MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963-A
RICHELIEU RIVER BASIN
WOODBURY, VT

NICHOLS POND DAM
VT 00184

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

APRIL 1980
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Nochols Pond Dam
NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS
U.S. ARMY CORPS OF ENGINEERS
NEW ENGLAND DIVISION

Nochols Pond Dam

DEPT. OF THE ARMY, CORPS OF ENGINEERS
NEW ENGLAND DIVISION, NEDED
424 TRÁPELO ROAD, WALTHAM, MA. 02254

April 1980

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DAMS, INSPECTION, DAM SAFETY,
Richelieu River Basin
Woodbury VT.
Nochols Brook

The dam is a 200 ft. long, 18 ft. high earth and masonry structure. The visual inspection of the dam revealed some minor problems. The general condition of the dam is considered fair. The dam is intermediate in size with a high hazard potential. There are a few recommendations which must be undertaken by the owner.
Identification Number: 00184
Name of Dam: Nichols Pond Dam
Town: Woodbury
County and State: Washington, Vermont
Stream: Nichols Brook
Date of Inspection: October 25, 1979

Nichols Pond Dam is a 200-foot-long, 18-foot-high earth and masonry structure. The dam was originally constructed in about 1900 to provide water supply for the generation of hydroelectric power at Mackville Dam, 2½ miles downstream. Water from Nichols Pond currently augments flows at Pottersville Dam on the Lamoille River. There is a concrete chute spillway approximately in the center of the earth structure which controls normal outflow. There is no emergency spillway and the service spillway is only 7' - 3" wide at its downstream end. Reportedly, a rectangular sluice (2 ft by 5 ft) controlled by two hand operated gates is located underneath the service spillway. The only engineering information available on the structure consisted of past inspection reports by two bureaus of the State of Vermont. There are no design calculations or construction data available.

The visual inspection of Nichols Pond Dam revealed some minor problems. The general condition of the dam is considered fair. The inspection revealed erosion on the crest of the dam, a large mass of debris that deflected flows toward the base of the downstream face, trees growing on the crest and overhanging the downstream channel, deterioration of the gate operating mechanism, no emergency spillway and trespassing on the crest. Based on the dam's intermediate size and High hazard classification in accordance with the Corps' guidelines, the test flood is the full PMF. The test flood for a drainage area of 4.6 square miles is approximately 8,300 cfs. Storage provided by the pond (1,335 acre-feet) will attenuate the test flood to a projected outflow of 5,870 cfs which will overtop the dam by 5.0 feet. The spillway will discharge 218 cfs (3.7% of the routed test flood outflow) with a water level at the top of the dam.
It is recommended that the owner engage a qualified registered engineer to design appropriate structures to control erosion at the base of the spillway and control the accumulation of debris, examine both upstream and downstream faces where not presently visible, perform a hydraulic analysis of the spillway, design an emergency spillway, evaluate the gate structure, and initiate an active maintenance program. The owner should develop a formal surveillance and downstream flood warning plan, including round-the-clock monitoring during heavy precipitation.

The recommendations and remedial measures are described in Section 7 and should be addressed within one year after receipt of this Phase I Inspection Report by the owner.

Very truly yours,

DuBois & King, Inc.

John J. Bilotta, P.E.
Project Manager
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 continued care and inspection can there be any chance that unsafe conditions be detected.

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 compli-
ance with OSHA rules and regulations is also excluded.
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6.1 Visual Observations

The visual observations did not disclose any indications of present structural instability. Undermining of the downstream wall next to the spillway, if worsened, could endanger the future stability of the dam. The debris at the spillway outfall prevented the gathering of data for an analysis of the sluice structure.

6.2 Design and Construction Data

There is practically no design and construction data available. Thus it is not possible to perform a formal analysis of the stability of the dam. A report of an inspection performed by the Public Service Commission in 1949 provides some details but the data is insufficient to perform any analysis.

6.3 Post Construction Changes

There are no post construction changes noted in the available records except for the repairs to the concrete of the upstream concrete face and spillway. The repaired concrete was observed to be in good condition.

6.4 Seismic Stability

The dam is located in Seismic Zone 2 and in accordance with the Phase I inspection guidelines does not warrant seismic analysis.
5.3 Experience Data

There are no recorded experiences of overtopping or any visual accounts of such. However, the rather limited capacity of the spillway (213 cubic feet per second) would tend to indicate that overtopping would occur on frequent basis. The scour or erosion noted on the top of the dam adjacent to the spillway may be an indication of overtopping.

5.4 Test Flood Analysis

The storage capacity of this structure (2840 acre-feet) puts it in the Intermediate size category. The hazard classification is High, since failure of Nichols Pond is likely to endanger the lives of more than a few people at Mackville and result in subsequent overtopping of Mackville Dam (two miles downstream). A failure of Nichols Pond Dam would likely endanger occupants of five dwellings located near Mackville Pond. Based upon "Recommended Guidelines for Safety Inspection of Dams" the test flood is the full Probable Maximum Flood (PMF). The drainage area for Nichols Dam consists of a regulated drainage area (3.4 square miles is controlled by East Long Pond Dam) and an independent drainage area (1.1 square miles). The PMF inflow to Nichols was obtained by adding the routed test flood outflow from East Long Pond Dam to the inflow projected from the Independent drainage area. The PMF envelope curve for Mountainous Areas was used to project inflows for the two drainage areas. The resulting test flood inflow (8300 cfs) for Nichols dam was then routed through the reservoir assuming the water surface to be initially at the crest of the dam (elevation 1130.5 NGVD). Calculations indicate that the dam would be overtopped by 5.0 feet (elevation 1135.5 NGVD). The resulting storage (1335 acre-feet) would attenuate the inflow to 5870 cfs outflow. The routed test flood outflow (5870 cfs) represents a 29% reduction of the test flood inflow.

5.5 Dam Failure Analysis

Utilizing the Corps' April, 1978, "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs," a dam failure analysis was performed for Nichols Pond. Prior to failure, the water level was assumed to be at the crest of the dam (elevation 1130.5 NGVD) and the breach height (upstream toe to water surface) would be 20 feet. A breach width of 70 feet was used in the Saint-Venant equation to compute a breach outflow of 10,500 cfs.

The breach would produce a 11.9-foot high flood wave and the resultant stage of Nichols Brook would be 13.7 feet above streambed at the initial impact area. Approximately two miles downstream lies Mackville Dam. The flood wave would cause subsequent overtopping of Mackville Dam. Appreciable damage could occur to five dwellings located at Mackville with flood levels up to twelve feet above the first floor of some of those dwellings. Another residential area one-half mile further downstream than Mackville has about ten more residences that would be subject to damages resulting by an 11.6-foot high flood wave. Further downstream the outskirts of Hardwick Village would be subjected to a flood wave 6.8 foot high. It is likely that more than a few lives may be lost if Nichols dam is breached, and therefore the dam is classified as High hazard.
5.1 General

Nichols Pond has a fixed crest weir for a principal spillway 37.0 feet wide set at elevation 1128.0. There is a 9 feet wide notch in the crest approximately 6-inches deep. The notch tapers to the width of 7 feet 3-inches and an elevation of 1126.0 at the downstream face. The only outlet for water at the downstream face of the spillway is a notch 7 feet 3 inches wide and 4 feet high. For various flows in the small range, the control for the pool level varies from the upstream to the downstream end. For flows less than 100 cfs, the upstream end of the spillway represents the control. For flows greater than 100 to 150 cfs, the downstream 4 feet deep notch represents the control of the spillway. It is suspected that when flow over the spillway is in a range of 75 to 150 cfs a hydraulic jump may occur in the middle of the spillway. Evidence of this phenomenon is represented by a scour mark approximately three-quarters of the way down the notch in the spillway.

The pond outlet is controlled by two gates with wooden stems which rise vertically in the center of the spillway in the upstream face. There is no information available on the size or invert of the outlet structure. Consequently, no rating or other analysis was performed for the outlet. The location of the gate operating mechanism in the center of the spillway would obviously prevent gate operation during periods of high water.

The watershed of Nichols Pond is relatively steep mountainous terrain covered for the most part with trees and forests. Approximately one-half mile upstream from Nichols Pond lies another large (for this watershed) lake named East Long Pond. The combination of East Long Pond and Nichols Pond have a total lake area at full pool of 350 acres. This represents 12 percent of the total watershed. It is likely that this large lake area will attenuate flood peaks. Both East Long Pond and Nichols Pond are owned and operated by the Village of Hardwick.

5.2 Design Data

The data on the hydrologic design of Nichols Pond Dam is not available. However, a preliminary analysis of the hydraulic characteristics of the spillway indicate that hydraulic control may switch from the upstream face to the throat of the spillway. This may result in a hydraulic jump occurring in the middle of the spillway.
SECTION 4
OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 Operational Procedures

a. General. Operational Procedures consists primarily of opening the gates in the summer time in order to augment flows to the power dam downstream on the Lamoille River. In order to operate the sluice gates, the operator must stand on the crest of the spillway and use a large wrench to turn the ratchet which raises and lowers the timbers attached to the gates. The wrench is kept at the Village maintenance shed approximately three miles downstream. There is no written procedure for lowering the pool level or opening the gates in preparation for a possible flooding event. A 1949 inspection report by an engineer for the Public Service Commission warned that both East Long Pond and Nichols Pond should not be kept full during flood season. The "flood season" was not defined. There is neither any indication that the policy was adopted nor any written operational tool for establishing the level of the two ponds.

b. Warning System. There is no system either to warn of an impending flood or to warn of possible overtopping.

4.2 Maintenance Procedures

a. General. There is no set program for maintaining the dam. Maintenance is performed on an "as-needed" basis. The only operating facilities on the dam are the two sluice gates. At the time of the inspection, the timber stems for both gates were deteriorated and showed signs of rot. There is no established procedure for maintaining these facilities.

4.3 Evaluation

There is a possibility of a serious problem at the downstream end of the spillway. Just beyond the vertical face there is an enormous mat of trees, branches, general trash, and other debris which has accumulated downstream of the spillway. Spillway flows impinge upon this debris and are scattered sideways, possibly causing an undermining of the downstream foundation. There is no written procedure for clearing the debris from the base of the spillway although it was reportedly a regular problem. The general operational and maintenance procedures can be described as poor. The rotten gate stems, the debris at the base of the spillway and the trees growing on the downstream area are indications of neglect.

Current procedures are considered to be inadequate to insure that all problems encountered can be remedied within a reasonable period of time. The owner should establish written procedures for operating and maintaining the structure.
3.2 Evaluation

On the basis of the visual inspection, the dam is judged to be in fair condition. The following features if left unattended could result in the deterioration of the dam:

a. Erosion of soil and resulting undermining of the downstream walls next to the spillways can endanger the stability of the walls. The erosion is probably worsened by the debris accumulated downstream of the spillway which in part deflects the flow laterally.

b. A cavity produced by erosion of earth fill against the left spillway wall, if enlarged, can result in damage to the spillway floor and left wall which in turn could cause flow into the cavity and further erosion. Enlarging of the cavity will develop rapidly in case of overtopping of the dam.

c. The roots of trees growing at the crest next to the downstream wall can exert pressures against the downstream wall.

d. The condition of the downstream spillway wall and its foundation requires inspection after removal of the debris at the spillway discharge.

e. The scour or spalling of the spillway walls may indicate a serious problem with the original design of the spillway. The unusual throat configuration at the downstream end of the spillway may become the hydraulic control, thereby forcing a hydraulic jump in the middle of the spillway. The resulting roller could be the origination of the scour to the left of the spillway as shown in photo 6.
These walls do not reach the elevation of the crest (photo 7). The stone wall appears in good condition and no evidence of seepage was observed in the wall. There are indications of minor lateral movement of the stone wall at mid-height. The concrete walls are also in good condition except for undermining that has occurred at the base of the concrete wall left of the spillway creating a void about 2 feet long and \( \frac{1}{2} \) inch deep. The concrete walls appear slightly bowed at mid-height. A seam observed at mid-height in the left concrete wall (Photo 8) may be the result of movement of concrete forms during initial construction. A masonry patch has been applied to the seam on the downstream face of the wall. Some efflorescence was also observed on the downstream face (Photo 9). There is some trespassing right of the spillway which has caused some deterioration of the downstream wall. The right bank downstream of the dam has an accumulation of debris. (photo 10).

