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NATIONAL DAM SAFETY PROGRAM. WARNER DAM, CHAUTAUQUA LAKE OUTLET--ETC(U)
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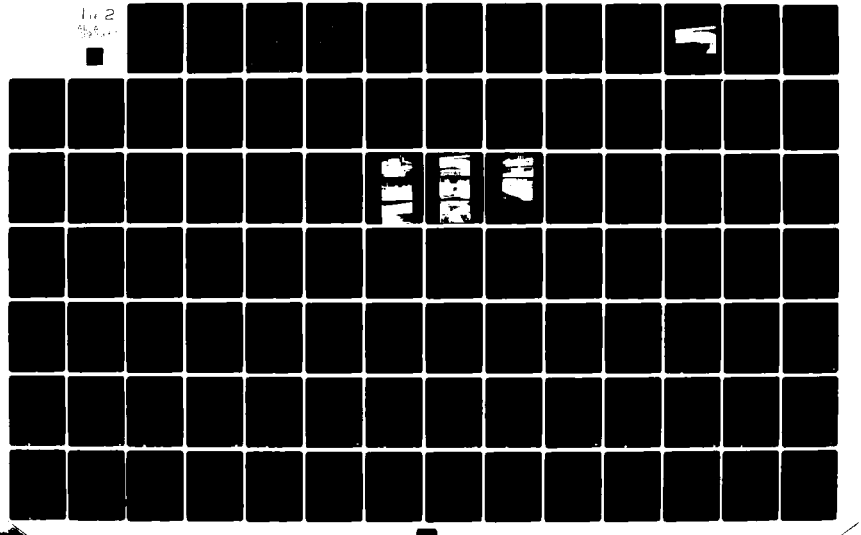

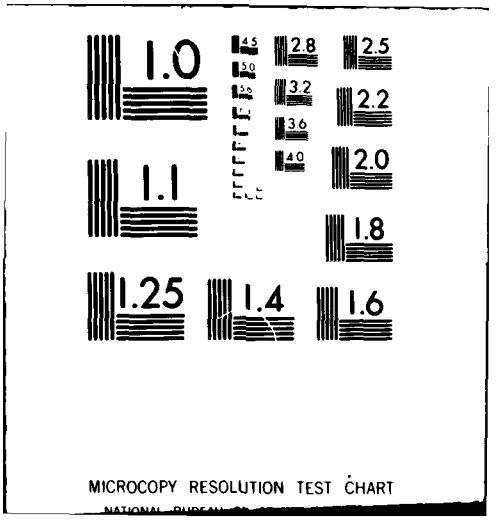
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Warner Dam
Chautauqua Lake
Chautauqua County
Allegheny River

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

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

Examination of available documents and a visual inspection of the dam did not reveal conditions which constitute an immediate hazard to human life or property. However, the dam has some

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deficiencies which require further investigation and remedial action.

Using the Corps of Engineers screening criteria for review of spillway adequacy, it has been determined that the dam would be overtopped for all storms exceeding approximately 75 percent of the PMF. The spillway is, therefore, considered to be "inadequate".

A review of the structural stability analysis performed by the designer indicated calculated safety factors against sliding to be less than recommended minimum value. A more detailed analysis which was performed as a part of this Phase I investigation produces a similar conclusion.

Therefore, it is recommended that within 6 months of notification to the owner a detailed field investigation and monitoring program be undertaken to determine the actual hydrostatic uplift pressures along the base of the dam and that further detailed structural stability analyses be performed using this information.

A warning system and evacuation plan should be developed and implemented within 3 months of notification to the owner.

In addition, the maintenance program which presently exists should be communicated to the dam operator for implementation within 3 months of notification to the owner.

Debris should be removed from the upstream face of the dam as part of the routine maintenance program.

Finally, leakage along the south side of the south tainter gate should be monitored on a daily basis with these observations documented in the dam operator's log book.

(6) National Dam Safety Program

ALLEGHENY RIVER BASIN

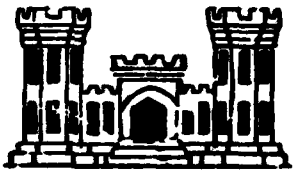
WARNER DAM, CHAUTAUQUA LAKE OUTLET

(Entry Number NY-750)

CHAUTAUQUA COUNTY, NEW YORK.

INVENTORY NO. NY-750

PHASE I INSPECTION REPORT. NATIONAL DAM SAFETY PROGRAM



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DEPARTMENT OF THE ARMY

NEW YORK DISTRICT, CORPS OF ENGINEERS

NEW YORK, NEW YORK

SEPTEMBER 1980

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PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

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

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

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

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
WARNER DAM
I.D. NO. N.Y. 750
ALLEGHENY RIVER BASIN
CHAUTAUQUA COUNTY, NEW YORK

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Appendix G - Drawings

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

NAME OF DAM:	Warner Dam-Chautauqua Lake Outlet Inventory No. N.Y. 750
STATE LOCATED:	New York
COUNTY:	Chautauqua
WATERSHED:	Allegheny River
STREAM:	Chadakoin River
DATE OF INSPECTION:	May 21, 1980 See Vicinity Map and Topographic Map Appendix G

ASSESSMENT

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

Using the Corps of Engineers screening criteria for review of spillway adequacy, it has been determined that the dam would be overtopped for all storms exceeding approximately 75 percent of the PMF. The spillway is, therefore, considered to be "inadequate".

A review of the structural stability analysis performed by the designer indicated calculated safety factors against sliding to be less than recommended minimum value. A more detailed analysis which was performed as a part of this Phase I investigation produces a similar conclusion.

Therefore, it is recommended that within 6 months of notification to the owner a detailed field investigation and monitoring program be undertaken to determine the actual hydrostatic uplift pressures along the base of the dam and that further detailed structural stability analyses be performed using this information.

A warning system and evacuation plan should be developed and implemented within 3 months of notification to the owner.

In addition, the maintenance program which presently exists should be communicated to the dam operator for implementation within 3 months of notification to the owner.

Debris should be removed from the upstream face of the dam as part of the routine maintenance program.

Finally, leakage along the south side of the south tainter gate should be monitored on a daily basis with these observations documented in the dam operator's log book.

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New York District Engineer

View of Downstream
face of Dam.

