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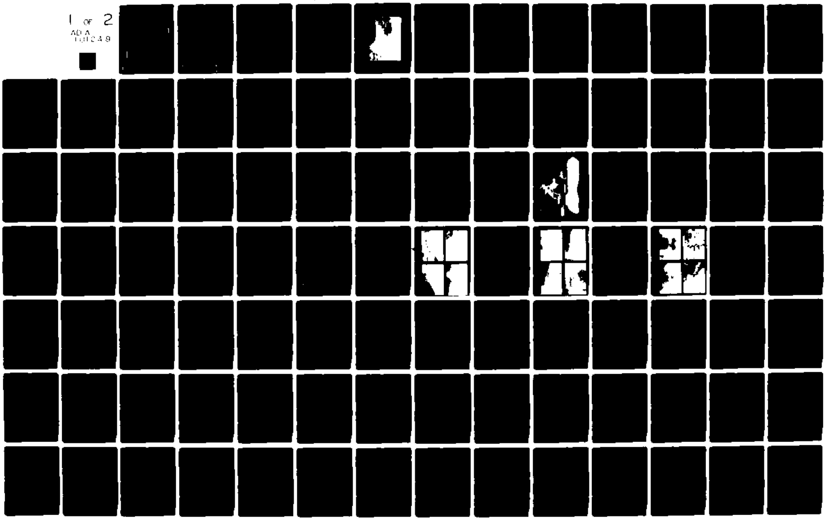

AD A101249

DELAWARE RIVER BASIN
BRANCH OF HORNBECKS CREEK, PIKE COUNTY

PENNSYLVANIA

RICKARDS DAM

LEVEL II

NDI I.D. NO. PA-00405
PENNDER I.D. NO. 52-82

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MRS. URBAN RICKARD

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

National Dam Inspection Program. Rickards Dam (NDI I.D. Number PA-00405, PennDER I.D. Number 52-82), Delaware River Basin, Branch of Hornbecks Creek. Pike County, Pennsylvania. Phase I Inspection Report,

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PREPARED FOR

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

DACW31-81-C-0015

PREPARED BY

GAI CONSULTANTS, INC.
570 BEATTY ROAD
MONROEVILLE, PENNSYLVANIA 15146

JUNE 1981

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PREFACE

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

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team.

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 (greatest reasonably possible storm runoff) for the region, 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.

Breach analyses are performed, when necessary, to provide data to assess the potential for downstream damage and possible loss of life. The results are based on specific theoretical scenarios peculiar to the analysis of a particular dam and are not applicable to other related studies such as those conducted under the Federal Flood Insurance Program.

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

ABSTRACT

Rickards Dam: NDI I.D. No. PA-00405

Owner:	Mrs. Urban F. Rickard
State Located:	Pennsylvania (PennDER I.D. No. 52-82)
County Located:	Pike
Stream:	Branch of Hornbecks Creek
Inspection Date:	17 October 1980
Inspection Team:	GAI Consultants, Inc. 570 Beatty Road Monroeville, Pennsylvania 15146

Based on a visual inspection, construction history, and available engineering data, the dam is considered to be in poor condition.

The size classification of the facility is small and its hazard classification is considered to be high. In accordance with the recommended guidelines, the Spillway Design Flood (SDF) for the facility ranges between the 1/2 PMF (Probable Maximum Flood) and the PMF. Since the facility is classified near the lower bounds of the small category, the SDF is considered to be the 1/2 PMF. Results of the hydrologic and hyraulic analysis indicate the facility will pass and/or store only about 29 percent of the PMF prior to embankment overtopping at the low area in the main embankment crest. Breach analysis indicates that failure under less than 1/2 PMF conditions could lead to increased downstream damage and potential for loss of life. Thus, based on screening criteria provided in the recommended guidelines, the spillway is considered to be seriously inadequate and the facility unsafe, non-emergency.

Calculations also indicate that if the embankment and dike crests were uniformly regraded to the elevation of the top of the spillway sidewalls at 1080.5 feet, the facility then could pass and/or store approximately 57 percent of the PMF and the spillway would be considered adequate.

Structural deficiencies observed by the inspection team included excessive settlement of both the main embankment and dike structures, general deterioration of the spillway and outlet works, and a general lack of routine maintenance. Historical correspondences also strongly questions the construction quality of the facility.

Rickards Dam: NDI I.D. No. PA-00405

It is recommended that the owner immediately:

a. Develop a formal warning system to notify downstream inhabitants should hazardous embankment conditions develop. The system should include provisions for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Have the main embankment and appurtenant dike evaluated by a registered professional engineer experienced in the design and construction of earth dams to assess their overall structural integrity and make remedial recommendations as required. At a minimum, the embankment and dike crests should be uniformly regraded to the top of the spillway sidewalls at elevation 1080.5 feet to make the facility hydraulically adequate.

c. Clear all excess vegetation from the slopes and crests of the embankment and appurtenant dike. In addition, remove the overgrowth and debris from the spillway forebay area.

d. Drain the inundated area along the downstream embankment toe and, subsequently, locate the source(s) of any seepage and/or leakage. Furthermore, any seepage and/or leakage observed, including the seepage encountered at the discharge end of the spillway channel, should be assessed in all future inspections, noting any turbidity or changes in rates of flow.

e. Repair the deteriorated concrete associated with the spillway.

f. Assess the operability of the outlet conduit and perform any remedial work deemed necessary to make the conduit fully functional.

g. Develop formal manuals of operation and maintenance to ensure the proper future care of the facility.

GAI Consultants, Inc.

Bernard M. Mihalcin

Bernard M. Mihalcin, P.E.

COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
BERNARD M. MIHALCIN
ENGINEER
20371-E

Date 3 JUNE 1981

Approved by:

James W. Peck

JAMES W. PECK
Colonel, Corps of Engineers
Commander and District Engineer

Date 19 JUNE 1981

OVERVIEW PHOTOGRAPH

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
RICKARDS DAM
NDI# PA-00405, PENNDER# 52-82

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.

1.2 Description of Project.

a. Dam and Appurtenances. Rickards Dam is an earth embankment constructed with a 1-foot thick (minimum) concrete corewall along its centerline. The structure is approximately nine feet high and 370 feet long, including spillway. An earth dike of similar design, about five feet high and 270 feet long, spans a low area about 1000 feet southwest of the main embankment. The facility is constructed with an uncontrolled, rectangular shaped, concrete spillway with an ogee-type weir, 72 feet in length, located near the left abutment. The total combined length of embankment, spillway and dike is about 640 feet. The outlet works consists of a 12-inch diameter, terra cotta pond drain encased in concrete and laid at the base of the earth fill. Control is provided at the inlet by means of a slide gate.

b. Location. Rickards Dam is located on a branch of Hornbecks Creek in Delaware Township, Pike County, Pennsylvania. The facility is situated in the Pocono Mountains of Pike County, less than five miles west of U.S. Route 209, which parallels the Delaware River in this area. The dam, reservoir and watershed are contained within the Lake Maskenozha, Pennsylvania-New Jersey, 7.5 minute U.S.G.S. topographic quadrangle (see Figure 1, Appendix E). The coordinates of the dam are N41° 13.5' and W74° 55.3'.

c. Size Classification. Small (9 feet high, 187 acre-feet storage capacity at top of dam).

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

e. Ownership. Mrs. Urban F. Rickard
Park Road
Box 94
Dingmans Ferry, Pennsylvania 18328

f. Purpose. Recreation.

g. Historical Data. Correspondence contained in PennDER files indicates the idea for a dam at the site of the present day Rickards Dam was originally conceived by Stoll Jagger of Stroudsburg, Pennsylvania, around 1930. Mr. Jagger had issued formal plans and specifications and was subsequently granted a construction permit. Construction never commenced under the direction of Mr. Jagger, however, reportedly due to the overall poor economic climate which prevailed in the 1930's. The land was eventually sold in 1936 to a New Jersey contractor, Urban F. Rickard. Mr. Rickard revised the original plans and began construction of the present facility in 1937.

Available correspondence indicates that construction of the facility was haphazard. Various instances are documented where actual construction differed significantly from the plans and specifications, or was not in compliance with the conditions of the construction permit. Finally, with the facility near completion, a state issued progress report noted that, "work has not been performed in a careful or satisfactory manner." Specifically, state officials cited various deficiencies including: 1) inadequately compacted embankment materials; 2) significant crest settlement; 3) a poorly designed spillway foundation that resulted in concrete distress; 4) leakage along the downstream embankment toe; and 5) an overly steep downstream embankment slope. Available correspondence gives no indication of whether or not any of the above deficiencies were corrected.

Since its completion the facility has been inspected at least three times by the state. Inspection reports dated 1941, 1956 and 1965 are contained in PennDER files. Generally, these reports cite an overall lack of maintenance along with the deficiencies previously stated. Typically, the facility was considered to be in fair to poor condition.

Rickards Dam is now owned by Mrs. Urban F. Rickard, widow of the original builder. No significant modifications have apparently been made since its completion, although a permit to draw down the reservoir to repair a "leak in dike" was issued in 1962. Also, it is noted that repair work on the spillway sidewall is dated 1962.

1.3 Pertinent Data.

a. Drainage Area (square miles). 1.2

b. Discharge at Dam Site.

Discharge Capacity of Outlet Conduit - Discharge curves are not available.

Discharge Capacity of Spillway at Maximum Pool ≅ 710 cfs (see Appendix D, Sheet 8).

c. Elevations (feet above mean sea level). The following elevations were obtained from available drawings and through field measurements based on the approximate elevation of normal pool at 1077.0 feet as estimated from Figure 1, Appendix E (also see Appendix D, Sheet 1).

Top of Dam	1081.0 (design). 1079.1 (field).
Top of Dike	1080.0 (design). 1078.9 (field).
Top of Spillway Sidewalls	1080.5 (field).
Maximum Design Pool	Not known.
Maximum Pool of Record	Not known.
Normal Pool	1077.0
Spillway Crest	1077.0
Upstream Inlet Invert	1068.5 (design).
Downstream Outlet Invert	1068.0 (design).
Streambed at Dam Centerline	1070 (estimate).
Maximum Tailwater	Not known.

d. Reservoir Length (feet).

Top of Dam	2800
Normal Pool	2700

e. Storage (acre-feet).

Top of Dam	187
Normal Pool	98

f. Reservoir Surface (acres).

Top of Dam	55
Normal Pool	26

g. Dam.

Type	Earth.
Length	298 feet (excluding spillway).
Height	Nine feet (field measured; embankment crest to downstream embankment toe).

Top Width	8 feet (design). 12 feet (field).
Upstream Slope	2H:1V (design). 1.75H:1V (field).
Downstream Slope	2H:1V (design). 1.75H:1V (field).
Zoning	Concrete corewall along embankment centerline supported on both sides with earthfill (see Figure 5).
Impervious Core	12-inch thick (minimum) concrete corewall along embankment centerline.
Cutoff	Corewall reportedly extends three feet into the impervious foundation; however, its effectiveness as a cutoff has been questioned since its construction.
Grout Curtain	None indicated.

h. Appurtenant Dike.

Type	Earth fill structure with a concrete or masonry corewall.
Location	Approximately 1000 feet southwest of the main embankment.
Height	Five feet (field measured; embankment crest to downstream embankment toe).
Length	270 feet.
Internal Features	(see Figure 7).

i. Diversion Canal and Regulating Tunnels. None.

j. Spillway.

Type	Uncontrolled, rectangular shaped, concrete spillway with an ogee shaped weir located near the left abutment.
Crest Elevation	1077.0 feet.
Crest Length	72 feet.

k. Outlet Conduit.

Type	12-inch diameter terra cotta pipe encased in concrete located to the right of the spillway.
Length	50 feet (estimated).
Closure and Regulating Facilities	Construction photographs indicated a slide gate at the inlet.
Access	The control mechanism is presently accessible by diver only. (Footbridge shown on design drawings.)

SECTION 2
ENGINEERING DATA

2.1 Design.

a. Design Data Availability and Sources. No formal design reports or calculations are available concerning any aspect of this facility. PennDER files contain correspondence and official documents, dated photographs, and several drawings, the most signficant of which have been included in Appendix E of this report (see Figures 2 through 7).

b. Design Features.

1. Embankment. Design features of the embankment are presented in Figures 3, 4 and 5. As indicated, the embankment is an earth fill structure, straight in plan, with a central concrete corewall. The corewall has a minimum thickness of one foot and reportedly extends one foot into impervious material. A cutoff wall that extends two feet below the corewall was reportedly added as indicated in Figure 5. Both the upstream and downstream embankment slopes were designed at 2H:1V, but field measured to be slightly steeper at 1.75H:1V. Likewise, the design embankment crest width is depicted as eight feet, but was field measured to be about 12 feet. The upstream embankment face is covered with a 12-inch layer of riprap (see Photograph 2).

2. Appurtenant Structures.

a) Spillway. Design features of the spillway are presented in Figures 3, 4 and 6. As indicated, the spillway is an uncontrolled, rectangular shaped, concrete structure located at the left abutment. A 72-foot long (field measured), concrete, ogee-type weir regulates flows through the channel which consists of a mortared riprap floor set between concrete sidewalls.

b) Outlet Conduit. The outlet conduit design is depicted in Figures 4 and 5. As indicated, the outlet conduit consists of a 12-inch diameter terra cotta pipe encased in concrete. Flows through the conduit are reportedly controlled at the inlet by means of a 12-inch diameter slide gate. The gate was originally operated from atop a small riser that extended upward from the outlet inlet and was accessible via a small footbridge.

c) Dike. Design features of the appurtenant earth dike are depicted in Figure 7. The dike is a five foot high earth fill structure about 270 feet long. The dike spans a low area approximately 1000 feet southwest of the main embankment. A comparison of Figures 5 and 7 show the internal design features of the main embankment and appurtenant dike to be very similar. The dike cross-section depicts a two foot thick, masonry corewall with a plastered upstream face as opposed to the concrete corewall indicated in Figure 5 for the main embankment.

c. Specific Design Data and Criteria. No specific design data or information relative to design procedures are available.

2.2 Construction Records.

Memoranda and correspondence contained in PennDER files document much of the construction history of the facility. It is apparent, from the available information, that construction of the facility was haphazard and prolonged. Construction began in 1937 and progressed without proper notification of state officials as required by the conditions contained in the state issued construction permit. Upon inspection of the facility the state required the owner to extend the corewall about two feet deeper into the foundation. This is depicted in both Figures 5 and 7. State officials also noted, prior to the final completion of the structure, that compaction of the embankment materials was less than satisfactory and was resulting in substantial settlement. (Note: Field measurements indicate differential settlement across the crests of both the main embankment and appurtenant dike in excess of one foot below the elevation of the top of the spillway sidewalls.) In addition, rocks and boulders were observed in the embankment fill which was not in compliance with the approved construction specifications. These and other instances prompted a state official to write in a construction progress report that, "work has not been performed in a careful or satisfactory manner."

2.3 Operational Records.

No records of the day-to-day operation of the facility are available.

2.4 Other Investigations.

No formal investigations, other than routine state inspections conducted in 1941, 1956 and 1965, have been performed on this facility subsequent to its completion. The results of the inspections are presented in brief reports contained in PennDER files.

2.5 Evaluation.

The available data are considered sufficient to make a reasonable Phase I evaluation of the facility.

SECTION 3

VISUAL INSPECTION

3.1 Observations.

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

b. Embankment. Observations made during the visual inspection indicate the embankment is in poor condition, primarily attributable to an overall lack of maintenance. The embankment crest and slopes are covered with small trees and lush, fern-like vegetation that effectively obscures view of the facility (see Photographs 1 and 2). No evidence of seepage through the downstream embankment face, animal burrows, sloughing or significant erosion was observed. A footpath is evident through the heavy overgrowth across the downstream embankment face between the embankment crest and the discharge end of the outlet conduit and is considered to be an area of minor erosion (see Photograph 3). Field measurements indicate the existence of differential settlement along the embankment crest in excess of one foot below the top of the spillway sidewalls (see "Profile of Main Embankment Crest from Field Survey," Appendix A). The field team observed minor seepage (≅1 gpm) emanating from the downstream end of the concrete spillway channel about 30 feet below the spillway crest. In addition, it was observed that the discharge end of the outlet conduit was completely inundated (see Photograph 4). Although the seepage observed at the spillway contributes to this ponded condition, it is possible that another seepage source, whose precise location could not be ascertained, exists along the downstream embankment toe. Leakage through the outlet conduit could be also contributing to this condition.

c. Appurtenant Structures.

1. Spillway. The condition of the spillway is considered to be fair. Concrete deterioration in the form of cracking, spalling and popouts is apparent throughout the structure (see Photographs 5, 6, 7, and 8). The deterioration, however, has not advanced sufficiently, as yet, to threaten the overall stability of the spillway. The spillway forebay is silted and heavily overgrown to the extent that spillway discharges would likely be adversely affected. In addition, the discharge channel is cluttered with large driftwood logs.

2. Outlet Conduit. The outlet conduit is considered to be in poor condition. The conduit was not operated in the presence of the inspection team nor did the owner's representative have any specific knowledge of when it was last successfully operated. Remnants of what apparently used to be a footbridge that lead to

the gate control (see Figure 5) were observed along the upstream embankment face. Thus, the control mechanism is presently inaccessible. The discharge end of the conduit is also completely inundated with only the top portion of the concrete headwall visible (see Photograph 4).

3. Dike. The appurtenant earth dike is considered to be in fair condition, due primarily to a general lack of maintenance. The slopes are heavily overgrown with brush and small trees (see Photographs 11 and 12). No evidence of sloughing, erosion, seepage through the downstream embankment face, or animal burrows was observed. Swamp-like conditions exist, however, along the downstream toe area, which suggests an inadequate cutoff.

d. Reservoir Area. The general area surrounding the reservoir is composed of moderate slopes that are heavily forested. No signs of slope distress were observed.

An impoundment known as Long Ridge Dam is located at the southwest corner of the Rickards Lake watershed, approximately one mile upstream of Rickards Dam. Long Ridge Dam (PennDER I.D. No. 52-185) is an earth embankment about 12 feet high and 274 feet long, including spillway. The facility is constructed with a spillway consisting of a small, rock lined, trapezoidal shaped channel with about two feet of available freeboard and a maximum discharge capacity of approximately 190 cfs.

e. Downstream Channel. Discharges from Rickards Dam flow almost immediately into a downstream reservoir known as Lower Rickards Lake. Lower Rickards Dam (PennDER I.D. No. 52-103) is an earth embankment about 10 feet high and 510 feet long, including spillway (see Appendix D, Sheets 19 and 20). The channel below Lower Rickards Dam is gently sloped and confined in a partially developed, wooded valley. It is estimated that between 10 to 20 persons inhabit several dwellings situated near the streambed in the 1700-foot long valley between Lower Rickards Dam and Little Fawn Lake Dam (no PennDER No. issued to date). Located immediately downstream of Little Fawn Lake Dam is Fawn Lake Dam (PennDER I.D. No. 52-182; see Appendix D, Sheets 23 and 24). Camp Log-N-Twig, a seasonal recreation camp located along the stream banks about 6,200 feet downstream of Fawn Lake Dam, likely houses several hundred persons during its peak season. A breach of Rickards Dam could result in substantial damage to the above mentioned downstream impounding facilities and possibly loss of life at the dwellings between Lower Rickards Dam and Little Fawn Lake Dam, as well as at Camp Log-N-Twig. Thus, 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. Deficiencies observed by the inspection team requiring remedial attention include: 1) regrading the crests of embankment and appurtenant dike and removing all excess vegetation from their slopes; 2) draining the inundated area along the downstream embankment toe and locating any areas of embankment seepage and/or outlet conduit leakage; 3) repairing all deteriorated portions of the spillway and removing the overgrowth from the forebay area; and 4) confirming the operability of the outlet conduit.

SECTION 4

OPERATIONAL PROCEDURES

4.1 Normal Operating Procedure.

Rickards Dam is essentially a self-regulating facility. Excess inflows are automatically discharged through the spillway and directed downstream. Typically, the outlet conduit is closed. No formal operations manual is presently available.

4.2 Maintenance of Dam.

According to information contained in PennDER files, Rickards Dam has a well documented history of inadequate maintenance. No maintenance is presently performed on any routine basis. No formal maintenance manual outlining maintenance procedures is available.

4.3 Maintenance of Operating Facilities.

See Section 4.2 above.

4.4 Warning System.

No formal warning system is in effect.

4.5 Evaluation.

No formal operations or maintenance manuals are presently available, but, are recommended to ensure the future proper care and operation of the facility. In addition, warning system procedures should be formalized and incorporated into these manuals.

SECTION 5

HYDROLOGIC/HYDRAULIC EVALUATION

5.1 Design Data.

No formal design data, calculations, or design reports are available.

5.2 Experience Data.

Daily records of reservoir levels and/or spillway dicharges are not available.

5.3 Visual Observations.

The spillway was observed by the inspection team to be in fair condition. Heavy overgrowth in the spillway forebay could potentially block free spillway flow, especially along portions of the weir adjacent to the sidewalls (see Photographs 5 and 7). Thus, the overall discharge capacity of the spillway would be effectively reduced. This condition can be easily rectified through normal maintenance, and consequently, it was assumed to have been corrected in the analysis.

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 D.

5.5 Summary of Analysis.

a. Spillway Design Flood (SDF). 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 Rickards 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). Since the facility is classified near the lower bounds of the small category, the SDF for the facility is considered to be the 1/2 PMF.

b. Results of Analysis. Rickards Dam was evaluated under normal operating conditions. That is, the reservoir was initially at its normal pool or spillway elevation of 1077.0 feet, with the spillway weir unobstructed and discharging freely. The outlet conduit was assumed to be non-functional for the purpose of analysis, since the total flow capacity of the conduit is not such that it would significantly increase the total discharge capabilities of the dam. The spillway consists of an uncontrolled, rectangular shaped, concrete channel with discharges regulated by a concrete, ogee-type weir.

Long Ridge Dam, located about 0.7-mile upstream from Rickards Dam, was also evaluated in this analysis. It, too, was evaluated under normal operating conditions. That is, the reservoir was initially at normal pool, the spillway was assumed to be discharging freely, and the outlet conduit was assumed to be closed. Outflow from Long Ridge Dam was routed directly into Rickards Lake. All pertinent engineering calculations relative to the evaluation of Rickards Dam, including those pertaining to the upstream Long Ridge Dam, are provided in Appendix D.

Overtopping analysis (using the modified HEC-1 computer program) indicated that the discharge/storage capacity of Rickards Dam can accommodate only about 29 percent of the PMF prior to embankment overtopping while Long Ridge Dam can accommodate about 60 percent of the PMF prior to overtopping. Under 1/2 PMF (SDF) conditions, the main embankment was overtopped for about 4.2 hours by depths up to 0.5-foot, while the dike was inundated for about 5.2 hours with a maximum depth of about 0.7-foot (Appendix D, Summary Input/Output Sheets, Sheets G and H). Since the SDF for Rickards Dam is the 1/2 PMF, it can be concluded that the dam has a high potential for overtopping, and thus, for breaching under floods of less than SDF magnitude. It must be noted that if the crest of the main embankment and the appurtenant dike were brought up to the elevation of the spillway sidewalls at 1080.5 feet, the facility would pass and/or store about 57 percent of the PMF.

Since Rickards Dam cannot safely pass a flood of at least 1/2 PMF magnitude, the possibility of embankment failure under floods of less than 1/2 PMF intensity was investigated (in accordance with Corps directive ETL-1110-2-234). Several possible alternative failure schemes were examined, since it is difficult, if not impossible, to determine exactly how or if a specific dam will fail. The major concern of the breaching analysis is with the impact of the various breach discharges on increasing downstream water surface elevations above those to be expected if breaching did not occur.

The modified HEC-1 computer program was used for the breaching analysis with the assumption that the breaching of an earth dam would begin once the embankment was overtopped. Also, in routing the outflows downstream, the channel bed was assumed to be initially dry.

Six possible modes of failure were investigated. First, two sets of breach geometry were evaluated for each of two failure times. The two breach sections chosen were considered to be the minimum and maximum probable failure sections. The maximum section was modeled to include the simultaneous failure of both the main embankment and the appurtenant dike. The two failure times (total time for each breach section to reach its final dimensions) under which the minimum and maximum breach sections were investigated were assumed to be a rapid time (0.5-hour) and a prolonged time (3.0 hours), so that a range of this most sensitive variable might be examined. In addition, an average possible set of breach conditions was analyzed with a failure time of 1.0-hour. Finally, a breach model which involved a failure only at the dike was examined, consisting of an average possible set of breach conditions and a failure time of 1.0-hour (Appendix D, Sheet 26).

The peak breach outflows (resulting from 0.32 PMF conditions) at Rickards Dam ranged from about 1,080 cfs for the minimum section-maximum fail time scheme to about 7,270 cfs for the maximum section-minimum fail time scheme. The peak outflow from the average breach scheme was approximately 2,380 cfs, while the resulting peak outflow from the potential failure at the dike only was about 1,330 cfs. The non-breach 0.32 PMF peak outflow was approximately 820 cfs (Appendix D, Sheet 27).

The breach outflows were first routed through Lower Rickards Dam. Although the various breach schemes, as well as the non-breach scheme, resulted in the overtopping of Lower Rickards Dam by depths ranging from 0.5 to 2.3 feet, the possible failure of this dam was not examined in this analysis (Appendix D, Sheet 28).

The discharges were then routed to Section 1 (see Figure 1, Appendix E), located about 0.2-mile downstream from Lower Rickards Dam, or about 0.4-mile downstream from Rickards Dam. Within this reach, the 0.32 PMF non-breach outflows rose to about 0.6-foot above the damage levels of the nearby residences. However, the water surface levels resulting from the breach models ranged from 0.3-foot to 3.0 feet above the non-breach levels, and thus, ranged up to 3.6 feet above the damage levels of the dwellings (Appendix D, Sheet 28). The consequences of dam failure can better be envisioned if not only the increase in the height of the floodwave is considered, but also the great increase in momentum of the larger and probably swifter moving volume of water.

