ACKENHEIL AND ASSOCIATES INC BALTIMORE MD
NATIONAL DAM INSPECTION PROGRAM COLONIAL DAM NUMBER 3, (NDI NU=EETC(U))
MAY 80  J P HANNAN
DACW31-80-C-0026

UNCLASSIFIED
OHIO RIVER BASIN
WASHWATER RUN
FAYETTE COUNTY

PENNSYLVANIA
NDI No. PA 00209
PENN DER No. 26-22

COLONIAL DAM No. 3
REDSTONE WATER COMPANY

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

ACKENHEIL & ASSOCIATES
DACW31-80-C-0026

REPARED FOR
DEPARTMENT OF THE ARMY
BALTIMORE DISTRICT, CORPS OF ENGINEERS
BALTIMORE, MARYLAND 21203

BY

ACKENHEIL & ASSOCIATES GEO SYSTEMS, INC.
CONSULTING ENGINEERS
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MAY 1980

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OHIO RIVER BASIN

COLONIAL DAM NO. 3
FAYETTE COUNTY, COMMONWEALTH OF PENNSYLVANIA
NDI NO. PA 00209
PennDER No. 26-22

REDSTONE WATER COMPANY

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

Prepared for: DEPARTMENT OF THE ARMY
Baltimore District, Corps of Engineers
Baltimore, Maryland 21203

Prepared by: ACKENHEIL & ASSOCIATES GEO SYSTEMS, INC.
Consulting Engineers
1000 Banksville Road
Pittsburgh, Pennsylvania 15216

Date: May 1980

This document has been approved for public release and sale; its distribution is unlimited.
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 Department of the Army, 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 visual observations and review of available data. Detailed investigations and analyses involving topographic mapping, subsurface investigations, materials testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify the need for such studies which should be performed by the owner.

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 the dam depends on numerous and constantly changing internal and external factors which are 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 time in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I investigations are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the spillway design flood is based on the estimated "Probable Maximum Flood" (PMF) for the region (greatest reasonably possible storm runoff), or fractions thereof. The spillway design flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition, and the downstream damage potential.
SYNOPSIS OF ASSESSMENT AND RECOMMENDATIONS

NAME OF DAM: Colonial No. 3
STATE LOCATION: Pennsylvania
COUNTY LOCATION: Fayette
STREAM: Washwater Run, a tributary of Redstone Creek
DATE OF INSPECTION: 31 October 1979
COORDINATES: Lat. 40° 00' 09"
               Long. 79° 45' 37"

ASSESSMENT

Based on a review of available design information, visual observations of conditions as they existed on the date of the field inspections, and hydrologic and stability calculations, the general condition of the Colonial Dam No. 3 is considered to be poor.

This assessment is based on:

1. Observation that the gravity dam structure has been partially breached at the right abutment.

2. Observations that physical deterioration of the dam and outlet works is significant.

3. Hydrologic/hydraulic calculations that indicate that the overflow crest has an "inadequate" discharge capacity.

The structure is classified as a "small" size, "significant" hazard dam. Corps of Engineers guidelines recommend a one hundred year flood to 1/2 the Probable Maximum Flood (PMF) for a "small" size, "significant" hazard dam. Colonial Dam No. 3 has a Spillway Design Flood of 1/2 the Probable Maximum Flood (PMF). Spillway capacity is "inadequate" because the non-overtopping flood discharge capacity, as estimated using the HEC-1 computer program, was found to be 0.09 PMF for an unbreached condition. The existing, breached condition, was not analyzed but is assessed to be hydrologically better than the unbreached condition.
SYNOPSIS OF ASSESSMENT AND RECOMMENDATIONS (CONT'D)
Colonial Dam No. 3

RECOMMENDATIONS

1. Additional Investigations: Immediately retain a professional engineer knowledgeable in dam design and construction to:
   
   a. Perform a detailed hydrologic/hydraulic analysis of the reservoir and dam and make recommendations on increasing the capacity of the system to make it adequate.
   
   b. Investigate the operability of the outlet works and provide recommendations on repair requirements.
   
   c. Provide recommendations on improving the physical condition of the deteriorated gravity dam and right abutment.

2. Emergency Operation and Warning Plan: Concurrent with the additional investigations recommended above, the owner should develop an Emergency Operation and Warning Plan including:
   
   a. Guidelines for evaluating inflow during periods of heavy precipitation or runoff.
   
   b. Procedures for around the clock surveillance during periods of heavy precipitation or runoff.
   
   c. Procedures for drawdown of the reservoir under emergency conditions.
   
   d. Procedures for notifying downstream residents and public officials, in case evacuation of downstream areas is necessary.

3. Inspection and Maintenance: The owner should immediately develop and implement formal inspection and maintenance procedures.
4. Orderly Breaching: In lieu of performing the above recommendations, the owner should engage the services of a professional engineer, knowledgeable in dam design and performance, to prepare specifications for completely breaching the structure, to make it incapable of impounding water. The structure should then be breached under the direction of the professional engineer, in accordance with applicable state and local regulations.
COLONIAL DAM No. 3

OVERVIEWS
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>PREFACE</strong></td>
<td>i</td>
</tr>
<tr>
<td></td>
<td><strong>SYNOPSIS OF ASSESSMENT AND RECOMMENDATIONS</strong></td>
<td>ii</td>
</tr>
<tr>
<td></td>
<td><strong>OVERVIEW PHOTOGRAPH</strong></td>
<td>v</td>
</tr>
<tr>
<td>SECTION 1</td>
<td><strong>PROJECT INFORMATION</strong></td>
<td></td>
</tr>
<tr>
<td>1.1</td>
<td>General</td>
<td>1</td>
</tr>
<tr>
<td>1.2</td>
<td>Description of Project</td>
<td>1</td>
</tr>
<tr>
<td>1.3</td>
<td>Pertinent Data</td>
<td>3</td>
</tr>
<tr>
<td>SECTION 2</td>
<td><strong>ENGINEERING DATA</strong></td>
<td></td>
</tr>
<tr>
<td>2.1</td>
<td>Design</td>
<td>5</td>
</tr>
<tr>
<td>2.2</td>
<td>Construction</td>
<td>6</td>
</tr>
<tr>
<td>2.3</td>
<td>Modification/Repair</td>
<td>6</td>
</tr>
<tr>
<td>2.4</td>
<td>Operation</td>
<td>7</td>
</tr>
<tr>
<td>2.5</td>
<td>Evaluation</td>
<td>7</td>
</tr>
<tr>
<td>SECTION 3</td>
<td><strong>VISUAL INSPECTION</strong></td>
<td></td>
</tr>
<tr>
<td>3.1</td>
<td>Findings</td>
<td>8</td>
</tr>
<tr>
<td>3.2</td>
<td>Evaluation</td>
<td>12</td>
</tr>
<tr>
<td>SECTION 4</td>
<td><strong>OPERATIONAL FEATURES</strong></td>
<td></td>
</tr>
<tr>
<td>4.1</td>
<td>Procedure</td>
<td>13</td>
</tr>
<tr>
<td>4.2</td>
<td>Maintenance of Dam and Operating Facilities</td>
<td>13</td>
</tr>
<tr>
<td>4.3</td>
<td>Inspection of Dam</td>
<td>13</td>
</tr>
<tr>
<td>4.4</td>
<td>Warning System</td>
<td>13</td>
</tr>
<tr>
<td>4.5</td>
<td>Evaluation</td>
<td>13</td>
</tr>
<tr>
<td>SECTION 5</td>
<td><strong>HYDROLOGY AND HYDRAULICS</strong></td>
<td></td>
</tr>
<tr>
<td>5.1</td>
<td>Evaluation of Features</td>
<td>14</td>
</tr>
<tr>
<td>SECTION 6</td>
<td><strong>STRUCTURAL STABILITY</strong></td>
<td></td>
</tr>
<tr>
<td>6.1</td>
<td>Available Information</td>
<td>16</td>
</tr>
<tr>
<td>6.2</td>
<td>Stability Analysis</td>
<td>17</td>
</tr>
<tr>
<td>6.3</td>
<td>Evaluation of Structural Stability</td>
<td>18</td>
</tr>
</tbody>
</table>
# TABLE OF CONTENTS (cont'd)

## SECTION 7 - ASSESSMENT AND RECOMMENDATIONS

| 7.1 Assessment | ........ | 20 |
| 7.2 Recommendations | ........ | 21 |

## APPENDIX A - VISUAL INSPECTION CHECKLIST
- Visual Observations Checklist II | A1
- Field Plan | A10
- Field Profile and Section | A11

## APPENDIX B - ENGINEERING DATA CHECKLIST

## APPENDIX C - PHOTOGRAPHS
- Photo Key Map | C1
- Photos 1 through 10 | C2
- Detailed Photo Descriptions | C7

## APPENDIX D - HYDROLOGY AND HYDRAULICS ANALYSES
- Methodology | D1
- Engineering Data | D3
- HEC-1 Data Base | D4
- Loss Rate and Base Flow Parameters | D5
- Elevation-Area-Capacity Relationship | D5
- Overtop Parameters | D6
- Program Schedule | D6
- Stage-Discharge Relationship | D7
- HEC-1 Computer Analysis | D9
- Reservoir/Spillway Hydrologic Performance Plot | D12

## APPENDIX E - PLATES
- List of Plates | E1
- Plates I through III | E2

## APPENDIX F - GEOLOGY
- Geomorphology | F1
- Structure | F1
- Stratigraphy | F1
- Geologic Map | F3
- Geologic Column | F4

## APPENDIX G - STABILITY ANALYSES
- List of Analyses | G1
- Department of Forests and Waters | G2
- Sliding Stability - Existing Conditions | G6
- Overturning Stability - Existing Conditions | G19
PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM
COLONIAL DAM NO. 3
NATIONAL I.D. NO. PA 00209
PennDER No. 26-22

SECTION 1
PROJECT INFORMATION

1.1 GENERAL

a. Authority: The Phase I investigation was performed pursuant to authority granted by Public Law 92-367 (National Dam Inspection Act) to the Secretary of the Army through the Corps of Engineers, to conduct inspections of dams throughout the United States.

b. Purpose: The purpose of the investigation is to make a determination on whether or not the dam constitutes a hazard to human life or property.

1.2 DESCRIPTION OF PROJECT

a. Dam and Appurtenances:

(1) Concrete Gravity Dam: Colonial Dam No. 3 is a reinforced concrete, gravity type dam, 28.5 feet high and 161 feet long. The upstream face is vertical and the downstream face is 1H:2V. The dam crest is 5 feet wide at Elev. 924 and contains a 2.5 foot high by 2 foot wide concrete parapet which is discontinuous across the crest. The missing sections of parapet are 39.7 feet and 13.4 feet in length and allow normal and storm flows to discharge over the dam crest. Discharge is to the bedrock streambed below.

(2) Outlet Works: The outlet works consists of an 18 inch diameter cast iron pipe that is controlled on the upstream face by a 2.5 foot square, inoperative, sluice gate. The pipe branches downstream of the dam into a 12 inch pond drain and an 8 inch water supply pipeline which is now disconnected.

(3) Downstream Conditions: Washwater Run flows through a narrow, uninhabited, and steep-sided valley. Approximately 2,000 feet below the dam, the Run passes through a large culvert beneath a railroad embankment and discharges to Redstone Creek.
b. **Location:** Colonial Dam No. 3 is located in Jefferson Township, Fayette County, Pennsylvania, approximately one mile east of Grindstone and 0.8 miles north of Rowes Run. The dam is situated across Washwater Run, which is tributary to Redstone Creek, which is tributary to the Monongahela River at Brownsville, Pennsylvania.

c. **Size Classification:** The dam has a maximum storage capacity of 23 acre-feet and a toe to crest height of 28.5 feet. Based on Corps of Engineers guidelines, this dam is classified as a "small" size structure.

d. **Hazard Classification:** Colonial Dam No. 3 is a "significant" hazard dam. In the event of catastrophic failure of the dam, it is unlikely that the railroad embankment and the few dwellings along Redstone Creek below would suffer great damage or loss of life would result.

e. **Ownership:** Colonial Dam No. 3 is owned by the Redstone Water Company. Correspondence can be addressed to:

Redstone Water Company, Inc.
Box 548
California, Pennsylvania 15419
Attention: Mr. Edward Yablonski
412-938-9164

f. **Purpose of Dam:** Colonial Dam No. 3 was originally constructed to provide water for industrial use by the Pittsburgh Coal Company. It was subsequently sold to the Redstone Water Company to supply water for residential and domestic use. The reservoir is not now used for water supply purposes and its current use is unknown.

g. **Design and Construction History:** The dam was designed in 1907 by E. J. Taylor, Chief Engineer and Construction Supervisor for the Pittsburgh Coal Company. It was constructed by Maynard and Flynn, Contractors, of Pittsburgh, Pennsylvania in 1907.

h. **Normal Operating Procedure:** Colonial Dam No. 3 was designed to operate as an uncontrolled structure. Under normal operating conditions, the pool level is maintained at Elev. 924.1 by the crest of the concrete gravity dam.
1.3 PERTINENT DATA

a. Drainage Area: 1.5 sq. miles

b. Discharge at Dam Facility:

- Maximum Known Flood at Dam Facility (flood of 4 June 1941): 600+ cfs*
- Overflow Crest: 280 cfs

(c. Elevation (Feet Above MSL)*:

- Constructed Top of Dam (Parapet): 926.5*
- Design Highwater: 926.5*
- Normal Pool (Unbreached): 924.1
- Overflow Crest: 924.1*
- Upstream Invert of Outlet Pipe: 899.2*
- Maximum Tailwater: Unknown
- Downstream Toe: 898.0

d. Reservoir Length:

- Length of Maximum Pool: 500 feet
- Length of Normal Pool: 450 feet

e. Total Storage:

- Constructed Top of Dam: 26 acre-feet
- Design Highwater: 26 acre-feet*
- Overflow Crest: 23 acre-feet*
- Normal Pool Level: 23 acre-feet*

f. Reservoir Surface:

- Constructed Top of Dam: 2 acres
- Design Highwater: 2 acres*
- Overflow Crest: 1.5 acres*
- Normal Pool: 1.5 acres*

g. Dam:

- Type: Reinforced Concrete, Gravity
- Length: 161 feet*
- Height: 28.5 feet
- Top Width:
  - At Crest: 5 feet
  - At Top of Parapet: 2 feet
- Slopes:
  - Upstream: Vertical
  - Downstream: 1H:2V
- Cutoff Provisions: Grouted Foundation*
h. **Outlet Works:**

Type
18 inch diameter cast iron pipe

Upstream Control
2.5 foot square sluice gate

Outlet
Branch to 12 inch diameter pond drain and 8 inch diameter water supply pipe

Gate Valves
Both branches, downstream
(neither observed)

i. **Overflow Crest:**

Discharge Sections are broad crested weirs:

(1) 39.7 feet long at Elev. 924.1
(2) 13.5 feet long at Elev. 924.2
(3) 5.5 feet long at Elev. 925.5

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*Taken or derived from original design drawings or design information in PennDER files.*
2.1 DESIGN

a. Data Available: The following written information and data may be obtained from the Pennsylvania Department of Environmental Resources, Harrisburg, Pennsylvania. The information was reviewed for this study.