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
WARNER DAM
I.D. NO. N.Y. 750
ALLEGHENY RIVER BASIN
CHAUTAUQUA COUNTY, NEW YORK

SECTION 1: PROJECT INFORMATION

1.1 GENERAL

a. Authority

This Phase I Inspection Report was authorized by the New York State Department of Environmental Conservation by Contract No. D-201458. This study was performed in accordance with the terms of the above contract and the Recommended Guidelines for Safety Inspection of Dams prepared by the Department of Army, Office of the Chief of Engineers to fulfill the requirements of the National Dam Inspection Act, Public Law 92-327.

b. Purpose of Inspection

This inspection was conducted to obtain available data concerning design and construction of the dam, to evaluate that data, to visually inspect existing conditions at the dam, to identify and evaluate deficiencies and/or hazardous conditions which, if present, may threaten life and property of the residents downstream of the dam and to recommend remedial measures to mitigate such deficiencies and hazardous conditions.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam

The Warner Dam is a reinforced concrete gravity dam with three (3) tainter gates having a total clear span of 80 feet.

The dam is located approximately 60 feet downstream of a former concrete gravity dam having the same structural elevations and hydraulic capacity of the new dam. The existing dam was intended by the designer to be a

"replacement in kind" type of project of the former concrete dam.

Control of underseepage and potential scour downstream of the dam is provided by steel sheet piling.

b. Location

The Warner Dam is located in the city of Jamestown, New York on the Chadakoin River approximately 3.3 miles downstream of Chautauqua Lake and about 700 feet upstream from the Main Street Bridge.

c. Size Classification

The dam has a maximum height of 11.5 feet above the stream bed at elevation 1300.0. However, this structure regulates the level of Chautauqua Lake, with its enormous storage capacity of 236,000 acre-feet between the stream bed and the top of the dam, and, therefore, is assigned the large size classification.

d. Hazard Classification

The dam is classified a high hazard structure due to the number of homes, businesses and bridges along the downstream channel.

e. Ownership

The dam is owned by the New York State Department of Environmental Conservation and operated by the city of Jamestown, Board of Public Utilities. Mr. Arthur Olsen, dam operator and superintendent of the city-owned electrical generating plant was contacted as part of the Phase I Inspection. Mr. Olsen's address is Board of Public Utilities, City of Jamestown, New York, 14701 and his telephone is 716-661-2309.

f. Purpose of Dam

The primary purpose of the dam is to maintain the level of Chautauqua Lake for recreational and fishery resources during low flow periods of the year, while ensuring that

minimum release (discharge) objectives are met for the Chadakoin River to satisfy water quality management.

g. Design and Construction History

In 1976 a report was prepared by Konski Engineers of Syracuse, New York entitled "Inspection and Analysis, Warner Dam, Jamestown, N.Y." which evaluated the condition of the former concrete gravity dam constructed in 1919. Based on the finding of that report, it was decided to replace the old concrete dam.

The firm of Erdman, Anthony, Associates was retained by the New York State Department of Environment Conservation to furnish professional services for assessing the environmental impact of replacing the old dam, and preparing detailed design documents and construction supervision of the new dam.

The dam was constructed primarily in 1979 by Herbert F. Darling, Contractors of Williamsville, New York and was placed in operation on October 11, 1979.

h. Normal Operation Procedure

In order to meet the objectives of the dams purpose an operation plan was developed by the New York State Department of Environmental Conservation. Included in Appendix E are two figures(No. 1 & No. 2) which depict the original operation plan for the dam. This operation plan has since been modified according to the operator and the present objective as of May 21, 1980 is shown on Figure No. 3 of Appendix E.

1.3 PERTINENT DATA

a. Drainage Area	187.2 sq.mi.

b. Discharge at Damsite (cfs)

Total spillway capacity at maximum pool elevation	6629

c. Elevation (ft. above MSL)

Steambed at centerline of dam and spillway crest	1300.00
Chautauqua Lake summer pool (also pool at dam)	1308.25
Top of dam and maximum design pool	1311.50

d. Storage (acre-feet) (Based on Corps of Engineers study in 1965)

Chautauqua Lake at spillway crest (Elevation 1300.0)	205,000
Chautauqua Lake summer pool (Elevation 1308.25)	305,000
Chautauqua Lake at flood pool (Elevation 1310.00)	325,000
Chautauqua Lake at Elevation 1317.32 (This corresponds to top of dam Elev.1311.5)	441,000

e. Reservoir Surface (acres)
(Based on Corps of Engineers Study - 1965)

Spillway crest (Elev. 1300.0)	8050
Design high water (Elev. 1310.0)	15,500
Top of dam (Elev. 1311.5)	15,750

f. Dam

Type: Concrete gravity - Tainter Gate Dam	
Length: (ft.)	96.0
Height: (ft.)	11.5
Top Width: (ft.)	9.0
Cutoff:	Steel Sheet Piling
Grout Curtain:	None

g. Spillway

Length of Weir:	80 ft.
Crest Elevation:	1300.0

Gates: 3-Tainter Gates 26'8" each in width

The data referred to above and in Section 5 which was obtained from the following publication of the Corps of Engineers:

"Lake Chautauqua and Chadakoin River, Jamestown, New York, Local Flood Protection, General Design Memorandum, Vol. II - Appendices", U. S. Army Engineer District - Pittsburgh, Corps of Engineers, dated March 1965.

SECTION 2: ENGINEERING DATA

2.1 GEOTECHNICAL DATA

a. General Geology

The dam on the Chautauqua Lake outlet (Chadakoin River) is located at the south end of Chautauqua Lake in Jamestown, New York.

This area lies within the Appalachian Uplands physiographic province, characterized by steep-sloped hills rising to elevations of 1600 feet or more and isolated by narrow ravine-like valleys.

Bedrock in the region consists of Upper Devonian shales, siltstones and sandstones which have been uplifted and dissected, but are essentially flat-lying. No active faults are known in the area. The city of Jamestown is situated in a region classified as Zone 3 seismicity, as shown on Figure No. 1 of the Recommended Guidelines for Safety Inspection of Dams.

The southwestern portion of New York State was the scene of repeated advances and recessions of continental ice during the Wisconsinan Stage of the Pleistocene; glacial till deposits on uplands in the Jamestown vicinity represent the terminal moraine of one such advance. Lowlands comprise part of the Chautauqua Lake basin and the lake itself is the remnant of a larger body created when ice dammed the outlet. Following final ice retreat, present and former meltwater channels were filled with granular outwash material deposited by streams emanating from the downwasting glacier.

b. Subsurface Conditions

The subsurface material reportedly encountered at the dam site is composed of some 15 feet of random man-placed fill underlain by a dense sand and gravel. The bedrock surface is on the order of 140 feet deep based on water well logs from the surrounding area.

