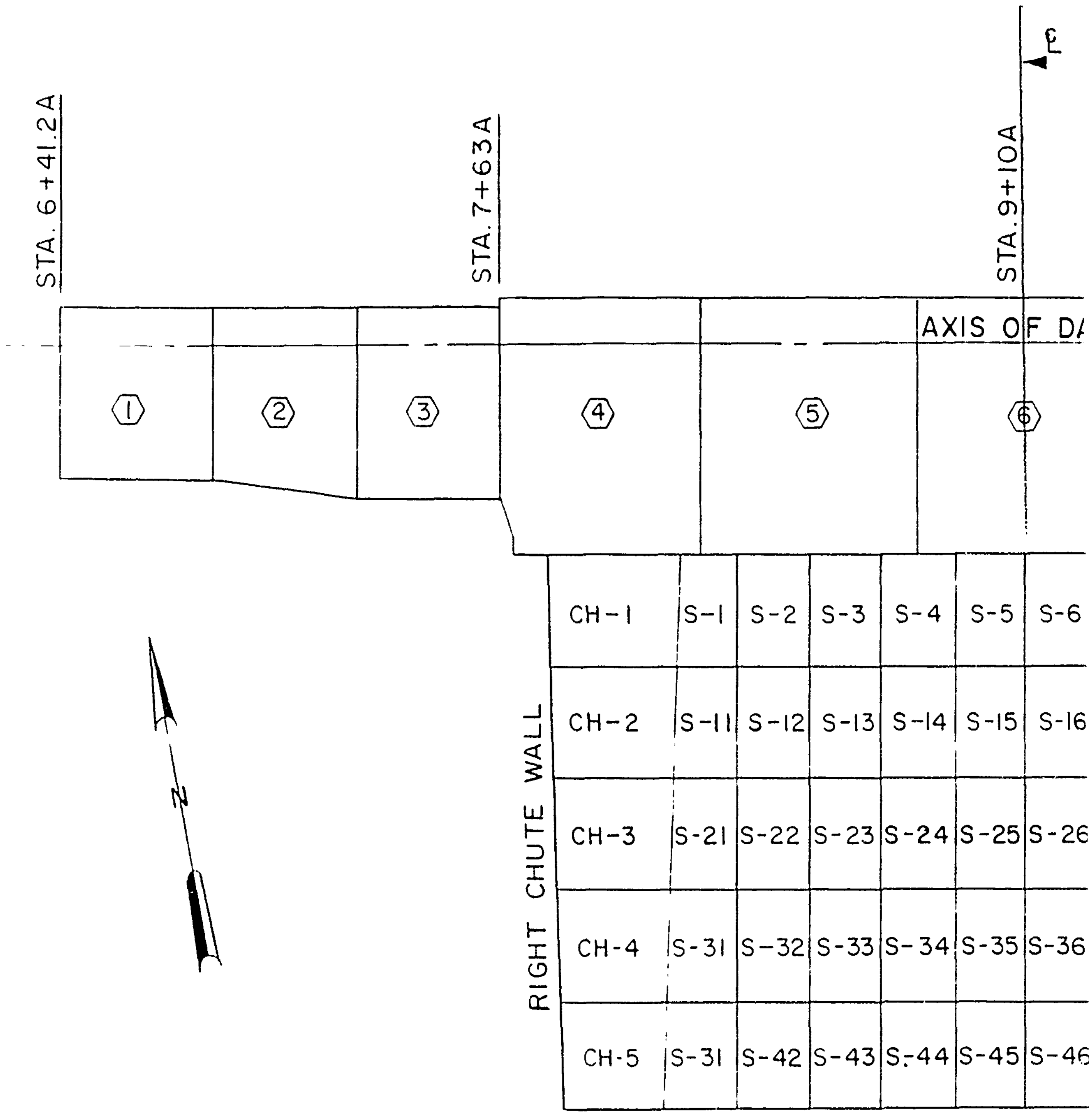
DESIGNED / DRAWN / TRACED / SUBMITTED BY: CHIEF, GEN. ENGRG. SEC. / RECOMMENDED: CHIEF, DESIGN BRANCH / APPROVED: CHIEF, ENGRG. DIV. FOR THE DISTRICT ENGINEER

DATE: AUG 1966

SCALE: 1 INCH = 30 FEET

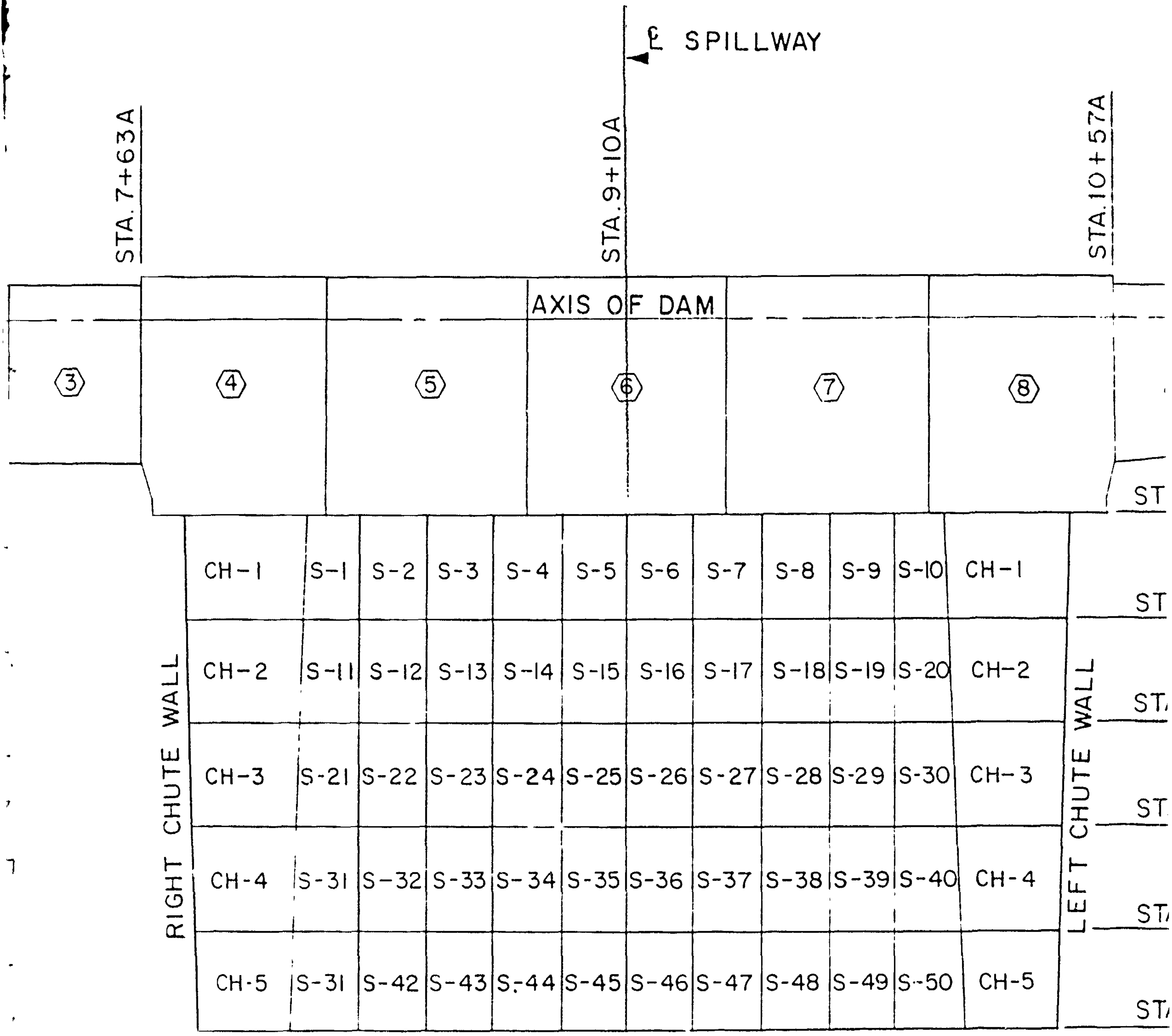
DRAWING NO 1770-C7-10/11

EMBANKME' CRITERIA AND PERFORMANCE REPORT - PLATE NO 5

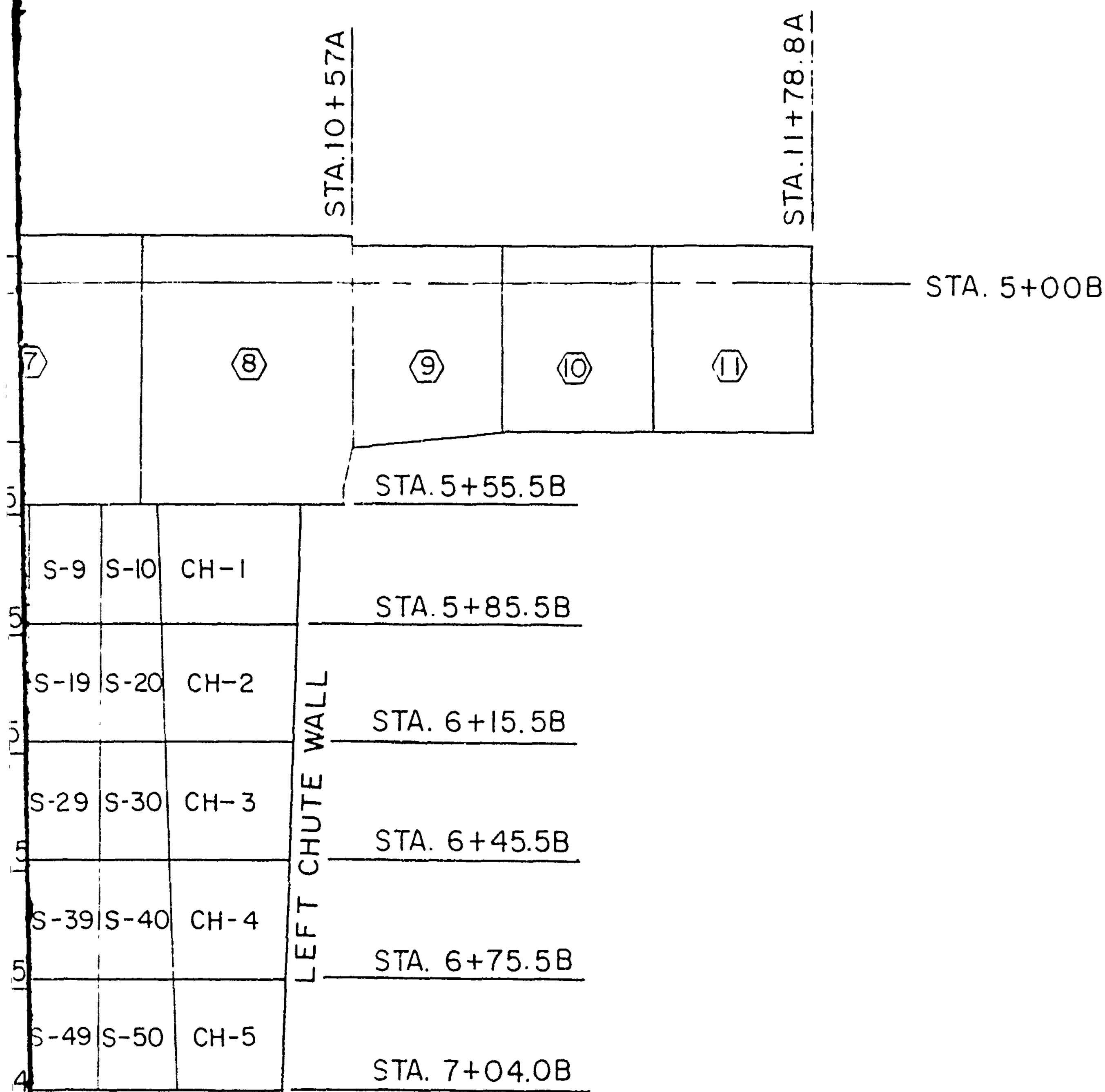


STA. 6+41.2A
STA. 7+63A
STA. 9+10A
AXIS OF DA
1
2
3
4
5
6
CH-1
S-1
S-2
S-3
S-4
S-5
S-6
CH-2
S-11
S-12
S-13
S-14
S-15
S-16
CH-3
S-21
S-22
S-23
S-24
S-25
S-26
CH-4
S-31
S-32
S-33
S-34
S-35
S-36
CH-5
S-31
S-42
S-43
S-44
S-45
S-46
RIGHT CHUTE WALL
N

KEY P


℄ SPILLWAY
STA. 7+63A
STA. 9+10A
STA. 10+57A
AXIS OF DAM
3
4
5
6
7
8
RIGHT CHUTE WALL
LEFT CHUTE WALL
CH-1 S-1 S-2 S-3 S-4 S-5 S-6 S-7 S-8 S-9 S-10 CH-1
CH-2 S-11 S-12 S-13 S-14 S-15 S-16 S-17 S-18 S-19 S-20 CH-2
CH-3 S-21 S-22 S-23 S-24 S-25 S-26 S-27 S-28 S-29 S-30 CH-3
CH-4 S-31 S-32 S-33 S-34 S-35 S-36 S-37 S-38 S-39 S-40 CH-4
CH-5 S-31 S-42 S-43 S-44 S-45 S-46 S-47 S-48 S-49 S-50 CH-5

KEY PLAN



RED RIVEP WATERSHED COSSATOT RIVER, ARK

GILLHAM DAM
FOUNDATION REPORT
KEY PLAN

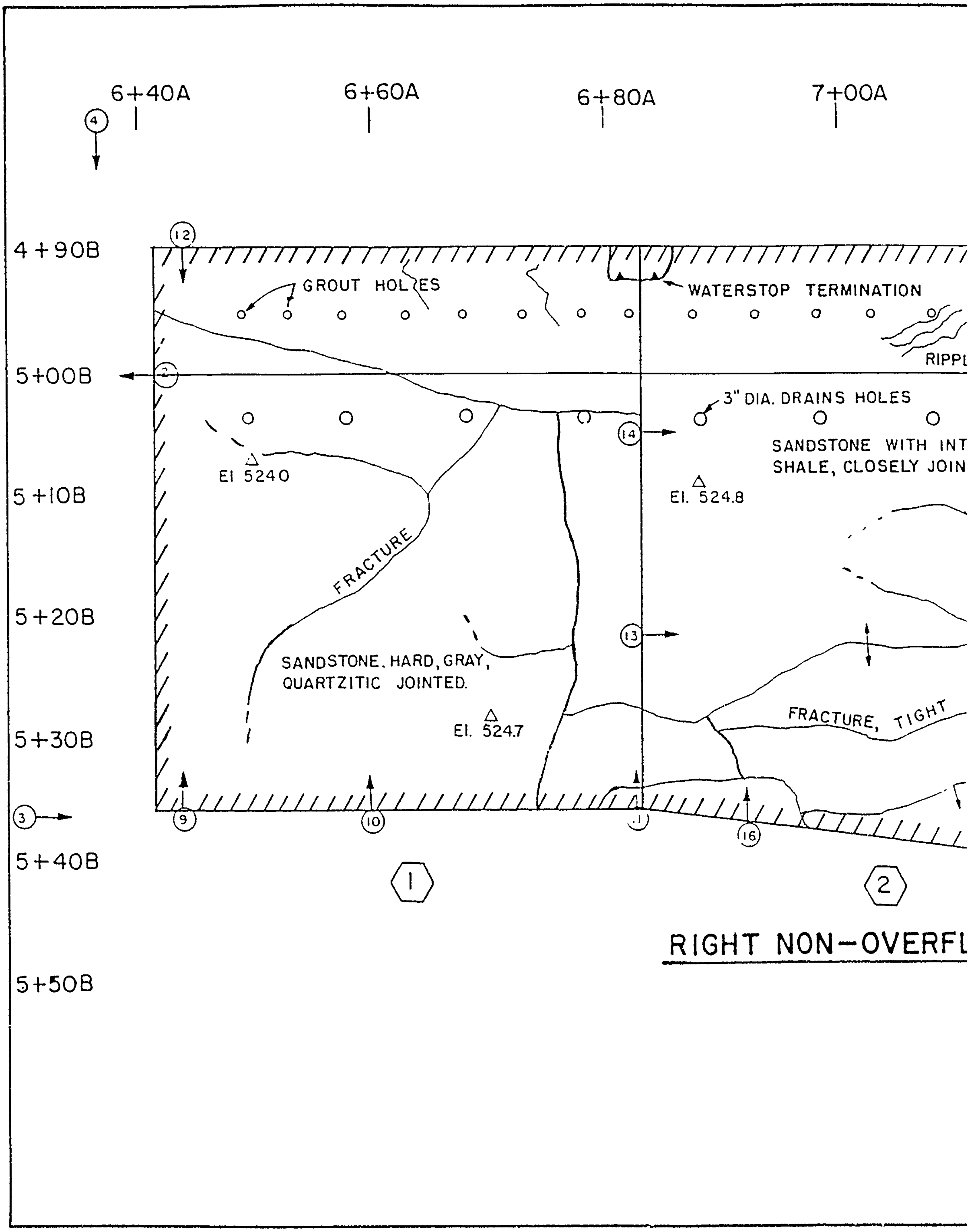
U S ARMY ENGINEER DIST TULSA CORPS OF ENGINEERS

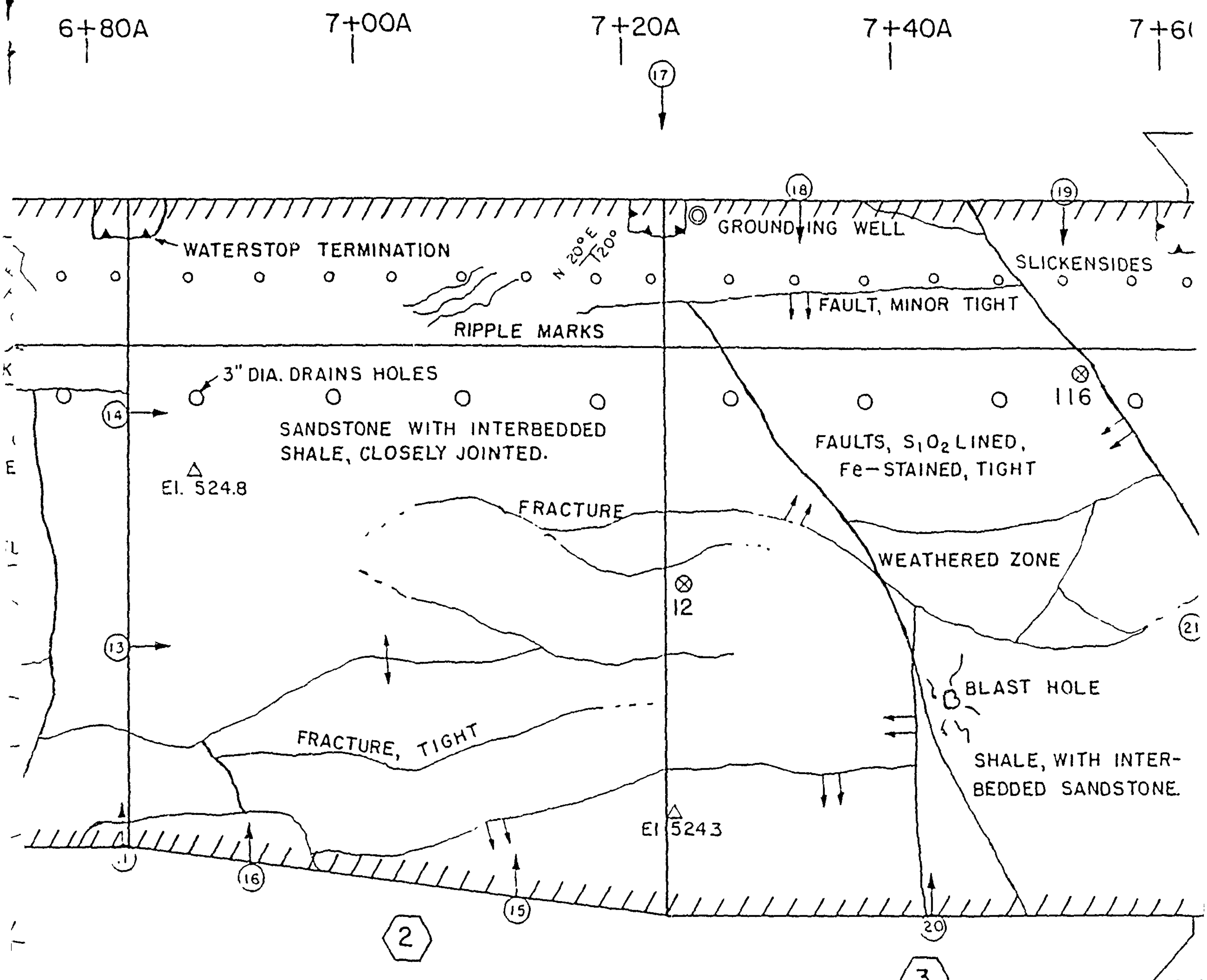
SUBMITTED: J. L. Williamson
GEOLOGIST

APPROVED:
RESIDENT ENGINEER

SCALE 1" = 40'

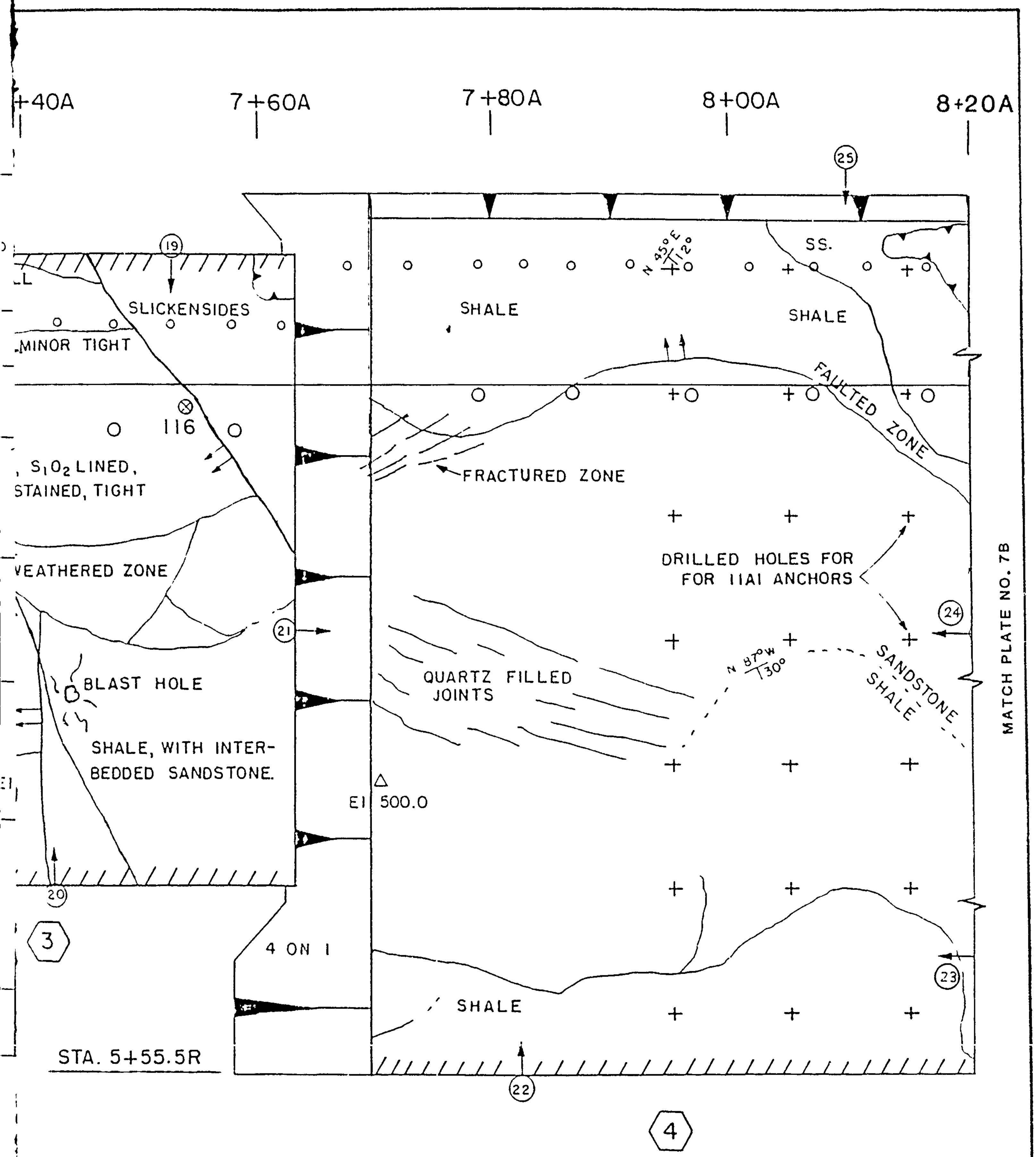
DATE 12-22-69

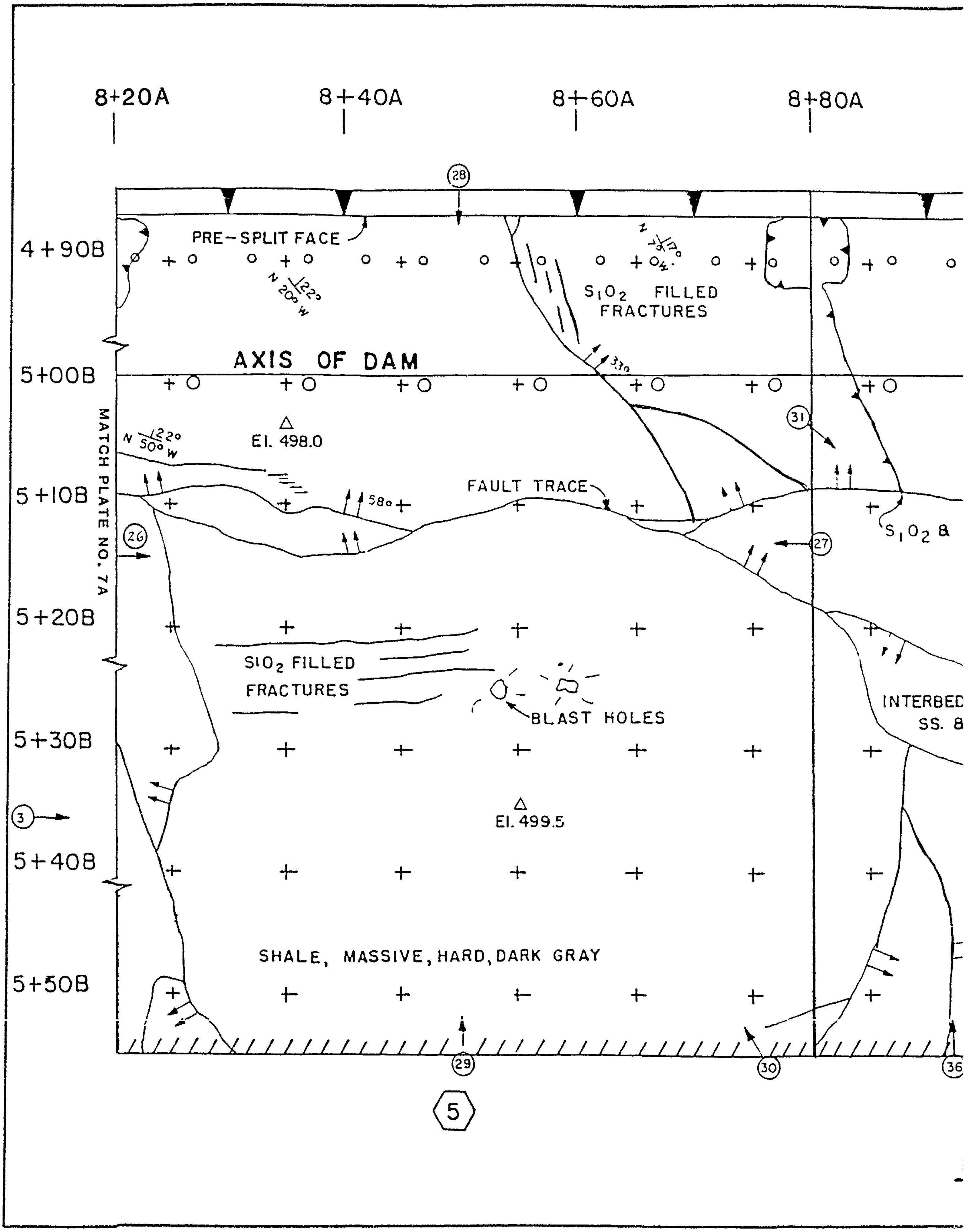

6+40A
6+60A
6+80A
7+00A
4+90B
5+00B
5+10B
5+20B
5+30B
5+40B
5+50B
GROUT HOLES
WATERSTOP TERMINATION
RIPPL
3" DIA. DRAINS HOLES
SANDSTONE WITH INT
SHALE, CLOSELY JOIN
El 5240
El. 524.8
FRACTURE
SANDSTONE. HARD, GRAY,
QUARTZITIC JOINTED.
El. 524.7
FRACTURE, TIGHT
1
2
3
4
9
10
11
12
13
14
16
RIGHT NON-OVERFL

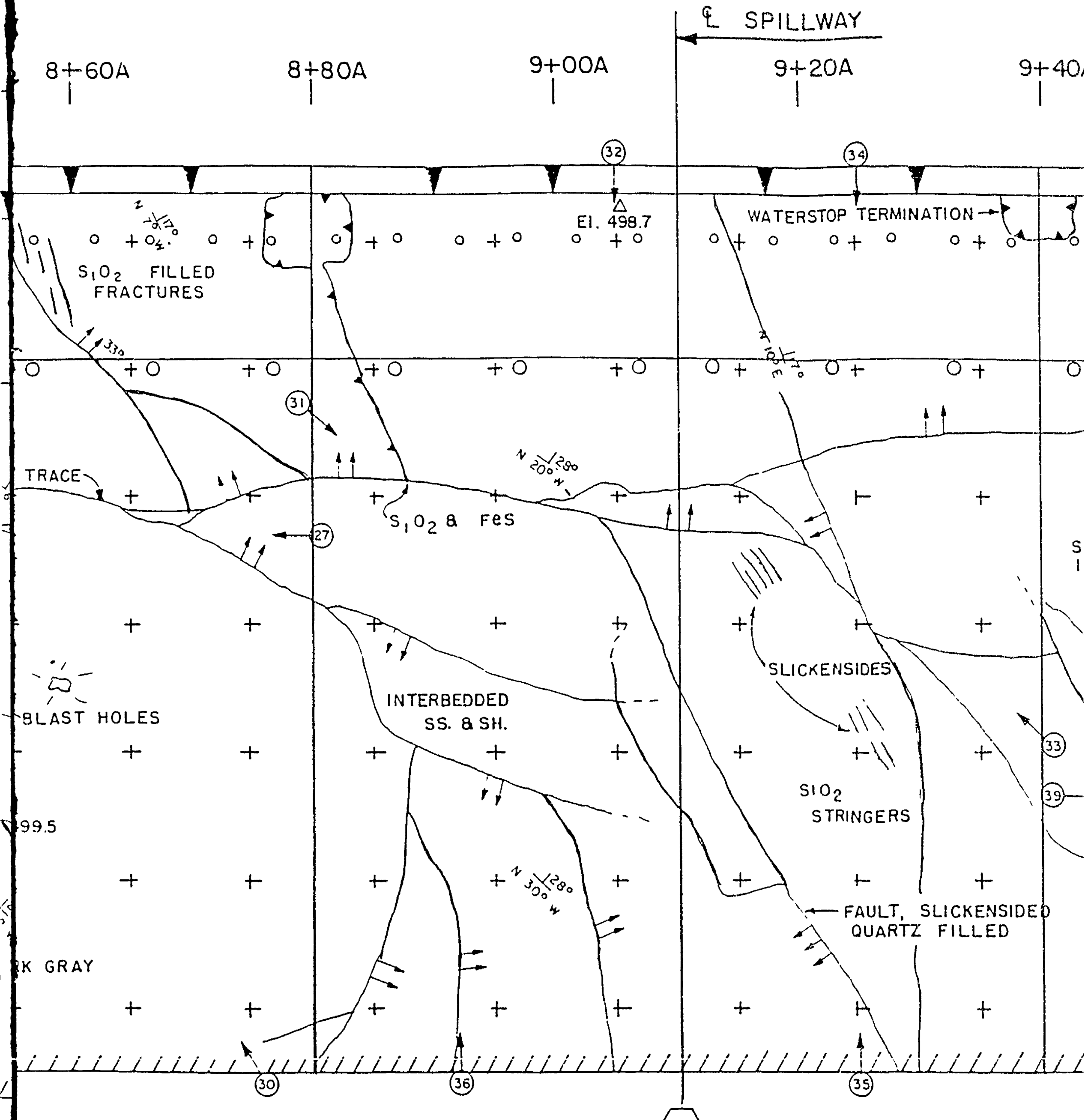


RIGHT NON-OVERFLOW

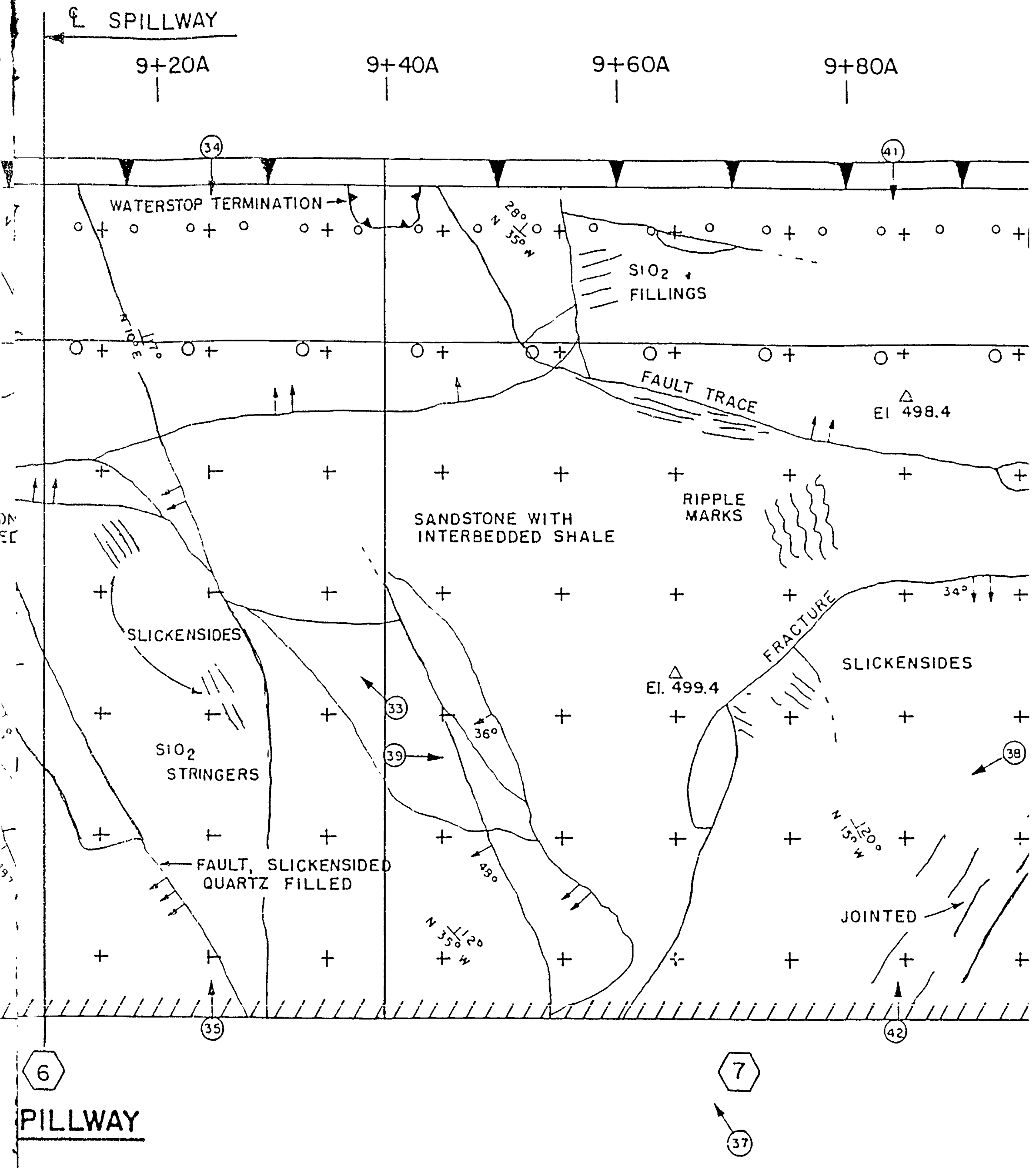
STA. 5+55.5R


+40A
7+60A
7+80A
8+00A
8+20A
25
19
SLICKENSIDES
SS.
SHALE
SHALE
MINOR TIGHT
N 45° E 12°
FAULTED ZONE
116
S_iO_2 LINED,
STAINED, TIGHT
FRACTURED ZONE
WEATHERED ZONE
DRILLED HOLES FOR
FOR 1IAI ANCHORS
21
24
BLAST HOLE
QUARTZ FILLED
JOINTS
N 87° W 30°
SANDSTONE
SHALE
SHALE, WITH INTER-
BEDDED SANDSTONE.
MATCH PLATE NO. 7B
El 500.0
20
3
4 ON 1
23
SHALE
STA. 5+55.5R
22
4


8+20A
8+40A
8+60A
8+80A
4+90B
5+00B
5+10B
5+20B
5+30B
5+40B
5+50B
MATCH PLATE NO. 7A
PRE-SPLIT FACE
AXIS OF DAM
SiO_2 FILLED FRACTURES
EI. 498.0
FAULT TRACE
SiO_2 FILLED FRACTURES
BLAST HOLES
EI. 499.5
SHALE, MASSIVE, HARD, DARK GRAY
INTERBED
SS. &
SiO_2 &
28
31
26
27
3
29
30
36
5

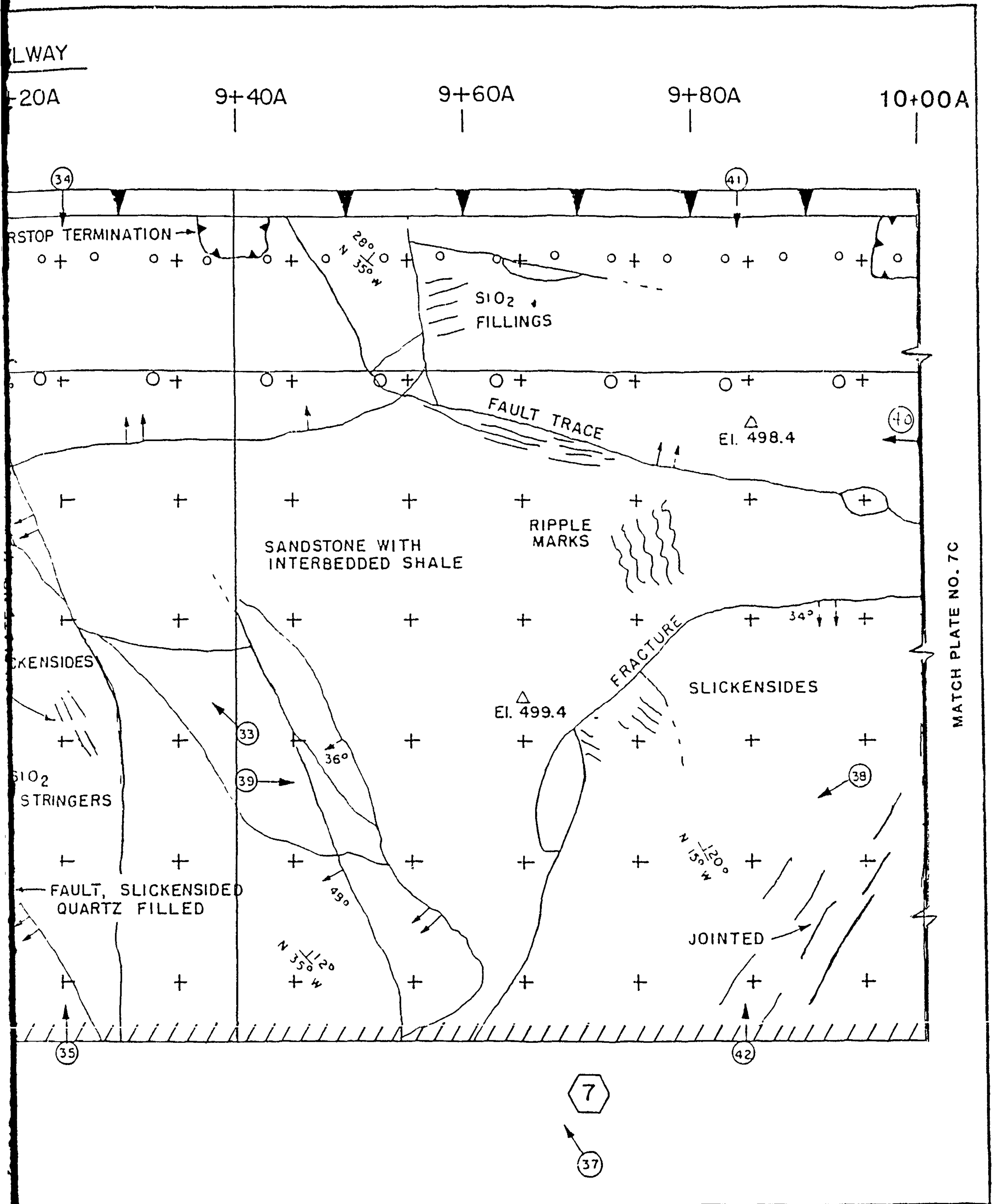


PLAN OF SPILLWAY

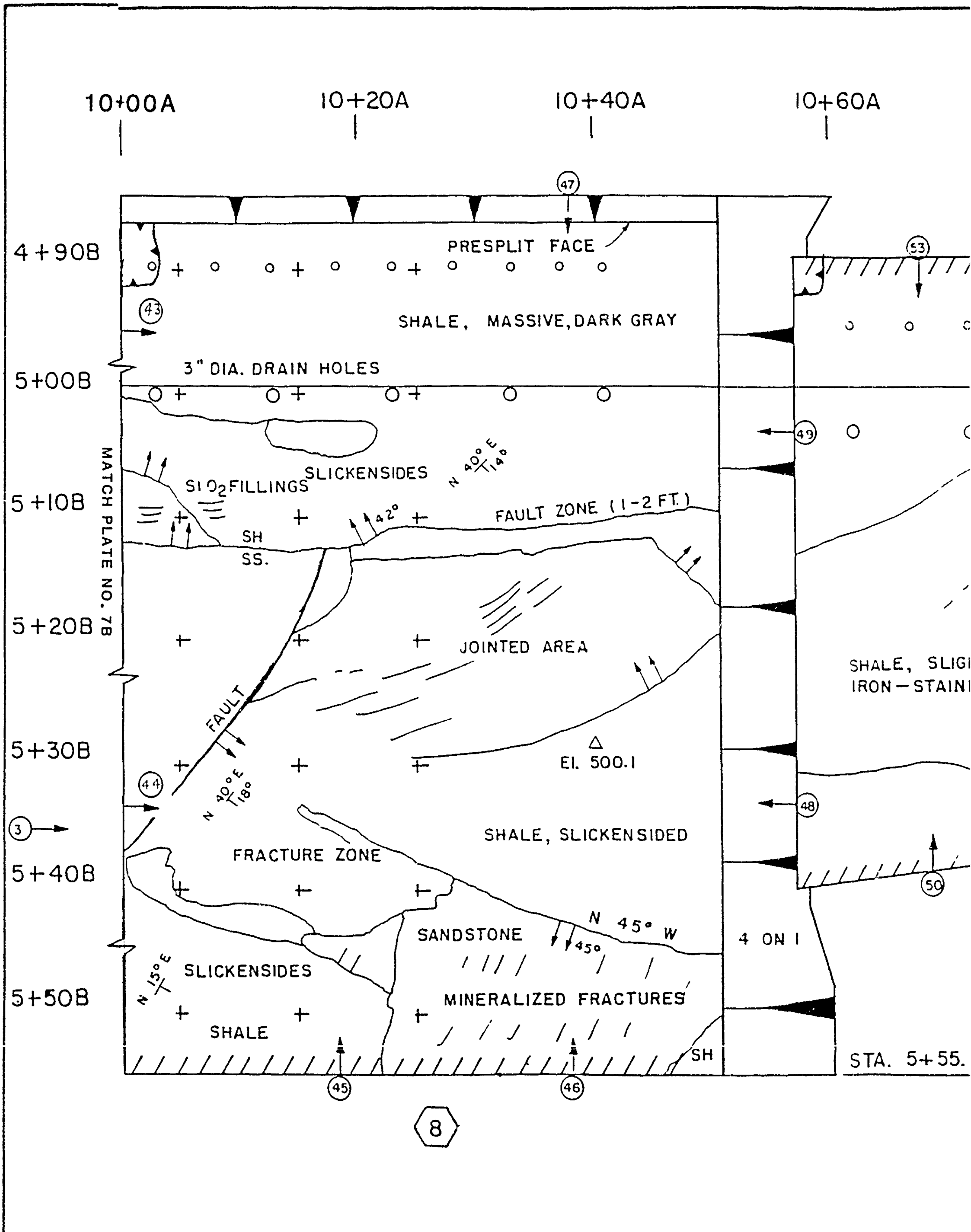


℄ SPILLWAY
9+20A
9+40A
9+60A
9+80A
34
41
WATERSTOP TERMINATION
28° N 35° W
N 10° E 17°
SIO2 FILLINGS
FAULT TRACE
El 498.4
SANDSTONE WITH INTERBEDDED SHALE
RIPPLE MARKS
SLICKENSIDES
FRACTURE
SLICKENSIDES
El. 499.4
33
36°
39
SIO2 STRINGERS
38
N 15° W 20°
FAULT, SLICKENSIDED QUARTZ FILLED
49°
N 35° W 12°
JOINTED
34°
35
42
6
7
37

PILLWAY

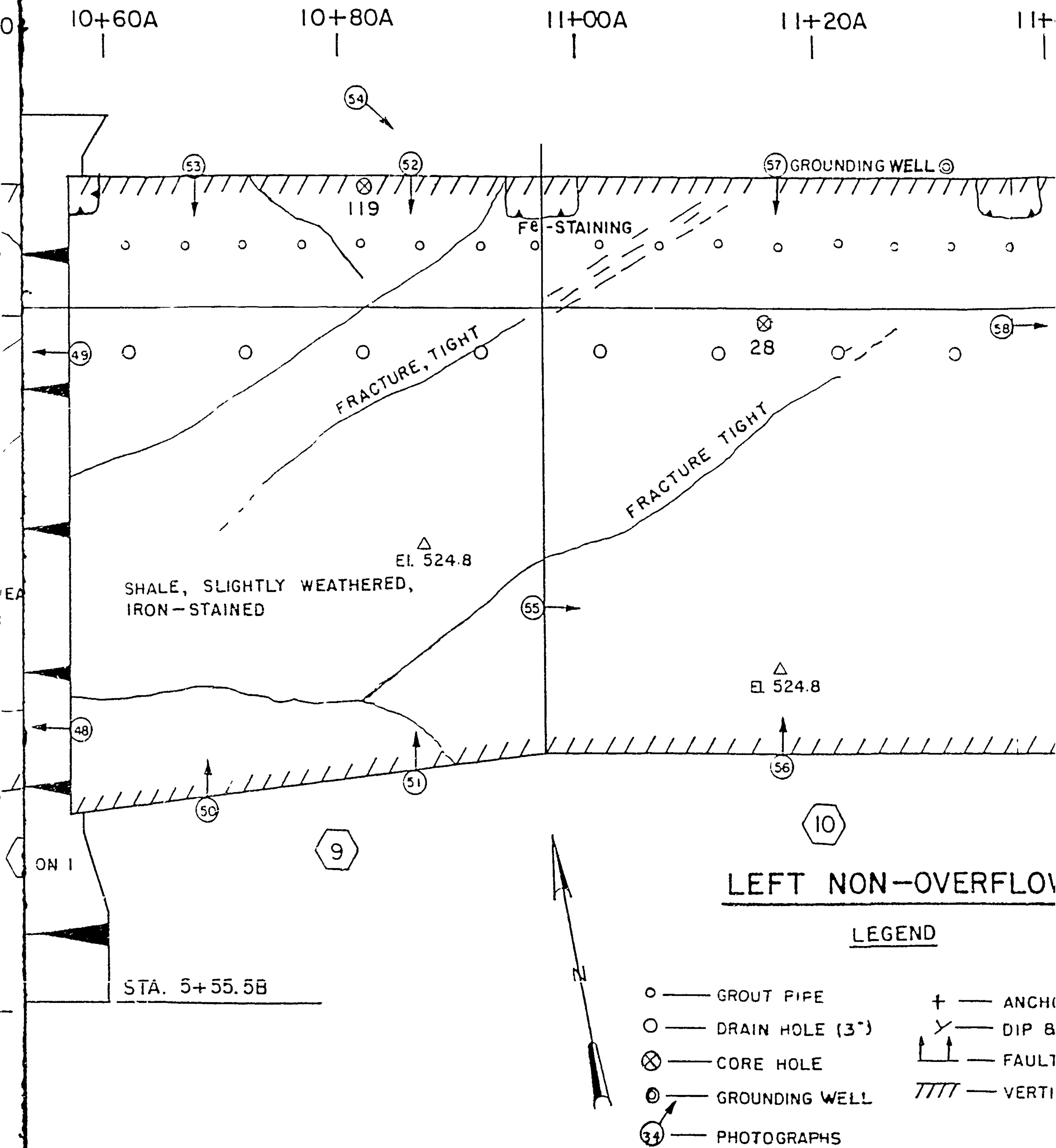


EMBANKMENT CRITERIA AND PERFORMANCE REPORT - PLATE NO. 7 B



10+00A
10+20A
10+40A
10+60A
4+90B
5+00B
5+10B
5+20B
5+30B
5+40B
5+50B
MATCH PLATE NO. 7B
PRESPLIT FACE
SHALE, MASSIVE, DARK GRAY
3" DIA. DRAIN HOLES
SLICKENSIDES
SiO_2 FILLINGS
FAULT ZONE (1-2 FT.)
SH
SS.
42°
N 40° E 14°
JOINTED AREA
FAULT
El. 500.1
N 40° E 18°
SHALE, SLICKENSIDED
FRACTURE ZONE
SANDSTONE
N 45° W
45°
SLICKENSIDES
N 15° E
MINERALIZED FRACTURES
SHALE
SH
4 ON 1
STA. 5+55.
SHALE, SLIG
IRON-STAIN
47
43
53
49
44
3
48
50
45
46

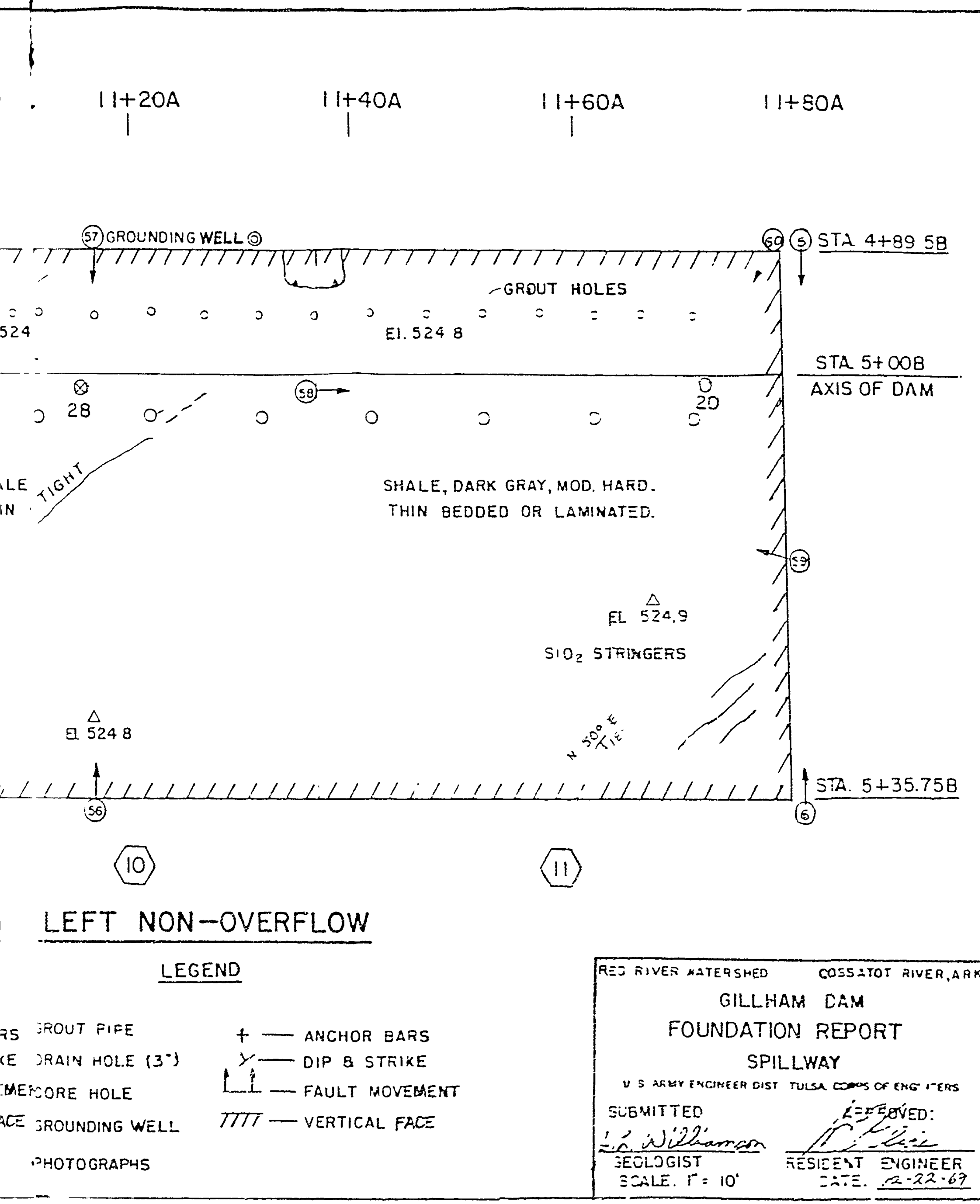
8



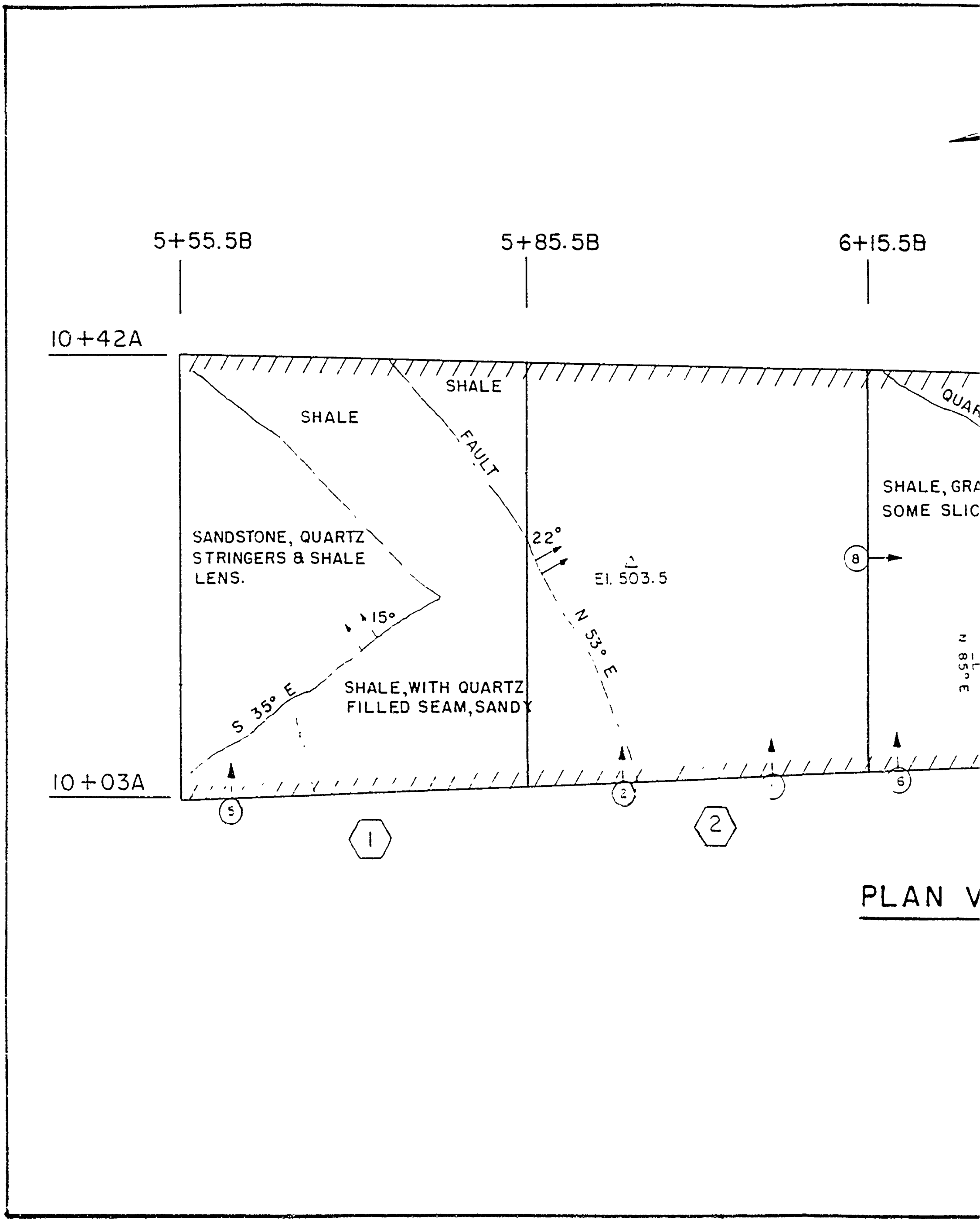
LEFT NON-OVERFLOW

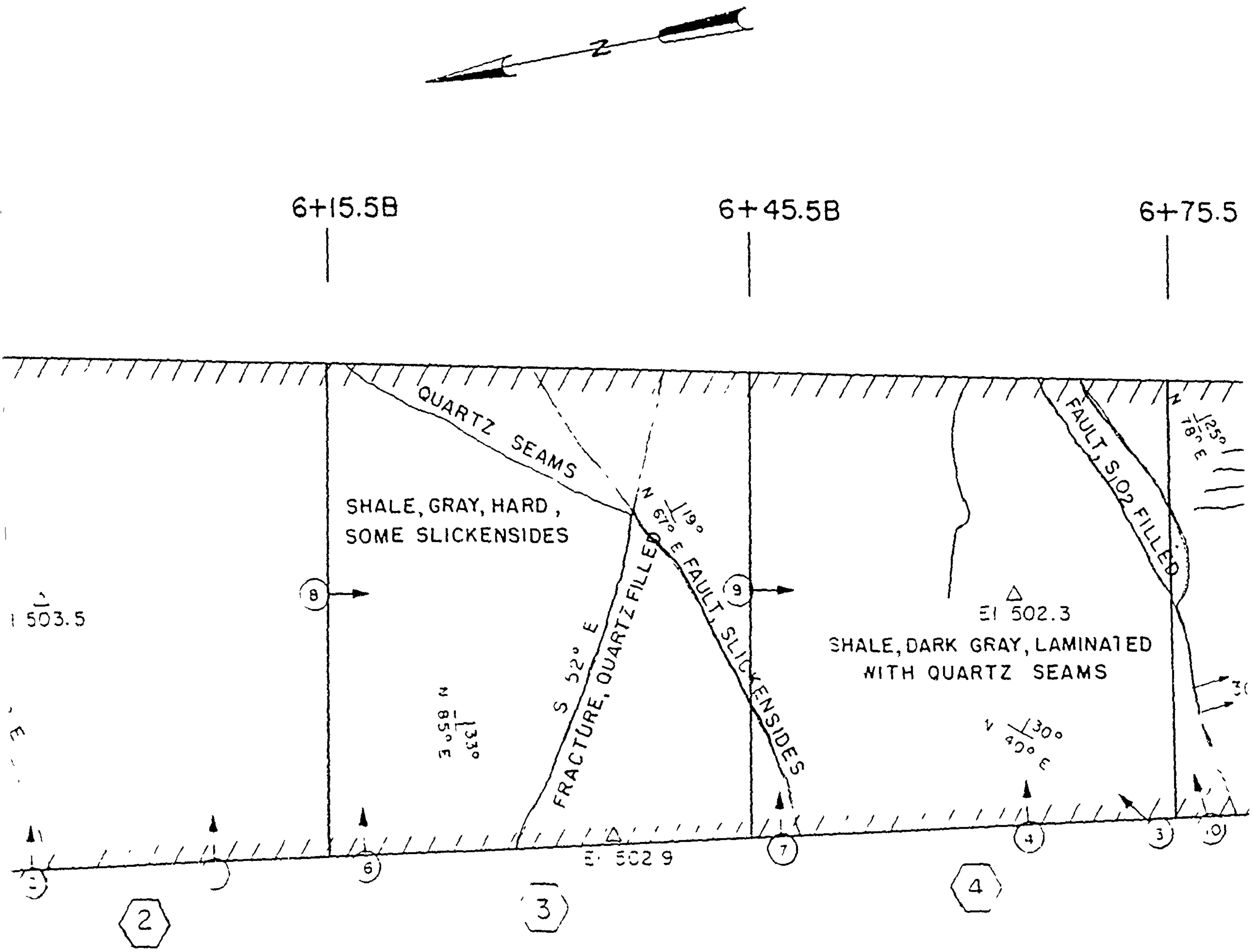
LEGEND

- GROUT PIPE
- DRAIN HOLE (3")
- CORE HOLE
- GROUNDING WELL
- PHOTOGRAPHS
- ANCH
- DIP &
- FAULT
- VERTI

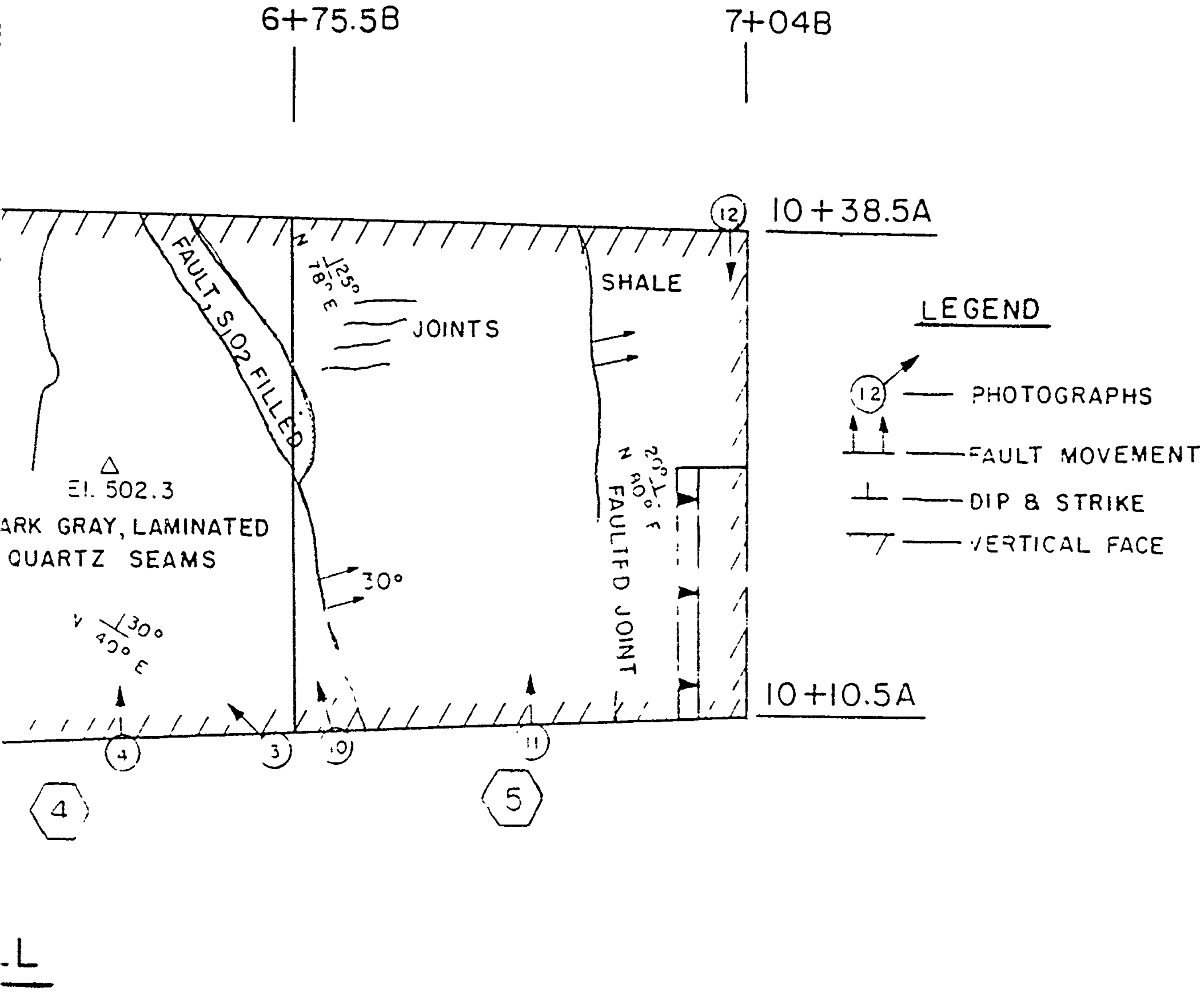


11+20A
11+40A
11+60A
11+80A
57 GROUNDING WELL
60
5 STA 4+89 5B
GROUT HOLES
524
El. 524 8
STA 5+00B
AXIS OF DAM
28
58
20
TIGHT
SHALE, DARK GRAY, MOD. HARD.
THIN BEDDED OR LAMINATED.
59
EL 524.9
SiO_2 STRINGERS
EL 524 8
56
STA. 5+35.75B
6
10
11
LEFT NON-OVERFLOW
LEGEND
GROUT PIPE
DRAIN HOLE (3")
CORE HOLE
GROUNDING WELL
PHOTOGRAPHS
ANCHOR BARS
DIP & STRIKE
FAULT MOVEMENT
VERTICAL FACE
RED RIVER WATERSHED
COSSATOT RIVER, ARK
GILLHAM DAM
FOUNDATION REPORT
SPILLWAY
U S ARMY ENGINEER DIST TULSA CORPS OF ENGINEERS
SUBMITTED
APPROVED:
GEOLOGIST
SCALE. 1" = 10'
RESIDENT ENGINEER
DATE. 12-22-69


5+55.5B
5+85.5B
6+15.5B
10+42A
10+03A
SHALE
SHALE
FAULT
QUAR
SHALE, GRA
SOME SLIC
SANDSTONE, QUARTZ
STRINGERS & SHALE
LENS.
22°
El. 503.5
8
15°
N 53° E
N 85° E
S 35° E
SHALE, WITH QUARTZ
FILLED SEAM, SANDY
5
2
6
1
2
PLAN V



