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NATIONAL DAM INSPECTION PROGRAM. WARNER GAP HOLLOW DAM (EDGEMON--ETC(U)
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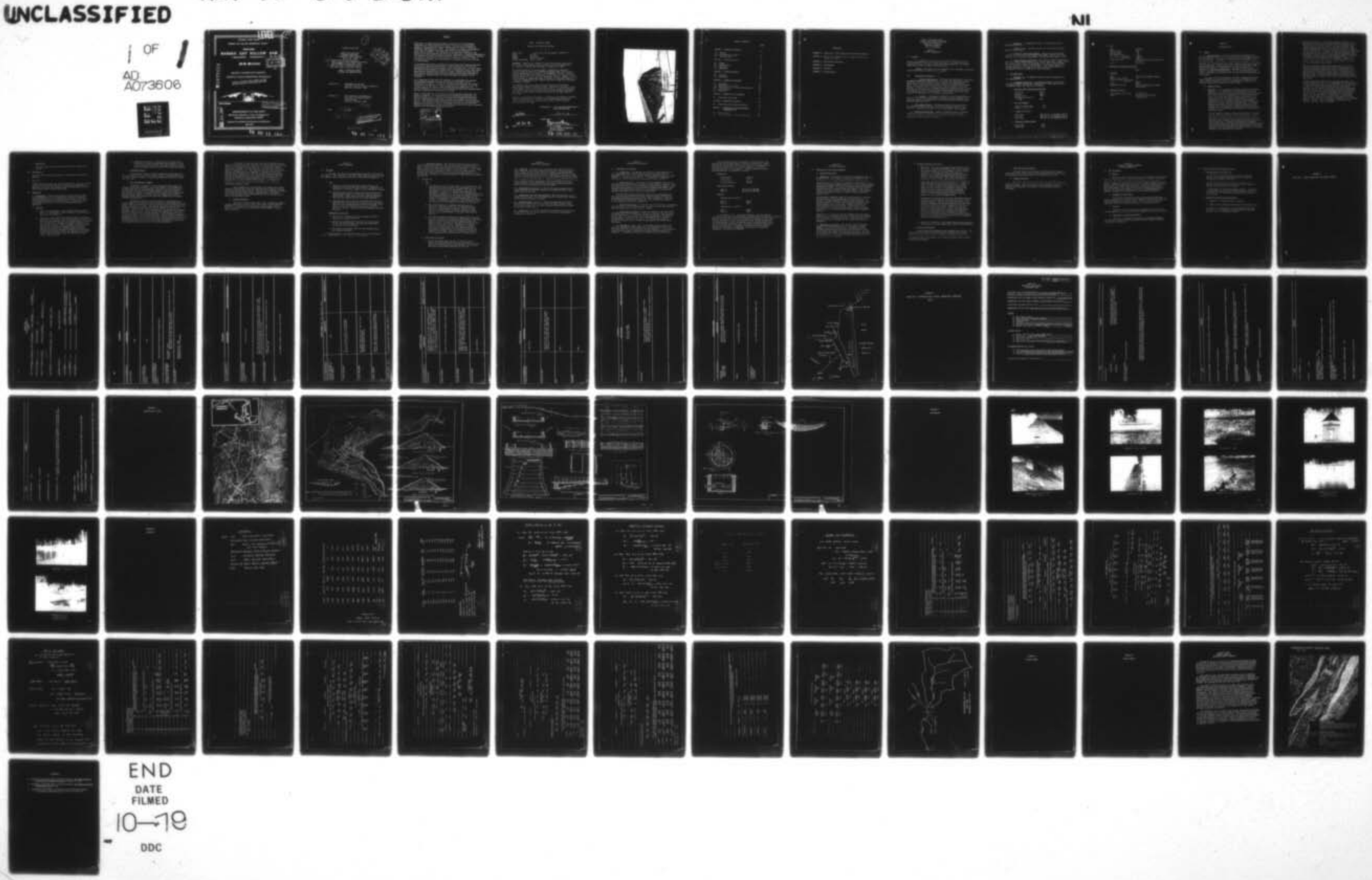

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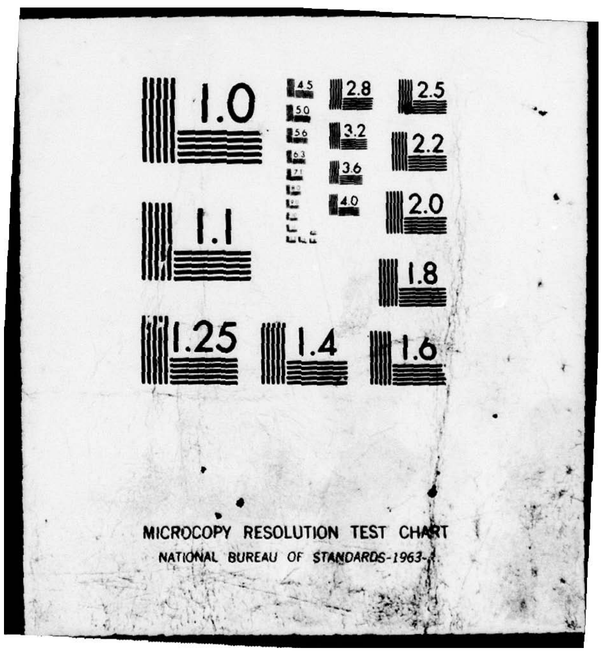
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POTOMAC RIVER BASIN

WARNER GAP HOLLOW, WASHINGTON COUNTY

MARYLAND

WARNER GAP HOLLOW DAM

(EDGEMONT RESERVOIR)

NDI NO. MD 00006

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

ORIGINAL CONTAINS COLOR PLATES: ALL DDC REPRODUCTIONS WILL BE IN BLACK AND WHITE.

DEPARTMENT OF THE ARMY

Baltimore District, Corps of Engineers

Baltimore, Maryland 21203

Prepared By: Maryland Water Resources Administration

MAY 1979

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POTOMAC RIVER BASIN

WARNER GAP HOLLOW DAM
(EDGEMONT RESERVOIR)
WASHINGTON COUNTY, MARYLAND
NDI NO. MD 00006

(6) National Dam Inspection Program.
Warner Gap Hollow Dam (Edgemont Reservoir)
(NDI MD 00006). Potomac River Basin,
Warner Gap Hollow, Washington County,
Maryland. Phase I Inspection Report.

PHASE I INSPECTION REPORT
NATIONAL DAM ~~SAFETY~~ PROGRAM

Prepared for: DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

Prepared by: WATER RESOURCES ADMINISTRATION
Department of Natural Resources
Tawes Building
Annapolis, Maryland 21401

(12) 83p.

Date: (11) May 1979

(10) Jeffrey O. Smith

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PREFACE

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

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

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

Phase I Inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. The spillway design flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

NAME OF DAM: Warner Gap Hollow Dam (Edgemont Reservoir)
STATE: Maryland
COUNTY: Washington
STREAM: Warner Gap Hollow
DATE OF INSPECTION: April 9, 1979

ASSESSMENT: Based on the evaluation of the conditions as they existed on the date of the inspection and as revealed by visual observations, the condition of Warner Gap Hollow Dam is assessed to be good.

The spillway capacity (17 percent PMF) is classified as inadequate because it will not pass the recommended spillway design flood of full Probable Maximum Flood according to the recommended criteria. Additional analyses of the downstream consequences in the event of a failure indicate that the dam is not in the seriously inadequate category. However, the owner should initiate an engineering study to evaluate the spillway capacity and to develop recommendations for remedial measures to reduce the overtopping potential of the dam.

The seepage at the left abutment should be monitored routinely for quantity and turbidity by means of a weir. Maintenance and repairs should be performed to restore the 30-inch drain valve to an operative condition and a positive seal should be provided on the upstream side. Maintenance work should include removal of woody vegetation on the downstream face of the dam.

Operation and maintenance procedures are unwritten and should be documented. A warning system should be developed to warn downstream residents of large spillway discharges during periods of heavy rainfall and run-off or failure of the dam.

SUBMITTED BY: WATER RESOURCES ADMINISTRATION
DAM SAFETY DIVISION

6/8/79
Date

Jeffrey O. Smith

APPROVED BY:

23 Jun 79
Date

G. K. Withers
G. K. WITHERS
Colonel, Corps of Engineers
District Engineer

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Edgemont Reservoir MD 00006

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
WARNER GAP HOLLOW DAM
(EDGEMONT RESERVOIR)
NDI NO. MD 00006

SECTION 1
PROJECT INFORMATION

1.1 General

a. Authority. The inspection was performed pursuant to the authority granted by the National Dam Inspection Act, Public Law 92-367, to the Secretary of the Army, through the Corps of Engineers, to conduct inspections of dams throughout the United States.

Purpose. The purpose of this inspection is to determine if the dam constitutes a hazard to human life or property.

1.2 Description of Project

ABSTRACT

a. Dam and Appurtenances. The Warner Gap Hollow Dam, known locall as Edgemont Reservoir, consists of a zoned earth fill embankment approximately 65 feet high and 700 feet long. A puddle wall approximately 5 feet wide by 15 feet deep near the upstream toe is detailed on the design plans. The rip-rap covered slopes rise at $1\frac{1}{2}$H:1V downstream and 2H:1V upstream. The spillway is a rectangular concrete channel with the entrance located near the right abutment and discharging approximately parallel to the longitudinal axis of the embankment. The spillway crest is at elevation 901.0, approximately 4 feet below the dam crest. ABSTRACT

b. Location. The Edgemont Reservoir is located approximately 1 mile east of the town of Smithsburg in Washington County, Maryland The structure impounds the headwaters of the Little Antietam Creek drainage, known as Warner Gap Hollow.

c. Size Classification. The maximum height of the dam is 65 fe . The reservoir volume to the top of the dam at elevation 905.0 is 296 acre-feet. Therefore, the dam is in the "intermediate" size category.

d. Hazard Classification. Damage to downstream roads, railroads, commercial buildings, residences and loss of more than a few lives would likely result from a failure of the dam. Accordingly, the dam is classified in the high hazard category.

e. Ownership. The Edgemont Reservoir is owned by the City of Hagerstown, Maryland.

f. Purpose of Dam. The single purpose of the dam is to provide municipal water supply.

g. Design and Construction History. The present structure was designed by the American Pipe Manufacturing Company, Engineers and Contractors of Philadelphia, Pennsylvania. Construction took place in 1902.

h. Normal Operating Procedures. The reservoir supplies approximately 50% of the average 10 MGD demand for the City of Hagerstown. Normal inflow and releases from storage are discharged to the City or the Smithsburg Reservoir through a system of supply mains. A low masonry dam on the adjacent Raven Rock watershed may be used to divert inflow directly to the Edgemont Reservoir through a 30" terra cotta pipe.

1.3 Pertinent Data

a. Drainage Area The Edgemont Reservoir has a drainage area of 2.34 square miles.

b. Discharge at Dam Site The maximum discharge at the dam site through the ungated spillway at elevation 901.0 is 991 cubic feet/sec. The maximum flood at the dam site is unknown.

c. Elevation (feet above mean sea level)

Top of Dam	905.0
Low Point	904.6
Spillway Crest	901.0
Streambed at centerline of dam	840

d, Reservoir (miles)

Length of maximum pool	0.20
Length of normal pool	0.17

e. Storage (acre feet)

Normal pool	246 acre ft. at elevation 901.00
Low point	292 acre ft. at elevation 904.65
Top of dam	296 acre ft. at elevation 905.00

f. Reservoir Surface (acres)

Top of dam	13.5
Normal pool	12.0

g. Dam

Type	Earthfill
Length (feet)	700
Height (feet)	65
Top width (feet)	16
Side slopes - Upstream	2H:1V
- Downstream	1½H:1V
Impervious Core	Zoned construction, material not specified
Cutoff	Puddle Wall

h. Diversion and Regulating Tunnel

None

i. Spillway

Type	Concrete rectangular channel
Width of weir (feet)	25
Crest elevation (feet above M.S.L.)	901.0
Gates	None
Downstream channel	Spillway discharges to Raven Rocks Creek.

j. Regulating Outlets

Water Supply mains (two)	12 inch C.I.P.
Drain	30-inch C.I.P. blow-off (inoperative)

SECTION 2

ENGINEERING DATA

2.1 Design:

a. Data Available: Warner Gap Hollow Dam was designed and probably constructed by the American Pipe Manufacturing Company, Engineers and Contractors, Philadelphia, Pennsylvania, during 1902. The only engineering data available for the design of the dam is contained on drawings entitled "Plan" and "Details of 100 Million Gallon Inpounding Reservoir for the Washington County Water Company, Hagerstown, Maryland", dated January 1902. These drawings were retraced by the City of Hagerstown Water Department in 1968 and the tracings are presented in Appendix C, "Location Map and Plans". Although test pit locations are shown on the plan view, subsurface data was not found during the data review. Hydrologic and hydraulic data was apparently not generated for this project.

Many years after construction of the dam, a seepage area at the right abutment was studied by Whitman, Requardt and Associates and a report entitled "Edgemont Reservoir, Investigation of Leakage with Remedial Grouting" was prepared in November 1968.

b. Design Features

1. Embankment - The construction drawings indicate the earthen embankment to be constructed in two zones with "selected material, compacted" beneath the upstream face and crest areas, and "ordinary material, compacted" in the downstream slope area. The dam was to be placed with a 2 horizontal to 1 vertical upstream slope and a 1½ horizontal to 1 vertical downstream slope. It should be noted that the graphic presentation on the drawings incorrectly shows the upstream slope to be steeper than the downstream slope. The slope configuration labels, however, indicate the intent of the designer and were apparently followed during construction. Rip-rap, 1 foot in thickness was provided on the upstream and downstream slopes.

   The fill reaches a maximum height of 60 feet near the center of the profile and is shown to be placed against a gently sloping left abutment and a steep right abutment. A puddle wall, five feet in width and fifteen feet in depth was constructed in the original stream valley along the upstream toe of the embankment. Although the material of the puddle wall was not specified on the plans, puddle is generally defined as a mixture of clay and gravel or sand, which are well mixed together, carefully moistened; and rammed in place. Foundation Material is not specified on the plans.