2.2 DESIGN RECORDS

The dam was designed by Erdman, Anthony, Associates of Rochester, New York who prepared a "Environmental Assessment, Warner Dam Replacement, City of Jamestown, New York" as well as structural stability analysis, engineering drawings and contract specification for the construction. Appendix F contains the above referenced report while selected engineering drawings are included in Appendix G. The structural stability analysis performed by the designer is contained in Appendix D. According to the designer, no formal hydrologic analysis was performed. Hydraulic analysis of the tainter gate capacity was performed, the results of which are contained as gate rating curves in Appendix C.

2.3 CONSTRUCTION RECORDS

Construction supervision was provided by both Erdman, Anthony, Associates and New York State Department of Environment Conservation, (NYSDEC). Extensive construction documentation is available in the files of NYSDEC, at 50 Wolf Road, Albany, New York. A Mr. Russell Wege of NYSDEC maintains the construction records and was contacted for certain information as part of the Phase I Inspection.

2.4 OPERATION RECORDS

Detailed operation records are maintained by Mr. Arthur Olsen of the Board of Public Utilities in Jamestown, New York. These records include, at the least, a daily observation of the stage: (lake level) at the U.S.G.S. Gauging Station at Bemus Point, New York, and recording of the pool elevation just upstream of the dam from a staff gauge. In addition, flow or discharge is obtained from the U.S.G.S. Gauging Station at Dow Street in the village of Falconer which is downstream of the dam, as well as which gate(s) is in operation and the total height of the gate(s) opening.

2.5 EVALUATION OF DATA

The data presented in this report has been compiled from information obtained from the city of Jamestown, the files of the New York State Department of Environmental Conservation, the Corps of Engineers office in Pittsburgh, Pennsylvania and the designer, Erdman, Anthony, Associates. The data reviewed is considered adequate and reliable.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General

A visual inspection of the dam was conducted on May 21, 1980. The weather at the time of the inspection was cloudy and rainy. The pool elevation upstream of the dam was 1307.41 with the tailwater at elevation 1300.1.

b. Dam

On the date of inspection only the middle tainter gate was in operation and discharging an estimated 240 cfs. The dam appeared to be in excellent condition. All concrete surfaces were in excellent condition, all mechanical parts appeared to be well lubricated and all gates were operated using the primary electrically driven motors.

No evidence of misalignment, structural cracking or seepage were detected. A slight amount of leakage was occurring along the south side of the southerly most tainter gate.

Debris had accumulated along the upstream face of the dam.

c. River Channel

The river banks for a distance of about 40 feet upstream of the dam are reinforced with an anchored bulkhead using steel sheet piling as the bulkhead wall and deadman. Upstream of the sheet piling the river bank is reinforced with the concrete abutments from the former concrete gravity dam. The concrete abutments were refaced and recapped. This concrete section ends approximately 135 feet upstream of the dam.

Downstream of the dam the anchored bulkhead system is utilized for erosion protection and river bank stabilization for a distance of 25 feet along the north bank to 50 feet along the south bank.

Downstream of the anchored bulkhead section, the north river bank is reinforced with a masonry wall for a distance of about 140 feet. This wall although, deteriorated, appeared to be stable.

Downstream of the anchored bulkhead along the south river bank, the bank has been provided with a laid-up stone protection (or stone paving) on a slope of about 2 horizontal to 1 vertical. This stone paving extends downstream, beyond the bulkhead, a distance of about 50 feet.

Some 700 feet downstream of the dam is the first structure which crosses the river. The structure, the Main Street Bridge is masonry stone arch bridge. Water passes through the two arches which span the river and based on the size of the arches, the structure would significantly constrict the flow during periods of large discharge.

3.2 EVALUATION

The visual inspection of this dam revealed that a slight amount of debris has collected along the upstream face of the dam.

Also, the dam operator had not been instructed on how to operate the tainter gates using the mechanical back-up system.

Finally, leakage was occurring along the seal between the concrete abutment and the south tainter gate.

SECTION 4: OPERATION AND MAINTENANCE PROCEDURE

4.1 PROCEDURE

The normal upstream pool and Chautauqua Lake level are controlled by the operation of the gates for this dam. Downstream flow, likewise is limited by the flow over the spillway. At low flow, the elevation of the pool just upstream of the dam matches that of Chautauqua Lake. However, due to head loss in the channel between the outlet of Chautauqua Lake and the dam, the elevation between Chautauqua Lake and the pool upstream of the dam varies depending on the discharge through the tainter gates. Based on historical records of the former dam the relationships were established for lake levels up to elevation 1310.0.

All gates are exercised regularly on a random basis. The dam operator maintains detailed records of which gate(s) is in use and the corresponding discharge.

All gates are operated with individually controlled electrically driven motors and each gate has a mechanical back-up system. The mechanical system has a gear reduction ratio of 2000:1 and can be operated using a battery powered hand drill.

The initial operation plan for this dam required a minimum release (discharge) rate of 60 cfs for the summer-fall period and 40 cfs for the winter-spring season. In addition, it was determined in the Chautauqua Lake-Chadakoin River Regulation Plan as prepared by the NYSDEC, that the water needs for the Jamestown Municipal Power Plant (located upstream of the dam) could be satisfied by maintaining the 60 cfs flow during the summer months.

4.2 MAINTENANCE OF DAM

The responsibility for maintaining the dam is, as near as can be determined, the city of Jamestown. Since the dam went into service in the fall of 1979 no maintenance as yet has taken place.

A formal maintenance program exists for this structure and is contained in the Operation Plan prepared by the NYSDEC. However, the dam operator was not aware of any maintenance program at the time of this investigation.

4.3 WARNING SYSTEM IN EFFECT

There is no warning system or evacuation plan in effect. The dam is closely monitored and the gates operated in accordance with the current operation plan during periods of heavy runoff.

4.4 EVACUATION

The operation procedure for this structure is satisfactory for periods when electrical power is available to operate the gates. However, provisions should be made to have the dam operator(s) knowledgeable in the operation of the gates when electrical power outages occur. In addition, the dam operator should be made aware of the existing maintenance program and this program should be implemented to ensure proper operation of all mechanical and electrical systems.