The discharges were subsequently routed through Little Fawn Lake Dam and Fawn Lake Dam. The embankment at Little Fawn Lake Dam was subjected to extensive overtopping in all cases, including the non-breach situation (Appendix D, Sheet 29). The potential failure of this dam was not examined herein. The embankment at Fawn Lake Dam was also overtopped under the 0.32 PMF non-breach model, by depths of up to 0.6-foot above the minimum embankment crest elevation. However, the discharges from the various breach schemes at Rickards Dam resulted in the overtopping of Fawn Lake Dam by up to 2.1 feet, producing a much greater likelihood for embankment failure (Appendix D, Sheet 29). Failure of Fawn Lake Dam would result in increased property damage and possibly loss of life according to the results of the Phase I Inspection Report entitled "Fawn Lake Dam", by GAI Consultants, Inc., dated June 1981 (see Appendix D, Sheet 19, Note 4).

Therefore, it is concluded that the failure of Rickards Dam would most likely lead to increased property damage and possibly loss of life in the downstream regions.

5.6 Spillway Adequacy.

As presented previously, Rickards Dam can accommodate only about 29 percent of the PMF prior to embankment overtopping. It has been shown that should an event of magnitude greater than this occur, the dam would be overtopped and could possibly fail, endangering downstream residences and increasing the potential for loss of life in the downstream regions. Therefore, the spillway is considered to be seriously inadequate. If, however, the embankment and appurtenant dike were regraded to the elevation of the top of the spillway sidewalls at 1080.5 feet, the facility could then pass and/or store approximately 57 percent of the PMF and the spillway would be considered adequate.

SECTION 6

EVALUATION OF STRUCTURAL INTEGRITY

6.1 Visual Observations.

a. Embankment. The embankment is considered to be in poor condition. The deficiencies noted by the inspection team are attributable to both inadequate adherence to construction specifications and an ongoing lack of adequate maintenance. Field measured settlement in excess of one foot below the elevation of the top of the spillway sidewalls effectively reduces the overall discharge capacity of the spillway prior to embankment overtopping. Furthermore, local low areas will tend to concentrate flows during overtopping, thus, increasing erosion and the potential for embankment failure. Compounding the problem is the fact that, based on available correspondence, lack of adequate compaction during construction likely increases the expected erodibility of the fill. Lack of adequate maintenance has resulted in overgrown slopes and a generally poor appearance. Nevertheless, no evidence of excess embankment stresses, slope instability, or seepage through the downstream embankment face was observed. Heavy overgrowth across the embankment slopes and along the downstream toe hamper visual observation of critical conditions and should be removed. Similarly, the ponded condition in the vicinity of the outlet conduit discharge obscures view of the specific location of any seepage which may be contributing to this condition. The ponded water should be drained and the seepage source(s) located, estimated and recorded on a regular basis.

b. Appurtenant Structures.

1. Spillway. The spillway appears structurally sound and is presently in fair condition. Observed overgrowth in the spillway forebay should be removed to afford maximum discharge capacity. Efforts should be made to repair areas of concrete deterioration. If deterioration were to continue, it is possible that high flows could damage the structure and possibly endanger the embankment.

2. Outlet Conduit. The outlet conduit may be functional; however, it was not operated in the presence of the inspection team. Access to the control mechanism above the elevation of normal pool should be reestablished. In addition, the ponded condition at the discharge end should be alleviated and the outlet kept clear and unobstructed.

3. Dike. The appurtenant earth dike is considered to be in fair condition exhibiting many of the same deficiencies associated with the main embankment. Similarly, the crest of the dike should be raised to the elevation of the top of the spillway sidewalls of the main embankment and excess overgrowth should be removed from the dike crest and slopes. In addition, specific

provisions should be made to include the dike area in any formal maintenance program eventually developed for the main embankment.

6.2 Design and Construction Techniques.

No information is available that details any design particulars. PennDER files do contain information relative to various aspects of the construction of the facility. Evidence of a lack of adherence to construction specifications and generally poor construction practices contributed to our overall poor evaluation of this facility.

6.3 Past Performance.

No records relative to the performance history of the facility are available; however, a drawdown permit was issued in 1962 to ostensibly repair a reported leak in the dike.

6.4 Seismic Stability.

The dam is located in Seismic Zone No. 1 and may be subject to minor earthquake induced dynamic forces. If subsequent engineering evaluations confirm that the structure is statically stable, it is believed that the facility, as constructed, will be able to withstand the expected dynamic forces; however, no calculations and/or investigations were performed to confirm this opinion.

SECTION 7

ASSESSMENT AND RECOMMENDATIONS FOR REMEDIAL MEASURES

7.1 Dam Assessment.

a. Safety. The results of this investigation indicate the facility is in poor condition.

The size classification of the facility is small and its hazard classification is considered to be high. In accordance with the recommended guidelines, the Spillway Design Flood (SDF) for the facility ranges between the 1/2 PMF (Probable Maximum Flood) and the PMF. Since the facility is classified near the lower bounds of the small category, the SDF is considered to be the 1/2 PMF. Results of the hydrologic and hydraulic analysis indicate the facility will pass and/or store only about 29 percent of the PMF prior to embankment overtopping at the low area in the main embankment crest. Breach analysis indicates that failure under 1/2 PMF conditions could lead to increased downstream damage and potential for loss of life. Thus, based on screening criteria provided in the recommended guidelines, the spillway is considered to be seriously inadequate and the facility unsafe, non-emergency.

Calculations also indicate that if the embankment and dike crests were uniformly regraded to the elevation of the top of the spillway sidewalls at 1080.5 feet, the facility could then pass and/or store approximately 57 percent of the PMF and the spillway would be considered adequate.

Structural deficiencies observed by the inspection team included excessive settlement of both the main embankment and dike structures, general deterioration of the spillway and outlet works, and a general lack of surface maintenance. Historical correspondence also strongly question the construction quality of the facility.

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

c. Urgency. The recommendations listed below should be implemented immediately.

d. Necessity for Additional Investigations. Additional investigations are considered necessary to further assess the overall structural integrity of the embankment and appurtenant dike.

7.2 Recommendations/Remedial Measures.

It is recommended that the owner immediately:

a. Develop a formal warning system to notify downstream inhabitants should hazardous embankment conditions develop. The system should include provisions for around-the-clock surveillance of the facility during periods of unusually heavy precipitation.

b. Have the main embankment and appurtenant dike evaluated by a registered professional engineer experienced in the design and construction of earth dams to assess their overall structural integrity and make remedial recommendations as required. At a minimum, the embankment and dike crests should be uniformly regraded to the top of the spillway sidewalls at elevation 1080.5 feet to make the facility hydraulically adequate.

c. Clear all excess vegetation from the slopes and crests of the embankment and appurtenant dike. In addition, remove the overgrowth and debris from the spillway forebay area.

d. Drain the inundated area along the downstream embankment toe and, subsequently, locate the source(s) of any seepage and/or leakage. Furthermore, any seepage and/or leakage observed, including the seepage encountered at the discharge end of the spillway channel, should be assessed in all future inspections noting any turbidity and/or changes in rates of flow.

e. Repair the deteriorated concrete associated with the spillway.

f. Confirm the operability of the outlet conduit and perform any remedial work deemed necessary to make the conduit fully functional. In addition, extend the gate control mechanism vertically upward so that it is accessible above normal pool.

g. Develop formal manuals of operation and maintenance to ensure the proper future care of the facility.

APPENDIX A

VISUAL INSPECTION CHECKLIST AND FIELD SKETCHES

CHECK LIST
VISUAL INSPECTION
PHASE 1

NAME OF DAM Rickards Dam STATE Pennsylvania COUNTY Pike

NDI # PA — 00405 PENNDER # 52-82

TYPE OF DAM Earth SIZE Small HAZARD CATEGORY High

DATE(S) INSPECTION 17 October 1980 WEATHER Clear TEMPERATURE 60° @ 10:00 am

POOL ELEVATION AT TIME OF INSPECTION 1076.2 feet M.S.L.

TAILWATER AT TIME OF INSPECTION N/A M.S.L.

INSPECTION PERSONNEL	OWNER REPRESENTATIVES	*OTHERS*
B. M. Mihalcin	RKR Hess Associates, Inc.	
D. J. Spaeder	Cliff Dennis	
D. L. Bonk		

RECORDED BY B. M. Mihalcin

EMBANKMENT

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA- 00405
SURFACE CRACKS	None observed. Embankment crest and slopes are covered with small trees and thick fern-like vegetation.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None observed.
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	No significant areas of sloughing or erosion were observed; however, dense overgrowth obscures view of the facility. Evidence of minor erosion was observed along the downstream embankment face where a footpath leads from the crest to the discharge end of the outlet conduit.
VERTICAL AND HORIZONTAL ALIGNMENT OF THE CREST	Horizontal - good. Vertical - see "Profile of Dam Crest from Field Survey," Appendix A.
RIPRAP FAILURES	Riprap zone along upstream embankment face appears to be composed of hand placed rock. No riprap failures were observed. Some randomly strewn rocks were also observed along the downstream embankment face.
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Good condition.

EMBANKMENT

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00405
DAMP AREAS IRREGULAR VEGETATION (LUSH OR DEAD PLANTS)	Lush vegetation covers entire embankment. Ponded water is located immediately downstream of the outlet conduit. Precise source of water is not known. The ponded water has submerged the discharge end of the outlet conduit.
ANY NOTICEABLE SEEPAGE	None observed through the downstream embankment face. Minor seepage (≈ 1 gpm) observed at the downstream end of spillway channel about 30 feet downstream of the spillway crest.
STAFF GAGE AND RECORDER	None.
DRAINS	None observed.
APPURTENANT DIKE	Five-foot high earth dike spans a low area approximately 1,000 feet southwest (right of the right abutment) of the main embankment. Downstream area is swamplike. Slopes are rock covered, but, very steep. Dense overgrowth covers both slopes.

OUTLET WORKS

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA- 00405
INTAKE STRUCTURE	Submerged, not observed. Remnants of a steel framed structure are visible along the upstream embankment face extending several feet into the reservoir prior to submerging.
OUTLET CONDUIT (CRACKING AND SPALLING OF CONCRETE SURFACES)	Not exposed and not observed.
OUTLET STRUCTURE	Portions of a concrete headwall visible along the downstream embankment toe. Outlet conduit is submerged by ponded water.
OUTLET CHANNEL	Discharges into a swampy area immediately below the dam. Inlet to Lower Rickards Lake is located less than 300 feet downstream of Rickards Dam.
GATE(S) AND OPERATIONAL EQUIPMENT	None observed.

EMERGENCY SPILLWAY

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA- 00405
TYPE AND CONDITION	Uncontrolled, rectangular shaped, concrete chute channel with an ogee-type crest located at the left abutment. Fair condition. Several cracks observed in weir. Probably shrinkage cracks - not significant at present.
APPROACH CHANNEL	Rock lined approach area partially obstructed by silt, brush and debris upstream of the weir adjacent the sidewalls.
SPILLWAY CHANNEL AND SIDEWALLS	Sidewalls are in fair condition exhibiting spalling, popouts, random cracking and general overall concrete deterioration. Unfinished concrete discharge channel floor apparently sprayed atop a layer of rock lining. Fair condition.
STILLING BASIN PLUNGE POOL	None.
DISCHARGE CHANNEL	See "Outlet Channel," page 4 of 8.
BRIDGE AND PIERS EMERGENCY GATES	None.

SERVICE SPILLWAY

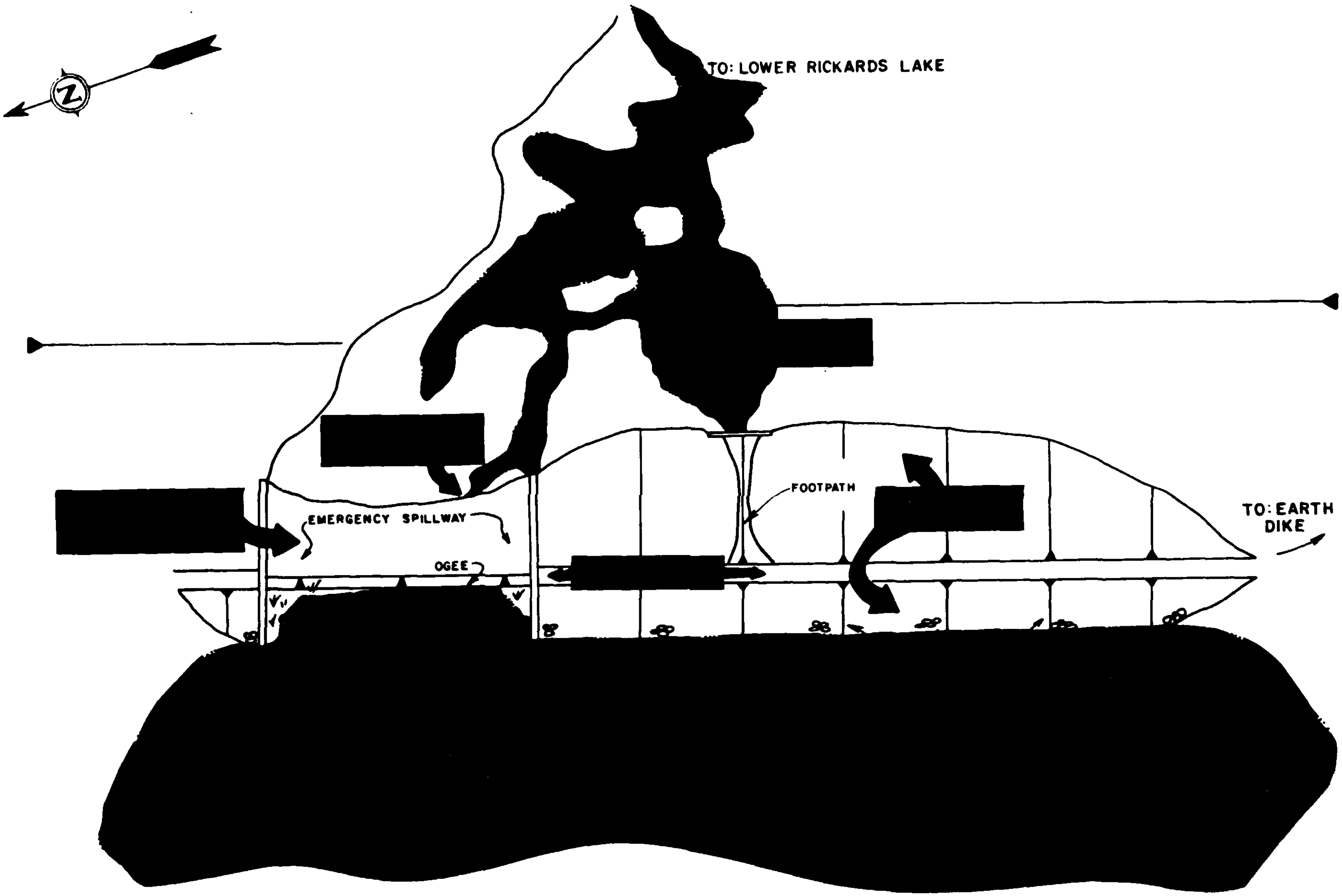
ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00405
TYPE AND CONDITION	N/A.
APPROACH CHANNEL	N/A.
OUTLET STRUCTURE	N/A.
DISCHARGE CHANNEL	N/A.

INSTRUMENTATION

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA - 00405
MONUMENTATION SURVEYS	None.
OBSERVATION WELLS	None.
WEIRS	None.
PIEZOMETERS	None.
OTHERS	None.

RESERVOIR AREA AND DOWNSTREAM CHANNEL

ITEM	OBSERVATIONS/REMARKS/RECOMMENDATIONS NDI# PA- 00405
SLOPES: RESERVOIR	Moderate slopes that are heavily forested.
SEDIMENTATION	None apparent.
DOWNSTREAM CHANNEL (OBSTRUCTIONS, DEBRIS, ETC.)	Discharges immediately into Lower Rickards Dam (PennDER I.D. No. 52-103) located less than 300 feet downstream of Rickards Dam.
SLOPES: CHANNEL VALLEY	Gently sloped channel confined in a forested valley with moderate to steep confining slopes between Lower Rickards Dam and Little Fawn Lake Dam.
APPROXIMATE NUMBER OF HOMES AND POPULATION	Several dwellings are located near the streambed in the reach between Lower Rickards Dam and Fawn Lake Dam (estimated population ≈ 10 to 20 persons). Camp Log-N-Twig, seasonal recreation camp, located about 6,200 feet downstream of Fawn Lake Dam, likely houses several hundred persons during its peak season.



RICKARDS DAM
GENERAL PLAN-FIELD INSPECTION NOTES

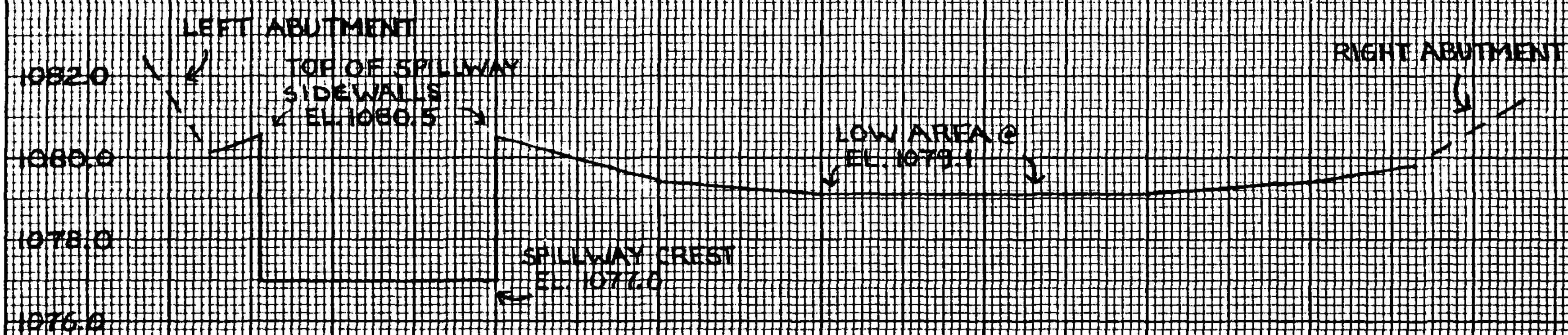
K+E 20 X 20 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.

46 1242

NDI # PA-00405

RICKARDS DAM

PROFILE OF MAIN EMBANKMENT
FROM FIELD SURVEY



SCALE:
VERTICAL: 1 IN. = 4 FT
HORIZONTAL: 1 IN. = 50 FT

SUBJECT RICKARDS DAM
BY DTS DATE 5-9-81 SHEET NO ___ OF ___
CHKD BY DLB DATE 5-11-81 PROJECT NO 80-238-405

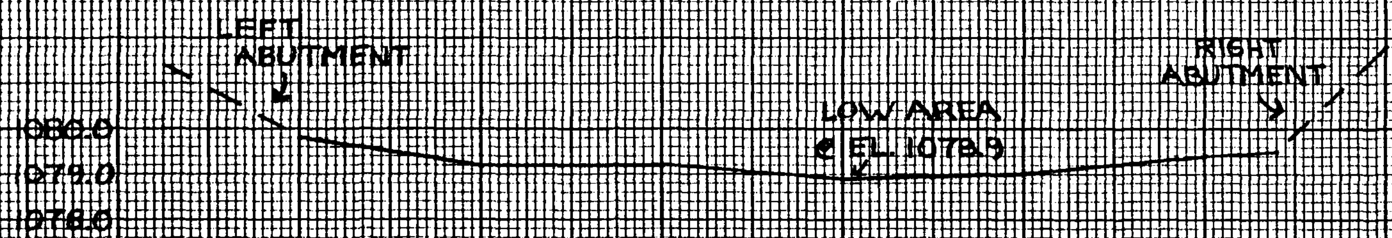
K•E 20 X 20 TO THE INCH • 7 X 10 INCHES
KEUFFEL & ESSER CO. MADE IN U.S.A. 46 1242

NDI # PA-00405

RICKARDS DAM

PROFILE OF APPURTENANT DIKE

FROM FIELD SURVEY



SCALE:
VERTICAL: 1 IN. = 4 FT.
HORIZONTAL: 1 IN. = 50 FT.

SUBJECT RICKARDS DAM
BY DJS DATE 5-9-81 SHEET NO. ___ OF ___
CHKD. BY DLB DATE 5-11-81 PROJECT NO. 80-238-405

APPENDIX B

ENGINEERING DATA CHECKLIST

CHECK LIST
ENGINEERING DATA
PHASE I

NAME OF DAM Rickards Dam

ITEM	REMARKS NDI# PA- 00405
PERSONS INTERVIEWED AND TITLE	Cliff Dennis - Project Engineer, RKR Hess Associates (engineer representing the owner, Mrs. Urban F. Rickard).
REGIONAL VICINITY MAP	See Figure 1, Appendix E.
CONSTRUCTION HISTORY	Constructed in 1937 by Urban F. Rickard, a contractor from Elizabeth, New Jersey. No significant modifications have apparently been made since its completion.
AVAILABLE DRAWINGS	Various design drawings are available from PennDER files. See Figures 2 through 7, Appendix E.
TYPICAL DAM SECTIONS	See Figures 5 and 7, Appendix E.
OUTLETS: PLAN DETAILS DISCHARGE RATINGS	See Figure 5, Appendix E. Discharge rating curves are not available.

CHECK LIST
ENGINEERING DATA
PHASE I
(CONTINUED)

ITEM	REMARKS NDI# PA· 00405
SPILLWAY: PLAN SECTION DETAILS	See Figures 3, 4, and 6, Appendix E.
OPERATING EQUIPMENT PLANS AND DETAILS	See Figure 5, Appendix E. 12-inch diameter slide gate controls flow at the inlet to the outlet conduit. Date of last operation is not known.
DESIGN REPORTS	None available.
GEOLOGY REPORTS	None available.
DESIGN COMPUTATIONS: HYDROLOGY AND HYDRAULICS STABILITY ANALYSES SEEPAGE ANALYSES	None available.
MATERIAL INVESTIGATIONS: BORING RECORDS LABORATORY TESTING FIELD TESTING	PennDER records refer to shallow test pits dug during construction to assess as-built foundation conditions.

CHECK LIST
ENGINEERING DATA
PHASE I
(CONTINUED)

ITEM	REMARKS NDI# PA. 00405
BORROW SOURCES	Correspondence indicates borrow taken from within reservoir area.
POST CONSTRUCTION DAM SURVEYS	None.
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	State inspection reports for the years 1941, 1956, and 1965 are contained in PennDER files.
HIGH POOL RECORDS	No formal records available.
MONITORING SYSTEMS	None.
MODIFICATIONS	None.

CHECK LIST
ENGINEERING DATA
PHASE I
(CONTINUED)

NDI# PA - 00405

ITEM	REMARKS
PRIOR ACCIDENTS OR FAILURES	None recorded.
MAINTENANCE: RECORDS MANUAL	No records or manual available.
OPERATION: RECORDS MANUAL	No records or manual available.
OPERATIONAL PROCEDURES	Self-regulating.
WARNING SYSTEM AND/OR COMMUNICATION FACILITIES	None.
MISCELLANEOUS	Five construction photographs dated 1937-38 are available in PennDER files which generally confirm details shown on construction drawings. Two photographs from state inspections in 1956 and 1965 are also available.

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CHECK LIST
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

NDI ID # PA-00405
PENNDER ID # 52-82

SIZE OF DRAINAGE AREA: 1.2 square miles (total); 1.1 square miles (local).

ELEVATION TOP NORMAL POOL: 1077.0 STORAGE CAPACITY: 98 acre-feet.

ELEVATION TOP FLOOD CONTROL POOL: - STORAGE CAPACITY: -

ELEVATION MAXIMUM DESIGN POOL: - STORAGE CAPACITY: -

ELEVATION TOP DAM: *1079.1 (field) STORAGE CAPACITY: 187 acre-feet.

SPILLWAY DATA

CREST ELEVATION: 1077.0 feet.

TYPE: Uncontrolled, rectangular concrete channel with ogee-type weir.

CREST LENGTH: 72 feet.

CHANNEL LENGTH: 30 feet.

SPILLOVER LOCATION: Left abutment.

NUMBER AND TYPE OF GATES: None.

OUTLET WORKS

TYPE: 12-inch diameter terra-cotta pipe encased in concrete.

LOCATION: Right of spillway.

ENTRANCE INVERTS: 1068.5 (design).

EXIT INVERTS: 1068.0 (design).

EMERGENCY DRAWDOWN FACILITIES: Slide gate at inlet.

HYDROMETEOROLOGICAL GAGES

TYPE: None.

