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NATIONAL DAM SAFETY PROGRAM. WESTONS MILLS ARCH DAM (NJ 00382),--ETC(U)
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RARITAN RIVER BASIN

LAWRENCE BROOK, MIDDLESEX COUNTY

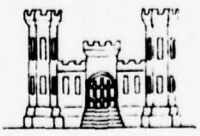
NEW JERSEY

WESTONS MILLS ARCH DAM

PHASE I INSPECTION REPORT

NATIONAL DAM SAFETY PROGRAM

NJ 00382



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

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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Middlesex County, N.J.

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Copies are obtainable from National Technical Information Service, Springfield, Virginia, 22151.

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National Dam Safety Program Phase I
Dam Safety
Westons Mills Arch Dam N.J.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

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 - 2D & CHESTNUT STREETS
PHILADELPHIA, PENNSYLVANIA 19106

IN REPLY REFER TO

NAPEN-D

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

30 AUG 1978

Dear Governor Byrne:

Inclosed is the Phase I Inspection Report for Westons Mills Arch Dam in Middlesex 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 four pages of the report.

Based on visual inspection, available records, calculations and past operational performance, Wastons Mills Arch Dam, initially listed as a high hazard potential structure but reduced to a significant hazard potential structure as a result of this inspection, is judged to be in fair overall condition. However, the dam's spillway is considered seriously inadequate since 9 percent of the Spillway Design Flood (SDF) would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

a. The spillway's adequacy should be determined by a qualified professional consultant engaged by the owner using more precise and sophisticated methods procedures and studies within six months from the date of approval of this report. Any practicable remedial measures necessary to insure the adequacy of the spillway and to prevent overtopping should be initiated within calendar year 1979.

b. Within six months of the date of approval of this report, the owner should initiate necessary engineering studies to determine the foundation conditions and assess the structural stability of the dam. The studies should address the possible effects of the collapse of the dam downstream on the subject dam. These studies should also include a determination of the need to repair the destroyed sluice gate. Any remedial measures found necessary, should be initiated in calendar year 1979.

c. During the next period of low flow, the reservoir should be drawn down as necessary to investigate the extent of spalling and surface deterioration of the concrete. Any remedial actions found necessary, should be initiated within six months of the drawdown.

d. Within one year of the date of approval of this report, the following actions should be initiated.

(1) Berms behind the abutments should be raised to at least abutment height and both the berms and abutments should be furnished with slope protection.

(2) Large debris and fallen trees should be removed from the upstream channel and reservoir.

(3) Initiate a system for recording operation and maintenance procedures.

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 Frank Thompson, Jr. and Edward Patton of the Fourth and Fifteenth District, 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

WESTONS MILLS ARCH DAM (NJ00382)

CORPS OF ENGINEERS ASSESSMENT OF GENERAL CONDITIONS

This dam was inspected on 14 and 17 June 1978 and 19 July 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 92-367.

Westons Mills Arch Dam, initially listed as a high hazard potential structure but reduced to a significant hazard potential structure as a result of this inspection, is judged to be in fair overall condition. However, the dam's spillway is considered seriously inadequate since 9 percent of the Spillway Design Flood (SDF) would overtop the dam. To insure adequacy of the structure, the following actions, as a minimum, are recommended:

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(3) Initiate a system for recording operation and maintenance procedures.

APPROVED: James G. Ton

JAMES G. TON
Colonel, Corps of Engineers
District Engineer

DATE: 30 Aug 78

PHASE I REPORT
NATIONAL DAM INSPECTION PROGRAM

Name of Dam Westons Mills Arch Dam NJ 00382

State New Jersey
County Located Middlesex
Coordinates Lat.4029.0 - Long.7424.9
Stream Lawrence Brook
Date of Inspection 19 June 1978

ASSESSMENT OF GENERAL CONDITIONS

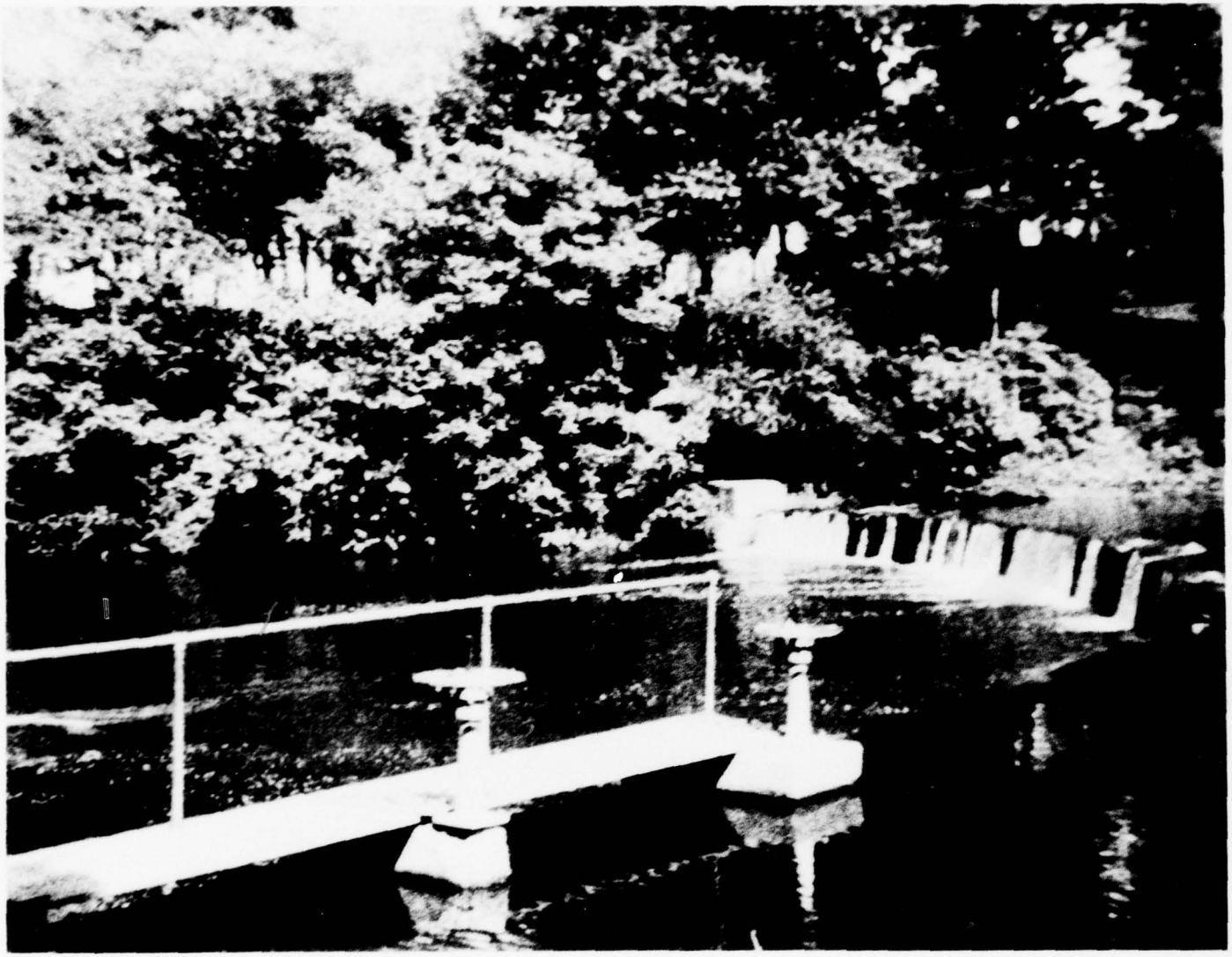
Westons Mills Arch Dam is in fair condition but the spillway is seriously inadequate. Little engineering information is available and it is recommended that the owner provide, in the near future, detailed foundation investigations and engineering studies. Remedial actions recommended are:

- Construction of slope protection around the abutments.
- Removal of large debris in the upstream channel.

The spillway capacity for the downgraded significant hazard category is 8 percent of the design flood. No appreciable improvement can be made to the existing spillway capacity. A collapse of the Lower Dam 600 feet downstream could endanger the Westons Mills Arch Dam by causing a sweepout of the downstream channel riprap.

F. Keith Jolls P.E.
Project Manager

Rudolph Wrubel
Vice President, Engineering



JUNE 1978

OVERVIEW OF WESTONS MILLS ARCH DAM

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PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
NAME OF DAM: WESTONS MILLS ARCH DAM NJ 00382

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 Corps of Engineers, 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 Westons Mills Arch 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

Westons Mills Arch Dam is a unreinforced concrete arch dam with three sluice gates on the west end. Each gate consists of a 30 inch cast iron pipe with a hand cranked sluice gate bolted to its face. The spillway extends across the entire length of the crest. The crest of the dam is 200 feet long, with a radius of 160 feet. At each end there is a concrete abutment 24 feet by 10 feet with a top elevation 2.5 feet above the spillway

crest. The height from spillway crest to the downstream channel invert is about 9 feet.

b. Location

Westons Mills Arch dam is located at Westons Mills, in the City of New Brunswick, Middlesex County: 0.5 miles northwest of Interchange 9 of the New Jersey Turnpike. It is immediately south of the concrete arch bridge carrying Route 18 over Lawrence Brook.

c. Size Classification

The maximum height of the dam is approximately 17 feet and the conservation storage is estimated to be 1050 acre feet. Therefore the dam is in the intermediate size category.

d. Hazard Classification

The dam was originally classified as a high hazard by the Corps of Engineers but as a result of this inspection, it is recommended that it be downgraded to a significant hazard classification. The town of Westons Mills lies immediately downstream; however, the residential areas are approximately 20 to 30 feet above the elevation of the dam and should failure occur, it appears there would be only minor property damage, principally involving boating facilities. The bridge to the immediate north (on Burnet Street) which spans Lawrence Brook and the Route 18 bridge just below the dam would probably not be harmed should this dam fail.

e. Ownership

The dam is owned by the City of New Brunswick, City Hall, 78 Bayard Street, New Brunswick, New Jersey 08903.

f. Purpose of Dam

The dam is used to increase the reservoir capacity for the city water supply system.

g. Design and Construction History

Westons Mills Arch Dam was designed by F. W. Schwiers Co. and constructed by the B. C. Coon Construction Company of Luzerne, Pennsylvania. Construction was completed in January 1919.

h. Normal Operating Procedures

A water supply intake dam exists 600 feet downstream and the only operations carried on at Westons Mills Arch dam is to regulate the lower reservoir elevation during periods of low flow.

