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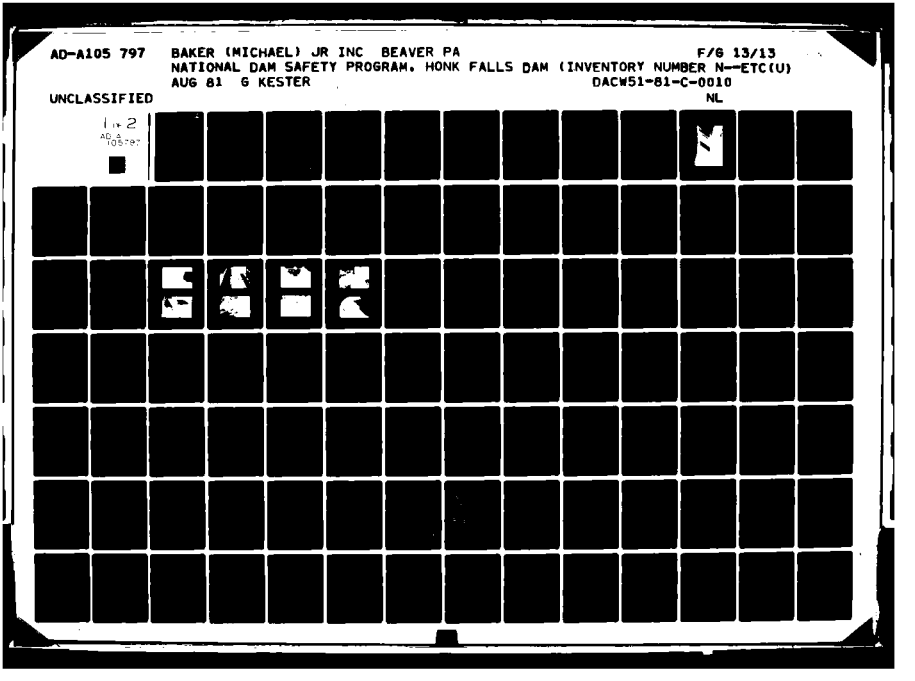


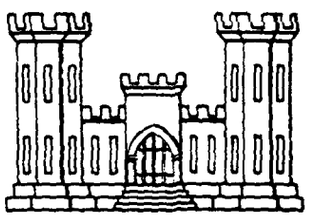
LEVEL II

LOWER HUDSON RIVER BASIN

HONK FALLS DAM

ULSTER COUNTY, NEW YORK
INVENTORY NO. N.Y. 73

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM



NEW YORK DISTRICT CORPS OF ENGINEERS

AUGUST 1981

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Lower Hudson River Basin

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

This report provides information and analysis on the physical condition of the dam as of the report date. Information and analysis are based on visual inspection of the dam by the performing organization.

Examination of available documents and a visual inspection of the dam and appurtenant structures did not reveal conditions which constitute an immediate hazard to human life or property.

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

Using the Corps of Engineers' screening criteria, it has been determined that the dam would be overtopped for all storms exceeding approximately 26 percent of the Probable Maximum Flood (PMF). The spillway is, therefore, adjudged as "seriously inadequate" and the dam is assessed as "unsafe, non-emergency."

The classification of "unsafe" means that, based on an initial screening and preliminary computations, there appears to be a serious deficiency in spillway capacity, so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream.

Structural stability analyses based on available information indicate that factors of safety against overturning and sliding are low for all conditions analyzed. When the dam is subjected to severe loading conditions such as ice loading or water levels overtopping the dam, the factors of safety are below recommended levels.

Therefore, a detailed stability analysis is considered necessary to determine actual stability conditions of the dam.

It is therefore recommended that, within three months of notification of the owner, detailed hydrologic and hydraulic investigations of the structure should be undertaken to more accurately determine the site-specific characteristics of the watershed and their effects upon the overtopping potential of the dam. At the same time, further analyses of the structural stability of the overflow and nonoverflow sections should be performed. The results of these investigations and analyses will determine the appropriate remedial measures which will be required. In the interim, a detailed emergency action plan must be developed and implemented during periods of unusually heavy precipitation. Also, around-the-clock surveillance must be provided during these periods.

At the present time, there exists no inspection or maintenance program at the dam. A program of regular inspection and maintenance should be established. The dam should be examined for seeps when the reservoir is below the spillway crest.

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PREFACE

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

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

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

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aide in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
HONK FALLS DAM
I.D. No. NY 73
DEC DAM No. 177C-735 LOWER HUDSON RIVER BASIN
ULSTER COUNTY, NEW YORK

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Honk Falls Dam (I.D. No. NY 73)
State:	New York
County:	Ulster
Stream:	Rondout Creek
Date of Inspection:	8 March 1981

ASSESSMENT

Examination of available documents and a visual inspection of the dam and appurtenant structures did not reveal conditions which constitute an immediate hazard to human life or property.

Using the Corps of Engineers' screening criteria, it has been determined that the dam would be overtopped for all storms exceeding approximately 26 percent of the Probable Maximum Flood (PMF). The spillway is, therefore, adjudged as "seriously inadequate" and the dam is assessed as "unsafe, non-emergency."

The classification of "unsafe" means that, based on an initial screening and preliminary computations, there appears to be a serious deficiency in spillway capacity, so that if a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream.

Structural stability analyses based on available information indicate that factors of safety against overturning and sliding are low for all conditions analyzed. When the dam is subjected to severe loading conditions such as ice loading or water levels overtopping the dam, the factors of safety are below recommended levels.

Therefore, a detailed stability analysis is considered necessary to determine actual stability conditions of the dam.

It is therefore recommended that, within three months of notification of the owner, detailed hydrologic and hydraulic investigations of the structure should be undertaken to more accurately determine the site-specific characteristics

of the watershed and their effects upon the overtopping potential of the dam. At the same time, further analyses of the structural stability of the overflow and nonoverflow sections should be performed. The results of these investigations and analyses will determine the appropriate remedial measures which will be required. In the interim, a detailed emergency action plan must be developed and implemented during periods of unusually heavy precipitation. Also, around-the-clock surveillance must be provided during these periods.

At the present time, there exists no inspection or maintenance program at the dam. A program of regular inspection and maintenance should be established. The dam should be examined for seeps when the reservoir is below the spillway crest.

The following remedial measures must be completed within one year:

1. The gates on the outlet pipes should be repaired, and their controls should be made operable.

2. Determine the source of and repair the area of seepage at the soil-concrete contact along the right wall.

3. Determine the source of and repair the area of seepage at the left wingwall.

4. Determine the cause of and repair the seepage below the auxiliary spillways.

5. Repair the eroded areas at the toe of the dam and at the construction joints.

6. Repair all areas on the dam where the concrete is spalled and deteriorated.

7. Remove all brush and trees growing near the right abutment and on the dam.

8. Remove the trees and debris from the channel below the auxiliary spillways.

9. A program of periodic inspections and maintenance of the dam should be established. All inspections and maintenance should be recorded for future reference.

10. A staff gage should be installed to monitor reservoir levels above normal pool.

SUBMITTED: _____
Granville Kester, Jr., P.E.
Vice President
MICHAEL BAKER, JR. of New York, INC.

APPROVED: _____
Colonel W.M. Smith, Jr.
New York District Engineer

DATE: 14 Aug 81

Overall View of Dam
Honk Falls Dam
I.D. No. NY 73
8 March 1981

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
HONK FALLS DAM
I.D. No. NY 73
DEC DAM No. 177C-735
LOWER HUDSON RIVER BASIN
ULSTER COUNTY, NEW YORK

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority - The Phase I inspection reported herein was authorized by the Department of the Army, New York District, Corps of Engineers, to fulfill the requirements of the National Dam Inspection Act, Public Law 92-367.

b. Purpose of Inspection - This inspection was conducted to evaluate the existing conditions of the dam, to identify deficiencies and hazardous conditions, to determine if these deficiencies constitute hazards to life and property, and to recommend remedial measures where required.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam - Honk Falls Dam is a concrete gravity structure 42.2 feet high, measured from the crest to the toe of the dam, and 294 feet long. The overflow section (principal spillway) is located in the center of the dam and consists of a 190.5-foot long broad-crested weir with a vertical upstream face and a sloping downstream face. The auxiliary spillway, located to the right of the principal spillway, has a total length of 22.3 feet and consists of a broad-crested concrete weir with vertical upstream and downstream faces. The non-overflow sections are 20 feet long on the left side and 61.2 feet long on the right side of the dam. Beneath the auxiliary spillways are two 6-foot diameter outlet pipes. A 3-foot diameter outlet pipe and a 12-inch outlet pipe are below the right and left sides of the spillway, respectively.

b. Location - Honk Falls Dam is located on Rondout Creek, a tributary of the Hudson River, 3 miles north of Ellenville, New York. The reservoir and dam are in Ulster County, New York. The coordinates

of the dam are N 41° 45' and W 74° 22.9'. The dam can be found on the Rondout Reservoir and Ellenville, New York, USGS 7.5 minute topographic quadrangles. A Location Map is shown in Appendix E.

c. Size Classification - Honk Falls Dam is 42.2 feet high, and the reservoir storage capacity at the minimum top of dam (elevation 579.1 feet T.B.M.) is 1504 acre-feet. Therefore, the dam is in the "intermediate" size category as defined by the Recommended Guidelines for Safety Inspection of Dams (Reference 13, Appendix D).

d. Hazard Classification - One home is 1700 feet downstream from the dam. Napanoch, New York, is 6000 feet downstream from the dam. Loss of life in the home and Napanoch is likely if the dam were to fail. Honk Falls Dam is therefore considered in the "high" hazard category as defined by the Recommended Guidelines for Safety Inspection of Dams.

e. Ownership - The dam and reservoir are owned by Recycled Paper Corp., c/o Albert I. Lonstein, Route 209, Terrace Hill, Ellenville, New York 12428.

f. Purpose of the Dam - Honk Lake is used for recreation.

g. Design and Construction - The dam was built about 1898. The designer and contractor are unknown.

h. Normal Operating Procedures - The reservoir is typically maintained at the spillway crest. There has been no maintenance or inspection of the dam for the past few years.

1.3 PERTINENT DATA

a. Drainage Area (square miles) - 104.09

b. Discharge at Dam (c.f.s.)

Spillway Capacity (at Minimum Top of Dam Elev. 579.1 ft. M.S.L.)	20,119.0

c. Elevation (Feet Above M.S.L.)[1] -

Minimum Top of Dam	579.1
Normal Pool (Spillway Crest)	569.0
Streambed at Toe of Dam	536.9

[1]All elevations are referenced to the spillway crest, elev. 569.0 ft. M.S.L., estimated from the USGS 7.5 minute topographic quadrangle, Rondout Reservoir, NY.

d. Reservoir Surface (Acres) -

Top of Dam (Elev. 579.1 ft. M.S.L.)	86.2
Spillway Crest (Elev. 569.0 ft. M.S.L.)	44.1

e. Reservoir Storage Capacity (Acre-Feet) -

Top of Dam (Elev. 579.1 ft. M.S.L.)	1504.0
Spillway Crest (Elev. 569.0 ft. M.S.L.)	860.0

f. Dam -

Type: Concrete gravity	
Length (Feet)	294.0
Height (Feet)	42.2
Top Width (Feet)	2.5
Side Slopes - Upstream	Vertical
Downstream	1V:1.3H

g. Principal Spillway -

Type: Concrete broad-crested weir	
Crest Length Perpendicular to Flow (Feet)	190.5
Crest Width Parallel to Flow (Feet)	3.6
Crest Elevation (Feet M.S.L.)	569.0

h. Auxiliary Spillway -

Type: Concrete broad-crested weir	
Total Crest Length Perpendicular to Flow (Feet)	22.3
Crest Width Parallel to Flow (Feet)	0.8
Crest Elevation (Feet M.S.L.)	576.6

i. Reservoir Drain -

Type: Two 6-foot diameter pipes, 15.4 feet below the spillway crest, with inoperative wooden gates. One 3-foot diameter pipe, 28.1 feet below the spillway crest, that appears to be plugged. One 12-inch cast iron pipe, 30.1 feet below the spillway crest, with a gate valve.

SECTION 2: ENGINEERING DATA

2.1 GEOLOGY

Honk Falls Dam is located along the western margin of the Rondout-Escpus Valley region, Valley and Ridge physiographic province - Hudson Lowland area. Bedrock in this region consists of faulted and folded Devonian and Silurian limestones, shales, and sandstones. Typical topographic relief in the region is 500 feet.

The Geologic Map of New York (Reference 2, Appendix D) indicates that bedrock units in the immediate vicinity of the dam are shales and siltstones of the undifferentiated Hamilton Group, Middle Devonian System. Outcrops of these units occur at the dam site.

The bedrock surface of Ulster County has been modified by the action of continental glaciation; therefore, most of the county is blanketed by ground moraine deposits.

There do not appear to be any geologic faults in the dam or reservoir area.

2.2 SUBSURFACE INVESTIGATION

No site-specific subsurface information was available for review by this investigation. However, as shown on the Field Sketch in Appendix E, extensive outcrops of steeply dipping bedrock are located immediately downstream from the dam. It can be assumed with a high degree of confidence therefore, that the dam is constructed primarily on bedrock.

The Soil Survey of Ulster County, New York (Reference 3, Appendix E) classifies the soil in the valley below the dam in the Arnot-Oquaga-Rock outcrop complex. These bouldery soils occur on steep slopes and are moderately to excessively well-drained loams with thickness up to 26 inches. They are said to be moderately permeable and with very rapid runoff characteristics. Free water, except for brief periods in spring and after heavy rain, is generally below the bedrock surface.

2.3 DAM AND APPURTENANT STRUCTURES

Honk Falls Dam was originally built around 1898 to provide power for a paper mill. Presently, the dam is

used for recreation purposes only. The dam is a concrete gravity dam approximately 42 feet high and 294 feet long. The dam has a 190.5-foot long main spillway which begins 20 feet from the left[1] abutment. It also has two auxiliary spillways on the right side of the dam. Beneath the auxiliary spillways are two 6-foot diameter outlet pipes. There is a 12-inch outlet pipe under the left side of the main spillway and a 3-foot diameter pipe under the right side. It appears that the dam is founded on bedrock, at least under the main spillway section. The auxiliary spillway and the right abutment for the dam may be founded on bedrock or soil.

The existing dam is illustrated by a field sketch which is included in Appendix E.

2.4 CONSTRUCTION RECORDS

No construction records were found during this investigation.

2.5 OPERATION RECORDS

No operation records were found during this investigation.

2.6 EVAULATION OF DATA

Engineering data was obtained entirely from files of the New York State Department of Environmental Conservation. The available data, while very limited, is considered adequate and reliable for Phase I Inspection purposes, with the exception that foundation conditions and details of the upstream face of the dam are not well-known.

[1]Looking downstream.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The inspection was performed on 8 March 1981. The weather was cloudy with temperatures in the low 30's. The reservoir surface was at the spillway crest.

b. Spillway - The principal spillway begins 20 feet from the left abutment. The spillway is a concrete broad-crested weir with a freeboard of 10.1 feet. The concrete on the spillway is spalled and generally deteriorated, and the construction joints are badly eroded.

c. Auxiliary Spillway - On the right side of the dam are two stepped-weir auxiliary spillways with a crest length of 10.1 feet and 7.2 feet. Both are broad-crested weirs with a freeboard of 2.5 feet. Trees and brush are growing in the discharge channel below the auxiliary spillway.

d. Dam - The dam is a concrete gravity structure 294 feet long with a height of 42.2 feet and a crest width of 2.5 feet. The concrete is spalled and deteriorated over the entire surface. The construction joints in the principal spillway are badly eroded with the copper seal visible in several areas. There are cracks in the downstream face of the auxiliary spillway and minor seeps are present (Photo 5, Appendix A). The wing wall along the right abutment downstream from the dam has seepage (5 to 10 gpm) at the soil-concrete contact along most of its length (Photo 6, Appendix A). There are three depressions, one at the upstream face of the dam and two behind the right wing wall that are possibly associated with the seepage under the right wing wall. The left wingwall has some seepage at its junction with the main portion of the dam. Some brush is growing in the joint between the upper and lower sections of the dam.

The rock-concrete interface at the toe of the dam is eroded in several areas (Photo 4, Appendix A). The rock is hard and appears tight, however, due to water over the spillway, it was not possible to see if any seepage was flowing beneath the dam.

e. Outlet Works - Two 6-foot diameter outlet pipes are below the auxiliary spillway (Photo 7, Appendix A). Wooden gates on the upstream ends of the pipes were used to control the flow. Both gates are leaking and have water flowing over their downstream face. The left gate has four valves mounted on its downstream side (Photo 8, Appendix A) and its lift arm is missing. A 3-foot diameter outlet pipe below the right side of the spillway appears to have been plugged. A 12-inch outlet pipe below the left side of the spillway has a gate valve on the downstream end that is leaking around the valve stem. None of the outlet works are operable.

f. Downstream Channel - The spillway discharges directly into the main channel; it is a deep rock channel with large boulders. The channel is extremely narrow from the dam to a point 1500 feet downstream where there is a sharp drop in the streambed forming a waterfall.

A home and a highway bridge are located 1500 feet downstream from the dam; several additional homes are located 1 mile downstream in Napanoch, New York.

g. Reservoir - The slopes of the reservoir are moderate to steep with woods and good cover. There were no signs of instability, and sedimentation did not appear to be a problem.

3.2 EVALUATION

Visual inspection revealed several deficiencies in the structure. The following items were noted:

1. The right wing wall has seepage at the soil-concrete contact along most of its length;

2. The wooden gates on the 6-foot diameter outlet pipes have water running down their face.

3. The mechanical equipment for the lift gates is inoperative.

4. The lift arm for the left gate on the 6-foot diameter outlet pipe is missing.

5. The left wingwall has seepage at the junction with the main structure of the dam.

6. There is seepage through the dam on the downstream face below the auxiliary spillways.

7. The junction of the concrete and rock at the toe of the dam is eroded.

8. The construction joints in the dam are badly eroded.

9. The entire concrete surface of the dam is spalled and deteriorated.

10. The valve on the 12-inch outlet pipe is leaking around the valve stem.

11. Brush is growing in the joint between the upper and lower sections of the dam.

12. Trees and brush are growing in the discharge channel below the auxiliary spillway.

13. Brush and trees are growing at the toe of the dam on the right abutment.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURES

4.1 PROCEDURES

There are no formal operating procedures. The operation of the dam is an automatic function controlled by the crest of the spillway.

4.2 MAINTENACE OF THE DAM

Maintenance of the dam is considered to be poor. The dam is not visually inspected and no maintenance has been performed for many years.

4.3 WARNING SYSTEM

There is no formal warning system or emergency action plan in effect.

4.4 EVALUATION

Maintenance and operating procedures for Honk Falls Dam are considered to be inadequate.

It is recommended that a maintenance program be implemented, and that formal records of examinations and necessary maintenance be recorded for future reference. A warning system and emergency action plan should be developed and put into operation. A staff gage should be installed to monitor reservoir levels above normal pool.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 DRAINAGE AREA CHARACTERISTICS

The drainage area for Honk Falls Dam is 104.09 square miles. Included in this drainage area is Merriman Dam (Fed. I.D. No. NY 74) which has a drainage area of 95 sq. mi. The delineation of that portion of the watershed below Merriman Dam was made using the USGS quadrangle for Rondout Reservoir, New York. The drainage basin between Merriman Dam and Honk Falls Dam consists of moderate to steep slopes which are well-covered with forests and ground vegetation. Some upland storage exists in the form of flat and swampy areas.

5.2 ANALYSIS CRITERIA

The hydraulic capacity of the dam, reservoir, and spillway was assessed by utilizing the U.S. Army Corps of Engineers' Flood Hydrograph Package, HED-1 DB (Reference 11, Appendix D). The hydrologic characteristics of the watershed below Merriman Dam, specifically the Snyder's unit hydrograph parameters, were average values obtained from Hydrologic Flood Routing Model for Lower Hudson River Basin (Reference 14, Appendix D). Rainfall losses were estimated at an initial loss of 1.0 inch and a constant loss of 0.1 inch per hour thereafter.

The outflow hydrograph for Merriman Dam was developed using material presented in the Phase I Inspection Report prepared for Merriman Dam by Justin and Courtney, Inc. (see Appendix F). The outflow hydrograph for Merriman Dam was routed to Honk Lake. The hydrograph for the drainage area between Merriman Dam and Honk Falls Dam was developed and combined with the hydrograph from Merriman Dam. The resulting hydrograph was then routed through Honk Falls Dam. The Probable Maximum Flood (PMF) and 1/2 Probable Maximum Flood (1/2 PMF) were developed and routed through the reservoir.

5.3 SPILLWAY CAPACITY

With the reservoir level at the minimum top of dam, the combined capacity of the service and auxiliary spillways were determined to be 20,122 c.f.s.

5.4 RESERVOIR CAPACITY

The storage capacity of Honk Falls Dam at normal pool is 860 acre-feet. The storage capacity of the reservoir

at the minimum top of dam is 1504 acre-feet. Therefore, flood control storage of the reservoir between the spillway crest and minimum top of dam is 644 acre-feet. This volume represents a total of 0.12 inch of runoff from the watershed.

5.5 FLOOD OF RECORD

No information concerning the effects of significant floods on the dam is available.

5.6 OVERTOPPING POTENTIAL

The maximum combined capacity of the spillways is 20,122 c.f.s. to the minimum top of dam. The peak outflows of the PMF and 1/2-PMF are 92,425 c.f.s. and 42,711 c.f.s., respectively. Therefore, the spillways are capable of passing 26 percent of the PMF before overtopping would occur.

Analyses of the dam and spillway show that the dam will be overtopped during the 1/2-PMF by a maximum depth of 5.11 feet for a total duration of 12.0 hours. The PMF results in overtopping by a maximum depth of 10.61 feet for a total duration of 18.0 hours.

5.7 RESERVOIR EMPYTING POTENTIAL

The reservoir has no operable outlet pipes to drawdown the reservoir.

5.8 EVALUATION

It was determined that the spillway is capable of passing 26 percent of the PMF before overtopping the dam. Structural stability analyses based on available information indicate that factors of safety against overturning and sliding are less than the recommended guidelines for all conditions overtopping the dam. The spillway is, therefore, judged to be "seriously inadequate".

Conclusions pertain to present conditions and the effect of future development on the hydrology has not been considered.

SECTION 6: STRUCTURE STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations - A number of significant deficiencies related to the stability of the structure were noted during the visual inspection. These include:

1. The face of the main spillway is spalled and badly deteriorated. The copper seals in the construction joints are visible on the spillway crest, and the joints are eroded on the downstream face of the spillway.

2. The downstream toe is eroded with exposed rebar on both ends of the main spillway. It is unknown if water is flowing along the rock concrete contact under the dam because water was flowing over the spillway at the time of inspection.

3. While there are erosion channels and depressions in the exposed bedrock downstream of the dam, the bedrock appears tight.