LOCATION: -

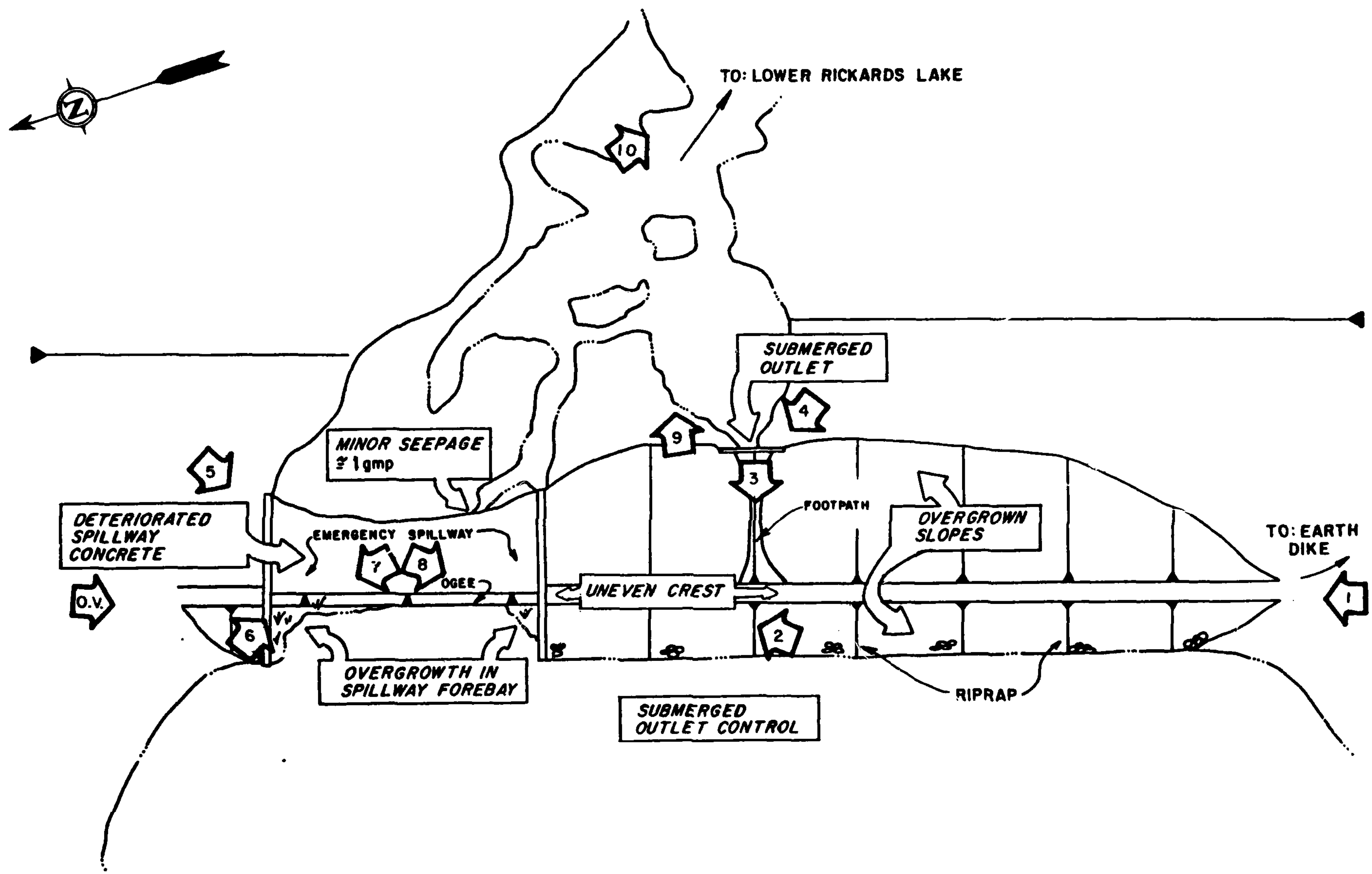
RECORDS: -

MAXIMUM NON-DAMAGING DISCHARGE: Not known.

*Elevation top of dike: 1078.9 (field).

APPENDIX C

PHOTOGRAPHS



RICKARDS DAM
PHOTOGRAPH KEY MAP

PHOTOGRAPH 1 Overview of the heavily overgrown embankment as seen from the right abutment.

PHOTOGRAPH 2 View of the upstream embankment face as seen from the right abutment.

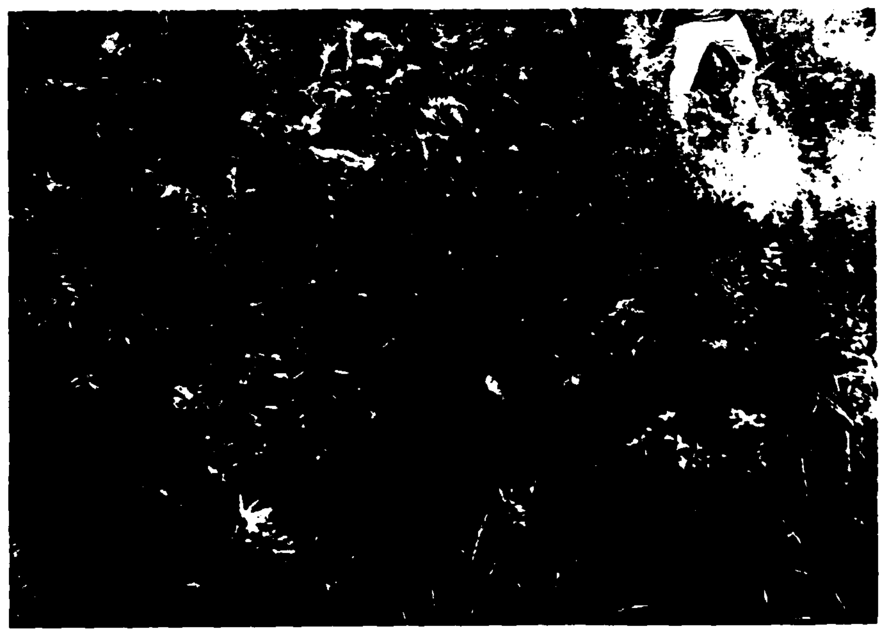
PHOTOGRAPH 3 View of a footpath along the downstream embankment face that leads from the crest to the discharge end of the outlet conduit.

PHOTOGRAPH 4 View of the presently inundated discharge end of the outlet conduit located along the downstream embankment toe to the right of the spillway.


1


2


3


4

PHOTOGRAPH 5 View of the spillway looking toward the right abutment.

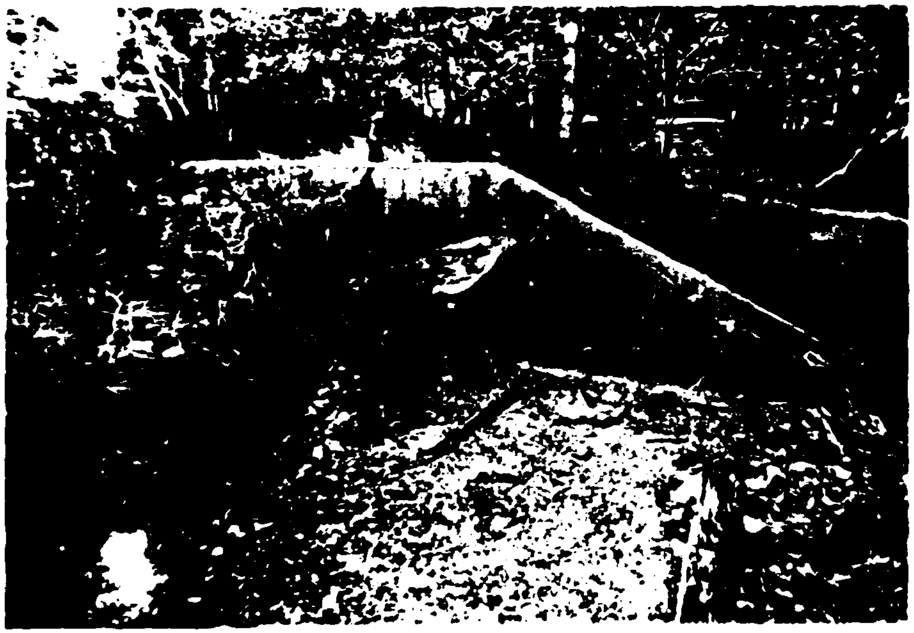
PHOTOGRAPH 6 View of the discharge channel beyond the spillway looking downstream.

PHOTOGRAPH 7 View of the deteriorated left spillway sidewall. Note the vegetation in the spillway forebay in the left center portion of the view.

PHOTOGRAPH 8 View of the deteriorated right spillway sidewall.


5


6


7


8

PHOTOGRAPH 9 View, looking downstream, of the area immediately downstream of the embankment as seen from the embankment crest.

PHOTOGRAPH 10 View, looking downstream, of the inlet to Lower Rickards Dam as seen from about 100 to 150 feet downstream of Rickards Dam.

PHOTOGRAPH 11 View of the earth dike located about 1000 feet southwest of Rickards Dam.

PHOTOGRAPH 12 View of the upstream dike face.

9

10

11

12

APPENDIX D

HYDROLOGIC AND HYDRAULIC ANALYSES

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), time(s) of occurrence the peak discharge(s), and the maximum stage(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 elevation(s) of failure hydrograph(s) for each location.

HYDROLOGY AND HYDRAULIC ANALYSIS
DATA BASE

NAME OF DAM: RICKARDS DAM

PROBABLE MAXIMUM PRECIPITATION (PMP) = 22.0 INCHES/24 HOURS (1)

STATION	1	2	3
STATION DESCRIPTION	LONG RIDGE DAM	RICKARDS DAM	
DRAINAGE AREA (SQUARE MILES)	0.1	1.1	
CUMULATIVE DRAINAGE AREA (SQUARE MILES)	0.1	1.2	
ADJUSTMENT OF PMF FOR DRAINAGE AREA LOCATION (%) (1)	Zone 1	Zone 1	
6 HOURS	111	111	
12 HOURS	123	123	
24 HOURS	133	133	
48 HOURS	142	142	
72 HOURS	-	-	
SNYDER HYDROGRAPH PARAMETERS			
ZONE (2)	1	1	
C_p (3)	0.45	0.45	
C_t (3)	1.23	1.23	
L (MILES) (4)	-	1.7	
L_{ca} (MILES) (4)	-	0.7	
L' (MILES) (4)	0.21	-	
t_p (MILES) (5)	0.48	1.30	
SPILLWAY DATA			
CREST LENGTH (FEET)	10	72	
FREEBOARD (FEET)	2.1	2.1	

(1) HYDROMETEOROLOGICAL REPORT 33, U.S. AMRY CORPS OF ENGINEERS, 1956.
(2) HYDROLOGIC ZONE DEFINED BY CORPS OF ENGINEERS, BALTIMORE DISTRICT, FOR DETERMINATION OF SNYDER COEFFICIENTS (C_p AND C_t).
(3) SNYDER COEFFICIENTS
(4) L = LENGTH OF LONGEST WATERCOURSE FROM DAM TO BASIN DIVIDE
L_{ca} = LENGTH OF LONGEST WATERCOURSE FROM DAM TO POINT OPPOSITE BASIN CENTROID.
L' = LENGTH OF LONGEST WATERCOURSE FROM RESERVOIR INLET TO DRAINAGE DIVIDE.
(5) $t_p = C_t (L \cdot L_{ca})^{0.3}$ or $t_p = C_t (L')^{0.6}$

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 3-19-81 PROJ. NO. 80-238-405
CHKD. BY DLB DATE 5-7-81 SHEET NO. 1 OF 29

GAI CONSULTANTS, INC.
Engineers • Geologists • Planners
Environmental Specialists

DAM STATISTICS

HEIGHT OF DAM = 9 FT (FIELD MEASUREMENT: TOP OF DAM TO DOWNSTREAM EMBANKMENT TOE; "TOP OF DAM" HERE AND ON ALL SUBSEQUENT CALCULATION SHEETS REFERS TO THE LOW AREA IN THE EMBANKMENT CREST)

NORMAL POOL STORAGE CAPACITY = 32×10^6 GALLONS (SEE NOTE 1)
= 98 AC-FT

MAXIMUM POOL STORAGE CAPACITY = 187 AC-FT (SHEET 4)
(@ TOP OF DAM)

DRAINAGE AREA = 1.2 SQ. MI. (PLANIMETERED ON USGS TOPO QUAD – LAKE MASKENOZHA, PA.)
LONG RIDGE LAKE SUB-BASIN = 0.1 SQ. MI.
RICKARDS DAM SUB-BASIN = 1.1 SQ. MI.

ELEVATIONS:

TOP OF DAM (DESIGN)	=	1081.0	(FIG. 3; SEE NOTE 2)
TOP OF DAM (FIELD)	=	1079.1	
TOP OF SPILLWAY SIDEWALLS	=	1080.5	(FIELD MEASURED)
TOP OF DIKE (DESIGN)	=	1080.0	(FIG 7; SEE NOTE 2)
TOP OF DIKE (FIELD)	=	1078.9	
NORMAL POOL	=	1077.0	(SEE NOTE 2)
SPILLWAY CREST	=	1077.0	(FIG 3, SEE NOTE 2)
UPSTREAM INLET INVERT (DESIGN)	=	1068.5	(FIG 5, SEE NOTE 2)
DOWNSTREAM OUTLET INVERT (DESIGN)	=	1068.0	(FIG 5, SEE NOTE 2)
DOWNSTREAM OUTLET INVERT (FIELD)	=	NOT KNOWN	
STREAMBED @ DAM CENTERLINE	=	1070	(EST; FIG. 3; SEE NOTE 2)

NOTE 1: OBTAINED FROM "REPORT UPON THE APPLICATION OF J.E. RICKARDS", FOR THE CONSTRUCTION OF A DAM ACROSS THE NORTH BRANCH OF DECKER CREEK, IN DELAWARE TOWNSHIP, PIKE COUNTY; NOVEMBER 4, 1936; FOUND IN PENN DER FILES.

SUBJECT	DAM SAFETY INSPECTION		
	RICKARDS DAM		
BY DJS	DATE 3-19-81	PROJ. NO. 80-238-405	
CHKD. BY DLB	DATE 5-7-81	SHEET NO. 2	OF 29

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NOTE 2: THE NORMAL POOL OR SPILLWAY CREST ELEVATION IS SOMEWHERE BETWEEN 1060.0 AND 1080.0, ACCORDING TO THE USGS 7.5 MINUTE TOPO QUAD FOR LAKE MASKENOZHA, PA. A NORMAL POOL ELEVATION OF 1077.0 WAS ASSUMED, IN ORDER TO BEST MATCH THE RESULTS OF THE FIELD SURVEY WITH THE CONTOURS INDICATED ON THE TOPO QUAD. SINCE THE DESIGN DRAWINGS ARE BASED ON A NORMAL POOL ELEVATION OF 100.0, A VALUE OF 977.0 FEET (OR 1077.0 - 100.0) MUST BE ADDED TO ALL ELEVATIONS INDICATED ON THESE DRAWINGS. IT IS NOTED THAT THE ELEVATIONS USED IN THIS ANALYSIS ARE CONSIDERED ESTIMATES, AND ARE NOT NECESSARILY ACCURATE.

DAM CLASSIFICATION

DAM SIZE:	SMALL	(REF 1, TABLE 1)
HAZARD CLASSIFICATION:	HIGH	(FIELD OBSERVATION)
REQUIRED SDF:	½ PMF TO PMF	(REF 1, TABLE 3)

HYDROGRAPH PARAMETERS

- LENGTH OF LONGEST WATERCOURSE: $L = 1.7$ MILES
- LENGTH OF LONGEST WATERCOURSE FROM DAM TO A POINT OPPOSITE BASIN CENTROID: $L_{CA} = 0.7$ MILES

(USGS TOPO QUAD – LAKE MASKENOZHA, PA)

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 3-20-81 PROJ. NO. 80-238-405
CHKD. BY DLB DATE 5-7-81 SHEET NO. 3 OF 29

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$C_p = 0.45$
$C_t = 1.23$

(SUPPLIED BY C.O.E., ZONE 1, DELAWARE RIVER BASIN)

SNYDER'S STANDARD LAG: $t_p = C_t \, (L \cdot L_{CA})^{0.3}$

$t_p = 1.23 \, (1.7 \times 0.7)^{0.3}$

$t_p = \underline{1.30}$ HOURS

(NOTE: HYDROGRAPH VARIABLES USED HERE ARE DEFINED IN REF. 2, IN SECTION ENTITLED "SNYDER SYNTHETIC UNIT HYDROGRAPH.")

RESERVOIR CAPACITY

RESERVOIR SURFACE AREAS:

	ELEVATION (FT)	SURFACE AREA (ACRES)
	1071.1	8
	1073.3	14
	1075.0	19
(NORMAL POOL)	1077.0	26
	1080.0	68
	1100.0	146

(SURFACE AREAS AT OR BELOW NORMAL POOL – PLANIMETERED ON FIG. 2 ; SURFACE AREAS ABOVE NORMAL POOL – PLANIMETERED ON USGS TOPO – LAKE MASKENOZHA.)

IT IS ASSUMED THAT THE MODIFIED PRISMOIDAL RELATIONSHIP ADEQUATELY MODELS THE RESERVOIR SURFACE AREA – STORAGE RELATIONSHIP. SINCE THE CAPACITY AT NORMAL POOL IS KNOWN, THE CALCULATED VOLUMES CAN BE ADJUSTED ACCORDINGLY.

$$\Delta V_{1-2} = \frac{h}{3}\left(A_1 + A_2 + \sqrt{A_1 \cdot A_2}\right) \qquad \text{(REF 14, p. 15)}$$

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

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WHERE ΔV_{1-2} = INCREMENTAL VOLUME BETWEEN ELEVATIONS 1 + 2, IN AC-FT,
h = ELEVATION 1 − ELEVATION 2, IN FT,
A_1 = SURFACE AREA AT ELEVATION 1, IN ACRES,
A_2 = SURFACE AREA AT ELEVATION 2, IN ACRES.

ALSO, IT WILL BE ASSUMED THAT THE SURFACE AREA VARIES LINEARLY BETWEEN ELEVATIONS 1077.0 AND 1080.0, AND BETWEEN ELEVATIONS 1080.0 AND 1100.0.

ELEVATION - STORAGE TABLE:

	RESERVOIR ELEVATION (FT)	A ① (AC)	ΔV_{1-2} (AC-FT)	INITIAL CALCULATED TOTAL VOLUME (AC-FT)	ADJUSTED ② FINAL VOLUME (AC-FT)
	1068.5	0	—	—	0
	1071.1	8	6.9	6.9	7
	1073.3	14	23.9	30.8	29
	1075.0	19	27.9	58.7	56
(NORMAL POOL)	1077.0	26	44.8	103.5	98
(TOP OF DAM)	1079.1	55	83.2	186.7	187
	1080.0	68	55.2	241.9	242
(DESIGN TOP OF DAM)	1081.0	72	70.0	311.9	312
	1082.0	76	74.0	385.9	386
	1083.0	80	78.0	463.9	464
	1084.0	84	82.0	545.9	546
	1085.0	88	86.0	631.9	632

① SURFACE AREAS TAKEN FROM SHEET 3. VALUES AT EL. 1079.1 AND ABOVE 1080.0 FOUND BY LINEAR INTERPOLATION.

② ADJUSTED FINAL VOLUME (BELOW NORMAL POOL) = INITIAL CALCULATED VOLUME × $\left(\frac{\text{KNOWN VOL. @ NORMAL POOL}}{\text{INITIAL CALC. VOL @ NORMAL P.}}\right)$
= INITIAL CALC. VOLUME × $\frac{98}{103.5}$
= INITIAL CALC. VOLUME × 0.947

— ZERO STORAGE ASSUMED AT UPSTREAM INLET INVERT OF OUTLET CONDUIT (SHEET 1,

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PMP CALCULATIONS

- APPROXIMATE RAINFALL INDEX = 22.0 INCHES (CORRESPONDING TO A DURATION OF 24 HOURS AND A DRAINAGE AREA OF 200 SQUARE MILES)

(REF. 3, FIG. 1)

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

- ASSUME DATA CORRESPONDING TO A 10-SQUARE MILE AREA MAY BE APPLIED TO THIS 1.1 SQUARE MILE BASIN.

DURATION (HRS)	PERCENT OF INDEX RAINFALL
6	111
12	123
24	133
48	142

(REF 3, FIG. 2)

HOP BROOK FACTOR (ADJUSTMENT FOR BASIN SHAPE AND FOR THE LESSER LIKELIHOOD OF A SEVERE STORM CENTERING OVER A SMALL BASIN) FOR A DRAINAGE AREA OF 1.1 SQUARE MILES IS 0.80.

(REF 4, p. 48)

SUBJECT DAM SAFETY INSPECTION

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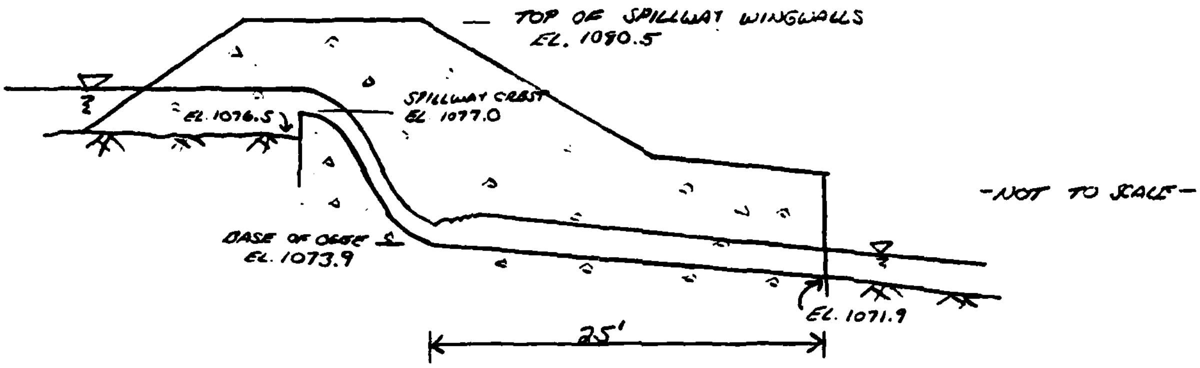
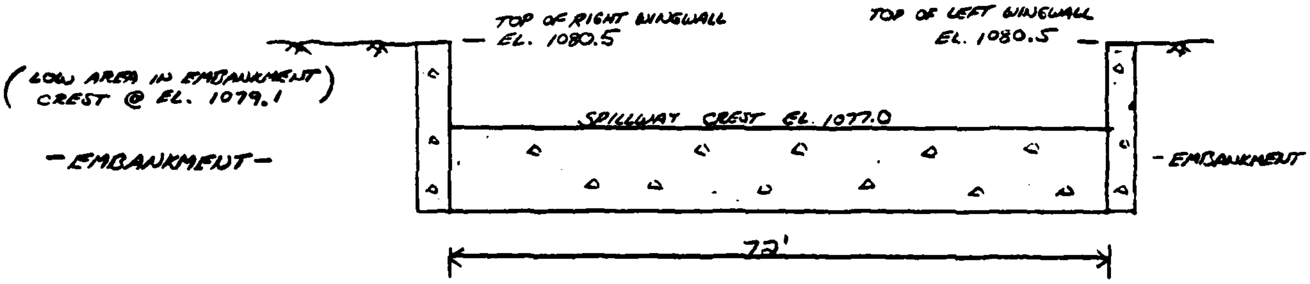
SPILLWAY CAPACITY

PROFILE:



CROSS-SECTION:

- LOOKING UPSTREAM -



- NOT TO SCALE -

(SKETCHES BASED ON FIELD MEASUREMENTS AND OBSERVATIONS AND DESIGN DRAWINGS)

THE SPILLWAY CONSISTS OF RECTANGULAR SHAPED CONCRETE CHANNEL WITH DISCHARGES REGULATED BY A CONCRETE OGEE-LIKE WEIR. ALTHOUGH THE WEIR WAS PARTIALLY OBSTRUCTED AT EITHER END AT THE TIME OF INSPECTION, THE ENTIRE WEIR LENGTH WAS ASSUMED TO BE EFFECTIVE IN THIS ANALYSIS.

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DISCHARGE OVER THE OGEE-LIKE WEIR CAN BE ESTIMATED BY THE RELATIONSHIP

$$Q = CLH^{3/2} \qquad \text{(REF 4, p. 373)}$$

WHERE Q = WEIR DISCHARGE, IN CFS,
C = COEFFICIENT OF DISCHARGE,
L = WEIR CREST LENGTH = 72 FT,
H = HEAD, IN FT.

IT IS ASSUMED THAT THE RELATIONSHIPS FOR OGEE-TYPE WEIRS, GIVEN IN REF 4, pp 372-382, CAN BE APPLIED TO THIS WEIR. THE DESIGN HEAD IS ASSUMED TO BE AT THE TOP OF THE WINGWALLS, OR 3.5 FT. FOR A FOREBAY DEPTH OF ABOUT 0.5 FT,

$$\frac{P}{H_o} = \frac{0.5}{3.5} = 0.14$$

$$\therefore C_o = \underline{3.47}$$

AS THE HEAD ON THE WEIR BECOMES SMALL, DISCHARGE IS REDUCED DISPROPORTIONATELY, DUE TO THE ROUGHNESS AND THE CONTACT PRESSURE BETWEEN THE WATER AND THE WEIR SURFACE. THUS, THE DISCHARGE COEFFICIENT (C) TAKES ON A LOWER VALUE THAN THAT OF DESIGN HEAD. THE OPPOSITE TREND OCCURS FOR HEADS GREATER THAN THAT OF DESIGN. THEREFORE, THE DISCHARGE COEFFICIENT WILL BE MODIFIED APPROPRIATELY, ACCORDING TO REF 4, FIG. 250.

IT WILL ALSO BE ASSUMED THAT THERE ARE NO APPROACH LOSSES HERE. THE SPILLWAY RATING TABLE IS PROVIDED ON SHEET 8.

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

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SPILLWAY RATING TABLE:

	RESERVOIR ELEVATION (FT)	H (FT)	H/H_o	C/C_o ①	C ②	Q ③ (CFS)
(NORMAL POOL)	1077.0	0	—	—	—	0
	1078.0	1.0	0.29	0.88	3.05	220
	1079.0	2.0	0.57	0.93	3.23	660
(TOP OF DAM)	1079.1	2.1	0.60	0.94	3.26	710
	1079.5	2.5	0.71	0.96	3.33	950
	1080.0	3.0	0.86	0.98	3.40	1270
(TOP OF SPILLWAY SIDEWALLS)	1080.5	3.5	1.00	1.00	3.47	1640
	1081.0	4.0	1.14	1.02	3.54	2040
	1081.5	4.5	1.29	1.04	3.61	2480
	1082.0	5.0	1.43	1.05	3.64	2930
	1083.0	6.0	1.71	1.07	3.71	3930
	1084.0	7.0	2.00	1.07	3.71	4950
	1085.0	8.0	2.29	1.07	3.71	6040

① FROM REF 4, FIG. 250, p. 378.

② $C = C/C_o \times C_o = C/C_o \times 3.47$

③ $Q = CLH^{3/2}$, WHERE $L = 72$ FT;
ROUNDED TO NEAREST 10 CFS.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 3-23-81 PROJ. NO. 80-238-405
CHKD. BY DLB DATE 5-7-81 SHEET NO. 9 OF 29

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EMBANKMENT RATING CURVE

ASSUME THAT THE EMBANKMENT BEHAVES ESSENTIALLY AS A BROAD-CRESTED WEIR WHEN OVERTOPPING OCCURS. THUS, THE DISCHARGE CAN BE ESTIMATED BY THE RELATIONSHIP

$$Q = CLH^{3/2}$$ (REF 5, p. 5-23)

WHERE Q = DISCHARGE OVER EMBANKMENT, IN CFS,
L = LENGTH OF EMBANKMENT OVERTOPPED, IN FT,
H = HEAD, IN FT; IN THIS CASE IT IS THE AVERAGE "FLOW AREA WEIGHTED HEAD" ABOVE THE LOW AREA IN THE EMBANKMENT CREST; AND
C = COEFFICIENT OF DISCHARGE, DEPENDENT UPON THE HEAD AND THE WEIR BREADTH.

