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NATIONAL DAM SAFETY PROGRAM. OCCOQUAN RESERVOIR SYSTEM DAM NUMB--ETC(U)
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POTOMAC RIVER BASIN

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Name Of Dam: OCCOQUAN RESERVOIR SYSTEM

Location: FAIRFAX AND PRINCE WILLIAM COUNTIES

Inventory Number: OCCOQUAN "UPPER" MAIN DAM, VA 15304
OCCOQUAN "LOWER" STORAGE DAM, VA 15305

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

AD A063580

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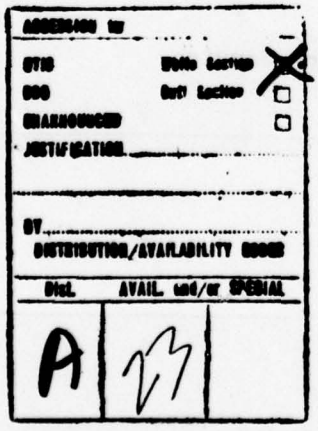
20. Abstract

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.

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PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of System: Occoquan Reservoir System
Name of Dams: Occoquan "Upper" Main Dam, # VA 15304
Occoquan "Lower" Pumping Dam, # VA 15305
Counties: Fairfax and Prince William
USGS Quadrangle Sheet: Occoquan
River: Occoquan

The Occoquan Reservoir System consists of an "upper" main dam and a "lower" water storage dam 2900 feet downstream. Both structures are concrete gravity dams with "ogee" spillway sections. The upper dam is 740 feet long with a 65-foot high, 523-foot long overflow section. The lower dam is 436 feet long with a 22-foot high, 387-foot long overflow section. The system is located on the Occoquan River just west of the Town of Occoquan. It is owned by the Fairfax County Water Authority (FCWA) and is the principle supply of water for portions of northern Virginia.

The spillway capacity of the upper dam is inadequate, but not considered seriously inadequate. The dam has been modified to pass overtopping flows of about two-thirds the Probable Maximum Flood (PMF). A stability check indicates that the overflow section is within Corps criteria for flows up to and including the PMF. However, further information is needed to perform a complete stability check on other portions of the structure. The spillway capacity of the lower dam, like the upper dam is inadequate, but not seriously inadequate. The dam has been modified to pass overtopping flows up to one-half the PMF and a failure during any flow would be inconsequential. Stability checks were not performed because available information was inadequate. There is no immediate need for remedial measures. However, recommendations presented in Section 7 should be completed within 6 months.

APPROVED:

Douglas L. Haller

DOUGLAS L. HALLER
Colonel, Corps of Engineers
District Engineer

Date: 8 Aug 78



UPPER DAM



LOWER DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
OCCOQUAN RESERVOIR SYSTEM

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, based on "Recommended Guidelines for Safety Inspections of Dams," National Program of Inspection of Dams (Vol. I, App. D), Department of the Army, Office of the Chief of Engineers, is to identify expeditiously those dams which might be a potential hazard to human life or property.

1.2 Project Description.

1.2.1 Description of System: The Occoquan Reservoir System consists of two dams, a "lower" water storage dam and an "upper" main dam. (Appendix A - Plate 3) The upper dam is a concrete gravity structure with an overflow "ogee" section (120 MSL), a non overflow section (130 MSL), and an intake structure (130 MSL) between the two sections. The overflow section ties into the left abutment and has a downstream concrete retaining wall to deflect high flows away from the abutment (Appendix B - Plate 1). The non overflow section ties into the right abutment (Appendix B - Plate 2). The total length of the dam is 740 feet. The overflow spillway section is 523 feet long and 65 feet high. The intake structure also serves pipelines leading to the water treatment plant. Three 6-foot diameter raw water intakes with invert elevations 107, 92 and 77 service one 6-foot diameter outlet pipe running downstream, from invert elevation 77 to a pumping station. Two turbines can discharge up to 290 CFS through a tail race with bottom elevation 54. One 36-inch diameter and two 24-inch diameter pipes also discharge into the tail race at invert elevations 61 and 60, respectively. With the reservoir pool at the spillway crest, elevation 120, the total discharge capacity of the three pipes is about 400 CFS. A small powerhouse with two 625 KVA generating units adjoins the intake structure on the downstream side. Power is generated for in-house use during periods of sufficient head, mostly from November to May. The reservoir impounded by the upper dam has a storage capacity of 30,300 acre feet and a water surface area of 1,700 acres at a pool elevation of 120 feet MSL. The main stem of the reservoir extends about 16 miles upstream along the Occoquan River to Lake Jackson (Appendix A - Plate 2). The upper dam does not have a gated spillway or diversion tunnel.

The lower dam is a concrete gravity structure with abutments tying into rock. The total length of the dam is 436 feet and consists of an overflow "ogee" spillway section and a raw water intake structure (Appendix B - Plate 1). The spillway section is 387 feet long and has a height of 22 feet at 52 feet above MSL. The raw water intake structure (top elevation 62 MSL) serves pipelines leading to a downstream water treatment plant. It has one 2 x 4-foot and two 4 x 4-foot raw water intakes, at invert elevation 37, which service one 6-foot diameter outlet pipe running downstream to a pumping station (Appendix A - Plate 10). A sluice gate in the intake structure can bypass flow to the channel through one 3 x 3-foot blowoff at invert elevation 31.5. An old 16-inch diameter cast iron pipe, located on the stream bed through the middle of the dam, is considered inactive by the FCWA. The reservoir impounded by the lower dam has a storage capacity of 170 acre feet at the spillway crest. The lower dam like the upper dam, has no gated spillway nor a diversion tunnel.

1.2.2 Location: The Occoquan Reservoir System is located on the Occoquan River, 1 mile upstream of Occoquan, Virginia (Appendix A - Plate 1). The dams span across the boundary between Fairfax and Prince William Counties.

1.2.3. Size Classification: The upper dam is classified a large size structure based on its storage capacity. The lower dam is classified as a small size structure based on its height and storage capacity.

1.2.4 Hazard Classification: The dams are located above an urban area and are, therefore, were originally given a high hazard classification in accordance with guidelines contained in Section 2.1.2 of the Recommended Guidelines for Safety Inspection of Dams. The hazard classification used to categorize dams is a function of location only and has nothing to do with its stability or probability of failure. However, since failure of the lower dam would not cause loss of life or damage to property it is given a low hazard classification.

1.2.5 Ownership: The system is owned and operated by the Fairfax County Water Authority (FCWA).

1.2.6 Purpose of System: The primary purpose of the reservoir system is water supply. The impounded supply provides a safe yield of 65 MGD on a 20-year recurrence freguency for Fairfax County, the City of Alexandria, and Prince William County, Virginia. Recreation is a secondary benefit of the upper reservoir.

1.2.7 Design and Construction History: Both dams were designed by the American Water Works Service Company and constructed by a subsidiary, The Alexandria Water Company. The lower dam was built in 1950 and the upper dam in 1955. Ownership of the system was transferred to the Fairfax County Water Authority in 1967. In June 1972, a major flood spawned by Tropical Storm Agnes overtopped the dams damaging their abutments and foundation. The area was subsequently declared a disaster area. The Baltimore District, Corps of Engineers, acting for the Office of Emergency Preparedness, took over the repairs to the dams. The Corps in turn engaged Harza Engineering Company of Chicago, Illinois to inspect the damage and to engineer the necessary repairs. All required work was completed by 1976. A copy of the report and a supplemental memo pertaining to the inspection of the dams are provided in Appendix E, Reports 1 and 2. Report 1, excluding exhibits, also includes a study by Harza Engineering Company to raise the height of the upper dam by 5 feet.

1.2.8 Normal Operational Procedures. The Occoquan Reservoir System has no method of flood control and, therefore, no normal operating procedure except for water supply and in-house electric power generation.

1.3 Pertinent Data.

1.3.1 Drainage Area. 595 Square Miles

1.3.2 Discharge at Dam Site.

Maximum known flood at dam site - 75,000 CFS (June 1972)
Ungated Spillway, capacity:
Upper Dam - Pool level at top of dam - 65,000 CFS
Lower Dam - Not determined

1.3.3 Dam and Reservoir Data. Pertinent data on the dam and reservoir are shown in the following table:

TABLE 1.1 - OCCOQUAN DAM AND RESERVOIR DATA

ITEM	ELEVATION FEET M.S.L.	RESERVOIR CAPACITY AREA ACRES	RESERVOIR CAPACITY ACRE FEET	WATERSHED INCHES	LENGTH MILES
UPPER OCCOQUAN DAM					
Top of Dam	130	3600+	56,000+	1.8	16
Ungated Spillway Crest	120	1700±	30,300±	1.0	16
Normal Stream Bed	55±	0	0	0	0
LOWER OCCOQUAN DAM					
Top of Dam, Right Abutment	62	27	450	0.02	0.6
Top of Dam, Left Abutment	57+	24	310	0.01	0.6
Ungated Spillway Crest	52+	21	170	0	0.5
Normal Stream Bed	30±	0	0	0	0

SECTION 2 - PROJECT INFORMATION

2.1 Design: Most original engineering data for the upper and lower dams were lost during the shift in ownership from the American Water Works Service Company to the Fairfax County Water Authority. A set of existing contract drawings has the majority of the details for the upper dam. These plans do not indicate cross-section properties through the upper dam at either the non-overflow section nor at the end of the overflow section adjoining the left abutment. The plans give very limited information on the lower dam.

In 1972, Harza Engineering Company prepared a report on the raising of the upper dam five feet. After Tropical Storm Agnes, Harza amended the report (Appendix E - Reports 1 through 3) to include storm damage repairs to both dams. The Fairfax County Water Authority has the report and a set of contract documents, Phase 2, Storm Agnes Repairs and Restoration, Upper and Lower Occoquan Dams, prepared by Harza for the remedial work. Contract drawings for raising the dam five feet were never produced. However, this phase is still a future consideration and Harza is presently preparing a supplementary report. In 1975, the powerhouse floor and downstream wall of the intake structure were post-tensioned to eliminate seepage. A two-sheet set of plans, on file with the water authority, indicates the extent of the post-tensioning.

2.2 Construction: Along with the original design data, all original construction records were lost when ownership of the dam changed. Fairfax County Water Authority has several photo albums which show the upper dam during repairs after Tropical Storm Agnes. The rock adjacent to the toe and left abutment of the dam was severely eroded. Overburden on the downstream side of the right abutment was also severely eroded. A large and extensive apron was added to the toe to prevent further erosion. The left abutment was faced with a downstream protective concrete wall to reflect overflow away from the abutment. The downstream right abutment contact was faced with grouted riprap.

Little storm damage occurred to the lower dam. The dam was undercut at both abutments and a few feet in the center of the structure. Aprons were added at each abutment and concrete grout was placed in the center. A more detailed account of the damage and remedial work is provided in Reports 3 and 4 of Appendix E.

Construction records of the powerhouse and intake structure post tensioning were not found.

2.3 Evaluation: Post 1972 records indicate that remedial work has been accomplished in accordance with current state-of-the-art methods. The engineering records were obtained from two sources, Harza Engineering Company and Fairfax County Water Authority. The water authority should have a complete file of all known records at its office. The available information was inadequate, because of the lack of cross sectional data.

SECTION 3 - VISUAL INSPECTION

3.1 Findings: The visual inspection was performed during the second highest flow since Tropical Storm Agnes. Information observed in the field is outlined in Appendix C. An inspection report by Harza Engineering on the dam shortly after Tropical Storm Agnes is included in Appendix E, Report 4.

3.2 Evaluation: The visual inspection revealed no apparent problems that would require immediate action.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: Principal flow at the upper dam is over the ogee spillway. Water is withdrawn from the reservoir as outlined in Section 1.2.1 for water supply at a maximum rate of about 150 CFS. Two vertical hydroelectric generators for in-house power are utilized when sufficient water is available (approximately 300 CFS).

The principal flow at the lower dam is also over its spillway. Water is withdrawn through the intake structure on the right abutment. Layout of the reservoir system is given on Plate 4 of Appendix A.

The intake structures do not have any significant effect as regulating outlets especially during high flows. The water level in the upper reservoir is generally near the crest of the spillway. The pool level at the lower dam is dependent upon the discharge over the upper dam.

4.2 Maintenance of the Dam: The FCWA does not perform any periodic check-list type inspections. Maintenance is performed as required.

4.3 Maintenance of Operating Facilities: The mechanical and electrical equipment appear to be in fair to good condition but somewhat antiquated. There was no maintenance or operation manual at the time of the inspection.

4.4 Warning System: The water authority does not have an elaborate warning system. In case of imminent problems, state and local emergency services are notified. Personnel maintain a 24-hour observance.

4.5 Evaluation: The regulating outlets do not offer flood protection. The FCWA should institute a regular inspection program. An elaborate warning system is not considered necessary because of the 24-hour observation.

SECTION 5 - HYDRAULIC/HYDROLOGY DATA

5.1 Design. Provision was made in the design and construction of the upper dam to permit a 5-foot increase in height to accommodate future water supply demands. Modifications to the toe and abutments, completed by 1976 to repair damages caused by Tropical Storm Agnes, are based on a design outflow of 150,000 CFS. This outflow is two-thirds of the Probable Maximum Flood (PMF) and would be accompanied by about a 7-foot overtopping of the dam. Modifications to Fairfax County Water Authority facilities downsteam of the upper dam, which includes the lower dam, are based on a design flow of 112,500 CFS.

5.2 Hydrologic Records. Streamflow records have been maintained at official stream gaging stations as shown on Plate 2 of Appendix A and listed in the following table:

TABLE 5.1 STREAMFLOW STATIONS

STATION	DRAINAGE AREA SQ.MI.	RECORDS AVAILABLE
Occoquan River near Manassas	343	Apr 1968-date
Bull Run near Manassas	148	Oct 1950-date
Occoquan River near Occoquan	570	Feb 1913-May 1916 Dec 1920-Sep 1923 Jun 1937-Jun 1956(a)

(a) Site presently submerged by Occoquan Main Dam Reservoir.

5.3 Flood Experience. The greatest flood known to have occurred was that of June 1972. Large floods of records at gages listed in the previous table are shown in the following table:

TABLE 5.2 - MAXIMUM FLOODS

ITEM	OCCOQUAN RIVER NEAR OCCOQUAN	OCCOQUAN RIVER NEAR MANASSAS	BULL RUN NEAR MANASSAS
Drainage area, Sq.Mi.	570	343	148
Maximum Discharges, CFS			
Jun 1972 (Agnes)	75,000	56,400	76,100
Oct 1942	37,000	--	--
Apr 1937	29,200	--	--
Sep 1975 (Eloise)	--	22,000	13,800

The maximum rise in the reservoir was to elevation 130.8 in June 1972 and 128.2 in September 1975. A maximum dischjarge of 34,400 cfs was computed for the Sectember 1975. Since flashboards were on the dam in the September 1975 flood, it is estimated that the water level would have risen to elevation 126.8 if the flashboards were not in place.

5.4 Flood Potential. A probable maximum flood was determined by Harza Engineering Company in a study leading to a proposal for modifying the upper dam to accommodate higher floods. Methodology used was essentially the same as that used by the Corps of Engineers and was accepted as computed. Floods of various frequencies were indicated in a preliminary report of a Flood Insurance Study for the Town of Occoquan by the Federal Insurance Administration. A preliminary determination of a 100-year peak discharge of 52,000 CFS was accepted as being a reasonable determination. The flood potential for the lower dam is essentially the same as for the upper dam.

5.5 Reservoir Regulation. Pertinent dam and reservoir data are shown in Table 1.1.

Spillway discharge capacity, reservoir area and volume data, and a tailwater rating curve from a report by Harza Engineering Company were deemed adequate for the upper dam and were extended upward to indicate probable values for the PMF. Routing of the PMF through the reservoir was accomplished by Harza using the computed hydrograph with appropriate spillway discharge and reservoir capacity curves. Reduction of other floods was estimated from the relative reduction in the PMF.

Available reservoir area and capacity data in the lower reservoir was extended upward to provide an estimate of these data at the top of the dam. Headwater and tailwater rating curves were not available or developed for the lower dam.

Lake Jackson is located on the Occoquan River about 16 miles upstream. The possibility of a break in the Lake Jackson Dam during a PMF flow, which could cause an increase in outflow from the Upper Occoquan Dam of as much as 15,000 CFS was considered as shown in Table 5.3. It will be noted that the effect of this increased flow would increase the headwater elevation about one foot and tailwater about three feet. An instantaneous break of Lake Jackson Dam with the reservoir at normal level would have a negligible effect at Occoquan.

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

TABLE 5.3 - UPPER OCCOQUAN RESERVOIR PERFORMANCE

ITEM	Normal Conditions	FLOOD			
		100 yr	Agnes	½ PMF	PMF(c)
Peak flow, CFS					
Inflow	200	52,000	77,000	113,000	226,000
Outflow	200	51,000	75,000	110,000	220,500
(Outflow)		(66,000)	(90,000)	(125,000)	(235,000)
Peak elevation, FT. MSL	120+	129	131	134	141.25
		(130)	(132)	(135)	(142)
Spillway (a)					
Depth of flow, FT.	1	9	11	14	21.25
Avg Velocity, FPS	1-2	11	13	15	18
Non-overflow section (b)					
Depth of flow, FT.	-9	-1	1	4	11.25
Avg Velocity, FPS	-	-	3	6	10
Tailwater elevation, FT. MSL	55	75	81	88	110
		(79)	(84)	(91)	(112)

(a) Crest elevation 120.0
(b) Top elevation 130.0
(c) 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.

NOTE: Items ( ) based on possible increase in peak if Lake Jackson Dam was breached.

Performance of the Lower Reservoir was not determined since data was not available. Observance of a relatively high flow of about 20,000 ± CFS on the day of inspection indicated a small drop, therefore, failure under extreme flood conditions would have little effect on downstream flows. Also failure under normal flow conditions would not produce damaging stages because of the limited storage in the reservoir.

5.7 Reservoir Emptying Potential. Assuming a median inflow of 200 CFS, it would take about 2½ weeks to draw down the upper reservoir from pool elevation 120 to elevation 66. This assumes that valves on all regulating outlets described in paragraph 1.2.1 are operable and fully opened. Assuming the same 200 CFS median inflow, it would take 2 to 3 days to draw down the lower reservoir about 10 feet, after which time outflow through the 3 x 3-foot blowoff would approximate inflow. This assumes the sluice gate in the intake structure to be fully operable.

5.8 Evaluation: Hydrologic evaluation guidelines recommend a Spillway Design Flood equivalent to the PMF for the upper dam. Having a maximum capacity of only about one-third of the peak PMF outflow, the spillway is therefore inadequate; however, this inadequacy must be qualified. Assuming that post-Agnes abutment modifications up to elevation 136 retain structural integrity during flooding to that height, the spillway and dam are then capable of passing about 140,000 CFS. This outflow is almost two-thirds of the PMF and, in keeping with the aforementioned preliminary Flood Insurance Study, would have an average return interval of several thousand years.

The guidelines recommend a range of Spillway Design Floods from one-half PMF to PMF for the lower dam. Because failure would have little effect on anything downstream, evaluating the spillway against one-half the PMF was selected. Having an estimated maximum capacity of approximately 15 percent of the peak one-half PMF outflow, the spillway is clearly inadequate. Post-Agnes channel and dam modifications were based on a design flow equal to one-half of the PMF, however, and failure induced damage would be inconsequential.

SECTION 6 - DAM STABILITY

6.1 Upper Dam

6.1.1 Structual Stability: Stability analyses have been performed on both the non-overflow and spillway sections of the upper dam. The non-overflow section analysis uses cross-section B-B, Plate 5, Appendix A and assumes the bottom is at elevation 75.0 and 95.0. The spillway section uses cross-section A-A on the same sheet. Stability of the non-overflow section is outside the Corps of Engineers criteria for loadings produced by water levels above the 100-year flood. The spillway section stability is within Corps criteria for all levels of water up to and including the PMF. A stability check was not performed on that portion of the overflow section adjoining the left abutment because of insufficient cross sectional data. The system is located in a seismic Zone II and seismic stability was not considered. Refer to Appendix D for stability calculations.

Stability analyses of the powerhouse and intake structure were not accomplished. The complexity of the reinforcement places the stability check outside the scope of the Phase I report.

6.1.2 Foundation and Abutments Conditions: The geology of the dam site and site preparations are documented in Report 1 and 2 of Appendix E. Refer to Plate 6 of Appendix A for a plan view of the foundation. Remedial measures performed are outlined in the contract documents noted in Section 2.1.

6.1.3 Evaluation: The spillway stability analysis is dependent on the foundation drains remaining functional. Without the foundation drains, the stabilities at PMF would not be acceptable under any criteria. Further information pertaining to foundation elevations is needed for those portions of the dam outlined in Section 6.1.1, in order to perform a more precise stability check.

6.2 Lower Dam

6.2.1 Structural Stability: A stability analysis of the lower dam was not accomplished. Adequate cross-sectional data could not be found. (Appendix A - Plates 8 and 9) Futhermore, this analysis was not necessary since failure would have neglible effect on downstream stages.

6.2.2 Foundation and Abutments: In September 1972, Harza Engineering directed an exploratory investigation of the lower dam. The investigation was performed by Soil Consultants, Inc. of Merrifield, Virginia. Ten core borings were drilled through the dam into the foundation. Records of the exploration were not available during the inspection. However, the findings are recorded in Appendix E, Report 3, which also includes an evaluation of the concrete dam. Remedial measures performed are also outlined in the contract documents noted in Section 2.1.

6.2.3 Evaluation. The cross sectional dimensions of the structure need to be better defined before a stability analysis can be performed. The unavailable exploration records should provide the exact foundation elevation needed for the analysis.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: Corps criteria recommends a spillway capacity equivalent to the PMF for the upper dam. The spillway of the upper dam is capable of passing only one-third the PMF before overtopping occurs. Therefore, the spillway is considered inadequate. However, it is not considered seriously inadequate, because the dam has been modified to accommodate overtopping flows of about two-thirds the PMF. A stability check indicates that the overflow section is within Corps criteria for flows up to and including the PMF. However, it is based on assuming foundation drains remain functional. Further information is needed to perform a complete stability check on other portions of the structure.

For the lower dam, Corp criteria recommends a spillway design flood equal to one-half to full PMF. The spillway can only pass about 15 percent of the one-half PMF without overtopping. Therefore, this spillway is inadequate. However, it is not considered seriously inadequate for the following two reasons: the dam has been modified to withstand overtopping flows up to one-half the PMF and a dam failure during any flow would be inconsequential. Stability checks of the dam were not performed.

Post 1972 work on the Occoquan Reservoir System was performed according to the current state of the art. Other available information was inadequate, because of lack of cross sectional data. The visual inspection revealed no apparent problems that would require immediate action. However, the FCWA should institute a regular inspection program.