SECTION 5: HYDROLOGY/HYDRAULIC

5.1 DRAINAGE AREA CHARACTERISTICS

The Chautauqua Lake/Chadekoin River Basin in which the dam is located is situated in the southwestern portion of New York State and is a tributary to the Allegheny River in the Ohio River Basin. The entire drainage area encompasses an area of 187.2 square miles. Of this total, the lake area is 20.6 square miles, while the surrounding land area contains 166.6 square miles.

Lake Chautauqua extends in a northwest, southeast direction for the majority of its 18 mile drainage basin length. Tributary inflow results from numerous small streams that range from 3 to 8 miles in length which enter around the lake periphery. The lake outflow occurs at its southeastern extremity and is controlled by Warner Dam located approximately 3.3 miles downstream on the Chadakoin River.

The area within the Lake Chautauqua Basin consists of rounded intervalley ridges that rise gradually from graded valley floors. The local ground relief varies from approximately 200 to 400 feet, maximum relief is only about 550 feet between the normal level of the lake, elevation 1308 and the western basin divide in the Goose Creek Headwaters, where an elevation of 1863 feet above sea level is attained.

5.2 ANALYSIS CRITERIA

The hydrologic analysis of this dam was performed using the Corps of Engineers HEC-1 computer program, Dam Safety Version. The spillway design flood selected for analysis was the PMF in accordance with the Recommended Guidelines of the U. S. Army Corps of Engineers.

A synthetic 6 hour unit hydrograph developed by U. S. Army Corps of Engineers, Pittsburgh District was used to compute the Probable Maximum Flood (PMF) hydrograph. The PMF

hydrograph was then routed through Chautauqua Lake and Warner Dam by the "Modified Puls" flood routing procedure to obtain the outflow hydrograph. It was assumed that all three tainter gates will be fully opened when the lake level exceeds elevation 1310.0.

5.3 SPILLWAY CAPACITY

The Warner Dam spillway structure consists of three (3) tainter gates, each being identical in size and each having a length of 26'-8" for a total spillway length of 80 feet. The elevation of the sill of the tainter gates is at 1300. The elevation of the top of the upstream wall on both ends of the dam is at 1311.5.

The spillway discharge rate is closely regulated for lake elevations under 1310.0. The maximum release rates at such elevations were taken from the Chautauqua Lake Maximum Release Rate Curve for Warner Dam. At lake elevations above 1310.0 all three gates are assumed to be fully opened and the corresponding spillway discharges were computed as weir flow with a discharge coefficient of 3.4. The discharges were adjusted to consider the effect of the submergence of the crest due to high tailwater elevation.

The spillways do not have sufficient capacity for discharging the peak outflow for the PMF but will pass one-half the PMF. For the PMF, the peak inflow is 190,189 cfs and the peak outflow is 8,740 cfs. For one-half the PMF, the peak inflow is 95,094 cfs and the peak outflow is 4,164 cfs. The computed spillway capacity for a water surface elevation at the top of the dam (elevation 1311.5) is 6,629 cfs.

We note, the Pittsburgh District Corps of Engineer computed the inflow and outlfow for the Probable Maximum Precipation. Their results indicate an inflow of 160,000 cfs and an outflow of 9700 cfs which reasonably matches the results obtained from our analysis. The loss rates

used by the Corps of Engineers was not published in their report of 1965, as these values may account for the difference in the two analyses.

5.4 RESERVOIR CAPACITY

The reservoir at normal pool impounded by this dam lies primarily within the limits of Chautauqua Lake and the existing channel of the Chadakoin River. The stage-storage curve for Chautauqua Lake was developed by the Pittsburgh District Corps of Engineers. The storage capacity of the reservoir at normal pool elevation is 305,000 acre feet. Within the normal range of lake storage, a one-foot change will accomodate about 1.3 inches of basin runoff.

5.5 FLOODS OF RECORD

The maximum elevation for Chautauqua Lake occurred on March 28, 1946 when the lake reached an elevation of 1311.8 above mean sea level. The highest known discharge recorded at the U.S.G.S. Gauging Station in Falconer, approximately 3 miles downstream of Warner Dam, was 2070 cfs on March 5, 1976.

5.6 OVERTOPPING POTENTIAL

The hydrologic analysis indicates that the dam has sufficient spillway capacity to discharge one-half the PMF, but does not have sufficient capacity to discharge the full PMF. For the PMF peak outflow of 8740 cfs the wall on both ends of the dam would be overtopped. For the peak outflow from one-half the PMF, the computed water surface elevation would be 2.45 feet below the elevation of the top of the dam. The dam would be overtopped by all storm events exceeding 75% of PMF. The elevation of the water surface can not be evaluated for discharges in excess of 75% of the PMF because the channel walls are overtopped.

5.7 EVALUATION

The spillway capacity of this structure is sufficient to discharge 75 percent of the PMF and is, therefore, judged to be inadequate.

SECTION 6: STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURE

a. Visual Observations

The dam and reinforced river banks both upstream and downstream of the dam appeared to be in excellent condition with no sign of instability.

b. Design and Construction Data

The firm of Erdman, Anthony, Associates provided the structural stability analysis performed as part of the design phase for this structure. The structural stability analysis considered both static and seismic loading conditions (inertial force only) with the upstream pool at elevation 1311.0 and the downstream pool at elevation 1297.0. This structural stability analysis indicated adequate safety factors against overturning for both static and seismic loading conditions. However, the computed sliding safety factors of 1.56 for static loads and 1.38 for static and seismic loads combined are both below the recommended minimum value of 3.0 and 1.5, respectively. Furthermore, this analysis was based on the assumption that the sheet piling below the dam is 100 percent effective in reducing hydrostatic uplift. Conversely, shear resistance along the abutments was not taken in account.

Since this design analysis indicated safety factors against sliding below the recommended minimum, a more detailed structural stability analysis was performed as part of this Phase I inspection.

The original design analysis followed by our critique thereof; along with the additional stability analysis which were performed in conjunction with this inspection, are all included in Appendix D. Our analysis considered only those loading conditions for normal winter pool conditions. A structural stability analysis at 1/2 PMF or PMF is not considered appropriate as all tainter gates

would be open and the forces on the dam would be less than with the gates closed, as is the case in our analysis.

Our analysis indicates that under the condition of maximum ice load the structure is stable, however, the factor of safety against sliding is 2.30 which is below the minimum recommended value of 3.0.

The sliding resistance computed assumed no shear resistance due to the embedded sheet piling as this would only be mobilized when the dam moves, which is considered failure. In addition, the analysis assumed the sheet piling below the spillway dam is completely ineffective and, therefore, full hydrostatic uplift pressure is applied along the upstream face with the typical triangular distribution to the downstream tailwater elevation.