4. The downstream face of the auxiliary spillways is cracked, and minor seepage is occurring.

5. The right wingwall next to the auxiliary spillway is cracked and has seepage at the soil-wall contact point. There are three depressions behind this wingwall which appear to be a result of this seepage.

6. Water is leaking around the wooden gates for the two 6-foot diameter pipes below the auxiliary spillway.

b. Design and Construction Data - No design information was available regarding the stability of the structure.

c. Operating Records - Operating records are not available.

d. Post Construction Changes - There were no known changes made in the dam, except that it is no longer used to provide power.

6.2 STRUCTURAL STABILITY ANALYSIS

The results of a previous stability analysis, if any, were not available for reference during this evaluation. A structural stability analysis has been conducted for the maximum section of the main spillway. The cases analyzed and respective results are as follows:

Case	Description of Loading Conditions
1	Normal operating conditions with reservoir at the spillway crest, full uplift and no tailwater.
2	Same as Case 1 with the addition of ice loading of 5000 pounds per lineal foot.
3	Reservoir level during 1/2-PMF (elev. 584.2 M.S.L.), with full uplift, and a tailwater of 9.1 feet.
4	Reservoir level during the PMF (elev. 589.7 M.S.L.), with full uplift, and a tailwater of 12.1 feet.

Case	Factor of Safety		Location of Resultant
	Overturning	Sliding	From Toe (ft.)
1	1.54	2.69	5.89
2	1.24	2.33	3.25
3	0.85	1.40	-3.79
4	0.64	0.97	-14.86

Notes: Location of middle 1/3 is 7.3 to 14.7 feet from downstream toe.

A negative sign above indicates that the resultant falls downstream of the toe.

In all cases analyzed, the factors of safety against overturning are low and the locations of the resultants fall outside of the middle 1/3. Therefore, the dam is not considered safe against overturning. In addition, the factor of safety against sliding was low for all cases. However, the structure has withstood normal loading conditions in the past without apparent structural damage and the analyses may not indicate the true field conditions or proper loading condition. Therefore, it is recommended that an in-depth engineering study of the structure be conducted to determine actual stability conditons prior to initiating any remedial measures.

6.3 SEISMIC STABILITY

The dam is lcoated in Seismic Zone 1 which presents no hazard from earthquakes, according to the Recommended Guidelines for Safety Inspection of Dams. This determination is contingent on the requirement that static stability conditions are satisfactory and conventional safety margins exist.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety - The Phase I Inspection of Honk Falls Dam revealed that the spillway is "seriously inadequate," based on the Corps of Engineers screening criteria: outflows from any storm in excess of 26 percent of the PMF will overtop the dam. For this reason, the dam has been assessed as unsafe, non-emergency.

The classification of "unsafe" means that, based on an initial screening and preliminary computations, there appears to be a serious deficiency in spillway capacity, so that is a severe storm were to occur, overtopping and failure of the dam would take place, significantly increasing the hazard to loss of life downstream.

Structural stability analyses based on available information indicate that factors of safety against overturning and sliding are low for all conditions analyzed. When the dam is subjected to severe loading conditions such as ice loading or water levels overtopping the dam, the factors of safety are below recommended levels.

b. Adequacy of Information - The information available and the observations and measurements made during the visual inspection are considered sufficient for this Phase I Inspection Report.

c. Need for Additional Investigation - Detailed hydrologic and hydraulic investigations of the structure are considered necessary to more accurately determine the overtopping potential of the dam and to determine appropriate mitigating measures in response to the spillway inadequacy. The reservoir should be drawn down to allow for a visual inspection of the upstream face of the dam and to check for seepage under the dam without water flowing over the spillway. Investigate the areas of seepage and determine their effect on the structural stability of the dam. A detailed stability analysis of the dam is considered necessary to determine actual stability conditions.

d. Urgency - The detailed hydrologic and hydraulic investigations and stability analysis must be initiated within three months of notification to the owner. Within one year, remedial measures

resulting from these investigations must be initiated, with completion of these measures during the following year. In the interim, a detailed emergency action plan must be developed and implemented during periods of unusually heavy precipitation. Also, around-the-clock surveillance must be provided during these periods.

7.2 RECOMMENDED MEASURES

The following remedial measures must be completed within one year:

1. The gates on the outlet pipes should be repaired, and their controls should be made operable.

2. Determine the source of and repair the area of seepage at the soil-concrete contact along the right wing wall.

3. Determine the source of and repair the area of seepage at the left wingwall.

4. Determine the cause of and repair the seepage below the auxiliary spillways.

5. Repair the eroded concrete at the toe of the dam and at the construction joints.

6. Repair all areas on the dam where the concrete is spalled and deteriorated.

7. Remove all brush and trees growing near the right abutment and on the dam.

8. Remove the trees and debris from the channel below the auxiliary spillways.

9. A program of periodic inspections and maintenance of the dam should be established. All inspections and maintenance should be recorded for future reference.

10. A staff gage should be installed to monitor reservoir levels above normal pool.

APPENDIX A

PHOTOGRAPHS

CONTENTS

Photo 1: Upstream Side of Dam

Photo 2: Downstream Face of Dam

Photo 3: Gate Works

Photo 4: Erosion at Toe on Left Abutment Showing Exposed Reinforcing Bars

Photo 5: Auxiliary Spillway and Wingwalls From Downstream Side

Photo 6: Seepage Along Right Wall

Photo 7: 6-Foot Diameter Outlet Pipes

Photo 8: Wooden Gate for Outlet Pipes

Note: Photographs were taken on 8 March 1981.

HONK FALLS DAM

Photo 1. Upstream Side of Dam
8 March 1981

Photo 2. Downstream Face of Dam
8 March 1981

Photo 3. Gate Works
8 March 1981

Photo 4. Erosion at Toe on Left Abutment
Showing Exposed Reinforcing Bars
8 March 1981

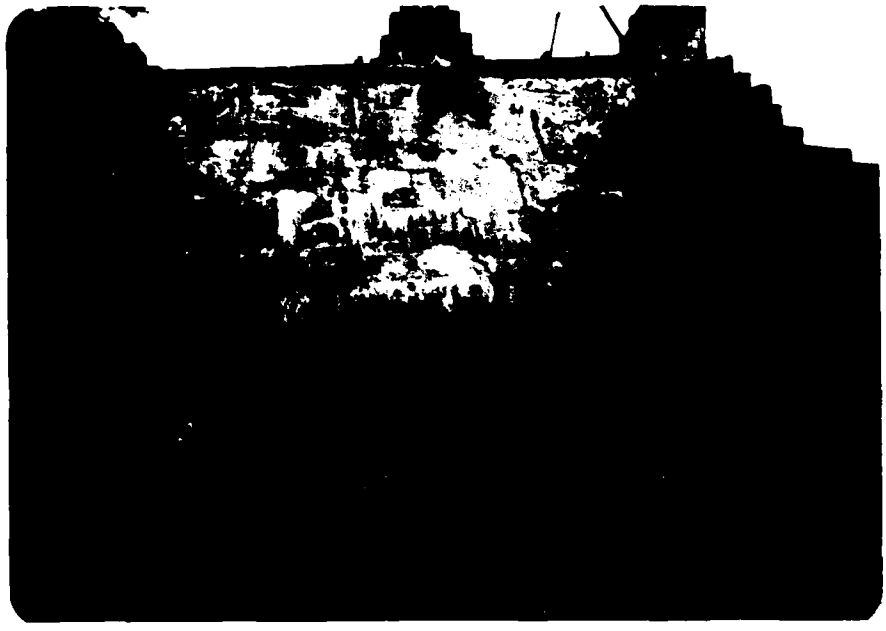

Photo 5. Auxiliary Spillway and Wingwalls
from Downstream Side
8 March 1981

Photo 6. Seepage Along Right Wall
8 March 1981

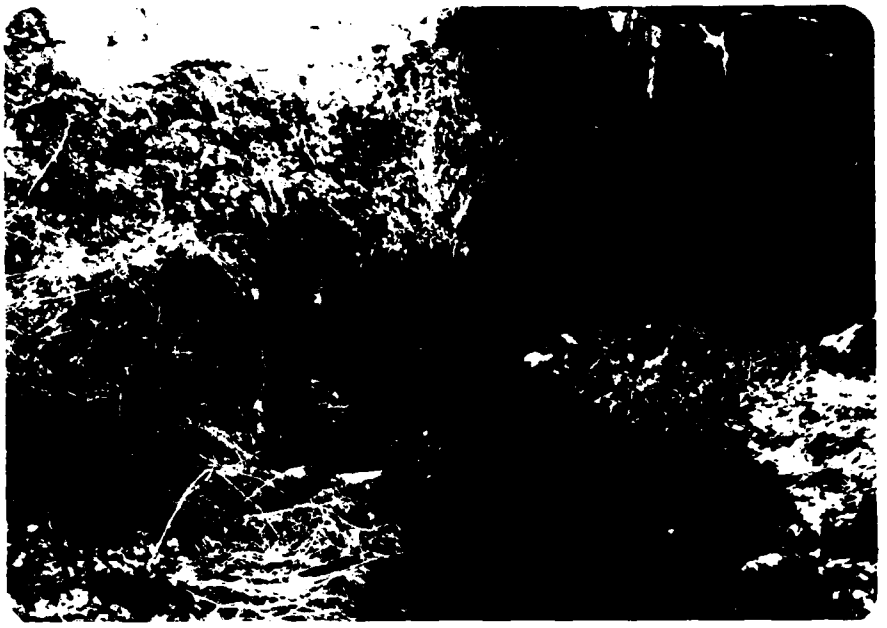

Photo 7. 6-Foot Diameter Outlet Pipes
8 March 1981

Photo 8. Wooden Gate for Outlet Pipe
8 March 1981

APPENDIX B

VISUAL INSPECTION CHECKLIST

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam Honk Falls Dam

Fed. I.D. # NY 73 DEC Dam No. 177C-735

River Basin Lower Hudson River

Location: Town Napanoch County Ulster

Stream Name Rondout Creek

Tributary of Hudson River

Latitude (N) 41° 45.0' Longitude (W) 74° 22.9'

Type of Dam Concrete Gravity Dam

Hazard Category High

Date(s) of Inspection 8 March 1981

Weather Conditions Cloudy, Low 30's

Reservoir Level at Time of Inspection 569.0 ft.

b. Inspection Personnel Terry S. Hawk, Gary W. Todd, Larry A. Diday

c. Persons Contacted (Including Address & Phone No.) 914-647-8500

Albert I. Lonstein

Route 209

Terrace Hill

Ellenville, NY 12428

d. History:

Date Constructed About 1897-8 Date(s) Reconstructed

Designer Unknown

Constructed By Unknown

Owner Recycled Paper Corporation

2) Embankment - Not Applicable

a. Characteristics

(1) Embankment Material _____

_____

(2) Cutoff Type _____

_____

(3) Impervious Core _____

_____

(4) Internal Drainage System _____

_____

(5) Miscellaneous _____

_____

b. Crest

(1) Vertical Alignment _____

_____

(2) Horizontal Alignment _____

_____

(3) Surface Cracks _____

_____

(4) Miscellaneous _____

_____

c. Upstream Slope

(1) Slope (Estimate) (V:H) _____

_____

(2) Undesirable Growth or Debris, Animal Burrows _____

_____

(3) Sloughing, Subsidence, or Depressions _____

_____

_____

(4) Slope Protection _____

_____

_____

(5) Surface Cracks or Movement at Toe _____

_____

d. Downstream Slope

(1) Slope (Estimate - V:H) _____

_____

(2) Undesirable Growth or Debris, Animal Burrows _____

_____

_____

(3) Sloughing, Subsidence or Depressions _____

_____

_____

(4) Surface Cracks or Movement at Toe _____

_____

(5) Seepage _____

_____

_____

(6) External Drainage System (Ditches, Trenches, Blanket) _____

_____

_____

(7) Condition Around Outlet Structure _____

_____

(8) Seepage Beyond Toe _____

e. Abutments - Embankment Contact _____

(1) Erosion at Contact _____

(2) Seepage Along Contact _____

3) Drainage System

a. Description of System None observed

b. Condition of System _____

c. Discharge from Drainage System _____

4) Instrumentation (Monumentation/Surveys, Observation Wells, Weirs, Piezometers, Etc.) _____

None

5) Reservoir

a. Slopes Moderate, to steep wooded slopes.

b. Sedimentation None observed

c. Unusual Conditions Which Affect Dam Merriman Dam is 5 mi. upstream.

6) Area Downstream of Dam

a. Downstream Hazard (No. of Homes, Highways, etc.) 1,500 ft. below the dam is a waterfall, a house, and a highway bridge. There are also several homes in Napanoch approximately 1 mile downstream.

b. Seepage, Unusual Growth None observed at the time of inspection.

c. Evidence of Movement Beyond Toe of Dam None observed at the time of inspection.

d. Condition of Downstream Channel Narrow rock channel with large boulders.

7) Spillway(s) (Including Discharge Conveyance Channel)

Overall condition is poor.

a. General The spillway is 184.5 ft. long with 2 auxiliary spillways at the right side of the dam.

b. Condition of Service Spillway The concrete spillway is spalled and deteriorated. The construction joints are badly eroded.

c. Condition of Auxiliary Spillway The 2 stepped auxiliary spillways are in fair condition with minor spalling on the concrete surfaces.

d. Condition of Discharge Conveyance Channel Discharges into main channel. Trees and brush are growing in the discharge channel below the auxiliary spillway.

8) Reservoir Drain/Outlet

Type: Pipe 2 Conduit 2 Other _____

Material: Concrete _____ Metal X Other _____

Size: 2 6-ft. diameter, 1 3-ft. diameter and 1 12-in. diameter Length 6-ft. diameter - 17 ft. long; 3-ft. & 12-in. diameter-length unknown

Invert Elevations: Entrance unknown
6 ft. - 553.6 ft.
Exit 3 ft. - 540.9 ft.
12 in. - 538.9 ft.

Physical Condition (Describe): Unobservable

Material: Iron

Joints: Good Alignment Good

Structural Integrity: 6 ft. pipes are rusted with some sedimentation in pipe.

Hydraulic Capability:

Means of Control: Gate X Valve ____ Uncontrolled ____

Operation: Operable ____ Inoperable X Other ____

Present Condition (Describe): 6 ft. pipes have wooden gates with water running down face. Left gate has 4 pipes with valves set in gate. Left gate lift arm is missing. 3 ft. pipe is plugged. The 12-inch pipe has a gate valve with the stem leaking.

9) Structural

a. Concrete Surfaces The concrete is spalled, deteriorated, with eroded construction joints. Some areas are cracked with seeps; rock interfaces are being undercut by erosion.

b. Structural Cracking The spillway of gate section and wingwalls are cracking.

c. Movement - Horizontal & Vertical Alignment (Settlement) No signs of movement of main structure were observed.

d. Junctions with Abutments or Embankments Rock concrete interfaces have been eroded. Right upstream abutment has some erosion with soil contact, riprap area on right abutment upstream has large depression, also undercut near wingwall. Downstream riprap is broken up. Two depressions near

(Continued next page)

right abutment, one in riprap where rock has settled, one small depression behind wall possibly as a result of the seep under the wall.

e. Drains - Foundation, Joint, Face None observed at the time of inspection.

f. Water Passages, Conduits, Sluices See drains

g. Seepage or Leakage Around wooden gates in 6 ft. pipes. Right wall has seepage existing from soil-concrete contact along most of length. Seepage under right wall is estimated at 5 to 10 GPM and appears clear, some penetration less than 2 ft. under wall with possible piping. Spillway face above 6 ft. discharge pipes is seeping. Left wingwall has some seepage near joint with main structure.

h. Joints - Construction, etc. Joints badly eroded along spillway face, copper seal visible in several places. Some brush growing in joint between upper and lower sections.

i. Foundation Some water is flowing along rock fissures. Rock concrete interface is eroded away in several areas. Rock is hard and appears tight, however, due to water over spillway, can not see if any seepage exists between rock and concrete or within rock itself.

j. Abutments See above

k. Control Gates Wood with water flowing down face.

l. Approach & Outlet Channels Approach channel unobservable. Outlet channel contains old broken concrete channel. Water has eroded an outlet in rock beyond wingwalls. The channel has trees and brush growing in it.

m. Energy Dissipators (Plunge Pool, etc.) None man-made. Erosion has created some pools.

n. Intake Structures

o. Stability Structure does not appear to have moved.

p. Miscellaneous Entire structure appears in poor condition, only real exit for water is over spillway causing dam to have a full lake.

10) Appurtenant Structures (Power House, Lock, Gatehouse, Other)

a. Description and Condition

Gate works are rusted and in poor condition.

Lift arm on left gate is missing.

APPENDIX C

HYDROLOGIC/HYDRAULIC

ENGINEERING DATA AND COMPUTATIONS

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject HONK FALLS DAM S.O. No. _____
APPENDIX C Sheet No. ____ of ____
HYDROLOGIC / HYDRAULIC CALC. Drawing No. _____
Computed by GWT Checked by _____ Date _____

SUBJECT	PAGE
CHECK LIST FOR DAMS	1
HYDRAULIC AND HYDROLOGIC DATA	5
TOP OF DAM PROFILE	8
TYPICAL CROSS SECTION	9
SPILLWAY RATING	10
AUXILIARY SPILLWAY RATING	12
COMBINED SPILLWAY RATING	14
SPILLWAY CAPACITY ANALYSIS	15
HEC-1 COMPUTER ANALYSIS	17

CHECK LIST FOR DAMS
HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA

AREA-CAPACITY DATA:

		Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1)	Top of Dam	579.1	86.2	1,504
2)	Design High Water (Max. Design Pool)	--	--	--
3)	Auxiliary Spillway Crest	576.6	75.8	1,345
4)	Pool Level with Flashboards	--	--	--
5)	Service Spillway Crest	569.0	44.1	860

DISCHARGES

		Volume (cfs)
1)	Average Daily	25
2)	Spillway @ Maximum High Water - Top of Dam -	19,876
3)	Spillway @ Design High Water	--
4)	Spillway @ Auxiliary Spillway Crest Elevation	12,895
5)	Low Level Outlet	--
6)	Total (of all facilities) @ Maximum High Water	20,122
7)	Maximum Known Flood	Unknown
8)	At Time of Inspection	25

CREST: ELEVATION: 579.1 ft.

Type: Concrete

Width: 2.5 ft. Length: 294 ft.

Spillover Broad-crested weir

Location Center of dam

SPILLWAY:

SERVICE		AUXILIARY
569.0 ft.	Elevation	576.6 ft.
Broad-crested weir	Type	Broad-crested weir
3.6 ft.	Width	0.8 ft.
	Type of Control	
X	Uncontrolled	X
	Controlled:	
--	Type (Flashboards; gate)	--
--	Number	--
--	Size/Length	--
	Invert Material	
	Anticipated Length of Operating Service	
	Chute Length	
20 ft.	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	27.6 ft.

HYDROMETEROLOGICAL GAGES:

Type: None

Location:

Records:

Date:

Max. Reading:

FLOOD WATER CONTROL SYSTEM:

Warning System: None

Method of Controlled Releases (mechanisms):

None

DRAINAGE AREA: 104.09 sq. mi.

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: Wooded

Terrain - Relief: Varies from flat to steep

Surface - Soil: Well-drained

Runoff Potential (existing or planned extensive alterations to existing surface or subsurface conditions)

There were no known plans for altering the existing runoff patterns at the time of inspection.

Potential Sedimentation problem areas (natural or man-made; present or future)

None observed. All slopes well-vegetated.

Potential Backwater problem areas for levels at maximum storage capacity including surcharge storage:

None observed at the time of inspection.

Dikes - Floodwalls (overflow & non-overflow) - Low reaches along the Reservoir perimeter:

Location: None

Elevation:

Reservoir:

Length @ Maximum Pool 2,500 ft.

Length of Shoreline (@ Spillway Crest) 7,200 ft.

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject NEW YORK DAMS — S.O. No. ____
HAWK FALLS DAM — Sheet No. 5 of 25
RAINFALL DATA — Drawing No. ____
Computed by WLS — Checked by GWT — Date 2/23/81

RAINFALL DATA

FROM HMR-33 —

DAM AND DRAINAGE AREA ARE IN ZONE 1

PMP (24 HR) 200 mi² = 21.3 IN.

DRAINAGE AREA = 9.083 SQ. MI.

PMP (6-HR) = 111% PMP (24 HR) 200 mi²
" (12-HR) = 123% " " "
" (24-HR) = 133% " " "
" (48-HR) = 142% " " "

FROM TP-40

100-YR, 24 HR RAINFALL = 7.8 IN.
" " 12 HR " = 6.5 IN.
" " 6 HR " = 5.3 IN.

Drainage Area above Roundout Reservoir = 95 Sq. Mi
Total D.A. above Hawk Falls = 104.09 Sq. Mi

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THE BAKER ENGINEERS

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Beaver, Pa. 15009

Subject LITTLE HONK CREEK — S.O. No.
HONK FALLS DAM — Sheet No. 6 of 25
Drawing No.
Computed by WLS — Checked by GWT — Date 2/[illegible]/81

HYDROLOGIC AND HYDRAULIC DATA

DRAINAGE AREA ABOVE HONK FALLS DAM NOT CONTROLLED BY RACCOUT RESERVOIR DAM = 63.31 SQ. MI. (MEASURED ON RACCOUT RESERVOIR, N.Y. QUAD = 9.068 SQ. MI.

L_{CA} = 17,200 FT = 3.26 MI
L = 48,600 FT = 9.20 MI

$T_p = C_T (L \times L_{CA})^{.3}$ $C_p = 0.63$
$= 2.0(9.2 \times 3.26)^{.3}$ $C_T = 2.0$
$T_p = 5.55$

STORAGE COMPUTATIONS

SURFACE AREA VS. ELEVATION MEASUREMENTS (TAKEN FROM QUAD)

ELEV. (FT.)	AREA (ACRES)	
569	44.1	→ NORMAL POOL AS SHOWN ON QUAD
580	90.0	
600	172.0	

DEPTH = 20 FT.

AT ELEV. 549 FT. AREA = 41.9 Ac. ASSUMING 1:1 S/S

$C_p = 0.63$, $C_T = 2.0$

$T_p = C_T (L \times L_{CA})^{.3}$
$= 2.0 (9.2 \times 3.26)^{.3}$
$T_p = 5.55$

ADJUSTMENT TO T_p FOR INTERVAL

$T_R = T_p/5.5 = 5.55/5.5 = 1.01$ HR INTERVAL USE 1 HR.

