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NATIONAL DAM SAFETY PROGRAM. TOWNES DAM (VA14102), DAN RIVER, P--ETC(U)
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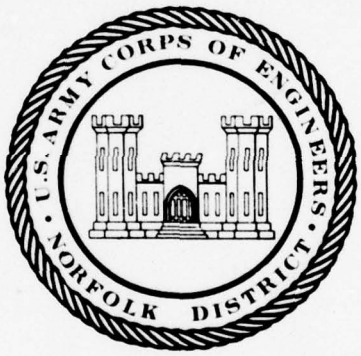
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Name Of Dam: TOWNES DAM
Location: PATRICK COUNTY
Inventory Number: VA 14102

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM



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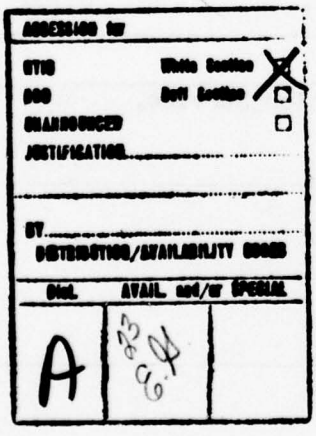
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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspections. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam and appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.



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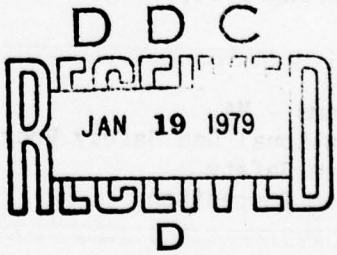


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PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Townes Dam
State: Virginia
County: Patrick
Coordinates: 3641.2 3023.8
Stream: Dan River
Date of Inspection: 27 July 1978

Townes Dam is an unreinforced concrete arch dam, 133 feet high, and 575 feet long at the crest, sited in a deep gorge at the head waters of the Dan River. Design calculations are not available. Searches by both the designer and the owner were unsuccessful in locating any design calculations. The Geology report and construction drawings indicating concrete strength and foundation grouting programs were reviewed and found to be adequate.

The dam is in good condition, however, seepage has occurred through an estimated 30 to 40% of all construction and monolith joints. The monolith joint located approximately 125 feet from the right abutment stairs appears to be open 1/8 inch. This condition occurs at the contact surface of the dam and abutment extending upward approximately 30 feet. No seepage was observed. Based on visual observation, this condition does not warrant serious concern at this time, however, further investigation is required.

The probable maximum flood (PMF) would overtop the dam by three feet reaching elevation 2233.5 with the flashboards removed. This exceeds the flood level of 2225 indicated on design drawings. This is cause for concern since no structural analysis is available. Analysis of dam stability for this dam is beyond the scope of a Phase I report.

Seepage was observed on the left abutment approximately 40 feet below the top of the dam within 5 feet of the dam/abutment contact. The seepage on the left abutment is not a problem at this time, however, volume of water should be monitored. (See Section 7)

Since the spillway passes less than one-half the PMF and the stability is unknown, the spillway must be considered seriously inadequate. If structural analysis proves the dam is stable under all loading conditions, the spillway may be considered inadequate in lieu of seriously inadequate. A dam with a seriously inadequate spillway is considered to be unsafe. Considering Townes Dam to be unsafe for overtopping conditions is a conservative assumption since essentially the stability of the dam is unknown. However, this approach is consistent with good engineering judgement since the lives of many people are affected. This should be considered an interim status until the stability of the dam can be determined by structural analysis.



TOWNES DAM
(PHOTO PRIOR TO 1940)

It is recommended that the owner, at his expense, take the following action:

a. Establish an effective warning system to provide for evacuation of the downstream flood plain if necessary. The warning system should be operable until it can be shown by analysis that Townes Dam and Talbott Dam upstream are stable with appropriate safety factors included under critical loading conditions. It must be considered that the Townes Dam is threatened whenever the reservoir level exceeds elevation 2225, or the reservoir level at Talbott reservoir upstream exceeds 2538, until analysis shows otherwise.

b. Engage a professional engineer to perform a structural analysis of the dam under all loading conditions including the evaluation of the open monolith joint at the right abutment. Structural analysis and evaluation should be completed within 6 months.

c. Engage the services of a professional consultant to complete a report of remedial mitigating measures which will correct the seriously inadequate spillway and have an agreement with the Commonwealth of Virginia to a reasonable time frame in which all remedial measures will be complete. This work shall be completed within six months from the date of notification to the Governor.

d. In addition the following recommendations described in Section 7 should be performed: (1) remove flashboards, (2) investigate stop log installation, (3) Inspect trash rack regularly, (4) verify maintenance of pipeline butterfly valve, (5) relocate transformer, fuse cutouts, and surge arrestors, (6) repair transformer oil leakage, (7) test transformer oil, (8) install emergency lighting, (9) clear reservoir debris.

Submitted By:
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SEP 27 1978

TOWNES DAM

SECTION 1 - PROJECT INFORMATION

1.1 General

1.1.1 Authority: Public Law 92-367, 8 Aug 72 authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspections of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose of the Phase I inspections is to identify expeditiously those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Townes Dam, originally named Point Lookout Dam, is an unreinforced concrete arch dam 133 feet high and 575 feet long with a radius of curvature to the upstream face of 240 feet. The top of the dam is 8'-0" wide and consists of a continuous walkway set at elevation 2227 MSL with reinforced concrete parapet walls 1'-0" thick on either side of the walkway extending to elevation 2230.5. Access to the walkway is provided by concrete stairs on the right abutment starting at elevation 2253. The walkway passes over an ungated spillway 120 feet long with a crest elevation of 2218. Removable timber flashboards are in place and raise the spillway crest 3 feet to elevation 2221. Flow passing over the spillway strikes the downstream face of the dam and washes into a stilling basin where curved wingwalls, symetrically placed on either side of the basin, divert the flow into a channel approximately 50 feet wide. The intake for power generation consists of a 120-inch diameter transition into a 70-inch pipeline passing through the dam near its base with the invert at elevation 2146. The intake is protected by a trash screen which can be raised or lowered by a motorized hoist. The pipeline carries water to a hydro-electric generating station located approximately 3 miles downstream from the dam. Flow in the pipeline to the three hydro-electric turbines is regulated by Dow type needle valves. In addition, a butterfly valve in the 70-inch pipeline, housed in a concrete valve house built on the downstream face of the dam at the right abutment, can control flow reaching the turbines. The butterfly valve can be operated electrically from the powerhouse or by an override at the dam. The butterfly valve is normally left in the open position (except when major repairs are made to the pipeline) and discharges are regulated at the powerhouse. A valved 12-inch line exiting the valve house allows water to bypass the 70-inch butterfly valve and be released into the stilling basin at Townes Dam. A small

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storage room, built into the dam on the right abutment, houses a private telephone for communication direct with the powerhouse and Talbott Dam upstream. Talbott and Townes Dams are both part of the Pinnacles Hydro-electric Project. Talbott Dam provides most of the storage for power generation.

1.2.2 Location: Townes Dam is located on the headwaters of the Dan River approximately 4 miles south of the town of Meadows of Dan and the point where Virginia route 58 intersects with the Blue Ridge Parkway. Townes Dam is located 5½ miles downstream from Talbott Dam.

1.2.3 Size Classification: The dam is classified as a large dam based on its height of 133 feet.

1.2.4 Hazard Classification: The dam has been given a high hazard classification in accordance with guidelines contained in Section 2.1.2 of "Recommended Guidelines for Safety Inspection of Dams" published by the Office, Chief of Engineers. The high hazard rating is required since if the dam failed more than a few lives would be lost. However, the hazard classification used to catagorize dams is a function of location only and has nothing to do with its stability or probability of failure.

1.2.5 Ownership: City of Danville, Va.

1.2.6 Purpose: The dam is used to generate hydro-electric power for the city of Danville.

1.2.7 Design and Construction History: The dam was designed by Charles T. Main, Inc. Engineers of Boston, Mass. Drawings are dated Jan 21, 1937. Construction was performed by Ligon and Ligon-Sammons Robertson. According to available drawings, foundation grouting was begun in June and completed in September of 1937. Concrete placement for the dam was completed in January 1938 followed by grouting of vertical joints through a system of keyways and grout slots built into the dam. The temporary sluiceway, which allowed river diversion during construction was, closed 1 April 1938.

1.2.8 Normal Operational Procedures: Operation of the project is regulated. The flow is monitored and controlled by the Pinnacles Hydro-Electric Power Plant for generation of electricity. At high pool levels, water is automatically passed over the spillway. Talbott Dam, located upstream, provides additional storage for power generation. Water from Talbott Dam is released into Townes Reservoir as required to maintain head for power generation.

1.3 Pertinent data:

1.3.1 Drainage Areas: The dam controls a total drainage area of 32.9 square miles. Of this total, 20.2 square miles is also controlled by the upstream Talbott Dam, leaving a total of 12.7 square miles attributed to the local drainage area.

1.3.2 Discharge at Dam Site:

Maximum known flood - 4000 cfs in August 1940 (estimated)

Aqueduct (3 - 12 inch outlets located at Pinnacles Hydro Station)
Pool level at spillway crest without flashboards - 188 cfs (avg.)

Spillway
Pool level at top of parapet wall with flashboards - 7500 cfs
Pool level at top of parapet wall without flashboards - 13,100 cfs

1.3.3 Dam and Reservoir Data: Pertinent data on the dam and reservoir are shown in the following table:

TABLE 1.1 DAM AND RESERVOIR DATA

Item	Elevation ft, msl	RESERVOIR CAPACITY Area Acres	Acre Feet	Watershed Inches	Length Miles
Top of Parapet Wall	2230.5	46	1830	1.04	1.8
Top of Walkway	2227.0	43	1650	0.95	1.7
Flood Level 1/	2225.0	41	1580	0.90	1.6
Top of Flashboards	2221.0	37	1400	0.80	1.6
Spillway Crest	2218.0	36	1325	0.76	1.6
Normal Pool	2212.0-2217.0	-	-	-	-
Streambed at Center-Line of Dam	2103.0	-	-	-	-

1/Elevation Noted on original drawings.

SECTION 2 - ENGINEERING DATA

2.1 Design:

2.1.1 Dam: The dam was designed by Charles T. Main, Inc. of Boston, Mass. Design drawings are available in the City of Danville Electric Department, Danville, Va. 24520, however, design calculations are lost. Specifications and Construction Regulations for Point Lookout Dam (Townes Dam) are also available from the City of Danville. Design drawings include core boring logs for preliminary studies of several possible sites for the Townes Dam as well as rock profiles at the present site of Townes Dam developed from core borings.

2.1.2 Geologic Investigation: An extensive geologic and foundation investigation was conducted for the dam by the design engineers, Charles T. Main, Inc. The investigations were directed by the Consulting Geologist, Wilbur A Nelson under contract to the design engineers. Geology reports are on file in the City of Danville Electric Dept. As part of the preliminary studies, eight separate sites were investigated. The investigations at each site included detailed geologic mapping and diamond core drilling. A total of 50 core borings were drilled with most concentrated at sites where the gorge was narrow. The final site selected was No. 17 where the gorge was narrowest. Site No. 16, 30 feet downstream, was the most geologically advantageous according to the consulting geologist. The selected site was, however, approved by the consulting geologist provided recommended foundation treatments were followed. In addition to the geologic mapping and drilling, the investigations also included several test pits and petrographic analysis of the foundation rocks.

A detailed geologic report of investigations with recommendations prepared by the consulting geologist is inclosed as Appendix IV. Geologic cross sections across the gorge at 4 of the 5 sites are shown on Plates 8 and 9, Appendix I.

The dam is located in the Blue Ridge Physiographic Province of Virginia and is underlain by metamorphic rocks of the Precambrian Age Lynchburg Formation. These rocks include predominately quartz-mica gneiss with minor amounts of schist and quartzite. The general strike of the schistosity of these rocks is from N45° to 50° with a dip to the southeast of 50° to 80°. The structural geology of the site is fairly complex. At least three periods of folding with associated jointing have occurred in the rocks of the area. Thrust and normal faulting are common in the Blue Ridge but neither occurs in the immediate dam site.

2.2 Construction: The dam was constructed by Ligon and Ligon-Sammons, Robertson Co., Inc. with construction inspection by Charles T. Main, Inc., Engineers. The available records prepared during construction follow:

a. Dwg. 1257-185, dated May 4, 1938 Pinnacles Hydro Electric Project, Point Lookout Dam (renamed Townes Dam) Pouring and Testing Record by Charles T. Main Inc. This drawing is a record of concrete placement and includes a tabulation of concrete test strengths. See Plate 5, Appendix I.

b. Dwg. 1257-186 and 1257-186A dated May 7, 1938 Pinnacles Hydro Electric Project, Point Lookout Dam, Grout Record. These drawings detail the foundation grouting program. See Plate 6 and 7, Appendix I.

c. An extensive photo file of construction progress is available.

The above construction information is on file in the City of Danville Electric Department.

The condition of the foundation rocks varied from abutment to abutment. Based on the core borings, overburden thicknesses ranged from 6 to 14 feet on the right abutment and 4 to 7 feet on the left abutment. Depth to unweathered rock varied from 26 to 44 feet on the right abutment and 16 to 26 feet on the left abutment. A zone of tallus from a nearby rock slide capped the upstream edge of the right abutment.

According to contract specifications inclosed in Appendix IV, excavations for the dam foundation were made to a sufficient depth to secure the foundation on sound bedrock free of open or soft seams. The excavations were made in horizontal steps at least 5 feet high as shown on Plate 5, Appendix I. After excavation of the foundation and before placement of the concrete structure, the base was thoughly cleaned of all loose materials and a ½" thick layer of slush grout was applied to the cleaned rock surface to secure a good bond with the first lift of concrete. Steel dowel bars were also grouted into the bedrock before concrete placement.

Weathered zones were also encountered within the underlying competent bedrock along narrow planes parallel to the schistosity. These weathered planes extended to the river elevation on the right abutment and were fairly continuous. Since the northeast strike of the schistosity planes cut across the dam site at approximately 45°, leakage through the weathered planes was anticipated. To stop the leakage under the dam and to provide a uniform and consolidated foundation, an extensive grouting program was completed before construction of the dam structure. The grouting was done in two

phases. The first phase included drilling 10% of the grout holes to 30 feet with a diamond core drill along the dam perimeter. After examining the cores and the grout records for these holes, the second phase grout holes were laid out and drilled to a minimum depth of 30 feet with jackhammers. Approximately 600 grout holes were drilled and grouted. The grouting procedure is outlined in the specifications in Appendix VI. Drawings showing the hole locations and grout records are shown on Plates 6 and 7, Appendix I.

2.3 Operation:

2.3.1 Underwater Inspection: In April of 1978 Logan Diving Inc. performed an underwater inspection of Townes Dam as part of a periodic general preventive maintenance program instituted by the Danville Electric Department. This report, titled "Underwater Inspection, Talbott and Townes Dams for City of Danville, Va., April 4 & 5, 1978," is included in this report as Appendix V.

2.3.2 Maintenance Logs: Daily maintenance logs are maintained for the dam and appurtenances. The City of Danville employs a reservoir patrolman who lives within walking distance of the dam.

2.3.2 Daily Logs: Daily station logs (operational logs) are maintained at the powerhouse and the following information is recorded:

a. General weather conditions.
b. Water level at Townes Reservoir.
c. Daily and monthly rainfall.
d. Power generation for the day and month. (This data can be converted to CFS released at the power station.)
e. Miscellaneous information concerning power plant operation.

2.3.3 Monitoring Program: 3/8" stainless steel eye-bolts (called deflection eye bolts on design drawings) were set adjacent to vertical monolith joints at regular intervals on the downstream face of the dam apparently to measure movement in the dam. City of Danville employees have no knowledge of any previous attempts to monitor movement in the dam and there is presently no ongoing monitoring program.

2.4 Evaluation: With the exception of original design calculations, engineering records maintained for the dam are good. Design drawings, although incomplete, do adequately describe the dam. Operational records are good.

SECTION 3 - VISUAL INSPECTION

3.1 General: Prior to the field inspection performed on 27 July 1978, the only inspection report available was for an underwater inspection performed by Logan Diving, Inc. The inspection revealed no significent defects with the exception of a broken lifting cable, which was replaced.

3.2 Findings:

3.2.1 Dam and Abutments: The results of the 27 July 1978 inspection are recorded in Appendix III. At the time of the inspection, the pool elevation was 2214.8 MSL (6.2 feet below the elevation of the spillway). Seepage was noted at the junction of the dam and wingwall on the left abutment where the seepage left red staining and calcium deposits. Seepage was also noted 5 feet from the dam contact on the left abutment approximately 40 feet below the top of the dam. Water was flowing approximately 10 gpm from the rock fill adjacent to the dam. Water was also noted flowing from the rock fill at two locations on the right abutment but because of the warmer temperature and variation in flow, it was felt the water was runoff and not seepage.

The monolith joint located approximately 125 feet from the right abutment stairs appears to be open 1/8 inch. This condition occurs at the contact surface of the dam and abutment extending upward approximately 30 feet. No seepage was observed. (See Photo 7, Appendix II)

There were calcium deposits on 30 to 40% of all monolith and construction joints, however, many of these areas were not seeping at the time of inspection.

3.2.2 Reservoir Area: Large tree limbs and sawdust were noted floating near the face of the dam. The side slopes of the reservoir were steep since the reservoir is sited in a gorge.

3.2.3 Downstream Areas: A U.S.G.S. quad sheet was used as a guide to assess the areas which might be affected by a dam failure. Approximately 25 houses are located on the floor of the gorge within 4 miles of the dam.

3.3 Evaluation:

3.3.1 Dam and Abutment: The observed open monolith joint does not warrant serious concern at this time, however, state of stress at this point should be verified by structural analysis. With the exception of some minor seepage, no defects in the dam or the abutment were noted.

3.3.2 Downstream Area: There are approximately 25 dwellings downstream of the dam. Due to the height of the dam and the steep slope of the gorge below the dam, a dam failure could destroy many dwellings.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: The Townes reservoir provides the required flow and head for the water wheel turbines in the powerhouse. Water from this reservoir flows through a combination type conduit that includes steel pipe, wood stave pipe, concrete-lined tunnels, and a steel penstock. The reservoir pool is normally maintained between elevations 2212 ft. msl and 2217 ft. msl, however, this is not always possible. Excess flow is automatically passed over the spillway.

Electrical power for the dam is supplied by a 2,300 volt, three-phase, transmission line from the Pinnacles Power plant. An oil filled distribution type transformer in the storage room provides power to the upper portion of the dam. Two oil filled distribution type transformers in the valve house provide power for operation of the valve. A backup power source is not availble. Location of the transformers violates the National Electrical Code. Emergency lighting is not available in the valve house.

4.2 Maintenance of Dam: Routine maintenance of the dam is generally not required due to the type of structure. Debris and logs are removed from the reservoir as required by a patrolman who is permanently stationed at the Townes Dam. His primary duties are to maintain and inspect the dam pipeline and operating facilities at the dam.

4.3 Maintenance of Operating Facilities: A maintenance checklist is maintained for all operating facilities as well as a list of all maintenance performed each year. Major items and frequency of inspection follow:

Check butterfly valve motor - monthly
Lubricate butterfly valve motor - monthly
Lubricate butterfly valve - annually
Inspect Transmission Line (from powerhouse to the dam) - monthly
Inspect pipelines - weekly
Inspect surge tank, penstock, control line - monthly
Inspect turbines - annually

In April 1978, as part of a contract between the City of Danville, Virginia, and Logan Diving, Inc. of Jacksonville, Florida, (Appendix V) minor underwater repairs were accomplished, and an inspection was made of the operating elements of this dam.

4.4 Warning System: No formal warning systems or evacuation plans exist.

4.5 Evaluation: The operational procedures and maintenance of Townes Dam by the City of Danville Electric Dept. appear good. Additions to the present procedures are listed in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC DESIGN

5.1 Design: There is no original hydraulic or hydrologic design data available for the Townes Dam.

5.2 Hydrologic Records: Records of reservoir pool elevations, air temperature, and precipitation are maintained at the Pinnacles Hydro-Electric Power Plant. The reservoir contents at the end of each month are published by the U. S. Geological Survey in "Water Resources for Data for Virginia". The U. S. Geological Survey has maintained flow records approximately 31 miles downstream of the Dan River near Francisco, N.C. (drainage area 124 square miles) since August 1924.

5.3 Flood Experience: The maximum pool level reported for the Townes Dam occurred 14 August 1940 at an elevation of 2226 ft, MSL, or 4.5 feet below the top of the parapet wall. The reservoir pool reportedly reached elevation 2224 during Tropical Storm Agnes in June 1972.

5.4 Flood Potential: The Probable Maximum Flood (PMF), 1/2 PMF, and one percent flood hydrographs were developed and routed through the reservoir by use of the HEC 1DB computer program (Reference 1, Appendix VII) and appropriate unit hydrograph, precipitation, and storage-outflow data. Clark's Tc and R coefficients for the local drainage area were estimated from basin characteristics and previously derived coefficients at the Francisco gage. The rainfall applied to the developed unit hydrograph was obtained from U. S. Weather Bureau Publications (References 2 and 3, Appendix VII). Losses were estimated at an inital loss of 0.80 inch and a constant loss thereafter of 0.05 inch/hour. Outflow hydrographs from Talbott reservoir were routed to Townes reservoir and combined with the local hydrographs to obtain the total inflow hydrographs to Townes reservoir.

5.5 Reservoir Regulation: Regulated releases from Talbott reservoir supplement local inflow to Townes. The Townes reservoir is regulated for the generation of electricity for the city of Danville by the Pinnacle Hydro-Electric Power Plant. The normal maximum withdrawl from Townes for power generation is 183 cfs. Under normal streamflow conditions, the reservoir pool ranges between elevation 2212 and 2217. Recent records indicate that it is not always possible to maintain the reservoir level in the normal pool range. Excess streamflows are automatically passed over the spillway or flashboards. The storage curve supplied by the owners was extended above the top of the flashboards by use of U. S. Geological Survey Quadrangle Maps. In the August 1940 flood, the flashboards did not blow out as designed. For this reason, two rating curves were developed for the spillway. One rating assumed the flashboards in place while the other assumed full conveyance for the ogee shaped spillway. A rating curve was also developed for the top of the

parapet wall. All ratings assumed that the parapet wall would withstand the pressures associated with overtopping. In routing hydrographs through the reservoir, it was assumed that the initial pool level was at the top of the flashboards for the condition with flashboards in place and at the spillway crest for the condition without flashboards.

5.6 Overtopping Potential: The probable rise of the reservoir for the two conditions with and without flashboards at both and other pertinent information on reservoir performance is shown in the following table:

TABLE 5.1 RESERVOIR PERFORMANCE

	FLOOD			
	1940	One Percent[1/]	1/2 PMF	PMF [2/]
	WITH FLASHBOARDS			
Peak Inflow, cfs	-	9,730	27,220	58,490
Peak Outflow, cfs	4,000	9,590	27,180	58,390
Peak Elevation, ft, msl	2226	2231.3	2235.2	2239.6
Spillway, 2221.0 ft, msl				
Depth of Flow, ft[3/]	4.5	4.5	4.5	4.5
Avg. Velocity, fps	7.4	15.5	18.6	22.6
Top of Parapet Wall, 2230.5 ft, msl				
Depth of Flow, ft [4/]	-	0.8	4.7	9.1
Duration, hrs	-	6.0	11.5	15.5
Avg. Velocity, fps	-	3.0	7.2	10.0
Tailwater Elevation, ft, msl	-	2109.6	2114.8	2120.4
	WITHOUT FLASHBOARDS			
Peak Inflow, cfs	-	9,790	25,230	57,400
Peak Outflow, cfs	-	9,650	25,020	57,330
Peak Elevation, ft, msl	-	2225.5	2233.5	2238.6
Spillway, 2218.0 ft, msl				
Depth of Flow, ft [3/]	-	7.5	7.5	7.5
Avg. Velocity, fps	-	10.7	18.1	20.6
Top of Parapet Wall, 2230.5 ft, msl				
Depth of Flow, ft [4/]	-	-	3.0	8.1
Duration, hrs	-	-	8.0	12.0
Avg. Velocity	-	-	5.7	9.4
Tailwater Elevation, ft, msl	-	2109.7	2114.3	2120.3

[1/] The One Percent Exceedence Frequency Flood has one chance in 100 of being exceeded in any given year.

[2/] The Probable Maximum Flood is an estimate of flood discharges that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

[3/] Depth of flow on spillway is limited due to walkway.

[4/] Difference between the maximum reservoir elevation and the top of the parapet wall.

5.7 Reservoir Emptying Potential: A 70-inch aqueduct at elevation 2146.0 ft, msl, which reduces to three 12-inch cast iron pipelines at the power plant is available for dewatering the reservoir. Assuming the power plant is able to operate at 100% capacity during drawdown, it would take approximately 4 days to lower the reservoir pool from the top of the flashboards to elevation 2146.0 ft, msl. There are no methods available for lowering the reservoir pool below this elevation.

5.8 Evaluation: The probability of the flashboards being blown out during flood conditions is uncertain; therefore, prime consideration was given to conditions of overtopping with the flashboards in place. Under this condition, the reservoir pool rose to within 4.5 feet of the top of the parapet wall in the August 1940 flood. With the flashboards in place, the parapet wall would be overtopped by 0.8 feet in the One Percent Flood, 4.7 feet in the 1/2 PMF, and 9.1 feet in the PMF for durations of 6.0, 11.5, and 15.5 hours respectively. The parapet wall would be just overtopped by 20% of the PMF. Conditions would be slightly less severe if the flashboards were not in place in that 35% of the PMF would just overtop the parapet wall with overtopping by other floods shown in Table 5.1.1. The spillway is considered seriously inadequate since it will not pass one-half of the PMF. If structural calculations indicate the dam is stable, the spillway should be considered inadequate but not seriously inadequate.

Conclusions pertain to present day conditions and the effect of furture development on the hydrology has not been considered.

SECTION 6 - STRUCTURAL STABILITY

6.1 Dam Foundation: Townes Dam is founded on competent metamorphic bedrock along the river valley and both abutments. Foundation treatments including contact and consolidation grouting were accomplished under the entire dam structure. Foundation drains were determined to be unnecessary and therefore not constructed. For a detailed description of site geology and foundation conditions, see Section 2.

6.2 Stability: Structural calculations are not available for the dam and, therefore, the stability cannot be checked. Analysis of the dam's stability and state of stress is beyond the scope of this report.

6.3 Evaluation:

6.3.1 Structural: Visual observations did not reveal any problems which indicate instability. The slightly open monolith joint noted in the field observations (125 feet from the right abutment stairs on the downstream face of the dam extending approximately 30 feet upward from the dam abutment contact) cannot be properly assessed without structural analysis. It does not appear to warrant serious concern at this time, however, this condition should be checked when structural analysis of this dam is performed.

6.3.2 Foundation: Dam foundations are usually evaluated on the basis of potential settlement, sliding and seepage. Settlement of the dam is not a problem because the foundation is competent bedrock. Abutment stability against sliding failure is critical in concrete arch dams since the abutments are absorbing most of the loads. In hard, competent metamorphic rocks like the gneiss comprising both abutments, sliding is only critical where low angle weathered schistosity planes or shear zones occur dipping in either an upstream or downstream direction away from the abutments. No shear zones were mentioned in the geology report and although weathered schistosity planes do occur dipping downstream, the magnitude of the dip is too great (60° - 80°) for sliding to occur. The potential for leakage did exist within the foundation rock, therefore, an extensive grouting program was completed under the entire dam structure. The grouting appeared to be effective as very little seepage was observed during the Corps visual inspection. The minor seepage noted is not presently affecting the integrity of the dam, however, these areas should be monitored to determine increased flows or piping of foundation materials. Based on the review of the site geology foundation conditions and treatments and on the the visual inspection, the dam's foundation appeared stable.

SECTION 7 - ASSESSMENT RECOMMENDATION REMEDIAL MEASURES

7.1 Assessment:

7.1.1 Safety: The Townes Dam, as observed 27 July 1978, appears sound without indication of structural instability or unsafe operation. The spillway will pass only 20% of the PMF without overtopping the parapet (flashboards in) and it will not pass the 100 year flood without exceeding the flood level, elevation 2225, indicated on the drawings. This is cause for concern since no stability analysis is availble and the dam may have only been structurally designed for the flood level shown on the drawings. With the flashboards removed, the spillway will pass 35% of the PMF. The spillway is therefore adjudged as seriously inadequate and the dam is assessed as unsafe, non-emergency.

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

Talbott Dam, upstream from Townes Dam, also passes only 35% of the PMF (flashboards removed) and is similar in construction to Townes Dam. See Phase I Inspection Report, National Dam Safety Program, Talbott Dam. The structrual stability of Talbott Dam must also be verified immediately since a failure of Talbott Dam would probably cause Townes Dam to fail.

7.1.2 Adequacy of Information: With the exception of structural analysis information on the dam is excellent.

7.1.3 Urgency: Preparation of structural analysis should begin immediately.

7.1.4 Necessity of Phase II: A Phase II inspection is not considered necessary.

7.2 Recommendation/Remedial Measures:

7.2.1 Structural Calculations: Structural analysis of the dam, including a check of the open joint noted in Section 6, should be started immediately and completed within 6 months.

