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NATIONAL DAM INSPECTION PROGRAM. UPPER DONOHOE DAM (NOW REFERRE--ETC(U)
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National Dam Inspection Program. Upper Donohoe Dam (now Referred to as) Twin Lakes Number 2 Dam, NDS I.D. PA-00478), Ohio River Basin, Little Crabtree Creek, Westmoreland County, Pennsylvania. Phase I Inspection Report.

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 (as was Upper Donohoe Reservoir), 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 REPORT
NATIONAL DAM INSPECTION PROGRAM

ABSTRACT

Upper Donohoe Dam: NDS I.D. No. PA-00478

Owner:	Westmoreland County
State Located:	Pennsylvania (PennDER I.D. No. 65-55)
County Located:	Westmoreland
Stream:	Little Crabtree Creek
Inspection Date:	13 December 1978
Inspection Team:	GAI Consultants, Inc. 570 Beatty Road Monroeville, Pennsylvania 15146

The visual inspection, operational history, hydrologic and hydraulic analysis and the engineering data obtained from a geotechnical consultant's recent report to the owner indicate that the structure is in poor condition. Apparent dislocation of the downstream slope paving, continuing seepage despite the current drawn down condition, and a dilapidated valve house with a valve mechanism of questionable reliability support this evaluation. In addition, the recently completed engineering study indicates a need for remedial stabilization. Records of past performance show numerous problems related to seepage both under and through the embankment.

The current owner has incorporated the Upper Donohoe Dam into Twin Lakes Park and has recently initiated a program of upgrading the facility which to date has included placement of durable riprap on the upstream slope, brick paving of the embankment crest, and the construction of a new spillway structure. Hydrologic and hydraulic calculations contained herein; however, indicate that the spillway system can only accommodate approximately 54 percent of the Probable Maximum Flood (PMF), which is considered to be the required Spillway Design Flood (SDF), before overtopping occurs. As the facility's hazard rating is "high", the present spillway is assessed as being inadequate, but not seriously inadequate.

The structural deficiencies of the embankment are of such a nature that if left uncorrected, they could result in failure of the dam with subsequent loss of life and/or substantial

property damage. Thus, the facility is considered unsafe. An emergency condition is not considered to exist because the current owner is aware of the deficiencies and the reservoir, therefore, is being maintained in a drawn down condition under close observation by park personnel. A remedial stabilization scheme has been developed and remedial plans are being finalized.

It is recommended that the owner:

a. Immediately activate a plan for emergency operation and a warning system for downstream residents. Included in the plan should be provisions for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Inspect the facility on a daily basis to insure the reservoir remains drawn down until rehabilitation is complete.

c. Remove the temporary earth road that presently crosses the discharge channel approximately 35 feet downstream of the spillway control. In addition, repairs to the breached left spillway sidewall at this location should be implemented. The thin layer of sediment lining the bottom of the concrete spillway channel should be removed.

d. Regrade the present embankment crest and restore all low areas to elevation 1130.0 feet (MSL).

e. Enlist the services of a professional engineer experienced in hydrology and hydraulics to perform a detailed evaluation of the facility. Included in the study should be a reevaluation of the adequacy of the existing spillway and discharge channel and the effects of any proposed modifications to the downstream Twin Lakes Dam No. 1. Subsequently, the owner should take whatever measures are deemed necessary to make the facility hydraulically adequate.

f. Rehabilitate the outlet works and provide a means of controlling or blocking flow at the inlet end of the blowoff line in the event a leak(s) develops beneath the embankment.

g. Develop an operations and maintenance manual for use at the facility.

h. Have the facility inspected on a yearly basis by a registered professional engineer experienced in the design and construction of earth dams to check for hazardous conditions that might develop. This should be done until repairs resulting from paragraph e above are accomplished.

GAI Consultants, Inc.	Approved by:
Bernard M. Mihalcin	G. K. Withers
Bernard M. Mihalcin, P.E.	G. K. WITHERS Colonel, Corps of Engineers District Engineer
Date 9 May 1979	Date 8 Jun 79

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NATIONAL DAM INSPECTION PROGRAM
UPPER DONOHOE DAM
NDI# PA-478, PENNDER# 65-55

SECTION 1
GENERAL INFORMATION

1.0 Authority.

The Dam Inspection Act, Public Law 92-367, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a program of inspection of dams throughout the United States.

1.1 Purpose.

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

ABSTRACT

1.2 Description of Project.

a. Dam and Appurtenances. The Upper Donohoe Dam is an earth embankment approximately 520 feet in length with a maximum height of 34 feet. According to PennDER records, the embankment was constructed with a puddle cutoff wall having a top and base width of 10 feet, a maximum height of approximately 18 feet and length of 500 feet. The depth of excavation for the cutoff wall is reportedly about 15 feet below the original ground surface. Both the upstream and downstream slopes were originally protected by "stone covered with mortar." Recently, however, a hard, siliceous limestone riprap was placed on the top 5 to 6 feet of the upstream slope and a brick deck was laid across the entire crest.

The facility is served by a rectangular-shaped concrete channel spillway constructed in 1977 and located at the right abutment. The discharge channel is cut in natural ground and rock below the spillway. A 12-inch diameter cast iron pipe passes beneath the center of the embankment and serves as the outlet conduit. The outlet conduit is valved at a valve house located at the downstream toe of the embankment.

~~b. Location.~~ Upper Donohoe Dam is located on the headwaters of Little Crabtree Creek in Hempfield and Unity Townships, Westmoreland County, Pennsylvania. This dam is immediately upstream of Twin Lakes No. 1 Dam and reservoir

ABSTRACT

(NDI# PA-487). Both facilities are integral components of Twin Lakes Park, a recreational facility located just northeast of Greensburg, Pennsylvania. The village of Luxor is about 0.8 mile downstream of the embankment. The dam, reservoir, and watershed are contained on the Latrobe, Pennsylvania, U.S.G.S. 7.5 minute topographic quadrangle (see Appendix G). The coordinates of the dam are N40° 19.3' and W79° 28.5'.

c. Size Classification. Small (34 feet high, 270 acre-feet storage capacity at top of dam).

d. Hazard Classification. High (see Section 3.1.e).

e. Ownership. Westmoreland County
Department of Parks and Recreation
Greensburg, Pennsylvania 15601

f. Purpose. Recreation.

g. Historical Data. The Upper Donohoe Dam was constructed in 1910 by Offutt and Bennett, contractors, of Greensburg, Pennsylvania, for the Jamison Coal and Coke Company. The dam, designed by the Jamison Coal and Coke Company, was used in conjunction with the lower dam and reservoir (Twin Lakes No. 1 Dam) as an industrial water supply facility for coal mining and coking operations. Later, it became the property of the Consolidation Coal Company. Approximately 15 years ago, ownership of the dam was transferred to the Westmoreland County, Department of Parks and Recreation, who have incorporated both the upper and lower dams into the Twin Lakes Park recreational facility.

Information and correspondence contained in PennDER files reveals the facility has had a history of problems including crest settlement, significant seepage, and an inadequate spillway. A lack of cooperation between state officials and the previous owners is evident from the available data. Occasionally, ad-hoc remedial measures were undertaken by previous owners most of which, however, proved to be ineffective.

In 1977, after complete rehabilitation of the lower Twin Lakes dam, the current owners replaced the spillway of the upper dam and added other improvements including the placing of riprap on the upper 5 to 6 feet of the upstream slope, paving the crest with brick and the addition of a metal hand rail on the downstream side of the crest.

In the spring of 1978, Westmoreland County retained Geo-Mechanics, Inc., consulting engineers of Belle Vernon, Pennsylvania, to perform a geotechnical engineering investigation of the facility. A detailed inspection performed by the consultant revealed serious structural deficiencies and, as a result, the reservoir has been maintained in a drawn down condition while detailed rehabilitation plans are being prepared.

1.3 Pertinent Data.

a. Drainage Area (square miles). 0.38 (total)

b. Discharge at Dam Site. Discharge records are not available.

c. Elevation (feet above mean sea level). The following elevations were obtained through field measurements that were based on the elevation of the emergency spillway crest at 1126.0 feet.

Top of Dam	1128.7 (field) 1130.0 (design)
Maximum Design Pool	Not known
Maximum Pool of Record	Not known
Normal Pool	1126
Emergency Spillway Crest	1126
Upstream Outlet Invert	Not known
Downstream Outlet Invert	1095 (estimate)
Streambed at Dam Centerline	1094 (estimate)
Maximum Tailwater	Not known

d. Reservoir Length (miles).

Top of Dam	0.3
Normal Pool	0.3

e. Storage (acre-feet).

Top of Dam	273
Normal Pool	215
Design Surcharge	Not known

f. Reservoir Surface (acres).

Top of Dam	24
Normal Pool	20
Maximum Design Pool	Not known

g. Dam.

Type	Earth embankment with a puddle cutoff wall and rock protected slopes.

Length	520 feet
Height	34 feet
Top Width	10 feet
Upstream Slope	3H:1V (exposed freeboard zone; field measured)
Downstream Slope	1.5H:1V (crest to toe; field measured)
Zoning	None indicated.
Impervious Core	The 1915 inspection report indicates a puddle cutoff wall near the upstream toe with a height of 18 feet, length of 500 feet, and a depth of excavation for foundation of 15 feet. The top of the cutoff wall is approximately 28 feet below the crest of the dam.
Cutoff	The puddle core trench serves as the only embankment cutoff.
Grout Curtain	None indicated.

h. <u>Diversion and Regulating Tunnels</u>. None.

i. <u>Spillway</u>.

Type	Uncontrolled, reinforced concrete chute spillway with an unlined discharge channel. The existing spillway was constructed in 1977 and is located at the right abutment.

Crest Elevation.	1126
Crest Length	20 feet

j. <u>Outlet Conduits</u>.

Supply Pipe	None.
Blowoff Pipe	12-inch diameter, cast iron, length unknown.
Closure	Manually operated gate valve in a valve house at the downstream toe of the dam.
Regulating Facilities	The valve house is located at the downstream toe and contains one 12-inch diameter gate valve for regulating discharge.
Access	Access to the valve house is from the left abutment.

SECTION 2
ENGINEERING DATA

2.1 Design.

a. Design Data Availability and Sources. No design reports are available for any aspect of the original embankment and outlet works. Design drawings for the 1977 emergency spillway reconstruction (see Figures 5 and 6) are available from the owner. A hydrologic and hydraulic assessment of the current spillway and a stability evaluation of the existing embankment were performed by Geo-Mechanics, Inc., and are presented in their 1978 report on the facility. Embankment and outlet conduit design features, presented below, are derived from an inspection report prepared by the Water Supply Commission of Pennsylvania in 1915.

b. Design Features.

1. Embankment. Available data indicate that the embankment is constructed of earth; however, placement and compaction procedures are unknown.

The earliest descriptive report pertaining to the facility was made by the Water Supply Commission of Pennsylvania on August 5, 1915. This report was prepared 5 years following construction and contains data supplied by the design engineer. The following excerpt is from this report and presumably describes the condition of the embankment following construction.

> "The surface soil was cleaned off and the dam was built on an underlying shale formation. The embankment material consists of rolled earth placed in layers with scrapers. Both the upstream and downstream slopes are protected with stone covered with mortar placed on a slope of 1.5H:1V. The crest of the embankment was seeded with grass. The length of the embankment along the crest is 500 feet with a crest width of 9 feet, maximum base width of 99 feet, and a maximum height above streambed of 30 feet. The puddle cutoff wall is in the upstream portion of the embankment. It is 10 feet thick and extends 15 feet below the original ground surface. The height of the cutoff wall is approximately 18 feet, with the top of the wall approximately 28 feet below the crest of the dam."

2. Appurtenant Structures.

a) Outlet Structure. The outlet structure is a square masonry building housing the valve mechanism for the outlet conduit and is located at the downstream toe of the embankment.

b) Conduits. A 12-inch diameter cast iron blowoff pipe passes beneath the embankment and is controlled by a single gate valve located in the valve house. Concrete anti-seep collars are reportedly spaced along its length to prevent leakage. The outlet of the 12-inch diameter conduit is located immediately downstream of the valve house.

c) Spillway. The spillway consists of a recently constructed reinforced concrete chute and an unlined discharge channel (see Figures 5 and 6). At the control section, the concrete channel is approximately 20 feet wide and 36 feet long. Discharge from the spillway enters a channel cut in natural ground and flows into the reservoir of Twin Lakes No. 1 Dam approximately 250 feet downstream of the Upper Donohoe Dam.

c. Design Data and Procedures. No design data are available for the original facility. Design parameters have been developed by the owner's consultant for the proposed rehabilitation of the facility and are discussed below.

1. Hydraulics and Hydrology. An assessment of the eroding spillway is presented in a recent report prepared by the owner's consultant. The analysis indicates that based on PennDER "C" Curve criteria, the spillway system is required to pass a peak flow of 645 cfs. Assuming a flow depth of 3 feet and applying Manning's equation for open channel flow, the maximum discharge of the spillway was determined to be 1349 cfs and the spillway was deemed adequate.

Analysis presented herein (see Section 5) indicates that the above assessment is in error in that a spillway flow depth in excess of 1.8 feet will cause the existing embankment to be overtopped. Also, the use of Manning's equation for determination of the spillway capacity is inappropriate, since the spillway discharges will be governed by critical flow relationships.

Designation of the existing blowoff conduit as the service or principal spillway facility and the concrete chute as the emergency spillway is questionable in that the normal pool appears to be set at the crest of the chute spillway, subjecting it to frequent flows.

2. Embankment. In 1978, six test borings were drilled along the downstream side of the embankment by the owner's consultant for the purpose of assessing the existing conditions within the embankment and to develop design parameters for the proposed rehabilitation (see Figure 3). Three of the test borings were drilled through the embankment and all six borings penetrated at least 25 feet into the underlying bedrock to determine the type, depth, and engineering properties of these materials. Shelby tube and bag samples were secured for subsequent direct shear and permeability tests and for laboratory compaction, permeability and classification tests, respectively. Soil and rock field permeability tests were performed in four of the test borings. Typical embankment cross-sections depicting existing conditions within the embankment are presented on Figure 4, Appendix F. The methods and results of the testing are presented in the consultants report.

Pertinent observations and test results in that study include:

- The top 4 to 9 feet of the embankment consists of relatively permeable granular soils.
- The underlying embankment and natural soils are fine grained (silt and clay) but are of variable permeability indicating questionable placement and/or compaction procedures.
- The in-place strength parameters for the embankment and natural soil foundation are $\phi = 35°$, $C = 0$, based on direct shear testing.
- The underlying bedrock is primarily sandstone with a permeability on the order of 10^{-3} cm/sec.

A stabilization scheme to provide an acceptable factor of safety for the facility is presented in the consultant's report and essentially consists of buttressing the downstream slope with a resistant rockfill toe, flattening the downstream slope and providing a filter drain to collect seepage attributed to the apparent high phreatic surface.

3. Appurtenant Structures.

a) Spillway. Based on construction drawings and data available from the owner and PennDER, the spillway appears to be adequately designed and constructed. No design calculations were made available to the inspection team for review.

b) Outlet Works. No data are available relative to the design of the outlet conduit.

2.2 Construction Records.

No construction records are available for any aspect of this facility. Construction drawings for the existing emergency spillway, constructed in 1977, are available from the Westmoreland County, Department of Parks and Recreation.

2.3 Operating Records.

No pool level, rainfall, or discharge records are available for the facility. Correspondence available from PennDER files indicate that discharges through the emergency spillway have been small due to the attenuation of flow caused by the upstream railroad embankment culvert.

2.4 Other Investigations.

Except for the previously discussed geotechnical study in 1978, no subsequent engineering related investigations have been conducted other than routine inspections of the facility by PennDER personnel.

2.5 Evaluation.

Sufficient data are available to make a Phase I assessment of the facility. A detailed geotechnical study has been conducted by a consultant to the owner. The stabilization scheme proposed as a result of the study appears adequate; however, the hydraulic and hydrologic assessment is questionable. In particular, the hydraulic capacity of the chute spillway appears in error. Designation and/or use of the spillway and its unlined downstream channel as the emergency facility is debatable.

SECTION 3
VISUAL INSPECTION

3.1 Observations.

a. General. The general appearance of this facility suggests the dam and its appurtenances are currently in poor condition.

b. Embankment. The visual inspection indicates the embankment is in poor condition. Some seepage flow and localized saturated areas were observed along the downstream toe to the right of the valve house in spite of the drawn down status of the reservoir. Just to the right of the valve house, the outlets of two cast iron toe drain pipes were observed discharging about 2-3 gallons per minute from the left drain and approximately 5 gallons per minute from the right drain. A full evaluation of the quantity and extent of leakage could not be determined due to the drawn down condition of the reservoir at the time of the inspection (see Photograph 3).

The crest of the embankment is well protected with an architectural paving brick (Unistone) installed in 1977. The brick is in excellent condition and covers the crest. In conjunction with the paving, a wood post and iron pipe fence railing were installed on the downstream side of the crest (see Photograph 1). The upstream slope is well protected against wave action by a durable limestone riprap also placed in 1977 (see Photograph 3). The hand-placed mortared stone covering the downstream slope was observed to be slightly irregular and to some extent disturbed. Most of the observed displacement was approximately one third of the way up from the toe and did not exhibit signs of recent movement. In addition, the downstream slope was covered with light vegetation making direct observation of the slope paving somewhat difficult.

c. Appurtenant Structures.

1. Spillway Structure. The spillway was completely reconstructed in 1977. Based on visual observations, the new spillway structure is in good condition (see Photographs 4 and 5).

2. Outlet Works. Complete submergence of the intake structure precluded the possibility of visual inspection. The outlet conduit passes beneath the embankment and discharges directly into Twin Lakes No. 1 Reservoir. The manually operated gate valve is located in a dilapidated, partially collapsed structure located just beyond the down-

stream toe of the embankment. The gate valve is currently operated by turning the valve stem with a pipe wrench. The gate valve was partially opened and an estimated discharge of approximately 200 gallons per minute was observed flowing from the outlet at the time of the inspection.

d. Reservoir Area. The general area surrounding the reservoir is characterized by moderate to steep, partially wooded slopes. Immediately upstream of the reservoir is a high railroad embankment crossing the principal stream flowing into the reservoir. The stream is carried beneath the embankment through a 3.0-foot by 2.5-foot culvert (Appendix C, Sheet 9). The outlet end of the culvert is completely submerged and partially blocked with silt and debris (see Photographs 11 and 12).

e. Downstream Channel. The concrete spillway structure discharges into a trapezoidal-shaped channel cut into natural ground on the right abutment (see Photograph 6). The channel is somewhat irregular with low and partially breached sidewalls on the downslope side. About 35 feet downstream of the spillway is a temporary construction road built across the channel. It is possible that modest discharges through the spillway could cause overtopping of the channel sidewall at the road crossing resulting in at least part of the flow being discharged near the toe of the dam. Approximately 250 feet downstream of the spillway, the channel discharges directly into the reservoir of Twin Lakes No. 1 Dam.

The nearest inhabited structure likely to be affected by a breach of this facility is downstream of Twin Lakes No. 1 Dam. The dwelling located in the floodplain of Little Crabtree Creek and the eastern end of the community of Luxor lie approximately 4,300 feet downstream of the Upper Donohoe Dam on Little Crabtree Creek (see Photograph 14). Many mobile homes are also located within the floodplain in this area. Little Crabtree Creek joins Crabtree Creek approximately 3.0 miles downstream of the embankment. Approximately 4.0 miles downstream of the dam, Crabtree Creek flows within the flood pool boundary of Loyalhanna Reservoir, a major flood control project. The intervening valley between the lower dam and the flood control project is generally sparsely vegetated; however, it is estimated that within this reach, more than one hundred people could be affected by an embankment breach of both the Upper Donohoe Dam and the Twin Lakes No. 1 Dam. Therefore, the hazard classification for the facility is considered to be "high".

3.2 Evaluation.

The overall condition of the facility is considered to be poor. The reservoir level has been drawn down due to previously observed seepage under normal pool conditions. The outlet works are dilapidated with no means of controlling flow at the inlet, and the spillway discharge channel requires remedial repair. These items are reportedly being considered in the proposed rehabilitation of the facility.

SECTION 4
OPERATIONAL PROCEDURES

4.1 Normal Operating Procedure.

According to the owner's representative, there are no formal operating procedures at this facility. Park personnel currently operate the gate valve to maintain the drawn down condition of the reservoir. The current drawn down condition is to be maintained until rehabilitation work is completed.

4.2 Maintenance of Dam.

The dam is maintained on an unscheduled basis by the Westmoreland County, Department of Parks and Recreation.

4.3 Maintenance of Operating Facilities.

Other than occasionally operating the gate valve, no regular maintenance has been performed on the operating mechanism.

4.4 Warning Systems.

There are no formal warning systems in effect at this facility.

4.5 Evaluation.

The facility is operated by park personnel on an unscheduled basis and no formal operations and maintenance manuals are available. There is no formal warning system in effect; however, the facility is under very close observation in order to maintain the current drawn down condition.

SECTION 5
HYDROLOGIC/HYDRAULIC EVALUATION

5.1 Design Data.

No original design data are available.

5.2 Experience Data.

Actual discharge records are not available for this facility. No data related to the performance of the facility and its appurtenances during major flooding events are available. The recently constructed spillway has yet to be subjected to major flooding and consequently, no significant data relative to the present spillway are available.