$T_{PR} = T_p + \frac{T_R - T_a}{4}$

$= 5.55 + \frac{1.01 - 1}{4}$

$T_{PR} = 5.55$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

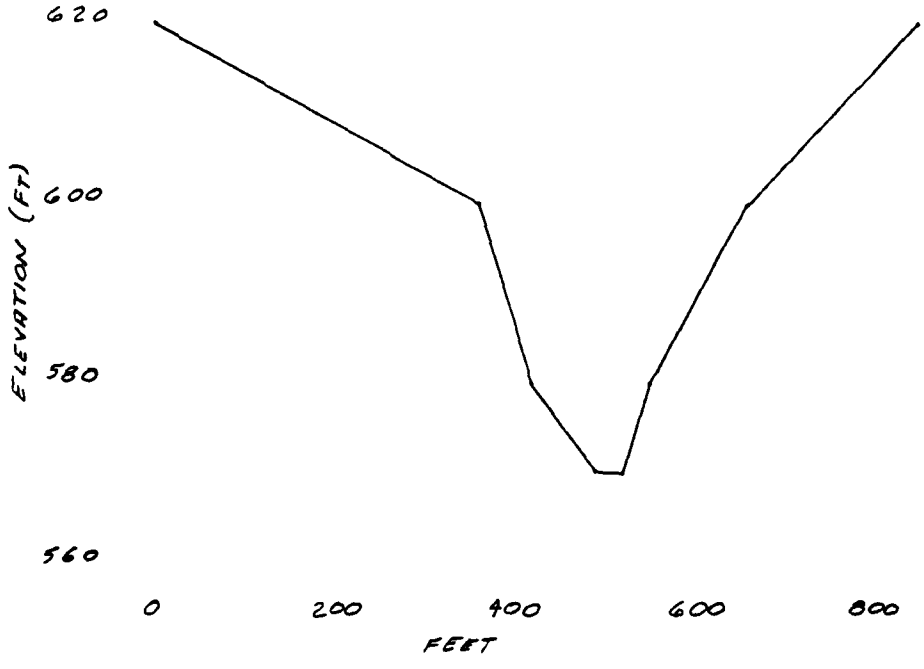
Subject COMPARISON OF DATA FROM MERRIMAN DAM (Roundout Reservoir) S.O. No. _____ Sheet No. 7 of 25 Drawing No. _____

Computed by GWT Checked by WDL Date 5/1/81

	MICHAEL BAKER, JR., INC.	JUSTIN AND COURTNEY, INC. PHASE I INSPECTION REPORT, 1978
DRAINAGE AREA	95 Sq. Mi.	95 Sq. Mi.
t_p / C_p	5.16 / 0.63	5.16 / 0.63
RAINFALL	24 HR. PMP	12 HR. PMP
PEAK OUTFLOW	84,597 C.F.S.	77,104 CFS

The hydrograph coefficients from the Merriman Dam Phase I Inspection Report were used to generate and match the outflow hydrograph from Merriman Dam for the 12 hour PMP. The 24 hour PMP values were then substituted to generate the PMF flows from Merriman Dam.

TYPICAL ROUTING CROSS SECTION
(CHANNEL FROM MERRIMAN DAM TO HONK FALLS)



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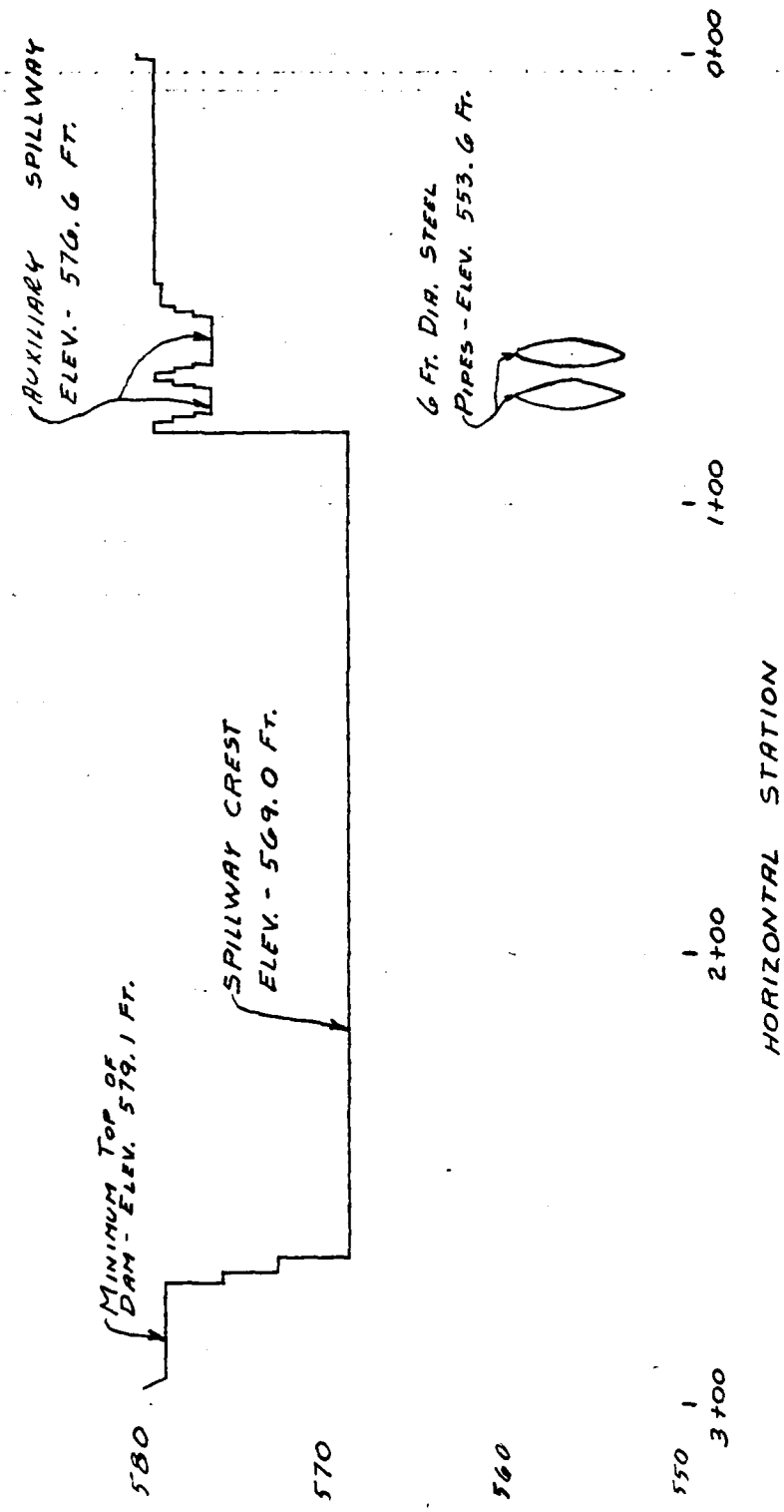
Beaver, Pa. 15009

Subject HONK FALLS DAM — S.O. No.
TOP OF DAM PROFILE — Sheet No. 8 of 25
Drawing No.
Computed by GWT — Checked by LAD — Date 3-11-81



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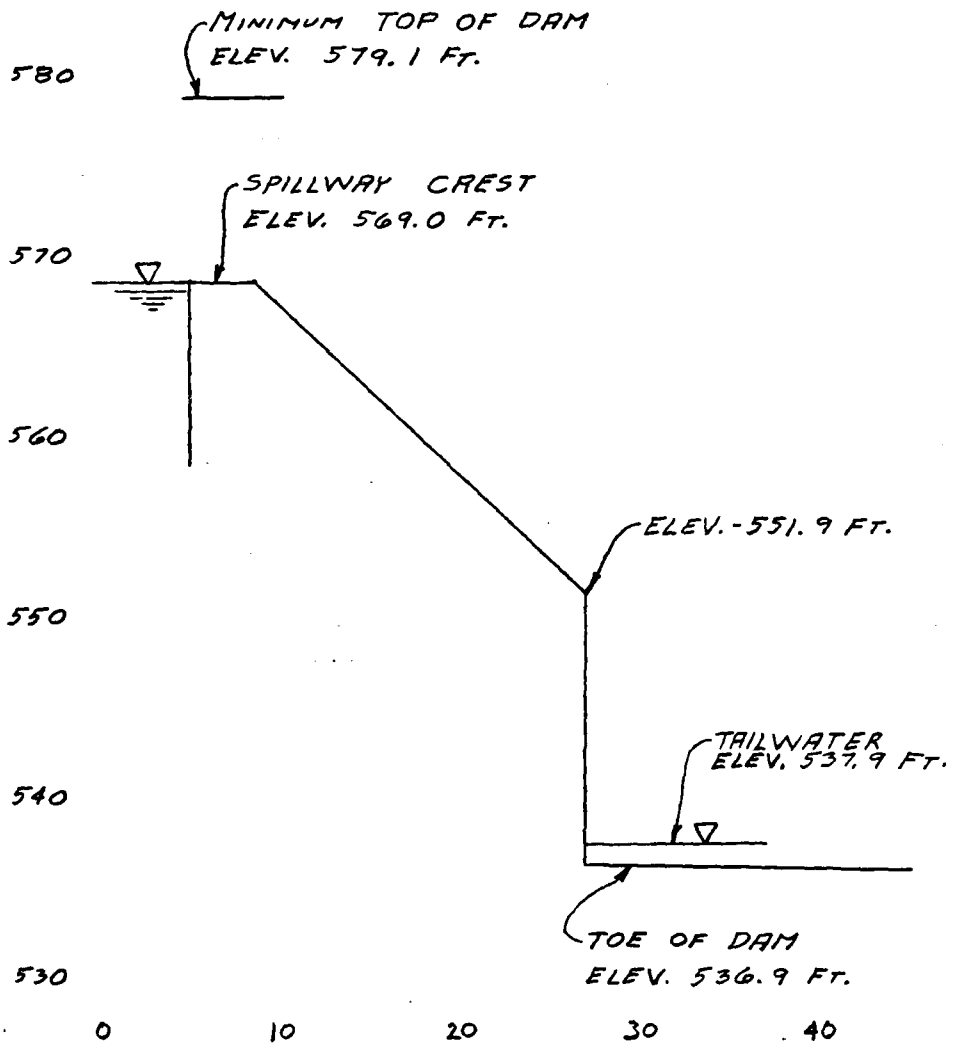
Subject HONK FALLS DAM S.O. No.
TYPICAL CROSS SECTION Sheet No. 9 of 25
Drawing No.
Computed by GWT Checked by LAD Date 3-11-81

TYPICAL CROSS SECTION AT STATION 2+14



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Subject HONK FALLS DAM S.O. No.
SPILLWAY RATING Sheet No. 10 of 25
Drawing No.
Computed by GWT Checked by LAD Date 3/11/81

WEIR FLOW /

$Q = CLH^{3/2}$

BREADTH OF CREST = 3.6 FT.
L = 184.5 FT
H VARIES FROM 0 FT. TO 21 FT.
C VARIES WITH H, Pg. 5-40 TABLE 5-3 BRATER & KING

ELEVATION, (FT)	H, / (FT)	L, / (FT)	C /	Q, (CFS)
569.0	0	184.5	0	0
570.0	1.0	184.5	2.67	492.6 /
571.0	2.0	184.5	2.68	1398.5 /
572.0	3.0	184.5	2.73	2617.2 /
573.0	4.0	184.5	2.79	4118.0 /
574.0	5.0	184.5	3.07	6332.7 /
575.0	6.0	184.5	3.32	9002.5 /
576.0	7.0	184.5	3.32	11,344.4 /
576.6	7.6	184.5	3.32	12,833.8 /
577.6	8.6	184.5	3.32	15,448.4 /
578.6	9.6	184.5	3.32	18,219.7 /
579.6	10.6	184.5	3.32	21,139.4 /
580.0	11.0	184.5	3.32	22,347.2 /
582.0	13.0	184.5	3.32	28,711.1 /
584.0	15.0	184.5	3.32	35,585.3 /
586.0	17.0	184.5	3.32	42,934.6 /
588.0	19.0	184.5	3.32	50,729.9 /
590.0	21.0	184.5	3.32	58,947.2 /

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Subject HOMER FALLS DAM S.O. No.
SPILLWAY RATING Sheet No. 11 of 25
(CONTINUED) Drawing No.
Computed by GWT Checked by LAD Date 3-19-81

WEIR FLOW

$Q = CLH^{3/2}$

BREADTH OF CREST = 3.6 FT. /
L = 3 FT. /
H VARIES

C VARIES WITH H, PG. 5-40
TABLE 5-3 BRATER + KING /

SPILLWAY STEP

ELEVATION (FT)	H (FT) /	L (FT) /	C /	Q (CFS)
573.0	0	3.0	0	0
574.0	1.0	3.0	2.67	8.0 /
575.0	2.0	3.0	2.68	22.7 /
576.0	3.0	3.0	2.73	42.6 /
576.6	3.6	3.0	2.79	57.2 /
577.6	4.6	3.0	3.07	90.9 /
578.6	5.6	3.0	3.32	131.9 /
579.6	6.6	3.0	3.32	168.9 /
580.0	7.0	3.0	3.32	184.5 /
582.0	9.0	3.0	3.32	268.9 /
584.0	11.0	3.0	3.32	363.4 /
586.0	13.0	3.0	3.32	466.8 /
588.0	15.0	3.0	3.32	578.6 /
590.0	17.0	3.0	3.32	698.1 /

SPILLWAY STEP

ELEVATION (FT)	H (FT) /	L (FT) /	C /	Q (CFS)
576.0	0	3.0	0	0
576.6	0.6	3.0	2.67	3.7 /
577.6	1.6	3.0	2.68	16.3 /
578.6	2.6	3.0	2.73	34.3 /
579.6	3.6	3.0	2.79	57.2 /
580.0	4.0	3.0	3.07	73.7 /
582.0	6.0	3.0	3.32	146.4 /
584.0	8.0	3.0	3.32	225.4 /
586.0	10.0	3.0	3.32	314.9 /
588.0	12.0	3.0	3.32	414.0 /
590.0	14.0	3.0	3.32	521.7 /

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Box 280
Beaver, Pa. 15009

Subject HONK FALLS DAM S.O. No.
AUXILIARY SPILLWAY RATING Sheet No. 12 of 25
Drawing No.
Computed by GWT Checked by LAD Date 3/11/81

WEIR FLOW

$Q = CLH^{3/2}$ /

RIGHT EMERGENCY SALLWAY

FLOW OVER CREST
ASSUMING VERTICAL SIDES

L = 17.3 FT. TOTAL CREST LENGTH COMBINING BOTH EMERGENCY SPILLWAYS /

H VARIES FROM 0 FT. TO 13.4 FT. /

C VARIES WITH H, TABLE 5-3 / BRATER + KING

LEFT EMERGENCY SPILLWAY

TOTAL CREST RATING

ELEVATION, (Ft)	H, (FT) /	L, (FT) /	C	Q, (CFS)
576.6	0	17.3	0	0
577.6	1.0	17.3	3.14	54.3 /
578.6	2.0	17.3	3.31	161.9 /
579.6	3.0	17.3	3.32	298.4 /
580.0	3.4	17.3	3.32	360.1 /
582.0	5.4	17.3	3.32	720.7 /
584.0	7.4	17.3	3.32	1,156.2 /
586.0	9.4	17.3	3.32	1,655.3 /
588.0	11.4	17.3	3.32	2,210.8 /
590.0	13.4	17.3	3.32	3,817.4 /

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject HONK FALLS DAM S.O. No.
AUXILIARY SPILLWAY RATING Sheet No. 13 of 25
(CONTINUED) Drawing No.
Computed by GWT Checked by LAD Date 3-11-81

FLOW OVER 1ST STEP

L = 2.5 FT TOTAL STEP WIDTH OF BOTH EMERGENCY SPILLWAYS
H VARIES FROM 0 FT. TO 12.4 FT.
C VARIES WITH H, TABLE 5-3 BRATER & KING
BREADTH OF WEIR = 2.5 FT.

TOTAL 1ST STEP RATING

ELEVATION (FT)	H (FT)	L (FT)	C	Q (CFS)
577.6	0	2.5	0	0
578.6	1.0	2.5	2.64	6.6
579.6	2.0	2.5	2.76	19.5
580.0	2.4	2.5	2.89	26.9
582.0	4.4	2.5	3.32	76.6
584.0	6.4	2.5	3.32	134.4
586.0	8.4	2.5	3.32	202.1
588.0	10.4	2.5	3.32	278.4
590.0	12.4	2.5	3.32	362.4

FLOW OVER 2ND STEP

L = 2.5 FT. TOTAL STEP WIDTH OF BOTH EMERGENCY SPILLWAYS
H VARIES FROM 0 FT. TO 11.4 FT.
C VARIES WITH H, TABLE 5-3 BRATER & KING
BREADTH OF WEIR = 2.5 FT.

TOTAL 2ND STEP RATING

ELEVATION (FT)	H (FT)	L (FT)	C	Q (CFS)
578.6	0	2.5	0	0
579.6	1.0	2.5	2.64	6.6
580.0	1.4	2.5	2.68	11.1
582.0	3.4	2.5	3.19	50.0
584.0	5.4	2.5	3.32	104.2
586.0	7.4	2.5	3.32	167.1
588.0	9.4	2.5	3.32	239.2
590.0	11.4	2.5	3.32	319.5

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Beaver, Pa. 15009

Subject HONK FALLS DAM S.O. No.
COMBINED SPILLWAY RATING Sheet No. 14 of 25
Drawing No.
Computed by GWT Checked by LAD Date 3-19-81

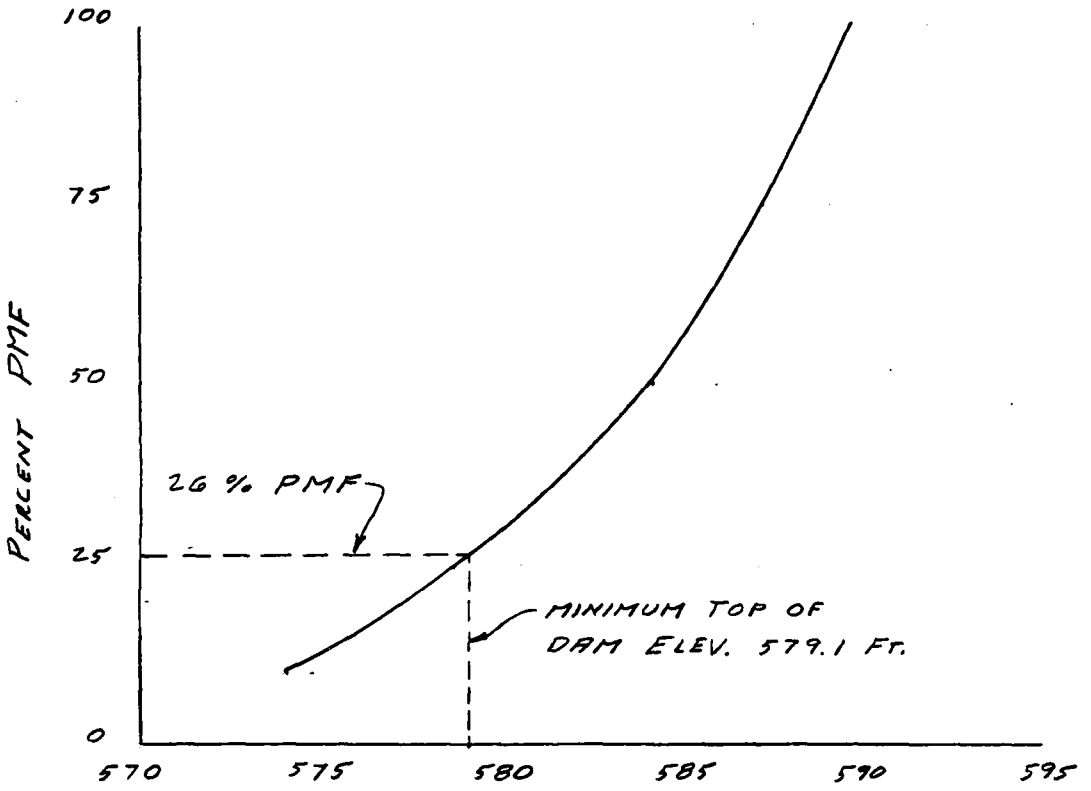
COMBINED SPILLWAY RATING

ELEVATION (FT)	SPILLWAY (CFS) ✓	SPILLWAY STEP (CFS) /	SPILLWAY STEP (CFS) /	AUXILIARY SPILLWAY CREST (CFS) /	AUXILARY SPILLWAY 1ST STEP (CFS) /	AUXILARY SPILLWAY 2ND STEP (CFS) /	TOTAL Q (CFS) /
569.0	0						0
570.0	492.6						492.6
571.0	1398.5						1398.5
572.0	2617.2						2617.2
573.0	4118.0	0					4118.0
574.0	6332.7	8.0					6340.7
575.0	9002.5	22.7					9025.2
576.0	11,344.4	42.6	0				11,387.0
576.6	12,833.8	57.2	3.7	0			12,894.7
577.4	15,448.4	90.9	16.3	54.3	0		15,609.9
578.6	18,219.7	131.9	34.3	161.9	6.6	0	18,554.4
579.6	21,139.4	168.9	57.2	298.4	19.5	6.6	21,690.0
580.0	22,347.2	184.5	73.7	360.1	26.9	11.1	23,003.5
582.0	28,711.1	268.9	146.4	720.7	76.6	50.0	29,973.7
584.0	35,585.3	363.4	225.4	1,156.2	134.4	104.2	37,568.9
586.0	42,934.6	466.8	314.9	1,655.3	202.1	167.1	45,740.8
588.0	50,729.9	578.6	414.0	2,210.8	278.4	239.2	54,450.9
590.0	58,947.2	698.1	521.7	2,817.4	362.4	319.5	63,666.3

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject HONK FALLS DAM S.O. No. _____
SPILLWAY CAPACITY ANALYSIS Sheet No. _ of 25
Drawing No. _____
Computed by GWT Checked by LAD Date 4/7/81



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**********
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
  4BJ UPDATE         04 JUN 79
**********
    1                A1      NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS
    2                A2      HYDROLOGIC AND HYDRAULIC ANALYSIS OF HONK FALLS DAM
    3                A3      UNIT HYDROGRAPH BY SNYDERS METHOD
    4                B    300       0      60       0       0       0       0       0      -4       0
    5                B1     5
    6                J      1       5       1
    7                J1   1.0     .75     .50     .25     .10
    8                K      0       1                               1
    9                K1     RUNOFF HYDROGRAPH TO ROUNDOUT RESERVOIR
   10                M      1       1      95
   11                P             21.3      85      98     109     115
   12                T                                                      0.10
   13                M   5.16    0.63
   14                X      0       0     1.0
   15                K      1       2                               1
   16                K1     ROUTING FOR MERRIMAN DAM
   17                Y                                1       1
   18                Y1     1                                        -840.0       0
   19                Y2     0    3420    7155   11935   17706   24197   47563   65915  118528  181739
   20                Y3     0    2085    4180    6285    8400   10525   16960   21300   32525   43800
   21                $S   7300  154000  164225  175300  186325  197000
   22                $E    720     840     849     850     855     860
   23                $$ 840.0     600     3.4     1.5
   24                $O    860    3.0     1.5    2500
   25                $L      0    2500
   26                $V    860   860.1
   27                K      1       3                               1
   28                K1     ROUTING THRU CHANNEL TO HONK LAKE
   29                Y                                1       1
   30                Y1     1                                        0
   31                Y6  0.08    0.04    0.08     570     620   27000  0.0035
   32                Y7     0     620     360     600     420     580     490     570     520     570
   33                Y7   550     580     660     600     850     620
   34                K      0       4                               1
   35                K1     RUNOFF HYDROGRAPH TO HONK LAKE
   36                M      1       1    9.09
   37                P             21.3     111     123     133     142
   38                T                                              1.0     0.10
   39                M   5.55    0.63
   40                X   -1.5   -0.05     2.0
   41                K      2       5                               1
   42                K1     COMBINE HYDROGRAPHS FROM STATIONS 3 AND 4
   43                K      1       6                               1
   44                K1     ROUTING FOR HONK FALLS DAM
   45                Y                                1       1
   46                Y1     1                                       -569.0      -1
   47                Y4   569     570     571     572     573     574     575     576   576.6   577.2
   48                Y4 578.6   579.5     580     582     584     586     588     590
   49                Y5     0   492.6  1398.5  2617.2  4118.0  5840.7  7825.2 11367.0 12834.7 15609.9
   50                Y5 17954   21690  23003.5  25973.7  37568.4  45740.3  54490.7  63060.3
```

51	$A	41.9	44.1	90.0	172.0					
52	$E	549	569	5EC	600					
53	$T	569								
54	$J	579.1	3.0	1.5	80					
55	$L	0	25	25	75	77	92	150	450	700
56	$V	579.1	579.3	579.7	579.8	580	581	582	585	590
57	K	99								

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**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  28 FEB 79
  ABJ UPDATE         04 JUN 79
**********************************

RUN  DATE 05/23/31
     TIME    07.27
```

NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS
HYDROLOGIC AND HYDRAULIC ANALYSIS OF HONK FALLS DAM
UNIT HYDROGRAPH BY SNYDERS METHOD

JOB SPECIFICATION

NO	NHR	NMIN	IDAY	IPR	IMOD	METRC	IPLT	IPRT	ISTAN
300	0	60	0	0	0	0	0	-4	0

	JOPER	NRT	LROPT	TRACE
	5	0	0	0

MULTI-PLAN ANALYSES TO BE PERFORMED
NPLAN= 1 NRTIO= 5 LRTIO= 1

RTIOS= 1.00 0.75 0.50 0.25 0.10

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SUB-AREA RUNOFF COMPUTATION

RUNOFF HYDROGRAPH TO ROUNDOUT RESERVOIR

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

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	95.00	0.0	95.00	0.0	0.0	0	0	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.0	21.30	85.00	90.00	100.00	115.00	0.0	0.0

TRSPC COMPUTED BY THE PROGRAM IS 0.866

LOSS DATA

LKOPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.0	0.0	1.00	0.0	0.0	1.00	0.0	0.10	0.0	0.0

UNIT HYDROGRAPH DATA
TP= 5.16 CP=0.63 NTA= 0

RECESSION DATA
STRTQ= 0.0 QRCSN= 0.0 RTIOR= 1.00

UNIT HYDROGRAPH 29 END-OF-PERIOD ORDINATES, LAG= 5.13 HOURS, CP= 0.64 VOL= 1.00

513.	2330.	4102.	6039.	7303.	7559.	6693.	5307.	4290.	3440.
2754.	2209.	1764.	1412.	1130.	905.	724.	580.	464.	371.
297.	238.	190.	152.	122.	98.	78.	63.		

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

HYDROGRAPH ROUTING

ROUTING FOR MERRIMAN DAM

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

ROUTING DATA

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

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.0	0.0	0.0	-840.	0

STORAGE	0.0	3420.00	7155.00	11993.00	17708.00	24197.00	47533.00	65915.00	116328.00	161739.00
OUTFLOW	0.0	2085.00	4180.00	6285.00	8400.00	10525.00	16900.00	21303.00	32325.00	43503.00

CAPACITY= 7300. 154000. 164525. 175300. 186325. 197600.

ELEVATION= 720. 840. 845. 850. 855. 860.

CREL	SPWID	CCOW	EXPW	ELEVL	COOL	CAREA	EXPL
840.0	600.0	3.4	1.5	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
860.0	3.0	1.5	2500.

CREST LENGTH AT OR BELOW ELEVATION 0. 2500.
860.0 860.1

PEAK OUTFLOW IS 34597. AT TIME 47.00 HOURS

PEAK OUTFLOW IS 61530. AT TIME 47.00 HOURS

PEAK OUTFLOW IS 38854. AT TIME 47.00 HOURS

PEAK OUTFLOW IS 17566. AT TIME 49.00 HOURS

PEAK OUTFLOW IS 5367. AT TIME 49.00 HOURS

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HYDROGRAPH ROUTING

ROUTING THRU CHANNEL TO MERRIMAN LAKE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO

	5	1	C	0	0	0	1	0	0

ROUTING DATA

JTYPE	QLOSS	AVG	IRES	ISAME	IOPT	IPMP		LSTR
0.0	0.0	0.0	1	1	0	0		0

NSTPS	NSTDL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	C	C	0.0	0.0	0.0	0.	0

NORMAL DEPTH CHANNEL ROUTING

JNEXT	QMAXI	QMAXO	ELNVT	ELMAX	RLNTH	SEL
1.0300	3.0400	0.0800	570.0	620.0	27300.	0.00360

CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC

0.0	620.00	360.00	600.00	420.00	580.00	490.00	570.00	520.00	570.00
550.00	580.00	660.00	600.00	850.00	620.00				

STORAGE	0.0	70.35	163.69	339.92	530.93	776.98	1033.92	1301.03	1733.32	2091.50
	2511.43	2967.88	3475.42	4094.42	4831.45	5686.50	6693.58	7750.67	8994.79	10236.73
OUTFLOW	0.0	402.20	1336.83	3500.08	6802.17	11894.19	18830.78	25825.93	34879.38	44876.79
	56336.99	69260.94	82824.31	98057.19	115023.44	135508.63	157918.38	183029.00	211015.31	242350.25
STAGE	570.00	572.63	575.26	577.89	580.53	583.16	585.79	588.42	591.05	593.68
	596.31	598.94	601.58	604.21	606.84	609.47	612.10	614.73	617.36	620.00
FLOW	0.0	402.20	1336.83	3500.08	6802.17	11894.19	18830.78	25825.93	34879.38	44876.79
	56336.99	69260.94	82824.31	98057.19	115023.44	135508.63	157918.38	183029.00	211015.31	242350.25

MAXIMUM STAGE IS 601.7

MAXIMUM STAGE IS 597.2

MAXIMUM STAGE IS 592.1

MAXIMUM STAGE IS 589.4

MAXIMUM STAGE IS 579.7

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

SUB-AREA RUNOFF COMPUTATION

RUNOFF HYDROGRAPH TO HONK LAKE

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPNT	INAME	ISTAGE	IAUTO
4	C	0	0	0	0	1	0	0

HYDROGRAPH DATA

IHYDG	IUNG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISMOM	ISAME	LOCAL
1	1	9.00	0.0	9.09	9.0	0.0	0	0	0

PRECIP DATA

SPFE	PAS	R6	R12	R24	R48	R72	R96
0.0	21.30	111.00	123.00	133.30	142.30	0.0	0.0

TRSPC COMPUTED BY THE PROGRAM IS 0.800

	0.0	0.0	1.00	0.0	0.0	1.00	1.00	0.00	0.0	0.0

UNIT HYDROGRAPH DATA
TP= 5.55 CP=0.63 NTA= 0

RECESSION DATA
STRTQ= -1.50 QRCSN= -0.05 RTIOR= 2.00

UNIT HYDROGRAPH 30 END-OF-PERIOD ORDINATES, LAG= 5.57 HOURS, CP= 0.63 VOL= 1.00

46.	161.	332.	498.	621.	670.	627.	526.	431.	353.
289.	236.	193.	158.	130.	106.	87.	71.	58.	48.
39.	32.	26.	21.	17.	14.	12.	10.	8.	6.

END-OF-PERIOD FLOW

J											
00.00	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP J	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP J
								SUM		24.20	20.50	3.70	125706.
										615.11	521.11	94.11	3559.60)

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

COMBINE HYDROGRAPHS

COMBINE HYDROGRAPHS FROM STATIONS 3 AND 4

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

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

HYDROGRAPH ROUTING

ROUTING FOR HONK FALLS DAM

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
6	1	0	0	0	0	1	0	0

ROUTING DATA

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

NSTPS	NSTOL	LAG	AMSKK	X	TSK	STORA	ISPRAT
1	0	0	0.0	0.0	0.0	-559.	-1

STAGE	569.00	570.00	571.00	572.00	573.00	574.00	575.00	576.00	577.00	577.60
	578.00	579.00	580.00	582.00	584.00	586.00	588.00	590.00		
FLOW	0.0	492.60	1398.50	2617.20	4119.00	6340.70	9029.20	11387.00	12994.70	15604.90
	18524.00	21690.00	25003.50	29973.70	37508.90	45740.80	54450.90	63666.30		
SURFACE AREA=	42.	44.	90.	172.						
CAPACITY=	0.	960.	1581.	4154.						
ELEVATION=	569.	569.	580.	600.						

	CREL	SPWID	COQW	EXPW	ELEVE	COQL	CAREA	EXPI
	569.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

EXIT DATA
IOPTE 0.0 EXPO SAVQ TO

SHEET 21 OF 25

				579.1	3.0	1.5	60.		
CREST LENGTH	0.	25.	25.	75.	77.	92.	137.	450.	735.
AT OR BELOW ELEVATION	579.1	579.2	579.7	579.8	580.0	581.0	582.0	585.0	590.0

PEAK OUTFLOW IS 92910. AT TIME 47.00 HOURS

PEAK OUTFLOW IS 67583. AT TIME 47.00 HOURS

PEAK OUTFLOW IS 42884. AT TIME 48.00 HOURS

PEAK OUTFLOW IS 13271. AT TIME 48.00 HOURS

PEAK OUTFLOW IS 6415. AT TIME 49.00 HOURS

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SHEET 22 OF 35

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

OPERATION	STATION	AREA	PLAN	RATIO 1 1.00	RATIO 2 0.75	RATIO 3 0.50	RATIO 4 0.25	RATIO 5 0.10
HYDROGRAPH AT	1	95.00 (246.05)	1	104925. (2971.26)	78691. (2228.44)	52464. (1485.63)	26232. (742.81)	10493. (297.13)
ROUTED TO	2	95.00 (246.05)	1	84557. (2395.52)	61596. (1744.22)	38864. (1100.50)	17566. (497.43)	5307. (150.25)
ROUTED TO	3	95.00 (246.05)	1	83687. (2369.75)	60788. (1721.32)	38719. (1096.39)	17377. (492.07)	5299. (150.20)
HYDROGRAPH AT	4	9.09 (23.54)	1	11325. (320.68)	8493. (240.51)	5662. (160.34)	2831. (80.17)	1132. (32.07)
2 COMBINED	5	104.09 (269.59)	1	93147. (2637.62)	67838. (1920.96)	42719. (1209.66)	19377. (548.70)	6441. (182.38)
ROUTED TO	6	104.09 (269.59)	1	92910. (2630.92)	67583. (1913.73)	42834. (1214.33)	19271. (545.68)	6419. (181.78)

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 .....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	840.00	849.00	863.50
STORAGE	15.000.	15.000.	197000.
OUTFLOW	0.	0.	152483.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	851.98	0.0	179867.	34597.	0.0	47.00	0.0
0.75	849.77	0.0	174046.	61596.	0.0	47.00	0.0
0.50	847.13	0.0	166122.	38364.	0.0	47.00	0.0
0.25	844.20	0.0	162843.	17261.	0.0	48.00	0.0
0.10	842.02	0.0	154267.	9067.	0.0	49.00	0.0

PLAN 1 STATION 3

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	33687.	601.7	47.00
0.75	60764.	597.6	46.00
0.50	33719.	592.1	46.00
0.25	17377.	585.4	48.00
0.10	5759.	579.7	50.00

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	565.00	5[illegible].00	579.10
STORAGE	860.	860.	15[illegible].
OUTFLOW	0.	0.	20122.

RATIO OF PMF	MAXIMUM RESERVOIR W.S.ELEV	MAXIMUM DEPTH OVER DAM	MAXIMUM STORAGE AC-FT	MAXIMUM OUTFLOW CFS	DURATION OVER TOP HOURS	TIME OF MAX OUTFLOW HOURS	TIME OF FAILURE HOURS
1.00	589.73	10.64	2632.	92910.	18.00	47.00	0.0
0.75	587.34	8.25	2341.	67583.	15.00	47.00	0.0
0.50	584.23	5.13	1975.	42834.	12.00	48.00	0.0
0.25	578.83	0.0	1480.	19271.	0.0	48.00	0.0
0.10	574.03	0.0	1128.	6419.	0.0	49.00	0.0

SHEET 25 OF 25

APPENDIX D

REFERENCES

REFERENCES

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5. Chow, Ven Te, Handbook of Applied Hydrology, McGraw - Hill Book Company, New York, 1964.

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14. George, Thomas S. and Taylor, Robert S., Hydrologic Flood Routing Model For Lower Hudson River Basin, Water Resources Engineers, Inc., 8001 Forbes Place, Suite 312, Springfield, Virginia, January 1977.

15. U.S. Army, Office of the Chief of Engineers, Engineering Circular EC-1110-2-163 (Draft Engineering Manual), "Spillway and Freeboard Requirements for Dams, Appendix C, Hydrometeorological Criteria and Hyetograph Estimates," (August 1975).

16. U.S. Army, Office of the Chief of Engineers, Engineering Circular EC-1110-2-188, "Engineering and Design, National Program of Inspection of Non-Federal Dams," Corps of Engineers, Washington, D.C., 30 December 1977.

17. U.S. Army, Office of the Chief of Engineers, Engineer Technical Letter No. ETL 1110-2-234, "Engineering and Design, National Program of Inspection of Non-Federal Dams, Review of Spillway Adequacy," Corps of Engineers, Washington, D.C., 10 May 1978.

18. Terzaghi, Karl, and Peck, Ralph B., Soil Mechanics in Engineering Practice 2d., ed., John Wiley & Sons, Inc., New York, 1967.

APPENDIX E

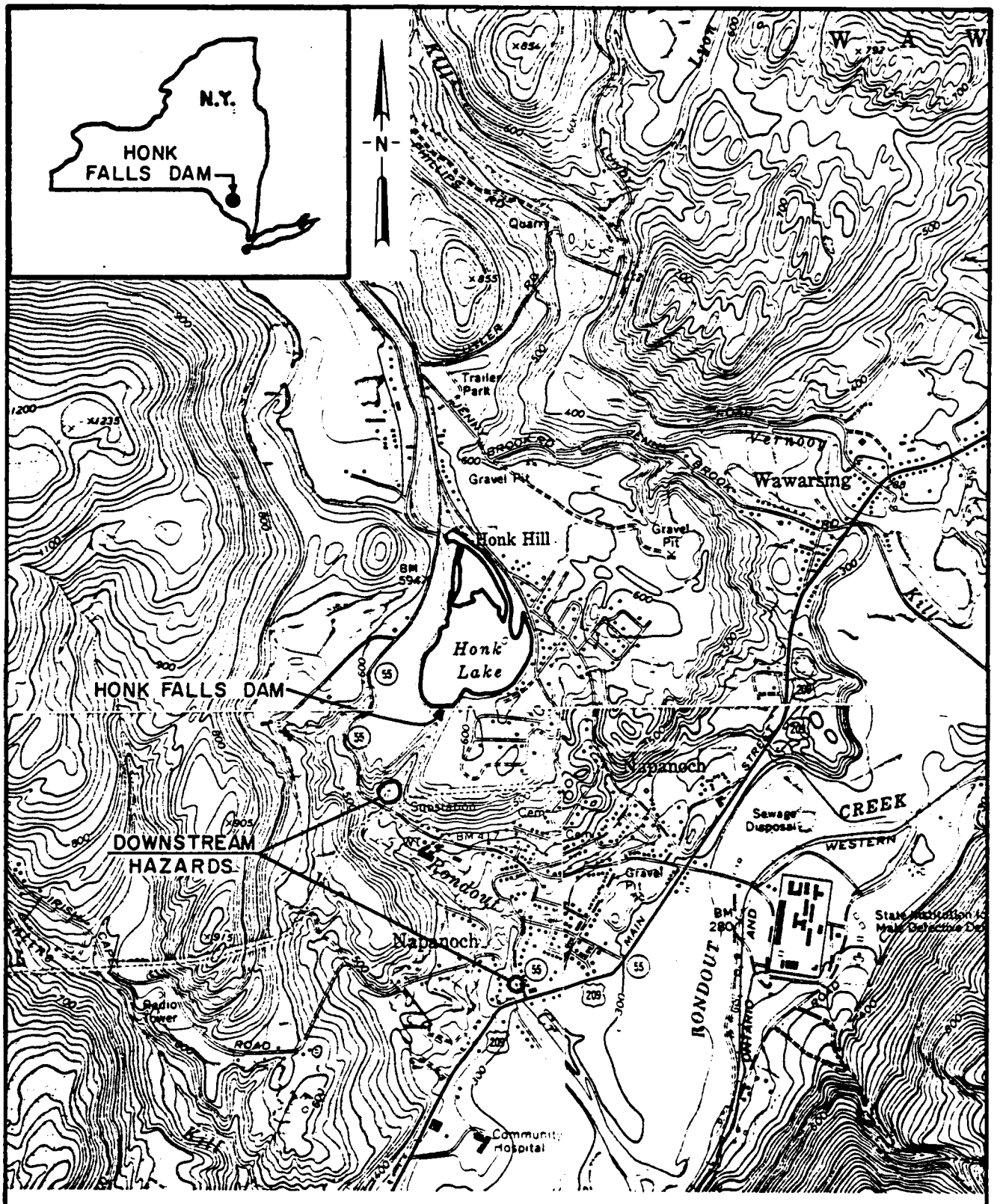
DRAWINGS

CONTENTS

Location Plan

Watershep Map

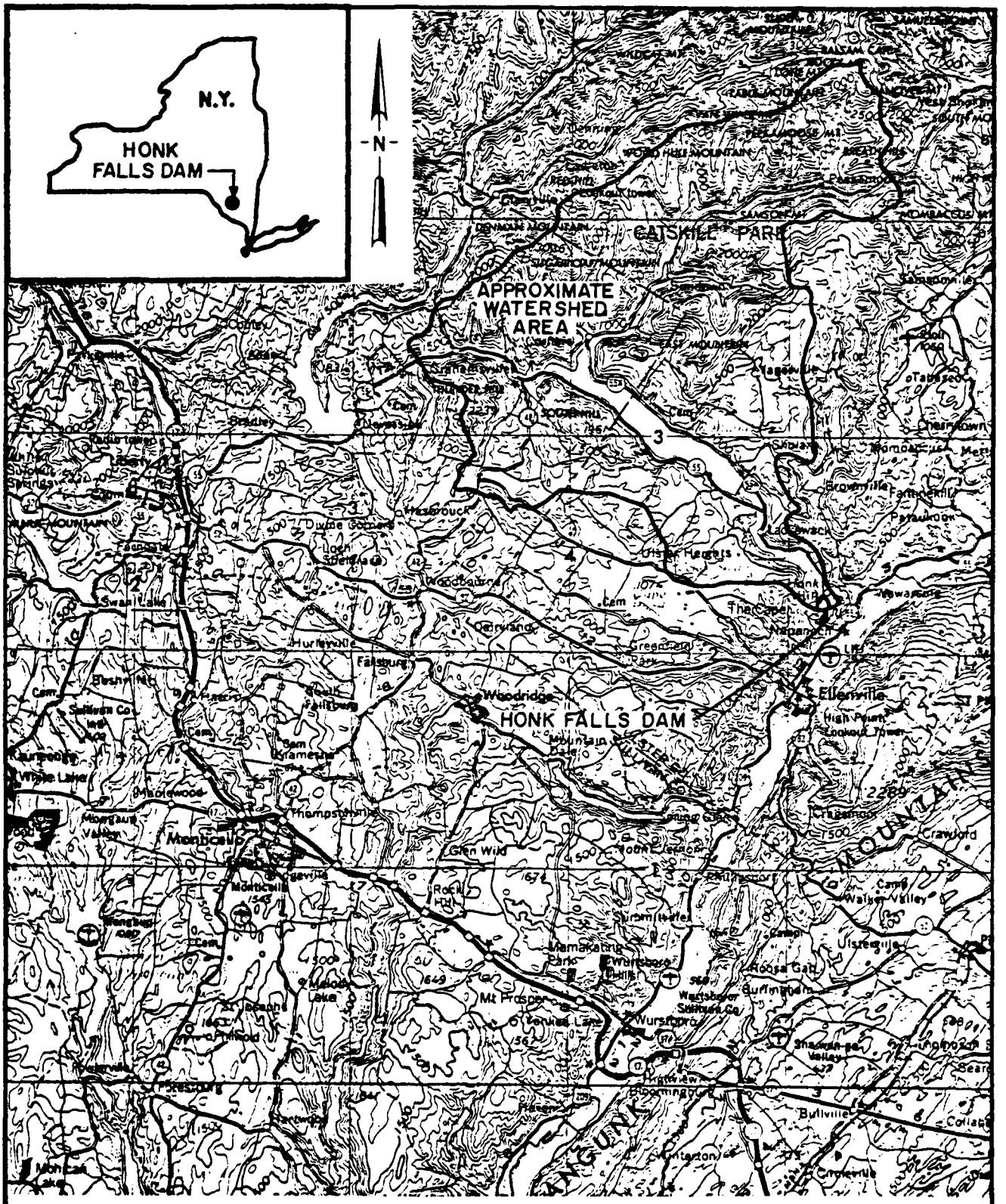
Field Sketch



REFERENCES:

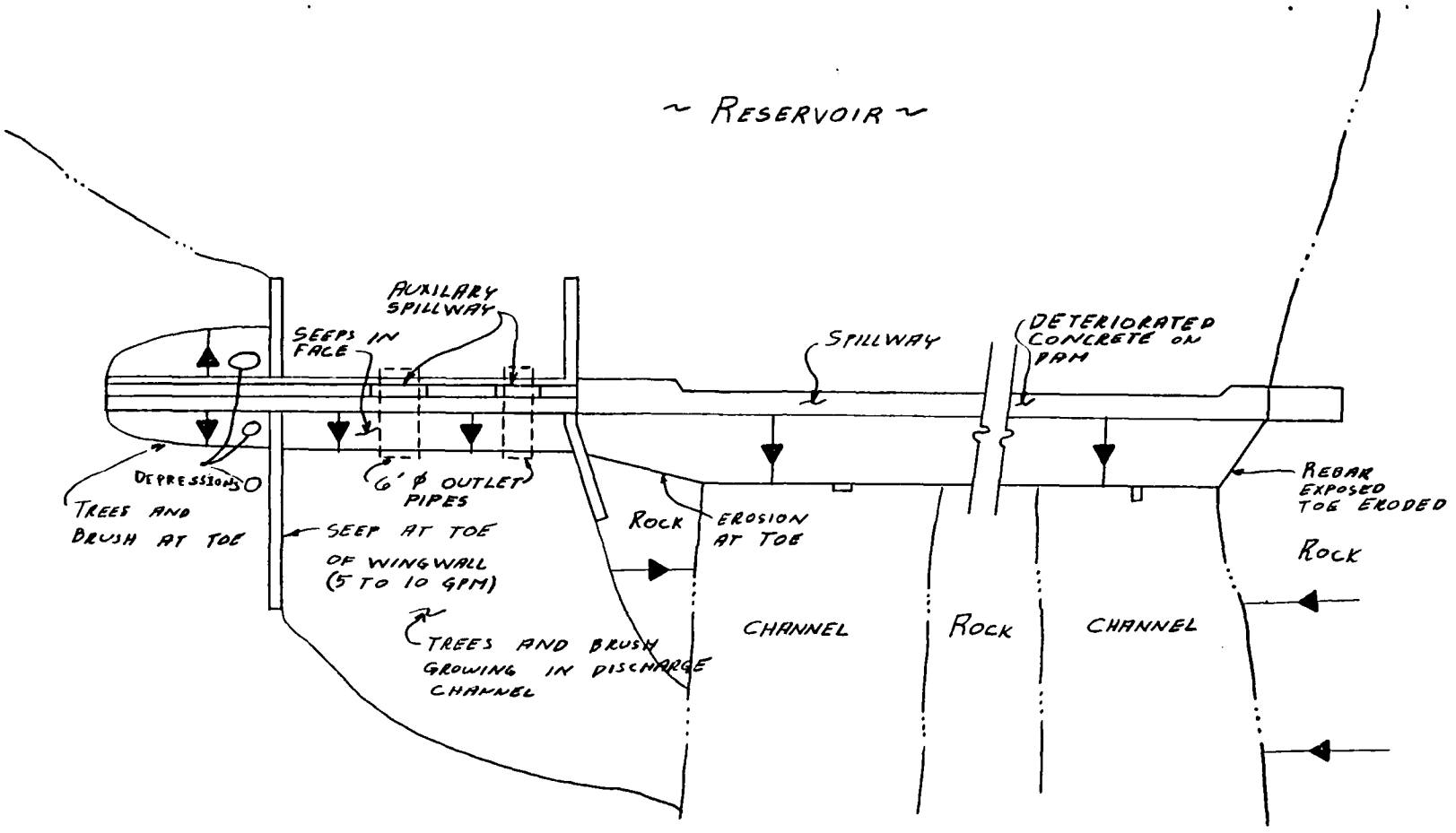
1. U.S.G.S. 7.5 NAPANOCH, N.Y. QUADRANGLE. 1956
2. U.S.G.S. 7.5' RONDOUT RESERVOIR, N.Y. QUADRANGLE. 1969
3. U.S.G.S. 7.5' KERHONKSON, N.Y. QUADRANGLE. 1969
4. U.S.G.S. 7.5' ELLENVILLE, N.Y. QUADRANGLE. 1969

LOCATION PLAN
HONK FALLS DAM



REFERENCES:
1. U.S.G.S. 1:250,000 SCRANTON, PA., N.Y., N.J. QUADRANGLE. REVISED 1976

WATERSHED MAP
HONK FALLS DAM



HONK FALLS DAM
FIELD SKETCH

APPENDIX F

BACKGROUND DOCUMENTS

Wap 178
Dam 735
LH

FORM IW31. 6 18 18 2000 (18-15288)

(NOTICE: After filling out one of these forms as completely as possible for each dam in your district, return it at once to the Conservation Commission, Albany.)

STATE OF NEW YORK

CONSERVATION COMMISSION

ALBANY

DAM REPORT

August 6, 1914
(Date)

CONSERVATION COMMISSION,

DIVISION OF INLAND WATERS.

GENTLEMEN:

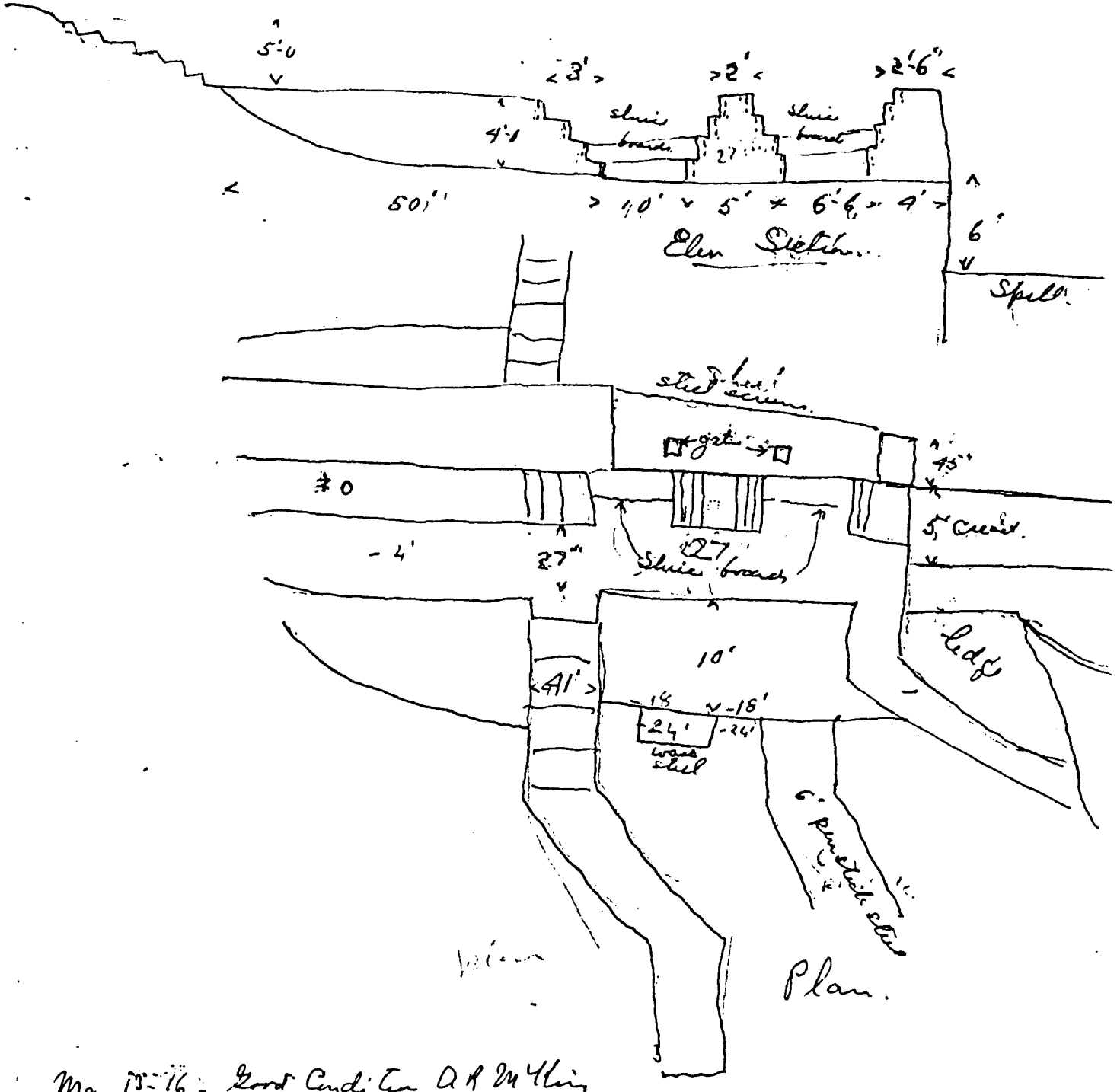
I have the honor to make the following report in relation to the structure known as the Hook Falls Dam.

This dam is situated upon the Kinderhook Creek
(Give name of stream)
in the Town of Canaan, Columbia County,
about 1/2 miles from the Village or City of Valatie.
(State distance)
The distance down stream from the dam, to the Albany Kinderhook Rail
(Up or down) (Give name of nearest important stream or of a bridge)
is about 2 miles. Hook Falls Power Co
(State distance)

The dam is now owned by Charles F. Luckenbach
(Give name in full)
and was built in or about the year 1897, and ~~was~~ extensively repaired or reconstructed ~~during the year~~.

As it now stands, the spillway portion of this dam is built of concrete
(State whether of masonry, concrete or timber)
and the other portions are built of concrete
(State whether of masonry, concrete, earth or timber with or without rock fill)

As nearly as I can learn, the character of the foundation bed under the spillway portion of the dam is rock formation and under the remaining portions such foundation bed is gravel.



May 13-16 - Good Condition A.R. McKim
3" water over crest

The total length of this dam is ~~250~~ 200 feet. The spillway or waste-weir portion, is about 175 feet long, and the crest of the spillway is about 6 feet below the top of the dam.

The number, size and location of discharge pipes, waste pipes or gates which may be used for drawing off the water from behind the dam, are as follows: one 6" thru & one 6" discharge waste pipes from west end.

State briefly, in the space below, whether, in your judgment, this dam is in good condition, or bad condition, describing particularly any leaks or cracks which you may have observed.)

Dam is in good condition. There are numerous leaks thru the concrete, largest about an inch in diameter, but these make less insignificant

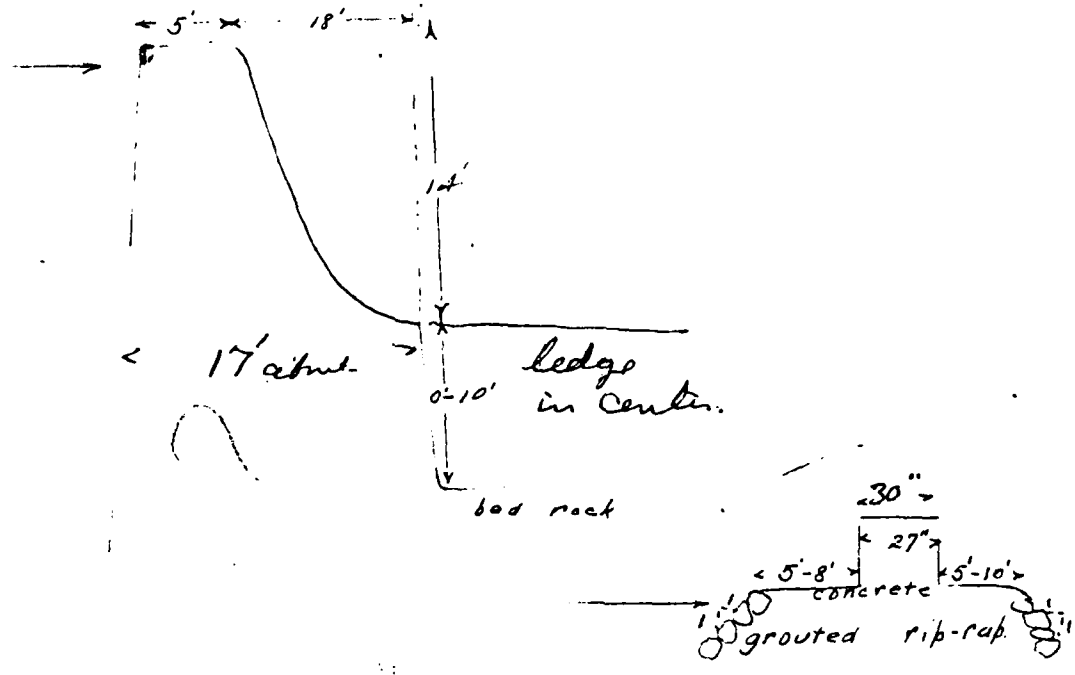
Reported by Richard A. Hyde
(Signature)

(Address—Street and number, P. O. Box or R. F. D. route)

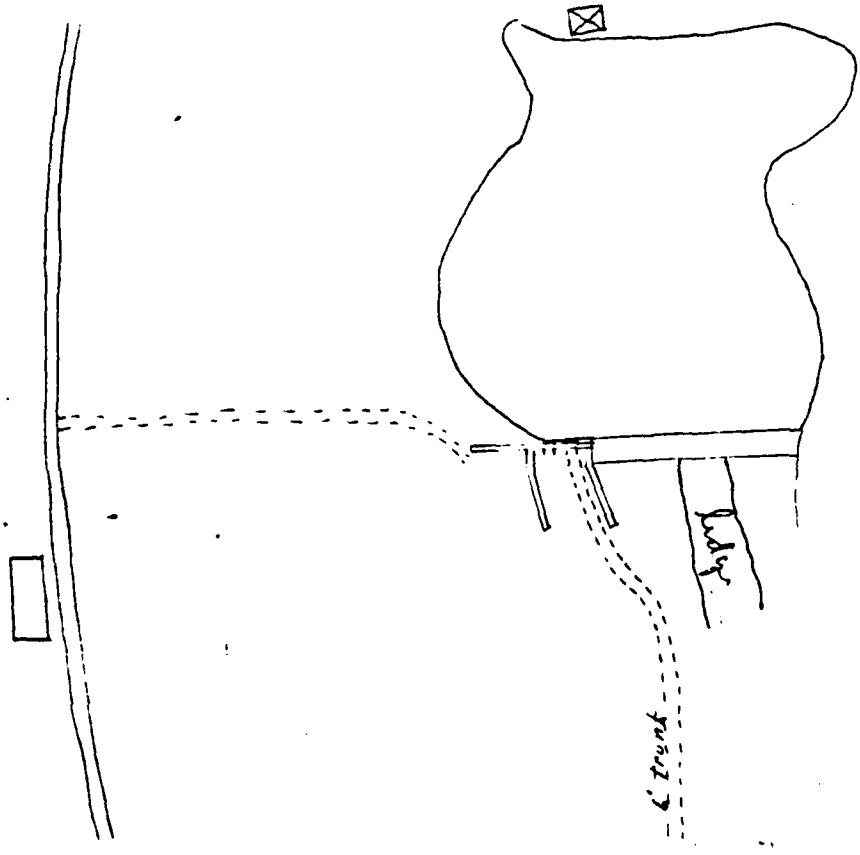
(Name of place)

(SEE OTHER SIDE)

(In the space below, make one sketch showing the form and dimensions of a cross section through the spillway or waste-weir of this dam, and a second sketch showing the same information for a cross section through the other portion of the dam. Show particularly the greatest height of the dam above the stream bed, its thickness at the top, and thickness at the bottom, as nearly as you can learn.)



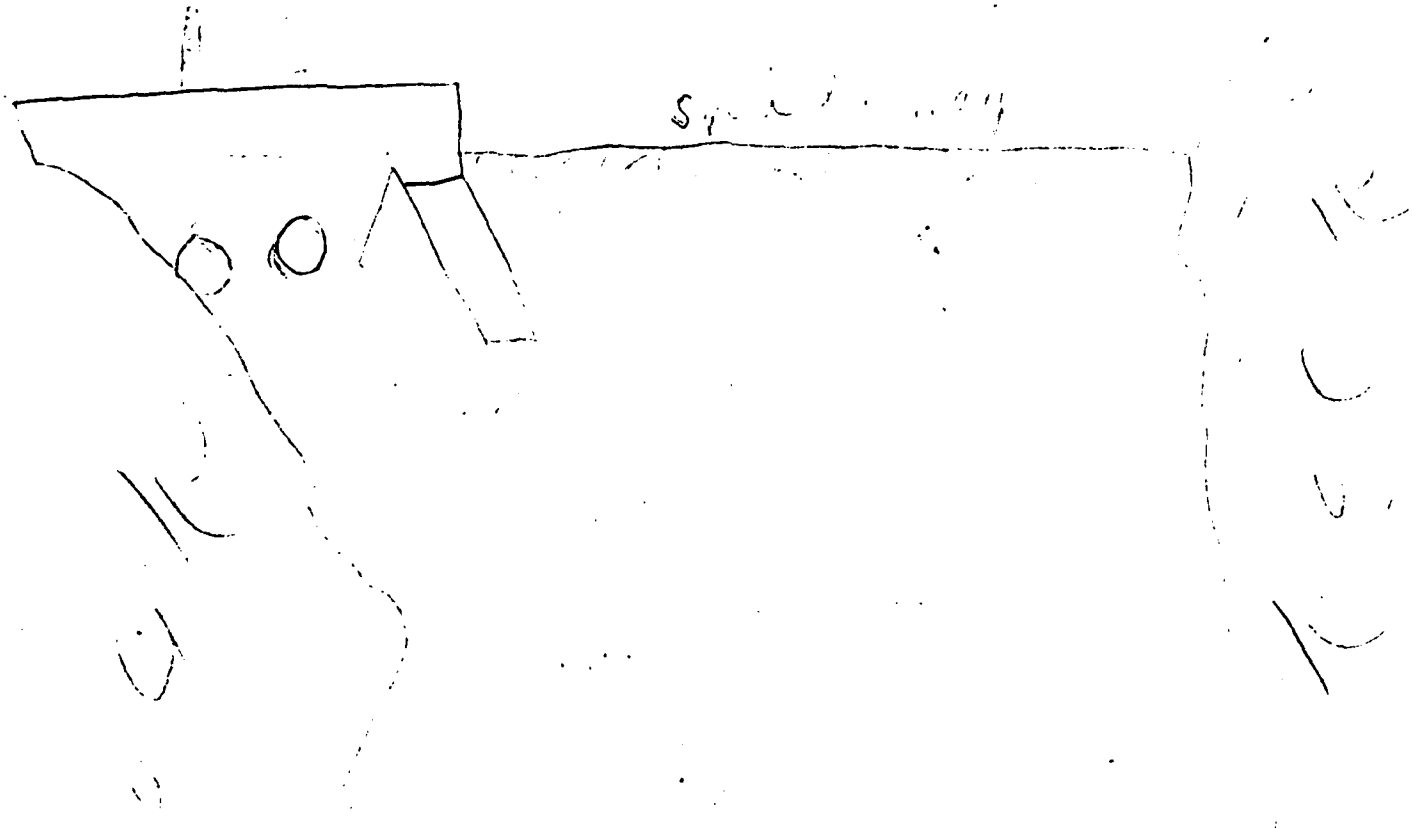
(In the space below, make a third sketch showing the general plan of the dam, and its approximate position in relation to buildings or other conspicuous objects in the vicinity.



State Conservation Commission
Albany N. Y.

GENTLEMEN:

Regarding our Dam at HONK FALLS near NAPANOCH N. Y.
OWNED BY C. P. DICKINSON
FITCHBURG MASS.
BUILT 1898
FOR a Hydro-electric plant
ON rock foundation
DAM and SPILLWAY, of concre te
Length of Spillway 200 ft
Total length of Dam and Spillway, 275 ft
Discharges through two 6ft Steel Penstocks 13 ft below top of Dam.



CONSERVATION COMMISSION
APR 6 1912
RECEIVED

MERRIMAN DAM NY 74

U.S. OF HONK FALLS DAM

SECTION 3 - HYDROLOGY/HYDRAULICS

The design flood used for Merriman Dam and Rondout Reservoir is the Probable Maximum Flood (PMF), according to the Recommended Guidelines for Safety Inspection of Dams. The PMF was derived from the adjusted 12 hour Probable Maximum Precipitation (PMP) and was routed through the reservoir using the U.S. Army Corps of Engineers computer program HEC-1. From this analysis, peak outflows and storages were determined for various percentages of the PMF. The routing analysis revealed that 23,645 acre-feet of water would be stored for the peak PMF outflow of 77,104 cfs. The storage input, as derived on sheet #A-16 in the appendix, shows that this storage corresponds to a reservoir elevation of 851.0, 11.0 feet above spillway crest and 9.0 feet below the top of the dam. Therefore, there is no danger of overtopping from a storm resulting in a flood equal to or less than the Probable Maximum Flood.

According to "The Delaware Water Supply News", March 1, 1940:

> "The maximum flood peak of record in the Rondout is that of August, 1928, reaching 26,715 cubic feet per second, with an indicated occurrence of once in 55 years; 14,000 cubic feet per second and over had been observed to occur three times in 25 years".

If inflow to the reservoir is assumed as 2 cfs per square mile of drainage area, drawdown from spillway crest to elevation 720 would take place in approximately 69 days.

HYDROLOGIC AND HYDRAULIC CALCULATIONS

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 1 OF _____
DATE 7/6/78
COMP. BY FEF
CHECKED BY DRC

NAME OF CLIENT _____
PROJECT Meriman Dam

STAGE - DISCHARGE RELATIONSHIP

$Q = CLH^{3/2}$ (Spillway Crest). $C = 3.4$, $L = 600'$ } $CL = 2040$

ELEVATION	HEAD	$H^{3/2}$	DISCHARGE
840.0	0	0	0
840.5	0.5	0.35	714
841.0	1.0	1.00	2040
842.0	2.0	2.83	5773
843.0	3.0	5.20	10608
844.0	4.0	8.00	16320
845.0	5.0	11.18	22807
848.0	8.0	22.63	46165
850.0	10.0	31.62	64505
855.0	15.0	58.09	117109
860.0	20.0	89.44	180311

→ $C = 3.2$, $L = 630$, $CL = 2016$

DETERMINE DISCHARGE THROUGH OUTLET WORKS.

MAXIMUM DISCHARGE @ EL. 840 = 890 MILLION GALLONS/DAY
= 1377 cfs.
DISCHARGE @ EL. 720 (OVERFLOW WEIR) = 700 MILLION GALLONS/DAY
= 1083 cfs.

STRAIGHT LINE RELATIONSHIP FROM EL. 840 TO EL. 720

→ THIS INFORMATION PROVIDED BY Mr. Kevin Cloonan
GRAHAMSVILLE, N.Y., 12740
POB 94

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 2 OF _____

DATE 7/25/78

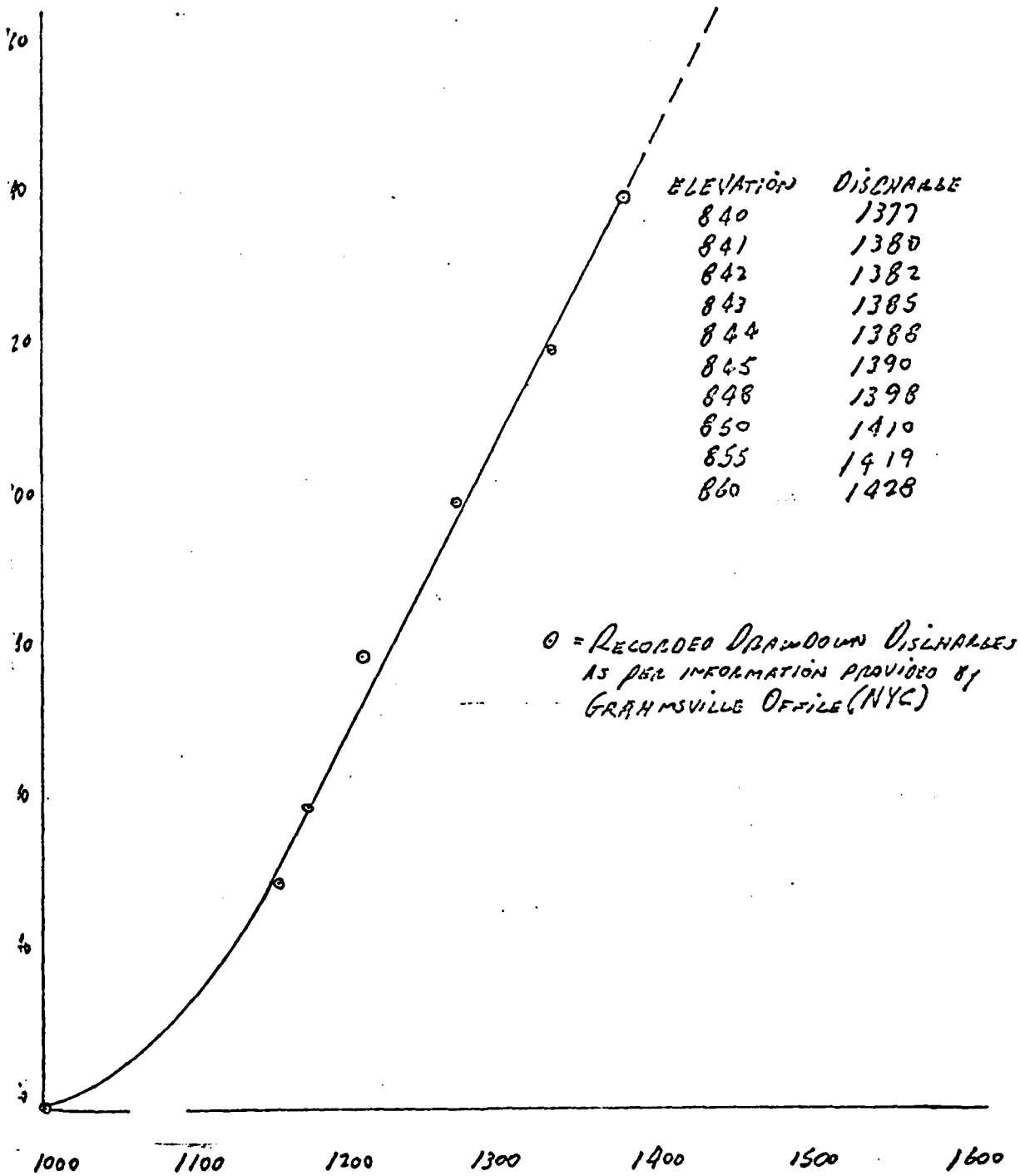
NAME OF CLIENT _____

COMP. BY FEP

PROJECT MERRIMAN DAM

CHECKED BY DBC

DISCHARGE THROUGH OUTLET WORKS



ELEVATION	DISCHARGE
840	1377
841	1380
842	1382
843	1385
844	1388
845	1390
846	1398
850	1410
855	1419
860	1428

⊙ = RECORDED DRAWDOWN DISCHARGES AS PER INFORMATION PROVIDED BY GRAHAMSVILLE OFFICE (NYC)

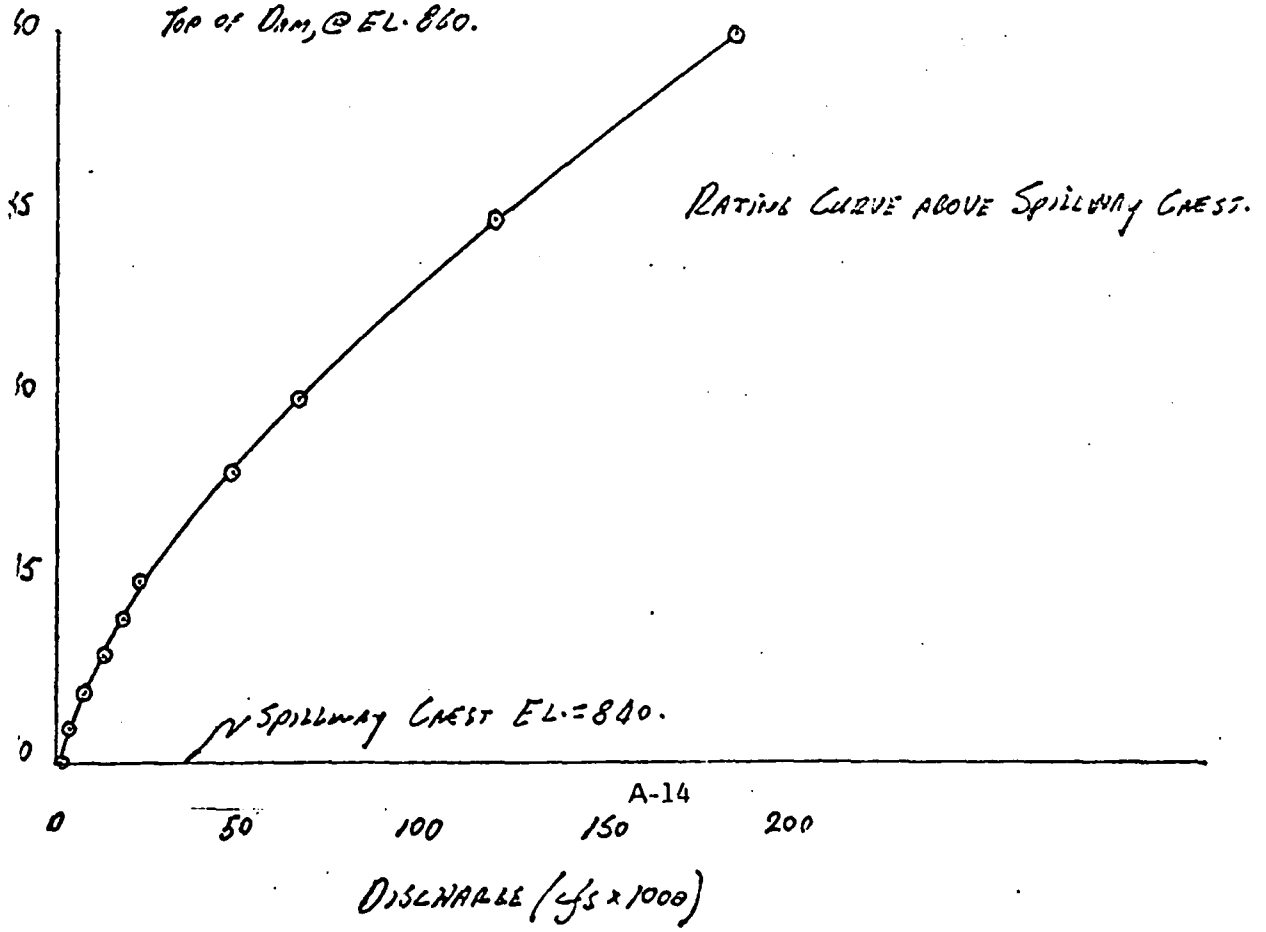
JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 3 OF _____
DATE 7/25/78
COMP. BY EEF
CHECKED BY DBC

NAME OF CLIENT _____
PROJECT Meriaman Dam

TOTAL DISCHARGE DETERMINATION

ELEVATION	OUTLET WORKS	SPILLWAY	TOTAL DISCHARGE
840	1377	0	1377
841	1380	2040	3420
842	1382	5773	7155
843	1385	10608	11993
844	1388	16320	17708
845	1390	22807	24197
848	1398	46165	47563
850	1410	64505	65915
855	1419	117109	118528
860	1428	180311	181739



JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 4 OF _____
DATE 7/6/78
COMP. BY FEF.
CHECKED BY DJC

NAME OF CLIENT _____
PROJECT MERRIMAN DAM

STAGE - STORAGE RELATIONSHIP

AREA @ 840, SPILLWAY CREST = 3.25 SQ. MI. = 2080 ACRES.

AREA @ 860, TOP OF DAM = 3.57 SQ. MI = 2285 ACRES.

$(2285-2080)/20' = 205/20 = 10.25 \approx 10$ ACRES/FT.

AREA = $10\,DEPTH + 2080$

STORAGE = $\int 10D + 2080 = 5D^2 + 2080\,D$

ELEVATION	DEPTH	$5D^2$	$2080\,D$	STORAGE
840.0	0	0	0	0
840.5	0.5	1.25	1040.0	1041
841.0	1.0	5.0	2080.0	2085
842.0	2.0	20.0	4160.0	4180
843.0	3.0	45.0	6240.0	6285
844.0	4.0	80.0	8320.0	8400
845.0	5.0	125.0	10400.0	10525
846.0	8.0	320.0	16640.0	16960
850.0	10.0	500.0	20800.0	21300
855.0	15.0	1125.0	31200.0	32325
860.0	20.0	2000.0	41600.0	43600

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 5 OF _____

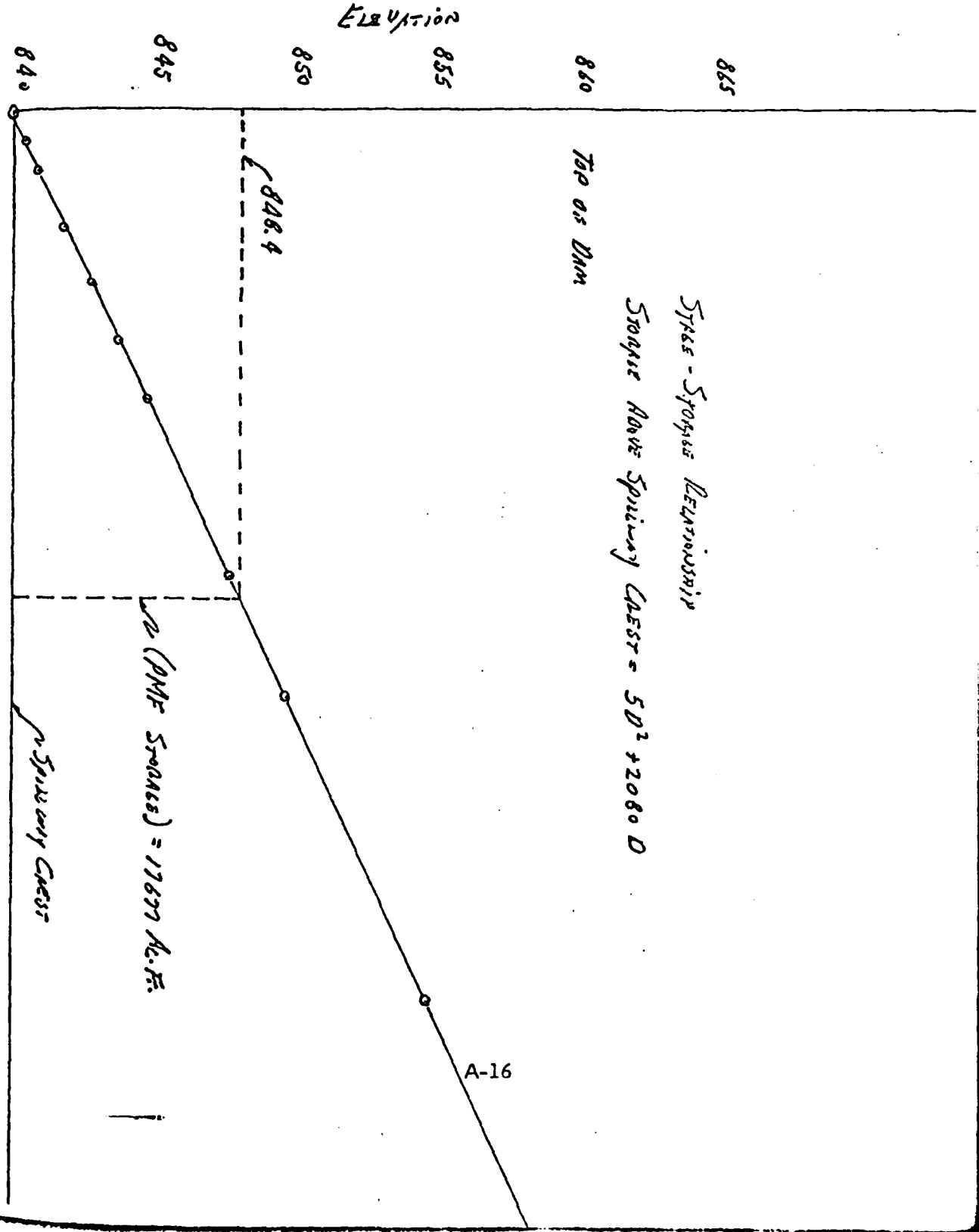
DATE 7/6/78

NAME OF CLIENT _____

COMP. BY F&F

PROJECT MERRIMAN DAM

CHECKED BY DBC



PROBABLE MAXIMUM FLOOD COMPUTATIONS

DRAINAGE AREA = 95 SQ. MI. PMP = 23"

PMP IN ZONE #1 - 75% OF 10 SQ. MILES, 6 HOUR VALUES

ISOHYETAL FIT REDUCTION FACTOR = 13.0%

R. ADJUSTED PMP = 23" x .13 = 2.99

REDUCTION = 23 - 2.99 = 20.01

20.01 x .75 = 15.01, USE 15 INCHES.

12 HR PMP = 20.01 x .84 = 16.8 INCHES.

$C_T = 2.0$
$C_p = .625$

$L = 11.80$ MILES
$L_{CA} = 2.0$ MILES

$T_p = C_T (L \times L_{CA})^{.3}$
$= 2.0 (11.8 \times 2.0)^{.3} = 5.16$

$T_R = T_P/5.5 = .94 \simeq 1.0$

Use 12 Hour Storm

Time (hours)	Rainfall Σ	Rainfall Incr
0-1	.20	.20
1-2	.40	.20
2-3	.80	.40
3-4	1.20	.40
4-5	2.25	1.05
5-6	3.45	1.20
6-7	10.80	7.35
7-8	13.20	2.40
8-9	14.70	1.50
9-10	16.20	1.50
10-11	16.50	.30
11-12	16.80	.30

Use minimum loss rate of .1 inch/hour

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 8 OF _____

NAME OF CLIENT _____ DATE 7/25/78

PROJECT MERRIMAN DAM COMP. BY FEF

CHECKED BY DBC

DRAWDOWN COMPUTATIONS — DRAWDOWN TIME = 69 DAYS

A_1 = AREA AT EL. 840 — S_1 = STORAGE AT EL. 840 = 154000 ACRE FT.

A_2 = AREA AT EL. 720 — S_2 = STORAGE AT EL. 720 = 7300 ACRE FT.

D = DEPTH.

$(A_1 + A_2)/2 \times D = S_1 - S_2$

$(2080 + A_2)/2 = (154000 - 7300)/D$

$2080 + A_2 = 2(154000 - 7300)/120$

$A_2 = 2445 - 2080$

$A_2 = 365$ ACRE

RISE = $(2080 - 365)/120' = 14.3$ ACRES/FT.

AREA = $14.30 + A_2$

AREA = $1430 + 365$

STORAGE = $7.15D^2 + 365D$

SEE SHEET #9

ELEVATION	DISCHARGE* (CFS)	FROM/TO (ELEVATION)	INCREMENT (STORAGE)	TIME (HRS).	TIME (DAYS)
840	1187	840/830	20095	205	8.54
820	1140	830/810	35900	381	15.88
800	1077	810/790	30180	339	14.13
780	1020	790/770	24460	290	12.08
760	987	770/760	10085	124	5.17
750	967	760/720	26040	326	13.58
720	0				

TOTAL TIME = 69 DAYS.

*ACCOUNTS FOR 2 CFS/SQ MI DRAINAGE AREA AS INFLOW

INFLOW = 2 × 95 = 190 CFS.

JUSTIN & COURTNEY, INC.
Division of O'Brien & Gere Engineers, Inc.
PHILADELPHIA, PA

SHEET NO. 9 OF _____

DATE 7/25/78

NAME OF CLIENT _____

COMP. BY FBF

PROJECT MERRIMAN DAM

CHECKED BY DBC

STORAGE = $7.15D^2 + 365D$

ELEVATION	DEPTH (FT.)	$7.15D^2$	$365D$	STORAGE (AC.-FT.)	INCREMENT STORAGE
840	120	102960	43800	146760	
					20095
830	110	86515	40150	126665	
					35900
810	90	57915	32850	90765	
					30180
790	70	35035	25550	60585	
					24460
770	50	17875	18250	36125	
					10085
760	40	11440	14600	26040	
					26040
720	0	0	0		

****
HEC-1 VERSION DATED JAN 1973
UPDATED AUG 74
CHANGE NO. 01
****

MERRIMAN DAM
PMF HYDROLOGY
NATIONAL DAM INSPECTION PROGRAM

JOB SPECIFICATION

NO	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
48	1	0	1	0	0	0	0	0	0

JOPER	NHT
3	0

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

SUB-AREA RUNOFF COMPUTATION

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

DC

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	1	95.00	0.00	0.00	0.00	0.000	0	0	0

PRECIP DATA

NP	STORM	DAJ	DAK
12	0.00	0.00	0.00

PRECIP PATTERN

.20	.20	.40	.40	1.05	1.20	7.35	2.40	1.50	1.50
.30	.30								

LOSS DATA

STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0.00	0.00	1.00	0.00	0.00	1.00	0.00	.10	0.00	0.00

UNIT HYDROGRAPH DATA
TP= 5.16 CP= .63 NTA= 0

RECESSION DATA
STRTQ= 0.00 QRCSN= 0.00 RTIOR= 1.00
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 6.06 AND R= 4.61 INTERVALS

UNIT HYDROGRAPH 28 END-OF-PERIOD ORDINATES, LAG= 5.14 HOURS, CP= .63 VOL= 1.00

568.	2062.	4050.	5954.	7210.	7474.	6635.	5346.	4301.	3459.
2783.	2234.	1801.	1448.	1165.	937.	754.	606.	488.	392.
316.	254.	204.	164.	132.	106.	86.	69.		

END-OF-PERIOD FLOW

	TIME		RAIN	EXCS	COMP Q
1	1	0	.20	.10	57.
1	2	0	.20	.10	263.
1	3	0	.40	.30	780.
1	4	0	.40	.30	1788.
1	5	0	1.05	.95	3635.

1	14	0	0.00	0.00	56427.
1	15	0	0.00	0.00	73784.
1	16	0	0.00	0.00	61056.
1	17	0	0.00	0.00	49585.
1	18	0	0.00	0.00	40013.
1	19	0	0.00	0.00	32188.
1	20	0	0.00	0.00	25892.
1	21	0	0.00	0.00	20827.
1	22	0	0.00	0.00	16753.
1	23	0	0.00	0.00	13476.
2	0	0	0.00	0.00	10840.
2	1	0	0.00	0.00	8720.
2	2	0	0.00	0.00	7014.
2	3	0	0.00	0.00	5642.
2	4	0	0.00	0.00	4538.
2	5	0	0.00	0.00	3645.
2	6	0	0.00	0.00	2927.
2	7	0	0.00	0.00	2338.
2	8	0	0.00	0.00	1864.
2	9	0	0.00	0.00	1447.
2	10	0	0.00	0.00	1103.
2	11	0	0.00	0.00	486.
2	12	0	0.00	0.00	264.
2	13	0	0.00	0.00	135.
2	14	0	0.00	0.00	31.
2	15	0	0.00	0.00	14.
2	16	0	0.00	0.00	0.
2	17	0	0.00	0.00	0.
2	18	0	0.00	0.00	0.
2	19	0	0.00	0.00	0.
2	20	0	0.00	0.00	0.
2	21	0	0.00	0.00	0.
2	22	0	0.00	0.00	0.
2	23	0	0.00	0.00	0.
3	0	0	0.00	0.00	0.
		SUM	16.80	15.60	951479.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	100494.	87684.	38931.	19822.	951478.
INCHES		8.59	15.25	15.53	15.51
AC-FT		43502.	77298.	78675.	78675.

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

HYDROGRAPH ROUTING

ISTAQ	ICOMP	ICON	ITAPE	JPLT	JPRT	INAME
2	1	0	0	0	0	0

ROUTING DATA

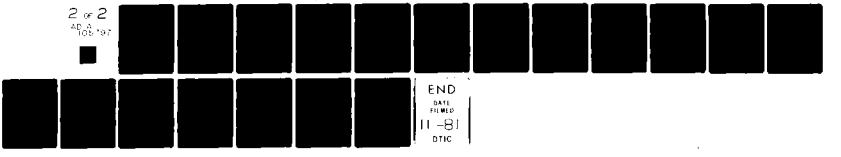
QLOSS	CLOSS	AVG	IRES	ISAME
0.0	0.000	0.00	1	0

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

0 2085. 4180. 6285. 8400. 10525. 16960. 21300. 32325. 43600.

AD-A105 797 BAKER (MICHAEL) JR INC BEAVER PA F/G 13/13
NATIONAL DAM SAFETY PROGRAM. HONK FALLS DAM (INVENTORY NUMBER N--ETC(U)
AUG 81 G KESTER DACW51-81-C-0010
UNCLASSIFIED NL

2 OF 2
AD A 105 797



END
DATE FILMED
11-81
DTIC

1	6	0	728.	5368.	1194.
1	7	0	1511.	11315.	2479.
1	8	0	3153.	23768.	5324.
1	9	0	5993.	42689.	11322.
1	10	0	9930.	64481.	22379.
1	11	0	14378.	84104.	38186.
1	12	0	18540.	96580.	54244.
1	13	0	21670.	98841.	67683.
1	14	0	23336.	91808.	75630.
1	15	0	23645.	80106.	77104.
1	16	0	22976.	67420.	73914.
1	17	0	21693.	55321.	67789.
1	18	0	20083.	44703.	60768.
1	19	0	18347.	36100.	53430.
1	20	0	16624.	29040.	46345.
1	21	0	14973.	23359.	40347.
1	22	0	13424.	18790.	34722.
1	23	0	12015.	15115.	29606.
2	0	0	10761.	12159.	25053.
2	1	0	9645.	9780.	21510.
2	2	0	8644.	7867.	18452.
2	3	0	7746.	6328.	15940.
2	4	0	6939.	5090.	13760.
2	5	0	6219.	4092.	11842.
2	6	0	5573.	3286.	10158.
2	7	0	4990.	2632.	9018.
2	8	0	4468.	2101.	7918.
2	9	0	4000.	1655.	6934.
2	10	0	3572.	1275.	6071.
2	11	0	3166.	794.	5347.
2	12	0	2783.	375.	4664.
2	13	0	2433.	193.	4052.
2	14	0	2134.	83.	3507.
2	15	0	1864.	22.	3058.
2	16	0	1628.	7.	2671.
2	17	0	1421.	0.	2332.
2	18	0	1241.	0.	2036.
2	19	0	1083.	0.	1777.
2	20	0	946.	0.	1552.
2	21	0	826.	0.	1355.
2	22	0	721.	0.	1183.
2	23	0	629.	0.	1032.
3	0	0	550.	0.	901.
		SUM			945698.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	77104.	70481.	36967.	19702.	945698.
INCHES		6.90	14.48	15.43	15.41
AC-FT		34967.	73362.	78197.	78197.