There is a wet area about 20 ft. downstream of the dam, left of the spillway (photo 1). No water flow was evident.

c. **Appurtenant Structures** The spillway walls and floor appear in good condition (photo 11) with some apparent spalling of the floor near the downstream end, (photo 12) Minor cracks in the spillway walls are typical of the cracks caused by concrete shrinkage (photo 13). The downstream face of the spillway could not be observed due to a large amount of debris accumulated against it (photo 14). An undesirable effect of the accumulated debris is a lateral deflection of the water flow resulting in erosion of the banks of the downstream channel adjacent to the dam. The erosion is evident on the left back of the downstream channel (photo 14) where the stump of a tree has rotated about 90°. This erosion could be responsible for the undermining of the concrete wall, as discussed in the previous section. Due to the debris accumulation, it was not possible to observe the condition of the downstream wall of spillway and of the downstream channel bottom immediately downstream for evidences of scour (photo 15).

The gate mechanism for a low-level outlet (photo 16) is a pair of wooden vertical elements which have deteriorated and require replacement. The gate mechanism would not be accessible during floods. The outlet conduit could not be observed due to the debris at the downstream end.

d. **Reservoir Area.** There were no evidences of instability along the reservoir edge in the vicinity of the dam.

e. **Downstream Channel.** The downstream channel is the natural streambed. A small timber bridge for a logging road about 100 feet downstream of the dam would not present a significant obstruction to the flow. There is an abandoned dam, approximately 5 feet high, located about 100 yards downstream. The structure has been breached from the streambed to the right bank. Consequently it was not considered a significant obstruction to flow. These are a few overhanging trees along the downstream channel (Photo 15).
SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. The field inspection of Nichols Pond Dam was performed on October 25 and 26, 1979. The weather was cloudy and cold with temperatures near 32°F. The inspection team included personnel from DuBois & King, Inc.; Geotechnical Engineers Inc.; Knight Consulting Engineers, Inc.; and a representative of the Village of Hardwick. A copy of the inspection checklist as completed during the field inspection is included as Appendix A. At the time of the inspection, the water was at full pool and flowng over the spillway. Consequently, no assessment could be made of the upstream face of the structure.

b. Dam. The dam consists of an earthen embankment with a dry masonry downstream face (photo 1) and a concrete upstream face (photo 2). The upstream face of the dam is a vertical concrete wall (photo 3). A new wall has been built immediately upstream of the original wall. The exposed part of the new wall appears in good condition with only minor spalling and cracking, while the old wall shows severe spalling in its exposed upper part (photo 4). The old wall appears to have settled slightly, on the right side of the spillway. The new wall does not show evidence of settlement.

The crest of the dam is grass covered with the exception of an area near the right abutment which is used for parking. Near the left abutment and also along the downstream edge, there are trees growing on the crest (photo 2). The elevation of the crest is somewhat irregular with areas higher and lower than the elevation of the top of the spillway walls. Adjacent to the left spillway wall there is a cavity about 3 to 4 ft. deep (photo 6). Further downstream along the left spillway wall, there is another cavity against the downstream wall (photo 5). It is possible that the two cavities are connected and may have formed when the dam has been overtopped and water has flowed into the upstream cavity and then downwards between the downstream wall and earth fill.

The downstream face of the dam consist of a dry masonry stone wall (photo 1) and concrete walls next to the spillway.
SECTION 2
ENGINEERING DATA

2.1 Design

Information on the design as well as specifications were not available for Nichols Pond Dam. The field sketch for this dam shows observable dimensions only.

2.2 Construction Data

Reports and records of construction were not available.

2.3 Operation

No operating manual was available for Nichols Pond Dam. Operating personnel reported that the facilities were operated annually to effect flow augmentation for hydro-power. There is no known schedule for monitoring the structure. There are records of past inspections performed by the Vermont Department of Water Resources and the Public Service Commission. These reports were valuable since they supplied additional dimensions which were unavailable at the time of the visual inspection.

2.4 Evaluation

a. Availability. The available information is not sufficient for stability analyses of the dam or the appurtenant structures. The only background data which could be located consisted of inspection reports by the Public Service Board and the Department of Water Resources of the state of Vermont.

b. Adequacy. The lack of engineering data did not allow for a definitive review. Therefore, the adequacy of this dam could not be assessed from the standpoint of reviewing design and construction data. All assessments were based primarily on the visual inspection, records of past performance, and sound hydrologic and structural engineering judgment.

c. Validity. Not applicable.
f. Reservoir Surface (acres).

1. Normal Pool 162
2. Flood-Control Pool N/A
3. Spillway Crest 162
4. Test Flood Pool 182
5. Top of Dam 167

g. Dam

1. Type earth and masonry structure
2. Length approximately 200 feet
3. Height approximately 18 feet
4. Top Width 30 feet (varies)
5. Side Slopes Vertical
6. Zoning N/A
7. Impervious Core N/A
8. Cut-Off Unknown
9. Grout Curtain Unknown
10. Other N/A

h. Diversion and Regulating Tunnel. Not Applicable.

i. Spillway.

1. Type Concrete overflow in center of dam
2. Length of Weir Varies from 37 feet to 7.25 feet
3. Crest Elevation Varies from 1127.5 to 1126.0
4. Gates N/A
5. Upstream Channel N/A
6. Downstream Channel Natural river bed

j. Regulating Outlets. Two sluice gates are located in the center of the dam. Reportedly, the outlet conduit is a 2 ft by 5 ft rectangular sluice. The gates are hand operated through a ratchet mechanism located in center of the principal spillway.
4) **Spillway Capacity at Test Flood Elevation.** The capacity of the spillway at test flood (elevation 1135.5 NGVD) is approximately 600 cfs. This represents approximately 10% of the routed test flood outflow.

5) **Total Project Discharge at Top of Dam.** At the top of the dam, the project will discharge 218 cfs at elevation 1130.5.

6) **Total Project Discharge for Test Flood Elevation.** The total project will discharge 5,870 cfs at 1135.5 elevation.

c. **Elevation (NGVD)**
   1. Stream Bed at Toe of Dam 1110 ±
   2. Bottom of Cut-off Unknown
   3. Maximum Tailwater Unknown
   4. Recreation Pool 1127.5
   5. Full Flood Control Pool N/A
   6. Spillway Crest (Ungated) 1127.5
   7. Design Surcharge (Original Design) Unknown
   8. Top of Dam 1130.5
   9. Test Flood Design Surcharge 1135.5

d. **Reservoir (length in feet).** Nichols Pond is approximately circular in plan, and it is 3,700 feet from the dam to the inflowing stream at normal pool. At the test flood elevation (1135.5) the pond would be about 5,000 feet long.

e. **Storage (acre-feet).**
   1. Normal Pool 2590
   2. Flood Control Pool N/A
   3. Spillway Crest Pool 2590
   4. Top of Dam 2841
   5. Test Flood Pool 3925
h. **Design and Construction History.** The history of the design and construction of Nichols Pond Dam is not available. It was reportedly constructed circa 1900.

i. **Normal Operating Procedure.** Nichols Pond Dam is maintained for flow augmentation for a power dam on the Lamoille River. The gates are reportedly opened in mid-summer and the pond level is maintained at approximately the spillway level (1127.5 NGVD). The gates are then closed in the spring to raise the pool level above the spillway level.

1.3 **Pertinent Data**

a. **Drainage Area.** The drainage basin of Nichols Pond Dam includes an area of 4.6 square miles. The land is mostly forested and the terrain is extremely steep and mountainous. One-half mile upstream along Nichols Brook, lies East Long Pond Dam which controls 3.5 square miles of the watershed. The basin is sparsely populated and there are very few houses and practically no paved roads.

The maximum reservoir area of 167 acres represents approximately 6% of the total drainage area. The predominant soils in the watershed are Glover-Calais and Calais-Buckland.

b. **Discharge at the Dam Site.**

(1) **Outlet Works.** Two sluice gates are located in the center of the structure. The gate operating mechanism consists of a timber riser attached to a ratchet system which is operated by a large wrench. In order to operate the gate mechanism, the operator has to stand in the center of the spillway. Consequently, the gates could not be operated during flood flows. Reportedly, the outlet conduit is a 2 ft by 5 ft rectangular sluice located in the center of the dam. The inlet of the sluice is located approximately 13.5 feet below the top of the dam.

(2) **Maximum Known Flood.** There were no records available nor were there any witnesses of any past flooding at the site.

(3) **Spillway Capacity at Top of Dam.** The principal spillway is a 37-foot wide structure which is notched approximately in the center. At the upstream end of the spillway, the notch is six inches deep and nine feet wide. At the downstream end of the spillway, the notch is four feet deep and 7 foot 3 inches wide. Above 100 cfs it is considered that this downstream throat would provide control by critical depth. This is the only uncontrolled outflow for the structure and its capacity at the top of the dam elevation 1130.5 is approximately 218 cfs (3.7% of the routed test flood outflow).
The upstream face has been cAPPED with two layers of concrete. The downstream face is a dry-stone masonry wall in some places and there has been concrete facing applied to a certain area in the center of this structure. The dam itself is bisected by a concrete spillway which varies in width from 37 feet at the upstream end to 7 feet 3 inches at the downstream end. The spillway varies in depth from 1.5 feet at the upstream end to 4 feet at the downstream end. The crest of the dam varies in elevation between 1130 feet above mean sea level to 1131 feet above mean sea level. The lowest point in the spillway is at elevation 1127.5. The inlet invert of the sluice is approximately at elevation 1117.0 NGVD.

There is neither any emergency spillway nor any other provision for discharging flood flows.

c. **Size Classification.** Nichols Pond Dam is 18 feet high and has a storage capacity of 2840 acre-feet. In accordance with article 2.1.1 of the Recommended Guidelines for Safety Inspection of Dams, the dam is Intermediate in size based upon its storage capacity which is greater than 1000 acre-feet and less than 50,000 acre-feet.

d. **Hazard Classification.** The dam has a hazard classification of High based upon its potential for damage. Approximately 2 miles downstream lies Mackville Dam. The flood wave generated by a breach of Nichols Pond Dam with a water level at the top of the dam would be approximately 11.4 feet high when it reached the Mackville Dam Pond. It is considered that the flood wave generated by a breach of Nichols Pond Dam would cause subsequent overtopping of Mackville Dam. Appreciable damage could occur to five dwellings located at Mackville with flood levels up to five feet above the first floor of some of those dwellings. Another residential area one-half mile further downstream than Mackville has about ten more residences that would be subject to the resultant flood by an 11.6-foot-high wave. The outskirts of the Village of Hardwick would be subjected to a flood wave 6.8 feet high. It is possible that more than a few lives may be lost if Nichols Pond Dam is breached.

e. **Ownership.** This dam is owned by the Village of Hardwick Electric Light Department. The dam was originally owned by Woodbury Granite Company and then by Green Mountain Power Corporation before it was acquired by its present owner.

f. **Operator.** The dam is operated and maintained by the Village of Hardwick, Vermont 05843. Mr. William Fee, Village Manager, is in charge of all Village equipment. His telephone number is 802/472-5201.

g. **Purpose.** The original purpose of the dam was to provide water supply to operate Mackville Dam for power generation. The power generating facilities of Mackville Dam have been eliminated; however, the outflow from Nichols Pond Dam is used to augment the flows for another dam on the Lamoille River at Pottersville which generates power for the Village of Hardwick Electric Light Department.
1.1 General

a. Authority. 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 England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. DuBois & King, Inc., has been retained by the New England Division to inspect and report on selected dams in the State of Vermont. Authorization and notice to proceed were issued to DuBois & King, Inc., under a letter of October 19, 1979 from William E. Hodgson, Jr., Colonel, Corps of Engineers. Contract No. DACW33-80-C-0003 has been assigned by the Corps of Engineers for this work.

b. Purpose

(1) To perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-Federal interests.

(2) To encourage and prepare the states to quickly initiate effective dam safety programs for non-Federal dams.

(3) To update, verify and complete the National Inventory of Dams.