PLAN VIEW—' EFT CHUTE WALL



RED RIVER W RSHED COSSATOT RIVER, ARK

GILLHAM DAM

FOUNDATION REPORT

LEFT CHUTE WALL

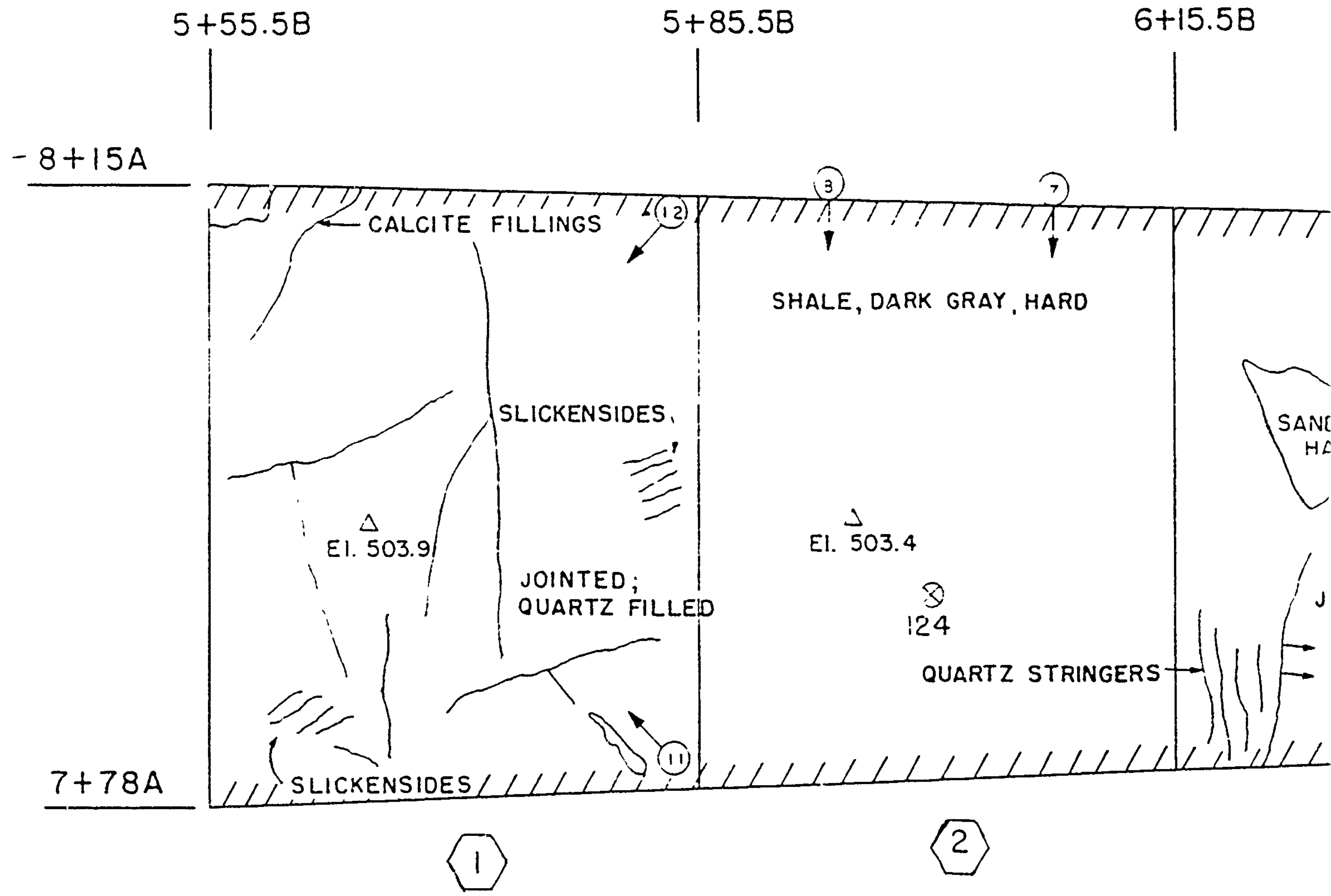
U S ARMY ENGINEER DIST TULSA CORPS OF ENGINEERS

SUBMITTED: J.L. Williamson — GEOLOGIST

APPROVED: — RESIDENT ENGINEER

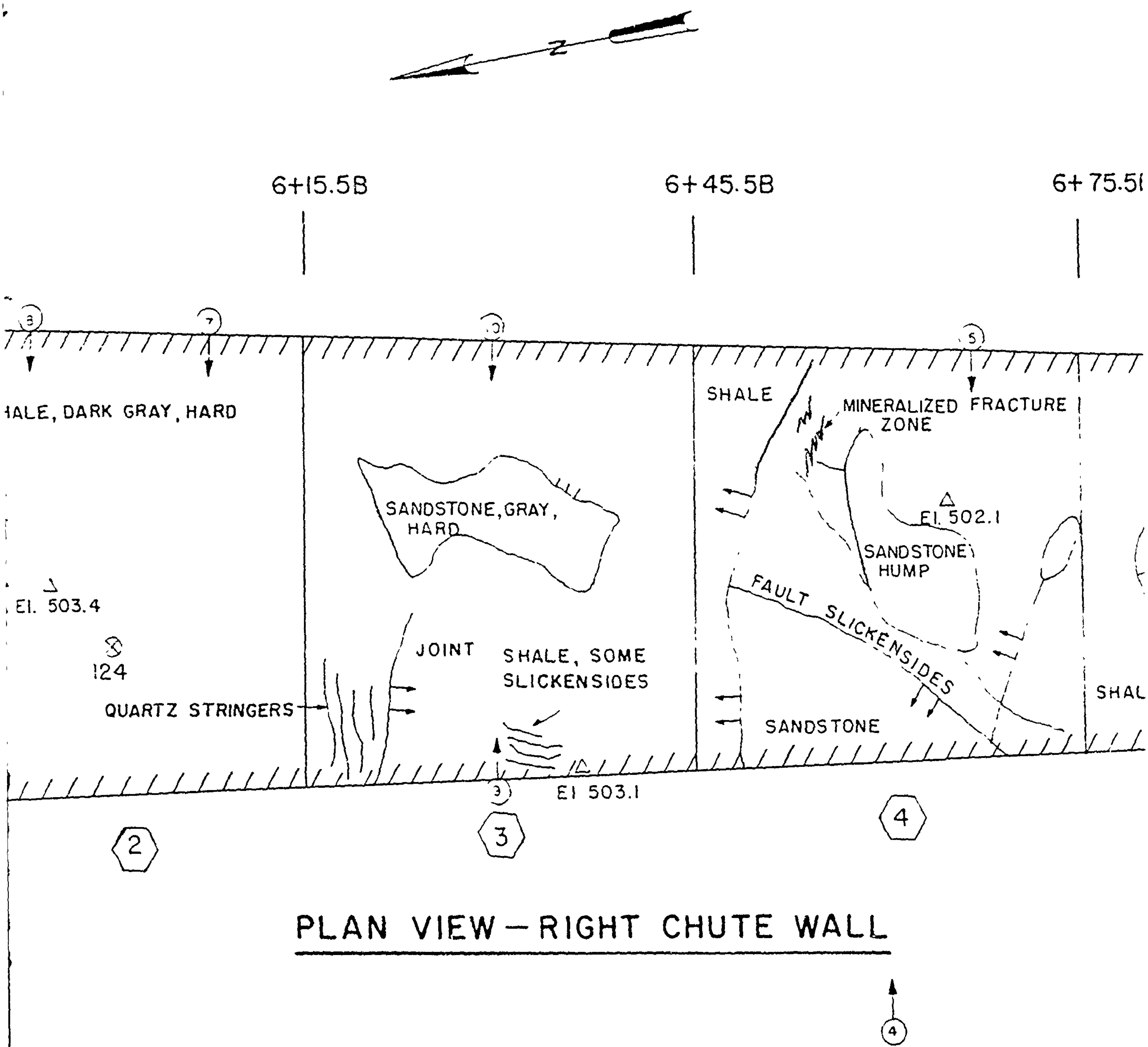
SCALE 1" = 10'

DATE: 12-22-69

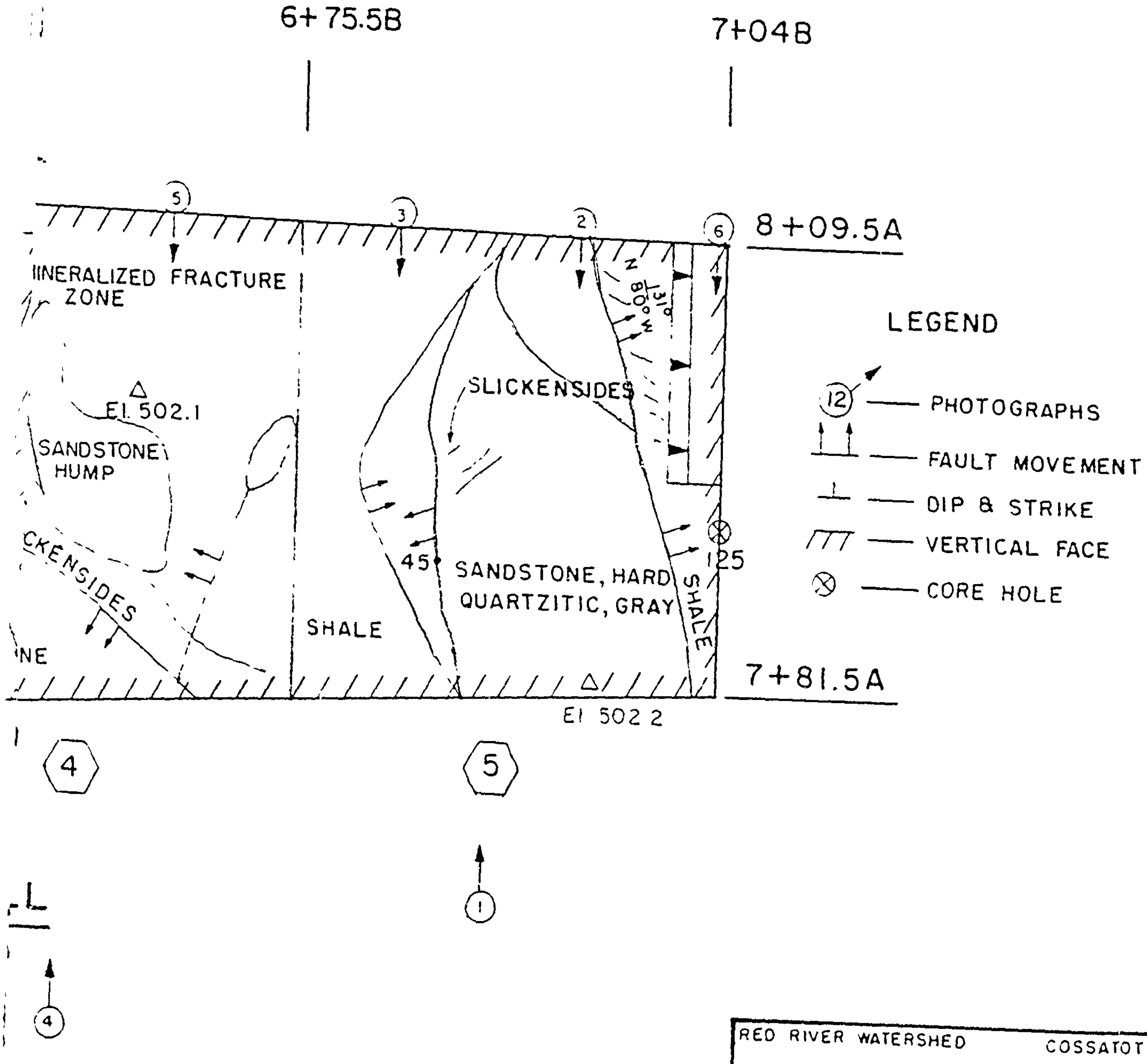


5+55.5B
5+85.5B
6+15.5B
8+15A
7+78A
CALCITE FILLINGS
12
3
7
SHALE, DARK GRAY, HARD
SLICKENSIDES
El. 503.9
El. 503.4
JOINTED; QUARTZ FILLED
124
QUARTZ STRINGERS
11
SLICKENSIDES
1
2

PLAN V



PLAN VIEW — RIGHT CHUTE WALL



RED RIVER WATERSHED COSSATOT RIVER, ARK

GILLHAM DAM
FOUNDATION REPORT
RIGHT CHUTE WALL

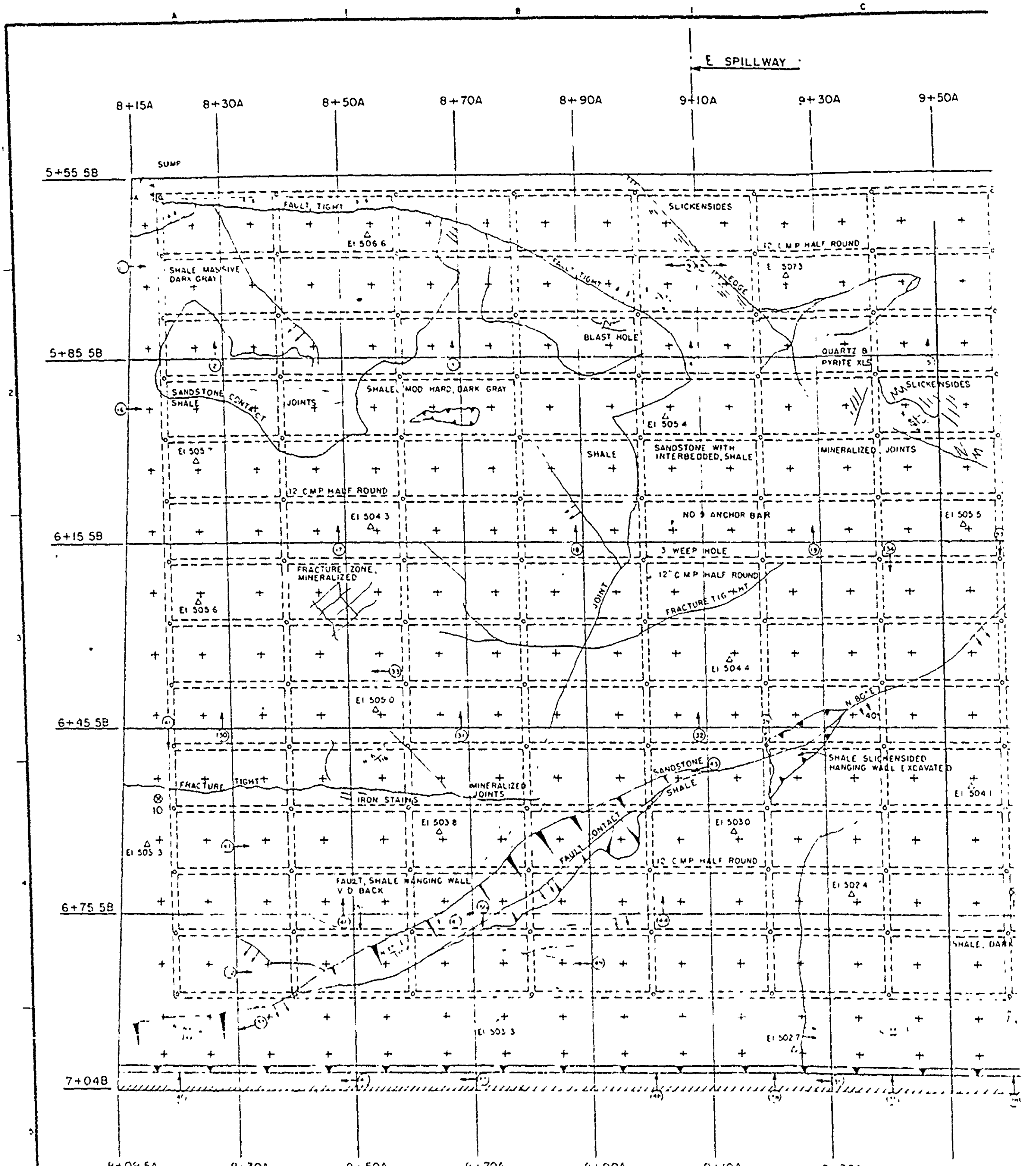
U S ARMY ENGINEER DIST TULSA CORPS OF ENGINEERS

SUBMITTED: J. L. Williamson, GEOLOGIST

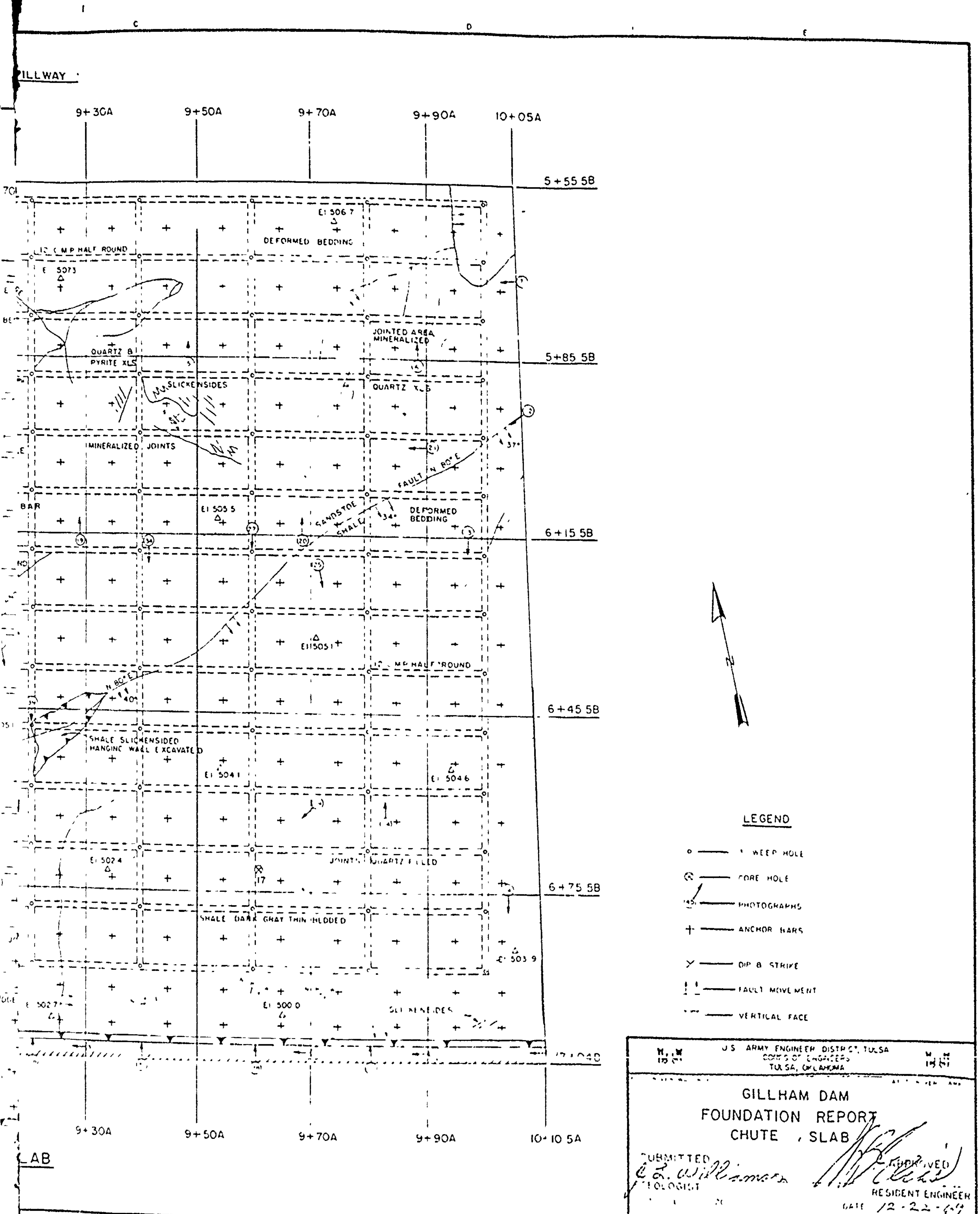
APPROVED: RESIDENT ENGINEER

SCALE. 1" = 10'

DATE: 12-22-69



PLAN OF CHUTE SLAB



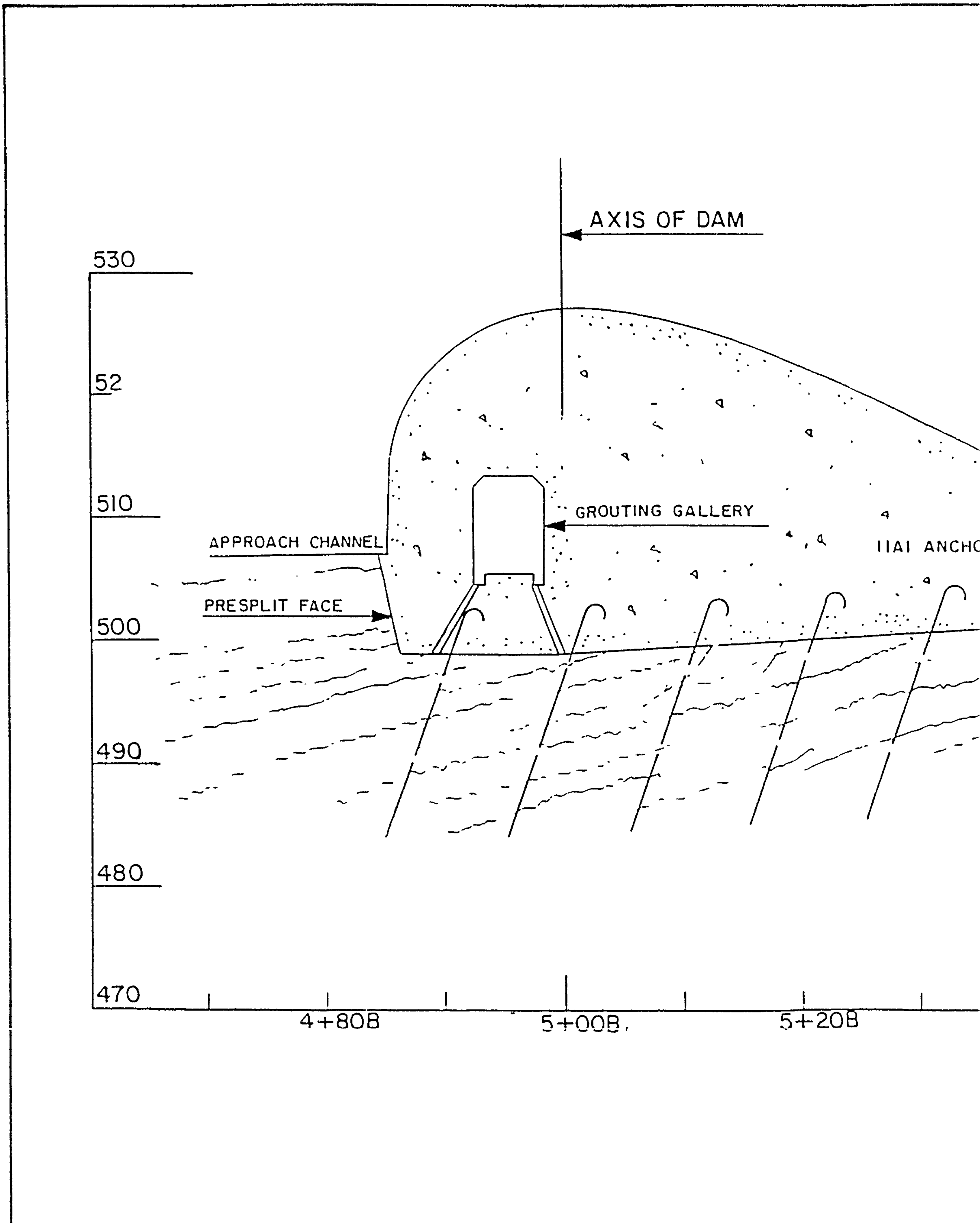
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

GILLHAM DAM
FOUNDATION REPORT
CHUTE SLAB

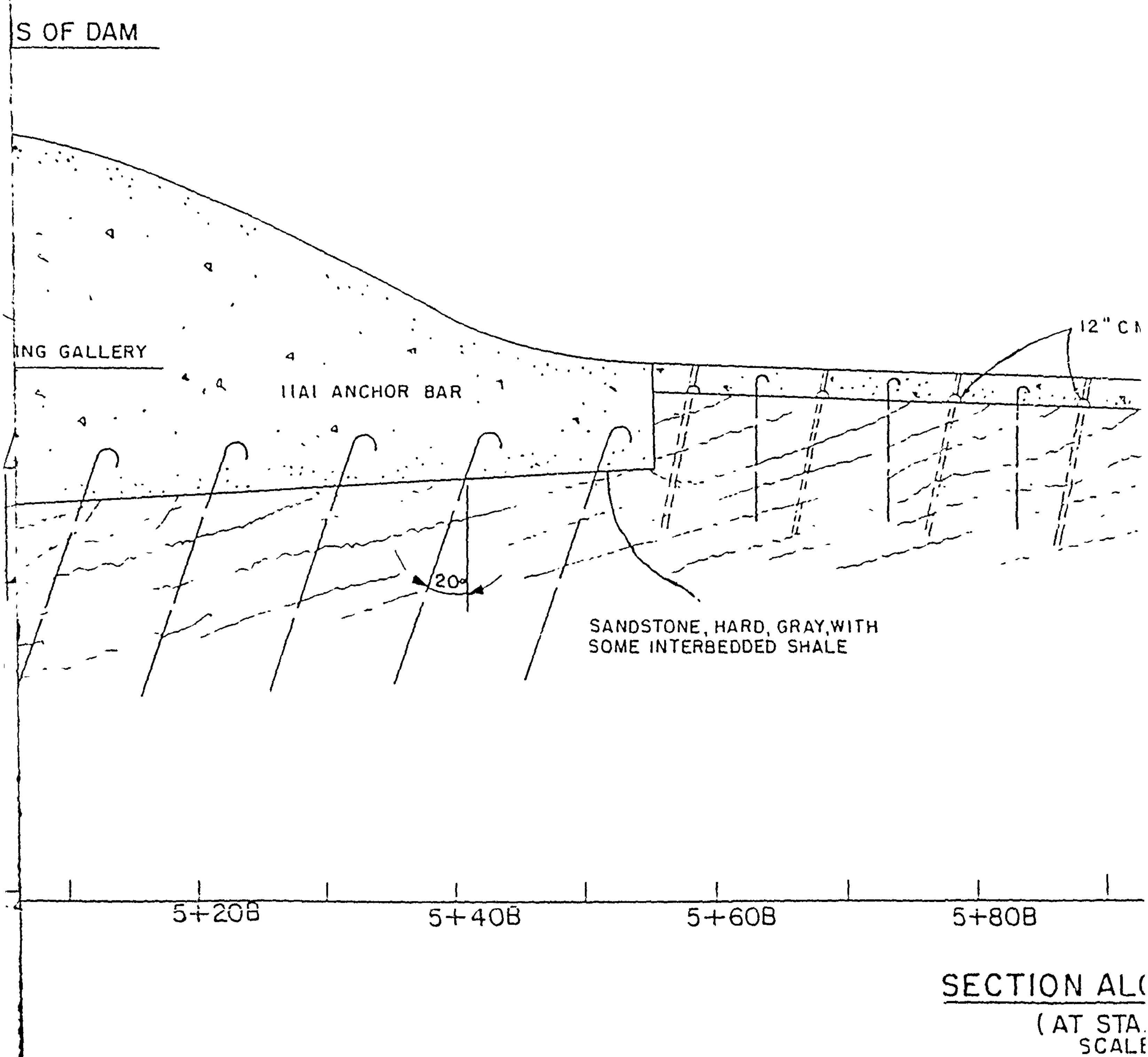
SUBMITTED GEOLOGIST

APPROVED RESIDENT ENGINEER

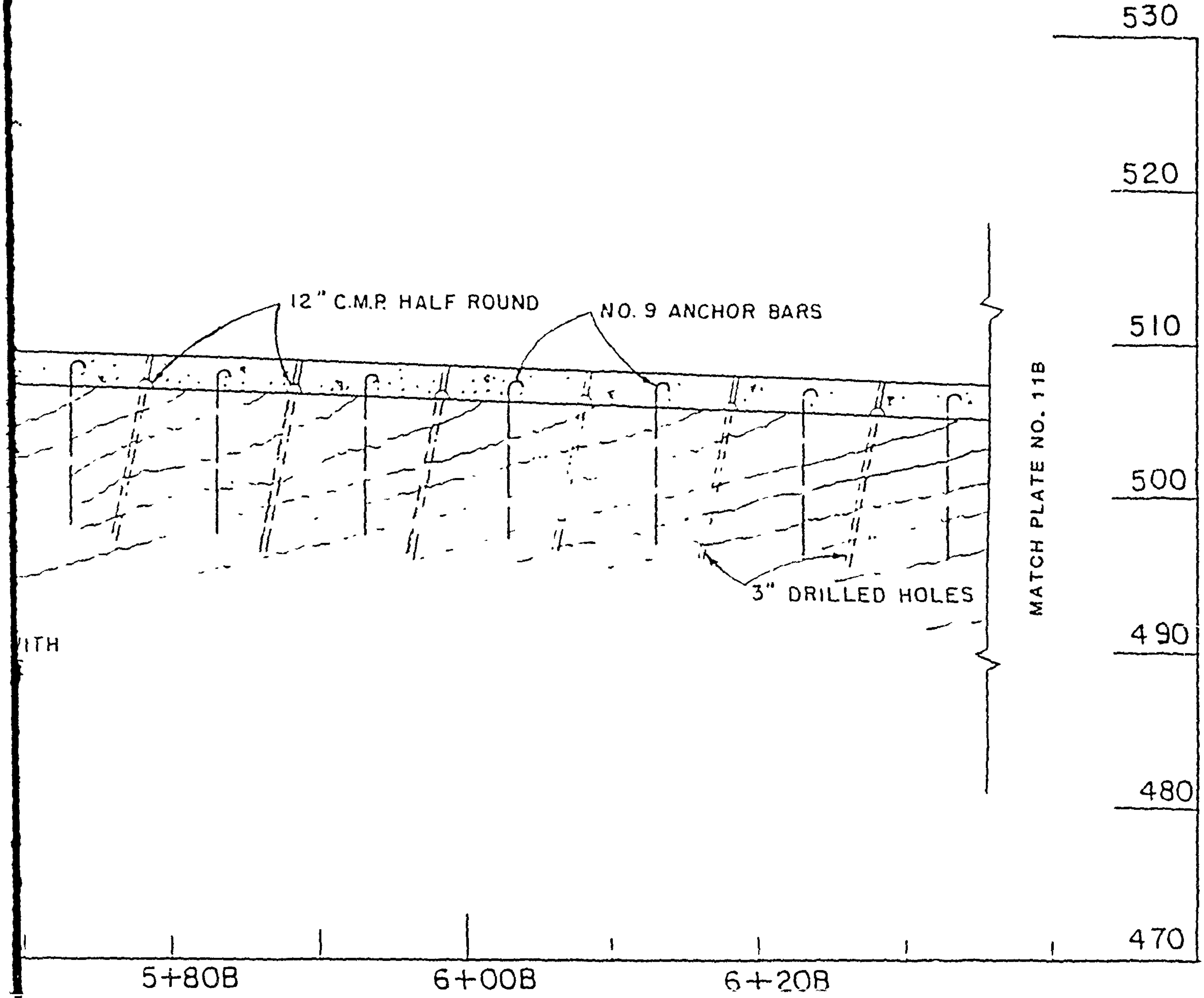
DATE 12-22-69



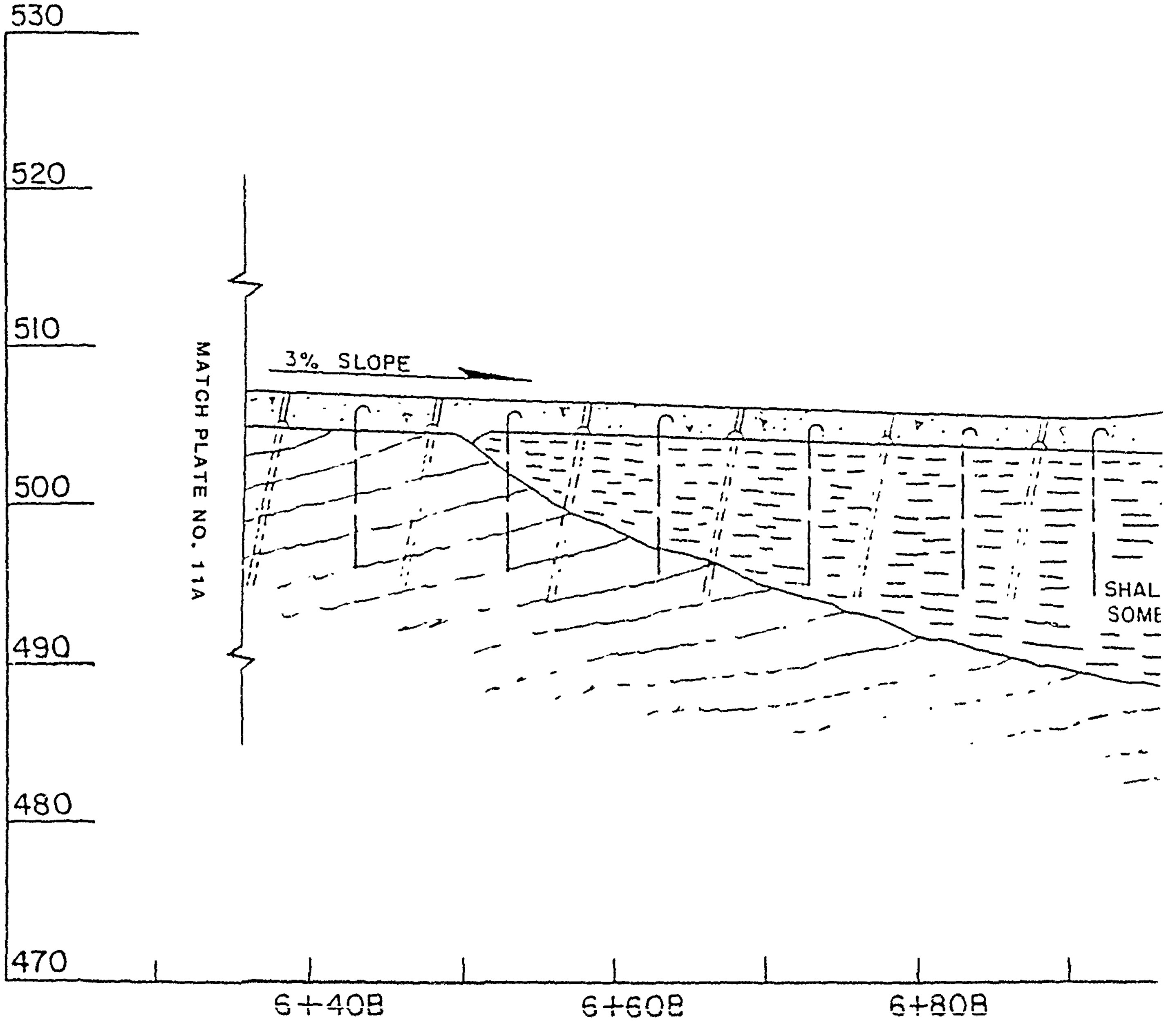
AXIS OF DAM
530
52
510
500
490
480
470
APPROACH CHANNEL
PRESPLIT FACE
GROUTING GALLERY
IIAI ANCHC
4+80B
5+00B
5+20B



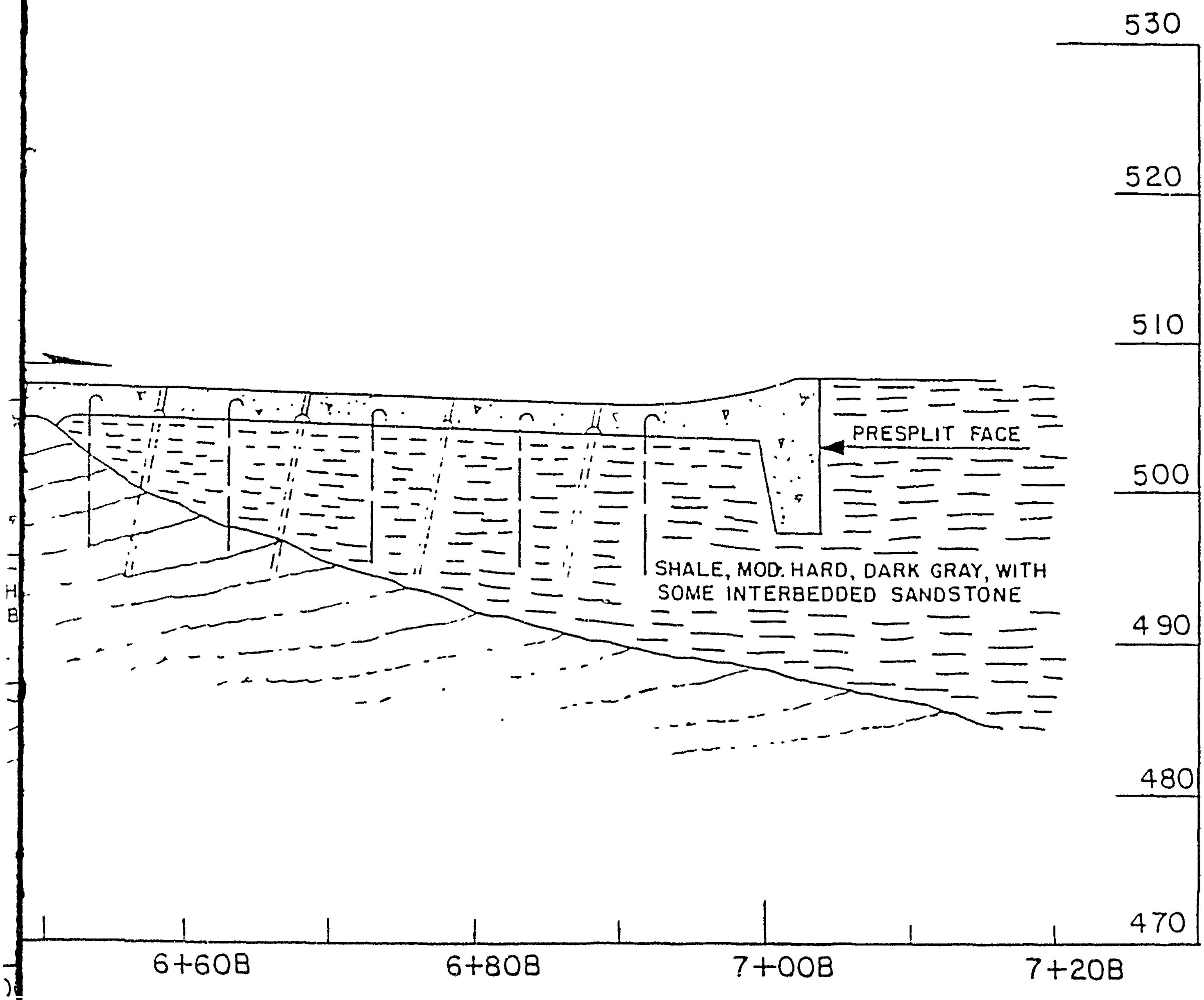
S OF DAM
ING GALLERY
11A1 ANCHOR BAR
12" C
20°
SANDSTONE, HARD, GRAY, WITH
SOME INTERBEDDED SHALE
5+20B
5+40B
5+60B
5+80B
SECTION AL
(AT STA.
SCALE



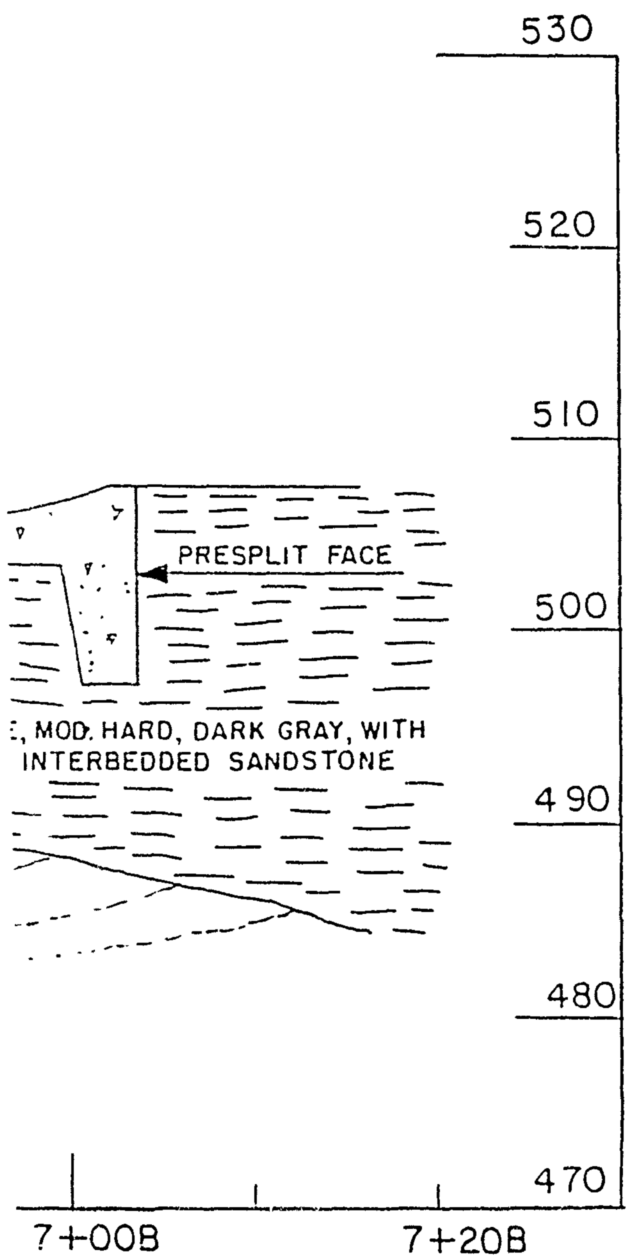
SECTION ALONG ℄ SPILLWAY
(AT STA. 9+10A)
SCALE: 1"=10'



530
520
510
500
490
480
470
MATCH PLATE NO. 11A
3% SLOPE
SHAL
SOME
6+40B
6+60B
6+80B



PRESPLIT FACE
SHALE, MOD. HARD, DARK GRAY, WITH SOME INTERBEDDED SANDSTONE
530
520
510
500
490
480
470
6+60B
6+80B
7+00B
7+20B



RED RIVER WATERSHED — COSSATOT RIVER, ARK

GILLHAM DAM

FOUNDATION REPORT

SECTION SPILLWAY & CHUTE SLAB

U S ARMY ENGINEER DIST TULSA. CORPS OF ENGINEERS

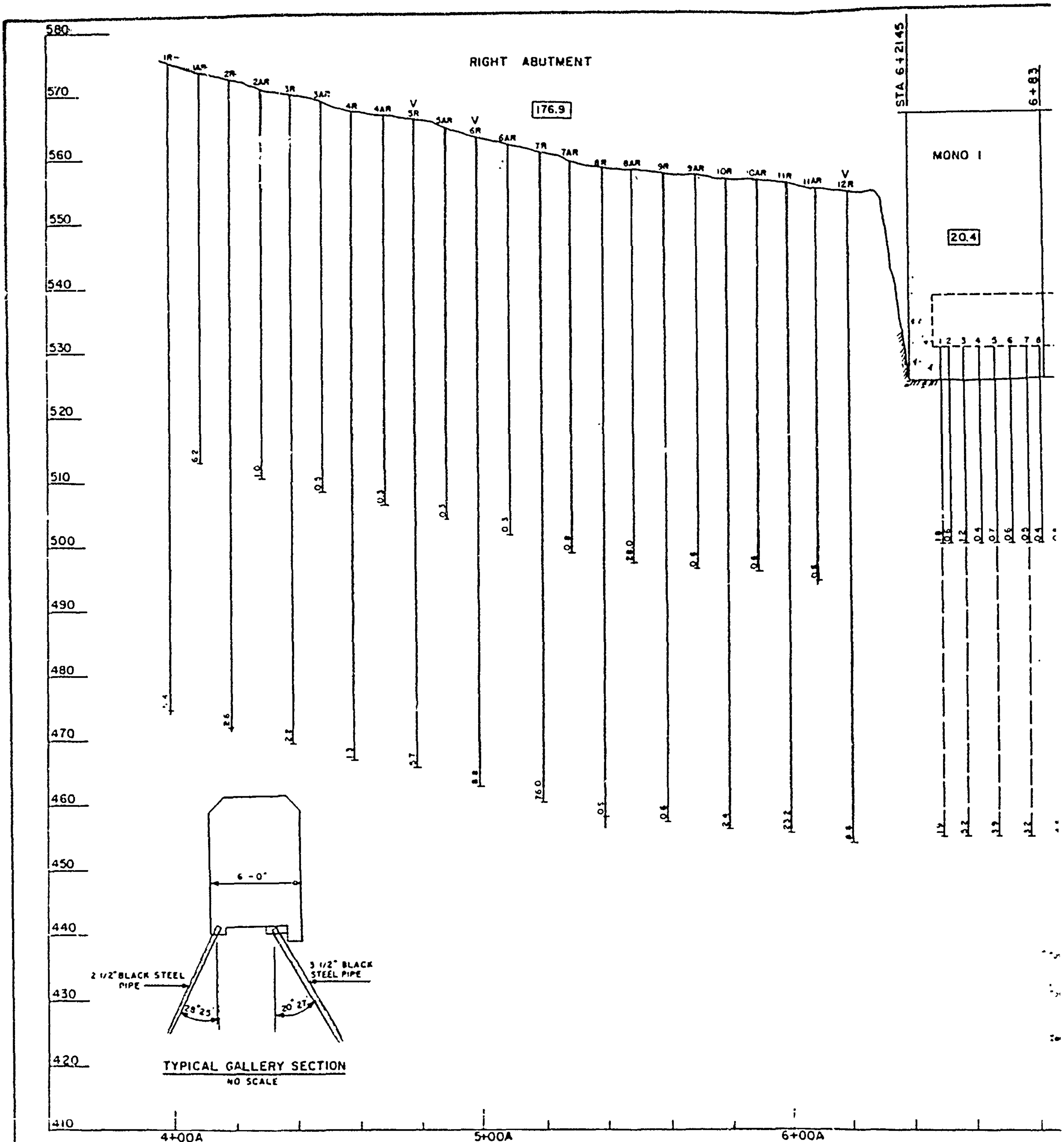
SUBMITTED: J. L. Williamson, GEOLOGIST

APPROVED: [signature], RESIDENT ENGINEER

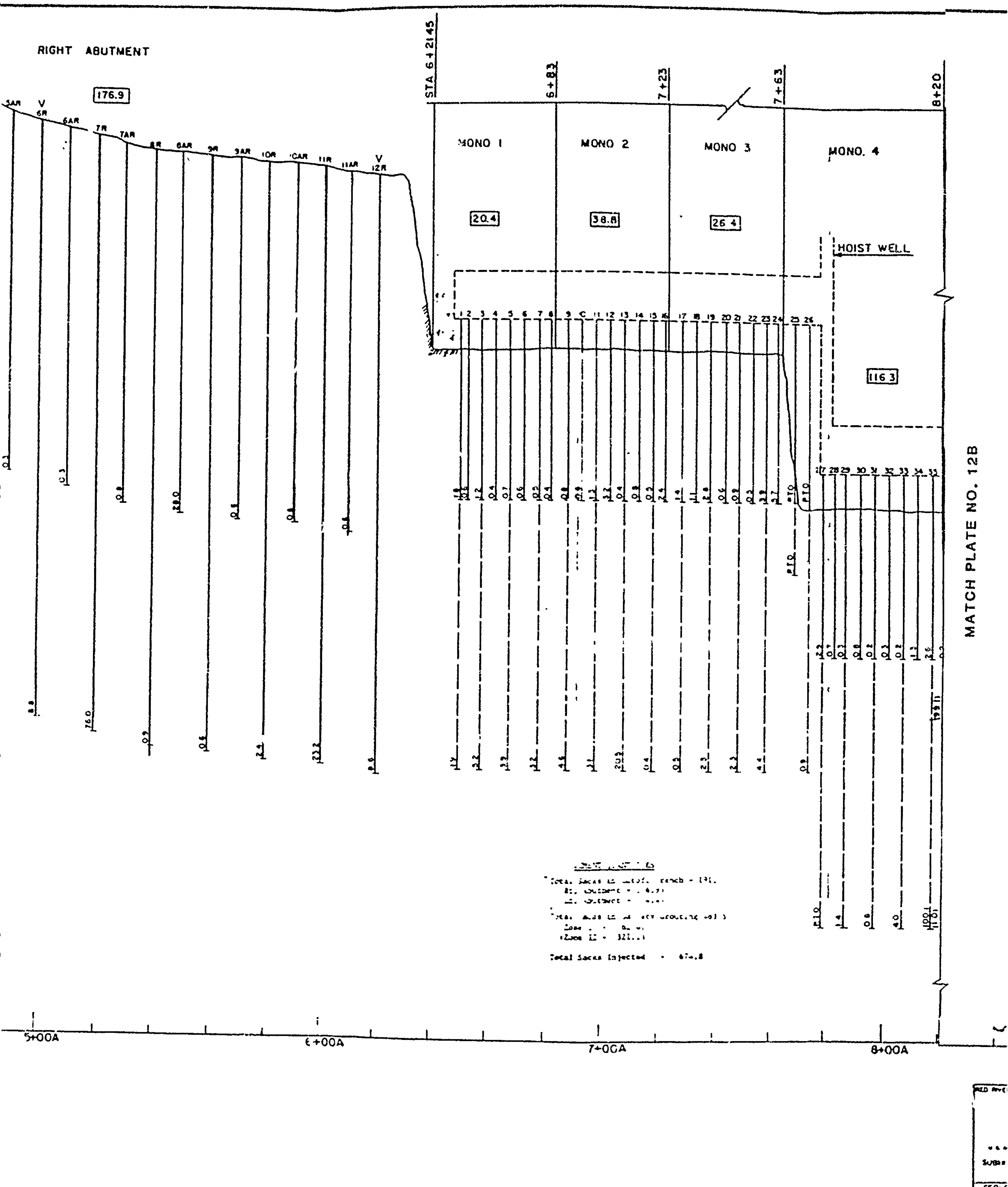
SCALE. 1" = 10'

DATE: 12-22-69

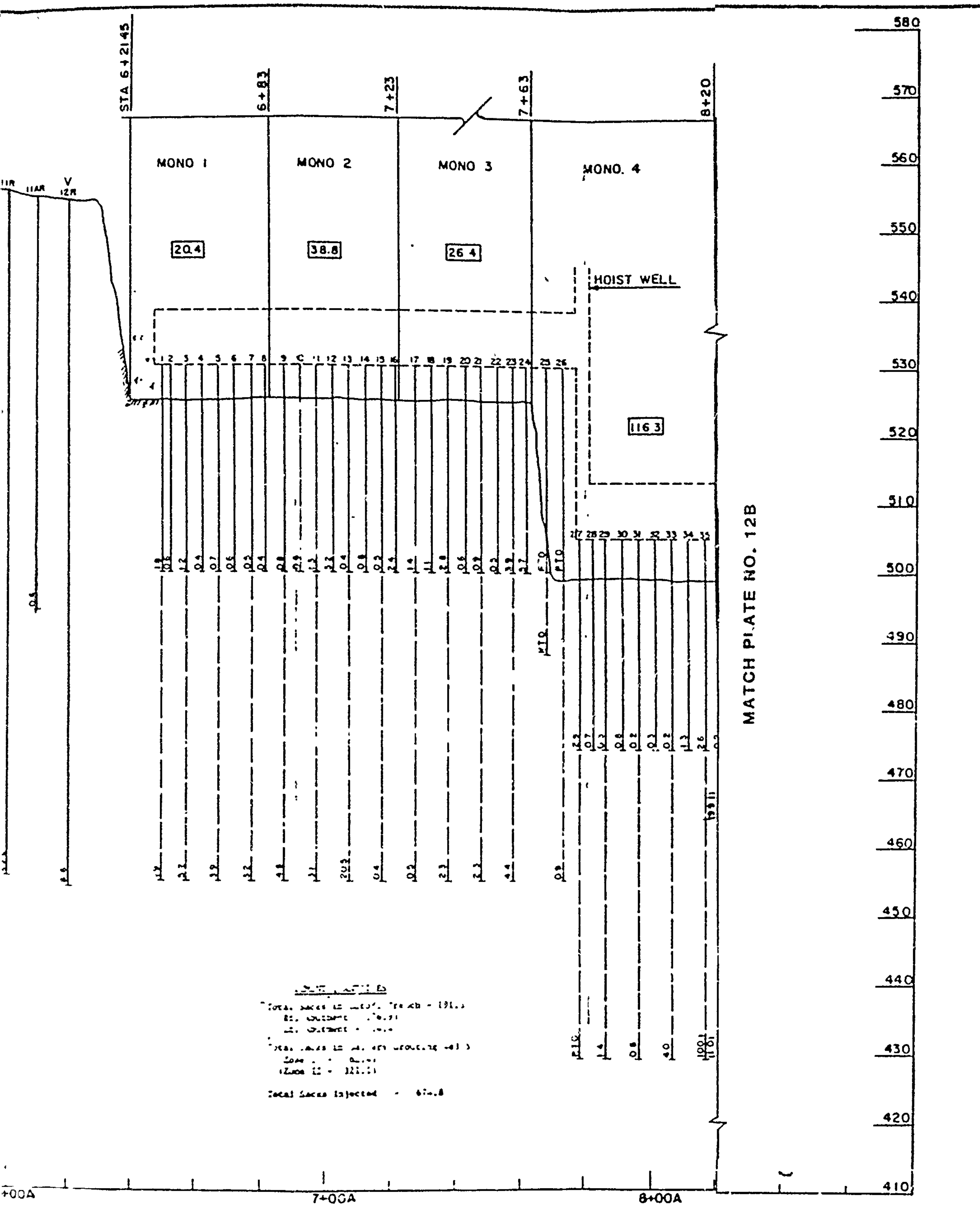
MB.



RIGHT ABUTMENT
176.9
STA 6+21.45
6+83
MONO 1
20.4
580
570
560
550
540
530
520
510
500
490
480
470
460
450
440
430
420
410
4+00A
5+00A
6+00A
6 - 0"
2 1/2" BLACK STEEL PIPE
3 1/2" BLACK STEEL PIPE
28°25'
20°27'
TYPICAL GALLERY SECTION
NO SCALE



EMBANKMENT CRITERIA AND PERFORMANCE R



RED RIVER WATERSHED — COSSATOT RIVER ARK

GILLHAM DAM

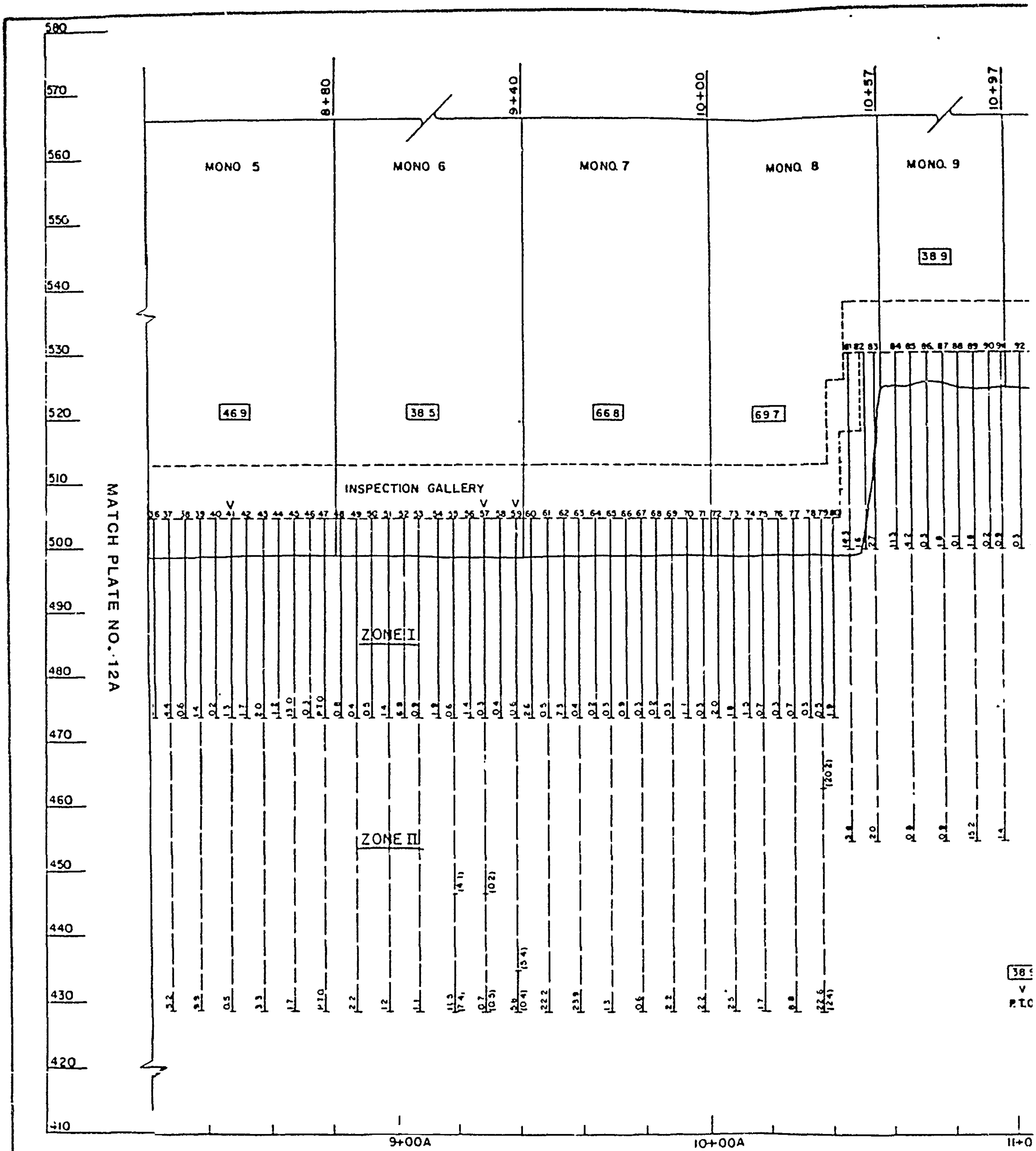
FOUNDATION REPORT

GROUTING PROFILE

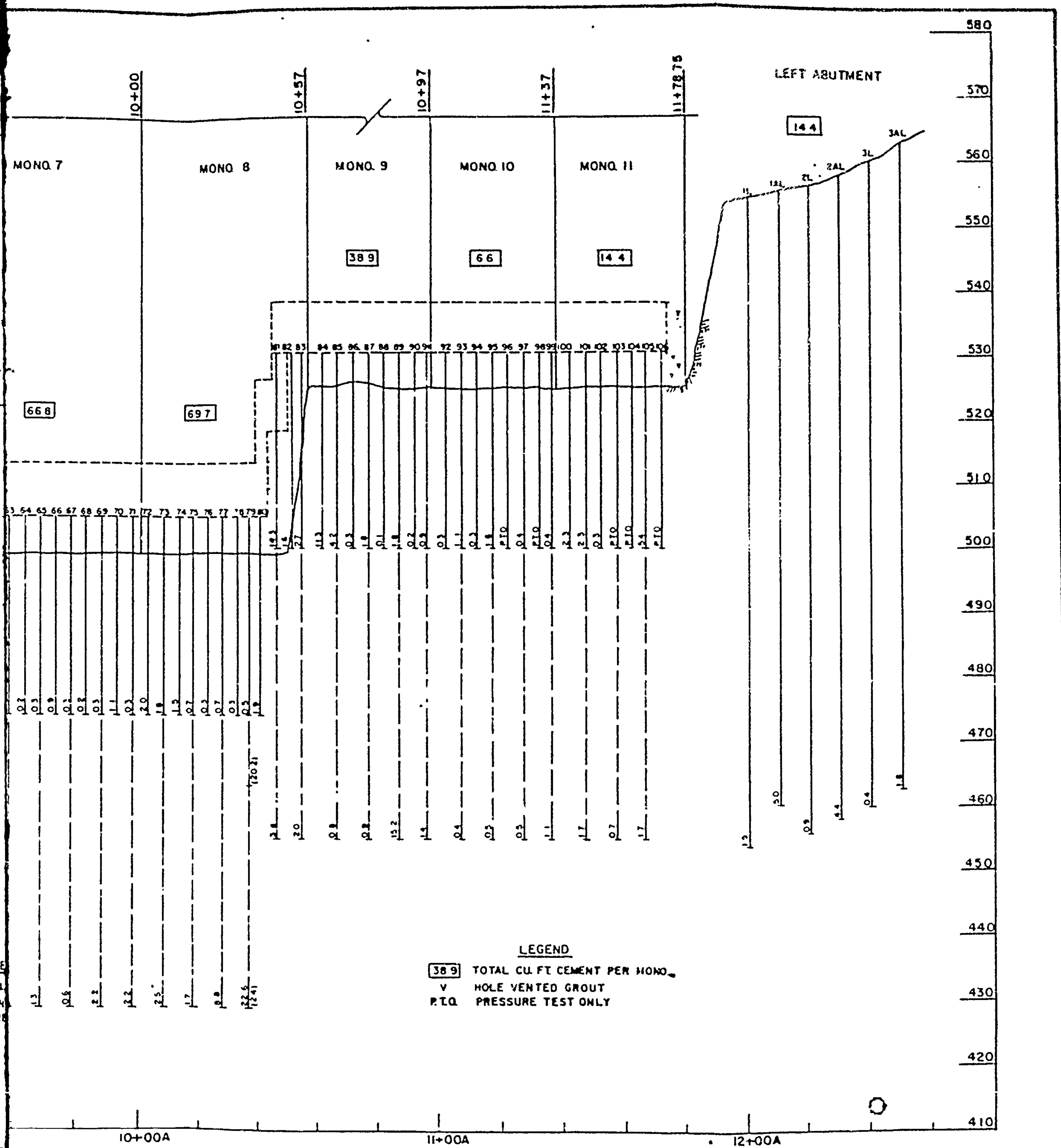
SUBMITTED

GEOLOGIST — RESIDENT ENGINEER

SCALE — DATE



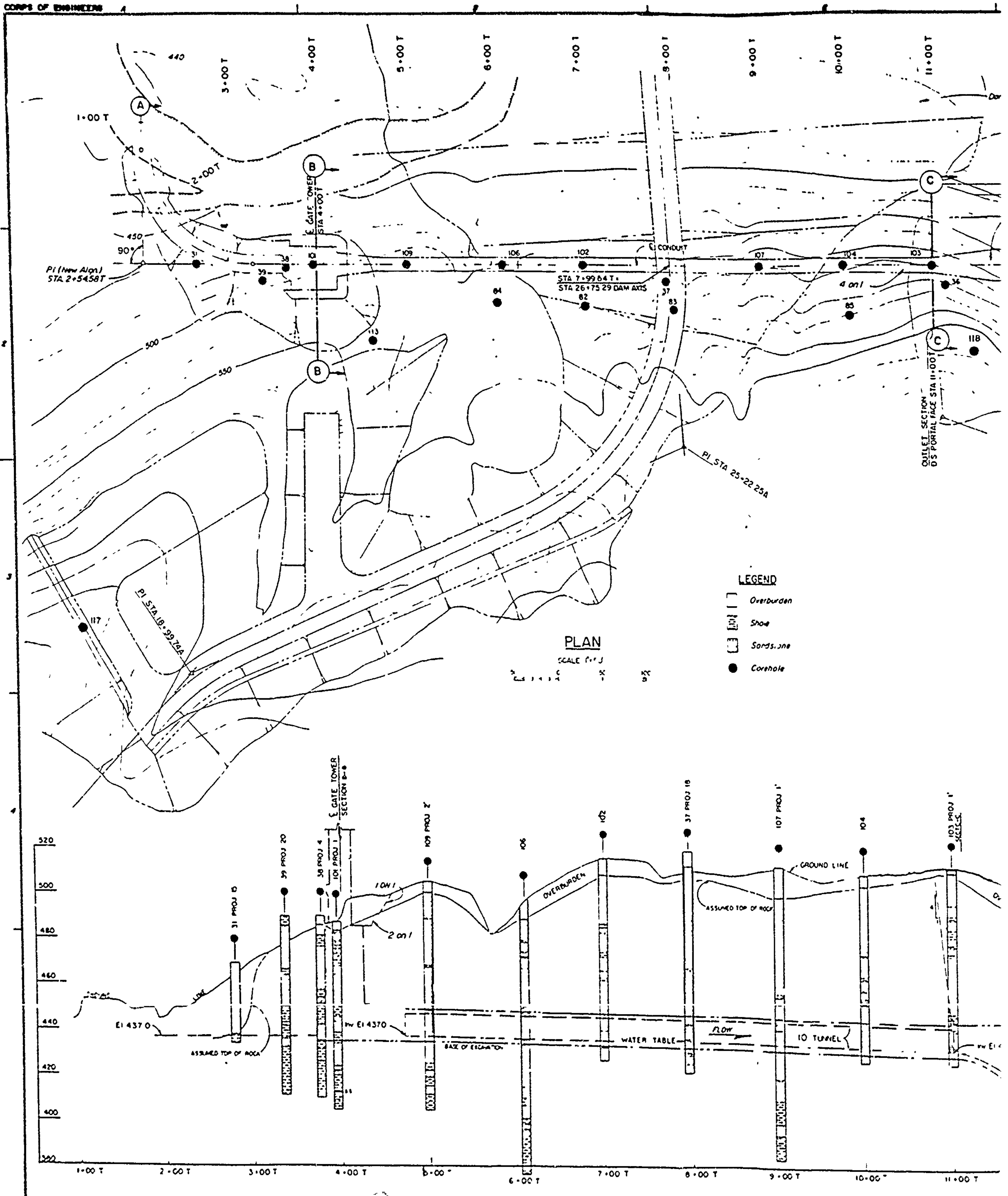
NOTE
CUBIC FEET (SACK) CEMENT INJECTED
IS INDICATED AT BOTTOM OF EACH
GROUT HOLE



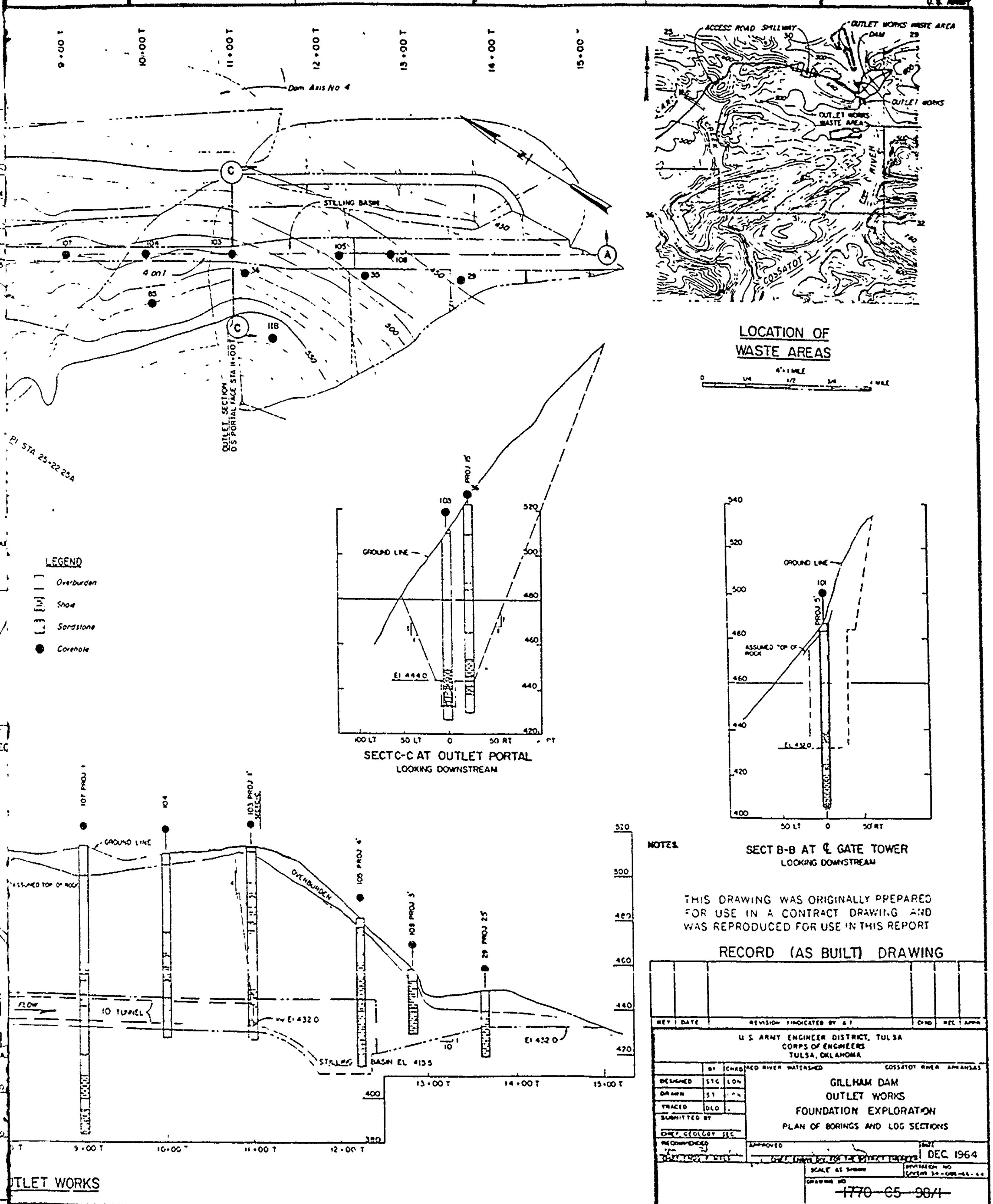
NOTE
CUBIC FEET (SACK) CEMENT INJECTED IS INDICATED AT BOTTOM OF EACH GROUT HOLE

RED RIVER WATERSHED — COSSATOT RIVER, ARK.
GILLHAM DAM
FOUNDATION REPORT
GROUTING PROFILE
SUBMITTED
GEOLOGIST — RESIDENT ENGINEER
DATE

