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NATIONAL DAM SAFETY PROGRAM. GEORGE F. BRASFIELD DAM. (VA-04101--ETC(U)

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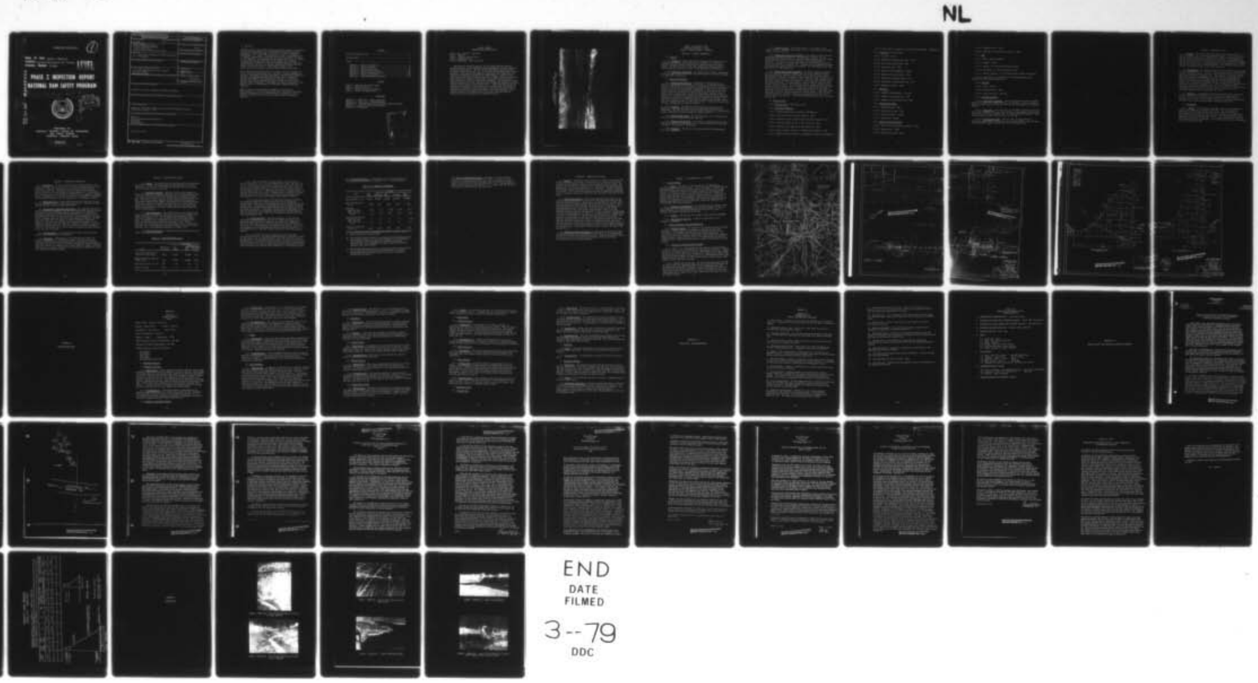

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APPOMATTOX RIVER BASIN

Name Of Dam: GEORGE F. BRASFIELD
Location: DINWIDDIE/CHESTERFIELD COS. VIRGINIA
Inventory Number: VA 04101

LEVEL II

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

(6) National Dam Safety Program. Dam George F. Brasfield (VA-04101), Appomattox River Basin, Dinwiddie/Chesterfield Counties, Virginia. Phase I Inspection Report.

(9) Final rept.

(12) 60p.

PREPARED BY
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

(11) MARCH 1978

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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 investigation 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 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.

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 and 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.

CONTENTS

PLATES

APPENDICES

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: George F. Brasfield
State: Virginia
County: Dinwiddie/Chesterfield
River: Appomattox River
Date of Inspection: 18 January 1978

The George F. Brasfield dam is a 55 feet high concrete, gravity structure located on the Appomattox River approximately 5 miles upstream of Petersburg, Virginia. The dam and its appurtenent structures were designed by Wiley and Wilson, Inc. in 1965 for the Appomattox River Water Authority. The geology and foundation conditions were evaluated by an experienced engineering geologist, Dr. Byron Cooper. A check of the stability analysis showed that the dam will be stable under the projected PMF. However, the PMF will overtop the non-overflow sections, thus jeopardizing the stability of the earth abutments. The inspection of the visible portions of the dam indicated minor seepage and spalling along the vertical and horizontal construction joints. These deficiencies are not considered detrimental to the stability of the structure, but should be included in a periodic inspection program. Severe erosion from surface runoff was noted on the downstream slope of the right abutment. This erosion should be corrected as soon as possible to prevent further deterioration of the abutment and a possible failure.

GOERGE F. BRASFIELD - OVERVIEW PHOTO
FROM D/S OF RIGHT ABUTMENT

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
GEORGE F. BRASFIELD DAM ID #VA04101

SECTION 1. 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 inspections 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 of the Phase I inspection is to identify expeditiously those dams which might be a potential hazard to human life or property.

1.2 Project Description

1.2.1 Dam and Appurtenances: The George F. Brasfield dam is a concrete gravity dam with a height of 55 feet at the maximum overflow section. The dam has a crest length of 1,250 feet and a crest width of 12 feet at the non-overflow section. Approximately, 840 feet of the dam's length serves as an ungated overflow spillway. A plan and profile and typical sections and details of the dam are shown on Plates 2 and 3, respectively. Appurtenances include four multilevel raw water intakes, twin 6'x 6' sluiceways with an upstream invert elevation of 114 MSL and a 36-inch cone valve with a downstream invert elevation of 124 MSL. The twin 6'x6' sluiceways and the cone valve sluiceways pass through the dam and are controlled by gates on the upstream face. Details of the wet well and gate arrangements are shown on Plate 4.

1.2.2 Location: The George F. Brasfield dam is located on the Appomattox River approximately five miles upstream from the City of Petersburg. The dam spans the boundary between Dinwiddie and Chesterfield County. Regional and vicinity maps are shown on Plate 1.

1.2.3 Size Classification: The dam classifies as an "Intermediate" size based on its height and storage capacity.

1.2.4 Hazard Classification: The George F. Brasfield dam was given a high hazard classification because of its close proximity to downstream residential and commercial development.

1.2.5 Ownership: The dam is owned and operated by the Appomattox River Water Authority.

1.2.6 Purpose of Dam: The primary purpose of the dam is water supply. However, numerous recreational areas are continuously developing adjacent to the reservoir.

1.2.7 Design and Construction History: The George F. Brasfield dam was designed by Wiley and Wilson, Inc. Lynchburg, Virginia for the Appomattox River Water Authority. Geologic investigations were performed by Dr. Byron N. Cooper, a past professor at Virginia Polytechnic Institute. A copy of his report and subsequent construction inspection reports are attached as Appendix III. Construction was completed in 1968 by the Wiley N. Jackson Construction Company. No modifications have been made since the 1968 completion.

1.2.8 Normal Operational Procedures: The dam is normally operated with the pool near elevation 158 MSL, the top of the ungated spillway. The adjacent water treatment plant draws 8 to 16 MGD for water supply. The project is required to release at least 100 cfs or the average of the previous 30 days inflow, whichever is less. The twin 6'x6' sluiceways are currently used to regulate this release. Excess flows are diverted over the ungated spillway. Instrumentation is used to monitor the pool level and the raw water intake. The recording devices on the overflow section provide a daily record of the pool level but are limited to a height of 24 inches above the spillway crest. A staff gauge mounted on the intake structure is used to measure levels above the two feet. In the past, this gauge has only been read during excessive runoff periods. The equipment associated with the raw water intake is inspected daily as part of a regular maintenance program. A maintenance program has not been developed for the dam.

1.3 Pertinent Data

1.3.1 Drainage Areas - 1335 square miles.

1.3.2 Discharge at Damsite

1.3.2.1 Maximum known flood at damsite - 405,000 cfs

1.3.2.2 Warm water outlet at pool elevation - N/A

1.3.2.3 Diversion tunnel low pool outlet at pool elevation - N/A

1.3.2.4 Diversion tunnel outlet at pool elevation - N/A

1.3.2.5 Gated spillway capacity at pool elevation - N/A

1.3.2.6 Gated spillway capacity at maximum pool elevation - N/A

1.3.2.7 Ungated spillway capacity at maximum pool elevation-99,000 cfs

1.3.2.8 Total spillway capacity at maximum pool elevation - 99,000 cfs

1.3.3 Elevation (ft. above MSL)

1.3.3.1 Top Dam - 167.5

1.3.3.2 Maximum pool-design surcharge - 167.5

1.3.3.3 Full flood control pool - N/A

1.3.3.4 Recreation pool - N/A

1.3.3.5 Spillway crest (ungated) - 158.0

1.3.3.6 Upstream sluiceway invert - 114

1.3.3.7 Downstream sluiceway invert - 113

1.3.3.8 Streambed at centerline of dam - 106

1.3.3.9 Maximum tailwater - 150.2

1.3.4 Reservoir

1.3.4.1 Length of maximum pool - 17 miles

1.3.4.2 Length of recreation pool - N/A

1.3.4.3 Length of flood control pool - N/A

1.3.5 Storage (acre-feet)

1.3.5.1 Recreation pool - N/A

1.3.5.2 Flood control pool - N/A

1.3.5.3 Design surcharge - 44,100

1.3.5.4 Top of dam - 79,500

1.3.6 Reservoir Surface (acres)

1.3.6.0 Top of dam (non-overflow section) - 6,750

1.3.6.2 Maximum pool - 6,750

1.3.6.3 Flood-control pool - N/A

1.3.6.4 Recreation pool - N/A

1.3.6.5 Spillway crest (overflow section) - 3,060

1.3.7 Dam

1.3.7.1 Type - Concrete Gravity

1.3.7.2 Length - 1,250'

1.3.7.3 Height - 55' @ maximum overflow section

1.3.7.4 Top Width - 12' @ non-overflow section

1.3.7.5 Concrete keyway - 5' x 5'(located U/S of dam's centerline)

1.3.7.6 Grout holes - 3-1/2" DIA by 50' randomly placed at the direction of the consulting geologist

1.3.8 Spillway

1.3.8.1 Type - ungated

1.3.8.2 Length of weir - 840'

1.3.8.3 Crest elevation - 158' MSL

1.3.9 Regulating Outlets

1.3.9.1 Twin 6'x6' Sluiceways: These sluiceways are used to regulate required releases and to lower the pool, if necessary. These sluiceways have a constant invert elevation of 114' MSL and are gated on the upstream intake.

1.3.9.2 Cone Valve: The cone valve was designed to precisely control the release quantity. The cone valve intake is a gated 48-inch pipe which necks down to a 36-inch diameter near the downstream discharge point. The invert elevation of the downstream face is 124' MSL.

1.3.9.3 48" Raw Water Pipe: The raw water is drawn through 4 multilevel gated intakes into the screened pumping chamber. The raw water is then pumped through a 48-inch pipe to the treatment plant.

like a bartender more ~~[illegible]~~

Operation

Produce elec. from 1900–1923
Water from Reser.
Debris & silt
2 Record Floods
No damage

SECTION 2: ENGINEERING DATA

2.1 Design: All readily available design data were furnished by Wiley and Wilson, Inc., the design engineering firm. These data included the site geology report by Dr. Byron Cooper, hydrologic and hydraulic design curves and structural stability calculations. The geology report and the results of the stability analyses have been attached as Appendices III and IV, respectively. A seepage analysis was performed by Dr. Cooper, but the results were not readily obtainable. The remaining data has been filed in the Norfolk District for future reference.

2.2 Construction: The dam was constructed and completed by the Wiley N. Jackson Construction Co. in 1968. Wiley and Wilson, Inc. furnished the quality assurance program. Copies of the foundation inspection reports have been attached with Appendix III. These reports were prepared by Dr. Cooper and include recommendations for river diversion, grouting and foundation preparation. Additional records including daily quality control reports and progress photographs are on file with Wiley and Wilson, Inc. in Lynchburg, Virginia. A copy of the contract specification is on file with Wiley and Wilson, Inc. and in the Norfolk District. Available construction documents indicate that the contract specifications and drawings were followed for the most part. However, a set of "as-built" drawings was not available for review.

2.3 Operation: The Appomattox River Water Authority is responsible for the operation and maintenance of the dam and its appurtenances. However, current operation procedures are generally limited to those necessary for control of water supply. Flow and normal operation records are filed at the water treatment plant adjacent to the dam.

2.4 Evaluation

2.4.1 Design: The available design data was more than adequate to evaluate the structural stability of the dam. The only possible deficiency in the design was the fact that the ends of the concrete non-overflow section were not tied into rock. However, economics would have prohibited a design with this feature. The relatively impervious abutments, if properly maintained, will perform satisfactorily until the non-overflow section is overtopped. It is estimated that this overtopping will occur during the PMF.

2.4.2 Construction: Available construction data was limited to the foundation inspection reports by Dr. Byron Cooper and visual observations made by the inspection team. Additional construction records can be obtained from Wiley and Wilson, Inc.; however, a great deal of time and expense would be required to research the daily quality control and quality assurance reports. This expenditure of time and money is not authorized or deemed necessary under a Phase I Inspection.

2.4.3 Operation: The operational procedures are adequate for the water supply facilities. However, the operation and maintenance program should be expanded to include a periodic inspection of the dam and an operation and maintenance program for the regulating outlets.

SECTION 3: VISUAL INSPECTION

3.1 General: A field inspection of the George F. Brasfield dam and its appurtenances was conducted on 18 January 1978. At the time of the inspection, the elevations of the pool and tailwater were approximately 160' and 121' MSL, respectively. These water levels prohibited the inspection of the overflow spillway section and the downstream toe area. In addition to Norfolk District personnel, the following personnel participated in the field inspection of the dam:

Mr. John Tabasko, Appomattox River Water Authority
Mr. I. P. Sandhu, State Water Control Board
Mr. Charles Martin, State Water Control Board

3.2 Findings: The results of the field inspection are attached as Appendix I.

3.3 Evaluations

3.3.1 Dam Structure: Minor seepage and spalling was noted on the visible portions of the non-overflow sections of the dam. In all cases, the minor seepage and spalling was confined to the construction or monolith joints. In most cases, the only evidence of seepage activity was staining in the form of precipitated calcite. The only active seepage was located on the right non-overflow section, approximately 6 feet above the abutment contact. This seepage appeared as a wet area and is not considered an immediate threat to the structural integrity of the dam. The spalling appears to be the result of attempts to patch monolith joints with lean grout. The minor seepage and spalled areas should be periodically inspected to determine if these deficiencies are progressing.