7.2.2 In accordance with paragraph 7.1.1, it is recommended that within two months from the date of notification to the Governor of the Commonwealth of Virginia, the owner engage the services of a professional consultant to determine by more sophisticated methods and procedures the adequacy of the spillway. Even though the seriously inadequate spillway would produce a dam failure primarily from hydrologic reasons, remedial measures in structural or geotechnical areas may be needed to remove the dam from an unsafe classification. Within 6 months of the date of notification to the governor, the professional consultant's report of appropriate remedial mitigating measures should have been completed and the owner should have an agreement with the Commonwealth of Virginia to a reasonable time frame in which all remedial measures will be complete. In the interim, a detailed emergency operation plan and warning system should be promptly developed. It must be considered that the dam is threatened whenever the reservoir level exceed elevation 225 MSL in Townes Dam or the reservoir level at Talbott reservoir upstream exceeds 2538 until analysis shows otherwise. Also, during periods of unusually heavy precipitation, around-the-clock surveillance should be provided.

7.2.3 Flashboards: Flashboards should be removed until the stability of the dam is verified. (This reduces the probability of activating the warning system to approximately one in one hundred for any given year.)

7.2.4 Stop Logs: It is suggested that the city determine the feasibility of incorporating a stop gate, or stop logs in the intake opening which serves the 70-inch pipeline. If the installation of a stop gate is found not feasible, major repairs on the butterfly valve would necessitate lowering the reservoir pool to an elevation below the sill of the intake opening.

7.2.5 Trash Rack: It is recommended as outlined in the Logan Diving, Inc. report, that the city perform a semi-annual operational inspection of the trash rack and its motorized hoist, the motorized 70-inch butterfly valve, and the 12-inch gate valve. The inspection should be performed in the presence of the superintendent of the Pinnacles Hydro Station.

7.2.6 Pipeline Butterfly Valve: It is felt that after 40 years of use, the city should contact the Chapman-Crane Co. to determine when major repairs would be anticipated on the butterfly disk sealing surface, the fixed seal, the trunnions and their bearings. An inquiry should be made also on whether it is possible to replace the operating stem packing while the valve is subject to the reservoir water pressure.

7.2.7 Electrical Relocation: The transformers, fuse cutouts, and surge arresters should be relocated outside of the storage room and valve house.

7.2.8 Transformer Oil Leakage: Transformer oil leakage should be repaired and the dielectic strength of the oil should be tested.

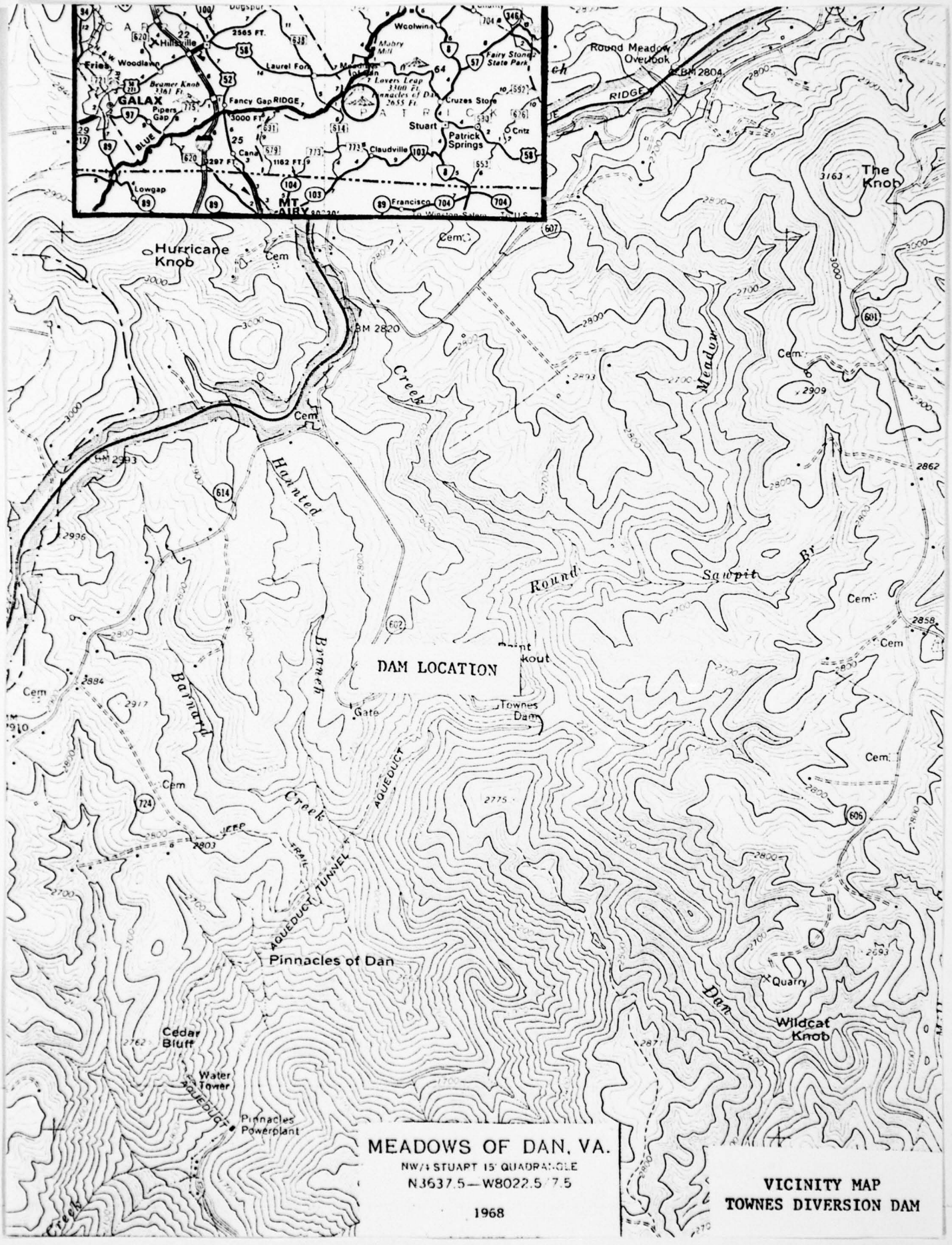
7.2.9 Emergency Lighting: A battery operated emergency lighting system should be provided in the valve house.

7.2.10 Monitoring Seepage: Seepage at the contact between the dam and left abutment should be measured and recorded every three months and additionally during periods when the reservoir is at a high level. A significant increase in flow rate might indicate that remedial foundation treatment would be required in the future.

7.2.11 Reservoir: The reservoir should be cleared of debris.

APPENDIX I

MAPS AND DRAWINGS



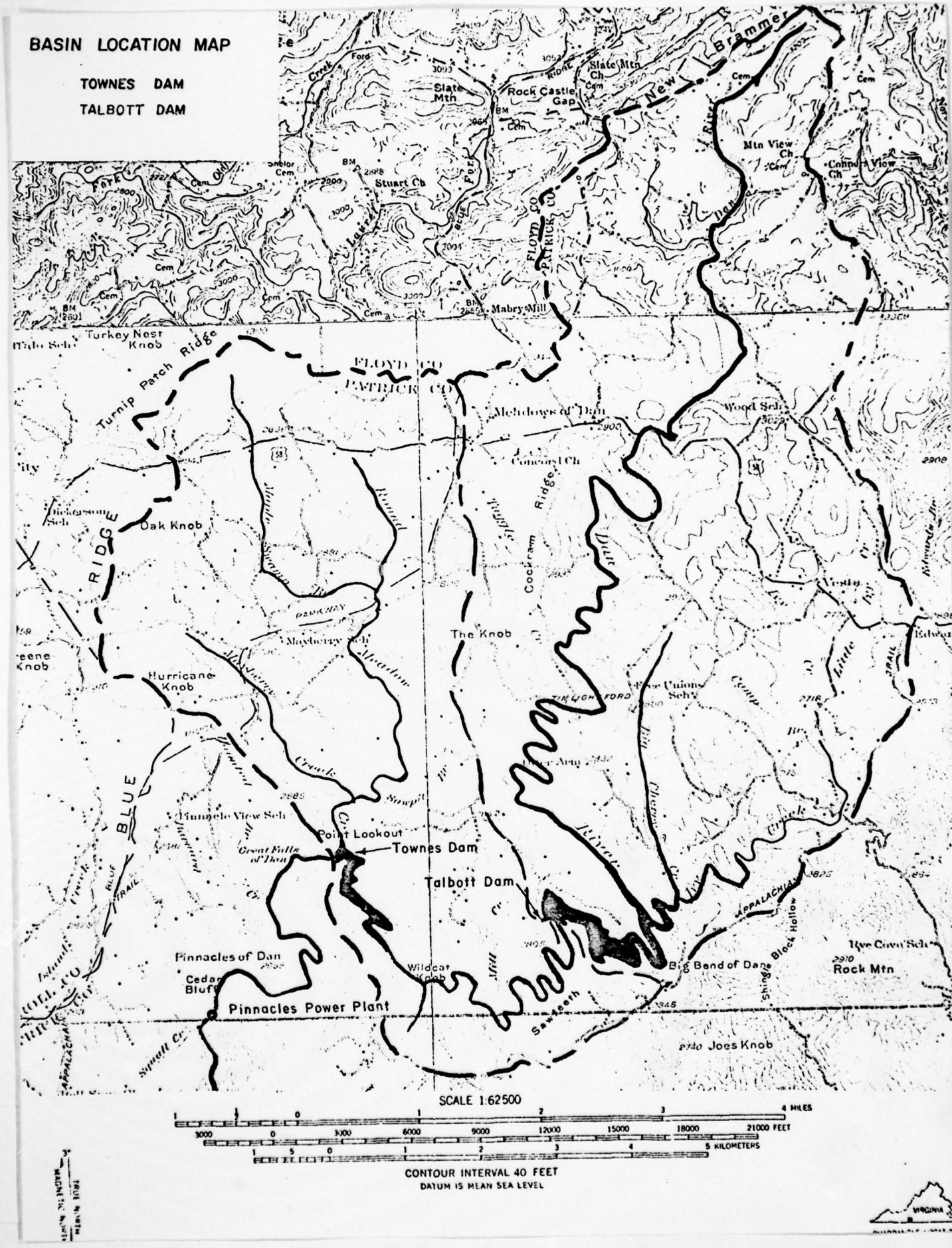
MEADOWS OF DAN, VA.

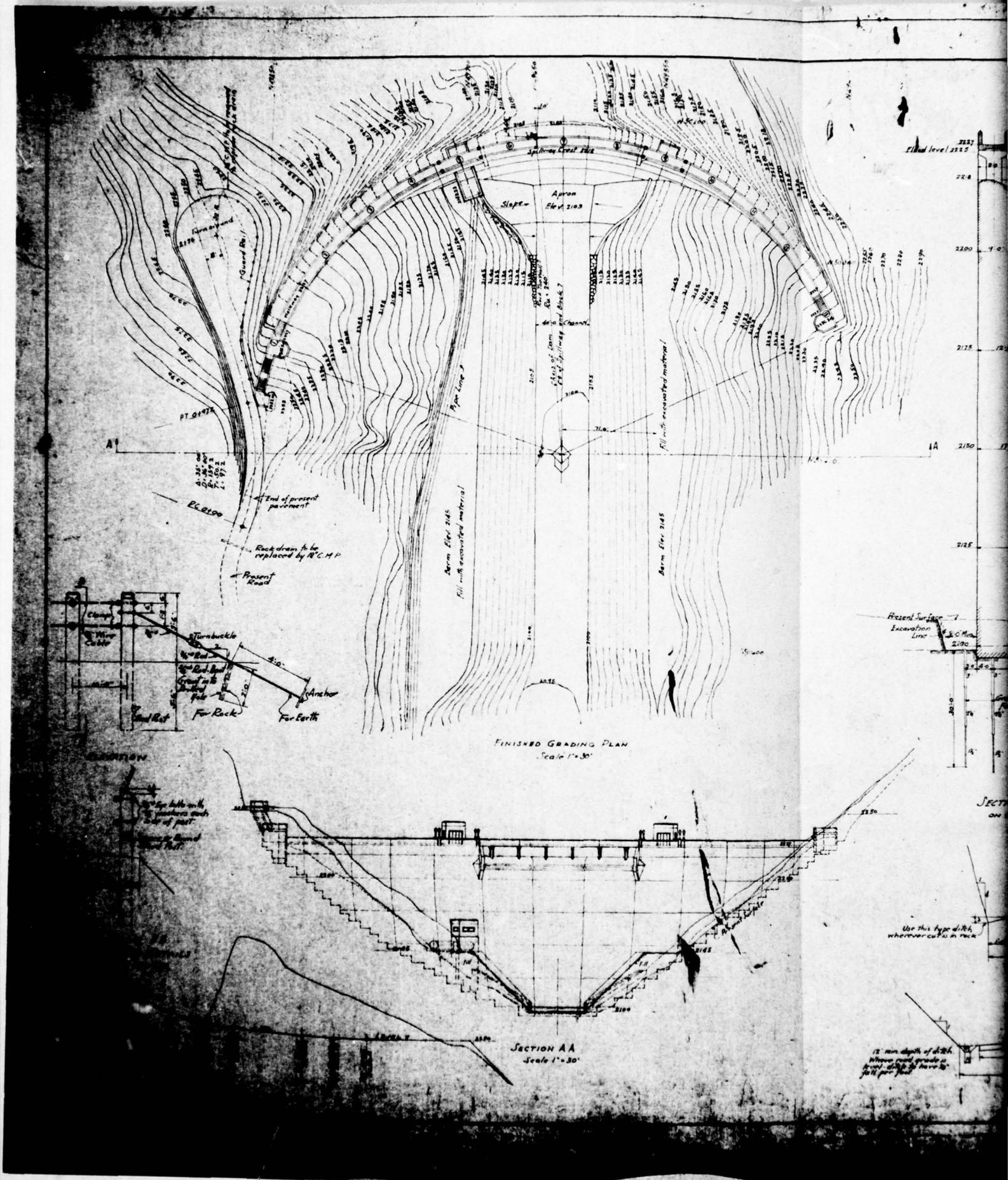
NW/4 STUART 15' QUADRANGLE

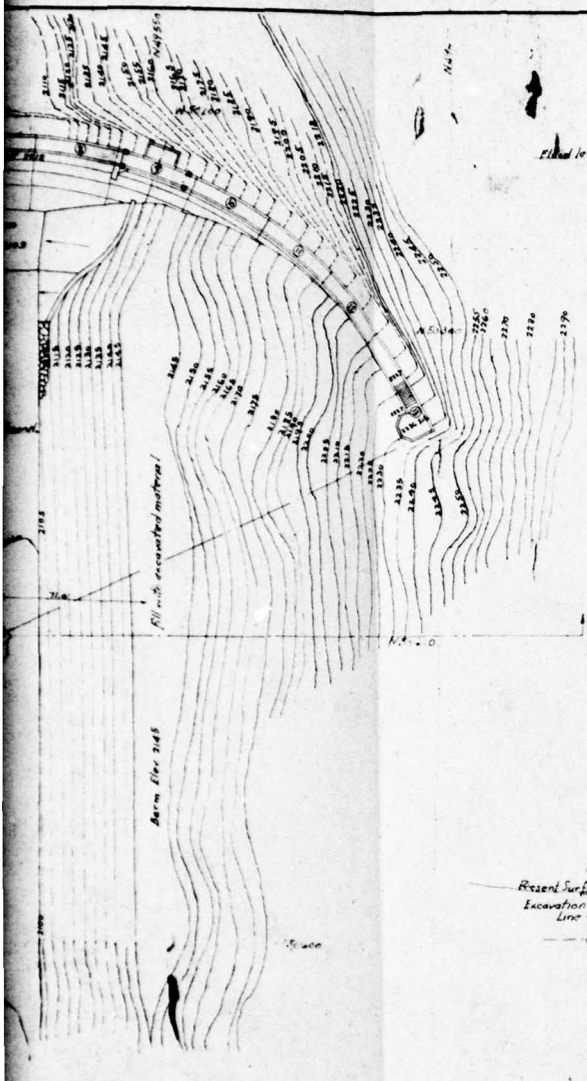
N3637.5—W8022.5/7.5

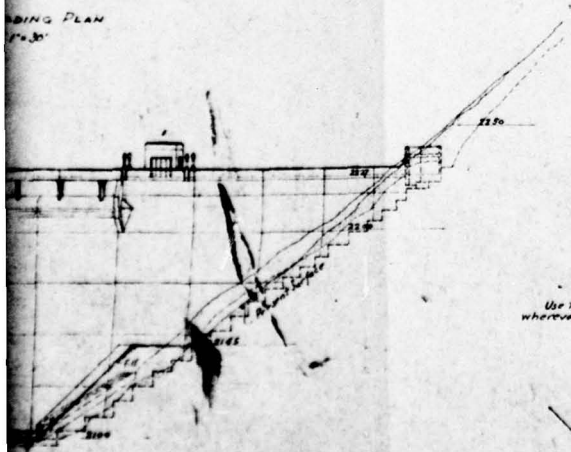
1968

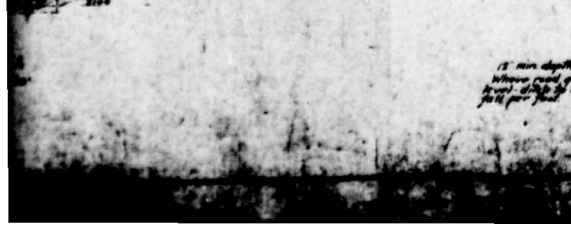
VICINITY MAP
TOWNES DIVERSION DAM


BASIN LOCATION MAP
TOWNES DAM
TALBOTT DAM
Slate Mtn
Rock Castle Gap
New River
Mtn View Ch
Stuart Cb
Mabry Mill
Turkey Nest Knob
Turnip Patch Ridge
FLOYD CO
PATRICK CO
Meadows of Dan
Concord Ch
Oak Knob
RIDGE
The Knob
Hurricane Knob
BLUE
Point Lookout
Townes Dam
Talbott Dam
Pinnacles of Dan
Cedar Bluff
Wildcat Knob
Pinnacles Power Plant
Big Band of Dan
Rock Mtn
Joes Knob
SCALE 1:62500
4 MILES
21000 FEET
5 KILOMETERS
CONTOUR INTERVAL 40 FEET
DATUM IS MEAN SEA LEVEL
VIRGINIA


Spillway Crest Elev
Apron Elev 2109
Slope
Guard Rail
Guard Rail
Flood level 2227.5
Axis of Dam
Ft of Spillway and Channel
Fill with excavated material
Berm Elev 2145
Fill with excavated material
Berm Elev 2145
Pipe Line
End of present pavement
Rock drain to be replaced by R.C.M.P.
Present Road
Present Surface
Excavation Line
Turnbuckle
Anchor
For Rock
For Earth
Finished Grading Plan
Scale 1" = 30'
Section AA
Scale 1" = 30'
Use this type ditch wherever cut is in rock
12" min. depth of ditch. Where road grade is level, ditch to have 1/2" fall per foot.



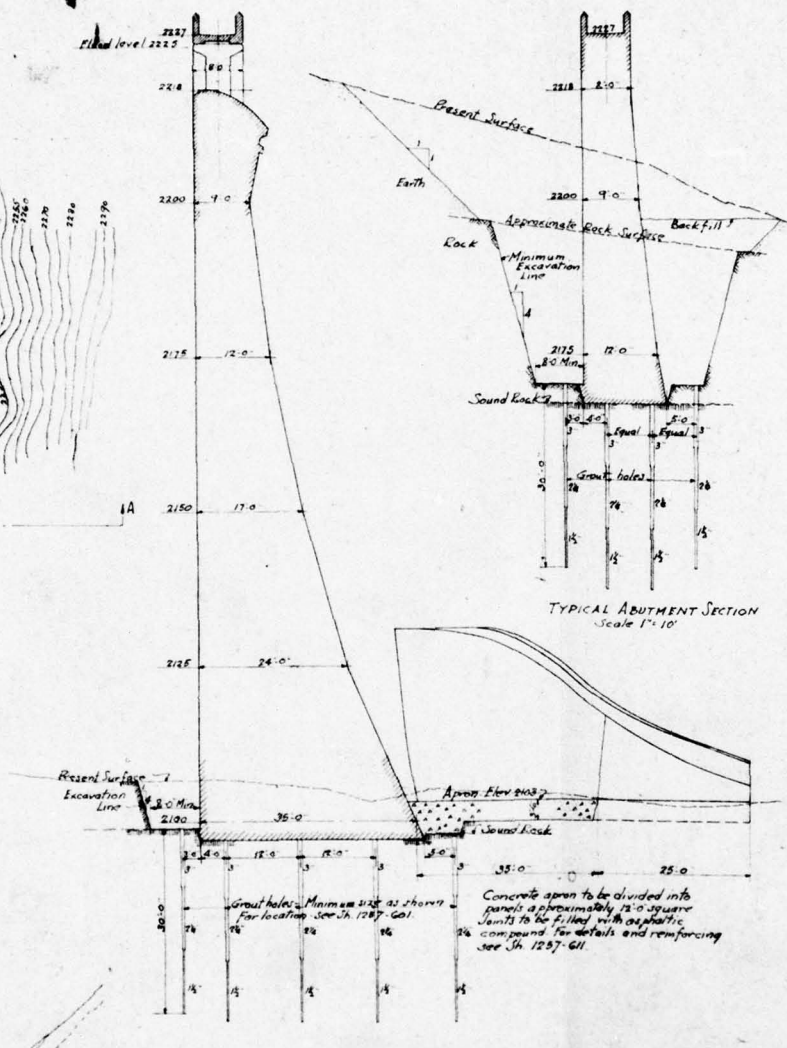




GRADING AND PAVING SECTION
ROAD STATION 0+92
Scale ½" = 1'-0"



GRADING AND PAVING SECTION
ROAD STATION 2+50
Scale ½" = 1'-0"



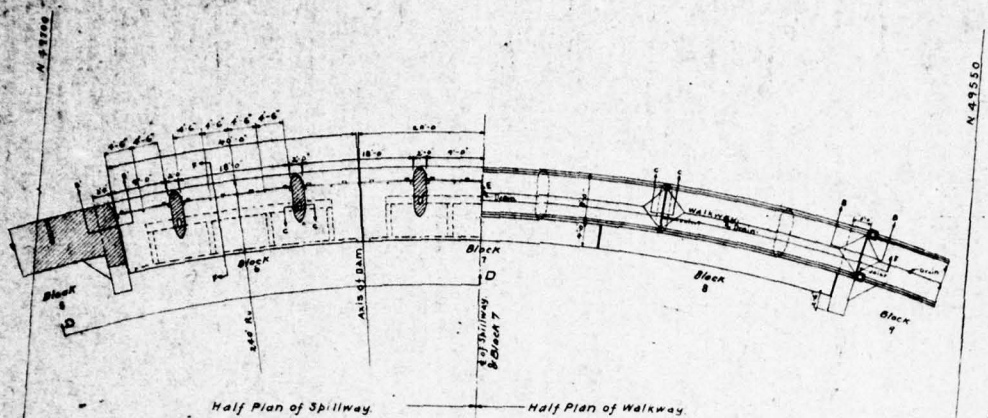
SECTION THROUGH DAM
ON ℄ OF SPILLWAY
Scale 1" = 10'-0"

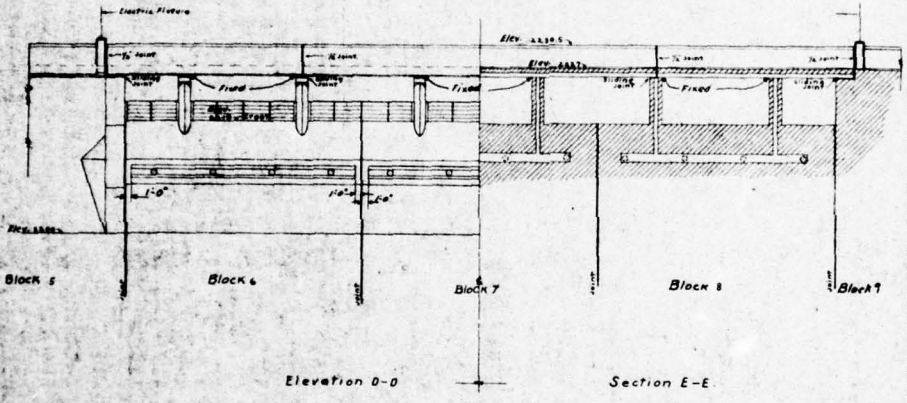
TYPICAL ABUTMENT SECTION
Scale 1" = 10'

PLATE 1

PWA DOCKET No. 1151 R
SHEET 12 OF 17 SHEETS

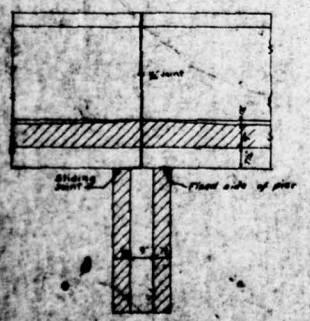
PINNACLES HYDRO-ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE, VIRGINIA
TOWERS - DAM
EXCAVATION, GROUTING, GRADING



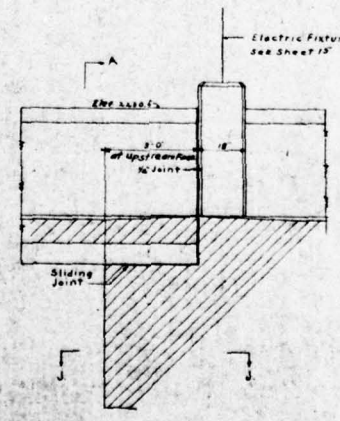


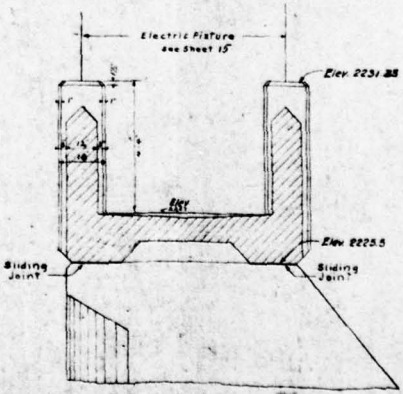
SCHEDULE FOR FLASHBOARDS
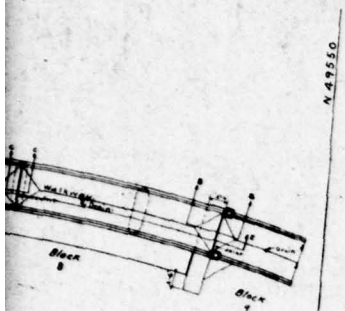
Item No.	No. Req'd.	Diagram	Remarks
①	17	Furr. ½ exact	Flashboard Pin Mild steel ultimate strength 55,000 lb.
②	17	3" Galv. Cap	Flashboard Socket 3" Galv. wrought steel pipe Standard weight
③	14	¾" Hole	End Support Clear stock Creosoted Pine
④	5	11'-10"	Flashboards Creosoted Pine Clear stock
⑤	2	8'-10"	Flashboards Creosoted Pine Clear stock
⑥	42		Truscon tapped inserts for ⅝" bolts Galvanized
⑦	42		⅝" dia. bolt and washer Galvanized
⑧	34		½" dia. U-bolts Galvanized

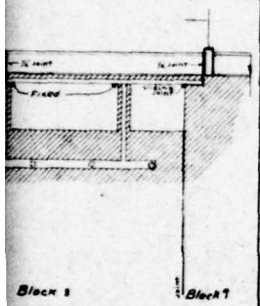


Section C-C
Scale ½" = 1'-0"



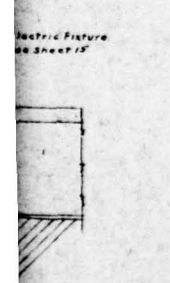


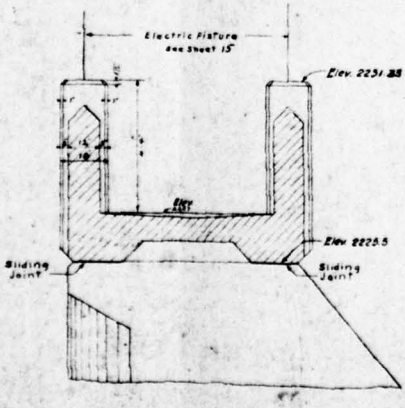




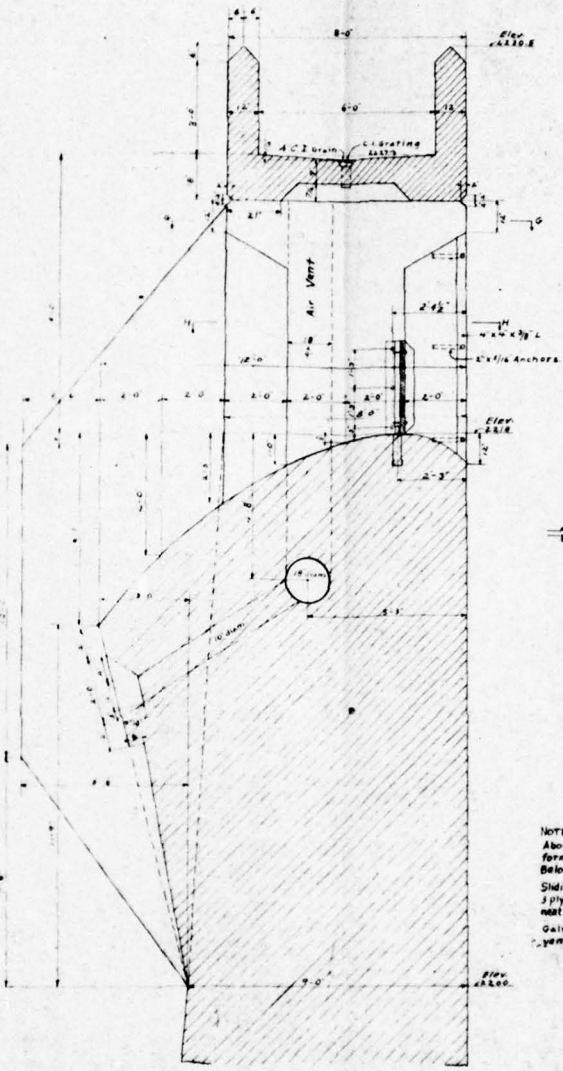
SCHEDULE FOR FLASHBOARDS

Item No.	No. Req'd	Diagram	Remarks
①	17	Turn ¾ exact	FLASHBOARD PIN Mild steel, ultimate strength 55,000 lb.
②	17	3 Galv. Cap 12" exact	FLASHBOARD SOCKET 3 Galv. wrought steel pipe, Standard weight
③	14	6 Hole	END SUPPORT Clear stock 4"x6" Creosoted Pine
④	5	17'-10"	FLASHBOARDS Creosoted Pine Clear stock
⑤	2	8'-10"	FLASHBOARDS Creosoted Pine Clear Stock
⑥	42		Truscon Tapped Inserts for ¾" bolts Galvanized
⑦	42		¾"x 4" bolt and washer Galvanized
⑧	38		¾ dia. U-bolts Galvanized

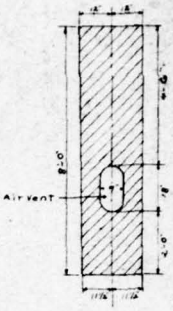




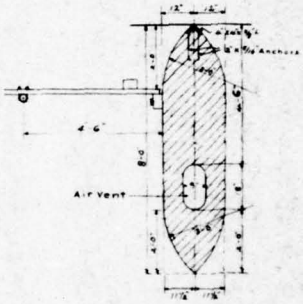
Section A-A
Scale ½"=1'-0"



Section F-F
Scale ½"=1'-0"



Section G-G.
Scale ½"=1'-0"



Section H-H
Scale ½"=1'-0"

NOTES
Above Elev. 2200, steel forms shall be used to form the spillway and the walkway piers. Below Elev 2200, wooden forms may be used.
Sliding joints in walkway to be provided by placing 3 plys of 15 lb. Asphalt Saturated felt trimmed neat to whole size of the joint area.
Galvanized steel forms for all air vents to be left in place.