(1) A design drawing, dated 8 August 1907, showing elevations and sections of the proposed dam and additional notes related to construction changes and subsequent dam modifications.

(2) Two "Reports Upon the H.C. Frick Coke Company's Dam" dated 25 June 1914 (draft) and 20 October 1914.

b. Design Features:

(1) Foundation: The dam was founded on sandstone bedrock, except near the crest, where shale was encountered. Bedrock was removed to an "average" depth of 2.5 feet and was reported to be quite dry and massive. Before construction of the dam, three irregularly spaced 3 inch diameter holes were drilled to a depth of 18 inches, tall pipes were inserted and the foundation was grouted with liquid grout under pipe head. The pipes were later encased in the concrete of the dam.

(2) Gravity Dam: The gravity dam was designed by E. J. Taylor of the Pittsburgh Coal Company in 1907. The dam had a two foot thick base slab 23 feet long and 39 feet wide (cross valley). The remainder of the dam, without base slab, was step-keyed into the abutment rock. The dam design called for a 5 foot wide crest, vertical upstream face, downstream face sloping at 1H:2V and a concrete parapet 18 inches high by two feet wide on the crest. The parapet had a 25 foot wide opening to allow normal and storm flow discharges. The concrete was to be reinforced with three inch square, 8 gage, wire mesh. The wire mesh sheets were to be placed adjacent to the dam surfaces and lapped one foot.
(3) Outlet Works: An 18 inch diameter cast iron outlet pipe was designed and was to be controlled on the upstream face by a 30 inch square sliding gate. Downstream, the pipe branched into eight inch and twelve inch lines, each with gate valve. The 12 inch line was a pond drain and the eight inch line was to supply water to the Colonial No. 3 mine.

2.2 CONSTRUCTION

a. Contractor: The dam was constructed for the Pittsburgh Coal Company by Maynard and Flynn, Contractors, of Pittsburgh, Pennsylvania in 1907. The work was performed under the direction of Ernest G. Taylor.

b. Field Changes: According to correspondence cited in Section 2.1a(2), that portion of the dam below the top of rock was made 3 feet wider than originally planned by stepping out 1 foot at the heel and 2 feet at the toe.

The dam was constructed to Elev. 925.5 instead Elev. 930 as originally intended.

2.3 MODIFICATION/REPAIR

a. 1908: One foot of concrete was added to the top of the parapet, bringing the dam top elevation to 926.5.

b. 1912: Following the storm of July 1912, when the dam was overtopped and the sides were washed out to bedrock, several corrective measures were undertaken.

(1) The reservoir was drained and all cracks on the upstream face were raked out and carefully pointed with a cement mortar and the entire upstream face was plastered with mortar.

(2) Extra masonry was placed on the dam's downstream face bringing the outline to the dimensions originally contemplated.

(3) Concrete blocks were placed along the downstream abutments to break the force of any water that might overflow the parapet.

(4) A portion of the dam was cut away and rebuilt about one foot deeper into the hillside.
(5) The length of the original overflow crest was increased from 25 feet to 39.7 feet by cutting out a portion of the concrete parapet.

(6) An additional spillway of 13.4 feet was cut out of the concrete parapet on the right side of the crest.

c. 1919: During the summer, holes were drilled into the dam and cement grout was forced into the concrete under pressure, stopping all leakage at the horizontal construction joints.

2.4 OPERATION

According to the Pennsylvania Department of Environmental Resources, the Redstone Water Company is responsible for operation of Colonial Dam No. 3.

The overflow crests are uncontrolled and performance and operation records are not maintained. The outlet works is not currently in use.

2.5 EVALUATION

a. Availability: Available design information and drawings were obtained from the Pennsylvania Department of Environmental Resources.

b. Adequacy: The available design information supplemented by field inspections and supporting engineering analyses in succeeding sections, is adequate for the purpose of this Phase I inspection report.

c. Validity: Based on the available data, there appears to be no reason to question the validity of the available design information and drawings.
SECTION 3
VISUAL INSPECTION

3.1 FINDINGS

a. **General:** The visual observations of Colonial Dam No. 3 and reservoir were performed on 31 October 1979, and consisted of:

   (1) Visual observations of the concrete gravity dam and abutments.

   (2) Visual observations of exposed sections of the outlet works.

   (3) Visual observations of downstream conditions and evaluation of the downstream hazard potential.

   (4) Visual observations of the reservoir shoreline and inlet stream channel.

   (5) Transit stadia survey of relative elevations along the dam crest centerline.

The visual observations were performed when the reservoir was below normal pool operating level due to an erosional breach at the right abutment. Tailwater conditions were normal.

The field plan, elevation, dam crest profile, and section and the visual observations checklist containing the observations and comments of the field inspection team are contained in Appendix A. Specific observations are illustrated on photographs in Appendix C. Detailed findings of the visual inspection are presented in the following sections.

b. **Concrete Gravity Dam:**

   (1) **Configuration:** The impounding structure is a concrete gravity dam with a vertical upstream face, steeply sloping downstream face, and an irregular crest profile developed to provide storm flow discharge capacity. A field plan, elevation, crest profile and section are included in Appendix A. At the time of the visual inspection, a breach was observed between the dam and the right abutment, that had lowered the reservoir pool approximately two feet below the normal
operating pool level. The inlet to the breach was clogged with debris that appeared to maintain the reservoir pool at a higher level than would have been possible with an unclogged breach. The breach was approximately eight feet wide at the top.

A grout pipe was observed near the downstream edge of the crest, approximately 50 feet right of the left abutment. Two steel Z's were attached, vertically, to the upstream face of the dam, about 70 feet right of the left abutment.

No evidence was found of reported concrete stilling blocks on the downstream abutments or the "extra masonry" splash pad placed on the downstream toe in 1912.

(2) Condition of Concrete: The impounding structure was observed to contain significant cracking of the parapet, significant spalling of concrete surfaces, and significant spalling and deterioration of construction joints. The downstream face of the dam had considerable moss, grass, and small trees growing.

No major structural cracks were observed.

(3) Vertical and Horizontal Alignment: No evidence was observed to indicate adverse vertical or horizontal movement of the dam. The crest was found by survey to be level and no skewing or offsets were observed. No evidence of tilting was observed.

(4) Seepage: Several seeps and considerable wetting were observed on the downstream face of the dam, particularly in the central portion of the structure. In general, the wet conditions were observed beginning at the approximate elevation of the water line on the upstream side and extended to the foundation. The seeping water appeared to be primarily associated with construction joint systems.

Three large seeps with flows estimated at five gallons per minute (gpm) each were observed about six feet below the dam crest near the toe of the dam on the left abutment.

(5) Foundation: The impounding structure is founded on bedrock which appeared to be a competent, fine grained sandstone. The portions of dam/foundation contact that were observed appeared tight and did not appear to be leaking.
(6) Abutments:

Left: The left abutment is a relatively steep, natural hillside which was heavily wooded upstream and downstream of the dam. No springs or seeps were observed in the abutment downstream of the dam, although a dense ground cover of leaves and brush may have obscured such observations.

A bedrock outcrop near the stream bed just below the toe of the dam was wet, but no flowing water was observed.

Right: The right abutment had generally the same conditions as the left abutment with respect to slope and cover. However, severe erosion of soil and rock has occurred in and just downstream of the breach described earlier. Normal inflow to the reservoir now passes into the breach, beneath the undercut right end of the dam, and is directed along the dam toe to the original creek channel below.

(7) Overflow Section: Under normal (unbreached) operating conditions, inflows to the reservoir are discharged over the dam crest, through the rectangular openings in the parapet. No other spillway facility was observed.

c. Outlet Works:

(1) Pond Drain: A 12 inch (nominal) diameter cast iron pipe pond drain outlets to the original Washwater Run channel just below the toe of the dam. The downstream end of the pipe was broken, badly deteriorated and dirt clogged. The concrete encasement around the pipe extending beyond the dam structure was also badly deteriorated and no flow control devices were observed. No discharge was observed.

The inlet end of the pipe was not observed due to the reservoir pool level. However, the two steel sluice gate slides on the upstream face of the dam were badly rusted.

(2) Water Supply Pipe: An eight inch (nominal) diameter cast iron water supply pipe was observed to pass through the dam immediately above the pond drain pipe. The eight inch pipe extended downstream several feet past the pond drain discharge point, but terminated at an open end condition. No flow controls were observed downstream of the dam and no water was discharging from the end of the pipe.
d. **Instrumentation:** No instrumentation was observed during the visual inspection.

e. **Reservoir:**

   (1) **Slopes:** The reservoir slopes were observed to be generally steep and densely wooded. No recent shoreline slope instability was noted.

   (2) **Sedimentation:** The upper end of the reservoir was silted in, but was covered with dense brush.

Water depth measurements along the upstream face of the dam indicated a sediment level four feet below the water surface.

   (3) **Watershed:** Conditions in the watershed did not appear to be significantly different from those indicated on the U.S.G.S. 7-1/2 minute quadrangle. The watershed is mostly farm and woodland and no mining or major construction operations were observed.

f. **Downstream Channel:**

   (1) **Flow Conditions:** The downstream channel in the first 2,000 feet below the dam, lies in a narrow, steep-sided valley, that is heavily wooded. The channel has a very rough bedrock bottom, undergoes several turns, and is generally clogged with down timber and debris.

   Several hundred feet downstream of the dam there is a twelve to fifteen foot high waterfall in Washwater Run. The waterfall appeared to be the result of erosion rather than differential bedrock displacement.

   (2) **Railroad Embankment:** A railroad embankment crosses the mouth of Washwater Run valley at the confluence with Redstone Creek. A concrete and masonry culvert transports the Run beneath the embankment. The culvert's controlling cross section is half an ellipse with a minor radius of five feet horizontally and a radius of seven feet vertically. The embankment is about 26 feet high, has a crest width of more than 30 feet, and contains a single railroad track and a gravel surfaced access road.
(3) Flood Plain Development: The first inhabited dwelling below Colonial Dam No. 3 lies on the floodplain of Redstone Creek at Rowes Run, about 2,500 feet below the dam. At least four inhabited dwellings lie on the flood plain in the first 6,000 feet below the dam. All the dwellings appeared to be more than four feet above the base of the Redstone Creek channel.

3.2 EVALUATION

a. Impounding Structure: The concrete gravity dam is considered to be in poor condition. This is based on the observed breach at the right abutment and the general condition of the structural concrete, particularly spalling at construction joints and deterioration of the parapet and crest. Significant seepage on the downstream face indicates water passages through the dam. Vertical and horizontal alignments appeared satisfactory and no tilting or skewing of the dam was noted. No major structural cracks were observed.

b. Right Abutment: The right abutment is considered to be in poor condition because of significant erosion due to water flowing in the breach. The base of the breach channel is on bedrock, but the sides consisted of barren, steep soil slopes. Continued, detrimental erosion can be anticipated.

c. Outlet Works: The outlet works are considered to be in poor condition. This is based on observed deterioration of pipes and sluice gate slides. Sluice gate operating controls or other valves were not observed.

d. Sedimentation: The reservoir was observed to be significantly silted; only four feet of standing water was observed at the dam.

e. Downstream Conditions: In the judgement of the evaluating engineer, the observed downstream conditions indicate that the hazard classification for Colonial Dam No. 3 is "significant". This is based on field observations of the downstream railroad embankment's height, crest width, and culvert capacity, and Redstone Creek floodplain development conditions. It was felt that even a catastrophic failure of Colonial Dam No. 3 would not cause a catastrophic failure of the railroad embankment. Rather, the railroad embankment culvert would throttle and control the discharge to Redstone Creek.
SECTION 4
OPERATIONAL FEATURES

4.1 PROCEDURE

Reservoir pool level is maintained by the uncontrolled concrete overflow crest of the gravity dam. Normal operating procedure does not require a dam tender.

When used for water supply purposes, the outlet works is normally open and discharge is to the 8 inch diameter cast iron water supply pipe. A gate valve (not observed) controls flow from the 12 inch diameter pond drain.

The water supply system is not now in use and flow controls are closed.