An effective sheet pile cut-off wall would significantly reduce the hydrostatic uplift pressures along the base of the dam as can be demonstrated readily through construction of a flow net. However, although this approach is valid in designing a dam, it is not rational in our opinion for a study of this nature to make an assumption regarding its effectiveness unless it can be so determined through actual field data.

Therefore, we recommend piezometers be installed and monitored to determine the actual magnitude and distribution of the hydrostatic uplift pressures below the dam and additional structural stability analysis be performed.

c. Seismic Stability

The designer's seismic stability analysis was based on criteria not presently recommended for a structure in Seismic Zone 3. For this reason a seismic structural stability analysis was performed as part of the Phase I inspection. Our analysis was performed using the Zanger

hydrodynamic pressure distribution which is similar to the Westergaard distribution recommended by the Corps of Engineers guidelines. Our analysis indicates adequate safety factors against sliding and overturning for the combined seismic and static loads analyzed.

SECTION 7: ASSESSMENT/RECOMMENDATIONS

7.1 ASSESSMENT

a. Safety

The Phase I Inspection of the Warner Dam did not reveal conditions which constitute an immediate hazard to human life and property of the downstream residents.

From the available data the total spillway capacity is capable of discharging 75 percent of the PMF without flow overtopping the upstream reinforced river banks. This spillway is, therefore, judged to be inadequate.

The structural stability analysis performed as part of this investigation indicates the dam has safety factors against sliding under maximum ice loading below that recommended by the Corps of Engineers Guidelines.

Since the designer's structural stability computations performed as part of the design phase indicates safety factors below those recommended and our analysis indicates potential stability problems, we recommend that field investigations and monitoring be performed to determine and monitor hydrostatic uplift pressures during the life of the structure.

b. Adequacy of Information

The information reviewed for this Phase I inspection is considered adequate.

c. Need for Additional Investigation

As discussed above, the structural stability analysis indicates a potentially unstable condition under maximum ice loading.

Therefore, a field investigation and monitoring program should be initiated within 6 months from the time of notification to the owner to determine the actual magnitude and distribution of hydrostatic uplift pressures at

the base of the dam. From the results of this investigation a detailed structural stability analysis should be performed.

7.2 RECOMMENDED REMEDIAL MEASURE

1) The revised stability analyses should be performed within three months after installation of the piezometers and whatever corrective measures may be dictated by the results initiated as soon as practical thereafter.

2) Develop and implement within 3 months, a warning system, and an evacuation plan for downstream residents in the event of large discharge. In addition, the dam operator should be provided with thorough instruction and the necessary equipment for operation of the mechanical tainter gate hoisting system.

3) The responsibility for providing routine and preventive maintenance should be clearly established and the appropriate parties notified and the maintenance program implemented within 3 months.

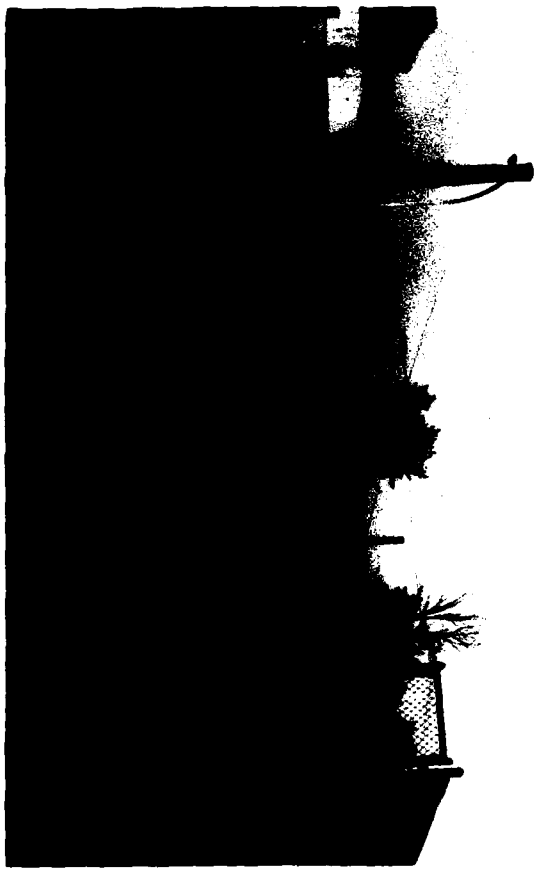
4) Remove debris from the upstream face of the dam within 3 months.

5) Leakage through the seal along the south tainter gate should be monitored on a daily basis and documented in the daily log book maintained by the dam operator so that it can be corrected if it becomes worse.

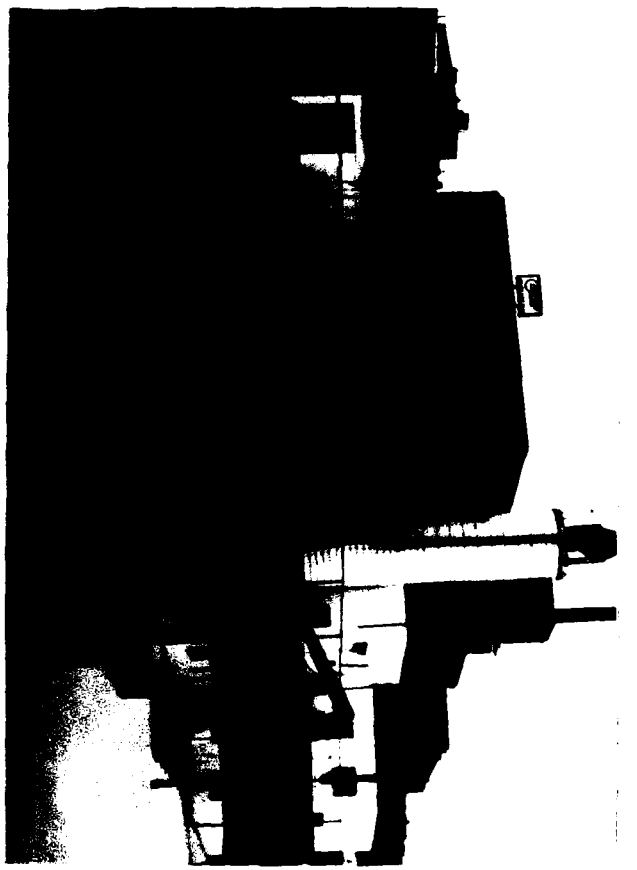
APPENDIX A

PHOTOGRAPHS

View of downstream face of dam



Continutaion of photo above. View of upstream face of dam.