LENGTH OF EMBANKMENT INUNDATED
VS. RESERVOIR ELEVATION:

	ELEVATION (FT)	LENGTH - MAIN EMBANKMENT (FT)	LENGTH - DIKE (FT)	TOTAL LENGTH (FT)
(TOP OF DIKE)	1078.9	—	0	0
	1079.0	—	65	65
(TOP OF DAM)	1079.1	0	95	95
	1079.4	200	215	415
	1079.8	250	275	525
	1080.0	265	280	545
	1080.2	285	290	575
	1080.5	315	300	615
	1080.7	320	305	625
	1081.0	330	315	645
	1081.5	345	335	680
	1082.0	365	350	715
	1083.0	405	375	780
	1084.0	445	400	845
	1085.0	485	430	915

(FROM FIELD SURVEY AND USGS TOPO QUAD - LAKE MASKENOZHA, PA.)

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

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ASSUME THAT INCREMENTAL DISCHARGES OVER THE EMBANKMENT FOR SUCCESSIVE RESERVOIR ELEVATIONS ARE APPROXIMATELY TRAPEZOIDAL IN CROSS-SECTIONAL FLOW AREA. THEN ANY INCREMENTAL AREA OF FLOW CAN BE ESTIMATED AS $H_i[(L_1+L_2)/2]$, WHERE L_1 = LENGTH OF EMBANKMENT OVERTOPPED AT HIGHER ELEVATION, L_2 = LENGTH AT LOWER ELEVATION, H_i = DIFFERENCE IN ELEVATIONS. THUS, THE TOTAL AVERAGE "FLOW AREA WEIGHTED HEAD" CAN BE ESTIMATED AS

H_w = (TOTAL FLOW AREA/L_1).

EMBANKMENT RATING TABLE:

	RESERVOIR ELEVATION (FT)	L_1 (FT)	L_2 (FT)	INCREMENTAL HEAD, H_i (FT)	① INCREMENTAL FLOW AREA, A_i (FT²)	TOTAL FLOW AREA, A_T (FT²)	② WEIGHTED HEAD, H_w (FT)	③ H_w/l	④ C	⑤ Q (CFS)
(TOP OF DIKE)	1078.9	0	–	–	–	–	–	–	–	0
	1079.0	65	0	0.1	3	3	0.05	0.004	2.90	0
(TOP OF DAM)	1079.1	95	65	0.1	8	11	0.12	0.01	2.94	10
	1079.4	415	95	0.3	77	88	0.21	0.02	2.97	120
	1079.8	525	415	0.4	188	276	0.53	0.04	3.02	610
	1080.0	545	525	0.2	107	383	0.70	0.06	3.03	970
	1080.2	575	545	0.2	112	495	0.86	0.07	3.03	1390
	1080.5	615	575	0.3	179	674	1.1	0.09	3.04	2160
	1080.7	625	615	0.2	124	798	1.3	0.11	3.04	2820
	1081.0	645	625	0.3	191	989	1.5	0.13	3.04	3600
	1081.5	680	645	0.5	331	1320	1.9	0.16	3.06	5450
	1082.0	715	680	0.5	349	1669	2.3	0.19	3.07	7660
	1083.0	780	715	1.0	748	2417	3.1	0.26	3.09	13,160
	1084.0	845	780	1.0	813	3230	3.8	0.32	3.09	19,340
	1085.0	915	845	1.0	880	4110	4.5	0.38	3.09	26,990

① $A_i = H_i[(L_1+L_2)/2]$

② $H_w = A_T/L_1$

③ l = BREADTH OF CREST = 12 FT (FIELD MEASURED - MAIN EMBANKMENT AND DIKE)

④ $C = f(H, l)$; FROM REF. 12, FIG. 24.

⑤ $Q = CL_1H_w^{3/2}$ (ROUNDED TO NEAREST 10 CFS)

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

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TOTAL FACILITY RATING TABLE

$Q_{TOTAL} = Q_{SPILLWAY} + Q_{EMBANKMENT}$

	RESERVOIR ELEVATION (FT)	① $Q_{SPILLWAY}$ (CFS)	② $Q_{EMBANKMENT}$ (CFS)	Q_{TOTAL} (CFS)
	1077.0	0	–	0
	1078.0	220	–	220
	1079.0	660	0	660
(TOP OF DAM)	1079.1	710	10 **	720
	1079.4	890 *	120	1010
	1079.5	950	240 *	1190
	1079.8	1140 *	610	1750
	1080.0	1270	970	2240
	1080.2	1420 *	1390	2810
	1080.5	1640	2160	3800
	1080.7	1800 *	2820	4620
	1081.0	2040	3600	5640
	1081.5	2480	5450	7930
	1082.0	2930	7660	10,590
	1083.0	3930	13,160	17,090
	1084.0	4950	19,340	24,290
	1085.0	6040	26,990	33,030

* – LINEARLY INTERPOLATED FROM RATING TABLE – SHEET 8.
(ROUNDED TO NEAREST 10 CFS)

** – DISCHARGE OVER DIKE ONLY.

① FROM SHEET 8.
② FROM SHEET 10.

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UPSTREAM DAM: LONG RIDGE DAM

– DAM STATISTICS:

- HEIGHT OF DAM = 12 FT (FIELD MEASURED: TOP OF DAM TO DOWNSTREAM EMBANKMENT TOE.)
- ELEVATION OF NORMAL POOL ≈ 1188.0 (SEE NOTE 3)
- ELEVATION OF TOP OF DAM ≈ 1190.1 (FIELD SURVEY)

– HYDROGRAPH PARAMETERS:

- LENGTH OF LONGEST WATERCOURSE FROM RESERVOIR INLET TO BASIN DIVIDE: $L' = 0.21$ MI.

(USGS TOPO QUAD – LAKE MASKENOZHA, PA)

$C_p = 0.45$, $C_t = 1.23$ (SHEET 3)

$$t_p = C_t (L')^{0.6} = (1.23)(0.21)^{0.6}$$
$$\approx 0.48 \text{ HOURS}$$

(NOTE: SINCE L_{CA}, THE LENGTH OF THE LONGEST WATERCOURSE FROM THE DAM TO A POINT OPPOSITE THE BASIN CENTROID, IS LESS THAN THE LENGTH OF THE RESERVOIR, THE SNYDER STANDARD LAG IS ESTIMATED AS $t_p = C_t (L')^{0.6}$, AS PER C.O.E. [BALTIMORE DISTRICT].)

NOTE 3: THE NORMAL POOL ELEVATION WAS ESTIMATED FROM THE USGS TOPO QUAD – LAKE MASKENOZHA, PA.

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LONG RIDGE DAM:

– RESERVOIR STORAGE CAPACITY:

SURFACE AREA (S.A) @ EL. 1180 ≃ 2 ACRES
S.A. @ NORMAL POOL (EL. 1188) ≃ 9 ACRES
S.A. @ EL. 1200 ≃ 18 ACRES

(PLANIMETERED ON USGS TOPO QUAD - LAKE MASKENOZHA)

S.A. @ TOP OF DAM (EL. 1190.1) = 10.6 ACRES
(BY LINEAR INTERPOLATION)

THE "ZERO-STORAGE" ELEVATION IS ASSUMED TO BE AT APPROXIMATELY THE SAME ELEVATION AS THE DOWNSTREAM TOE OF THE DAM, AT ELEV. 1178.

– ELEVATION STORAGE RELATIONSHIP:

THE ELEVATION-STORAGE RELATIONSHIP IS COMPUTED INTERNALLY IN THE HEC-1 PROGRAM, BY USE OF THE CONIC METHOD, BASED ON THE SURFACE AREA DATA GIVEN ABOVE (SEE SUMMARY INPUT/OUTPUT SHEETS).

– PMP DATA – SEE SHEET 5.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

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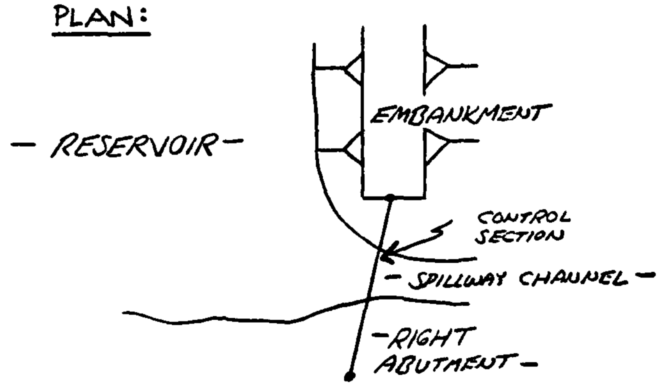
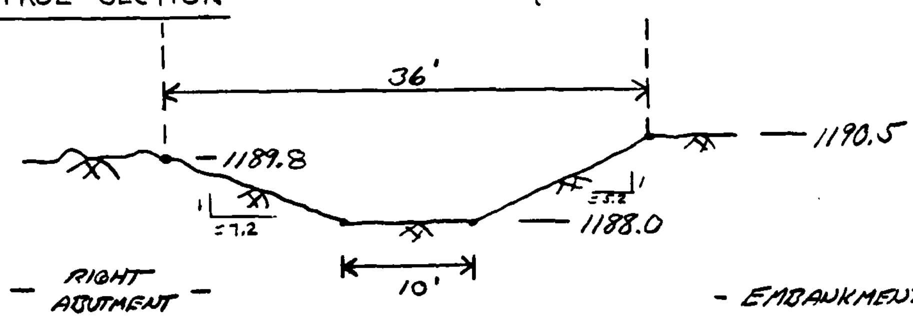
LONG RIDGE DAM:

– SPILLWAY CAPACITY:

PLAN:



CONTROL SECTION: (LOOKING UPSTREAM)



(NOT TO SCALE)

– SKETCHES BASED ON FIELD NOTES AND OBSERVATIONS.

THE SPILLWAY CONSISTS OF AN UNCONTROLLED, TRAPEZOIDAL SHAPED, PARTIALLY ROCK-LINED CHANNEL CUT THROUGH THE EMBANKMENT NEAR ITS RIGHT ABUTMENT. THE CONTROL SECTION IS LOCATED NEAR THE RESERVOIR OUTLET, AS SKETCHED ABOVE.

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LONG RIDGE DAM:

BASED ON THE ASSUMPTION OF CRITICAL FLOW AT THE CONTROL SECTION,

$$\frac{Q^2 T}{g A^3} = 1.0 \qquad \text{(REF 5, p. 8-7)}$$

WHERE Q = DISCHARGE, IN CFS,
T = TOP WIDTH OF FLOW AREA, IN FT,
g = GRAVITATIONAL ACCELERATION CONSTANT = 32.2 FT/SEC^2,
A = FLOW AREA, IN FT.^{2}

ALSO,

$$H_m = D_c + \frac{D_m}{2}$$

AND

$$D_m = A/T \qquad \text{(REF 5, p. 8-8)}$$

WHERE

H_m = TOTAL HEAD AT CRITICAL DEPTH, OR MINIMUM SPECIFIC ENERGY, IN FT,
D_c = CRITICAL DEPTH, IN FT,
D_m = MEAN DEPTH OF FLOW AREA, IN FT.

THE RESERVOIR ELEVATION CORRESPONDING TO ANY PARTICULAR DISCHARGE IS THEN $H_m + 1188.0$ (WHERE INVERT OF CONTROL SECTION = 1188.0). THIS IS BASED ON THE ASSUMPTION OF ZERO-VELOCITY HEAD AT THE RESERVOIR JUST UPSTREAM OF THE CONTROL SECTION, AND NEGLIGIBLE HEAD LOSS TO THE CONTROL SECTION → NO APPROACH LOSSES.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

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LONG RIDGE DAM:

SPILLWAY RATING TABLE:

D_c (FT)	$A^{①}$ (FT^2)	$T^{②}$ (FT)	$D_m^{③}$ (FT)	$H_m^{④}$ (FT)	$Q^{⑤}$ (CFS)	RESERVOIR ELEVATION⑥ (FT)	
0.5	6.6	16.2	0.41	0.7	24	1188.7	
1.0	16.2	22.4	0.72	1.4	78	1189.4	
1.5	29.0	28.6	1.01	2.0	166	1190.0	
1.6	31.9	29.8	1.07	2.1	187	1190.1	(TOP OF DAM)
1.9	41.4	32.9	1.26	2.5	264	1190.5	
2.2	51.5	34.4	1.50	3.0	358	1191.0	
2.5	62.1	36.0	1.73	3.4	463	1191.4	
2.8	72.9	36.0	2.03	3.8	589	1191.8	
3.3	90.9	36.0	2.53	4.6	820	1192.6	

① FOR $D_c \le 1.8$, $A = 10D_c + \frac{7.2}{2}D_c^2 + \frac{5.2}{2}D_c^2 = 10D_c + 6.2D_c^2$

FOR $1.8 \le D_c \le 2.5$, $A = 38.1 + 32.4(D_c - 1.8) + \frac{5.2}{2}(D_c - 1.8)^2$

FOR $D_c \ge 2.5$, $A = 62.1 + 36.0(D_c - 2.5)$

② FOR $D_c \le 1.8$, $T = 10 + 7.2D_c + 5.2D_c = 10 + 12.4D_c$

FOR $1.8 \le D_c \le 2.5$, $T = 10 + 13 + 5.2D_c = 23 + 5.2D_c$

FOR $D_c \ge 2.5$, $T = 36$

③ $D_m = A/T$

④ $H_m = D_c + D_m/2$

⑤ $Q = \sqrt{gA^3/T}$

⑥ RESERVOIR ELEVATION $= H_m + 1188.0$

SUBJECT DAM SAFETY INSPECTION
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LONG RIDGE DAM:

— EMBANKMENT RATING CURVE:

ASSUME THAT THE EMBANKMENT CREST BEHAVES ESSENTIALLY AS A BROAD-CRESTED WEIR WHEN OVERTOPPING OCCURS. THUS, THE DISCHARGE CAN BE ESTIMATED BY THE RELATIONSHIP

$$Q = CLH^{3/2}$$ (SEE SHEET 9)

THE ASSUMPTIONS AND METHODOLOGY USED ON SHEETS 9 + 10, FOR THE RICKARDS DAM EMBANKMENT RATING TABLE, ARE USED HERE.

EMBANKMENT RATING TABLE:

	ELEVATION (FT)	L_1 ① (FT)	L_2 (FT)	INCREMENTAL HEAD, H_i (FT)	INCREMENTAL FLOW AREA, A_i ② (FT²)	TOTAL FLOW AREA, A_T (FT²)	WEIGHTED HEAD, H_w ③ (FT)	H_w/l ④	C ⑤	Q ⑥ (CFS)
(LOW AREA-RIGHT ABUTMENT)	1189.8	0	—	—	—	—	—	—	—	0
(TOP OF DAM)	1190.1	5	0	0.3	1	1	0.20	0.02	2.97	0
	1190.2	65	5	0.1	4	5	0.08	0.01	2.92	0
	1190.3	135	65	0.1	10	15	0.11	0.01	2.94	10
	1190.5	180	135	0.2	32	47	0.26	0.03	2.99	70
	1190.7	195	180	0.2	38	85	0.44	0.05	3.01	170
	1191.0	220	195	0.3	62	147	0.67	0.07	3.03	370
	1191.3	255	220	0.3	71	218	0.85	0.09	3.03	610
	1191.6	255	255	0.3	77	295	1.16	0.13	3.04	970
	1192.0	260	255	0.4	103	398	1.53	0.17	3.06	1510

① L_1 = LENGTH OF EMBANKMENT OVERTOPPED; FROM FIELD SURVEY AND USGS TOPO - LAKE MASKENOZHA.
② $A_i = H_i\,[(L_1 + L_2)/2]$
③ $H_w = A_T/L_1$
④ l = BREADTH OF CREST = 9 FT (FIELD MEASURED)
⑤ $C = f(H_w, l)$; FROM REF. 12, FIG 24.
⑥ $Q = CL_1H_w^{3/2}$ (TO NEAREST 10 CFS)

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DSS DATE 3-30-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 18 OF 29

GAI CONSULTANTS, INC.
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LONG RIDGE DAM:

TOTAL FACILITY RATING TABLE:

$$Q_{TOTAL} = Q_{SPILLWAY} + Q_{EMBANKMENT}$$

	ELEVATION (FT)	① $Q_{SPILLWAY}$ (CFS)	② $Q_{EMBANKMENT}$ (CFS)	Q_{TOTAL} (CFS)
	1188.0	0	-	0
	1188.7	20	-	20
	1189.4	80	-	80
	1190.0	170	-	170
(TOP OF DAM)	1190.1	190	0	190
	1190.2	210*	0	210
	1190.3	230*	10	240
	1190.5	260	70	330
	1190.7	300*	170	470
	1191.0	360	370	730
	1191.3	440*	610	1050
	1191.6	530*	970	1500
	1192.0	650*	1510	2160

* - LINEARLY INTERPOLATED FROM RATING TABLE - SHEET 16. (ROUNDED TO NEAREST 10 CFS).

① FROM RATING TABLE - SHEET 16 (ROUNDED TO NEAREST 10 CFS)
② FROM SHEET 17.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 4-21-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 19 OF 29

GAI CONSULTANTS, INC.
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DOWNSTREAM DAMS

1) LOWER RICKARDS DAM:

- HEIGHT OF DAM = 10 FT (SEE NOTE 4)
- ELEVATION OF NORMAL POOL = 1070.0 "
- ELEVATION OF TOP OF DAM = 1071.7 "

- RESERVOIR STORAGE CAPACITY:

THE ELEVATION-STORAGE RELATIONSHIP IS COMPUTED INTERNALLY IN THE HEC-1 PROGRAM, BASED ON THE DATA GIVEN BELOW: (SEE NOTE 4)

RESERVOIR ELEVATION (FT)	SURFACE AREA (AC)
1055.0 *	0
1070.0	15
1071.7	17.4
1080.0	29

* – ELEVATION REQUIRED IN ORDER TO MAINTAIN A NORMAL POOL STORAGE OF 75 AC-FT (SEE NOTE 4).

NOTE 4: DATA TAKEN FROM "PHASE I INSPECTION REPORT, NATIONAL DAM INSPECTION PROGRAM, FAWN LAKE DAM", PENN DER I.D. NO. 52-182, NDI I.D. NO. PA-00822, PREPARED BY GAI CONSULTANTS, INC.; JUNE 1981.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 4-22-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 20 OF 29

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LOWER RICKARDS DAM:

– SPILLWAY CAPACITY:

THE SPILLWAY RATING CURVE IS COMPUTED INTERNALLY IN THE HEC-1 COMPUTER PROGRAM, BY USE OF THE WEIR EQUATION AND THE DATA GIVEN BELOW:

$$Q = CLH^{3/2}$$ (SHEET 7)

WHERE Q = DISCHARGE, IN CFS,
C = DISCHARGE COEFFICIENT = 2.7,
L = WEIR LENGTH = 35 FT,
H = HEAD, IN FT.

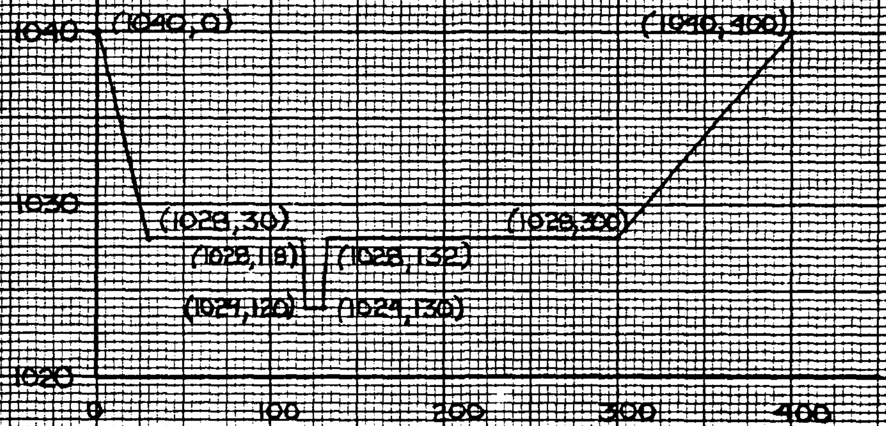
(SEE NOTE 4)

– EMBANKMENT RATING TABLE:

DISCHARGE OVER THE EMBANKMENT WILL BE COMPUTED INTERNALLY IN THE HEC-1 PROGRAM, BASED ON THE WEIR EQUATION. THE LENGTH OF EMBANKMENT INUNDATED WILL BE ASSUMED TO REMAIN CONSTANT AT 510 FEET FOR ALL RESERVOIR ELEVATIONS. THE DISCHARGE COEFFICIENT WILL BE ASSUMED TO BE ON THE ORDER OF 3.0.

(SEE NOTE 4)

DOWNSTREAM ROUTING SECTION



SECTION 1,
≈ 1080 FT. D.S. FROM LOWER
RICKARDS LAKE DAM

INVERT ≈ 1024.0
CHANNEL SLOPE ≈ 0.016
$n_{LOB} \approx n_{ROB} \approx 0.080$
$n_{CH} \approx 0.045$
DAMAGE LEVEL ≈ 1028

(NOTE: SECTION BASED ON FIELD MEASUREMENTS AND OBSERVATIONS AND USGS TOPO QUAD – LAKE MASKENOZHA, PA. ELEVATIONS ARE CONSIDERED ESTIMATES AND ARE NOT NECESSARILY ACCURATE.)

46 1242
K-E 20 X 20 TO THE INCH • 7 X 10 INCHES KEUFFEL & ESSER CO. MADE IN U.S.A.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 4-22-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 22 OF 29

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2) LITTLE FAWN LAKE DAM

- HEIGHT OF DAM = 9 FT (SEE NOTE 4)

- ELEVATION OF NORMAL POOL = 1010.0 "

- ELEVATION OF TOP OF DAM = 1012.4 "

- RESERVOIR STORAGE CAPACITY :

THE ELEVATION - STORAGE RELATIONSHIP IS COMPUTED INTERNALLY IN THE HEC-1 PROGRAM, BASED ON THE DATA GIVEN BELOW : (SEE NOTE 4)

RESERVOIR ELEVATION (FT)	SURFACE AREA (ACRES)
1003.0	0
1010.0	2.5
1012.4	3.5
1020.0	6.5

- SPILLWAY RATING TABLE : (SEE NOTE 4)

RESERVOIR ELEVATION (FT)	OUTFLOW (CFS)
1010.0	0
1010.7	20
1011.4	50
1012.1	100
1012.4	130
1013.0	190
1014.0	320
1015.1	470
1016.0	610
1017.0	790

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 4-22-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 23 OF 29

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LITTLE FAWN LAKE DAM:

- EMBANKMENT RATING TABLE:

DISCHARGE OVER THE EMBANKMENT WILL BE COMPUTED INTERNALLY IN THE HEC-1 PROGRAM, WITH THE ASSUMPTION THAT CRITICAL DEPTH OCCURS ON THE CREST, AND WITH THE CREST PROFILE REPRESENTED BY A SERIES OF TRAPEZOIDS. THE INPUT DATA IS GIVEN BELOW:

(SEE NOTE 4)

RESERVOIR ELEVATION (FT)	LENGTH OF EMBANKMENT INUNDATED (FT)
1012.4	10
1012.7	50
1013.0	90
1013.2	210
1013.5	300
1014.0	350
1015.0	360
1016.0	370
1018.0	390

3) FAWN LAKE DAM

- HEIGHT OF DAM ≈ 22 FT (SEE NOTE 4)

- ELEVATION OF NORMAL POOL ≈ 997.0 "

- ELEVATION OF TOP OF DAM ≈ 999.7 "

- RESERVOIR STORAGE CAPACITY:

THE ELEVATION-STORAGE RELATIONSHIP IS COMPUTED INTERNALLY IN THE HEC-1 PROGRAM, BASED ON THE RESERVOIR SURFACE AREA DATA PROVIDED ON SHEET 24.

SUBJECT	DAM SAFETY INSPECTION
	RICKARDS DAM
BY DJS	DATE 4-22-81	PROJ. NO. 80-238-405
CHKD. BY DLB	DATE 5-7-81	SHEET NO. 24 OF 29

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FAWN LAKE DAM:

RESERVOIR ELEVATION (FT)	SURFACE AREA (AC)	
978.0	0	
997.0	7	
999.7	10.6	
1000.0	11	(SEE NOTE 4)
1020.0	20	

FACILITY RATING CURVE:

RESERVOIR ELEVATION (FT)	OUTFLOW (CFS)	
997.0	0	
997.7	30	(SEE NOTE 4)
998.3	90	
999.0	200	
999.6	370	
999.7	390	
999.9	470	
1000.1	580	
1000.2	660	
1000.4	930	
1000.7	1630	
1001.0	2610	
1001.5	4640	
1002.0	7210	
1003.0	13,000	

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DSS DATE 4-22-81 PROJ. NO. 80-238-405
CHKD. BY DLB DATE 4-22-81 SHEET NO. 25 OF 29

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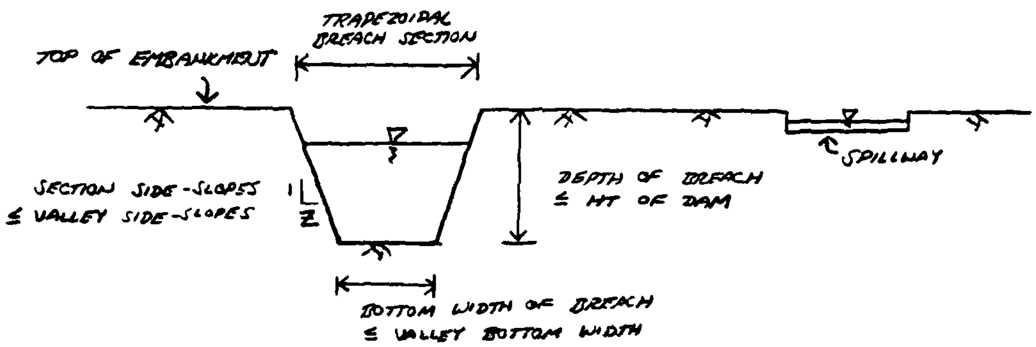
BREACH ASSUMPTIONS

TYPICAL BREACH SECTION:



HEC-1 DAM BREACHING ANALYSIS INPUT:

PLAN	BREACH BOTTOM WIDTH (FT)	MAX. BREACH DEPTH (FT)	SECTION SIDE-SLOPES	BREACH TIME (HRS)	ELEVATION AT WHICH FAILURE COMMENCES (FT)
① MIN. BREACH SECTION, MIN. FAIL TIME	10	9.2	1H : 1V	0.5	1079.1
② MAX. BREACH SECTION, MIN. FAIL TIME	150	9.2	23 : 1	0.5	1079.1
③ MIN. BREACH SECTION, MAX. FAIL TIME	10	9.2	1 : 1	3.0	1079.1
④ MAX. BREACH SECTION, MAX. FAIL TIME	150	9.2	23 : 1	3.0	1079.1
⑤ AVERAGE POSSIBLE CONDITIONS	25	9.2	1 : 1	1.0	1079.1
⑥ AVERAGE POSSIBLE CONDITIONS – DIKE ONLY	15	4.8	2 : 1	1.0	1078.9

FOR PLANS ①-⑤ THE BREACHING IS ASSUMED TO COMMENCE WHEN THE RESERVOIR LEVEL REACHES THE ELEVATION OF THE LOW AREA ALONG THE MAIN EMBANKMENT CREST, TO ENSURE OVERTOPPING OF BOTH THE MAIN EMBANKMENT AND THE DIKE. FOR PLAN ⑥, BREACHING COMMENCES WHEN LOW AREA ALONG THE DIKE IS OVERTOPPED.