4.2 MAINTENANCE OF DAM AND OPERATING FACILITIES

Colonial Dam No. 3 and appurtenances are not maintained.

4.3 INSPECTION OF DAM

The Redstone Water Company is required by the State of Pennsylvania to inspect the dam annually and make needed repairs.

4.4 WARNING SYSTEM

There are no warning systems or formal emergency procedures to alert or evacuate downstream residents upon threat of a dam failure.

4.5 EVALUATION

There are no written operation, maintenance or inspection procedures, nor is there a warning system or formal emergency procedure for this dam. These procedures should be developed in the form of checklists and step by step instructions, and should be implemented as necessary.
5.1 EVALUATION OF FEATURES

a. Design Data: Colonial Dam No. 3 has a watershed of 960 acres which is vegetated primarily by woodland and farmland. The watershed is about two miles long, one mile wide and has a maximum elevation is 1,240 feet above Mean Sea Level (MSL). At normal pool, the dam impounds a reservoir with a surface area of 1.5 acres and a storage volume of 23 acre-feet. Normal pool elevation was maintained at Elev. 924.1 by the overflow crest, prior to partial breaching.

According to Penn DER files, the overflow crest capacity was made sufficient to accommodate 506 cubic feet per second per square mile which was considered adequate for this structure and watershed. The Colonial Dam No. 3 capacity for the observed cross section and existing freeboard condition was computed to be 280 cfs. No additional hydrologic calculations were found relating reservoir/spillway performance to the Probable Maximum Flood or fractions thereof.

b. Experience Data: Continuous records of reservoir level or rainfall amounts are not kept. Records of the structure being overtopped include a flow depth of 0.5 foot in July 1912 and a depth of 1.1 feet during the storm of 4 June 1941.

c. Visual Observations: On the date of the field reconnaissance, severe deterioration of the concrete overflow sections was observed. An 8 foot wide erosion breach was observed in the right abutment. In order to perform the HEC-1 analysis, it was assumed that this condition was repaired and the structure performed as designed. This assumption was made because the repaired condition is considered to be hydrologically more unsafe than the existing breached condition.

d. Overtopping Potential: Overtopping potential was investigated through the development of the Probable Maximum Flood (PMF) for the watershed and the subsequent routing of the PMF and fractions of the PMF through the reservoir and overflow crest. The Corps of Engineers guidelines recommend the 100 year flood to 1/2 the PMF for "small" size, "significant" hazard dams. Based on the observed downstream conditions, Colonial Dam No. 3 has a Spillway Design Flood (SDF) of one half PMF.
Hydrometeorological Report No. 33 indicates the adjusted 24 hour Probable Maximum Precipitation (PMP) for the subject site is 19.4 inches. No calculations are available to indicate whether the reservoir and overflow crest are sized to pass a flood corresponding to 9.7 inches of rainfall in 24 hours (1/2 PMP). Consequently, an evaluation of the reservoir/overflow crest system was performed to determine whether the overflow crest capacity is adequate under current Corps of Engineers guidelines.

The Corps of Engineers, Baltimore District, has directed that the HEC-1 Dam Safety Version computer program be utilized. The program was prepared by the Hydrologic Engineering Center (HEC), U.S. Army Corps of Engineers, Davis, California. The major methodologies and key input data for this program are discussed briefly in Appendix D.

The peak inflow to Colonial Dam No. 3 for the SDF was determined by HEC-1 to be 1637 cfs.

   e. Adequacy: For the design condition (without breach), the capacity of the combined reservoir and overflow crest system was determined to be 0.09 PMF. An initial pool elevation of 924.1 was assumed prior to commencement of the storm. According to Corp of Engineer's guidelines, Colonial Dam No. 3 overflow crest capacity is "inadequate."

   At 0.50 PMF, the Colonial Dam No. 3 parapet is overtopped by 1.18 feet of water for a duration of 10 hours (unbreached condition).
SECTION 6
STRUCTURAL STABILITY

6.1 AVAILABLE INFORMATION

a. Visual Observations: On the date of the field reconnaissance, four observations were made that are significant to an evaluation of the structural stability of Colonial Dam No. 3. These observations are:

(1) Extensive erosion of the right abutment (breach) that has reduced the effective foundation contact area of the dam.

(2) Extensive leakage on the downstream face of the dam, particularly in the area of construction joints, that indicates the existence of openings through the dam.

(3) A considerable accumulation of silt in the reservoir.

(4) Lack of a concrete "batter" at the toe of the dam that was reported to have been placed in 1912. The observed downstream dam slope was more or less constant from crest to toe at 1H:2V.

b. Design and Construction Data: Available design and construction data pertinent to the structural stability of the dam includes:

(1) The drawing referenced in Section 2.1a(1) above and reproduced as Plate III in Appendix E.

(2) Reports in the correspondence that the dam foundation was cement grouted under low head during construction and that the dam itself was grouted in 1919 to seal leaks.

c. Performance Data: According to information in the PennDER files, Colonial Dam No. 3 has been overtopped without failure, twice in its lifetime - by 0.5 foot in July 1912 and by 1.1 feet on 4 June 1941.
Previous Stability Analysis: A comprehensive stability analysis of the dam was performed by engineers of the Department of Forest and Waters in 1914. The analysis evaluated both overturning and sliding stability and was presented in the reports referenced in Section 2.1a(2) above. Results of the analysis and two pages of supporting calculations, also obtained from PennDER files, are contained in Appendix G.

6.2 STABILITY ANALYSIS

a. General: An analysis of sliding and overturning stability was performed to evaluate the dam for existing conditions.

b. Assumptions:

(1) The right abutment breach length is not affected by uplift pressures and offers no sliding or passive resistance.

(2) The unit weight of wire mesh reinforced concrete is 145 pcf.

(3) Sediment behind the dam has a total unit weight of 80 pcf, an effective angle of internal friction of 3°, and no cohesive strength.

(4) The foundation bedrock has a total unit weight of 140 pcf, an effective angle of internal friction of 45°, and no cohesive strength.

(5) The coefficient of friction between mass concrete and clean bedrock is 0.70 (Ref. NAVFAC-DM7, Table 10-1).

(6) Weight and drag forces caused by the flow of water over the dam are negligible.

(7) Foundation uplift pressure varies linearly from a maximum of 100 percent hydrostatic at the upstream edge of the dam to zero at the downstream edge.

(8) Rankine empirical formulas are applicable for determining passive and active lateral earth pressures for the downstream rock key and reservoir sediment, respectively.
The dam is keyed 2.5 feet into rock.

The breach will be debris clogged during large meteorological events, such that unbreached reservoir pool levels will exist.

c. **Sliding Stability:** Sliding stability of the existing dam configuration was analyzed for static conditions to estimate the amount of cohesion required between the dam and foundation to give a safety factor (SF) against sliding of 3. The dam was analyzed as a three dimensional, rigid body to more accurately characterize the relationship between driving and resisting forces. The pool level associated with the SDF (Elev. 927.7) was employed in the analysis. To achieve a SF of 3, it was determined that a cohesion of 20 psi, acting over the unbreached portion of the foundation, is required. The calculations are contained in Appendix G.

d. **Overturning Stability:** Overturning stability of the existing dam was analyzed for static and seismic conditions using a two dimensional rigid body model (one foot wide section).

For static conditions and the SDF pool level (Elev. 927.7), the safety factor against overturning was found to be 1.11 and the resultant force was located 2.2 feet upstream of the downstream toe of the dam. Since the base third point is 6.7 feet upstream of the downstream toe, the toe stress condition was evaluated and found to be 4.3 tons per square feet (TSF).

Corresponding conditions were analyzed for a seismic load equivalent to a static horizontal load of 0.025g (Zone I condition). The safety factor against overturning was determined to be 1.09 and the resultant force was located 1.8 feet above the downstream toe. Accordingly, the toe stress was analyzed to be 5.4 TSF.

6.3 **EVALUATION OF STRUCTURAL STABILITY**

a. **Sliding Stability:** Colonial Dam No.3 is considered to have an adequate margin of safety against sliding. This is based on the analysis indicating that a dam/foundation cohesion of 20 psi is necessary to achieve a sliding factor of safety of three. A dam/foundation cohesion of this magnitude is considered to be reasonable based on visual observations and general experience.
a. Overturning Stability: Colonial Dam No. 3 is considered to have an adequate margin of safety against overturning for both static and seismic conditions. Although the resultant force is not located within the middle third of the dam for the analyzed conditions, toe pressures were calculated to be considerably less than the allowable maximums for the dam's concrete and foundation rock.
7.1 **ASSESSMENT**

a. **Evaluation:**

(1) **Gravity Dam:** The concrete gravity dam is considered to be in poor condition. This is based on:

i. The observed breach at the right end of the dam that has resulted in considerable erosion of the dam and abutment.

ii. The observed condition of concrete surfaces, that showed significant spalling, deterioration, seepage and leakage.

iii. The observed deteriorated condition of the outlet works pipes and flow controls.

iv. The "inadequate" capacity rating of the overflow crest using the HEC-1 computer program.

(2) **Hazard Classification and Spillway Design**

    *Flood:* Visual observations of flood plain conditions below Colonial Dam No. 3 indicate the structure has a "significant" hazard classification that requires a SDF of 0.5 PMF.

b. **Adequacy of Information:** The information available on design, construction, operation and performance, in combination with visual observations and hydrology, hydraulic and stability calculations were sufficient to evaluate the dam in accordance with the Phase I investigation guidelines.

c. **Urgency:** The recommendations presented in Section 7.2a and 7.2c should be implemented immediately
7.2 RECOMMENDATIONS

a. Additional Investigations: Immediately retain a professional engineer knowledgeable in dam design and construction to:

(1) Perform a detailed hydrologic/hydraulic analysis of the reservoir and dam and make recommendations on increasing the capacity of the system to make it adequate.

(2) Investigate the operability of the outlet works and provide recommendations on repair requirements.

(3) Provide recommendations on improving the physical condition of the deteriorated gravity dam and right abutment.

b. Emergency Operation and Warning Plan: Concurrent with the additional investigations recommended above, the owner should develop an Emergency Operation and Warning Plan including:

(1) Guidelines for evaluating inflow during periods of heavy precipitation or runoff.

(2) Procedures for around the clock surveillance during periods of heavy precipitation or runoff.

(3) Procedures for drawdown of the reservoir under emergency conditions.

(4) Procedures for notifying downstream residents and public officials, in case evacuation of downstream areas is necessary.

c. Inspection and Maintenance: The owner should develop and implement formal inspection and maintenance procedures.

d. Orderly Breaching: In lieu of performing the above recommendations, the owner should engage the services of a professional engineer, knowledgeable in dam design and performance, to prepare specifications for completely breaching the structure, to make it incapable of impounding water. The structure should then be breached under the direction of the professional engineer, in accordance with applicable state and local regulations.
APPENDIX A

VISUAL INSPECTION CHECKLIST
VISUAL OBSERVATIONS CHECKLIST II
(MASONRY IMPOUNDING STRUCTURE)

Name Dam Colonial No. 3  County Fayette  State Pennsylvania  National ID # PA 00209
Type of Dam Concrete Gravity  Hazard Category Significant
Date (s) Inspection 31 October 1979  Weather Clear, Warm  Temperature 68°
Pool Elevation at Time of Inspection 922 + (MSL) Tailwater at Time of
Inspection 898 + (MSL)

Inspection Personnel:  J. E. Barrick, P.E.  Ackenheil & Associates, Hydrologist and
  Project Manager.
  J. P. Hannan,  Ackenheil & Associates, Geotechnical Engineer
  S. G. Mazzella,  Ackenheil & Associates, Civil Engineer
  E. Yablonski,  Redstone Water Company, Owner's Representative

Recorder  J. E. Barrick  Geo Project G79153-H
  PennDER I.D. No. 26-22
**CONCRETE/MASONARY DAM**

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>GENERAL DESCRIPTION</td>
<td>Colonial Dam No. 3 is a concrete gravity structure which appears to be keyed into bedrock. The crest is five feet wide and contains three - two foot wide crest blocks that are 2.5 feet high. The upstream face is vertical and the downstream face is approximately 1H:2V. The dam is an overflow structure and has no detached spillway. However, a breach has developed at the right abutment and currently serves as the dam's principal spillway.</td>
<td></td>
</tr>
<tr>
<td>SURFACE CRACKS</td>
<td>Crest blocks (parapet) are badly cracked. Downstream face is badly cracked vertically and horizontally. Downstream face is badly spalled with grass and small trees growing on it.</td>
<td></td>
</tr>
<tr>
<td>CONCRETE SURFACES</td>
<td></td>
<td></td>
</tr>
<tr>
<td>STRUCTURAL CRACKING</td>
<td>No significant structural cracks were observed.</td>
<td></td>
</tr>
<tr>
<td>VERTICAL AND HORIZONTAL ALIGNMENT</td>
<td>There does not appear to have been any vertical or horizontal displacement of the structure. Crest is straight and appears level. No evidence of settlement is apparent.</td>
<td></td>
</tr>
<tr>
<td>MONOLITH JOINTS</td>
<td>Spalled cracks observed at several locations.</td>
<td></td>
</tr>
<tr>
<td>CONSTRUCTION JOINTS</td>
<td>Downstream face construction joints are noticeable due to considerable spalling.</td>
<td></td>
</tr>
<tr>
<td>STAFF GAGE AND RECORDER</td>
<td>None observed.</td>
<td></td>
</tr>
</tbody>
</table>
## Visual Examination of Observations

### Any Noticeable Seepage
A seep or wet spot approximately 20 feet up the dam face on right side near the end of the right crest block. An actively flowing seep on right face of dam, just left of the end of right crest block and 7 feet below crest - estimated flow 1 gallon per minute. Significant flow around right abutment beneath the concrete structure in vicinity of breach. This flow is directed along the toe of the dam to the channel below. Numerous seeps in the central portion of the downstream face, the highest of which is 2 feet below crest; other seeps emanating from spalled and cracked sections four and more feet below crest. Three seeps, each discharging approximately 5 gallons per minute, are located six feet below the crest and near the left abutment.