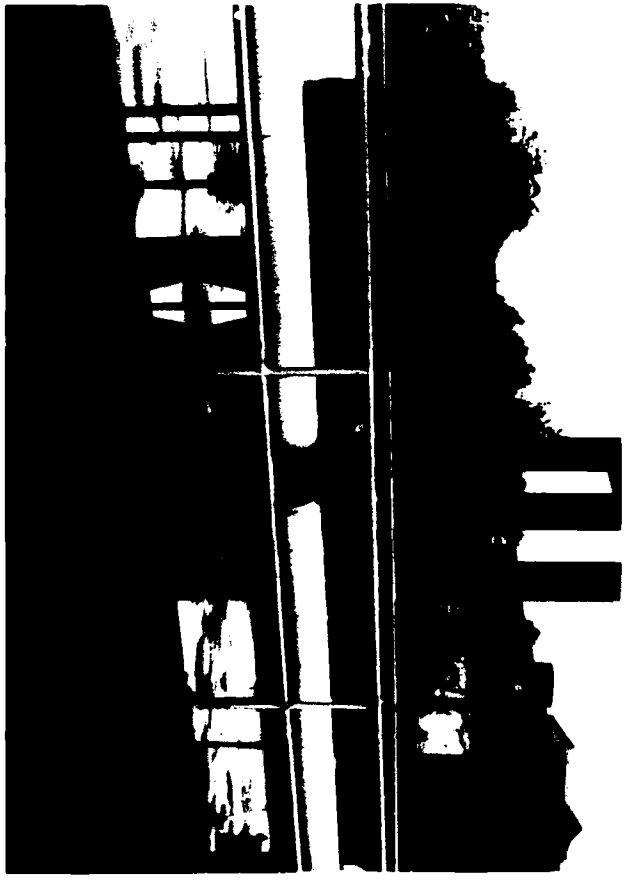
View of retaining structures along north side of upstream channel. (Note concrete in center is part of old masonary dam

Continuation of photo above. View of downstream face of dam.

View of downstream channel from dam.

View of pin pedestal and attachment of tainter gate arm to concrete pier.

View of trash behind (upstream side) tainter gate.



View of electric motor, control panel and covered drive shaft for controlling tainter gate.

APPENDIX B

VISUAL INSPECTION CHECKLIST

THOMSEN ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS & GEOLOGISTS

VISUAL INSPECTION CHECKLIST

1) **Basic Data**

a. General

Name of Dam Warner Dam (Chautauqua [illegible])

I.D. # 92-16[illegible] DEC. Dam No. NY 7[illegible]

River Basin Allegheny

Location: Town [illegible] County Chautauqua

U.S.G.S. Quadrangle Jamestown

Stream Name Chadakoin River - Chautauqua Lake

Tributary of Allegheny River

Latitude (N) 42° 05.[illegible]' Longitude (W) 79° 14.5'

Type of Dam Concrete Gravity Dam

Hazard Category C

Date(s) of Inspection [illegible]

Weather Conditions [illegible]

Reservoir Level at Time of Inspection [illegible]

Tailwater Level at Time of Inspection 1300 [illegible]

b. Inspection Personnel [illegible] II - [illegible]

Charles E. [illegible] - [illegible] Engineers Inc

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

[illegible] - [illegible] Dam [illegible], [illegible] Public Utilities, Jamestown NY 14701 Telephone # 716-[illegible] - 2309

d. History:

Date Constructed 1978-1979 Date(s) Reconstructed

Designer Erdman, Anthony, Associates Rochester New York

Constructed by Herbert F. Darling [illegible]

Owner NYS Department of Environmental Conservation

e. Seismic Zone Zone [illegible]

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VISUAL INSPECTION CHECKLIST

2) Embankment

a. Characteristics

1) Embankment Material Not Applicable

2) Cutoff Type _____

3) Impervious Core _____

4) Internal Drainage System _____

5) Miscellaneous _____

b. Crest

1) Vertical Alignment _____

2) Horizontal Alignment _____

3) Surface Cracks _____

4) Miscellaneous _____

c. Upstream Slope

1) Slope(Estimate)(V:H) N. A.

2) Undesirable Growth or Debris, Animal Burrows _____

3) Sloughing, Subsidence or Depressions _____

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VISUAL INSPECTION CHECKLIST

4) Slope Protection _____

5) Surface Cracks or Movement at Toe _____

d. Downstream Slope

1) Slope (Estimate - V:H) N.A.

2) Undesirable Growth or Debris, Animal Burrows _____

3) Sloughing, Subsidence or Depressions _____

4) Surface Cracks or Movement at Toe _____

5) Seepage _____

6) External Drainage System (Ditches, Trenches; Blanket)

7) Condition Around Outlet Structure _____

8) Seepage Beyond Toe _____

e. Abutments-Embankment Contact

N.A.

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VISUAL INSPECTION CHECKLIST

1) Erosion at Contact _____

2) Seepage Along Contract _____

3) Drainage System

a. Description of System None

b. Condition of System _____

c. Discharge from Drainage System _____

4) Instrumentation (Monumentation/Surveys, Observation Wells, Weirs, Piezometers, Etc.) _____

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VISUAL INSPECTION CHECKLIST

5) ~~Reservoir~~ Upstream Pool

a. Slopes Very gentle to nearly level

b. Sedimentation None Suspected due to recent construction of this structure

c. Unusual Conditions Which Affect Dam Intake Pipe for City Generation Plant upstream of dam requires upstream pool to be maintained at least above Elev. 1306.0

6) Area Downstream of Dam

a. Downstream Hazard (No. of Homes, Highways, etc.) Many Homes & Businesses along Downstream Channel

b. Seepage, Unusual Growth None

c. Evidence of Movement Beyond Toe of Dam None

d. Condition of Downstream Channel Good; Bridge downstream of Dam appears to be a constriction.