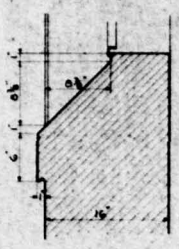
Section J-J See Section B-B
Scale ½"=1'-0"

PWA DOCKET Nº 1151-R

SHEET 6 OF 17 SHEETS

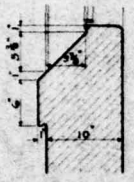
PINNACLES HYDRO-ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE VIRGINIA
POINT LOOKOUT DAM
BLOCKS 5, 6, 7, 8 AND 9
SPILLWAY
CHAS. T. MAIN, INC. ENGINEERS

PLATE 2

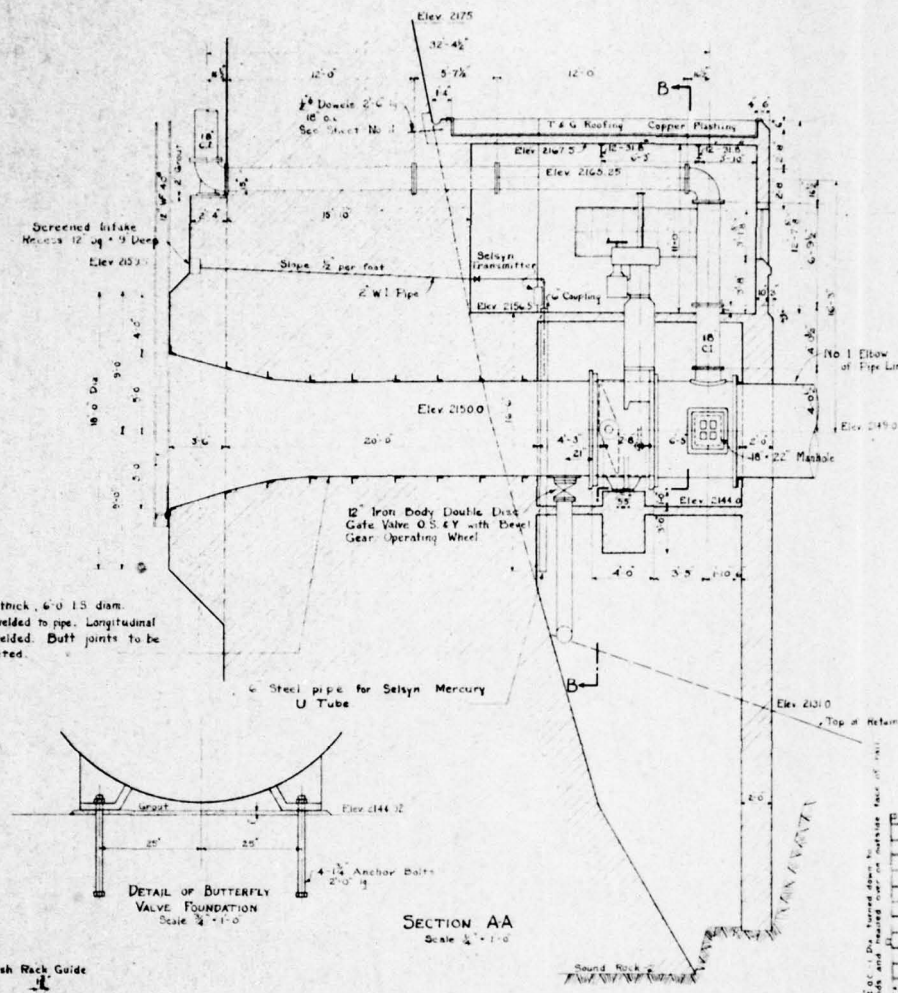


DETAIL OF FIRST FLOOR WINDOW SILL
Scale 1½" = 1'-0"

Contractor to provide head, jamb & mullion slots for sash erection.

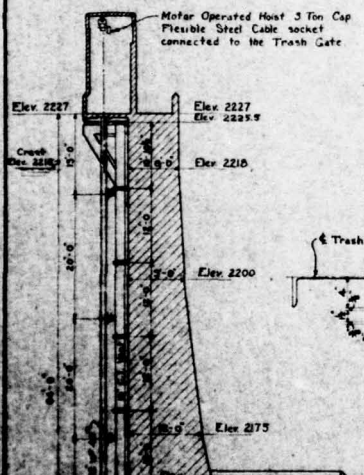


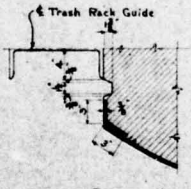
DETAIL OF SECOND FLOOR WINDOW SILLS
Scale 1½" = 1'-0"



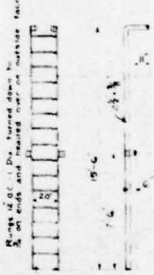
SECTION A-A
Scale ⅛" = 1'-0"

DETAIL OF BUTTERFLY VALVE FOUNDATION
Scale ½" = 1'-0"

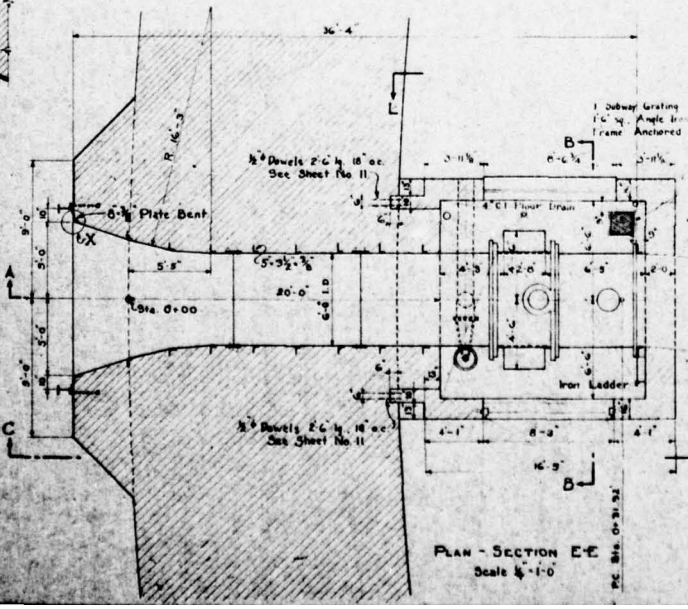




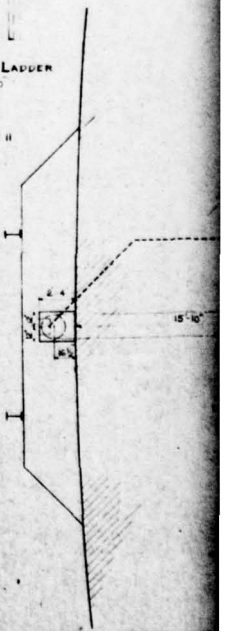
DETAIL X
Scale 1½" = 1'-0"



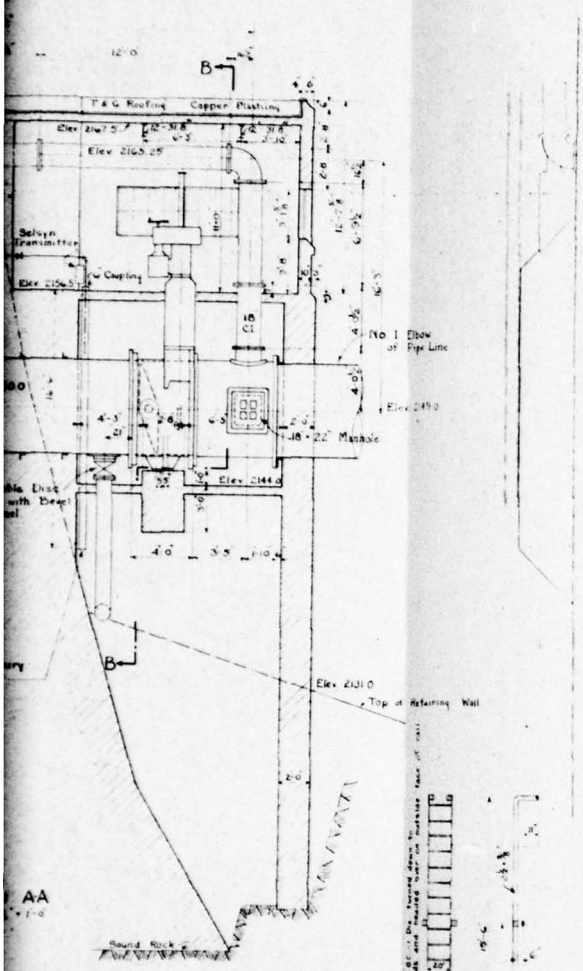
DETAIL OF IRON LADDER
Scale ½" = 1'-0"



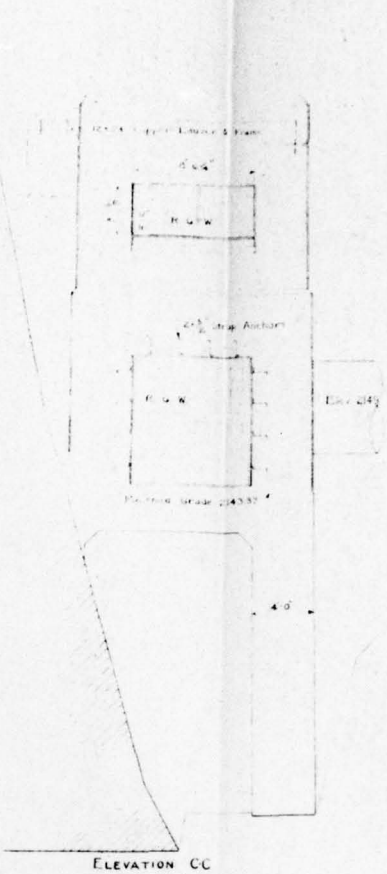
PLAN - SECTION E-E
Scale ¼" = 1'-0"



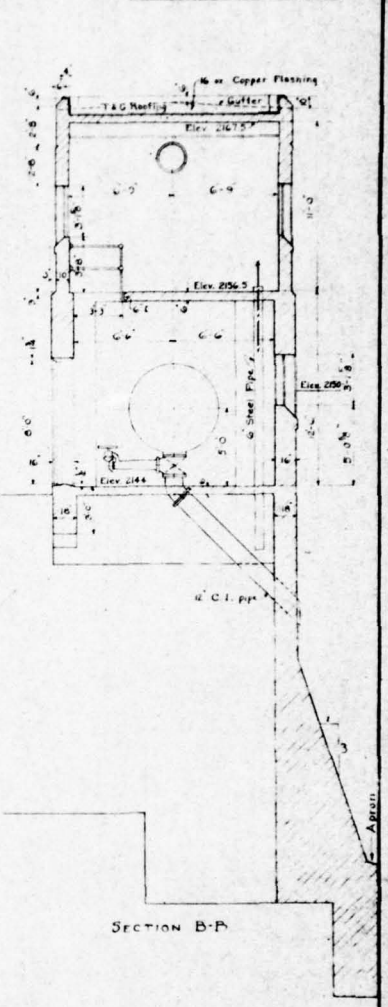
Notes
R.G.W. Indicates
For Location
Electrical Eq

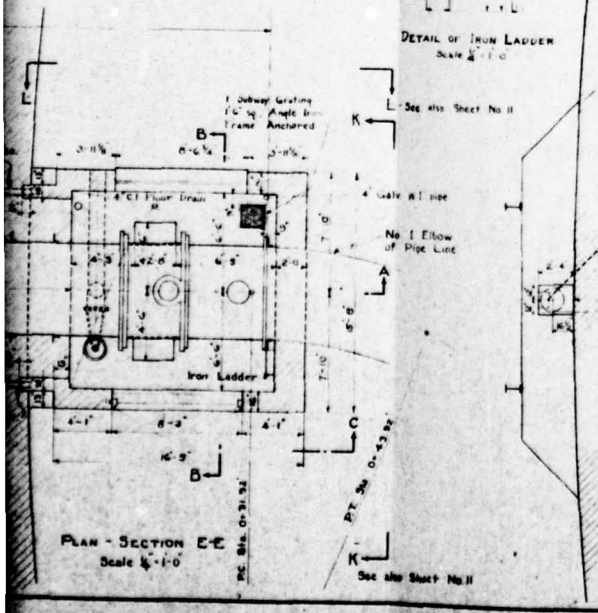


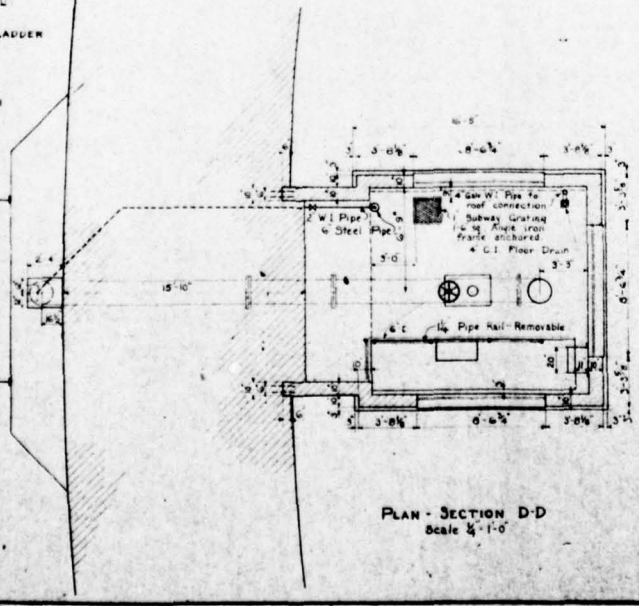




Notes
R.G.W. Indicates Rough Glass Wired
For Location of Conduit Pull Boxes & other Electrical Equipment, See Sheet No. 14







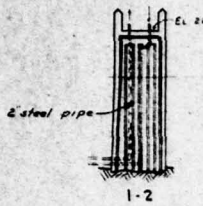
Sheet 4 of 17 Sheets
This drawing supersedes No 1257-604, dated 11-3-37
P.W.A. Docket 1151 PR

Pinnacles Hydro Electric Project
City of Danville Electric Department
Danville, Virginia
Point Lookout Dam
Valve House

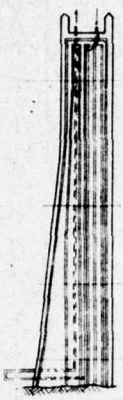
Chas. T. Main, Inc. Engineers
201 Devonshire St. Boston, Mass. U.S.A.

Revisions

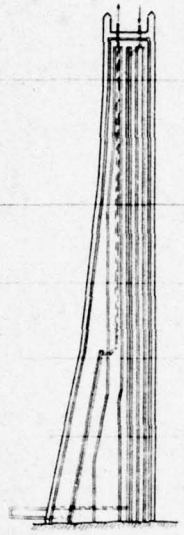
PLATE 3



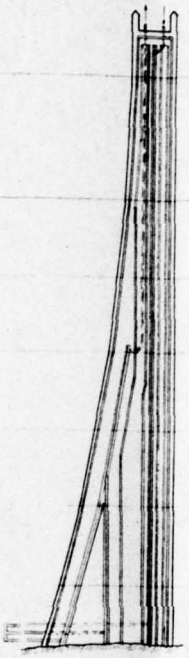
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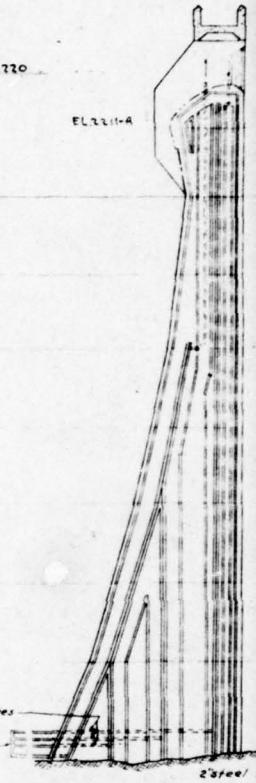
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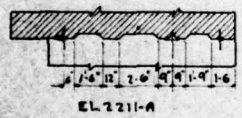
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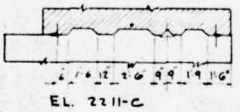


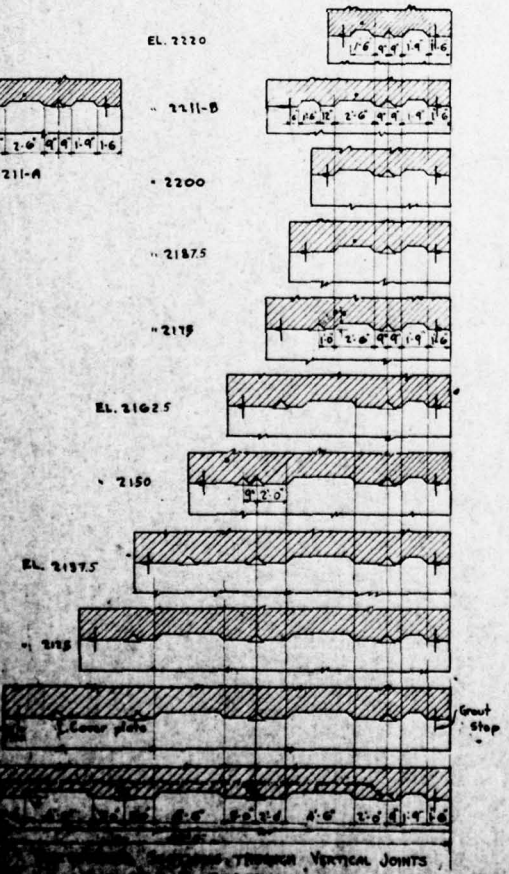
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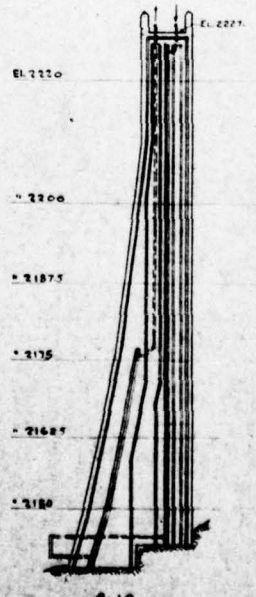


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9-10



10-11

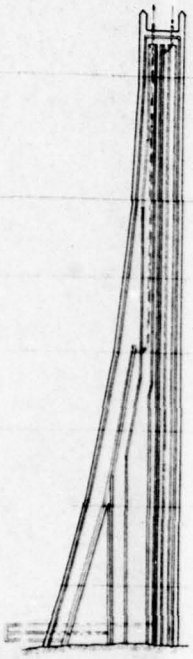


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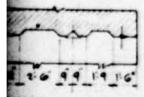
ELEVATIONS OF VERTICAL JOINTS
Scale

NUMBERS UNDER EACH ELEVATION INDICATE
KEEP GROUT SLOTS CLEAN BY BLOWING COMPRES
↑ ↓ ARROWS SHOW DIRECTION OF FLOW OF

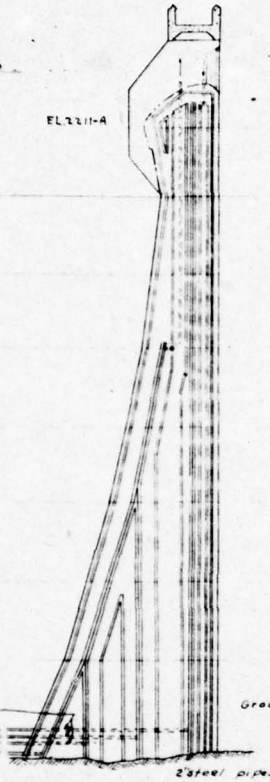




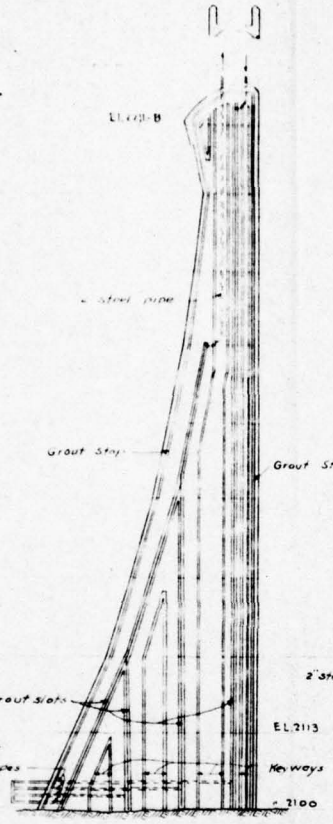
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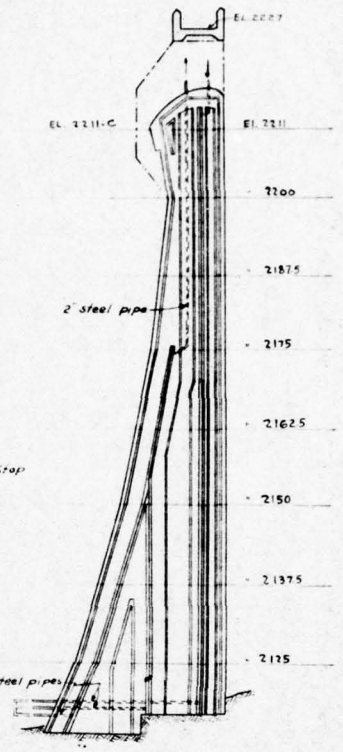
11-C



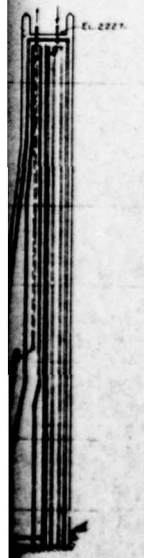
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6-7, 7-8 Similar

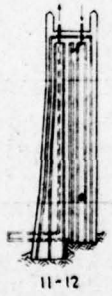


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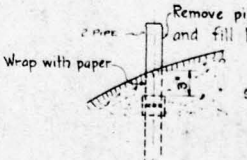
10-11



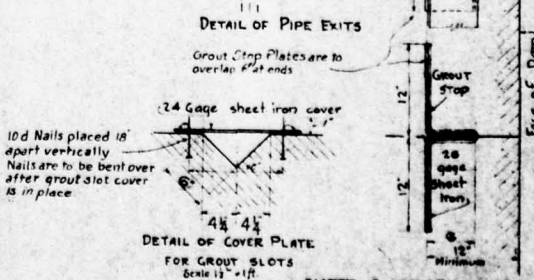
11-12



12-13



DETAIL OF PIPE EXITS



DETAIL OF COVER PLATE FOR GROUT SLOTS
Scale 1½" = 1'-0"

ELEVATIONS OF VERTICAL JOINTS SHOWING KEYWAYS AND GROUTING PROVISIONS
Scale 1" = 10'-0"

NUMBERS UNDER EACH ELEVATION INDICATE JOINTS BETWEEN SIMILARLY NUMBERED BLOCKS

KEEP GROUT SLOTS CLEAN BY BLOWING COMPRESSED AIR THROUGH THEM AFTER PLACING OF EACH LAYER OF CONCRETE.

↑↓ ARROWS SHOW DIRECTION OF FLOW OF GROUT.

PLATE 4

SHEET 8 OF 17 SHEETS

PINNACLES HYDRO-ELECTRIC PROJECT

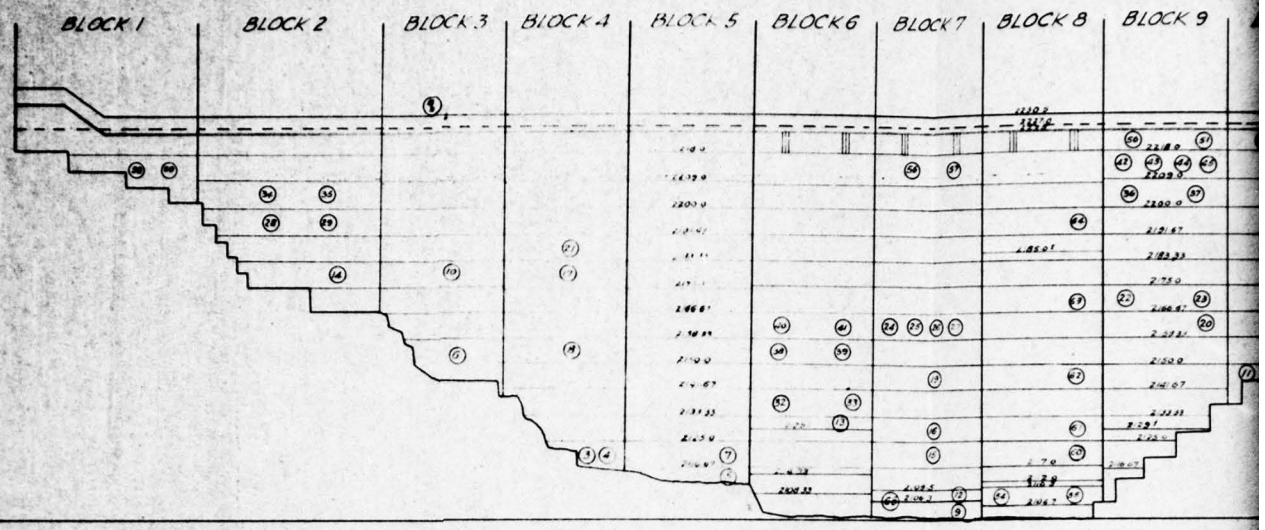
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE, VIRGINIA

POINT LOOKOUT DAM

VERTICAL CONSTRUCTION JOINTS

CHAS. T. MAIN, INC. ENGINEERS
201 DEVONSHIRE ST. BOSTON

CONCRETE TEST DATA



TABULATION OF TESTS

RESULTS OF COMPRESSION TESTS ON CLASS 'A' CONCRETE POINT LOOKOUT DAM

No. Cyl.	Size	7 Days	28 Days
D-A 3	6 x 12	3882	—
D-A 4	6 x 12	—	5697
D-A 5	6 x 12	3672	—
D-A-6	6 x 12	—	5288
D-A-7	6 x 12	—	4129
D-A-8	6 x 12	—	5808
D-A-9	6 x 12	—	5982
D-A-10	6 x 12	—	4892
D-A-11	6 x 12	—	5072
D-A-12	6 x 12	—	5336
D-A-13	6 x 12	—	5538
D-A-14	6 x 12	—	5984
D-A-15	6 x 12	—	5089
D-A-16	6 x 12	—	5488
D-A-17	6 x 12	—	4962
D-A-18	6 x 12	—	5700
D-A-19	6 x 12	—	5906
D-A-20	6 x 12	—	5845
D-A-21	6 x 12	—	4868
D-A-22	6 x 12	Record lost	

No. Cyl.	Size	7 Days	28 Days
D-A-23	6 x 12	—	3791
D-A-24	6 x 12	2529	—
D-A-25	6 x 12	—	3228
D-A-26	6 x 12	3 days 2495	
D-A-27	6 x 12	6 days 3148	
D-A-28	6 x 12	2370	—
D-A-29	6 x 12	—	3678
D-A-30	6 x 12	2239	—
D-A-31	6 x 12	—	3512
D-A-32	6 x 12	2812	—
D-A-33	6 x 12	—	4265
D-A-34	6 x 12	2592	—
D-A-35	6 x 12	—	4757
D-A-36	6 x 12	2978	—
D-A-37	6 x 12	—	4393
D-A-38	6 x 12	2122	—
D-A-39	6 x 12	—	3519
D-A-40	6 x 12	2873	—
D-A-41	6 x 12	—	4792
D-A-42	6 x 12	6 days 3060	