CORPS OF ENGINEERS

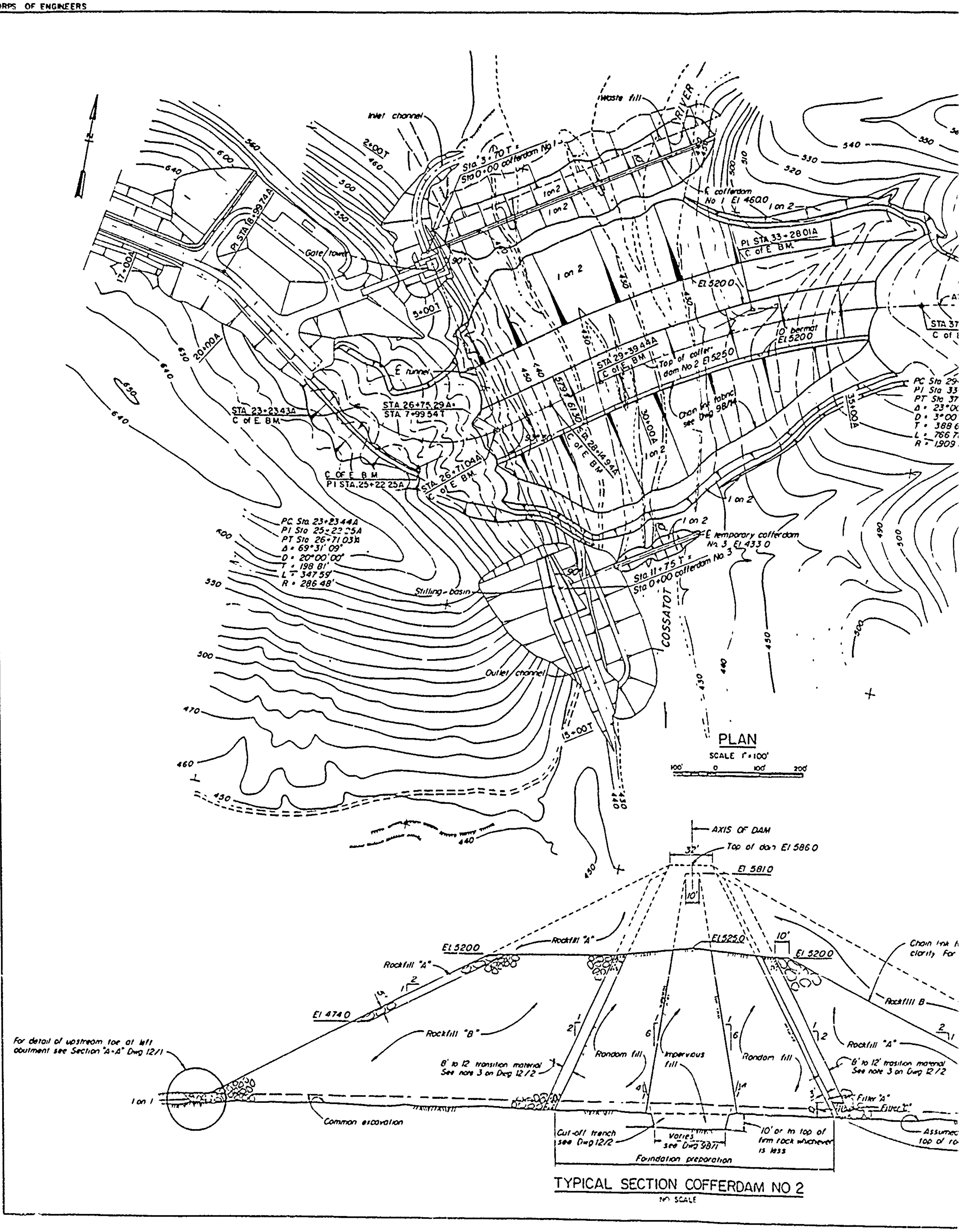


SECT A-A ALONG CENTER LINE OF OUTLET WORKS

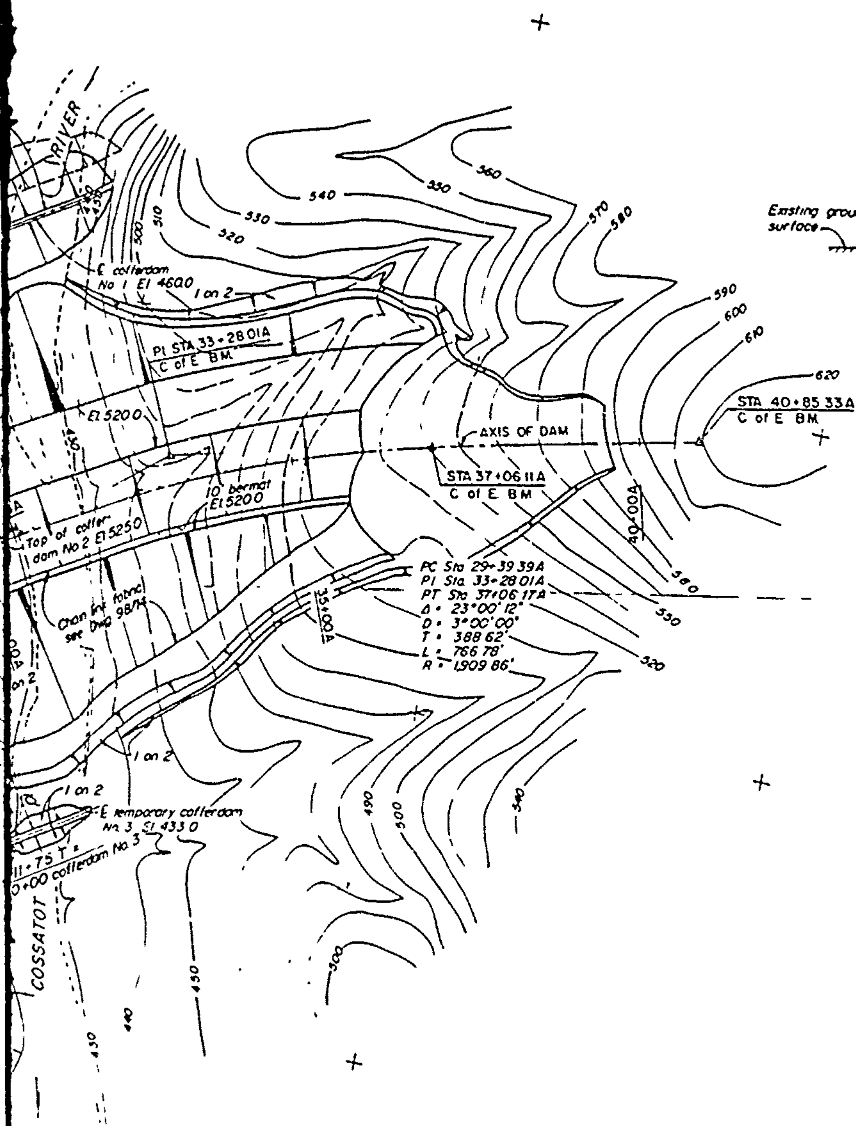


U.S. ARMY
9+00 T
10+00 T
11+00 T
12+00 T
13+00 T
14+00 T
15+00 T
Dam Axis No 4
STILLING BASIN
4 on 1
OUTLET SECTION
D/S PORTAL FACE STA 11+00T
PI STA 25+22.254
ACCESS ROAD SPILLWAY
OUTLET WORKS WASTE AREA
DAM
OUTLET WORKS
OUTLET WORKS WASTE AREA
COSSATOT
RIVER
LOCATION OF WASTE AREAS
LEGEND
Overburden
Shale
Sandstone
Corehole
GROUND LINE
EL 444.0
SECT C-C AT OUTLET PORTAL
LOOKING DOWNSTREAM
ASSUMED TOP OF ROCK
EL 432.0
SECT B-B AT ℄ GATE TOWER
LOOKING DOWNSTREAM
ASSUMED TOP OF ROCK
OVERBURDEN
FLOW
10 TUNNEL
EL 432.0
STILLING BASIN EL 415.5
NOTES.
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
RECORD (AS BUILT) DRAWING
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER ARKANSAS
GILLHAM DAM
OUTLET WORKS
FOUNDATION EXPLORATION
PLAN OF BORINGS AND LOG SECTIONS
DEC 1964
1770-C5-98/1
OUTLET WORKS

CORPS OF ENGINEERS

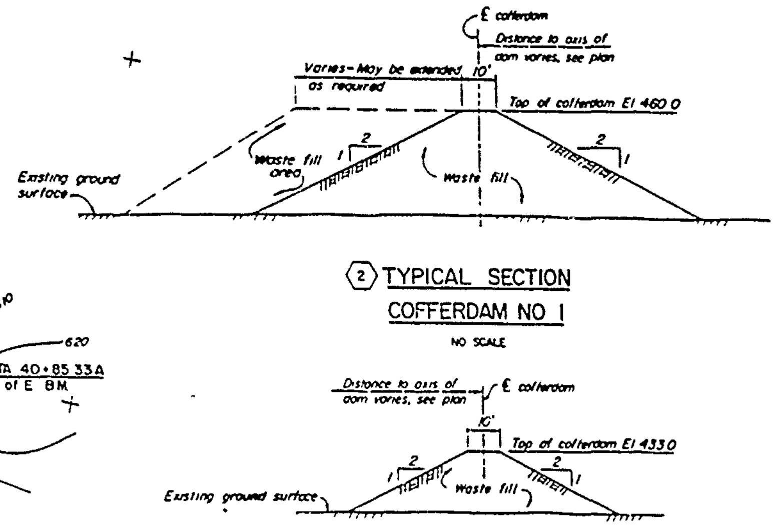


U.S. ARMY



PLAN

SCALE 1" = 100'



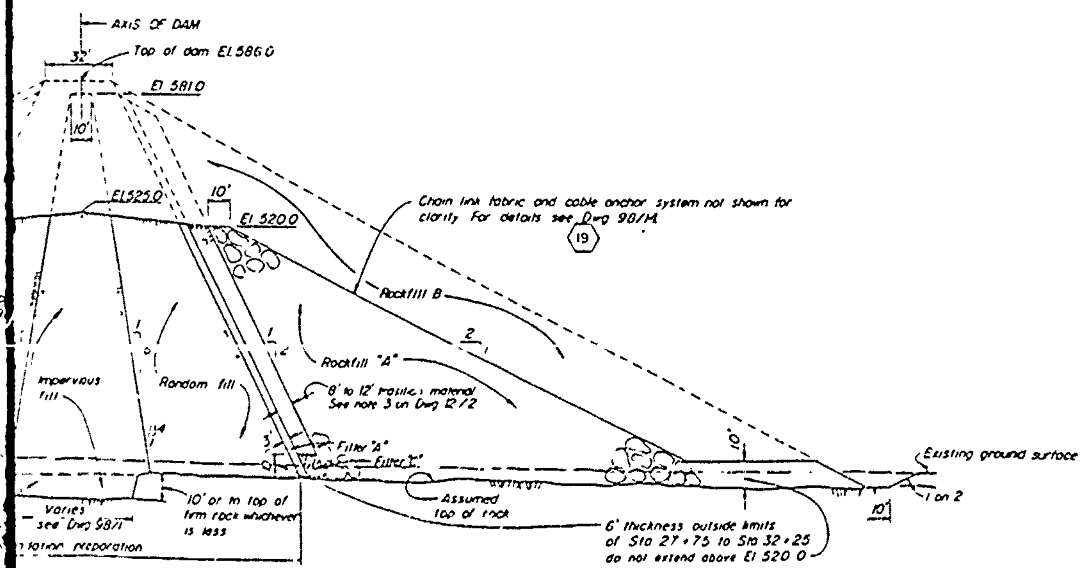
(2) TYPICAL SECTION COFFERDAM NO 1

NO SCALE

(2) TYPICAL SECTION TEMPORARY COFFERDAM NO.3

NO SCALE

NOTES



TION COFFERDAM NO 2

U S ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

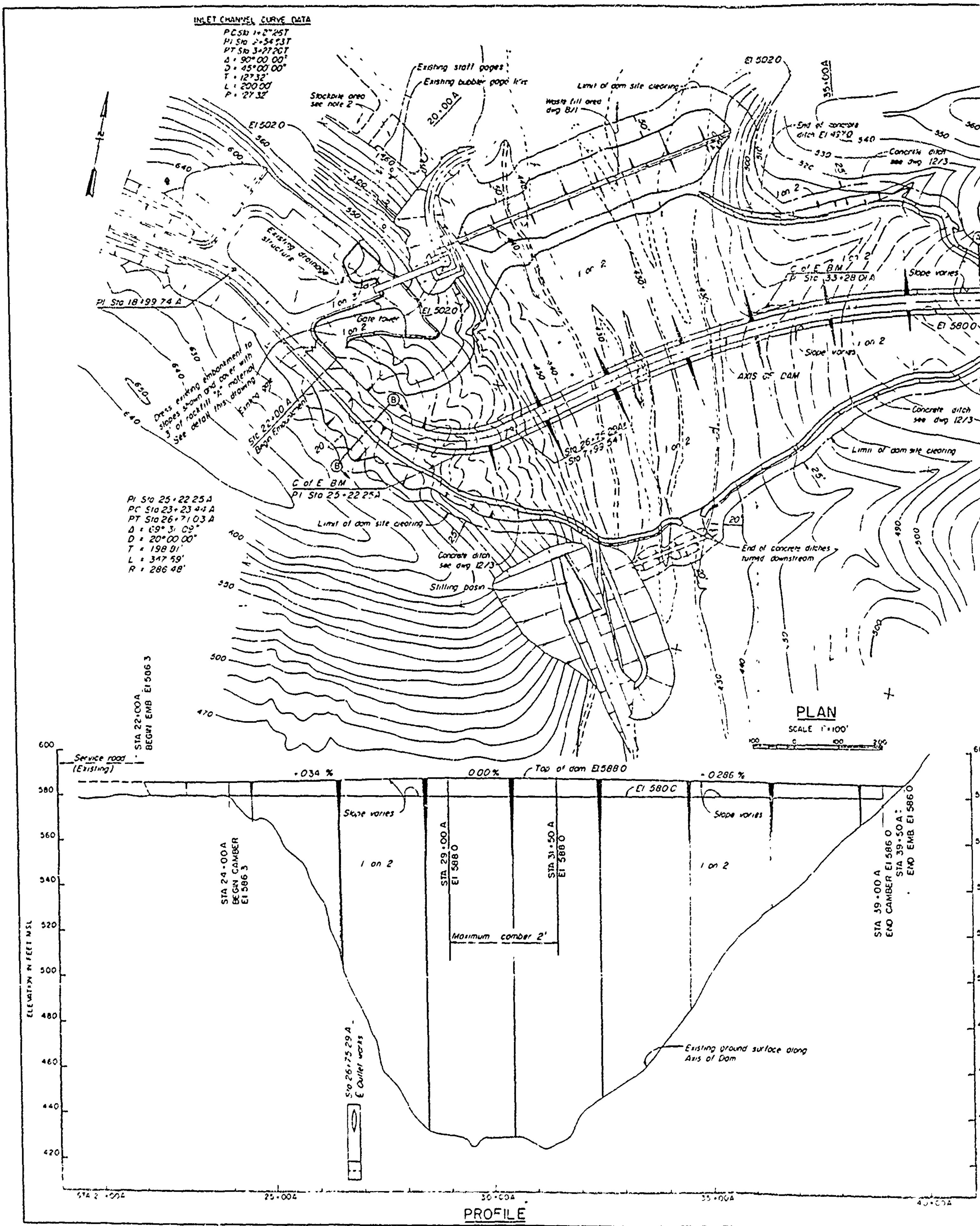
GILLHAM DAM
EMBANKMENT
DIVERSION
PLAN AND SECTIONS

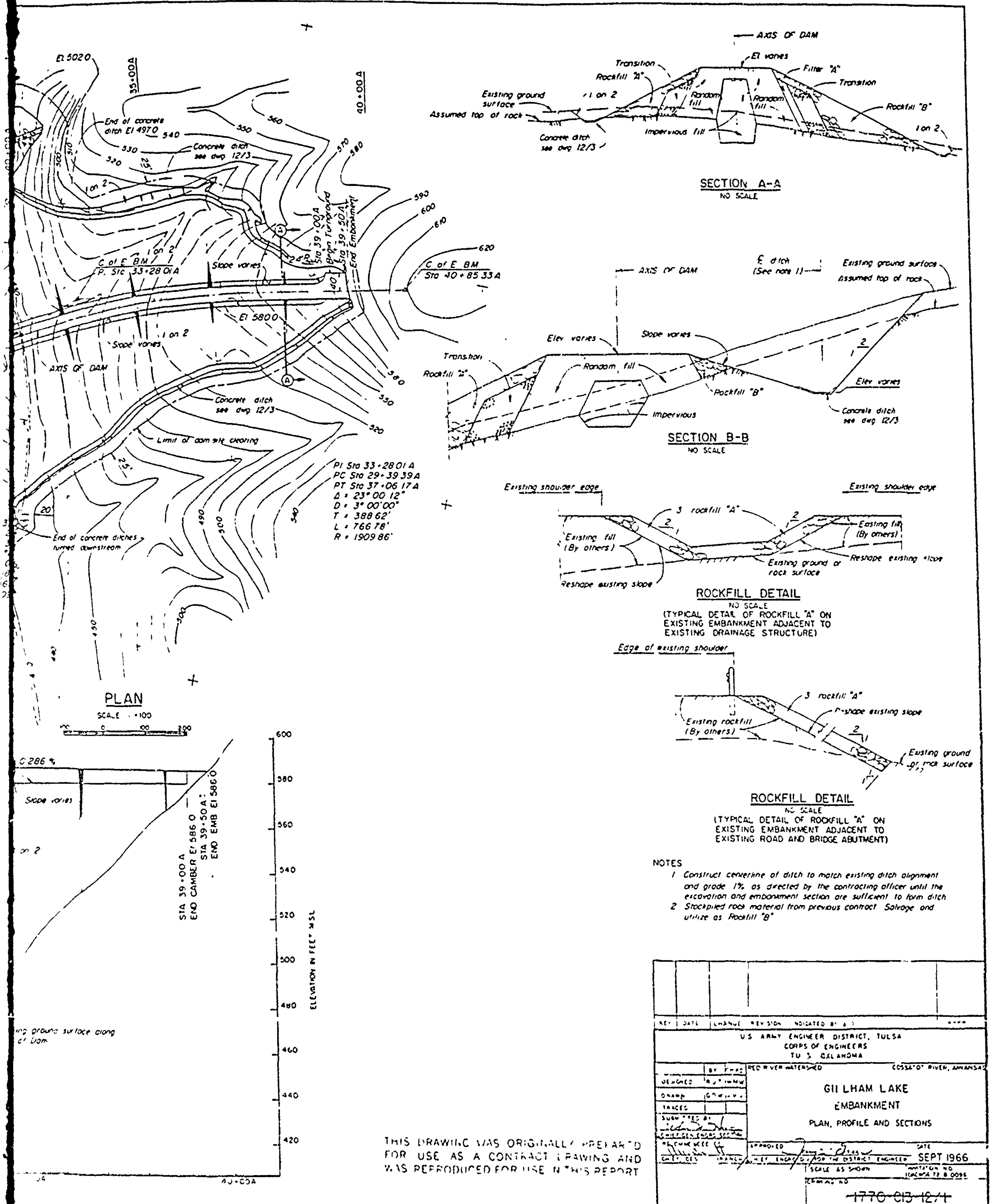
DESIGNED R J T / M W W
DRAWN R J T / J V N
TRACED R D M / J V N

DATE APR 1971

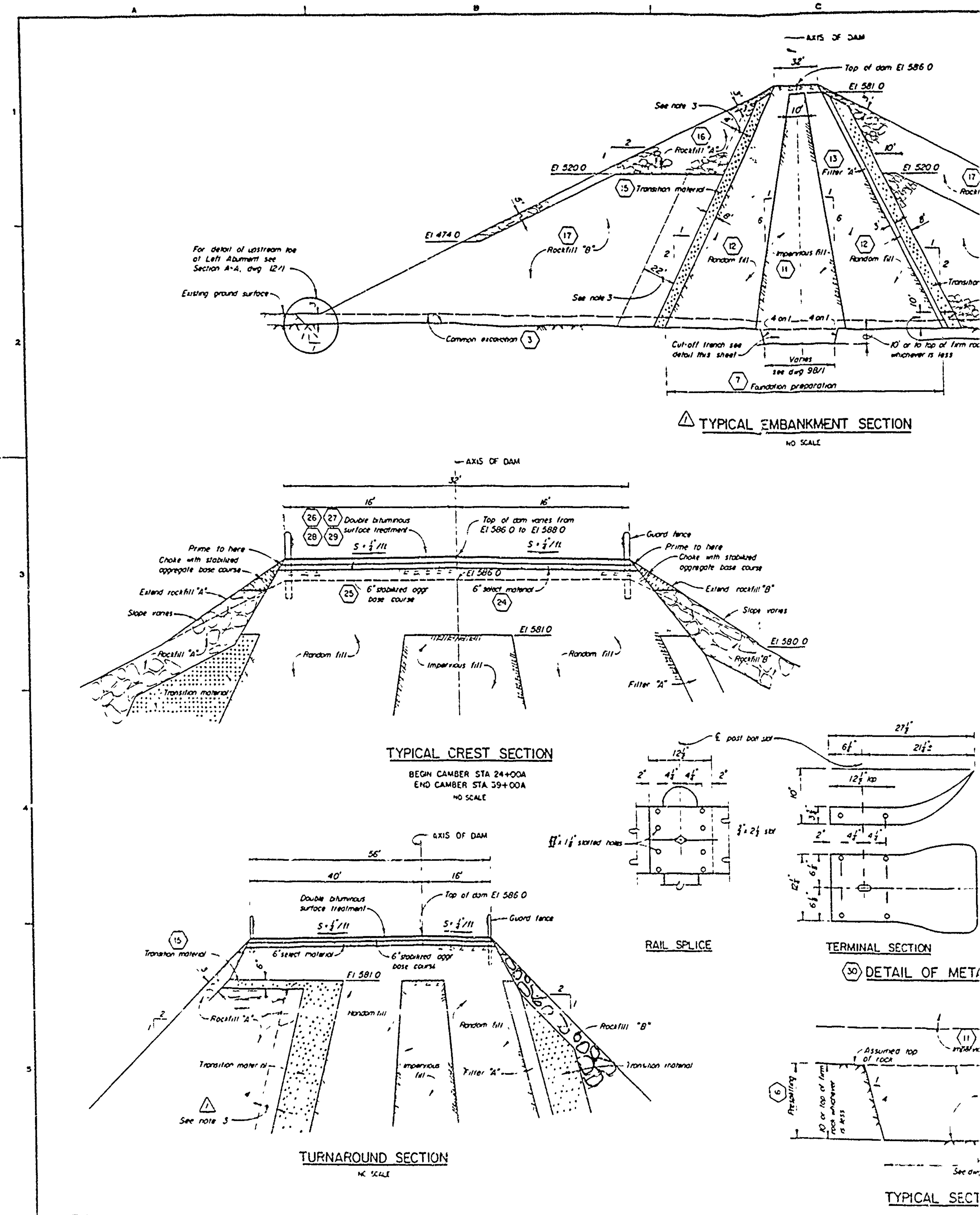
SCALE AS SHOWN

DRAWING NO 1770-DM9-8/2

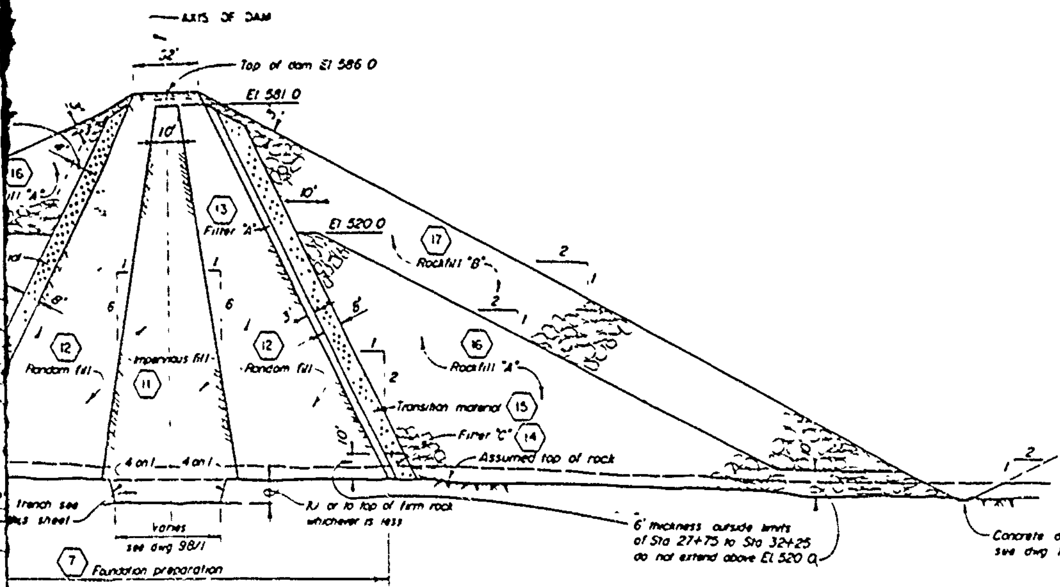

INLET CHANNEL CURVE DATA
Existing staff gages
Existing bubbler gage lines
Stockpile area see note 2
Limit of dam site clearing
Waste fill area dwg 8/1
Existing drainage structure
Gate tower
PI Sta 18+99.74 A
Dress existing embankment to slopes shown and cover with 3' of rockfill "A" material See detail this drawing
Sta 22+00 A Begin Embankment
C of E BM
PI Sta 25+22.25 A
PC Sta 23+23.44 A
Concrete ditch see dwg 12/3
Stilling basin
End of concrete ditches turned downstream
AXIS OF DAM
Slope varies
1 on 2
El 580.0
El 502.0
PLAN
SCALE 1"=100'
Service road (Existing)
Top of dam El 588.0
STA 22+00A BEGIN EMB El 586.3
STA 24+00A BEGIN CAMBER El 586.3
STA 29+00 A El 588.0
STA 31+50 A El 588.0
Maximum camber 2'
STA 39+00 A END CAMBER El 586.0
STA 39+50 A END EMB El 586.0
Existing ground surface along Axis of Dam
ELEVATION IN FEET MSL
PROFILE



EMBANKMENT CRITERIA AND PERFORMANCE REPORT - PLATE NO. 15

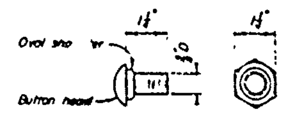


AXIS OF DAM
32'
Top of dam El 586 0
El 581 0
See note 3
Rockfill "A"
Filter "A"
El 520 0
Transition material
El 474 0
Rockfill "B"
Random fill
Impervious fill
For detail of upstream toe of Left Abutment see Section A-A, dwg 12/1
Existing ground surface
Common excavation
Cut-off trench see detail this sheet
Varies
see dwg 98/1
Foundation preparation
TYPICAL EMBANKMENT SECTION
NO SCALE
Double bituminous surface treatment
Top of dam varies from El 586 0 to El 588 0
Guard fence
Prime to here
Choke with stabilized aggregate base course
Extend rockfill "A"
Extend rockfill "B"
Slope varies
6" stabilized aggr base course
6" select material
El 586 0
El 581 0
El 580 0
TYPICAL CREST SECTION
BEGIN CAMBER STA 24+00A
END CAMBER STA 39+00A
NO SCALE
RAIL SPLICE
TERMINAL SECTION
DETAIL OF META
56'
40'
16'
Top of dam El 586 0
TURNAROUND SECTION
NO SCALE
Assumed top of rock
Presplitting
TYPICAL SECT



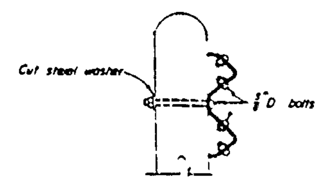
TYPICAL EMBANKMENT SECTION

NO SCALE

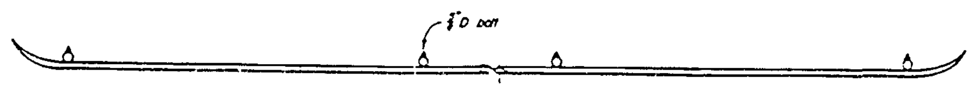


SPLICE BOLT
POST BOLT
(SIMILAR EXCEPT LENGTH)

NUT

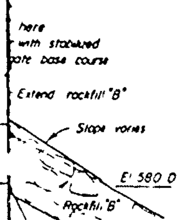


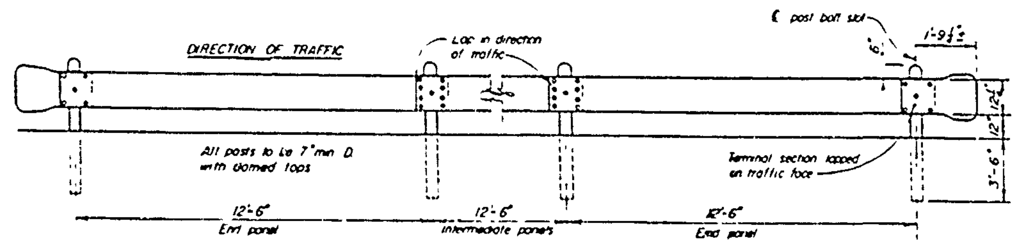
SECTION AT POST



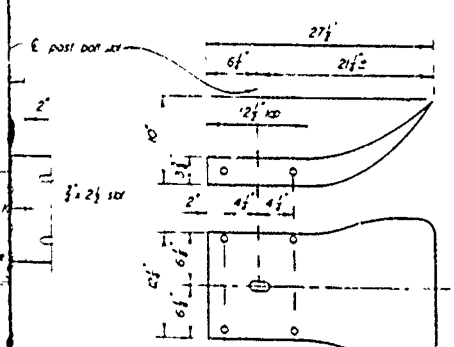
PLAN

Not less than No 12 U.S.S. gauge steel

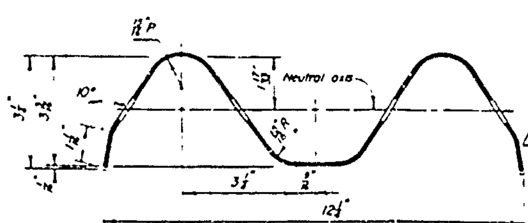




ELEVATION



TERMINAL SECTION



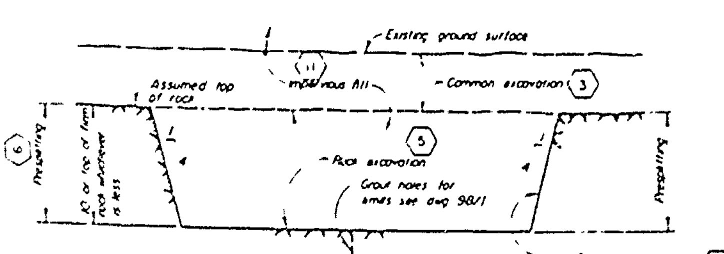
SECTION THRU RAIL ELEMENT

(30) DETAIL OF METAL PLATE GUARD FENCE

NO SCALE

NOTES
1. Limits of guard fence from Sta 22+00A to Sta 39+50A±, right and left
2. Final depth of cut-off trench will be determined by the Contracting Officer; after the foundation is exposed. For cuts less than 5 feet conventional excavation procedures will be followed with slopes to conform to adjacent presplit face
3. Excess by-products generated from processing Rockfill "A", may be placed in this zone

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT



TYPICAL SECTION CUT-OFF TRENCH

NO SCALE

KEY	DATE	CHANGE	REVISION INDICATED BY △	APPR
1	6 16 72	Am 0001	Revise transition B note 3	J W

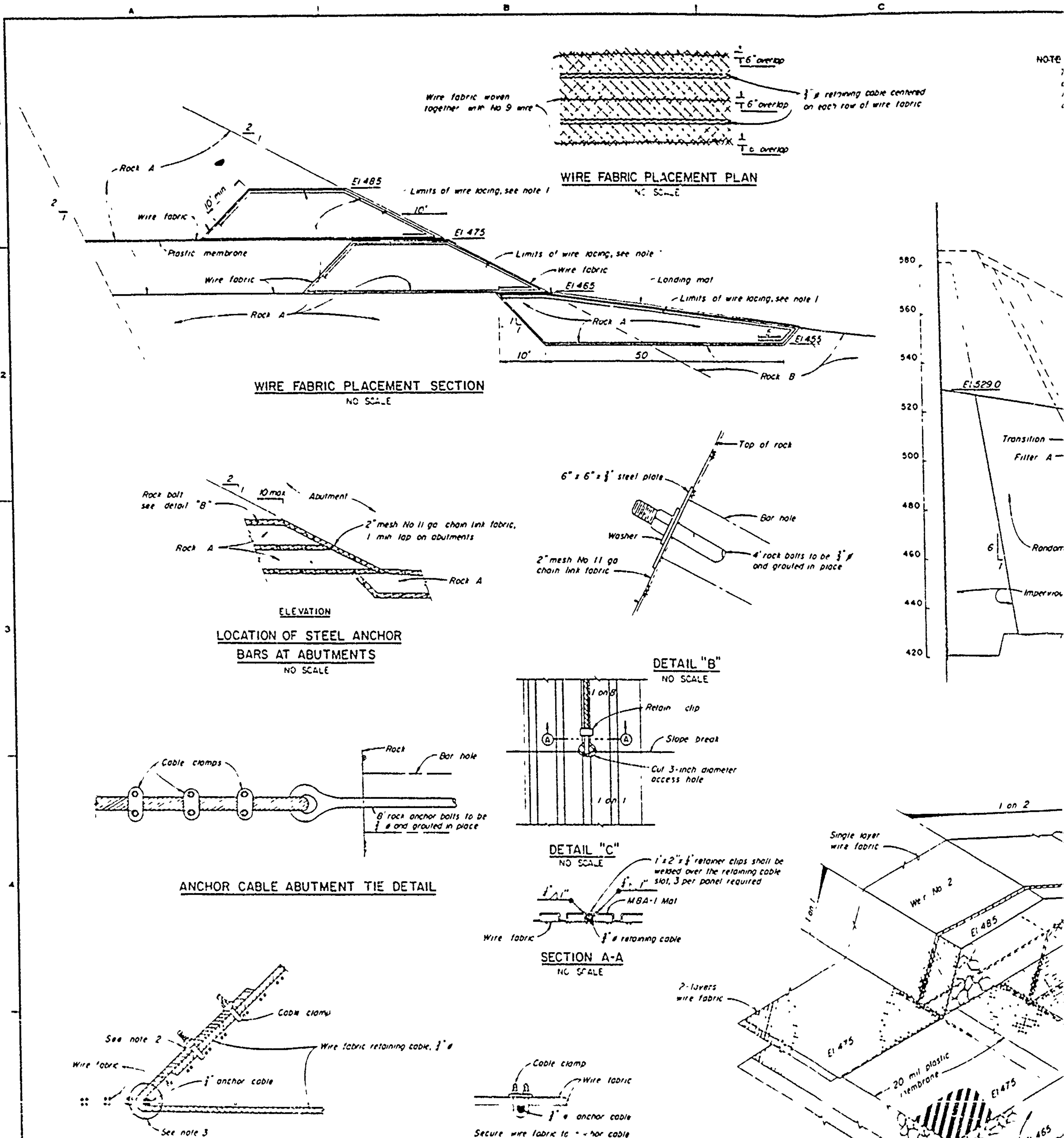
U.S. ARMY ENGINEER DISTRICT TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER ARKANSAS

GILLHAM LAKE
EMBANKMENT
SECTIONS AND DETAILS I

DESIGNED BY / DRAWN BY / CHECKED BY / SUBMITTED
Chief Engineering Sec
DATE NOV 1970

INVITATION NO DACW56 72-B-0098
SCALE AS SHOWN
DRAWING NUMBER ~~1770-C13-12/2-1~~



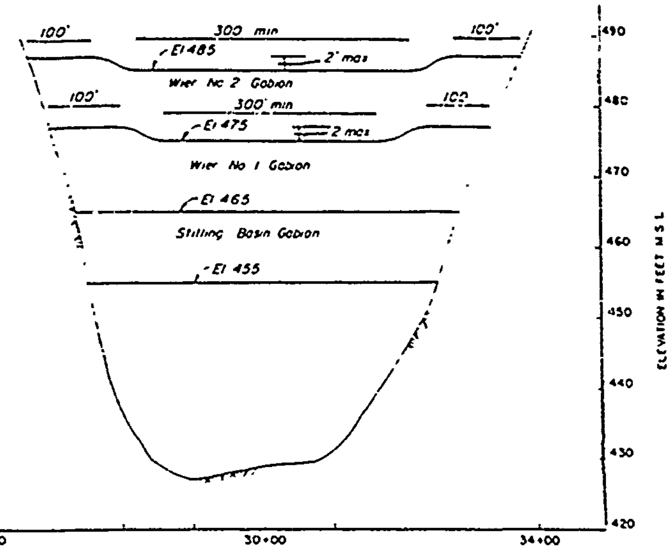
MBA-1 LANDING MAT PLACEMENT NOTES

1. The cover plates and sliding bolts in the slots along which the retaining cables are to be placed, shall be cut out and removed
2. The landing mat shall be placed with its longest demension perpendicular to the dam axis
3. Placement of the landing mat shall begin at the upstream toe of the stilling basin at the right abutment and be placed continuous to the left abutment
4. The end laps shall be staggered as shown in the "Typical Oblique Of Wire Fabric Installation" with the under lap ends downstream End and side laps shal made as shown on information drawing
5. The bend of the downstream toe of the stilling basin shall be made a er placement by driving a he vy [illegible] along the slope b eak or by [illegible] approved means

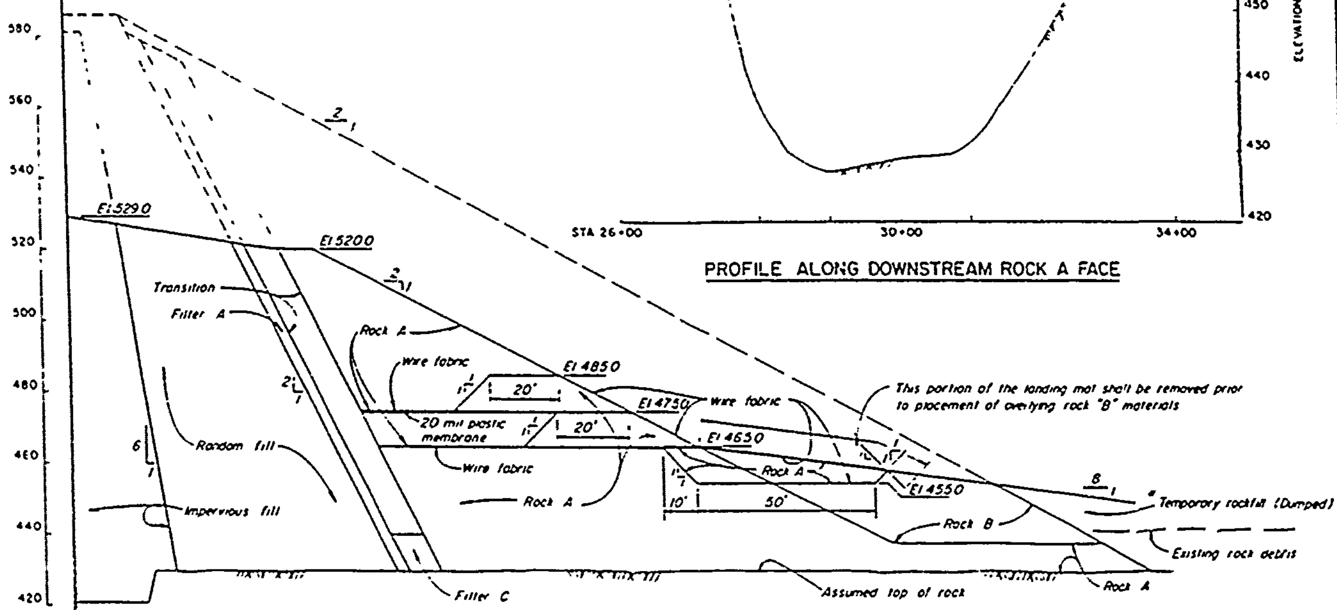
NOTES

1. The wire fabric shall be woven together with No 9 wire along the longitudinal overlaps shown in the "Wire fabric placement section"
2. The upstream end of the 3/4" ø wire retaining cable shall be temporarily secured to the 3/4" ø anchor cable until the rockfill and wire fabric installation is completed at which time detail "A" can be assembled
3. The 3/4" ø anchor cable and both layers of wire fabric shall be laced together with No 9 wire
4. Plastic membrane shall be overlapped 12" at all joints
5. Prior to final tying of cables and mesh the cables extending across the [illegible] on the slope shall be stretched to remove slack

NOTE-
The weir crest extending 100' from abutment contacts, shall be constructed not more than two feet higher than the crest elevation in the center. see "Profile Along Downstream Rock Face"

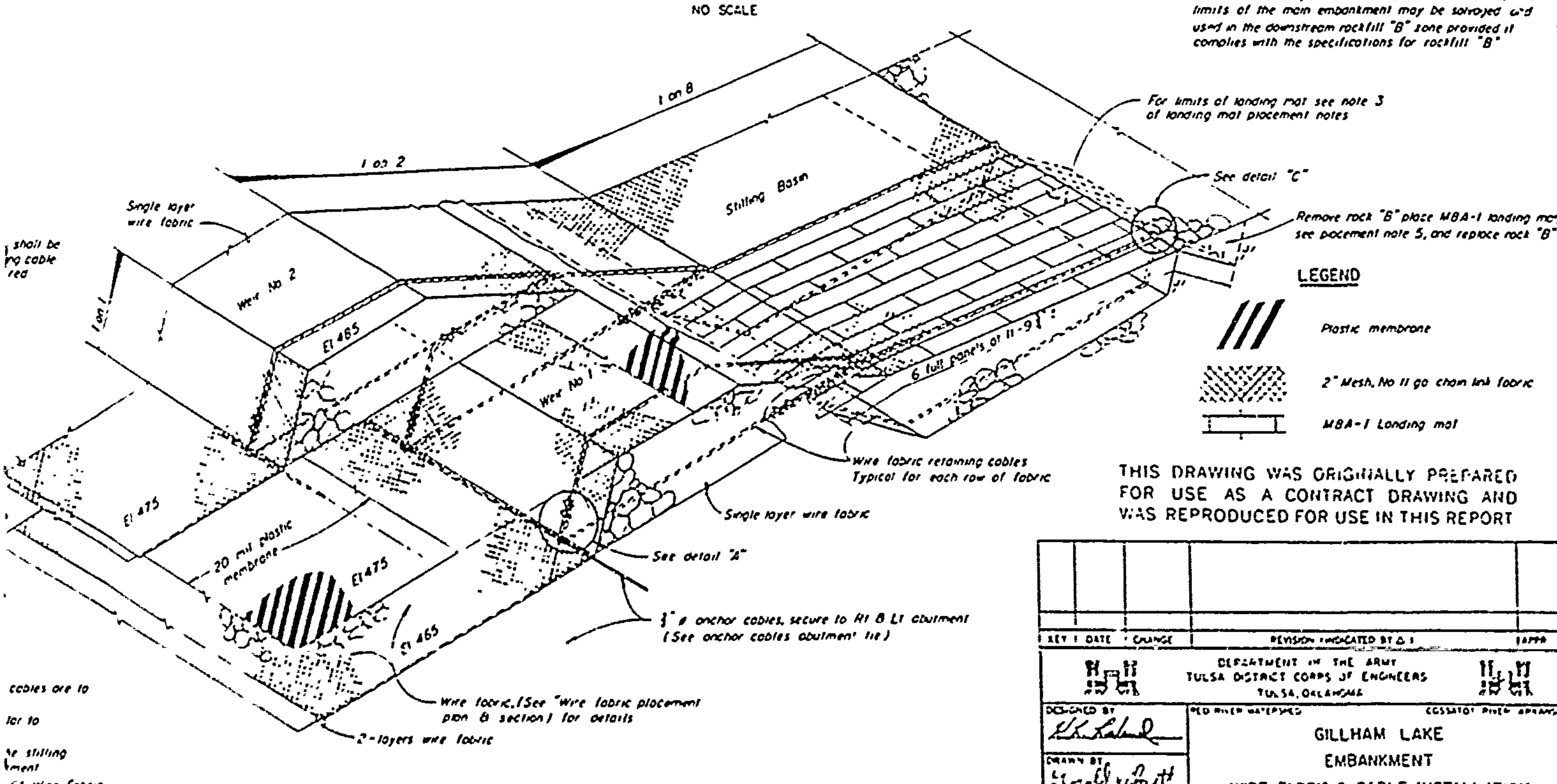


PROFILE ALONG DOWNSTREAM ROCK A FACE



DOWNSTREAM SECTION
NO SCALE

* After cofferdam No 2 has been completed to El 529.0 the 8 1 temporary rockfill (dumped) berm outside the limits of the main embankment may be salvaged and used in the downstream rockfill "B" zone provided it complies with the specifications for rockfill "B"



LEGEND

- Plastic membrane
- 2" Mesh, No 11 ga chain link fabric
- MBA-1 Landing mat

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

TYPICAL OBLIQUE OF WIRE FABRIC INSTALLATION
NO SCALE

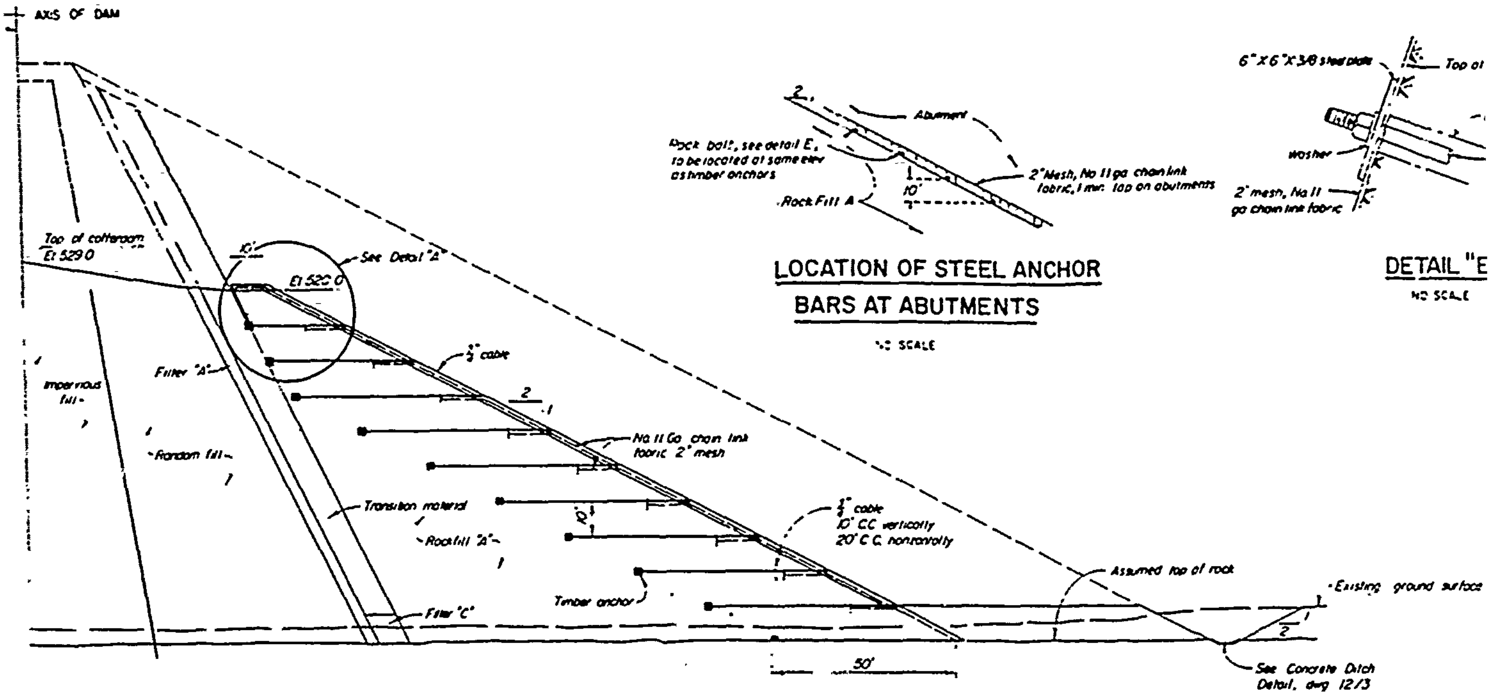
DEPARTMENT OF THE ARMY
TULSA DISTRICT CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERWAYS — COSSATOT RIVER ARKANSAS

GILLHAM LAKE
EMBANKMENT
WIRE FABRIC & CABLE INSTALLATION
DETAILS

DESIGNED BY / DRAWN BY / CHECKED BY / SUBMITTED
INVITATION NO. DACW56- -8
SCALE AS SHOWN
DATE NOV 73

A B C



TYPICAL SECTION

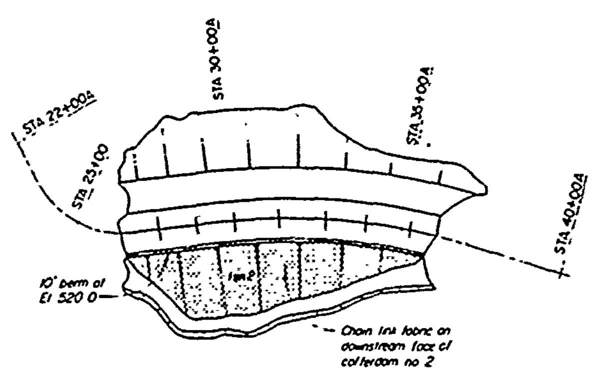
NO SCALE

LOCATION OF STEEL ANCHOR BARS AT ABUTMENTS

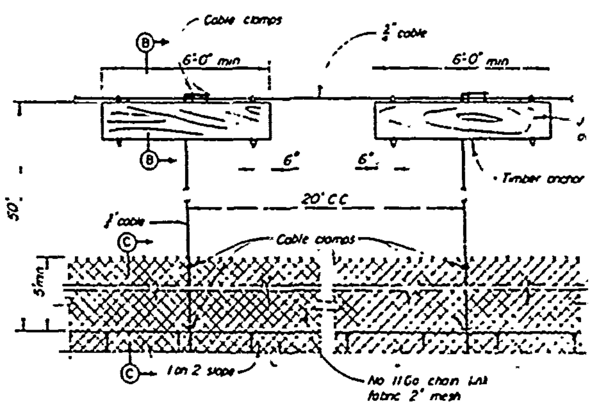
NO SCALE

DETAIL "E

NO SCALE

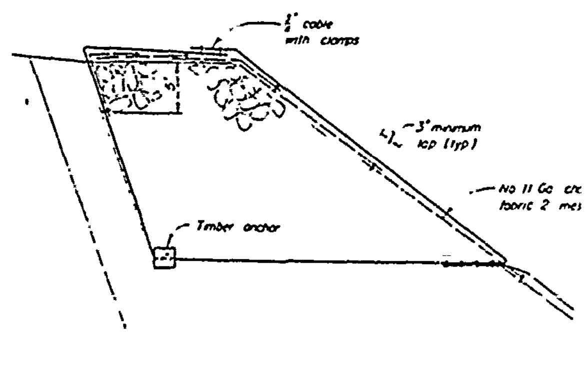


PLAN



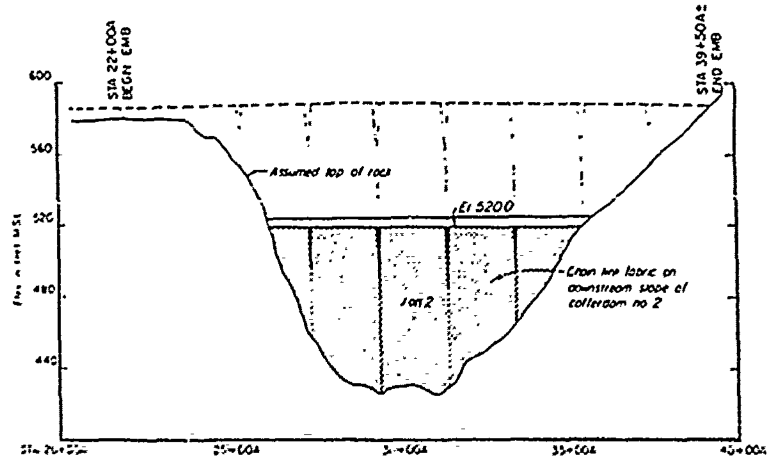
TIMBER ANCHOR PLAN

NO SCALE



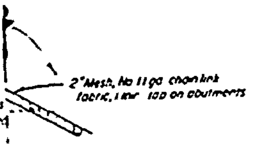
DETAIL "A"

NO SCALE



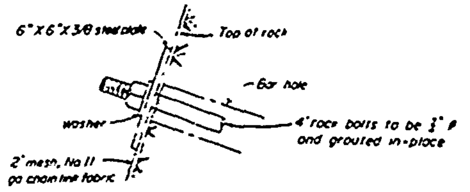
PROFILE

1



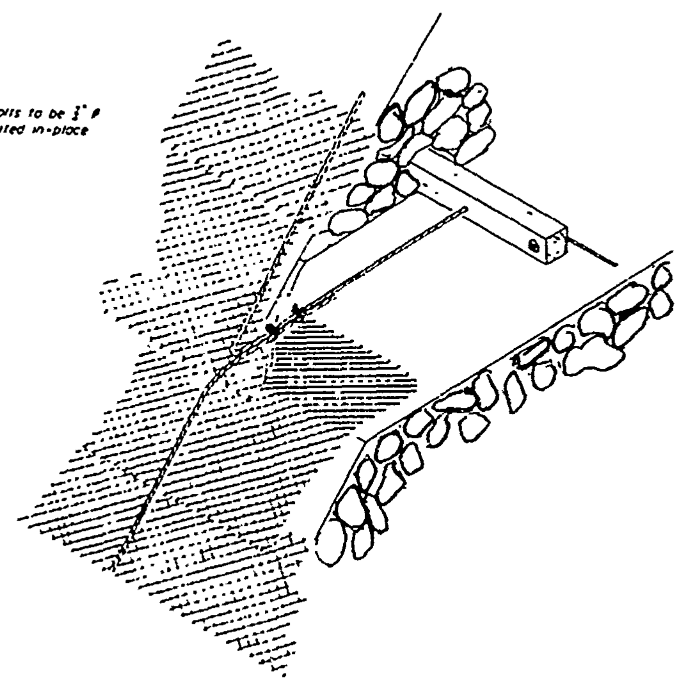
EEL ANCHOR

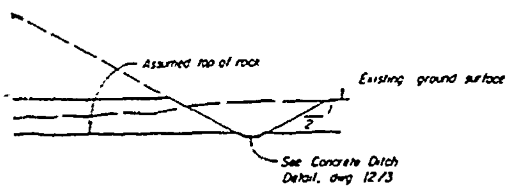
IENTS



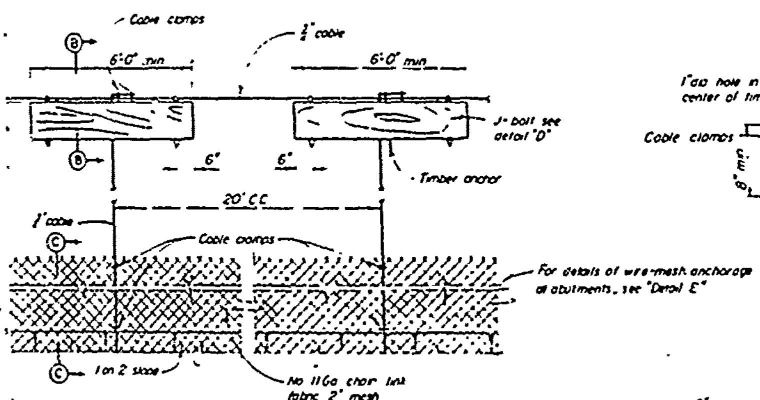
DETAIL "E"

NO SCALE



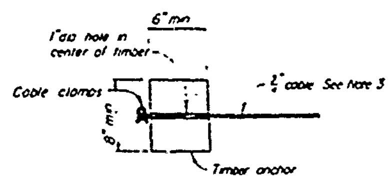


CHAIN LINK FABRIC ANCHOR SYSTEM



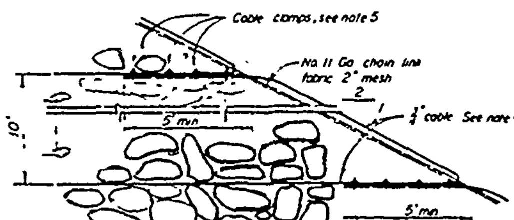
TIMBER ANCHOR PLAN

NO SCALE



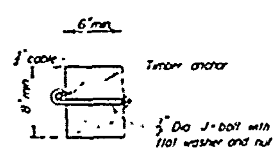
SECTION B-B

NO SCALE



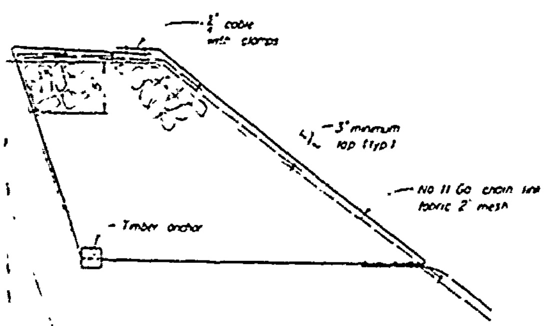
SECTION C-C

NO SCALE



DETAIL "D"

NO SCALE



DETAIL "A"

NO SCALE

NOTES

1. All joints in chain link fabric to be overlapped a minimum of 3" and tied on 12" centers
2. All work shown on this sheet will be paid for under bid item no (19)
3. Where ¾" cable is indicated on this drawing, cable or wire rope 25,000 lb bursting strength (min), may be used
4. Prior to final tying of cables and mesh, the cables extending across the fill and up the slope shall be stretched to remove slack
5. Number of cable clamps used at each tie shall be that number necessary to develop the full strength of the cable

KEY	DATE	CHANGE	REVISION INDICATED BY △	APPR

U. S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

DESIGNED BY | DRAWN BY | CHECKED BY

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

GILLHAM LAKE
EMBANKMENT
CABLE ANCHORAGES
DETAILS

SUBMITTED — CHIEF SOILS DESIGN SEC.

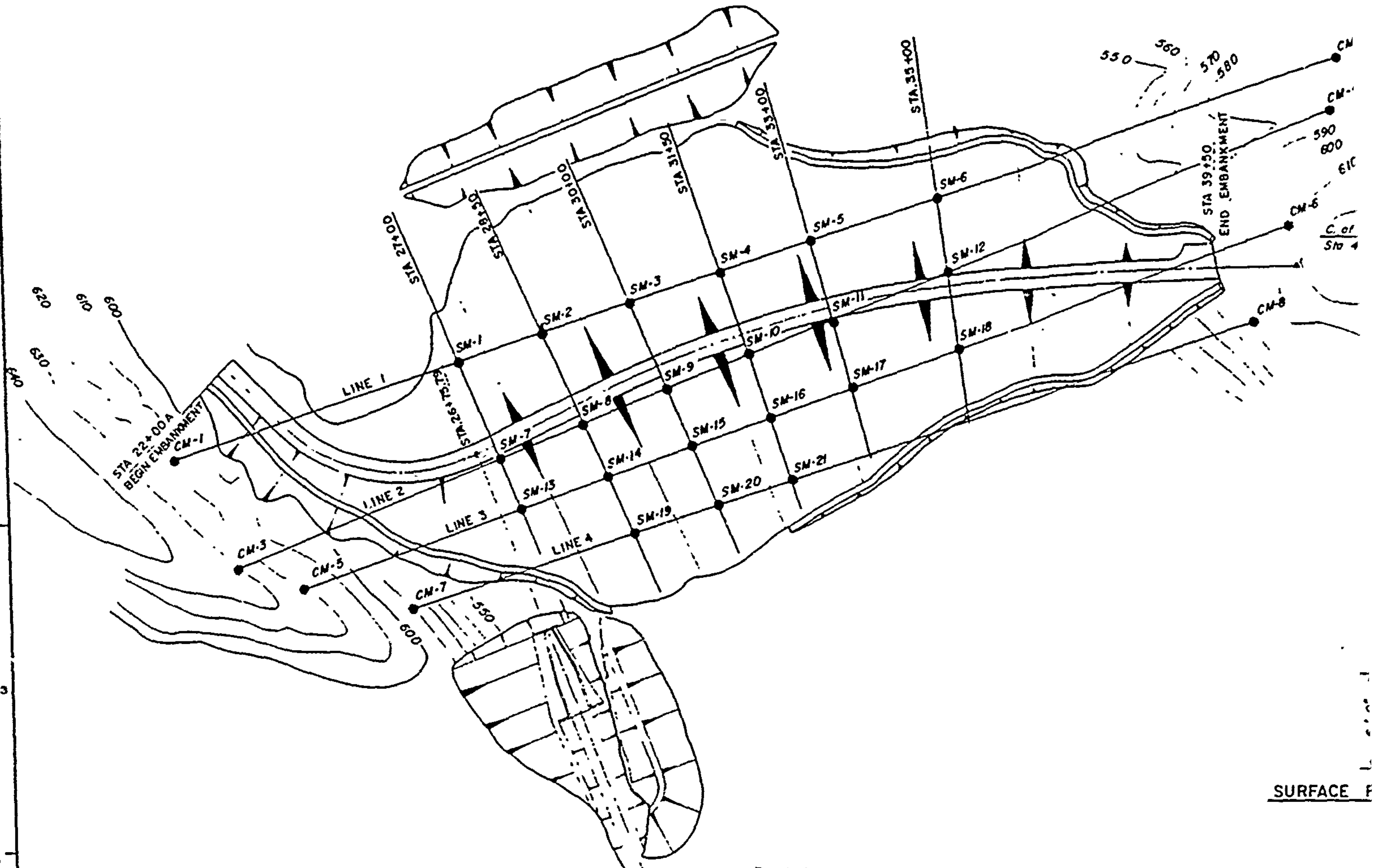
DATE NOV 1970

INVITATION NO DACW56-72-B-0096

SCALE AS SHOWN

DRAWING NUMBER ~~1779-03-872~~

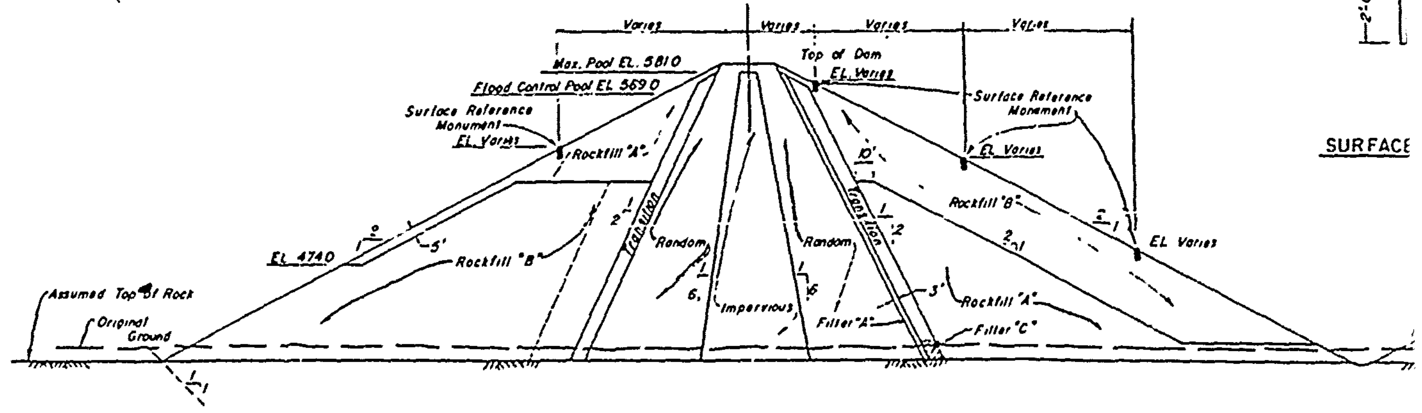
SUPERSEDED



PLAN

SCALE 1"=100'

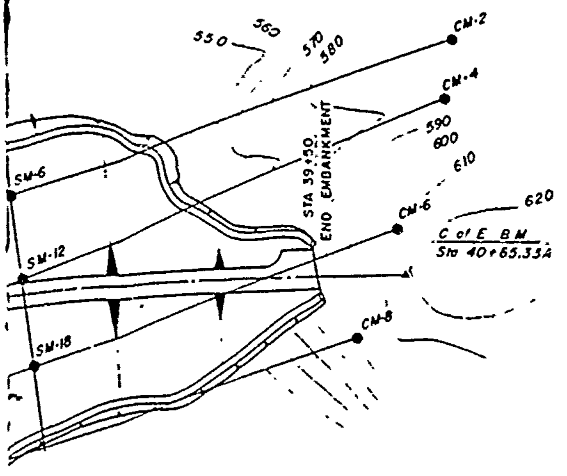
SURFACE F



TYPICAL SECTION

NO SCALE

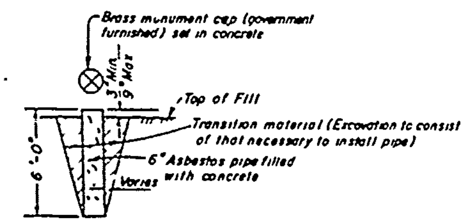
SURFACE



HUB NO	STATION	OFFSET	HUB NO.	STATION	OFFSET
LINE 1			**LINE 3**		
CM-1	STA.22+45	170'DS	CM-5	STA.24+50	216' DS
SM-1	STA 27+00	142'US	SM-13	STA.27+00	110' DS
SM-2	STA 28+50	124'US	SM-14	STA 28+50	126' DS
SM-3	STA 30+00	108'US	SM-15	STA.30+00	136' DS
SM-4	STA 31+50	101'US	SM-16	STA.31+50	141' DS
SM-5	STA 33+00	109'US	SM-17	STA.33+00	131' DS
SM-6	STA.35+00	138'US	SM-18	STA.35+00	101' DS
CM-2	STA.41+55	335'US	CM-6	STA.40+70	72' US
LINE 2			**LINE 4**		
CM-3	STA.23+85	235'DS	CM-7	STA.25+50	216' DS
SM-7	STA 27+00	24' DS	SM-19	STA.28+50	224'DS
SM-8	STA 28+50	32' DS	SM-20	STA 30+00	238'DS
SM-9	STA.30+00	38' DS	SM-21	STA.31+50	246'DS
SM-10	STA.31+50	36' DS	CM-8	STA.40+10	80' DS
SM-11	STA.33+00	20' DS			
SM-12	STA 35+00	20' US			
CM-4	STA 41+40	255'US			

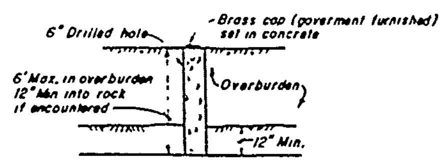
NOTE:

Locations of control monuments and offsets upstream and downstream of the dam axis for surface monuments are approximate. Exact locations to be determined in the field.