1.2 Description of Project

a. Location. Nichols Pond Dam is located in the Town of Woodbury, Vermont on Nichols Brook approximately three miles upstream from its confluence with Cooper Brook. The dam is shown on the 15 minute U.S.G.S. Quadrangle for Plainfield, Vermont, with coordinates approximately 72° 20.6' west longitude, 44° 27.7' north latitude, Washington County, Vermont. The location of Nichols Dam is shown on the location map immediately preceding this page.

b. Description of Dam and Appurtenances. Nichols Pond Dam is an earth and masonry structure approximately 18 feet high with vertical walls both upstream and downstream. The breadth of the structure varies from 28 to 44 feet with an average breadth of approximately 30 feet.
7.1 Dam Assessment

a. Condition. The visual inspection indicated the dam to be generally in fair condition. Items that could result in deterioration of this condition are:

- Erosion of soil downstream of dam, next to spillway, and resulting undermining of the downstream wall of the dam.
- A cavity at the crest, next to the left spillway wall.
- Trees growing next to the downstream wall.
- The timber risers that serve as gate stems are badly deteriorated.
- No emergency spillway.
- Scouring and spalling of the spillway.

The assessment of the present condition of the dam is subject to verification by inspection of the downstream wall of the spillway which could not be observed due to accumulation of debris.

b. Adequacy of Information. The information available was practically nil and thus the assessment of the condition of the dam is based solely on the visual inspection.

c. Urgency. The recommendations presented in Section 7.2 and 7.3 should be carried out within one year of receipt of this report by the owner.

7.2 Recommendations

The following investigations and needed corrections should be performed under the direction of a registered engineer qualified in the design and construction of dams.

Removal of debris that has accumulated downstream of the spillway and examination of the downstream spillway wall and its foundation for scour and possible undermining, and design of any repairs which might be needed.
Design measures to prevent accumulation of debris at the spillway discharge. This may include the design and installation of a log boom across the intake channel.

Design an appurtenant structure for the base of the spillway to prevent erosion of banks immediately downstream of the spillway. This may include an additional structure such as a "plunge pool" or a re-regulating weir to provide backwater.

Hydraulic analysis of the spillway to determine whether or not a hydraulic jump will form within the confines of the spillway. This may precipitate the redesign of the spillway structure to eliminate this undesirable occurrence.

Hydrologic and economic evaluation of the installation of an emergency spillway of sufficient capacity to protect the safety of the dam. This may include the raising of the structure to safely pass a design flood.

Replacement of deteriorated timber stems and a thorough examination of the gates and trash rack.

7.3 Remedial Measures

a. Operation and Maintenance Procedures

1. Remove trees and bushes growing on the crest and overhanging the downstream channel.

2. Fill the cavity next to the spillway left wall with lean concrete or compacted clayey soil.

3. Repair cracked and spalled concrete on the principal spillway.

4. Restrict trespassing from the crest of the structure.

5. Establish a program of annual technical inspections by a registered qualified engineer.

6. Develop a formal surveillance and downstream flood warning plan including round-the-clock monitoring during heavy precipitation.

7.4 Alternatives

There are no alternatives which are consistent with the present uses of the dam.
APPENDIX A

VISUAL CHECKLIST WITH COMMENTS
<table>
<thead>
<tr>
<th>PARTY:</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. John Bilotta D&amp;K</td>
<td></td>
</tr>
<tr>
<td>2. Jeffrey Spaulding D&amp;K</td>
<td></td>
</tr>
<tr>
<td>3. Gonzalo Castro GEI</td>
<td></td>
</tr>
<tr>
<td>4. Stephen Knight KCE</td>
<td></td>
</tr>
<tr>
<td>5. Erwin Gileris Village of Hardwick</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>PROJECT FEATURE</th>
<th>INSPECTED BY</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Structure</td>
<td>S. Knight</td>
<td></td>
</tr>
<tr>
<td>2. Foundations</td>
<td>G. Castro</td>
<td></td>
</tr>
<tr>
<td>3. Hydraulics/Electric Mechanical</td>
<td>J. Bilotta</td>
<td></td>
</tr>
</tbody>
</table>
# INSPECTION CHECK LIST

<table>
<thead>
<tr>
<th>AREA EVALUATED</th>
<th>CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAM ENBANKMENT</td>
<td></td>
</tr>
<tr>
<td>Crest Elevation</td>
<td>Earth with upstream vertical concrete face and downstream vertical stone face.</td>
</tr>
<tr>
<td>Current Pool Elevation</td>
<td>1-inch over spillway crest.</td>
</tr>
<tr>
<td>Maximum Impoundment to Date</td>
<td>Unknown</td>
</tr>
<tr>
<td>Surface Cracks</td>
<td>A few cracks on upstream face.</td>
</tr>
<tr>
<td>Pavement Condition</td>
<td>N.A.</td>
</tr>
<tr>
<td>Movement or Settlement of Crest</td>
<td>Possible settlement of old upstream concrete face to right of spillway.</td>
</tr>
<tr>
<td>Lateral Movement</td>
<td>None observed.</td>
</tr>
<tr>
<td>Vertical Alignment</td>
<td>Crest very irregular.</td>
</tr>
<tr>
<td>Horizontal Alignment</td>
<td>Downstream stone wall bowed.</td>
</tr>
<tr>
<td>Condition at Abutment</td>
<td>Downstream concrete bowed-likely due to defective form.</td>
</tr>
<tr>
<td>Indications of Movement of Structural Items on Slopes</td>
<td>Minor erosion at end of concrete wall at right abutment (u.s.).</td>
</tr>
<tr>
<td>Trespassing on Slopes</td>
<td>N.A.</td>
</tr>
<tr>
<td>Vegitation on Slopes</td>
<td>Downstream edge of crest right of spillway - footpath. Also logging road around right end which is at same elevation as dam. Some at earth slope near right abutment.</td>
</tr>
<tr>
<td>Sloughing or Erosion of Slopes or Abutments</td>
<td>Erosion of crest against left wall of spillway.</td>
</tr>
<tr>
<td>Rock Slope Protection - Riprap Failures</td>
<td>N.A.</td>
</tr>
</tbody>
</table>
## Inspection Check List

**Project** Nichols Pond Dam  
**Date** 10-25/26-79  
**Name** J. Bilotta D&K  
**Name** S. Knight KCE  
**Name** G. Castro GEI

<table>
<thead>
<tr>
<th>Area Evaluated</th>
<th>Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outlet Works - Intake Channel and Intake Structure</td>
<td></td>
</tr>
<tr>
<td>a. Approach Channel</td>
<td>None observable.</td>
</tr>
<tr>
<td>Slope Conditions</td>
<td></td>
</tr>
<tr>
<td>Bottom Conditions</td>
<td></td>
</tr>
<tr>
<td>Rock Slides or Falls</td>
<td></td>
</tr>
<tr>
<td>Log Boom</td>
<td></td>
</tr>
<tr>
<td>Debris</td>
<td></td>
</tr>
<tr>
<td>Condition of Concrete Lining</td>
<td></td>
</tr>
<tr>
<td>Drains or Weep Holes</td>
<td></td>
</tr>
<tr>
<td>b. Intake Structure</td>
<td></td>
</tr>
<tr>
<td>Condition of Concrete</td>
<td></td>
</tr>
<tr>
<td>Stop Logs and Slots</td>
<td>Riser stems - vertical timbers that are used to raise gates are deteriorated.</td>
</tr>
<tr>
<td>AREA EVALUATED</td>
<td>CONDITIONS</td>
</tr>
<tr>
<td>------------------------------------</td>
<td>-----------------------------------------------------------------------------</td>
</tr>
<tr>
<td>OUTLET WORKS - CONTROL TOWER</td>
<td>None as such.</td>
</tr>
<tr>
<td>A. Concrete and Structural</td>
<td></td>
</tr>
<tr>
<td>General Condition</td>
<td></td>
</tr>
<tr>
<td>Condition of Joints</td>
<td></td>
</tr>
<tr>
<td>Spalling</td>
<td></td>
</tr>
<tr>
<td>Visible Reinforcing</td>
<td></td>
</tr>
<tr>
<td>Rusting or Staining of Concrete</td>
<td></td>
</tr>
<tr>
<td>Any Seepage or Leaks in Gate Chamber</td>
<td></td>
</tr>
<tr>
<td>Cracks</td>
<td></td>
</tr>
<tr>
<td>Rusting or Corrosion of Steel</td>
<td></td>
</tr>
<tr>
<td>b. Mechanical and Electrical</td>
<td></td>
</tr>
<tr>
<td>Air Vents</td>
<td>None.</td>
</tr>
<tr>
<td>Float Wells</td>
<td>None.</td>
</tr>
<tr>
<td>Crane Hoist</td>
<td>None.</td>
</tr>
<tr>
<td>Elevator</td>
<td>None.</td>
</tr>
<tr>
<td>Hydraulic System</td>
<td>None.</td>
</tr>
<tr>
<td>Service Gates</td>
<td>Hand operated by a large wrench - we were told that it is operated at least annually not accessible when water flowing over spillway, wrench is kept at garage.</td>
</tr>
<tr>
<td>Emergency Gates</td>
<td>None.</td>
</tr>
<tr>
<td>Lighting Protection System</td>
<td>None.</td>
</tr>
<tr>
<td>Emergency Power System</td>
<td>None.</td>
</tr>
<tr>
<td>Wiring and Lighting System in Gate Chamber</td>
<td>None.</td>
</tr>
<tr>
<td>DAM EMBANKMENT CONTINUED</td>
<td>N.A. but there is a cavity under concrete wall left of spillway (0.5' x 2.0').</td>
</tr>
<tr>
<td>----------------------------------------------------------------</td>
<td>--------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Unusual Movement or Cracking at or near Toes</td>
<td>5' x 10' wet area 20' downstream of dam opposite stonewall on left of spillway.</td>
</tr>
<tr>
<td>Embankment or Downstream Seepage</td>
<td>None observed.</td>
</tr>
<tr>
<td>Piping or Boils</td>
<td>None known.</td>
</tr>
<tr>
<td>Foundation Drainage Features</td>
<td>None known.</td>
</tr>
<tr>
<td>Toe Drains</td>
<td>None known.</td>
</tr>
<tr>
<td>Instrumentation System</td>
<td>None known.</td>
</tr>
</tbody>
</table>
# Inspection Check List

**Project**: Nichols Pond Dam  
**Date**: 10-25/25-79  
**Project Feature**:  
**Discipline**:

<table>
<thead>
<tr>
<th>Area Evaluated</th>
<th>Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outlet Works - Transition and Conduit</td>
<td>Outlet of conduit not visible because of debris at discharge covering outlet completely.</td>
</tr>
<tr>
<td>General Condition of Concrete</td>
<td></td>
</tr>
<tr>
<td>Rust or Staining on Concrete</td>
<td></td>
</tr>
<tr>
<td>Spalling</td>
<td></td>
</tr>
<tr>
<td>Erosion or Cavitation</td>
<td></td>
</tr>
<tr>
<td>Cracking</td>
<td></td>
</tr>
<tr>
<td>Alignment of Monoliths</td>
<td></td>
</tr>
<tr>
<td>Alignment of Joints</td>
<td></td>
</tr>
<tr>
<td>Numbering of Monoliths</td>
<td></td>
</tr>
</tbody>
</table>
## Inspection Check List

**Project** Nichols Pond Dam  
**Date** 10-25/26-79  
**Name** J. Bilotta D&K  
**Discipline** S. Knight KCE  
**Name** G. Castro GEI

<table>
<thead>
<tr>
<th>Area Evaluated</th>
<th>Conditions</th>
</tr>
</thead>
</table>
| **Outlet Works - Outlet Structure and Outlet Channel** | No outlet structure.  
See spillway for condition of channel. |
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS

a. Approach Channel
   General Condition Non visible.
   Loose Rock Overhanging Channel N.A.
   Trees Overhanging Channel N.A.
   Floor of Approach Channel N.A.

b. Weir and Training Walls
   General Condition of Concrete Good.
   Rust or Staining Minor staining.
   Spalling Concrete spall in bottom of spillway channel apparently due to cavitation.
   Any Visible Reinforcing None observed.
   Any Seepage or Efflorescence None observed.
   Drain Holes None observed.

c. Discharge Channel
   General Condition Full of debris for 30' downstream of dam
   Loose Rock Overhanging Channel None.
   Trees Overhanging Channel Yes, Several - some have fallen into channel.
   Floor of Channel Boulderers
   Other Obstructions Logging road bridge at about 100' downstream