No. Cyl.	Size	7 Days	28 Days
D-A-43	8 x 16	—	3979
D-A-44	6 x 12	—	5942
D-A-45	6 x 12	—	5818
D-A-46	8 x 16	3478	—
D-A-47	8 x 16	—	3979
D-A 48	6 x 12	—	6 032
D-A 49	6 x 12	—	7054
D-A 50	6 x 12	—	5561
D-A-51	8 x 16	—	3971
D-A-52	8 x 16	—	3352
D-A 53	6 x 12	—	4739
D-A 54	8 x 16	—	3909
D-A-55	6 x 12	—	5217
D-A 56	8 x 16	—	3829
D-A-57	6 x 12	—	5370
D-A 58	6 x 12	—	4906
D-A-59	8 x 16	—	3953
D-A-60	6 x 12	—	4 990
D-A 61	6 x 12	—	4 27
D-A 62	6 x 12	—	4810

No. Cyl.	Size	7 Days	28 Days
D-A 63	6 x 12	—	3316
D-A 64	6 x 12	—	4608
D-A 65	6 x 12	—	5371

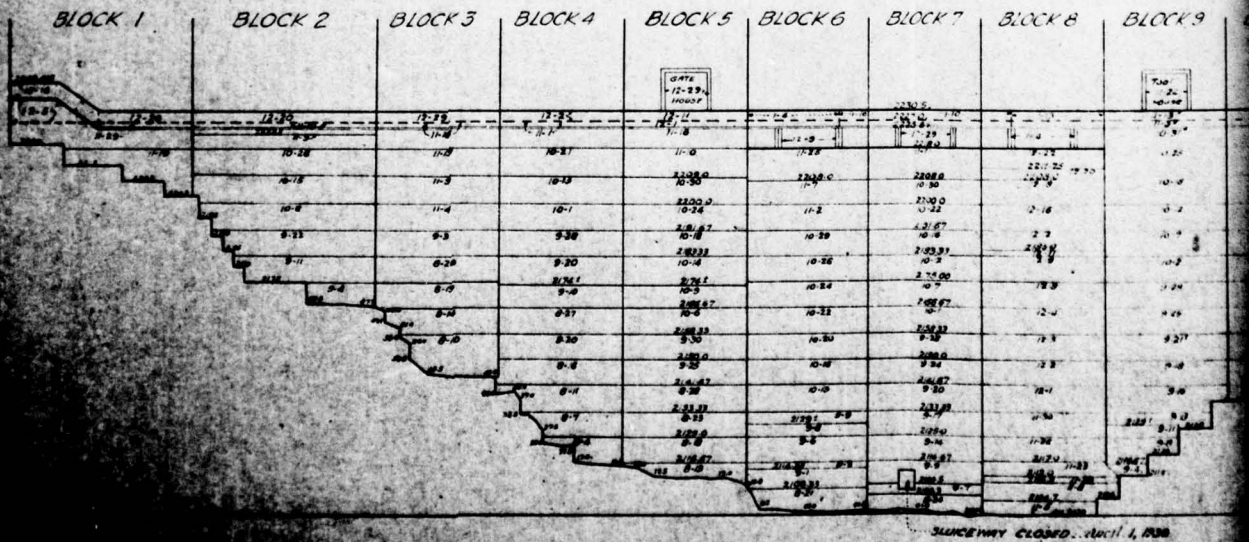
* D-A 65 Taken when placed on line Aug. of April 1, 1938

No. Cyl.	Size	7 Days	28 Days*
*D-1	6 x 12	3395	—
*D-2	6 x 12	—	3820
D-B-1	6 x 12	2023	—
D-B-2	6 x 12	—	3961
D-B-3	6 x 12	—	3827

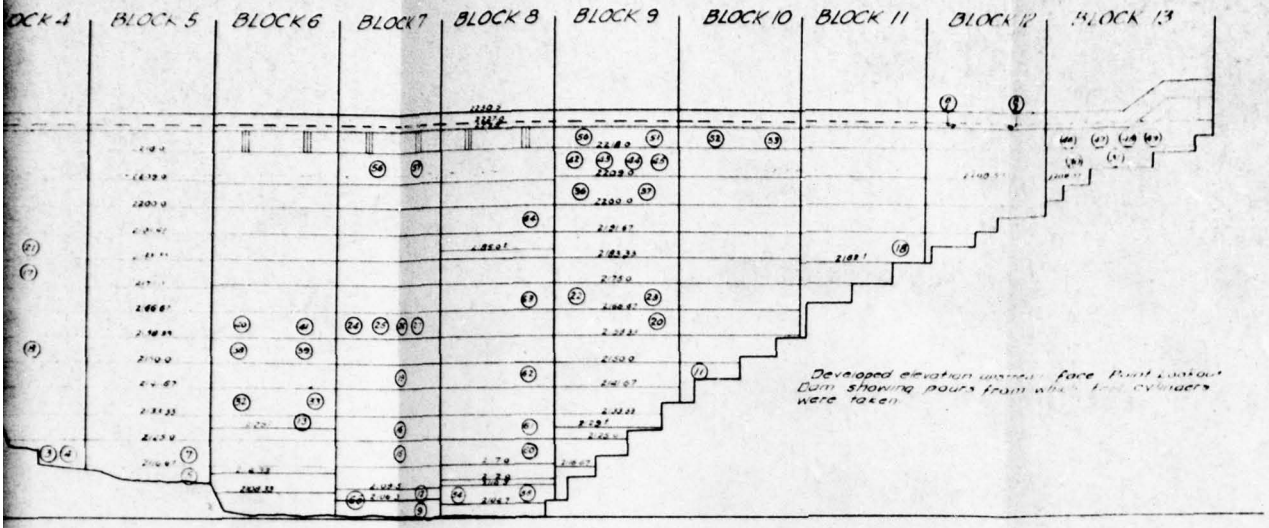
* By 6 day rate taken from Instrument plot

TEST CYLINDERS WERE MADE FOR APPROXIMATELY EVERY 513 C. Yds OF CONCRETE POURED

DEVELOPED ELEVATION OF UPSTREAM FACE POINT LOOKOUT DAM SHOWING POURING DATES FOR EACH BLOCK



CONCRETE TEST DATA



Developed elevation upstream face Point Lookout Dam showing pours from which test cylinders were taken.

Block	Size	Date	Age Days
5	6 x 12	-	33.6
6	6 x 12	-	46.8
6	6 x 12	-	53.41

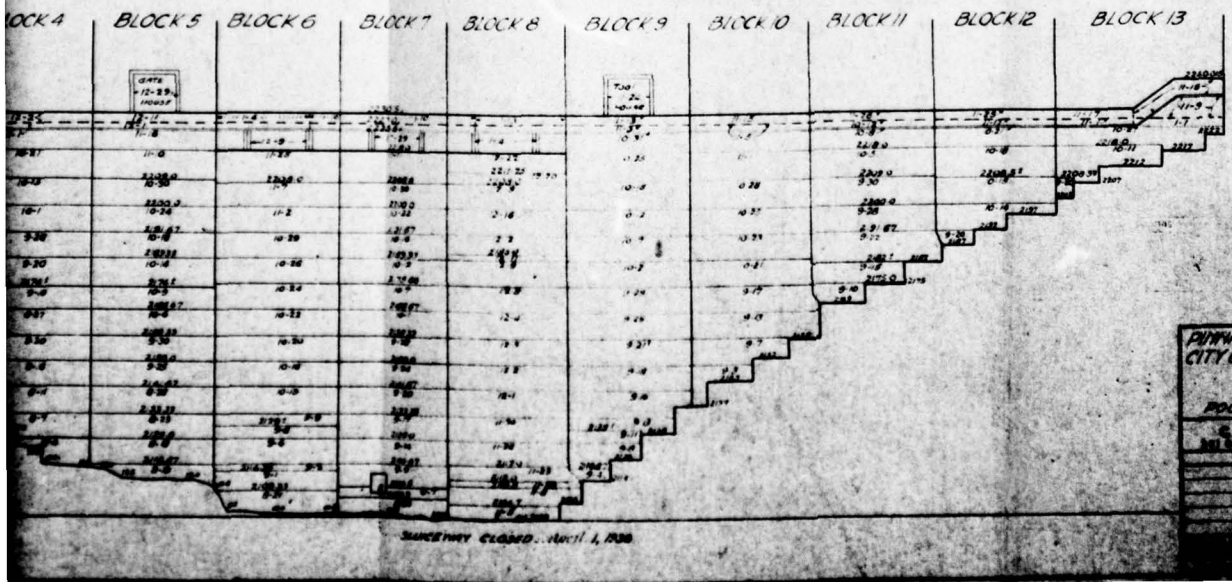
Cylinders taken when concrete was poured after April 1, 1938

Compression Tests of Concrete			
No. Cyl.	Size	Date	Weight P.C.
C-1	6 x 12	3325	-
C-2	6 x 12	-	3810
C-B-1	6 x 12	2023	-
C-C-2	6 x 12	-	3961
C-B-3	6 x 12	-	3827

* C-1 & C-B were taken from test from Retirement Dam

ONE CYL. EVERY 513 C. Yds.

DEVELOPED ELEVATION OF UPSTREAM FACE POINT LOOKOUT DAM
SHOWING POURING DATES FOR EACH BLOCK

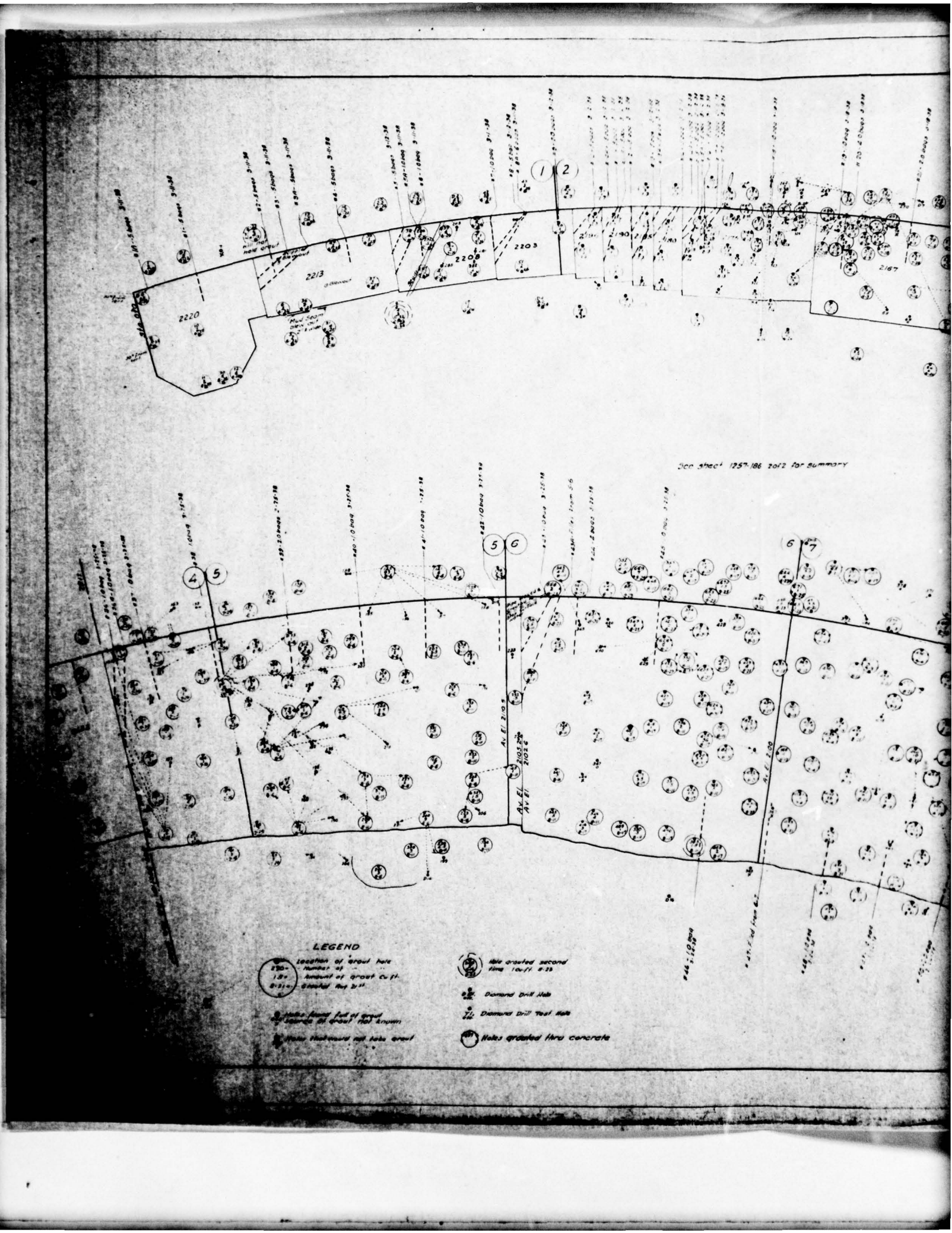


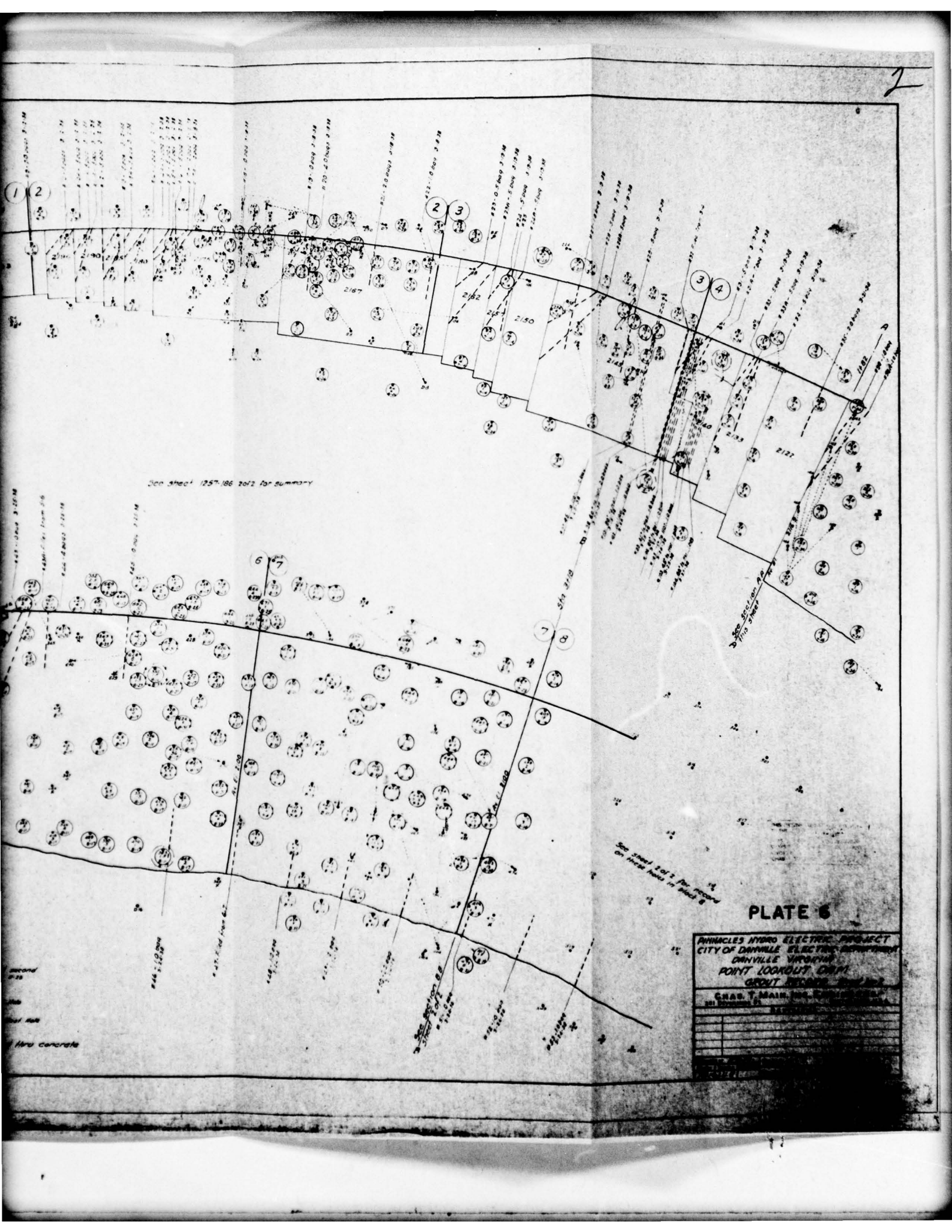
NOTE
First concrete was poured Aug 4'37 in block 4

SLUICEWAY CLOSED - April 4, 1938

PLATE 5

PINNACLES HYDRO-ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC
DANVILLE
POINT LOOKOUT DAM
POURING AND TESTING

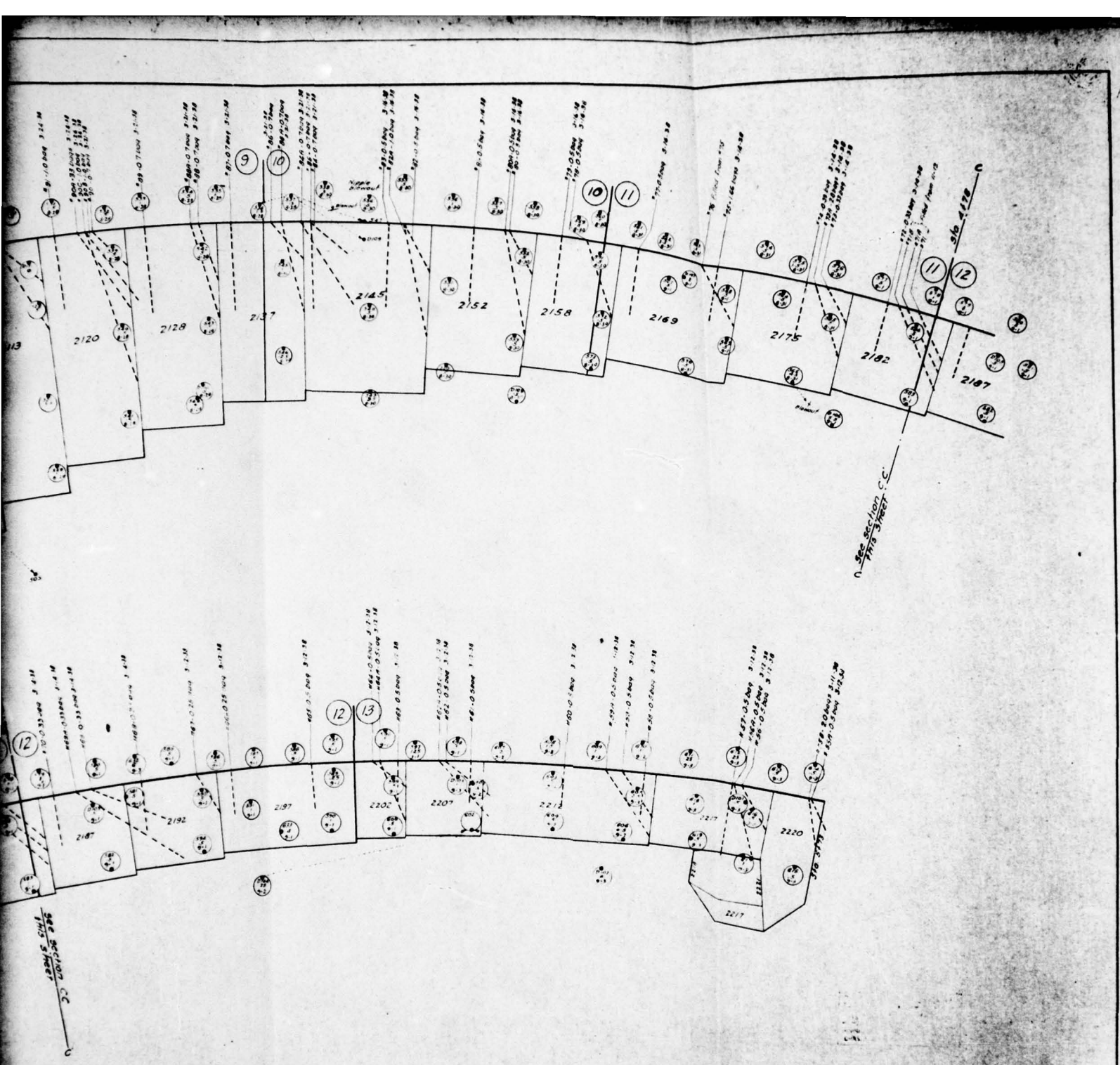

See sheet 1257-186 2012 for summary
1
2
3
4
5
6
7
2220
2213
2208
2203
2167
Av. El. 2103
Av. El. 2102.6
LEGEND
Location of grout hole
Number of ...
Amount of grout Cu. ft.
Stacked thus 2"
Holes having had of grout
No record of grout not known
Holes that would not take grout
Hole grouted second time 1 cu. ft. & 2s
Diamond Drill Hole
Diamond Drill Test Hole
Holes grouted thru concrete


See sheet 1257-186 2012 for summary
See Section A-A this sheet
See sheet 2 of 2 for details on this sheet
See Section B-B Sheet 1 of 2
PLATE 6
PINNACLES HYDRO ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE VIRGINIA
POINT LOOKOUT DAM
GROUT RECORD

GROUTING VERTICAL JOINTS

Adjoining Blocks	Date	Cu Ft Grout
1-2	3-10-38	29
2-3	3-16-38	38
3-4	3-18-38	102
4-5	3-22-38	98
5-6	3-29-38	150
6-7	3-28-38	175
7-8	3-31-38	102
8-9	3-13-38	19
9-10	3-14-38	9
10-11	3-17-38	8
	Total	723 Cu Ft

[illegible] 10866 Lin Ft | 2698.5 Cu Ft Grout
16376 Lin Ft
[illegible] 1781 Lin Ft 147 Cu Ft Grout
140 Cu Ft Grout
[illegible] above Grout 398 Cu Ft were
[illegible] grout in nipples, 13 cu ft
[illegible] used to grout in nipples to
[illegible] grouting Steamway, only 75 cu ft
[illegible] for all existing vertical joints.



PINNACLES HYDRO-ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE VIRGINIA
POINT LOOKOUT DAM
GROUT RECORD Sheet 2 of 2

CHAS. T. MAIN, INC. ENGINEERS
201 Devonshire St. Boston Mass. U.S.A.

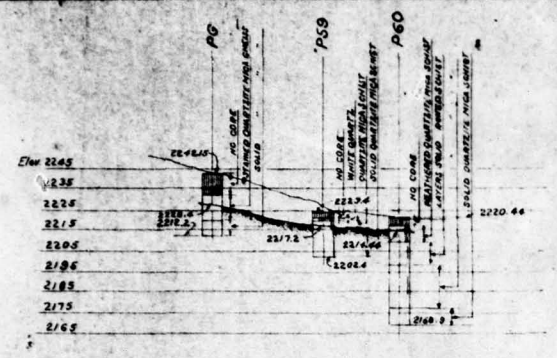
REVISIONS

DATE | BY | APPD

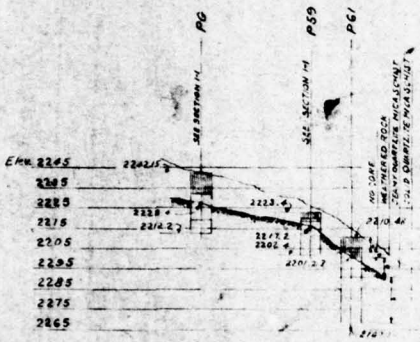
PROJECT 1257-1864

PLATE 7

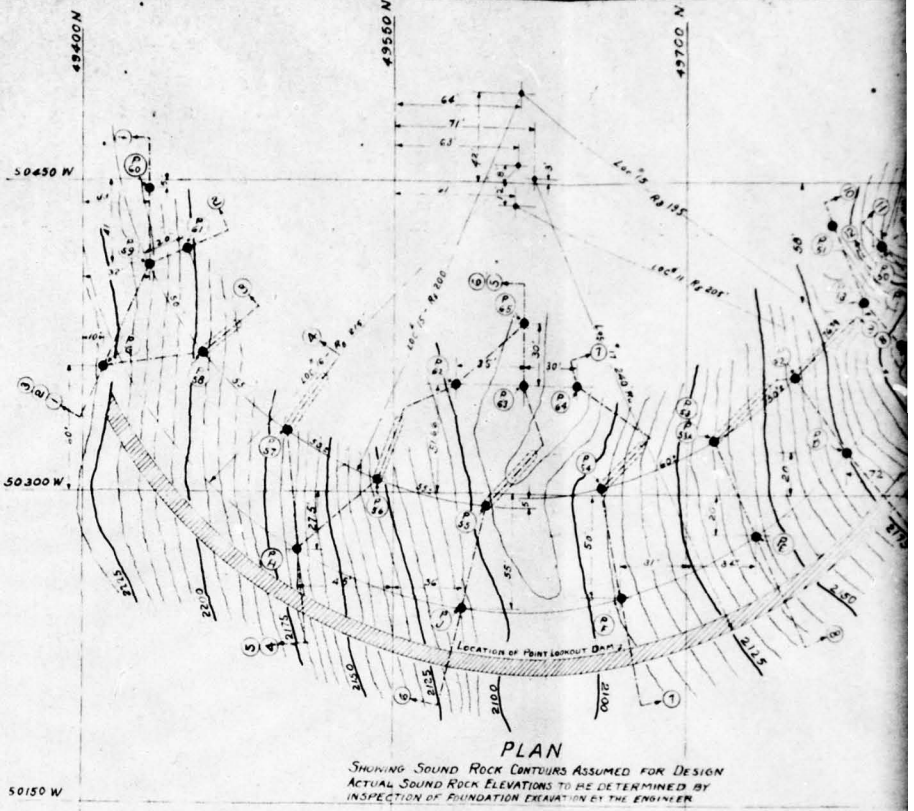
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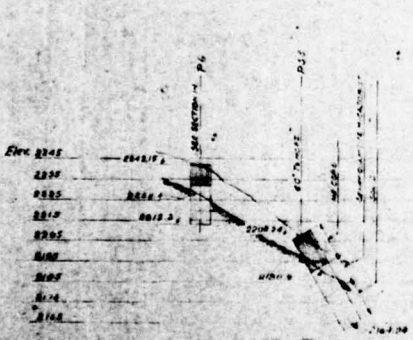


SECTION (2) (2)

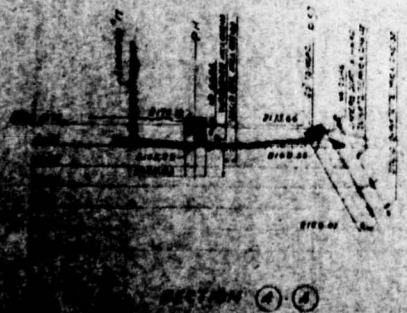


PLAN

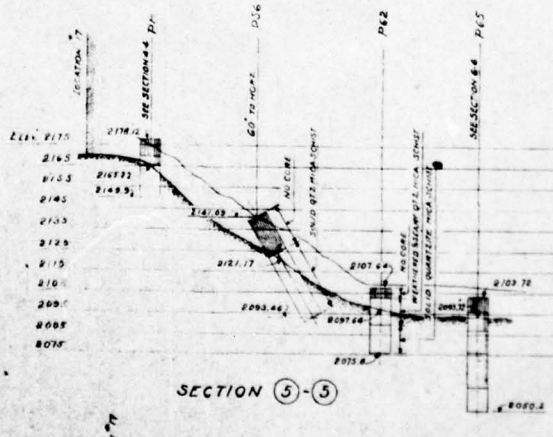
SHOWING SOUND ROCK CONTOURS ASSUMED FOR DESIGN
ACTUAL SOUND ROCK ELEVATIONS TO BE DETERMINED BY
INSPECTION OF FOUNDATION EXCAVATION BY THE ENGINEER



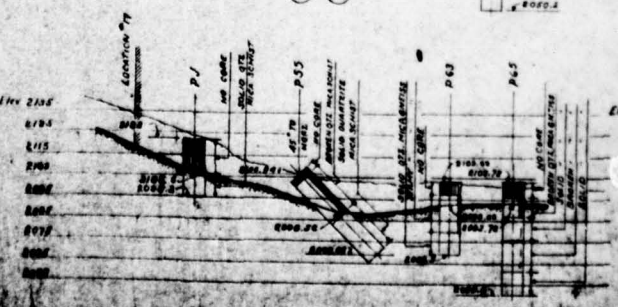
SECTION (3)-(3)



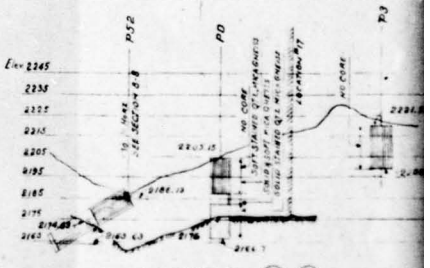
SECTION (4)-(4)