RUNOFF SUMMARY, AVERAGE FLOW

		PEAK	6-HOUR	24-HOUR	72-HOUR	AREA
HYDROGRAPH AT	1	100434.	87684.	38931.	19822.	95.00
ROUTED TO	2	77104.	70481.	36967.	19702.	95.00

APPENDIX G

STRUCTURAL STABILITY

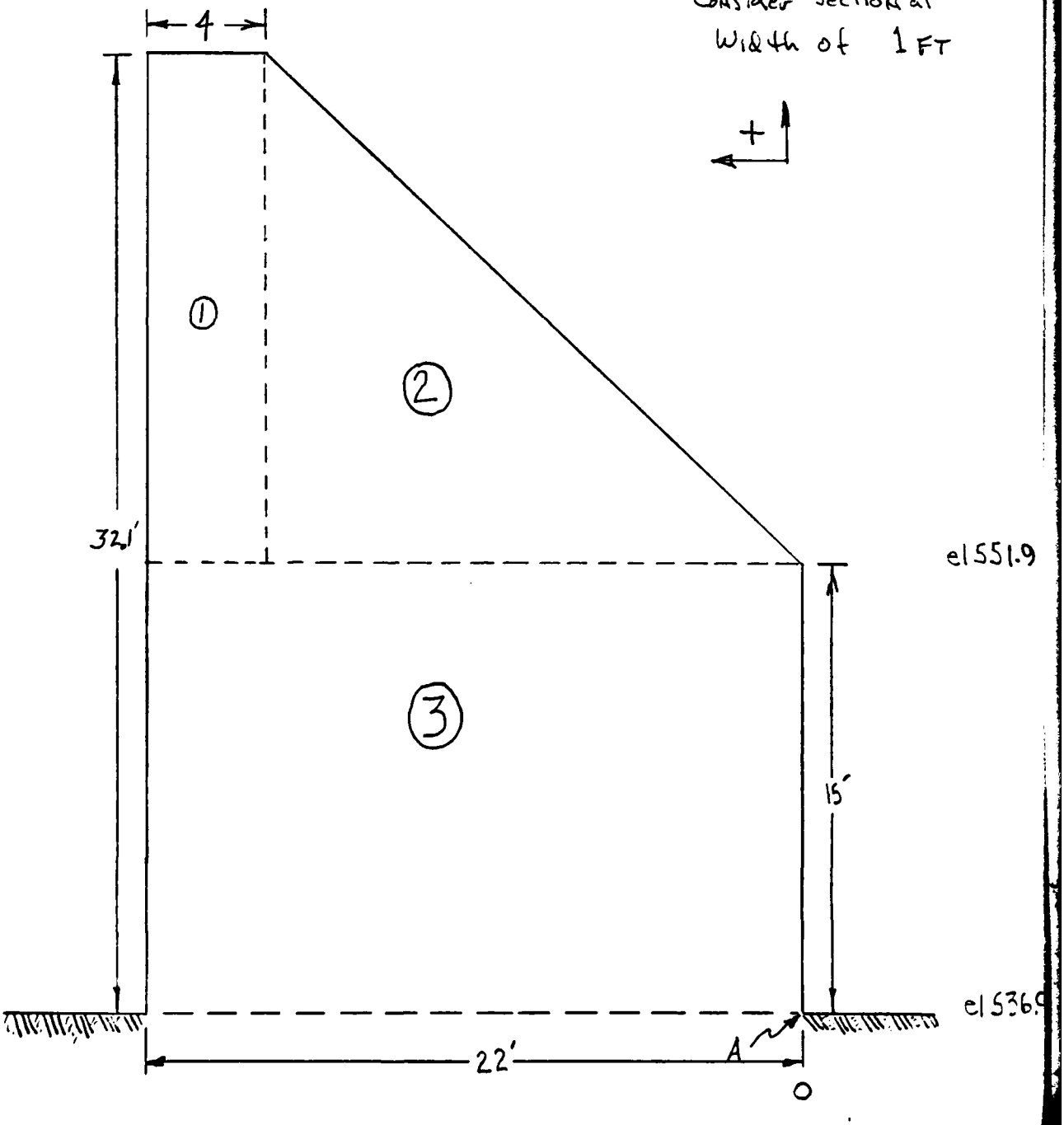
MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hoyt Falls Dam — S.O. No. 13838-00-ALA
Typical Cross Section — Sheet No. 1 of 16
Station 2+14 — Drawing No.
Computed by DWM. — Checked by JH — Date May 6, 1981

Stability Analysis

$\gamma_c = 150$ PCF
Consider Sectional Width of 1 FT



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Honk Falls Dam — S.O. No. 13839-0-ARA
Stability Analysis — Sheet No. 2 of 16
Drawing No. —
Computed by DWM — Checked by TSH — Date May 6, 1981

Section	W (lb)	Arm (ft)	Moment (k-ft)
①	4(150)(17.1)	20	205.2
②	½(18)(17.1)150	⅔(18)	277.02
③	22(15)150	11	544.5
	ΣW = 82845 lb		ΣM = 1026.72 k-ft

Resultant = 82.845 k

$\bar{X} = \frac{1026.72 \text{ k-ft}}{82.845 \text{ k}} = 12.39'$ O.K., within Middle Third $7.33' < \bar{X} \leq 14.67'$

Case I — Normal operating conditions with reservoir level at the spillway crest, full uplift, and no tailwater

Case II — Same as Case I with the addition of ice loading of 5,000 lb/ft of wall

Case III — Reservoir level during ½PMF, Full uplift as in Case I, with tailwater ≅ 60% full value for overflow section or full value for non-overflow section

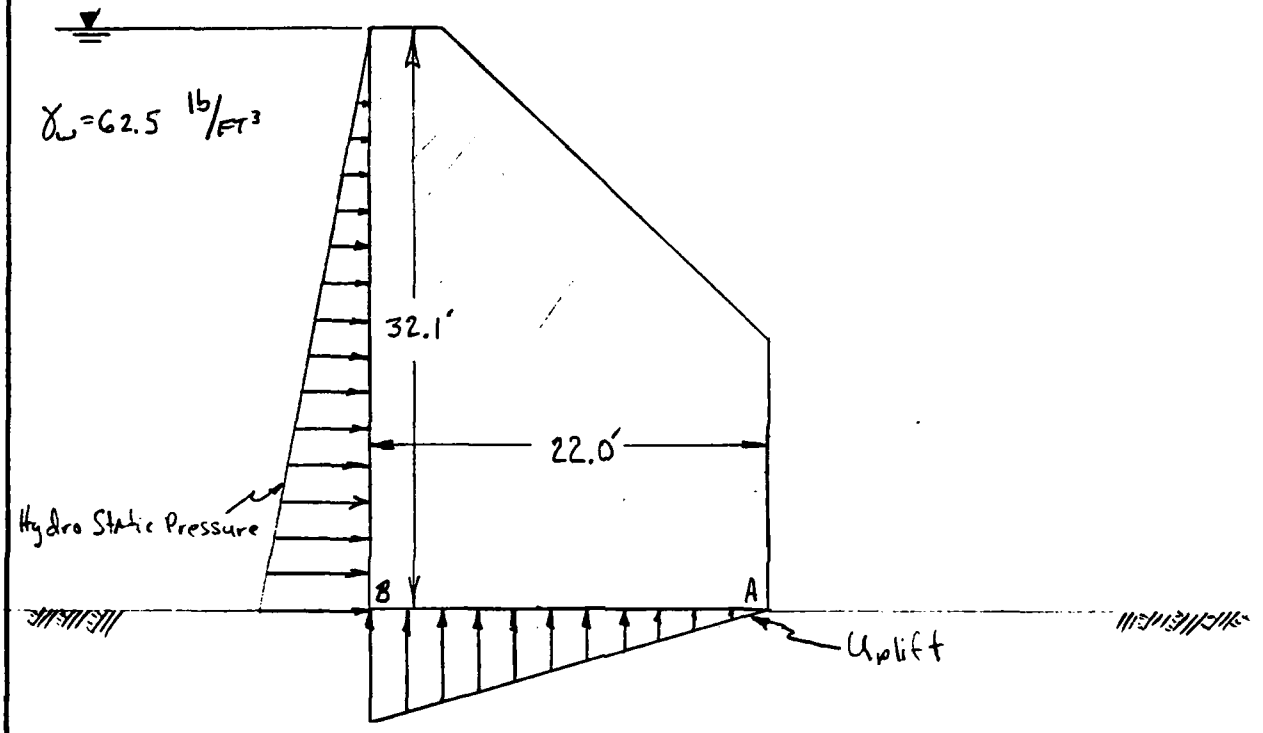
Case IV — Reservoir level during PMF, Full uplift as in Case I, with tailwater ≅ 60% full value for overflow section or full value for non-overflow section

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hawk Falls Dam — S.O. No. 13998-01-AKA
Stability Analysis — Sheet No. 3 of 16
Drawing No.
Computed by DWM — Checked by TSH — Date May 6, 1991

Case I (over flow Section)



Pressure at Base of Dam = 32.1 (62.5) = 2,006.25 lb/ft² = 2.006 K/ft²

Resultant Hydro Static Force = ½ (32.1) (2.006) (1) = 32.2 Kips

Moement Arm (Resultant location) = ⅓ (32.1) = 10.7 ft above base of Dam

Resultant Uplift Force = ½ (2.006) (22) = 22.07 K

Moment Arm (resultant location) = ⅔ (22) = 14.67 from Right side of Dam.

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hank Falls Dam — S.O. No. 13383-03-AKA
Stability Analysis — Sheet No. 4 of 16
Drawing No.
Computed by DWM — Checked by TSU — Date May 6, 1991

Now Consider Bearing Pressure, i.e. Reaction of Foundation

$BP_B = \frac{\Sigma W}{T}\left(1 - \frac{e(6)}{T}\right)$

$BP_A = \frac{\Sigma W}{T}\left(1 + \frac{e(6)}{T}\right)$

$\Sigma W = W_c = 82.845$ kips (Weight)

$T = 22'$

ΣH 32.2 kips (Horizontal Load)

MA_H 10.7 Ft

$\Sigma M_C \Rightarrow r$

$r = \frac{\Sigma H (MA_H)}{\Sigma W} = \frac{32.2\,(10.7)}{82.845}$

$r = 4.159'$ $\quad (12.39 - 4.159 = 8.231')$ $\quad e = \frac{22}{2} - 8.231 = 2.769$

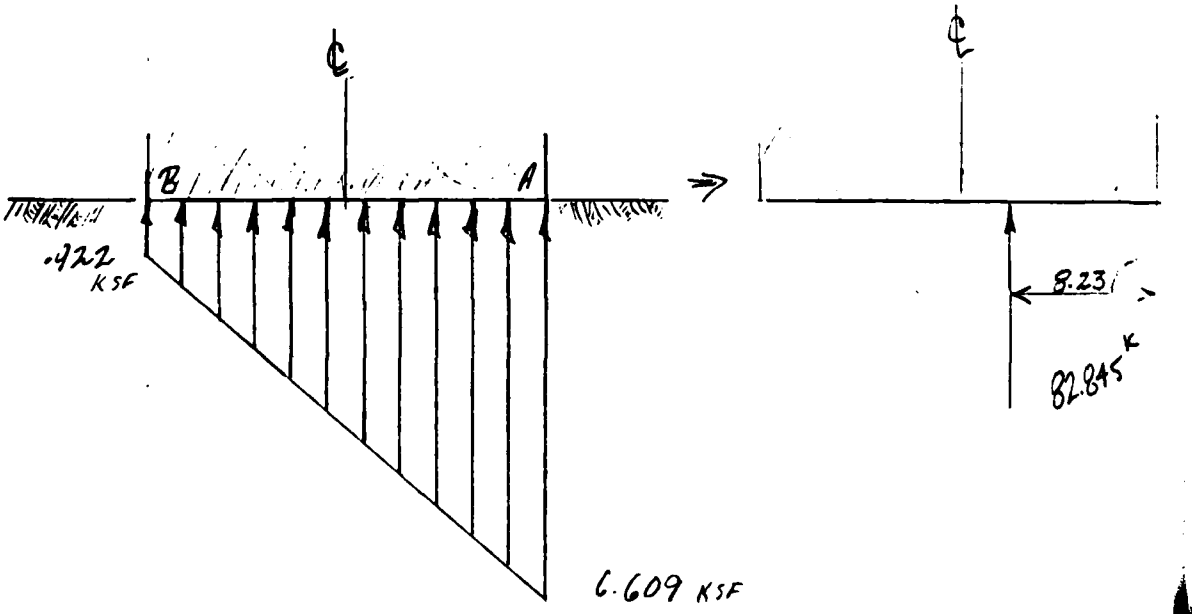
$BP_B = \frac{82.845}{22}\left(1 - \frac{2.769(6)}{22}\right)$ $\quad BP_B = .922$ KSF

$BP_A = \frac{82.845}{22}\left(1 + \frac{2.769(6)}{22}\right)$ $\quad BP_A = 6.609$ KSF

This initial Calculation does Not Consider Uplift

Uplift pressure At B $\rightarrow UP_B = 2.006$ KSF

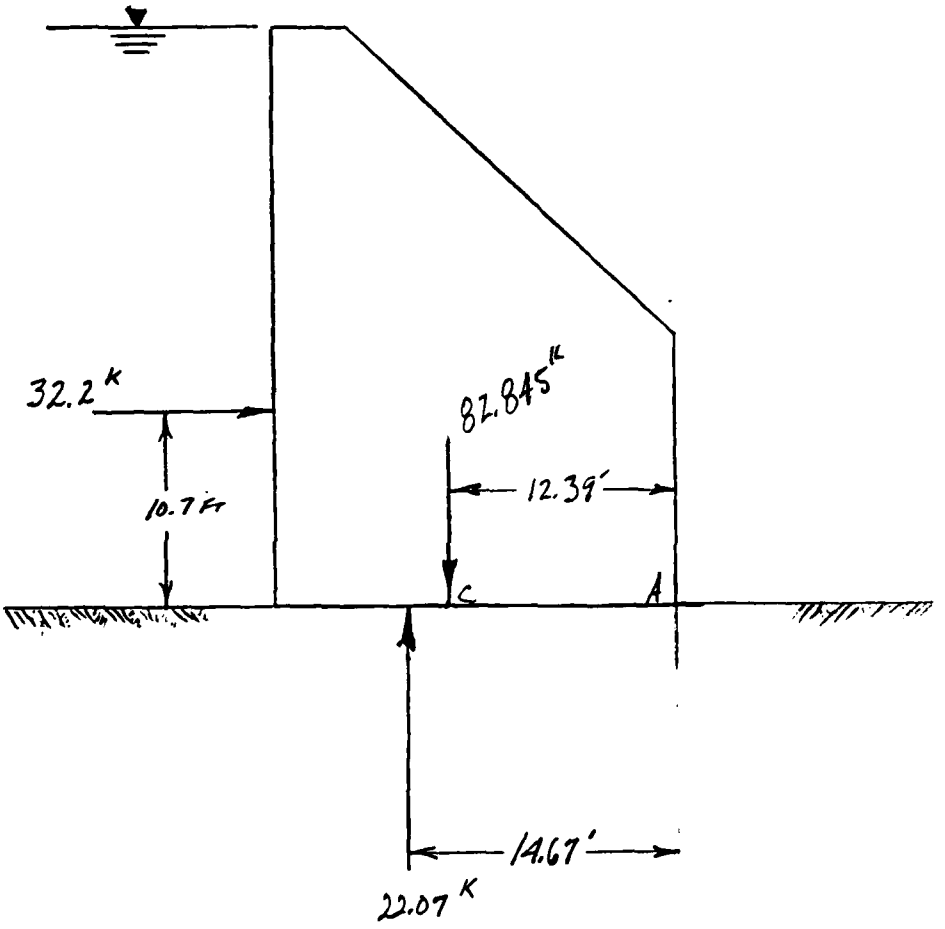
$UP_A = 0$



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hank Falls Dam — S.O. No. 13333-00-ARA
Stability Analysis — Sheet No. 5 of 16
Drawing No.
Computed by DWM — Checked by — Date May 6, 1991



Overturning

$$F.S._{overturning} = \frac{\Sigma M_A \text{ Resisting Overturning}}{\Sigma M_A \text{ Causing Overturning}} = \frac{82.845(12.39)}{32.2(10.7) + 22.07(14.67)}$$

$F.S._{OVERTURNING} = 1.536$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hoyle Falls Dam — S.O. No. 13333-03-AA
Stability Analysis — Sheet No. 6 of 16
Drawing No.
Computed by DWM — Checked by TSH — Date May 6, 1991

Sliding

General Eq. Forces Resisting Sliding

$$R = (\Sigma F_{vertick}) \tan \phi + s A$$

ϕ = Angle of internal friction of Foundation Material

ΣF_v = Summation of Vertiele Forces

s = Unit Shear Strength at zero Normal loading along potential failure plane

A = Area of potential failure plane developing "s"

$\phi = 35^\circ$ $\quad s = 2$ KSF $\quad$ TAN $35^\circ = .700$

$\Sigma F_v = 82.845 - 22.07 = 60.775$ kips

$A = 22(1) = 22$ FT^2

$R = (60.775)(.700) + 2(22)$

$= 86.543$ kips.

$H = 32.2$ $\quad F.S. = \frac{R}{H} = \frac{86.543}{32.2}$

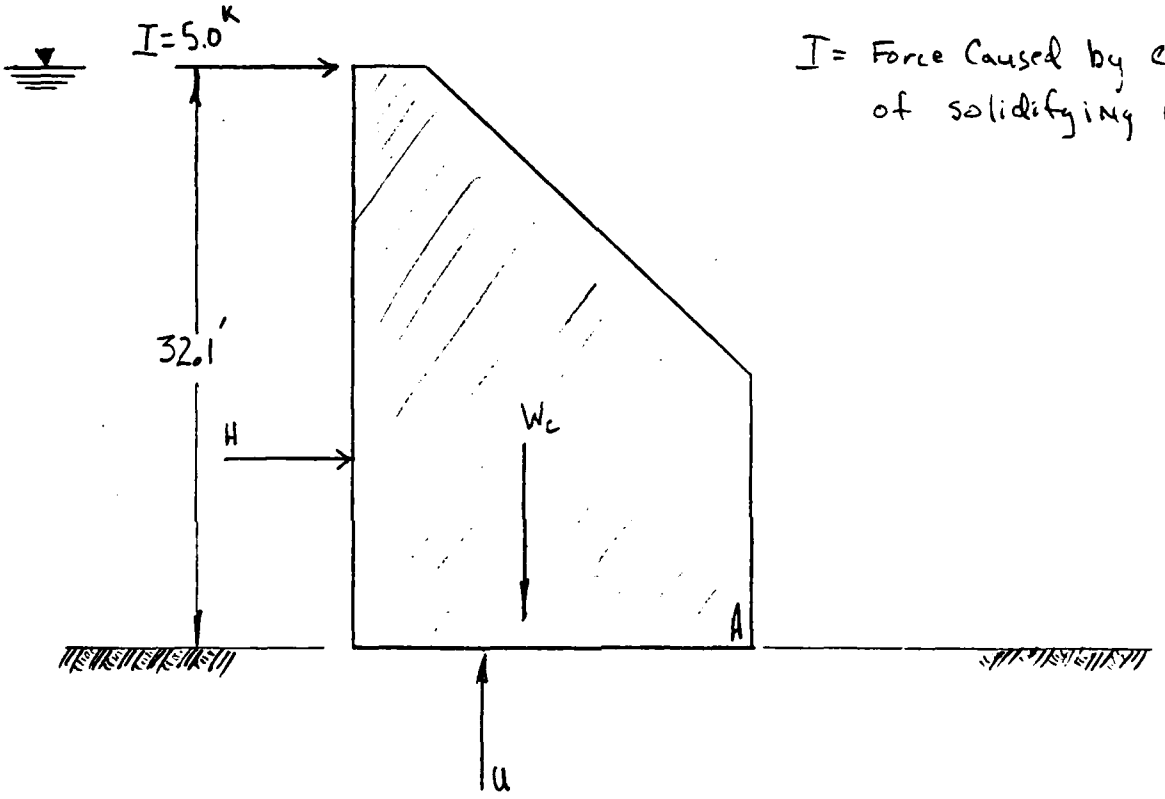
$$F.S._{sliding} = 2.688$$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hank Falls Dam — S.O. No. 13333-00-AAA
Stability Analysis — Sheet No. 7 of 16
Drawing No.
Computed by DWM — Checked by TH — Date May 6, 1991

Case II (Overflow Section)



I = Force Caused by expansion of solidifying ice

Overturning

$$F.S. = \frac{82.845(12.39)}{32.2(10.7) + 22.07(14.67) + 5(32.1)}$$

F.S. = 1.238 overturning

Sliding $H = 32.2 + 5 = 37.2$

$$F.S. = \frac{86.543}{37.2}$$

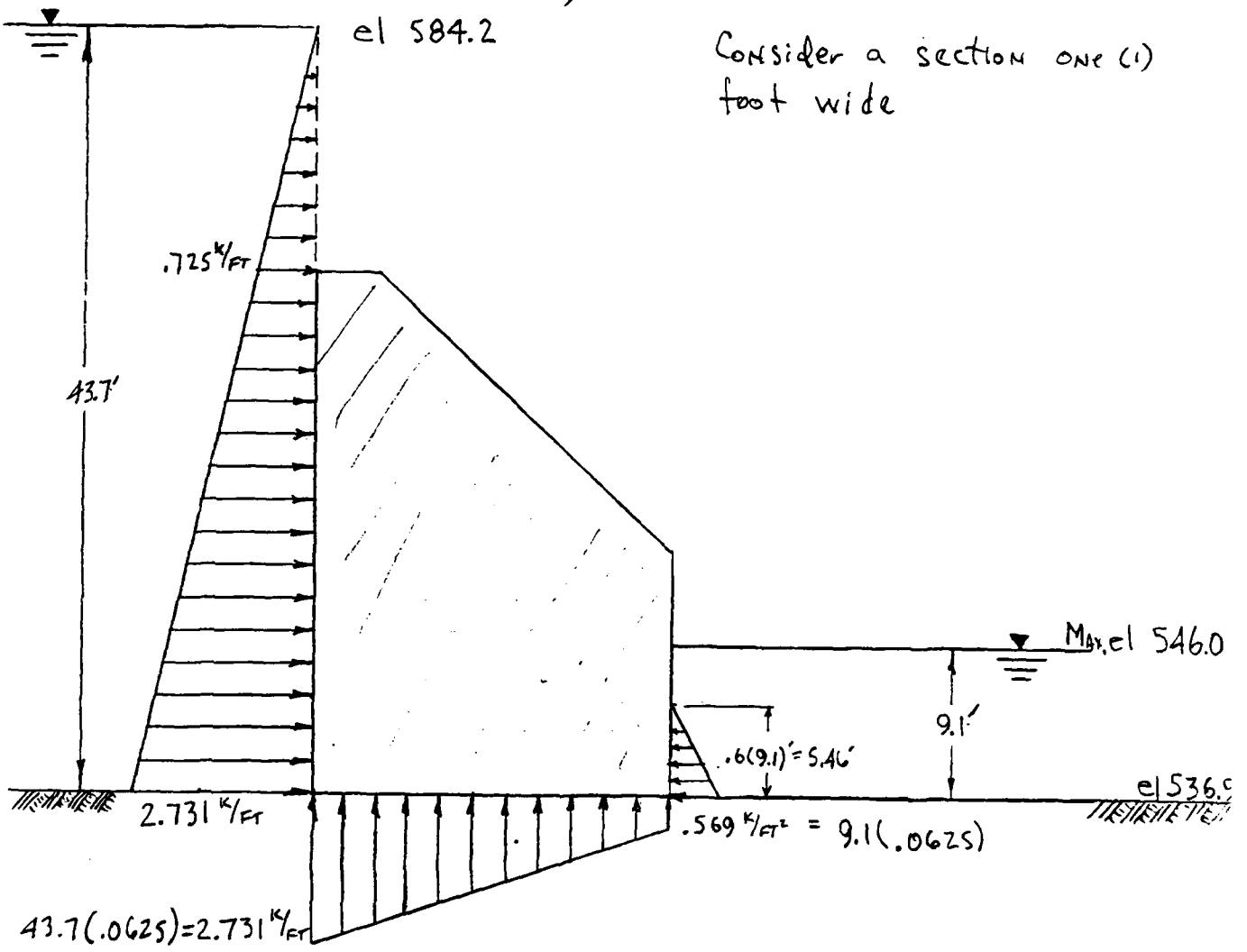
F.S. = 2.326 overturning

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject HONK FALLS DAM — S.O. No. 13833-33-AKA
Stability Analysis — Sheet No. 8 of 16
Drawing No.
Computed by DWM — Checked by BH — Date May 6, 1981

