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SCHNABEL ENGINEERING ASSOCIATES RICHMOND VA
NATIONAL DAM SAFETY PROGRAM, LAKE CAROLINE DAM (INVENTORY NUMBER--ETC(U)
APR 81 R E MARTIN, C S ANDERSON, J G STARR

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ROANOKE RIVER DAM

Name Of Dam:

LAKE CHARLES DAM

Location:

CHARLES COUNTY, VIRGINIA

Inventory Number:

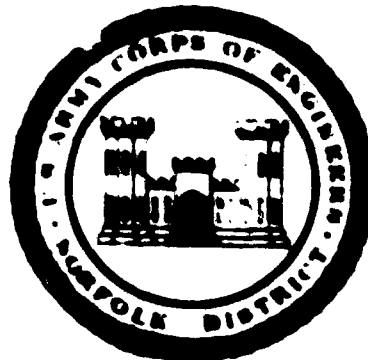
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PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

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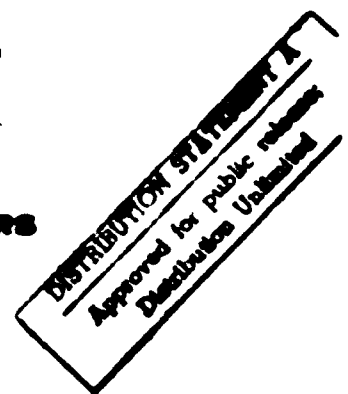


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PREPARED FOR
NORFOLK DISTRICT CORPS OF ENGINEERS
603 FRONT STREET
NORFOLK, VIRGINIA 23510

BY

SCHWAB ENGINEERING ASSOCIATES, P.C./
J. K. TIDWELL AND ASSOCIATES, INC.



APRIL 1981

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I Inspection is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspection. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

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MATTAPONI RIVER BASIN

NAME OF DAM: LAKE CAROLINE DAM
LOCATION: CAROLINE COUNTY
INVENTORY NUMBER: VA. NO. 03324

④ PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Lake Caroline Dam (Inventory Number
VA 03324), Mattaponi River Basin,
Caroline County, Virginia. Phase I
Inspection Report.

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② Final rept.

(15) DACW65-81-D-0020

BY

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF DAM

Name of Dam: Lake Caroline Dam
State: Virginia
Location: Caroline County
USGS QUAD Sheet: Hewlett, Virginia
Coordinates: Lat 37° 59.2' Long 77° 30.4'
Date of Inspection: April 20, 1981

Lake Caroline Dam is a homogeneous earthfill structure about 1425 ft long and 48 ft high. The principal spillway consists of a 400 ft long reinforced concrete overflow weir. The weir overflows onto a 20 ft wide concrete apron 1.5-2.0 ft below the weir crest. The concrete apron is also used as a roadway for access across the dam. The dam is an intermediate size structure and is assigned a significant hazard classification. The dam is located on Stevens Mill Run, in Caroline County, Virginia. The lake is used for water supply and recreation, and is owned and maintained by the Lake Caroline Association.

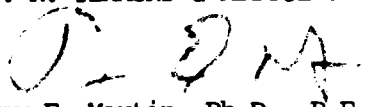
Based on the criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the appropriate Spillway Design Flood (SDF) for the dam is the $\frac{1}{2}$ PMF. The spillway will pass 100 percent of the Probable Maximum Flood (PMF) without overtopping. The spillway is rated adequate.

The visual inspection did not reveal any problems which would require immediate attention. The dam is considered stable and a stability analysis is not required. An emergency operation and warning

plan should be developed. Furthermore, a staff gage should be installed to monitor water levels. Small sapplings growing in the riprap on the upstream slope should be removed. Bare areas present along the downstream toe and areas of sparse vegetation on the downstream slope should be reseeded. Seepage along the downstream toe should be monitored during routine maintenance. It is also recommended that attempts be made to halt shoreline erosion in order to prevent sediment buildup in the lake. For monitoring purposes it is recommended that the downstream beaver dam(s) be removed, so as to prevent submergence of drain outlets.

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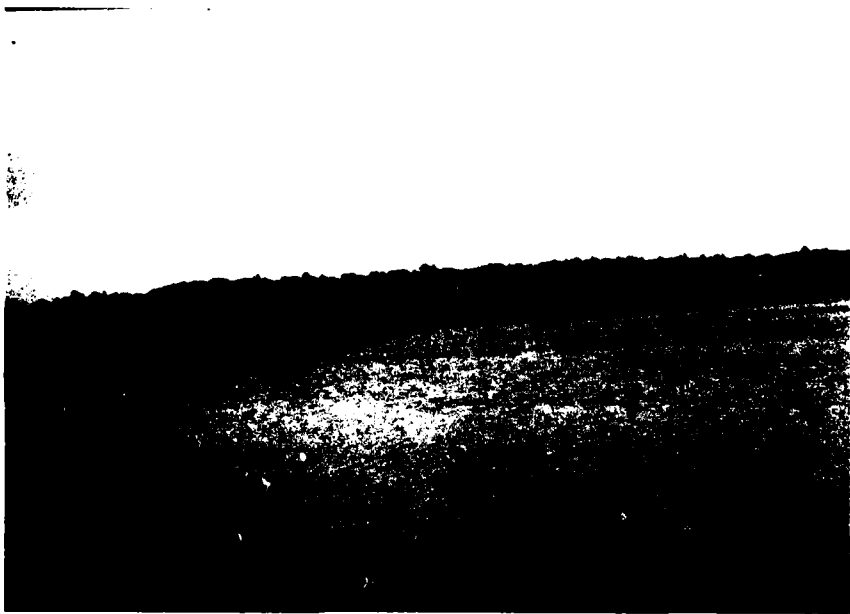
Original signed by
JACK G. STARR

Jack G. Starr
Chief, Engineering Division

Date: **SEP 2 1981**



Lake Caroline



Dam

Overview Photographs

SECTION I - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspection of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (see Reference 1, Appendix VI). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Caroline Lake Dam is a homogeneous earthfill structure approximately 1425 ft long and 48 ft high.* The crest of the dam is 21 ft wide, and side slopes are approximately 3 horizontal to 1 vertical (3H:1V) on the upstream slope, and 2.5H:1V on the downstream slopes of the dam. Field measurements indicate the downstream slope approaches 2½H:1V locally. A 7 ft wide berm is shown on drawings at elevation 194 msl along the upstream slope. The crest of the dam is at elevation 210 msl.

* Height is measured from the top of the dam to the downstream toe at the centerline of the stream.

The dam is keyed into the foundation and there is a rock toe drain. An internal drainage system was not provided. Existing vegetation on the embankment slopes and riprap along the upstream slope at normal pool levels provide slope protection.

The principal spillway consists of a 400 ft long rectangular shaped reinforced concrete overflow weir. The weir discharges onto a 20 ft wide concrete apron 1.5-2.0 ft below the weir crest. The spillway apron also serves as a roadway across the dam. The weir crest elevation is 198 msl. An 8 ft wide low flow weir is located at the center of the principal spillway at elevation 197.75 msl.

The discharge channel below the weir apron is a grouted riprap channel at a 7.5% slope. The channel width varies from 400 ft at the weir to 300 ft at the lower portion where it discharges to Stevens Mill Run.

A 24 inch diameter butterfly valve which is used to drain the lake is located approximately 150 ft upstream of the dam centerline approximately 800 ft from the right abutment with an intake elevation of 170 msl. The drain pipe is a 324 ft long, 24 inch diameter ductile iron pipe with anti-seep collars at 18 ft intervals. The invert elevation at the outlet structure is 162.5 msl. (See Plate 4, Appendix I.)

A water supply intake pipe, located approximately 700 ft from the right abutment, passes through the embankment and discharges into the water treatment plant immediately downstream of the dam. The water supply pipe is 8 inch ductile iron with anti-seep collars 18 ft on center. The water supply is controlled by an eight inch gate valve at an invert elevation of 170 msl.

1.2.2 Location: Lake Caroline Dam is located on Stevens Mill Run approximately 1½ miles south of Ladysmith, Virginia on U. S. Route 1. (See Plate 1, Appendix I.)

1.2.3 Size Classification: The dam is classified as an "intermediate" size structure based on its height and maximum lake storage potential.

1.2.4 Hazard Classification: The dam is located in a small community, and based upon the proximity of the water treatment plant located immediately downstream, the dam is assigned a "significant" hazard classification. The hazard classification used to categorize a dam is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: The dam is owned and maintained by the Lake Caroline Association.

1.2.6 Purpose: Provide water supply and recreation to the Lake Caroline community.

1.2.7 Design and Construction History: The dam is designed and constructed under the supervision of American Realty Service Corporation. The structure was constructed by Bailey and Associates and completed in 1968.

1.2.8 Normal Operation Procedures: The principal spillway is ungated, therefore, water rising above the crest of the weir is automatically discharged downstream. Normal pool is maintained at elevation 198 msl at the crest of the weir. The 24 inch diameter valve at intake elevation 170 msl is manually operated and is used to

lower the lake below normal pool. The 8 inch diameter valve at intake elevation 171 msl is manually operated and is used as a secondary control on the water supply intake.

1.3 Pertinent Data:

1.3.1 Drainage Area: The drainage area is 9.6 square miles.

1.3.2 Discharge at Dam Site: According to Mr. R. J. Miller, the maximum known flood at the dam site occurred in August, 1969 as a result of Hurricane Camille. The maximum pool rise was 7 ft (elevation 205). This corresponds to an approximate discharge of 25,928 CFS.

Principal Spillway Discharge:

Pool Elevation at Crest of Dam (elevation 210) 58,197 CFS

1.3.3 Dam and Reservoir Data: See Table 1.1, below:

Table 1.1 - DAM AND RESERVOIR DATA

Item	Reservoir				
	Elevation feet msl	Area Acres	Volume Acre Feet	Storage	
				Watershed Inches	Length Miles
Crest of Dam	210	586	8265	16.2	2.5
Principal Spillway Crest	198	273	2821	5.5	2.3
Streambed at Down- stream Toe of Dam	162	-	-	-	-

SECTION 2 - ENGINEERING DATA

2.1 Design: The dam was designed and constructed under the supervision of American Realty Service Corporation, Memphis, Tennessee. The embankment was initially designed by Robert D. Sayre and Associates, Richmond, Virginia and later modified by G. K. Jewell and Associates, Columbus, Ohio prior to its construction.

A subsurface investigation was conducted at the site by Mr. Robert D. Sayre, P.E., Richmond, Virginia in July, 1968, for the design of the project. The purpose of the investigation was to determine the subsurface soil and rock conditions for the dam embankment and spillway. A subsurface profile is shown on Plate 2, Appendix I, but the report was not available for review. Additional geotechnical engineering analyses were performed by G. K. Jewell and Associates, Columbus, Ohio to analyze the soil conditions and stability for a modified embankment section. This report and the soils laboratory test results are included as Appendix IV.

The dam was originally designed as a homogeneous, compacted earth fill embankment with a 20 ft wide cutoff trench approximately 65 ft upstream from the embankment center line, a combined principal-emergency spillway at the left abutment, and a blanket drain below the toe. The modified embankment design shown on Plate 12, Appendix IV eliminated the blanket drain and recommended a rock toe. Upstream and downstream slopes were designed at 3H:1V and 2.5H:1V, respectively with a 7 ft wide upstream berm at elevation 194 msl, 4 ft below the normal pool level. Details are provided on Plate 3, Appendix I.

A review of the design data indicates the dam was founded on overburden with the cutoff trench excavated to fresh rock (non-rippable rock). No permeability test data was included with the information reviewed. Details of the cutoff trench are provided on Plates 2 and 3 of Appendix I. No internal drainage system was indicated in the design data reviewed. The spillway is a combined principal-emergency spillway between Stations 14+00 and 20+00 at the left abutment. The spillway is concrete lined over the crest with upstream and downstream channels lined with riprap. Details of the spillway are shown on Plates 2, 5 and 6, Appendix I.

The laboratory test data describing the engineering properties of the borrow materials from the emergency spillway area which were used to construct the embankment are included in Appendix IV, Plates 1 through Plate 11. The shear strength parameters of this material determined by remolded, unconsolidated-undrained and remolded saturated consolidated-undrained triaxial compression tests are as follows:

<u>Borrow Source</u>	<u>Soil</u>	<u>Shear Strength Parameters</u>	
		<u>Friction Angle</u>	<u>Cohesion</u>
Emergency spillway	SW - SC	$\phi = 17^{\circ}$	1200 psf(1)
Emergency spillway	SW - SC	$\phi = 21^{\circ}$	1200 psf(2)
Emergency spillway	SW - SC	$\phi' = 38^{\circ}$	0 psf(2)

- (1) Remolded Unconsolidated-Undrained Triaxial Compression Test (UU)
- (2) Remolded, Saturated, Consolidated Undrained Triaxial Compression Tests (CU)

A stability analysis was performed on the dam section by G. K. Jewell and Associates and the results are included in Appendix IV. The conditions analyzed were: as-built (or end of construction case), steady state seepage for the downstream slope, and rapid drawdown for the upstream slope. Minimum values of strength parameters from the laboratory test results were used for the stability analysis. Effective strength values obtained from the CU test were used for the analysis of rapid drawdown and steady state seepage conditions and values from unconfined compression tests and UU tests were used for the analysis of the as-built condition. The results of the stability analysis are:

<u>Condition</u>	<u>Factor of Safety</u>
As-built	3.0 [†]
Steady State Seepage	1.3
Rapid Breakdown	1.25

The specifications for Lake Caroline Dam required that the borrow materials to be used for embankment construction have a Plasticity Index not less than 10 percent, a maximum dry unit weight greater than 110 pcf and particle sizes less than 6 inches in diameter. The selection of suitable borrow materials was to be under the direction of the engineer at all times. Soils in the proposed emergency spillway borrow area were reported to be weathered gneiss which after soaking for 24 hours, classified as an MH material according to the Unified Soil Classification System, ASTM D-2487. The material in the cutoff trench and embankment

was to be compacted to 97 percent of Standard Proctor maximum dry density according to ASTM D-698. Before compaction of the embankment fill, the moisture content was to be brought to plus or minus 2 percent of the optimum moisture content. Fill material was to be placed in successive lifts not exceeding 6 inches in compacted thickness.

The spillway was designed as an overflow weir structure consisting of reinforced concrete weir, a reinforced concrete overflow apron below the weir and a 400 ft wide discharge channel with a plunge pool at the end of the channel. The discharge channel was designed with a 12 ft wide concrete channel along the centerline with an earth overflow channel.

The spillway apron was designed as a roadway for access across the dam, and the spillway was designed to accommodate the Probable Maximum Flood with a 4 ft freeboard. Details of the spillway are presented on Plates 5 and 6 of Appendix I.

2.2 Construction: The embankment was constructed by Bailey and Associates of Stafford, Virginia. No construction records are available for this structure. Field inspection was under the direction of Robert D. Sayre and Associates, Richmond, Virginia. According to Mr. Sayre, the only construction problem he could remember was some difficulty in controlling water within the cutoff trench in order to obtain the specified compaction.

2.3 Evaluation: Design drawings are representative of the structure, however, hydrologic and hydraulic calculations were not available for evaluation. There is sufficient information to evaluate embankment stability and the foundation conditions.

SECTION 3 - VISUAL INSPECTION

3.1 Findings: At the time of inspection, the dam was in good condition. Field observations are outlined in Appendix III.

3.1.1 General: An inspection was made on April 20, 1981. The weather was partly cloudy and windy with a temperature of 65° F. The pool and tailwater levels at the time of the inspection were 198 and 162.5 msl, respectively, which corresponds to normal pool and tailwater elevations. Ground conditions were damp at the time of inspection as a result of early morning light rain. No previous inspection reports were available.

3.1.2 Dam and Spillway: The upstream slope was moderately vegetated with grass while the downstream slope was sparsely vegetated with grass and includes some bare areas. Scattered small sapplings were growing between the riprap. The crest of the dam was occupied by a paved road used for access to residences around the lake shore. The road appeared to be in good condition. Field measurements indicate the upstream slope to be 3H:1V and the downstream slope to be 2½H:1V at the center of the dam. Riprap, consisting of 1 to 3 ft fresh granite gneiss boulders, placed on the upstream slope appears to be in good condition. The riprap is visible below pool level and extends 5 to 7 ft above pool level on the upstream slope. The embankment appears to be constructed of residual material consisting of silty sand and clayey silty sand mixtures visually classifying SM to SC in accordance with the Unified Soils Classification System. No surface erosion was observed on the embankment slopes except for minor surface washing at scattered locations along the downstream slope.