2. Appurtenant Structures - An overflow spillway consisting of a rectangular concrete waste channel 550 feet in length, 25 feet in width and 3 to 4 feet in height, is located near the right abutment approximately parallel to the axis of the dam. The construction drawings show the channel floor to have a slope of 1 foot in 26 feet at the entrance rising to a peak at el. 901, and an exit slope of 1% transitioning to the natural grade of 20%. The spillway floor consists of plain concrete slabs 18 inches in thickness. In the vicinity of the spillway entrance, a cutoff is provided by increasing the slab thickness in steps to a maximum of 6 feet at the entrance. The drawings indicate the channel floor to be lined with pine boards attached to wooden stringers embedded in the concrete.

The side walls of the spillway are indicated to be of plain concrete, ranging in height from 4 feet at the entrance to 3 feet along the natural grade and varying in width from 3 feet at the base to 2 feet at the top of wall. The plans do not provide for any construction joints within or between the walls and floor slabs. A rip-rap filled timber crib, 24 feet long by 41 feet wide, is shown at the outfall of the spillway in the plunge pool.

A drain consisting of 30-inch cast iron bell and spigot pipe entends through the center of the embankment with an inlet invert at approximately el. 850 and an outlet invert at approximately el. 840. Two 12-inch water supply pipes of unspecified material pass through the right side of the dam at approximate els. 850 and 860. The drain and supply lines are controlled by valves and blow offs positioned just beyond the downstream toe of the dam. All pipes are provided with cut off walls within the upstream side of the embankment. An additional 12-inch supply pipe, existing prior to placement of the dam near the right abutment, was apparently intended to be preserved during construction, but no treatment, remedial work, or additions are specified on the plans.

c. Design Data

The only design data available consists of the construction drawings.

2.2 Construction.

The only construction data available is contained on construction drawings.

2.3 Operation.

Formal operating records have not been maintained. The only written records consist of daily pool levels within the impoundment, rain gage data at the dam, and limited stream flow data above the reservoir on the stream in Warners Gap Hollow.

2.4 Evaluation.

a. Availability. Construction drawings and the remedial study report prepared by Whitman, Requardt and Associates constitute the engineering data and are available in the files of the State of Maryland Department of Natural Resources. Daily pool levels are contained in the files of the City of Hagerstown Water Department.

b. Adequacy.

1. Hydrology and Hydraulics - The original design considerations are unavailable. Refer to Section 5, Hydrology and Hydraulics and Appendix E.

2. Embankment - The construction drawings address the embankment configuration only. Test pit locations are shown on the drawings, but subsurface data and interpretation of the data are not available. Although the original design data, if any existed, relative to soil strengths, foundation capacities, slope stability and seepage analyses were not available for review, the remedial study performed by Whitman, Requardt and Associates provides some indication of subsurface conditions within the embankment and foundation. Refer to Section 2.4.d. for discussion of the remedial grouting study.

3. Appurtenant Structures - Design data for the appurtenant structures is limited to dimensions and locations as shown on the construction drawings. Structural analyses of the concrete spillway were not available for review and comment.

c. Operating Records.

The daily pool levels, rainfall records and stream gage data for a limited period are the only written records. This data alone does not allow complete assessment of operating procedures relative to stability of the dam.

d. Post Construction Changes.

Subsequent to completion of the dam, pronounced seepage was observed at the right abutment. In the late 1930's, a hole appeared in the downstream face of the embankment, 8 feet down from the crest and 25 feet along the crest from the right abutment. The superintendent at the time, Mr. Lee Harne, filled the hole with concrete and no additional holes have been reported. In 1968, the City of Hagerstown retained Whitman, Requardt and Associates to study the seepage area and prepare remedial recommendations.

The Whitman, Requardt study consisted of a preliminary subsurface exploration by test pits in the vicinity of the seepage points and utility trenches, monitoring of seepage outflow at several locations, test borings, and grouting program. The seepage monitoring indicated that the quantity of seepage varied directly with the height of water in the reservoir. Test pit observations suggested that the seepage was originating through the foundation and not through the embankment or along utility trenches. Anticipating a grouting program, six test borings were advanced at the right side of the dam along the crest. The test holes penetrated the embankment, extended into the rock foundation, and obtained soil samples, standard penetration blow counts, rock cores, and ground water levels. Dye was introduced into two of the test holes and was observed to issue from the seepage zones downstream of the dam shortly thereafter.

The results of the subsurface exploration indicated the embankment to consist of stiff silty clay with rock fragments corresponding to the "Selected Material, Compacted" zone shown on original construction drawings. No test holes were drilled in the "Ordinary Material, Compacted" zone. The foundation materials were found to be phyllite and shale which was weathered, fractured and relatively pervious in the seepage area. The initial test holes were pressure grouted and an additional 17 holes were drilled from 20 to 30 feet into the foundation and pressure grouted. Upon completion of the grouting program, the seepage stopped and has not re-occured to date. Another seepage source beyond the left abutment was judged to be not directly related to the reservoir and dam and was not included in the Whitman, Requardt study.

Since completion of the spillway, the original pine planks lining the spillway floor have been removed and the concrete of both the floor and side walls restored. This work was alledgedly performed in the 1930's by the Works Progress Administration. The timber crib at the spillway outfall has disappeared, but the plunge pool is rock lined and appears stable.

e. Seismic Stability.

The dam is located within seismic zone 1 immediately adjacent to zone 2 and static stability with normal safety factors should be sufficient to withstand minor earthquake induced dynamic forces. However, no calculations or studies have been performed to confirm static stability.

SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. The dam and its appurtenant structures were found to be in good overall condition at the time of the inspection, April 9, 1979. The complete visual inspection check list is presented in Appendix A.

b. Dam.

1. There is a six-inch depression in the vicinity of the observation well near the right abutment. Otherwise, the horizontal and vertical alignment of the embankment is good.

2. There are minor surface irregularities in the rip-rap on the downstream slope. The slope has scattered trees and brush growth. Based upon an October, 1978, photograph at low pool, the upstream rip-rap is in good condition.

3. Flow from left side of the dam and downstream of the toe (springheads) are conveyed via a system of tile and plastic pipe to a stone outlet beyond the 30" blow-off pipe. The flow was clear and estimated to be between 10 and 15 gallons per minute. The pipe system was damaged in several locations.

c. Appurtenant Structures.

1. The concrete associated with the rectangular overflow spillway was in good condition.

2. Debris and sediment partially choke the outlet channel for the 30" blow-off pipe. The valve was inoperative at the time of the inspection.

3. The control and by-pass valves for the discharge mains are regularly operated.

d. Reservoir Area. The reservoir slopes are steep and heavily wooded. Sedimentation is not reported to be a problem.

e. Downstream Channel. The discharge from the overflow spillway joins the Raven Rock Stream to form the headwaters of Little Antietam Creek. There are boulder deposits within the stream channel as the slope becomes milder about 2000 feet downstream of the reservoir. In the event of a dam failure, approximately 5 homes, a commercial establishment, several State and County roads and the Western Maryland Railroad could be affected. Consequently, a hazard category of "high" appears appropriate for this dam.

3.2 Evaluation.

a. Dam

1. The depression near the right abutment is purportedly in the same general location where a hole appeared in the downstream face near the crest in the late 1930's. This area also corresponds to the area of seepage investigated in 1968. Referring to discussion in Section 2.4.d. and Section 6.1.a.(the hole and depression is the result of internal erosion, loss of embankment material and subsequent "day-lighting" of voids. No indications of recent settlement were observed in this area during the inspection. The depression increases the potential for over-topping as discussed in Section 5.

2. The recent dumping of replacement rip-rap over the downstream slope may account for the non-uniform slope surface noted during the inspection. Tension cracks, toe heave, and/or alignment irregularities which might indicate deep seated movement of the dam slopes were not observed.

3. The 1968 investigation judged the seepage activity beyond the left abutment not as serious as the leakage at the right abutment and further evaluation was deferred until the spring of 1969. No record of the deferred evaluation was found during the data review. Based upon the relative clarity and apparent seasonal fluctuation of seepage, the source of discharge at the left abutment appears to be primarily spring activity not directly affecting the stability of the dam. However, the quantity and turbidity of seepage should be monitored on a routine basis.

b. Appurtenant Structures

1. Due to the annual draft and lack of formal operating policy, the 30-inch blow-off valve has not been exercised since a reservoir cleaning was accomplished in the 1940's. The valve is considered inoperable at this time.

SECTION 4
OPERATIONAL PROCEDURES

4.1 Procedure. The purpose of this dam is to provide for municipal water supply for the City of Hagerstown, Maryland. The normal pool varies widely with the annual pattern of runoff, demand and diversion from the adjacent Raven Rock watershed. Discharges to the water supply system are accomplished by operation of valves downstream from the embankment toe. The drain valve (30-inch blow-off pipe) is presently thought to be inoperable. Uncontrolled discharges are through the overflow spillway at elevation 901.0, approximately four feet below the dam crest. Though unwritten, the diverted discharges from the Raven Rock watershed are ceased during periods of heavy rainfall.

4.2 Maintenance of the Dam. No written maintenance program has been established, but the general appearance of the dam indicates a high degree of pride and care.

4.3 Maintenance of Operating Facilities. With the exception of the inoperative drain gate, the operating facilities receive daily attention and as needed maintenance and repair.

4.4 Warning System. There is no formal warning system in effect although the resident manager at the dam site calls rainfall totals and reservoir stages into the County's Central Alarm which is linked to the Civil Defense System.

4.5 Evaluation. The general operational procedures are satisfactory except that no formal warning system is in effect and maintenance procedures are unwritten.

SECTION 5
HYDRAULICS AND HYDROLOGY

5.1 Evaluation of Features.

a. Design Data. The Edgemont Reservoir has a watershed area of 2.34 square miles and impounds a reservoir with a surface area of approximately 13 acres. The concrete overflow spillway beyond the right abutment can safely discharge 991 cfs. No hydrologic or hydraulic design data were available for the preparation of this report.

b. Experience Data. As previously stated, Edgemont Reservoir is classified as an intermediate size dam in the high hazard category. Under the recommended criteria for evaluating spillway discharge capacity, such structures are required to pass the Probable Maximum Flood (PMF). The maximum flood magnitude at the dam site is unknown but is believed by operating personnel to have occured in March, 1936.

The PMF inflow hydrograph for the reservoir was determined utiliing the Dam Safety Version of the HEC-I computer program developed by the Hydrologic Engineering Center of the Corps of Engineers. The peak of the PMF inflow hydrograph is 5920 cfs. The input data and results of the program are presented in Appendix E.

c. Visual Observations. On the date of the inspection, no conditions were observed that would indicate that the spillway of the dam could not operate satisfactorily in the event of a flood.

d. Overtopping Potential. Various percentages of the PMF inflow hydrograph were routed through the reservoir to determine the percentage of PMF inflow that the dam can pass without overtopping. The analyses indicate that the 10% PMF level can be discharged without overtopping the embankment. Should the low point be filled and the dam crest brought to a uniform level, the dam can pass 20% of PMF without being overtopped.

e. Spillway Adequacy. Since the spillway can pass neither PMF nor ½ PMF, the degree of inadequacy must be determined by first estimating the effect of overtopping upon the dam embankment. If overtopping causes the dam to fail, a breach analysis must then be performed to determine if a significant increase in hazard to loss of life exists.

The overtopping analysis of Appendix E indicates that ½ PMF overtops the dam by approximately 1 foot for a period of 6.5 hours. The velocity over the dam crest at the low point is computed to be 4.8 feet per second which is considered sufficient to initiate erosional failure of the embankment. A breach analysis was therefore performed and is summarized as follows:

Assumption:

Breach Width	100 feet
Side Slopes	1V:1H
Failure Time:	2 hours
Failure Elevation:	905.65

Downstream Analyses:

Plan 1:	Dam fails @ 50% PMF
Plan 2:	No Failure @ 50% PMF

Results:

Maximum Stage, Section 3

Plan 1:	842.8
Plan 2:	840.1

Maximum Stage, Section 4

Plan 1:	786.6
Plan 2:	785.2

At the 50% PMF level, the analysis indicates that the breached structure would increase the downstream discharge rate from 2960 cfs to 5955 cfs However, the wide flood plain in the danger reach/damage center results in minor increases in stage of less than 2 feet. The flood peak from the breached dam arrives at the damage center 15 minutes earlier than the peak assuming overtopping only. Based upon the visual observation of the downstream area, no significant increase in loss of life or damages are envisioned. Accordingly, the spillway is considered inadequate, not seriously inadequate.

SECTION 6
STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations

1. Embankment. The majority of the earthen embankment appears stable with no visible signs of cracking, settlement, or differential movement. The depression observed near the right abutment is judged to be the result of the internal seepage and loss of embankment material through the hydraulically open fracture system of the rock foundation which occured prior to the grouting program implemented in 1968. Based upon the present absence of seepage, the grouting work is judged to have successfully treated the causes of internal erosion at the right abutment. The depressed area does not exhibit cracks or signs of recent movement and appears stable.

The downstream embankment slope of 1.5 horizontal to 1 vertical and the upstream slope configuration of 2 horizontal to 1 vertical are steep compared to modern design standards. The continued stability of these slopes under past operating conditions suggests that insitu soil strengths sufficient to resist sliding have been available. Since complete soil descriptions and laboratory strength tests are not available, the actual stability and factor of safety against sliding are unknown.

Seepage in the vicinity of the left abutment is clear with no signs of loss of embankment material. Considering the uniformity of the downstream slope and crest on the left side of the dam, the existence of seepage has not visibly affected the embankment.