5.3 Visual Observations.

Based on visual observations, the recently constructed spillway is considered to be in good condition. Several deficiencies were noted that include: 1) several inches of sediment lining the channel bottom of the concrete spillway; 2) a temporary earth road crossing the discharge channel about 35 feet downstream of spillway control section; 3) a breach of the left channel sidewall at the road; and, 4) an irregular shaped unlined discharge channel. The above conditions could adversely affect the operation of the spillway (Appendix C, Sheets 14 and 15); however, for the purpose of the analysis, all of the deficiencies were neglected as it is assumed that they will be corrected in the proposed rehabilitation program.

5.4 Method of Analysis.

The facility has been analyzed in accordance with the procedures and guidelines established by the U. S. Army Corps of Engineers, Baltimore District, for Phase I hydrologic and hydraulic evaluations. The analysis has been performed utilizing a modified version of the HEC-1 program developed by the U. S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, California. Analytical capabilities of the program are briefly outlined in the preface contained in Appendix C.

5.5 Summary of Analysis

a. Spillway Design Flood. In accordance with procedures and guidelines contained in the National Guidelines for Safety Inspection of Dams for Phase I Investigations, the Spillway Design Flood (SDF) for Upper Donohoe Dam ranges between the 1/2 PMF (Probable Maximum Flood) and the PMF. This classification is based on the relative size of the dam (small), and the potential hazard of dam failure to downstream developments (high). Due to the presence of another impoundment just downstream, and the high damage potential of dam failure to both the downstream impoundment and residences, the SDF for this facility is considered to be the PMF.

b. Results of Analysis. The Upper Donohoe Dam was evaluated under assumed normal operating conditions. That is, the reservoir was initially at its normal pool or spillway elevation of approximately 1126.0 feet, with the low-level "blowoff" conduit closed. Actually, the conduit is usually kept slightly open. However, the opening provides very little discharge capacity. Also, the reservoir level was found to be drawn down at the time of inspection due to structural considerations. The spillway is presently an unlined chute channel with a flat concrete critical flow control crest. The chute channel was in a state of disrepair at the time of inspection, but it was assumed to be in good condition in the analysis.

A railroad embankment with a small culvert for flow passage is located just upstream from the reservoir, and controls about 50 percent of the possible reservoir inflows. In order to account for the effects of this embankment on inflows, the embankment was considered to function like a dam in the analysis, with the small culvert providing the only means of discharge. The culvert outflows were then added to the local reservoir inflows to determine the total inflow hydrographs into the Upper Donohoe Dam Reservoir. All pertinent engineering calculations relative to the evaluation of this facility are provided in Appendix C.

Overtopping analysis (using the Modified HEC-1 Computer Program) indicated that the discharge/storage capacity of the Upper Donohoe Dam could accommodate only about 54 percent of the PMF prior to overtopping of the dam (Appendix C, Summary Input/Output Sheets, Sheet H). The peak PMF (SDF) inflow of about 750 cfs was attenuated by the discharge/storage capabilities of the dam and reservoir such that the resulting peak outflow was about 640 cfs (Summary Input/Output Sheets, Sheets E and G). Under the PMF, the dam embankment was overtopped for approximately 4.3 hours, with a maximum depth of inundation of about 0.6 feet (Summary Input/Output Sheets, Sheet H).

Although this analysis implies that the dam facility can handle a flood of greater than 1/2 PMF magnitude, the effect of the railroad embankment on reservoir inflows cannot be overemphasized (since the actual configuration and capacity of the discharge culvert is unknown). The peak PMF flow on the upstream side of the railroad embankment was about 640 cfs, the peak 1/2 PMF flow was about 320 cfs, with the culvert possibly causing backwater flooding upstream due to the resulting large depths of headwater (Summary Input/Output Sheets, Sheet B). As can be seen, the presence of the embankment and small culvert caused a large portion (approximately 50 percent) of the total reservoir inflow to be greatly attenuated. Had the embankment not been present, or had the culvert been large enough to pass at least a 1/2 PMF size peak flow, the total peak inflow into the reservoir would have been approximately 910 cfs for the PMF, or approximately 600 cfs for the 1/2 PMF (these estimates are based on the detailed HEC-1 computer output). If the 1/2 PMF peak inflow was 600 cfs, the corresponding outflow would be in excess of 390 cfs which would result in a depth of embankment inundation in excess of 0.3 feet (based on the detailed HEC-1 output). In this case, the discharge/storage capacity of the Upper Donohoe Dam would accommodate less than 50 percent of the PMF. Therefore, any increase in the discharge values of the rating curve of the railroad embankment culvert above those estimated in this analysis (i.e. any increase in the size of the culvert) will probably lead to a more serious classification of the facility.

5.6 Spillway Adequacy.

Hydrologic and hydraulic analysis of the Upper Donohoe Dam indicates that the existing facility can pass and/or store approximately 54 percent of the recommended Spillway Design Flood (SDF) which for this facility is considered to be the Probable Maximum Flood (PMF). Therefore, the spillway system is considered to be inadequate, but not seriously inadequate.

SECTION 6
EVALUATION OF STRUCTURAL INTEGRITY

6.1 Visual Observations.

a. Embankment. Based on the visual inspection, the embankment appears to be in poor structural condition. Moderate seepage was observed along the toe of the embankment particularly along the right abutment. A full evaluation of the seepage, however, could not be made due to the current drawn down condition of the reservoir. PennDER files indicate a history of severe and chronic seepage that may be a contributing factor in displacing the mortared stone facing on the downstream slope. A high phreatic surface, identified by a geotechnical consultant to the owner when the reservoir was full produced a low factor of safety against sliding which resulted in the decision to draw down the reservoir and initiate the design of a remedial stabilization scheme.

b. Appurtenant Structures

1. Spillway. The concrete spillway structure at the right abutment is in good condition. Concrete surfaces are in good condition with no evidence of cracking, spalling, or other deterioration. The discharge channel is obstructed, poorly maintained and in need of remedial repair.

2. Outlet Works. The outlet works are in poor condition. The valve house is in a dilapidated state creating a difficult and hazardous access. The owners representative reports difficulty in operating the gate valve and there is no upstream control on the outlet conduit.

6.2 Design and Construction Techniques.

The first inspection report issued in 1915 indicates the embankment was placed in layers and rolled. No drawings or other records are available detailing the methods of the actual design and construction. Construction drawings are available for the existing spillway structure.

6.3 Past Performance.

No formal records of past performance are available from the owner; however, historical accounts and inspection reports available from PennDER files recount a history of severe and chronic seepage with little attempt to remedy the situation by the former owners. Field inspection, however, did reveal that two cast iron toe drain pipes were installed at some time in the past. Despite the current drawn down condition of the reservoir, leakage continues along the downstream toe and particularly to the right of the valve house.

PennDER inspection reports also suggest a long history of spillway inadequacy and replacement. Field inspection also indicated that the channel sidewall immediately below the emergency spillway has been breached in the past possibly discharging flow onto the right abutment and along the embankment toe.

6.4 Seismic Stability

The dam is located in Seismic Zone No. 1 and thus subject to minor earthquake induced forces. Since the structure has a history of excessive seepage both beneath and through the embankment, it is possible that even minor earthquake induced dynamic forces could be significant at high pool levels. However, no investigations or calculations were performed to confirm this opinion.

SECTION 7
ASSESSMENT AND RECOMMENDATIONS FOR REMEDIAL MEASURES

7.1 Dam Assessment.

a. Safety. The visual inspection, operational history, hydrologic and hydraulic analysis, and the engineering data obtained from a geotechnical consultant's report to the owner indicate that the structure is in poor condition. Apparent dislocation of the downstream slope paving, continuing seepage despite the current drawn down condition, and a dilapidated valve house with a valve mechanism of questionable reliability support this evaluation. In addition, the recently completed engineering study of the embankment indicates the need for remedial stabilization. Records of past performance show numerous problems related to seepage both under and through the embankment.

The current owner has incorporated the Upper Donohoe Dam into Twin Lakes Park and has recently initiated a program of upgrading the facility which to date has included placement of durable riprap on the upstream slope, brick paving of the embankment crest, and the construction of a new spillway structure. Hydrologic and hydraulic calculations contained herein indicate that the spillway system can only accommodate approximately 54 percent of the Probable Maximum Flood (PMF), which is considered to be the required Spillway Design Flood (SDF), before overtopping of the embankment occurs. As the facility's hazard rating is "high", the present spillway, therefore, is assessed as being inadequate, but not seriously inadequate.

Structural deficiencies of the embankment are of such a nature that if left uncorrected, they could result in failure of the dam with subsequent loss of life and/or substantial property damage. Thus, the facility is considered unsafe. An emergency condition is not considered to exist because the current owner is aware of the deficiencies and is maintaining the reservoir in a drawn down condition under close observation by park personnel. A remedial stabilization scheme has been developed and remedial plans are being finalized.

b. Adequacy of Information. The available data are considered sufficient to make an accurate Phase I assessment of the facility.

c. Urgency. It is recommended that the additional investigation and remedial measures listed below be implemented immediately.

d. Necessity for Additional Investigation. A reevaluation of the existing spillway system is considered necessary.

7.2 Recommendations/Remedial Measures.

It is recommended that the owner:

a. Immediately activate a plan for emergency operation and a warning system for downstream residents. Included in the plan should be provisions for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Inspect the facility on a daily basis to insure the reservoir remains drawn down until the proposed rehabilitation is complete.

c. Remove the temporary earth road that presently crosses the discharge channel approximatly 35 feet downstream of the spillway control. In addition, repairs to the breached left spillway sidewall at this location should be implemented. The thin layer of sediment lining the bottom of the concrete spillway channel should be removed.

d. Regrade the present embankment crest and restore all low areas to elevation 1130.0 feet (MSL).

e. Enlist the services of a professional engineer experienced in hydrology and hydraulics to perform a detailed evaluation of the facility. Included in the study should be a reevaluation of the adequacy of the existing spillway and discharge channel and the effects of any proposed modifications to the downstream Twin Lakes Dam No. 1. Subsequently, the owner should take whatever measures are deemed necessary to make the facility hydraulically adequate.

f. Rehabilitate the outlet works and provide a means of controlling or blocking flow at the inlet end of the blowoff line in the event a leak(s) develops beneath the embankment.

g. Develop an operations and maintenance manual for use at the facility.

h. Have the facility inspected on a yearly basis by a registered professional engineer experienced in the design and construction of earth dams to check for hazardous conditions that might develop. This should be done until repairs resulting from paragraph e above are accomplished.

APPENDIX A

CHECK LIST - ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
PHASE I

NAME OF DAM: Upper Donohoe Dam
NDI#: PA-478 PENNDER#: 65-55

ITEM	REMARKS NDI# PA - 478
PERSONS INTERVIEWED AND TITLE	Adrian Horvath - Maintenance Development Coordinator. William Paxton - Planning Coordinator (landscape architect) Westmoreland County, Department of Parks and Recreation, Greensburg, Pennsylvania
REGIONAL VICINITY MAP	See Appendix G. U.S.G.S. 7.5 minute series topographic quadrangle, Latrobe, Pennsylvania, dated 1054 and photorevised in 1969.
CONSTRUCTION HISTORY	Construction history of the original embankment is inferred from PennDER correspondence. Purchased by Westmoreland County in 1964. Rehabilitation design prepared by Geo-Mechanics, Inc., and presented to PennDER. Plan to rehabilitate the facility in spring, 1979 (see Section 1.2.g).
AVAILABLE DRAWINGS	No drawings of the original facility are available. Rehabilitation drawings and existing conditions by Geo-Mechanics, Inc., dated 1978. See Appendix F, Figures 2 through 6.
TYPICAL DAM SECTIONS	See Appendix F, Figures 3 and 4.
OUTLETS: PLAN DETAILS DISCHARGE RATINGS	 Not available. Not available. Not available.

ITEM	REMARKS NDI# PA - 478
SPILLWAY: PLAN SECTION DETAILS	See Appendix F, Figures 5 and 6.
OPERATING EQUIPMENT PLANS AND DETAILS	Not available.
DESIGN REPORTS	Not available for the original embankment. Rehabilitation design report prepared by Geo-Mechanics, Inc., Belle Vernon, Pennsylvania, entitled, "Subsurface Exploration and Geotechnical Engineering Investigation, Upper Dam at Twin Lakes Park, Westmoreland County, Pennsylvania." Report and related correspondence available from Westmoreland County, Department of Parks and Recreation, Greensburg, Pennsylvania.
GEOLOGY REPORTS	Contained within the above-referenced report.
DESIGN COMPUTATIONS: HYDROLOGY AND HYDRAULICS STABILITY ANALYSES SEEPAGE ANALYSES	Contained within the above-referenced report.
MATERIAL INVESTIGATIONS: BORING RECORDS LABORATORY TESTING FIELD TESTING	Contained within the above-referenced report and related drawings; Appendix F, Figures 3 and 4.

ITEM	REMARKS NDI# PA - 478
BORROW SOURCES	See Appendix F, Figures 5 and 6.
POST CONSTRUCTION DAM SURVEYS	Not available.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Not available for the original embankment. Rehabilitation design report entitled, "Subsurface Exploration and Geotechnical Engineering Investigation, Upper Dam at Twin Lakes Park, Westmoreland County, Pennsylvania," by Geo-Mechanics, Inc., Belle Vernon, Pennsylvania. Report and related correspondence available from Westmoreland County, Department of Parks and Recreation, Greensburg, Pennsylvania.
HIGH POOL RECORDS	Contained within the above-referenced report.
MONITORING SYSTEMS	Contained within the above-referenced report.
MODIFICATIONS	Contained within the above-referenced report and also displayed on the drawings. See Appendix F, Figures 3 and 4.

ENGINEERING DATA (CONTINUED)

ITEM	REMARKS NDI# PA - 478
PRIOR ACCIDENTS OR FAILURES	None.
MAINTENANCE: RECORDS MANUAL	There are no formal maintenance or operation programs in effect at this facility. Informal maintenance is accomplished through periodic mowing, visual inspection, etc., by county park personnel.
OPERATION: RECORDS MANUAL	Pool elevation, daily discharge, or operational records are not kept for this facility. No formal operations manual is available.
OPERATIONAL PROCEDURES	There are no formal operational procedures associated with this facility. Excess inflow is discharged through the embankment spillway. The outlet works are operated manually to maintain the current drawn down condition of the reservoir.
WARNING SYSTEM AND/OR COMMUNICATION FACILITIES	Informal contacts with the local fire department are maintained by park personnel. The park police are also aware of the potential hazard to the downstream population. There is no job assignment for someone to watch the reservoir during periods of high rainfall.
MISCELLANEOUS	

CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

NDI ID # 478

PENN DER ID # 65-55

SIZE OF DRAINAGE AREA: 0.38 square miles

ELEVATION TOP NORMAL POOL: 1126 STORAGE CAPACITY: 215 acre-feet

ELEVATION TOP FLOOD CONTROL POOL: - STORAGE CAPACITY: -

ELEVATION MAXIMUM DESIGN POOL: - STORAGE CAPACITY: -

ELEVATION TOP DAM: 1128.7 (low spot) STORAGE CAPACITY: 270 acre-feet

SPILLWAY DATA

CREST ELEVATION: 1126 (assumed datum)

TYPE: Concrete open rectangular channel

WIDTH: 20 feet

LENGTH: 26 feet

SPILLOVER LOCATION: right abutment

NUMBER AND TYPE OF GATES: None

OUTLET WORKS

TYPE: 12-inch diameter cast iron conduit

LOCATION: Beneath the center of the embankment

ENTRANCE INVERTS: Not known

EXIT INVERTS: 1095 (estimate)

EMERGENCY DRAWDOWN FACILITIES: 12-inch diameter gate valve containe with the valve house located at the downstream toe.

HYDROMETEOROLOGICAL GAGES

TYPE: None

LOCATION: -

RECORDS: -

MAXIMUM NON-DAMAGING DISCHARGE: Not known

APPENDIX B

CHECK LIST - VISUAL INSPECTION

CHECK LIST
VISUAL INSPECTION
PHASE 1

NAME OF DAM Upper Donohoe Dam STATE Pennsylvania COUNTY Westmoreland

NDI# PA - 00478 PENNDER# 65-55

TYPE OF DAM Earth and rockfill SIZE Small HAZARD CATAGORY High

DATE(S) INSPECTION 13 December 1978 WEATHER Cold and windy TEMPERATURE 30° @ 9:00 a.m.

POOL ELEVATION AT TIME OF INSPECTION 1120.9 M.S.L.

TAILWATER AT TIME OF INSPECTION 1094 M.S.L.

INSPECTION PERSONNEL	OWNER REPRESENTATIVES	OTHERS
B. M. Mihalcin	Adrian Horvath	
D. L. Bonk		
S. R. Michalski		
W. J. Veon		

RECORDED BY D. L. Bonk

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 478
SURFACE CRACKS	None observed. The upstream slope is covered with a layer of dumped rock riprap composed of a siliceous limestone known locally as the "Loyalhanna Limestone." The embankment crest is covered with brick paving.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	The downstream slope is constructed of hand-placed mortared stone that is covered to a large extent by thick clumps of grass and topsoil. No unusual cracking or movement was observed, however, the slope is slightly irregular. The toe area has been disturbed somewhat by a recent drilling program carried out as part of an overall study of this facility.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	None observed (see above comments).
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	The embankment is well aligned from abutment to abutment. Differential settlements across the crest were measured to be approximately 1.3 feet maximum.
RIPRAP FAILURES	The riprap layer covering the upstream slope was placed in August 1977, and is presently in good condition. The hand-placed stone covering the downstream slope was observed to be slightly irregular and to some extent disturbed.
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Good condition.

EMBANKMENT

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA -478
DAMP AREAS IRREGULAR VEGETATION (LUSH OR DEAD PLANTS)	No flow was observed directly through the downstream embankment face. No irregularities were noted regarding local vegetation. The downstream toe was observed in a saturated condition with water evident along the right side. A recent drilling program along the toe is suspected to be contributing to this condition but to what extent could not be determined.
ANY NOTICEABLE SEEPAGE	As noted above, the downstream toe is saturated with noticeable flow.
STAFF GAGE AND RECORDER	None.
DRAINS	Two cast iron drains are located to the right of the valve house. Both pipes apparently serve as toe drains. During the inspection, flow from the left drain was estimated at approximately 2-3 gpm while flow from the right drain was estimated at approximately 5 gpm.

OUTLET WORKS

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 478
INTAKE STRUCTURE	Submerged.. Not observed.
OUTLET CONDUIT (CRACKING AND SPALLING OF CONCRETE SURFACES)	12-inch diameter cast iron blowoff with discharge end located immediately downstream of the valve house. The conduit valve was partially opened and the flow was estimated at approximately 200 gpm.
OUTLET STRUCTURE	Dilapidated masonry valve house located at the base of the downstream toe. Replacement of the valve house is to be part of renovation program currently being formulated by a geotechnical consultant.
OUTLET CHANNEL	Discharges directly into Twin Lakes No. 1 Reservoir.
GATE(S) AND OPERATIONAL EQUIPMENT	The only operable device associated with the outlet works at this facility is a gate valve on the blowoff conduit. The valve is located within the valve house at the downstream toe. Presently, the valve is operable, but with difficulty.

EMERGENCY SPILLWAY

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 478
TYPE AND CONDITION	Small concrete channel with no regulating weir. The spillway is located at the right side of the embankment adjacent the right abutment.
APPROACH CHANNEL	No well defined approach to the emergency spillway exists.
SPILLWAY CHANNEL AND SIDEWALLS	New concrete structure roughly 20 feet wide and 26 feet in length. Good condition.
STILLING BASIN PLUNGE POOL	None. Flow through the concrete channel is discharged into a rough cut earth channel of varying cross-section along the downstream right abutment hillside and eventually into Twin Lakes No. 1 Reservoir.
DISCHARGE CHANNEL	Earth channel roughly trapezoidal in cross-section. The channel was reconstructed after the flood of June 1972 (Hurricane Agnes) severely damaged it. A temporary construction road below the spillway has locally lowered the sidewall of the channel. Large discharges may overflow the channel and direct flow along the right abutment and toe area of the embankment.
BRIDGE AND PIERS	A small wooden bridge spans the concrete spillway and connects the right abutment to the embankment crest. The base of the bridge is 2.7 feet above the spillway channel.
EMERGENCY GATES	None.

SERVICE SPILLWAY

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA - 478
TYPE AND CONDITION	N/A - See 5 of 8.
APPROACH CHANNEL	N/A
OUTLET STRUCTURE	N/A
DISCHARGE CHANNEL	N/A

INSTRUMENTATION

ITEM	OBSERVATIONS AND/OR REMARKS NDI# PA -478
MONUMENTATION SURVEYS	None
OBSERVATION WELLS	None
WEIRS	None
PIEZOMETERS	None
OTHERS	Stakes marking the positions of recently drilled test boring holes were observed along the downstream toe of the embankment. A noticeable flow was observed issuing from one probable boring location near the right abutment toe area.

RESERVOIR AREA AND DOWNSTREAM CHANNEL

NDI# PA - 478

ITEM	OBSERVATIONS AND/OR REMARKS
SLOPES: RESERVOIR	Steep and forested.
SEDIMENTATION	During the visual inspection, the pool was being maintained at a drawn down level. Much of the surrounding shore was therefore visible and only a small amount of sedimentation was observed.
DOWNSTREAM CHANNEL (OBSTRUCTIONS, DEBRIS, ETC.)	Discharge from this facility flows directly into Twin Lakes No. 1 Reservoir.
SLOPES: CHANNEL VALLEY	Discharge is directly into Twin Lakes No. 1 Reservoir.
APPROXIMATE NUMBER OF HOMES AND POPULATION	Approximately one-half dozen homes could be affected by minor flooding due to their close proximity to the stream. A major flood could possibly endanger the lives of those persons residing in a trailer park near the community of Luxor approximately 1/2 mile downstream. It is estimated that at least 100 persons could be affected by flows resulting from a sudden embankment breach and subsequent failure of the Twin Lakes No. 1 Dam.

