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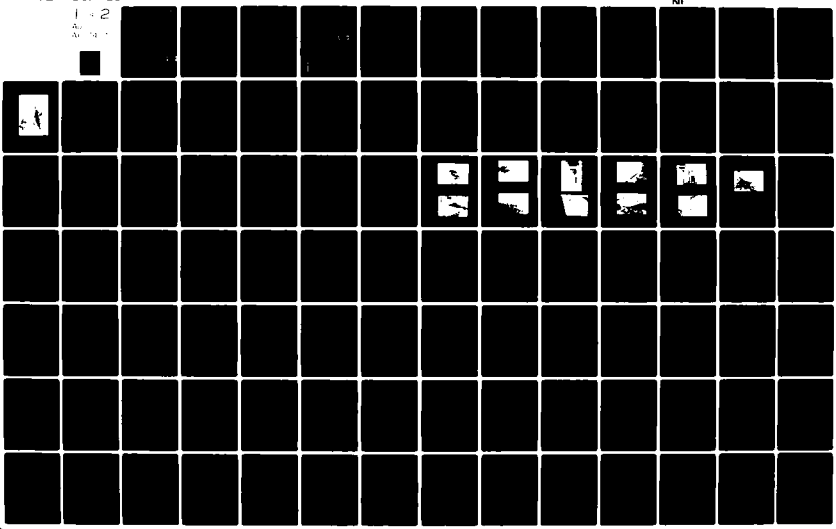

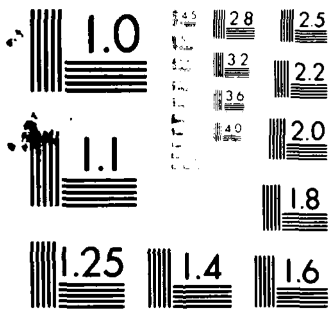

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LONG ISLAND BASIN

BYRAM LAKE RESERVOIR DAM

WESTCHESTER COUNTY, NEW YORK
INVENTORY NO. N.Y. 1175

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NEW YORK DISTRICT CORPS OF ENGINEERS

SEPTEMBER 1981

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Long Island 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.

The examination of documents and the visual inspection findings of Byram Lake Dam did not reveal conditions which constitute an immediate hazard to life and property. However, the dam has some deficiencies which require further investigation and remedial action.

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Using the Corps of Engineers screening criteria for initial review of the spillway adequacy, it has been determined that the dam would be overtopped by all floods exceeding 61 percent of the PMF. The maximum spillway discharge capacity is 18.4 percent of the peak PMF outflow. The spillway is therefore judged to be inadequate.

The structural stability analysis based on available information, assumed strength parameters and material properties and visual inspection indicates that the dam is inadequate in overturning and sliding for all loading conditions except for the normal loading.

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LONG ISLAND BASIN

BYRAM LAKE RESERVOIR DAM

WESTCHESTER COUNTY, NEW YORK
INVENTORY NO. N.Y. 1175

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NEW YORK DISTRICT CORPS OF ENGINEERS

SEPTEMBER 1981

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 investigations, 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 aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
BYRAM LAKE RESERVOIR DAM
I.D. NO. N.Y. 1175
D.E.C. NO. 345
LONG ISLAND BASIN
WESTCHESTER COUNTY, NEW YORK

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APPENDICES

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NAME OF DAM	Byram Lake Dam, N.Y. 1175
STATE LOCATED	New York
COUNTY LOCATED	Westchester
STREAM	Byram River
BASIN	Long Island
DATE OF INSPECTION	14 May 1981

ASSESSMENT

The examination of documents and the visual inspection findings of Byram Lake Dam did not reveal conditions which constitute an immediate hazard to life and property. However, the dam has some deficiencies which require further investigation and remedial action.

Using the Corps of Engineers screening criteria for initial review of the spillway adequacy, it has been determined that the dam would be overtopped by all floods exceeding 61 percent of the PMF. The maximum spillway discharge capacity is 18.4 percent of the peak PMF outflow. The spillway is therefore judged to be "inadequate"

The structural stability analysis based on available information, assumed strength parameters and material properties and visual inspection indicates that the dam is inadequate in overturning and sliding for all loading conditions except for the normal loading.

It is therefore recommended that within 3 months of notification to the owner an in-depth engineering study be undertaken to more accurately evaluate the stability of the dam and to recommend remedial measures, if required. Within eighteen (18) months of the date of notification to the owner, any modification to the structure as a result of this investigation to achieve stability of the dam under the half (1/2) PMF and full PMF events should be completed. In the interim, a detailed emergency action plan and warning system should be promptly developed. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided. In addition, the dam has a number of problem areas which, if left uncorrected, have the potential to develop into hazardous conditions and must be corrected within twelve (12) months.

The following are the recommended measures which must be corrected:

1. The upstream embankment surfaces should be cleared of vegetation and debris, regraded to their original geometry with suitable embankment material, and protected with riprap. Prior to regrading, the stone training walls at the upstream embankment side should be repaired.

2. The reservoir drain and its control facilities should be made operational to insure that continued deterioration of these structures will not adversely affect the dam.

3. Heavy brush, shrubs, trees and debris should be removed from all locations on the embankment and in the spillway channel. Provide a program of cutting and mowing of the embankment surfaces and spillway channel.

4. Replace the deteriorated mortar joints between the stone tiers of the downstream stone training walls. Monitor by visual inspection and continual leakage through these joints; record estimated flow quanitities and describe the clarity of the flow.

5. Investigate the leakage which is occurring within the valve chamber. Monitor periodically by visual inspection the leakage in this area and record estimated flow quantities and describe the clarity of the flow.

6. Provide a program of periodic inspection and maintenance of the dam and its appurtenances, including

yearly operation and lubrication of the reservoir drain and its control facilities. Document this information for future reference. Develop an emergency action plan and periodically update the plan during the life of the structure.

Eugene O'Brien

Eugene O'Brien, P.E.
New York No. 29823

Approved By:

Col. W. M. Smith, Jr.
New York District Engineer

Date: 18 AUG 1981

OVERVIEW

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
BYRAM LAKE RESERVOIR DAM
I.D. NO. N.Y. 1175
D.E.C. NO. 345
LONG ISLAND BASIN
WESTCHESTER 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 Contract No. DACW 51-81-C-0008 in a letter dated 14 December 1980 in fulfillment of the requirements of the National Dam Inspection Act, Public Law 92-367 dated 8 August 1972.

b. Purpose of Inspection

This inspection was conducted to evaluate the existing condition 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 THE PROJECT

a. Description of the Dam and Appurtenant Structures

Byram Lake Dam consists of an embankment section divided by a center ungated spillway section. The dam is approximately 185 feet in total length and has a maximum height of approximately 27 feet.

The crest width of the embankment section is 15 feet. The downstream slope varies from approximately 1V:2 to 2.5H (Vertical to Horizontal). The upstream slope measured to be 1V:10H to a distance of 15 feet from the upstream crest edge (See Section 3). The upstream slope is partially protected with riprap.

The spillway section is constructed of stone/masonry and it is located at the approximate center of the dam. The crest width is 10 feet and is 6 feet lower than the embankment crest. A single permanent concrete flashboard, 1.9 feet high and 1 foot thick, extends longitudinally along the crest. Stone/masonry training walls form the sidewalls for the spillway and extend upstream and downstream of the section.

The reservoir drain for the project consists of a 20-inch diameter pipe located at the base of the spillway. Discharge is controlled by a valve located within the section; access to the valve is via a chamber and connecting gallery.

Discharge through the drain and over the spillway enters a 4 ft high rectangular masonry channel. The channel extends downstream perpendicular to the axis of the dam.

A concrete swale is located at the right abutment contact. The channel collects water from another basin located downstream and west of the dam.

b. Location

Byram Lake Dam is located at the southern end of Byram Lake in the township of North Castle, Westchester County, New York. The dam is located approximately 0.1 mile from Byram Lake Road, one to two miles north of Armonk, New York.

c. Size Classification

The dam is 27 feet high and the reservoir has a storage capacity of 2909 acre-feet. The dam is classified as "intermediate" in size.

d. Hazard Classification

The dam is classified as high hazard due to the large number of homes located approximately 2 miles downstream in the town of Armonk, New York.

e. Ownership

The dam is owned and operated by the Village of Mt. Kisco. The persons to contact concerning operations of the dam are Mr. Howard Zane, Village Engineer, and Mr. James Canero, Water Foreman, both at 104 Main Street, Mt. Kisco, New York 10549. Telephone No. (914) 241-0500.

f. Purpose of Dam

The purpose of the dam is to create a water supply reservoir for the Village of Mt. Kisco and for smaller outlying communities located in the Town of Bedford.

g. Design and Construction History

It is unknown as to when the dam was designed or constructed. However, according to available documents (see Appendix G), Byram Lake was part of the New York City water supply system until 1958, at which time it was purchased by the Village of Mt. Kisco.

h. Normal Operating Procedure

According to the available documents which are presented in Appendix G, water for village use is drawn from the reservoir at a pumping station located at the north end of the lake. The water is pumped through a 12-inch main to two open reservoirs located one-half mile west of the lake along Byram Lake Road. Under normal water system operations, the water level in the lake is at El 551 (MSL), or approximately 0.5 feet below the top of the permanent concrete flashboard.

1.3 PERTINENT DATA

a. Drainage Area (square miles)	1.18
b. Discharge at Dam Site (cfs)	
Ungated Spillway (Top of Embankment)	272.8
Reservoir Drain	Inoperable
c. Elevation (Feet above MSL, USGS Datum)	
Top of Embankment	455.55
Spillway Crest	451.47
d. Reservoir	
Length of Maximum Pool (Top of Embankment) (miles)	Unknown
Length of Normal Pool (Top of Flashboard) (miles)	1.34
Surface Area (acres)	163.9
e. Storage (acre-feet)	
Top of Embankment (Maximum)	3610
Top of Flashboard (Normal)	2909
f. Embankment Sections	
Type	Earthfill
Length (feet)	175 (Total)
Upstream Slope	1V:10H (See Section 3)
Downstream Slope	1V:2 to 2.5H
Height (feet)	27
Crest Width (feet)	15
Cutoff	Unknown
g. Spillway	
Type	Stone/Masonry overflow sill with vertical and sloping upstream and downstream faces respectively

Length (feet)	12
Crest Width (feet)	10
Height (feet)	21
Apron	Unknown

h. <u>Reservoir Drain</u>

Type	Unknown
Diameter (inches)	20
Control	Valve

SECTION 2 - ENGINEERING DATA

2.1 GEOLOGY

Byram Lake Dam is located in the New England Upland Section of the New England Maritime Physiographic Province (Ref. 4). The bedrock in this Section consists of metamorphic, igneous and sedimentary rocks which have undergone a complex sequence of deposition, folding, faulting and erosion. The rock at the damsite is Fordham gneiss of Precambrian Age; the rock is not exposed at the site (Ref.5).

2.2 SUBSURFACE INVESTIGATIONS

There are no subsurface investigation data available for the project. The surface soils of this Section are of glacial origin and are composed of sands, silts and gravels.

2.3 DAM AND APPURTENANT STRUCTURES

The only available design records show a cross-section of the spillway section prepared by the City of New York, Department of Water Supply, Gas and Electricity. This section is shown in Appendix A.

2.4 CONSTRUCTION RECORDS

No information has been located regarding the construction of the dam and its appurtenances.

2.5 OPERATION RECORDS

The dam impounds water for use by the Village of Mt. Kisco. Records are kept for the water supply pumping operations which are performed at the northern end of the lake. No records of discharge at the dam are kept for the project.

2.6 EVALUATION OF DATA

The information obtained from the available documents and a visual inspection is considered adequate for a Phase I inspection and evaluation.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General

A visual inspection of Byram Lake Dam was made on 14 May 1981. The weather was partly cloudy and the temperature was 65° F. At the time of this inspection the reservoir level was 2 feet below the spillway crest.

b. Embankment

The horizontal and vertical alignment of the embankment section appears to be good. The crest of the embankment to the left of the spillway is grassed; the crest to the right contains some small bushes (see PHOTOGRAPH 2).

The general condition of the upstream embankment surface is poor. Debris and vegetation consisting of small bramble bushes exist along the slope (see PHOTOGRAPHS 1 and 3). The riprap has been deteriorated and/or eroded, resulting in the erosion of the dam, particularly adjacent to the spillway training walls and along the upstream crest edge (see PHOTOGRAPH 4 and 8).

The downstream slope of the embankment section is covered with fallen trees and vegetation consisting of small brambles to large trees approximately 18 inches in diameter. The slope appears to be stable with no signs of shallow slope failures.

There is no emergency action plan for the project.

c. Spillway

The exposed surfaces of the spillway section appear to be in good condition. There is no evidence of cracking or other structural distress (see PHOTOGRAPH 5). The condition of the permanent concrete flashboard and sill is also good (see PHOTOGRAPH 6).

The stone/masonry training walls are in good condition. Some deterioration has occurred, however, along the upstream embankment side, probably due to wave action (see PHOTOGRAPH 7); it appears that the mortar joints between the tiers has deteriorated allowing seepage to exit through the joints.

d. Appurtenant Structures

The reservoir drain was not operated during this inspection. According to Mr. Canero, the drain control facilities were destroyed by vandals in the late 1960's. Since then, the drain has not been operated and the metal doors to the valve chamber have been welded shut (see PHOTOGRAPH 9). Wetness was observed on the sill beneath the valve chamber door.

e. Downstream Channel

The downstream channel is rectangular with 4 ft high stone sidewalls and a boulder bottom (see PHOTOGRAPH 10). For the most part the channel is clear of debris, except at the base of the spillway section which contains logs, boards and other debris (see PHOTOGRAPH 7).

f. Reservoir Area

The reservoir area consists of moderately rolling to steep terrain. The slopes appear stable, with no signs of past movement. There appears to be no sedimentation problems in the reservoir area.

g. Abutments

The concrete swale located at the right abutment contact is in good condition (see PHOTOGRAPH 11). There were no signs of major distress at either abutment contacts.

3.2 EVALUATION OF OBSERVATIONS

Visual observations made during the course of this inspection did not reveal serious problems which would affect the adequacy of the dam and its appurtenant facilities. The following summarizes in order of importance, the encountered problem areas with the recommended remedial action:

1. The upstream embankment surfaces should be cleared of vegetation and debris, regraded to their original geometry with suitable embankment material, and protected with riprap. Prior to regrading, the stone training walls at the upstream embankment side should be repaired.

2. The reservoir drain and its control facilities should be made operational to insure that continued deterioration of these structures will not adversely affect the dam.

3. Heavy brush, shrubs, trees and debris should be removed from all locations on the embankment and in the

spillway channel. Provide a program of cutting and mowing of the embankment surfaces and spillway channel.

4. Replace the deteriorated mortar joints between the stone tiers of the downstream stone training walls. Monitor by visual inspection any continual leakage through these joints; record estimated flow quantities and describe the clarity of the flow.

5. Investigate the leakage which is occurring within the valve chamber. Monitor by visual inspection the leakage in this area; record estimated flow quantities and describe the clarity of the flow.

6. Provide a program of periodic inspection and maintenance of the dam and its appurtenances, including yearly operation and lubrication of the reservoir drain and its control facilities. Document this information for future reference. Develop an emergency action plan and periodically update the plan during the life of the structure.

SECTION 4 - OPERATIONS AND MAINTENANCE

4.1 PROCEDURES

The reservoir drain and its control facilities have not been operational for over 10 years. Discharge from the lake is controlled from the water supply pumping station located at the north end of the reservoir. There are no operation procedures, aside from water supply pumping operations, which control discharge over the spillway.

4.2 MAINTENANCE OF THE DAM

According to Mr. Canero, there is no formal procedure for maintaining the dam. Maintenance is carried out by the Village of Mt. Kisco on an "as-needed" basis.

4.3 WARNING SYSTEM IN EFFECT

No warning system is in effect or in preparation.

4.4 EVALUATION

The overall maintenance of the dam is considered to be inadequate, as follows:

1. The deterioration of riprap along the upstream embankment surfaces has caused erosion of the embankment and deterioration of the stone training walls.

2. Vegetation consisting of small bushes to large diameter trees have been allowed to grow on embankment surfaces.

3. Leakage is occurring through the deteriorated mortar joints of the downstream training walls and within the valve chamber as evidenced by leakage beneath chamber door.

4. The reservoir drain and its control facilities are not operational.

5. No formal operation and maintenance manual exists for the project.

SECTION 5 - HYDROLOGIC/HYDRAULIC

5.1 DRAINAGE BASIN CHARACTERISTICS

The Byram Lake Dam is located at the upstream end of Byram River in the North Castle Township, Westchester County. The Hydrologic Unit Code Number is 01100006. The drainage basin extends north into Bedford Township and is roughly rectangular in shape with an area of 1.18 square miles. The basin, which consists of a north/south oriented valley with very steep side slopes, has little storage capacity.

During normal flow periods, additional runoff from a 0.20 square mile area southwest of the dam is drained into the lake by a concrete channel. In the event of the Probable Maximum Flood (PMF), it is assumed that the high lake elevation will cause overbank flow in the channel thereby resulting in no contribution to the lake level. Therefore, in the PMF analysis, this small area was not considered.

5.2 ANALYSIS CRITERIA

The analysis of the adequacy of the spillway is performed by developing a design flood, using the unit hydrograph method and the Probable Maximum Precipitation (PMP). The all season 200 square miles 24 hours PMP for the Byram Lake area, taken from Weather Bureau sources, is 22 inches. For computational convenience, the basin including the lake area is divided into three sub-basins. Inflow hydrograph from each sub-basin is computed using the U.S. Army Corps of Engineers HEC-1DB computer program (Ref. 1). For unit hydrograph computations, the Snyder coefficients C_T and C_P are assigned as 2 and 0.5, respectively. Initial loss of 1.0 inch and constant loss of 0.1 inch/hour were estimated as representative of the sub-basins for the design storm.

In accordance with the recommended guidelines for Safety Inspection of Dams (Ref. 3), the adequacy of the spillway is analyzed using the PMF. A multi-ratio analysis was performed for the full, 0.75, 0.50 and 0.25 PMF.

5.3 SPILLWAY CAPACITY

The ungated stone masonry spillway with a permanent concrete flashboard at crest elevation 451.47 ft (MSL) is 10.0 feet long and has vertical wingwalls up to elevation 455.55 ft. The computed maximum discharge with the water surface at elevation 455.55 ft (top of dam) is 272.8 cfs.

5.4 RESERVOIR CAPACITY

The normal reservoir capacity is 2909 acre-feet. The computed surcharge storage of 701 acre-feet is equivalent to approximately 11 inches of runoff over the entire basin.

5.5 FLOODS OF RECORD

There are no records available of floods or maximum lake elevation.

5.6 OVERTOPPING POTENTIAL

The potential of the dam being overtopped is investigated based on the spillway discharge capacity and the available surcharge storage to meet selected design flood inflows.

The analysis is performed using the above mentioned HEC-1DB computer package and assuming that the water surface in the reservoir is at spillway crest elevation at the beginning of the flood event. Table 1 summarizes the computer analysis.

TABLE 1

RATIO OF PMF (%)	PEAK INFLOW (cfs)	PEAK OUTFLOW (cfs)	OVERTOPPING (Ft)
100	3631	1486	1.50
75	2723	651	0.66
50	1816	209	0.00
25	908	80	0.00

The analysis indicates that the dam would be overtopped by all floods exceeding 61 percent of the PMF. The maximum spillway discharge capacity is 18.4 percent of the peak PMF outflows

5.7 EVALUATION

The spillway is inadequate to pass the routed PMF outflow without overtopping; however, the spillway will pass the 1/2 PMF outflow. The spillway is inadequate for all storms in excess of 61% of the PMF.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

Visual observations did not reveal conditions which would adversely affect the stability of the dam at the present time. The dam and appurtenances do have some deficiencies, however, which if left uncorrected, could potentially affect the stability of the dam. These deficiencies are as follows:

1. Erosion of the upstream slope, particularly along the crest edge, has occurred due to the lack of adequate slope protection.

2. Leakage is occurring through the joints in the downstream stone training wall and within the valve chamber as evidenced by seepage under the chamber door.

3. The reservoir drain and its control facilities are not operational.

b. Design and Construction Data

The original design computations regarding the structural stability of the dam are not available. Any construction data are also not available.

c. Operating Records

No operating records are kept for the project. No major operation problems which would affect the stability of the dam were reported.

d. Post-Construction Changes

There are no recorded post-construction changes for the project.

e. Seismic Stability

According to the recommended Corps of Engineers guidelines, the dam is located in Seismic Zone No. 1; therefore, no seismic stability analysis for this dam was performed.

6.2 STRUCTURAL STABILITY ANALYSIS

A structural stability analysis was performed for the spillway section presented in Appendix A and in accordance with recommended Corps of Engineers guidelines. The following lists the cases analyzed and the results of the analysis.

Case	Description of Loading Conditions
I	Normal Loading, Lake Level at El 451.47, No Tailwater, Full Uplift
II	Same as Case I, with 5K/LF, Ice Load
III	Unusual Loading, 1/2 PMF, Lake Level at El 454.90, Tailwater Depth 2.4 feet
IV	Extreme Loading, Full PMF, Lake Level at El 457.0, Tailwater Depth 7.0 feet

SUMMARY OF RESULTS

Case	Location of Resultant	Sliding Factor of Safety
I	Inside Middle Third	4.80
II	4.45 Feet Outside Middle Third	2.69
III	0.17 Feet Outside Middle Third	2.62
IV	2.10 Feet Outside Middle Third	2.13

Structural stability analyses based on available information and the visual inspection indicate that the spillway section is inadequate in overturning and sliding except for the normal loading case.

SECTION 7 - ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

Phase I investigation of Byram Lake Dam did not indicate conditions which constitute an immediate hazard to human life and property. Based on engineering judgment and the past performance record of the structure, the project appears to be in fair condition. The project, however, does have deficiencies and inadequacies which, if not remedied, have the potential for developing into hazardous conditions.

Using Corps of Engineers screening criteria for review of spillway adequacy, it has been determined that the dam would be overtopped for all storms exceeding approximately 61 percent of the Probable Maximum Flood (PMF). The spillway is, therefore judged to be inadequate.

The results of the stability analysis indicate that the dam is inadequate in overturning and sliding for all loading conditions except for the normal loading. The analysis however, may not incorporate the actual material properties of the foundation nor the actual loading conditions. It is therefore recommended that an in-depth engineering investigation be performed to more accurately evaluate, based on field investigations, the stability of the structure and to propose remedial measures, if required.

b. Adequacy of Information

The information and data available were adequate for the performance of this investigation.

c. Need for Additional Investigations

An in-depth engineering investigation should be undertaken to more accurately evaluate the structural stability of the spillway. The investigation should include, but not be limited to, a field investigation to determine the material properties of the spillway and foundation. The investigation should provide remedial measures such that the dam is stable under flood conditions equal to half (1/2) PMF and PMF.

d. Urgency

The in-depth engineering investigation which is required must be initiated within three months from the date of notification. Within 18 months of notification, remedial measures as a result of this investigation must be initiated with completion of these measures within the following year. In the interim, develop an emergency action plan for notification of downstream residents and proper around-the-clock surveillance of the dam during periods of extreme runoff. The other problem areas listed below must be corrected within one year of notification.

7.2 RECOMMENDED MEASURES

1. The results of the aforementioned structural stability investigation will determine the appropriate remedial measures required.

2. The upstream embankment surfaces should be cleared of vegetation and debris, regraded to their original geometry with suitable embankment material, and protected with riprap. Prior to regrading, the stone training walls at the upstream embankment should be repaired.

3. The reservoir drain and its control facilities should be made operational to insure that continued deterioration of these structures will not adversely affect the dam.

4. Heavy brush, shrubs, trees and debris should be removed from all locations on the embankment and in the spillway channel. Provide a program of cutting and mowing of the embankment surfaces and spillway channel.

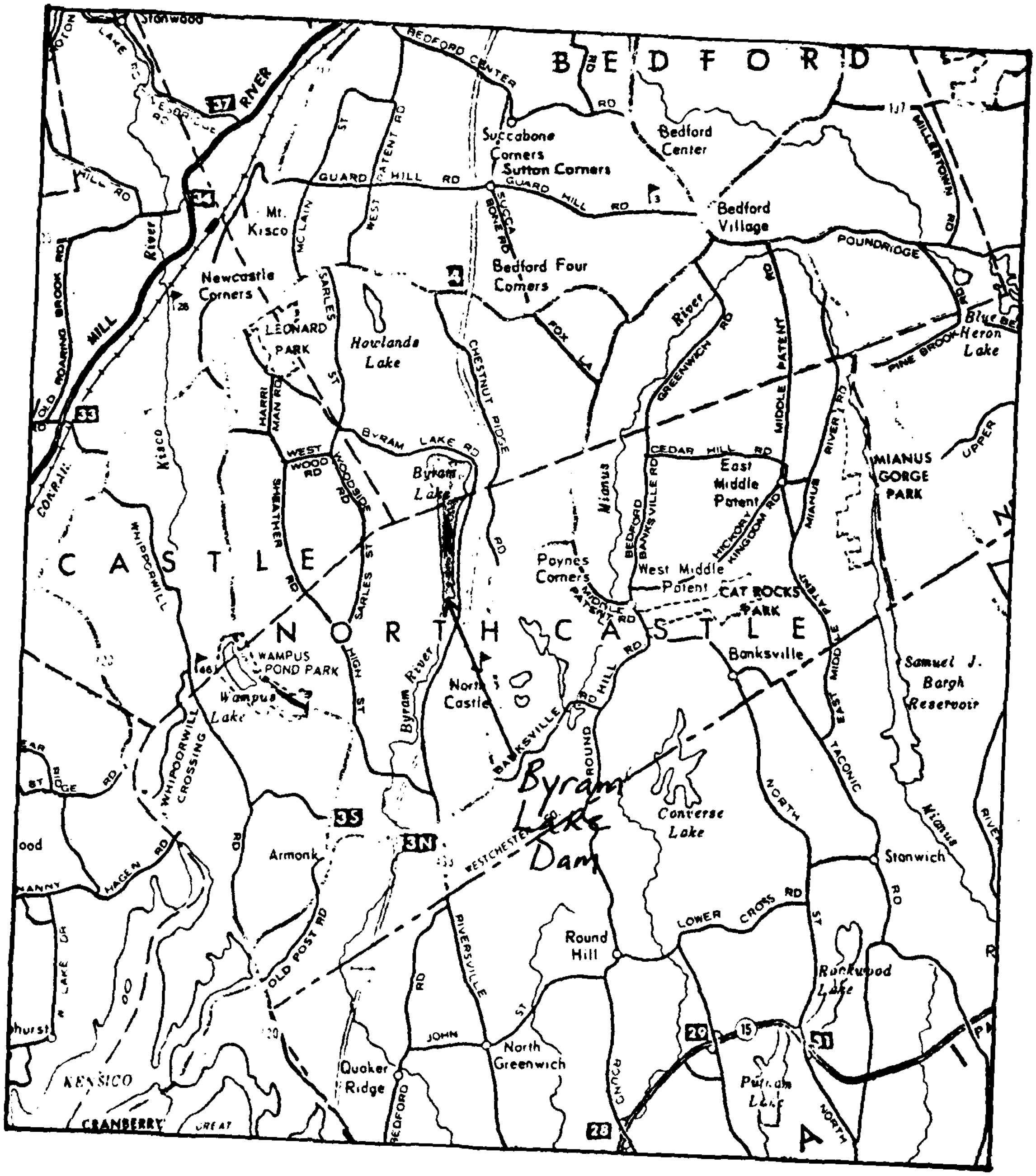
5. Replace the deteriorated mortar joints between the stone tiers of the downstream stone training walls. Monitor by visual inspection any continual leakage through these joints; record estimated flow quantities and describe the clarity of the flow.