2. Appurtenent Structures. The concrete overflow spillway appears to be in good condition with only minor cracking, spalling, and slight deterioration. Although flow in the spillway prevented direct observation of construction joints on the floor, some joints are suspected to be open. It could not be determined whether flow was occurring beneath the slabs, but the floor and walls appeared stable and settlement from undermining was not indicated.

b. <u>Design and Construction Data</u>

1. Embankment - The only design and construction data available are the construction drawings. The details shown on the drawings appear to be in accordance with the design practice of the period and reflect the project as completed. The remedial study and grouting work undertaken in 1968 provide subsurface data for at least a portion of the embankment. Split spoon sampling and standard penetration tests indicate the fill materials to be stiff to very stiff silty clays with rock fragments displaying penetration resistances of 15 to 35 blows per foot. Although the blow counts from the penetration tests could be affected by the presence of rock fragments, the test results are consistently high and the fill materials are judged to have been reasonably compacted during placement and subsequent consolidation. The test borings indicate that the fill was placed on rock and dense decomposed rock.

   It should be noted that the silty clay embankment material indicated by the borings of the 1968 study is not a free draining material. As such, pore water pressures in the upstream embankment slope could become high and the factor of safety against shear failure reduced when the reservoir level is lowered rapidly. Ordinarily, upstream embankment slopes constructed of silty clay and subject to rapid drawdown (more than 6 inches of drawdown per day) would be designed at a configuration of between 3 to 4 horizontal to 1 vertical. The design slope configuration of 2 horizontal to 1 vertical is not in accordance with good engineering practice.

2. Appurtenant Structures - The details shown on the construction drawings are in accordance with accepted design practice.

c. <u>Operating Procedures</u>

Detailed operating procedures were unavailable for review. The daily pool level records and withdrawal rates indicate that drawdown on the order of 5 feet per day is possible. Considering the steeper than normal upstream slope and the relatively impervious clayey soils in the upstream zone of embankment, some stability problems could be generated by rapid drawdown.

d. Post Construction Changes

The post construction changes of foundation grouting and replacement of the timber spillway lining with concrete were performed as remedial work and should enhance the stability of the dam.

e. Seismic Stability

Edgemont Reservoir is located adjacent to seismic zone 2 and seismic stability is predicated upon static stability with conventional margins of safety. Since the static stability cannot be fully assessed with the data available, the stability of the dam under seismic activity is unknown.

SECTION 7
ASSESSMENT, REMEDIAL MEASURES AND RECOMMENDATIONS

7.1 Dam Assessment:

a. Safety

Based upon visual inspection and review of design and construction data, the dam at Edgemont Reservoir appears to presently be in good condition. Some potential slope stability problems as addressed in section 2 and section 6 could exist, but do not pose an immediate safety hazard. Preliminary hydrologic and hydraulic analyses indicate the overflow spillway is capable of passing only 17 percent of PMF before the dam is overtopped. Since dam failure will not significantly increase loss of life or property damage in the event of overtopping by PMF, the spillway is judged to be inadequate, but not seriously inadequate.

b. Adequacy of Information

The available information consists of contruction drawings, a remedial grouting study report, limited streamflow and rainfall data, and daily withdrawal rates and pool levels. With the exception of slope stability, this data is generally adequate to assess the project.

c. Urgency

Although immediate action is not required at this time, the recommendations of this section should be implemented as soon as possible.

d. Necessity for additional studies

Due to the inadequacy of the spillway, detailed hydrologic and hydraulic analyses should be performed so that remedial recommendations can be formulated.

7.2 Remedial Measures and Recommendations:

a. Dam and Appurtenant Structures

1. Initiate an engineering study to evaluate spillway capacity and determine remedial measures to reduce the overtopping potential at the dam.

2. The 30-inch drain valve should be restored to an operable condition and should be provided with a positive seal on the upstream side of the dam.

3. The seepage at the left abutment should be monitored on a routine basis to detect any changed conditions in quantity or turbidity. Flow measurements should be made with the use of a weir.

b. Operation and Maintenance Procedures

1. Document operating procedures in writing.

2. Remove woody vegetation from downstream embankment slope.

3. Develop a warning system to warn downstream residents of large spillway discharges during periods of heavy rainfall and runoff or failure of the dam.

APPENDIX A

CHECK LIST - VISUAL INSPECTION, SITE SKETCH, PHASE I

Check List
Visual Inspection
Phase I

Name of Dam Edgemont Reservoir County Washington State Maryland ID# MD-00006

Type of Dam Earth Hazard Category 1

Date(s) of Inspection 4/9/79 Weather Cloudy/Rain Temperature 45°F

Pool Elevation at Time of Inspection 901+ M.S.L. Tailwater 840± M.S.L.

Inspection Personnel:		Owners Representatives:
J. O. Smith		William M. Breichner, Superintendent City of Hagerstown Water Department
T. J. Moynahan		John Harne, Manager, Edgemont Reservoir
D. L. Moore		J. O. Smith, Recorder

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Riprap downstream slope-surface irregularities.	
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Horizontal - good Vertical - approximately 1/2 foot depression in vicinity of observation well #19 approximately 100 feet from right abutment	
RIPRAP FAILURES	Upstream slope - good Downstream slope - see above	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	No erosion noted	
ANY NOTICEABLE SEEPAGE	Flow from left side conveyed via system of tile and plastic pipe. precise location of sources, if any, within dam foundation unknown: flow clear and approximated at 15 gpm.	
STAFF GAGE AND RECORDER	Stakes in pool area used by Manager to record daily pool levels to nearest inch	
DRAINS	No foundation drains on plans or noted in field	

OUTLET WORKS- 30" C.I.P. blow off - drain

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	N/A	
INTAKE STRUCTURE	30 " C.I.P. stub in pool at approximate elevation 850 condition not observable	
OUTLET STRUCTURE	30" geared valve and C.I.P. stub at toe at approximate elevation 840 operating condition unknown	
OUTLET CHANNEL	Debris noted, channel not used for reservoir releases since draining accomplished in 1940's.	
EMERGENCY GATE - Drain	Not operated since 1940's sediment level in	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Spillway consists of 25' wide rectangular concrete channel approximately 550 ft. long. Adverse approach slope, 25 ft. long @ 4%. Exit slope, approx. 125 ft. long @ 1%. Remainder of channel conforms to steep preconstruction natural grades and exits at approx. elevation 845.	
APPROACH CHANNEL	Clear of debris, concrete channel in good condition.	
DISCHARGE CHANNEL	Clear of debris, concrete channel in good condition slight spalling on side walls noted bituminous joint filler applied at construction joints approx. 2 yrs. ago. No differential movement or cracking noted.	
BRIDGE AND PIERS	Good condition.	

INSTRUMENTATION

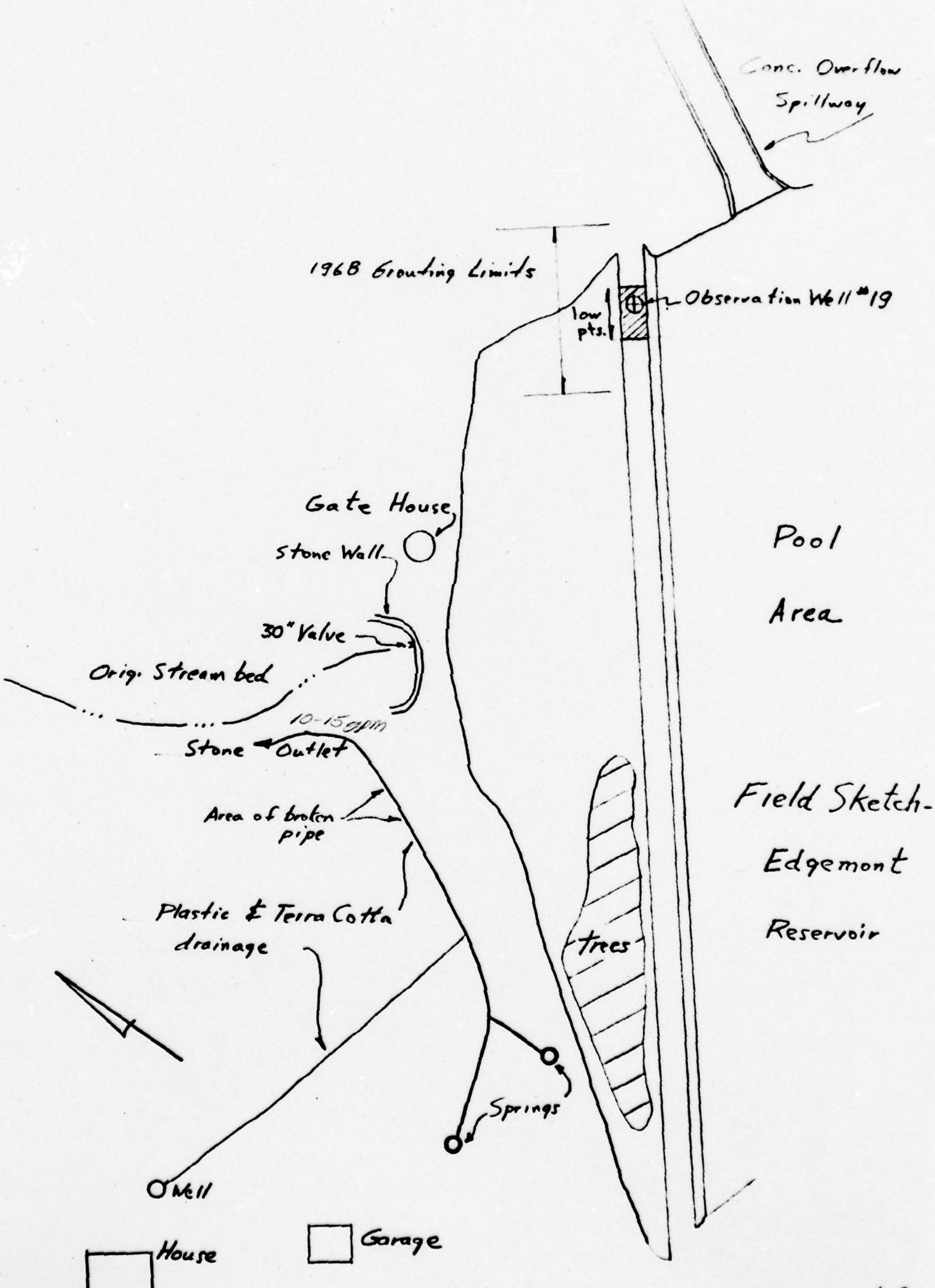
ISUAL EXAMINATION OF ONUMENTATION/SURVEYS	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
	None noted	
BSERVATION WELLS	Observation well number 19 near right abutment remains in place from 1968 remedial grouting readings not recorded since 1968 study.	
EIRS	None	
IEZOMETERS	None	

RESERVOIR

ISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
LOPES	Steep and wooded. No movement noted.	
EDIMENTATION	Not a great problem according to resident Manager. Reservoir drained and cleaned in 1940's. Volume removed unknown. 1976 police search by divers reported clear conditions on bottom.	

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Confluence of Warner Gap Hollow and Raven Rocks stream form Little Antietam Creek; boulder and cobble bed load deposits noted.	
SLOPES	Some degradation noted.	
APPROXIMATE NO. OF HOMES AND POPULATION	5 to 10 homes and some comercial in danger reach.	



Conc. Overflow
Spillway
1968 Grouting Limits
low pts.
Observation Well #19
Gate House
Stone Wall
30" Valve
Orig. Stream bed
10-15gpm
Stone
Outlet
Area of broken pipe
Plastic & Terra Cotta drainage
Springs
Well
trees
House
Garage
Pool
Area
Field Sketch-
Edgemont
Reservoir

APPENDIX B

CHECK LIST - ENGINEERING DATA, DESIGN, CONSTRUCTION, OPERATION,

PHASE I

DAM NAME: Edgemont Reservoir
ID # MD 00006

CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 2.34 mi^2, City owns 907 acres moderate to steeply sloping valley walls, heavily wooded.

ELEVATION TOP OF NORMAL POOL (STORAGE CAPACITY): 901.0 (246 Ac-Ft)

ELEVATION TOP OF FLOOD CONTROL POOL (STORAGE CAPACITY): -----

ELEVATION MAXIMUM DESIGN POOL: ---------------------------------

ELEVATION TOP OF DAM: 905 (296 Ac-Ft), low pt. 904.65 (292 Ac-Ft)

CRESTS

a. Elevation 901.0
b. Type Concrete, rectangular channel
c. Width 25 feet
d. Length
e. Location Spillover Discharges parallel to dam axis beyond rt. abutment.
f. Number and Type of Gates None

OUTLET WORKS:

a. Type Two 12" C.I.P. water supply mains
b. Location Near right abutment
c. Entrance Inverts
d. Exit Inverts
e. Emergency Drawdown Facilities 30" C.I.P. inoperable

HYDROMETEOROLOGICAL GAGES:

a. Type Standard 8 inch precipitation gage (non-recording)
b. Location Immediately below dam at site superintendant's residence
c. Records Daily totals phoned to National Weather Service

MAXIMUM NON-DAMAGING DISCHARGES:

ITEM		REMARKS
SPILLWAY PLAN		
SECTIONS		Yellow pine flooring and sides of spillway is not present, allegedly due to damages sustained in flood of March, 1936 spillway is smoothly finished concrete.
	Available	
DETAILS		
OPERATING EQUIPMENT PLANS & DETAILS	No operating equipment	

Avialable plans and details are on three plans sheets dated October 1, 1968 which were traced from original plan sheets dated January 1902 by the American Pipe Manufacturing Company, Engineers and Contractors of Philadelphia, Penna.