1.3 PERTINENT DATA

a. Drainage Area

The drainage area for the Westons Mills Arch Dam is 42.0 square miles.

b. Total spillway capacity at maximum pool elevation - 2290 c.f.s.

c. Elevation (M.S.L.)

Top of dam	-	21.09
Maximum pool	-	21.09
Recreation	-	18.43

d. Reservoir

Length of maximum pool	-	13500 feet
Length of recreation pool	-	12600 feet

e. Storage

Recreation pool	-	1050 acre feet
Top of dam	-	1600 acre feet

f. Reservoir Surface

Top of dam	-	180 acres
Maximum pool	-	180 acres
Recreation pool	-	160 acres
Spillway crest	-	160 acres

g. Dam

Type - Concrete arch dam
Length - 248 feet
Height - 17 feet (9 feet - spillway crest to downstream invert)
Top width - 3 feet
Zoning - No zoning information available

h. Diversion and Regulating Tunnel

None

i. Spillway

Type - narrow crest (radial)
Length of weir - 200 feet
Crest elevation - 18.43 feet

j. Regulating Outlets

Three 30 inch diameter pipes with sluice gates attached.

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

The information available for review of the Westons Mills Arch Dam included:

1) Drawing entitled "Plan for Arched Dam" dated June 19, 1917, together with bidding documents and specifications (partially complete).

2) 1917-1919 correspondence between the City Engineer and the Consulting Engineer (numerous letters).

3) Partial stress calculations made by Consulting Engineer, F. W. Schwiers Jr. Co., 90 West Street, New York.

4) Photographs of dam during and after construction, 1918-1919.

2.2 CONSTRUCTION

The information regarding the original construction included photographs, progress reports and correspondence between the City Engineer and Consulting Engineer indicates the work was carried on in a controlled workmanlike fashion. The dam was built by the B. C. Coon Construction Company of Luzerne, Pennsylvania. An additional 18 inches of height was added to the contract plan height during construction when it was recorded that the rock bedding on which the dam is founded was located about one foot higher than the original plan elevation.

The dam immediately downstream was constructed prior to the study dam and replaced a wood structure which burned down.

2.3 OPERATION

See Section 4. An inspection was ordered by the New Jersey Bureau of Water Control in 1968 but according to Bureau correspondence of 3 August 1973 to Robert

C. Kane, City Engineer, the inspection had not been carried out as of that date. No records of subsequent inspections were located.

2.4 EVALUATION

a. Availability

The original engineering data reviewed indicates that the construction was carefully prosecuted and proper supervision was in evidence. Additional information required for a complete evaluation should include:

1) Concrete cores for strength evaluation.

2) Visual inspection (dewatering or with divers) of the dam foundations.

b. Adequacy

The concrete mix specified was 1:2.5:5 and in light of the visible portions viewed during the field inspection, was properly mixed and placed. No attempt was formulated in this phase regarding the present compressive strength. From a summary of quantities in the 1917 bid documents, the arch structure is not reinforced, hence shrinkage cracks could be expected in the arch ring.

The foundation conditions for this dam consist of a shallow depth of fine granular and fine grained material overlying shale bedrock. The dam is founded on the shale according to the design plans. It is unknown whether the base was keyed into the rock of cast directly on the surface.

The overburden soil is a silty sand to sandy silt with varying amounts of intermixed gravel. The depth to bedrock is estimated at less than ten feet and is described in general as thin to thick beds of soft shale, with occasional beds of fine-grained sandstone, all dipping gently to the northwest. The New Brunswick shales weather readily into small fragments and these quickly revert to silt and clay sized particles.

c. Validity

Based upon field observations, the existing engineering data appears valid insofar as the existing structure's configuration and condition. See Section 6 for comment on the structural stability regarding the width/height ratio.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General

On-site inspections of the dam took place on June 14 and 27 and July 19, 1978. Although there are 1968 records of the NJDEP requesting the City of New Brunswick to inspect the dam, no evidence that this inspection was undertaken were available.

b. Dam

The concrete arch is in fairly good condition but appears to be stabilized to a considerable degree by heavy siltation and dumped riprap on both the upstream and downstream sides. There is evidence of minor reconstruction and asphalt patching of the berm on the upstream side of the west abutment. There was no evidence of major fractures, or pieces missing from the crest of the dam. The location or condition of construction joints in the crest arch are unknown as they were underwater at the time of inspection. There is considerable spalling and surficial deterioration of the exposed concrete. The 19 July reinspection was held to verify this condition and the material in the downstream channel.

c. Appurtenant Structures

One of the three 30 inch sluice gates appears to be destroyed but the remaining two are operable to control reservoir elevation for the water intake dam immediately downstream.

d. Reservoir Area

There is a fair amount of debris and several large fallen trees in both the upstream and downstream reservoirs. The Weston Mills Pond extends

several miles up Lawrence Brook to the dam at Farrington Lake but the reservoir width is quite restricted by the relatively steep natural banks and narrow stream channel. Much of the urban contiguous development is well above normal flood elevation.

Judging from the difficulty of access for dredging and the drainage, the inspection confirmed that the upstream reservoir is quite extensively silted.

e. *Downstream Channel*

The existing banks of lower reservoir between the two dams are also quite steep which, together with the Route 18 concrete arch bridge, seriously restrict the downstream channel hydraulic capacity. Immediately north of the lower dam, the flow is further restricted by a newer bridge (built in 1965) over Burnet Street.

3.2 EVALUATION

The major concern of the inspection team is the status of hydraulic conditions during periods of high flow and collapse possibilities should the dam immediately downstream rupture. From available photographs at the office of the City Engineer, the July 1975 flood did considerable damage to the lower dam water intake structure. Further structural investigation of the concrete spillway, jointery and foundations can only be made if the dam is dewatered (see Section 7).

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

Operational procedures were not observed by the inspection team. Because of the city water supply intake facilities at the dam immediately downstream, city personnel are normally on duty 24 hours a day. From discussions with the City Engineer, operational activity at the Westons Mills Arch dam consist primarily of periodic inspections and the removal of floating drift and debris when the sluice gates are adjusted to control the lower pool intake elevation. Water Department personnel also monitor the dam whenever there are major storms.

4.2 MAINTENANCE OF DAM

The dam is periodically inspected and repairs undertaken when required and funds are available. Several years ago, the dam was inspected by scuba divers and leaks were repaired. Additional riprap was also placed on the downstream side of the spillway.

4.3 MAINTENANCE OF OPERATING FACILITIES

The only operating facilities in use are the two 30 inch gate valves which are periodically inspected by the City.

4.4 DESCRIPTION OF WARNING SYSTEM

None exists except the monitoring by City personnel during major storms.

4.5 EVALUATION

The present operational procedures and safeguards during periods of heavy flows were deemed to be adequate.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data

Utilizing the Guidelines for the Safety Inspection of Dams, it has been determined that the dam at Westons Mills Arch Dam is intermediate in size and falls into the significant hazard category due to the presence of urban development immediately downstream. Accordingly, the spillway design flood (SDF) was determined to be one half the PMF and the inflow hydrograph was calculated from the probable maximum precipitation (PMP).

The entire 200' length of the arch dam functions as a spillway. Abutments on either side of the dam are 24 feet long and 2.5 feet higher than the spillway crest. The maximum discharge over the dam which does not overtop the abutments is 2290 cfs.

The PMF hydrograph for this drainage area was calculated using the SCS curvilinear unit hydrograph. Peak inflow to the reservoir for the PMF and 1/2 PMF was 62,000 cfs and 31,000 cfs respectively, indicating that the discharge capacity of the dam is significantly inadequate. The 1/2 PMF was routed through the reservoir and the discharge decreased insignificantly from 31,000 cfs to 29,000 cfs.

In accordance with Corps of Engineers, Philadelphia District, directives, the inflow hydrograph and flood routing was additionally derived utilizing the HEC-1 program. A slight reduction in the PMF and 1/2 PMF to 58,500 cfs and 29,200 cfs respectively was noted. Flood routing for 1/2 the PMF yielded a peak discharge of 27,340 cfs. Employing the routed SDF, the spillway discharge capacity will accommodate approximately 8% of the SDF.

