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NATIONAL DAM INSPECTION PROGRAM. TILLSON LAKE DAM (NY00083), LO--ETC(U)
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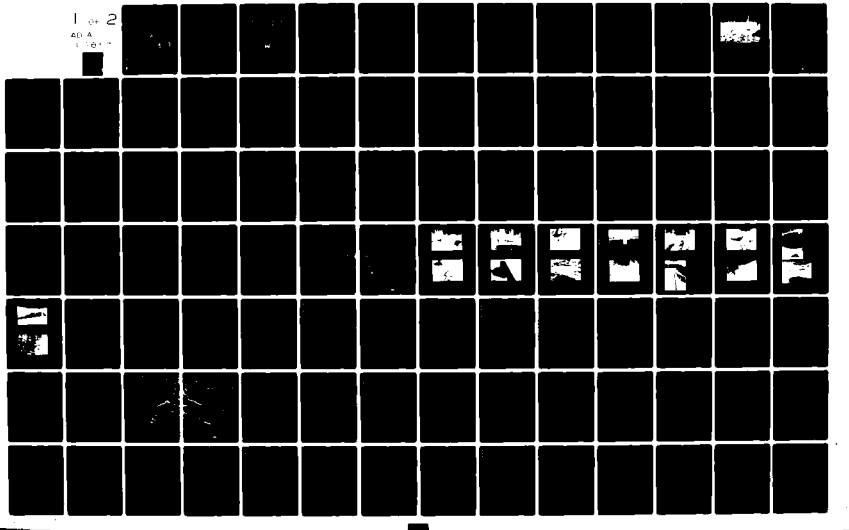

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Tillson Lake Dam
Ulster County
Lower Hudson River Basin

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

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

Examination of available documents and visual inspection of the dam did not reveal conditions which constitute an immediate hazard to human life or property. However, the dam has some serious deficiencies which require further investigation and remedial work.

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Hydrologic and hydraulic analysis indicates that maximum spillway discharge capacity without flashboards is only about 39% of the PMF peak outflow. The 1/2 PMF would overtop the earth embankment and would probably cause failure. Therefore, in accordance with Corps of Engineers' screening criteria for review of spillway adequacy, spillway capacity is considered "seriously inadequate" and the dam is assessed as "unsafe, non-emergency".

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean that there appears to be a serious deficiency in spillway capacity and if a severe storm were to occur, overtopping and failure of the dam could take place, significantly increasing the hazard to loss of life downstream of the dam.

Therefore, it is recommended that within 3 months after

after receipt of this report by the Owner, any appropriate remedial work should be completed. The detailed analysis and the design and construction observation of any remedial work should be done by a qualified, registered professional engineer.

In the meantime, the flashboards should immediately be removed from the spillway and kept removed pending the results of the detailed hydrologic and hydraulic analysis. Also, the Owner should immediately institute a program to visually inspect the dam and its appurtenances at least once a month. Within 3 months after receipt of this report the Owner should complete an emergency action plan outlining action to be taken to minimize the downstream effects of an emergency, together with an effective warning system.

Structural stability analysis of the spillway section indicates that it has unsatisfactory stability for all cases except normal spring and fall conditions (with flashboards removed) and that the right training wall is critically unstable for normal conditions. Therefore, it is recommended that a detailed structural stability analysis of the spillway section for all loading conditions be started within 3 months after receipt of this report by the Owner. This analysis should include investigation of foundation conditions, embankment loading conditions, and structural details. The large crack in the right training wall should be taken into account. Any necessary remedial work should be completed within 18 months after receipt of this report by the Owner. The investigation and the design and construction observation of any remedial work should be done by a qualified, registered professional engineer.

LOWER HUDSON RIVER BASIN

TOWN OF GARDINER

ULSTER COUNTY, NEW YORK

TILLSON LAKE DAM

NY 00083

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY

NEW YORK DISTRICT, CORPS OF ENGINEERS

26 FEDERAL PLAZA

NEW YORK, NY 10278

JULY 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 investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

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

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

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

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

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TILLSON LAKE DAM, NY 00083

PHASE I INSPECTION REPORT

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NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.:	NY 00083
Name of Dam:	Tillson Lake Dam
State Located:	New York
County:	Ulster
Municipality:	Town of Gardiner
Watershed:	Lower Hudson River Basin
Stream:	Palmaghatt Kill
Date of Inspection:	April 8, 1981

ASSESSMENT

Examination of available documents and visual inspection of the dam did not reveal conditions which constitute an immediate hazard to human life or property. However, the dam has some serious deficiencies which require further investigation and remedial work.

Hydrologic and hydraulic analysis indicates that maximum spillway discharge capacity without flashboards is only about 39% of the PMF peak outflow. The 1/2 PMF would overtop the earth embankment and would probably cause failure. Therefore, in accordance with Corps of Engineers' screening criteria for review of spillway adequacy, spillway capacity is considered "seriously inadequate" and the dam is assessed as "unsafe, non-emergency".

The classification of "unsafe" applied to a dam because of a seriously inadequate spillway is not meant to connote the same degree of emergency as would be associated with an "unsafe" classification applied for a structural deficiency. It does mean that there appears to be a serious deficiency in spillway capacity and if a severe storm were to occur, overtopping and failure of the dam could take place, significantly increasing the hazard to loss of life downstream of the dam.

36.3 Therefore, it is recommended that within 3 months after receipt of this report by the Owner, a detailed hydrologic and hydraulic analysis be started to better assess spillway capacity. This should include a more accurate determination of the site specific characteristics of the watershed. Within 18 months

after receipt of this report by the Owner, any appropriate remedial work should be completed. The detailed analysis and the design and construction observation of any remedial work should be done by a qualified, registered professional engineer.

In the meantime, the flashboards should immediately be removed from the spillway and kept removed pending the results of the detailed hydrologic and hydraulic analysis. Also, the Owner should immediately institute a program to visually inspect the dam and its appurtenances at least once a month. Within 3 months after receipt of this report the Owner should complete an emergency action plan outlining action to be taken to minimize the downstream effects of an emergency, together with an effective warning system.

Structural stability analysis of the spillway section indicates that it has unsatisfactory stability for all cases except normal spring and fall conditions (with flashboards removed) and that the right training wall is critically unstable for normal conditions. Therefore, it is recommended that a detailed structural stability analysis of the spillway section for all loading conditions be started within 3 months after receipt of this report by the Owner. This analysis should include investigation of foundation conditions, embankment loading conditions, and structural details. The large crack in the right training wall should be taken into account. Any necessary remedial work should be completed within 18 months after receipt of this report by the Owner. The investigation and the design and construction observation of any remedial work should be done by a qualified, registered professional engineer.

Because of other deficiencies, the following additional investigations should be started within 3 months after receipt of this report by the Owner. The investigations should be performed by a qualified, registered professional engineer.

1) Investigate the character of the spoil material on the downstream slope to determine whether it should be removed and, if so, provide the procedure for removal.

2) Determine whether major repairs should be made to the core wall, which has multiple cracks.

3) Investigate the origin of the seeps through the floor of the spillway discharge channel.

92.2 Any remedial work deemed necessary as a result of these investigations should be completed within 18 months after receipt of this report by the Owner. A qualified, registered professional engineer should design and observe the construction of any necessary remedial work.

The following remedial work should be completed by the Owner within 12 months after his receipt of this report. Where engineer-

ing assistance is indicated, the Owner should engage a qualified, registered professional engineer. Assistance by such an engineer may also be useful for some of the other work.

1) Remove trees and brush and their root systems from all surfaces of the dam and for 20 feet downstream of the toe in accordance with procedures established by an engineer. Continue to keep these same areas clear by cutting brush and trees and mowing grass at least annually.

2) Repair the eroded zones of the embankment adjacent to the spillway and along the upstream slope in accordance with a design by an engineer.

3) Monitor the seep adjacent to the outlet conduit and have the data evaluated in accordance with procedures established by an engineer.

4) Dewater and clean the outlet conduit and have it inspected by an engineer.

5) Restore the outlet conduit sluice gate to operation and exercise it regularly.

6) Contingent on the results of the detailed stability analysis by an engineer, repair the zones of eroded and deteriorated concrete of the spillway, discharge channel, and training walls in accordance with a design by an engineer.

7) Develop and implement effective routine operation and maintenance procedures for the dam and its appurtenances.

8) Institute a program of comprehensive technical inspection of the dam and its appurtenances by an engineer on a periodic basis of at least once every two years.

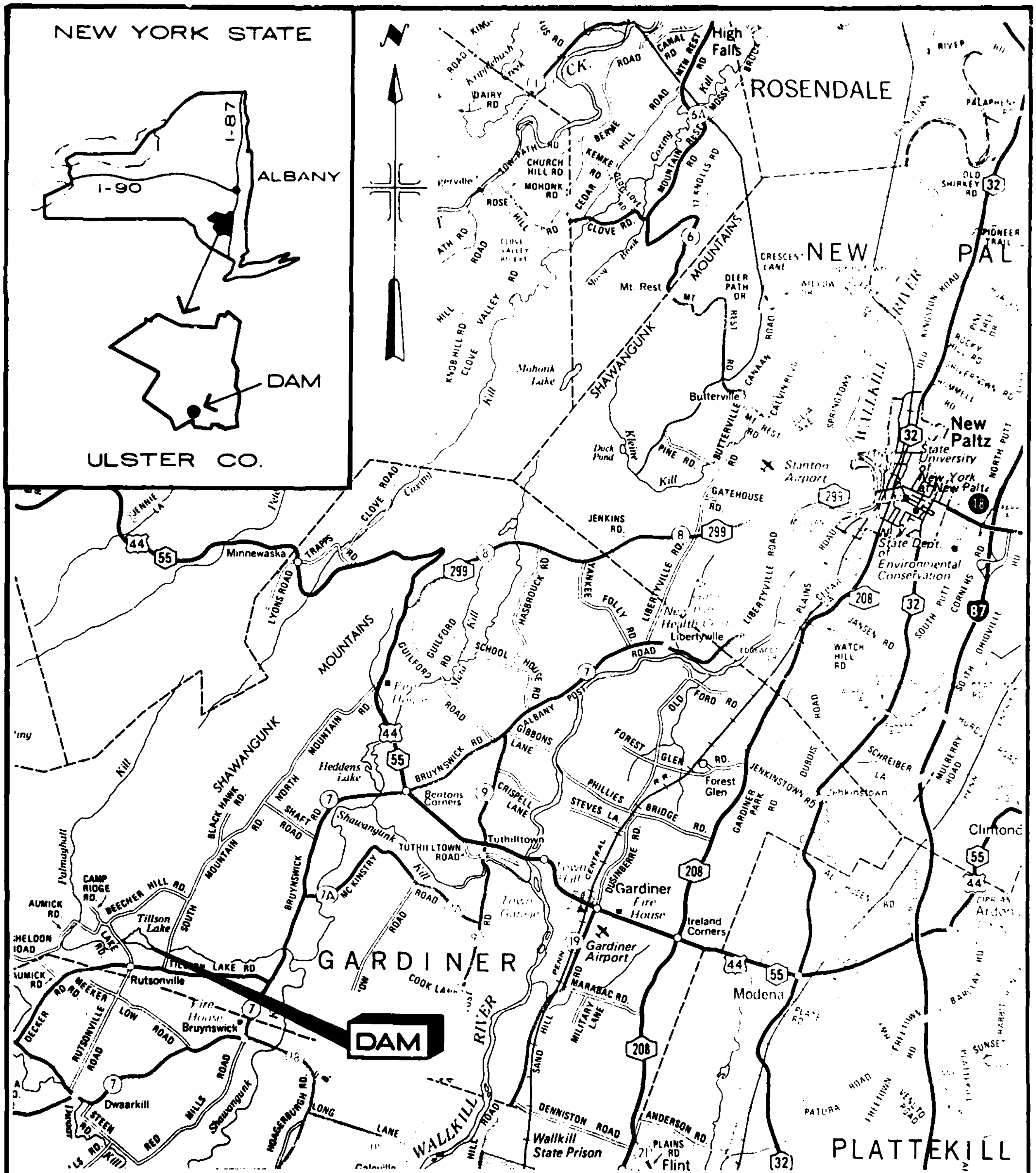
Approved by:

Kenneth J. Male
President
C. T. Male Associates, P.C.
NY PE 25004

Col. W. M. Smith, Jr.
New York District Engineer
Corps of Engineers

Date: 31 Aug 81

Overview Photo - Tillson Lake Dam. Note trees on slope and irregular crest - 4/8/81



SCALE OF MILES

0 1 2 3

BASE MAP: ULSTER CO. HIGHWAY MAP - 1977

PROJECT NO. 58.01.004/80.844

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TILLSON LAKE DAM
VICINITY MAP

TOWN OF GARDINER	ULSTER CO., NY
SCALE: 1" = 1.5 MILES	DATE JANUARY 1981

C. T. MALE ASSOCIATES, P. C.

3000 TROY ROAD, SCHENECTADY, N.Y. 12309

PROFESSIONAL ENGINEERS LAND SURVEYORS LAND PLANNING CONSULTANTS

NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

NAME OF DAM: TILLSON LAKE DAM, ID NO. NY 00083

SECTION 1

PROJECT INFORMATION

1.1 GENERAL

a. Authority

The National Dam Inspection Act, Public Law 92-367, August 8, 1972, authorized the Secretary of the Army through the Corps of Engineers to initiate a national program of dam inspection throughout the United States. The New York District of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within New York State. C. T. Male Associates, P.C., has been retained by the New York District to inspect and report on selected dams in the State of New York. Authorization and notice to proceed was issued to C. T. Male Associates, P.C., under a letter from Michael A. Jezior, LTC, Corps of Engineers. Contract No. DACW51-81-C-0014 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection

The purpose of the inspection program is to perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public, and thus permit correction in a timely manner by non-Federal interests.

1.2 DESCRIPTION OF PROJECT

a. Location

The dam is located on the Palmaghatt Kill about 10 miles southwest of the Village of New Paltz. The dam at its maximum section is at Latitude 41 degrees - 40.3 minutes North, Longitude 74 degrees - 14.8 minutes West.

Access to the dam is from County Route 7 to the east, then via Tillson Lake road to either South Mountain Road or Lake Road, and then via a private gravel road that runs south of the lake between South Mountain Road and the Tillson Lake Recreation Park and pavilion on Lake Road (see Vicinity Map, and Drainage Area Map Appendix C-5).

737 The official and popular name of the dam is Tillson Lake Dam and the official and popular name of the reservoir is Tillson Lake.

b. Description of Dam and Appurtenances

Tillson Lake Dam is an earth embankment with a central concrete core wall. The dam has an ogee-like spillway about 30 feet from the left abutment. The irregular brush, tree, and debris-covered embankment is about 308 feet long (including the spillway) by about 39 feet high. The upstream and downstream slopes are irregular with the upstream slope above the beach estimated at 1.5H:1V and with portions of the downstream slope as steep as 1.25H:1V. The upstream slope is covered with rock riprap to about 4 feet below the top of the dam. The dam has an irregular crest width averaging about 15 feet.

The dam has a reinforced concrete core wall about 18 inches wide at portions of the top which are exposed. The wall increases in section as it extends down to the original ground surface. The core wall is partially exposed on both sides of the spillway and the top of the core wall, at EL 376, has been considered to be the top of the dam.

The ogee-like spillway is a concrete gravity overflow section about 3.5 feet wide at the top with a crest length of about 55 feet. The spillway overflow section varies uniformly in downstream height from left to right, being 4 feet high on the left and about 20 feet high on the right. The spillway discharge channel is formed by training walls on each side of the spillway extending downstream about 80 feet. The channel bottom consists of concrete pavement over bedrock (and possibly over some hardpan) and it slopes downward left to right, as well as downstream (see Photo A-6A).

The spillway crest has 4 sections of 3-foot-high wooden flashboards, each section being 13.75 feet long. The sections are supported at their ends by 3 railroad rails embedded in the spillway crest (see Photo A-6B). The two end sections are each additionally supported by three 1.5-inch pipes and the middle sections by two 1.5-inch pipes. At the time of inspection one section of flashboards was removed. Also, just downstream of the weir crest there are 3 additional railroad rails embedded in the spillway overflow section which are apparently not used.

The dam has a concrete outlet conduit about 3 feet square on the inside and about 170 feet long. In the reservoir there is a concrete control tower with a floor stand control mechanism for a 30-inch-diameter sluice gate at the end of the conduit at the base of the tower. The handwheel for the control mechanism is missing and the gate is inoperable. The conduit discharges into the streambed at the downstream toe and presently is about half silted shut at its downstream end.

73. c. Size Classification

In accordance with Recommended Guidelines (Reference 1), Tillson Lake Dam is classified as "small" in size because its height is 39 feet (within the 25 to 40-foot range) and the maximum storage capacity at the top of the dam is 394 acre-feet (within the 50 to 1,000-acre-foot range).

d. Hazard Classification

In accordance with Recommended Guidelines (Reference 1), Tillson Lake Dam is classified as having a "high" hazard potential. This is because it is judged that failure of the dam would significantly increase flows downstream which could cause loss of more than a few human lives and appreciable property damage. Downstream development that could be damaged or destroyed by a dam failure includes: a home to the left of the stream about 300 feet downstream and the associated driveway bridge over the stream (both are visible in Photo A-9B), and several homes even closer and lower to the stream about 900 feet downstream near South Mountain Road (vertical drop from the dam to the homes near South Mountain Road is about 40 feet).

e. Ownership

It is suspected that the dam was constructed in the 1920's or early 1930's for Hassey A. Tillson. Presently the dam and reservoir are owned by:

U & U Realty, Inc.
100 Seaview Drive
Secaucus, New Jersey 07094

Attention: Joseph Uanue, President
(201) 348-4900

f. Operator

Day-to-day operation of the dam is the responsibility of:

Tillson Lake Recreation Park, Inc.
Gardiner, NY 12525

Attention: Henry S. Cuney, President
(914) 564-2718

and

George Surinach, Vice-President
(Mr. Cuney's Son)
35 Utterby Rd.
Malverne, NY 11565
(516) 887-7859

Tillson Lake Recreation Park, Inc. is the leasee of the property upon which the dam and the associated recreational facilities are located.

93 g. Purpose of Dam

The dam was originally constructed to impound water for recreational purposes. The impoundment is presently used for the same purpose by the Operator who runs a swimming beach and pavilion at the western end of the lake. The dam is at the eastern end of the lake.

h. Design and Construction History

It is suspected that the dam was constructed in the 1920's or early 1930's for Hassey A. Tillson. The original designer and construction contractor are unknown. No direct data concerning the original design or construction could be found.

On September 21, 1938 the dam was overtopped and a large section to the right of the spillway failed causing violent flooding and damage downstream. The spillway was completely flashboarded shut prior to and during the flood with 4-foot-high flashboards. In 1939 the dam was reconstructed for the original owner. The reconstruction consisted of repairing the breach in the core wall and embankment. In addition, the core wall and the dam to the right of the spillway were raised about 2 feet, new fill was placed over the entire downstream slope, and new flashboards 2.5 feet high were installed.

In August 1955 the portion of the dam to the left of the spillway, which had never been raised as intended during the 1939 work, was overtopped. A portion of the toe of the dam was washed away, and the spillway discharge channel was damaged. In 1956 major repairs were undertaken for the owner, Dominick Porco. The repair work consisted of raising the core wall and dam on the left side of the spillway to match the right side, rebuilding portions of the spillway training walls and discharge channel bottom, filling in washed out areas, and possibly adding riprap to the upstream slope. The sluice gate, then inoperable, was also supposedly repaired at that time.

Refer to Section 2 of this report, as well as to the Engineering Data Checklist in Appendix F2, for a complete discussion of the design and construction history. Drawings and other engineering data are included in Appendices F3 and G.

i. Normal Operating Procedures

The Operator visits the dam site at least twice a week during the summer and randomly at other times. The 3-foot-high

flashboards are up May through September and are normally removed by the Operator's son for the period of October through April. Last winter, however, only one of the four sections of flashboards was removed.

The outlet conduit sluice gate, normally closed, is inoperable. The control tower over the gate can only be reached by boat and the operating handwheel is missing. The sluice gate was last operated 15 or 20 years ago. The lake used to be drained for cleaning about every 10 years, but this was last done in the 1960's according to the Operator's son.

1.3 PERTINENT DATA

a. Drainage Area (square miles)	4.78
b. Discharge at Dam Site (cfs)	
Spillway (W.S. at top of dam)	
- with flashboards	950
- without flashboards	2,690
Outlet Conduit (normally closed and presently inoperable - estimated potential with W.S. at spillway crest w/o flashboards)	130
Maximum Known Flood (estimated at 2 ft. over flashboard crest in August 1955)	500

c. Elevation (feet - NGVD)

The elevation base of the reconstruction drawings in Appendix G is about 90 feet lower than NGVD (National Geodetic Vertical Datum of 1929) based on the water surface elevation listed in the Gazetteer of Lakes (Reference 25). USGS mapping shows no specific elevation on the water surface but is consistent with the Gazetteer elevation. Therefore, all elevations used in this report are 90 feet higher than those on the drawings in Appendix G and are in feet above mean sea level NGVD.

Top of Dam (top of core wall)	376
Design High Water (for 1,250 cfs)	373.5 ±
Spillway Crest - with flashboards	373
- without flashboards	370
Entrance Invert of Outlet Conduit	341 ±

d. Reservoir Length (feet) - at spillway crest	1,700 ±

e. Reservoir Surface Area (acres)	
Top of Dam	28.5 ±
Spillway Crest - with flashboards	25.6 ±
- without flashboards	22.7

f. Reservoir Storage (acre-feet)	
Top of Dam	394
Spillway Crest - with flashboards	312
- without flashboards	230

490

g. Dam

Type - Earth embankment.
Length - 308 feet including spillway.
Height - 39 feet.
Top Width - Irregular, averages about 15 feet.
Side Slopes - Upstream - 1.5H:1V above beach, 8H:1V on beach, below water presumed original 2.5H:1V.
- Downstream - Original 2H:1V. Present steepest slope of spoil over original surface is 1.25H:1V
Zoning - Homogeneous with central concrete core wall and miscellaneous spoil on downstream slope.
Impervious Core - Central concrete core wall (cracked and therefore not an impervious barrier) reported to be 28 inches thick at its base and 12 to 15 inches thick at its top (measures 18 inches thick on portions of top exposed).
Cutoff - Concrete core wall extends to bedrock or to hardpan.
Grout Curtain - None known.

h. Spillway

Type - Concrete ogee-like with 3-foot flashboards.
Length of Weir - 55 feet.
Upstream Channel - Reservoir immediately upstream of weir crest. Bottom of reservoir is silted up level with weir crest at each end of spillway.
Downstream Channel - About an 80-foot-long concrete paved channel with concrete training walls. Channel slopes down steeply toward right side as well as toward downstream.

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i. Outlet Conduit (reservoir drain)

Size - Reported 3 feet square by about 170 feet long (measures 3.8 feet wide at outlet).
Description - Concrete box culvert from control tower in reservoir, through dam to downstream toe. The downstream end of the conduit is about half silted shut.
Control - Reported 30-inch-diameter sluice gate at upstream end at base of control tower with floor stand on top of control tower. Control tower only accessible by boat, handwheel is missing, and sluice gate is inoperable.

SECTION 2

ENGINEERING DATA

2.1 DESIGN DATA

a. Geology

There is no geologic information available in the data for this dam. The following information was obtained from current geologic maps and publications (References 28 and 29), as well as from the site visit.

Tillson Lake Dam is located in the Hudson-Mohawk lowlands of the Valley and Ridge physiographic province in southeast New York State. Bedrock in the vicinity of the dam is shale, argillite, and siltstone of the middle Ordivician period (approximately 460 million years old). The dam is located on the eastern flank of generally flat-lying basin rocks that underlie the Catskill Mountains.

The bedrock at the dam is a slate or argillite with closely-spaced (approximately 1/2 inch) foliations. The strike and dip measured about 20 feet downstream from the dam on the right side of the discharge channel is N 30° E, 28° N. Since the dam is oriented northeast-southwest, the horizontal thrust of the dam tends to close the north-dipping foliation planes. (Note: In the application for repairs, dated May 12, 1939 (see Appendix F3-3), the bedrock was called slate and indicated to have a dip of 50°W. It is not known where this measurement was taken.)

There is no surficial geology map available for this site.

b. Subsurface Investigations

There are no subsurface investigations available for this dam site.

A drawing by Solomon LeFevre dated March 22, 1939 (see Appendix G-1) shows the dam to be founded on slate from Sta 0+00 to Sta 2+70 and on hardpan from Sta 2+70 to the right abutment. The same drawing shows that the spillway was founded chiefly on bedrock but that hardpan was left unexcavated under the central portion (looking in transverse section) of the spillway.

The visual inspection showed bedrock exposed about 10 feet to the right of the low-level outlet conduit, about at Sta 1+30. Also, till (hardpan) is evident on top of bedrock to the left of the spillway. A scarp in till is evident in a zone just downstream of the right side of the dam. This zone seems to be a former borrow area.

c. Dam and Appurtenances

It is suspected that the dam was designed and constructed in the 1920's or early 1930's for Hassey A. Tillson. The original designer is unknown. No direct data concerning the original design could be found.

There are no direct data available on the composition of the dam. In a letter by Mr. Fred Briehl dated September 27, 1938 (see Appendix F3-1) describing the failure of the dam, he referred to the fill in the dam as "dirt" and as "rock and dirt." In a drawing by Solomon LeFevre dated March 22, 1939 (see Appendix G-1) he refers to the old and the proposed fill as "earth fill."

Immediately downstream from the dam, on the right side, there are the remains of an old borrow pit, which may be the source of the fill for the embankment.

The top surfaces of the crest and downstream slope are very irregular. According to the Owner's representative, the lake has been drained in the past and the bottom cleaned. He indicated that the spoil may have been dumped on the crest and downstream shell.

2.2 CONSTRUCTION HISTORY

a. Initial Construction

The original contractor for the dam is unknown and no records concerning the actual construction of the dam and appurtenances are known to exist. A brief review of the construction history can be found in Appendix F2, Checklist for General Engineering Data and Interview with Dam Owner.

b. Modifications and Repairs

On September 21, 1938 the dam was overtopped and breached. The spillway was completely flashboarded shut prior to and during the flood with 4-foot-high flashboards. As a result of this condition water from the storm rose in the reservoir, and aided somewhat by high winds, proceeded to spill over the dam. The flow of water over the dam washed out the fill section on the downstream side of the core wall to the right of the spillway. The core wall, then unsupported on the downstream side, burst. A portion of the core wall about 90 feet wide at the top and 30 feet deep failed. A letter by Fred Briehl, dated September 27, 1981 (see Appendix F3-1) describes the dam failure and the resulting damage.

In 1939 the dam was reconstructed for the original owner, H. A. Tillson. An application for the dam's reconstruction, dated May 12, 1939, appears as Appendices F3-3 to F3-6.

The core wall was repaired with new reinforced concrete which was doweled to the old remaining concrete with 3/4-inch steel rods. The core wall and embankment were both raised about 2 feet on the right side of the spillway. The earth fill washed away by the breach was replaced and additional fill was added to the entire downstream slope of the dam to the right of the spillway. New 2.5-foot-high flashboards were also installed on the spillway, resulting in 3.5 feet of freeboard on the right side but only 1.5 feet on the left. The core wall to the left of the spillway was never raised during the 1939 reconstruction.

The engineer for the 1939 reconstruction was Solomon LeFevre, New Paltz, New York. Correspondence concerning the 1939 work can be found in Appendices F3-7 to F3-8. A drawing concerning the reconstruction of the dam appears as Appendix G-1. The construction contractor for the 1939 work is not known.

In August 1955 another storm and subsequent flood caused a portion of the dam and core wall to the left of the spillway, which had never been raised during the 1939 reconstruction, to be overtopped and the embankment was eroded. There appears to have been about 2-foot-high flashboards on the spillway just prior to and during the flood. The lower ends of the spillway channel training walls were overtopped by water flowing in the channel. This flooding eroded the ground area behind the left training wall and caused a portion of the earth dam at the toe, to the right of the spillway, to be washed away. A section of the right spillway training wall and part of the channel bottom were also undermined and washed away. A report on damage to the dam due to the storm can be found on Appendix F3-9.

In 1956 the dam was repaired and reconstructed for the owner, Dominick Porco. A listing of the proposed 1956 reconstruction, as well as an application for the dam's reconstruction in 1956, appear as Appendices F3-10 to F3-14.

The core wall and dam to the left of the spillway were raised about 2 feet to match the rest of the dam. The left training wall of the spillway was raised about 1.5 feet and extended downstream. Some concrete work to promote better flow may have been done in the trough along the right side of the spillway discharge channel. The downstream end of the right training wall and the downstream end of the concrete bottom of the spillway discharge channel were replaced. The washed out fill areas were also repaired. At this time additional riprap may have been placed on the upstream slope and the sluice gate was supposedly repaired.

45, The engineer for the 1956 reconstruction was T.W. ("Don") Westlake, P.E., Holmes Road, RD 1, Box 66, Newburgh, New York. Some spillway and flashboard computations, possibly done by Westlake and/or the State reviewers in 1956, appear on Appendices F3-15 to F3-

17. Drawings concerning the 1956 reconstruction can be found as Appendices G-2 and G-3. The construction contractor for the 1956 work is not known.

c. Maintenance and Pending Remedial Work

There are no known plans for any maintenance or remedial work on the dam by the Owner.

2.3 OPERATION RECORD

a. Inspections

There is no known record of inspection of the dam by the Owner.

One inspection report by the New York State Department of Environmental Conservation (NYS-DEC), dated April 23, 1973, was found (see Appendix F3-18). This inspection report indicated that concrete surfaces at the dam needed some minor repairs that could be undertaken as maintenance items. The growth of trees on the downstream slope and minor cracks in the concrete were noted. The report indicated that some periodic maintenance was being performed and that the dam was in good condition.

b. Performance Observations, Water Levels, and Discharges

There are no known records of performance observations or of routine water levels and discharges at the dam.

c. Past Floods and Previous Failures

On September 21, 1938 the dam was overtopped and breached. In August 1955 high water again overtopped and damaged the dam. The details of these failures have previously been discussed in Section 2.2b.

2.4 EVALUATION

a. Availability

As listed on Appendix F1, engineering data and records for the dam were available from the Dam Safety Section of the NYS-DEC. This data was reviewed, and copies of all the records found are included in chronological order in Appendices F3 and G. Appendix F2, Checklist for General Engineering Data and Interview with Dam Owner, also contains pertinent engineering information.

461 b. Adequacy

Available data consisted of drawings, letters, and a report concerning the two failures and subsequent reconstructions,

applications for reconstruction, and an inspection report. Such data as original design drawings, construction specifications, design calculations, record drawings, complete data on foundation and embankment soils, and operation and performance data were not available. The lack of such in-depth engineering data does not permit a comprehensive review. Therefore, the available data was not adequate by itself to permit an assessment of the dam.

c. Validity

Based on field observation and checking, some of the data is not valid. The flashboards now used at the dam are 3 feet high and do not resemble the 2.5-foot-high flashboards of the 1956 reconstruction design (see Appendix G-3), which were designed to trip at various water elevations. The present flashboards are shown in Photo A-6B.

The outlet conduit is reported as being 3 feet square (see Appendix F3-1), but actually measures 3.8 feet wide at its downstream end.

The dike shown on the 1956 reconstruction drawing to the left of the spillway (see Appendix G-2) is not apparent in the field.

The 1939 and 1956 reconstruction drawings (see Appendix G) show that the core wall was to be raised 2.5 feet, for a total height of 6.5 feet over the spillway crest. Measurements show that only about 2 feet was added, for a total height of 6 feet over the spillway crest.

The 1939 reconstruction drawing (see Appendix G-1) indicates that the downstream slope should be 2H:1V and the upstream slope 2.5H:1V. Measurements show that the downstream slope is very irregular and is about 1.25H:1V at its steepest portion. The upstream slope below the water level could not be estimated, but the beach is about 8H:1V and the slope above the beach is about 1.5H:1V.

SECTION 3

VISUAL INSPECTION

3.1 FINDINGS

a. General

Tillson Lake Dam was inspected on April 8, 1981. The inspection party (see Appendix B-1) was accompanied by Mr. George Surinach, Vice-President of Tillson Lake Recreation Park, Inc., leasee of the dam and lake, who represented the Owner. Mr. Surinach is the son of Mr. Henry Cuney, who is the normal Operator of the dam and is President of the Recreation Park. The weather was sunny and warm at the time of the inspection. The water surface was at about EL 370.2 or about 2 inches over the spillway crest. The Visual Inspection Checklist is included as Appendix B, while selected photos taken during the inspection are included in Appendix A and as the Overview Photo at the beginning of this report. Appendix A-1 is a photo index map.

b. Dam

There is no evidence of sloughs or slides of the embankment.

Cracked Core Wall - The core wall is cracked at its junction with the right spillway training wall, as shown in Photo A-3B. It is also cracked at Sta 1+07, 1+18, 1+35, 1+50, and 1+60. The core wall was covered with soil beyond Sta 1+75.

These cracks probably render the core wall ineffective as a barrier to seepage through the embankment. That is, observation wells on each side of the wall and installed at the same depth would probably show practically the same water level on both sides of the wall. For this reason, this dam should be considered essentially an earth dam without a core wall for the purposes of judging its susceptibility to piping, until further information is available.

The core wall does serve the function of halting further erosion if the upstream slope is eroded away. It also would considerably retard any breach that might begin to form during an overtopping. The concrete core wall is also a positive barrier to any animals burrowing in the embankment. One woodchuck hole was observed at about midheight on the downstream slope among boulders at about Sta 1+70.

Trees and Shrubs - The crest, the entire downstream slope, and the upstream slope along the normal water line are all fully

forested with trees and shrubs, as shown in the Overview Photo and Photo A-2A. This vegetation prevents any effective observation of seeps that may occur through the embankment.

465 Spoil on Crest and Downstream Slope - The drawing for repair of the dam in 1939 (see Appendix G-1) shows a downstream slope of 2H:1V. The measured downstream slope at its steepest portion is 1.25H:1V (see Photo A-3A). Also, the crest is at a higher elevation along much of its length, by about 2 feet, than is shown in the 1939 and 1956 reconstruction drawings. At about Sta 1+70 to 2+20 on the downstream slope there is a pile of discarded boulders, also shown in Photo A-3A.

It is probable that this steeper-than-designed downstream slope and higher crest were built up from spoil removed from the bottom of the lake. This spoil material, if it became saturated by leakage through the core wall, high rainfall, or minor overtopping, would be unstable. In addition, if the spoil is less pervious than the embankment, it could act as a cap on the downstream slope, preventing proper drainage and reducing stability.

Surface Erosion - Extensive erosion of the upstream slope has occurred both to the left and to the right of the spillway (see Photos A-2A and A-7A). This erosion has not proceeded further into the dam due to the core wall.

The entire upstream slope is wave-cut at the normal lake level and a beach has formed in the riprap.

Soil has been extensively eroded from the downstream toe of the dam to the right of the right spillway training wall (see Photo A-4A). This latter erosion may have occurred during the flood in 1955 and was not repaired, or it may have been repaired and re-eroded.

Seepage - One seep was observed on the downstream side. It was exiting from a point 18 inches below and 12 inches to the right of the low-level outlet conduit (shown as an iron-stained zone on the left in Photo A-4B). This seep was clear, running at 6 to 10 gpm, and appeared to be exiting from the top of bedrock.