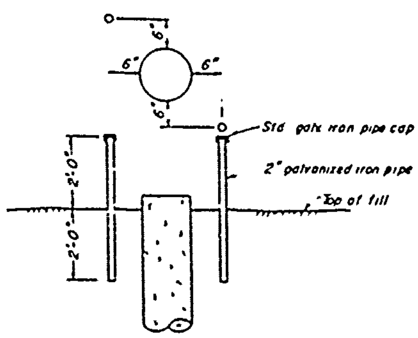
SURFACE REFERENCE MONUMENT

NO SCALE



CONTROL REFERENCE MONUMENT

NO SCALE



SURFACE MONUMENT GUARD

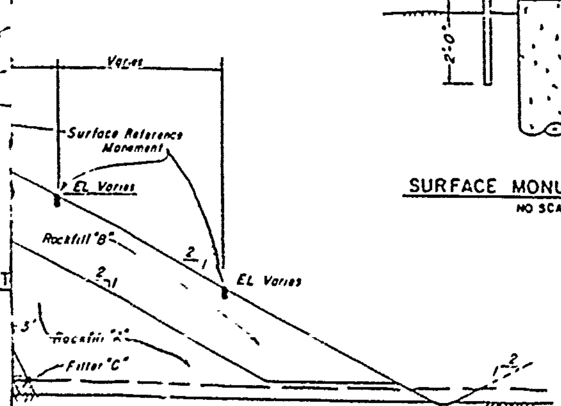
NO SCALE

LEGEND

- SM – Surface reference monument
- CM – Control reference monument

NOTE

1. All work shown on this drawing will be paid for under bid item (20)



THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

KEY	DATE	CHANGE	REVISION INDICATED BY △	APPR

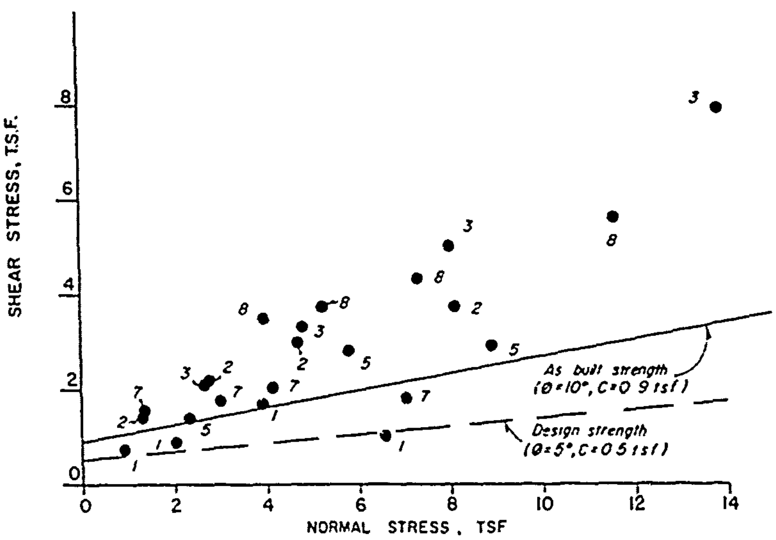
U S ARMY ENGINEER DISTRICT TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, OKLAHOMA

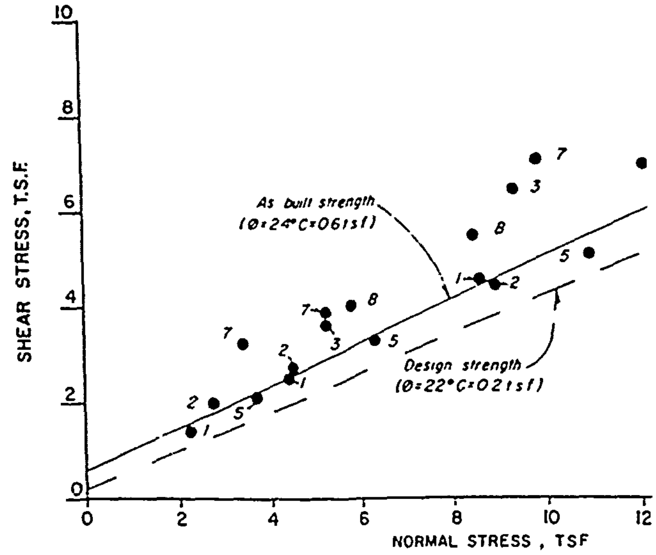
DESIGNED BY · DRAWN BY · CHECKED BY · SUBMITTED — CHIEF, SOILS DESIGN SEC

GILLHAM LAKE
EMBANKMENT
ENGINEERING MEASUREMENT DEVICES
PLAN, SECTION AND DETAILS

INVITATION NO DACW56-72-B-0056
SCALE AS SHOWN
DRAWING NUMBER ~~1770-613-13/1~~
DATE NOV 1970

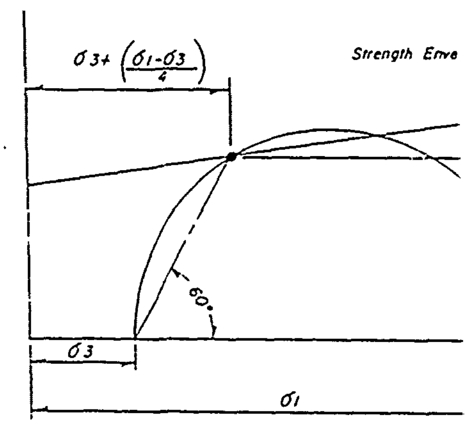


Q STRENGTHS



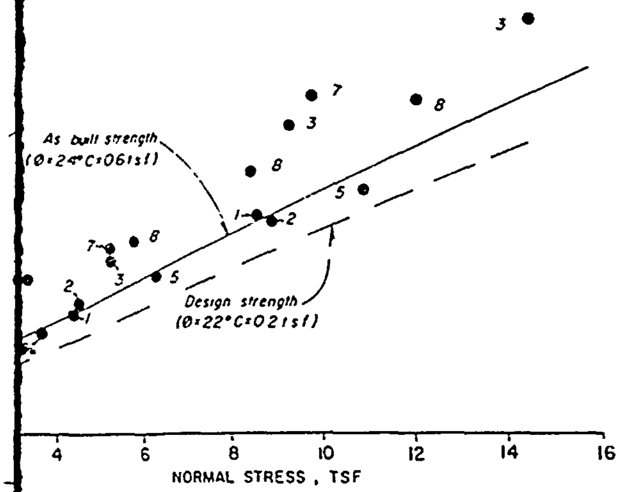
R STRENGTHS

EMBANKMENT EARTH FILL SAMPLES										
SAMPLE NO	RECORD SAMPLE NUMBER	TYPE SPECIMEN	STA.	OFFSET ft.	ELEV. ft M.S.L.	CLASS	LL %	%Fi.	WC %	γ_d p.c.f.
1	1-3	REMOLDED	31+00	℄	440	CL	23	60	16	113 4
2	R-5	REMOLDED	31+00	60' DS	438	CL-ML	22	49	13 8	116 8
3	R-23	UNDIST	30+00	50' US	460	CL-ML	18	57	11 8	115 1
4	#4	UNDIST	31+00	℄	480	SC	27	48	14 7	116
5	#5	UNDIST	32+00	50'DS	480	SC	22	43	13 4	118
6	#6	REMOLDED	28+70	45'US	504	SC	21	41	15 1	121
7	1-44A	REMOLDED	30+00	℄	532	CL	22	65	14 2	119
8	#8	UNDIST	35+00	15'DS	550	CL	29	79	17 1	115 7

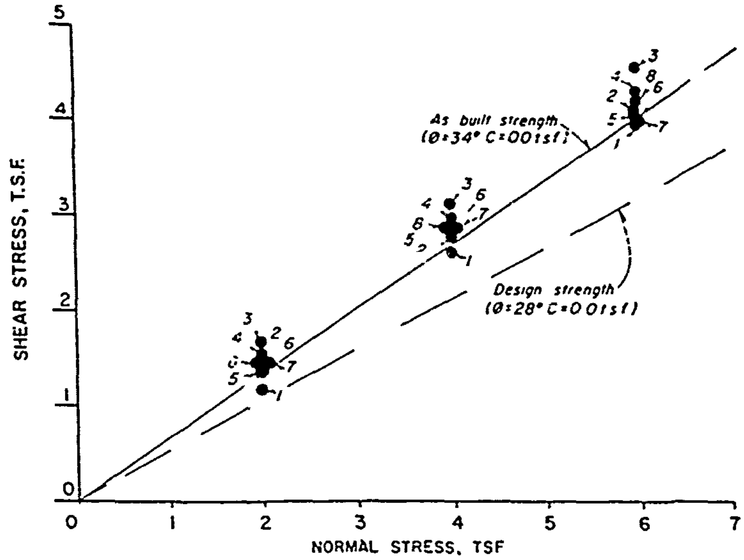


MOHR'S CIRCLE DIAGRAM
REMOLDED SAMPLES

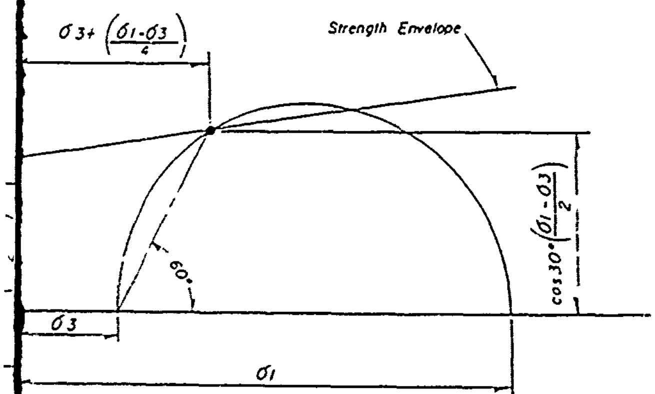
C D E



R STRENGTHS

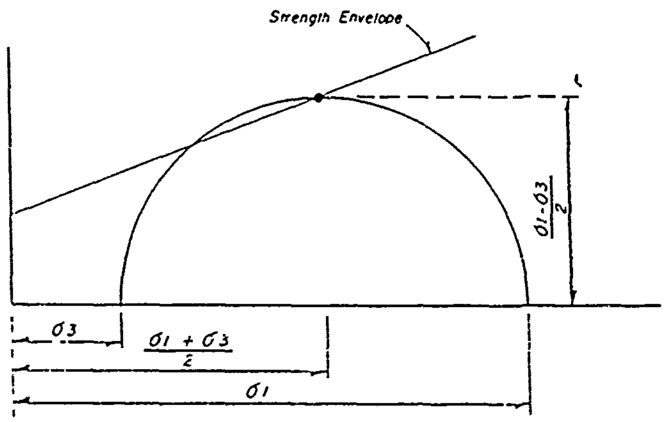


S STRENGTHS



MOHR'S CIRCLE DIAGRAM

REMOLDED SAMPLES



MOHR'S CIRCLE DIAGRAM

UNDISTURBED SAMPLES

LEGEND

σ_1 = Major principal stress at failure

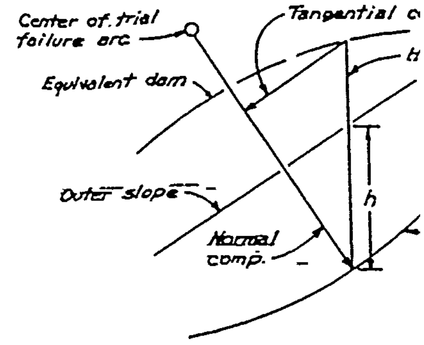
σ_3 = Minor principal stress at failure

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

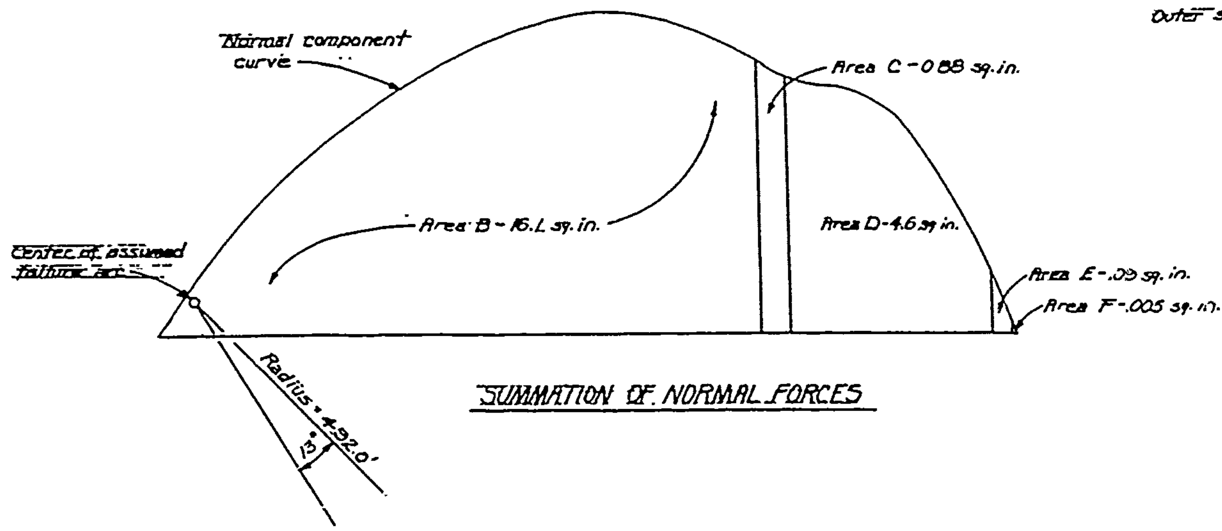
RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

GILLHAM LAKE
EMBANKMENT RECORD SAMPLES
'AS BUILT' SHEAR STRENGTHS
EARTH FILL

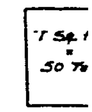
DESIGNED BY: [signature]
DRAWN BY: Sewell
CHECKED BY: [signature]
SUBMITTED: [signature]
CHIEF, SO. MECH. SEC.
DATE JAN 1975
SCALE AS SHOWN
DRAWING NO.



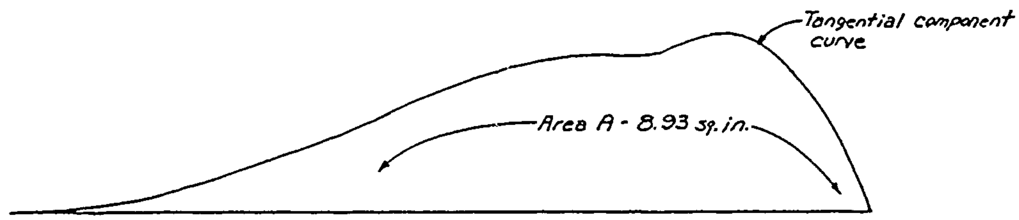
TYPICAL FORCE DI



SUMMATION OF NORMAL FORCES



.AREA S



SUMMATION OF TANGENTIAL FORCES

SAFETY	
Radius (feet)	Coo (Cen X (f
210.8	-191.
326.2	-233
483.7	-321.
207.5	-210
442.5	-293
165.2	-155
492.0	-318
299.3	-24
394.1	-30
186.9	-170

NOTE: The ab
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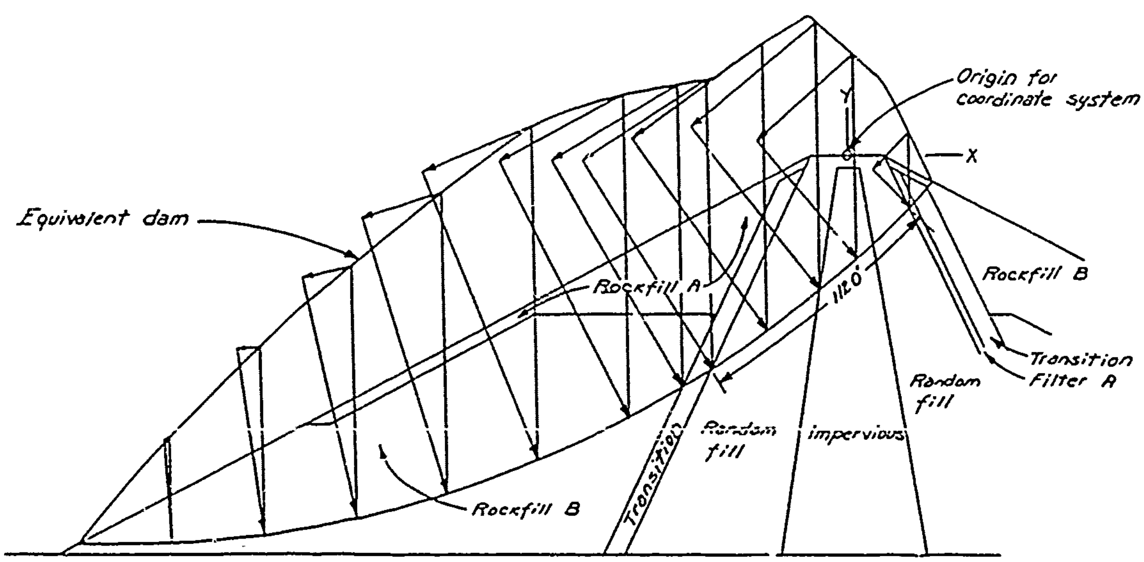
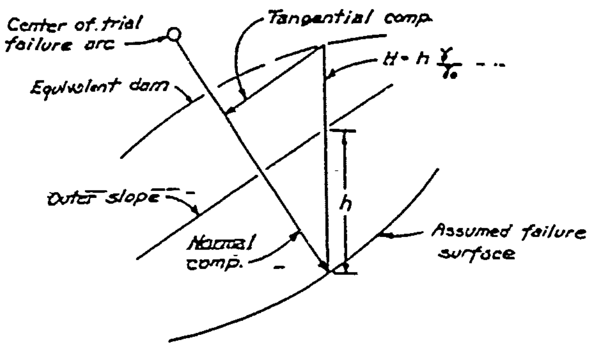


STABILITY ANALYSIS OF UPSTREAM SLOPES
CONDITION OF END OF CONSTRUCTION

SCALE: 1" = 40'

40 0 40 80



TYPICAL FORCE DIAGRAM

1 Sq. in. = 50 Tons

AREA SCALE

SAFETY FACTOR SUMMARY			
Radius (feet)	Coordinates (Center of arc) X (ft.)	Y (ft.)	Safety factor
210.8	-191.0	47.0	1.759
326.2	-233.5	161.0	1.582
483.7	-321.0	324.0	1.564
207.5	-210.0	42.0	1.574
442.5	-293.0	281.0	1.560
165.2	-159.5	20.0	1.854
492.0	-318.0	329.0	1.555
293.3	-244.5	146.5	1.663
394.1	-305.0	228.5	1.627
186.9	-170.0	22.0	1.791

NOTE: The above table includes results of all arcs analyzed for end of construction condition. Coordinates for arc centers are referenced from ℄ and crest of the dam.

ADOPTED DESIGN DATA						
MATERIALS	Soil wts. Lbs/Ft³ Saturated	Submerged	Shear strengths *	Q	R	S
ROCKFILL A	105 ** 130	67.6	φ	42	42	42
			C	0	0	0
ROCKFILL B	135	72.6	φ	36	36	36
			C	0	0	0
TRANSITION	125	62.5	φ	33	33	33
			C	0	0	0
FILTER A	125	62.5	φ	33	33	33
			C	0	0	0
RANDOM FILL	125	62.5	φ	5	22	28
			C	0.5	0.2	0
IMPERVIOUS	125	62.5	φ	5	22	28
			C	0.5	0.2	0

* φ = Angle of internal friction (Degrees)
** = Saturated surface dry weight (40% voids)
C = Cohesion (Tons/ft²)

Forces acting on failure arc

Positive tangential force - Area A = 446.5 Tons
ΣTotal = 446.5 Tons

Normal and resisting forces

Normal force Area B = 805 Tons
Normal force Area C = 44.0 Tons
Normal force Area D = 230.0 Tons
Normal force Area E = 4.5 Tons
Normal force Area F = 0.25 Tons

Cohesion (LC) = 112.0 × 0.5 = 56.0 Tons

Safety factor computations

Formula: $S.F. = \frac{\Sigma N \times \tan\phi + LC}{\Sigma T}$

For unconsolidated-undrained (Q) condition

$S.F. = \frac{805.25 \tan 36° + 48.5 \tan 33° + 230 \tan 5° + 56.0}{446.5}$

$S.F. = 1.55$

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

	BY	CHKD
DESIGNED	DEW	R.L.
DRAWN	DEW	DEW
TRACED	DEW	DEW

GILLHAM DAM AND RESERVOIR
EMBANKMENT
STABILITY ANALYSIS - END OF CONSTRUCTION UPSTREAM

SUBMITTED: Chief, Soil Mech. Section
RECOMMENDED: Chief, F&M Branch
APPROVED: Chief, Engrg Div for the Dist. Engineer
DATE: FEB. 1971
SCALE: AS SHOWN
DRAWING NO. 1770-DM9-98/7

(2)

ADOPTED DESIGN DATA						
MATERIALS	SOIL WTS. LBS/FT³		SHEAR STRENGTH			
	SATURATED	SUBMERGED	*	Q	R	S
ROCKFILL A	105** 130	67.6	ϕ	42	42	42
			C	0	0	0
ROCKFILL B	135	72.6	ϕ	36	36	36
			C	0	0	0
TRANSITION	125	62.5	ϕ	33	33	33
			C	0	0	0
FILTER A	125	62.5	ϕ	33	33	33
			C	0	0	0
RANDOM FILL	125	62.5	ϕ	5	22	28
			C	0.5	0.2	0
IMPERVIOUS	125	62.5	ϕ	5	22	28
			C	0.5	0.2	0

*ϕ = Angle of internal friction (Degrees)
** = Saturated surface dry weight (40% Voids)
C = Cohesion (Tons/ft²)

...S ACTING ON FAILURE ARC.

Positive tangential force - Area A = 522 Tons
Negative tangential force - Area B = -16 Tons
Σ Total = 506 Tons

NORMAL AND RESISTING FORCES

Normal force - Area C	= 1 Ton.
Normal force - Area D	= 3.5 Tons
Normal force - Area E	= 407 Tons
Normal force - Area F	= 80 Tons
Normal force - Area G	= 812.5 Tons
Normal force - Area H	= 2.5 Tons
Cohesion (LC) = 148 x 0.5	= 74 Tons

SAFETY FACTOR COMPUTATIONS

Formula: $S.F. = \frac{\Sigma N \tan\phi + \Sigma C}{\Sigma T}$

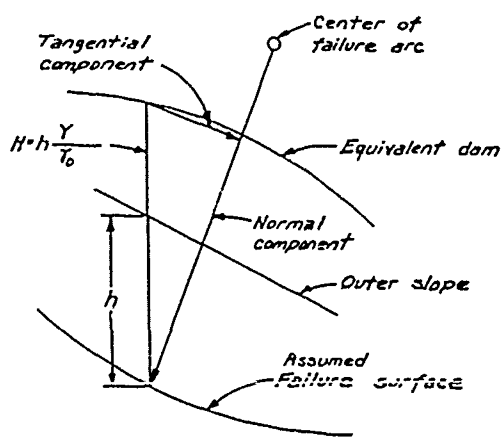
For unconsolidated undrained (Q) condition

$S.F. = \frac{812.5 \tan 42° + 83.5 \tan 33° + 407 \tan 5° + 2.5 \tan 36° + 74}{506}$

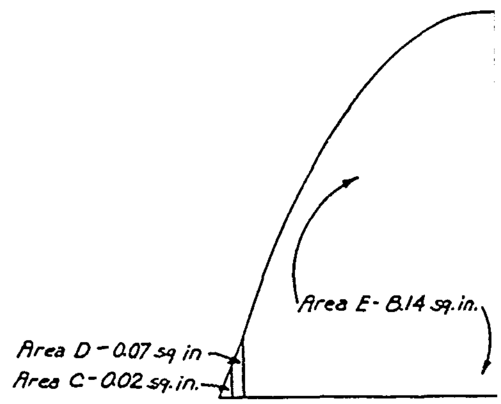
S.F. = 1.775

SAFETY FACTOR SUMMARY			
Radius (Feet)	Coordinates (Center of Arc)		Safety Factor
	X (Ft.)	Y (Ft.)	
210.8	191.0	47.0	1.915
326.2	233.5	161.0	1.774
463.7	321.0	324.0	1.809
207.5	210.0	42.0	2.096
442.5	293.0	281.0	1.794
165.2	159.5	20.0	2.004
492.0	318.0	329.0	1.798
299.3	244.5	146.5	1.874
394.1	305.0	228.5	1.884
186.9	172.0	22.0	1.913

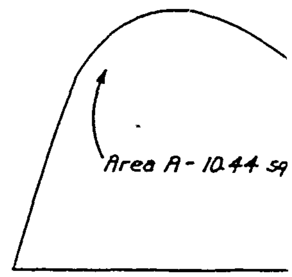
NOTE:
The above table includes results of all arcs analyzed for the end of construction condition. Coordinates for arc centers are referenced from the ℄ and crest of the dam.



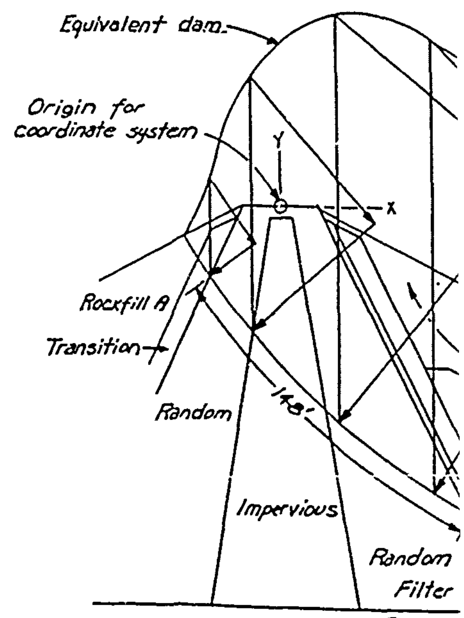
TYPICAL FORCE DIAGRAM



SUMMAT



SUMMATIO



Filter C

STABILITY AN
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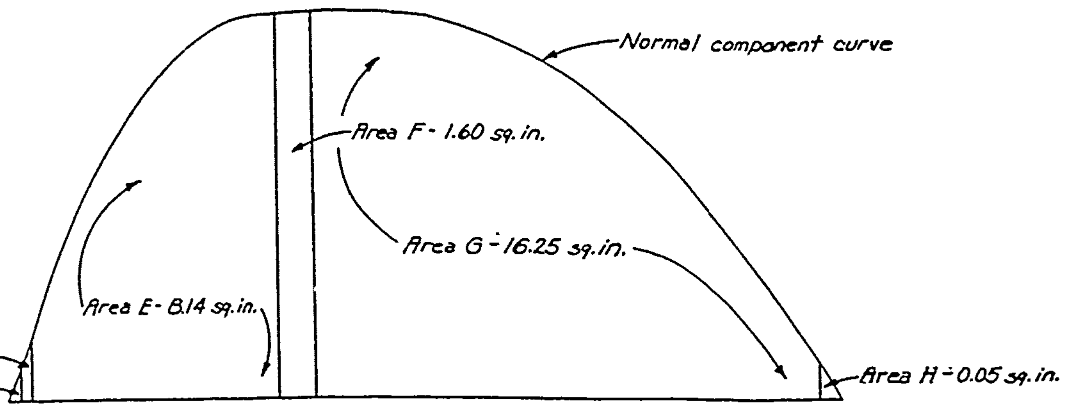
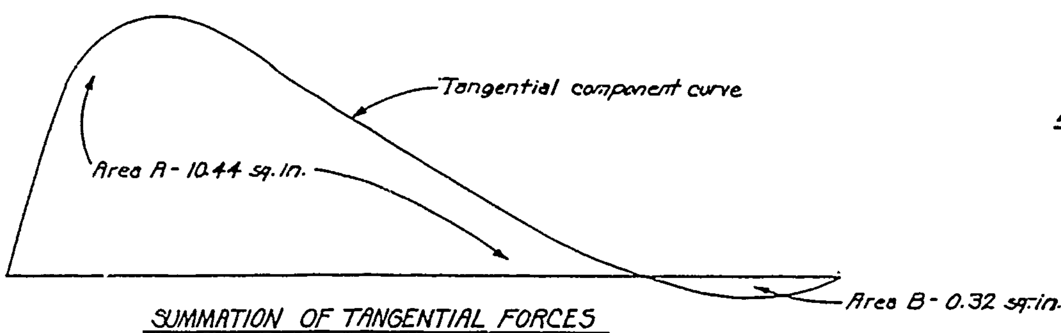


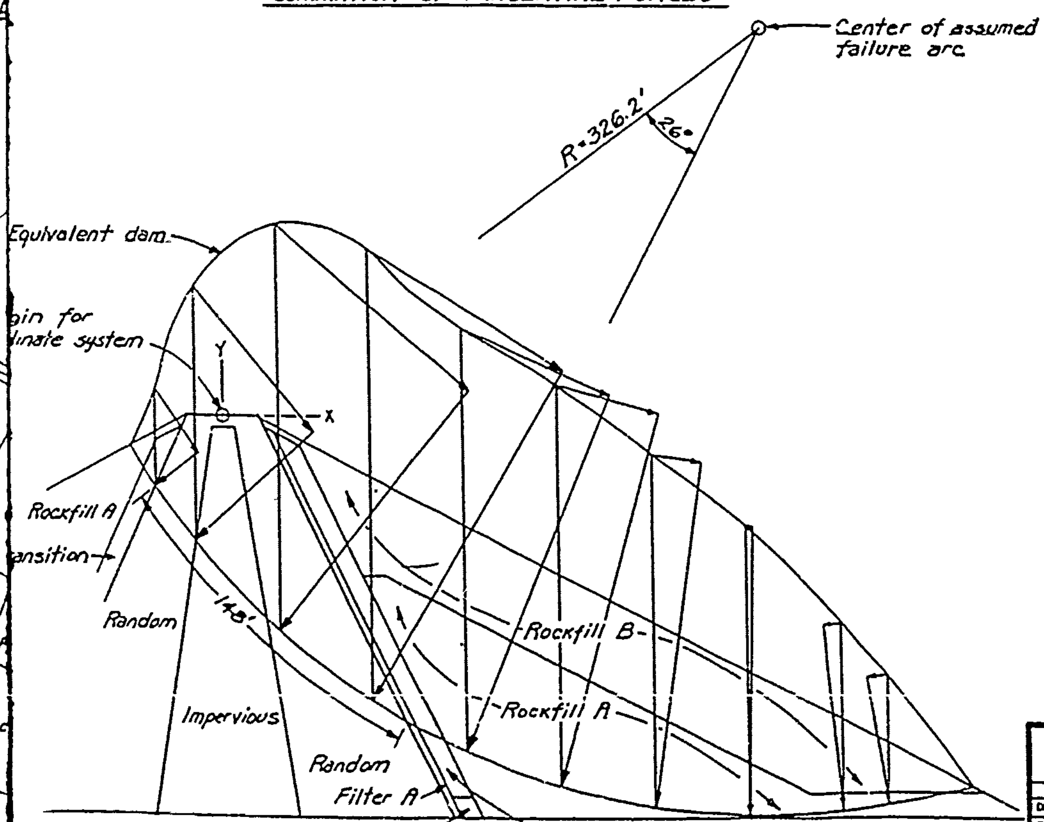
SUMMATION OF NORMAL FORCES

1 Sq. In. = 50 Tons

AREA SCALE



SUMMATION OF TANGENTIAL FORCES



STABILITY ANALYSIS OF DOWNSTREAM SLOPES UNDER CONDITION OF END OF CONSTRUCTION

40 0 40 80

SCALE: 1" = 40'

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — CARNATOT RIVER, ARKANSAS

	BY	CHKD
DESIGNED	D.E.W.	K.L.
DRAWN	D.E.W.	D.E.W.
TRACED	D.E.W.	D.E.W.

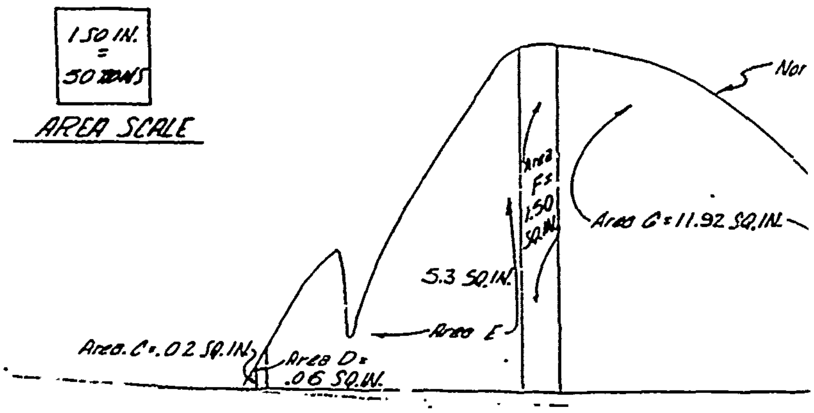
GILLHAM DAM AND RESERVOIR
EMBANKMENT
STABILITY ANALYSIS - END OF CONSTRUCTION DOWNSTREAM

SUBMITTED: CHIEF, S.& M. SECTION
RECOMMENDED: CHIEF, F.& M. BRANCH
APPROVED: CHIEF, ENGRG. DIV. FOR THE DISTRICT ENGINEER

DATE: FEB. 1971
SCALE: AS SHOWN
DRAWING NO. 1770-DM9-98/8

ADOPTED DESIGN VALUES						
MATERIALS	SOIL WTS. LBS/FT³		SHEAR STRENGTHS			
	Saturated	Submerged	*	Q	R	S
Rockfill A	105** 130	67.6	ϕ	42	42	42
			C	0	0	0
Rockfill B	135	72.6	ϕ	36	36	36
			C	0	0	0
Transition	125	62.5	ϕ	33	33	33
			C	0	0	0
Filter A	125	62.5	ϕ	33	35	33
			C	0	0	0
Random Fill	125	62.5	ϕ	5	22	28
			C	0.5	0.2	0
Impervious	125	62.5	ϕ	5	22	28
			C	0.5	0.2	0

*ϕ = Angle of Internal friction (degrees)
** = Saturated Surface Dry weight (40% Voids)
C = Cohesion (Tons/Ft²)



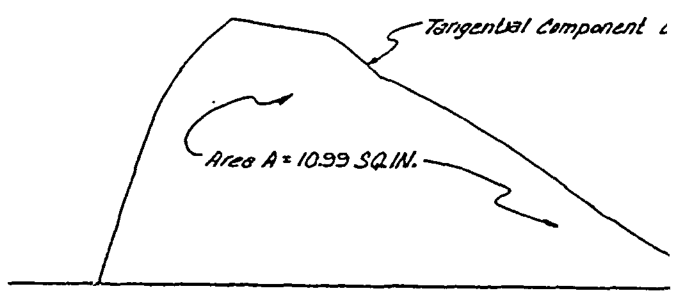
SUMMATION OF NORMAL ...

FORCES ACTING ON FAILURE ARC

Positive tangential force - Area A = 549.5 tons
Negative tangential force Area B = -64.0 tons
Σ TOTAL = 485.5 tons

NORMAL AND RESISTING FORCES

Normal force Area C	1.0	Tons
Normal force Area D	3.0	Tons
Normal force Area E	265.0	Tons
Normal force Area F	75.0	Tons
Normal force Area G	596.0	Tons
Normal force Area H	2.0	Tons



SUMMATION OF TANGENTIAL

SAFETY FACTOR COMPUTATIONS

FORMULA: $S.F. = \frac{\Sigma N \times \tan\phi + LC}{\Sigma T}$

For consolidated-drained (S) condition

$$SF = \frac{597 \tan 42° + 78 \tan 33° + 265.0 \tan 28° + 2.0 \tan 36°}{485.5}$$

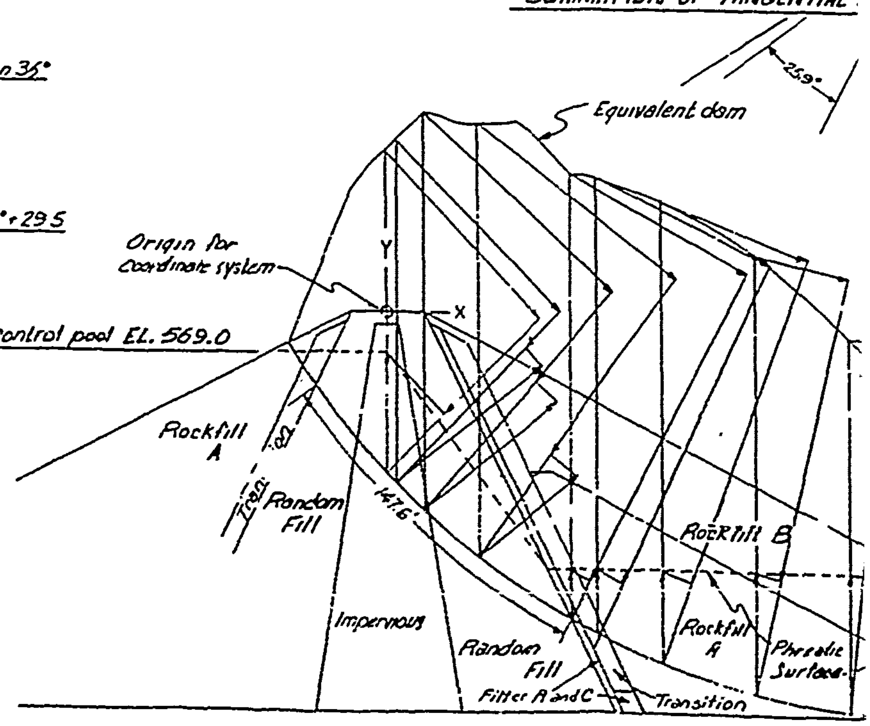
SF = 1.50

For consolidated-undrained (R) condition

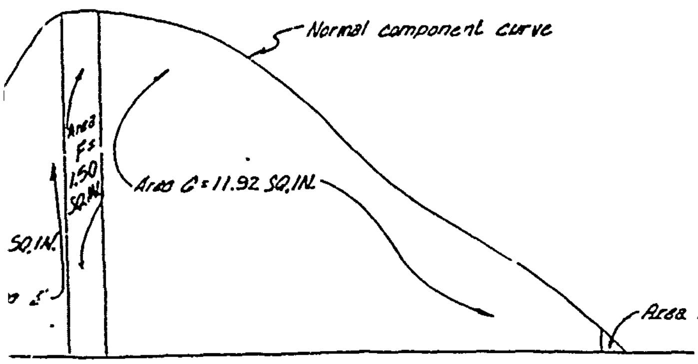
$$S.F. = \frac{597 \tan 42° + 78 \tan 33° + 265.0 \tan 22° + 2 \tan 36° + 29.5}{485.5}$$

S.F. = 1.495

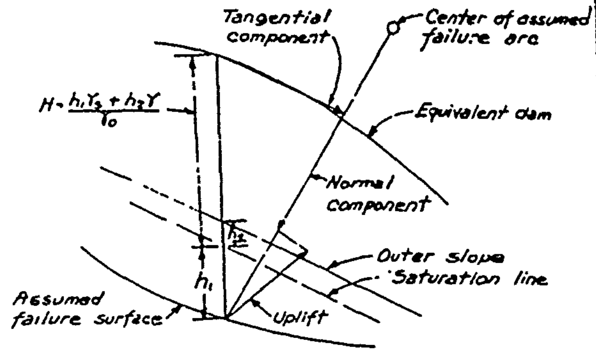
Average R and S = 1.50



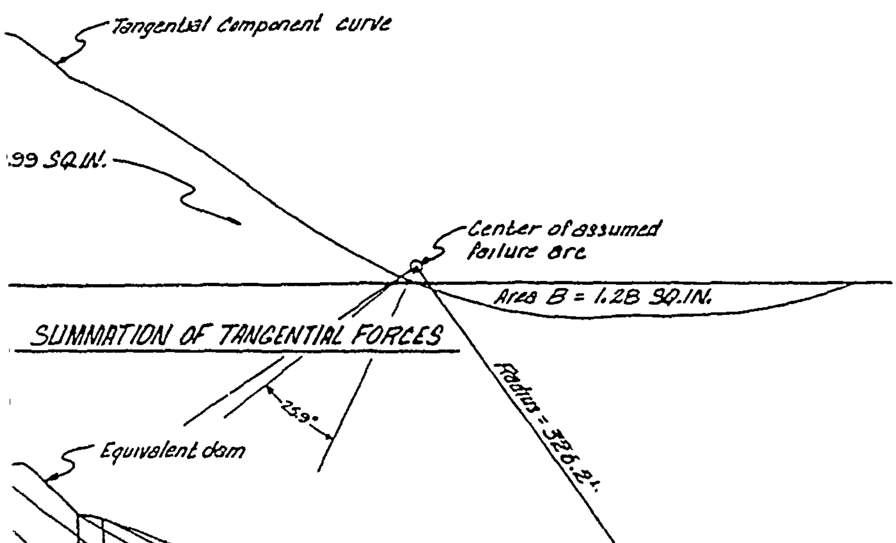
STABILITY ANALYSIS OF DOWNSTREAM SLOPE
CONDITION OF STEADY SEEPAGE
SCALE: 1" = 40'



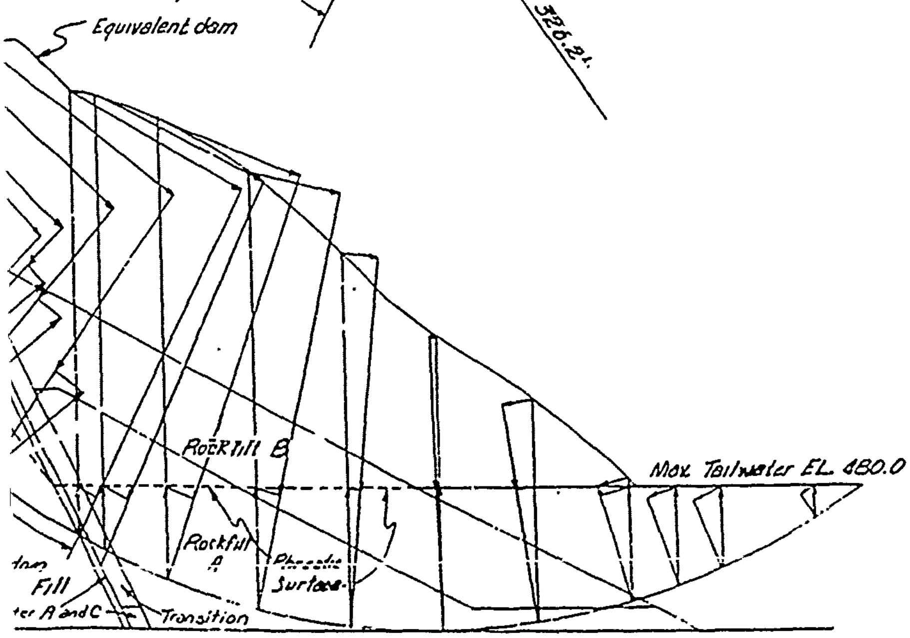
UMMATION OF NORMAL FORCES



TYPICAL FORCE DIAGRAM



SUMMATION OF TANGENTIAL FORCES



…IS OF DOWNSTREAM SLOPES UNDER
…OF STEADY SEEPAGE

SCALE: 1" = 40'

SAFETY FACTOR SUMMARY				
RADIUS (FT.)	Coordinates (Center of Arc)		Safety Factor	
	X (ft)	Y (ft)	R strength	S strength
210.8	191.0	47.0	1.519	1.436
326.2	233.5	161.0	1.486	1.497
483.7	321.0	324.0	1.595	1.578
207.5	210.0	42.0	1.638	1.593
442.5	293.0	281.0	1.557	1.551
165.2	159.5	20.0	1.642	1.597
492.0	318.0	329.0	1.579	1.570
299.3	244.5	146.5	1.575	1.542
394.1	305.0	228.5	1.615	1.585
186.9	170.0	22.0	1.533	1.505

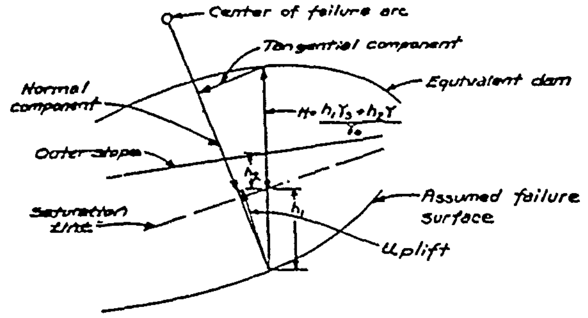
Note:
The above table includes results of all arcs analyzed for the steady seepage condition. Coordinates for arc centers are referenced from the ℄ and Crest of the dam.

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

	BY	CHKD	
DESIGNED	DEW	KL	RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS
DRAWN	DEW	DEW	GILLHAM DAM AND RESERVOIR
TRACED	DEW	DEW	EMBANKMENT

STABILITY ANALYSIS-STEADY SEEPAGE

SUBMITTED: CHIEF, SOILS DESIGN SECTION
RECOMMENDED: CHIEF, F&M BRANCH
APPROVED: CHIEF, ENGRG DIV. FOR THE DIST. ENGINEER
DATE: FEB. 1971
SCALE: AS SHOWN
DRAWING NO. 1770-DM9-98/9



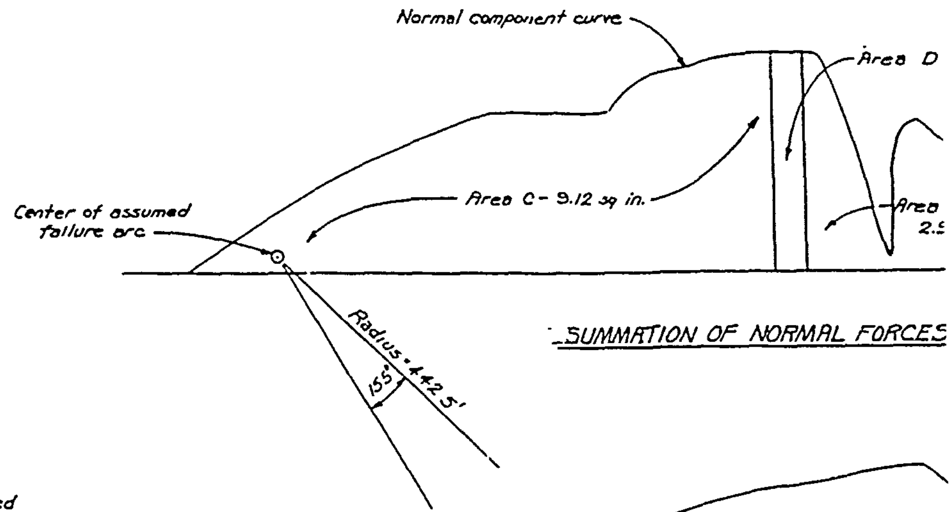
TYPICAL FORCE DIAGRAM

1 Sq. In. = 50 Tons

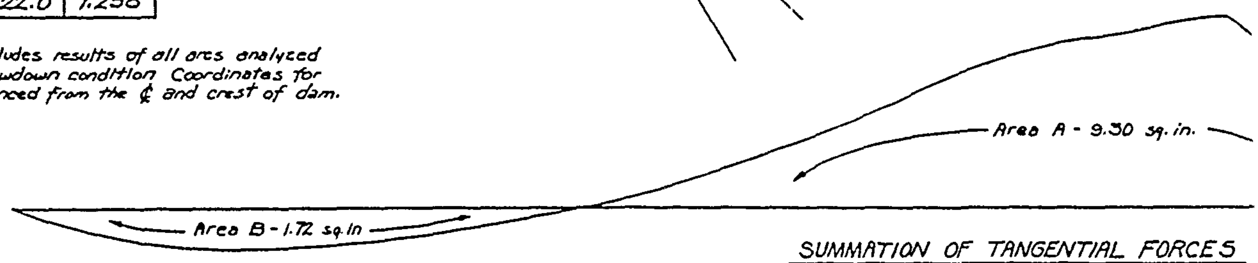
AREA SCALE

Radius (ft.)	Coordinates (Center of Arc) X (ft.)	Y (ft.)	Safety Factor
218.8	-191.0	47.0	1.224
326.2	-233.5	161.0	1.181
483.7	-321.0	324.0	1.179
207.5	-210.0	42.0	1.287
442.5	-293.0	281.0	1.174
165.2	-159.5	20.0	1.246
492.0	-318.0	329.0	1.178
295.3	-244.5	146.5	1.179
394.1	-305.0	228.5	1.202
186.9	-170.0	22.0	1.258

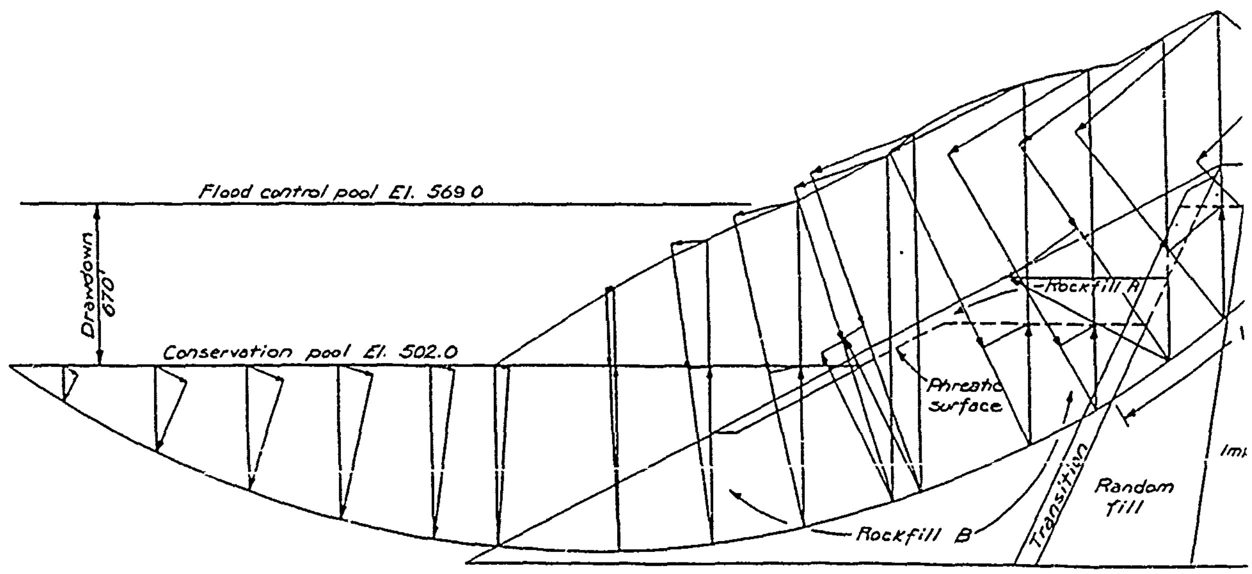
NOTE:
The above table includes results of all arcs analyzed for the sudden drawdown condition. Coordinates for arc centers referenced from the ℄ and crest of dam.



SUMMATION OF NORMAL FORCES



SUMMATION OF TANGENTIAL FORCES



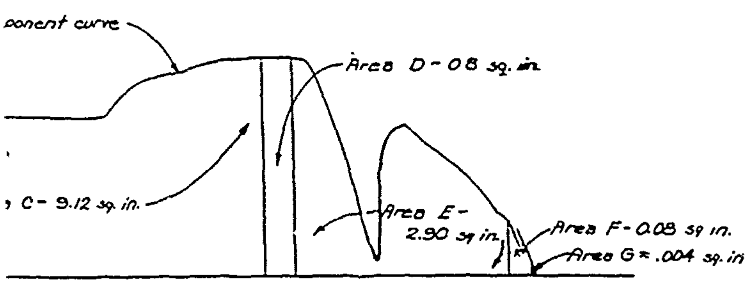
STABILITY ANALYSIS OF UPSTREAM SLOPE
CONDITION OF SUDDEN DRAWDOWN

SCALE: 1" = 40'

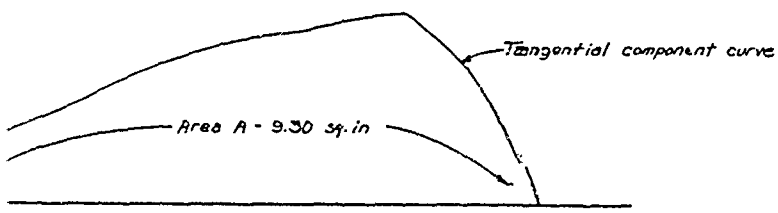
(1)

1 Sq. In. = 50 Tons

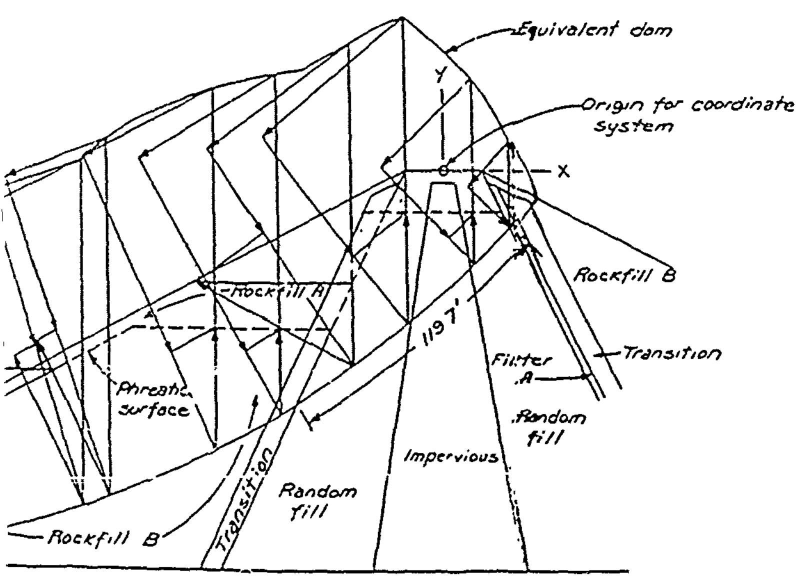
AREA SCALE



SUMMATION OF NORMAL FORCES



SUMMATION OF TANGENTIAL FORCES



STABILITY ANALYSIS OF UPSTREAM SLOPES UNDER CONDITION OF SUDDEN DRAWDOWN

SCALE: 1" = 40'

40 0 40 80

ADOPTED DESIGN DATA						
MATERIALS	Soil Wts. Lbs/Ft³		Shear Strengths			
	Saturated	Submerged	*	Q	R	S
ROCKFILL A	105** 130	67.6	φ	42	42	42
			c	0	0	0
ROCKFILL B	135	72.6	φ	36	36	36
			c	0	[illegible]	0
TRANSITION	125	62.5	φ	33	33	33
			c	0	[illegible]	0
FILTER A	125	62.5	φ	33	33	33
			c	0	[illegible]	0
RANDOM FILL	125	62.5	φ	5	22	28
			c	0.5	0.2	0
IMPERVIOUS	125	62.5	φ	5	22	28
			c	0.5	0.2	.0

* φ = Angle of internal friction (degrees)
** = Saturated surface dry weight (40% Voids)
c = Cohesion (Tons/Ft²)

Forces acting on failure arc

Positive tangential force - Area A = 465.0 Tons
Negative tangential force - Area B = -86.0 Tons
ΣTotal = 379.0 Tons

Normal and resisting forces

Normal force Area C ———— = 456.0 Tons
Normal force Area D ———— = 40.0 Tons
Normal force Area E ———— = 145.0 Tons
Normal force Area F ———— = 4.0 Tons
Normal force Area G ———— = 0.2 Tons

Cohesion (LC) = 119.7' × 0.2 = 23.9 Tons

Safety factor computations

Formula: $S.F. = \frac{\Sigma N \times \tan\phi + LC}{\Sigma T}$

For consolidated undrained (R) condition:

$$S.F. = \frac{456.2 \tan 36° + 44 \tan 33° + 145.0 \tan 22° + 23.9}{379.0}$$

S.F. = 1.168

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — CASSATOT RIVER, ARKANSAS

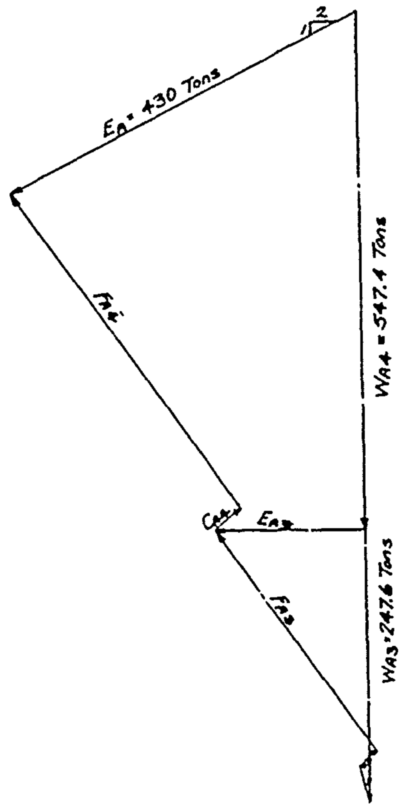
GILLHAM DAM AND RESERVOIR
EMBANKMENT
STABILITY ANALYSIS - SUDDEN DRAWDOWN

DESIGNED / DRAWN / TRACED — SUBMITTED: CHIEF, SOIL MECH. SECTION — RECOMMENDED: CHIEF, F&M BRANCH — APPROVED: CHIEF, ENGRG. DIV. FOR THE DISTRICT ENGINEER

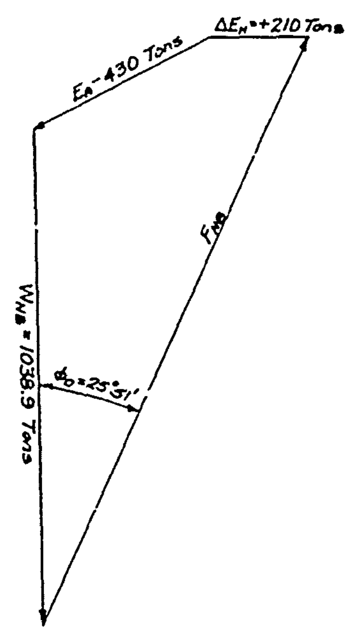
DATE: FEB. 1971

SCALE: AS SHOWN

DRAWING NO. 1770-DM9-98/10



FORCE POLYGON FOR ACTIVE WEDGE FOR TRIAL F.S = 1.50
1" = 100 Tons

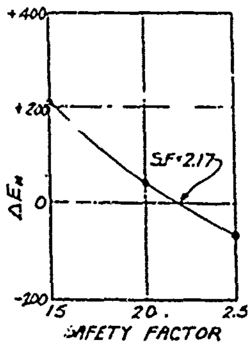


FORCE POLYGON FOR NEUTRAL BLOCK FOR TRIAL F.S = 1.50
1" = 200 Tons

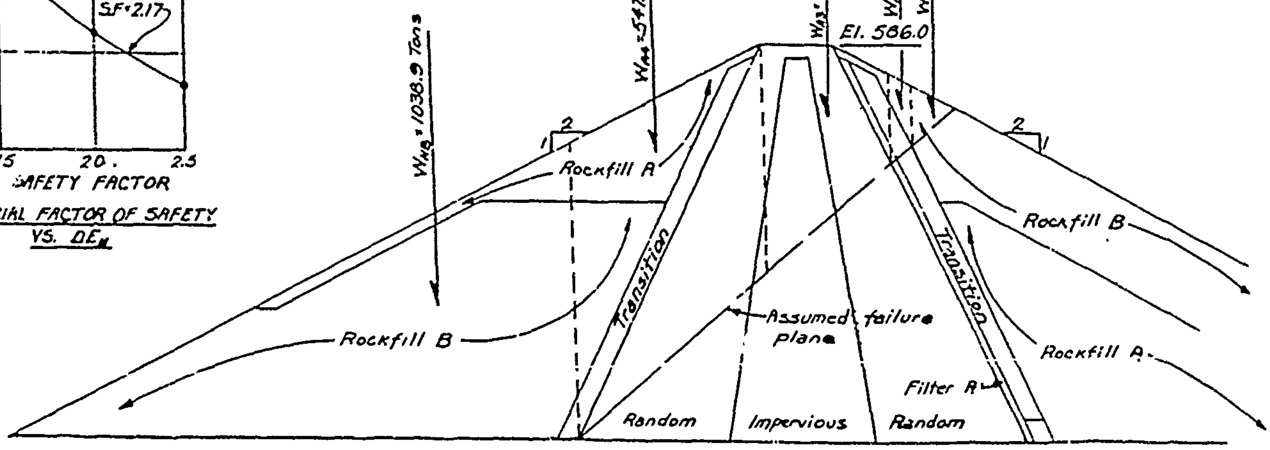
ADOPTED DESIGN DATA				
MATERIALS	SOIL WTS. LBS/FT³ SATURATED	SOIL WTS. LBS/FT³ SUBMERGED	SHEAR STRENGTH (Q TEST)	
Rockfill A	105** 130	67.6	φ*	42
			C	0
Rockfill B	135	72.6	φ	36
			C	0
Transition	125	62.5	φ	33
			C	0
Filter	125	62.5	φ	33
			C	0
Random fill	125	62.5	φ	5
			C	0.5
Impervious	125	62.5	φ	5
			C	0.5

*φ = Angle of internal friction (Degrees)
** = Saturated surface dry weight (40% voids)
C = Cohesion (Tons/ft²)

$C_D = C/F.S.$
$\tan \phi_D = (\tan \phi)/F.S.$



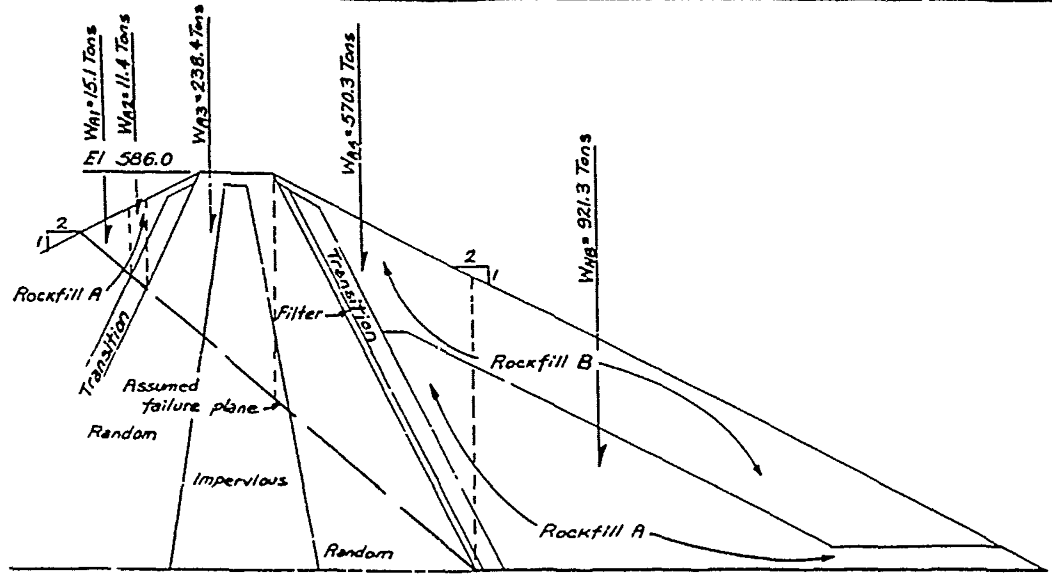
TRIAL FACTOR OF SAFETY VS. ΔE_H



STABILITY ANALYSIS OF UPSTREAM SLOPE UNDER CONDITION OF END OF CONSTRUCTION
1" = 40'

40 0 40 80

(1)



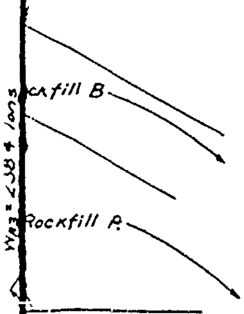
STABILITY ANALYSIS OF DOWNSTREAM SLOPE UNDER CONDITION OF END OF CONSTRUCTION

1" = 40'

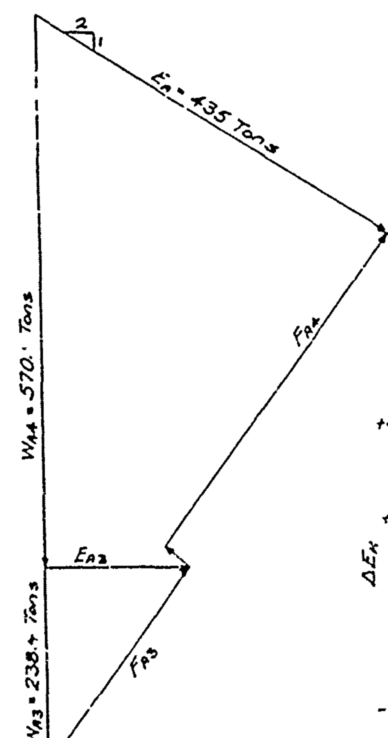
D DESIGN DATA			
SOIL WTS. LBS/FT³		SHEAR STRENGTH (Q TEST)	
SATURATED	SUBMERGED		
105** 130	67.6	φ*	42
		c	0
135	72.6	φ	36
		c	0
125	62.5	φ	33
		c	0
125	62.5	φ	33
		c	0
125	62.5	φ	5
		c	0.5
125	62.5	φ	5
		c	0.5

f internal friction (Degrees)
ed surface dry weight (40% voids)
on (Tons/ft²)

S.
an φ)/F.S.

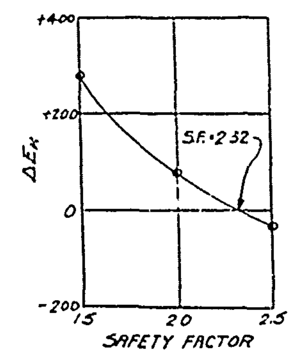


ITION OF

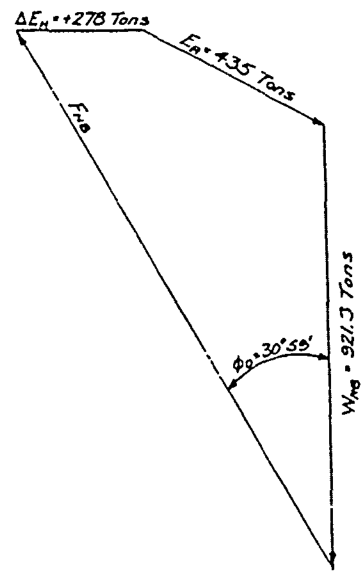


FORCE POLYGON FOR ACTIVE WEDGE FOR TRIAL F.S = 1.50

1" = 100 Tons



TRIAL FACTOR OF SAFETY VS ΔE_H



FORCE POLYGON FOR NEUTRAL BLOCK FOR TRIAL F.S = 1.50

1" = 200 Tons

U.S. ARMY ENGINEER DISTRICT, TULSA CORPS OF ENGINEERS TULSA, OKLAHOMA		
	BY	CHKD
DESIGNED	R.E.W.	K.L.
DRAWN	D.E.W.	D.E.W.
TRACED	D.E.W.	D.E.W.