### Structure to Abutment/Embankment Juncions
Structure to abutment seals appear to be generally adequate with exception of breach at right end of structure and three seeps on the left abutment.

### Drains
None observed.

### Water Passages
Leaks, wet areas observed at many construction joints and cracks.

### Foundation
Appears to be adequate. Lower part of structure observed to be founded on competent sandstone.
**CONCRETE/MASONRY DAM**

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>UNUSUAL MOVEMENT OR</td>
<td>None observed.</td>
<td></td>
</tr>
<tr>
<td>CRACKING AT OR BEYOND</td>
<td></td>
<td></td>
</tr>
<tr>
<td>THE TOE</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLOUGHING OR EROSION</td>
<td>None observed.</td>
<td></td>
</tr>
<tr>
<td>OF NON-OVERFLOW</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SECTION SLOPES</td>
<td></td>
<td></td>
</tr>
<tr>
<td>RIPRAP FAILURES</td>
<td>No riprap observed.</td>
<td></td>
</tr>
</tbody>
</table>
### UNGATED OVERFLOW SECTION
(DAM CREST)

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONCRETE WEIR</td>
<td>Badly deteriorated concrete with construction joint cracks.</td>
<td></td>
</tr>
<tr>
<td>APPROACH CHANNEL</td>
<td>Reservoir - unblocked access to weir crest.</td>
<td></td>
</tr>
<tr>
<td>DISCHARGE CHANNEL</td>
<td>Consists of natural rock with steep, heavily wooded side slopes.</td>
<td></td>
</tr>
<tr>
<td>BRIDGE AND PIERS</td>
<td>None observed.</td>
<td></td>
</tr>
</tbody>
</table>
### OUTLET WORKS

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>CRACKING AND SPALLING OF CONCRETE SURFACES IN OUTLET CONDUIT</td>
<td>Concrete encasement for outlet pipes below toe of dam is badly deteriorated.</td>
<td></td>
</tr>
<tr>
<td>INTAKE STRUCTURE</td>
<td>Not observed due to pool level. Above water level, sluice gate slides are badly rusted and do not appear capable of supporting load. Sluice gate width is 30 inches.</td>
<td></td>
</tr>
<tr>
<td>OUTLET STRUCTURE</td>
<td>Outlet pipe 12 inch C.I. and abandoned 8 inch C.I. water supply pipe lies immediately above cast iron outlet pipe. Outlet pipe is broken near end and partially clogged with mud.</td>
<td></td>
</tr>
<tr>
<td>OUTLET CHANNEL</td>
<td>Discharge directly to Washwater Run below toe of dam.</td>
<td></td>
</tr>
<tr>
<td>EMERGENCY GATE</td>
<td>See &quot;Intake Structure&quot; above.</td>
<td></td>
</tr>
</tbody>
</table>
## INSTRUMENTATION

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>MONUMENTATION/SURVEYS</td>
<td>None observed.</td>
<td></td>
</tr>
<tr>
<td>OBSERVATION WELLS</td>
<td>None observed.</td>
<td></td>
</tr>
<tr>
<td>WEIRS</td>
<td>None observed.</td>
<td></td>
</tr>
<tr>
<td>PIEZOMETERS</td>
<td>None observed.</td>
<td></td>
</tr>
<tr>
<td>OTHER</td>
<td>None observed.</td>
<td></td>
</tr>
</tbody>
</table>
### RESERVOIR

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>SLOPES</td>
<td>Generally steep, heavily wooded. No observed recent slope instability around shoreline except at breach area, where erosion of slope is significant.</td>
<td></td>
</tr>
<tr>
<td>SEDIMENTATION</td>
<td>Entire upper end of pond sedimented and covered with vegetation. Sedimentation measured to be four feet below water level along upstream face of dam.</td>
<td></td>
</tr>
<tr>
<td>WATERSHED</td>
<td>Conditions appear to be similar to those indicated on the most recent U.S.G.S. 7-1/2 minute topographic map. No new mining operations or major construction sites were observed. Watershed is mostly farm and woodland.</td>
<td></td>
</tr>
</tbody>
</table>
**DOWNSTREAM CHANNEL**

<table>
<thead>
<tr>
<th>VISUAL EXAMINATION OF</th>
<th>OBSERVATIONS</th>
<th>REMARKS OR RECOMMENDATIONS</th>
</tr>
</thead>
<tbody>
<tr>
<td>CONDITION</td>
<td></td>
<td></td>
</tr>
<tr>
<td>(OBSTRUCTIONS,</td>
<td>Width of downstream channel varies from 50 to 100 feet. Channel banks densely wooded. Considerable brush and downturn. High &quot;n&quot; conditions. Extends approximately 2000 feet where it flows into Redstone Creek.</td>
<td></td>
</tr>
<tr>
<td>DEBRIS, ETC.)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SLOPES</td>
<td>Slopes very steep, heavily wooded. Considerable rock outcrop in valley.</td>
<td></td>
</tr>
<tr>
<td>RAILROAD CULVERT</td>
<td>Immediately upstream of confluence with Redstone Creek, Wahwater Run channel flows through a culvert under a railroad embankment. The entrance is a horseshoe type culvert with flared 45° wingwalls. Culvert entrance is 12 feet high, base width 10.5 feet. Embankment above is 3.5H:2.5V and is 26 feet on the slope to the crest. (Total height above stream bed is 27 feet). Crest width greater than 30 feet and contains a single railroad track and gravel surfaced road.</td>
<td></td>
</tr>
<tr>
<td>APPROXIMATE NO.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>OF HOMES AND</td>
<td>One uninhabited dwelling present in flood plain immediately downstream of culvert outlet. Below this is a beverage distribution building with a residence on the 2nd floor. Across the road from this structure is an uninhabited building. Next nearest inhabited dwelling on floodplain is at Grindstone, 0.9 miles below the railroad culvert (1.7 miles below dam).</td>
<td></td>
</tr>
<tr>
<td>POPULATION</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX B

ENGINEERING DATA CHECKLIST
<table>
<thead>
<tr>
<th>ITEM</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Drawings</td>
<td>**Drawing E 10281, &quot;Section on Line &quot;C&quot;, Washwater Run, for Proposed Dam&quot;, dated 8 August 1907. Unnamed and undated drawing showing details of winch operated sluice gate control.</td>
</tr>
<tr>
<td>As-Built Drawings</td>
<td>See Design Drawings above for added notes and information related to construction changes and subsequent modifications.</td>
</tr>
<tr>
<td>Regional Vicinity Map</td>
<td>U.S.G.S. 7-1/2 Minute Fayette City, Pennsylvania Quadrangle. See Appendix E.</td>
</tr>
<tr>
<td>ITEM</td>
<td>REMARKS</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Construction History</td>
<td>Plans were prepared by E. J. Taylor, Chief Engineer and Construction Supervisor, Pittsburgh Coal Company, 1907.</td>
</tr>
<tr>
<td></td>
<td>Built in 1907 by Maynard and Flynn, Contractors of Pittsburgh, Pennsylvania.</td>
</tr>
<tr>
<td>Typical Sections of Dam</td>
<td>See Design Drawings above.</td>
</tr>
<tr>
<td>Outlets - Plans</td>
<td>See Design Drawings above.</td>
</tr>
<tr>
<td>Details</td>
<td></td>
</tr>
<tr>
<td>Constraints</td>
<td></td>
</tr>
<tr>
<td>Outlet Discharge Ratings</td>
<td>See Design Reports below.</td>
</tr>
<tr>
<td>Rainfall/Reservoir Records</td>
<td>See Design Reports below.</td>
</tr>
<tr>
<td>Design Reports</td>
<td>See &quot;Report Upon the H.C. Frick Coke Company's Dam&quot; dated Harrisburg, 20 October 1914. Also, draft report, dated 25 June 1914.</td>
</tr>
<tr>
<td>Geology Report</td>
<td>None available.</td>
</tr>
<tr>
<td>Hydrology and Hydraulics</td>
<td>See Design Reports above.</td>
</tr>
<tr>
<td>ITEM</td>
<td>REMARKS</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>-------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Design Computations</td>
<td>See Design Reports above. Also, pencil drawing titled &quot;H. C. Frick Coke Company, Dam on Washwater Run, for Colonial #3 Plant&quot; undated, showing dam cross-section.</td>
</tr>
<tr>
<td>Dam Stability</td>
<td>None available.</td>
</tr>
<tr>
<td>Seepage Studies</td>
<td></td>
</tr>
<tr>
<td>Materials Investigations</td>
<td></td>
</tr>
<tr>
<td>Boring Records</td>
<td></td>
</tr>
<tr>
<td>Laboratory Field</td>
<td></td>
</tr>
<tr>
<td>Post-Construction Surveys</td>
<td>See As-Built Drawings above.</td>
</tr>
<tr>
<td>of Dam</td>
<td></td>
</tr>
<tr>
<td>Borrow Sources</td>
<td>Not applicable.</td>
</tr>
<tr>
<td>Modifications</td>
<td>See As-Built Drawings above.</td>
</tr>
</tbody>
</table>

In 1908 the parapet was increased 1 foot in height from elevation 925.5 to 926.5.

Following the storm of July 1912 when the dam was overtopped and the sides were washed out down to bedrock, several corrective measures were undertaken during October and November of 1912:
<table>
<thead>
<tr>
<th>ITEM</th>
<th>REMARKS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modifications (continued)</td>
<td>(1) Reservoir was drained and all cracks which showed on the upstream face were raked out and carefully pointed with a cement mortar and the entire upstream face was plastered with mortar.</td>
</tr>
<tr>
<td></td>
<td>(2) Extra masonry was placed on the dam's downstream face carrying out the outline to the dimensions originally contemplated.</td>
</tr>
<tr>
<td></td>
<td>(3) Concrete blocks were placed along the side hill downstream to break the force of any water that might overflow the parapet.</td>
</tr>
<tr>
<td></td>
<td>(4) A portion of the dam was cut away and rebuilt, being carried about 1 foot deeper into the hillside than before.</td>
</tr>
<tr>
<td></td>
<td>(5) The length of the original spillway near the center of the dam was increased from 25 feet to 39.7 feet by cutting out a portion of the concrete.</td>
</tr>
<tr>
<td></td>
<td>(6) An additional spillway of 13.4 feet in length was cut out of the concrete on the right side.</td>
</tr>
<tr>
<td>ITEM</td>
<td>REMARKS</td>
</tr>
<tr>
<td>-------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>*Modifications (continued)</td>
<td>During the summer of 1919, holes were drilled into the concrete and cement grout was forced into the concrete under pressure which stopped all leakage at the horizontal construction joints.</td>
</tr>
<tr>
<td>*High Pool Records</td>
<td>Miscellaneous correspondence indicates that the storm of July 1912 overtopped the parapet of the dam by 6 inches. The flood of 4 June 1941 overtopped the parapet by 1.1 feet.</td>
</tr>
<tr>
<td>Prior Accidents or Failure of Dam Description Reports</td>
<td>None reported.</td>
</tr>
<tr>
<td>Maintenance Operation Records</td>
<td>None available.</td>
</tr>
<tr>
<td>*Spillway - Plan Section Details</td>
<td>See Design Drawing above.</td>
</tr>
<tr>
<td>Specifications</td>
<td>None available.</td>
</tr>
</tbody>
</table>

* Information and data may be obtained from the Pennsylvania Department of Environmental Resources, Harrisburg, Pennsylvania.

** Included as plates in Appendix E.
APPENDIX C

PHOTOGRAPHS
COLONIAL DAM No 3

PHOTO 1 UPSTREAM FACE

PHOTO 2 DOWNSTREAM FACE
COLONIAL DAM No 3

PHOTO 3. RIGHT ABUTMENT BREACH

PHOTO 4. RIGHT ABUTMENT BREACH
COLONIAL DAM No 3

PHOTO 5  BREACH FLOW

PHOTO 6  BREACH
COLONIAL DAM No 3

PHOTO 7  BREACH DISCHARGE CHANNEL

PHOTO 8  SEEPAGE
PHOTO 9. OUTLET WORKS PIPES

PHOTO 10. POND DRAIN OUTLET
DETAILED PHOTO DESCRIPTIONS

Photo 1  **Upstream Face** of dam from left abutment, showing parapet openings and right abutment. Note debris in vicinity of the breach.

Photo 2  **Downstream Face** showing deteriorated concrete surfaces and vegetal growth on dam.

Photo 3  **Right Abutment Breach** looking downstream. Showing dam crest overhang.

Photo 4  **Right Abutment Breach** looking upstream. Note erosion of abutment.

Photo 5  **Breach Flow** as seen from below dam. Water is discharging from beneath the concrete dam.

Photo 6  **Breach** looking upstream showing overhanging condition of dam crest.

Photo 7  **Breach Discharge Channel** looking downstream from breach. Dam toe is at left of photo.

Photo 8  **Seepage** through dam on left abutment.

Photo 9  **Outlet Works Pipes** at toe of dam. Water supply pipe above and pond drain pipe below.