Since the Lawrence Brook at this location is rather confined in a narrow channel between steep-sided banks, the overtopping discharge capacity was extrapolated to accommodate 1/2 the PMF. It was determined that a flood height 12 feet over the spillway crest (9.5 feet above the abutments) would result during a storm equivalent to 1/2 the PMF (assuming no tailwater control).

b. Experience Data

Although there is no recorded stream flow data immediately downstream from Westons Mills Arch Dam, there is a gaging station 4 miles upstream at the Farrington Dam. Log-Pearson type III flood frequency analyses were performed by the U.S. Geological Survey utilizing weighted WRC map skews on the historical data available from this station. The transposed 100 and 500-year floods are 5800 cfs and 9680 cfs respectively. Floods of these magnitudes would overtop the embankments by approximately 2.0 feet and 3.5 feet respectively. The period of record at Farrington is 50 years. Observations made by City personnel during the storm of July, 1975, (and confirmed by their photographs) indicate the river overtopped the embankments by about 2 to 3 feet. The only discerned damage resulting from that storm was some erosion of the west embankment. This area has since been backfilled with concrete slope protection.

c. Visual Observation

The most westerly of the three sluice gates is destroyed and the usefulness of the remaining sluices is inconsequential during the periods of high flow as they are submerged on both sides of the dam and would have little hydraulic capacity. Therefore, this dam has no drawdown capability.

d. Overtopping Potential

The spillway has a maximum capacity of 8% of the design flood (1/2 PMF)before overtopping the abutments and is clearly inadequate in the significant hazard category. As indicated above, 1/2 the PMF, when routed thru the reservoir, results in a overtopping of the dam by over 9 feet. There is hearsay evidence that this dam has been repeatedly overtopped in the past and the overtopping potential will continue to exist, regardless of the hazard category considered. Due to the present physical geometry, the overtopping potential cannot be related to hazard. However, it is felt that a failure of the dam would not significantly contribute to the downstream water surface elevation (hence the damage potential) as long as the lower dam does not collapse.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

The alignment of the concrete structure is plumb and true and no significant tilting or differential settlements were observed. It is noted that the entire spillway is continuously passing several inches of water hence close visual examination is limited.

b. Design and Construction Data

Referring to Section 2.4.b, only minor cracking and spalling were observed during the field check but because the structure is unreinforced, temperature, rib shortening and shrinkage cracks may be expected in the concrete arch. The reentrant corners of the abutments are deteriorated with the edges broken off but this is of minor consequence.

The original design computations for overturning and sliding stability were unavailable. Maximum concrete stresses appear to have been computed using a three foot hydraulic crest. Although there exists some knowledge of the engineering design techniques employed in 1917, the method actually used for deriving the stability of the dam is extremely questionable in view of the geometry of the structure. The height to base width ratio indicates that some arching action was taken into account in the original design but it appears to have been done on an intuitive basis rather than employing statical methods available in 1917. Based on the SDF head established in this study, the dam appears to be statically unstable and has a negative factor of safety. Without knowledge of the foundation

conditions, further stability investigations in this phase are conjectural.

If the dam is considered to be a gravity structure without arch action it is unstable. Its ability to sustain arch action is directly tied to the contact between the abutment and shale bedrock. Since the New Brunswick shales decompose readily when exposed to air and/or water; further geotechnical explorations must be made to evaluate its present rock supporting capability. It is thought that additional stability is being provided by the downstream face riprap.

c. Operating Records

Performance records are unavailable regarding the dam's stability under maximum loading conditions but it should be noted that the downstream New Brunswick water supply intake dam (just north of Route 18) maintains a tailwater within 4.5 to 5 feet of the spillway crest. The dam appeared to suffer little damage during the July 1975 flood.

d. Post Construction Changes

The only structural modification noted was the raising of the spillway crest by 18 inches during the initial construction (as noted in paragraph 2.2). There is evidence of patching on the structure and riprap has been placed by City forces a few years ago just below the spillway.

e. Seismic Stability

As the dam is located in Seismic Zone 1, little hazard exists from earthquake forces and the potential vulnerability is negligible. The inertial forces relating to Zone 1 earthquake coefficients should be taken into account in the structural analyses in further studies but from the consultant's experience with this type of dam geometry, it will have little effect on the calculated stability and factor of safety compliance relating to the shale foundations.

SECTION 7 - ASSESSMENTS/RECOMMENDATIONS/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Conditions

On the basis of the Phase I visual examination, the existing concrete dam appears to be in fair structural condition and functions adequately as part of the New Brunswick water supply system (although the spillway is extremely inadequate to pass the design flood). No detrimental findings, excepting the physical geometry/design characteristics which, in order to render a complete structural review and analysis, will require the further gathering of information and review, were revealed.

It is believed that the safety of this dam would be substantially decreased if the Lower Dam were to fail. This is due principally to the short 600 foot distance between them which maintains a tailwater on the study dam during overtopping flood conditions.

b. Adequacy of Information

The information gathered for Phase I is thought to be adequate but the available data is insufficient to fully evaluate the structural stability of the dam in detail. This will have to be done in further studies and will require geotechnical investigations of the foundation rock and visual inspection of the spillway and abutments.

c. Urgency

A collapse of the Lower Dam (north of Route 18) could endanger the integrity of the Westons Mills Arch dam by eventually causing a sweepout of the lower stream face channel bed fill material and riprap that is thought to presently help stabilize the dam.

Consequently, further investigations should be undertaken in the near future regarding the foundation conditions and stability of the dam.

d. Necessity for Further Study

The inspection indicates that improvements to the spillway are impractical although its capacity does not meet the requirements of the Recommended Guidelines for Safety Inspection of Dams, passing only 8 percent of the SDF. However, due to the unknown condition of the foundations and the concrete in the spillway structure, additional studies and structural analyses appear to be warranted.

7.2 RECOMMENDATIONS/REMEDIAL ACTIONS

a. Alternatives

Inasmuch as original stability analyses and design computations are unavailable and this dam is classified in the significant hazard category, it is recommended that further studies be undertaken regarding these aspects. It is recommended that the owner provide, at his own expense, stability computations, including a trial arch analysis, and additional investigative data on the foundation conditions. This information is considered essential to complete assess the continued stability and to determine if the dam constitutes a hazard to human life and property. Its structural condition is classified as questionable pending receipt of the further investigations.

Remedial measures recommended are the construction of concrete or riprap slope protection above and below the abutments and to remove the large debris presently in the upstream channel. The berms behind the abutments should be raised to at least the height of the abutments and protected.

b. O&M Maintenance and Procedures

Because the City of New Brunswick presently maintains a close monitoring of the Westons Mills Arch Dam, little is foreseen as improvements to O&M procedures. However, a check list should be developed for periodic maintenance inspections so records of conditions and repairs can be maintained.