Some dampness was observed at the level of the toe of the pile of boulders on the downstream slope.

c. Appurtenant Structures

1) Intake Structure and Control Tower

The intake structure and control tower are one and the same concrete structure located upstream of the dam, in the reservoir, surrounded by water (see Photo A-5A). Only the upper part of the control tower was visible for inspection. The lower part of the tower and the intake structure were submerged.

13 From what was readily visible from shore, the control tower is in poor condition. The concrete is eroded and stained. The brackets for the slide gate control mechanism are also rusted and appear to be loose.

On top of the control tower there is a control mechanism (see Photo A-5A) for the 30-inch-diameter slide gate on the outlet conduit. The gate stem, which runs up the downstream side of the tower, and the control mechanism are rusted and in poor condition. The control mechanism is inoperable, according to the Operator's son, and its handwheel is missing. The slide gate is presently closed and has not been operated for a number of years.

2) Outlet Structure and Outlet Conduit

The outlet structure consists of just the exposed end of the square outlet conduit (see Photo A-4B). The outlet structure concrete is scaling and discolored. The outlet conduit is also silted in to about one-half its normal depth as far upstream as could be seen. The remainder of the inside of the outlet conduit was not observable.

3) Spillway and Discharge Channel

The spillway is at the left side of the dam looking downstream (see Overview Photo). The spillway consists of a concrete ogee-like weir section with flashboards and a concrete discharge channel with concrete training walls (see Photos A-5B, A-6A, and A-6B). In general, the concrete of the spillway and discharge channel is in fair to poor condition.

The upper one foot of concrete of the left training wall is crumbling (see Photos A-6B and A-7A). There is deterioration of the cold joints as well as cracking and efflorescence of the concrete. Available records indicate that the top of the left training wall was raised during the 1956 reconstruction (see Section 2.2b).

The right training wall has a crack its full height, at about the toe of the ogee section (see Photo A-8A). There is a large spall at the right training wall contact with the downstream side of the ogee crest (see Photo A-7B) and erosion of this training wall at the water line on the upstream side near the spillway (see Photo A-2A). There is erosion of the concrete along the base of the right training wall due to the flow of water (see Photo A-8A). Efflorescence, minor cracking, and the location of cold joints can be seen in Photos A-7B and A-8A.

454 The face of the ogee-like spillway section is eroded and the concrete is spalling and scaling (see Photo A-7B). There is also considerable erosion and spalling of the spillway-to-channel bottom transition joint (see Photo A-6B) and the erosion is as much as one foot deep in some places.

The discharge channel slab is cracking (see Photo A-8B) and there are spalls along its construction joints. The channel concrete along the base of the right training wall is also eroding due to flow which concentrates there. There are also two seeps through cracks in the concrete floor of the spillway discharge channel. One seep is flowing clear at about 1 gpm and the other is flowing clear at about 25 gpm (see Photo A-8B). There are iron stains where the seeps exit. The seepage may be entering the pavement through cracks higher up on the floor of the spillway.

d. Reservoir Area

The reservoir area is grassed or forested with hardwoods (see Photo A-9A). Slopes are gentle and there was no indication that excessive erosion or slope failures into the reservoir might occur.

e. Downstream Channel

The downstream channel (see Photo A-9B) is a continuation of the Palmaghatt Kill starting from the toe of the dam and the downstream end of the spillway discharge channel. Downstream of the dam the Palmaghatt Kill is a somewhat rocky channel that is wooded along both sides.

3.2 EVALUATION

The cracked core wall makes it necessary to assume that the core wall is absent for the purpose of evaluating potential piping.

The spoil material that was placed on the downstream side of the dam probably was merely dumped loosely. If it becomes saturated due to seepage, high rainfall, or minor overtopping, it is likely to fail since it was dumped to a slope of 1.25H:1V.

The spoil, trees, brush and the boulders on the downstream slope make it impossible to inspect the slope adequately.

455 Erosion that is occurring adjacent to the spillway on the upstream and downstream side should be repaired. Also, the wave-cut upstream slope should be repaired by replacing riprap and removing the trees and brush.

The outlet pipe slide gate does not work. It should be repaired and then exercised regularly.

The outlet conduit should be dewatered, cleaned, and then inspected to ascertain its condition.

The large vertical crack in the right training wall should be checked periodically for possible worsening condition.

The seeps through the spillway discharge channel floor should be investigated further when the water level is below the spillway crest to try and find their origin.

The zones of eroded and deteriorated concrete of the discharge channel, spillway crest, and training walls should be repaired.

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SECTION 4

OPERATION AND MAINTENANCE PROCEDURES

4.1 OPERATION PROCEDURES

There are no written operation procedures for the dam.

Tillson Lake is used for recreational purposes. The outlet conduit sluice gate is normally shut. The dam has 3-foot-high flashboards which are in place from May through September (essentially the summer season) and are normally removed by the Operator's son for the period of October through April (fall-winter-spring).

At the time of inspection on April 8, 1981 the lake level was about 2 inches higher than the concrete weir crest with outflow estimated to be about 5 cfs. Three of the four 13.75-foot-long sections of flashboards were still in place on the weir crest since only one section had been removed prior to this last winter.

4.2 MAINTENANCE OF DAM AND OPERATING FACILITIES

There are no maintenance procedures for the dam.

The Operator visits the dam site at least twice a week during the summer and randomly at other times. The outlet conduit sluice gate, normally closed, is inoperable. It can only be reached by boat and the operating handwheel is missing. The sluice gate was reportedly last operated 15 or 20 years ago. In about 1979 a diver casually looked at the sluice gate while looking for a lost watch. The Operator's son indicated that the diver verbally reported that there was a buildup of debris in front of the gate and that it was corroded.

The lake used to be drained for cleaning about every 10 years, but this was last done in the 1960's according to the Operator's son. The flashboards are replaced or repaired as required.

4.3 EMERGENCY ACTION PLAN AND WARNING SYSTEM

There is no emergency action plan and warning system for the dam.

4.4 EVALUATION

Maintenance of the dam is unsatisfactory. The condition of the dam and its appurtenances seems to indicate that it receives little to no routine maintenance. Large trees, brush, and debris cover the upstream and downstream slopes. There is erosion damage to the upstream slope near the right training wall of the spillway. The outlet sluice gate is in a state of disrepair and the downstream

end of the outlet conduit is silted in for half its depth. Effective operation and maintenance procedures need to be developed and implemented by the Owner in order to avoid continued deterioration of the dam.

The Owner should develop an emergency action plan outlining action to be taken to minimize the downstream effects of an emergency, together with an effective warning system.

SECTION 5

HYDROLOGY AND HYDRAULICS

5.1 DRAINAGE AREA CHARACTERISTICS

Tillson Lake Dam and Tillson Lake are located on the Palmaghatt Kill in southeastern New York. About 1.5 miles downstream of the dam, the Palmaghatt Kill joins the Shawangunk Kill. The Shawangunk Kill drains to the northeast into the Wallkill River, which in turn drains to the Rondout Creek. The Rondout Creek flows east and discharges into the Hudson River at Kingston.

The total drainage area at the dam is about 4.78 square miles, of which about 0.035 square miles (22.7 acres), or less than one percent, is actual reservoir surface at the spillway crest (see Appendix C-6). Being in the Shawangunk Mountains, the topography is characterized by slopes of from 10% to 25%. Elevations in the drainage area vary from EL 370 to EL 2180.

5.2 ANALYSIS CRITERIA

The U.S. Army Corps of Engineers Hydrologic Engineering Center's Program HEC-1 DB (Reference 3) was used to develop the test flood hydrology and perform the reservoir routing.

The purpose of this analysis was to evaluate the dam and spillway with respect to their surcharge storage and spillway capacity. Accordingly, it was assumed that the water surface was at the spillway crest, with flashboards removed (normal fall-winter-spring condition), at the start of the flood routing. In addition, the outlet conduit was assumed to be closed, as it is normally. The outlet conduit gate is presently inoperable anyway.

A constant base flow of 2 cfs per square mile was chosen to represent average conditions in the drainage area and was inputted into the program for all subareas.

The index PMP (probable maximum precipitation) input to the HEC-1 DB program was 21 inches for a 24-hour duration all-season storm over a 200-square-mile basin, according to HMR 33 (Reference 4). Maximum 6-hour, 12-hour, and 24-hour precipitation for the actual size of the drainage area (same for 10 square miles or less) were inputted to the program as percentages of the index PMP in accordance with HMR 33. A storm reduction coefficient was then applied internally by the program in order to transpose or center the storm over the actual total drainage area. Thus, the corrected 24-hour PMP for the actual total drainage area became 22.2 inches. All rainfall was distributed using the Standard Project Storm arrangement embedded in the program. (Note: Only a 24-hour PMP was modeled. If a 48-hour PMP had been used, as is customary, the corrected 48-hour PMP would have been 23.9 inches,

inflow to the reservoir would have been slightly more, and spillway capacity would have been slightly more inadequate than shown by the analysis in this report.)

76 Appendix C-8 summarizes the subarea, loss rate, and unit hydrograph data input to the program. Only two subareas were used. Subarea 1 consists of all the drainage area around the reservoir, and Subarea 2 consists of just the reservoir surface. For the land in Subarea 1, loss rates were assumed to be 1.0 inch initially and a constant 0.1 inch per hour thereafter. Snyder unit hydrograph parameters were chosen from the 1977 Lower Hudson River Basin Flood Routing Model (Reference 20). A conservative standard lag time was computed. The program uses the inputted lag time and Snyder peaking coefficient to solve by iteration for approximate Clark coefficients, which are then used to calculate the runoff hydrograph.

For the reservoir surface making up Subarea 2, loss rates were set to zero so that rainfall would equal rainfall excess, or runoff. Assuming no delay in the rainfall/runoff response, a constant unit hydrograph for a rainfall duration equal to the HEC-1 DB calculation interval was developed per Appendix C-8 and inputted to the program.

The floods selected for analysis were full and 1/2 PMF (probable maximum flood). Floods as ratios of the PMF (e.g., 1/2 PMF) were taken as ratios of runoff, not of precipitation. Peak inflow for the PMF is about 6,870 cfs or 1,437 csm (cfs per square mile). Peak outflow is not reduced at all by reservoir routing and is the same as peak inflow. For 1/2 PMF the peak inflow is about 3,440 cfs (720 csm) and the routed peak outflow is about 3,430 cfs (718 csm).

5.3 RESERVOIR CAPACITY

Storage capacity for the reservoir, assumed to be at the spillway crest without flashboards, EL 370, was obtained from applications for the two reconstructions of the dam in 1939 and in 1956 (see Appendicies F3-3 & F3-11). USGS contour mapping (see Appendix C-5) was used to obtain area measurements inside contour elevations above the spillway crest and the capacity of the reservoir for these areas was computed by the method of conic sections. A hand tabulation of the reservoir volumes inputted to the program is on Appendix C-6.

At the spillway crest without flashboards, EL 370, the reservoir has a capacity of 230 acre-feet. At the spillway crest with flashboards, EL 373, the reservoir has a capacity of 312 acre-feet. At the top of dam, EL 376, the reservoir has a capacity of 394 acre-feet. Maximum surcharge storage between the spillway crest without flashboards and the top of dam amounts to 164 acre-feet, or about 0.6 of an inch of runoff from the total 4.78-square-mile drainage area. Therefore, the reservoir has almost no capacity to attenuate peak inflow.

5.4 SPILLWAY CAPACITY

The dam has a 55-foot-long concrete ogee-like spillway. During the summer the spillway is used with 3-foot-high flashboards, but for modeling purposes the flashboards were assumed not to be in place as is normal during fall, winter, and spring. The top of the dam is about 6 feet higher than the spillway crest without flashboards.

The discharge capacity for the service spillway was computed assuming critical flow over a sharp-crested weir. Since the spillway weir is not a true ogee and has a shallow approach depth due to silt buildup, the sharp-crested weir approximation is considered adequate for this analysis. Reduction in discharge capacity due to abutment contractions was neglected. The spillway discharge computations are presented on Appendix C-7. With water 6 feet over the spillway crest without flashboards (i.e., water level at top of dam) the spillway discharges about 2,690 cfs. With the 3-foot flashboards in place and the same water level at top of dam, the spillway discharge is reduced to about 950 cfs.

The 1956 application for reconstruction of the dam (see Appendix F3-12) indicates that the spillway was designed to safely discharge 1,250 cfs at a pool level 3.5 feet above the spillway crest. Present discharge computations in this analysis are slightly more conservative and show that a pool level of about 3.6 feet above the spillway crest is required to achieve the design discharge.

Total discharge from the dam consists of just flow from the spillway. As discussed previously in Section 5.2, the capacity of the outlet conduit was neglected since it is normally closed and presently inoperable. The weir parameters for the service spillway were inputted to the HEC-1 DB program which did the spillway discharge calculations during the flood routing.

5.5 FLOODS OF RECORD

As noted in Section 2.3c, the dam was overtopped and breached by a flood on September 21, 1938, and again overtopped and damaged by a flood in August 1955. The spillway was flashboarded just prior to both flood events and available records imply that the boards did not fail during either flood. For the 1938 flood, it appears that there were 4-foot-high flashboards level with the top of the dam and water spilled over a portion of the top of the dam. The depth of flow over the top is unknown. For the 1955 flood, the maximum pool level is reported in the 1956 Engineer's Report (see Appendix F3-9) to have been about 2 feet over the flashboards (boards appear to have been about 2 feet high at that time). Using the spillway capacity data developed in Section 5.4, the corresponding flood discharge in 1955 is estimated to have been about 500 cfs (105 csm), or only about 7% of the PMF peak outflow predicted. 500 cfs is probably equal to or greater than the 1938 flood, since during that event 500 cfs would have required a depth of flow over the entire top of dam and flashboarded spillway of about 8 inches.

5.6 OVERTOPPING POTENTIAL

The results of the overtopping analysis using the HEC-1 DB program are summarized in Table 5.1. The overtopping analysis computer input and output for the PMF and 1/2 PMF are included starting on Appendix C-9.

As noted from Table 5.1, the PMF overtops the dam by about 2.2 feet maximum with duration of overtopping of about 7.2 hours. 1/2 PMF also overtops the dam but only by 0.6 of a foot maximum with duration of overtopping of about 3.3 hours. Peak inflows are 6,870 cfs for the PMF and 3,440 cfs for 1/2 PMF. PMF peak outflow is the same as inflow, while 1/2 PMF peak outflow is reduced slightly by reservoir routing to 3,430 cfs. Time to maximum stage, or the time from the start of the 24-hour storm to peak outflow, is about 20 hours for both PMF and 1/2 PMF. The peak portion of the inflow and outflow hydrographs for the PMF and 1/2 PMF are shown by the computer plots on Appendices C-15 and C-16. Total project discharge capacity at the top of dam is due only to the spillway (no flashboards, outlet conduit closed) and is about 2,690 cfs, or only about 39% of the PMF peak outflow and about 78% of the 1/2 PMF peak outflow.

Tillson Lake Dam was also modeled with the 3-foot-high flashboards in place on the spillway. For this case the total project discharge capacity at the top of dam is only about 950 cfs, or only about 14% of the PMF peak outflow. The PMF overtops the dam by about 3.0 feet and 1/2 PMF overtops the dam by about 1.6 feet. The computer input and output are included starting on Appendix C-17 and the results are summarized by footnote (e) on Table 5.1.

5.7 EVALUATION

Maximum spillway discharge capacity without flashboards is only about 39% of the PMF peak outflow. The 1/2 PMF would overtop the earth embankment and would probably cause failure. The dam has failed completely due to overtopping once in the past in 1938 causing violent flooding and damage downstream. It is judged that failure due to overtopping would significantly increase the hazard to loss of life downstream from that which would exist just prior to failure. Therefore, in accordance with Corps of Engineers' screening criteria for review of spillway adequacy, spillway capacity is considered "seriously inadequate" and the dam is assessed as "unsafe, non-emergency".

TABLE 5.1

TILLSON LAKE DAM

OVERTOPPING ANALYSIS

CONDITIONS

Total Drainage Area = 4.78 square miles
Start Routing at Spillway Crest EL 370
Top of Dam EL 376
Total Project Discharge Capacity at Top of Dam = 2,690 cfs ± due to spillway (flashboards removed).
Outlet conduit closed.
Some values rounded from computed results.

	PMF	1/2 PMF (a)
INFLOW		
24-hour Rainfall (inches)	22.2	12.4 (b)
24-hour Rainfall Excess (inches)(c)	19.5	9.7 (d)
Peak Inflow (cfs)	6,870	3,440
Peak Inflow (csm)	1,437	720
OUTFLOW		
Peak Outflow (cfs)	6,870	3,430
Peak Outflow (csm)	1,437	718
Time to Peak Outflow (hours)	20.0	20.1
Maximum Storage (acre-feet)	455	410
Max. W.S. Elevation (feet-NGVD)	378.2 (e)	376.6 (e)
Minimum Freeboard (feet)	overtopped	overtopped
Maximum Depth over Dam (feet)	2.2	0.6
Duration of Overtopping (hours)	7.2	3.3

(a) One-half of PMF total runoff, including base flow. For PMF base flow = 2 cfs per square mile = 10 cfs ±

(b) Approximation assuming total losses are the same as for the PMF.

(c) Rainfall Excess = Rainfall for the Reservoir Surface. For the rest of the drainage area, losses are assumed to be 1.0 inch initially and 0.1 inch per hour thereafter.

(d) Equal to one-half of PMF value.

(e) If 3-foot high flashboards are in place and do not fail, total discharge capacity at top of dam = 950 cfs ± ; for PMF, peak outflow = 6,870 cfs ± and dam overtopped by 3.0 feet ; for 1/2 PMF, peak outflow = 3,430 cfs ± and dam overtopped by 1.6 feet.

SECTION 6

STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations

The steep (1.25H:1V) downstream slope apparently was formed by dumping spoil from the lake bottom loosely on top of the original dam. This slope is very likely to slide downstream if it becomes saturated by seepage through the cracked core wall, heavy rain, or minor overtopping of the dam. The failure of this mass of material may or may not cut into the original slope of the dam.

b. Design and Construction Data

The design and construction data indicate that the downstream slope should be 2H:1V. The steeper existing downstream slope was discussed above.

No existing stability analysis was found for any part of the dam.

c. Operating Records

No operating records were found or operational problems reported which would adversely affect the stability of the dam.

d. Post-Construction Changes

The post-construction change was discussed in 6.1a above.

e. Seismic Stability

This dam is in Seismic Zone 1. According to Recommended Guidelines (Reference 1) a seismic stability analysis is not required.

6.2 STABILITY ANALYSIS

The concrete spillway is a gravity structure varying in height from about 4 feet to 20 feet. An independent structural stability analysis was performed on a representative section about 16 feet high. The cross section for analysis was chosen about 10 feet from the right training wall where the effects of lateral support due to the training wall are considered minimal. The cross section geometry is based on a 1939 reconstruction drawing (see Appendix G-1) and on visual observation (see Photos A-6B and A-7B). The following loading cases were analyzed:

29 Case 1 - Normal pool at flashboard crest 3 feet above spillway crest, full headwater uplift, no tailwater, silt load starting 3 feet below spillway crest based on observation.

Case 2 - Normal pool at spillway crest, no flashboards, ice load of 5 kips per linear foot for ice 1.0 foot thick, full headwater uplift, tailwater and silt load same as Case 1.

Case 3 - Half PMF pool at EL 376.6 or 6.6 feet above spillway crest, tailwater estimated at 5 feet deep or 11 feet below spillway crest, full headwater and tailwater uplift, no flashboards, silt load same as Case 1.

Case 4 - Full PMF pool at EL 378.2 or 8.2 feet above spillway crest, tailwater estimated at 6 feet deep or 10 feet below spillway crest, remaining conditions same as Case 3.

The results of the stability analysis are summarized in Table 6.1. The computations are included in Appendix D.

For all the loading cases analyzed, minimum satisfactory overturning stability is considered to be a factor of safety of 1.5 with the resultant passing through the middle third of the base. For sliding stability, because of the method of analysis used and the conservative assumptions that were made about foundation material properties, a minimum satisfactory factor of safety of 2.0 is considered appropriate for all the loading cases analyzed, rather than the customary 3.0. Both overturning and sliding stability must be satisfactory in order for stability of the section to be satisfactory.

As noted from Table 6.1, the spillway has unsatisfactory stability for all four primary loading cases. Included in the unsatisfactory rating are the normal summer condition and winter ice load condition, Cases 1 and 2 respectively. Case 1A, normal pool at spillway crest with no flashboards, which represents normal spring and fall conditions under present operating procedures, is the only case where the spillway appears to have acceptable stability, and then only by a small margin. This normal spring condition essentially prevailed on the day of the visual inspection.

29b Case 1B is the same as Case 1 except that the very large shear key at the heel of the section is assumed to help resist overturning. This causes the overturning stability to become barely satisfactory, but tensile stresses must exist in the concrete in order to allow the shear key to be effective. The concrete is assumed to be unreinforced since no data was found to the contrary. Development of tensile stresses in unreinforced

TABLE 6.1

TILLSON LAKE DAM

STABILITY ANALYSIS OF GRAVITY SECTIONS

CASE	OVERTURNING			SLIDING FACTOR OF SAFETY (c)	
	FACTOR OF SAFETY (a)		LOCATION OF RESULTANT (b)		
Spillway Section					
1- Normal Pool with Flashboards	1.24	unsatisfactory	0.22b	1.75	unsatisfactory
1A- no Flashboards	1.63		0.38b	2.51	
1B- with Flashboards and U/S shear key	1.53	(tensile stresses)	0.39b	1.75	unsatisfactory
2- Normal Pool, no Flashboards, Ice Load	1.05	unsatisfactory	0.05b	2.16	
3- Half PMF Pool, no Flashboards	<1.17	unsatisfactory	<0.14b	< 1.57	unsatisfactory
4- Full PMF Pool, no Flashboards	<1.10	unsatisfactory	<0.09b	< 1.45	unsatisfactory
Right Training Wall of Spillway Discharge Channel					
1- Normal Conditions	0.78	unstable	-0.14b	7.56	

(a) Overturning factor of safety is ratio of resisting moments to driving moments taken about the toe.

(b) Distance from toe to point where resultant passes through base, expressed in terms of base dimension "b". Middle third of base is 0.33b to 0.67b.

(c) For spillway section, sliding factor of safety is ratio of shear resistance moment to driving moments taken about the center of a circular arc failure plane. For spillway training wall, sliding factor of safety is ratio of shear resistance force to driving forces taken along a horizontal failure plane.

concrete is itself considered unsatisfactory. Therefore, the stability of Case 1B must be considered just as unsatisfactory as Case 1.

For Cases 3 and 4, the 1/2 PMF and PMF conditions, it should be noted that the full weight of the flowing water on the face of the spillway was taken into account as a resisting force. Considering the relatively steep face of the spillway and the high head and discharge for the 1/2 PMF and PMF conditions, it is probable that the flowing water would exert little to no pressure - or even negative pressure - on face of the spillway. Therefore, the actual stability might be even more unsatisfactory than presently computed, even to the point where the spillway would become unstable under 1/2 PMF and PMF conditions.

In view of the apparent unsatisfactory stability of the spillway, it is recommended that a detailed structural stability investigation of the spillway be conducted to better assess its stability under all loading conditions. The investigation should include appropriate field and laboratory work to determine foundation material properties and structural details. Also, the effect of lateral support offered by the spillway abutments may need to be evaluated. The investigation should determine what modifications, if any, are necessary to achieve satisfactory stability.

The right concrete training wall of the spillway discharge channel is also a significant gravity structure. Its failure would threaten the safety of the embankment behind it. A section of this wall was analyzed under normal earth load conditions with an assumed normal water level in the embankment of 5 feet above the spillway channel floor. The cross section for analysis was chosen at the maximum unsupported height of about 21 feet which occurs at about the spillway toe. The results of the analysis are summarized in Table 6.1, while the computations are included starting on Appendix D-19.

As noted from Table 6.1, the training wall is critically unstable against overturning for normal conditions. Sliding stability does not appear to be a problem. Since the wall has not in fact failed, the present analysis must not reflect the true support system of the wall and/or the actual loading conditions. The wall was assumed to be unreinforced since no data was found to the contrary.

143 In view of the apparent instability of the right training wall of the spillway discharge channel, it is recommended that the training wall be included in the detailed structural stability investigation previously recommended for the spillway itself. Similar to the spillway study, the investigation of the right training wall should include determination of embankment loading conditions and structural details, the possible need to evaluate

the affects of lateral support offered by the spillway, and the determination of any modifications necessary to achieve satisfactory stability.

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SECTION 7

ASSESSMENT AND RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

Visual inspection of Tillson Lake Dam revealed the following deficiencies which affect the safety of the dam:

1) Discarded spoil and boulders on top of the original downstream slope and crest.

2) Multiple cracks in the concrete core wall.

3) Trees and shrubs on the entire downstream slope, the crest, and along the water line upstream.

4) Erosion adjacent to the spillway and along the upstream slope.

Hydrologic and hydraulic analysis indicates that maximum spillway discharge capacity without flashboards is only about 39% of the PMF peak outflow. The 1/2 PMF would overtop the earth embankment and would probably cause failure. It is judged that failure due to overtopping would significantly increase the hazard to loss of life downstream from that which would exist just prior to failure. Therefore, in accordance with Corps of Engineers' screening criteria for review of spillway adequacy, spillway capacity is considered "seriously inadequate" and the dam is assessed as "unsafe, non- emergency".

Structural stability analysis of the spillway section indicates unsatisfactory stability for normal summer conditions and winter ice load conditions, as well as for 1/2 PMF and PMF conditions. Normal spring and fall conditions (with flashboards removed) result in satisfactory stability by only a small margin. Structural stability analysis of the right training wall of the spillway discharge channel indicates critical instability for normal conditions.

b. Adequacy of Information

Available information together with that gathered during the visual inspection, while considered adequate for this Phase I Inspection, is deficient in the following respects:

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1) The downstream slope and crest are covered with miscellaneous fill, boulders, and trees, making it impossible to observe their condition adequately.

2) There are no data available on material properties of the foundation under the spillway, embankment loading conditions behind the right training wall of the spillway discharge channel, and structural details inside the spillway and right training wall. Such data critically affect the structural stability analysis of these two sections.

3) Minor inconsistencies in the engineering data available, based on field observation and checking, are itemized in Section 2.4c.

c. Need for Additional Investigations

The following detailed engineering investigations should be performed by a registered professional engineer qualified by training and experience in the design of dams:

1) Perform a detailed hydrologic and hydraulic analysis to better assess spillway adequacy. This should include a more accurate determination of the site specific characteristics of the watershed.

2) Investigate the character of the spoil material on the downstream slope to determine whether it should be removed and, if so, provide the procedure for removal.

3) Determine whether major repairs should be made to the core wall.

4) Investigate the origin of the seeps through the floor of the spillway discharge channel.

5) Perform a detailed structural stability analysis of the spillway and of the right training wall of the spillway discharge channel to better assess their stability under all loading conditions. This should include investigation of foundation conditions, embankment loading conditions, and structural details. The large vertical crack in the right training wall should be taken into account.

29? d. Urgency

As recommended below in Section 7.2a, the flashboards should be removed from the spillway immediately. Also, a program to visually inspect the dam at least once a month should be instituted immediately. As recommended below in Section 7.2b, development of an emergency action plan should be completed within 3 months after receipt of this Phase I Inspection Report by the Owner. While the action plan is being developed, and within 3

months after receipt of this report by the Owner, the investigations recommended above in Section 7.1c should be started.

Any remedial work deemed necessary as a result of these investigations should be completed within 18 months after receipt of this report by the Owner.

Measures recommended below in Section 7.2c should be completed within 12 months after receipt of this report by the Owner.

7.2 RECOMMENDED MEASURES

The following work should be performed by the Owner. Where engineering assistance is indicated, the Owner should engage a registered professional engineer qualified by training and experience in the design of dams. Assistance by such an engineer may also be useful for some of the other work.

a. Complete Immediately

1) Remove the flashboards from the spillway and keep them removed pending the results of the detailed hydrologic and hydraulic analysis.

2) Institute a program to visually inspect - not just casually look at - the dam and its appurtenances at least once a month.

b. Complete Within 3 Months

Develop an emergency action plan outlining action to be taken to minimize the downstream effects of an emergency, together with an effective warning system.

c. Complete Within 12 Months

1) Remove trees and brush and their root systems from all surfaces of the dam and for 20 feet downstream of the toe in accordance with procedures established by an engineer. Continue to keep these same areas clear by cutting brush and trees and mowing grass at least annually.

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2) Repair the eroded zones of the embankment adjacent to the spillway and along the upstream slope in accordance with a design by an engineer.

3) Monitor the seep adjacent to the outlet conduit and have the data evaluated in accordance with procedures established by an engineer.

4) Dewater and clean the outlet conduit and have it inspected by an engineer.

5) Restore the outlet conduit sluice gate to operation and exercise it regularly.

6) Contingent on the results of the detailed stability analysis by an engineer, repair the zones of eroded and deteriorated concrete of the spillway, discharge channel, and training walls in accordance with a design by the engineer.

7) Develop and implement effective routine operation and maintenance procedures for the dam and its appurtenances.

8) Institute a program of comprehensive technical inspection of the dam and its appurtenances by an engineer on a periodic basis of at least once every two years.

d. Complete Within 18 Months

The following remedial work should be completed by the Owner. A qualified, registered professional engineer should design and observe the construction of the remedial work.

1) Appropriate modifications as a result of the detailed hydrologic and hydraulic analysis.

2) Appropriate modifications as a result of investigating the spoil material on the downstream slope.

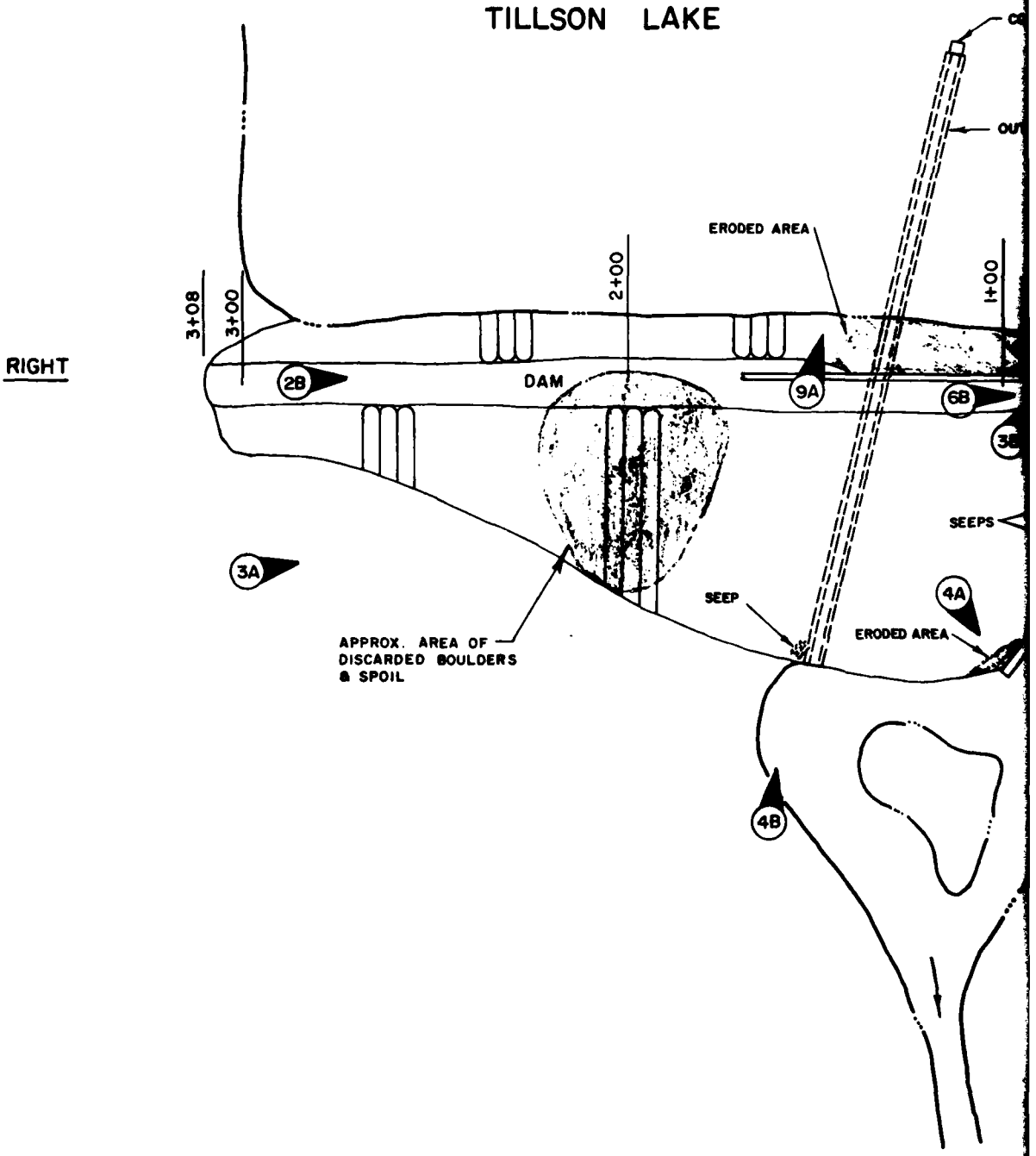
3) Appropriate modifications as a result of investigating the cracks in the core wall.

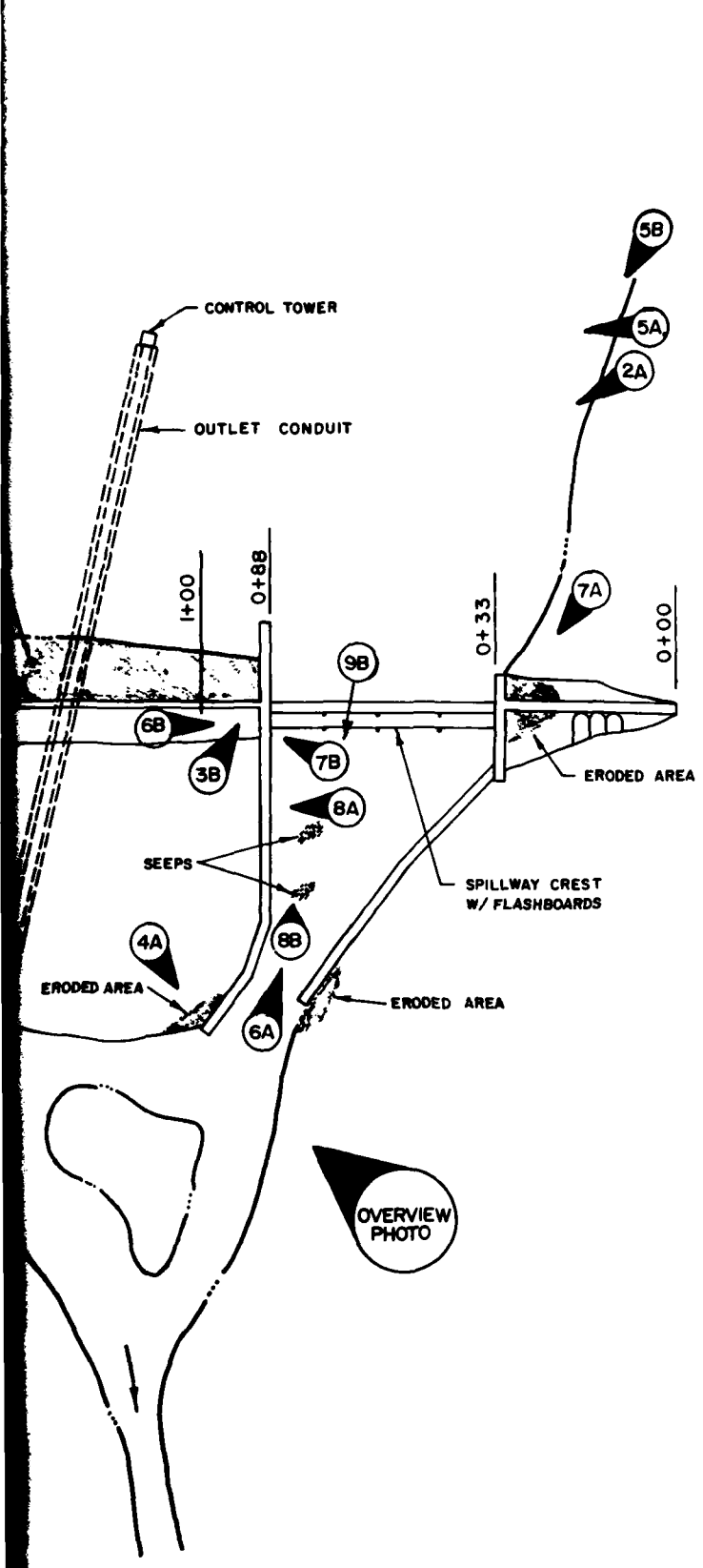
4) Appropriate modifications as a result of investigating the seeps through the floor of the discharge channel.