Saturated or wet areas were encountered along the toe of the down-

stream slope beginning about 455 ft to the right of the principal spillway and extending an additional 305 ft. Some discoloration and flow estimated at 1 to 2 gpm were observed originating in an eroded V-notch at the extreme right side of the saturated area. This is shown in the Field Sketch, Sheet 1 in Appendix III. The bottom 5 ft[±] of the downstream slope is essentially void of vegetation and is very moist, extending from the V-notch to a point 325 ft[±] left of the notch. No sloughing or bulging of the slope was observed in this area.

The abutments were well vegetated and only minor erosion was observed on the upstream right abutment. Surface soils in the surrounding area included fine to medium clayey sands and sandy clays.

An overflow spillway and apron are located at the left abutment and consist of a 400 ft wide concrete weir with a 8 ft wide by 3 inch deep low flow notch. No signs of deterioration were noted and the concrete appeared to be in good condition. There was no separate approach channel to the spillway. The reservoir area upstream of the weir was riprap lined and appeared to be in good condition. The discharge channel is approximately 300 ft wide and slopes at about 7.5 percent. Immediately downstream from the weir, the discharge channel is occupied by the access road which crosses the dam crest. The road is concrete through the spillway area and bordered by a cable guardrail along the downstream edge. The discharge channel is lined with stone riprap and grouted along the center portion below the roadway. Evidence of severe erosion during Hurricane Camille is still visible in the lower reaches of the discharge area. It was reported that 11 ft deep gullies were carved into the discharge channel below the concrete section. These have been backfilled with riprap and concrete slurry and appeared stable.

No toe drain outlets were observed because water was backed up into the plunge pool area from obstructions downstream. The lake drain discharge pipe was also submerged and not observed. The emergency gate consists of a 24 inch butterfly valve. An 8 inch discharge pipe for an existing water treatment plant is located approximately 100 ft to the right from the lake drain. It is controlled by an 8 inch gate valve. The gate and butterfly valves on the water intake structure and drain facility were in good operating condition according to Mr. Miller.

3.1.3 Reservoir Area: The reservoir area was free of debris and the perimeter consisted of lawn area and wooded sections. (Overview Photograph, Page 3). Some shoreline erosion was observed near the right end of the dam. The reservoir is located in a natural valley with side slopes at approximately 10H:1V. No sediment build-up was observed.

3.1.4 Downstream Area: The downstream channel is located in a heavily wooded flood plain with 5H:1V side slopes above the channel banks (Photograph No. 5, Appendix II). The channel is approximately 2 ft deep with 1H:1V side slopes. A water treatment facility for the Lake Caroline community and U. S. Route 1 are both located immediately below the dam.

3.1.5 Instrumentation: No instrumentation (monuments, observation wells, piezometers, etc.) was encountered for the structure. A staff gage consisting of painted numbers on the lake drain intake structure was no longer visible because of weathering of the paint.

3.2 Evaluation: Overall, the dam was in good condition at the time of the inspection.

3.2.1 Dam and Spillway: The vegetative cover on the abutments and upstream embankment slopes appeared well maintained. The small

sapplings growing in the riprap on the upstream slope should be removed. The observed bare areas along the downstream toe and areas of sparse vegetation on the downstream slope should be seeded. Seepage at the toe area is apparently the result of a properly functioning rock toe drain. It is recommended that this seepage be monitored during routine maintenance to verify that detrimental surface erosion has not developed along the downstream toe. If such conditions should develop, a Professional Engineer with an expertise in Geotechnical Engineering should be contacted to evaluate the problem and make recommendations for required corrective measures. The minor surface erosion on the right upstream abutment contact does not require any attention. The overflow spillway is functioning well. A staff gage should be installed to monitor water levels. For monitoring purposes it is recommended that the downstream beaver dam(s) be removed, so as to prevent the submergence of drain outlets.

3.2.2 Reservoir Area: Shoreline erosion should be halted to prevent sediment buildup.

3.2.3 Downstream Area: A breach in the dam during extreme flooding could damage the water treatment facility and U. S. Route 1.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: The normal storage pool is elevation 198 msl or 3 inches above the crest of the low flow weir. The lake provides recreation and water supply as its principal uses. Water passes automatically through the spillway as the water level in the reservoir rises above the spillway crest. A 24 inch butterfly valve and discharge pipe are provided to drawdown the reservoir below normal pool.

4.2 Maintenance of Dam and Appurtenances: Maintenance is the responsibility of the owner. Maintenance consists of inspection, debris removal, mowing of vegetative cover and repair. Maintenance is routinely performed.

4.3 Warning System: At the present time, there is no warning system or evacuation plan for the dam.

4.4 Evaluation: The dam and appurtenances are in good operating condition, and maintenance of the dam appeared to be adequate.

An emergency operation and warning plan should be developed. It is recommended that a formal emergency procedure be prepared and furnished to all operating personnel. This should include:

- a. How to operate the dam during an emergency.
- b. Who to notify, including public officials, in case evacuation from the downstream area is necessary.

SECTION 5 - HYDRAULICS/HYDROLOGIC DATA

5.1 Design: Lake Caroline Dam was designed as a single purpose dam. Hydrologic and hydraulic data were not available.

5.2 Hydrologic Records: There are no records available.

5.3 Flood Experience: According to Mr. R. J. Miller, the flood of August 1969 resulting from Hurricane Camille created an increase of 7 ft[±] above normal pool (elevation 205 msl[±]).

5.4 Flood Potentials: In accordance with the established guidelines, the Spillway Design Flood is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region), or fractions thereof. The Probable Maximum Flood (PMF) and $\frac{1}{2}$ PMF and 100 year flood hydrographs were developed by the HEC-1 method (Reference 4, Appendix VI). Precipitation amounts for the flood hydrographs of the PMF and 100 year flood were taken from U. S. Weather Bureau Information (Reference 5 and 6, Appendix VI). Appropriate adjustments for basin size and shape were accounted for. These inflow hydrographs were routed through the reservoir to determine maximum pool elevations.

5.5 Reservoir Regulations: For routing purposes, the pool at the beginning of flood was assumed to be at elevation 198 msl. Reservoir stage-storage data and stage-discharge data were computed from construction plans and U.S. G. S. topographic maps. Floods were routed through the

reservoir using the spillway discharge up to a pool storage elevation of 210 msl. Discharges above pool elevation 210 msl were routed over the non-overflow section of the dam.

5.6 Overtopping Potential: The predicted rise of the reservoir pool and other pertinent data were determined by routing the flood hydrographs through the reservoir as previously described. The results for the flood conditions PMF, $\frac{1}{2}$ PMF and 100 year flood are shown in the following Table 5.1:

TABLE 5.1 - RESERVOIR PERFORMANCE

	Hydrograph			
	Normal Flow	100 Yr. Flood	$\frac{1}{2}$ PMF	PMF
Peak Flow, CFS				
Inflow	9	6921	18,423	36,845
Outflow	9	4312	13,794	29,576
Maximum Pool Elevation				
Ft, msl	198	200.5	203.1	206.2
Non-Overflow Section (Elev 210 msl)				
Depth of Flow, Ft	-	-	-	-
Duration, Hours	-	-	-	-
Velocity, fps *	-	-	-	-
Tailwater Elevation				
Ft, msl	162.5	173.4	176.2	179.5

* Critical velocity

5.7 Reservoir Emptying Potential: A 24 inch diameter gate at elevation 170 msl is capable of draining the reservoir through a 24 inch outlet pipe. Assuming that the lake is at normal pool elevation (198 msl) and there is 9 cfs inflow, it would take approximately 40 days to lower the reservoir to elevation 170 msl. This is equivalent to an approximate drawdown rate of 0.02 ft/day based on the hydraulic height measured from normal pool to the invert of the drawdown pipe divided by the time to dewater the reservoir.

5.8 Evaluation: The U. S. Army, Corps of Engineers guidelines indicate the appropriate Spillway Design Flood (SDF) for an intermediate size, significant hazard dam is the $\frac{1}{2}$ PMF to PMF. Because of the risk involved, the $\frac{1}{2}$ PMF has been selected as the SDF. The spillway will pass 100 percent of the PMF without overtopping the crest of the dam.

Hydrologic data used in the evaluation pertains to present day conditions with no consideration given to future development.

SECTION 6 - DAM STABILITY

6.1 Foundation and Abutments: The dam is located along the eastern edge of the Piedmont physiographic province of Virginia. This site is underlain by a thin veneer of Pleistocene terrace deposits and Recent alluvial soils. These sediments consist of a heterogeneous mixture of sand, silt, clay and gravel materials. They are underlain by residual soils, which are derived from the in place weathering of the underlying granite gneiss bedrock of Precambrian age. A profile and description of the materials encountered in the design test borings are included in Plate 2, Appendix I. Bedrock exposed below the spillway consists of differentially weathered and jointed granite gneiss. The formation observed was essentially horizontal. No faults have been identified in the immediate area.

The design report and drawings show the embankment being founded on overburden with a cutoff trench extending to the top of fresh or non-rippable rock. The trench was planned to have a 20 ft wide bottom and 1H:1V side slopes.

Based upon the design geologic data and performance history of the dam to date, a stable foundation is assumed. Gradual consolidation of underlying soils would be expected during application of fill materials. The underlying soils had probably essentially consolidated under the applied load not long after completion of construction.

6.2 Embankment:

6.2.1 Materials: Design drawings show the dam as a homogeneous earth fill embankment. Laboratory tests performed on borrow materials show the fill ranging from well-sorted sands (SW) to clayey sands (SC) in accordance with the Unified Soil Classification System. It was noted

in the design report that once this material was broken down and soaked for several days, it became a highly plastic silt (MH). It was specified that all fill material in the cutoff trench and embankment was to be placed in "successive layers of not more than 6 inches depth loose measure" and "compacted to a minimum of 97% of the standard proctor density." It was further specified that, "Tamping rollers shall be used for compacting the earthfill to the above density."

6.2.2 Subdrains and Seepage: Earthwork specifications required that the subgrade for the embankment be prepared by "scalping" to a 6 inch depth or deeper if so indicated on the drawings (Plate 2, Appendix I). The subgrade was to be cleared of all loose and objectionable material. The cutoff was to extend to fresh or non-rippable rock. No mention was made of the natural permeability of the bedrock, however, in the presence of joints and fractures natural permeabilities would be expected to range from low to high.

To control seepage, a rock toe drain was specified in design as shown on Plate 2, Appendix I. Gradation specifications are included in Appendix V. In attempt to prevent piping around the lake drain and water supply pipes, anti-seep collars were included. The seepage observed along the downstream is apparently the result of normal functioning of the rock toe drain.

6.2.3 Stability: The dam is 48 ft high and has a crest width of approximately 21 ft. Design slopes are 2.5H:1V on the downstream side and 3H:1V on the upstream side. Measurements taken at the center of the dam indicate the downstream slope is $2\frac{1}{2}$ H:1V. A 7 ft wide berm exists on the upstream slope at approximately elevation 194 msl.

The final report included no findings that proved the dam to be unsafe. Factors of safety calculated in the design stability analysis were in accordance with Corps of Engineers guidelines (Reference 1, Appendix VI) except for the steady state seepage condition. A factor of safety of 1.3 was calculated for the downstream slope which is slightly less than the 1.5 factor of safety recommended in Reference 1, Appendix VI. Based upon the design data, quality control used during construction, and performance history of the structure, no further studies are recommended. A routine maintenance program exists for the structure and maintenance is considered adequate. At the present time, there is no warning system or evacuation plan for the dam.

7.2.1 Emergency Operation and Warning Plan: It is recommended that a formal emergency procedure be prepared, prominently displayed, and furnished to all operating personnel. This should include:

- ### 7.3 Required Maintenance:

7.3.1 A staff gage should be installed to monitor water levels.

7.3.2 Small sapplings growing in the riprap on the upstream slope should be removed.

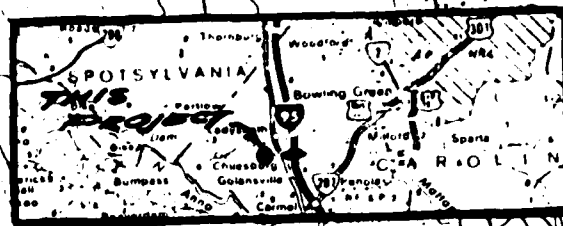
7.3.3 Bare areas present along the downstream toe and areas of sparse vegetation on the downstream slope should be reseeded.

7.3.4 Seepage along the downstream toe should be monitored during maintenance to verify that detrimental surface erosion has not developed along the downstream toe. If such conditions should develop, a Professional Engineer with an expertise in Geotechnical Engineering should be contacted to evaluate the problem and make recommendations for required corrective measures.

7.3.5 Attempts should be made to halt shoreline erosion in order to prevent sediment build up in the lake.

7.3.6 For monitoring purposes, it is recommended that the downstream beaver dam(s) be removed, so as to prevent submergence of drain outlets.