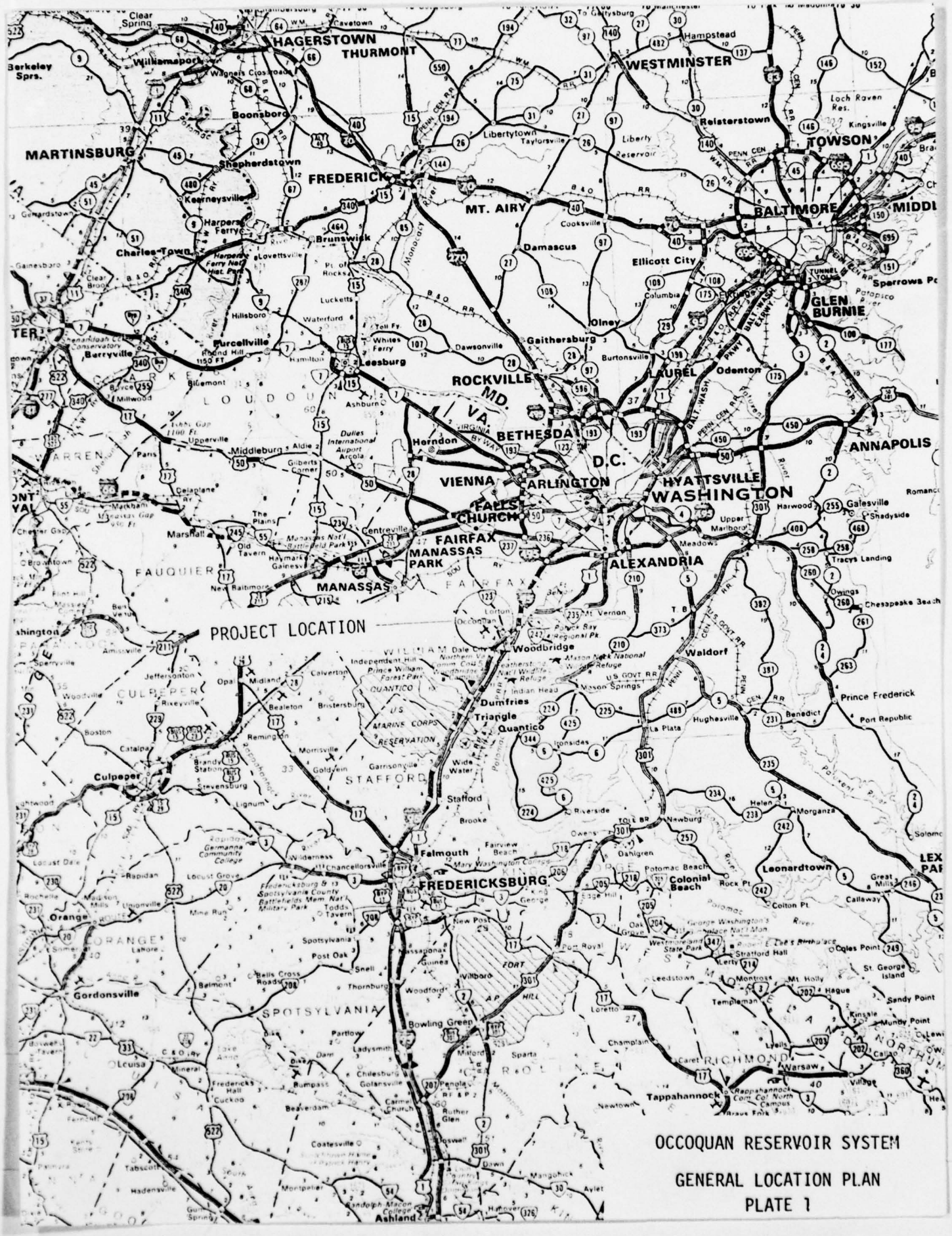
7.2 Remedial Measures: There is no immediate need for remedial measures. However, the following recommendations are offered and should be completed within 6 months.

7.2.1 The FCWA should maintain a complete file of all known records at its office.

7.2.2 A yearly inspection of the system should be performed. This includes a systematic and regular check of all foundation drains to insure proper functioning. Any drain found to be non-functional should be immediately corrected.

7.2.3 Exact foundation elevations for the upper dam along the non-overflow section and along the portion adjoining the left abutment should be determined.

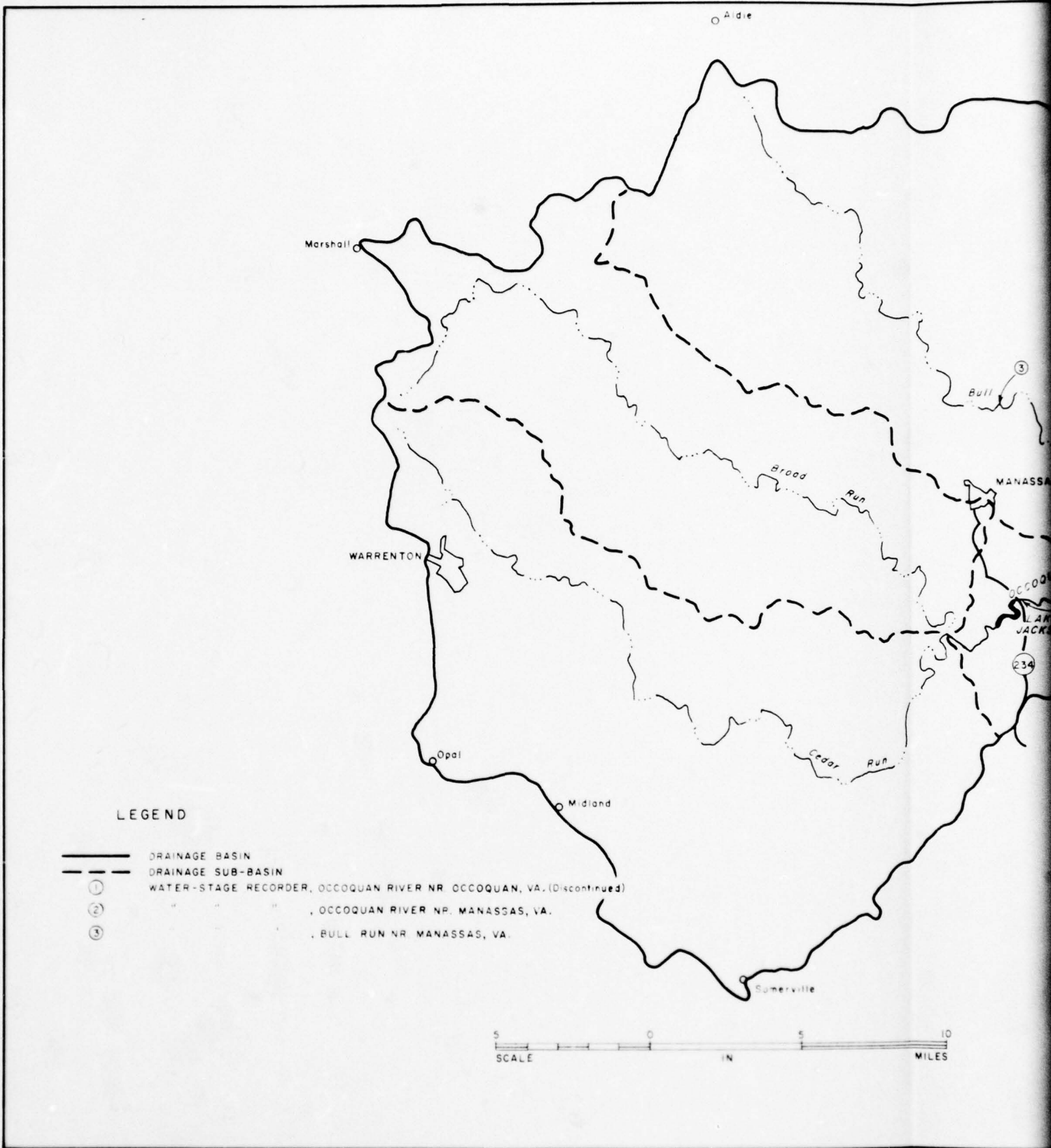
APPENDIX A - MAPS AND DRAWINGS

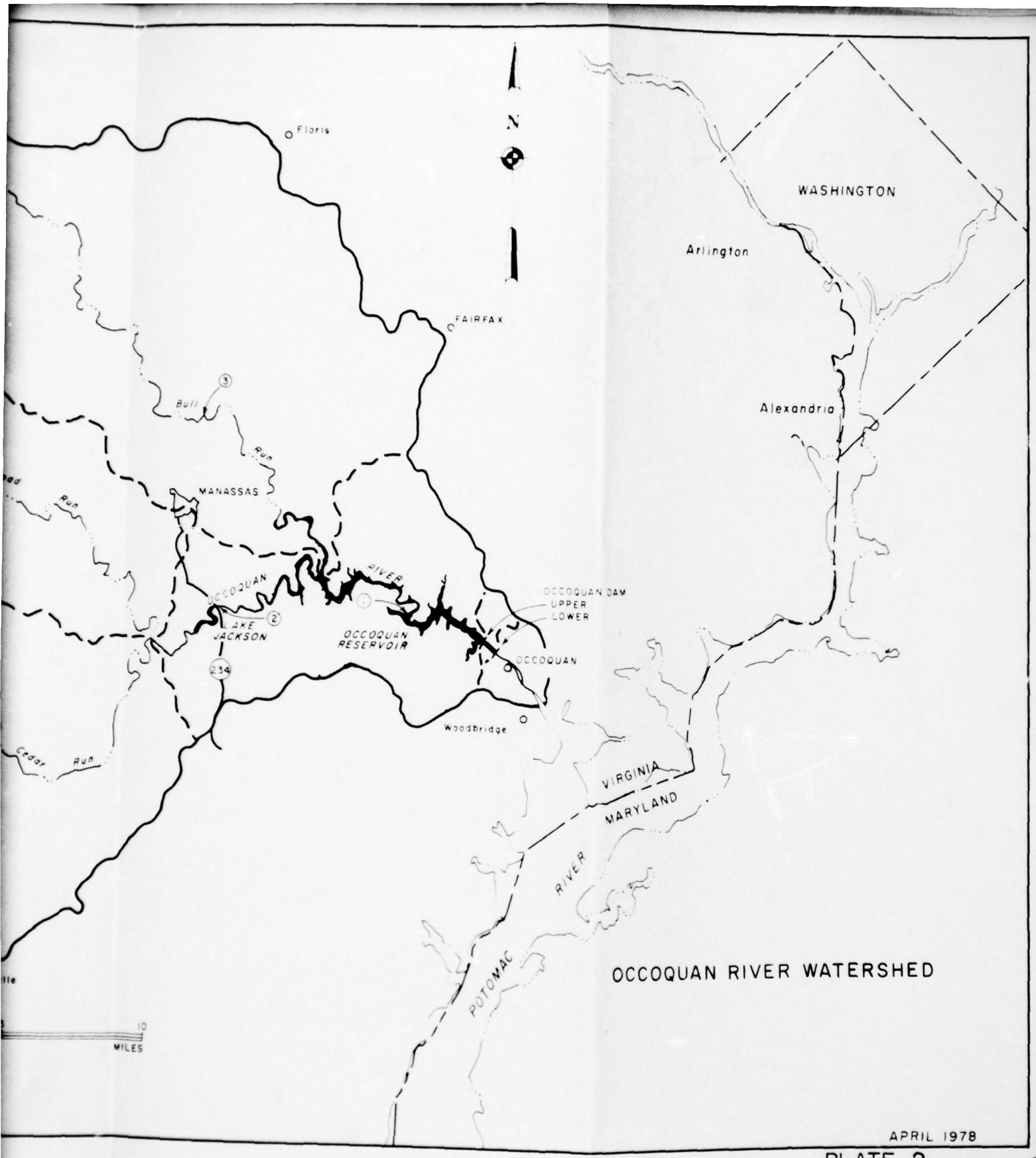


OCCOQUAN RESERVOIR SYSTEM

GENERAL LOCATION PLAN

PLATE 1

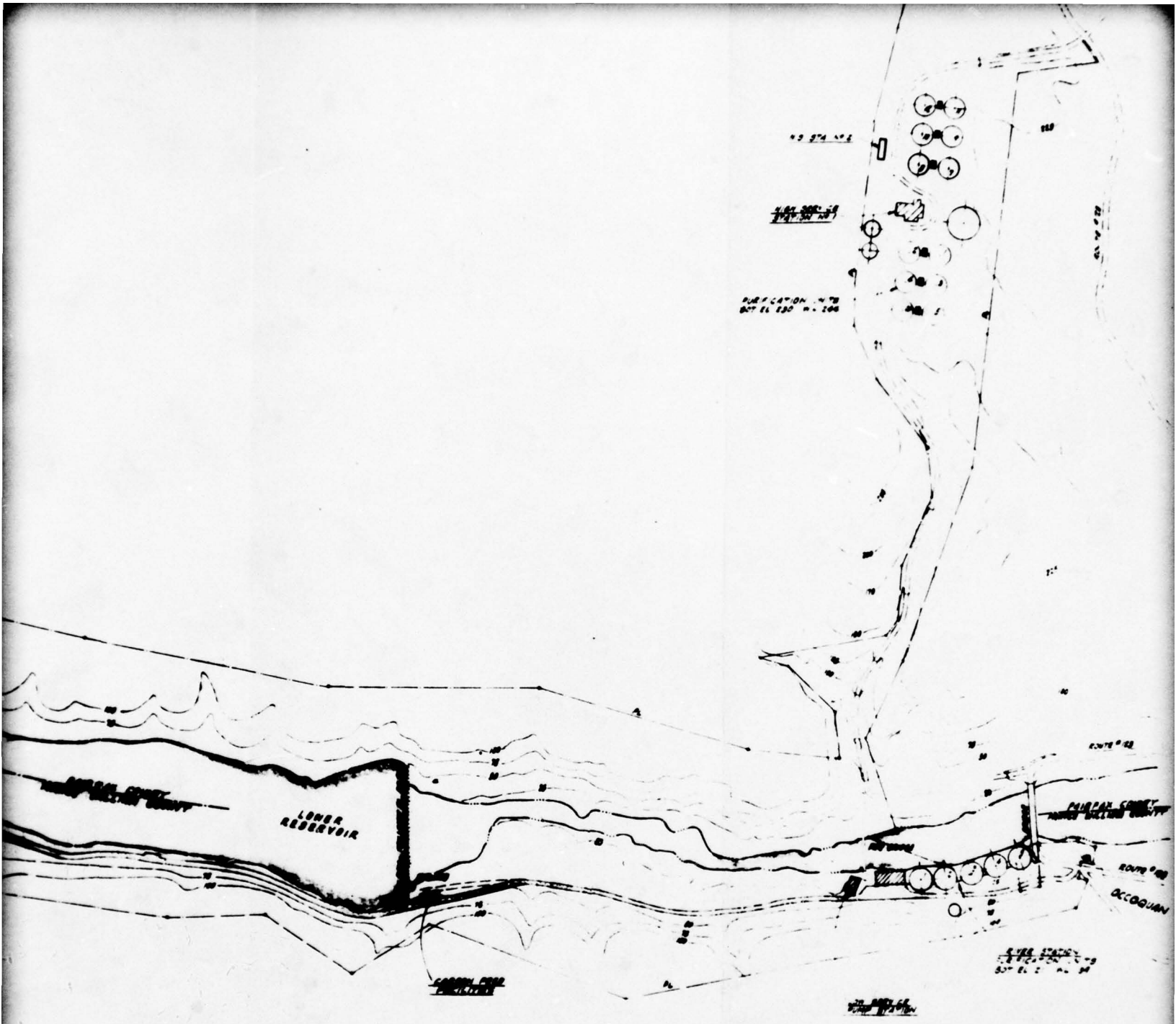

Aldie
Marshall
Bull
Run
Broad
Run
MANASSA
WARRENTON
OCCOQ
LAK
JACK
234
Opal
Cedar
Run
Midland
Somerville
LEGEND
DRAINAGE BASIN
DRAINAGE SUB-BASIN
① WATER-STAGE RECORDER, OCCOQUAN RIVER NR. OCCOQUAN, VA. (Discontinued)
② " " " , OCCOQUAN RIVER NR. MANASSAS, VA.
③ " " " , BULL RUN NR. MANASSAS, VA.
5 0 5 10
SCALE IN MILES



APRIL 1978

PLATE 2


N
OCCOQUAN CREEK
UPPER RESERVOIR
LOWER RESERVOIR



REVISIONS

OCCOQUAN SUPPLY
UPPER RESERVOIR DAM & INTAKE
GENERAL PLAN

THE ALEXANDRIA WATER COMPANY
ALEXANDRIA, VA.

AMERICAN WATER WORKS SERVICE COMPANY, INC.
PHILADELPHIA, PA.

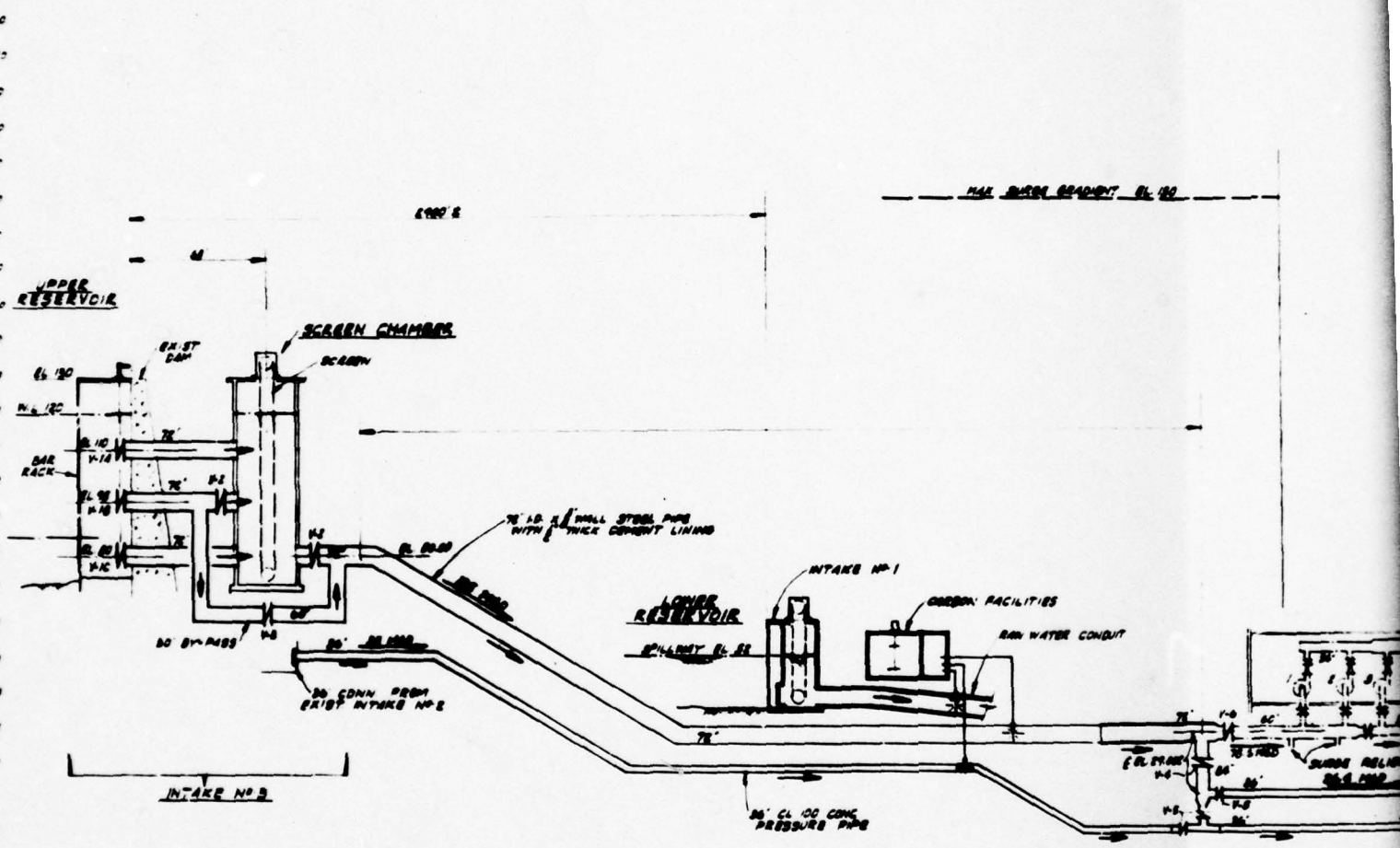
SCALE: 1" = 200'
USE ENGINEERING ONLY
DRAWN BY JB
DATE
CHECKED BY
APPROVED
PROJECT MGR.
SUPV. ENGR.