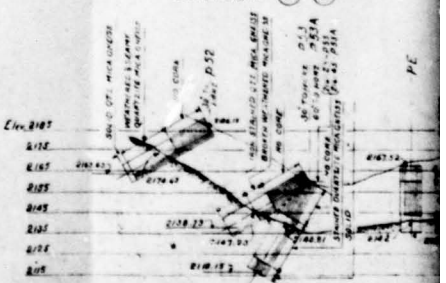
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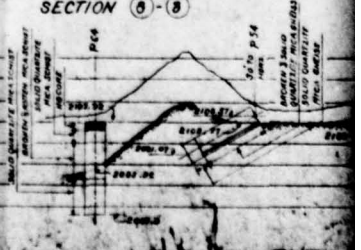
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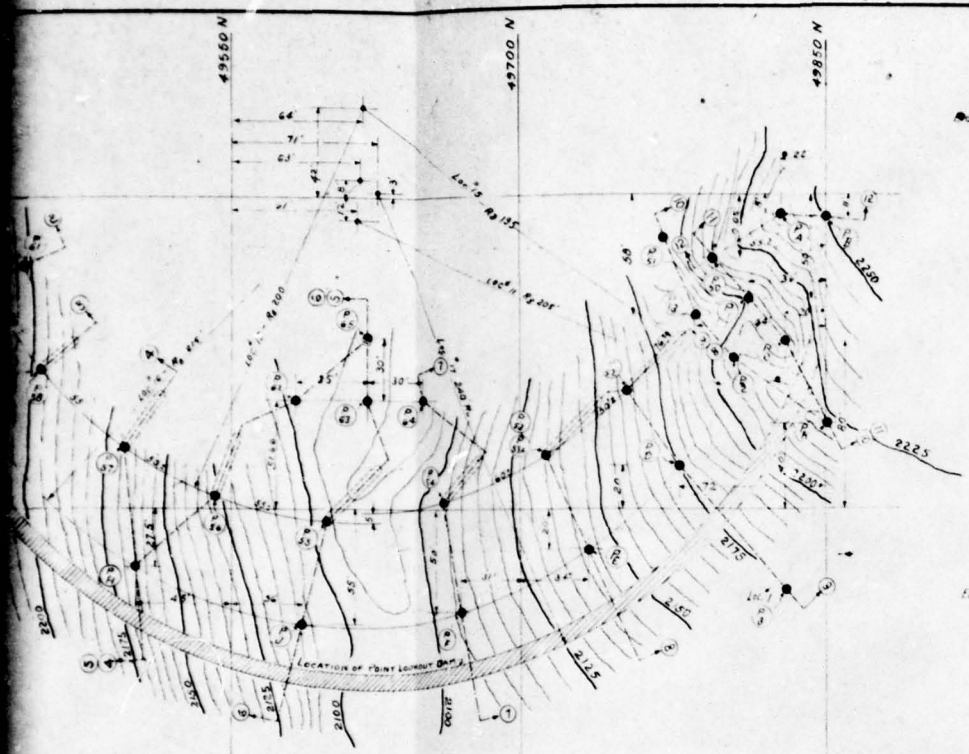
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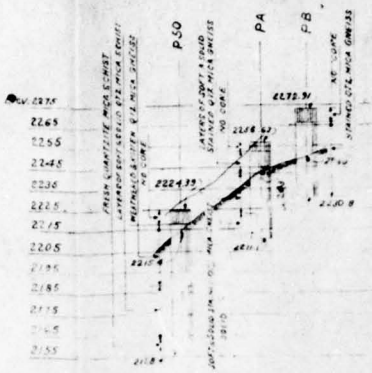


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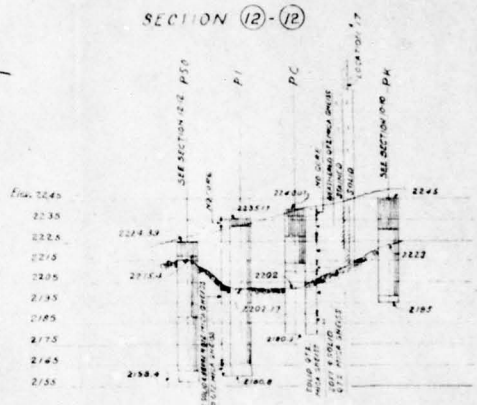


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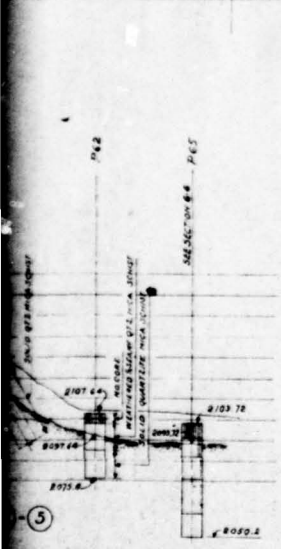
SHOWING SOUND ROCK CONTOURS ASSUMED FOR DESIGN
ACTUAL SOUND ROCK ELEVATIONS TO BE DETERMINED BY
INSPECTION OF FOUNDATION EXCAVATION BY THE ENGINEER

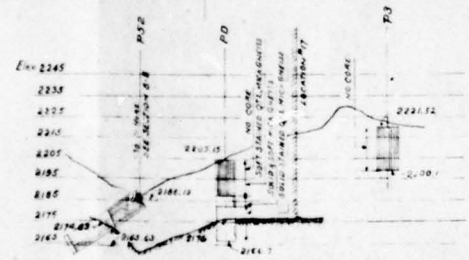


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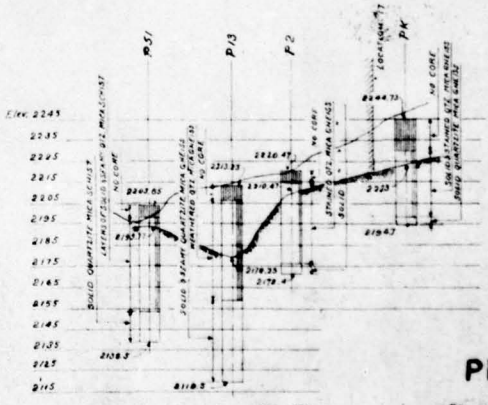


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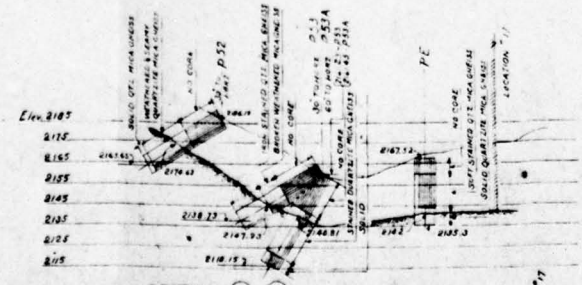




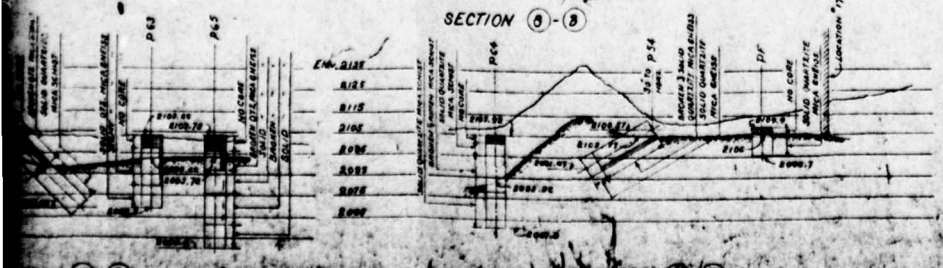
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SECTION (10)-(10)



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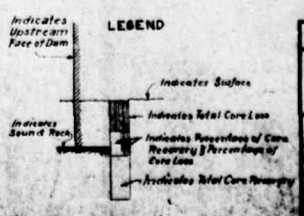
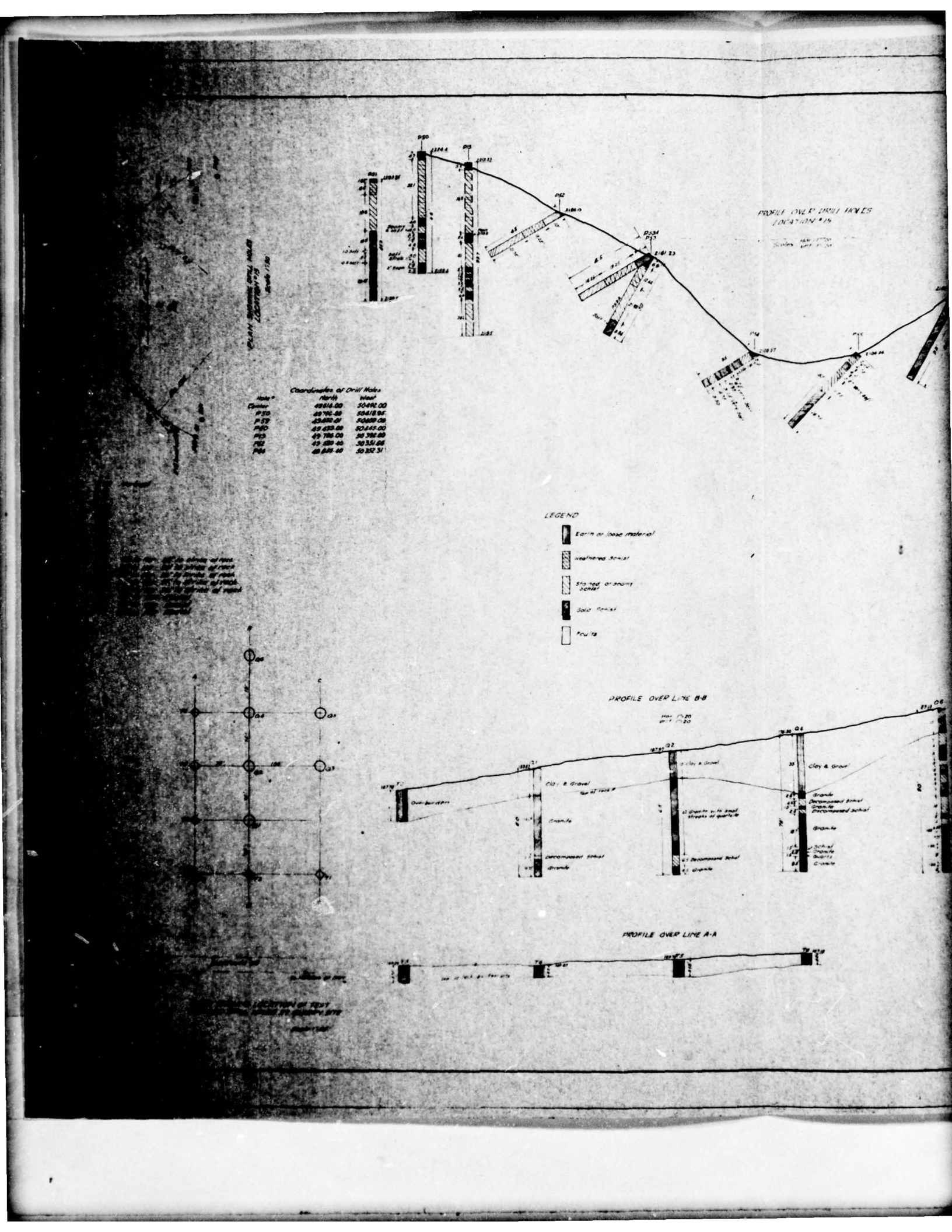


PLATE 8

PWA DOCKET Nº 1151-R

SHEET 9 OF 17 SHEETS

PINNACLES HYDRO-ELECTRIC PROJECT
CITY OF DANVILLE ELECTRIC DEPARTMENT
DANVILLE VIRGINIA
POINT LOOKOUT DAM
DIAMOND DRILL CORE BORINGS
CHAS. T. MAIN, INC. ENGINEERS
201 Devonshire St. Boston
REVISIONS



PLAN SHOWING LOCATION OF DRILL HOLES

Coordinates of Drill Holes

Hole #	North	West
Center	49816.00	50492.00
P50	49796.88	50418.96
P57	49882.01	50489.08
P60	49 693.00	50445.00
P53	49 786.00	50 382.80
P52	49 689.40	50 351.68
P54	49 849.40	50 352.31

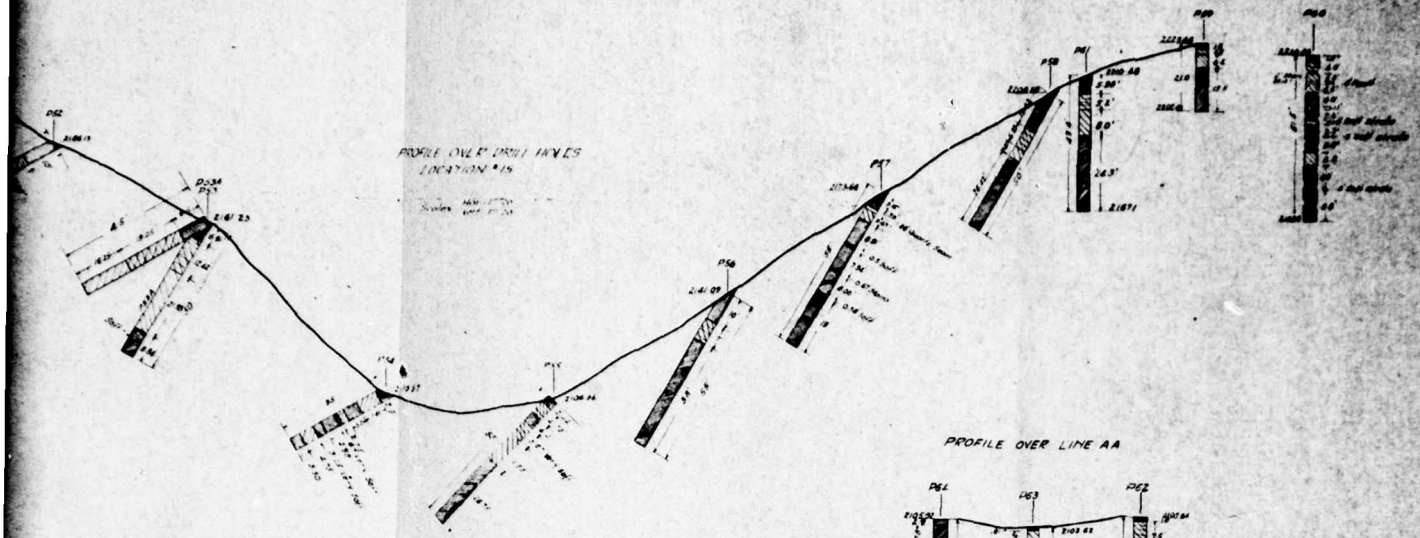
PROFILE OVER DRILL HOLES LOCATION #14

LEGEND

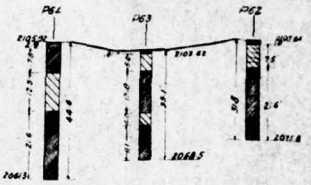
- Earth or loose material
- Weathered Schist
- Stratified or jointy Schist
- Solid Schist
- Faults

PROFILE OVER LINE B-B

PROFILE OVER LINE A-A



PROFILE OVER LINE AA

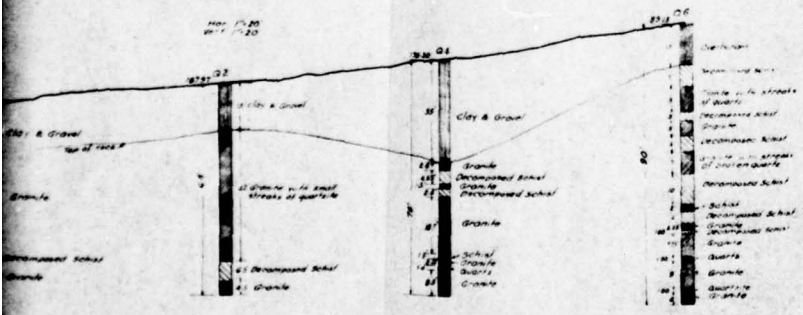




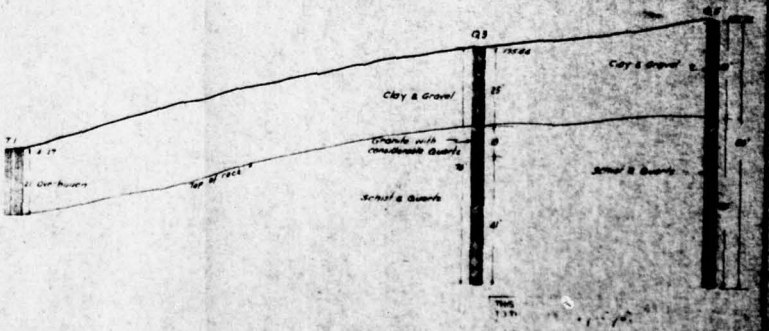
LEGEND

- Earth or loose material
- Weathered Schist
- Stratified or stony Schist
- Solid Schist
- Faults

PROFILE OVER LINE B-B



PROFILE OVER LINE C-C



PROFILE OVER LINE A-A



PWA PROJECT NO.

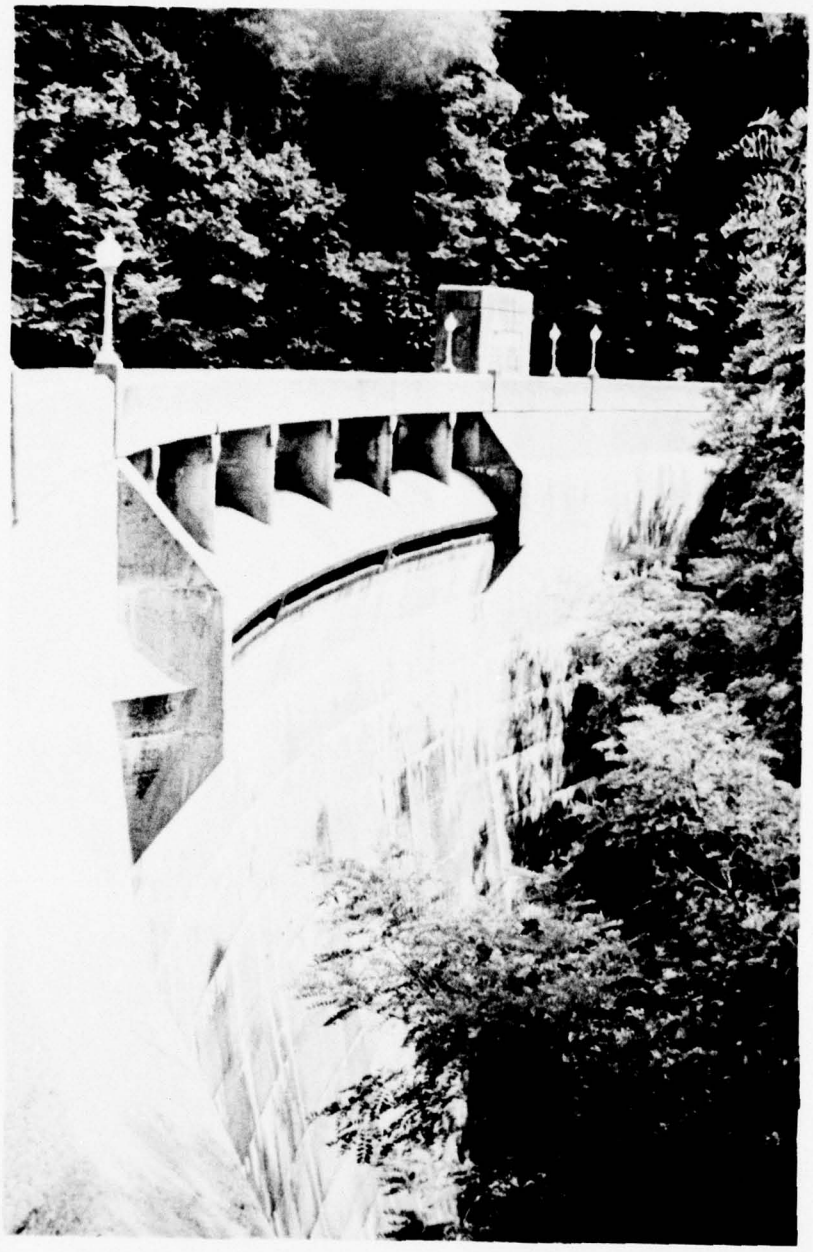
PINNACLES HYDRO-ELECTRIC
CITY OF DANVILLE ELECTRIC
DANVILLE
POINT LOOKOUT DAM
CORE BORING

CHAS. T. MAIN, INC.

PLATE

APPENDIX II

PHOTOGRAPHS



TOWNES DAM
NOTE LEACHING OF CALCIUM THROUGH CONSTRUCTION JOINTS

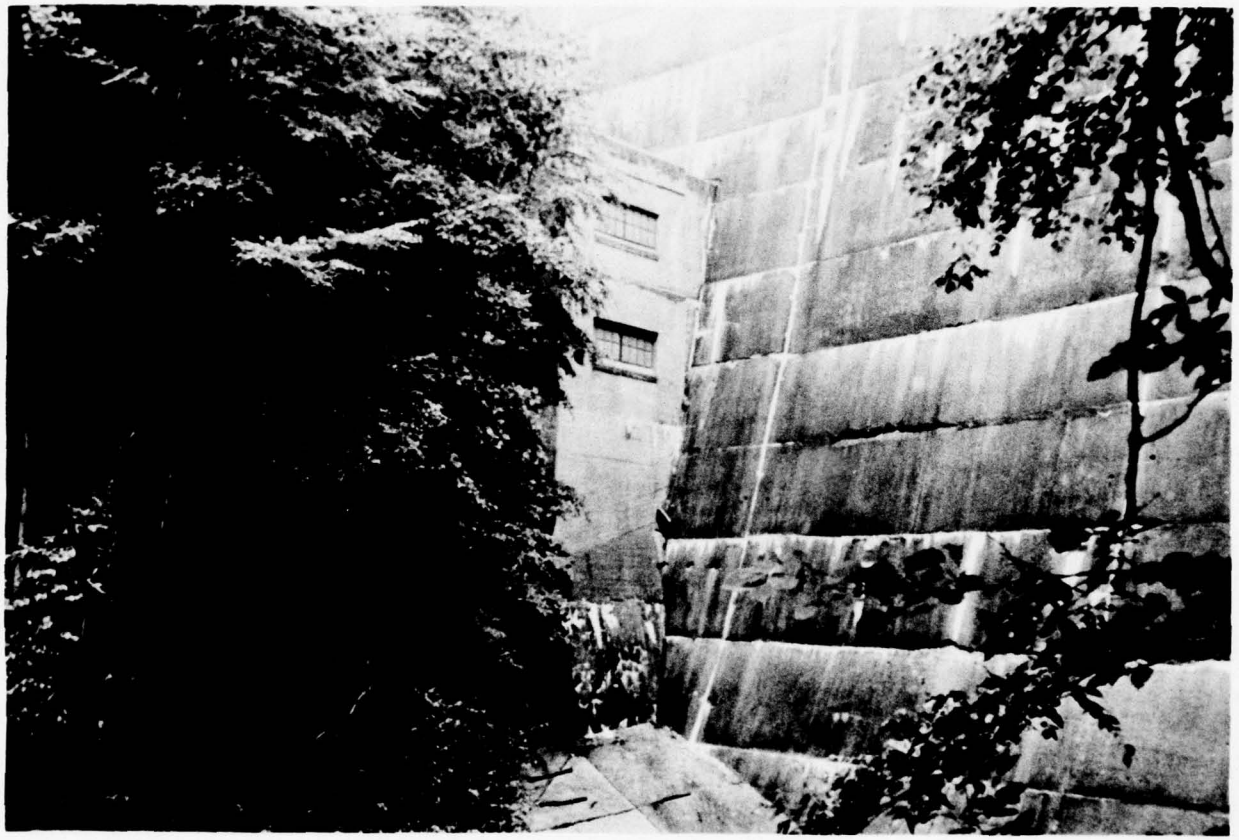
PHOTO 1



TOWNES DAM---VIEW OF STILLING BASIN
TAKEN FROM TOP OF DAM

PHOTO 2

PHOTO 3



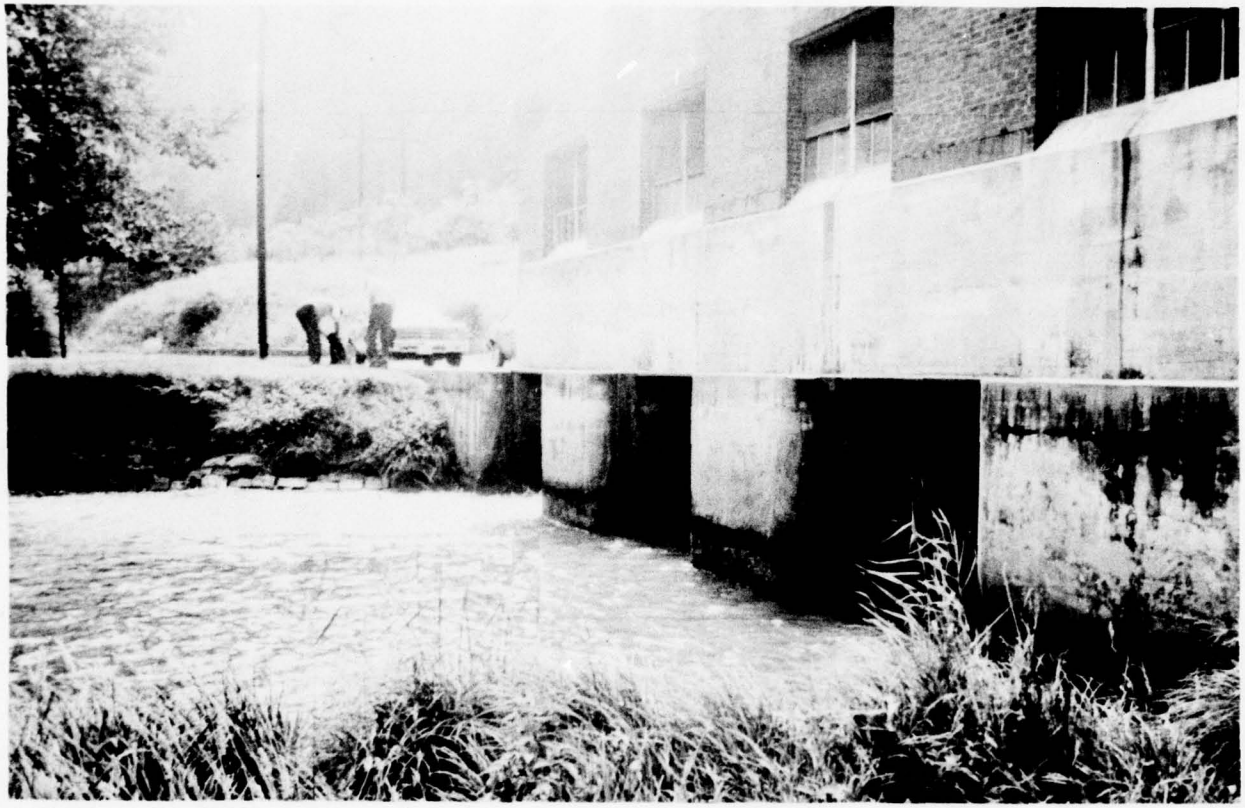
TOWNES DAM---OUTLET WORKS
TAKEN FROM LEFT WINGWALL OF STILLING BASIN



IOWMES DAM---72 INCH WOOD STAVE
PIPELINE TAKEN AT THE OUTLET WORKS
LOOKING DOWNSTREAM

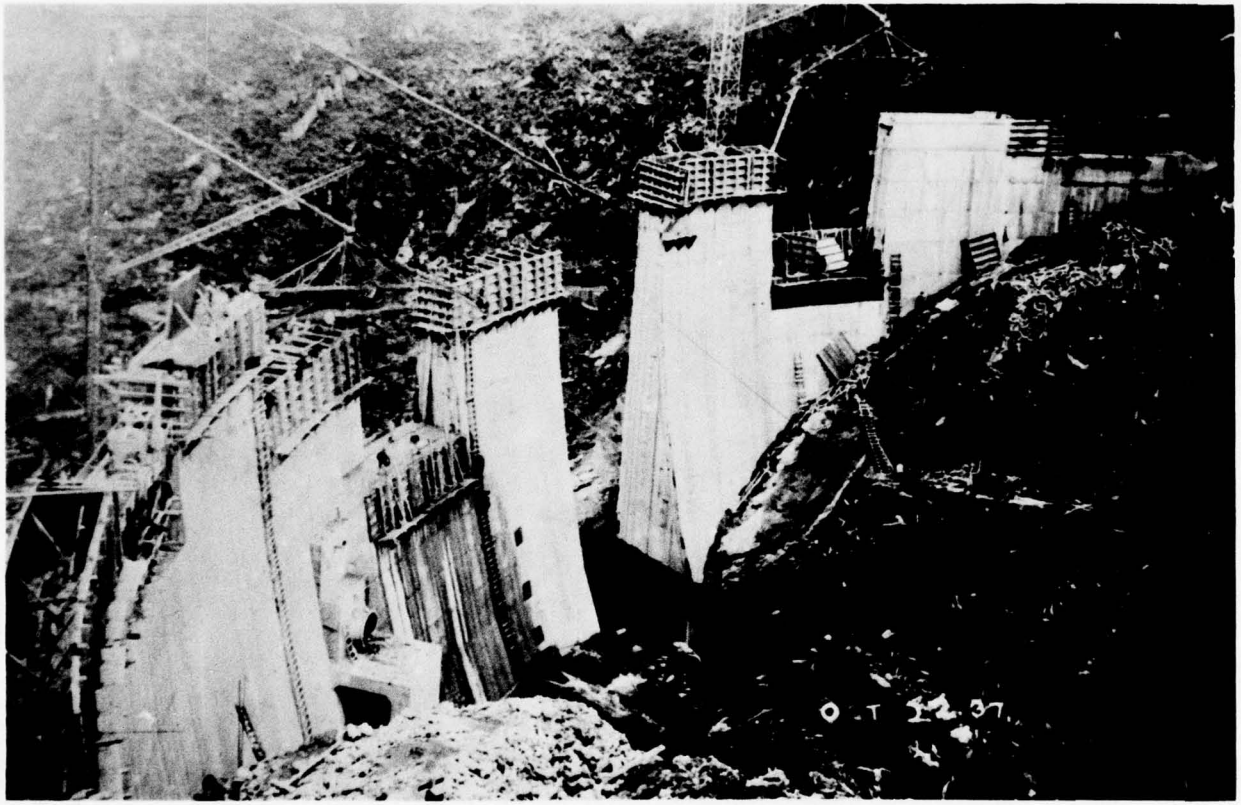
PHOTO 4

PHOTO 5



TOWNES DAM/PINNACLES HYDRO ELECTRIC
POWER PLANT---VIEWS OF DISCHARGE OUTLETS
FOR TURBINE GENERATORS

PHOTO 6



TOWNES DAM - 22 OCTOBER 1937



TOWNES DAM--MONOLITH JOINT OPEN 1/8"
AT DAM/ABUTMENT CONTACT

PHOTO 7

APPENDIX IV

GEOLOGY REPORT

APPENDIX III

FIELD OBSERVATIONS

PHASE I - FIELD OBSERVATIONS

Name of Dam: Townes Dam (VA 14102)
County: Patrick State: Virginia
Coordinates: Lat. 3641.2 Long. 3023.8
Date of Inspection: 27 July 1978
Weather: Partly cloudy with scattered thundershowers
Temperature: 80°F
Pool Elevation at Time of Inspection: 2214.8 MSL
Tailwater at Time of Inspection: 2100 MSL

Norfolk District Inspection Personnel:

Jeff Irving, Structural Engineer
Meade Stith, Structural Engineer
Lonnie Baird, Electrical Engineer
Henry Hammond, Mechanical Engineer
Bill Barker, Geologist
Jim Robinson, Hydrologist
Rob Schonk, Geotechnical Engineer (Recorder)

Other attendees:
Sam Stryker, City of Danville
Dave Lucado, State Water Control Board
Clyne Willis, Pinnacles Hydro Station Supervisor
Jesse Thompson, Talbott Dam Patrolman
John Bowman, Townes Dam Patrolman

1. GENERAL: At the time of the inspection, the level of the water in the reservoir was 6.2 feet below the level of the spillway.