APPENDIX C

HYDROLOGY AND HYDRAULICS

PREFACE

The modified HEC-1 program is capable of performing two basic types of hydrologic analyses: 1) the evaluation of the overtopping potential of the dam; and 2) the estimation of the downstream hydrologic-hydraulic consequences resulting from assumed structural failures of the dam. Briefly, the computational procedures typically used in the dam overtopping analysis are as follows:

a. Development of an inflow hydrograph(s) to the reservoir.

b. Routing of the inflow hydrograph(s) through the reservoir to determine if the event(s) analyzed would overtop the dam.

c. Routing of the outflow hydrograph(s) from the reservoir to desired downstream locations. The results provide the peak discharge(s) of each routed hydrograph at the downstream end of each reach.

The evaluation of the hydrologic-hydraulic consequences resulting from an assumed structural failure (breach) of the dam is typically performed as shown below.

a. Development of an inflow hydrograph(s) to the reservoir.

b. Routing of the inflow hydrograph(s) through the reservoir.

c. Development of a failure hydrograph(s) based on specified breach criteria and normal reservoir outflow.

d. Routing of the failure hydrograph(s) to desired downstream locations. The results provide estimates of the peak discharge(s), time(s) to peak and maximum water surface elevations of failure hydrographs for each location.

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-5-79 PROJ. NO. 79-217-473

CHKD. BY DLB DATE 4-16-79 SHEET NO. 1 OF 15

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DAM STATISTICS

HEIGHT OF DAM ≈ 34 FT (FIELD MEASURED)

MAXIMUM POOL STORAGE CAPACITY ≈ 270 AC-FT [OBTAINED FROM HEC-1 OUTPUT]
@ TOP OF DAM

NORMAL POOL STORAGE CAPACITY ≈ 215 AC-FT (SEE NOTE 1)

DRAINAGE AREA ≈ 0.38 SQ.MI. (TOTAL) [PLANIMETERED OFF USGS 7.5 MINUTE LATROBE, PA QUAD]
0.16 SQ.MI (LOCAL)
0.22 SQ.MI (US OF RAILROAD EMBANKMENT)

NOTE 1: STORAGE VALUE OBTAINED FROM "DAMS, RESERVOIRS AND NATURAL LAKES", WATER RESOURCES BULLETIN No 5, COMMONWEALTH OF PENNSYLVANIA, DEPARTMENT OF FORESTS AND WATER, HARRISBURG, PA. THE REPORTED VALUE WAS 70 MILLION GALLONS.

DAM CLASSIFICATION

DAM SIZE - SMALL (REF 1, TABLE 1)

HAZARD CLASSIFICATION - HIGH (FIELD OBSERVATION)

REQUIRED SDF - 1/2 PMF TO PMF (REF 1, TABLE 3)

SUBJECT	DAM SAFETY INSPECTION		
	UPPER DONOHOE DAM		
BY WJV	DATE 4-5-79	PROJ. NO. 78-617-478	
CHKD. BY DLB	DATE 4-16-79	SHEET NO. 2	OF 15

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HYDROGRAPH PARAMETERS

a) FOR SUB-BASIN UPSTREAM OF 75 FT RAILROAD EMBANKMENT (WHICH IS LOCATED JUST US OF THE UPPER DONOHOE RESERVOIR):

LENGTH OF LONGEST WATERCOURSE $(L) \approx 0.78$ MI

$L_{CA} \approx 0.27$ MI (MEASURED ALONG LONGEST WATERCOURSE FROM EMBANKMENT CULVERT INLET TO CENTROID OF SUB-BASIN)

NOTE 2: VALUES OF L AND L_{CA} ARE MEASURED FROM USGS 7.5 MINUTE LATROBE, PA QUAD.

$C_t \approx 1.6$
$C_p \approx 0.45$
} [SUPPLIED BY COE; ZONE 24, OHIO RIVER BASIN]

t_p = SNYDER'S STANDARD LAG $\approx 1.6 (L \times L_{CA})^{0.3}$

$\therefore t_p \approx 1.6 \left[(0.78)(0.27)\right]^{0.3} \approx 1.0$ HR.

b) FOR LOCAL RESERVOIR SUB-BASIN:

SINCE THE SUB-BASIN CENTROID IS LOCATED WITHIN THE RESERVOIR:

$$^{*}t_p \approx 1.6 (L')^{0.6}$$

WHERE L' = LENGTH ALONG LONGEST WATERCOURSE FROM THE RESERVOIR BOUNDARY TO THE DRAINAGE DIVIDE ≈ 0.18 MI

$\therefore t_p \approx 1.6 (0.18)^{0.6} \approx 0.57$ HR ; $C_p \approx 0.45$ (AS ABOVE)

SUBJECT	DAM SAFETY INSPECTION		
	UPPER DONOHOE DAM		
BY WJV	DATE 4-5-79	PROJ. NO. 79-617-478	
CHKD. BY DLB	DATE 4-16-79	SHEET NO. 3	OF 15

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RESERVOIR SURFACE AREAS

a) FOR UPSTREAM SUB-BASIN :

ELEVATION (FT)	SURFACE AREA (AC)
≈ 1135	0
1140	2.8
1160	10.1
1190	26.6

NOTE 3 : SURFACE AREAS PLANIMETERED OFF THE 7.5 MINUTE LATROBE, PA QUAD. THE "0" SURFACE AREA ELEVATION WAS ESTIMATED BASED ON THE ASSUMPTION THAT THE ≈ 200 FT EMBANKMENT CULVERT SLOPE WAS EQUAL TO THE 2.5% SLOPE OF THE STREAM IMMEDIATELY UPSTREAM FROM THE CULVERT W/ THE CULVERT OUTLET INVERT @ ABOUT EL 1130 (FIG 2, APPENDIX F)

b) FOR UPPER DONOHOE RESERVOIR :

SURFACE AREA (SA) @ NORMAL POOL EL 1126 ≈ 20.1 ACRES

NOTE 4 : SURFACE AREAS PLANIMETERED OFF FIGURE 2, APPENDIX F. NORMAL POOL ELEVATION ALSO OBTAINED FROM THIS FIGURE. THE NOTES OF FIGURE 2 DO NOT REFER TO THE UPPER DONOHOE DAM.

SA @ EL. 1130 ≈ 24.2 ACRES

SA @ EL. 1135 ≈ 29.4 ACRES

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-6-79 PROJ. NO. 78-617-478

CHKD. BY DLB DATE 4-16-79 SHEET NO. 4 OF 15

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RATE OF AREA CHANGE PER FOOT OF RESERVOIR RISE ⇒

$$\Delta A/\Delta H \approx (24.2 - 20.1)\text{ ACRES} / (1130.0 - 1126.0)\text{ FEET} \approx 1.03 \frac{AC}{FT}$$

SA @ TOP OF DAM EL. 1128.7 ≈ $[(1128.7 - 1126.0) \times 1.03 \frac{AC}{FT}] + 20.1$ AC
(LOW TOP OF DAM ELEVATION) ≈ 22.9 ACRES
FIELD MEASURED

RESERVOIR ELEVATION @ "O" STORAGE

NORMAL POOL VOLUME ≈ 1/3 HA ≈ 215 AC-FT (CONIC METHOD)

SA @ NORMAL POOL EL. 1126.0 ≈ 20 AC

$$\therefore \quad H = \frac{3V}{A} \approx 3(215 \text{ AC-FT}) / 20.1 \text{ AC} \approx 32.1 \text{ FT}$$

ZERO VOLUME ELEVATION ≈ 1126.0 FT − 32.1 FT ≈ 1093.9 FT

NOTE 5: ACTUAL MINIMUM ELEVATION @ "O" STORAGE IS PROBABLY LESS THAN THE ABOVE VALUE (BASED ON INFORMATION CONTAINED IN PENN DER FILES). HOWEVER, IN ORDER TO COMPUTE A STORAGE - ELEVATION RELATIONSHIP AND STILL MAINTAIN A STORAGE OF 215 AC-FT @ EL 1126.0, THE ABOVE "O" STORAGE ELEVATION MUST BE INPUT INTO THE HEC-1 PROGRAM

STORAGE - ELEVATION RELATIONSHIP

COMPUTED INTERNALLY BY THE HEC-1 PROGRAM FOR BOTH THE UPSTREAM SUB-BASIN AND THE RESERVOIR BASED ON THEIR RESPECTIVE GIVEN SURFACE AREA VS ELEVATION INFORMATION (SEE SUMMARY INPUT / OUTPUT SHEETS).

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-5-79 PROJ. NO. 79-617-479

CHKD. BY DLB DATE 4-16-79 SHEET NO. 5 OF 15

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PMP CALCULATIONS (FOR BOTH THE UPSTREAM AND LOCAL SUB-BASINS)

- APPROXIMATE RAINFALL INDEX = 24 IN (REF 3, FIG 1)
(CORRESPONDING TO A DURATION OF 24 HR AND AN AREA OF 200 SQ MI LOCATED IN SOUTHWESTERN PENNSYLVANIA)

- DEPTH - AREA - DURATION ZONE #7 (REF 3, FIG 1)

- LOCAL DRAINAGE AREA ≈ 0.16 SQ.MI. AND UPSTREAM DRAINAGE AREA ≈ 0.22 SQ.MI. HOWEVER, THE STORM WILL BE CENTERED OVER THE TOTAL DRAINAGE AREA ABOVE TWIN LAKE Nº1 DAM ≈ 1.99 SQ.MI. (APPENDIX C-1, SHEET 1)
⇒ ASSUME THAT DATA CORRESPONDING TO A 10 SQ MI. DA IS REPRESENTATIVE OF THIS BASIN:

DURATION (HR)	PERCENT OF INDEX RAINFALL (%)
6	102.0
12	120.0
24	130.0

NOTE 6: A 24-HR RATHER THAN A 48-HR DURATION IS USED SO THAT A TIME STEP OF 5-MINUTES CAN BE USED IN THE HEC-1 PROGRAM

- HOP BROOK FACTOR (ADJUSTMENT FOR BASIN SHAPE AS WELL AS FOR THE LESSER LIKELIHOOD OF A SEVERE STORM CENTERING OVER A SMALLER BASIN) CORRESPONDING TO A DA ≈ 1.99 SQ.MI. (<10 SQ.MI.) ≈ 0.80 (REF 4, PG. 48).

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-6-79 PROJ. NO. 73-617-478

CHKD. BY DLB DATE 4-16-79 SHEET NO. 6 OF 15

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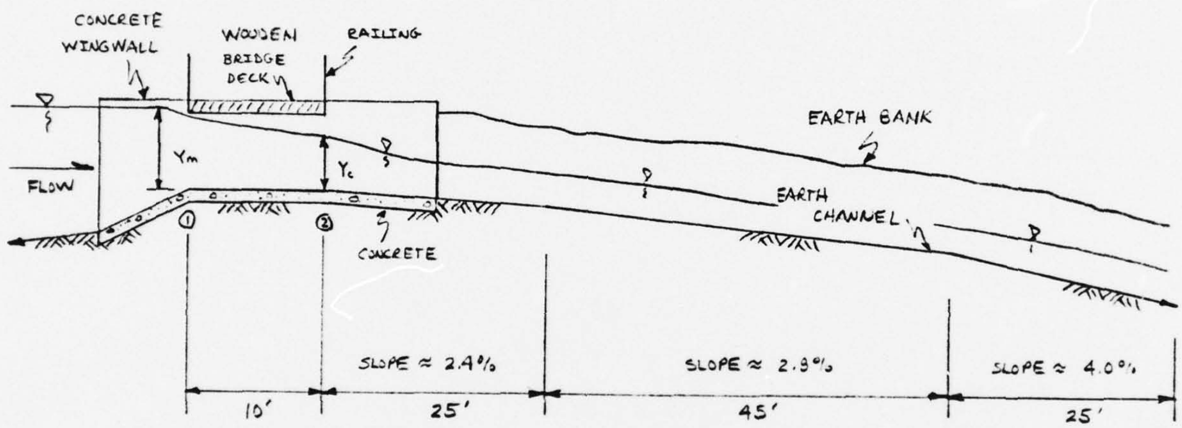
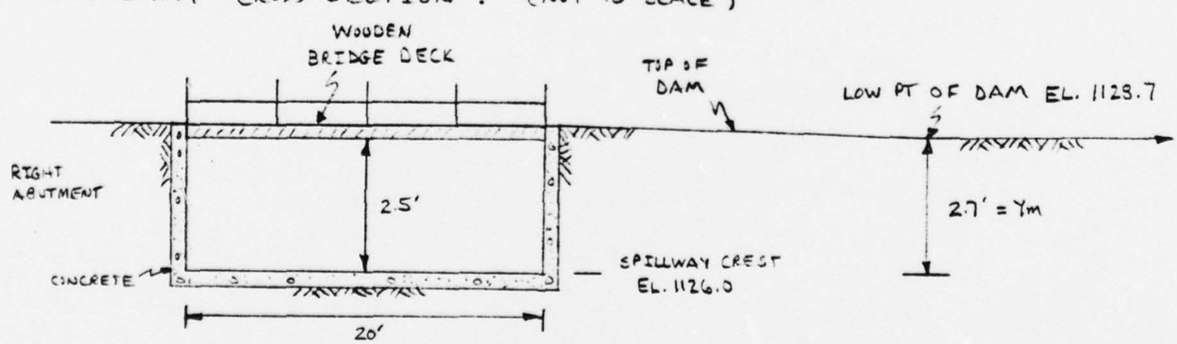
SPILLWAY CAPACITY

- PROFILE OF SPILLWAY : (NOT TO SCALE)



- SPILLWAY CROSS-SECTION : (NOT TO SCALE)



NOTE 7: SPILLWAY PROFILE SLOPES WERE MEASURED IN THE FIELD AS WERE THE LOW TOP OF DAM ELEVATION AND THE SPILLWAY OPENING DIMENSIONS.

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM
BY WJV DATE 4-3-79 PROJ. NO. 73-617-473
CHKD. BY DLB DATE 4-16-79 SHEET NO. 7 OF 15

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- ASSUME THAT THE FLOW CONTROL SECTION IS LOCATED @ SECTION ② AS SHOWN ON SHEET 6 W/ Y_c = CRITICAL DEPTH AND Y_m = MAXIMUM RESERVOIR DEPTH ABOVE SPILLWAY CREST PRIOR TO EMBANKMENT OVERTOPPING ≈ 2.7 FT.

ENERGY BALANCE BETWEEN ① AND ②:

$$Y_m + v_1^2/2g + z_1 = Y_c + v_c^2/2g + z_2 + H_L \qquad \text{(REF 7, PG 40)}$$

WHERE v_1 = RESERVOIR VELOCITY ≈ 0 FPS;
z_1 = CHANNEL ELEVATION @ ① IN FT;
v_c = CRITICAL VELOCITY IN FPS;
z_2 = CHANNEL ELEVATION @ ② IN FT;
H_L = HEAD LOSS BETWEEN ① AND ② ≈ 0 FT

SINCE $z_1 - z_2 \approx 0$ (SECTIONS ① AND ② ARE CLOSE ENOUGH TOGETHER SUCH THAT ΔELEVATION ≈ 0)

$$Y_m = 2.7\text{ FT} = Y_c + v_c^2/2g$$

- SINCE THE CRITICAL SECTION IS RECTANGULAR IN SHAPE,

$$v_c^2/2g = Y_c/2 \qquad \text{(REF 13, PG 143)}$$

$$\therefore \quad 2.7\text{ FT} = Y_c + v_c^2/2g = Y_c + Y_c/2 = \tfrac{3}{2} Y_c$$

$$Y_c \approx 1.8\text{ FT}$$

- SINCE $Y_c \approx 1.8$ FT ⇒ $A_c = 20\,Y_c \approx (20)(1.8) \approx 36\text{ FT}^2$
$v_c \approx \sqrt{2g\,(Y_c/2)} \approx \sqrt{g\,Y_c} \approx \sqrt{(32.2\text{ FT/SEC}^2)(1.8)}$
≈ 7.6 FPS

∴ CAPACITY OF SPILLWAY = $Q = A_c v_c \approx (36\text{ FT}^2)(7.6\text{ FPS}) \approx 274$ CFS
SAY 270 CFS

- CHECK TO SEE IF CRITICAL DEPTH DOES CONTROL @ ② :

CHANNEL SLOPE DS OF ② ≈ 2.4% (SHEET 6)

CRITICAL SLOPE IS DEFINED BY MANNING'S EQ.:

$$S_C \approx \left(\frac{n\, v_c}{1.49\, R^{2/3}} \right)^2 \qquad \text{(REF 13, PG 143)}$$

WHERE n = CHANNEL ROUGHNESS COEFFICIENT ≈ 0.025 (FIELD ESTIMATE FOR EARTH CHANNEL); $v_c \approx 7.6$ FPS (SHEET 7), AND R = HYDRAULIC RADIUS = $\frac{\text{FLOW AREA}}{\text{WETTED PERIMETER}}$ $\approx A_c/P \approx 36\,FT^2/(20+1.8+1.8) \approx 1.53$ FT

$$\therefore\ S_C \approx \left[(0.025)(7.6) / (1.49)(1.53)^{2/3} \right]^2 \approx 0.92\% < 2.4\%$$

⇒ CRITICAL DEPTH CONTROLS @ SECTION ②

NOTE 8: IF THE EMBANKMENT WAS ACTUALLY LEVEL AND AT DESIGN EL 1130 ⇒ $Y_m \approx 4$ FT ⇒ $Y_c \approx 2.67$ FT ⇒ $v_c \approx 9.3$ FPS. THEREFORE, $A_c \approx 20(2.67) \approx 53.4\ FT^2$ ⇒ $Q = A_c v_c \approx (53.4\ FT^2)(9.3\ FPS)$ $Q \approx 500$ CFS (ASSUMING THAT THE SPILLWAY BRIDGE WILL BE WASHED AWAY). IF THE SPILLWAY CAPACITY ACTUALLY WAS ≈ 500 CFS ⇒ THE FACILITY COULD ACCOMMODATE A FLOOD IN EXCESS OF 70% OF THE PMF (SUMMARY INPUT/OUTPUT SHEETS, SHEET H).

SPILLWAY RATING CURVE

COMPUTED INTERNALLY BY HEC-1 VIA THE TRAPEZOIDAL RATING CURVE ROUTINE, BASED ON THE SPILLWAY GEOMETRY AS PRESENTED ON SHEET 6. THE TRAPEZOIDAL ROUTINE CALCULATES CRITICAL CONTROL DISCHARGES IN A WAY SIMILAR TO THAT OUTLINED ON SHEET 7 (SEE SUMMARY INPUT/OUTPUT SHEETS).

SUBJECT	DAM SAFETY INSPECTION				
	UPPER DONOHOE DAM				
BY	WJV	DATE	4-9-79	PROJ. NO.	78-617-479
CHKD. BY	DLB	DATE	4-16-79	SHEET NO.	9 OF 15

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RAILROAD EMBANKMENT CULVERT RATING CURVE

- CULVERT INLET ≈ 2.5 FT (DEPTH) × 3.0 FT (WIDTH) RECTANGULAR MASONRY OPENING (FIELD MEASURED)

- CULVERT OUTLET => REPORTED TO BE A 6 FT DIAMETER PIPE. HOWEVER, ON THE DAY OF INSPECTION THE OUTLET WAS SUBMERGED WITH WATER, DUE TO THE ACCUMULATION OF ABOUT 5± FT OF SEDIMENT IN THE CULVERT AND EXIT CHANNEL. THIS ACCUMULATION OF SEDIMENT IS REPORTED TO BE A COMMON OCCURRENCE W/ DREDGING DONE INFREQUENTLY.

SINCE THE ACTUAL CULVERT OUTLET OPENING SIZE IS NOT KNOWN, AND DUE TO THE LARGE SEDIMENT ACCUMULATIONS WHICH CONSTANTLY CLOG MOST OF THE OUTLET OPENING, THE PERFORMANCE OF THE OUTLET WILL BE ASSUMED TO BE REPRESENTED BY THE PERFORMANCE OF A 2.5 FT × 3.0 FT RECTANGULAR OPENING.

- CULVERT DISCHARGES ARE CONTROLLED BY EITHER THE INLET OR THE OUTLET OF THE CULVERT; DEPENDING ON SUCH FACTORS AS CROSS-SECTIONAL AREA, LENGTH, ROUGHNESS, SLOPE, AND ENTRANCE CONDITIONS OF THE CULVERT BARREL, AS WELL AS HEADWATER AND TAILWATER LEVELS.