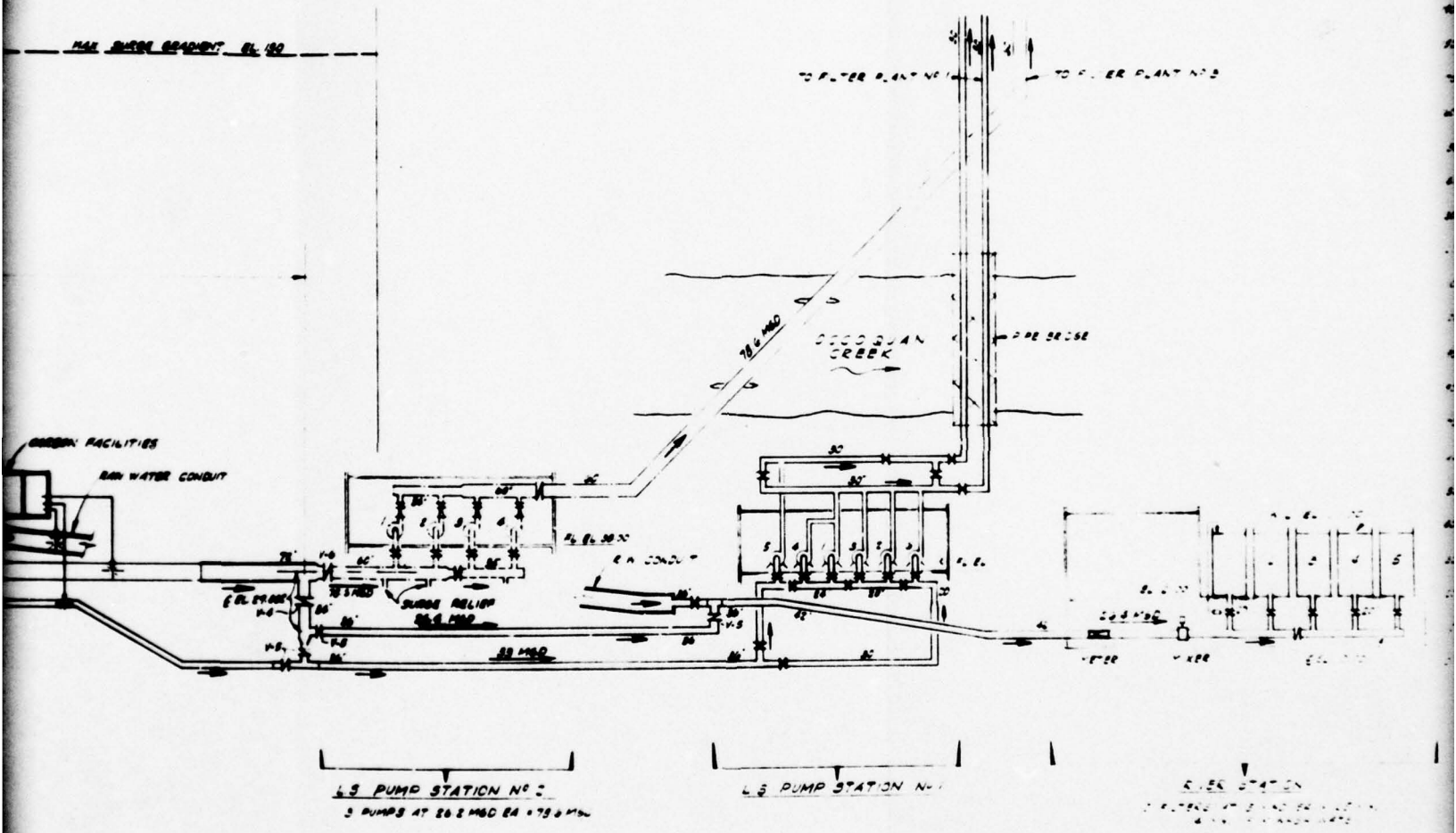
PLANS PREPARED BY
E. H. PLESSER
VIRGINIA CERT. NO. 7715

USE APPROVED DRAWINGS ONLY
FOR CONSTRUCTION PURPOSES

84-640-2

PLATE 3


MAX SURGE GRADIENT EL 180
UPPER RESERVOIR
SCREEN CHAMBER
EX. ST. DAM
SCREEN
EL 190
W.L. 180
EL 110
W.L. 98
EL 60
BAR RACK
78" I.D. X 1/4" WALL STEEL PIPE WITH 6" THICK CEMENT LINING
EL 80.88
50' BY-PASS
20" CONN FROM FIRST INTAKE NO 2
INTAKE NO 1
LOWER RESERVOIR
SCREEN FACILITIES
RAW WATER CONDUIT
INTAKE NO 2
96" CL 100 CONC PRESSURE PIPE
L.S PUMP STATIO
3 PUMPS AT 26.2 MG



REVISIONS

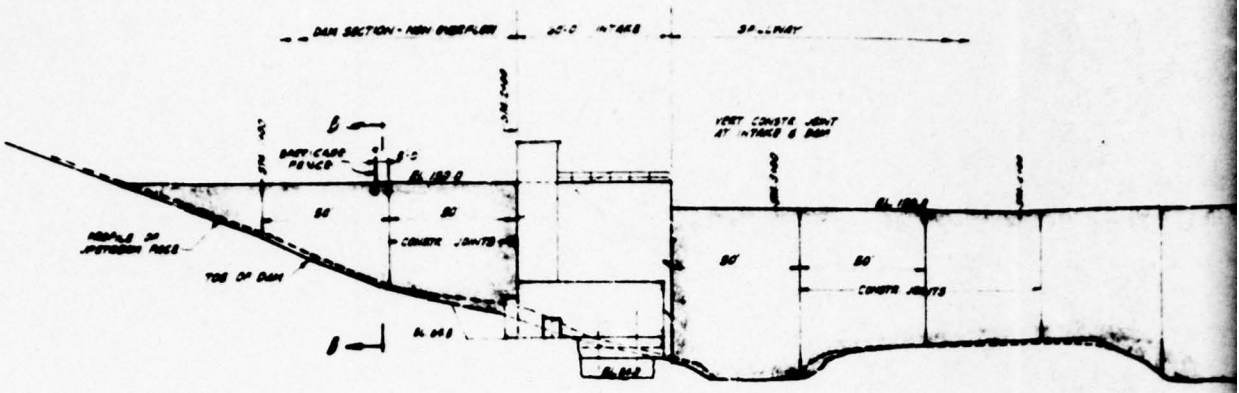
AMERICAN WATER WORKS SERVICE COMPANY, INC.
THREE PENN CENTER PLAZA PHILADELPHIA PA 19102

SCALE USE DIMENSIONS ONLY
DRAWN BY DATE CHECKED BY
APPROVED PROJECT INSP BY

USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

84-360

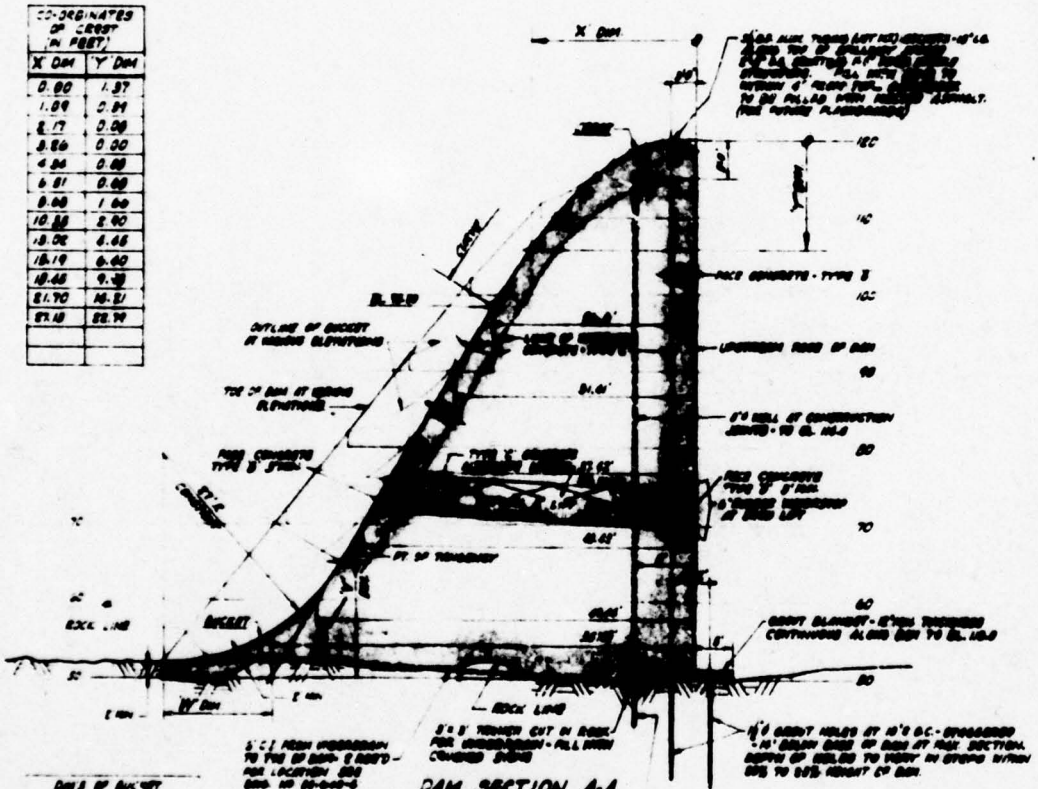
PLATE 4



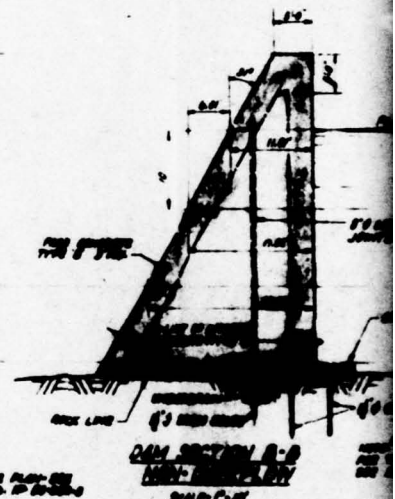
ELEVATION OF DAM

SCALE: 1"=80'

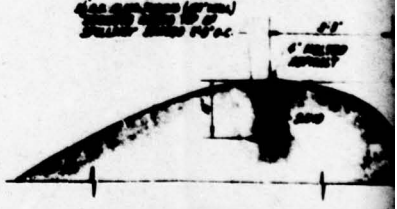
CO-ORDINATES OF CREST (IN FEET)	
'X' DAM	'Y' DAM
0.00	1.37
1.09	0.91
2.17	0.09
3.26	0.00
4.34	0.09
6.51	0.69
8.68	1.60
10.85	2.90
13.02	4.46
15.19	6.40
17.36	9.13
21.70	16.21
27.12	22.79



DAM SECTION A-A
SPILLWAY
SCALE: 1"=10'

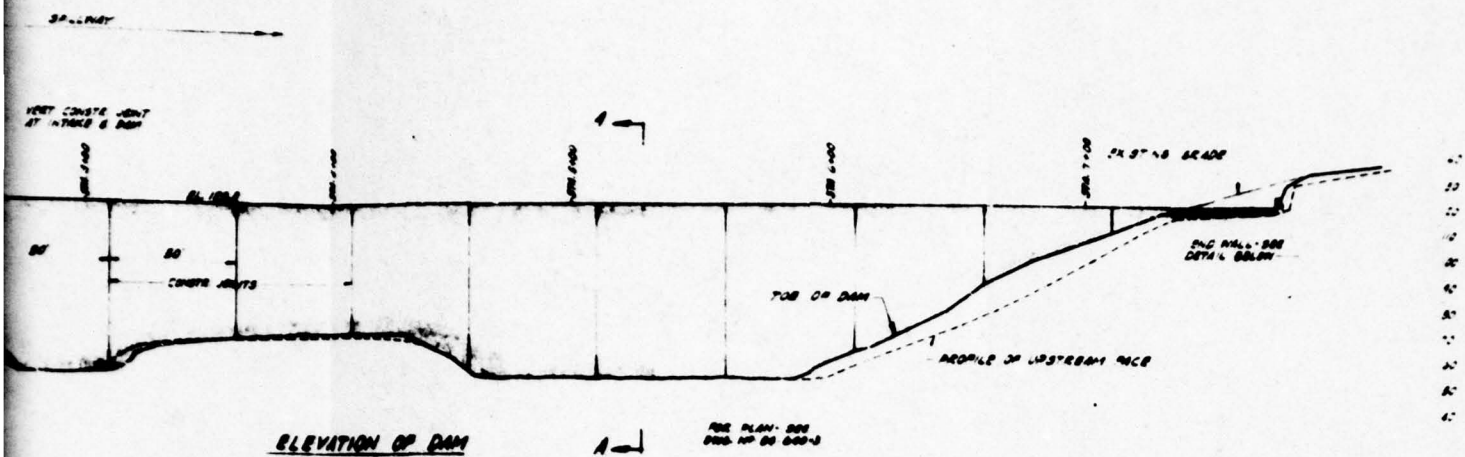


DAM SECTION A-A
SCALE: 1"=10'



DETAIL OF PIPE SOCKETS ALONG DAM
SCALE: 1"=40'

DIMS OF BUCKET AT VARIOUS ELEVATIONS (IN FEET)		
EL. Ev	'H' DIM WIDTH	'V' DIM DEPT
80	14	15
78	13	4 10
76	12	4 89
74	11	14 55
72	10	14
70	9	13 70
68	8	13 39
66	7	13 05
64	6	12
62	5	12 75
60	4	12 2
58	3	11 90
56	2	11 5



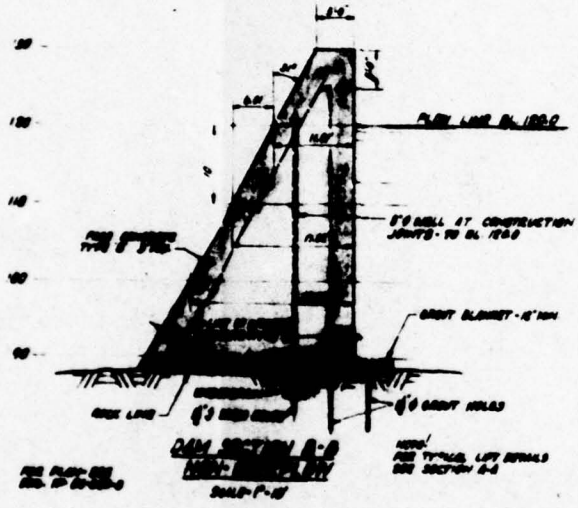
ELEVATION OF DAM

SCALE: 1"=50'

A

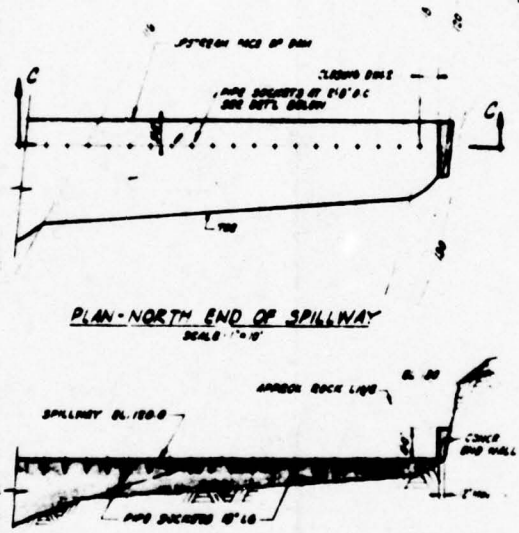
A

FOR PLAN: SEE DWG. NO 84-640-3



DAM SECTION A-A

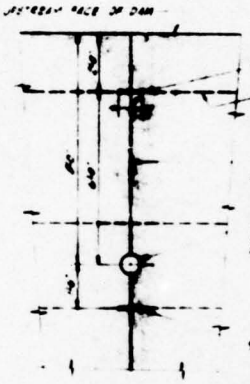
SCALE: 1"=10'



PLAN - NORTH END OF SPILLWAY

SCALE: 1"=10'

SECTION C-C



SECTIONAL PLAN OF VERT. CONSTR. JT

SCALE: 1"=10'

NOTE:
9" RUBBER WATERSTOP TO BE FLAT DUMBBELL TYPE BY: SERVICISED PRODUCTS CO CHICAGO ILL.



DETAIL OF PIPE SOCKETS ALONG DAM

SCALE: 1"=5'

PLANS PREPARED BY
E. M. ALDRICH
VIRGINIA CORP. NO 776

REVISIONS

OCCOQUAN SUPPLY
UPPER RESERVOIR DAM & INTAKE DAM
SECTIONS & DETAILS
THE ALEXANDRIA WATER COMPANY
ALEXANDRIA VA.

AMERICAN WATER WORKS SERVICE COMPANY, INC.
PHILADELPHIA, PA.

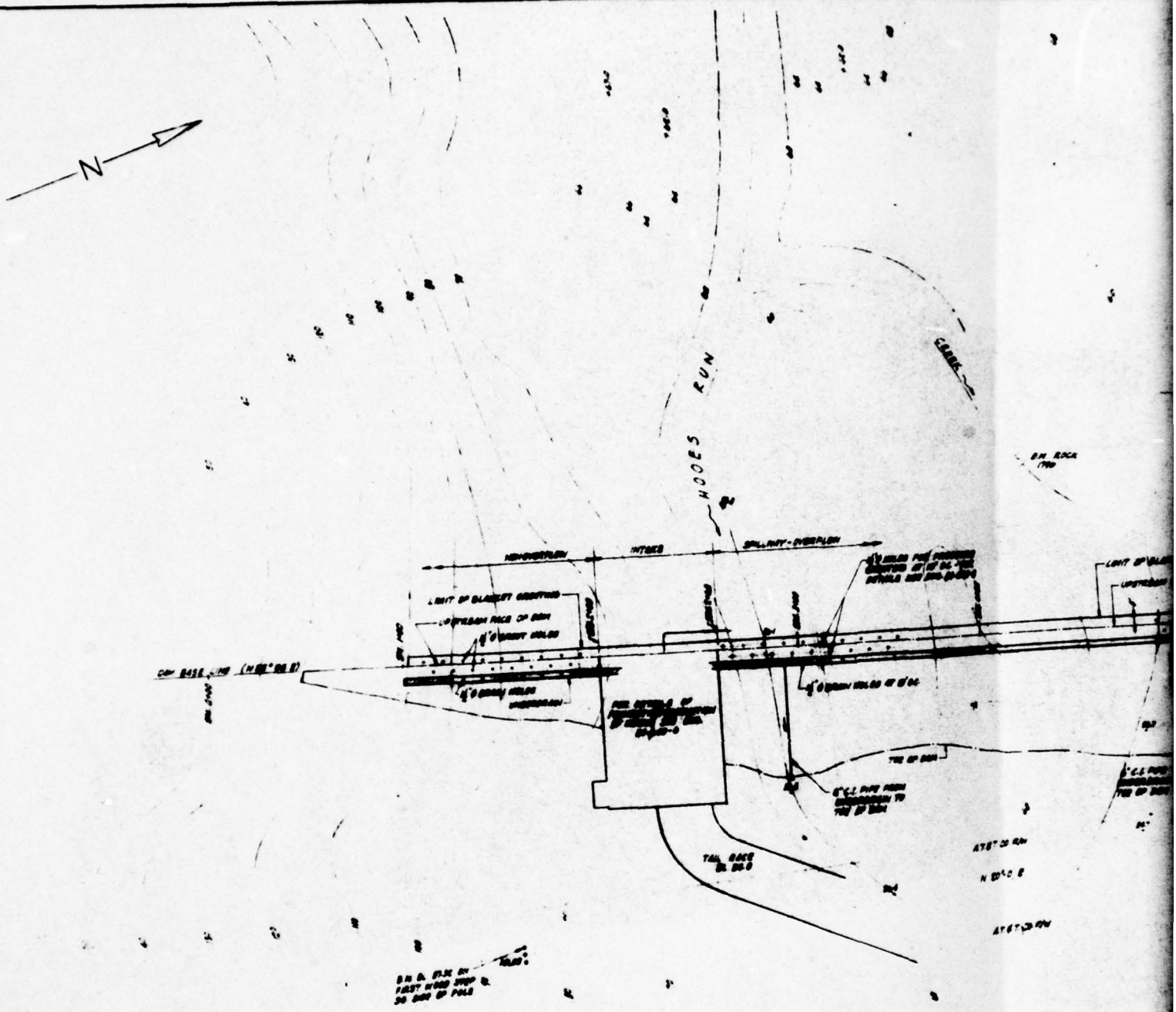
SCALE: AS SHOWN
DRAWN BY J.T.B.
APPROVED M.J.C.
DATE: OCT. 30, 1951 CHECKED BY E.L.S.
PROJECT P-55 SHEET NO.

USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

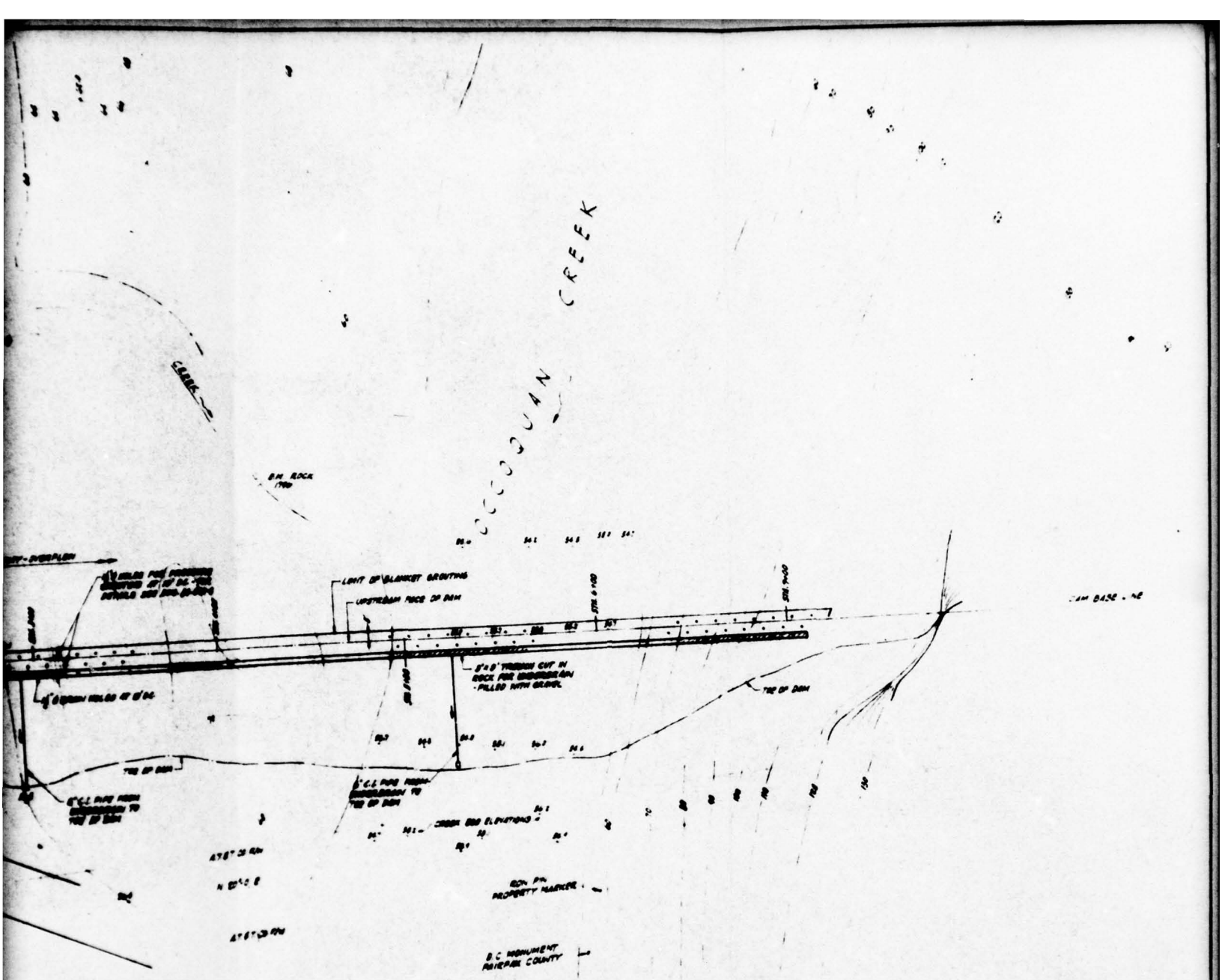
USE SUPERSEDED ONLY

84-640-5

PLATE 5



EXCAVATION PLAN OF DAM
SCALE: 1" = 80'



EXCAVATION PLAN OF DAM
SCALE: 1" = 80'