5) Appropriate modifications as a result of the detailed structural stability analysis of the spillway and of the right training wall of the spillway discharge channel.

APPENDIX A

PHOTOGRAPHS


TILLSON LAKE
RIGHT
3+08
3+00
2+00
1+00
ERODED AREA
DAM
2B
9A
6B
3B
SEEPS
3A
SEEP
4A
ERODED AREA
APPROX. AREA OF
DISCARDED BOULDERS
& SPOIL
4B



LEFT

TILLSON LAKE DAM
PHOTO INDEX MAP

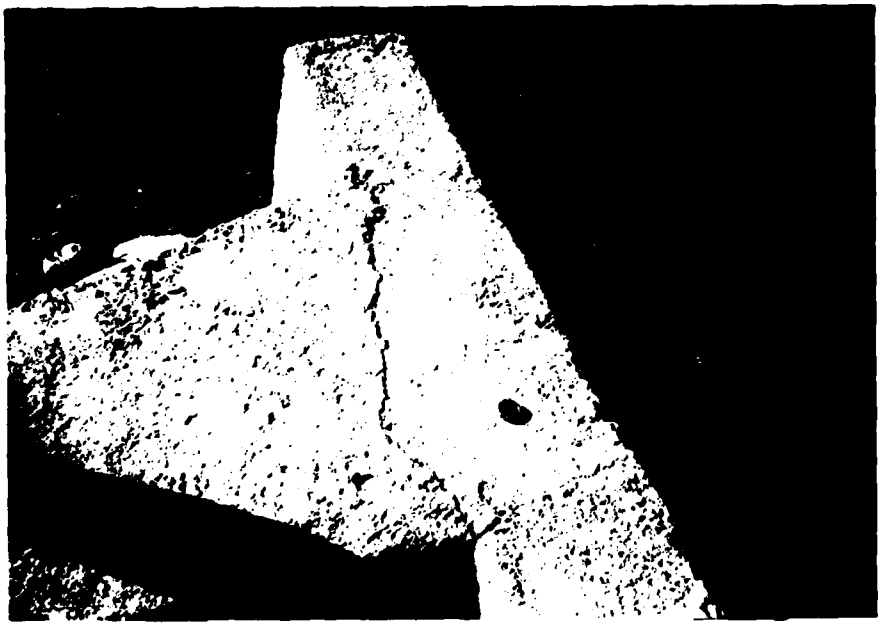
TOWN OF GARDINER	ULSTER CO., N.Y.
SCALE: NONE	DATE JULY 1981

C.T. MALE ASSOCIATES, P.C.
50 TROY ROAD, SCHENECTADY N.Y. 12306
PROFESSIONAL ENGINEERS LAND SURVEYORS LAND PLANNING CONSULTANTS

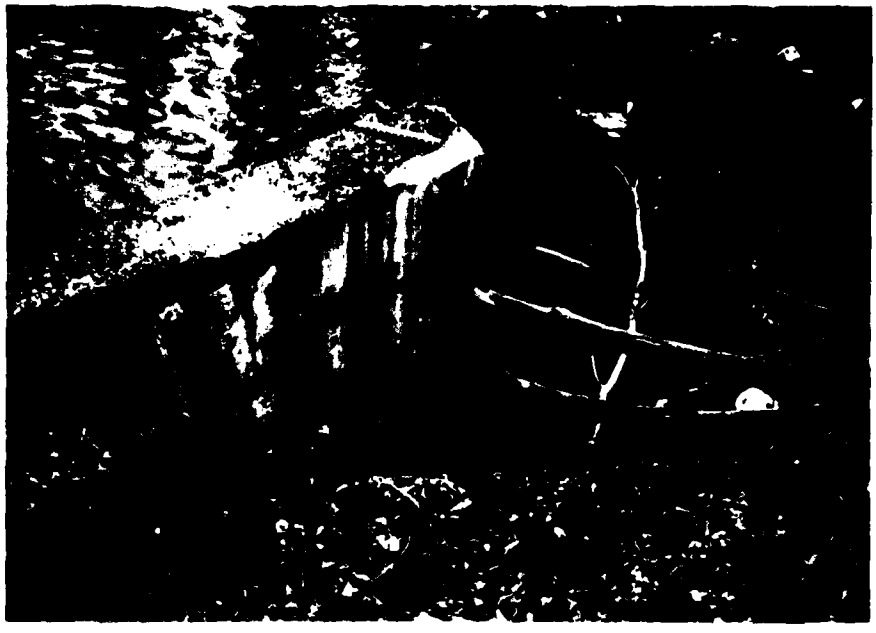
A-2A Upstream slope of dam looking toward right abutment. Note erosion of training wall and upstream slope near spillway 4/8/81

A-2B Top of dam looking toward left abutment - 4/8/81

A-3A Downstream slope of dam from 20 feet downstream of toe at about Sta 3 + 00 - 4/8/81



A-3B Crack at core wall and right training wall junction - 4/8/81



A-4A Erosion at right side of downstream end of right training wall of spillway discharge channel. Eddy probably caused the erosion - 4/8/81

A-4B Outlet conduit looking upstream. A seep is barely visible at the left side of the outlet conduit - 4/8/81

A-5A Control tower and slide gate control mechanism - 4/8/81

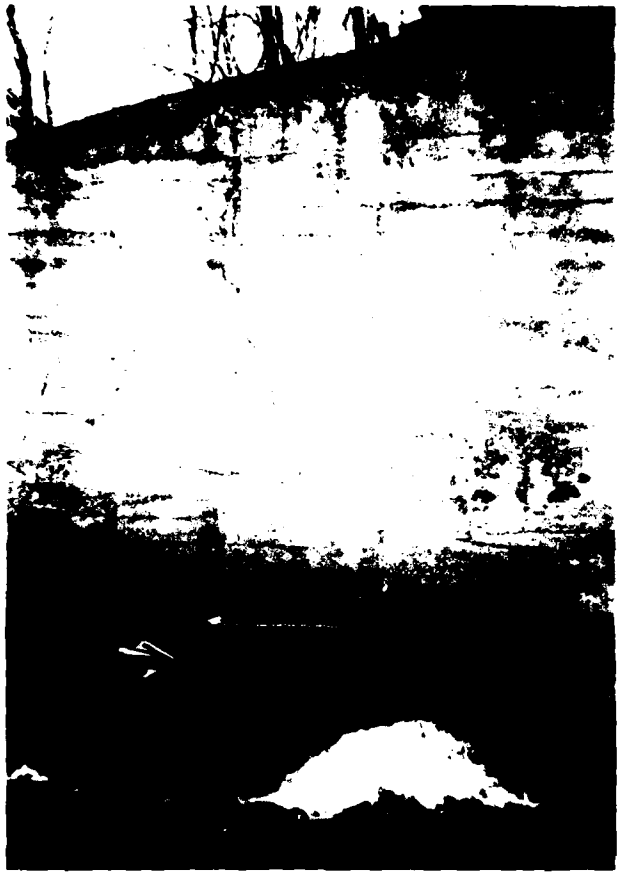
A-5B Spillway crest with flashboards, one section removed, looking downstream - 4/8/81

A-6A Spillway discharge channel looking upstream. Note erosion at downstream end of left training wall and seep in middle through channel bottom - 4/8/81

A-6B Spillway crest looking toward left abutment. Note erosion at ogee and channel transition and siltation upstream of weir - 4/8/81

A-7A Left training wall and exposed core wall at spillway crest. Note cold joints and deteriorated condition of concrete 4/8/81

A-7B Right training wall at spillway crest. Note spalling at training wall and ogee intersection and erosion and scaling of downstream side of ogee section - 4/8/81



A-8A Right training wall at downstream end of ogee section. Note vertical crack in wall and flow concentration and resulting erosion along base of wall - 4/8/81

A-8B Close-up of seep through floor of spillway discharge channel 4/8/81

A-9A Reservoir shoreline looking upstream from dam. Note control tower at left - 4/8/81

A-9B Downstream channel from spillway crest - 4/8/81

APPENDIX B

VISUAL INSPECTION CHECKLIST

PHASE I

VISUAL INSPECTION CHECKLIST

1. BASIC DATA

a. General

Name of Dam Tillson Lake Dam

Fed. I.D.# NY00083 DEC Dam No. 194-2420

River Basin LOWER HUDSON

Location: Town GARDINER County ULSTER

Stream Name PALMAGHATT KILL

Tributary of SHAWANGUNK KILL

Latitude (N) 41° 40.3' Longitude (W) 74° 14.8'

Type of Dam EARTH FILL W/ CONCRETE CORE WALL

Hazard Classification HIGH

Date(s) of Inspection APRIL 8, 1981

Weather Conditions SUNNY & WARM

Reservoir Level at Time of Inspection 370.17 ±
(2"± HIGHER THAN CONCRETE WEIR CREST)

b. Inspection Personnel (*Recorder) THOMAS BENNEDUM - CTM, EDWIN VOPELAK JR.* - CTM, STEVE J. POULOS* - GEI

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

GEORGE SURINACH, VICE-PRESIDENT OF TILLSON LAKE REC. PARK, INC.
35 UTTERBY RD., MALVERNE, NY 11565 → (LEASEE & OPERATOR OF LAKE & DAM)
BUS. (516) 489-0505 HOME (516) 887-7859

ALSO MET HENRY CUNEY, PRES OF TILLSON LAKE REC. PARK & OPERATOR OF DAM AT SITE PRIOR TO INSPECTION

d. History

Date Constructed SUSPECTED 1920's-1930's Date(s) Reconstructed 1939 & 1956

Designer ORIGINAL-UNKNOWN, 1939-S. LEFEVRE, 1956-T.W. WESTLAKE

Constructed By UNKNOWN

Owner U & U REALTY, INC., 100 SEAVIEW DR., SECAUCUS, NJ 07094

ATTN: JOSEPH UANUE, PRESIDENT (201) 348-4900

1568 Name of Dam Tillson Lake Dam Date Apr 8, 81 2

2. EMBANKMENT

a. Characteristics

GEI 1) Embankment Material Probably glacial till. (An apparent former borrow area is on downstream side at right.)

GEI 2) Cutoff Type None

GEI 3) Impervious Core Concrete core wall 18 in. thick at top. Cold joints in core wall to left of spillway and to right of spillway.

GEI 4) Internal Drainage System None

GEI 5) Miscellaneous Appears that dredged material from pond has been discarded on crest and downstream slope. Both are very irregular.

GEI b. Crest

GEI 1) Vertical Alignment Irregular ±1 ft.

GEI 2) Horizontal Alignment Irregular due to piled spoil.

GEI 3) Lateral Movement Not observable.

GEI 4) Surface Cracks Corewall is cracked at spillway and at Sta 1+35, 1+18, 1+07, 1+50, and 1+60. Core covered at stations beyond 1+75.

GEI 5) Miscellaneous Crest is forested with trees to 10 in. and brush.

GEI c. Upstream Slope

GEI 1) Slope (Estimate H:V) 1.5H:1V above beach. 8H:1V on beach.

GEI 2) Undesirable Growth or Debris, Animal Burrows Trees to 8 in. Brush.

GEI 3) Sloughing, Subsidence or Depressions Wave erosion at pool level. Eroded to core wall at spillway on both sides.

GEI 4) Slope Protection 3 to 12 in. stone to top el. of flashboards. Grass and brush and trees above.

GEI 5) Surface Cracks or Movement at Toe Not observable

GEI d. Downstream Slope

GEI 1) Slope (Estimate - H:V) 1.25H:1V

GEI 2) Undesirable Growth or Debris, Animal Burrows Forested. Trees to 20 in. Pile of rocks at Sta 1+70 midslope.

GEI 3) Sloughing, Subsidence or Depressions A large amount of excess material has been placed where 1938 washout occurred. One woodchuck hole among the boulders.

GEI 4) Surface Cracks or Movement at Toe Not observable.

GEI 5) Seepage Damp at toe of pile of boulders (about 12 ft above tailwater). No seepage evident at bedrock/embankment interface, where exposed near toe at Sta. 1+25.

GEI 6) External Drainage System (Ditches, Trenches, Blanket) None.

GEI 7) Condition Around Outlet Structure No movement. Clear seepage at 6-10 gpm immediately to right of outlet structure. Iron stained on bottom. No silt.

GEI 8) Seepage Beyond Toe Seepage exits 18 in below top of structure. None

GEI e. Abutments - Embankment Contact

Good. Spillway is at left and founded on bedrock. Erosion on right side of downstream toe of spillway due to eddies during high water.

GEI 1) Erosion at Contact None (see item above)

GEI 2) Seepage Along Contact None.

3. DRAINAGE SYSTEM

GEI a. Description of System None

GEI b. Condition of System N.A.

GEI c. Discharge from Drainage System N.A.

4. INSTRUMENTATION (Monumentation/Surveys, Observation Wells, GEI Weirs, Piezometers, Etc.)

None

5. RESERVOIR

GEI a. Slopes Gentle. 3H:1V at shore and 10 ft up. Otherwise < 10H:1V.

GEI b. Sedimentation Not observed

GEI c. Unusual Conditions Which Affect Dam None

1285 Name of Dam Tillson Lake Dam Date Apr 8, 81 5

6. AREA DOWNSTREAM OF DAM

a. Downstream Hazard (No. of Homes, Highways, etc.) BRIDGE 400'± D/S, HOUSE 300'± D/S BUT FAIRLY HIGH ABOVE CHANNEL, SEVERAL DWELLINGS FURTHER D/S

GEI b. Seepage, Growth Fully forested. No seepage observed.

GEI c. Evidence of Movement Beyond Toe of Dam None noted

d. Condition of Downstream Channel ROCKY CHANNEL, BRIDGE D/S, SOME TREE & BRUSH ENCROACHMENT.

7. SPILLWAY(S) (Including Discharge Channel)

a. General CONCRETE OGEE SET ON MOSTLY BEDROCK, RAILROAD RAILS & 1½" PIPES SUPPORT 3' WOODEN FLASHBOARDS W/ NO PROVISION FOR AUTOMATIC FAILURE DURING HIGH WATER. SPILLWAY SECTION ABOUT 4'± HIGH ON LEFT, 20'± HIGH ON RIGHT, PROBABLY NOT A TRUE OGEE

b. Conditon of Service Spillway GENERALLY FAIR TO POOR, LEFT TRAINING WALL @ SPILLWAY & D/S END TOP 1' OF CONCRETE IS CRUMBLING, COLD JOINTS IN CONCRETE, MINOR CRACKING & EFFLORESCENCE RIGHT TRAINING WALL - CRACK FULL HEIGHT OF WALL ABOUT 20' FROM SPILLWAY CREST, LARGE SPALL AT RIGHT TRAINING WALL CONTACT W/ D/S SIDE OF OGEE CREST, CRACK BETEEN RIGHT TRAINING WALL &

~~c. Condition of Auxiliary Spillway~~ CORE WALL CONTACT, EROSION AT WATER LINE NEAR SPILLWAY ON U/S SIDE TRAINING WALL. OGEE SECTION - EROSION, SCALING, & SPALLING ALONG ENTIRE D/S FACE GENERAL - SOME HAIRLINE CRACKING, EFFLORESCENCE OF MOST SPILLWAY & TRANNING WALL CONCRETE, SPALLING AT CONSTRUCTION JOINTS

c. NO AUXILIARY SPILLWAY

d. Condition of Discharge Channel CONCRETE ERODED AS DEEP AS 1' BETWEEN OGEE AND CHANNEL TRANSITION, CHANNEL BOTTOM SLAB CRACKING AT 90° TO JOINTS IN SLAB, EROSION OF CHANNEL CONCRETE ALONG BASE OF RIGHT TRAINING WALL WERE DISCHARGES OVER SPILLWAY CONCENTRATE, SPALLING AT JOINTS OF CHANNEL BOTTOM.

8. RESERVOIR DRAIN/OUTLET

a. Type: Pipe _____ Conduit ✓ Other _____

b. Material: Concrete ✓ Metal _____ Other _____

c. Size: 3.8'x3.8'± INSIDE AT OUTLET Length _____ } SEE H+H DATA CHECKLIST, APPENDIX C

d. Invert Elevations: Entrance _____ Exit _____ }

e. Physical Condition (Describe)

Unobservable ✓ (CONDUIT SILTED IN ABOUT 2'±)

1) Material D/S EXPOSED END IS SCALED & DISCOLORED

2) Joints UNOBSERVABLE Alignment UNOBSERVABLE

3) Structural Integrity UNKNOWN

4) Hydraulic Capability CONDUIT SILTED IN 2'±

f. Means of Control: Gate ✓ Valve _____ Uncontrolled _____

Operation: Operable _____ Inoperable ✓ Other _____

Present Condition (Describe) GATE STEM & CONTROL MECHANISM RUSTED & INOPERABLE, HANDWHEEL MISSING

g. Other Outlets (water mains, diversion pipes) _____

N/A

9. STRUCTURAL

a. Concrete Surfaces MOST SURFACES SHOW SIGNS OF EROSION, HAIRLINE CRACKING, SCALING, AND EFFLORESCENCE, SPALLING AT SOME CONSTRUCTION JOINTS. CRUMBLING OF TOP OF EXPOSED COREWALL & PART OF TOP OF RIGHT TRAINING WALL

b. Structural Cracking SEVERAL THROUGH CORE WALL [SEE 2.b.4)], LEFT TRAINING WALL [SEE 7.b.], & CHANNEL SLAB. ALSO COLD JOINTS IN MANY CONCRETE SURFACES

c. Movement - Horizontal & Vertical Alignment(Settlement) APPEARS OKAY

GEI d. Junctions with Abutments or Embankments Good condition.

GEI e. Drains - Foundation, Joint, Face None.

f. Water Passages, Conduits, Sluices DOWNSTREAM END OUTLET CONDUIT CONCRETE IS SCALING & DISCOLORED

GEI g. Seepage or Leakage Two seeps from floor of spillway through cracks in concrete. Flowing clear at 1 gpm and 25 gpm. Iron stain where seeps exit. Seepage may be entering pavement through cracks higher up on the floor of spillway.

0798 Name of Dam Tillson Lake Dam Date Apr 8, 1981 8

h. Joints - Construction, etc. MANY COLD JOINTS WITH SPALLING AT SOME JOINTS, ESPECIALLY OGEE & DISCHARGE CHANNEL CONTACT AND ALONG TOP OF LEFT TRANNING WALL

GEI i. Foundation Bedrock is slate, (argillite,) and/or shale, closely jointed. Strike N30E Dip 23° N. Dam probably founded on bedrock all the way across, but at least ~~to~~ Sta 0 to Sta 1+30. [Old (1939) drawing shows foundation is hardpan beyond Sta 2+20]

GEI j. Abutments Satisfactory

k. Control Gates OUTLET CONDUIT GATE (ONLY ONE) IS INOPERABLE

l. Approach & Outlet Channels APPROACH IS OKAY, OUTLET CHANNEL CURVES D/S TOWARD TOE & BOTTOM SLOPES TOWARD LEFT TRANNING WALL. THIS CAUSES FLOW TO BE CONCENTRATED AT OGEE "TOE" & ALONG BASE OF RIGHT TRANNING WALL & CAUSES CONCRETE EROSION AT THE LOCATIONS.

m. Energy Dissipators (Plunge Pool, etc.) ROCK AT D/S END OF CHANNEL APPEARS ADEQUATE SOME EROSION AT D/S END RIGHT TRAINING WALL (SEE 2.e.)

n. Intake Structures UNOBSERVABLE

o. Stability

p. Miscellaneous N/A

8876 Name of Dam Tillson Lake Dam Date Apr. 8, 1981 9

10. APPURTENANT STRUCTURES (Power House, Lock, Gatehouse, Service Bridge, Other)

a. Description: CONTROL TOWER IN LAKE WITH CONTROL MECHANISM FOR SLIDE GATE, ACESSABLE ONLY BY BOAT

b. Condition: ONLY OBSERVABLE FROM SHORE, CONTROL MECHANISM IS INOPERABLE. CONCRETE IS ERODED + STAINED, STEEL BRAKETS FOR MECHANISM + GATE APPEAR LOOSE + IN POOR CONDITION

11. MISCELLANEOUS MECHANICAL/ELECTRICAL EQUIPMENT

a. Description: N/A

b. Condition:

12. OTHER

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APPENDIX C

HYDROLOGIC AND HYDRAULIC ENGINEERING DATA

CHECKLIST AND COMPUTATIONS

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Overtopping Analysis (flashboards in place)	
Computer Input	C-17
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PHASE I INSPECTION

HYDROLOGIC AND HYDRAULIC
ENGINEERING DATA CHECKLIST

Name of Dam TILLSON LAKE DAM Fed. Id.# NY00083

1. AREA-CAPACITY DATA

	Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
a. Top of Dam *	376	28.5 EST.	394
b. Design High Water (Max. Design Pool)	373.5±	26.1 EST.	315 EST.
c. Auxiliary Spillway Crest	N/A	—	—
d. Pool Level with Flashboards	373	25.6 EST.	312
e. Service Spillway Crest	370	22.7	230

* CREST IS IRREGULAR. ELEVATION IS FOR EXPOSED COREWALL & TOP OF ABUTMENT.

2. DISCHARGES

	Volume (cfs)
a. Average Daily	UNKNOWN
b. Spillway @ Top of Dam (W/O FLASHBOARDS)	2,690
c. Spillway @ Design High Water (APPENDIX F3-12)	1250 = 262CSM
d. Service Spillway @ Auxiliary Spillway Crest Elevation	N/A
e. Low Level Outlet — NORMALLY CLOSED, 30" ¢ GATE EL 341.3±, EST. Q = 130 cfs W/ WATER SURFACE @ EL 370 & CONDUIT CLEAN. PRESENTLY INOPERABLE.	0
f. Total (of all facilities) @ Top of Dam	2,690
g. Maximum Known Flood — SEPT. 21, 1938 DAM FAILED; AUGUST, 1955 DAM OVERTOPPED	*
h. At Time of Inspection	~5

* GREATER THAN SPILLWAY CAPACITY W/ FLASHBOARDS IN PLACE. ESTIMATED AT 500 cfs = 105 csm FOR 1955 EVENT BASED ON 2' FLOW OVER FLASHBOARDS PER APPENDIX F3-9.

4596

3. TOP OF DAM

Elevation 376

a. Type EARTH FILL w/ CONC. CORE WALL + GRAVITY SPILLWAY SECTION

b. Width VARIES AVG. 15' Length 308' (255' w/o SPILLWAY)

c. Spillover SERVICE SPILLWAY

d. Location 30' FROM LEFT ABUTMENT · LOOKING D/S

4. SPILLWAY

	SERVICE		AUXILIARY
a.	370 w/o FLASHBOARDS * 373 w/ FLASHBOARDS	Elevation	NONE
b.	OGEE (PROBABLY NOT A TRUE OGEE)	Type	
c.	55'	Width	
		Type of Control	
d.	N/A	Uncontrolled	
		Controlled:	
e.	FLASHBOARDS	Type (Flashboards; gate)	
f.	4 SECTIONS	Number	
g.	3' HIGH @ 13.75' EACH	Size/Length	
h.	CONCRETE	Invert Material	
i.		Anticipated Length of Operating Service	
j.	80' ±	Chute Length	
k.	~3' DUE TO SILT	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	
l.		Other	

* USGS ELEVATION FROM NEW YORK GAZETTER OF LAKES, REF. 25

4597

5. OUTLET STRUCTURES/EMERGENCY DRAWDOWN FACILITIES

a. Type: Gate____ Sluice____ Conduit ✓ Penstock____

b. Shape CONC. BOX CULVERT W/ CONTROL GATE ON U/S END

c. Size 3' x 3', ~ 170' LONG (MEASURES 3.8' WIDE AT OUTLET)
30" SLUICE GATE AT U/S END

d. Elevations: Entrance Invert 341 ± PER DRAWINGS

Exit Invert 335 ± PER FIELD SURVEY (SILTED IN ABOUT 2')

e. Tailrace Channel: Elevation N/A

6. FLOOD WATER CONTROL SYSTEM

a. Warning System NONE

b. Method of Controled Releases (mechanisms) CAN ONLY REGULATE RELEASES BY REMOVING FLASHBOARDS, OUTLET CONDUIT SILTED IN AND CONTROL GATE IS INOPERABLE

7. CLIMATOLOGICAL GAGES REFERENCES 21 & 22

a. Type NON-RECORDING PRECIPITATION GAGE INDEX # 3138

b. Location TOWN OF GARDINER LAT. 41° 4', LONG. 74° 09'
~ 3 MILES EAST OF DAM

c. Period of Record 1956 TO PRESENT

d. Maximum Reading UNKNOWN Date____

8. STREAM GAGES REFERENCES 23 & 24

a. Type SURFACE WATER STATION USGS GAGE # 01371500

b. Location WALLKILL RIVER AT GARDINER (REGULATED)
LAT. 41° 41' 10", LONG. 74° 09' 56", ~5 MILES WEST OF DAM

c. Period of Record 1924 - PRESENT

d. Maximum Reading 30,800 cfs = 433 cms Date OCT. 16, 1955

9. OTHER

PER REF. 24 MAXIMUM KNOWN DISCHARGE OF SHAWANGUNK KILL @ GANAHGOTE, ABOUT 3.5 MILES N.E. OF DAM, = 14,000 cfs = 95.2 cms 8/19/55 & OF DWAAR KILL @ DWAARKILL, ABOUT 2 MILES S.W. OF DAM, = 1,870 cfs = 179.8 cms 6/1/52

6169

10. DRAINAGE BASIN CHARACTERISTICS

a. Drainage Area 4.780 SQ. MILES OR 3059.6 ACRES

b. Land Use - Type HEAVILY WOODED

c. Terrain - Relief SLOPES OF 10%-25%, ELEVATIONS FROM EL 370 TO EL 2180

d. Surface - Soil GLACIAL TILL

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

NONE KNOWN.

f. Potential Sedimentation Problem Areas (natural or man-made; present or future)

NONE KNOWN.

g. Potential Backwater Problem Areas for Levels at Maximum Storage Capacity (including surcharge storage)

NONE

h. Dikes - Floodwalls (overflow & non-overflow) - Low Reaches Along the Reservoir perimeter

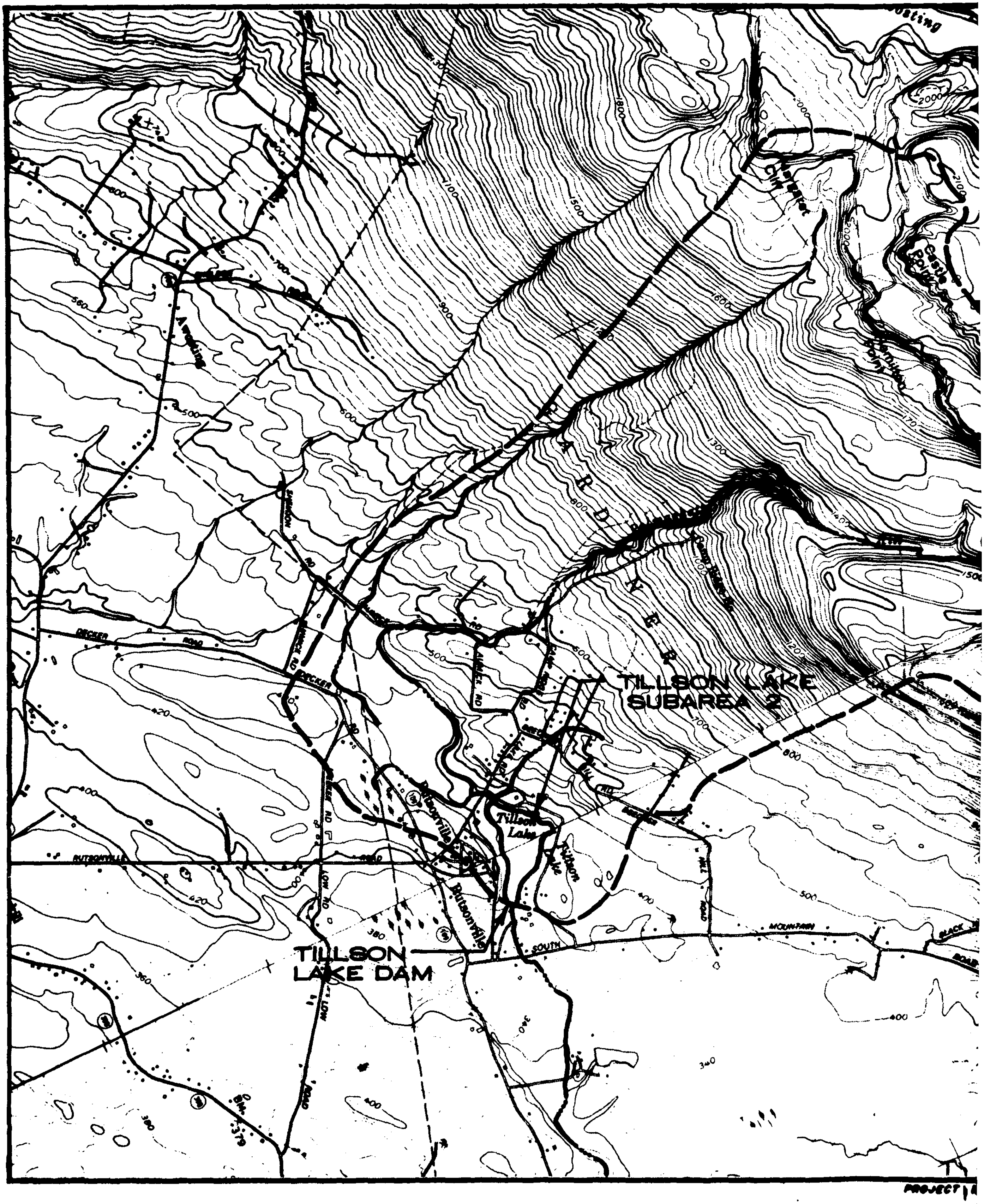
Location N/A

Elevation

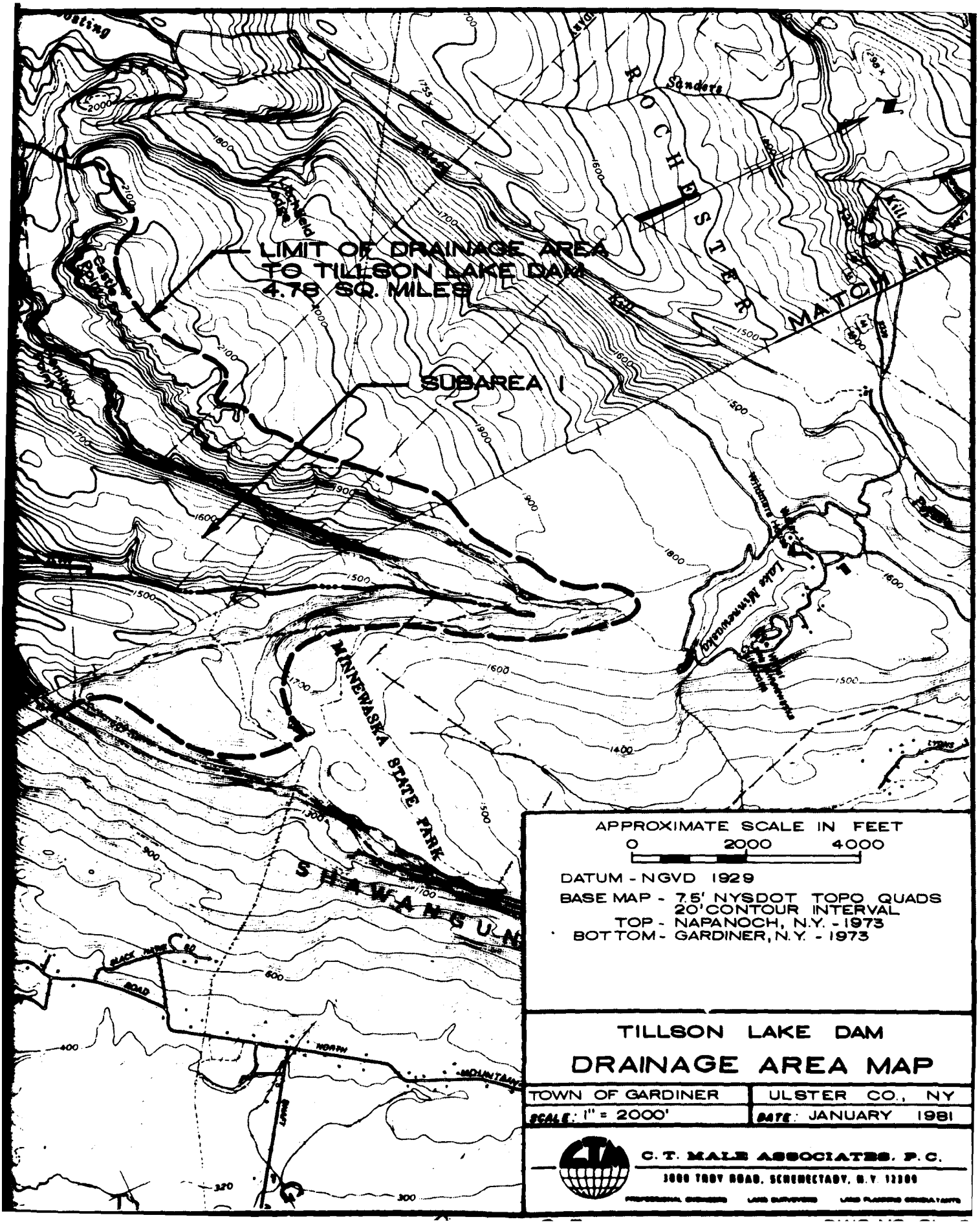
i. Reservoir

Length @ Maximum Design Pool ~1700' (AT SPILLWAY CREST) (feet)

Length of Shoreline (@ Service Spillway Crest) ~5300 (feet)


TILLSON LAKE
SUBAREA 2
TILLSON
LAKE DAM
DECKER ROAD
RUTSONVILLE ROAD
Tillson Lake
MOUNTAIN
SOUTH

PROJECT



APPROXIMATE SCALE IN FEET

0 2000 4000

DATUM - NGVD 1929

BASE MAP - 7.5' NYSDOT TOPO QUADS
20' CONTOUR INTERVAL
TOP - NAPANOCH, N.Y. - 1973
BOTTOM - GARDINER, N.Y. - 1973

TILLSON LAKE DAM

DRAINAGE AREA MAP

TOWN OF GARDINER	ULSTER CO., NY
SCALE: 1" = 2000'	DATE: JANUARY 1981

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COMPUTER SERVICES LANDSCAPE ARCHITECTURE LABORATORY SERVICES

JOB TILLSON LAKE DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 4/14/81
CHECKED BY JPB DATE 5/30/81
SCALE SB.01.00004

ELEVATION - AREA - STORAGE COMPUTATIONS

RESERVOIR VOLUME: FOR STORAGE ABOVE SPILLWAY CREST VOLUME COMPUTED BY METHOD OF CONIC SECTIONS $\Delta V_{1,2} = h/3 (A_1 + A_2 + \sqrt{A_1 A_2})$

	ELEVATION (NGVD - ft.) (1)	AREA (acres)	INPUT VOLUME (acre-feet)	
	340	—	0	
SPILLWAY CREST	370	22.7 (3)	230 (2)	
SPILLWAY CREST W/ FLASHBOARDS	373	25.6	312	CALC. BY COMPUTER
TOP OF DAM	376 (4)	28.5	394	CALC. BY COMPUTER
	380	32.3 (3)	504	

(1) CONSTRUCTION DRAWING ELEVATION BASE IS APPROXIMATELY 90' LOWER THAN NGVD ELEVATION PER NY GAZETTEER OF LAKES (REF. 25)

(2) IMPOUNDING CAPACITY AT SPILLWAY CREST FROM RECONSTRUCTION APPLICATIONS DATED 5/15/39 & 5/4/56 (SEE APPENDICES E3-3 & F3-11)

(3) FROM USGS TOPOGRAPHIC MAPPING, RECONSTRUCTION APPLICATIONS SHOW 25 ACRES.