- *INLET CONTROL DISCHARGES ARE INDEPENDENT OF TAILWATER DEPTH, AND ARE CONTROLLED BY HEADWATER LEVEL AND ENTRANCE GEOMETRY. FOR H/D (HEADWATER DEPTH TO CULVERT DEPTH RATIO) < 1.2, THE DISCHARGE EQUATION IS:

$$Q = \frac{2}{3} C_B B H \sqrt{\frac{2}{3} g H} \quad \text{(CONSTRICTED FLOW)}$$

WHERE Q = DISCHARGE IN CFS; C_B = END CONSTRICTION COEFFICIENT ≈ 0.9 (SQUARE EDGED ENTRANCE), B = WIDTH OF CULVERT = 3.0 FT, H = HEADWATER DEPTH ABOVE INLET INVERT ELEVATION OF 1135.0 FT, AND g = 32.2 FT/SEC^2.
↳(ESTIMATED, SEE SHEET 3)

FOR H/D > 1.2:

$$Q = C_h B D \sqrt{2g (H - C_h D)}$$

WHERE Q, B, g, AND H ARE AS BEFORE, D = DEPTH OF CULVERT = 2.5 FT, AND C_h = CONTRACTION COEFFICIENT = 0.6 (SQUARE-EDGED ENTRANCE).

*INFORMATION OBTAINED FROM: OPEN CHANNEL FLOW BY F.M. HENDERSON, MACMILLAN PUBLISHING CO., INC., NEW YORK, NEW YORK. 1966 (PG 263)

- INLET CONTROL FLOWS:

ELEVATION (FT)	H (FT)	H/D (FT/FT)	Q (CFS)	ELEVATION (FT)	H (FT)	H/D (FT)	Q (CFS)
1135.0	0	0	0	1142.0	7	2.8	85
1136.0	1	0.4	10	1143.0	8	3.2	90
1137.0	2	0.8	20	1144.0	9	3.6	100
1138.0	3	1.2	40	1145.0	10	4.0	105
1139.0	4	1.6	60	1146.0	11	4.4	110
1140.0	5	2.0	70	1147.0	12	4.8	115
1141.0	6	2.4	80	1148.0	13	5.2	120

- ** OUTLET CONTROL DISCHARGES ARE ESPECIALLY DEPENDENT ON TAILWATER LEVEL, ALONG WITH ALL OTHER CHARACTERISTICS OF THE CULVERT BARREL. OUTLET CONTROL CAN OCCUR IF $H > 0.75\ D$, WITH DISCHARGE DEFINED BY ITS RELATIONSHIP TO HW IN THE EQUATION BELOW:

$$HW = \left[1 + K_e + \frac{29 n^2 L}{R^{1.33}}\right] \frac{Q^2}{2gA^2} + TW - LS_o$$

WHERE HW = WATER SURFACE ELEVATION @ INLET IN FT; K_e = ENTRANCE LOSS COEFFICIENT ≈ 0.4 (WINGWALLS @ 30°-75° TO CULVERT, SEE REF BELOW); $n \approx 0.020$ (FIELD ESTIMATE); $A = 7.5\ FT^2$; $R = \frac{\text{FLOW AREA}}{\text{WETTED PERIMETER}} = 7.5/[3+3+2.5+2.5] \approx 0.69$ FT; L = LENGTH OF CULVERT ≈ 200 FT (ESTIMATED); S_o = SLOPE OF CULVERT ≈ 0.025 (SHEET 3); Q = CULVERT DISCHARGE IN CFS; TW = TAILWATER ELEVATION = ELEVATION OF OUTLET INVERT (≈ 1130.0 FT) + DEPTH OF CULVERT (2.5 FT) FOR Q UP TO FLOW AT WHICH OUTLET CONTROL OVERTAKES INLET CONTROL, THEN ASSUME THAT TW INCREASES ABOVE THIS ELEVATION BY 0.1 FT FOR EVERY 10 CFS OR SO INCREASE IN FLOW.

** INFORMATION OBTAINED FROM: "HYDRAULIC CHARTS FOR THE SELECTION OF HIGHWAY CULVERTS", HEC Nº 5, BUREAU OF PUBLIC ROADS.

- OUTLET CONTROL FLOWS:

Q (CFS)	TW (FT)	LS_o (FT)	HW (FT)	Q (CFS)	TW (FT)	LS_o (FT)	HW (FT)
60	1132.5	5	1132.7	120	1132.6	5	1148.6
70	1132.5	5	1134.6	130	1132.7	5	1152.3
80	1132.5	5	1136.8	140	1132.8	5	1156.3
90	1132.5	5	1139.3	150	1132.9	5	1160.7
100	1132.5	5	1142.1	160	1133.0	5	1165.3
110	1132.5	5	1145.1	170	1133.1	5	1170.2

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 4-9-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-16-79 SHEET NO. 12 OF 15

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- TOTAL CULVERT RATING CURVE:

HEADWATER ELEVATION (FT)	① Q_{INLET} (CFS)	② Q_{OUTLET} (CFS)	Q (CFS)
1135.0	0	—	0
1136.0	10	76	10
1137.0	20	81	20
1138.0	40	85	40
1139.0	60	89	60
1140.0	70	93	70
1141.0	80	96	80
1142.0	85	100	85
1143.0	90	103	90
1144.0	100	106	100
1145.0	105	110	105
1146.0	110	113	110
1147.0	115	116	115
1147.4	117	117	117
1148.6		120	120
1152.3		130	130
1156.3		140	140
1160.7		150	150
1165.3		160	160
1170.2		170	170

① FROM SHEET 10
② FROM SHEET 11

SUBJECT	DAM SAFETY INSPECTION UPPER DONOHOE DAM
BY WJY	DATE 4-9-79
PROJ. NO.	78-617-478
CHKD. BY DLB	DATE 4-16-79
SHEET NO.	13 OF 15

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DAM EMBANKMENT RATING CURVE

- ALTHOUGH THE EMBANKMENT CREST IS LINED WITH ≈ 0.6 FT WOODEN RAILROAD TIES, ANY ADDITIONAL STORAGE WHICH THEY MIGHT PROVIDE WILL BE NEGLECTED SINCE THEY ARE NOT CONSIDERED TO BE PERMANENT STRUCTURES ⇒ ASSUME THE WOODEN TIES ARE REMOVED.

- FLOWS OVER THE EMBANKMENT WILL BE COMPUTED INTERNALLY BY HEC-1 VIA THE ASSUMPTION THAT CRITICAL DEPTH OCCURS ON THE CREST W/ THE CREST PROFILE REPRESENTED BY A SERIES OF TRAPEZOIDS. (SEE SUMMARY INPUT/OUTPUT SHEETS FOR RATING INFORMATION).

- INPUT INFORMATION : (BASED ON FIELD MEASUREMENTS)

RESERVOIR ELEVATION (FT)	DEPTH OF WATER ABOVE CREST (FT)	LENGTH OF CREST INUNDATED (FT)	
1128.7	0	0	
1128.8	0.1	75	
1128.9	0.2	175	
1129.0	0.3	225	
1129.1	0.4	325	
1129.2	0.5	375	
1129.3	0.6	420	
1129.5	0.8	500	
1129.6	0.9	520	
1129.7	1.0	521	ASSUME 20% SLOPES TO THE RIGHT AND LEFT OF THE EMBANKMENT
1130.2	1.5	526	
1130.7	2.0	531	

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY WJV DATE 5-4-79 PROJ. NO. 79-617-487

CHKD. BY DLB DATE 5-4-79 SHEET NO. 14 OF 15

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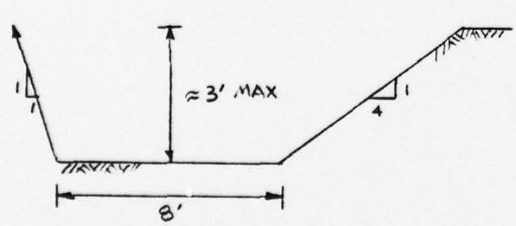
ESTIMATE OF ACTUAL SPILLWAY CAPACITY

- ALTHOUGH THE HYDROLOGIC / HYDRAULIC EVALUATION OF THIS FACILITY WILL BE PERFORMED ASSUMING THAT THE SPILLWAY CHUTE CHANNEL IS IN IDEAL CONDITION (SEE SECTION 5.3), SOME ESTIMATE OF THE ACTUAL PRESENT SPILLWAY SYSTEM CONSEQUENCES AND CAPACITY SHOULD BE MADE.

- DUE TO THE PRESENCE OF THE BREACH IN THE LEFT CHUTE CHANNEL WALL (CAUSED BY THE PLACEMENT OF A TEMPORARY EARTH AND ROCK ROAD ACROSS THE CHUTE CHANNEL @ ABOUT 35 FT DS FROM THE SPILLWAY CREST), SOME SPILLWAY DISCHARGE WILL FLOW TOWARD THE TOE WHEN THE CHANNEL DEPTH EXCEEDS ABOUT 1.5 FT OR SO.

- THE ACTUAL PRESENT CRITICAL FLOW CONTROL SECTION IS LOCATED ABOUT 70 FT DS FROM THE SPILLWAY (SEE SKETCH ON SHEET 6). THE APPROXIMATE CROSS-SECTION DIMENSIONS ARE GIVEN IN THE SKETCH BELOW. ASSUMING UNIFORM FLOW ABOVE THIS SECTION (REF 7, PG. 5), THE MAXIMUM DEPTH OF FLOW UPSTREAM FROM THE SECTION WILL BE ≈ 2.7 FT (CORRESPONDING TO THE MAXIMUM SPILLWAY DEPTH PRIOR TO OVERTOPPING W/O THE CRITICAL CONTROL ON THE CREST). ALSO, IGNORING THE POSSIBLE FLOW THROUGH THE LEFT CHUTE CHANNEL WALL BREACH THE SPILLWAY CAPACITY CAN BE FOUND



(NOT TO SCALE)

FROM: $Y_m = Y_c + v_c^2/2g \Rightarrow 2.7 = Y_c + v_c^2/2g$. SINCE $A_c \approx 8Y_c + 2.5Y_c^2$ AND B_c = (TOP WIDTH) = $8 + 5Y_c$ (FROM GEOMETRY); $Q = A_c v_c$; AND $Q^2 B_c = g A_c^3$ (REF 13, PG 141) $\Rightarrow$

$$Q \approx \sqrt{g[8Y_c+2.5Y_c^2]^3/[8+5Y_c]}\quad \text{, AND}$$

(ASSUMING APPROACH VELOCITY HEAD AND CHANNEL LOSSES ARE NEGLIGIBLE)

$$2.7 = Y_c + \frac{Q^2}{2g\,(8Y_c+2.5Y_c^2)^2}$$

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM
BY WJV DATE 5-4-79 PROJ. NO. 78-617-487
CHKD. BY DLB DATE 5-4-79 SHEET NO. 15 OF 15

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THEREFORE, BY TRIAL AND ERROR $\Rightarrow$ $Y_c \approx 2.0$ FT
$Q \approx 180$ CFS

THUS, THE CAPACITY OF THE ACTUAL PRESENT SPILLWAY SYSTEM IS ABOUT 2/3 OF THAT COMPUTED FOR THE PROPOSED REHABILITATED SPILLWAY SYSTEM (SHEET 7). HOWEVER, DUE TO THE BREACH IN THE LEFT SPILLWAY CHANNEL WALL, THE ABOVE COMPUTED CAPACITY PRIOR TO OVERTOPPING WILL ACTUALLY BE SOMEWHAT MORE.

SUBJECT: DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY: WJV DATE: 4-20-79 PROJ. NO. 79-617-473