THE BREACH ASSUMPTIONS LISTED ON THE PREVIOUS SHEET ARE BASED ON THE SUGGESTED RANGES PROVIDED BY THE COE, (BALTIMORE DISTRICT), AND ON THE PHYSICAL CONSTRAINTS OF THE DAM AND SURROUNDING TERRAIN.

- DEPTH OF BREACH:
  - = 9.2 FT FOR PLANS ① - ⑤ (TOP OF DAM TO DOWNSTREAM EMBANKMENT TOE)
  - = 4.8 FT FOR PLAN ⑥ (TOP OF DIKE TO DOWNSTREAM TOE)

  (FIELD SURVEY)

- TOTAL LENGTH OF BREACHABLE EMBANKMENT =

  300 FT (MAIN EMBANKMENT) + 270 FT (DIKE) = 570 FT

  (FIELD SURVEY)

- VALLEY BOTTOM WIDTH:

  MAIN EMBANKMENT = 150 FT

  DIKE = 150 FT

  (FIELD NOTES)

(FOR THE MAXIMUM BREACH SECTION, IT IS ASSUMED THAT BOTH THE MAIN EMBANKMENT AND THE DIKE FAIL SIMULTANEOUSLY. THUS, IT IS MODELED AS A SINGLE SECTION, WITH A BOTTOM WIDTH OF 150 FT, AND A TOP WIDTH OF 570 FT.)

SUBJECT: DAM SAFETY INSPECTION
RICKARDS DAM

BY: DJS DATE: 4-27-81 PROJ. NO.: 80-238-405

CHKD. BY: DLB DATE: 5-7-81 SHEET NO. 27 OF 29

GAI CONSULTANTS, INC.
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HEC-1 DAM BREACHING ANALYSIS OUTPUT SUMMARY:

RESERVOIR DATA: (UNDER 0.32 PMF BASE FLOW CONDITIONS)

PLAN NUMBER *	VARIABLE BREACH BOTTOM WIDTH (FT)	ACTUAL MAX. FLOW DURING FAIL TIME (CFS)	CORRESPONDING TIME OF PEAK (HRS)	INTERPOLATED OR HEC-1 ROUTED MAX. FLOW DURING FAIL TIME (CFS)	CORRESPONDING TIME OF PEAK (HRS)	ACTUAL PEAK FLOW THROUGH DAM (CFS)	CORRESPONDING TIME OF PEAK (HRS)	TIME OF INITIAL BREACH (HRS)
1	10	1993	41.83	1993	41.83	1993	41.83	41.33
2	150	7269	41.71	6804	41.67	7269	41.71	41.33
3	10	1081	44.33	1081	44.33	1081	44.33	41.33
4	150	2042	42.72	2033	42.67	2042	42.72	41.33
5	25	2377	42.33	2377	42.33	2377	42.33	41.33
6	15	1326	42.00	1326	42.00	1326	42.00	41.00

* - FROM SHEET 25.

(THE NON-BREACH 0.32 PMF PEAK OUTFLOW ≅ 823 CFS; SEE HEC-1 SUMMARY INPUT/OUTPUT SHEETS)

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 4-27-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 28 OF 29

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. .TREAM ROUTING DATA: (UNDER 0.32 PMF BASE FLOW CONDITIONS)

. . RICKARDS DAM:

	MAXIMUM INFLOW: LOWER RICKARDS DAM RESERVOIR (CFS)	MAXIMUM OUTFLOW: LOWER RICKARDS DAM RESERVOIR (CFS)	MAXIMUM WATER SURFACE ELEVATION (FT)	MAXIMUM DEPTH ABOVE TOP OF DAM (EL. 1071.7) (FT)
	1993	1859	1072.7	1.0
2	6804	6201	1074.0	2.3
3	1081	1068	1072.3	0.6
4	2033	2002	1072.7	1.0
5	2377	2224	1072.8	1.1
6	1326	1273	1072.4	0.7
NON-BREACH	821	820	1072.2	0.5

SECTION 1; 1080 FT D.S. FROM LOWER RICKARDS DAM:

BREACH* PLAN	PEAK FLOW (CFS)	CORRESPONDING WATER SURFACE ELEVATION (FT)	WATER SURFACE ELEVATION W/O BREACH (FT)	ELEVATION DIFFERENCE (FT)	APPROXIMATE DAMAGE LEVEL OF DWELLINGS (FT)
1	1821	1029.4	1028.6	+0.8	1028
2	6341	1031.6	1028.6	+3.0	1028
3	1057	1028.9	1028.6	+0.3	1028
4	1978	1029.5	1028.6	+0.9	1028
5	2241	1029.7	1028.6	+1.1	1028
6	1260	1029.1	1028.6	+0.5	1028

* - FROM SHEET 25.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 4-27-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-7-81 SHEET NO. 29 OF 29

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DOWNSTREAM ROUTING DATA: (UNDER 0.32 PMF BASE FLOW CONDITIONS)

LITTLE FAWN LAKE DAM:

BREACH* PLAN	MAXIMUM INFLOW (CFS)	MAXIMUM OUTFLOW (CFS)	MAXIMUM WATER SURFACE ELEVATION (FT)	MAXIMUM DEPTH ABOVE TOP OF DAM (EL. 1012.4) (FT)
1	1821	1794	1014.3	1.9
2	6341	6279	1016.1	3.7
3	1057	1055	1013.9	1.5
4	1978	1982	1014.4	2.0
5	2241	2239	1014.6	2.2
6	1260	1249	1014.0	1.6
NON-BREACH	820	819	1013.7	1.3

FAWN LAKE DAM:

BREACH* PLAN	MAXIMUM INFLOW (CFS)	MAXIMUM OUTFLOW (CFS)	MAXIMUM WATER SURFACE ELEVATION (FT)	MAXIMUM DEPTH ABOVE TOP OF DAM (EL. 999.7) (FT)
1	1794	1833	1000.8	1.1
2	6279	5959	1001.8	2.1
3	1055	1055	1000.5	0.8
4	1982	1981	1000.8	1.1
5	2239	2205	1000.9	1.2
6	1249	1250	1000.5	0.8
NON-BREACH	819	818	1000.3	0.6

* - FROM SHEET 25.

SUBJECT: DAM SAFETY INSPECTION
RICKARDS DAM

BY: DJS | DATE: 5-8-81 | PROJ. NO.: 80-238-405
CHKD. BY: DLB | DATE: 5-12-81 | SHEET NO.: A OF X

GAI CONSULTANTS, INC.
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SUMMARY INPUT/OUTPUT SHEETS

OVERTOPPING ANALYSIS

```
DAM SAFETY INSPECTION
RICKARDS DAM, W/U.S. LONG RIDGE DAM *** OVERTOPPING ANALYSIS ***
10-MINUTE TIME STEP AND 48-HOUR STORM DURATION

                           JOB SPECIFICATION
   NQ     NHR    NMIN    IDAY     IHR    IMIN   METRC    IPLT    IPRT   NSTAN
  300       0      10       0       0       0       0       0       0       0
                        JOPER     NWT   LROPT   TRACE
                            5       0       0       0

                     MULTI-PLAN ANALYSES TO BE PERFORMED
                        NPLAN= 1 NRTIO= 5 LRTIO= 1
   RTIOS=    .20     .30     .40     .50    1.00

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

                  SUB-AREA RUNOFF COMPUTATION

   INFLOW HYDROGRAPHS- LONG RIDGE RESERVOIR

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

                              HYDROGRAPH DATA
   IHYDG   IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
       1      1     .10    0.00    1.20    0.00   0.000       0       1       0

                               PRECIP DATA
          SPFE     PMS      R6     R12     R24     R48     R72     R96
          0.00   22.00  111.00  123.00  133.00  142.00    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=   .48     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= 3.15 AND R= 4.32 INTERVALS

        UNIT HYDROGRAPH 25 END-OF-PERIOD ORDINATES, LAG=    .48 HOURS, CP=  .45   VOL= 1.00
     10.      36.      58.      58.      47.      37.      29.      23.      18.      15.
     12.       9.       7.       6.       5.       4.       3.       2.       2.       1.
      1.       1.       1.       1.       0.
```

INITIAL + CONSTANT RAINFALL LOSSES AS PER C.O.E.

BASE FLOW PARAMETERS AS PER C.O.E.

	RAIN	EXCS	LOSS	COMP Q
SUM	24.99	22.60	2.39	8749.
	(635.)	(574.)	(61.)	(247.74)

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-8-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. 13 OF X

GAI CONSULTANTS, INC.

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LONG RIDGE DAM-INFLOW

0.20 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	85.	39.	12.	6.	1753.
CMS	2.	1.	0.	0.	50.
INCHES		3.63	4.43	4.53	4.53
MM		92.20	112.64	115.08	115.08
AC-FT		19.	24.	24.	24.
THOUS CU M		24.	29.	30.	30.

0.30 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	128.	59.	18.	9.	2630.
CMS	4.	2.	1.	0.	74.
INCHES		5.45	6.65	6.80	6.80
MM		138.43	168.96	172.61	172.61
AC-FT		29.	35.	36.	36.
THOUS CU M		36.	44.	45.	45.

0.50 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	213.	98.	30.	15.	4383.
CMS	6.	3.	1.	0.	124.
INCHES		9.08	11.09	11.33	11.33
MM		230.71	281.60	287.69	287.69
AC-FT		48.	59.	60.	60.
THOUS CU M		60.	73.	74.	74.

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	427.	195.	60.	29.	8767.
CMS	12.	6.	2.	1.	248.
INCHES		18.17	22.17	22.65	22.65
MM		461.42	563.20	575.38	575.38
AC-FT		97.	118.	121.	121.
THOUS CU M		119.	146.	149.	149.

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

HYDROGRAPH ROUTING

ROUTE THROUGH LONG RIDGE RESERVOIR

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
GRD	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	-1188.	-1

STAGE	1188.00	1188.70	1189.40	1190.00	1190.10	1190.20	1190.30	1190.50	1190.70
	1191.30	1191.60	1192.00						
FLOW	0.00	20.00	80.00	170.00	190.00	210.00	240.00	330.00	470.00
	1050.00	1500.00	2160.00						

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJJ DATE 5-9-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. C OF X

GAI CONSULTANTS, INC.
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SURFACE AREA=	0.	2.	9.	11.	18.
CAPACITY=	0.	1.	42.	63.	203.
ELEVATION=	1178.	1180.	1188.	1190.	1200.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
1188.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1190.1	0.0	0.0	0.

LONG RIDGE
DAM-OUTFLOW

0.20PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	46.	30.	11.	5.	1626.
CMS	1.	1.	0.	0.	46.
INCHES		2.83	4.13	4.20	4.20
MM		71.94	104.83	106.72	106.72
AC-FT		15.	22.	22.	22.
THOUS CU M		19.	27.	28.	28.

0.30PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	75.	49.	17.	8.	2473.
CMS	2.	1.	0.	0.	70.
INCHES		4.56	6.28	6.39	6.39
MM		115.83	159.48	162.32	162.32
AC-FT		24.	33.	34.	34.
THOUS CU M		30.	41.	42.	42.

0.50PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	146.	87.	29.	14.	4186.
CMS	4.	2.	1.	0.	119.
INCHES		8.12	10.63	10.82	10.82
MM		206.37	270.04	274.77	274.77
AC-FT		43.	57.	58.	58.
THOUS CU M		53.	70.	71.	71.

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	375.	183.	58.	28.	8486.
CMS	11.	5.	2.	1.	240.
INCHES		17.06	21.56	21.93	21.93
MM		433.44	547.51	556.96	556.96
AC-FT		91.	115.	117.	117.
THOUS CU M		112.	142.	144.	144.

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

SUB-AREA RUNOFF COMPUTATION

LOCAL INFLOW- RICKARDS DAM RESERVOIR

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

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DDS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. D OF X

GAI CONSULTANTS, INC.
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HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	1.10	0.00	1.20	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	22.00	111.00	123.00	133.00	142.00	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= 1.30 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= 8.13 AND R=12.53 INTERVALS

UNIT HYDROGRAPH 71 END-OF-PERIOD ORDINATES, LAG= 1.30 HOURS, CP= .45 VOL= 1.00

10.	37.	76.	122.	169.	209.	236.	248.	241.	223.
206.	190.	176.	162.	150.	138.	128.	118.	109.	100.
93.	86.	79.	73.	67.	62.	57.	53.	49.	45.
42.	39.	36.	33.	30.	28.	26.	24.	22.	20.
19.	17.	16.	15.	14.	13.	12.	11.	10.	9.
8.	8.	7.	7.	6.	6.	5.	5.	4.	4.
4.	4.	3.	3.	3.	3.	2.	2.	2.	2.
2.									

	RAIN	EXCS	LOSS	COMP Q
SUM	24.99	22.60	2.39	94837.
	(635.)(	574.)(	61.)(	2685.48)

LOCAL INFLOW-
RICKARDS DAM.

0.20 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	551.	371.	129.	63.	18957.
CMS	16.	10.	4.	2.	537.
INCHES		3.14	4.37	4.45	4.45
MM		79.64	110.99	113.11	113.11
AC-FT		184.	256.	261.	261.
THOUS CU M		227.	316.	322.	322.

0.30 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	826.	556.	194.	95.	28435.
CMS	23.	16.	5.	3.	805.
INCHES		4.70	6.55	6.68	6.68
MM		119.46	166.49	169.66	169.66
AC-FT		276.	384.	392.	392.
THOUS CU M		340.	474.	483.	483.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. E OF X

GAI CONSULTANTS, INC.
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0.50PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1377.	927.	323.	158.	47392.
CMS	39.	26.	9.	4.	1342.
INCHES		7.84	10.92	11.13	11.13
MM		199.10	277.48	282.77	282.77
AC-FT		460.	641.	653.	653.
THOUS CU M		567.	790.	805.	805.

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2754.	1854.	646.	316.	94784.
CMS	78.	52.	18.	9.	2684.
INCHES		15.68	21.85	22.27	22.27
MM		398.20	554.97	565.54	565.54
AC-FT		919.	1281.	1306.	1306.
THOUS CU M		1134.	1580.	1610.	1610.

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

COMBINE HYDROGRAPHS

COMBINE LONG RIDGE DAM OUTFLOW WITH RICKARDS DAM RESERVOIR LOCAL INFLOW

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

SUM OF RICKARDS DAM LOCAL INFLOW AND LONG RIDGE DAM OUTFLOW.

0.20PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	596.	401.	140.	69.	20583.
CMS	17.	11.	4.	2.	583.
INCHES		3.11	4.35	4.43	4.43
MM		78.90	110.48	112.58	112.58
AC-FT		199.	278.	284.	284.
THOUS CU M		245.	343.	350.	350.

0.30PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	901.	605.	211.	103.	30908.
CMS	26.	17.	6.	3.	875.
INCHES		4.69	6.53	6.66	6.66
MM		119.12	165.91	169.05	169.05
AC-FT		300.	418.	426.	426.
THOUS CU M		370.	515.	525.	525.

0.50 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1523.	1014.	352.	172.	51578.
CMS	43.	29.	10.	5.	1461.
INCHES		7.86	10.90	11.11	11.11
MM		199.71	276.86	282.10	282.10
AC-FT		503.	697.	710.	710.
THOUS CU M		620.	860.	876.	876.

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3072.	2035.	704.	344.	103270.
CMS	87.	58.	20.	10.	2924.
INCHES		15.78	21.82	22.24	22.24
MM		400.78	554.35	564.83	564.83
AC-FT		1009.	1396.	1422.	1422.
THOUS CU M		1245.	1722.	1755.	1755.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. F OF X

GAI CONSULTANTS, INC
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Environmental Specialists

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

                    HYDROGRAPH ROUTING

    ROUTE TOTAL HYDROGRAPH THROUGH RICKARDS DAM

              ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
              RD          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  -1077.      -1

STAGE     1077.00   1078.00   1079.00   1079.10   1079.40   1079.50   1079.80   1080.00   1080.20
          1080.70   1081.00   1081.50   1082.00   1083.00   1084.00   1085.00

FLOW         0.00    220.00    660.00    720.00   1010.00   1190.00   1750.00   2240.00   2810.00
          4620.00   5640.00   7930.00  10590.00  17090.00  24290.00  33030.00

CAPACITY=      0.        7.       29.       56.       98.      187.      242.      312.      386.      464.
             546.      632.

ELEVATION=  1069.     1071.     1073.     1075.     1077.     1079.     1080.     1081.     1082.     1083.
            1084.     1085.

              CREL   SPWID   COQW   EXPW   ELEVL   COQL   CAREA   EXPL
            1077.0     0.0    0.0    0.0     0.0    0.0     0.0    0.0

                                DAM DATA
                        TOPEL    COQD    EXPD  DAMWID
                       1079.1     0.0     0.0      0.
```

RICKARDS DAM – OUTFLOW.

0.20PMF

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                PEAK   6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
CFS             482.     362.      133.       65.         19491.
CMS              14.      10.        4.        2.           552.
INCHES                   2.81      4.12      4.20           4.20
MM                      71.37    104.77    106.61         106.61
AC-FT                    180.      264.      268.           268.
THOUS CU M               222.      325.      331.           331.
```

0.30 PMF

```
                PEAK   6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
CFS             762.     565.      201.       98.         29430.
CMS              22.      16.        6.        3.           833.
INCHES                   4.38      6.23      6.34           6.34
MM                     111.26    158.21    160.97         160.97
AC-FT                    280.      398.      405.           405.
THOUS CU M               346.      491.      500.           500.
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SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-13-81 SHEET NO. G OF X

GAI CONSULTANTS, INC.

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0.50 PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1411.	969.	338.	165.	49531.
CMS	40.	27.	10.	5.	1403.
INCHES		7.51	10.48	10.67	10.67
MM		190.84	266.31	270.91	270.91
AC-FT		481.	671.	682.	682.
THOUS CU M		593.	827.	842.	842.

PMF

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2977.	2001.	685.	334.	100279.
CMS	84.	57.	19.	9.	2840.
INCHES		15.51	21.23	21.59	21.59
MM		394.01	539.28	548.47	548.47
AC-FT		992.	1358.	1381.	1381.
THOUS CU M		1224.	1675.	1704.	1704.

STAGE

1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2
1077.3	1077.3	1077.3	1077.3	1077.3	1077.3	1077.3	1077.3	1077.3	1077.3
1077.4	1077.4	1077.4	1077.4	1077.4	1077.4	1077.4	1077.4	1077.4	1077.4
1077.5	1077.5	1077.5	1077.6	1077.6	1077.7	1077.7	1077.8	1077.9	1078.0
1078.1	1078.1	1078.2	1078.3	1078.4	1078.5	1078.6	1078.7	1078.8	1078.9
1079.0	1079.2	1079.3	1079.4	1079.5	1079.6	1079.6	1079.6	1079.6	1079.6
1079.6	1079.6	1079.6	1079.6	1079.5	1079.5	1079.5	1079.5	1079.4	1079.4
1079.4	1079.3	1079.3	1079.2	1079.2	1079.1	1079.1	1079.0	1079.0	1078.9
1078.8	1078.8	1078.7	1078.6	1078.6	1078.5	1078.5	1078.4	1078.4	1078.3
1078.3	1078.2	1078.2	1078.1	1078.1	1078.1	1078.0	1078.0	1078.0	1078.0
1077.9	1077.9	1077.9	1077.8	1077.8	1077.8	1077.8	1077.7	1077.7	1077.7

STAGE HYDROGRAPH FOR ½ PMF EVENT; DIKE (EL. 1078.9) IS OVERTOPPED FOR ≈ 5.2 HOURS.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJJ DATE 5-11-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-13-81 SHEET NO. H OF X

CONSULTANTS, INC.
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STAGE HYDROGRAPH FOR PMF EVENT; DIKE (EL. 1078.9) IS OVERTOPPED FOR ≈ 8.3 HOURS.

1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0	1077.0
1077.0	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.2	1077.2
1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2
1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2
1077.2	1077.2	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1	1077.1
1077.1	1077.1	1077.1	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2
1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.2	1077.3
1077.3	1077.3	1077.3	1077.3	1077.4	1077.4	1077.4	1077.4	1077.5	1077.5
1077.5	1077.5	1077.5	1077.6	1077.6	1077.6	1077.6	1077.7	1077.7	1077.7
1077.7	1077.7	1077.7	1077.8	1077.8	1077.8	1077.8	1077.8	1077.8	1077.9
1077.9	1078.0	1078.0	1078.1	1078.2	1078.3	1078.4	1078.5	1078.7	1078.8
1078.9	1079.1	1079.2	1079.3	1079.4	1079.4	1079.5	1079.6	1079.7	1079.7
1079.9	1080.0	1080.1	1080.2	1080.2	1080.2	1080.3	1080.2	1080.2	1080.2
1080.2	1080.2	1080.1	1080.1	1080.1	1080.1	1080.0	1080.0	1079.9	1079.9
1079.8	1079.8	1079.7	1079.7	1079.6	1079.6	1079.5	1079.5	1079.4	1079.4
1079.3	1079.3	1079.2	1079.2	1079.2	1079.1	1079.1	1079.0	1078.9	1078.9
1078.8	1078.7	1078.7	1078.6	1078.6	1078.5	1078.5	1078.4	1078.4	1078.3
1078.3	1078.3	1078.2	1078.2	1078.1	1078.1	1078.1	1078.0	1078.0	1078.0

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SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1188.00	1188.00	1190.10
STORAGE	42.	42.	63.
OUTFLOW	0.	0.	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
.20	1189.01	0.00	51.	46.	0.00	41.17	0.00
.30	1189.35	0.00	55.	75.	0.00	41.00	0.00
.40	1189.61	0.00	57.	112.	0.00	40.83	0.00
.50	1189.84	0.00	60.	146.	0.00	40.83	0.00
1.00	1190.56	.46	68.	375.	2.50	40.33	0.00

LONG RIDGE DAM – OVERTOPS @ ≈ 0.60 PMF

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.20	1078.60	0.00	166.	482.	0.00	42.33	0.00
.30	1079.14	.04	190.	762.	1.33	42.17	0.00
.40	1079.44	.34	208.	1074.	3.33	41.83	0.00
.50	1079.62	.52	219.	1411.	4.17	41.50	0.00
1.00	1080.25	1.15	260.	2977.	7.33	41.17	0.00

RICKARDS DAM – OVERTOPS @ ≈ 0.29 PMF

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405
CHKD. BY DLB DATE 5-12-81 SHEET NO. I OF X

gai CONSULTANTS, INC.
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BREACHING ANALYSIS.

(INPUT SAME AS FOR OVERTOPPING ANALYSIS, WITH THE ADDITION OF BREACH CRITERIA GIVEN HERE.)

- UNDER 0.32 PMF CONDITIONS:

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DAM SAFETY INSPECTION
RICKARDS DAM *** BREACH ANALYSIS, W/U.S. AND D.S. FACILITIES INCLUDED ***
10-MINUTE TIME STEP AND 48-HOUR STORM DURATION

                              JOB SPECIFICATION
   NQ      NHR     NMIN    IDAY     IHR    IMIN   METRC    IPLT    IPRT   NSTAN
  300        0       10       0       0       0       0       0       0       0
                          JOPER     NWT   LROPT   TRACE
                              5       0       0       0

                    MULTI-PLAN ANALYSES TO BE PERFORMED
                       NPLAN= 7 NRTIO= 1 LRTIO= 1
   RTIOS=     .32

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

               HYDROGRAPH ROUTING

     ROUTE TOTAL HYDROGRAPH THROUGH RICKARDS DAM
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PLAN

(1)

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                              DAM BREACH DATA
          BRWID       Z     ELBM    TFAIL     WSEL   FAILEL
           10.     1.00  1069.90      .50  1077.00  1079.10

               STATION   RD   , PLAN 1  RATIO 1

   BEGIN DAM FAILURE AT 41.33 HOURS

   PEAK OUTFLOW IS     1993. AT TIME  41.83 HOURS
```

(2)

```
                              DAM BREACH DATA
          BRWID       Z     ELBM    TFAIL     WSEL   FAILEL
          150.    23.00  1069.90      .50  1077.00  1079.10

               STATION   RD   , PLAN 2  RATIO 1

   BEGIN DAM FAILURE AT 41.33 HOURS

   PEAK OUTFLOW IS     7269. AT TIME  41.71 HOURS
```

(3)

```
                              DAM BREACH DATA
          BRWID       Z     ELBM    TFAIL     WSEL   FAILEL
           10.     1.00  1069.90     3.00  1077.00  1079.10

               STATION   RD   , PLAN 3  RATIO 1

   BEGIN DAM FAILURE AT 41.33 HOURS

   PEAK OUTFLOW IS     1081. AT TIME  44.33 HOURS
```

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405
CHKD. BY DLB DATE 5-12-81 SHEET NO. J OF X

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PLAN

4

DAM BREACH DATA

BRWID	Z	ELBM	TFAIL	WSEL	FAILEL
150.	23.00	1069.90	3.00	1077.00	1079.10

STATION RD , PLAN 4 RATIO 1

BEGIN DAM FAILURE AT 41.33 HOURS

PEAK OUTFLOW IS 2042. AT TIME 42.72 HOURS

5

DAM BREACH DATA

BRWID	Z	ELBM	TFAIL	WSEL	FAILEL
25.	1.00	1069.90	1.00	1077.00	1079.10

STATION RD , PLAN 5 RATIO 1

BEGIN DAM FAILURE AT 41.33 HOURS

PEAK OUTFLOW IS 2377. AT TIME 42.33 HOURS

6

DAM BREACH DATA

BRWID	Z	ELBM	TFAIL	WSEL	FAILEL
15.	2.00	1074.10	1.00	1077.00	1078.90

STATION RD , PLAN 6 RATIO 1

BEGIN DAM FAILURE AT 41.00 HOURS

PEAK OUTFLOW IS 1326. AT TIME 42.00 HOURS

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. K OF X

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THE DAM BREACH HYDROGRAPH WAS DEVELOPED USING A TIME INTERVAL OF .010 HOURS DURING BREACH FORMATION.
DOWNSTREAM CALCULATIONS WILL USE A TIME INTERVAL OF .167 HOURS.
THIS TABLE COMPARES THE HYDROGRAPH FOR DOWNSTREAM CALCULATIONS WITH THE COMPUTED BREACH HYDROGRAPH.
INTERMEDIATE FLOWS ARE INTERPOLATED FROM END-OF-PERIOD VALUES.