7) Spillway(s) (Including Discharge Conveyance Channel)

a. General Tainter Gate Dam with 3 Gates each measuring 26' 8" wide separated by concrete piers. Gates are electrically hoisted with a mechanical backup system

b. Condition of Service Spillway Excellent

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VISUAL INSPECTION CHECKLIST

c. Condition of Auxiliary Spillway none

d. Condition of Discharge Conveyance Channel Good. Bridge Downstream of Structure appears to be a constriction in the channel

8) Reservoir Drain/Outlet None

Type: Pipe _____ Conduit _____ Other _____
Material: Concrete _____ Metal _____ Other _____
Size: _____ Length _____
Invert Elevations: Entrance _____ Exit _____
Physical Condition (Describe): Unobservable _____
Material: _____
Joints: _____ Alignment _____
Structural Integrity: _____
Hydraulic Capability: _____
Means of Control: Gate _____ Valve _____ Uncontrolled _____
Operation: Operable _____ Inoperable _____ Other _____
Present Condition (Describe): _____

Additional Remarks and/or Observations:

No Warning System
Dam is monitored on a regular basis during heavy precipitation
Daily Records of Operation are maintained by Dam Operators

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CONSULTING GEOTECHNICAL ENGINEERS SEATTLE, WASH.

9) Structural

a. Concrete Surfaces Excellent

b. Structural Cracking None Observed

c. Movement - Horizontal & Vertical Alignment (Settlement)
None Observed

d. Junctions with Abutments or Embankments
Excellent

e. Drains - Foundation, Joint, Face None

f. Water Passages, Conduits, Sluices Dam Spillway is
the only water passage

g. Seepage or Leakage Slight amount of water
along south side of south water gate

h. Joints - Construction, etc. Excellent

i. Foundation Unobservable

j. Abutments

k. Control Gates 3 Taintor Gates each 26'8" wide by 11' high. Primary Control - Electric Driven Hoisting Gear. Back-up Control - Mechanical Drive Hoisting Gear

l. Approach & Outlet Channels Unobservable

m. Energy Dissipators (Plunge Pool, etc.) None

n. Intake Structures N.A.

o. Stability Appeared Satisfactory

p. Miscellaneous All gates operated electrically on date of inspection, back up mechanical system not operated as flow rate is not conducive with the procedure

APPENDIX C

HYDROLOGIC/HYDRAULIC ENGINEERING DATA AND COMPUTATIONS

THOMSEN ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS & GEOLOGISTS

CHECK LIST FOR DAMS HYDROLOGIC AND HYDRAULIC ENGINEERING DATA

AREA-CAPACITY DATA:

		Elevation (ft.)	Surface Area (acres)	Storage Capacity (acre-ft.)
1)	Top of Dam	1311.5	[illegible]	441,000
2)	Design High Water (Max. Design Pool)	1310.0*	15,000	412,000
3)	Auxiliary Spillway Crest	N.A. →		
4)	Pool Level with Flashboards	N.A. →		
5)	Service Spillway Crest	1300.0	[illegible]	[illegible]

* Maximum Design Elevation of Chautauqua Lake

DISCHARGES

		Volume (cfs)
1)	Average Daily	343*
2)	Spillway @ Maximum High Water (Top of Dam)	[illegible]
3)	Spillway @ Design High Water (Elev. 1310.0)	[illegible]
4)	Spillway @ Auxiliary Spillway Crest Elevation	N.A.
5)	Low Level Outlet	N.A.
6)	Total (of all facilities) @ Maximum High Water	[illegible]
7)	Maximum Known Flood March 5, 1976	2070*

* - Based on Records of the USGS Gauging Station at Falconer, N.Y. from 1935 to 1976 for Chadakoin River Flows

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OUTLET STRUCTURES/EMERGENCY DRAWDOWN FACILITIES: NONE

Type: Gate_____ Sluice _____ Conduit _____ Penstock_____

Shape: _____

Size: _____

Elevations: Entrance Invert _____

Exit Invert _____

Tailrace Channel: Elevation _____

HYDROMETEROLOGICAL GAGES:

Type: _____

Location: Jamestown Airport

Records:

Date - _____

Max. Reading - _____

FLOOD WATER CONTROL SYSTEM:

Warning System: None

Method of Controlled Releases (mechanisms):

Electrically Controlled Hoisting Equipment with a Mechanical Backup System

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CREST: Top of [illegible] wall ELEVATION: 13[illegible]

Type: Tainter Gate Dam - Concrete [illegible]

Width: 47' Length: 9[illegible]'

Spillover Concrete [illegible]

Location Spillover occurs in 90 ± the 9[illegible] foot length, separated only by concrete piers

SPILLWAY:

PRINCIPAL		EMERGENCY
1300.00	Elevation	None
	Type	
80 feet	Width	
	Type of Control	
	Uncontrolled	
	Controlled:	
Tainter Gates	Type (Flashboards; gate)	
3	Number	
26'8" wide by 11' high	Size/Length	
	Invert Material	
	Anticipated Length of operating service	
	Chute Length	
The same Elevation 1300.0	Height Between Spillway Crest & Approach Channel Invert (Weir Flow)	

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DRAINAGE AREA: 190 sq miles

DRAINAGE BASIN RUNOFF CHARACTERISTICS:

Land Use - Type: Ranging from recreational to agricultural and forested

Terrain - Relief: Ranges from nearly level to steep

Surface - Soil: Highly variable ranging from glacio-lacustrine silt and clay to very dense glacial till deposits

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

Runoff Potential is not expected to change significantly because of the size of the drainage area

Potential Sedimentation problem areas (natural or man-made; present or future)

Considered Minimal

Potential Backwater problem areas for levels at maximum storage capacity including surcharge storage:

Chautauqua Lake levels above Elevation 1313.0 would cause inundation of many residences surrounding the lake shore

Dikes - Floodwalls (overflow & non-overflow) - Low reaches along the Reservoir perimeter: None