CHKD. BY: DLB DATE: 4-21-79 SHEET NO. A OF H

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OVERTOPPING

SUMMARY INPUT/OUTPUT SHEETS

```
DAM SAFETY INSPECTION
UPPER DONOHOE + TWIN LAKES NO 1 DAMS ****** OVERTOPPING ANALYSIS ******
5-MINUTE TIME STEPS AND 24-HOUR STORM DURATION

                                JOB SPECIFICATION
   NQ      NHR     NMIN     IDAY      IHR     IMIN    METRC     IPLT     IPRT    NSTAN
  288        0        5        0        0        0        0        0        0        0
                           JOPER      NWT    LROPT    TRACE
                               5        0        0        0

                          MULTI-PLAN ANALYSES TO BE PERFORMED
                            NPLAN= 1 NRTIO= 6 LRTIO= 1
  RTIOS=      .30      .40      .50      .60      .70     1.00

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

                              SUB-AREA RUNOFF COMPUTATION

  INFLOW INTO ARTIFICIAL RESERVOIR CAUSED BY RAILROAD EMBANKMENT AND CULVERT

              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      .22     0.00     0.38     0.00    0.000        0        1        0

                                      PRECIP DATA
              SPFE      PMS       R6      R12      R24      R48      R72      R96
              0.00    24.00   102.00   120.00   130.00     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=  1.00      CP= .45      NTA=  0

                                    RECESSION DATA
                        STRTQ=    -1.50    QRCSN=     -.05     RTIOR= 2.00
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC=12.50 AND R=19.13 INTERVALS
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SUBJECT DAM SAFETY INSPECTION

UPPER DONOHOE DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-21-79 SHEET NO. 8 OF 14

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UNIT HYDROGRAPH10 END-OF-PERIOD ORDINATES, LAG= 1.01 HOURS, CP= .45 VOL= .99

1.	5.	11.	17.	25.	33.	41.	49.	55.	60.
64.	65.	65.	67.	59.	56.	53.	50.	48.	45.
43.	41.	39.	37.	35.	33.	31.	30.	28.	27.
25.	24.	23.	22.	21.	20.	19.	18.	17.	16.
15.	14.	14.	13.	12.	12.	11.	10.	10.	9.
9.	8.	8.	8.	7.	7.	7.	6.	6.	6.
5.	5.	5.	5.	4.	4.	4.	4.	3.	3.
3.	3.	3.	3.	3.	2.	2.	2.	2.	2.
2.	2.	2.	2.	2.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.

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	24.96	23.08	1.88	38450.
										(634.)	(586.)	(48.)	(1088.78)

INFLOWS US OF RAILROAD EMBANKMENT CULVERT

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	643.	396.	133.	133.	38435.
CMS	18.	11.	4.	4.	1088.
INCHES		16.75	22.57	22.57	22.57
MM		425.33	573.32	573.32	573.32
AC-FT		196.	265.	265.	265.
THOUS CU M		242.	327.	327.	327.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	321.	198.	67.	67.	19217.
CMS	9.	6.	2.	2.	544.
INCHES		8.37	11.29	11.29	11.29
MM		212.66	286.66	286.66	286.66
AC-FT		98.	132.	132.	132.
THOUS CU M		121.	163.	163.	163.

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	386.	238.	80.	80.	23061.
CMS	11.	7.	2.	2.	653.
INCHES		10.05	13.54	13.54	13.54
MM		255.20	343.99	343.99	343.99
AC-FT		118.	159.	159.	159.
THOUS CU M		145.	196.	196.	196.

SUBJECT DAM SAFETY INSPECTION
UPPER DONOHOE DAM
BY WJV DATE 4-20-79 PROJ. NO. 79-617-478
CHKD. BY DLB DATE 4-21-79 SHEET NO. C OF H

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HYDROGRAPH ROUTING

ROUTE THROUGH ARTIFICIAL RESERVOIR AND INTO UPPER DONOHOE DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
101	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	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-1135.	-1

STAGE	1135.00	1136.00	1137.00	1138.00	1139.00	1140.00	1141.00	1142.00	1143.00
	1145.00	1146.00	1147.00	1147.40	1148.60	1152.30	1156.30	1160.70	1165.30
FLOW	0.00	10.00	20.00	40.00	60.00	70.00	80.00	85.00	90.00
	105.00	110.00	115.00	117.00	120.00	130.00	140.00	150.00	160.00

SURFACE AREA=	0.	3.	10.	27.
CAPACITY=	0.	5.	126.	480.
ELEVATION=	1135.	1140.	1160.	1180.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
1180.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1180.0	0.0	0.0	0.

OUTFLOWS FROM RAILROAD EMBANKMENT CULVERT

PEAK OUTFLOW IS 152. AT TIME 20.33 HOURS

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	152.	150.	73.	73.	21088.
CMS	4.	4.	2.	2.	597.
INCHES		6.35	12.38	12.38	12.38
MM		161.29	314.56	314.56	314.56
AC-FT		74.	145.	145.	145.
THOUS CU M		92.	179.	179.	179.

PEAK OUTFLOW IS 125. AT TIME 19.42 HOURS

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	125.	122.	55.	55.	15846.
CMS	4.	3.	2.	2.	449.
INCHES		5.15	9.31	9.31	9.31
MM		130.81	236.36	236.36	236.36
AC-FT		60.	109.	109.	109.
THOUS CU M		74.	135.	135.	135.

SUBJECT: DAM SAFETY INSPECTION — UPPER DONOHOE DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-21-79 SHEET NO. D OF H

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0.6 PMF

OUTFLOW FROM RR CULVERT

PEAK OUTFLOW IS 132. AT TIME 19.67 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	132.	129.	60.	60.	17182.
CMS	4.	4.	2.	2.	487.
INCHES		5.46	10.09	10.09	10.09
MM		138.60	256.29	256.29	256.29
AC-FT		64.	118.	118.	118.
THOUS CU M		79.	146.	146.	146.

SUB-AREA RUNOFF COMPUTATION

LOCAL INFLOW INTO UPPER DONOHOE DAM RESERVOIR

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

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	.16	0.00	0.38	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	24.00	102.00	120.00	130.00	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= .57 CP= .45 NTA= 0

RECESSION DATA

STRTQ= -1.50 QRCSN= -.05 RTIOR= 2.00

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 7.18 AND R=10.95 INTERVALS

UNIT HYDROGRAPH 62 END-OF-PERIOD ORDINATES, LAG= .57 HOURS, CP= .45 VOL= 1.00

4.	15.	30.	48.	64.	76.	82.	80.	73.	67.
61.	56.	51.	46.	42.	39.	35.	32.	29.	27.
24.	22.	20.	19.	17.	15.	14.	13.	12.	11.
10.	9.	8.	7.	7.	6.	6.	5.	5.	4.
4.	4.	3.	3.	3.	2.	2.	2.	2.	2.
2.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.								

SUBJECT DAM SAFETY INSPECTION

UPPER DONOHOE DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-21-79 SHEET NO. E OF 14

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END-OF-PERIOD FLOW

0 MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
									SUM	24.96	23.08	1.88	28250.
										(634.)(	586.)(	48.)(	799.95)

LOCAL UPPER DONOHOE RESERVOIR INFLOW

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	626.	309.	98.	98.	28239.
CMS	18.	9.	3.	3.	800.
INCHES		17.97	22.80	22.80	22.80
MM		456.35	579.19	579.19	579.19
AC-FT		153.	194.	194.	194.
THOUS CU M		189.	240.	240.	240.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	313.	155.	49.	49.	14119.
CMS	9.	4.	1.	1.	400.
INCHES		8.98	11.40	11.40	11.40
MM		228.18	289.59	289.59	289.59
AC-FT		77.	97.	97.	97.
THOUS CU M		95.	120.	120.	120.

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	375.	185.	59.	59.	16943.
CMS	11.	5.	2.	2.	480.
INCHES		10.78	13.68	13.68	13.68
MM		273.81	347.51	347.51	347.51
AC-FT		92.	117.	117.	117.
THOUS CU M		113.	144.	144.	144.

COMBINE HYDROGRAPHS

COMBINE RAILROAD EMBANKMENT OUTFLOWS W/ LOCAL INFLOWS FOR TOTAL INFLOW

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

TOTAL UPPER DONOHOE RESERVOIR INFLOWS

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	749.	431.	171.	171.	49327.
CMS	21.	12.	5.	5.	1397.
INCHES		10.54	16.77	16.77	16.77
MM		267.83	425.98	425.98	425.98
AC-FT		214.	340.	340.	340.
THOUS CU M		263.	419.	419.	419.

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	410.	253.	104.	104.	29965.
CMS	12.	7.	3.	3.	849.
INCHES		6.20	10.19	10.19	10.19
MM		157.45	258.78	258.78	258.78
AC-FT		126.	206.	206.	206.
THOUS CU M		155.	255.	255.	255.

SUBJECT DAM SAFETY INSPECTION

UPPER DONOHOE DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-21-79 SHEET NO. F OF H

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0.6 PMF

TOTAL UPPER DONOHOE RESERVOIR INFLOW

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	481.	290.	118.	118.	34125.
CMS	14.	8.	3.	3.	966.
INCHES		7.10	11.60	11.60	11.60
MM		180.23	294.70	294.70	294.70
AC-FT		144.	235.	235.	235.
THOUS CU M		177.	290.	290.	290.

HYDROGRAPH ROUTING

ROUTE TOTAL INFLOW HYDROGRAPH THROUGH UPPER DONOHOE DAM RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
202	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	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-1126.	1