Photo 10  **Pond Drain Outlet** showing broken and deteriorated conditions.
APPENDIX D

HYDROLOGY AND HYDRAULICS
ANALYSES
APPENDIX D
HYDROLOGY AND HYDRAULICS

Methodology: The dam overtopping analysis was accomplished using the systemized computer program HEC-1 (Dam Safety Version), July, 1978, prepared by the Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, California. A brief description of the methodology used in the analysis is presented below.

1. Precipitation: The Probable Maximum Precipitation (PMP) is derived and determined from regional charts prepared from past rainfall records including "Hydrometeorological Report No. 33" prepared by the U.S. Weather Bureau.

The index rainfall is reduced from 10% to 20% depending on watershed size by utilization of what is termed the HOP Brook adjustment factor. Distribution of the total rainfall is made by the computer program using distribution methods developed by the Corps.

2. Inflow Hydrograph: The hydrologic analysis used in development of the overtopping potential is based on applying a hypothetical storm to a unit hydrograph to obtain the inflow hydrograph for reservoir routing.

The unit hydrograph is developed using the Snyder method. This method requires calculation of several key parameters. The following list gives these parameters, their definition and how they were obtained for these analyses.

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<tr>
<th>Parameter</th>
<th>Definition</th>
<th>Where Obtained</th>
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<tr>
<td>Ct</td>
<td>Coefficient representing variations of watershed</td>
<td>From Corps of Engineers</td>
</tr>
<tr>
<td>L</td>
<td>Length of main stream channel</td>
<td>From U.S.G.S. 7.5 minute topographic map</td>
</tr>
<tr>
<td>Lca</td>
<td>Length on main stream to centroid of watershed</td>
<td>From U.S.G.S. 7.5 minute topographic map</td>
</tr>
</tbody>
</table>
Cp Peaking coefficient From Corps of Engineers*

A Watershed size From U.S.G.S. 7.5 minute topographic map

3. Routing: Reservoir routing is accomplished by using Modified Puls routing techniques where the flood hydrograph is routed through reservoir storage. Hydraulic capacities of the outlet works, spillways and the crest of the dam are used as outlet controls in the routing.

The hydraulic capacity of the outlet works can either be calculated and input or sufficient dimensions input and the program will calculate an elevation-discharge relationship.

Storage in the pool area is defined by an area-elevation relationship from which the computer calculates storage. Surface areas are either planimetered from available mapping or U.S.G.S. 7.5 minute series topographic maps or taken from reasonably accurate design data.

4. Dam Overtopping: Using given percentages of the PMF the computer program will calculate the percentage of the PMF which can be controlled by the reservoir and spillway without the dam overtopping.

*Developed by the Corps of Engineers on a regional basis for Pennsylvania.
HYDROLOGIC AND HYDRAULIC ENGINEERING DATA

DRAINAGE AREA CHARACTERISTICS: Mostly woodland and
pasture

ELEVATION TOP NORMAL POOL (STORAGE CAPACITY): 924.1 (23 acre-feet).

ELEVATION TOP FLOOD CONTROL POOL (STORAGE CAPACITY): 925.7 (26 acre-feet).

ELEVATION MAXIMUM DESIGN POOL: 925.7

ELEVATION TOP DAM: 925.7 (minimum) 926.5 (average)

OVERFLOW SECTIONS

a. Elevation 924.1 (right) 924.2 (left)
b. Type Free overfall
c. Width 5 feet
d. Length 39.7 feet (right) 13.3 feet (left)
e. Location Spillover right and left of center
f. Number and Type of Gates None

OUTLET WORKS

a. Type 12 inch cast iron pipe
b. Location left of center
c. Entrance Inverts + 899.25
d. Exit Inverts + 899
e. Emergency Drawdown Facilities Sluice gate (inoperative)

HYDROMETEOROLOGICAL GAGES

a. Type None
b. Location N/A
c. Records None

MAXIMUM REPORTED NON-DAMAGING DISCHARGE Parapet overtopped by 1.1 feet during storm of
4 June 1941 also by 0.5 foot in July 1912.
NAME OF DAM: Colonial Dam No. 3  
NDI ID NO.  
PA 209

Probable Maximum Precipitation (PMP)  
24.2*

Drainage Area  
1.5 sq. mi.

Reduction of PMP Rainfall for Data Fit  
Reduce by 20%, therefore PMP rainfall =  
=19.4 in.

Adjustments of PMF for Drainage Area (Zone 7)  
6 hrs.  
102%  
12 hrs.  
120%  
24 hrs.  
130%

Snyder Unit Hydrograph Parameters  
Zone  
29**  
Cp  
0.5  
Ct  
1.6  
L  
2.0 mile  
Lca  
1.1 mile  
\[ t_p = C_t (L \cdot Lca)^0.3 \]  
2.0 hours

Loss Rates  
Initial Loss  
1.0 inch  
Constant Loss Rate  
0.05 inch/hour

Base Flow Generation Parameters  
Flow at Start of Storm  
1.5 cfs x 1.5 sq.mi. = 2.25 cfs  
Base Flow Cutoff  
0.5 peak  
Recession Ratio  
2.0

Overflow Section Data  
Crest Length  
13.3 and 39.7 feet  
Freeboard  
1.6 feet  
Discharge Coefficient  
2.3-3.08  
Exponent  
1.5  
Discharge Capacity  
316.4 cfs

---

* Hydrometeorological Report 33  
** Hydrological zone defined by Corps of Engineers,  
Baltimore District, for determining Snyder's Coefficients  
(Cp and Ct).
LOSS RATE AND BASE FLOW PARAMETERS

As Recommended by Corps of Engineers, Baltimore District

STETL = 11 inch
CNSTL = 0.05 inches/hour
STETQ = 1.5 cfs/m²
QRCSTN = 0.05 (570 of Peak Flow)
ETIOE = 2.0

ELEVATION - AREA - CAPACITY - RELATIONSHIPS

From USGS, 7.5 min Quad, Friends and Field Inspection data

AT ELEVATION 924.1
Initial Storage = 23 Acre Feet
Pond Surface Area = 1.5 Acres

AT ELEVATION 940, AREA = 3.7 Acres

AT ELEVATION 960, AREA = 14.7 Acres

From Conc Method for Reservoir Volume
Flood Hydrograph Package (Hec-1)
Dam Safety Version (Users Manual)

\[ H = \frac{2V}{A} = \frac{2(23)}{1.5} = 4.6 \text{ Feet} \]

Elevation where Area Equals Zero
924.1 - 4.6 = 878.1

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<tr>
<td>940.0</td>
<td>3.7</td>
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<tr>
<td>960.0</td>
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Overtop Parameters

Top of Parapet Elevation (minimum) = 925.7
Length of Dam (excluding Spillway) = 115.5
Coefficient of Discharge "C" = 3.1
SL max = 120.0  $V_{max} = 930.0$

Program Schedule

1. Inflow Colonial Dam
2. Reservoir
3. Route Colonial Dam
4. END
Overflow Section on Right

\( Q = CLH^{1/2} \)

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<th>( Q ) (cfs)</th>
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Values of \( C \) interpolated from Table 5-3 page 5-46

"King and Breder", Handbook of Hydraulics
Overflow Section on Left

\[ Q = CLH^{3/2} \]

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<th>( Q^2 (\text{cfs}) ) Total Flow</th>
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PREVIEW OF SEQUENCE OF STREAM NETWORK CALCULATIONS

RUNOFF HYDROGRAPH AT 1
ROUTE HYDROGRAPH TO 2
END OF NETWORK

---

NATIONAL PROGRAM FOR THE INSPECTION OF NON FEDERAL DAMS
HYDROLOGIC AND HYDRAULIC ANALYSIS OF COLONIAL NO. 3 DAM
PROBABLE MAXIMUM FLOOD PMF/UNIT HYDROGRAPH BY SNYDER'S METHOD

JOB SPECIFICATION

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JOPER: MAT: LRCPT: TRACE

MULTI-PLAN ANALYSES TO BE PERFORMED

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D9
**SUB-AREA RUNOFF CALCULATION**

**INFLOW HYDROGRAPH FOR COLONIAL NO. 3 DAM**

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**END-OF-PERIOD FLOW**

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<th>EXCS</th>
<th>LOSS</th>
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<td>593.1</td>
<td>48.1</td>
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**HYDROGRAPH ROUTING**

**ROUTING AT COLONIAL NO. 3 DAM**

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**UNIT HYDROGRAPH DATA**

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<td>982</td>
<td></td>
<td></td>
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<tr>
<td>655</td>
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<td></td>
</tr>
<tr>
<td>327</td>
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</table>

**SUMMARY**

**PLAN-RATIO ECONOMIC COMPUTATIONS**

<table>
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<th>RATIO</th>
<th>1.00</th>
<th>0.90</th>
<th>0.80</th>
<th>0.70</th>
<th>0.60</th>
<th>0.50</th>
<th>0.40</th>
<th>0.30</th>
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<tbody>
<tr>
<td>1</td>
<td>327</td>
<td>2947</td>
<td>2619</td>
<td>2292</td>
<td>1964</td>
<td>1637</td>
<td>982</td>
<td>655</td>
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<td>2</td>
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<td>92.7</td>
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<td>92.7</td>
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</table>

**SUMMARY OF DAM SAFETY ANALYSIS**

<table>
<thead>
<tr>
<th>PLAN</th>
<th>ELEVATION</th>
<th>INITIAL VALUE</th>
<th>SPILLWAY CREST</th>
<th>TOP OF DAM</th>
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<tr>
<td>1</td>
<td>924.10</td>
<td>924.10</td>
<td>925.70</td>
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<td>3.27</td>
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<td>3</td>
<td>928.74</td>
<td>3.24</td>
<td>31</td>
<td>294.7</td>
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<td>4</td>
<td>928.50</td>
<td>2.80</td>
<td>31</td>
<td>261.9</td>
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<tr>
<td>5</td>
<td>928.24</td>
<td>2.54</td>
<td>30</td>
<td>229.2</td>
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<tr>
<td>6</td>
<td>927.98</td>
<td>2.28</td>
<td>30</td>
<td>206.4</td>
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<tr>
<td>7</td>
<td>927.66</td>
<td>1.68</td>
<td>29</td>
<td>161.7</td>
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<tr>
<td>8</td>
<td>927.31</td>
<td>1.31</td>
<td>26</td>
<td>98.2</td>
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<tr>
<td>9</td>
<td>926.58</td>
<td>0.88</td>
<td>26</td>
<td>99.5</td>
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<tr>
<td>10</td>
<td>926.06</td>
<td>0.16</td>
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<td>327</td>
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</table>

**PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY**

<table>
<thead>
<tr>
<th>OPERATION</th>
<th>STATION</th>
<th>AREA</th>
<th>PLAN</th>
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<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
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<tbody>
<tr>
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<td>1.50</td>
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<td>1964</td>
<td>1637</td>
<td>982</td>
<td>655</td>
<td>327</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>1.50</td>
<td>327</td>
<td>2947</td>
<td>2619</td>
<td>2292</td>
<td>1964</td>
<td>1637</td>
<td>982</td>
<td>655</td>
<td>327</td>
<td></td>
</tr>
</tbody>
</table>
HYDROLOGIC PERFORMANCE PLOT

Maximum Reservoir Water Surface Elevation

Minimum Dam Elevation

- 90% UNDER EXISTING (NON-BEACH) CONDITIONS

07. DMF
APPENDIX E

PLATES
LIST OF PLATES

Plate I  Regional Vicinity Map
Plate II  Plan Showing Topography Below the Dam
Plate III Cross Section of Dam
Colonial Dam No. 3 is located in the Pittsburgh Plateau section of the Appalachian Plateau physiographic province. This region is characterized by essentially flat lying strata at an altitude great enough to have permitted deep valley cutting by streams. Colonial Dam No. 3 is located along a narrow steep sided valley formed by Washwater Run, a tributary of Redstone Creek. The confluence of these two streams is about Elev. 810 ft.

**Structure**

General: The axis of the Lambert Syncline lies approximately 0.3 miles west of the dam. This syncline trends N14°E and plunges to the northeast. This syncline is typical of the alternating series of broad anticlines and synclines which characterize the Pittsburgh Plateau section. Based on estimates obtained from the "Coal and Surface Structure Map of Fayette County, Pennsylvania" the rock strata in the vicinity of the dam dip to the northwest.

Faults: No observations were made that would indicate faulting in the rocks outcropping around the dam site. In general, only a few evidences of faulting have been observed in all of Fayette County.

**Stratigraphy**

General: Rocks outcropping in the immediate vicinity of Colonial Dam No. 3 belong to the Uniontown Formation, Monongahela Group of Upper Pennsylvanian age; and the Waynesburg Formation, Dunkard Group of Permian age.

Waynesburg Coal: The Waynesburg Coal is the lowest bed in the Dunkard Group. It is often used as the markerbed to separate the Dunkard Group from the Monongahela Group. It outcrops at approximately Elev. 980 ft., or about 55 ft. above the reservoir surface. This coal is multiple bedded and about 5 ft. thick. It is separated into 2 or 3 benches by gray clay shale or plastic clay partings.
Arnoldsburg Sandstone: Rock excavated at the embankment foundation was described in the report cited in Section 2.1 b(1) as a sandstone overlain by a shale. The sandstone is believed to be the Arnoldsburg Sandstone of the Waynesburg Formation. This stratum has been described as a medium grained, soft gray brown sandstone occurring in thin to medium beds. Its thickness averages 10 feet in this area.

Uniontown Limestone: The Uniontown Limestone of the Uniontown Formation lies immediately above the Arnoldsburg sandstone. The limey shale beds at the base of this unit are believed to be the shales noted as occurring above the sandstone in a report cited in Section 2.1a(2). This unit averages 15 feet in thickness.