APPENDIX I
MAPS AND DRAWINGS



LAKE CAROLINE



Run

Cem

Lake
Caroline

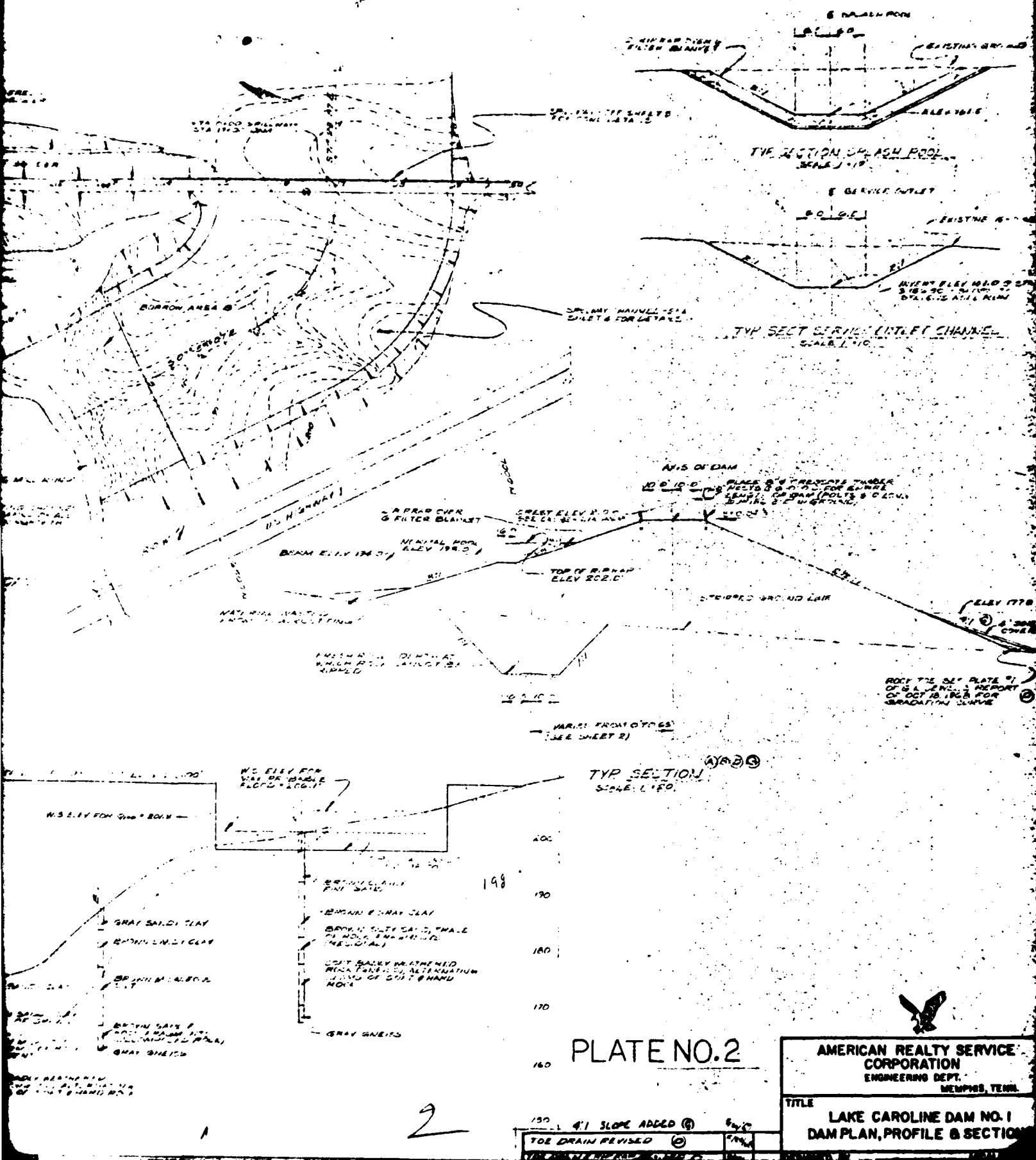
Golansville

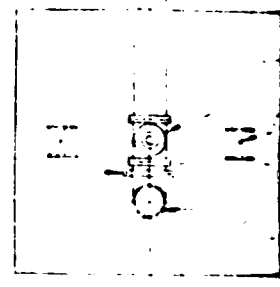
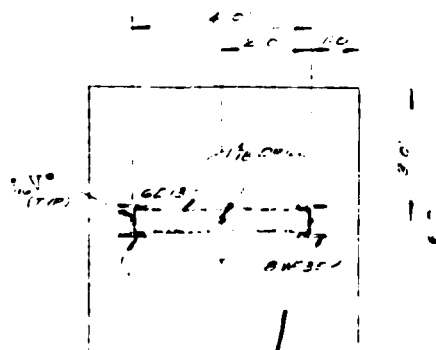
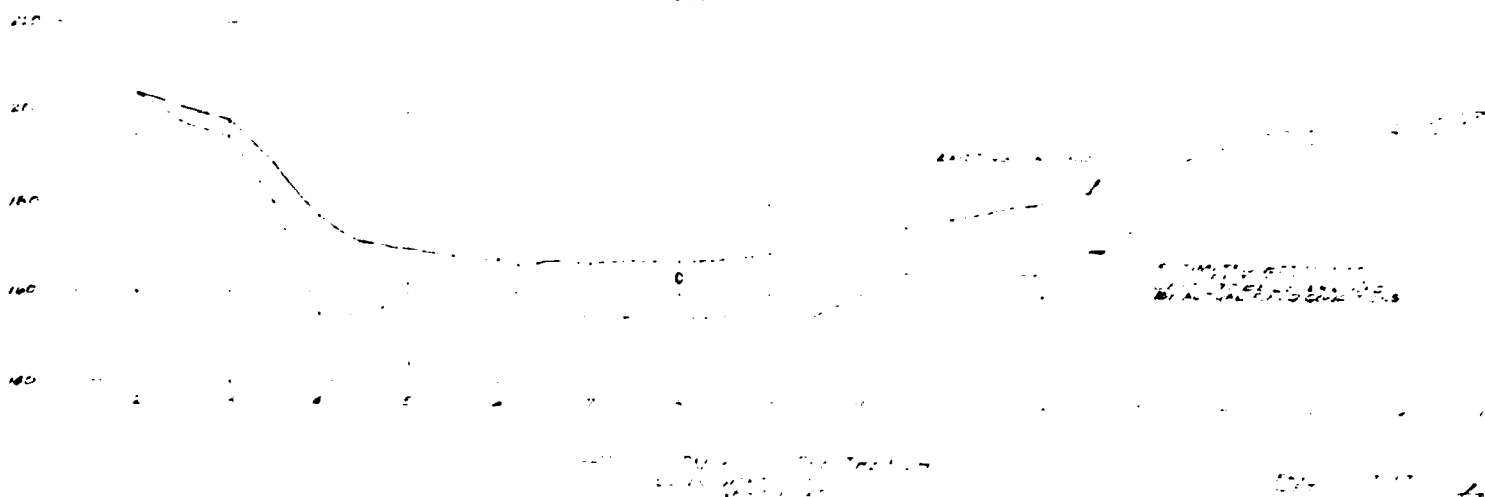
PLATE NO. 1

SCALE: 1" = 2,000'

Cedar
Pork

HEWLETT, VA.
N3752.5—W7730.7.5





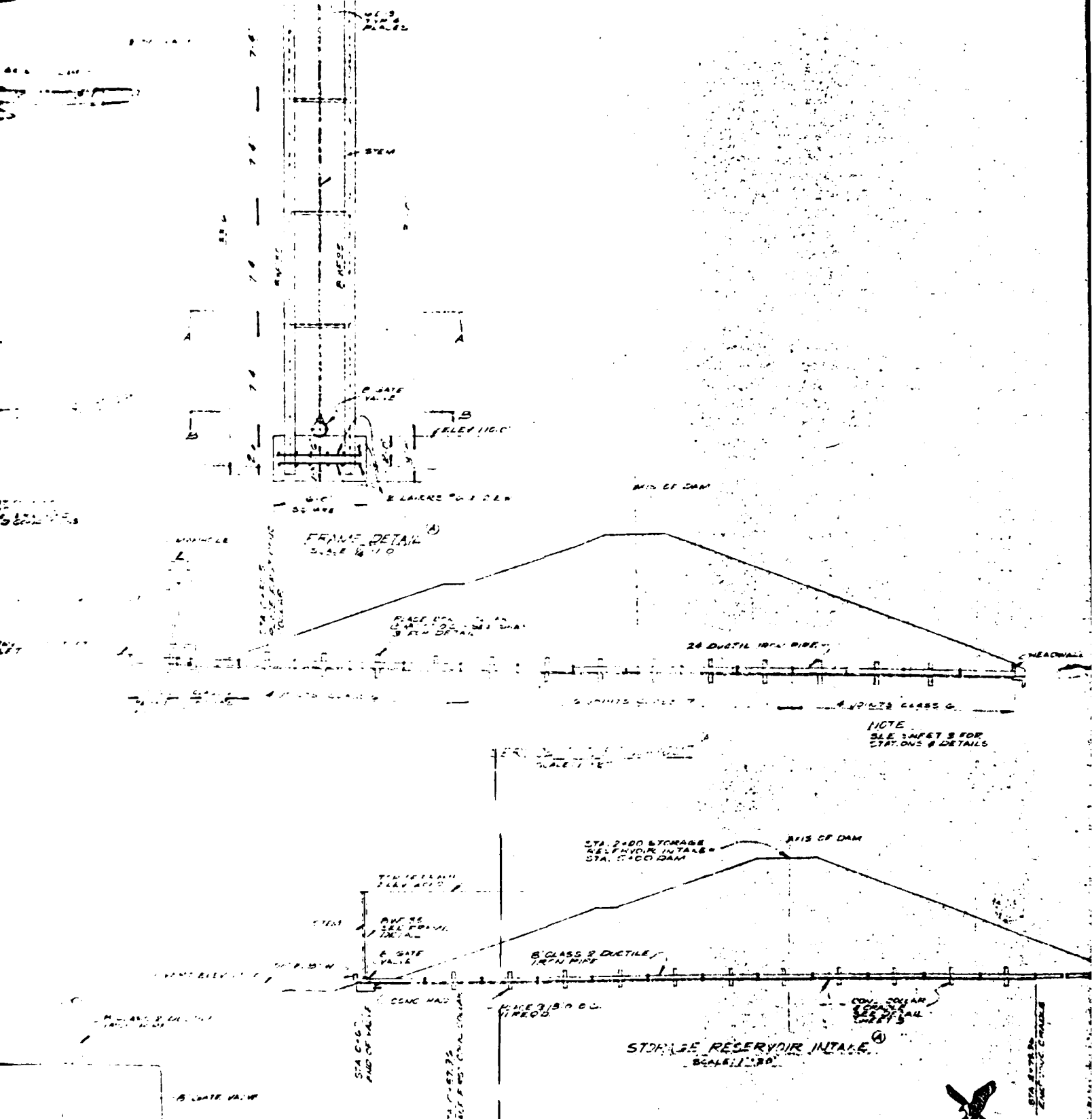



PLATE NO.3

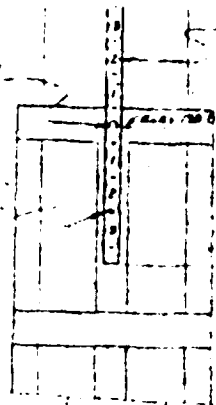


AMERICAN REALTY SERVICE CORPORATION
ENGINEERING DEPT.
MEMPHIS, TENN.

**TITLE LAKE CAROLINE DAM NO. 1
TOE DRAIN & CUT-OFF TRENCH
PLAN, PROFILE & DETAILS**

MOUNT GASE TAILING MAN
WITH 2" DIA. ANCHOR AT
ELEV. 198.1

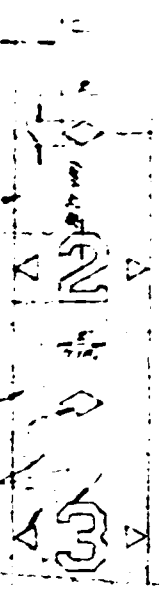
SECT. F. MOUNT WITH CHANNEL
PLATES AS SHOWN. ANCHOR
PLATE TO BE PLACED IN TAIL
ANCHOR BOLT TO S.F.O. B.C.



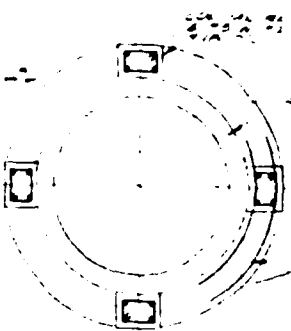
ELEVATION GASE
SCALE 1/8" = 1'-0"

APPLY WHITE SPENT
PAINTING AS PER
SPEC.

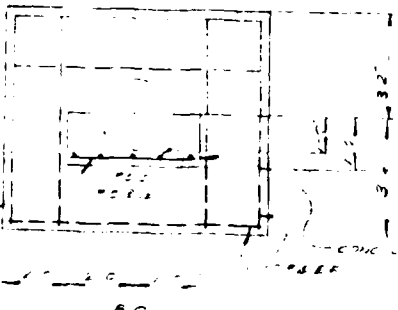
APPLY BLACK SPENT
PAINTING TO MARKER
AS PER
SPEC.



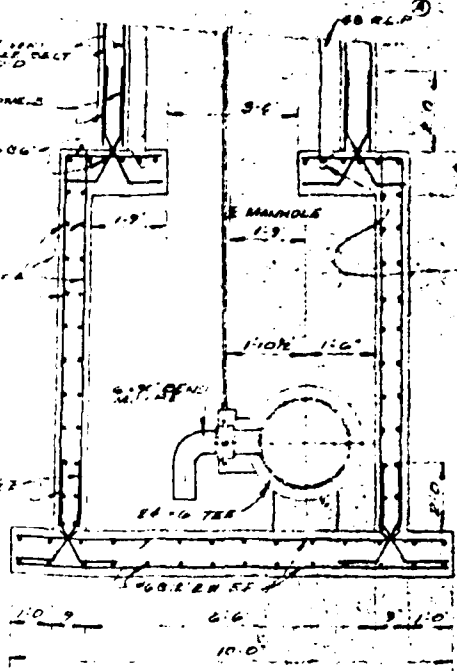
DETAIL OF GASE
TAILING
SCALE 1/4" = 1'-0"



SECT. D-D
SCALE 1/8" = 1'-0"

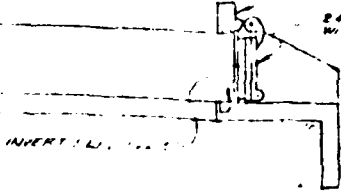


SECT. B-B
SCALE 1/8" = 1'-0"



SECT. A-A
SCALE 1/8" = 1'-0"

AREA OF TONGUE AS
REQD. TO MAKE LEAK
TIGHT & APPROX.



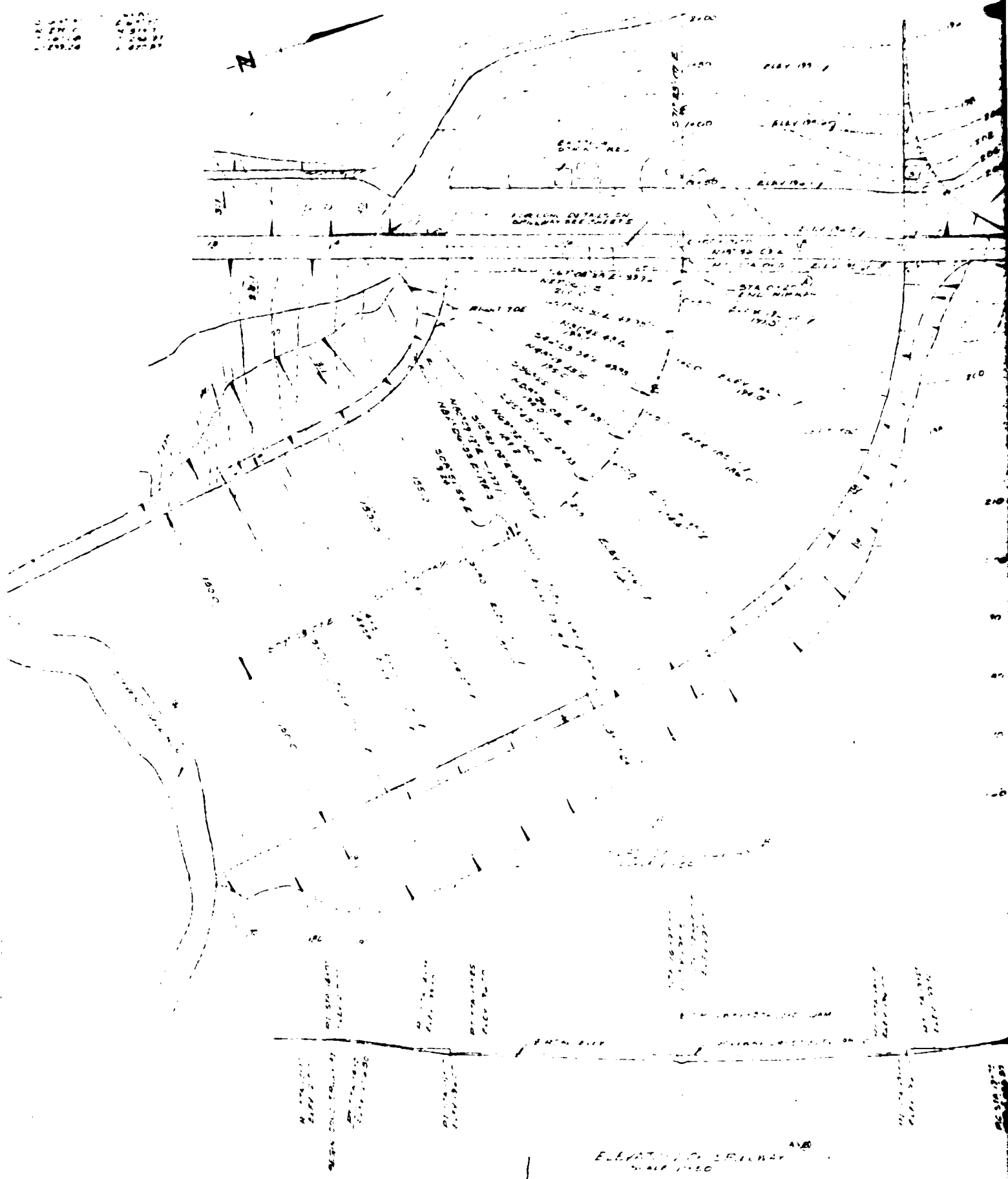
SECT. C-C
SCALE 1/8" = 1'-0"

AMERICAN REALTY SERVICE
CORPORATION
ENGINEERING DEPT.
MEMPHIS, TENN.