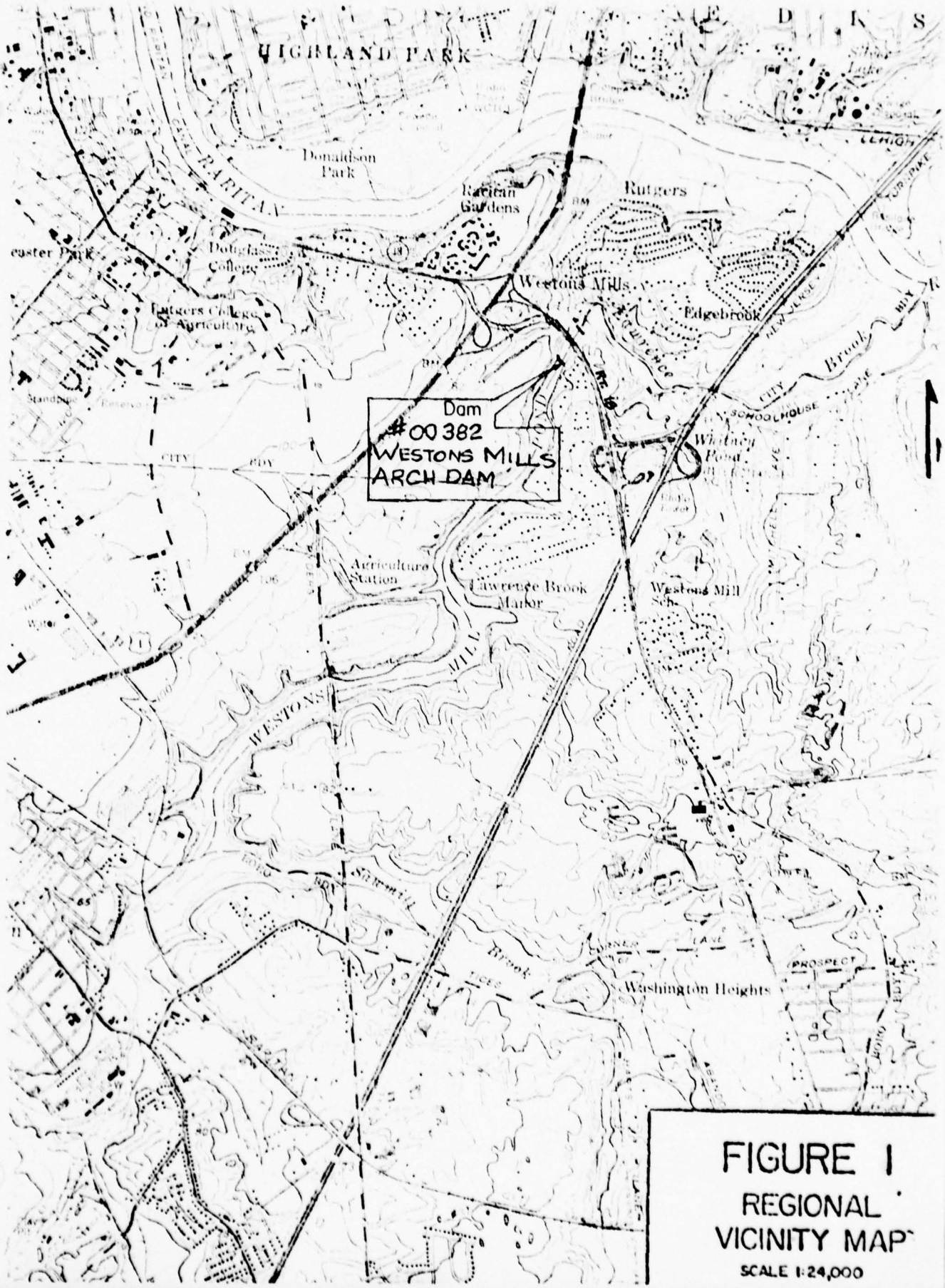


FIGURE 1

REGIONAL VICINITY MAP

SCALE 1:24,000

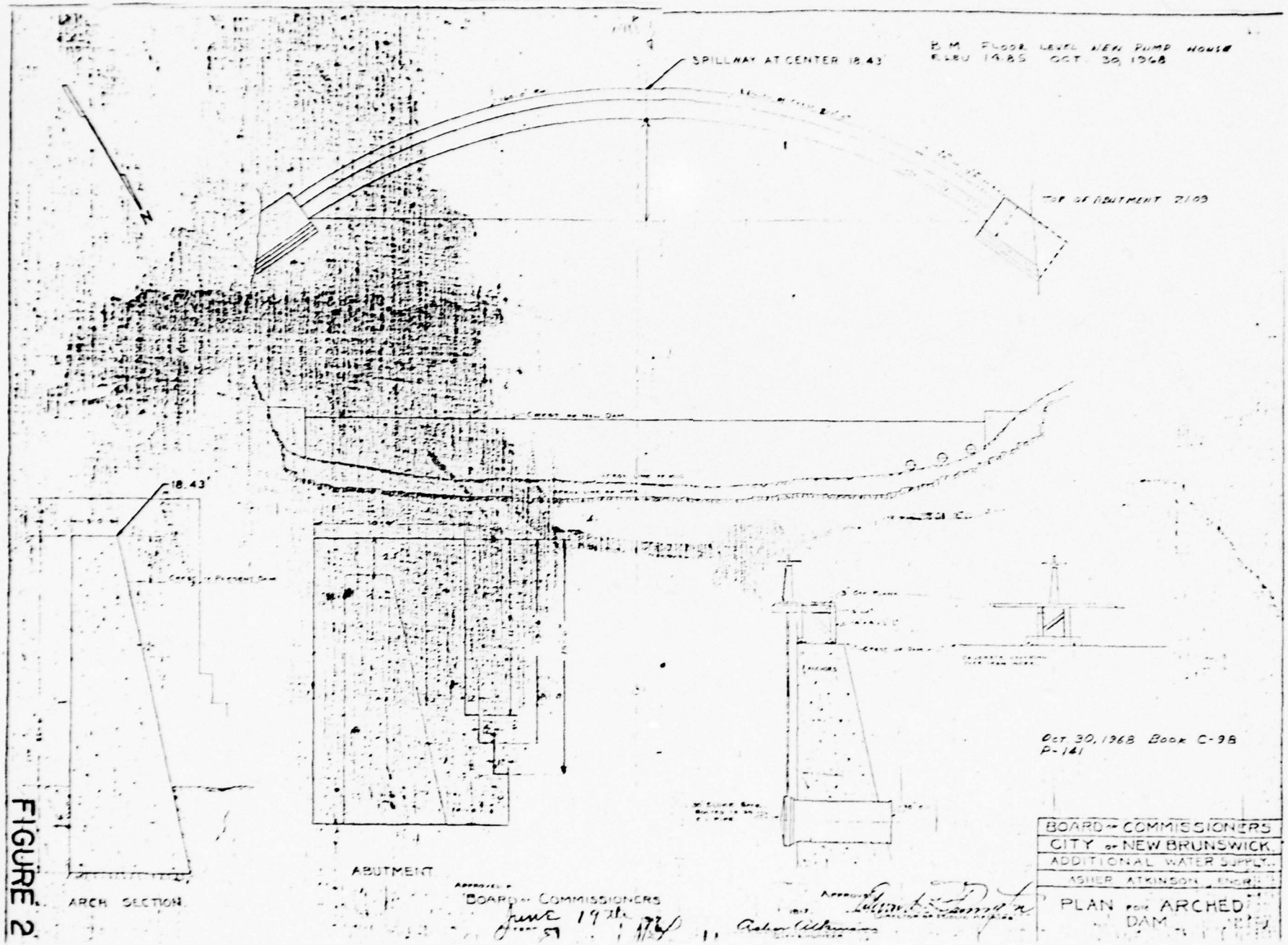

SPILLWAY AT CENTER 18.43'
B.M. FLOOR LEVEL NEW PUMP HOUSE
ELEV 19.85 OCT. 30, 1968
TOP OF ABUTMENT 21.03
18.43'
ABUTMENT
ARCH SECTION
Oct 30, 1968 Book C-98
P-141
BOARD OF COMMISSIONERS
CITY OF NEW BRUNSWICK
ADDITIONAL WATER SUPPLY
ASHLEY ATKINSON, ENGR.
PLAN FOR ARCHED DAM
Approved
BOARD OF COMMISSIONERS
June 19 1931
Robert Atkinson

FIGURE 2.

Check List
Visual Inspection
Phase 1

Name Dam Weston Mills Arch Dam County Middlesex State New Jersey Coordinators NJDEP

Date(s) Inspection June 14,19,27 July 19,1978 Weather Clear Temperature 80°

Pool Elevation at Time of Inspection 18.6 M.S.L. Tailwater at Time of Inspection 14.0 M.S.L.

Inspection Personnel:

T. Chapter K. Jolls

M. Carter R. Lang

C. Hoffman

K.P. Jolls Recorder

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SEEPAGE OR LEAKAGE	Unknown (entire spillway submerged)	Unreinforced concrete arch. There are some type of joints in structure (unknown)
STRUCTURE TO ABUTMENT/EMBANKMENT JUNCTIONS	Satisfactory. No seepage observed. Left embankment junction patched.	Berms behind abutments should be regraded up to abutment grade.
DRAINS	None	
WATER PASSAGES	None	
FOUNDATION	Unknown. Shale bedrock exists within 10-15 feet of ground. Steep banks indicate rock close to surface.	N. Brunswick Shale area.

VISUAL EXAMINATION OF	OBERSVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS CONCRETE SURFACES	Unknown in spillway. (Continuously submerged)	Concrete very old but only surface is eroded. Appears to be monolithically solid in interior. (Only abutments observed).
STRUCTURAL CRACKING	Unknown in spillway. (Continuously submerged)	
VERTICAL AND HORIZONTAL ALIGNMENT	Satisfactory. No differential settlement observed at top elevations.	Plans indicate foundation is on solid rock.
MONOLITH JOINTS	None observed.	
CONSTRUCTION JOINTS	Poor condition at abutments.	Joint deterioration not critical in abutments Spalling indicates concrete is not reinforced.

EMBANKMENT

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SURFACE CRACKS	N/A	Narrow berms at abutments. Except for slope protection, embankment minor concern re zoning and classification.
UNUSUAL MOVEMENT OR CRACKING AT OR BEYOND THE TOE	None	
SLOUGHING OR EROSION OF EMBANKMENT AND ABUTMENT SLOPES	Some erosion. Minor patching with concrete and asphalt at west abutment.	
VERTICAL AND HORIZONTAL ALINEMENT OF THE CREST	N/A	
RIPRAP FAILURES	Riprap dumped in channel. None behind abutments.	

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM	See page 2	
ANY NOTICEABLE SEEPAGE	No	
STAFF GAGE AND RECORDER	None	
DRAINS	None. Drains in concrete spillway only.	

OUTLET WORKS

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT	Unknown (continuously submerged)	
INTAKE STRUCTURE	3-30" Ø sluice gates.	2 gates operable. 1 vandalized. Temporary wood catwalk built over gates.
OUTLET STRUCTURE	30" Ø pipes(approximately 9' below spillway crest) at each gate.	
OUTLET CHANNEL	See page 7.	
EMERGENCY GATE	None	No drawdown capability.

UNGATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE WEIR	Narrow crest (3' x 200') on 160' radius. Entire spillway built at same elevation.	Base width to height ratio small for gravity structure. Determine arching effect in further studies.
APPROACH CHANNEL	Natural stream channel confined by natural, rather steep river banks.	
DISCHARGE CHANNEL	See approach cannel.	Stability depends on Lower Dam bridge. Danger of sweepout.
BRIDGE AND PIERS	None	

GATED SPILLWAY

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONCRETE SILL	N/A	
APPROACH CHANNEL	N/A	
DISCHARGE CHANNEL	N/A	
BRIDGE AND PIERS	N/A	
GATES AND OPERATION EQUIPMENT	N/A	

INSTRUMENTATION

VISUAL EXAMINATION	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
MONUMENTATION/SURVEYS	None	
OBSERVATION WELLS	None	
WEIRS	None	
PIEZOMETERS	None	
OTHER	There is a gage at Farrington dam (4 miles upstream).	Current records are available. New Jersey Water Resources U.S.G.S. Survey NJ-76-1.

RESERVOIR

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
SLOPES	Length extends up Lawrence River approximately 5 miles. Slopes are steep in many areas.	Flow thru reservoir restricted by bridge at Ryder's Lane.
SEDIMENTATION	Unknown. Heavily silted up in upper reaches of pond but not near dam.	Difficult access due to residential areas for dredging and/or silt removal.

DOWNSTREAM CHANNEL

VISUAL EXAMINATION OF	OBSERVATIONS	REMARKS OR RECOMMENDATIONS
CONDITION (OBSTRUCTIONS, DEBRIS, ETC.)	Considerable debris (large trees) in channel.	Route 18 concrete arch bridge between study and lower dam. Minor hydraulic constriction.
SLOPES	Steep natural banks (rock apparently in close to surface).	Trees on sides in many areas.
APPROXIMATE NO. OF HOMES AND POPULATION	Very few homes in zones of flooding. There are boat basins and similar facilities below lower dam.	All residential areas are on sufficient high ground.