RED RIVER WATERSHED — CASSATOT RIVER, ARKANSAS

GILLHAM DAM AND RESERVOIR
EMBANKMENT
STABILITY ANALYSIS
END OF CONSTRUCTION

SUBMITTED: CHIEF, SOIL MECH. SECTION
RECOMMENDED: CHIEF, F&M BRANCH
APPROVED: CHIEF, ENGRG DIV. FOR THE DIST. ENGINEER
DATE FEB. 1971
SCALE: AS SHOWN
DRAWING NO. 1770-DM9-98/11

(2)

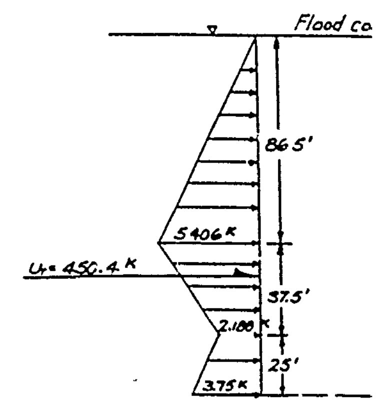
ADOPTED DESIGN DATA					
MATERIALS	SOIL WTS. LBS/FT³		SHEAR STRENGTH		
	SATURATED	SUBMERGED	*	R	S
Rockfill A	105** 130	67.6	Ø	42	42
			C	0	0
Rockfill B	135	72.6	Ø	36	36
			C	0	0
Transition	125	62.5	Ø	33	33
			C	0	0
Filter A	125	62.5	Ø	33	33
			C	0	0
Random	125	62.5	Ø	22	28
			C	0.2	0
Impervious	125	62.5	Ø	22	28
			C	0.2	0

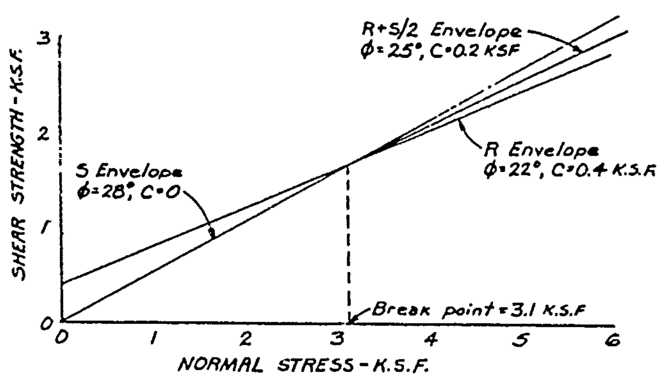
*Ø = Angle of internal friction (Degrees)
** = Saturated surface dry weight (40% voids)
C = Cohesion (Tons/Ft²)

$C_D = C/F.S.$
$\tan \phi_D = (\tan \phi / F.S.)$

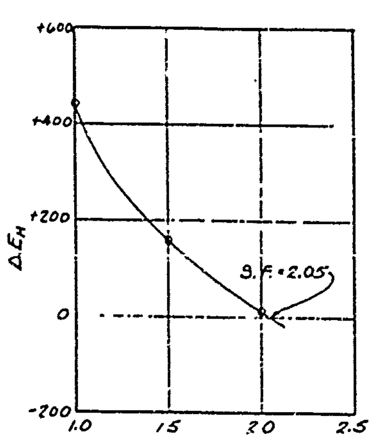
SHEAR STRENGTH TO USE FOR RANDOM AND IMPERVIOUS MATERIAL

Solve for h for $N = 3.1$ K.S.F. (See section)
$[(586-569)\cdot 0.125 + 0.0625h] \cos 40° = 3.1$ K.S.F.
$h = 30.75'$ (Use R+S/2)

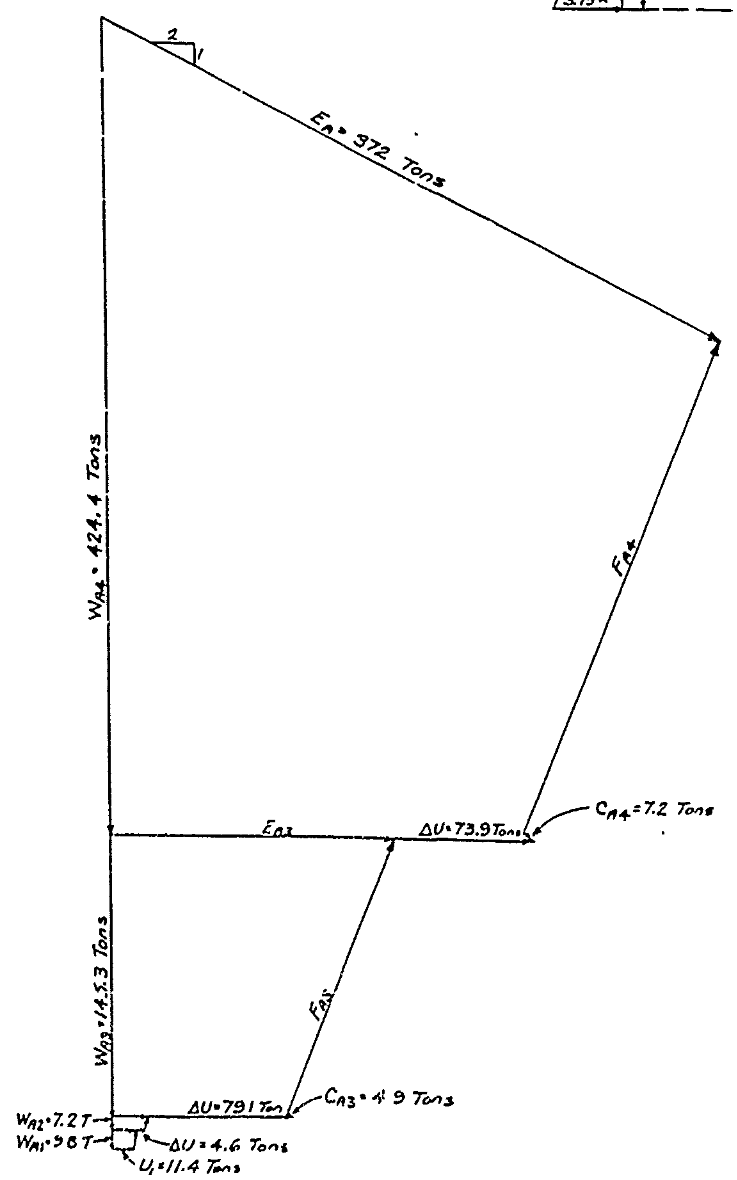




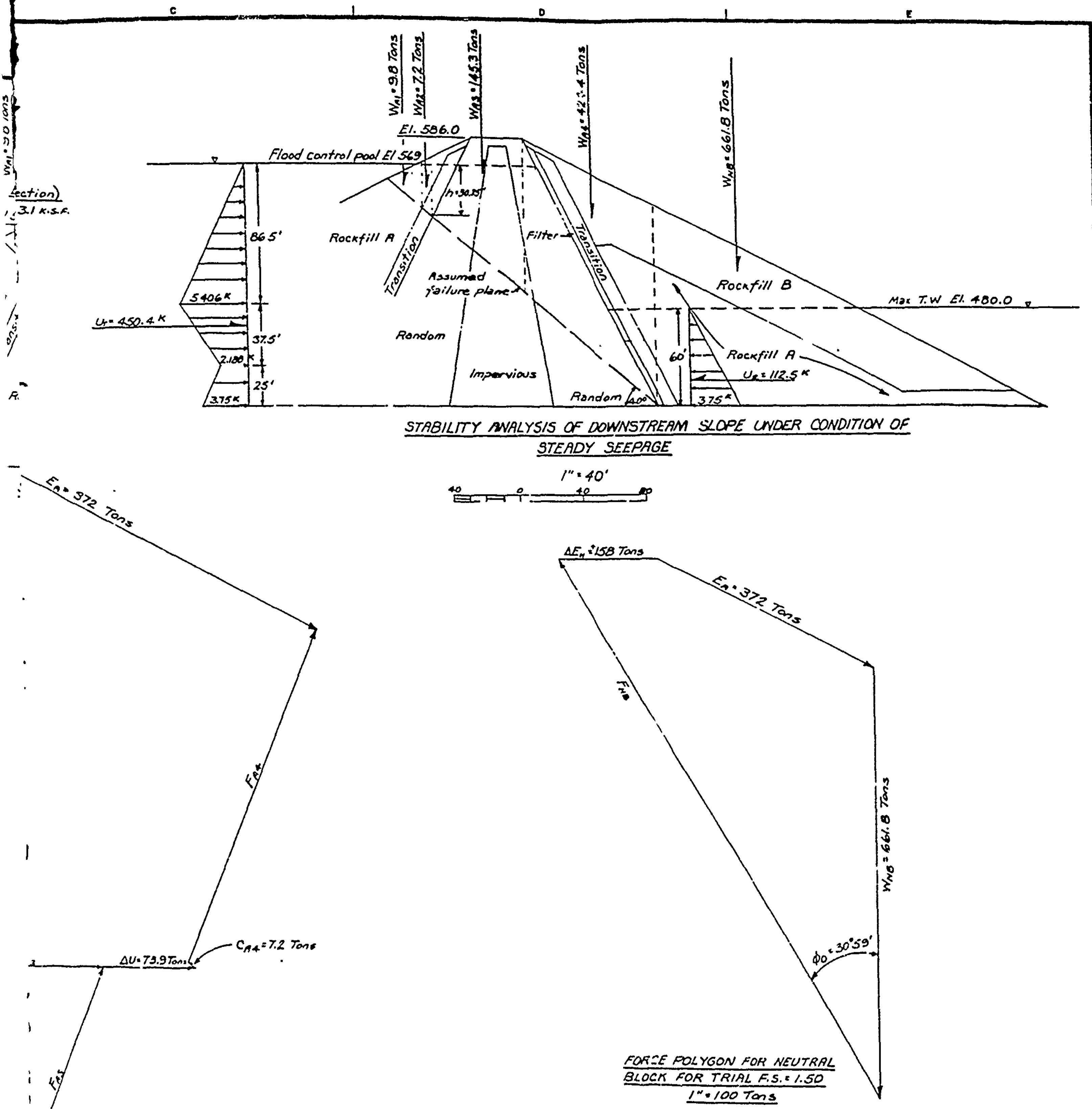
COMPOSITE STRENGTH ENVELOPES FOR RANDOM AND IMPERVIOUS MATERIALS



TRIAL FACTOR OF SAFETY VS. ΔE_H



FORCE POLYGON FOR ACTIVE WEDGE FOR TRIAL F.S. = 1.50
1" = 50 Tons



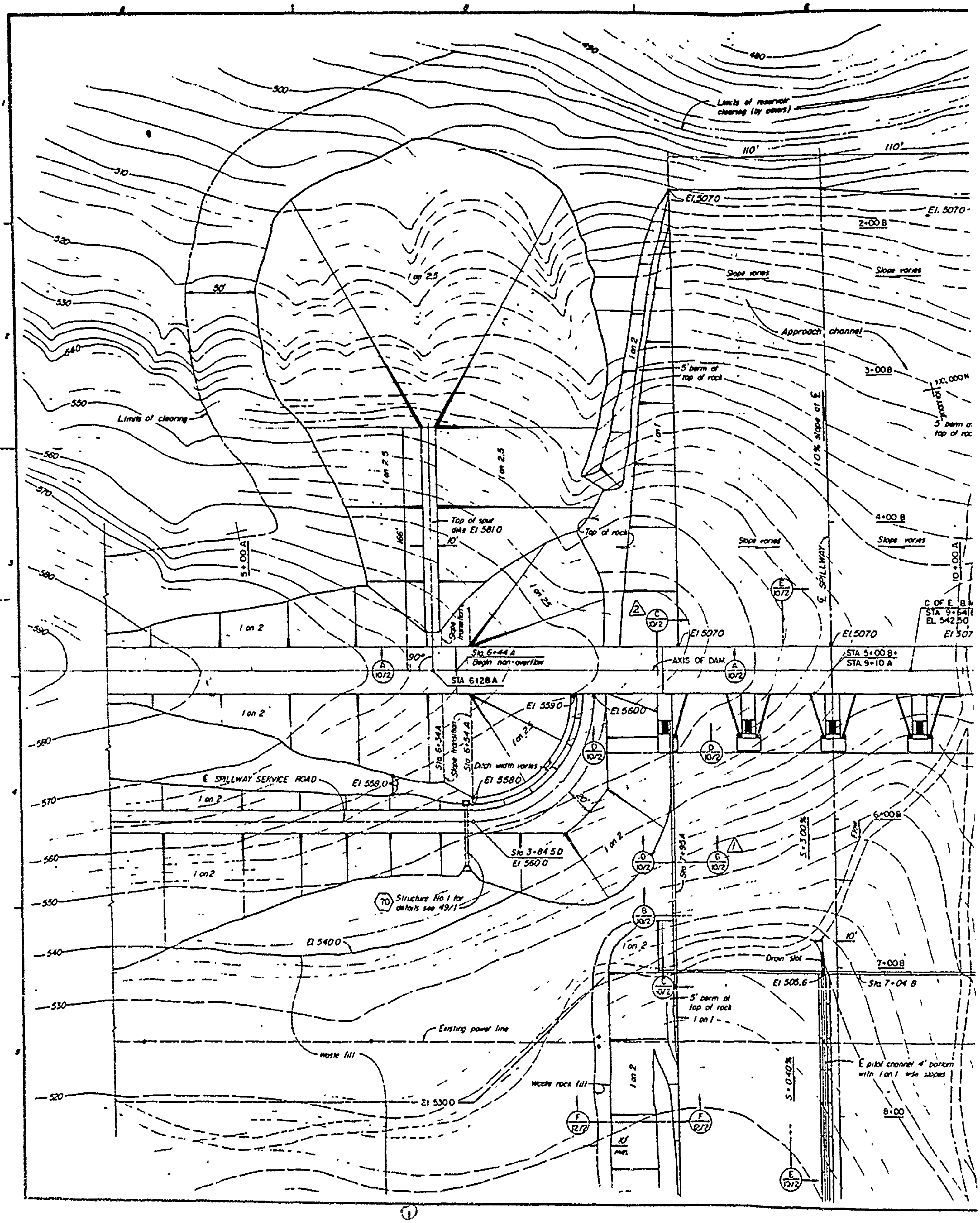
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — CASSATOT RIVER, ARKANSAS

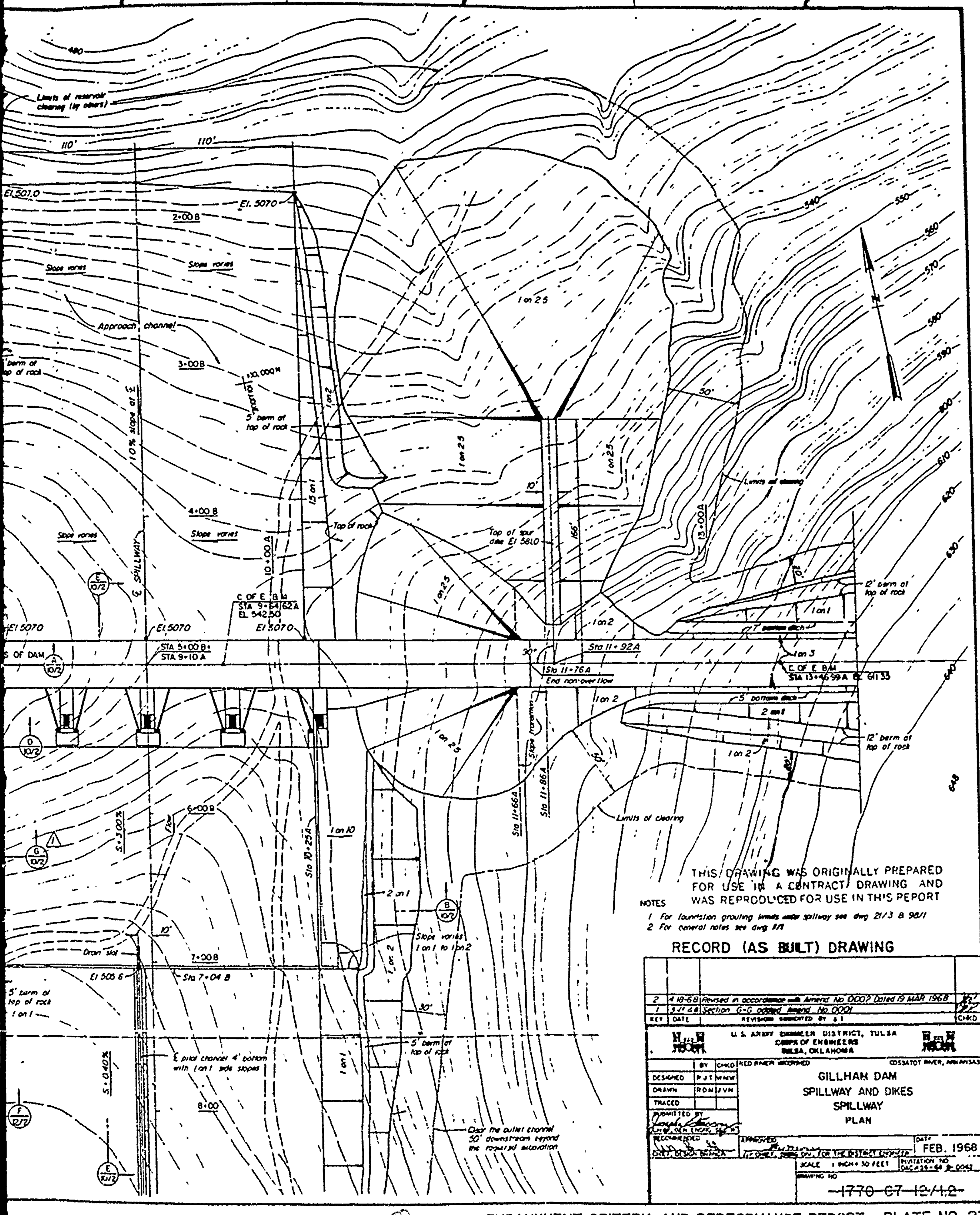
GILLHAM DAM AND RESERVOIR
EMBANKMENT
STABILITY ANALYSIS - STEADY SEEPAGE

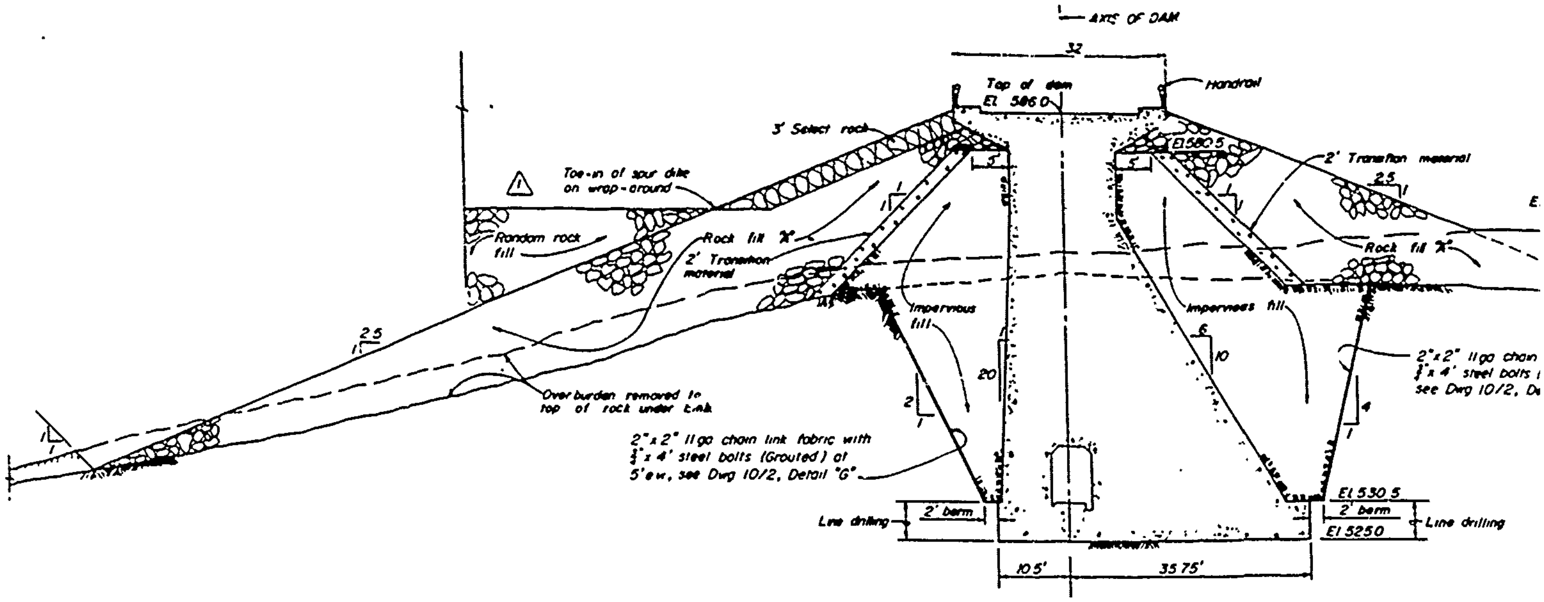
	BY	CHKD
DESIGNED	D.E.W.	K.L.
DRAWN	D.E.W.	D.E.W.
TRACED	D.E.W.	D.E.R.

SUBMITTED: CHIEF, SOIL MECH. SECTION
RECOMMENDED: CHIEF, F&M BRANCH
APPROVED: CHIEF, ENGRG. DIV. FOR THE DIST. ENGINEER
DATE: FEB. 1971
SCALE: AS SHOWN
DRAWING NO. 1770-DM9-98/12



Limits of reservoir clearing (by others)
110'
El 5070
El. 5070
2+00 B
Slope varies
1 on 2.5
Approach channel
5' berm at top of rock
3+00 B
10% slope at ℄
Limits of clearing
1 on 2.5
Top of spur dike El 5810
4+00 B
Top of rock
Slope varies
℄ SPILLWAY
5+00 A
10+00 A
1 on 2
Slope transition
Sta 6+44 A
Begin non-overflow
AXIS OF DAM
STA 6+28 A
STA 5+00 B
STA 9+10 A
El 559.0
El 560.0
Ditch width varies
℄ SPILLWAY SERVICE ROAD
El 558.0
El 558.0
Sta 3+84.5 D
El 560.0
Structure No 1 for details see 49/1
El 540.0
Drain slot
7+00 B
El 505.6
Sta 7+04 B
5' berm at top of rock
1 on 1
Existing power line
Waste fill
Waste rock fill
El 530.0
℄ pilot channel 4' bottom with 1 on 1 side slopes
8+00
6+00 B
Flow

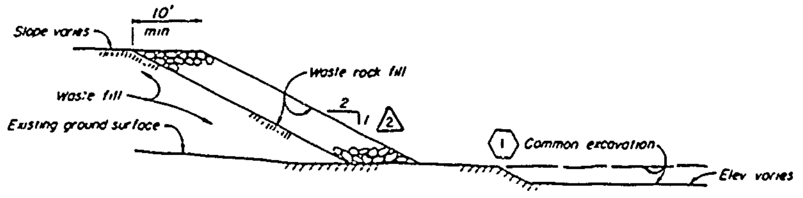

Limits of reservoir clearing (by others)
Approach channel
Slope varies
10% Slope at ℄
℄ SPILLWAY
5' berm at top of rock
1 on 25
1 on 2
15 on 1
Top of rock
Top of spur dike El 581.0
Limits of clearing
12' berm at top of rock
7' bottom ditch
5' bottom ditch
C OF E B.M STA 9+64.62A EL 542.50
C OF E B.M STA 13+46.59A EL 611.33
STA 5+00 B= STA 9+10 A
Sta 11+92A
Sta 11+76A End non-overflow
Sta 11+66A
Sta 11+86A
Sta 10+25A
S= 3.00%
S= 0.040%
Drain slot
Sta 7+04 B
℄ pilot channel 4' bottom with 1 on 1 side slopes
Clear the outlet channel 50' downstream beyond the required excavation
Slope varies 1 on 1 to 1 on 2
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
NOTES
1 For foundation grouting limits under spillway see dwg 21/3 8 98/1
2 For general notes see dwg 1/1
RECORD (AS BUILT) DRAWING
2 4-18-68 Revised in accordance with Amend No 0007 Dated 19 MAR 1968
1 Section G-G added Amend No 0001
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY
PLAN
FEB. 1968
SCALE 1 INCH = 30 FEET
1770-67-12/1.2



TYPICAL WRAP-AROUND SECTION

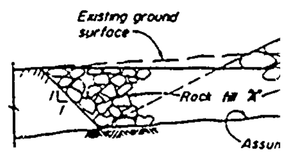
LEFT ABUTMENT

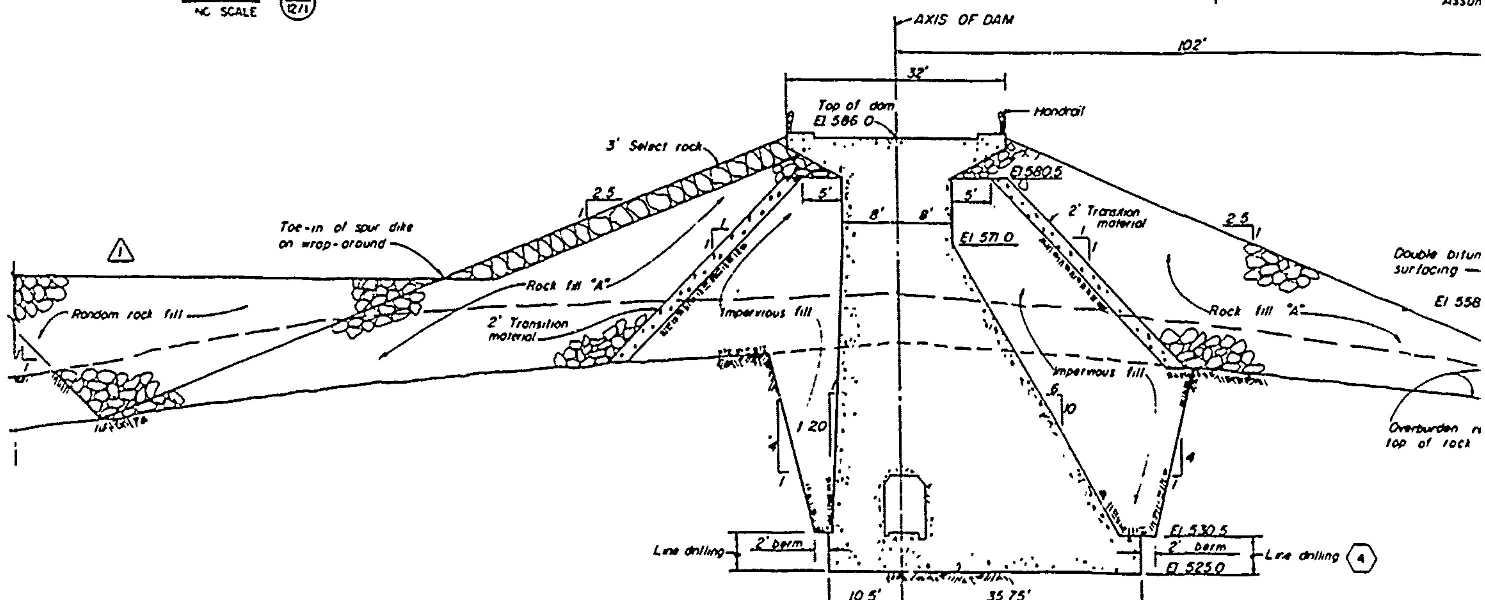
NO SCALE



SECTION F/12/1

NO SCALE

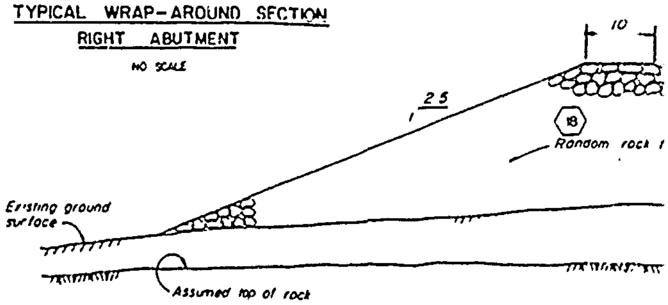




TYPICAL WRAP-AROUND SECTION

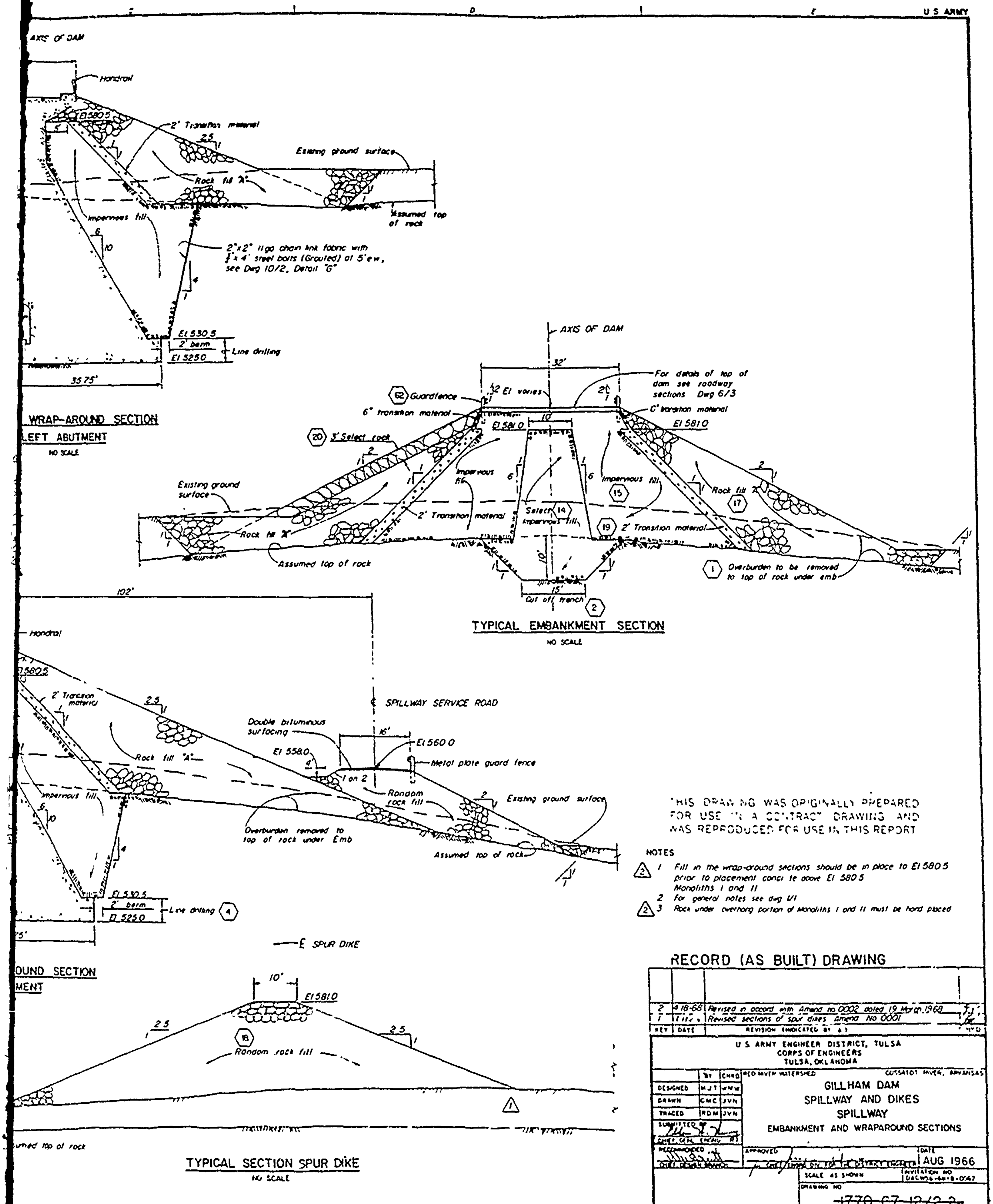
RIGHT ABUTMENT

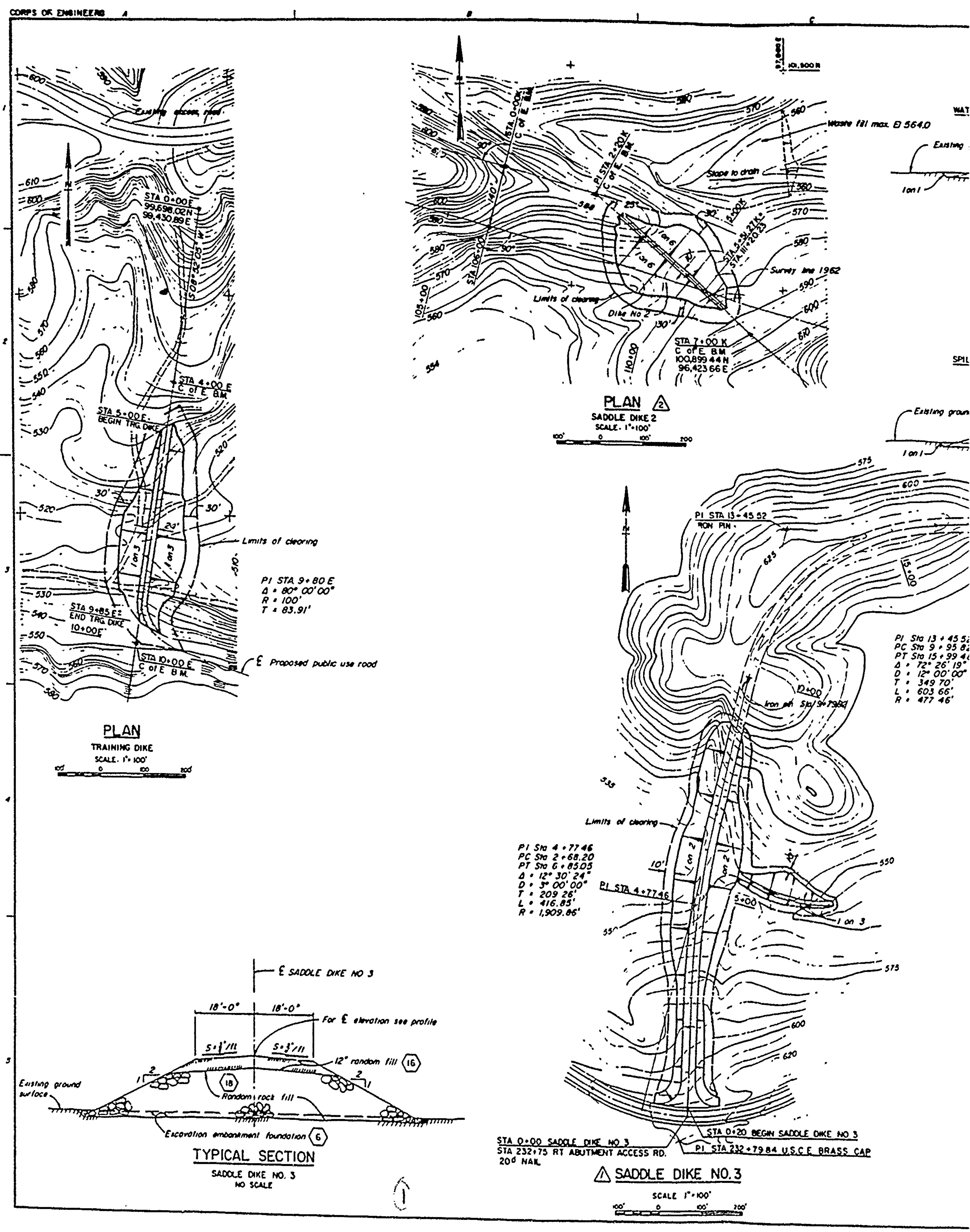
NO SCALE

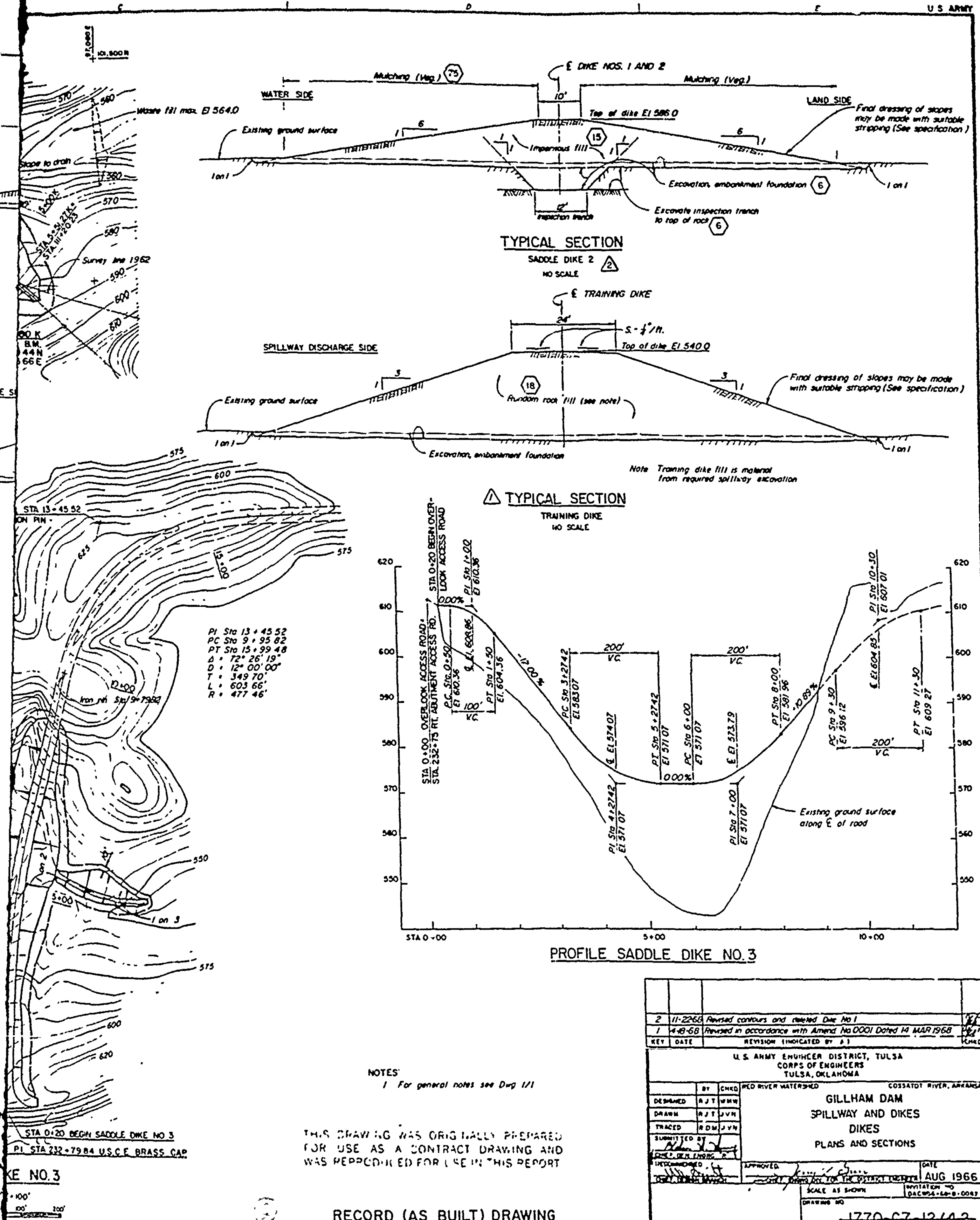


TYPICAL SECTION S...

NO SCALE

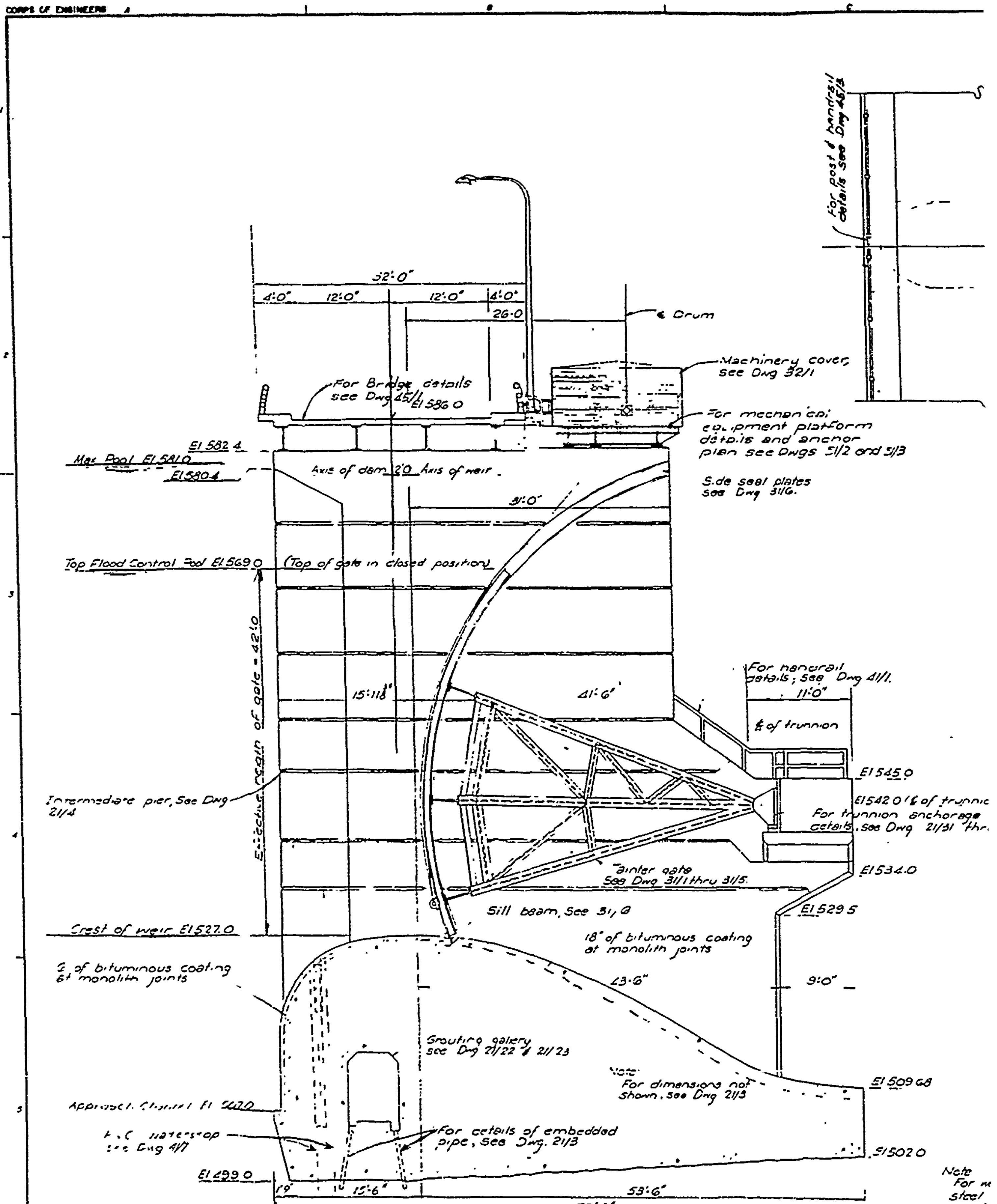

U S ARMY
AXIS OF DAM
Handrail
El 580.5
2' Transition material
2.5
Existing ground surface
Rock fill "A"
Impervious fill
Assumed top of rock
2"x2" 11ga chain link fabric with 3/4"x 4' steel bolts (Grouted) at 5' e.w., see Dwg 10/2, Detail "G"
El 530.5
2' berm
El 525.0
Line drilling
35.75'
WRAP-AROUND SECTION
LEFT ABUTMENT
NO SCALE
AXIS OF DAM
32'
Guardfence
El varies
For details of top of dam see roadway sections Dwg 6/3
6" transition material
El 581.0
10'
3' Select rock
Impervious fill
Existing ground surface
2' Transition material
Select impervious fill
Rock fill "A"
2' Transition material
Assumed top of rock
Overburden to be removed to top of rock under emb
15'
Cut off trench
TYPICAL EMBANKMENT SECTION
NO SCALE
102'
Handrail
El 580.5
2' Transition material
SPILLWAY SERVICE ROAD
Double bituminous surfacing
16'
El 560.0
El 558.0
Metal plate guard fence
1 on 2
Rock fill "A"
Random rock fill
Impervious fill
Existing ground surface
Overburden removed to top of rock under Emb
Assumed top of rock
El 530.5
2' berm
El 525.0
Line drilling
SPUR DIKE
OUND SECTION
MENT
10'
El 581.0
2.5
Random rock fill
umed top of rock
TYPICAL SECTION SPUR DIKE
NO SCALE
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
NOTES
1 Fill in the wrap-around sections should be in place to El 580.5 prior to placement conc te above El 580.5 Monoliths I and II
2 For general notes see dwg I/1
3 Rock under overhang portion of Monoliths I and II must be hand placed
RECORD (AS BUILT) DRAWING
2 4-18-68 Revised in accord with Amend no 0002 dated 19 March 1968
1 Revised sections of spur dikes Amend No 0001
KEY DATE REVISION (INDICATED BY Δ)
U S ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY
EMBANKMENT AND WRAPAROUND SECTIONS
DESIGNED MJT WMW
DRAWN CMC JVN
TRACED RDM JVN
SUBMITTED BY
RECOMMENDED
APPROVED
CHIEF, DESIGN BRANCH
CHIEF, ENGR DIV. FOR THE DISTRICT ENGINEER
DATE AUG 1966
SCALE AS SHOWN
DRAWING NO
1770-C7-12/2.2


CORPS OF ENGINEERS
Existing access road
STA 0+00E
99,698.02N
98,430.89E
STA 4+00E
C of E BM
STA 5+00E
BEGIN TRG DIKE
1 on 3
Limits of clearing
PI STA 9+80 E
Δ = 80° 00' 00"
R = 100'
T = 83.91'
STA 9+85 E
END TRG DIKE
10+00E
STA 10+00 E
C of E BM
℄ Proposed public use road
PLAN
TRAINING DIKE
SCALE: 1" = 100'
Waste fill max El 564.0
Slope to drain
Survey line 1962
Limits of clearing
Dike No 2
1 on 6
STA 7+00 K
C of E BM
100,899.44 N
96,423.66 E
PLAN
SADDLE DIKE 2
SCALE: 1" = 100'
PI STA 13+45.52
IRON PIN
PI Sta 13+45.52
PC Sta 9+95.82
PT Sta 15+99.4
Δ = 72° 26' 19"
D = 12° 00' 00"
T = 349.70'
L = 603.66'
R = 477.46'
Iron pin Sta 9+79.80
Limits of clearing
PI Sta 4+77.46
PC Sta 2+68.20
PT Sta 6+85.05
Δ = 12° 30' 24"
D = 3° 00' 00"
T = 209.26'
L = 416.85'
R = 1,909.86'
PI STA 4+77.46
1 on 2
1 on 3
℄ SADDLE DIKE NO 3
18'-0"
18'-0"
For ℄ elevation see profile
12" random fill (16)
Existing ground surface
Random rock fill
(18)
Excavation embankment foundation (6)
TYPICAL SECTION
SADDLE DIKE NO. 3
NO SCALE
STA 0+00 SADDLE DIKE NO 3
STA 232+75 RT ABUTMENT ACCESS RD.
20d NAIL
STA 0+20 BEGIN SADDLE DIKE NO 3
PI STA 232+79.84 U.S.C.E. BRASS CAP
SADDLE DIKE NO. 3
SCALE 1" = 100'



U S ARMY
℄ DIKE NOS. 1 AND 2
Mulching (Veg.)
WATER SIDE
LAND SIDE
10'
Top of dike El 586.0
Waste fill max. El 564.0
Existing ground surface
Impervious fill
1 on 1
Final dressing of slopes may be made with suitable stripping (See specification)
Excavation, embankment foundation
Excavate inspection trench to top of rock
12'
Inspection trench
TYPICAL SECTION
SADDLE DIKE 2
NO SCALE
℄ TRAINING DIKE
24'
Top of dike El 540.0
SPILLWAY DISCHARGE SIDE
Random rock fill (see note)
Excavation, embankment foundation
Note: Training dike fill is material from required spillway excavation
TYPICAL SECTION
TRAINING DIKE
NO SCALE
PI Sta 13 + 45.52
PC Sta 9 + 95.82
PT Sta 15 + 99.48
Δ = 72° 26' 19"
D = 12° 00' 00"
T = 349.70'
L = 603.66'
R = 477.46'
STA 13 + 45.52
Survey line 1962
Slope to drain
1 on 2
1 on 3
STA 0+00 OVERLOOK ACCESS ROAD = STA 232+75 RT. ABUTMENT ACCESS RD.
STA 0+20 BEGIN OVERLOOK ACCESS ROAD
200' VC
100' VC
0.00%
-12.00%
+10.69%
Existing ground surface along ℄ of road
PROFILE SADDLE DIKE NO. 3
NOTES:
1. For general notes see Dwg 1/1
STA 0+20 BEGIN SADDLE DIKE NO 3
PI STA 232+79.84 U.S.C.E. BRASS CAP
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
RECORD (AS BUILT) DRAWING
2 11-22-66 Revised contours and deleted Dike No 1
1 4-8-68 Revised in accordance with Amend No 0001 Dated 14 MAR 1968
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
DIKES
PLANS AND SECTIONS
AUG 1966
SCALE AS SHOWN
1770-C7-12/4.2

CORPS OF ENGINEERS

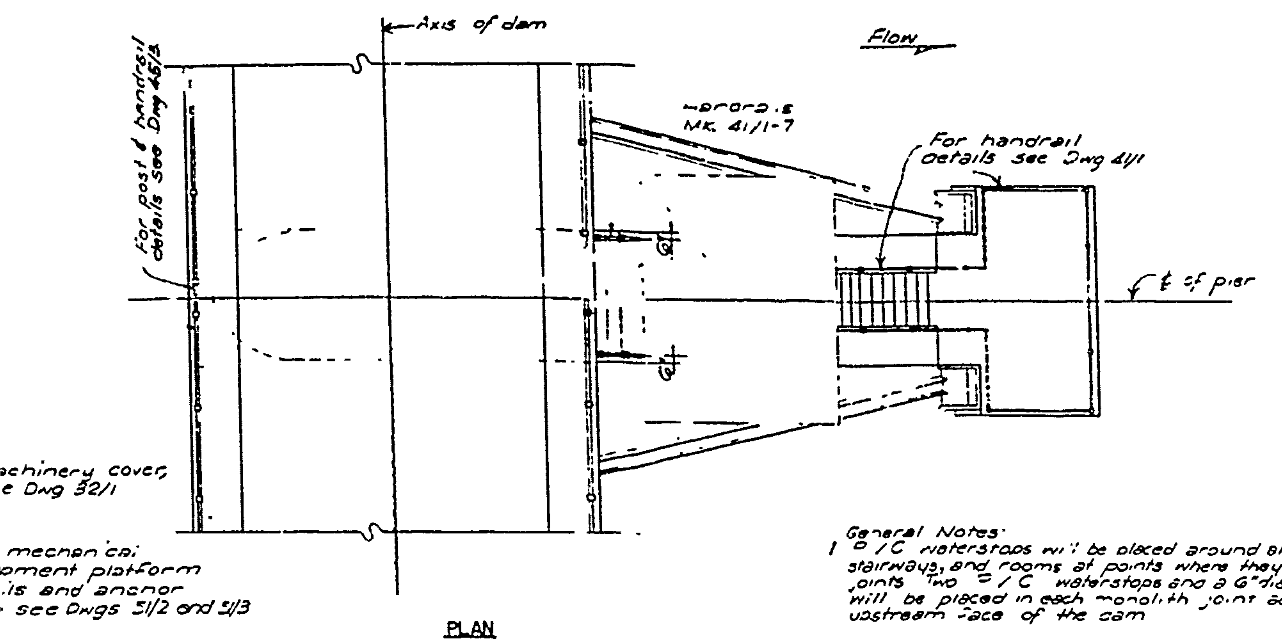


ELEVATION

TYPICAL STRUCTURAL ARRANGEMENT

SCALE 1/8" = 1'-0"

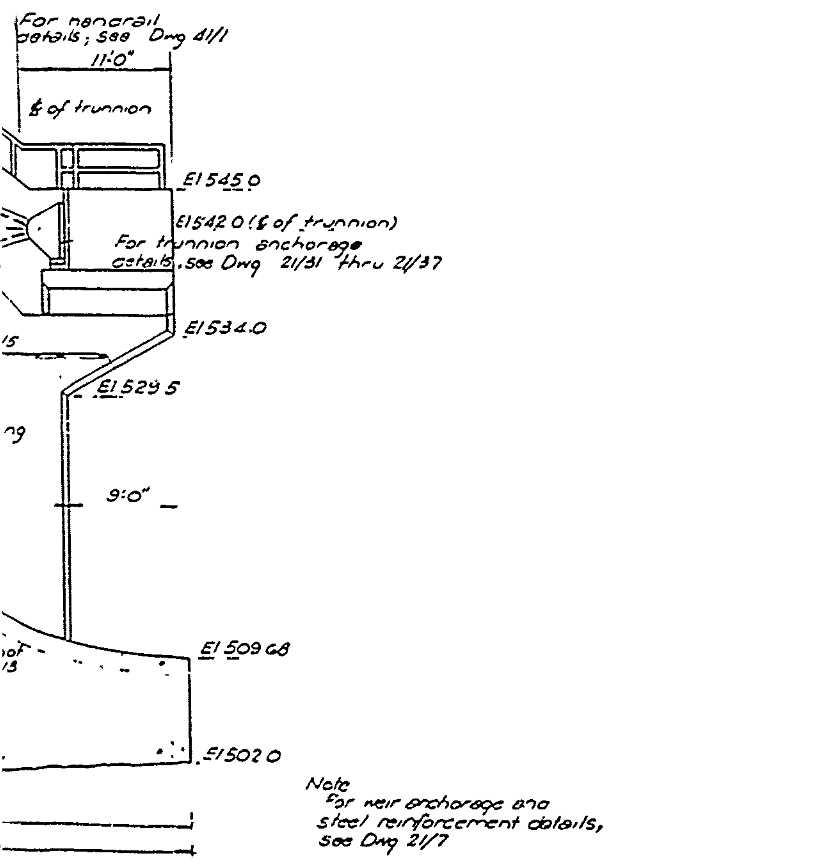
(1)



General Notes:

1 P/C waterstops will be placed around all passageways, stairways, and rooms at points where they cross monolith joints. Two P/C waterstops and a 6" diameter formed hole will be placed in each monolith joint adjacent to the upstream face of the dam.

2 All exterior corners and edges of concrete [illegible] chamfered 1/2" unless otherwise noted.
3 Exterior construction joints shall be chamfered [illegible] where shown on drawings. See 21/3 for details.
4 The drawings shall not be scaled. Use dimensions and elev shown.
5 The minimum Conc cover for reinforcing bars shall be not less than 4" unless otherwise noted.
6 Forms which support the roof of all galleries, passageways, stairwells, and rooms shall be left in place for a period of not less than 14 days after completion of the pour forming the roof.
7 Straight bars: The identification mark consists of the bar size designation number followed by the bar length in feet and inches. For even feet lengths, the 0 denoting inches is omitted. For lengths less than 10 feet, the number denoting feet is preceded by 0.
8 Bent bars: The identification mark consists of two numbers separated by one or more letters. The first number is the bar size number. The particular letter or letters indicate the pattern of bend with variable dimensions. The final number following the shape designation letter identifies the particular set of dimensions shown on the bar bending diagram to which the bar must conform.
9 Bent bar details are shown on Dwg 2[illegible]
[illegible]



THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

RECORD (AS BUILT) DRAWING

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

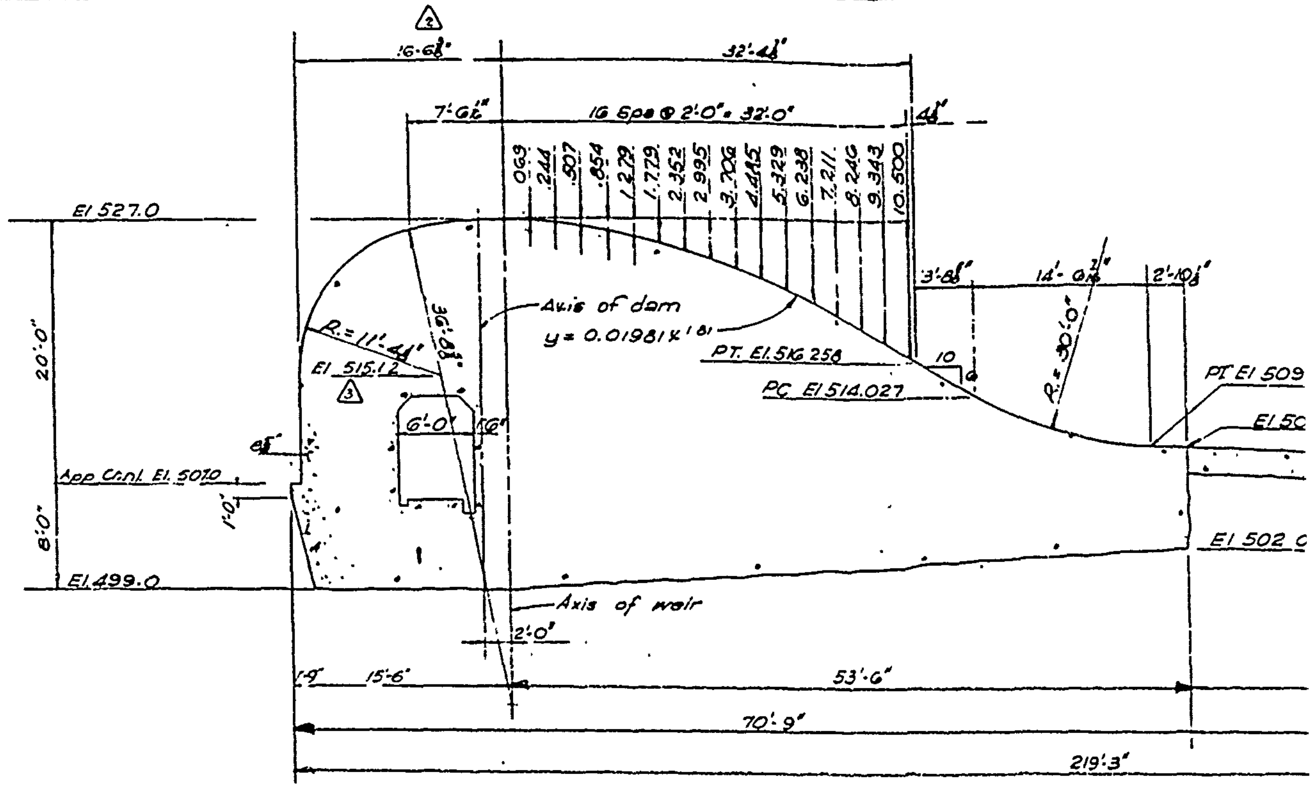
GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY—STRUCTURAL
TYPICAL STRUCTURAL ARRANGEMENT

	BY	CHKD
DESIGNED	DE	MR
DRAWN	CL	DE
TRACED	VM	MR

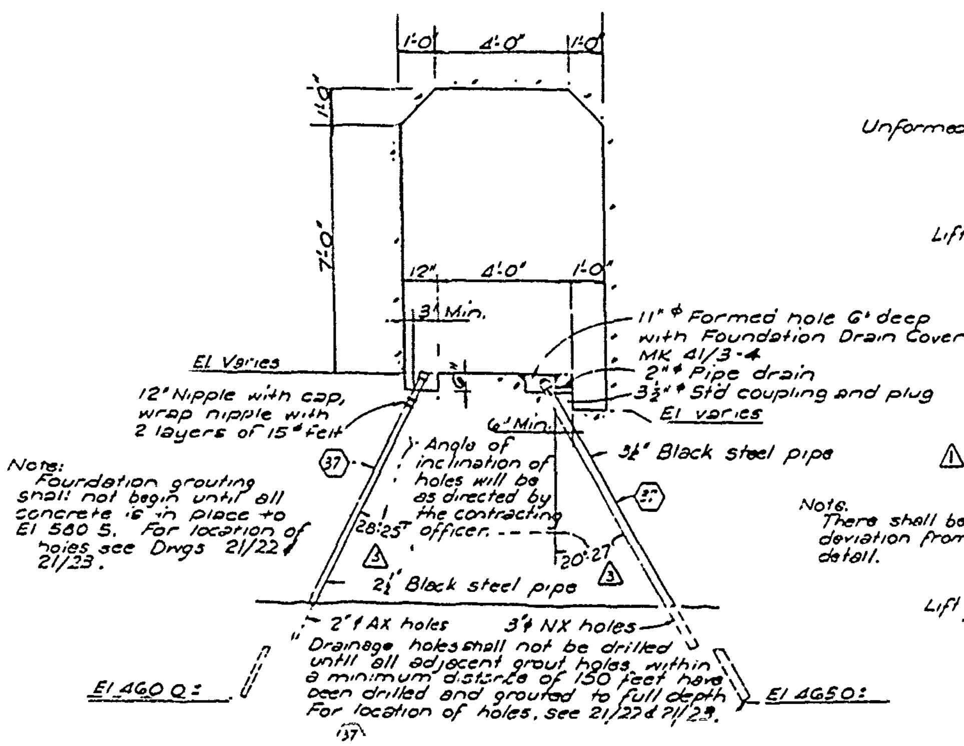
SUBMITTED BY: CHIEF, STRUCTURAL SECT.
RECOMMENDED: CHIEF, DESIGN BRANCH
APPROVED: CHIEF, ENGR. DIV. FOR THE DISTRICT ENGINEER
DATE: AUG 1966
SCALE AS SHOWN
DRAWING NO. ~~1770-67-21/2~~

(2)

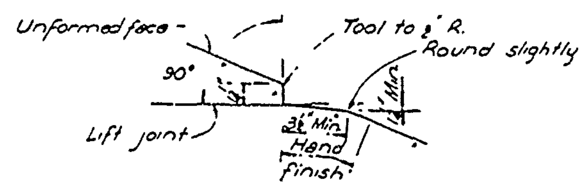
CORPS OF ENGINEERS



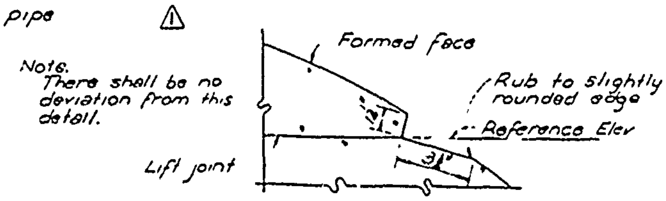
WEIR AND CHUTE PROFILE
SCALE 3/16" = 1'-0"



SECTION A 21/22 A 21/23
(TYPICAL GALLERY CROSS SECTION)
SCALE 1/4" = 1'-0"

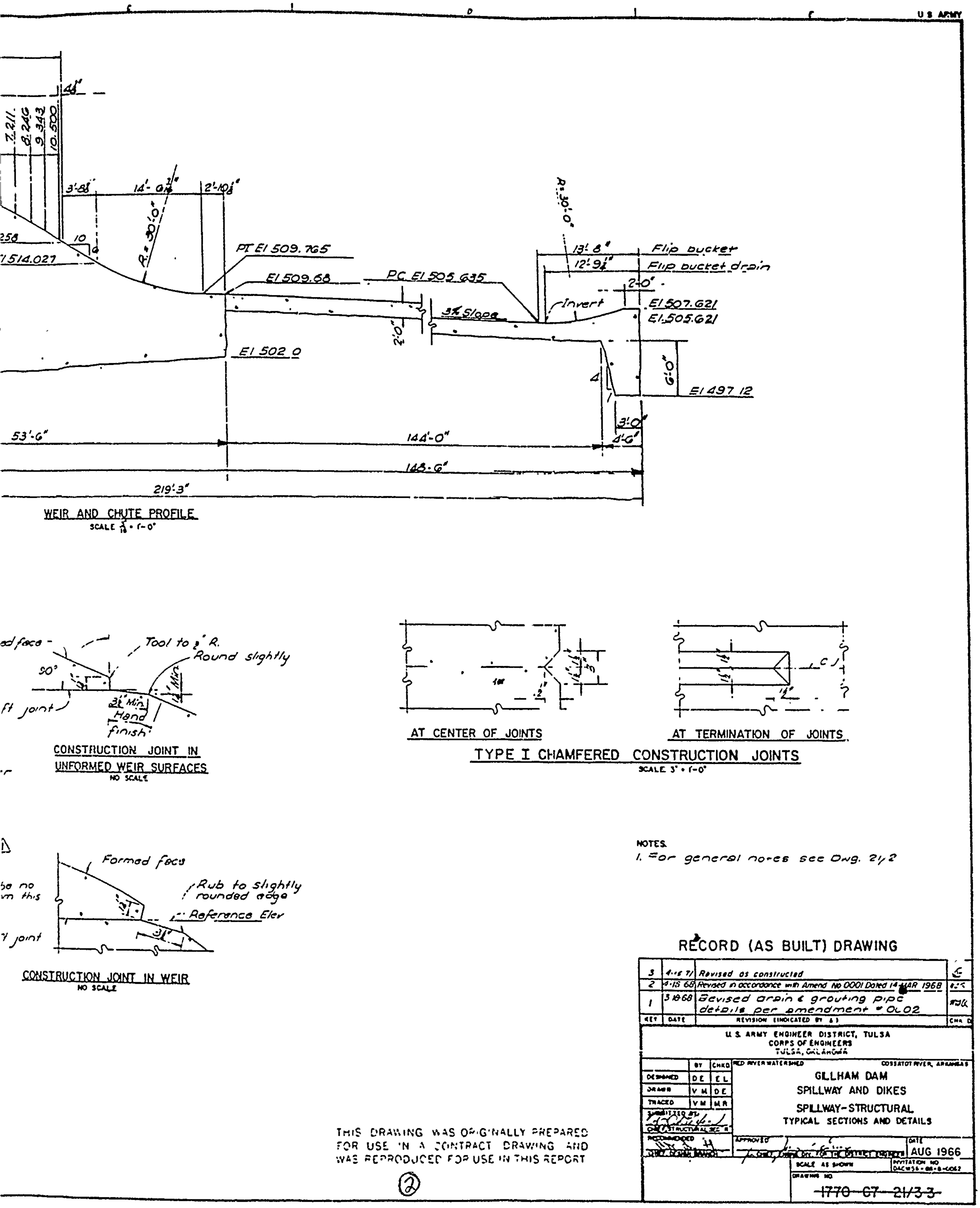


CONSTRUCTION JOINT IN UNFORMED WEIR SURFACES
NO SCALE

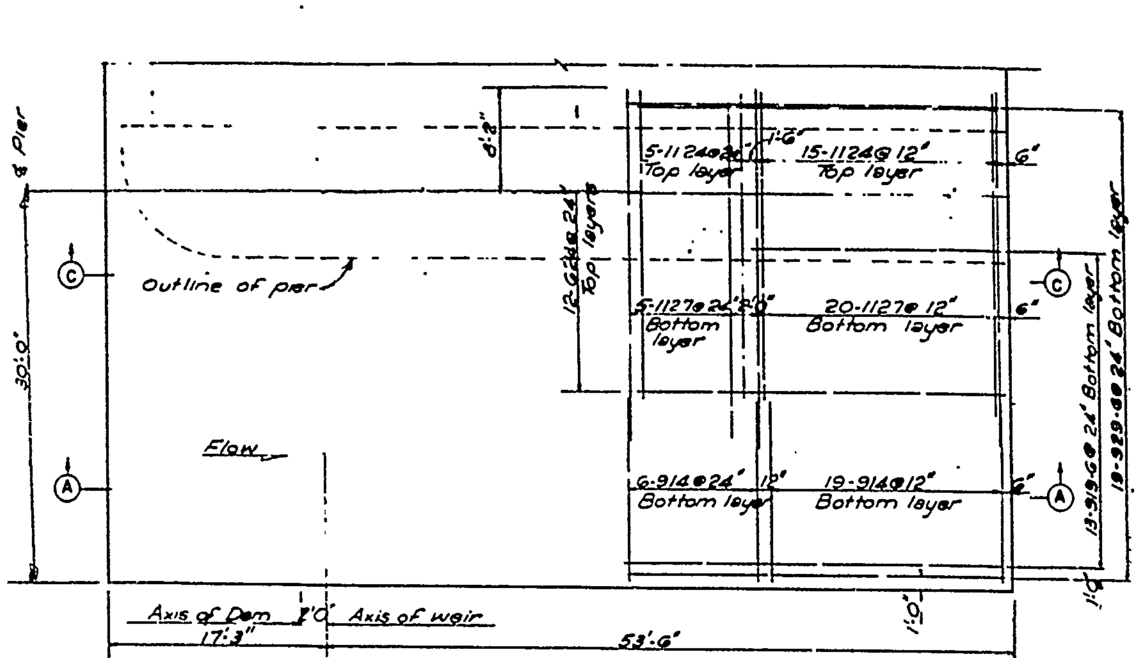


CONSTRUCTION JOINT IN WEIR
NO SCALE

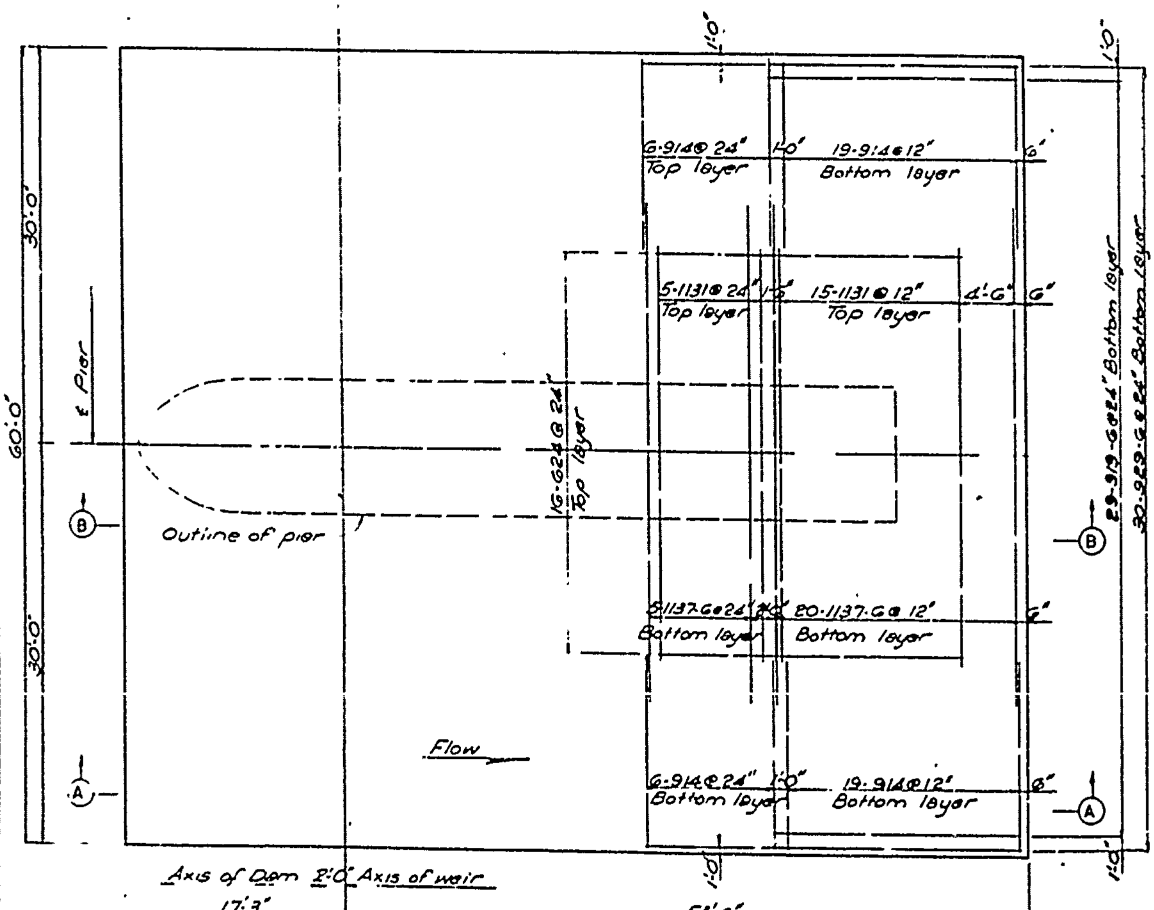
(1)


U S ARMY
PT El 509.765
El 509.68
PC El 505.635
3% Slope
Invert
Flip bucket
Flip bucket drain
El 507.621
El 505.621
El 502.0
El 497.12
144'-0"
148'-6"
219'-3"
53'-6"
WEIR AND CHUTE PROFILE
Tool to ⅛" R.
Round slightly
Hand finish
CONSTRUCTION JOINT IN UNFORMED WEIR SURFACES
NO SCALE
AT CENTER OF JOINTS
AT TERMINATION OF JOINTS
TYPE I CHAMFERED CONSTRUCTION JOINTS
Formed face
Rub to slightly rounded edge
Reference Elev
CONSTRUCTION JOINT IN WEIR
NO SCALE
NOTES.
1. For general notes see Dwg. 21/2
RECORD (AS BUILT) DRAWING
3 4-6-71 Revised as constructed
2 4-15-68 Revised in accordance with Amend No 0001 Dated 14 MAR 1968
1 3-18-68 Revised drain & grouting pipe details per amendment # 0002
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY-STRUCTURAL
TYPICAL SECTIONS AND DETAILS
AUG 1966
SCALE AS SHOWN
1770-67-21/3.3
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

CORPS OF ENGINEERS

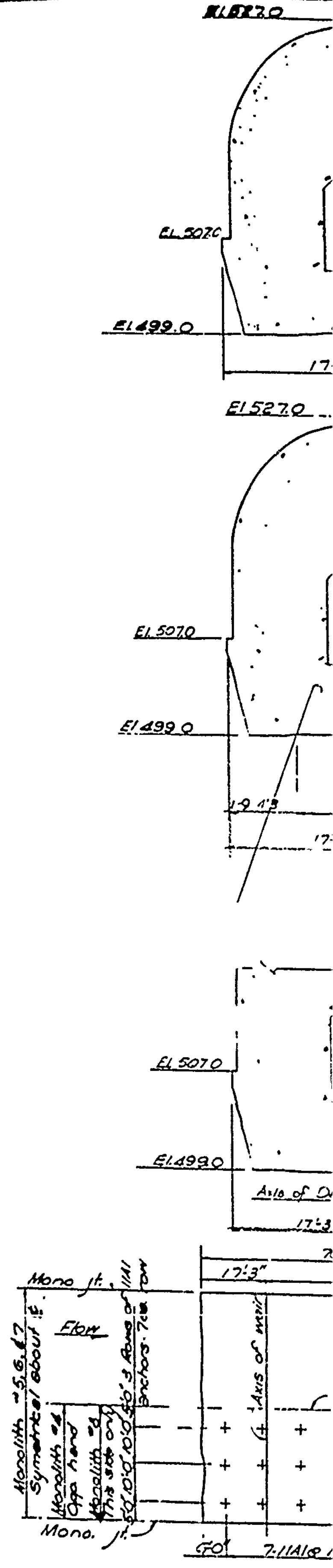


PLAN END PIER MONOLITH
(MONOLITH 8 AS SHOWN)
(MONOLITH 4 OPPOSITE HAND)
SCALE 3/16" = 1'-0"

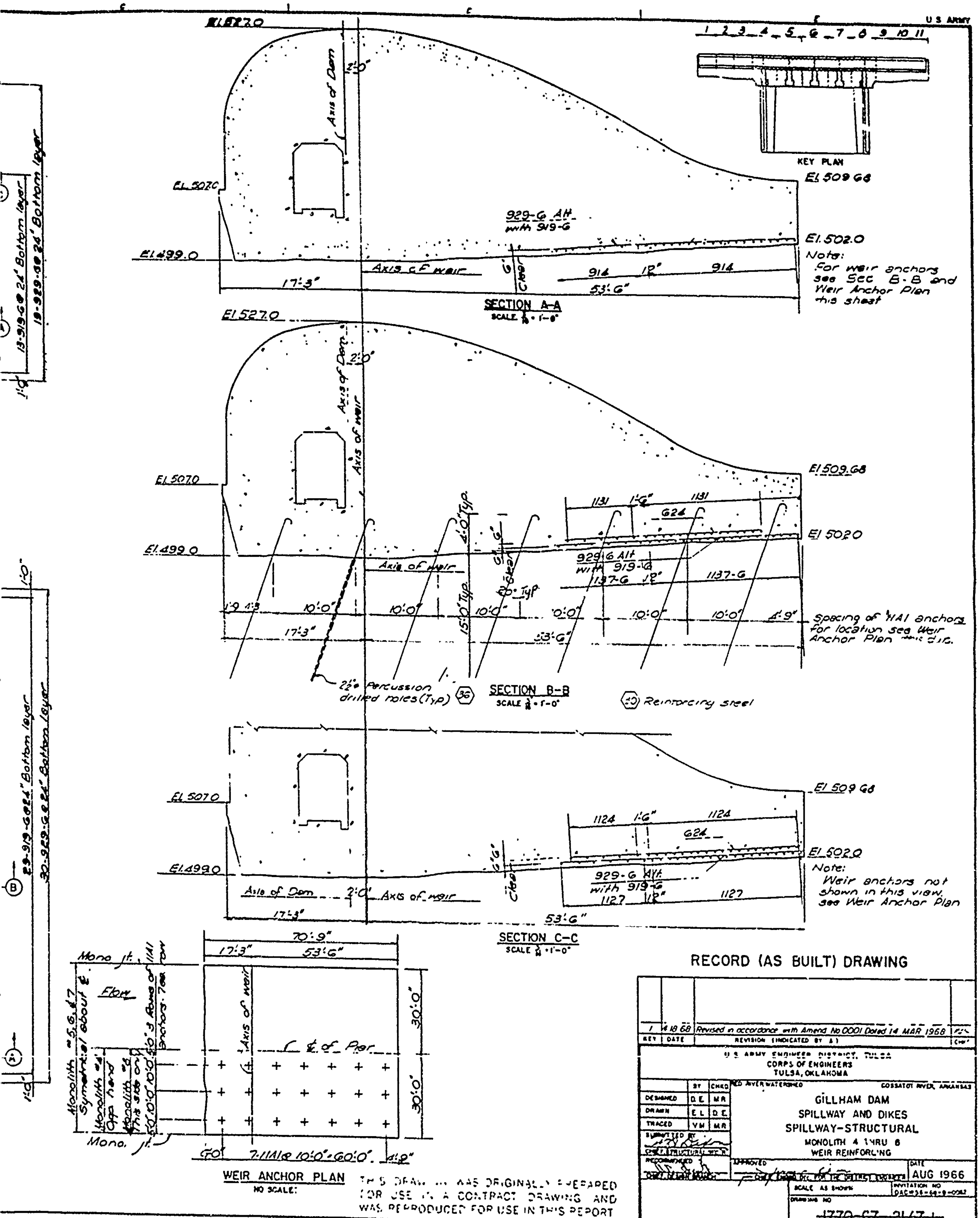


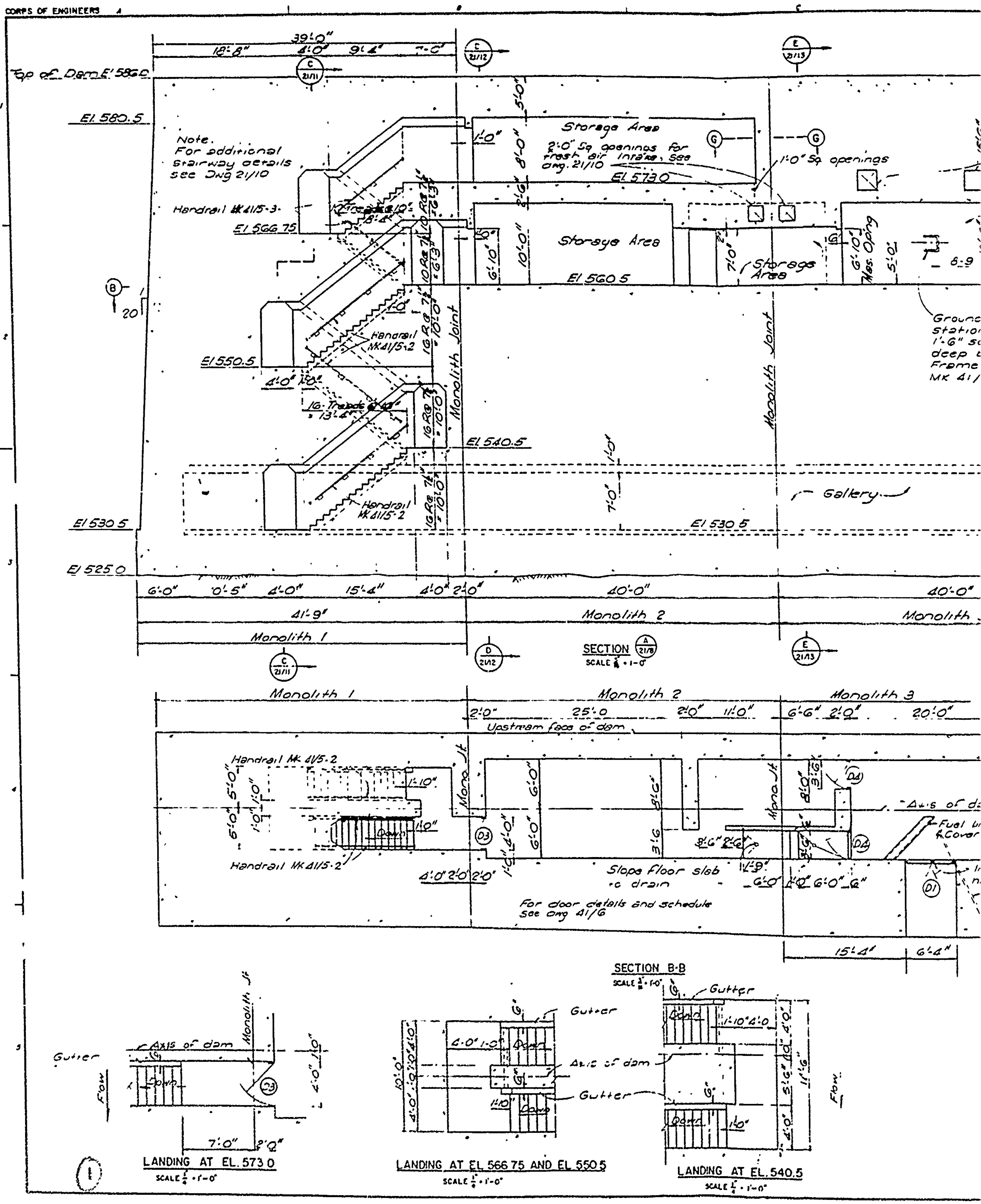
PLAN INTERMEDIATE PIER MONOLITH
(MONOLITHS 5, 6 AND 7)
SCALE 3/16" = 1'-0"