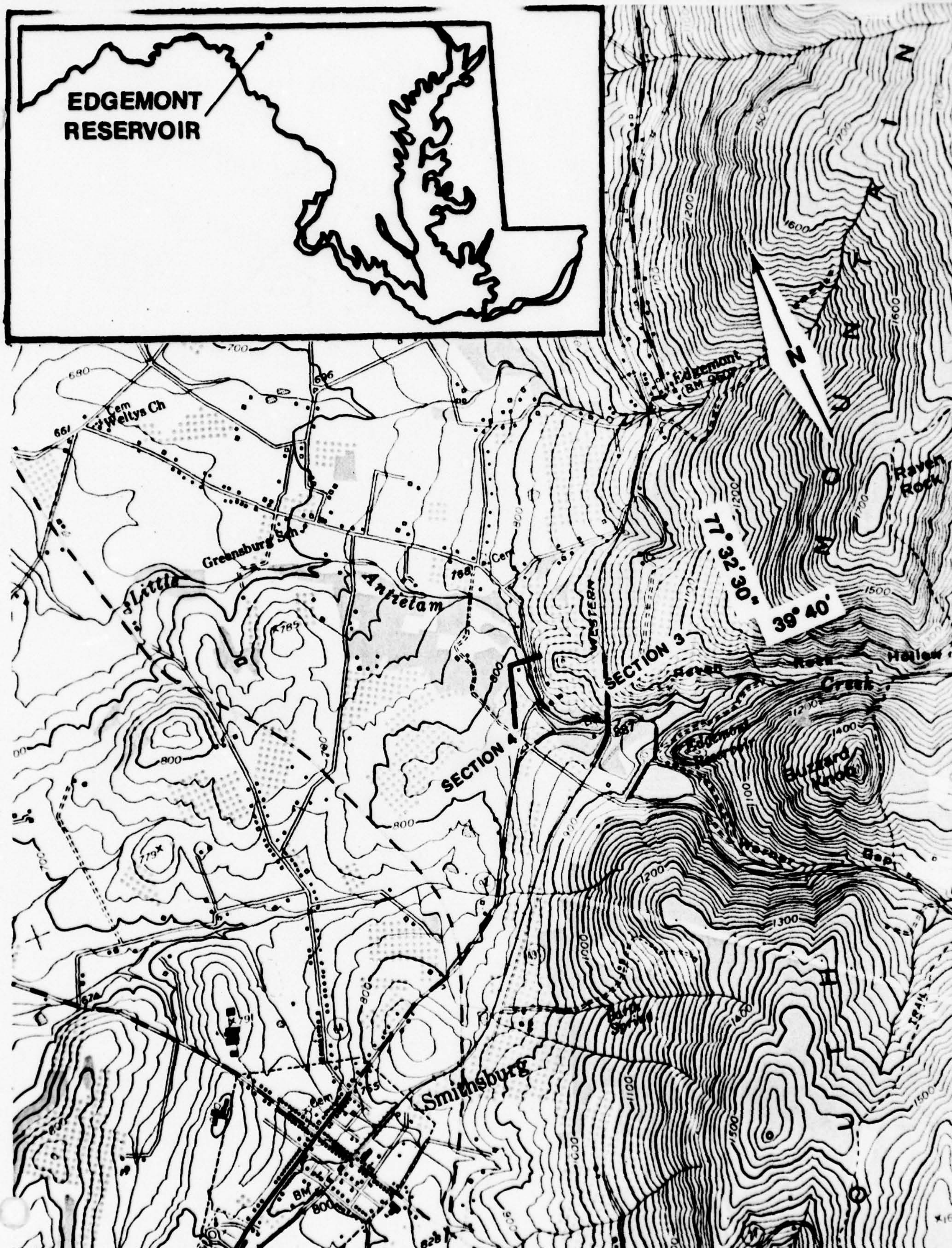
ITEM	REMARKS
MONITORING SYSTEMS	Well #19 from 1968 grouting program. No data available since 1968.
MODIFICATION	No known modifications.
HIGH POOL RECORDS	Daily pool levels maintained by resident superintendent and recorded at Hagerstown Water Department offices 790-3200 x 218.
POST CONSTRUCTION ENGINEERING STUDIES & REPORTS	"Edgemont Reservoir Investigation of Leakage with Remedial Grouting" dated November, 1968 by Whitman, Reguardt and Associates.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	Apparent internal erosion failure near right abutment in late 1930's. No technical reports available.
MAINTENANCE OPERATION RECORDS	Records of 1968 grouting program.

ITEM	REMARKS
DESIGN REPORTS	None
GEOLOGY	None
DESIGN COMPUTATIONS	None
HYDROLOGY & HYDRAULICS	None
DAM STABILITY	None
SEEPAGE STUDIES	No original, *foundation seepage investigated* in 1968
MATERIALS INVESTIGATIONS	Test pits mentioned on plans, but information not available
BORING RECORDS	None
LABORATORY	None 23 borings taken thru embankment and into foundation, May - July, 1968
FIELD	None
POST CONSTRUCTION SURVEY OF DAM	None
BORROW SOURCES	Thought to be in present pool

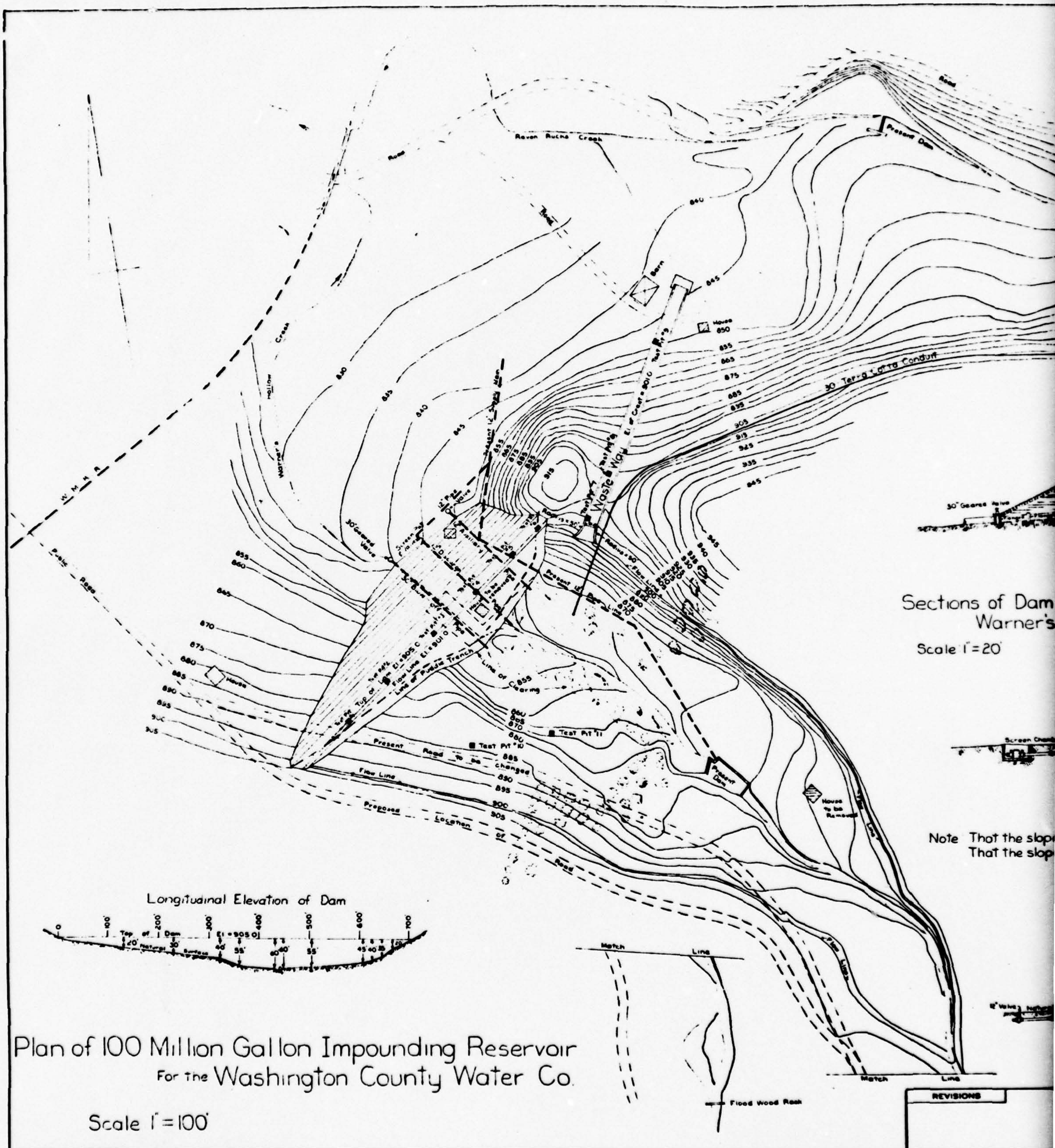
ITEM	REMARKS
AS BUILT DRAWINGS	None
REGIONAL VICINITY MAP	Available
CONSTRUCTION HISTORY	None
TYPICAL SECTIONS OF DAM	Available, note, upstream and downstream slope designators are correct, but drawing is reversed. i.e. the steeper slope, 1½H:1V, is the downstream slope.
OUTLETS - PLANS	Available
- DETAILS	Not Available
-CONSTRAINTS	Drain not operable, additional valves have been added to by-pass gate house
- DISCHARGE RATINGS	NoT Available
RAINFALL/RESERVOIR RECORDS	Daily records kept at Hagerstown Water Department Edgemont Gage- 40 yrs. of record.

APPENDIX C

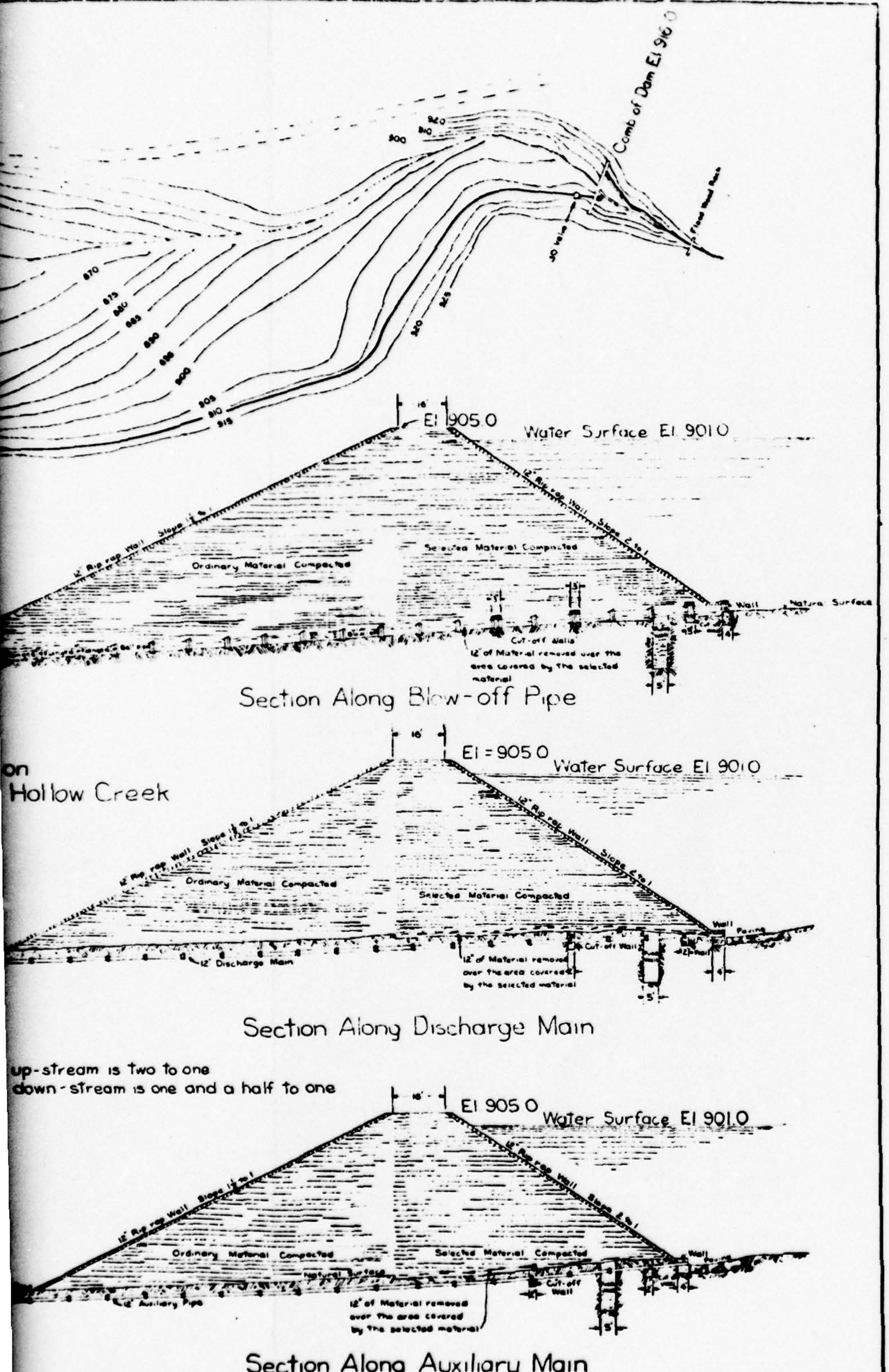
LOCATION MAP & PLANS


EDGEMONT
RESERVOIR
SECTION 3
SECTION 4
Edgemont
Raven Rock
Buzzard Knob
Smithsburg
Greensburg Sch
Welty Ch
Little
Antietam
Beaver
77° 32' 30"
39° 40'
N
LOCATION MAP
SCALE 1:24000