Upper Benwood Limestone: This unit occurs stratigraphically below the Arnoldsburg Sandstone and is composed of a light to dark gray fine grained interbedded with clay, limey shale and sandstone. Its average thickness is 50 feet.
<table>
<thead>
<tr>
<th>AGE</th>
<th>COLUMN</th>
<th>PROLREN BEDS</th>
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<tbody>
<tr>
<td>CENOEMERI</td>
<td></td>
<td>PLEISTOCENE GLACIAL, OUTWASH, RIVER TERACE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>DEPOSITS AND ALLUVEUM</td>
</tr>
<tr>
<td>PERMAN</td>
<td></td>
<td>UPPER WASHINGTON LIMESTONE</td>
</tr>
<tr>
<td>DEVONIAN</td>
<td></td>
<td>WASHINGTON COAL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WAYNESBURG SANDSTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WAYNESBURG COAL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WAYNETOWN SANDSTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>WAYNETOWN COAL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BOWGWOOD LIMESTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BOWGLEY COAL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>PITTSBURGH SANDSTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>PITTSBURGH COAL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CONNELLVILLE SANDSTONE</td>
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<tr>
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<tr>
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<td>MORGANTOWN SANDSTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>AMES LIMESTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>PITTSBURGH RED BEDS</td>
</tr>
<tr>
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<td></td>
<td>PALTIBURGH SANDSTONE</td>
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<tr>
<td></td>
<td></td>
<td>MANOWING SANDSTONE</td>
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<tr>
<td></td>
<td></td>
<td>UPPER FREEPORT COAL</td>
</tr>
<tr>
<td></td>
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<td>UPPER KITTANNING COAL</td>
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<tr>
<td></td>
<td></td>
<td>WORTHINGTON SANDSTONE</td>
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<td></td>
<td></td>
<td>LOWER KITTANNING COAL</td>
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<td></td>
<td></td>
<td>HOMERWOOD SANDSTONE</td>
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<td></td>
<td></td>
<td>MERCER SANDSTONE, SHALE &amp; COAL</td>
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<tr>
<td></td>
<td></td>
<td>CONGOSMENESSING SANDSTONE</td>
</tr>
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<td></td>
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<td>BURGOSN SANDSTONE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CUYAHOGA SHALE</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BEREON SANDSTONE</td>
</tr>
</tbody>
</table>

DATE: MAY 1980
SCALE: NONE
OR: JF CK: JPH
A. C. ACKENHEIL & ASSOCIATES, INC.
CONSULTING ENGINEERS
PITTSBURGH, PA., CHARLESTON, W. VA. & BALTIMORE, MD.
APPENDIX G

STABILITY ANALYSES
<table>
<thead>
<tr>
<th>Analysis</th>
<th>Page No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Department of Forests and Waters</td>
<td>G2</td>
</tr>
<tr>
<td>Sliding Stability - Existing Conditions</td>
<td>G6</td>
</tr>
<tr>
<td>Overturning Stability - Existing Conditions</td>
<td>G19</td>
</tr>
</tbody>
</table>
The forces assumed as acting on the section are:

1. Weight of masonry, taken as 145 pounds per cubic foot, and acting vertically.

2. The horizontal pressure of the water against the vertical face of the dam, with water at elevation 934 ft.

3. Upward water pressure on the base, considered as 2/3 that due to the hydrostatic head, applied at the heel, decreasing uniformly to zero at the toe, and acting 1/3 the distance from the heel to the toe.

4. No ice action, the lip of the spillway being rounded so that ice cannot cling to it.

The overflow section of the dam was uncontrolled, it being under more extreme conditions than the section containing the parapet. The forces, moments, etc. acting on this dam at various elevations are shown in the following tabulation:

<table>
<thead>
<tr>
<th>TABLE</th>
<th>918</th>
<th>912</th>
<th>906</th>
<th>900</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1) - Elevation of Joint</td>
<td>918</td>
<td>912</td>
<td>906</td>
<td>900</td>
</tr>
<tr>
<td>(2) - Elevation water surface</td>
<td>926.5</td>
<td>926.5</td>
<td>926.5</td>
<td>926.5</td>
</tr>
<tr>
<td>(3) - Top of dam</td>
<td>934</td>
<td>934</td>
<td>934</td>
<td>934</td>
</tr>
<tr>
<td>(4) - Width at elevation of (3) in ft</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>(5) - - - (1) - -</td>
<td>0</td>
<td>12</td>
<td>16</td>
<td>20</td>
</tr>
<tr>
<td>(6) - Weight of masonry above joint in lbs.</td>
<td>5600</td>
<td>12840</td>
<td>20740</td>
<td>28630</td>
</tr>
<tr>
<td>(7) - Lever arm of masonry about bottom in ft.</td>
<td>4.64</td>
<td>4.84</td>
<td>5.06</td>
<td>5.28</td>
</tr>
<tr>
<td>(8) - Upward pressure of water in lbs.</td>
<td>1620</td>
<td>3380</td>
<td>5930</td>
<td>11040</td>
</tr>
<tr>
<td>(9) - Lever arm about bottom in ft.</td>
<td>5 1/3</td>
<td>7 1/3</td>
<td>9 1/3</td>
<td>13 1/3</td>
</tr>
<tr>
<td>(10) - Net Vertical Forces (6) - (8) in lbs.</td>
<td>4160</td>
<td>10560</td>
<td>19440</td>
<td>28320</td>
</tr>
<tr>
<td>(11) Distance of line of action of (10) from toe</td>
<td>4.44</td>
<td>6.46</td>
<td>8.48</td>
<td>10.70</td>
</tr>
<tr>
<td>(12) - Horizontal pressure of water in lbs.</td>
<td>2040</td>
<td>4840</td>
<td>8640</td>
<td>12700</td>
</tr>
<tr>
<td>(13) - Lever arm of (12) about toe</td>
<td>4.64</td>
<td>6.64</td>
<td>8.64</td>
<td>10.64</td>
</tr>
<tr>
<td>(14) - Shear stress pressure on joint</td>
<td>6600</td>
<td>12900</td>
<td>22300</td>
<td>31700</td>
</tr>
</tbody>
</table>
(15) - Distance from toe to which N cuts joint.  3.24  5.90  4.14  7.04
(16) - Distance from middle third (-- indicates outside)  0.57  0.23  0.13  0.07
(17) - Total overturning moment - ft. lbs.  12440  98720  142000  530400
(18) - N resisting  26220  94800  217700  647000
(19) - Ratio 16\psi x(11) \begin{pmatrix} 2 \theta \end{pmatrix}  2.07  1.76  1.56  1.40
(20) - Ratio......\frac{2\theta}{2\theta + \theta} \times (11)  2.66  2.45  2.25  2.06
(21) - Ratio......\frac{(11)}{(12)}  \begin{pmatrix} 2 \theta \end{pmatrix} x (11)  2.66  2.45  2.25  2.06
(22) - Pressure at toe, Reservoir full - tons per sq. ft.  6.41 6.04 5.69 5.59
(23) - Pressure at heel, Reservoir empty - tons per sq. ft.  5.53 5.06

Under the above conditions, with the reservoir full, the resultant cuts the joints investigated inside the middle third point, with one exception, namely section 900, where it falls 0.53 ft. outside; however, for section 900 it is again well inside. For reservoir empty the resultant is within the middle third, with the exception of the joint at elevation 900, where it is 0.25 ft. outside. The tension caused by the above is practically negligible. Line 19 indicates the ratio of resisting to overturning moment, and shows the factor of strength possessed by the section against rotation about its toe.

Line 21 shows the ratio of horizontal to vertical forces, and indicates the resistance of the section to sliding on the base. The maximum pressure on the toe is 1.36 tons per square foot, with the reservoir full, and on the heel 0.06 tons per square foot, with the reservoir empty, which are very moderate. In line 21 it is seen that for the section at elevation 900, the tangent of the angle at which the resultant cuts the base is greater than allowable, indicating that the dam is about to slide on its base. This is due to assuming an upward hydrostatic pressure on the base, even though the foundation was gouged, and very little water was encountered; it neglects any strength being developed by the reservoir, due to the embedded stones, or re-anchoring of the base. In my opinion, the section is sufficiently heavy.
Assume upward pressure due to full head at upper face and none at toe and figure resultant.

\[ \frac{1212}{1} = \text{upward pressure} \]

\[ \begin{align*}
B & = 1.4 \times 10^4 \\
T & = 1.4 \times 10^3 \\
N & = 1.4 \times 10^2
\end{align*} \]

Spillway capacities:

\[ \frac{Q_1}{Q_2} = \frac{0.125}{0.15} = 0.833 \]

Flow over dam in \( 1913 \):

\[ \begin{align*}
\frac{1}{2} & = 102.16 \times 102.16 \\
& = 2.53 \times 10^6 \\
\frac{1}{7} & = 171.6 \times 102.16 \\
& = 1.71 \times 10^5 \\
\end{align*} \]
Subject: Sliding Stability

Sheet 1 of 12

Job: Colonial Loan A-2
Job No: 7953H

GEO Systems, Inc.
1000 Banksville Road
PITTSBURGH, PA. 15216
(412) 331-7111

Date: [Blank]

ACKENHEIL & ASSOCIATES

41' 13' 21'
46' 40'

G6

ELEVATION (LOOKING DOWNSTAIRS)
WEIGHT OF DAM

SECTION A-A

Area of Concrete = \( 2(20) + \frac{1}{2}(24)(5 + 17) \)
= 304 sq ft

Area of Concrete without base = 264 sq ft

VOLUME OF CONCRETE

LENGTH A

\[ V = \frac{1}{2}bh \]

\[ = \frac{1}{3}(264)(64) = 5632 \text{ cf} \]

LENGTH B

\[ V = 39(304) = 11856 \text{ cf} \]

LENGTH C-D

\[ V = \frac{1}{3}bh \]

\[ = \frac{1}{3}(264)(68) = 5104 \text{ cf} \]

PARAPET

\[ V = 2(2.5)(41 + 21 + 46) = 540 \text{ cf} \]

TOTAL VOLUME = 23132 cf

TOTAL WEIGHT @ 145 pcf* = 3354 kips

AREA OF BASE

LENGTH A

\[ W_1 = 5 \quad W_2 = 17 \quad L = -\sqrt{20^2 + 64^2} = 68.4 \text{ ft} \]

\[ A_A = \frac{68.4}{2} \cdot (5 + 17) = 752 \text{ sf} \]

LENGTH B, C, and C

\[ W_1 = 2 \quad W_2 = 20 \]

\[ A_{B_1} = A_{B_3} = 2(20) = 40 \text{ sf} \]

LENGTH B

\[ W_1 = 39 \quad W_2 = 20 \]

\[ A_B = 780 \text{ sf} \]

* Unreinforced concrete
LENGTH C  \[ W_1 = 17 \quad W_e = 10.8 \quad L = \sqrt{23.2 + 11.6^2} \]
\[ = 32.3 \]
\[ A_c = \frac{32.3}{2} (17 + 10.8) = 421 \text{ SF} \]

\[ \text{AREA OF ROOF} = 2033 \text{ SF} \]

*ERROR NOTED ON CHECK  \[ W_2 = 11.2 \]

SUBSEQUENT CALCULATIONS NOT CHANGED

AS ERROR HAS NO EFFECT ON FINAL ANSWER

JEB
SLIDING RESISTANCE

Assumptions: Neglect weight of water above dam crest.

Assume $\lambda = 0.70$ as per NAVFAC DM-7 for mass concrete on clean sound rock. See Table 10-1.

Breach length has no resistance to sliding or uplift pressure.

Foundation uplift varies from full hydrostatic pressure at upstream edge to zero at the downstream edge.

Red level is at:
(1) Top of parapet
(2) 5' above parapet

(1) RED LEVEL AT TOP OF PARAPET:

LENGTH A

Base length at right = 17' $\rightarrow$ $U_{uplift, r} = \frac{1}{2}(62.4)(26.5)(17) = 14.1 \text{ K}$

Base length at left = 5' $\rightarrow$ $U_{uplift, l} = \frac{1}{2}(62.4)(2.5)(5) = 0.4 \text{ K}$

Uplift over length A

$U_{uplift, A} = \frac{L}{3} (P_c + \sqrt{P_c P_e} + P_e) = 360 \text{ K}$
**LENGTH B**

Base length = 20'  
\[ U_{\text{strip}} = \frac{1}{2} (6.24) (2.5) (20) \]
\[ = 17.5 K/ft \]

Uplift over length B

\[ U_{\text{up}}(B) = 17.5 \times 20 = 350 K \]

**LENGTH C**

Base length at left = 17'  
\[ U_{\text{strip}} = 4.1 K \]

Base length at right = 10.8'  
\[ U_{\text{strip}} = \frac{1}{2} (6.24)(4.1)(10.8) \]
\[ = 4.3 K \]

Uplift over length C

\[ U_{\text{up}}(C) = \frac{28}{3} (14.1 + 4.8 + \sqrt{4.1 	imes 8.8}) K = 253 K \]

Total Uplift = 350K + 694K + 253K = 1307K

Effective normal load on foundation

\[ \bar{N} = 3354K - 1307K = 2047K \]

\[ \text{Surface} = 2047K (0.7) \]
\[ \text{Resulting} = 1433 K \]
(2) POOL LEVEL 5' ABOVE POROLPET

**LENGTH A**

Right end \( U_{\text{uplift}} = \frac{1}{2} (62.4)(31.5)(17) = 16.7 \text{k} \)

Left end \( U_{\text{uplift}} = \frac{1}{2} (62.4)(7.5)(5) = 1.2 \text{k} \)

Uplift over length A = \( \frac{64}{3} (16.7 + 1.2 + \sqrt{16.7(1.2)}) \text{k} = 477 \text{k} \)