PLAN 2

TIME (HOURS)	TIME FROM BEGINNING OF BREACH (HOURS)	INTERPOLATED BREACH HYDROGRAPH (CFS)	-	COMPUTED BREACH HYDROGRAPH (CFS)	=	ERROR (CFS)	ACCUMULATED ERROR (CFS)	ACCUMULATED ERROR (AC-FT)
41.333	0.000	755.		755.		0.	0.	0.
41.343	.010	916.		802.		114.	114.	0.
41.353	.020	1077.		876.		200.	315.	0.
41.363	.029	1238.		972.		266.	580.	0.
41.373	.039	1398.		1085.		313.	893.	1.
41.382	.049	1559.		1215.		345.	1238.	1.
41.392	.059	1720.		1358.		362.	1600.	1.
41.402	.069	1881.		1515.		366.	1966.	2.
41.412	.078	2042.		1683.		358.	2324.	2.
41.422	.088	2202.		1863.		340.	2664.	2.
41.431	.098	2363.		2049.		314.	2978.	2.
41.441	.108	2524.		2239.		285.	3263.	3.
41.451	.118	2685.		2435.		250.	3512.	3.
41.461	.127	2846.		2636.		210.	3722.	3.
41.471	.137	3007.		2846.		161.	3883.	3.
41.480	.147	3167.		3058.		109.	3992.	3.
41.490	.157	3328.		3273.		55.	4047.	3.
41.500	.167	3489.		3489.		-0.	4047.	3.
41.510	.176	3684.		3705.		-21.	4026.	3.
41.520	.186	3879.		3920.		-42.	3984.	3.
41.529	.196	4074.		4134.		-61.	3923.	3.
41.539	.206	4269.		4346.		-77.	3846.	3.
41.549	.216	4464.		4556.		-92.	3754.	3.
41.559	.225	4659.		4761.		-102.	3652.	3.
41.569	.235	4854.		4963.		-109.	3543.	3.
41.578	.245	5049.		5161.		-112.	3431.	3.
41.588	.255	5244.		5354.		-110.	3321.	3.
41.598	.265	5439.		5542.		-103.	3218.	3.
41.608	.275	5634.		5725.		-91.	3127.	3.
41.618	.284	5829.		5912.		-83.	3044.	2.
41.627	.294	6024.		6103.		-79.	2965.	2.
41.637	.304	6219.		6288.		-69.	2896.	2.
41.647	.314	6414.		6467.		-53.	2843.	2.
41.657	.324	6609.		6638.		-30.	2813.	2.
41.667	.333	6804.		6804.		-0.	2813.	2.
41.676	.343	6722.		6962.		-240.	2573.	2.
41.686	.353	6640.		7115.		-474.	2099.	2.
41.696	.363	6559.		7260.		-702.	1397.	1.
41.706	.373	6477.		7269.		-792.	605.	0.
41.716	.382	6395.		7088.		-693.	-88.	-0.
41.725	.392	6313.		6927.		-614.	-701.	-1.
41.735	.402	6232.		6784.		-553.	-1254.	-1.
41.745	.412	6150.		6658.		-508.	-1762.	-1.
41.755	.422	6068.		6545.		-477.	-2239.	-2.
41.765	.431	5986.		6444.		-458.	-2697.	-2.
41.775	.441	5905.		6354.		-450.	-3147.	-3.
41.784	.451	5823.		6273.		-451.	-3598.	-3.
41.794	.461	5741.		6117.		-376.	-3973.	-3.
41.804	.471	5659.		5901.		-241.	-4215.	-3.
41.814	.480	5578.		5714.		-137.	-4352.	-4.
41.824	.490	5496.		5554.		-58.	-4410.	-4.
41.833	.500	5414.		5414.		0.	-4410.	-4.

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. L OF X

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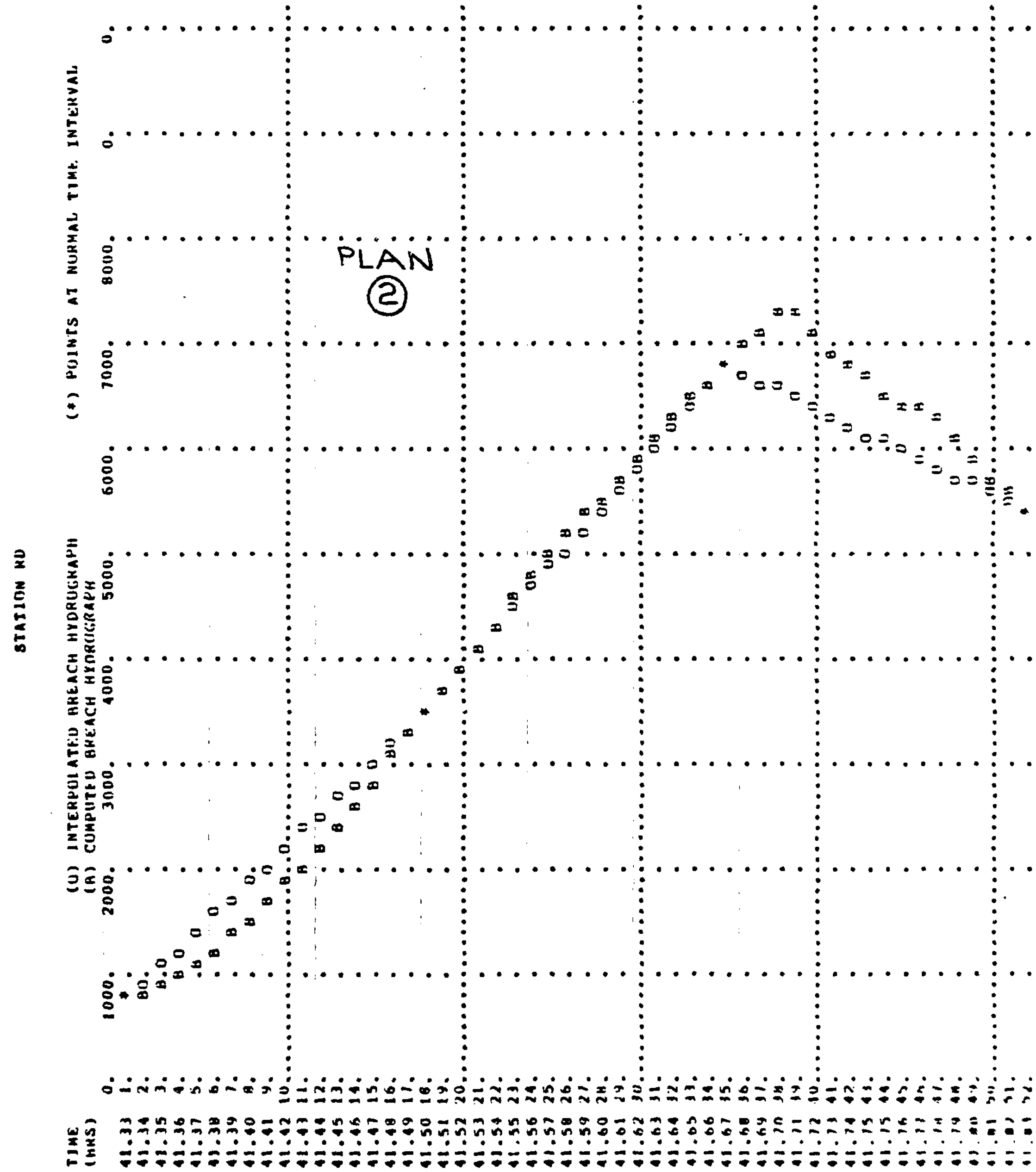

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NATIONAL DAM INSPECTION PROGRAM. RICKARDS DAM (NDI I.D. NUMBER --ETC(U)
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SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

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THE DAM BREACH HYDROGRAPH WAS DEVELOPED USING A TIME INTERVAL OF .021 HOURS DURING BREACH FORMATION.
DOWNSTREAM CALCULATIONS WILL USE A TIME INTERVAL OF .167 HOURS.
THIS TABLE COMPARES THE HYDROGRAPH FOR DOWNSTREAM CALCULATIONS WITH THE COMPUTED BREACH HYDROGRAPH.
INTERMEDIATE FLOWS ARE INTERPOLATED FROM END-OF-PERIOD VALUES.

PLAN 5

TIME (HOURS)	TIME FROM BEGINNING OF BREACH (HOURS)	INTERPOLATED BREACH HYDROGRAPH (CFS)	- COMPUTED BREACH HYDROGRAPH (CFS)	= ERROR (CFS)	ACCUMULATED ERROR (CFS)	ACCUMULATED ERROR (AC-FT)
41.333	0.000	753.	753.	0.	0.	0.
41.354	.021	775.	765.	10.	10.	0.
41.375	.042	797.	782.	15.	25.	0.
41.396	.063	820.	802.	17.	42.	0.
41.417	.083	842.	825.	17.	60.	0.
41.438	.104	864.	849.	15.	75.	0.
41.458	.125	887.	875.	11.	86.	0.
41.479	.146	909.	903.	6.	93.	0.
41.500	.167	931.	931.	0.	93.	0.
41.521	.188	964.	961.	3.	95.	0.
41.542	.208	996.	992.	4.	100.	0.
41.563	.229	1028.	1023.	5.	105.	0.
41.583	.250	1061.	1055.	5.	110.	0.
41.604	.271	1093.	1088.	5.	115.	0.
41.625	.292	1125.	1122.	4.	118.	0.
41.646	.313	1158.	1156.	2.	120.	0.
41.667	.333	1190.	1190.	0.	120.	0.
41.688	.354	1227.	1225.	2.	122.	0.
41.708	.375	1263.	1261.	3.	125.	0.
41.729	.396	1300.	1297.	3.	128.	0.
41.750	.417	1337.	1334.	3.	131.	0.
41.771	.437	1374.	1371.	2.	133.	0.
41.792	.458	1410.	1408.	2.	135.	0.
41.813	.479	1447.	1445.	2.	137.	0.
41.833	.500	1484.	1484.	0.	137.	0.
41.854	.521	1522.	1523.	-0.	137.	0.
41.875	.542	1561.	1562.	-1.	136.	0.
41.896	.562	1600.	1600.	-1.	135.	0.
41.917	.583	1638.	1639.	-1.	134.	0.
41.938	.604	1677.	1678.	-1.	133.	0.
41.958	.625	1716.	1716.	-1.	132.	0.
41.979	.646	1754.	1755.	-1.	132.	0.
42.000	.667	1793.	1793.	0.	132.	0.
42.021	.687	1830.	1831.	-1.	131.	0.
42.042	.708	1866.	1868.	-2.	128.	0.
42.063	.729	1903.	1906.	-3.	126.	0.
42.083	.750	1940.	1942.	-3.	123.	0.
42.104	.771	1976.	1979.	-3.	120.	0.
42.125	.792	2013.	2015.	-2.	118.	0.
42.146	.812	2050.	2051.	-1.	117.	0.
42.167	.833	2086.	2086.	0.	117.	0.
42.188	.854	2123.	2122.	1.	118.	0.
42.208	.875	2159.	2157.	2.	120.	0.
42.229	.896	2195.	2191.	4.	125.	0.
42.250	.917	2231.	2224.	7.	132.	0.
42.271	.937	2268.	2257.	10.	142.	0.
42.292	.958	2304.	2290.	14.	156.	0.
42.313	.979	2340.	2332.	9.	165.	0.
42.333	1.000	2377.	2377.	0.	165.	0.

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. N OF X

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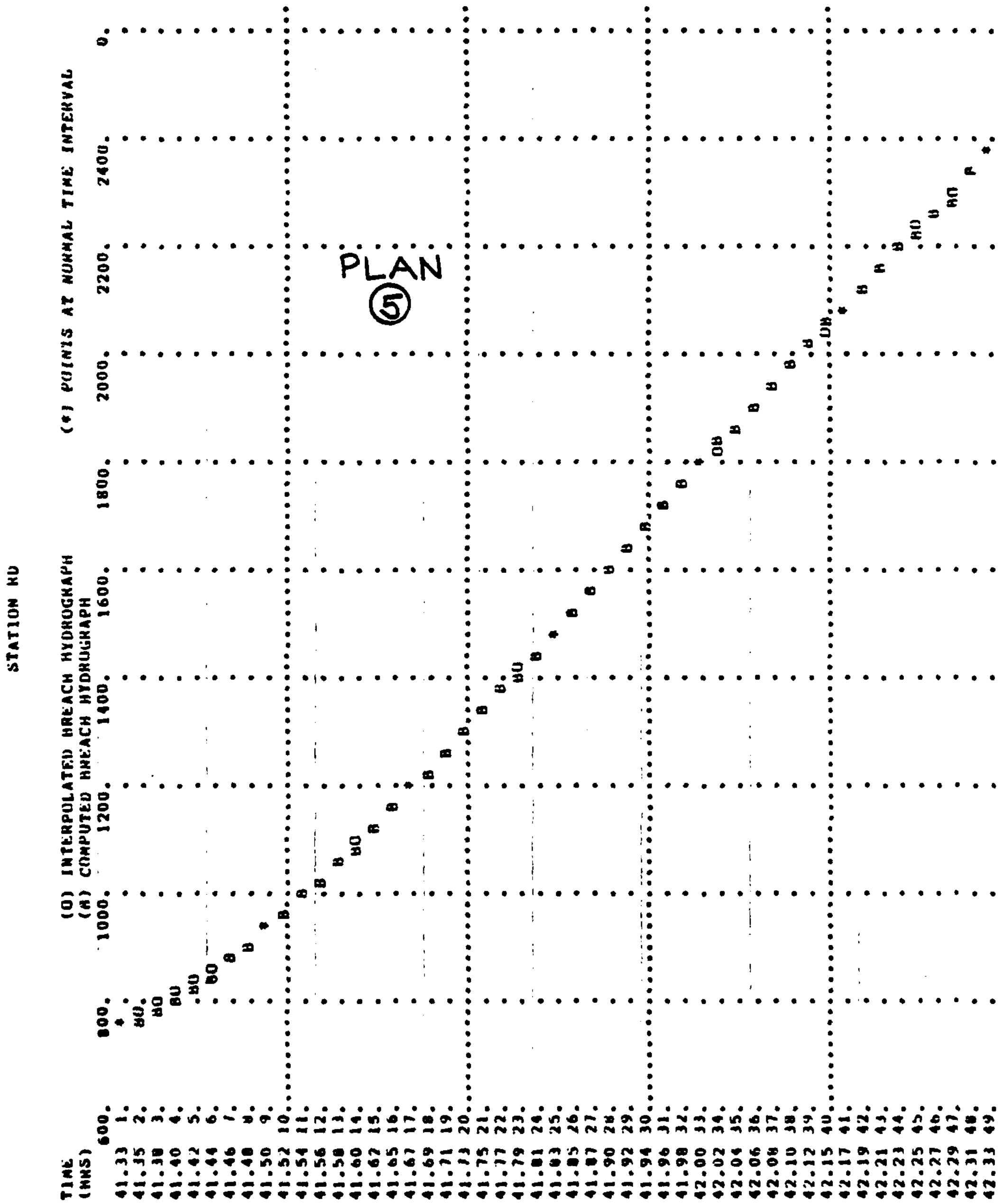

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405
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HYDROGRAPH ROUTING

ROUTE TOTAL HYDROGRAPH THROUGH LOWER RICKARDS LAKE DAM

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

ALL PLANS HAVE SAME
ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-1070.	0

SURFACE AREA=	0.	15.	17.	29.
CAPACITY=	0.	75.	103.	293.
ELEVATION=	1055.	1070.	1072.	1080.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
1070.0	35.0	2.7	1.5	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
1071.7	3.0	1.5	510.

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

HYDROGRAPH ROUTING

ROUTE FROM LOWER RICKARDS DAM TO SECTION 1, 1080 FT DOWNSTREAM

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
SEC1	1	0	0	0	0	1	0	0

ALL PLANS HAVE SAME
ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

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

NORMAL DEPTH CHANNEL ROUTING

QN(1)	QN(2)	QN(3)	ELNVT	ELMAX	RLNTH	SEL
.0800	.0450	.0800	1024.0	1040.0	1080.	.01600

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

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CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
      0.00 1040.00    30.00 1028.00   118.00 1028.00   120.00 1024.00   130.00 1024.00
    132.00 1028.00   300.00 1028.00   400.00 1040.00

STORAGE        0.00        .22        .45        .71        .98       2.61       8.39      14.36      20.52
              33.41      40.14      47.07      54.18      61.48      68.98      76.66      84.54      92.60

OUTFLOW        0.00      30.03      92.33     177.54     282.61     457.65    1245.34    2550.45    4285.78
            8904.71   11749.88   14936.93   18458.80   22310.63   26489.19   30992.44   35819.27   40969.30

STAGE       1024.00    1024.84    1025.68    1026.53    1027.37    1028.21    1029.05    1029.89    1030.74
            1032.42    1033.26    1034.11    1034.95    1035.79    1036.63    1037.47    1038.32    1039.16

FLOW           0.00      30.03      92.33     177.54     282.61     457.65    1245.34    2550.45    4285.78
            8904.71   11749.88   14936.93   18458.80   22310.63   26489.19   30992.44   35819.27   40969.30


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

          HYDROGRAPH ROUTING

    ROUTE TOTAL HYDROGRAPH THROUGH LITTLE FAWN LAKE DAM

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

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

                      NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                          1       0       0   0.000   0.000   0.000  -1010.      -1

STAGE      1010.00    1010.70    1011.40    1012.10    1012.40    1013.00    1014.00    1015.10    1016.00

FLOW          0.00      20.00      50.00     100.00     130.00     190.00     320.00     470.00     610.00

SURFACE AREA=      0.         3.         4.         7.

CAPACITY=          0.         6.        13.        50.

ELEVATION=      1003.      1010.      1012.      1020.

                 CREL   SPWID    COQW    EXPW   ELEVL    COQL   CAREA    EXPL
               1010.0     0.0     0.0     0.0     0.0     0.0     0.0     0.0

                                     DAM DATA
                             TOPEL    COQD    EXPD   DAMWID
                            1012.4     0.0     0.0       0.