2. CONCRETE ARCH DAM

2.1 Seepage: Seepage was noted at the junction of the dam and the left wingwall of the stilling basin. The seepage had left a red stain. Leeching of calcium from the concrete has stained the dam face white at many construction and monolith joints, however, most of these joints did not appear to be seeping at the time of inspection. (This was difficult to determine since rain began at the start of the inspection.)

2.2 Structure to Abutment Junction: Two suspected seepage areas were noted at the dam-abutment contact. The first area noted was on the left abutment approximately 40 feet below the top of the dam. Water was noted flowing at about 10 gpm from under the rock backfill 5 feet from the dam contact. Because of the location and coolness of the water, it is very probable the spring represents seepage and not runoff. It wasn't possible to determine the exact location of the seepage due to the fill along the dam contact. According to the dam

operator, the spring flow has not fluctuated. The second area is located on the right abutment approximately 50 feet below the top of the dam and 3 to 10 feet downstream of the dam contact. Water was noted flowing from 2 areas in the rock fill at at rate of approximately 5 gpm each. The temperature of the water was much warmer than from the spring on the left abutment and according to the dam operator the flow is variable irrespective of the reservoir level. For these reasons, it is felt the water is runoff and not seepage.

2.3 Drains: No drains were observed during the inspection.

2.4 Water Passages: In addition to the ungated spillway, there is a 70-inch wood stave outlet conduit and a 12-inch bypass line.

2.5 Foundation: The foundation along both abutments and channel section is fairly competent quartz, mica gneiss. The strike of the schistosity was measured at N 45° to 50° E with a tip of 50° to 80° to the southeast.

2.6 Concrete Surfaces: Only minor cracks requiring no remedial action were found. The concrete is in generally good condition.

2.7 Structural Cracking: There was no evidence of structural cracking.

2.8 Vertical and Horizontal Alignment: There were no apparent displacements. Poor formwork on the downstream face resembles horizontal displacement in places, however, no horizontal displacement was observed.

2.9 Monolith Joints: Leaching and calcium staining was noted along some of the monolith joints. The monolith joint located approximately 125 ft from the right abutment stairs appears to be open 1/8-inch. This condition occurs at the contact surface of the dam and abutment extending upward approximately 30 feet. No seepage was obvserved. (See Photo 7)

2.10 Construction Joints: Concrete formwork on the downstream face of the dam was poorly aligned with adjacent concrete. Cement mortar was placed over the construction joints (See Photo 3) probably to smooth the appearance of the dam. It is possible that the mortar "patchwork" was an attempt to stop seepage through the dam which can be seen in early photos. Most of these "patches" have spalled off of the dam. Calcium deposits are built up at construction joints from seepage through the joints.

3. OUTLET WORKS:

3.1 Intake Structure: The intake structure is submerged for normal pool elevation. It consists of a 70-inch intake and a trash screen. The trash screen can be raised or lowered by a motorized hoist.

3.2 Outlet Structure: The outlet structure consists of a valve house near the foot of the right abutment and a pipe line to convey the water under pressure from the valve house to the powerhouse. The water is carried through a 70-inch wood stave pipeline, 70-inch horseshoe shaped tunnel, 70-inch welded steel pipe and penstock ranging from 60 to 54 inches in diameter. The 54-inch penstock branches into three 12-inch lines that serve three turbines. The flow of water generates power and then discharges into the Dan River at a point 3 miles downstream of the Townes Dam.

3.3 Emergency Gate: There is no emergency gate.

4. UNGATED SPILLWAY:

4.1 Concrete Weir: The concrete weir appeared to be in good condition with no apparent spalls or cracks. 2 x 6 inch flashboards are in place on the spillway. Conversations with the dam operator indicates that the reservoir level rarely reaches the flashboards.

4.2 Bridge and Piers: The parapet walls and walkway appeared to be in good condition with no signs of concrete spalling or cracking.

5. INSTRUMENTATION

5.1 Monumentation/Surveys: There is a surveying platform and monument located 240 feet downstream of the dam. Eyebolts have been placed in the face of the dam for measuring deflection and pipes extend into the dam for measuring temperature.

6. RESERVOIR

6.1 Slopes: Reservoir slopes are estimated to be 30° or greater.

6.2 Sedimentation: Conversations with the engineers representing the City of Danville indicated sedimentation along the base of the dam. An underwater inspection of the dam was performed by Logan Diving, Inc. of Jacksonville, Florida on 4 and 5 April 1978. (Appendix V) Large tree limbs and sawdust were noted floating near the face of the dam. The reservoir should be cleared of debris.

6.3 Water Levels: There is a staff gage at the dam and the water levels are recorded every day at 7:30 AM and 3:30 PM. Water levels are also monitored remotely from the Pinnacles hydro-electic station.

7. DOWNSTREAM CHANNEL

7.1 Conditions: The stream bed itself is covered with vegetation due to infrequent use of the spillway.

7.2 Slopes: Slopes in the vicinity of the dam are approximately 50 degrees.

7.3 Approximate Number of Homes and Population: There are more than 25 homes located in the bottom of the gorge within three river miles of the dam.

APPENDIX IV

GEOLOGY REPORT

GEOLOGICAL REPORT
ON THE
PINNACLES HYDRO-ELECTRIC DEVELOPMENT
ON THE DAN RIVER
IN PATRICK COUNTY, VIRGINIA
P.W.A. PROJECT 1151-R

FOR
THE CITY OF DANVILLE
DANVILLE, VIRGINIA

JANUARY 1937

BY
WILBUR A. NELSON
Consulting Geologist

SUPPLEMENT
TO ENGINEERING REPORT BY
CHAS. T. MAIN, INC
BOSTON, MASSACHUSETTS

GEOLOGICAL REPORT
ON THE
PINNACLES HYDRO-ELECTRIC DEVELOPMENT
ON THE DAN RIVER
IN PATRICK COUNTY, VIRGINIA
P.W.A. PROJECT 1151-R

By

WILBUR A. NELSON

CONSULTING GEOLOGIST

INTRODUCTION

Field work in connection with this Report was begun in November, 1935, and was completed in November, 1936. From November, 1935 to July 1, 1936, several trips were made each month to the project. From July 1st to September 1st almost continuous work was done on this project.

Trips were made several times during each month to the different dam sites, which were studied and drilled during this period. In addition to a thorough checking of the cores of the drill holes, a detailed study was made of the geology of the dam site areas. Geological sections were obtained from surface outcroppings, from road



Pinnacles of the Dan

Photo by W.A.N.



Winter Scene in Gorge of Dan River at Point Lookout Dam Site

Photp by W.A.N.

cuts and from the study of the core drilling done in this region. Considerable time was spent in studying the underground water conditions in the narrow ridge forming the east abutment of the Big Bend Dam Site.

After September 1, 1936 two trips were made to the project, one in September and one in November, and considerable office work has been done in the preparation of this Report.

This Report is made for the City of Danville, Danville, Virginia, and will be supplementary to the Engineering Report made by Charles T. Main, Inc., on this same Project for the City of Danville.

LOCATION

The Point Lookout Dam Site and the Big Bend Dam Site are located in the extreme west central part of Patrick County, Virginia, in the gorge of the Dan River near the eastern edge of the Blue Ridge Mountains.

The location of the Point Lookout Dam Site is approximately 9½ miles slightly north of east of the town of Stuart, while the Big Bend Dam Site is approximately 7½ miles slightly north of east of the town of Stuart.

The nearest railroad to the dam site is a branch of the Southern Railway which stops at the town of Stuart.

Service roads have been built into both dam sites by the City of Danville. These roads connect with the main state highway at Cruzes Store on top of the Blue Ridge Mountains, about 5 miles roughly north of the two dam sites.

Patrick County is located in that part of Virginia known as the Blue Ridge Mountain Province, which borders on the east the Great Valley Region of Virginia.

The Point Lookout and Big Bend Dam Sites, and surrounding territory, are shown in the northeast quarter of the U. S. Geological Survey Topographic Sheet known as the "Stuart Quadrangle", a topographic sheet on the scale of 1 to 62500, having a contour interval of 20 feet. (See Plate 1).

GEOLOGY

The rocks comprising this part of Virginia are known as metamorphic rocks and belong to the great group of crystalline rocks designated as of pre-Cambrian Age.

All of the rocks outcropping and occurring around the two dam sites belong to this group of intensely metamorphosed and altered crystalline rocks. The rocks consist of a schistose series of gneisses of various compositions, some layers being graphitic, some sericitic, and some almost pure quarts. Most of the layers can be

designated a quartz mica gneiss, due to the large but varying amounts of mica and quartz comprising them. In addition smaller amounts of other rock-forming minerals occur in these layers. Interbedded in these series of quartz mica gneisses occur a number of layers of a rock known as greenstone, which is composed primarily of hornblend.

It is considered from an examination of thin sections of rock from the drill holes at both dam sites that the quartz mica gneisses of various compositions were originally sedimentary rocks, containing in some of the layers a small amount of calcium carbonate, but not sufficient to cause solution channels or danger from leakage due to a dissolving out of such calcium carbonate content. Interbedded with these altered sediments at the Big Bend Dam Site occur several layers of greenstone. These layers of greenstone, now composed almost entirely of hornblend, may have originally either been sills of basic igneous rock which came into this sedimentary series in pre-Cambrian Times, or may have been originally just other layers of the sedimentary series which contained a higher calcium content which, when subjected to the metamorphic forces to which this region has been subjected in the past, could have been altered to the present type of greenstone rock which occurs in this section.

Irrespective of the original origin of these greenstone layers, they are now dense layers of rock made up almost entirely of hornblende and lying parallel to the quartz mica gneisses of varying composition which occur between them.

All of the rock series at both dam sites show varying degrees of foliation and schistosity. Strike and dip observations taken on these rocks relate entirely to the planes of schistosity, which are prominently developed throughout this region. It is, however, thought that the schistosity developed at both dam sites is locally at least parallel to the old original bedding planes of these highly altered pre-Cambrian sedimentary rocks.

A study of these rocks shows at least three periods of folding, which can be distinctly recognized at the present time, but it is not known how many other periods of folding ante-dated the three that can now be distinguished. These distinct periods of folding have naturally fractured the rocks in many different ways and have developed planes of weakness along definite lines in certain places.

On the eastern front of the Blue Ridge, about five miles east of the dam site area, there is evidence of thrust faulting and also normal faulting, but as these lines of disturbances roughly parallel the front of the Blye Ridge, which has a direction of approximately N.50°E.,

these lines do not cut into dam site and reservoir areas.

At the Big Bend Dam Site, the service road built into the selected dam site, shows in one of the cuts made in September, 1936, the presence of a normal fault, which is almost vertical and approximately parallel to the top of the ridge just to the west of the Dan River at Dam Sites Nos. 21 and 22. Although this fault is only several hundred feet from the Dan River, its direction is such as not to affect adversely the dam sites just mentioned.

A detailed study of all of these complicated geological factors has been necessary in order to determine where proper foundation conditions exist for the location of dams at Point Lookout and in the Big Bend region.

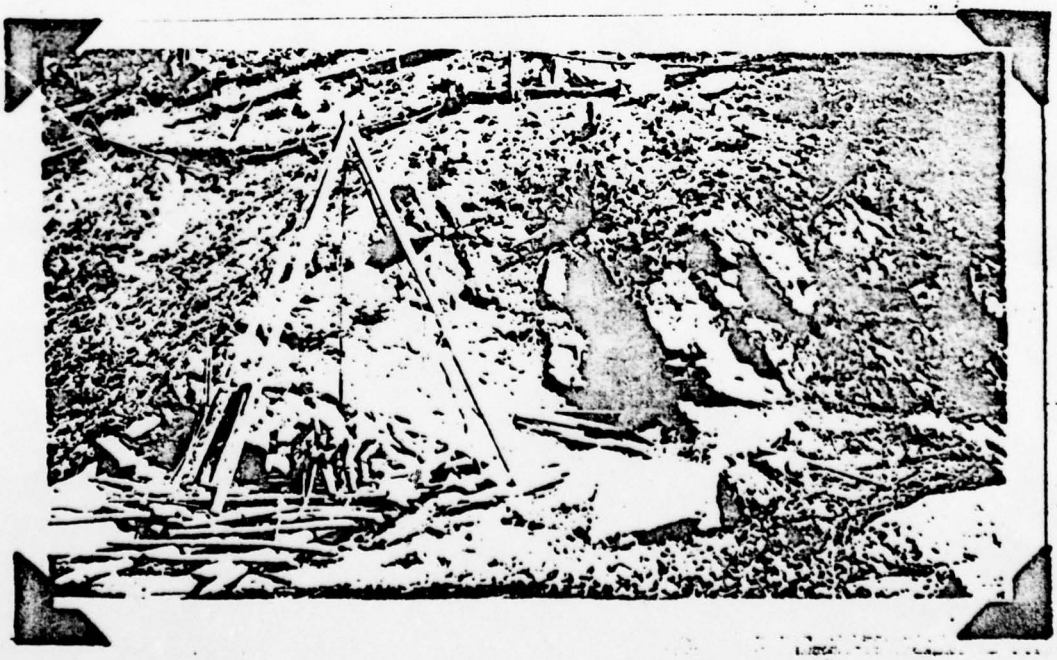
GEOLOGY AND STRUCTURE OF ROCKS AT POINT LOOKOUT DAM SITES

At the Point Lookout Dam Sites, the pre-Cambrian crystalline rocks consist of a series of quartz mica gneisses formed by metamorphism of pre-Cambrian sediments. Certain of these gneissic layers contain about three or four percent calcium carbonate, which in thin rock sections is shown now to be in the form of crystalline calcium carbonate acting as a cementing material in isolated and not in contiguous places.



Looking Downstream. Dam Site No. 16 in the Foreground

Photo by W.A.N.



Drilling at Dam Site No. 16 on West Bank of the Dan River

Photo by W.A.N.

An analysis of a large sample of rock from the service road to the Point Lookout Dam Sites, at a point about half way from the top to the bottom of the gorge on this road, shows that particular layer of rock to contain 3.72 percent calcium carbonate. This analysis was made of a sample of stone collected by the Consulting Geologist, and analyzed by Froehling and Robertson, Inc., of Richmond, Virginia.

The original location at Point Lookout was known as Dam Site No. 1. It was located near the center line of the rock slide from the Point Lookout bluff, and was condemned on geological grounds discussed further on in this Report.

The Consulting Geologist recommended a site to be drilled, which was originally designated as Dam Site No. 1. This site was downstream about 25 feet from the downstream side of the rock slide just mentioned. This is practically the same site as was designated later Dam Site No. 16 by the Engineers. Instead of drilling the site recommended by the Consulting Geologist a site, known as Dam Site No. 5, about 75 feet downstream from the junction of Round Meadow Creek and the Dan River and approximately 200 feet upstream from the rock slide area above-mentioned, was selected by the Engineers and drilling was started without the knowledge of the Consulting

Geologist, although this stretch of the river had been previously condemned by the Consulting Geologist. Details of the geological reasons for condemning this area are given later on in this Report.

Dam Site No. 17 is about 25 or 30 feet upstream from Dam Site No. 16, the site selected by the Designing Engineer. Dam Site No. 16, as already mentioned, is practically identical with the original Dam Site No. 11, the two sites being only 3 or 4 feet apart. Dam Site No. 16 is discussed after the "Character and Weathering of Schistosity" and before Dam Site No. 17, as the nearest drilling done to Dam Site No. 17 is on the location of Dam Site No. 16, and the information on which the location of Dam Site No. 17 is based is only the partial information obtained from drilling Dam Site No. 16.

Character and Weathering of Schistosity for the Point Lookout Dam Sites: For all of the Point Lookout Dam Site locations the general strike of the schistosity of the quartz mica gneiss in this region is from N. 45° E. to N. 55° E., and the dip of the schistosity varies from 50° to the South East to 80° to the South East. A cross section taken at right angles to the strike of the schistosity and showing the general dip of the schistosity

at the Point Lookout Dam Site No. 15 is attached. This cross section shows that the schistosity is steeper in the bottom of the gorge than it is on the sides of the gorge at the level of the top of the proposed dam.

A detailed study of the core drill holes drilled at Dam Site locations 11, 15 and 16, along this section, shows that lines of weathering, that is the lines along which the ground water has percolated downward and caused oxidation and weathering of the rocks, follows definitely the lines of schistosity and that these lines of weakness extend down on the west side of the Point Lookout Dam Site down to at least the level of the stream.

Placing Drill Holes P-A, 50 and 13 in this same plane, it can be shown that the line of weathered rock occurring in Drill Hole P-A at from 41'1½" to 41'6½ occurs again in the bottom of Drill Hole No. 50 at an elevation of 2160 feet above sea level, and again occurs in the bottom of Drill Hole 13 at an approximate elevation of 2121 feet where the core lose at this elevation amounts to 2½". In the same way soft, weathered rock layers paralleling the schistosity are found to extend down to approximately this same depth by using data in Holes P-13, P-52, P-53 and P-53A, drilled for Dam Site locations 11 and 15. In all of the drill holes on the west abutment of the Point Lookout Dam locations 11, 15 and 16, soft, weathered core

rock is found within a few feet of the bottom of each hole. In Drill Hole No. 55, for Dam Site No. 15, a 45° hole drilled from near the center of the river under the western bank of the stream, we find iron stained joints at a depth of 48'1", or just 7 feet above the bottom of this hole, showing that at a depth of approximately 2075 feet we have rocks showing weathering and rocks through which leakage may occur.

Dam Site No. 16: Thin sections of the rock from the drill cores nearest to the proposed dam location at Point Lookout, which were drilled for Dam Site No. 16, show only very minor amounts of calcium carbonate occurring in the rocks at the dam site. Thin rock sections were made from an average part of the core from Drill Holes P-B, P-C, P-D, P-F, and P-J. Examinations of these thin sections were made under the microscope to determine the amount of calcium carbonate present in crystalline form. In Drill Hole P-B the rock, which was shown to be a quartz mica gneiss with definite banding of quartz and mica, showed a few crystals of calcite in one part of the thin section, as well as other rock-forming minerals in minor amounts. There was not enough calcite present to make the rock porous through the action of ground water in dissolving out the calcite.

Two thin rock sections were made from the core of Drill Hole P-C, one near the bottom of the hole and one about half way down. The thin rock section from the bottom of the hole showed no calcite, but the thin section made from the core about halfway down in this hole showed several calcite crystals acting as a cementing material between crystals of quartz. The rock in both of these thin sections was seen to be a quartz mica gneiss containing some garnets, and a number of other rock-forming minerals in minor amounts.

The thin rock section from the core of Drill Hole P-D showed this rock to be a quartz mica gneiss with one small aggregate of calcite as a cementing material between quartz crystals, as well as other rock-forming minerals in minor amounts. Again this microscopic investigation showed that there was not sufficient calcite present in the rock to cause rock porosity by the solution of this mineral.

The thin section from the core of Drill Hole P-F showed several small crystals of calcite with mica and quartz. The rock was seen to be a quartz mica gneiss, with some pyrite present and also a few garnets, as well as other rock-forming minerals in minor amounts. This thin rock section showed more calcite than was seen in any of the other thin sections made from the cores of the

drill holes at the Point Lookout Dam Site. However, the amount of calcite in this thin rock section was not sufficient to cause any trouble by solution through the action of circulating ground water in this type of rock as the several calcite crystals were all more or less segregated together in one part of the thin rock section.

The thin rock section made from the core of Drill Hole P-H did not show any calcite. The rock was again seen to be a quartz mica gneiss, with alternating layers of mica and quartz containing garnets and pyrite and also additional rock-forming minerals in minor amounts.

The thin rock section made from a sample of the core of Drill Hole P-J showed one or two small calcite crystals in the mixture of quartz and mica. This thin rock section also showed some pyrite, as well as additional rock-forming minerals in minor amounts. The rock was seen to be a quartz mica gneiss.

In all of the thin rock sections examined and mentioned above, the calcite in no case was found to be scattered throughout the entire thin rock section, but, wherever it was found, the calcite occurred only in isolated spots in the rock and then only in minor amounts in crystalline form with the quartz and mica and other rock-forming minerals. In only one drill hole in the Point

Lookout area, which was one of the holes drilled for Dam Site location No. 15, was any calcite seen in sufficiently large crystal form to be recognized by the eye. In Drill Hole P-60, at 21' depth, one calcite crystal at least ½ inch across was seen in the drill core.

Dam Site No. 17: The final location of the Point Lookout Dam, Dam Site No. 17, made by the Designing Engineer, has moved the dam location upstream approximately 30 feet from Dam Site No. 16, the area in which the final drilling was done, and has placed the west side of this dam on the west side of the gorge in the edge of the rock slide from the high Point Lookout bluff.

Although core drilling has been done on the downstream side of this final location none has been done on the upstream side, except Hole P-K for abutment purposes, so that no definite information is available as to the thickness of the rock slide on the west side of the gorge where the dam is now located. As this location is on the edge of the rock slide, it is logical to assume that the slide has a less thickness than it would have near its center part, another 90 to 100 feet upstream.

As each rock slide from a bluff acts in a slightly different manner, there is no way of definitely determining the amount of rock debris lying on the west side of the

gorge at the present dam site location unless additional drilling is done at this point, and any subsurface contours made to solid rock at this point are purely hypothetical. The same is true of any contour lines drawn on the top of unweathered rock on the east side of the present dam location, as no drill holes have been dug except on the downstream side of this location, the nearest ones being about 30 feet away from the center line of the proposed location. As the schistosity of the rock on this side cuts across the dam location at about an angle of 45° we have again no way of determining the depth to solid rock except by deductions. Therefore, no maps showing contour elevations of solid rock are given in this Report.

At Dam Site No. 16, soil or loose rock covering varies from 14'4" at Hole P-K, approximately 30 feet upstream from the edge of the rock slide, to 6 feet along the hillside line of the dam site on the west side of the gorge, while on the river banks the soil overburden is about 4 feet. On the east side of the gorge the overburden to weathered rock varies from 4'8" to 7'1" along the line of the dam site. So it is logical to assume that at Site No. 17 one will find at least this much overburden and probably more.

The depth from the surface to the unweathered rock on the hillside line of Dam Site No. 16, on the west side of the gorge, varies from 26'3" to 35'7", but in Hole P-K, an abutment hole on the west side of the gorge, the depth to good rock is 44'3½". Taking Holes A, C, and K, which in order named are located toward the edge of the rock slide, it is noted that the depth to good rock increases, being 34'4" in Hole P-A, 35'7" in Hole P-C, and 44'3½" in Hole P-K. This indicates that it will be approximately 40 feet to unweathered rock at the west end of Dam Site No. 17.

If the depth to unweathered rock increases regularly upstream from each of the other holes drilled on the line of Dam Site No. 16, it might be estimated that the depth to unweathered rock along the arc of Dam Site No. 17 on the west side of the gorge would vary from 30 to 35 feet, plus any additional depth due to the edge of the rock slide. These facts are given as nothing more than an estimate based on the facts given above.

On the east side of the gorge the depth to good rock varies from 16'2½" to 25'9", these data being obtained from three holes drilled on the line of Dam Site No. 16. A study of the surface topography on this side of the gorge would indicate that the depth to weathered rock on the line

of Dam Site No. 17 would be at least as deep as shown at Site No. 16, and there is always the possibility that it might be a few feet deeper.

In order to take care of any leakage which may occur under the Point Lookout Dam Site, selected by the Designing Engineer, on its west side and under the river part of the dam, grout holes should be put down to at least an elevation of 2,070 feet above sea level as the strike of the schistosity does not parallel the dam but extends across the line of the dam at an angle of approximately 45°, which would permit of leakage from the reservoir along such lines under the dam into the river below. (See page 10).

From a geological standpoint, it is considered that a site about 30 to 50 feet downstream from the final location is better than the location decided upon by the Designing Engineer, unless additional drilling had been done to prove all of the facts in regard to this final location to be definitely satisfactory. A location 30 to 50 feet downstream would have kept entirely away from the edges of the rock slide on the west side of the gorge, and one would be dealing with definitely known rock conditions.

Other Dam Sites Investigated at Point Lookout: The original dam site at Point Lookout, Dam Site No. 1, was located in the narrowest part of the gorge, which happened to be in the center of the rock slide from Point Lookout, this heavy slide of loose rock having filled in the gorge and made the stream narrowest at this point. Detailed geological work showed, from a study of the strike and dip measurements of the outcrops of rock on the west side of the gorge, that none of these rock outcrops were in place, as the schistosity of these rock outcrops when plotted on the map showed a fan-shaped pattern. Although the geological investigation showed definitely the presence of this rock slide, the Engineers wished to drill several holes at this point in order to prove the correctness of the geological facts, and, therefore, Drill Holes P-1, 2 and 3 were drilled in the rock slide area and showed that the geological facts as stated by the Consulting Geologist were correct.

Dam Site No. 5: The site recommended by the Consulting Geologist as the one then to be drilled was designated as Dam Site No. 11 and was about 25 feet downstream from the rock slide. This is practically the same site as was later given the Number 16 by the Engineers. Instead of

drilling this site, a new site, known as Dam Site No. 5, about 75 feet downstream from the junction of Round Meadow Creek and the Dan River, and approximately 200 feet upstream from the rock slide area first drilled, was selected by the Engineers and drilling was started without the knowledge of the Consulting Geologist, although previously this stretch of the river had been condemned by the Consulting Geologist, on geological grounds as being unsuitable for the location of the Point Lookout Dam. Nineteen holes were drilled on this location, none of which gave information of any geological value, as all of this drilling resulted in only condemning a site which, as stated above, had been condemned by the Consulting Geologist. All of these holes were unnecessary and none of them should have been drilled.

At this particular site, the schistosity of the rocks paralleled the thread of the Dan River, and just upstream from this site the Dan River turned sharply to the southeast at the point where Round Meadow Creek came in from a direction slightly west of north, so that the schistosity of the rocks on the east side of the gorge cut across the Dan River above and below the proposed location and made leakage through the abutment of a proposed dam at this point too dangerous and made the abutment conditions too bad to consider this site. On the

west side, the schistosity likewise cut through back of the abutment from Round Meadow Creek above this proposed site into the rock slide on the west side of the gorge below this site, again making leakage from the reservoir through the west abutment into the stream below this proposed site a strong possibility. All of these conditions were readily worked out by the detailed geological investigations made by the Consulting Geologist and were the reasons for condemning this site without drilling.

Summary of all Sites Investigated at Point Lookout: When the geological studies of this region were completed it was determined that the only satisfactory location, from a foundation standpoint, for the dam site was in that section of the river below Dam Site Location 16 and above Dam Site Location 15, a distance of approximately 90 feet and this fact was reported to the proper officials of the City of Danville and to Charles T. Main, Inc.

In the section of the river between Dam Site No. 15 and Dam Site No. 17, the strike of the schistosity of the rock which varies from N. 45°E., to N. 50° E., crosses the thread of the river at an oblique angle, so that any leakage along lines of weakness paralleling the planes of schistosity will be much more easily stopped

than where the schistosity parallels the thread of the river above the rock slide, the lower edge of which is at Dam Site No. 17. In addition to this fact the rock outcrops are more massive and more continuous on both sides of the river in the section selected than in the sections above and below. Within this section, between Dam Site Locations 15 to 17, a distance of approximately 90 feet, it is desirable, as already mentioned, to stay slightly downstream from Dam Site No. 17, so as to get away from the edge of the rock slide of the high bluff. Recommendations consistently have been made throughout the progress of this work that a site slightly downstream from 17 and upstream from 15 be selected as the best site from a geological standpoint.