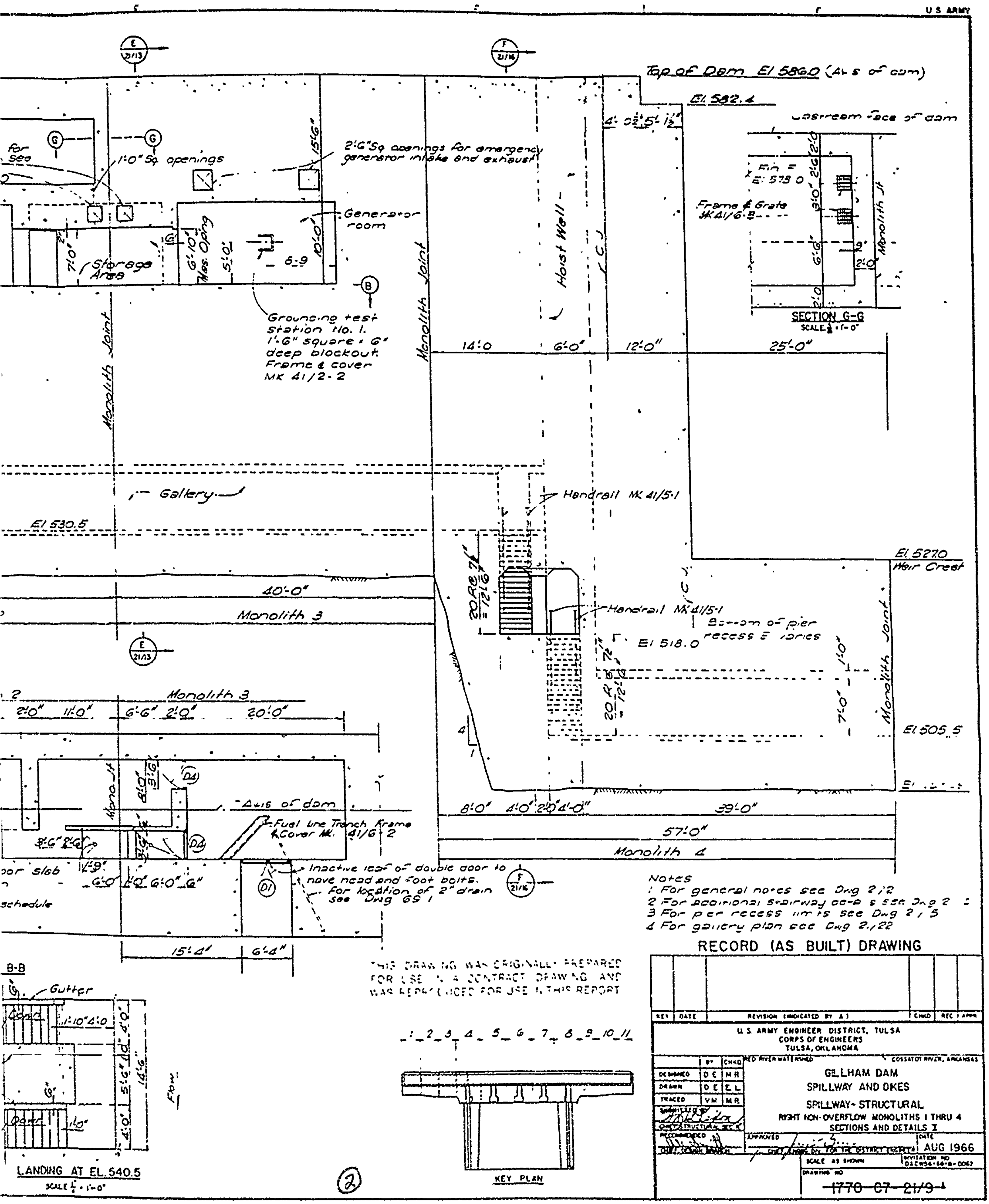
(1)



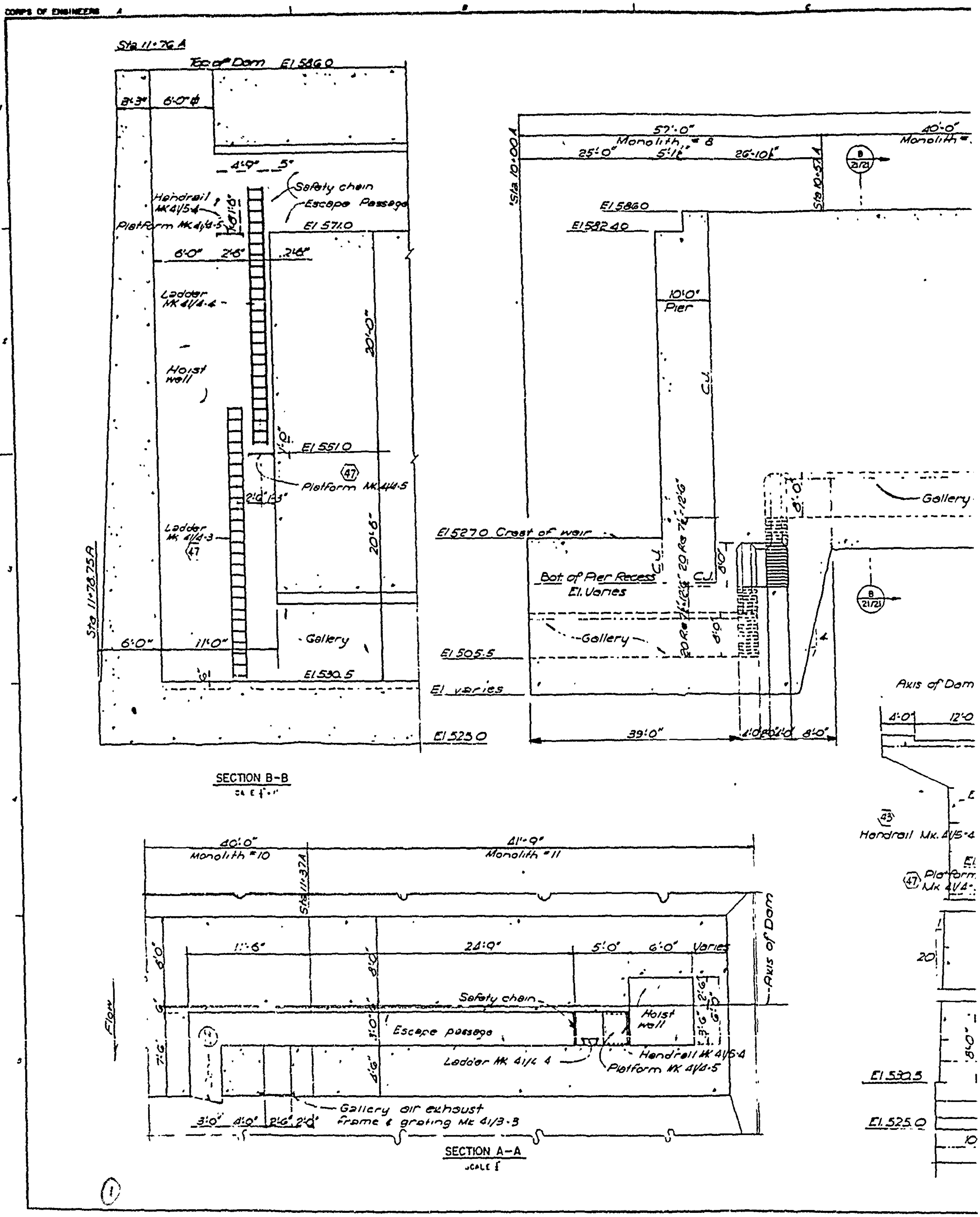
WEIR ANCHOR
NO SCALE


U S ARMY
KEY PLAN
SECTION A-A
SCALE 1/8" = 1'-0"
El 509.68
El 502.0
El 507.0
El 499.0
929-6 Alt with 919-6
Axis of Dam
Axis of weir
Note: For weir anchors see Sec B-B and Weir Anchor Plan this sheet
17'-3"
53'-6"
El 527.0
SECTION B-B
SCALE 1/8" = 1'-0"
2 1/2" Ø Percussion drilled holes (Typ)
Reinforcing steel
Spacing of 11A1 anchors for location see Weir Anchor Plan this dwg.
SECTION C-C
SCALE 1/8" = 1'-0"
Note: Weir anchors not shown in this view, see Weir Anchor Plan
WEIR ANCHOR PLAN
NO SCALE
Mono. Jt.
Flow
C of Pier
70'-9"
30'-0"
RECORD (AS BUILT) DRAWING
Revised in accordance with Amend No 0001 Dated 14 MAR 1968
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY-STRUCTURAL
MONOLITH 4 THRU 6
WEIR REINFORCING
AUG 1966
1770-67-21/7.1
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT


CORPS OF ENGINEERS
Top of Dam El 586.0
El 580.5
Note.
For additional stairway details see Dwg 21/10
Handrail MK 41/5-3
El 566.75
El 550.5
El 540.5
El 530.5
El 525.0
Storage Area
2'-0" Sq openings for fresh air intake, see dwg. 21/10
1'-0" Sq openings
El 573.0
Storage Area
El 560.5
Storage Area
Handrail MK 41/5-2
Monolith Joint
Monolith Joint
Gallery
Monolith 1
Monolith 2
Monolith 3
SECTION A
SCALE ⅛" = 1'-0"
Upstream face of dam
Axis of dam
Slope floor slab to drain
For door details and schedule see dwg 41/6
SECTION B-B
SCALE ⅛" = 1'-0"
Gutter
Flow
LANDING AT EL. 573.0
SCALE ¼" = 1'-0"
LANDING AT EL 566.75 AND EL 550.5
SCALE ¼" = 1'-0"
LANDING AT EL. 540.5
SCALE ¼" = 1'-0"

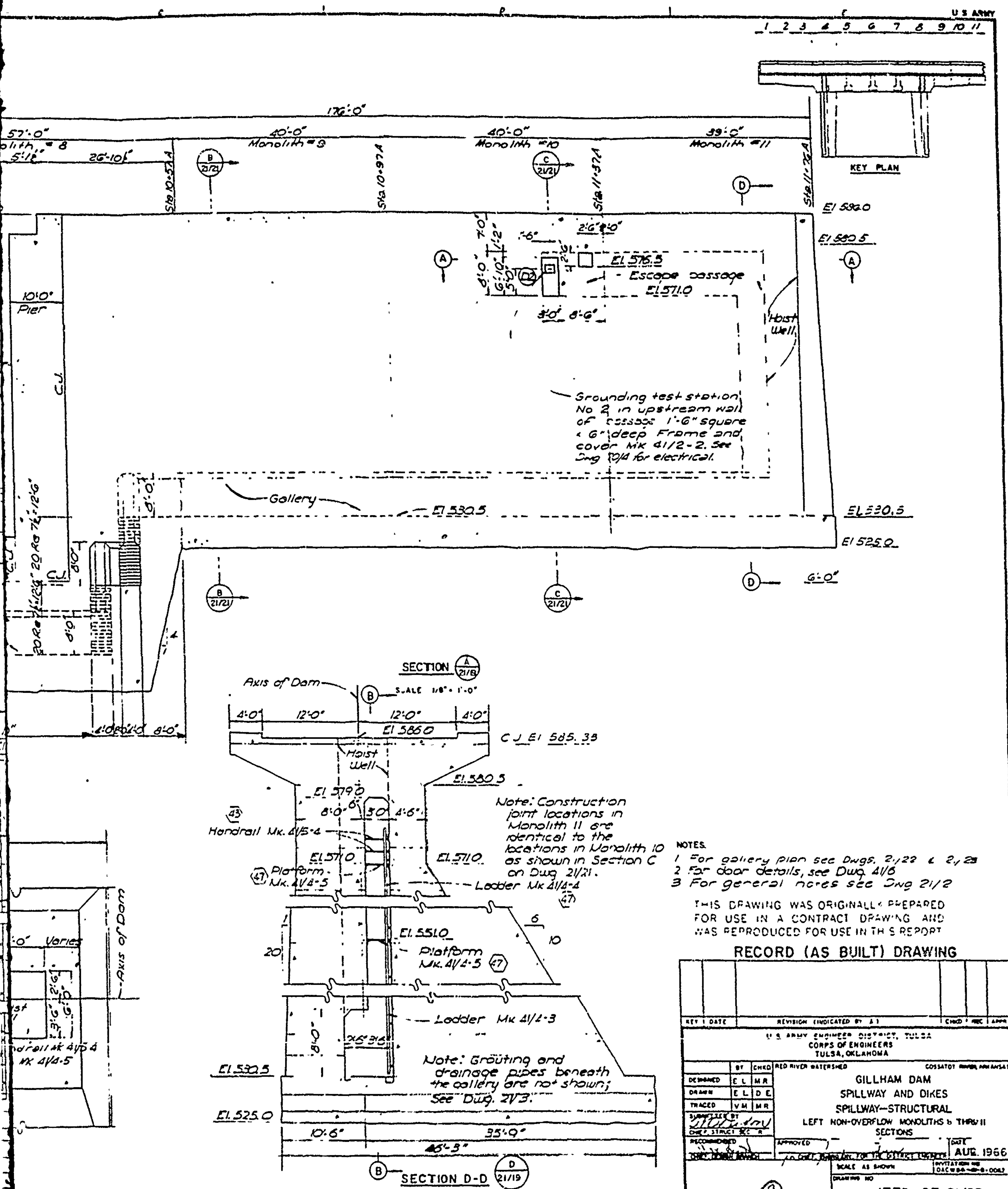


EMBANKMENT CRITERIA AND PERFORMANCE REPORT - PLATE NO. 33

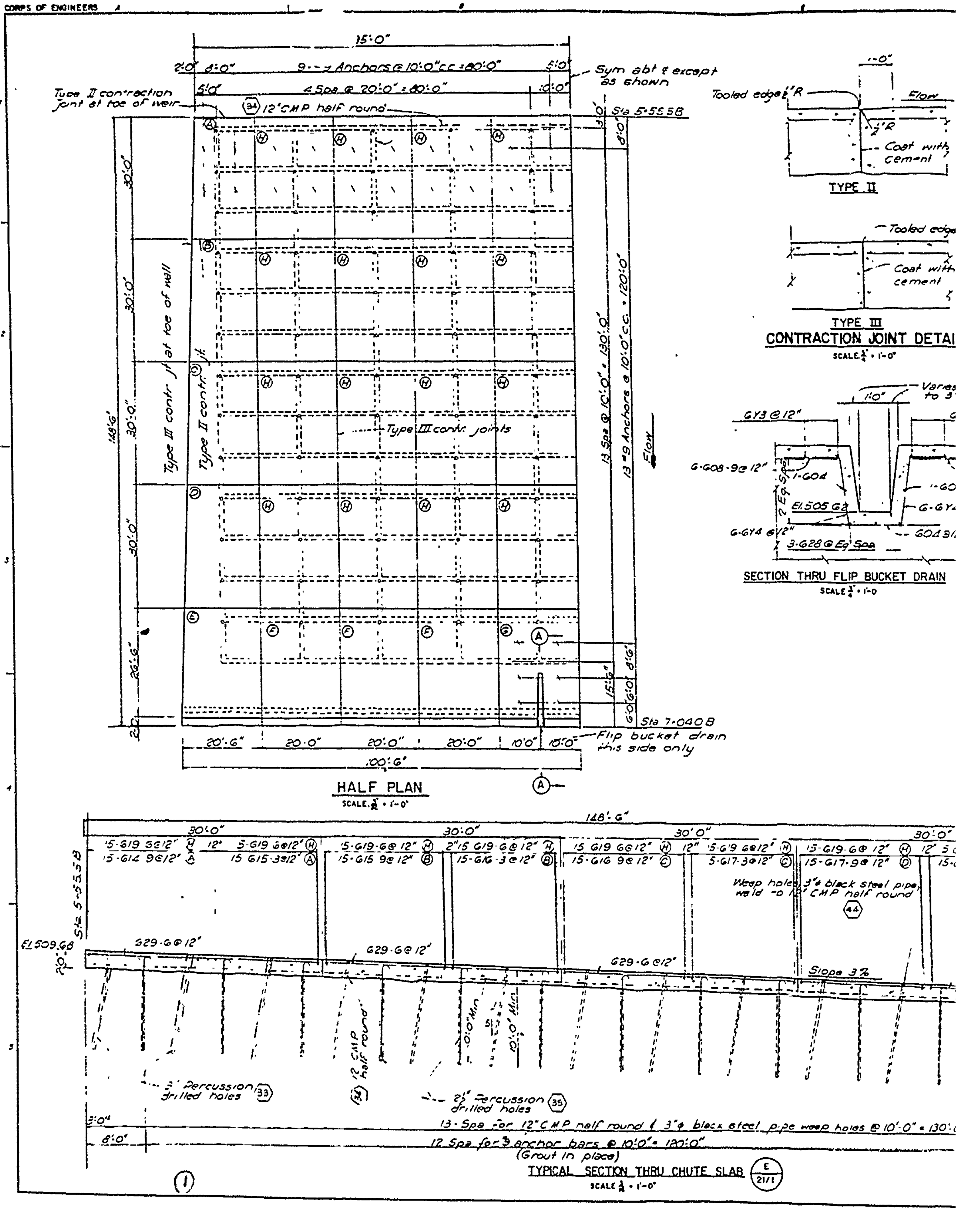


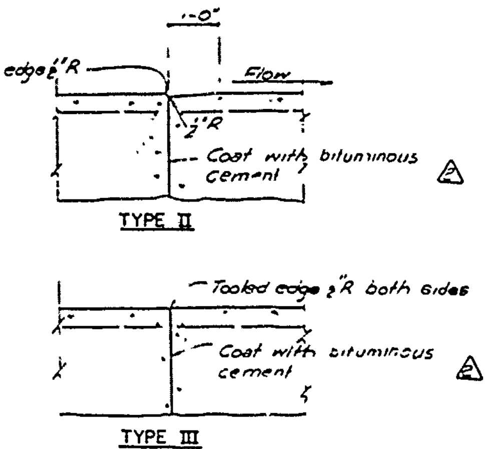
SECTION B-B

SECTION A-A



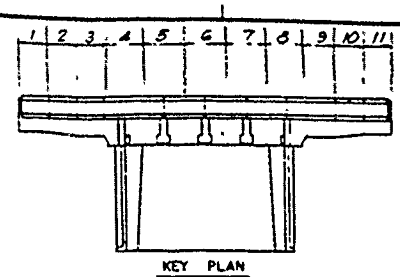
U S ARMY
KEY PLAN
176'-0"
57'-0"
40'-0" Monolith #9
40'-0" Monolith #10
39'-0" Monolith #11
El 590.0
El 580.5
El 576.5
Escape passage El 571.0
Hoist Well
10'-0" Pier
Grounding test station No 2 in upstream wall of passage 1'-6" square & 6" deep Frame and cover Mk 41/2-2. See Dwg 70/4 for electrical.
Gallery
El 530.5
El 525.0
SECTION A 21/18
SCALE 1/8" = 1'-0"
Axis of Dam
El 586.0
CJ El 585.35
Hoist Well
El 580.5
El 579.0
Handrail Mk. 41/5-4
El 571.0
Platform Mk. 41/4-5
Ladder Mk 41/4-4
Note: Construction joint locations in Monolith 11 are identical to the locations in Monolith 10 as shown in Section C on Dwg 21/21.
El 551.0
Platform Mk. 41/4-5
Ladder Mk 41/4-3
Note: Grouting and drainage pipes beneath the gallery are not shown; See Dwg. 21/3.
El 530.5
El 525.0
10'-6"
35'-9"
46'-3"
SECTION D-D 21/19
SCALE 3/16" = 1'-0"
NOTES.
1 For gallery plan see Dwgs. 21/22 & 21/23
2 For door details, see Dwg. 41/6
3 For general notes see Dwg 21/2
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
RECORD (AS BUILT) DRAWING
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY—STRUCTURAL
LEFT NON-OVERFLOW MONOLITHS 8 THRU 11
SECTIONS
AUG. 1966
1770-67-21/20

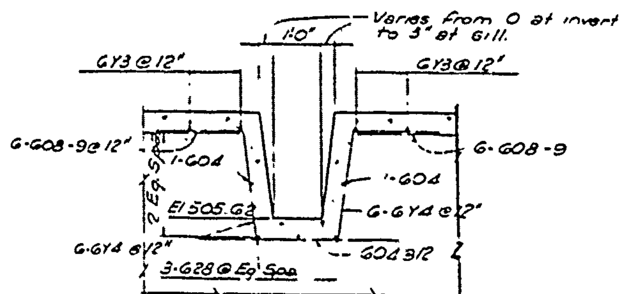

CORPS OF ENGINEERS
35'-0"
2'-0" 8'-0"
9 - Anchors @ 10'-0" cc = 80'-0"
5'-0"
Sym abt ℄ except as shown
Type II contraction joint at toe of weir
5'-0"
4 Spa @ 20'-0" = 80'-0"
10'-0"
12" CMP half round
Sta 5+55.58
148'-6"
30'-0"
30'-0"
30'-0"
30'-0"
26'-6"
Type III contr jt at toe of wall
Type II contr jt
Type III contr joints
13 Spa @ 10'-0" = 130'-0"
13 #9 Anchors @ 10'-0" c.c. = 120'-0"
Flow
8'-0"
8'-6"
15'-6"
6'-0"
Sta 7+04.08
Flip bucket drain this side only
20'-6"
20'-0"
20'-0"
20'-0"
10'-0"
10'-0"
100'-6"
HALF PLAN
1'-0"
Tooled edge
Flow
Coat with cement
TYPE II
Tooled edge
Coat with cement
TYPE III
CONTRACTION JOINT DETAIL
SCALE 3/4" = 1'-0"
1'-0"
G-Y3 @ 12"
G-G08-9 @ 12"
1-G04
El. 505.62
G-GY4 @ 12"
3-G28 @ Eq Spa
SECTION THRU FLIP BUCKET DRAIN
SCALE 3/4" = 1'-0
148'-6"
30'-0"
30'-0"
30'-0"
30'-0"
15-G19 3 @ 12"
15-G14 9 @ 12"
5-G19 6 @ 12"
15 G15-3 @ 12"
15-G19-6 @ 12"
15-G15 9 @ 12"
15 G19-6 @ 12"
15-G16-3 @ 12"
15 G19 6 @ 12"
15-G16 9 @ 12"
5-G19 6 @ 12"
5-G17-3 @ 12"
15-G19-6 @ 12"
15-G17-9 @ 12"
Weep holes 3"ø black steel pipe, weld to 12" CMP half round
Sta 5+55.58
El. 509.68
G29-6 @ 12"
G29-6 @ 12"
G29-6 @ 12"
Slope 3%
12" CMP half round
10'-0" Min
10'-0" Min
Percussion drilled holes
2½" Percussion drilled holes
3'-0"
8'-0"
13 - Spa for 12" CMP half round & 3"ø black steel pipe weep holes @ 10'-0" = 130'-0"
12 Spa for anchor bars @ 10'-0" = 120'-0"
(Grout in place)
TYPICAL SECTION THRU CHUTE SLAB
SCALE 1/4" = 1'-0"
E
21/1



CONTRACTION JOINT DETAILS

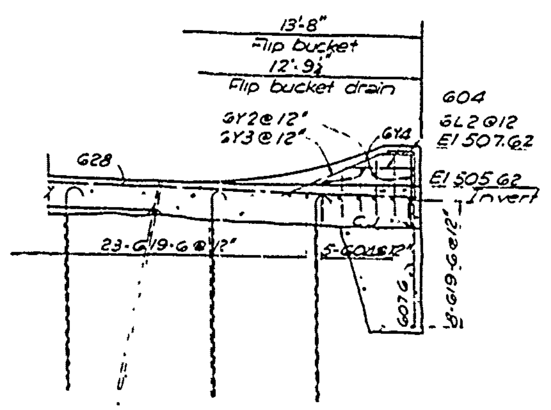
SCALE ¾" = 1'-0"





SECTION THRU FLIP BUCKET DRAIN

SCALE ¾" = 1'-0



SECTION A-A

SCALE ½" = 1'-0"

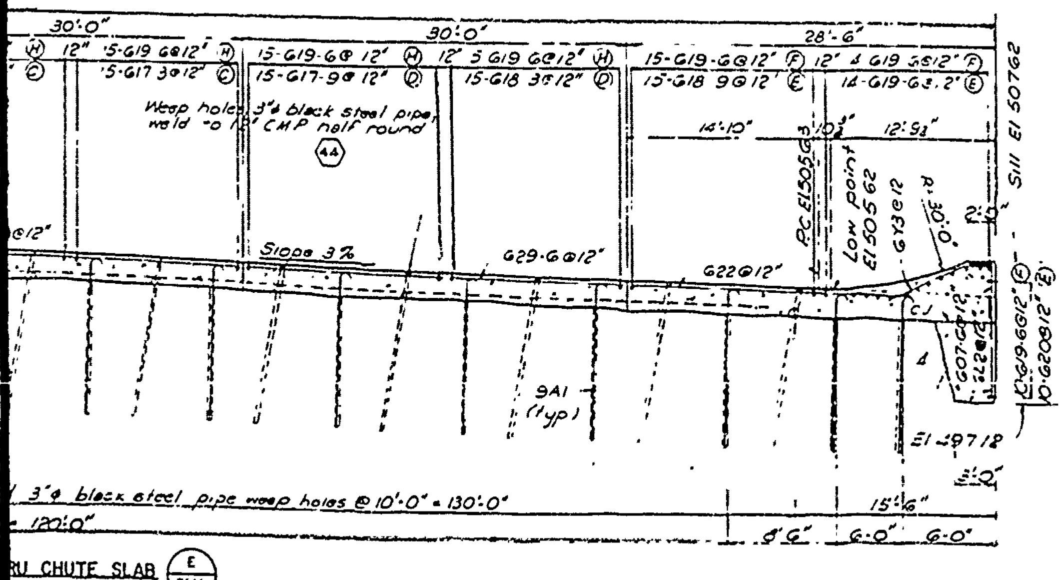
SLABS	NO OF SLABS	STRAIGHT BAR MARK	LGTH	NO REQ	BENT BAR MARK	LGTH	NO REQ
A	2	G14-9	14-9	30	9A1	11-3	12
		G15-3	15-3	30			
		G29-6	29-6	30			
B	2	G15-9	15-9	30	9A1	11-3	12
		G16-3	16-3	30			
		G29-6	29-6	32			
C	2	G16-9	16-9	30	9A1	11-3	12
		G17-3	17-3	30			
		G29-6	29-6	34			
D	2	G17-9	17-9	30	9A1	11-3	12
		G18-3	18-3	30			
		G29-6	29-6	36			
E	2	G07-6	7-6	38	9A1	11-3	12
		G18-9	18-9	30	GL2	5-9	38
		G19-6	19-6	28	GY3	8-6	38
		G20	20-0	20			
		G22	22-0	38			
F	7	G07-6	7-6	140	9A1	11-3	42
		G19-6	19-6	273	GL2	5-9	140
		G22	22-0	140	GY3	8-6	140
G	1	G04	4-0	1	9A1	11-3	6
		G07-6	7-6	18	GL2	5-9	18
		G08-6	8-6	12	GY3	8-6	18
		G19-6	19-6	31	GY4	3-4	12
		G22	22-0	18	GY2	4-0	6
		G28	23-0	3			
H	32	G19-6	19-6	960	9A1	11-3	192
		G29-6	29-6	720			

Note For details of bent bars listed in this table see Dwg 21/3?

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

NOTES

1 Concrete in the chute slab shall be placed in an alternating "checkerboard" pattern so that adjacent slabs will not be placed in sequence
2 Concrete in chute slab shall be paid for under bid item (28)
3 Reinforcement in chute slab shall be paid for under bid item (30)
4 For general notes see dwg 21/2
5 Alignment of the half rounds may vary to suit foundation conditions.



RECORD (AS BUILT) DRAWING

KEY	DATE	REVISION
2	4-15-71	Revised as constructed
1	4-18-68	Revised in accordance with Amend No 0001 Dated 14 MAR 1968

U. S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

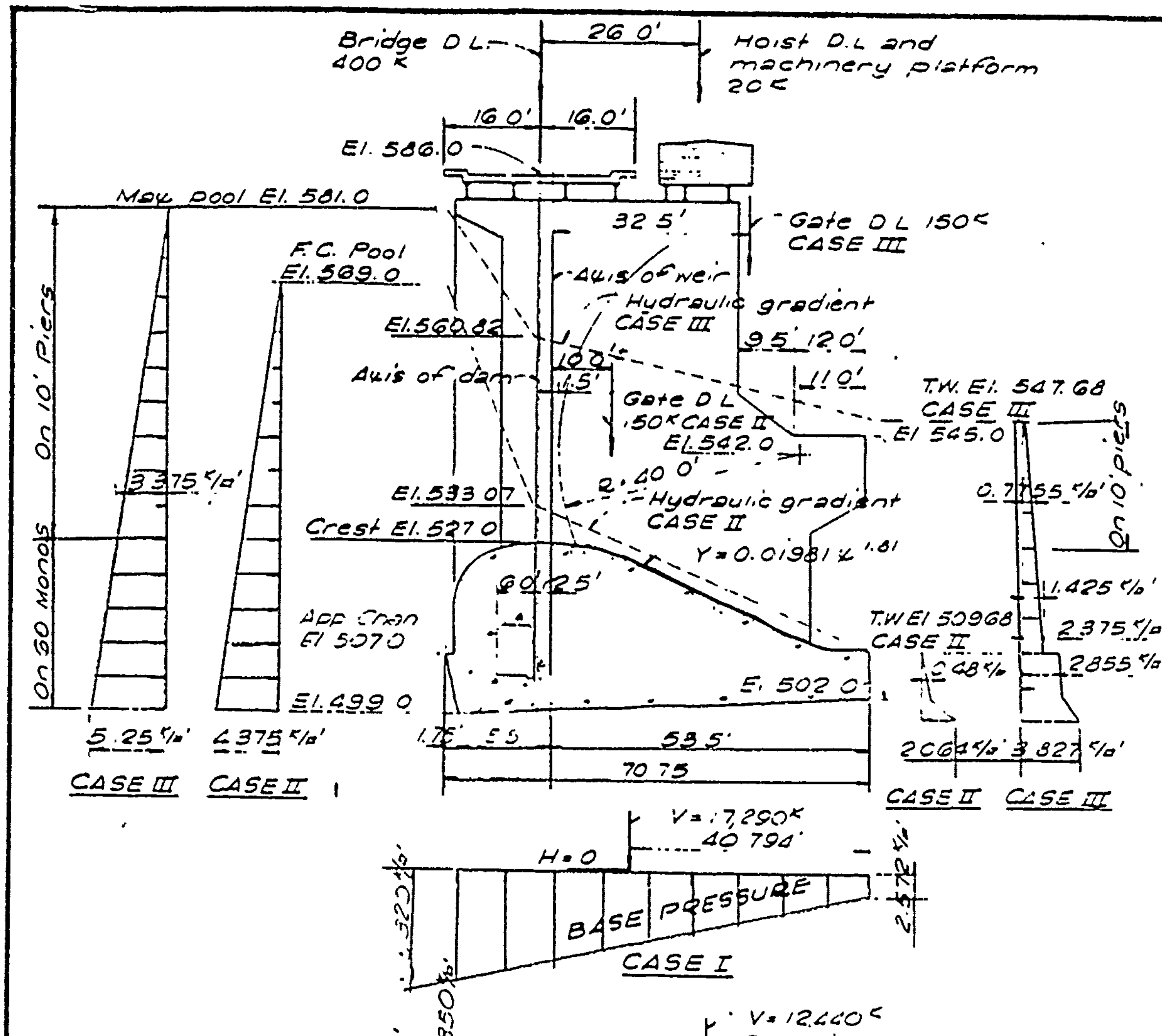
RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

GILLHAM DAM
SPILLWAY AND DIKES
SPILLWAY—STRUCTURAL
CHUTE SLAB

DESIGNED WM / DE — DRAWN WM / EL — TRACED VM / MR

DATE AUG 1966 — SCALE AS SHOWN

~~1770 67 21/30.2~~



RESISTANCE TO SLIDING

Rs = Pv tan φ + CLB

φ = 25°   φ* = 45°

C = 8 K/a'   C = 14 K/a'*

L = 66.75'   B = 60'

(*) Crossbed shear

DESIGN LOADS

Concrete Wc = 150#/C.F.

Water W = 62.5#/C.F., P = 62.5#/S.F.

LOADING CONDITIONS

CASE I Reservoir empty, gate resting on weir

CASE II Upper pool at El 569.0, Tailwater at El 509.68. Gate resting on weir. Uplift assumed fully effective at heel and toe. Uplift at the gallery assumed to be one-half the difference between the undrained hydraulic gradient and the tailwater, added to the tailwater.

CASE III Upper pool at El 58'0. Tailwater at El. 547.68. Gate full open. Uplift assumed as in CASE II

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A DESIGN MEMORANDUM AND WAS REPRODUCED FOR USE IN THIS REPORT

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

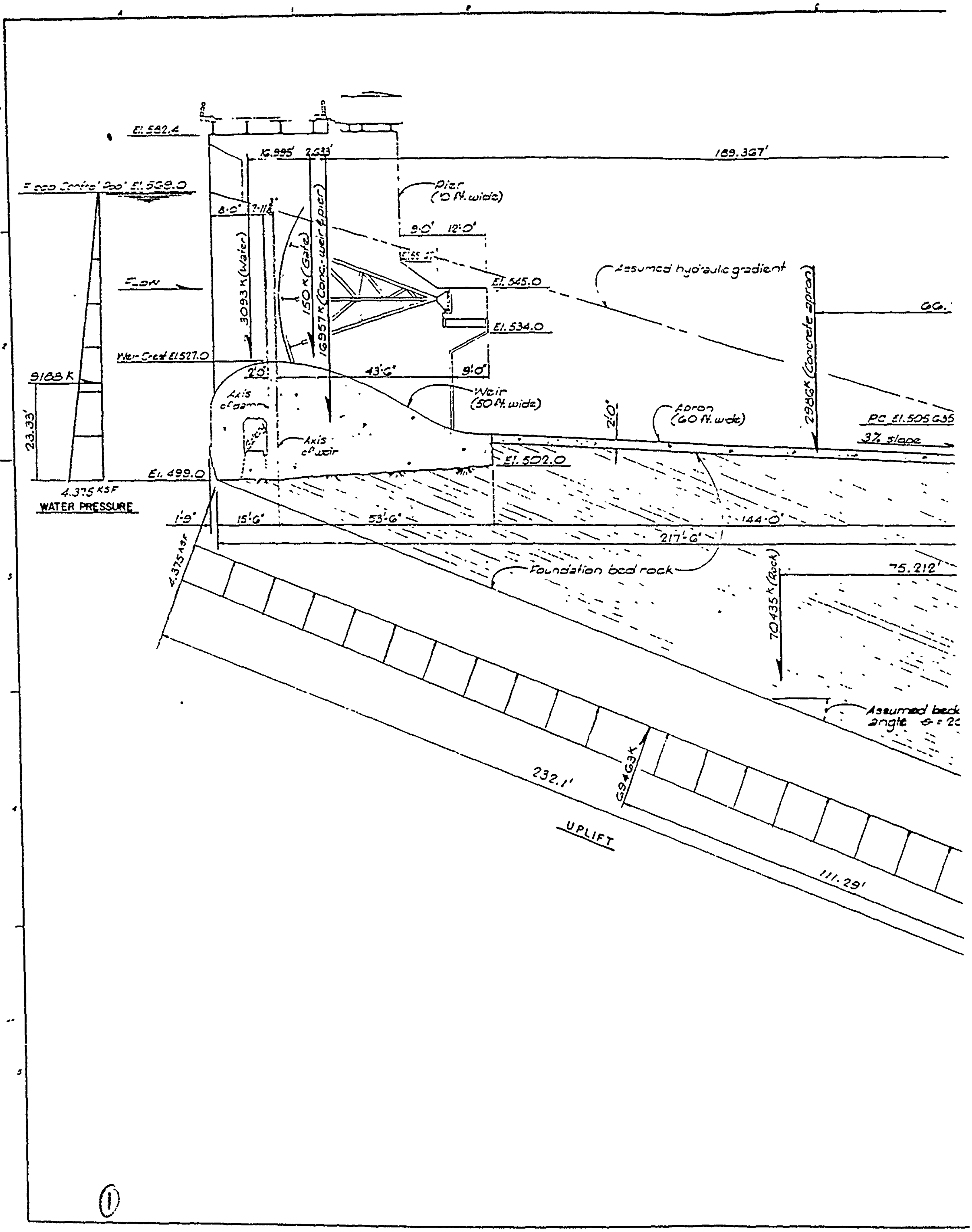
GILLHAM DAM AND RESERVOIR
SPILLWAY AND DIKES
SPILLWAY
OVERFLOW STABILITY ANALYSIS

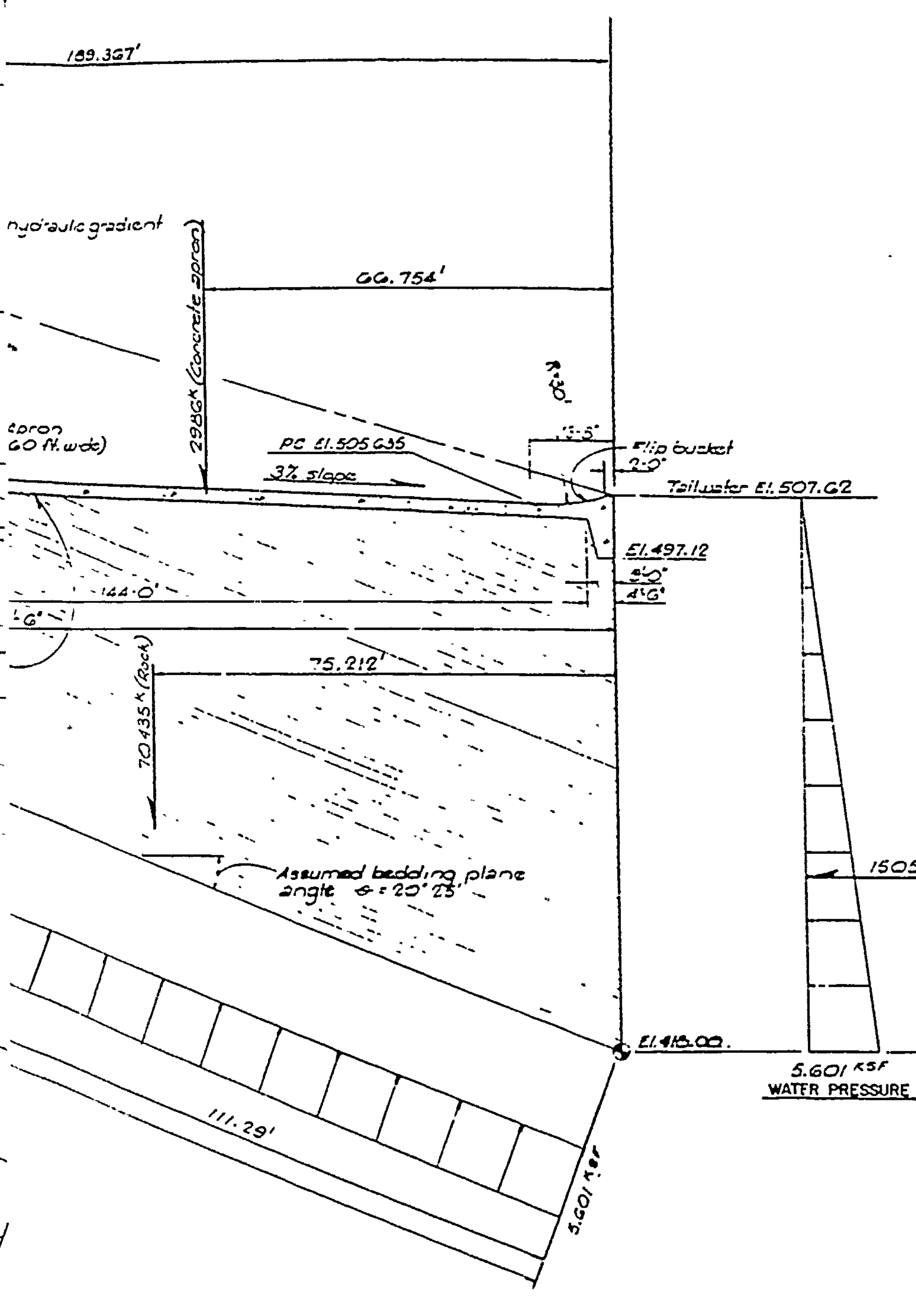
DATE AUG. 1969

SCALE AS SHOWN

DRAWING NO. 1770-DM12-21/4

COL. CORPS OF ENGINEERS DISTRICT ENGINEER


El. 582.4
16.995'
2.633'
189.367'
Flood Control Pool El. 569.0
Pier
(10 ft. wide)
8.0'
9.0'
12.0'
Assumed hydraulic gradient
El. 545.0
FLOW
3093 K (Water)
150 K (Gate)
16957 K (Conc.-weir & pier)
El. 534.0
29866K (Concrete apron)
Weir Crest El. 527.0
9188 K
2'0"
43'6"
9'0"
Weir
(50 ft. wide)
Axis of dam
Axis of weir
2'0"
Apron
(60 ft. wide)
PC El. 505.635
3% slope
23.33'
El. 502.0
El. 499.0
4.375 KSF
WATER PRESSURE
1'9"
15'6"
53'6"
144'0"
217'6"
4.375 KSF
Foundation bed rock
75.212'
70435 K (Rock)
Assumed bedding angle
232.1'
69463K
UPLIFT
111.29'
(1)



DESIGN DATA

Concrete: W_c = 150 pcf

Water: W_w = 62.5 pcf, P_w = 62.5 p.s.f.

Rock: W_R = 120 pcf

RESISTANCE TO SLIDING

$Rs = \Sigma V \tan\phi + C L B$ $\quad Rs^* = \Sigma V \tan\phi + C L^* B$

$\phi = 0$, $C = 8^{KSF}$, $B = 60$ ft.

Rs = Total resistance to sliding due to 100% of bedding plane in contact

Rs* = Total resistance to sliding due to 55.3% of bedding plane in contact

L = 232.1 ft. (Inclined length from heel of weir along the bedding plane to the downstream end of the rock contact)

L* = 128.4 ft. (55.3% of base with positive bearing on the bedding plane considering overturning stability with rotation at the downstream end of the rock contact)

LOADING CONDITION

Upper pool at flood control pool @ El. 589.0. Tailwater at end sill of apron @ El. 507.62. Gate resting on weir. Assumed a deep scour hole about 90 ft. immediately downstream from the apron and uplift to be fully effective at heel and toe with a straight line gradient.

RESULT OF SPILLWAY DESIGN STUDY					
ΣV (KIPS)	ΣH (KIPS)	Rs (KIPS)	Rs* (KIPS)	Rs/ΣH	Rs*/ΣH
28,528	18,371	104,400	61,632	5.68	3.35

ΣV = Total vertical force

ΣH = Total horizontal force

REV	DATE	REVISION AND BY	

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

DESIGNED BY: C.Y. Chang
DRAWN BY: [illegible]
CHECKED BY: L.S. M.R.
SUBMITTED
CHIEF, STR SEC

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

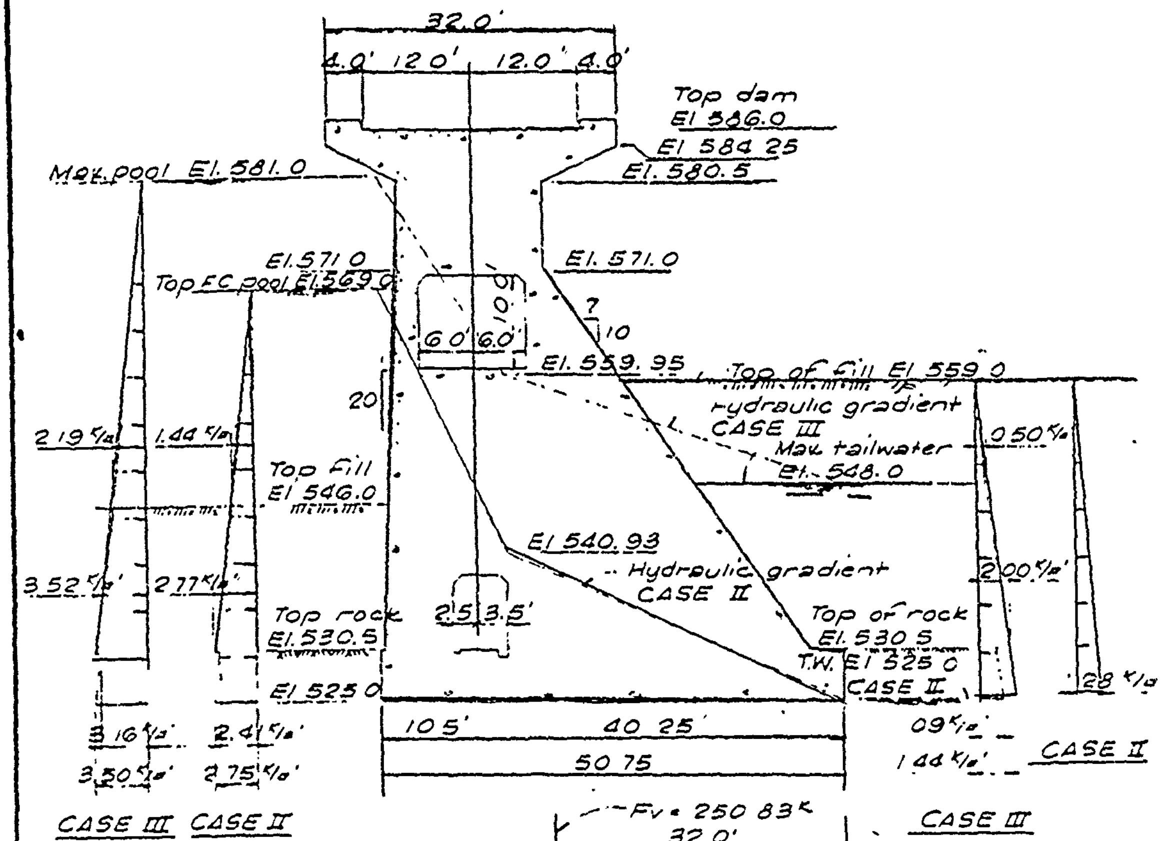
GILLHAM LAKE

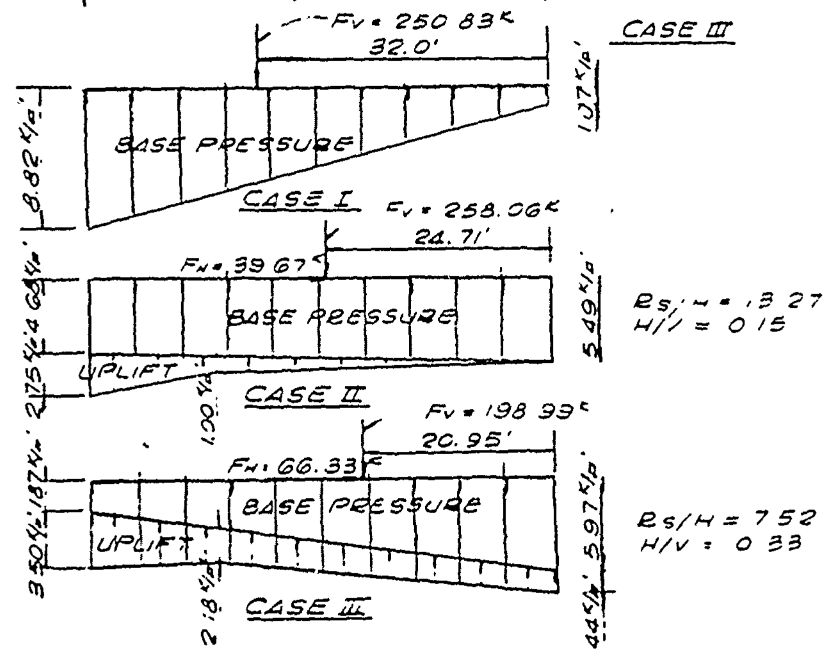
SPILLWAY DESIGN STUDY

INVITATION NO. DACW56- -B-00
SCALE AS SHOWN
DRAWING NUMBER
DATE

(2)

U.S. ARMY





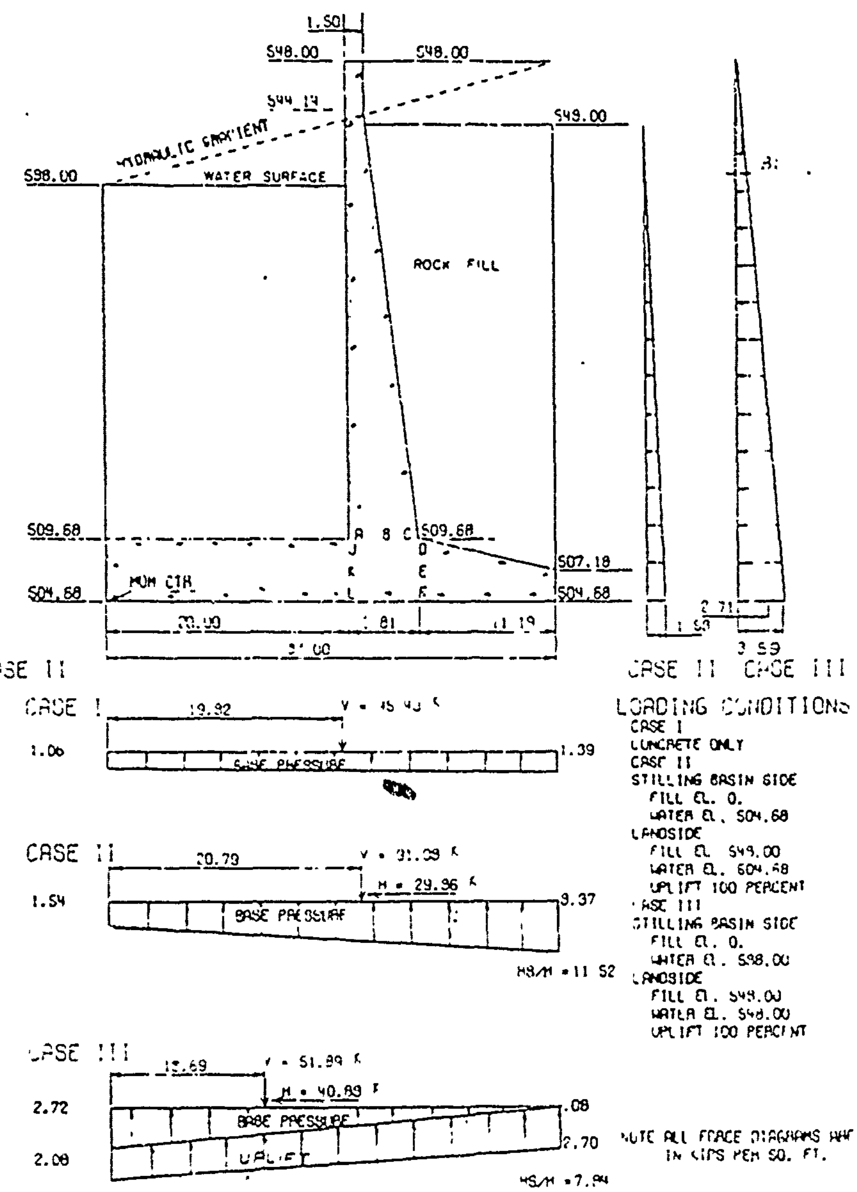
DESIGN LOADS
Concrete:
Wc = 150#/C.F.
Water
W = 62.5#/C.F.
P = 62.5#/S.F.
Moist soil
W = 120#/C.F.
P = 45#/S.F.
Sat soil
W = 125#/C.F.
P = 86#/S.F.

RESISTANCE TO SLIDING
Rs = Fv tan φ + CLB
φ = 25°
C = 8 K/a'
L = 50.75'

LOADING CONDITIONS
CASE I Concrete only
CASE II Pool at El 569.0 Tailwater at El. 525.0 Uplift assumed fully effective at heel and toe uplift at the gallery assumed to be one-half the difference between the undrained hydraulic gradient and the tailwater added to the tailwater
CASE III Pool at El. 581.0 Tailwater at El. 548.0 Uplift assumed as in CASE I

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A DESIGN MEMORANDUM AND WAS REPRODUCED FOR USE IN THIS REPORT

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS
GILLHAM DAM AND RESERVOIR
SPILLWAY AND DIKES
SPILLWAY
NON-OVERFLOW STABILITY ANALYSIS
DESIGNED / DRAWN / TRACED / SUBMITTED / RECOMMENDED / APPROVED
CHIEF, DESIGN BRANCH — CHIEF, ENGR. DIV.
DATE AUG. 1969
SCALE AS SHOWN
COL., CORPS OF ENGINEERS, DISTRICT ENGINEER
DRAWING NO. ~~1770-DM12-21/5~~



DESIGN LOADS
CONCRETE
WC = 150 LB/CF
FILL
WM = 100.0 LB/CF
PM = 40.0 LB/SF
WS = 120.0 LB/CF
PS = 85.5 LB/SF
WATER
W = 62.5 LB/CF
P = 62.5 LB/CF

RESISTANCE TO SLIDING
HS = PV TAN O + CL
O = 25 DEGREES
C = 80 KSF
L = 37 FT.

LOADING CONDITIONS
CASE I
CONCRETE ONLY
CASE II
STILLING BASIN SIDE
FILL EL. 0.
WATER EL. 504.68
LANDSIDE
FILL EL. 549.00
WATER EL. 504.68
UPLIFT 100 PERCENT
CASE III
STILLING BASIN SIDE
FILL EL. 0.
WATER EL. 538.00
LANDSIDE
FILL EL. 549.00
WATER EL. 548.00
UPLIFT 100 PERCENT

NOTE ALL FORCE DIAGRAMS ARE IN KIPS PER SQ. FT.

TABLE OF STRESSES (PSI)				98
POINT	CASE I	CASE II	CASE III	
A	—	fc = 521	fc = 975	—
B	—	v = 33	v = [illegible]	—
C	—	fc = [illegible]	fs = [illegible]	[illegible]
D	fc = 90	fs = 6200	fs = 16930	[illegible]
E	v = 4	v = 17	v = 35	—
F	fs = 7735	fc = 200	fc = 46	—
G				
H				
I				
J	fc = 222	fc = [illegible]	fc = 4	—
K	v = 15	v = 48	v = 55	—
L	fs = 3600	fs = 2349	fs = 25'33	[illegible]

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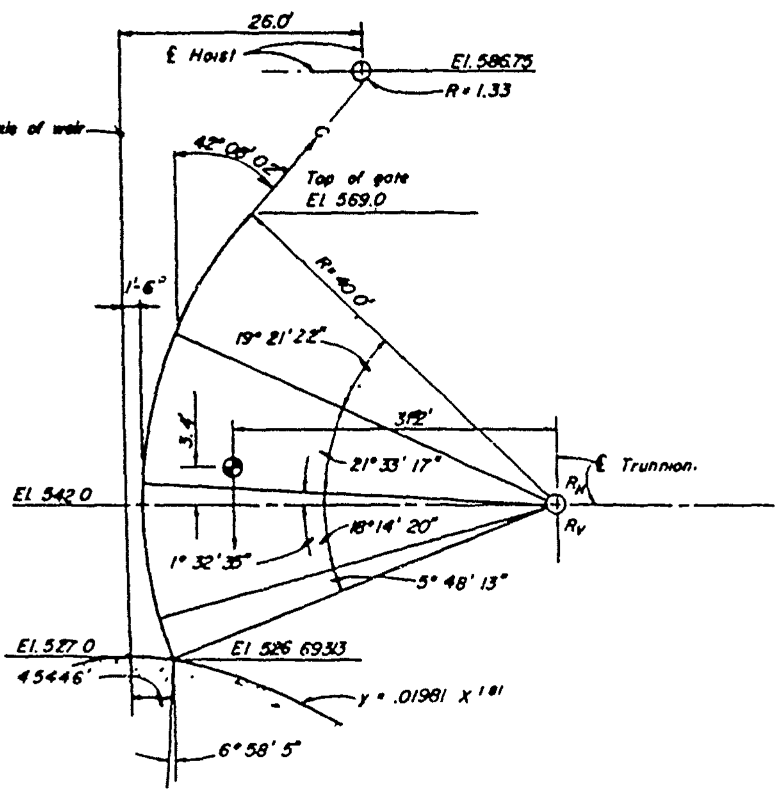
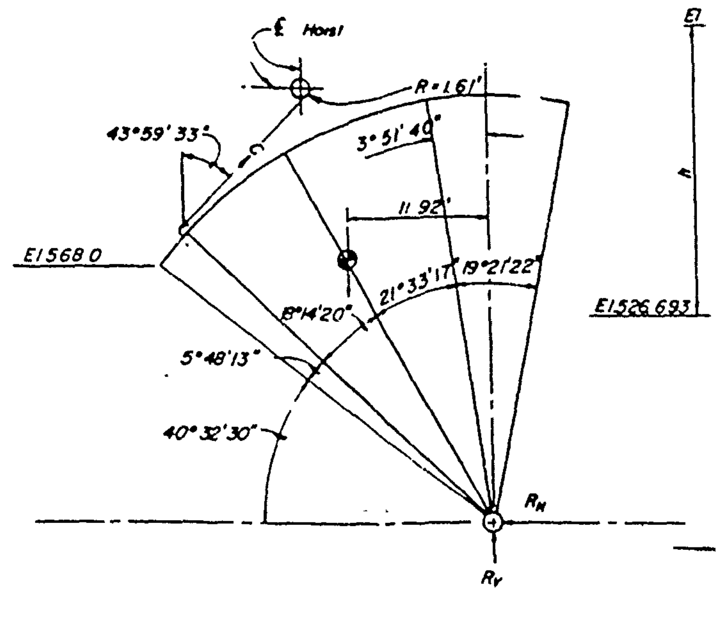
U.S. ARMY ENGINEER DISTRICT TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

GILLHAM DAM AND RESERVOIR
SPILLWAY AND DIKES
SPILLWAY
CHUTE WALL STABILITY ANALYSIS
AUG 1969
1770-DM12-21/6

*U.S. GPO:1989-662-871



GATE CLOSED



GATE FULL OPEN

C – Chain pull
W – Water load
α – Angle between horizontal and resultant of water load
S – Sill reaction
R_H – Horizontal trunnion reaction parallel to pier face
R_V – Vertical trunnion reaction parallel to pier face
H – Horizontal trunnion reaction perpendicular to pier face (Thrust)
M_H – Moment in horizontal plane @ trunnion due to strut reaction
M_V – Moment in vertical plane @ trunnion due to strut reaction

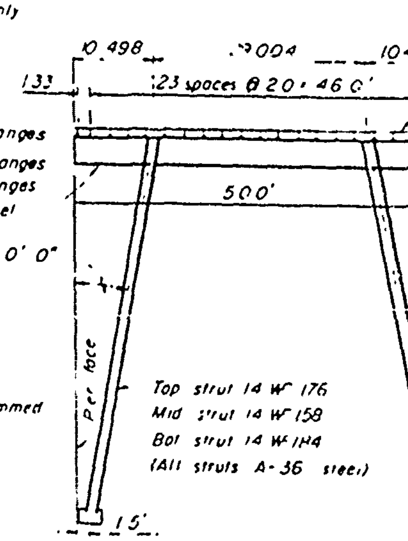
Total weight of gate = 1480[K]

Upper dihedral angle = 4° 42' 15"

Lower dihedral angle = 3° 59' 12"

LOADING CONDITIONS

CASE I	Water to top of gate Gate resting on sill No chain pull
CASE II	Water to top of gate ½ total chain pull on each side No sill reaction
CASE III	Gate full open ½ total chain pull on each side No water load No sill reaction
CASE IV	Case I with wave action
CASE V	Case II with wave action
CASE VI	Water to top of gate ½ total chain pull, one side only No sill reaction
CASE VII	Case VI with wave action



TYPICAL GATE FRAME

NO SCALE

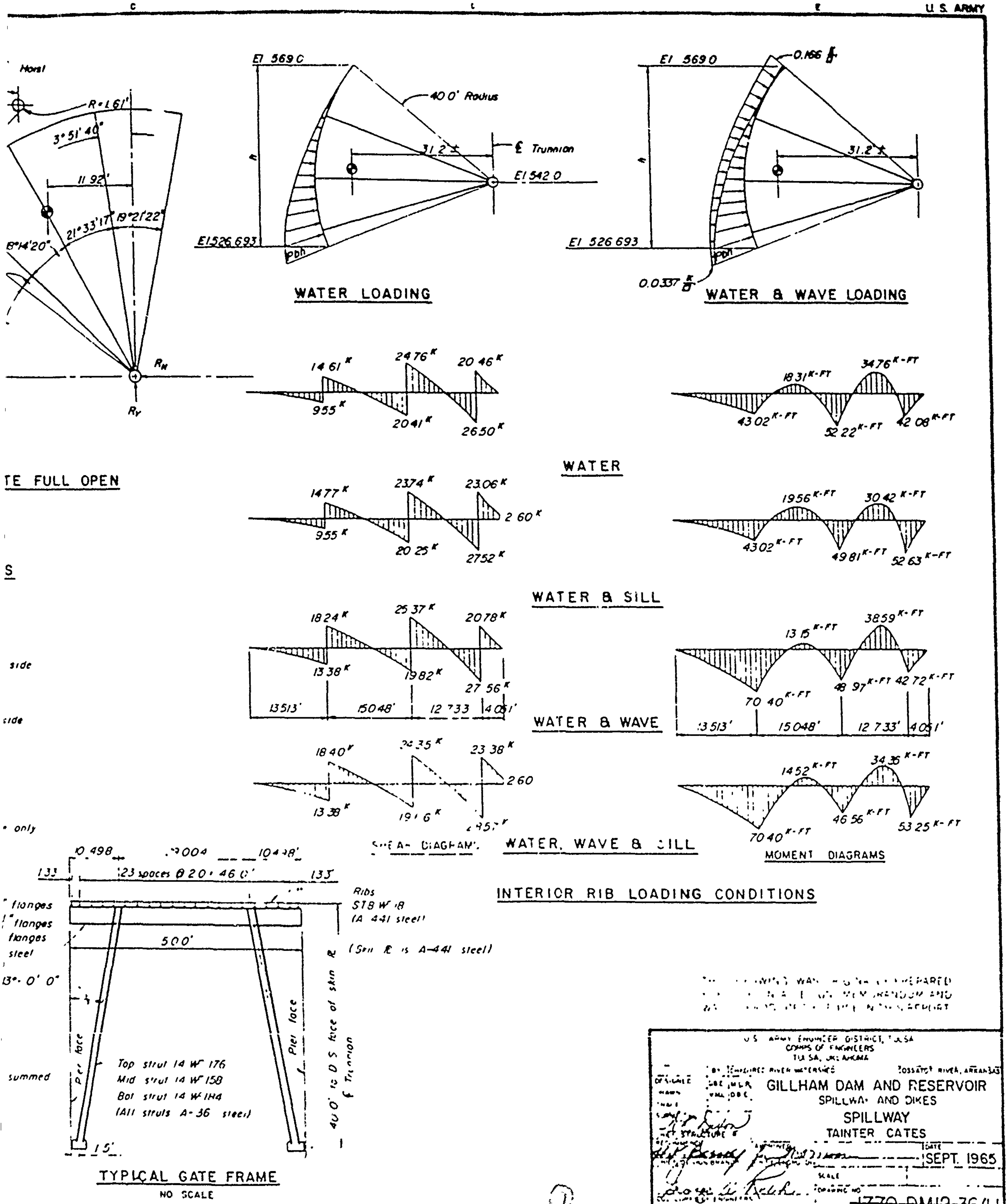
FORCES AND REACTIONS ON 1/2 GATE

CASE & SIDE FRAME	C KIPS	WATER P_H	WATER P_V	S KIPS	R_H KIPS	R_V KIPS	M_H * KIP-FT	M_V * KIP-FT	H * KIPS
I - BOTH	0 0	1398 3	-38 6	66 1	1406 4	-30 42	89 87	-1 83	357 21
II - BOTH	57 6	1398 3	-38 6	0 0	1437 1	-7 18	39 56	-12 36	362 70
III - BOTH	21.7	0 0	0 0	0 0	15 1	58 37	†	†	†
IV - BOTH	0 0	1504 1	-10 6	66 1	1512 2	-2 44	97.55	-1 31	385 09
V - BOTH	57 6	1504 1	-10 6	0 0	1542 9	2[illegible] 80	47 25	-9 81	390 60
VI - LEFT	76 8	1398 3	-38 6	0 0	1450 0	-13 73	-168 51	-57 15	358 05
VI - RIGHT	0 0	1398 3	-38 6	0 0	1398 3	-37 56	-279.94	-40 34	358 05
VII - LEFT	76 8	1504 1	-10 6	0 0	1555 8	+20 17	-100 85	-54 59	385 93
VII - RIGHT	0 0	1504 1	-10 6	0 0	1504 1	-3 14	-267 62	-42 91	385 93