NOTE: Both banks of discharge channel severely eroded adjacent to dam.
<table>
<thead>
<tr>
<th>AREA EVALUATED</th>
<th>CONDITIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTLET WORKS - SERVICE BRIDGE</td>
<td>No service bridge.</td>
</tr>
<tr>
<td>a. Super Structure</td>
<td></td>
</tr>
<tr>
<td>Bearings</td>
<td></td>
</tr>
<tr>
<td>Anchor Bolts</td>
<td></td>
</tr>
<tr>
<td>Bridge Seat</td>
<td></td>
</tr>
<tr>
<td>Longitudinal Members</td>
<td></td>
</tr>
<tr>
<td>Under Side of Deck</td>
<td></td>
</tr>
<tr>
<td>Secondary Bracing</td>
<td></td>
</tr>
<tr>
<td>Deck</td>
<td></td>
</tr>
<tr>
<td>Drainage System</td>
<td></td>
</tr>
<tr>
<td>Railings</td>
<td></td>
</tr>
<tr>
<td>Expansion Joints</td>
<td></td>
</tr>
<tr>
<td>Paint</td>
<td></td>
</tr>
<tr>
<td>b. Abutment &amp; Piers</td>
<td></td>
</tr>
<tr>
<td>General Condition of Concrete</td>
<td></td>
</tr>
<tr>
<td>Alignment of Abutment</td>
<td></td>
</tr>
<tr>
<td>Approach to Bridge</td>
<td></td>
</tr>
<tr>
<td>Condition of Seat &amp; Backwall</td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX B

ENGINEERING DATA
APPENDIX B

1. There are no known records of design, construction or maintenance.

2. A copy of an inspection performed by Stephen Haybrook for the Public Service Commission can be found on pages B-2 through B-5. The inspection was dated October 26, 1949. A copy of an analysis performed in 1954 by Louis M. Laushey for the Public Service Commission appears on pages B-6 through B-11. A copy of an inspection performed in 1979 by A. Peter Baranaco for the Department of Water Resources can be found on pages B-12 through B-14. A letter report by Mr. Barranco describes some of the history on page B-15.

3. Plans and sketches prepared by DuBois & King, Inc., appear on figure B-1, page B-16. Information shown on these plans and sketches is based upon information in past inspection reports and observations made during the visual inspection. Dimensions or materials indicated at the time of inspection were not verified. Elevations shown are based upon USGS datum.

4. There are no known records of subsurface investigations.
Concrete wall 6" x 1/8" = 1" thick.
Concrete spillway - good.

Possible undermining.

Survey/Inspection 7-12-87
Heat, Clear
Hand held, C-rule & clock

Notes (c)
MEMORANDUM

To: File
From: A. Peter Barranco, Jr., P.E., Dam Safety Engineer
Subject: Nichols Pond Dam - Woodbury (252-1)

The writer inspected subject dam, obtained dimensions and photographs on July 12, 1979.

Overall the dam appears to be in good condition but in need of brush and tree removal on crest and downstream slopes and debris removal below spillway.

The only items of some concern are two areas where erosion has occurred. One is a cavity about 10' long, 2-3' wide and 3' deep adjacent to the left spillway wall. It could not be determined if this resulted from overtopping or from subsidence due to soil being removed internally. The downstream face was so packed with debris (logs and branches) that an examination of the downstream base of the wall was not possible. The other area is about 30' left of the spillway on the downstream side of the crest. Since there is only about 1.5' of freeboard at normal pool and a relatively small spillway that is subject to blockage by debris, overtopping of this dam would appear likely.

There may be some leakage and undermining below downstream wall, however, debris prevented a close examination. Concrete in spillway and walls in generally good condition. Water level was about 0.4' above crest of low flow notch, or about 1.5' below crest. Crest is somewhat irregular. Wood in gate stems may be nearing end of its useful life.
DAM INSPECTION STATUS

Name: Nicheles Res
DWR No.: 252-1
Town: Crockett
NDS No.: VTOO 184
Owner: Village of Mulberry (Electric Dept)
Hazard Class: II
Address: Mulberry St
Size Category: II
Telephone: 422-522 (Cty. Village Elec. Dept)
Inspect every 4 years * 4

Type: EF/S, Height: 18
Storage: 1200
Use: P(5) Juris. PSB

INSPECTION RECORD
* * Not required. PSB jurisdiction
Date inspects when an area on
informed basis.

<table>
<thead>
<tr>
<th>Inspection Date</th>
<th>Inspected By</th>
<th>Report Date</th>
<th>Owner Notified</th>
<th>Condition Summary</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-9-47</td>
<td>SHH (PSC)</td>
<td>10-26-49</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5-9-53</td>
<td>SHH (PSC)</td>
<td>5-9-53</td>
<td></td>
<td></td>
</tr>
<tr>
<td>11-22-54</td>
<td>PSC (Cty.)</td>
<td>11-26-54</td>
<td>12-1-54</td>
<td></td>
</tr>
<tr>
<td>7-27-73</td>
<td>PHS (PSC)</td>
<td>7-27-73</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-12-79</td>
<td>A-13 (PSC)</td>
<td>10-12-79</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

POTENTIAL DOWNSTREAM HAZARDS

<table>
<thead>
<tr>
<th>Description</th>
<th>Miles Downstream</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mulberry Dam (Res)</td>
<td>2.5</td>
<td>Subject to overflowing by MULberry Pond Rd.</td>
</tr>
<tr>
<td>Mulberry (Village)</td>
<td>2.6 - 3.1</td>
<td>TH bridge, power lines, structures, trailers</td>
</tr>
<tr>
<td>Route 4 (Highway)</td>
<td>3.1</td>
<td>Bridge</td>
</tr>
</tbody>
</table>

* Hazard Class II based on possible overflowing and failure of Mulberry Pond Dam

INFORMATION AVAILABLE

Plans
Dimensions (field check)
Photos

INFORMATION NEEDED NEXT INSPECTION

Dimensions (field check)
Detailed Survey
Photos
Nichols Pond Dam can handle floods safely. The rate of 800 cubic feet per second discharged from East Long Pond will be reduced some by channel storage, and Nichols Dam might possibly have to discharge 600 cubic feet per second plus the estimated 200 to 320 cubic feet per second from its own watershed. The sum would be of the order of magnitude of more than 1000 cubic feet per second—which it could not do safely with its estimated capacity of not more than one-half this amount.

Some of those figures were estimated simply to get an approximate answer. With more complete information on the nature of the one-half mile long connecting stream and the reservoir characteristics at East Long Pond, more detailed calculations could give a more exact answer. However, it seems clear that the outflow capacity at Nichols Dam is adequate as long as both reservoirs are not full at the same time; under these latter circumstances the Nichols outflow capacity (spillway and sluice) would be about fifty percent adequate to handle a major flood on both areas.

**Recommendations:**

1. Repair head wall, spillway slab and wall, and earth embankment as described previously.

2. Definitely avoid having both reservoirs full at the same time during seasons when heavy runoffs can be expected.

*Louis M. Laushey*
Professional Engineer
Nichols Pond Dam

Fanning, for New England streams (maximum flow)

\[ Q = 200(1) = 200 \text{ cubic feet per second} \]

Rational Formula (2" rain per hour, 25\% runoff)

\[ Q = (0.25) (2) (640) = 320 \text{ cubic feet per sec.} \]

All of these empirical formulas show rates less than the estimated 485 cubic feet per second capacity available. Although there have been many instances of much higher run-off rates from a one square mile area, because the area is forested it is believed that the discharge capacity of Nichols Dam is sufficient if inflows from East Long Pond are small or non-existent during a heavy storm and "full" reservoir at Nichols Pond.

Condition "b" above.

When both reservoirs are in flood simultaneously the discharge capacity at Nichols Dam can be estimated as follows: add to the flow from the Nichols watershed the estimated inflow rate minus the effect of storage.

From the previous calculations of the flood rates, corrected for the new drainage area of 3 square miles, it is likely that a peak flow between 500 and 1000 cubic feet per second can enter the East Long Pond reservoir. Information is lacking on storage capacity, but it can be estimated that say 800 cubic feet per second could be discharged. Although complete information is lacking, it is known that the sluice capacity alone is about three times that at Nichols Dam, and assuming the spillway is of the same order of relative magnitude, it appears likely that East Long Pond
Nichols Pond Dam

**Sluice Capacity:** The approximate sluice capacity is
\[
Q = (5 \times 2)(8.02) \sqrt{\frac{(12 - 1) + 5}{1 + 0.5 + 0.02 \times \frac{4.0}{14}}} 
\]
\[
Q = 240 \text{ cubic feet per second}
\]

**Combined Capacity:** The combined capacity of sluice and spillway (at a head above the crest of 1.5 feet) is 240 + 245 = 485 cubic feet per second. This should be sufficient to handle the drainage from a severe storm on the one square mile watershed - assuming no inflow from the East Long Pond Dam.

**Inflow from East Long Pond Dam:** The required discharge capacity of Nichols Dam will be computed for two conditions which depend on the method of operation of the reservoir and the extent of the storm.

(a) no inflow from East Long Pond Dam, but "full" reservoir in Nichols Dam during a storm on the Nichols Dam Watershed only.

(b) inflow from East Long Pond Dam assuming both reservoirs "full" and a storm over both watersheds. This is of course, the most serious possibility.

**Required Capacity:**
Condition "a" above - Nichols Dam should have a combined spillway and sluice capacity within the following limits to handle safely runoff from it's own watershed only.

**Kuichling (for frequent floods)**
\[
Q = \sqrt{\frac{1,000}{1 + 370} + 20} \times 1.0 = 133 \text{ cubic feet per second}
\]
Nichols Pond Dam

The Embankment: - The embankment is well-sodded and quite stable. The downstream face is in good condition; no leakage was observed, although the pond level was low and leakage might occur through the disintegrated wall during high water.

Minor repairs are needed on a small section of the right bank beyond the head wall to fill and stabilize this section which has eroded out. The damage is not serious, but should be rectified when the head wall is repaired.

The Spillway: - The 6-inch thick spillway slab and the spillway guide walls are cracked in several places, probably due to settlement of the slab on the earth fill under the slabs. The bituminous filler previously used to attempt repairs is not effective, and leakage might occur through the cracks into the earth fill under the slab. The rock face which retains the earth fill on the downstream side of the spillway is in good condition.

The spillway slab and guide walls should be patched with new concrete to prevent leakage into the earth fill under the spillway slab when the spillway is in operation.

The Spillway Capacity: - The maximum safe capacity of the spillway is estimated to be:

\[ Q = 3.7 \times 36(1.5)^{3/2} = 245 \text{ cubic feet per second} \]
The non-overflow section of the Nichols Pond Dam is an earth embankment with these approximate dimensions: length, 160 feet, width 26 feet, and height 14 feet. A concrete head wall, 12 inches wide extends the length of the embankment on the upstream side. The spillway is nearly in the center of the embankment, being 36 feet long at the crest, and capable of a maximum safe head of 1.5 feet. A rectangular sluice, 5 feet x 2 feet, controlled by 2 hand operated gates, also passes water downstream along the longitudinal center-line of the spillway.

The Head Wall: - The 12-inch wide concrete head wall is badly spalled and disintegrated. A bituminous joint sealer applied years ago to seal the cracks in the wall is not now effective. It is expected that leakage would occur when the pond is high, although none was noticed during the inspection because the pond was several feet below the spillway crest, and most of the serious disintegration was at a higher elevation. The stability of the dam is not affected by the disintegrated wall as long as leakage does not occur.

The head wall should be resurfaced by chipping out all unsound sections and replacing with new concrete adequately tied into the sound portion of the existing wall. This would not be a major project because most of the disintegration is at a high elevation.
Mr. Oscar L. Shepard  
Chairman, State of Vermont  
Public Service Commission  
Montpelier, Vermont

Subject: Nichols Pond Dam

Dear Mr. Shepard:

Nichols Pond Dam was inspected on November 22, 1954, in accordance with the policy of the Vermont Public Service Commission to periodically check on the safety of the dams in the state. The inspection party consisted of Mr. Silas C. Carpenter, Engineer for the Public Service Commission, Mr. Larrabee, Superintendent for the Village of Hardwick, and the writer.

The owner and operator of the Nichols Pond Dam is the Village of Hardwick, Vermont. The dam is located near Woodbury, Vermont which is downstream from the dam. East Long Pond Dam and Reservoir are approximately one-half mile upstream from the Nichols Pond Dam.