Dam Sites 11, 15, 16 and 17 are all located very close together, Dam Site No. 15 being approximately 50 feet downstream from Dam Site No. 16, and Dam Site No. 16 being approximately 25 feet downstream from Dam Site No. 17. Dam Sites 11 and 16 are at almost the same or identical locations, as they are located only 3 or 4 feet apart. The drilling in connection with these four dam sites was started at first for Dam Site No. 11. Due to the fact that the Designing Engineer did not consider that the western abutment was satisfactory, drilling was

moved downstream to Dam Site No. 15, then upstream to Dam Site No. 16, and Dam Site No. 17 was finally located approximately 25 feet upstream from the drilling done for Dam Site No. 16.

RECOMMENDATIONS

Recommendations on the Point Lookout Dam Site, as well as the Big Bend Dam Site, are given at the end of the entire Report.

GEOLOGY AND STRUCTURE OF ROCKS AT BIG BEND DAM SITES

Original Big Bend Dam Site at Big Bend (Site A): At the original Big Bend Dam Site (Site A) the pre-Cambrian rocks consist of a series of quartz mica gneisses. At this point the general strike of the schistosity of these gneisses is from N. 45° E., to N. 60° E., and the dip of the schistosity, which is to the southeast, varies from 60° to 80°.

The original Big Bend Dam Site was located on the Dan River just below the mouth of Ivy Creek. The first geological investigations were made at this point and showed that the west abutment of the dam was against a long narrow ridge formed by the Big Bend of the Dan River, while the east abutment of the dam was against a narrow ridge lying between the Dan River and Ivy Creek. Both of the abutments to this propo sed dam were on narrow ridges where leakage conditions are undoubtedly bad. The schistosity of the gneissic rocks was at right angles to both of these ridges, so that the lines of weathering along the planes of schistosity extend across the narrow point of both of these ridges and would undoubtedly permit leakage from the reservoir into the Dan River below the proposed dam site. This would have called for excessive grouting on both of

these narrow ridges, with also the possibility that such grouting would not have stopped the desired amount of leakage. These detailed studies of the geology at this original site showed the geological conditions were so unfavorable for the location of a dam the height desired by the Engineers that this site was condemned and abandoned.

DamSites 1 to 8, All Downstream from Big Bend: Following the request of the Engineers that the first site below this point on the Dan River where geological conditions were satisfactory would be the point where they would prefer to locate a dam, a study was started of the upper Dan River Gorge. Sites from 1 to 8 were investigated (see attached map for locations). Cross sections of the gorge made by the Engineers showed that the stretch of the river at Sites 5 and 6 were narrowest. Detailed geological investigations were made at and between these points, which were only a few hundred feet apart. The final locations, known as Dam Sites 21 and 22, are approximately at the location of original Site No. 5.

Dam Sites Nos. 5 and 6, Approximately 2 Miles Downstream from Original Big Bend Site: In the stretch of the river at original locations 5 and 6, a study of the detailedgeology shows that the crystalline pre-Cambrian rocks consist of a series of quartz mica gneisses of varying composition

interbedded with beds of greenstone composed mostly of hornblende. The strike of the schistosity varies from N. 45° E., to N. 60° E., and the dip of the schistosity varies from 50° to 85° to the southeast. The strike of the schistosity extends across the Dan River at almost right angles, and the dip of the schistosity, which is in general almost vertical, is downstream.

Dam Site No. 1 (new series of numbers) Approximately Same as Dam Site No. 6: The best site in this stretch of the river from a geological standpoint is what is shown as Site No. 1 on the attached map showing the general geology of this area. This site is, from a geological standpoint, the best of the sites studied, as both the east and west abutments of this proposed dam are against the sides of the gorge where the bends of the river do not make narrow ridges and where no leakage would occur around their abutment. Due to the cross section at this point being greater than at the other points studied the Engineers considered it was necessary to attempt to find a narrower cross section where geological conditions were satisfactory, or where a study of these conditions would show that any weakness in the foundation or abutments could be taken care of by proper engineering practice.

The narrowest cross section on this section of the river was upstream several hundred feet from Site No. 1,

and to this narrowest cross section the designations Dam Sites Nos. 21 and 22 were given. Between these two sites, 1 to 22, a distance of approximately 550 feet, several drill holes were put down on the west side of the gorge at the then proposed reservoir height to see if satisfactory cores could be obtained which would show the presence of a type of unweathered rock which would suit the Designing Engineer for abutment purposes. Although two of these holes showed considerable good rock in the drill cores and there was also a 15 foot exposed ledge of hard quartz mica gneiss, it was not considered by the Designing Engineer that sufficient good rock was present in this stretch of the river for satisfactory abutments. Since then the blasting done for the service road into the dam site has shown for a distance of 250 feet downstream from the west abutment of Dam Site No. 22 quartz mica gneiss which is considered satisfactory for abutment purposes by the Consulting Geologist.

Dam Sites Nos. 21 and 22 (new series of numbers) Approximately same as Dam Site No. 5: Drilling was then concentrated at Dam locations 21 and 22 (the axes of these two locations cross on the east bank of the river) although the east abutment of both of these dam sites is against a narrow ridge made by the bend of the Dan River where geological conditions are most unfavorable and where leakage

will undoubtedly occur, which will have to be stopped by proper engineering practice.

Drilling at Dam Sites Nos. 21 and 22 showed satisfactory foundation conditions to exist along the arc of either of these dam sites, and satisfactory abutment conditions to exist at the west abutment of both sites.

Final drill holes put down to determine the foundation conditions for Dam Site No. 22 showed overburden to vary from two feet to sixteen feet, and the top of unweathered rock to vary from six feet below the surface to 54 feet below the surface, the greater depths to the unweathered rock occurring on the upper portion of the arc on the west side of the river, where, although the rock is deeply weathered, abutment conditions are excellent.

The entire west bank of the river between Dam Site No. 1 and Dam Site No. 22 is more deeply weathered than on the east side, and depths to unweathered rock are much greater than on the east side of the gorge. Below the first unweathered rock there occur weathered schistose rock in narrow planes parallel to the schistosity, which zones will continue below the dam foundations and should be grouted if excessive leakage is to be prevented.

However, the geological conditions along the arc of Sites Nos. 21 and 22 and the character of rocks along these arcs are considered satisfactory for dam foundation purposes.

Dam Site No. 22: Thin rock sections were made of cores of holes drilled along the arc or in close proximity to the arc of Dam Site No. 22. The thin rock section made from part of the core from Drill Hole B-222 showed the rock to be a quartz mica gneiss containing much pyrite as an impurity, and also other rock-forming minerals. No calcite was present.

The thin rock section from part of the core of Drill Hole No. 223 was in what is known as greenstone and showed that this rock was made up almost entirely of thin crystalline hornblende, with accessory rock-forming minerals.

The thin rock section of part of the core of Drill Hole No. B-226 was also in greenstone and showed that this rock was of the same character as rock in Drill Hole No. B-223.

The thin rock section from part of the core of Drill Hole No. B-224 shows this rock to be primarily a mica schist, full of small drag folds and having considerable pyrite following these small folds. No calcite is present but quartz and garnets do occur, as well as other rock-forming minerals.

The thin rock section from part of the core of Drill Hole No. B-230 showed this rock to be a quartz mica gneiss, banded with graphite gneiss. It contains much

so small that it would not affect the solubility of these rocks when considered from the standpoint of dam foundations.

The schistosity of these crystalline rocks appears to be roughly parallel to the original bedding of these rocks. These rocks, with the exception of the greenstone, are of a type that would weather more deeply than the quartz mica gneisses found at the Point Lookout Site which contain, as a rule, more quartz although there are one or two 10 or 15 foot layers of the quartz mica gneiss at the sites being discussed which contain much quartz and are very massive and resistent to weathering. They show up beautifully in the cuts of the service road which was completed in September, 1936, to the Big Bend Site.

Dam Site No. 22, which it is understood is the final selection of the Designing Engineer for the location of the so-called Big Bend Dam, lies entirely in quartz mica gneiss of varying composition. The west abutment of this dam is in quartz mica gneiss and approximately 50 feet from the upstream edge of a massive greenstone layer, while the east abutment is approximately 25 feet from the upstream edge of this same massive greenstone layer.

These statements are based on the location of the dam as shown on Exhibit G, Sheet I of the Pinnacles Hydro-Electric Project, with a reservoir level elevation of 2532 feet.

Detailed Studies of Narrow Ridge on which East Abutment of Dam Site No. 22 is Located: The location of the east abutment of the dam is of considerable importance, as this abutment is located on a long, narrow ridge through which leakage will undoubtedly occur from the reservoir formed by the dam into the river below unless this leakage is taken care of by proper engineering practice. Between this east abutment and the top of the narrow ridge which occurs on this side of the river at Drill Hole No. 302 there are several hundred feet of rock through which leakage will occur, as the water table in this ridge between the abutment and to within a few feet of Drill Hole No. 302 is below the elevation of the proposed reservoir water level.

Drill Hole No. 301, put down on the top of the ridge near the east abutment and in the greenstone above-referred to, had a bottom elevation of approximately 2510 feet and no water remained in this hole although it was tested continuously throughout the entire summer and fall of 1936 to see if it contained water at any time. This greenstone is a dense rock, fractured by jointing produced through the folding forces to which all of these rocks have been subjected.

A cross section of the ridge showing this hole and eight other drill holes which were put down along the crest of this ridge is attached, which shows the level of



Gorge of Dan River
Photo by W.A.N.



Narrow Ridge forming East Abutment of Dam Sites 21 and 22
Photo by W.A.N.

the water table on July 28, 1936, on August 17, 1936, and on September 21, 1936, which later date was the date on which the lowest water table elevation was reached, for in October the water table started to rise slightly.

In addition to the water table, the attached cross section shows the depth to hard rock and the depth of soil overburden.

Nine drill holes were put down along the top of this ridge as it was known that the geological conditions along this ridge at the east abutment of the proposed dam were very poor. It should be noted that the water table line, given on July 28, 1936, was only between Holes No. 303 and 302. From Drill Holes No. 302 to 301 no data was available on the top of the water table, as on that date the top of the water table was below the bottom of Hole No. 301 and below elevation 2522 in Hole No. 309, which was as deep as any tests could be made in that hole on that date as the lower two feet of this hole had caved in. The reservoir water table is given on the attached cross section at 2532 feet above sea level. If this is the elevation used when the dam is constructed the water table of July 28 would be above this elevation, except between Drill Holes No. 302 and the east abutment of the dam, a distance of about 240 feet. The lowest water table obtained to date was on September 21, 1936,

at which time this water table parallels the water table of July 28 between Drill Holes No. 302 and 306, being approximately 6 to 10 feet below the July 28 level. No water table level is given on the September date for Hole No. 300A, which lies between Drill Holes Nos. 306 and 308, due to the fact that this hole evidently caved at a depth of 77 feet below the surface,or at an elevation of 2585 feet, but in Drill Hole No. 308 the September 21 water level is approximately 8 feet lower than the July 28 level. As Hole No. 300-A caved at an elevation of 2585, the water table in this hole on the September date was below this elevation, and it is estimated it was on the September date at approximately 2578 feet elevation or 7 feet below the point at which the hole caved. The information on this hole caving was obtained from Mr. D. J. Shea, Resident Engineer, in letter dated January 16, 1937.

Attached are the observations made under the direction of the Engineers at the request of the Consulting Geologist on the water level in the drill holes on the ridge at the east abutment of the dam. It will be noted that throughout the entire time that observations were made no water level occurred in Holes Nos. 301 and 309, and that from August 1 to August 5 no observations were made in Hole No. 302, and that after August 17 no water was found in Holes Nos. 300A or 307. The 8 foot drop of water

ELEVATION OF WATER IN RIDGE HOLES

Month of July 1936

HUB ELEV. / HOLE NO. / DATE	2662.0 300A	2613.3 301	2620.3 302	2650.0 306	TIME	REMARKS
21	2606.1	NoWater	2559.0	2566.5	8:30 A.M.	(Shower from 11:4
22	2606.1	" "	2558.8	2566.5	8:30 A.M.	(to 1:00 -21
23	2586.2	" "	2558.8	2566.4	9:00 A.M.	(Hard rain 4:30-
24	2586.2	" "	2558.8	2566.5	9:00mA.M.	(5:30P.M. -23rd.
25	2586.5	" "	2558.6	2566.5	8:30 A.M.	
27	2586.4	" "	2558.5	2566.5	8:30 A.M.	(40'-1 & 60' of (1¼" put in 302
28	2586.3	" "	2558.8	2566.5	9:00 A.M.	(Hard Rain 2:00-
30	2585.9	" "	2559.8	2566.4	9:00 A.M.	(4:00 P.M. -29th
31	2585.9	" "	2558.8	2566.5	9:00 A.M.	

Month of August 1936

1	2585.7	" "	2558.0	2566.0	9:00 A.M.	
3	2585.7	" "		2566.0	8:45 A.M.	(Tester would not (go by 53.5 in 302
4	2585.5	" "		2566.0	9:00 A.M.	Ditto
5	2585.4	" "		2566.0	9:00 A.M.	(Hard rain 11:45- (12:45 P.M.
6	2585.3	" "	2560.2	2566.0	10:00 A.M.	Rain 3 to 4 P.M.
7	2585.2	" "	2561.4	2565.8	9:00 A.M.	Rain 3 to 7 P.M.
8	2585.2	" "		2566.0	8:30 A.M.	to 10.00 A.M.
10	2585.0	" "	2562.0	2565.7	9:45 A.M.	Rain 12:30-5 P.M.
11	2584.8	" "	2562.1	2565.6	9:45 A.M.	
12	2584.8	" "	2562.0	2565.5	9:15 A.M.	
13	NoWater	" "	2561.9	2565.5	1:00 P.M.	
14	2584.6	" "	2561.7	2565.5	9:00 A.M.	
15	2584.6	" "	2561.5	2563.4	9:15 A.M.	(Rain, slow 3:30 (to 6:00 P.M.
17	2584.6	" "	2562.0	2565.2	8:45 A.M.	(Rain 8-10 P.M. (Aug. 16.
18	NoWater	" "	2561.9	2565.2	9:00 A.M.	
21	" "	" "	2561.4	2564.9	9:00 A.M.	
31	" "	" "	2560.8	2564.4	8:45 A.M.	

AD-A063 583 ARMY ENGINEERING DISTRICT NORFOLK VA F/G 13/2
NATIONAL DAM SAFETY PROGRAM. TOWNES DAM (VA14102), DAN RIVER, P--ETC(U)
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Month of September 1936

HUB ELEV. / HOLE NO. / DATE	2662.0 300A	2613.3 301	2620.3 302	2650.0 306	TIME	REMARKS
1	No Water	No Water	2560.2	2564.4	2:00 P.M.	
2	" "	" "	2560.2	2564.3	8:45 A.M.	
7	" "	" "	2560.0	2564.1	8:45 A.M.	
14	" "	" "	2559.1	2563.6	9:00 A.M.	
21	" "	" "	2558.5	2563.2	9:45 A.M.	

Month of October 1936

2	" "	" "	2559.9	2562.7	8:30 A.M.	(Heavy Rains (Sept. 29 & 30.
12	" "	" "	2559.8	2562.2	8:30 A.M.	(Rain 7, 8 & 9.
19	" "	" "	2563.1	2562.0	8:30 A.M.	(Heavy Rain 16th (Light Rain 17th
26	" "	" "	2561.9	2561.7	8:45 A.M.	

ELEVATION OF WATER IN RIDGE HOLES (CONT'D)

Month of July 1936

HUB ELEV. HOLE NO. DATE	2635.0 307	2654.7 308	2616.0 309	TIME	REMARKS
21	2537.0	2573.1	No Water	8:30 A.M.	309 Caved at 2520.(
22	2536.9	2572.9	" "	8:30 A.M.	309 Caved at 2520.((Shower from 11.45- (1:00 on 21st.
23	2536.8	2572.9	" "	9:00 A.M.	309 Caved at 2522.(
24	2536.9	2573.1	" "	9:00 A.M.	309 Caved at 2522.((Hard Rain 2 - 4-23
25	2536.8	2572.4	" "	8:30 A.M.	
27	2536.7	2572.4	" "	8:30 A.M.	
28	2536.7	2572.4	" "	9:00 A.M.	(Hard Rain 2 to 4 or (July 29.
30	2536.6	2571.9	" "	9:00 A.M.	
31	2536.7	2572.2	" "	9:00 A.M.	

Month of August 1936

1	2536.5	2571.9	" "	9:00 A.M.	
3	2536.0	2571.9	" "	8:45 A.M.	(Tester wou
4	2536.0	2571.8	" "	9:00 A.M.	
5	2535.9	2571.8	" "	9:00A.M.	(Hard Rain 11:45 to (12.45 P.M.
6	2535.8	2571.8	" "	10:00 A.M.	Rain 3-4 P.M.
7	2535.8	2571.2	" "	9:00 A.M.	(Slow Rain 3 to 7 PM
8	2535.7	2571.4	" "	8:30 A.M.	(to 10:00 A.M.-8th
10	2535.7	2571.4	" "	9:45 A.M.	Rain 12:30-5:00P.M.
11	2535.0	2571.1	" "	9:45 A.M.	
12	2534.9	2571.1	" "	9:15 A.M.	
13	2535.1	2571.0	" "	1:00 P.M.	
14	2535.1	2571.1	" "	9:00 A.M.	
15	2534.8	2570.8	" "	9:15 A.M.	(Rain, slow 3:30- (6:00 P.M.
17	2534.7	2570.4	" "	8:45 A.M.	Rain 8-10 P.M. 16th
18	Water Past Tape	2570.5	" "	9:00 A.M.	
21	Below Tester	2569.8	" "	9:00 A.M.	
31	NoWater	2567.8	" "	8:45 A.M.	

Month of September 1936

HUB ELEV. / HOLE NO. / DATE	2635.0 307	2654.7 308	2616.9 309	TIME	REMARKS
1	NoWater	2567.4	No Water	2:00 P.M.	
2	" "	2567.5	" "	8:45 A.M.	
7	" "	2566.5	" "	8:45 A.M.	
14	" "	2565.4	" "	9:00 A.M.	
21	" "	2565.0	" "	9:45 A.M.	

Month of October 1936

2	" "	2564.6	" "	8:30 A.M.	(Heavy Rains Sept. (29 & 30.
12	" "	2563.7	" "	8:30 A.M.	Rain 7, 8 and 9.
19	" "	2564.3	" "	8:30 A.M.	Heavy rain Oct. 16.
26	" "	2563.5	" "	8:45 A.M.	Light rain Oct. 17.

which was the average lowering in the water table from July 28 to September 21 would carry the water table to just below the bottom of H ole No. 307, and would carry it below the elevation of 2532, which it is understood is the proposed water level of the reservoir. This would mean that special attention should also be paid to stopping reservoir leakage around Drill Hole No. 307.

It should be noted that a particularly soft layer of gneiss containing a certain amount of sericite occurring at Drill Hole 302 is responsible for the deep overburden at this point, and that the layer of greenstone starting at Drill Hole No. 309 and extending down the ridge approximately 115 feet shows very little weathering compared to the weathering of the gneissic layers. It should also be noted that the greenstone occurring at this point, where the ridge is very narrow, is badly fractured and that the water table drops to an unknown depth in this rock. It is a type of rock which can be grouted easily.

It should also be noted that where a similar greenstone ledge occurs where the ridge is wide at Drill Hole No. 308 that the water table is found to occur at a satisfactory level.

The fact that the schistosity of all of these rock layers extends at right angles across the narrow part of the ridge makes a bad geological condition, as leakage will

occur along the weathered planes of schistosity which cut across the narrow part of the ridge at almost a right angle.

The results of the core drilling of this ridge shows that between Drill Hole No. 302 and the east abutment of the proposed dam site there is a stretch of rock partly greenstone and partly gneiss of varying composition through which leakage will occur, but it is considered that this leakage can be stopped by proper engineering practice. This is a distance of approximately 240 feet. Throughout most of the remainder of this distance the top of the water table, which is only based on records kept for several months since these drill holes were completed, is above the top of the proposed reservoir to be formed by the building of the proposed dam.

Normal Fault in Big Bend Dam Site Area: A normal fault occurs on the west side of the ridge which forms the west abutment to Dam Sites Nos. 21 and 22. It is exposed at Station 220-50 on the service road into this dam site area, where this normal fault cuts off part of the greenstone layer which crosses the river a few feet down stream from Dam Sites Nos. 21 and 22. This fault extends along the west side of the ridge in a direction N. 8° W., so it does not cut into the reservoir area or the Dam Sites Nos. 21 and 22.

RECOMMENDATIONS

Point Lookout Dam Site No. 17

From a geological standpoint Dam Site No. 17, the final location made by the Designing Engineer, is approved, with the understanding that, due to the fact that no core drilling has been done on the upstream side of this dam site and, further, due to the fact that the west abutment and part of the west side of the arc of this dam is located on the edge of the rock slide from Point Lookout, it is impossible to determine accurately the depth to solid rock. No sub-surface contours to solid rock could be drawn at this point as such would be purely hypothetical. It is estimated that at least 35 feet of overburden and weathered rock will be found on part of the west side of the arc of Dam No. 17.

It is also estimated from geological conditions that lines of weathering parallel to the planes of schistosity will extend down to an elevation of 2070 feet above sea level at places on the west side of the arc of this dam, so that grouting should be done to at least this depth.

Although from a geological standpoint Dam Site No. 16 is considered best, the site selected by the

Designing Engineer, Dam Site No. 17, is approved from a geological standpoint with the understanding that considerable grouting will have to be done and that considerable overburden will be found on part of the west arc of this dam site.

Big Bend Dam Site No. 22

Either Dam Site No. 21 or 22 is approved from a geological standpoint with the understanding that the provisions which are outlined below will be followed by the Engineers.

After the dam is built at or approximately at location No. 21 or 22 and after the leakage which will occur between the east abutment and Drill Hole No. 302, a distance of approximately 240 feet, has been taken care of by proper engineering practice, it is recommended that the reservoir be filled so that it can be determined just where any minor leakage may occur through the remainder of the ridge on the east side of the dam back of Drill Hole No. 302, and when the location of such leakage is determined that the water level of the reservoir be lowered and that this leakage be stopped by grouting that part of the ridge where the leakage occurred. Data obtained through the summer and early fall of 1936 on the

water table on this ridge shows that this water table is practically at the elevation of the top of the proposed reservoir level or above it throughout the remainder of the ridge, if the reservoir level is at 2532 feet above sea level. At Drill Hole No. 307 the water table was below the proposed reservoir level during the latter part of August, and the months of September and October. Leakage can, therefore, be expected through the ridge at this point, which leakage can be stopped in the manner suggested above.

Although from a geological standpoint it is considered that the site between 200 and 250 feet downstream from sites Nos. 21 and 22 is best, the sites selected by the Designing Engineer, that is Sites Nos. 21 and 22, are approved from a geological standpoint provided the recommendations made above are followed.

Respectfully submitted,

Wilbur A. Nelson

Wilbur A. Nelson,
Consulting Geologist.

January, 1937.

APPENDIX V

UNDERWATER INSPECTION, REPORT

LOGAN DIVING, INC.
5731 ST. AUGUSTINE ROAD - CODE 904 731-0000
JACKSONVILLE, FLORIDA 32207

UNDERWATER INSPECTION

TALBOTT & TOWNES DAMS

FOR

CITY OF DANVILLE, VA.

APRIL 4 & 5, 1978

An underwater inspection of the Talbott and Townes Dams, also known as the Pinnacles Hydro-Electric Development, was requested and performed as part of a periodic general preventive maintenance program instituted by the Danville Electric Department.

The work was coordinated by Messrs. Eldred C. Yerks, H. E. Cole, and Clyne Willis of the Danville Electric Department. Their efforts included scheduling of personnel and materials to effect repairs anticipated to a 6" by-pass valve at the Talbott Dam and replacement of a lifting cable for the trash rack at Townes Dam; provide personnel to operate various valves and winches at both dams; coordinate water flow outages; provide safety liaison with Logan Diving, Inc.; and general coordination of the overall project.

Logan Diving, Inc. was represented by Jack S. Mixer, John Redon and Thomas E. Thurston. Their responsibilities were to inspect the upstream intake structures and appurtenances and downstream dam toes and footings.

The following procedure was followed at both dams prior to commencement of diving operations:

1.) Conference with the Danville project coordinators to detail inspection procedure, safety measures and exchange of other information.

2.) Physical operation of all gates and valves by Danville personnel in the presence of Logan representative to assure their good operation and to determine the extent of any leakage.

3.) Inspection of the downstream side of the dam by Logan for undermining, spalling concrete, leakage and other deleterious conditions.

4.) Groundline soundings and measurements by Logan on the upstream side of the dams in the vicinity of the intake.

5.) Set up and testing of diving equipment before each dive and after all flow valves had been closed.

TALBOTT DAM

DOWNSTREAM AREAS INSPECTED

Footing, stilling basin, drain holes in stilling basin wall, construction joints, discharge pipes, concrete deterioration.

DOWNSTREAM FINDINGS

A.) Two stilling basin wall drain holes were clogged with gravel and debris. Both drains were opened during the inspection.

B.) Two of the 6" ø discharge pipes had blockages which prevented totally free water flow. These blockages were not removed though minor attempts were made. This problem was not deemed significant enough to warrant extensive effort for removal and has existed for many years.

UPSTREAM AREAS INSPECTION

Gate guides, gate cable, groundline build-up concrete deterioration, trashrack, trashrack guides and supports.

UPSTREAM FINDINGS

In all regards the upstream portion was found to be in good condition with the one exception of extensive groundline build-up in front of the trash rack which is restricting water flow and placing pressure against the trash rack when the gate is open. (See drawing for details)

The groundline materials are a composite of silt, trash, sticks and tree stumps accumulated since the time of construction.

At the conclusion of the inspection, the gate was opened and the cable clearly marked to indicate the open position.

TOWNES DAM

DOWNSTREAM AREAS INSPECTED

Footing, spillway apron footing for undermining, construction joints, concrete deterioration and general condition.

DOWNSTREAM FINDINGS

No defects were found.

UPSTREAM AREAS INSPECTED

Trash rack guides, trash rack, groundline build-up concrete deterioration. Specific attention was given to the deteriorated and broken lifting cable, single wheel block and trash rack lifting eye.

UPSTREAM FINDINGS

No significant defects were found with the single exception of the broken lifting cable and corroded block.

Divers removed the old block and cable and attached the new block and cable furnished by Danville.

The new block was placed through the trash rack lifting eye and wired in position with approximate No. 9 copper wire.

Tension on the new stainless steel was pulled to remove all slack and the cable clearly marked in the closed trash rack position at the operating floor with red paint.

Rust blisters were observed on the trash rack guides which poses no structural problem at this time. However, diver should scrape and clean the guides before lifting of the trash rack to prevent any binding of the rack in the guides. (see drawing for groundline details)

RECOMMENDATIONS FOR TALBOTT DAM

The most significant problem is the heavy amount of silt and debris accumulated against the trash racks. Although this condition presents no imminent danger because of the structures good condition, it does restrict water flow and place undue stress on the trash racks supports.

Should this groundline build-up cause the trash rack to fail, the corrective cost, possible long term outage, and associated problems would be excessive.

We strongly recommend excavation of the groundline in front of the trash rack to allow full flow of water through the trash rack.

This work would require careful coordination with plant operations, careful excavation to avoid snagging the trash rack or structure which could cause damage and divers to monitor the work.

Our firm is designing a procedure and quotation for performance of this work.

Although the two clogged discharge pipes are not a serious problem at this time, they should be checked periodically (semi-annual) to see if additional pipes become closed which would warrant the cost of correction.

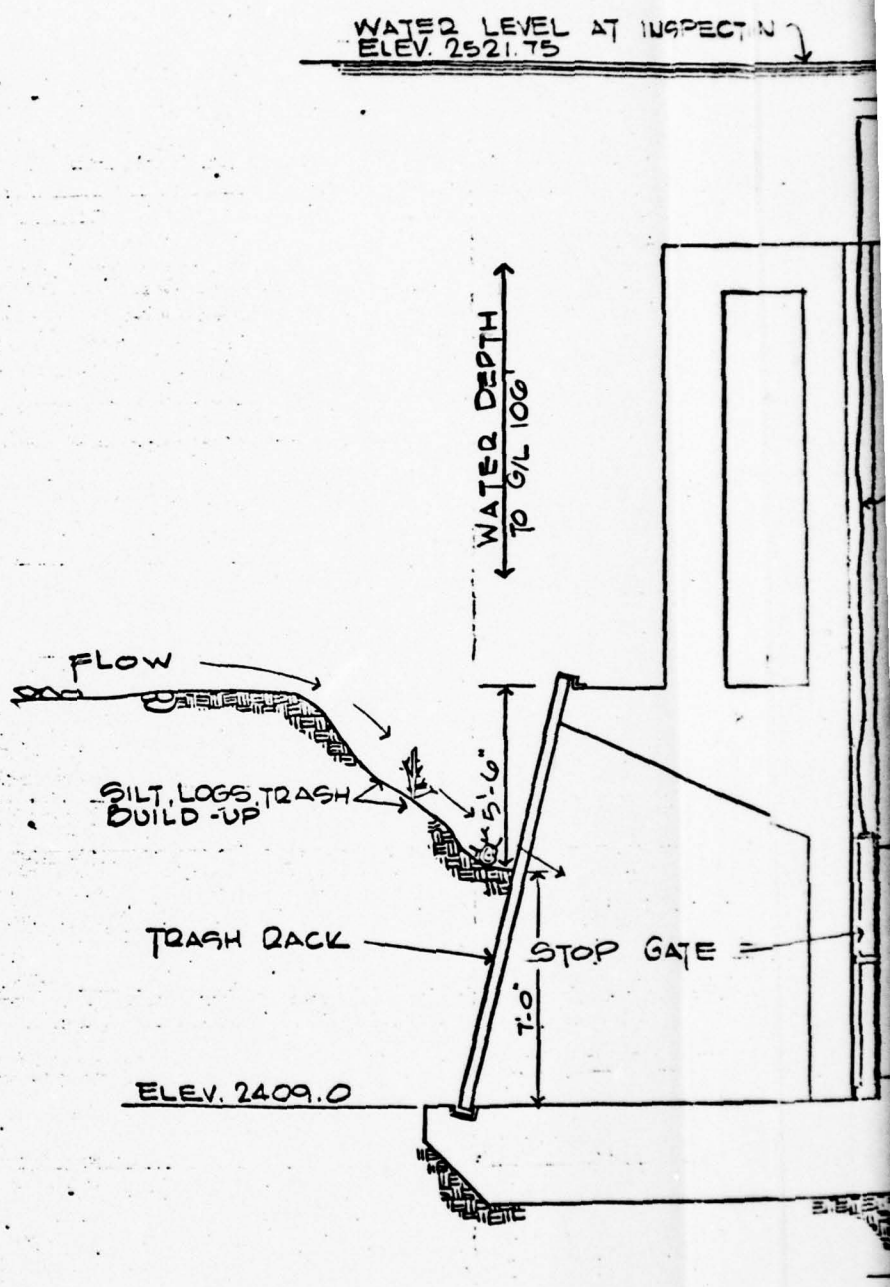
We also recommend semi-annual opening and closing of the head gate and downstream valves to assure their continued functions.