C-1

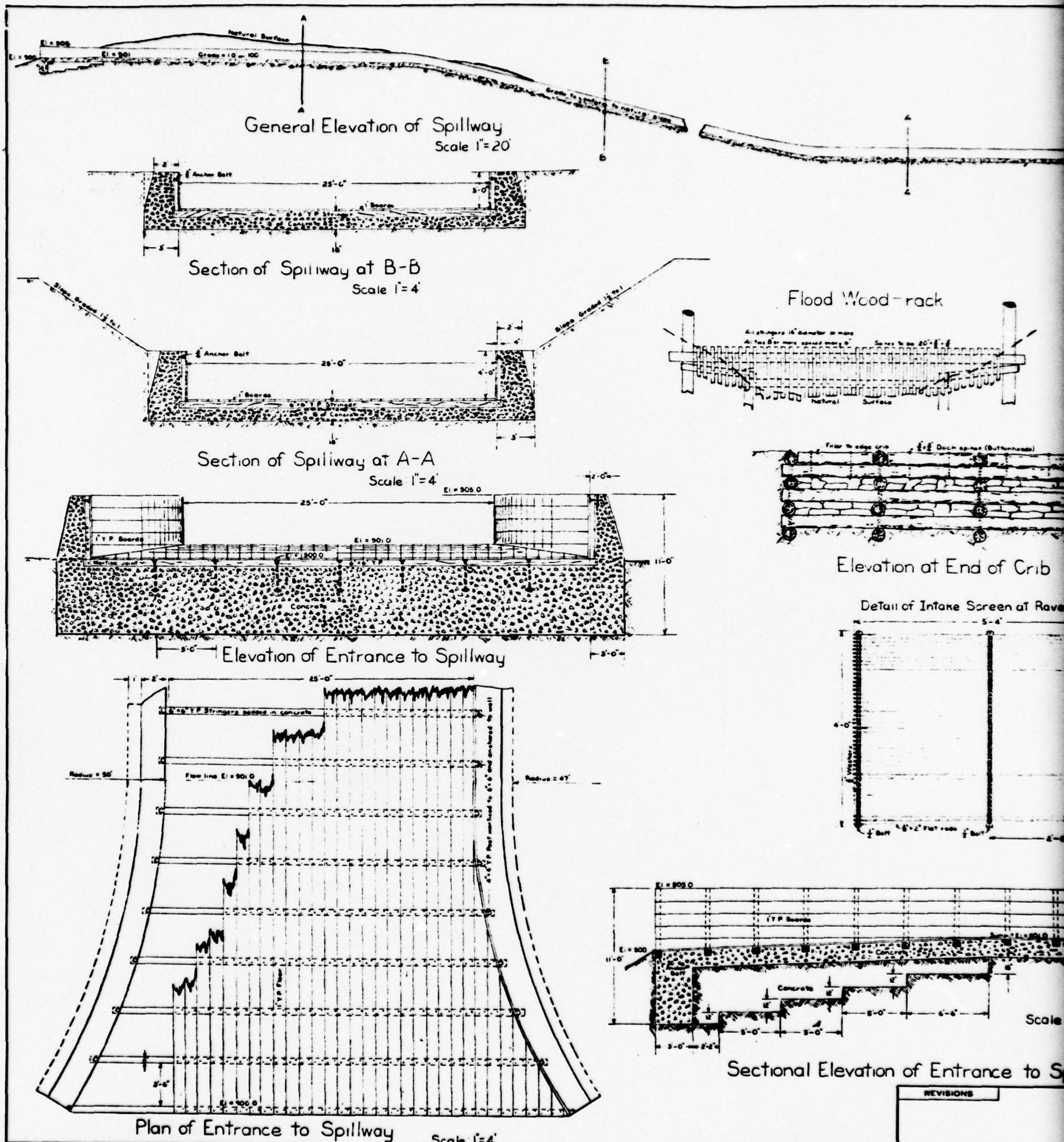


Plan of 100 Million Gallon Impounding Reservoir
For the Washington County Water Co.
Scale 1" = 100'
Longitudinal Elevation of Dam
Sections of Dam
Warner's
Scale 1" = 20'
Note That the slop
That the slop
Present Dam
Raven Rucks Creek
Hollow Creek
Warner's
W. M. R. R.
Public Road
House
Barn
Waste Way
30" Terra Cotta Conduit
Present Road to be changed
Flow Line
Proposed Location of Road
Test Pit 10
Test Pit 11
Line of Clearing
Line of Puddle Trench
House to be Removed
Match Line
Flood Wood Rack
REVISIONS



TITLE 100 Million Gallon Impounding Reservoir	CITY OF HAGERSTOWN, MARYLAND BOARD OF WATER COMMISSIONERS	
LOCATION Edgemont Reservoir		FILE NUMBER
SCALE As Shown DRAWN M.L.S. DATE Oct. 1, 1968	APPROVED SUPERINTENDENT DATE	6-3

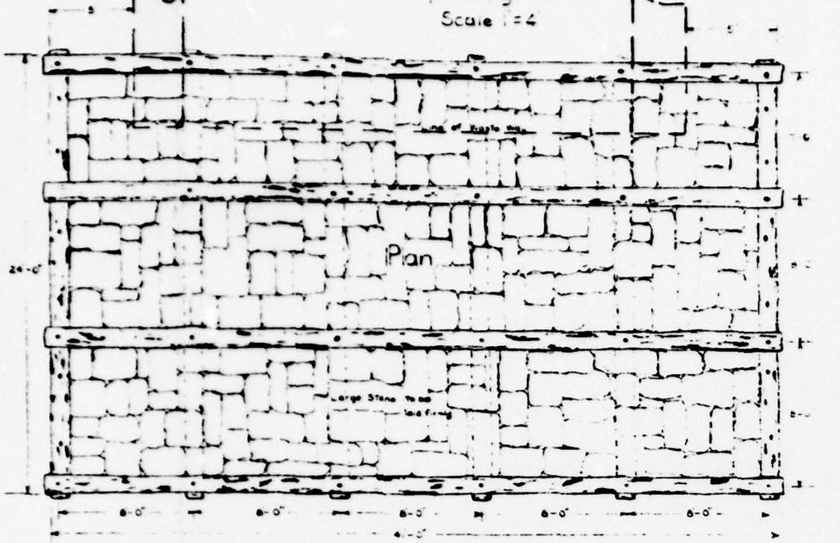
2



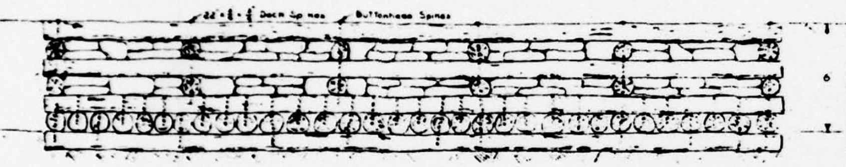
Natural Surface
El = 901
General Elevation of Spillway
Scale 1"=20'
25'-0"
Section of Spillway at B-B
Scale 1"=4'
Flood Wood-rack
natural Surface
Anchor Bolt
Boards
Section of Spillway at A-A
Scale 1"=4'
El = 905.0
El = 901.0
Concrete
11'-0"
Elevation of Entrance to Spillway
Elevation at End of Crib
Detail of Intake Screen at Rave
5'-4"
4'-0"
Flow line El = 901.0
Radius = 50'
Radius = 47'
El = 900.0
Plan of Entrance to Spillway
Scale 1"=4'
1"T P Boards
Scale
Sectional Elevation of Entrance to Sp
REVISIONS

1

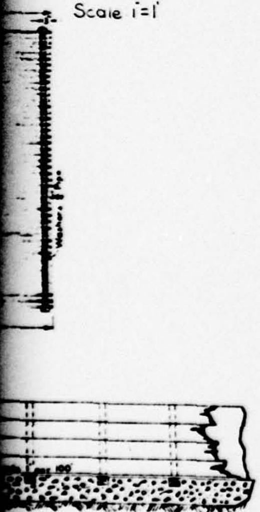
Crib at End of Spillway
Scale 1"=4'



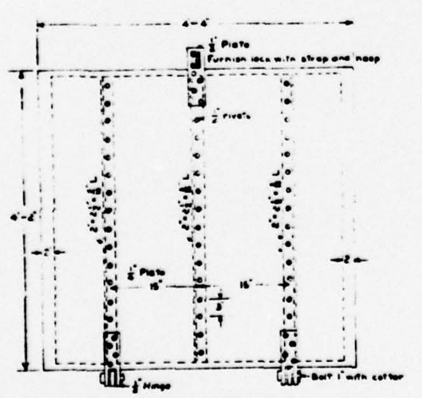
Front Elevation



...ks Creek Dam
Scale 1"=1'

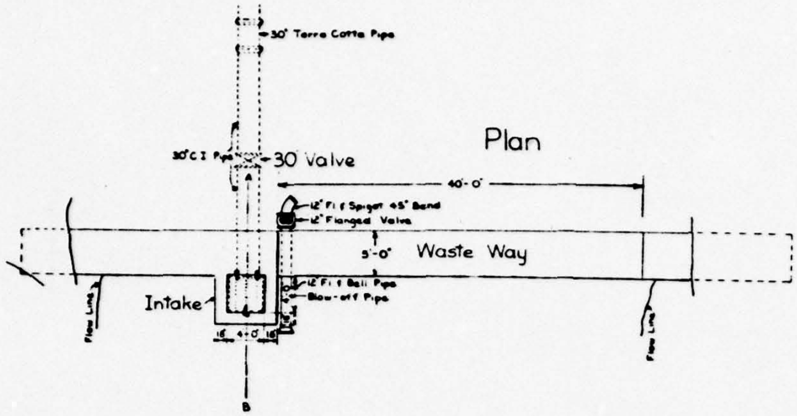


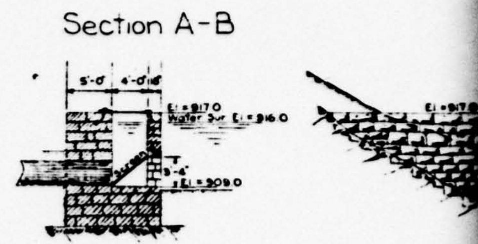
...lway



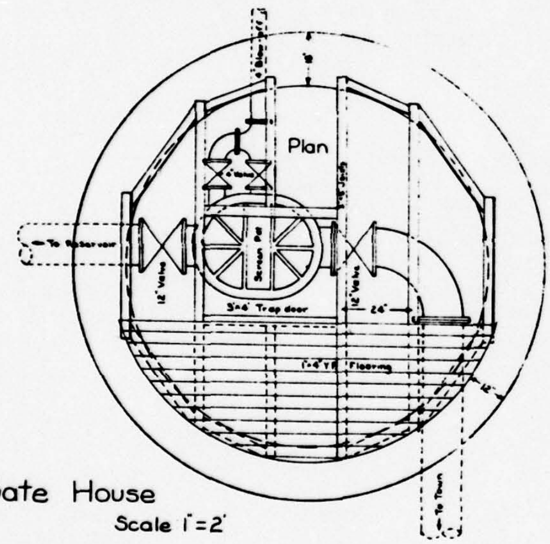
Detail of Iron Door on Intake
at Raven Rocks Creek Dam
Scale 1"=1'

TITLE 100 Million Gallon Impounding Reservoir	CITY OF HAGERSTOWN, MARYLAND BOARD OF WATER COMMISSIONERS
LOCATION Edgemont Reservoir	APPROVED ______ SUPERINTENDENT ______ DATE
SCALE As Shown DRAWN BY M.L.S DATE Oct 1, 1966	FILE NUMBER C-3

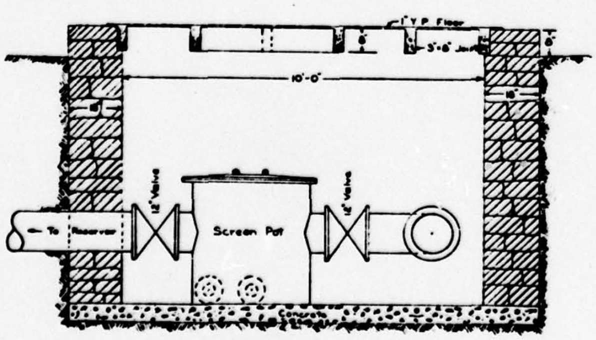




Details of Dam on Raven Rocks Creek
Scale 1"=8'



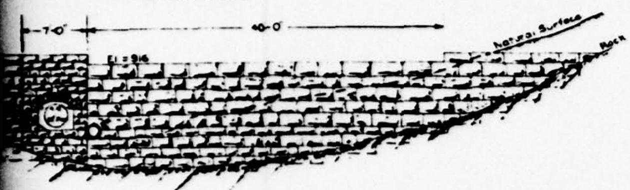
Gate House
Scale 1"=2'



Elevation

REVISIONS

Elevation



TITLE 100 Million Gallon Impounding Reservoir	CITY OF HAGERSTOWN, MARYLAND BOARD OF WATER COMMISSIONERS
LOCATION Edgemont Reservoir	APPROVED ________ SUPERINTENDENT ________ DATE — FILE NUMBER 6-3
SCALE As Shown DRAWN BY M.L.S. DATE Oct. 1, 1966	