6. Investigate the leakage which is occurring within the valve chamber. Monitor by visual inspection the leakage in this area; record estimated flow quantities and describe the clarity of the flow.

7. Provide a program of periodic inspection and maintenance of the dam and its appurtenances, including yearly operation and lubrication of the reservoir drain and its control facilities. Document this information for future reference. Develop an emergency action plan and periodically update the plan during the life of the structure.

DRAWINGS

APPENDIX A



SCALE

0 1 2 3 4 Miles

VICINITY MAP

BYRAM LAKE DAM

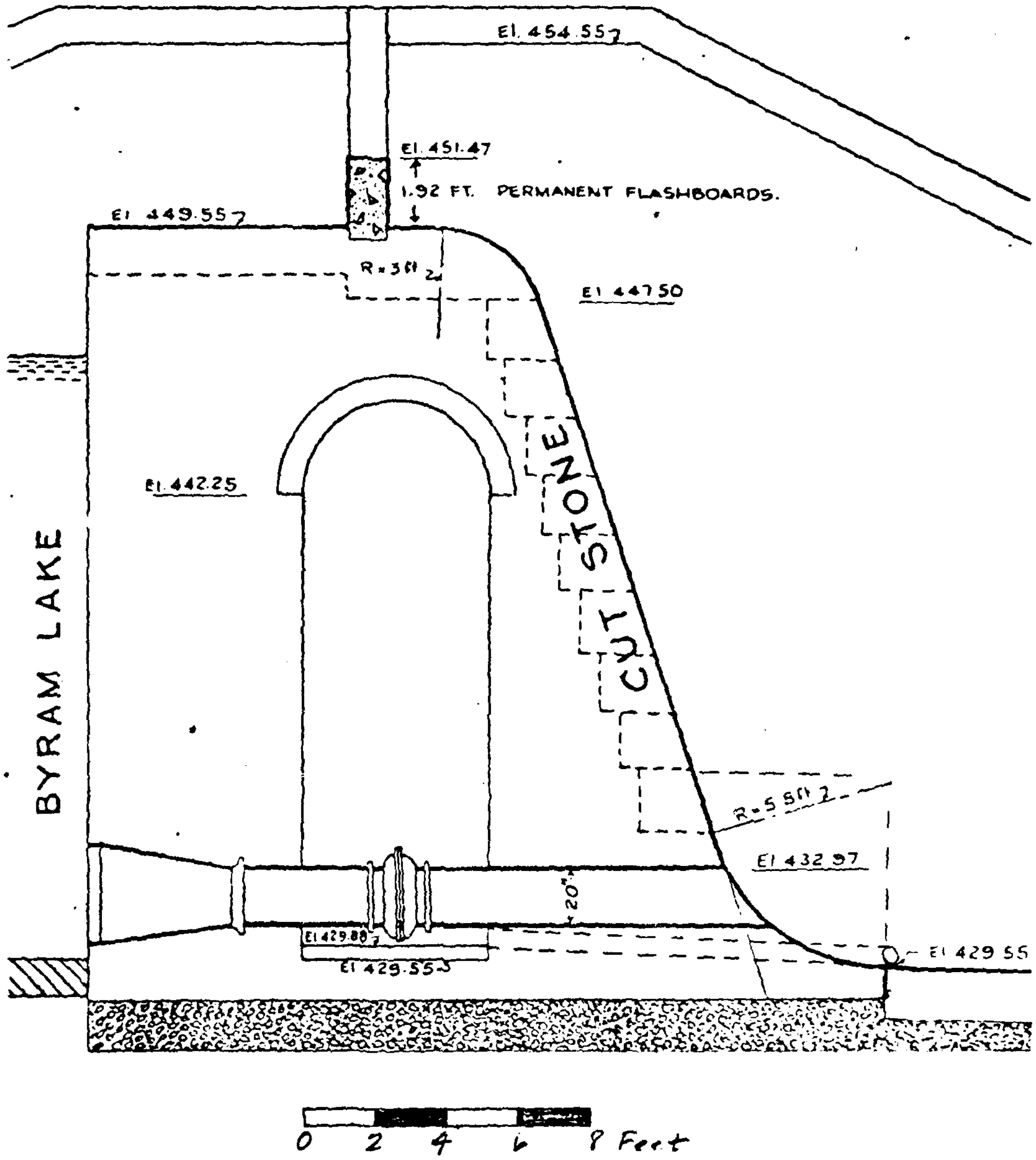
MT. KISCO, N.Y. QUAD

TOPOGRAPHIC MAP

BYRAM LAKE DAM

WSC 469

File -TR17



AREA OF BYRAM LAKE WATER SURFACE.

Original Pond = 121.1 acres.

Lake at masonry spillway level = 158.0 acres

(See Real Estate File # 8200992, Tube # 377)

Note: This Section is copied from the "Map of the Byram River in the State of New York - 1889" Dr 276 X

Elevations refer to Sandy Hook. To convert to Croton Datum, add 0.45 ft

DRAWN BY S.O

CITY OF NEW YORK

DEPARTMENT OF WATER SUPPLY, GAS AND ELECTRICITY

SECTION OF

SPILLWAY

OF DAM AT BYRAM LAKE

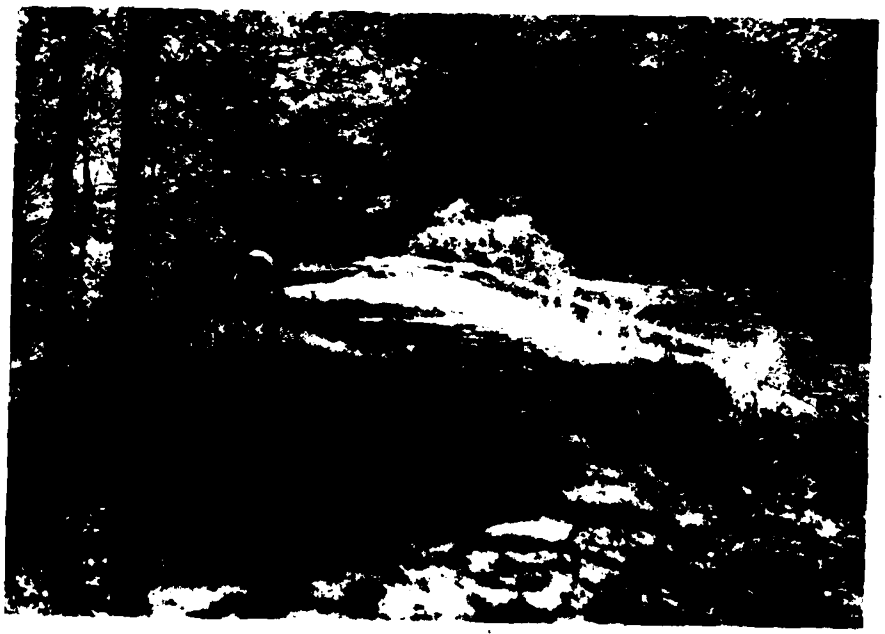
DIVISION OF WATER SUPPLY CONTROL

SCALE 1in = 4ft - NOVEMBER 1944

PHOTOGRAPHS

APPENDIX B

1. UPSTREAM SLOPE OF LEFT EMBANKMENT SECTION.



2. CREST OF LEFT EMBANKMENT SECTION.

3. UPSTREAM SLOPE OF RIGHT EMBANKMENT SECTION. (NOTE: Debris)

4. EROSION OF UPSTREAM CREST EDGE OF LEFT EMBANKMENT SECTION.

5. SPILLWAY VIEWED FROM THE DOWN-STREAM CHANNEL. (NOTE: Debris at Base of Spillway)

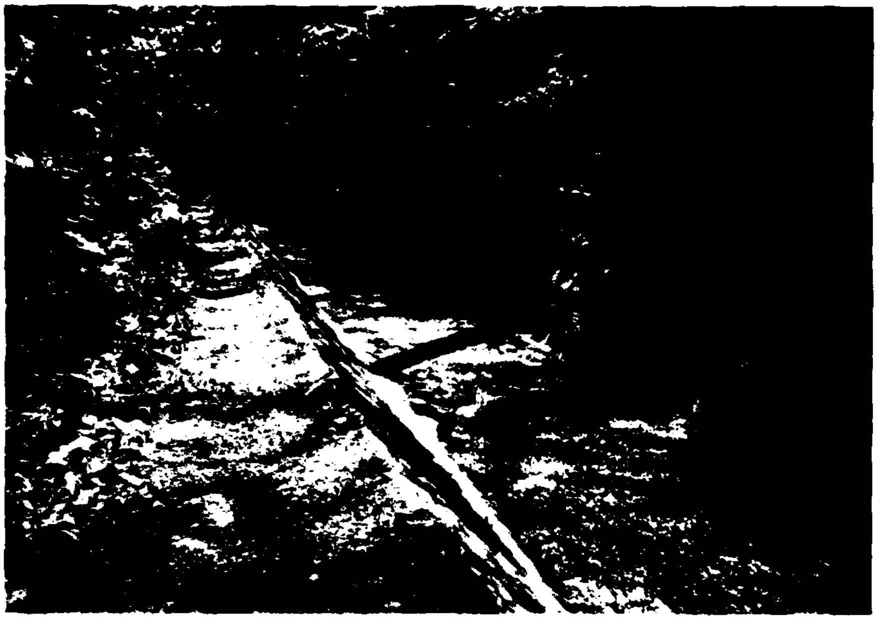
6. VIEW OF SPILLWAY SILL, PERMANENT FLASHBOARD AND STONE/MASONRY SIDEWALLS.

7. DEBRIS AND SEEPAGE THROUGH DOWNSTREAM TRAINING WALL AT BASE OF SPILLWAY.

8. UPSTREAM SPILLWAY TRAINING WALLS. (OBSERVE EROSION OF EMBANKMENT AND DETERIORATION OF STONE/MASONRY TRAINING WALLS)

9. GATE CHAMBER, METAL DOORS ARE WELDED SHUT. (OBSERVE WET AREA ON SILL BELOW DOORS).

10. DOWNSTREAM SPILLWAY CHANNEL. (OBSERVE DEBRIS AND FALLEN TREES)



11. CONCRETE SWALE AT LEFT ABUTMENT.

VISUAL INSPECTION CHECKLIST

APPENDIX C

VISUAL INSPECTION CHECKLIST

1) Basic Data

a. General

Name of Dam Byram Lake Reservoir Dam

Fed. I.D. # NY1175 DEC Dam No. 346

River Basin Long Island

Location: Town Bedford and North Castle County Westchester

Stream Name Byram River

Tributary of Unknown

Latitude (N) 41° - 09' - 18" Longitude (W) 073° - 41' - 36"

Type of Dam Earth Embankment with center stone/masonry spillway

Hazard Category High

Date(s) of Inspection 14 May 1981

Weather Conditions Partly Cloudy 60°F

Reservoir Level at Time of Inspection 2.0 ft below spillway crest

b. Inspection Personnel Mr. Harvey S. Feldman and Mr. Albert DiBernardo

c. Persons Contacted (Including Address & Phone No.)

Mr. Howard Zane, Village Engineer, Village of Mt. Kisco, 104 Main Street, Mt. Kisco, New York 10549 (914) 241-0500 & Mr. James Cancro of the same address

d. History:

Date Constructed Unknown Date(s) Reconstructed Unknown

Designer Unknown

Constructed By Unknown

Owner Village of Mount Kisco

1. Embankment

a. Characteristics

(1) Embankment Material *Earthfill.*

(2) Cutoff Type *Unknown, however, it does not appear that a cutoff exists within the dam.*

(3) Impervious Core *Unknown*

(4) Internal Drainage System *Unknown*

(5) Miscellaneous *None*

b. Crest

(1) Vertical Alignment *Appears to be good*

(2) Horizontal Alignment *Appears to be good.*

(3) Surface Cracks *None were observed along the exposed surfaces of the dam*

(4) Miscellaneous *There were some shrubs and tall thickets along the crest of the right embankment section*

c. Upstream Slope

(1) Slope (Estimate) (V:H) *1:10 as measured about 10 feet from the crest.*

(2) Undesirable Growth or Debris, Animal Burrows *Some vegetation consisting of small brambles exist along the left embankment upstream slope.*

(3) Sloughing, Subsidence or Depressions *Erosion has occurred along the slope due to lack of riprap*

(4) Slope Protection The slope protection consist of small stone. The upper 5 to 8 feet of the slope is unprotected. There appears to be some erosion at the upstream crest edge, particulary at the left embankment

(5) Surface Cracks or Movement at Toe None was observed, however the reservoir surface was at a higher elevation than the toe.

d. Downstream Slope

(1) Slope (Estimate - V:H) 1:2

(2) Undesirable Growth or Debris, Animal Burrows Small brush to large diameter trees were observed.

(3) Sloughing, Subsidence or Depressions The slope is fairly regular. There does not appear to be any evidence of sloughing or subsidence No depressions were observed.

(4) Surface Cracks or Movement at Toe None observed

(5) Seepage None observed. No swamp-like vegetation (marsh-grass) was observed. No wet areas or swampy areas were observed along the embankment surfaces or downstream of the dam

(6) External Drainage System (Ditches, Trenches; Blanket) None

(7) Condition Around Outlet Structure The outlet structure is at the base of the spillway. Debris exists here.

(8) Seepage Beyond Toe None observed. Seepage does exist however in the bottom three tiers of the d/s training wall This is probably due to the deterioration of the mortar joints

e. Abutments - Embankment Contact

A diversion channel is located at the right abutment The channel collects water from runoff downstream of the dam and diverts it to the reservoir.

(1) Erosion at Contact None observed

(2) Seepage Along Contact None observed

3) Drainage System

a. Description of System None Exists

b. Condition of System Not Applicable

c. Discharge from Drainage System Not Applicable

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

Not Applicable

5) Reservoir

a. Slopes *The reservoir slopes are relatively steep and are bedrock with soil overburden*

b. Sedimentation *No signs of excessive sedimentation was observed. No indications of any activities which may increase sediment load in the near future.*

c. Unusual Conditions Which Affect Dam *Interstate 684 exists along the east side of the reservoir. None of the water from this roadway is channelled to the reservoir*

6) Area Downstream of Dam

a. Downstream Hazard (No. of Homes, Highways, etc.) *The town of Armonk is located approx 2mi. d/s*

b. Seepage, Unusual Growth *None observed*

c. Evidence of Movement Beyond Toe of Dam *None Observed*

d. Condition of Downstream Channel *Except for the debris at the base of the spillway, the d/s channel is in relatively good, clear condition.*

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

The stone/masonry spillway is located at the approximate center of the dam

a. General *The spillway is a stone/masonry structure about 10' wide & 6' below the emb^{MT} crest. The section has a concrete sill with a permanent concrete flashboard to a height ≅ 1.9' above the sill*

b. Condition of Service Spillway *Good. There are no cracks or other evidence of structural distress. The ~~co~~ stone training walls form the spillway sidewalls and are also in good condition, except for the seepage condition previously noted.*

c. Condition of Auxiliary Spillway None

d. Condition of Discharge Conveyance Channel The downstream channel is approx. 4' high with ~~ss~~ stone sidewalls and a boulder bottom. It is in relatively good condition.

8) Reservoir Drain/Outlet

Type: Pipe ✓ Conduit ______ Other ______

Material: Concrete ______ Metal ______ Other Unknown

Size: 20" ϕ Length 21' approx

Invert Elevations: Entrance Unknown Exit Unknown

Physical Condition (Describe): Unobservable ✓

Material: ______

Joints: ______ Alignment ______

Structural Integrity: Inoperable since 1960's when it was vandalized by dynamite

Hydraulic Capability: Unknown

Means of Control: Gate ______ Valve ✓ Uncontrolled ______

Operation: Operable ______ Inoperable ✓ Other ______

Present Condition (Describe): Could not be observed since the valve chamber door is welded shut.

) Structural

a. Concrete Surfaces See (7)

b. Structural Cracking None observed

c. Movement - Horizontal & Vertical Alignment (Settlement) None Observed; Alignment appears to be good in [illegible] ntal and vertical alignment

d. Junctions with Abutments or Embankments The spillway abuts into the adjacent training walls. No leakage was observed

e. Drains - Foundation, Joint, Face N.A.

f. Water Passages, Conduits, Sluices [illegible] description of reservoir drain

g. Seepage or Leakage None observed through exposed surfaces of spillway

h. Joints - Construction, etc. No construction joints were observed along the d/s surface of the spillway

i. Foundation Could not be observed. No seepage or undermining was observed

j. Abutments N.A.

k. Control Gates N.A.

l. Approach & Outlet Channels N.A.

m. Energy Dissipators (Plunge Pool, etc.) N.A

n. Intake Structures N.A.

o. Stability Appears Stable (See Stability Computations)

p. Miscellaneous N.A

HYDROLOGIC DATA AND COMPUTATIONS

APPENDIX D

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	455.55		
2) Design High Water (Max. Design Pool)	Unknown	Unknown	Unknown
3) Auxiliary Spillway Crest	N.A.	N.A.	N.A.
4) Pool Level with Flashboards	See (5)	—	—
5) Service Spillway Crest	451.47		

DISCHARGES

	Volume (cfs)
1) Average Daily	Unknown
2) Spillway @ Maximum High Water	382
3) Spillway @ Design High Water	Unknown
4) Spillway @ Auxiliary Spillway Crest Elevation	N.A.
5) Low Level Outlet	Unknown
6) Total (of all facilities) @ Maximum High Water	382
7) Maximum Known Flood	Unknown
8) At Time of Inspection	2' below S/W Crest

CREST: ELEVATION: EL 455.55

Type: Earth Embankment

Width: 15' Length: 175' (total)

Spillover Uncontrolled Center Spillway

Location Approximate Center of Dam

SPILLWAY:

SERVICE		AUXILIARY
El. 451.47 (Top of Flashboard)	Elevation	N.A.
Sill with perm. flashboard	Type	N.A.
10'	Width	N.A.
	Type of Control	
✓	Uncontrolled	N.A.
	Controlled:	
Although there is a permanent concrete	Type (Flashboards; gate)	N.A.
flashboard, it is not	Number	N.A.
controlled	Size/Length	N.A.
	Invert Material	N.A
	Anticipated Length of operating service	N.A
↑	Chute Length	N.A
N.A.	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	N.A

HYDROMETEROLOGICAL GAGES:

Type : None

Location: NA

Records:

Date - NA

Max. Reading - NA

FLOOD WATER CONTROL SYSTEM:

Warning System: None

Method of Controlled Releases (mechanisms):

Pumping station with 12 inch main for Mt. Kisco water supply is at north end of lake.

DRAINAGE AREA: 1.18 sq. miles

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: Wooded

Terrain - Relief: Steep Slopes

Surface - Soil: Glacial Origin

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

Unknown

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

None

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

None

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

Location: None

Elevation:

Reservoir:

Length @ ~~Maximum~~ Normal Pool 1.34 miles ~~(Miles)~~

Length of Shoreline (@ Spillway Crest) 3.13 miles (Miles)

TAMS

Job No. 1579 - 15

Project BYRAM LAKE RES. DAM

Subject

Sheet 1 of 34

Date JAN 20, 19[illegible]

By D L C

Ch'k. by

LAKE EL				451 MSL
LAKE PERIMETER		8.25"		16500 ft / 3.13 mile
FETCH.				7100 ft / 1.34 mile
LAKE AREA.	5182			
	5002	1.79 / 1.78	1.785 in² -	163.91
	4825			
				22% [illegible]
DRAINAGE AREA.	7600			
	.6778	8.22 / 8.19	8.205 in² -	753.44 ac. / 1.18 mi²
	.5959			
460' CONTOUR.	.5663			
	.5395	2.12 / 2.13	2.13 in² -	195.59 ac.
	.5182			

TAMS

Job No. 1579-15

Project [illegible]

Subject Hydrologic / Hydraulic Computations

Sheet 2 of 34

Date [illegible]

By [illegible]

Ch'k. by

Normal Storage capacity = 2909 Ac. ft.

ELEVATION VS. ~~[illegible]~~ STORAGE

EL	ΔH	AREA (Ac.)	MEAN AREA (Ac.)	VOL. Ac. ft.	STORAGE Ac. ft.
					2909
451.47	0	163.91			
454	2.53	173.31	168.61	427	3336
456	2	180.73	177.[illegible]	354	3690
458	2	188.16	184.45	369	4059
460	2	195.59	192	384	4443

$L_1 = 0.8 = 1600$ ft $= 0.3$ mile

$L_{c1} = 0.4 = 800$ ft $= 0.15$ mile

$L_2 = 1.3 = 2600$ ft $= 0.49$ mile

$L_{c2} = 1000$ ft $= 0.19$ mile

$L_3 = 1.3 = 2600$ ft $= 0.49$ mile

$L_{c3} = 0.2 = [illegible] = 0.08$ mile

TAMS

Job No. 1579-15
Sheet 3 of 34
Project BYRAM LAKE
Date 5/ /
Subject Hydrologic/Hydraulic [illegible]
By [illegible]
Ch'k. by

SUB AREA (1)	DESCRIPTION (2)	PLANIMETER READING (3) (IN²)	AREA (4) (AC.)	AREA SQ. MILES	A.TIMP. (A_K/A)
1	LAND (A_{LD1})	1.37	125.8	0.2	
	LAKE (A_{LK1})	0.74	67.95	0.106	
	TOTAL (A_1)			$\boxed{0.306}$	0.346
2	LAND (A_{LD2})	2.27	208.45	0.326	
	LAKE (A_{LK2})	0.67	61.52	0.096	
	TOTAL (A_2)			$\boxed{0.422}$	0.227
3	LAND (A_{LD3})	2.76	253.44	0.4	
	LAKE (A_{LK3})	0.33	30.30	0.047	
	TOTAL (A_3)			$\boxed{0.447}$	0.105

TAMS

Job No. 15 79-15

Sheet 4 of 34

Project BYRAM LAKE

Date July 21, 19..

Subject HYDROLOGIC / HYDRAULIC COMPUTATIONS

By

Ch'k. by

Sub-area 1.

$$t_p = C_t (L L_{CA})^{0.3}$$

$$= 2(0.3 \times 0.15)^{0.3}$$

$$= 0.79.$$

$$t_r = 0.79/5.5$$

$$= 0.14 \text{ hrs.}$$

for $t_R = 0.5$

$$t_{PR} = 0.79 + 0.25(t_R - t_r)$$

$$= 0.79 + 0.25(0.5 - 0.14)$$

$$t_{PR} = \underline{\underline{0.88}} \text{ hrs.}$$

TAMS

Job No. 1579-15 Sheet 5 of 34

Project BYRAM LAKE Date JULY 21, 1981

Subject HYDROLOGIC / HYDRAULIC COMPUTATIONS By D. K. B.

Ch'k. by

SUB-AREA 2.

$$t_p = C_t (L L_{CA})^{0.3}$$

$$= 2 (0.49 \times 0.19)^{0.3}$$

$$= 0.98$$

$$t_r = 0.98 / 5.5$$

$$= 0.178$$

for $t_R = 0.5$

$$t_{PR} = 0.98 + 0.25 (0.5 - 0.178)$$

$$= \underline{1.06 \text{ hrs}}.$$

SUB-AREA 3.

$$t_p = C_t (L L_{CA})^{0.3}$$

$$= 2 (0.49 \times 0.08)^{0.3}$$

$$= 0.76$$

$$t_r = 0.76 / 5.5$$

$$= 0.138$$

for $t_R = 0.5$

$$t_{PR} = 0.76 + 0.25 (0.5 - 0.138)$$

$$= \underline{0.85 \text{ hrs}}.$$

TAMS

Job No. 1579-13 Sheet 6 of 34

Project Byram Lake Dam Date 7/20/1981

Subject By

Ch'k. by

BYRAM LAKE SITUATED AT HEADWATERS of BYRAM RIVER

D/A N/S LENGTH ≃ 9800'

LAKE N/S LENGTH ≃ 7100'

ESTIMATED T_p 1/2 hour. (less on W steep slopes) (slightly more to the N + East.)

Use $640 C_p$ = 320 $C_p = 0.5$

All Season 200 SQMI 24 hour PMP for Westchester Co (Z1) 22 inches

Dur	6 hr	12 hr	24 hr	48 hr
%	112	123		

SUB AREA	1	2	3
D/A	=	=	
L	= 0.3 mi	= 0.49	= 0.49
L_{CA}	= 0.15 mi	= 0.19	= 0.08
T_p	= 0.79	= .98	0.76
A (mile²)			

$Q = \frac{640 C_p}{T_p}$ 400

TAMS

Job No. 1579-15 | Sheet 7 of 34
Project BYRAM LAKE | Date JULY 20. 1981
Subject HYDROLOGIC / HYDRAULIC COMPUTATIONS. | By D.L.C
Ch'k. by

L = 10

C = 3.31 (Kings Handbook of Hydraulics)

CREST EL. = 451.47

Spillway dimensions on $$ CARD.

Rating computed by HEC-1DB program.

EL.	H.	C	Q
451.47		3.31	0
453	1.53	3.31	62.6
455	3.53	3.31	219 5
455.55	4.08	3.31	272.8

TAMS

Job No. 1579-15 | Sheet 8 of 34

Project BYRAM LAKE | Date 5/25/81

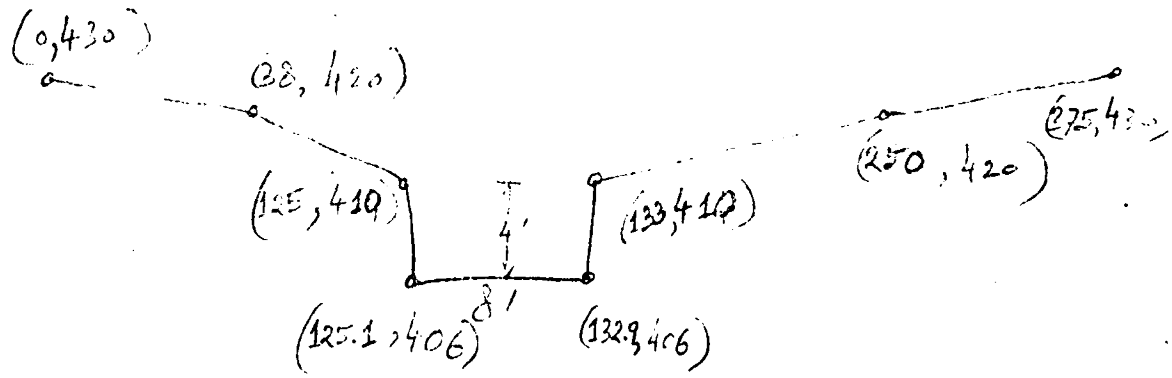
Subject D/S CHANNEL CHARACTERISTICS | By D.K. B...