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
WESTONS MILLS ARCH DAM

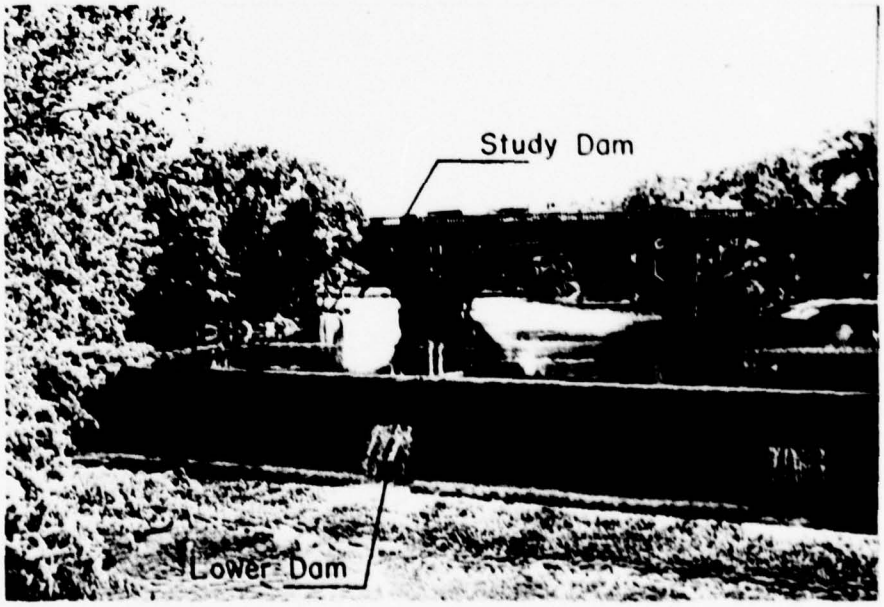
ITEM	REMARKS
PLAN OF DAM	Available for structural geometry; No details available.
REGIONAL VICINITY MAP	Available
CONSTRUCTION HISTORY	Partially available(photographs)
TYPICAL SECTIONS OF DAM	Available
HYDROLOGIC/HYDRAULIC DATA	Available at Farrington Dam gage
OUTLETS - PLAN	None
- DETAILS	
-CONSTRAINTS -DISCHARGE RATINGS	
RAINFALL/RESERVOIR RECORDS	Not available

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

ITEM	REMARKS
MONITORING SYSTEMS	None
MODIFICATIONS	Unknown
HIGH POOL RECORDS	Not available
POST CONSTRUCTION ENGINEERING STUDIES AND REPORTS	None
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	Not available
MAINTENANCE OPERATION RECORDS	Hearsay information from City of New Brunswick.

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

Downstream view of spillway



Dam downstream of study dam

CHECK LIST
HYDROLOGIC AND HYDRAULIC DATA
ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: Area = 42.0 square miles

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 18.43 (1050 acre-ft.)

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 21.09 (1600 acre-ft.)

ELEVATION MAXIMUM DESIGN POOL: 21.09

ELEVATION TOP DAM: 21.09

CREST:

a. Elevation 18.43
b. Type Narrow Crest Weir
c. Width 3 ft.
d. Length 200 ft.
e. Location Spillover None
f. Number and Type of Gates 3 - 18" Ø sluices

OUTLET WORKS: None

a. Type
b. Location
c. Entrance inverts
d. Exit inverts
e. Emergency draindown facilities None

HYDROMETEOROLOGICAL GAGES:
a. Type Water stage recorder
b. Location Farrington Dam (4 mi. upstream)
c. Records 1927 - present

MAXIMUM NON-DAMAGING DISCHARGE: 2290 c.f.s.

BY CH DATE 7-78 **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A-1 OF

CHKD. BY DATE Dam Inspection PROJECT 3-2[illegible]

SUBJECT Weston Mills - PRECIPITATION DATA FOR SYNTHETIC HYDROGRAPH

PMF HYDROGRAPH

Fig 1, P-29 Small Dams - Zone 6 - New Jersey
PMF 10 sq mi 6 hr duration = 26"

Fig 2, P-30 Zone 6 Drainage Area = 42 sq mi
Rainfall = .87 × 26 = 22.6"

Fig 4 Zone 6 Distribution of 6 hr rainfall

Time	Dist % of 6 hr	Cumul. Rainfl	Δ R.	R Rearrang (6,4,3,1,2,5[illegible])	Rearrang Cum.	Runoff C.N. 70	Δ Runoff
0.5	30	6.78	6.78	0.68	0.68	0	0
1.0	48	10.85	4.07	0.90	1.58	0.10	0.10
1.5	58	13.11	2.26	0.90	2.48	0.45	0.35
2.0	65	14.69	1.58	1.13	3.61	1.10	0.65
2.5	71	16.05	1.36	1.13	4.74	1.85	0.75
3.0	76	17.18	1.13	1.36	6.10	2.90	1.05
3.5	81	18.31	1.13	4.07	10.17	6.40	3.50
4.0	85	19.21	0.90	6.78	16.95	12.80	6.40
4.5	89	20.11	0.90	2.26	19.21	15.00	2.20
5.0	93	21.02	0.91	1.58	20.79	16.40	1.40
5.5	97	21.92	0.90	0.91	21.70	17.40	1.00
6.0	100	22.60	0.68	0.90	22.60	18.20	0.80

$$T_c = \left(\frac{11.9 \times L^3}{H}\right)^{0.385} \qquad L = 11 \text{ mi.} \qquad H = 70$$

$$T_c = 8.06 \text{ hr.}$$

$$T_p = \frac{D}{2} + 0.6\,T_c = \frac{0.5}{2} + 0.6(8.06) = 5.09 \text{ hr.}$$

$$T_b = 2.67 \times T_p = 13.58$$

$$q_p = \frac{484 \times A \times Q}{T_p} = 3994\ Q$$

Above values utilized in derivation of synthetic hydrograph only.

BY _____ DATE _____

CHKD. BY _____ DATE _____

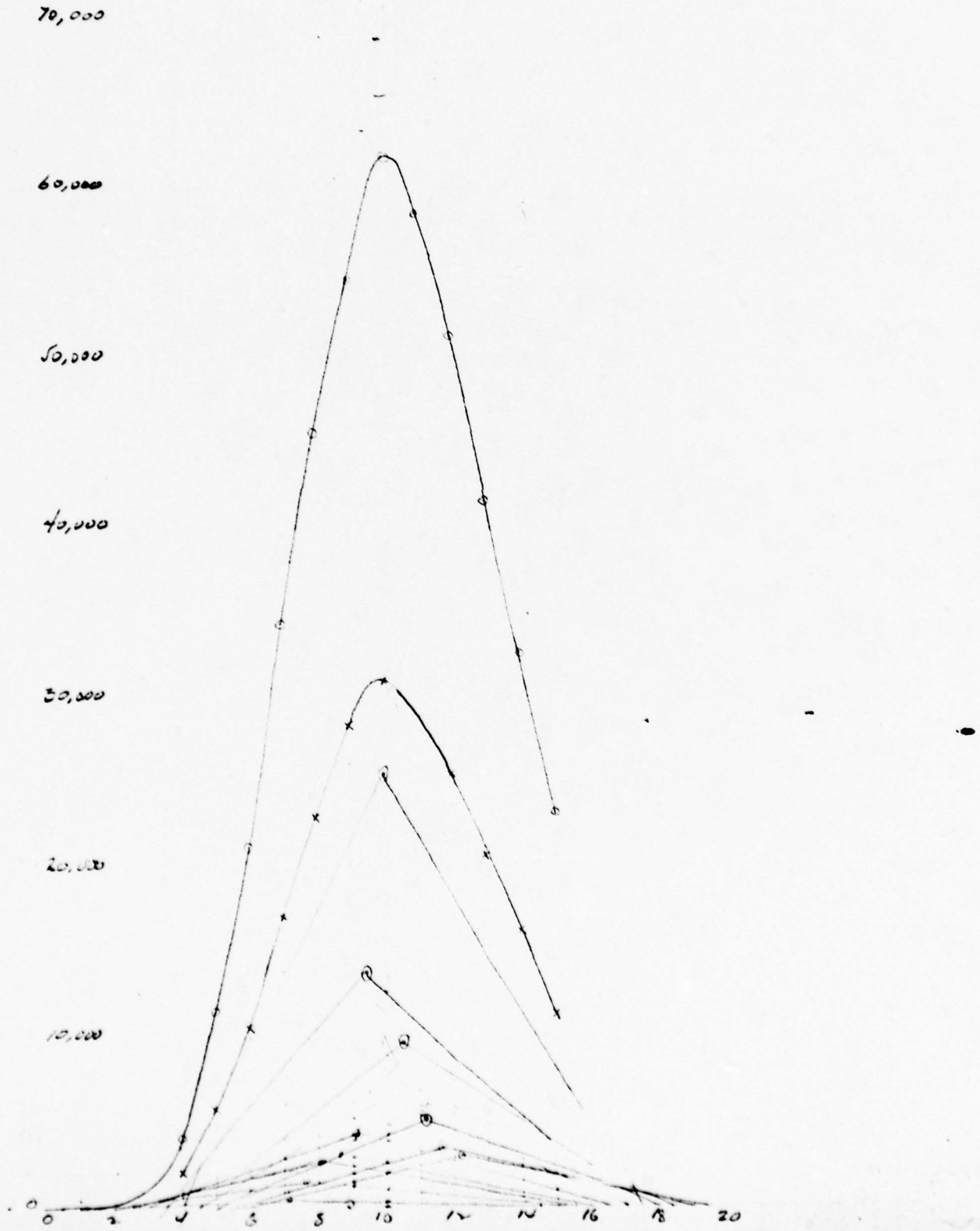
SUBJECT WESSONS MILL

DAM INSPECTION

HYDROGRAPH (Triangular Method)

PROJECT C-222

Ar



BY CM DATE 1-18 | LOUIS BERGER & ASSOCIATES INC. | SHEET NO. A 3 OF
CHKD. BY DATE | DAM INSPECTION | PROJECT C-222
SUBJECT WESTON MILL DAM.

