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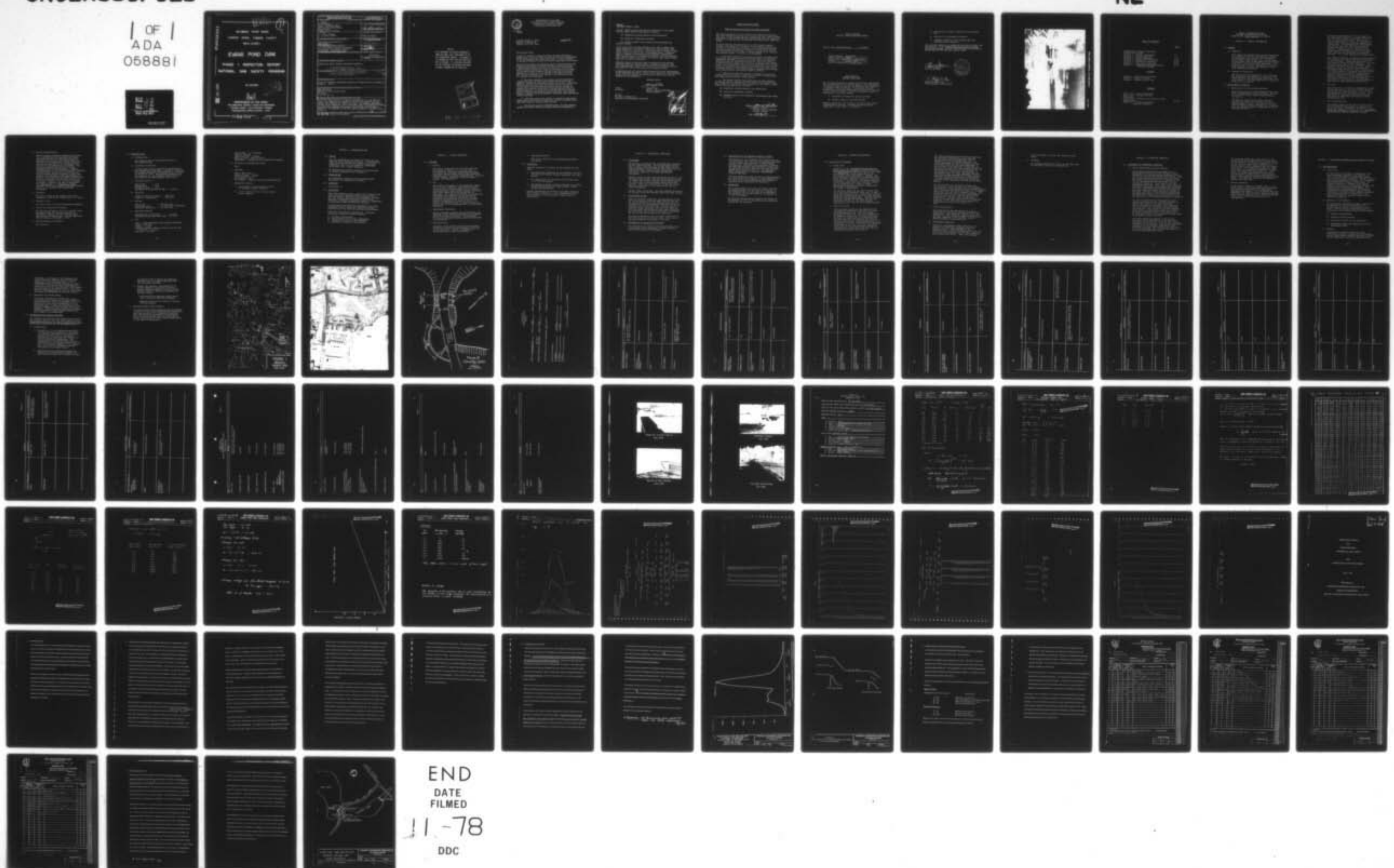

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DELAWARE RIVER BASIN

COOPER RIVER, CAMDEN COUNTY

NEW JERSEY

EVANS POND DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NJ 00394

DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

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This report cites results of a technical investigation as to the dam's adequacy. The inspection and evaluation of the dam is as prescribed by the National Dam Inspection Act, Public Law 92-367. The technical investigation includes visual inspection, review of available design and construction records, and preliminary structural and hydraulic and hydrologic calculations, as applicable. An assessment of the dam's general condition is included in the report.

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DEPARTMENT OF THE ARMY
PHILADELPHIA DISTRICT, CORPS OF ENGINEERS
CUSTOM HOUSE—2 D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO
NAPEN-D

Honorable Brendan T. Byrne
Governor of New Jersey
Trenton, New Jersey 08621

29 AUG 1978

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Evans Pond Dam in Camden County, New Jersey which has been prepared under authorization of the Dam Inspection Act, Public Law 92-367. A brief assessment of the dam's condition is given on the first three pages of the report.

Based on visual inspection, available records, calculations and past operational performance, the Evans Pond Dam, initially listed as a "High" hazard potential structure, but reduced to "Significant" hazard potential category as a result of this inspection, is judged to be in poor overall condition. The dam's spillway is considered inadequate as 61 percent of the 100-year flood would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. The actual capacity of the spillway should be determined using more precise and sophisticated methods and procedures by a qualified, professional consultant, engaged by the owner. This study should be completed within four months from the date of approval of this report. To afford protection against loss of the dam, an auxilary crest spillway should then be designed and constructed in calendar year 1979. In the interim, a detailed emergency operation plan and warning system should be promptly develped. Also, during periods of unusually heavy precipitation, around the-clock surveillance should be provided.

b. Within four months from the date of approval of this report, graded stone riprap should be installed at the dam's corrugated metal pipe (CMP) outlets to prevent further erosion.

c. The remedial measures recommended above are only temporary actions and will not alleviate the basic structural unsoundness of

NAPEN-D
HONORABLE Brendan T. Byrne

the dam. Within one year from the date of approval of this report, further studies should be undertaken to evaluate:

(1) Feasibility of major repairs to the existing dam.

(2) Design of a replacement structure.

(3) Complete removal of the existing dam and enlarging the present reservoir.

A copy of the report is being furnished to Mr. Dirk C. Hofman, New Jersey Department of Environmental Protection, the designated State Office contact for this program. Within five days of the date of this letter, a copy will also be sent to Congressmen James J. Florio and Edwin B. Forsythe of the First and Sixth Districts, respectively. Under the provisions of the Freedom of Information Act, the inspection report will be subject to release by this office, upon request, thirty days after the date of this letter.

Additional copies of this report may be obtained from the National Technical Information Services (NTIS), Springfield, Virginia, 22161 at a reasonable cost. Please allow four to six weeks from the date of this letter for NTIS to have copies of the report available.

An important aspect of the Dam Safety Program will be the implementation of the recommendations made as a result of the inspection. We accordingly request that we be advised of proposed actions taken by the State to implement our recommendations.

Sincerely yours,

James G. Ton

JAMES G. TON
Colonel, Corps of Engineers
District Engineer

1 Incl
As stated

Cy furn:
Mr. Dirk C. Hofman, P.E.
Department of Environmental Protection

EVANS POND DAM (NJ 00394)

CORPS OF ENGINEERS ASSESSMENT OF GENERAL CONDITIONS

This dam was inspected on 14 June 1978 by Louis Berger and Associates, Inc. under contract to the State of New Jersey. The state, under agreement with the U. S. Army Engineer District, Philadelphia, had this inspection performed in accordance with the National Dam Inspection Act, Public Law 96-367.

The Evans Pond Dam, initially listed as a "High" hazard potential structure, but reduced to "Significant" hazard potential category as a result of this inspection, is judged to be in poor overall condition. The dam's spillway is considered inadequate as 61 percent of the 100-year flood would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. The actual capacity of the spillway should be determined using more precise and sophisticated methods and procedures by a qualified, professional consultant, engaged by the owner. This study should be completed within four months from the date of approval of this report. To afford protection against loss of the dam, an auxilary crest spillway should then be designed and constructed in calendar year 1979. In the interim, a detailed emergency operation plan and warning system should be promptly develped. Also, during periods of unusually heavy precipitation, around the-clock surveillance should be provided.

b. Within four months from the date of approval of this report, graded stone rip-rap should be installed at the dam's corrugated metal pipe (CMP) outlets to prevent further erosion.

c. The remedial measures recommended above, are only temporary actions and will not alleviate the basic structural unsoundness of the dam. Within one year from the date of approval of this report, further studies should be undertaken to evaluate:

(1) Feasibility of major repairs to the existing dam.

(2) Design of a replacement structure.

(3) Complete removal of the existing dam and enlarging the present reservoir.

APPROVED: James G. Ton

JAMES G. TON
Colonel, Corps of Engineers
District Engineer

DATE: 29 Aug 78

PHASE I REPORT
NATIONAL DAM INSPECTION PROGRAM

Name of Dam Evans Pond Dam NJ 00394

State Located	New Jersey
County Located	Camden
Coordinates	Lat. 3954.0 - Long. 7501.3
Stream	Cooper River
Date of Inspection	14 June 1978

ASSESSMENT OF GENERAL CONDITIONS

The concrete spillway is badly deteriorated structurally and is undermined to an unknown degree. The embankment is adequate only as long as the Wallworth dam remains at its present crest elevation. Despite apparent low probability of serious downstream damage or loss of life in the event of failure, corrective measures should be undertaken in the near future:

1) Construct an auxiliary crest spillway

2) Install riprap at the CMP outlets

However, these will not alleviate the basic structural unsoundness of the dam. Further studies should be undertaken in the future to evaluate:

1) Feasibility of major repairs to the existing dam

2) Design of a replacement structure

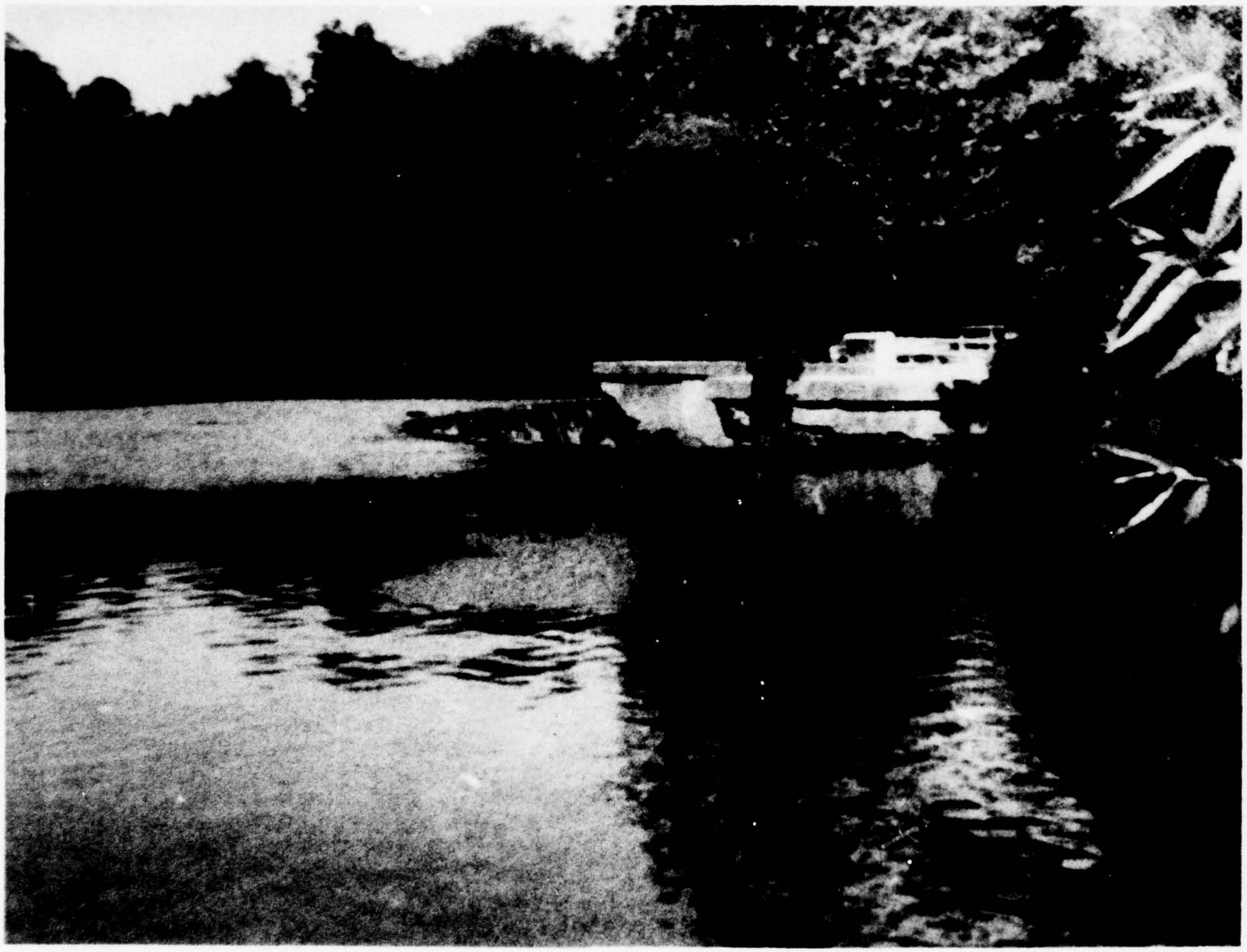
3) Complete removal of the existing dam and redredging the reservoir

The spillway capacity is inadequate and does not meet the requirements of the Recommended Guidelines for Safety Inspection of Dams, having a capacity before overtopping of only 60% of the spillway design flood.

STATE OF KENTUCKY
F. KEITH
JOLLS
5432
REGISTERED
PROFESSIONAL ENGINEER

F. Keith Jolls P.E.
Project Manager

Rudolph Wrubel P.E.
Vice President, Engineering



JUNE 1978

OVERVIEW OF EVANS POND DAM

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
NAME OF DAM EVANS POND DAM NJ 00394

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority

This report is authorized by the Dam Inspection Act, Public Law 92-367, and has been prepared in accordance with contract FPM-36 between Louis Berger & Associates, Inc. and the State of New Jersey and its Department of Environmental Protection, Division of Water Resources. The State in turn, is under agreement with the U.S. Army Engineer District, Philadelphia to have this inspection performed.

b. Purpose of Inspection

The purpose of this inspection is to evaluate the structural and hydraulic condition of the Evans Pond Dam and appurtenant structures, and to determine if the dam constitutes a hazard to human life or property.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances

Evans Pond Dam is an earth embankment structure with a deteriorated concrete spillway and two new corrugated metal relief pipes (recently installed).

The dam was constructed in 1917 and has a variable top width in the earth section of approximately 25 feet. The crest is covered with an asphalt roadway pavement but the bridge over the spillway apron is presently closed to vehicular traffic.

The dam is approximately 370 feet long and the spillway structure is a semicircular ogee weir of 78 feet crest length with a sluice gate at each end of its arc. The spillway is a reinforced concrete structure supported on timber piles. The abutments are incorporated into bridge piers for the structure which crosses the pond at a point just below the spillway. This bridge is a steel stringer timber decked structure twenty feet in width with wingwalls and abutments of reinforced concrete. It is also supported on timber piles.

The circular spillway is located at approximately the center of the dam and has two sluice gates at each abutment. These sluiceways have 18 inch outfall lines and were employed in the past to control the level of the reservoir. The location and grade of these lines are unknown. Within the last two years, two additional corrugated metal elitipical pipes, approximately 6 x 4 feet have been installed just west of the concrete spillway to reduce the undercutting problem as delineated in the appended 1975 Report by Edward H. Richardson Associates, Inc. The invert elevations of the new CMPA pipe spillways are constructed at approximately the same grade as the concrete spillway crest.

b. Location

Evans Pond Dam is located in Haddonfield, Camden County, New Jersey. The dam is built across the Cooper River approximately 7.2 miles from its confluence with the Delaware River. It is approximately 200 yards above the Wallworth Dam which is immediately southeast of Kings Highway(Route 41).

c. Size Classification

The maximum height of the dam is about 13 feet and the conservation storage is estimated to be 50 acre feet. Therefore, the dam is in the <u>small size category</u> as defined by the <u>Recommended Guidelines for Safety Inspection of Dams</u>.

d. Hazard Classification

Several densely populated communities; Cherry Hill, Haddonfield, Collingswood and Camden are below the dam site but practically all residential areas are above flood elevation. Flooding in the downstream reaches below the dam are confined mainly to the Cooper River Lake basin and Camden County Parklands. The historic highwater datum fairly closely approximates parkland boundaries. Based on available data, it is felt the existing structure is potentially unstable and its failure and ensuing mudwave could conceivably trigger the failure of the Wallworth dam just downstream. Further, the existing Kings Highway bridge just below the Wallworth Dam is quite old and its structural stability, due to such a collapse, is suspect. However, the dam is downgraded from high hazard to a significant hazard category as the only economic loss most probably would be the aforementioned downstream structures.

e. Ownership

The dam is owned by the Camden County Park Commission, Park Drive, Cherry Hill, N.J. 08054

f. Purpose of Dam

The dam is used for scenic/recreation purposes.

g. Design and Construction History

The dam was designed in 1917 as a rolled earth embankment with the concrete spillway by Remington & Vosbury for the original owner, the Borough of Haddonfield. The two additional CMPA sluiceways were added in 1976.

h. Normal Operating Procedures

See Section 4

1.3 PERTINENT DATA

a. Drainage Area

The drainage area of the Evans Pond Dam is 17.4 square miles.

b. Discharge at Dam Site

A water level gage is located at the Wallworth dam immediately downstream. According to records, the maximum discharge recorded there is 3300 cfs on August 28, 1971. The spillway capacity with the reservoir at the abutment top elevation is calculated to be approximately 2750 c.f.s.

c. Elevation (M.S.L.)

Top of dam - 17.0
Maximum pool - 17.0
Recreation pool - 12.5
Streambed at centerline of dam - 4± feet

d. Reservoir

Length of recreation pool - 3430 feet
Length of maximum pool - 7700 feet

e. Storage

Top of dam - 220 acre feet
Recreation pool - 50 acre feet (estimated)
Design for surcharge - 170 acre feet

f. Reservoir Surface

Maximum pool (top of dam) 70 acres
Recreation pool (spillway crest) 25 acres

g. Dam

Type - earth embankment with concrete spillway
Length - 375 feet
Height - 13 feet
Freeboard between normal reservoir and the top of the dam - 4.5 feet
Top width - 25 feet

Side slopes - 2:1 (maximum)
Zoning - Unknown
Impervious core - unknown
Grout curtain - none recorded
Embankment - composition and compactness unknown

h. Diversion and Regulating Tunnel

None

i. Spillway

Type - Ogee crest
Length of weir - 78 feet
Crest elevation - 12.5
U/S Channel - none
D/S Channel - reservoir pond for Wallworth Dam

j. Regulating Outlets

1) 2 wood gates (ratchet-operated) with 18" Ø pipes (Inverts unknown)

2) 2 6'x4' CMPA Invert El. 12.5± (Outlet invert unknown)

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

The only information available for review for the Evans Pond Dam included the report: "Investigative Study of Evans Pond Dam for Camden County Park Commission, July 1975, by Edward H. Richardson Associates, Inc., Newark Delaware."

No construction contract drawings, specifications or as-built documents were available.

2.2 CONSTRUCTION

No information regarding the dam construction, maintenance or repairs was available.

2.3 OPERATION

See Section 4

2.4 EVALUATION

The field inspection and a review of the boring logs taken during the appended report study indicate that the subsoil underlying the embankment is weak. The lack of detailed construction records and additional geotechnical analyses render it impossible to make more cogent subsurface evaluation.

An underwater investigation undertaken by the Park Commission about 5 years ago indicated serious undermining of the concrete spillway and abutments.

Additional information required for a detailed structural evaluation should include:

1) As-built measurements
2) Soils borings in the earth embankment
3) Piezometric levels in the embankment
4) Additional underwater inspections

SECTION 3 - VISUAL INSPECTION

3.1 <u>FINDINGS</u>

a. General

The visual inspections of the Evans Pond dam took place on June 14, 21, and 28, 1978. An underwater inspection was undertaken about five years ago by Camden County Park Commission employees but the original documentation, save for a brief summary contained in the Richardson report, was unavailable.

b. Dam

The surface and slopes of the embankment appear to be in fair condition. Some minor erosion has occurred at various spots and the recently installed sluice pipes exhibit outlet velocities that could cause scour during periods of high flow. No evidence of seepage or significant settlement were observed but at the time of inspection the backwater elevation from the Wallworth dam downstream limited the exposed height of embankment to about 3.5 feet. It appears the upstream face is heavily silted up and the toe of the downstream face is continually submerged.

c. Appurtenant Structures

Serious concrete spalling and deterioration was observed at the concrete abutments and wingwalls below the concrete spillway. As previously stated, the spillway structure and access bridge are undermined.

d. Reservoir Area

According to the Camden County Park Commission officials, nothing is done regarding siltation of the reservoir. Debris is removed as a continuing part of Park maintenance.

e. Downstream Channel

Some minor erosion of the downstream reservoir was noted.

3.2 EVALUATION

The main subjects of concern to the inspection team were:

a. The structural condition of the abutment walls and spillway with special concern regarding the undermining.

b. The capacity of the spillway and the additional 6 x 4 foot CMPA pipes.

c. The potential hazard of Evans Pond dam in relation to the Wallworth dam and Kings Highway bridge immediately downstream.

Further discussion and evaluation of these subjects are covered in Section 7. The recommendations set forth on page 12 of Richardson's report were not implemented.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

Operational procedures were not physically observed by the inspection team. From discussions with Mr. John E. Kern, Superintendent of the Camden County Park Commission, it was learned that except for the removal of debris blocking the spillway and sluiceways, there are no formal operational procedures.

However, prior to the installation of the two 6' x 4' corrugated metal pipes, the two wooden gates located on either side of the spillway (each controlling an 18" pipe) were ratchet operated during high flow periods. Since the installation of the two relief pipes the operation of the wooden gates has not proved necessary.

During normal conditions, the water surface elevation of the pond is at the spillway crest and CMPA inverts.

4.2 MAINTENANCE OF DAM

Complete periodic inspection and maintenance of the dam is unfeasible since the upper face of the dam is almost completely silted and the lower face is approximately 80% continuously submerged due to the backwater of the Wallworth Dam. The two ratchet-operated wooden sluiceways are the only means of lowering Evans Pond below spillway crest elevation. The successful use of these sluiceways is doubtful due to the silting condition and vandalism.

Draining of Wallworth Lake to permit inspection of the lower face is not feasible since there are no apparent drawdown facilities at that dam.

4.3 MAINTENANCE OF OPERATING FACILITIES

No maintenance is presently being performed on the gated pipes except occasional removal of debris from the spillway and sluiceways.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

Throughout the year following the installation of the two CMPA's, the flow was continuously monitored by Park Commission personnel to insure proper installation. Moreover, they continue to monitor the area during heavy flow periods to insure that the upstream reservoir does not rise too high and impede the reservoir intake for the Haddonfield water supply which is located a considerable distance upstream.

The Park Commission does not have a formalized plan for contacting civil defense or other authorities but rely on their own monitoring and methods of alerting local authorities as necessary.

4.5 EVALUATION

Since the drawdown facilities for Evans Pond Dam are hydraulically poor and none are apparent at the Wallworth Dam, in the event of an emergency the stability of the dam could be in jeopardy if it were overtopped.

The present operational procedures are deemed to be adequate in view of the physical and hydraulic aspects of the location.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data

According to the Recommended Guidelines For Safety Inspection of Dams, the Evans Pond dam is a small size but was classified by the Corps of Engineers as a high hazard due to the surrounding urban development. However, the site inspection revealed the downstream flood plain is almost completely clear of habitable structures. In addition, the storage capacity of the pond itself has been severely reduced due to sedimentation. Based on these observations the hazard rating is downgraded from high to significant.

The spillway length of the dam at Evans Pond is 78 feet with abutments which are 4.5 feet above the spillway crest. Additional discharge capacity is provided by two 6'x4' CMPA passing under the western embankment. Maximum combined discharge through the conduits and over the spillway is 2750 cfs, the CMPA conduits contributing only 250 cfs at overtopping head.

b. Experience Data

From the gage records at the Wallworth dam, the maximum flood within the past ten years occurred August 27-29, 1971 with a peak discharge of 3300 cfs. This was adjudged to be somewhat greater than a 50-year frequency. However, the consultant has determined that the SDF should be the based on the 100-year precipitation event. This determination is in conformance with the aforementioned inspection guidelines and is the result of a subjective evaluation of the various hazard potential considerations associated with this dam and discussed throughout this report.

The inflow hydrograph was calculated utilizing the 100-year precipitation event from the Precipitation Intensity-Duration Curves prepared by the U.S. Weather Service. The inflow hydrograph for this drainage area was calculated using the SCS unit hydrograph. A peak inflow to the reservoir of 4900 cfs for the 100-year flood event was routed through the reservoir resulting in a minor reduction in the discharge to 4700 cfs. Additionally, a Log-Pearson Type III flood frequency analysis performed on data obtained at a gaging station 1000 feet downstream from Evan's Pond yields a 100-year flood of 4970 cfs which correlates with the inflow discharge obtained by the prior method.

At the direction of the Corps of Engineers, the 100-year frequency flood and its respective discharge were also computed utilizing precipitation values obtained from Technical Paper No. 40 which were input into the HEC-1 computer program. The values obtained for the 100-year flood before and after routing were 4634 cfs and 4498 cfs respectively. Based on this reevaluation program, the spillway capacity will accommodate about 60% of the SDF which would overtop the embankments by slightly more than 1.0 feet.

c. Visual Observations

The stability of the dam relies to a large extent on the backwater from the Wallworth dam downstream. This backwater extends up to the spillway at Evans Pond but causes a reduction in spillway capacity of the two CMPA sluices during periods of high discharge.

d. Overtopping Potential

Using the recommended results obtained, the spillway is marginally inadequate for the design criteria. The discharge of the reservoir for a 100-year storm would be 4498 cfs with a spillway capacity of 2750 cfs; therefore overtopping would occur. Thus, the capacity of the spillway is 60% of the 100-year flood event.

e. Drawdown

No drawdown capabilities exist at this dam since the two small sluices are inoperative.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations and Data Review

The original ogee concrete spillway is in extremely poor condition with extensive major cracking on the flared abutment sidewalls which direct the overflow beneath the park bridge. They are deteriorated beyond sound structural repair. The present spillway capacity is reduced by the lack of freeboard beneath the bridge and the converging sidewalls. According to the Park Commission officials, their scuba divers observed extensive undermining of the foundation structure but little differential settlement was observed; thus the supporting timber piling is thought to be sound. Because of the continued undermining and subsequent loss of soil, the bridge and the spillway could collapse should the piling shift or rupture or if the erosion cavities are breached.

The two corrugated metal pipe spillways were installed principally as a remedial measure to control the level of the upper reservoir. (The recommendations setforth in the appended report by Edward H. Richardson Associates were not adopted). These pipes do not have adequate earth cover to support heavy vehicular traffic and have inadequate outfall scour protection. As previously stated, the road over the dam crest is presently closed to traffic.

The earth embankment appears to be in fair condition but is suspect due to the spillway structure undermining and the weak soil strata evidenced by the borings. Much of the downstream embankment is continuously submerged by the reservoir tailwater from the Wallworth dam and the upstream slopes are silted up to within 2 or 3 feet of the embankment crest.

Four borings conducted at the dam site (two on the crest of the dam and two 150 feet north of the spillway) revealed the uppermost 20 feet of soil to be a loose sand, underlain by a medium compact silty micaceous sand to a depth of 40 feet. Soil conditions below this forty foot depth are categorized in general as sand, silty and clayey sand and sandy silt. Some gravel is always intermixed with the major soil fraction and this gravel, together with coarser sand, becomes increasingly abundant with depth. The depth to bedrock is estimated at greater than 100 feet.

b. Seismic Stability

As the dam is located in Seismic Zone 1, only minor hazard exists from earthquake forces and the potential vulnerability is negligible regarding this aspect. It is believed that the embankment was constructed by compaction methods rather than by hydraulic fill, and liquefaction from seismic activities would not be a consideration.

SECTION 7 - ASSESSMENTS/RECOMMENDATIONS/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Conditions

On the basis of the Phase I examination, the earth embankment appears to be adequate as long as the Wallworth dam downstream remains at its present crest elevation. However, should this dam be removed or collapse, the conditions would immediately worsen at Evans Pond. The concrete spillway is thought to be beyond economic repair and is structurally unsound, due mainly to the foundation undermining. The installation of the additional CMPA pipes is a satisfactory stopgap measure but is mainly effective in controlling the upstream reservoir crest during periods of normal flow. The embankment stability against seepage failure remains in question.

b. Adequacy of Information

The information gathered for Phase I appears to be adequate in view of the urgency and recommendations stated in Paragraphs c and d below. However should additional studies be undertaken, the following data would be needed:

1) As-built measurements

2) Additional soils borings

3) Piezometric levels in the embankment

4) Additional underwater inspections of the undermined areas

c. Urgency

A collapse of the Evans Pond dam could conceivably trigger a failure of the Wallworth dam and additionally endanger the Kings Highway bridge immediately below the Wallworth dam.

Conversely, a collapse of the Wallworth dam would ultimately endanger the questionable stability of the Evans Pond Dam. It is recommended that if further studies are undertaken that the Wallworth Dam be analyzed in conjunction with this and any other restrictions on this reach of the Cooper River. It is felt that the recommendations set out for this study dam should be undertaken in the near future.

d. Necessity for Further Study

An overall assessment of conditions at the Evans Pond Dam, in spite of its poor condition and juxtiposition with the Wallworth dam is deemed to be not unduly significant as it is determined that it does not constitute a major hazard to human life and only minor danger to property. Further studies regarding its safety, unless directed towards complete restoration, are thought to be unnecessary.

7.2 RECOMMENDATIONS/REMEDIAL MEASURES

The attached calculations have shown that the spillway capacity does not meet the requirements of the Recommended Guidelines for Safety Inspection of Dams, being able to pass only 60% of the design flood.

a. Alternatives

1) As stated in 7.1.d, additional structural studies could be undertaken to ascertain the economic feasibility of restoring the present structure and if this proved unfeasibile, to undertake the design of a new dam immediately downstream. The undermining of the present dam could be rectified but it is believed this would not solve the inherent weakness of the embankment zoning and the unknown structural condition of the timber piling.

2) Excepting for the adverse environmental effects to the surrounding parkland, the most prudent solution is to remove the dam

and spillway and to dredge the upstream reservoir out to a depth compatible with the downstream reservoir.

3) Despite the apparent low probability of serious damage in the event of failure, certain remedial measures relating to the present structure should be undertaken in the near future:

   - Constructing an auxiliary crest spillway in the east approach embankment.
   - Placing riprap at the outlet of the two CMPA sluiceways.

b. O&M Maintenance and Procedures

The Camden County Park Commission should develop a check list for periodic maintenance inspections and keep records of all findings and repairs. Also, their present procedures for monitoring the site during storms could be formalized to insure notification of Civil Defense Authorities in the event of emergencies.

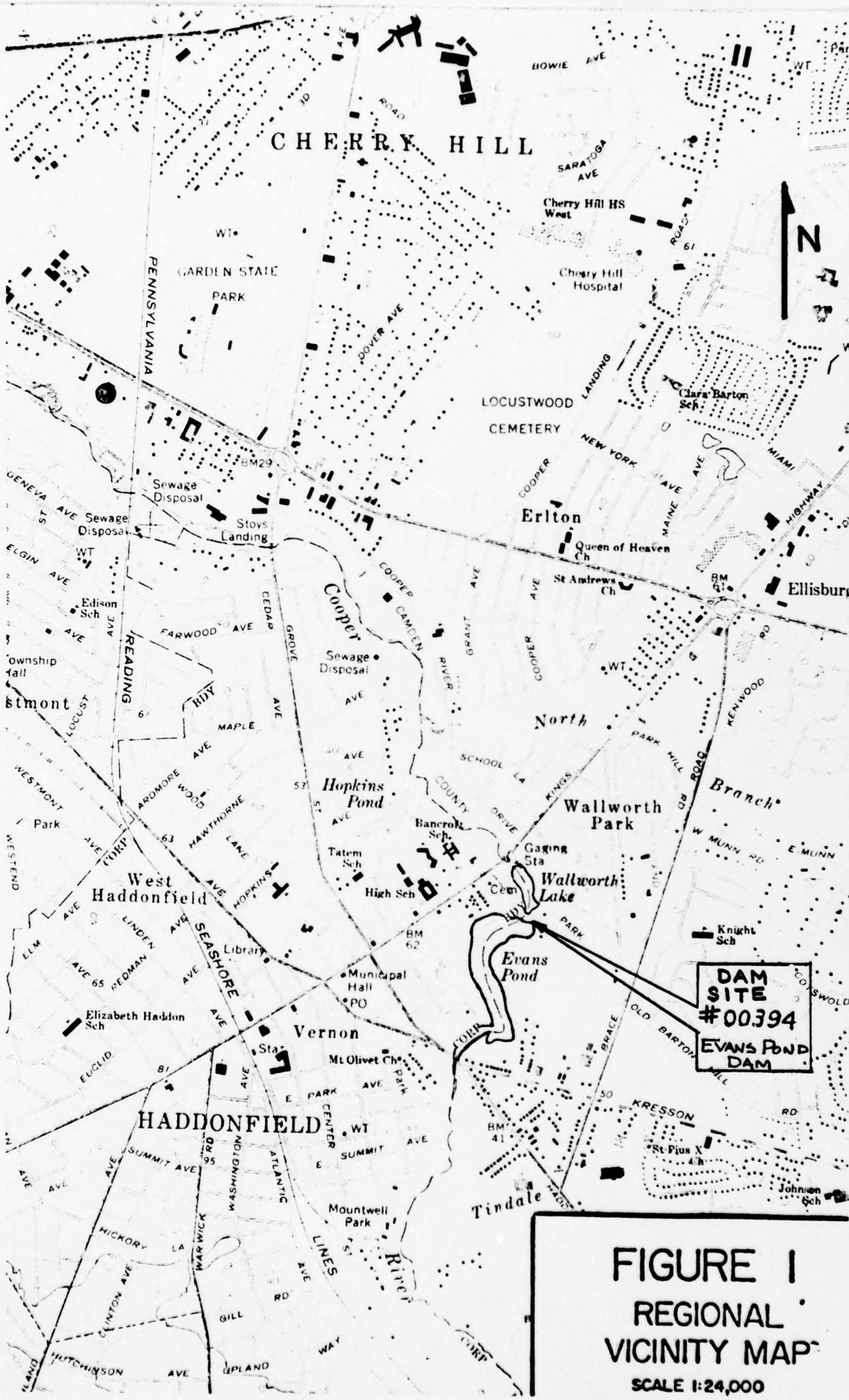


FIGURE I
REGIONAL VICINITY MAP
SCALE 1:24,000


COOPER RIVER DRIVE
KINGS HIGHWAY
00394

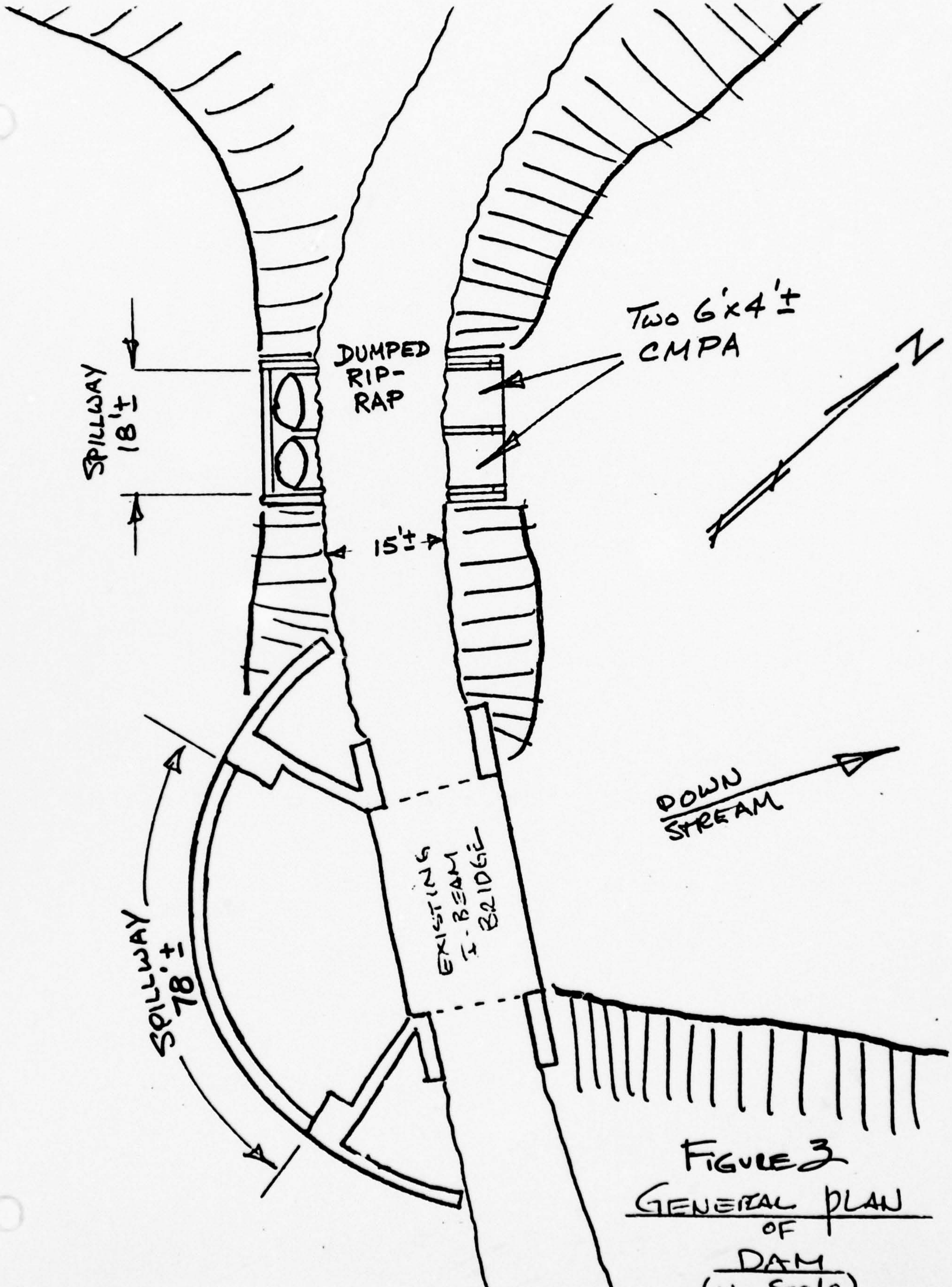


FIGURE 3

GENERAL PLAN OF DAM

(No Scale)

SHEET 1

Check List
Visual Inspection
Phase 1

Name Dam Evans Pond Dam   County Camden   State New Jersey   Coordinators NJDEP

Date(s) Inspection June 14,21, 28, 1978   Weather Sunny   Temperature 80°F

Pool Elevation at Time of Inspection 12.7 M.S.L.   Tailwater at Time of Inspection 9.7 M.S.L.

Inspection Personnel:

T. Chapter	H. Grout	
M. Carter	R. Lang	
K. Jolls		

K. Jolls Recorder

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEEPAGE OR LEAKAGE	No seepage observed All joints in concrete structure leak.	
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Satisfactory	
DRAINS	Not visible	
WATER PASSAGES	2 CMP elliptical pipes	Installed recently (1976±)
FOUNDATION	Nothing visible Appears undercut, especially in discharge channel under bridge.	(See Richardson Report)

CONCRETE/MASONRY DAMS

VISUAL EXAMINATION OF	OBERSVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Yes - very poor condition	Much spalling Concrete surfaces at spillway elevation badly deteriorated.
STRUCTURAL CRACKING	Yes	All concrete in very poor condition. Unfeasible to repair.
VERTICAL AND HORIZONTAL ALIGNMENT	Ok. Structure on timber piling.	Retaining wall joints at bridge badly eroded.
MONOLITH JOINTS	Not visible	
CONSTRUCTION JOINTS	Bad condition	Structural repairs required.

EMBANKMENT

SHEET 4

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	None. Top of berm is roadway pavement.	
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	None	Approach roadway on dam presently closed to traffic.
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	Satisfactory	
RIPRAP FAILURES	No riprap.	

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	Satisfactory	No erosion
ANY NOTICEABLE SEEPAGE	No	
STAFF GAGE AND RECORDER	None	
DRAINS	2 - CMP drains added in left roadway approach section.	Spillway (18'±) submerged at time of inspection.

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Yes	Concrete deteriorated
INTAKE STRUCTURE	2 ratchet-operated gates	
OUTLET STRUCTURE	2-18" Ø pipes (submerged)	Presently not being operated. One gate wheel vandalized.
OUTLET CHANNEL	Some minor erosion. Condition of channel under bridge unknown but evidence of scour exists.	
EMERGENCY GATE	None	

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Poor condition. Spillway is circular concrete (all submerged at time of inspection).	
APPROACH CHANNEL	None	
DISCHARGE CHANNEL	Restricted by bridge (low soffit)	
BRIDGE AND PIERS	Abutments spalled. Superstructure satisfactory.	Bridge immediately downstream from dam.

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	None	
APPROACH CHANNEL	None	
DISCHARGE CHANNEL	N/A	
BRIDGE AND PIERS	None	
GATES AND OPERATION EQUIPMENT	None	

SHEET 9

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None	
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	None	

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Grassed; flat	No draw-down facility Sluice gates inoperative
SEDIMENTATION	Heavily silted up	Reservoir capacity considerably reduced by silting.

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	No obstructions Little debris	
SLOPES	Grassed-stable shoreline	
APPROXIMATE NO. OF HOMES AND POPULATION	None within apparent high water levels observed.	

SHEET 1

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
EVANS POND DAM

ITEM	REMARKS
PLAN OF DAM	General Plan avail. (not to scale) (Sketch from Park Commission)
REGIONAL VICINITY MAP	Available
CONSTRUCTION HISTORY	Available
TYPICAL SECTIONS OF DAM	Not available
HYDROLOGIC/HYDRAULIC DATA	Not available
OUTLETS - PLAN	Not available
- DETAILS	Not available
-CONSTRAINTS	Not available
-DISCHARGE RATINGS	Not available
RAINFALL/RESERVOIR RECORDS	Not available

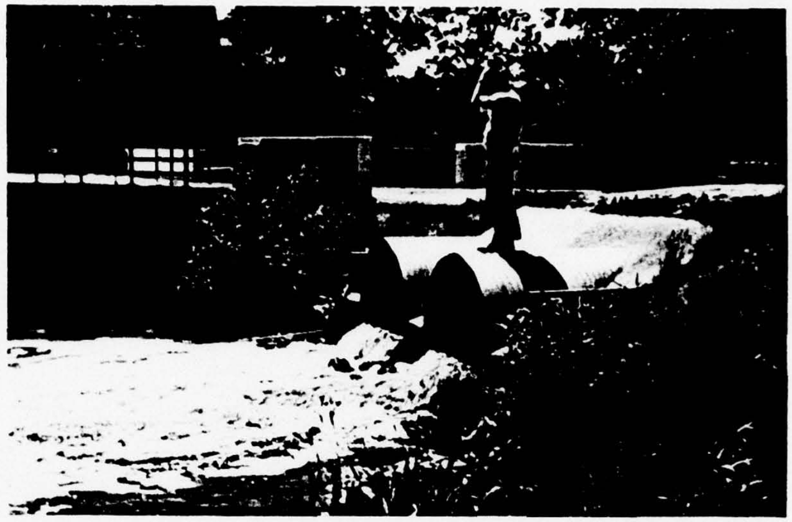
ITEM	REMARKS
DESIGN REPORTS	Richardson report available
GEOLOGY REPORTS	None available
DESIGN COMPUTATIONS	Not available
HYDROLOGY & HYDRAULICS	Not available
DAM STABILITY	Not available
SEEPAGE STUDIES	Not available
MATERIALS INVESTIGATIONS	
BORING RECORDS	Available (See Richardson report)
LABORATORY	
FIELD	
POST-CONSTRUCTION SURVEYS OF DAM	None
BORROW SOURCES.	Unknown

ITEM	REMARKS
MONITORING SYSTEMS	None
MODIFICATIONS	Installation of 2-6'x4' CMPA
HIGH POOL RECORDS	None available
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	Richardson report available
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None
MAINTENANCE OPERATION RECORDS	None available

ITEM	REMARKS
SPILLWAY PLAN	
SECTIONS	None available
DETAILS	None available
OPERATING EQUIPMENT PLANS & DETAILS	None available

Bridge and 2-6'x4' C.M.P.A.
June 1978

Spillway and West Abutment
June 1978



Discharge of Sluices
June 1978

View West towards bridge
June 1978

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: 17.4 sq. miles

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 12.5 (50 acre-ft.)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 17.0 (220 acre-ft.)

ELEVATION MAXIMUM DESIGN POOL: Unknown

ELEVATION TOP DAM: 17.0

CREST:

a. Elevation 12.5
b. Type Earth embankment with concrete spillway
c. Width 25 feet
d. Length 375 feet
e. Location Spillover None
f. Number and Type of Gates 2-ratchet operated gates

OUTLET WORKS: Spillway; Sluices

a. Type Ogee; 2-6'x4' CMPA and 2-18" Ø pipes
b. Location Left abutment (CMPA)
c. Entrance inverts 12.5
d. Exit inverts Unknown
e. Emergency draindown facilities None

HYDROMETEOROLOGICAL GAGES: Gage at Wallworth Dam
a. Type Water-stage recorder
b. Location Approximately 200' east of Kings Highway (Rt. 41)
c. Records 1964 - current

MAXIMUM NON-DAMAGING DISCHARGE: 2750 cfs

BY T.C. DATE 8-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A1 OF

CHKD. BY DATE EVANS POND DAM INSPECTION PROJECT C-222

SUBJECT Precipitation Data - TP-40 -

Data from TP-40 100 year event

Time	*Rainfall	Δ	Rearrange Δ	Cumul. Δ	C.N.-65 R.	ΔR
0.5	2.24	2.24	.13	.13	0	0
1.0	2.94	.70	.14	.27	0	0
1.5	3.49	.55	.17	.44	0	0
2.0	3.91	.42	.18	.62	0	0
2.5	4.10	.19	.18	.80	0	0
3.0	4.28	.18	.19	.99	0	0
3.5	4.46	.18	.70	1.69	0.1	0.1
4.0	4.63	.17	2.24	3.93	1.0	0.9
4.5	4.80	.17	.55	4.48	1.32	0.32
5.0	4.94	.14	.42	4.9	1.6	0.28
5.5	5.08	.14	.17	5.07	1.72	0.12
6.0	5.21	.13	.14	5.21	1.8	0.08

* Corrected for area

Time of concentration

method 1.

L = 8.16 miles H = 130'

$$T_c = \left(\frac{11.9 \times 8.16^3}{130}\right)^{0.385} = 4.50 \text{ hours}$$

method 2. U.S. Navy & Texas Hwy. Dpt. - (Incremental method)

Watercourse (overland negligible)

$$Slp = \frac{130' \times 100}{8.16 \times 5280} = 0.3\%$$ Avg veloc = 2.0 ft/sec

$$T_c = \frac{8.16 \times 5280}{2} \text{ Seconds} = 6.0 \text{ hours}$$

BY D. J. M. DATE 8-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A2 OF
CHKD. BY ______ DATE ______ EVANS POND DAM PROJECT C222
SUBJECT UNITGRAPH HEC 1 INPUT

Time of concentration = 4.5 hours

Lag time = $\frac{0.6\ Tc}{0.85}$ = 3.18

$T_s = 3.18 + \frac{D}{2}$ = 3.43 hours

Drainage area = 17.4 sq miles
DSF (for 1 inch) = 26.89 × 17.4 = 467.9

100/Ts = 29.15

DSF/Ts = 136.41

Time	100/Ts	Dimensionless ordinate	Q (cfs)
0.5	14.58	1.0	136
1.0	29.15	4.2	573
1.5	43.73	9.6	1310
2.0	58.31	15.7	2142
2.5	72.89	19.8	2701
3.0	87.47	20.9	2851
3.5	102.05	19.4	2646
4.0	116.63	16.3	2223
4.5	131.21	13.0	1773
5.0	145.79	10.2	1391
5.5	160.37	8.1	1105
6.0	174.95	6.2	846
6.5	189.53	4.9	655
7.0	204.11	3.7	505
[illegible]	218.69	2.8	382
8.0	233.27	2.2	300
8.5	247.85	1.6	218
9.0	262.43	1.28	175
[illegible]	277.01	0.98	134
10.0	291.59	0.76	104
10.5	306.17	0.59	80
11.0	220.75	0.46	63

BY D. J. M. DATE 8-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A3 OF

CHKD. BY ______ DATE ______ EVANS POND DAM PROJECT C222

SUBJECT UNITGRAPH HEC 1 INPUT

Time	100/t	Dimensionless ordinate	Q
11.5	335.33	0.37	50
12.0	349.91	0.28	38
12.5	364.49	0.24	33
13.0	379.07	0.19	26
13.5	393.65	0.16	22
14.0	408.23	0.14	19
14.5	422.81	0.12	16

L, LAG TIME AS DEFINED BY THE SCS IS THE TIME IN HOURS FROM THE MIDPOINT OF EXCESS RAINFALL, TO THE TIME OF PEAK DISCHARGE.

L, LAG TIME AS DEFINED BY THE BUREAU OF RECLAMATION IS FROM THE CENTRE OF MASS OF RAINFALL TO THE CENTER OF MASS OF RUNOFF.

Tc IS EQUAL TO $\left(\frac{11.9 L^3}{H}\right)^{0.385}$ FROM THE CALIFORNIA CULVERTS PRACTICE

SCS L IS APPROXIMATELY 0.6 Tc

EXAMPLES OF DETERMINING L (LAG) BY BUREAU OF RECLAMATION DEFINITION,

$$L = \frac{T_P - (D/2)}{0.85}$$ WHERE D IS THE TIME INTERVAL OF THE UNITGRAPH

THE SCS CURVELINEAR UNIT HYDROGRAPH CAN BE DERIVED BY FIRST TAKING BUREAU OF RECLAMATION L, (LAG) PLUS $\frac{D}{2}$ AFTER BEING DIVIDED BY 100, THEN MULTIPLIED BY EACH ABSCISSA (IN HOURS) BY THE QUOTIENT. THEN READING THE DIMENSIONLESS ORDINATE FOR THE GIVEN PERCENTAGES FROM THE PREVIOUSLY DETERMINED SCS CURVELINEAR DIMENSIONLESS GRAPH, (COPY ATTACHED)

TO OBTAIN Q IN CFS FOR EACH ORDINATE MULTIPLY EACH DIMENSIONLESS ORDINATE BY A FACTOR OBSERVED FOR ONE INCH,

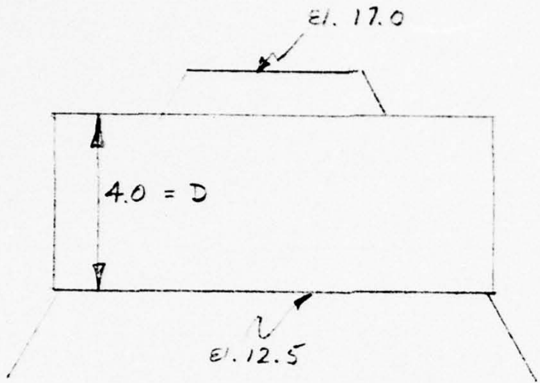
26.89 × AREA

SCS Curvilinear Dimensionless Graph A5

T₀	0	1	2	3	4	5	6	7	8	9
0	0.0	0.0	0.1	0.1	0.2	0.2	0.2	0.3	0.3	0.4
10	0.4	0.5	0.6	0.7	0.9	1.0	1.2	1.3	1.4	1.6
20	1.8	2.0	2.2	2.5	2.8	3.1	3.3	3.6	3.9	4.2
30	4.5	4.8	5.1	5.4	5.8	6.2	6.5	6.9	7.2	7.6
40	8.0	8.4	8.8	9.2	9.6	10.0	10.4	10.8	11.3	11.7
50	12.2	12.6	13.1	13.5	14.0	14.5	14.9	15.2	15.6	16.0
60	16.4	16.7	17.0	17.3	17.7	18.0	18.2	18.5	18.7	19.0
70	19.2	19.4	19.6	19.8	20.0	20.2	20.3	20.4	20.5	20.7
80	20.8	20.8	20.9	20.9	21.0	21.0	20.9	20.9	20.8	20.8
90	20.7	20.6	20.5	20.5	20.4	20.3	20.2	20.1	20.0	19.9
100	19.8	19.6	19.4	19.2	19.1	18.9	18.7	18.5	18.3	18.1
110	17.9	17.6	17.4	17.1	16.9	16.6	16.4	16.2	16.0	15.7
120	15.5	15.2	15.0	14.7	14.5	14.2	14.0	13.8	13.6	13.4
130	13.2	13.0	12.8	12.6	12.4	12.2	12.0	11.8	11.6	11.5
140	11.3	11.1	10.9	10.7	10.5	10.4	10.2	10.0	9.9	9.8
150	9.6	9.4	9.3	9.1	9.0	8.8	8.7	8.5	8.4	8.2
160	8.1	8.0	7.8	7.7	7.5	7.4	7.3	7.2	7.0	6.9
170	6.8	6.7	6.6	6.5	6.4	6.2	6.1	6.0	5.9	5.8
180	5.7	5.6	5.5	5.4	5.3	5.2	5.1	5.0	5.0	4.9
190	4.8	4.7	4.6	4.6	4.5	4.4	4.3	4.2	4.2	4.1
200	4.0	3.9	3.8	3.8	3.7	3.6	3.6	3.5	3.4	3.4
210	3.3	3.2	3.2	3.1	3.1	3.0	3.0	2.9	2.8	2.8
220	2.7	2.7	2.6	2.6	2.6	2.5	2.5	2.4	2.4	2.3
230	2.3	2.2	2.2	2.2	2.1	2.1	2.0	2.0	2.0	1.9
240	1.9	1.8	1.8	1.8	1.7	1.7	1.7	1.6	1.6	1.6
250	1.6	1.5	1.5	1.5	1.4	1.4	1.4	1.4	1.3	1.3
260	1.3	1.3	1.2	1.2	1.2	1.2	1.2	1.1	1.1	1.1
270	1.1	1.0	1.0	1.0	1.0	1.0	1.0	0.9	0.9	0.9
280	0.9	0.9	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7
290	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.6
300	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.5
310	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.4	0.4
320	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
330	0.4	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
340	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2
350	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
360	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
370	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1
380	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
390	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
400	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
410	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
420	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
430	0.1	0.1	0.1	0.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Capacity of sluices

Capacity based on chart 6 from Hydraulic Engineering circular #5



Water level above El. 12.5	Hw/D	Discharge cfs (Q)	For two Sluices (Qx2)
0.5	0.125		
1.0	0.25		
1.5	0.375	24	48
2.0	0.50	40	80
2.5	0.625	59	59
3.0	0.75	80	80
3.5	0.875	103	103
4.0	1.00	124	124
4.5	1.125	150	150

BY D. J. M. DATE 8-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A7

CHKD. BY ______ DATE ______ EVANS POND DAM INSPECTION PROJECT C272

SUBJECT SPILLWAY DISCHARGE

Discharge over crest (EL. 12.5)

L = 78.0' C = ± 3.3

Water level above El. 12.5	Discharge over crest (cfs)	Combined discharges Crest + 2 Sluices
0.5	91	91
1.0	257	257
1.5	472	520
2.0	728	808
2.5	1017	1135
3.0	1337	1497
3.5	1685	1891
4.0	2059	2307
4.5	2457	2737

BY D.J.M DATE 8-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A8 OF
CHKD. BY DATE EVANS POND DAM INSPECTION PROJECT C222
SUBJECT

dam length = 375 feet
crest length = 78 fee

ΔL = 375-78 = 297 feet

assuming 1 feet overtopping of dam

discharge over crest

$c = 3.0 \pm \quad h = 5.5$

$Q = 3.0 \times 5.5^{1.5} \times 78 = 3018$ cfs

discharge over dam :-

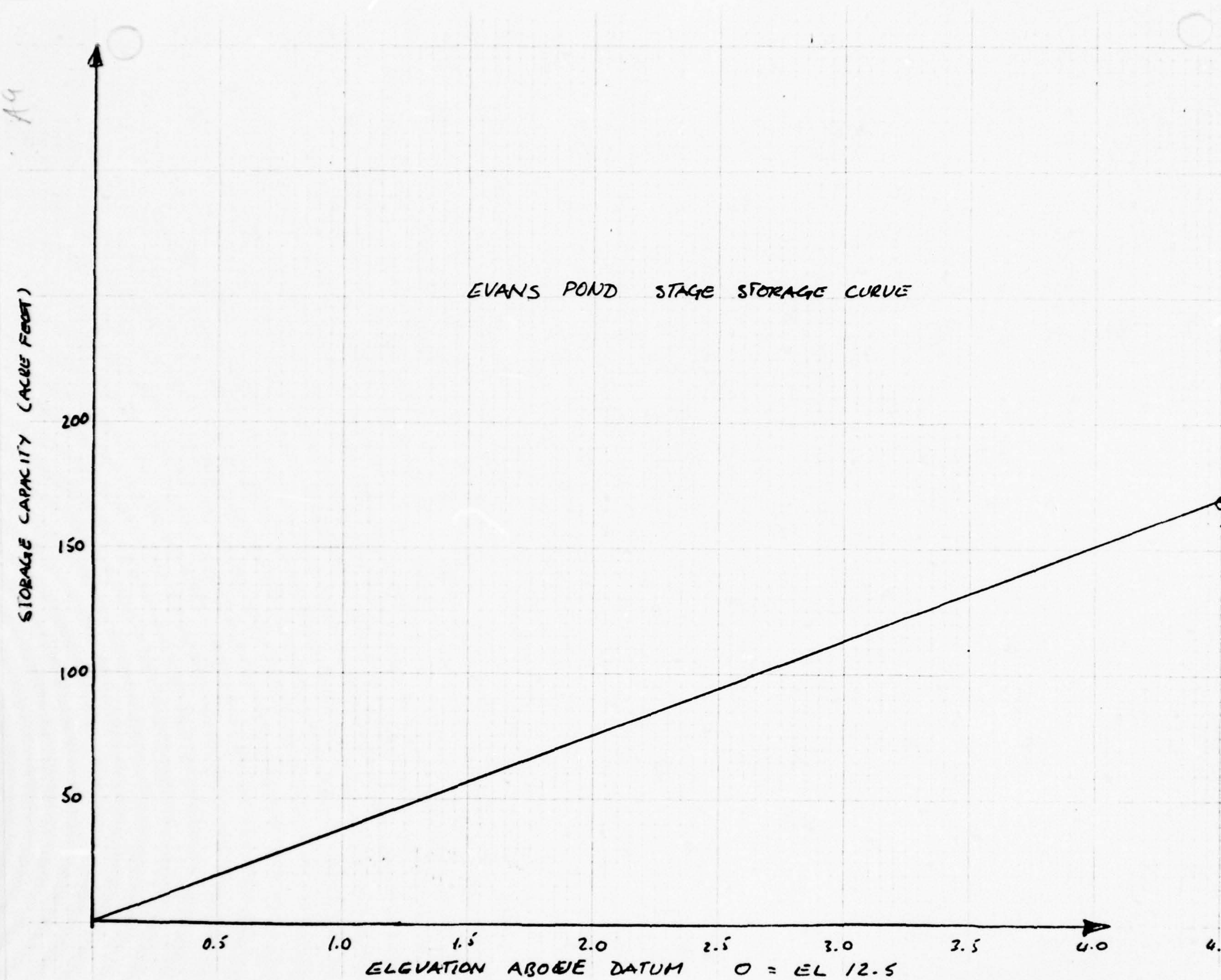
$c = 2.8 \pm \quad h = 1 \quad L = 297$

$Q = 2.8 \times 297 \times 1.0^{1.5} = 832$ cfs

discharge through pipe from attached nomograph ≅ 195 cfs

for two pipes = 390 c.fs.

Total Q at elevation 18.0 = 4240


EVANS POND STAGE STORAGE CURVE
STORAGE CAPACITY (ACRE FEET)
200
150
100
50
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
ELEVATION ABOVE DATUM 0 = EL 12.5

BY D.J.M. DATE ________ **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A10 OF ____
CHKD. BY ______ DATE ________ EVANS POND DAM INSPECTION PROJECT C222
SUBJECT ________

Summary

Δh feet	Total discharge cfs	Storage Acre feet
0	0	0
1.5	520	57
2.0	808	75
2.5	1135	93
3.0	1497	113
3.5	1891	132
4.0	2307	151
4.5	2757	170
5.5	4240	208

These values used in Y2 & Y3 cards of HEC 1 input.

EFFECT OF BRIDGE

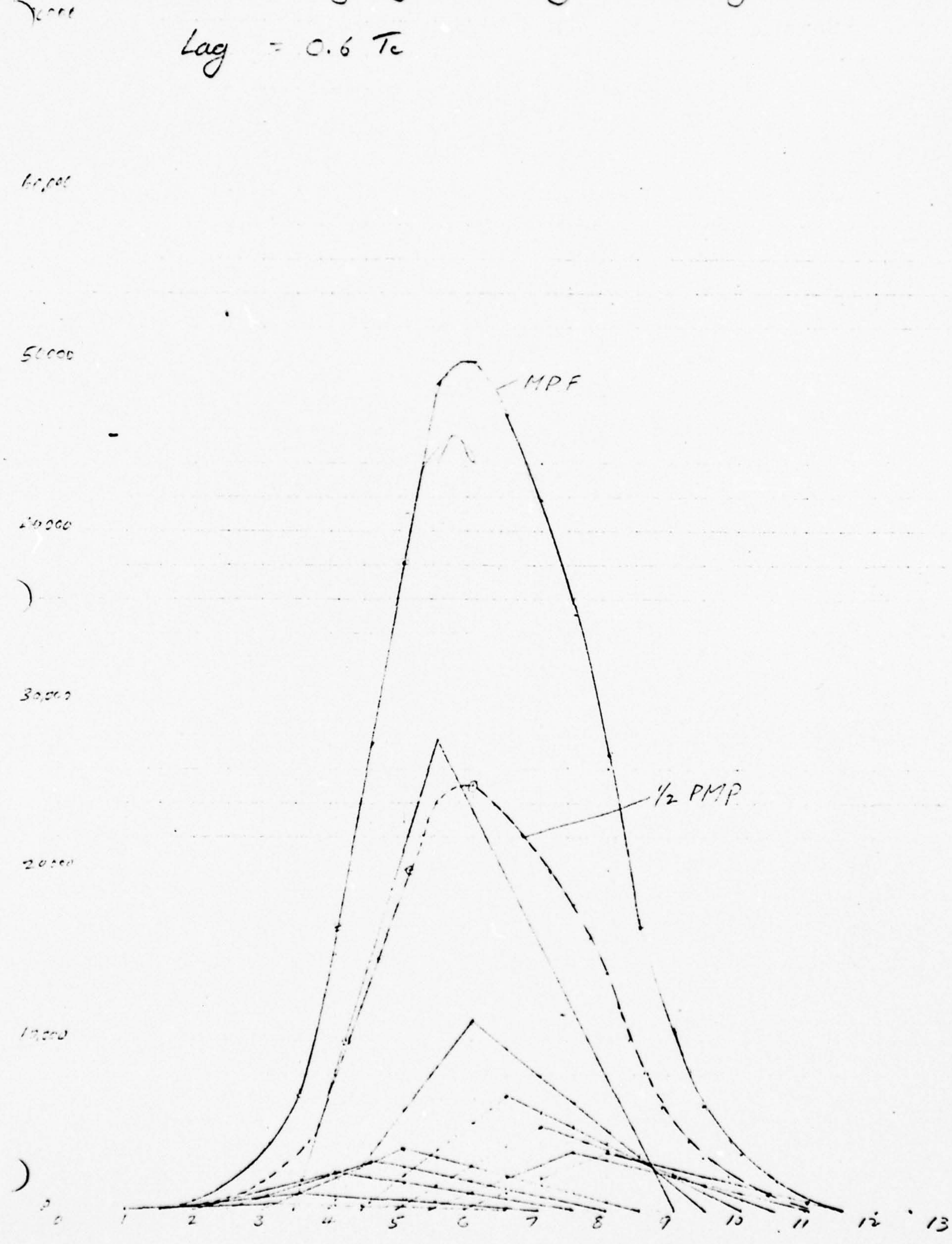
The geometry of the spillway and its areal relationship to the bottom of the bridge precludes the development of a pressure head at peak discharge

CHKD. BY______ DATE________

SUBJECT______ SHEET A 10 A

Manual hydrograph using SCS Lagtime where

Lag = 0.6 Tc



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*****************************
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
*****************************
                    EVANS POND DAM INSPECTION
                    BY D.J.MULLIGAN
                    THURSDAY AUGUST 3RD 1978

                                  JOB SPECIFICATION
                    NQ    NHR   NMIN  IDAY   IHR  IMIN METRC  IPLT  IPRT NSTAN
                    50      0     30     0     0     0     0     0     0     0
                                       JOPER      NWT
                                           3        0

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

                             SUB-AREA RUNOFF COMPUTATION

              INFLOW HYDROGRAPH    (100-YEAR FREQUENCY EVENT)
                    ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME
                        1       0       0       0       2       0       1

                                  HYDROGRAPH DATA
         IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
             0      -1   17.40     0.0   17.40     0.0     0.0       0       0       0

                                    PRECIP DATA
                              NP   STORM     DAJ     DAK
                              12     0.0     0.0     0.0
                                   PRECIP PATTERN
     0.0       0.0       0.0       0.0       0.0       0.0      0.10      0.90      0.32      0.28
    0.12      0.08

                                      LOSS DATA
        STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
          0.0     0.0    1.00     0.0     0.0    1.00     0.0     0.0     0.0     0.0

                             GIVEN UNIT GRAPH, NUHGG=  29
    136.      573.     1310.     2142.     2701.     2851.     2646.     2223.     1773.     1391.
   1105.      846.      655.      505.      382.      300.      218.      175.      134.      104.
     80.       63.       50.       38.       33.       26.       22.       19.       16.
              UNIT GRAPH TOTALS  22517. CFS OR 1.00 INCHES OVER THE AREA

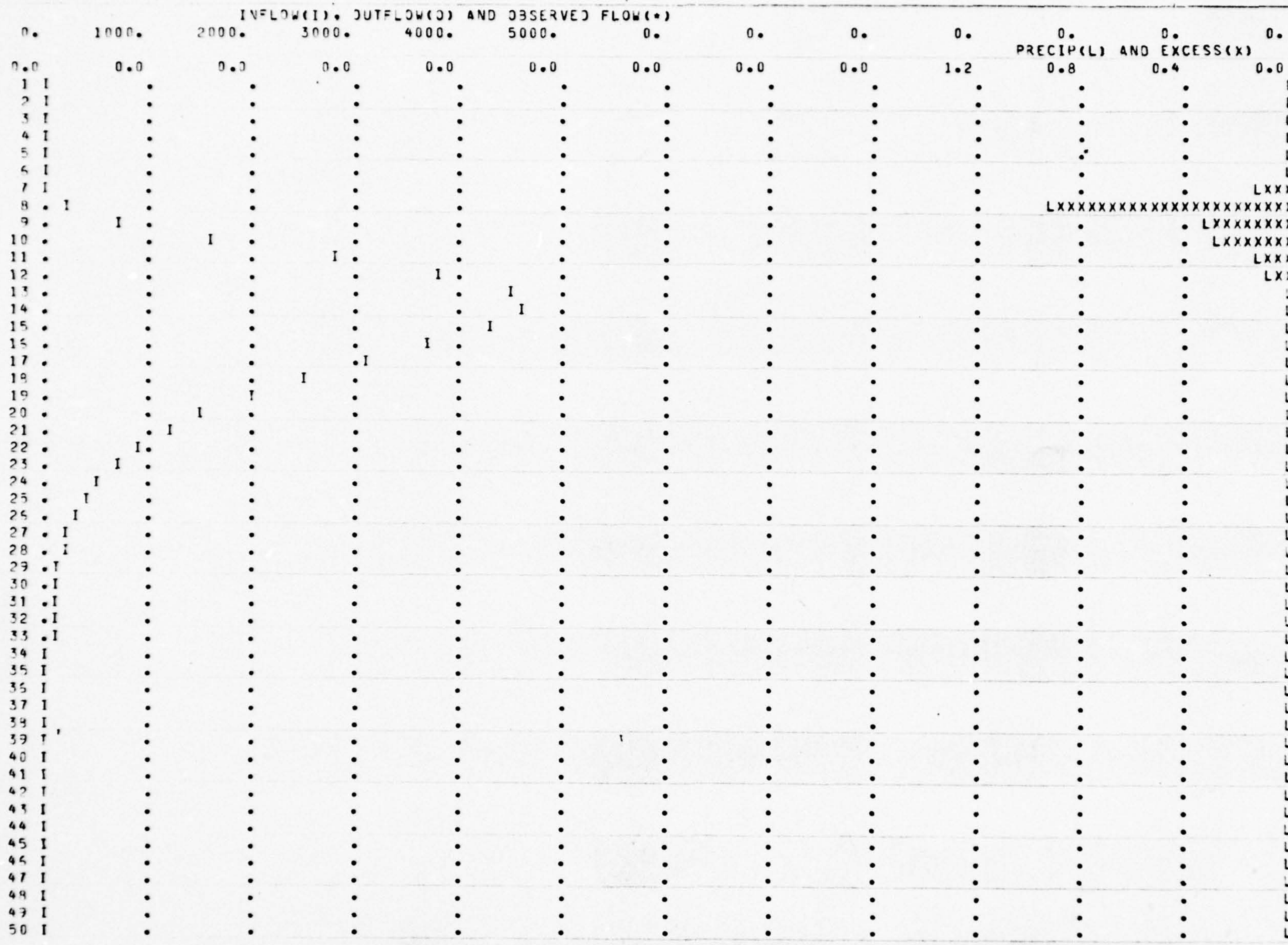
                                   RECESSION DATA
                   STRTQ=      0.0    QRCSV=      0.0     RTIOR= 1.00

                                  END-OF-PERIOD FLOW
                             TIME   RAIN    EXCS      COMP Q
                                1    0.0     0.0          0.
                                2    0.0     0.0          0.
                                3    0.0     0.0          0.
                                4    0.0     0.0          0.
                                5    0.0     0.0          0.
                                6    0.0     0.0          0.
                                7   0.10    0.10         14.
                                8   0.30    0.30        180.
                                9   0.32    0.32        690.
```

10	0.28	0.28	1615.
11	0.12	0.12	2794.
12	0.38	0.38	3848.
13	0.0	0.0	4498.
14	0.0	0.0	4634.
15	0.0	0.0	4318.
16	0.0	0.0	3745.
17	0.0	0.0	3098.
18	0.0	0.0	2499.
19	0.0	0.0	1961.
20	0.0	0.0	1529.
21	0.0	0.0	1183.
22	0.0	0.0	909.
23	0.0	0.0	702.
24	0.0	0.0	530.
25	0.0	0.0	411.
26	0.0	0.0	315.
27	0.0	0.0	244.
28	0.0	0.0	188.
29	0.0	0.0	146.
30	0.0	0.0	115.
31	0.0	0.0	89.
32	0.0	0.0	72.
33	0.0	0.0	58.
34	0.0	0.0	48.
35	0.0	0.0	40.
36	0.0	0.0	32.
37	0.0	0.0	15.
38	0.0	0.0	9.
39	0.0	0.0	3.
40	0.0	0.0	1.
41	0.0	0.0	0.
42	0.0	0.0	0.
43	0.0	0.0	0.
44	0.0	0.0	0.
45	0.0	0.0	0.
46	0.0	0.0	0.
47	0.0	0.0	0.
48	0.0	0.0	0.
49	0.0	0.0	0.
50	0.0	0.0	0.
SUM	1.80	1.80	40533.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4634.	2977.	844.	811.	40531.
INCHES		1.59	1.81	1.81	1.81
AC-FT		1477.	1676.	1676.	1676.

STATION 1



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HYDROGRAPH ROUTING

FLOOD ROUTING

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
11	1	0	0	2	0	1

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.0	0.0	1	0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA
1	0	0	0.0	0.0	0.0	-1.

STORAGE=	0.	57.	75.	93.	113.	132.	151.	170.	208.	0.
OUTFLOW=	0.	520.	808.	1135.	1497.	1891.	2307.	2757.	4240.	0.

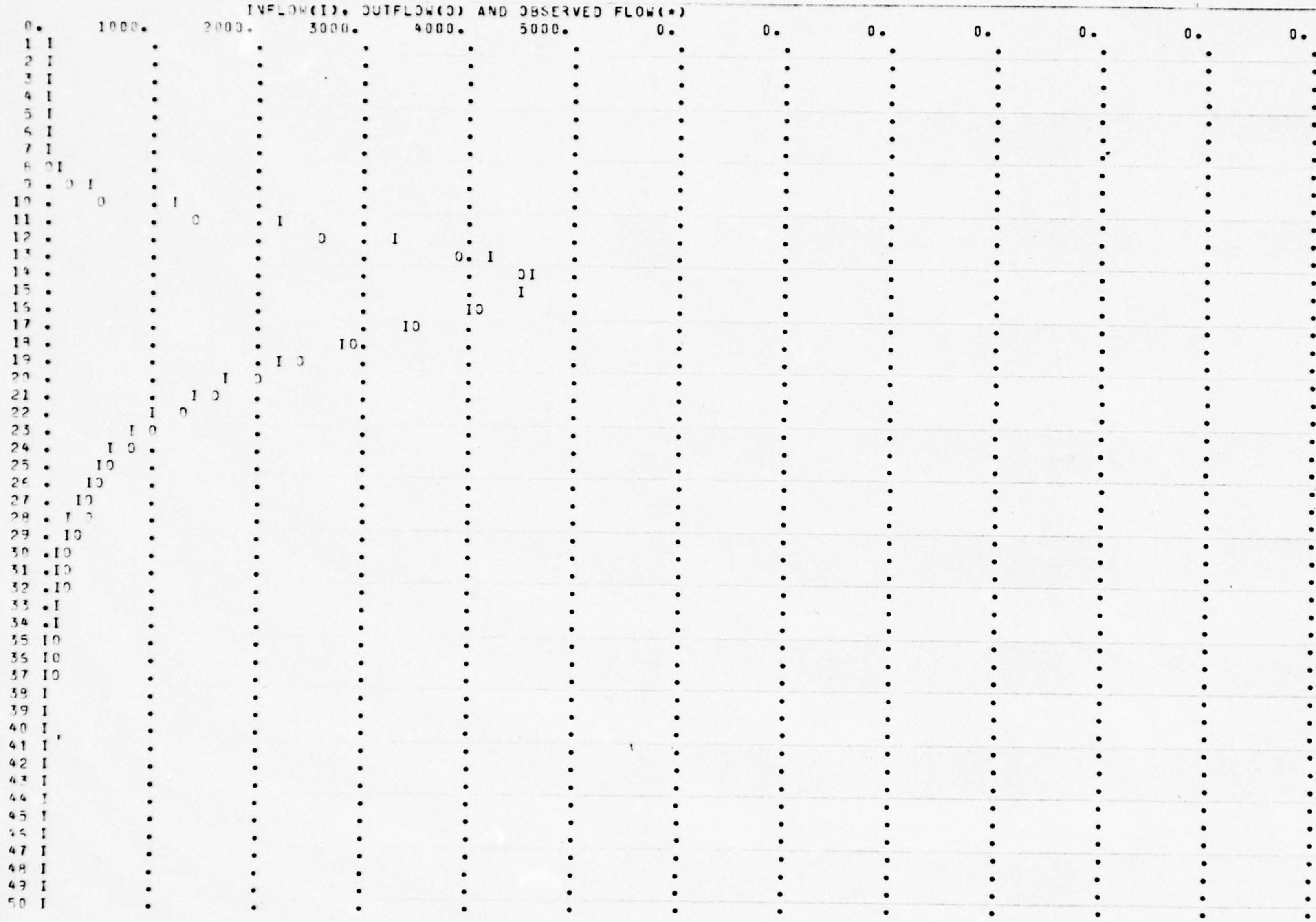
TIME	EOP STOR	AVG IN	EOP OUT
1	0.	0.	0.
2	0.	0.	0.
3	0.	0.	0.
4	0.	0.	0.
5	0.	0.	0.
6	0.	0.	0.
7	0.	7.	2.
8	4.	97.	32.
9	13.	435.	160.
10	52.	1152.	475.
11	105.	2204.	1357.
12	162.	3321.	2558.
13	200.	4173.	3928.
14	215.	4566.	4498.
15	214.	4476.	4479.
16	204.	4032.	4080.
17	189.	3422.	3492.
18	173.	2798.	2873.
19	156.	2230.	2420.
20	137.	1745.	1993.
21	118.	1356.	1607.
22	102.	1046.	1291.
23	87.	805.	1026.
24	75.	616.	803.
25	64.	470.	638.
26	56.	363.	508.
27	48.	279.	435.
28	40.	216.	366.
29	33.	167.	303.
30	27.	131.	248.
31	22.	102.	202.
32	18.	81.	163.
33	14.	65.	132.
34	12.	53.	107.
35	10.	44.	87.
36	8.	36.	71.
37	6.	24.	56.
38	5.	12.	42.
39	3.	6.	31.

40	2.	2.	22.
41	2.	1.	15.
42	1.	0.	10.
43	1.	0.	7.
44	1.	0.	5.
45	0.	0.	3.
46	0.	0.	2.
47	0.	0.	2.
48	0.	0.	1.
49	0.	0.	1.
50	0.	0.	0.
SUM			40529.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	4498.	2881.	844.	811.	40529.
INCHES		1.54	1.81	1.81	1.81
AC-FT		1430.	1676.	1676.	1676.

VF.

STATION 11



RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	4534.	2977.	844.	811.	17.40
ROUTED TO	11	4498.	2881.	844.	811.	17.40

Evan Pond
Fed # 394

INVESTIGATIVE STUDY

OF

EVANS POND DAM

HADDONFIELD, NEW JERSEY

FOR

CAMDEN COUNTY PARK COMMISSION

JULY, 1975

PREPARED BY

EDWARD H. RICHARDSON ASSOCIATES, INC.

CONSULTING ENGINEERS

NEWARK, DELAWARE AND BRIDGETON, NEW JERSEY

1. INTRODUCTION

THE CAMDEN COUNTY PARK COMMISSION HAS BEEN ENGAGED FOR THE LAST SEVERAL YEARS IN AN EFFORT TO UPGRADE PENNYPACKER PARK AND DOWNSTREAM AREAS UNDER THEIR JURISDICTION TO ALLEVIATE FLOODING PROBLEMS AND TO ENHANCE THE DESIRABILITY OF THE PARK FOR PUBLIC USAGE. THE PARK, LOCATED IN A SCENIC REACH OF THE COOPER RIVER BASIN NEAR HADDONFIELD, NEW JERSEY, WAS CONSTRUCTED FOUR DECADES AGO AND SUBSEQUENTLY EXPERIENCED AN EXTENDED PERIOD OF LIMITED MAINTENANCE UNTIL RECENT TIMES.

SEVERAL ROAD WASHOUTS IN RECENT YEARS AT EVANS MILL DAM PROMPTED PARKS COMMISSION PERSONNEL TO UNDERTAKE AN UNDERWATER INVESTIGATION OF THE DAM, SPILLWAY AND ROAD EMBANKMENTS. THE MAGNITUDE AND SEVERITY OF THE DAMAGE FOUND RESULTED IN THE AUTHORIZATION OF THIS INVESTIGATIVE STUDY OF EVANS POND SPILLWAY TO DETERMINE THE IMMEDIATE AND LONG TERM ACTIONS NECESSARY TO INSURE THE INTEGRITY OF THE STRUCTURE AND TO PRESERVE THE PUBLIC SAFETY AND ENJOYMENT OF THE AREA.

2. ANALYSIS OF PRESENT BRIDGE AND SPILLWAY AT EVANS MILL POND

The present spillway structure at Evans Mill Pond, constructed in 1917, is a semicircular ogee weir of eighty feet crest length with a sluiceway at each end of its arc. The spillway is a reinforced concrete structure supported on timber piles. The abutments are incorporated into bridge piers for Evans Mill Road which crosses the pond at this point. The dam (and road) embankment is of earth construction. The bridge is a steel supported timber decked structure only twenty feet in width. The wingwalls and abutments are of reinforced concrete and are supported on piles. Because of extensive sedimentation of the pond above the dam, the water surface is maintained at the elevation of the spillway crest to reduce the possibility of fish kills. The water appears to be of poor quality, possibly resulting from the effluent of several upstream sewage treatment plants entering the pond. The exclusive presence of such coarse fish species as carp tends to verify this judgement.

The findings of the underwater inspection by the Park Commission as presented on their exhibits dated 6-23-75 and 7-18-75 (see back pocket) reveal (NOT AVAILABLE 6/16/78) JK the extent and severity of the damage afflicting the structure. Summarizing those findings briefly, the damage to the concrete portions of the structure consists of extensive major cracks, deterioration of the concrete, exposure of the reinforcing bars and loss of support from undermining. Few areas remain that have not been undermined by loss of soil carried under the

STRUCTURE. SEVERAL AREAS OF SOIL LOSS HAVE RESULTED IN LARGE UNDERMINED AREAS. A SERIES OF UNDERMINED AREAS ALONG THE EAST ABUTMENT HAS THE POTENTIAL OF CONNECTING AT ANY TIME AND PRECIPITATING PARTIAL OR TOTAL FAILURE OF THE STRUCTURE. MANY OF THE PILES HAVE BEEN EXPOSED AS A RESULT OF THE LOSS OF SOIL, BUT THEY APPEAR TO BE IN REASONABLY GOOD CONDITION.

HEAVY SEDIMENTATION OF THE POND HAS ELIMINATED USAGE OF THE RESERVOIRS FLOOD CONTROL VOLUME. FIGURES 1 AND 2 DEMONSTRATE THE DESIRABILITY OF THIS FLOOD CONTROL FEATURE AND POINT UP THE NEED FOR EXTENSIVE DREDGING OF THE POND.

ON THE BASIS OF ON-SITE INSPECTIONS, SOIL BORINGS, THE UNDERWATER INSPECTION, THE PAST HISTORY OF THE STRUCTURE AND PRELIMINARY HYDRAULIC AND STRUCTURAL ANALYSES, IT IS FELT THAT THE EXISTING STRUCTURE IS POTENTIALLY UNSTABLE AND UNSAFE, THAT IT IS BEYOND REASONABLE REPAIR AND THAT IMMEDIATE ACTIONS MUST BE TAKEN TO PROTECT THE DAM AND SPILLWAY AGAINST FURTHER DETERIORATION AND POSSIBLE FAILURE FOR THE MINIMUM POSSIBLE TIME PERIOD IN WHICH A NEW STRUCTURE CAN BE CONSTRUCTED.

AS INTERIM EMERGENCY MEASURES, IT IS RECOMMENDED THAT THE ROAD BE IMMEDIATELY CLOSED TO ALL TRAFFIC AND THAT A SURCHARGE RELIEF NOTCH BE CUT THROUGH THE EASTERLY ROAD EMBANKMENT. AS SHOWN IN FIGURE 3, SURCHARGE NOTCH ILLUSTRATION, FOR ANY GIVEN STORM, A TOTAL SPILLWAY LENGTH OF TWICE THE EXISTING

WOULD RESULT IN A HEAD OVER THE SPILLWAY OF ONE HALF THAT PRESENTLY REALIZED. WATER BEHIND AN EMBANKMENT CREATES PRESSURE DIRECTLY PROPORTIONAL TO THE DEPTH. REDUCING THE DEPTH REDUCES THE TOTAL PRESSURE AGAINST THE EMBANKMENT AND REDUCES THE CHANCES OF FAILURE FOR A WEAKENED STRUCTURE SUCH AS THIS. THE NOTCH SHOULD BE CUT TO AN ELEVATION JUST BELOW THAT OF THE SPILLWAY AND SHOULD BE A MINIMUM OF TWENTY FEET IN BOTTOM WIDTH WITH THREE TO ONE SIDE SLOPES. LINING WITH HOT-MIX ASPHALTIC CONCRETE ON A STONE BASE IS RECOMMENDED TO PREVENT ENLARGEMENT OF THE NOTCH BY FLOWING WATERS (SEE FIGURE 4) WHICH MIGHT THEN TIE IN WITH THE INTERCONNECTING CAVERNS UNDER THE EAST ABUTMENT.

IT IS FURTHER RECOMMENDED THAT THE WATER LEVEL IN THE POND BE LOWERED THE MAXIMUM EXTENT POSSIBLE TO FURTHER REDUCE THE PRESSURE AGAINST THE EMBANKMENT. IT IS REALIZED THAT THIS LOWERING OF THE POND LEVEL INCREASES THE RISK OF SUSTAINING A FISH KILL, BUT ALL MEASURES TO REDUCE THE POSSIBILITY OF FAILURE OF THE STRUCTURE SHOULD BE TAKEN. SHOULD THE STRUCTURE FAIL, ADDED TO THE DEBRIS OF THE STRUCTURE AND THE EMBANKMENT WILL BE LARGE QUANTITIES OF THE HIGHLY ORGANIC MUDS PRESENTLY TRAPPED IN EVANS POND. IN ADDITION TO THE EXPENSE OF THE CLEAN-UP OPERATION, THE EXPOSURE OF THESE MUDS TO AIR WILL RESULT IN AN UNSIGHTLY AND FOUL SMELLING CONDITION. ADDITIONALLY, THE ECOLOGICAL DAMAGE WOULD BE SUBSTANTIAL WITH EXTENSIVE FISH KILLS AND LOSS OF WILDLIFE HABITAT IN BOTH THE UPPER AND LOWER PONDS.

THE EXISTING STRUCTURE IS BEYOND REPAIR. THE DAMAGE TO THE SPILLWAY, ABUTMENTS AND EMBANKMENT ARE SO EXTENSIVE AS TO PRECLUDE REPAIR ATTEMPTS. THE ONLY POSSIBLE REPAIR WOULD INVOLVE DRIVING SHEET PILES UPSTREAM OF THE SPILLWAY FROM BANK TO BANK. THIS WOULD BE AN EXPENSIVE AND NOT ESPECIALLY AESTHETIC PROPOSITION; AND AN OPERATION THAT RUNS THE REAL RISK OF A FAILURE OF THE STRUCTURE INDUCED BY THE INTENSIVE VIBRATIONS THAT COULD BE SET UP DURING THE DRIVING PROCESS. FAILURE COULD OCCUR BY COLLAPSE OF THE SPILLWAY OR EMBANKMENT OR BY LIQUIFICATION OF THE SETTLE MUDS WHICH COULD PUSH UNDER THE SPILLWAY OR EMBANKMENT. SETTLE MUDS TEND TO DEVELOP A DEGREE OF LATERAL COHESIVENESS THAT CONCEIVABLY COULD BE BROKEN BY VIBRATIONS FROM THE PILE DRIVING OPERATION.

3. HYDROLOGIC ANALYSIS

Long duration rainfall records and a short duration streamflow record near the site were correlated to determine an appropriate flood event for design purposes. The maximum flood within the past ten years occurred August 27 - 29, 1971 with a peak discharge of 3,300 cfs. Rainfall records indicate a rainfall on August 27, 1971 of approximately 6.1 inches. The rainfall considered to be of 100 year frequency is in the range of seven to eight inches. The event of August 27, 1971 is, therefore, probably somewhat greater than a fifty year frequency. This event was chosen to be the design condition. (See Figure 5.)

Using the existing structure as a starting point, the design flood event was routed through the Evans Pond spillway and the Wallworth Dam spillway to determine the maximum water surface elevations that could be expected in each pond. The results are shown schematically in Figure 6. On the basis of this routing, substantial depths of flow over the existing bridge deck will occur.

Overtopping of the bridge is caused primarily by a lack of capacity in the box culvert carrying water under the bridge. Backwater from the lower dam extends up to the spillway at Evans Pond and fills this culvert to within a foot of its capacity. Little capacity remains for the flow coming over the spillway; and as a consequency, the storage behind the embankment continues

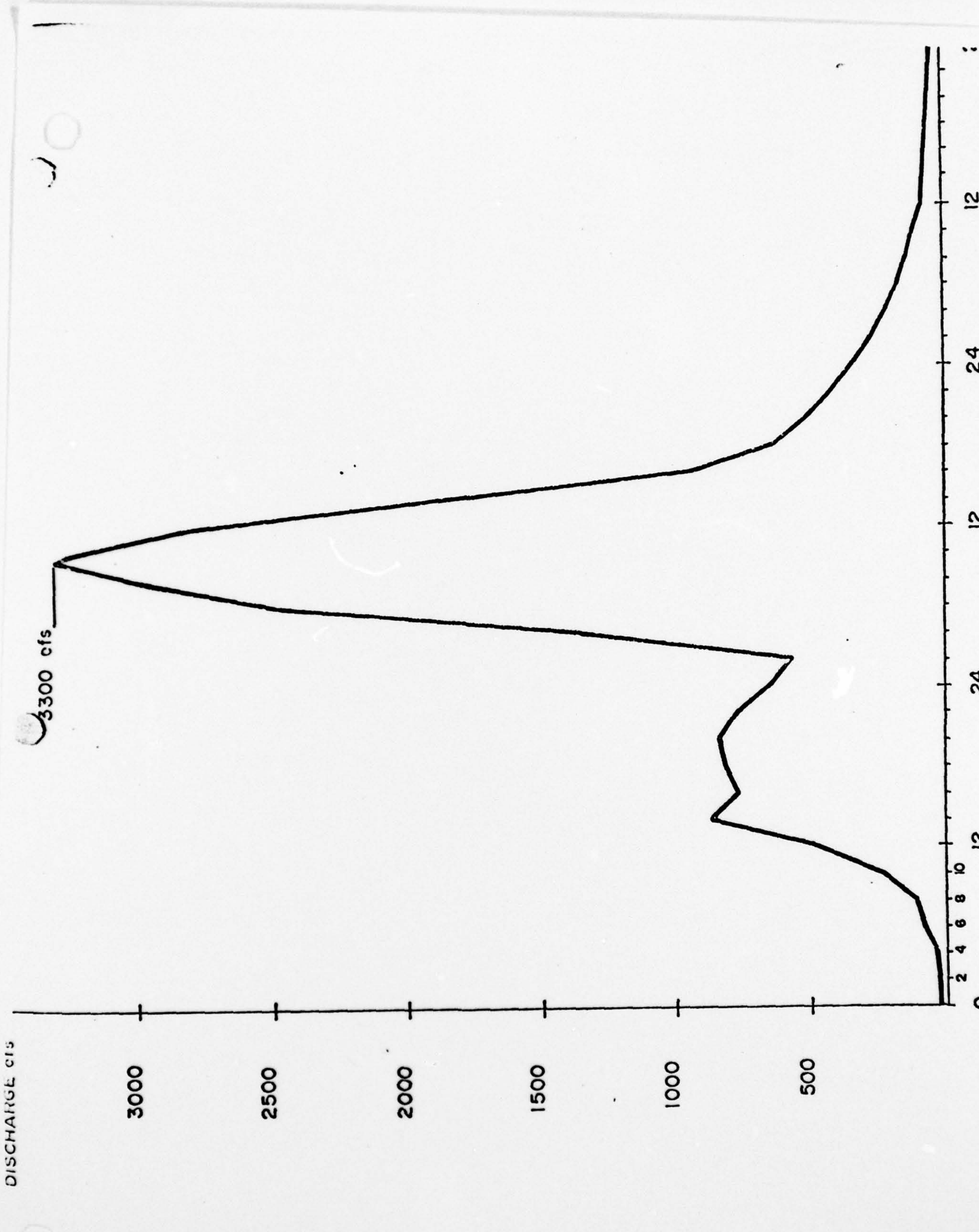
TO INCREASE UNTIL FLOW OVER THE BRIDGE OCCURS. THE DEPTH OF SUBMERGENCE FOR THE DESIGN FLOOD (SEVERAL FEET) IS SUCH THAT RAISING THE ELEVATION OF THE DECK TO REDUCE SUBMERGENCE IS POINTLESS. * ~~THE ELEVATION OF THE DECK OF THE PROPOSED BRIDGE HAS BEEN RAISED APPROXIMATELY A FOOT TO HELP VEHICULAR PASSAGE DURING EVENTS OF LOWER FREQUENCY.~~

THE WATER SURFACE ELEVATION OF THE LOWER POND IS APPROXIMATELY 3.5 FEET BELOW THE ELEVATION OF BORDERING EVANS MILL ROAD, PERMITTING CONSIDERATION OF RAISING THE ELEVATION OF WALLWORTH DAM. THIS OPTION WILL BE DISCUSSED IN THE RECOMMENDATIONS SECTION OF THIS REPORT.

THE ROUTING VERIFIED THAT THE SPILLWAY ITSELF AT EVANS DAM IS HYDRAULICALLY ADEQUATE ALTHOUGH A MORE DETAILED ANALYSIS MAY RECOMMEND A SLIGHTLY LONGER CREST LENGTH. * ~~FOR THE PURPOSES OF THIS PRELIMINARY INVESTIGATION, THE NEW STRUCTURE REFLECTS MANY OF THE DIMENSIONS AND FEATURES OF THE EXISTING STRUCTURE.~~

A CONSIDERABLY MORE DETAILED HYDROLOGIC AND HYDRAULIC ANALYSIS WOULD BE REQUIRED FOR FINAL DESIGN PURPOSES.

* DISREGARD. THE RECOMMENDED CONST. CONTAINED HEREIN WAS NEVER UNDERTAKEN.
JHG 6/16/78



EVAN'S POND - DAM AND SPILLWAY
DISCHARGE HYDROGRAPH
COOPER RIVER AT HADDENFIELD, N.J.
AUGUST 27 - 29, 1971
RESULTING FROM
RAINFALL OF 6.1 INCHES

FIGURE 5

EDWARD H. RICHARDSON ASSOCIATES, INC.
CONSULTING ENGINEERS
NEWARK, DEL.

DATE:			
COMM. NO. 7018	FILE NO.		SCALE:

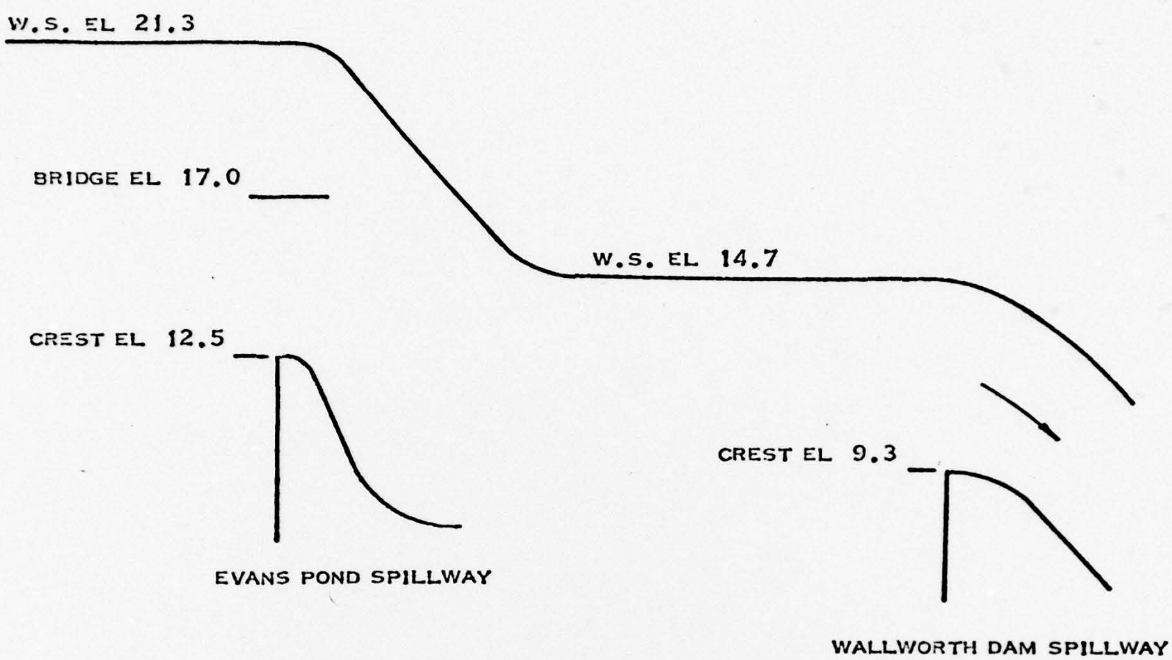


FIGURE 6
WATER SURFACE ELEVATIONS RESULTING FROM
3,300 CFS DISCHARGE

EDWARD H. RICHARDSON ASSOCIATES, INC.
CONSULTING ENGINEERS
NEWARK, DEL.

DATE:		
COMM. NO.	FILE NO.	SCALE:

4. PRELIMINARY SUBSURFACE RECONNAISSANCE

A PRELIMINARY SUBSURFACE RECONNAISSANCE WAS PERFORMED FOR THE PROPOSED RELOCATION OF THE EVANS POND SPILLWAY (SEE APPENDIX).

FOUR (4) TEST BORINGS WERE PERFORMED AT THE SITE. TWO (2) OF THESE TEST BORINGS, NOS. 2 AND 3, WERE PERFORMED ON THE EAST AND WEST SIDES OF THE EXISTING BRIDGE STRUCTURE. TEST BORINGS NOS. 1 AND 4 WERE PERFORMED APPROXIMATELY 150 FT. DOWNSTREAM FROM THE BRIDGE STRUCTURE. THE DEPTH OF TEST BORINGS AVERAGED BETWEEN 30 AND 40 FT.

THE SOILS ENCOUNTERED AT THE SITE MAY BE DESCRIBED BY THE FOLLOWING GENERALIZED PROFILE:

<u>BRIDGE SECTION</u>

AVERAGE RANGE OF STRATA DEPTH	DESCRIPTION
0 – 5'	SANDY AND CLAY SILT-FILL
5 – 20'	DARK GREY ORGANIC SILT CLAY WITH SOME SAND
20 – 23'	DARK GREY MICACEOUS SILTY SAND
23 – 40'	DARK GREY MICACEOUS SILT

<u>DOWNSTREAM SECTION</u>

0 – 7'	SANDY AND CLAY SILT-FILL
7 – 17'	DARK GREY SILTY SAND
17 – 30'+	DARK GREY MICACEOUS SILT

BASED ON THE RESULTS OF THE FIELD TEST BORINGS, VISUAL CLASSIFICATIONS AND LIMITED LABORATORY DATA, THE FOLLOWING CONCLUSIONS ARE DRAWN:

1. AT THE BRIDGE LOCATION, A DEFINITE WEAK ZONE BY THE EMBANKMENT WAS ENCOUNTERED. THIS WEAK ZONE OF LOW STRENGTH IS COINCIDENT WITH THE REPORTED VOIDS BELOW THE STRUCTURE. THEREFORE, IF HYDROSTATIC PRESSURES ARE NOT RELEASED OR REDUCED, THERE IS A POTENTIAL FOR SOIL MOVEMENT IN A LATERAL DIRECTION. ALSO, THE SUPPORT FOR THE OVERBURDEN AND ROADWAY SECTION IS DEFINITELY DECREASING.

2. THE SOILS ENCOUNTERED AT THE DOWNSTREAM AREA REPRESENT A BETTER SUBSURFACE CONDITION FOR STRUCTURE PLACEMENT. THE UNDERLYING SILT MATERIAL IS STIFF AND WILL PROVIDE SUITABLE PILE SUPPORT. ALTHOUGH THE SILT SOILS MAY BE COMPRESSIBLE UNDER ANY PROPOSED EMBANKMENT LOAD, IT MAY BE POSSIBLE TO CONSTRUCT EARTH EMBANKMENTS IN THIS AREA WITH A MINIMUM OF SETTLEMENT.

IN SUMMARY, THE TEST BORINGS AT THE BRIDGE LOCATION SUBSTANTIATE THE VISIBLE POOR CONDITIONS OF THE STRUCTURE AND INDICATE THAT CONTINUED DETERIORATION IS PROBABLY. AT THE DOWNSTREAM LOCATION, THE SOILS ENCOUNTERED REPRESENT A MORE FAVORABLE SUBSURFACE CONDITION FROM A CONSTRUCTION POINT OF VIEW. ONCE A PROPOSED STRUCTURE AND ALIGNMENT HAVE BEEN FINALIZED, ADDITIONAL TEST BORING INFORMATION WILL BE REQUIRED TO DETERMINE DESIGN PARAMETERS BASED ON ACTUAL SUBSURFACE CONDITIONS AT THE PROPOSED SITE.

Drilling Contractor

P. O. BOX 1097, NEWARK, DELAWARE 19711

BORING LOG

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NAME: Evans Pond .... PROJECT NO. [illegible]

Camden County .... SUPERVISOR [illegible]

BORING NO.	DRILLER	DATE
[illegible]B-1	G. Truver	7-3-75
WEATHER	**SURFACE ELEVATION**	**DATUM**
Hot	[illegible]	[illegible]

Sample No.	Sample Depth - Feet From	Sample Depth - Feet To	Depth Strata Feet From	Depth Strata Feet To	Driller's Description of Materials	*Blows A		
			0	1.0	Top Soil			
1	1.0	2.5			Gray Silty Sand w/Tr. Clay	3	4	2
2	2.5	4.0		5.0	Same as above	1	2	2
3	5.0	6.5	5.0	8.0	Brn. & Gray Silty Sand w/Tr. Clay	1	1	1
4	8.5	10.0	8.0	12.0	Brn. Silty Sand w/Tr. Gravel	WT.	H.	1
5	14.0	15.5	12.0	16.5	Dk. Gray Silty Sand w/Tr. Mica	2	6	11
S-1	17.0	19.0	16.5		Dk. Gray Silt Rec. 2.0	SHELBY		
6	19.0	20.5		23.0	Same as above	5	10	15
7	24.0	25.5	23.0		Dk. Gray Silty Sand w/Tr. Mica	4	7	10
8	29.0	30.5			Dk. Gray Silty Sand w/Tr. Shells	7	10	13
9	34.0	35.5			Dk. Gray Silty Sand	7	10	13
10	39.0	40.5		40.5	Dk. Gray Silty Sand	8	12	16

*A Number of blows of 140 lb. hammer dropped 30 in. required to drive 2 in. split-spoon sampler for each of three 6 in. increments.

*B Number of blows of 300 lb. hammer dropped 18 in. required to drive .... in. casing 12 inches.

REMARKS: 24.0-40.5 Very Stiff

GROUND WATER		
W/L	5.0	
Caved	3.5	

CASING B: 0-1, 1-2, 2-3, 3-4, 4-5, 5-6, 6-7, 7-8, 8-9, 9-10, 10-11, 11-12, 12-13, 13-14, 14-15, 15-16, 16-17, 17-18, 18-19, 19-20, 20-21, 21-22, 22-23, 23-24, 24-25, 25-26, 26-27, 27-28, 28-29, 29-30, 30-31, 31-32, 32-33, 33-34, 34-35, 35-36, 36-37, 37-38, 38-39, 39-40, 40-41, 41-42, 42-43, 43-44, 44-45, 45-46, 46-47, 47-48, 48-49, 49-50, 50-51, 51-52, 52-53, 53-54, 54-55, 55-56, 56-57, 57-58, 58-59, 59-60, 60-61

WALTON CORPORATION
Drilling Contractor

P. O. BOX 1097, NEWARK, DELAWARE 19711

BORING LOG

ME Evans Pond .... PROJECT NO. [illegible]

Camden Co., N.J. .... SUPERVISOR [illegible]

RING NO.	DRILLER	DATE
2	G. Truver	7-3-75
ATHER	**SURFACE ELEVATION**	**DATUM**
Hot	[illegible]	[illegible]

mple No.	Sample Depth - Feet From	Sample Depth - Feet To	Depth Strata Feet From	Depth Strata Feet To	Driller's Description of Materials	*Blows A		
			0	0.5	Blacktop			
1	0.5	2.0	0.5	2.0	Brn. Fine Sand	5	5	9
2	2.5	4.0	2.0	3.0	Brn. Sandy Silt	4	6	4
			3.0	4.5	Brn. Sand (Fine)			
3	5.0	6.5	4.5		Brn. Sandy Silt(Void)	1	1	2
4	8.5	10.0			Brn. Sandy Silt (Void)	WT.	H.	1
5	14.0	15.5		18.0	Brn. & Gray Silt & Sand(Void)	4	2	2
6	19.0	20.5	18.0		Dk. Green Silty Sand w/Mica	5	5	8
	24.0	25.5		28.0	Dk. Green Silty Sand w/Mica	5	9	11
[illegible]	29.0	30.5	28.0	32.0	Sand Lost Most of Sample	7	12	17
9	34.0	35.5	32.0		Dk. Gray Sandy Silt w/Mica	4	6	11
1	37.0	39.0			Dk. Gray Sandy Silt Rec. 2.0	SHELBY		
10	39.0	40.5		40.5	" " " "	6	8	12

BLOWS O CASING I
0- 1
1- 2
2- 3
3- 4
4- 5
5- 6
6- 7
7- 8
8- 9
9-10
10-11
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51-52
52-53
53-54
54-55
55-56
56-57
57-58
58-59
59-60

umber of blows of 140 lb. hammer dropped 30 in. required to drive 2 in. split-spoon sampler for each of three increments.

umber of blows of 300 lb. hammer dropped 18 in. required to drive .... in. casing 12 inches.

RKS: 1.0 Void in Sample #3

1.0 " " " #4

[illegible] Lost Most Sample

GROUND WATER

WALTON CORPORATION
Drilling Contractor
P. O. BOX 1097, NEWARK, DELAWARE 19711

BORING LOG

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.ME Evans Pond .... PROJECT NO. [illegible]

Camden Co., N. J. .... SUPERVISOR [illegible]

ORING NO.	DRILLER	DATE
T33	G. Truver	7-7-75
EATHER	**SURFACE ELEVATION**	**DATUM**
Cloudy & Cool	[illegible]	[illegible]

ample No.	Sample Depth - Feet From	Sample Depth - Feet To	Depth Strata Feet From	Depth Strata Feet To	Driller's Description of Materials	*Blows A		
	0	0.5	0	0.5	Blacktop			
1	0.5	2.0	0.5	3.0	Brn. F/M Sand	13	6	8
2	2.5	4.0	3.0	5.0	Brn. Clayey Fine Sand	6	3	3
3	5.0	6.5	5.0	8.5	Green Fine Sand	Wt.	Hammer	
4	8.5	10.0	8.5		Blk. Silty Sand		1	H
5	14.0	15.5		18.5	Black Silty Sand	Wt.	H	1
6	19.0	20.5	18.5		Blk. & Gray Sandy Silt	Wt.	of	H.
7	24.0	25.5		25.0	Blue/Green Sand w/Tr. Gravel	1	4	2
			25.0	28.5	Brn. Sandy Silt			
8	29.0	30.5	28.5		Blk. Sandy Silt w/Mica	7	8	12
9	34.0	35.5		35.5	Blk. Sandy Silt w/Mica & Shells	5	10	11
					(Very Stiff)			

BLOWS C CASING
0- 1
1- 2
2- 3
3- 4
4- 5
5- 6
6- 7
7- 8
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*A Number of blows of 140 lb. hammer dropped 30 in. required to drive 2 in. split-spoon sampler for each of three 6 in. increments.

*B Number of blows of 300 lb. hammer dropped 18 in. required to drive .... in. casing 12 inches.

MARKS: 24.0-25.5 Augers Dropped to Ground Surface
29.0-30.5 Had to wash out because of 5' Heave
34.0 35.5 Washed Out

GROUND WATER

5.5		
11.0	Caved	

WALTON CORPORATION

Drilling Contractor

P. O. BOX 1097, NEWARK, DELAWARE 19711

BORING LOG

[illegible]E ..... Evans Pond ..... PROJECT NO. 7018

..... Camden Co. N.J. ..... SUPERVISOR F.A.S.

[illegible]RING NO. 4	DRILLER G. Truver	DATE 7-7-75
[illegible]ATHER Cloudy & Hot	SURFACE ELEVATION [illegible]	DATUM [illegible]

[illegible]mple No.	Sample Depth - Feet From	Sample Depth - Feet To	Depth Strata Feet From	Depth Strata Feet To	Driller's Description of Materials	*Blows A		
			0	0.5	Top Soil			
1	0.5	2.0	0.5	2.0	Brn. Fine Sand w/Silt	3	3	3
					Sandy Clay			
2	2.5	4.0	2.0	5.0	Green/Gray Silty Fine Sand	1	1	
3	5.0	6.5	5.0	8.0	Brn. Clayey Sand(F) Wood Chunks)	4	1	3
4	8.5	10.0	8.0	12.0	Silty Fine Sand	2	1	1
5	14.0	15.5	12.0	18.0	Gray M Sand w/Tr. Silt	1	4	1
6	19.0	20.5	18.0		Dk. Gray Sandy Silt w/Mica(Shell)	3	5	8
7	24.0	25.5			Same as above	3	6	10
8	29.0	30.5		30.5	Same as above	5	7	10

BLOWS ON CASING B
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1- 2
2- 3
3- 4
4- 5
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*Number of blows of 140 lb. hammer dropped 30 in. required to drive 2 in. split-spoon sampler for each of three increments.

Number of blows of 300 lb. hammer dropped 18 in. required to drive .... in. casing 12 inches.

[illegible] ..... 14.0-15.5 Wash Out .....

..... [illegible] Accumulation 4.0 .....

[illegible] 10.0 .....

GROUND WATER

5. RECOMMENDATION

On the basis of this preliminary investigation, it is recommended that a replacement structure be constructed to replace the existing unsafe structure at Evans Dam.* The soil borings indicate better subsurface conditions downstream of the present site and the downstream relocation would allow the present structure to perform the function of a cofferdam during construction of the new structure. This cofferdam role is believed safe assuming the recommended emergency measures are performed.

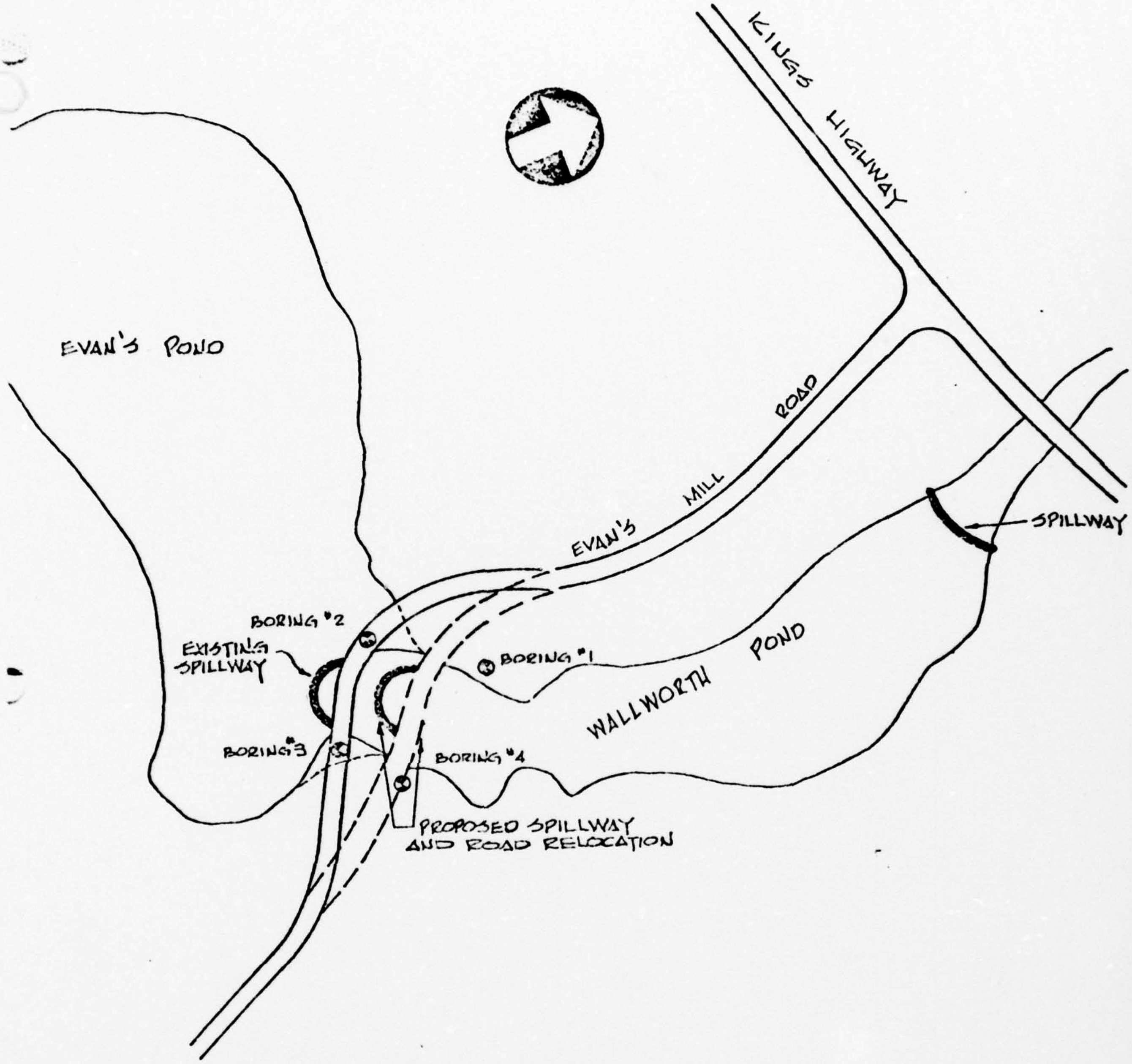
As shown on Figure 7, the proposed replacement structure and roadway would be located approximately one hundred feet downstream (north) of the present site. This relocation provides for a smoother road alignment as well as better subsurface conditions to minimize foundation costs. Two eleven foot wide travel lanes, a four foot shoulder on both sides and an adjoining six foot bicycle/pedestrian way on the spillway side make up the proposed roadway section. (See Figure 8.) Although somewhat narrower than the twelve foot travelway/ten foot shoulder combination recommended by AASHO, the new road will be a major improvement over the existing road in safety and convenience for all classes of users. The shoulders and bicycle/pedestrian path shall be carried across the proposed replacement structure. Cable guard rail, which affords a more unobstructed view of the ponds, is recommended for both sides of the proposed roadway for the full embankment length.

* Not implemented. JHJ

IT WILL BE NECESSARY TO DREDGE EVANS POND SUBSTANTIALLY TO REGAIN THE VOLUME LOST TO SEDIMENTATION. THIS WILL ALLOW FOR THE UTILIZATION OF ADDITIONAL FLOOD STORAGE AND WILL ENHANCE WATER QUALITY OVER A PERIOD OF TIME.

AN ALTERNATIVE TO REPLACING THE EVANS POND STRUCTURE WOULD CONSIST OF RAISING THE WALLWORTH DAM APPROXIMATELY TWO FEET AND REMOVING THE SPILLWAY AT EVANS PONT. EVANS MILL ROAD WOULD STILL BE MORE THAN A FOOT ABOVE THE ELEVATION OF THE FIFTY YEAR FLOOD. EVANS POND WOULD HAVE A NORMAL WATER SURFACE APPROXIMATELY A FOOT LOWER THAN AT PRESENT. HYDROSTATIC PRESSURE AGAINST THE EMBANKMENT WOULD BE ELIMINATED FOR DAILY FLOWS AND GREATLY REDUCED FOR FLOOD FLOWS.

THE DISADVANTAGES TO THIS ALTERNATIVE ARE: THAT EXTENSIVE REMEDIAL WORK WOULD STILL BE REQUIRED ON THE ROADWAY EMBANKMENT; MORE DREDGING WOULD BE REQUIRED BECAUSE OF THE LOWER WATER SURFACE; SOME PRESENTLY UTILIZED LANDS ALONG THE LOWER POND WOULD BE SUBMERGED; THE FLOOD PEAK ATTENUATION EFFECT POSSIBLE WITH TWO PONDS WOULD BE REDUCED; AND THE FIFTY YEAR TRADITION OF TWIN PONDS WOULD BE DESTROYED. THE PREFERABLE COURSE WOULD SEEM TO BE TO REPLACE THE EVANS POND STRUCTURE.



EVAN'S POND - DAM AND SPILLWAY PROPOSED SPILLWAY AND ROAD RELOCATION FIGURE 7	EDWARD H. RICHARDSON ASSOCIATES, INC CONSULTING ENGINEERS NEWARK, DEL. DATE: COMM. NO. 7018 — FILE NO. — SCALE:

DRAWN BY______ CHECKED BY______ Book______ Page______