Ch'k. by

$$\text{LENGTH} = \frac{3.6''}{8} \times \frac{\cancel{24000}^{\,2000}}{\cancel{12}} \text{ ft} = 900 \text{ ft}$$

$$\text{Slope}: \frac{40}{900} = 0.044$$

Cross section



```
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION   1 APR
**********************************

    1          A1                            BYRAM LAKE DAM
    2          A2                           PHASE 1 INSPECTION
    3          A3                        HEC-1DB PMF ANALYSIS  MAY 81
    4          B     100      0      30      0      0      0      0      0      0      0
    5          B1      5
    6          J       1      4      1
    7          J1      1   0.75    0.5   0.25
    8          K       0      1                                 1
    9          K1      1 SUB-BASIN 1 INFLOW HYDROGRAPH
   10          M       1      1  0.306           1.18                        1
   11          P             22    112    123    133    141
   12          T                                                1    0.1          0.346
   13          W    0.88    0.5
   14          X      -1   -0.1    1.5
   15          K       0      2                                 1
   16          K1      2 SUB-BASIN 2 INFLOW HYDROGRAPH
   17          M       1      1  0.422           1.18                        1
   18          P             22    112    123    133    141
   19          T                                                1    0.1          0.227
   20          W    1.06    0.5
   21          X      -1   -0.1    1.5
   22          K       2     10                                 1
   23          K1      3 COMBINE HYDROGRAPHS OF SUB-BASINS 1 AND 2
   24          K       0      3                                 1
   25          K1      4 SUB-BASIN 3 INFLOW HYDROGRAPH
   26          M       1      1  0.447           1.18                        1
   27          P             22    112    123    133    141
   28          T                                                1    0.1          0.105
   29          W    0.85    0.5
   30          X      -1   -0.1    1.5
   31          K       2     11                                 1
   32          K1      5 COMBINE HYDROGRAPHS OF SUB-BASIN 3 AND THE REST
   33          K       1      3                                 1
   34          K1      6 ROUTE THROUGH LAKE
   35          Y                            1      1
   36          Y1      1                                     2909      0
   37          SS   2909   3336   3690   4059   4443
   38          SE451.47    454    456    458    460
   39          $$451.47     10   3.31    1.5
   40          $D455.55   3.09    1.5    185
   41          K       1      4                                 1
   42          K1      6 CHANNEL ROUTE D/S OF DAM
   43          Y                            1      1
   44          Y1      1
   45          Y6  0.035  0.017  0.035    406    430    900  0.044
   46          Y7      0    430     38    420    125    410  125.1    406  132.9    406
   47          Y7    133    410    250    420    375    430
   48          K      99

                  PREVIEW OF SEQUENCE OF STREAM NETWORK CALCULATIONS

                          RUNOFF HYDROGRAPH AT            1
                          RUNOFF HYDROGRAPH AT            2
                          COMBINE  2 HYDROGRAPHS AT      10
                          RUNOFF HYDROGRAPH AT            3
                          COMBINE  2 HYDROGRAPHS AT      11
                          ROUTE HYDROGRAPH TO             3
                          ROUTE HYDROGRAPH TO             4
                          END OF NETWORK
```

Sheet 9 of 34

PREVIEW OF SEQUENCE OF STREAM NETWORK CALCULATIONS

```
RUNOFF HYDROGRAPH AT             1
RUNOFF HYDROGRAPH AT             2
COMBINE  2 HYDROGRAPHS AT       10
RUNOFF HYDROGRAPH AT             3
COMBINE  2 HYDROGRAPHS AT       11
ROUTE HYDROGRAPH TO              3
ROUTE HYDROGRAPH TO              4
END OF NETWORK
```

Sheet 10 of 34

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*************************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
LAST MODIFICATION   01 APR 80
*************************************
```

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******************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  01 APR 80
******************************

RUN  DATE* 81/05/27.
     TIME* 07.42.16.

                                  BYRAM LAKE DAM
                                PHASE 1 INSPECTION
                             HEC-1DB PMF ANALYSIS  MAY 81

                                JOB SPECIFICATION
          NQ     NHR    NMIN    IDAY     IHR    IMIN   METRC    IPLT    IPRT   NSTAN
         100       0      30       0       0       0       0       0       0       0
                                 JOPER     NWT   LROPT   TRACE
                                     5       0       0       0

                         MULTI-PLAN ANALYSES TO BE PERFORMED
                            NPLAN= 1 NRTIO= 4 LRTIO= 1
           RTIOS=   1.00    .75    .50    .25

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

                          SUB-AREA RUNOFF COMPUTATION

          1 SUB-BASIN 1 INFLOW HYDROGRAPH

               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     .31    0.00    1.18    0.00   0.000       0       1       0

                                   PRECIP DATA
                 SPFE     PMS      R6     R12     R24     R48     R72     R96
                 0.00   22.00  112.00  123.00  133.00  141.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS  .800

                                    LOSS DATA
        LROPT   STRKR   DLTKR   RTIOL   ERAIN   STRKS   RTIOK   STRTL   CNSTL   ALSMX   RTIMP
            0    0.00    0.00    1.00    0.00    0.00    1.00    1.00     .10    0.00     .35

                               UNIT HYDROGRAPH DATA
                           TP=   .88   CP= .50   NTA= 0

                                  RECESSION DATA
                     STRTQ=   -1.00   QRCSN=    -.10   RTIOR= 1.50
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 1.82 AND R= 2.12 INTERVALS

           UNIT HYDROGRAPH 13 END-OF-PERIOD ORDINATES, LAG=    .89 HOURS, CP=  .50  VOL= 1.00
         43.     102.      95.      59.      36.      23.      14.       9.       5.       3.
          2.       1.       1.
```

Sheet 11 of 34

```
       0                                END-OF-PERIOD FLOW
   MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q    MO.DA  HR.MN  PERIOD   RAIN   EXCS   LOSS    COMP Q

    1.01    .30       1    .00    .00    .00        0.     1.02   1.30      51    .06    .03    .03        7.
    1.01   1.00       2    .00    .00    .00        0.     1.02   2.00      52    .06    .03    .03        8.
    1.01   1.30       3    .00    .00    .00        1.     1.02   2.30      53    .06    .03    .03        9.
    1.01   2.00       4    .00    .00    .00        1.     1.02   3.00      54    .06    .03    .03        9.
```

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	END-OF-PERIOD FLOW COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.00	.00	.00	0.	1.02	1.30	51	.06	.03	.03	7.
1.01	1.00	2	.00	.00	.00	0.	1.02	2.00	52	.06	.03	.03	8.
1.01	1.30	3	.00	.00	.00	1.	1.02	2.30	53	.06	.03	.03	9.
1.01	2.00	4	.00	.00	.00	1.	1.02	3.00	54	.06	.03	.03	9.
1.01	2.30	5	.00	.00	.00	1.	1.02	3.30	55	.06	.03	.03	10.
1.01	3.00	6	.00	.00	.00	1.	1.02	4.00	56	.06	.03	.03	10.
1.01	3.30	7	.00	.00	.00	1.	1.02	4.30	57	.06	.03	.03	10.
1.01	4.00	8	.00	.00	.00	1.	1.02	5.00	58	.06	.03	.03	10.
1.01	4.30	9	.00	.00	.00	1.	1.02	5.30	59	.06	.03	.03	10.
1.01	5.00	10	.00	.00	.00	1.	1.02	6.00	60	.06	.03	.03	10.
1.01	5.30	11	.00	.00	.00	1.	1.02	6.30	61	.16	.13	.03	15.
1.01	6.00	12	.00	.00	.00	1.	1.02	7.00	62	.16	.13	.03	25.
1.01	6.30	13	.01	.00	.01	1.	1.02	7.30	63	.16	.13	.03	35.
1.01	7.00	14	.01	.00	.01	1.	1.02	8.00	64	.16	.13	.03	41.
1.01	7.30	15	.01	.00	.01	1.	1.02	8.30	65	.16	.13	.03	45.
1.01	8.00	16	.01	.00	.01	1.	1.02	9.00	66	.16	.13	.03	47.
1.01	8.30	17	.01	.00	.01	1.	1.02	9.30	67	.16	.13	.03	48.
1.01	9.00	18	.01	.00	.01	1.	1.02	10.00	68	.16	.13	.03	49.
1.01	9.30	19	.01	.00	.01	1.	1.02	10.30	69	.16	.13	.03	50.
1.01	10.00	20	.01	.00	.01	1.	1.02	11.00	70	.16	.13	.03	50.
1.01	10.30	21	.01	.00	.01	1.	1.02	11.30	71	.16	.13	.03	50.
1.01	11.00	22	.01	.00	.01	1.	1.02	12.00	72	.16	.13	.03	51.
1.01	11.30	23	.01	.00	.01	1.	1.02	12.30	73	.99	.95	.03	86.
1.01	12.00	24	.01	.00	.01	1.	1.02	13.00	74	.99	.95	.03	170.
1.01	12.30	25	.06	.02	.04	2.	1.02	13.30	75	1.18	1.15	.03	257.
1.01	13.00	26	.06	.02	.04	4.	1.02	14.00	76	1.18	1.15	.03	326.
1.01	13.30	27	.07	.02	.05	6.	1.02	14.30	77	1.48	1.45	.03	388.
1.01	14.00	28	.07	.02	.05	7.	1.02	15.00	78	1.48	1.45	.03	448.
1.01	14.30	29	.09	.03	.06	8.	1.02	15.30	79	1.80	1.77	.03	509.
1.01	15.00	30	.09	.03	.06	10.	1.02	16.00	80	5.69	5.66	.03	738.
1.01	15.30	31	.11	.04	.07	11.	1.02	16.30	81	1.38	1.35	.03	995.
1.01	16.00	32	.34	.15	.20	17.	1.02	17.00	82	1.38	1.35	.03	959.
1.01	16.30	33	.08	.05	.03	25.	1.02	17.30	83	1.08	1.05	.03	783.
1.01	17.00	34	.08	.05	.03	26.	1.02	18.00	84	1.08	1.05	.03	652.
1.01	17.30	35	.07	.03	.03	23.	1.02	18.30	85	.09	.06	.03	518.
1.01	18.00	36	.07	.03	.03	20.	1.02	19.00	86	.09	.06	.03	360.
1.01	18.30	37	.01	.00	.00	16.	1.02	19.30	87	.09	.06	.03	231.
1.01	19.00	38	.01	.00	.00	11.	1.02	20.00	88	.09	.06	.03	150.
1.01	19.30	39	.01	.00	.00	7.	1.02	20.30	89	.09	.06	.03	101.
1.01	20.00	40	.01	.00	.00	5.	1.02	21.00	90	.09	.06	.03	96.
1.01	20.30	41	.01	.00	.00	3.	1.02	21.30	91	.09	.06	.03	92.
1.01	21.00	42	.01	.00	.00	3.	1.02	22.00	92	.09	.06	.03	88.
1.01	21.30	43	.01	.00	.00	2.	1.02	22.30	93	.09	.06	.03	85.
1.01	22.00	44	.01	.00	.00	2.	1.02	23.00	94	.09	.06	.03	82.
1.01	22.30	45	.01	.00	.00	2.	1.02	23.30	95	.09	.06	.03	78.
1.01	23.00	46	.01	.00	.00	2.	1.03	0.00	96	.09	.06	.03	75.
1.01	23.30	47	.01	.00	.00	2.	1.03	.30	97	0.00	0.00	0.00	72.
1.02	0.00	48	.01	.00	.00	2.	1.03	1.00	98	0.00	0.00	0.00	69.
1.02	.30	49	.06	.03	.03	2.	1.03	1.30	99	0.00	0.00	0.00	67.
1.02	1.00	50	.06	.03	.03	4.	1.03	2.00	100	0.00	0.00	0.00	64.
									SUM	24.82	22.42	2.40	9385.
										(630.)	(569.)	(61.)	(265.75)

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	998.	577.	190.	94.	9355.
CMS	28.	16.	5.	3.	265.
INCHES		17.54	23.05	23.70	23.70
MM		445.46	585.54	601.97	601.97
AC-FT		286.	376.	387.	387.
THOUS CU M		353.	464.	477.	477.

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	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CMS	28.	16.	5.	3.	265.
INCHES		17.54	23.05	23.70	23.70
MM		445.46	585.54	601.97	601.97
AC-FT		286.	376.	387.	387.
THOUS CU M		353.	464.	477.	477.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 1

0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	2.	4.	6.	7.	8.	10.
11.	17.	25.	26.	23.	20.	16.	11.	7.	5.
3.	3.	2.	2.	2.	2.	2.	2.	2.	4.
7.	8.	9.	9.	10.	10.	10.	10.	10.	10.
15.	25.	35.	41.	45.	47.	48.	49.	50.	50.
50.	51.	86.	170.	257.	326.	388.	445.	509.	738.
993.	959.	783.	652.	518.	360.	231.	150.	101.	96.
92.	88.	85.	82.	78.	75.	72.	69.	67.	64.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	998.	577.	190.	94.	9355.
CMS	28.	16.	5.	3.	265.
INCHES		17.54	23.05	23.70	23.70
MM		445.46	585.54	601.97	601.97
AC-FT		286.	376.	387.	387.
THOUS CU M		353.	464.	477.	477.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 2

0.	0.	0.	0.	0.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	2.	3.	4.	5.	6.	7.
8.	13.	19.	19.	17.	15.	12.	8.	5.	4.
2.	2.	2.	2.	2.	2.	2.	2.	1.	3.
5.	6.	7.	7.	7.	7.	8.	8.	8.	8.
11.	19.	26.	31.	34.	35.	36.	37.	37.	38.
38.	39.	65.	128.	193.	245.	291.	336.	381.	553.
748.	719.	587.	489.	389.	270.	173.	113.	75.	72.
69.	66.	64.	61.	59.	56.	54.	52.	50.	48.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	748.	433.	142.	70.	7016.
CMS	21.	12.	4.	2.	199.
INCHES		13.15	17.29	17.77	17.77
MM		334.10	439.15	451.48	451.48
AC-FT		215.	282.	290.	290.
THOUS CU M		265.	348.	358.	358.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 3

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	2.	3.	4.	4.	5.
5.	8.	12.	13.	11.	10.	8.	6.	4.	2.
2.	1.	1.	1.	1.	1.	1.	1.	1.	2.
3.	4.	4.	5.	5.	5.	5.	5.	5.	5.
7.	13.	17.	20.	22.	24.	24.	25.	25.	25.

25.	25.	43.	85.	129.	163.	194.	224.	254.	359.
499.	480.	392.	326.	259.	180.	115.	75.	50.	48.
46.	44.	42.	41.	39.	38.	36.	35.	33.	32.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	499.	288.	95.	47.	4678.
CMS	14.	8.	3.	1.	132.
INCHES		8.77	11.53	11.85	11.85
MM		222.73	292.77	300.98	[illegible]

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					24.	24.	25.	25.	25.
25.	25.	43.	85.	129.	163.	194.	224.	254.	359.
499.	480.	392.	326.	259.	180.	115.	75.	50.	48.
46.	44.	42.	41.	39.	38.	36.	35.	33.	32.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	499.	288.	95.	47.	4678.
CMS	14.	8.	3.	1.	132.
INCHES		8.77	11.53	11.85	11.85
MM		222.73	292.77	300.98	300.98
AC-FT		143.	188.	193.	193.
THOUS CU M		176.	232.	238.	238.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 4

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	2.	2.	2.
3.	4.	6.	6.	6.	5.	4.	3.	2.	1.
1.	1.	1.	1.	1.	1.	1.	1.	0.	1.
2.	2.	2.	2.	2.	2.	3.	3.	3.	3.
4.	6.	9.	10.	11.	12.	12.	12.	12.	13.
13.	13.	22.	43.	64.	82.	97.	112.	127.	194.
249.	240.	195.	163.	130.	90.	52.	38.	25.	24.
23.	22.	21.	20.	20.	19.	19.	17.	17.	16.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	249.	144.	47.	23.	2339.
CMS	7.	4.	1.	1.	66.
INCHES		4.38	5.76	5.92	5.92
MM		111.37	146.38	150.49	150.49
AC-FT		72.	94.	97.	97.
THOUS CU M		88.	116.	119.	119.

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

SUB-AREA RUNOFF COMPUTATION

2 SUB-BASIN 2 INFLOW HYDROGRAPH

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

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	.42	0.00	1.12	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	22.00	112.00	123.00	133.00	141.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS .800

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.00	0.00	1.00	0.00	0.00	1.00	1.00	.10	0.00	.23

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UNIT HYDROGRAPH DATA
TP= 1.05 CP= .50 NTA= 0

RECESSION DATA
STRTQ= -1.00 QRCSN= -.10 RTIOR= 1.50
APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 2.27 AND R= 2.81 INTERVALS

UNIT HYDROGRAPH 16 END-OF-PERIOD ORDINATES, LAG= 1.07 HOURS, CP= .50 VOL= 1.00
34. 101. 119. 88. 61. 43. [illegible] [illegible]

UNIT HYDROGRAPH DATA

TP= 1.05 CP= .50 NTA= 0

RECESSION DATA

STRTQ= -1.00 QRCSN= -.10 RTIOR= 1.50

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 2.27 AND R= 2.81 INTERVALS

UNIT HYDROGRAPH 16 END-OF-PERIOD ORDINATES, LAG= 1.07 HOURS, CP= .50 VOL= 1.00

34.	101.	119.	98.	61.	43.	30.	21.	15.	10.
7.	5.	3.	2.	2.	1.				

0

END-OF-PERIOD FLOW

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.00	.00	.00	0.	1.02	1.30	51	.06	.02	.04	6.
1.01	1.00	2	.00	.00	.00	0.	1.02	2.00	52	.06	.02	.04	7.
1.01	1.30	3	.00	.00	.00	1.	1.02	2.30	53	.06	.02	.04	8.
1.01	2.00	4	.00	.00	.00	1.	1.02	3.00	54	.06	.02	.04	9.
1.01	2.30	5	.00	.00	.00	1.	1.02	3.30	55	.06	.02	.04	10.
1.01	3.00	6	.00	.00	.00	1.	1.02	4.00	56	.06	.02	.04	10.
1.01	3.30	7	.00	.00	.00	1.	1.02	4.30	57	.06	.02	.04	10.
1.01	4.00	8	.00	.00	.00	1.	1.02	5.00	58	.06	.02	.04	10.
1.01	4.30	9	.00	.00	.00	1.	1.02	5.30	59	.06	.02	.04	11.
1.01	5.00	10	.00	.00	.00	1.	1.02	6.00	60	.06	.02	.04	11.
1.01	5.30	11	.00	.00	.00	1.	1.02	6.30	61	.16	.12	.04	14.
1.01	6.00	12	.00	.00	.00	1.	1.02	7.00	62	.16	.12	.04	25.
1.01	6.30	13	.01	.00	.01	1.	1.02	7.30	63	.16	.12	.04	37.
1.01	7.00	14	.01	.00	.01	1.	1.02	8.00	64	.16	.12	.04	46.
1.01	7.30	15	.01	.00	.01	1.	1.02	8.30	65	.16	.12	.04	52.
1.01	8.00	16	.01	.00	.01	1.	1.02	9.00	66	.16	.12	.04	57.
1.01	8.30	17	.01	.00	.01	1.	1.02	9.30	67	.16	.12	.04	60.
1.01	9.00	18	.01	.00	.01	1.	1.02	10.00	68	.16	.12	.04	62.
1.01	9.30	19	.01	.00	.01	1.	1.02	10.30	69	.16	.12	.04	63.
1.01	10.00	20	.01	.00	.01	1.	1.02	11.00	70	.16	.12	.04	64.
1.01	10.30	21	.01	.00	.01	1.	1.02	11.30	71	.16	.12	.04	65.
1.01	11.00	22	.01	.00	.01	1.	1.02	12.00	72	.16	.12	.04	66.
1.01	11.30	23	.01	.00	.01	1.	1.02	12.30	73	.99	.95	.04	94.
1.01	12.00	24	.01	.00	.01	1.	1.02	13.00	74	.99	.95	.04	178.
1.01	12.30	25	.06	.01	.05	2.	1.02	13.30	75	1.18	1.14	.04	283.
1.01	13.00	26	.06	.01	.05	2.	1.02	14.00	76	1.18	1.14	.04	375.
1.01	13.30	27	.07	.02	.05	4.	1.02	14.30	77	1.48	1.44	.04	459.
1.01	14.00	28	.07	.02	.05	6.	1.02	15.00	78	1.48	1.44	.04	541.
1.01	14.30	29	.09	.02	.07	7.	1.02	15.30	79	1.80	1.76	.04	624.
1.01	15.00	30	.09	.02	.07	8.	1.02	16.00	80	5.69	5.65	.04	840.
1.01	15.30	31	.11	.02	.09	9.	1.02	16.30	81	1.38	1.34	.04	1162.
1.01	16.00	32	.34	.11	.23	13.	1.02	17.00	82	1.38	1.34	.04	1241.
1.01	16.30	33	.08	.04	.04	20.	1.02	17.30	83	1.09	1.05	.04	1097.
1.01	17.00	34	.08	.04	.04	23.	1.02	18.00	84	1.09	1.05	.04	953.
1.01	17.30	35	.07	.03	.04	24.	1.02	18.30	85	.09	.05	.04	804.
1.01	18.00	36	.07	.03	.04	22.	1.02	19.00	86	.09	.05	.04	622.
1.01	18.30	37	.01	.00	.00	17.	1.02	19.30	87	.09	.05	.04	447.
1.01	19.00	38	.01	.00	.00	15.	1.02	20.00	88	.09	.05	.04	320.
1.01	19.30	39	.01	.00	.00	11.	1.02	20.30	89	.09	.05	.04	231.
1.01	20.00	40	.01	.00	.00	8.	1.02	21.00	90	.09	.05	.04	168.
1.01	20.30	41	.01	.00	.00	6.	1.02	21.30	91	.09	.05	.04	125.
1.01	21.00	42	.01	.00	.00	4.	1.02	22.00	92	.09	.05	.04	119.
1.01	21.30	43	.01	.00	.00	3.	1.02	22.30	93	.09	.05	.04	115.
1.01	22.00	44	.01	.00	.00	2.	1.02	23.00	94	.09	.05	.04	110.
1.01	22.30	45	.01	.00	.00	2.	1.02	23.30	95	.09	.05	.04	106.
1.01	23.00	46	.01	.00	.00	2.	1.03	0.00	96	.09	.05	.04	101.

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1.01	23.30	47	.01	.00	.00	2.	1.03	.30	97	0.00	0.00	0.00	97.
1.02	0.00	48	.01	.00	.00	2.	1.03	1.00	98	0.00	0.00	0.00	94.
1.02	.30	49	.06	.02	.04	2.	1.03	1.30	99	0.00	0.00	0.00	90.
1.02	1.00	50	.06	.02	.04	3.	1.03	2.00	100	0.00	0.00	0.00	86.

SUM 24.92 21.98 2.94 12431.

(633.)(558.)(72.)([illegible])

									96	.09	.05	.04		100.
1.01	23.30	47	.01	.00	.00	2.	1.03	.30	97	0.00	0.00	0.00	97.	
1.02	0.00	48	.01	.00	.00	2.	1.03	1.00	98	0.00	0.00	0.00	94.	
1.02	.30	49	.06	.02	.04	2.	1.03	1.30	99	0.00	0.00	0.00	90.	
1.02	1.40	50	.06	.02	.04	3.	1.03	2.00	100	0.00	0.00	0.00	85.	

SUM	24.52	21.98	2.84	12431.
	(633.)(	558.)(	72.)(	352.01)

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1241.	762.	253.	124.	12386.
CMS	35.	22.	7.	4.	351.
INCHES		16.79	22.29	22.75	22.75
MM		426.41	566.06	577.93	577.93
AC-FT		378.	501.	512.	512.
THOUS CU M		466.	618.	631.	631.

HYDROGRAPH AT STA 2 FOR PLAN 1, RTIO 1

0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	2.	3.	4.	6.	7.	8.
9.	13.	20.	25.	24.	22.	19.	15.	11.	8.
6.	4.	3.	2.	2.	2.	2.	2.	2.	3.
6.	7.	8.	9.	10.	10.	10.	10.	11.	11.
14.	25.	37.	46.	52.	57.	60.	62.	63.	64.
65.	66.	94.	178.	283.	375.	459.	541.	624.	840.
1162.	1241.	1097.	933.	804.	622.	447.	320.	231.	168.
125.	119.	115.	110.	106.	101.	97.	94.	90.	86.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1241.	762.	253.	124.	12386.
CMS	35.	22.	7.	4.	351.
INCHES		16.79	22.29	22.75	22.75
MM		426.41	566.06	577.93	577.93
AC-FT		378.	501.	512.	512.
THOUS CU M		466.	618.	631.	631.

HYDROGRAPH AT STA 2 FOR PLAN 1, RTIO 2

0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	2.	3.	4.	5.	6.
7.	10.	15.	18.	18.	17.	14.	11.	8.	6.
4.	3.	2.	2.	2.	2.	2.	2.	1.	2.
4.	5.	6.	7.	7.	8.	8.	8.	8.	8.
11.	19.	28.	34.	39.	42.	45.	46.	47.	48.
49.	49.	70.	133.	212.	281.	344.	406.	468.	630.
871.	931.	823.	715.	603.	467.	335.	240.	173.	126.
93.	89.	86.	82.	79.	76.	73.	70.	67.	65.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	931.	571.	190.	93.	9290.
CMS	26.	16.	5.	3.	263.
INCHES		12.59	16.71	17.06	17.06
MM		319.81	424.54	433.45	433.45
AC-FT		283.	376.	384.	384.
THOUS CU M		349.	464.	474.	474.

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HYDROGRAPH AT STA 2 FOR PLAN 1, RTIO 3

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	2.	3.	3.	4.
4.	6.	10.	12.	12.	11.	10.	7.	[illegible]	4.
3.	2.	2.	1.	1.	1.	1.	1.	1.	[illegible]
[illegible]	4.	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

HYDROGRAPH AT STA 2 FOR PLAN 1, RTIO 3

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	2.	3.	3.	4.
4.	6.	10.	12.	12.	11.	10.	7.	5.	4.
3.	2.	2.	1.	1.	1.	1.	1.	1.	2.
3.	4.	4.	5.	5.	5.	5.	5.	5.	5.
7.	12.	18.	23.	26.	28.	30.	31.	32.	32.
33.	33.	47.	89.	141.	187.	229.	271.	312.	420.
581.	621.	549.	477.	402.	311.	223.	160.	115.	84.
62.	60.	57.	55.	53.	51.	49.	47.	45.	43.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	621.	391.	125.	62.	6193.
CMS	18.	11.	4.	2.	175.
INCHES		8.39	11.14	11.38	11.38
MM		213.21	283.03	288.97	288.97
AC-FT		189.	251.	256.	256.
THOUS CU M		233.	309.	316.	316.

HYDROGRAPH AT STA 2 FOR PLAN 1, RTIO 4

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	1.	1.	1.	2.	2.
2.	3.	5.	6.	6.	6.	5.	4.	3.	2.
1.	1.	1.	1.	1.	1.	1.	1.	0.	1.
1.	2.	2.	2.	2.	3.	3.	3.	3.	3.
4.	6.	9.	11.	13.	14.	15.	15.	16.	16.
16.	16.	23.	44.	71.	94.	115.	135.	156.	210.
290.	310.	274.	238.	201.	156.	112.	80.	58.	42.
31.	30.	29.	27.	26.	25.	24.	23.	22.	22.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	310.	190.	63.	31.	3097.
CMS	9.	5.	2.	1.	88.
INCHES		4.20	5.57	5.69	5.69
MM		106.60	141.51	144.48	144.48
AC-FT		94.	125.	128.	128.
THOUS CU M		116.	155.	158.	158.

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COMBINE HYDROGRAPHS

3 COMBINE HYDROGRAPHS OF SUB-BASINS 1 AND 2

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
10	2	0	0	0	0	1	0	0

SUM OF 2 HYDROGRAPHS AT 10 PLAN 1 RTIO 1

1.	1.	1.	1.	1.	1.	1.	1.	1.	1.