Location - On Lawrance Br (Tributary of Raritan) at East Brunswick N.J.
Comp. 1917 Height = 17'± DRAINAGE AREA (PLANIMETERED) = 42.0 Mi²
Indicated spillway L = 200' Q = 2290
Dam lies upstream from gravity dam which submerges all but abut 6' of Weston Mill Dam.
AREA OF RECREATION POOL = 160 ACRES (PLANIMETERED) × AVG DEPTH (N. BRUNSWICK CITY DAM. = 1050 Acre
1917 Drawing available - Spillway MW = 1600 "
Shows 200' crest on 160' radius Δ = 2.5' @ Dam
Abutments 2.5' higher than crest ± 24' long
Abutments beyond dam - Foundation - ON Rock

Size - Intermediate Hazard - Significant
Hydrograph ½ PMF to PMF

Log Pearson Frequency @ (@ FARRINGTON DAM)

$\text{Area}_{FARR.} = \frac{42.0}{34.4} = 1.22$

		TRANSPOSED Q's
50 yr	= 3770	
100 yr	= 4760 × 1.22 =	5800 cfs
500 yr	= 7930 × 1.22 =	9650 cfs

Spillway discharges

Over main crest - L = 200'

H	C	Q
0.5	2.9	205
1.0	2.9	580
1.5	2.9	1065
2.0	2.9	1640
2.5	2.9	2293
3.0	2.9	3014
3.5	2.9	3798
4.0	2.9	4640
5.0	2.9	6485
6.0	2.9	8525
8.0	2.9	13124
10.0	2.9	18341
12.0	2.9	24110
13.1	2.9	27185
14.0		30382

Over Abutments

H	C	Q	Total Q
			205
			580
			1065
			1640
0	2.5	0	2293
0.5	"	47	3061
1.0	"	134	3932
1.5	2.8	247	4887
2.5		531	7016
3.5		880	9405
5.5		1733	14857
7.5		2760	21100
9.5		3935	28045
10.5		4573	31355
11.5		5242	35624

Surcharge Capacity

Elevation	Area	Ave Area	Δ Vol	Σ Vol
18.43				
	30	280	3240	3240

From "Small Dams":

PMP for 10 sq mi = 26"/6 hr. ;

Reduction factor for 42 sq mi = 87%
thus: .87 x 26" = 22.6" rainfall

Zone C distribution of 6 hr rainfall

Time	Dist %	Cumul. R.	ΔR.	R. Rearrng	Rearrng Cum	Runoff C.N. 70	Δ Runoff
1	48	10.85	10.85	1.58	1.58	0.10	0.10
2	65	14.69	3.84	2.03	3.61	1.10	1.0
3	76	17.18	2.49	2.49	6.10	2.90	1.8
4	85	19.21	2.03	10.85	16.95	12.80	9.9
5	93	21.02	1.81	3.84	20.79	16.40	3.6
6	100	22.60	1.58	1.81	22.60	18.2	1.8

$T_c = \left(\frac{11.9 \times L^3}{H}\right)^{.385}$ L = 11 mi. H = 70

T_c = 8.06 hr.

Alternate determination of T_c per U.S. Navy TP-PW-5

Channel slope = $\frac{70 \times 100}{11 \times 5280}$ = 0.12 %

min. Avg. vel. for slopes < 2% = 2 ft/sec

$\frac{11 \times 5280}{2 \times 3600}$ = 8.07 hr

BY HG DATE

CHKD. BY DATE

SUBJECT DIMENSIONLESS UNITGRAPH (1 HR)

LOUIS BERGER & ASSOCIATES INC.

SHEET NO. A5 OF

PROJECT C-222

WESTONS MILL DAM

Unit Time T_r = 1 hr; T_{p0} = 5.09 hrs; $L + D/2 = \frac{5.09}{0.81} = 6.93 = 7$

Area = 26.89 mi² DSF (2 in) = 26.89 × A = 1129.4

Time	% T_p 100/T_p (hr)	Dimensionless * Ordinate Q/(T/DSF)	Q DSF/T_p (D.O.)
1	14.43	0.9	147
2	28.85	4.2	684
3	43.29	9.2	1500
4	57.72	15.5	2526
5	72.15	19.6	3194
6	86.6	20.9	3406
7	101	19.6	3194
8	115	16.6	2705
9	130	13.2	2151
10	144	10.5	1711
11	159	8.2	1336
12	173	6.5	1059
13	188	5.0	815
14	202	3.8	619
15	216	3.0	489
16	231	2.28	372
17	245	1.68	274
18	260	1.3	212
19	274	1.0	163
20	289	0.78	127
21	303	0.6	98
22	317	0.4	65
23	332	0.3	49
24	346	0.28	46
25	361	0.23	38
26	375	0.2	33
27	390	0.17	27
28	404	0.14	23
29	418	0.11	18

* Value for Dimensionless Ordinate read from graph

BY H. G. DATE ..... **LOUIS BERGER & ASSOCIATES INC.** SHEET NO. A6

CHKD. BY ..... DATE ..... PROJECT C222

SUBJECT BUREAU OF RECLAMATION DEFINITION OF TERMS USED IN UNITGRAF

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

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

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

SCS L IS APPROXIMATELY $0.6\,T_c$

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

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

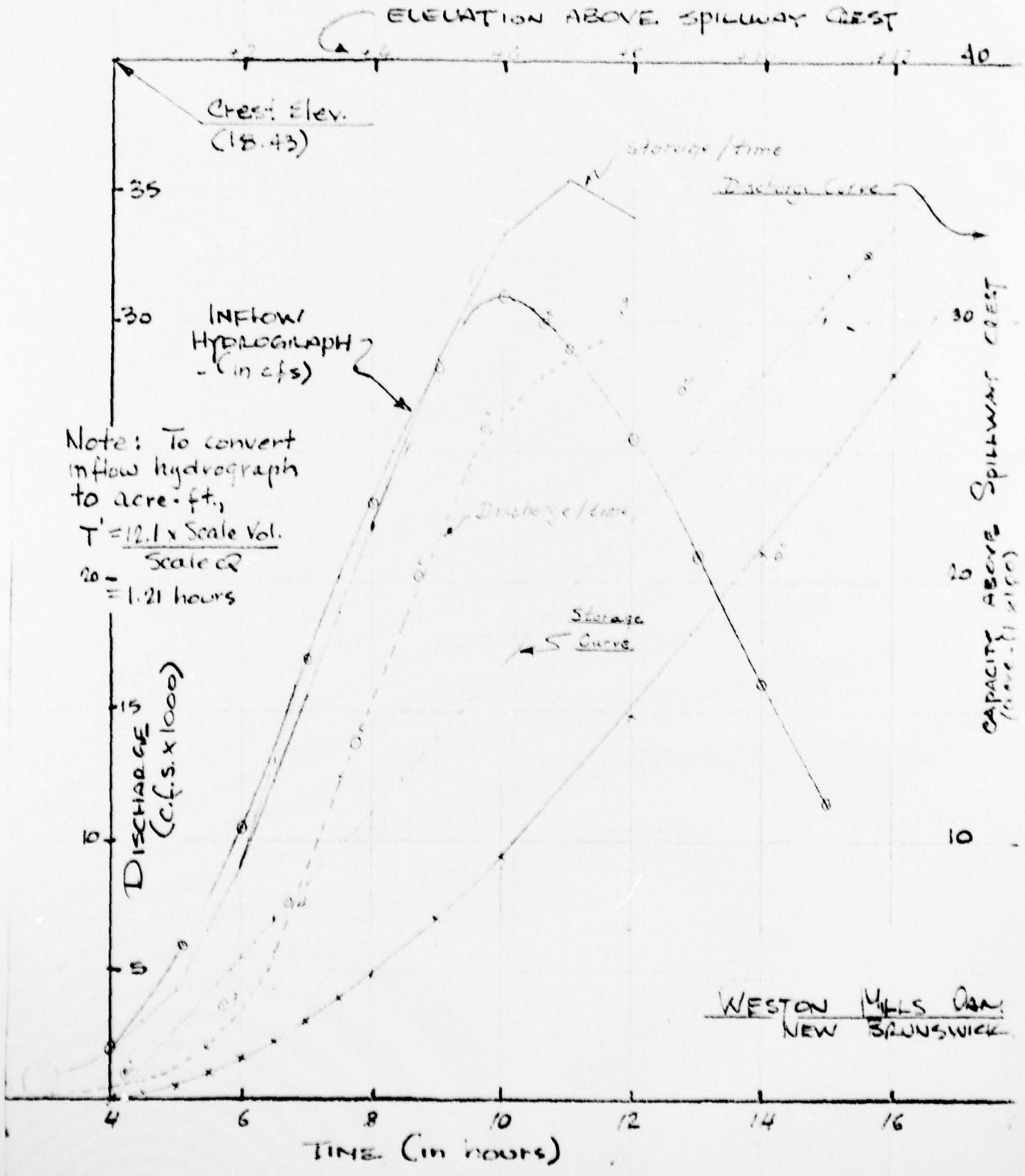
THE SCS CURVELINEAR UNIT HYDROGRAPH CAN BE DERIVED BY FIRST TAKI BUREAU OF RECLAMATION L, (LAG) PLUS $\frac{D}{2}$ AFTER BEING DIVIDED BY 100, THEN MULTIPLIED BY EACH ABSCISSA (IN HOURS) BY THE QUOTIENT. THEN READING THE DIMENSIONLESS ORDINATE FOR THE GIVEN PERCENTAGES FROM THE PREVIOUSLY DETERMINED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

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

$$26.89 \times \text{AREA}$$

T.	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	15.4	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	19.9	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	19.8	19.7	19.6	19.5	19.4	19.3	19.2	19.3	19.1
110	19.9	19.6	19.9	19.1	19.7	16.2	16.4	16.2	16.0	15.7
120	15.5	15.2	15.0	14.7	14.5	14.2	14.0	13.9	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	9.9
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	4.5	4.5	4.4	4.3	4.2	4.2	4.1
200	4.0	3.9	3.9	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	0.1	0.1	0.1	0.1	0.1	0.1	0.1

SUMMARY OF HYDRAULIC CHARACTERISTICS



BY TC DATE

CHKD. BY DATE

SUBJECT Storage / Discharge Summary Sheet

LOUIS BERGER & ASSOCIATES INC.