SURFACE AREA=	0.	20.	23.	24.	29.
CAPACITY=	0.	215.	273.	304.	437.
ELEVATION=	1094.	1126.	1129.	1130.	1135.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
1126.0	20.0	0.0	0.0	0.0	0.0	0.0	0.0

IABCOA	ISPITW	ISPCTW
10	0	0

SS	NGATES	DESHD	APEL	APWID	APLOSS	POPTH
0.00	1	2.7	0.0	0.0	0.0	0.0

1093.90	0.	0.	0.0	0.	0.	0.0	0.	0.	0.
1126.00	215.	0.	0.0	0.	0.	0.0	0.	0.	0.
1128.70	273.	267.	1.8	267.	0.	0.0	0.	267.	267.
1130.00	304.	474.	2.7	474.	0.	0.0	0.	474.	474.
1135.00	437.	1518.	6.0	1518.	0.	0.0	0.	1518.	1518.

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1128.7	0.0	0.0	0.

CREST LENGTH AT OR BELOW	0.	175.	225.	325.	375.	420.	500.	521.	526.	531.
ELEVATION	1128.7	1128.9	1129.0	1129.1	1129.2	1129.3	1129.5	1129.7	1130.2	1130.7

SUBJECT: DAM SAFETY INSPECTION
UPPER DONOHOE DAM

BY: WJV DATE: 4-20-79 PROJ. NO. 79-617-479

CHKD. BY: DLB DATE: 4-21-79 SHEET NO. 6 OF 14

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UPPER DONOHOE OUTFLOWS

OVERTOPPING @ ≈0.54 PMF

PEAK OUTFLOW IS 640. AT TIME 16.58 HOURS

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	640.	389.	151.	151.	43404.
CMS	18.	11.	4.	4.	1229.
INCHES		9.52	14.76	14.76	14.76
MM		241.72	374.84	374.84	374.84
AC-FT		193.	299.	299.	299.
THOUS CU M		238.	369.	369.	369.

PEAK OUTFLOW IS 250. AT TIME 18.33 HOURS

0.5 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	250.	219.	88.	88.	25458.
CMS	7.	6.	3.	3.	721.
INCHES		5.37	8.66	8.66	8.66
MM		136.40	219.85	219.85	219.85
AC-FT		109.	175.	175.	175.
THOUS CU M		134.	216.	216.	216.

PEAK OUTFLOW IS 300. AT TIME 18.08 HOURS

0.6 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	300.	251.	101.	101.	29140.
CMS	9.	7.	3.	3.	825.
INCHES		6.14	9.91	9.91	9.91
MM		156.07	251.65	251.65	251.65
AC-FT		124.	201.	201.	201.
THOUS CU M		154.	248.	248.	248.

SUBJECT DAM SAFETY INSPECTION

UPPER DONOHOE DAM

BY WJV DATE 4-20-79 PROJ. NO. 79-617-478

CHKD. BY DLB DATE 4-21-79 SHEET NO. H OF H

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SUMMARY OF DAM SAFETY ANALYSIS

RAILROAD EMBANKMENT

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1135.00	1180.00	1180.00
STORAGE	0.	480.	480.
OUTFLOW	0.	190.	190.

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
.30	1144.68	0.00	21.	103.	0.00	18.92	0.00
.40	1147.68	0.00	34.	118.	0.00	19.17	0.00
.50	1150.53	0.00	50.	125.	0.00	19.42	0.00
.60	1153.15	0.00	68.	132.	0.00	19.67	0.00
.70	1155.53	0.00	86.	138.	0.00	19.83	0.00
1.00	1161.66	0.00	144.	152.	0.00	20.33	0.00

UPPER DONOHOE DAM

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1126.00	1126.00	1128.70
STORAGE	215.	215.	273.
OUTFLOW	0.	0.	267.

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
.30	1127.72	0.00	251.	170.	0.00	18.50	0.00
.40	1128.14	0.00	260.	211.	0.00	18.42	0.00
.50	1128.53	0.00	269.	250.	0.00	18.33	0.00
.60	1128.85	.15	277.	300.	2.08	18.08	0.00
.70	1129.03	.33	281.	380.	3.08	17.50	0.00
1.00	1129.30	.60	287.	640.	4.33	16.58	0.00

1137.0	2	0.8	20	1144.0	9	3.6	100
1138.0	3	1.2	40	1145.0	10	4.0	105
1139.0	4	1.6	60	1146.0	11	4.4	110
1140.0	5	2.0	70	1147.0	12	4.8	115
1141.0	6	2.4	80	1148.0	13	5.2	120

LIST OF REFERENCES

1. "Recommended Guidelines for Safety Inspection of Dams," prepared by Department of the Army Office of the Chief of Engineers, Washington, D. C. (Appendix D).

2. "Unit Hydrograph Concepts and Calculations," by Corps of Engineers, Baltimore District (L-519).

3. "Seasonal Variation of Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Duration of 6, 12, 24, and 48 Hours," Hydrometeorological Report No. 33, prepared by J. T. Riedel, J. F. Appleby and R. W. Schloemer Hydrologic Service Division Hydrometeorological Section, U. S. Department of the Army, Corps of Engineers, Washington, D. C., April 1956.

4. <u>Design of Small Dams</u>, U. S. Department of the Interior, Bureau of Reclamation, Washington, D. C., 1973.

5. <u>Handbook of Hydraulic</u>, H. W. King and E. F. Brater, McGraw-Hill, Inc., New York, 1963.

6. <u>Standard Handbook for Civil Engineers</u>, F. S. Merritt McGraw-Hill, Inc., New York, 1968.

7. <u>Open-Channel Hydraulics</u>, V. T. Chow, McGraw-Hill, Inc., New York, 1959.

8. <u>Weir Experiments, Coefficients, and Formulas</u>, R. E. Horton, Water Supply and Irrigation Paper No. 200, Department of the Interior, United States Geological Survey, Washington, D. C., 1907.

9. "Probable Maximum Precipitation Susquehanna River Drainage Above Harrisburg, Pennsylvania," Hydrometeorological Report 40, prepared by H. V. Goodyear and J. T. Riedel, Hydrometeorological Branch Office of Hydrology, U. S. Weather Bureau, U. S. Department of Commerce, Washington, D. C., May 1965.

10. <u>Flood Hydrograph Package (HEC-1) Dam Safety Version</u>, Hydrologic Engineering Center, U. S. Army Corps of Engineers, Davis, California, July 1978.

11. "Simulation of Flow Through Broad Crest Navigation Dams with Radial Gates," R. W. Schmitt, U. S. Army Corps of Engineers, Pittsburgh District.

12. "Hydraulics of Bridge Waterways," BPR, 1970, Discharge Coefficient Based on Criteria for Embankment Shaped Weirs, Figure 24, page 46.

13. Applied Hydraulics in Engineering, Morris, Henry M. and Wiggert, James N., Virginia Polytechnic Institute and State University, 2nd Edition, The Ronald Press Company, New York, 1972.

14. Standard Mathematical Tables, 21st Edition, The Chemical Rubber Company, 1973, page 15.

15. Engineering Field Manual, U. S. Department of Agriculture, Soil Conservation Service, 2nd Edition, Washington, D. C. 1969.

APPENDIX C-1

SUPPLEMENTAL CALCULATIONS

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-2-79 PROJ. NO. 78-617-497

CHKD. BY DLB DATE 4-13-79 SHEET NO. 1 OF 8

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DAM STATISTICS

HEIGHT OF DAM ≈ 31 FEET (FIELD MEASURED)

MAXIMUM POOL STORAGE CAPACITY ≈ 470 AC-FT [OBTAINED FROM HEC-1 OUTPUT]
@ TOP OF DAM

NORMAL POOL STORAGE CAPACITY ≈ 340 AC-FT (SEE NOTE 1)

DRAINAGE AREA ≈ 1.51 SQ.MI. (LOCAL) [PLANIMETERED OFF USGS 7.5 MINUTE SERIES QUAD, LATROBE, PA.]
1.89 SQ.MI. (TOTAL)

NOTE 1: STORAGE CAPACITY VALUE WAS OBTAINED FROM "DAMS, RESERVOIRS, AND NATURAL LAKES", WATER RESOURCES BULLETIN Nº 5, COMMONWEALTH OF PENNSYLVANIA, DEPARTMENT OF FORESTS AND WATERS, HARRISBURG, PA., 1970. THE REPORTED VALUE WAS 110 MILLION GALLONS. THIS VALUE WAS ALSO INDICATED ON FIGURE 2, APPENDIX F.

DAM CLASSIFICATION

DAM SIZE - SMALL (REF 1, TABLE 1)

HAZARD CLASSIFICATION - HIGH (FIELD OBSERVATION)

REQUIRED SDF - 1/2 PMF TO PMF (REF 1, TABLE 3)

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No. 1 DAM

BY WJV DATE 4-2-79 PROJ. NO. 78-617-497

CHKD. BY DLB DATE 4-13-79 SHEET NO. 2 OF 8

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HYDROGRAPH PARAMETERS

LENGTH OF LONGEST WATERCOURSE (L) ≈ 1.36 MI. (SEE NOTE 2)

$L_{CA} \approx 0.81$ MI [MEASURED ALONG THE LONGEST WATERCOURSE FROM THE DAM CREST TO THE CENTROID OF THE REPRESENTATIVE SUB-BASIN]

NOTE 2: THREE INDEPENDENT STREAMS (BESIDES THE SMALL STREAM GENERATED BY THE OUTFLOWS OF THE UPSTREAM UPPER DONOHOE DAM) DRAIN THE LOCAL 1.51 SQ.MI. BASIN. EACH OF THE STREAMS ENTERS THE RESERVOIR AT A DISTINCTLY DIFFERENT POINT AND COLLECTS RUNOFF FROM ABOUT 1/3 OF THE LOCAL AREA (SEE REGIONAL VICINITY MAP, APPENDIX G). THE L AND LCA PARAMETERS ARE ALSO APPROXIMATELY THE SAME FOR EACH STREAM SUB-BASIN. THEREFORE, INSTEAD OF CONSIDERING A SEPARATE LOCAL RESERVOIR INFLOW HYDROGRAPH FOR EACH OF THE STREAMS, ONLY ONE LARGER LOCAL INFLOW HYDROGRAPH WILL BE COMPUTED IN THE HEC-1 ANALYSIS. THIS IS DONE UNDER THE ASSUMPTION THAT A HYDROGRAPH GENERATED BY APPLYING A RAINFALL DISTRIBUTION TO A NUMBER (3) OF SEPARATE BUT QUANTITATIVELY EQUAL UNIT HYDROGRAPHS AND ADDING THE RESULTS CAN BE APPROXIMATED BY APPLYING THE RAINFALL DISTRIBUTION TO A UNIT HYDROGRAPH WHICH IS A NUMBER (3) TIMES LARGER THAN ANY ONE OF THE SEPERATE BUT EQUAL UNIT GRAPHS. THE LARGE UNIT GRAPH TO BE COMPUTED BY HEC-1 WILL BE BASED ON THE ENTIRE LOCAL DRAINAGE AREA AS WELL AS THE L AND LCA VALUES ABOVE WHICH WERE MEASURED FOR THE SUB-BASIN WHICH CONTAINED THE LARGEST OF THE THREE STREAMS (SEE REGIONAL VICINITY MAP, REPRESENTATIVE SUB-BASIN). (VALUES OF L AND LCA WERE MEASURED FROM THE USGS 7.5 MINUTE LATROBE, PA QUAD)