REVISIONS

PLANS PREPARED BY
E. M. ALDRICH
VIRGINIA CERT. N° 775

OCCOQUAN SUPPLY
UPPER RESERVOIR DAM & INTAKE
DAM
FOUNDATION PLAN

THE ALEXANDRIA WATER COMPANY
ALEXANDRIA, VA

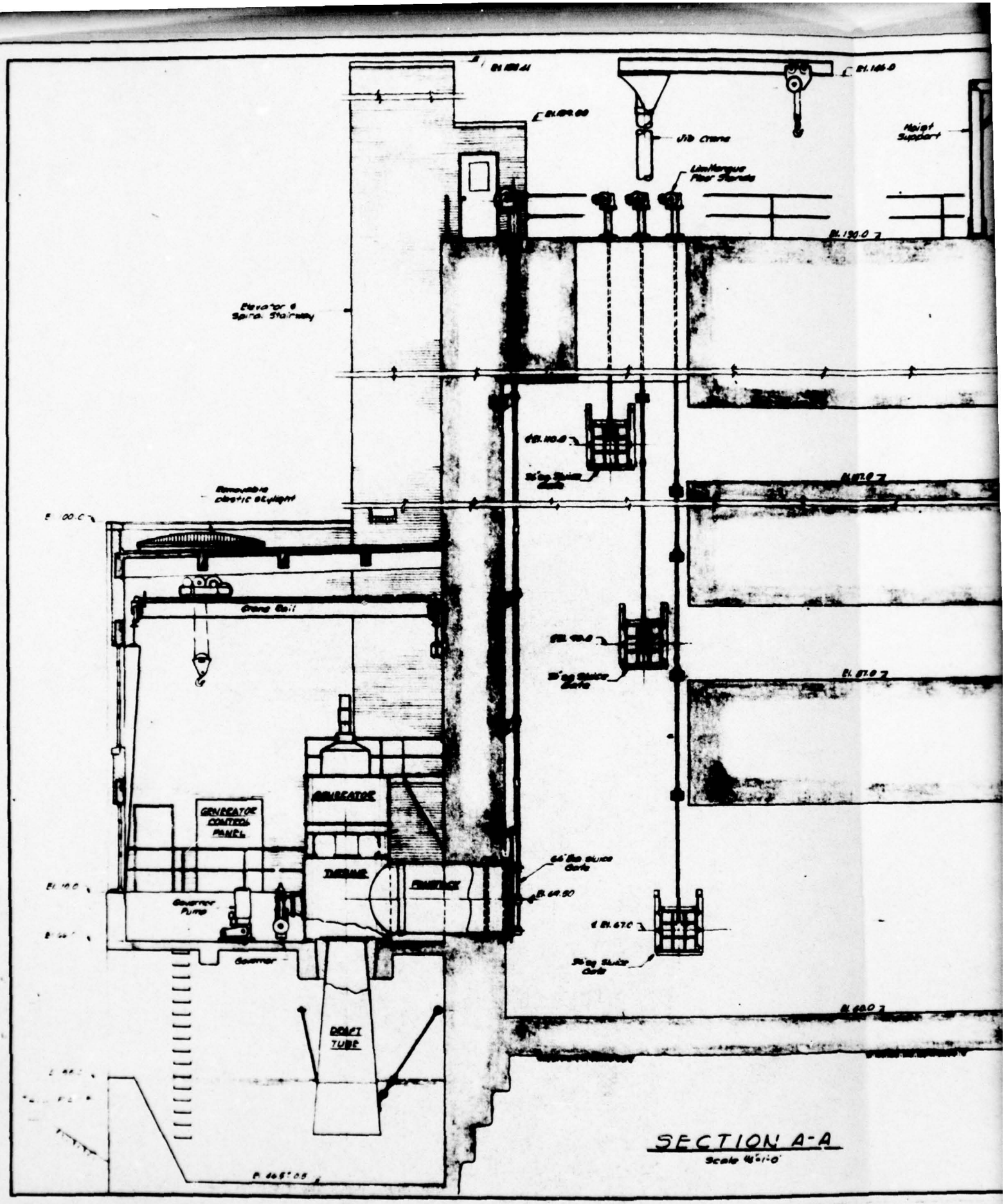
AMERICAN WATER WORKS SERVICE COMPANY, INC.
THREE PENN CENTER PLAZA PHILADELPHIA 2 PA

SCALE 1"=80' USE DIMENSIONS ONLY
DRAWN BY J.S. DATE 12/ 1/ 66 CHECKED BY E. J. S.
APPROVED M.J.C. PROJECT A-66 INSP BY

USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

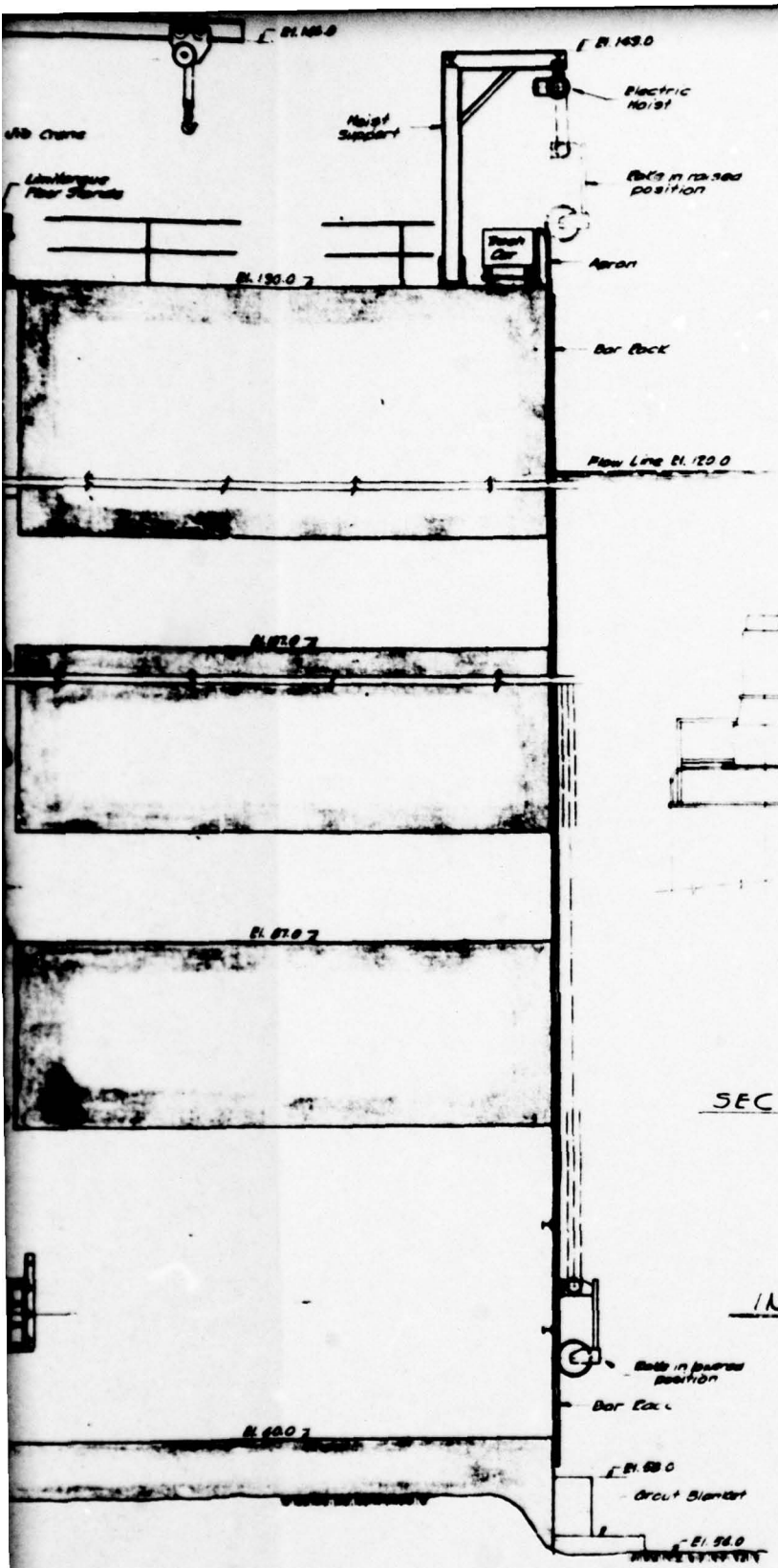
84-640-4

PLATE 6

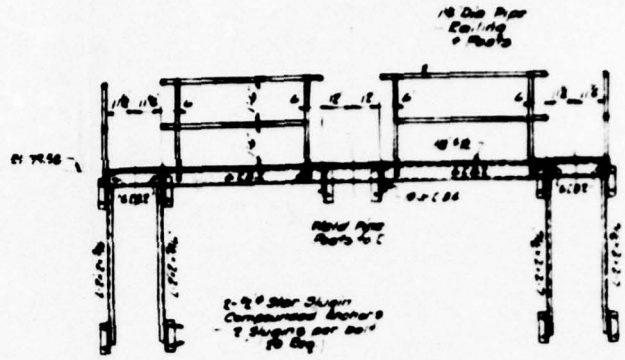


SECTION A-A

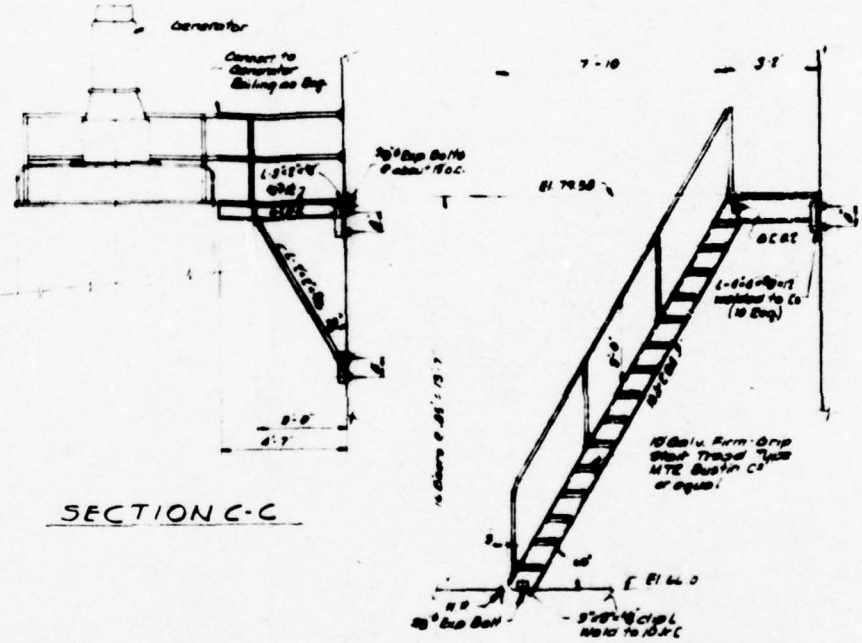
Scale 1/8"=1'-0"



CTION A-A
Scale 1/8"=1'-0"



SECTION B-B



SECTION C-C

SECTION D-D

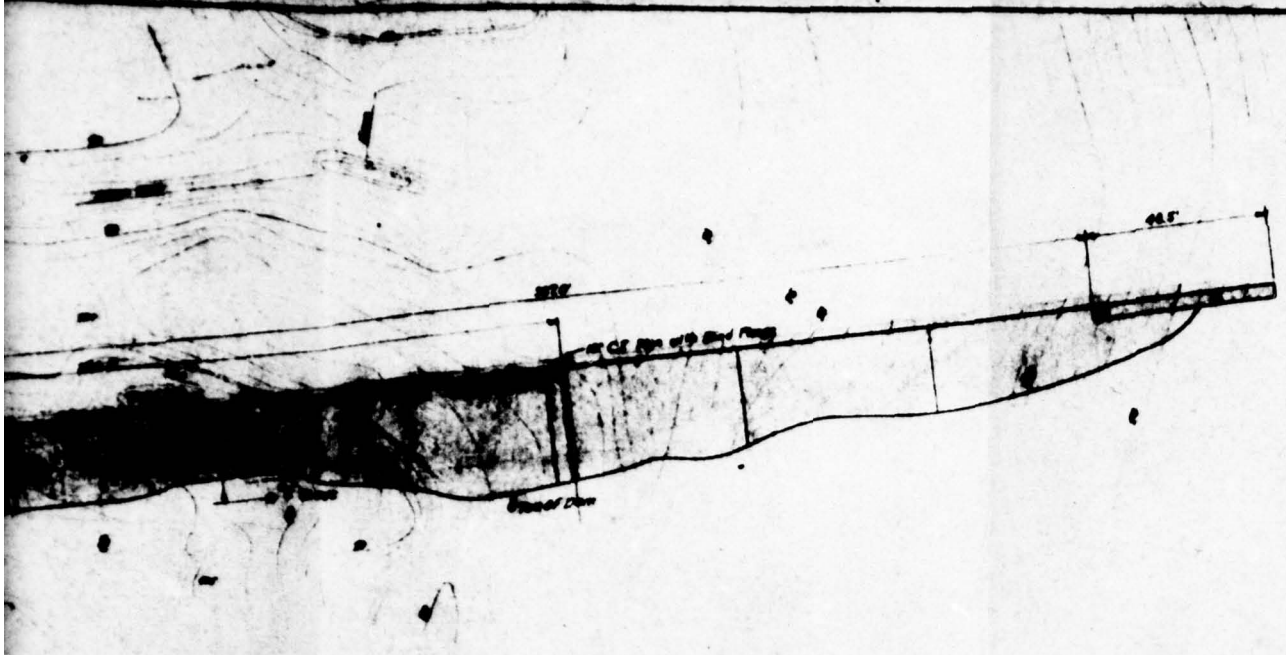
INSPECTION PLATFORM DETAILS
Scale 1/8"=1'-0"

OCCOQUAN SUPPLY
UPPER RESERVOIR DAM & INTAKE
POWER HOUSE - MECHANICAL
EQUIPMENT SECTIONS
THE ALEXANDRIA WATER COMPANY
ALEXANDRIA, VA.
AMERICAN WATER WORKS SERVICE COMPANY, INC.
SCALE: AS NOTED
DRAWN BY P.J.V.
APPROVED A.J.G.
84-660-32

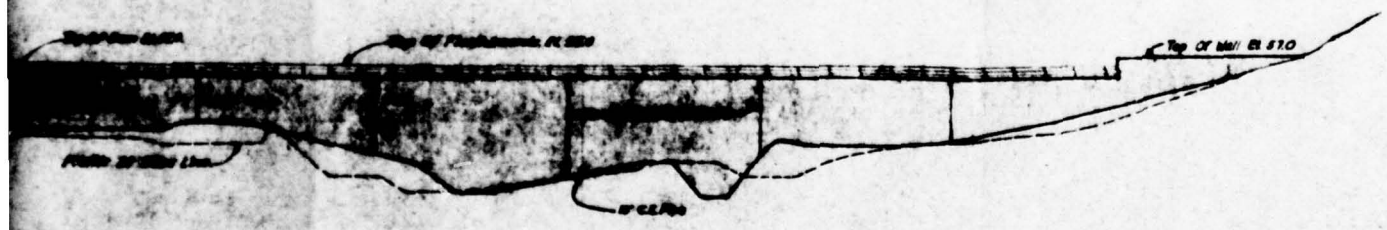
PLANS PREPARED BY
E.H. ALDRICH
VIRGINIA CERT. No 778

PLATE 7

PLAN



PLAN



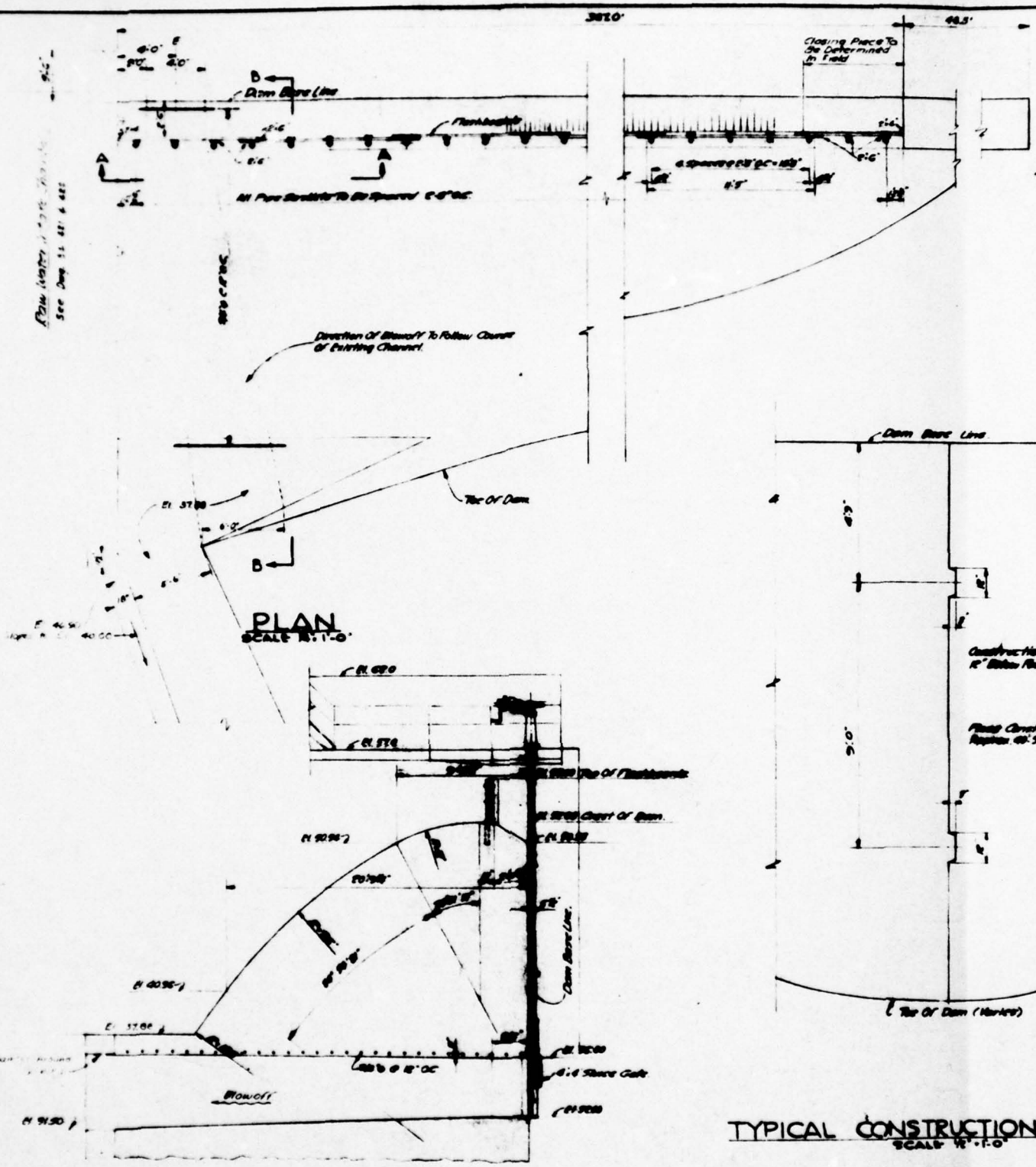
ELEVATION

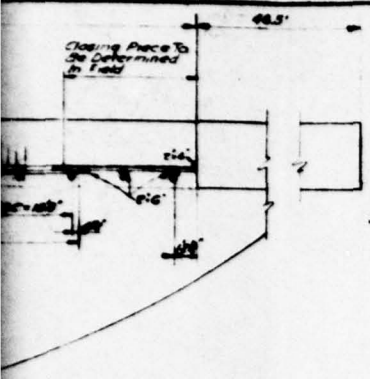
For Location & General Plan see Dwg. 86-619.

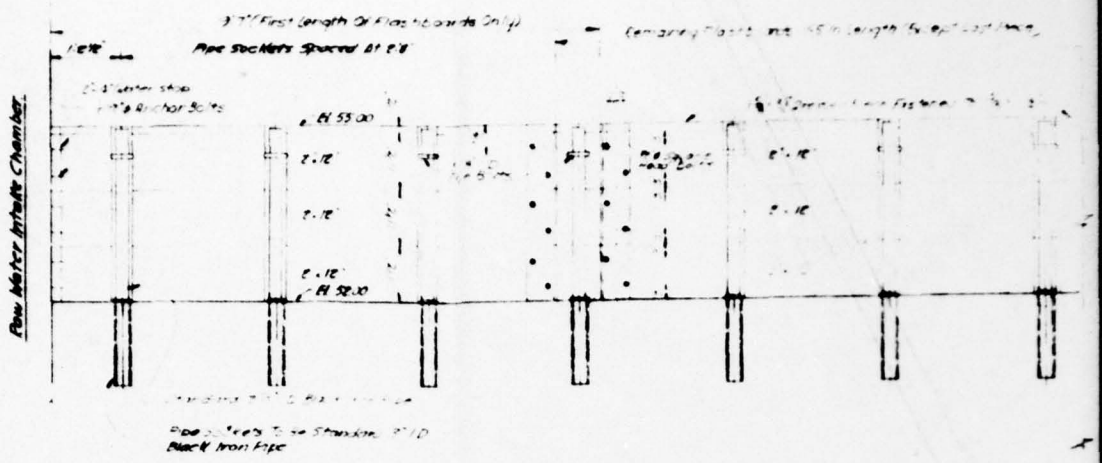
OCCOQUAN SUPPLY

DAM AND INTAKE
GENERAL PLAN

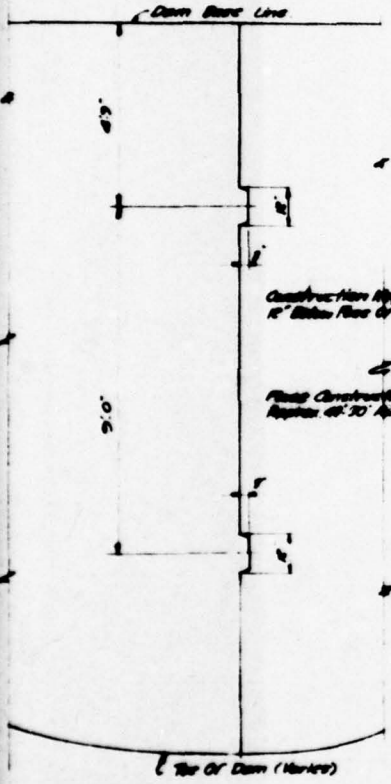
THE ALEXANDRIA WATER COMPANY
ALEXANDRIA, VA.

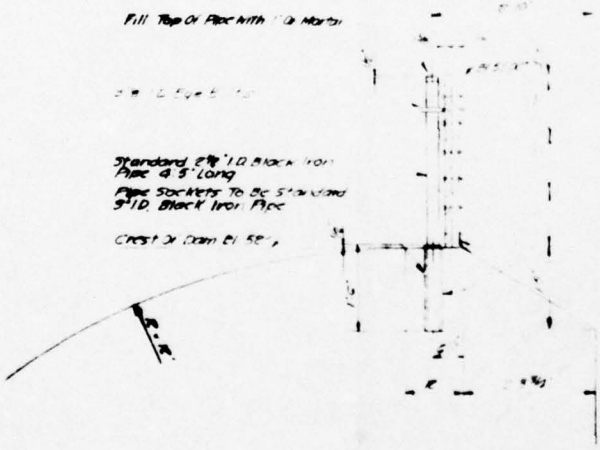

Closing Piece To Be Determined In Field
Dam Base Line
Direction Of Blowoff To Follow Course Of Existing Channel
Toe Of Dam
PLAN
SCALE 1" = 1'-0"
Dam Base Line
Construction Keys To End 15' Below Floor Of Dam.
Toe Of Dam (Varies)
TYPICAL CONSTRUCTION JOINT
SCALE 1" = 1'-0"
Blowoff
SECTION BB
SCALE 1" = 1'-0"





SECTION AA
SCALE 1/8"=1'-0"





DETAIL OF FLASHBOARD
SCALE 1/8"=1'-0"

TYPICAL CONSTRUCTION JOINT
SCALE 1/8"=1'-0"

NOTES
1. Concrete To Have A 28 Day Compressive Strength Of 3000 PSI
2. All Lumber To Be Cypress S4S Tongue & Groove
3. Fill Pipe Sockets With Stand & Cap With Asphalt After Pipe Supports Have Been Placed

For General Plan, see Dwg. 86-620
For Detail of end condition at earth bank see Dwg 86-620

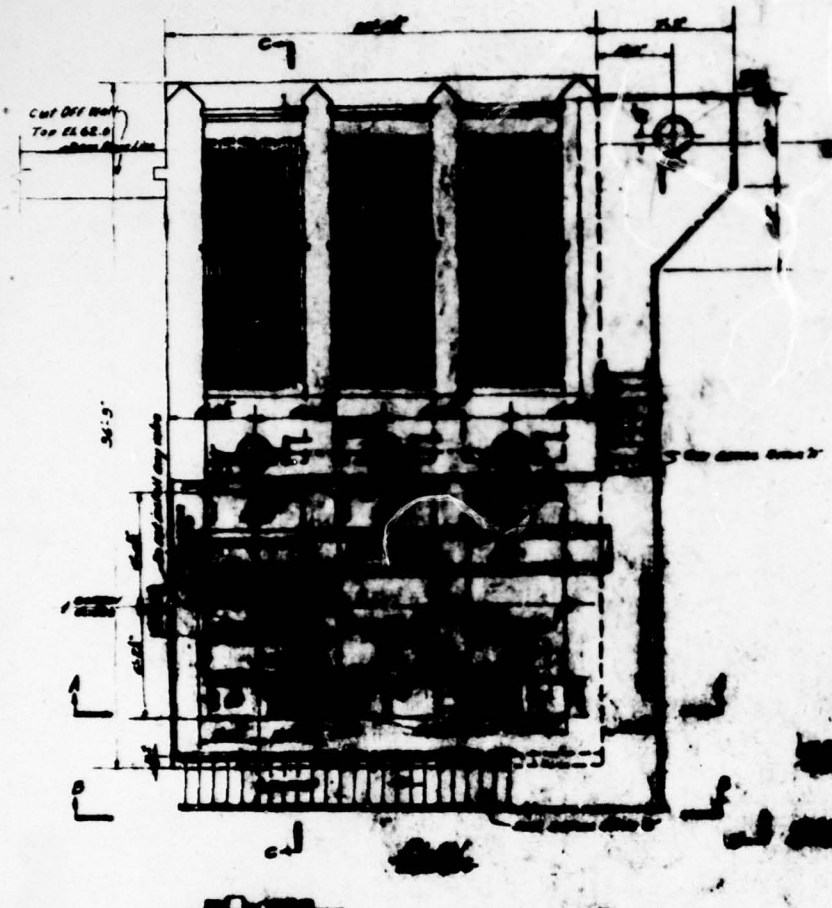
OCCOQUAN SUPPLY

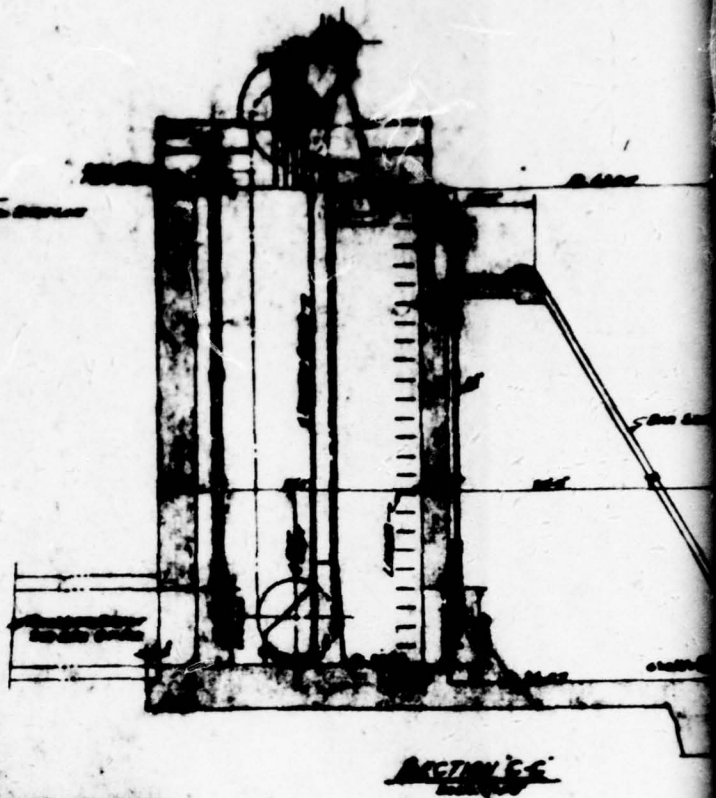
OCCOQUAN CREEK DAM
DETAILS
THE ALEXANDRIA WATER COMPANY
ALEXANDRIA, VA.

WATER WORKS SERVICE COMPANY, INC.
50 CHURCH STREET NEW YORK, N.Y.
SCALE: AS SHOWN USE DESIGNATED ONLY
DRAWN BY D.V.G. DATE: CHECKED BY R.G.
APPROVED JOB ED-A DES. BY

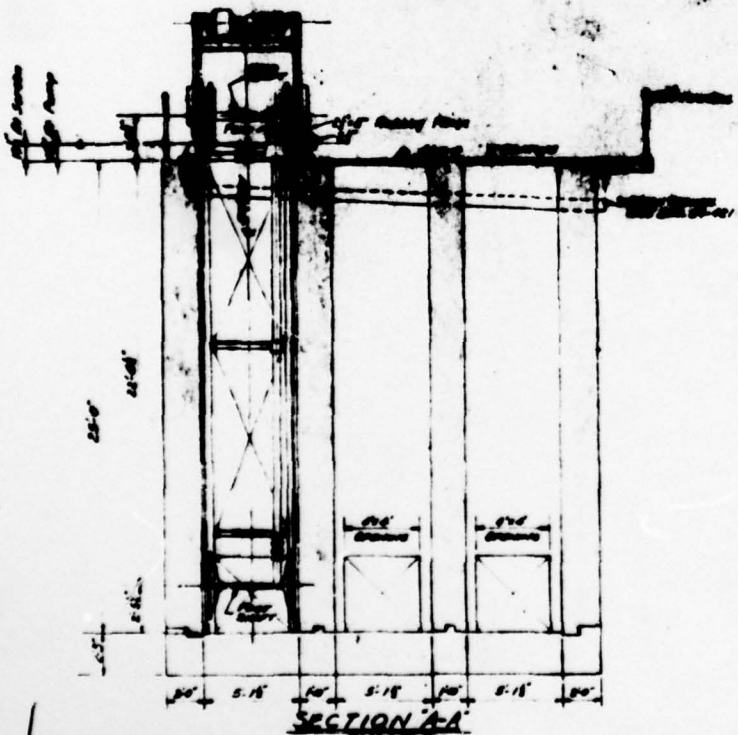
USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES

PLATE 9

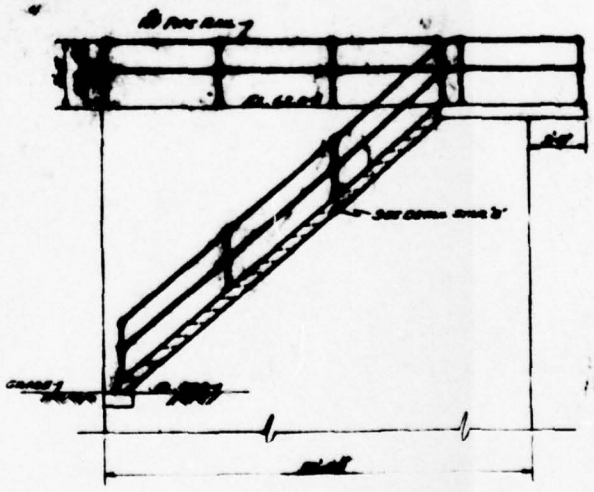




SECTION 'C-C'



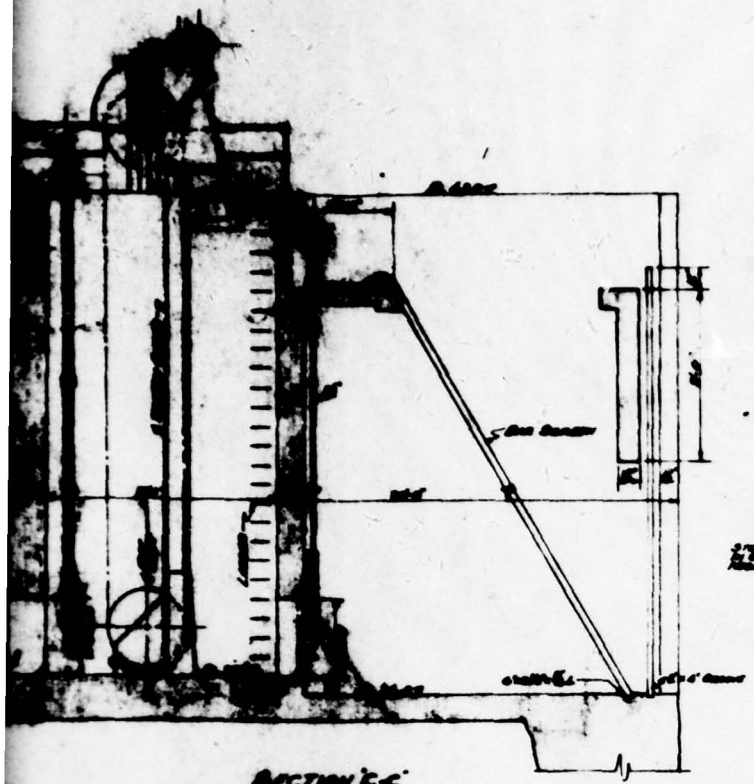
SECTION 'A-A'



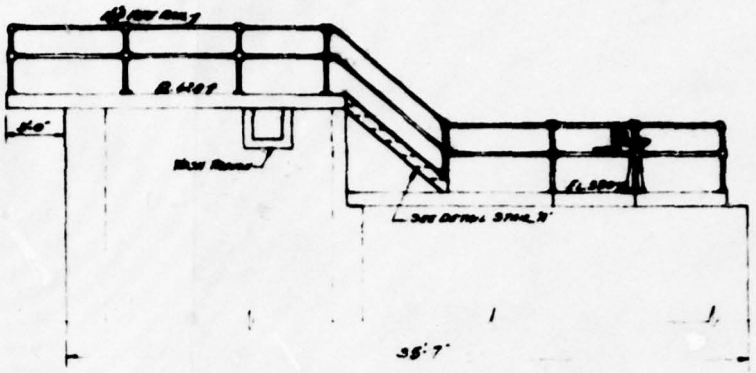
SECTION 'D-D'

REFERENCE DWG :-

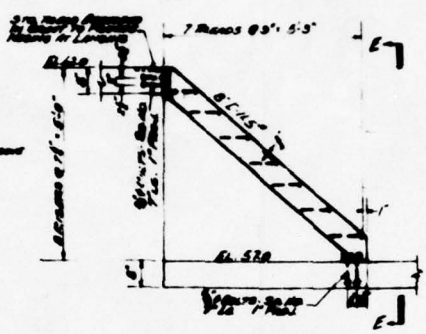
For additional plans, sections & details see dwg. #84-62



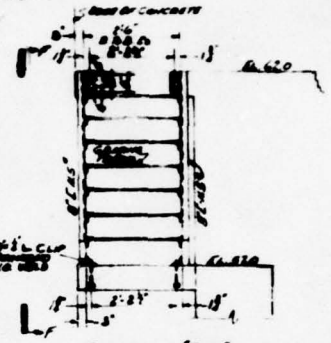
SECTION C-C



SECTION D-D

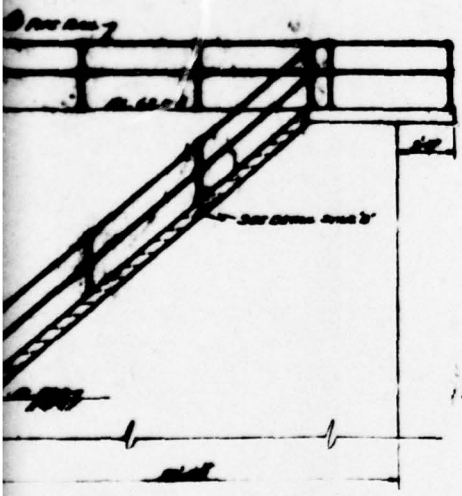


SECTION F-F

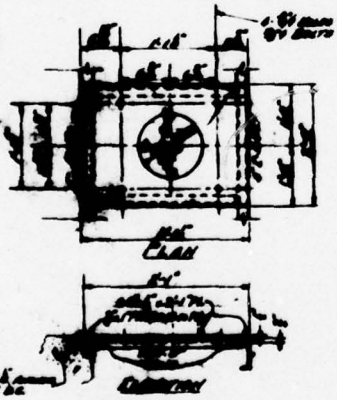


SECTION E-E

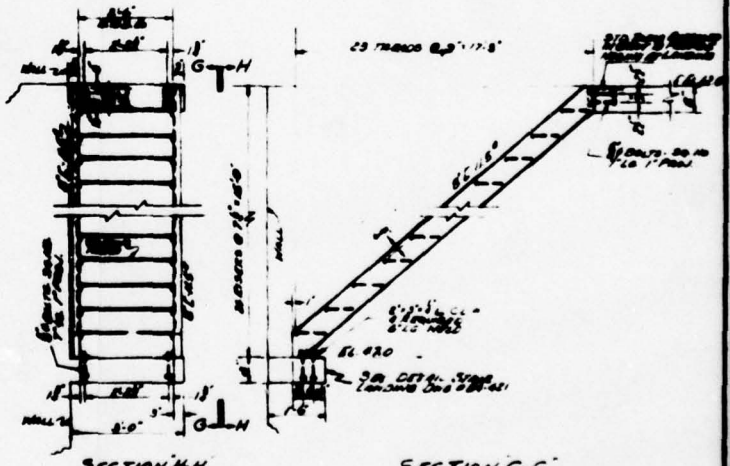
DETAIL STAIR A



SECTION B-B



DETAIL PUMP



SECTION H-H

SECTION G-G

DETAIL STAIR B

OCCOQUAN SUPPLY

RAW WATER INTAKE DETAILS

THE ALEXANDRIA WATER COMPANY ALEXANDRIA, VA.

WATER WORKS SERVICE COMPANY, INC.

84-42

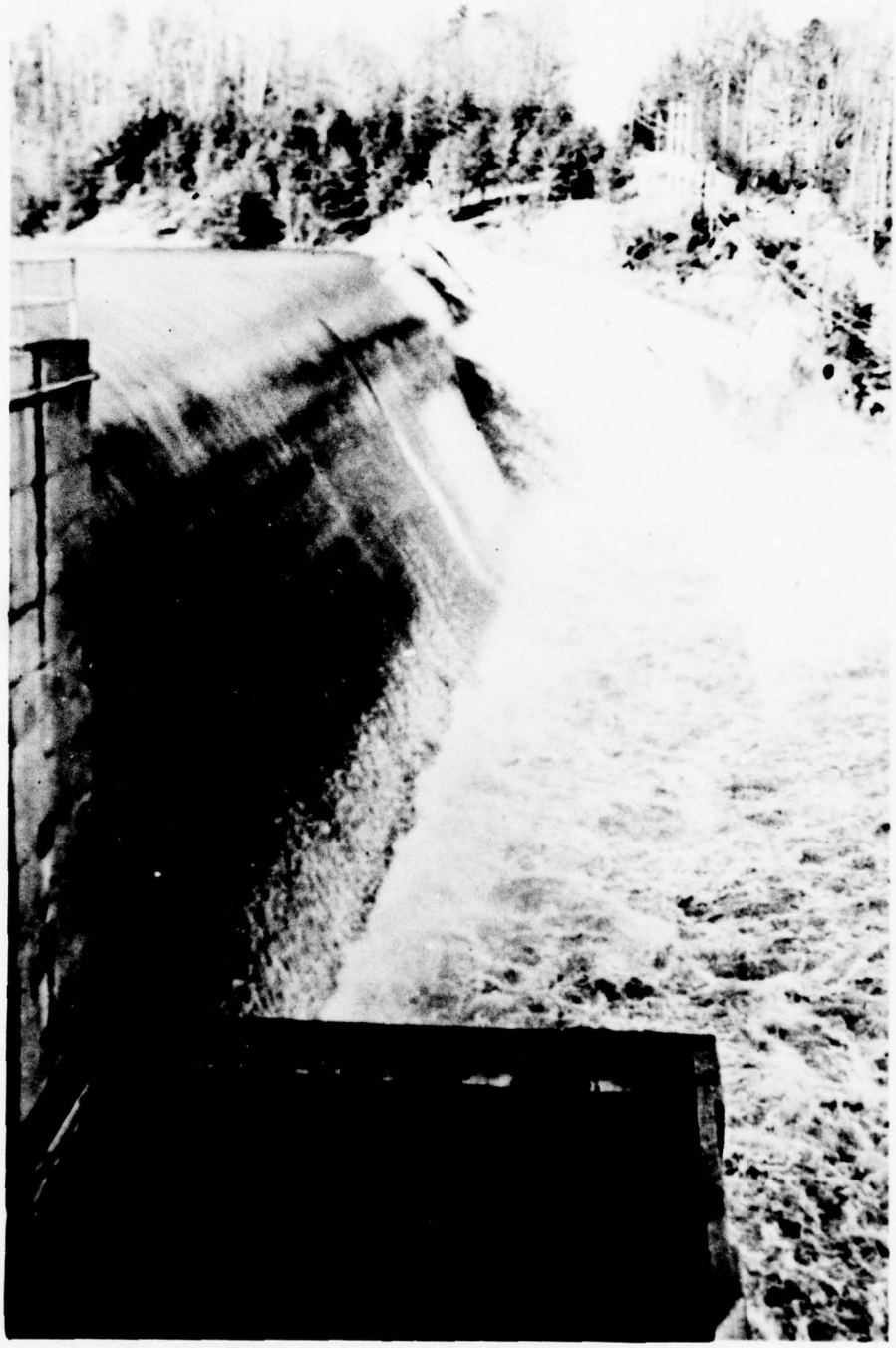
PLATE 2

RENCE DWG. -

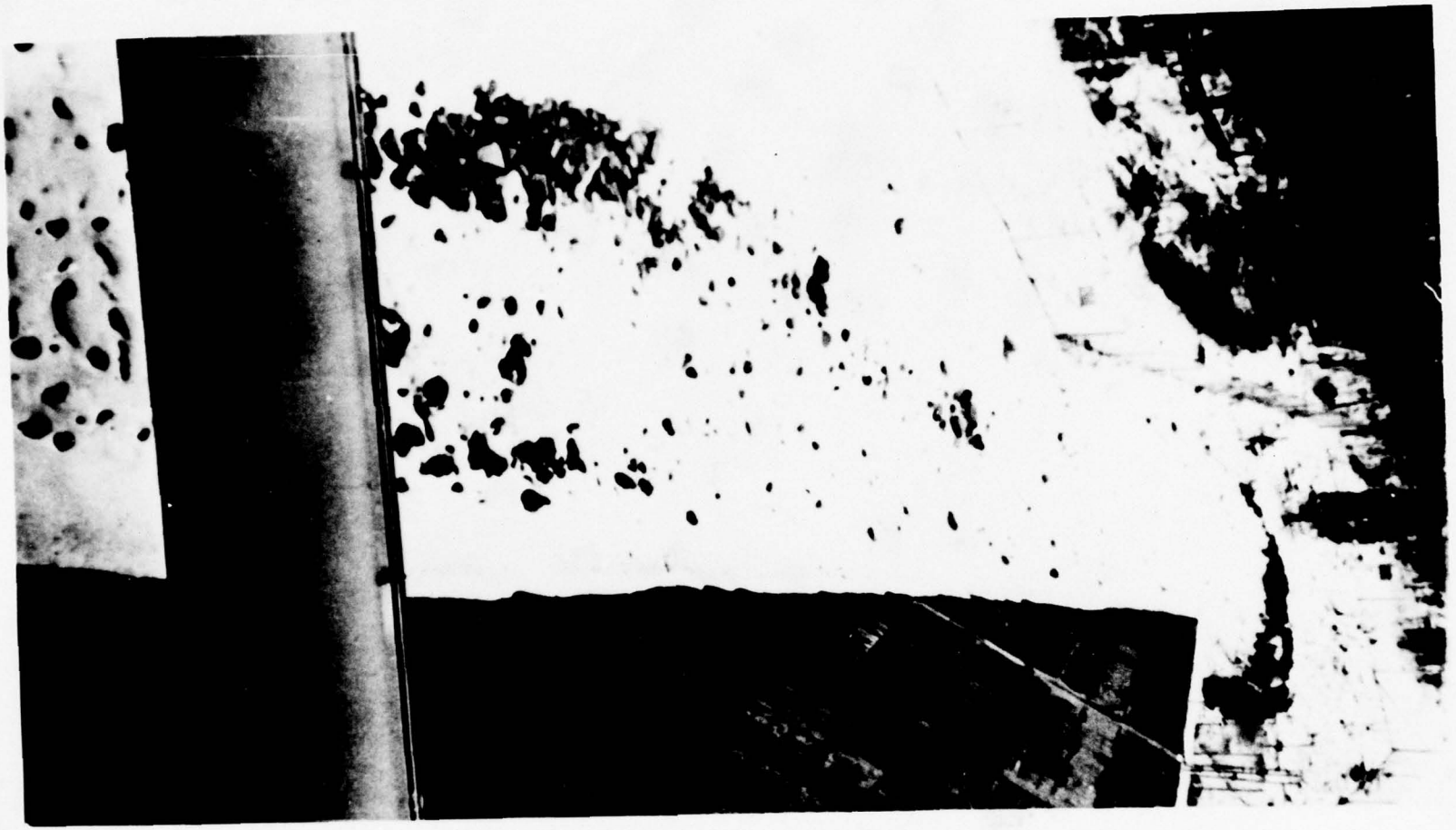
For additional plans, sections & details see dwg. #84-42

APPENDIX B - PHOTOGRAPHS

PLATE 1

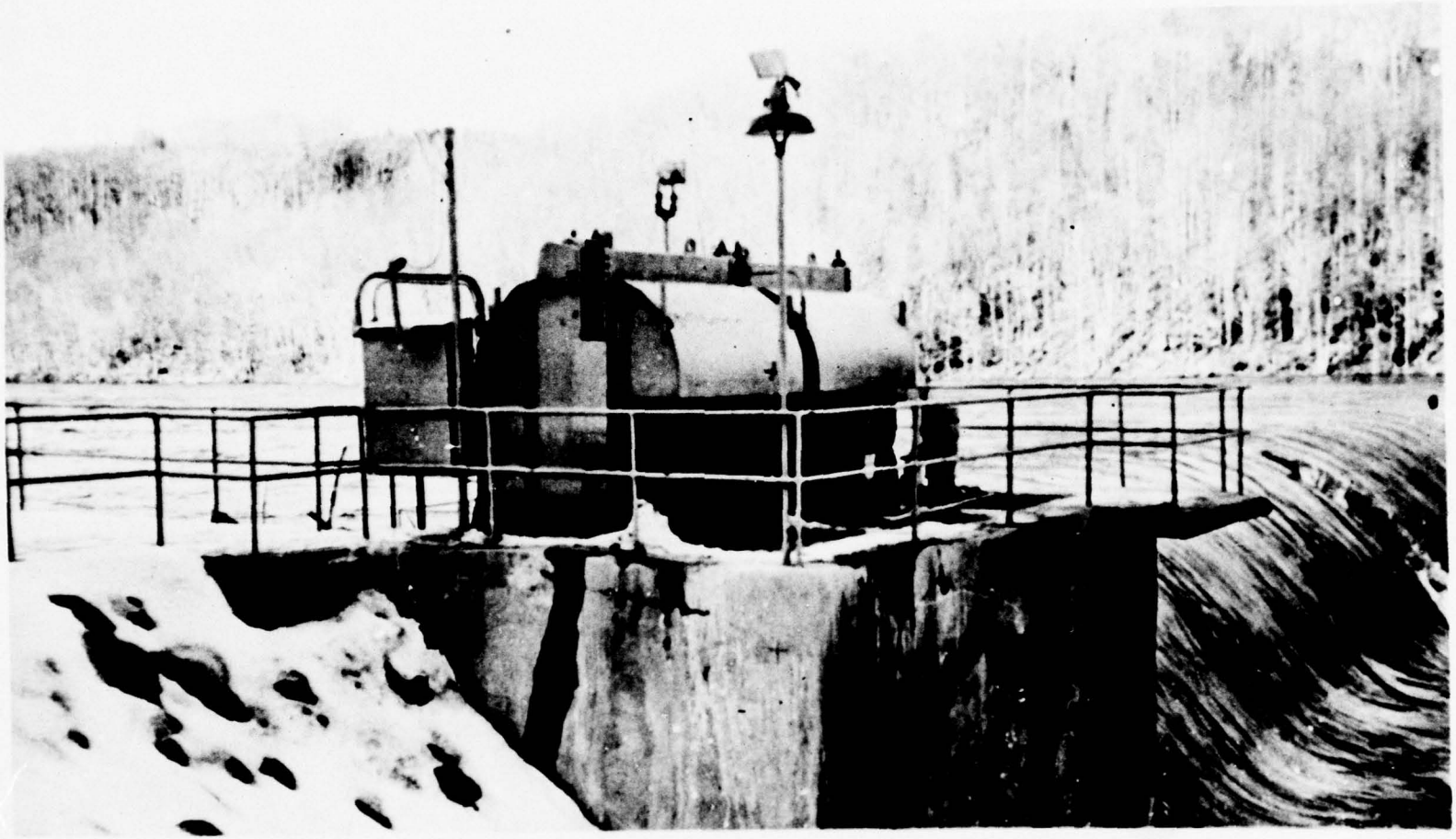


SPILLWAY AND LEFT ABUTMENT
UPPER DAM



DOWNSTREAM RIGHT ABUTMENT
UPPER DAM

PLATE 2



INTAKE STRUCTURE
LOWER DAM

PLATE 3



CHANNEL DOWNSTREAM OF
LOWER DAM

PLATE 4

APPENDIX C - FIELD OBSERVATIONS

APPENDIX C

FIELD OBSERVATIONS

The visual inspection was conducted 26 January 1978. The sky was clear and the temperature was around 40 degrees. There was no wind and the ground was covered with a blanket of snow. The river was swollen due to heavy rains during the past week.

UPPER DAM:

At the time of the inspection, 3:30 PM, 4.82 feet of water was flowing over the spillway (120 MSL). The flow eventually crested at 125.25 MSL at 11:00 PM that night. This was the second highest recorded flow since Tropical Storm Agnes in June 1972. Most of the dam was unobservable. The downstream face of the non-overflow section showed very little deterioration. There were no signs of calcium deposits on the face. There is one vertical joint running through the face at the contact between the non-overflow section and the intake structure. There is horizontal cracking along several visible lift joints, but no spalling. The right side of the powerhouse has vertical and horizontal jointing with calcium staining. The powerhouse was wet, but the water appeared to be runoff from melting snow.

A concrete retaining wall starts at the end of the dam and projects 45 degrees in front of the downstream face of the dam. This wall was installed after Tropical Storm Agnes to protect the weathered rock in the left abutment.

The heavy flow during the inspection was the second true test of the stability of the wall. The overflow induced a tremendous impact, however, there were no apparent signs of excess stress.

On the right side of the dam, the non-overflow section extends from the intake structure tying into the abutment. The contact was unobservable due to snow cover. The downstream contact was protected with grouted riprap.

LOWER DAM:

Water was flowing freely over the length of the lower dam. The high water was overtopping the trash screen in the intake structure at elevation 57 MSL. The left abutment could not be directly observed due to wet conditions and difficult access. Based on observation from the right abutment, the left end of the dam was barely visible due to the high flows. It appears to tie into rock. On the right side, the dam ties into the intake structure. A wing wall extends from the intake structure and ties into a riprapped abutment.

RESERVOIR AND DOWNSTREAM CHANNELS:

The area surrounding the reservoir is heavily wooded. The downstream channel on the left is an exposed rock bluff. The right side is riprapped approximately 1500 feet downstream of the lower dam to a pipebridge traversing the river (Appendix B - Photographs: Plate 4). Portions of the shoreline along the Town of Occoquan were flooding with up to 2 feet of water.

ATTENDEES:

Fairfax County Water Authority
- Floyd Eunpu
- Warren Hunt

State Water Control Board
- J. Roy Murphy

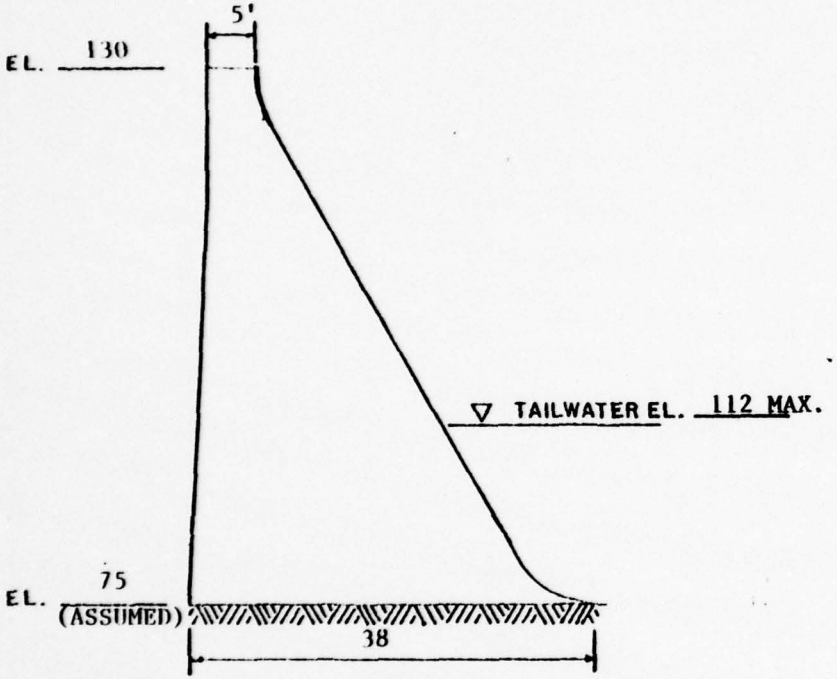
Corps of Engineers
- L. F. Baird
- K. R. Brooker
- M. L. Cheshire, Jr.
- J. C. Irving
- D. A. Pezza

APPENDIX D - STRUCTUAL CALCULATIONS

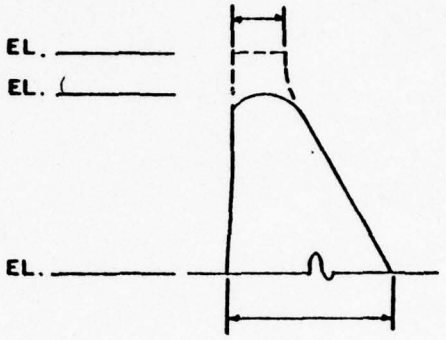
GRAVITY DAM DESIGN
STABILITY ANALYSIS

ANALYSIS DONE ON X FULL SECTION ___ PARTIAL SECTION
LOCATION OF SECTION NON OVERFLOW SECTION
ANALYSIS PREPARED BY U.S. ARMY CORPS OF ENGINEERS (IRVING)

	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
	NORMAL OPERATION	120	-	123.9	63.2	0.51	17.46	100	10.68	4.05	2.47
*	NORMAL OPERATION	120	-	150.8	63.2	0.42	18.82	100	10.93	4.08	3.86
	P.M.F.	142	112	54.1	92.9	1.72	OUTSIDE BASE	0	6.81	-	-
*	P.M.F.	142	112	115.8	92.9	0.8	7.73	61	7.21	9.99	-



FULL SECTION



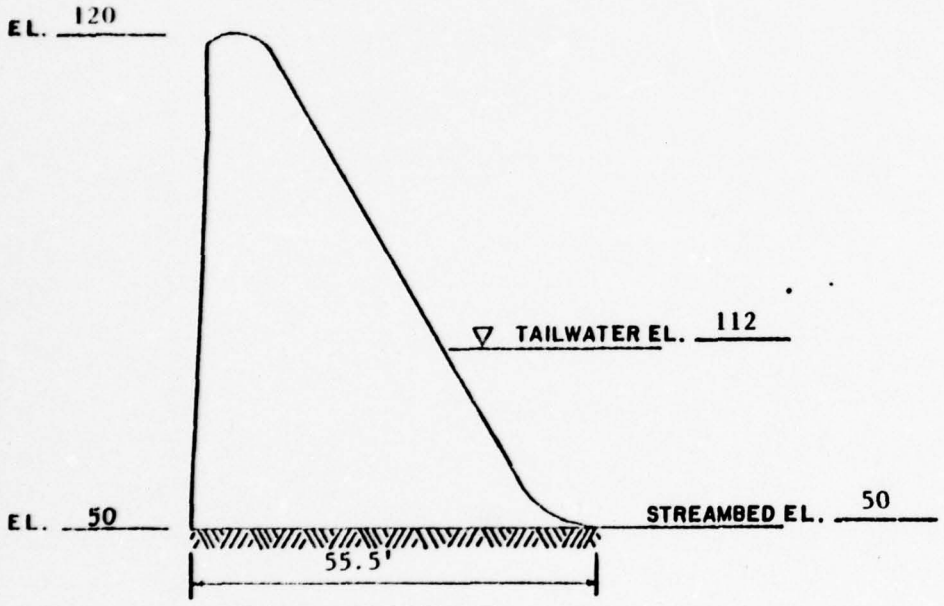
PARTIAL SECTION

GRAVITY DAM DESIGN
STABILITY ANALYSIS

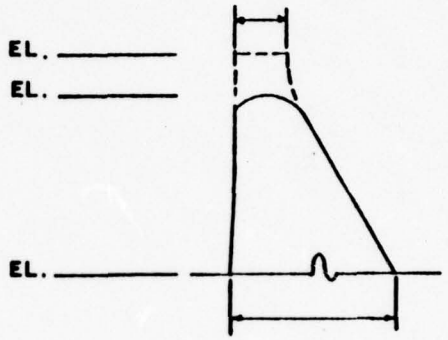
ANALYSIS DONE ON X FULL SECTION ___ PARTIAL SECTION
LOCATION OF SECTION FULL SPILLWAY
ANALYSIS PREPARED BY U.S. ARMY CORPS OF ENGINEERS (IRVING)

	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
	NORMAL OPERATION	120	–	233.^{K}	153^{K}	0.66	20.2'	100%	6.65	7.62	0.74
*	NORMAL OPERATION	120	–	288.2^{K}	153^{K}	0.53	23.1'	100%	6.86	7.8	2.58
	P.M.F.	142	112	87.6^{K}	130^{K}	1.48	4.68'	25%	7.15	12.48	–
*	P.M.F.	142	112	221.0^{K}	130^{K}	0.59	19.69'	100%	7.77	7.45	0.51

*WITH DRAINS WORKING.



FULL SECTION



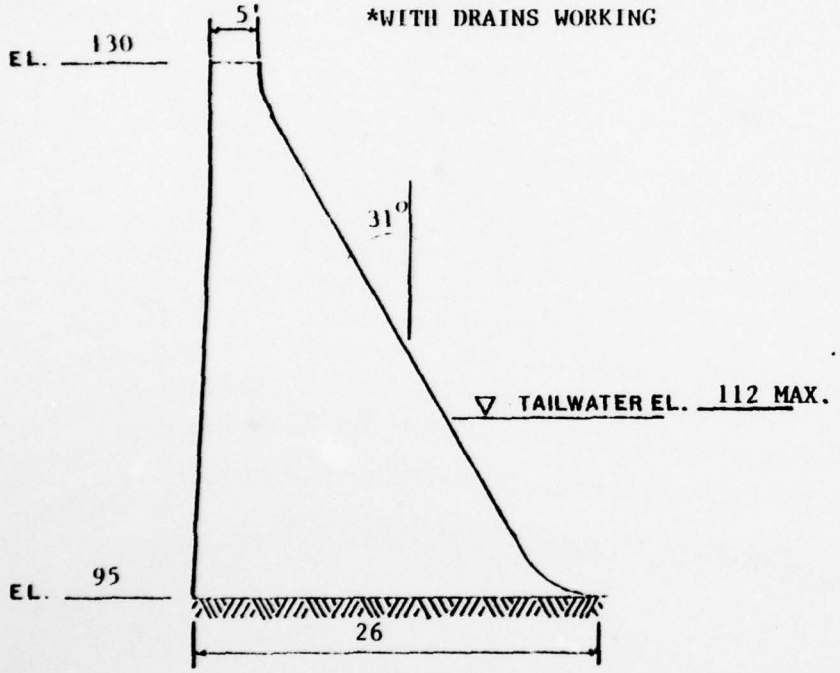
PARTIAL SECTION

GRAVITY DAM DESIGN
STABILITY ANALYSIS

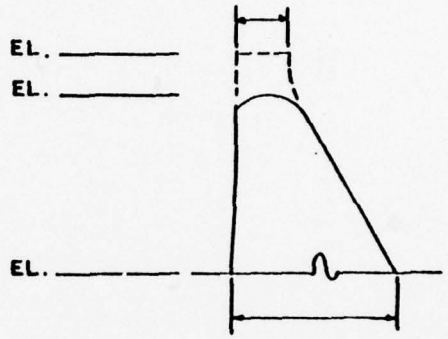
ANALYSIS DONE ON X FULL SECTION ___ PARTIAL SECTION
LOCATION OF SECTION NON OVERFLOW SECTION
ANALYSIS PREPARED BY U.S. ARMY CORPS OF ENGINEERS (IRVING)

	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
	NORMAL OPERATION	120	-	61.2	19.5	0.32	14.29	100	23.	1.65	3.05
*	P.M.F.	142	112	55.4	55.4	1	2.6	30	8.	14.2	-
	½ P.M.F.	135	91	48.9	49.1	1	3.8	45	9.	8.6	-
*	½ P.M.F.	135	91	65.4	49.1	0.75	7.2	83	9.	6.0	-

*WITH DRAINS WORKING



FULL SECTION



PARTIAL SECTION

APPENDIX E - REPORTS

REPORT - I

FAIRFAX COUNTY WATER AUTHORITY, VIRGINIA

REPORT ON

INSPECTION AND RAISING OF
THE OCCOQUAN DAM

BY

HARZA ENGINEERING COMPANY

APRIL 1975

HARZA ENGINEERING COMPANY CONSULTING ENGINEERS

April 5, 1973

Mr. James J. Corbalis, Jr.
Engineer-Director
Fairfax County Water Authority
4121 Chatelain Road
P.O. Box 91
Annandale, Virginia 22003

Subject: Upper Occoquan Dam

Dear Mr. Corbalis:

We are pleased to present our report on the raising of Upper Occoquan Dam. The draft of this report was completed and submitted for your review shortly before the great flood of June 1972. The size of the flood and the damage it caused to the dam foundations led to a reappraisal of some of the ideas and the recommendations of the report.

Subsequently the report was brought up-to-date and revised with the exception of the gate study. The results of the gate study which is a comparative study are still valid. The tainter gates remain the preferred type and the cost of different arrangements of gates (as to their number and size) does not differ significantly leaving the final choice to other considerations.

The 1972 flood disclosed the vulnerability of the foundation rock to scour at high flows. A few large gates with the resulting high flow concentration would require construction of an expensive, large stilling basin. For this reason we recommend now smaller gates occupying most of the crest length and distributing flood flow more evenly over the river channel width.

Scheme E with eight tainter gates, 46 ft wide and 7.5 ft high, is the recommended scheme. Its estimated construction cost is $1,920,000. This sum includes a 20% contingency which appears proper at this preliminary stage of design. The cost of engineering, construction supervision and owner's overhead was also included but not the interest during construction.

150 SOUTH WACKER DRIVE CHICAGO, ILLINOIS 60606
TEL (312) 855-7000 CABLE: HARZAENG CHICAGO TELEX 25-0340

Report on
Inspection and Raising of
the Occoquan Dam
in Virginia

Object of the Report

This report presents an investigation of the safety of Upper Occoquan Dam in Virginia and recommendations regarding its proposed raising. The dam is owned by the Fairfax County Water Authority. The Authority is considering increasing the height of the dam by five feet and raising the reservoir level by the same amount. Harza Engineering Company was authorized by the Authority on November 3, 1971 to: 1) inspect the dam in order to determine its present condition and 2) make a feasibility study of increasing the height of the dam and to prepare a construction cost estimate. After hurricane Agnes in June, 1972, we were also instructed to make a probable maximum flood study of Occoquan River.

Description and History of the Dam

The dam was built in 1955 by the Alexandria Water Company in Alexandria, Virginia. It was designed by the American Water Works Service Company in Philadelphia. The dam is a concrete gravity structure of 70-foot maximum height above foundation with the exception of the intake structure which is taller (Exhibit 1). The overall length of the structure is about 730 feet. Most of this length (520 feet) is represented by a free overflow weir. The crest of the weir and the normal reservoir level are at El. 120.0. The intake deck and the top of the south bank non-overflow section are at El. 130.0.

The intake structure serves the pipelines leading to the water treatment plant located less than a mile downstream. Three additional intakes were installed in 1966 in the right or south non-overflow section of the dam. A small powerhouse with two 350-kW generating units adjoins the intake structure on the downstream side. Power is generated only when there is excess water available. This is mostly between November and May.

In June 1972 a major flood (estimated at 75,000 cfs) spawned by the tropical storm Agnes went over the dam damaging its foundations. Subsequent to the hurricane the area was declared a disaster area and the Corps of Engineers, acting for the Office of Emergency Preparedness, took over the repairs to the dam. The Corps in turn, with the consent of the Fairfax County Water Authority, engaged Harza Engineering Company to inspect the damage and to engineer the necessary repairs. The first phase of this work, the repairs to the toe of the dam, are now underway.

The report on raising the dam was drafted and submitted in a preliminary form for your review shortly before the flood of 1972. The lesson of the flood led to revisions in the text and in the recommendations of the report. The revision was confined mainly to selecting a larger number of smaller gates over a few deeper gates for more uniform flow distribution and better energy dissipation.

Inspection of the Dam

The initial inspection of the dam took place on November 22, 1971 following a brief meeting in the Authority's office with Mr. James J. Corbalis, Jr., Engineer-Director of the Authority, and his staff. The inspection was made by Mr. Richard C. Acker, Geologist, and Mr. Andrew Eberhardt, Vice President and Chief Structural Engineer, both of Harza. They were accompanied by Messrs. Warren Hunt and Jerry Hasky of the Water Authority.

After the June 1972 flood Mr. Eberhardt of Harza accompanied by Mr. Peter Conroy, a Harza geologist, visited the site and recommended to the Corps of Engineers unwatering of the toe of the dam to permit inspection of the full extent of the damage. Subsequently a cofferdam was constructed according to a design prepared by Harza and the unwatering of the toe of the dam was accomplished on August 16, 1972. The representatives of the Corps of Engineers, of the Fairfax County Water Authority and of Harza Engineering Company witnessed the unwatering and inspected the damage. Harza's reports on both inspections are enclosed as Exhibit 17.

Foundations

The geologic map of Virginia indicates that the damsite and entire reservoir are underlain by granite gneiss of undetermined age (probably Paleozoyic or PreCambrian).

The bedrock exposed at and near the dam was observed to be a granite gneiss in which parallel alignment of dark minerals gives a streaky (gneissic) appearance. The rock was seen to be very hard and unweathered at all outcrops. The rock mass is thoroughly intersected by joints of which there appear to be three principal sets oriented as follows:

(A) Approximately normal to the stream valley, dip steeply upstream.

(B) Approximately parallel to the stream valley, dipping steeply riverward on the left bank and steeply into the right bank.

(C) Approximately normal to the stream valley, dipping gently (15° - 20°) downstream.

Randomly oriented joints also occur. Spacing of the joints in each set varies from less than 1-foot to several feet. The "C" set of joints above appears to have a somewhat wider average spacing than do sets "A" and "B". The joints are slightly open at the surface but apparently become tight at shallow depth judging from the very slight seepage noted in a nearby quarry excavated to well below river level and separated from the river by a rock wall less than one hundred feet thick.

One shear zone about 5 feet in width was noted. It is oriented parallel to the "A" set of joints and is well exposed on the left abutment about 15 feet downstream of the dam toe. The shear zone appeared tight and unweathered. Projected toward the south abutment, the shear would intersect the dam foundation at about the midpoint of the dam.

According to Mr. Jerry Hasky, who observed foundation preparation during construction of the dam, the contractor excavated to sound rock whose surface was carefully cleaned prior to concrete placement. A number of available pictures of the excavated foundation confirm this.

The foundation rock is competent to sustain not only the loads imposed by the existing structure but also the load of the 5-foot added height of structure. Before the 1972 flood there was some evidence of only limited scouring along the toe of the dam. The probings did not disclose any significant undermining (Exhibit 3).

The 1972 flood, however, which more than doubled the maximum flow of 35 year record, plucked out a large volume of rock at the toe of the dam and also from the cliffs of the north abutment. The survey made after unwatering (Exhibit 18) disclosed the maximum depth of scour to be 15 feet below the top of the bucket. There was, however, hardly any undermining of the concrete structure itself.

The proposed repairs, now underway under the direction of the Corps of Engineers, are aimed at protecting the toe of the dam against possible future undermining and at improving energy dissipation conditions below the dam. Accordingly the dam toe will be buttressed and protected with concrete extending downstream in the form of a short apron. All concrete will be well anchored into the rock. The deep stilling pools due by the flood will be retained to improve energy dissipation. In another area (in the middle of the dam) the rock will be excavated before the construction of the apron to provide the required tailwater depth.

The subsequent phases of repair work will include the north and south abutments and the intake wall. The proposed work in the north abutment will consist of enlarging (by rock removal) and paving with concrete the side channel or chute carved by the flood in the cliffs of the abutment. The work on the south abutment when erosion was limited to the shallow overburden will be much more limited. The condition of the intake wall is still to be investigated at this stage. (see "Intake Wall," p.7).

Drainage

As shown on the available construction drawings, the dam has a well-designed seepage control system consisting of a grout curtain, a gravel drain which runs the length of the dam and 8-in. diameter wells at the vertical construction (contraction) joints between dam monoliths or every 50 feet.

The grout curtain at the heel of the dam consists of two rows of grout holes extending to the maximum depth of 14 feet. The drain is a 3 ft x 3 ft trench cut in rock 8 feet from the face of the dam and filled with crushed stone. A curtain of drain holes extends from the trench down into rock to half depth of the grout holes. The 8-in. wells formed in the monolith joints discharge directly into the 3' x 3' drain. The latter discharges to the tailwater through 6" cast iron drain pipes. There are two such pipes in the weir section of the dam and one in the non-overflow section.

As the drain outlets were submerged during the original inspection in 1971, it was not possible to observe their functioning. For this reason it was recommended to the Authority that the existing 8-in. diameter vertical wells be used to check on the drains and the existing uplift pressure (Harza letter of December 7, 1971).

Accordingly, a local contractor was hired to drill holes through the five feet of concrete in the crest of the dam to gain access to the vertical wells. Four holes were drilled, one at each alternate contraction joint between dam monoliths, beginning with the joint located 50 feet from the intake structure (Test hole No. 1 - Exhibit 2).

The dye placed in holes No. 1 and 3 appeared at the downstream toe within 5 minutes. Hole No. 2 missed the well below completely. The last well (Hole No. 4) was found blocked. After it was cleaned with compressed air and water, the dye was placed in the hole (Exhibit 3). A considerable dye quantity appeared at the toe of the dam but some dye also appeared at various levels at the contraction joint in the dam. Air bubbles were also evident at the face of the dam. Both results indicated leakage past the rubber waterstops placed on two sides of the well in the contraction joint. The water level in the well was about 11 feet below the reservoir level.

These findings show that some of the vertical wells may not be operative or only partially operative, possibly due to damage or accidental plugging during construction. Others, however, are open and the

dye tests indicate that the foundation trench drain is not plugged. Reaming and cleaning of plugged wells can be accomplished fairly easily during the proposed raising of the dam. Where leakage past the waterstops occurs, grouting of the contraction joints and redrilling of the well could be considered. Proper functioning of the wells, however, is of relatively little importance in a dam with well constructed and tight lift joints. The foundation trench drain has generous proportions (3 x 3 ft) and, in view of the type of rock, is not expected to ever plug up.

After the dam toe was unwatered in 1972, it was possible to observe seepage firsthand. Both discharge pipes from the foundation trench drain were flowing even if the south pipe was flowing very little. There was no trace of seepage at the concrete-to-rock contact and the rock face exposed underneath the bucket by scour was dry. It was concluded again that the trench drain was functioning well.

Silting

Soundings made 3 years ago disclosed only several feet of silt at the upstream face of the dam. Considerable silting exists at an old dam located approximately 6 miles upstream and submerged by the reservoir.

Concrete

The concrete structures are generally in excellent shape. There are very few places where traces of seepage could be observed. In the non-overflow section on the south abutment there are a few small white spots of calcium carbonate indicating spots where a little leakage occurred in the past. There seemed to be one wet spot below the lowest of the three intake pipes installed in 1966. Similar white deposits are visible at the horizontal construction joints in the south (or right) wall of the intake structure. However, there is no leakage at present and the wall looks dry. No cracks could be seen in the top wall (El. 130) of the intake structure. This was quite significant

because there are some rectangular openings in the intake deck covered with gratings. Normally cracks are likely to appear from the corners of such openings.

The spillway concrete is obviously of good quality. The cores obtained from drilling in the crest of the dam confirmed this observation even if the mix appeared oversanded. Two samples tested afterward in a laboratory showed compressive strength of 7202 and 8436 psi (Exhibit 4). Slight roughening of the concrete is visible at the waterline on the upstream side. The dam is also quite free of cracks. Walking its full length on the downstream side, only two vertical localized and narrow cracks could be observed.

The downstream face was wet because of the spray carried over the dam by a very strong wind. Wetting of the surface could possibly conceal some minor seeps which would be visible on a dry surface. On the other hand, if seepage through the joints was present, one would expect to see deterioration of concrete caused by freezing or thawing. There was no indication of any damage whatsoever to the concrete surface. Some minor seeps could be observed only near the north abutment. The very toe of the dam was submerged by a narrow pond of water contained between the concrete and the rock outcrops downstream. Soundings indicated that there was no damage to the concrete or any undermining.

After unwatering in 1972 all of the concrete toe could be inspected in the dry. No damage was found to the concrete even if the damage to the rock foundation and the north abutment was extensive.

Intake Wall

The upstream bulkhead wall of the powerhouse which is 6-feet thick displays some evidence of leakage. The lower portion of the wall inside the powerhouse could not be examined because it has been covered with corrugated metal panels. The panels were installed by the former owner probably to cover up seepage and improve the appearance of the interior of the powerhouse. Some water was standing in the gutter extending along the bottom of the wall but no flow could be observed.

There have also been some leaks in the upper portion of the wall which extends above the powerhouse roof and is accessible from the roof. The leaks have been patched up with epoxy and very little or no water could be observed on the wall. The wall has been painted over. Several rusty streaks appear on it, all close together. Some irregularities on the concrete surface are also visible, as if a poor vertical construction joint or a cold joint had developed in the concrete.

The indication of some trouble in the powerhouse wall seemed at odds with the appearance of the rest of the project. Its concrete in general is practically crack-free.

An examination of the drawings gave a possible clue; the wall is reinforced on the upstream side only. There is no steel in the downstream face (if the drawings are corect). Consequently the wall acting as a horizontal beam was likely to develop vertical cracks, hence, the seepage.

Our figures indicate that the wall is not in immediate danger. Even if it cracked, it could still act as two horizontal cantilevers capable of taking the load. The proposed raising the dam 5 feet will increase the stresses in the wall but not very significantly. However, the absence of steel in one face causes the concrete to be in tension. Such design is defective by accepted engineering standards. We recommend that the wall be inspected closely. To this end the paneling and paint should be removed over a width of at least 16 feet (or the wall horizontal span), and the concrete cleaned and examined. Cleaning is best accomplished by sand blasting, but this method is not suitable inside the powerhouse because of the presence of machinery. Consequently other methods will have to be used, such as: power brushed, bush hammering or chipping. Spraying with water could be used to prevent dust. Possibly a rubber or plastic hose could be attached to a power brush to discharge water directly where the concrete is being cleaned.

One or more 3-inch (or so) deep vertical slots about 2 feet long should be cut in the wall to check whether there is any steel in the downstream face of the wall. Core drilling may be used to ascertain the depth of cracks.

After the wall is examined thoroughly, the proper remedial action, if any, can be decided.

The excerpts from the condemnation proceedings for the dam received at a later date confirm our own observations regarding the seepage through the east wall of the intake chamber. They mention a vertical crack in "the wall of the intake structure," which we assume is the east wall.

Raising the Dam

General

The Fairfax County Water Authority proposes to increase the reservoir capacity by raising the dam five feet. Raising the normal pool level, however, has to be accomplished without increasing present flood levels. The Authority owns flood easement rights up to El. 130 at the dam and to slightly higher elevations upstream. The present extent of flooding adjacent lands during high river flows cannot be exceeded.

According to a study made in 1957 by Edward S. Holland, professional engineer, a flow of 62,200 cfs over the dam corresponds to headwater elevation of 130.8. Holland established that this discharge corresponds to a 100-year storm for ultimate development of the watershed.

Our calculations indicate that headwater El. 130.8 would result in a discharge over the existing dam of about 72,000 cfs which would indicate a somewhat less frequent occurrence for this headwater elevation or a lower elevation for 62,200 cfs discharge.

By installing regulating gates of adequate capacity, the dam can be raised without exceeding the present backwater levels during high river flows. The discharge of 72,000 cfs with headwater at El. 130.8 was used to determine the required gate size and number.

A flood peak inflow of 226,000 cfs was arrived at based on the probable maximum storm and existing conditions (Appendix A). Future urbanization will result in a higher flood peak. The increase, however, will be moderated by the suburban, low density type development expected for this area. For the purpose of checking the dam stability, a discharge of 150,000 cfs (or about 2/3 of the probable maximum flood) was considered adequate. The same discharge was used in the recommended scheme for raising the dam (Scheme E) to determine the required clearance under opened spillway gates and their trunnion setting.

Stability Analysis

Stability of the dam was studied to determine whether a five-foot increase in height would require any additional stabilizing measures. The studies included both the dam as is and the dam raised five feet.

Stability analysis was made for two assumptions regarding the foundation drain performance:

Case 1. - Drain functioning
Case 2. - Drain plugged

The second condition, as pointed out earlier, is not expected to ever occur. Nevertheless it was considered to provide more insight into the degree of stability of the dam.

Case 1. The uplift pressures were assumed to be distributed as shown on Exhibit 5. The full reservoir head acts at the heel of the dam. At the drain line the head is reduced to one-fourth of the full head on the dam or to one-fourth of the difference between the headwater and tailwater. The tailwater depth is significant only at flood discharges. From the drain line the uplift pressure decreases along a straight line to the tailwater level or to zero as the case may be.

Case 2. Both the effect of drains and of the grout curtain were disregarded. The uplift pressure diagram represents a straight line variation from full headwater at the upstream face of the dam to tailwater (or zero) at the toe of the dam.

For the dam raised five feet, stability studies included both the free overflow section and the proposed gate section (to be discussed later on).

The results listed in Exhibit 5 show that the dam as built has a high degree of stability. Even with the drains completely inoperative, the resultant force falls within the middle third of the base. This indicates that all of the base is in compression and no theoretical tensions develop even under the most adverse assumptions.

This reserve of stability permits raising the dam without increasing its base width. With the drains functioning and the reservoir level at El. 125.0 (the new proposed level) still no tensions develop at the base of the dam whether in the weir or gate section. It is only when both the drainage system and the grout curtain are assumed fully ineffective that the resultant moves out of the middle third. However, theoretical maximum compressive stress under the toe of the dam does not exceed 10 ksf and the structure remains stable.

Under flood condition, headwater elevation 130.8 (flow of about 72,000 cfs), the dam is also stable. Only a small reduction in the uplift pressure produced by the drains or the grout curtain, or both would be required to keep the resultant within the middle third, eliminating all theoretical tensions on the base.

At 150,000 cfs discharge (2/3 of probable maximum flow), the headwater rises to about elevation 136.0. At this stage 6 feet of water overtops the intake deck but the dam remains stable with the resultant well within the base.

Shear friction factor of safety against sliding is well in excess of the required minimum as is usually the case on hard rock foundations.

Types of Gates

Three basic types of crest gates could be considered for Occoquan Dam: vertical wheel gates, flap gates and tainter gates.

Vertical wheel gates sometimes used in spillways require heavy tracks for their wheels and tall superstructures to guide and support the gates when open. Their wheel assemblies require a number of machined parts and some maintenance. The cost of such gates with appurtenances is higher than that of tainter gates. In addition the gate openings at Occoquan have to be fairly wide and shallow in order to avoid excessive cutting of the existing crest. Such proportions are not the most suitable for wheel gates.

Flap gates can be built low and long. For this reason they have been used often on long weirs with small heads and also for raising existing dams. They do not require any superstructure or bridges and need few, if any, piers. Since they are overflow gates, they are particularly suitable where ice or trash have to be released periodically over the dam.

An example of a successful flap gate installation is at Decatur Dam owned by the City of Decatur, Illinois. The project designed by Harza Engineering Company in 1954 consisted of installing two gates, each about 233 feet long and 5 feet high on top of an existing dam and making the necessary structural alterations. The gates were furnished by S. Morgan Smith Company (later acquired by Allis-Chalmers) and were of the type known as "Bascule" gates.

Tainter gates, very popular in the U.S., are of rugged and simple construction. They are economical, dependable and require little or no maintenance other than occasional painting.

In view of the above, only flap gates and tainter gates were considered in this report.*)

Gate Sizes and Arrangement (Initial Study)

In addition to the selection of the type of gates, it was necessary to explore the effect of the gate size and number on the cost of the

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*) See also Exhibit 6.

project. As the reservoir cannot be lowered below the existing crest for the purpose of construction (except three to four feet in summer), it was necessary to consider the effect of the gate size and number not only on the volume of concrete to be removed but also on the cost of cofferdamming.

The problem was bracketed by studying three possible tainter gate sizes and arrangements and an arrangement utilizing flap gates:

Scheme A - Few deep gates occupying only a portion of the crest length: 3 gates, 46 ft wide by 15 ft high (Exhibit 7).

Scheme B - Shallow gates occupying nearly the full length of the crest: 9 gates, 47 ft wide and 6 ft high (Exhibit 8).

Scheme C - Eight shallow gates, 47 ft wide and 5 ft high and one deep gate 46 ft x 15 ft. In this scheme the reduced height of the gates does away with the need and expense of cutting down the crest of the dam. The reduced gate capacity, however, has to be made up by one deep gate (Exhibit 9).

Scheme D - Three flap (or Pelican) gates each 171 ft long and 6 ft deep (Exhibit 10).

A flood routing study was made to check the effect of the gate operation on the reservoir elevations. The shape of the flood hydrograph was developed from the existing flow records furnished by the U.S. Geological Survey in Richmond, Virginia. It was assumed that the gates would automatically maintain constant pool elevation at El. 125 until fully opened.

At this point the reservoir would begin to rise. It was found that the 72,000 cfs maximum discharge at the dam corresponded to a peak inflow of 78,400 cfs. The surcharge of the reservoir could accommodate the peak inflow of this large flood without exceeding the 130.8 stage at the dam. Only in the early stages of the flood would the reservoir levels be somewhat higher than those for the existing dam. In order to arrive at the construction cost of the project it was necessary to consider how the work will be actually carried out. The problems of access, cofferdamming and river diversion had to be studied as they appeared to affect the project cost materially.

Since all of the proposed work is to be carried out on the crest of the dam (or at the height of up to 70 feet above the riverbed), the first thought was to use construction barges or pontoons. Such barges would be stationed along the crest of the dam, carrying cranes, materials and supplies. It was found, however, that there is lack of good access to the reservoir within a reasonable distance from the dam. In addition there would be danger of having the barges and equipment swept over the dam by a flood.

Carrying out the work from the intake deck was also considered. This would require building an access road to the top of the dam along the south or right abutment. The top of the south non-overflow section would have to be widened with a temporary deck. It was concluded that even if the cost of these measures were reasonable, the scheme would not be practical. A substantial and fairly wide bridge would be required for the full length of the dam and all concrete work would have to be carried from the end of the bridge as its construction advanced. This would likely create a bottleneck resulting in slow and costly work.

Accordingly the construction scheme was based on carrying out all work from the downstream side of the dam. A work trestle and fill would be built along the toe of the dam to carry a construction crane capable of reaching the crest of the dam (Exhibit 11). All equipment and materials would be brought in on trucks using the powerhouse access road. A storage area is available nearby along the access road. This construction scheme was used in estimating the cost of all gate arrangements studied.

Scheme A

Scheme A calls for constructing only three tainter gates (Exhibit 7). In order to provide the required discharge capacity, the gates have to be 15 feet deep. This leads to removing the ten top feet of the existing dam. The work, however, is concentrated within a limited area

which simplifies cofferdamming problems and only a short bridge is required to provide the necessary access for hoist maintenance or manual operation from the intake deck.

Fewer gates than three would require removing more than 10 feet of concrete. This would increase the cost of cofferdamming. In addition, if only one or two gates were provided the consequences of a hoist failure and inability to open the gate during a flood would be more serious.

The removal of concrete to create gate openings in the dam will have to be done carefully in order not to injure the adjoining concrete which is to remain in place. Based on experience with similar jobs, it is proposed to separate the concrete to be removed by drilling closely spaced vertical holes (Exhibit 11). The holes would be 3 1/2 inches in diameter placed 6 inches on centers. After these holes are drilled to the desired depth (about 10 feet maximum) the concrete between the holes would be removed by drilling overlapping holes in between the original holes. The latter would be used to guide the drill by having a 3 inch pipe welded to the drill frame. In this manner slots about 3 inches wide would be created. Then the concrete between the two slots could be removed by light blasting.

This method was used to arrive at the construction cost estimate. It is conceivable of course that a different method of concrete removal would be used in the actual construction.

The remaining length of the dam (outside of the gate section) requires only a limited amount of concrete work. The crest is raised 5 feet by building a concrete extension on it. The new concrete will be anchored to the existing concrete with reinforcing bars grouted in drilled holes. The surface of the crest would be properly roughened to assure good bond. The use of epoxies for bonding should be considered in the final design. This portion of the dam will continue to function as a free overflow weir.

The gates would be operated with electric motor hoist and stainless steel cables or sling chains. Rubber seals would be provided to insure watertightness. Automatic gate controls would maintain constant pool level. If desired, to reduce the wear and maintenance of the hoists, smaller river flows could be allowed to go uncontrolled over the weir portion of the dam. In this case the gates would not open until the reservoir level rose a foot or so. Flood routing computations indicated that this mode of operation should be permissible. The critical 130.8 stage at the dam would not be exceeded unless the reservoir was allowed to rise considerably more than one foot before the gates started to open.

A concrete paving slab will be placed downstream of the dam opposite the gate bays. The slab will provide protection against possible scour caused by greater flow concentration.

Removal of the top ten feet in the dam crest requires a cofferdam. The reservoir level, as noted earlier, cannot be drawn down more than 4 feet or to El. 116. At times of high river flows, the crest is overtopped. This can occur at any time of the year.

Due to the height of the dam, a cofferdam founded on the reservoir bottom would be much too costly. Any reasonably priced cofferdam has to be supported on the dam itself. It also must consist of elements light enough to be erected by the construction crane moving along the downstream trestle and fill.

The cofferdamming cost was estimated on this basis. The scheme calls for building two piers first. Each pier would be built behind a semicircular steel cofferdam supported on steel brackets resting on and anchored to the crest of the dam (Exhibit 11). The cofferdams would be sealed with rubber strips bearing on concrete. Concrete removal and new concrete construction would proceed behind the two cofferdams. After the piers are constructed, the two cofferdams would be removed and steel stop logs placed between the piers. Removal of concrete and construction of a new ogee would then be carried out behind the stop logs. The two cofferdams and the stop logs would be reused in the construction of the two remaining piers and ogees.

During this stage of work flashboards are erected over the gate section of the project and the river is diverted over the left or north portion of the dam. After the gate section is completed, the river is diverted through it and the concrete work on the north portion of the dam begins. For this stage of work the work trestle is extended by placing a fill downstream of the dam.

The overall estimated construction cost of Scheme A including engineering and the owner's overhead is $1, 217,000 (Exhibit 13).

Scheme B

In Scheme B the gates cover nearly the full length of the crest with the exception of two short end bays (Exhibit 8). The end bays are a fixed free overflow weir. The top of the weir is at the future reservoir Elev. 125.0. The dam is raised to this elevation by simply adding concrete on top of the existing crest.

There is one gate for each dam monolith. Each gate is centered on the contraction joint. The width of the gate opening is the length of the monolith minus the pier thickness (3 feet). The gates are operated with electrical hoists mounted on piers. In view of the relatively small size and weight of the gate, a single hoist is proposed for each pair of gates. The odd or ninth gate, however, will require a separate hoist.

A service bridge is required for the full length of the dam to provide access to all hoists. The bridge will connect to the intake deck at El. 130.

A part of the original crest will be occupied now by the gate piers. In addition the end fixed weir bays will have less flood discharge capacity than the corresponding length of the original lower crest. For this reason it was necessary to increase the depth of gates to six feet. This in turn requires lowering the existing crest from El. 120 to El. 119.0. As a result, removal of concrete and resurfacing

of the crest over nearly the full length of the dam is necessary.

The gate (or bridge) piers and the gates have been set back or downstream of the high point of the crest. This will permit erecting a rather simple cofferdam on top of the crest. The cofferdam will consist of steel pins or pipes inserted into the existing sockets in the crest and of timbers laid across the pins.

The crest, however, will have to be cut and resurfaced also upstream of the pins. Consequently the work on the crest will have to be done in two stages: first behind the cofferdam and second, upstream of the cofferdam when the reservoir is drawn down in summer months. The work behind the cofferdam will also have to be carried out in two stages: first over one half of the dam length, then over the remaining half.

The concrete in the crest surface will be removed to at least 8 inches below the new lowered profile. The new concrete surface will be tied to the existing concrete with steel anchors placed at about 2 feet o.c. each way and grouted in drilled holes. The use of epoxy as a bonding agent will be considered. The piers will be anchored similarly with larger size reinforcing bars grouted in the old concrete.

The deck construction was assumed (for the purpose of this study) to consist of precast, prestressed beams and a cast-in-place deck. The construction work will be carried out with the help of a crane moving along a work trestle and fill as in Scheme A.

The overall estimated cost of Scheme B including engineering and owner's overhead is $1,152,000 (Exhibit 13).

Scheme C

In Scheme B the crest of the dam has to be lowered one foot in order to accommodate 6 foot high gates (Exhibit 9). Concrete removal and reshaping of the ogee has to be carried out nearly the full length of the dam. In addition the work at each point has to be accomplished

in two stages: behind a flashboard type cofferdam and also upstream of it when the reservoir is below its normal full level.

The object of Scheme C is to eliminate this work by using lower gates (only 5 feet high) which would not require crest modifications except for pier construction. The reduced gate discharge capacity, however, has to be made up somewhere. This is accomplished by installing one deep gate of the same size as the gates proposed in Scheme A. There will be 8 gates 47 ft wide by 5 ft high and one gate 46 ft wide and 15 ft high. An access bridge will extend the full length of the dam as in Scheme B.

Work would be carried out behind a flashboard type cofferdam erected on the crest of the dam as in Scheme B. The work on the deep gate bay will require the same type of cofferdams as proposed in Scheme A. The work trestle and fill will be the same as in the preceding scheme.

The overall estimated construction cost of Scheme C including engineering and owner's overhead is $1,210,000 (Exhibit 13).

Scheme D

Allis Chalmers Company, a designer and manufacturer of flap gates sold under the trademark of "Pelican", suggested several different arrangements of such gates for Occoquan Dam and furnished preliminary cost estimates (Exhibit 12).

The least expensive set of Pelican gates proposed consists of three gates each 43 ft wide and 16 ft high. The estimated cost of these gates F.O.B. job site is $390,000.

The second in cost scheme consists of two gates each 250 ft long and 5 ft high at the price of $440,000. In this scheme each gate has three hydraulic operators and the gates seal against each other eliminating the center pier. Their total discharge capacity, however, is not adequate.

In this respect the next in price scheme: 2 gates 250 ft x 6 ft is more satisfactory. Its estimated cost is $508,000.

In all schemes Allis Chalmers proposed placing hydraulic cylinder operators behind the gates and eliminating the bridge over the spillway. In this arrangement the access to the operators is limited to the space behind a closed gate.

Based on this information three flap gates each 171 ft long and 6 ft high were selected (Exhibit 10). The gates are separated by piers to permit isolating individual gates for maintenance or repair. Such a feature is considered very desirable. The overall discharge capacity of the gates equals that of the tainter gate schemes. Each flap (or Pelican) gate is operated by two hydraulic cylinders placed, following Allis-Chalmers recommendation, behind the gate. The cylinders are placed inside pits excavated in the concrete mass of the dam. Also following Allis-Chalmers recommendations, the spillway bridge has been eliminated. Its usefulness would be quite limited as it does not provide direct access to the hoist pits.

The cost of the gates used in the estimate was that furnished by Allis-Chalmers for two 250 ft long gates 6 ft high. The overall length of the gates, however, was increased slightly (to 3 x 171 ft = 513 ft) to provide the same discharge capacity as that of the existing spillway and of the tainter gate schemes.

The deep flap gate scheme (three gates, each 43 ft wide x 16 ft high) was considered less desirable because it calls for a single hydraulic operator for each gate. In case of its failure the gate could open fully and a good portion of the water stored behind the dam would be lost. Installation of hydraulic operators would require excavating 18 ft deep pits in the existing concrete.

The construction work required by flap gate installation would be carried out in the manner similar to that described for Scheme B.

The overall estimated construction cost of Scheme D including engineering and the owner's overhead is $1,314,000 (Exhibit 13).

Discussion

The estimated construction costs of all tainter gate schemes (A,B & C) do not differ significantly:

Scheme A	$1,217,000
Scheme B	$1,152,000
Scheme C	$1,210,000
Scheme D	$1,314,000

The cost of Scheme D, the flap or Pelican gates 6 ft high, runs somewhat higher. If the objection against deep flap gates were set aside (that is the use of a single hydraulic operator per gate), the price of flap gates and hoists would be reduced but the overall cost of such a scheme would be about the same as of Scheme D.

At this point it will be helpful to ignore the relative costs of all schemes and to look at their advantages and disadvantages instead.

Fewer gates (Scheme A or 3 deep flap gates) offer the advantage of having less machinery to maintain. In addition the long free overflow crest associated with any three gate scheme will accommodate smaller fluctuations in the river flow without the need to operate the gates. This will result in less wear and maintenance than when nearly all of the crest length is covered with a string of low height gates (Schemes B, C and D).

Flap gates being overflow gates have the advantage of passing ice and trash at small gate openings and without expenditure of manpower. However, ice is not a problem at Occoquan. Dumping of trash downstream is objectionable when there is general clamor for clean streams. Flap gates also offer safety during floods; the gates will open when the hoists fail to work.

This feature at the same time represents the most serious drawback of flap gates when used in a water supply reservoir. Due to malfunctioning of a hydraulic operator or operators, the gate may open and cause a large loss of stored water. During the spill the hoist is not accessible for repairs. For this reason flap gates often have more than a single hydraulic cylinder per gate. The cylinders are designed so the loss of pressure in one of them would not cause the gate to open.

In Decatur only failures of flexible hoses connecting cylinders to the oil pressure lines have been experienced. In each case a large amount of oil was spilled into the river. The City representatives, although giving the gate a good mark, also expressed some concern for the consequences if one of the gates were to open uncontrollably causing a sudden flood wave downstream.

Tainter gates are superior in this respect. They do not open when there is a hoist failure. The only loss of water can occur if the gate already opened fails to close due to loss of power. In such a case, however, the hoist can be operated from a portable power drive or by hand. The same method of operation can be used in case of emergency during a flood.

In the worst case, in order to protect the water supply, the hoist chains or cables could be cut allowing the gate to drop. It should also be noted that the hydraulic cylinders of the flap gates sunk in deep pits are less accessible for maintenance than the conventional tainter gate hoists located on the piers or the deck.

Hydraulically, both types of gates perform well if correctly designed and properly located in relation to the crest. Flap gates require venting of the underside of the jet. This can be accomplished at small gate openings with flow splitters attached to the top edge of the gate. At larger gate openings the flow splitters are submerged and venting is provided through the air ducts in the piers and in the crest.

Usually the weir crest is made wide enough to accommodate the gate in a fully open position without any gate overhang on the downstream side. The jet of water falling away from the dam surface could cause an excessive air demand. Violent oscillation of the jet with loud accompanying noises would result.

At Occoquan the crest of the dam is fairly narrow. A six foot high flap gate cannot be fitted into it without creating some downstream overhang. To avoid this the crest would have to be widened in the upstream direction at some additional expense. Alternately extensive concrete work on the downstream face of the dam would be required to eliminate the overhang.

Scheme E (Final Design)

The 1972 flood, which occurred after completion of the above gate study, amply demonstrated vulnerability of the hard rock foundation to high flows. The deep gates, with their high flow concentration, would require construction of a large and expensive stilling basin with training walls. The additional cost involved would price any "deep gate" scheme out of contention.

The 1972 flood also brought out hydraulic inadequacies of the side channel on the north abutment. For the above reasons another low gate scheme was developed (Scheme E - Exhibit 14). It consists of eight gates 46 ft wide and 7.5 ft high. The scheme is similar to Scheme B except that the ninth gate on the north abutment was eliminated and the remaining gates were somewhat increased in height to maintain the required discharge capacity. Removal of the ninth gate will decrease flood flows over the north abutment and reduce the undesirable lateral or cross flow. In addition there will be no need to buy a separate hoist for the ninth gate. The spillway bridge was raised to clear the nappe of a 150,000 cfs discharge.

The estimated construction cost of Scheme E is $1,920,000. It reflects the changes in the riverbed topography after the 1972 flood and the effects of the repairs to the dam toe and the north abutment now under way on the cost of the construction trestle and fill. The construction scheme for work on the crest of the dam is shown in Exhibit 15. The work on the crest will be carried out in a single operation behind a steel cofferdam supported first on the crest, then on the new piers. This is considered a more realistic and conservative approach (than relying on the reservoir drawdown to complete the work on the crest as in Scheme B) in view of the unpredictability of the river flows. Some unit prices were revised upward to conform with the results of recent bidding on the dam toe repairs.

For the above reasons and also due to the escalation, the cost of Scheme E cannot be compared directly with that of any of the previously studied gate schemes which were not updated. There is actually no need for this comparison as Scheme E is simply an updated version of

Scheme B, the least expensive scheme, modified to suit the lessons of the big flood of 1972 and the changes caused by it.

Conclusions & Recommendations

The Occoquan Dam is a fairly modern structure, well designed and built. It rests on a hard rock foundation. It shows little wear and it is adequately stable to carry an additional load of 5 feet of water. The 1972 flood damage limited to the foundation is now being repaired. The steps being taken will assure greater safety against similar future floods.

Raising of the reservoir level by 5 feet will require installing gates on the dam crest. Otherwise flood backwater would rise above present levels.

Of the several gate types available, tainter gates are considered the most desirable. They are of simple rugged construction, require little maintenance other than painting and in a sense are self-closing which is a desirable feature in a water supply project. Price wise they also offer some advantage.

It is recommended the eight smaller gates (46 ft wide by 7.5 ft high - Scheme E) be used. Smaller (shallower) gates are preferred for energy dissipation reasons. The remaining 116 feet of the dam crest will be retained as a free-overflow weir.

The estimated construction cost of the recommended scheme (Scheme E) is:

Construction cost	$1,600,000
Engineering, construction supervision and owner's overhead (20%)	$ 320,000
Total estimated cost	$1,920,000

Harza Engineering Company

Restoration of Upper Occoquan Dam

Design Memo

REPORT-2

Introduction

The Upper Occoquan Dam owned by the Fairfax County Water Authority is located on Occoquan Creek, Virginia, three miles upstream of its confluence with the Potomac River.

The dam was built in 1955 by the Alexandria Water Company in Alexandria, Virginia. It was designed by the American Water Works Service Company in Philadelphia. The dam is a concrete gravity structure of 70-foot maximum height above foundation with the exception of the intake structure which is taller (Exhibit 1). The overall length of the structure is about 730 feet. Of this 520 feet is a free overflow weir. The crest of the weir and the normal reservoir level are at El. 120.0. The intake deck and the top of the south bank non-overflow section are at El. 130.0.

The intake structure serves the pipelines leading to the water treatment plant located less than a mile downstream. Three additional intakes were installed in 1966 in the right or south non-overflow section of the dam. A small powerhouse with two 350-kW generating units adjoins the intake structure on the downstream side. Power is generated only when there is excess water available. This is mostly between November and May.

In June 1972 a major flood (estimated at 75,000 cfs) spawned by hurricane Agnes went over the dam damaging its foundations. Subsequent to the hurricane the area was declared a disaster area and the Corps of Engineers, acting for the Office of Emergency Preparedness, took over the repairs to the dam. The Corps in turn engaged Harza Engineering Company to inspect the damage and to engineer the necessary repairs.

The first visit of Harza engineers to the dam after the 1972 flood took place on July 12, 1972 and the observations made were recorded in Harza letter to the Corps of July 18, 1972. Harza's recommendation was to unwater the toe of the dam in order to fully examine and assess the extent of damage. Subsequently Harza sent one of its engineers to the site to design and write specifications for the construction of an earth cofferdam necessary for unwatering.

On August 16 and 17 Harza inspected the dam again after successful unwatering. Harza's recommendation for repairs were submitted to the Corps of Engineers on August 21, 1972. The Corps, trying to complete the most urgent repairs the same fall, established a rush schedule for preparation of contract drawings and specifications for repairs to the toe of the dam. These were completed and submitted to the Corps on September 25, 1972. The drawings showing the proposed repairs to the north abutment were submitted on March 27, 1973.

Damage

After unwatering, it was found that the 1972 flood caused considerable damage to the dam foundation including the north abutment. The damage to the south abutment protected by the non-overflow section (10 feet higher than the rest of the dam) was less significant. Basically damage consisted of deep scour right at the dam toe. The force of the water falling over the dam (an uncontrolled overflow weir) plucked out a large volume of rock,much of it in the form of large boulders or blocks of rock. Near the north abutment scour reached a depth of 20 feet below the toe bucket. Near the south end of the weir the scour reached a depth of 15 feet at a point less than 15 feet away from the dam. The dam was at a steep or in some places vertical dropoff in rock with some local undercutting. The presence of a narrow fault or shear zone along the dam toe was a contributing factor in the scour.

The flood removed a very large volume of rock on the north abutment that blocked the flow path stopping short of undermining the dam concrete. On the south abutment where the dam was overtopped by only about one foot of water, the damage was limited to erosion of soil and loose rock along the dam toe. The extent of damage is described in detail in Harza letter-report of August 21, 1972.

Dam Safety

The dam itself is a well built and well preserved structure. The concrete is of good quality and the uplift control provisions: a grout curtain and a system of drains shown on the construction drawings represent modern practice. The foundation consisting of hard granitic gneiss is excellent. The dam is well keyed into the rough surface of the rock as evidenced by the photos of the dam construction. The concrete-to-rock contract and the foundation itself are very tight as demonstrated by the very small drain discharge and the dryness of the underlying rock observed after unwatering of the tailrace.

The very quality of the foundation could have lead the designers of the dam to believe that the rock below the dam did not need any protection. In addition they did not make any provisions for flow going over the north end of the dam to return to the river channel. The resulting adverse hydraulic conditions caused extensive erosion in the north abutment. They also produced the deepest scour.

After unwatering the tailrace, the dam was not found in immediate danger. It was obvious, however, that future floods could be expected to increase the scour and further undermine the dam.

Remedial Measures

General

The remedial measures recommended in Harza's letter-report of August 21, 1972 were directed at insuring the dam safety by: 1) protecting the rock at the dam toe and the foundation rock undercut beneath

the dam with concrete, 2) improving hydraulic action and energy dissipation conditions of the spillway.

Dam Toe

The deeply scoured areas were retained as natural stilling basins and paved. Excavation of high points or rock was required to smooth out the surface for paving. In the center portion of the dam some excavation was necessary to assure adequate depth for hydraulic jump at lower and medium flows.

The length of the apron was set arbitrarily at 20 feet plus a seven foot wide (at the base) end sill or deflector. The length of the apron is considerably shorter than that normally provided for area of energy dissipation. It was felt, however, that a longer apron was not warranted on hard rock foundation. A relatively short apron provided with an end sill should assure that any future scour would occur away from the dam.

Minimum apron thickness was set at two feet. The apron slab was anchored to rock with #8 bars spaced at five feet on centers. Contraction joints with waterstops were placed at 25 foot intervals along the dam.

The horizontal apron was connected to the spillway bucket with a concrete chute sloped at 1 on 2.5 to fit topography of the rock. The chute was made tangent to the curvature of the bucket. This required removal of some of the bucket concrete and underlying rock. Drain holes were provided in the sloping chute drilled into the rock foundation.

Due to the large differences in the depth of scour along the dam, the apron was stepped. Otherwise, the amount of concrete or of excavation or both would increase considerably.

North Abutment

The north abutment presented much more of a problem than the fairly straightforward treatment of the dam toe. Ideally the flow over the north end of the dam should be returned gradually to the streambed without interfering excessively with the action of the hydraulic jump below the main central portion of the dam. This, however, would require very extensive rock excavation in the steep abutment.

A less expensive solution was to treat the north end of the weir as a side channel spillway. A chute for this could be excavated reasonably by taking advantage of the partial cut in the abutment cliffs already made by the flood. An effort was made to fit the chute to the irregular rock topography. However considerable rock excavation was still required particularly in the upper part of the abutment. A large knob of rock on the abutment directly downstream of the spillway and to the north of Station 4 + 50 remained after the 1972 flood. A large portion of this knob had to be removed to relieve the existing condition of extreme turbulence and flow concentration.

The bottom and the downstream wall (rock face) of the chute were paved. This was considered a necessary measure in view of the vulnerability of the rock to high flows amply demonstrated by the flood of 1972. All concrete pavement was anchored to rock, similarly to the apron in the riverbed section of the dam. A paved transition slope was provided between the dam toe and the chute floor.

The final design was worked out using detailed rock topography surveyed after the flood by Robert R. Kim and Associates Inc. for the Fairfax County Water Authority.

The area located at the toe of the north abutment (between Stations 3+10 and 4+12.50) required special attention. At this point the cross flow from over the abutment collides with the discharge over the center portion of the dam. The resulting turbulence caused more extensive scour here than at any other point along the dam.

Accordingly the elsewhere short apron was extended beginning with Station 3+10 to protect a larger area.

REPORT-3

October 17, 1973

Department of the Army
Baltimore District
Corps of Engineers
P.O. Box 1715
New Federal Building
Baltimore, Maryland 21201

Attention: Mr. Dick Strong

Subject: Lower and Upper Occoquan Dam Repairs as a Result of Tropical Storm "Agnes"

Gentlemen:

We are pleased to report that the exploratory drilling of the Lower Dam was completed last month. We had our Mr. Kim de Rubertis, who assisted in setting up the original exploration plan, visit the dam twice during drilling; once at the beginning of drilling in order to locate the holes and to set up the necessary procedures; the second time at the end of drilling to review its results, to re-inspect the structure in this light and to determine the scope of repairs. We also re-inspected the south abutment damage at the Upper Dam and studied the scope of necessary repairs.

This letter reports our findings and lists our recommendations.

Lower Dam

Findings

Soil Consultants, Inc. from Merrifield, Virginia was hired by Fairfax County Water Authority to do the drilling in accordance with authorization received from OEP. Drilling in general followed the foundation investigation plan proposed by us in our letter of August 10, 1972. All told 10 core holes were drilled located as shown in the enclosed Exhibit 1. Core logs can be found in Exhibit 2. The work was supervised by Mr. Stan Kiefer, Structural Engineering Inspector, the Fairfax County Water Authority. The core logs and the cores were reviewed at the site by Mr. de Rubertis and his comments as to the character and

The quality of both the concrete and the foundation are noted on the core logs. Seepage tests were made by placing dye upstream of the dam and observing the time required for the dye to appear downstream.

a. Concrete. Four concrete cores were broken in the laboratory of Froehling & Robertson, Inc. to determine their compressive strength. The results are listed in Exhibit 3. The compressive strength varies from 2,381 psi to 4,435 psi.

Visual examination of the concrete cores indicated the following:

Lift joints generally are not visible in cores,

There is little or no honeycombing effect, concrete is over-sanded and dense and only slightly pitted,

Most drilling breaks took place around coarse aggregate,
Coarse aggregate comes from stream gravels composed of sound gneiss, schist, phyllite and quartz pebbles, round to subnormal,
Common core lengths are 6 inches,
Concrete has a good ring when struck.

In four out of ten holes the concrete in the upper ten feet of the dam appeared to be less sound. There is little seepage through the concrete lift joints.

The pool was drawndown during drilling exposing most of the upstream face of the dam. In general the concrete is quite crackfree.

b. Foundation. Generally the dam is founded on massive rock of good quality (granitic gneiss), not highly jointed. No bond was found between concrete and rock in some of the holes but the contact was tight. The dam is exceptionally well keyed into the rock.

Weathered rock was encountered in the holes near the north abutment where most of the flood damage occurred. Dye test indicated little underseepage.

In general, considering the low height of the structure, the foundation is more than adequate. The weathered rock at the north end of the dam will not present a problem if repairs include protection of the foundation at the dam toe against scour or undercutting.

c. Stability of the Dam

A low dam of relatively massive proportions well keyed into the hard rock foundations as Lower Occoquan Dam normally does not present any stability problems. This does not apply to the north end of the dam in its present state. Water levels observed in the drill holes did not indicate excessive uplift.

d. Flood Damage. Flood damage is concentrated at the north end of the dam. Flood waters carried away the very end of the dam (a low block of concrete about five feet long at the crest and about 10 ft long at the bottom), scoured the rock of the abutment directly below the dam and undercut the dam toe for a distance of over 100 feet starting at the north end. The cavity reaches a height of several feet and extends upstream into or under the body of the dam for up to three feet. Some concrete over the cavity was eroded and carried away. All damage in this portion of the dam seems to be confined to the area of weathered rock.

Moving away from the north abutment toward the center of the stream a deep water filled hole is encountered right at the toe of the weir. Soundings did not disclose any undercutting in this location. It will be advisable, however, to confirm this when the hole can be unwatered during the repairs to the dam.

The sluiceway outlet slab (near the south abutment) is undermined. The sluiceway training wall is severly undermined over a distance of over 15 feet. Failure of the wall could possibly endanger the carbon slurry house. The sluice gate hoist stem is bent and the gate hoist is askew. This area requires restoration.

Recommendations

The extent of needed repairs at the north end of the dam is fairly obvious.

The cavity at the toe has to be chipped and cleaned of all loose material, filled with concrete to the original ogee outline and grouted where conconcrete may not penetrate all voids. Some anchor bars will be required. Drypacking along the toe is recommended in other areas.

The weathered rock along the damaged toe should be paved over with a concrete slab one foot thick well anchored.

The washout concrete section at the very north end should be replaced and anchored to rock.

The deep hole in the center section of the weir should be unwatered for inspection. If any undercutting is found, repairs would be similar to those at the north end.

The undermined section of the sluiceway training wall should be repaired by casting a new foundation extended to solid rock. The sluiceway slab should be removed and recast.

We also recommend that the south abutment slope be riprapped. The sluice gate, hoist and stem should be repaired or replaced or both as necessary to make them operable. We understand that this work is being repaired by the Authority under a separate authorized contract.

The enclosed drawing (Exhibit 4) will assist you in reviewing our recommendations.

Upper Dam South Abutment

As reported in our letter of August 21, 1972 the south non-overflow section of the dam was overtopped during the Hurricane Agnes flood by about one foot of water. The falling water washed out overburden (to a maximum depth of up to 20 ft) and loose rock along the toe of the dam and also some overburden above the end of the dam. No undercutting of the dam concrete took place. Since a still larger flood has to be anticipated in the future with the accompanying more serious damage, means of repairing the dam foundation needs to be considered.

We recommend that the existing trench-like scour be cleaned of debris and partially filled with riprap. The riprap would be grouted with concrete using small aggregate. Grouting is necessary as otherwise the impact and velocity of water would be too excessive for any reasonable size riprap. It may be advisable to let the individual rocks project above the concrete (grout). The rough surface thus created will tend to slow down the water rushing down the slope of the abutment. The proposed scheme is outlined on the enclosed sketches (Exhibit 5).

We also considered the need to protect the steel pipes of the water intake, the water tank and the powerhouse south brick wall. None of these structures, however, suffered any visible damage during the last flood. The reason appears to be a high rise in tailwater which submerged or flooded everything within the lower half of the non-overflow

section. This provided a cushioning effect for the falling water and rather effectively protected the structures.

For this reason we do not think that any other restoration measures need to be undertaken on the south abutment except for grouted riprap protection outlined above.

Powerhouse Intake Wall

We were not able to inspect the powerhouse wall because the paneling covering the wall has not been removed yet. We did, however, examine the original drawings of the wall. They show no reinforcing steel in its downstream face. If the wall was actually built this way, cracks and seepage could be expected. In addition the drawings show that the north end of the wall is not tied back into the cross wall, i.e., into the north wall. Consequently, there are theoretically at least some tensile stresses present in unreinforced concrete. It is our understanding that the Authority is proceeding to investigate in detail any damage that may have occurred in this area.

In view of the above, it is mandatory that the wall be inspected rather thoroughly. Some core drilling may be necessary.

Your comments and approval to proceed with detailed construction plans and specifications relative to all of the items described herein (except the Powerhouse Intake Wall) are awaited.

Very truly yours,

Andrew Eberhardt
Chief Structural Engineer

Encl: As Noted

cc: Mr. Floyd Eunpu, Fairfax County Water Authority
(w/encl.)

Washouts occurred at both dam ends. At either end, however, the dam is abutting against rock and the water pouring over the dam has not undermined this contact. The washouts are limited to the overburden and loose or jointed rock at the toe of the dam.

On the left (or north) abutment a large volume of rock was removed from the rocky bluff or knob which blocked the path of water going over the spillway left end. No undermining of the dam, however, took place in this area.

The configuration of the river bottom immediately downstream of the dam was changed considerably by the flood. The width of the plunge pool (filled with water) scoured along the toe of the dam has about doubled. A bar made up of large blocks of rock has formed parallel to the dam and approximately 100 feet downstream of the toe. Only smaller rocks or stones existed there before. Soundings indicated that at the same time the depth of the scour has increased up to 8 feet. The concrete toe of the dam has been undermined by pockets of erosion in a few places to a depth of several or more feet. The powerhouse tailrace channel has been blocked with rocks and the water discharged through the valves or leaks in the intake or the powerhouse now flows along the toe of the dam and across the riverbed until it reaches the vicinity of the left (or north) bank.

Lower Occoquan Dam

The flood washed out the very left or north end of the dam. The dam at this point is a low concrete gravity structure. The washout is only several feet deep but the flood waters rushing down the abutment along the toe of the dam also undermined the latter. There is an open space or gap between the concrete and the rock surface along the toe extending at several or more feet upstream. The gap extends toward the river up to where the central portion of the dam is founded on massive rock. A good portion of the dam toe to rock contact is in the dry exposed to view and is obviously intact. One exception is a deep pocket in the foundation where the toe of the dam is submerged and could not be examined.

A concrete wing wall at the right (or south) abutment displays a vertical crack in the middle of its length. This could be an old shrinkage crack. A fairly large volume of rock was eroded from the left abutment downstream of the dam. The training wall along the right abutment was undermined for much of its length.

HARZA ENGINEERING COMPANY CONSULTING ENGINEERS

Mr Vestmeta

#4

July 18, 1972

REPORT-4

Baltimore District
Corps of Engineers
Area 11
T.C. Williams High School
3330 King Street
Alexandria, Virginia 22314

Attention: Mr. Ira E. Reed, P.E.
Area Engineer

Gentlemen:

This letter-report is in answer to your request made at the conference held at the Occoquan Upper Dam on July 12, 1972 between the Corps of Engineers representatives, Mr. Warren Hunt of Fairfax County Water Authority, and Harza Engineering Company of Chicago represented by Messrs. Conroy and Eberhardt. We have been working as consultants for the Fairfax County Water Authority and this letter is being submitted with their knowledge.

It was agreed that we would submit to you:

1. Our findings resulting from an inspection of both Occoquan dams, Upper and Lower.
2. Our recommended program of investigations of the extent of damage.
3. An order-of-magnitude type of estimate on the probable construction cost of the required repairs.

Observations made during the Inspection

Upper Occoquan Dam

There is no visible damage to the concrete structures. The contraction joints between the dam monoliths that were tight before the flood remained dry, suggesting that no differential or significant movement has taken place. The only possible exception is the intake wall. This wall had developed some leaks in the past which could be attributed to the cracks caused by lack of steel reinforcement in the downstream face. Now, after the flood, the wall portion above the powerhouse roof seems to have sprung new leaks or opened the old ones. These leaks, however, are very small.