Dam Inspection

SHEET NO. A9 OF

PROJECT C-222

Elev (Ft. Above Crest)	Storage (Acre Ft.)	Discharge (cfs)
1	275	580
2	550	1640
3	825	3061
4	1100	4887
5	1400	7016
6	1675	9405
7	1950	12050
8	2250	14857
9	2525	18600
10	2800	21100
11	3090	24500
12	3370	28045

FINAL RUN

HEC-1 VERSION DATED JAN 1973
UPDATED AUG 73
CHANGE NO. 01

WESTONS MILLS DAM INSPECTION JOB#C222
COMPOSED BY DJM
TUESDAY VII/XVIII/MCMLXXVIII

JO- SPECIFICATION

NQ	NMR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
100	1	0	0	0	0	0	0	0	0

JOPER IWT
3 0

SUB-AREA RUNOFF COMPUTATION

INFLOW HYDROGRAPH

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
5	0	0	0	2	0	1

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	-1	42.00	0.0	42.00	0.0	0.500	0	0	0

PRECIP DATA

VP	STORM	DAU	DAK
6	0.0	0.0	0.0

PRECIP PATTERN

0.10	1.00	1.40	9.90	3.40	1.80

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

147.	684.	1900.	2826.	3194.	3406.	3194.	2705.	2151.	1711.
1336.	1059.	815.	619.	469.	372.	274.	212.	163.	127.
90.	69.	49.	45.	38.	33.	27.	23.	18.	

UNIT GRAPH TOTALS 27080. CFS OR 1.00 INCHES OVER THE AREA

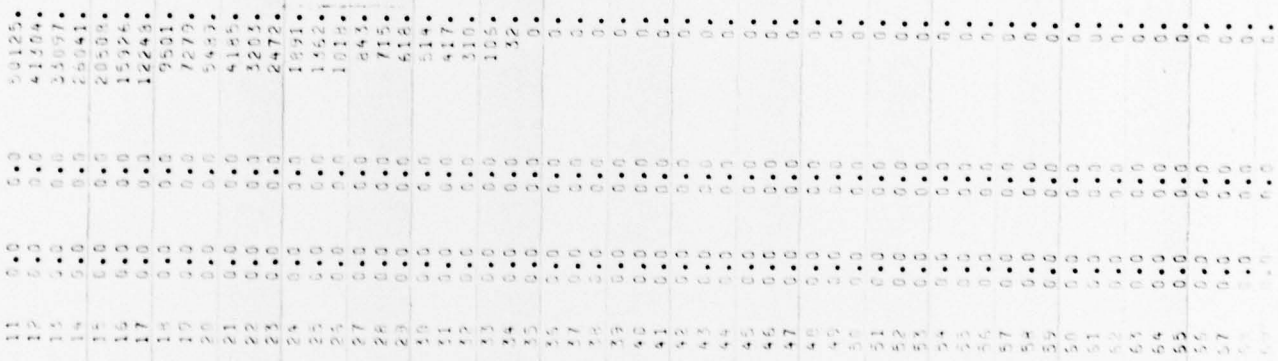
RECESSION DATA

STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

END-OF-PERIOD FLOW

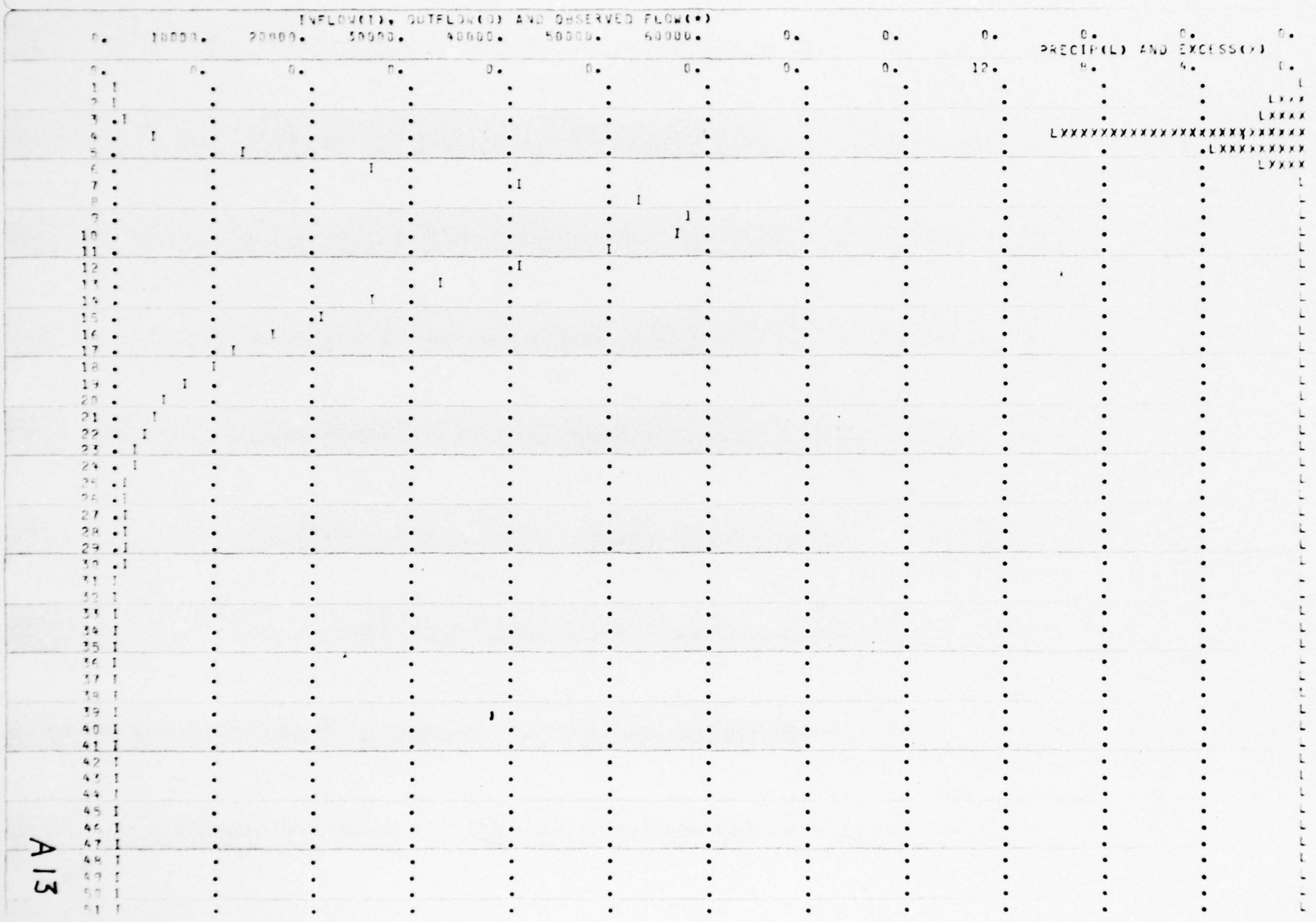
TIME	RAIN	EXCS	COMP Q
1	0.10	0.10	15.
2	1.00	1.00	215.
3	1.80	1.80	1099.
4	9.90	9.90	4433.
5	3.60	3.60	12844.
6	1.80	1.80	28458.
7	0.0	0.0	41117.
8	0.0	0.0	53009.
9	0.0	0.0	58434.
10	0.0	0.0	56822.

A10



72	0.0	0.0	0.
73	0.0	0.0	0.
74	0.0	0.0	0.
75	0.0	0.0	0.
76	0.0	0.0	0.
77	0.0	0.0	0.
78	0.0	0.0	0.
79	0.0	0.0	0.
80	0.0	0.0	0.
81	0.0	0.0	0.
82	0.0	0.0	0.
83	0.0	0.0	0.
84	0.0	0.0	0.
85	0.0	0.0	0.
86	0.0	0.0	0.
87	0.0	0.0	0.
88	0.0	0.0	0.
89	0.0	0.0	0.
90	0.0	0.0	0.
91	0.0	0.0	0.
92	0.0	0.0	0.
93	0.0	0.0	0.
94	0.0	0.0	0.
95	0.0	0.0	0.
96	0.0	0.0	0.
97	0.0	0.0	0.
98	0.0	0.0	0.
99	0.0	0.0	0.
100	0.0	0.0	0.
SUM	18.20	18.20	492854.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	59494.	50125.	20378.	6845.	492855.
INCHES		11.10	10.05	18.19	18.19
AC-FT		24873.	40440.	40753.	40753.


STATION 5
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)
0. 10000. 20000. 30000. 40000. 50000. 60000.
0. 0. 0. 12.
PRECIP(L) AND EXCESS(X)
0. 4. 8.