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 78-617-497

CHKD. BY DLB DATE 4-13-79 SHEET NO. 3 OF 8

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$$\left.\begin{array}{l} C_t \approx 1.6 \\ C_p \approx 0.45 \end{array}\right\}$$

[SUPPLIED BY COE ; ZONE 24, OHIO RIVER BASIN]

$$\therefore \quad t_p = \text{SNYDER'S STANDARD LAG} = 1.6\,(L \times L_{CA})^{0.3}$$

$$t_p = 1.6\,[(1.36)\times(0.81)]^{0.3} \approx 1.65 \text{ HR}$$

RESERVOIR SURFACE AREAS

SURFACE AREA (SA) @ NORMAL POOL EL. 1094.0 ≈ 33.3 ACRES

NOTE 3: SURFACE AREA VALUES WERE OBTAINED FROM FIGURE 2, APPENDIX F, BY PLANIMETERING THE AREAS BETWEEN THE RESPECTIVE CONTOUR LINES AND THE DAM CREST. ACTUAL NORMAL POOL ELEVATION OF 1094.0 WAS OBTAINED FROM A COMBINATION OF FIGURES 2 AND 6, APPENDIX F. (CONSTRUCTION DRAWINGS IN APPENDEX F ARE 3 FT LOWER THAN ACTUAL ELEVATIONS ⇒ SEE NOTES ON FIG. 2)

SA @ EL. 1100 FT ≈ 42.4 ACRES

RATE OF AREA CHANGE PER FOOT OF RESERVOIR RISE:

$$\Delta A / \Delta H = (42.4 - 33.3) \text{ ACRES} / (1100.0 - 1094.0) \text{ FEET}$$

$$\Delta A / \Delta H \approx 1.5 \text{ ACRES/FOOT}$$

SA @ TOP OF DAM EL. 1097.7 ≈ (1.5 AC/FT)(1097.7 - 1094.0) + 33.3 AC
(LOW TOP OF DAM ELEVATION ⇒ FIELD MEASURED)

SA @ EL. 1097.7 ≈ 38.9 ACRES

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 79-617-497

CHKD. BY DLB DATE 4-13-79 SHEET NO. 4 OF 8

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RESERVOIR ELEVATION @ "O" STORAGE

NORMAL POOL VOLUME $\approx$ 1/3 HA $\approx$ 340 AC-FT (CONIC METHOD)

SA @ NORMAL POOL EL. 1094.0 $\approx$ 33.3 ACRES

$\therefore H = \frac{3V}{A} \approx 3(340 \text{ AC-FT})/(33.3 \text{ ACRES}) \approx 30.6 \text{ FT}$

ZERO VOLUME ELEVATION $\approx$ 1094.0 - 30.6 $\approx$ 1063.4 FT

NOTE 4: ALTHOUGH THE ACTUAL MINIMUM RESERVOIR ELEVATION @ "O" STORAGE IS $\approx$ 1072.0 FT. (FIG. 7, APPENDIX F), IN ORDER TO COMPUTE A STORAGE-DISCHARGE RELATIONSHIP AND STILL MAINTAIN A STORAGE OF 340 AC-FT @ NORMAL POOL, THE ABOVE CALCULATED "O" STORAGE ELEVATION OF 1063.4 MUST BE INPUT INTO THE HEC-1 PROGRAM.

STORAGE - ELEVATION RELATIONSHIP

COMPUTED INTERNALLY BY THE HEC-1 PROGRAM, BASED ON GIVEN SURFACE AREA VS ELEVATION INFORMATION. (SEE SUMMARY INPUT/OUTPUT SHEETS)

SUBJECT	DAM SAFETY INSPECTION		
	TWIN LAKES Nº 1 DAM		
BY WJV	DATE 4-3-79	PROJ. NO. 79-617-487	
CHKD. BY DLR	DATE 4-13-79	SHEET NO. 5	OF 8

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PMP CALCULATIONS

- APPROXIMATE RAINFALL INDEX = 24 INCHES (REF. 3, FIG. 1)
  (CORRESPONDING TO A DURATION OF 24 HRS AND A DRAINAGE AREA OF 200 SQ. MI LOCATED IN SOUTHWESTERN PENNSYLVANIA)

- DEPTH - AREA - DURATION ZONE #7 (REF 3, FIG 1)

- ALTHOUGH THE LOCAL DRAINAGE AREA ≈ 1.51 SQ. MI., THE AREA OVER WHICH THE PMP WILL BE CENTERED IS THE TOTAL 1.89 SQ. MI. BASIN AREA ⇒ ASSUME THAT DATA CORRESPONDING TO A 10 SQ. MI. AREA IS REPRESENTATIVE OF THIS BASIN :

DURATION (HR)	PERCENT OF INDEX RAINFALL (%)
6	102.0
12	120.0
24	130.0

NOTE 5: A 24-HR RATHER THAN A 48-HR DURATION IS USED SO THAT A TIME STEP OF 5-MINUTES CAN BE USED IN THE HEC-1 PROGRAM

- HOP BROOK FACTOR (ADJUSTMENT FOR BASIN SHAPE AS WELL AS FOR THE LESSER LIKELIHOOD OF A SEVERE STORM CENTERING OVER A SMALLER BASIN) CORRESPONDING TO A DA = 1.89 SQ. MI. (< 10 SQ. MI.) ≈ 0.80 (REF 4, PG 48).

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 79-617-437

CHKD. BY DLB DATE 4-13-79 SHEET NO. 6 OF 8

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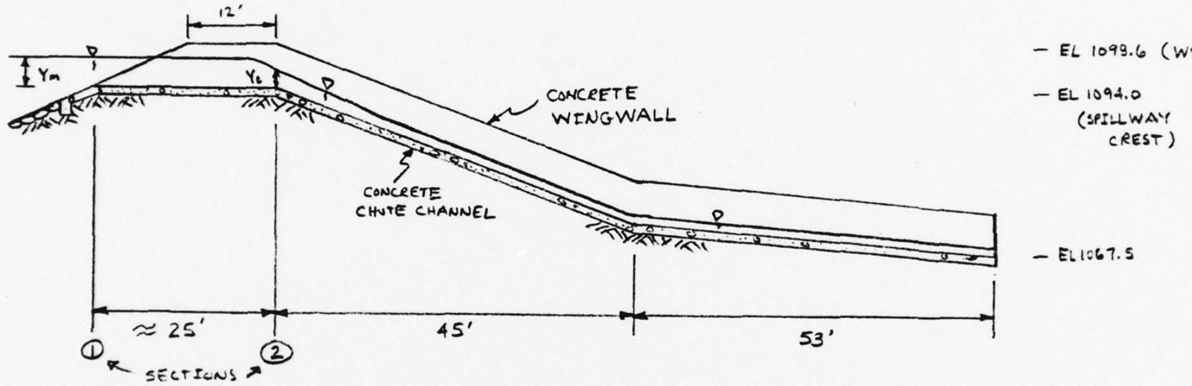
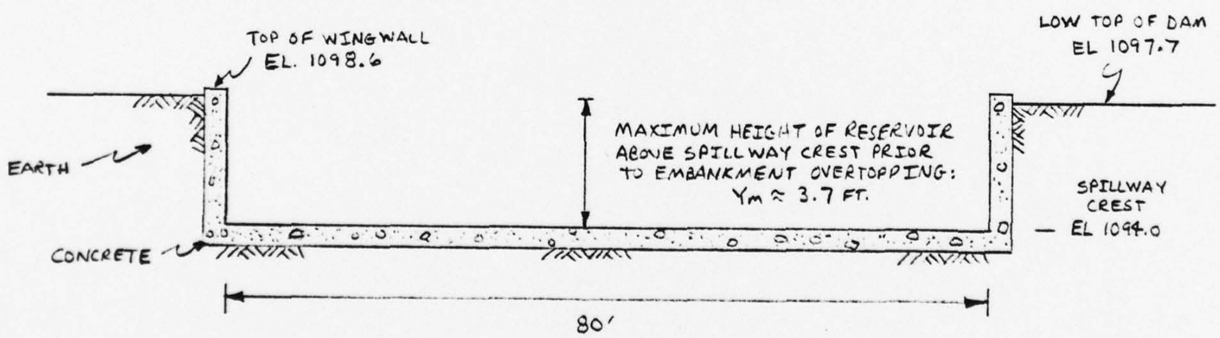
SPILLWAY CAPACITY

- SPILLWAY PROFILE: (NOT TO SCALE)



- SPILLWAY CREST SECTION: (NOT TO SCALE)



- ASSUMING THAT THE WATER SURFACE PROFILE PASSES THROUGH CRITICAL DEPTH @ SECTION ②: ENERGY BALANCE BETWEEN ① AND ② ⇒

$$Y_m + \cancel{v_1^2/2g}^{\,0} + z_1 = Y_c + v_c^2/2g + z_2 + \cancel{H_L}^{\,0} \quad \text{(REF 7, PG. 4}$$

WHERE v_1 = RESERVOIR VELOCITY ≈ 0 FPS.,

z_1 = ELEVATION @ ① IN FT.,

v_c = CRITICAL VELOCITY @ ② IN FPS.,

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES Nº 1 DAM

BY WJV DATE 4-3-79 PROJ. NO. 78-617-497
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Z_2 = ELEVATION @ ② IN FT., AND
H_L = HEAD LOSS BETWEEN ① AND ② ≈ 0

- SINCE $Z_1 - Z_2 \approx 0$ (SECTIONS ① AND ② ARE CLOSE TOGETHER)

$$Y_m = Y_c + v_c^2/2g \qquad \text{w/ } Y_m = 3.7\text{ FT}$$

- FOR CRITICAL DEPTH IN A RECTANGULAR SECTION:

$$v_c^2/2g = Y_c/2 \qquad \text{(REF 7, PG. 55)}$$

$$\therefore \quad Y_m = 3.7\text{ FT} = Y_c + Y_c/2 = 3/2\, Y_c$$

$$Y_c \approx 2.47\text{ FT}$$

- CRITICAL AREA = $A_c \approx (80\text{FT})(Y_c) = (80\text{FT})(2.47\text{FT}) \approx 197.6\text{ FT}^2$

- CRITICAL VELOCITY ⇒ $v_c = \sqrt{g Y_c}$ (FROM ABOVE)

$$v_c = \sqrt{g\,(2.47\text{FT})}$$

$$v_c \approx 8.92\text{ FPS}$$

∴ SPILLWAY CAPACITY = $Q = A_c v_c = (197.6\text{ FT}^2)(8.92\text{ FPS})$

$$Q \approx 1760\text{ CFS}$$

NOTE 6: IF DAM CREST WAS LEVEL @ DESIGN ELEVATION 1098.0 FT
⇒ $Y_m = 4 = 3/2\, Y_c$ ⇒ $Y_c \approx 2.67$ FT ; $v_c^2/2g \approx 1.33$ FT ⇒
$v_c \approx 9.25$ FPS ; $Q = A_c v_c \approx [80\,(2.67)]\,[9.25] \approx 1980$ CFS

SUBJECT DAM SAFETY INSPECTION
TWIN LAKES No 1 DAM
BY WJV DATE 4-3-79 PROJ. NO. 78-617-487
CHKD. BY DLB DATE 4-13-79 SHEET NO. 8 OF 8

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SPILLWAY RATING CURVE

COMPUTED INTERNALLY BY HEC-1 VIA THE TRAPEZOIDAL RATING CURVE ROUTINE, BASED ON THE SPILLWAY GEOMETRY AS PRESENTED ON SHEET 6. THE TRAPEZOIDAL ROUTINE CALCULATES CRITICAL CONTROL DISCHARGES IN A WAY SIMILAR TO THAT OUTLINED ON SHEETS 6 AND 7. (SEE SUMMARY INPUT / OUTPUT SHEETS).

DAM EMBANKMENT RATING CURVE

- COMPUTED INTERNALLY BY HEC-1 VIA THE ASSUMPTION THAT CRITICAL DEPTH OCCURS ON THE CREST (WHEN OVERTOPPED), W/ THE CREST PROFILE REPRESENTED BY A SERIES OF TRAPEZOIDS. (SEE SUMMARY INPUT/OUTPUT SHEETS FOR RATING INFORMATION).

- INPUT INFORMATION: (BASED ON FIELD MEASUREMENTS)

	RESERVOIR ELEVATION (FT)	DEPTH OF WATER ABOVE CREST (FT)	LENGTH OF CREST INUNDATED (FT)
TOP OF DAM -	1097.7	0	300
	1097.8	0.1	430
	1098.0	0.3	520
	1098.1	0.4	740
	1098.5	0.8	800
	1099.0	1.3	830
	1099.5	1.8	860
	1100.0	2.3	890

APPENDIX D

PHOTOGRAPHS

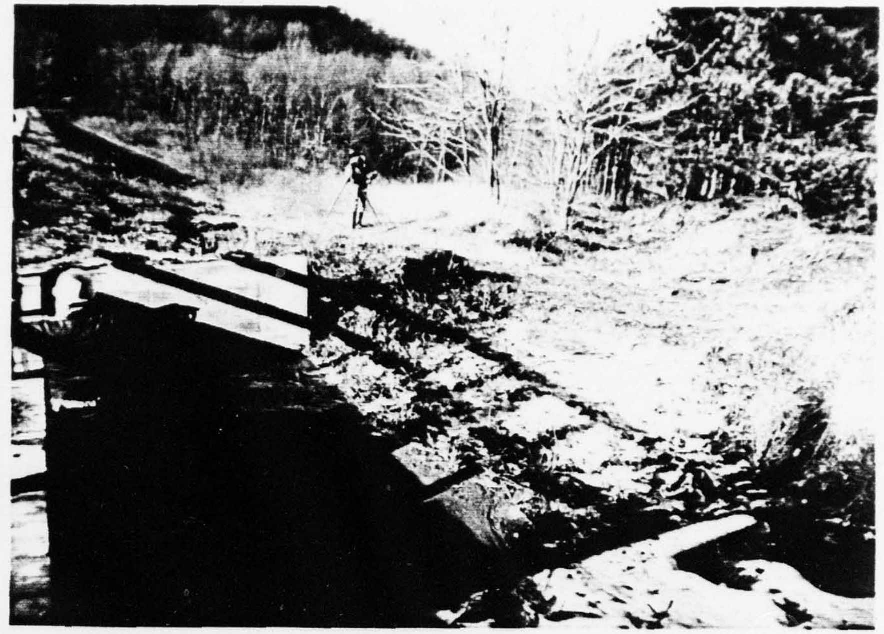

1

2

3

4


5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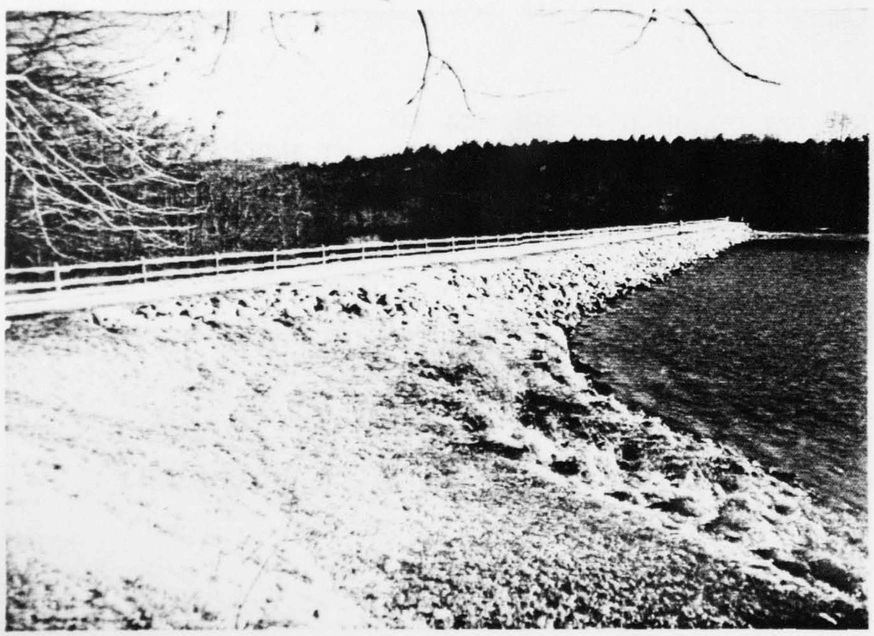
7


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14

APPENDIX E

GEOLOGY

Geology

The Upper Donohoe Dam is located in the Pittsburgh Plateau Section of the Appalachian Plateaus Physiographic Province. The Pittsburgh Plateau Section is characterized by flat-lying to gently folded sedimentary rock strata of Pennsylvanian age. Major structural axes trend from southwest to northeast with flanking strata dipping northwest and southeast. The amplitude of folding in this section is quite low, consequently, surface expression of the major structures is not evident.

Structurally, the Upper Donohoe Dam is located midway between the Greensburg syncline to the northwest and the Fayette anticline to the southeast. The axial trace of these two structures are nearly parallel and trend N45° to 50°E. Each of these structures is doubly plunging adjacent to the site. The Greensburg syncline forms an elongated structural basin west of the site, whereas the Fayette anticline forms an elongated dome on its axis to the east. In the immediate vicinity of the dam, bedrock generally strikes N40° to 45°E and dips to the northwest at approximately 350 feet per mile or about 3 to 4 degrees.

The dam and reservoir are located on sedimentary rock strata of the Conemaugh Group of Pennsylvanian age. Based on published data, the bedrock underlying the dam consists of those members of the Conemaugh Group which generally lie approximately 320 to 350 feet below the base of the Pittsburgh

Coal. The generalized stratigraphic column for this area indicates the Saltsburg sandstone is to be expected in this interval.