Case III (overflow Section)



Consider a section one (1) foot wide

Resultant Hydro Static Force on Face $= \left[\frac{1}{2}(2.731 - .725)32.1 + .725(32.1)\right](1)$

$= 55.469$ Kips

Resultant location $= \dfrac{\left[\frac{1}{2}(2.731-.725)32.1\right]\frac{32.1}{3} + \left[.725(32.1)\right]\frac{32.1}{2}}{55.469}$

$= 12.945'$ above base level

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hook Falls Dam — S.O. No. 12333-05-AHA
Stability Analysis — Sheet No. 9 of 16
Drawing No.
Computed by DWH — Checked by TEL — Date 1/4, 7, 1981

Resultant Hydro Static Force on Down Stream Face

$= \frac{1}{2}[.6(9.1)]^2 62.5 = 931.616 = .932^{kips}$

Resultant Location $= \frac{.6(9.1)}{3} = 1.82$ FT above base level

Resultant Uplift Force $= \left(\frac{.569 + 2.731}{2}\right) 22 = 35.3^{kips}$

Resultant location $= \frac{\left[\frac{1}{2}(2.731 - .569)22\right]^{22/3} + \left[.569(22)\right]^{22/2}}{36.3}$

$= 8.598$ FT from upstream face

$22 - 8.598 = 13.402$ FT from Down stream face

Bearing Pressure (excluding Uplift)

$\Sigma M_C \Rightarrow r \qquad r = \frac{(55.467)(12.745) - (.932)(1.82)}{82.845}$

$r = 8.647'$

$12.39 - 8.647 = 3.743'$ From Down stream face

$e = 22/2 - 3.743 = 7.257$ Not within middle Third

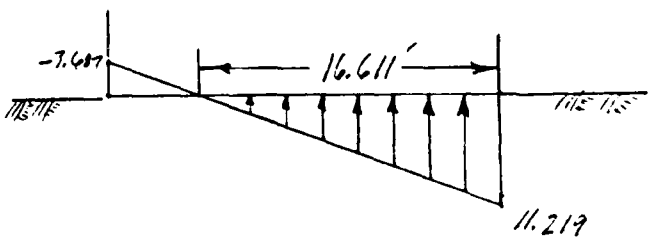
$7.257 > 3.667$

$BP_D = \frac{82.845}{22}\left(1 - \frac{7.257(6)}{22}\right)$

$BP_D = -3.687^{K}$

$BP_A = \frac{82.845}{22}\left(1 + \frac{7.257(6)}{22}\right)$

$= 11.219^{K}$



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

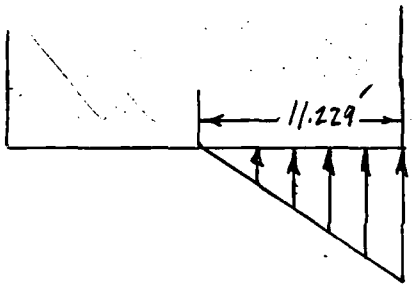
Subject Hank Falls Dam — S.O. No. 13993-05-AAA
Stability Analysis — Sheet No. 10 of 16
Drawing No.
Computed by DWM — Checked by EL — Date May 7, 1931

Negative Bearing Pressure (Tension) Does not exist

Revise Bearing Pressure Diagram

$\frac{1}{2}[3(3.743)]BP_A = 82.845$

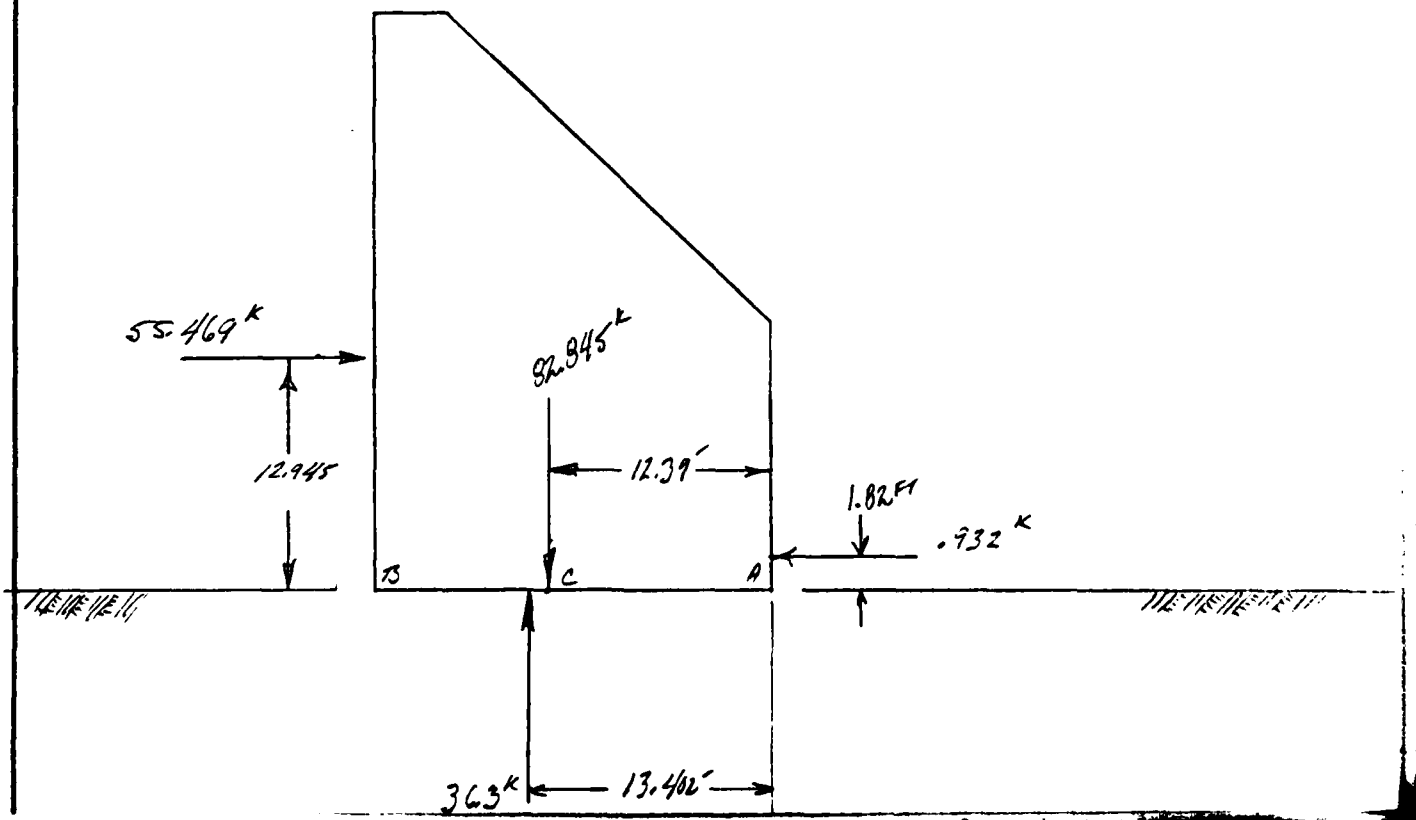
$BP_A = 14.756 \ ^K/_{FT^2}$



(Excluding Uplift)

14.756 KSF

Resultant Loadings



MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hull Falls Dam — S.O. No. 13383-03-ARH
Stability Analysis — Sheet No. 11 of 16
Drawing No.
Computed by DWM — Checked by ~~A.4~~ — Date Mar. 7, 1991

Overturning $\quad \Sigma M_A \Rightarrow$

$$F.S._{overturning} = \frac{(82.845)(12.39) + (1682)(.932)}{(55.469)(12.945) + (363)(13.432)}$$

$$\boxed{F.S._{overturning} = 0.854}$$

Sliding

$$R = (F_v)\ \text{TAN}\ \phi + SA \qquad R = (46.545).7 + 2(22)$$

$$H = 55.469 - .932 = 54.537^{K}$$

$$F.S. = R/H = \frac{76.582}{54.537}$$

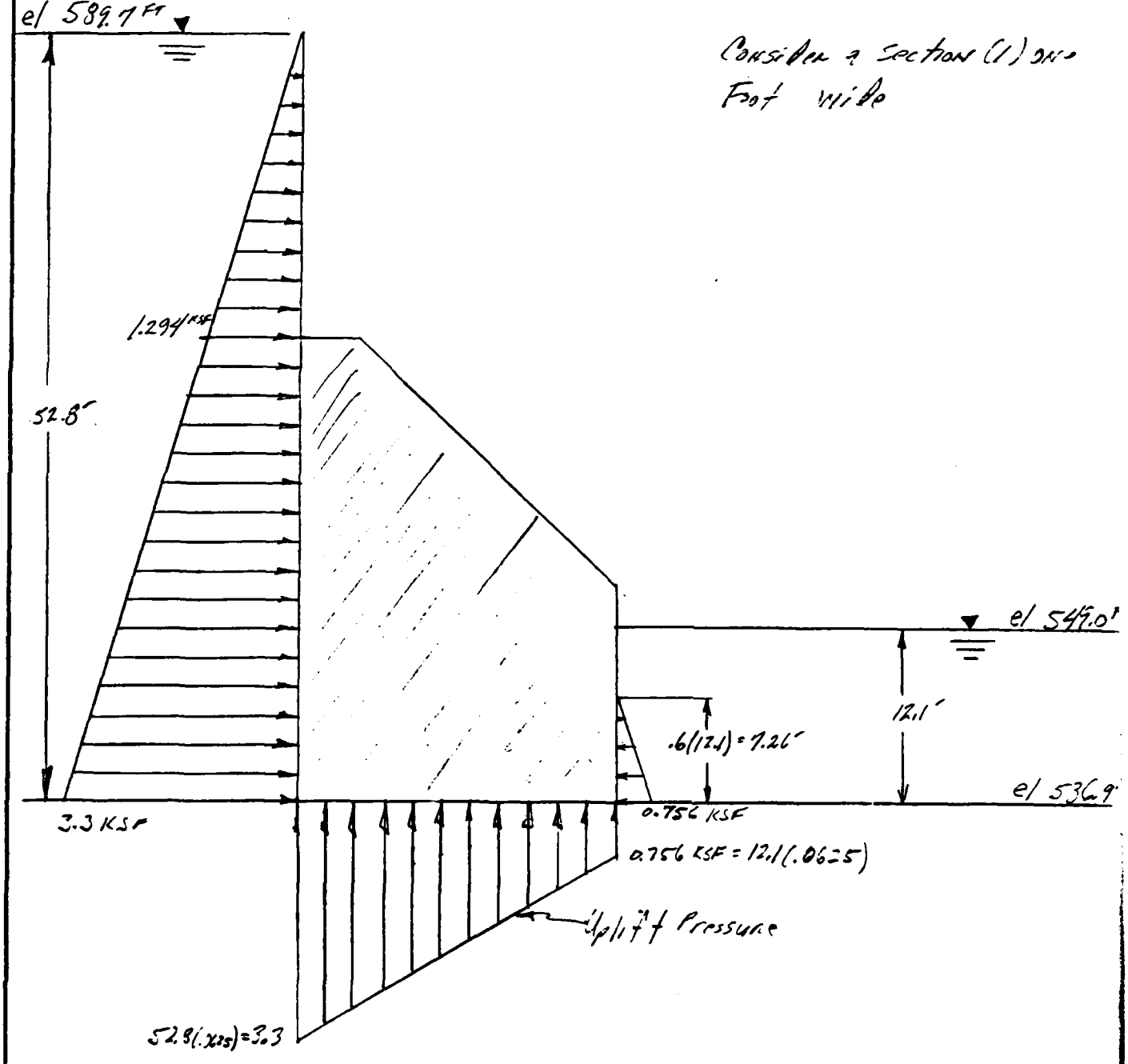
$$\boxed{F.S._{sliding} = 1.404}$$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Howe Falls Dam S.O. No. 13883-03-AMA
Stability Analysis Sheet No. 12 of 16
Drawing No.
Computed by GWM Checked by EH Date May 7, 1981

Case II (Over Flow Section)



Consider a Section (1) One Foot Wide

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Honk Falls Dam — S.O. No. 13333-00-ARA
Stability Analysis — Sheet No. 13 of 16
Drawing No.
Computed by DWM. Checked by TEH Date May 7, 1981

Resultant of Hydrostatic Force on Upstream Face

$$\left(\frac{1.294 + 33}{2}\right) 32.1 = 73.730 \text{ kips}$$

Resultant location =

$$\frac{\left[(1.294)(32.1)\right]\frac{32.1}{2} + \left[\tfrac{1}{2}(33 - 1.294)\,32.1\right]\frac{32.1}{3}}{73.73}$$

$$= 13.715 \text{ FT above base level}$$

Resultant on Down stream Face

$$.6(12.1)(.0625) = 0.454 \text{ kips}$$

Resultant Location = $\frac{.6(12.1)}{3} = 2.42$ FT above base level

Resultant Uplift Force

$$\left(\frac{3.3 + .756}{2}\right) 22 = 44.616 \text{ kips}$$

Resultant location

$$\frac{\left[(.756)(22)\right]\frac{22}{2} + \left[\tfrac{1}{2}(3.3 - .756)(22)\right]22/3}{44.616}$$

$$= 8.7 \text{ FT From Upstream Face}$$

$22 - 8.7 = 13.3$ FT From Downstream Face

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hawk Falls Dam — S.O. No. 13889-00-ATA
Stability Analysis — Sheet No. 14 of 16
Drawing No.
Computed by DWM — Checked by [illegible] — Date May 7, 1991

Bearing Pressure (excluding Uplift)

$M_C = Pr$

$$r = \frac{73.73(13.715) - .454(2.42)}{82.845}$$

$r = 12.193$ FT

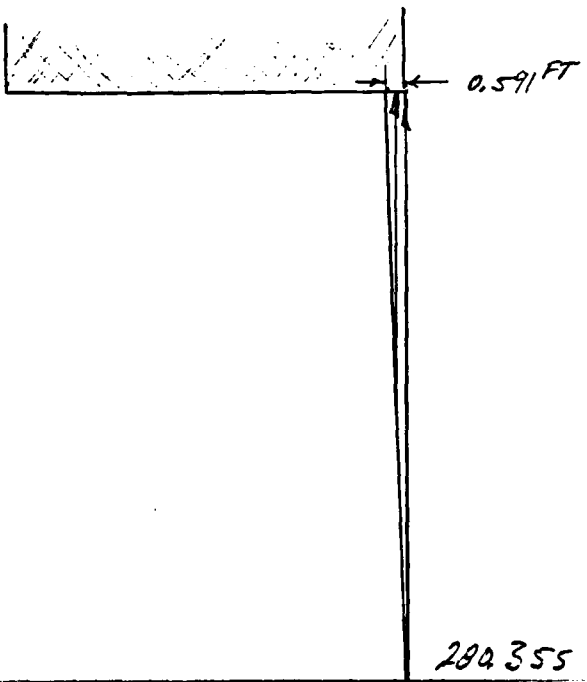
$12.39 - 12.193 = .197$ FT From down stream Face

$e = 22/2 - .197 = 10.803$

Bearing Pressure at toe

$\frac{1}{2}[3(.197)]\,BP_A = 82.845$

$BP_A = 280.355$ KSF



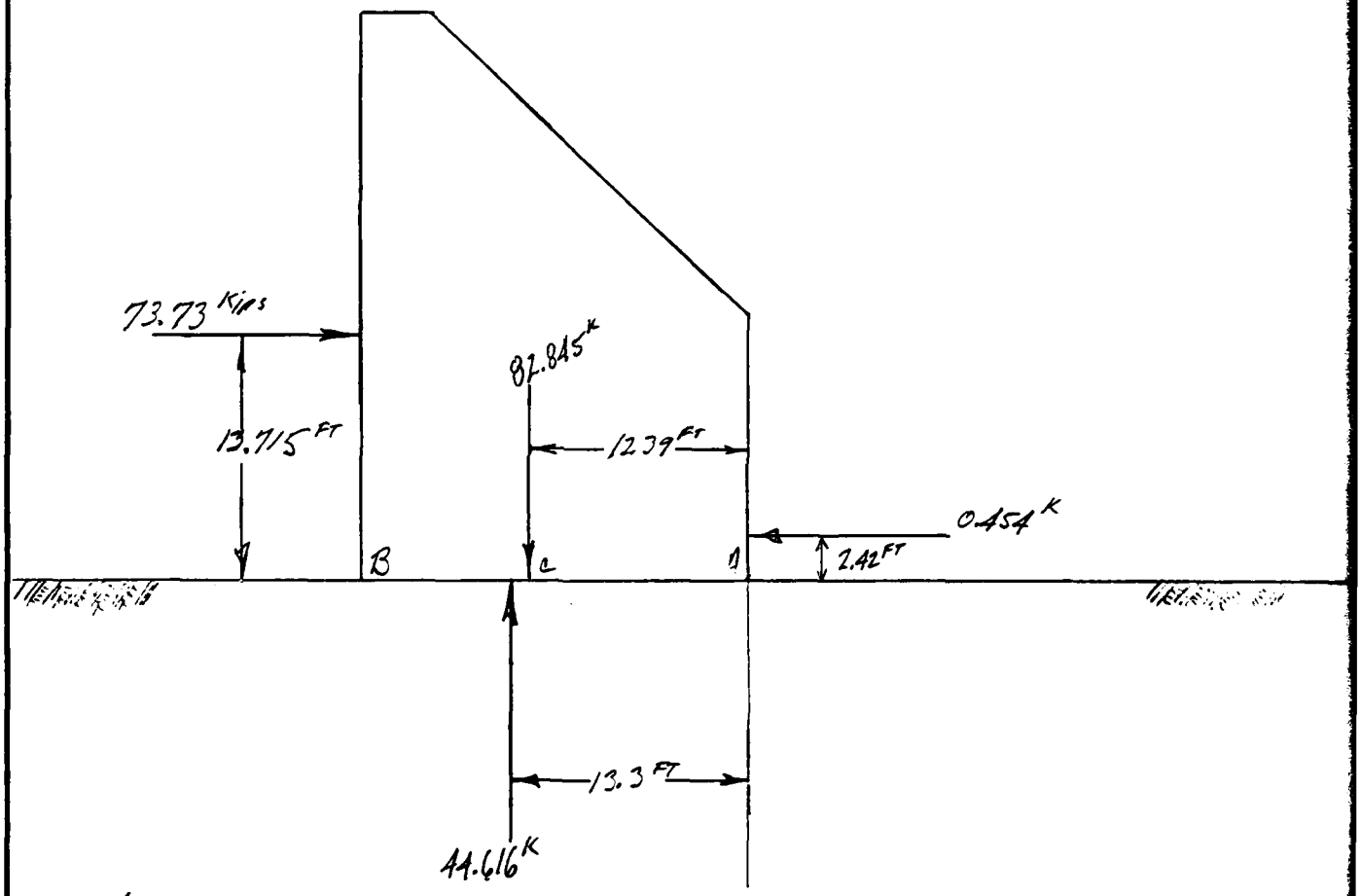
Excluding Uplift

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS

Box 280
Beaver, Pa. 15009

Subject Hulk Falls Dam — S.O. No. 13883---ARA
Stability Analysis — Sheet No. 15 of 16
Drawing No. ____
Computed by DWM — Checked by TS4 — Date May 7, 1981

Resultant Location



Overturning

$$\Sigma M_A \Rightarrow \frac{82.845(12.39) + 0.454(2.42)}{73.73(13.715) + (44.616)(13.3)} = F.S._{overturning} = 0.640$$

Sliding

$R = (38.229)(.7) + 2(22) = 70.76$

$H = 73.73 - .454 = 73.276$

$F.S. = R/H = 70.76/73.276$

$F.S._{sliding} = 0.966$

MICHAEL BAKER, JR., INC.
THE BAKER ENGINEERS
Box 280
Beaver, Pa. 15009

Subject Hulk Falls Dam S.O. No. 15333-00-ALA
Stability Analysis Sheet No. 16 of 16
Drawing No.
Computed by DCHM Checked by TSH Date May 7, 1991

Reaction Locations (including Uplift)

Case I $\frac{\Sigma M_A}{\Sigma V} = \frac{32.2(10.7) + 22.07(14.67) - 82.845(12.39)}{-82.845 + 22.07}$

$= 5.893$ FT FROM TOE UNDER DAM NOT WITHIN MIDDLE THIRD

Case II $\frac{\Sigma M_A}{\Sigma V} = \frac{32.2(10.7) + 22.07(14.67) + 5(32.1) - 82.845(12.39)}{-82.845 + 22.07}$

$= 3.252$ FT FROM TOE UNDER DAM NOT WITHIN MIDDLE THIRD

Case III $\frac{\Sigma M_A}{\Sigma V} = \frac{58.469(12.945) + 36.3(13.402) - 82.845(12.39) - 1.3(.752)}{36.3 - 82.945}$

$= -3.79$ FT FROM TOE NOT UNDER DAM SECTION

Case IV $\frac{73.73(13.715) + 44.66(13.3) - 82.945(12.39) - .454(2.42)}{44.66 - 82.845}$

$= -14.861$ FT FROM TOE NOT UNDER DAM SECTION