**LENGTH B**

Uplift over length B = \( 39(\frac{1}{2})(62.4)(33.5)(20) \)

= 815 \text{k}

**LENGTH C**

Left end \( U_{\text{uplift}} = 16.7 \text{k} \)

Right end \( U_{\text{uplift}} = \frac{1}{2} (62.4)(19.1)(10.8) \)

= 6.4 \text{k}

Uplift over length C = \( \frac{64}{3} (16.7 + 6.4 + \sqrt{16.7(6.4)}) \text{k} = 312 \text{k} \)

Total uplift for (2) = 477 \text{k} + 815 \text{k} + 312 \text{k} = 1604 \text{k}

\( N = 3354 \text{k} - 1604 \text{k} = 1750 \text{k} \)

\( P_{\text{down}} = 1225 \text{k} \)
PASSIVE RESISTANCE

Assumptions:  Dom is constructed a minimum two feet into competent rock

Rock has $\theta = 45^\circ$ $c = 0$ $r = 140$ pcf

No passive resistance exist over
break length

\[ K_p = \tan^2 (45^\circ + \frac{\theta}{2}) \]
\[ = \tan^2 (67.5^\circ) = 5.8 \]

\[ P_p = \frac{1}{2} K_p \gamma H^2 \]
\[ = \frac{1}{2} (5.8) (140) (2)^2 \]
\[ = 2k \]

TOTAL RESISTANCE TO SLIDING

(1) 1435K

(2) 1227K.
ACKENHEIL AND ASSOCIATES INC
BALTIMORE MD
F/9
13/13
NATIONAL DAM INSPECTION PROGRAM
COLONIAL DAM NUMBER 3
(MAY 80)
DACW31-80-C-0026
UNCLASSIFIED
DRIVING FORCES

Assumptions:
- Real level is at:
  (1) Top of Parapet
  (2) 5' above parapet
- Parapet is continuous across top of dam
- Fluid drag forces are neglected.
- Sediment has M=8000 lb, d=30, c=10 psf

(1) POOL LEVEL AT TOP OF PARAPET

LENGTH A - WATER

$F_{right} = \frac{1}{2} (62.4)(76.5)^2 = 21.5\text{K}$
$F_{left} = \frac{1}{2} (62.4)(7.5)^2 = 0.2\text{K}$

$F_{A} = \frac{64}{3} (21.5+0.2+\sqrt{21.9\cdot0.2})$
$= 516\text{K}$

LENGTH B - WATER

$F_{left} = \frac{1}{2} (62.4)(25.5)^2 = 25.3\text{K}$
$F_{B} = 99(25.3\text{K}) = 2537\text{K}$

LENGTH C - D - WATER

$F_{left} = 21.9\text{K}$
$F_{right} = 0.2\text{K}$

$F_{CD} = \frac{58}{3} (21.9+0.2+\sqrt{21.9\cdot0.2})$
$= 468\text{K}$.
Total Hydrostatic Pressure Load

\[ F_h = 197.1K \]

Length A - Segment

\[ k_a = \tan^2 \left( 45^\circ - \frac{\phi}{2} \right) = 0.9 \quad \phi = 3^\circ \]

\[ f = \frac{1}{2} (5.5 - 62.4 \times 0.9) (18)^2 = 2.6K \]

\[ f_{eff} = 0 \]

\[ L_A = \frac{18}{2} (64) = 48.0 \]

\[ S_A = \frac{1}{3}(48)(2.6) = 42K \]

Length B -

\[ f = 2.6K \]

\[ S_B = 2.6(39) = 102K \]

Length C-D

\[ f_{eff} = 2.6K \]

\[ f_{fict} = 0.1 \]

\[ L_{CD} = \frac{18}{2} (68) = 43.5 \]

\[ S_{CD} = \frac{1}{3}(2.6)(43.5) = 38K \]

Total Active Segment Pressure

\[ S = 182K \]

Total Gravity Force (1)

\[ F_h = 197.1K + 182K = 2153K \]
(2) POOL LEVEL 5' ABOVE TOP OF PUMPSET

\[ \text{LENGTH A - WATER} \]
\[ f_{\text{LCA}} = \frac{1}{2} (52.4)(3.5)^2 = 210 \text{ k} \]
\[ f_{\text{LCA}} = \frac{1}{2} (52.4)(7.5)^2 = 1.8 \text{ k} \]
\[ F_A = \frac{f_A}{3} \left( 52.0 + 1.8 + \sqrt{3.0(18)} \right) \]
\[ = 859 \text{ k} \]

\[ \text{LENGTH B} \]
\[ f = \frac{1}{2} (52.4)(33.5)^2 = 35.0k \]
\[ F_B = 35(33.5) = 1365k \]

\[ \text{LENGTH LD} \]
\[ f_{\text{LCA}} = 31.0 \text{ k} \]
\[ f_{\text{LCA}} = 1.8 \text{ k} \]
\[ F_D = \frac{f_D}{3} \left( 31.0 + 1.8 + \sqrt{3.0(18)} \right) \]
\[ = 779 \text{ k} \]

Total Hydrostatic Pressure Load
\[ F_{L.2} = 859k + 1365k + 779k = 3003k \]

Total Driveline Force (2)
\[ F_{L.1} = 3003k + 182k = 3185k \]
SAFETY FACTORS AGAINST SLIDING

\[ R_2 = \sqrt{\tan(\alpha + \phi)} \cdot \frac{cA_2}{\cos \alpha (1 - \tan \phi \tan \alpha)} \]

For \( \alpha = 0 \) (HORIZONTAL BASE)

\[ R_2 = \sqrt{\tan \phi} \cdot cA_2 \]

For \( \tan \phi = 0.7 \)

\[ A_2 = 2033 \]

\[ R_2 = 0.7 \sqrt{2033} \]

For the load of the porous top \( 0.7V = 1435 \) k

For a sunder top SF = 3

\[ R_2 = 2153 (3) k = 1435k + 2033c \]

\( \Delta \theta \) \( C = 2.47 \text{ ksf} = 17 \text{ psi} \)

For the soil level 5' above the porous top, SF = 3

\[ R_L = 3 (3185) k = 1257k + 2033c \]

\[ = 4096 \text{ ksf} = 28 \text{ psi} \]

For SF = 3, 0 cohesive bond of between 17 ksf to 28 psi must be maintained over the entire base. Since this is not an unreasonable value, the structure can be assumed to be safe against sliding failure.
FOR THE SDF = ½ DF & EL 927.7

THEN:

\[ C = 17,000 \text{ psi} \leq \text{EL 926.5} \]
\[ C = 28,000 \text{ psi} \leq \text{EL 931.5} \]

THEN:

\[ C = 19,600 \text{ psi} \leq 20,000 \text{ psi} \leq \text{FAI} \]
\[ \Delta w SF = 3 \]
**ACKENHEIL & ASSOCIATES**

GEO Systems, Inc.
1000 Banksville Road
PITTSBURGH, PA 15216
(412) 531-7111

Job: COLONIAL Dam No. 3
Job No.: 79-53-4
Subject: Factor of Safety Against Over-turning

Made By: Sam
Date: 1/28/80
Checked: PA
Date: 1/4/80

**CASE II, EL. 929.0**

- $h = 5.0'$
- $w = 2.0'$
- $T_r = 62.4$ pce.
- $G' = 145$ pce.

**CASE I, EL. 924.0**

- $x_c = 45$ pce.
- $x_0 = 140$ pce.
- $G = 45^\circ$
- $C = 0$ pce.

---

$$K_a = \frac{1 - \sin \theta}{1 + \sin \theta}$$

- $\theta = 3^\circ 
  K_a = \frac{1 - \sin 3^\circ}{1 + \sin 3^\circ} = 0.90$

- $\theta = 45^\circ 
  K_a = \frac{1 - \sin 45^\circ}{1 + \sin 45^\circ} = 0.172
  K_c = \frac{1}{0.172} = 5.828$
**Weight of Gravity Section — Magnitude & Location**

\[ A_1 = 2.0' \times 2.5' = 5 \text{ ft}^2 \times 17' \]
\[ A_2 = 5.0' \times 24' = 120 \text{ ft}^2 \times 16.5' \]
\[ A_3 = \frac{1}{2} \times 2' \times 24' = 144 \text{ ft}^2 \times 10' \]
\[ A_4 = 20' \times 2' = 40 \text{ ft}^2 \times 10' \]

<table>
<thead>
<tr>
<th>Section</th>
<th>( A_1 ) (\text{ft}^2)</th>
<th>( x ) (\text{ft})</th>
<th>( W \times A_1 ) (\text{lb})</th>
<th>( Wr ) (\text{ft-lb})</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>17.0</td>
<td>72.5</td>
<td>1.12,325</td>
</tr>
<tr>
<td>2</td>
<td>120</td>
<td>14.5</td>
<td>17,400</td>
<td>287,100</td>
</tr>
<tr>
<td>3</td>
<td>144</td>
<td>10.0</td>
<td>20,800</td>
<td>208,800</td>
</tr>
<tr>
<td>4</td>
<td>40</td>
<td>10.0</td>
<td>5,800</td>
<td>58,000</td>
</tr>
</tbody>
</table>

\[ \sum W = 4,4805', \quad \sum W r = 5622.5' \text{ ft} \]

\[ W = \sum W = 4,4805', \quad \sum W r = 5622.5' \text{ ft} \]

\[ x = \sum \frac{W r}{W} = \frac{5622.5'}{4,4805'} = 12.64 \text{ ft} \text{ to right} \]

\[ \text{of pt. Q} \]
CASE II  

WATER SURFACE AT CREST

\[ P = h_y \gamma_w \]
\[ P_1 = 6 \times 62.4 \text{pcf} = 374.4 \text{psf} \]

\[ R = (x + h_y) \gamma_w + \frac{h_y^2 \gamma_s}{K_0} \]
\[ R = (6 + 18) \times 62.4 \text{pcf} + 18 \times (80 - 62.4 \text{pcf}) (0.90) \]
\[ R = 1497.6 \text{psf} + 285.1 \text{psf} = 1782.7 \text{psf} \]

\[ R_2 = (h_x + h_y) \gamma_w + \frac{h_x \gamma_s}{K_a} + \frac{h_y \gamma_s}{K_a} \]
\[ R_2 = 26 \times 62.4 \text{pcf} + 285.1 \text{psf} + 2 \times (140 - 62.4 \text{pcf}) (0.172) \]
\[ R_2 = 1622.4 \text{psf} + 285.1 \text{psf} = 1934.2 \text{psf} \]

\[ F_2 = \frac{h_x}{2} P_2 = 3 \times 374.4 \text{psf} = 1123 \text{"/ft. or section length} \]
\[ F_2 = h_x P_1 = 18 \times 374.4 \text{psf} = 6739 \text{"/ft.} \]
\[ F_2 = \frac{h_x}{2} (P_2 - P_1) = 9 (1782.7 - 374.4) = 12675 \text{"/ft.} \]
\[ F_2 = h_x P_2 = 2 \times 1782.7 \text{psf} = 3565 \text{"/ft.} \]
\[ F_2 = h_x (P_2 - P_1) = 1 (1934.2 - 1782.7) \text{psf} = 152 \text{"/ft.} \]
CASE II - WATER SURFACE 5 FT. ABOVE CREST

\[ P'_1 = P_1 + 5 \times 62.4 \text{ lb/ft}^2 = 686.4 \text{ lb/ft}^2 \]

\[ F'_1 = F_1 + 3 \text{ in} = 2094.7 \text{ lb/ft} \]

\[ P'_2 = P_2 + 3 \text{ in} = 2246.2 \text{ lb/ft} \]

\[ F'_2 = \frac{1}{2} \sqrt{P_1} \times 5.5 = 686.4 \text{ lb/ft} \times \frac{3775}{4} \text{ ft. of section length} \]

\[ F'_3 = \frac{1}{2} \sqrt{P_2} = 12355 \text{ lb/ft} \]

\[ F'_4 = \frac{F_3}{2} = 12675 \text{ lb/ft} \]

\[ F'_5 = F_5 = 2 \times 2094.7 \text{ lb/ft} = 4189 \text{ lb/ft} \]

\[ F'_6 = \frac{F_5}{2} = 1527 \text{ lb/ft} \]

\[ W = \left[ h_1 \gamma_w + h_2 \gamma_s \right] \times 1 \text{ ft.} \]

\[ W = 11 \times 62.4 \text{ lb/ft}^2 + 18 \times 80 \text{ lb/ft}^2 = 1814 \text{ lb/ft} \text{ (CASE I)} \]

\[ X = 20 \text{ ft} - \left( \frac{10}{2} \right) = 19.5 \text{ ft} \]

\[ W = \left[ h_1 \gamma_w + h_2 \gamma_s \right] \times 1 \text{ ft.} \text{ (CASE II)} \]

PASSIVE RESISTANCE FORCE

\[ F_p = \frac{1}{2} \gamma_t h^2 k_p \]

\[ F_p = \frac{1}{2} \left( 140 \text{ lb/ft}^2 \right) \left( 2 \text{ ft} \right)^2 \left( 5.2 \text{ lb/ft}^2 \right) \]

\[ F_p = 1632 \text{ lb/ft} \]

\[ Y_p = \frac{h}{2} h = 2/3 \text{ ft.} \]
F.S. OVERTURNING = Mr / OVERTURNING MOMENT

\[ M_r = x_e W_e + y_p F_p + x_w I_w \]

\[ M_r = \Sigma WZ + y_p F_p + x_w I_w \]

\[ M_r = 561.25 \, \text{lb} \, \text{ft} + \left( \frac{3}{16} \times 16 \, \text{lb} \, \text{ft} \right) + (19.5 \times 18.3) \]

\[ M_r = 602.686 \, \text{lb} \, \text{ft} \quad \text{(602.7 m-k)} \quad \text{for CASE I} \]

\[ M_r' = \Sigma WZ + y_p F_p + x_w I_w \]

\[ M_r' = 561.25 \, \text{lb} \, \text{ft} + 1088 \, \text{lb} \, \text{ft} + (19.5 \times 21.26) \]

\[ M_r' = 608.670 \, \text{lb} \, \text{ft} \quad \text{(608.7 m-k)} \quad \text{for CASE II} \]

\[ M_o = \Sigma F'y + U' \times y \]