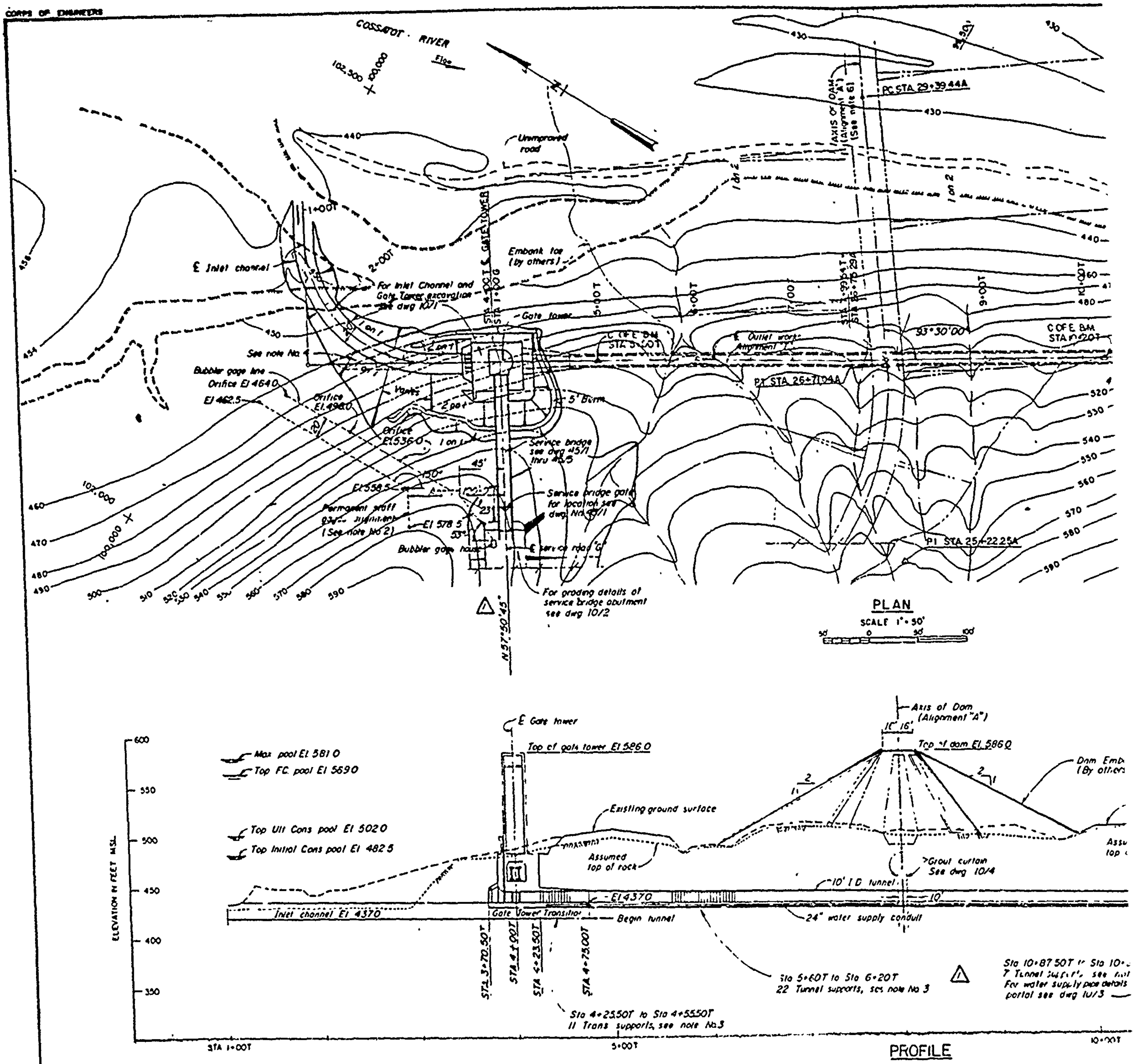
† Not computed * Vectorially summed from struts Minimum R_V's = -8.97[K]

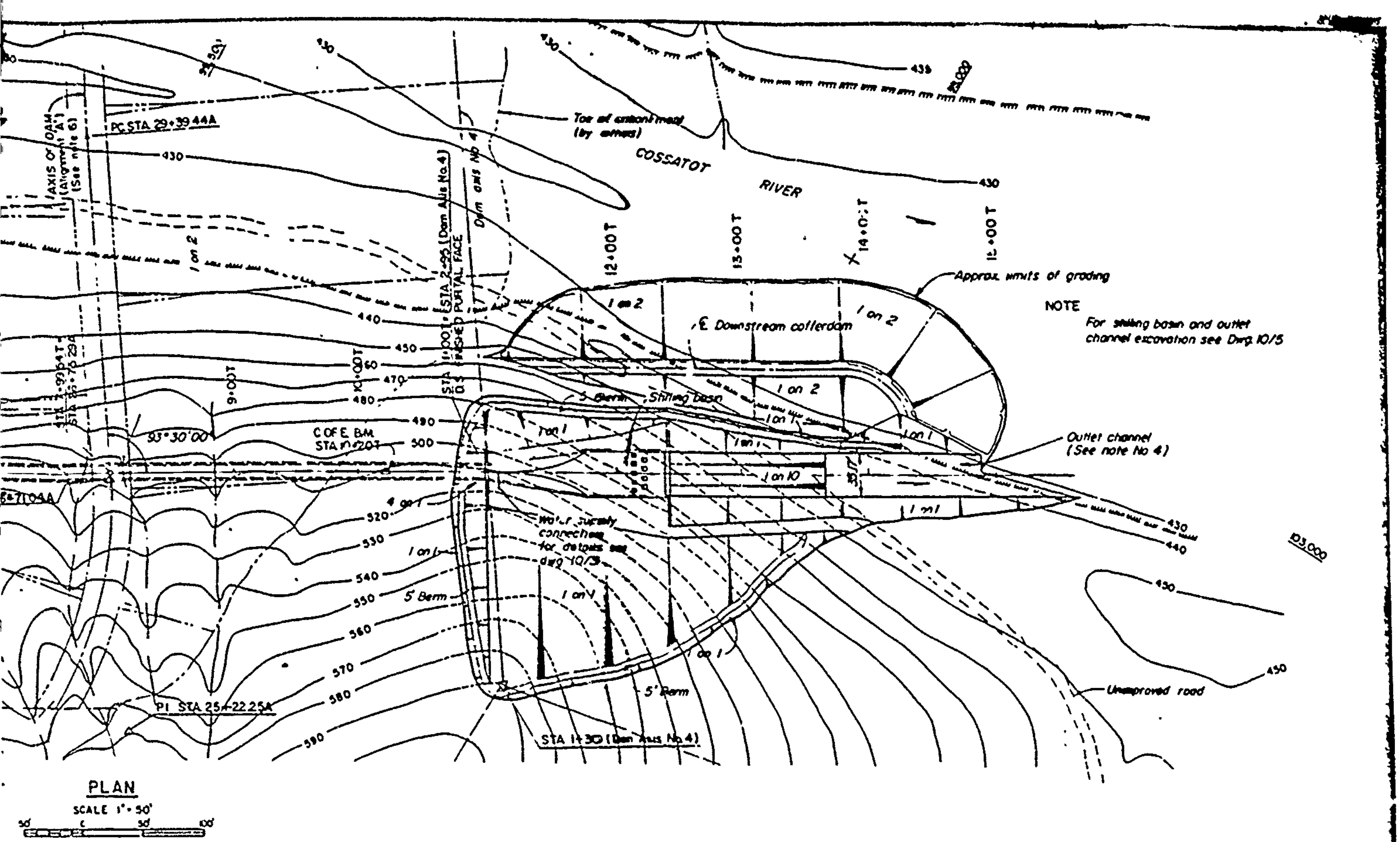
* Vectorially summed from struts

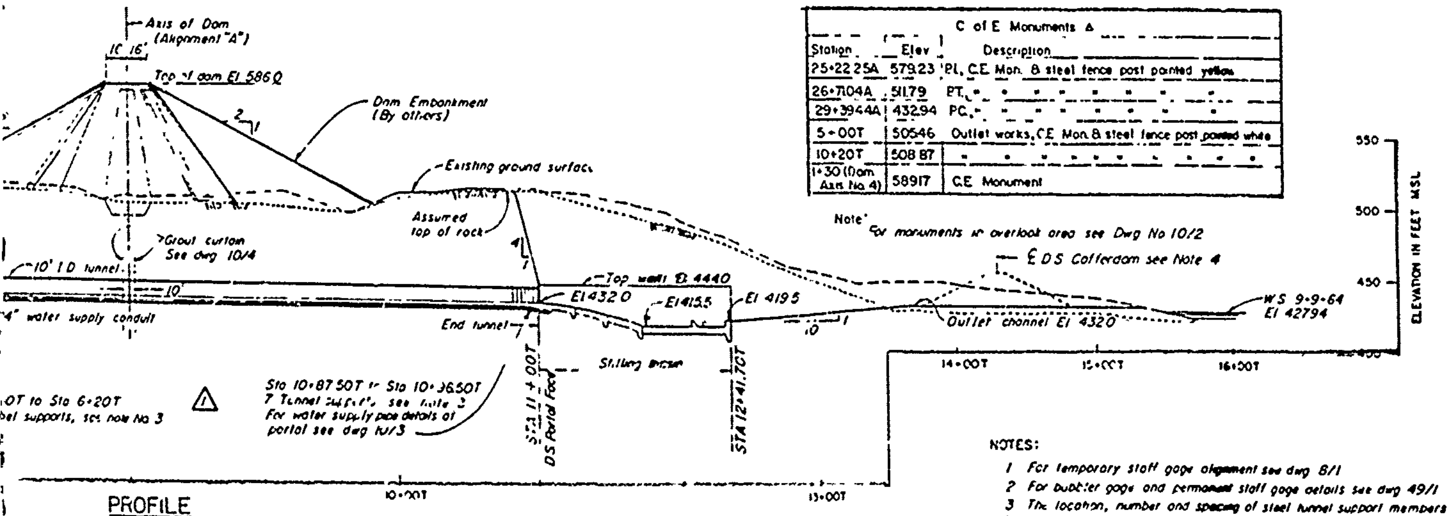
(1)


U. S. ARMY
El 569.0
40.0' Radius
℄ Trunnion
El 542.0
El 526.693
WATER LOADING
0.166 K/□'
0.0337 K/□'
WATER & WAVE LOADING
R=1.61'
3°51'40"
11.92'
21°33'17"
19°21'22"
8°14'20"
TE FULL OPEN
WATER
WATER & SILL
WATER & WAVE
WATER, WAVE & SILL
MOMENT DIAGRAMS
INTERIOR RIB LOADING CONDITIONS
Ribs
ST8 W 18
(A 441 steel)
(Sill ℞ is A-441 steel)
5.00'
Top strut 14 W 176
Mid strut 14 W 158
Bot strut 14 W 184
(All struts A-36 steel)
Pier face
TYPICAL GATE FRAME
NO SCALE
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
GILLHAM DAM AND RESERVOIR
SPILLWAY AND DIKES
SPILLWAY
TAINTER GATES
SEPT. 1965
1770-DM12-36/41

CORPS OF ENGINEERS







C of E Monuments △		
Station	Elev	Description
25+22 25A	579 23	PI, C.E. Mon. & steel fence post painted yellow
26+71 04A	511 79	PT, " " " " " " " "
29+39 44A	432 94	PC, " " " " " " " "
5+00T	505 46	Outlet works, C.E. Mon. & steel fence post painted white
10+20T	508 87	" " " " " " " " " "
1+30 (Dam Axis No 4)	589 17	C.E. Monument

Note: For monuments in overlook area see Dwg No 10/2

NOTES:

1. For temporary staff gage alignment see dwg 8/1
2. For bubbler gage and permanent staff gage details see dwg 49/1
3. The location, number and spacing of steel tunnel support members are approximations only and are subject to change to meet conditions as encountered during construction. Actual placement of steel supports will be subject to approval of the Contracting Officer. See dwg 10/3 for support details
4. For US and DS cofferdam details see dwg 8/1
5. For limits of work areas see dwg No 2/1
6. For dam axis curve data see dwg 10/2

RECORD (AS BUILT) DRAWING

U S ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

GILLHAM DAM

OUTLET WORKS

PLAN & PROFILE

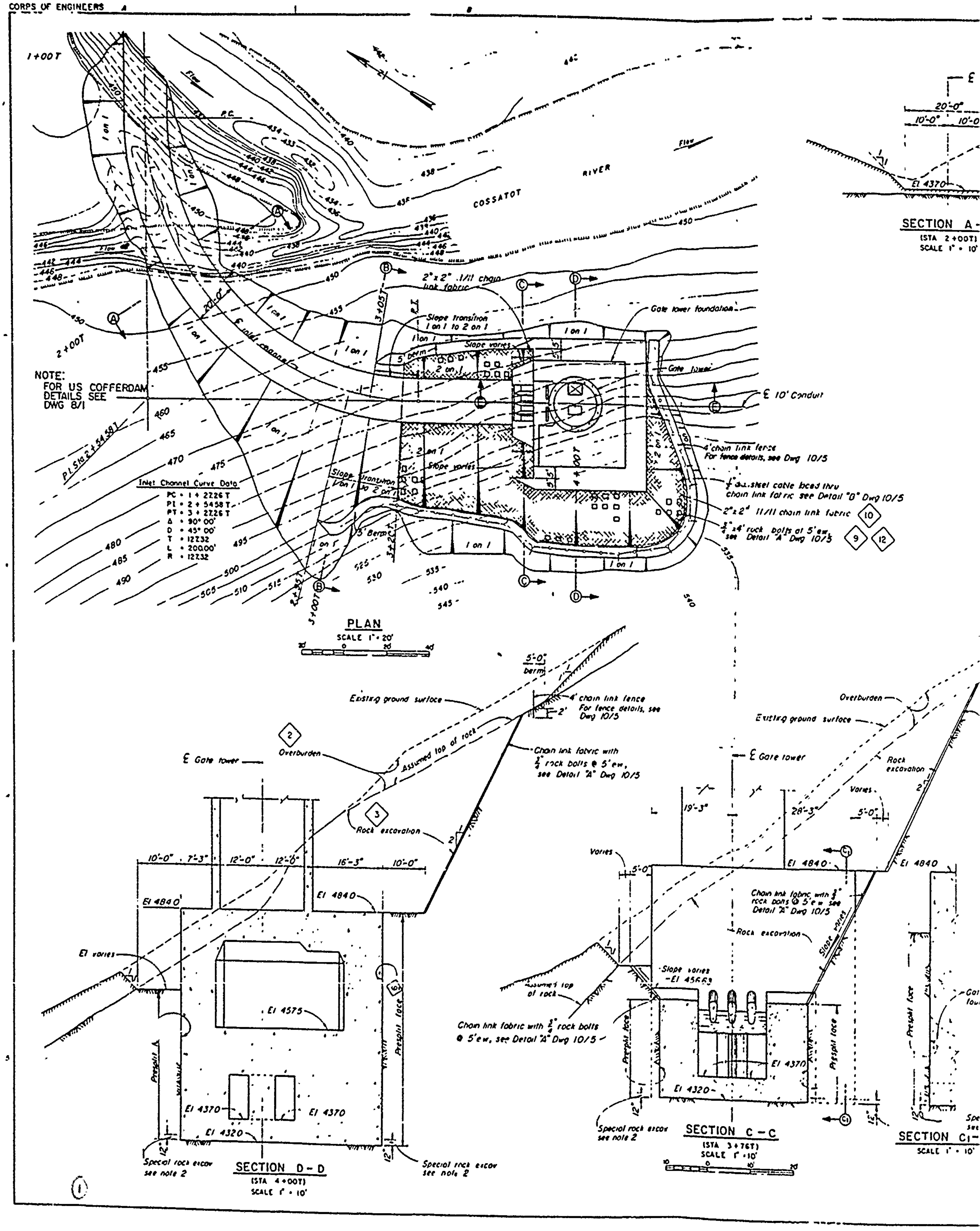
	BY	CHKD
DESIGNED	MAN	WMW
DRAWN	MAN	WMW
TRACED	MS	

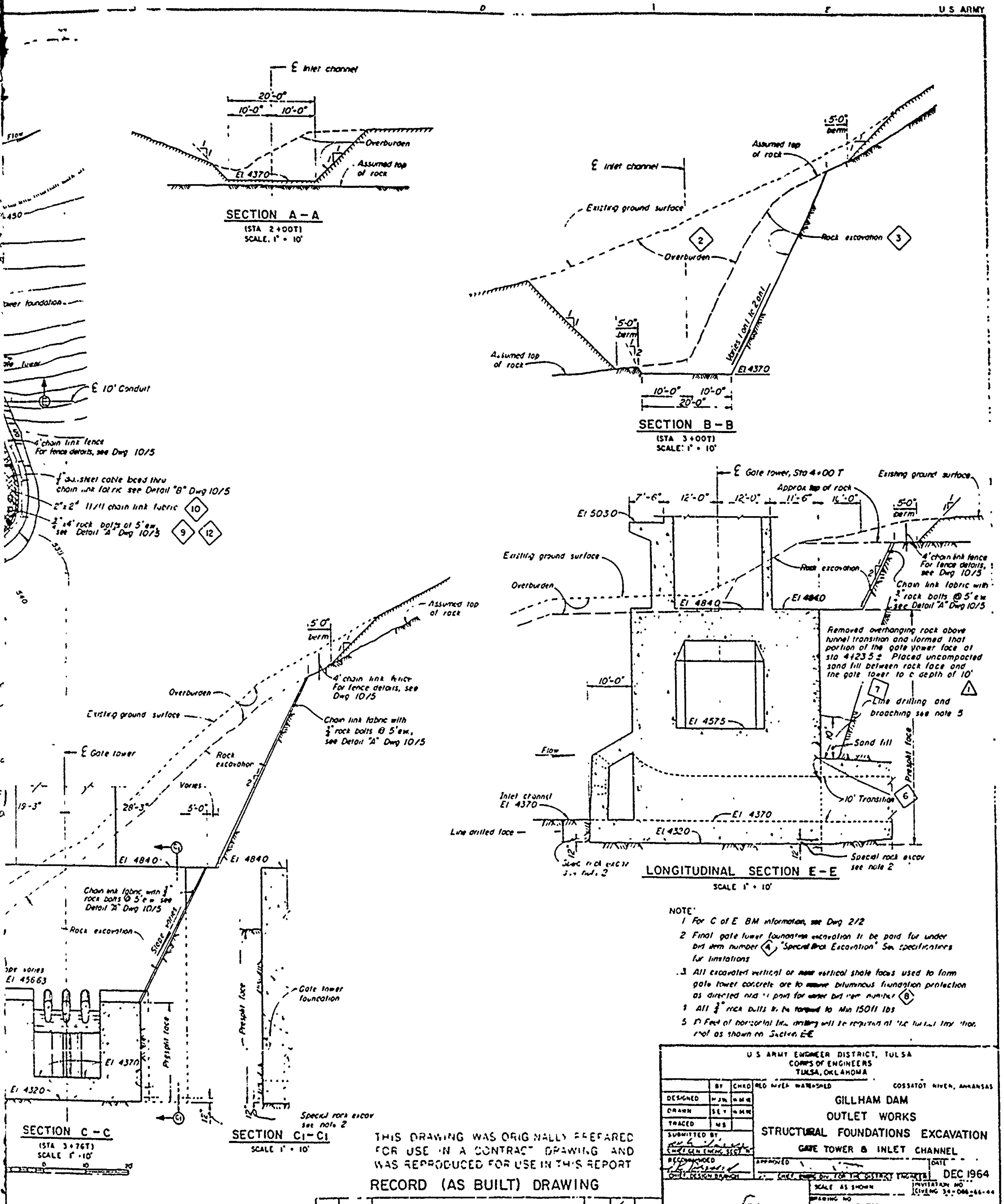
SUBMITTED BY: Chief, Gen Engrg Section
RECOMMENDED: Chief, Design Branch
APPROVED: Chief, Engrg Div for the District Engineer
DATE: DEC. 1964
SCALE AS SHOWN
DRAWING NO 1770-C5-2/2.1

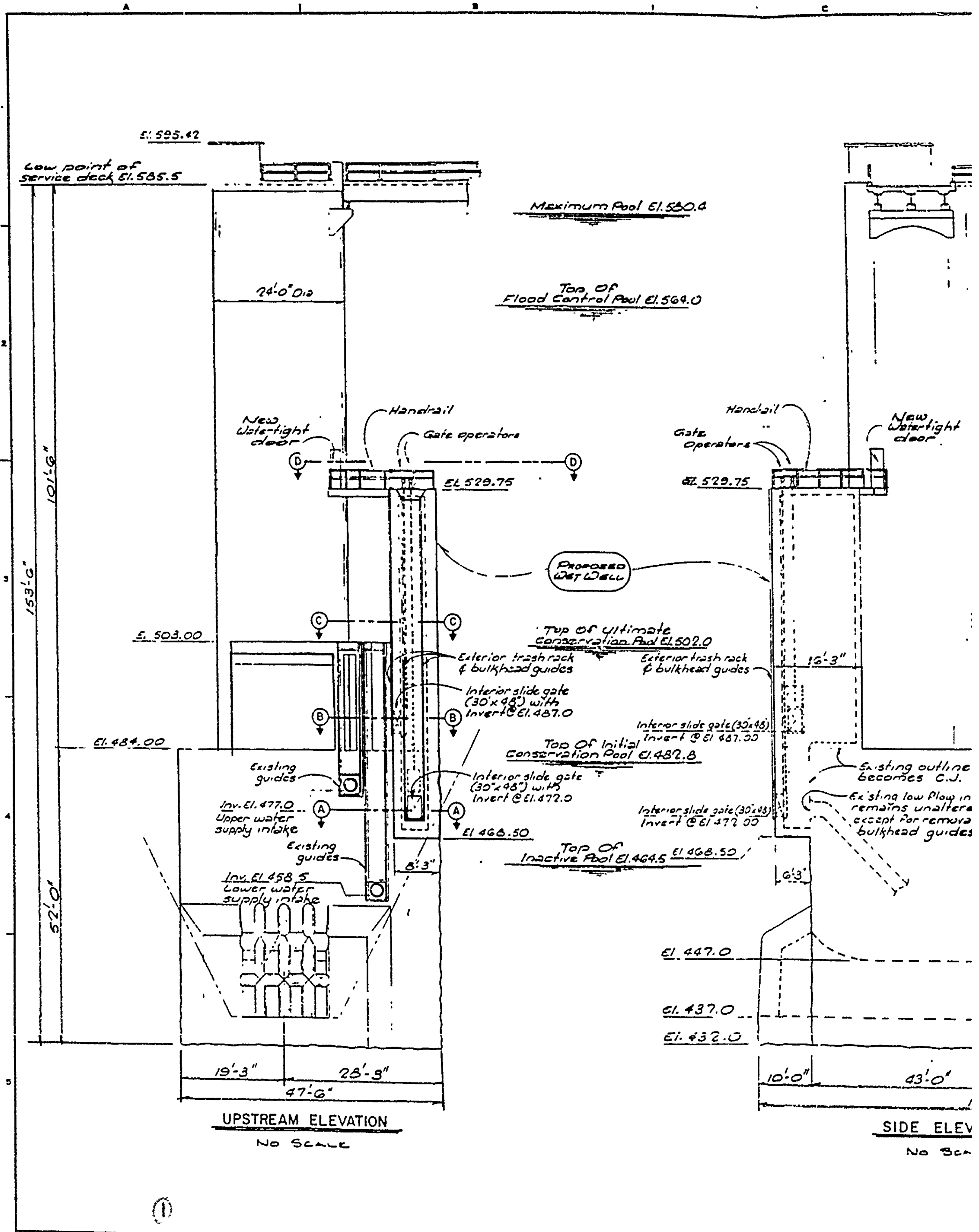
1 | 2-7-?? | Revised as constructed

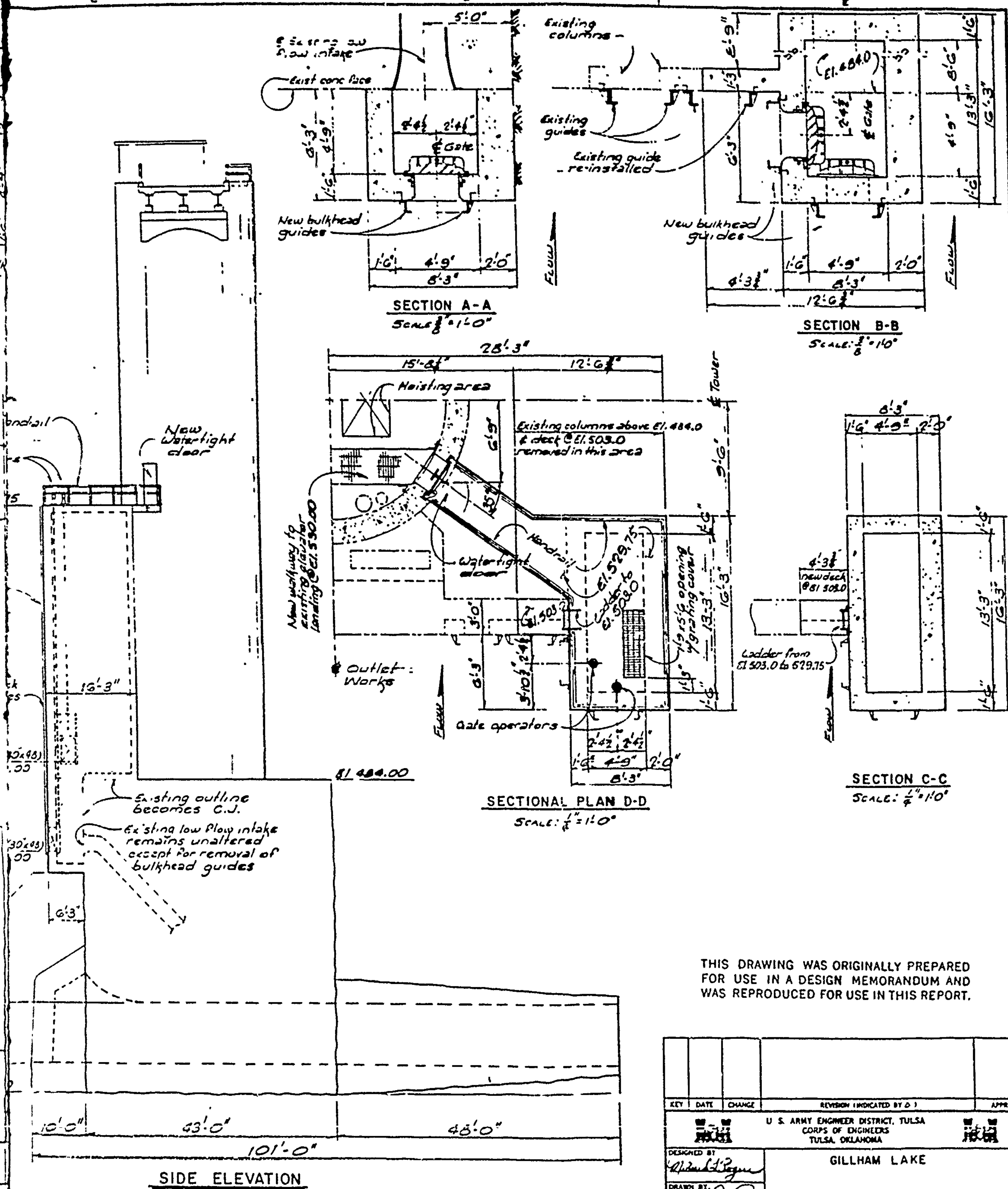
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

(3)


CORPS OF ENGINEERS
1+00T
2+00T
COSSATOT
RIVER
Flow
P.C.
1 on 1
2"x2" 11/11 chain link fabric
Slope transition 1 on 1 to 2 on 1
Slope varies
5' berm
2 on 1
Gate tower foundation
Gate tower
℄ 10' Conduit
℄ Inlet channel
NOTE:
FOR US COFFERDAM DETAILS SEE DWG 8/1
P.I. Sta 2+54.58T
4' chain link fence For fence details, see Dwg 10/5
1/4" dia. steel cable laced thru chain link fabric see Detail "B" Dwg 10/5
2"x2" 11/11 chain link fabric
3/4"x4' rock bolts at 5' e.w. see Detail "A" Dwg 10/5
Inlet Channel Curve Data
PC = 1+22.26T
PI = 2+54.58T
PT = 3+22.26T
Δ = 90° 00'
D = 45° 00'
T = 122.32
L = 200.00'
R = 122.32
Slope transition 1 on 1 to 2 on 1
5' Berm
4+00T
3+05T
3+00T
PLAN
SCALE 1" = 20'
5'-0" berm
Existing ground surface
4' chain link fence For fence details, see Dwg 10/5
Overburden
Assumed top of rock
Chain link fabric with 3/4" rock bolts @ 5' e.w., see Detail "A" Dwg 10/5
℄ Gate tower
Rock excavation
10'-0" 7'-3" 12'-0" 12'-0" 16'-3" 10'-0"
El 4840
El varies
El 4575
Presplit face
El 4370
El 4320
Special rock excav see note 2
SECTION D-D
(STA 4+00T)
SCALE 1" = 10'
Overburden
Existing ground surface
℄ Gate tower
Rock excavation
Varies
19'-3"
28'-3"
5'-0"
El 4840
Chain link fabric with 3/4" rock bolts @ 5' e.w. see Detail "A" Dwg 10/5
Rock excavation
Slope varies
Slope varies -El 4566.3
Assumed top of rock
Chain link fabric with 3/4" rock bolts @ 5' e.w., see Detail "A" Dwg 10/5
Presplit face
El 4370
El 4320
Special rock excav see note 2
SECTION C-C
(STA 3+76T)
SCALE 1" = 10'
20'-0"
10'-0" 10'-0"
El 4370
SECTION A-
(STA 2+00T)
SCALE 1" = 10'
El 4840
SECTION C1-
SCALE 1" = 10'


U S ARMY
SECTION A – A
(STA 2+00T)
SCALE: 1" = 10'
℄ Inlet channel
Overburden
Assumed top of rock
El 4370
SECTION B – B
(STA 3+00T)
SCALE: 1" = 10'
Existing ground surface
Rock excavation
Varies 1 on 1 to 2 on 1
5'-0" berm
℄ 10' Conduit
4' chain link fence For fence details, see Dwg 10/5
2"x2" 11/11 chain link fabric
LONGITUDINAL SECTION E – E
SCALE 1" = 10'
℄ Gate tower, Sta 4+00 T
El 5030
El 4840
El 4575
El 4320
Inlet channel El 4370
Line drilled face
Line drilling and broaching see note 5
Sand fill
10' Transition
Presplit face
Special rock excav see note 2
Removed overhanging rock above tunnel transition and formed that portion of the gate tower face at sta 4+23.5 ± Placed uncompacted sand fill between rock face and the gate tower to a depth of 10'
Chain link fabric with 3/4" rock bolts @ 5' e.w., see Detail "A" Dwg 10/5
SECTION C – C
(STA 3+76T)
SCALE 1" = 10'
SECTION C1 – C1
SCALE 1" = 10'
Gate tower foundation
NOTE:
1 For C of E BM information, see Dwg 2/2
2 Final gate tower foundation excavation to be paid for under bid item number 4, "Special Rock Excavation". See specifications for limitations
3 All excavated vertical or near vertical shale faces used to form gate tower concrete are to receive bituminous foundation protection as directed and is paid for under bid item number 8
4 All 3/4" rock bolts to be torqued to Min 150 ft lbs
5 ... Feet of horizontal line drilling will be required at the tunnel transition roof as shown on Section E-E
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
RECORD (AS BUILT) DRAWING
Revised as constructed
U S ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
OUTLET WORKS
STRUCTURAL FOUNDATIONS EXCAVATION
GATE TOWER & INLET CHANNEL
DEC 1964
SCALE AS SHOWN
1770-C5-10/1.1


El. 595.42
Low point of service deck El. 585.5
Maximum Pool El. 580.4
24'-0" Dia
Top Of Flood Control Pool El. 564.0
153'-6"
101'-6"
New Water-tight door
Handrail
Gate operators
El. 529.75
Proposed Wet Well
El. 503.00
Top of ultimate Conservation Pool El. 502.0
Exterior trash rack & bulkhead guides
Interior slide gate (30"x48") with Invert @ El. 487.0
El. 484.00
Top Of Initial Conservation Pool El. 482.8
Existing guides
Interior slide gate (30"x48") with Invert @ El. 472.0
Inv. El. 477.0 Upper water supply intake
El 468.50
Top Of Inactive Pool El. 464.5
Existing guides
Inv. El 458.5 Lower water supply intake
8'-3"
52'-0"
19'-3"
28'-3"
47'-6"
UPSTREAM ELEVATION
No Scale
Handrail
Gate operators
New Watertight door
El. 529.75
16'-3"
Exterior trash rack & bulkhead guides
Interior slide gate (30x48) Invert @ El 487.00
Existing outline becomes C.J.
Existing low flow in remains unaltere except for remova bulkhead guides
Interior slide gate (30x48) Invert @ El 472.00
El 468.50
6'-3"
El 447.0
El. 437.0
El. 432.0
10'-0"
43'-0"
SIDE ELEV
No Sca



THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A DESIGN MEMORANDUM AND WAS REPRODUCED FOR USE IN THIS REPORT.

KEY	DATE	CHANGE	REVISION (INDICATED BY ◇)	APPR

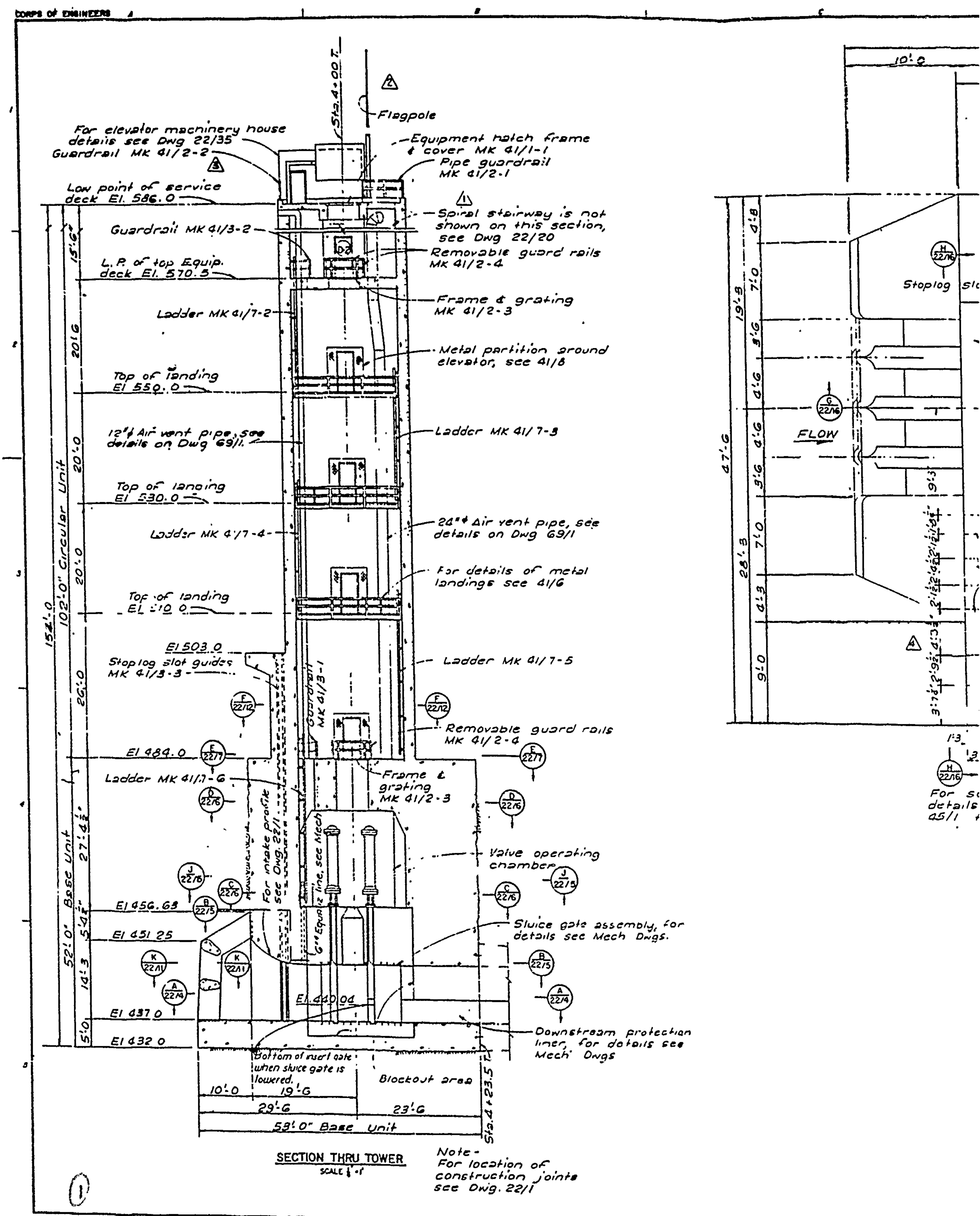
U. S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

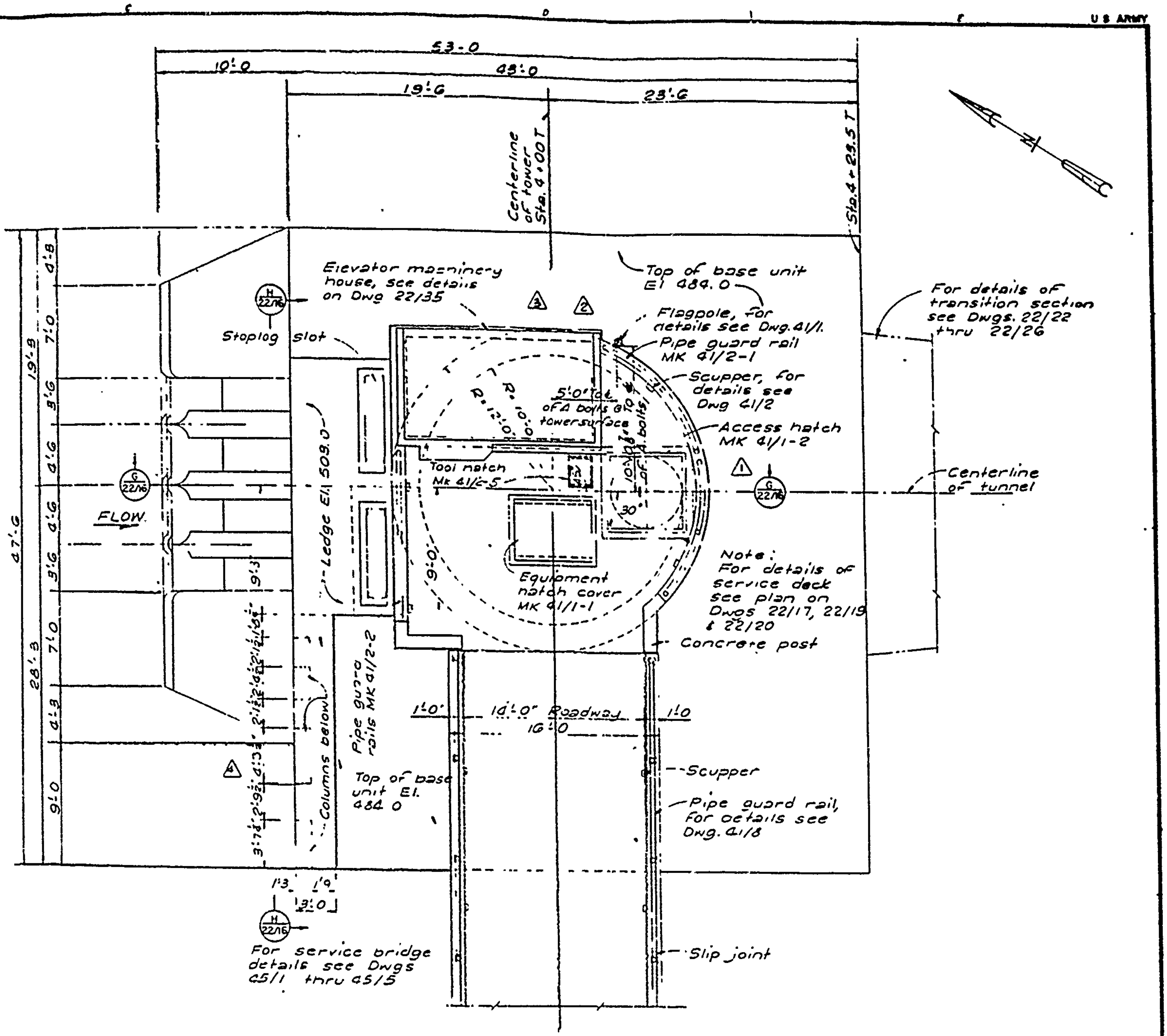
GILLHAM LAKE

ADDITION TO INTAKE STRUCTURE

LOW FLOW WET WELL FACILITIES

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED:
CHIEF, DATE
SCALE AS SHOWN
DRAWING NUMBER ~~1770-DM10-22/9~~


CORPS OF ENGINEERS
For elevator machinery house details see Dwg 22/35
Guardrail MK 41/2-2
Flagpole
Sta. 4+00 T.
Equipment hatch frame & cover MK 41/1-1
Pipe guardrail MK 41/2-1
Low point of service deck El. 586.0
Spiral stairway is not shown on this section, see Dwg 22/20
Guardrail MK 41/3-2
Removable guard rails MK 41/2-4
L.P. of top Equip. deck El. 570.5
Frame & grating MK 41/2-3
Ladder MK 41/7-2
Metal partition around elevator, see 41/8
Top of landing El 550.0
12"ϕ Air vent pipe, see details on Dwg 69/1.
Ladder MK 41/7-3
Top of landing El 530.0
Ladder MK 41/7-4
24"ϕ Air vent pipe, see details on Dwg 69/1
For details of metal landings see 41/6
El 503.0
Stoplog slot guides MK 41/3-3
Ladder MK 41/7-5
Guardrail MK 41/3-1
Removable guard rails MK 41/2-4
El 484.0
Ladder MK 41/7-6
Frame & grating MK 41/2-3
For intake profile see Dwg. 22/1
6"ϕ Equaliz line, see Mech
Valve operating chamber
El 456.63
El 451.25
Sluice gate assembly, for details see Mech Dwgs.
El 440.04
El 437.0
El 432.0
Downstream protection liner, for details see Mech Dwgs
Bottom of insert gate when sluice gate is lowered.
Blockout area
Sta. 4+23.5 T.
10'-0
19'-6
29'-6
23'-6
53'-0" Base Unit
152'-0
102'-0" Circular Unit
52'-0" Base Unit
15'-6"
20'-6
20'-0
20'-0
26'-0
27'-4 3/8"
5'-4 5/8"
14'-3
5'-0
SECTION THRU TOWER
Note -
For location of construction joints see Dwg. 22/1
10'-0
Stoplog
FLOW
47'-6
19'-3
28'-3
For details 45/1



PLAN AT TOP DECK

SCALE 1/8"=1'

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

RECORD (AS BUILT) DRAWING

KEY	DATE	REVISION
4		Revised as constructed
3		Revise elevator machinery house
2		Added flagpole
1		Revised access hatch, added tool hatch & spiral stairway

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

GILLHAM DAM
OUTLET WORKS
STRUCTURAL

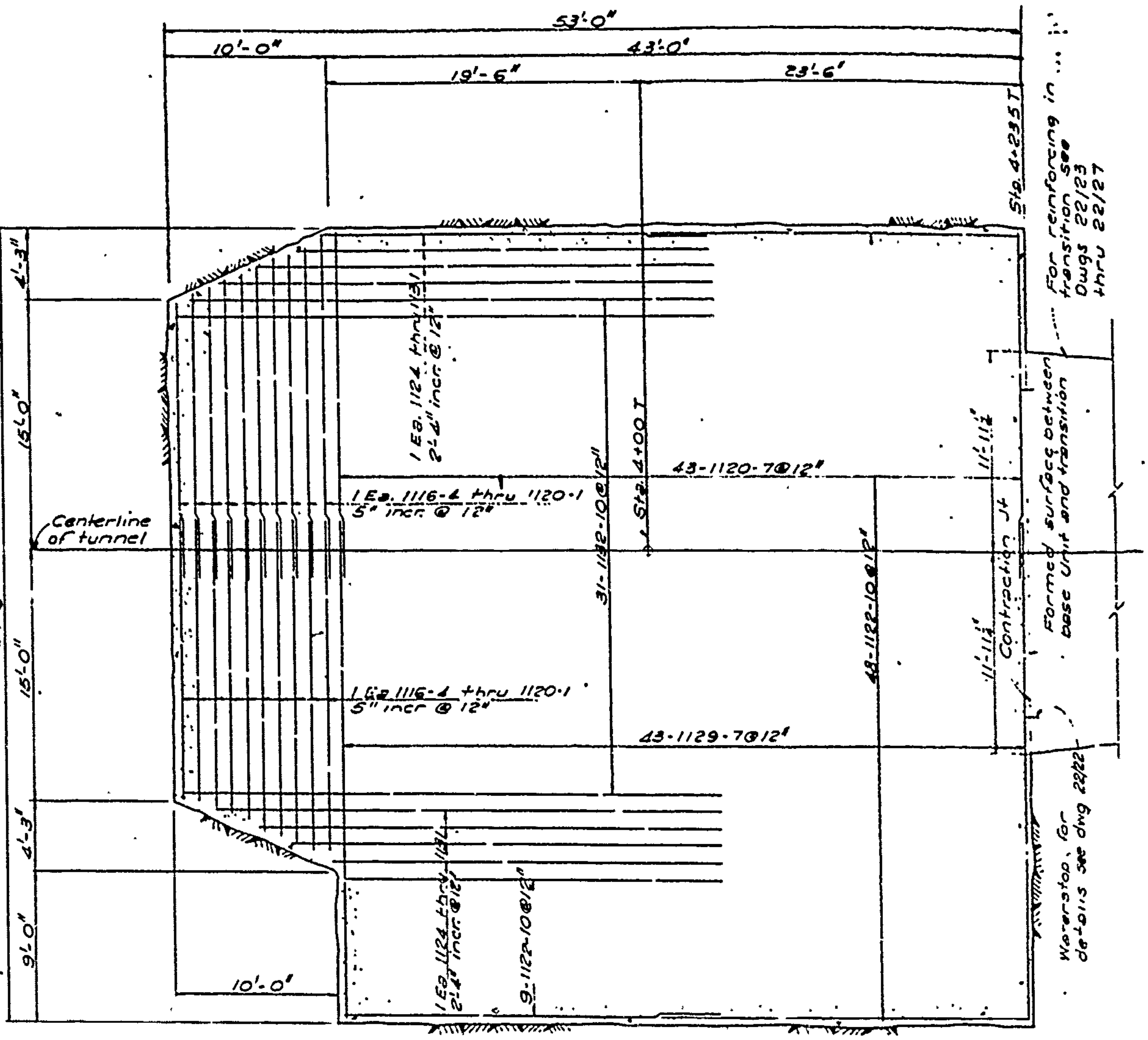
GATE TOWER - PLAN AND SECTION

DEC 1964

SCALE AS SHOWN

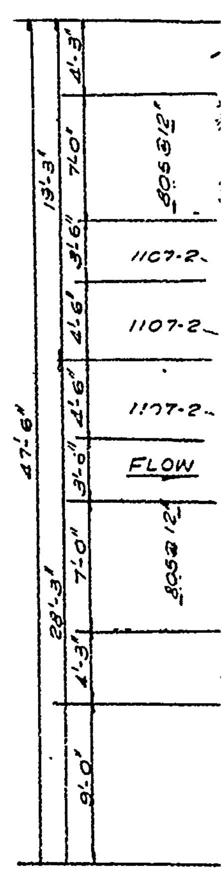
DRAWING NO. 1770-C5-22/2.4

CORPS OF ENGINEERS



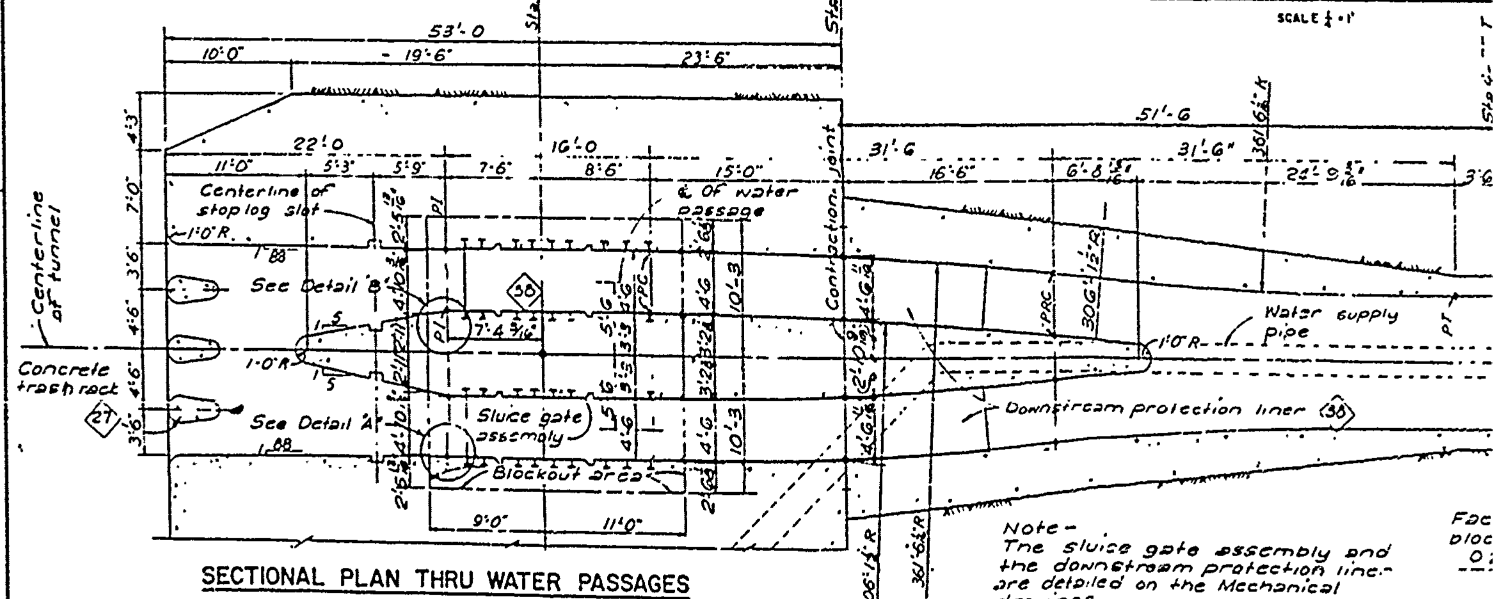
BOTTOM STEEL

SCALE: ¼" = 1'-0"



BASE SLAB REINFORCING (EL.432.0-

SCALE ¼" = 1'

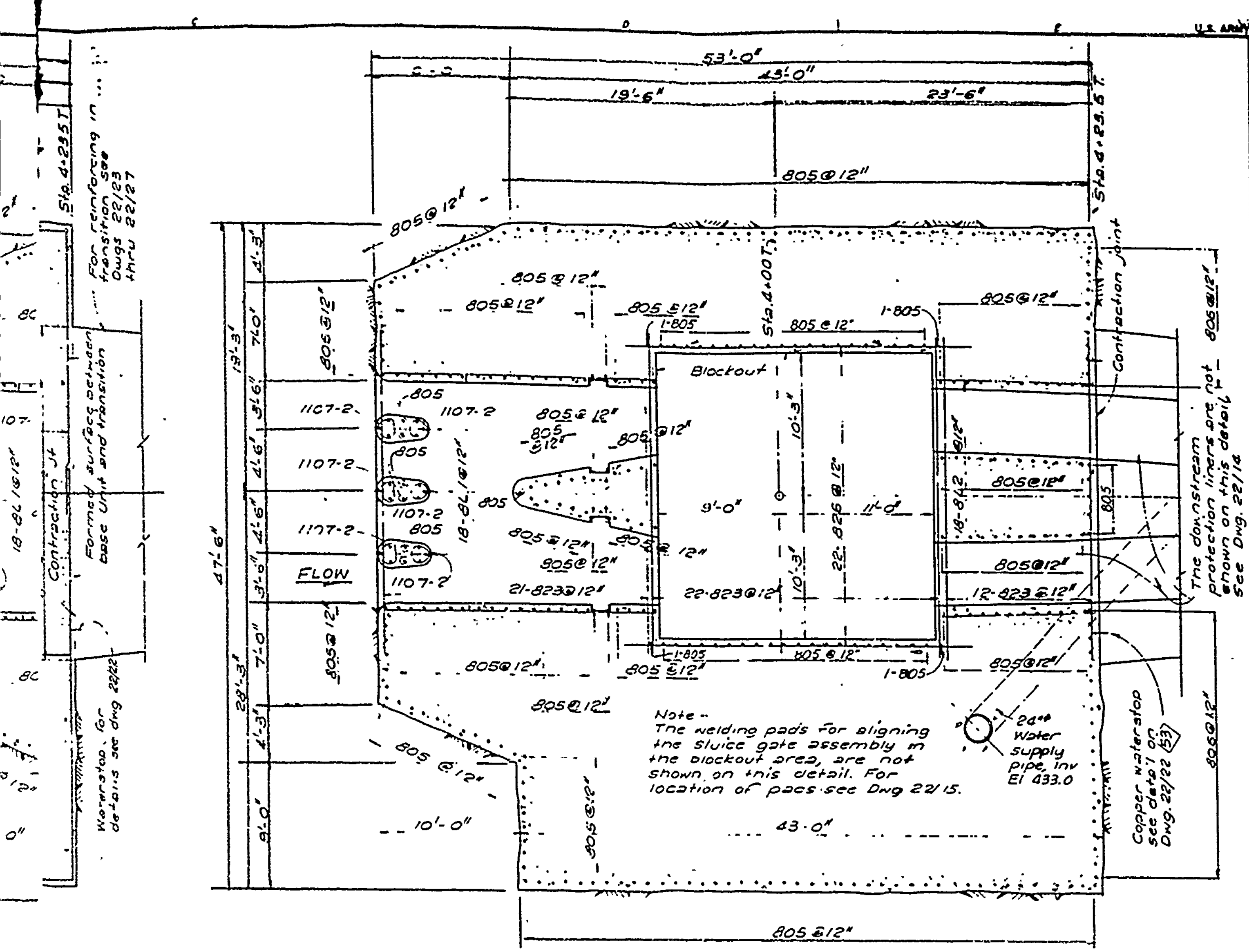


SECTIONAL PLAN THRU WATER PASSAGES

SCALE ¼" = 1'-0"

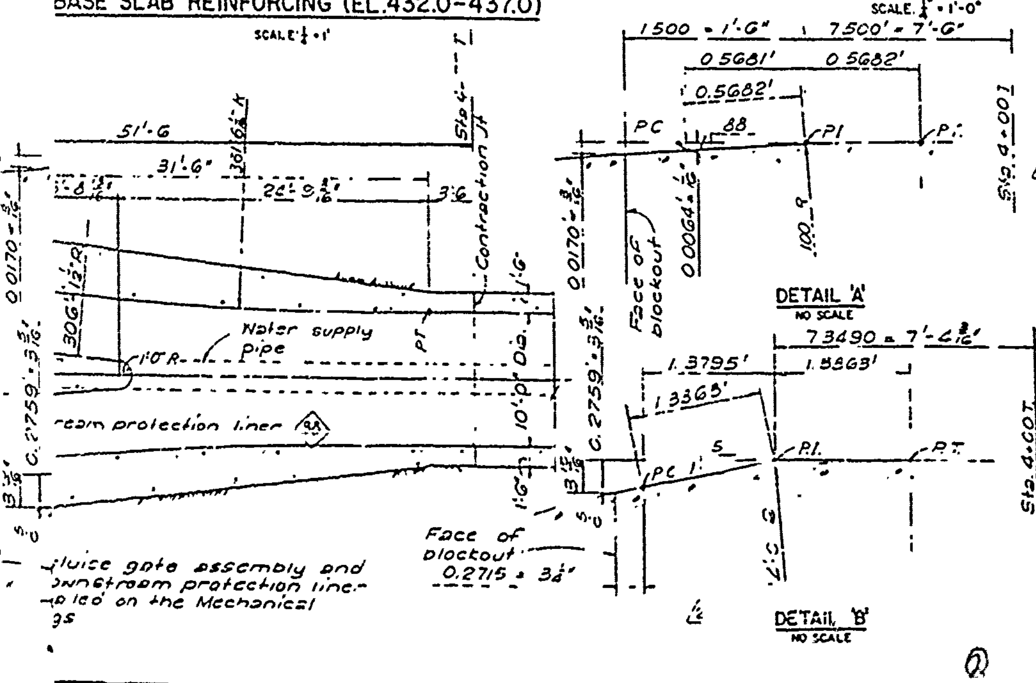
Note - The sluice gate assembly and the downstream protection liner are detailed on the Mechanical drawings

①



BASE SLAB REINFORCING (EL.432.0-437.0)

SCALE $\frac{1}{8}$" = 1'



THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT.

NOTES:

1. Concrete in blockout paid for under bid item (34)
2. Concrete in base unit except in blockout and concrete trash rack paid for under bid item (25)

RECORD (AS BUILT) DRAWING

KEY	DATE	REVISION	CHKD	REC	APPR
2	9 13 66	Curve data revision			
1	12-65	Changed note, Amendment #1			

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

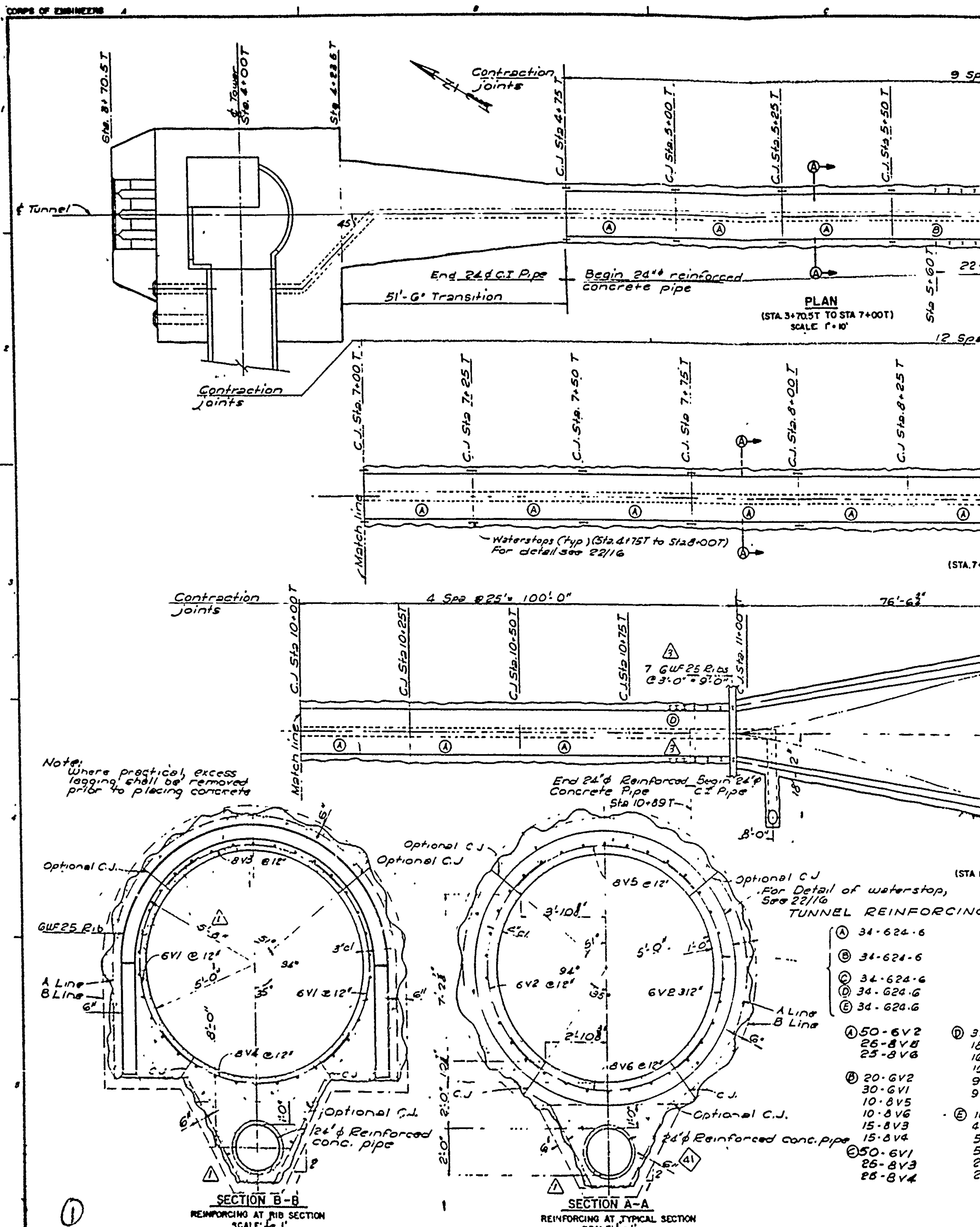
GILLHAM DAM
OUTLET WORKS
STRUCTURAL
GATE TOWER - BASE UNIT
HORIZONTAL SECTIONS I

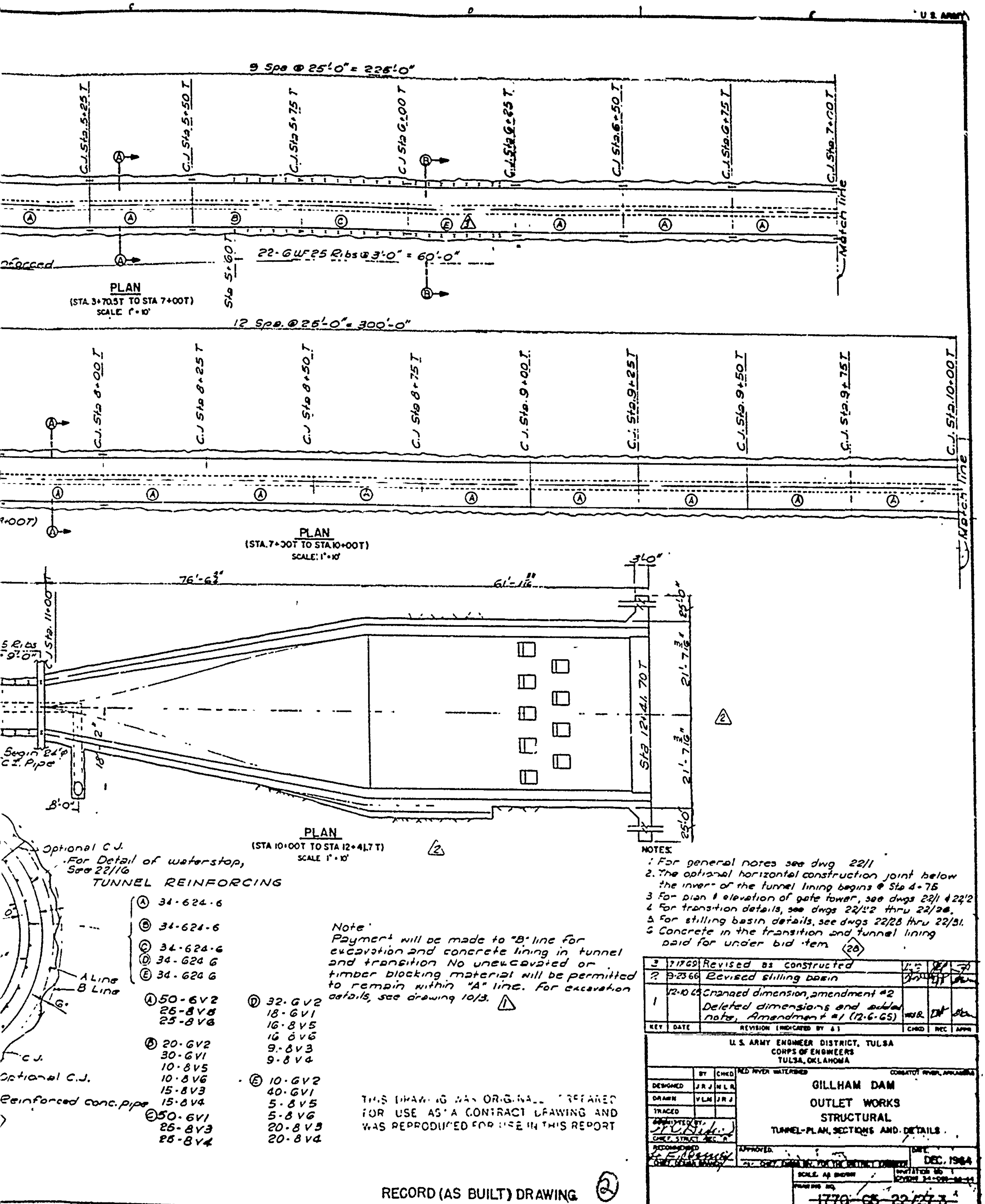
	BY	CHKD
DESIGNED	J.R.J.	W.L.R.
DRAWN	C.F.K.	L.W.P.
TRACED		

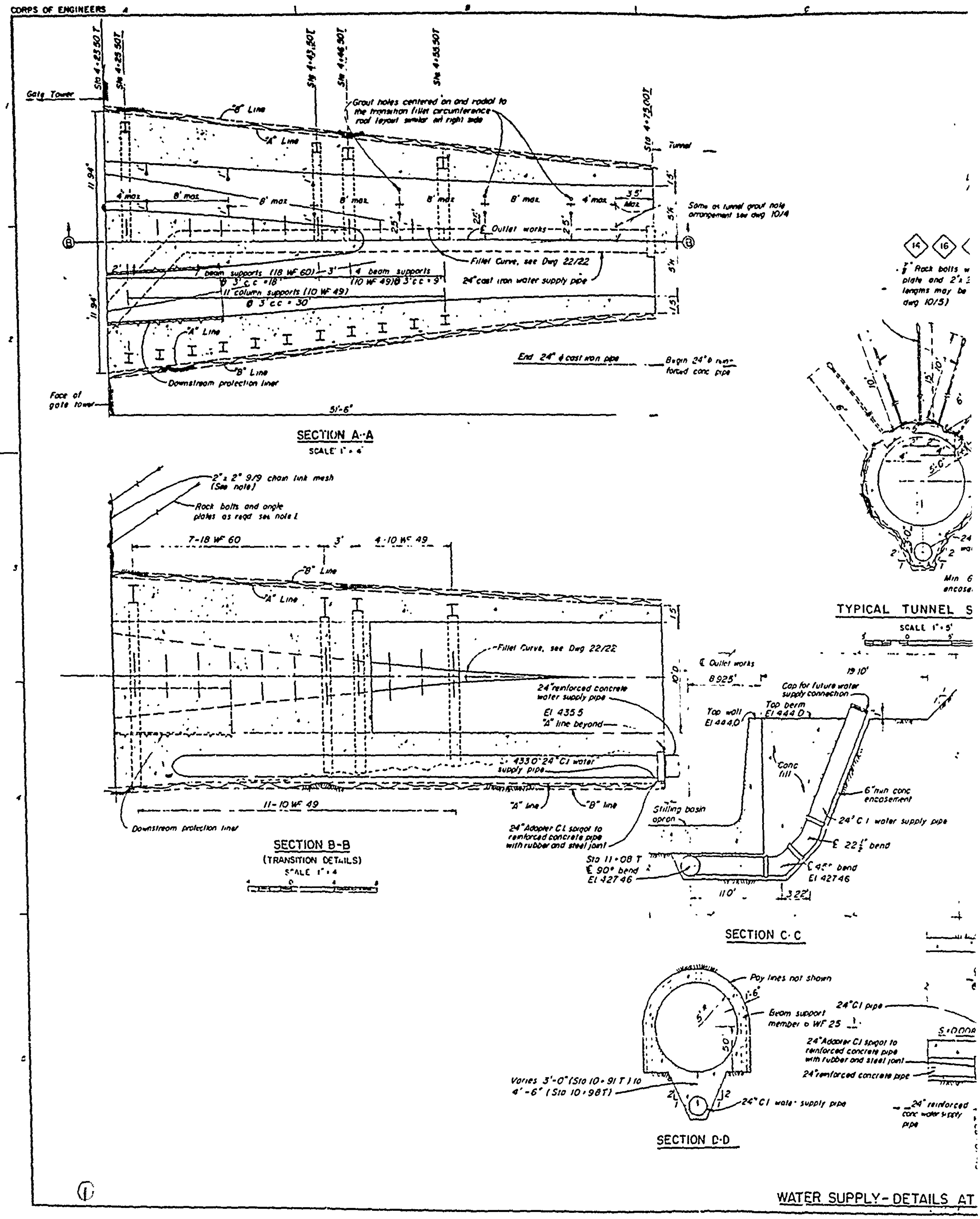
DATE DEC. 1964

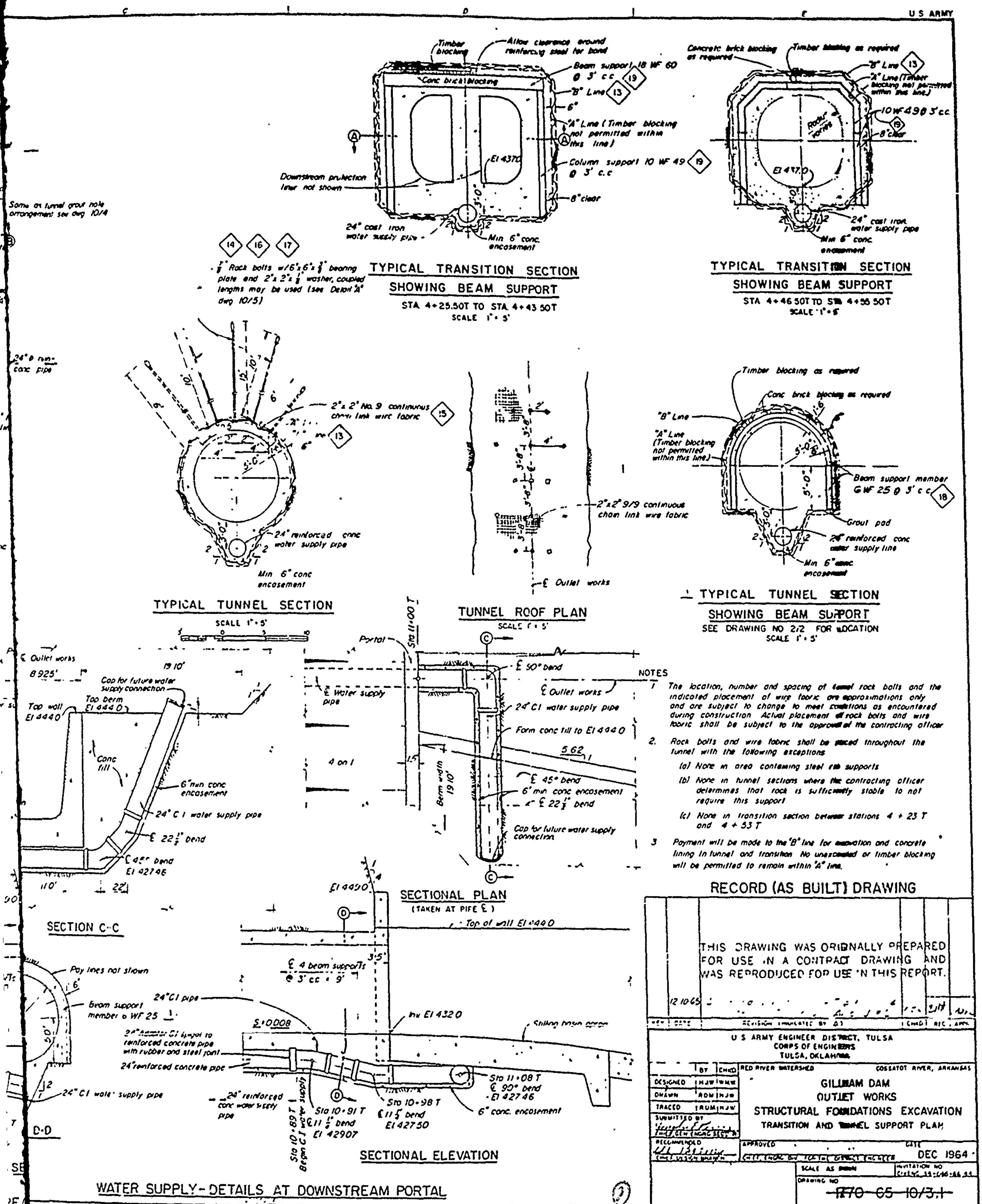
SCALE AS SHOWN

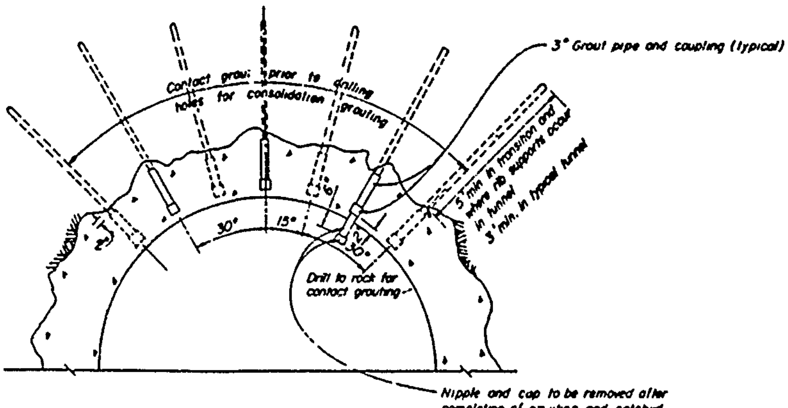
~~1770-C5-22/3.2~~


CORPS OF ENGINEERS
Contraction joints
Sta. 3+70.5T
℄ Tower Sta. 4+00T
Sta. 4+23.6T
C.J. Sta. 4+75 T
C.J. Sta. 5+00 T.
C.J. Sta. 5+25 T
C.J. Sta. 5+50 T
9 Spa
℄ Tunnel
45°
End 24"ϕ C.I. Pipe
Begin 24"ϕ reinforced concrete pipe
51'-6" Transition
Sta. 5+60T
PLAN
(STA. 3+70.5T TO STA 7+00T)
SCALE 1"=10'
12 Spa
Contraction joints
C.J. Sta. 7+00 T.
C.J. Sta. 7+25 T
C.J. Sta. 7+50 T
C.J. Sta. 7+75 T
C.J. Sta. 8+00 T
C.J. Sta. 8+25 T
Match line
Waterstops (typ) (Sta. 4+75T to Sta. 8+00T)
For detail see 22/16
Contraction joints
4 Spa @ 25' = 100'-0"
76'-6¾"
C.J. Sta. 10+00 T
C.J. Sta. 10+25T
C.J. Sta. 10+50T
C.J. Sta. 10+75T
C.J. Sta. 11+00
7 6WF25 Ribs @ 3'-0" = 9'-0"
Match line
End 24"ϕ Reinforced Concrete Pipe
Begin 24"ϕ C.I. Pipe
Sta 10+89T
8'-0"
Note: Where practical, excess lagging shall be removed prior to placing concrete
Optional C.J.
6WF25 Rib
A Line
B Line
8V3 @ 12"
6V1 @ 12"
8V4 @ 12"
24"ϕ Reinforced conc. pipe
SECTION B-B
REINFORCING AT RIB SECTION
SCALE: ½" = 1'
Optional C.J.
8V5 @ 12"
6V2 @ 12"
6V2 @ 12"
8V6 @ 12"
For Detail of waterstop, See 22/16
24"ϕ Reinforced conc. pipe
SECTION A-A
REINFORCING AT TYPICAL SECTION
SCALE: ½" = 1'
TUNNEL REINFORCING
Ⓐ 34-624-6
Ⓑ 34-624-6
Ⓒ 34-624-6
Ⓓ 34-624-6
Ⓔ 34-624-6
Ⓐ 50-6V2
26-8V8
25-8V6
Ⓑ 20-6V2
30-6V1
10-8V5
10-8V6
15-8V3
15-8V4
Ⓒ 50-6V1
25-8V3
25-8V4
①