(4) FROM FIELD MEASUREMENTS. (TOP OF EXPOSED CORE WALL)

DRAINAGE AREA	AREA (acres)	(square miles)
WATERSHED DIRECT TO RESERVOIR (SUBAREA 1)	3036.9	4.745
RESERVOIR SURFACE (SUBAREA 2) @ SPILLWAY CREST EL = 370	22.7	.035
TOTAL	3059.6	4.780 (5)

(5) RECONSTRUCTION APPLICATIONS SHOW 3.5 SQ. MI.

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JOB TILLSON LAKE DAM
SHEET NO. ____ OF ____
CALCULATED BY CLV DATE 4/14/81
CHECKED BY [illegible] DATE 5/30/81
SCALE 58.01.00004

DISCHARGE COMPUTATIONS

DAM APPURTENANCE	ELEVATION (NGVD)	SIZE
SPILLWAY	CREST EL = 370 w/o FLASHBOARDS CREST EL = 373 w/ FLASHBOARDS	55' CREST LENGTH
DAM	CREST EL = 376 (LOW POINT, TOP OF CORE WALL)	253' CREST LENGTH (EXCLUDES SPILLWAY)
OUTLET CONDUIT (30" SLUICE GATE @ U/S END)	INVERT EL = 341	3'x3' CONCRETE BOX (MEASURES 3.8' WIDE AT OUTLET)

FOR FLOW OVER SPILLWAY: $Q = 3.33 L H^{1.5}$ (FORMULA FOR CRITICAL FLOW OVER SHARP-CRESTED WEIR, REFERENCE 9.)
↑ INPUT
FOR FLOW OVER DAM: $Q = 3.087 L H^{1.5}$ (FORMULA FOR CRITICAL FLOW OVER BROAD-CRESTED WEIR, REFERENCE 9)

	ELEVATION (NGVD)	$H_{SPILLWAY}$ w/o FLASHBOARDS (feet)	H_{DAM} (feet)	$Q_{OUTLET\ CONDUIT}$ (cfs)	$Q_{SPILLWAY}$* w/o FLASHBOARDS (cfs)	Q_{DAM}* (cfs)	Q_{TOTAL} (cfs)
SPILLWAY CREST	370	0	0	0	0	0	0
	372	2	0	ASSUMED CLOSED ↓	518	0	518
FLASHBOARD CREST	373	3	0		952	0	952
	374	4	0		1,465	0	1,465
	375	5	0		2,048	0	2,048
TOP OF DAM	376	6	0		2,692	0	2,692 SAY 2,690
	377	7	1		3,392	781	4,173
	378	8	2		4,144	2,209	6,353
	379	9	3		4,945	4,058	9,003
	380	10	4	0	5,792	6,248	12,040

* HAND COMPUTATION TO MATCH COMPUTATION BY HEC-1 DB PROGRAM.

Form CTM-405

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COMPUTER SERVICES LANDSCAPE ARCHITECTURE LABORATORY SERVICES

JOB TILLSON LAKE DAM
SHEET NO. ____ OF ____
CALCULATED BY ELV DATE 4/14/81
CHECKED BY JPG DATE 5/30/81
SCALE 58.01.00004

DRAINAGE AREA DATA FOR HEC-1 DB MODEL

SUBAREA 1: AREA TRIBUTARY DIRECTLY TO RESERVOIR
AREA = 4.745 SQUARE MILES

LOSS RATES: 1.0" - INITIALLY
0.1"/HOUR - CONSTANT LOSS RATE

UNIT HYDROGRAPH PARAMETERS: USE SNYDER METHOD

A = DRAINAGE AREA = 4.745 SQUARE MILES
L = LENGTH OF MAIN WATERCOURSE TO UPSTREAM LIMIT OF DRAINAGE AREA = 4.73 MILES
L_{CA} = LENGTH ALONG MAIN WATERCOURSE TO POINT OPPOSITE THE CENTROID OF THE DRAINAGE AREA = 2.46 MILES
C_t = SNYDER'S BASIN COEFFICIENT = 2.2 (FROM REF. 20)
C_p = SNYDER'S PEAKING COEFFICIENT = .650 (FROM REF. 20)
t_p = STANDARD LAG IN HOURS = $C_t(LL_{CA})^{0.3}$ = 4.59 HOURS

∴ USE t_p = 4.6 HOURS

SUBAREA 2: RESERVOIR SURFACE, AREA = .035 SQ. MILES = 22.7 ACRES

LOSS RATES: NONE BECAUSE RAINFALL ≈ RUNOFF FOR WATER SURFACE

UNIT HYDROGRAPH PARAMETERS:

FOR U.H. W/ 5 MINUTE DURATION + 1" RAIN

$$\bar{Q} = \frac{A(1")}{t} = \frac{22.7 \text{ acres } (1")}{5 \text{ minutes}} \left(\frac{43\,560 \text{ sq FT}}{1 \text{ acre}}\right)\left(\frac{1 \text{ FT}}{12 \text{ inches}}\right)\left(\frac{1 \text{ minute}}{60 \text{ seconds}}\right)$$

$\bar{Q}$ = 275 cfs (W/O LOSS RATE)

```
A NYD DAM INSPECTION: DACW51-81-C-0014
A NY00083, TILLSON LAKE DAM, 58.01.0004 / 80.00844
A OVERTOPPING ANALYSIS TLD2  W/O FLASHBOARDS
B    288       0       5                                                  4
B1     5
J      1       2       1
J1   1.0     0.5
K      0     SA-1                                          1
K1 SUBAREA 1 RUNOFF COMPUTATION
M      1       1   4.745                 10                              1
P                 21     111     123     132
T                                                  1.0     .10
N    4.6     .65
X     -2       1                                                                  1 PART
K      0     SA-2                                          1
K1 SUBAREA 2 (RESERVOIR) RUNOFF COMPUTATION
M      1      -1   0.035                 10                              1
P                 21     111     123     132
T                                                    0       0
U      1
U1   275
X     -2                1
K      2   SA-2C                                         1
K1 COMBINING HYDROGRAPHS 1 & 2
K      1     RES                       2                 1
K1 ROUTING FLOWS THROUGH RESERVOIR
Y                                  1       1
Y1     1                                              -370       0
SS     0     230     504
SE   340     370     380
SV   370      55    3.33     1.5
SD   376   3.087     1.5     253
K     99
A
A
A
A
A
```

```
**********
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
**********

RUN  DATE: 5/27/81
     TIME: 3:42 PM

                    NYO DAM INSPECTION: DACW51-81-C-0014
                    NY00083, TILLSON LAKE DAM, 58.01.0004 / 80.00844
                    OVERTOPPING ANALYSIS TLU2  W/O FLASHBOARDS

                                    JOB SPECIFICATION
                    NO     NHR    NMIN    IDAY     IHR    IMIN   METRC    IPLT    IPRT   NSTAN
                    288      0       5       0       0       0       0       0       4       0
                                          JOPER     NWT   LROPT   TRACE
                                              5       0       0       0

                                    MULTI-PLAN ANALYSES TO BE PERFORMED
                                          NPLAN= 1  NRTIO= 2  LRTIO= 1
                              RTIOS=    1.00    0.50

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

                                    SUB-AREA RUNOFF COMPUTATION

                    SUBAREA 1 RUNOFF COMPUTATION
                                  ISTAO   ICOMP   IECON   ITAPE    JPLT    JPRT   INAME  ISTAGE   IAUTO
                                   SA-1       0       0       0       0       0       1       0       0

                                    HYDROGRAPH DATA
                              IHYDG    IUNG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                                  1       1    4.74    0.00   10.00    0.00   0.000       0       1       0

                                    PRECIP DATA
                                    SPFE     PMS      R6      R12     R24     R48     R72     R96
                                    0.00   21.00  111.00  123.00  132.00    0.00    0.00    0.00
TRSPC COMPUTED BY THE PROGRAM IS 0.600

                                    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    0.10    0.00    0.00

                                    UNIT HYDROGRAPH DATA
                                          TP=  4.60    CP=0.65    NTA=  0

                                    RECESSION DATA
                                    STRTQ=  -2.00    QRCSN=   0.00    RTIOR= 1.00

                    UNIT HYDROGRAPH100 END-OF-PERIOD ORDINATES, LAG=  4.60 HOURS, CP= 0.65  VOL= 0.76
                      1.      4.      9.     15.     22.     29.     37.     45.     54.     63.
                     73.     83.     93.    104.    114.    126.    137.    148.    160.    172.
                    184.    196.    208.    220.    232.    245.    258.    270.    283.    296.
                    308.    321.    332.    344.    354.    364.    374.    383.    391.    399.
                    406.    413.    419.    425.    430.    435.    439.    442.    445.    447.
                    449.    450.    451.    451.    450.    449.    446.    443.    439.    434.
                    426.    418.    409.    401.    392.    384.    376.    368.    361.    353.
                    346.    339.    332.    325.    318.    311.    305.    298.    292.    286.
                    280.    274.    269.    263.    258.    252.    247.    242.    237.    232.
                    227.    222.    218.    213.    209.    204.    200.    196.    192.    188.
```

```
        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

                                                                         SUM  22.18  19.49   2.68   524631.
                                                                             ( 563.)( 495.)(  68.)(14855.68)

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

                                    SUB-AREA RUNOFF COMPUTATION

                 SUBAREA 2 (RESERVOIR) RUNOFF COMPUTATION
                            ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE  IAUTO
                             SA-2       0       0       0      0      0       1       0      0

                                              HYDROGRAPH DATA
                  IHYDG    IUHG   TAREA    SNAP   TRSDA   TRSPC   RATIO   ISNOW   ISAME   LOCAL
                      1      -1    0.03    0.00   10.00    0.00   0.000       0       1       0

                                                PRECIP DATA
                           SPFE     PMS      R6     R12     R24     R48     R72     R96
                           0.00   21.00  111.00  123.00  132.00    0.00    0.00    0.00
 TRSPC COMPUTED BY THE PROGRAM IS 0.800

                                                 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    0.00    0.00    0.00    0.00

                                               RECESSION DATA
                              STRTQ=   -2.00    QRCSN=    0.00    RTIOR= 1.00
        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

                                                                         SUM  22.18  22.18   0.00    6136.
                                                                             ( 563.)( 563.)(   0.)(  173.75)

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

                                        COMBINE HYDROGRAPHS

                 COMBINING HYDROGRAPHS 1 & 2
                            ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE  IAUTO
                            SA-2C       2       0       0      0      0       1       0      0

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

                                        HYDROGRAPH ROUTING

                 ROUTING FLOWS THROUGH RESERVOIR
                            ISTAQ   ICOMP   IECON   ITAPE   JPLT   JPRT   INAME  ISTAGE  IAUTO
                              RES       1       0       0      2      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   -370.       0

   CAPACITY=        0.      230.      504.
```

ELEVATION= 340. 370. 380.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
370.0	55.0	3.3	1.5	0.0	0.0	0.0	0.0

DAM DATA

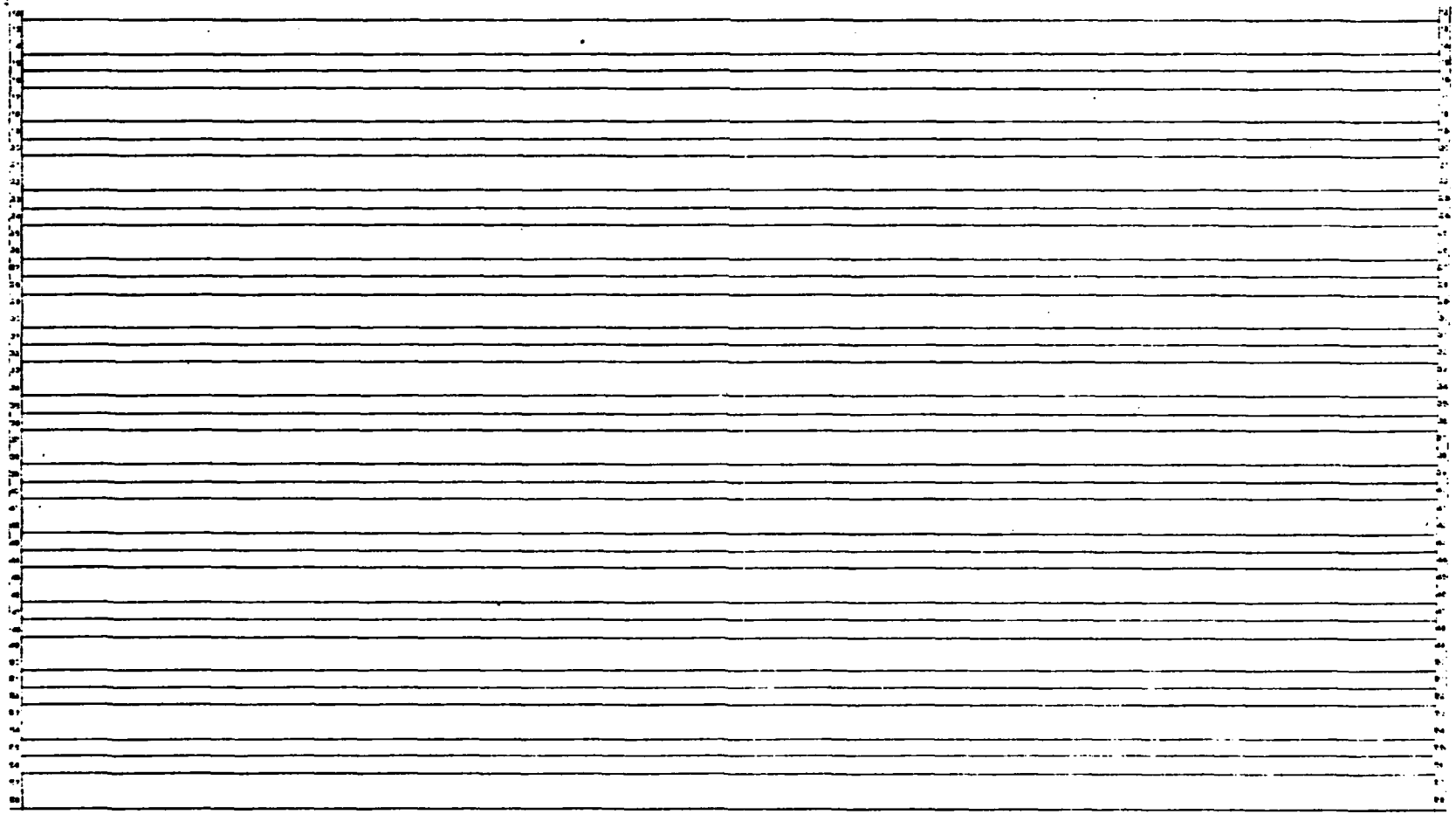
TOPEL	COQD	EXPD	DAMWID
376.0	3.1	1.5	253.

PEAK OUTFLOW IS 6866. AT TIME 20.00 HOURS

1 PART

DVN

PEAK OUTFLOW IS 3426. AT TIME 20.08 HOURS



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	RATIOS APPLIED TO FLOWS RATIO 1	RATIO 2
				1.00	0.50
HYDROGRAPH AT	SA-1	4.74	1	6871.	3435.
		(12.29)		(194.56)(	97.28)(
HYDROGRAPH AT	SA-2	0.03	1	546.	273.
		(0.09)		(15.45)(	7.73)(
2 COMBINED	SA-2C	4.78	1	6874.	3437.
		(12.38)		(194.66)(	97.33)(
ROUTED TO	RES	4.78	1	6866.	3426.
		(12.38)		(194.43)(	97.03)(

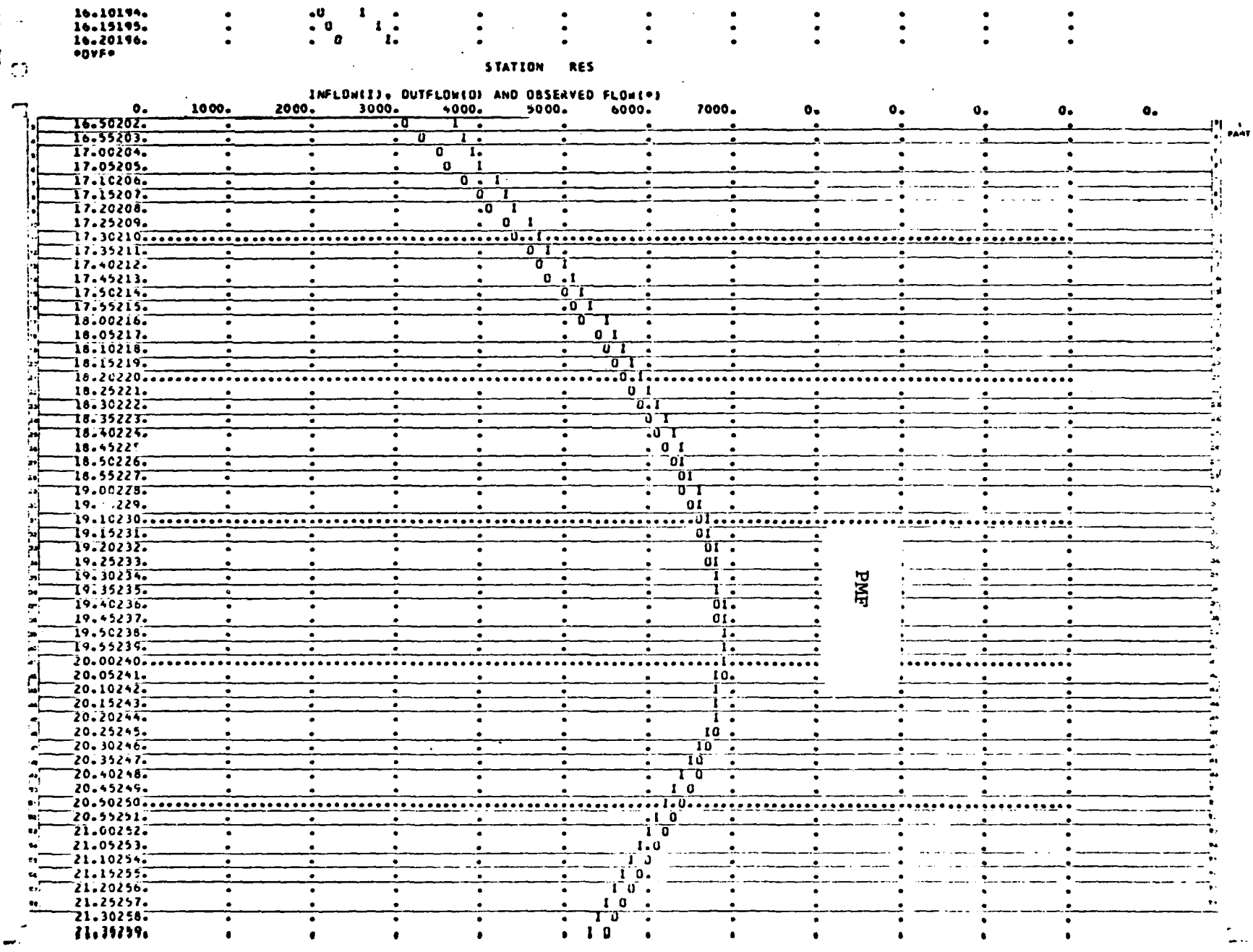
1 PART

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 .....

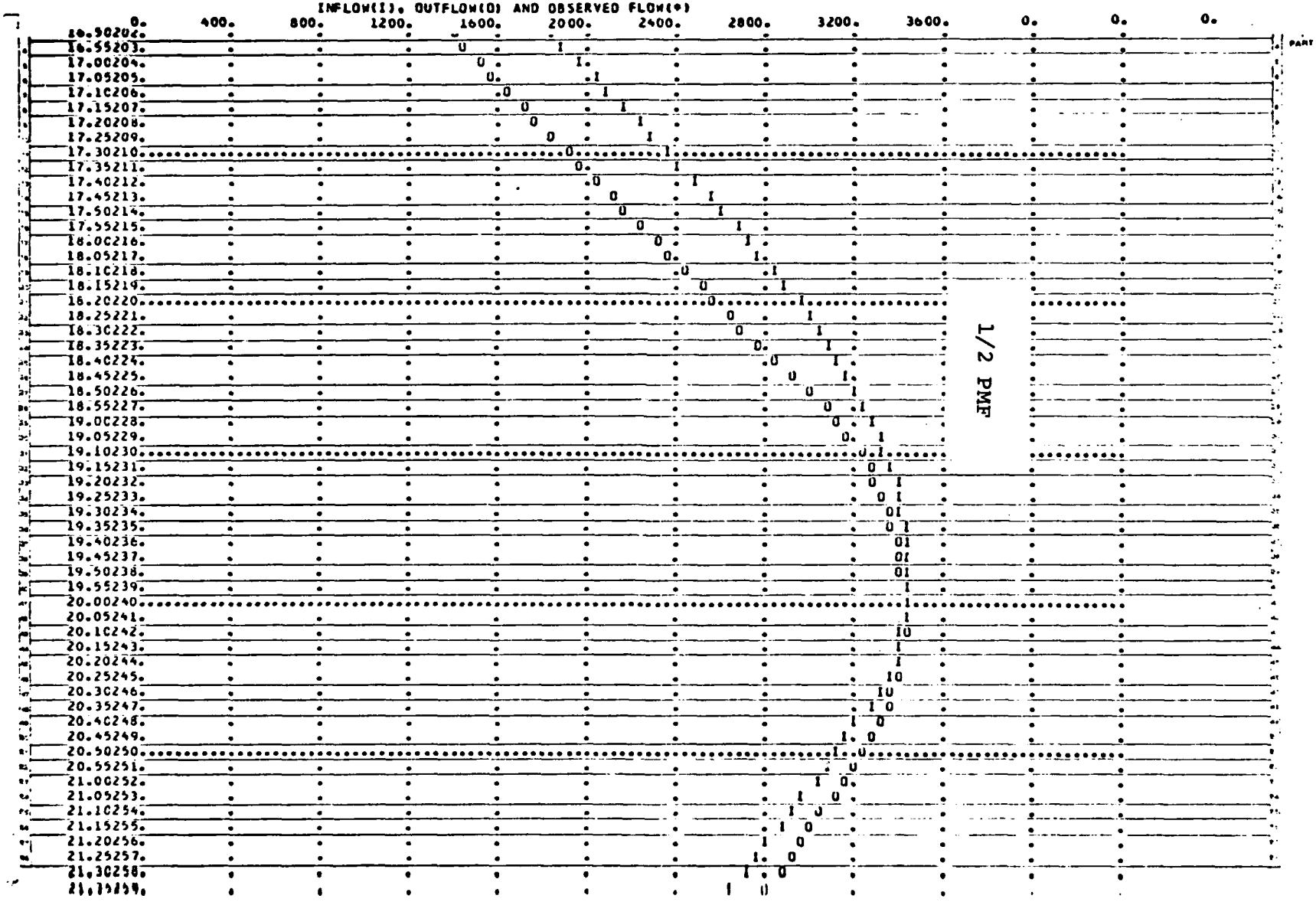
	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	370.00	370.00	376.00
STORAGE	230.	230.	394.
OUTFLOW	0.	0.	2692.

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	378.21	2.21	455.	6866.	7.17	20.00	0.00
0.50	376.57	0.57	410.	3426.	3.33	20.08	0.00


16.10194.
16.15195.
16.20196.
•DYF•
STATION RES
INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(•)
0. 1000. 2000. 3000. 4000. 5000. 6000. 7000. 0. 0. 0. 0. 0.
16.50202.
16.55203.
17.00204.
17.05205.
17.10206.
17.15207.
17.20208.
17.25209.
17.30210.
17.35211.
17.40212.
17.45213.
17.50214.
17.55215.
18.00216.
18.05217.
18.10218.
18.15219.
18.20220.
18.25221.
18.30222.
18.35223.
18.40224.
18.45225.
18.50226.
18.55227.
19.00228.
19.05229.
19.10230.
19.15231.
19.20232.
19.25233.
19.30234.
19.35235.
19.40236.
19.45237.
19.50238.
19.55239.
20.00240.
20.05241.
20.10242.
20.15243.
20.20244.
20.25245.
20.30246.
20.35247.
20.40248.
20.45249.
20.50250.
20.55251.
21.00252.
21.05253.
21.10254.
21.15255.
21.20256.
21.25257.
21.30258.
21.35259.
PMF

STATION RES

INFLOW(I), OUTFLOW(O) AND OBSERVED FLOW(*)



```
A NYD DAM INSPECTION: DACW51-81-C-0014
A NY00383, TILLSON LAKE DAM, 56.01.0004 / 80.00844
A OVERTOPPING ANALYSIS TLD1  W/ FLASHBOARDS
B    288       0       5                                                4
B1     5
J      1       2       1
J1   1.0     0.5
K      0    SA-1                                        1
K1 SUBAREA 1 RUNOFF COMPUTATION
M      1       1   4.745          10                                    1
P              21     111     123     132
T                                          1.0     .10
W    4.6     .65
X     -2       1
K      0    SA-2                                        1
K1 SUBAREA 2 (RESERVOIR) RUNOFF COMPUTATION
M      1      -1   0.035          10                                    1
P              21     111     123     132
T                                          0       0
U      1
U1   275
X     -2       1
K      2    SA-2C                                       1
K1 COMBINING HYDROGRAPHS 1 & 2
K      1     RES                                        1
K1 ROUTING FLOWS THROUGH RESERVOIR
Y                                  1       1
Y1     1                                   -373       0
SS     0     230     504
SE   340     370     360
S*   373      55    3.33     1.5
SD   376   3.087     1.5     253
K     99
A
A
A
A
A
```

********** **********
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION JULY 1978
LAST MODIFICATION 26 FEB 79
********** **********

RUN DATE: 8/09/81
TIME: 11:36 AM

1 PART

NYO DAM INSPECTION: DACW51-81-C-0014
NY00043, TILLSON LAKE DAM, 58.01.0004 / 80.00644
OVERTOPPING ANALYSIS TL01 W/ FLASHBOARDS

JOB SPECIFICATION

NQ	NHR	NMIN	IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
288	5	5	0	0	0	0	0	4	0

JOPER	NWT	LROPT	TRACE
5	0	0	0

MULTI-PLAN ANALYSES TO BE PERFORMED
NPLAN= 1 NRTIO= 2 LRTIO= 1

RTIOS= 1.00 0.50

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

SUB-AREA RUNOFF COMPUTATION

SUBAREA 1 RUNOFF COMPUTATION

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

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	1	4.74	0.00	10.00	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	21.00	111.00	125.00	132.00	0.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS 0.800

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	0.10	0.00	0.00

UNIT HYDROGRAPH DATA
TP= 4.60 CP=0.65 NTA= 0

RECESSION DATA
STRTQ= -2.00 QRCSN= 0.00 RTIOR= 1.00

UNIT HYDROGRAPH 100 END-OF-PERIOD ORDINATES, LAG= 4.60 HOURS, CP= 0.65 VOL= 0.76

1.	4.	9.	15.	22.	29.	37.	45.	54.	63.
73.	83.	93.	104.	114.	126.	137.	148.	160.	172.
184.	196.	208.	220.	232.	245.	258.	270.	283.	296.
308.	321.	332.	344.	354.	364.	374.	383.	391.	399.
406.	413.	419.	425.	430.	435.	439.	442.	445.	447.
449.	450.	451.	451.	450.	449.	446.	443.	439.	434.
426.	418.	409.	401.	392.	384.	376.	368.	361.	353.
346.	339.	332.	325.	318.	311.	305.	298.	292.	286.
280.	274.	269.	263.	258.	252.	247.	242.	237.	232.
227.	222.	218.	213.	209.	204.	200.	196.	192.	188.

END-OF-PERIOD FLOW

0 NO. DA	HR. MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	NO. DA	HR. MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
									SUM	22.18	19.49	2.68	524631.
										(563.)(	495.)(	68.)(14855.68)

PART 1

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

SUB-AREA RUNOFF COMPUTATION

SUBAREA 2 (RESERVOIR) RUNOFF COMPUTATION

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

HYDROGRAPH DATA

IHYDG	IUHG	TAREA	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
1	-1	0.03	0.00	10.00	0.00	0.000	0	1	0

PRECIP DATA

SPFE	PMS	R6	R12	R24	R48	R72	R96
0.00	21.00	111.00	123.00	132.00	0.00	0.00	0.00

TRSPC COMPUTED BY THE PROGRAM IS 0.000

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	0.00	0.00	0.00	0.00

RECESSION DATA

STRTQ= -2.00 QRCSN= 0.00 RTIOR= 1.00

END-OF-PERIOD FLOW

0 NO. DA	HR. MN	PERIOD	RAIN	EXCS	LOSS	COMP Q	NO. DA	HR. MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
									SUM	22.18	22.18	0.00	5136.
										(563.)(	563.)(	0.)(	173.75)

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

COMBINE HYDROGRAPHS

COMBINING HYDROGRAPHS 1 & 2

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

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

HYDROGRAPH ROUTING

ROUTING FLOWS THROUGH RESERVOIR

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

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	-373.	0

CAPACITY= 0. 230. 504.

ELEVATION= 340. 370. 380.

CREL	SPWID	COQW	EXPW	ELEVL	COQL	CAREA	EXPL
373.0	55.0	3.3	1.5	0.0	0.0	0.0	0.0

DAM DATA

TOPEL	COQD	EXPD	DAMWID
376.0	3.1	1.5	253.

1 PART

PEAK OUTFLOW IS 6867. AT TIME 20.00 HOURS

PEAK OUTFLOW IS 3431. AT TIME 20.00 HOURS

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	RATIO 2
				1.00	0.50
HYDROGRAPH AT	SA-1	4.74	1	6871.	3435.
		(12.29)		(194.56)(	97.28)(
HYDROGRAPH AT	SA-2	0.03	1	546.	273.
		(0.09)		(15.45)(	7.73)(
2 COMBINED	SA-2C	4.78	1	6874.	3437.
		(12.38)		(194.66)(	97.33)(
ROUTED TO	RES	4.78	1	6867.	3431.
		(12.38)		(194.45)(	97.17)(

RATIOS APPLIED TO FLOWS

SUMMARY OF DAM SAFETY ANALYSIS

PLAN 1 .....

	INITIAL VALUE	SPILLWAY CREST	TOP OF DAM
ELEVATION	373.00	373.00	376.00
STORAGE	312.	312.	394.
OUTFLOW	0.	0.	952.