<table>
<thead>
<tr>
<th>( y )</th>
<th>( F )</th>
<th>( F'y )</th>
<th>( \Sigma F'y )</th>
<th>( y' )</th>
<th>( F' )</th>
<th>( F'y' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22.12</td>
<td>244.706</td>
<td>1223.56</td>
<td>1</td>
<td>23.67</td>
<td>377.51</td>
</tr>
<tr>
<td>2</td>
<td>11.67</td>
<td>74.129</td>
<td>1235.50</td>
<td>2</td>
<td>12.55</td>
<td>135.90</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>101.400</td>
<td>1267.5</td>
<td>3</td>
<td>10.40</td>
<td>188.9</td>
</tr>
<tr>
<td>4</td>
<td>3.595</td>
<td>3.595</td>
<td>18.99</td>
<td>4</td>
<td>4.189</td>
<td>8.758</td>
</tr>
<tr>
<td>5</td>
<td>2/3</td>
<td>152</td>
<td>101</td>
<td>5</td>
<td>2/3</td>
<td>152</td>
</tr>
<tr>
<td></td>
<td>202.91</td>
<td>330.949</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\[ \Sigma F'y = 202.91 \, \text{lb} \, \text{in} \]

\[ \Sigma F'y' = 330.949 \, \text{lb} \, \text{in} \]
\[ U = \frac{1}{2} \times 62.4 \times 20 \times 26 \text{''} = 16224 \text{''/ft} \]
\[ U'' = \frac{1}{2} \times 62.4 \times 20 \times 31 \text{''} = 19344 \text{''/ft} \]
\[ x_v = \frac{2}{3} \times 20 = 13.33 \text{ ft} \]
\[ U_x = \frac{20}{3} \times 16224 \text{''} = 216,320 \text{''} \]
\[ U_x = \frac{20}{3} \times 19344 \text{''} = 257,920 \text{''} \]

AT 100% UPLIFT

CASE I

\[ F_S = \frac{M_2}{M_0} \]
\[ F_S = \frac{608,686}{203,901 + 216,320} = 1.43 \]

CASE II (h = 5')

\[ F_S = \frac{M_2'}{M_0'} \]
\[ F_S = \frac{608670}{330,949 + 257,920} = 1.03 \]
**ACKENHEIL & ASSOCIATES**
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Job COLONIAL DAM No. 3
Job No. 79/58

Subject: RESULTANT FORCE LOCATION

Made By: SGM Date: 5/28/80
Checked: FRB Date: 6/10/80

### ANALYSIS No. 2

#### CASE I
\( h = 0' \) (Dam Crest)

#### CASE II
\( h = 5' \) (5 ft. above crest)

<table>
<thead>
<tr>
<th>CASE</th>
<th>( y )</th>
<th>( F )</th>
<th>( F'y )</th>
<th>( y' )</th>
<th>( F' )</th>
<th>( F'y' )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>22</td>
<td>1123</td>
<td>24,706</td>
<td>1</td>
<td>23.67</td>
<td>3,775</td>
</tr>
<tr>
<td>2</td>
<td>11</td>
<td>6739</td>
<td>74,129</td>
<td>2</td>
<td>11</td>
<td>12,555</td>
</tr>
<tr>
<td>3</td>
<td>8</td>
<td>12675</td>
<td>101,400</td>
<td>3</td>
<td>8</td>
<td>12,675</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>3565</td>
<td>3565</td>
<td>4</td>
<td>1</td>
<td>4189</td>
</tr>
<tr>
<td>5</td>
<td>2/3</td>
<td>152</td>
<td>101</td>
<td>5</td>
<td>2/3</td>
<td>152</td>
</tr>
<tr>
<td>6</td>
<td>2/3</td>
<td>1032</td>
<td>-1088</td>
<td>6</td>
<td>2/3</td>
<td>-1632</td>
</tr>
</tbody>
</table>

\[ \sum F = \sum F'y = \Sigma F' = \Sigma F'y' \]

\[ R' = 22,622', 202,813' \]

\[ y' = 22,622', 202,813' \]

\[ \sum F' = \sum F'y' = \Sigma F = \Sigma F' = \Sigma F'y' \]

\[ y = 8.97 \text{ ft. above base} \]

\[ y = 10.47 \text{ ft. above base} \]
\[ U = 16.32 \text{ kN} \]
\[ U' = 19.34 \text{ kN} \]
\[ W_L = 44.8 \text{ kN} \]
\[ x_c = 12.6 \text{ m} \]
\[ R_f' = 31.5 \text{ kN} \]
\[ R_f = 22.6 \text{ kN} \]
CASE II
\[ ZF_V = 0 \quad R_v = u_c - u + u_i \]
\[ R_v = 44.8^k - 19.3^k + 2.1^k = 27.6^k \]
\[ \bar{x} = 608.9^k - 588.9^k \]
\[ 27.6^k \]
\[ \bar{x} = 0.72 \text{ ft} \]
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---

Both  $R$'s are outside middle third of base $b$.

**EXAMINE FOR TOE PRESSURES.**

$$ P = \frac{2Rv}{3x} $$

**CASE I**  \[ Rv = 30.4' \],  \[ x = 6.0' \]

$$ P_{\text{max}} = \frac{2 \times 30.4'}{3 \times 6.0'} = 3.4 \text{ ksf} $$

$$ P_{\text{max}} = \frac{5.4 \times 1000}{144 \text{ in}^2/\text{ft}^2} = 23.6 \text{ psi} $$

**CASE II**  \[ Rv = 27.6' \],  \[ x = 0.72' \]

$$ P_{\text{max}} = \frac{2 \times 27.6'}{3 \times 0.72'} = 25.6 \text{ ksf} $$

$$ P_{\text{max}} = \frac{25.6 \times 1000}{144} = 177.8 \text{ psi} $$

---

**SDF POOL LEVEL**

For pool @ El. 927.7 (SDF = 1/2 PMF)

$$ \gamma'' = \left[ \frac{1934.4 - 1622.4(3.7) + 1622.4}{5} \right] / 1000 \text{ sf} $$

$$ \gamma'' = 18.5' $$
\[ U'' = 18.5^\prime + 13.33^\prime \]
\[ U'' = 246.6'' \]

\[ M_R'' = \Sigma w_i + y_p F_p + x_i \left[ \frac{\left( \omega_i - \omega_1 \right) 3.7 + \omega_i}{5} \right] \]
\[ M_R'' = 567.2''^\prime + 19.5'' \left[ \frac{\left( 2.1''\cdot 1.8'' \right) 3.7 + 1.8''}{5} \right] \]
\[ M_R'' = 600.6''^\prime \]

\[ M_0'' = \Sigma F_y + \frac{3.7}{5} (2F_y' - 2F_y) \]
\[ M_0'' = 203.9''^\prime + 3.7 \left( \frac{330.9''^\prime - 203.9''^\prime}{5} \right) \]
\[ M_0'' = 297.9''^\prime \]

\[ F.S._{SOF} = \frac{M_R''}{M_0'' + U''} \]
\[ F.S._{SOF} = \frac{600.6''^\prime}{297.9''^\prime + 246.6''^\prime} = 1.11 \]

\[ R_V'' = w_o - U'' + \omega_i \]
\[ R_V'' = 448^\prime - 18.5^\prime + 2.0^\prime = 28.3^\prime \]
\[ R'' = M_R'' - M_0'' - U''^\prime \]
\[ R'' = 606.6''^\prime - 207.9''^\prime - 246.6''^\prime = 22.2''^\prime \]
\[ D_{100} = \frac{2 \cdot 28.3''^\prime}{3 \cdot 2.2''^\prime} = 8.6''\text{ref} \times 5.97''\text{per} = 4.3''\text{per} \]
SEISMIC ANALYSIS:

1) CENTER OF GRAVITY SECTION

\[
\begin{align*}
\Delta_1 &= 2.5' \times 2' = 5 \text{ m}^2 \\
\Delta_2 &= 24' \times 5' = 120 \text{ m}^2 \\
\Delta_3 &= 1/2 \times 24' \times 12' = 144 \text{ m}^2 \\
\Delta_4 &= 2' \times 20' = 40 \text{ m}^2
\end{align*}
\]

<table>
<thead>
<tr>
<th>( \Delta_1 ) (m²)</th>
<th>( \Delta_2 ) (m²)</th>
<th>( \Delta_3 ) (m²)</th>
<th>( \Delta_4 ) (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>120</td>
<td>144</td>
<td>40</td>
</tr>
</tbody>
</table>

\( \Sigma \Delta_1 = 309 \text{ m}^2 \)

\( \Sigma \Delta_2 = 3410 \text{ m}^3 \)

\( \Sigma \Delta_3 = 3246.25 \text{ m}^3 \)

\( \bar{X} = \frac{\Sigma \Delta_2}{\Sigma \Delta} = \frac{3910}{309} = 12.65 \text{ m} \)

\( \bar{Y} = \frac{\Sigma \Delta_3}{\Sigma \Delta} = \frac{3246.25}{309} = 10.67 \text{ m} \)
VALUES FROM "RESULTANT FORCE LOCATION"

\[ \theta' = 19.3^\circ \]
\[ y' = 13.3 \text{ ft} \]

\[ R_p' = 31.5^\circ \]
\[ R_p = 31.5^\circ \]
\[ y = 10.47 \text{ ft} \]
\[ x = 8.97 \text{ ft} \]

\[ h = 5' \text{ (CASE III)} \]
\[ h = 0' \text{ (CASE II)} \]

\[ F_{o} = 0.025 W \]

The factor, 0.025, is from the recommended guidelines for safety evaluation of dams (Zone 1)

\[ F_{o} = 0.025 \times 44.8^\circ \]

\[ F_{o} = 1.12^\circ \]
\[ FS = \frac{M_2}{M_0} \]

\[ M_a = 602.69 \text{ kN} \]
\[ M_a' = 608.77 \text{ kN} \]

**100\% UPLIFT**

**CASE I**

\[ m_{b1} = V \xi x_v + R \xi y_v + F_{d0} \gamma \]

\[ m_{b1} = 16.22 \times 13.33 \text{ kN} + 22.6 \times 8.97 \text{ kN} + 1.12 \times 10.67 \text{ kN} \]

\[ M_a = 430.89 \]

\[ FS_1 = \frac{M_e}{M_{b1}} = \frac{602.69}{430.89} = 1.40 \]

**CASE II**

\[ m_{b2} = V \xi x_v + R \xi y_v' + F_{d0} \gamma \]

\[ m_{b2} = 19.34 \times 13.33 \text{ kN} + 31.5 \times 10.47 \text{ kN} + 1.12 \times 10.67 \text{ kN} \]

\[ M_{b2} = 599.56 \text{ kN} \]

\[ FS_2 = \frac{M_e}{M_{b2}} = \frac{608.77}{599.56} = 1.02 \]
SDF:

\[ M_r^2 = 606.6 \text{ ft}^2 \]

\[ m_o = M_r^2 + U^x + F_0 \cdot x \]

\[ m_o = 297.9 \text{ ft}^2 + 246.6 \text{ ft}^2 + (1.12 \cdot 10.67') \]

\[ m_o = 556.5 \text{ ft}^2 \]

\[ F.S. = \frac{M_r^2}{m_o} = \frac{606.6}{556.5} = 1.09 \]

**Toe Pressures**

For \( F.S. = 1.0 \):

\[ R = \frac{M_r^2 - m_o}{R_v} \]

**Case 1**: \( \Sigma F_y = 0 \)

\[ R_v = W_0 - U^x + w \]

\[ R_v = 148^2 - 16.2^2 + 1.8^2 = 30.4^2 \]

\[ R = \frac{602.69 - 330.89}{30.4^2} = 5.65 \text{ ft} \]

\( R \) is outside middle third (6.67' - 13.33')

\[ P = \frac{2R_v}{3R} = \frac{2 \cdot 30.4^2}{3 \cdot 5.65} \]

\[ 25 \text{ psi} = 1.8 \text{ tons} \]
CASE II:

$$F_v = 0 \quad R_v = w_c - U' + w_i$$

$$R_v = 44.8 - 19.3 + 2.1 = 27.6$$

$$R = \frac{608.77 - 599.56}{27.6} = 0.33 \text{ ft.}$$

If outside middle third:

$$\rho = \frac{2R_v}{3}$$

$$\rho = \frac{2 \times 27.6}{3 \times 0.33} = 55.8 \text{ ksf} = 388 \text{ psf} = 27.9 \text{ TFS}$$

$$F_v = 0 \quad R'' = w_c - U'' + w_i$$

$$R'' = 44.8 - 18.5 + 2.0 = 28.3$$

$$R = \frac{606.6}{28.3} = 21.77 \text{ ft.}$$

If outside middle third:

$$\rho = \frac{2R_v}{3}$$

$$\rho = \frac{2 \times 28.3}{3 \times 1.77} = 10.7 \text{ ksf} = 74 \text{ psf} = 55.4 \text{ TFS}$$

Sheets 1 of 5