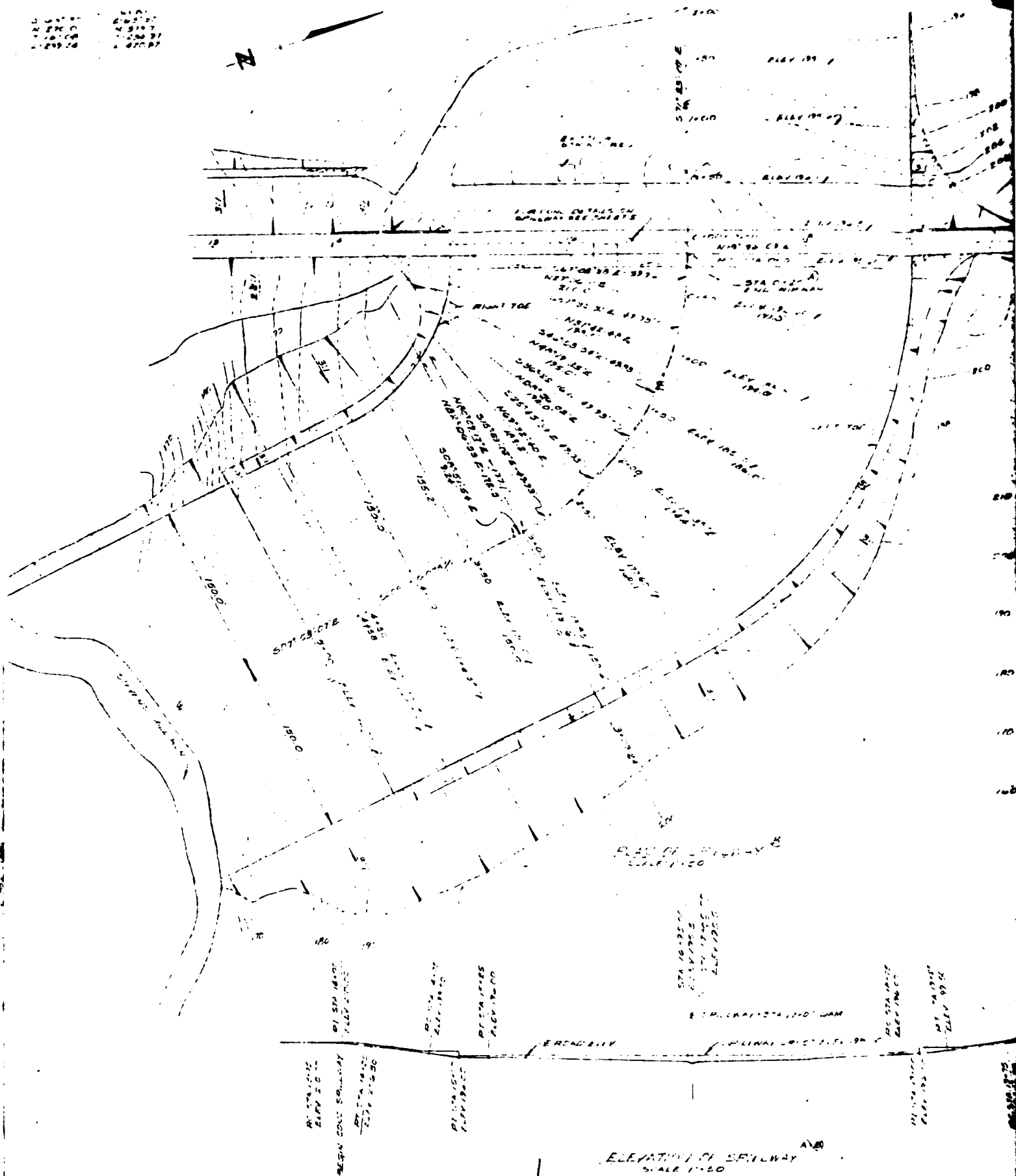
TITLE LAKE CAROLINE DAM NO. 1
SERVICE OUTLET
PLAN & SECTIONS

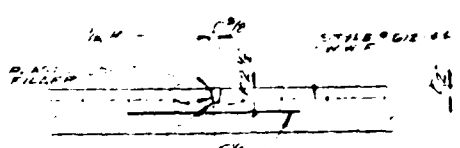
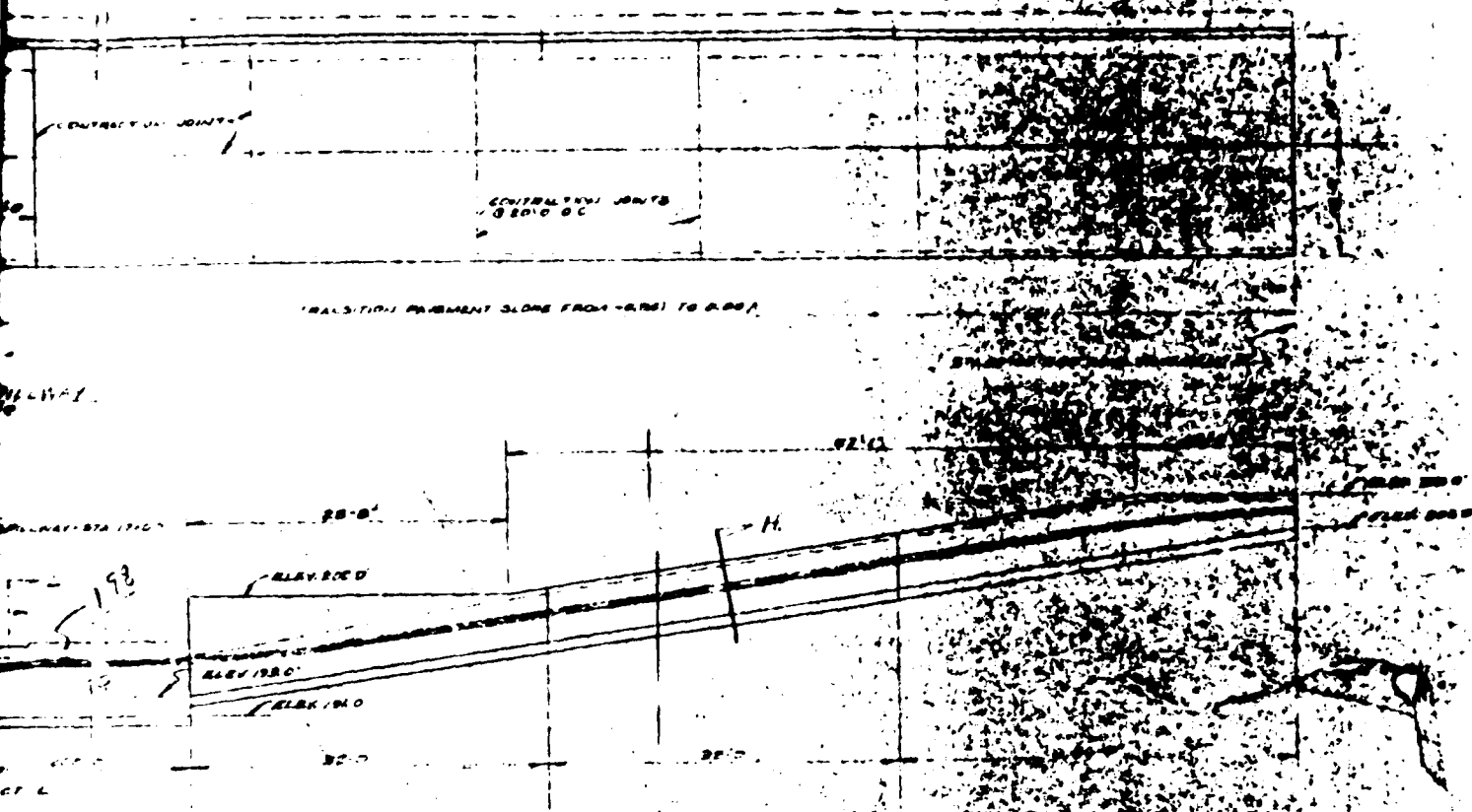
1000
1000
1000

N

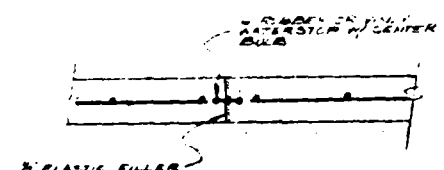


ELEVATION OF RAILWAY
SCALE 1:1000

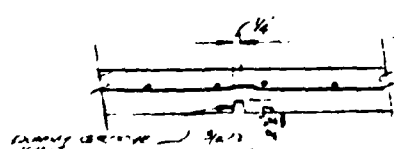




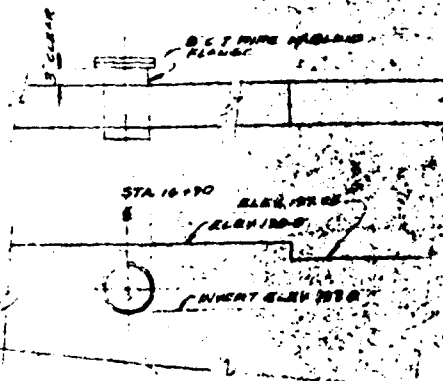
TYP. DRAINAGE DITCH
SCALE 3/4" = 1'-0"



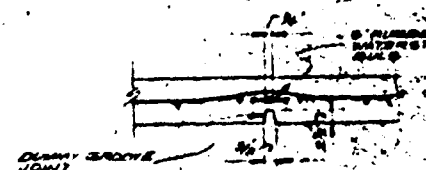
TYP. WALL EXPANSION JOINT
SCALE 3/4" = 1'-0"



TYP. WALL CONTRACTION JOINT
SCALE 3/4" = 1'-0"



ALT. OUTLET DETAIL
SCALE 3/4" = 1'-0"



TYP. CONTRACTION JOINT FOR SPILLWAY SECTION
SCALE 3/4" = 1'-0"

- CONCRETE NOTES**
1. ALL CONCRETE IS TO BE 3000 PSI COMPRESSIVE STRENGTH IN 28 DAYS.
 2. REINFORCING STEEL SHALL HAVE A MINIMUM YIELDING STRENGTH OF 60,000 PSI.
 3. REINFORCING STEEL SHALL BE PLACED IN THE CENTER OF THE WALL.
 4. REINFORCING STEEL SHALL BE PLACED IN THE CENTER OF THE WALL.
 5. REINFORCING STEEL SHALL BE PLACED IN THE CENTER OF THE WALL.
 6. ALL SPILLWAYS SHALL BE STAGGERED UNLESS OTHERWISE NOTED.

NOTE
ALL CONTRACTION JOINTS SHALL BE PLACED AT THE CENTER OF THE WALL.
ALL SPILLWAYS SHALL BE STAGGERED UNLESS OTHERWISE NOTED.

APPENDIX II

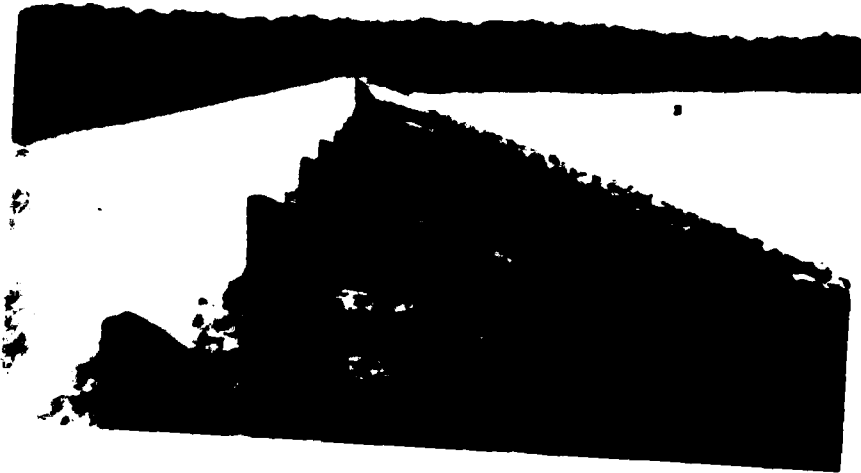
PHOTOGRAPHS



Photograph No. 1 - Spillway
Note Access Road Across Spillway



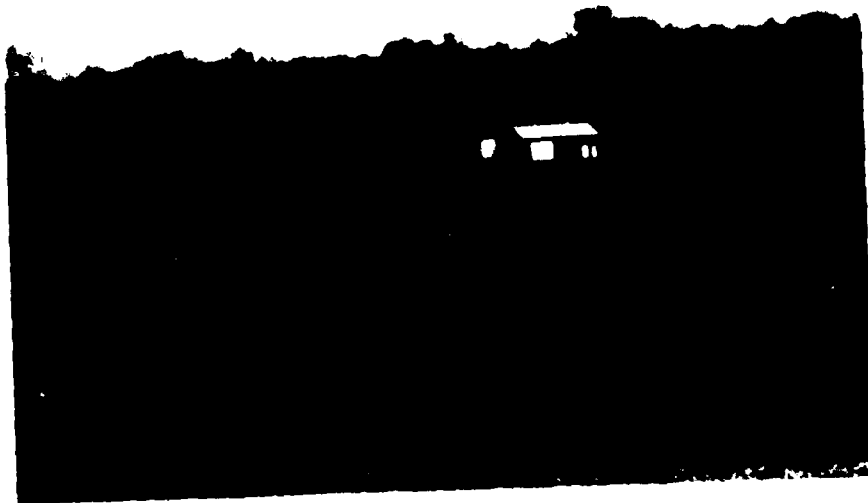
Photograph No. 2 - Low Flow Weir



Photograph No. 3 - Upstream Face of Dam
and Access Road



Photograph No. 4 - Downstream Face of Dam



Photograph No. 5 - Water Treatment Plant
Immediately Downstream

APPENDIX III
FIELD OBSERVATIONS

Check List
Visual Inspection
Phase I

Lat 37° 59.2'
Long 77° 30.4'

Name Dam Lake Caroline County Caroline State Virginia Coordinates

Date(s) Inspection April 20, 1981 Weather Partly Cloudy Temperature 65°F
Windy

Pool Elevation at Time of Inspection 193.0 msl Tailwater at Time of Inspection 162.5 msl

Inspection Personnel:

Schnabel Engineering Associates
Gilbert T. Seese
Stephen G. Werner
Raymond A. DeStephen, P.E.*

J. K. Timmons & Associates
Robert G. Roop, P.E.
Steven Oddi

Recorder
Gilbert T. Seese

State Water Control Board
Hugh M. Gildea, P.E.

Owner's Representative

R. J. ("Slim") Miller

*Not present during this inspection but visited the dam on May 18, 1981.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	The slopes, crest and abutment contacts were inspected and no cracks were noted. Ground conditions were damp at the time of inspection as a result of early morning rains.	-
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	No unusual movements or cracking were noted on the dam or beyond the embankment toe.	-
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	No serious erosion was noted on the upstream embankment. Minor surface washing was observed on the downstream slope. The downstream slope was measured as 2½H:1V and the upstream slope as 3H:1V at the center of the dam.	-
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	The vertical and horizontal alignment of the dam appeared to be good. A paved road occupies the crest of the dam. The road is in good condition.	-
RIPRAP FAILURES	Riprap is present on the upstream slope and appears to be in good condition. It consists of fresh granite gneiss boulders 1 to 3 ft. in length. The riprap is visible below pool level and extends 5 to 7 ft above pool level on the upstream slope. Scattered small sapplings were observed growing through the riprap.	-

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
FOUNDATION	The junction of the embankment and abutments were well vegetated with only minor erosion noted on the upstream abutment. Surface soils in the surrounding area include fine to medium clayey sands and sandy clays.	-
ANY NOTICEABLE SEEPAGE	Seepage and saturated ground conditions were observed along the right downstream toe of the embankment. See Field Sketch, Sheet 1. No sloughing or bulging of the embankment was observed in the area.	-
DRAINS	Lake drain discharge area was submerged because of water backed up from a beaver pond downstream. No drains observed.	For monitoring purposes, the outlet should not be submerged.
MATERIALS	The embankment appears to be constructed of fine to medium silty SAND to clayey silty SAND (SM to SC) brown-moist with fine to coarse gravel, probably residual material	-
VEGETATION	Upstream slope moderately vegetated with grass. Downstream slope sparsely vegetated with grass and includes scattered bare areas.	Downstream slope should be reseeded/

PRINCIPAL SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
CONTROL SECTIONS	Principal spillway consists of a 400 ft wide concrete wier with a 7 ft wide x 3" deep low flow notch. See Field Sketch, Sheet 1.	Combined principal-emergency spillway in good condition.
APPROACH CHANNEL	No separate approach channel. Area upstream of wier is riprap lined and in good condition.	-
DISCHARGE CHANNEL	The discharge channel is approximately 300 ft wide and slopes at about 7.5%. Immediately below the wier the discharge channel is connected and bordered by a cable guard rail along the downstream edge. This area serves as part of the road crossing the dam. Just downstream from the roadway the discharge channel is covered with riprap. The discharge channel area downstream from the concrete section was severely eroded in 1969 during hurricane Camille. 11 ft deep gullies eroded in the channel areas during the storm were backfilled with riprap and concrete slurry. Evidence of severe erosion during the storm is still visible in the lower reaches of the discharge area. The area appeared stable.	This information was received from Mr. Miller during the inspection.
BRIDGE AND PIERS	None	-
EMERGENCY GATE	A 24 in butterfly valve is at invert El 166.5 According to Mr. Miller the value is in good operating condition.	The valve has not been operated recently.
GATES AND OPERATION	An 8 in intake for an existing water treatment plant at El 170 ⁺ with an 8 in valve. According to Mr. Miller the value is in good operating condition.	Operational

EMERGENCY SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONTROL SECTIONS	Combined principal-emergency spillway See principal spillway.	-
APPROACH CHANNEL	None	-
DISCHARGE CHANNEL	None	-
BRIDGE AND PIERS	None	-
MISCELLANEOUS	None	-

INSTRUMENTATION

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATION
MONUMENTATION/SURVEYS	None Observed	-
OBSERVATION WELLS	None	-
WEIRS	None	-
PIEZOMETERS	None	-
STAFFGAGES	A staff gage consisting of numbers painted on the lake drain intake structure is shown on the design drawings. These numbers are no longer visible because of weathering of the paint.	A staff gage should be installed.
OTHER	None	-

RESERVOIR

VISUAL EXAMINATION	OBSERVATIONS	REMARKS AND RECOMMENDATIONS
	<p>The reservoir is located in a natural valley with gentle to moderate slopes (10H:1V), mostly wooded with some residential lawn areas. Some shoreline erosion near the right lower end of the reservoir was observed. The reservoir appeared to be free of debris.</p>	<p>Shoreline erosion should be stabilized.</p>

SLOPES

None observed. Water was clear.

SEDIMENTATION

DOWNSTREAM CHANNEL

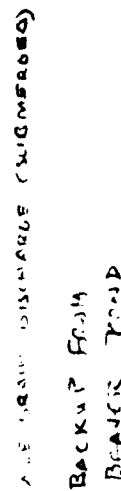
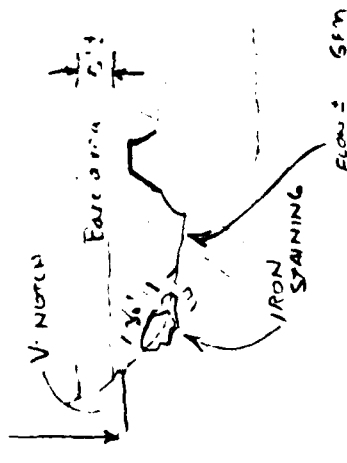
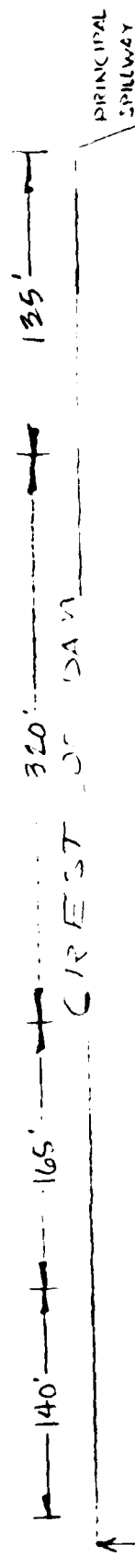
VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Heavily wooded downstream channel. Four lane highway bridge (U.S. Rt. 1) crosses channel 900 ft± downstream from dam. The bridge opening is 30 ft wide by 8 ft high (N = 0.1) The channel is approximately 2 ft deep with 1H:1V side slopes.	-
SLOPES	Between dam on Route 1 bridge, area is level and swampy with numerous saturated, heavily vegetated areas. Flood plain and channel are narrow with 5H:1V± slopes.	-
APPROXIMATE NO. OF HOMES AND POPULATION	Community water treatment and supply facility approximately 500± ft downstream from dam.	-

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

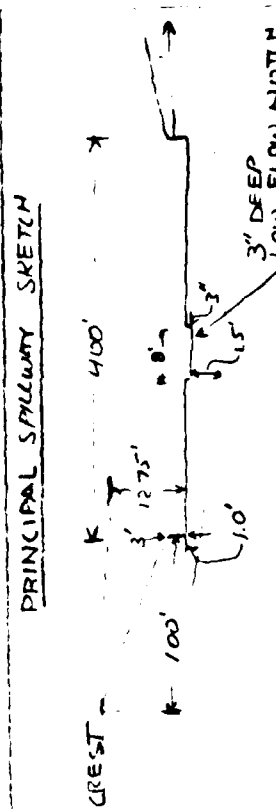
ITEM	REMARKS
REGIONAL VICINITY MAP	Hewlett, Virginia USGS 7½ Minute Quadrangle -
DESIGN/CONSTRUCTION HISTORY	Design and construction under the supervision of American Realty Service Corporation. Constructed by Bailey and Associates and completed in 1968. -
PLAN OF DAM	Plans were provided by R. J. Miller, Caretaker/Owner representative and are included in Appendix I. -
TYPICAL SECTIONS OF DAM	See plans, Appendix I -
OUTLETS - PLAN DETAILS CONSTRAINTS DISCHARGE RATINGS	See plans, Appendix I -
SPILLWAY - PLAN SECTION DETAILS	See plans, Appendix I -
OPERATING EQUIPMENT - PLAN DETAILS	See plans, Appendix I -

ITEM	REMARKS
MONITORING SYSTEMS	None
RAINFALL/RESERVOIR HIGHPOOL RECORDS	None
GEOLOGY REPORTS	See Test Boring Plans, Appendix I
BORROW SOURCES	Reservoir, spillway
MATERIALS INVESTIGATIONS BORING RECORDS LABORATORY-FIELD TEST DATA	See report, Appendix IV
HYDROLOGIC/HYDRAULIC DATA	None

ITEM	REMARKS
DESIGN REPORTS	See Appendix IV -
DESIGN COMPUTATIONS HYDROLOGY & HYDRAULICS DAM STABILITY SEEPAGE STUDIES	No design computations available for hydrology and hydraulics of the dam. Stability analysis included in Appendix IV. -
POST CONSTRUCTION ENGINEERING STUDIES RECORDS, SURVEYS	Not available -
MODIFICATIONS	None -
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	Erosion up principal spillway discharge channel area -
MAINTENANCE OPERATION RECORDS	Very little maintenance prior to 1977. R.J. Miller began regular maintenance when employed at Lake Caroline 4 years ago. Information from R. J. Miller during inspection



SEEING
FRAMES



LAKE CAROLINE DAM

FIELD SKETCH

NO SCALE DATE 4-20-81

SHEET 1

400' CONCRETE WEIR

NO SCALE

INDICATES
MAGNIFICATION

APPENDIX IV
DESIGN REPORT

G. K. JEWELL AND ASSOCIATES

Soils Engineering Consultants

195 CHITTENDEN AVENUE

COLUMBUS, OHIO 43201

TELEPHONE 294-3748

G. K. JEWELL, P.E.
BURR D. BENEDICT, JR., P.E.

WILLIAM W. HOLMAN, P.E.
TREVOR P. CRAIG, P.E.

October 18, 1968
1112

American Realty Service, Inc.
P. O. Box 4831
Crosstown Station
Memphis, Tennessee 38104

Attention: Mr. Robert Sanderson

Re: Lake Caroline Dam
Fredericksburg, Virginia

Gentlemen:

Submitted here is the report of the work done on the Lake Caroline Dam. This work has been done in accordance with an agreement reached in the meeting between Mr. Robert Sanderson, Mr. Robert D. Sayre and Mr. G. K. Jewell, on August 22, 1968. Mr. Robert D. Sayre's report of July 1968, which was loaned to this office by American Realty Service, is also enclosed.

Please contact this office if you have questions in connection with any phase of this work.

Very truly yours,

G. K. JEWELL AND ASSOCIATES

V. V. Rajadhyaksha

V. V. Rajadhyaksha

G. K. Jewell
G. K. Jewell

VVR/ea
Submitted: 3 copies
cc: Mr. Robert D. Sayre

Enc: Soil Study and Embankment Design
Lake Caroline Dam

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ANALYSIS AND RECOMMENDATIONS - - - - -	2

APPENDIX

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Standard Proctor Compaction Test - - - - -	2
Unconfined Compression Test - - - - -	3
Unconsolidated-undrained Triaxial Compression Tests - - - - -	4 through 6
Consolidated-undrained Triaxial Compression Tests - - - - -	6 through 9
Strength Envelopes - - - - -	10 and 11
Recommended Dam Section - - - - -	12

INTRODUCTION

At the request of Mr. Robert Sanderson, Mr. G. K. Jewell met with representatives of the American Realty Service and Mr. Robert D. Sayre, the soils consultant for the proposed Lake Caroline Dam, to discuss the need for a blanket drain below the toe of the proposed embankment. As a result of this meeting, Mr. Jewell agreed to determine if strength data were available in the files of G. K. Jewell and Associates for soils comparable to the soils with which the proposed embankment will be constructed. If such data were available, stability analyses were to be performed to determine the factor of safety of the structure without a blanket drain. These considerations were contained in a letter submitted to Mr. Sanderson on August 23, 1968.

Strength data for material comparable to the project soil were not available in the files of G. K. Jewell and Associates, and samples of the proposed borrow material were requested so that laboratory tests could be performed. The soil received for testing was obtained from the emergency spillway area. A testing program was developed for the proposed borrow material, and subsequent to the completion of this program, stability analyses were performed.

LABORATORY TESTING

Based upon visual identification, the proposed borrow samples were found to be similar and were mixed. The following amount of testing was performed on the combined sample.

- 1 Visual Identification
- 1 Liquid and Plastic Limit
- 1 Sieve Analysis
- 1 Sieve and Hydrometer Analysis
- 1 Standard Proctor Compaction Test
- 1 Unconfined Compression Test
- 3 Unconsolidated-undrained Triaxial Compression Tests
- 3 Consolidated-undrained Triaxial Compression Tests
with pore-pressure measurements.

G. K. JEWELL AND ASSOCIATES

SOILS ENGINEERING CONSULTANTS

The results of the liquid and plastic limit test are given on Plate 1. Results of all other tests are presented in curve form on Plates 1 through 9.

ANALYSIS AND RECOMMENDATIONS

The soil in the proposed emergency spillway borrow area is reported to be weathered Baltimore gneiss. The soil when air-dried appears to be granular upon visual examination, but upon the addition of water and application of finger pressure, tends to break down into a fine-grained soil. A dry sieve analysis resulted in the gradation presented as Curve 2 on Plate 1. Curve 1 on Plate 1 shows the gradation of the soil after it has been soaked in water overnight and agitated in a standard mixing cup used in the preparation for an hydrometer test. When a plastic limit test was performed on a sample by adding water directly to an air-dried specimen, the soil does not appear to be plastic, but when soaked for several hours, a specimen of soil had liquid limit and plastic limit values of 59 and 44 percent respectively. It was also noted, that compacting the soil at practical moisture content values does not break down the soil significantly. After compaction, the gradation of the soil is more like that of Curve 2 than Curve 1 (Plate 1). Therefore, it is believed that at the time of construction of the embankment, the borrow soil will be represented by Curve 2 but that after passage of time, the gradation of the embankment soil will lie somewhere between Curves 1 and 2. Evidence of this phenomenon was also found while consolidating a test specimen for a remolded consolidated-undrained triaxial compression test. The test specimen continued to consolidate under a constant pressure for longer than 24 hours. This indicated that the apparently coarse-grained particles continued to break down to finer particles under the confining load.

All of these observations indicate that the sample is mainly composed of weathered rock particles which are in the process of breaking down to soil.

Stability analyses based on the results of the remolded laboratory tests were performed on the dam section, proposed in Mr. Robert Sayre's report of July 1968, after deleting the downstream drainage blanket. The conditions for which the analysis was performed were: as-built and steady seepage for the downstream slope, and sudden drawdown for the upstream slope. Effective strength values obtained from the consolidated-undrained triaxial compression tests were used for the analysis of the sudden drawdown and steady seepage conditions. The minimum value obtained from the results of unconfined compression and unconsolidated-undrained triaxial compression tests were used for the analysis of the as-built condition. Following are the minimum acceptable factors of safety considered for this project and the minimum values obtained for the various conditions of analysis:

Condition	Factor of Safety	
	Acceptable	Actual
As-built	1.3	3.0 ⁺
Steady seepage	1.5	1.33
Sudden drawdown	1.2	1.25

For the stability analyses, it was assumed that seepage would occur through the embankment and would break out on the downstream slope. Because of the granular-like nature of the fill material, seepage through the embankment is possible even though there appear to be granular materials below the embankment which could act as an in-place blanket drain. Generally, the embankment was safe using the strength values determined in the laboratory. It was found, however, that the toe of the embankment, in the zone where seepage might emerge, was potentially unsafe, and analyses at this location

resulted in the factor of safety of 1.3 which is listed in the foregoing table. If the toe were to slough, the rest of the embankment would be endangered.

The rock toe material has been designed as a filter to prevent the embankment material (in a granular-like condition) from infiltrating into the rock toe and causing piping. It should be noted however, because of the peculiar nature of the borrow soil, there may be no protection from infiltration if the embankment soil in the form of Curve 2 should deteriorate, in time, to the condition represented by Curve 1.

Based upon this investigation, it is believed that the blanket drain, which was originally designed for the downstream slope, may be omitted, but that a rock toe should be placed as shown on Plate 12. The rock toe should be composed of materials having the gradation limits shown as the hatched zone on Plate 1 and should be covered with 4-inch layer of borrow soil. Actually, the rock toe will be necessary only if seepage occurs through the embankment and breaks out on the downstream slope.

It is believed that the soil proposed for borrow will erode severely. The surface of the completed embankment should be protected from erosion as quickly as possible.

A concluding statement in connection with the soil evaluated for this project is appropriate. It has been observed that the borrow soil deteriorates under pressure, especially when wet. These conditions will occur when the soil is used in a water-retaining embankment. It is possible therefore, that the structure and the characteristics of the borrow soil will change. The results of these short duration tests do not necessarily indicate the long-term strength characteristics of the soil in a broken down condition. It is

repeated that it is impossible to evaluate the long-term performance and the safety of this embankment based upon the information developed in the course of this investigation. It would be desirable therefore to determine if these materials have been used locally for earth structures and if there are performance records available in connection with the long-term behavior of these soils. If such information is available, it should be used to determine if additional modification of the design section is desirable.

APPENDIX

[illegible]

STANDARD PROCTOR COMPACTION TEST

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Test Pit: Emergency Spillway

Sample: Bag

Material: See Plate 1 and report.