CREST LENGTH       10.        50.        90.       210.       300.       350.       360.       370.       390.
AT OR BELOW
ELEVATION       1012.4     1012.7     1013.0     1013.2     1013.5     1014.0     1015.0     1016.0     1018.0
```

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405
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LONG RIDGE DAM

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HYDROGRAPH ROUTING

ROUTE TOTAL HYDROGRAPH THROUGH FAWN LAKE DAM

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

ALL PLANS HAVE SAME
ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP	LSTR
0.0	0.000	0.00	1	1	0	0	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.000	0.000	0.000	-997.	-1

STAGE	997.00	997.70	998.30	999.00	999.60	999.70	999.90	1000.10	1000.20
	1000.70	1001.00	1001.50	1002.00	1003.00				
FLOW	0.00	30.00	90.00	200.00	370.00	390.00	470.00	580.00	660.00
	1630.00	2610.00	4640.00	7210.00	13000.05				

SURFACE AREA	0.	7.	11.	11.	20.
CAPACITY=	0.	44.	68.	71.	377.
ELEVATION=	978.	997.	1000.	1000.	1020.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
997.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
999.7	0.0	0.0	0.

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SUMMARY OF DAM SAFETY ANALYSIS

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1188.00	1188.00	1190.10
STORAGE	42.	42.	63.
OUTFLOW	0.	0.	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
.32	1189.41	0.00	55.	82.	0.00	41.00	0.00

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY RJS DATE 5-11-81 PROJ. NO. 81-238-405

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RICKARDS DAM

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.15	.05	190.	1993.	.47	41.83	41.33

PLAN 2 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.14	.04	189.	7269.	.25	41.71	41.33

PLAN 3 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.18	.08	192.	1081.	1.22	44.33	41.33

PLAN 4 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.15	.05	190.	2042.	.44	42.72	41.33

PLAN 5 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.15	.05	190.	2377.	.52	42.33	41.33

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. 5 OF X

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PLAN 6 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.11	.01	188.	1326.	.33	42.00	41.00

PLAN 7 ....
(NON-BREACH)

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1077.00	1077.00	1079.10
STORAGE	98.	98.	187.
OUTFLOW	0.	0.	720.

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
.32	1079.20	.10	193.	821.	2.00	42.00	0.00

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

LOWER RICKARDS DAM

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1072.66	.96	120.	1859.	7.67	42.00	0.00

PLAN 2 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1074.03	2.33	146.	6201.	6.00	41.83	0.00

PLAN 3 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1072.31	.61	113.	1068.	8.00	44.33	0.00

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. 7 OF X

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PLAN 4 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1072.72	1.02	121.	2002.	6.00	42.67	0.00

PLAN 5 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1072.81	1.11	123.	2224.	6.67	42.33	0.00

PLAN 6 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1072.41	.71	115.	1273.	7.83	42.17	0.00

PLAN 7 ....
(NON-BREACH)

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1070.00	1070.00	1071.70
STORAGE	75.	75.	103.
OUTFLOW	0.	0.	209.

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
.32	1072.18	.48	111.	820.	7.67	42.17	0.00

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 81-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. U OF X

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SECTION 1

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

PLAN 1 STATION SEC1

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	1821.	1029.4	42.00

PLAN 2 STATION SEC1

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	6341.	1031.6	41.83

PLAN 3 STATION SEC1

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	1057.	1028.9	44.33

PLAN 4 STATION SEC1

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	1978.	1029.5	42.67

PLAN 5 STATION SEC1

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	2241.	1029.7	42.50

PLAN 6 STATION SEC1

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	1260.	1029.1	42.17

PLAN 7 STATION SEC1
(NON-BREACH)

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
.32	820.	1028.6	42.33

SUBJECT DAM SAFETY INSPECTION
RICKARDS DAM

BY DES DATE 5-11-81 PROJ. NO. 81-238-405
CHKD. BY DLB DATE 5-12-81 SHEET NO. V OF X

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LITTLE FAWN LAKE

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

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1014.32	1.92	20.	1794.	10.33	42.17	0.00

PLAN 2 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1016.12	3.72	28.	6279.	8.83	41.83	0.00

PLAN 3 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1013.88	1.48	19.	1055.	10.33	44.50	0.00

PLAN 4 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1014.42	2.02	21.	1982.	8.83	42.83	0.00

PLAN 5 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1014.55	2.15	21.	2239.	9.50	42.50	0.00

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 80-238-405

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FAWN LAKE DAM

PLAN 6 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1014.01	1.61	19.	1249.	10.33	42.17	0.00

PLAN 7 ....
(NON-BREACH)

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	1010.00	1010.00	1012.40
STORAGE	6.	6.	13.
OUTFLOW	0.	0.	130.

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
.32	1013.71	1.31	18.	819.	10.33	42.33	0.00

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

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

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
.32	1000.76	1.06	80.	1833.	5.33	42.17	0.00

PLAN 2 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

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
.32	1001.76	2.06	91.	5959.	3.83	41.83	0.00

PLAN 3 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

SUBJECT DAM SAFETY INSPECTION

RICKARDS DAM

BY DJS DATE 5-11-81 PROJ. NO. 80-238-405

CHKD. BY DLB DATE 5-12-81 SHEET NO. X OF X

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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
.32	1000.45	.75	76.	1055.	5.83	44.50	0.00

PLAN 4 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

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
.32	1000.81	1.11	80.	1981.	4.17	42.83	0.00

PLAN 5 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

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
.32	1000.88	1.18	81.	2205.	4.50	42.50	0.00

PLAN 6 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

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
.32	1000.54	.84	77.	1250.	5.00	42.33	0.00

PLAN 7 ....
(NON-BREACH)

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	997.00	997.00	999.70
STORAGE	44.	44.	68.
OUTFLOW	0.	0.	390.

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
.32	1000.32	.62	75.	818.	5.00	42.50	0.00

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APPENDIX E

FIGURES

LIST OF FIGURES

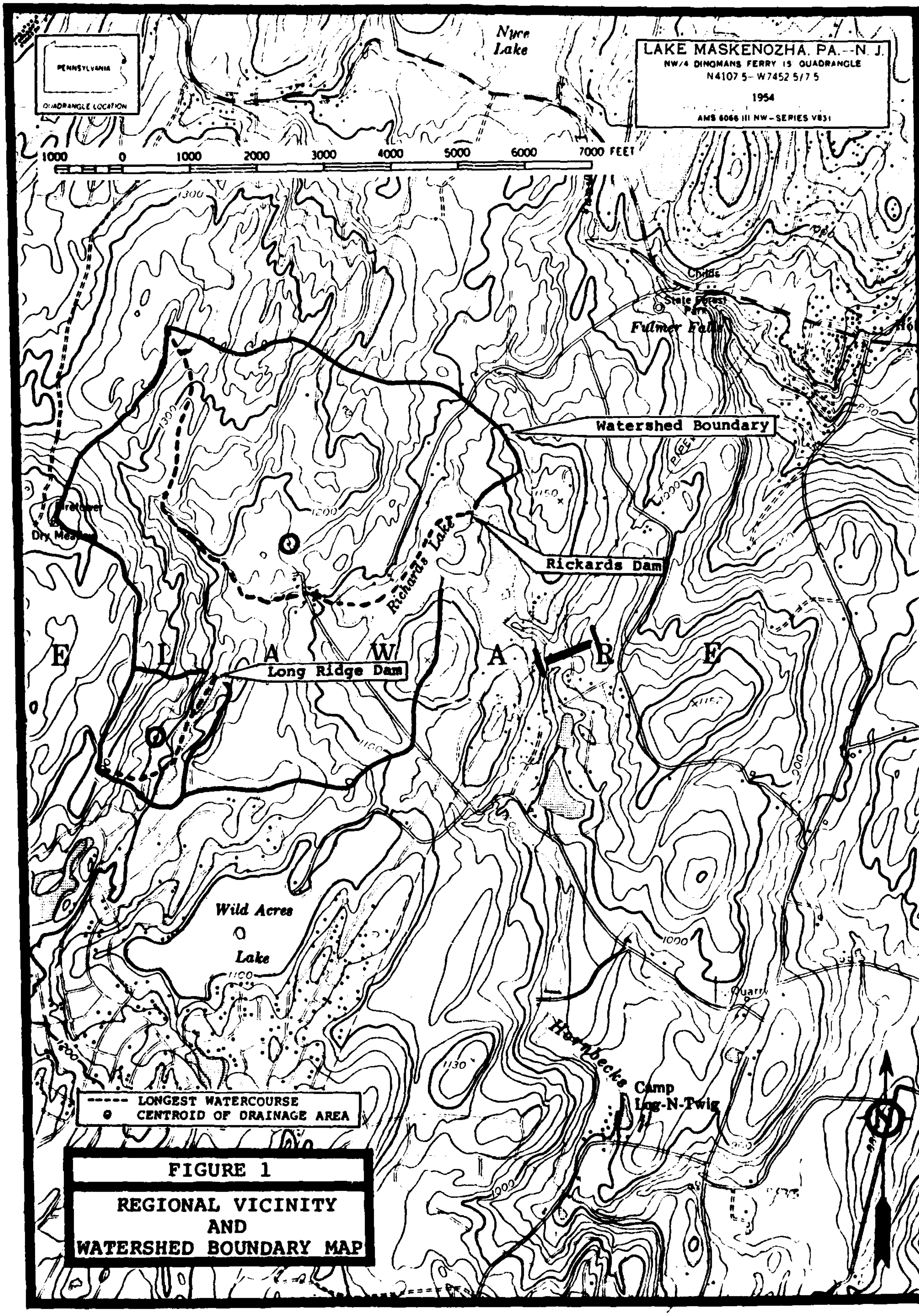
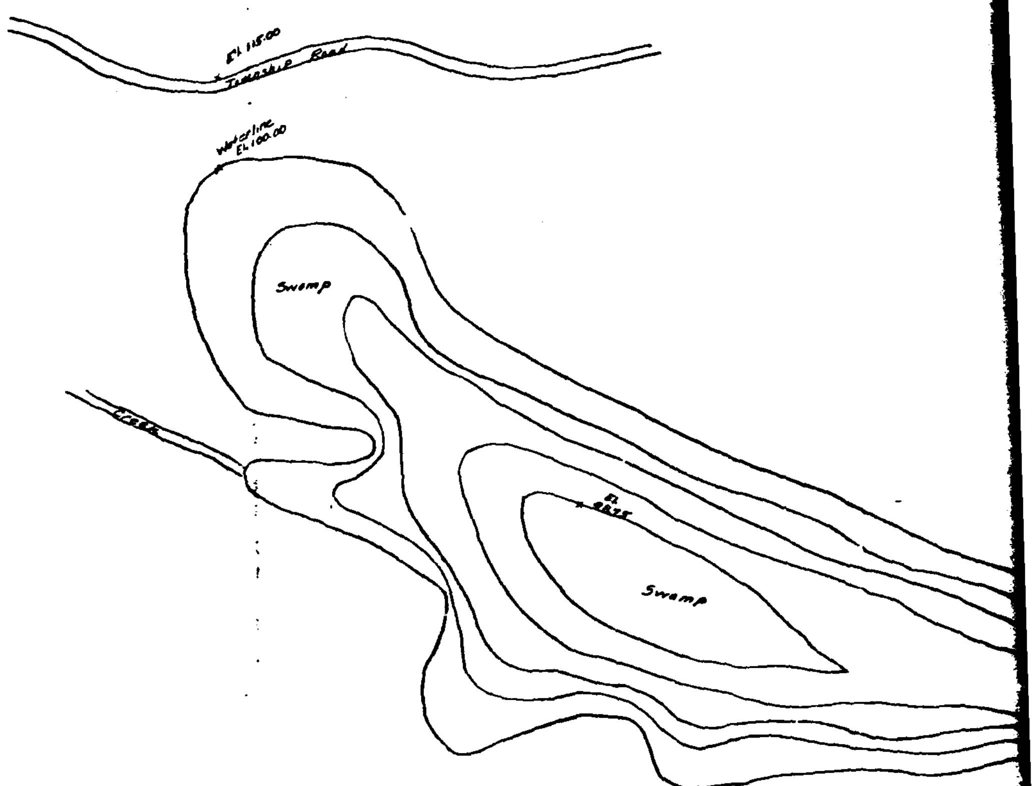


FIGURE 1

REGIONAL VICINITY AND WATERSHED BOUNDARY MAP



PLAN

STOLL JAGGE

G. AL

ST

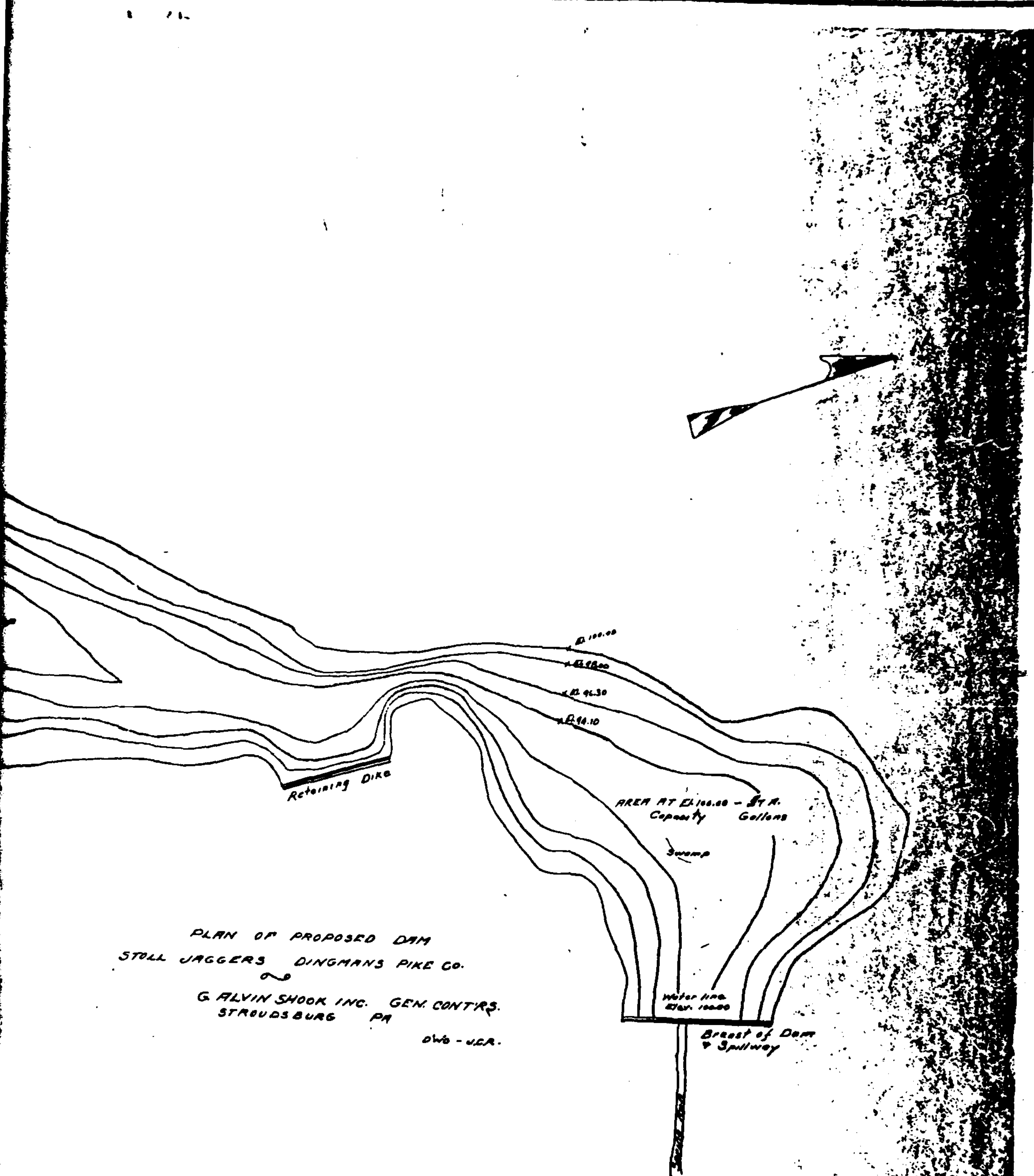


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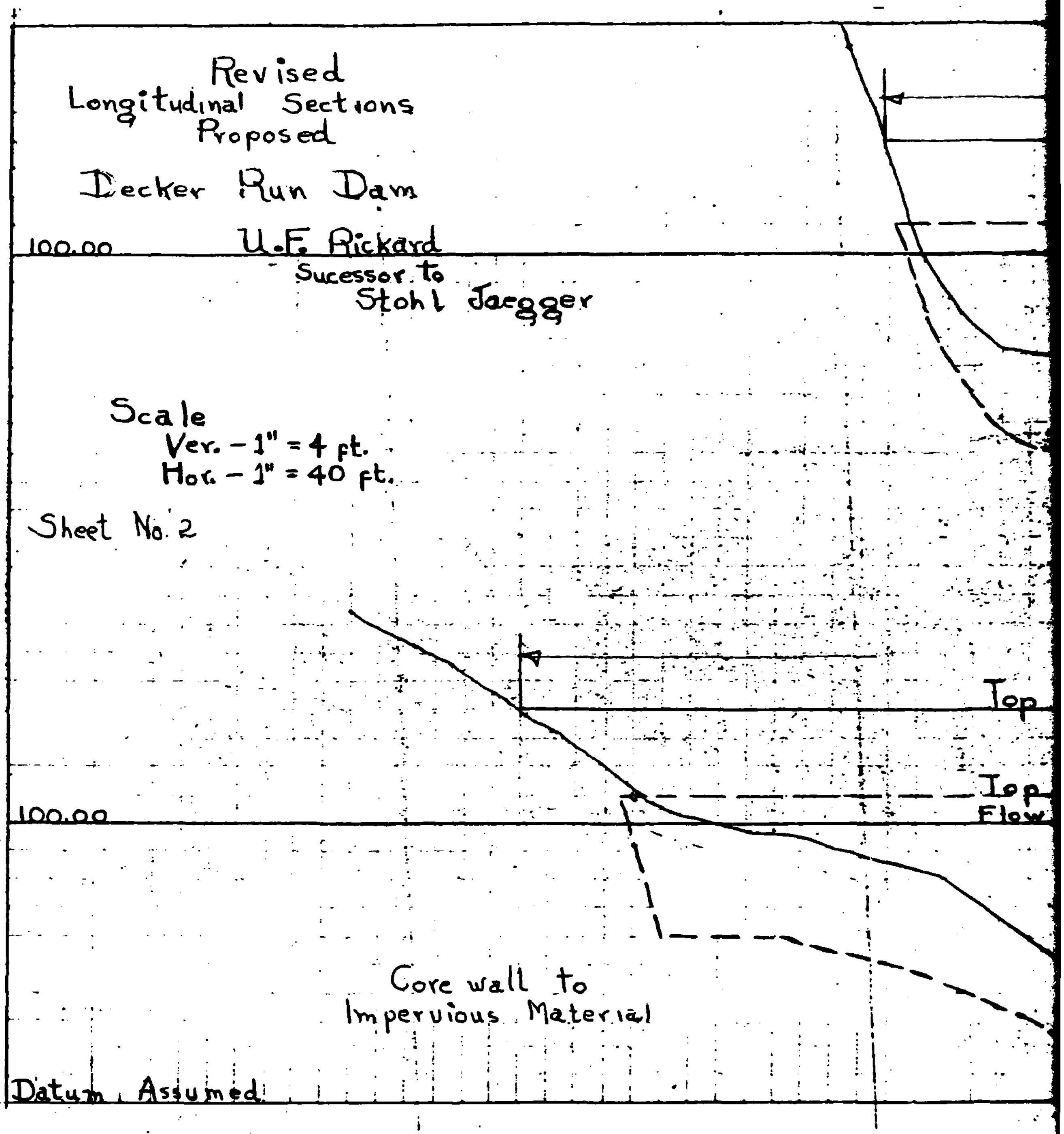
FIGURE 2

2


Revised
Longitudinal Sections
Proposed
Decker Run Dam
U.F. Rickard
Sucessor to
Stohl Jaegger
100.00
Scale
Ver. - 1" = 4 ft.
Hor. - 1" = 40 ft.
Sheet No. 2
Top
Top
Flow
100.00