Nichols Pond Dam forms a lake of approximately 125 acres, with a storage of approximately 54 million cubic feet. The drainage area is about 1 square mile. The East Long Pond overflow also discharges into the Nichols Pond. The drainage area above East Long Pond is approximately 3 square miles; the pond has an approximate area of 250 acres and storage of 43 million cubic feet.
Plan View

Sketch of

Nichols' Pond Dam

Drawn by SHH
6/9/49
Conclusions:

The writer concludes that this dam is in a satisfactory structural condition but lacks adequate spillway capacity. Keeping one or both ponds below spillway crest level provides a margin of safety against overtopping and probable destruction of the dam. If, at some future time, it becomes desirable to maintain a full pond level at both Nichols Pond and East Long Pond, then consideration should be given to enlarging the spillway capacity.

[Signature]

STEPHEN A. HAYBROOK
HYDRAULIC ENGINEER

Public Service Commission
Montpelier, Vermont
October 26, 1949

Report No. 79
through seepage. Although the wall appeared in a weakened condition, it was considered stable enough to retain the embankment.

On date of visit the pond level was drawn down to about 5 ft. below crest level. With this low water it was impossible to determine what seepage, if any, occurred through the dam.

The embankment was well consolidated and sufficiently contained between its outside walls. Its top was protected by a sod cover. Both sluice gates were in good working order. The outlet and overflow structures were in good condition.

From all appearances the dam was being provided with the usual maintenance.

Comments re Dam:

At this dam the spillway capacity is limited. With both Nichols Pond and East Long Pond full at a time of maximum flood inflow, the spillway could not handle, simultaneously, the runoff from its own drainage area and the overflow from East Long Pond.

According to the operator, both ponds are never full at the same time. Either one or the other is generally drawn down below crest-level. With this method of operation, the possibility of overtopping Nichols Pond dam is greatly reduced.

It will be noted that both ponds are located in an isolated, wooded section. Consequently, the possibility of flood damage is also reduced.
REPORT ON NICHOL'S POND DAM

Supplementing the storage of East Long Pond is Nichols Pond about 1/2 mile further downstream and on the same brook in the town of Woodbury, Vermont. This storage is used according to the needs of the owner's hydro-electric plant in the course of the stream. It is owned and operated by the Village of Hardwick.

The dam at the outlet creates a pond having a surface area of about 125 acres and a useable volume estimated at 54 million cubic feet. Besides the discharge from East Long Pond it receives the drainage from a catchment area of 1 square mile.

Description of Dam:

In general, the dam consists of an earth-fill contained between a concrete wall on the upstream side and a dry stone masonry wall on the downstream side. It is about 200 ft. in total length, 26 ft. in width and 14 ft. in height. A sketch of the dam is appended herein.

Discharge past the dam is provided by a rectangular concrete sluiceway, 5 ft. by 2 ft., and controlled by two hand operated wooden gates. Overflow is accommodated by a concrete-paved spillway trough 1.5 ft. below the top of the dam and located through its middle.

Notes from Inspection of June 9, 1949:

The concrete head wall raking up the upstream face of the dam showed a battered effect due to wave and ice action. A bituminous material has been applied to the cracks to control
MEMORANDUM

To: File
From: A. Peter Barranco, Jr., P.E., Dam Safety Engineer
Subject: Mackville Pond Dam - Hardwick (93-2)

On July 12, 1979 the writer inspected subject structure and obtained photographs and additional dimensions.

The dam is in fair-poor condition, particularly because of lack of maintenance and repairs. Brush and tree growth made access and inspection difficult, however, despite its rundown condition the dam appears to be stable. The concrete training wall on the right downstream channel wall is in good condition. Leakage was noted along part of downstream face, however, it is about what one would expect of a dam of this construction and condition. Spillways are somewhat irregular due to loss of concrete cap and type of construction. Mortar on left upstream face has deteriorated.

While at the site, the writer spoke with Mr. Carroll Rowell who has lived in the house at the right end of the dam since 1913. Mr. Rowell is familiar with the history of this dam and the ones on Nichols Pond and East Long Pond - all of which were built by the Woodbury Granite Company. Mackville Pond Dam was apparently built about 1900. During the 1927 flood, the dam and bridge were overtopped but held, however, the right side of the pond (apparently old ground) washed out and destroyed several homes. The washout left a very deep ravine next to Mr. Rowell's house but did not damage the house because the erosion on that side was halted by ledge. After the flood, the washed out area and road were filled, however, the fill would not hold and it was necessary to drive steel sheet piling from near the right abutment across the town road a distance of 200-300' to hold it. The houses destroyed in the flood were not rebuilt.

During the 1973 flood, according to high water marks pointed out by Mr. Rowell, the pond level rose to 3.5' above spillway crest which would mean that the "non-overflow" sections were overtopped by about a foot.
PLAN
SCALE 1"=20'

ELEVATION
SCALE 1"=20'

LEGEND
⊙ DIRECTION OF PHOTOGRAPH
⊙ POINT ELEVATION

NATIONAL DAM INSPECTION PROGRAM
NICHOLS POND DAM
WOODBURY, VERMONT

PLAN AND ELEVATION VIEW

292
APPENDIX C

PHOTOGRAPHS

FOR LOCATION OF PHOTOS, SEE FIGURE B-1
LOCATED IN APPENDIX B
1. Downstream face of left side of dam

2. Upstream view of dam from left
3. Upstream view of dam from right abutment

4. Close-up of upstream concrete wall.
5. Base of wall, left side of dam

6. Cavity to the left of spillway
7. Wall to right of spillway

8. Wall to left of spillway
9. Left wall showing efflorescence

10. Right bank, downstream of dam
11. Spillway looking downstream

12. Spalled area of spillway
13. Condition of spillway walls

14. Downstream face of spillway
15. View of dam from downstream

16. Gate operating mechanism
APPENDIX D

HYDROLOGIC AND HYDRAULIC CALCULATIONS
surcharge height 5 = surcharge height 4 = 5.3' x 5.3' (adj) values will not change, no further iterations necessary.

Since dam is overtopped, \( \frac{1}{2} \) PMF must be routed to determine spillway adequacy.

\[ Q_r = 4050 \text{ cfs} \]

\[ \text{surcharge height}_5 = 4.6' \text{ (el 1212.6')} \]

\[ \text{STORE}_1 = 4120 - 3251 = 869 \text{ a-f} \]

\[ \text{STORE}_2 = \frac{869 \text{ a-f} \times 12''}{3.44 \text{ ft}^2 \times 640 \text{ sec/m}^2} = 4.7366'' \]

\[ \text{STORE}_2 = Q_r (1 - \frac{\text{STORE}_1}{9.5''}) = 4050 \left(1 - \frac{4.7366}{9.5''}\right) = 2031 \text{ cfs} \]

\[ \text{surcharge height}_2 = 2.3' \text{ (el 1211.3')} \]

\[ \text{STORE}_3 = 3860 - 3251 = 609 \text{ a-f} \]

\[ \text{STORE}_4 = \frac{609 \times 12''}{3.44 \times 640} = 3.3174'' \]

\[ \text{STORE}_{41''} = \left(\frac{3.3174 + 4.7366}{2}\right) = 4.0280'' \]

\[ Q_{P_2} = 4050 \left(1 - \frac{4.0280}{9.5''}\right) = 2333 \text{ cfs} \]

\[ \text{surcharge height}_3 = 3.5' \text{ (el 1211.5')} \]

\[ \text{STORE}_3 = 3800 - 3251 = 649 \text{ a-f} \]

\[ \text{STORE}_4 = \frac{649 \times 12''}{3.44 \times 640} = 3.5374'' \]

\[ \text{storeave} = \left(\frac{3.5374 + 4.0280}{2}\right) = 3.7827'' \]

\[ Q_{P_3} = 4050 \left(1 - \frac{3.7827}{7.5''}\right) = 2437 \text{ cfs} \]

\[ \text{surcharge height}_4 = 3.65' \text{ (el 1211.65')} \]
**EFFECT OF SURCHARGE STORAGE ON PM F**

\[ Q_p = 8100 \text{ cfs} \quad \text{Height of surcharge} = 6.4' \quad \text{(0.1214) sap rating curve} \]

**SURCHARGE VOLUME = TOTAL VOLUME - NORMAL POOL VOLUME (from elevation volume curve)**

\[ \text{stor}_1 = 4480 - 3251 = 1229 \text{ a-f} \]

\[ \frac{\text{stor}_1}{3.44 \text{ mi}^2 \times 640 \text{ acre/mi}^2} = 6.6968' \]

\[ Q_p = Q_p \left(1 - \frac{\text{stor}_1}{19'}\right) = 8100 \left(1 - \frac{6.6968}{19}\right) = 5244 \text{ cfs} \]

**SURCHARGE HEIGHT \(_2\) = 5.18' (at 1213.18')**

\[ \text{stor}_2 = 4260 - 3251 = 1009 \text{ a-f} \]

\[ \text{stor}_2 \times 12'' \times 640 \text{ acre/mi}^2 = 5.4996' \]

\[ \text{stor}_{\text{ave}} = \frac{(5.4996 + 6.6968)}{2} = 6.0992' \]

\[ Q_p_2 = 8100 \left(1 - \frac{6.0992}{19}\right) = 5500 \text{ cfs} \]

**SURCHARGE HEIGHT \(_3\) = 5.3' (at 1213.3')**

\[ \text{stor}_3 = 4280 - 3251 = 1029 \text{ a-f} \]

\[ \text{stor}_3 \times 12'' \times 640 \text{ acre/mi}^2 = 5.6853' \]

\[ \text{stor}_{\text{ave}} = \frac{(5.6853 + 5.0992)}{2} = 5.3956' \]

\[ Q_p_3 = 8100 \left(1 - \frac{5.3956}{19}\right) = 5604 \text{ cfs} \]

**SURCHARGE HEIGHT \(_4\) = 5.33' (at 1213.33')**

\[ \text{stor}_4 = 4290 - 3251 = 1039 \text{ a-f} \]

\[ \text{stor}_4 \times 12'' \times 640 \text{ acre/mi}^2 = 5.6632' \]

\[ \text{stor}_{\text{ave}} = \frac{(5.6632 + 5.8539)}{2} = 5.7586' \]

\[ Q_p_4 = 8100 \left(1 - \frac{5.7586}{19}\right) = 5645 \text{ cfs} \]

**SURCHARGE HEIGHT \(_5\) = 5.34' (at 1213.34')**
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<th>(cf)</th>
<th>Head</th>
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<th>(cf)</th>
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<td>4.5</td>
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DAM CREST - ELEVATION 1211.5'
RIGHT EMBANKMENT

Majority of 25' foot right embankment is a hill which will not be overtopped (2.5' above left embankment). But small length of embankment before and after hill will reduce flow - length ≈ 20' - equation 1211.5

\[ Q = C L H^{3/2} \]
\[ Q = 2.6(20)(H^{3/2}) \]
\[ Q = 52 H^{3/2} \]

36'' Ø OUTLET PIPE - ASSUMED NON-EFFECTIVE IN FLOW COMPUTATIONS BECAUSE IT HAS A GATED OPENING. OPENING MECHANISM HAS BEEN REMOVED, AND IF REINSTALLED WOULD BE IN MIDDLE OF SPILLWAY, MAKING USE DURING FLOOD IMPOSSIBLE.
EMERGENCY SPILLWAY - CREST ELEVATION 1208'

\[ Q_{es} = C_w L H^{3/2} \]

\[ L = 80' \]

\[ C_w = 3.1 \text{ (conservative value chosen due to field conditions)} \]

\[ Q_{es} = 3.1 \times 80 H^{3/2} \]

\[ Q_{es} = 248 H^{3/2} \]

DAM CREST - ELEVATION 1210'

Left Embankment near auxiliary spillway

\[ Q = C_w L H^{3/2} \]

\[ L = 95' \]

\[ C_w = 2.6 \text{ (conservative value chosen due to field conditions)} \]

\[ Q = 2.6 \times 95 H^{3/2} \]

\[ Q = 247 H^{3/2} \]

DAM CREST - ELEVATION 1211.3'

Left Embankment near left abutment

\[ Q = C_w L H^{3/2} \]

\[ L = 80' \]

\[ C_w = 2.6 \text{ (conservative value chosen due to field conditions)} \]

\[ Q = 2.6 \times 80 H^{3/2} \]

\[ Q = 208 H^{3/2} \]
**Step 1**

**Calculation of Spillway Design Flood**

**Classification Size - Intermediate**

**Hazard - High**

**DAM SAFETY GUIDELINES RECOMMEND**

PMF

PMF = 2350 cfs/ mi²

PMF = 2350 \( \frac{cfs}{mi^2} \) \( \times \) 3.44 mi² = 8084 cfs \( \approx \) 8100 cfs

PMF = 8100 cfs

\( \frac{1}{2} \) PMF = 4050 cfs

**Step 2 (cont.)**

**Calculation of Surchage by Full PMF**

**Auxiliary Spillway - Crest Elevation 1208.8'**

\[ q_s = C_w H^{3/2} \]

\[ q_{ms} = 2.9 \times 12 \times H^{3/2} \]

\[ q_{ns} = 3.4 \times 8.8 \times H^{3/2} \]

**L = 12**

\[ C_w = 2.9 \text{ (Conservative value chosen due to field debris could obstruct weir)} \]
Project: Nichols Pond Dam
Subject: Hydraulics / Hydroslogy

**Step 1:** Classification of Spillway Design Flood

Classification: Size - Intermediate
Hazard - High

DAM SAFETY GUIDELINES RECOMMEND FULL PMF

PMF not found on PMF curve envelope, due to drainage area being less than 2 mi². New England Division, Corps of Engineers consulted on November 27, 1979. Engineer Gary James recommended using same PMF / mi² as 1761 Long Pond Dam (1750' upriver) and adding the outflow of East Long Pond reservoir to get the input PMF for Nichols Pond reservoir.