Sheet 17 of 34

1.	1.	1.	2.	2.	2.	3.	3.	3.	3.
3.	3.	3.	3.	4.	7.	10.	13.	15.	17.
20.	30.	45.	51.	47.	42.	35.	26.	18.	12.
9.	7.	6.	5.	5.	4.	4.	4.	4.	8.
12.	15.	17.	18.	19.	20.	20.	21.	21.	21.
29.	50.	72.	87.	97.	104.	109.	111.	113.	115.
116.	116.	180.	348.	540.	701.	846.	989.	1133.	1578.
2160.	2200.	1880.	1605.	1323.	983.	677.	[illegible]	[illegible]	[illegible]

..	1.	1.	1.	1.	1.	1.	1.

1.	1.	1.	2.	2.	2.	3.	3.	3.	3.
3.	3.	3.	3.	4.	7.	10.	13.	15.	17.
20.	30.	45.	51.	47.	42.	35.	26.	18.	12.
9.	7.	5.	5.	5.	4.	4.	4.	4.	8.
12.	15.	17.	18.	19.	20.	20.	21.	21.	21.
29.	50.	72.	87.	97.	104.	108.	111.	113.	115.
116.	116.	180.	348.	540.	701.	846.	989.	1133.	1578.
2160.	2200.	1880.	1605.	1323.	983.	677.	470.	331.	264.
217.	208.	199.	192.	184.	177.	170.	163.	156.	150.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2200.	1334.	442.	217.	21742.
CMS	62.	38.	13.	6.	616.
INCHES		17.04	22.61	23.15	23.15
MM		432.94	574.25	588.04	588.04
AC-FT		661.	877.	898.	898.
THOUS CU M		816.	1082.	1108.	1108.

SUM OF 2 HYDROGRAPHS AT 10 PLAN 1 RTIO 2

1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	2.	2.	2.	2.	2.	2.
2.	2.	2.	2.	3.	5.	8.	10.	11.	13.
15.	22.	34.	35.	35.	31.	26.	19.	13.	9.
7.	5.	4.	4.	3.	3.	3.	3.	3.	6.
9.	11.	13.	14.	15.	15.	15.	15.	16.	16.
22.	37.	54.	65.	73.	78.	81.	83.	85.	86.
87.	87.	135.	261.	405.	526.	635.	742.	849.	1184.
1620.	1650.	1410.	1204.	992.	737.	508.	353.	248.	198.
162.	156.	150.	144.	138.	132.	127.	122.	117.	113.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1650.	1000.	332.	163.	16306.
CMS	47.	28.	9.	5.	462.
INCHES		12.78	16.96	17.36	17.36
MM		324.71	430.68	441.03	441.03
AC-FT		496.	658.	674.	674.
THOUS CU M		612.	812.	831.	831.

SUM OF 2 HYDROGRAPHS AT 10 PLAN 1 RTIO 3

0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	2.	3.	5.	6.	8.	9.
10.	15.	23.	25.	24.	21.	17.	13.	9.	6.
4.	3.	3.	2.	2.	2.	2.	2.	2.	4.
6.	8.	9.	9.	10.	10.	10.	10.	10.	10.
14.	25.	36.	43.	48.	52.	54.	56.	57.	57.
58.	58.	90.	174.	270.	350.	423.	495.	566.	789.
1080.	1100.	940.	803.	661.	491.	339.	235.	166.	132.
108.	104.	100.	96.	92.	88.	85.	81.	78.	75.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1100.	667.	221.	109.	10871.
CMS	31.	19.	6.	3.	308.
INCHES		8.52	11.30	11.58	11.58
MM		216.47	287.12	294.02	294.02
AC-FT		331.	439.	449.	449.
THOUS CU M		408.	541.	554.	554.

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SUM OF 2 HYDROGRAPHS AT 10 PLAN 1 [illegible]

0.	0.

	8.52	11.30	11.58	11.58
MM	216.47	287.12	294.02	294.02
AC-FT	331.	439.	449.	449.
THOUS CU M	408.	541.	554.	554.

SUM OF 2 HYDROGRAPHS AT 10 PLAN 1 RTIO 4

0.	0.	3.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	2.	3.	3.	4.	4.
5.	7.	11.	13.	12.	10.	9.	6.	4.	3.
2.	2.	1.	1.	1.	1.	1.	1.	1.	2.
3.	4.	4.	5.	5.	5.	5.	5.	5.	5.
7.	12.	1^.	22.	24.	26.	27.	28.	28.	29.
29.	29.	45.	87.	135.	175.	212.	247.	283.	39[illegible].
540.	550.	470.	401.	331.	246.	169.	118.	83.	[illegible].
54.	52.	50.	48.	46.	44.	42.	41.	39.	[illegible]8.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	550.	333.	111.	54.	5435.
CMS	16.	9.	3.	2.	154.
INCHES		4.26	5.65	5.79	5.79
MM		108.24	143.56	147.01	147.01
AC-FT		165.	219.	225.	225.
THOUS CU M		204.	271.	277.	277.

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SUB-AREA RUNOFF COMPUTATION

4 SUB-BASIN 3 INFLOW HYDROGRAPH

ISTAQ	ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
3	0	0	0	0	0	1	0	0

HYDROGRAPH DATA

IHYDG	IUHS	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	.45	0.00	.13	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	22.00	112.00	123.00	133.00	141.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS .800

LOSS DATA

LROPT	STRKR	DLTKR	RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
0	0.20	0.00	1.00	0.00	0.00	1.00	1.00	.10	0.00	.11

UNIT HYDROGRAPH DATA

TP= .85 CP= .50 NTA= 0

RECESSION DATA

STRTQ= -1.00 QRCSN= -.10 RTIOR= 1.50

APPROXIMATE CLARK COEFFICIENTS FROM GIVEN SNYDER CP AND TP ARE TC= 1.67 AND R= 2.11 INTERVALS

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UNIT HYDROGRAPH 12 END-OF-PERIOD ORDINATES, LAG= .86 HOURS, CP= .50 VOL= 1.00

71.	154.	135.	83.	51.	32.	20.	12.	7.	5.
3.	2.								

0

END-OF-PERIOD FLOW

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.00	.00	.00	0.	1.02	1.30	51	.04	.04	.04	[illegible]

... ICE 1.67 AND R= 2.11 INTERVALS

UNIT HYDROGRAPH 12 END-OF-PERIOD ORDINATES, LAG= .86 HOURS, CP= .50 VOL= 1.00

71.	154.	135.	83.	51.	32.	20.	12.	7.	5.
3.	2.								

0

MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	END-OF-PERIOD FLOW COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
1.01	.30	1	.00	.00	.00	0.	1.02	1.30	51	.06	.01	.04	5.
1.01	1.00	2	.00	.00	.00	0.	1.02	2.00	52	.06	.01	.04	6.
1.01	1.30	3	.00	.00	.00	1.	1.02	2.30	53	.05	.01	.04	7.
1.01	2.00	4	.00	.00	.00	1.	1.02	3.00	54	.06	.01	.04	7.
1.01	2.30	5	.00	.00	.00	1.	1.02	3.30	55	.06	.01	.04	8.
1.01	3.00	6	.00	.00	.00	1.	1.02	4.00	56	.05	.01	.04	8.
1.01	3.30	7	.00	.00	.00	1.	1.02	4.30	57	.06	.01	.04	8.
1.01	4.00	8	.00	.00	.00	1.	1.02	5.00	58	.06	.01	.04	8.
1.01	4.30	9	.00	.00	.00	1.	1.02	5.30	59	.06	.01	.04	8.
1.01	5.00	10	.00	.00	.00	1.	1.02	6.00	60	.06	.01	.04	8.
1.01	5.30	11	.00	.00	.00	1.	1.02	6.30	61	.16	.12	.04	15.
1.01	6.00	12	.00	.00	.00	0.	1.02	7.00	62	.16	.12	.04	31.
1.01	6.30	13	.01	.00	.01	1.	1.02	7.30	63	.16	.12	.04	45.
1.01	7.00	14	.01	.00	.01	1.	1.02	8.00	64	.16	.12	.04	53.
1.01	7.30	15	.01	.00	.01	1.	1.02	8.30	65	.16	.12	.04	59.
1.01	8.00	16	.01	.00	.01	1.	1.02	9.00	66	.16	.12	.04	62.
1.01	8.30	17	.01	.00	.01	1.	1.02	9.30	67	.16	.12	.04	64.
1.01	9.00	18	.01	.00	.01	1.	1.02	10.00	68	.16	.12	.04	65.
1.01	9.30	19	.01	.00	.01	1.	1.02	10.30	69	.16	.12	.04	66.
1.01	10.00	20	.01	.00	.01	1.	1.02	11.00	70	.16	.12	.04	66.
1.01	10.30	21	.01	.00	.01	1.	1.02	11.30	71	.16	.12	.04	67.
1.01	11.00	22	.01	.00	.01	1.	1.02	12.00	72	.16	.12	.04	67.
1.01	11.30	23	.01	.00	.01	1.	1.02	12.30	73	.99	.94	.04	125.
1.01	12.00	24	.01	.00	.01	1.	1.02	13.00	74	.99	.94	.04	252.
1.01	12.30	25	.06	.01	.05	1.	1.02	13.30	75	1.18	1.14	.04	377.
1.01	13.00	26	.06	.01	.05	2.	1.02	14.00	76	1.15	1.14	.04	476.
1.01	13.30	27	.07	.01	.06	3.	1.02	14.30	77	1.48	1.43	.04	566.
1.01	14.00	28	.07	.01	.06	3.	1.02	15.00	78	1.48	1.43	.04	654.
1.01	14.30	29	.09	.01	.08	4.	1.02	15.30	79	1.80	1.75	.04	743.
1.01	15.00	30	.09	.01	.08	4.	1.02	16.00	80	5.69	5.65	.04	1109.
1.01	15.30	31	.11	.01	.10	5.	1.02	16.30	81	1.38	1.34	.04	1471.
1.01	16.00	32	.34	.07	.27	10.	1.02	17.00	82	1.38	1.34	.04	1373.
1.01	16.30	33	.08	.04	.04	17.	1.02	17.30	83	1.38	1.34	.04	1121.
1.01	17.00	34	.08	.04	.04	21.	1.02	18.00	84	1.38	1.34	.04	933.
1.01	17.30	35	.07	.02	.04	20.	1.02	18.30	85	.09	.04	.04	734.
1.01	18.00	36	.07	.02	.04	13.	1.02	19.00	86	.09	.04	.04	501.
1.01	18.30	37	.01	.00	.00	14.	1.02	19.30	87	.09	.04	.04	317.
1.01	19.00	38	.01	.00	.00	10.	1.02	20.00	88	.09	.04	.04	204.
1.01	19.30	39	.01	.00	.00	6.	1.02	20.30	89	.09	.04	.04	144.
1.01	20.00	40	.01	.00	.00	4.	1.02	21.00	90	.09	.04	.04	140.
1.01	20.30	41	.01	.00	.00	3.	1.02	21.30	91	.09	.04	.04	135.
1.01	21.00	42	.01	.00	.00	2.	1.02	22.00	92	.09	.04	.04	129.
1.01	21.30	43	.01	.00	.00	2.	1.02	22.30	93	.09	.04	.04	124.
1.01	22.00	44	.01	.00	.00	2.	1.02	23.00	94	.09	.04	.04	119.
1.01	22.30	45	.01	.00	.00	2.	1.02	23.30	95	.09	.04	.04	115.
1.01	23.00	46	.01	.00	.00	2.	1.03	0.00	96	.09	.04	.04	110.
1.01	23.30	47	.01	.00	.00	2.	1.03	.30	97	0.00	0.00	0.00	105.
1.02	0.00	48	.01	.00	.00	2.	1.03	1.00	98	0.00	0.00	0.00	101.
1.02	.30	49	.06	.01	.04	2.	1.03	1.30	99	0.00	0.00	0.00	97.
1.02	1.00	50	.06	.01	.04	3.	1.03	2.00	100	0.00	0.00	0.00	93.
									SUM	24.62	21.53	3.29	13288.
										(630.)	(547.)	(83.)	(376.27)

Sheet 20 of 34

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1471.	836.	272.	132.	13235.
CMS	42.	24.	8.	4.	375.
INCHES		17.39	22.[illegible]	22.95	22.95
MM		441.66	574.88	582.[illegible]	[illegible]
AC-FT		[illegible]	[illegible]	[illegible]	[illegible]

(630.)(547.)(83.)(374 27)

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1471.	836.	272.	132.	13235.
CMS	42.	24.	8.	4.	375.
INCHES		17.39	22.63	22.95	22.95
MM		441.66	574.88	582.98	582.98
AC-FT		414.	539.	547.	547.
THOUS CU M		511.	665.	675.	675.

HYDROGRAPH AT STA 3 FOR PLAN 1, RTIO 1

0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	2.	3.	3.	4.	4.
5.	10.	17.	21.	20.	18.	14.	10.	6.	4.
3.	2.	2.	2.	2.	2.	2.	2.	2.	3.
5.	6.	7.	7.	8.	8.	8.	8.	8.	8.
15.	31.	45.	53.	59.	62.	64.	65.	66.	66.
67.	67.	125.	252.	377.	476.	566.	654.	743.	1109.
1471.	1373.	1121.	933.	734.	501.	317.	204.	146.	140.
135.	129.	124.	119.	115.	110.	106.	101.	97.	93.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1471.	836.	272.	132.	13235.
CMS	42.	24.	8.	4.	375.
INCHES		17.39	22.63	22.95	22.95
MM		441.66	574.88	582.98	582.98
AC-FT		414.	539.	547.	547.
THOUS CU M		511.	665.	675.	675.

HYDROGRAPH AT STA 3 FOR PLAN 1, RTIO 2

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	2.	2.	3.	3.
4.	7.	13.	15.	15.	13.	10.	7.	5.	3.
2.	2.	1.	1.	1.	1.	1.	1.	1.	3.
4.	5.	5.	6.	6.	6.	6.	6.	6.	6.
11.	23.	34.	40.	44.	46.	43.	49.	49.	50.
50.	50.	94.	189.	263.	357.	424.	490.	557.	832.
1104.	1030.	841.	700.	550.	375.	233.	153.	110.	105.
101.	97.	93.	89.	86.	82.	79.	76.	73.	70.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1104.	627.	204.	99.	9926.
CMS	31.	18.	6.	3.	281.
INCHES		13.04	16.97	17.21	17.21
MM		331.25	431.16	437.24	437.24
AC-FT		311.	404.	410.	410.
THOUS CU M		383.	499.	506.	506.

HYDROGRAPH AT STA 3 FOR PLAN 1, RTIO 3

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	2.	2.	2.
2.	5.	9.	10.	10.	9.	7.	5.	3.	2.

Sheet 2 of 34

1.	1.	1.	1.	1.	1.	1.	1.	1.	2.
3.	3.	3.	4.	4.	4.	4.	4.	4.	4.
8.	16.	22.	27.	29.	31.	32.	33.	33.	33.
33.	33.	63.	126.	189.	238.	283.	327.	371.	554.
736.	646.	560.	466.	367.	250.	159.	102.	73.	70.
67.	65.	62.	60.	57.	55.	53.	51.	49.	47.

2.	5.	9.	11.	[illegible]	1.	1.	2.	2. 3.	2. 2.

1.	1.	1.	1.	1.	1.	1.	1.	1.	2.
3.	3.	3.	4.	4.	4.	4.	4.	4.	4.
8.	16.	22.	27.	29.	31.	32.	33.	33.	33.
33.	33.	63.	126.	189.	238.	283.	327.	371.	554.
736.	686.	560.	466.	367.	250.	159.	102.	73.	70.
67.	65.	62.	60.	57.	55.	53.	51.	49.	47.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	736.	418.	136.	66.	6617.
CMS	21.	12.	4.	2.	187.
INCHES		8.59	11.32	11.48	11.48
MM		220.83	287.44	291.49	291.49
AC-FT		207.	270.	273.	273.
THOUS CU M		256.	333.	337.	337.

HYDROGRAPH AT STA 3 FOR PLAN 1, RTIO 4

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	1.	1.	1.	1.
1.	2.	4.	5.	5.	4.	3.	2.	2.	1.
1.	1.	0.	0.	0.	0.	0.	0.	0.	1.
1.	2.	2.	2.	2.	2.	2.	2.	2.	2.
4.	8.	11.	13.	15.	15.	16.	16.	16.	17.
17.	17.	31.	67.	94.	119.	141.	163.	186.	277.
368.	343.	290.	233.	133.	125.	79.	51.	37.	35.
34.	32.	31.	30.	29.	27.	26.	25.	24.	23.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	368.	209.	66.	33.	3309.
CMS	10.	6.	2.	1.	94.
INCHES		4.35	5.66	5.74	5.74
MM		110.42	143.72	145.75	145.75
AC-FT		104.	135.	137.	137.
THOUS CU M		125.	166.	169.	169.

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

COMBINE HYDROGRAPHS

5 COMBINE HYDROGRAPHS OF SUB-BASIN 3 AND THE REST

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

SUM OF 2 HYDROGRAPHS AT 11 PLAN 1 RTIO 1

1.	1.	2.	2.	2.	2.	2.	2.	2.	2.
2.	2.	2.	2.	3.	3.	3.	3.	3.	4.
4.	4.	4.	4.	5.	9.	13.	15.	19.	22.
25.	40.	62.	71.	67.	59.	47.	35.	24.	16.
11.	9.	7.	7.	6.	6.	6.	6.	5.	11.
17.	21.	24.	26.	27.	28.	29.	29.	29.	29.
44.	81.	117.	140.	156.	166.	172.	176.	179.	181.
182.	183.	306.	600.	917.	1177.	1412.	1643.	1875.	2687.

Sheet 22 of 34

3631.	3573.	3001.	2538.	2056.	1483.	995.	674.	477.	434.
351.	337.	324.	311.	298.	287.	275.	264.	254.	244.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3631.	2169.	714.	360.	34276.
CMS	103.	61.	[illegible]	10.	[illegible]
INCHES		17.10	22.72	23.08	23.08
MM		434.26	576.49	586.33	586.33

		[illegible]	600.	917.	[illegible]	177.	176.	179.	191
					1177.	1712.	343.	875.	2687.
3631.	3573.	3001.	2538.	2056.	1483.	995.	674.	477.	434.
351.	337.	324.	311.	298.	287.	275.	264.	254.	244.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	3631.	2169.	714.	350.	34976.
CMS	103.	61.	20.	10.	990.
INCHES		17.18	22.62	23.08	23.08
MM		436.26	574.49	586.11	586.11
AC-FT		1076.	1417.	1445.	1445.
THOUS CU M		1327.	1747.	1783.	1783.

SUM OF 2 HYDROGRAPHS AT 11 PLAN 1 RTIO 2

1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	2.	2.	2.	2.	2.	3.	3.	3.
3.	3.	3.	3.	4.	7.	10.	12.	14.	16.
19.	30.	47.	53.	50.	45.	37.	27.	18.	12.
8.	7.	6.	5.	5.	5.	4.	4.	4.	8.
13.	15.	18.	19.	20.	21.	21.	21.	22.	22.
33.	61.	88.	105.	117.	124.	129.	132.	134.	136.
137.	137.	229.	450.	688.	883.	1059.	1232.	1406.	2015.
2723.	2660.	2251.	1903.	1542.	1112.	746.	506.	355.	303.
263.	253.	243.	233.	224.	215.	206.	198.	190.	183.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	2723.	1627.	536.	262.	26232.
CMS	77.	46.	15.	7.	743.
INCHES		12.88	16.96	17.31	17.31
MM		327.19	430.87	439.58	439.58
AC-FT		807.	1062.	1084.	1084.
THOUS CU M		995.	1311.	1337.	1337.

SUM OF 2 HYDROGRAPHS AT 11 PLAN 1 RTIO 3

1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	2.	2.	2.	2.	2.
2.	2.	2.	2.	3.	4.	6.	8.	9.	11.
12.	20.	31.	36.	33.	30.	24.	18.	12.	8.
6.	4.	4.	3.	3.	3.	3.	3.	3.	5.
9.	11.	12.	13.	14.	14.	14.	14.	14.	14.
22.	40.	58.	70.	78.	83.	86.	88.	90.	91.
21.	92.	153.	300.	459.	589.	706.	822.	938.	1344.
1816.	1787.	1501.	1269.	1028.	742.	497.	337.	239.	202.
176.	169.	162.	155.	149.	143.	137.	132.	127.	122.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1816.	1085.	357.	175.	17488.
CMS	51.	31.	10.	5.	495.
INCHES		8.59	11.31	11.54	11.54
MM		218.13	287.24	293.06	293.06
AC-FT		538.	708.	723.	723.
THOUS CU M		663.	874.	891.	891.

Sheet 23 of 34

SUM OF 2 HYDROGRAPHS AT 11 PLAN 1 RTIO 4

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	2.	3.	4.	5.	5.
6.	10.	16.	18.	17.	15.	12.	9.	6.	4.
3.	2.	2.	2.	2.	2.	1.	1.	1.	3.
4.	5.	5.	6.	7.	7.	7.	7.	7.	7.
11.	20.	29.	35.	39.	41.	43.	44.	[illegible]	[illegible]

```
                    SUM OF 2 HYDROGRAPHS AT     11  PLAN 1  RTIO 4
      0.      0.      0.      0.      0.      0.      0.      0.      0.      0.
      0.      0.      1.      1.      1.      1.      1.      1.      1.      1.
      1.      1.      1.      1.      1.      2.      3.      4.      5.      5.
      5.     10.     16.     18.     17.     15.     12.      9.      6.      4.
      3.      2.      2.      2.      2.      2.      1.      1.      1.      3.
      4.      5.      6.      6.      7.      7.      7.      7.      7.      7.
     11.     20.     29.     35.     39.     41.     43.     44.     45.     45.
     46.     46.     76.    150.    229.    294.    353.    411.    469.    672.
    908.    893.    750.    634.    514.    371.    249.    169.    119.    101.
     88.     84.     81.     78.     75.     72.     69.     66.     63.     61.

                     PEAK   6-HOUR   24-HOUR   72-HOUR   TOTAL VOLUME
            CFS      908.     542.      179.       87.        8744.
            CMS       26.      15.        5.        2.         248.
         INCHES               4.29      5.65      5.77         5.77
             MM             109.06    143.62    146.53       146.53
          AC-FT               269.      354.      361.         361.
     THOUS CU M               332.      437.      446.         446.


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

                              HYDROGRAPH ROUTING

        6 ROUTE THROUGH LAKE

                 ISTAQ   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                     3       1       0       0       0       0       1       0       0
                                         ROUTING DATA
         QLOSS   CLOSS     AVG    IRES   ISAME    IOPT    IPMP            LSTR
           0.0   0.000    0.00       1       1       0       0               0

                 NSTPS   NSTDL     LAG   AMSKK       X     TSK   STORA  ISPRAT
                     1       0       0   0.000   0.000   0.000   2909.       0

    CAPACITY=    2909.     3336.     3690.     4059.     4443.

   ELEVATION=     451.      454.      456.      458.      460.

                 CREL   SPWID    COQW    EXPW   ELEVL    COQL   CAREA    EXPL
                451.5    10.0     3.3     1.5     0.0     0.0     0.0     0.0

                                              DAM DATA
                                   TOPEL    COQD    EXPD  DAMWID
                                   455.6     3.1     1.5    185.

                            STATION      3, PLAN 1, RATIO 1

                           END-OF-PERIOD HYDROGRAPH ORDINATES

                                           OUTFLOW
      0.      0.      0.      0.      0.      0.      0.      0.      0.      0.
      0.      0.      0.      0.      0.      0.      0.      0.      0.      0.
      0.      0.      0.      0.      0.      0.      0.      0.      0.      0.
      0.      0.      0.      1.      1.      1.      1.      0.      0.      0.
      2.      2.      2.      2.      2.      2.      2.      2.      2.      2.


      2.      2.      2.      2.      2.      3.      3.      3.      3.      3.
      3.      4.      4.      5.      6.      7.      8.     10.     11.     12.
     14.     15.     18.     22.     30.     43.     60.     83.    112.    153.
    214.    328.    758.   1150.   1401.   1486.   1429.   1296.   1141.    997.
    877.    778.    700.    635.    583.    538.    500.    467.    438.    413.

                                           STORAGE
   2909.   2909.   2909.   2909.   2909.   2909.   2909.   2910.   2910.   2910.
```

Sheet 24 of 34

	0.	0.	1.	1.	[illegible]	0.	0.	0.	0.
						2.	2.	2.	2.
2.	2.	2.				2.	2.	2.	2.

2.	2.	2.	2.	2.	3.	3.	3.	3.	3.
3.	4.	4.	5.	6.	7.	8.	10.	11.	12.
14.	15.	18.	22.	30.	43.	60.	83.	112.	153.
214.	328.	758.	1150.	1401.	1486.	1429.	1296.	1141.	997.
877.	778.	700.	635.	583.	538.	500.	467.	438.	413.

STORAGE

2909.	2909.	2909.	2909.	2909.	2909.	2909.	2910.	2910.	2910.
2910.	2910.	2910.	2910.	2910.	2910.	2910.	2911.	2911.	2911.
2911.	2911.	2911.	2911.	2912.	2912.	2912.	2913.	2914.	2914.
2915.	2917.	2919.	2922.	2924.	2927.	2929.	2931.	2932.	2933.
2933.	2934.	2934.	2934.	2934.	2934.	2935.	2935.	2935.	2935.
2936.	2936.	2937.	2938.	2939.	2940.	2941.	2942.	2943.	2944.
2946.	2948.	2952.	2957.	2963.	2969.	2976.	2983.	2990.	2997.
3004.	3011.	3023.	3039.	3068.	3110.	3161.	3221.	3290.	3379.
3592.	3640.	3753.	3828.	3870.	3884.	3875.	3853.	3826.	3800.
3777.	3757.	3740.	3726.	3713.	3702.	3692.	3684.	3676.	3668.

STAGE

451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.6	451.6	451.6	451.6	451.6	451.6
451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6
451.6	451.6	451.6	451.6	451.6	451.7	451.7	451.7	451.7	451.7
451.7	451.7	451.7	451.8	451.8	451.8	451.9	451.9	451.9	452.0
452.0	452.1	452.1	452.2	452.4	452.7	453.0	453.3	453.7	454.2
454.9	455.7	456.3	456.7	457.0	457.0	457.0	456.9	456.7	456.6
456.5	456.4	456.3	456.2	456.1	456.1	456.0	456.0	455.9	455.9

PEAK OUTFLOW IS 1486. AT TIME 43.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1486.	1047.	345.	166.	16601.
CMS	42.	30.	10.	5.	470.
INCHES		8.29	10.93	10.95	10.95
MM		210.49	277.53	278.19	278.19
AC-FT		519.	685.	686.	686.
THOUS CU M		640.	844.	846.	846.

STATION 3, PLAN 1, RATIO 2

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	2.	2.	2.	2.	2.	2.	2.
2.	2.	3.	3.	4.	5.	5.	6.	7.	8.
9.	10.	12.	15.	20.	28.	40.	55.	74.	101.
142.	193.	242.	302.	469.	597.	651.	647.	612.	568.
525.	486.	452.	423.	398.	376.	357.	340.	325.	312.

STORAGE

Sheet 25 of 34

2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2910.
2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.
2910.	2911.	2911.	2911.	2911.	2911.	2911.	2912.	2912.	2913.
2914.	2915.	2916.	2918.	2921.	2922.	2924.	2925.	2926.	2927.
2927.	2927.	2928.	2928.	2928.	2928.	2928.	2928.	2928.	2929.
2929.	2930.	2930.	2931.	2932.	2932.	2933.	2934.	2935.	2936.
2937.	2939.	2942.	2945.	2950.	2955.	[illegible]	[illegible]	[illegible]	[illegible]

[illegible] [illegible] 423. 398. 376. 357. 340. 325. 312.

AGE

2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2910.
2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.
2910.	2911.	2911.	2911.	2911.	2911.	2911.	2912.	2912.	2913.
2914.	2915.	2916.	2918.	2921.	2922.	2924.	2925.	2926.	2927.
2927.	2927.	2928.	2928.	2928.	2928.	2928.	2928.	2928.	2929.
2929.	2930.	2930.	2931.	2932.	2932.	2933.	2934.	2935.	2936.
2937.	2939.	2942.	2945.	2950.	2955.	2960.	2965.	2970.	2975.
2961.	2985.	2993.	3006.	3029.	3061.	3099.	3145.	3197.	3254.
3357.	3461.	3554.	3629.	3684.	3717.	3729.	3728.	3720.	3710.
3699.	3689.	3679.	3671.	3664.	3657.	3650.	3644.	3639.	3633.

STAGE

451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.6	451.6	451.6	451.6
451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6
451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6
451.6	451.6	451.7	451.7	451.7	451.7	451.8	451.8	451.8	451.9
451.9	451.9	452.0	452.0	452.2	452.4	452.6	452.9	453.2	453.6
454.1	454.7	455.2	455.7	456.0	456.1	456.2	456.2	456.2	456.1
456.0	456.0	455.9	455.9	455.9	455.8	455.8	455.7	455.7	455.7

PEAK OUTFLOW IS 651. AT TIME 43.50 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	651.	514.	181.	87.	8709.
CMS	18.	15.	5.	2.	247.
INCHES		4.07	5.73	5.74	5.74
MM		103.35	145.55	145.92	145.92
AC-FT		255.	359.	360.	360.
THOUS CU M		314.	443.	444.	444.

STATION 3, PLAN 1, RATIO 3

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	2.	2.	2.	3.	3.	3.	4.	4.
5.	6.	6.	8.	11.	16.	22.	30.	41.	56.
79.	108.	136.	160.	180.	194.	203.	207.	209.	209.
209.	208.	207.	206.	205.	204.	202.	201.	200.	198.

STORAGE

2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.
2909.	2909.	2909.	2910.	2910.	2910.	2910.	2910.	2910.	2910.
2910.	2910.	2910.	2910.	2910.	2910.	2911.	2911.	2911.	2912.
2912.	2913.	2914.	2915.	2917.	2918.	2919.	2920.	2921.	2921.
2921.	2921.	2921.	2922.	2922.	2922.	2922.	2922.	2922.	2922.
2922.	2923.	2923.	2924.	2924.	2925.	2925.	2926.	2926.	2927.
2928.	2929.	2931.	2933.	2936.	2940.	2943.	2947.	2950.	2954.

Sheet 26 of 34

2957.	2961.	2965.	2975.	2990.	3011.	3037.	3067.	3102.	3147.
3210.	3280.	3343.	3394.	3435.	3464.	3481.	3490.	3493.	3493.
3493.	3491.	3489.	3487.	3485.	3483.	3480.	3477.	3475.	3471.

STAGE

451.5	451.5	451.5	451.[illegible]	451.5	451.5	451.5	451.5	451.5	451.5
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

2922.	2923.	2923.	[illegible]	[illegible]	[illegible]	[illegible]	2922.	2922.	2922.
2928.	2929	2931	2977	[illegible]	2925.	2925.	2926.	2926.	2927.
							7.	50.	954.

2957.	2961.	2965.	2975.	2990.	3011.	3037.	3067.	3102.	3147.
3210.	3280.	3343.	3394.	3435.	3464.	3481.	3490.	3493.	3493.
3493.	3491.	3489.	3487.	3485.	3483.	3480.	3477.	3475.	3471.

STAGE

451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6	451.6
451.6	451.6	451.6	451.6	451.6	451.7	451.7	451.7	451.7	451.7
451.8	451.8	451.8	451.9	451.9	452.1	452.2	452.4	452.6	452.9
453.3	453.7	454.0	454.3	454.6	454.7	454.8	454.9	454.9	454.9
454.9	454.9	454.9	454.9	454.9	454.8	454.8	454.8	454.8	454.8

PEAK OUTFLOW IS 209. AT TIME 45.00 HOURS

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	209.	206.	80.	39.	3872.
CMS	6.	6.	2.	1.	110.
INCHES		1.63	2.55	2.55	2.55
MM		41.39	64.69	64.89	64.89
AC-FT		102.	160.	160.	160.
THOUS CU M		126.	197.	197.	197.

STATION 3, PLAN 1, RATIO 4

END-OF-PERIOD HYDROGRAPH ORDINATES

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	2.
2.	2.	2.	3.	4.	6.	8.	11.	15.	20.
28.	39.	49.	59.	66.	72.	76.	78.	79.	79.
80.	80.	80.	80.	80.	79.	79.	79.	79.	79.

STORAGE

2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.
2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.	2909.
2909.	2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.	2910.
2911.	2911.	2911.	2912.	2913.	2913.	2914.	2914.	2915.	2915.
2915.	2915.	2915.	2915.	2915.	2915.	2915.	2916.	2916.	2916.
2916.	2916.	2916.	2916.	2917.	2917.	2917.	2918.	2918.	2918.
2918.	2919.	2920.	2921.	2923.	2925.	2926.	2928.	2930.	2932.
2933.	2935.	2939.	2942.	2950.	2960.	2974.	2989.	3007.	3029.
3061.	3097.	3129.	3155.	3176.	3192.	3202.	3207.	3210.	3211.
3212.	3212.	3212.	3212.	3212.	3212.	3211.	3211.	3210.	3210.

STAGE

451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5

Sheet 27 of 34

451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5	451.5
451.5	451.5	451.5	451.5	451.5	451.6	451.6	451.6	451.6	451.6
451.6	451.6	451.6	451.7	451.7	451.8	451.8	451.9	452.0	452.2

```
 451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5
 451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5
 451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5
 451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5    451.5
 451.5    451.5    451.5    451.5    451.6    451.6    451.6    451.6    451.6    451.6
 451.6    451.6    451.6    451.7    451.7    451.8    451.9    451.9    452.0    452.2
 452.4    452.6    452.8    452.9    453.1    453.1    453.2    453.2    453.3    453.3
 453.3    453.3    453.3    453.3    453.3    453.3    453.3    453.3    453.3    453.3

PEAK OUTFLOW IS       80. AT TIME  46.50 HOURS

                    PEAK   6-HOUR  24-HOUR  72-HOUR  TOTAL VOLUME
           CFS       80.     79.      30.      15.      1466.
           CMS        2.      2.       1.       0.        42.
        INCHES               .63      .96      .97       .97
            MM             15.94    24.50    24.57     24.57
         AC-FT               39.      60.      61.       61.
     THOUS CU M              48.      75.      75.       75.

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

                              HYDROGRAPH ROUTING

            6 CHANNEL ROUTE D/S OF DAM

                ISTAQ  ICOMP  IECON  ITAPE   JPLT   JPRT  INAME ISTAGE  IAUTO
                    4      1      0      0      0      0      1      0      0
                                    ROUTING DATA
         QLOSS  CLOSS    AVG   IRES  ISAME   IOPT   IPMP          LSTR
           0.0  0.000   0.00      1      1      0      0             0

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

  NORMAL DEPTH CHANNEL ROUTING
  ----------------------------

          QN(1)   QN(2)   QN(3)   ELNVT   ELMAX   RLNTH    SEL
          .0350   .0170   .0350   406.0   430.0    900.  .04400

          CROSS SECTION COORDINATES--STA,ELEV,STA,ELEV--ETC
             0.00  430.00   38.00  420.00  125.00  410.00  125.10  406.00  132.90  406.00
           133.00  410.00  250.00  420.00  375.00  430.00

  STORAGE ..     0.00        .20        .41        .62       1.06       2.17       3.94       6.39       9.52      13.31
                17.79      22.92      28.68      34.97      41.80      49.17      57.08      65.52      74.50      84.02

  OUTFLOW        0.00     176.74     488.17     857.03    1432.66    2526.43    4416.06    7337.03   11497.55   17088.80
             24288.67   33265.73   44557.92   57883.91   73300.27   90901.87  110783.99  133041.65  157769.20  185060.12

  STAGE        406.00     407.26     408.53     409.79     411.05     412.32     413.58     414.84     416.11     417.37
               418.63     419.89     421.16     422.42     423.68     424.95     426.21     427.47     428.74     430.00

  FLOW           0.00     176.74     488.17     857.03    1432.66    2526.43    4416.06    7337.03   11497.65   17088.80
             24288.67   33265.73   44557.92   57883.91   73300.27   90901.87  110783.99  133041.65  157769.20  185060.12

                              STATION      4, PLAN 1, RTIO 1

                                          OUTFLOW
```

Sheet 28 of 34

						110783.99	133041.65	157769.20	185060.12

STAGE	406.00	407.26	408.53	409.79	411.05	412.32	413.58	414.84	416.11	417.37
	418.63	419.89	421.16	422.42	423.68	424.95	426.21	427.47	428.74	430.00
FLOW	0.00	176.74	488.17	857.03	1432.66	2526.43	4416.06	7337.03	11497.65	17088.80
	24288.67	33265.73	44557.92	57883.91	73300.27	90901.87	110783.99	133041.65	157769.20	185060.12

STATION 4, PLAN 1, RTIO 1

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	[illegible]
0.	0.	0.	1.	1.	1.	1.	2.	2.	2.
2.	2.	2.	2.	2.	2.	2.	2.	2.	2.
2.	2.	2.	2.	2.	3.	3.	3.	3.	3.
3.	4.	4.	5.	6.	7.	8.	10.	11.	12.
14.	15.	18.	22.	30.	43.	60.	83.	111.	151.
212.	325.	748.	1147.	1396.	1487.	1430.	1299.	1143.	1030.
278.	780.	700.	636.	583.	539.	501.	468.	438.	414.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	0.	0.	0.	0.	0.	0.	0.

STAGE

406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.1	406.1	406.1	406.1	406.1
406.1	406.1	406.1	406.2	406.2	406.3	406.4	406.6	406.8	407.1
407.4	407.9	409.4	410.4	411.0	411.1	411.0	410.8	410.4	410.1
409.8	409.5	409.3	409.0	408.9	408.7	408.6	408.4	408.3	408.2

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	1487.	1047.	345.	166.	16592.
CMS	42.	30.	10.	5.	470.
INCHES		8.29	10.92	10.95	10.95
MM		210.52	277.48	278.04	278.04
AC-FT		519.	684.	686.	686.
THOUS CU M		640.	844.	846.	846.

MAXIMUM STORAGE = 1.

MAXIMUM STAGE IS 411.1

STATION 4, PLAN 1, RTIO 2

Sheet 29 of 34

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
		1.							

S N , PL , RTI C

OUTFLOW									
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	2.	2.	2.	2.	2.	2.	2.
2.	2.	3.	3.	4.	5.	[illegible]	6.	7.	8.
9.	10.	12.	14.	20.	28.	39.	54.	73.	100.
141.	192.	241.	301.	465.	597.	650.	649.	611.	569.
524.	488.	452.	424.	398.	377.	357.	341.	325.	313.
STOR									
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	1.	1.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
STAGE									
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.1	406.1
406.1	406.1	406.1	406.1	406.1	406.2	406.3	406.4	406.5	406.7
407.0	407.3	407.5	407.8	408.4	408.9	409.1	409.1	408.9	408.8
408.7	408.5	408.4	408.3	408.2	408.1	408.0	407.9	407.9	407.8

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	650.	514.	181.	87.	8701.
CMS	18.	15.	5.	2.	246.
INCHES		4.07	5.73	5.74	5.74
MM		103.33	143.44	145.80	145.80
AC-FT		255.	359.	360.	360.
THOUS CU M		314.	442.	443.	443.

MAXIMUM STORAGE = 1.

MAXIMUM STAGE IS 409.1

STATION 4, PLAN 1, RTIO 3

OUTFLOW									
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.

Sheet 200 of 34

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.

1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	2.	2.	2.	3.	3.	3.	4.	4.
5.	5.	5.	8.	11.	15.	22.	30.	40.	55.
78.	107.	136.	159.	179.	194.	203.	207.	209.	209.
209.	208.	207.	206.	205.	204.	202.	201.	200.	198.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

STAGE

406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.1	406.1	406.1	406.2	406.2	406.3	406.4
406.5	406.8	407.0	407.1	407.3	407.3	407.4	407.4	407.4	407.4
407.4	407.4	407.4	407.4	407.4	407.4	407.4	407.4	407.4	407.3

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	209.	206.	80.	39.	3857.
CMS	6.	6.	2.	1.	109.
INCHES		1.63	2.54	2.55	2.55
MM		41.39	64.60	64.90	64.80
AC-FT		102.	159.	160.	160.
THOUS CU M		126	196	197.	197.

MAXIMUM STORAGE = 0.

MAXIMUM STAGE IS 407.4

STATION 4, PLAN 1, RTIO 4

OUTFLOW

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	1.	1.	1.	1.	1.	2.
2.	2.	2.	3.	4.	6.	8.	11.	14.	20.
28.	39.	49.	58.	65.	72.	75.	78.	79.	79.
80.	80.	80.	80.	80.	80.	79.	79.	79.	79.

STOR

Sheet 31 of 34

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

STOR

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

STAGE

406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	405.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	405.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	405.0
406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0	406.0
406.0	406.0	406.0	406.0	406.0	406.0	406.1	406.1	406.1	406.1
406.2	406.3	406.3	406.4	406.5	406.5	406.5	406.6	406.6	406.6
406.6	406.6	406.6	406.6	406.6	406.6	406.6	406.6	406.6	406.6

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
CFS	80.	79.	30.	15.	1464.
CMS	2.	2.	1.	0.	41.
INCHES		.63	.96	.97	.97
MM		15.94	24.43	24.53	24.53
AC-FT		39.	60.	60.	60.
THOUS CU M		48.	74.	75.	75.

MAXIMUM STORAGE = 0.

MAXIMUM STAGE IS 406.6

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

Sheet 22 of 34

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)

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 .75	RATIO 3 .50	RATIO 4 .25
HYDROGRAPH AT	1	.31 (.79)	1 (	998. 28.26)(	748. 21.19)(	499. 14.13)(	249. 7.06)(
HYDROGRAPH AT	2	.42 (1.09)	1 (	1241. 35.15)(	931. 26.36)(	621. 17.57)(	310. 8.79)(
2 COMBINED	10	.73 (1.89)	1 (	2200. 62.31)(	1650. 46.73)(	1100. 31.16)(	550. 15.58)(
HYDROGRAPH AT	3	.45 (1.16)	1 (	1471. 41.66)(	1104. 31.25)(	736. 20.83)(	368. 10.42)(
2 COMBINED	11	1.18 (3.04)	1 (	3631. 102.82)(	2723. 77.11)(	1816. 51.41)(	908. 25.70)(
ROUTED TO	3	1.18 (3.04)	1 (	1486. 42.07)(	651. 18.44)(	209. 5.93)(	80. 2.26)(
ROUTED TO	4	1.18 (3.04)	1 (	1487. 42.11)(	650. 18.39)(	209. 5.93)(	80. 2.25)(

Sheet 33 of 34

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	451.47	451.47	455.55
STORAGE	2909.	2909.	3610.
OUTFLOW	0.	0.	273.

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 ....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	451.47	451.47	455.55
STORAGE	2909.	2909.	3610.
OUTFLOW	0.	0.	273.

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	457.05	1.50	3984.	1486.	9.50	43.00	0.00
.75	456.21	.66	3729.	651.	8.50	43.50	0.00
.50	454.89	0.00	3493.	209.	0.00	45.00	0.00
.25	453.27	0.00	3212.	80.	0.00	46.50	0.00

PLAN 1 STATION 4

RATIO	MAXIMUM FLOW,CFS	MAXIMUM STAGE,FT	TIME HOURS
1.00	1417.	411.1	43.00
.75	610.	409.1	43.50
.50	209.	407.4	45.00
.25	80.	406.6	46.50

Sheet 34 of 34

STABILITY COMPUTATIONS

APPENDIX E

TAMS

Job No. 1579-15 — Sheet 1 of [illegible]

Project: Bayou Lake Dam — Date: 6-3-8[illegible]

Subject: Stability Analyses — By: [illegible]

Ch'k. by:

Assumptions:

1) The unit weight of masonry is 150 lb/ft³

2) Ice load of 5000 lb/ft [illegible] 1 ft from top of [illegible] (COE criteria)

3) Friction angle of foundation material is 25° & cohesion = 1 ksf

4) Dam Site is in seismic zone 1 - [illegible]

5) Normal Operating Lake Level is at EL 451.47

6) [illegible]
7) Stability analysis done in accordance with Corps of Engineers criteria

Additional Data:

1) [illegible] analyses the following headwater and tailwater elevations [illegible]

[illegible]	Headwater EL.	Tailwater EL.
½ PMF	[illegible]	433.75
1 PMF	[illegible].0	435.55

Cases Evaluated

Case I - Normal Loading; lake level at [illegible] crest EL. 451.47 - No ice load, Full Uplift

Case II - Case I condition with the [illegible] of [illegible] ice [illegible] - 1 ft [illegible]

Case III - Unusual Loading; Lake level at ½ PMF Stage

Case IV - Extreme Loading; Lake level at 1 PMF Stage

TAMS

Job No. 1577.15 | Sheet 2 of 10
Project Byllesby [illegible] Dam | Date 6-3-[illegible]
Subject Stability Analysis | By [illegible]
Ch'k. by

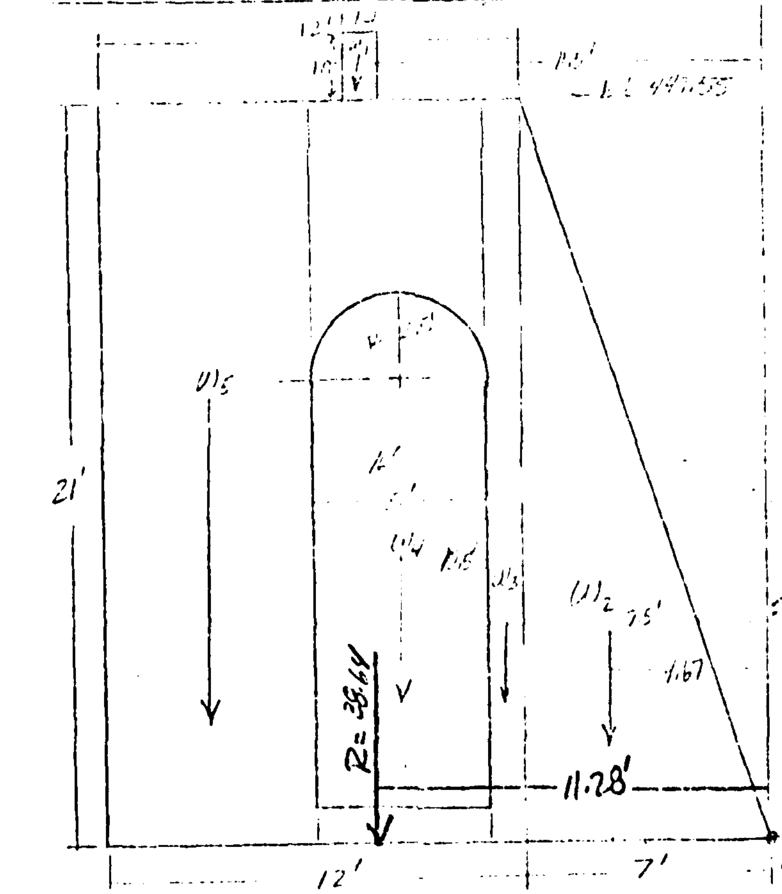
Stability Criteria

a) overturning – resultant force shall be contained within the middle 1/3 of the base for Cases I thru IV

b) Sliding – [illegible] of safety [illegible] shall be 3.0 or greater for Cases I thru IV

Case I – Force Calculations

Calculation of [illegible]



Scale 1" = 5'

$W_1 = 1.92 \times 1.0 \times 150 = .289\,k$

$W_2 = \frac{1}{2}(7 \times 21)(150) = 11.025\,k$

$W_3 = 1 \times 21 \times 150 = 3.15\,k$

$W_4 = [(5 \times 21) - (\text{[illegible]}) - (\text{[illegible]})]150$

$= [105 - 9.8 - \text{[illegible]}]\,150 = 5.2\text{[illegible]}$

$W_5 = 6 \times 21 \times 150 = 18.90\,K$

W	X_A	M
.289	11.5'	3.21 k-ft
11.025	4.7'	51.49
3.15	7.5'	23.62
5.28	10.5	55.44
18.90	16.0	302.40
ΣW: 38.64	ΣM_A	436.26 k-ft

$\overline{X}_A = \frac{436.26}{38.64} = 11.28'$

OK – within middle 1/3
6.33 ≠ 12.66

EL 425.55

TAMS

Job No. 1579-15 — Sheet 3 of 15

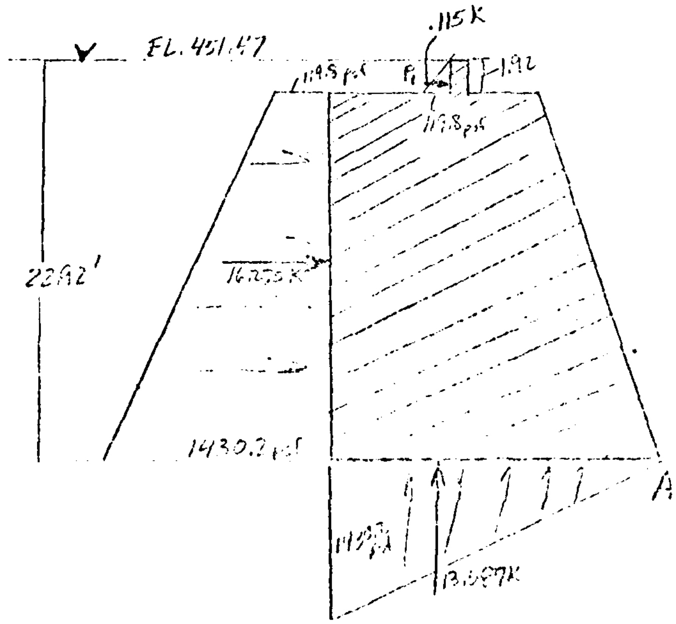
Project Bynum Lake Dam — Date 6-3-81

Subject Stability Analysis — By JRW

Ch'k. by

Case I cont.

Calculation of Hydrostatic Forces



Scale 1" : 10'

$P_1 = 119.8 \times 1.92 \times 1/2 = .115\ k$

$P_2 = (119.8 \times 21) + (\frac{1}{2} \cdot 1310.4 \times 21) =$ 16.275 K

$U = (1430.2 \times 19)\frac{1}{2} = 13.587\ K$

$MA_{P_1} = \frac{1.92}{3} + 21 = 21.64'$

$MA_{P_2} = 11.35'$

$MA_U = 2/3(19) = 12.67'$

W 8428.55

$\Sigma M_1 = (.115)(21.64) = 2.5$ k-ft
$(16.275)(11.35) = 184.7$ k-ft
$+ (13.587)(12.67) = 172.1$ k-ft
357.3 k-ft

$\Sigma F_V = 13.587\ K$
$\Sigma F_H = 16.39\ K$

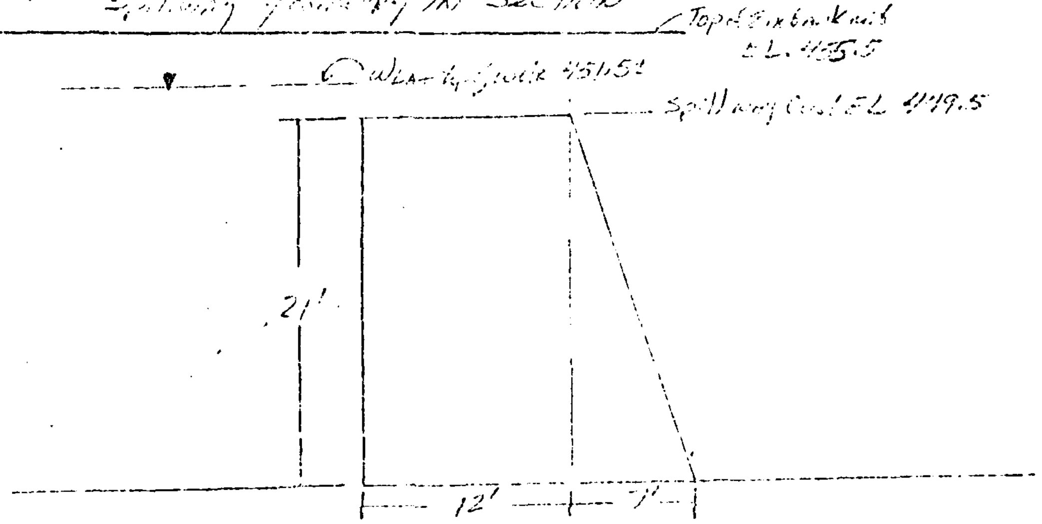
TAMS

Job No. 1579-15 | Sheet 4 of 10
Project [illegible] Dam | Date [illegible]
Subject [illegible] Analyses | By [illegible]
Ch'k. by

Case I Cont.

Calculate End Sheering Resistance for spillway section

Spillway Geometry in Section



1) [illegible] is assumed [illegible] of the spillway [illegible] [illegible]

2) The following Soil properties [illegible]

Embankment fill is silty sand

$\gamma_T = 125$ pcf $\quad \gamma = 62.4$ pcf

$K_0 = .40$ $\quad$ Frictional coefficient $\tan \delta = 0.30$

TAMS

Job No. 1579-15 [illegible]

Project [illegible]

Subject [illegible]

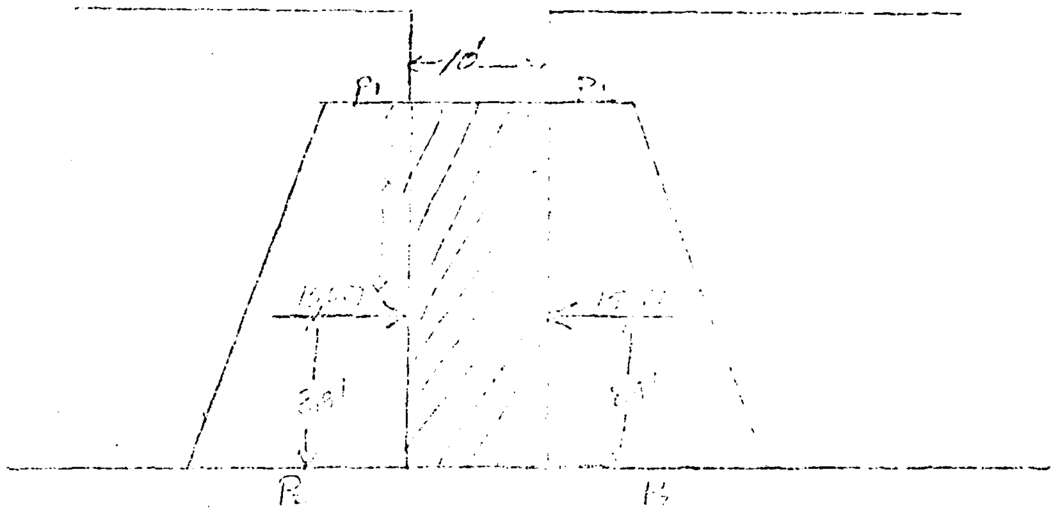
Sheet [illegible] of [illegible]

Date [illegible]

By [illegible]

Ch'k. by

Resulting from pressure [illegible] ([illegible])



$P_1 = (4 \times 125 + 2 \times 10.6) \times .4 = 250$ psf

$P_2 = (4 \times 125 + 23 \times 10.6) \times .4 = 776$ psf

Resultant force / [illegible] is $\left(\frac{250+776}{2}\right) \times 21 = 10{,}773$ plf

[illegible]

[illegible] $= 10.773 \times$ [illegible]

[illegible]

$= 166.9$ Kips

For [illegible] Solve

[illegible] $= \frac{10.773}{2}$

$x^2 + 46x - 215 = 0 \quad x = 12.8$

Resultant is 21-12.8' [illegible]

Total shear, [illegible]

$(166.9 \times 2) \times \frac{23.0}{[illegible]} = 100.14$ Kips

[illegible] $\frac{100.14}{10}$ [illegible] $= 10.0$ Kips

[illegible]

[illegible] $10.0 \times 8.2 = 82.0$ [illegible]

TAMS

Job No. 1579-15

Project Bypass Lake Dam

Subject Stability Analyses

Sheet 6 of

Date 6-3-91

By

Ch'k. by

Case II Force Calculations

Extreme Ice Loading

1' thick ice cap @ max water level exerting 5000 plf

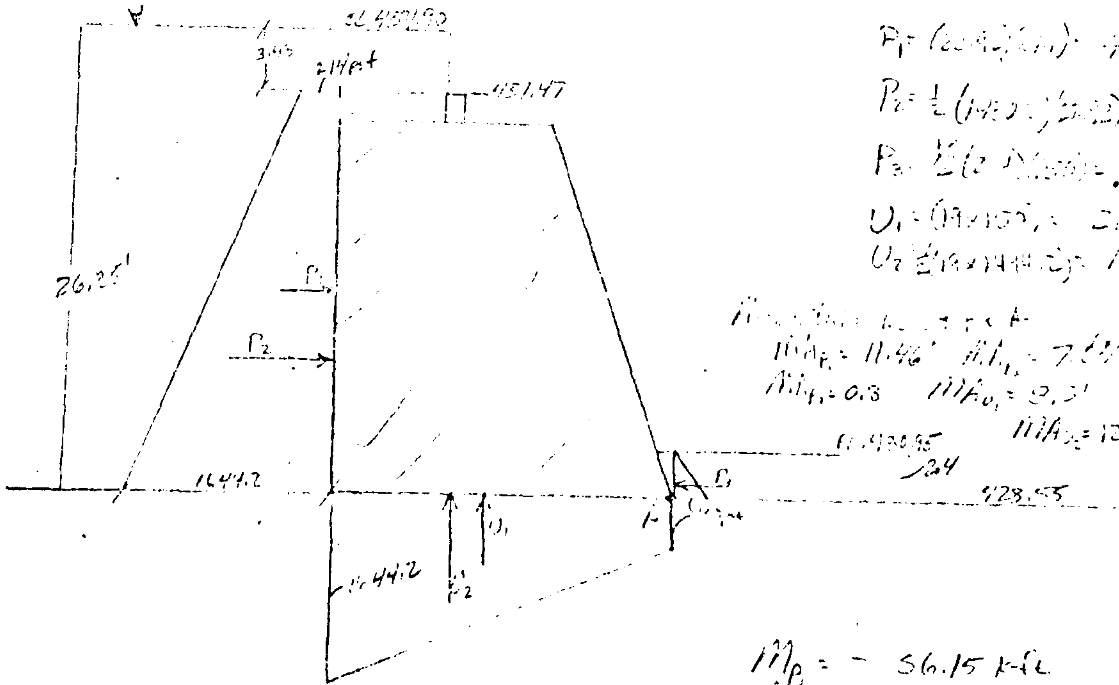
F_H = 5000 plf (1 ft)

MA. 22.98 - 0.5 = 22.48'

M_I = 5.00 k × 22.48 = 112.4 k-ft

Case III Force Calculations

[illegible] forces at ½ PMP



P_1 = (ca.[illegible]) [illegible]

P_2 = ½ ([illegible]) [illegible]

P_3 = [illegible]

U_1 = ([illegible]) = [illegible]

U_2 = ½ ([illegible]) [illegible]

Moment arms about A

MA_{P_1} = 11.46' MA_{P_2} = 7.84'

MA_{P_3} = 0.3 MA_{U_1} = 9.31'

MA_{U_2} = 12.[illegible]

M_{P_1} = − 56.15 k-ft

M_{P_2} = − 125.30

M_{P_3} + .16

M_{U_1} = − 25.8

M_{U_2} = − 177.8

ΣM = 385.0 k-ft

TAMS

Job No.

Project

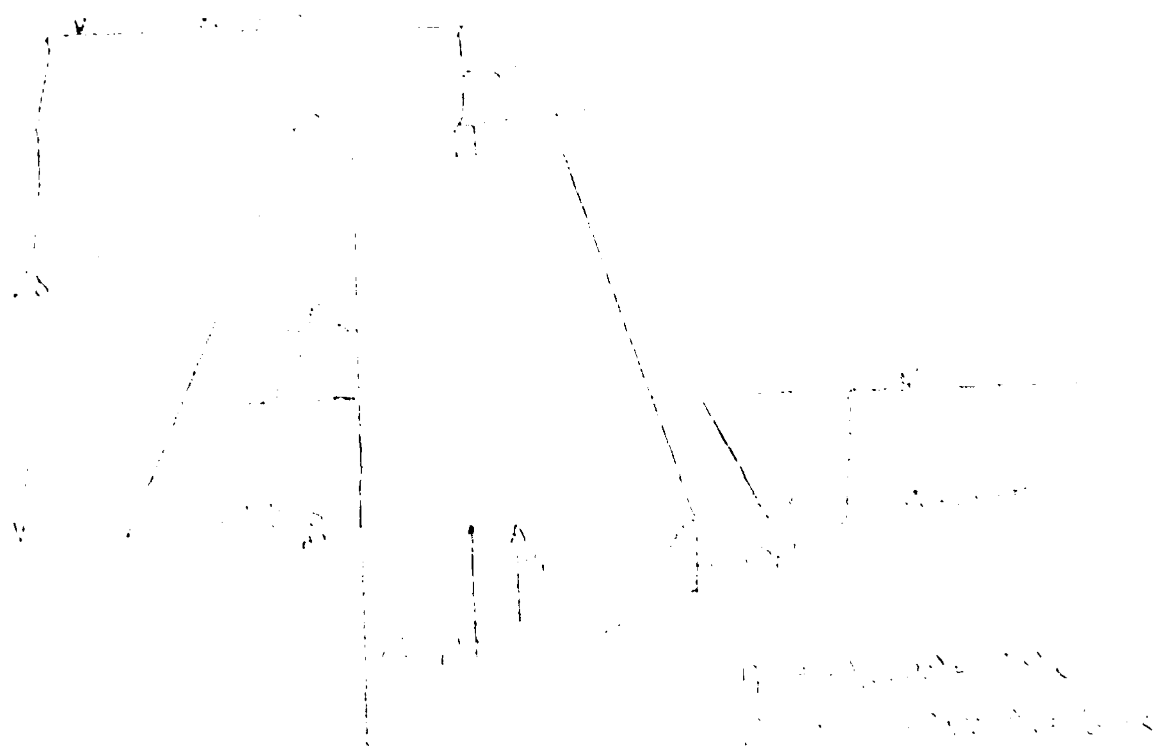
Subject

Sheet of

Date

By

Ch'k. by



TAMS

Job No. 1579-15

Project Byron Lake Dam

Subject [illegible]

Date 6-3-81

By [illegible]

Ch'k. by

Analyses

Sign Convention M - Skidding; + [illegible]

F + Down and Downstream; - Up & Upstream

Case I - Normal Loading - No Ice

	V	F_H	M
Dead Load	38.64 k	----	+ 436.2[illegible] k-ft
Uplift [illegible]	- [illegible] k	[illegible]	- [illegible] k-ft
[illegible]	[illegible]	[illegible]	+ [illegible] k-ft
	[illegible]	6.3[illegible]	[illegible].76

$e = \frac{19}{2} - \frac{[illegible]}{[illegible]} = 3.15$

$\frac{19}{6} - 3.15 \geq 0$ ([illegible]) OK

[illegible]

[illegible]

Factor of Safety Against Sliding

$F.S. = \frac{[illegible]}{\Sigma F_H}$

$F.S. = \frac{[illegible]}{6.39}$

$F.S. = 4.39 > 3$ OK

AD-A107 417 TIPPETTS-ABBETT-MCCARTHY-STRATTON NEW YORK F/G 13/13
NATIONAL DAM SAFETY PROGRAM. BYRAM LAKE RESERVOIR DAM (INVENTOR--ETC(U)
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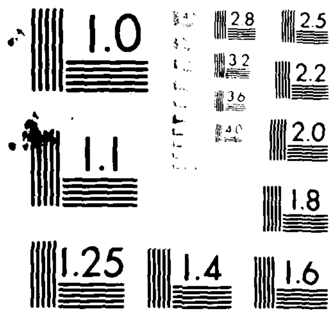


MICROCOPY RESOLUTION TEST CHART

TAMS

Job No. 1579.15

Project Byram Lake Dam

Subject Stability Analyses

Date 6-?-81

By J.M.

Ch'k. by

Case II - Normal Load plus Ice Load

	F_V	F_H	M
Dead Load	38.64K	—	+436.26 KFt
Hydrostatic Forces	-13.59K	16.89K	-359.30
Ice Load	—	5.0K	-112.1kft
Side Shear	—	-10.0K	+ 82.0kft
	25.05K	11.89K	46.86

$\bar{e} = \frac{19}{2} - \frac{46.86}{25.05} = 7.62$ ft

is $\frac{19}{6} - 7.62 = 0$ (-4.45) not within middle 1/3

Factor of Safety Against Sliding

$F.S. = \frac{25.05 \tan 25 + 1(19)}{11.89}$

$F.S. = 2.69 < 3$

Case III - 1/2 PMF

	F_V	F_H	M
Dead Load	38.64 k	—	436.26 kft
Side Shear	—	-10.0	+ 82.00
Hydrostatic Forces	-17.0 k	21.1	-385.2 kft
	+ 21.64	11.1	+ 133.26 kft

$\bar{e} = \frac{19}{2} - \frac{133.26}{21.64} = 3.34$ ft

is $\frac{19}{6} - 3.34 = 0.17$ ft, outside middle 1/3 NG.

Factor of Safety Against Sliding

$F.S. = \frac{21.64 \tan 25 + 1(19)}{11.1}$

$F.S. = 2.62 < 3$ NG

TAMS

Job No. 1573-15 | Sheet 10 of 10
Project Equinunk Lake Dam | Date 6-3-81
Subject Stability Analysis | By J.F.
Ch'k. by

Case IV PMP

	F_V	F_H	M
Dead Load	38.64 k	—	436.26 kft
Side Thrust	—	-10.0 k	+ 82.00 kft
Hydrostatic & Uplift	-21.0 k	22.8 k	-443.70 kft
	17.64	12.8 k	74.56 kft

$$e = \frac{19}{2} - \frac{74.56}{17.64} = 5.27$$

$$\frac{19}{6} - 5.27 = -2.10 \text{ ft}$$ outside of middle 1/3 — because not in middle — N.G.

$$F.S. = \frac{17.64 k \tan 25^\circ + c(19)}{22.8^*}$$

$$F.S. = 1.19 < 3$$

*Note that side force was not used to compute the F.S. value, since this force will not exist due to overtopping of the embankment.

REFERENCES

APPENDIX F

REFERENCES

1. "Flood Hydrograph Package (HEC-1) Users Manual for Dam Safety Investigations", U.S. Army Corps of Engineers, Hydrologic Engineering Center, September 1979.

2. "Seasonal Variation of the Probable Maximum Precipitation, East of the 105th Meridian for Areas from 10 to 1,000 Square Miles, and Durations of 6, 12, 24 and 48 Hours", Hydrometeorological Report No. 33. Weather Bureau, U.S. Department of Commerce, April 1956.

3. "Recommended Guidelines for Safety Inspection of Dams", Department of the Army, Office of the Chief of Engineers, Appendix B.

4. "New Englad Upland Section", Internal Report, Civil Engineering Department, Purdue University, West Lafayette, Indiana, August 1977.

5. Geologic Map of New York, The University of the State of New York, The State Education Department, Map and Chart Series No. 5, Albany, New York, 1962.

OTHER DATA

APPENDIX G

VILLAGE OF MOUNT KISCO

WATER UTILITIES STUDY

FEBRUARY 1981

DRAFT

Hazen and Sawyer, P.C.
Engineers
Mount Kisco, New York
New York, New York

STUDY

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

VILLAGE OF MOUNT KISCO

WATER UTILITIES STUDY

I. INTRODUCTION

This study provides a preliminary assessment of Mount Kisco's existing water supply and distribution facilities to meet projected future water demands. Long term projections made in 1966 were updated to reflect the newly proposed land use plan for the Village and the feasibility of alternative projects to meet a projected supply deficit was evaluated. Such projects include purchases of supplementary supplies from neighboring municipalities and an examination of alternative measures to increase the safe yield of Byram Lake.

A preliminary analysis of the water distribution system was undertaken to determine in general, those areas of the Village with adequate and/or inadequate domestic service and fire protection. Based on this investigation, required improvements were identified and both a preliminary cost estimate and order of priority for implementation established.

2. EXISTING WATER SYSTEM

General Description

The Mount Kisco water system is owned and operated by the Village and serves the entire Village and small outside areas in the Town of Bedford. Mount Kisco's sources of supply, shown on Figure 1, are Byram Lake, the primary source of supply, and a well field located on Green Lane in the Town of Bedford. Water is drawn from the lake through either an 8-inch or 12-inch intake pipe. The Byram Lake pumping station is equipped with three manually primed pumps with rated capacities of 2.9, 1.7 and 1.1 mgd (million gallons per day). The largest pump was installed in 1961, and new motor controllers were installed on all pumps in 1978.

Water is pumped from Byram Lake through a 12-inch main to two open reservoirs located approximately one-half mile west of the lake along Byram Lake Road. Under normal operation, the upper reservoir with a flow line at elevation 551 feet and a capacity of 1.5 MG (million gallons) feeds the 4.5 MG lower reservoir at elevation 539 feet. Water flows into the distribution system from the lower reservoir, where chlorine is added to the outlet pipe. The transmission system consists of parallel unlined 8-inch and 16-inch mains which were installed in 1928, and a new 16-inch cement lined ductile iron main. New chlorination equipment and a venturi flow meter were installed in 1978 when the new transmission main was constructed.

The well supply consists of three wells drilled in glacial deposits in the low lying area along the Kisco River. In 1955, the New York Water Power and Control Commission estimated the combined yield of the wells at about 0.4 mgd, and the existing output is approximately 0.2 mgd. The existing well pumps have rated capacities of 190, 65 and 35 gpm (gallons per minute) and the water is chlorinated before it is fed to the distribution system through a 10-inch main on Green Lane and parallel 6-inch and 8-inch lines along North Bedford Road. Because of the limited drainage area tributary to the well field (0.68 square miles) and the fact that there is little likelihood of the aquifer being supplied from other drainage areas, it is felt that the safe yield of the well supply should be estimated at the present 0.2 mgd pumpage rate.

The distribution system is divided into two service zones. The area east of the railroad tracks, is served by gravity from the Byram Lake Road reservoirs while west of the tracks above elevation 400± feet, the area is served by a high service booster pumping station and 500,000 gallon storge tank which was constructed in 1968. System storage is adequate to meet peak demands, fire protection requirements and other emergency situations.

Byram Lake

Byram Lake was part of the New York City water supply system until 1958, at which time it was purchased by Mount Kisco for water supply purposes. The drainage area tributary to the lake is 1.37 square miles, including the lake surface area of approximately 0.25 square miles (see Figure 1). In addition, runoff from a 0.20 square mile area on the southwest side of the lake is drained to the reservoir by a diversion channel. The storage volume with the reservoir filled to the flow line of the spillway is estimated at 948 MG with storage available for water supply purposes with the present intakes limited to 493 MG in the upper 9-1/2 feet of the lake.

The lake, which covers approximately 16 percent of the Byram Lake drainage area and provides approximately 300 MG of storage per square mile of watershed area, is a more highly developed water supply than other nearby systems. It is estimated that almost 90 percent of the average annual runoff of 1.1 mgd per square mile can be developed for supply. After correcting for evaporation and allowing for the additional runoff which is diverted to the lake, the safe yield of Byram Lake is estimated at about 1.2 mgd as described later.

Generally, water quality in Byram Lake is typical of a lake in the early stages of eutrophication. Oxygen levels are generally about 5 mg/l (milligrams per liter) throughout, high enough to support fish life. Nutrients such as phosphorus and nitrogen are present in sufficient quantity to support algae blooms. The clarity of the water is good, with turbidity levels reported as normally below 5 NTU (Nephelometric Turbidity Units), and suspended solids below 10 mg/l.

Taste and odor problems with Byram Lake water were reported as early as 1960 by the County Department of Health. The problems are believed to be related to increases in emergent vegetation and algae. Starting in the spring of 1980, chemical control of algae and protozoas by the application of copper sulfate was undertaken. The control of emergent vegetation was considered, but NYS Department of Environmental Conservation regulations controlling the use of Diquat in water supplies ruled out its use at that time.

Although the quality of Byram Lake water is acceptable at this time, requirements set forth in the Safe Drinking Water Act dictate that planning for future water supply facilities consider treatment beyond the present practice of chlorination.

3. VILLAGE GROWTH AND WATER DEMAND FORECASTS

Projections of future population and commercial/light industrial development are presented to demonstrate the anticipated growth potential of Mount Kisco and to confirm the need for expanded sources of water supply. Growth estimates were developed by the Village's planning consultant, Raymond, Parish, Pine and Weiner, Inc.

Residential/Commercial Development

Projections of expected short term, one to five years, and long term, five to fifteen years, population increases are presented in Table 1. The projections are for individual parcels as identified on the Village zoning map and are estimated based on either zoning capacity or firm development proposals.

It is estimated that the population within the Village may increase by 36 percent to about 12,500 by the year 2000. The population served by the water system, including customers outside of the Village, may increase from 9600 in 1980 to 13,200 in the year 2000. These estimates are based on full service within the Village and the assumption that there will be only a minimal increase in the customers served outside of the Village.

It is projected that about one million square feet of additional commercial (including office and retail space) and/or light industrial floor area will be developed by the year 2000. As indicated on Table 2, it is anticipated that about 40 percent of this total will be developed by the year 1985.

Water Demands

Historical metered water sales for 1974 through 1979 are presented on Table 3. As indicated, average daily sales increased from 1.04 mgd in 1974 to 1.14 mgd in 1978. Per capita sales for the same period ranged from 112 gcd (gallons per capita per day) in 1976 to 121 gcd in 1978. Sales within the Village increased by 10 percent from 1974 to 1979, while sales to outside customers decreased by approximately 25 percent. The relatively high per capita useage is attributable to the significant amount of water used by non-residential development within the Village, e.g. offices, light manufacturing, hospital, etc.

Complete records of total water demand, which includes both metered sales and unaccounted for water, are not available for the period from 1974 through 1979. Therefore, unaccounted for water, maximum day demand (which is used as a basis for design of treatment facilities) and peak hour demand (used to calculate required system storage and transmission capacity) must be estimated. Unaccounted for water includes losses due to leakage, water used for fire fighting and flushing of mains, and meter slippage. Considering the age of the village distribution system, when projecting future demand it will be assumed that unaccounted for water will decrease from 25 percent of total demand in 1980 (based on leakage reported in the 1966 Report by Hazen and Sawyer) to 15 percent by the year 2000. This decrease assumes that the Village will undertake a continuing program of leakage detection and meter repair.

In projecting future metered sales, it is assumed that per capita sales will decrease from the present level of 120 gcd to 106 gcd in the year 2000, reflecting some conservation and the lower consumption rate for proposed multi-family housing. Average demands for the proposed commercial/light industrial developments is estimated at 0.2 gpd per square foot of floor area. Projections of future water demands are presented on Table 4. The projections indicate that the total average daily demand will increase from an estimated 1.55 mgd in 1980 to about 1.90 mgd by the year 2000.

In estimating maximum daily and hourly demands, peaking factors of 180 percent and 260 percent were assumed based on operational experience in neighboring communities. By the year 2000, maximum day and hourly demands are estimated to increase to about 3.4 and 4.9 mgd, respectively.

TABLE I

VILLAGE OF MOUNT KISCO

FUTURE POPULATION

		Total[1] Village Population	Estimated Population Served by Water System: Inside Village	Outside[4] Village	Total
1970		8,172	8,172	500	8,672
1980[2]		9,120	9,120	500	9,620
1980-1985					
. CD - 350 units @ 2.5 persons/unit	875				
. R-6 - 130 units (Senior Citizen)	325				
. R-4 - 120 units @ 2.5 persons/unit	300				
Subtotal	1,500	10,620	10,620	550	11,170
1985-2000					
. R.O - 165 units @ 2.8 persons/unit	465				
. RRR - 200 units @ 2.8 persons/unit	560				
. R-4 - 150 units @ 2.5 persons/unit	375				
. Misc. - 200 units @ 2.8 persons/unit	560[3]				
Subtotal	1,960	12,580	12,580	600	13,180

Notes:

(1) Projections by Raymond, Parish, Pine and Weiner, Inc., September 1980. Reference zoning map for location of development areas.
(2) Local estimate. Preliminary count of U.S. Census 7726.
(3) Includes 100 units which could be developed on the Golf Course.
(4) Estimated based on 200 customers in 1970 and 1980 and 2.5 persons per customer.

TABLE 2

VILLAGE OF MOUNT KISCO

FUTURE COMMERCIAL/LIGHT INDUSTRIAL DEVELOPMENT(1)

Project Location	Estimated Additional Floor Space, sf. 1980 to 1985	1985 to 2000	Total
. Route 172 to Village Line	150,000	150,000	300,000
. Radio Circle	-	250,000	250,000
. Lexington Avenue - From Radio Circle to Moore Avenue	-	100,000	100,000
. South Moger Avenue	-	60,000	60,000
. Kisco Avenue to Industrial Drive	50,000	-	50,000
. North Bedford Road - Village Line to Baker Street	200,000	-	200,000
Total	400,000	560,000	960,000

Note:

(1) Projections by Raymond, Parish, Pine and Weiner, Inc., September 1980.

TABLE 3

VILLAGE OF MOUNT KISCO

HISTORICAL WATER CONSUMPTION

	Total Metered Sales MG	Average Daily Metered Sales mgd Inside Village	Outside Village	Total	Estimated Population Served	Estimated Per Capita Metered Sales gcd
1974	380	.96	.08	1.04	9050	115
1975	385	.97	.08	1.05	9150	115
1976	375	.97	.06	1.03	9250	111
1977	400	1.03	.07	1.10	9350	117
1978	415	1.08	.06	1.14	9450	120
1979	410	1.07	.06	1.13	9550	118

TABLE 4

VILLAGE OF MOUNT KISCO

FUTURE WATER DEMANDS

		Metered Water Sales						
	Population Served	Daily Per Capita (1) Consumption gcd	Total mgd	Additional (2) Commercial mgd	Unaccounted (3) for Water mgd	Total Average Daily Demand mgd	Maximum (4) Day Demand mgd	Maximum (5) Hour Demand mgd
1980	9,600	120	1.15	-	.40	1.55	2.8	4.0
1985	11,200	112	1.25	.08	.32	1.65	3.0	4.3
2000	13,200	106	1.40	.20	.30	1.90	3.4	4.9

Notes:

(1) Assumes future residential population per capita consumption rate of 65 gcd.
(2) Estimated at 0.2 gpd per square foot.
(3) Estimated to decrease from 25 percent of total demand in 1980 to 15 percent in 2000.
(4) Estimated at 180 percent of Average Daily Demand.
(5) Estimated at 260 percent of Average Daily Demand.

4. WATER SUPPLY ALTERNATIVES

Need for Expansion

Mount Kisco's Byram Lake and North Bedford Road well supplies have a combined safe yield of approximately 1.4 mgd. As indicated below, average daily consumption presently exceeds the system safe yield by 0.2 mgd and the deficit is projected to reach about 0.6 mgd by the year 2000.

	1980	1985	2000
Maximum Day Demand - mgd	2.80	3.00	3.40
Average Daily Consumption - mgd	1.55	1.65	1.90
Safe Yield of Existing Supplies - mgd	1.40	1.40	1.40
Projected Average Daily Supply Deficit-mgd	0.15	0.25	0.50

Except for the summer and fall of 1980, rainfall has been greater than normal during the past few years, enabling the Village to overdraft its' Byram Lake supply. If the 1980 drought continues through the winter and spring of 1981 and Byram Lake is further depleted (it was estimated that the lake dropped to about 40 percent of capacity as of February 1, 1981), the Village may have to seek an immediate supplement to its water supply and/or impose strict conservation measures. Emergency drought measures are discussed later.

For the short-term, it is recommended that the Village formalize arrangements to purchase water from New Castle to supplement the present supply and to supply proposed developments. (New Castle's primary supply is New York City's Catskill Aqueduct and the secondary source is the New Croton Aqueduct as dissuced later.) Water should be purchased during New Castle's "off-peak" supply period of November through early May to reduce the draft on Byram Lake and increase its rate of recovery. Such an arrangement may be acceptable to New Castle as "off-peak" sales to Mount Kisco would not affect service to existing customers during the peak summer demand period. However, should arrangements with New Castle not be attainable for any number of reasons, the Village could not take on additional water customers without subjecting present users to the possibility of more stringent conservation measures to protect the limited supply during prolonged droughts. (A similar off-peak arrangement was recently implemented between Westchester Joint Water Works (WJWW) which serves the Village and Town of Mamaroneck and the Town/Village of Harrison, and the Port Chester Water Company. The agreement provides for the WJWW to supply up to about 2 mgd to the Company's system in the City of Rye to reduce the draft on it's surface supply from the Greenwich Water Company. WJWW furnishes water it purchases from New York City's Catskill/Delaware system.)

The timing of the start of an off-peak supply from New Castle may be dependent upon the recovery of the New York City reservoir system from the present drought. The City system was at about 30 percent of capacity on February 1st. While the Village might be legally entitled to draw City water from New Castle, we do not feel that it would be ethical to start use City water as long as the City's reservoirs are proportionately lower than Byram Lake and appear to be dropping at a faster rate. However, the situation should be reviewed each month and, if conditions change, the Village should be prepared to start drawing City water providing New Castle can furnish it.

Long-Term Sources of Supply

Sources of additional supply secured by Mount Kisco should be adequate to meet the projected long term water demand. Alternative sources of future supply which were investigated include expansion of the Byram Lake drainage area, purchase of supplementary supplies from New Castle, and development of a joint supply with New Castle. A direct supply from the New York City system has been considered un-economical in the case of the Croton system and not feasible in the case of the Delaware system. The Village has existing interconnections with the New Castle water system which is supplied from New York City's Catskill and Croton Aqueducts as shown on Figure 2. Modifications to the existing Byram Lake intakes to increase available storage are discussed herein. However, modifications to raise the level of Byram Lake dam without an expansion of the drainage area is not recommended.

Rock wells are used throughout Westchester County for private domestic supplies, but, generally, they do not yield enough water for large municipalities. Additional wells south of the Village might be developed if glacial deposits exist and can be located. However, a preliminary hydrogeologic study, test drilling and pumping would be needed to determine the quantity of water available. While we recommend that such a program be implemented, it would not be prudent to assume at this time that any additional supply can be obtained. If the program proceeds as far as test drilling and pumping, the results can be analyzed and an economic comparison can be made with the alternatives evaluated in this report.

The safe yield of a water supply system is the amount of water which can be continuously drawn during a prolonged drought and, for a surface supply with a reservoir, it is a function of the rainfall and runoff in the watershed, the volume of storage provided in the reservoir and the evaporation from the surface of the reservoir. The maximum yield that can be developed from any system with an infinitely large reservoir is equal to the average runoff less the evaporation from the reservoir. Unfortunately there are no runoff records available for the Byram Lake watershed, but on the basis of information available on small watersheds in Westchester County, the long-term average runoff is estimated at about 1.1 mgd per square mile. Therefore, the maximum safe yield that could be developed from the Byram Lake watershed with an unlimited amount of storage is estimated at about 1.1 mgd/square mile x 1.57 square miles = 1.73 mgd, less about 0.3 mgd of surface evaporation, or about 1.4 mgd.

From detailed hydrologic studies in other watersheds, it has been estimated that the safe yield which can be developed from the Bryam Lake watershed during severe droughts would be approximately as shown on Figure 3. (Also shown on Figure 3 is a plot of yield versus storage for larger streams on the east side of the Hudson River where the average runoff is about 0.9 mgd/square mile. This plot was developed in the Comprehensive Water Supply Study for Westchester County and New York City.) Using the estimated curve for the Byram Lake watershed, the safe yield which can be developed is estimated as follows:

		Storage		Estimated Safe Yield			
		mg	Storage/ mg/ sq.mi.	Yield mgd/ sq.mi.	Total Yield mgd	(Less) Evaporation mgd	Net Yield mgd
Present	-	493	314	0.91	1.46	0.3	1.16
Possible Future	-	700±	440	0.98	1.54	0.3	1.24

From the foregoing, it is evident that the existing Byram Lake watershed is close to ultimate development, i.e., adding more storage will increase the safe yield only slightly. Therefore, if more of the present storage capacity was utilized by lowering the existing intake and pumping facilities, the safe yield could be increased only slightly as shown. The entire capacity of the reservoir could not be used as water quality, intake, and pumping considerations usually make the lower 25 per cent of a reservoir unuseable.

Considering the accuracy of the estimates of drought yields and assuming that some increase in useable storage capacity of Byram Lake may be secured either on an emergency or permanent basis, it is felt that the safe yield of the supply can be assumed as between 1.2 and 1.3 mgd. For purposes of this report, we have used th lower limit with present storage and the upper limit with increased storage. Taking into account the yield of the existing wells, the combined yield of the system will be between 1.4 and 1.5 mgd depending upon the total storage used.

Future Supply Alternatives

Three alternative water supply projects were investigated. In evaluating the projects, it was assumed that if Byram Lake is to be used as a primary source in the future, permanent modifications to the existing intake, a new raw water pumping and treatment facilities will eventually be required, although the timing of these improvements cannot be established now. To increase the safe yield during severe droughts to about 1.3 mgd, the existing intake will have to be modified and new raw water pumping facilities constructed at a lower elevation to permit the use of an additional 200± MG of storage. This will increase the utilization to about 73 percent of the total volume of storage. Treatment facilities would be constructed in the vicinity of the existng low service reservoirs on Byram Lake Road. The facilities would have to be sized to handle future maximum day demands less the output of the existing wells or about 3.2 mgd, with provision for only limited expansion beyond the year 2000.

- Alternative 1 - Develop Additional Local Surface Supplies to Meet Total Demand

  The Village would increase the safe yield of Byram Lake to approximately 2.0 mgd by developing runoff diversion facilities on the 0.75 square mile drainage area to the west of the lake identified on Figure 4. Runoff from the new drainage area would be diverted by a small dam and pumped approximately 7,000 feet to Byram Lake.

- Alternative 2 - Participate in a Joint Water Supply Project to Meet All Water Demands

  Under this plan, Mount Kisco would secure its primary supply through a joint project with the Town of New Castle with Byram Lake and the North Bedford Road wells maintained only as standby sources of supply. (The Byram Lake supply would not be modified and local treatment facilities would not be added under this alternative.) The Village could either contract to purchase water on a wholesale basis or seek to participate in a recently proposed construction program to expand and eventually treat New Castle's Catskill Aqueduct supply. A new transmission main would be constructed on Bedford Road from Roaring Brook Road in New Castle to the 16-inch transmission main at the intersection of Byram Lake and Bedford Roads in Mount Kisco. Some improvements in the New Castle transmission system may also be required to transmit water from the proposed New Catskill Aqueduct connection in Millwood to Roaring Brook Road.

- Alternative 3 - Purchase Water to Supplement Existing Supply

  Under this plan, the Village would secure an off-peak supplementary supply from New Castle. The Byram Lake supply would be retained as the primary source of supply and it would be used to its maximum capabilities, i.e. it would be "over-drafted" when above average runoff is available. However, the storage would have to be carefully managed so that Byram Lake could handle the entire system demand (less the well supply) during the critical summer months. This will permit the Village to supplement the Byram Lake supply as required during the off-peak months. This type of program should minimize the total amount of water to be purchased from New Castle over a long period of time and should minimize the unit cost of such water since it will be purchased in off-peak months.

If consumption reaches the total estimated in the year 2000 and a severe drought occurs, the amount of water to be purchased would average about 0.4 mgd, (1.9 mgd demand less 1.5 mgd system yield) on an annual basis. Assuming that the purchase must be made during a six month off-peak period, the actual supply rate will be about 0.8 mgd. On a long term basis, the amount of water purchased would be less than in the drought years since the actual available runoff would be used as much as possible. However, since the average runoff is estimated at only about 0.1 mgd more than the safe yield of the system and there will be unavoidable inefficiencies in the management of a reservoir system with the supplementary supply, in this instance the average amount of water to be purchased can be estimated on the basis of drought years. Accordingly, the amount of water purchased under this scheme would be approximately as follows:

	1985	2000
Average Daily Consumption - mgd	1.65	1.90
Safe Yield With Lowered Intake - mgd	1.50	1.50
Amount of Water Purchased:		
Daily Average - mgd	.15	.40
6-month Average - mgd	.30	.80
Total - mg/year	55	146

A comparison of the design capacities and preliminary capital costs of the three alternatives is presented in Table 5. The cost estimates are based on 1980 dollars and would have to be updated in the future to coincide with actual construction schedules. The largest single element of each project is the cost of treatment facilities which, as explained later, would not be constructed in the immediate future.

Estimates of the cost of treatment faclities in Alternative 2 - New Joint Supply Project are based on preliminary cost estimates developed by Hazen and Sawyer in a recently completed study for the Town of New Castle and "scaled-up" estimates as necessary for joint requirements. The "scaled-up" estimates have not been presented to the Town of New Castle and if Alternative 2 is selected, both municipalities would have to review the estimates, capacities to be provided for each participant, allocation of costs, institutional arrangements to accomplish a joint project, etc. The cost estimates presented in the New Castle report and the "scaled-up" joint estimates used herein for Alternative 2 are compared as follows:

	Initial Design Capacity mgd	Capital Cost Total	Capital Cost $/mgd
New Castle Alone:			
1. Catskill A. Connection tion and Pumping Station	6.6	$1,100,000(1)	$167,000
2. Future Treatment Plant	6.6	$5,000,000	$758,000
Alternative 2 - Joint Project:			
1. Catskill Aqueduct Connection and Pumping Station	10.0	$1,500,000±	$150,000
2. Future Treatment Plant	10.0(2)	$6,300,000	$630,000

(1) Does not include allowance for purchase of site for pump station and future treatment plant.

(2) Capacity based on Option 3 in New Castle report. This option assumes treatment of 100 percent of Catskill supply.

The three alternatives are compared in Table 6 on an annual cost basis using 1980 dollars and projected water consumption for the year 2000. Present electric power costs were used and maintenance costs estimates for joint and separate treatment plants reflect some economies of scale. For the joint project under Alternative 2, a service charge by New Castle was not included in the estimates because, as explained later, a joint project offers other economic advantages to New Castle. Also, the capital cost estimates include an allowance for Mount Kisco to pay for transmission main improvements within the New Castle system to permit full service to the Village. For the off-peak supplementary supply under Alternative 3, a New Castle service charge of $150 per million gallon has been assumed. (This is, of course, in addition to N.Y.C. purchase cost, electric power costs for pumping and future treatment costs.) The assumptions and allowances used in the joint project and off-peak supply alternatives will require review, discussion and refinement by both municipalities, but could be used as a starting point in a joint discussion.

The cheapest project for the Village is Alternative 3 - Purchase Water to Supplement Existing Supply which has both the lowest capital and annual costs. For the Town of New Castle, this alternative offers the opportunity to utilize its otherwise idle capacity during off-peak months. The Village can also defer any permanent capital improvements to the intake and pumping station as long as possible, as it appears that it will be cheaper to purchase water than increase the yield of the exisiting Byram Lake supply. When the existing Byram Lake pumping station has to be replaced in the future because of age, the new station and the modification to the intake can be designed to utilize the lower 200± mg of the lake at a minimal increase in cost. (As discussed later, temporary emergency modifications should be implemented and these will suffice until permanent modifications are made.)

Alternatives 1 and 2 are approximately equal in annual costs but the latter is cheaper in capital cost. Alternative 2 - Joint Water Supply Project offers significant economic advantages to New Castle which, if New Castle so elects, might be used to reduce Mount Kisco's cost to make Alternative 2 more attractive vis-a-vis Alternative 3. The capital costs for separate and joint projects presented previously indicate that

New Castle's capital costs would be reduced approximately as follows in a joint project where costs are allocated in direct proportion to capacity provided as follows:

1.	Catskill Aqueduct Connection and Pumping Station	6.6 x ($167,000-$150,000) =	$112,000
2.	Future Treatment Plant	6.6 x ($758,000-$630,000) =	845,000
		Total Reduction	$ 957,000

If about one-half of this capital savings was shifted from New Castle to Mount Kisco as an inducement to participate in a joint project, the annual costs of Alternative 2 would be decreased from about $605,000 to about $565,000. This compares with about $522,000 for Alternative 3. Therefore, within the accuracy of the preliminary estimates, the two alternatives would be almost equal.

Since the viability and economics of Alternative 3 - Purchase Water to Supplement Existing Supply depends upon the position adopted by the Town of New Castle, the latter could conceivably establish terms for Alternative 3 such that it would be more expensive than Alternative 2. This would force the Village to opt for Alternative 1 or 2, or to do nothing to increase the safe yield of the system except for required pumping station and intake improvements. On the other hand, at least until and perhaps beyond such time that treatment of the Catskill and Byram Lake supplies is required, it would be attractive to the Town of New Castle to sell off-peak water to the Village and, of course, to continue to be able to secure Byram Lake water from the Village on an emergency basis through the Bedford Road connection and pumping station. In similar situations, a municipality often [illegible] a minimum payment each year, regardless of whether it is a dry or wet year. In wet years, the purchaser may elect not to take delivery of the water if the purchaser can supply its own water at a cheaper unit cost. (This would be the case for the Village where Byram Lake pumping costs are cheaper than New Castle pumping plus N.Y.C. purchase costs.) As the seller is assured of a minimum annual payment, both parties are protected during the wet years.

It is apparent that the choice of both short and long term solutions to the Village's supply problems will be affected by the decisions of the Town of New Castle. The capital and annual costs presented in Tables 5 and 6 are for preliminary comparison purposes and will require further refinement as the timing of construction of treatment facilities is defined by regulating agencies and inter-municipal negotiations proceed.

The uncertainty of the timing of future construction of treatment facilities for each municipality, and the fact that it may not be the same for each municipality preclude definitive discussion between municipalities on Alternative 2 at this time. However, the preliminary annual cost comparison and the separate analysis on the possible economic advantage to New Castle indicate that Alternative 2 should be explored in detail before either municipality proceeds with their own treatment plant. Therefore, we recommend that any short-term arrangement with New Castle to provide an off-peak supplementary supply be sufficiently flexible so that it will permit the future implementation of Alternative 2 if both municipalities so wish.

Emergency Drought Measures

As discussed previously, the existing Byram Lake supply can be supplemented with New York City water purchased from New Castle during the off-peak season. However, the unprecedented rapid decline in the storage of the New York City system during the past few months make this alternative less feasible at the present time. With Byram Lake at about 40 percent of capacity and the City's reservoirs at only about 30 percent and dropping at a faster rate than Byram Lake, we feel that this alternative should not be implemented until the relative storage capacities are reversed. However, the Village should take the necessary steps with New Castle and New York City to permit the use of this supply as soon as possible.

Because of the uncertainty in the timing of a supplementary supply from New Castle due to of New York City's situation and the fact that New Castle may not have spare capacity available in the summer months, the Village should have other emergency alternatives. The cheapest and most readily implementable alternative is to modify the existing intake and provide potable pumping and piping facilities to permit the use of another 200± MG of Byram Lake storage.

When the lake level drops about 9.5 feet to about elevation 442 feet, the total suction lift for the existing pumps becomes excessive and the pumps lose prime and/or cavitation takes place. This situation can be overcome by constructing a separate intake pit on shore with new valved connections to the existing intakes. The pit would receive water pumped from potable pumps installed either along the lowered shoreline or, if submersible types are used, in the lake itself. Temporary piping would be installed from the pumps to the pit. Depending upon the type of pumps available, they would have direct engine drives or, for submersible pumps, a separate engine-generator unit located on the shore. (Westchester Joint Water Works used a similar pumping arrangement when the Kensico Reservoir fell below it's intake level in the summers of 1979 and 1980.) For preliminary purposes, we estimate that an installation of this type would cost from $35,000 to $50,000. If emergency pumps can be obtained from civil defense sources or rented, the initial costs would be considerably less.

If the emergency measures include pumping water from nearby ponds into Byram Lake, this alternative would be expensive to implement and to operate. Furthermore, since there are no large ponds or lakes nearby, the quantity will be limited and the quality of the water is likely to be poor. We do not recommend further investigation of this alternative unless the situation became more critical.

A preliminary hydrogeologic study to determine if there are potential sites for additional wells has been recommended. If the study indicates potential sites, a test drilling program followed by test wells would be required to determine the feasibility of developing additional ground water. The wells on Green Lane are in a glacial formation and such water bearing formation are limited in extent and yield in Westchester County. Therefore, we have not included any allowance for additional groundwater in our estimates of future supply. However, because of the critical drought, it would be prudent to investigate the sources that might be developed quickly and which would also serve as a supplement to Byram Lake in the future. We have discussed this matter with a firm of ground hydrogeologists familiar with the area and a study of this type under the supervision of Hazen and Sawyer would cost in the order of $1,500 to $2,000.

T/ E 5

VILLAGE OF MOUNT KISCO

COMPARISON OF DESIGN CRITERIA AND CAPITAL COSTS OF ALTERNATIVE SUPPLY PROJECTS
(1981 Dollars)

	Alternative 1 Increased Local Supply	Alternative 2 New Joint Supply	Alternative 3 Supplement to Existing Supply
Design Criteria for Year 2000			
. Maximum Daily Demand - mgd	3.4	3.4	3.4
. Average Daily Consumption - mgd	1.9	1.9	1.9
. Local Well Supply Utilized - mgd	0.2	0	0.2
. Existing Surface Supply Utilized - mgd	1.3	0	1.3
. Diversion Supply Utilized - mgd	0.4	0	0
. Average Daily Supply from Outside Sources - mgd	0	1.9	0.4
. Capacity of Local Water Treatment Plant - mgd	3.2	-	3.2
. Treatment Capacity from Outside Source - mgd	0	3.4	-
. Capacity of Diversion Supply - mgd	5.0	-	-
Preliminary Capital Costs			
1. Byram Lake Intake and Pumping Facilities	$ 500,000	-	$ 500,000
2. Diversion Facilities	1,500,000	-	-
3. Catskill Aqueduct Connection and Pumping Facilities	-	$ 510,000(1)	-
4. Transmission Main Extension	-	1,600,000(3)	100,000
5. Treatment Plant	3,300,000	2,100,000(2)	3,300,000
Total Capital Cost	$5,300,000	$4,210,000	$3,900,000

(1) Pro-rate share of $1,500,000± project to serve New Castle and Mount Kisco see text.
(2) Pro-rate share of $6,300,000± project to serve New Castle and Mount Kisco see text.
(3) Includes cost of transmission main from Boering Brook Road to 16-inch Village main plus allowance for improvements in New Castle system.

TABLE 6

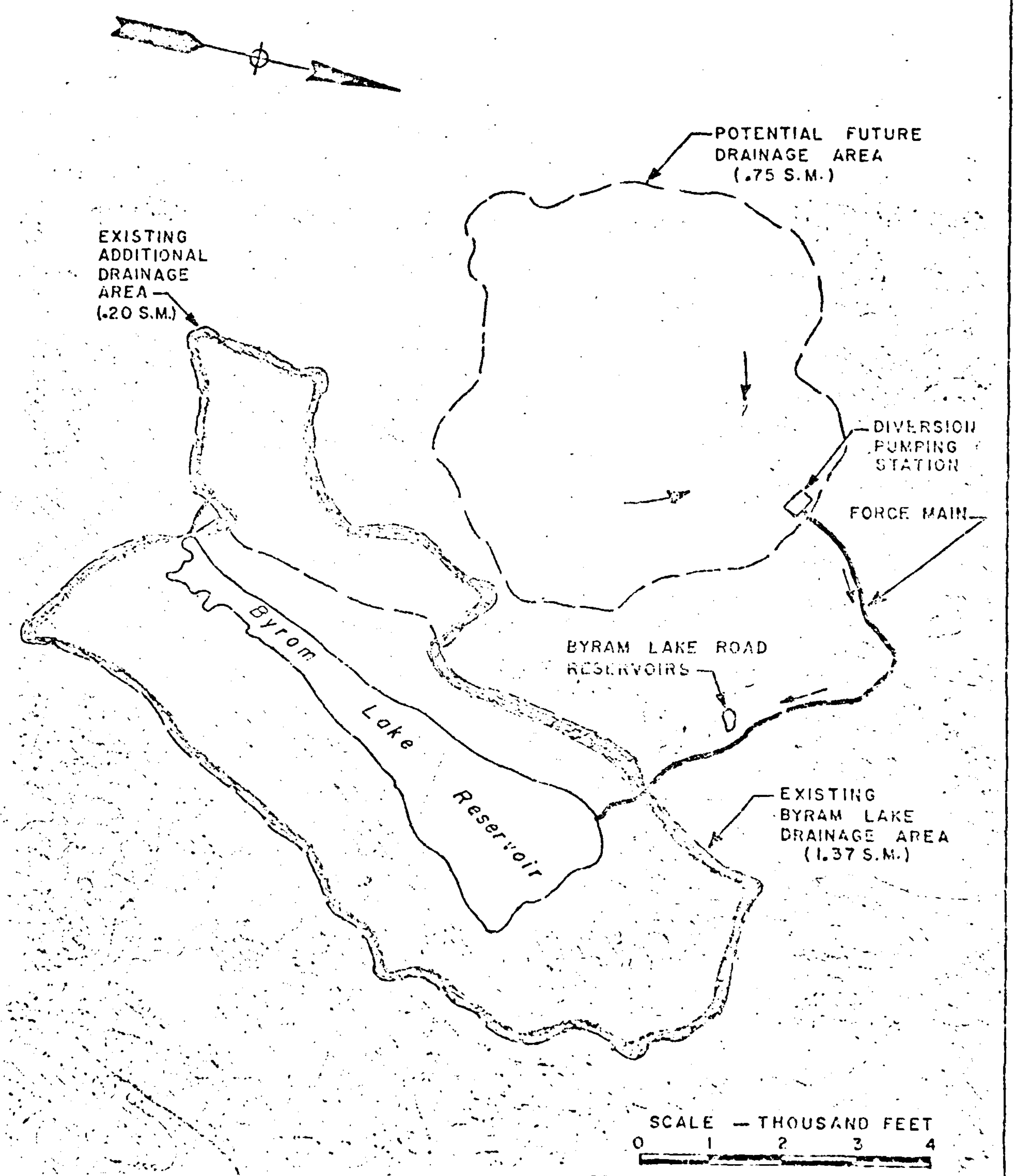
VILLAGE OF MOUNT KISCO

COMPARISON OF ANNUAL COSTS IN YEAR 2000 OF ALTERNATIVE SUPPLY PROJECTS
(1981 Dollars)

Item	Alternative 1 Increased Local Supply	Alternative 2 New Joint Supply	Alternative 3 Supplement to Existing Supply
1. Electric Power for Pumping:			
. Raw water from Byram Lake at $50/mg	$24,000	-	$24,000
. Well Supply at $100/mg	7,000	-	7,000
. Water from Outside Sources at $130/mg(1)	-	$90,000	19,000
. Diversion Facilities at $100/mg(2)	15,000	-	-
Subtotal	$46,000	$90,000	$50,000
2. Purchase of Water:			
. NYC Charge at $104/mg	-	$72,000	$15,000
. Assumed Surcharge by New Castle at $150/mg	-	-	22,000
Subtotal	0	$72,000	$37,000
3. Cost of Treatment:			
. Byram Lake Plant at $150/mg	$93,000	-	71,000
. New Castle Joint Plant at $100/mg	-	69,000	-
. New Castle Plant at $125/mg	-	-	18,000
Subtotal	$ 93,000	$ 69,000	$ 89,000
4. Average Debt Service on Capital Cost(4)	$470,000	$374,000	$346,000
Total Annual Costs	$609,000	$605,000	$522,000
$/MG	$878	$873	$756

(1) Approximate present power costs for pumping from Catskill Aqueduct into New Castle System.
(2) Includes cost of repumping all of diverted water out of Byram Lake to treatment plant.
(3) Assumed charge for illustrative purposes only. Actual off-peak rate to be negotiated.
(4) Average annual payment for 30 year bonds at 8% rate of interest.

FIGURE 4

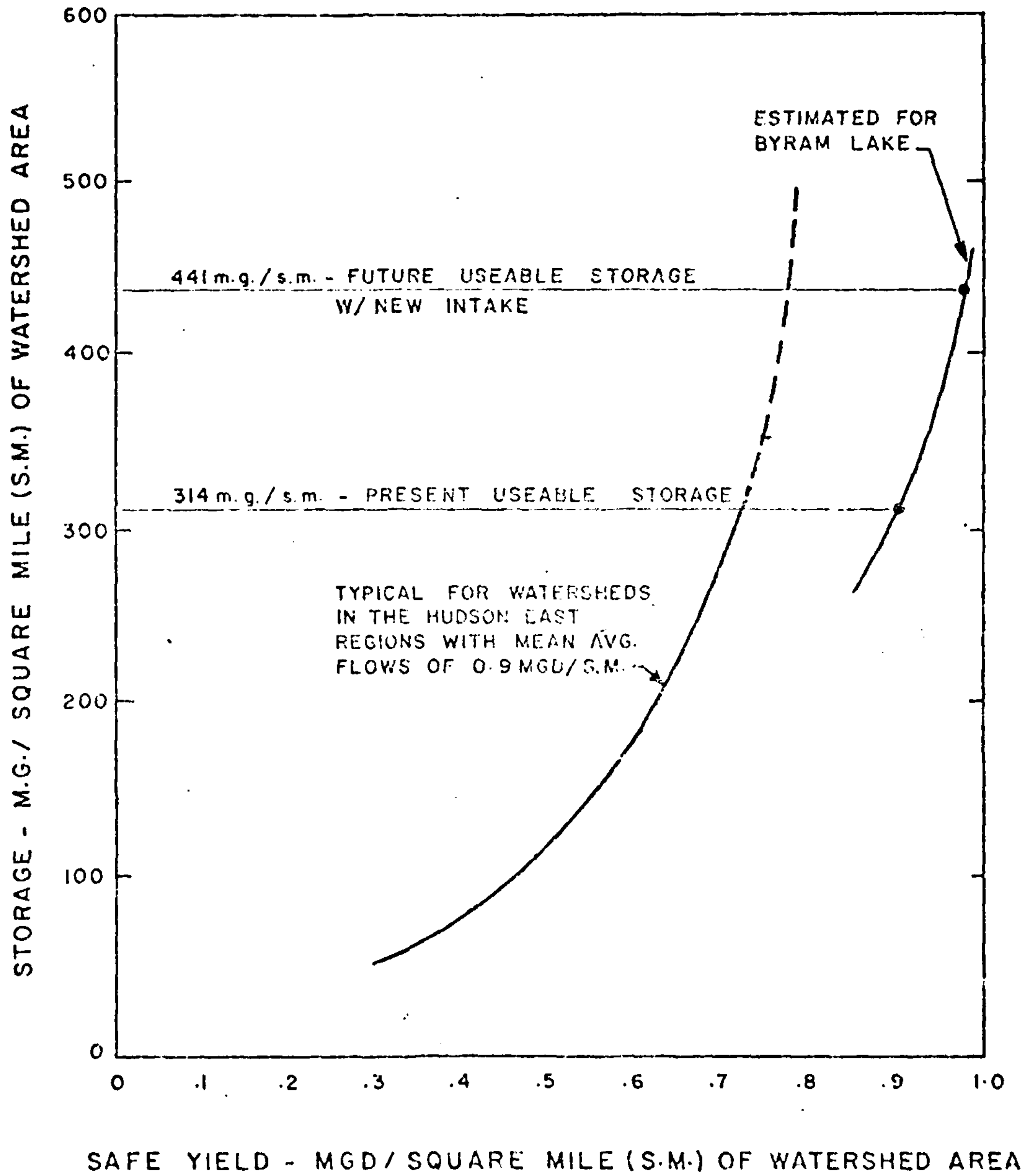


HAZEN AND SAWYER, P.C.
Engineers

VILLAGE OF MOUNT KISCO
WATER UTILITIES STUDY

POTENTIAL FUTURE
DRAINAGE AREA

FIGURE 3



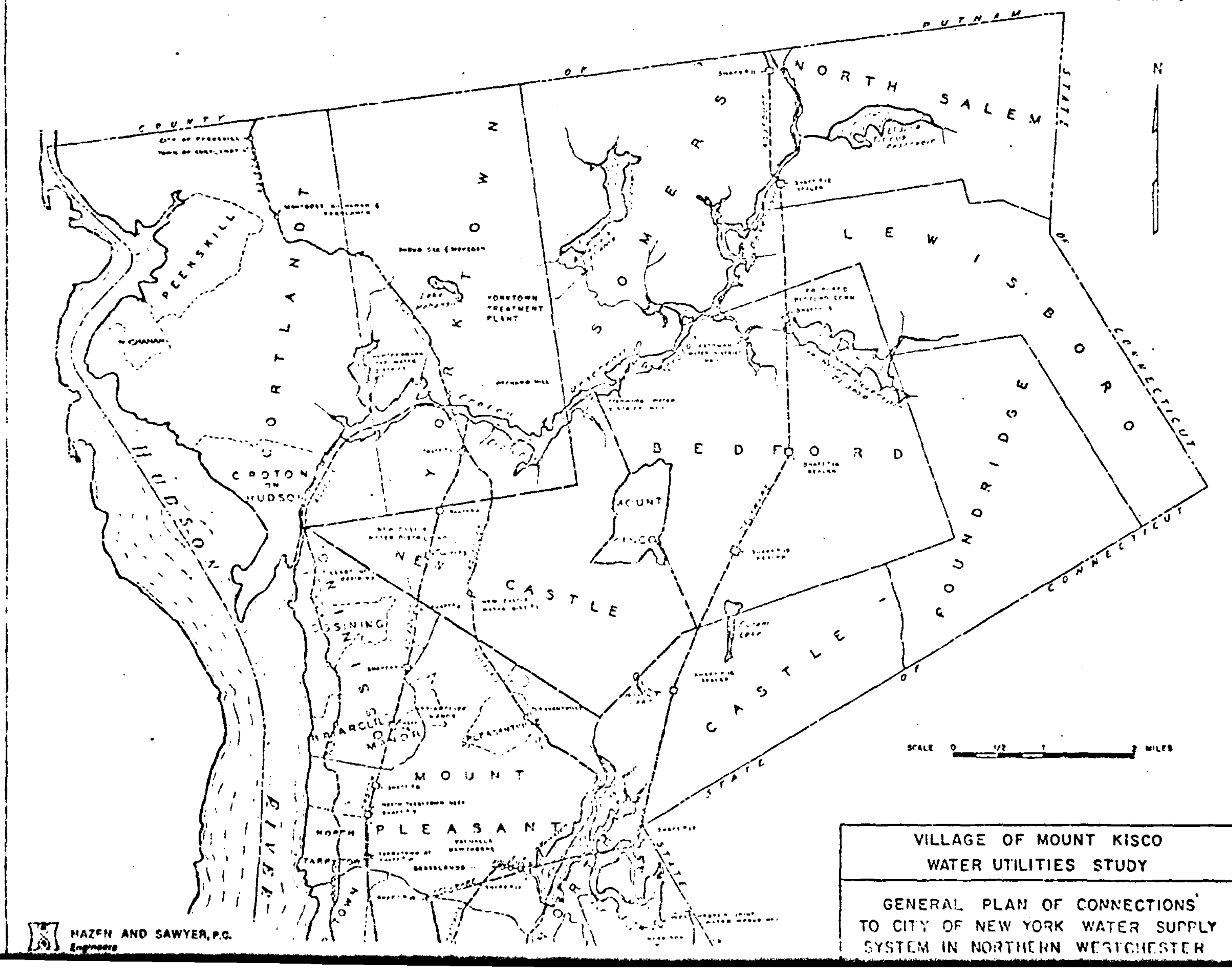
SOURCE: COMPREHENSIVE PUBLIC WATER SUPPLY STUDY FOR THE CITY OF NEW YORK AND COUNTY OF WESTCHESTER, 1967.

HAZEN AND SAWYER, P.C.
Engineers

VILLAGE OF MOUNT KISCO
WATER UTILITIES STUDY

BYRAM LAKE
STORAGE VS SAFE YIELD

FIGURE 2

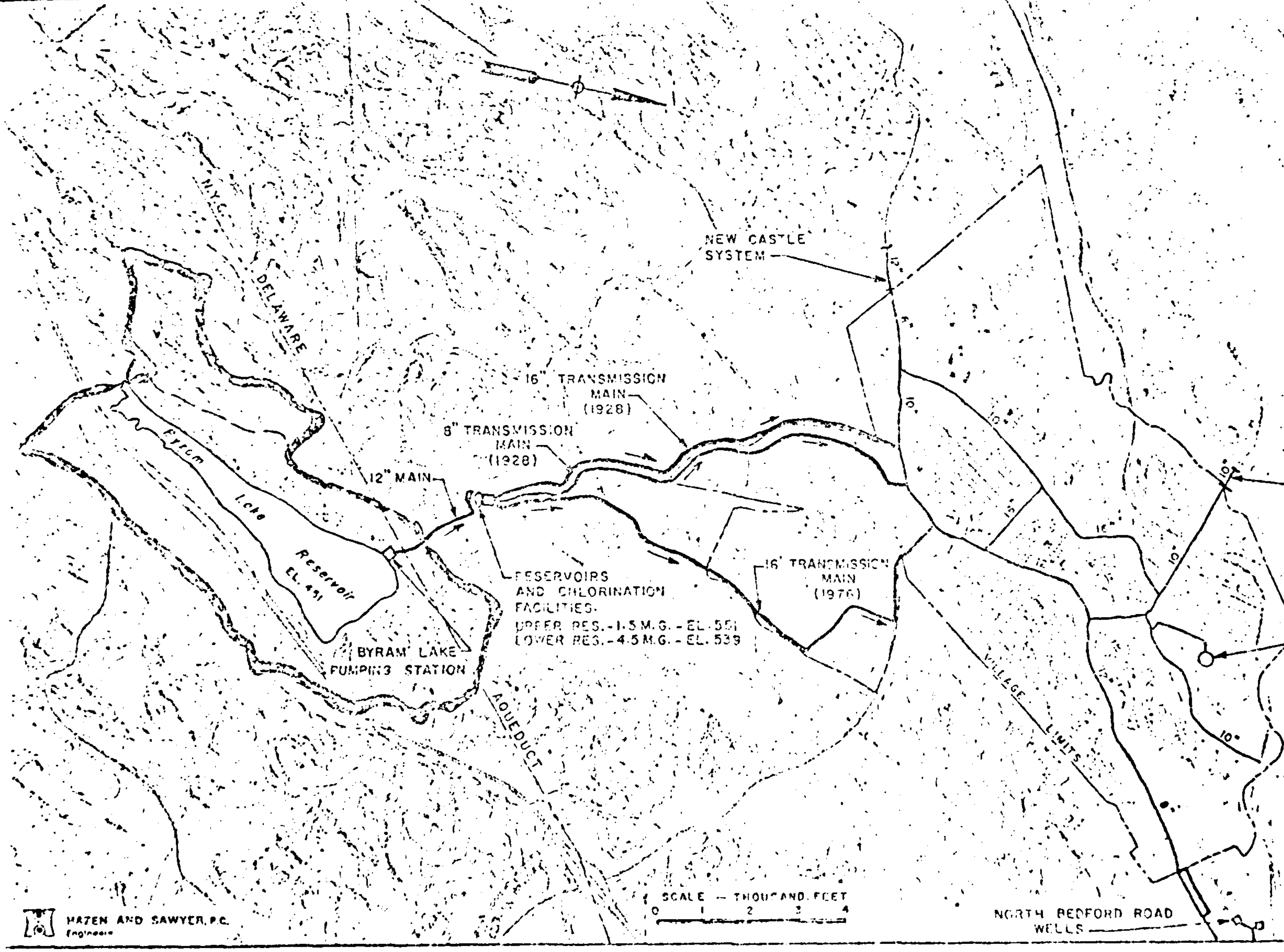


VILLAGE OF MOUNT KISCO
WATER UTILITIES STUDY

GENERAL PLAN OF CONNECTIONS TO CITY OF NEW YORK WATER SUPPLY SYSTEM IN NORTHERN WESTCHESTER

HAZEN AND SAWYER, P.C.
Engineers

FIGURE 1



VILLAGE OF MOUNT KISCO
WATER UTILITIES STUDY

EXISTING SOURCES OF SUPPLY

5. REQUIRED DISTRIBUTION SYSTEM IMPROVEMENT

Preliminary Analysis of the Distribution System

The Village distribution system consists of 4, 6, 8, 10, 12 and 16-inch mains, as shown on Figure 5. In analyzing the water distribution system, it has been considered that mains 10-inches and above (trunk mains) are generally required for water transmission to each localized area, while smaller mains are used for local water distribution and to carry fire flows.

Generally, a distribution system is judged upon its ability to: (1) meet peak customer demands at a reasonable pressure; (2) to provide adequate fire flows; and (3) to supply the projected needs of proposed new developments. A further consideration is that the structural condition of the distribution system should be such as to continue to meet these requirements in the forseeable future.

The existing Mount Kisco water distribution system is capable of meeting all present customer demands. Static pressure levels in the downtown area under normal flow conditions are at about 90 psi (pounds per square inch). In the areas of higher elevation in the low service zone, the static pressure levels are about 35 psi under normal flow conditions, sufficient for domestic purposes.

The capability to carry adequate fire flows is routinely determined for fire insurance purposes. The most recent survey for this purpose was conducted during September 1979 by the Insurance Service Office of New York. Municipalities are rated for fire protection on a scale from 1 to 10, with 1 the highest rating. The Village received an improved rating of Class 4 (formerly Class 5) which permits certain types of property to be eligible for reduced premiums on fire insurance policies.

The insurance survey provides the results of fire hydrant flow tests that were performed throughout the Village (see Exhibit 1). These tests were evaluated by Hazen and Sawyer to determine what improvements are required to increase inadequate distribution system fire flow pressures. In general, it was determined that the benefits brought about by the construction of the new 16-inch main from Byram Lake are not being extended to the northern portion of the system. To accomplish this, the trunk main system must be reinforced to serve the area along North Bedford Road. In addition, localized problems occur because of bottlenecks and because many of the 4-inch mains cannot supply required fire flows.

1. Trunk Main Extensions

As mentioned above, a trunk main is required to increase fire flow capacities in the northern section of the system. To meet this requirement, a 12-inch main extension is proposed from the 16-inch transmission main on South Bedford Road through the American Ultramar and Glass properties and connecting to the 12-inch transmission main on North Bedford Road. This main extension would provide substantial additional capacity in the system, as well as completing the transmission supply loop of 16-inch and 12-inch mains. As outlined in Report No. 1, it has been recommended that the Village discuss some cost sharing arrangement with the developers of the American Ultramar property for the easement and a 12-inch main to the Glass property. The Village would also have to work out an agreement with the developer of the Glass property to install a 12-inch main as recommended in Report No. 1.

On Emery Street, the 4-inch main between Croton Avenue and the 10-inch main to the high service storage tank should be replaced by a new 10-inch line. This would serve to both reduce pressure losses in those streets at a high elevation such as the north end of Croton Avenue and to strengthen local fire flow capabilities. The cost of the improvement is estimated at $70,000.

2. Improvements to Eliminate "Dead Ends" and "Bottlenecks"

. Connect "dead-ends" at Barker Street, Allan Street and Knowlton Avenue	700 ft. of 6-inch.
. Connect 10-inch and 4-inch mains on Lexington Avenue and Maplewood Drive	30 ft. of 6-inch.
. Connect 10-inch and 4-inch mains on Lexington Avenue at Gregory Avenue	30 ft. of 6-inch.

- Connect 8-inch and 6-inch mains mains on Quaker Place — 30 ft. of 6-inch.

The cost of these improvements is estimated at $50,000.

3. <u>Capital Improvements for Increased Fire Protection</u>

There are approximately 35,000 feet of old, unlined 4-inch mains presently in service in the Village. As part of a long term improvement program these mains should either be taken out of service or replaced with 6-inch and 8-inch cement lined pipe.

The existing 4-inch mains have been classified into two categories based on deficiency. The first group includes those mains which should be replaced to increase fire protection, while the latter group includes mains which should be replaced to improve overall system reliability. This group will be discussed in the next section.

Streets served by 4-inch mains that are considered to provide unsatisfactory fire protection are listed below. Discussions with the Water Department superintendent have determined that in each case there are known deficiencies.

Replace 4-inch mains with 6-inch or 8-inch mains as follows:

Armonk Road, Park Avenue and Fairways Drive	1,360 ft. of 8-inch.
Sarles Street, Highland Avenue and Dakin Avenue	1,930 ft. of 8-inch.
Orchard Street	1,520 ft. of 6-inch.
Sands Street	250 ft. of 6-inch.
Willetts Road	1,410 ft. of 6-inch.
Washburn Road	1,220 ft. of 6-inch.
Marion Road	830 ft. of 6-inch.
Manchester Drive	1,280 ft. of 6-inch.
Columbus Drive	600 ft. of 8-inch.
Terrace Place	380 ft. of 8-inch.
Green Street	700 ft. of 6-inch.
Grove Street	3,000 ft. of 6-inch.

The total length of the improvements is estimated at 14,480 feet and the cost at $800,000.

4. Capital Improvements to Increase System Reliability

There are approximately 23,000 ft. of 4-inch mains that are in service and which have not been proposed for replacement or abandonment because of deficient fire protection. Some 19,000 feet of these mains (such as the one on Lexington Avenue) are paralleled by a larger main. In these cases, service connections should be transferred to the larger mains and the 4-inch main should be abandoned. This will improve customer service pressures and eliminate any undetected leakage from the old mains.

The replacement of these mains is recommended for inclusion in a long term construction program. However, the priority for completing this work is considered to be the lowest of the proposed recommendations.

Replace 4-inch mains with 6-inch mains as follows:

Turner Road Ext.	300 ft.
Turner Road	250 ft.
Sands Street	300 ft.
Hillside Avenue	350 ft.
Barker Street	800 ft.
High Street	500 ft.
West Street	1,100 ft.

The length of the improvement is estimated at 3,600 feet and the cost is estimated at $250,000. The cost for moving about 190 service connection taps is estimated at $100,000.

5. Required Capital Improvements to Serve Proposed New Developments

It is not expected that proposed future development (see Tables 1 and 2) would involve abnormally high water demands for either normal consumption or to meet fire flows. With the exception of the back of the Glass above elevation 425 ft., existing trunk mains will provide sufficient capacity to meet projected demands. For large parcels, developers should be required to construct service lines to trunk mains rather than to smaller distribution mains. Development on the Glass property above the 425 foot elevation should be served by a hydroneunmatic system, to be owned and operated by the cooperative association (see Report No. 1).

<u>Recommended Phasing of Capital Improvements</u>

A phased capital improvement program for the construction of the proposed distribution system improvements is recommended. The improvements as shown on Figure 6, should be constructed in three, five year duration programs, the priority of which is set by the benefits to be provided.

Generally, the program shown in Table 7 is directed toward the initial construction of larger mains. As indicated, the proposed 12-inch main through the American Ultramar and Glass properties should be included as a Phase I project. The elimination of system bottlenecks should be also be undertaken during the first phase. Under the second phase, 4-inch mains which provide inadequate fire protection would be replaced, with the construction of the Emery Street transmission main and the replacement of the remaining 4-inch main conducted thereafter. It is further recommended that a program of eliminating existing service connections to 4-inch mains when these pipes are paralleled by larger pipes, be undertaken by Water Department personnel as soon as practicable.

<u>Need for Computer Modelling</u>

In a water distribution system it is possible to mathematically calculate the flows and pressure which can be expected under any given operational condition. Mathematical calculations are not generally performed manually since the number of repeat calculations necessary for a large distribution network can be literally in the hundreds of thousands. This type of rigorous analysis can be readily performed by a computer, which can mathematically simulate the physical characteristics of a distribution system.

Some of the major advantages for computer modelling are:

1. The computer is able to perform the necessary thousands of iterative calculations for a fraction of the cost of manual calculations.

2. Once the physical characteristics have been entered into the computer, many different flow conditions can be readily examined.

3. For a given day, observed actual conditions can be used to calibrate the model.

4. The impact of projected future flow conditions can be readily simulated.

However, the value of a computer model is directly related to the amount and accuracy of routine information that is used in setting it up. The following list, although not exhaustive, shows the types of information that would be needed:

Daily Diurnal Flow Records

Village officials should routinely maintain and keep flow charts for the supply to the system. When collected over the years, they can be used to observe gradual changes in the system as well as changes in demand, and track the volume of unaccounted water flows.

Spot Measurements of Hydrant Pressures

On a routine basis, the Water Department personnel should observe hydrant pressures under average flow conditions. When logged periodically on a map, these pressures become a valuable tool in spotting local distribution problems.

Records of Water Main Failures

In any distribution system, failures occur. During the temporary emergency that results, it is sometimes possible to observe the internal condition of the pipe involved. Long term periods of failure locations, causes, and pipe conditions are a valuable programs planning for the replacement of water mains and hydrants.

Records of Customer Complaints

Customer complaints about pressures or poor water quality provide an insight into the condition of the system. A log should be kept of each complaint.

A computer model of the Mt. Kisco water distribution system is not recommended at the present time, because to be valid, substantial amounts of field testing would be necessary. As a first step, the maintenance of records such as those listed above should be initiated. This will have the benefit of familiarizing the Water Department personnel with proper procedures and record keeping, improve the reliability of the model and reduce the cost of preparing the model.

TABLE 7

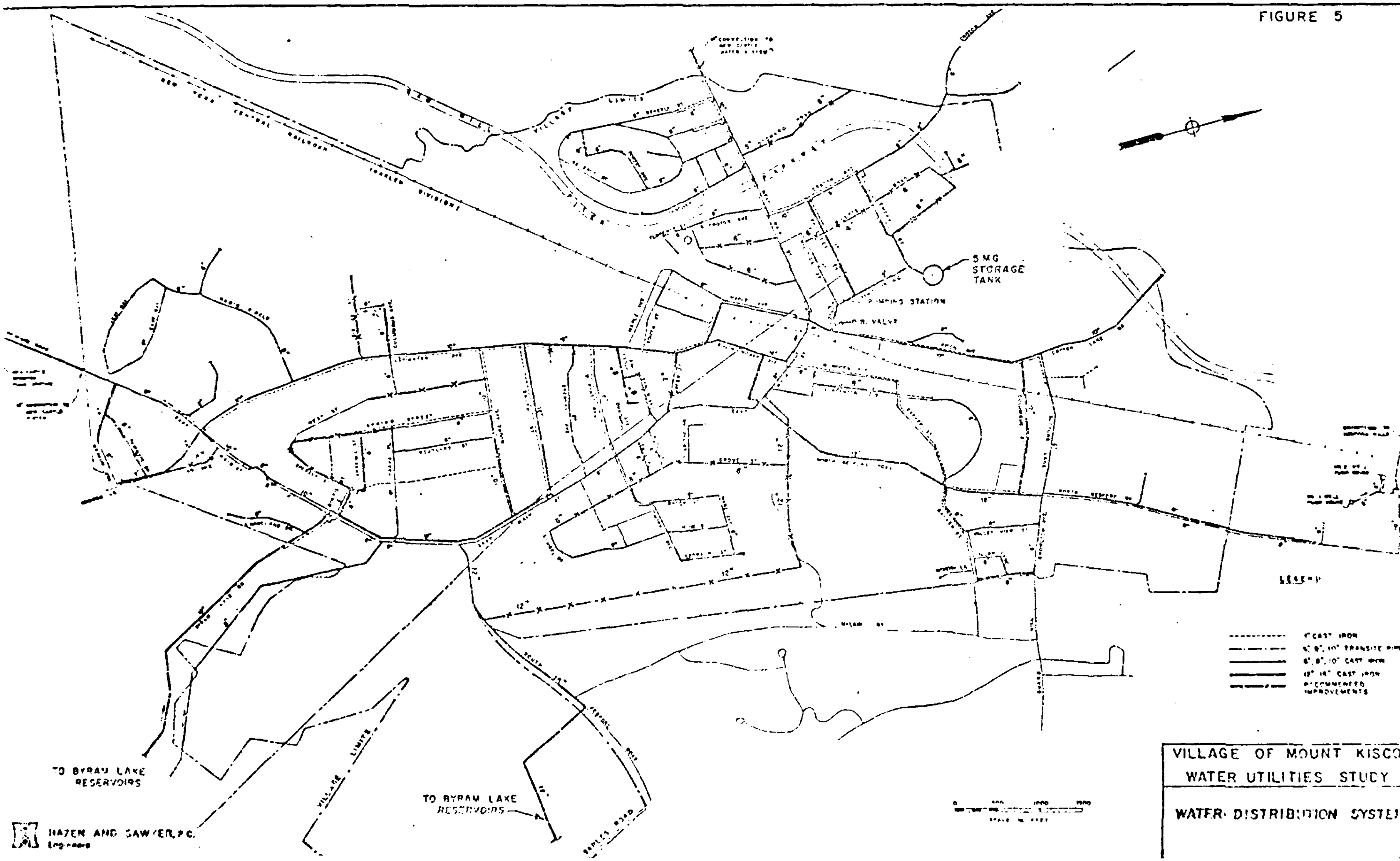
VILLAGE OF MOUNT KISCO

PROPOSED DISTRIBUTION SYSTEM IMPROVEMENTS

Project	Phase 1	Estimated Capital Cost (1981 Dollars) Phase 2	Phase 3	Total
1. Construction of New Trunk* Mains	$ 70,000	-	-	$70,000
2. Elimination of Bottle-Necks and Dead-Ends	$ 50,000	-	-	$50,000
3. Improvements to Increase Local Fire Protection	-	$800,000	-	$800,000
4. Improvements to Increase System Reliabiltiy	-	-	$350,000	$350,000
Total	$120,000	$800,000	$350,000	$1,270,000

*Does not include costs for 12-inch main through American Ultramar and Glass properties. These costs cannot be determined until after negotiations between Village and developer are completed.

FIGURE 5


5 MG
STORAGE
TANK
PUMPING STATION
TO BYRAM LAKE
RESERVOIRS
TO BYRAM LAKE
RESERVOIRS
VILLAGE LIMITS
VILLAGE OF MOUNT KISCO
WATER UTILITIES STUDY
WATER DISTRIBUTION SYSTEM
HAZEN AND SAWYER, P.C.
Engineers

6. WATER CONSERVATION MEASURES

To establish the potential savings which could be achieved by a water conservation program in Mount Kisco, the areas of major usage must be identified and measures for achieving a savings in consumption assessed. In the Village, residential customers constitute the major water demand, while Northern Westchester Hospital is the single largest consumer.

Water use conservation measures which can be considered include:

- The Installation of Water Saving Devices,
- Modification of the Water Rate Structure, and
- Reduction of Distribution System Leakage and Meter Repair.

Installation of Water Saving Devices

Recent studies by the Public Health Service and other researchers have indicated that in a typical home with traditional plumbing fixtures, the following water uses can be expected for a family of three:

	Use gpd	Percent of Total
. Dishwashing -	15	7
. Laundry -	35	17
. Drinking and Cooking -	10	5
. Bathing -	60	30
. Oral Hygiene -	5	3
. Toilet Flushing -	75	35
. Misc. Cleaning -	5	3
Total	205	100

Of the above uses, the greatest are for laundry, bathing and toilet flushing, which amount of 83 percent of the total. While water savings can be achieved by the customer in all three areas, flow reducing devices are only applicable for shower head, faucet, and toilet installations. Savings in laundering can be achieved by washing larger loads a fewer number of times.

Shower Heads and Faucets

Several devices are available which limit the maximum flow of water from shower heads and faucets. The simplest and least expensive of these devices are plastic orifices which can be inserted in shower head and faucet feed lines. The more expensive devices are newly marketed shower heads which use either mixing valves or are pressure compensating. Both types of devices are easily installed and are applicable to both new and retrofit installations.

Installed on shower heads, these devices are reported to limit the flow rate to 3.5 gpm, while on faucets they limit flow to 2.5 gpm. Total water savings can only be estimated as it is dependent on the available water pressure and individual habits. However, potential savings of from 30 to 50 percent have been reported using flow restrictors.

Toilets

A standard U.S. water closet with a four gallon tank uses about five to six gallons during a normal flushing cycle, considerably more water than is necessary. U.S. manufacturers are currently marketing shallow trap toilets which limit consumption to 3.5 gallons per flush, while European manufacturers are attempting to market units in the U.S. which limit consumption to less than 1 gallon per flush. If specified for new construction or replacement of existing fixtures, these units would decrease the water consumed for flushing by standard water closets from 50 to 85 percent.

By reducing the volume of the tank on standard water closets, significant water savings can be achieved from presently installed units at a minimum of expense. Using weighted plastic bottles or by installing toilet dams, savings of 0.5 to 1.0 gallons or about 15 percent per flush can be achieved.

Effectiveness of Water Savings Devices

The installation of water savings devices by residential and commercial customers can reduce consumption by up to 10 percent if installed on existing fixtures. However, public education is mandatory for a conservation program to be effective.

During 1980, the Township of East Brunswick, N.J. undertook a demonstration program to determine the effectiveness of water savings devices when retrofitted to existing fixtures. After informing about 560 homeowners that they were selected to participate in the program, packages of water savings devices were delivered by township employees. The packages, each of which cost the township $10 for materials, contained toilet tank dams, an orifice valve for a showerhead, faucet flow reducers and a brochure giving detailed installation instructions and estimated savings in water and heating costs from each device.

It was determined that a savings of from 10 to 15 percent of total consumption could be achieved for a family of four if all the devices were installed. However, even with an intensive educational effort, not all homeowners utilized the devices. Township officials concluded that flow reducing devices do significantly reduce demand, but that if a conservation program to be implemented, public education must be effective or severe shortages must exist..

If low flow shower heads, faucets, and shallow trap toilets are specified for new residential construction, savings of 20 percent could be achieved. Similar savings in commercial developments could also be realized if flush valve toilets are specified. Such requirements if written into the Village plumbing code should not be objectionable, as they would not significantly affect the cost of construction and the equipment is easily obtained.

Modification of the Water Rate Structure

The present water rates, which were adopted in 1977, includes a fixed ready to serve charge based on meter size and a usage charge based on metered consumption (see Table 8). The usage charge to all customers is based on a rate structure under which consumption is priced at a flat rate. Although a flat rate structure is a fair method of allocating costs, if further conservation would be necessary during a drought emergency, an escalating block rate structure could be adopted.

With an escalating block rate an initial block could be established at the present rate, with usage thereafter priced at a rate which will severely penalize over consumption. The initial block could be reduced to about 3000 cu.ft. per quarter, 250 gpd or 82 gcd for a family of three. Although fair to single family residential customers, the decreased initial block would unfairly penalize larger commercial and/or industrial customers. These large customers which can be distinguished by meter size, could be billed based on a higher initial block rate system.

It is to be noted that while the escalating block rate structure system would encourage conservation, it can lower revenue if rates are not adjusted to reflect decreased sales while it is in place. Before considering a modification to its present rate system, the Village should conduct a cost of service study to establish actual system expenditures and therefore, revenue requirements to support the water system if an escalating block rate structure was to be considered.

Elimination of Distribution System Leakage

The core of the Village distribution system was constructed in the 1920's and its overall tighiness is at best uncertain. As previously discussed, accurate meter records of water supplied from Byram Lake and the North Bedford Wells are unavailable for recent years, making it impossible to determine the loss of water due to leakage. The latest determination was made in 1966, at which time it was estimated that leakage was approximately 25 percent of total system demand.

In projecting future Village demand, it was assumed that leakage would be reduced from the 25 percent rate estimated in 1966 to about 15 percent by the year 2000. To achieve this goal, the Village will have to undertake an effective leakage detection and repair program. This could be accomplished by Village public works personnel, the county, or by an outside specialty contractor.

TABLE 8

VILLAGE OF MOUNT KISCO

EXISTING WATER RATE SYSTEM

1. Service Charge

Size of Meter	Quarterly Charge Inside and Outside Village
5/8"	$ 2.50
1"	6.90
1-1/4"	10.00
1-1/2"	12.50
2"	20.00
3"	39.40
4"	62.50
6"	125.00
8"	200.00

2. Usage Charge

Rate per 1000 cu.ft.		
Inside Village	Outside Village	Bulk Sales
$6.90	$13.80	$10.35

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