Hammer: 5.5 pounds

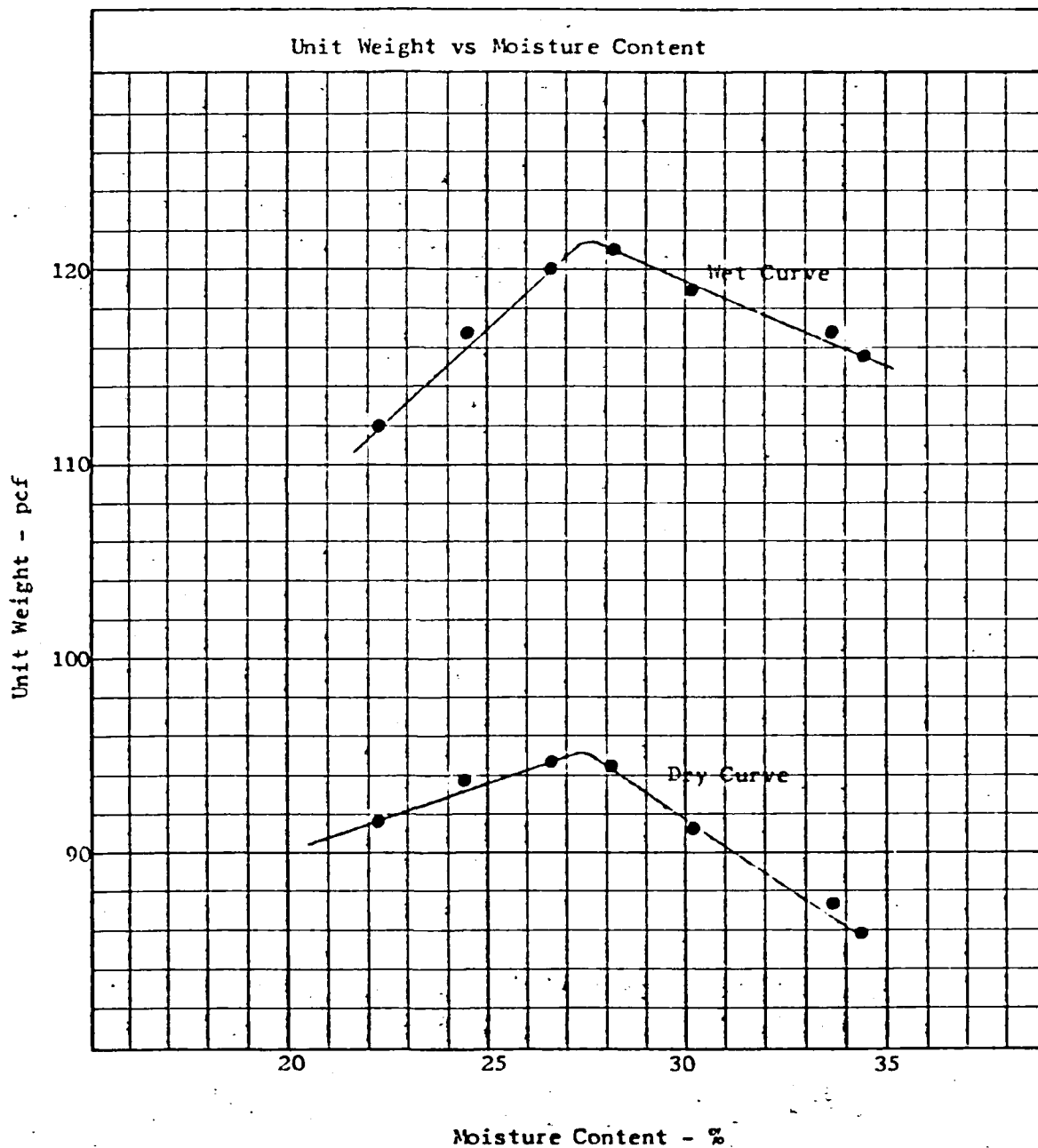
Blows: 25/layer

Drop: 12 inches

Layers: 3

Optimum Moisture Content=27.5%

Maximum Dry Density=95.2 pcf



REMAINED UNCONFINED COMPRESSION TEST

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

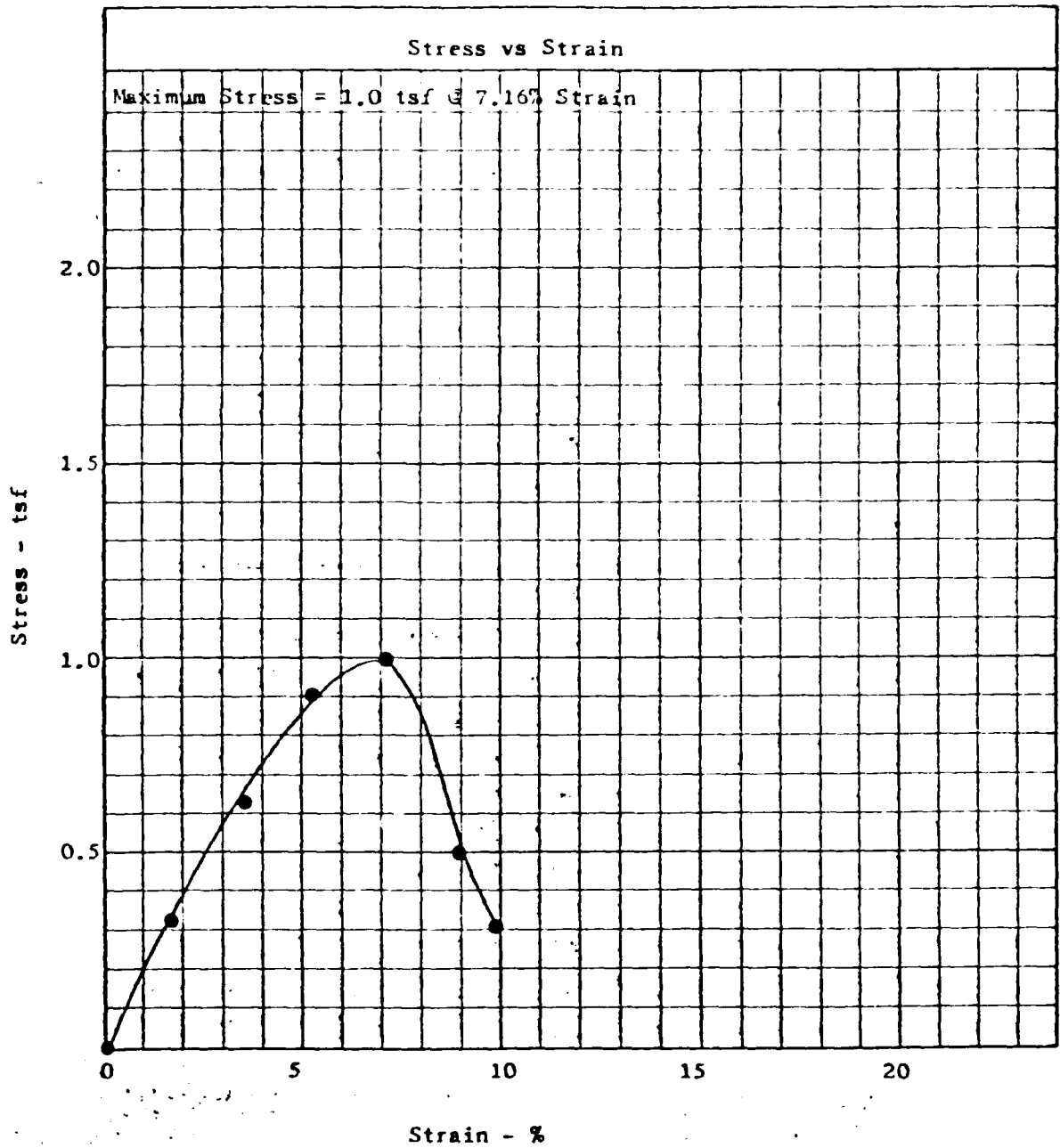
Material: See Plate 1 and report.

Moisture Content: 27%

Unit Dry Weight: 95 pcf

Size: 5.59" x 2.83" dia.

Unit Wet Weight: 120 pcf



TRIAXIAL COMPRESSION TEST - REMOLDED SAMPLE

Project: Lake Caroline

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

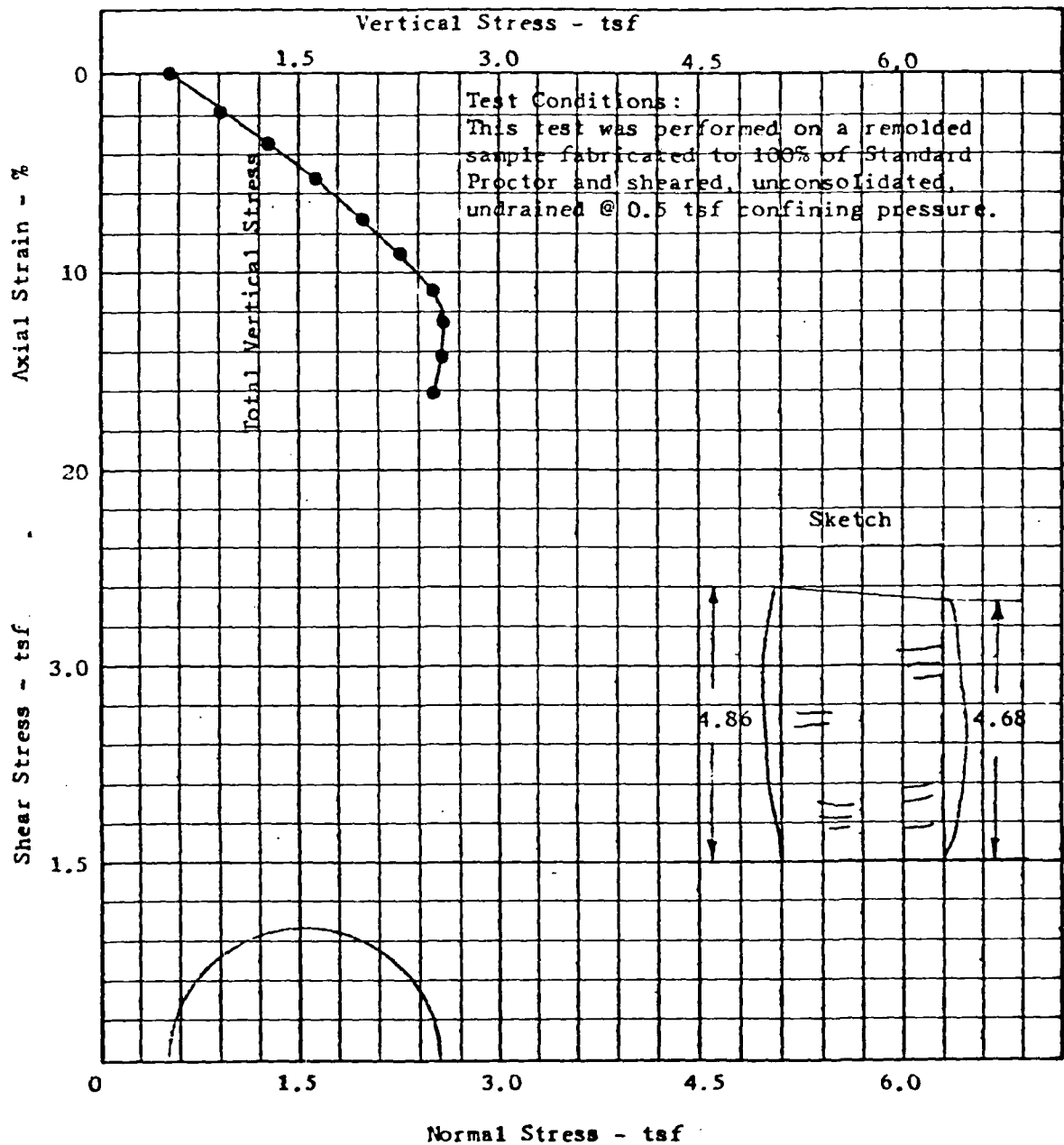
Material: See Plate 1 and report.

Initial Moisture Content: 28%

Unit Dry Weight: 95 pcf

Size: 5.59" x 2.83" dia.

Unit Wet Weight: 121 pcf



TRIAXIAL COMPRESSION TEST--FLOODED SAMPLE

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

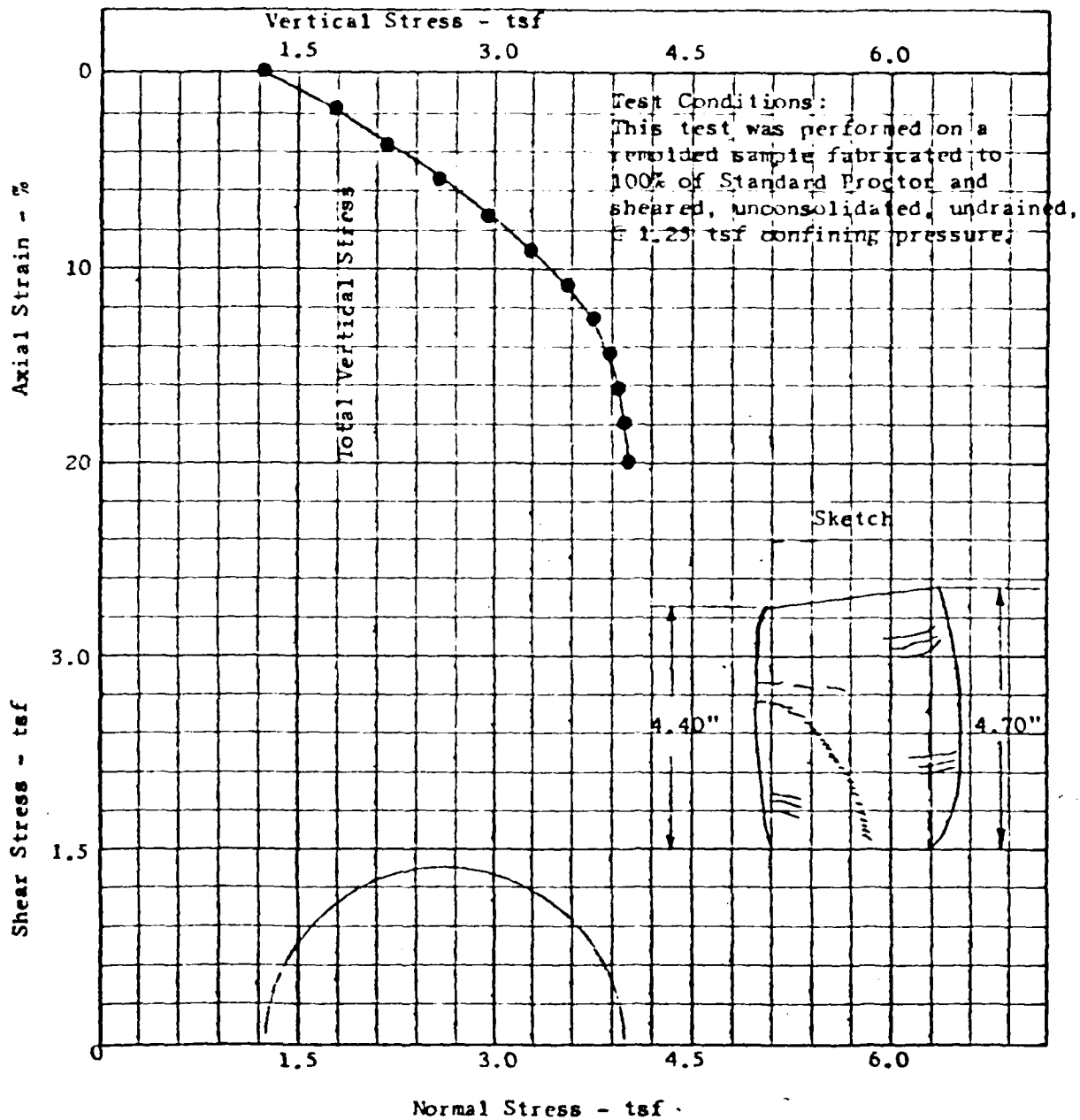
Material: See Plate 1 and report.

Initial Moisture Content: 27%

Unit Dry Weight: 95 pcf

Size: 5.59" x 2.83" dia.

Unit Wet Weight: 120 pcf



TRIAxIAL COMPRESSION TEST - REMOLDED SAMPLE

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

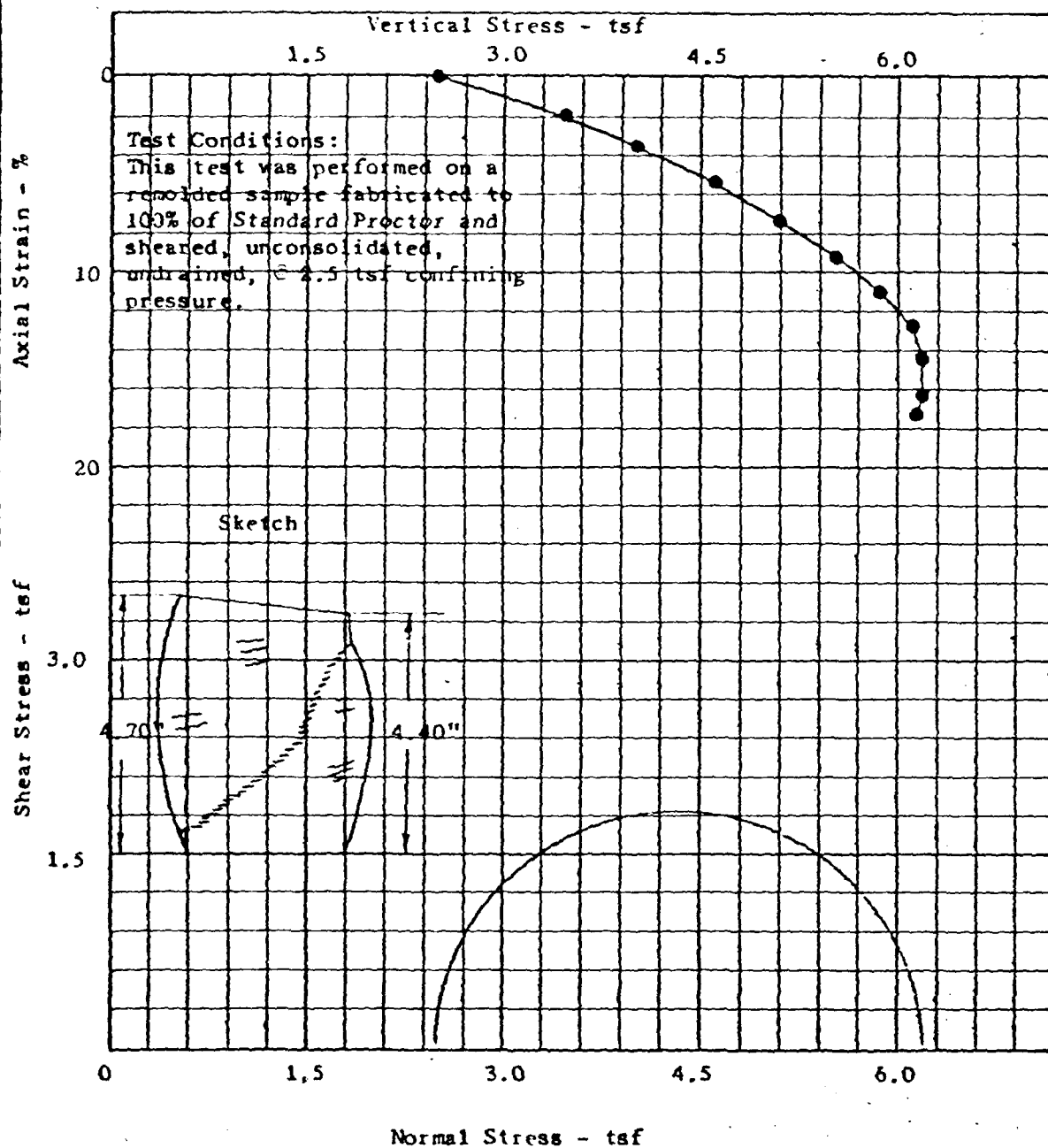
Material: See Plate 1 and report.

Initial Moisture Content: 27%

Unit Dry Weight: 95 pcf

Size: 5.59" x 2.83" dia.

Unit Wet Weight: 120 pcf



TRIAXIAL COMPRESSION TEST - REMOLDED SAMPLE - WITH PORE PRESSURE MEASUREMENTS

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

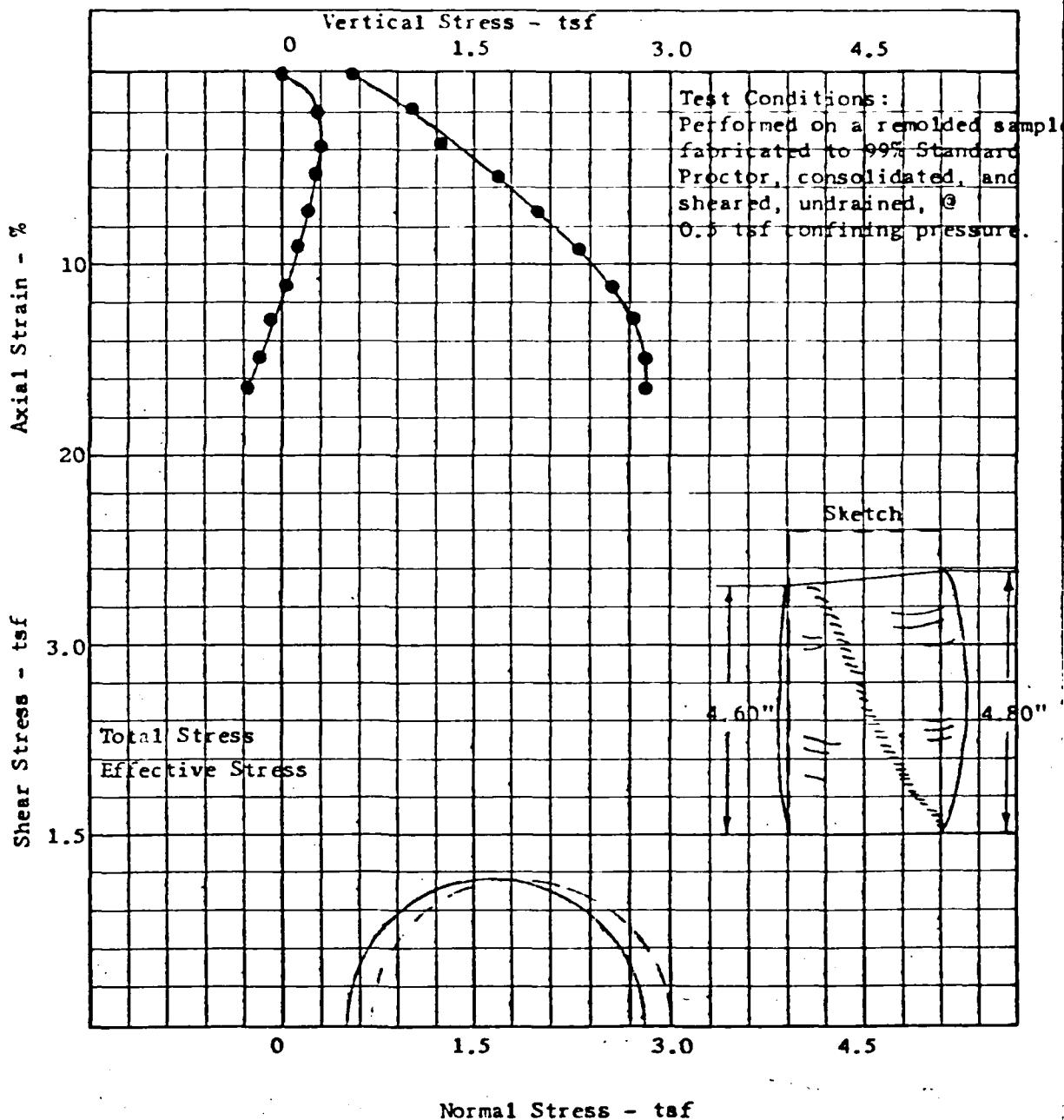
Material: See Plate 1 and report.

Initial Moisture Content: 28%

Unit Dry Weight: 94 pcf

Size: 5.57" x 2.83" dia.

Unit Wet Weight: 121 pcf



Unit Wet Weight: 120 pcf



TRIAXIAL COMPRESSION TEST--REMOLDED SAMPLE--WITH PORE-PRESSURE MEASUREMENTS

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

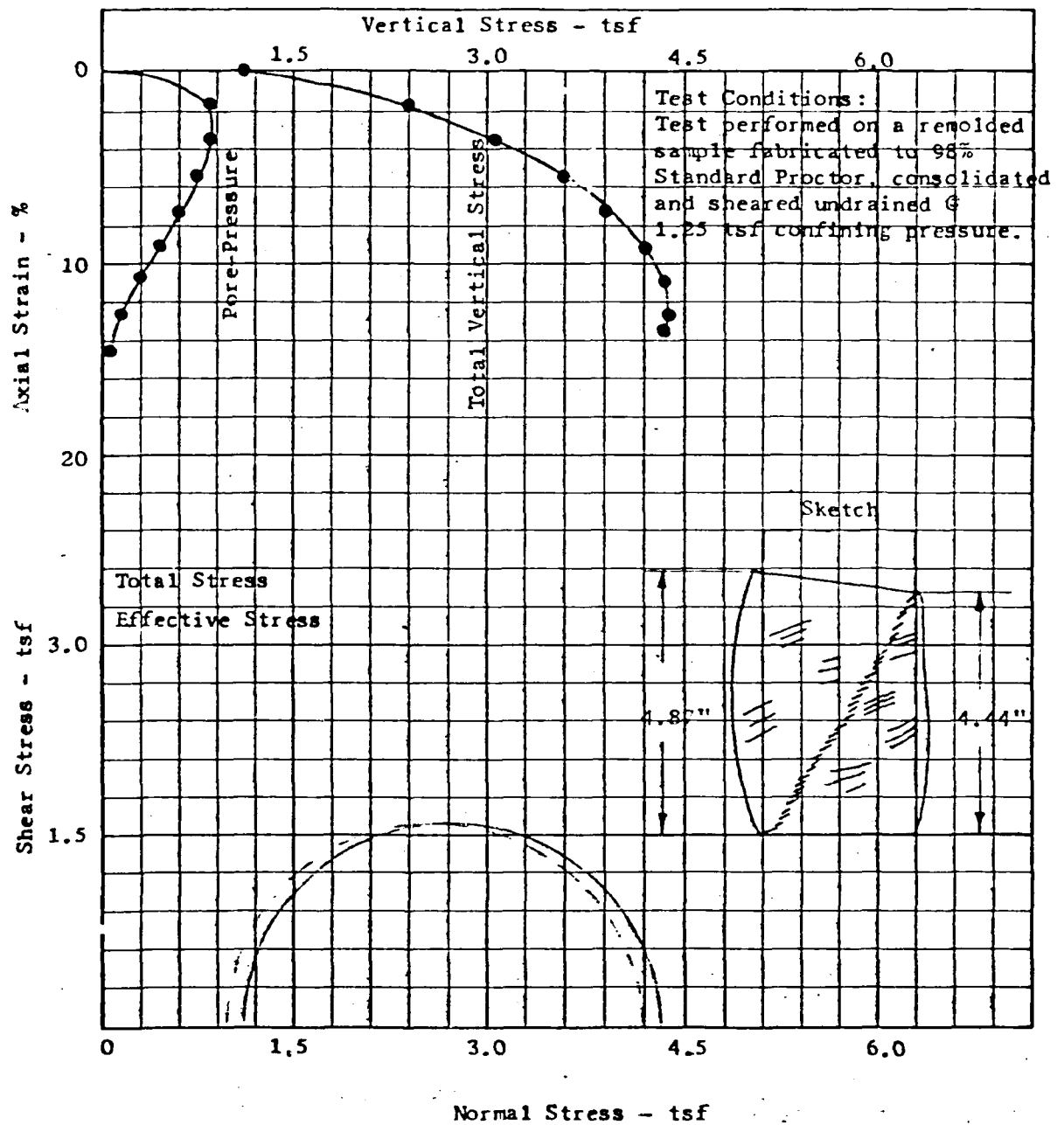
Material: See Plate 1 and report.

Initial Moisture Content: 28%

Unit Dry Weight: 93 pcf

Size: 5.58" x 2.83" dia.

Unit Wet Weight: 120 pcf



TRIAXIAL COMPRESSION TEST--REMOLDED SAMPLE--WITH PORE-PRESSURE MEASUREMENTS

Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

Boring: Emergency Spillway

Sample: Bag

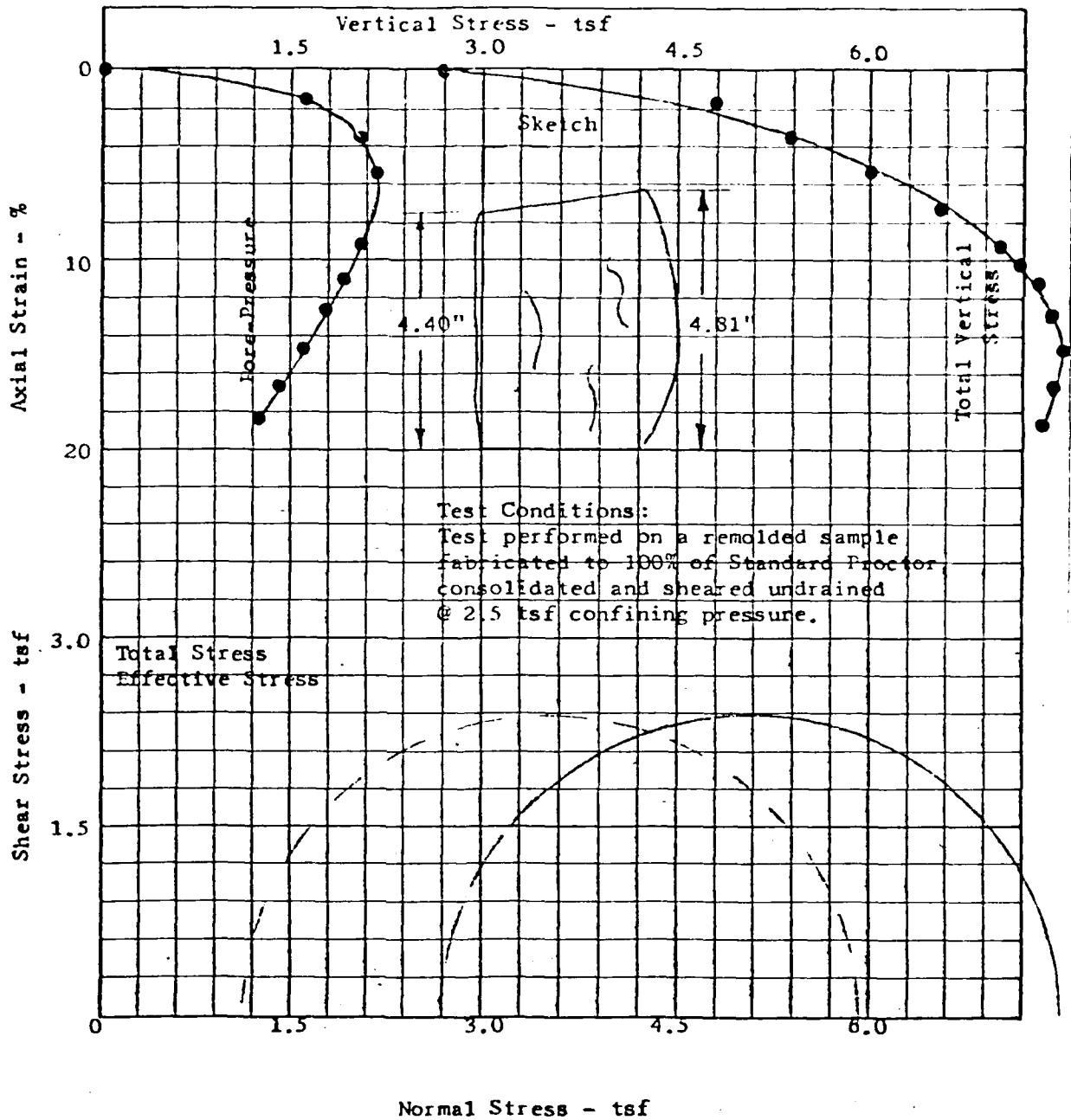
Material: See Plate 1 and report.

Initial Moisture Content: 27%

Unit Dry Weight: 95 pcf

Size: 5.58" x 2.83" dia.

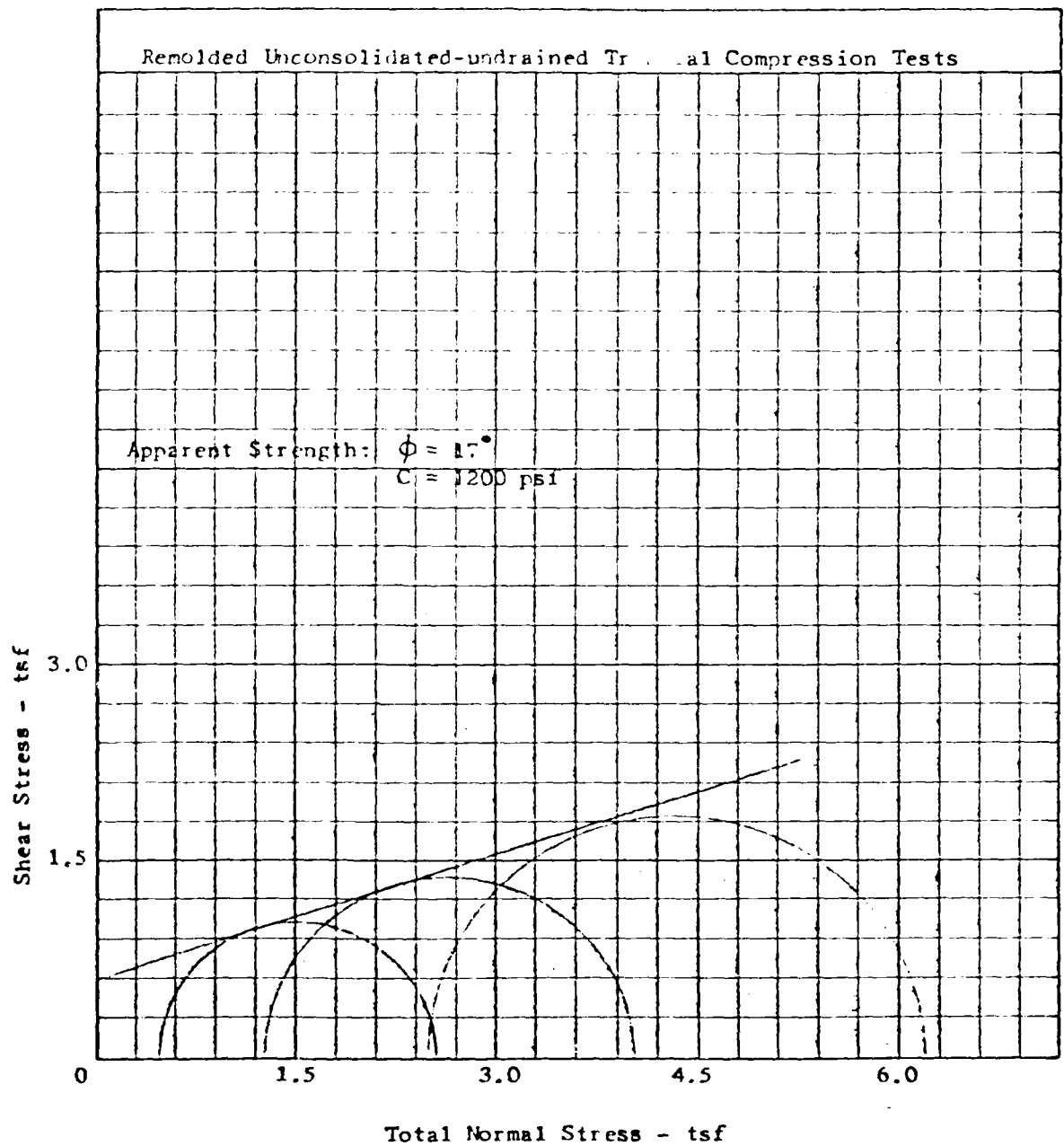
Unit Wet Weight: 121 pcf



Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

MOHR STRENGTH ENVELOPE



Project: Lake Caroline Dam

Location: Fredericksburg, Virginia

STRENGTH ENVELOPE

Remolded, Saturated, Consolidated-undrained Triaxial Compression Tests

Apparent Strength: $\phi = 21^\circ$
 $C = 1200$ psf

Effective Strength: $\phi' = 38^\circ$
 $C' = 0$

Vertical Stress - tsf

3.0

Effective

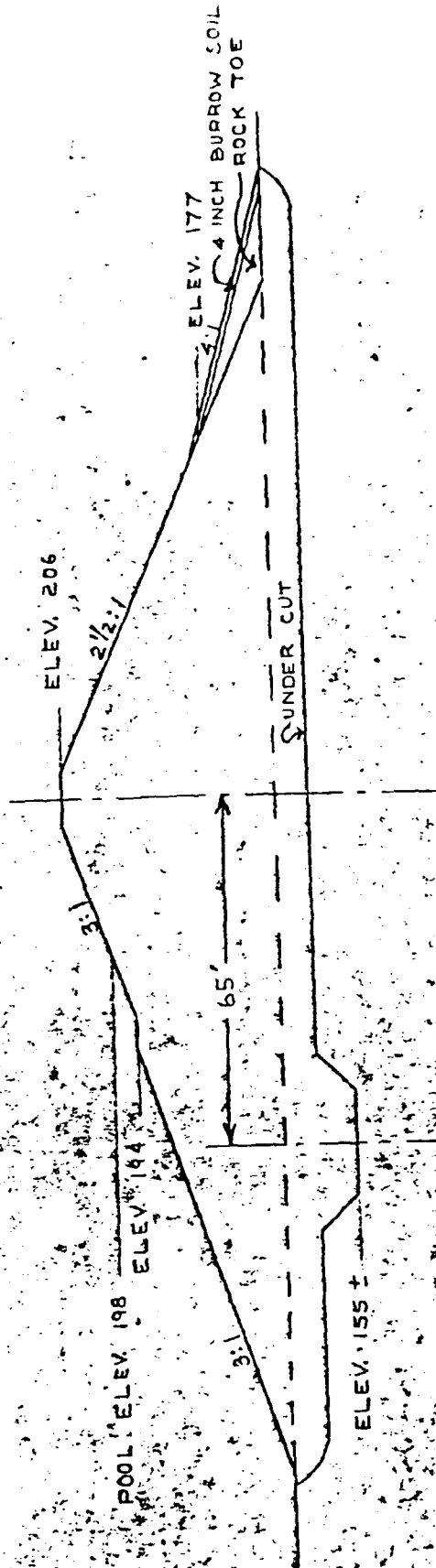
Apparent

3.0

4.5

6.0

Horizontal Stress - tsf



MODIFIED MAXIMUM EMBANKMENT SECTION LAKE CAROLINE DAM FREDERICKSBURG, VIRGINIA

SCALE: 1" = 30'

MODIFIED FROM SECTION BY ROBERT D. SAYRE P.E.

G. K. JEWELL AND ASSOCIATES
SOILS ENGINEERING CONSULTANTS

APPENDIX V

EARTHWORK AND TOE DRAIN SPECIFICATIONS

SECTION I

EARTHWORK

1.1. CLEARING AND GRUBBING:

(1) Work Included: The work under this Section includes furnishing all labor, materials and equipment for clearing and grubbing the site to be occupied by the permanent construction and the surface areas of all borrow and waste areas.

(2) Clearing: The areas to be occupied by permanent construction together with the surface areas of all borrow pits and stockpile sites shall be cleared of all trees, stumps, roots, brush, fences, poles, rubbish and other objectionable material. Such material shall become the property of the Contractor and shall be burned, removed from the job site or otherwise disposed of as approved by the Engineer.

(3) Burning: All material to be burned shall be piled neatly and when in suitable condition shall be burned completely. Piling for burning shall be done in such a manner and in such locations as to cause the least possible fire risk. Burning shall reduce all materials to ashes. The Contractor shall at all times take special precautions to prevent the spread of fires beyond cleared areas and shall have available equipment for use in preventing and suppressing fires.

1.2. EXCAVATION

(1) Work Included: This section of work shall include the stripping of the work area as shown on the drawings, the removal and disposition of unsuitable materials from the work area, the excavation of the cut-off trench, preparation of the embankment foundation area for backfill and the backfilling of all areas other than the embankment proper and around structures.

(2) Stripping: After the work area has been cleared and grubbed, it shall be stripped of sod, topsoil, organic material or other objectionable material to a minimum depth of one foot or to a greater depth if topsoil exists to such greater depth.

Suitable topsoil shall be stockpiled for later use upon the embankment or other filled areas. Objectionable material and excess topsoil shall be disposed of within the reservoir area as directed by the Engineer.

(3) Excavation of Foundation and Cut-off Trench: Excavation shall be to the lines and grades shown on the drawings or as established by the Engineer. During the progress of the work it may be found desirable to vary the slopes or dimensions of the excavations shown on the drawings.

All excavations for the embankment, cut-off trench or structures shall be under conditions free from flowing, standing or seeping water. During the progress of the work the surface shall be maintained so that it shall be well drained at all times. The excavation shall be maintained in the above-stated condition until such time as the embankment or structure has been constructed thereon. No excavation shall be made into frozen materials without the written permission of the Engineer.

(4) Disposition of Excavated Material: Excavated material suitable for use in the fill area may be used directly for that purpose or stock-piled for such later use. All such material must be approved for use by the Engineer. Materials excavated from the embankment foundation shall not be used in the construction of embankments without the express permission of the Engineer. Excess or undesirable excavated material shall be wasted within the reservoir area in the general vicinity of the construction as directed by the Engineer. All such waste areas shall be fine graded to drain properly and present a neat appearance.

(5) Excavation for Buried Structures: If the material at or below the normal grade of the bottom of the structure is unsuitable for foundation, it shall be replaced to such depths and widths as necessary with stabilized gravel.

(6) Preparation of Foundation Subgrade: The subgrade for the embankment shall be prepared by scalping to a six inch (6") depth or deeper if so indicated on the drawings, compacting and leveling so that the surface materials of the foundation will be as compact as hereinafter specified for the embankment proper. Surfaces upon which the earth-fill embankment is to be placed shall be cleared of all loose and objectionable material in an approved manner by hand work or other effective means.

The surface of the subgrade, immediately prior to placing the fill, shall have all surface water removed, soft and unstable areas excavated and shall be properly moistened and sufficiently clean to obtain a suitable bond with the earthfill.

1.3. EMBANKMENT:

(1) Work Included: The work under this Section includes furnishing all labor, materials, equipment and transportation necessary to complete the compacted earthfill embankment to the lines and grades shown on the drawings, including the backfill of the cut-off trench.

(2) Borrow sites other than those designated on the plans shall require the approval of the Engineer prior to excavation in that area.

Clearing, grubbing and stripping of the borrow areas shall be as required by other sections of these specifications.

The Engineer shall designate the depths of cut in all parts of the borrow pits; cuts shall then be made to such depths. The earthfill material delivered on the dam embankment shall be equivalent to a mixture of materials obtained from an approximately uniform cutting from the full height of the designated face of the borrow pit excavation. The type of equipment used and the Contractor's operation in the excavation of borrow material shall be such as will produce the required uniformity of mixture of the borrow materials. The plasticity index of the borrow material shall not be less than 10%. All dry, hard lumps of soil from the borrow pit shall be broken down so that the largest dimension of the lump is less than three (3) inches.

Excavated surfaces of borrow pits shall be graded to slopes not steeper than 1 1/2 to 1. The borrow pit shall be operated so as not to impair the usefulness or mar the appearance of any of the property of the Owner. Borrow pits will be maintained with sufficient slopes to prevent standing water and shall be provided with drainage ditches to convey storm water to natural outlets. Seepage zones, if encountered within the borrow area, must be controlled by properly placed drains to prevent the borrow material from becoming wet, weak and compressible.

The selection of suitable borrow materials shall be under the direction of the Engineer at all times. Materials having a dry weight of less than 110 pounds per cubic foot shall not be used in the embankments. Particles larger than 6 inches in diameter shall not be used in making the embankment.

(3) Placing Fill Material: Fill material shall be placed in successive layers of not more than 6 inches depth loose measure, placed approximately horizontal for the full length and width of the section. A crown of 2% slope shall be maintained on the cross-section of the fill area to provide drainage. Materials shall not be placed on areas softened by standing water. If it is necessary to provide a break in the length of the earthfill, the ends of the sections shall slope not more than 4 to 1. No frozen material shall be placed in the fill nor shall the embankment be placed on frozen ground. If the surface of a previously accepted lift of embankment becomes wet and unstable, such material shall be removed and replaced to such extent as directed by the Engineer without further compensation.

Before compaction in the fill areas, the borrow materials shall be brought to from 2% below to 2% above optimum moisture content. Optimum moisture shall be defined as that water content which will result in a maximum dry unit weight of the soil when subjected to the Standard Proctor Compaction Test, AASHTO T-99 or A.S.T.M. D-698.

Optimum moisture content for the borrow material shall be determined by the Engineer based on compaction control curves. Field density tests will also be performed on materials placed in the compacted fill during construction. Material which does not contain sufficient moisture to meet the above requirements shall be sprinkled with water at the borrow site or on the embankment as directed by the Engineer. Materials containing excess moisture shall be dried to meet the above limits prior to compaction. Drying of wet materials shall be accomplished by the use of plows, discs, harrows, scarifiers or power-driven mixing machines as approved by the Engineer.

The material in the cut-off trench and embankment shall be compacted to a minimum of 97% of the standard proctor density.

Tamping rollers shall be used for compacting the earthfill to the above density. The Contractor shall at all times maintain sufficient compacting equipment operating on the fill to balance the numbers of hauling equipment used to deliver and spread the borrow material.

When each layer of loose material has been prepared to the moisture content herein provided, it shall be compacted by passing the compacting roller, as specified above, over the layer a minimum of six times. Each pass shall overlap the preceding pass by not less than one foot. If tests indicate that less than the required degree of compaction has been obtained by this method, the weight of the roller may be increased, the number of passes of the roller may be increased, or the thickness of the layer of material may be decreased.

It shall be the responsibility of the Contractor to properly bond each succeeding layer of material to the layer previously rolled down. If the rolled surface of any layer is too smooth or too dry to bond properly with the succeeding layer, it shall be roughened by harrowing and moistened to the satisfaction of the Engineer before the succeeding layer is placed in the fill.

1.4. BACKFILL AROUND STRUCTURES:

Materials placed against, over and around the permanent concrete structures shall be classified as "Backfill around Structures." Applicable provisions of Section 1.3. "Embankment" shall apply to this work except that material shall be placed in layers not to exceed four inches loose measure and compacted by mechanical tampers. Mechanized equipment such as bulldozers or front end loaders shall not be used to push backfill material against the completed masonry. Care shall be taken to bring all fill up evenly.

SECTION 5

TOE DRAINS

5.1. WORK INCLUDED:

This item includes all labor, equipment and material not specifically listed as provided by the Owner, necessary to complete the work as stipulated in this specification and other contract documents.

(1) Material Furnished by Owner:

- [a] All fiber perforated pipe and fittings.

5.2. INSPECTION AND ACCEPTANCE OF MATERIAL BY CONTRACTOR:

In general sections 4.2. through 4.6. of these specifications shall govern the work included under this section.

5.3. TOE DRAIN FILTER:

The sand for the toe drain filter shall conform to the following gradation:

Passing 3/8" sieve	100%
Passing No. 4 sieve	95 - 100%
Passing No. 8 sieve	85 - 100%
Passing No. 30 sieve	60 - 90%
Passing No. 50 sieve	45 - 70%
Passing No. 200 sieve	5% maximum

The gravel for the toe drain filter shall be crushed rock or pit run gravel conforming to Virginia Department of Highways grading No. 19 or 20, except that the amount of material passing the No. 200 sieve shall not exceed 5%.

The sand and gravel used in the toe drain filter shall be free from dirt and organic matter. The material in the filter shall be compacted to the same density as the embankment.

APPENDIX VI - REFERENCES

1. Recommended Guidelines for Safety Inspection of Dams, Department of Army, Office of the Chief of Engineers, 46 pp.
2. Design of Small Dams, U. S. Department of Interior, Bureau of Reclamation, 1974, 816 pp.
3. Geologic Map of Virginia, Virginia Division of Mineral Resources, 1963.
4. HEC-1 Dam Break Version, Flood Hydrograph Package, Users Manual for Dam Safety Investigations, the Hydrologic Engineering Center, U. S. Army Corps of Engineers, September, 1978.
5. Hydrometeorological Report No. 33, U. S. Department of Commerce, Weather Bureau, U. S. Department of Army, Corps of Engineers, Washington, D. C., April, 1956.
6. Technical Paper No. 40, U. S. Department of Commerce, Weather Bureau, Washington, D. C., May, 1961.