Basin - Mountainous

PMF for East Long Pond: 2350 cfs / mi²

\[ \text{DA for Nichols Pond} \]

\[ \text{PMF} = \frac{2350 \times 1}{\text{mi}^2} \times 1.111 = 2608.5 \text{ cfs} \]

Outflow from East Long Pond (PMF)

\[ \text{PMF} = 2608.5 + 5645 \text{ cfs} = 8253.5 \approx 8300 \text{ cfs} \]

Calculations shown on pages 7-14.

PMF = 8300 cfs - outflow from East Long Pond (mostly by PMF)

\[ \frac{1}{2} \text{PMF} = \frac{2608.5}{2} + \frac{5645}{2} = 3757.3 \text{ cfs} \approx 3800 \text{ cfs} \]

**Step 2:** Calculation of Discharge by PMF

\[ Q = C_w L H^{3/2} \]

\[ Q = 200 \cdot (2.63) H^{3/2} \]

\[ Q = 526 \cdot H^{3/2} \]

Broad crested weir

C\(_w\) = 2.63 from Kingard Brater
Page 5-46, Table E.
NICHOLS POND DAM - located in Woodbury, VT

CLASSIFICATION: SIZE - INTERMEDIATE (based on storage)
HAZARD - HIGH (based upon 100 year flood of downstream homes)

BASIC DATA:
DRAINAGE AREA: INDEPENDEANT 1.11 mi²
TOTAL 4.55 mi²
RESERVOIR: NORMAL POOL ELEVATION 1128' (USGS)
STORAGE 2590.4 a-f
MAXIMUM POOL ELEVATION 1130.5'
STORAGE 2840.9 a-f
SURFACE AREA
161.9 acres (NORMAL POOL)
167 acres (MAXIMUM POOL)

DAM: EARTH FILL WITH CONCRETE UPSTREAM WALL AND STONE MASONARY WALL DOWNSTREAM - BOTH WALLS VERTICAL
MAXIMUM HEIGHT - 16'
LENGTH - 200'

SPILLWAY: PRIMARY - TRIANGULAR SHAPED WEIR, TAPERS FROM 27' TO 7.5'
EMERGENCY - NONE

OUTLET: 5'x2' SLUICEWAY
INVERT 1117' AS PER STATE OF VERMONT SURVEY
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<th>(\frac{A_1 + A_2}{2}) (acres)</th>
<th>Height (ft)</th>
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<td>Subject: Nichols Pond Dam</td>
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**Normal Pool Surface** (Elev. 1128') USGS

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<th>Reading 3</th>
</tr>
</thead>
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<td>0.27</td>
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<td>0.78/3 = 0.26</td>
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</table>

Average = 0.78/3 = 0.26

Area: 0.26 x 0.973 = 0.25 mi² x 640 = 161.9 acres

Minimum pool to be obtained via graph on page 2, giving pool area at next contour level (1120')

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<th>Reading 3</th>
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<td>0.61/2 = 0.32</td>
<td>0.95/3 = 0.32</td>
</tr>
</tbody>
</table>

Average = 0.95/3 = 0.32

Area: 0.923 x 0.32 = 0.31 mi² = 197 acres

Normal Pool Storage (1128') (vertical walls assumed)

Height to spillway = 16'

16' x 161.7 = 2588.8 cubic ft

Surge storage (1129.5')

1.5' x 167 = 250.5 cubic ft

Minimum Real Storage

Sum of surfacce and normal storage

2588.8 + 250.5 = 2839.3 acre-ft
Job No.: 91110  
Project: East Long Pond  
Subject: Hydraulics

\[ \text{STOR}_4 = 3930 - 3251 = 679 \text{ a.-f.} \]
\[ \text{STOR}_4 = \frac{679}{12} \times 640 = 3700.9''\]
\[ \text{STOR}_{ave} = \frac{3.7009 + 3.7827}{2} = 3.7418'' \]

\[ Q_{p5} = 4050 \left(1 - \frac{3.7418}{9.5}\right) = 2455 \text{ cfs} \]

\[ \text{auxiliary height}_b = 3.65' \ (3611.69) \]

\[ \text{auxiliary height}_b = \text{auxiliary height}_s = 3.65' = 3.7' \ (3611.7') \]

**Conclusions**

1) Reservoir storage will reduce the full PMF test inflow to an outflow of 5645 cfs (30% reduction). The 1/2 PMF test inflow will be reduced, due to reservoir storage, to 2455 cfs (39% reduction).

2) The spillways can only pass 747 cfs before the dam is overtopped (13% of test outflow of 5645 cfs; 30% of test outflow of 2455 cfs).

3) The PMF will cause a dam overtopping of 3.3' (3612.3'). 1/2 PMF causes the dam to be overtopped by 1.7' (3612.7').
OUTLET ASSUMED NON EFFECTIVE IN FLOW COMPUTATIONS
BECAUSE OUTLET IS GATED. GATE OPENING MECHANISM
HAS BEEN REMOVED. ALSO, THE OPERATOR WOULD HAVE TO
STAND IN THE MIDDLE OF THE SPILLWAY TO OPERATE
THE GATES, MAKING ITS USE UNLIKELY DURING A
HURRICANE.

SPILLWAY

IRREGULAR SPILLWAY REQUIRES SPECIAL
COMPUTATIONS TO DETERMINE IF INLET OR OUTLET
CONTROLS FLOW. A RATING CURVE FOR THE
SPILLWAY (SHOWN ON PAGE 6.514) INDICATES A CAPACITY OF
2350 F³/S AT DAM CREST (ELEVATION 1130.5') CONSEQUENTLY
WHEN THE ENTIRE DAM IS OVER-TOPPED (ELEV. 1130.5'), THE
SPILLWAY WILL BECOME INSIGNIFICANT IN FLOW
CALCULATIONS, THE WEIR-LIKE FLOW OVER THE DAM
CRUSH WILL DOMINATE.
1. Rating curve for downstream end
   a. Sketch:

   ![Sketch](image)

2. Final discharge for various depths at Critical Depth
   Per King & Varner p 8-8 (Formulas 8-29)

   \[ Q = \sqrt{g} \cdot D_e^{3/2} \cdot \left( \sqrt{g} = 5.17; \ b = 7.25 \right) \]

   \[ Q = 41.1 \cdot D_e^{3/2} \]

   \[ EGL = Z_o + d + H_a \]

   where \( Z_o = 1126.0 \); \( \frac{H_a}{2} = \frac{D_e}{2} \)

<table>
<thead>
<tr>
<th>d (ft)</th>
<th>( D_e^{3/2} )</th>
<th>Q</th>
<th>( H_a )</th>
<th>EGL</th>
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<tbody>
<tr>
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<td>22.6</td>
<td>923</td>
<td>4.0</td>
<td>1138.0</td>
</tr>
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</table>
2. Rating curve for upstream end of spillway

a. Sketch

Using same formula \( Q = \sqrt{g} \cdot L \cdot D_a^{3/2} \)
we will combine \( b_1 \) and \( b_2 \) and investigate

\( b_1 = 6, \ b_2 = 28' \)

b. Determine two rating curves and combine

\[ Q = 2.2' \cdot Q = 158.8 \cdot D_a^{3/2} \]

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<th>( d_1 )</th>
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<th>( H_v )</th>
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<tbody>
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<tr>
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<td>159</td>
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<tr>
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<td>3.02</td>
<td>449</td>
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<td>3.95</td>
<td>628</td>
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</table>

\[ z = 112.8, \ q = 9.0 = \sqrt{61/5} \]

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<thead>
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<th>( d_1 )</th>
<th>( D_a^{3/2} )</th>
<th>( Q )</th>
<th>( H_v )</th>
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<td>1.5</td>
<td>1132.0</td>
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</table>
### Table: Primary Spillway and Dam Crest Flow

<table>
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<th>Water Surface Elevation (ft)</th>
<th>Primary Spillway</th>
<th>Dam Crest (e1130.5)</th>
<th>Total Flow (cfs)</th>
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<td>Head (ft)</td>
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<tr>
<td>1113</td>
<td>-</td>
<td>-</td>
<td>8.5</td>
</tr>
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</table>

*Damage on spillway not included since e1132 because configuration of dam is assumed to control.*
STEP 3  EFFECT OF SURCHARGE STORAGE ON PMF

\[ G_p = 8300 \text{ cfs} \]

\[ \text{HEIGHT OF SURCHARGE} = 8.8' \quad (\text{at 1136.8') } \]

\[ \text{SURCHARGE VOLUME} = \text{TOTAL VOLUME} - \text{NORMAL POOL VOLUME} \] (see elevation volume curve p. 4)

\[ \text{STOR}_1 = 6175 - 2590.4 = 1584.6 \text{ a-f} \]

\[ \text{STOR}_2 = \frac{1584.6 \text{ a-f} \times 12''}{4.55 \text{ ft}^2 \times 640 \text{ a-ft} / \text{ft}^2} = 6.5299'' \]

\[ G_p = G_p \left(1 - \frac{\text{STOR}_1}{19''}\right) = 8300 \left(1 - \frac{6.5299}{19''}\right) = 544.7 \text{ cfs} \]

\[ \text{SURCHARGE HEIGHT}_2 = 7.3' \quad (\text{at 1135.3') } \]

\[ \text{STOR}_2 = 3850 - 2590.4 = 1257.6 \text{ a-f} \]

\[ \text{STOR}_3 = \frac{1257.6 \times 12}{4.55 \times 640} = 5.1907'' \]

\[ \text{STOR}_{ave} = \left(5.1907 + 6.5299\right)/2 = 5.8603'' \]

\[ G_p = 8300 \left(1 - \frac{5.8603}{19''}\right) = 5740 \text{ cfs} \]

\[ \text{SURCHARGE HEIGHT}_3 = 7.45' \quad (\text{at 1135.45') } \]

\[ \text{STOR}_3 = 2700 - 2590.4 = 1399.6 \text{ a-f} \]

\[ \text{STOR}_4 = \frac{1399.6 \times 12}{4.55 \times 640} = 5.3967'' \]

\[ \text{STOR}_{ave} = \left(5.3967 + 5.8603\right)/2 = 5.6285'' \]

\[ G_p = 8300 \left(1 - \frac{5.6285}{19''}\right) = 5841 \text{ cfs} \]

\[ \text{SURCHARGE HEIGHT}_4 = 7.52' \quad (\text{at 1135.62') } \]

\[ \text{STOR}_4 = 3926 - 2590.4 = 1334.6 \text{ a-f} \]

\[ \text{STOR}_4 = \frac{1334.6 \times 12}{4.55 \times 640} = 5.4997'' \]
Job No. 91118
Project
Nicola Island Dam
Subject