In 1978, a subsurface investigation of the existing embankment and underlying foundation was conducted to evaluate the structure and develop rehabilitation design parameters. A total of six test borings were drilled on and downstream of the existing embankment. All of the borings sampled at least 25 feet of bedrock. A medium to moderately massive sandstone was encountered in all the test borings. This sandstone was incorrectly identified as the Morgantown sandstone which occurs well above elevation 1200 in the immediate vicinity of the site. The sandstone underlying the embankment is most likely the Saltsburg sandstone. The subsequent analysis performed by the consultant, however, remains the same since the prominent sandstone members of the Conemaugh Formation share common characteristics. The following excerpt is taken from the consultant's report.

> "The foundation (natural) soils on which the embankment fill has been placed consist of a relatively thin (average thickness 6 feet) layer of residual soils. These residual soils have been formed by the in-place weathering of the underlying bedrock and reflect both their lithology and fabric. The foundation soils vary in consistency from stiff to very stiff which indicates low consolidation, moderate shear strength properties and in-situ low permeabilities.
>
> The surface of bedrock slopes at approximately the same rate as the natural ground surface. Lithologically, the rock sampled in the test borings consists of fine to medium grained sandstones separated by 5 to 7 feet thick layers of claystone and clayey shales.

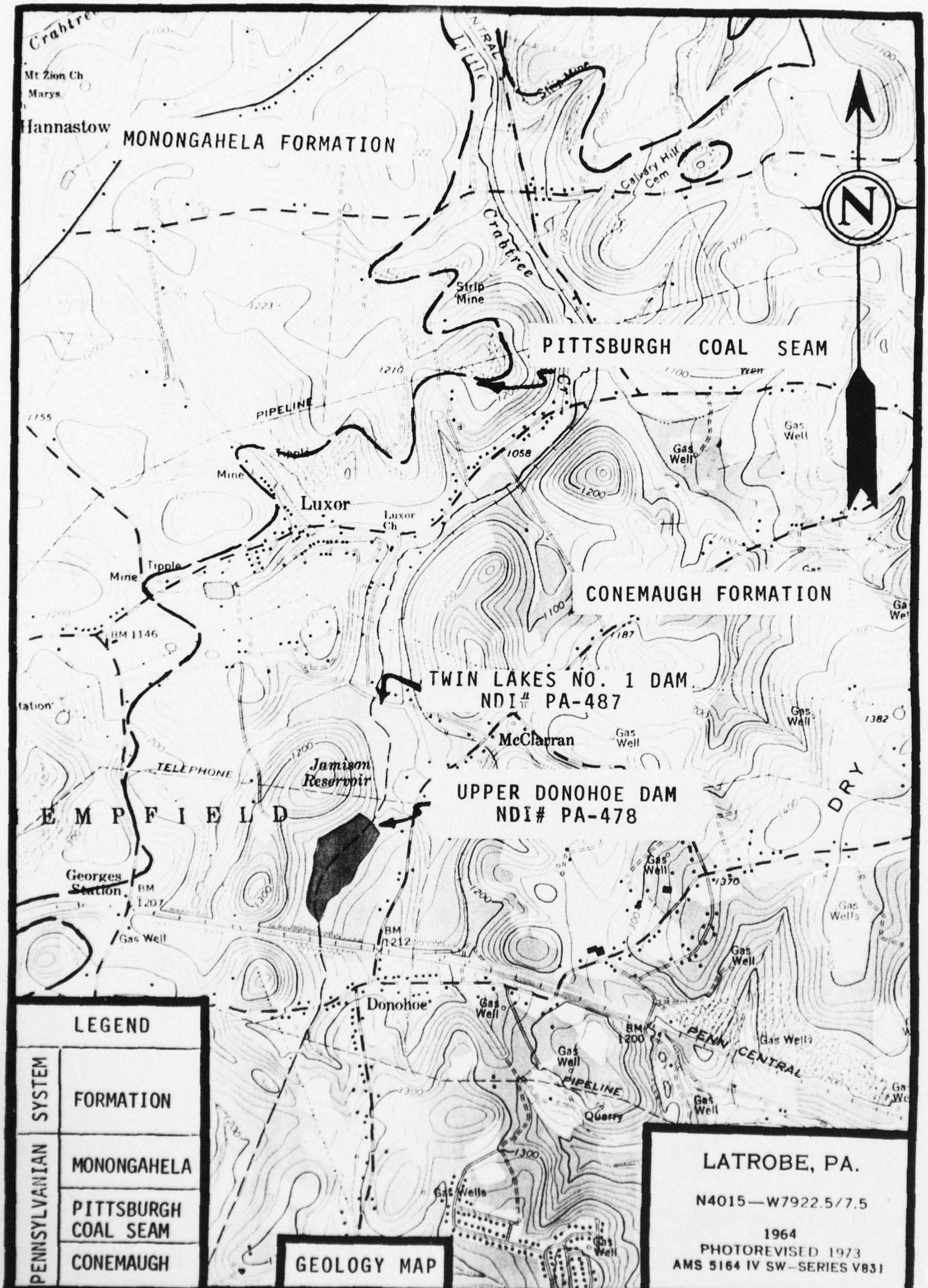
The top 5 to 10 feet of bedrock strata exhibited extensive leaching and water staining, reflecting the long and continuous movement of water."

---

[1]"Subsurface Exploration and Geotechnical Engineering Investigation, Upper Dam and Twin Lakes Park, Westmoreland County, Pennsylvania," prepared by Geo-Mechanics, Inc., for the Westmoreland County, Department of Parks and Recreation, Greensburg, Pennsylvania, September 1978.

[2]"Geologic Atlas of the United States, Latrobe Folio, Pennsylvania," U. S. Geological Survey, No. 110, 1904.

[3]"Mineral Resources of the Greensburg Quadrangle, Westmoreland County, Pennsylvania," M. E. Johnson, Topographic and Geologic Survey, Atlas 37, Harrisburg, Pennsylvania, 1925.



MONONGAHELA FORMATION
PITTSBURGH COAL SEAM
CONEMAUGH FORMATION
TWIN LAKES NO. 1 DAM
NDI# PA-487
UPPER DONOHOE DAM
NDI# PA-478
Hannastow
Mt Zion Ch
Marys
Crabtree
Strip Mine
Calvary Hill Cem
PIPELINE
Luxor
Luxor Ch
Mine
Tipple
BM 1146
McClarran
Jamison Reservoir
TELEPHONE
EMPFIELD
Georges Station
BM 1207
Gas Well
BM 1212
Donohoe
BM 1200
PENN CENTRAL
Quarry
Gas Wells
DRY
LEGEND
SYSTEM
FORMATION
PENNSYLVANIAN
MONONGAHELA
PITTSBURGH COAL SEAM
CONEMAUGH
GEOLOGY MAP
LATROBE, PA.
N4015—W7922.5/7.5
1964
PHOTOREVISED 1973
AMS 5164 IV SW—SERIES V831

APPENDIX F

FIGURES

LIST OF FIGURES

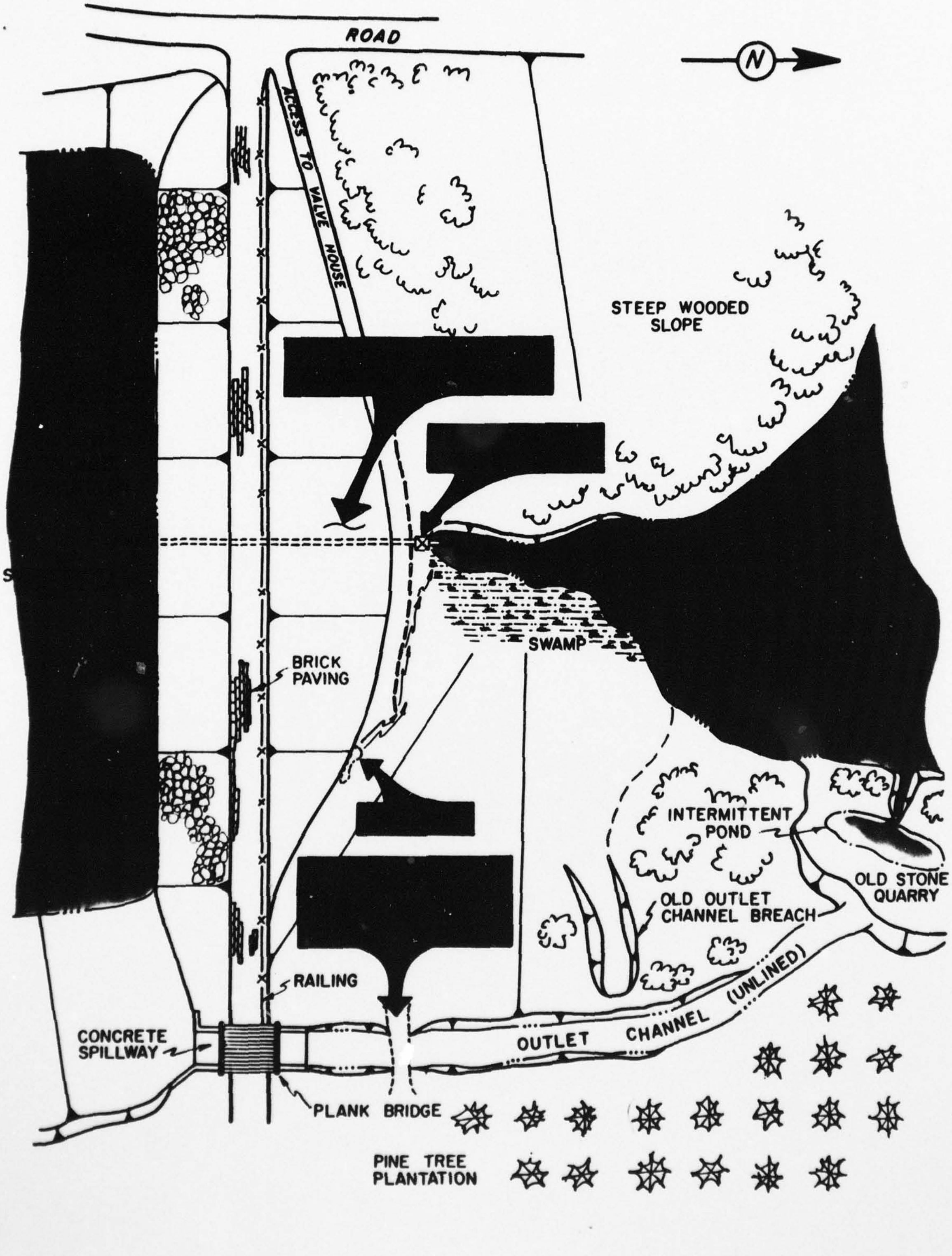
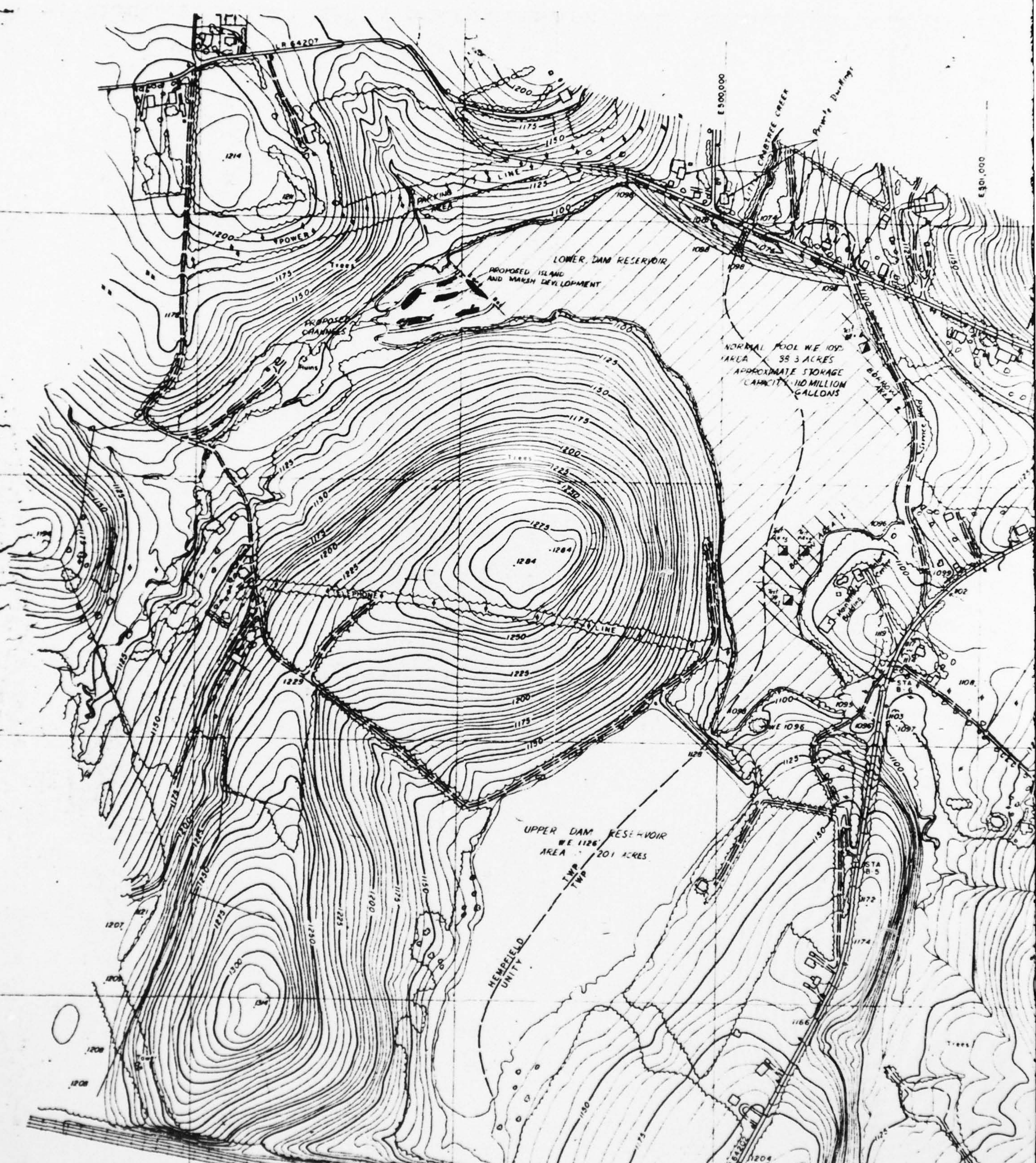
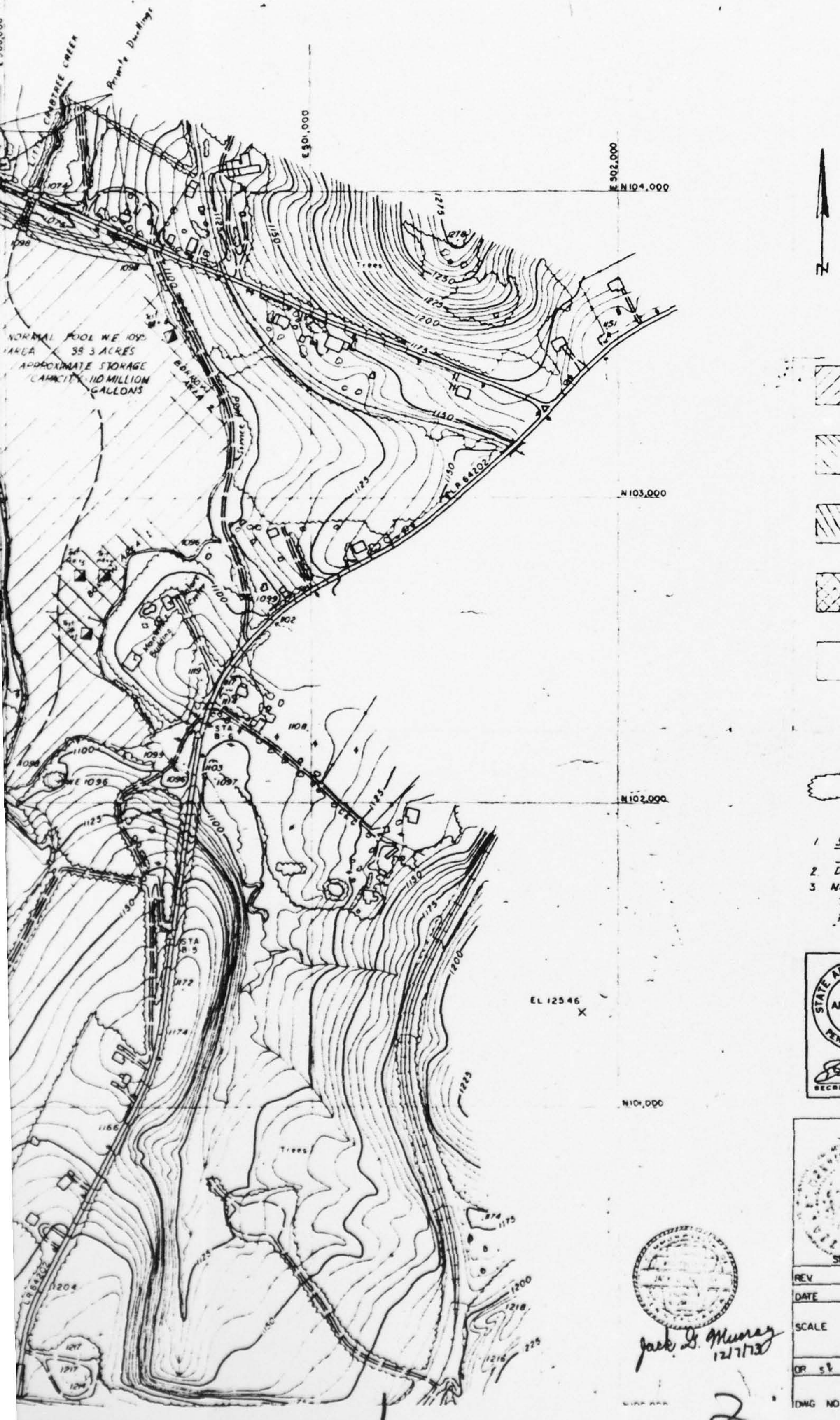


FIGURE 1 - UPPER DONOHUE DAM
GENERAL PLAN
FIELD INSPECTION NOTES



L.R. 64207
CRABTREE CREEK
Private Dwellings
E 300,000
E 301,000
POWER LINE
PARKING AREA
LOWER DAM RESERVOIR
PROPOSED ISLAND AND MARSH DEVELOPMENT
PROPOSED CHANNELS
Ruins
NORMAL POOL W.E. 1098
AREA 38.3 ACRES
APPROXIMATE STORAGE CAPACITY 110 MILLION GALLONS
Trees
TELEPHONE LINE
UPPER DAM RESERVOIR
W.E. 1126'
AREA 201 ACRES
TWP
TWP
HEMPFIELD
UNITY
PENNSYLVANIA
RR

1



LEGEND

PROPOSED AND EXISTING LOWER DAM RESEVIOR AREA

EMBANKMENT WORK AREA (TO BE SEEDED)

SPILLWAY (TO BE REBUILT)

APPROXIMATE BORROW AREAS

WILDLIFE REFUGE AREA

TREES

NOTES

1. SITE PLAN ELEVATIONS = CONSTRUCTION PLAN ELEVATIONS PLUS 3 FEET
2. DEVELOPMENT AREA 35.1 ACRES.
3. NEW LOWER DAM RESERVOIR IS ESSENTIALLY THE SAME AREA AS THE OLD LOWER DAM RESERVOIR WHICH WAS DRAINED

STATE ART COMMISSION APPROVED PENNSYLVANIA

19,30?
SUBMISSION NUMBER

TWIN LAKES LOWER DAM RECONSTRUCTION
TITLE OF SUBMISSION

GREENSBURG
WESTMORELAND
LOCATION

9/3/74
DATE

SECRETARY, STATE ART COMMISSION

WESTMORELAND COUNTY COMMISSIONERS APPROVAL	SIGNATURE	DATE
DOROTHY K. SHOPE	Dorothy K. Shope	9-9-74
ROBERT G. SHIREY	Robert G. Shirey	9/9/74
TED SIM		9-9-74

FIGURE 2

SEAL

REV	Aug. 1974
DATE	Nov 1973
SCALE	1" = 200'
DR SR	CK JM
DWG NO	7342-A

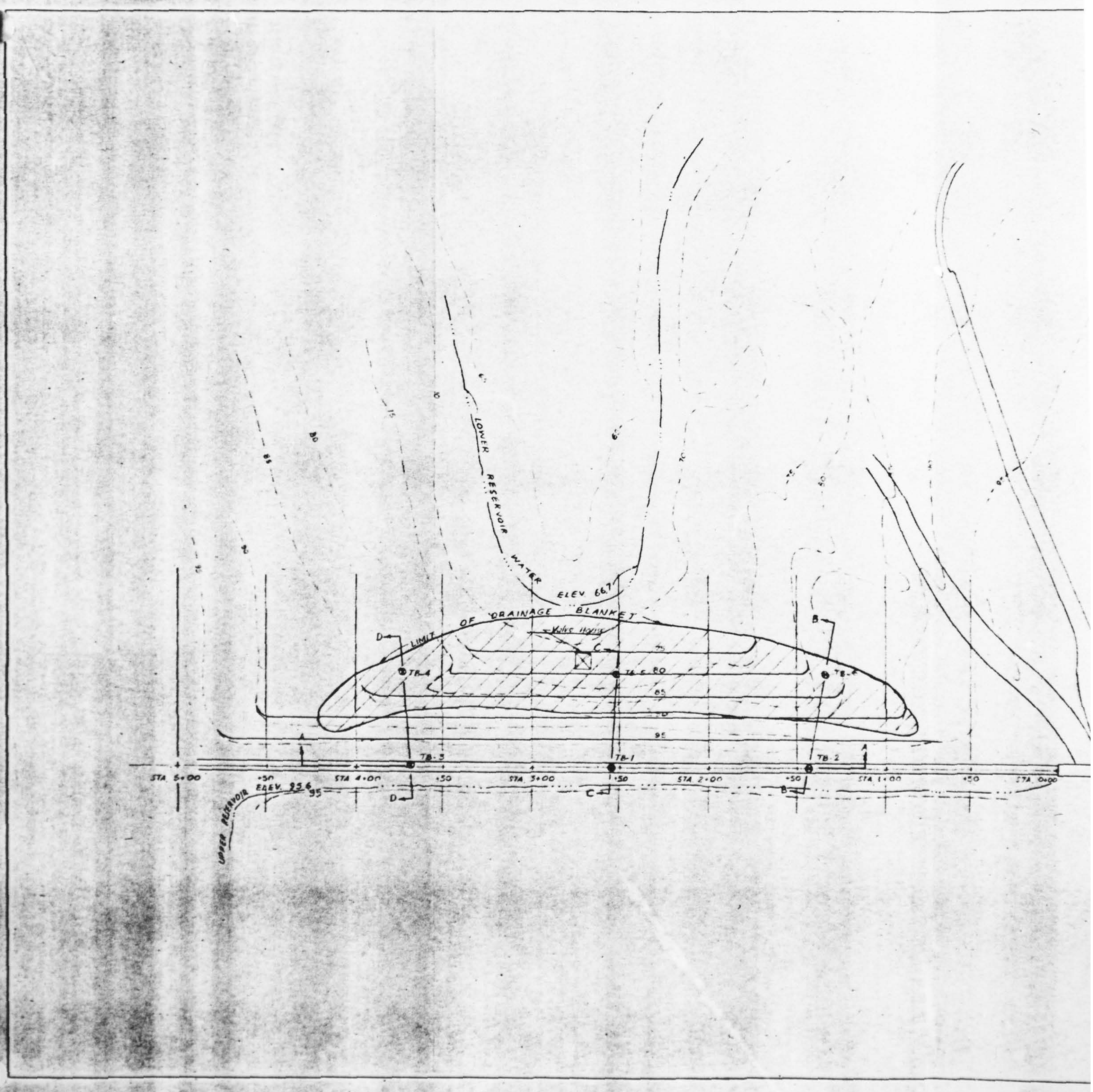
LOWER DAM RECONSTRUCTION
TWIN LAKES PARK
WESTMORELAND COUNTY DEPT OF PARKS & RECREATION

SITE PLAN

GEO-MECHANICS, INC., MONESSEN, PA.

Jack G. Murray
12/7/73

2


LOWER RESERVOIR WATER ELEV. 66.7
LIMIT OF DRAINAGE BLANKET
UPPER RESERVOIR ELEV. 95.6
STA. 5+00
STA. 4+00
STA. 3+00
STA. 2+00
STA. 1+00
STA. 0+00
TB-1
TB-2
TB-3
TB-4
A
B
C
D

1

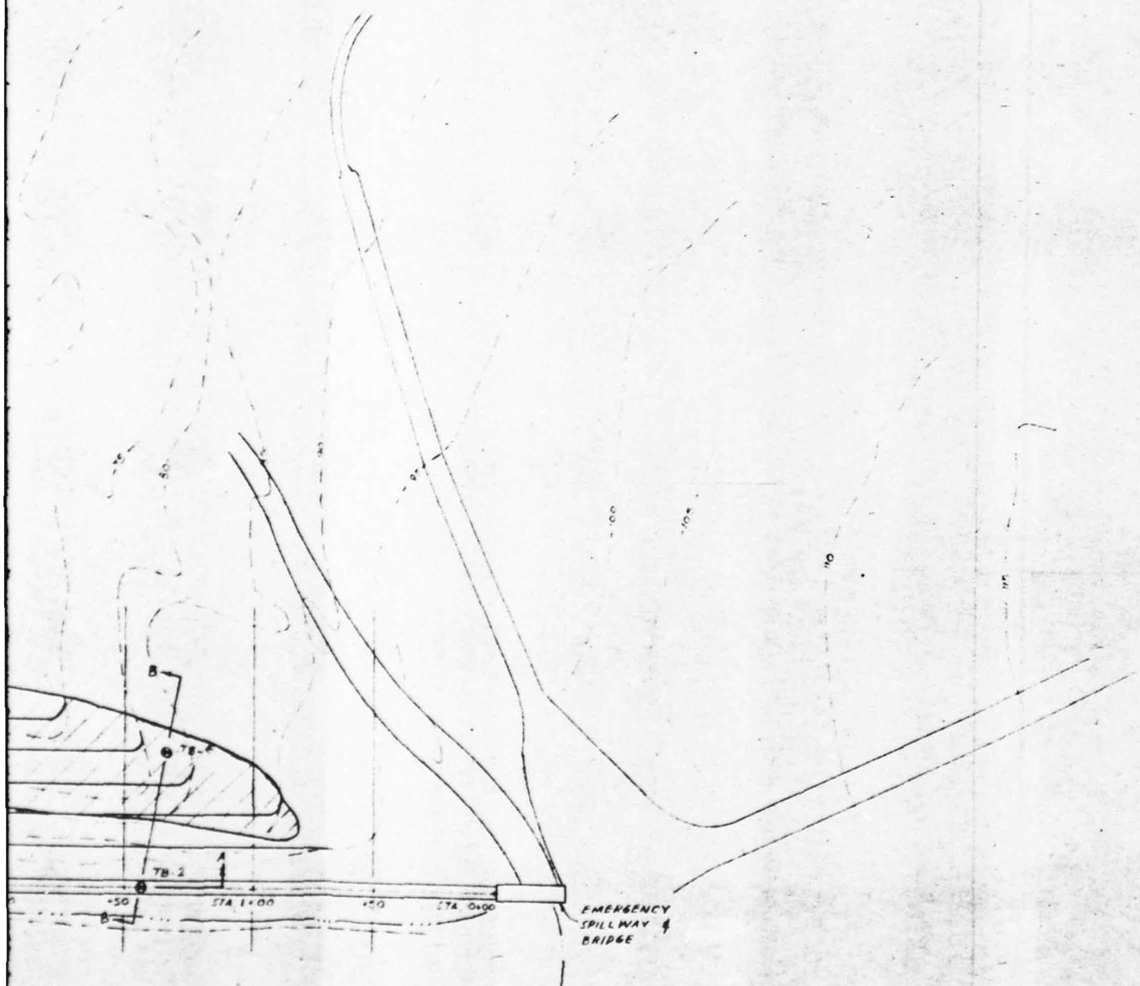


FIGURE 3

UPPER DAM AT TWIN LAKES PARK

WESTMORELAND COUNTY, PA.

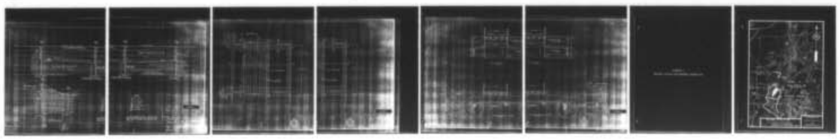
TEST BORING LOCATION PLAN AND REGRADING PLAN

Revision No	Date	Scale	Drawn By	Checked By	Job No.	Sheet No.
	Sept., 1978	1"=30'	S.K	J.A	7856	1 of 6

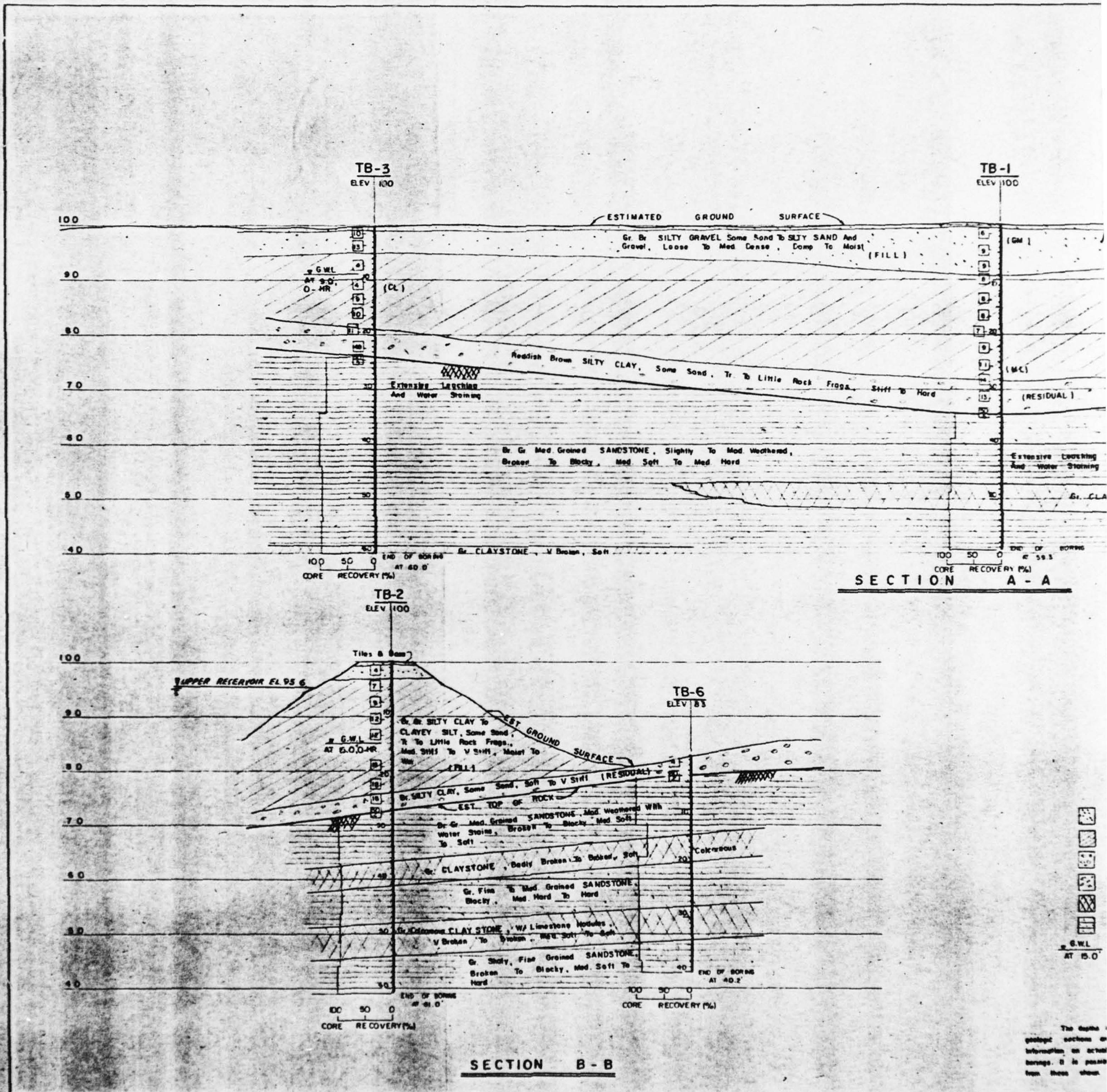
GEO - MECHANICS, INC.
GEOTECHNICAL CONSULTANTS BELLE VERNON, PENNSYLVANIA

AD-A070 584 GAI CONSULTANTS INC MONROEVILLE PA F/G 13/2
NATIONAL DAM INSPECTION PROGRAM. UPPER DONOHOE DAM (NOW REFERRE--ETC(U)
APR 79 DACW31-79-C-0013
UNCLASSIFIED NL

2 OF 2
AD
A070584

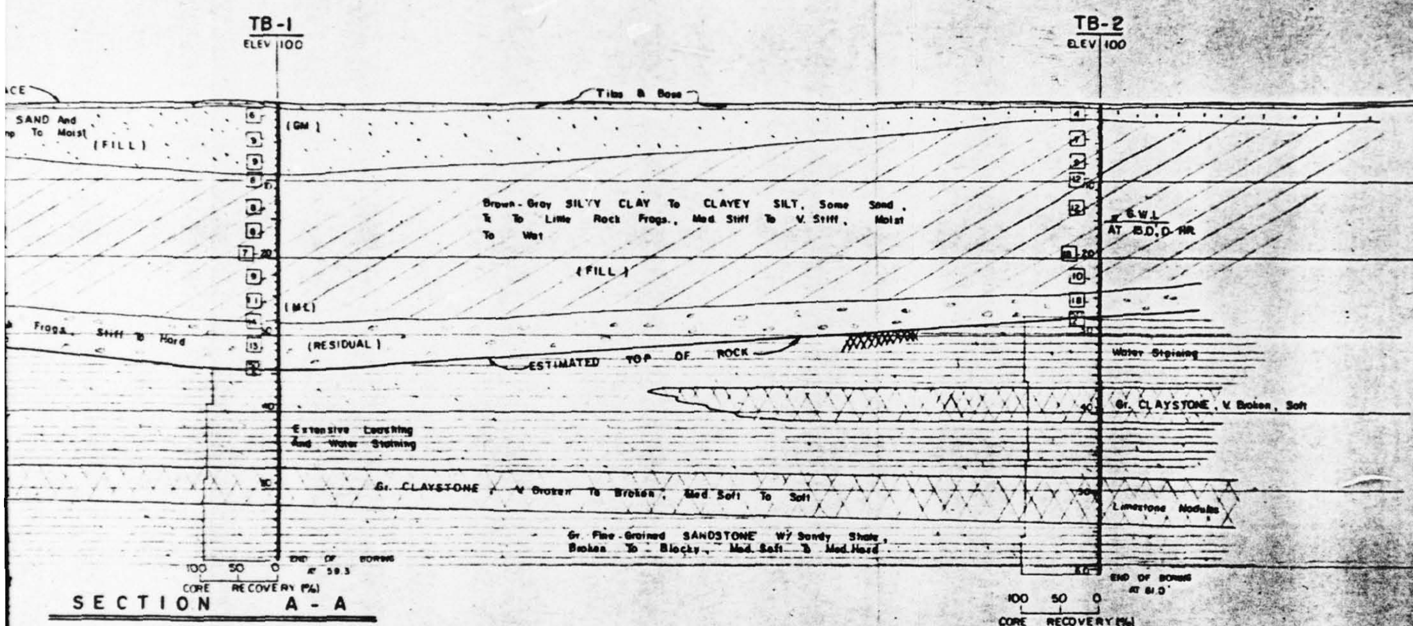


END
DATE
FILMED
8-79
DDC



TB-3
TB-1
TB-2
TB-6
ESTIMATED GROUND SURFACE
Gr. Br SILTY GRAVEL Some Sand To SILTY SAND And Gravel, Loose To Med. Dense, Damp To Moist (FILL)
Reddish Brown SILTY CLAY, Some Sand, Tr. To Little Rock Frags., Stiff To Hard
(RESIDUAL)
Extensive Leaching And Water Staining
Br. Gr. Med. Grained SANDSTONE, Slightly To Mod. Weathered, Broken To Blocky, Med. Soft To Med. Hard
Gr. CLAYSTONE, V Broken, Soft
END OF BORING AT 60.0'
CORE RECOVERY (%)
SECTION A - A
UPPER RESERVOIR EL 95.6
Tiles & Base
EST. GROUND SURFACE
EST. TOP OF ROCK
Gr. CLAYSTONE, Badly Broken To Bedded, Soft
Gr. Fine To Med. Grained SANDSTONE, Blocky, Med. Hard To Hard
Calcareous
END OF BORING AT 40.2'
END OF BORING AT 61.0'
SECTION B - B

1



SECTION A - A

LEGEND

- SAND (FILL)
- CLAY / SILT (FILL)
- CLAY (RESIDUAL)
- ROCK FRAGMENTS
- CLAYSTONE / CLAYSHALE
- SANDSTONE
- G.W.L. AT 15.0' GROUND WATER LEVEL

The depths and thicknesses of the soil and rock strata indicated on these geologic sections are generalized from and interpolated between the test borings. Information on actual subsurface conditions exist only at the location of the test borings. It is possible that surface conditions between the test borings may vary from those shown.

FIGURE 4

UPPER DAM AT TWIN LAKES PARK

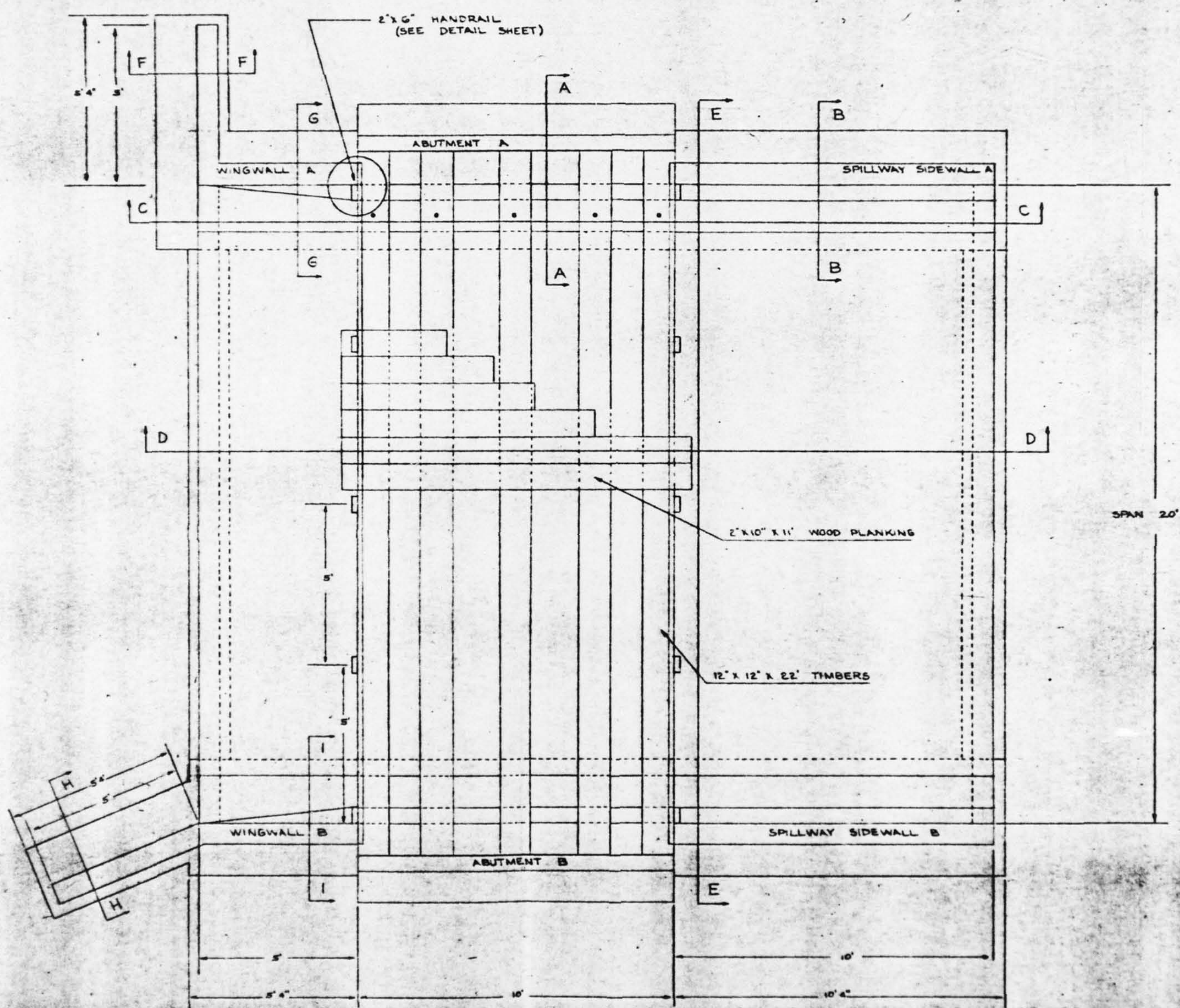
WESTMORELAND COUNTY, PA.

GEOLOGIC CROSS SECTIONS

Revision No.	Date	Scale	Drawn By	Checked By	Job No.	Sheet No.
	Sept., 1978	Ver. 1" = 10' Hor. 1" = 10'	S:K	J.A	7856	2 of 2

GEO - MECHANICS, INC.
GEOTECHNICAL CONSULTANTS BELLE VERNON, PENNSYLVANIA

2



WESTMORELAND COUNTY DI			
SEAL	REVISION NO.	DATE	BY

1

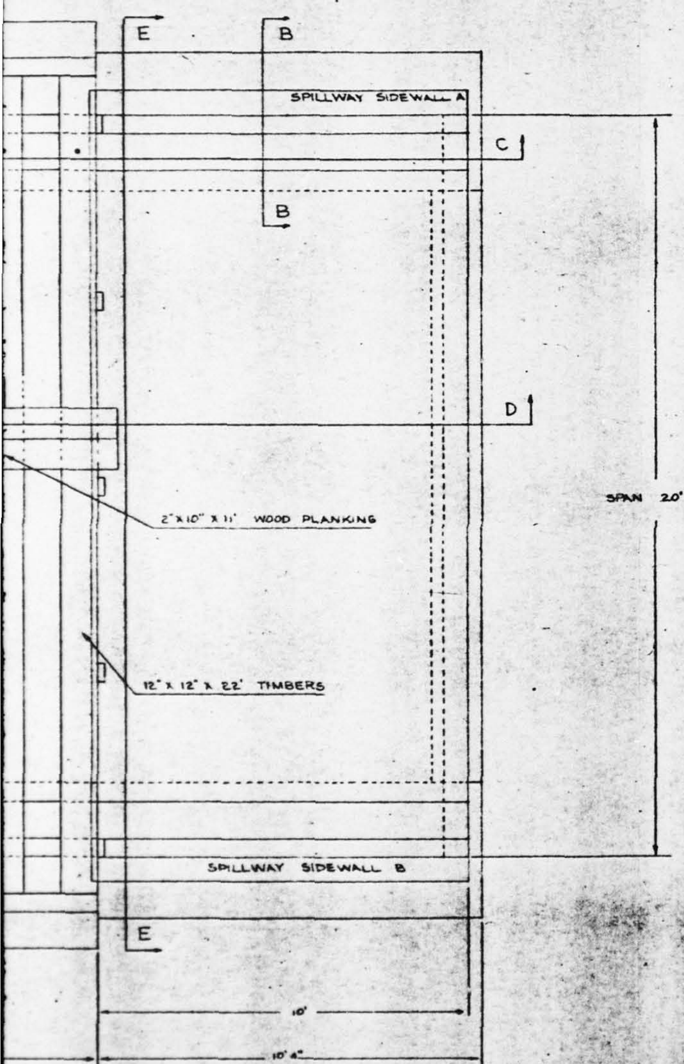
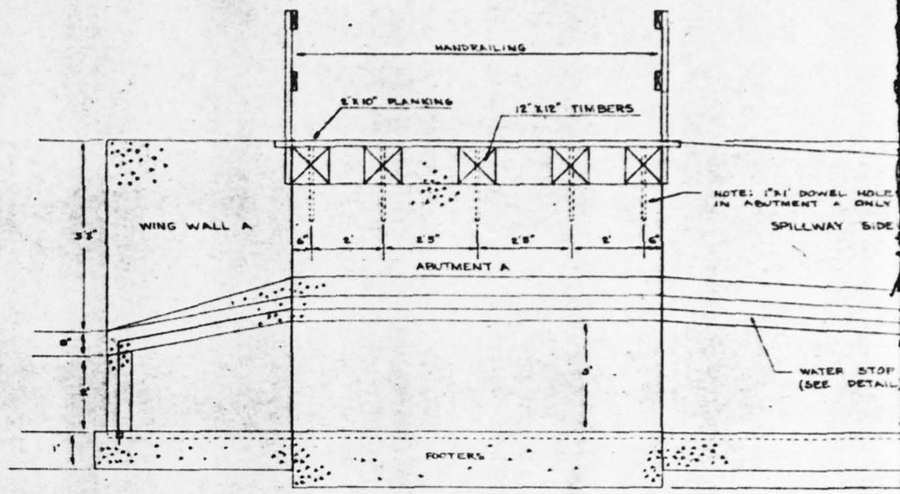


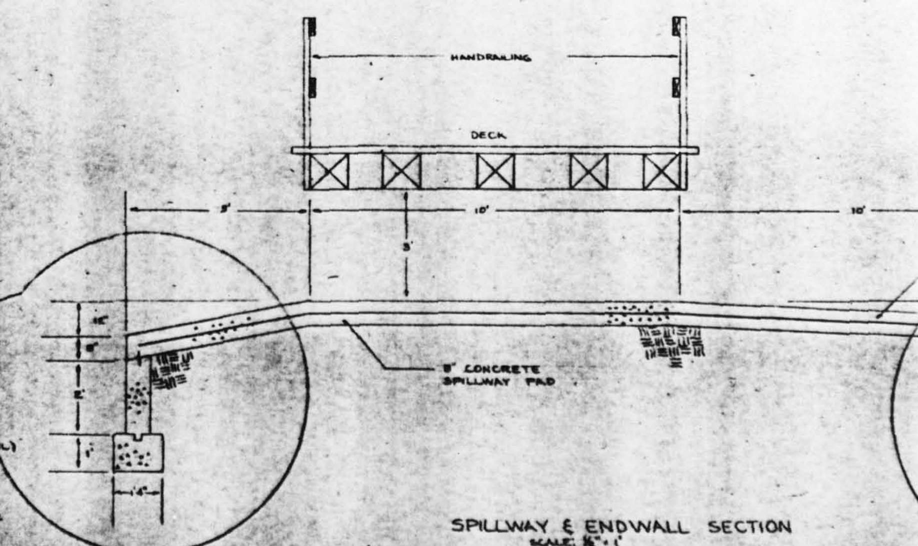
FIGURE 5

WESTMORELAND COUNTY DEPARTMENT OF ENGINEERING

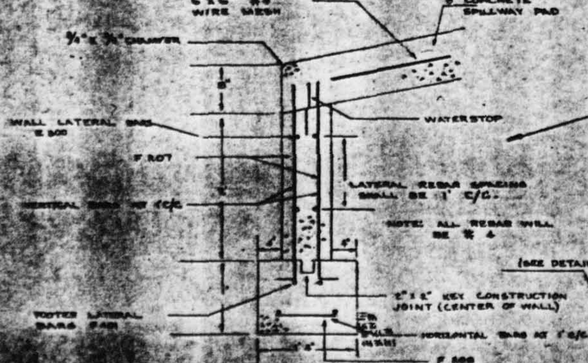
SEAL	REVISION NO.	DATE	BY	
ROBERT J. PIRONDER 24726-E				TWIN LAKES PARK UPPER LAKE PLAN
				drawn by D.R.P.



WALL SECTION CC
SCALE: 1/2"=1'



SPILLWAY & ENDWALL SECTION
SCALE: ½"=1'



END WALL A

1

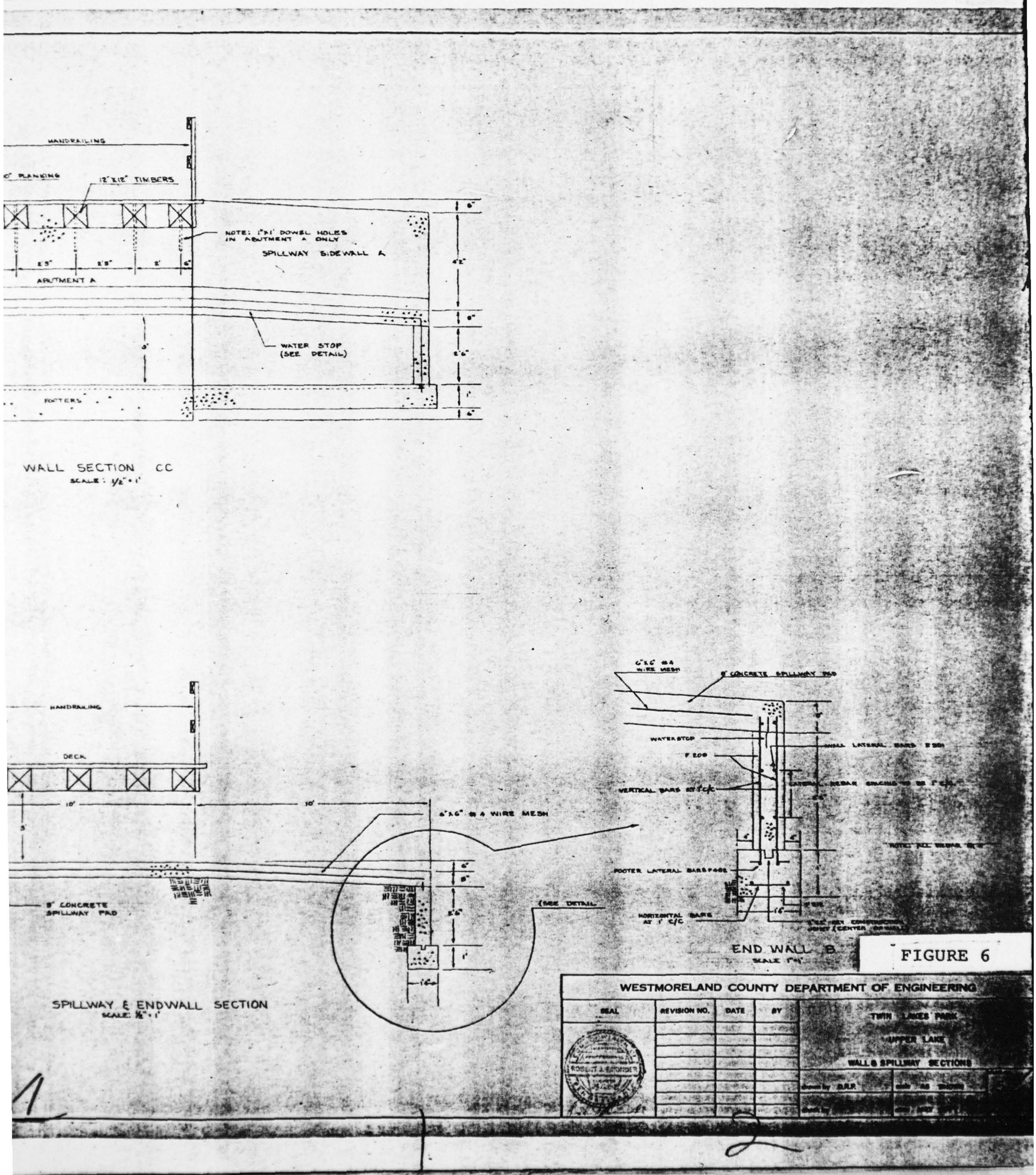
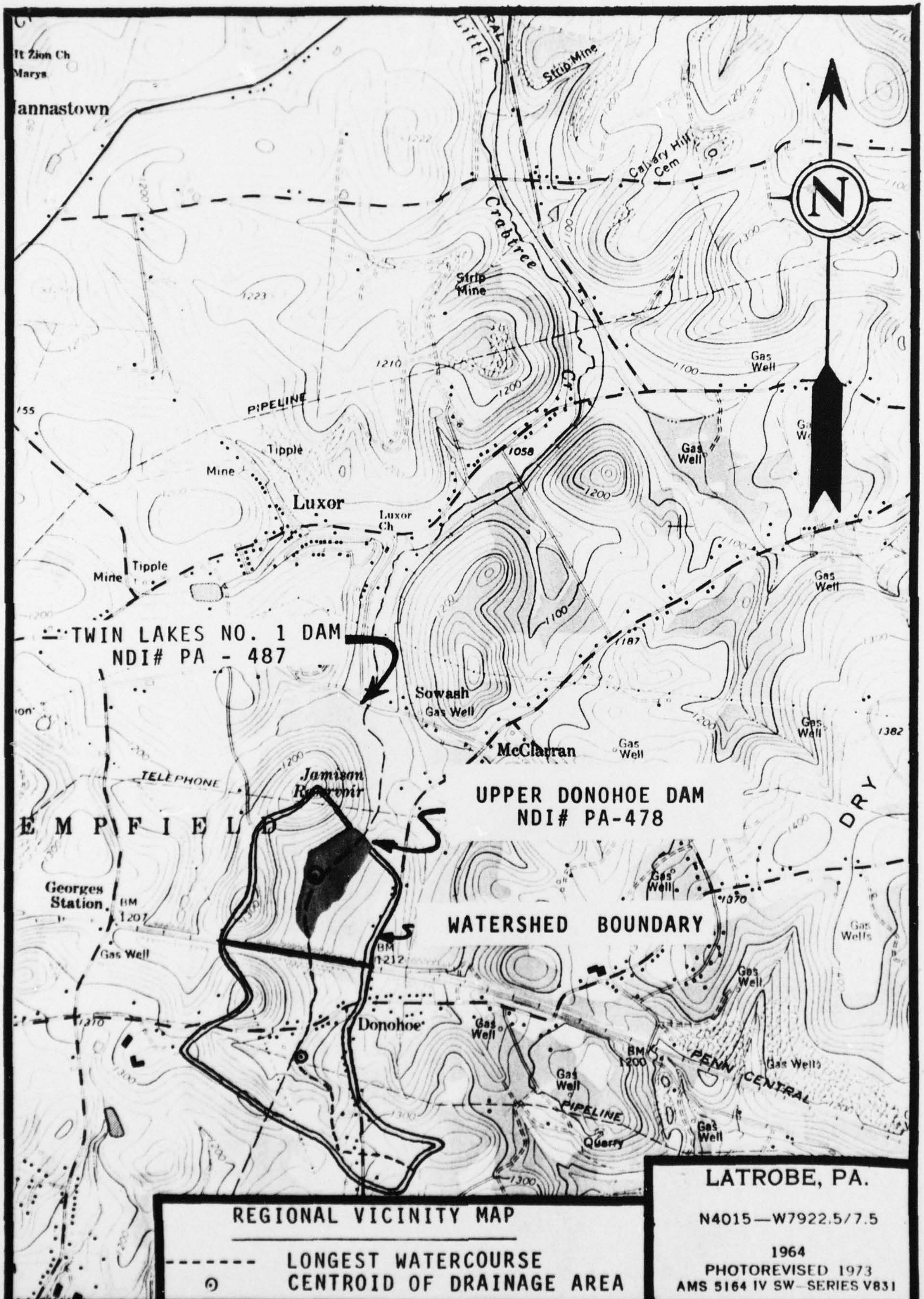

HANDRAILING
PLANKING
12"X12" TIMBERS
NOTE: 1"x1' DOWEL HOLES IN ABUTMENT A ONLY
SPILLWAY SIDEWALL A
ABUTMENT A
WATER STOP (SEE DETAIL)
FOOTERS
WALL SECTION CC
SCALE: 1/2"=1'
HANDRAILING
DECK
6"X6" #4 WIRE MESH
8" CONCRETE SPILLWAY PAD
(SEE DETAIL
SPILLWAY & ENDWALL SECTION
SCALE: 1/2"=1'
8" CONCRETE SPILLWAY PAD
WATER STOP
F 200
VERTICAL BARS AT 1' C/C
FOOTER LATERAL BARS
HORIZONTAL BARS AT 1' C/C
END WALL B
WESTMORELAND COUNTY DEPARTMENT OF ENGINEERING
SEAL
REVISION NO.
DATE
BY
TWIN LAKES PARK
UPPER LAKE
WALL & SPILLWAY SECTIONS

FIGURE 6

APPENDIX G

REGIONAL VICINITY AND WATERSHED BOUNDARY MAP


TWIN LAKES NO. 1 DAM
NDI# PA - 487
UPPER DONOHOE DAM
NDI# PA-478
WATERSHED BOUNDARY
Jannastown
Luxor
Luxor Ch
Tipple
Mine
Strip Mine
Crabtree
Cr
Little
Calvary Hill Cem
PIPELINE
TELEPHONE
Sowash
Gas Well
McClarran
Jamison Reservoir
EMPFIELD
Georges Station
BM 1207
BM 1212
BM 1200
Donohoe
PENN CENTRAL
Quarry
DRY
1058
1187
1382
1070
N
REGIONAL VICINITY MAP
LONGEST WATERCOURSE
CENTROID OF DRAINAGE AREA
LATROBE, PA.
N4015—W7922.5/7.5
1964
PHOTOREVISED 1973
AMS 5164 IV SW—SERIES V831