1 PART

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	379.04	3.04	478.	6867.	9.17	20.00	0.00
0.50	377.62	1.62	439.	3431.	7.92	20.00	0.00

APPENDIX D

STABILITY ANALYSIS

TABLE OF CONTENTS

	Page
Spillway Section	D-1
Right Training Wall of Spillway Discharge Channel	D-19

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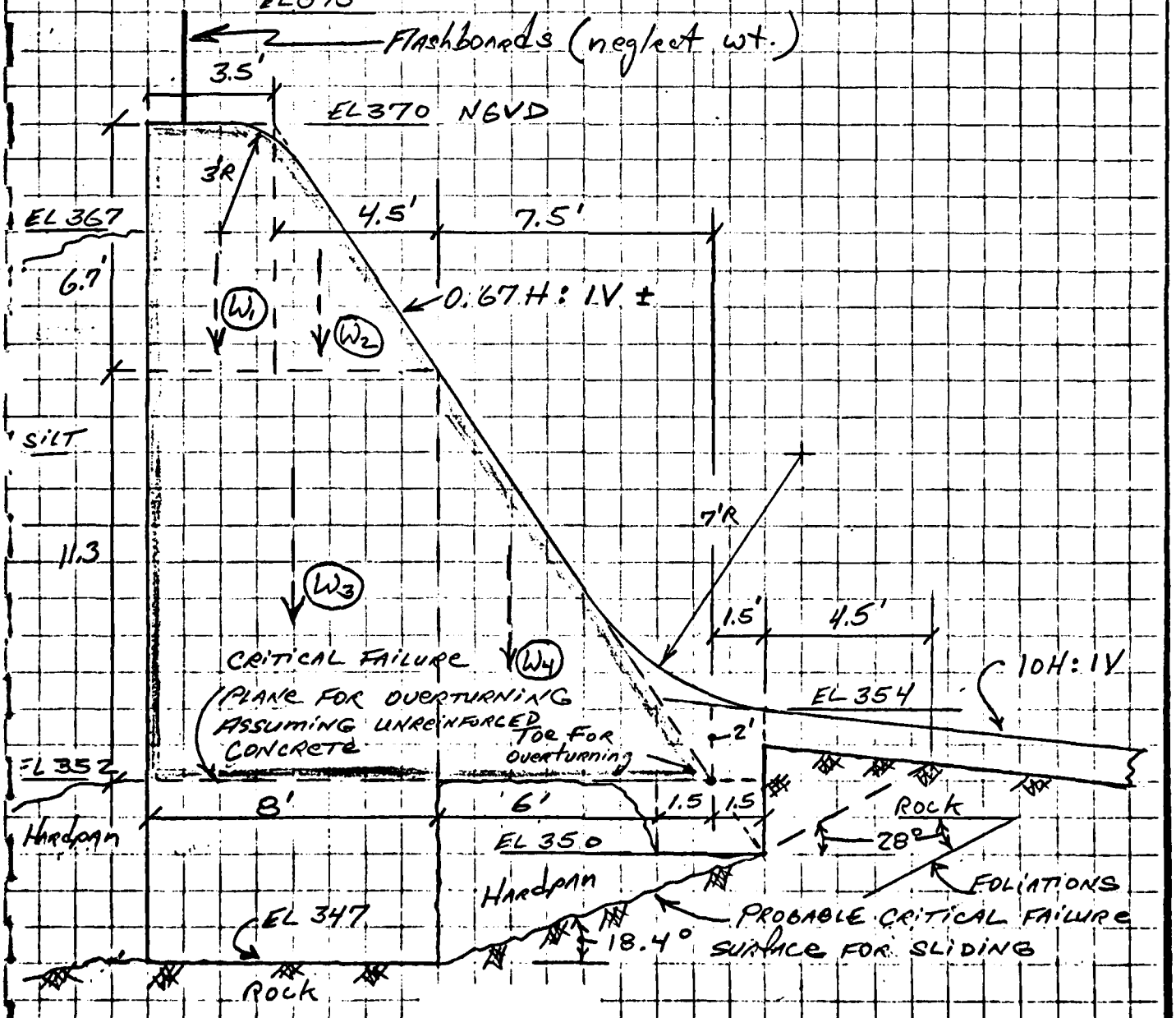
(518) 785-0976

JOB TILLSON LAKE DAM
SHEET NO. 1 OF 18
CALCULATED BY JHB DATE 5/16/81
CHECKED BY DATE
SCALE 1/4" = 1'

STABILITY ANALYSIS OF SPILLWAY

CROSS SECTION FOR ANALYSIS (10' from right training wall, less than max. height and somewhat free from effects of lateral support of training wall, see Photos A-6A, 6B & 7B & dwg. Appendix G-1, NGVD base 90' higher than old dwg. base)



Form CIM-405A

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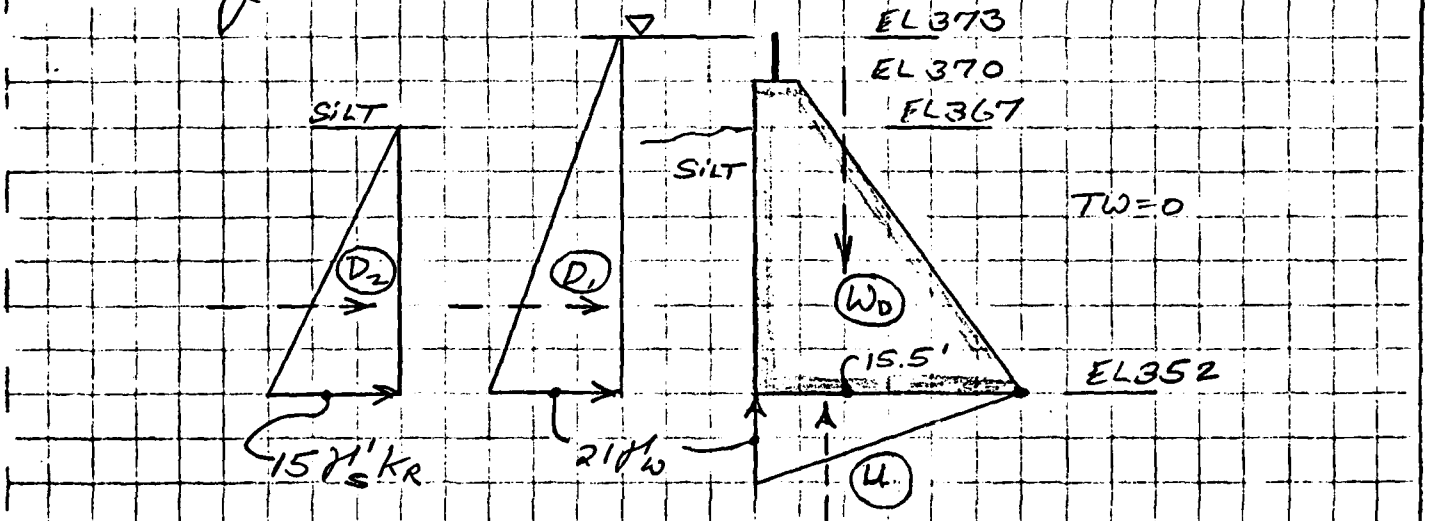
JOB TILLSON LAKE DAM
SHEET NO. 2 OF 18
CALCULATED BY JAB DATE 5/16/81
CHECKED BY DATE
SCALE None

Dead Load	Volume	× Unit Wt(γ) =	W	× Arm about toe =	Horiz. Moment M
W_1	3.5×6.7×1	0.150 k/cf	3.52 k	(3.5/2)+4.5+7.5	48.37
W_2	1/2×4.5×6.7×1	0.150	2.26	(4.5×2/3)+7.5	23.74
W_3	8×11.3×1	0.150	13.56	(8/2)+7.5	155.94
W_4	1/2×7.5×11.3	0.150	6.36	7.5×2/3	31.78
		W_D = 25.70 k		ΣM = 259.83 Ftk	

CASE 1 - Normal pool at flashboard crest, full HW uplift, silt 3' below spillway by observation, negligible TW by observation

Overturning



Resisting Forces × Moment Arm about toe = M_R

W_D = dead load = 25.70 k As before × As before = 259.83 Ftk

Driving Forces M_D

D_1 = water pressure
= (1/2 × 21 × 0.0624) 21 = 13.76 k × 21/3 = 96.31

D_2 = submerged silt pressure - where γ'_s = 90 #/CF - 62.4
γ'_s = 27.6, say γ'_s = 30 #/CF = 0.030 ksf, ξ
K_R = Coeff. of horiz. earth pressure at rest = 0.5
= (1/2 × 15 × 0.03 × 0.5) 15 = 1.69 k × 15/3 = 8.44

Form CTM-405A

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(518) 783-0976

JOB TILLSON LAKE DAM
SHEET NO. 3 OF 18
CALCULATED BY JAB DATE 5/16/81
CHECKED BY DATE
SCALE None

CASE 1 - Overturning (Cont'd) x Moment Arm about toe = M_o

U = HW uplift

$= (1/2 \times 21 \times 0.0624)\,15.5 = 10.16\,k$ $\quad 15.5 \times 2/3 = 104.94$

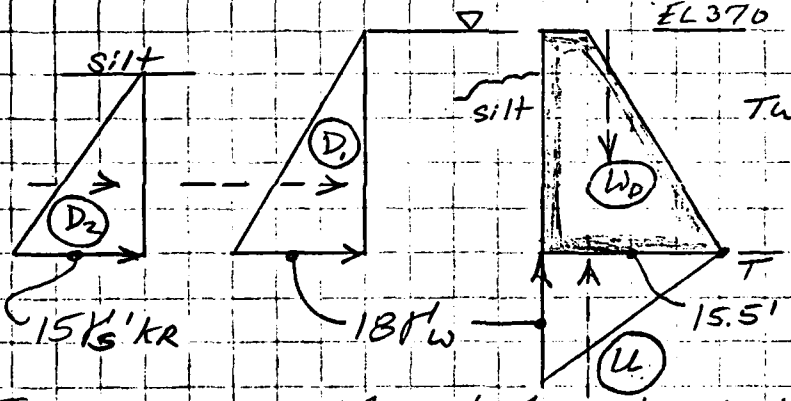
209.69 Ft k

$FS = \Sigma M_R / \Sigma M_o = 259.83 / 209.69 = (1.24)$

Resultant from toe $= d = \Sigma M_T / \Sigma V = \frac{\Sigma M_R - \Sigma M_o}{W_D - U} = \frac{259.83 - 209.69}{25.70 - 10.16}$

$d = \frac{+50.14}{15.54} = +3.23' \times \frac{b}{15.5} = (+0.22b) < 1/3\,b$

CASE 1A - Same as case 1, except no flashboards



Resisting Forces x Moment Arm about toe = M_R

W_D = Head load = 25.70 k per sheet 2 x sheet 2 $= 259.83$ Ft k

Driving Forces $\qquad M_D$

D_1 = Water pressure $= (1/2 \times 18 \times 0.0624)\,18 = 10.11\,k \times 18/3 = 60.65$

D_2 = sub. silt press. = same as Case 1, sheet 2 $= 8.44$

U = HW uplift $= (1/2 \times 18 \times 0.0624)\,15.5 = 8.70 \times 15.5 \times 2/3 = 89.95$

159.04 Ft k

$FS = \Sigma M_R / \Sigma M = 259.83 / 159.04 = (1.63)$

Resultant from toe = $d = \Sigma M_T / \Sigma V = \frac{\Sigma M_R \Sigma M_o}{W_D - U} = \frac{259.83 - 159.04}{25.70 - 8.70}$

$d = 100.79 / 17.0 = 5.93' \times \frac{b}{15.5} = (0.38b) > 1/3\,b$

FORM CTM-35A

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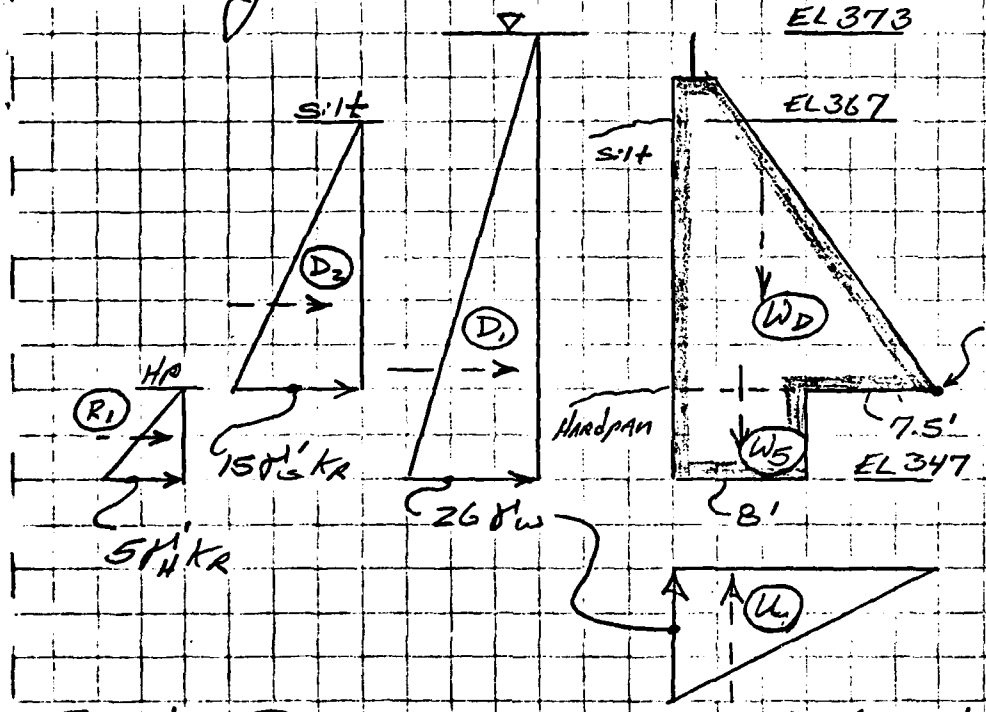
JOB TILLSON LAKE DAM
SHEET NO. 4 OF 18
CALCULATED BY JMR DATE 6/2/81
CHECKED BY DATE
SCALE None

CASE 1B → - Same as Case 1, except assume that u/s shear key can help resist overturning due to:
- unusually large cross section of key
- possible reinforcement in key to take tension

Overturning



Resisting Forces × Moment Arm about toe = M_R

W_D = dead load = 25.70 k per sheet 2 × sheet 2 = 259.83 Ftk

W_S = dead load of u/s shear key
= 5 × 8 × 0.150 = 6 k × $\left(\frac{8}{2} + 7.5\right)$ = 69.0

R_1 = submerged hardpan pressure where
γ'_H = 140 #/CF - 62.4 = 77.6 say 75 #/CF = 0.075 ksf
K'_R = coeff. of horiz. earth press. at rest = 0.5
= $\left(\frac{1}{2} \times 5 \times 0.075 \times 0.5\right)5 = 0.47$ k × $5 \times \frac{2}{3}$ = 1.56

= 330.39 Ftk
M_R

Driving Forces

D_1 = water pressure
= $\left(\frac{1}{2} \times 26 \times 0.0624\right)26 = 21.09$ k × $\left(\frac{26}{3} - 5\right)$ = 77.33

D_2 = submerged silt pressure = same as Case 1, sheet 2 → = 8.44

Form CTM-405 A

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JOB TILLSON LAKE DAM
SHEET NO. 5 OF 18
CALCULATED BY JMB DATE 6/2/81
CHECKED BY DATE
SCALE None

CASE 1B - Overturning (cont'd)

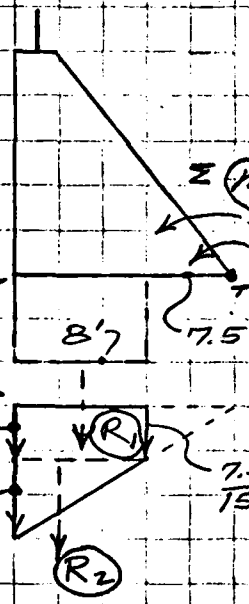
U_1 = 1/W uplift
$= (1/2 \times 26 \times 0.0624)15.5 = 12.57\text{k}$ x $15.5 \times 2/3 =$ 129.93
215.70 Ftk

$FS = \Sigma M_R / \Sigma M_O = 330.39/215.70 = (1.53)$

Resultant from toe $= d = \Sigma M_T / \Sigma V = \frac{\Sigma M_R - \Sigma M_O}{W_D + W_S - U} = \frac{330.39 - 215.70}{25.70 + 6 - 12.57}$

$d = \frac{+114.69}{19.13} = 6.00 \times \frac{b}{15.5} = (0.39b) > 1/3\,b$

What is Max. Tensile Stress in Concrete?



ΣM_R = 330.39 w/ key
$\Sigma M'_R$ = 259.83 w/o key

T_c = max. tensile stress in concrete

$M_{R_1} = \frac{7.5}{15.5} T_c\, 8 \left(\frac{8}{2} + 7.5\right) = 44.52\, T_c$

$M_{R_2} = \left[1/2 \left(1 - \frac{7.5}{15.5}\right) T_c\, 8\right]\left[\frac{8 \times 2}{3} + 7.5\right] = \left[2.06\, T_c\right](12.83) = 26.49\, T_c$

$M_{R_1} + M_{R_2} + \Sigma M'_R = \Sigma M_R$

$(44.52 + 26.49)T_c = 330.39 - 259.83$

$T_c = \frac{70.56}{71.01} = 0.99$ ksf

$T_c = 0.99$ ksf $\cong$ (7 psi)

$T_{c\,allowable} = 3$ to $6\sqrt{f'_c}$ (ACI Code), Assume $f'_c = 3000$ psi conc.

$T_{c\,a} = 3$ to $6\sqrt{3000} \cong 160$ to 320 psi.

$T_c = 7$ psi $<<$ 160 psi $= T_{c\,a}$ ok,
but tension in concrete not a good condition unless reinforcement present.

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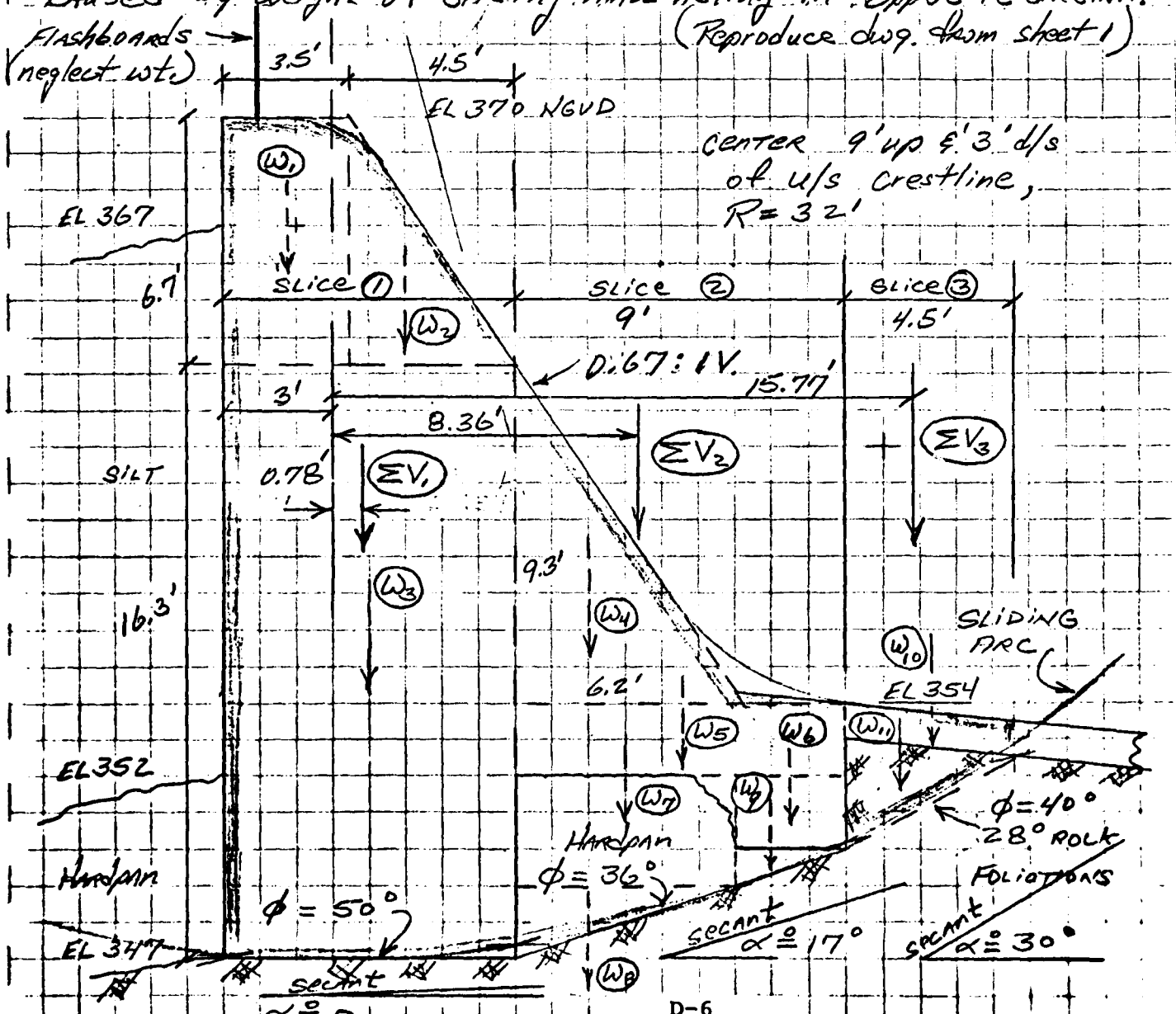
JOB TILLSON LAKE DAM
SHEET NO. 6 OF 18
CALCULATED BY GPB DATE 6/15/81
CHECKED BY DATE
SCALE 1/4" = 1'

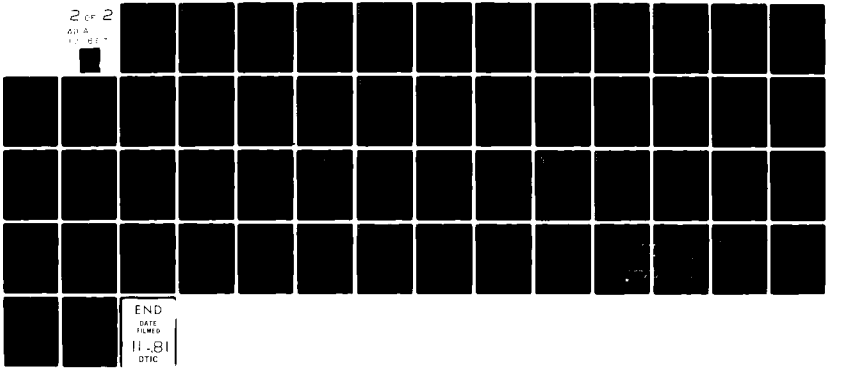
CASE I - SLIDING Rock foliations strike N 30° E - almost parallel w/ spillway crest - but dip 28° N., i.e. u/s, see dwg. Appendix G-1 for direction. Since dip angle is large and is against direction of sliding, critical failure surface is a circular arc of best fit. To analyze use method of slices (Reference 30) & compute moments about center of arc. This takes into account reduction in net driving moment caused by weight of sliding mass acting in opposite direction.

(Reproduce dwg. from sheet 1)



Form CIM-40D A

AD-A106 657 MALE (C T) ASSOCIATES SCHENECTADY NY F/G 13/13
NATIONAL DAM INSPECTION PROGRAM. TILLSON LAKE DAM (NY00083), LO--ETC(U)
AUG 81 K J MALE, W M SMITH DACW51-81-C-0014
UNCLASSIFIED NL



END
DATE FILMED
11-81
DTIC

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JOB TILLSON LAKE DAM
SHEET NO. 7 OF 18
CALCULATED BY MB DATE 6/15/81
CHECKED BY DATE
SCALE None

CASE I - SLIDING (cont'd)

		Horiz. Moment Arm about ARC center	M
Vertical Load - Slice 1			
W_1 = Concrete, same as on sheets 1 & 2			
=	= 3.52 K	$-(\frac{3.5}{2} - 0.5)$	= - 4.40
W_2 = Concrete, same as on sheets 1 & 2			
=	= 2.26 K	$(4.5/3 + 0.5)$	= 4.52
W_3 = Concrete			
$= (8' \times 16.3 \times 1)\ 0.150\ kcf$	= 19.56 K	$(8/2 - 3)$	= 19.56
	ΣV_1 = (25.34 K)		ΣM = + 19.68
$\bar{X}_1 = \Sigma M / \Sigma V_1$ = (0.78')			
Vertical Load - Slice 2			
W_4 = Concrete			
$= (1/2 \times 6.2 \times 9.3 \times 1)\ 0.150\ kcf$	= 4.32 K	$6.2/3 + 4.5 + 0.5$	= 30.56
W_5 = Concrete			
$= (2 \times 9 \times 1)\ 0.150$	= 2.70	$9/2 + 4.5 + 0.5$	= 25.65
W_6 = Concrete			
$= (2 \times 3 \times 1)\ 0.150$	= 0.90	$3/2 + 6.0 + 4.5 + 0.5$	= 11.25
W_7 = hardpan c γ_H = 140 #/CF			
$= (3 \times 6 \times 1)\ 0.140$	= 2.52	$6/2 + 4.5 + 0.5$	= 20.16
W_8 = hardpan			
$= (1/2 \times 2 \times 6 \times 1)\ 0.140$	= 0.84	$6/3 + 4.5 + 0.5$	= 5.88
W_9 = hardpan			
$= (1/2 \times 1 \times 3 \times 1)\ 0.140$	= 0.21	$3/3 + 6.0 + 4.5 + 0.5$	= 2.52
	ΣV_2 = (11.49 K)		ΣM = 96.02
$\bar{X}_2 = \Sigma M / \Sigma V_2$ = (8.36')			
Vertical Load - Slice 3			
W_{10} = Concrete			
$= (1 \times 4.5 \times 1)\ 0.150$	= 0.68 K	$4.5/2 + 9 + 4.5 + 0.5$	= 10.97
W_{11} = Rock e γ_R = 165 #/CF typical for the Slate or Argillate observed			
$= (1/2 \times 3 \times 4.5 \times 1)\ 0.165$	= 1.11 K	$4.5/3 + 9 + 4.5 + 0.5$	= 17.26
	ΣV_3 = (1.79 K)		ΣM = 28.23
$\bar{X}_3 = \Sigma M / \Sigma V_3$ = (15.77')			

FORM CM-403A

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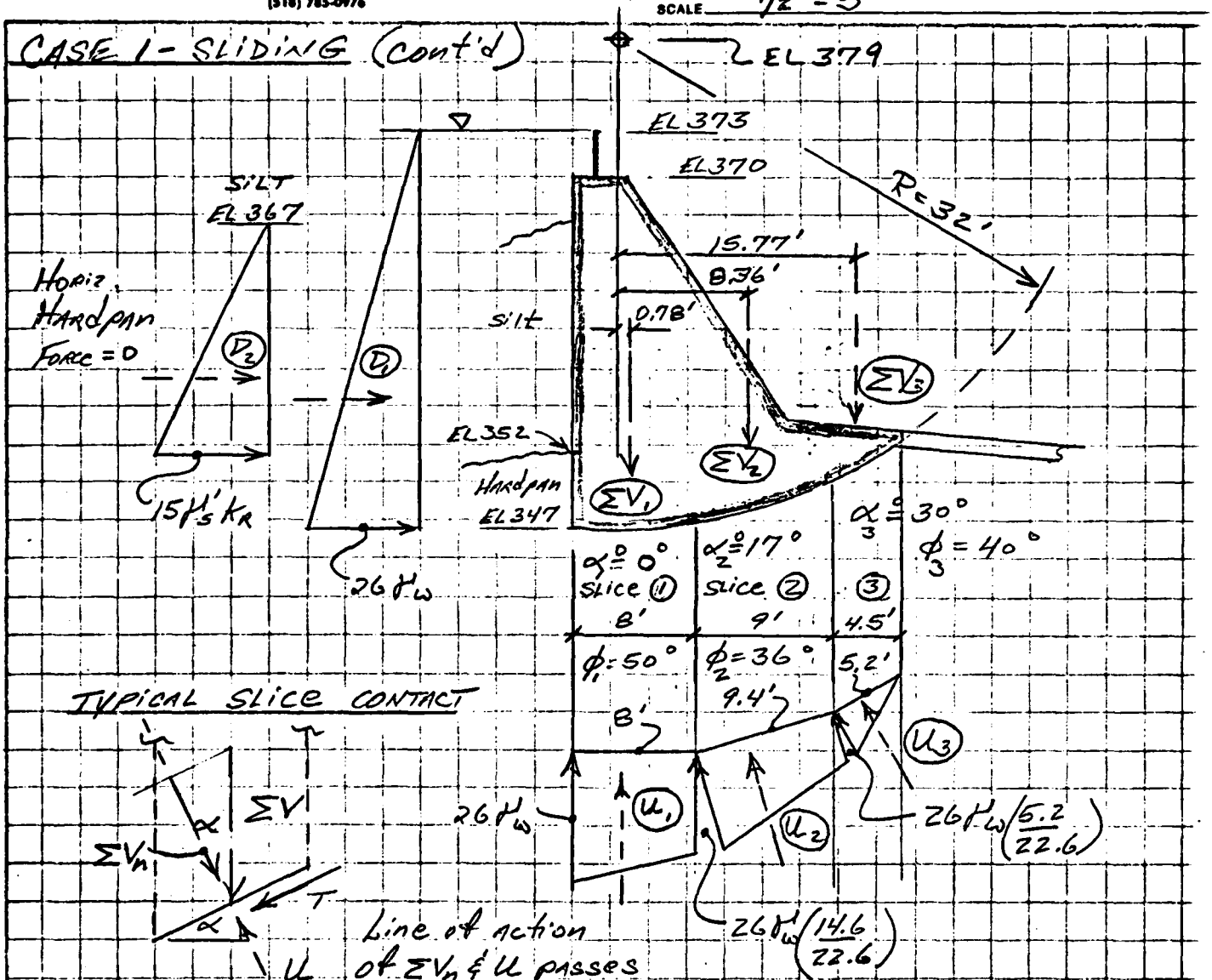
JOB Tillson Lake Dam
SHEET NO. 8 OF 18
CALCULATED BY [illegible] DATE 6/16/81
CHECKED BY DATE
SCALE 1/2" = 5'

CASE 1 - SLIDING (cont'd)



TYPICAL SLICE CONTACT

Line of action of ΣV_n & U passes thru ARC center, ∴ no moment

T = shear force mobilized by driving forces & developed from normal effective force on slice contact

$= N \tan\phi + cA$, c = cohesion = 0 for all slices

$N = \Sigma V_n - U = \Sigma V \cos\alpha - U$

So $T = [\Sigma V \cos\alpha - U] \tan\phi$

where ϕ = angle of sliding friction

For slice 1, concrete/rock contact, assume $\phi = 50°$ due to rough interface caused by edges of rock foliations

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JOB Tillson Lake Dam
SHEET NO. 9 OF 18
CALCULATED BY PMB DATE 6/16/81
CHECKED BY DATE
SCALE None

CASE I - SLIDING (cont'd)

For Slice 2, hardpan/rock contact, Assume $\phi_2 = 36°$ avg. for till
" " 3, contact along rock foliations, Assume $\phi_3 = 40°$

$U_1 = Hw$ uplift $= \left(26 H_w + 26 H_w \frac{14.6}{22.6}\right)\frac{B}{2} = 26 \times 0.0624 \left(1 + \frac{14.6}{22.6}\right) 4$

$= 10.68\,K = U_1$

$U_2 = 26 \times 0.0624 \left(\frac{14.6 + 5.2}{22.6}\right)\frac{9.4}{2} = 6.68\,K = U_2$

$U_3 = 1/2 \times 26 \times 0.0624 \left(\frac{5.2}{22.6}\right) 5.2 = 0.97\,K = U_3$

Resisting Forces		Moment Arm About Arc Center = M_R
T_1 = shear force along slice 1 contact		
$= (\Sigma V_1 \cos\alpha_1 - U_1) \tan\phi_1$		
$= (25.34 \cos 0° - 10.68) \tan 50°$	$= 17.47\,K$	
$T_2 = (11.49 \cos 17° - 6.68) \tan 36°$	$= 3.13\,K$	
$T_3 = (1.79 \cos 30° - 0.97) \tan 40°$	$= 0.49\,K$	
	$21.09\,K \times 32' =$	674.88

Driving Forces

D_1 = Water pressure
$= 1/2 \times 26 \times 0.0624 \times 26 = 21.09\,K \times [(26 \times 2/3) + 6] = 492.13$

D_2 = Submerged silt pressure
where $H_s' = 0.03$ ft & $K_R' = 0.5$ (see sheet 2)
$= 1/2 \times 15 \times 0.03 \times 0.5 \times 15 = 1.69\,K \times [(15 \times 2/3) + 12] = 37.13$

ΣV_1 = wt. of slice 1 from sheet 7 $= -25.34\,K \times 0.78' = -19.77$
ΣV_2 = " " " 2 " " " $= -11.49 \times 8.36' = -96.06$
ΣV_3 = " " " 3 " " " $= -1.79 \times 15.77' = -28.23$
$\Sigma M_D = 385.20$

$FS = \Sigma M_R / \Sigma M_D = 674.88 / 385.20 = \boxed{1.75}$

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SHEET NO. 10 OF 18
CALCULATED BY WRB DATE 6/17/81
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SCALE 1/2" = 5'

CASE 1 - SLIDING (cont'd)

Check Possible Horiz. Failure Plane thru U/S Shear Key

R_s = resistance to sliding = at least $V_c A$ where A = shear area of concrete key & V_c = conc. shear strength, $V_c \doteq 2.0\sqrt{f'_c}$ (ACI Code), Assume $f'_c = 3000$ psi, then $V_c = 2.0\sqrt{3000} = 110$ psi, say $V_c \doteq 100$ psi $= 14.4$ ksf

$R_s = 14.4(8 \times 1) = 115.2$ K

H_s = horizontal driving force $= D_1 + D_2$

D_1 = water pressure down to failure plane @ EL 352
$= 1/2 \times 21 \times 0.0624 \times 21 = 13.76$ K (see sheet 2)

D_2 = submerged silt pressure = 1.69 K from sheets 8 & 9

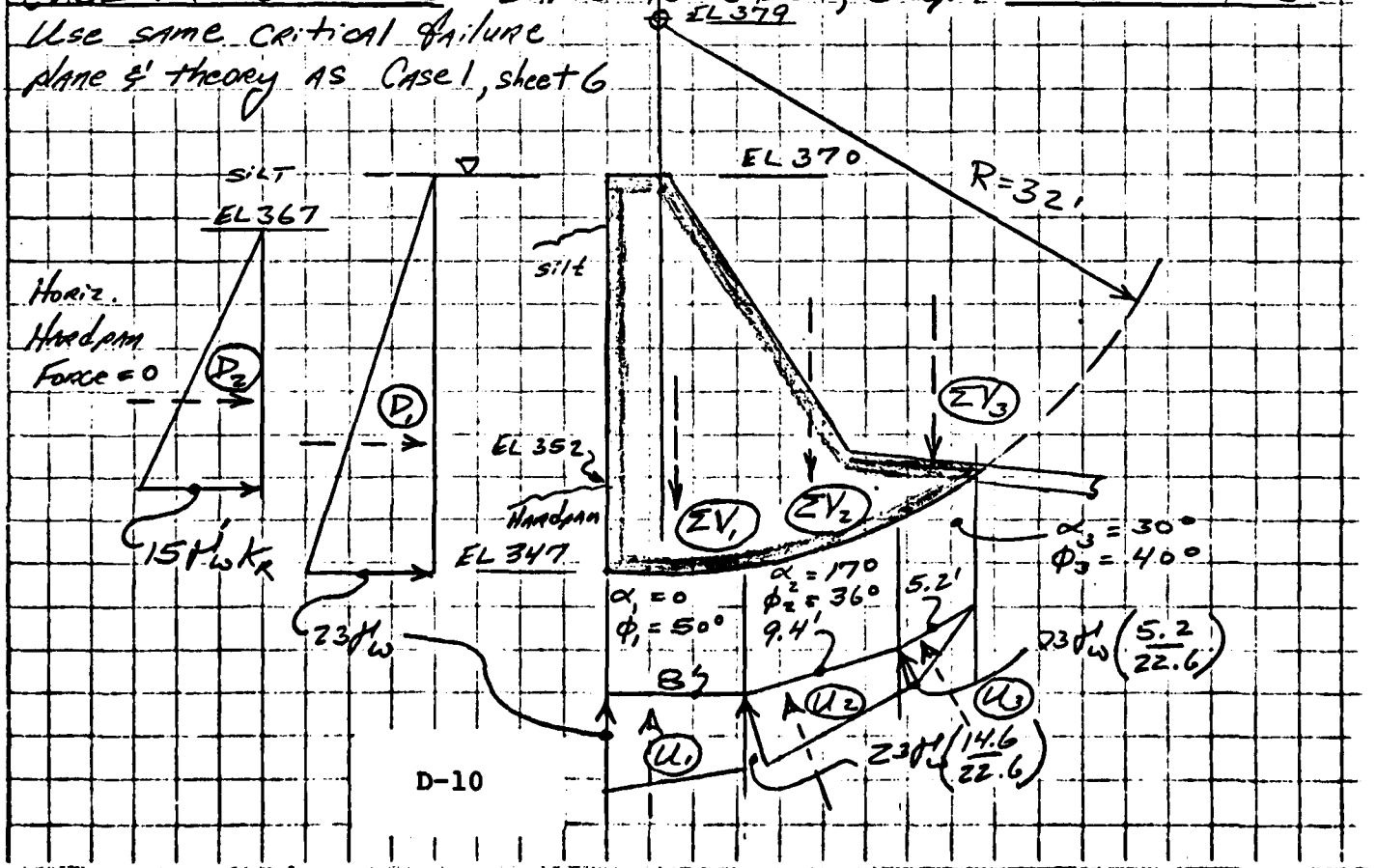
$H_s = D_1 + D_2 = 15.45$ K

$FS = R_s / H_s = 115.2 / 15.45 = 7.46 >> 1.75$ using circular arc

∴ Circular arc remains as critical failure surface

CASE 1A - SLIDING - Same as Case 1, except no flashboards

Use same critical failure plane & theory as Case 1, sheet 6



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SHEET NO. 11 OF 18
CALCULATED BY GAB DATE 6/17/81
CHECKED BY DATE
SCALE None

CASE 1A - SLIDING (cont'd)

U_1 = HW uplift = $23 \times 0.0624 \left(1 + \frac{14.6}{22.6}\right) \frac{B}{2} = 9.45$ k

$U_2 = 23 \times 0.0624 \left(\frac{14.6 + 5.6}{22.6}\right) \frac{9.4}{2} = 6.03$ k

$U_3 = \frac{1}{2} \times 23 \times 0.0624 \left(\frac{5.2}{22.6}\right) 5.2 = 0.86$ k

Resisting Forces — × Moment Arm About Ave Center = M_R

T_1 = shear force along slice 1 contact = $(\Sigma V_1 \cos\alpha - U_1) \tan\phi$
where ΣV all slices same as Case 1A, sheet 7

$T_1 = (25.34 \cos 0° - 9.45) \tan 50° = 18.94$
$T_2 = (11.49 \cos 17° - 6.03) \tan 36° = 3.60$
$T_3 = (1.79 \cos 30° - 0.86) \tan 40° = 0.58$
$23.12 \text{ k} \times 32' = 739.84$

Driving Forces

D_1 = Water pressure
$= \frac{1}{2} \times 23 \times 0.0624 \times 23 = 16.50 \times \left[(23 \times \frac{2}{3}) + 9\right] = 401.62$
D_2 = submerged silt pressure same as Case 1, sheet 9 = 37.13
ΣV_1 = wt. of slice 1 same as Case 1, sheet 9 = −19.77
ΣV_2 = " " " 2 " " " " " " " = −96.06
ΣV_3 = " " " 3 " " " " " " " = −28.23
$\Sigma M_D = 294.69$

$FS = \Sigma M_R / \Sigma M_D = 739.84 / 294.69 = 2.51$

−144.06

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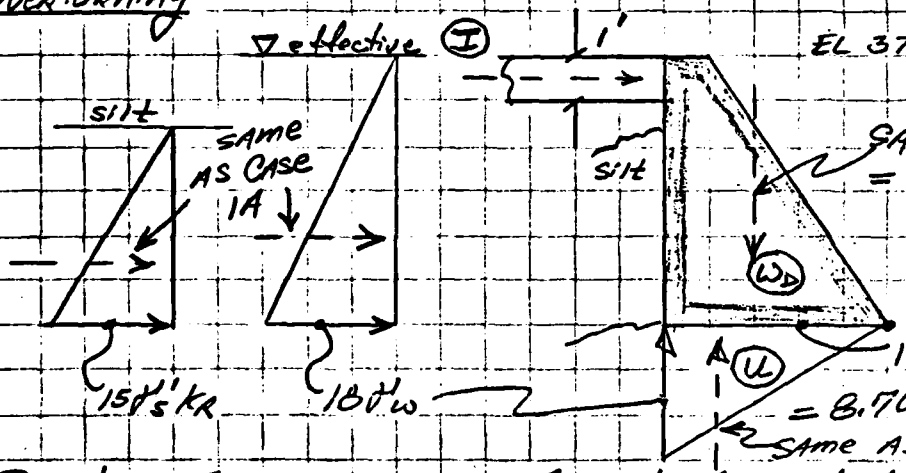
JOB TILLSON LAKE DAM
SHEET NO. 12 OF 18
CALCULATED BY JMB DATE 6/19/81
CHECKED BY DATE
SCALE None

CASE 2 - Normal Pool plus Ice Load. Flashboards are normally removed for winter. Max. recommended ice load (Ref. 1) = 10 K/Ft & ice 2' thick. Use 5K/Ft & ice 1.0' thick as appropriate for conditions at this dam.

Overturning



Resisting Forces	× Moment Arm about toe	=	M_R
Only W_D, same as Case 1A, sheet 3 →		=	259.83
			M_D →
Driving Forces			
HW pressure, silt pressure & uplift same as Case 1A, sheet 3 →			159.04
I = ice load = 5K × (18 − 0.5) =			87.50
		ΣM_D =	246.54

$FS = \Sigma M_R / \Sigma M_D = 259.83 / 246.54 = (1.05)$

Resultant from toe = $d = \Sigma M / \Sigma V = \Sigma M_R - \Sigma M_D = 259.83 - 246.54$ / $W_D - U$ / $25.70 - 8.70$

$d = 13.29 / 17.0 = 0.17' \times b / 15.5' = (0.05b) < 1/3\,b$

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JOB TILLSON LAKE DAM
SHEET NO. 13 OF 18
CALCULATED BY MAR DATE 6/19/81
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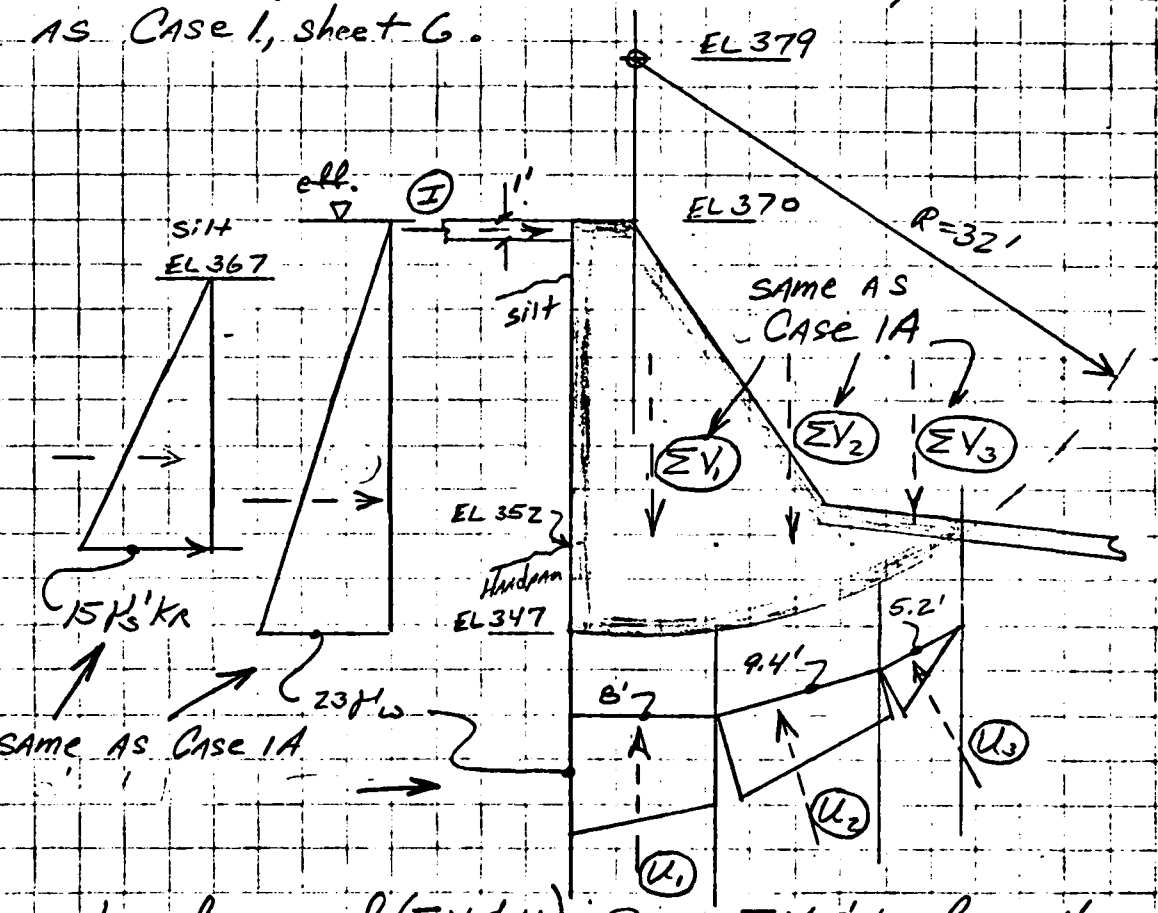
SCALE 1/2" = 5'

CASE 2 - SLIDING Use same critical failure plane & theory AS Case 1, sheet 6.



Resisting shear force = f(ΣV & U). Since ΣV & U for all slices same as for Case 1A, sheet 11, ΣM_R = 739.84

Driving Forces x Moment Arm About ARC center = M_D

HW pressure, silt pressure, uplift & ΣV's same as Case 1A, sheet 11 → = 294.69

I = ice load = 5k x (9+0.5) = 47.50

ΣM_D = 342.19

$FS = \Sigma M_R / \Sigma M_D = 739.84 / 342.19 = (2.16)$

F CT. 05/

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SHEET NO. 14 OF 18
CALCULATED BY JMB DATE 6/17/81
CHECKED BY DATE
SCALE None

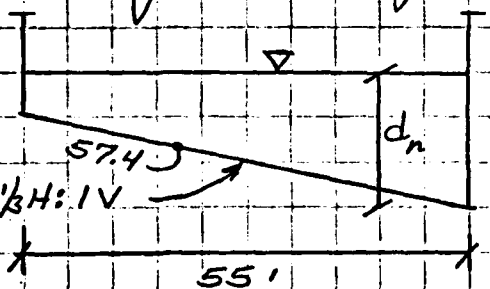
CASE 3 - 1/2 PMF Pool, no flashboards, full headwater & tailwater uplift, remainder SAME AS CASE 1A

Estimate Tailwater For Flood Conditions

For 1/2 PMF & PMF dam is overtopped per H&H Analysis, Table 5.1
1/2 PMF Max. W.S. EL 376.6 - spillway EL 370 = 6.6' over spillway
PMF " " " 378.2 - " " " = 8.2' " "
Per Appendix C-7, $Q_{spillway} = 3.33 L H^{1.5}$, where L = 55' clear
so For 1/2 PMF, H = 6.6', Q = 3105, say Q = (3100) cfs
" PMF, H = 8.2', Q = 4301, say Q = (4300) cfs
Assume uniform flow in spillway discharge channel where:
$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$ (Manning's equation, Ref. 8)

where n = roughness coefficient ≅ 0.014 for concrete channel
A = cross sectional Area of flow, ft²
R = hydraulic radius = A/wetted perimeter (P), ft.
S = slope of energy gradient, assume equal to avg. slope of channel, see dwg. Appendix G-1 & sheet 6 of this stability Analysis, S = 10H:1V = 0.10

Average Spillway Discharge Channel X-Sect. For 7 to 8' D/S of spillway toe, looking downstream (see dwg. Appendix G-1)



$Q = \frac{1.486}{0.014} (0.10)^{1/2} A R^{2/3}$

$Q \cong 33.57 A R^{2/3}$

d_n	Top Width	A	P	$R^{2/3} = (A/P)^{2/3}$	$Q \cong 33.57 A R^{2/3}$
3	10	15	13.4	1.08	543 cfs
6	20	60	26.9	1.71	3448 > 1/2 PMF
4	13.3	26.6	17.9	1.30	1164 cfs
5	16.7	41.8	22.4	1.52	2131 ← 1/2 PMF
6	20	60	26.9	1.71	3448 ← PMF
7	23.3	81.6	31.3	1.90	5205

105. CT

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CALCULATED BY JPA DATE 6/19/81
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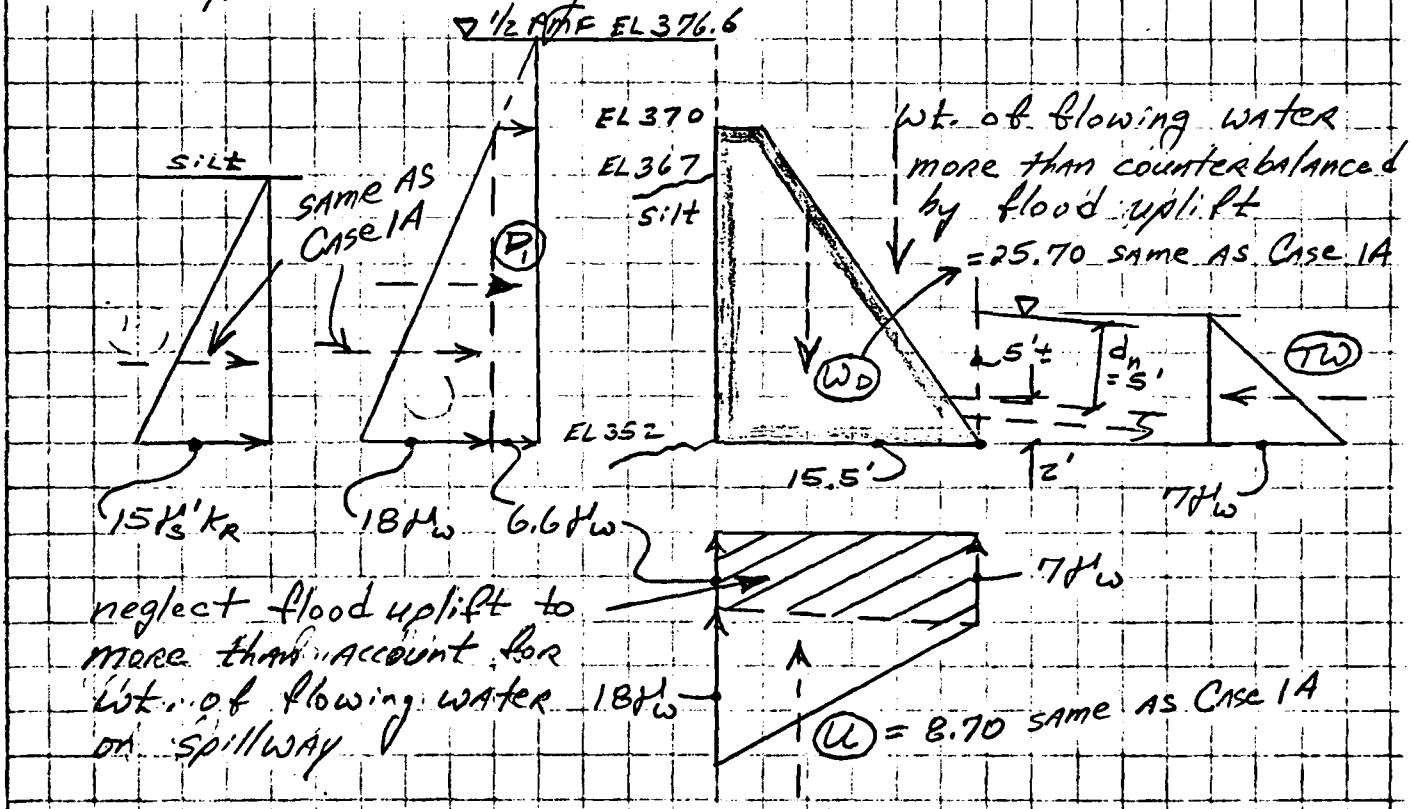
SCALE None

CASE 3 - Estimate Tailwater (cont'd)

By interpolation, for 1/2 PMF $Q = 3100$ cfs, $d_n \doteq 5.7'$, say (5.0')
& for PMF $Q = 4300$ cfs, $d_n \doteq 6.5'$, say (6.0')
(Round down to be conservative for stability)

CASE 3 - 1/2 PMF Overturning



Resisting Forces	x Moment Arm About Toe	=	M_R
W_D = same as Case 1A, sheet 3		=	259.83
TW = flood tailwater = $1/2 \times 7 \times 0.0624 \times 7 = 1.53 \times 7/3$		=	3.57
		ΣM_R =	263.40

Driving Forces			M_D
Normal HW pressure, silt pressure & uplift			
P - same as Case 1A, sheet 3 →		=	159.04
P_1 = flood HW pressure			
$= 6.6 \times 0.0624 \times 18 = 7.41 \times 18/2$		=	66.72
		ΣM_D =	225.76

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SHEET NO. 16 OF 18
CALCULATED BY JMB DATE 6/19/81
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SCALE None

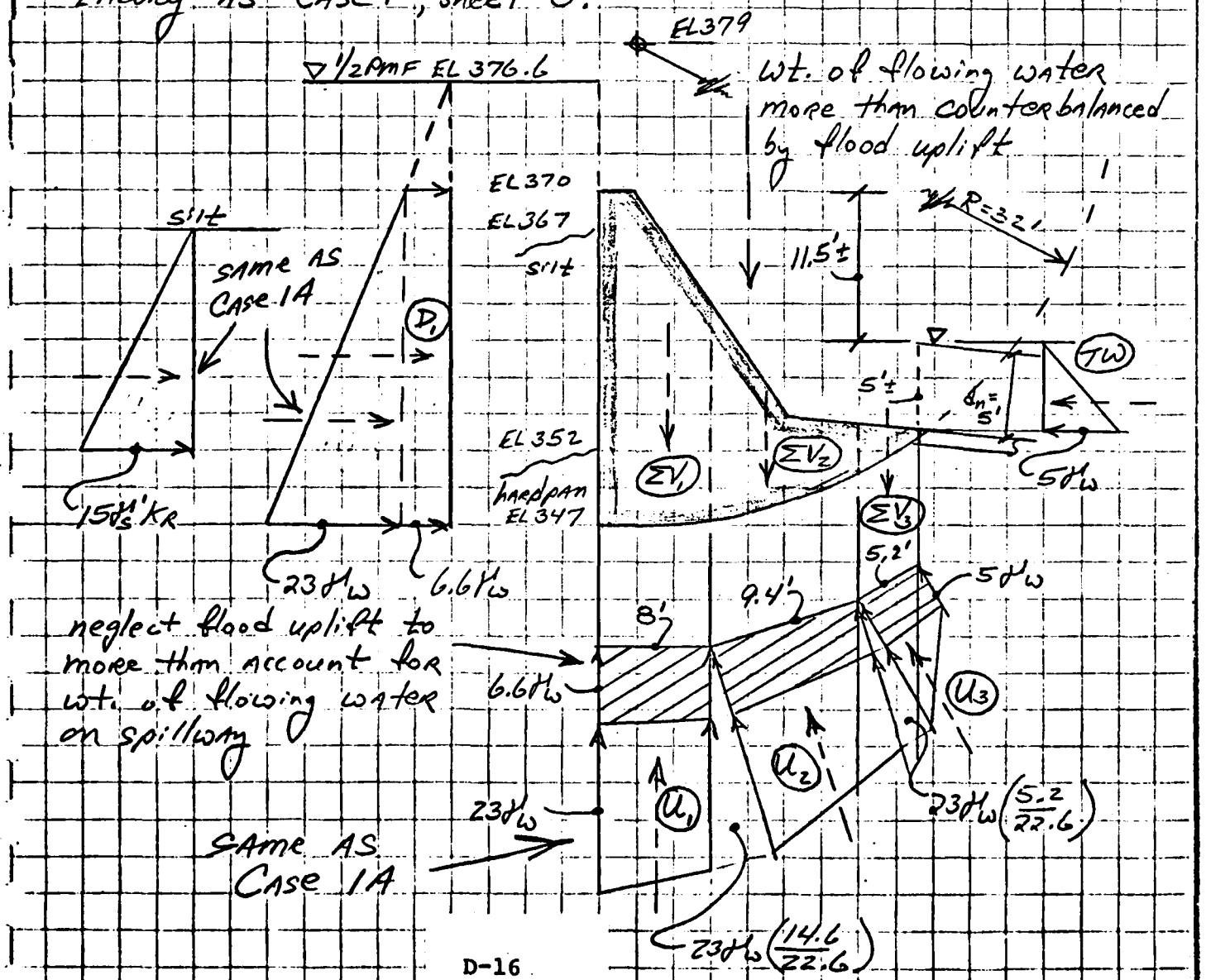
CASE 3 - Overturning (cont'd)

$FS = \Sigma M_R / \Sigma M_O = 263.40 / 225.76 = (1.17)$

Resultant from toe $= d = \Sigma M_T / \Sigma V = \frac{\Sigma M_R - \Sigma M_O}{W_D - U} = \frac{263.40 - 225.76}{25.70 - 8.70}$

$d = 37.64 / 17.0 = 2.21' \times \frac{b}{15.5} = (0.14\,b) < 1/3\,b$

CASE 3 - SLIDING Use same critical failure plane & theory as Case 1, sheet 6.



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SHEET NO. 17 OF 18
CALCULATED BY DMR DATE 6/19/81
CHECKED BY DATE
SCALE None

CASE 3 - SLIDING (cont'd)

Resisting shear force = $f(\Sigma V \& \Sigma U)$. Since ΣV & ΣU for all slices same as for Case 1A, sheet 11, ΣM_R = 739.84

Driving Forces × Moment Arm About Arc Center = M_D

Normal HW pressure, silt pressure, uplift & ΣV's same as Case 1A, sheet 11 → = 294.69

P_1 = flood HW pressure = $6.6 \times 0.0624 \times 23 = 9.47$ × $(23/2 + 9)$ = 194.18

TW = flood tailwater $\ominus 1/2 \times 5 \times 0.0624 \times 5 = \ominus 0.78$ × $[(5 \times 2/3) + 11.5 + 9]$ = $\ominus$18.59

ΣM_D = 470.28

$FS = \Sigma M_R / \Sigma M_D = 739.84 / 470.28 = (1.57)$

CASE 4 - PMF Overturning - Refer to Case 3, sheet 15 methodology.

TW $d_n \cong 6'$, so $d_v \cong 6'$ & d_v Above toe = 8'

Resisting Forces × Moment Arm About toe = M_R

W_D = same as Case 3, sheet 15 = 259.83

TW = flood tailwater = $1/2 \times 8 \times 0.0624 \times 8 = 2.00 \times 8/3$ = 5.32

ΣM_R = 265.15

Driving Forces M_D

Normal HW pressure, silt pressure & uplift same as Case 3, sheet 15 = 159.04

P_1 = flood HW pressure @ EL 378.2, 8.2' Above spillway = $8.2 \times 0.0624 \times 18 = 9.21 \times 18/2$ = 82.89

ΣM_D = 241.93

$FS = \Sigma M_R / \Sigma M_D = 265.15 / 241.93 = (1.10)$

Resultant from toe = $d = \frac{\Sigma M_T}{\Sigma V} = \frac{\Sigma M_R - \Sigma M_D}{W_D - U} = \frac{265.15 - 241.93}{25.70 - 8.70}$

$d = 23.22 / 17.0 = 1.37' \times b/15.5 = (0.09b) < 1/3\,b$

Form C1M-405A

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JOB TILLSON LAKE DAM
SHEET NO. 18 OF 18
CALCULATED BY WB DATE 6/19/81
CHECKED BY DATE
SCALE None

CASE 4 - PMF SLIDING - Refer to Case 3, sheet 16 methodology.
TW $d_n \doteq 6'$, so $d_v \doteq 6'$
Resisting shear force = $f(\Sigma V \& U)$. Since $\Sigma V \& U$ for all slices same as for Case 3, sheet 17, $\Sigma M_R = 739.84$

Driving Forces — M_D
Normal HW pressure, silt pressure, uplift & ΣV's same as Case 3, sheet 17 = 294.69
D = flood HW pressure @ EL 378.2, 8.2' above spillway
$= 8.2 \times 0.0624 \times 23 = 11.77 \times (23/2 + 9) =$ 241.26
TW = flood tailwater = $\ominus 1/2 \times 6 \times 0.0624 \times 6$
$= \ominus 1.12 \times [(6 \times 2/3) + 10.5 + 9] =$ $\ominus$26.40
$\Sigma M_D =$ 509.55

$FS = \Sigma M_R / \Sigma M_D = 739.84 / 509.55 =$ (1.45)

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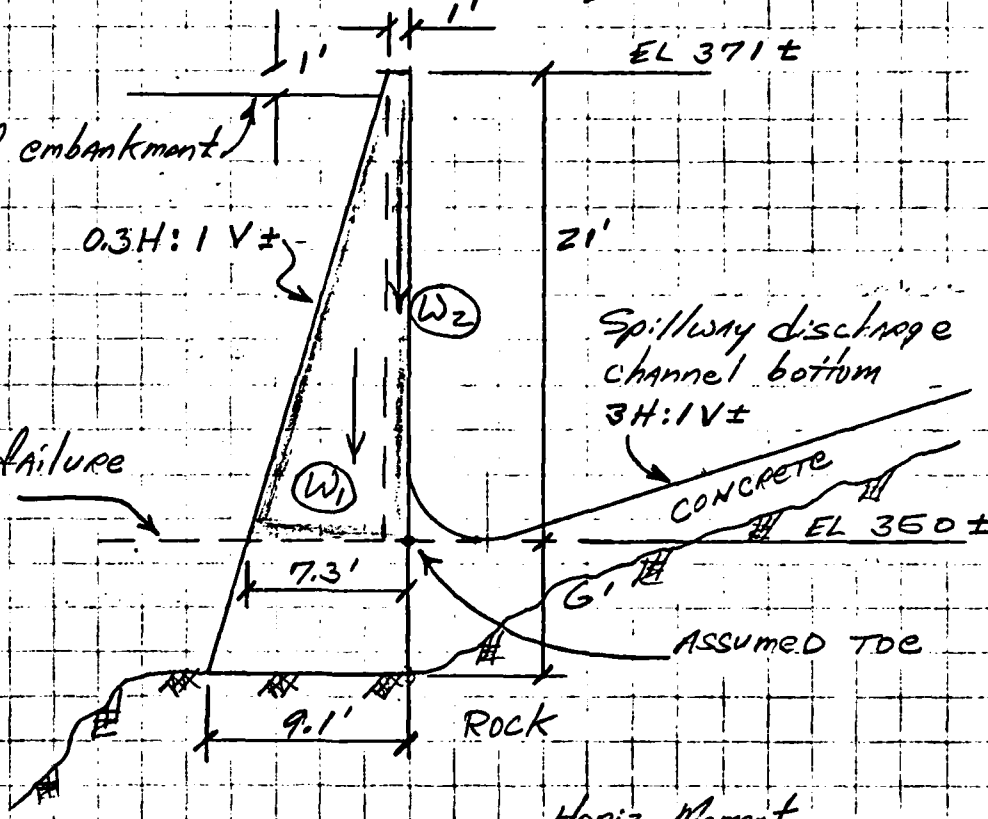
JOB TILLSON LAKE DAM
SHEET NO. 1 OF 4
CALCULATED BY PAB DATE 5/11/81
CHECKED BY DATE
SCALE 1/4" = 2'

STABILITY ANALYSIS OF RIGHT TRAINING WALL OF SPILLWAY

CROSS SECTION FOR ANALYSIS (At max. unsupported height which occurs just d/s of spillway toe, see dwg. Appendix G-1 & photos Appendix A)



Dead Load	Volume	× Unit Wt. (γ) =	W	× Horiz. Moment Arm About Toe =	M
W_1	$\frac{6.3}{2} \times 21 \times 1$	0.150 K/Ft³	9.92 K	$\frac{6.3}{3}+1$	30.75
W_2	$1 \times 21 \times 1$	0.150	3.15 K	0.5	1.58
		W_D =	13.07 K	ΣM =	32.33 Ft K

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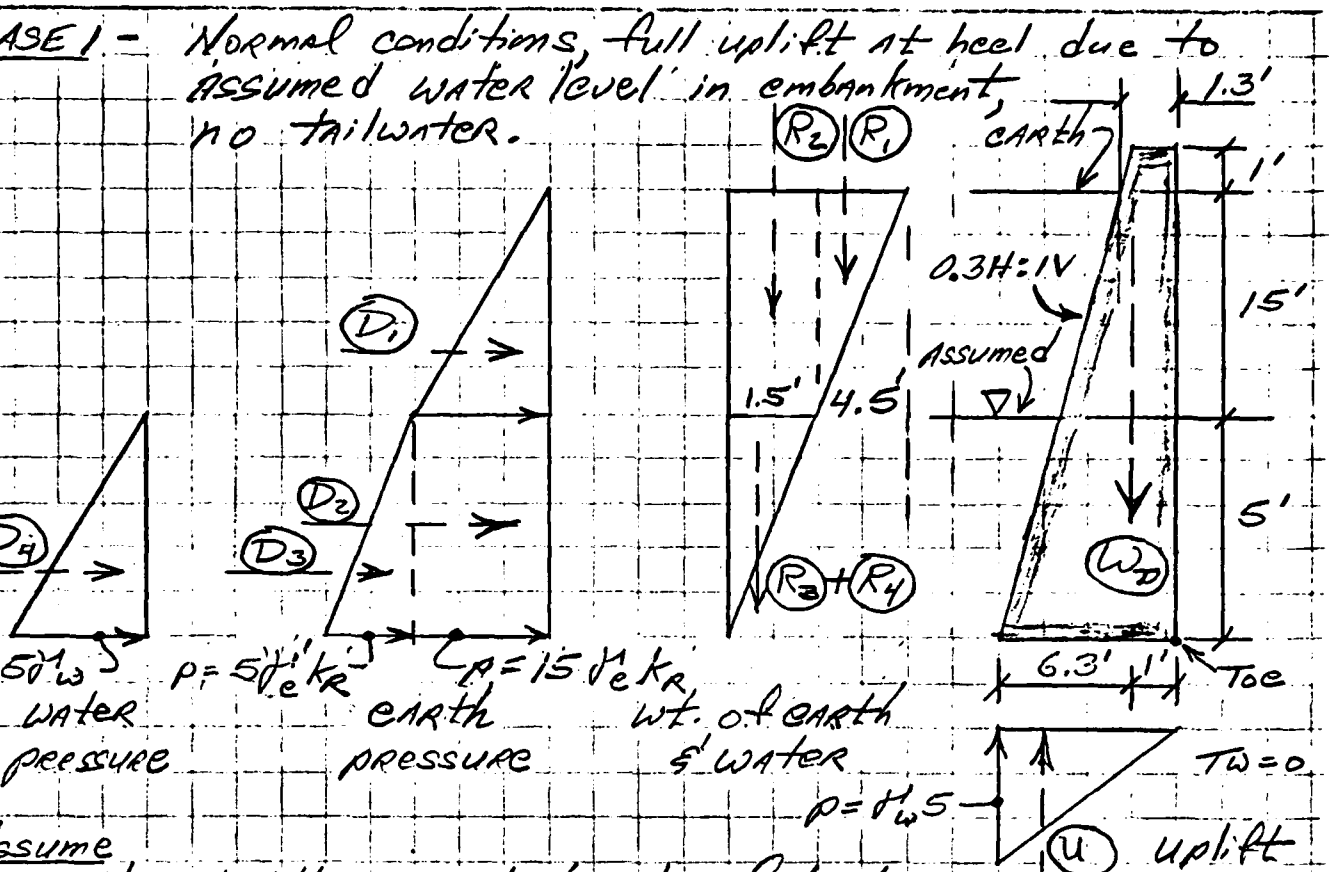
JOB TILLSON LAKE DAM
SHEET NO. 2 OF 4
CALCULATED BY JMR DATE 5/12/81
CHECKED BY DATE
SCALE None

CASE I – Normal conditions, full uplift at heel due to assumed water level in embankment, no tailwater.



water pressure — earth pressure — wt. of earth & water

Assume

- earth = lightly compacted glacial till
  $\gamma_e = 135\ \#/Ft^3$, $\gamma'_e = 135 - 62.4 = 72.6 \doteq 75\ \#/Ft^3$
- coefficient of horizontal earth pressure at rest = $K_R = 0.5 \pm$ empirical

Resisting Forces		Overturning Moment Arm	M_R
W_D = dead load = 13.07 as before		as before	32.33
R_1 = wt. of earth = $(4.5/2)15 \times 0.135$ =	4.56K	$(4.5 \times 2/3) + 1.3$ =	19.61
R_2 = wt. of earth = $1.5 \times 15 \times 0.135$ =	3.04	$(1.5/2) + 4.5 + 1.3$ =	19.91
R_3 = wt. of submerged earth = $(1.5/2)5 \times 0.075$ =	0.28	$(1.5 \times 2/3) + 4.5 + 1.3$ =	1.90
R_4 = wt. of water = $(1.5/2)5 \times 0.0624$ =	0.23	$(1.5 \times 2/3) + 4.5 + 1.3$ =	1.56
		ΣM_R =	75.31 Ft K

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SHEET NO. 3 OF 4
CALCULATED BY JRB DATE 5/12/81
CHECKED BY DATE
SCALE None

Driving Forces		x Moment Arm	= M_o
D_1 = earth pressure = $(1/2 \times 15 \times 0.135 \times 0.5)15$	= 7.59 k	$(15/3)+5$	= 75.90
D_2 = earth pressure = $(15 \times 0.135 \times 0.5)5$	= 5.06	5/2	= 12.65
D_3 = submerged earth pressure = $(1/2 \times 5 \times 0.075 \times 0.5)5$	= 0.47	5/3	= 0.78
D_4 = water pressure = $(1/2 \times 5 \times 0.0624)5$	= 0.78	5/3	= 1.30
U = uplift = $(1/2 \times 5 \times 0.0624)7.3$	= 1.14	$7.3 \times 2/3$	= 5.55
			ΣM_o = 96.18 Ft k

Overturning $FS = \Sigma M_R / \Sigma M_o = 75.31 / 96.18 = (0.78) < 1.0$ unstable

Resultant outside base since $FS < 1.0$

Resultant from toe $= d = \frac{\Sigma M_T}{\Sigma V} = \frac{M_R - M_o}{W_D + \Sigma R - U} = \frac{75.31 - 96.18}{13.07 + 8.11 - 1.14}$

$d = \frac{-20.87}{20.04} = -1.04' \times \frac{6}{7.3'} = (-0.146)$

Sliding

- Assume 3000 psi concrete w/ shear stress = $V_c = 2.0\sqrt{f'_c}$ (per ACI Code), $V_c = 2.0\sqrt{3000} = 110$ psi, say $V_c = 100$ psi or $V_c = 14{,}400$ psf

Resistance = R_s = shear of concrete = $V_c A = 14.4 \times 7.3 = 105.12$ k

Sliding Force = Horiz. $H_s = D_1 + D_2 + D_3 + D_4 = 13.90$ k

$FS = R_s / H_s = 105.12 / 13.90 = (7.56) >> 2.0$ ok

Sliding below failure plane should be no problem because of embedment.

Form CTM-405 A

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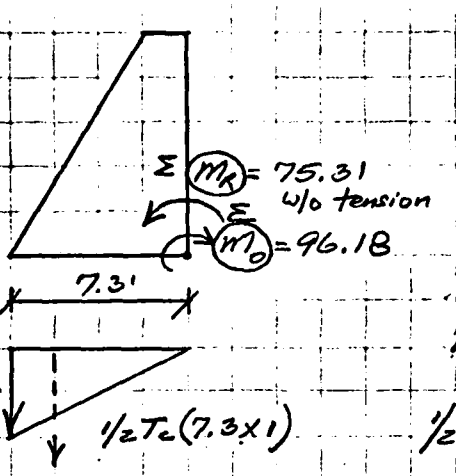
JOB TILLSON LAKE DAM
SHEET NO. 4 OF 4
CALCULATED BY MMR DATE 6/2/81
CHECKED BY DATE
SCALE None

Tensile Stress In Concrete For Overturning Stability



T_c = max. tensile stress in concrete
$T_{c\,allowable}$ = 3 to 6 $\sqrt{f'_c}$ (ACI Code)
Assume f'_c = 3000 psi concrete
T_{cA} = 3 to 6 $\sqrt{3000}$ ≐ 160 to 320 psi

$FS = \frac{\Sigma M_R'}{\Sigma M_O}$, OR $\Sigma M_R' = (FS)\Sigma M_O$

$\frac{1}{2} T_c' 7.3\left(7.3 \times \frac{2}{3}\right) + \Sigma M_R = (FS)\Sigma M_O$

$17.76\, T_c = (FS)\Sigma M_O - \Sigma M_R$

$T_c = \frac{(FS)\,96.18 - 75.31}{17.76}$

FOR FS = 1.0 (barely stable)

$T_c = (1.0)\frac{96.18 - 75.31}{17.76} = 1.18 \text{ ksf} \times \frac{1000\#}{K} \times \frac{SF}{144 in^2} \doteq 8 \text{ psi}$

(8 psi) < 160 psi = T_{cA} OK

FOR FS = 1.5 (Acceptable stability)

$T_c = (1.5)\frac{96.18 - 75.31}{17.76} = 3.88 \text{ ksf} \doteq 27 \text{ psi}$

FOR Resultant in Middle 1/3 (Also req'd for Acceptable stability)

from page 3, $d = \frac{1}{3} b_{min} = \frac{7.3}{3} = \frac{\Sigma M_T}{\Sigma V} = \frac{\Sigma M_R - \Sigma M_O + 17.76\, T_c}{W_D + \Sigma R - U_i + \frac{7.3}{2} T_c}$

$\frac{7.3}{3}(13.07 + 8.11 - 1.14 + 3.65\, T_c) = 75.31 - 96.18 + 17.76\, T_c$

$\frac{7.3}{3}(20.04 + 3.65\, T_c) - 17.76\, T_c = -20.87$

$48.76 + 8.88\, T_c - 17.76\, T_c = -20.87$

$-8.88\, T_c = -69.63$

$T_c = 7.84 \text{ ksf} \doteq (54 \text{ psi})$ controls

54 psi > 27 psi

54 psi < 160 psi = T_{cA} !!

∴ Wall will stand because concrete is in tension less than allowable, but this is not a good condition. Other factors: - possible reinforcement
- support by spillway

Form CIM-405A

APPENDIX E

REFERENCES

TILLSON LAKE DAM, NY 00083

PHASE I INSPECTION REPORT

REFERENCES

This is a general list of references pertinent to dam safety investigations. Not all references listed have necessarily been used in this specific report.

1. "Engineering and Design, National Program For Inspection of Non-Federal Dams", ER 1110-2-106, Dept. of the Army, Office of the Chief of Engineers, 26 September 1979, with Change 1 of 24 March 1980. Included as Appendix D of the ER is "Recommended Guidelines For Safety Inspection of Dams".

2. "HEC-1 Flood Hydrograph Package, Users Manual", The Hydrologic Engineering Center, U.S. Army Corps of Engineers, January 1973.

3. "Flood Hydrograph Package (HEC-1), Users Manual for Dam Safety Investigations", The Hydrologic Engineering Center, U.S. Army Corps of Engineers, September 1978.

4. HMR 33, "Seasonal Variations of Probable Maximum Precipitation, East of the 105th Meridian for Areas 10 to 1000 Square Miles and Durations from 6 to 48 Hours," U.S. Dept. of Commerce, NOAA, National Weather Service, 1956.

5. HMR 51, "All-Season Probable Maximum Precipitation, U.S. East of 105th Meridian for Areas from 1000 to 20,000 Square Miles and Durations from 6 to 72 Hours", U.S. Dept. of Commerce, NOAA, National Weather Service, 1974.

6. HYDRO-35, "Five-to-60 Minute Precipitation Frequency for the Eastern and Central United States", U.S. Dept. of Commerce, NOAA, National Weather Service, June 1977.

7. "Technical Paper No. 40, Rainfall Frequency Atlas of the United States", U.S. Dept. of Commerce, Weather Bureau, 1961.

8. Design of Small Dams, United States Dept. of the Interior, Bureau of Reclamation, Second Edition, 1973, Revised Reprint, 1977.

9. King, Horace W. and Brater, Ernest F., Handbook of Hydraulics, fifth edition, McGraw-Hill Book Co., Inc., New York, N. Y., 1963.

10. "Flood Hydrograph Analyses and Computations", EM 1110-2-1405, U.S. Army Corps of Engineers, 31 August 1959.

11. "Technical Release No. 55, Urban Hydrology for Small Watersheds", U.S. Dept. of Agriculture, Soil Conservation Service (Engineering Division), January 1975.

12. National Engineering Handbook, Section 4, Hydrology, U. S. Dept. of Agriculture, Soil Conservation Service, August 1972.

13. "Hydraulic Design of Spillways", EM 1110-2-1603, U.S. Army Corps of Engineers, 31 March 1965, with Change 1 included.

14. "Standard Project Flood Determinations", EM 1110-2-1411, U.S. Army Corps of Engineers, 26 March 1952.

15. "Hydrologic and Hydraulic Assessment", Appendix D of EC 1110-2-188, U.S. Army Corps of Engineers, 30 December 1977.

16. "Reviews of Spillway Adequacy, National Program of Inspection of Non-Federal Dams", ETL 1110-2-234, U.S. Army Corps of Engineers, 10 May 1978.

17. Hammer, Mark J., Water and Waste-Water Technology, John Wiley & Sons, Inc., New York, 1975.

18. "Hydraulic Charts For the Selection of Highway Culverts", Hydraulic Engineering Circular No. 5, U.S. Department of Commerce, Bureau of Public Roads, December 1965.

19. "Guide for Making a Condition Survey of Concrete in Service", American Concrete Institute (ACI) Journal, Proceedings Vol. 65, No. 11, November 1968, pages 905-918.

20. "Lower Hudson River Basin, Hydrologic Flood Routing Model", New York District, Corps of Engineers, January 1977.

21. "Climatological Data, Annual Summary, New York, 1979", Volume 91, No. 13, National Oceanic and Atmospheric Administration, Asheville, North Carolina.

22. "Climatological Data, New York, September 1980", Volume 92, No. 9, National Oceanic and Atmospheric Administration, Asheville, North Carolina.

23. "Water Resources Data For New York, Water Year 1979", Volume 1, USGS Water-Data Report NY-79-1, U.S. Geological Survey, Albany, New York, 1980.

24. "Maximum Known Stages and Discharges of New York Streams Through 1973", Bulletin 72, U.S. Geological Survey, 1976.

25. "Characteristics of New York Lakes (Gazetteer)", Bulletin 68, U.S. Geological Survey and NYS Department of Environmental Conservation, 1970.

26. "Gravity Dam Design", EM 1110-2-2200, U.S. Army Corps of Engineers, 25 September 1958, with Changes 1 & 2 included.

27. "Gravity Dam Design - Stability", ETL 1110-2-184, U.S. Army Corps of Engineers, 25 February 1974.

28. "Landforms and Bedrock Geology of New York State", New York State Museum and Science Service, University of the State of N.Y., State Education Dept., Albany, N.Y., reprinted 1973.

29. Geologic Map of New York, Lower Hudson Sheet, New York State Museum and Science Service, University of the State of N.Y., State Education Dept., Albany, N.Y., reprinted 1973.

30. Terzaghi, Karl and Peck, Ralph B., Soil Mechanics in Engineering Practice, second edition, John Wiley & Sons, Inc., New York, N.Y., 1967.

APPENDIX F

AVAILABLE ENGINEERING DATA AND RECORDS

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Checklist for General Engineering Data and Interview with Dam Owner	F2
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APPENDIX F

SECTION F1

LOCATION OF AVAILABLE ENGINEERING DATA AND RECORDS

1. Owner: U & U Realty, Inc.
100 Seaview Drive
Secaucus, NJ 07094
Attn: Joseph Uanue, President
201-348-4900

Available: No data.

2. Operator (Leasee): Tillson Lake Recreation Park, Inc.
Gardiner, NY 12525
Attn: Henry S. Cuney, President
914-564-2718

George Surinach, Vice-President
(Mr. Cuney's Son)
35 Utterby Rd.
Malverne, NY 11565
516-887-7859

Available: No data.

3. Designer: Unknown.

4. Construction Contractor:

Unknown, but owner was:
Hassey A. Tillson
Walden, NY (deceased)

5. Designer For 1939 Reconstruction:

Solomon LeFevre
New Paltz, NY
(business status unknown, not contacted)

6. Designer For 1956 Reconstruction:

T.W. ("Don") Westlake, P.E.
Holmes Rd.
RD 1, Box 66
Newburgh, NY
(business status unknown, not contacted)

7. Construction Contractors For Reconstructions: Unknown

8. Agency: NYS Department of Environmental Conservation
50 Wolf Rd.
Albany, NY 12233
Attn: George Koch, P.E., Chief, Dam Safety Section
518-457-5557

Available: Drawings, letters and report describing failures and reconstructions, applications for reconstruction, and inspection report.

4585

PHASE I INSPECTION

CHECKLIST FOR GENERAL ENGINEERING DATA
& INTERVIEW WITH DAM OWNER

Name of Dam Tillson Lake Dam Fed. Id.# NY 00083

Date April 8, 1981 Interviewer(s) Thomas P. Bennedum

Dam Owner/Representative(s) Interviewed, Title & Phone#

George Surinach, V-P of Tillson Lake Recreation Park
35 Utter by Rd.
Bus. 516-489-0505, Home 516-887-7859 Malverne, NY 11565
(Son of Operator)

1. OWNERSHIP (name, title, address & phone #)
U&U Realty, Inc., 100 Seaview Dr., Secaucus, NJ 07094
Attn: Joseph Uanue, President 201-348-4900

2. OPERATOR (name, title, address & phone # of person responsible for day-to-day operation) Henry S. Cuney, President, Tillson Lake Recreation Park, Inc. (lessee), Gardiner NY 12525 914-564-2718 (met @ beg'n of site inspection)
Also George Surinach, see above.
a. Operator Full/Part time Part time

3. PURPOSE OF DAM
a. Past Recreation. Original owner Hassey A. Tillson, then Dominick Porco
b. Present Recreation. U&U bought pavilion & bench in 1974±. In 1980 bought dam & remainder of land around lake.

4. DESIGN DATA
a. Designed When unknown, suspect 1920's-1930's
b. By (name, address, phone #, business status)
unknown
c. Geology Reports None known
d. Subsurface Investigations None known
e. Design Reports/Computations (H&H, stability, seepage)
None known

f. Design Drawings (plans, sections, details) None known

g. Design Specifications None known

h. Other 1939 & 1956 reconstruction drawings (see Appendix G-1 & G-2) show some details of original design.

5. CONSTRUCTION HISTORY

a. Initial Construction

1) Completed When unknown, suspect 1920's - 1930's

2) By (name, address, phone #, business status) Actual contractor unknown. Original owner: Hassey A. Tillson, Walden, NY (deceased)

3) Borrow Sources/Material Tests None known

4) Construction Reports/Photos No photos known. See Appendix F3-1 for short description of construction by a victim of the first failure of the dam.

5) Diversion Scheme/Construction Sequence None known

6) Construction Problems None known

7) As-Built Drawings (plans, sections, details) None known

8) Data on Electrical & Mechanical Equipment Affecting Safe Operation of Dam No electrical at the dam. No data on the gate mechanism.

9) Other n/a

4576

b. Modifications (review design data & initial construction items as applicable & describe) • 1939 raised conc. core wall to rt. of spillway 2.0'± (records say 2.5') & raised top of embankment to about same elevation. Put 2.5' high flashboards on spillway, leaving 3.5' of freeboard to top of core wall on rt, but only 1.5' on left due to left core wall not being raised as intended. (See 9-OTHER)

c. Repairs & Maintenance (review design data & initial construction items as applicable & describe) • 1939 repaired breach (about 30' deep x 90' long on top) in core wall just to rt. of spillway & replaced washed out embankment. Engineer was LeFevre as noted in 5b-Modifications. See Appendix F3-1 thru F3-8 & App. G-1. • 1956 replaced about 30' of d/s end of Rt. training wall & conc. bottom of spill. disch. channel (See 9-OTHER)

6. OPERATION RECORD

a. Past Inspections (dates, by, authority, results) Only record - April 03, 1973 by NYS-DEC, see Appendix F3-18.

b. Performance Observations (seepage, erosion, settlement, post-construction surveys, instrumentation & monitoring records) No instrumentation, monitoring records, or other items recorded.

c. Post-Construction Engineering Studies/Reports None known except those in conjunction with 1939 & 1956 repairs, see Appendix F3 & Appendix G.

d. Routine Rainfall, Reservoir Levels & Discharges None known.

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e. Past Floods That Threatened Safety (when, cause, discharge, max. pool elevation, any damage)
• Sept. 21, 1938 flood, dam o/T, embankment washed out & dam breached see Appendix F3-1.
• August 1955 flood, low core wall left of spillway o/T by "several inches", damage to spillway disch. ch. training walls, part of d/s toe washed out, see Appendix F3-9.

f. Previous Failures (when, cause, describe)
• Sept 21, 1938, dam o/T & breached & August 1955 partially o/T & wash out damage (see 6e - Past Floods).

g. Earthquake History (seismic activity in vicinity of dam)
None known

7. VALIDITY OF DESIGN, CONSTRUCTION & OPERATION RECORDS (note any apparent inconsistencies)
• Flashboards now 3' high & don't resemble 1956 design (see App. G-3)
• Outlet conduit reported 3' square (Appendices F3-1 & G-1) but measures about 3.8' square on d/s end. (see 9-OTHER)

8. OPERATION & MAINTENANCE PROCEDURES

a. Operation Procedures in writing? No Obtain copy or describe. (reservoir regulation plan, normal pool elevation and status of operating facilities, who operates & means of communication to controller, mode of operating facilities, i.e., manual, automatic, remote) • 3' flashboards up May 1 - end Sept. Normally removed by Operator's son Oct. - April, but only 1 of 4 - 13'-9" sections removed this past winter.
• Outlet conduit sluice gate (30" ϕ) normally shut. (see 9-OTHER)

b. Maintenance Procedures in writing? No Obtain copy or describe. • Flashboards are repaired/replaced as req'd.
• 1979 ± diver casually looked at outlet sluice gate while looking for lost watch. His verbal report was that there was a build-up of debris in front & it looked corroded.

c. Emergency Action Plan & Warning System in Writing? No
Obtain copy or describe. (actions to be taken to minimize the D/S effects of an emergency)

- Some thought given, but no plan in place.
- Would assume that Operator would contact Ulster Co. Sheriff's Dept. thru section sgt. who rents house on lake from U&U Realty.

9. OTHER

56) Modifications • (1939 cont'd) Breach in core wall was repaired (see 5c-Repairs). Engineer was Solomon LeFevre, New Paltz, NY (business status unknown, not contacted). See Appendix F3-1 thru F3-8 & Appendix G-1. Contractor unknown.

• 1956 raised conc. core wall to left of spillway 2.0' to match rest of dam. Raised left training wall of spillway disch. ch. about 1.5' & extended end of wall d/s. Some concrete work to promote better flow may have been done in trough along Rt. side of spillway disch. channel. The d/s part of the right training wall of the spillway disch. ch. was replaced, as was the d/s end of the conc. bottom of the spillway disch. ch. Engineer was T.W. ("Don") Westlake, P.E., Holmes Rd., RD1, Box 66, Newburgh, NY (business status unknown, not contacted). See Appendix F3-9 thru F3-17 & Appendix G-2. The flashboard work proposed on Appendix G-3 appears not to have been done. Contractor unknown.

5c) Repairs • (1956 cont'd) ... which had been undermined & washed out. Replaced part of d/s toe of embankment which had been washed out. Engineer was Westlake as noted in 56-Modifications. See Appendix F3-9 thru F3-17 & Appendix G-2.

7) Validity • Dike shown on 1956 reconstruction dwg. (Appendix G-2) to the left of the spillway is not apparent in field.

8a) Operation Is inoperable & can only be reached by boat. Handwheel is missing. Last operated 15-20 yrs ago.

- Lake used to be drained for cleaning every 10 yrs. Last done in 60's per memory.
- Operator visits dam at least twice/week during summer & randomly at other times.

7) Validity • 1939 & 1956 reconstruction dwgs (see Appendix G) show that core wall was to be raised 2.5', for a total of 6.5' over spillway crest. Measurement shows only about 2.0' added, total 6.0' over spillway crest.

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APPENDIX F

SECTION F3

COPIES OF ENGINEERING DATA AND RECORDS

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Application for Reconstruction of the Dam, by Dominick Porco - April 30, 1956	F3-11
Calculations on Spillway Capacity & Flashboards, suspect by Westlake or State Reviewers in 1956	F3-15
Inspection Report, by NYS-DEC - April 23, 1973	F3-18

C O P Y

Wallkill, N.Y.
Sept 27, 1938

OCT 1 1938

The State Conservation Dept.
Albany, N.Y.

Gentlemen:

On the night of Sept. 21st at about 8:30 o'clock, our farm was subjected to a violent flood. For some days previous there had been intermittent rain. The brook,running some 50 to 100 ft. below the house,had risen somewhat, but not in any sense to dangerous proportions. On a number of previous occasions we saw it considerably higher and after receding left us entirely no damage. On the night in question, however, a private dam (Tillson Lake) about three quarters of a mile upstream burst. This dam held back thirty five acres of water, thirty five feet deep in the channel. While this dam had an ample spillway to take care of high waters, it was the constant practice of the owners to keep this spillway planked up. Only when there was already threatening high water was any attempt made to remove the planks or to open the 3' square drain, a difficult job in both cases even in normal weather to say nothing of its being an impossible job during a storm. To open the drain it was necessary to row out to a concrete tower. To remove the planks from the spillway was at all times dangerous, for it required working at a 35' height against water pressure. There were no mechanical means to remove these planks which quite completely blocked the entire spillway. Such were the conditions on the night of Wednesday, Sept.21st. It was a physical impossibility to do either of the two jobs. The high water then proceeded, aided somewhat by high velocity wind, to spill with considerable volume and force over the dam proper.

At this point I wish to describe briefly the construction of the dam. At the right (facing upstream from below the dam) there was a spillway shaped somewhat like a half funnel except that the sides were more square than round. The left hand side of this spillway had a concrete wall at right angles to the dam proper and running diagonally from the bed stream to the top of the dam. The lead off of the spillway was concrete. A few feet to the left of this wall and running at the very bottom of the original channel was the three foot square concrete drain running from a point some distance in the lake directly under the concrete tower to a point on the other side of the dam a few feet beyond the edge of the dirt fill. The rest of the dam other than the spillway, was a concrete wall 28" wide at the base, 12" wide at the top and approximately 35' high. Through this concrete there were some steel reinforcing bars of about 3/4" diameter. These bars appeared to be spaced about four or five feet apart. On both sides of this concrete wall there was a rock and dirt fill. Since the spillway was planked shut, the rising water spilled directly over the dam proper and kept washing away quite rapidly the down stream part of the fill, leaving the concrete wall entirely unsupported since it stood there without any other permanent bracing. The answer - it burst from top to bottom about 100 feet wide.

DEC

A number of people caught in the rush of water very narrowly escaped with their lives; bridges were promptly washed away; houses flooded; many, many trees uprooted, and general damage that goes with a flood. My farm is approximately three quarters of a mile below this dam, and while others were closer to it than we were our farm was the hardest hit because the water upon coming to our land concentrates between two high banks giving it great volume and force. Hundreds of tons of gravel, rocks and boulders, some of the latter weighing from five to ten ton, were deposited on our lawn but a few feet from the house. The foundation of the corner of the house was undermined, cellar flooded and contents destroyed; one hundred pullets were drowned and swept away, likewise a stack of straw all the winter bedding for the cattle; a tractor was carried 60 ft. and turned over considerably damaged; farm machinery, wagons, etc. swept away and recovered in a damaged condition. Many of our prize shade trees are gone. The well was flooded leaving us still without drinking water. The electric meter and switches in the cellar are so water soaked we are still without light or power. Two bridges leading from the farm to the outside road, our only means of travel, were washed away. Our two gardens have been made completely useless being strewn with boulders, gouged with numerous holes and the top soil gone. And worse yet, the course of the brook has been so altered that the next high water can do us several times the damage suffered this time.

It should be unnecessary to mention in detail all the havoc that a flood can cause. Our harvesting of corn and other important fall work has been unceremoniously interrupted by the emergency of this flood. While we realize the problem created by floods, all people hereabouts are of one opinion, to wit, that this one was caused unnecessarily. This dam probably was originally passed by State inspectors; the spillway, even in my own opinion, was constructed large enough to take care of a swollen stream. But why, in the name of common sense, a spillway is so constructed and then its purpose nullified by being planked up, (something that should never have been permitted at any time regardless of weather conditions) I don't know.

It seems quite certain that reconstruction of this Tillson Lake dam will start within a few days, so herein is the purpose of this letter. If the owners of this dam are permitted to do a job no better than the last in the way of construction and reinforcement; if they are permitted to plank up the spillway and thereby again endanger lives and cause thousands of dollars worth of damage, I for one along with my neighbors, most strenuously object. It seems to me in view of the seriousness of the damage just done and the possibility of its repetition, that somewhat more stringent regulations be enforced. It is not pleasant to anticipate these calamities with every rainy spell and to feel in constant uncertainty of the security of one's family and property, to say nothing of the inability to sustain such financial losses. It is difficult for a farmer to sustain himself under normal conditions.

I should like to have some assurance from you or whatever authorities have jurisdiction in these matters, that this will be given due consideration and action.

Yours truly,
(Signed) Fred Briehl

DEC

Form E-61
1-16-37-500 (11E-5273)

STATE OF NEW YORK

Superseded by 194-2420

DEPARTMENT OF PUBLIC WORKS

DIVISION OF ENGINEERING

ALBANY

Received May 15, 1939 Dam No. 194-842

Disposition May 19, 1939 Watershed Lower Hudson

Foundation inspected

Structure inspected

Application for the Construction or Reconstruction of a Dam

Application is hereby made to the Superintendent of Public Works, Albany, N. Y., in compliance with the provisions of Section 948 of the Conservation Law (see last page of this application) for the approval of specifications and detailed drawings, marked Reconstruction "Tillson Lake" Dam, Near Rutsonville, N. Y. herewith submitted for the { ~~Construction~~ / reconstruction } of a dam herein described. All provisions of law will be complied with in the erection of the proposed dam. It is intended to complete the work covered by the application about June 1, 1939 (Date)

1. The dam will be on Brook flowing into Shawangunk Kill in the town of Gardiner, County of Ulster and 5.5 miles north west of Wallkill N. Y. (give exact distance and direction from a well-known bridge, dam, village main cross-roads or mouth of a stream)
2. Location of dam is shown on the Newburgh and Ellenville quadrangle of the United States Geological Survey.
3. The name of the owner is W. A. Tillson,
4. The address of the owner is Walden, N. Y.
5. The dam will be used for Lake for Summer Club and bungalows
6. Will any part of the dam be built upon or its pond flood any State lands? no
7. The watershed above the proposed dam is 3.5 (including Palmaghatt) square miles.
8. The proposed dam will create a pond area at the spillcrest elevation of 25. acres and will impound 10,000,000 cubic feet of water.

DEC

9. The maximum height of the proposed dam above the bed of the stream is 33 feet 6 inches.

10. The lowest part of the natural shore of the pond is 15 feet vertically above the spillcrest, and everywhere else the shore will be at least 50 to 1200 feet above the spillcrest.

11. State if any damage to life or to any buildings, roads or other property could be caused by any possible failure of the proposed dam 1.5 miles of farm land to the Shawangunk Kill. stream crosses an improved County Road near the Kill

12. The natural material of the bed on which the proposed dam will rest is (clay, sand, gravel, boulders, granite, shale, slate, limestone, etc.) Hudson River Slate

13. Facing down stream, what is the nature of material composing the right bank? Hudson River Slate

14. Facing down stream, what is the nature of the material composing the left bank? Hudson River Slate

15. State the character of the bed and the banks in respect to the hardness, perviousness, water bearing, effect of exposure to air and to water, uniformity, etc Slate dips 50 degrees to west and t the harder strata resist wear, and form riffles, which have withstood the flow of water for ages along bed of stream for several hundred ft

16. Are there any porous seams or fissures beneath the foundation of the proposed dam? no

17. WASTES. The spillway of the above proposed dam will be 55 feet long in the clear; the waters will be held at the right end by a concrete wall the top of which will be 6.5 feet above the spillcrest, and have a top width of 3.5 feet; and at the left end by a concrete wall the top of which will be 6.5 feet above the spillcrest, and have a top width of 3.5 feet.

18. The spillway is designed to safely discharge 1250 cubic feet per second.

19. Pipes, sluice gates, etc., for flood discharge will be provided through the dam as follows:

1 - 30" x 30" sluice gate

20. What is the maximum height of flash boards which will be used on this dam? 2.5 ft.

21. APRON. Below the proposed dam there will be an apron built of concrete 55. feet long across the stream, 5.5 feet wide and 2 to 5 feet thick.

22. Does this dam constitute any part of a public water supply? no

DEC

INSTRUCTIONS

Read carefully on the last page of this application the law setting forth the requirements to be complied with in order to construct or reconstruct a dam.

Each application for the construction or reconstruction of a dam must be made on this standard form, copies of which will be furnished upon request to the Chief Engineer, Division of Engineering, Department of Public Works, Albany, N. Y. The application must be accompanied by three sets of plans, and specifications. The information furnished must be in sufficient detail in order that the stability and safety of the dam can be determined. In cases of large and important dams assumptions made in calculating stresses and stability should be given.

Samples of materials to be used in the dam and of the material on which the dam is to be founded may be asked for, but need not be furnished unless requested.

If the dam constitutes a part of a public water supply, application should be made to the Water Power and Control Commission under Article XI of the Conservation Law.

An application for the construction or reconstruction of a dam must be signed by the prospective owner of the dam or his duly authorized agent. The address of the signer and the date must be given as provided for on the last page of the application form.

DEC

SECTION 948 OF THE CONSERVATION LAW

§ 948. **Structures for impounding water; inspection of docks; penalties.** No structure for impounding water and no dock, pier, wharf or other structure used as a landing place on waters shall be erected or reconstructed by any public authority or by any private person or corporation without notice to the superintendent of public works, nor shall any such structure be erected, reconstructed or maintained without complying with such conditions as the superintendent of public works may by order prescribe for safeguarding life or property against danger therefrom. No order made by the superintendent of public works shall be deemed to authorize any invasion of any property rights, public or private, by any person in carrying out the requirements of such order. The superintendent of public works shall have power, whenever in his judgment public safety shall so require, to make and serve an order directing any person, corporation, officer or board, constructing, maintaining or using any structure hereinbefore referred to, remove, repair or reconstruct the same within such reasonable time and in such manner as shall be specified in such order, and it shall be the duty of every such person, corporation, officer or board, to obey, observe and comply with such order and with the conditions prescribed by the superintendent of public works for safeguarding life or property against danger therefrom, and every person, corporation, officer or board failing, omitting or neglecting so to do, or who hereafter erects or reconstructs any such structure hereinbefore referred to without submitting to the superintendent of public works and obtaining his approval of plans and specifications for such structures when required so to do by his order or who hereafter fails to remove, erect or to reconstruct the same in accordance with the plans and specifications so approved shall forfeit to the people of this state a sum not to exceed five hundred dollars to be fixed by the court for each and every offense; every violation of any such order shall be a separate and distinct offense, and, in case of a continuing violation, every day's continuance thereof shall be and be deemed to be a separate and distinct offense. This section shall not apply to a dam where the area draining into the pond formed thereby does not exceed one square mile, unless the dam is more than ten feet in height above the natural bed of the stream at any point or unless the quantity of water which the dam impounds exceeds one million gallons; nor to a dock, pier, wharf or other structure under the jurisdiction of the department of docks, if any, in a city of over one hundred and seventy-five thousand population. This section as hereby amended shall not impair the effect of an order heretofore made by the conservation commission or commissioner under this section prior to the taking effect of chapter four hundred and ninety-nine of the laws of nineteen hundred and twenty-one, nor require the approval by the superintendent of public works of plans and specifications heretofore approved by such commission or commissioner under this section.

The foregoing information and accompanying plans and specifications are correct to the best of my knowledge and belief.

H A Tiller, Owner

By _____, authorized agent of owner.

Address of signer Walden Wis Date May 12, 1939

DEC

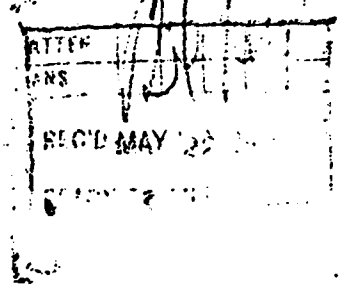
STATE OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
ARTHUR W. BRANDT
SUPERINTENDENT

THOMAS F. FARRELL
CHIEF ENGINEER

DIVISION OF ENGINEERING

ALBANY, N. Y., May 20, 1939

H. A. Tillson Dam
L.H.W. 194-84
Town of Gardiner
Ulster Co.



Mr J. S. Bixby
District Engineer
Poughkeepsie, N.Y.

Dear Sir:-

There is being sent to you enclosed herewith approved plans for the reconstruction of a dam owned by H. A. Tillson, Walden, N.Y. The dam is located in the Town of Gardner, Ulster County, 5.5 miles northwest of Wallkill, N.Y. This dam failed sometime ago due to the fact that the spillway was obstructed by flash boards during flood.

Very truly yours,

T. F. FARRELL
Chief Engineer

JPN/CG

enc

DEC

May 20, 1939

Mr S. LeFevre,
Forest Glen,
New Paltz, N.Y.

Dear Sir:-

An application and plans filed by you for the reconstruction of a dam in the Town of Gardner, Ulster County, 5.5 miles northwest of Wallkill, N.Y., for Mr H. A. Tillson, are hereby approved to the extent of our authority under the provisions of section 948 of the Conservation Law.

This dam is designated by us as 194-842 Lower Hudson Watershed.

One set of plans for this dam, stamped with the approval of this department, is being sent to you enclosed herewith.

Very truly yours

T. F. FARRELL
Chief Engineer

JPN/CC

enc.

DEC

T. W. ("DON") WESTLAKE, P. E.
HOLMES ROAD
R. D. 1, BOX 66
PROFESSIONAL ENGINEER
LAND SURVEYOR
NEWBURGH, N. Y.
TEL. 318W2

4-30-56

REPORT ON TILLSON LAKE DAM
Rutsonville, N. Y.
Owner - Dominick Porco

Results To Dam From Rain and Flood of August 1955

North Core Wall

A portion of this wall had not been built higher during the reconstruction of 1939, and the top of the wall is still at elevation 284. The lake level topped this by several inches, and the overflowing water scoured the earth somewhat. The sluice gate has not been operative for years and could not be opened to help lower the water level.

Spillway Side Walls

At the lower end of the spillway the walls were of insufficient height to carry the flow. The overflowing on the northeast side resulted in scouring the earth to slate rock close below the surface. But on the southwest side, a portion of the toe of the earth dam was washed out, and a section of spillway wall and bottom were undermined and washed out.

Depth of Water Above Flashboards - August 1955 Flood

Judging by the amount of scouring at the north core wall, the wall was topped by several inches; and the depth of water above flashboards would thusly have been about 2 feet.

Note - With the raising of the north core wall, the top of wall will then be 4 feet above top of flashboards.

DEC

T. W. ("DON") WESTLAKE, P. E.
HOLMES ROAD
R. D. 1. BOX 66
PROFESSIONAL ENGINEER
LAND SURVEYOR
NEWBURGH, N. Y.
TEL. 318W2

Proposed Reconstruction - 1956

Plan

The proposed reconstruction is shown on Dwg. 1, Job No. 56-12 of T. W. Westlake, P. E.

Specifications

Specifications for the work are outlined on the drawing.

Outline of Work and Completion Dates

Work	Completion Date
North Core wall and Dike	5-30-56
Spillway Sidewall and Bottom Rebuilding	7-30-56
Filling In Washed Out Areas	7-30-56
Rip Rap, Or Concrete Walls To Raise Pool Level	8-30-56
Repair Sluice Gate	10-30-56

(All work to be completed by 10-30-56)

DEC

Form E-61. 8-14-52-2M (2D-282)
Ord. 7-05-58

STATE OF NEW YORK

DEPARTMENT OF PUBLIC WORKS

See [illegible] 44-642

ALBANY

Received 5/4/56 G. Dam No. 194-2420

Disposition 5/25/56 Watershed Lower Hudson

Foundation inspected .....

Structure inspected .....

Application for the Construction or Reconstruction of a Dam

Application is hereby made to the Superintendent of Public Works, Albany, N. Y., in compliance with the provisions of Section 948 of the Conservation Law (see third page of this application) for the approval of specifications and detailed drawings, marked 1956 Reconstruction of "Tillson Lake" Dam near Rutsonville, N. Y.

herewith submitted for the {construction / reconstruction} of a dam herein described. All provisions of law will be complied with in the erection of the proposed dam. It is intended to complete the work covered by the application about Nov. 1, 1956 (Date)

1. The dam will be on Brook flowing into Shawangunk Kill in the town of Gardiner County of Ulster and 5.5 miles northwest of Wallkill, N. Y.
(Give exact distance and direction from a well-known bridge, dam, village, main cross-roads or mouth of a stream)

2. Location of dam is shown on the Newburgh and Ellenville quadrangle of the United States Geological Survey.

3. The name of the owner is Dominick Porco

4. The address of the owner is R. D. 1, Route 9W, Newburgh, N. Y.

5. The dam will be used for Lake for Summer Club and Bungalows

6. Will any part of the dam be built upon or its pond flood any State lands? no

7. The watershed above the proposed dam is 3.5 (including Palmaghatt) square miles.

8. The proposed dam will create a pond area at the spillcrest elevation of 25 acres and will impound 10,000,000 cubic feet of water.

DEC

9. The maximum height of the proposed dam above the bed of the stream is 33 feet 6 inches.

10. The lowest part of the natural shore of the pond is 15 feet vertically above the spillcrest, and everywhere else the shore will be at least 50 to 1200 feet above the spillcrest.

11. State if any damage to life or to any buildings, roads or other property could be caused by any possible failure of the proposed dam 1.5 miles of farm land to the Shawangunk Kill; some individual homes; stream crosses under an improved Cty. Road near the Kill

12. The natural material of the bed on which the proposed dam will rest is (clay, sand, gravel, boulders, granite, shale, slate, limestone, etc.) Hudson River Slate

13. Facing downstream, what is the nature of material composing the right bank? Hudson River Slate

14. Facing downstream, what is the nature of the material composing the left bank? Hudson River Slate

15. State the character of the bed and the banks in respect to the hardness, perviousness, water bearing, effect of exposure to air and to water, uniformity, etc. Slate dips 50 degrees to the west, and the harder strata resist wear for several hundred feet downstream from the spillway.

16. Are there any porous seams or fissures beneath the foundation of the proposed dam? no; there are no signs of boils on the downstream face.

17. WASTES. The spillway of the above proposed dam will be 55 feet long in the clear; the waters will be held at the right end by a concrete wall the top of which will be 6.5 feet above the spillcrest, and have a top width of 3.5 feet; and at the left end by a concrete wall the top of which will be 6.5 feet above the spillcrest, and have a top width of 3.5 feet.

18. The spillway is designed to safely discharge 1250 cubic feet per second. (based on flow depth of 3.5')

19. Pipes, sluice gates, etc., for flood discharge will be provided through the dam as follows:
one 30" x 30" sluice gate

20. What is the maximum height of flash boards which will be used on this dam? 2.5 feet

21. APRON. Below the proposed dam there will be an apron built of concrete 55 feet long across the stream, 5.5 feet wide and 2 to 5 feet thick.

22. Does this dam constitute any part of a public water supply? no

DEC

INSTRUCTIONS

Read carefully on the third page of this application the law setting forth the requirements to be complied with in order to construct or reconstruct a dam.

Each application for the construction or reconstruction of a dam must be made on this standard form, copies of which will be furnished upon request to the Department of Public Works, Albany, N. Y. The application must be accompanied by three sets of plans, and specifications. The information furnished must be in sufficient detail in order that the stability and safety of the dam can be determined. In cases of large and important dams assumptions made in calculating stresses and stability should be given.

Samples of materials to be used in the dam and of the material on which the dam is to be founded may be asked for, but need not be furnished unless requested.

If the dam constitutes a part of a public water supply, application should be made to the Water Power and Control Commission under Article XI of the Conservation Law.

An application for the construction or reconseruction of a dam must be signed by the prospective owner of the dam or his duly authorized agent. The address of the signer and the date must be given as provided for on the last page of the application form.

SECTION 948 OF THE CONSERVATION LAW

§ 948. Structures for impounding water; inspection of docks; penalties. No structure for impounding water and no dock, pier, wharf or other structure used as a landing place on waters shall be erected or reconstructed by any public authority or by any private person or corporation without notice to the superintendent of public works, nor shall any such structure be erected, reconstructed or maintained without complying with such conditions as the superintendent of public works may by order prescribe for safeguarding life or property against danger therefrom. No order made by the superintendent of public works shall be deemed to authorize any invasion of any property rights, public or private, by any person in carrying out the requirements of such order. The superintendent of public works shall have power, whenever in his judgment public safety shall so require, to make and serve an order, setting forth therein his findings of fact and his conclusions therefrom, directing any person, corporation, officer or board, constructing, maintaining or using any structure hereinbefore referred to, either remove the said structure or to repair or reconstruct the same within such reasonable time and in such manner as shall be specified in such order, and it shall be the duty of every such person, corporation, officer or board, to obey, observe and comply with such order and with the conditions prescribed by the superintendent of public works for safeguarding life or property against danger therefrom, and every person, corporation, officer or board failing, omitting or neglecting so to do, or who hereafter erects or reconstructs any such structure hereinbefore referred to without submitting to the superintendent of public works and obtaining his approval of plans and specifications for such structures when required to do so by his order or hereafter fails to remove, erect or to reconstruct the same in accordance with the plans and specifications so approved shall forfeit to the people of the State a sum not to exceed five hundred dollars to be fixed by the court for each and every offense; every violation of any such order shall be a separate and distinct offense, and, in such case of a continuing violation, every day's continuance thereof shall be and be deemed to be a separate and distinct offense. Such order shall not contain any provision to compel the owner to make repairs or proceed with reconstruction as specified in this section by any type of construction other than that of the dam itself. In addition to said forfeiture upon the violation of any such order, the superintendent of public works shall have power to enter upon the lands and waters where such structures are located, for the purpose of removing, repairing or reconstructing the same, and to take such other and further precautions which he may deem necessary to safeguard life or property against danger therefrom. In removing, repairing and reconstructing such dam the superintendent shall not deviate from the method, manner or specifications contained in the original order. The superintendent of public works shall certify the amount of the costs and expenses incurred by him for the removal, repair or reconstruction aforesaid, or in anywise connected therewith, to the board of supervisors of the county or counties in which the said lands and waters are located, whereupon it shall be the duty of such board of supervisors to add the amount so certified to the assessment rolls of such locality or localities as a charge against the real property upon which the dam is located designated or described by the superintendent of public works as chargeable therewith, and to issue its warrant or warrants for the collection thereof. Thereupon it shall become the duty of such locality or localities through their proper officers to collect the amounts so certified in the same manner as other taxes are collected in such locality or localities, and when collected, to pay the same to the superintendent of public works

who shall thereupon pay the same into the treasury. Any amount so levied shall thereupon become a lien upon the real property affected thereby, to the same extent as any tax levy becomes and is a lien thereon.

Any person in interest may, within thirty days from the service of any such order, appeal to the supreme court to determine the reasonableness of such order. At any time during such appeal to the supreme court upon at least three days' notice, the party appealing may apply for an order directing any question of fact to be tried and determined by a jury, and the court shall thereupon cause such question to be stated for trial accordingly and the findings of the jury upon such question shall be conclusive. Appeals may be taken from the supreme court to the appellate division of the supreme court and to the court of appeals in such cases, subject to the limitations provided in the civil practice act.

This section shall not apply to a dam where the area draining into the pond formed thereby does not exceed one square mile, unless the dam is more than ten feet in height above the natural bed of the stream at any point or unless the quantity of water which the dam impounds exceeds one million gallons; nor to a dock, pier, wharf or other structure under the jurisdiction of the department of docks, if any, in a city of over one hundred and seventy-five thousand population. This section as hereby amended shall not impair the effect of an order heretofore made by the conservation commission or commissioner under this section prior to the taking effect of chapter four hundred and ninety-nine of the laws of nineteen hundred and twenty-one, nor require the approval by the superintendent of public works, of plans and specifications theretofore approved by such commission or commissioner under this section.

The foregoing information is correct to the best of my knowledge and belief, and the construction will be carried out in accordance with the approved plans and specifications.

Dominick Parisi ....., Owner

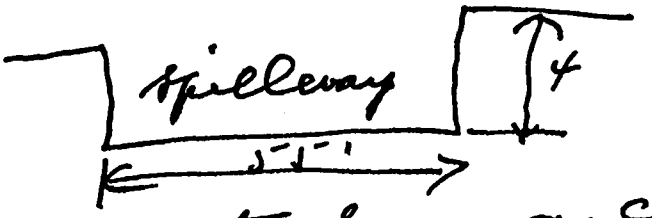
By ....., authorized agent of owner.

Address of signer R. D. 1, Route 9W, Newburgh, N. Y. Date 4-30-56

DEC

— Comments

Dam was originally built without flashboards



and was supposed to have a capacity of 1210 c.f.s.

$Q = 3.1 \times 5.5 \times \sqrt{\frac{6.53}{3.5}} = 1210$ c.f.s.

$H = 3.5'$

This allowed only 0.5' free-board which is not enough & dam was washed out. They water level was raised 2½ feet by putting on 2+1' flashboards & raising the elevation of the dam. This left the spillway opening 5'-1" x 4' same as original assuming that the flash boards were strong enough not to go out. However they are not strong enough. Probably the best way to handle it is to have the flashboards go out when the water level gets to not more than 1' above flashboards, providing this does not flood other peoples lands.

Sluice gate should be made to work. other repairs as noted on Engr. Report.

Subject to flash floods as ratio of $\frac{21}{640} = .04$ is very small.

DEC

Flashboards

Flashboards

Spillway Designed for 1250 c.f.s. according to appl.

$Q = 3.5 \times 55 \times \sqrt{3.5}^{6.13} = 1250$ c.f.s.

Actual Run-Off

$Q = CIA$ $\quad A = 3.5 \times 640 = 2240$

Woods 6% grade White County

$V = 2$

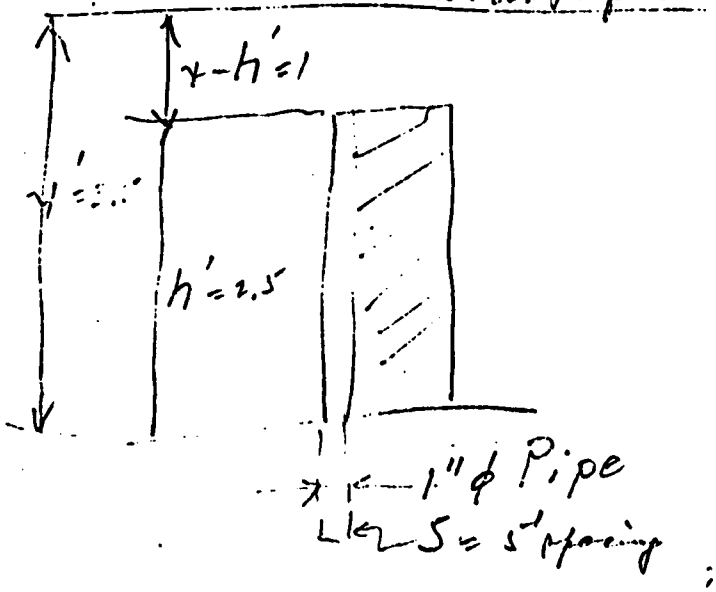
$\frac{21300}{2} = \frac{1260}{60} = 210$ Min. $= T = 3.5$ hr.

$I = \frac{4.06}{3.5} = 1.16$" per hr.

$Q = .35 \times 1.16 \times 2240 = 910$ c.f.s. (1250 c.f.s. is OK)

Flashboards should go out when water is 1' above top of them.

DEC



$$12\left[\left(\frac{62.5 \times 5 \times 2.5 \times 2.5 \times 2.5}{3}\right) + 62.5 \times 5\left(3.5-2.5\right)\frac{2.5 \times 2.1}{2}\right] = \frac{.087}{.6575} = .132$$

$$\text{stress} = \frac{21400}{.132} = 161500 \text{ psi}$$

DEC DAM INSPECTION REPORT

RB	CTY	YR. AP.	DAM NO.	INS. DATE	USE	TYPE
01	56	39	2420 (194)	04 23 73	002	2

AS BUILT INSPECTION

- [1] Location of Spillway and outlet
- [1] Size of Spillway and outlet
- [1] Elevations
- [1] Geometry of Non-overflow section

GENERAL CONDITION OF NON-OVERFLOW SECTION [1]

[1] Settlement	[2] Cracks	[1] Deflections
[1] Joints	[2] Surface of Concrete	[1] Leakage
[1] Undermining	[1] Settlement of Embankment	[1] Crest of Dam
[2] Downstream Slope Trees	[1] Upstream Slope	[1] Toe of Slope

GENERAL CONDITION OF SPILLWAY AND OUTLET WORKS [1]

[C] Auxiliary Spillway	[1] Service or Concrete Spillway	[1] Stilling Basin
[2] Joints	[2] Surface of Concrete	[1] Spillway Toe
[1] Mechanical Equipment	[1] Plunge Pool	[1] Drain

[1] Maintenance	[B] Hazard Class
[3] Evaluation	[3] Inspector

COMMENTS:

Dam in good condition

Trees growing on downstream slope

DEC

DEC DAM INSPECTION REPORT CODING

1. River Basin - Nos. 1-23 on Compilation Sheets
2. County - Nos. 1-62 Alphabetically
3. Year Approved -
4. Inspection Date - Month, Day, Year
5. Apparent use -
   1. Fish & Wildlife Management
   2. Recreation
   3. Water Supply
   4. Power
   5. Farm
   6. No Apparent Use
6. Type -
   1. Earth with Aux. Service Spillway
   2. Earth with Single Conc. Spillway
   3. Earth with Single non-conc. Spillway
   4. Concrete
   5. Other
7. As-Built Inspection - Built substantially according to approved plans and specifications

Location of Spillway and Outlet Works

1. Appears to meet originally approved plans and specifications.
2. Not built according to plans and specifications and location appears to be detrimental to structure.
3. Not built according to plans and specifications but location does not appear to be detrimental to structure.

Elevations

1. Generally in accordance to approved plans and specifications as determined from visual inspection and use of hand level.
2. Not built according to plans and specifications and elevation changes appear to be detrimental to structure.
3. Not built according to plans and specifications but elevation changes do not appear to be detrimental to structure.

Size of Spillway and Outlet Works

1. Appears to meet originally approved plans and specifications as determined by field measurements using tape measure.
2. Not built according to plans and specifications and changes appear detrimental to structure.
3. Not built according to plans and specifications but changes do not appear detrimental to structure.

Geometry of Non-overflow Structures

1. Generally in accordance to originally approved plans and specifications as determined from visual inspection and use of hand level and tape measure.
2. Not built according to plans and specifications and changes appear detrimental to structure.
3. Not built according to plans and specifications but changes do not appear detrimental to structure.

General Conditions of Non-Overflow Section

1. Adequate - No apparent repairs needed or minor repairs which can be covered by periodic maintenance.
2. Inadequate - Items in need of major repair.

(Items) For boxes listed on condition under non-overflow section.
1. Satisfactory.
2. Can be covered by periodic maintenance.
3. Unsatisfactory - Above and beyond normal maintenance.

DEC

DEC DAM INSPECTION REPORT CODING (cont.)

General Condition of Spillway and Outlet Works

1. Adequate - No apparent repairs needed or minor repairs which can be covered by periodic maintenance.
2. Inadequate - Items in need of major repair.

(items) For boxes listed conditions listed under spillway and outlet works.

1. Satisfactory.
2. Can be covered by periodic maintenance.
3. Unsatisfactory - Above and beyond normal maintenance.
4. Dam does not contain this feature.

Maintenance

1. Evidence of periodic maintenance being performed.
2. No evidence of periodic maintenance.
3. No longer a dam or dam no longer in use.

.S.)

Hazard Classification Downstream

1. (A) Damage to agriculture and county roads.
2. (B) Damage to private and/or public property.
3. (C) Loss of life and/or property.

Evaluation - Based on Judgment and Classification in Box Nos.

Evaluation for Unsafe Dam

1. Unsafe - Repairable.
2. Unsafe - Not Repairable.
3. Insufficient evidence to declare unsafe.

River Basins	Counties	
(1) LOWER HUDSON	1 Albany	36 Orange
(2) UPPER HUDSON	2 Allegany	37 Orleans
(3) MOHAWK	3 Bronx	38 Oswego
(4) LAKE CHAMPLAIN	4 Broome	39 Otsego
(5) DELAWARE	5 Cattaraugus	40 Putnam
(6) SUSQUEHANNA	6 Cayuga	41 Queens
(7) CHEMUNG	7 Chautauqua	42 Rensselaer
(8) OSWEGO	8 Chemung	43 Richmond
(9) GENESEE	9 Chenango	44 Rockland
(10) ALLEGHENY	10 Clinton	45 St. Lawrence
(11) LAKE ERIE	11 Columbia	46 Saratoga
(12) WESTERN LAKE ONTARIO	12 Cortland	47 Schenectady
(13) CENTRAL LAKE ONTARIO	13 Delaware	48 Schoharie
(14) EASTERN LAKE ONTARIO	14 Dutchess	49 Schuyler
(15) SALMON RIVER	15 Erie	50 Seneca
(16) BLACK RIVER	16 Essex	51 Steuben
(17) WEST ST. LAWRENCE	17 Franklin	52 Suffolk
(18) EAST ST. LAWRENCE	18 Fulton	53 Sullivan
(19) RACQUETTE RIVER	19 Genesee	54 Tioga
(20) ST. REGIS RIVER	20 Greene	55 Tompkins
(21) HOUSATONIC	21 Hamilton	56 Ulster
(22) LONG ISLAND	22 Herkimer	57 Warren
(23) OSWEGATCHIE	23 Jefferson	58 Washington
(24) GLASSE	24 Kings	59 Wayne
	25 Lewis	60 Westchester
	26 Livingston	61 Wyoming
	27 Madison	62 Yates
	28 Monroe	
	29 Montgomery	
	30 Nassau	
	31 New York	
	32 Niagara	
	33 Oneida	
	34 Onondaga	
	35 Ontario	

DEC

APPENDIX G

DRAWINGS

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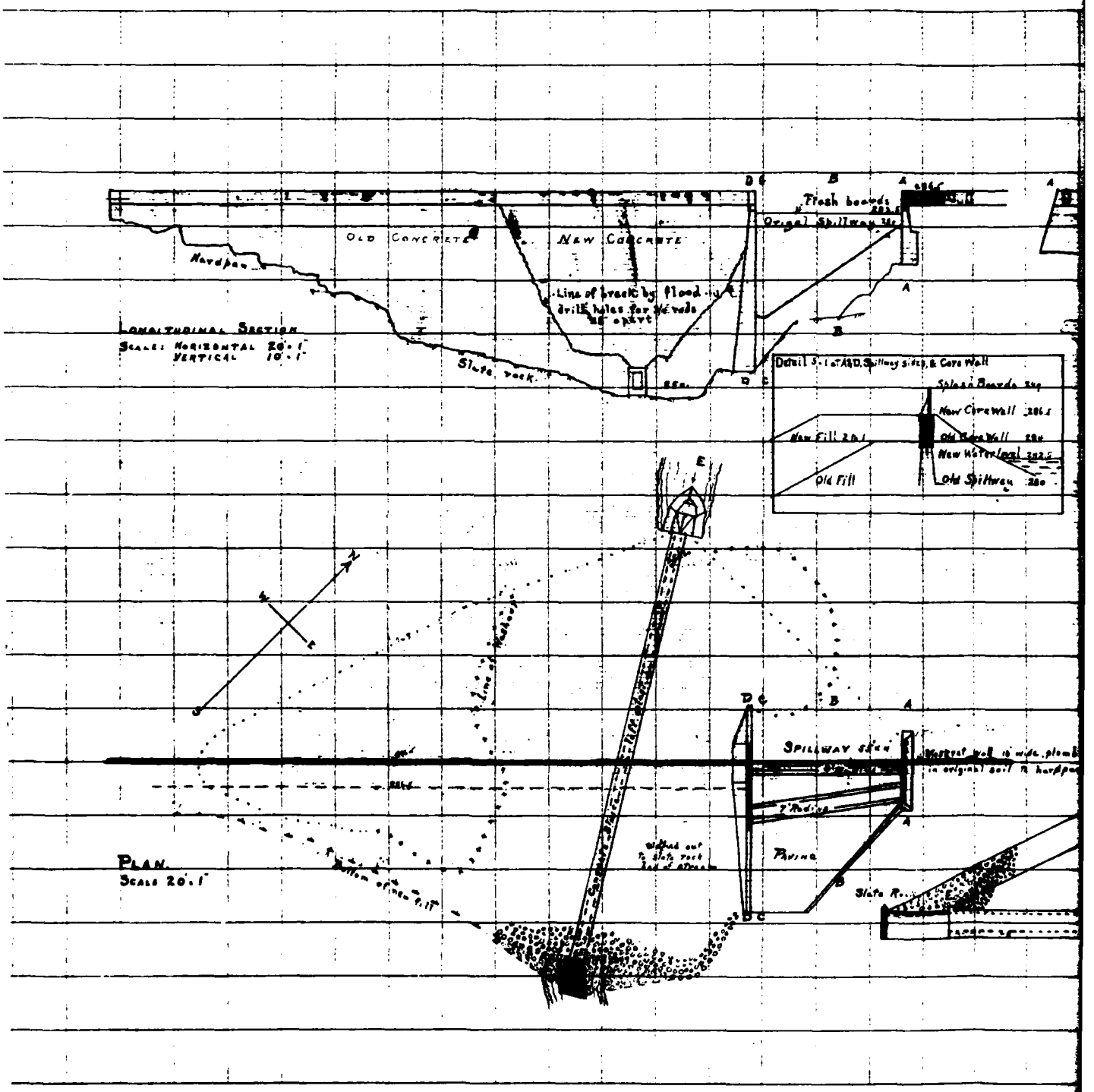
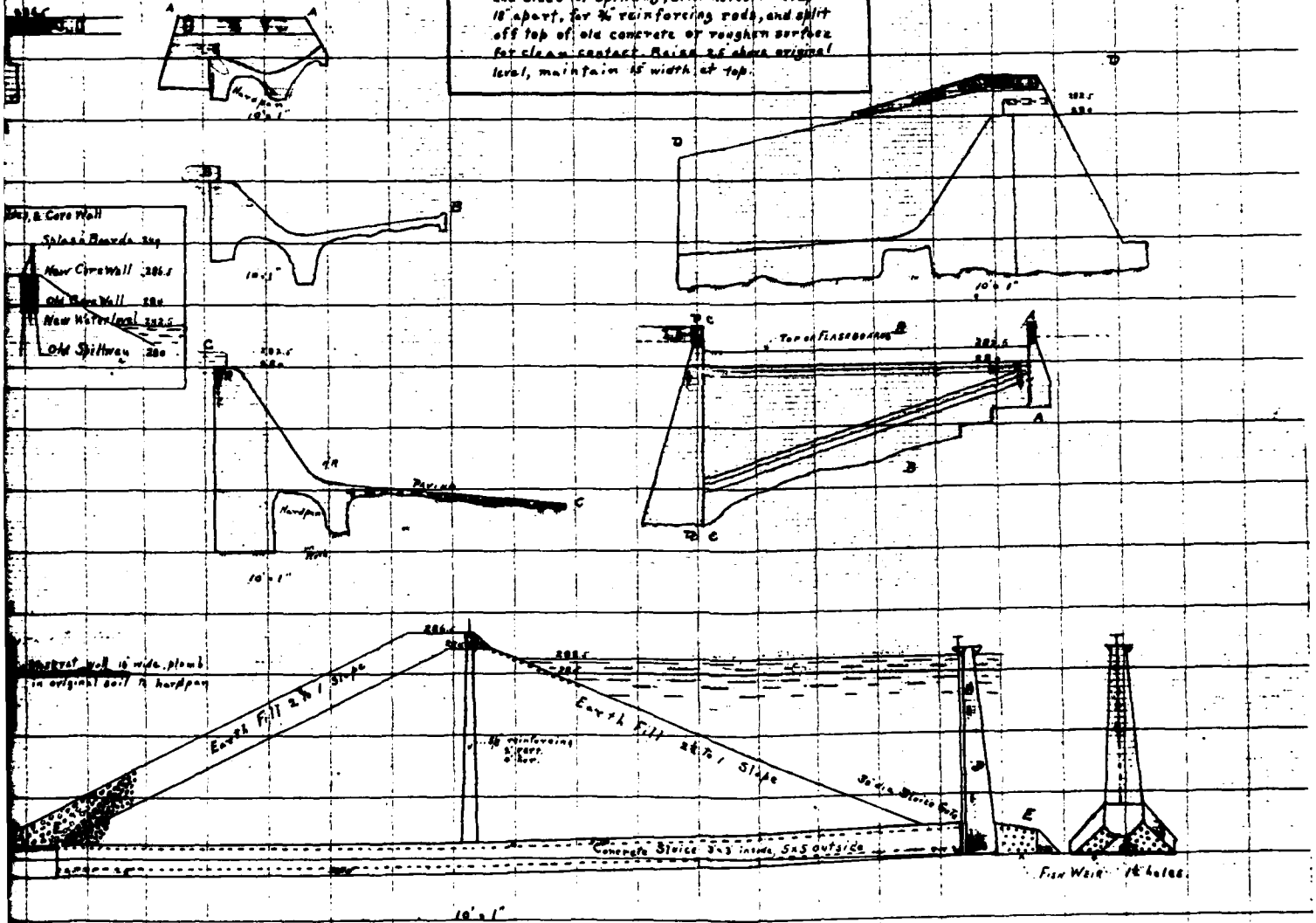
1956 Reconstruction, by T.W. Westlake	
Overall Plan - April 25, 1956	G-2
Flashboard Details - May 10, 1956	G-3



FROM DEC
REDUCED TO 40% OF ORIGINAL

RECONSTRUCTION "TILLSON LAKE"
DAM NEAR RUTSONVILLE, N.Y.
BY H. A. TILLSON, WALDEN, N.Y.
TOWN OF GARDINER ULSTER CO.
SCALE 20'-1" & 10'-1" AS NOTED

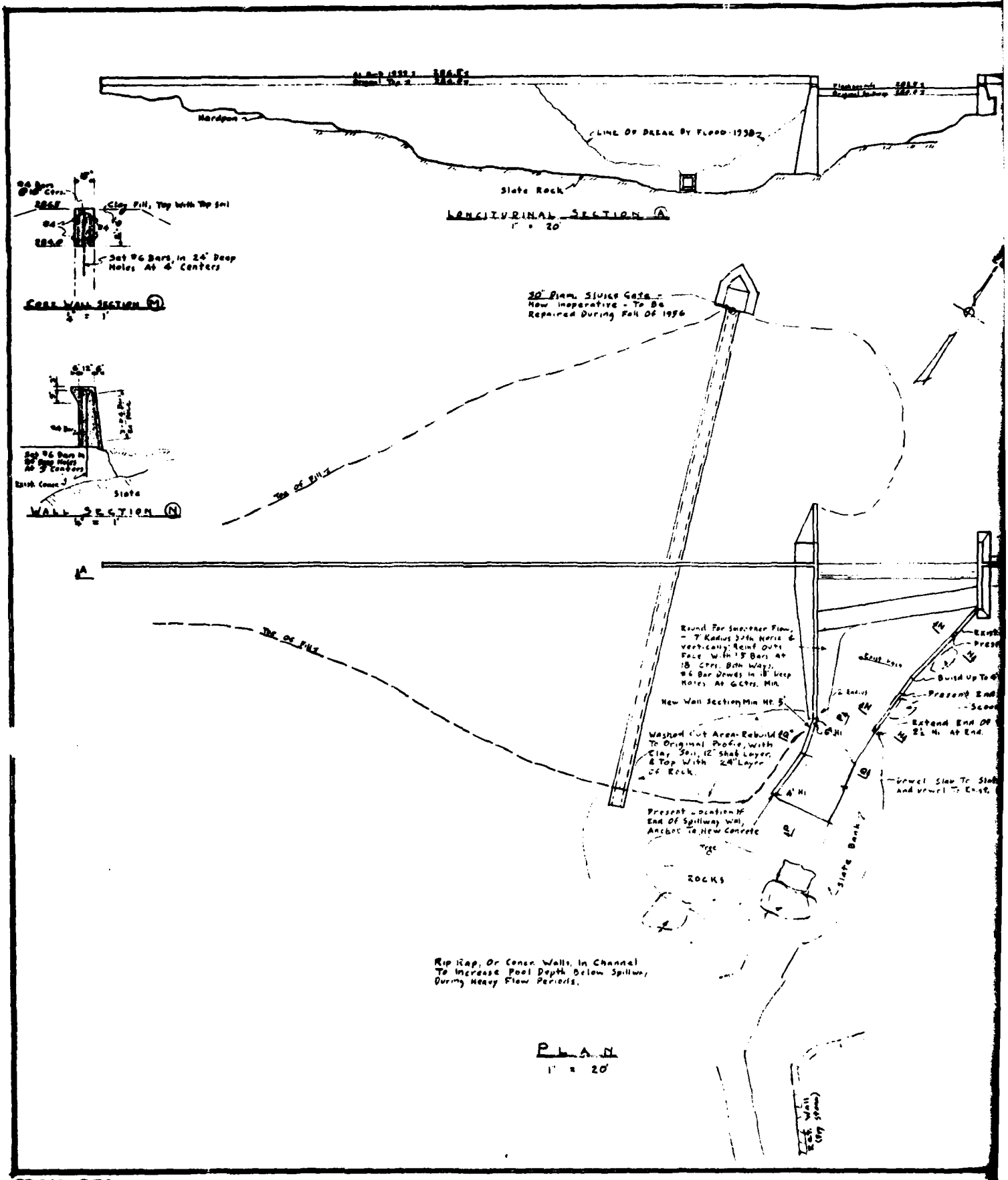
Notes:- For new concrete to raise core wall and sides of spillway, drill holes 12" deep 18" apart, for ¾" reinforcing rods, and split off top of old concrete or roughen surface for clean contact. Raise 2.5' above original level, maintain 15' width at top.



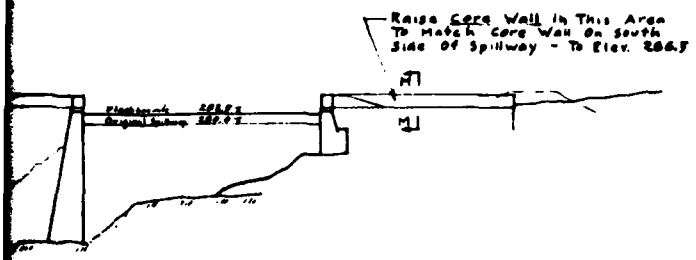
Johnson Le Fevre
Former Sec.
New York, N.Y.
March 28, 1939

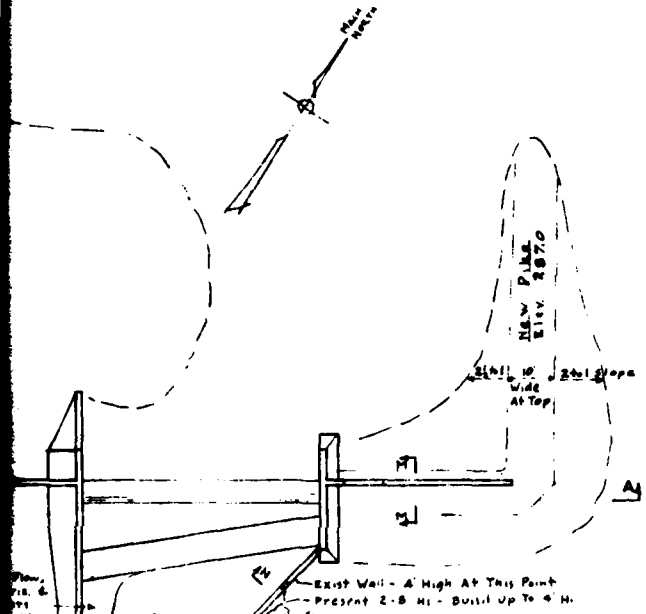
G-1 CTM DWG NO. 81-3

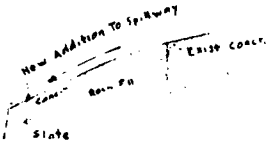
2



FROM DEC
REDUCED TO 50% OF ORIGINAL







SECTION ①

SECTION ②

GENERAL NOTES

1. All Repair Work Indicated On This Plan To Be Done Immediately Except The Sluice Gate - That Is To Repaired At End Of The '56 Summer Season.
2. The Engineer Has Not Checked The Stability Of The Dam Design, And He Has Responsibility Only For The Repairs Indicated.

CONCRETE NOTES

1. Concrete To Be 2,500# Min. Strength.
2. Min. Cover Of Concrete Over Reinf. Bars To Be 2".
3. Roughen Old Concrete Where It Is To Be Joined By New, #6 Bar Dowels In 18" Dp Holes At 4' Ctrs Unless Noted.
4. Minimum New Slab Thickness 15". Reinf. Both Ways With #5 Bars At 18" Ctrs Unless Noted.
5. Dowel New Concrete To Slate Rock Where Possible.

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TILLSON LAKE DAM

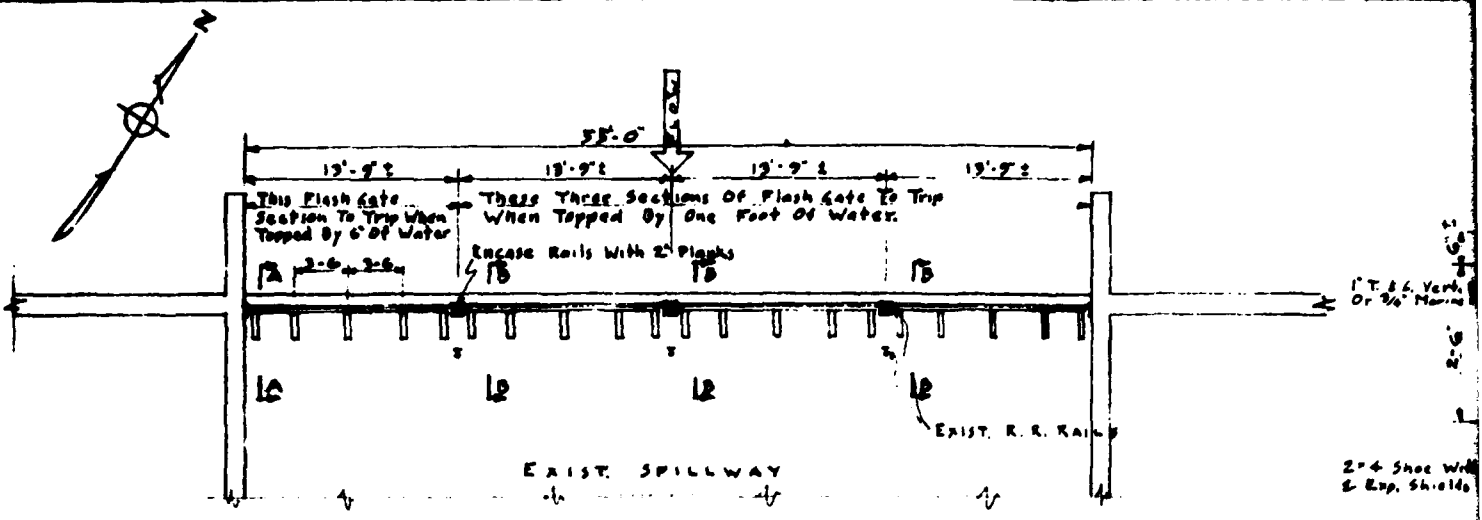
RUTSONVILLE, N.Y.
OWNER - DOMINICK POCCO

TW	4-25-56
SCALE -	AS NOTED
DWG	JOB No. 56-12
	SHEET No. 1
	REV. No. 0

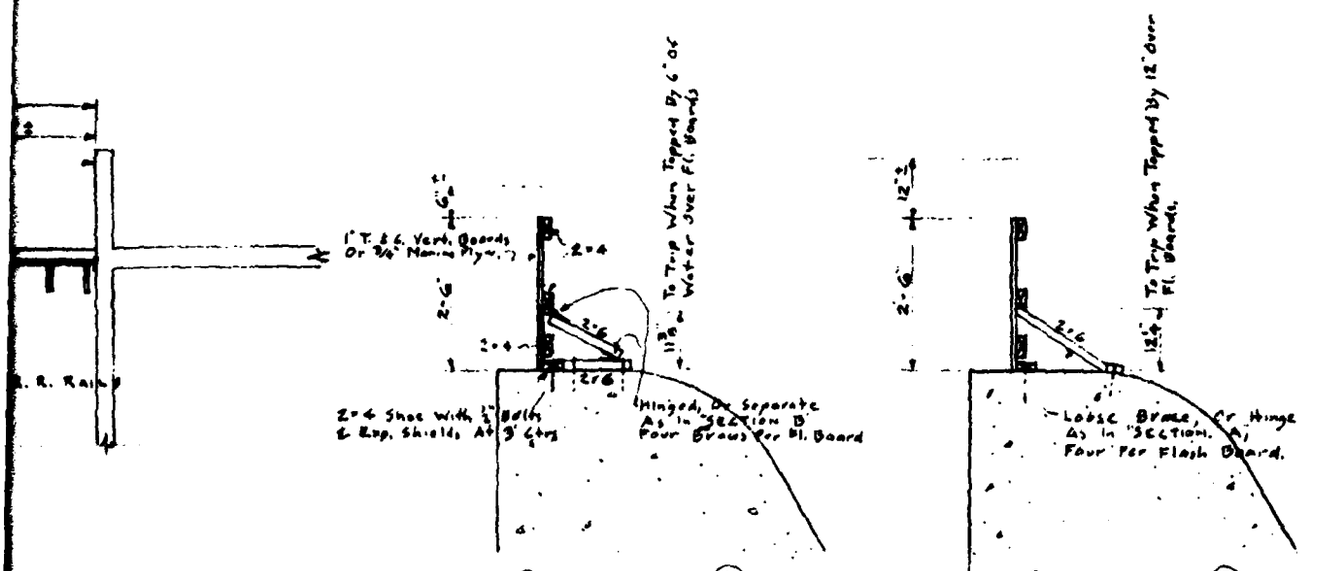
1956 RECONSTRUCTION

G-2 CTM DWG NO. 81-3

2



PLAN VIEW
OF SPILLWAY
1/8" = 1'



SECTION (A)
TO TRIP WITH 6" DEPTH
½" = 1'

SECTION (B)
TO TRIP WITH 12" DEPTH
½" = 1'

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TILLSON LAKE DAM

RUTSONVILLE, N. Y.
OWNER - DOMINICK PORCO

TW	5-10-56
SCALE-	AS NOTED
FILE	JOB NO. 5612 DWG. 2 REV. 0

1956 RECONSTRUCTION
FLASH BOARD DETAILS

G-3 CTM DWG NO. 81-3


DATE
ILME