Sheet 22 of 44
Date 4/10/80
By En Chk. by 

\[ Q_{R1} = 2300 \left( 1 - \frac{5.5641}{19} \right) = 5869 \text{ cfs} \]

\[ \text{Surcharge height} = 7.52 \text{ ft} \] (sl. 1135.52)

\[ \text{Surcharge height} = \text{Surcharge height} (7.52' = 7.5') \]

\[ Q_{R1} = 3800 \text{ cfs} \]

\[ \text{Surcharge height} = 6.4' \] (sl. 1134.4')

\[ \text{Surcharge Volume} = \text{Total Volume} - \text{Normal pool volume} \]

\[ \text{Surge} = 3750 - 2570 = 1180 \text{ ft} \]

\[ \text{Surcharge Volume} = \frac{1180 \text{ ft} \times 12}{4.55 \text{ ft} \times 640 \text{ ft}^2/\text{acre}} = 4.7802 \text{ acre ft} \]

\[ Q_{R1} = 2300 \left( 1 - \frac{4.7802}{9.5} \right) = 1288 \text{ cfs} \]

\[ \text{Surcharge height} = 4.8' \] (sl. 1132.8')

\[ \text{Surge} = 2300 - 2600 = 300 \text{ cfs} \]

\[ \text{Surcharge} = \frac{300 \times 12}{4.55 \times 640} = 2.2379 \text{ ft} \]

\[ \text{Surcharge} = \left( 2.3379 + 4.7802 \right) / 2 = 4.0591 \text{ ft} \]

\[ Q_{R1} = 2300 \left( 1 - \frac{4.0591}{7.5} \right) = 2176 \text{ cfs} \]

\[ \text{Surcharge height} = 5.05' \] (sl. 1133.05')

\[ \text{Surge} = 3425 - 2670 = 755 \text{ cfs} \]

\[ \text{Surcharge} = \frac{755 \times 12}{4.55 \times 640} = 3.4407 \text{ ft} \]

\[ \text{Surcharge} = \left( 3.4407 + 4.0591 \right) / 2 = 3.7500 \text{ ft} \]

\[ Q_{R1} = 2300 \left( 1 - \frac{3.7500}{9.5} \right) = 2300 \text{ cfs} \]
Job No. [Blank]  
Project [Blank]  
Subject Hydrology  
Date 4/18/70  
By [Blank]  

Sheet 22 of 44

**Calculation**

Surchage height = 5.15' (el 1133.15')

\[ s_4 = 1133.00 - 1127.85 = 5.15' \]

\[ s_4 = \frac{850 \times 12}{4.55 \times 6.4} = 3.544' \]

\[ s = \frac{(3.544 + 3.750)}{2} = 3.647' \]

\[ Q = 2300 \left(1 - \frac{3.647}{7.5}\right) = 2341 \text{ cfs} \]

Surchage height = 5.18' (el 1133.18') < 1133.2'

Surchage height c = 5.18' (el 1133.2')

**Conclusion**

Reservoir storage will reduce the full PMF test intake to an outflow of 5869 cfs (29% reduction).

The full test intake will be reduced due to reservoir storage, to 2341 cfs (38% reduction).

The spillway can only pass 218 cfs before the test is overtopped. (4% of test discharge of 5869 cfs; 7% of test discharge of 2341 cfs)

The PMF will cause a dam overtopping of 5.0' (el. 1135.5'). \( \frac{1}{2} \) PMF causes the dam to be overtopped by 2.7' (el. 1133.2').
Corps of Engineers recommends this procedure - Do breach analysis w/ water at top of dam (full spillway capacity). Check to see if one or more homes will be affected. If so, use this case. If not, try analysis w/ water at top of spillway (negligible downstream flow). Using this or of analysis, a case will be found which will cause damage or loss of life (the object of the analysis).

Case 1. Water at top of dam (A: 1130.5)

\[ h = \frac{2}{3} \left( \frac{V^2}{g} \right)^{3/2} \]

\[ y_0 = \frac{1}{3} (0.40)(175) \sqrt{32} \]

\[ V^{3/2} = 10,500 \text{ cfs} \]

- breach width
- height of water above dam

Initial downstream discharge = 218 ft³/s, Stage = 1.8'

Total flow after breach = 10,500 + 218 = 10,718 ft³/s, Stage = 13.9'

Front wave = 4.8 ft Stage = 13.9 - 1.8 = 12.1'

A 12.1' wave will cause damage to the dam, hence use 4.

**STEP 1** Reservoir Storage

\[ @ A: 1130.5' \]

\[ 2841 \text{ a-ft} \]

**STEP 2** Peak failure outflow

\[ Q_p = 10500 \text{ cfs} \]

**STEP 3** Chezy Discharge Routing Curve
<table>
<thead>
<tr>
<th>CHANNEL</th>
<th>RIGHT OVERBANK</th>
<th>LEFT OVERBANK</th>
<th>TOTAL AREA</th>
<th>TOTAL FLOW</th>
</tr>
</thead>
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<td>Flow</td>
<td>Type</td>
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<td>5</td>
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<td>23.25</td>
<td>348</td>
<td>29</td>
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</table>

**Equations:**

\[ L = 10,503 \]

\[ S = \frac{1123 - 723}{5} = 0.017 \text{ in} \]

\[ n = 0.05 \]

\[ n = 0.08 \]
9/1/6

HYDRAULICS / HYDROLOGY

SPILLWAY ELEVATION 925.0'

\[ Q = C_u L H^{3/2} \]
\[ Q = 3.1(3.9) \times 3/2 \]
\[ Q = 120.9 \times 3/2 \]

L = 23 + 16 = 39'

C_u = 3.1 (based upon field conditions)

DAM CREST ELEVATION 927.0'

\[ Q = C_u L H^{3/2} \]
\[ Q = 3.0(41.5) \times 3/2 \]
\[ Q = 124.5 \times 3/2 \]

L = 80.5 - 39 = 41.5'

C_u = 3.0 (based upon field conditions)

DIKE ELEVATION 929.3'

THE ROADWAY TO THE LEFT OF THE DAM HAS HAD FLOOD WATERS USE IT AS AN EMERGENCY SPILLWAY. IT WILL BE CONSIDERED AS A WEIR WITH A LENGTH OF 5', VERTICAL WALLS ARE ASSUMED TO STAY CONSERVATIVE.

\[ Q = C_u L H^{3/2} \]
\[ Q = 2.6(75) \times 3/2 \]
\[ Q = 195 \times 3/2 \]

L = 75'

C_u = 2.6 (based upon field conditions)

OUTLET ASSUMED NON EFFECTIVE IN FLOW COMPUTATIONS. WASTE GATE IS INOPERABLE AND THE PENSTOCK GATE OPENING MECHANISM IS LOCATED IN THE MIDDLE OF THE SPILLWAY, MAKING ITS USE UNLIKELY DURING A FLOOD.
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<tr>
<th>Elevation (ft)</th>
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<th></th>
<th>H</th>
<th>Q (cfs)</th>
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</tbody>
</table>
**STEP 3**

**STAGE-DISCHARGE ROUTING CURVE**

**DOWNSTREAM XS APPROXIMATED FROM**

**X SECTIONAL DATA SURVEYED BY DUBOIS AND KING PERSONNEL**

**RELATING TO 1'000 INSURANCE STUDY FOR TOWN OF HARDWICK VT**

**REACH 1**

\[ L = 1500' \]
\[ A_{elev} = 23' \]
\[ S = \frac{23}{1500} = 0.0153 \]

\[ n = 0.08 \]

\[ S = \frac{6.7}{500} = 0.0134 \]
### Mannings Equation Used

\[ Q = \frac{1.49}{n} A^{1/3} S^{1/2} \]

\[ R = \frac{A}{P} \]

<table>
<thead>
<tr>
<th>Stage</th>
<th>Area</th>
<th>Wetted Perimeter</th>
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<th>Right</th>
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</tr>
<tr>
<td>2</td>
<td>26</td>
<td>17.2</td>
<td>158</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>26</td>
<td>158</td>
</tr>
<tr>
<td>4</td>
<td>58</td>
<td>17.2</td>
<td>600</td>
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<td>85</td>
<td>24</td>
<td>12.6</td>
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<td>8</td>
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<td>17.2</td>
<td>2072</td>
<td>54</td>
<td>19</td>
<td>249</td>
<td>54</td>
<td>19</td>
<td>230</td>
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<td>10</td>
<td>154</td>
<td>17.2</td>
<td>3054</td>
<td>96</td>
<td>25.3</td>
<td>537</td>
<td>96</td>
<td>25.3</td>
<td>346</td>
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<tr>
<td>12</td>
<td>186</td>
<td>17.2</td>
<td>4125</td>
<td>150</td>
<td>31.6</td>
<td>966</td>
<td>150</td>
<td>31.6</td>
<td>486</td>
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<td>15</td>
<td>234</td>
<td>17.2</td>
<td>6338</td>
<td>253.5</td>
<td>41.1</td>
<td>1940</td>
<td>253.5</td>
<td>41.1</td>
<td>741</td>
</tr>
<tr>
<td>18</td>
<td>282</td>
<td>17.2</td>
<td>8231</td>
<td>384</td>
<td>50.6</td>
<td>3370</td>
<td>384</td>
<td>50.6</td>
<td>1050</td>
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<td>21</td>
<td>330</td>
<td>17.2</td>
<td>10685</td>
<td>541.5</td>
<td>60.1</td>
<td>5323</td>
<td>541.5</td>
<td>60.1</td>
<td>1413</td>
</tr>
</tbody>
</table>

---

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Project: Mackville Dam  
Subject: Hydraulics  

Reach Z

Reach Length = 500'

\[ \Delta e_y = 887 \cdot 820 = 67' \]

\[ S = \frac{\Delta e_y}{L} = \frac{67}{500} = 0.134 \]

\[ b = 20' \]

2) Normal depth found via Manning's equation
\[ Q = \frac{1.49}{n} A R^{\frac{3}{2}} S^{\frac{1}{2}} \]

3) Critical depth from table 8-4 in King and Brater, p. 8-53

* Refer to Hydraulic and Excavation table USBR

Assume critical depth at throat

<table>
<thead>
<tr>
<th>Stage</th>
<th>Area (ft²)</th>
<th>Radius (ft)</th>
<th>( R^{\frac{3}{2}} )</th>
<th>Normal Flow (cfs)</th>
<th>Critical Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>44</td>
<td>1.71</td>
<td>1.43</td>
<td>4.29</td>
<td>3.38</td>
</tr>
<tr>
<td>4</td>
<td>96</td>
<td>3.07</td>
<td>2.11</td>
<td>13.77</td>
<td>10.08</td>
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<td>6</td>
<td>156</td>
<td>4.22</td>
<td>2.61</td>
<td>27.70</td>
<td>19.53</td>
</tr>
<tr>
<td>8</td>
<td>224</td>
<td>5.25</td>
<td>3.02</td>
<td>46.03</td>
<td>31.68</td>
</tr>
<tr>
<td>10</td>
<td>300</td>
<td>6.21</td>
<td>3.34</td>
<td>68.09</td>
<td>46.58</td>
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<tr>
<td>12</td>
<td>384</td>
<td>7.12</td>
<td>3.65</td>
<td>95.38</td>
<td>64.35</td>
</tr>
<tr>
<td>14</td>
<td>476</td>
<td>7.99</td>
<td>3.94</td>
<td>12.758</td>
<td>8.506</td>
</tr>
<tr>
<td>16</td>
<td>576</td>
<td>8.83</td>
<td>4.21</td>
<td>16.492</td>
<td>10.875</td>
</tr>
</tbody>
</table>
AT CONFLUENCE W/ COOPER BROOK, FLOOD WAVE WILL MEET A LARGE OPEN AREA, WHICH ACTS AS A RESERVOIR. ELEVATION- STORAGE CURVE WILL BE DERIVED. EFFECTS ON FLOOD WAVE WILL BE DETERMINED USING OUTLET CHANNEL AS A CONTROL.