9 Spa @ 25'-0" = 225'-0"
C.J. Sta. 5+25 T
C.J. Sta. 5+50 T
C.J. Sta. 5+75 T
C.J. Sta. 6+00 T
C.J. Sta. 6+25 T
C.J. Sta. 6+50 T
C.J. Sta. 6+75 T
C.J. Sta. 7+00 T
Match line
22-6WF25 Ribs @ 3'-0" = 60'-0"
Sta 5+60 T
PLAN
(STA. 3+70.5T TO STA 7+00T)
SCALE: 1" = 10'
12 Spa. @ 25'-0" = 300'-0"
C.J. Sta 8+00 T
C.J. Sta 8+25 T
C.J. Sta 8+50 T
C.J. Sta 8+75 T
C.J. Sta. 9+00 T
C.J. Sta. 9+25 T
C.J. Sta. 9+50 T
C.J. Sta. 9+75 T
C.J. Sta. 10+00 T
Match line
PLAN
(STA. 7+00T TO STA. 10+00T)
SCALE: 1" = 10'
76'-6¾"
61'-1⅛"
3'-0"
25'-0"
21'-7 3/16"
Sta 12+41.70 T
C.J. Sta. 11+00
Begin 24" φ C.I. Pipe
8'-0"
PLAN
(STA 10+00T TO STA 12+41.7 T)
SCALE: 1" = 10'
Optional C.J.
For Detail of waterstop, See 22/16
TUNNEL REINFORCING
Ⓐ 34-624-6
Ⓑ 34-624-6
Ⓒ 34-624-6
Ⓓ 34-624 6
Ⓔ 34-624 6
A Line
B Line
C.J.
Optional C.J.
Reinforced conc. pipe
Ⓐ 50-6V2 26-8V8 25-8V6
Ⓑ 20-6V2 30-6V1 10-8V5 10-8V6 15-8V3 15-8V4
Ⓒ 50-6V1 26-8V3 26-8V4
Ⓓ 32-6V2 18-6V1 16-8V5 16-8V6 9-8V3 9-8V4
Ⓔ 10-6V2 40-6V1 5-8V5 5-8V6 20-8V3 20-8V4
Note:
Payment will be made to "B" line for excavation and concrete lining in tunnel and transition. No unexcavated or timber blocking material will be permitted to remain within "A" line. For excavation details, see drawing 10/3.
NOTES:
1. For general notes see dwg 22/1
2. The optional horizontal construction joint below the invert of the tunnel lining begins @ Sta 4+75
3. For plan & elevation of gate tower, see dwgs 22/1 & 22/2
4. For transition details, see dwgs 22/22 thru 22/26.
5. For stilling basin details, see dwgs 22/28 thru 22/31.
6. Concrete in the transition and tunnel lining paid for under bid item 26
3 | 7-17-69 | Revised as constructed
2 | 9-23-66 | Revised stilling basin
1 | 12-10-65 | Changed dimension, amendment #2. Deleted dimensions and added note, Amendment #1 (12-6-65)
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
OUTLET WORKS
STRUCTURAL
TUNNEL-PLAN, SECTIONS AND DETAILS
DEC. 1964
1770-GS-22/27.3
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
RECORD (AS BUILT) DRAWING


CORPS OF ENGINEERS
Gate Tower
"B" Line
"A" Line
Grout holes centered on and radial to the transition fillet circumference roof layout similar at right side
Tunnel
Same as tunnel grout hole arrangement see dwg 10/4
Outlet works
Fillet Curve, see Dwg 22/22
24" cast iron water supply pipe
7 beam supports (18 WF 60) @ 3' c.c = 18'
4 beam supports (10 WF 49) @ 3' c.c = 9'
11 column supports (10 WF 49) @ 3' c.c = 30'
End 24" cast iron pipe
Begin 24" reinforced conc pipe
Downstream protection liner
Face of gate tower
51'-6"
SECTION A-A
SCALE 1" = 4'
2" x 2" 9/9 chain link mesh (See note)
Rock bolts and angle plates as reqd see note L
7-18 WF 60
4-10 WF 49
Fillet Curve, see Dwg 22/22
24" reinforced concrete water supply pipe
El 435.5 "A" line beyond
433.0' 24" CI water supply pipe
11-10 WF 49
Downstream protection liner
SECTION B-B
(TRANSITION DETAILS)
SCALE 1" = 4'
24" Adapter C.I. spigot to reinforced concrete pipe with rubber and steel joint
Sta 11+08 T
℄ 90° bend
El 427.46
Outlet works
Cap for future water supply connection
Top wall El 444.0
Top berm El 444.0
Conc fill
Stilling basin apron
6" min conc encasement
24" C.I. water supply pipe
℄ 22½° bend
℄ 45° bend
El 427.46
SECTION C-C
TYPICAL TUNNEL S
SCALE 1" = 5'
Rock bolts w plate and 2" x lengths may be dwg 10/5)
Pay lines not shown
Beam support member 6 WF 25
Varies 3'-0" (Sta 10+91 T) to 4'-6" (Sta 10+98 T)
24" CI water supply pipe
24" CI pipe
24" Adapter CI spigot to reinforced concrete pipe with rubber and steel joint
24" reinforced concrete pipe
24" reinforced conc water supply pipe
SECTION D-D
WATER SUPPLY-DETAILS AT

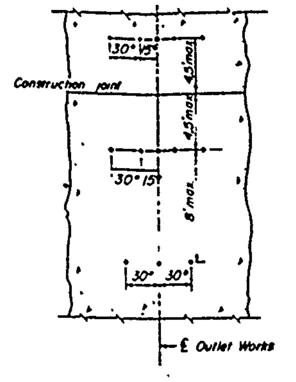

U S ARMY
TYPICAL TRANSITION SECTION
SHOWING BEAM SUPPORT
STA 4+25.50T TO STA 4+43 50T
SCALE 1" = 5'
TYPICAL TRANSITION SECTION
SHOWING BEAM SUPPORT
STA 4+46 50T TO STA 4+56 50T
SCALE 1" = 5'
Timber blocking
Allow clearance around reinforcing steel for bond
Conc brick blocking
Beam support 18 WF 60 @ 3' c.c
"B" Line
"A" Line (Timber blocking not permitted within this line)
Column support 10 WF 49 @ 3' c.c
Downstream protection liner not shown
8" clear
24" cast iron water supply pipe
Min 6" conc encasement
Concrete brick blocking as required
Timber blocking as required
10 WF 49 @ 3' c.c
24" cast iron water supply pipe
Same as tunnel grout hole arrangement see dwg 10/4
7/8" Rock bolts w/6"x6"x3/8" bearing plate and 2"x2"x1/8" washer, coupled lengths may be used (see Detail "A" dwg 10/5)
2"x2" No. 9 continuous chain link wire fabric
24" reinforced conc water supply pipe
Min 6" conc encasement
TYPICAL TUNNEL SECTION
SCALE 1" = 5'
2"x2" 9/9 continuous chain link wire fabric
℄ Outlet works
TUNNEL ROOF PLAN
SCALE 1" = 5'
Timber blocking as required
Conc brick blocking as required
"B" Line
"A" Line (Timber blocking not permitted within this line)
Beam support member 6WF 25 @ 3' c.c
Grout pad
24" reinforced conc water supply line
Min 6" conc encasement
TYPICAL TUNNEL SECTION
SHOWING BEAM SUPPORT
SEE DRAWING NO 2/2 FOR LOCATION
SCALE 1" = 5'
℄ Outlet works
Cap for future water supply connection
Top berm El 444.0
Top wall El 444.0
Conc fill
6" min conc encasement
24" C I water supply pipe
℄ 22 1/2° bend
℄ 45° bend El 427.46
SECTION C-C
Portal
Sta 11+00 T
℄ 50° bend
℄ Water supply pipe
℄ Outlet works
24" CI water supply pipe
Form conc fill to El 444.0
4 on 1
Berm width 19.10'
℄ 45° bend
6" min conc encasement
℄ 22 1/2° bend
Cap for future water supply connection
SECTIONAL PLAN
(TAKEN AT PIPE ℄)
NOTES
1. The location, number and spacing of tunnel rock bolts and the indicated placement of wire fabric are approximations only and are subject to change to meet conditions as encountered during construction. Actual placement of rock bolts and wire fabric shall be subject to the approval of the contracting officer.
2. Rock bolts and wire fabric shall be placed throughout the tunnel with the following exceptions:
(a) None in area containing steel rib supports
(b) None in tunnel sections where the contracting officer determines that rock is sufficiently stable to not require this support
(c) None in transition section between stations 4 + 23 T and 4 + 53 T
3. Payment will be made to the "B" line for excavation and concrete lining in tunnel and transition. No unexcavated or timber blocking will be permitted to remain within "A" line.
RECORD (AS BUILT) DRAWING
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT.
Pay lines not shown
Beam support member 6 WF 25
24" CI pipe
24" Adapter CI spigot to reinforced concrete pipe with rubber and steel joint
24" reinforced concrete pipe
24" CI water supply pipe
Top of wall El 444.0
℄ 4 beam supports @ 3' cc = 9'
Inv El 432.0
Stilling basin apron
Sta 11+08 T ℄ 90° bend El 427.46
Sta 10+98 T ℄ 11 1/4° bend El 427.50
Sta 10+91 T ℄ 11 1/4° bend El 429.07
6" conc. encasement
SECTION D-D
SECTIONAL ELEVATION
WATER SUPPLY - DETAILS AT DOWNSTREAM PORTAL
U S ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
COSSATOT RIVER, ARKANSAS
GILLHAM DAM
OUTLET WORKS
STRUCTURAL FOUNDATIONS EXCAVATION
TRANSITION AND TUNNEL SUPPORT PLAN
DEC 1964



TYPICAL TUNNEL HALF SECTION

GROUT HOLE ARRANGEMENT

SCALE 1" = 2'

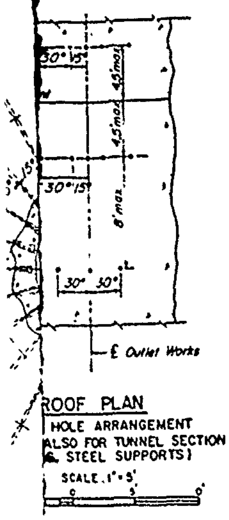


ROOF PLAN

GROUT HOLE ARRANGEMENT
(TYPICAL ALSO FOR TUNNEL SECTION REQUIRING STEEL SUPPORTS)

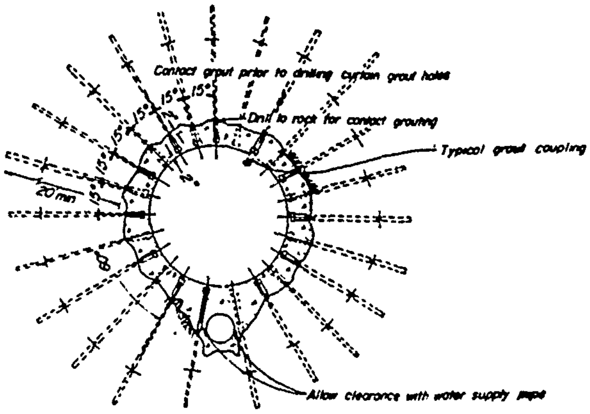
SCALE 1" = 5'

U S ARMY



ROOF PLAN
(HOLE ARRANGEMENT ALSO FOR TUNNEL SECTION & STEEL SUPPORTS)
SCALE 1"=5'

CU
STA 7
STA 8



CURTAIN GROUTING
STA 7+94.44 T (SHOWN SOLID)
STA 8+04.44 T (SHOWN DASHED)
SCALE 1"=5'

NOTES:
1. See Dwg No 10/3 for grout hole layout at transition
2. Additional grout connections may be required to meet conditions encountered during construction and shall be installed as directed by the contracting officer
3. Curtain grout holes will be drilled normal to ℄ grade of tunnel
4. All drilling and grouting details will be paid for at contract prices for items 24a thru 24l.

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

RECORD (AS BUILT) DRAWING

U S ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

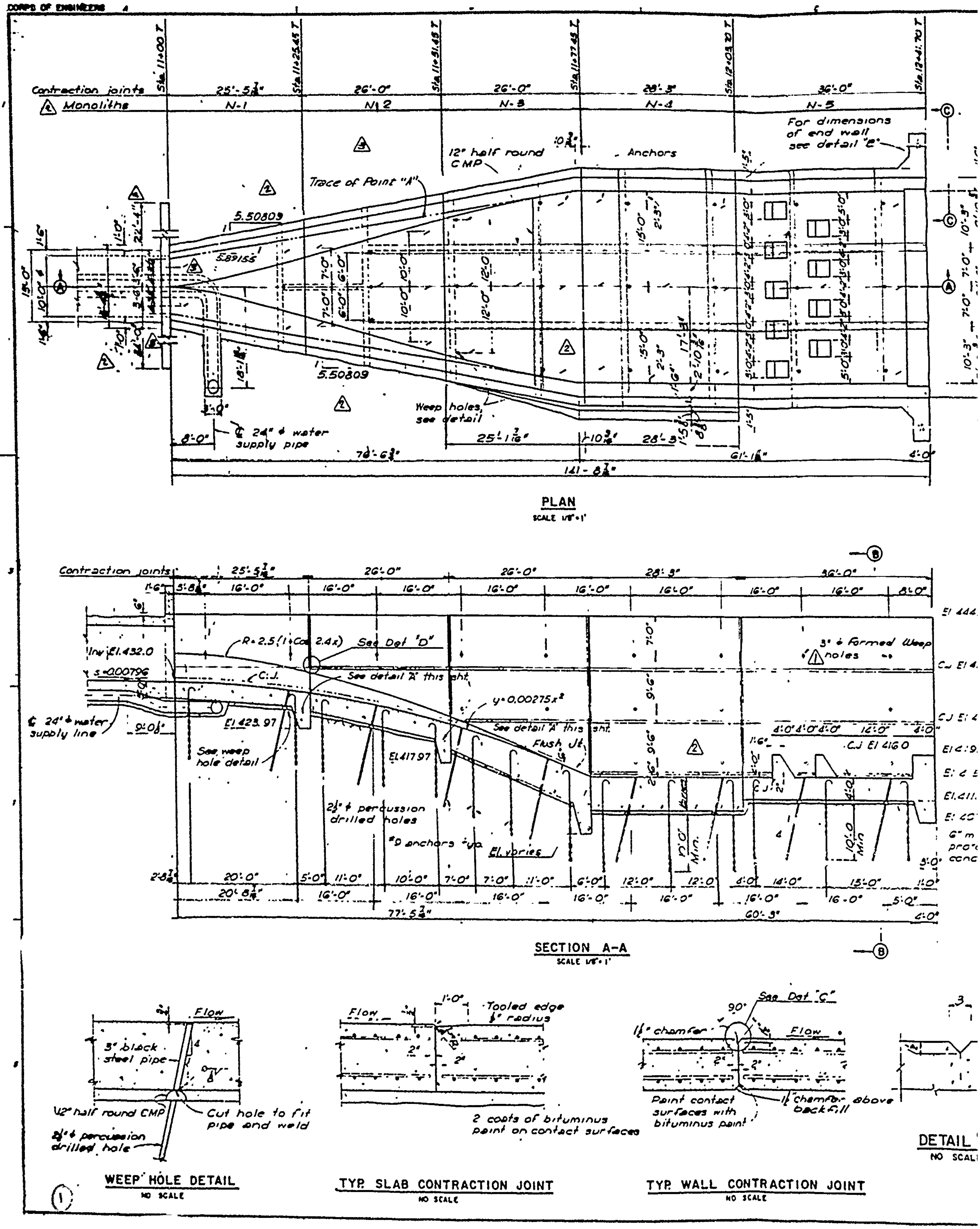
RED RIVER WATERSHED — COSSATOT RIVER ARKANSAS

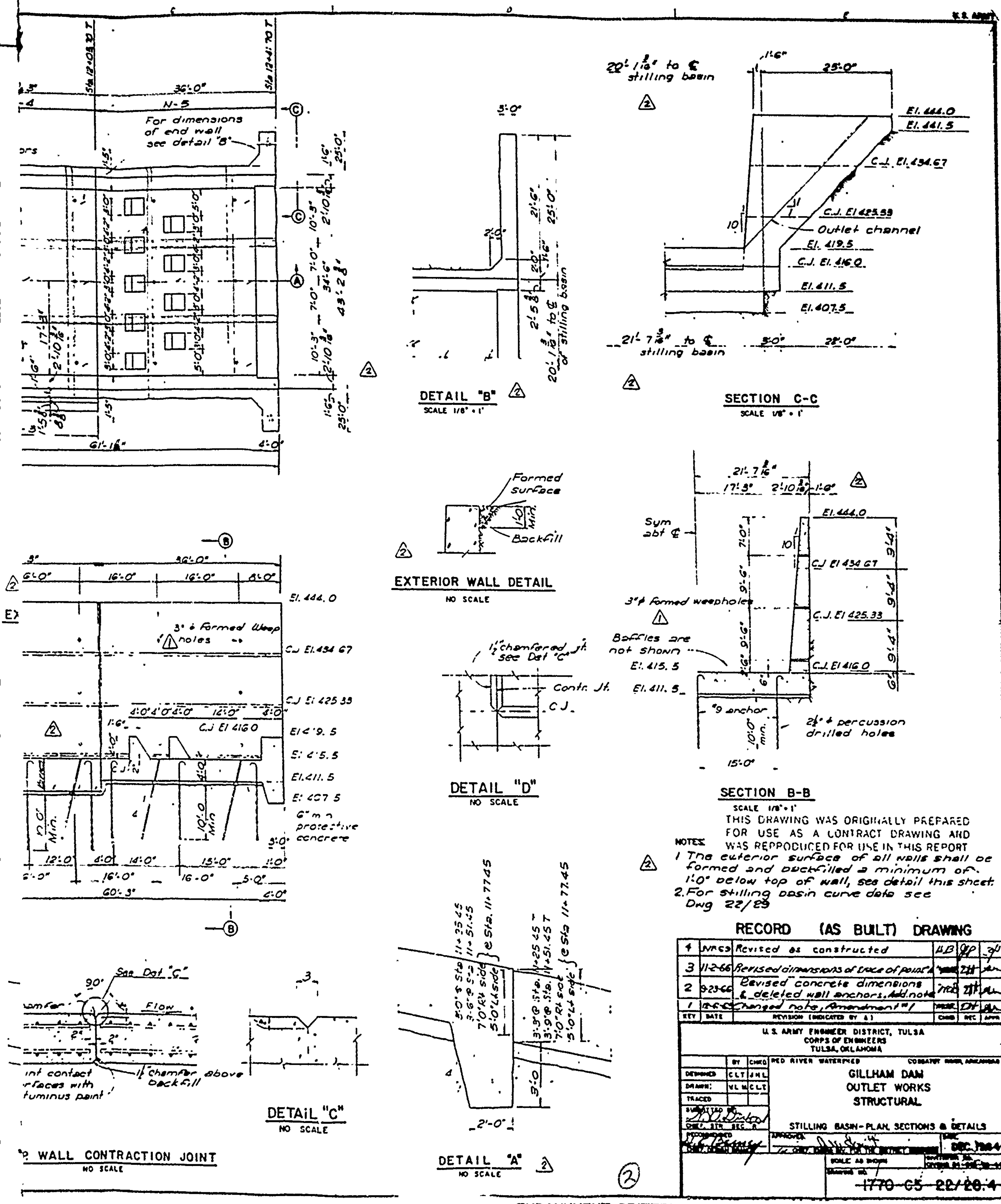
GILLHAM DAM
OUTLET WORKS
STRUCTURAL FOUNDATIONS EXCAVATION
TUNNEL AND TRANSITION GROUTING DETAILS

	BY	CHKD
DESIGNED	STG	LEJ
DRAWN	CDW	HJW
TRACED	PDM	HJW

SUBMITTED BY
RECOMMENDED
APPROVED
DATE DEC., 1964
SCALE AS SHOWN
DRAWING NO ~~1770 C5 10/4~~

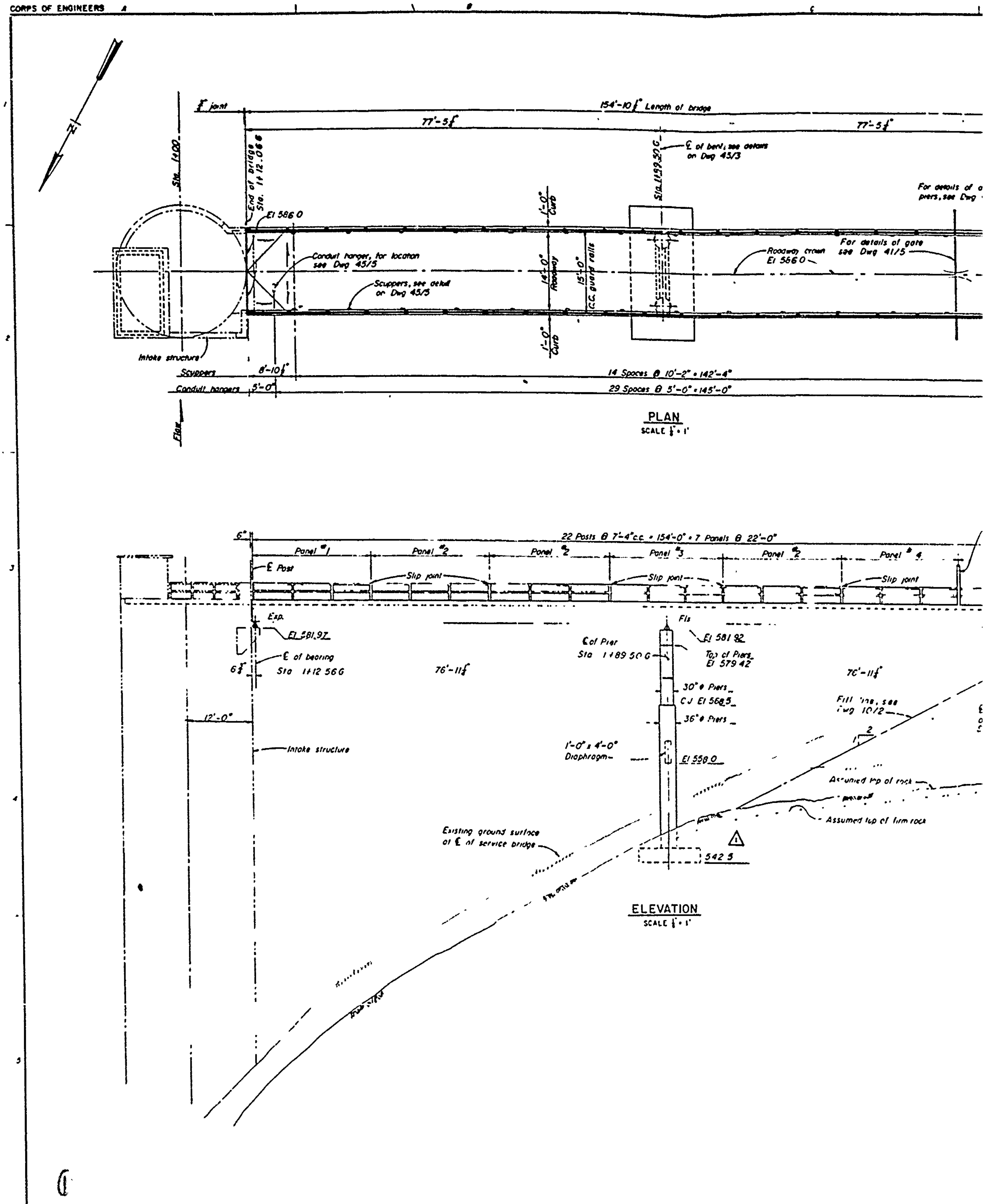
(3)

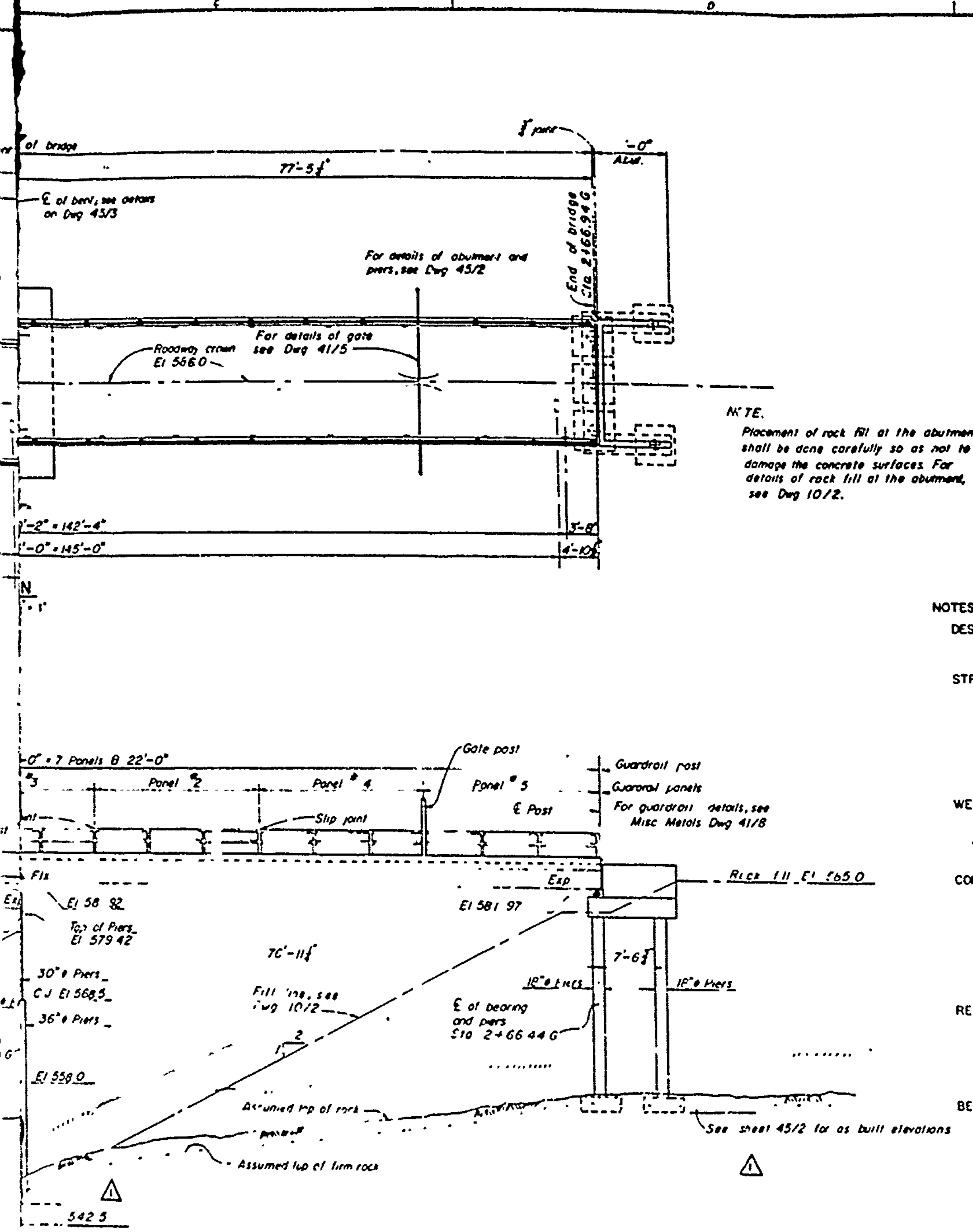

CORPS OF ENGINEERS
Contraction joints
Monoliths
N-1
N-2
N-3
N-4
N-5
25'-5 3/4"
26'-0"
26'-0"
28'-3"
36'-0"
For dimensions of end wall see detail "E"
12" half round CMP
Anchors
Trace of Point "A"
5.50809
5.89155
Weep holes, see detail
24" φ water supply pipe
78'-6 3/4"
25'-1 1/16"
28'-3"
61'-1 1/8"
141'-8 3/4"
4'-0"
8'-0"
3'-0"
PLAN
SCALE 1/8"=1'
Contraction joints
16'-0"
Inv El.432.0
s=0.00796
R=2.5(1+Cos 2.4x)
See Det 'D'
See detail 'A' this sht.
C.J.
y=0.00275x²
Flush Jt.
24" φ water supply line
El.423.97
El.417.97
See weep hole detail
2 1/2" φ percussion drilled holes
#9 anchors typ.
El. varies
3" φ formed weep holes
C.J El 416.0
10'-0" Min.
20'-0"
20'-8 3/4"
77'-5 3/4"
60'-3"
SECTION A-A
SCALE 1/8"=1'
Flow
3" black steel pipe
1/2" half round CMP
Cut hole to fit pipe and weld
2 1/2" φ percussion drilled hole
WEEP HOLE DETAIL
NO SCALE
Tooled edge 1/2" radius
2 coats of bituminus paint on contact surfaces
TYP. SLAB CONTRACTION JOINT
NO SCALE
See Det "C"
1 1/2" chamfer
Paint contact surfaces with bituminus paint
1 1/2" chamfer above backfill
TYP. WALL CONTRACTION JOINT
NO SCALE
DETAIL
NO SCALE



DETAIL "B"
SCALE 1/8" = 1'
SECTION C-C
SCALE 1/8" = 1'
EXTERIOR WALL DETAIL
NO SCALE
DETAIL "D"
NO SCALE
SECTION B-B
SCALE 1/8" = 1'
DETAIL "C"
NO SCALE
DETAIL "A"
NO SCALE
WALL CONTRACTION JOINT
NO SCALE
THIS DRAWING WAS ORIGINALLY PREPARED FOR USE AS A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT
NOTES
1 The exterior surface of all walls shall be formed and backfilled a minimum of 1'-0" below top of wall, see detail this sheet.
2. For stilling basin curve data see Dwg 22/29
RECORD (AS BUILT) DRAWING
4 Revised as constructed
3 11-2-66 Revised dimensions of trace of point
2 9-23-66 Revised concrete dimensions & deleted wall anchors. Add. note
1 Changed note, Amendment #1
U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA
RED RIVER WATERSHED
GILLHAM DAM
OUTLET WORKS
STRUCTURAL
STILLING BASIN-PLAN, SECTIONS & DETAILS
1770-65 22/26.4

CORPS OF ENGINEERS





NOTES:

DESIGN LOAD

Design load H{illegible} -S12-44 in accordance with A A S H.O 1961 specifications.

STRUCTURAL STEEL

All structural steel shall be ASTM A-36

All bolts shall be ¾" diameter ASTM A325 bolts Bolt holes shall be 13/16" diameter

Backfilling of the pier and abutment shall be completed before erecting any structural steel

All structural steel shall be erected before any of the deck is placed

WELDING

All welding shall conform to the current specifications for Welded Highway and Railway Bridges, by the American Welding Society

Beams shall be aligned and bolted in place before welding.

CONCRETE

All concrete shall have a 28 day strength of 3000 P.S.I.

The deck pouring sequence shall be followed as shown on Dwg 45/5.

All false work shall be removed before any of the deck is placed.

All exposed edges shall have a ¾" chamfer, unless otherwise noted.

The deck forms shall be set to take the dead load deflections into account, so that the roadway shall be to grade

REINFORCING STEEL

Reinforcing steel shall be intermediate grade billet steel, f_s = 20,000 P.S.I.

Reinforcing clearances unless otherwise noted are as follows:

Deck	1 inch
Pier & Abutment	3 inches

BEARING DEVICES

All bearing devices shall be fabricated with ASTM A-36 Structural Steel Rockers and bolsters shall be sufficiently true to form that they will fit machined surfaces of the pins without bending No machining or grinding of the rockers shall be permitted that will reduce the bearing surfaces on the pins to less than that shown on the drawings Masonry bearing plates shall be placed on three layers of twelve to fourteen ounce duck, each layer shall be thoroughly coated with red lead paint The masonry surfaces under the bearing plates shall be smooth and finished to grade with a carborundum brick Rocker fins on the expansion shoes shall be set vertical based on 70°F. For each 20°F temperature variation the rocker fins shall be rotated ⅛ inch at the beam flange The direction of rotation shall be such that expansion or contraction to 70°F will leave the rocker fins in a vertical position.

RECORD (AS BUILT) DRAWING

KEY	DATE	REVISION (INDICATED BY △)	CHKD	REC	APPR
1	[illegible]	Revised as constructed	[illegible]	[illegible]	[illegible]

U.S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER, ARKANSAS

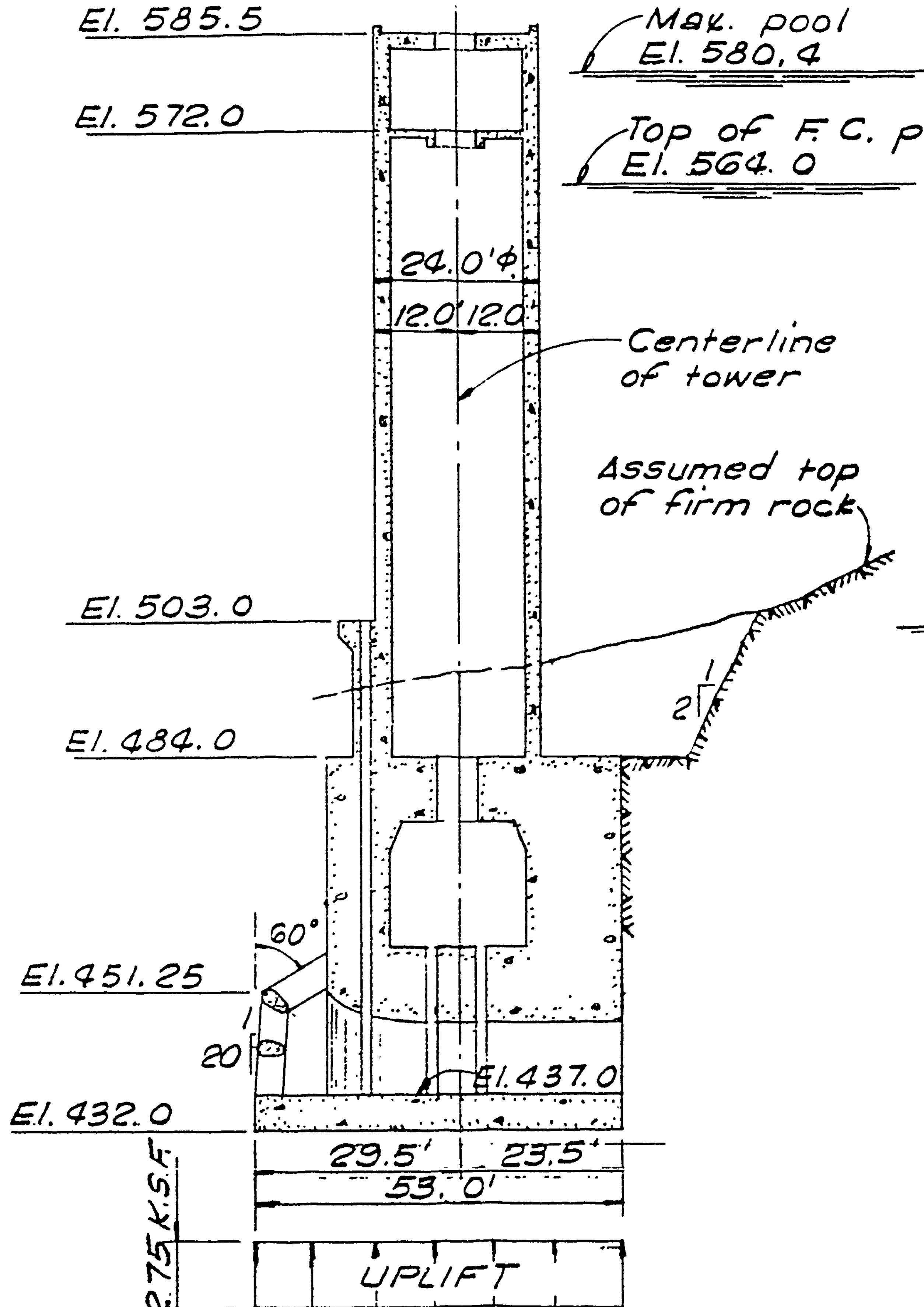
GILLHAM DAM
OUTLET WORKS
STRUCTURAL
SERVICE BRIDGE-PLAN AND ELEVATION

	BY	CHKD
DESIGNED	C.L.T.	M.B.C.
DRAWN	C.L.T.	M.L.R.
TRACED	P.D.G.	J.R.J.

SUBMITTED BY: [illegible]
CHIEF, STRUCT. SEC.
RECOMMENDED: [illegible]
CHIEF, DESIGN BRANCH
APPROVED: [illegible] FOR THE DISTRICT ENGINEER
DATE: DEC. 1964
SCALE: AS SHOWN
INVITATION NO: CIVENG-34-066-65-44
DRAWING NO: ~~1770-65-45/11~~

THIS DRAWING WAS ORIGINALLY PREPARED FOR USE IN A CONTRACT DRAWING AND WAS REPRODUCED FOR USE IN THIS REPORT

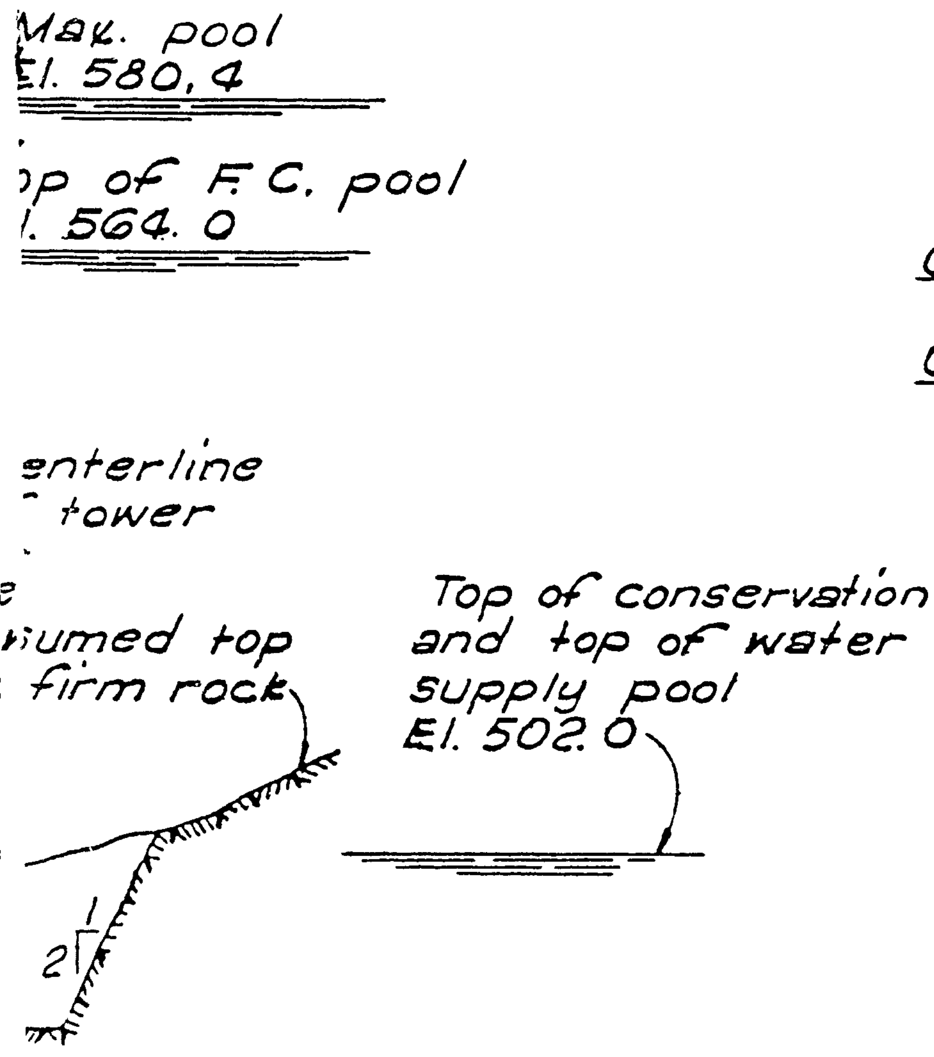
(2)



SECTION THRU TOWER

SCALE: 1" = 20'

BASE PR	
POINT	CA
A	+4
B	+5
C	+9
D	+4
E	+5
F	+5
G	+9



GILLHA

STAB

DESIGN LOAD ASS
AND BASE PRE

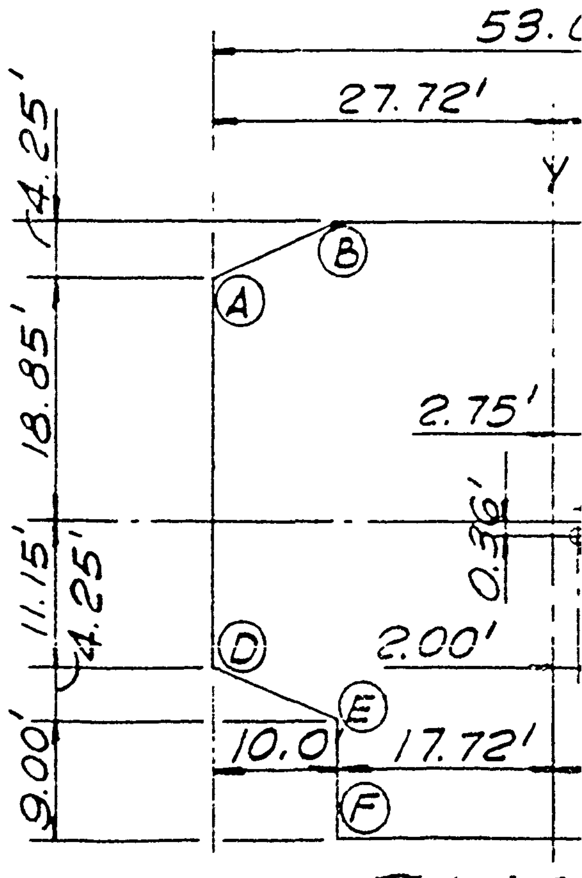
CASE I (Construction co
Concrete only.

CASE II Water to El. 580
service gate clos
log closed and
acting over tota

DESIGN LO

Concrete: Wc =
Water: W_1 =
P_1 =

BASE PRESSURES(KSF)		
POINT	CASE I	CASE II
A	+4.97	+1.84
B	+5.78	+2.16
C	+9.13	+3.71
D	+4.76	+2.11
E	+5.51	+2.51
F	+5.45	+2.59
G	+9.14	+4.14



PLAN O

SCALE:

R

GILLHAM GATE TOWER

STABILITY ANALYSIS

DESIGN LOAD ASSUMPTIONS AND BASE PRESSURES

CASE I (Construction condition)
Concrete only.

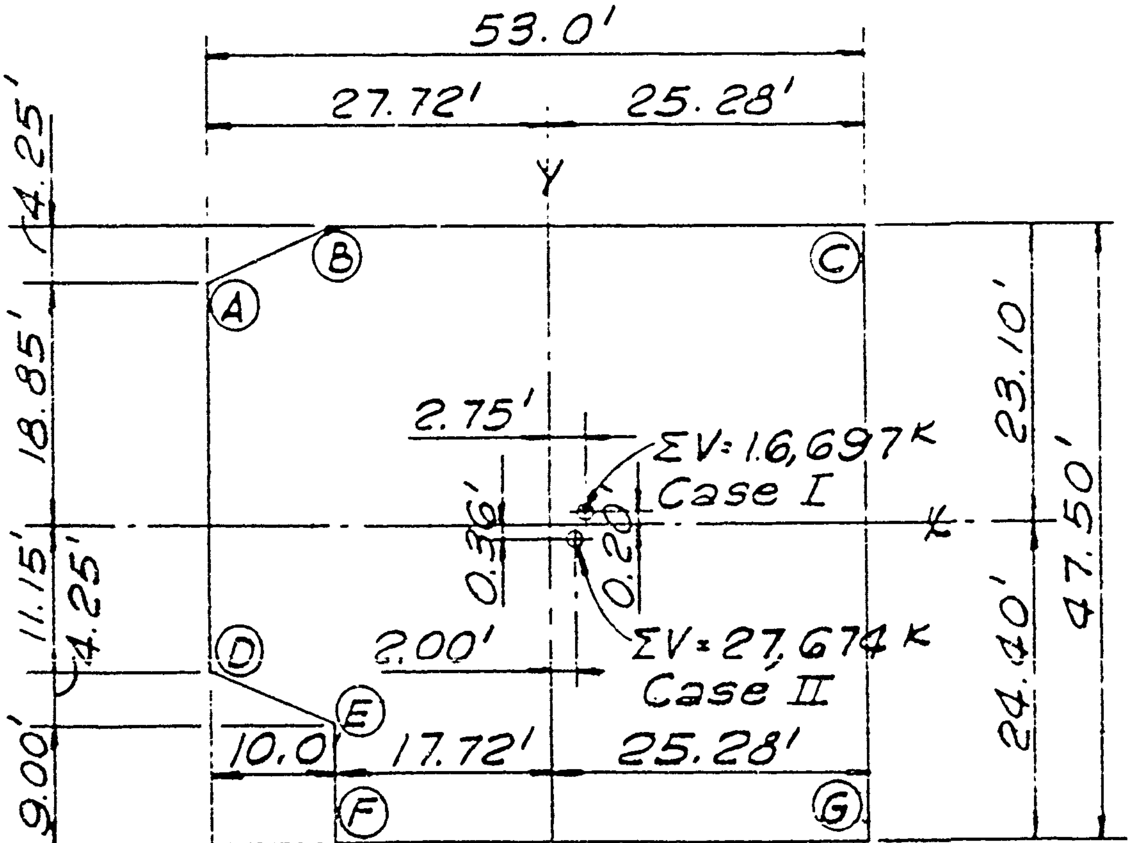
CASE II Water to El. 580.4 with right service gate closed and left stop log closed and with 100 % uplift acting over total base area.

nservation
of water
ool

DESIGN LOADS

Concrete: $W_c = 150$ #/C.F.
Water: $W_1 = 62.5$ #/C.F.
$P_1 = 62.5$ #/S.F.

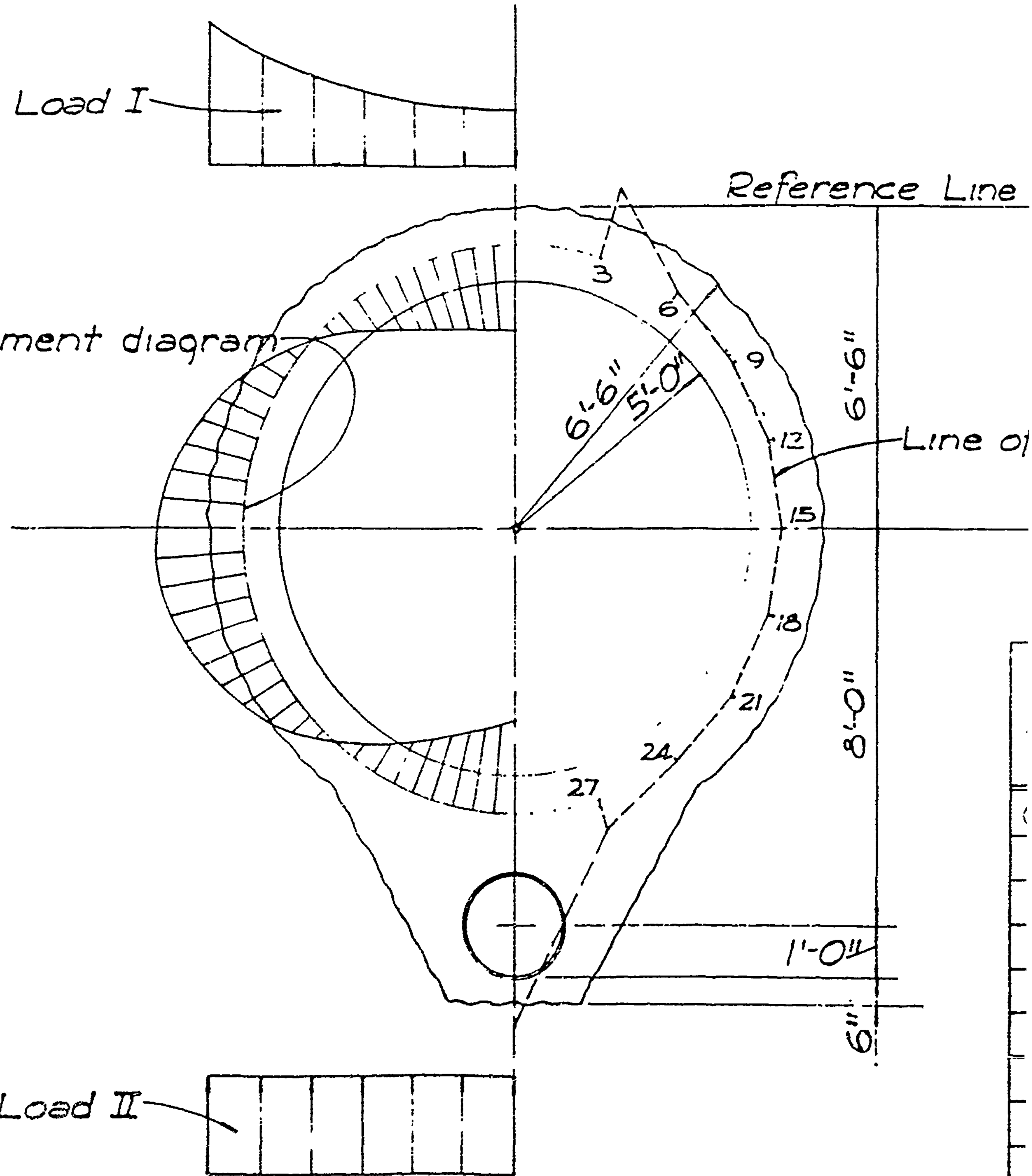
SSURES(KSF.)	
E I	CASE II
97	+1.84
78	+2.16
43	+3.71
76	+2.11
51	+2.51
45	+2.59
4	+4.14



PLAN OF BASE

SCALE: 1" = 15'

3



Load I = Rock load (from dropout)
Load II · Base Pressure

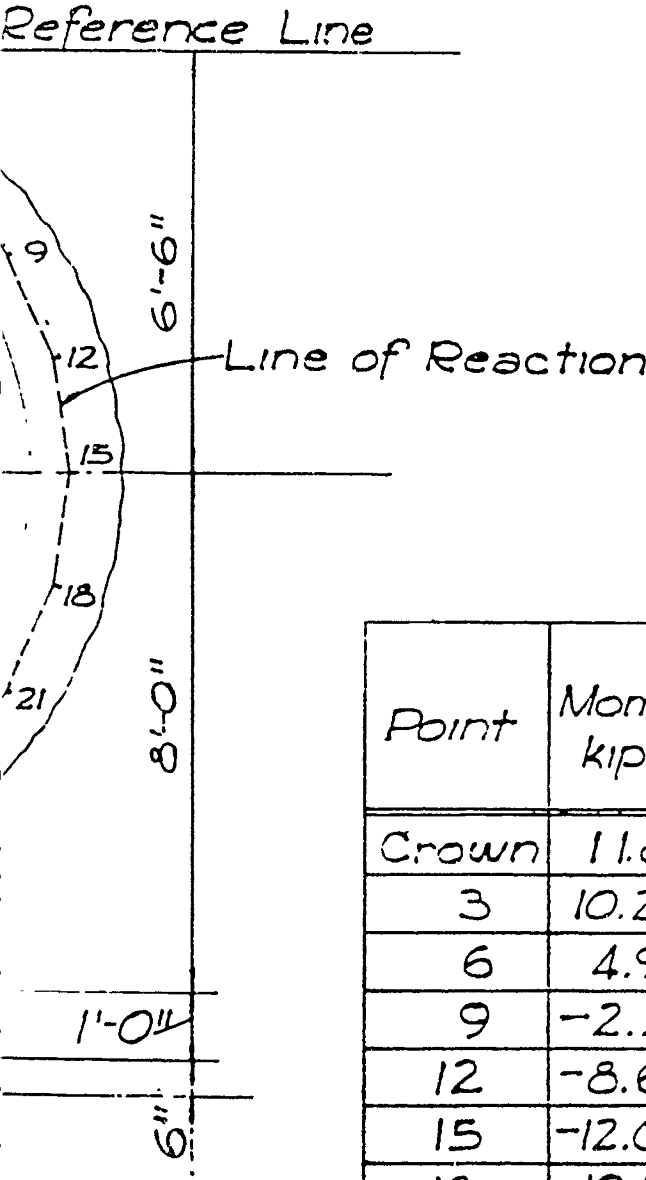
GILLHAM

10' ϕ TUNNEL

Dimension Scale 3' 0

Moment Scale 20 0

Line of Reaction Scale 40' 0



NOTES:

1. Analysis presented represent severe moment loading that anywhere in the zone the sectio
2. Section was also checked agai failure by low moment high thr
3. All loads are computed on basis of Computer Program Number
4. Minimum steel of #6@12" will b section where steel is not othe This includes longitudinal steel

Point	Moment kip-ft.	Shear kips	Thrust kips	As Req'd	Bars	As Supplied
Crown	11.81	0	- .216	0.606	#8@12	0.79
3	10.279	2.340	.534	.528	#8@12	0.79
6	4.992	3.838	2.522	.152	#6@12	0.44
9	-2.205	3.944	5.062	0	#6@12	0.44
12	-8.692	2.595	7.287	.180	#6@12	0.44
15	-12.037	.216	8.453	.309	#6@12	0.44
18	-10.823	-2.374	8.005	.263	#6@12	0.44
21	- 5.320	-4.095	6.003	.056	#6@12	0.44
24	2.377	-4.202	3.319	0	#6@12	0.44
27	9.215	-2.628	1.081	.402	#6@12	0.44
Invert	12.455	0	.216	0.639	#8@12	0.79

(2)

[illegible]

GILLHAM DAM
10'ϕ TUNNEL SECTION

Dimension Scale 3' 0 3' 6'

Moment Scale 20 0 20 40 K:FT.

Line of Reaction Scale 40' 0 20' 40'

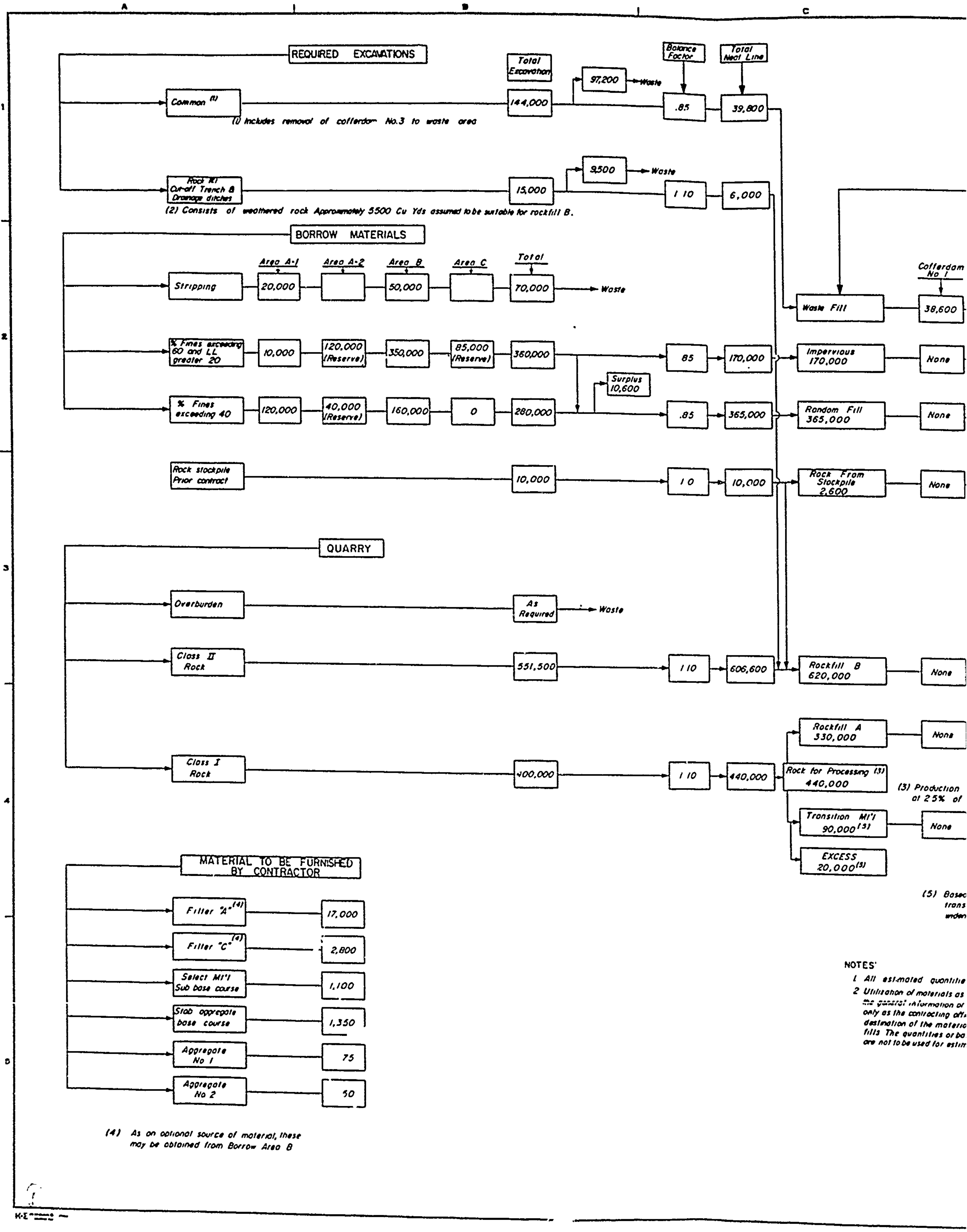
eaction

NOTES:

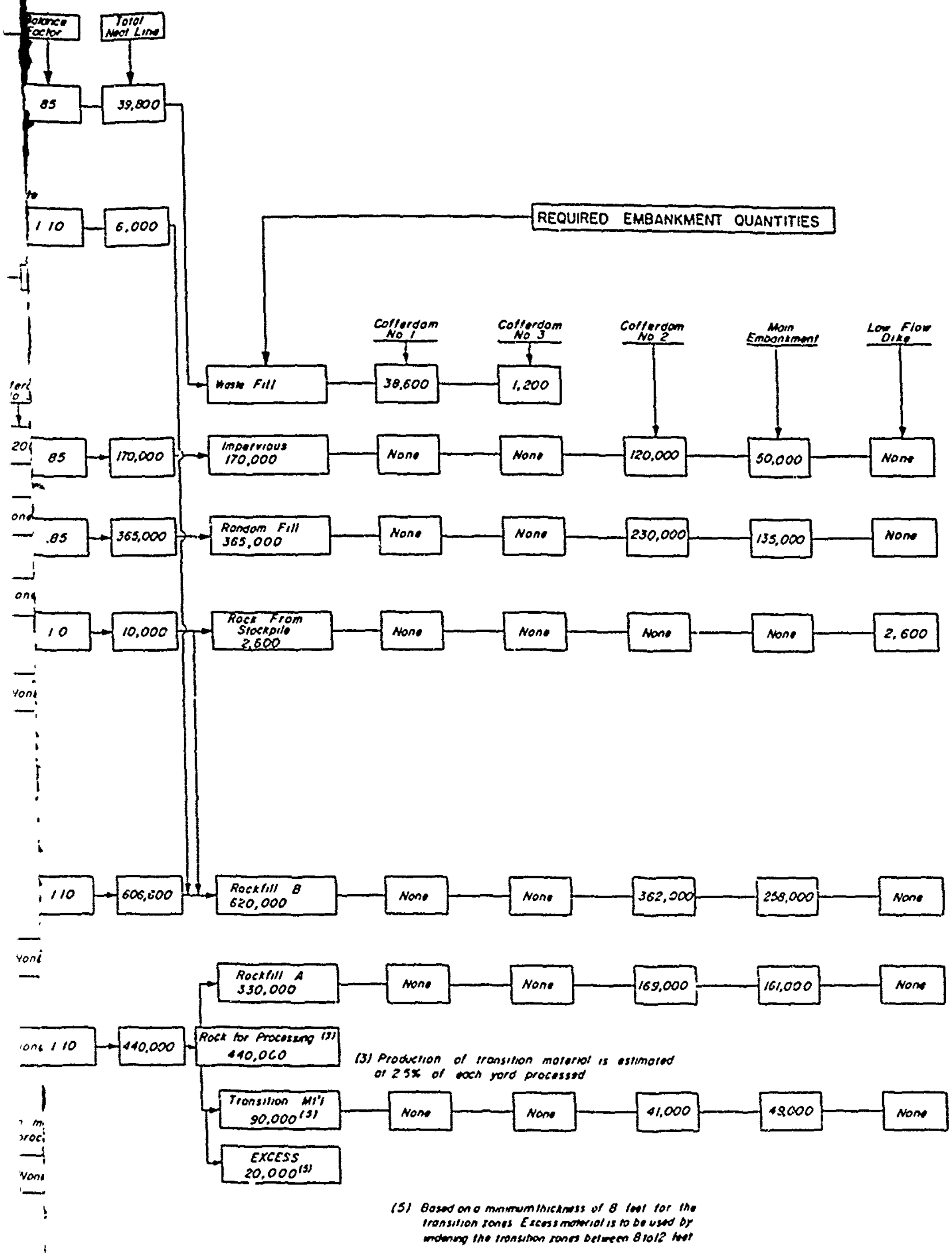
1. Analysis presented represents the most severe moment loading that occurs anywhere in the zone the section is to be used.
2. Section was also checked against compression failure by low moment high thrust loads.
3. All loads are computed on basis of equations of Computer Program Number 13-G1-G515.
4. Minimum steel of #6@12" will be used in section where steel is not otherwise shown. This includes longitudinal steel.

nt	Moment kip-ft.	Shear kips	Thrust kips	As Req'd	Bars	As Supplied	f'c 1800 psi allow.	v 127 psi allow.
wn	11.81	0	- .216	0.606	#8@12	0.79	595	0
3	10.279	2.340	.534	.528	#8@12	0.79	518	16
6	4.992	3.838	2.522	.152	#6@12	0.44	288	25
9	-2.205	3.944	5.062	[illegible]	#6@12	0.44	200	27
2	-8.692	2.595	7.287	[illegible]	#6@12	0.44	549	21
5	-12.037	.216	8.453	.309	#6@12	0.44	804	2
	-10.823	-2.374	8.005	.263	#6@12	0.44	712	-19
	- 5.320	-4.095	6.003	.056	#6@12	0.44	299	- 33
4	2.377	-4.202	3.319	0	#6@12	0.44	125	-30
7	9.215	-2.628	1.081	.402	#6@12	0.44	573	- 17
ert	12.455	0	.216	0.639	#8@12	0.79	628	0

(2)



REQUIRED EXCAVATIONS
Total Excavation
Balance Factor
Total Neat Line
Common (1)
144,000
97,200
Waste
.85
39,800
(1) Includes removal of cofferdam No.3 to waste area
Rock (2) Cut-off Trench & Drainage ditches
15,000
9,500
Waste
1.10
6,000
(2) Consists of weathered rock Approximately 5500 Cu Yds assumed to be suitable for rockfill B.
BORROW MATERIALS
Area A-1
Area A-2
Area B
Area C
Total
Stripping
20,000
50,000
70,000
Waste
% Fines exceeding 60 and LL greater 20
10,000
120,000 (Reserve)
350,000
85,000 (Reserve)
360,000
.85
170,000
Surplus 10,600
% Fines exceeding 40
120,000
40,000 (Reserve)
160,000
0
280,000
.85
365,000
Rock stockpile Prior contract
10,000
1.0
10,000
Waste Fill
Cofferdam No 1
38,600
Impervious 170,000
None
Random Fill 365,000
None
Rock From Stockpile 2,600
None
QUARRY
Overburden
As Required
Waste
Class II Rock
551,500
1.10
606,600
Rockfill B 620,000
None
Rockfill A 330,000
None
Class I Rock
400,000
1.10
440,000
Rock for Processing (3) 440,000
(3) Production of 25% of
Transition Mt'l 90,000 (5)
None
EXCESS 20,000 (5)
(5) Based ... trans... under...
MATERIAL TO BE FURNISHED BY CONTRACTOR
Filter "A" (4)
17,000
Filter "C" (4)
2,800
Select Mt'l Sub base course
1,100
Stab aggregate base course
1,350
Aggregate No 1
75
Aggregate No 2
50
(4) As an optional source of material, these may be obtained from Borrow Area B
NOTES:
1. All estimated quantitie...
2. Utilization of materials as ... the general information of ... only as the contracting offi... destination of the materia... fills. The quantities or ba... are not to be used for estim...



THIS DRAWING FOR INFORMATION ONLY

NOTES:

1. All estimated quantities are expressed in cubic yards
2. Utilization of materials as shown on this drawing is presented for the general information of the contractor and is to be considered only as the contracting officers interpretation as to the origin and destination of the materials necessary to construct the required fills. The quantities or balance factors shown on this drawing are not to be used for estimating or pay purposes

KEY	DATE	CHANGE	REVISION (INDICATED BY Δ)	APPR

U. S. ARMY ENGINEER DISTRICT, TULSA
CORPS OF ENGINEERS
TULSA, OKLAHOMA

RED RIVER WATERSHED — COSSATOT RIVER ARKANSAS

GILLHAM DAM
EMBANKMENT
FOUNDATION EXPLORATION
MATERIALS USAGE CHART

	BY	CHKD
DESIGNED	DEW	
DRAWN	RDM	DEW
TRACED	RDM	DEW

SUBMITTED BY: CHIEF, SOILS MECH SEC
RECOMMENDED: CHIEF, FDN & MATLS BR
APPROVED: CHIEF, ENGR DIV FOR THE DISTRICT ENGINEER
DATE: APR 1971
SCALE: AS SHOWN
DRAWING NO: 1770-DM9-98/13

END

FILMED

DATE: 9-90

DTIC