Core wall to
Impervious Material
Datum Assumed

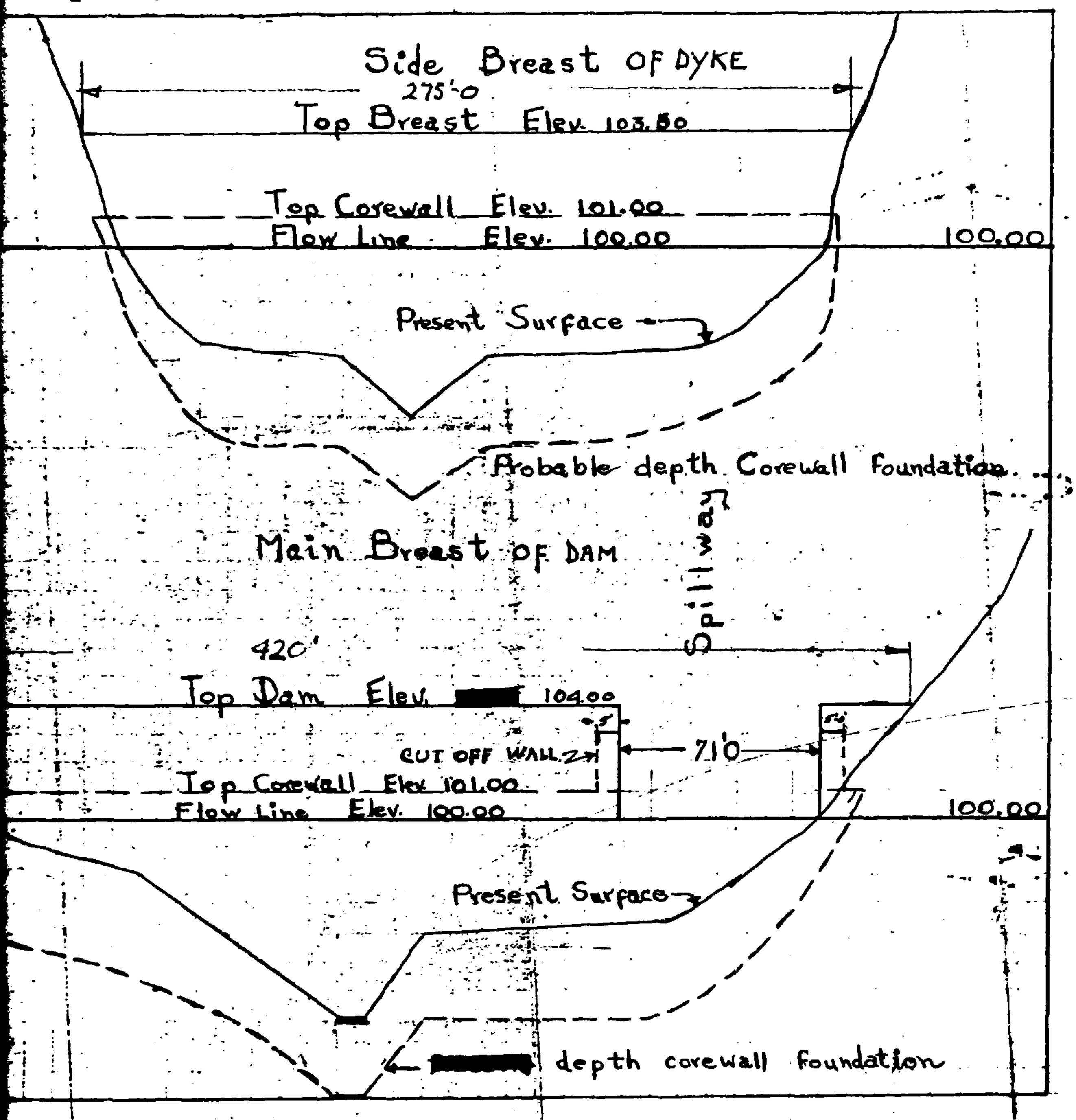


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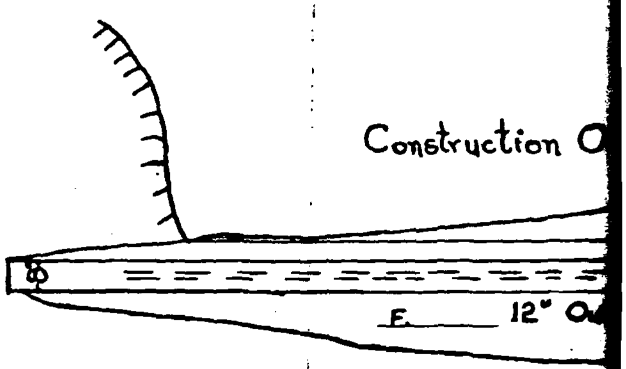
FIGURE 3



Revised General Plan
Proposed
Decker Run Dam
Scale 1" = 40 ft.

U. F. Rickard
566 Irvington Ave.
Elizabeth, N.J.
Contractor

Sheet No 3

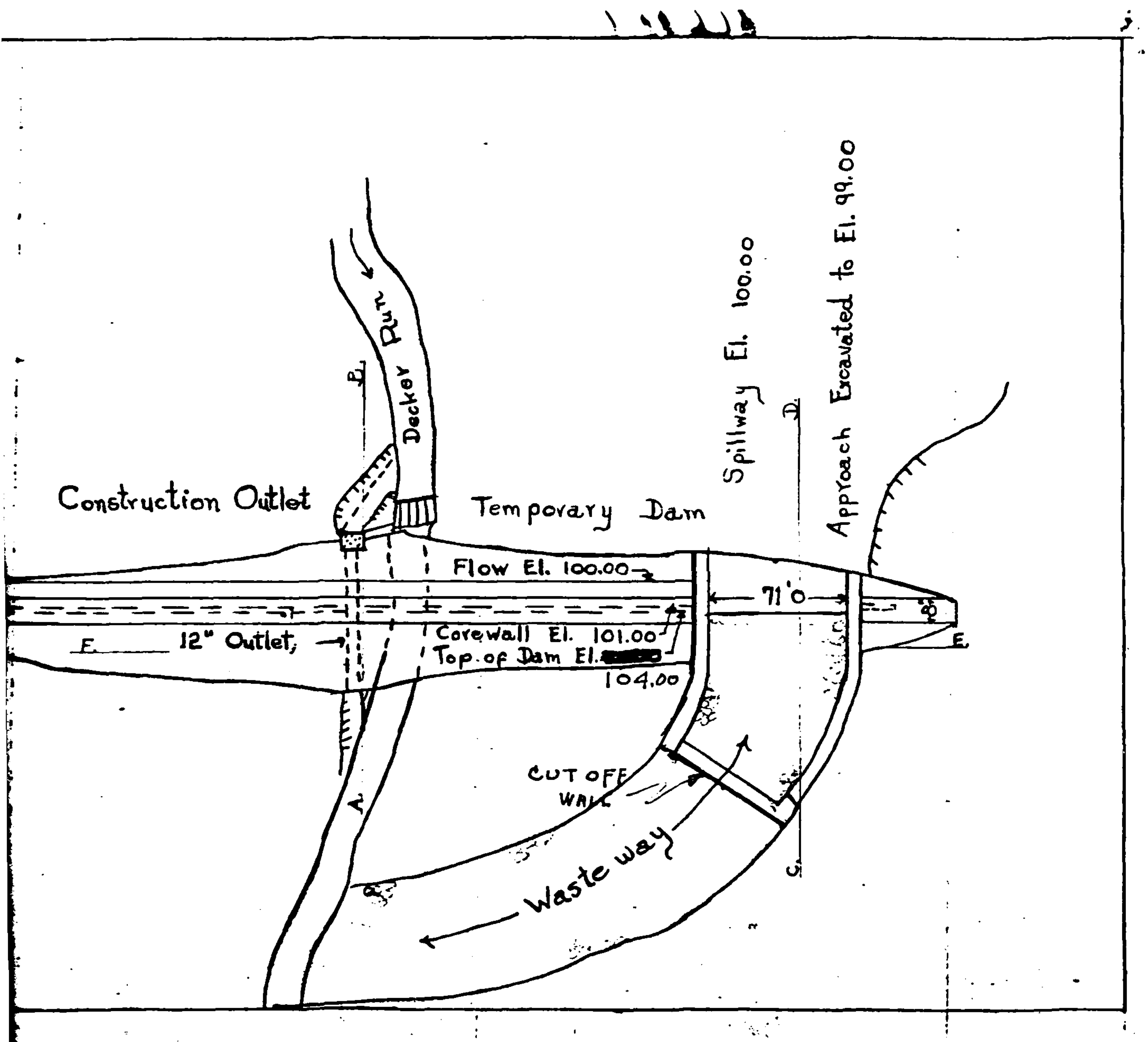


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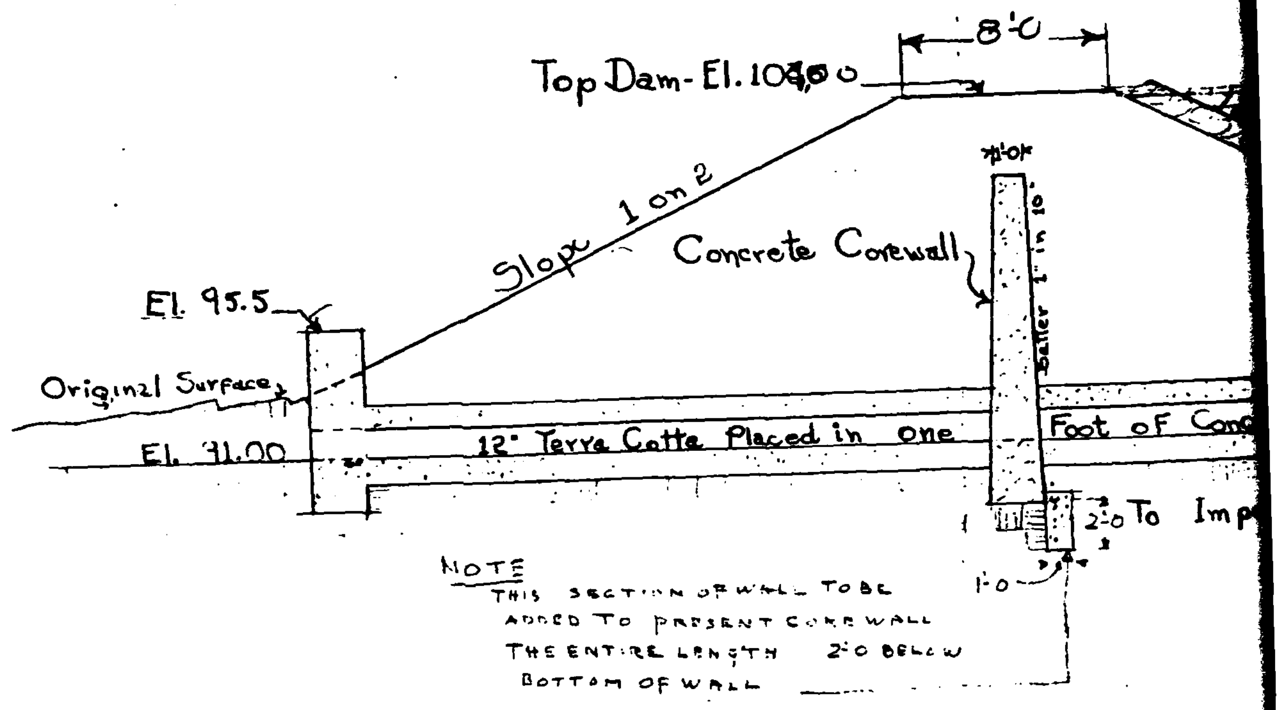
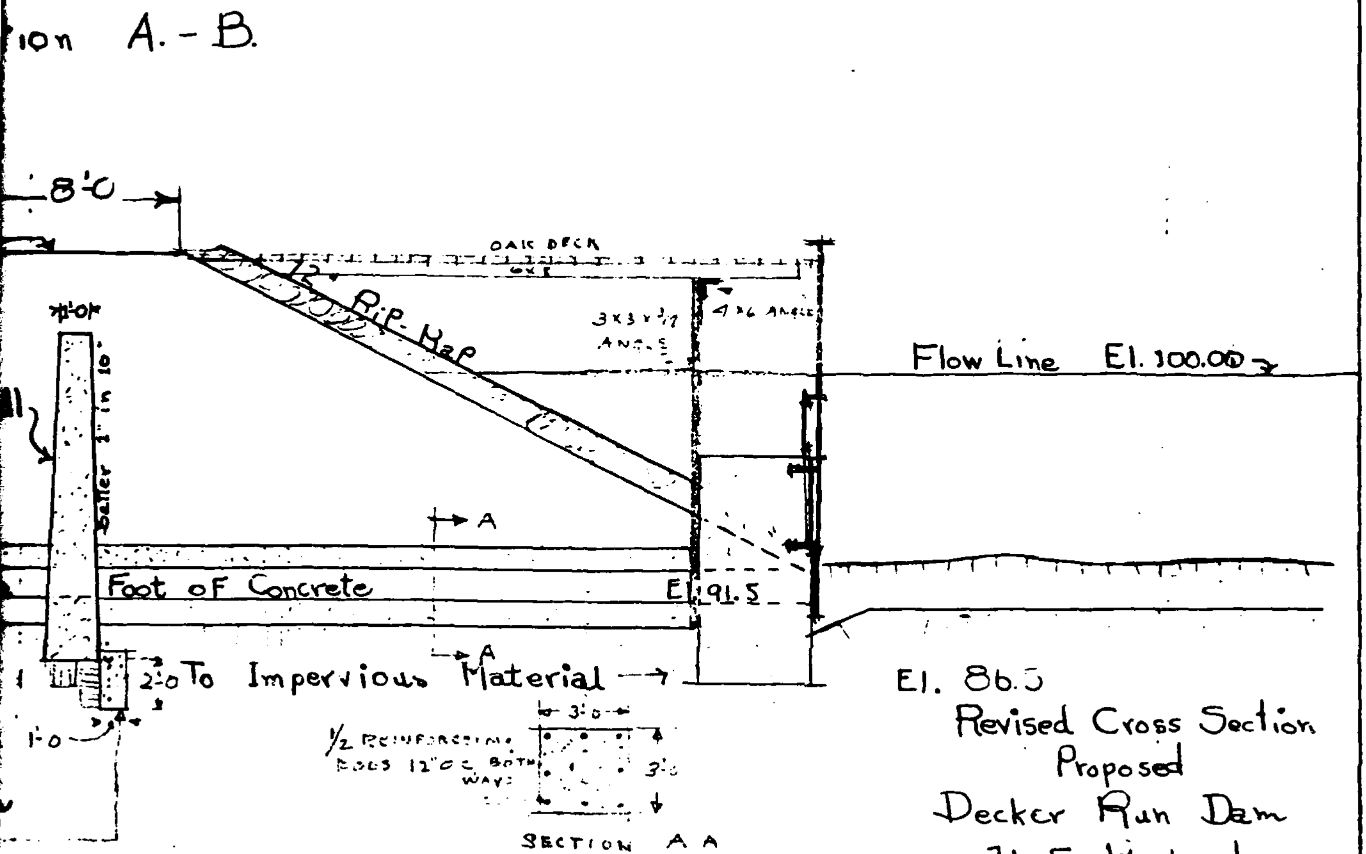
FIGURE 4

Section A.-B.


8'0
Top Dam-El.
1'0
Slope 1 on 2
Concrete Corewall
Batter 1 in 10
El. 95.5
Original Surface
El. 91.00
12" Terra Cotta Placed in one
Foot of Conc
2'0 To Imp
1'0
NOTE
THIS SECTION OF WALL TO BE
ADDED TO PRESENT CORE WALL
THE ENTIRE LENGTH 2'0 BELOW
BOTTOM OF WALL



Revised Cross Section
Proposed
Decker Run Dam
U. F. Rickard
566 Irvington Ave, Eliz., N.J.
Contractor
Scale 1" = 5 ft.

Sheet No 4

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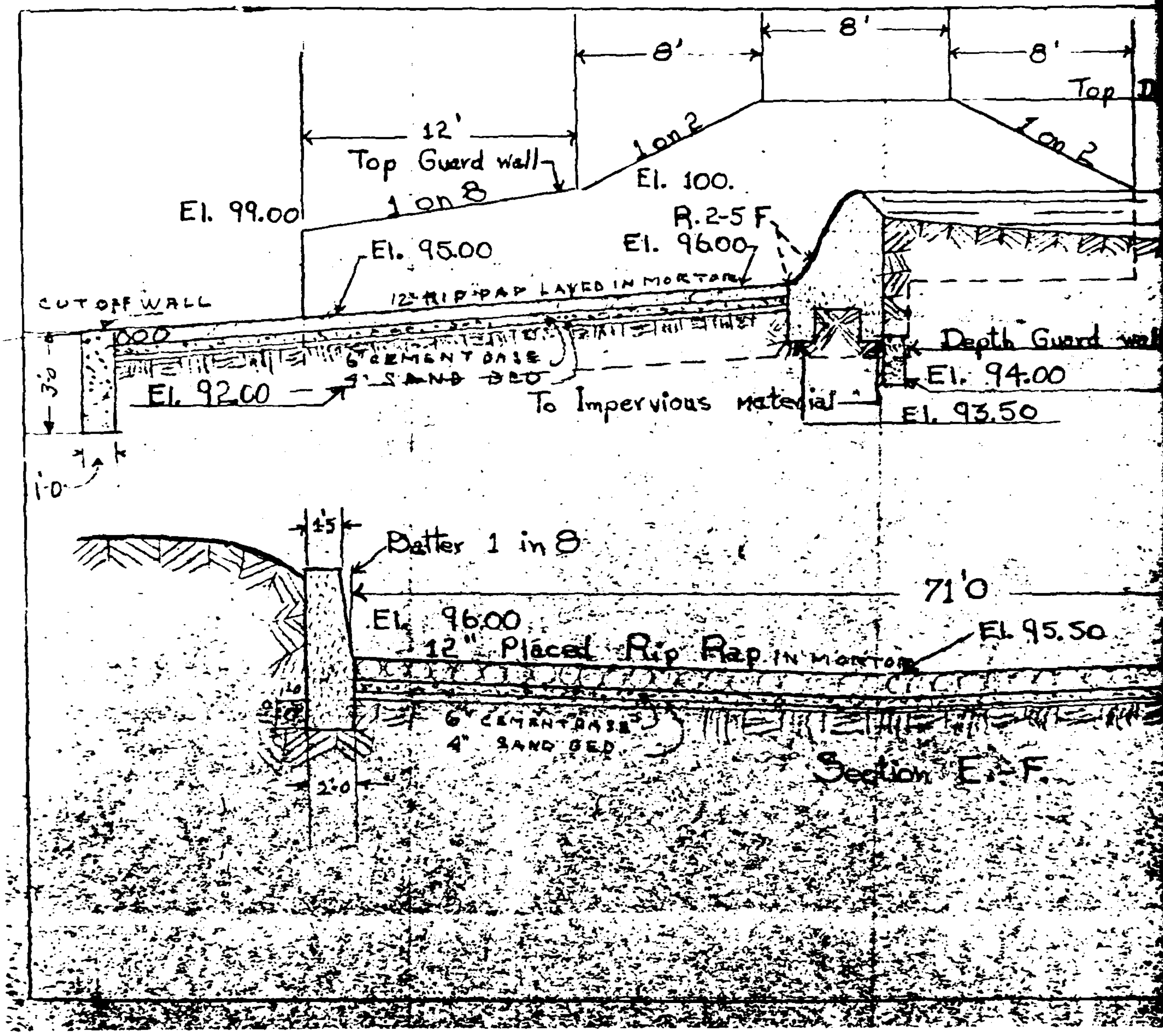
FIGURE 5



8'
8'
8'
Top D
12'
Top Guard wall
1 on 2
1 on 2
1 on 8
El. 100.
El. 99.00
R. 2-5 F
El. 95.00
El. 96.00
12" RIP RAP LAYED IN MORTAR
CUT OFF WALL
Depth Guard wall
3'0
6" CEMENT BASE
4" SAND BED
El. 92.00
El. 94.00
To Impervious material
El. 93.50
1'0
1'5
Batter 1 in 8
71'0
El. 96.00
12" Placed Rip Rap IN MORTOR
El. 95.50
6" CEMENT BASE
4" SAND BED
2'0
Section E-F.

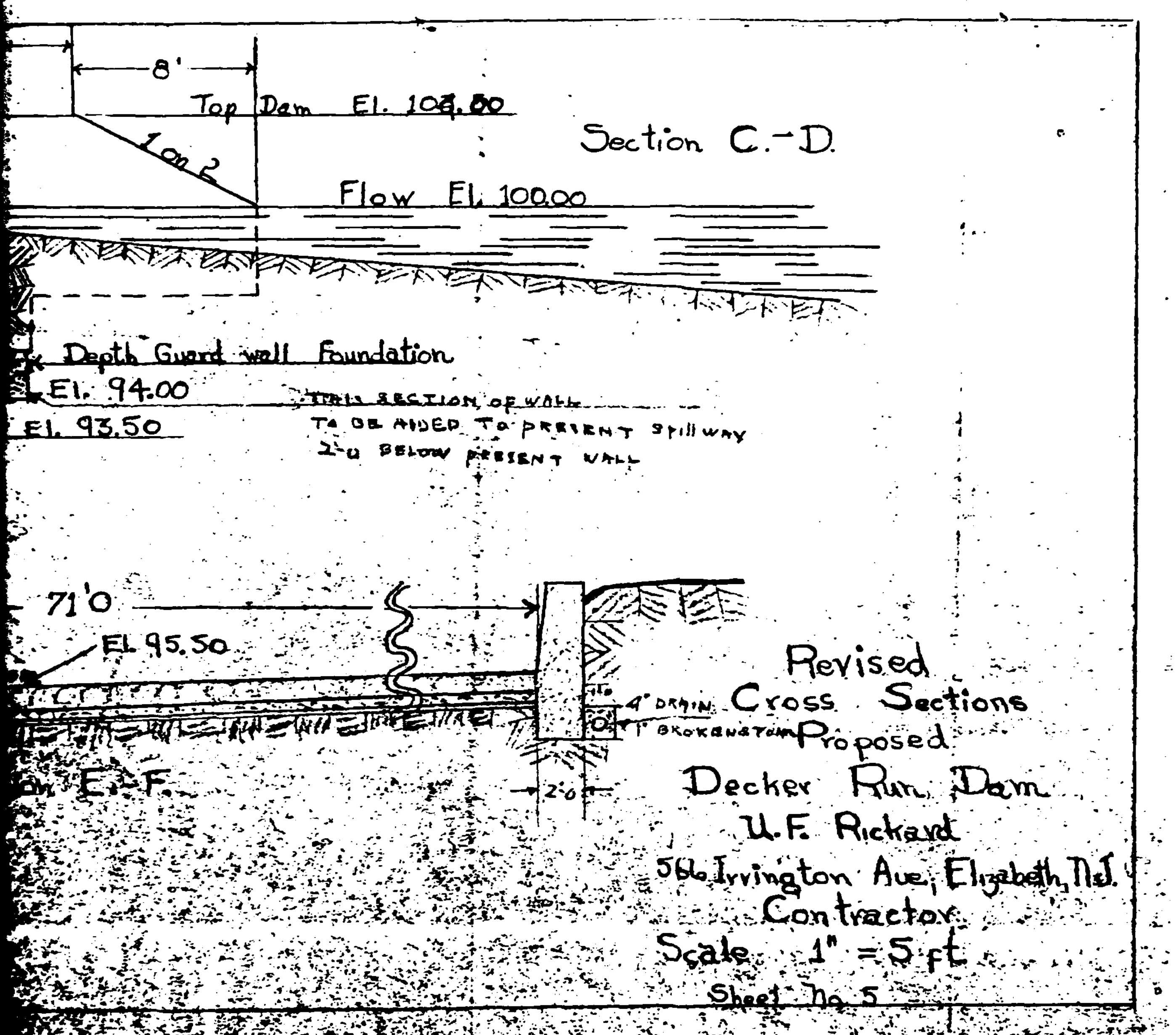


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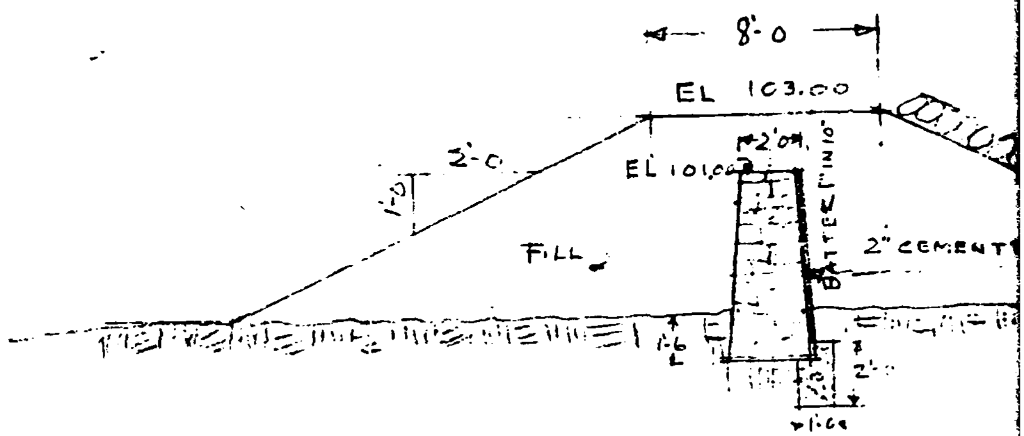
FIGURE 6

SECTION THRU



NOTE
THIS SECTION OF WALL TO BE
ADDED TO PRESENT CORE WALL
THE ENTIRE LENGTH 2'-0 BELOW
PRESENT WALL

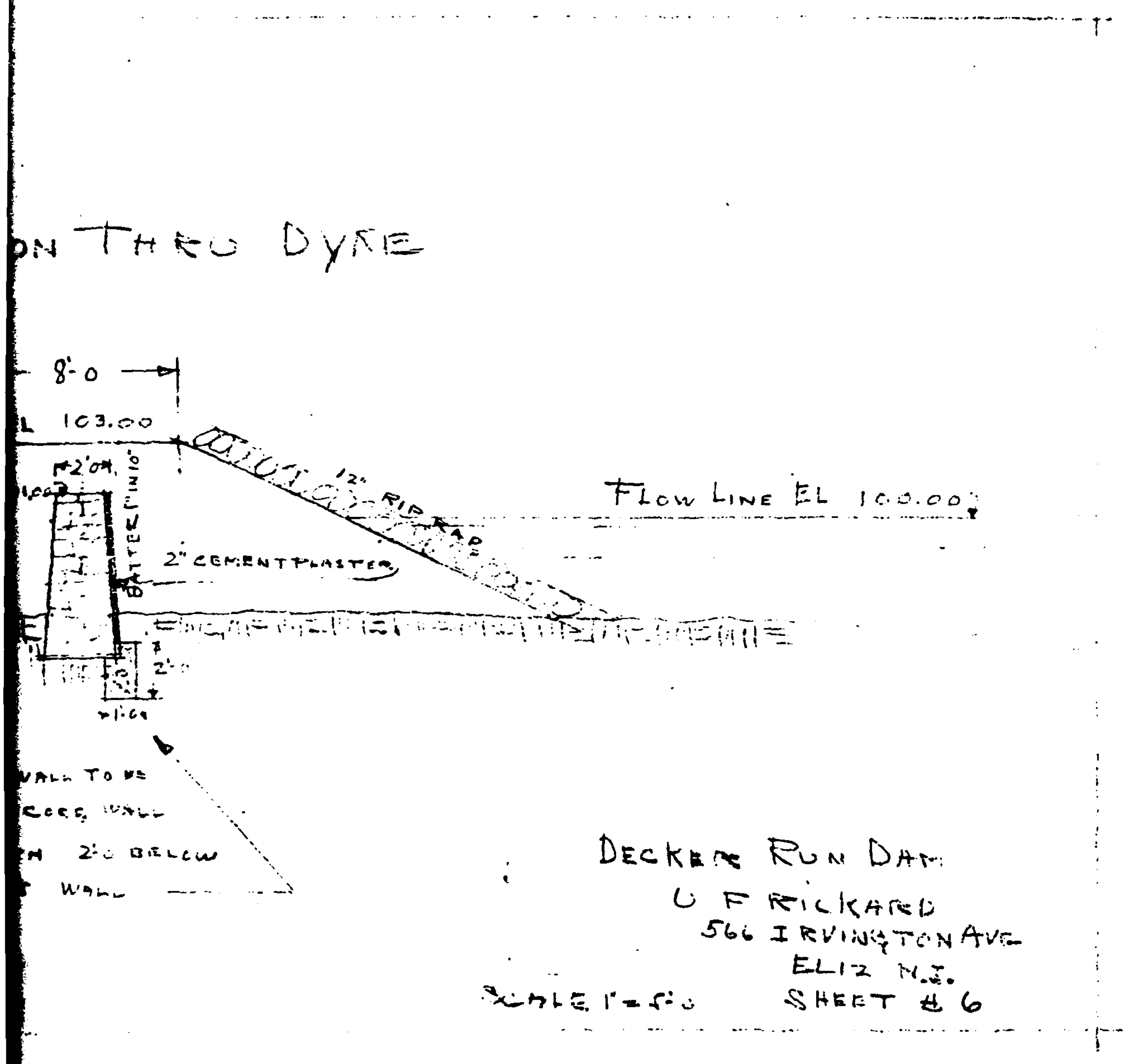


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FIGURE 7

APPENDIX F

GEOLOGY

Geology

Rickards Dam is located in the glaciated Low Plateaus section of the Appalachian Plateaus physiographic province of eastern Pennsylvania. In this area, the Appalachian Plateaus province is characterized topographically by flat-topped, hummocky hills formed as a result of glaciation and subsequent stream dissection of nearly flat-lying strata. The Devonian age sedimentary rock strata in Pike County regionally strike N35°E and dip gently to the northwest. The Delaware River is the major drainage basin in the area. Major tributary streams intersect the Delaware River at right angles; whereas, smaller streams display a slightly more random tributary pattern. Both major and minor tributary stream systems are joint controlled and exhibit modified rectangular and trellis-type drainage patterns.

Structurally, the area containing Pike County lies on the south flank of a broad, asymmetrical synclinorium that plunges to the southwest. Superimposed on this broad structural basin are numerous anticlinal and synclinal folds characterized by planar limbs and narrow hinges. Due to prior glaciation, low relief and surficial soil cover, fold axes are difficult to trace.

The sedimentary rock sequences in the vicinity of the dam and reservoir are probably members of the Susquehanna Group of Upper Devonian age (see Geology Map). The sedimentological changes observed in the Catskill Formation indicate that the rate of sedimentation exceeded the rate of basin subsidence, resulting in a facies change from marine to non-marine strata. On the accompanying geology map the delineation between the Middle and Upper Devonian age sedimentary rock sequences represents the Allegheny Front, which separates the Valley and Ridge physiographic province from the Appalachian Plateaus physiographic province.

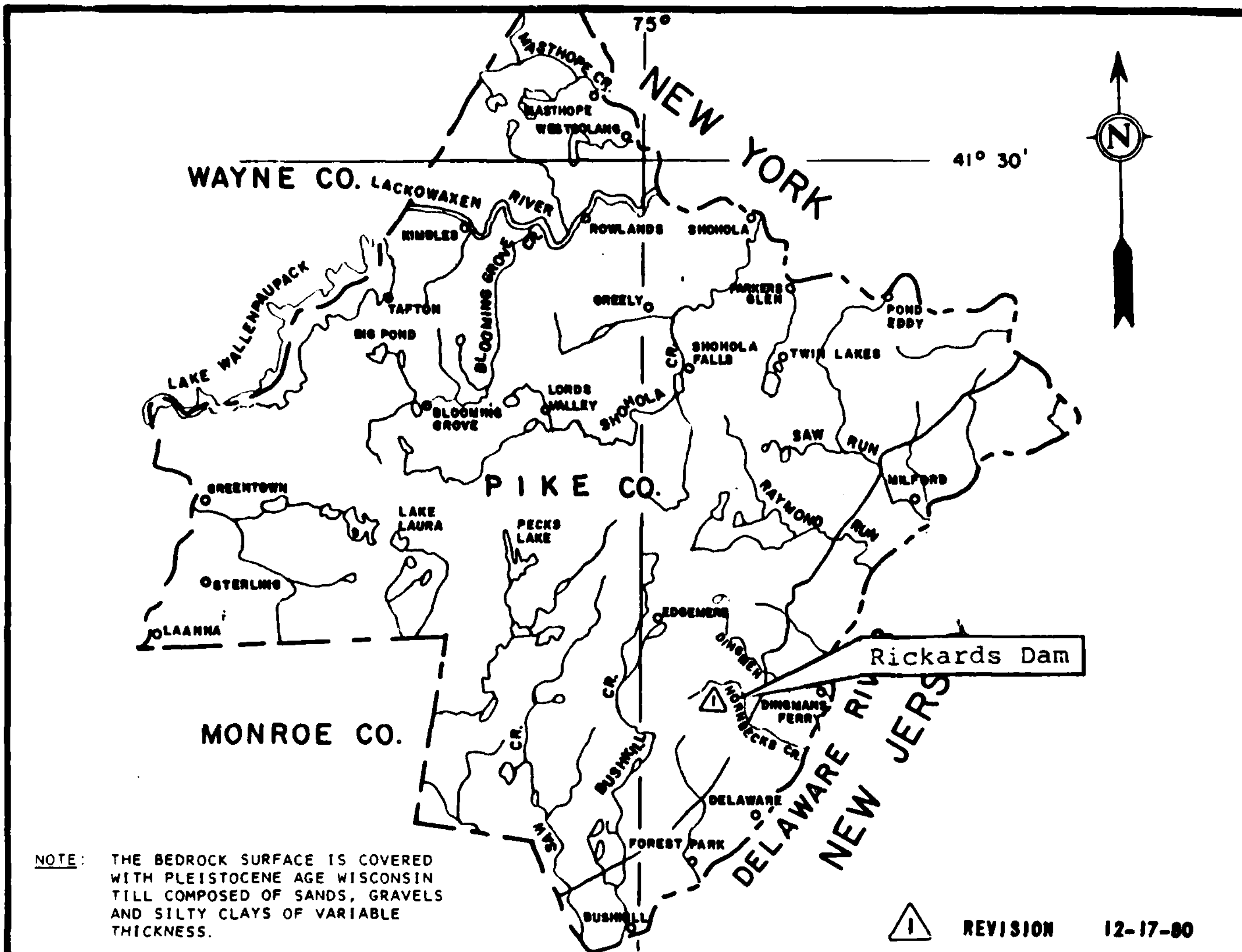
Approximately half of Pike County, including the dam site, is covered by a blanket of Wisconsin age (most recent) glacial drift which, based on the degree of weathering, was probably deposited during the Woodfordian stage. Valley bottoms are typically covered by recent alluvium and Woodfordian outwash of variable thickness, but typically less than 10 feet. These deposits are characteristically unconsolidated stratified sand and gravel, usually with more gravel than sand and some small boulders. The direction of the Wisconsin ice advance was from the northeast over the Catskill Mountains and from the north over the Appalachian Plateau. The terminal moraine resulting from the southern most advance of the Wisconsin ice sheet in this area is located in the southern portion of Monroe County, which borders Pike County to the South.

References:

1. Fletcher, F. W., Woodrow, D. L., "Geology and Economic Resources of the Pennsylvania Portion of the Milford and Port Jervis 15 minute U.S.G.S. Topographic Quadrangles," Pennsylvania Geological Survey, Fourth Series, Harrisburg, Atlas 223, 1970.

2. Sevon, W. D., Berg, T. M., "Geology and Mineral Resources of the Skytop Quadrangle, Monroe and Pike Counties, Pennsylvania", Pennsylvania Geological Survey, Fourth Series, Harrisburg, Atlas 214A., 1978.

3. Sevon, W., Personal Communication, Commonwealth of Pennsylvania Department of Environmental Resources, Harrisburg, December 3, 1980.



NOTE: THE BEDROCK SURFACE IS COVERED WITH PLEISTOCENE AGE WISCONSIN TILL COMPOSED OF SANDS, GRAVELS AND SILTY CLAYS OF VARIABLE THICKNESS.

LEGEND

UPPER DEVONIAN — **SUSQUEHANNA GROUP**

Catskill Formation - *Shohola Member interbedded 5- to 25-foot thick units of greenish-gray and grayish-red very fine to medium-grained sandstone and sandy shale and lesser medium-gray to medium-dark-gray sandstone and shale. Sandstones are predominantly low-rank graywackes. Beds are thin to very thick and most have simple or planar sets of small- to medium-scale, generally low-angle cross stratification. Contacts with shale units are abruptly disconformable to gradational. Sandstones are poorly cleaved. Shale is thinly laminated and well cleaved. Mud cracks, convolute bedding, and sole marks are present near contacts with sandstone units. Member is more than 2,300 feet thick. Lower contact is gradational and is placed at top of highest red bed of the underlying Analomink. Analomink Red Shale Member, medium-grayish red silty, micaceous, finely laminated well-cleaved shale containing thin beds of brownish-gray sandy siltstone and silty very fine grained sandstone. Unit is the "first red" going up section in Upper Devonian sequence. member is about 100 feet thick. Lower contact is gradational and is placed at the base of lowest red bed. Delaware River Flags Member, grayish-green, micaceous, laminated sandstone and lesser interbedded sandy shale. Beds range from a few inches to as much as 4 feet thick. Sandstones are low-rank graywackes and contain no marine fossils. Member is about 300 feet thick. Lower contact is gradational.*

MIDDLE DEVONIAN — **HAMILTON GROUP**

Mahantango Formation - *Upper member medium-dark-gray, fairly coarse grained, thin-bedded siltstone and silty shale; member is about 700 feet thick and is separated from lower member by the "Centerfield Reef," a calcareous siltstone biostrome containing abundant horn corals. The Centerfield is about 25 feet thick, lower member, virtually same lithology as upper member. Unit is about 1,100 feet thick. Lower contact is gradational.*

Marcellus Shale - *Dark-gray, evenly laminated, silty clay shale and clayey silt shale. Unit commonly contains very hard limy concretions and is well cleaved; bedding is generally obscured. Member is about 75-feet thick. Lower contact is gradational.*

SCALE

0 6 12 18 MILES

REFERENCE:
GEOLOGIC MAP OF NORTHEASTERN PENNSYLVANIA. COMPILED BY GEO. W. STOSE AND O.A. LJUNGSTEDT COMMONWEALTH OF PENNSLYVANIA DEPT. OF INTERNAL AFFAIRS DATED 1932, SCALE 1" = 6 MILES.

GEOLOGY MAP

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