MAP SCALE 1" = 400'

CONVERSION FACTOR FOR PLANIMETER

\[ 1' = (400)^2 \times \frac{1 \text{ acre}}{43,560 \text{ ft}^2} = 3.673 \text{ acre} \]

\[ 1' = 3.673 \text{ acre} \]

<table>
<thead>
<tr>
<th>PLANIMETER</th>
<th>READING</th>
<th>AVERAGE</th>
<th>AREA</th>
<th>SLICE</th>
<th>INCREMENTAL</th>
<th>TOTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>MEAS.</td>
<td>#1</td>
<td>#2</td>
<td>#3</td>
<td>AREA</td>
<td>HEIGHT</td>
<td>VOLUME</td>
</tr>
<tr>
<td>313</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0</td>
</tr>
<tr>
<td>316</td>
<td>4.27</td>
<td>4.29</td>
<td>4.31</td>
<td>4.29</td>
<td>7.88</td>
<td>3</td>
</tr>
<tr>
<td>320</td>
<td>5.32</td>
<td>6.30</td>
<td>6.30</td>
<td>8.31</td>
<td>23.14</td>
<td>4</td>
</tr>
<tr>
<td>324</td>
<td>12.50</td>
<td>12.40</td>
<td>12.40</td>
<td>12.43</td>
<td>38.09</td>
<td>4</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>


**REACH CHARACTERISTICS**

\[ L = 2600' \]
\[ \Delta \text{Up} = 814 - 806 = 8' \]
\[ S = \frac{\Delta \text{Up}}{L} = \frac{8}{2600} = 0.0031 \]

**RANNINGS EQUATION USED**

\[ Q = \frac{1.49}{n} \left( \frac{R}{2}\right)^{3/2}, \quad p = \frac{A}{P} \]

\[ n = 0.05 \quad \text{Rock channel, brush overbanks} \]

<table>
<thead>
<tr>
<th>Stage</th>
<th>Area</th>
<th>Perimeter</th>
<th>( R )</th>
<th>( R^{3/2} )</th>
<th>( Q )</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>410</td>
<td>210.8</td>
<td>1.945</td>
<td>1.559</td>
<td>1061</td>
</tr>
<tr>
<td>4</td>
<td>840</td>
<td>221.5</td>
<td>3.792</td>
<td>2.433</td>
<td>3391</td>
</tr>
<tr>
<td>6</td>
<td>1290</td>
<td>232.3</td>
<td>5.953</td>
<td>3.138</td>
<td>6716</td>
</tr>
<tr>
<td>8</td>
<td>1750</td>
<td>243.1</td>
<td>7.199</td>
<td>3.731</td>
<td>10,833</td>
</tr>
<tr>
<td>10</td>
<td>2250</td>
<td>253.9</td>
<td>8.862</td>
<td>4.285</td>
<td>15,977</td>
</tr>
<tr>
<td>12</td>
<td>2760</td>
<td>264.6</td>
<td>10.130</td>
<td>4.777</td>
<td>21,876</td>
</tr>
</tbody>
</table>

\( \Delta \text{Up} \) approximated from topography.

Channel itself neglected due to extreme overbank widths.
STEP 4

\[ q_1 = 10,250 \text{ cfs} \]

**ENTER NICHOLS POND REACH 1 (refer to p. 23-25)**

\[ V_1 = \frac{10,500 \times 775}{13560 \div 2} = 186.80 \text{ ft} \leq \frac{2841}{\pi} \text{ ft.} \]

\[ L_1 = 10,500 \text{ ft} \]

\[ q_{L1,0} = 10,250 \left(1 - \frac{186.8}{2841}\right) = 9576 \text{ cfs} \]

\[ C_{1,0} = 12.3' \quad \text{area} = 735' \]

\[ V_2 = \frac{735' \times 10,500}{13560} = 177.2 \text{ ft} \]

\[ V_{ave} = \frac{(177.2 + 186.8)}{2} = 182.0 \text{ ft} \]

\[ q_{P_2} = 10,250 \left(1 - \frac{182}{2841}\right) = 9593 \text{ cfs} \times 9600 \text{ cfs} \]

**OUTFLOW** = 7600 cfs 

**Stage** = 13.3'

---

**ENTER MACKVILLE POND (assumed filler to elevation 927.0')**

**INVESTIGATE SORCHARGE STORAGE EFFECTS ON FLOW**

\[ q_{P_1} = 7600 \text{ cfs} \quad \text{HEIGHT}_I = 10.2' (2193.2) \text{ refer to Mackville Rating Curve, page 28} \]

\[ V_1 = \text{SURCHARGE STORAGE} = 377.5 - 182 = 195.5 \text{ ft} \]

\[ q_{P_2} = q_{P_1} \left(1 - \frac{V_1}{2841}\right) = 7600 \left(1 - \frac{377.5}{2841}\right) = 6737 \text{ cfs} \]

**SURCHARGE HEIGHT** = 7.8' (elev. 934.8')

\[ V_2 = 377.5 - 182 = 195.5 \text{ ft} \]
\[ Q = 7600 \left( 1 - \frac{2841}{2841} \right) = 8965 \text{ cfs} \]

**Surcharge Height** = 9.8' (at 934.8')

**Surcharge Height** = **Surcharge Height** = 9.8', no further iterations necessary (note: flow divided immediately downstream)

**ENTER REACH 1 - MACKVILLE DAM (Refer P. 30-32)**

\[ Q_{1} = 8965 \text{ cfs} \quad \text{stage} = 14.4' \quad \text{area} = 680 \text{ ac} \]

\[ V_1 = \frac{680 \text{ ac} \times 1500'}{43560 \text{ ft}^2/\text{ac} \cdot 2' \text{ = 23.4 ac-ft} \leq \frac{2841 \text{ ac-ft}}{2' \text{ = length, ok}} \]

\[ Q_{\text{trial}} = Q_{1} \left( 1 - \frac{V_1}{2841} \right) = 8965 \left( 1 - \frac{23.4}{2841} \right) = 8891 \text{ cfs} \]

\[ V_1 = \frac{680 \text{ ac} \times 1500'}{43560} = 23.4 \text{ ac-ft} = \text{Vmax} \]

\[ Q_{2} = \frac{8965 \left( 1 - \frac{23.4}{2841} \right)}{2} = 8891 \text{ cfs} \]

Outflow = 8891 cfs

**Stage = 14.4'**

**ENTER REACH 2 - MACKVILLE DAM (Refer P. 33-34)**

\[ Q_{2} = 8891 \text{ cfs} \quad \text{stage} = 14.4' \quad \text{area} = 4780 \text{ ac} \]

Note: flow is critical at throat.

\[ V_1 = \frac{500' \times 4950'}{43560} = 5.7 \text{ ac-ft} \leq \frac{2841 \text{ ac-ft}}{2' \text{ = length, ok}} \]

\[ L_2 = 500' \quad Q_{\text{trial}} = Q_{2} \left( 1 - \frac{V_1}{2841} \right) = 8891 \left( 1 - \frac{5.7}{2841} \right) = 8873 \text{ cfs} \]
Subject: Channel Routing

\[ V_2 = \frac{376^2 \times 500}{43560 \text{ ft}^2/\text{acre}} = 4.3 \text{ a-f} \]

\[ V_{ave} = \left( 5.7 + 4.3 \right) / 2 = 5 \text{ a-f} \]

\[ Q_p = 8875 \left( 1 - \frac{5.0}{2841} \right) = 88.75 \text{ cfs} \]

\[ d_{ave} = 11.6' \quad \text{OUTFLOW} = 88.75 \text{ cfs} \]

**ENTER CONFLUENCE NICHOLS-COOPERS BROOK (FOR A 35-36)**

1. \( Q_p = 8875 \text{ cfs} \)
2. Rating curve for avil channel controls, from stage-discharge curve
   \( d_1 = 7.1' \)
3. Elevation of valley floor = 814.0 (at avil channel)
4. Elevation of water surface = 814.0 + 7.1 = 821.1'
5. Elevation - elevation curve, \( V_{storage} = 152 \text{ a-f} \leq 2841 \text{ a-f} \): OK

\[ Q_p = 8875 \left( 1 - \frac{152}{2841} \right) = 8400 \text{ cfs} \]

\[ d_2 = 6.7' (at 820.9) \quad V_2 = 144.5 \text{ a-f} \]

\[ V_{ave} = \left( 152 + 144.5 \right) / 2 = 148.25 \text{ a-f} \]

\[ Q_p = 8875 \left( 1 - \frac{148.25}{2841} \right) = 6412 \text{ cfs} \]

\[ d_3 = 6.9' (at 820.9) \quad V_2 = 144.5 \text{ a-f} \]

\[ V_{ave} = \left( 148.25 + 144.5 \right) / 2 = 146.38 \text{ a-f} \]
\[ Q_3 = 8875 \left( 1 - \frac{1463}{2841} \right) = 8418 \text{ cfs} \]

\[ d_4 = 6.7' \text{ (e' 8.20.9')} \]

No further iterations, values will not change significantly.

Outflow = 8418 cfs  Stage (Exit) = 6.9'

---

ENTER MACKVILLE DAM REACH 3 (Refer p37-38)

Inflow = 8418 cfs  Stage = 6.9'  o.m. = 1450

\[ V_1 = \frac{2600' \times 14800'}{43560 \text{ acre}} = 88.3 \text{ a.-f} \leq \frac{2841 \text{ a.-f}}{2} \]

\[ L_3 = 2600' \]

\[ Q_{\text{trial}} = 8418 \left( 1 - \frac{883}{2841} \right) = 8156 \text{ cfs} \]

\[ d_4 = 6.8' \text{  area = } 1460 \text{ a.'} \]

\[ V_2 = \frac{2600' \times 1460}{43560 \text{ acre}} = 87.1 \text{ a.-f} \]

\[ V_{\text{ave}} = \frac{(87.1 + 88.3)}{2} = 87.7 \text{ a.-f} \]

\[ Q_{p_2} = 8418 \left( 1 - \frac{87.7}{2841} \right) = 8158 \text{ cfs} \]

Outflow = 8158 cfs  Stage = 6.8'

ENTER OUTSKIRTS VILLAGE OF HARDWICK
**Flood Routing Summary**

<table>
<thead>
<tr>
<th>Reach</th>
<th>Discharge (cfs)</th>
<th>Stage (ft)</th>
<th>Wave (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>At Nichols Pond Dam</td>
<td>10,250</td>
<td>13.7</td>
<td>11.9</td>
</tr>
<tr>
<td>At Confluence with Mackville Pond (10,500' downstream of Nichols Dam)</td>
<td>9,600</td>
<td>13.3</td>
<td>11.4</td>
</tr>
<tr>
<td>At Mackville Dam (13,000' downstream of Nichols Dam)</td>
<td>8,965</td>
<td>14.4</td>
<td>14.4</td>
</tr>
<tr>
<td>Downstream of Mackville Dam</td>
<td>8,965</td>
<td>14.4</td>
<td>14.4</td>
</tr>
<tr>
<td>1,500' downstream of Mackville Dam (14,500' DS of Nichols Dam)</td>
<td>8,891</td>
<td>14.4</td>
<td>14.4</td>
</tr>
<tr>
<td>2,000' downstream of Mackville Dam (15,000' DS of Nichols Dam)</td>
<td>8,875</td>
<td>11.6</td>
<td>11.6</td>
</tr>
<tr>
<td>3,800' downstream of Mackville Dam (16,800' DS of Nichols Dam)</td>
<td>8,418</td>
<td>6.7</td>
<td>6.9</td>
</tr>
<tr>
<td>After large open area</td>
<td>8,418</td>
<td>6.7</td>
<td>6.9</td>
</tr>
<tr>
<td>6,400' downstream of Mackville Dam (19,400' downstream of Nichols Dam)</td>
<td>8,158</td>
<td>6.8</td>
<td>6.8</td>
</tr>
</tbody>
</table>

Outskirts of Village of Hardwick
NOT TO SCALE

TEST INFLOW = 8300 cfs

<table>
<thead>
<tr>
<th>CONDITION AT DAM</th>
<th>WATER SURFACE ELEVATION</th>
<th>TOTAL DISCHARGE (cfs)</th>
<th>PRIMARY SPILLWAY DISCHARGE (cfs)</th>
<th>EMERGENCY SPILLWAY DISCHARGE (cfs)</th>
<th>% OF TOTAL DISCHARGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>ENTIRE CREST OF DAM</td>
<td>1135.5</td>
<td>5667</td>
<td>600</td>
<td>N/A</td>
<td>10%</td>
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<tr>
<td>RESERVOIR FILLED TO DAM CREST</td>
<td>1120.5</td>
<td>218</td>
<td>218</td>
<td>N/A</td>
<td>100%</td>
</tr>
<tr>
<td>WATER AT SPILLWAY INVERT</td>
<td>1128</td>
<td>0</td>
<td>0</td>
<td>N/A</td>
<td>0%</td>
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</table>
APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS