ANDROSCOGGIN RIVER BASIN
BERLIN, NEW HAMPSHIRE

SITE NO.1, DEAD RIVER DAM
N H 00473

STATE NO 24.14

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

JULY 1979

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<td>ABSTRACT</td>
<td>The dam has a hydraulic height of 46.5 ft and is 3035 ft. long. The dam is good condition with a few minor concerns which need attention. It is intermediate in size with a high hazard potential. A major breach at normal or recreation pool level could result in the loss of 10 or more lives and excessive property damage.</td>
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THIS DOCUMENT IS BEST QUALITY PRACTICABLE. THE COPY FURNISHED TO DTIC CONTAINED A SIGNIFICANT NUMBER OF PAGES WHICH DO NOT REPRODUCE LEGIBLY.
Honorable Hugh J. Gallen
Governor of the State of New Hampshire
State House
Concord, New Hampshire 03301

Dear Governor Gallen:

Inclosed is a copy of the Site No. 1, Dead River Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Water Resources Board, the cooperating agency for the State of New Hampshire. In addition, a copy of the report has also been furnished the owner, City of Berlin, Berlin, New Hampshire 03570.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Water Resources Board for your cooperation in carrying out this program.

Sincerely,

[Signature]

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer
Identification No.: NH00473
Name of Dam: Site No. 1, Dead River
City: Berlin
County and State: Coos County, New Hampshire
Stream: Jericho Brook
Date of Inspection: June 7, 1979

BRIEF ASSESSMENT

Site No. 1, Dead River Dam (the Project) has a hydraulic height of 46.5 feet, is 16 feet wide at its crest, and is 3,035 feet long. It is a zoned compacted earth embankment with a 100-foot wide grass covered earthen emergency spillway. The dam spans a reach of Jericho Brook, and is located in northeastern New Hampshire approximately 4 miles northwest to the City of Berlin. Maximum storage capacity is about 4,720 acre-feet. The Project is a dual-purpose structure, providing recreation and flood control. The pond (Jericho Lake) is approximately 3,700 feet in length with a normal surface area of about 132 acres.

The dam is in good condition. Minor concerns are: extensive vehicular trespassing on the crest, the upstream slope, the downstream slope, and the area immediately downstream of the toe of the dam is destroying the grassy ground cover and leaving ruts in the ground surface. Two minor seepages near the outlet of the principal spillway discharge pipe and the two corrugated metal underdrain pipes; and one seepage approximately 300 feet south of the bend in the dam and 100 feet downstream from the toe of the dam.

Based on an intermediate size and a high hazard classification in accordance with Corps guidelines, the test flood is the Probable Maximum Flood (PMF). A test flood inflow of 16,870 cfs was routed, resulting in a test flood outflow of 7,975 cfs (1,231 csm) would not overtop the dam although the reservoir would rise to elevation 1,370.9 feet MSL, which is 0.1 feet below the top of the dam. The spillway will pass 8,102 cfs or about 102 percent of the test flood. A major breach at normal or recreation pool level could result in the loss of 10 or more lives and excessive property damage.

The owner, City of Berlin, should implement the results of the recommendations and remedial measures given in Sections 7.2 and 7.3 within 2 years after receipt of this Phase I Inspection Report.

Warren A. Guinan
Project Manager
N.H. P.E. 2339
This Phase I Inspection Report on Site No. 1, Dead River Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

JOSEPH A. MCELROY, MEMBER
Foundation & Materials Branch
Engineering Division

CARNEY M. TERZIAN, MEMBER
Design Branch
Engineering Division

JOSEPH W. FINEGAN, JR., CHAIRMAN
Chief, Reservoir Control Center
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division
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.
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Figure 1 - Overview of Site No. 1, Dead River Dam.
NATIONAL DAM INSPECTION PROGRAM
PHASE I INSPECTION REPORT
SITE NO. 1, DEAD RIVER DAM

SECTION 1
PROJECT INFORMATION

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. Anderson-Nichols & Company, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to Anderson-Nichols under a letter of March 22, 1979 from John P. Chandler, Colonel, Corps of Engineers. Contract No. DACW33-79-C-0050 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 initiate quickly 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. Site No. 1, Dead River Dam, (the Project) is the primary structure in the Soil Conservation Service Dead River Watershed project and is located in the City of Berlin, Coos County, New Hampshire. The project consists of a multiple-purpose dam and an earthen dike. The dike is located on the watershed divide approximately 4,150 feet to the northwest of the dam. The dam, constructed for flood protection and recreation, spans Jericho Brook and creates Jericho Lake. After discharging through the dam, Jericho Brook flows easterly for a distance of approximately 1.75 miles before turning sharply to the southeast. Here, approximately 0.25 miles downstream of its crossing by Route 110, Jericho Brook becomes known as the Dead River, although both names refer to the same watercourse. The Dead River continues to the southeast approximately 3.0 miles to its confluence with the Androscoggin River; for the last 0.5 miles of its course, the Dead River flows through an urbanized area of
b. Description of the Dam and Appurtenances. The Project consists of an earthen dam and an earthen dike. The latter is located approximately 4,150 feet to the northwest of the dam. The dam, the primary structure at Site No. 1 is oriented in a north-south direction and has a 60° 30' bend to the southeast located 525 feet south along the dam from the principal outlet structure. The dam, about 3,035 feet long, 46.5 feet high (hydraulic height) and 16 feet wide at its crest consists of: a 2,935-foot long earthen dam embankment, a 100-foot wide grass-covered emergency spillway, which is located between the north end of the dam and the north abutment, a 3.5' x 10.5' two-stage drop-inlet riser, discharging into a 42-inch diameter reinforced concrete conduit extending through the dam and a 30-inch diameter low-level reservoir drain pipe extending into the pond (Jericho Lake) behind the dam.

The dam is a zoned and compacted earthen structure with a cutoff through alluvium, glacial till, and decomposed rock to firm bedrock. The upstream slope of the embankment is inclined at 3H:1V and downstream face has a 2.5H:1V slope. Design plans indicate that two 12-inch diameter perforated corrugated metal pipes are located near the downstream toe of the dam to collect seepage. These pipes drain into the downstream outlet channel in the vicinity of the principal spillway discharge pipe.

The two-stage riser is the principal outlet structure or spillway, consisting of two 2'H x 2.7'W inlets at the lower level and two 1.9'H x 10.5'W inlets at the second level discharging into a 3.5' x 10.5' shaft connected to a 42-inch diameter reinforced concrete pipe. Trash racks prevent debris from entering the inlets causing blockage of the riser shaft or the 42-inch reinforced concrete discharge conduit. The low-level 30-inch reservoir drain pipe from the upstream reservoir pool is connected to the riser shaft at the invert of the 42-inch diameter reinforced concrete conduit, which discharges into a riprapped plunge pool at the downstream toe of the dam. Invert data on all outlets is provided in Section 1.3. The reservoir drain is sealed by a regular sluice-type gate (cast iron disk) which is controlled by a geared hand-crank gate lift mechanism located on the concrete platform on top of the riser. A manhole located on the platform nearby permits entrance into the structure for servicing the principal outlet works and the reservoir drain gate lift mechanism.

The dike is a zoned earthen structure with a cutoff along the center line of the structure through alluvium, glacial till, and decomposed rock to firm bedrock. The dike embankment is approximately 1,665 feet long, 14 feet high (structural and hydraulic height), 10 feet wide at its crest, and the upstream and downstream slopes are inclined at 3H:1V. Its crest elevation is equal to that of the main dam embankment.
Breach of the dam at normal pool would result in a flood stage of 20.0 feet, 7.0 feet over the roadway, at the Route 110 bridge, 1.4 miles downstream of the dam. It was determined that the breach would result in a stage of about 13.2 feet at the Converse Building (currently closed) on the north side of Route 110, 0.4 miles southeast of the Route 110 crossing of Jericho Brook and approximately 1.8 miles downstream of the dam. At this stage, the Converse Building would be inundated by 3 to 5 feet of water causing extensive property damage. However, the reach further downstream where the Dead River flows through a heavily urbanized area of the City of Berlin, is a far greater hazard area. Previous floods in 1927, 1936, and 1953 caused great property damage in this area. An estimated 70 residential and commercial structures in this area are subject to flood damage. The SCS manuscript Work Plan for Watershed Protection, Flood Prevention, and Recreation, Dead River Watershed (June 1965) discusses at length the Dead River major flooding events. The most devastating flood (1936), with a discharge estimated to have been 2,000 cfs, has a return period (recurrence interval) of approximately 100 years. It was determined that breach of the dam, generating a discharge of approximately 18,200 cfs, would result in a stage of about 21.1 feet at the Main Street bridge which could cause excessive damages and considerable loss of life (10 or more) in the most heavily urbanized area of Berlin. This downstream hazard reach begins approximately 4.0 miles downstream of the dam and 0.75 miles above the confluence of the Dead River with the Androscoggin River.

Breath of the dam at emergency spillway elevation would result in a flood stage of 21.5 feet, about 8.5 feet over the roadway at the Route 114 bridge. The resulting stage at the Converse Building downstream of the bridge was determined to be 17.0 feet, causing a 7 foot inundation of the building. At the Main Street bridge, the breach discharge of 31,700 cfs would result in a stage of about 23 feet.

As a result of this analysis and due to major historical flooding events on the Dead River in the City of Berlin, the dam is considered to be of High Hazard potential. For more detailed information regarding the breach analysis and the SCS Watershed Work Plan see Appendix D.
e. Test Flood Analysis. The dam is classified as being intermediate in size, having a hydraulic height of 46.5 feet and a maximum (top of dam) storage of approximately 4,720 acre-feet. Using the Recommended Guidelines for Safety Inspection of Dams, the test flood was determined to be the Probable Maximum Flood (PMF). The watershed above the dam, determined by the SCS to have an average slope of 685 feet/mile, is classified as being mountainous. From the PMF Peak Flow Rates graph the discharge for a mountainous watershed of 6.48 square miles is 2,130 cubic feet per second per square mile (csm) of drainage area. Thus, the peak inflow into the site is 13,800 cubic feet per second (cfs). However, SCS developed a freeboard hydrograph which generated a higher peak inflow (16,870 cfs or 2,600 csm) which has been considered to be the PMF because of the greater detail employed in the SCS analysis. An outflow of 7,975 cfs or 1,230 csm was determined by the SCS from a routing and has been employed as the test flood. Analysis of the elevation versus discharge curves indicates that the principal and emergency spillways combined are capable of passing the test flood without dam overtopping. A discharge of the test flood magnitude would result in a reservoir pool elevation of 1,370.9 feet MSL, which is 0.1 feet below the top of the dam. The total project capacity is 8,102 cfs which is 102 percent of the test flood discharge.

f. Dam Failure Analysis. The impact of the failure of the dam with the reservoir at recreation or normal pool elevation (principal spillway crest) was assessed using the Guidance for Estimating Downstream Dam Failure Hydrographs issued by the Corps of Engineers. Owing to the capacity of the dam project, it was assumed that assessing the impact of dam failure at top of dam (elevation 1,371.0 feet MSL) would be an unrealistic analysis. Therefore the dam failure analysis with Jericho Lake at normal pool level (1,352.0 feet MSL) and at the crest of the emergency spillway (1,362.0 feet MSL) was assessed. Though analysis of dam failure at normal flow conditions was sufficient to identify the hazard potential, computations for failure at emergency spillway crest were also completed.

It was also unrealistic to assume that the dam's breach width would be 640 feet (0.4 times the dam's length at mid-height) as is suggested in the "Rule of Thumb" Guidance for Estimating Downstream Dam Failure Hydrographs. A breach width of 100 feet has been assumed to be a more realistic value. The analysis covered the downstream reach extending from the dam to the urbanized area of the City of Berlin, approximately 4.0 miles below on the Dead River. Although the dam spans Jericho Brook and creates Jericho Lake, it has been determined from field inspection that the Dead River and Jericho Brook are the same watercourse. Approximately 1.75 miles east of the dam, Jericho Brook turns sharply to the southeast and is renamed the Dead River. The watercourse is known as the Dead River from this point to its confluence with the Androscoggin River 3.0 miles downstream.
SECTION 5
HYDROLOGIC/HYDRAULIC

5.1 Evaluation of Features

a. General. The Project consists of an intermediate height earthen dam and an earthen dike which impounds a reservoir (Jericho Lake) of intermediate size. The total length of the dam embankment is 3,035 feet which includes 100 feet of grass-covered earthen emergency spillway. The top of the dam (1,371.0 feet MSL) is 9.0 feet above the emergency spillway crest and 19.0 feet above the low-stage of the principal spillway. The dike, approximately 4,150 feet west of the dam, on the watershed divide, is an earthen structure 1,665 feet long with a crest elevation of 1,371.0 feet MSL. The normally maintained recreation pool (1,352.0 feet MSL) extends approximately 3,700 feet upstream of the dam; the flood control pool (1,362.0 feet MSL) extends approximately 4,150 feet upstream of the dam to the dike embankment.

SCS has calculated the time required to draw down Jericho Lake, first, from the flood-control pool elevation to the principal spillway crest and, secondly, from the principal spillway crest to the level of the reservoir drain (see Appendix B). A drawdown estimate was calculated assuming no inflow, the two-stage riser was the only discharging outlet, and reservoir pool level initially at the crest of the emergency spillway (1,362.0 feet MSL). An analysis of the drawdown capacity under falling-head conditions determined that it would take 10.1 days to draw down the lake to the level of the principal spillway crest (1,352.0 feet MSL). The second drawdown estimate was calculated assuming an average inflow rate of 10.5 cfs (1.62 csm) into the reservoir, the 30-inch diameter reinforced concrete low-level reservoir drain pipe was open and flowing full, and the reservoir pool level was initially at the crest of the principal spillway. It was determined that it would take 10.8 days to drain Jericho Lake under these conditions.

b. Design Data. Detailed hydrologic and hydraulic data for the Project were disclosed (Appendix B). SCS design details give a principal spillway capacity of 302 cfs and an emergency spillway capacity of 7,800 cfs. The as-built plans indicate that the design capacities are representative of as-built conditions.

c. Experience Data. The owner of the dam has indicated that the maximum stage at the damsite occurred when the reservoir pool elevation reached the second-stage outlet of the principal spillway. Flow was estimated to have been 90 cfs.

d. Visual Observation. No evidence of damage to the dam as the result of flood flow was visible at the time of this inspection. The reservoir pool has never reached the dike embankment.
SECTION 4
OPERATIONAL PROCEDURES

4.1 Procedures

Formal operating procedures (see Appendix B, pages B-1 to B-5) have been agreed upon by the Soil Conservation Service, the designer of the Project, and the City of Berlin, the current owner and sponsor of the project. The City is required to inspect all structural works of improvement annually and after every major storm or occurrence of any unusual adverse conditions that affect their operation. However, the only operable outlet structure at the Project, is the 30" low-level reservoir control which allows the pool to be drawn down for cleaning or repairs at the site. The principal and emergency spillways function automatically as the reservoir pool elevation changes. The SCS provides consultative assistance to the City of Berlin for the operation of the structures.

4.2 Maintenance of the Structures

The City of Berlin has been responsible for the maintenance of Site No. 1, Dead River Dam since 1973. As set forth in the Operation and Maintenance Agreement between the City and the SCS, the City must obtain SCS approval of all plans, designs, and specifications for maintenance work involving major repairs "... which include: (1) repairing separated joints, cracks, or breaks in the principal spillway, (2) correcting seepage, (3) replacing significant backfill around structures resulting from major erosion damage, (4) major vegetation due to failure to obtain an adequate vegetative cover, and (5) restoring areas with significant erosion caused by unusual flow in emergency spillways." The City is required to undertake all maintenance that the SCS has determined to be necessary. The SCS will provide the City with consultative assistance in the preparation of plans, designs, and specifications for needed repairs of the structures.

4.3 Maintenance of Operating Facilities

The City of Berlin is required to inspect all structural works annually and after every major storm or occurrences of any unusual adverse conditions that affect their operation.

4.4 Description of Any Warning System in Effect

No written warning system for the Project was disclosed.

4.5 Evaluation

Minor maintenance is good; however, extensive trespassing by vehicles on the crest, upstream and downstream slopes, and the area immediately downstream of the toe requires attention. The dike is not as well maintained as the main dam.
Extensive trespassing by vehicles on the crest, the upstream slope, the downstream slope, and the area immediately downstream of the toe of the dam is destroying the grassy ground cover and leaving ruts in the ground surface. If the trespassing is not controlled, it may lead to serious erosion and seepage problems.

Two minor seepages near the outlets of the spillway pipe and the corrugated metal pipe underdrains may be indicative of a problem. The seepage that is located more than 100 feet downstream of the toe of the dam may or may not be associated with flow from reservoir, and therefore may not be indicative of a problem and presents no problem to discharge.

The saplings growing on the slopes of the dike could pose a long-term problem if allowed to grow into trees.
to the top of the concrete slab. Inside dimensions of the box are 10.5 feet long by 3.5 feet wide. Two 2.7-foot wide by 2-foot high lower inlets are cast into the ends of the riser section 6.4 feet below the top slab and the two higher-level spillway inlets have a crest 4.0 feet above the crest of the lower inlets. The higher-level inlets are 10.5 feet long and are 1.9 feet high. All spillway openings in the riser are protected with steel trash gate openings to contain floating debris. All visible portions of the concrete riser appear to be in very good condition with no signs of deterioration or corrosion. The outlet end of the 42-inch diameter outlet pipe (see Appendix C - Figures 8 and 9) is reinforced concrete also observed to be in very good condition. The outlet ends of the corrugated metal toe drains discharging to the spillway outlet pipe from downstream face are in good condition.

The emergency spillway (see Appendix C - Figure 15) is an open channel about 100 feet wide at the bottom and with a 1H:1V side slope of exposed rock and 4H:1V side slope of earth embankment to the north and a 4H:1V side slope of earth embankment to the south. The channel bottom appears to be mostly glacial till. Several of the rock exposures in the channel appear to be bedrock, while others are large boulders.

d. Dike. The dike (see Appendix C - Figures 16 and 17) is located on the drainage divide along the west side of the reservoir. The dike is about 1,665 feet long, 14 feet high, and 10 feet wide at the crest. The reservoir elevation at the time of the inspection was well below the upstream toe of the dike. The crest of the dike is covered with grass and has an elevation equal to that of the dam embankment crest. The upstream and downstream slopes of the dike are inclined at 3H:1V. Both slopes are covered with grass and some brush and saplings are beginning to grow on the slopes. Some swampy areas near the downstream toe of the dike are obviously the result of a high natural water table and are not the result of seepage from the reservoir.

e. Reservoir Area. The watershed (see Appendix C - Figure 18) above the reservoir is rolling and heavily wooded. A bathing beach, bathhouse, and boat ramp are located on the north shore of the reservoir; no other structures exist on the shores of the lake. No evidence of significant sedimentation was observed.

f. Downstream Channel. The spillway outlet conduit (see Appendix C - Figure 19) discharges into a riprap-lined plunge pool. Riprap extends in the downstream channel for a distance of about 75 feet from the outlet conduit. Beyond that point are trees and brush growing adjacent to the channel. Downstream from the emergency spillway are trees and brush growing in that channel and presents no problem to discharge.

3.2 Evaluation

Based on the visual inspection, the Project is in good condition.
SECTION 3
VISUAL INSPECTION

3.1 Findings

a. General. The Project consists of a dam of intermediate height which impounds a reservoir of intermediate size and a dike 4,150 feet to the northwest of the dam. The watershed above the reservoir is rolling and heavily wooded. The downstream area is rolling and partially wooded.

b. Dam. The Project is an earthen embankment about 46.5 feet high (hydraulic and structural height), 2,935 feet long, and 16 feet wide at the crest. (See Appendix C - Figures 2, 3, and 4.) A sand-and-gravel roadway, completely bare of vegetation, runs along the crest of the dam. The upstream slope of the embankment is inclined at 3H:1V and is covered with grass which is kept mowed. Riprap extends from an elevation about 3 feet above the reservoir level at the time of the inspection to an elevation about 15' below dam embankment crest. The riprap was in good condition. Near the south abutment there are some ruts (see Appendix C - Figures 5 and 6) where a vehicle was apparently driven on the upstream slope during wet weather. The downstream slope is also covered with grass which is kept mowed. Ruts are at many locations on the downstream slope where vehicles have apparently been driven during wet weather. There are also many ruts (see Appendix C - Figure 7) in the relatively flat grassy area immediately downstream of the dam. Two corrugated metal drain pipes (see Appendix C - Figures 8 and 9) discharge at the downstream toe, one on either side of the spillway outlet pipe. Water was flowing from both drain pipes at the time of the inspection. Two minor seepage areas are located on the south side of the spillway outlet pipe. One seepage (see Appendix C - Figure 10) was observed approximately 300 feet south of the bend in the dam and 100 feet downstream from the toe of the dam. It is not known whether this seepage is coming from the reservoir or whether it is natural spring. The north end of the dam is an earthen slope (see Appendix C - Figures 11, 12, and 13) which serves as part of the south bank of the emergency spillway channel. Vehicles have been driven up two paths on this slope, destroying the vegetation and leaving the end of the embankment susceptible to erosion.

c. Appurtenant Structures. A concrete outlet structure (see Appendix C - Figure 14) is located near the center of the dam. This structure serves as a low-level outlet, and two-stage principal spillway, all constructed in one vertical riser. The reservoir drain outlet is a gated 30-inch diameter pipe located at the base of the riser. The mechanical gate operating mechanism mounted on the top of the riser appeared to be well maintained and in good condition. The concrete riser section is 20 feet high from base to the crest of the higher-level spillway and an additional 2.4 feet
The City is required to inspect all structural works of improvement annually and after every major storm or occurrence of any unusual adverse conditions that affect their operation. However, the only manually operated structure at Site No. 1, Dead River Dam, is the low-level reservoir outlet which allows the pool to be drawn down for cleaning or repairs at the site. The principal and emergency spillways function automatically as the reservoir pool elevation changes. The SCS provides consultative assistance to the City of Berlin for the operation of the structures.

2.4 Evaluation

a. Availability. Complete SCS engineering plans and sketches, hydrologic and hydraulic calculations, construction field notes, and detailed geologic data are on file at the SCS Durham office.

b. Adequacy. Field inspection of Site No. 1, Dead River Dam, indicated that the SCS plans and sketches were adequate. Final assessments and recommendations are based upon the SCS plans, sketches, and hydrologic and hydraulic calculations in conjunction with the visual inspection of the dam and dike sites.

c. Validity. The visual inspection disclosed that the present conditions are consistent with the SCS as-built plans and sketches.
SECTION 2
ENGINEERING DATA

2.1 Design
Site No. 1, Dead River Dam, was designed by the U.S. Department of Agriculture, Soil Conservation Service (SCS) in 1968. Design data were found in the SCS Durham, New Hampshire, office. These data consisted of:

(1) Complete design report with all hydrologic and hydraulic calculations (see Appendix B).

(2) Detailed geologic investigations of the dam and dike sites (see Appendix B).

(3) Dam, dike, and emergency spillway design data on 39 sheets and other data include (selected 9 of 39 sheets in Appendix B):

(a) Plans of dike, dam, principal spillway, and emergency spillway, toe drains, and pool.

(b) Sketches of typical sections of dam and dike embankment and emergency spillway.

(c) Profiles along centerline of dike, dike cutoff trench, dam, dam cutoff trench, and emergency spillway.

(d) Detailed listing of quantities of construction materials.

(e) Sketches of fill placement in dam and dike embankments.

(f) Logs of test holes (not readily reproducible).

2.2 Construction
The design plans were revised in 1971 to reflect as-built conditions. Field notes and revisions made to the design plans are available in the SCS Durham office.

2.3 Operation
The City of Berlin is responsible for operation of the dam for structural measures as set forth by an agreement (see Appendix B, pages B-1 to B-5) between the SCS, the project's designer, the New Hampshire Water Resources Board (NHWRB), and the City of Berlin, the sponsor of the Dead River Watershed project.
(6) U/S Channel - The dam spans Jericho Brook and creates Jericho Lake. The watershed above the reservoir is steep and heavily wooded. A bathing beach, bathhouse, and boat ramp are located on the northshore of the reservoir and comprise Jericho Lake Park. No other development exists on the shores of Jericho Lake. The dike is not clearly visible from the dam.

(7) D/S Channel - The channel immediately downstream is about 20 feet in width. The channel bottom is a mixture of sand, gravel, and boulders. Trees and heavy brush cover the valley sides. In the immediate vicinity of the principal spillway outlet erosion of the channel walls is apparent.

k. Emergency Spillway

(1) Type - a grass covered earthen channel having side slopes 1H:1V of exposed rock and 4H:1V of earth embankment to the north and 4H:1V side slope of earth embankment to the south.

(2) Width - 100'

(3) Crest elevation - 1,362.0' MSL

(4) Length of level section - 60' (approximate)

(5) U/S Channel - The approach channel originates at the northeast bank of the reservoir north of the dam and is grass covered.

(6) D/S Channel - The downstream channel is not well defined with standing trees and brush noted. It joins the downstream channel of the principal spillway several hundred yards downstream of the principal spillway outlet.

1. Regulating Outlet. A 60-foot long, 30-inch diameter reinforced concrete pipe serves as reservoir drain, originating at upstream pool bottom with invert at 1,329.3 feet (MSL) and discharges into 42-inch diameter concrete. The crank-operated gate lift mechanism is located on top of the cover of the two-stage drop-inlet riser.
(8) Cutoff - core trench excavated through alluvium, glacial till and decomposed rock to firm bedrock as shown on SCS design plans

(9) Grout curtain - none

(10) Toe drain - two 12" diameter perforated corrugated metal pipes

h. Dike

(1) Type - earthen embankment on unconsolidated glacial deposits along the watershed divide

(2) Length - 1,665'

(3) Height - 14'

(4) Topwidth - 10'

(5) Side Slopes - 3H:1V upstream and downstream

(6) Zoning - 2 zones in dike embankment indicated on SCS design plans (see Appendix B)

(7) Impervious core - none indicated on SCS design plans

(8) Cutoff - core trench excavated through alluvium, glacial till, and decomposed rock to firm bedrock along centerline shown on SCS design plans

(9) Grout curtain - none

(10) Toe drain - none

i. Diversion and Regulating Tunnel - not applicable

j. Principal Spillway

(1) Type - a vertical reinforced concrete two-stage drop-inlet riser having a covered top with two 2'H x 2.7'W rectangular inlets at the lower level and two 1.9'H x 10.5'W inlets at the higher level which ultimately discharge into a 42-inch horizontal conduit.

(2) Size - 3.5' x 10.5' drop-inlet riser discharging into a 42-inch diameter horizontal conduit with a length of 204 feet through dam embankment

(3) Crest Elevation - 1,352.0' MSL for first level inlets and 1,356.0' MSL for second level inlets

(4) Gates - none

(5) Low-level - 30-inch diameter reinforced concrete pipe which originates in pool bottom and discharges into the 42-inch conduit.
(4) Length of flood control pool - 4,150

e. **Storage (acre-feet)**

(1) Recreation pool - 1,240
(2) Flood control pool - 2,800
(3) Principal spillway crest pool - 1,240
(4) Emergency spillway crest pool - 2,800
(5) Top of dam - 4,720
(6) Test flood pool - 4,695

f. **Reservoir Surface (acres)**

(1) Recreation pool - 132
(2) Flood control pool - 177
(3) Principal spillway crest - 132
(4) Emergency spillway crest - 177
(5) Test flood pool - 208
(6) Top of dam - 215

g. **Dam**

(1) Type - earthen embankment on unconsolidated glacial deposits with drop-inlet spillway (principal) and grassed emergency spillway; consists of 333,700 cubic yards of fill.

(2) Length - 3,035' (includes 100-foot wide emergency spillway)

(3) Height - 46.5' (hydraulic and structural height)

(4) Topwidth - 16'

(5) Side slopes - 3H:1V upstream and 2.5H:1V downstream

(6) Zoning - 3 zones in dam embankment and riprap on the upstream slope as indicated on SCS design plans.

(7) Core - trapezoidal section consisting of silty sands with a 40' width at base, 10' topwidth, and height of 30'; elevation at top of impervious core is 1,352.0' MSL as shown on SCS design plans.
(1) Low-level outlet (reservoir drain) 30" RCP capacity @ principal spillway elevation - 112 cfs @ 1,352' MSL

(2) Principal (drop-inlet) spillway capacity @ test flood elevation - 302 cfs @ 1,370.9' MSL (low stage: 113 cfs; high stage: 189 cfs)

(3) Emergency spillway discharge @ test flood elevation - 7,673 cfs @ 1,370.9' MSL

(4) Total project discharge @ test flood elevation - 7,975 cfs @ 1,370.9' MSL

(5) Principal (drop-inlet) spillway capacity @ top of dam - 302 cfs @ 1,371.0' MSL (low stage: 113 cfs; high stage: 189 cfs)

(6) Emergency spillway capacity @ top of dam - 7,800 cfs @ 1,371.0' MSL

(7) Total spillway capacity @ top of dam - 8,102 cfs @ 1,371.0' MSL

c. Elevation (feet above MSL as shown on available "as built" SCS plans in Appendix B)

(1) Streambed @ centerline of dam - 1,324.5 (approximate, at downstream toe)

(2) Bottom of cutoff trench - 1,317.0

(3) Maximum tailwater - unknown

(4) Upstream invert low-level outlet - 1,329.3

(5) Recreation pool - 1,352.0 (principal spillway)

(6) Full flood control pool - 1,362.0

(7) Drop-inlet spillway crest - lower level - 1,352.0, higher level - 1,356.0

Emergency spillway crest - 1,362.0

(8) Design surcharge (original design) - 1,371.0

(9) Top of dam - 1,371.0

(10) Test flood pool - 1,370.9

d. Reservoir (feet)

(1) Length of maximum pool - 4,150

(2) Length of pool at principal spillway crest - 3,700 (approximate)

(3) Length of pool at emergency spillway crest - 4,150 (approximate)
c. **Size Classification.** Intermediate (hydraulic height - 46.5 feet; storage - 4,720 acre-feet) based on criteria (intermediate size corresponds to dam with height ≥ 40 feet and < 100 feet and storage ≥ 1,000 and < 50,000 acre-feet) in Recommended Guidelines for Safety Inspection of Dams.

d. **Hazard Classification.** High Hazard. A major breach could result in extensive property damage and considerable loss of life (10 or more) downstream in the urbanized area of the City of Berlin. Major floods in 1927, 1936, and 1953 on the Dead River, with significantly smaller discharges than the breach discharge used in this report, support this contention (See Section 5.1 f.).

e. **Ownership.** The Project is owned by the City of Berlin, New Hampshire.

f. **Operator.** The dam is operated by the City of Berlin Recreation and Parks Department, First Avenue, Berlin, New Hampshire, 03570. Telephone (603) 752-2010.

g. **Purpose of the Project.** The Project was constructed for the purposes of storing floodwaters and providing recreational opportunities.

h. **Design and Construction History.** The Project was designed by the Soil Conservation Service (SCS), Durham, New Hampshire office. Construction was begun in May 1969 by Rogers Construction Co., Inc. of Brattleboro, Vermont and the project was 99 percent complete by December 1969. All remaining construction work was completed by September 1970 by the same firm.

i. **Normal Operating Procedures.** The City of Berlin, as is set forth in the Operation and Maintenance Agreement for Structural Measures (between the City and SCS), is required to inspect all structural works of improvement annually and after every major storm or occurrence of any unusual adverse conditions that affect their operation. The only operable outlet at the Project is the low-level reservoir drain which allows the reservoir pool level to be drawn down for cleaning or repairs at the site. The principal spillway (two-stage riser) and the emergency spillway function automatically as the reservoir pool elevation changes.

1.3 **Pertinent Data**

a. **Drainage Area.** The watershed above the dam consists of 6.48 square miles (4,147 acres) of steep forested terrain. No storage areas exist in the watershed above the upstream limit of Jericho Lake. The normal surface area (recreation pool) is 132 acres which constitutes 3.2 percent of the total watershed area.

b. **Discharge at Damsite.** According to the director of the City of Berlin Park and Recreation Department, the maximum known stage at the damsite occurred at an unknown date when the reservoir pool elevation approximately reached the second stage inlet of the principal spillway. Using the rating curve calculated for the dam, the project discharge was estimated to have been 90 cfs.
6.1 Evaluation of Structural Stability

a. Visual Observations. The visual examination indicates the following evidence of potential problems:

(1) Extensive vehicular trespassing on the crest, the upstream slope, the downstream slope, and the area immediately downstream of the toe of the dam is destroying the grassy ground cover and leaving ruts in the ground surface.

(2) Two minor seepages near the outlet of spillway discharge pipe and the two underdrain pipes.

(3) One seepage more than 100 feet downstream from the toe of the dam.

b. Design and Construction Data. Site No. 1, Dead River Dam, was designed by the U.S. Department of Agriculture, Soil Conservation Service (SCS) in 1968. Design data were found in the SCS Durham, New Hampshire, office. These data consisted of:

(1) Complete design report with all hydrologic and hydraulic calculations (see Appendix B).

(2) Detailed geologic investigations of the dam and dike sites (see Appendix B).

(3) Dam, dike, and emergency spillway design data on 39 sheets; include (selected 9 of 39 sheets in Appendix B):

(a) Plans of dike, dam, principal spillway, and emergency spillway, toe drains, and pool.

(b) Sketches of typical sections of dam and dike embankments and emergency spillway.

(c) Profiles along centerline of dike, dike cutoff trench, dam, dam cutoff trench, and emergency spillway.

(d) Detailed listing of quantities of construction materials.

(e) Sketches of fill placement in dam and dike embankments.

(f) Logs of test holes (not readily reproducible).

The design plans were revised in 1971 to reflect as-built conditions. Field notes and revisions made to the design plans are available in the SCS Durham office.
c. Operating Records. Detailed inspections records obtained from the SCS Durham office are in Appendix B.

d. Post-Construction Changes. There is no record of post-construction changes.

e. Seismic Stability. This dam is located in Seismic Zone 2 and in accordance with the Phase I guidelines does not warrant seismic analysis.
7.1 Dam Assessment

a. Condition. The visual examination indicates that Dead River Dam is in good condition. The only concerns with respect to the long-term stability of the dam are:

(1) Extensive vehicular trespassing on the crest, the upstream slope, the downstream slope, and the area immediately downstream of the toe of the dam is destroying the grass ground cover and leaving ruts in the ground surface.

(2) Two minor seepages near the outlet of spillway discharge pipe and the two corrugated metal underdrain pipes.

(3) One seepage approximately 300 feet south of the bend in the dam and 100 feet downstream from the toe of the dam may or may not be associated with flow from the reservoir and, therefore, may not be indicative of a problem.

b. Adequacy of Information. The visual examination and the complete design data available are adequate for the purposes of the Phase I inspection.

c. Urgency. The recommendations and remedial measures made in 7.2 and 7.3 below should be implemented by the owner within two years after receipt of this Phase I inspection.

d. Need for Additional Investigation. There is no need for additional investigation for the purposes of this Phase I report.

7.2 Recommendations

The owner should engage a registered professional engineer to investigate the two minor seepages near the spillway outlet and the one seepage which is more than 100 feet downstream from the dam.

7.3 Remedial Measures

a. Operating and Maintenance Procedures. The owner should:

(1) Repair ruts and control trespassing on the dam and the area immediately downstream of the dam.

(2) Visually inspect the dam and appurtenant structures and monitor seepage once a month.

(3) Engage a registered professional engineer to make a comprehensive technical inspection of the dam once every two years.

(4) Establish an around-the-clock surveillance program
for use during and immediately after heavy rainfall and also a warning program to follow in case of emergency conditions or imminent dam failure.

(5) Mow and cut saplings on dike embankment seasonally.

(6) Observations at the dam should be made after significant events, but while substantial storage is still being utilized to determine if seepage exists which could not be detected when the reservoir pool is at normal elevation.

7.4 Alternatives

None.
APPENDIX A

VISUAL INSPECTION CHECKLIST
VISUAL INSPECTION CHECKLIST
PARTY ORGANIZATION

PROJECT Site No. 1, Dead River Dam DATE June 7, 1979
(Berlin, NH) TIME 1400

WEATHER Sunny, warm

W.S. ELEV. U.S. DN.S.
1353.1 1321.0

PARTY:
1. Warren Guinan
2. Stephen Gilman
3. Katherine Hivley
4. Pattu Keshevan
5. Ronald Hirschfeld
6. 
7. 
8. 
9. 
10. 

PROJECT FEATURE INSPECTED BY REMARKS
1. Hydrology/Hydraulics W. Guinan
2. Structural Stability S. Gilman
3. Soils & Geology R. Hirschfeld
PERIODIC INSPECTION CHECKLIST

PROJECT Site No. 1, Dead River (Berlin, NH)  DATE June 7, 1979

PROJECT FEATURE Dam Embankment  NAME 

DISCIPLINE  NAME 

<table>
<thead>
<tr>
<th>AREA EVALUATED</th>
<th>CONDITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>DAM EMBANKMENT</td>
<td></td>
</tr>
<tr>
<td>Crest Elevation</td>
<td>1371.0 MSL</td>
</tr>
<tr>
<td>Current Pool Elevation</td>
<td>1353.1 MSL</td>
</tr>
<tr>
<td>Maximum Impoundment to Date</td>
<td>1356.0 MSL (approximate)</td>
</tr>
<tr>
<td>Surface Cracks</td>
<td>None observed</td>
</tr>
<tr>
<td>Pavement Condition</td>
<td>Not paved</td>
</tr>
<tr>
<td>Movement or Settlement of Crest</td>
<td>None observed</td>
</tr>
<tr>
<td>Lateral Movement</td>
<td>None observed</td>
</tr>
<tr>
<td>Vertical Alignment</td>
<td>Good</td>
</tr>
<tr>
<td>Horizontal Alignment</td>
<td>Good</td>
</tr>
<tr>
<td>Condition at Abutment and at Concrete Structures</td>
<td>Good</td>
</tr>
<tr>
<td>Indications of Movement of Structural Items on Slopes</td>
<td>None observed</td>
</tr>
<tr>
<td>Trespassing on Slopes</td>
<td>Extensive vehicle tracks on downstream face of dam at and beyond toe.</td>
</tr>
<tr>
<td>Sloughing or Erosion of Slopes or Abutments</td>
<td>None observed</td>
</tr>
<tr>
<td>Rock Slope Protection - Riprap Failures</td>
<td>Riprap on upstream face - no failures observed.</td>
</tr>
<tr>
<td>Unusual Movement or Cracking at or Near Toe</td>
<td>None observed</td>
</tr>
<tr>
<td>Unusual Embankment or Downstream Seepage</td>
<td>Two minor seepage areas right of spillway outlet pipe</td>
</tr>
<tr>
<td>Piping or Boils</td>
<td>None observed</td>
</tr>
<tr>
<td>Foundation Drainage Features</td>
<td>Two CMP toe drains discharging water on either side of spillway discharge.</td>
</tr>
<tr>
<td>Toe Drains</td>
<td>None observed</td>
</tr>
<tr>
<td>Instrumentation System</td>
<td>Grass on both upstream and downstream slopes. Some grass on crest, but vehicle tracks on crest are bare soil.</td>
</tr>
<tr>
<td>Vegetation</td>
<td></td>
</tr>
</tbody>
</table>
# PERIODIC INSPECTION CHECKLIST

**PROJECT**  Site No. 1, Dead River (Berlin, Nil)  
**DATE**  June 7, 1979

**PROJECT FEATURE**  Dike Embankment

**DISCIPLINE**  

**AREA EVALUATED** | **CONDITION**
--- | ---
DIKE EMBANKMENT |  
Crest Elevation | 1371.0 MSL
Current Pool Elevation | 1353.1 MSL
Maximum Impoundment to Date | 1356.0 MSL (approximate)
Surface Cracks | None observed
Pavement Condition | Not paved
Movement or Settlement of Crest | None observed
Lateral Movement | None observed
Vertical Alignment | Good
Horizontal Alignment | Good
Condition at Abutment and at Concrete Structures | Good
Indications of Movement of Structural Items on Slopes | None
Trespassing on Slopes | None observed
Sloughing or Erosion of Slopes or Abutments | None observed
Rock Slope Protection - Riprap Failures | No riprap
Unusual Movement or Cracking at or Near Toes | None observed
Unusual Embankment or Downstream Seepage | No water against upstream side of dike
Piping or Boils | None
Foundation Drainage Features | None observed
Tie-Ins | None observed
Instrumentation System | None observed
Vegetation | Grass on crest, grass, brush, and small trees on upstream slope and downstream slopes.
PERIODIC INSPECTION CHECKLIST

PROJECT Site No. 1, Dead River (Berlin, NH)  DATE June 7, 1979

PROJECT FEATURE Principal Spillway (Two-stage riser)  NAME

DISCIPLINE  NAME

<table>
<thead>
<tr>
<th>AREA EVALUATED</th>
<th>CONDITION</th>
</tr>
</thead>
<tbody>
<tr>
<td>OUTLET WORKS - CONTROL TOWER</td>
<td>Good</td>
</tr>
<tr>
<td>a. Concrete and Structural</td>
<td>Good - no indications of movement.</td>
</tr>
<tr>
<td>General Condition</td>
<td>None visible</td>
</tr>
<tr>
<td>Condition of Joints</td>
<td>None</td>
</tr>
<tr>
<td>Spalling</td>
<td>None visible</td>
</tr>
<tr>
<td>Visible Reinforcing</td>
<td>None</td>
</tr>
<tr>
<td>Rusting or Staining of Concrete</td>
<td>None visible</td>
</tr>
<tr>
<td>Any Seepage or Efflorescence</td>
<td>None visible</td>
</tr>
<tr>
<td>Joint Alignment</td>
<td>Good - no apparent movement.</td>
</tr>
<tr>
<td>Unusual Seepage or Leaks in Gate Chamber</td>
<td>Not visible</td>
</tr>
<tr>
<td>Cracks</td>
<td>None visible</td>
</tr>
<tr>
<td>Rusting or Corrosion of Steel</td>
<td>None visible - galvanized steel in good condition.</td>
</tr>
<tr>
<td>b. Mechanical and Electrical</td>
<td>Hand-operated crank serves as a gate lift mechanism for low-level reservoir drain; located on concrete platform on top of two-stage riser. Appears to be in good condition.</td>
</tr>
<tr>
<td>Air Vents</td>
<td></td>
</tr>
<tr>
<td>Float Wells</td>
<td></td>
</tr>
<tr>
<td>Crane Hoist</td>
<td></td>
</tr>
<tr>
<td>Elevator</td>
<td></td>
</tr>
<tr>
<td>Hydraulic System</td>
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</tr>
<tr>
<td>Service Gates</td>
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</tr>
<tr>
<td>Emergency Gates</td>
<td></td>
</tr>
<tr>
<td>Lightning Protection System</td>
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</tr>
<tr>
<td>Emergency Power System</td>
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</tr>
<tr>
<td>Wiring and Lighting System</td>
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</tbody>
</table>
**PERIODIC INSPECTION CHECKLIST**

**PROJECT** Site No. 1, Dead River (Berlin, NH)  
**DATE** June 7, 1979

**PROJECT FEATURE** Emergency Spillway  
**NAME**

**DISCIPLINE**

**NAME**

<table>
<thead>
<tr>
<th>AREA EVALUATED</th>
<th>CONDITION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</strong></td>
<td></td>
</tr>
<tr>
<td>d. Approach Channel</td>
<td></td>
</tr>
<tr>
<td>General Condition</td>
<td>Good</td>
</tr>
<tr>
<td>Loose Rock Overhanging Channel</td>
<td>None</td>
</tr>
<tr>
<td>Trees Overhanging Channel</td>
<td>None</td>
</tr>
<tr>
<td>Floor of Approach Channel</td>
<td>Glacial till</td>
</tr>
<tr>
<td>b. Weir and Training Walls</td>
<td></td>
</tr>
<tr>
<td>General Condition of Concrete</td>
<td>None - earth spillway</td>
</tr>
<tr>
<td>Rust or Staining</td>
<td>None</td>
</tr>
<tr>
<td>Spalling</td>
<td>None</td>
</tr>
<tr>
<td>Any Visible Reinforcing</td>
<td>None</td>
</tr>
<tr>
<td>Any Seepage or Efflorescence</td>
<td>None</td>
</tr>
<tr>
<td>Drain Holes</td>
<td>None</td>
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<tr>
<td>e. Discharge Channel</td>
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<tr>
<td>General Condition</td>
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</tr>
<tr>
<td>Loose Rock overhanging Channel</td>
<td>None</td>
</tr>
<tr>
<td>Trees Overhanging Channel</td>
<td>Some trees in channel</td>
</tr>
<tr>
<td>Floor of Channel</td>
<td>Glacial till</td>
</tr>
<tr>
<td>other obstructions</td>
<td>None</td>
</tr>
<tr>
<td>AREA EVALUATED</td>
<td>REMARKS</td>
</tr>
<tr>
<td>--------------------------------------</td>
<td>----------------------------------------------</td>
</tr>
<tr>
<td>Stability of Shoreline</td>
<td>Good</td>
</tr>
<tr>
<td>Sedimentation</td>
<td>None visible</td>
</tr>
<tr>
<td>Changes in Watershed</td>
<td>None visible</td>
</tr>
<tr>
<td>Runoff Potential</td>
<td>None visible</td>
</tr>
<tr>
<td>Upstream Hazards</td>
<td>None visible</td>
</tr>
<tr>
<td>Downstream Hazards</td>
<td>Urbanized area of City of Berlin Converse Building</td>
</tr>
<tr>
<td>Alert Facilities</td>
<td>None visible</td>
</tr>
<tr>
<td>Hydrometeorological Gages</td>
<td>None visible</td>
</tr>
<tr>
<td>Operational &amp; Maintenance Regulations</td>
<td>None visible</td>
</tr>
</tbody>
</table>
APPENDIX B

ENGINEERING DATA
Dead River - Topdressing

Recommendations for this season.

Use 500#/acre of "Sul-po-mag". This is sulfate of potash magnesia. This may be purchased at Merrimack Farmers' Exchange in 80# bags. The ton price is $125. There may be other local sources.

There are 6½ acres involved in the dam. Of these, 2½ acres are on the upstream face of the dam and 4½ on the downstream face.

Basis for recommendations.

The dam is covered with a good population of birdsfoot trefoil. The vigor of these plants is very poor on the upstream face and fairly good but deteriorating on the downstream face.

Soil tests taken on 5/27/77 revealed a deficiency of magnesium and potassium in relation to the needs of birdsfoot trefoil.

The objective of the recommendation is to try and increase the plant cover of birdsfoot trefoil in order to prevent erosion and to prevent encroachment of woody plant species that would require annual eradication.

Recommendations for next spring and early summer.

A topdressing of 300#/acre of muriate of potash next spring is very desirable. If this can be accomplished, further topdressing should be based on soil tests taken every other year.

One ton of muriate of potash would cover the entire dam. It costs approximately $170/ton and is readily available through local grain companies.

Attachment
This maintenance checklist is a guide for determining the maintenance required for Public Law 566 flood control structures in New Hampshire. It doesn’t take the place of experience and judgment and is not inclusive. Items of a difficult nature to check, such as principal spillway conduit condition, are not included. Intensive checks of these items are necessary at proper intervals. Review of As-Built drawings, the design folder, structure history, and previous maintenance reports should be part of the inspection. Except maintenance is a vital part of safe and effective operation.

Except where otherwise indicated, completion of this form may be facilitated by ranking maintenance items on a 1 to 4 basis where

1 = satisfactory
2 = satisfactory, but check carefully at next inspection
3 = requires maintenance this season
4 = requires immediate attention.

<table>
<thead>
<tr>
<th>WATERSHED</th>
<th>DEAD RIVER</th>
<th>SITE</th>
<th>DATE</th>
<th>6/12/78</th>
</tr>
</thead>
</table>

INSPECTED BY Keith MacPherson, Richard Haldeman, Mitchell Berkowitz, M. L. Glidden

1. GENERAL ITEMS

Access Road.
Site Fencing.
Traffic Conditions.
Vandalism Control.
Trash Control.

COMMENTS Gate is being installed across left end of dam.

2. RESERVOIR

Timber stand at reservoir.
Debris and slash.
Sediment level in relation to low stage inlet.

COMMENTS Brush in the inlet and outlet channel of the Emergency Spillway should be removed. Debris and slash on the upstream embankment.

B-2
June 19, 1975

Mr. James Smith
City Treasurer
City Hall
Berlin, WI 02570

Dear Mr. Smith:

Attached is the annual Operations and Maintenance Inspection Report for the Dead River Watershed conducted on June 17, 1975.

The report lists items that require attention by the City as their obligation under the Operation and Maintenance Agreement of August 26, 1969.

I trust that you will see that these actions are carried out and following completion will send me a letter to that effect.

Sincerely,

Calvin J. Perkins
District Conservationist

cc: Mr. Holley, Acting City Engineer

Dr. Tingele, Assistant State Conservationist (17)
An operation and maintenance inspection of Site 1 (Jericho Lake) in the Dead River Watershed, Berlin, NH was conducted on May 23, 1976. The inspection was made by representatives from the local sponsoring organization, the Coos County Conservation District, and the Soil Conservation Service. Those present were:

Mitchell Berkovitz, Berlin Recreation Director
Randell Harbert, Coos County Conservation District
Richard Hallock, District Conservationist, USDA, SCS
Eddie Wood, USDA, SCS
Tillman Marshall, USDA, SCS
Nick Luhtala, USDA, SCS

Following is a list of items discussed and the necessary action needed:

1. Remove debris from the principle spillway trash rack.
2. Close off, repair erosion and revegetate old access road below dam.
3. Close off and vegetate present road running across the top of the dam.
4. Improve vegetative cover on the dam slopes by:
   A. Removing woody vegetation which is starting to develop in several areas.
   B. Lime and fertilize slopes, especially the upstream slope. Lime and fertilizer should be applied according to soil test recommendations which can be obtained through the County Agent's office, Coos County Extension Service, Lancaster, NH. However, if this information can not be obtained a general recommendation of 2 tons per acre (or 100 lbs. per 1,000 sq. ft.) of ground limestone and 400 lbs. per acre (or 10 lbs. per 1,000 sq. ft.) of 10-10-10 or equivalent fertilizer may be used.
5. Remove alders and other woody vegetation that is starting to grow in the stream overflow area, between the upstream slope of the dam and the inlet stream.
June 3, 1976

Mr. James Smith
City Manager
City Hall
Perlin, WI 03570

Dear Mr. Smith:

Attached is the annual Operations and Maintenance Inspection Report for the Dead River Watershed conducted on May 23, 1976.

The report lists those items which require maintenance by the City. I trust that these items will be carried out as soon as possible under the City's obligation in the Operation and Maintenance Agreement, for this project, of August 26, 1968.

When the items listed are completed please send me a letter so indicating.

If additional technical assistance is needed to carry out any of these maintenance items please feel free to call on us.

Very truly yours,

Richard S. Haldeman
District Conservationist
all of the undesirable vegetation along the dam and the over-flow areas. Much of our new operations and maintenance program included the 1968 project agreement. In future years we hope to improve this type of maintenance based on the changes in the park's areas.

Our summer workers also controlled the erosion along the new access road with the introduction of bark. This has already started new vegetation which should enhance the roadway next year. Two important areas of erosion were also brought to a control. The first was the entrance way which has been carefully cribbed and seeded. The second area was along the steep slope of the parking lot. Again, cribbing and seeding along with a drainage ditch has halted any erosion. During the 1977 summer we will complete the parking lot erosion control, re-landscape the roadway and develop 12 special tent campsites in the upper field.

As always, I look forward to another year at Jericho Lake Park and the needed improvements we hope to fulfill. Your inspection in May 1977 should also provide us with additional recommendations.

Cordially,

Mitchell A. Berkowitz,
Director

CC: Mayor Maurice Lincourt
   City Manager James C. Smith
17 December 1976

Mr. Richard S. Halderman
District Recreationist
U.S.C.S.
Piche Building
99 Main Street
Lancaster, N.H.

Dear Mr. Halderman,

The Recreation Department Summer Staff has worked very diligently to complete many of the inspection recommendations. To date we have removed debris from the principle spillway trash rack and consider this as part of our weekly operation once the park is open each year. In the Early spring this would be checked once the access rood is clear of ice. The Old access road has been partially closed off and we intend to completely close this road as well as the road on top of the dam. All erosion in these areas has been initially halted and further repairs and vegetation will commence in the late spring and early summer. One of the major difficulties in this area is accessibility by EPA. During the 1977 summer we feel that we will be better able to control their use through more additional unauthorized and barrier improvements.

During this Summer we were able to apply lime to the slope and in many cases we fertilized, limed, added lawn and re-seeded have spots along the dam. This will continue during the 1977 summer.

In the area of poor vegetation control I feel we went beyond the recommendations and will continue to control
Concrete: Cracking; Spalling; Other deterioration; Excessive movement (check joints); Waterstops; Joint sealant; Other.

Trashracks: low and high stage
Condition of protective coatings; Corrosion; Damaged parts; Condition of fastenings; Need of gratings due to beaver; Safety condition (protruding fastenings, sharp edges, etc.); Other.

Gates: including lifting device, stem, guides, disc, flap
Condition of protective coating; Corrosion; Damaged parts; Condition of fastenings; Stem alignment; Operation; Lubrication; Wood decay; Other.

Structure Drainage:
Report under "Embankment and Other Drains"

Structure, Railing, Grates, Barriers, etc.
Condition of protective coating; Corrosion; Damaged parts; Condition of fastenings; Wood decay; Safety condition (protruding fastenings, sharp edges, etc.); Other.

Safety Items:
Condition of warning signs; Condition of safety equipment; Other.

COMMENTS
stop log in stream below structure channels for stop log rusty - paint

9. CHANNEL
Stream obstructions.
Debris in stream.
Sediment bars controlled.
Plunge pool stability.
Fish habitat appurtenances.
Riprap -- Report under "Riprap" (item 4)

COMMENTS


B-11
Caution Be extremely careful when using ladders. Check condition before using. Ladders are sometimes broken, loose, corroded, and or slippery. Use safety harness.

Ladders:
inside and out
- Condition of protective coating
- Corrosion
- Damaged parts
- Loose
- Other

Concrete:
inside and out
- Cracking
- Spalling
- Other deterioration
- Excessive movement (check joint at riser and conduit)

Trashracks:
low and high stage
- Condition of protective coatings
- Corrosion
- Damaged parts
- Condition of fastenings
- Need of gratings due to beaver
- Safety condition (protruding fastenings, sharp edges, etc.)

Manhole:
- Condition of protective coatings
- Corrosion
- Damage
- Lock operable
- Other

Gate:
including lifting device, stem, guides, disc
- Condition of protective coating
- Corrosion
- Damaged parts
- Condition of fastenings
- Stem alignment
- Lubrication
- Operation
- Other

Safety Items:
- Condition of warning signs
- Condition of safety equipment

Comments
Trash in racks need cleaning. Didn't check ladder - no wrench to get M.H. cover off. Manhole cover rusty. Gate lift should be painted & checked for operation. Grease & oil if necessary.
5. VEGETATION

<table>
<thead>
<tr>
<th>Condition of stand (including need for lime and fertilizer)</th>
<th>Dam</th>
<th>Left</th>
<th>Right</th>
<th>Dike</th>
<th>Channel</th>
<th>Outlet wave</th>
<th>Flood (Parking)</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undesirable vegetation</td>
<td>3</td>
<td>3</td>
<td></td>
<td></td>
<td>1</td>
<td>3</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Drainage (surface)</td>
<td>1/</td>
<td>1</td>
<td></td>
<td>1</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Erosion</td>
<td>1/</td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Sedimentation</td>
<td>1/</td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Condition of planting</td>
<td>1/</td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Pest control</td>
<td>1/</td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Fire control</td>
<td>1/</td>
<td>1</td>
<td></td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

COMMENTS
- Entire dam should be fertilized based on soil test results.
- Entire dam may need to be limed based on soil test results.
- Shrub growth on dam should be eradicated.
- Spillway should be limed and fertilized based on soil test results.
- Shrub growth in outfall should be controlled.
- Bank above parking lot should be limed & fertilized based on soil test.

6. EMBANKMENT, STRUCTURAL, & OTHER DRAINS

<table>
<thead>
<tr>
<th>Depth of Flow (in inches above invert)</th>
<th>Dam left right</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>With any obstruction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without any obstruction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Turbidity of Discharge (yes, no)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>With any obstruction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without any obstruction</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition of Protective Coating</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outside</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inside</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Obstruction in Flow (yes, no)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Animal Guard Condition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outlet Condition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Retarding Pool Elevation (ft. msl)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(ft. above)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

COMMENTS
- Seepage around riser.
### 3. EMBANKMENT AND EXCAVATED SLOPES

(Report riprap and vegetation and erosion condition under Items 4 and 5.)

<table>
<thead>
<tr>
<th></th>
<th>Dam</th>
<th>Others</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding or sloughing</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Holes (rodent and other)</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>(check especially at embankments)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excessive settlement (embankments)</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Cracks</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Traverse</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Longitudinal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seepage 2/</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Piping 2/</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**COMMENTS:** Could be gouging by ice.

**Upfill:** 200' east of east at 40' long-top about halfway upfill. 2nd area 120' LT at PI of curve. 2 holes burrowing downstream face (right) opposite rock rib outlet. Vehicle tracks should be filled in, particularly those running up & down slope.

### 4. RIPRAP

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Dam</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upstream berm</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Principal Spillway Outlet</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Embankment Gutters left</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Emergency Spillway location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Right.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waterways location</td>
<td>FLOODWAY CHANNEL</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outlet Channel</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**COMMENTS:** Riprap around riser displaced. Some minor displacement in riprap on upstream face along floodway.

---

1/Looking downstream.  
2/Check especially at downstream face of embankments.
This maintenance checklist is a guide for determining the maintenance required for Public Law 506 flood control structures in New Hampshire. It doesn't take the place of experience and judgment and is not inclusive. Items of a difficult nature to check, such as principal spillway conduit condition, are not included. Intensive checks of these items are necessary at proper intervals. Review of As Built drawings, the design folder, structure history, and previous maintenance reports should be part of the inspection. Prompt maintenance is a vital part of safe and effective operation.

Except where otherwise indicated, completion of this form may be facilitated by ranking maintenance items on a 1 to 4 basis where

1 = satisfactory
2 = satisfactory, but check carefully at next inspection
3 = requires maintenance this season
4 = requires immediate attention.

<table>
<thead>
<tr>
<th>WATERSHED</th>
<th>DEAD RIVER</th>
<th>SITE</th>
<th>DATE</th>
<th>INSPECTED BY</th>
</tr>
</thead>
</table>

1. GENERAL ITEMS

- Access Road
- Site Fencing
- Traffic Conditions
- Vandalism Control
- Trash Control

**COMMENTS:**
- **UPSTREAM FACE OF DAM:** EMBR. SPHWY FLOODWAY
- **DOWNSTREAM EASE:** Hole under crane 4" wide 3" depth, flowing full, travel access top of dam, old road to dam not fenced as required in access road construction

2. RESERVOIR

- Timber stand at reservoir
- Debris and slash
- Sediment level in relation to low stage inlet

**COMMENTS:** Water along beach look clearer this year.

5/77

SOIL CONSERVATION SERVICE
### 8. IMPACT BASIN, SAF, BOX INLET, & MISCELLANEOUS CONCRETE STRUCTURES

(specify) Recirculating System - Inlet and dam

<table>
<thead>
<tr>
<th>Concrete: inside and out</th>
<th>Cracking 1; Spalling 1; Other deterioration 1; Excessive movement (check joints) 1; Waterstops 1; Joint sealant 1; Other 1.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Trashracks: low and high stage</td>
<td>Condition of protective coatings 1; Corrosion 1; Damaged parts 1; Condition of fastenings 1; Need of gratings due to beaver 1; Safety condition (protruding fastenings, sharp edges, etc.) 1; Other 1.</td>
</tr>
<tr>
<td>Gates: including lifting device, stem, guides, disc, flap</td>
<td>Condition of protective coating 2; Corrosion 2; Damaged parts 2; Condition of fastenings 2; Stem alignment 2; Operation 3; Lubrication 2; Wood decay 2; Other 2.</td>
</tr>
<tr>
<td>Structure Drainage:</td>
<td>Report under &quot;Embankment and Other Drains&quot;</td>
</tr>
<tr>
<td>Structure, Railing, Grates, Barriers, etc.</td>
<td>Condition of protective coating 2; Corrosion 2; Damaged parts 2; Condition of Fastenings 2; Wood decay 2; Safety condition (protruding fastenings, sharp edges, etc.) 2; Other 2.</td>
</tr>
<tr>
<td>Safety Items:</td>
<td>Condition of warning signs 2; Condition of safety equipment 2; Other 2.</td>
</tr>
</tbody>
</table>

**COMMENTS**

---

### 9. CHANNEL

<table>
<thead>
<tr>
<th>Stream obstructions</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Debris in stream</td>
<td>1</td>
</tr>
<tr>
<td>Sediment bars controlled</td>
<td>1</td>
</tr>
<tr>
<td>Plunge pool stability</td>
<td>1</td>
</tr>
<tr>
<td>Fish habitat appurtenances</td>
<td>---</td>
</tr>
<tr>
<td>Riprap -- Report under &quot;Riprap&quot; (item 4)</td>
<td></td>
</tr>
</tbody>
</table>

**COMMENTS**

---
Caution: Be extremely careful when using ladders. Check condition before using. Ladders are sometimes broken, loose, corroded, and or slippery. Use safety harness.

Ladders: None Condition of protective coating_; Corrosion_; Damaged parts_; Loose_; Other_.

Concrete: Cracking 0.; Spalling 0.; Other deterioration 0.; Excessive movement (check joint at riser and conduit) 0.; Other 0.

Trashracks: Condition of protective coatings_; Corrosion 0.; Damaged parts_; Condition of fastenings 1.; Need of gratings due to beavers 0.; Safety condition (protruding fastenings, sharp edges, etc.) 1.; Other 0.

Manhole: Condition of protective coatings 3.; Corrosion _; Damage_; Lock operable_; Other_.

Gate: Condition of protective coating_; Corrosion _; Damaged parts_; Condition of fastenings_; Stem alignment_; Lubrication_; Operation_; Other_.

Safety Items: Condition of warning signs_; Condition of safety equipment_; Other_.

COMMENTS: Paint manhole cover.

INLET TO RECIRCULATING PIPE SYSTEM - Remove debris from trash rack. (3)
5. **VEGETATION**

<table>
<thead>
<tr>
<th>Condition of stand (including need for lime and fertilizer)</th>
<th>Dam</th>
<th>Left</th>
<th>Right</th>
<th>Dike</th>
<th>Outlet Channel</th>
<th>Waterway</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Desirable vegetation</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drainage (surface)</td>
<td>3</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Erosion</td>
<td>3</td>
<td>3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sedimentation</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Condition of planting</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pest control</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fire control</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**COMMENTS**
Pockets need to be drained and revegetated. Remove woody plants from the dam. Bare spots should be topdressed and reseeded. There is erosion from damage to vegetation by ATV's. Remove the woody vegetation from the dike.

---

6. **EMBANKMENT, STRUCTURAL, & OTHER DRAINS**

<table>
<thead>
<tr>
<th>Depth of Flow (in inches above invert)</th>
<th>Dam left</th>
<th>Dam right</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>With any obstruction</td>
<td>None</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Without any obstruction</td>
<td>1/2</td>
<td>1/2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Turbidity of Discharge (yes, no)</th>
<th>With any obstruction</th>
<th>No</th>
<th>No</th>
</tr>
</thead>
<tbody>
<tr>
<td>Without any obstruction</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Condition of Protective Coating</th>
<th>Outside</th>
<th>Inside</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>2</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Obstruction in Flow (yes, no)</th>
<th>No</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Animal Guard Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Outlet Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Retarding Pool Elevation (ft. msl)</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>or (ft.) above msl below</td>
<td></td>
</tr>
<tr>
<td>Other</td>
<td>1</td>
</tr>
</tbody>
</table>

**COMMENTS**

---

1/Looking downstream.
2/Including runs, surface, stream, marshes, and livestock crossings.
3. EMBANKMENT AND EXCAVATED SLOPES

(Report riprap and vegetation and erosion condition under Items 4 and 5.)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Dam</th>
<th>Dike</th>
<th>Left</th>
<th>Right</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sliding or sloughing</td>
<td>2</td>
<td>1</td>
<td>2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Holes (rodent and other)</td>
<td>3</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>(check especially at embankments)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Excessive settlement (embankments)</td>
<td>3*</td>
<td>1</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cracks</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Traverse</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Seepage</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Piping</td>
<td>1</td>
<td>1</td>
<td>2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

COMMENTS: Ruts in the spillway should be filled. If the spillway operates, ruts in the outlet channel will accelerate erosion. The city should consider fencing or erecting barriers to keep ATVs out of the spillway and off the dam.

*Fill in low areas on top of the dam.

4. RIPRAP

<table>
<thead>
<tr>
<th>Location</th>
<th>Displ. of Rock</th>
<th>Loss of Spalls</th>
<th>Loss of Bedding</th>
<th>Erosion of Found.</th>
<th>Breakdown of Rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream berm</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Principal Spillway Outlet</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Embankment Gutters</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>left</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>right</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Emergency Spillway</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waterways</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flooding Br.</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>location</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outlet Channel</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>Other</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

COMMENTS

1/Looking downstream.  
2/Check especially at downstream face of embankments.
An operation and maintenance inspection of Site 1 (Jericho Lake) in the Dead River Irrigated, Berlin, N. H., was conducted on June 17, 1975. The inspection was made by representatives from the local sponsoring organization and the Soil Conservation Service. Included were:

James Smith, City Manager  
Maurice Theler, Acting City Engineer  
Mitchell Berkowitz, Berlin Recreation Director  
Calvin Perkins, District Conservationist, USDA, SCS  
Charles Dietz, USDA, SCS  
Wallace Jolly, USDA, SCS  
Thomas Rogers, USDA, SCS  
Rick Luhtala, USDA, SCS

Following is a list of items discussed and the necessary action needed:

1. Remove the small amount of debris from the principal spillway trash rack and along the rip-rap material of the dam.

2. Cut the alders and other woody plant growth that is encroaching the emergency spillway and in a few spots along the dam.

3. All roadways through the emergency spillway, along the downstream side of the dam, and across the top of the dam should be seeded and mulched to provide these areas with a vegetative cover.

4. The upstream slope of the dam should be fertilized to encourage a more dense vegetative cover. Apply 10 lbs. per 1000 square feet of 15-10-10 commercial fertilizer. Application should be made as soon as possible to obtain maximum benefit this growing season.

5. The borrow area where the new access road is being constructed is also void of desirable vegetative cover. This area should also be fertilized, but the slope of the dam is the most critical at this time.

6. It was noted the repairs to the spillway on top of the concrete riser have been made.
UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Picke Building, 99 Main St., Lancaster, NH 03524

SUBJECT: Operations and Maintenance Inspection - Dead River Watershed

DATE: May 21, 1974

TO:
Mr. James Smith
City Manager
City Hall
Berlin, New Hampshire 03570

Dear Mr. Smith:

Attached is the annual Operations and Maintenance Inspection Report for the Dead River Watershed conducted on May 17, 1974.

The report lists items that require attention by the City as their obligation under the Operation and Maintenance Agreement of August 26, 1968.

I trust that you will see that these actions are carried out and following completion will send me a letter to that effect.

Sincerely,

Calvin J. Perkins
District Conservationist

cc: M. Wheeler
    C. Dirle
    W. Nelson
    K. MacPherson
An operation and maintenance inspection of Site 1 (Jericho Lake) in the Dead River Watershed, Berlin, NH, was conducted on May 17, 1974. The inspection was made by representatives from the local sponsoring organization and the Soil Conservation Service. Included were:

- Maurice Wheeler, Assistant City Engineer
- Mitchell Berkowitz, Berlin Recreation Department
- Gus Kozar, Member, Berlin Recreation Commission
- Keith MacPherson, Design Engineer, USDA, Soil Conservation Service
- Calvin J. Perkins, District Conservationist, USDA, Soil Conservation Service

Following is a list of items discussed and the necessary action needed:

1. Remove the accumulation of debris from the principal spillway trash rack and along the rip-rap material of the dam and in the approach to the emergency spillway.

2. Remove the accumulation of bottles and litter at the trash barrels near the top of the dam.

3. Repair the two areas of concrete spalling on top of the principal spillway. Directions for making this repair are contained in a letter from Mr. Dingle of SCS to Mr. Brungot, City Engineer, dated June 26, 1973. (Copy attached)

4. Remove sediment from the entrance to the water circulation pipe at the water control structure.

5. Repair and vegetate wheel track ruts on the downstream slope of the dam to prevent further erosion.

6. Keep vehicles off dam by placing large boulders close together at the pipeline end of dam. Once vehicle traffic across top of dam has stopped then vegetate the top of dam.

7. The most serious item is to develop better vegetative cover in the emergency spillway area and borrow areas adjacent to the spillway. This area is approximately 20 acres in size and should be fertilized to promote grass growth. Apply 250 lbs./acre of 15-10-10 or closest equivalent. This application should take place immediately to be effective in this growing season. If this is not done erosion will begin to occur and should the emergency spillway function severe problems could occur without the presence of good vegetative cover.

8. A visit to the site indicated that no maintenance was required at this time.
Mr. Sylvio Grotcan, Mayor
City of Berlin
City Hall
Berlin, N. H. 03570

Dear Mr. Grotauc:

On October 1, 1973 Soil Conservation Service personnel and Maurice Wheeler representing Clarence Brunot visited the Jeddo River Site #1, Jeddo Lake. During the visit several operational observations were made that require the immediate attention of the city. They are as follows:

1. A stanchion bracket is missing on the inside of the concrete riser. This needs repair or replacement to assure proper functioning of the sluice gate. This equipment was made by:

   Coldwell - Wilcox Division
   S. S. Rockwell Co.
   200 Fliot Street
   Fairfield, Conn. 06430

   The missing part is a cast iron stanchion bracket, Part # A-207, Drawing No. SK-13b8, (62-I-652, Class 30). Refer to Drawing No. C-57-530 of the original specifications. Please have repaired as soon as possible.

2. There are two areas of spalling on top of the concrete riser. These should be repaired and a letter of June 26, 1973 to Clarence Brunot explaining the details of making the repairs. (Copy attached.)

3. The stop logs should be removed from the diversion channel during the winter months. The sections of old metal pipe that have floated into the diversion channel should be removed.

4. The water storage at the site should be sufficient to cover the recreation circulation pipe with at least 6 feet of water before winter to prevent freezing of this pipe system.

B-20
I trust that you will see that this work is performed promptly to assure proper functioning of the flood control structure. If I can be of any assistance please contact me.

Sincerely,

[Signature]

Calvin J. Perkins
District Conservationist

CC: Clarence Brummet, City Engineer, Berlin
Charles Hixley, SCS, Durkan
Valt Nelson, SCS, Durkan
May 21, 1973

Mr. Clarence Brungot
City Engineer
Public Works Department
City Hall
Berlin, N. H. 03570

Dear Mr. Brungot:

Since the City of Berlin will soon be having a change in the City Manager position I am sending the attached annual Operations and Maintenance Inspection Report for the Dead River Watershed to you.

The report lists items # 1, 2, 3, 4, 5, 6, that require action by the City of Berlin.

I trust that you will see that these actions are carried out and following completion will send me a letter to that effect.

Sincerely,

Calvin J. Perkins
District Conservationist

CJP:

cc: C. Dingle, Asst. State Conservationist (W), SCS, Durham, N. H.
W. Jolly, State Conservation Engineer, SCS, Durham, N. H.
W. Nelson, Soil Conservationist, SCS, Durham, N. H.
OPERATIONS AND MAINTENANCE INSPECTION REPORT
of Site 1 (Jericho Lake)
Dead River Watershed Project
May 22, 1973

An operation and maintenance inspection of Site 1 (Jericho Lake) in the Dead River Watershed was conducted on May 22, 1973. The inspection was made by representatives from the local sponsoring organization and the Soil Conservation Service. Included were:

Clarence Brunnot, Berlin City Engineer
Hannice Wheeler, Assistant to City Engineer
Walter Nelson, USDA, Soil Conservation Service
Calvin J. Perkins, USDA, Soil Conservation Service

Following is a list of items discussed and the necessary action needed.

1. Remove heavy debris accumulation from principal spillway trash rack and on the rip-rap material along the dam and in the approach to the emergency spillway.

2. Two areas on top of the concrete riser (each about 2 square feet in area) were showing evidence of spalling. These areas were from 1/8 to 1/3 inch deep and had water within. Cause of this action is unknown and correction procedure was not decided upon.

3. In general the vegetation looked better than last year. However, it is apparent that legumes will not grow on the exposed upstream slope of dam. Grasses in this area and other flat areas is present but appeared to need fertilization. Suggested fertilizing a total of 11 acres with 20-10-10 at a rate of 200 lbs./acre. Downstream slope of dam has good legumes and grass cover.

4. Clean debris from water control structure, retrieve stop logs that are in downstream channel, and remove sections of old steel pipe in upstream and downstream channel.

5. Slight bank erosion at end of rip-rap on left side of channel below water control structure. This should be watched closely and if conditions increase the rip-rap should be extended.

6. Old road along base of dam should be seeded to prevent erosion and discourage use of this area by unauthorized vehicles.

7. No vegetation on top of dam, however, it appeared that this was causing no problem.

8. Impossible to control vehicle traffic to the site. Cable at gate on Route 110 is destroyed as fast as it is installed.
9. Although records indicate that a plaque was ordered by Water Resources Board in 1971 there is no evidence of installation. The question of who is responsible for installation was brought up but could not be answered.

10. Inclement weather prohibited a visit to the dike area. Mr. Brunsot stated that the dike was in good vegetative cover and that brush was not encroaching on same.
June 26, 1972

Mr. Joseph Burke
City Manager
City Hall
Berlin, N. H. 03570

Dear Mr. Burke:

Attached is the annual Operations and Maintenance Inspection Report for the Road River Watershed conducted on June 20, 1972.

The report lists items #1 and 2 that require maintenance work by the local sponsors.

Following completion of the work please forward me a letter to that effect.

Sincerely,

Enclosure

Calvin J. Perkins
District Conservationist

cc: C. Brungot, Public Works Superintendent, Berlin, N. H. 03570
    G. Bingle, Asst. State Conservationist (II), SCS, Durham, N. H. 03824
Operations and Maintenance Inspection Report
of Site 1 (Jericho Lake)

Dead River Watershed Project

JULY 20, 1972

An operation and maintenance inspection of Site 1 (Jericho Lake) in the Dead River Watershed was conducted on June 20, 1972. The inspection was made by representatives from the local sponsoring organization and the Soil Conservation Service. Included were:

Mayor Sylvio Croteau, Berlin
Joseph F. Burke, Berlin City Manager
Maurice Wheeler, Berlin
Calvin J. Perkins, USDA, Soil Conservation Service

Following is a list of items discussed and the necessary action needed.

1. Remove debris from principal spillway and on the rip-rap material along the dam.

2. Remove stones under planks at the water control structure in the Jericho Brook channel plus the section of metal pipe that floated downstream and lodged at the structure.

3. Vegetation is not good on upstream slope of dam but may improve during summer growth period. Good vegetation on downstream slope of dam. Winter kill of vegetation is apparent on the downstream flat areas and in the emergency spillway and adjacent borrow area. There is no present damage being done by the lack of vegetation cover but this bears a close watch.

4. The top of the dam is being used extensively by vehicular traffic of various types. They enter onto the dam via the City pipeline and this type of traffic is almost impossible to control. All traffic at the main entrance to the site has been stopped except for ATV and similar two-wheeled transportation. With so much traffic on the top of the dam it is void of vegetation but this is not causing any problems at the present time.
OPERATIONS & MAINTENANCE INSPECTION REPORT
OF SITE 1 (JERICHO LAKE)
DEAD RIVER WATERSHED PROJECT

June 9, 1971

An operation and maintenance inspection of Site 1 (Jericcho Lake) in the Dead River Watershed was conducted on June 9, 1971. The inspection was made by representatives from the local sponsoring organizations and the Soil Conservation Service. Included were:

Clarence Enngot, City of Berlin
Forrest Hodgdon, N. H. Water Resources Board
Peter Barkes, N. H. Water Resources Board
Henry Stackel, USDA, Soil Conservation Service
Charles Brown, USDA, Soil Conservation Service
Ray Werninger, USDA, Soil Conservation Service
Edward Hutchinson, USDA, Soil Conservation Service
Charles Holden, USDA, Soil Conservation Service
Walter Zwearcan, USDA, Soil Conservation Service

Following is a list of items discussed and the necessary action needed.

1. The dike was in good repair with an excellent vegetative cover established. Woody plant growth should be controlled in the future by either moving or use of spray.

2. Trash to be cleared away from the principal spillway and along the dam at the water edge. The N. H. Water Resources Board agreed to take care of this item.

3. Two washouts on the back side of the dam above the principal spillway pipe need repair. The City of Berlin agreed to make this repair by filling and seeding the washout holes. Also to remove stones and smooth out material down slope from the washouts.

4. Inspection of dam, floodway, emergency spillway and borrow area indicated no need for maintenance at this time.

Being early in the growing season, it was difficult to determine the quality of the grass cover. It is possible that there will be a need for fertilizing some of these areas another year.
5. There was no vegetative cover in wheel tracks over the entire length of the dam. This condition will continue to exist until vehicle traffic can be controlled. The condition, however, was not considered harmful to the dam.
Dear Mr. Brungot:

Attached is the annual Operations and Maintenance Inspection Report for the Dead River Watershed conducted on June 9, 1971. The report lists two items on Site 1 that require maintenance work by the local sponsors.

Following completion of the work, please forward me a Letter of Certification and the approximate cost for doing the work. The cost information will be helpful to us in estimating operation and maintenance in future projects.

Sincerely,

Edward F. Hutchinson
District Conservationist

Enclosure - O&M Inspection Report
Site 1, Dead River Watershed Project

cc: Coos Conservation District, Lancaster, N.H.
    C. Dingle, Asst. State Conservationist (W), Soil Conservation Service, Durham, New Hampshire

USDA: SCS: E. F. Hutchinson: hs
Piche Building, 99 Main Street, Lancaster, New Hampshire 03524

June 30, 1971

Mr. Peter Merkes
New Hampshire Water Resources Board
Concord, New Hampshire 03301

Dear Mr. Merkes:

Attached is the annual Operations and Maintenance Inspection Report for the Dead River Watershed conducted on June 9, 1971. The report lists two items on Site 1 that require maintenance work by the local sponsors.

Following completion of the work, please forward me a Letter of Certification and the approximate cost for doing the work. The cost information will be helpful to us in estimating operation and maintenance in future projects.

Sincerely,

Edward F. Hutchinson
District Conservationist

Enclosure - O&M Inspection Report
Site 1, Dead River Watershed Project

cc: C. Dingle, Asst. State Conservationist (W), Soil Conservation Service, Durham, New Hampshire

USDA: 503: E. F. Hutchinson: ha
I. Watershed Data
   A. Structure Class .............................................. C
   B. Drainage Area .............................................. 41.47 Ac.
   C. Time of Concentration - Tc ................................ 4.7 Hrs.
   D. Hydrologic Curve Number - Cn
      1. Moisture Condition II* .................................. 68
      2. Moisture Condition III* .................................. 88

II. Principal Spillway
   A. Conduit
      1. Inside Dia .............................................. 42.5 In.
      2. Length ................................................... 4.33 Ft.
   B. Riser
      1. Inside Dimensions ....................................... 3.5 x 0.5 Ft.
      2. Height (Floor to Crest) ................................ 29.0 Ft.
   C. Weir Length ................................................. 41.0 Ft.
   D. Orifice Dimensions ......................................... 3.0 x 0.72 In.
   E. Reservoir Drain Size ....................................... 70.0 In.
   F. Type of Energy Dissipater ................................ Plunge Pool

III. Emergency Spillway
   A. Width .......................................................... 150 Ft.
   B. Side Slopes ................................................... 6:1 EARTH / 1:6 SLOP.
   C. Length of Level Section .................................... 80 Ft.
   D. Exit Slope .................................................... 6:1
   E. Max. Velocity in Exit Section D H. W. .................. 2.3 Ft./Sec.
   F. Duration of Flow thru Emer. Spillway D H. W. .......... 4.5 Hrs.
   G. Frequency of Use ........................................... LESS THAN ONCE IN 100 YRS.

IV. Earth Fill
   A. Height .......................................................... 46 Ft.
   B. Volume ......................................................... 56 C Y.
   C. Compaction .................................................... ZONE A CLASS A

Fill Placement

*AS PER WORK PLAN
This multiple-purpose dam is located in Coos County, New Hampshire, on the Jerico Brook tributary of the Dead River. The site is approximately four miles northwest of Berlin, New Hampshire.

Sheet 4 of this report, together with the Mt. Washington, New Hampshire, 15-minute quadrangle published by the U. S. Geological Survey, may be used to locate the structure more definitely.

A summary of pertinent design information is given on sheet 2 of this report.

This is the only proposed floodwater retarding dam in the Dead River Watershed. This will be a multiple-purpose structure designed to retard a 100-year frequency storm without discharge occurring in the emergency spillway.

The results of the hydrologic and hydraulic computations are given on sheet 3 of this report.

The primary structure consists of a zoned compacted earthfill with a cutoff through alluvium, glacial till, and decomposed rock to firm bedrock. A drainage system is located under the downstream portion of the earthfill to control the phreatic surface and to collect seepage.

The principal spillway is a drop inlet, closed conduit structure consisting of a two-stage, reinforced, concrete riser, 2-inch diameter reinforced concrete water pipe, and a riprapped plunge pool.

The emergency spillway is designed as an earth and rock cut in the left abutment. An engineering cost analysis was performed which resulted in the least combined cost of fill and emergency spillway rock excavation.
DESIGN REPORT
DEAD RIVER
WATERSHED PROJECT
SITE 1
COOS COUNTY
NEW HAMPSHIRE
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

INDEX
GENERAL
LAYOUT
HYDRAULICS
GEOLOGY
SOIL TESTING
E. & F. DESIGN
STRUCTURAL
QUANTITIES
SPECIFICATIONS
Notification No. 4 was issued to provide for over-runs in common excavation, rock excavation, and flooding rock excavation.

Modification No. 5 was issued to provide for over-runs in concrete for the circulation pipe and rock excavation for the circulation pipe.

Modification No. 6 was issued to provide for the construction of a concrete cut-off wall adjacent to the left pier of the drop structure. This was deemed necessary to minimize abutment leakage around the drop structure.

Modification No. 7 was issued to provide for an over-run in riprap for the diversion channel.

Modification No. 8 was issued to allow compensation for a claim submitted by the contractor on Bid Item 8 and Bid Item 16. The contractor based his claim on changed conditions on both items. Bid Item 8 was excavation for the circulation pipe. Backfilling of this item was subsidiary to the excavation item. It was designated that the excavation be backfilled with selected granular material around the pipe. The material contained a great deal of cobbles and boulders making it extremely difficult to provide suitable granular backfill. Granular backfill was obtained from another source with Modification No. 8 providing compensation for additional cost incurred by increased haul distance.

Bid Item 16 was earthfill. Bedrock was exposed above grades shown on the plans creating a shortage of fill. The borrow area had to be expanded in order to obtain the necessary material. The contractor claimed a changed condition due to the presence of the ledge and filed for compensation due to lost efficiency and extra wear on equipment, etc., due to working on bedrock and the confined area of the expanded borrow area. Compensation was granted for approximately one-half of the amount claimed. Presently, the Contracting Officer's final decision is pending. It appears as though the contractor will have to go to the courts in order to obtain any further compensation.

The contractor completed the job well within the allotted time. He actually worked during a 206 calendar day period cut of 341 calendar days.

Arthur N. Luhtala
Project Engineer

ANLUHTALA:pak
At first the contractor attempted to place the drain fill around a form constructed cut of sheet steel and angle iron that was to be dragged along the trench by a bulldozer. This method did not prove to work. It was extremely difficult to keep the form moving along the centerline of the trench and when there was drain pipe in the form the friction of drain fill particles tended to keep the entire unit moving as one solid mass. This approach was then abandoned and plywood panels were used to complete the job.

The drain fill was supplied by Lessadi Sand and Gravel from Gorham, New Hampshire, although the fine drain fill was trucked from Guildhall, Vermont.

The principal spillway pipe was furnished by Interpace Corporation, Wharton, New Jersey. The pipe was a 42" pre-stressed cylinder pipe (SP-5) double wrap.

Northeastern Culvert Corporation of Westminster, Vermont, supplied the bituminous coated corrugated metal pipe and the trash racks.

Bancroft and Martin, Inc., of South Portland, Maine, supplied the reinforcing steel, cast iron and asbestos cement pipe along with the appurtenances for the circulation system.

Coldwell-Wilcox Company, a division of W. S. Rockwell Company, Fairfield, Connecticut, supplied the back pressure gate.

The R. L. Lecaron Foundry Company of Brockton, Massachusetts, supplied the manhole assembly.

The quality of furnished materials was very good.

The contractor's overall performance was excellent. The experience, equipment, organization and supervision provided was of high quality. The relations between ES3 and the contractor were excellent. The service should not hesitate in awarding future contracts to this contractor.

Eight (8) modifications were issued on this job. Modification No. 1 resulted due to an over-run in the quantity of reinforcing steel and a change in drain trench dimensions. The width of the drain trench had to be increased in order that the flow area required by design be maintained.

Modification No. 2 was issued to apply vegetative treatment on the dike and related areas.

Modification No. 3 was issued to provide plans for the access tee of the circulation pipe. These were apparently overlooked in quantity summaries.
Available land rights forced the borrow pit extension into an area where excavation was very difficult. This area was located on the northwest border of Borrow Area A. Numerous boulders and the presence of bedrock caused the extreme difficulty. If it had been possible to extend the pit in a northerly direction, I believe that this would have been a more favorable source. This area was not as confined and it appeared as though there was less oversize material present.

In future contracts that require considerable quantities of fill, I suggest that borrow areas be designed to supply at a minimum one and one half times the estimated fill requirement. It is easier to make a borrow area smaller during construction operations than it is to make it larger.

Shrinkage factors, excess amounts of unsuitable material from borrow pits, and other unforeseen factors warrant the need for adequate quantities of available borrow on any particular job.

The concrete was supplied by Brideau Ready Mix of Berlin, New Hampshire.

Six (6) concrete cylinders were broken after seven (7) days with an average strength of 3,800 psi. The minimum seven day break was 2,860 psi and the maximum was 3,750 psi.

Eighteen (18) concrete cylinders were broken after twenty-eight (28) days with an average strength of 5,310 psi. The minimum twenty-eight day break was 3,600 psi and the maximum break was 5,340 psi.

The drainage system for this job consisted of a four (4) foot wide trench drain with a double drain filter. The coarse drain fill was encased by an eight (8) inch layer of fine drain fill. It was extremely difficult to maintain this eight-inch layer with any uniformity due to the nature of the existing foundation soils. Numerous boulders and cobbles are typical of many New Hampshire soils. In the future I suggest that designers allow a minimum of one (1) foot for fine drain fill in a double filter design. This would be much easier to control in the field, allowing placement with less chance of contamination. Contamination appeared to be the most difficult thing to control in placement of the fine drain fill.

Plywood forms were utilized in placement of the drain fill. As drain fill was placed around the plywood sections, the forms would be raised and back filled again. A gradall proved to be an extremely versatile machine for this operation because of the ability to tilt the bucket in all directions.
Borrow sources on the job were infested with boulders and oversize material. Specifications allowed the use of material up to eighteen (18) inches in size. A large vibratory roller, as specified by construction specifications, attained required compaction with reasonable efficiency.

The material in borrow areas was very dense. Double pushing of scrapers and on occasions triple pushing was utilized in loading operations. Along with ripping this provided for an efficient earth moving operation. It is estimated that ten thousand (10,000) cubic yards per day was moved on occasion. The average production was in the range of thirty-five hundred (2500) cubic yards per day.

A total of seventy-six (76) compaction tests were made. Eleven (11) tests showed compaction below an acceptable density. Most of the unsatisfactory tests resulted with material being on the dry side of optimum. This necessitated adding water to the fill to obtain satisfactory densities.

Zone 1 required a minimum density of 95% of Standard Proctor. Eighteen (18) tests were taken with only two (2) failing. The average of all acceptable densities was 101.2% of Standard Proctor.

Zones 2, 3, 4 and 5 required class V compaction as defined by construction specification 5. In the field this was evaluated by Standard Proctor control due to the nature of the material. The soil had enough silt so that a definite sharp created Proctor curve could be established. Ninety-five (95) percent of Standard Proctor was considered as acceptable.

In Zones 2 and 3, fifty-two (52) tests were taken with only eight (8) failing to meet 95%. The average of all acceptable densities was 103.5% of Standard Proctor.

In Zones 4 and 5, of the dike, six (6) tests were taken with only one (1) test failing. The average of all acceptable densities was 105.5% of Standard Proctor.

A shortage of earth fill developed on the job. Bedrock appeared at elevations above the grade shown in the plans in Borrow Area A, the major borrow source, thus decreasing the amount of available fill. Another factor was that approximately seventeen thousand (17,000) cubic yards of rock had to be removed from the foundation instead of an anticipated two thousand (2,000) cubic yards. This resulted in excavating Borrow Area A deeper than expected along with having to extend the pit beyond the limits indicated on the plans.
Henry S. Stamato1
January 26, 1971
Page 2

The contract provided for two hundred and eighty (280) calendar days of performance time to complete the work. Work commenced on May 26, 1969. A winter shutdown was issued on December 11, 1969, with approximately ninety-nine (99) percent of the work complete. The contractor used just over sixty-eight (68) percent to perform this amount of work. Performance time again commenced on June 15, 1970, with all remaining construction being completed between August 17, 1970, and August 28, 1970. The final value of the contract was $651,100.59.

In addition to winter shutdowns, the contractor's work was suspended because of weather for a total of five (5) days, all occurring during November 1969. The contractor was also granted sixty-one (61) additional days of performance time due to contract modifications.

Personnel directly concerned with the work were as follows:

Vernon A. Knowlton, Contracting Officer, New Hampshire Water Resources Board, Concord, New Hampshire
Norman A. McDonald, Project Manager, Rogers Construction Co., Inc., Brattleboro, Vermont
Glen Gibs, General Superintendent, Rogers Construction Co., Inc., Brattleboro, Vermont
Arthur N. Luhtala, Government Representative and Project Engineer, Soil Conservation Service, Berlin, New Hampshire

Other personnel were assigned to the job from time to time as required for assistance in inspection and layout work.

The contractor furnished good equipment for the job. The major items of equipment included:

a. Crawler tractors: Two D-3H's with ripper attachments, 82-h0, D-7, D-5, JD-450, and other tractors as needed.
b. Rubber-tired dozers, Cat. 834-B and 834-B.
c. Front end loader, Cat. 530.
e. SPV-7 vibratory roller.
f. Cat. 14G round grader.
g. L-Tex 73-24 double barreled scrapers.
h. Gardner Denver 940 cfm wagon drill.
i. 6 large dump trucks (10-15 cu. yd. capacity).
j. Miscellaneous equipment such as a water wagon, pulp, small compressor, vibrating and trenching hand compactors, chain saws, sand blasting equipment, and other necessary equipment as required.
Plymouth, N.H.

S2G - Construction Report, Site No. 1, Dead River Watershed

January 26, 1971

Henry S. Starzel
State Conservation Engineer
Soil Conservation Service
Durham, New Hampshire 03824

The location of this site is in northern New Hampshire in the city of Berlin, Coos County. This dam is a multiple purpose dam with recreation being the secondary purpose. The job was contracted by the New Hampshire Water Resources Board under Project Agreement No. 12-10-270-75. The City of Berlin and the Coos County Soil Conservation District were sponsors of the work.

The City of Berlin cost shared 23.16% of the costs incurred by construction of the dam and dike, with the Soil Conservation Service paying the remaining 76.84% of the costs. The City of Berlin and SCS cost shared on a 50-50 basis on construction of the circulation system which is considered as a basic recreation facility. The roadway was constructed with SCS paying 100% of the costs because the intended nature is specifically for flood control purposes.

A public viewing of the site was conducted on January 21, 1969. The unique thing about this viewing was that snow machines were required for access to this remote site. Snow depths of seven feet or more on the level were reported at the time of the site showing.

Bids for this job were opened on February 7, 1969. The following shows the bidders and the amount of their respective bids:

- Rogers Construction Co., Inc., Brattleboro, Vt. $561,692.39
- Caledonia, Inc., St. Johnsbury, Vermont 736,756.00
- Cappy-Simone, Inc., Freeport, Long Island, N.Y. 747,913.10
- Welch & Corr Construction Corp., West Springfield, Massachusetts 774,317.30
- Weaver Brothers Construction Co., Inc., Concord, NH 831,856.05

The engineer's estimate was $566,286.55

Contract No. W2G-SSS-36 was awarded to Rogers Construction Company, Inc. of Brattleboro, Vermont, on February 27, 1970.
V \hspace{1cm} \textbf{RECORDS}

The Sponsor will maintain in a centralized location a record of all inspections performed both individually and jointly by the Sponsor and the Service, and of all significant actions taken by the Sponsor with respect to operation and maintenance. The Service may inspect these records at any reasonable time.

VI, \textbf{GENERAL}

A. The Sponsor will:

1. Prohibit the installation of any structures or facilities that will interfere with the operation or maintenance of the structural measures.

2. Obtain prior Service approval of the Plans and Specifications for any alteration or improvement to the structural measures.

3. Obtain prior Service approval of any agreement to be entered into with other parties for the operation or maintenance of all or any part of the structural measures, and provide the Service with a copy of the agreement after it has been signed by the Sponsor and the other party.

B. Service personnel will be provided the right of free access to the structural measures at any reasonable time for the purpose of carrying out the terms of this Plan.

C. The responsibilities of the Sponsor under this Plan are effective simultaneously with the acceptance of the works of improvement in whole or in part.
III ESTABLISHMENT PERIOD (continued 2)

resulting from major erosion damage, (h) major revegetation due to failure to obtain an adequate vegetative cover, and (5) restoring areas with significant erosion caused by unusual flow (volume, recurrence or extended period of time) in emergency spillways.

F. No action with respect to needed repairs during the Establishment Period will be taken by the Sponsor or the Service which would lessen or adversely affect any legal liability of any contractor or his surety for payment of the cost of the repairs.

IV INSPECTIONS AND REPORTS

A. During the Establishment Period the Sponsor and the Service will jointly inspect the structural measures at least annually and after unusually severe floods or the occurrence of any other unusual condition that might adversely affect the structural measures. It is desirable the annual inspections be performed during the month shown below. Any supplemental inspections then determined necessary will be scheduled and agreed to at that time.

May
(Month)

B. After the Establishment Period the structural measures will be inspected annually by the Sponsor, preferably during the month shown below, and after unusually severe floods or the occurrence of any other unusual condition that might adversely affect the structural measures.

May
(Month)

C. After the Establishment Period the Service may inspect the structural measures at any reasonable time.

D. A written report will be made of each inspection. The report of joint inspections will be prepared by the Sponsor with the assistance of the Service. A copy of each report will be provided by the party preparing the report to the other party within ten days of the date on which the inspection was made.
III ESTABLISHMENT PERIOD (continued)

2. Repairs determined by the Service to have been occasioned by improper operation or maintenance, or both.

3. Repairs applicable to municipal or industrial water supply or to any other purpose for which construction costs are not authorized to be paid for in whole or in part with funds appropriated to the Service.

4. Repairs that are mutually determined by the Sponsor and the Service as being items of normal maintenance rather than major repair and are not therefore in keeping with the spirit and intent of the Establishment Period provisions.

B. The Establishment Period for structural measures (exclusive of any associated vegetative work) is a period of three years ending at midnight on the third anniversary of the date on which the structural measure is accepted.

C. The Establishment Period for vegetative work associated with a structural measure is a period from date of acceptance of the initial vegetative work to midnight of the date on which the Service writes the Sponsor advising that an adequate vegetative cover has been obtained. However, this period shall not exceed two growing seasons or the end of the Establishment Period for the associated structural measure whichever is greater in time.

D. As used in the two preceding paragraphs, and elsewhere in this Plan, the following words have the meanings described below:

ACCEPTED, ACCEPTANCE: The date structural or vegetative measures are accepted from the contractor when a contract is involved, or the date structural or vegetative measures are completed to the satisfaction of the Service when force account operations are involved.

ADEQUATE VEGETATIVE COVER: A minimum of seventy percent (70%) cover of the desirable species, with no active rilling that cannot be controlled by the vegetation.

E. Major repair may involve such things as (1) repairing separated joints, cracks or breaks in the principal spillway, (2) correcting seepage, (3) replacing significant backfill around structures.
OPERATION AND MAINTENANCE PLAN

I OPERATIONS

A. The Sponsor will be responsible for and will operate or have operated without cost to the Service the structural measures in compliance with any applicable Federal, State and local laws, and in a manner that will assure that the structural measures will serve the purpose for which installed as set forth in the Work Plan.

B. The Service will, upon request of the Sponsor and to the extent that its resources permit, provide consultative assistance in the operation of the structural measures.

II MAINTENANCE

A. The Sponsor will:

1. Be responsible for and promptly perform or have performed without cost to the Service except as provided in Paragraph III, Establishment Period, all maintenance of the structural measures determined by either the Sponsor or the Service to be needed.

2. Obtain prior Service approval of all plans, designs and specifications for maintenance work involving major repair.

B. The Service will, upon request of the Sponsor and to the extent that its resources will permit, provide consultative assistance in the preparation of plans, designs and specifications for needed repair of the structural measures.

III ESTABLISHMENT PERIOD

A. During an Establishment Period, as herein defined, the Service will bear such part of the cost of any needed major repairs to the structural measures, including associated vegetative work, as is proportionate to the original construction costs borne by the Service in the construction of the structural measures except that the Service will not bear any of the cost for:

1. Repairs to channels or portions thereof which do not have permanent linings such as concrete, riprap, or grouted rock.
OPERATION AND MAINTENANCE AGREEMENT
FOR
STRUCTURAL MEASURES

THIS AGREEMENT made on August 26, 1949, is between the Soil Conservation Service, United States Department of Agriculture, hereinafter referred to as the Service, and the following organizations, hereinafter referred to as the Sponsor:

City of Berlin, New Hampshire
Coos County Conservation District

The Sponsor and the Service agree to carry out the plan on the attached four pages for the operation and maintenance of structural measures in the Dead River Watershed Project, State of New Hampshire. The measures covered by this agreement are identified as:

one multiple purpose structure plus Jericho Brook diversion channel, diversion dike and floodway, known as Site No. 1, Dead River Watershed, located in Berlin, Coos County, New Hampshire, along Jericho Brook approximately 1.3 miles west (upstream) of the intersection of Route #110 and Jericho Brook.

Name of Sponsor City of Berlin, New Hampshire

By ___________________________ Title ___________________________

This action was authorized at an official meeting of the Sponsor named immediately above on August 26, 1949, at Berlin, New Hampshire

Attest ___________________________ Title ___________________________

Name of Sponsor Coos County Conservation District

By ___________________________ Title ___________________________

This action was authorized at an official meeting of the Sponsor named immediately above on August 26, 1949, at

Attest __________________________________ Title ___________________________

Soil Conservation Service, United States Department of Agriculture

By ___________________________ Title State Conservationist

8-31
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Total: 270

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Total: 270

10.06 Days

**L Routines Started at 6 Day Drawdown from E.S. Crest**
DEAD RIVER WATERSHED
MULTIPLE PURPOSE DAM - NO. 1
COOS COUNTY
NEW HAMPSHIRE

Reference:
U.S.G.S. 15' Quadrangle
Mt. Washington, New Hampshire
LAYOUT CONTENTS

ITEM

LOCATION OF POST @ RANGE

PRINCIPAL SPILLWAY LAYOUT

EM, SPILLWAY CURVE DATA

ANGLE BETWEEN DAM & E
Location of Post @ Range 344 & Lot 14415 with respect to Base Line of Survey @ Dom

\[ \alpha = 200^\circ + 180^\circ - 338^\circ 22 = 41^\circ 38' \]

\[ L = 189.5 \sin 41^\circ 38' \approx 189.5 \times 0.66436 = 125.896 \text{ Say 125.9} \]

\[ d = 189.5 \cos 41^\circ 38' \approx 189.5 \times 0.74741 = 141.634 \text{ Say 141.6} \]

Station of Post along B of Survey

Station \((49 + 40) + 14 + 41.6 = 50 + 81.6\)
COMPUTATION SHEET

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

STATE: I-4
PROJECT: DEAD POND SITE

CHECKED BY: KAW
DATE: 2/25/65
JOB NO.: 7700-L

SUBJECT: PRINCIPAL SPILLWAY LAYOUT

Sheets of

\[ M = \frac{1}{800} (G_1 - G_2) = \frac{178}{800} (0.00 + 1.753) = 0.212 \]

\[
\begin{align*}
Z_2 &= M \cdot \left( \frac{\theta}{2} \right) = 0.312 \cdot \frac{256}{200} = 0.319 \\
Z_4 &= 0.979 \\
Z_5 &= 0.173 \\
Z_6 &= 0.123 \\
Z_7 &= 0.164 \\
Z_8 &= 0.102 \\
Z_9 &= 0.066 \\
Z_{10} &= 0.045
\end{align*}
\]

NOTE: ABOVE DIMENSIONS FOR LENGTHS OF PIPES ARE BASED ON NOMINAL LENGTHS.

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OUTLET: 201.33

B-60
Subject: EM. SP. CURVE DATA (INLET)  

1. \( \Delta = 37^\circ \) \( \frac{\Delta}{2} = 18^\circ 30' \)

2. \( R = 125' \)

3. \( D = \frac{5729.65}{R} = \frac{5729.65}{125} = 45.83720 \)

4. \( L = 100 \left( \frac{\Delta}{D} \right) = 100 \left( \frac{37}{45.83720} \right) = 80.72 \approx 80.72 \)

5. \( T = R \tan \frac{\Delta}{2} = (125)(0.33460) = 41.825' \approx 41.82 \)

6. \( E = R \sec \frac{\Delta}{2} = (125)(0.65449) = 80.72 \approx 80.72 \)

7. \( M = R \cos \frac{\Delta}{2} = (125)(0.80517) = 100.00 \approx 100.00 \)

8. \( Lc = 2 R \sin \frac{\Delta}{2} = (250)(0.41730) = 79.325' \approx 79.33 \)

\[ \frac{d}{100} = \frac{18^\circ 30'}{80.720} = 22.91873' \]

\[ d = 0.2291873' \]

\[ d_{25} = (0.2291873)(25) = 5.72968' \approx 5^\circ 43' 7.8' \]

\[ d_{57.72} = (0.2291873)(5.72) = 1.3109 = 1^\circ 18.7' \]

\[ C_{25} = 2 R \sin \Delta_{25} = (250)(0.41730) = 24.96' \]

\[ C_{57.72} = 2 R \sin \Delta_{57.72} = (250)(0.41730) = 5.85' \]

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<th>Deflection in deg. &amp; min.</th>
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PL = (3+00) + 41.82 = 3+41.82
EM SPILLWAY CURVE DATA (OUTLET)

1. $\Delta = 35^\circ \quad \frac{\Delta}{2} = 17^\circ 30'$

2. $R = 250'$

3. $D = \frac{3725.65}{250} = 22.0662$

4. $L = 100 \left( \frac{\Delta}{D} \right) = 100 \left( \frac{35}{22.0662} \right) = 150.7144$

5. $T = R \tan \frac{\Delta}{2} = (250) \left( \frac{35}{22.0662} \right) = 78.325$

6. $E = R \sec \frac{\Delta}{2} = (250) \left( \frac{60}{22.0662} \right) = 12.325$

7. $M = R \cos \frac{\Delta}{2} = (250) \left( \frac{30}{22.0662} \right) = 11.570$

8. $L_c = 2R \sin \frac{\Delta}{2} = (500) \left( \frac{60}{22.0662} \right) = 150.355$

\[ d_{100} = \frac{\Delta}{2} + L = \frac{17.60}{202.1144} = 11.4592 \]
\[ C_{15} = 2R \sin \frac{d_{15}}{2} = (500) \left( \frac{60}{22.0662} \right) = 24.99 \]
\[ C_{1744} = 2R \sin \frac{d_{1744}}{2} = (500) \left( \frac{60}{22.0662} \right) = 7.715 \]
I. LEFT ABUTMENT

\[ \tan \alpha = \frac{100}{500} = 0.20 = 11^\circ 18' 36'' \]

II. RIGHT ABUTMENT

\[ \tan \theta = \frac{100}{500} = 0.20 = 11^\circ 18' 36'' \]
General Considerations:

Drilling and geologic investigations have shown more extensive bedrock in the Emergency Spillway sites than was anticipated at the time the Watershed Work Plan was prepared.

This finding dictates that the emergency spillway be raised and narrowed to reduce costly rock excavation. Studies were made to obtain optimum costs between rock excavations and extra fill caused by increased fill heights.

The studies showed the most economical combination to be: the alignment selected with a crest elevation of 1362.0 and an emergency spillway width of approx. 100 feet.

In the Watershed Work Plan, the principal spillway was planned using a 1-stage inlet. In this design, a two-stage inlet was used for the following reasons:

A. TRASH PROBLEM

1. The flood pool is very wooded and is expected to produce much trash.
2. Beavers inhabit the general site area and are usually attracted to PL 566 impoundments. They can be expected to make attempts at modifying water levels.
3. The two-stage inlet allows use of the modified grating type trash rack at the low stage, particularly when beavers shop trash into the riser. When the beavers are not active, the gratings are to be removed.
4. The two-stage trash rack gives more trash area before the principal spillway becomes...
interval between clogging of the stages.

5. The site is relatively remote during the snow season so that inspections may become limited.

6. A 2 stage inlet has desirable trash features.

B. Economics

1. The two stage alternative raises the water levels a small amount. However, the bedrock in the emergency has made raising the E.S. Crest an economical measure in any case. The increase in water level caused by the 2 stage alternative does not result in any additional cost in fill.

2. The cost of additional concrete and trash racks cost is considered worth while.

C. OTHER

1. With a 2 stage the Gara lift will not become inundated as frequently as with a single stage.

2. Class "C" Dam with recreation
MAX. HEIGHT OF DAM  38.5 FT.

ELEV. TOP OF DAM  1366.5

STORAGE BELOW TOP OF DAM (FROM PROJECTED
STAGE-STORAGE CURVE) = 3750 AC-FT (EST)

\[ K_s = \frac{(38.5)(3750)}{300} = 411.29 \quad \Rightarrow \quad K_s = 25 \]

\[ K_p \]

FLOOD AREA, CITY OF BERLIN  250 PEOPLE (EST)  \[ K_p = 20 \]

\[ K_w \]

EST. AVE. FLOOD PLAIN WIDTH = 300 FT  \[ K_w = 5 \]

\[ K_d \]

DISTANCE FROM STRUCTURE TO
DAMAGE CENTER = 3.2 MILES  \[ K_d = 1.0 \]

\[ K = \frac{K_s + K_p + K_w}{K_d} = \frac{25 + 20 + 5}{1.0} = 50 \]

\[ 7.30 \]

OTHER FACTORS

FAILURE MAY CAUSE LOSS OF LIFE

COMMERCIAL & RESIDENTIAL BUILDINGS ARE
IN FLOOD PLAIN

8-57
## New Hampshire Dead River Site

### Stage - Storage Data

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<th>Area (sq ft)</th>
<th>Area (sq acres)</th>
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<td>6.00</td>
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</tr>
</tbody>
</table>

### Scale: 1" = 300 ft, 1' = 2.062 acres
**Reservoir Sedimentation Design Summary**

**Location:** Coos County  
**Rivershed:** Dead River  
**Date:** 9-21-67  
**State:** New Hampshire

**Data Computed by:** William A. Bonin  
**Title:** Geologist

**Type of Erosion**  
<table>
<thead>
<tr>
<th>Culivated Land</th>
<th>Future (After Cons. Treatment)</th>
<th>Present Conditions</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Acres</td>
<td>Soil Loss (Tons/AC)</td>
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<tr>
<td>B</td>
<td>272</td>
<td>0.039</td>
</tr>
<tr>
<td>C</td>
<td>129</td>
<td>0.692</td>
</tr>
<tr>
<td>D</td>
<td>125</td>
<td>0.136</td>
</tr>
</tbody>
</table>

**Sediment Sources (Average Annual):**

- **Total Sheet Erosion:** 20, Delivery Rate: 2.83, Delivery: 42.7
- **Total Delivery:** 86.7

**Average Dry Weight of Upland Soils:** 23 lbs/cu ft

**Texture of Sediment:**
- % Clay:  
- % Silt:  
- % Coarse: 

**Average Annual Sediment Delivered to Site from All Sources (Tons):**

- **Sheet Erosion:** 42.7, Trap Efficiency: 59, Annual Deposition: 42.8, Design Period: 100, Period Total Deposition: 4,280
- **Floodplain Scour:** 42.0, Annual Deposition: 42.0, Design Period: 100, Period Total Deposition: 4,200

**Design Totals:** 8,400

**Sediment Storage Requirements:**

<table>
<thead>
<tr>
<th>Condition of Sediment</th>
<th>% of Total</th>
<th>Deposition (Tons)</th>
<th>Volume Weight of Sediment</th>
<th>Storage Required</th>
<th>Storage Allocation (Acre Feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Lbs/cu ft</td>
<td>Tons/AC ft</td>
<td>Acre-feet</td>
</tr>
<tr>
<td>JEREGED</td>
<td>100</td>
<td>320</td>
<td>6.0</td>
<td>4.00</td>
<td>3.28</td>
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<tr>
<td>MATERED</td>
<td>40</td>
<td>1280</td>
<td>6.0</td>
<td>2.10</td>
<td>1.68</td>
</tr>
<tr>
<td>TOTALS</td>
<td>52</td>
<td>1600</td>
<td>6.0</td>
<td>5.10</td>
<td>3.28</td>
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</table>

**Design:** 8-60
**Drainage Area:** 6.48 SQ MI = 4147 ACRE

<table>
<thead>
<tr>
<th>Sub. No.</th>
<th>Land Use</th>
<th>Soil Group</th>
<th>Acres</th>
<th>Complex No. TIMES ACRES</th>
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<tbody>
<tr>
<td>A</td>
<td>Forest</td>
<td>A-B</td>
<td>1425</td>
<td>65.5 93338</td>
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<tr>
<td></td>
<td>Forest</td>
<td>C-D</td>
<td>1180</td>
<td>72.9 86022</td>
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<tr>
<td>C</td>
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<td>A-B</td>
<td>830</td>
<td>65.5 54365</td>
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<td>C-D</td>
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<tr>
<td>E-1</td>
<td>Forest</td>
<td>A-B</td>
<td>477</td>
<td>65.5 31293</td>
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<tr>
<td></td>
<td>Idle</td>
<td>D</td>
<td>115</td>
<td>83  9595</td>
</tr>
</tbody>
</table>

**Weighted No.** = \( \frac{283261}{4147} \) = 68.3

**Use 68, AMC II**

**85, AMC III**

*Data: Taken from:*
1) Forest Service LTR to Mr. Addison DTD 10-30-63
2) Soil Cover Complex No.'s, Present Future by E.L.R., 12-29-64

Both references in Dead River W/S Hydrology Book.
**Subject**

Tc

by 15-1015

Subarea A

50% of area with these slopes

<table>
<thead>
<tr>
<th>Slope</th>
<th>Value</th>
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<tbody>
<tr>
<td>1300</td>
<td>17%</td>
</tr>
<tr>
<td>7850</td>
<td></td>
</tr>
<tr>
<td>1300</td>
<td>19%</td>
</tr>
<tr>
<td>6750</td>
<td></td>
</tr>
<tr>
<td>900</td>
<td>19%</td>
</tr>
<tr>
<td>4700</td>
<td></td>
</tr>
<tr>
<td>500</td>
<td>16.7%</td>
</tr>
<tr>
<td>5000</td>
<td></td>
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</table>

**Average slope** = 18%

**Weighted slope**

<table>
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<th>Value</th>
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<tbody>
<tr>
<td>0.5</td>
<td>9</td>
</tr>
<tr>
<td>0.5</td>
<td>4</td>
</tr>
</tbody>
</table>

**Weighted slope** = 13%

\[ L = 12250 \text{ ft} \]
\[ Y = 13\% \]
\[ CN = 0.9 \]

\[ T_c = 1.67 \times 0.95 = 1.6 \text{ hrs} \]
channelized flow through subareas B & C

<table>
<thead>
<tr>
<th>Length</th>
<th>Fall</th>
<th>Slope</th>
<th>V.</th>
<th>Tc</th>
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</thead>
<tbody>
<tr>
<td>7500'</td>
<td>180'</td>
<td>1.9%</td>
<td>3.1</td>
<td>18 hrs.</td>
</tr>
</tbody>
</table>

\[ T_c = T_{c subarea A} + \text{channel travel time in subarea } C \]

\[ T_{c subarea A} + 0.8 = 2.4 \text{ hrs.} \]

AS PER WORK PLAN
Low stage design
Designing as a 2 stage riser

As per work plan:
- Set permanent pool level at elevation 1352.0 (Recreation Pool).
- 4AC.Ft. sediment considered deposited below elevation 1352.0

Ideally, release rate thru 1st stage should be as near as possible to the rate that would have been obtained using a single stage riser with the water surface at the same elevation.

This cannot easily be done, therefore, proportion orifice dimensions as large as possible yet compatible with standard riser.

Set second stage elevation as close to elevation of first stage as possible & still have reasonable trash rack arrangement.
Possible Relation of Sizes & Elevations Using 42" Pipe

Scale 1" = 3'0"

By EM SCS-50

\[ H_2 = (2D + 6') \text{ to } 20' \text{ for standard on this scheme} \]

\[ H_2 = 2 \times 3.5 + 0.5 + 7.5' \text{ for provided} \]

But still considered a good solution for the conditions.
ORIFICE FLOW

From Sketch
Try: Following 2 Second Stage crest at elev. 1356.0

\[ E_4 = 1352.0 \]

USE 2 ORIFICES

\[ C_0 = 0.60 \quad C_4 = 3.1 \]

LOW STAGE WEIR FLOW

\[ Q_{wl} = C_4 \frac{H^3}{4} = 3.1 \times (2)(2.625)^{3/2} \]

\[ Q_{wl} = 16.28 \text{ ft}^3/\text{sec} \]

LOW STAGE ORIFICE FLOW

\[ Q_{ol} = C_0 a_0 \sqrt{2g} h_0 = 0.60 \times (2)(2)(2.625)(8.020) h_0^{1/2} \]

\[ Q_{ol} = 50.53 \text{ ft}^2/\text{sec} \]

Q when second stage begins operating

\[ h_0 = 1356 - 1353 = 3.0 \quad 5^{1/2} = 1.732 \]

\[ Q_{hl} = 50.53 \times 1.732 = 87.52 \text{ cfs} \]

or \[ 87.52 = 13.5 \text{ cfs} \]

B-66
PRINCIPAL SPILLWAY DIMENSIONS

Assuming:

<table>
<thead>
<tr>
<th>Weir Crest</th>
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<tr>
<td>Release</td>
<td>1352.5</td>
</tr>
<tr>
<td>Release 1352.5</td>
<td>1351.0</td>
</tr>
<tr>
<td>16' Berm</td>
<td>4'</td>
</tr>
<tr>
<td>1'</td>
<td></td>
</tr>
<tr>
<td>73.80</td>
<td>101.38</td>
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</table>

plunge pool
maybe used
RISER CREST WEIR & CONDUIT FLOW.

**WEIR FLOW**

\[ Q_{wh} = C_h \cdot L_h \cdot H_h^{3/2} \]
\[ = 3.1 (0.5 \times \varepsilon) H_h^{3/2} \]
\[ Q_{wh} = 65.10 \cdot H_h^{3/2} \]

**CONDUIT FLOW**

\[ Q_{ph} = C_p \cdot h_p^{1/2} \]

**TRY 3.5' I.D. R/C**

\[ C_p = A_p \sqrt{\frac{\varepsilon^3}{1 + K_e + K_p \cdot h_p}} \]
\[ A_p = 9.6 \text{ cm} \]
\[ M = 0.01 \text{ cm} \]
\[ K_e = 1.0 \text{ cm} \]
\[ K_p = 0.0050 \text{ cm} \]

\[ h_p = (137.6 - 132.1) + 16' + (137.6 - 133.4) \times 2.5 \]
\[ + 4 + 1' = 176.18 \text{ Say/76' cm} \]

\[ C_p = 9.6 \sqrt{\frac{64.32}{1 + 1.0 + (0.0050 \times 176)}} \]
\[ C_p = 45.43 \text{ cm} \]
\[ Q_{ph} = 45.43 \cdot h_p^{1/2} \]

\[ Q_{ph max} = 45.43 (136.2 - 132.6) \times 75^{1/2} = 269.7 \text{ cfs < 276 cfs} \]

**USE 42'' PIPE**

\[ B = 68 \]
DATA TAKEN FROM TP-40, TP49, & WORK PLAN

<table>
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<tr>
<th>TIME</th>
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<th>IOYRAMCII +1/DAY SNOWMELT</th>
<th>IOYRAMCII CN: 85</th>
<th>IOYRAMCII CN: 68</th>
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<td>Q(HR)</td>
<td>P(HR)</td>
<td>Q(HR)</td>
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<td>.50</td>
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<td>2.78</td>
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<td>10</td>
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8-69
Elevation of top of dam

Freeboard Hydrograph Routing Elevation 1362.24

Freeboard Hydrograph Routing Elevation 1362.24

Wave Freeboard

\( F \): Fetch = 4.4500 ft = 0.85 miles

\( U \): Velocity of wind = 100 MPH

\( H \): HT of waves = \( 0.171U^2 + 2.50 \) ft

\( H = 3.11 \) ft

\( 1362.24 + 3.11 = 1365.35 \)

Frost Depth

Assume Max. Frost Depth = 5.0 ft

\( 1362.24 + 5.0 = 1367.24 \)

Maximum Freeboard Elevation

Required = 370.9

Set Top of Dam at

Elevation: 1371.0
Iw = FRAC. APPR. LEVEE MIN. 5' \\
Q_0 = MAXIMUM DISCHARGE AT Iw 3650 CFM \\
Q_p = PIPE DISCHARGE AT Iw 3260 CFM \\
Q_e = Q_o - Q_p 390 CFM \\
Q_1 = PEAK INFLOW 4275 CFM \\
V_1 = TOTAL INFLOW VOLUME 555 AC FT \\
Iw(Q_1/V_1)^{1/2} = 2.08 \\
V_ww/V_1 = 0.84 \\
V_ww = INFLOW VOLUME AT Iw 1690 CF \\
I = Iw - Ie 1.2 \\
\[
\frac{Iw - Ie - \left(\frac{Q_p}{Q_0}\right) \cdot Iw}{Q_p} = 12 \\
1562 - 1527 - 1603 - 1569 \\
2.71 = 2.9 \\
\frac{2.9}{2.4} \\
B-83
CRITICAL OUTLET CHANNEL SLOPE

\[ b = 100 \text{ FT}, \quad M = 0.04, \quad Q_E = 39 \text{ CFS} \quad \text{EL.} \quad 1362.24 \]
\[ Q_E = 0.39 \text{ CFS/FT} \quad \rho = 25\% \quad \rho_E = 0.0975 \text{ CFS/FT} \]
\[ d_c = \sqrt{\frac{\rho_E}{\rho}} = \sqrt{\frac{(0.0975)^2}{0.39}} = 0.67 \text{ FT} \quad \text{SEP ES.08} \]
\[ S_c = \frac{2.142 \times \rho^2 (100)}{\rho_E^{3/2}} = 2.142 \times \left(\frac{0.0975}{0.39}\right)^{3/2} = 5.8\% \quad \text{SEP ES.08} \]

\[ \cdot \quad \text{USE 4\%} \quad \text{EM.87} \]

MAX VELOCITY THRU E.S. @ CONTROL SECTION

**E.S. HYDROGRAPH**
\[ Q_E = 0.39 \text{ CFS/FT} \]
\[ d_c = 0.7 \text{ FT} \]
\[ V_c = 2.32 \text{ FPS} \]

**FREEBOARD HYDROGRAPH**
\[ Q_E = 7975 - 302 = 7673 \text{ CFS} \]
\[ Q_E = 76.73 \text{ CFS/FT} \]
\[ d_c = 5.7 \text{ FT} \]
\[ V_c = 13.6 \text{ FPS} \]
**HYDROGRAPH COMPUTATION**

**FREEBOARD HYDROGRAPH**

**WATERSHED OR PROJECT** DEAD RIVER

**STATE** NEW HAMPSHIRE

**STRUCTURE SITE OR SUBAREA** SITE 1

**DR AREA** 6.48 SQ. MI.  
**T** 2.4 HR.  
**RUNOFF CONDITION NO.** II

**RUNOFF CURVE NO.** 68  
**STORM DISTRIBUTION CURVE** B

**STORM DURATION** 6 HR.  
**RAINFALL:**  
**POINT 20.2 IN.**  
**AREAL 20.3 IN.**

**Q** 15.57 IN.  
**COMPUTED** $t_p$ 1.68 HR.  
**$T_o$** 5.83 HR.

$\frac{(T_o - T_p)}{T_p} = \frac{3.17}{2.5} = 1.27$  
**USED** 3  
**REVISED** $T_p$ 1.78

$q_p = \frac{464 A}{\text{REV. } T_p} = 1762 \text{ CFS.}$

$q = 27134 \text{ CFS.}$

_DATA FROM TABLE 3.217 (2.70FS8)_

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<thead>
<tr>
<th>LINE NO.</th>
<th>t HOURS</th>
<th>q CFS</th>
<th>LINE NO.</th>
<th>t HOURS</th>
<th>q CFS</th>
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### Emergency Spillway Hydrograph Computation

**Watershed or Project:** Dead River  
**State:** N. H.  
**Structure Site or Subarea:** Site 1  

**Dr. Area:** 6.48 sq. mi.  
**Tc:** 2.4 hr.  
**Runoff Condition No.:**  

**Runoff Curve No.:** 68  
**Storm Distrib. Curve:** B  
**Hydrograph Family No.:** 3  

**Storm Duration:** 6 hr.  
**Point Rainfall:** 3.3 in.  
**Areal Rainfall:** 8.3 in.  
**Qp:** \( 4.49 \) in.  
**Computed** \( Tp = 1.68 \) hr.  
**To:** \( 4.70 \) hr.  
**\( Tp + \frac{2}{3} \):** \( 2.90 \) hr.  
**Used:** 3  

\[ q_p = \frac{684}{A} \]  
**REV. Tp:** 1.56 hr.  
**Qp:** 8.98 CFS.  

\( q_{c} = \frac{\text{COLUMN}}{\text{REV. Tp}} \) \( q_{c} = (Qc/Qp) Qp \)

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**Notes:**
- Commence flood routing at ELEV. 1352.4
- Drawdown started from E. 5 CFS
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DFAD river site I
Emergency spillway stage & discharge data (cont.)
### Tabular Computations

#### SOIL CONSERVATION SERVICE

#### DEAD RIVER SITE 1

**EMERGENCY SPILLWAY STAGE & DISCHARGE DATA**

**REF:** ES-124

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**B.76**
### DEAD RIVER SITE

#### EMERGENCY SPILLWAY STAGE & DISCHARGE DATA (cont.)

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13-74
REQUIRED STORAGE:

\[ Q = 4.10 \text{ INCHES} \]
\[ Q = \frac{4.10 \times 4147}{12} = 1416.9 \text{ AC-FT} \]

TOTAL SEDIMENT = 5 AC-FT

RECREATION = 1235.6 AC-FT

TOTAL NECESSARY STORAGE TO E.S. CREST = 2657.5 AC-FT

FROM STAGE-STORAGE CURVE:

\[ 2657.5 \text{ AC-FT} \rightarrow \text{EL. 1361.25} \]

FROM ECONOMIC DATA, USE EL. 1362.0

Storage to elev. 1362.0 = 2800 AC-FT

\[ \frac{1240.6}{1559.4} \]

\[ \text{or } 1559.4 \times 12 = 4.51'' \rightarrow 4.10'' \text{ CK.} \]

\[ \frac{4147}{4147} \text{ (PROVIDED) (NEEDED)} \]
Reservoir Drain Pipe Size

Ave. annual runoff = 22 inches
or \( \frac{22}{365} = 0.0603 \) inches/day

\( 0.0603 = \frac{1.62 \text{ csm}}{0.03719} \)

\( 1.62 \times 6.48 = 10.5 \text{ cfs} \) \( \) (average rate of runoff)

Try 30" RCP reservoir drain \( S = 0.005 \)

\( Q_{30} = 4.909 \text{ ft}^2 \)

From "Design" Seelye

\( Q = 25 \text{ cfs} \) \( \) (water level with top of entrance OK.

using 24" RCP \( Q = 14 > 10.5 \) also OK.

Estimate time to empty reservoir

\( S = 1236 \text{ A.F. (Beneficial Stor)} + 5 \text{ (Sed. Stor)} = 1241 \text{ A.F.} \)

Using 30" RCP \( \eta Q_{\text{max}} = \frac{0.62 + \sqrt{2g}h}{h} \)

\( h = \frac{1352 - (1329.5 + \frac{25}{2})}{1.21} = 21.25' \)

\( Q_{\text{max}} = 0.62 \times 4.909 \times 8.02 \times \sqrt{21.25} \)

\( = 112.3 \text{ cfs} \)

\( Q_{\text{ave}} = \left(\frac{112.3 + 25}{2}\right)^\frac{1}{2} = 68.7 \text{ cfs} \)

\( Q_{\text{net}} = 68.7 - 10.5 = 58.2 \text{ cfs} \)

\( 1 \text{ cfs} = 1.99 \text{ A.F.D.} \)

\( \frac{T = \text{time to empty} = \frac{1241}{58.2 \times 1.98} = 10.8 \text{ Days} \text{ OK.}}{\text{}} \)

\( T = \text{time with 24" Pipe} = 20.1 \text{ Day (Excessive)}} \)

Use 30" RCP Reservoir Drain

\( Q_{\text{max}} = 112.3 \) \( (30") \)

\( \frac{112.3}{6.48} = 17.3 \text{ csm} \) \( B-85 \)
Tailwater in Outlet Channel

\[ Q_{\text{max}} = 270 \text{ cfs} \]
\[ m = 0.035 \]
\[ b = 16' \]
\[ s = 0.005 \]
\[ t = 2' \]

\[ \frac{Q}{b} = \frac{270}{16} = 16.9 \text{ cfs/ft} \]

\[ \frac{m'Q}{b^\frac{3}{2}} = \frac{0.035 \times 270}{16^{\frac{3}{2}} \times 0.005^{\frac{3}{2}}} = \frac{9.45}{1630 \times 0.07071} = 0.082 \]

\[ \frac{d}{b} = 0.165 \quad \therefore \quad d = 1.165 \times 16 = 2.64' \]

\[ A = d (b + 2d) = 2.64 (16 + 2.64 \times 2) = 56.4 \text{ ft}^2 \]

\[ V = \frac{Q}{A} = \frac{270}{56.4} = 4.8 \text{ fps}, \text{ OK.} \]

When \( Q = 270 \text{ cfs} \) & Channel incl @ 1320.5

Elev of W.S. = 1323.14

Set top of Rip Prop @ Elev 1323.5
FLOODWAY - ELEV. TOP OF DAM

The right side (looking downstream) of the dam must be raised above the elevation normally required to provide freeboard for the floodway flow.

The top elevations of the dam adjacent to the floodway will be determined by considering the following:

1. That all the Jericho Brook flow enters the flood pool thru the floodway & diversion channel.

2. That the peak freeboard flow pass the floodway, without overtopping the dam.

   \[
   \text{Peak Inflow} = (16872)(5.66/6.48) = 1475^*  
   \]

   \[
   \text{Pool Elevation} \cap \text{Peak Flow} = 1363.8^*  
   \]

3. That the floodway flow, when the pool elev. is maximum, pass without overtopping the dam.

   \[
   \text{Max. Pool Elev.} = 1370.9  
   \]

   \[
   \text{Flow in brook when pool elev. equals 1370.9 is} \ (79.75)(5.66/6.48) = 6966 \text{ CFS}  
   \]

4. That an arbitrary floodway flow (value between those in para. 2&3 above) pass without overtopping the dam.

   \[
   \text{Say flow} = 13280 \text{ CFS}  
   \]

   \[
   \text{Pool Elevation} = 13280 = 1367.0  
   \]

* See freeboard routing - Brook drainage area.
Subject: Floodway

Selection of N

STA 22+00 to 17+00

Channel in Rock & Earth: 0.025

Irregularity: 0.015

Size & Shape: 0.010

Obstructions: 0.020

Vegetation: 0.067

Degree of Meandering: 0.000

Total: 0.077
STATE: NEW HAMPSHIRE  
PROJECT: DEAD RIVER  
SITE:  

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<td>0.010</td>
<td>VARIATION IN SIZE &amp; SHAPE LARGE &amp; SMALL SECTIONS ALTERNATING, SHAPE CHANGES</td>
</tr>
<tr>
<td>------</td>
<td>-------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>STEP</td>
<td>0.008</td>
<td>MINOR EFFECT OF OBSTRUCTIONS</td>
</tr>
<tr>
<td>------</td>
<td>-------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>STEP</td>
<td>0.007</td>
<td>LOW DEGREE OF EFFECT CAUSED BY VEGETATION</td>
</tr>
<tr>
<td>------</td>
<td>-------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>STEP</td>
<td>0.000</td>
<td>DEGREE OF MEANDERING</td>
</tr>
<tr>
<td>------</td>
<td>-------</td>
<td>--------------------------</td>
</tr>
<tr>
<td>TOTAL</td>
<td>0.060</td>
<td></td>
</tr>
</tbody>
</table>

$N = 0.060$

THIS VALUE BASED ON FOLLOWING:

FLOODWAY CLEARED & GRUBBED
PLANED & SEEDED
DAM PARTLY RIP.RAPPED
**Selection of \( n \)**

From STA 900 to End of Study (STA 400)

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Channel in Earth/rock</td>
<td>0.025</td>
</tr>
<tr>
<td>2</td>
<td>Moderate degree of irregularity</td>
<td>0.015</td>
</tr>
<tr>
<td>3</td>
<td>Variation in size &amp; shape, large &amp; small sections alternating shape changes</td>
<td>0.010</td>
</tr>
<tr>
<td>4</td>
<td>Appreciable effect of obstructions</td>
<td>0.030</td>
</tr>
<tr>
<td>5</td>
<td>Very high effect caused by vegetation</td>
<td>0.070</td>
</tr>
<tr>
<td>6</td>
<td>Degree of meandering</td>
<td>0.000</td>
</tr>
<tr>
<td>Total</td>
<td></td>
<td>0.15</td>
</tr>
</tbody>
</table>

**Notes**

\[ \sum_{i=1}^{6} \text{Value} = 0.15 \]

\[ \sum_{i=1}^{6} \text{Value} = 0.15 \]

\[ \sum_{i=1}^{6} \text{Value} = 0.15 \]

\[ \sum_{i=1}^{6} \text{Value} = 0.15 \]
Memorandum

TO: C. H. Dingle, State Conservation Engineer,
SCS, Durham, New Hampshire

DATE: April 22, 1968

FROM: Lorn P. Dunnigan, Head, Soil Mechanics Laboratory,
SCS, Lincoln, Nebraska

SUBJECT: ENG 22-5, New Hampshire WP-08, Dead River, Site No. 1 (Coos County)

ATTACHMENTS

1. Form SCS-354, Soil Mechanics Laboratory Data, 4 sheets.
2. Form SCS-355, Triaxial Shear Test Data, 3 sheets.
3. Form SCS-352, Compaction and Penetration Resistance Report, 5 sheets.
4. Form SCS-353, Soil Classification, 1 sheet.
5. Form SCS-130, Drain Materials, 1 sheet.

DISCUSSION

FOUNDATION

A. Bedrock: Bedrock underlies the till on the abutments and the alluvium in the floodplain at depths of from 0 to 27 feet. The bedrock is described in the geology report for this site.

Durability tests were made on the rock cores from test hole 505. During the Los Angeles abrasion test the loss was 45.8 percent. Gradation A (Federal specification) was used for the test. The ledge rock procedure was used for the soundness test and the loss during 5 cycles was only 0.09 percent.

B. Soil Classification: The soil on the abutments is primarily glacial till. Sample 4.1 and 214.1 are considered to be representative of the majority of the till on the left abutment. The grain-size curve for these two samples and curves for two other samples from the spillway are shown on the attached form SCS-353 for comparison. The gradation of these four samples fall within a relatively narrow range and the properties of the materials are expected to be comparable. The till on this abutment is stratified in some zones as indicated by variable and high permeability rates and by sample 503.2 which represents a water worked till from the 11 to 14-foot depth. Sample 4.1 contains 15 percent fines and is classified as non-plastic SM. Sample 214.1 contains 10 percent fines and is an EM-SP. The water worked till contains 22 percent fines and is classified as non-plastic SM. There is some muck and alluvium in the vicinity of centerline station 54+00 on this abutment.
Lorn P. Dunnigan

Subj: ENG 22-5, New Hampshire WP-08, Dead River, Site No. 1

The mantle material in the floodplain between approximate centerline stations 21+50 and 27+50 consist of a surface zone of muck overlying pervious alluvium that is logged primarily as SP and SP-SM. Sample 502.1 was submitted from approximately the toe area downstream from centerline station 24+50. The material represented contains 13 percent fines and is classed as non-plastic SM.

The muck averages about 1.5 feet thick and ranges up to 6.5 feet thick. The sandy alluvium ranges from 0 to about 20 feet thick.

The till on the right abutment and diversion area ranges from about 30 feet at approximately centerline station 20+00 to from 0 to about 5 feet on the rest of the abutment right of centerline station 19+00. Samples were submitted from test holes 7 and 10 on the abutment. These samples are somewhat finer grained than the till on the left abutment. They contain about 30 percent fines and they are classed as non-plastic SM and low plasticity SC-SM.

C. Shear Strength: The blow count in the till and the alluvium is relatively high and, based on this data, it appears that the shear strength of the till and the alluvium is adequate for the embankment planned. It is planned to strip the muck from the foundation and its shear strength will not be a problem.

D. Consolidation: With the muck removed the consolidation potential of the till and of the alluvium is expected to be very low.

E. Permeability: The permeability of the till and of the alluvium is expected to be highly variable. Field permeability tests were made and the data are reported in the geology report.

EMBANKMENT

A. Classification: The majority of the fill material at this site is represented by samples 4.1, 59.1, 207.1, 214.1 and 221.1. The gradation curves for each of these samples falls within a relatively narrow range as shown on the attached form SCS-353. Sample 4.1 contains 14 percent fines and is classed as a non-plastic SM. The other four samples referred to above contain from 8 percent to 11 percent fines and they are classed as SP-SM.

The bulk dry density of the gravel size material is in excess of 160pcf.
B. Compacted Density: Standard Proctor compaction tests were made on the minus No. 4 fraction of samples 207.1, 214.1, 221.1, 4.1 and 7.1. The maximum dry densities obtained fell within a relatively narrow range of from 120 pcf to 124 pcf.

In addition to the standard compaction tests a relative density test was made on the minus 1 1/2-inch fraction of sample 214.1. The test was made in accordance with ASTM designation D2049. The minimum density obtained is 115 pcf and the maximum density obtained is 139.1 pcf.

C. Permeability: Permeability consolidation tests were made on the minus No. 4 fraction of sample 214.1 (68W1663). The tests were made at densities of 95 percent of Proctor and at 100 percent of Proctor.

The test data obtained are summarized as follows:

<table>
<thead>
<tr>
<th>Test No</th>
<th>% Proctor</th>
<th>k in fps</th>
<th>Consolidation ft/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>2000 pcf</td>
</tr>
<tr>
<td>114.5</td>
<td>95</td>
<td>3.000</td>
<td>5.000</td>
</tr>
<tr>
<td>120.9</td>
<td>100</td>
<td>0.055</td>
<td>0.038</td>
</tr>
</tbody>
</table>

The test data for the test at 95 percent of Proctor is not conclusive. You will note that the permeability rate increased as the load was increased and that the consolidation potential was less under these loadings than it was at 100 percent of Proctor. We don't know the reason for this apparent discrepancy but we will set up a check test and report the data when the test is complete.

D. Shear Strength: Consolidated undrained triaxial shear tests were made on sample 214.1 (68W1663). One test was made on the minus No. 4 material compacted to 95 percent of standard Proctor density and one test was made on the material finer than one inch compacted at a density of approximately 128 pcf. The test specimens for the minus one inch material were graded so that they contained 40 percent gravel. The test specimen diameter was 4.0 inches.

The test data are summarized as follows:

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>% Gravel in Test Specimen</th>
<th>Test Specimen Diameter (inches)</th>
<th>Test Density (pcf)</th>
<th>Degree of Saturation</th>
<th>Shear Strength Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>68W1663</td>
<td>0</td>
<td>1.4</td>
<td>114</td>
<td>76.3</td>
<td>Ø deg</td>
</tr>
<tr>
<td>68W1663</td>
<td>40</td>
<td>4.0</td>
<td>128</td>
<td>80.9</td>
<td>42.0</td>
</tr>
</tbody>
</table>
You will note that the degree of saturation is low for both tests. The 1.4 inch diameter test specimens were soaked prior to testing and the 4.0 inch specimens were molded at saturation. The computations for the degree of saturation are based on the moisture content after test and weight loss during the test. The material tested contains a relatively low percentage of non-plastic fines and the specimens drain very rapidly. This is believed to be the reason for the low degree of saturation. The test results have been interpreted as $c = 0$ and this was done to make allowance for the low degree of saturation.

The test density used for the specimens containing gravel is equal to 95 percent of the minus No. 4 Proctor density adjusted for 40 percent gravel. It is interesting to note that this density is within 3 pec of 75 percent of relative density on the minus 1 1/2-inch fraction.

**E. Durability of Rock:** The Los Angeles abrasion test and the sodium sulfate soundness tests were made on five samples. The test data obtained are summarized as follows:

<table>
<thead>
<tr>
<th>Sample Number</th>
<th>Los Angeles Abrasion</th>
<th>Sodium Sulfate Soundness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field</td>
<td>Laboratory</td>
<td>Gradation</td>
</tr>
<tr>
<td>18</td>
<td>1666</td>
<td>G</td>
</tr>
<tr>
<td>227</td>
<td>1667</td>
<td>G</td>
</tr>
<tr>
<td>230</td>
<td>1670</td>
<td>G</td>
</tr>
<tr>
<td>505</td>
<td>1671</td>
<td>A</td>
</tr>
<tr>
<td>4.1</td>
<td>1653</td>
<td>A</td>
</tr>
</tbody>
</table>

It was noted that thin sections of CSW1668 (Field No. 228) could be flaked with the fingers, therefore, no tests were made.

The gradation referred to in the table under Los Angeles Abrasion corresponds to grading in the Federal specification for this test.
Subj: EKG 22-5, New Hampshire WP-08, Dead River, Site No. 1

<table>
<thead>
<tr>
<th>Slope</th>
<th>Condition</th>
<th>( \gamma ) Sat.</th>
<th>( \varphi ) deg</th>
<th>( c ) psi</th>
<th>( F_s )</th>
</tr>
</thead>
<tbody>
<tr>
<td>3:1 Upstream</td>
<td>Horizontal flow lines</td>
<td>128.5</td>
<td>42</td>
<td>0</td>
<td>1.22</td>
</tr>
<tr>
<td>3:1 Upstream</td>
<td>Parallel flow lines</td>
<td>128.5</td>
<td>42</td>
<td>0</td>
<td>1.37</td>
</tr>
<tr>
<td>2 1/2:1 Downstream</td>
<td>No Drain - Horizontal flow lines</td>
<td>128.5</td>
<td>42</td>
<td>0</td>
<td>0.99</td>
</tr>
<tr>
<td>2 1/2:1 Downstream</td>
<td>No Drain - Parallel flow lines</td>
<td>128.5</td>
<td>42</td>
<td>0</td>
<td>1.14</td>
</tr>
<tr>
<td>3:1 Downstream</td>
<td>No Drain - Horizontal flow lines</td>
<td>128.5</td>
<td>42</td>
<td>0</td>
<td>1.22</td>
</tr>
<tr>
<td>2 1/2:1 Downstream</td>
<td>Drained</td>
<td>128.5</td>
<td>42</td>
<td>0</td>
<td>2.25</td>
</tr>
</tbody>
</table>

CONCLUSIONS AND RECOMMENDATIONS

A. Site Preparation: We concur with the proposal to remove the muck from the foundation area of the dam. The water table is high and dewatering will be necessary.

B. Cutoff: A multiple-purpose flood-control and recreation dam is planned. The till and alluvium overlying bedrock is primarily an SF-SM. These materials are stratified and contain zones of high permeability. In order to insure that the dam functions as intended we suggest that the cutoff trench bottom on bedrock on the abutment and in the floodplain section. Cutoff to bedrock can be obtained with trench depths of 11 feet or less except where the centerline cross a "topographic high" on each side of the floodplain. In the areas of the topographic high cuts in the range of 30 feet will be necessary to bottom the trench on rock. The till may not be as variable as it appears to us and it may be possible to reduce seepage by upstream blanketing or by some other method of cutoff.

Glacial till like sample 7.1 (68W1654) contains 33 percent fines and 14 percent finer than 0.005 mm. It is classified as an SC-7. If this type of material is available in sufficient quantity we suggest that it be used for backfill. The permeability rate of this material placed at 9 percent of Proctor density would be very low.

If material like sample 7.1 is not available or if it cannot be easily identified in the field, material like sample 4.1 or sample 214.1 may be used. We suggest that it be placed at approximately 105 percent of Proctor density with the control on the minus No. 4 fraction in order to include uniform low permeability fill. We suggest a placement moisture content of optimum if possible.

It might be possible to reduce excavation by locating the cutoff upstream from centerline.
C. Principal Spillway: The proposed location for the principal spillway is near the base of the left abutment. This location was selected in the field as best location of several alternates studied. The bedrock profile at this location is somewhat irregular and it occurs at depths of from about 5 feet to 14 feet. The alluvium overlying the bedrock is described as loose to very compact and pervious. The water table occurs at or above ground surface and the blow count varies from two blows per foot to more than 200 blows per foot. It appears, on the basis of this information, that it would be desirable to excavate the principal spillway trench to a uniform gradient in or on bedrock and to backfill the trench with material like that suggested for the cutoff trench. Consolidation of the backfill under the conduit would be very low as indicated by the consolidation data obtained on the permeability test specimens.

D. Drain: A drain is suggested to control the phreatic line in the embankment and to provide a safe outlet for seepage from the foundation. As a minimum, a trench drain is suggested at about c/b = 0.6. We suggest that the drain tap the most pervious stratum in the upper 10 feet and that it extend up the abutment to permanent pool elevation. We suggest that the filter-drain completely encompass the conduit on the drain line.

The suggested filter limits are shown on the attached form SCS-130.

E. Embankment Design:

1. Placement of Materials: The materials available for use in the embankment are those represented with the gradation curves plotted on the attached form SCS-353. It is primarily on SP-SM. If it is possible to select enough material like sample 7.1 for a thin core section in the embankment this would be highly desirable because it would result in a core with low permeability and the placement density could be controlled to a minimum of 95 percent of standard Proctor.

If it is not possible to use the finer grained materials as discussed in the cutoff section of the report then we suggest that a core section be constructed of the till like the sample represented with the grain size plots on form SCS-353 by placing them at near 100 percent of Proctor density with the density controlled on the minus #4 fraction. We suggest a placement moisture content slightly on the wet side of standard Proctor optimum.
Subj: L.C 22-5, New Hampshire WF-08, Dead River, Site No. 1

The soil sections would also consist of till but the control could be by method specification. We suggest that the method specified result in a density of about 128pcf or higher.

The consolidation data on the permeability samples indicate that there will be very little difference in the consolidation potential of the material placed at 95 percent of Proctor and the material placed at 100 percent of Proctor so differential settlement between the sections should not be a problem.

2. Slopes: The data and analyses indicate that proposed slopes have acceptable factors of safety.

3. Settlement: An overfill allowance of 0.5 foot is suggested to compensate for residual settlement in the fill and foundation.

F. Dike: It is proposed to construct the dike out of material like that used in the embankment and 3:1 slopes are planned. This appears to be adequate.

Lorn P. Dunnigan

CC:
C. H. Dingle, Durham (3)
N. P. Hogner, Upper Derry
DETAILED GEOLOGIC INVESTIGATION OF DAM SITE

Report No. NH-7703

Investigated by:

William A. Eoin, Geologist
Durham, New Hampshire

C. R. Penney, Field Assistant
Plymouth, New Hampshire

November 21, 1967

GENERAL

State : New Hampshire  Dam No. : 1
County : Coos  Fund Class : WP-08 (2005)
Township : Berlin  Structure Class : c
Watershed: Dead River  Site Group : 1

SITE DATA

Drainage area : 6,46 square miles (4,117 acres)
Type of structure : earth
Purpose : flood control and recreation
Valley trend : east (downstream)
Maximum height of fill : 45 feet
Length of fill : 2,635 feet
Volume of completed fill : 256,000 cubic yards
Steepness of abutments : 50% and 10% left; 10% and 2% right
Emergency spillway width : 100 feet
# Detailed Geologic Investigation of Dam Site

**Report No. NH-77/2**

**Investigated by:** 12/12/67  
**William A. Bonini**, Geologist

**Durham, New Hampshire**  
**C. R. Penney, Field Assistant**  
**Plymouth, New Hampshire**  
**November 21, 1967**

## General

<table>
<thead>
<tr>
<th>State</th>
<th>New Hampshire</th>
<th>Dam No.</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>County</td>
<td>Gooys</td>
<td>Fund Class</td>
<td>WP-06 (2006)</td>
</tr>
<tr>
<td>Township</td>
<td>Berlin</td>
<td>Structure Class</td>
<td>c</td>
</tr>
<tr>
<td>Watershed</td>
<td>Dead River</td>
<td>Site Group</td>
<td>1</td>
</tr>
</tbody>
</table>

## Site Data

<table>
<thead>
<tr>
<th>Drainage area</th>
<th>6.48 square miles (4,147 acres)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of structure</td>
<td>earth</td>
</tr>
<tr>
<td>Purpose</td>
<td>flood control and recreation</td>
</tr>
<tr>
<td>Valley trend</td>
<td>east (downstream)</td>
</tr>
<tr>
<td>Maximum height of fill</td>
<td>45 feet</td>
</tr>
<tr>
<td>Length of fill</td>
<td>2,935 feet</td>
</tr>
<tr>
<td>Volume of compacted fill</td>
<td>226,000 cubic yards</td>
</tr>
<tr>
<td>Steepness of abutments</td>
<td>50% and 10% left; 18% and 2% right</td>
</tr>
<tr>
<td>Emergency spillway width</td>
<td>100 feet</td>
</tr>
</tbody>
</table>
19. Aside from the two low saddles, the foundation and borrow soils are represented by samples 53.1 and 59.1. This overburden is generally semi-pervious and dense. It correlates to the fill material (See item).

20. Up to 5 feet of muck was probed in the low, wet saddles. Beneath the muck there is 1/2 to 2 feet of compact and semi-pervious silty sands (Sample 52.1) over compact and very pervious, gravels and sands (Sample 52.2) with 15% hard, rounded cobbles and boulders to 18 inches.

21. The depth to bedrock is an assumed depth in this area and the thickness of the outwash is unknown.
to be very pervious. The most pervious material is the "open-work" cobbles and boulders (See TF-5 from 2 to 5 feet). Sample 502.1 represents the finer gradation of the variable outwash.

11. The bedrock has overall adequate "in-place" strength. Leakage rates through the bedrock are minor or insignificant.

DIVERSION AREA

12. There are many surface boulders. The overburden is generally shallow. Sample 10.1 represents most of this material. The water level is at or a few feet above the hard bedrock surface.

13. Sample 502.1 represents a limited area of pervious, coarse-grained soil.

Borrow and emergency spillway area

14. The topographic map of the bedrock is contoured to the top of the bedrock surface. A small amount of this rock, as in TP-211 (5 to 9 feet) and DH-226 (10 to 18 feet), can be excavated as earth.

15. Additional borrow is available to the west of this area (See TF's -102, -201, -202).

16. The pegmatite and Olivarian granite are obviously durable rocks. The schistose rock is a non-durable, soft, degradable rock. These rock types can be selected or rejected for use as riprap by field examination.

17. The selection or rejection of the biotite gneisses for use as riprap will present problems as noted:

(a) Where granitized, it has the hardness and durability of the granite.

(b) Elsewhere it is hard and granular with friable surfaces or de- 
composed to some depth (4' in TP-211, 3' in TP-501, 5' in 
TP-502).

Abrasion and soundness tests may assist in the determination of suit-
ability for riprap.

Additional borro area

18. The high knob centered at DH-153 is a good source of fill. More borrow is available to the west of this area (See TF's -151 and -152). The material is represented by sample 2.1. It correlates to the fill material (See item I).
CONTENTS AND CONCLUSIONS

1. Sample 211.1 and the correlation samples (namely: 1.1, 50.1, 207.1 and 211.1) represent our fill material and most of the overburden at the site. This material is relatively dry (4.6% moisture), somewhat cemented and difficult to excavate. It includes hard, subangular to subrounded rocks as follows:

<table>
<thead>
<tr>
<th>Size</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>3%</td>
<td>3&quot; - 6&quot;</td>
</tr>
<tr>
<td>2%</td>
<td>6&quot; - 12&quot;</td>
</tr>
<tr>
<td>1%</td>
<td>12&quot; - 18&quot; plus occasional larger boulders</td>
</tr>
</tbody>
</table>

This soil in-place is dense and semi-pervious. Most of the cobbles and boulders are near the surface.

2. Surface boulders in the borrow and emergency spillway area total:

- 53 cubic yards per acre 6" - 18"
- 311 cubic yards per acre 18" - 36"

3. Surface boulders elsewhere on the site total:

- 35 cubic yard per acre 6" - 18"
- 173 cubic yards per acre 18" - 36"

4. There is also 4,250 cubic yards of boulder rock excavation within the grid survey (See drawing sheets 2 and 3).

5. Core material and quality drain fill materials in sufficient quantity are not available on site or within reasonable distance of the site.

AT THE DAM

6. Sample 7.1 represents the overburden in the high "topo" feature that crosses the right abutment.

7. Sample 211.1 is similar to the overburden in the high "topo" feature along the left abutment. Sample 503.2 represents a very pervious inclusion in an otherwise semi-pervious abutment.

8. The foundation is wet except for the high ground along the centerline.

9. There is up to 6.5 feet of rock across the low area in the foundation. The bedrock surface is very irregular in this area. There are some 10 feet of near vertical relief on the bedrock surface. There are outcrops and bedrock was cored at depths of up to 19 feet.

10. The overburden across the low area is thick and variable in type, strength and permeability. Some strata within this outwash are observed.
The underlying bedrock generally consists of hard, round and durable, weakly foliated olivine granite, hard granular textured biotite gneiss and occasional highly weathered schists.

Exceptions:

Four feet of bedrock (biotite gneiss) was easily excavated from 5 - 9 feet in test pit (TP-111).

Eight feet of very soft biotite-hornblende schist was cored from 10 to 18 feet. Core recovery in similar material was very low.

DIKE

There are no bedrock outcrops in this area. Surface rocks are common but they are not as numerous as at the dam. The areal extent of outwash and till material was determined in the test pitting program. The vertical extent of these materials and the depth to bedrock was not determined.

Except for the area of the saddles, the overburden is glacial till. Up to 5 feet of muck was probed in the saddles. The underlying outwash deposits consist of 0.5 to 2.0 feet of compact and semi-pervious gray-sheets (15% silt and 10% hard, subangular to subrounded gravel) over compact and pervious SP-2Y and CP-SM soil. (40 - 60% hard-rounded gravels to 3 inches, 5% silt and 15% hard, rounded cobbles and boulders to 18 inches). The water level is at or near the surface and the pit walls were caving. In test pit (TP-57) the ground water was a bright iridescent orange color.

The glacial till in the foundation and adjacent borrow areas is an SM and SP-SM soil. It has 10 - 25% hard, subangular gravel to 3 inches and 10 to 15% silt. It is dense and semi-pervious to pervious. It includes up to 25% hard, subangular rocks from 3 to 24 inches.

In the borrow areas, oversize material includes hard, subangular to subrounded rock as follows:

<table>
<thead>
<tr>
<th>Percentage</th>
<th>Size</th>
</tr>
</thead>
<tbody>
<tr>
<td>5%</td>
<td>3&quot; - 6&quot;</td>
</tr>
<tr>
<td>5%</td>
<td>6&quot; - 12&quot;</td>
</tr>
<tr>
<td>10%</td>
<td>12&quot; - 24&quot;</td>
</tr>
</tbody>
</table>
There are numerous hard, subangular surface rocks in the above discussed areas, that is:

- 35 cubic yards per acre from 6 to 18" in size
- 173 cubic yards per acre from 18" to 36" in size
- (rocks over one cubic yard as noted on sheets 2 and 3)

**EMERGENCY SPILLWAY AND PRIMARY BORROW AREA**

This area has many hard, subangular surface rocks. There are 53 cubic yards per acre from 3 to 18 inches and 311 cubic yards per acre from 18 to 36 inches. Boulders well over one cubic yard are common. Much of the topsoil in this area has been lost to sheet erosion following forest fires.

The upper few feet of the till (usually one foot but up to 6 feet) are a more rocky CM soil. The overburden is otherwise a less silty sand (SP-SM) with 25% hard, subangular gravel to 3 inches and 10% silt (i.e. non-plastic fines with no dry strength). This material is light olive-tan color, only slightly moist, very dense and semi-pervious. It contains 5% hard subangular rocks to 16 inches. Some of the gravel and plus three inch material is decomposed. Occasional large boulders (some over one cubic yard) occur within the overburden.

**IN PLACE TEST DATA**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth</th>
<th>Dry Density</th>
<th>Moisture</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP - 205 @ 5.0'</td>
<td>dry density 118.9 #/Ft³</td>
<td>4.6% moisture on - 3/4&quot;</td>
<td></td>
</tr>
<tr>
<td>TP - 207 @ 5.0'</td>
<td>dry density 106.2 #/Ft³</td>
<td>4.6% moisture on - 3/4&quot;</td>
<td></td>
</tr>
</tbody>
</table>

The following particle-size distribution curve (field determination on dry sieve) is typical of the borrow material:
The bedrock encountered in these excavations included hard pegmatite, granite and biotite gneiss.

AREA UPSTREAM FROM THE RIGHT Wing OF THE DAM
BEHIND JERICO BROOK AND THE RESERVOIR

It is through this area that Jericho Brook will be diverted into the reservoir basin. An aqueduct system, as along the broken profile from test pits (TP-11) to (TP-907), may be constructed to maintain water quality through circulation in the reservoir basin.

In test pits (TP-A, -B, -C, -D, -E, -F) hard and granular bedrock (biotite gneiss) was encountered at depths of 2.5 to 4.5 feet. The overburden is a loamy and pervious alluvium-till with 20% silt and less than 5% gravel. The water level is at or immediately above the bedrock surface. In test pit (TP-F), the furthest downslope of the lettered pits, hard, pink granite was encountered at 6 feet.

Test pits (TP-11, -901 to -903) encountered hard bedrock within 3.5 feet of the surface. The overburden is a moist, silty sand with less than 5% hard angular gravel to 3 inches and 20% silt. Test pits (TP-904, -905) were dug to 10 feet. Bedrock was probably encountered at the bottom of test pit (TP-905). The overburden is dry, pervious and compact gravelly sands and sandy gravels with 10% silt and 40 to 60% hard subangular to subrounded gravel. This coarse material (SP-8M and GP-21) has 30% hard subangular to subrounded cobbles and boulders to 24 inches. Test pits (TP-906 to -910) encountered hard bedrock within 6 feet of the surface. The overburden is wet. It consists of muck, boulders and very little SM soil with 15% inclusions. The overburden includes up to 60% hard angular to subangular cobbles and boulders to 24 inches. The underlying bedrock includes granular textured biotite gneiss, coarse grained granite and pegmatite.

ADDITIONAL BORROW AREAS

Section XX', through the high topographic feature, demonstrates a thick section of till. Drill hole (DH-153) penetrated 69 feet of overburden, which is 25 feet below the water surface elevation in the swamp. The water level in this boring, measured 3 days after completion of the boring, was at 10.5 feet. The bottom 10 feet of this boring consist of lodgment till that was sheared into a depression in the bedrock surface. The overburden is otherwise the usual sandy till; i.e., silty sands with 15% silt and 15% hard angular gravel, semi-pervious to pervious, and dense. Test pit (TP-A) showed the water-worked inclusions within this sandy till. In one such inclusion at 22 feet in drill hole (DH-153) a permeability rate of 2.6 feet per day was recorded.

Test pit (TP-151) and test pit (TP-152) on section LL' were excavated 5 to 10 feet into the sandy till (30% with 10% silt). Refusal at these depths was presumably near bedrock. The overburden contains up to 35% subangular cobbles and boulders to 1 cubic yard.

REFERENCE:
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
B106

DATE
SHEET
D6
<table>
<thead>
<tr>
<th>D.H. No.</th>
<th>Test Section (NX) (Fr. - to)</th>
<th>Pressure Loss (psl/min)</th>
<th>M.L. (It)</th>
<th>Hard (It)</th>
<th>C (psi)</th>
<th>X (fpa)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>greater than 10</td>
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<td>18.1</td>
<td>0.2</td>
<td>0.09</td>
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<td>25.5 - 26.5</td>
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<td>2.1</td>
<td>18.1</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>305</td>
<td>16.5 - 21.5</td>
<td>less than 5</td>
<td>2.6</td>
<td>18.6</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>306</td>
<td>21.5 - 26.5</td>
<td>less than 10</td>
<td>2.6</td>
<td>18.6</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>307</td>
<td>20.0 - 25.0</td>
<td>less than 5</td>
<td>0.0</td>
<td>16.0</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>308</td>
<td>17.0 - 20.0</td>
<td>none</td>
<td>0.0</td>
<td>16.0</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>309</td>
<td>29.0 - 34.0</td>
<td>none</td>
<td>0.0</td>
<td>16.0</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>310</td>
<td>12.0 - 17.0</td>
<td>less than 10</td>
<td>0.0</td>
<td>34.5</td>
<td>none</td>
<td>--</td>
</tr>
</tbody>
</table>

An occasional partial loss of drill water was noted while coring in drill hole (DH-16). A temporary artesian flow, estimated at 2 gpm, was noted from 15 to 16 feet in drill hole (DH-302) and from 37.5 feet in drill hole (DH-16).

**Right Abutment (base of abutment to top of dam)**

Except for the high topographic feature investigated with drill hole (DH-19) and test pit (TP-7), this is an area of extensive outcrops and very thin overburden.

Test pit seven (TP-7) was excavated 11 feet into very dense and impervious till. This material is a silty fine to medium sand with 20% silt and less than 5% gravel. There are occasional decomposed and hard subangular pink granite cobbles to 12 inches. The drill hole (DH-17) encountered similar soil to 27 feet and pegmatite was cored from 27 to 52 feet. The overburden took no water; and except for the test section from 37.0 to 42.0, where packers could not be seated, the bedrock took no water.

The outcrops include hard and durable pegmatite and associated graphic granite. Relatively hard to highly weathered bitite gneiss outcrops between sills of hard pegmatite in the channel bank.

Test pits (TP-9, -11, and -13) were dug 2 feet to bedrock. Test pit (TP-10) was dug to refusal at 5 feet (presumably bedrock). The overburden is generally a firm and pervious E1 till with 5% hard angular rock to eighteen inches. In test pit (TP-10) the water level is at 3 feet; in test pit (TP-11) the water level is at the surface.

Test pit (TP-14), the highest excavation on this abutment, was dug in an obscenely stratified till to bedrock at 9 feet. The material is firm and pervious to semi-pervious. It includes silty sands and sandy silts with pebbles of medium to coarse sand. Some boulders to one cubic yard were encountered. The water level in November 1966 was at 3 feet. In August 1967 water was brought to the surface by running test pitting equipment over the backfilled pit.
the proposed embankment.

The water level is at or near the surface.

Bedrock outcrops in this area. The outcrops include hard and durable pegmatite, graphic granite and some biotite gneiss.

The overburden is up to 19 feet thick. It consists of firm to dense, semi-pervious sands with 15% silt and 10% gravel (51); loose to very compact, pervious, stratified sands with up to 20% silt and 30% gravel (Q t. 51); and cobbles rounded and subrounded cobbles and boulders (40% larger than 3 inches, the minus 3 inch is a loose and very pervious GP with 60% larger than 1/4 inch).

The corel bedrock varied in type and quality. Core recoveries as low as 15, 20 3/4 and 12% for 5 foot runs are recorded. It appears that only hard pegmatite was recovered in core runs that penetrated both the pegmatite and biotite gneiss. The poorest recoveries occurred in the granular, friable and weathered decomposed biotite gneiss. In drill hole (BH-503) an A-rod was driven 1.5 feet into the biotite gneiss between two core runs.

Bedrock was excavated in test pits (T-501, -502). In test pit (T-501) the bedrock from 5 to 6 feet were cut in decomposed rock fragments that broke down to SM soil and thin angular slabs of fresh rock up to 10 inches in minimum dimension. Much water was flowing through this zone. The bedrock (biotite gneiss) at 6 feet is hard and fractured. In test pit (T-502) the biotite gneiss was decomposed and easily excavated from 3 to 6 feet. At 6 feet the rock is hard.

Permeability tests in the overburden are not indicative of the actual leakage through the variable, alluvial outwash material. The inability to effectively seal the casing and high water table and low head conditions resulted in negative test results and insignificant flow.

The overall quality of the bedrock hindered or precluded pressure testing much of the bedrock.

(1) In DI-302, packers couldn't enter the cored rock.
(2) In DI-502, packers couldn't be seated at all.
(3) In DI-901, the packers couldn't be seated from 7.5 to 9.5.
(4) In DI-502, the packers couldn't be seated from 10.5 to 12.5.
(5) In DI-19, the packers couldn't be seated from 17.0 to 18.0

The following pressure test data was obtained:
In drill holes (DH-16, -17) the water level is deep; i.e., 21.4 and 20.5 feet.

In drill hole (DH-15), near the top of dam elevation, the water level is shallow; i.e., 1.7 feet.

The bedrock includes pegmatite, granite, granitized biotite gneiss and biotite gneiss. With the exception of the biotite gneiss, which is slightly friable along fracture, the bedrock is otherwise hard, sound and durable.

Summary of Pressure Testing Results:

<table>
<thead>
<tr>
<th>D.H. No.</th>
<th>Test Section (HX)</th>
<th>Pressure Loss (psi/min)</th>
<th>W.L. (ft)</th>
<th>Head (ft)</th>
<th>Q (gpm)</th>
<th>K (fps)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>11.0 - 16.0</td>
<td>less than 10</td>
<td>1.7</td>
<td>58.7</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>15</td>
<td>15.0 - 20.0</td>
<td>none</td>
<td>1.7</td>
<td>58.2</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>16</td>
<td>20.0 - 25.0</td>
<td>less than 10</td>
<td>1.7</td>
<td>52.4</td>
<td>7.2</td>
<td>3.1</td>
</tr>
<tr>
<td>16</td>
<td>26.3 - 46.0</td>
<td>greater than 10</td>
<td>21.4</td>
<td>67.4</td>
<td>1.6</td>
<td>0.5</td>
</tr>
<tr>
<td>17</td>
<td>31.0 - 40.0</td>
<td>less than 5</td>
<td>20.5</td>
<td>66.5</td>
<td>none</td>
<td>--</td>
</tr>
</tbody>
</table>

Test pit (TP-503) was dug into the steep abutment slope in the downstream section of this glacial till feature. The upper 11 feet of till is typical of the feature; i.e., silty sands with 5% hard angular gravel to 3 inches, 15% silt, moist, dense and semi-porous. The section from 11 to 21 feet is a water-worked inclusion in the till deposit. It is a silty sand with 20% gravelly sand to 4 inch, 20% silt, wet, dense and porous. The high permeability rates in drill hole (DH-17) probably occurred in a similar material.

Foundation (bet on base of abutments)

This area was investigated with four test pits (TP-501, -502, -5, -502) and six drill holes (DH-501, -502, -503, -504, -505 and -16).

This low area was also extensively probed on 50-foot centers. There is up to 6.5 feet of sand, average thickness 1.5 feet, within the base width of...
SUMMARY OF FINDINGS

GENERAL

From October 27 to November 7, 1966, a Unit 3/4-yard backhoe (model 1020) and Caterpillar D-6 bulldozer were used to excavate 61 test pits in the foundation area of the dam and emergency spillway-borrow area. Twenty-six additional test pits were excavated from August 1 to August 8, 1967, using a John Deere backhoe (model JD-350) and Caterpillar D-6 bulldozer. The additional test pits were dug in the foundation and borrow areas of the dike, the area between Jericho Brook and the basin, and emergency spillway area.

The drilling program, 21 borings totaling 676 feet, was accomplished from May 25 to July 13, 1967, using 2 skid-mounted drill rigs -- Acker 85, belt drive, hand feed, 36 h.p. and SKH 35 H, hydraulic, 23 h.p. with an Oliver OC-4 tractor.

Thirteen (13) large disturbed samples have been submitted to the Soils Lab for testing and correlation. Some 75 feet of MX core from six (6) borings have also been submitted for durability tests.

FOUNDATION AREA OF THE DAM

Left Abutment (top of dam to base of abutment)

Four test pits (TP-1, -2, -601, -503) and three drill holes (DH-15, -16, -17) delineate this feature. It is a thick deposit of till (26 to 27 feet) which thins to 10 feet near the top of dam elevation.

Test pits (TP-1, -2, -601) located at the base of the hill on the upstream side of the feature, encountered bedrock at 5 to 7.5 feet.

In test pit (TP-1), 1.5 feet of the bedrock (biotite gneiss) was decomposed and easily excavated. In test pit (TP-2), the biotite gneiss was hard and durable from the surface. The water level in these pits is at the surface. This overburden is a rather non-disruptive bouldery till (SP-SM, CM and ML) with a foot of surface muck. It is wet, loose to firm and pervious to semi-pervious.

Drill holes (DH-15, -17) put down through the top of the feature, encountered the thick section of CM till with 15% silt. This overburden is dense to very dense and generally semi-pervious. In drill hole (DH-16), the overburden accepted no water. In drill hole (DH-17), drill water was lost in a very pervious zone from 18.5 to 20.0 feet and the following permeability test results were obtained:
The outwash material between the till features in the foundation of the dam includes sands, gravel, and openwork cobbles and boulders. This material is pervious to very pervious and is generally compact. It is up to 19 feet thick and directly overlies bedrock. Outwash also occurs in both saddles in the foundation of the dike. This material includes compact and pervious to very pervious sands and gravels to a depth of over 11 feet.

Muck in the foundation of the dam is up to 6.5 feet thick. In the foundation of the dike up to 5.0 feet of muck was probed.

In contrast to the homogeneous glacial till overburden, the bedrock is structurally complex and variable in type and quality. Oliverian granite intrudes the Ammonoosuc volcanics at the dam site.

The Oliverian granite is a weakly foliated, pink to gray, medium to coarse-grained rock. The principal dark mineral is biotite, which constitutes only a few percent of the rock. Pink pegmatite with occasional graphic intergrowths of quartz and feldspar are associated with the granite. The pegmatites range in size from small dikes and sills (defined by our shallow borings) to larger tabular bodies (greater than 25 feet thick). The granite and pegmatite are hard, sound and durable.

The Ammonoosuc volcanics include biotite gneiss, biotite schist and biotite-hornblende schist.

The gneiss is a fine-grained, dark gray rock consisting of quartz, biotite, plagioclase and potash feldspar. It is generally well foliated and has a fragmental texture. As a rule the weathered surface is granular, friable and disintegrates rather readily. The unweathered rock is apparently sound. There is a granitized zone within the gneiss in contact with the Oliverian granite. The more schistose rock is deeply weathered and decomposed.

The rock structure strikes North 10 to 30° East and dips 5 to 30° Southeast.
The site is located in the very rugged White Mountain Region of northern New Hampshire. It is within the dissected and glaciated Upland Section of the New England Physiographic Province.

Drainage from the northeast flank of Black Crescent Mountain forms Jericho Brook which flows past the reservoir basin, a large upland swamp near the watershed divide. The drainage area controlled is 6.5 square miles, of which 5.7 square miles is diverted from Jericho Brook. Continental glaciation in a mountainous area has created a situation where a large reservoir basin is without a correspondingly large drainage area.

Except for the area of the swamp, the overburden consists of glacial till as ground moraine. This till was carried in and on the ice and not deposited until the ice became stagnant and melted. With continued thinning and retreat of the ice sheet to the northwest, a large volume of glacial meltwater flowed over the watershed divide. This huge glacial river entered the watershed at the location of the dike. It flowed through the area of the upland swamp. And in a complex cycle of glacial-fluvial erosion and deposition, it created and destroyed a temporary glacial lake and cut the spectacular "pothole" at the cutlet of the swamp. When deglaciation of the area was complete, the glacial water course was left high and virtually "dry" on the watershed divide. Since immediate post glacial time, the only water that flows into the pothole results from direct rainfall in the basin and unusual flood flows which spill over from Jericho Brook into the upland swamp. In more recent time, that is about 70 years ago, the area was burned over and much of the topsoil was lost to sheet erosion.

The perimeter of the basin is extremely bouldery. Very large boulders, 50 to 150 cubic yards or so, are common. With the exception of the topographic high features in the vicinity of the centerline of the dam and the closed high feature north of the emergency spillway, the glacial till is generally thin or absent. This till was somewhat water-worked at the time of deposition. Lenses of clean, sorted sands with obscure stratification and flow structure around cobbles occurs occasionally within an otherwise massively structured till. The till is rather homogeneous throughout the area. It is an SP-3M and SM soil with 20 to 25% subangular gravel to 3 inches, 10 to 15% silt, 10% coarse sand and 55% fine to medium sands. It is only slightly resist and generally dense and semi-pervious. The coarse fraction, except for occasional decomposed gravel particles and cobbles, is hard, sound and durable. The fine fraction is non-plastic and has no dry strength. The till was excavated with difficulty due to the cobbles, boulders and high percentage of gravel being held firmly within the matrix. This matrix is relatively dry and shows slight iron oxide cementation. The upper few feet of the till (usually one foot but up to 5 or 6 feet depending on the amount of post glacial and recent erosion) contains 5% - 3 to 6 inch, 12% - 5 to 12 inch and 1% - 12 to 18 inch subangular rocks. Beneath the surface till 3 to 16-inch rock constitute 5% of the material. Oversize cubic yard boulders can occur anywhere in the glacial till overburden.
NOTE: This site involves construction of the following:

(a) Dike, 1,665 feet long and 14 feet high, along the watershed divide.

(b) Diversion and flood way to conduct Jericho Brook, 5.66 square miles (3,622 acres) drainage area, into the reservoir basin.

(c) An aqueduct system to maintain water quality through circulation in the reservoir basin.

<table>
<thead>
<tr>
<th>Volume (acre-feet)</th>
<th>Surface Area (acres)</th>
<th>Depth at Dam (feet)</th>
<th>Depth at Dike (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediment pool</td>
<td>5</td>
<td>3</td>
<td>--</td>
</tr>
<tr>
<td>Recreation pool</td>
<td>1,257</td>
<td>135</td>
<td>25</td>
</tr>
<tr>
<td>Floodwater pool</td>
<td>1,417</td>
<td>174</td>
<td>35</td>
</tr>
</tbody>
</table>
NOTE: This site involves construction of the following:

(a) Dike, 1,665 feet long and 14 feet high, along the watershed divide.

(b) Diversion and flood way to conduct dike into brook, 5.66 square miles (3,622 acres) drainage area, into the reservoir basin.

(c) An aqueduct system to maintain water quality through circulation in the reservoir basin.

### STORAGE ALLOCATION

<table>
<thead>
<tr>
<th>Volume (acre-feet)</th>
<th>Surface Area (acres)</th>
<th>Depth at Dam (feet)</th>
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<td>35</td>
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</tbody>
</table>
The site is located in the very rugged White Mountain region of northern New Hampshire. It is within the drained and glaciated Upland Section of the New England Physiographic Province.

Drainage from the northeast flank of Black Pond Mountain forms Jericho Brook which flows past the reservoir basin. A large amount of runoff near the watershed divide. The drainage area centered as 88 square miles, of which 9.7 square miles is diverted from Jericho Brook. Continental glaciation in a mountainous area has created a situation where a large reservoir basin is without a correspondingly large drainage area.

Except for the area of the swamp, the overburden consists of glacial till at ground surface. This till was deposited after the ice had retreated during the period when the ice became stagnant and melted. With continued thinning and retreat of the ice sheet to the northwest, a large volume of glacial meltwater flowed over the watershed divide. This large glacial river entered the watershed at the location of the dam. It flowed through the area of the upland swamp. And in a complex cycle of glacial-fluvial erosion and deposition, it created and destroyed a temporary glacial lake and cut the spectacular "pothole" at the outlet of the swamp. As glaciation of the area was complete, the glacial water courses left high and virtually "dry" on the watershed divide. Since immediate post-glacial time, the only water that flows into the pothole results from direct rainfall in the basin and unusual flood flows which spill over from Jericho Brook into the upland swamp. In more recent time, that is about 70 years ago, the area was burned over and much of the topsoil was lost to sheet erosion.

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The outwash material between the till features in the foundation of the dam includes sands, gravels and openwork cobbles and boulders. This material is pervious to very pervious and is generally compact. It is up to 19 feet thick and directly overlies bedrock. Outwash also occurs in both saddles in the foundation of the dike. This material includes compact and pervious to very pervious sands and gravels to a depth of over 11 feet.

Muck in the foundation of the dam is up to 6.5 feet thick. In the foundation of the dike up to 5.0 feet of muck was probed.

In contrast to the homogeneous glacial till overburden, the bedrock is structurally complex and variable in type and quality. Oliverian granite intrudes the Ammonoosuc volcanics at the dam site.

The Oliverian granite is a weakly foliated, pink to gray, medium to coarse-grained rock. The principal dark mineral is biotite, which constitutes only a few percent of the rock. Pink pegmatite with occasional graphic intergrowths of quartz and feldspar are associated with the granite. The pegmatites range in size from small dikes and sills (defined by our shallow borings) to larger tabular bodies (greater than 25 feet thick). The granite and pegmatite are hard, sound and durable.

The Ammonoosuc volcanics include biotite gneiss, biotite schist and biotite-hornblende schist.

The gneiss is a fine-grained, dark gray rock consisting of quartz, biotite, plagioclase and potash feldspar. It is generally well foliated and has a fragmental texture. As a rule the weathered surface is granular, friable and disintegrates rather readily. The unweathered rock is apparently sound. There is a granitized zone within the gneiss in contact with the Oliverian granite. The more schistose rock is deeply weathered and decomposed.

The rock structure strikes North 10 to 30° East and dips 5 to 30° Southeast.
SUMMARY OF FINDINGS

GENERAL

From October 27 to November 7, 1965, a Unit 3/2-yard backhoe (model 102C) and Caterpillar D-6 bulldozer were used to excavate 41 test pits in the foundation area of the dam and emergency spillway-borrow area. Twenty-six additional test pits were excavated from August 1 to August 8, 1967, using a John Deer backhoe (model 45-350) and Caterpillar D-6 bulldozer. The additional test pits were dug in the foundation and borrow areas of the dike, the area between Jericho Brook and the basin, and emergency spillway area.

The drilling program, 21 borings totaling 674 feet, was accomplished from May 28 to July 11, 1967, using skid-mounted drill rigs -- Aker PG, belt drive, hand feed, 35 h.p. and 26435 H, hydraulic, 26 h.p. with an Oliver 0C-4 tractor.

Thirteen (13) large disturbed samples have been submitted to the Soils Lab for testing and correlation. Some 7.5 feet of NX core from six (6) borings have also been submitted for durability tests.

FOUNDATION AREA OF THE DAM

Left Abutment (top of dam to base of abutment)

Four test pits (TP-1, -2, -501, -503) and three drill holes (DH-15, -16, -17) delineate this feature. It is a thick deposit of till (26 to 27 feet) which thins to 10 feet near the top of dam elevation.

Test pits (TP-1, -2, -601) located at the base of the hill on the upstream side of the feature, encountered bedrock at 5 to 7.5 feet.

In test pit (TP-1), 1.5 feet of the bedrock (biotite gneiss) was decomposed and easily excavated. In test pit (TP-2), the biotite gneiss was hard and durable from the surface. The water level in these pits is at the surface. This overburden is a rather non-discrict bouldery till (SP-SM, SM and ML) with a foot of surface muck. It is wet, loose to firm and pervious to semi-pervious.

Drill holes (DH-15, -17) put down through the top of the feature, encountered the thick section of CM till with 15% silt. This overburden is dense to very dense and generally semi-pervious. In drill hole (DH-16), the overburden accepted no water. In drill hole (DH-17), drill water was lost in a very pervious zone from 18.5 to 20.0 feet and the following permeability test results were obtained:

**REFERENCE:**

**U.S. DEPARTMENT OF AGRICULTURE**

**SOIL CONSERVATION SERVICE**

**PLANTING NO.**

**SHEET 5 OF 15**

**DATE**
<table>
<thead>
<tr>
<th>Text Depth (ft)</th>
<th>Casing Size</th>
<th>W.L. (ft)</th>
<th>Head (ft)</th>
<th>Q (gpm)</th>
<th>K (l/d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.0</td>
<td>4&quot;</td>
<td>20.5</td>
<td>10.6</td>
<td>1.1</td>
<td>22.0</td>
</tr>
<tr>
<td>20.0</td>
<td>4&quot;</td>
<td>20.5</td>
<td>21.0</td>
<td>12.0</td>
<td>175.0</td>
</tr>
<tr>
<td>20.0</td>
<td>4&quot;</td>
<td>18.2</td>
<td>19.2</td>
<td>9.6</td>
<td>105.0</td>
</tr>
<tr>
<td>25.0</td>
<td>4&quot;</td>
<td>20.5</td>
<td>20.5</td>
<td>0.3</td>
<td>3.2</td>
</tr>
</tbody>
</table>

1/ Turbulent flow \( Q/A = 0.34 \text{ fps} \)
2/ Turbulent flow \( Q/A = 0.25 \text{ fps} \)

In drill holes (DH-16, -17) the water level is deep; i.e., 21.4 and 20.5 feet.
In drill hole (DH-15), near the top of dam elevation, the water level is shallow; i.e., 1.7 feet.
The bedrock includes pegmatite, granite, granitized biotite gneiss and biotite gneiss. With the exception of the biotite gneiss, which is slightly friable along fracture, the bedrock is otherwise hard, sound and durable.

Summary of Pressure Testing Results:

<table>
<thead>
<tr>
<th>D.H. No.</th>
<th>Test Section(WX)</th>
<th>Pressure Loss</th>
<th>W.L. (ft)</th>
<th>Head (ft)</th>
<th>Q (gpm)</th>
<th>K (l/d)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>11.0 - 16.0</td>
<td>less than 10</td>
<td>1.7</td>
<td>54.7</td>
<td>none</td>
<td>--</td>
</tr>
<tr>
<td>15.0 - 20.0</td>
<td>none</td>
<td>1.7</td>
<td>51.2</td>
<td>none</td>
<td>--</td>
<td></td>
</tr>
<tr>
<td>16</td>
<td>26.3 - 46.0</td>
<td>greater than 10</td>
<td>21.4</td>
<td>67.4</td>
<td>7.2</td>
<td>3.1</td>
</tr>
<tr>
<td>17</td>
<td>31.0 - 40.0</td>
<td>less than 5</td>
<td>20.5</td>
<td>66.5</td>
<td>none</td>
<td>--</td>
</tr>
</tbody>
</table>

Test pit (TP-503) was dug into the steep abutment slope in the downstream section of the glacial till feature. The upper 11 feet of till is typical of the feature; i.e., silty sands with 5% hard subangular gravel to 3 inches, 15% silt, moist, dense and semi-pervious. The section from 11 to 21 feet is a water-worked inclusion in the till deposit. It is a silty sand with 20% coarse sand to ¾ inch, 20% silt, wet, dense and pervious. The high permeability rates in drill hole (DH-17) probably occurred in a similar material.

Foundation (between base of abutments)
This area was investigated with four test pits (TP-501, -502, -5, -602) and six drill holes (DH-301, -302, -303, -501, -505 and -18).
This low area was also extensively prospected on 50-foot centers. There is up to 4.5 feet of rock, average thickness 1.5 feet, within the line width of...
the proposed embankment.

The water level is at or near the surface.

Bedrock cutouts in this area. The cutouts include hard and decomposed pegmatite, graphitic granite and some biotite gneiss.

The overburden is up to 19 feet thick. It consists of firm to dense, non-porous sands with 15% silt and 10% gravel (20); loose, very compact, porous, stratified sands with up to 2% silt and 50% gravel (5); and openwork rounded and subrounded cobbles and boulders (all larger than 3 inches). The minus 3 inch is a loose and very pervious gravel (1... larger than 4 inch).

The cored bedrock varied in type and quality. Core recoveries as low as 12, in. in 0.8 and 2.5 for 5 feet runs are recorded. It appears that only hard pegmatite can recover in core runs that penetrated both the pegmatite and biotite gneiss. The poorest recoveries occurred in the granular, friable and sometimes decomposed biotite gneiss. In drill hole (P-505) an A-rod was driven 1.5 feet into the biotite gneiss between two core runs.

Bedrock was excavated in test pit (P-501, 502). In test pit (P-501) the bedrock from 5 to 6 feet were cut in decomposed rock fragments that broke down to 124 soil and thin angular slabs of fresh rock up to 18 inches in maximum dimension. Each water was flowing through this zone. The bedrock (biotite gneiss) at 6 feet is hard and fractured. In test pit (P-502) the biotite gneiss was decomposed and easily excavated from 3 to 6 feet. At 6 feet the rock is hard.

Permeability tests in the overburden are not indicative of the actual leakage through the variable, alluvium outwash material. The inability to effectively test the cored and high water table and low head conditions resulted in negative test results and insignificant flow.

The overall quality of the bedrock hindered or precluded pressure testing high if the bedrock.

(1) In BH-112, packers couldn't enter the cored rock.
(2) In BH-12, packers couldn't be seated at all.
(3) In BH-17, the packers couldn't be seated from 14.5 to 20.5.
(4) In BH-11, the packers couldn't be seated from 14.5 to 14.5.
(5) In BH-11, the packers couldn't be seated from 17.5 to 19.

The following pressure test data was obtained:
An occasional partial loss of drill water was noted while boring in drill hole (DH-18). A temporary artesian flow, estimated at 2 gpm, was noted from 15 to 18 feet in drill hole (DH-302) and from 37.5 feet in drill hole (DH-18).

Right Abutment (Base of abutment to top of dam)

Except for the high topographic feature investigated with drill hole (DH-19) and test pit (TP-7), this is an area of extensive outcrops and very thin overburden.

Test pit seven (TP-7) was excavated 11 feet into very dense and impervious till. This material is a silty fine to medium sand with 25% silt and less than 5% gravel. There are occasional decomposed and hard subangular pink granite cobbles to 12 inches. The drill hole (DH-19) encountered similar soil to 27 feet and pegmatite was cored from 27 to 52 feet. The overburden took no water; and except for the test section from 37.0 to 42.0, where packers could not be seated, the bedrock took no water.

The outcrops include hard and durable pegmatite and associated graphic granite. Relatively hard to highly weathered bitite gneiss outcrops between sills of hard pegmatite in the channel bank.

Test pits (TP-9, -11, and -13) were dug 2 feet to bedrock. Test pit (TP-10) was dug to refusal at 6 feet (presumably bedrock). The overburden is generally a firm and pervious SM till with 5% hard angular rock to eighteen inches. In test pit (TP-10) the water level is at 3 feet; in test pit (TP-11) the water level is at the surface.

Test pit (TP-11), the highest excavation on this abutment, was dug in an obscurely stratified till to bedrock at 9 feet. The material is firm and pervious to semi-pervious. It includes silty sands and sandy silts with pockets of medium to coarse sand. Some boulders to one cubic yard were encountered. The water level in November 1966 was at 3 feet. In August 1967 water was brought to the surface by running test pitting equipment over the backfilled pit.
The bedrock encountered in these excavations includes hard pegmatite, granite and biotite gneiss.

**AREA UNTREATED FROM THE PRESUMED AREA OF THEfällt**

It is through this area that Jericho Brook will be diverted into the reservoir basin. An aqueduct system, as along the broken profile from test pits (TP-11) to (TP-907), may be constructed to maintain water quality through circulation in the reservoir basin.

In test pits (TP-A, -B, -C, -D, -E, -F) hard and granular bedrock (biotite gneiss) was encountered at depths of 1.5 to 2.5 feet. The overburden is a layer and part an alluvial tilled with 20% silt and less than 2% gravel. The water level is at or immediately above the bedrock surface. In test pit (TP-F), the furthest drilling of the lettered pits, hard, pink granite was encountered at 6 feet.

Test pits (TP-11, -902 to -903) encountered hard bedrock within 3.5 feet of the surface. The overburden is a moist, silty sand with less than 5% hard angular gravel to 3 inches and 20% silt. Test pits (TP-904, -905) were dug to 1 feet. Bedrock was probably encountered at the bottom of test pit (TP-905). The overburden is dry, pervious and compact gravely sand and sandy gravel with 10% silt and 60 to 80% hard subangular to subrounded gravel. The coarse material (SP-SM and CP-CN) has 30% hard subangular to subrounded cobbles and boulders to 1 inch. Test pits (TP-906 to -910) encountered hard bedrock within 6 feet of the surface. The overburden is wet. It consists of mud, mudstone and very little SM soil with SM-EL inclusions. The overburden includes up to 60% hard angular to subangular cobbles and boulders to 1 inch. The underlying bedrock includes granular textured biotite gneiss, coarse grained granite and pegmatite.

**ADDITIONAL EXCAVATION AREAS**

Section 911, through the high topographic feature, demonstrates a thick section of LL. Drill hole (DH-15) penetrated 8 feet of overburden, which is more than 2 feet below the water surface elevation in the swamp. The water level in this boring, measured 9 days after completion of the boring, was at 11.6 feet. The bottom 10 feet of this boring consist of loam and till that is sheared into a depression in the bedrock surface. The overburden is mostly the usually sandy till: i.e., silty sand with 15% silt and 10% hard angular gravel. Semi-pervious to pervious, and dense. Test pit (TP-11) showed the water-marked inclinations within this sandy till. In one such inclusion at 22 feet in drill hole (DH-195) a permeability rate of 2.6 feet per day was recorded.

Test pits (TP-105) and test pit (TP-151) on section 911 were excavated 5 to 1 feet into the sandy till (M with 15% silt). Bedrock at these depths was irregularly in bedrock. The excavation contains up to 10% subangular cobbles and boulders to 1 cubic yard.  

**REFERENCES:**

**U.S. DEPARTMENT OF AGRICULTURE**
**SOIL CONSERVATION SERVICE**

**DRAWING NO.:** B-20

**DATE:**
There are numerous hard, subangular surface rocks in the above discussed areas, that is:

35 cubic yards per acre from 6 to 18" in size
173 cubic yards per acre from 18" to 36" in size
plus
rocks over one cubic yard as noted on sheets 2 and 3

EMERGENCY SPILLWAY AND PRIMARY BORROW AREA

This area has many hard, subangular surface rocks. There are 53 cubic yards per acre from 3 to 18 inches and 311 cubic yards per acre from 18 to 36 inches. Boulders well over one cubic yard are common. Much of the topsoil in this area has been lost to sheet erosion following forest fires.

The upper few feet of the till (usually one foot but up to 6 feet) are a more rocky CM soil. The overburden is otherwise a less silty sand (SP-SM) with 25% hard, subangular gravel to 3 inches and 10% silt (i.e. non-plastic fines with no dry strength). This material is light olive-tan color, only slightly moist, very dense and semi-pervious. It contains 5% hard subangular rocks to 18 inches. Some of the gravel and plus three inch material is decomposed. Occasional large boulders (some over one cubic yard) occur within the overburden.

IN PLACE TEST DATA

TP - 205 @ 5.0'  dry density 118.9 #/Ft³
                   4.6% moisture on - 3/4"
TP - 207 @ 5.0'  dry density 106.2 #/Ft³
                   4.6% moisture on - 3/4"

The following particle-size distribution curve (field determination on dry sieve) is typical of the borrow material:

REFERENCE:  U.S. DEPARTMENT OF AGRICULTURE
            SOIL CONSERVATION SERVICE

DRAWING NO.  B-121

SHEET OF  
DATE
The underlying bedrock generally consists of hard, sound and durable, weakly foliated Oliverian granite, hard granular textured biotite gneiss and occasional highly weathered schists.

Exceptions:

Four feet of bedrock (biotite gneiss) was easily excavated from 5 - 9 feet in test pit (TP-211)

Eight feet of very soft biotite-hornblende schist was cored from 10 to 19 feet. Core recovery in similar material was very low.

DIKE

There are no bedrock outcrops in this area. Surface rocks are common but they are not as numerous as at the dam. The areal extent of outwash and till material was determined in the test pitting program. The vertical extent of these materials and the depth to bedrock was not determined.

Except for the area of the saddles, the overburden is glacial till. Up to 5 feet of rock was probed in the saddles. The underlying outwash deposits consist of 0.5 to 2.0 feet of compact and semi-pervious gray SM soil (15% silt and 15% hard, subangular to subrounded gravel) over compact and pervious SP-SM and GP-SM soil. (40 - 40% hard rounded gravels to 3 inches, 5% silt and 15% hard, rounded cobbles and boulders to 18 inches). The water level is at or near the surface and the pit walls were caving. In test pit (TP-57) the ground water was a bright iridescent orange color.

The glacial till in the foundation and adjacent borrow areas is an SM and SP-SM soil. It has 10 - 25% hard, subangular gravel to 3 inches and 10 to 15% silt. It is dense and semi-pervious to pervious. It includes up to 25% hard, subangular rocks from 3 to 24 inches.

In the borrow areas, oversize material includes hard, subangular to subrounded rock as follows:

- 5% 3" - 6"
- 5% 6" - 12"
- 10% 12" - 24"

REFERENCE:

U.S. Department of Agriculture
SOIL CONSERVATION SERVICE

7-22

drawing no.
1. Sample 221.1 and the correlation samples (namely: 4.1, 59.1, 207.1 and 221.1) represent our fill material and most of the overburden at the site. This material is relatively dry (4.6% moisture), somewhat cemented and difficult to excavate. It includes hard, subangular to subrounded rocks as follows:

- 3% 3" - 6"
- 2% 6" - 12"
- 1% 12" - 18" plus occasional larger boulders

This soil in-place is dense and semi-pervious. Most of the cobbles and boulders are near the surface.

2. Surface boulders in the borrow and emergency spillway area total:

- 53 cubic yards per acre 6" - 18"
- 31 cubic yards per acre 18" - 36"

3. Surface boulders elsewhere on the site total:

- 35 cubic yards per acre 6" - 18"
- 17 cubic yards per acre 18" - 36"

4. There is also 4,250 cubic yards of boulder rock excavation within the grid survey (See drawing sheets 2 and 3).

5. Core material and quality drain fill materials in sufficient quantity are not available on site or within reasonable distance of the site.

AT THE DAM

6. Sample 7.1 represents the overburden in the high "topo" feature that crosses the right abutment.

7. Sample 511.1 is similar to the overburden in the high "topo" feature along the left abutment. Sample 503.2 represents a very pervious inclusion in an otherwise semi-pervious abutment.

8. The foundation is wet except for the high ground along the centerline.

9. There is up to 6.5 feet of muck across the low area in the foundation. The bedrock surface is very irregular in this area. There is some 10 feet of near vertical relief on the bedrock surface. There are outcrops and bedrock was cored at depths of up to 19 feet.

10. The overburden across the low area is thick and variable in type, strength and permeability. Some strata within this outcrop are observed.
to be very pervious. The most pervious material is the "open-work" cobbles and boulders (see TP-9 from 2' to 5 feet). Sample 5061 represents the finer gradation of the variable outwash.

11. The bedrock has overall adequate "in-place" strength. Leakage rates through the bedrock are minor or insignificant.

DIVERSION AREA

12. There are many surface boulders. The overburden is generally shallow. Sample 1.1 represents most of this material. The water level is at or a few feet above the hard bedrock surface.

13. Sample 506.1 represents a limited area of pervious, coarse-grained soil.

BOUNDARY AND PREDOMINANT UTILITIES AREA

14. The topographic map of the bedrock is contoured to the top of the bedrock surface. A small amount of this rock, as in TI-11 (5 to 9 feet) and DI-5 (10 to 16 feet), can be excavated as earth.

15. Additional borrow is available to the west of this area (See TP's -102, -101, -100).

16. The pegmatite and Oliverian granite are evidently durable rocks. The schistose rock is a non-durable, soft, degradable rock. These rock types can be selected or rejected for use as riprap by field examination.

17. The selection or rejection of the biotite gneiss for use as riprap will present problems as noted:

(a) Where granitized, it has the hardness and durability of the granite.

(b) Elsewhere, it is hard and granular with friable surfaces or decomposed to some depth (4' in TP-111, 3' in TI-161, 5' in TI-101).

Abrasion and soundness tests may assist in the determination of suitability for riprap.

ADDITIONAL BORROW AREA

18. The Nine Knob centered at DH-15 is a good source of fill. More borrow is available to the west of this area (See TP's -151 and -152). The material is represented by sample 2.1. It correlates to the fill material (See item 1).

REFERENCE:

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

DATE
DIKE

19. Aside from the two low saddles, the foundation and borrow soils are represented by sample 53.1 and 59.1. This overburden is generally semi-pervious and dense. It correlates to the fill material (see item).

20. Up to 5 feet of slack was probed in the low, wet saddle. Beneath the slack there was 1 to 2 feet of compact and semi-pervious clayey sands (Sample 5.1) over compact and very pervious, gravelly and sandy (Sample 6.1) with 15% hard, rounded cobbles and boulders to 18 inches.

21. The depth to bedrock is an assumed depth in this area and the thickness of the outwash is unknown.
ENGINEERING INTERPRETATION, RECOMMENDATIONS AND CONCLUSIONS

General

The structure, a multiple-purpose flood control - recreation dam, is located in a large basin on the northerly side of Jericho Brook, in Berlin, New Hampshire.

Centerline of Dam

Various centerline of dam locations have been studied. A centerline along the high topographical features, as proposed in planning, would probably require the least amount of fill; but this particular centerline would propose difficulty during construction due to the number of angles required for layout along the high features.

The demonstrated centerline was chosen for the following reasons:

1. On the left abutment, the proposed centerline is located on the downstream side of the high features. By locating the dam upstream of these features, pervious zones, such as found in NS-27 from 16.5 to 20.0 feet, could be cut-off with the blanket effect of the fill against the slope. Also, the pervious zones would be in a more desirable location in the foundation.

2. The high topographical feature upstream of the centerline in the vicinity of station 20+00 and 0+00 could be used as a source of borrow. This could be excavated to a level that is approximately the elevation of the surrounding area. Any stratifications of sand, such as found in TP-4, therefore, would not be in the foundation of the dam.

3. On the right abutment, in the vicinity of station 20+00 and 0+00, the centerline goes through a high feature. Laboratory tests did not indicate any stratification of pervious zones in the feature. The bulk of the feature lies upstream of the centerline; and if any stratification is present, it would be more desirable having it downstream of the centerline rather than upstream.

4. For the proposed centerline, there would be a better opportunity to extend the cut-off to 64 rock in the area of rock features. This possibility should be further considered.

REFERENCES

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

B-126
DEAD RIVER WATERSHED PROJECT
MULTIPLE PURPOSE DAM NO. 1

RAINAGE AREA
FLOOD STORAGE
WATER SURFACE AREA
HEIGHT OF DAM
VOLUME OF HILL

41.47 ACRES
653 ACRE FT
132 ACRES
46 FEET
21,946 CUBIC YARDS
151,620 CUBIC YARDS

BUILT UNDER THE WATERSHED PROTECTION AND FLOOD PREVENTION ACT

by
COOS COUNTY SOIL CONSERVATION DISTRICT,
CITY OF BERLIN, NEW HAMPSHIRE

and the
NEW HAMPSHIRE WATER RESOURCES BOARD

with assistance of
U.S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE
1968

INDEX
turn over of the water throughout the site if the Jericho Brook base flow is allowed to enter the recreation pool in the vicinity of the principal spillway. To lessen such a potential short circuiting, flows as large as possible will be diverted into the reservoir as far upstream as is feasible.

It is quite probable that this water quality diversion will be a small pipe or channel conducting flow from Jericho Brook to a point well upstream of the dam.
The dike will be subject to small heads during flood times and no head during normal conditions. It is anticipated that no unusual design measures are required.

The only foundation treatment required appears to be the removal of the muck encountered in the two saddles.

A cut-off will be excavated 3 or 4 feet into the ground to penetrate through the zone of frost action.

Drainage measures are expected to be accomplished by selective placement. Outwash material from the saddles and the coarsest of the borrow materials will be placed in a downstream blanket zone. In the saddles and low areas, where the dike is subject to the greatest head, oversize rocks can be wasted on the downstream face and toe of the structure.

Proposed side slopes for the dike are 3:1 both upstream and downstream.

Adequate borrow material is available for construction of the dike from the high topographical features located both upstream and downstream of the centerline. It is important that all of these areas used are dressed and neatly graded and provided with positive drainage because of the recreation involved.

Local complications preclude diverting waters from one watershed to another. Therefore, the use of this saddle for an emergency spillway will not be allowed.

Figure 1: Rock Diversion

This diversion is primarily required to divert a major portion of the dike rock into this relatively economical site.

The color, surface Outcrops and the irregular topography indicate that the diversion will involve rock excavation.

Attempts are being made to determine the most economical method to accomplish the diversion. The varying rock depth and irregular rock surface complicates these studies.

The final solution is expected to be a wide floodway involving special rock excavation along with some ailing and ripraping techniques.

This structure is to be used for multi-purpose recreation. It is interesting that the water quality be adequate for recreation. The question exists whether there will be sufficient
If the quantity of oversize is adequate, a zone of riprap on
the lower upstream face of the dam will be called for. A zone
of this nature, particularly on the right abutment where the
dam also serves as a dike for the Jericho Brook diversion,
would keep the face of the slope from eroding during flood
flows. Bedrock excavated from the emergency spillway could be
used as riprap. The quantity of oversize material appears
large enough so that any excess rock not used as riprap will
be considered for use in a fill zone containing oversize
material up to 18", or as a rock fill.

Due to the expansive pool area and the easterly trend of the
valley, the prevailing west winds would make the dam vulner-
able to wave action. A zone of riprap would help in stabil-
izing the upstream slope of the dam.

Proposed side slopes for the dam are 3:1 upstream and 2½:1
downstream.

The primary borrow source is the emergency spillway area. The
emergency spillway excavation (50,000 c.y.) will not provide
sufficient borrow, so the area adjacent to the spillway will
be used for borrow. From the recreation aspect, it is impor-
tant that the area be sloped, neatly dressed, and provided
with positive drainage.

The high topographical feature located upstream of the center-
line in the area to the right of the principal spillway will
be used as a source of borrow. It will be excavated to some
predetermined elevation well below recreation pool level.

The area west of the high feature was investigated as a borrow
source. The overburden was relatively rocky and somewhat thin
in places.

The Jericho Brook diversion will be another source of borrow
materials. These materials will be used in any necessary
diking to contain flows in the water quality circulation
system, or in the dam itself, depending on the volume of the
excavations. Rock excavation in this area is expected to be
small. Rock excavation from this source will be used as rip-
rap.

Dike

A 1,000-foot dike is required on the upper end of the storage
bassin to prevent flood flows from entering the Ammoncusuc
Watershed.
2. Any unsuitable materials in the foundation will be removed particularly in the area of the left abutment from station 31+00 to 37+00.

3. The steep slope on the left abutment next to the principal spillway location should be cut back to an acceptable slope.

Emergency Spillway

The emergency spillway is located on the left abutment. The layout was based on a study of the bedrock contours of the area. An economic study was developed to determine the most economic proportioning of the structure. The study shows a spillway with a 100-foot bottom width, a crest elevation of 1342.0, and a top of dam elevation of 1371.0 to be the most economical layout.

The emergency spillway layout does not yield sufficient fill to construct the dam. It will be necessary to go to a borrow area for additional fill. The rock excavated from the spillway will be used for riprap as required.

The control section will bottom on bedrock. The side slopes proposed are 1:1 in rock and 4:1 in earth.

An additional safety factor has been provided. The emergency spillway crest was set one foot above the elevation required for a 100-year long-duration storm. This reduced the amount of rock excavation and still left the control section on bedrock.

The emergency spillway outlets into a drainage way that enters the stream approximately 2,000 feet below the structure. Therefore, flowing water in the emergency spillway will not impinge on the dam.

The profile of the emergency spillway is such that it provides maximum head. The exit channel is directed away from the structure and a considerable distance downstream before discharge is permitted on natural relief.

Borrow and Borrow Areas - Dam

All of the available borrow, from the emergency spillway and borrow areas, appears to be homogeneous.

It appears as if more compactive effort could be used in a core mix because of the lack of good quality core material. The core zone could be widened to incorporate oversized
required. Material for such a drain would undoubtedly have to come from materials located off site.

It may be possible to relieve the pervious zone found in DH-17 from 16.5 to 20.0 by tapping into that strata with a trench if it is encountered during foundation excavations.

The approximately 5,000 cubic yards of GP-EM material as found in TP-04 might be considered a usable pit run blanket material in the low area (station 22+00 to 28+00).

The geologist has indicated that leakage through bedrock is minor.

Other factors to be considered in drainage design are as follows:

1. Sand and gravel of significant quantity and quality are not available on site or within close proximity (except as noted above).

2. The closest source of pit run sands and gravels occurs in the Lead River Valley adjacent to NH Route 110, approximately 4 miles away.

Principal Spillway

The principal spillway location is proposed at the base of the hill feature on the left abutment. This appears to be the best location. By moving the alignment to the left into the slope, a sizable excavation will be required. Moving the alignment to the right would result in trigging the two outcrops of bedrock which would be undesirable. Moving the alignment to a location where the pipe would be all on bedrock would not allow a pond drain system without excessive excavation of rock.

It appears that a cantilever outlet with a plunge pool will be the appropriate energy dissipater for the pipe outlet. If bedrock is encountered in the plunge pool, it will be blasted out to the required grades.

压实处理 - 水坝

1. Any muck in the foundation area of the dam will be excavated to a suitable material prior to the placement of any fill. It will be important during construction of the spillway excavation include a muck around nested boulders and bottoms out on a firm material.

REFERENCE:  U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
B-129
From the high feature on the right abutment to Jericho Brook a cut-off can be extended to bedrock with shallow cuts of 0 to 7 feet.

**Drainage - Dam**

Stratification of the water-worked till and the perviousness of the alluvium outwash indicates the need for drainage measures.

On the left abutment from DH-15 toward the high features, bedrock could be reached with a standard trench drain. In the area of the high feature, bedrock is over 20 feet deep as indicated by DH-16 and DH-17. To what depth should a trench drain be extended in the area of these features? A pipe collector could be installed and outleted in the drainage way, between the high features, in the vicinity of station 31+400.

At the base of the high feature, in the area of the principal spillway, a trench could be extended around the toe, beyond the embankment, to intercept any stratifications or lenses present.

In the flat area between the base of the abutments, the outwash is up to 19 feet deep as indicated by DH-505. A pipe collector drain is recommended in this area. A blanket drain may also be necessary if the embankment material is not compatible with the foundation material.

Openwork cables and boulders are described as representing portions of the overburden in this area. What measures should be taken in the design and construction phases if it is anticipated that they present a serious problem?

The foundation in this area appears as if consolidation will present no problem.

With the foundation being so pervious and ground water at the ground surface, it is expected that dewatering to an elevation close to bedrock will be extremely difficult. To what depth should a trench be extended across this area? Should any other special measures be taken in this area?

The high feature investigation by DH-19 shows 27 feet of overburden. To what depth should the trench be extended in the area of deep overburden?

Because the fill is generally homogeneous, and there is some question as to obtaining a downstream shell none significantly different than the core form, a secondary drain may be...
Increased if the cut-off were placed upstream of the centerline.

5. Having only one angle in the dam would simplify construction.

Cut-off - Dam

The fact that this structure is a multiple-purpose recreation dam, with a 126-acre lake and a maximum depth of 25 feet, would indicate the need for a cut-off.

A cut-off will disclose and obstruct any stratifications or lenses found in the water-worked till of the foundation. A good quality core material is not available on site, so any cutting off will have to be accomplished with the available soils on site.

It appears as though an upstream cut-off can be extended to bedrock on the left abutment with cuts of 10 feet or less until you reach the high feature near the stream. In this particular feature, bedrock is 27 feet deep as indicated by PN-17. It is not believed necessary to extend the cut-off to bedrock in this area. Possibly extending it 10 feet or so to lengthen the flow path would be adequate. A blanket could be placed on the face of the slope and extended some distance upstream if necessary.

In the area of the principal spillway, bedrock was encountered from 6 to 11 feet. Some of it was highly weathered and easily excavated with a backhoe. It is anticipated that zones of weathered rock carrying much water will be removed. To the right of the principal spillway there is an outcrop of bedrock, indicating some sharp relief in the bedrock surface.

Found the foundation area, between the base of the abutments, was encountered from 0-11 feet upstream of the centerline. It appears that a positive cut-off can be realized.

The water is at the ground surface in much of this area, so it is expected that water will be a major problem during construction.

A cut-off near the base of the right abutment, bedrock at 7 feet as indicated by PN-17. To what depth the cut-off be extended in this area? PN-16 and CP-7 are to be very dense and impervious. A depth of 5 feet to penetrate the surface zone may be adequate for
Figure 2 - Looking north along the top of the dam embankment from the alignment break.

Figure 3 - Looking south along the top of the dam embankment from the alignment break.
Figure 4 - Looking along the top of the dam embankment from the south abutment.

Figure 5 - Looking toward south abutment at vehicle tracks near downstream toe of dam.
Figure 6 - Looking toward north abutment at vehicle tracks near downstream toe of dam.

Figure 7 - Vehicle tracks along downstream toe near principal spillway discharge channel.
Figure 8 - Looking north at principal spillway outlet pipe and toe drains.

Figure 9 - Closeup of principal spillway outlet pipe and toe drains.
Figure 10 - Looking at a spring 100 feet downstream of the toe of dam near the south abutment.

Figure 11 - Looking southeast across emergency spillway and at north end of dam embankment.
Figure 12 - Looking at north end of dam embankment.

Figure 13 - Looking downstream from north bank of Jericho Lake at emergency spillway discharge channel.
Figure 14 - Looking north at principal spillway riser.

Figure 15 - Looking east at emergency spillway discharge channel.
Figure 16 - Looking south along crest of dike.

Figure 17 - Looking north along crest of dike from south abutment.
Figure 18 - Looking upstream from the crest of the dam embankment at Jericho Lake.

Figure 19 - Looking downstream at the principal spillway discharge channel.
APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
SITE NO. 1 DEAD RIVER DAM
BERLIN, NEW HAMPSHIRE
REGIONAL VICINITY MAP

LEGEND
- First Class Roads
- Second Class Roads
- Watershed Boundary
- Streams
- Railroad
- Township Line
- Books
- Towns
- White Mountain National Forest Boundary
- Woods Road

SCALE IN MILES

MAP BASED ON, DEAD RIVER WATERSHED PROJECT COVER SHEET MAP. U.S. DEPARTMENT OF AGRICULTURE, SOIL CONSERVATION SERVICE.
DEAD RIVER SITE #1  
TEST FLOOD ANALYSIS

(JERICHO BROOK)

DRAINAGE AREA: 698 M^2

STORAGE: 
1. NORMAL POOL (1352 m^3) = 1239.6 ACRE-FEET
2. EMERGENCY SPILLWAY CREST (182 m^2) = 2795.0 ACRE-FEET

SIZE CLASSIFICATION: INTERMEDIATE
HAZARD CLASSIFICATION: SIGNIFICANT

TEST FLOOD: PMF

DETERMINATION OF PROBABLE MAXIMUM FLOOD (PMF)

1. USE TOE/NEE PMF PEAK FLOW RATES GRAPH-W/BASIN (SLOPE IS 685 FT/MI.)
   IN MOUNTAINOUS CATEGORY; FLOW RATE IS 2130 CSM

   PMF = 2130 CFS/SQ. MILE x 6.48 SQ. MILES = 13802 CFS

2. FROM SOIL CONSERVATION SERVICE DESIGN DATA, A FREEBOARD
   HYDROGRAPH HAS BEEN DEVELOPED FOR A 6-HOUR
   STORM WITH 20.3 IN. RAINFALL WHICH GENERATES 15.6 IN.
   RUNOFF RESULTING PEAK INFLOW IS 16870 CFS

DUE TO GREATER DETAIL OF SCS STUDY, THE TEST FLOOD
WILL BE 16870 CFS. THIS WILL BE CONSIDERED THE PMF AS
THE PROJECT DESIGN FLOOD > RECOMMENDED SPILLWAY DESIGN FLOOD (PMF)
ROUTING OF AN INFLOW VALUE OF 16870 BY SCS RESULTED
IN AN OUTFLOW DISCHARGE OF 7975 CFS. THIS CORRESPONDS
TO AN ELEVATION OF 1370.9 MSL ON THE RATING CURVE
DEVELOPED FOR THE PRINCIPAL AND EMERGENCY SPILLWAYS
(SEE APPENDIX B). THE CREST THE DAM IS 1371.0 (MSL).

SUMMARY: THE TEST FLOOD IS THE MAXIMUM CAPACITY
OF THE PRINCIPAL AND EMERGENCY SPILLWAYS COMBINED
JUST BEFORE THE DAM EMBANKMENT IS OVERTOPPED.

NOTE: SEE PLATE TITLED, "FLOOD ROUTING - EMERGENCY
SPILLWAY HYDROGRAPH", APPENDIX "B" FOR
DAM RATING CURVE.

D-2
DEAD RIVER SITE #1 (TERICO BROOK)

I. GENERAL DISCUSSION OF ANALYSIS

A. DETERMINE DOWNSTREAM HAZARD CLASSIFICATION, FAILURE OF THE DAM WILL BE CONSIDERED WITH THE WATER SURFACE ELEVATION OF UPSTREAM IMPOUNDMENT AT TWO DIFFERENT ELEVATIONS:

1) WATER SURFACE AT NORMAL POOL (RECREATION POOL) ELEVATION OF 1352' MSL.

2) WATER SURFACE AT MAXIMUM POOL ELEVATION OR THE ELEVATION BEFORE FLOW OCCURS IN THE EMERGENCY SPILLWAY; CORRESPONDS TO CREST ELEVATION OF EMERGENCY SPILLWAY AT 1362' MSL. (NOTE: DUE TO THE EXTREMELY LARGE CAPACITY OF THE EMERGENCY SPILLWAY, IT IS UNREALISTIC TO ASSUME MAXIMUM POOL ELEVATION AT TOP OF DAM EMBANKMENT — ELEVATION 1371' MSL.)

IN BREACH ANALYSIS IT IS UNREALISTIC TO ASSUME THE BREACH WIDTH WOULD BE \( \frac{1}{4} \) (DAM WIDTH) = \( \frac{1}{4} \times 3035' = 1214' \). THEREFORE, ASSUME BREACH WIDTH OF 100'.

B. METHODOLOGY EMPLOYED FOR THE CALCULATION OF BREACH DISCHARGES, USING FORMULA:

\[ Q_1 = \frac{8}{27} W_0 \sqrt{Y_0}^{3/2} \]

WHERE:

- \( Q_1 \) = BREACH DISCHARGE, CUBIC FEET PER SECOND
- \( W_0 \) = BREACH WIDTH, FEET
- \( Y_0 \) = ACCELERATION DUE TO GRAVITY, 32.2 ft/sec²
- \( Y_0 \) = DIFFERENCE OF POOL ELEVATION FROM UPSTREAM INVERT, FEET

NOTE: UPSTREAM INVERT = 1329.3' MSL/

BREACH DISCHARGE FORMULA FROM "RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPH, COE, APRIL 1978.

D-3
DESCRIPTION OF DOWNSTREAM REACHES
FROM SITE NO. 1, DEAD RIVER DAM

BASED UPON PHYSICAL CHARACTERISTICS THE WATER-COURSE HAS TWO DISTINCTLY DIFFERENT REACHES BELOW THE DAM.

(1) REACH 1 IS FROM THE DAM TO A POINT APPROXIMATELY 0.25 MILES DOWNSTREAM OF THE JERICHO BROOK BY ROUTE 110, OR A TOTAL DISTANCE OF APPROXIMATELY 1.5 MILES DOWNSTREAM (EAST) OF THE DAM. THIS REACH HAS A STEEP SLOPE, HEAVILY WOODED OVERBANKS, AND A ROCKY CHANNEL BOTTOM. THERE ARE NO INHABITED STRUCTURES LOCATED IN THIS REACH WHICH WOULD BE AFFECTED BY THE BREACH OF THE DAM (AT NORMAL POOL ELEVATION).

(2) REACH 2 IS FROM THE POINT (1.75 MILES DOWNSTREAM) WHERE JERICHO BROOK TURNS SHARPLY TO THE SOUTHEAST AND BECOMES KNOWN AS DEAD RIVER. FIELD INSPECTION HAS REVEALED THAT JERICHO BROOK AND DEAD RIVER ARE THE SAME WATERSHED TO THE JUNCTION OF THE DEAD RIVER WITH THE ANDROSCOGGIN RIVER 3.0 MILES DOWNSTREAM. IN THIS REACH THE DEAD RIVER HAS A WIDE FLAT FLOODPLAIN WITH CONSIDERABLE STORAGE AND HAS AN EXTREMELY FLAT CHANNEL GRADIENT. ON THE SOUTH BOUNDARY OF THE FLOODPLAIN IS THE GRAND TRUNK CANADIAN NATIONAL RAILROAD, ROUTE 110 AND NUMEROUS COMMERCIAL, INDUSTRIAL, AND RESIDENTIAL BUILDINGS. IN ADDITION, FOR THE LAST 0.75 MILES OF ITS COURSE, THE DEAD RIVER FLOWS THROUGH A HEAVILY URBANIZED AREA OF THE CITY OF BERLIN INCLUDING THE CENTRAL BUSINESS DISTRICT.
II. SUMMARY OF BREAK WITH RESERVOIR AT NORMAL OR RECREATION POOL LEVEL (1352.0 FT. MSL)

(A) BREACH DISCHARGE, Q, DETERMINATION

\[ Q_1 = \frac{8}{27} \times \frac{W^2}{g} \times \frac{1}{Y_1^{3/2}} \]

- \( W = 100\) FEET
- \( Y_1 = 5.67 \) FEET / SECOND
- \( Y_2 = (1352.0 - 1329.3) \) FEET = 22.7 FEET

\[ Q_1 = \frac{8}{27} \times 100 \times 5.67 (22.7)^{3/2} = 18169 \text{ CFS} \]

ASSUME MINIMAL ANTecedENT DISCHARGE, \( Q_2 \)

\[ Q_2 = \text{DISCHARGE OVER PRINCIPAL SPILLWAY (TWO-STAGE RISER)} = 25 \text{ CFS} \] (STAGE IS 2 FEET)

TOTAL BREACH DISCHARGE IS \( 18169 + 25 = 18194 \text{ CFS} \)

(B) DISCUSSION OF RESULT OF II.1) BREACH AT NORMAL POOL

UPON DOWNSTREAM REACHES, INCLUDES HISTORICAL FLOODING

1. REACH 1 - NO IMPACT ON INHABITED STRUCTURES; THE RT. 110 CROSSINGS OF JERICHO BROOK COULD BE COMPLETELY INUNDATED WITH A DEPTH OF FLOODING OF APPROXIMATELY 7.0 FEET (SEE CALCULATIONS ON PAGES 10-15 OF 18)

2. REACH 2 -

THE FIRST STRUCTURE AFFECTED BY THE DAM'S BREAK IS LIKELY TO BE THE CONVERSE BUILDING (CLOSED) ON THE NORTH SIDE OF ROUTE 116 (BETWEEN ROUTE 116 AND THE RR TRACKS) 0.4 MILES BELOW OR SOUtheast OF ROUTE 116'S CROSSING OF JERICHO BROOK.

ASSUMING:

(i) NO REACH STORAGE ABOVE CONVERSE BUILDING OR IT'S STEEP SLOPE OF STEEPS

(ii) MINIMAL INTELLIGENT FLOW, 25 CFS

THE BREAK DISCHARGE CORRESPONDS TO A DEPTH OF FLOODING OF APPROXIMATELY 13.2 FEET ON THE STEEPEST DISCHARGE RATING CURVE DEVELOPED FOR REPRESENTATIVE CROSS-SECTION AT CONVERSE BUILDING (SEE PAGES 6-9 OF 18 FOR CALCULATIONS). ONCE FLOW OVERTOPS THE MUIRROAD TRACK EMBANKMENT, THE CONVERSE BUILDING COULD BE FLOODED; THE EMBANKMENT "D..."
IS APPROXIMATELY 100' ABOVE THE CHANNEL INVERT AND THE CONVERSE BUILDING'S ILL IS
2 FEET BELOW THE TOP OF THE EMBANKMENT. THEREFORE, A 3-5 FOOT DEPTH OF FLOODING
COULD BE EXPECTED AT THE CONVERSE BUILDING WITH THE DAM BEING BREACHED
(HIGH NORMAL POOL ELEVATION), CAUSING
CONSIDERABLE DAMAGE.

Flooding History - Downstream in Reach 2 is
the urbanized area of Berlin along the
Dead River Watercourse, which is the greater
downstream hazard area. This area has
experienced excessive flood damages on
at least three occasions in this century.
Major flooding events in this area in
1927, 1936, and 1953 were responsible for
the construction of the multiple-purpose
(flood control and recreation) structures,
including the dam and the dike, at Site No. 1,
Dead River Watershed. The considerable
flooding problems experienced on the
Dead River in the city of Berlin are
discussed extensively in the work plan for
watershed protection, flood prevention, and
recreation, Dead River Watershed (June 1965,
USDA Soil Conservation Service, Coos County
Soil Conservation District).

Several excerpts from this publication
include:

"Past records show that flooding from the
Dead River has constantly plagued the citizenry
of Berlin. On a number of occasions, floods have
disrupted transportation and caused serious
damage to residences and business and commercial
establishments. Local citizens in Berlin feel
fortunate there has never been loss of life in
the treacherous Dead River channel during periods
of flooding."

"The principal floodwater damages are to
INDUSTRIAL, COMMERCIAL, AND RESIDENTIAL PROPERTY AND ROADS AND BRIDGES IN BERLIN. IN THE FLOODPLAIN BETWEEN HILLSDIE AVE. AND MAIN ST. THERE ARE APPROXIMATELY 26 RESIDENTIAL AND 44 COMMERCIAL BUILDINGS SUBJECT TO FLOOD DANGER FROM LARGE FLOODS. PAGE 7

SUMMARY OF BREECH ANALYSIS ON REACH 2—THE 1936 AND 1953 FLOODS WITH DISCHARGES ESTIMATED TO HAVE BEEN 2000 CFS AND 1000 CFS AND RECURRENCE INTERVALS OF 100 AND 33 YEARS, RESPECTIVELY, CAUSED CONSIDERABLE DAMAGE TO BERLIN. BREECH OF THE DAM, WITH TERRA HOLE AT NORMAL OR RECREATION POOL, COULD RESULT IN A DISCHARGE OF 18200 CFS, WHICH WOULD OVERTOP THE MAIN ST BRIDGE BY APPROXIMATELY 4 FEET (SEE CALCULATIONS ON PAGES 15-18 OF 18) CAUSING AN EXCESSIVE AMOUNT OF DAMAGE TO MAIN ST. STORES AND POSSIBLY CONSIDERABLE LOSS OF LIFE (10 OR MORE). THE DAM IS THEREFORE CONSIDERED TO BE OF HIGH HAZARD POTENTIAL DUE TO THIS LIKELIHOOD OF SEVERE FLOODING IN THE CITY OF BERLIN.

BREAK OF DAM WITH RESERVOIR AT CREST OF EMERGENCY SPILLWAY (136.0 FT. MSL)

THOUGH ANALYSIS NOT REQUIRED TO DETERMINE HAZARD BASED ON NORMAL POOL BREACH COMPUTATIONS, A BREACH OF DAM AT EMERGENCY SPILLWAY CREST WOULD RESULT IN DISCHARGE OF 31415 CFS AND Q = 8/17 - 100 (5.67) (1362 - 1329.3)^3/2

31915 CFS

CONSIDERABLY GREATER DAMAGES IN THE CITY OF BERLIN.

NOTE: RECONSTRUCTION OF MAIN ST. BRIDGE TO PASS 100-YEAR EVENT (2000 CFS) WOULD NOT ELIMINATE SEVERE FLOODING WITH DISCHARGE OF 18200 CFS.
JOB NO. 3573-1

$Q_2 = \text{DISCHARGE OVER PRINCIPAL SPILLWAY (TWO STAGE RISER)} = 280 \text{ cfs (FROM SPILLWAY RATING CURVE)}$

$Q_{\text{TOTAL}} = Q_1 + Q_2 = 31,415 + 250 = 31,665 \text{ cfs}$

(2) REACH 2:
FROM 2HT 9 OF 18 - FLOOD DEPTH = 17.0'
ABOVE CHANNEL INVERT AT CONVERSE BLDG.
\[ \therefore \text{7' OF FLOODING WOULD BE EXPECTED AT THE BLDG ITSELF} \]

@ MAIN ST - 3-4 FT OF FLOODING COULD BE EXPECTED (SEE 2HT 16A, 18 OF 18)

(1) REACH 1: 8.5 - 9.0' FLOOD DEPTH OVER BRIDGE (SEE 2HT 12 AND 13 OF 18)

NOTE: SEE DIATE TITLED, "FLOOD ROUTING - EMERGENCY SPILLWAY HYDROGRAPH" APPENDIX "B" FOR DAM RATING CURVE.
BREACH CALCULATIONS FOR:
A. IMPACT AT CONVERSE BLDG. ON RT. 110

DEAD RIVER CROSS SECTIONAL GEOMETRY (AT CONVERSE BLDG.)
Using cross sectional geometry and channel slope in Manning's equation, stage-discharge rating curve can be developed for the cross section.

\[ Q = \frac{1.486 A R^{2/3}}{n} \]

Where:
- \( Q \): Discharge, (cfs)
- \( A \): Cross-sectional area, (ft\(^2\))
- \( R \): Hydraulic radius, (ft)
- \( S \): Slope, (ft/ft)
- \( n \): Friction factor

5 (slope) is estimated to be 0.002 from USGS quad sheet.

\( n \) (Friction factor) is 0.04, also overbanks are essentially gravel with little vegetation, therefore, use 0.04 for composite \( n \) value for channel and overbanks.

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* See discharge calculations on page 8 of 15.
## INVENTORY OF DAMS IN THE UNITED STATES

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### REMARKS

20-1970

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### INSPECTION

- **INSPECTION BY**: ANDESON-NICHOLS AND COMPANY, INC
- **INSPECTION DATE**: 07JUN 9
- **AUTHORITY FOR INSPECTION**: PL92-367

### REMARKS

32-INO-STAGE RIVER TOTALING 20 FEET, EMERGENCY SPILLWAY 100 FEET WIDE
APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS
FOR THE CASE OF BREACH AT EMERGENCY SPILLWAY ELEVATION A RESULTING FLOOD DEPTH BETWEEN 3 TO 4 FEET WOULD BE EXPECTED AT MAIN ST.

(SEE SHEET 18 OF 18)
Determination of Flood Depths at Main St Bridge, Berlin

Bridge Specifications: Concrete Box, 10' H x 13' W, Weir Coeff. = 2.6

1. Q: At top of road, El. 993 MSL (Invert 976.0 MSL)
   a. Orifice Flow
      Headwater Depth in terms of depth (HW/D) = 17/10 = 1.7
      From BPR Chart: Q = 2210 CFS (170 CFS/ft x 13 ft)

2. Q: Flood depth of 5' over road, El. 998 MSL
   a. Orifice Flow
      Headwater Depth in terms of depth (HW/D) = 22/10 = 2.2
      From BPR Chart: Q = 2600 CFS (200 CFS/ft x 13 ft)
   b. Weir Flow
      Q = CLH²/2 = 2.6 (5.017 + 5.01)(5)²/2 = 23084 CFS
      Total Flow = Orifice and Weir Flow = 2600 + 23084 = 25684 CFS

3. Q: Flood depth of 2.5' over road, El. 995.5 MSL
   a. Orifice Flow
      Headwater Depth in terms of depth (HW/D) = 19.5/10 = 1.95
      From BPR Chart: Q = 2470 CFS (190 CFS/ft x 13 ft)
   b. Weir Flow
      Q = CLH²/2 = 2.6 (2.5/0.17 + 2.5/0.1)(2.5)²/2 = 4081 CFS
      Total Flow = 2470 + 4081 = 6551 CFS

Summary:

From rating curve, constructed for Main St Bridge, a flood depth of approximately 4 feet over the road (stage of 21.7' 99.7 MSL) could result if the dam were breached. This would result in first floor flooding, by 2-3 feet of numerous stores on Main St, as is indicated by elevations (1st Floor) shown on sewer drawings, causing excessive damage.

D-19
CHART FOR DETERMINING ORIFICE FLOW FOR:
(1) ROUTE 110 BRIDGE
(4) MAIN ST. BRIDGE

### CHART I

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<tbody>
<tr>
<td>$5' \times 9' \text{ Box} \quad Q/8 = 15 \text{ cfs/ft}$</td>
<td>6</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>Inlet $W$ {\text{ in.}}</td>
<td>7.5</td>
<td>9</td>
<td>10</td>
</tr>
<tr>
<td>$O$ foot</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
<tr>
<td>(1)</td>
<td>7.5</td>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>(2)</td>
<td>9.0</td>
<td>3.8</td>
<td></td>
</tr>
<tr>
<td>(3)</td>
<td>205.4</td>
<td>4.1</td>
<td></td>
</tr>
</tbody>
</table>

**RATIO OF DISCHARGE TO WIDTH ($Q/W$) IN CFS PER FOOT**

**HEADWATER DEPTH IN TERMS OF HEIGHT OF BOX IN FEET**

**HEIGHT OF BOX IN FEET**

**ANGLE OF WINGWALL FLARE**

**WINGWALL FLARE**

- **(1)** 30° to 75°
- **(2)** 30° and 45°
- **(3)** 0° (recessed to depth of flare)

To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through $D$ and $O$ scales, or reverse as illustrated.

**HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL**

**HYDRAULIC CHART FOR BOX CULVERTS Route 110 MAIN STREET**

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**D-18**
5'-DIAMETER CMP ON
DIRT RD. (UNNAMED)
WITH PROJECTING ENTRANCE

EXAMPLE
D-36 Inch (2:0 Tilt)
D-66 gfs
- (1) 1.0 9.4
- (2) 2.1 4.3
- (3) 2.2 6.6
*0 in feet

OVERTOPPING
DISCHARGE

HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH DIRT CONTROL

CHART 4

BUREAU OF PUBLIC ROADS
STAGE-DISCHARGE RELATIONSHIP (SEE PAGE 13) INDICATES THAT THE BREACH DISCHARGE (18,200 CFS) WOULD RESULT ROUGHLY A 7.0-FOOT DEPTH OF FLOODING OVER THE ROUTE 110 ROADWAY AT THE BRIDGE OVER JERICHO BROOK.

BREACH DISCHARGE (31,700 CFS) WOULD RESULT IN A 8.5 TO 9.0 FOOT DEPTH OF FLOODING OVER RT 110 ROADWAY AT THE BRIDGE OVER JERICHO BROOK.
RT 110
Q = CLH^{3/4} WHERE:
Q = WEIR DISCHARGE (CFs)
L = WEIR SECTION LENGTH (FT)
H = AVERAGE DEPTH OF FLOODING, IS HALF THE FLOOD DEPTH AT BRIDGE &

TRIAL #1: FLOOD DEPTH \( E = 2' \)
WEIR FLOW (LEFT ROADWAY) =
\[ 2.6 \times (2.09)^{3/2} = 130 \text{ CFs} \]
WEIR FLOW (RIGHT ROADWAY) =
\[ 2.6 \times (2.01)^{3/2} = 520 \text{ CFs} \]
ORIFICE FLOW (HWD = 9 + 6/9 = 7.7) IS 150 CFs/FT x 22 FT = 3210 CFs
TOTAL FLOW = 2' FLOOD DEPTH IS COMBINED
PRESSURE AND WEIR FLOW = 3080 + 130 + 520 = 3730 CFs

TRIAL #2: FLOOD DEPTH \( E = 4' \)
WEIR FLOW (LEFT) =
\[ 2.6 \times (4.09)^{3/2} = 735 \text{ CFs} \]
WEIR FLOW (RIGHT) =
\[ 2.6 \times (4.01)^{3/2} = 2942 \text{ CFs} \]
ORIFICE FLOW (HWD = 4 + 8/9 = 5.9) IS 155 CFs/FT x 22 FT = 3410 CFs
TOTAL FLOW IS 3410 + 735 + 2942 = 7087 CFs

TRIAL #3: FLOOD DEPTH \( E = 6' \)
WEIR FLOW (LEFT) =
\[ 2.6 \times (6.09)^{3/2} = 2026 \text{ CFs} \]
WEIR FLOW (RIGHT) =
\[ 2.6 \times (6.01)^{3/2} = 8106 \text{ CFs} \]
ORIFICE FLOW (HWD = 6 + 10/9 = 7.1) IS 165 CFs/FT x 22 FT = 3630 CFs
TOTAL FLOW IS 3630 + 2026 + 8106 = 13762 CFs

TRIAL #4: FLOOD DEPTH \( E = 8' \)
WEIR FLOW (LEFT) =
\[ 2.6 \times (8.09)^{3/2} = 4160 \text{ CFs} \]
WEIR FLOW (RIGHT) =
\[ 2.6 \times (8.01)^{3/2} = 16640 \text{ CFs} \]
ORIFICE FLOW (HWD = 8 + 12/9 = 9.2) IS 180 CFs/FT x 22 FT = 3960
TOTAL FLOW IS 3960 + 4160 + 16640 = 24760 CFs

D-14
ANALYSIS OF DOWNSTREAM BRIDGES AND CULVERTS

A 5-FOOT DIAMETER CORRUGATED METAL PIPE ON DIRT RD. 5MI DOWNSTREAM OF DAM.

Top of road (centerline of culvert) is 2 feet above low chord. Therefore, headwater, in terms of diameter (HW/D), is 1.9 before structure is overtopped.

Capacity of culvert, before overtopping, is 190 CFS (see page 14 of 18). Breach discharge would probably wash out the low earthen embankment.

B 9' x 22' CONCRETE BOX CULVERT ON ROUTE 110.

Top of road (centerline of bridge) is approximately 4 feet above low chord. Therefore, headwater in terms of depth before roadway is overtopped is 1.4 = (9 + 4)/9.

Capacity of structure before roadway is overtopped is roughly 2690 CFS (22' wide times 120 CFS per foot of width). See page 15.

To determine impact of breach, must develop stage-discharge curve for structure to get depth of flooding. Requires estimation of weir shape of the roadway; must estimate effective flow limit on right (southeast) roadway of RT. 110 as it slopes continuously downhill to city of Berlin. An estimate of 0.01 slope for the right roadway will suffice for estimating weir flow; left roadway has a 0.09 slope.

Calculate weir flow; assume WEIR C = 2.6, and try different flood depths to get rating curve.

Use standard weir equation:

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

D-13
RATING CURVE (DEAD RIVER) 9 CF 18 RHINOCERS
CROSS-SECTION AT CONVERSE BUILDING

Discharge at Emergency Spillway: 31,400 CFS
Stage 111.5 (Above Invert)

Discharge
Normal Pool
19,100 CFS
Approximate Stage 13.5
(Above Invert)

Approximate Converse Building Elevation
Stage 10
(Above Invert)

Rating Curve at Converse Building

D-12
<table>
<thead>
<tr>
<th>Stage (ft)</th>
<th>Formula</th>
<th>Calculation</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>( Q = \frac{1.986 \times (15)(1)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 25 \text{ cfs} )</td>
</tr>
<tr>
<td>4</td>
<td>( Q = \frac{1.986 \times (245)(1.19)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 444 \text{ cfs} )</td>
</tr>
<tr>
<td>6</td>
<td>( Q = \frac{1.986 \times (875)(1.407)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 1520 \text{ cfs} )</td>
</tr>
<tr>
<td>8</td>
<td>( Q = \frac{1.986 \times (1705)(2.09)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 4631 \text{ cfs} )</td>
</tr>
<tr>
<td>10</td>
<td>( Q = \frac{1.986 \times (2535)(3.11)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 8977 \text{ cfs} )</td>
</tr>
<tr>
<td>12</td>
<td>( Q = \frac{1.986 \times (3365)(4.12)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 14367 \text{ cfs} )</td>
</tr>
<tr>
<td>14</td>
<td>( Q = \frac{1.986 \times (4195)(5.12)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 20705 \text{ cfs} )</td>
</tr>
<tr>
<td>16</td>
<td>( Q = \frac{1.986 \times (5025)(6.12)^{2/3}(0.002)^{1/2}}{0.04} )</td>
<td>( Q = 27934 \text{ cfs} )</td>
</tr>
</tbody>
</table>
END

FILMED

8-85

DTIC