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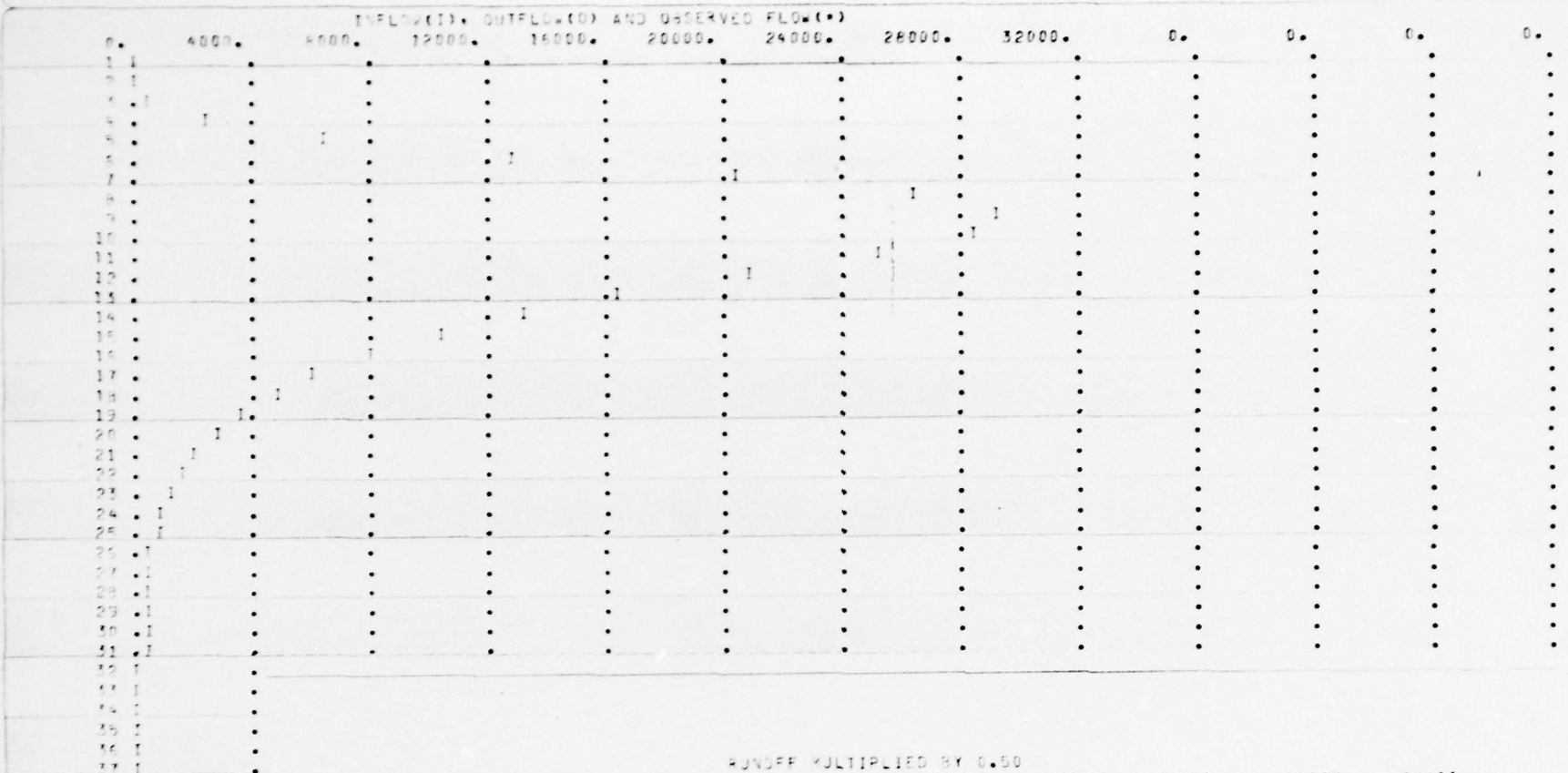
•OVF•

STATION 5



RUNOFF MULTIPLIED BY 0.50

	1.	104.	543.	2220.	5423.	12829.	20527.	24505.	23217.	20411.
	26063.	20052.	16348.	13020.	10234.	7963.	5124.	4751.	3659.	2745.
	2092.	1602.	1276.	945.	681.	503.	421.	358.	309.	257.
	208.	155.	53.	16.	0.	0.	0.	0.	0.	0.
	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	23217.	23067.	10188.	3423.	244427.
INCHES		5.55	9.83	9.10	9.10
AC-FT		12436.	20220.	20374.	20376.

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HYDROGRAPH ROUTING

FLOOD ROUTING

ISTAN	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME
55	1	0	0	2	0	1

ROUTING DATA

CLOSS	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	0.

STORAGE=	0.	0.	270.	550.	840.	1110.	1400.	1675.	3090.	3370.
OUTFLOW=	0.	0.	500.	1540.	3081.	4887.	7016.	9405.	24500.	28045.

TIME	END STOR	AVG IN	END OUT
1	1.	7.	1.
2	5.	58.	10.
3	29.	328.	62.
4	199.	1384.	278.
5	427.	4321.	1173.
6	1001.	9626.	4176.
7	1775.	18693.	10474.
8	2524.	23531.	18463.
9	3083.	27861.	24214.
10	3414.	28814.	27339.
11	3482.	26737.	29025.
12	3353.	23857.	24171.
13	2740.	19500.	20762.
14	2337.	14784.	17104.
15	2083.	11537.	13759.
16	1815.	9109.	10913.
17	1590.	7043.	8663.
18	1393.	5437.	6946.
19	1217.	4195.	5676.
20	1059.	3192.	4552.
21	920.	2419.	3646.
22	801.	1847.	2915.
23	693.	1410.	2336.
24	610.	1091.	1944.
25	532.	816.	1571.
26	462.	595.	1307.
27	402.	465.	1079.
28	353.	390.	892.
29	313.	333.	741.
30	280.	285.	617.
31	251.	233.	540.
32	224.	182.	481.
33	195.	104.	420.
34	166.	35.	357.
35	140.	8.	300.
36	117.	0.	251.

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40	57.	0.	123.
41	48.	0.	103.
42	40.	0.	86.
43	34.	0.	72.
44	28.	0.	60.
45	24.	0.	51.
46	20.	0.	42.
47	17.	0.	35.
48	14.	0.	30.
49	12.	0.	25.
50	10.	0.	21.
51	8.	0.	17.
52	7.	0.	15.
53	6.	0.	12.
54	5.	0.	10.
55	4.	0.	9.
56	3.	0.	7.
57	3.	0.	6.
58	2.	0.	5.
59	2.	0.	4.
60	2.	0.	4.
61	1.	0.	3.
62	1.	0.	2.
63	1.	0.	2.
64	1.	0.	2.
65	1.	0.	1.
66	1.	0.	1.
67	0.	0.	1.
68	0.	0.	1.
69	0.	0.	1.
70	0.	0.	1.
71	0.	0.	0.
72	0.	0.	0.
73	0.	0.	0.
74	0.	0.	0.
75	0.	0.	0.
76	0.	0.	0.
SUM			246429.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	27339.	23546.	10045.	3423.	246429.
INCHES		5.24	8.30	9.10	9.10
AC-FT		11731.	19934.	20376.	20377.

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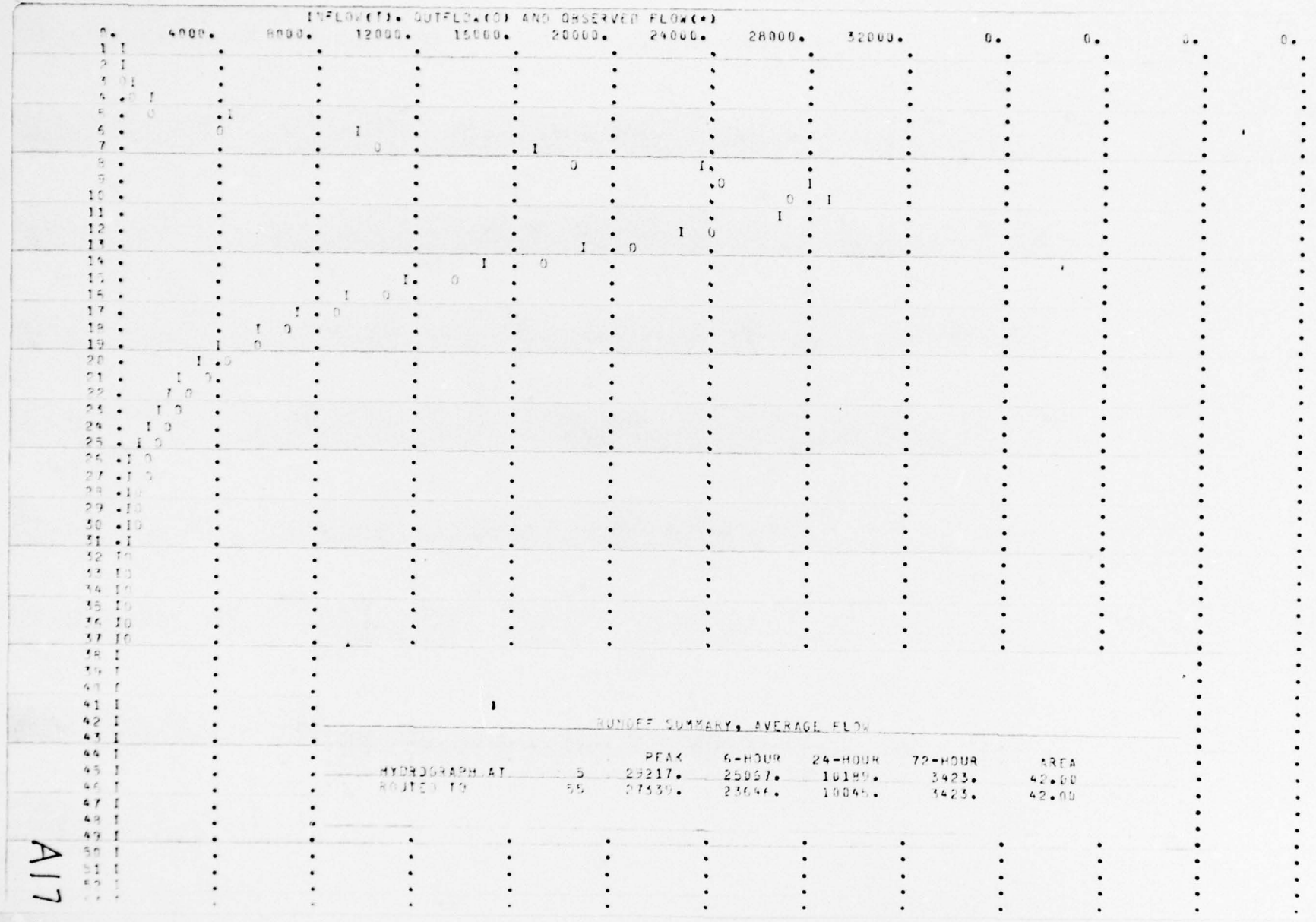
•SVE•

STATION 55



RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	5	29217.	25057.	10189.	3423.	42.00
ROUTED TO	55	27339.	23044.	10045.	3423.	42.00

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