RECOMMENDATIONS FOR TOWNES DAM

Following replacement of the trash rack lifting cable, we believe the system to be in good condition.

We do recommend cleaning of the trash rack guides prior to any needs to raise and lower the trash rack.

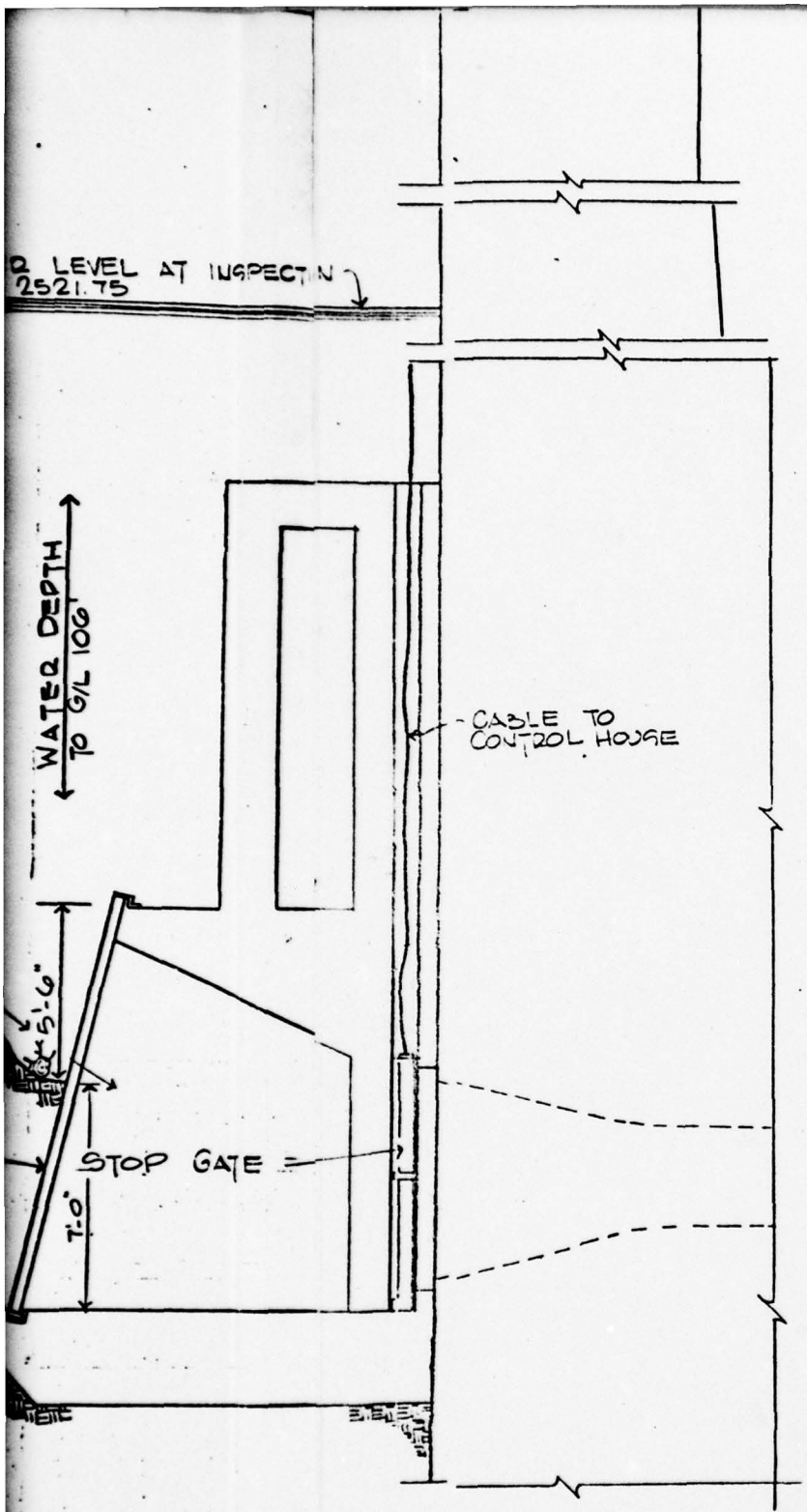
We also recommend semi-annual operation of downstream valve to assure to continued functions.



SECTION THRU TRASH RACK

SCALE 1/4"=1'-0"

TALBOTT DAM



TION THRU

SH RACK

¼"=1-0"

DAM

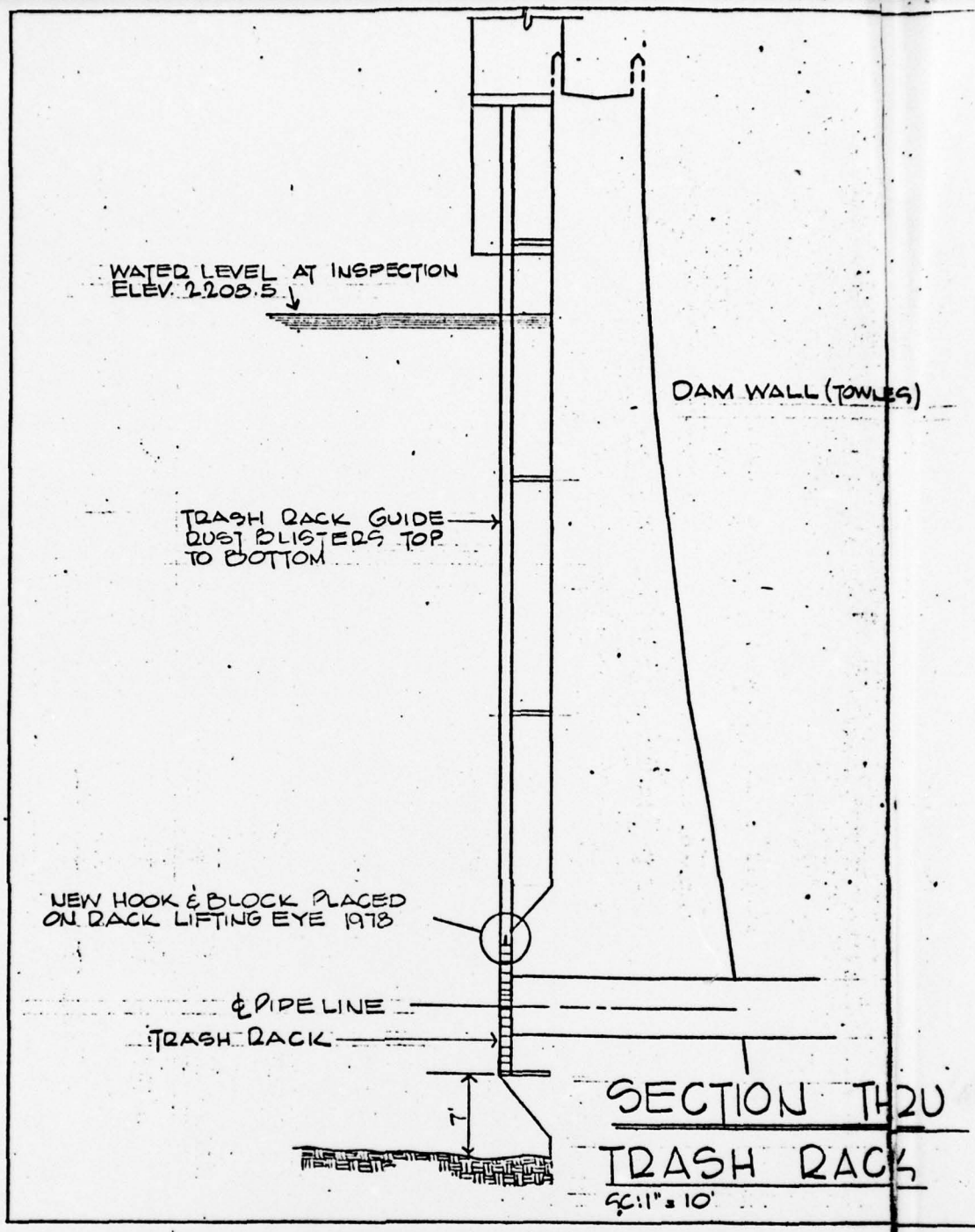
UNDERWATER INVESTIGATION

PINNACLES HYDRO-ELECTRIC PROJ.
UPSTREAM INSPECTION

TALBOTT & TOWNES DAMS
FOR
DANVILLE ELECTRIC DEPT.

BY
LOGAN DIVING, INC.
JACKSONVILLE, FLA. 32207

DRAWN BY	J.W.H.
CHECKED BY	JGM
DATE	APR 78
SCALE	HOR. AS NOTED VER.
DRWG. NO.	1 of 2



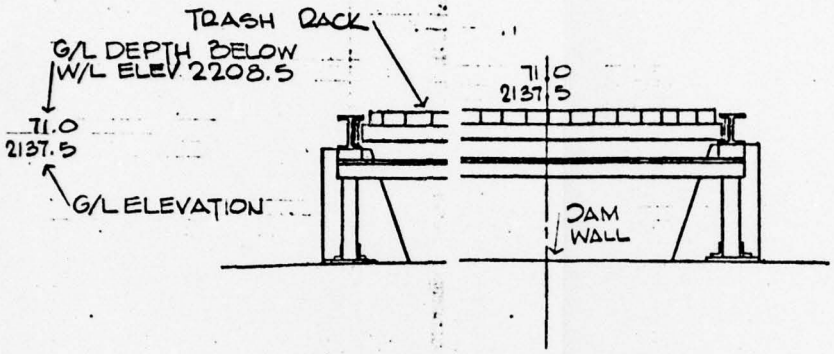
G/L
W/L
71.0
2137.5
G/

68.0
2140.5

72.8
2135.7

71.5'
2137.0

DAM WALL (TOWERS)



PLAN AT TRASH RACK

SC 1/4" = -0"
TOWERS DAM

CTION THRU
ASH RACK
10'

BY
LOGAN DIVING, INC.
JACKSONVILLE, FLA. 32207

DWG 2 OF 2

2

APPENDIX VI

CONTRACT SPECIFICATIONS
(SELECTED PAGES)

PART I.

SPECIFICATION

for

POINT LOOKOUT DAM

I. GENERAL

(a) Purpose

These specifications and accompanying contract documents, including the drawings and schedules are intended to cover the clearing of the Point Lookout Reservoir and the building of the Point Lookout Dam, which is part of the Pinnacles Hydro-Electric Project. The Contractor shall do all of the work indicated in these specifications and on the plans and such additional work as may be necessary to complete the dam and appurtenances in a substantial and acceptable manner and leave the work in a neat and finished condition.

(b) Description

The Point Lookout Dam will be constructed across the Dan River about 475 feet below its confluence with Round Meadow Creek. The dam is located about 4 miles south of Meadows of Dan in Patrick County, Virginia. Meadows of Dan is 16 miles west on Route 58 from Stuart, Virginia, the terminal of the Danville and Western Railway, and 34 miles East on Route 58 from Galax, Virginia, on the Norfolk & Western Railway. Both railroads have storage tracks and unloading space which can be made available to the Contractors.

From Meadows of Dan an improved road leads on an easy grade to the top of the gorge of the Dan River, from which a macadam surfaced road leads into the gorge, on a 13% grade a distance of about 4000 feet to the dam site.

The width of the Dan River at the dam site is about 50 feet and the depth about 18 inches. The average flow is approximately 85 second feet and the estimated maximum monthly flow in the past 15 years has been about 300 second feet, the flood water occuring either in the Spring or in the Fall of the year. It is believed that a small coffer dam and flume leading across the dam site would be sufficient to unwater the site preliminary to excavating for the foundation.

The dam is a massive concrete arch dam built in sections, or blocks about 40 feet long. The dam is about 130 ft. high and 573 ft. long.

The intake will be built in the center of one of these blocks and consists of a 72" steel pipe through the dam controlled by an electrically operated Dow type butterfly valve. On the upstream side of the intake, a removable screen will be provided, controlled by an electrical hoist in a gate house, located at the crest of the dam. A similar house is located symetrically about the center line of the spillway. At the

down-stream side of the intake a valve house is to be built to house the butterfly valve and its operating mechanism. At the crest of the dam a walkway is provided with stairways at each abutment.

(c) Vicinity Map

The attached print shows a general location of the project, of the roads leading to it, and diagramatically, the general scope of the work.

(d) Construction Camp Facilities

The City operates a Construction Camp near the site of the work and is prepared to furnish food and lodging to the Contractor's men at nominal rates per man day as approved by the PWA Resident Engineer Inspector, but not to exceed $1.00 per day per man.

The City will also furnish office space for the Contractor including light and heat at a rate not to exceed $ .10 per square foot per month.

II. DRAWINGS

The following general drawings are a part of these specifications and contract:

Drawing No.	Title
1257-601	General Plan
1257-602	Block 1 - Platforms
1257-603	Block 1 Section 7 - Sluiceway
1257-604	Block 5 - Valve House
1257-605	Blocks 5 & 9, Valve, Gate & Tool Houses
1257-606	Blocks 5, 6, 7, 8 & 9- Spillway
1257-607	Block 13 Platform - Instrument Pier
1257-608	Vertical Construction Joints
1257-609	Diamond Drill Core Borings
1257-610	Blocks 1,9, 13 Reinforced Concrete Details
1257-611	Blocks 4,5,6,7, & 9 " " "
1257-612	Excavation, Grouting & Grading
1257-613	Steel & Iron Work
1257-614	Blocks 1 to 9 - Electrical Work
1257-615	Blocks 1 to 9 - " "
1257-616	Deflection Measurement Apparatus
1257-617	Deflection and Temperature Stations, Location of Construction Joints

These drawings show the approximate dimensions and shape of the dam and related structures and the general arrangement of all features. Such additional drawings as may be required for the details of the work will be furnished to the Contractor as required. The Contractor will not be held responsible for the correctness or sufficiency of the designs,

trees, bushes and other perishable matter. Trees shall be cut off not higher than 18 inches above the ground. All material so removed shall be piled in windrows and burned. The burning shall be done at such time and in such manner as may be required by the Engineer. Any available timber within the area to be flooded by the reservoir may be cut and used by the Contractor for construction purposes.

(b) Measurement and Payment

Measurement and payment for reservoir clearing shall be made on the unit prices bid in the schedule. It is stipulated and agreed that the planimeter shall be considered an instrument of precision adapted to the measurement of cleared areas.

VII. EXCAVATION

(a) Classification

Except as otherwise provided in these specifications, all material removed from excavation will be measured in excavation only to the neat lines shown on the drawings or prescribed by the Engineer and will be classified as follows:

Rock Excavation

All solid rock in place which cannot be removed until loosened by blasting, barring or wedging, and all boulders or detached pieces of solid rock more than 1/2 cubic yard in volume. No material except boulders and detached pieces of solid rock will be classed as rock excavation which is not actually loosened by blasting before removal, unless blasting is prohibited, and barring, wedging or similar methods are prescribed by the Engineer.

Disintegrated Rock

All disintegrated and loose rock which can be efficiently removed by excavating machinery and without blasting.

Earth Excavation

All soft overburden not including disintegrated rock which can be removed by sluicing or with excavating machinery.

No additional allowance over the unit prices bid for excavation will be made on account of any of the material being wet or frozen.

(b) Blasting

Blasting will be permitted only when proper precautions are taken for the protection of persons, the work and private property and any damage done to the work or to property shall be repaired by the Contractor at the Contractor's expense. Exploders shall in no case be stored, transported or kept in the same place in which dynamite or other explosives are stored, transported or kept. In general the precautions

taken to prevent accidents shall be subject to the approval of the Engineer, but the Contractor shall be liable for all injuries to, or death of persons, or damage to property caused by blasting or explosives. Electric blasting machines shall be used on all work.

(c) Excavation for Foundations

The excavation for the dam foundations shall be made to a sufficient depth to secure foundation on sound ledge rock, free from open seams or other objectionable features. Unusual precaution shall be taken to preserve the rock outside of the line of excavation in the soundest possible condition. Blasting may be done only to the extent directed by the Engineer with explosives of such moderate power and in such locations as will neither crack nor damage the rock outside of the prescribed excavation limits. Whenever in the opinion of the Engineer, blasting is likely to injure the rock upon or across which concrete is to be placed, the use of explosives shall be discontinued and the excavation completed by wedging, barring and picking, or other suitable methods. The excavation shall be roughly shaped, as directed by the Engineer to approximately horizontal steps, at least 5 feet high and separated by approximately vertical radial planes. Special care shall be taken in excavating these steps to avoid shattering or damaging the adjacent rock.

(d) Limits of Foundation Excavation

Earth and loose rock shall be excavated on the steepest practical slopes with the following modification. Where soft overburden is encountered the slopes shall not be less than 1:1 and the toe of the slopes shall not be less than 30" from the beginning of the rock cut. Where rock is encountered the slopes shall not be less than 1/4:1. Solid rock shall be excavated as close as practical to the neat lines of the structure, except for the protection of the upstream face of the dam the minimum excavation shall consist of an 8 ft. ledge along the entire face of the dam.

The drawings refer to, and show "sound rock" contoursand elevations determined from diamond drill core borings. These contours and elevations were used in the design of the dam. The actual sound rock elevation will be determined by the Engineer after an inspection of excavation. If the approved sound rock elevation so determined differs more than 5 ft. from the sound rock elevation and contours shown on the contract drawings it may be necessary to redesign the dam. The Contractor shall have no right for claims due to delay that may be necessary to make changes in the design.

(e) Disposal of Materials

At the upstream face of the dam no excavated material shall be disposed of in a distance of 500 ft. and the trench along the upstream face shall not be backfilled. At the downstream side of the dam backfill shall be provided and grading shall be done in accordance with the drawings.

Excavated material not used for backfill shall be disposed of in a manner and at locations approved by the Engineer. All spoil banks shall be located where they will not interfere with the natural flow of the river or with the discharge from the spillway. Spoil banks shall be located where they will not detract from the appearance of the structure or interfere with the accessibility of the structure for operation. Spoil banks shall be leveled and trimmed to reasonably regular lines and the Contractor shall not be entitled to any additional compensation on account of such refinement. All combustible material such as trees, logs, brush or roots required to be removed from the excavation for the dam or other works, or from the sites of the spoil banks or otherwise shall be piled and burned under the direction of the Engineer. The cost of disposing of all excavated material that is wasted and all other work described in this paragraph shall be included in the price bid in the schedule for excavation.

In general the manner of disposing of materials is indicated on drawing 1257-612, which shows two spoil banks on either side of the river extending from the downstream face of the dam a distance of approximately 300 ft. downstream.

The top of these spoil banks is at elevation 2145. and the toes of the banks are 40 ft. apart on either side of the river, leaving a straight channel for water passing over the spillway.

(f) Measurement and Payment

Measurement and payment will be made to the prescribed neat lines of the excavation and at the unit prices bid in the schedule. Backfill will be measured for payment in place and paid for at the price bid in the schedule. Any excavation outside of the limits prescribed which is required to be backfilled shall be done by and at the expense of the Contractor.

VIII. FOUNDATION PREPARATION

(a) Description

After the sound rock elevation has been determined and the minimum excavation required on the upstream face of the dam has been completed the base rock shall be prepared for the building of the dam.

The surface of the rock shall be left rough so as to bond well with the concrete, and where necessary shall be cut to rough benches or steps as directed by the Engineer to secure the required roughness. Care must be taken not to shatter or disturb rock foundations unnecessarily. All loose fragments, dirt and spalls must be removed from the rock surface before concrete is poured. Wire brooms, hammers, picks or streams of water, air or steam or other effective means shall be used to clean the foundation.

(b) Grouting Base Rock

The base rock underneath the dam shall be prepared by thorough cement grouting. The purpose of this grout is to provide a water tight curtain below the dam along its upstream face and also to provide a uniform and consolidated foundation for the dam. While the general scheme of grouting to be employed is shown on the drawings, the depth and location of borings is illustrative only, and may be different when the excavation is made and the exploration holes are drilled.

Approximately 10% of the total number of grout holes shall be drilled with diamond core drills, 1-1/2" core. The location of these exploration holes will be determined by the Engineer, and the record of the holes and the cores shall be kept and the cores properly preserved. Each of the diamond drill borings shall extend a minimum of 30 feet below the sound rock elevation and if the last 10 feet of the hole does not establish a satisfactory rock, the hole shall be extended up to a depth of 50 feet.

After a study of the cores of the exploration holes has been made, grout holes shall be drilled to such depth as the Engineer may determine. All grout holes shall be drilled with jack hammers, or by any other method, and shall have a minimum diameter of 1-1/2".

Immediately preceding grouting, each hole shall be thoroughly cleaned by lowering a 1 inch pipe to the bottom of the hole and pumping water through the pipe until a uniform flow of clean water is returned from the hole. After such flow is established the pumping of water shall be continued at least 15 minutes, at the end of which time the hole shall be dried with compressed air, the pipe carefully removed and the hole filled with grout, plugged, and connected to the grout pump which shall be kept pumping until a pressure of 100# per square inch is established. The pressure shall be maintained at 100# until the hole refuses to take additional grout and for 15 minutes longer. If this cannot be accomplished, additional holes shall be drilled in the vicinity of the unsatisfactory hole and all shall be grouted in the same manner.

Holes shall be drilled and grouted in the sequence shown on the drawings. A complete record shall be kept of the time each hole was drilled, washed and grouted, and also the amount of grout admitted during each 10 minute interval shall be recorded.

Grout for the base rock shall be composed of cement and water, or cement, sand and water in proportions to be determined by the Engineer. Sand shall be clean and of such fineness that 100% will pass a screen with 64 openings per square inch and 50% will pass a screen with 1600 openings per square inch. The apparatus for mixing and placing the grout shall be of a type approved by the Engineer and capable of mixing and stirring the grout and forcing it continually into holes at any desired pressure up to 100 lbs. per square inch. If during the grouting of any grout hole, grout is found to flow from adjacent holes in sufficient

(c) Classification

Class A, 3000# concrete shall be used in the apron and the main body of the dam up to elevation 2225.5. Class B, 2000# concrete shall be used for the stairways and gate, tool and valve houses and the parapet and walkway above elevation 2225.5, and instrument pier.

(d) Forms

It is suggested that the Contractor use panel forms for the main body of the dam with suitable devices to hold adjacent ends and edges of panels tight and in accurate alignment. All forms shall be wired each time before being used with suitable non-stretching wire satisfactory to the Engineer. Metal rods or similar devices to hold the forms will be allowed in the structure provided proper means are used to take out a portion of each rod nearest the surface, at least 2" in length. All holes left after removal of rods shall be filled immediately and completely covered with cement mortar and the surface left smooth and in good condition. If wire ties are used they shall be cut off closely to the concrete after forms are removed. Wood forms shall be wet thoroughly before placing concrete. Where forms are placed in sufficient units for continuous surfacing care shall be exercised to set the forms tightly over the completed surface so as to prevent leakage of mortar from the concrete. Forms shall be left in place until their removal is authorized by the Engineer and shall then be removed by the Contractor with care so as to avoid injury to the concrete. Steel forms shall be used above elevation 2200 to form the spillway, walkway supporting piers, and walkway above the spillway wing walls. Permanent galvanized steel forms shall be furnished and left in place for all air vents in the spillway. The cost of forms shall be included in the prices bid for concrete.

(e) Defective Concrete

Damaged concrete from any cause or defective concrete which shall be found defective at any time before the completion and acceptance of the work shall be removed and replaced by the Contractor at no expense to the City.

(f) Reinforcing Steel

Reinforcing steel shall be deformed bars rolled from intermediate grade new billet steel, free from rust, scale or coatings which would tend to reduce the bond. The Contractor shall furnish the Engineer with detailed shop drawings for approval. Reinforcing bars shall be so secured that they will not be displaced during the placing of concrete.

Payment for reinforcing steel will be made at the unit prices bid in the schedule which price shall include the delivered cost of the steel and the cutting, bending, placing, wiring and maintaining of same as shown on the drawings or as directed by the Engineer.

and stairways shall be checked before concrete is poured. The entire surface of the parapet above elevation 2225.5 shall be stripped green and finished with wooden floats. The surface of the walkway shall be troweled smooth and finished with wooden floats, particular care being taken to maintain drainage to the 4" cast iron pipes provided for this purpose.

(d) Concrete in Valve and Gate Houses

The concrete in the Valve, Tool and Gate Houses shall be class B concrete built to the lines and dimensions shown on the drawings or prescribed by the Engineer. All reinforcement and imbedded steel and iron work shall be checked before concrete is poured. The roof of the gate and tool houses at the top of the dam shall be waterproofed and finished integrally with steel trowel and left smooth. No roofing will be required for these structures. The roof covering of the valve house shall be a 5 ply tar and gravel Barrett specification, or equal, and provision shall be made for drainage including the installation of a 4" Holt, or equal, drainage connection.

(e) Measurement and Payment

Concrete in the main dam and apron will be measured by the cubic yards actually placed and paid for at the unit price bid in the schedule for Class A concrete, which price shall include all labor and all material excepting reinforcement.

Concrete in parapets and stairway will be measured by the cubic yards actually placed and paid for at the unit price bid in the schedule for Class B concrete, which price shall include all material and all labor excepting reinforcement.

The gate house and tool house at the crest of the dam and the valve house will be paid for at the lump sum prices bid in the schedule for these items.

(f) Vertical Construction Joints

Each vertical construction joint shall be provided with keyways, grout stops, grouting slots, and grouting pipes as shown on the drawings. The grouting slots are provided in the concrete block in which the concrete is placed first, by nailing a wooden strip to the forms and providing pipe connections and nails as shown. After the forms are stripped, the grouting slot is protected by a sheet iron cover fastened by bending the nails as shown on the drawings. When the concrete is placed in the adjoining blocks the grouting slots are to be kept clean by blowing compressed air through them, connecting the compressed air supply individually to each grout pipe at the bottom of the block. After the dam is finished, the grout pipes at the bottom of the different blocks are connected by a header immediately preceding the grouting.

(g) Grouting Construction Joints

The grouting of the joints is to be done by the use of two grouting pumps. The grouting shall be done at two symetrically located vertical joints, starting with the two joints at the extreme ends of the dam and moving toward the center, always grouting two symetrically located joints simultaneously. After the installation of the pumps at two joints, water should be pumped through the grout pipes until all air is removed and a uniform flow of water established through the outgoing pipe, after which the water should be pumped through the joint for at least once hour. After water has been pumped through the joint for one hour, without stopping the pump and without interrupting the flow, grout should be gradually admitted in the pump and the pumping continued until a uniform flow of grout will be established through the outlet pipe. Then the outlet pipe should be plugged by use of a vent provided on this pipe, the pump working continuously, increasing the pressure in the grouting system until the specified pressure of 100 lbs. per sq.in. is obtained. The pump should be kept pumping under this pressure until the joint refuses to admit any grout and one half hour afterwards.

If, during the grouting operation the pressure cannot be built up, due to leakage, the whole procedure should be reversed, water being admitted in the grouting pipes and pumped until all grout is washed out. Afterwards the leak must be found and stopped and the whole grouting procedure repeated until the joint is satisfactorily grouted under the whole specified pressure of 100 lbs. per sq. in.

Grout for vertical construction joints shall be composed of cement and water in proportions to be determined by the Engineer. Grouting will be paid for at the unit price bid in the schedule and shall include all labor, material, plant and incidentals necessary to the operation, including metal grout stops and plates over V slots. Measurement of dry materials will be made at the grouting pump and payment will be made for each full batch or fractional batch actually forced into the vertical construction joints. No payment will be made for lost, wasted or rejected grout.

XI. STRUCTURAL STEEL

(a) Description

The principal structural steel items are the trash rack, its guides and supports and the slot and cover plate in the floor of the gate house, the roof beams, gratings, ladder, pipe railing and miscellaneous items in the gate house, the tool house and the valve house.

(b) Materials and Workmanship

All structural steel shall conform to the latest specification of the American Society for Testing Materials for structural steel for buildings. Insofar as applicable, the design, fabrication, and erection shall be in accordance with the standard specifications of the American

APPENDIX VII

REFERENCES

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1. HEC-1DB Flood Hydrograph Package, (Hydrologic Engineering Center, U. S. Army Corps of Engineers, July 1978).

2. "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian," Hydrometeorological Report No. 33, (U. S. Weather Bureau, April 1956).

3. "Rainfall Frequency Atlas of the United States," Technical Paper No. 40, (U. S. Weather Bureau, May 1961).