3.3.2 Abutments: The erosion on the downstream slope of the right abutment is considered a problem that will require corrective action. If allowed to continue, the unimpeded erosion could possibly cut the abutment slopes back to the abutment and end of dam contact. This action would result in a shorter seepage path and a possible failure of the abutment. Additional erosion was observed on the upstream shoreline of the left abutment. This erosion can be corrected by the use of approved shore protection.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Procedures: The only operational procedures pertain to the intake structure. No set pool level has been established; however, the pool level is usually maintained at or near the overflow spillway elevation of 158' MSL. A minimum release as previously specified in Section 1 has been established by the City of Hopewell. In addition to this minimum release, 8 to 16 MGD are withdrawn by the adjacent treatment plant. The pumps used to pump the raw water to the plant are operated on an alternating sequence to insure operation in an emergency situation.

4.2 Maintenance Dam: Actual maintenance of the structure has not been accomplished since the construction of the dam. This type of dam usually does not require periodic maintenance.

4.3 Maintenance of Operating Facilities: The operating facilities associated with the pumping of raw water are maintained on a daily basis by personnel from the Appomattox River Water Authority. Additional preventive maintenance in the form of lubrication is performed on the accessible valve stems for the sluiceways. The operability of the twin 6'x 6' sluiceway gates has only been tested during periods of extremely high flows when they were opened to prevent flooding of the pump chambers. The cone valve has only been used on one occasion. Plant personnel indicated that the valve was extremely difficult to close and eroded the downstream slope. Therefore, after the valve was finally closed, it was decided not to use it unless circumstances prevented the use of the twin sluiceways.

4.4 Warning System: At the present time, there are no warning systems or evacuation plans in operation.

4.5 Evaluation: Maintenance of the operating facilities are considered adequate for the raw water supply. However, a periodic operation and maintenance program should be established for the twin sluiceways and the cone valve. The effect of permanently abandoning the cone valve should be evaluated by the design engineer. Also, a plan for removing debris from the intake bar screen should be established as part of a periodic maintenance program.

SECTION 5. HYDRAULIC/HYDROLOGIC

5.1 Design. The engineering firm designing the dam determined the probable maximum flood discharge would be 130,000 c.f.s. which would cause a rise of 12.0 feet over the spillway and 2.5 feet over the non-overflow section of the dam.

5.2 Hydrologic Records. Records of flow in the Appomattox River are maintained at Mattoax (drainage area 726 square miles) which is located 36 miles upstream from the dam and at Matoaca (drainage area 1,344 square miles) which is located 3 miles downstream from the dam. These stream gaging stations are maintained by the Virginia State Water Control Board (SWCB). The operators of the dam maintain a continuous record of the pool level from 0 up to 24 inches above the crest of the spillway. Periodic readings are made on a staff gage during periods when the pool level is more than 24 inches above the crest of the spillway.

5.3 Flood Experience. The largest flood in over 50 years of record occurred in October 1972 when the water reached a height of 63 inches (elevation 163.25) over the ungated spillway when a peak discharge of 40,800 c.f.s. was recorded at the downstream Matoaca gage. A highwater mark established by the Corps of Engineers just below the dam indicated a tailwater elevation of 132. This is estimated to be approximately a 1 percent chance flood. The flood of June 1972 rose to a height of 44.50 inches (elevation 161.71) in the reservoir with a corresponding discharge of 22,800 c.f.s.

5.4 Reservoir Operation. Pertinent dam and reservoir data are shown in the following table:

Table 5.1 DAM AND RESERVOIR DATA

		Reservoir		
			Capacity	
Item	Elevation ft., m.s.l.	Area acres	Acre feet	Watershed inches
Top of dam, non-overflow section 365 feet long	167.5	6,750	79,500	1.12
Ungated spillway, 840 feet long, crest	158	3,060	35,400	0.50
Two gated sluices, 6' X 6'	114	0	0	0
Normal riverbed	109	0	0	0

5.4.1 Water withdrawals varying roughly between 8 and 16 M.G.D. (12 and 24 c.f.s.) are made from the reservoir. The two 6 ft. x 6 ft. sluices are set to release enough flow to maintain a flow of 100 c.f.s. at Hopewell which is at the mouth of the river. There is no controlled flood storage space in the reservoir and the average flow is about 1,150 c.f.s. Therefore pool level is slightly above the spillway crest most of the time. Determination of probable discharge from reservoir in various floods utilized surcharge storage above spillway crest with no release assumed from low-level outlets.

5.4.2 A spillway rating curve furnished by Wiley and Wilson was adjusted slightly to reflect reservoir rise and discharge experienced in downstream stream gaging station in floods of June and October 1972. A reservoir storage capacity curve furnished by Wiley and Wilson was extended above elevation 164 based on the area of the elevation 170 contour. A tailwater rating curve was developed on the basis of the October 1972 highwater mark and water surface profiles made in connection with a flood plain information study for the area downstream from the dam.

5.5 Flood Potential. The flood potential at the dam was determined by application of rainfall to a hydrologic model of the watershed which had been developed by the Hydrologic Engineering Center in connection with a study following wide spread flooding in June 1972 in the Middle Atlantic and Northeastern States. Because of the broadness of the hydrograph and limited surcharge storage it is considered that the reduction of peak flow by the reservoir would be minimal.

5.5.1 Rainfall for floods computed was taken from NWS publication HMR 33 for the Probable Maximum Flood, and Technical Paper No. 40; atlas 2 for the 100-Year Flood. EM 1110-2-1411 was used for determination of the Standard Project Flood. The peak of the 100-Year Flood determined for various points in the basin based on a frequency analysis of peak discharge records indicated very close agreement with that determined from rainfall and utilization of the hydrologic model.

5.6 Overtopping Potential. The probable rise in the reservoir and other pertinent information is summarized in the following table:

Table 5.2. RESERVOIR PERFORMANCE

Item	Avg. flow	Flood			
		One Percent 1/	SPF 2/	1/2 PMF	PMF 3/
Peak outflow, c.f.s.	1,150	40,000	62,000	87,500	175,000
Peak elevation, ft. m.s.l.	158.5	163	165	167	175
Spillway					
Depth of flow	0.5	5.2	6.95	8.75	13.5
Avg. velocity, f.p.s.	2.7	9.2	10.6	11.9	15.4
Non-overflow section					
Depth of flow	0	0	0	0	4.0
Avg. velocity, f.p.s.	0	0	0	0	5.2
Tailwater elevation, ft. m.s.l.	118	132	136	140	150

1/ The 1 Percent Exceedence Frequency Flood has 1 chance in 100 of being exceeded in any given year.

2/ The Standard Project Flood is an estimate of flood discharges that may be expected from the most severe combination of meteorologic and hydrologic conditions that are considered reasonably characteristic of the geographical region involved, excluding extremely rare combinations.

3/ The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

5.7 Reservoir Emptying Potential. Assuming an average inflow of 1,150 c.f.s. and a minimum tailwater of 116.0 feet, m.s.l. exists during the drawdown operation, it would take a minimum of 10 days to bring the pool level to elevation 120.0 feet m.s.l. This operation is based on the assumption that the two 6' x 6' sluices and the 36 inch cone valve are all operative and fully open.

SECTION 6: STRUCTURAL STABILITY

6.1 General: The dam has been analyzed for sliding, overturning and foundation pressure conditions. Calculations for stability at both the overflow and non-overflow sections indicate that the dam is stable. The results of the stability analysis for the overflow section is attached as Appendix IV. Two loading conditions were considered. The PMF condition was considered first (headwater elevation = 170 feet, tailwater elevation = 153 feet). The second load condition involved a hypothetical condition having a headwater elevation of 170 feet and a tailwater elevation of 116 feet.

6.2 Visual Observations: The upstream and downstream faces of the right and left non-overflow sections of the dam show waterstains at most of the construction joints. These stains are probably due to water from drainage scuppers flowing down the dam faces and into the joints. The construction joints extend through the monoliths of the dam spaced 40' to 65' horizontally and 6' vertically. Through several joints at the downstream face of the right non-overflow section, there is visible evidence of minor seepage. The most active joint is located approximately 6' above existing grade and is closest to the spillway wall. Seepage from this joint is estimated at less than one gallon per minute (1 GPM). The presence of efflorescence, concrete spalling and minor cracking was observed along the faces of the joints. The concrete decks of the right and left non-overflow sections of the dam showed transverse and minor diagonal cracking. Transverse cracks up to 1/8" in width appear mainly at each construction joint of the parapets and extend across the deck between the parapet faces. The diagonal cracking occurs randomly throughout the parapet deck.

6.3 Design and Construction Data: Design data concerning stability of the dam conforms with the requirements specified in the "Recommended Guidelines for Safety of Inspection of Dams". As-built drawings to verify the accuracy of the design data and dimensions used in the design calculations were not readily available.

SECTION 7: ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment

7.1.1 Safety: The dam will pass the SPF and 1/2 PMF without overtopping the non-overflow section. Wiley and Wilson indicated that the PMF would overtop the non-overflow section by 2-1/2 feet. The Norfolk District calculations predicted a slightly higher PMF discharge which results in a depth of 4 feet on the non-overflow section. Stability calculations indicate that the concrete gravity dam will be structurally stable during the PMF flood conditions indicated by Wiley and Wilson. However, the integrity of the abutment slopes would be jeopardized by an overtopping condition and would, therefore, govern the safety of the project under PMF flood conditions.

7.1.2 Adequacy of Information: The available data is adequate with the exception of the non-availability of as-built drawings. A set of design drawings and corresponding design calculations were used to evaluate the physical parameters of the dam.

7.1.3 Urgency: The dam will not require urgent remedial treatment.

7.1.4 Necessity for Phase II: A Phase II inspection is not considered necessary.

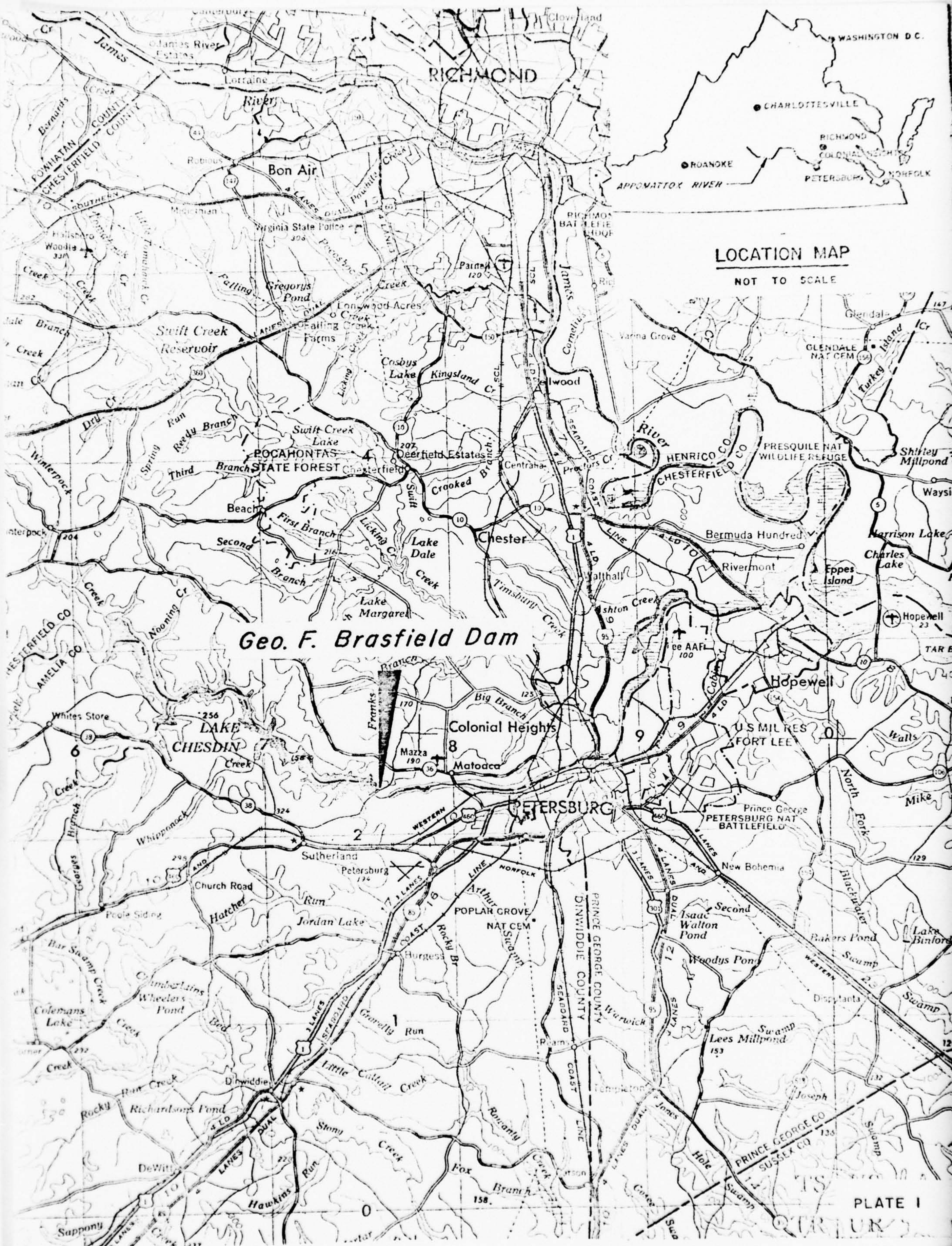
7.2 Remedial Measures and Recommendations

7.2.1 Remedial Action: The erosion of the abutment slopes should be corrected as soon as possible to prevent a potential problem from developing. It is suggested that the areas affected by the surface erosion be stabilized by proper grading and seeding techniques. In conjunction with this treatment, the surface runoff from the surrounding slopes should be diverted to the river via a paved channel to prevent further erosion. Riprap should be used to arrest the upstream shoreline erosion.

7.2.2 Operation and Maintenance Programs:

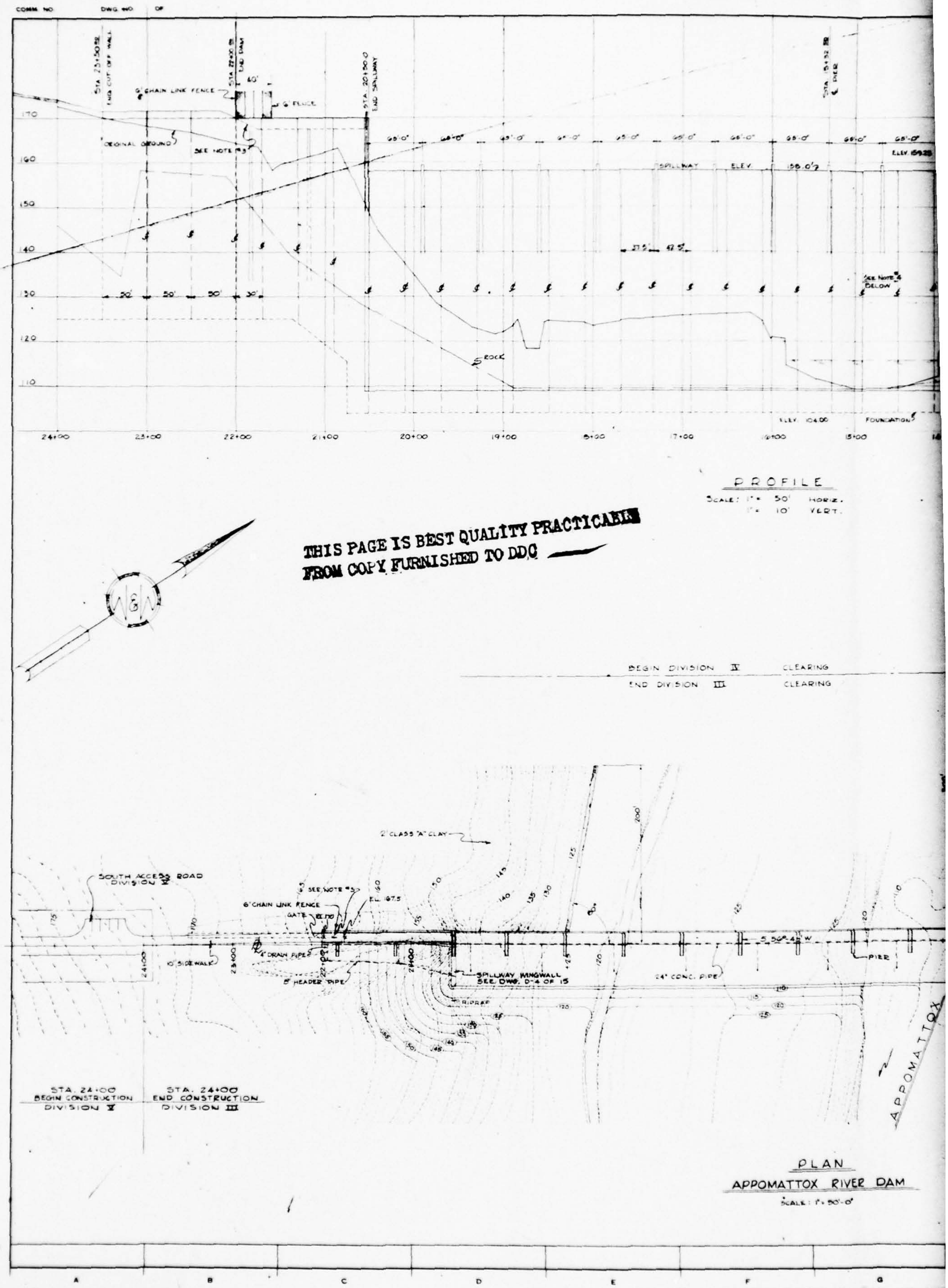
7.2.2.1 Dam: It is recommended that an operation and maintenance program be developed for the dam and its appurtenant structures. Because the dam is a concrete gravity structure, the program for the dam, itself, can be limited to periodic visual inspections. These visual inspections should cover known seepage and spall areas, the overflow sections and the foundation drain laterals. It will be necessary to inspect these latter features during low flow conditions.

7.2.2.2 Appurtenant Structures: The facilities associated with the raw water intake are adequately operated and maintained by personnel of the Appomattox River Water Authority. It is recommended that this operation and maintenance program be expanded to include the twin 6'x6' sluiceways and the 36-inch cone valve. Also, procedures for removing trash and/or debris from the bar screen should be established as part of the adopted program.



LOCATION MAP
NOT TO SCALE
WASHINGTON D.C.
CHARLOTTESVILLE
RICHMOND
COLONIAL HEIGHTS
ROANOKE
PETERSBURG
NORFOLK
APPOMATTOX RIVER
Geo. F. Brasfield Dam
RICHMOND
Bon Air
Swift Creek Reservoir
POCAHONTAS STATE FOREST
Chesterfield
Chester
LAKE CHESDIN
Colonial Heights
Matoaca
PETERSBURG
Hopewell
U S MIL RES FORT LEE
PETERSBURG NAT BATTLEFIELD
Sutherland
POPLAR GROVE NAT CEM
Dinwiddie
DINWIDDIE COUNTY
PRINCE GEORGE COUNTY
PRESQUILE NAT WILDLIFE REFUGE
HENRICO CO.
CHESTERFIELD CO.

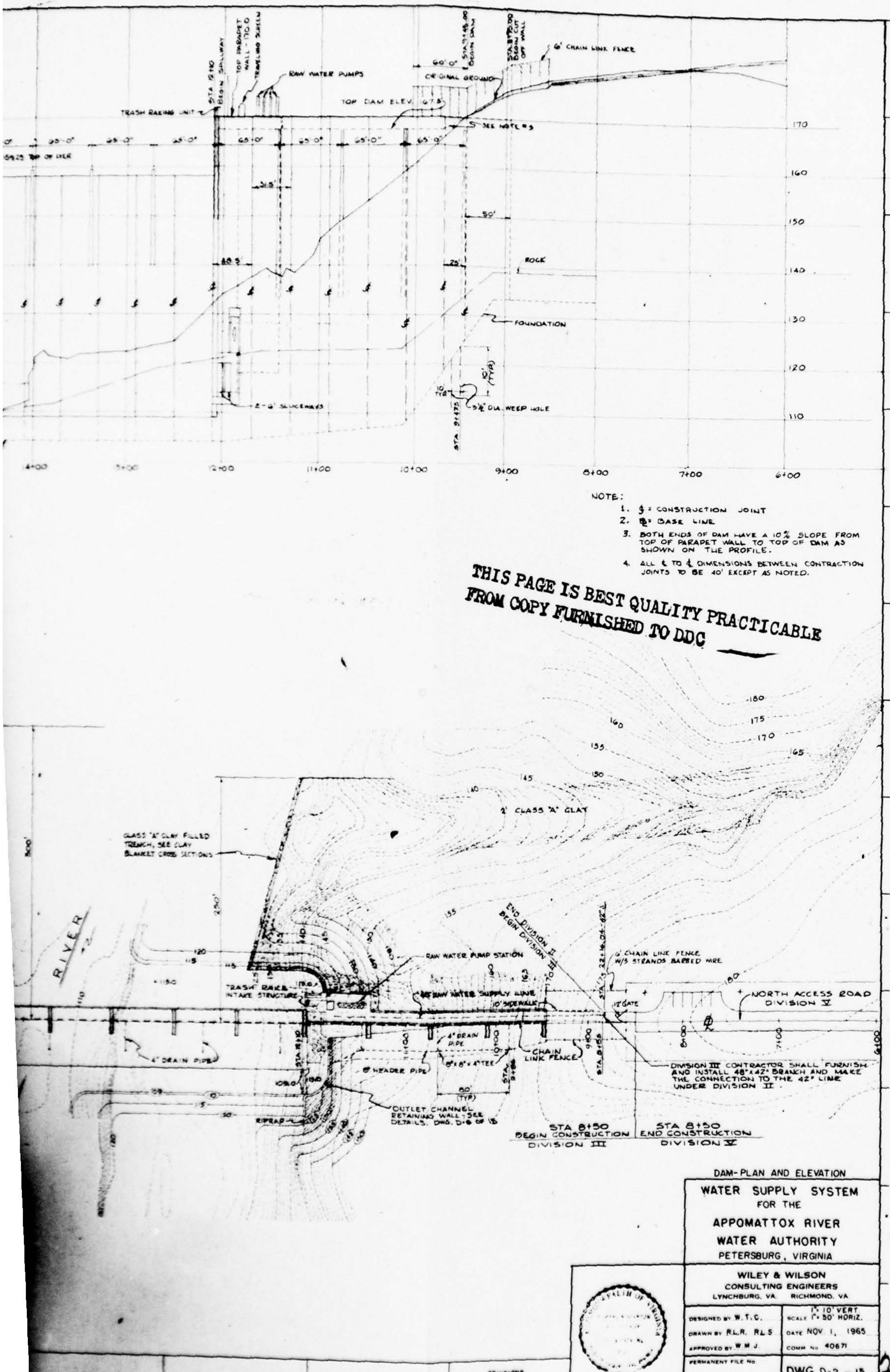
PLATE I



PLAN

APPOMATTOX RIVER DAM

SCALE: 1" = 50'-0"



NOTE:

1. ℊ = CONSTRUCTION JOINT
2. Ɓ = BASE LINE
3. BOTH ENDS OF DAM HAVE A 10% SLOPE FROM TOP OF PARAPET WALL TO TOP OF DAM AS SHOWN ON THE PROFILE.
4. ALL ℄ TO ℄ DIMENSIONS BETWEEN CONTRACTION JOINTS TO BE 40' EXCEPT AS NOTED.

DAM-PLAN AND ELEVATION

WATER SUPPLY SYSTEM
FOR THE
APPOMATTOX RIVER
WATER AUTHORITY
PETERSBURG, VIRGINIA

WILEY & WILSON
CONSULTING ENGINEERS
LYNCHBURG, VA. RICHMOND, VA.

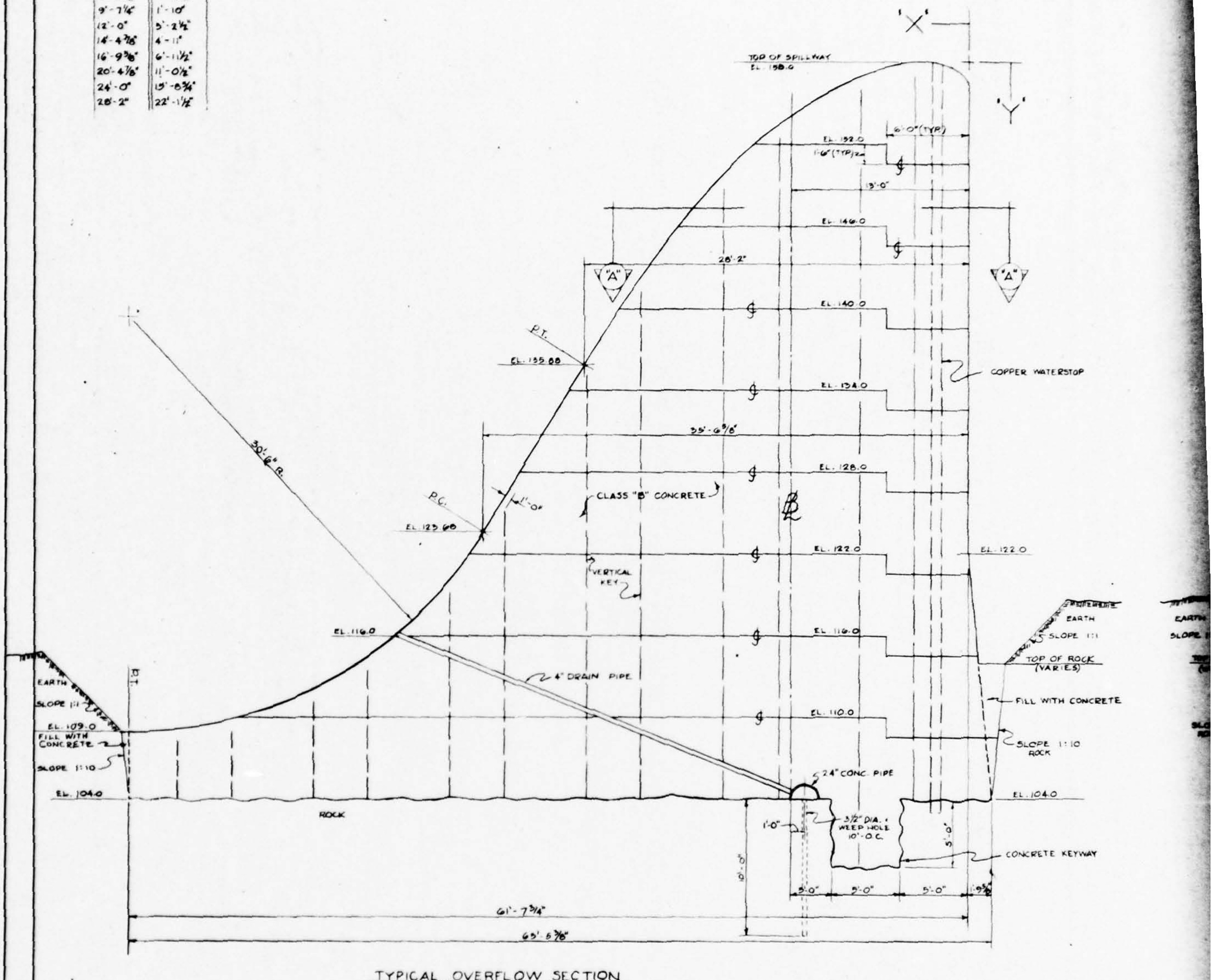
DESIGNED BY W.T.C.	SCALE 1"=10' VERT. 1"=50' HORIZ.
DRAWN BY R.L.R. R.L.S.	DATE NOV. 1, 1965
APPROVED BY W.M.J.	COMM. No. 40671
PERMANENT FILE No.	DWG. D-2 of 15

REVISIONS

PLATE 2

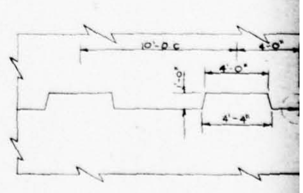
COMM. NO. DWG. NO. OF

SPILLWAY CREST DIMENSIONS	
"X"	"Y"
	1'-6⅛"
1'-2½"	5¼"
2'-4⅞"	1"
3'-7¼"	—
4'-9⅝"	1"
7'-2½"	9⅛"
9'-7¼"	1'-10"
12'-0"	3'-2½"
14'-4⅞"	4'-11"
16'-9⅜"	6'-11½"
20'-4⅞"	11'-0½"
24'-0"	15'-8¾"
28'-2"	22'-1½"



TYPICAL OVERFLOW SECTION

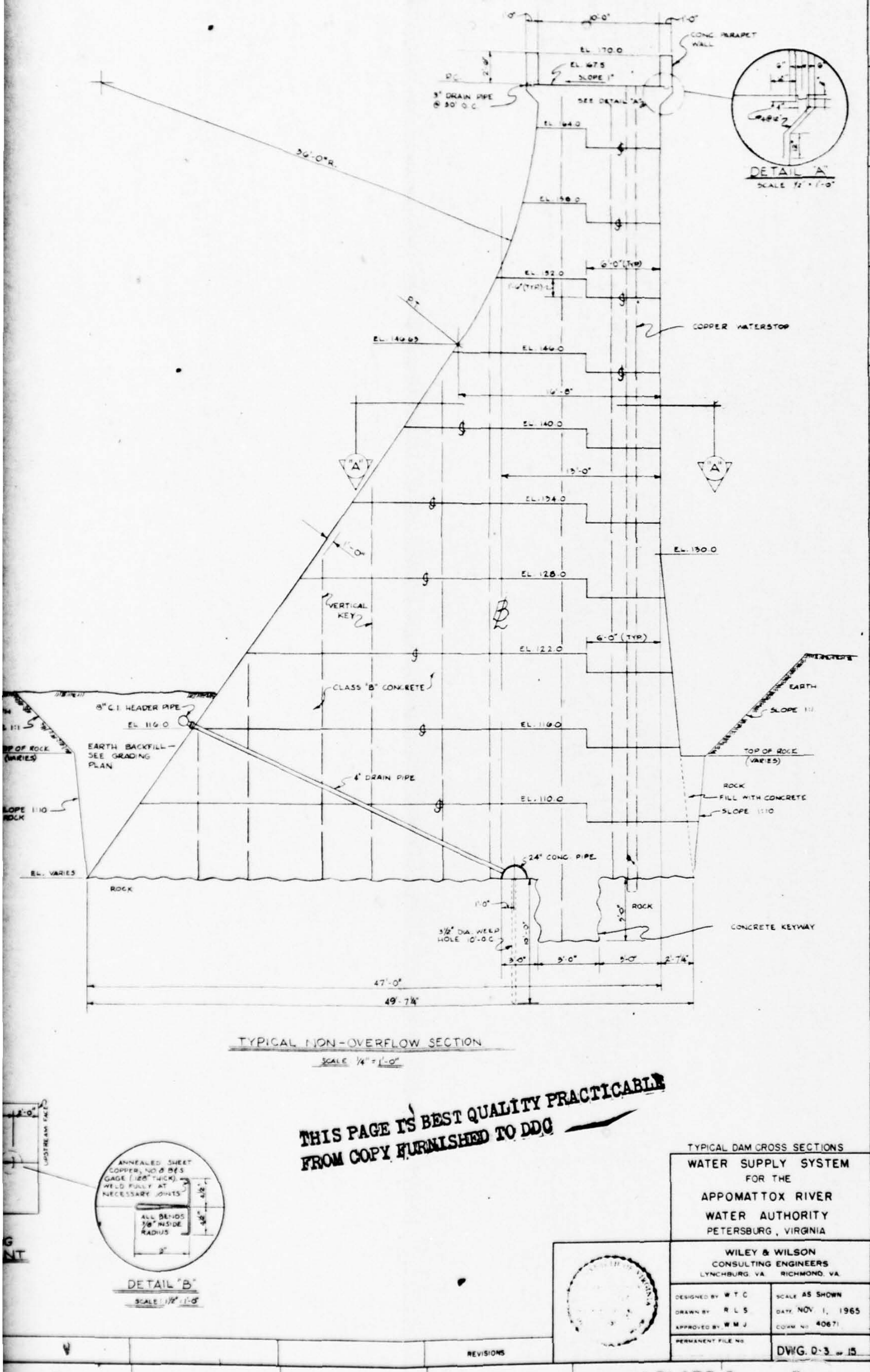
SCALE: ¼" = 1'-0"



SECTION "A-A" SHOWING VERTICAL CONTRACTION JOIN

SCALE: ¼" = 1'-0"

A B C D E F G



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TYPICAL DAM CROSS SECTIONS

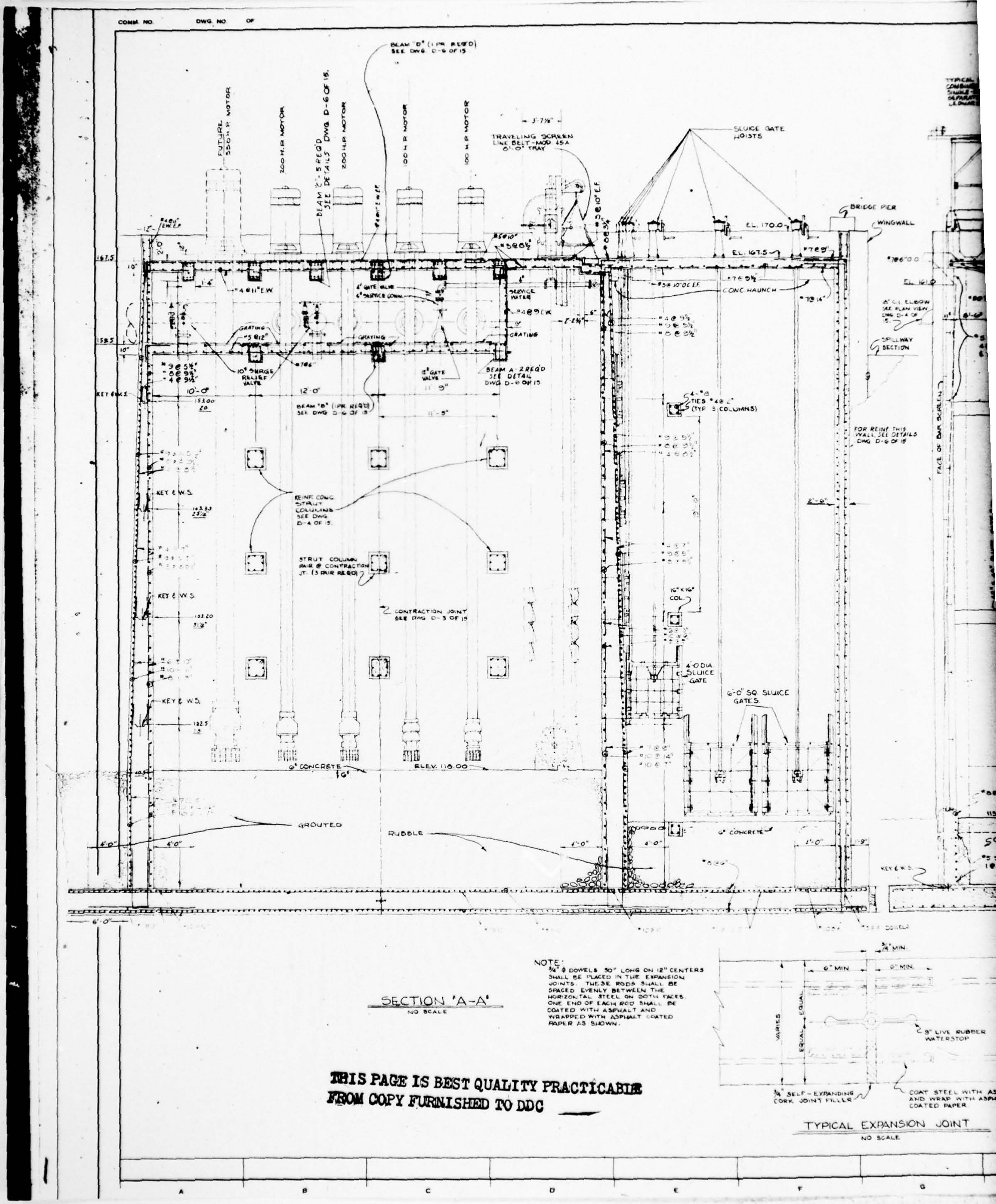
WATER SUPPLY SYSTEM
FOR THE
APPOMATTOX RIVER
WATER AUTHORITY
PETERSBURG, VIRGINIA

WILEY & WILSON
CONSULTING ENGINEERS
LYNCHBURG, VA. RICHMOND, VA.

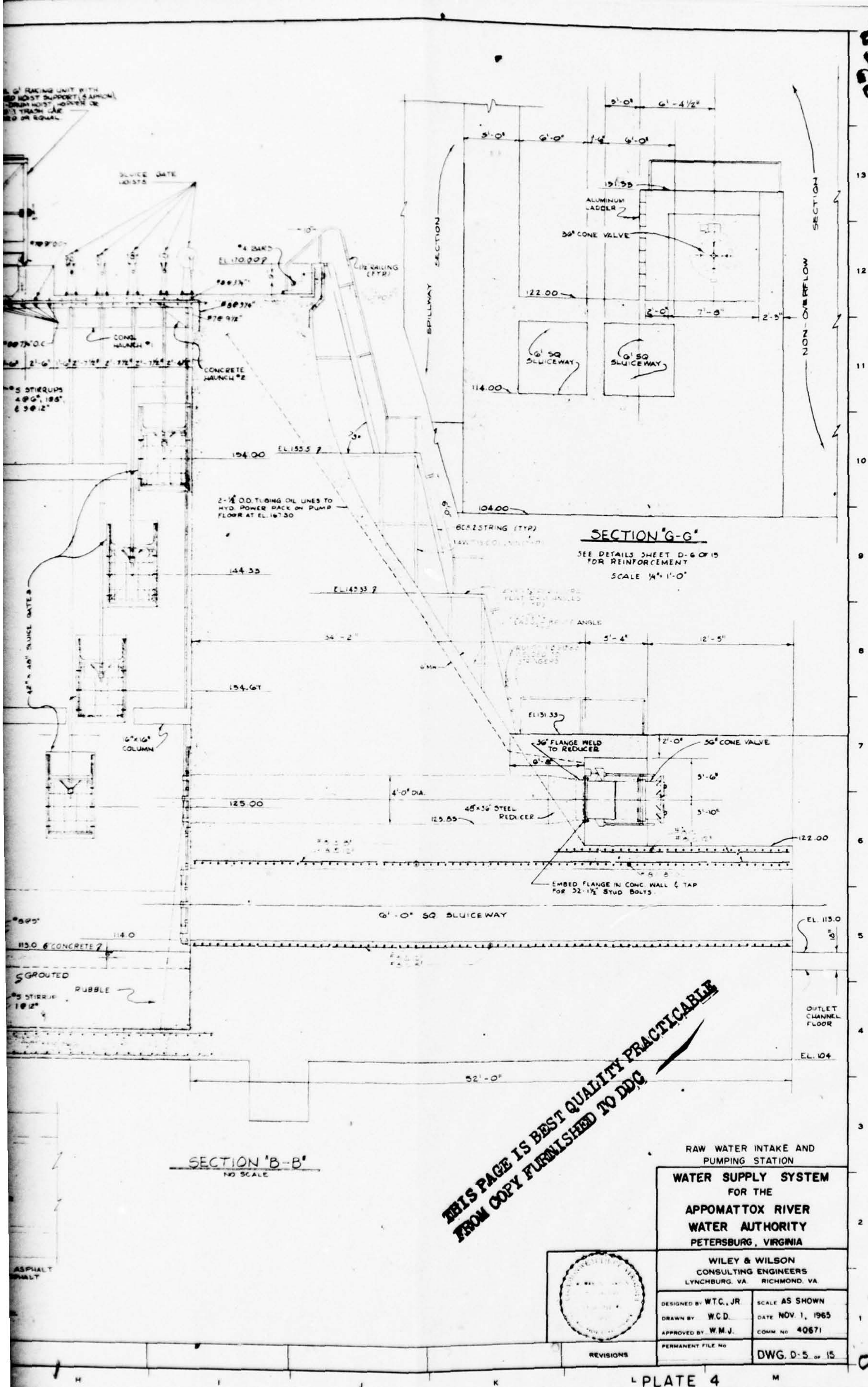
DESIGNED BY W.T.C.	SCALE AS SHOWN
DRAWN BY R.L.S.	DATE NOV. 1, 1965
APPROVED BY W.M.J.	COMM. NO. 40671
PERMANENT FILE NO.	DWG. D-3 of 15

REVISIONS

PLATE 3



COMM. NO.
DWG. NO.
OF
BEAM "D" (1 PR. REQ'D)
SEE DWG. D-6 OF 15
FUTURE 350 H.P. MOTOR
200 H.P. MOTOR
BEAM "C" - 5 REQ'D SEE DETAILS DWG. D-6 OF 15.
200 H.P. MOTOR
100 H.P. MOTOR
100 H.P. MOTOR
TRAVELING SCREEN
LINK BELT - MOD 45A
6'-0" TRAY
SLUICE GATE HOISTS
BRIDGE PIER
WINGWALL
EL. 170.0
EL. 167.5
CONC HAUNCH
SERVICE WATER
GRATING
4" GATE VALVE
4" SERVICE CONN.
10" SURGE RELIEF VALVE
12" GATE VALVE
BEAM A - 2 REQ'D SEE DETAIL DWG. D-6 OF 15
BEAM "B" (1 PR. REQ'D) SEE DWG. D-6 OF 15
SPILLWAY SECTION
18" C.I. ELBOW SEE PLAN VIEW DWG. D-4 OF 15
4-#8 TIES #4 @ 12" (TYP. 3 COLUMNS)
FOR REINF. THIS WALL SEE DETAILS DWG. D-6 OF 15
FACE OF BAR SCREEN
REINF. CONC. STRUT COLUMNS SEE DWG. D-4 OF 15.
STRUT COLUMN PAIR @ CONTRACTION JT. (3 PAIR REQ'D)
CONTRACTION JOINT SEE DWG. D-3 OF 15
16" X 16" COL.
4'-0 DIA. SLUICE GATE
6'-0" SQ. SLUICE GATES
KEY & W.S.
6" CONCRETE
ELEV. 118.00
GROUTED
RUBBLE
NOTE:
3/4" ϕ DOWELS 30" LONG ON 12" CENTERS SHALL BE PLACED IN THE EXPANSION JOINTS. THESE RODS SHALL BE SPACED EVENLY BETWEEN THE HORIZONTAL STEEL ON BOTH FACES. ONE END OF EACH ROD SHALL BE COATED WITH ASPHALT AND WRAPPED WITH ASPHALT COATED PAPER AS SHOWN.
SECTION "A-A"
NO SCALE
3/4" MIN.
6" MIN.
VARIES
EQUAL
9" LIVE RUBBER WATERSTOP
3/4" SELF-EXPANDING CORK JOINT FILLER
COAT STEEL WITH AS... AND WRAP WITH ASP... COATED PAPER
TYPICAL EXPANSION JOINT
NO SCALE
A
B
C
D
E
F
G


SECTION "G-G"
SEE DETAILS SHEET D-6 OF 15 FOR REINFORCEMENT
SCALE 1/4" = 1'-0"
SPILLWAY SECTION
NON-OVERFLOW SECTION
ALUMINUM LADDER
36" CONE VALVE
6' SQ SLUICEWAY
122.00
114.00
104.00
SLUICE GATE HOISTS
RAILING (TYP)
CONC HAUNCH #1
CONCRETE HAUNCH #2
#5 STIRRUPS
154.00
EL. 155.5
2-1/2" O.D. TUBING OIL LINES TO HYD. POWER PACK ON PUMP FLOOR AT EL. 167.30
6C8.2 STRINGER (TYP)
144.33
EL. 143.33
42" x 48" SLUICE GATES
134.67
EL. 131.33
36" FLANGE WELD TO REDUCER
36" CONE VALVE
16"x16" COLUMN
4'-0" DIA.
48"x36" STEEL REDUCER
125.00
125.33
122.00
EMBED FLANGE IN CONC. WALL & TAP FOR 32-1 1/2" STUD BOLTS
6'-0" SQ SLUICEWAY
EL. 113.0
114.0
113.0 6" CONCRETE
GROUTED RUBBLE
OUTLET CHANNEL FLOOR
EL. 104
52'-0"
SECTION 'B-B'
NO SCALE
ASPHALT
RAW WATER INTAKE AND PUMPING STATION
WATER SUPPLY SYSTEM FOR THE
APPOMATTOX RIVER WATER AUTHORITY
PETERSBURG, VIRGINIA
WILEY & WILSON
CONSULTING ENGINEERS
LYNCHBURG, VA. RICHMOND, VA
DESIGNED BY W.T.C., JR.
DRAWN BY W.C.D.
APPROVED BY W.M.J.
PERMANENT FILE No
SCALE AS SHOWN
DATE NOV. 1, 1965
COMM. No. 40671
DWG. D-5 of 15
REVISIONS
PLATE 4

APPENDIX I

VISUAL OBSERVATIONS

APPENDIX I

CHECK LIST
VISUAL INSPECTION
PHASE I

Name of Dam: George F. Brasfield

County: Chesterfield State: Virginia

Coordinates: Lat. 3713.2 Long. 7731.7

Date of Inspection: 18 Jan 78

Weather: Sunny Temperature: 50°F.

Pool Elevation at Time of Inspection: 160' MSL

Tailwater at Time of Inspection: 121' MSL

Norfolk District Inspection Personnel:

Ken Brooker
Gus Malamos
Dave Pezza
Bill Barker
Ed Strawsnyder (Recorder)

1. Concrete Structure

1.1 Seepage or Leakage

1.1.1 Observations: Minor seepage (less than 1 GPM) was noted on the downstream face of the south non-overflow section of the dam. The seepage outcropped along an existing horizontal construction joint and appeared as a wet area on the face of the dam. Additional evidence of minor seepage was noted in the form of calcite precipitation along horizontal and vertical construction joints. This efflorescence of the calcite from the concrete appeared on both the south and north non-overflow sections of the dam. In all cases, this condition was confined to the existing joint system. It was not possible to inspect the overflow section of the dam because of the two feet of water passing over the spillway.

1.1.2 Recommendations: The minor seepage is attributed to the expansion and contraction of the construction joints and is, therefore, not considered a serious threat to the integrity of the dam. It is recommended that an annual inspection program be implemented to monitor the minor seepage and possible joint deterioration.

1.2 Structure to Abutment Contact

1.2.1 Observations: The abutments of the concrete dam are composed of weathered granite and residual clays. A majority of the abutment slopes are grassed. Despite the grass cover, serious erosional gullies have developed on the downstream slope of the right abutment. This erosion has steepened the downstream slopes (1v to 1h) and reduced the fill adjacent to the dam. In addition to the erosion, minor shoreline erosion was noted on the upstream slope of the left abutment.

1.2.2 Recommendations: The erosion on the downstream slope of the right abutment should be corrected immediately to prevent further deterioration of the abutment. In addition to grading and seeding, a paved channel should be constructed to divert surface runoff from the upper slopes to the river. The erosion of the upstream shoreline adjacent to the dam can be arrested by the use of rip-rap and filter cloth.

1.3 Drains

1.3.1 Observations: Flows over the spillway prevented inspection of the foundation drains. A drainage gallery consisting of a 24-inch pipe, relief wells and drain laterals were installed during construction. The drain laterals outcrop on the downstream face of the overflow section above the normal tailwater pool. Observations during the inspection indicated that the drains are probably submerged most of the year because of the almost continuous flow over the spillway.

1.3.2 Recommendations: The outcropping foundation drains should be annually inspected during periods of low flow when the drains are visible and access is possible. Any measurable flows noted during these inspections should be reported with the annual maintenance inspection previously recommended.

1.4 Water Passages

1.4.1 Observations: In addition to the 840 feet of ungated, overflow spillway, there are two 6'x6' gated sluiceways and one 3' diameter cone valve which allow the passage of water through the dam. The 6'x6' sluiceways are controlled by a gate on the upstream side. A bar screen has been placed on the upstream side to prevent debris from clogging the outlet structures; however, there are no workable means of readily removing debris from this screen. The sluiceways were completely submerged at the time of the inspection. The cone valve was closed at the time of our inspection. Representatives from the Water Authority indicated that the cone valve has only been used once. However, because of a mechanical problem with the closure gate and downstream erosion, the valve has been closed indefinitely.

1.4.2 Recommendations: The cone valve should be maintained in an operational state as originally designed. A means of cleaning the intake bar screen should also be established. These later suggestions should be added to the maintenance program for the intake structure.

1.5 Foundation

1.5.1 Observations: The available geology reports indicate that the concrete gravity dam is founded on competent granite. A high tailwater during the inspection prevented actual observations of the foundation. Several downstream rock outcrops verified the bed rock described in the geology report.

1.5.2 Remarks: A review of the foundation construction reports indicated that every effort was made to found the dam on competent rock. The reports also indicated that a grouting program was utilized where competent rock was questioned.

1.6 Surface Cracks

1.6.1 Observations: No significant surface cracks were noted on the visible portion of the upstream and downstream faces of the dam. Minor temperature cracks were noted on the deck or parapet slabs on top of both non-overflow sections. These cracks ranged from hairline to 1/8-inch.

1.6.2 Recommendations: Open surface cracks should be sealed to prevent infiltration of surface runoff.

1.7 Monolith Joints

1.7.1 Observations: The concrete monoliths were placed in 6' lifts. Some spalling was noted on the visible portion of the non-overflow sections. The spalling is more pronounced at the intersection with the vertical construction joints.

1.7.2 Recommendations: The observed spalling is not considered detrimental to the structural integrity of the dam. However, the spalling should be periodically inspected as a part of an annual maintenance program. The downstream face of the overflow sections should be inspected when the flows permit access.

1.8 Construction Joints

1.8.1 Observations: Spalling and efflorescence are prevalent along the construction joints especially at the intersection with the horizontal monolith joints. According to the design documents, a copper waterstop has been installed between the vertical construction joints to prevent leakage during temperature changes.

1.8.2 Remarks: The spalling along the vertical construction joints is not considered critical. It appears that the spall zones are areas in which lean grout was used to patch voids during construction..

2. Outlet Works

2.1 Intake Structure

2.1.1 Observations: The opening to the intake structure is approximately 47'x15' and is protected by a bar screen. Only the upper 4 feet of the bar screen was visible during the inspection. Four gated intakes located at different levels divert influent into the pumping chamber for water supply. The remaining flow is diverted through the two 6'x6' sluiceways which are presently adjusted to maintain a flow of at least 100 cfs.

2.1.2 Recommendations: A means of clearing debris from the bar screen should be established. The gates of the 6' x 6' sluiceways should be periodically operated to insure performance during emergency situations.

2.2 Outlet Structure

2.2.1 Observation: The outlet structure was totally submerged during the inspection. The outlet structure consists of the two gated, 6'x6' sluiceways and a 36" diameter cone valve which was not open during our inspection.

2.3 Outlet Channel

2.3.1 Observations: The outlet channel is approximately 52' in length. The downstream discharge channel is protected by a concrete retaining wall. Riprap and concrete have been added above the retaining wall to protect the upper slopes from erosion when the cone valve is operational. Surface runoff from the adjacent slopes have partially undermined portions of the concrete and riprap blanket causing severe cracking and partial collapses.

2.3.2 Recommendations: The collapse of the protective rip-rap and concrete blanket is not considered detrimental to the operation of the outlet channel. However, if the cone valve is used in the future, the downstream slope should be protected against erosion.

3. Ungated Spillway

3.1 Concrete Weir

3.1.1 Observation: The concrete weir or overflow section of the dam is 840' in length with twelve piers equally spaced along this length. The overflow weir is approximately 10' below the crest of the dam. Except for the 12 piers, the flow is unimpeded.

3.1.2 Recommendations: The overflow weir should be inspected during low flows when the water can be controlled by the intake structure. An automated monitoring system should be added to accurately measure flows in excess of 2' over the weir. A staff gauge is currently used to accomplish this measurement.

3.2 Bridge Piers: Bridge piers were constructed on the overflow weir for an anticipated bridge over the dam. The piers occasionally trap debris but do not severely impede the flow over the weir.

4. Instrumentation: The only instrumentation associated with the dam are the gauges that measure the weir overflow and the stream gauges located 3 miles downstream and 36 miles upstream. These latter gauges are currently operational and recording.

5. Reservoir

5.1 Slopes: The slopes bordering the reservoir are heavily wooded and relatively undeveloped. The terrain can be described as gently rolling.

5.2 Sedimentation: No sedimentation studies have been initiated to date.

6. Downstream Channel

6.1 Conditions: The downstream channel narrows from approximately 840 feet at the base of the dam to approximately 300 feet at a distance of 200 feet from the toe. Miscellaneous fill from the foundation excavation has been randomly placed on the downstream slopes adjacent to the existing channel. The fill does not appear to impede the flow. The overbank areas immediately downstream of the dam are heavily wooded.

6.2 Slopes: The downstream slope of the channel is approximately 0.3 percent to Petersburg.

6.3 Approximate Population: The City of Petersburg with a population of 36,103 is located approximately 5 miles downstream of the dam. The river flows through the center of the Petersburg business district. The nearest source of habitation is a place called Matoca which is approximately 3 miles downstream of the dam. The population in this area is estimated at 200.

APPENDIX II

CHECK LIST - ENGINEERING DATA

APPENDIX II

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION

1. Plan of Dam - A complete set of plans are available in the Norfolk District. A plan view of the dam is inclosed with the Phase I Inspection Report.

2. Regional Vicinity - Map - A U.S.C.G.S. quad sheet was used as a vicinity map and attached as Plate 1.

3. Construction History - The dam was constructed by Wiley N. Jackson, Inc. in 1968 under the inspection of the design firm, Wiley and Wilson, Inc.

4. Typical Sections of Dam - Typical sections of the dam are inclosed with the Phase I Inspection Report.

5. Hydrologic/Hydraulic Data - These data were taken from Wiley and Wilson's design calculations. A summary of these data is attached.

6. Outlets - Plans and details of the outlets are available at the Norfolk District. Constraints and discharge ratings are included with the design calculations furnished by Wiley and Wilson.

7. Rainfall/Reservoir Records - A majority of the rainfall and reservoir records are available at the Appomattox River Water Authority. Additional rainfall data are available from Virginia climatological records.

8. Design Reports - Contract specifications and design calculations were obtained from Wiley and Wilson.

9. Geology Reports - Geology reports were prepared by Dr. Byron H. Cooper, professor at VPI&SU for the design and construction phases of the project. The initial report and subsequent construction inspection reports are inclosed as an appendix with the Phase I Inspection Report.

10. Design Computations - Design computations were furnished by Wiley and Wilson and included hydrology and hydraulic analyses and structural stability analyses. Seepage analyses were made by Dr. Cooper but the results were not readily available.

11. Materials Investigation - Reference is made to the material investigation and testing in the geology report. The subsurface exploration and laboratory testing were performed by Froehling and Robertson, Inc. The results were interpreted by Dr. Cooper.

12. Post-Construction Surveys of Dam - Except for the daily visual observations of the dam by maintenance personnel, no purposeful surveys have been made since completion of the dam.

13. Borrow Sources - The designated borrow sources were located on the left abutment above elevation 165' MSL near the existing water treatment plant.

14. Spillway Plan - Sections and details of the ungated, overflow spillway are inclosed.

15. Operating Equipment - Plans and details of the pumping station equipment are available in the Norfolk District.

16. Monitoring Systems - There are no monitoring systems at the dam except for the gauges used to measure the height of water flowing over the spillway and the raw water intake flow.

17. Modifications - The modifications made prior to and during construction are noted in the specifications and the geology inspection reports.

18. High Pool Records - Records for high pools are available at the Appomattox River Water Authority.

19. Post Construction Engineering Studies and Reports - No post studies have been accomplished.

20. Prior Accidents or Failure of Dam - None.

21. O & M Records - Operation and maintenance records are available for the pumping station only.

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

1. DRAINAGE AREA CHARACTERISTICS: 1,335 Square Miles

2. ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 158.0' MSL (35,400 A-F)

3. ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): Not Applicable

4. ELEVATION MAXIMUM DESIGN POOL: Unknown (170.0' MSL PMF)

5. ELEVATION TOP DAM: 167.5' MSL

6. CREST:

   6.1 Elevation - 158.0
   6.2 Type - Ogee Service Spillway
   6.3 Width - 10 feet
   6.4 Length - 840 feet with 12 piers
   6.5 Location Spillover - Center of dam
   6.6 Number and Type of Gates - Ungated

7. OUTLET WORKS

7.1	Type - Two 6'x6' Sluices	One 36" Coned Valve
7.2	Location - Base of Dam	Center of dam
7.3	Entrance inverts - 114.0	124.0
7.4	Exit inverts - 114.0	124.0 MSL
7.5	Emergency draindown facilities - Sluices & Cone Valve	

8. HYDROMETEOROLOGICAL GAUGES

8.1	Type - Recording Streamgaging Stations	Pool Level (Recording)
8.2	Location - 3 mi. D.S. & 36 mi. U.S.	Spillway
8.3	Records - 1926 to present	

9. MAXIMUM NON-DAMAGING DISCHARGE: Unknown

APPENDIX III

GEOLOGY REPORT AND FOUNDATION INSPECTION REPORTS

BYRON N. COOPER
GEOLOGIST
BOX 13
BLACKSBURG, VIRGINIA 24060

V. P. I. OFFICE
108 HOLDEN HALL
TELEPHONE
AREA CODE 703, 772-6261
EXT. 270

RESIDENCE
COUNTRY CLUB DR.
TELEPHONES
AREA CODE 703, 772-87

REPORT OF INVESTIGATIONS OF FOUNDATION CONDITIONS AT TWO TENTATIVE SITES FOR A DAM ON APPOMATTOX RIVER NEAR PETERSBURG, VIRGINIA

Since March, 1965, subsurface conditions along two tentative center lines for a proposed dam on Appomattox River a few miles upstream from Matoaca, Virginia, have been under investigation. Soil sampling, core drilling in rock, and some auger drilling have been conducted on either side of Appomattox River, and toward the end of field investigations three exploratory trenches were 'dozed out to allow inspection of the weathered granitic residue that lies between a capping zone of red plastic clay and the fresh granite at depths of up to nearly 80 feet below the surface. The two possible sites for the dam are delineated on Wiley and Wilson Drawing designated "Reservoir Basin Map, Sheet 1". Boreholes given simple numbers in a broken series from 1 to 28 explore the downstream or easterly dam site. Boreholes in the "BL-2" series relate to the upstream site as shown on that map.

The area of investigation is situated near the western edge of the Fall Zone, a series of rapids created by resistant ledges of Petersburg Granite which is the bedrock within the area studied. The main rapids are about 3 miles downstream.

The Petersburg Granite is a variegated pink to gray to dark-gray and very coarse-grained granite with a grain that varies from that of an alaskite to coarse pegmatite. The weakest and most deeply weathered rock is generally the coarse pegmatitic zones. The surface of contact between fresh rock and the overlying soil and mantle rock is exceedingly irregular and consists of deeply weathered pockets of granite residue separated by pinnacles of medium-grained to coarse-grained granite. Holes drilled in the vicinity of 1 show the vagaries of soil-bedrock relations.

As shown in Figure 1, the normal soil profile is divisible into several zones. The topsoil which is humus-bearing is sandy and in most places is not over 2 feet thick. Immediately underneath it is a zone of plastic red clay which is breached generally in the bottoms of the short tributary gullies in either valley wall. The red clay layer is 1 to 15 feet thick. Directly under this zone is a body of gray disintegrated granite which has the general texture and consistency of sawdust when disturbed but which in the undisturbed condition is reasonably tight and coherent. The upper contact of this zone is sharp, but the lower one is gradational down into weathered friable granite bedrock--some of which is soft enough to broken by hand in the core samples.

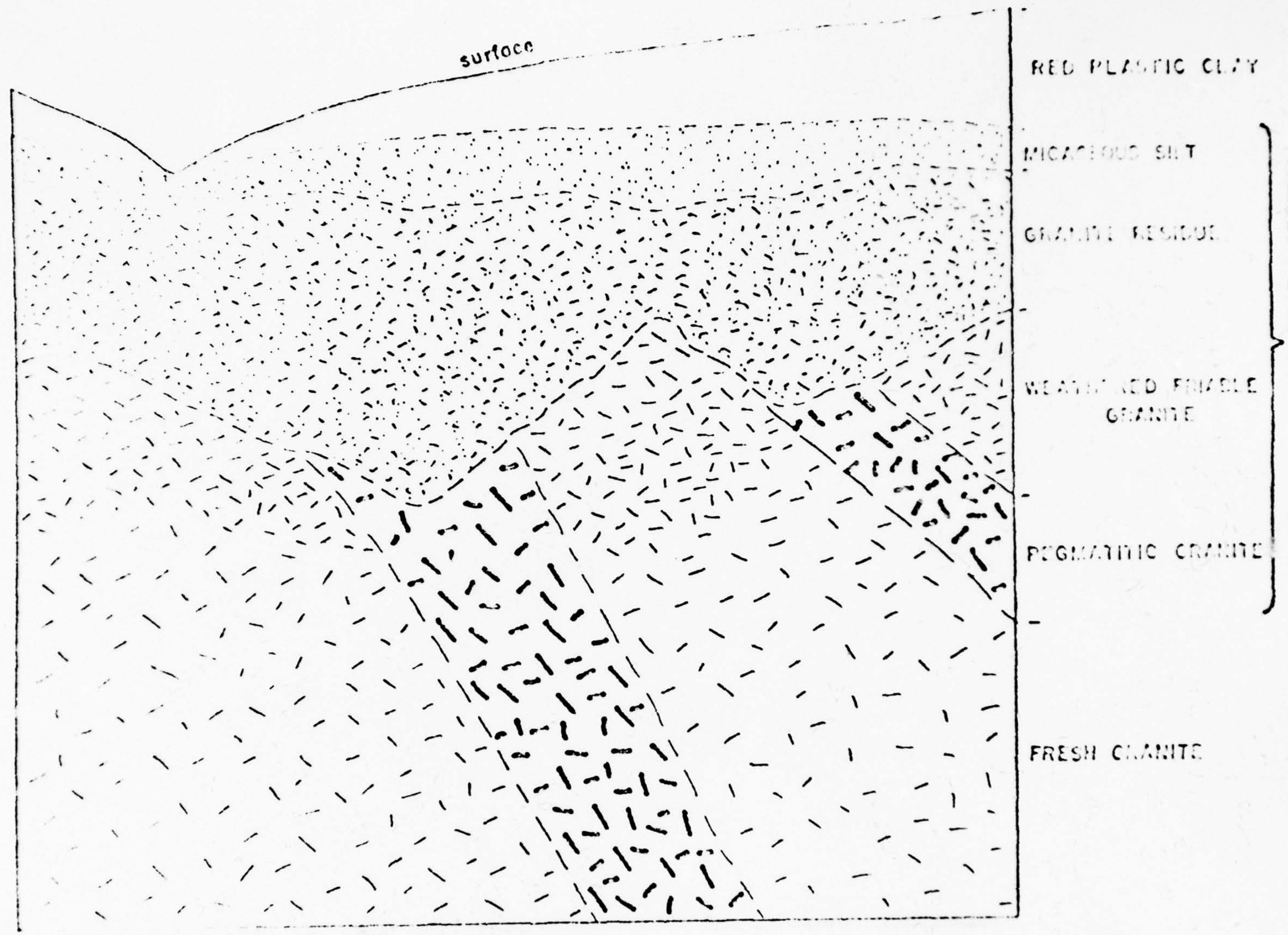


Figure 1.- Diagrammatic profile of weathered zone on the Petersburg Granite in area of BL-2 base line, across Appomattox River.

A short distance downstream from the original center line, the river bends abruptly northward. This change in course brings the river so close and so nearly parallel to the north section of any dam that might be built on that center line as to invite massive seepage under a favorable hydraulic gradient. The effective permeability of the granite residue will depend to a great extent upon the width of the zone through which the seepage would have to take place in order to by-pass the dam. The width of the seepage zone is so narrow between the original center line and the north deflection of the river that it militates against the use of the original center line.

The BL-2 line borings show subsurface conditions like those on the original line investigated, but the distance that seepage water would have to travel to by-pass the north wing of the dam and emerge along the west bank of the river where it flows northward would be hundreds of feet longer, and certainly the hydraulic gradient would be much lower. This extra distance is added protection against seepage. The low coefficient of permeability of the granite pulp means great frictional loss of head the farther any seepage water by-passing the dam would have to travel to a point of egress. Therefore, seepage will be minimized by utilizing the BL-2 line.

Fresh bedrock is close to the surface along the river, but in either valley wall it is generally deeply buried beneath a thick cover of red clay and subjacent disintegrated granite. The permeability of the latter will depend upon the proportion of fines. Minus 200-mesh material makes up nearly 20 per cent of the bulk. These fines are undoubtedly the cause for the low permeability as measured by Froehling and Robertson tests. The fines are readily mobilized and clog the micropores in the granite residue. No effective seepage will take place through the friable oxidized granite zone below the zone of granite pulp. The granite residue is the only material that presents any significant problem. Examination of the soil samples in the granite residue zone first led me to consider the residue to be rather permeable. The low permeability coefficients obtained by Froehling and Robertson in that material seemed incongruous.

This question was resolved by the 'dozing of three deep trenches into the granite residue--two on the south side and one on the north side of the river. The trenches on the south side of the river revealed that the granite residue is quite clayey and plastic, and the coherence of the material is attested by the vertical faces maintained by the residue even after heavy rains. The residue exposed in the trench on the north side of the river was a little less clayey and plastic but still of much more coherent consistency than had been indicated by the soil samples obtained from borings made in the general vicinity of the trench. The soil samples evidently represent not only disturbed material but also material from which the fines have been washed out. They are therefore deceiving. The trenches provided better information on the granite residue.

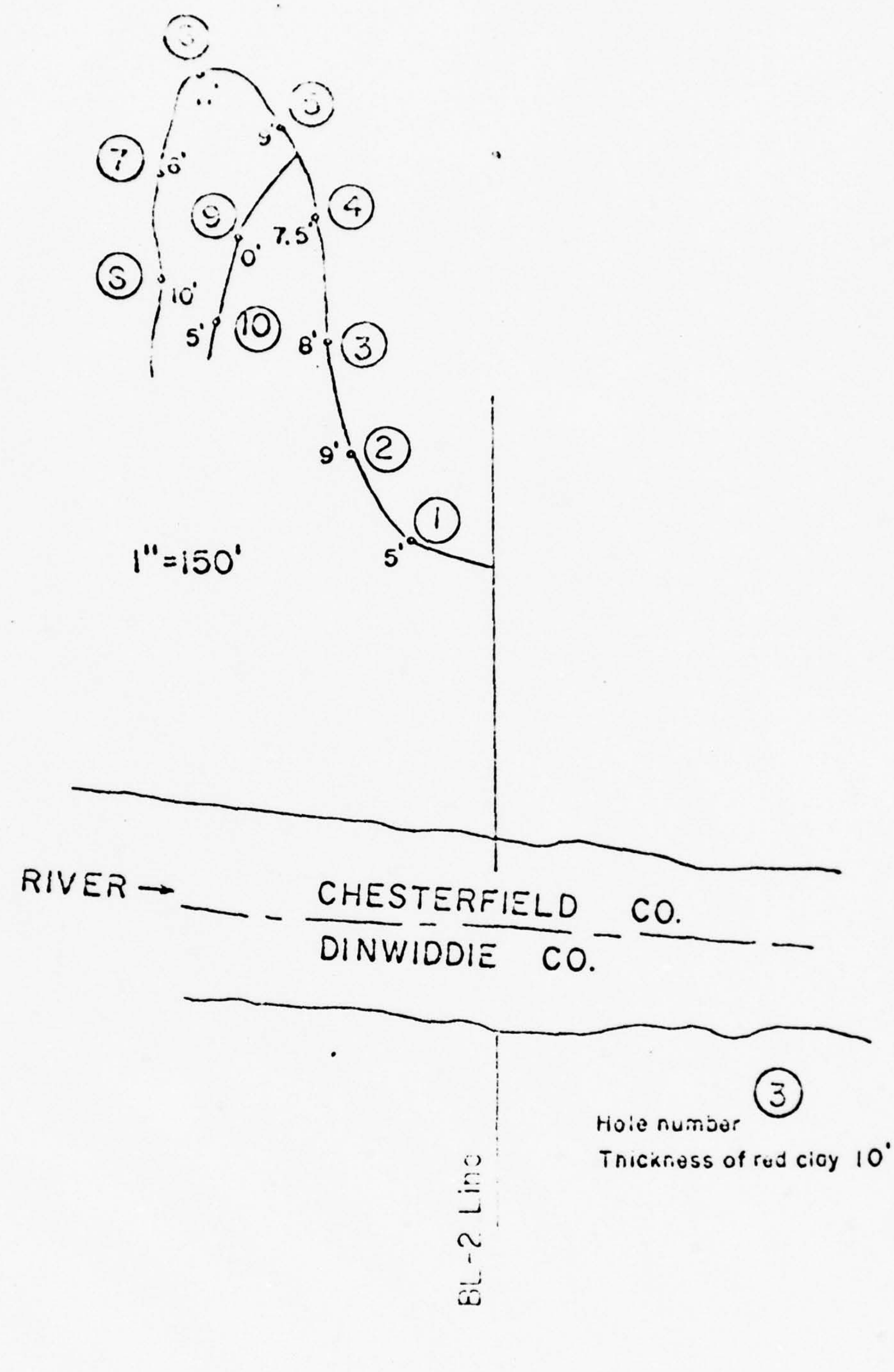


Figure 2.– Sketch of auger holes with thickness of red clay encountered.

The tests on permeability of the granite pulp, conducted by Froehling and Robertson, Inc., show relatively low permeability and the permeability coefficients derived from those tests are considerably lower than one would have estimated by visual inspection of the material. A flow net made to determine the seepage characteristics of the granite residue suggests that significant seepage would affect only a very limited section of the granite residue associated with the dam on the north side of the river, where downstream from the center line a reentrant gully in the north valley wall is deep enough to create locally a possibly favorable gradient for some seepage. This possibility can, so I understand, be mitigated by building up the ground contour in that small ravine. These tests and procedures corroborate my conclusions based upon observations of the granite residue exposed in the walls of the three deep exploratory cuts that were made. The disintegrated granite in place and undisturbed is not very permeable and contains sufficient clay fines so that when it is soaked with water these fines will be mobilized in the slow seepage, with the mass effect of sealing off and reducing the size of openings in the granite pulp through which some little seepage could take place.

Examination of the upper contacts of the granite residue with the persistent zone of red clay shows thin stringers of the latter extending downward for many feet into the disintegrated material. This condition certainly is a retardant to ground-water seepage through the residuum.

The gradational nature of the contact between the soft granite residue and the firmer, but weathered and friable, bedrock below does introduce one unfortunate condition. No arbitrary elevation determining depth of excavation in the granite weathered zones, which will reach relatively strong material, can be predetermined from the exploratory work that has been done. The depth to bottom of the foundation excavations will be discernible only when excavations are made. Therefore, the remaining uncertainty as to how much yardage of excavation that will have to be done to yield satisfactory foundations will mean a considerable contingency factor in construction estimations that have to be made in advance.

Examination of the three open trenches which were prepared for study of the granite residue disclosed the overlying shroud of red plastic clay or saprolite. This material constitutes a strong deterrent to soil percolation by impounded water. If the red clay were completely continuous over the hilly terrain, the clay would constitute an ideal sealant membrane opposing penetration of the impounded waters. Examination of the ground surface indicates that the only place where incurrent seepage of impounded water could be expected is in the general area of the gully lying immediately west of the BL-2 dam site. It was suggested that auger drilling of the soils around the gully should disclose the nature and extent of the red-clay layer or zone which is so impermeable. Figure 2 is a copy of Mr. Smith's plot of the auger borings as referenced to the BL-2 base line. Only Bore Hole 9 of this series of auger holes showed no red clay. This exploration signifies that in the area where any seepage of impounded water is likely to occur a protective zone of impermeable saprolite occurs close to the surface, except for a narrow zone probably averaging less than 25 yards wide down the middle trench-like section of the ravine. That portion can be rendered impermeable also by filling in the bottom portion of this ravine with red clay obtained from borrow area situated above the elevation of maximum reservoir impoundment. With this minor remedial treatment, the gully area as a source of possible incurrent seepage of impounded water will have been eliminated.

The wing sections of the proposed dam will, of course, be founded in the partially disintegrated granite. The presence of a zone of red plastic clay near the surface of the ground raises the possibility for use of this material in construction of the wings of the dam. The availability of the corewall material adjacent to the dam site is worthy of consideration in determining possible design of those sections. The entire center section which will extend across the river and the flats on either side would constitute an adeuqate spillway providing assurance that the higher wing sections would never be topped even by a flood.

In summary, the geologic conditions at the Appomattox River dam sites have proved to be different from what was inferred from the abundance of granite exposures in the general area. The thick zone of disintegrated granite and friable bedrock are much thicker than expected. I would have diagnosed these subsurface materials as permeable if I had had only the soil samples to make an evaluation of them. However, Froehling and Robertson, Inc., has established in a typical location pertinent to the upper dam site that the granite pulp has a relatively low permeability. Choice of the upper or BL-2 center line reduces the possibility for significant seepage under any condition of permeability. Presence of a protective cover of impermeable plastic clay presents a great deterrent to ready infiltration of impounded water. This impermeable cover can be made even more effective by emplacement of identical material from borrow areas so as to make the clay cover entirely continuous.

Therefore, I have been able to reach the conclusion that the BL-2 location for the proposed dam and that its erection and subsequent impoundment will not incur serious seepage.

If there are questions which you would like for me to try to answer to provide information that may not have been covered in the above summary report, please forward them to me for reply.

May 19, 1965

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Byron N. Cooper
P.O. Box 13
Blacksburg, Virginia 24060

REPORT OF INSPECTION TRIP TO OBSERVE FOUNDATION CONDITIONS OF NORTH HALF OF DAM ON THE APPOMATTOX RIVER
Friday, July 8
1966

The portion of the excavations that had been made by the time of the writer's visit represents about 300 feet of the total length of planned excavations. The river had been diverted by a coffer dam into the old canal thus freeing the old main channel for examination, as well as the north wing, spillway, and part of the massive center section's foundation. This report summarizes my findings.

The uncovered ledges of granite under the old river bed are characterized by closed joints and seemingly little disintegration along the joints. From all surface indications, fresh and tight rock will be found at the projected depth of excavation.

In the east wall of excavation for the main section of the dam, bedrock conditions are somewhat varied as a result of rather close jointing and thorough disintegration of the granitic rocks along the margins of the joints. Fortunately, the major set of shear joints, which is well defined by slickensides along the northwest wall of the excavation, trends virtually parallel to the axis of the dam. This set is the more open set of joints. The excavations to designed bottom in the center section did not disclose rock sufficiently tight for good foundations, and an additional lift of 5 feet thickness must be taken out. Open jointing prevails in the walls of the main cut all the way down to the originally designed bottom of excavations, but exploratory drilling is reported to be in fresh tight rock between 3 and 5 feet below that level. There is no alternative to removing the additional 5 feet of rock referred to.

The granite in the north end of the main section is less closely jointed and the top of fresh tight granite rises so that the bottom as originally planned will provide satisfactory foundation conditions, although the rock surface will be rather irregular.

The foundation conditions in the north wing section show a weathering profile identical to that visualized from borings and trenching. The soft mealy weathered granite which was found by empirical tests to have low permeability was examined all the way down to fresh rock. The kaolinization of the feldspars in the weathered granite has closed the pores sufficiently to make the rock essentially impermeable. When the rather irregular excavation for the east wing has had its concrete floor and core poured, as planned, Class A clay will be packed into the excavation. I would suggest that the lower portion of the clay fill be introduced with a relatively high moisture content which will be lost by lateral wicking out into the now somewhat dried soft granite.

I believe that a good bond can be obtained between the irregular excavation and the placed Class A clay. One of my earlier misgivings about the dam was the north end or wing section, but conditions are better than I expected them to be.

Out in the main section of the excavation, there is one more deeply weathered section of granite which is bordered on the southwest side by a dike of basic igneous rock. This rock is characterized by close jointing such as characteristic of diabase dikes; however, the rock is not a diabase and will have to have further study for precise identification. The close jointing in this dike, which extends diagonally northwestward across the main part of the excavation. The dike dips northeastward at about 65° and being only about 3 feet thick will have to be grouted by drilling the grout holes in the middle of and at the same inclination as the dike.

The joints can expectably be figured to be open enough to take thin grout to a depth of 50 feet below finished bottom excavations. Casing for the grout holes will be carried up through the dam so that grouting can be done with better success.

Since the dike extends in a direction that carries its trace rapidly toward the reservoir boundary, I do not believe that any great amount of grouting will need to be done in this dike on the upstream side of the excavation. However at least 2 and possibly 3 groute holes should be put down along the same orientation so as to stay in the narrow dike to a depth of 80 feet. These grout holes should be spaced as follows: (1) 20 feet from the excavation line on the west side and midway in the dike; (2) the second one 10 feet northwest of #1; and (3) possibly a third hole 10 feet along trend of the dike from #2. The location of these grout holes and their angle of inclination is of critical importance, and I would like to lay these holes out precisely after the bottom section has been cleaned off thoroughly so that the precise trend of the dike is unmistakably defined. The grout holes that should be drilled northwest of the excavation on the upstream side of the dam will be hard to position precisely, but this can be done best right after the bottom lift has been all taken out and cleaned but before any holes have been drilled.

The two or three holes recommended northwest of the dam can be carefully staked and then drilled after concrete has been poured for at least the lower two-thirds of the main section.

In general, I think the foundation conditions are somewhat better than I thought that they would be. The cross-cutting dike is an unsuspected and hitherto undetected nuisance that will have to be dealt with rather carefully. Spacing of the grout holes in the bottom can be argued about; personally, I regard the rock to be treated to require spacing of the holes at 10-foot intervals. It would be very helpful if the initial grout hole drilled was diamond-drilled to yield a core of the rock, which would allow better understanding of the open character of the joints and depth of weathering of the walls of the joints. Would this be possible?

7-9-66

Byron N. Cooper

Byron N. Cooper
P.O. Box 13
Blacksburg, Virginia 24060

REPORT OF FINDINGS FROM SECOND VISIT TO APPOMATTOX RIVER DAM SITE ON JULY 27, 1966

This memorandum reports on the observations of bottom conditions after removal of an additional 5 to 6 feet of rock from the bottom of the excavation for the north side of Appomattox River Dam.

As discussed in the previous report, the bedrock in a considerable sector of the north area of excavation did not prove to be tight at the planned base of excavations, and after examination of the conditions, it was recommended that an additional 5 to 6 feet of rock be removed in an effort to find better conditions for founding the base of the dam.

On July 27, the writer reinspected the foundation conditions after removal of the recommended material. In the area that was considered of critical importance, namely the section exposing the cross-cutting dike of black, closely jointed igneous rock, and also the granite rock immediately southwest of the dike, the foundation conditions were examined in detail. The granitic rock next to the dike is free of open jointing. Two carefully washed sections all the way across the bottom section, which can be considered typical for the base of this portion of the dam, were examined virtually inch by inch, all joints probed, and the rock found essentially water-tight. The bottom surface is somewhat irregular, but the rock is fresh and hard.

With careful cleaning in advance of pouring the initial basal layer of the concrete section, absolutely fresh rock should insure a very satisfactory bonding surface. The main set of joints which almost parallel the axis of the dam strike N. 65° E. and dip 70° SE. This set at the new foundation level is essentially closed. The weakest set of joints, which are vertical and which strike N. 20° W. were not seen on the bottom of the new, lower level of excavation. However, a series of gently dipping and convergent sheet fractures showed up in the west wall when the additional 5 to 6 feet of rock was removed. Some open clefts were probed and found to be unusually coarse pegmatite rock composed of crystals of quartz and feldspar up to 1.25 inches in diameter. Such rock cleaved along grain boundaries and the nut-size crystals of quartz and feldspar fell of of place. These clefts were probed and found to be solid and tight back but a few inches from the west face. These joints appear to die out downward and are not noticeable on the floor of the lower excavation line.

Some overbreak is being experienced in the cut-off trench being excavated in rock on the upstream side of the foundation. This cannot be avoided. The rock is being shot very lightly but tends to overbreak in irregular patterns. Care should be taken to excavate all of the rock that is not absolutely tight on the bottom.

Foundation conditions in the bottom section at the new, lower level appear to be quite satisfactory, and I do not believe that better conditions could be found any deeper.

The cross-cutting dike was found much better defined when examined on July 27. The dike strikes N. 15-20° W. and dips 65° NE. Good exposures disclose intimate fracturing on even closer spacing and more irregularity of orientation of fractures than first believed. Weathering of this rock has developed selvages of sticky, putty-like clay which will doubtless seal contacts with the bordering granite. The dike which averages 30 inches in thickness possesses so many fractures that it will have to be grouted with thin grout under a considerable pressure to insure sealing of the many thin openings in the fresh part of the dike.

After I had returned to Blacksburg, additional excavation on the bottom indicated that the dike "disappeared" down dip at about 3 feet below the deepened bottom. The condition as described over the telephone by Mr. Dodl strongly suggests that the dike is offset in some fashion either by a fault that essentially parallels the sheet fractures described above, or else it pinches down to perhaps a few inches wide and swells again at greater depth.

Considering the nature of the dike rock and its cross-cutting relations, it will be advisable to do some exploring to disclose what actually happens at depth to the dike. In this connection, it should be mentioned that a small but well-defined irregular lenticle of dike rock occurs at the intersection of two joints in granite on the downstream wall of the excavation for the dam. The effect of the dike in that sector has been to close the joints and anneal them with the dike material. Such squeeze-out extensions are not at all uncommon and may not interfere in any way with tightness of the rock.

Suffice it to say, however, that the dike rock surely must have come up from below and must certainly have downward extensions below the depth where it is supposed to play out. The nature of the offset and locus of downward continuation of the dike-rock body must be ascertained.

Based upon discussions with Mr. Major and Mr. Dodl before leaving the site on July 27, it was agreed that I would return and inspect the bottom conditions shortly before August 10.

I am awaiting further word from Mr. Dodl regarding the time of my next visit.

August 1, 1966

Byron N. Cooper

Byron N. Cooper
P.O. Box 13
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REPORT OF OBSERVATIONS AT APPOMATTOX RIVER DAM SITE
August 26, 1966

On August 26, 1966, I examined two sections approximately 50 feet long, which had been cleaned preparatory to pouring the initial pour of concrete on the rock foundation of the Appomattox River Dam. Half of an additional section south of these two sections was also examined.

Conditions in the cut-off trench and also over the floor of the two cleaned sections and the adjacent half section show excellent founding conditions in the bedrock. The northerly-trending cross joints although readily identifiable are closed and individual breaks discontinuous and somewhat offset en echelon. Prominent twin, parallel pegmatite veins exposed in the most northerly section examined anneal one set of northerly joints but are cut by another more northeasterly-trending set which is not particularly well developed.

With the exception of a small portion in the corner of the next section to the south, which was examined, there was no observed weathering oxidation or weakening of the bedrock, all being fresh. One small section in the southeast corner of the second section examined did show some slight kaolinization of the feldspars, but the material was essentially rock hard and without bordering open fractured.

In both sections the bottoms were sufficiently rough to provide an excellent lock with the concrete. The rock surfaces show a variety of rock types including cross-cutting pegmatites and either roof pendants or xenoliths of dark-colored country rock, but contacts between different kinds of rock are tight.

Farther south in sections being prepared for cleaning, the founding surface is essentially on the original grade elevation. The greater depth of weathering in the sections previously covered with concrete, which necessitated excavating to greater depth to obtain required bedrock foundation conditions, seems to have been caused by the cross-cutting dike of basic igneous rock, which was adequately tested and found to be tight below the lowered base of the dam.

Conditions observed during the inspection of August 26 were highly satisfactory. A continuing series of photographs documenting geologic details in the various sections examined is being kept for the record.

August 29, 1966

Byron N. Cooper
BNC

Byron N. Cooper
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SEP

REPORT ON EXAMINATIONS OF PORTIONS OF THE ROCK FOUNDATION
OF THE APPOMATTOX RIVER DAM

On Wednesday, September 21, and again on Saturday, September 24, 1966, the dam site was visited for the purpose of inspecting cleaned sections of the rock foundation which had been readied for pouring concrete. The inspection undertaken on Wednesday, September 21, was futile. During the night of September 20, and exceedingly heavy rain that fell between 1:30 and 5:30 A.M. led to flooding of the portions to be inspected and cover of the bedrock with a layer of sand and silt. Portions of the higher parts of the foundation were exposed over a part of the area, but the water covered better than half of the critical portions that had been previously cleaned preparatory to pouring concrete. Consequently, the inspection had to be rescheduled and was conducted on Saturday, September 24.

At the time of the Saturday visit, approximately 160 feet of length of the foundation was examined in detail (sections 15, 16, 17, and 18). In Section 15 a very curious weathered and oxidized zone of highly fractured granite (originally red but now altered to buff-gray except for centers of roundish, joint-bound masses) which has been partially kaolinized trends across the foundation and actually cuts across the southwestern corner of previously poured section 14. The border of this fractured and broken zone, which is on the side close to section 14, is composed of thin wavy quartz veins and hydrothermally altered rock that spalls readily on exposure. The softness of the broken zone is attested by the deeper excavation that was found necessary to get down to firm rock. As shown in the downstream face of the northeast corner of section 15, this curious zone thickens downward from a surface width of only a few feet. In all probability, this weathered zone with its criss-crossing joints extends for at least 50 feet below the bottom of the present rock exposure and thus is too much to deal with by further excavation. The excavation, as prepared, exposes rock with sufficient strength to support the dam, and the criss-crossing fractures are sufficiently closed so that possibility of piping water under the dam is slight. Nevertheless, it was deemed important to take precautionary measures to insure plugging of the joints and fractures by the drilling of three grout holes in the southwest corner of Section 14. These holes will not be drilled or pressure grouted until after the initial pour of concrete for section 15 has been poured and hardened. The three grout holes carried down to a depth of 50 feet below the base of the first pour in Section 14 should be ample to pressure grout the width of the fractured, weathered zone near the upstream face of the dam so that the foundation is tight. One grout hole is 2.5 feet northeast of the lock rentrant in Section 14 closest to the upstream face; another hole was laid out 10 feet to the northeast; and the third grout hole is 10 feet from the first one and close to the second reentrant lock from the upstream side of Section 14.

Also in Section 15, but southwest of the fracture zone, there is an open cross-cutting joint that is bleeding a little water from the downstream rock face. This joint was carefully traced to see if it persisted across the entire foundation as a discrete crack. The joint closes at a point about two-thirds of the way from the downstream to the upstream face and is closed in the cutoff trench. Several of these cross joints belonging to the system that trends N. 10° W. were traced n and carefully examined to see if they were persistent and could pipe appreciable water. None were found, and all faded out in the hard, monolithic central portion of the rock foundation for Sections 16, 17, and 18. Those sections have such tight rock with ideally rough surface that they will provide an excellent foundation and bonding surface for the initial pour of concrete upon them.

Beginning in Section 17, the reddish to pinkish granite rocks which characterized portions of the rock foundation cleared and poured earlier in construction work undergoes a gradual change to gray granite which will prevail for quite some distance across the bed of the river. This change of color is of no particular significance in itself, but the gray granite is of more uniform grain and contains fewer joints, fewer cross-cutting pegmatite veins, less saussuritization of the feldspars, and is on the whole tighter rock and sounder than the reddish phases of the Petersburg Granite.

The width of ideal foundation rock exposed in Sections 16, 17, and 18 will insure water-tightness. With the treatment of the 10-foot wide zone of fractured oxidized granite first described in this report, Sections 14, 15, 16, 17, and 18 can be passed as acceptable for pouring concrete.

Two sets of joints, as before recognized are identifiable: one trending N. 57° E. and dipping about 48° SE; the other trending N. 10° W. and dipping 60° to the east. A third but weaker and less well defined set of joints trending almost east-west shows in Sections 17 and 18. All these joints individually come and go and none are open for any great distance or far enough to constitute appreciable possibility for subsurface piping of water.

September 27, 1966

Geologist

January 27, 1967

EXAMINATION OF SECTIONS 25 and 24 OF THE FOUNDATION
OF APPOMATTOX RIVER DAM

On January 26, 1967, Sections 24 and 25 were examined after Sections 22 and 23 had been poured.

In Section 24, the predominant gray granite is criss-crossed and interlaced by thin pegmatite dikes ordered along the N. 5° E. joints and the N. 35° W joints. The foundation is excellent as prepared, except for a narrow weak weathered zone emerging from under the southwest border of Section 23 at the recessed lock inset closest to the downstream border of the foundation. This narrow fracture zone grades into a series of very narrow by? discrete fractures lined by, but not completely closed by drusy quartz growths. The fractures containing the quartz selvages are not continuous individually and where the rock is fresh these minor openings will not pipe water after the conrete is poured. The narrow zone cleaned out near the border of Section 23 was reexamined after the weathered rock had been picked out. Along the southwest side of this section, there is a zone of inclusions of greenish-gray schist enclosed by coarse pegmatite. This zone trends N. 10° W. and beginning about 6 feet from the upstream wall, where there is no rock at the form line, there is a weak zone of weathered rock about 1 foot wide. Because of lack of any cut-off trench along the upstream side of this section, the possibility of seepage through the foot-wide weathered zone six feet long was removed by cutting down the weak section to a depth of about 1 additional foot. Good fresh rock all the way around the excavated slice was proven by jack-hammering until fresh dry rock powder indicated that hard rock had been reached.

In Section 24 there are more sets of joints than noted in any previous section. One set trends north-south, another east west. The N. 70° E. set instead of dipping 45° or less as usual are practically vertical. The best developed set of joints is the north-south set, and the usual predominating influence of the N. 5°E., N. 35° W., and N. 70° E. joint sets is lacking. The effect of all these five joint systems is to produce a toothed surface that is ideal for bonding. This section was approved after the two minor areas referred to had been cleaned out and down to fresh rock.

Section 25 is composed mainly of gray granite with only very minor pegmatite bodies. The N. 70° E. set of joints, subparallel to the axis of the dam are better developed in this section. There are five such joints running all the way across this section, one of which will be poured against on the upstream side. None of the joints are open, and the rock surface is sufficiently rough to be passed practically as is. Narrow quartz veins close up many of the joints. One minor weak zone only about a foot wide cuts across the northeast corner of the section and emerged from under the most southerly recess lock joint form on the southwestern edge of Section 24.

Once this minor weak zone is picked out to fresh rock, the entire section will be in excellent shape for concrete. The haulage road crosses a portion of the downstream face of this section, so that not all of it is exposed. However, the bedrock is fresh under a thin veneer of broken rock, and when the loose material is cleaned off, the bedrock under the old roadway will be quite satisfactory for foundations.

Sections 24 and 25 were approved for pouring before leaving the job.

B. N. Cooper

REPORT OF EXAMINATION OF SECTIONS 22 and 23
OF THE FOUNDATION OF APPOMATTOX RIVER DAM

January 12, 1967

In Section 22 there is a weathered, oxidized zone 15 feet from the border of Section 21, which was observed to be issuing seepage water. The solid, sloping projection of the upstream rock next to Section 21 should be left in place by the north wall in the area of seepage needs to be cut downsomewhat, possibly restoring the trench along the upstream side of the foundation. Out in Section 22 and continuing in line with the strike of the seepage area in the northwest wall is a weak zone following the N. 5° E strike joint system. Some cleaning out of weathered rock along this seepage zone will be necessary to obtain rock to which concrete will bond.

In Section 22 near the southeast corner, there is a zone of highly fractured, oxidized granite that is still not solid enough to be approved. This oxidized area needs to be cleaned out perhaps down a foot or so to get below the weathered, kaolinized top rock. The more of this weathered oxidized rock that is removed, the better the bond. Most of the rock in Section 22 is good and firm and fresh, and the rock surface though on the whole somewhat lower shows the three intersecting sets of joints, as in the three previous sections. The central part of the section contains many thin pegmatite bodies, and the very coarse feldspar is somewhat shattered, but the rock is fresh and solid.

Section 23 shows two prominent oxidized zones in the north wall, one of which opens out into an area about 10' northeast-southwest by 22 feet upstream-downstream which is weathered into a soft friable and crumbly pulp. This somewhat diamond-shaped area in Section 23 needs to be lowered sufficiently so that the major joints bounding the weathered zone at a depth of 2 feet or so will be reached. The amount of material that will need to be removed and the uncertainties of what will be found suggest that this section should be observed before the concrete is poured. This zone plays out into fresh but highly fractured red granite that predominates in the downstream one-third of Section 23. The pegmatites help to seal up the N. 35° W. and N. 5° E. joint systems. The N. 70° E. set of joints in section 23 are not well developed.

It is anticipated that the cleanup work in Sections 23 and 24 will be completed in time for reexamination prior to pouring by January 17.

B. N. Cooper

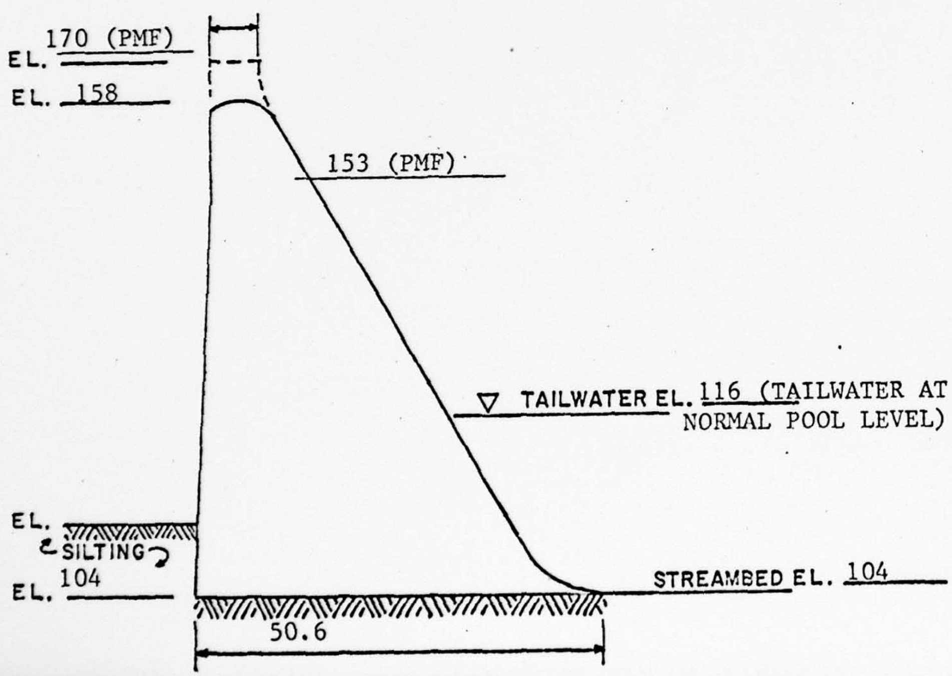
APPENDIX IV

RESULTS OF STABILITY ANALYSES

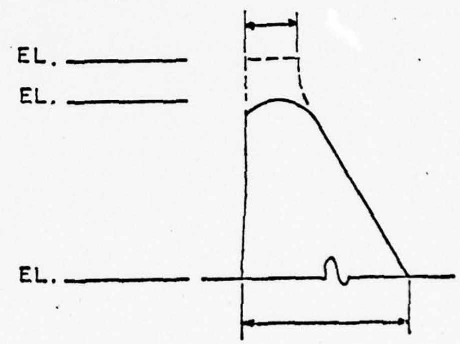
GRAVITY DAM DESIGN
STABILITY ANALYSIS

ANALYSIS DONE ON x FULL SECTION ___ PARTIAL SECTION
LOCATION OF SECTION OVERFLOW
ANALYSIS PREPARED BY C.N. MALAMOS

LOADING CASE	ELEV. HEAD WATER	ELEV. TAIL WATER	ΣV	ΣH	ΣH/ΣV	LOCATION RESULTANT FROM TOE	% BASE IN COMPRESSION	FACTOR SAFETY SLIDING	FOUNDATION PRESSURE	
									TOE	HEEL
P.M.F.	170	153	119,565	56,594	0.47	26.27	100%	9.1	4,717	6,783
*	170	116	194,344	127,125	0.68	22.08	100%	4.5	5,302	2,382



FULL SECTION



PARTIAL SECTION

*Hypothetical extremes -

Maximum headwater (PMF) and minimum tailwater (116)

APPENDIX V

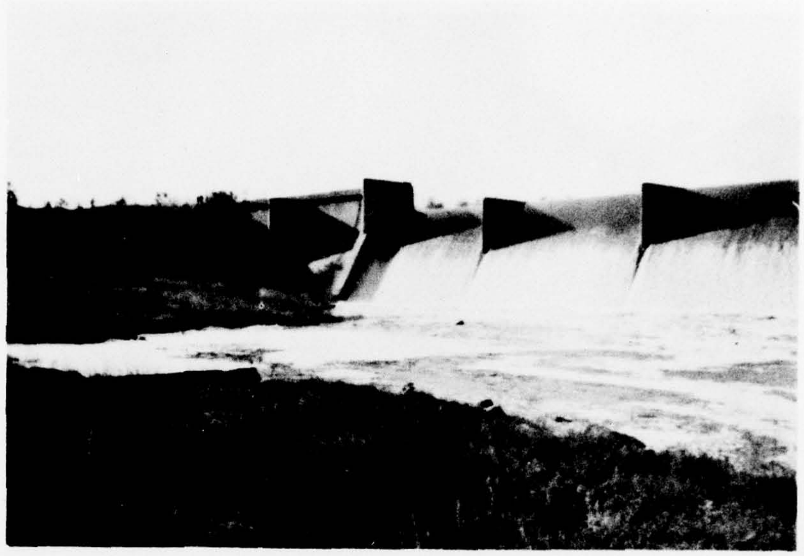
PHOTOGRAPHS

GEORGE F. BRASFIELD - VIEW OF BANK EROSION ON D/S SLOPE OF RIGHT ABUTMENT

GEORGE F. BRASFIELD - VIEW OF BANK EROSION ON U/S SLOPE OF LEFT ABUTMENT

GEORGE F. BRASFIELD - VIEW OF SPALLING ON D/S FACE
ALONG JOINTS

GEORGE F. BRASFIELD - VIEW OF DOWNSTREAM CHANNEL



GEORGE F. BRASFIELD - VIEW OF RIGHT ABUTMENT

GEORGE F. BRASFIELD - VIEW OF LEFT ABUTMENT AND SPILLWAY OUTLET CHANNEL UNDER PUMPING STATION