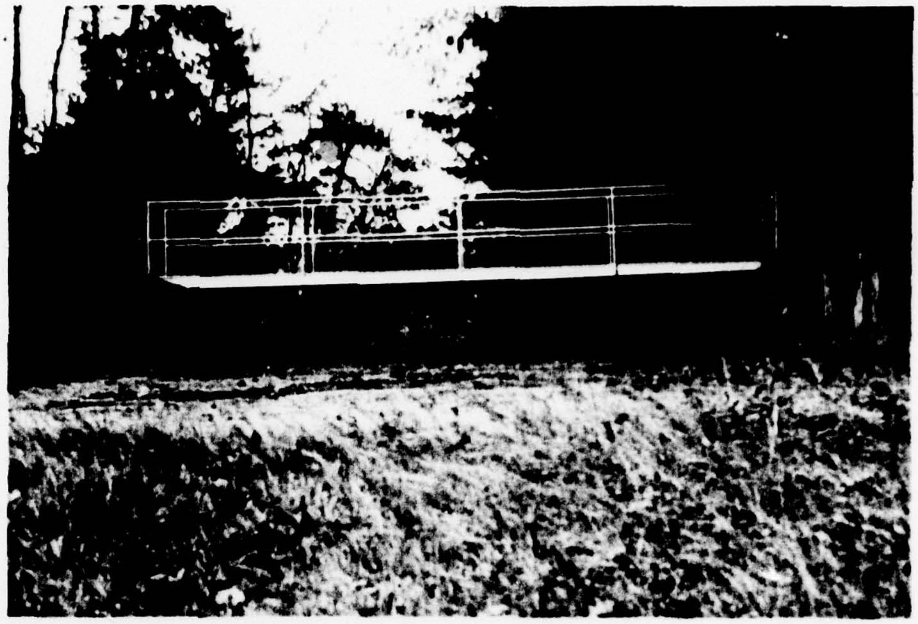
2

APPENDIX D

PHOTOGRAPHS

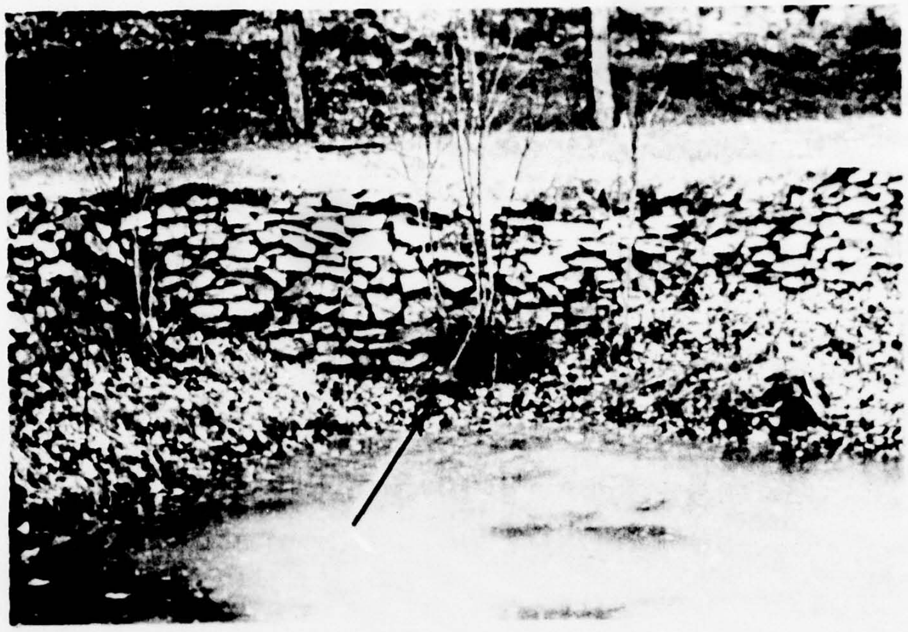
CREST

DOWNSTREAM FACE-VIEWED FROM
RIGHT ABUTMENT



OVERFLOW SPILLWAY-ENTRANCE

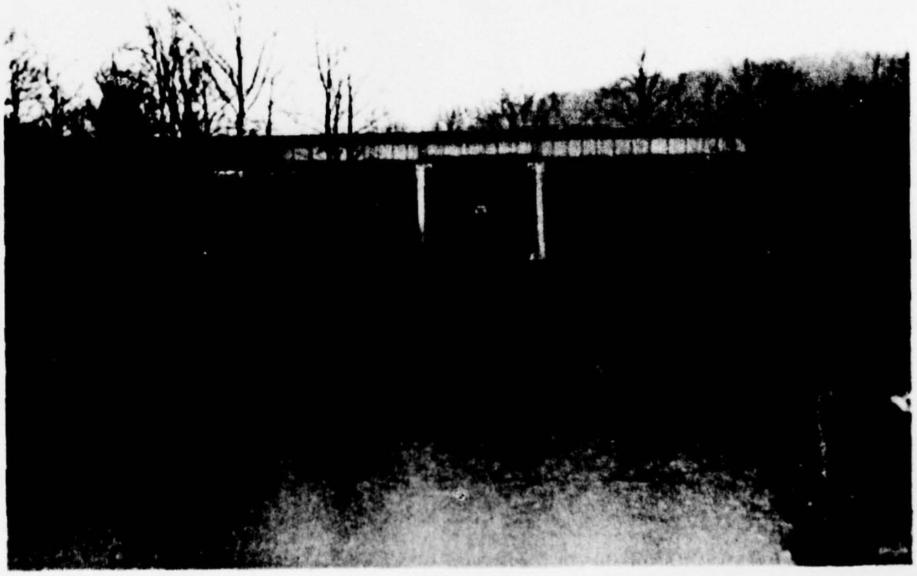
OVERFLOW SPILLWAY



30" BLOW-OFF (DRAIN)

SEEPAGE, LEFT SIDE BEYOND TOE

GATE HOUSE



DOWNSTREAM R.R.CROSSING
(STATION #3 OF
HYDRAULIC ROUTINGS)

DOWNSTREAM COMMERCIAL

DOWNSTREAM DWELLING
(STATION #4 OF
HYDRAULIC ROUTINGS)

APPENDIX E

ANALYSES

Contents

Elev.	Area, in²	Area, Ac	Avg. Area	ΔVol, Ac-Ft	Cumul. Vol.
855	1.85	0.425	–	0	0
860	3.47	0.797	0.611	3.05	3.05
865	7.94	1.82	1.31	6.55	9.6
870	12.19	2.80	2.31	11.5	21.1
875	18.22	4.18	3.49	17.45	38.55
880	23.98	5.50	4.84	24.2	62.75
885	31.09	7.13	6.32	31.6	94.35
890	37.0	8.49	7.81	39.05	133.4
895	42.52	9.76	9.12	45.6	179.0
900	51.77	11.9	10.83	54.15	233.1
901					246 interpolated
905	58.8	13.5	12.7	63.5	296.65
910	Est.	15	14.25	71.25	367.9

Edgemont

Stage-Area-Volume

from 1"=100ft. plan sheet (see C-2)

Stage	Depth	Area, Ft^2	Wet. Perim.	Hyd. Radius	$R^{1/3}$	$R^{4/3}$
901	0	0	0	0	0	0
901.5	.5	12.5	26	.48	.61	.38
902	1	25	27	.93	.95	.90
902.5	1.5	37.5	28	1.34	1.22	1.47
903	2	50	29	1.72	1.44	2.06
903.5	2.5	62.5	30	2.08	1.64	2.65
904	3	75	31	2.42	1.81	3.24
904.5	3.5	87.5	32	2.73	1.96	3.81
905	4	100	33	3.03	2.10	4.37
906	5	125	35	3.57	2.35	5.44
907	6	150	37	4.05	2.55	6.43
908	7	175	39	4.49	2.73	7.36

Spillway Data

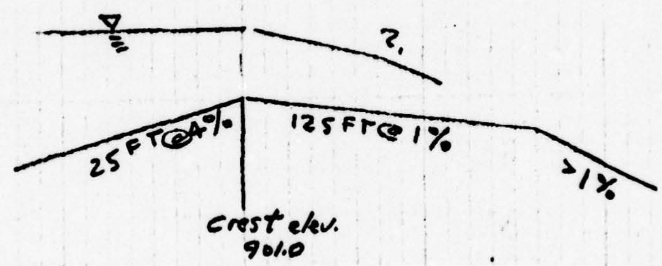
Crest elev. - 901.0

adverse entrance slope, 4%, length, 25 FT

exit slope, 1%, length, 125 FT

remaining length, 400 FT, >1%

width = 25 FT, vertical sides



Edgemont Reservoir
Stage-Discharge Calcs

Spillway Capacity at top of Dam

at stage 905, $d = 4$, $A = 100$, $T = 25$, $R^{4/3} = 4.37$

Using: $Q^2/g = A^3/T$, S_c, critical slope, $= \dfrac{14.56 n^2 d_m}{R^{4/3}}$

$d_m = Q_c^2/A^2 g$ all referenced from Soil Cons. Service

NEH-5 p. 5.4-8 and 10

Assuming $n = 0.013$ for concrete

$Q_c = (gA^3/T)^{1/2} = (32.2 \times (100)^3/25)^{1/2} = 1135$ cfs

$d_m = Q_c^2/A^2 g = (1135)^2/(100)^2\, 32.2 = 4$ FT

$S_c = \dfrac{14.56 n^2 d_m}{R^{4/3}} = 14.56\,(0.013)^2\, 4/4.37 = 0.0023$ FT/FT

$0.01 > 0.0023$ ∴ critical depth

occurs at control & discharge $= Q_c = 1135$ cfs

Overtopping Discharge thru Spillway

at stage 908, $d = 7$, $A = 175$, $T = 25$, $R^{4/3} = 7.36$

$Q_c = (32.2\,(175)^3/25)^{1/2} = 2627$ cfs

$d_m = (2627)^2/(175)^2\, 32.2 = 7$ FT

$S_c = 14.56\,(0.013)^2\, 7/7.36 = 0.0023 < 0.01$ OK

$Q = Q_c = 2627$ cfs

Additional Discharges - Spillway

at stage 902, $d = 1$, $A = 25$, $T = 25$, $R^{4/3} = 0.90$

$Q_c = (32.2 (25)^3/25)^{1/2} = 142$ cfs

$d_m = (142)^2/(25)^2 32.2 = 1$ FT

$S_c = 14.56 (0.013)^2 1/0.90 = 0.0027 < 0.01$ ok

$Q = Q_c = 142$ cfs

at stage 903, $d = 2$, $A = 50$, $T = 25$, $R^{4/3} = 2.06$

$Q_c = (32.2 (50)^3/25)^{1/2} = 401$ cfs

$d_m = 2$ FT [$d_m = d$ due to channel uniformity]

$S_c = 14.56 (0.013)^2 2/2.06 = 0.0024 < 0.01$ OK

$Q = Q_c = 401$ cfs

at stage 904, $d = 3$, $A = 75$, $T = 25$, $R^{4/3} = 3.24$

$Q_c = (32.2 (75)^3/25)^{\frac{1}{2}} = 737$ cfs

$d_m = 3$, $S_c = 14.56 (0.013)^2 3/3.24 = 0.023 < 0.01$ OK

$Q = Q_c = 737$ cfs

at stage 904.5, $d = 3.5$, $A = 87.5$, $T = 25$, $R^{4/3} = 3.81$

$Q_c = (32.2 (87.5)^3/25)^{1/2} = 929$ cfs

$d_m = 3.5$, $S_c = 14.56 (0.013)^2 3.5/3.81 = 0.0023 < 0.01$ OK

$Q = Q_c = 929$ cfs

Spillway Stage - Discharge Summary

	Stage, MSL	Discharge, cfs
spillway crest	901	0
	902	142
	903	401
	904	737
approx. low pt.	904.5	929
dam crest	905	1135
	908	2627

Snyders UH Coefficients

using Balto. District regional curves

Zone 32 → $C_p = 0.75$

C_t = Plate K, Potomac River West of Monocacy

use $t_p = 1.90\,(L L_{ca})^{0.3} = 2.95$

where $L = 7.4" \times 2000 \frac{FT}{in} = 14800\,FT = 2.8$ mile

$L_{ca} = 4.1" \times 2000 = 8200 = 1.55$ mile

From Hydromet 33, PMP Index = 23.7 inch, Zone 6

read R_6 R_{12} R_{24} from 10 mi² (pt. rainfall)

113% 123% 132%

```
*********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION       JULY 1978
  LAST MODIFICATION  26 FEB 79
*********************************
   1      A1      SNYDER UNIT HYDROGRAPH,FLOOD ROUTING,AND DAM OVERTOPPING        ANALYSIS
   2      A2      WARNERS HOLLOW CREEK DAM,WASHINGTON CO.,MD. N.D.I. MD00006
   3      A3      FOR 10%,20%,30%,40%,50%,60%,70%,80%,AND 100% PMF
   4      B     160       0      15       0       0       0       0       0      -4       0
   5      B1      5
   6      J       1       9      .1
   7      J1    .10     .20     .30     .40     .50     .60     .70     .80    1.00
   8      K       0       1                                       1
   9      K1      CALCULATION OF PMF RATIOS TO WARNERS HOLLOW CREEK DAM
  10      M       1       1    2.34            2.34                                 1
  11      P            23.7     113     123     132
  12      T                                                       1     .05
  13      W    2.95    0.75
  14      X    -1.0   -0.05     2.0
  15      K       1       2                                       1
  16      K1      ROUTED FLOWS THROUGH EDGEMONT RESERVOIR
  17      Y                               1       1
  18      Y1      1                                             246      -1
  19      Y4    901     902     903     904   904.5     905     908
  20      Y5      0     142     401     737     929    1135    2627
  21      $S      0     9.6    38.5    94.3   133.4   179.0   233.1   246.0   296.6   367.9
  22      $E    855     865     875     885     890     895     900     901     905     910
  23      $$    901
  24      $D904.65      3.1     1.5     700
  25      $L      0      50     700
  26      $V904.65    904.9     905
  27      K      99
```

```
**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION        JULY 1978
 LAST MODIFICATION  26 FEB 79
**********************************

RUN  DATE* 79/04/23.
     TIME* 08.47.58.

                    SNYDER UNIT HYDROGRAPH,FLOOD ROUTING,AND DAM OVERTOPPING        ANALYSIS
                    WARNERS HOLLOW CREEK DAM,WASHINGTON CO.,MD. N.D.I. MD00006
                    FOR 10%,20%,30%,40%,50%,60%,70%,80%,AND 100% PMF

                                 JOB SPECIFICATION
          NQ     NHR    NMIN    IDAY     IHR    IMIN   METRC    IPLT    IPRT   NSTAN
         160       0      15       0       0       0       0       0      -4       0
                                 JOPER     NWT   LROPT   TRACE
                                     5       0       0       0

                         MULTI-PLAN ANALYSES TO BE PERFORMED
                              NPLAN= 1 NRTIO= 9 LRTIO= 1
          RTIOS=     .10     .20     .30     .40     .50     .60     .70     .80    1.00

     **********      **********      **********      **********      **********

                              SUB-AREA RUNOFF COMPUTATION

               CALCULATION OF PMF RATIOS TO WARNERS HOLLOW CREEK DAM

                    ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                        1       0       0       0       0       0       1       0       0

                                    HYDROGRAPH DATA
          IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
              1       1    2.34    0.00    2.34    0.00   0.000       0       1       0

                                     PRECIP DATA
                  SPFE     PMS      R6     R12     R24     R48     R72     R96
                  0.00   23.70  113.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS  .800

                                      LOSS DATA
     LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
         0    0.00    0.00    1.00    0.00    0.00    1.00    1.00     .05    0.00    0.00
```

```
                                   UNIT HYDROGRAPH DATA
                              TP=  2.95    CP= .75    NTA=  0

                                   RECESSION DATA
                         STRTQ=    -1.00    ORCSN=    -.05    RTIOR= 2.00

                 UNIT HYDROGRAPH 46 END-OF-PERIOD ORDINATES, LAG=   2.95 HOURS, CP=  .75  VOL= 1.00
         10.        38.        77.       120.       167.       215.       263.       308.       346.       372.
        387.       392.       387.       370.       336.       293.       255.       222.       193.       169.
        146.       127.       110.        96.        83.        73.        63.        55.        48.        41.
         36.        31.        27.        24.        21.        18.        16.        14.        12.        10.


          9.         8.         7.         6.         5.         4.

    0                                    END-OF-PERIOD FLOW
 MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP O       MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP O

                                                                  SUM  25.03  23.18   1.65   141613.
                                                                      ( 636.)( 589.)(  47.)( 4010.03)

                                        HYDROGRAPH ROUTING

                         ROUTED FLOWS THROUGH EDGEMONT RESERVOIR

                                ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                                  2       1       0       0        0       0      1       0       0
                                                    ROUTING DATA
                        QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP           LSTR
                          0.0   0.000    0.00      1       1       0       0              0

                                NSTPS   NSTOL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                                  1       0       0    0.000   0.000   0.000   246.     -1

   STAGE     901.00     902.00     903.00     904.00     904.50     905.00     909.00

   FLOW        0.00     142.00     401.00     737.00     929.00    1135.00    2627.00

    CAPACITY=      0.        10.        39.        94.       133.       179.       233.       246.       297.       368.

   ELEVATION=    855.       865.       875.       885.       890.       895.       900.       901.       905.       910.

                           CREL   SPWID   COQW   EXPW   ELEVL   COOL   CAREA   EXPL
                          901.0     0.0    0.0    0.0     0.0    0.0     0.0    0.0

                                                   DAM DATA
                                        TOPEL   COQD   EXPD   DAMWID
                                        904.7    3.1    1.5    700.

   CREST LENGTH         0.        90.       700.
   AT OR BELOW
   ELEVATION        904.7      904.9      905.0
```

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIOS APPLIED TO FLOWS RATIO 1 .10	RATIO 2 .20	RATIO 3 .30	RATIO 4 .40	RATIO 5 .50	RATIO 6 .60	RATIO 7 .70	RATIO 8 .80	RATIO 9 1.00
HYDROGRAPH AT	1	2.34	1	592.	1184.	1776.	2368.	2960.	3552.	4144.	4736.	5920.
		(6.06)		(16.76)(	33.53)(	50.29)(	67.06)(	83.82)(	100.58)(	117.35)(	134.11)(	167.64)
ROUTED TO	2	2.34	1	577.	1177.	1775.	2367.	2960.	3552.	4144.	4736.	5920.
		(6.06)		(16.33)(	33.32)(	50.26)(	67.04)(	83.81)(	100.58)(	117.34)(	134.11)(	167.65)

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	901.00	901.00	904.65
STORAGE	246.	246.	292.
OUTFLOW	0.	0.	991.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.10	903.52	0.00	278.	577.	0.00	19.00	0.00
.20	905.00	.35	297.	1177.	2.25	18.75	0.00
.30	905.31	.66	301.	1775.	4.25	18.50	0.00
.40	905.52	.87	304.	2367.	5.50	18.50	0.00
.50	905.71	1.06	307.	2960.	6.50	18.50	0.00
.60	905.88	1.23	309.	3552.	7.25	18.50	0.00
.70	906.04	1.39	311.	4144.	7.75	18.50	0.00
.80	906.18	1.53	313.	4736.	8.00	18.50	0.00
1.00	906.46	1.81	317.	5920.	9.00	18.50	0.00

Overtopping Velocities

thru low section: assume parabolic shape, topwidth = 50 FT
@ 0.2 PMF stage = 905.0 depth = 0.35 FT

$$Area = 2/3(0.35)50 = 11.67\ FT^2$$

$$Q_c = \left(32.2\ (11.67)^3/50\right)^{1/2} = 32\ cfs$$

$$V = Q/A = 32/11.67 = 2.7\ fps$$

over dam @ 0.5 PMF, stage = 905.71

$$Area = 700\ FT \times 0.71\ FT = 497\ FT^2$$

$$Q_c = \left(32.2\ (497)^3/700\right)^{1/2} = 2376\ cfs$$

$$V = Q/A = 2376/497 = 4.8\ fps$$ for 6.5 hours

velocities are high enough to assume an erosional failure would begin during $\frac{1}{2}$ PMF

∴ run breach analysis to assess the degree of spillway inadequacy

Breach Parameters

as suggested by HEC-1 Dam Safety Version
& file information

Breach Width : ht. of dam = 60 FT

$\frac{D}{2}$ 60/2 < breach width < 180 $3D$

or 30 < breach width < 180

USE 100 FT

Side Slope : 0 < Z < 1 USE Z = 1

Failure Time : 0.5 < TFAIL < 4

USE TFAIL = 2 hrs. based upon

rock slope protection & engineered fill

Failure Elevation: USE 905.65 for FAILEL

or 1 FT above low pt. or 0.65 FT

above nominal dam crest

Run Multi-plan analysis @ 50% PMF

with 1 plan allowing a breach and 1 plan

not allowing a breach, so that downstream

stages and time to peaks can be compared in the

assessment of spillway inadequacy — seriously inadequate or not

```
**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION       JULY 1978
 LAST MODIFICATION  26 FEB 79
**********************************
A1        SNYDER UNIT HYDROGRAPH,FLOOD ROUTING,AND DAM BREACH       ANALYSIS
A2        WARNERS HOLLOW CREEK DAM,WASHINGTON CO.,MD. N.D.I. MD00006
A3        FOR 50% PMF PLAN 1 IS FAILURE, PLAN 2 IS NO BREACH
B     160       0      15       0       0       0       0       0      -4       0
B1      5
J       2       1       1
J1    .50
K       0       1                                       1
K1        CALCULATION OF PMF RATIOS TO WARNERS HOLLOW CREEK DAM
M       1       1    2.34            2.34                               1
P            23.7     113     123     132
T                                                       1     .05
W    2.95    0.75
X    -1.0   -0.05     2.0
K       1       2                       2               1
K1        ROUTED FLOWS THROUGH EDGEMONT RESERVOIR
Y                               1       1
Y1      1                                             246      -1
Y4    901     902     903     904   904.5     905     908
Y5      0     142     401     737     929    1135    2627
$S      0     9.6    38.5    94.3   133.4   179.0   233.1   246.0   296.6   367.9
$E    855     865     875     885     890     895     900     901     905     910
$$    901
$D904.65      3.1     1.5     700
$L      0      50     700
$V904.65    904.9     905
$B    100       1     860       2     901  905.65
$B    100       1     860       2     901  906.00
K       1       3
K1        CHANNEL ROUTING-MOD PULS REACH 2-3
Y                               1       1
Y1      1
Y6    .04     .03     .04     835     880    1000     .02
Y7      0     880      20     860      35     845      45     835      75     835
Y7     76     845      96     845     130     880
K       1       4
K1        CHANNEL ROUTING-MOD PULS REACH 3-4
Y                               1       1
Y1      1
Y6    .04     .03     .04     780     830    1500    .037
Y7      0     830     800     820    1400     783    1401     780    1416     780
Y7   1417     783    1437     783    1540     830
K      99
```

E-14

```
**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION       JULY 1978
  LAST MODIFICATION  26 FEB 79
**********************************

RUN  DATE* 79/04/23.
     TIME* 09.02.35.

            SNYDER UNIT HYDROGRAPH,FLOOD ROUTING,AND DAM BREACH      ANALYSIS
            WARNERS HOLLOW CREEK DAM.WASHINGTON CO.,MD. N.D.I. MD00006
            FOR 50% PMF PLAN 1 IS FAILURE. PLAN 2 IS NO BREACH

                                   JOB SPECIFICATION
      NO      NHR     NMIN     IDAY      IHR     IMIN    METRC     IPLT     IPRT    NSTAN
     160        0       15        0        0        0        0        0       -4        0
                              JOPER      NWT    LROPT    TRACE
                                  5        0        0        0

                           MULTI-PLAN ANALYSES TO BE PERFORMED
                               NPLAN= 2 NRTIO= 1 LRTIO= 1
     RTIOS=     .50
```

```
**********          **********          **********          **********          **********

                                  SUB-AREA RUNOFF COMPUTATION

                        CALCULATION OF PMF RATIOS TO WARNERS HOLLOW CREEK DAM

                      ISTAQ    ICOMP    IECON    ITAPE     JPLT     JPRT    INAME   ISTAGE   IAUTO
                          1        0        0        0        0        0        1        0       0

                                          HYDROGRAPH DATA
          IHYDG    IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO    ISNOW    ISAME    LOCAL
              1       1     2.34     0.00     2.34     0.00    0.000        0        1        0

                                            PRECIP DATA
                     SPFE      PMS       R6      R12      R24      R48      R72      R96
                     0.00    23.70   113.00   123.00   132.00     0.00     0.00     0.00
TRSPC COMPUTED BY THE PROGRAM IS   .800

                                             LOSS DATA
        LROPT   STRKR   DLTKR   RTIOL   FRAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
            0    0.00    0.00    1.00    0.00    0.00    1.00    1.00     .05    0.00    0.00

                                        UNIT HYDROGRAPH DATA
                                   TP=  2.95     CP=  .75     NTA=  0

                                           RECESSION DATA
                          STRTO=    -1.00    ORCSN=     -.05    RTIOR=  2.00

       UNIT HYDROGRAPH 46 END-OF-PERIOD ORDINATES, LAG=   2.95 HOURS, CP=  .75  VOL= 1.00
         10.       38.       77.      120.      167.      215.      263.      308.      346.      372.
        387.      392.      387.      370.      336.      293.      255.      222.      193.      168.
        146.      127.      110.       96.       83.       73.       63.       55.       48.       41.
         36.       31.       27.       24.       21.       18.       16.       14.       12.       10.

          9.        8.        7.        6.        5.        4.

         0                                  END-OF-PERIOD FLOW
   MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q      MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS   COMP Q

                                                                    SUM   25.03   23.18    1.85   141613.
                                                                        ( 636.)( 589.)(  47.)( 4010.03)
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**********          **********          **********          **********          **********

                                   HYDROGRAPH ROUTING

                 ROUTED FLOWS THROUGH EDGEMONT RESERVOIR

                         ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                             2       1       0       0       2       0       1       0       0

                                       ALL PLANS HAVE SAME
                                          ROUTING DATA
                 QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP             LSTR
                   0.0   0.000    0.00       1       1       0       0                0

                         NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                             1       0       0   0.000   0.000   0.000    246.      -1

STAGE       901.00      902.00      903.00      904.00      904.50      905.00      908.00

FLOW          0.00      142.00      401.00      737.00      929.00     1135.00     2627.00

 CAPACITY=        0.       10.       39.       94.      133.      179.      233.      246.      297.      368.

FLEVATION=      855.      865.      875.      885.      890.      895.      900.      901.      905.      910.

                      CREL   SPWID    COQW    EXPW   ELEVL    COQL   CAREA    EXPL
                     901.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0

                                                 DAM DATA
                                       TOPEL    COQD    EXPD  DAMWID
                                       904.7     3.1     1.5    700.

CREST LENGTH          0.        50.      700.
AT OR BELOW
FLEVATION          904.7      904.9     905.0

                                             DAM BREACH DATA
                                  BRWID       Z    ELBM   TFAIL    WSEL  FAILEL
                                   100.    1.00  860.00    2.00  901.00  905.65

BEGIN DAM FAILURE AT 17.75 HOURS

PEAK OUTFLOW IS      5955. AT TIME  18.29 HOURS
```

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*OVN*

                                        DAM BREACH DATA
                          BRWID       Z    ELBM   TFAIL    WSEL  FAILEL
                           100.    1.00  860.00    2.00  901.00  906.00

PEAK OUTFLOW IS     2960. AT TIME  18.50 HOURS


          **********     **********     **********     **********     **********

                                     HYDROGRAPH ROUTING

                  CHANNEL ROUTING-MOD PULS REACH 2-3

                        ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                            3       1       0       0       0       0       0       0       0

                                         ALL PLANS HAVE SAME
                                            ROUTING DATA
                  QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP            LSTR
                    0.0   0.000    0.00       1       1       0       0               0

                        NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                            1       0       0   0.000   0.000   0.000      0.       0


NORMAL DEPTH CHANNEL ROUTING
----------------------------

      QN(1)    QN(2)    QN(3)    ELNVT    ELMAX    RLNTH      SEL
      .0400    .0300    .0400    835.0    890.0    1000.   .02000


      CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
          0.00  880.00    20.00  860.00    35.00  845.00    45.00  835.00    75.00  835.00
         76.00  845.00    96.00  845.00   130.00  880.00

 STORAGE        0.00      1.70      3.55      5.53      7.66     10.81     14.45     19.34     22.49     26.99
               31.55     36.46     41.52     47.04     52.71     58.64     64.82     71.25     77.94     84.88

 OUTFLOW        0.00    847.15   2606.91   5017.50   7995.14  12336.05  18105.12  25015.95  33025.95  42122.03
            52204.34  63580.23  75961.39  89462.29 104099.28 119889.99 136852.97 155007.41 174372.97 194969.63

   STAGE      835.00    837.37    839.74    842.11    844.47    846.84    849.21    851.59    853.95    856.32
              858.69    861.05    863.42    865.79    868.16    870.53    872.89    875.26    877.63    880.00

    FLOW        0.00    847.15   2606.91   5017.50   7995.14  12336.05  18105.12  25015.95  33025.95  42122.03
            52304.34  63580.23  75961.39  89462.29 104099.28 119889.99 136852.97 155007.41 174372.97 194969.63

MAXIMUM STAGE IS     842.0

MAXIMUM STAGE IS     840.1
```

********** ********** ********** ********** **********

HYDROGRAPH ROUTING

CHANNEL ROUTING-MOD PULS REACH 3-4

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
4	1	0	0	0	0	0	0	0

ALL PLANS HAVE SAME
ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
C.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	0.	0

NORMAL DEPTH CHANNEL ROUTING

QN(1)	QN(2)	QN(3)	ELNVT	ELMAX	RLNTH	SEL
.0400	.0300	.0400	780.0	830.0	1500.	.03700

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
0.00 830.00 800.00 820.00 1400.00 783.00 1401.00 780.00 1416.00 780.00
1417.00 783.00 1437.00 783.00 1540.00 830.00

STORAGE	0.00	1.44	6.16	15.48	29.20	47.30	69.79	96.67	127.94	163.6
	203.65	248.10	296.93	350.15	407.76	469.75	541.01	631.56	741.72	871.4
OUTFLOW	0.00	640.68	3001.97	8939.63	19605.02	36080.53	59361.86	90378.26	130006.69	179081.6
	239402.29	308737.65	390830.75	435401.77	593150.59	714758.95	784453.54	896829.79	1033762.55	1223235.8
STAGE	780.00	782.63	785.26	787.89	790.53	793.16	795.79	798.42	801.05	803.6
	806.32	808.95	811.58	814.21	816.84	819.47	822.11	824.74	827.37	830.0
FLOW	0.00	640.69	3001.97	8939.63	19605.02	36080.53	59361.86	90378.26	130006.69	179081.6
	239402.29	308737.65	390830.75	435401.77	593150.59	714758.95	794453.54	896829.79	1033762.55	1223235.8

MAXIMUM STAGE IS 795.6

MAXIMUM STAGE IS 795.2

PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

RATIOS APPLIED TO FLOWS

OPERATION	STATION	AREA	PLAN	RATIO 1 .50
HYDROGRAPH AT	1	2.34	1	2960.
		(6.06)	(	83.82)(
			2	2960.
			(	83.82)(
ROUTED TO	2	2.34	1	5877.
		(6.06)	(	166.42)(
			2	2960.
			(	83.80)(
ROUTED TO	3	2.34	1	5918.
		(6.06)	(	167.58)(
			2	2959.
			(	83.79)(
ROUTED TO	4	2.34	1	5912.
		(6.06)	(	167.41)(
			2	2959.
			(	83.80)(

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	901.00	901.00	904.65
STORAGE	246.	246.	292.
OUTFLOW	0.	0.	991.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.50	905.65	1.00	306.	5955.	2.46	18.29	17.75

PLAN 2 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	901.00	901.00	904.65
STORAGE	246.	246.	292.
OUTFLOW	0.	0.	991.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
.50	905.71	1.06	307.	2950.	6.50	18.50	0.00

PLAN 1 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.50	5918.	842.9	18.25

PLAN 2 STATION 3

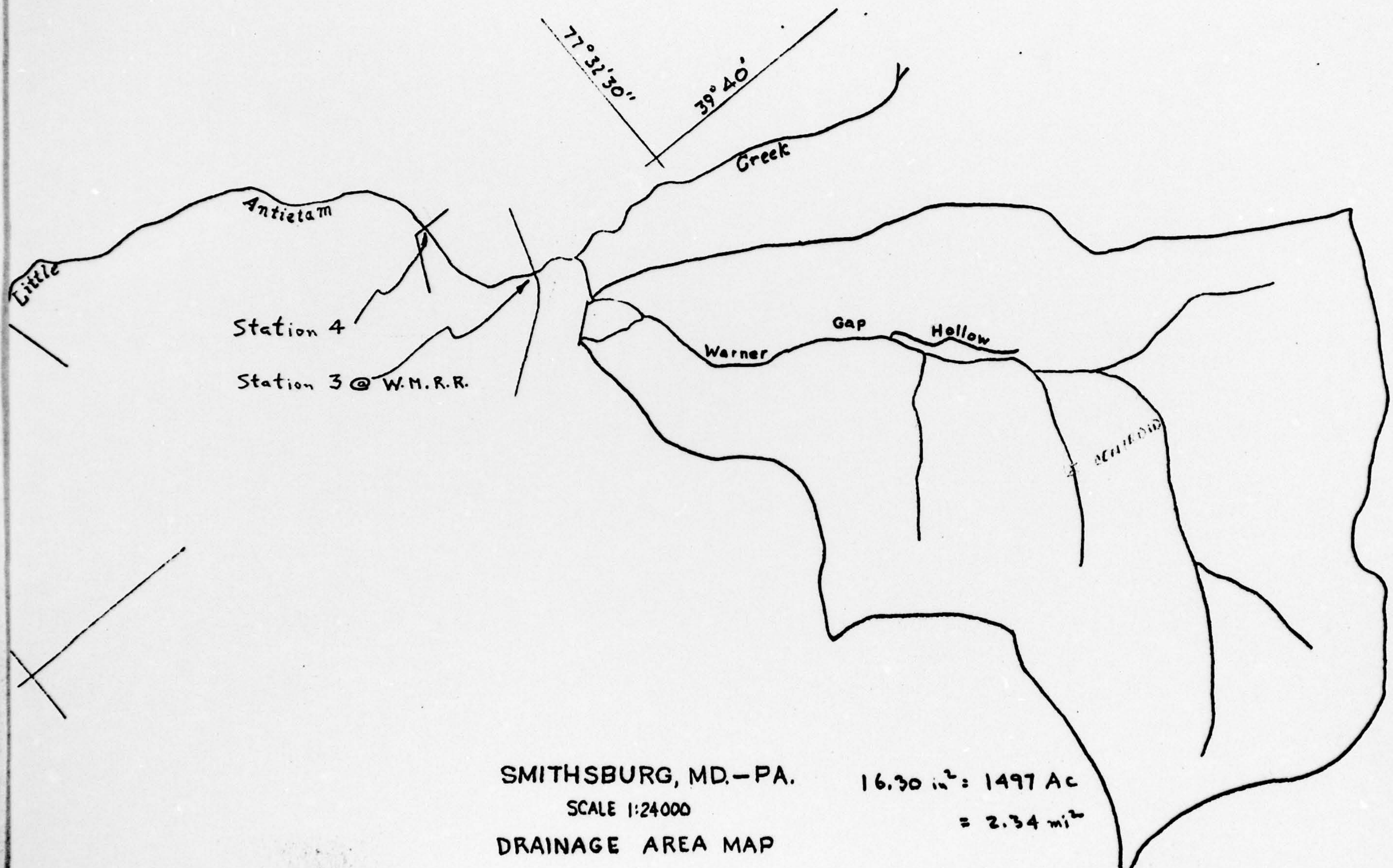
RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.50	2959.	840.1	18.50

PLAN 1 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.50	5912.	786.6	18.25

PLAN 2 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.50	2959.	785.2	18.50



SMITHSBURG, MD.–PA.
SCALE 1:24000
DRAINAGE AREA MAP

APPENDIX F

GEOLOGY REPORT

APPENDIX F

GEOLOGY REPORT

GEOLOGY REPORT
EDGEMONT RESERVOIR
WASHINGTON COUNTY, MARYLAND

Washington County is located in the Appalachian Highlands geologic division and is bounded by the western slope of the Blue Ridge province on the east and the center of the Ridge and Valley province on the west. The eastern portion of the county is traversed by the major structual unit known as the South Mountain Anticlinorium. Edgemont Reservoir is situated within this unit on the western slope of the South Mountain anticline.

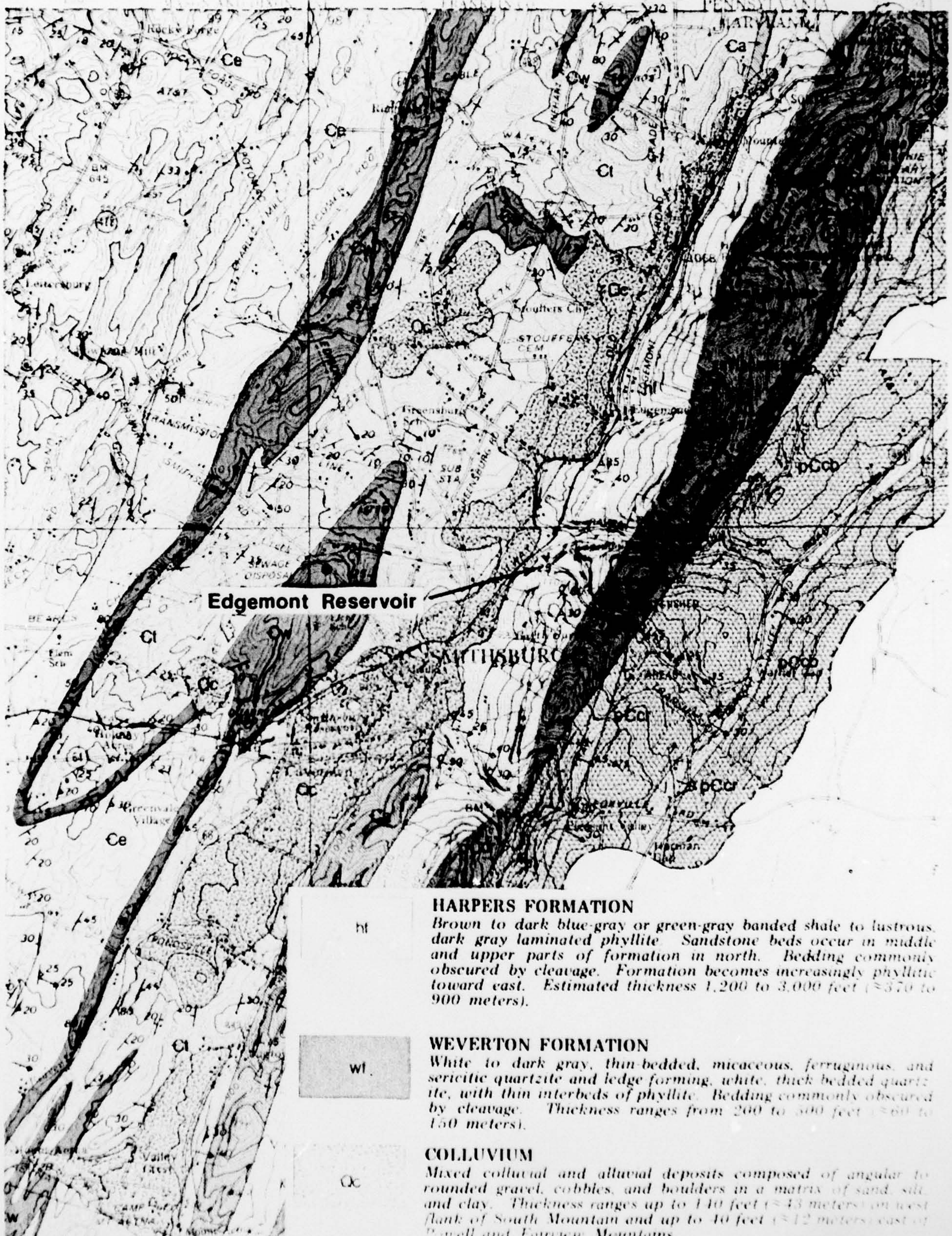
The reservoir, dam, and appurtenant structures are all underlain by the Harper's formation which is characterized by brown to dark blue gray banded shale and dark gray banded laminated phyllite of Cambrian age. The bedding of this formation is vertical or overturned and cleavage is intense. The cleavage strikes N45°E and dips 35°SE.

The Harper's formation weathers to a silty clay and silt of low plasticity with many rock fragments designated as Edgemont - Laidig channery loam and Chewacla silt loam by the USDA Soil Conservation Service. The soil cover is relatively thin ranging from 3 to 5 feet in thickness. The results of the limited subsurface explorations performed for the remedial grouting study in 1968 indicate the embankment to consist of clayey silt and silty clay with rock fragments, corresponding to the natural soil available in the reservoir area. These materials were placed on weathered phyllite of the Harper's formation. The rock in the upper portion of the foundation was found to be highly fractured and dye tests revealed the fractures to be hydraulically open. The consultant concluded that the primary source of leakage at the right abutment was through the fractured rock foundation and a successfull grouting program was implemented.

The orientation of the cleavage planes alone, with strike paralleling the dam and dipping toward the dam, would ordinarily lead to predictions of only minor seepage through the foundation. The concentrated leakage at the right abutment and the seepage at the left side of the dam possibly indicate hydraulically open fractures oriented transverse to the axis of the dam. Although the foundation strength and competency should not be affected by seepage, the integrity of the embankment could be vulnerable to internal erosion.

WASHINGTON COUNTY GEOLOGIC MAP

Scale 1:62500



ht	**HARPERS FORMATION** *Brown to dark blue-gray or green-gray banded shale to lustrous, dark gray laminated phyllite. Sandstone beds occur in middle and upper parts of formation in north. Bedding commonly obscured by cleavage. Formation becomes increasingly phyllitic toward east. Estimated thickness 1,200 to 3,000 feet (≈370 to 900 meters).*
wt	**WEVERTON FORMATION** *White to dark gray, thin-bedded, micaceous, ferruginous, and sericitic quartzite and ledge forming, white, thick bedded quartzite, with thin interbeds of phyllite. Bedding commonly obscured by cleavage. Thickness ranges from 200 to 500 feet (≈60 to 150 meters).*
Qc	**COLLUVIUM** *Mixed colluvial and alluvial deposits composed of angular to rounded gravel, cobbles, and boulders in a matrix of sand, silt and clay. Thickness ranges up to 140 feet (≈43 meters) on west flank of South Mountain and up to 40 feet (≈12 meters) east of Powell and Fairview Mountains.*

REFERENCES

1. Department of Geology, Mines, and Water Resources, The Water Resources of Allegany and Washington Counties, Bulletin 24, 1961.

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3. United States Department of Agriculture, Soil Conservation Service, Soil Survey of Washington County; Maryland, October 1962.