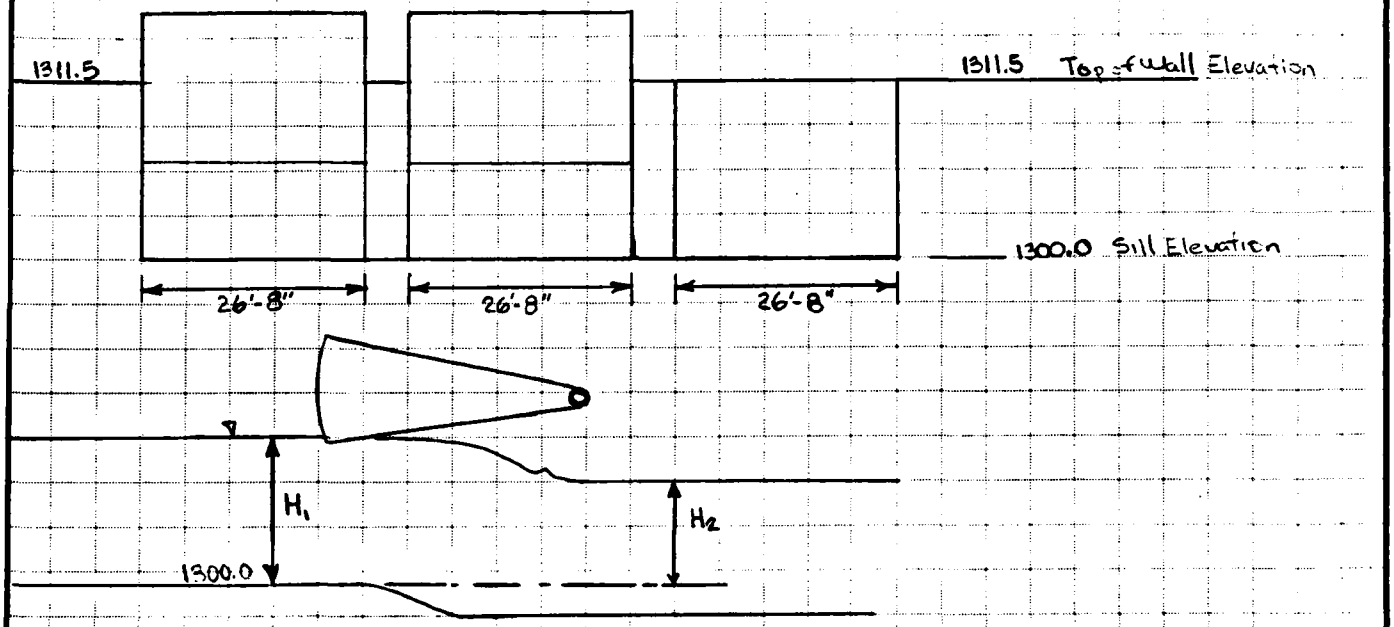
Location: _____

Elevation: _____

McFarland-Johnson Engineers, Inc.
171 Front Street
BINGHAMTON, NEW YORK 13905

JOB Wappinger Hydraulic computations
SHEET NO. _____ OF _____
CALCULATED BY R. Wolst DATE 9/3/80
CHECKED BY _____ DATE _____
SCALE _____

Stage - Discharge Computation:



$C = 3.4$, $L = 80'$

$Q_1 = CLH_1^{3/2}$

Lake Elevation	Elev. @ Dam	H_1	Free Discharge Q_1	H_2	$\left(\frac{H_2}{H_1}\right)^{1.5}$	$\frac{Q}{Q_1}$	Q (submerged)
1314.15	1308.5	8.5	6740	7.40	.81	.53	3570
1314.67	1309.0	9.0	7314	7.65	.78	.56	4112
1315.20	1309.5	9.5	7964	7.96	.76	.57	4539
1315.74	1310.0	10.0	8601	8.25	.75	.58	4988
1316.27	1310.5	10.5	9254	8.58	.74	.59	5459
1316.79	1311.0	11.0	9923	8.81	.72	.62	6152
1317.32	1311.5	11.5	10,607	9.10	.70	.625	6629

McFarland-Johnson Engineers, Inc.
171 Front Street
BINGHAMTON, NEW YORK 13905

JOB [illegible]
SHEET NO. _____ OF _____
CALCULATED BY R. WOLFT DATE 9/2/80
CHECKED BY _____ DATE _____
SCALE _____

Stage-Discharge Data

Lake Elevation	Elevation Upstream of Dam	Tailwater Elevation	Discharge (cfs)
1308.25	1308.25		950
1314.15	1308.5	1307.40	3570
1314.67	1309.0	1307.65	4112
1315.20	1309.5	1307.96	4539
1315.74	1310.0	1308.25	4988
1316.27	1310.5	1308.58	5459
1316.79	1311.0	1308.81	6152
1317.32	1311.5	1309.10	6629

Assumptions

① All 3 gates will be opened when Lake Elevations exceed 1310.0. At Lake Elevations under 1310.0 the Discharge was obtained from the Chautauqua Lake-Chadakoin River Maximum Release Rate vs. Lake Elevation Curve.

② The discharge for each gate was determined by the weir flow formula $Q = CLH^{3/2}$ where $C = 3.4$. In cases where the tailwater elevation submerges the crest of the dam (elevation 1300.0) a discharge for a submerged weir condition was computed utilizing the method developed in page 5-18 of the Handbook of Hydraulics by King & Brater.

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171 Front Street
BINGHAMTON, NEW YORK 13905

JOB [illegible] Dam # [illegible]
SHEET NO _____ OF _____
CALCULATED BY [illegible] DATE 9/3/[illegible]
CHECKED BY _____ DATE _____
SCALE _____

Stage - Storage Data

Lake Elevation	Elevation @ Dam (Upstream)	(Acre-ft.) Storage Volume
1308.25	1308.25	305,000
1314.25	1308.5	390,000
1314.67	1309.0	397,000
1315.20	1309.5	405,000
1315.74	1310.0	412,000
1316.27	1310.5	420,000
1316.79	1311.0	430,000
1317.32	1311.5	441,000

Normal Pool Elevation - 1308.25

Assumptions

① Elevation @ Warner Dam and at Lake Chautauqua are approximately equal at the Normal Pool Elevation of 1308.25

② At Lake elevations above normal pool (1308.25) the elevation at the dam is lower than Lake elevation. This is due to the head loss throughout the distance from the Lake to the Dam when the outlet gates of the Dam are opened. To approximate this differential at higher Lake Stages, historical Lake vs. Dam Elevations were plotted against each other and a best-fit line showing their relationship was extrapolated to a top of dam elevation of 1311.5. (see Graph Warner Dam Elevation vs Chautauqua Lake)

③ At Lake elevations above 1310.0 all 3 gates are assumed to be open. The historical dam vs. Lake Chautauqua Elevations that were utilized to determine the Dam-Lake elevation differential all met this condition.

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171 Front Street
BINGHAMTON, NEW YORK 13905

JOB Warren Town Hydrologic Study

SHEET NO. _____ OF _____

CALCULATED BY R. Waidt DATE 7/2/84

CHECKED BY _____ DATE _____

SCALE _____

Stage - Storage Data (cont.)

④. Stage-Storage Relationships were taken from the Stage-Storage Curve for Lake Chautauqua - Chadakoin River prepared by the U.S. Army Corps of Engineers, Pittsburgh District. Note that for each stage at the Dam a corresponding Lake Elevation was used to determine the storage.

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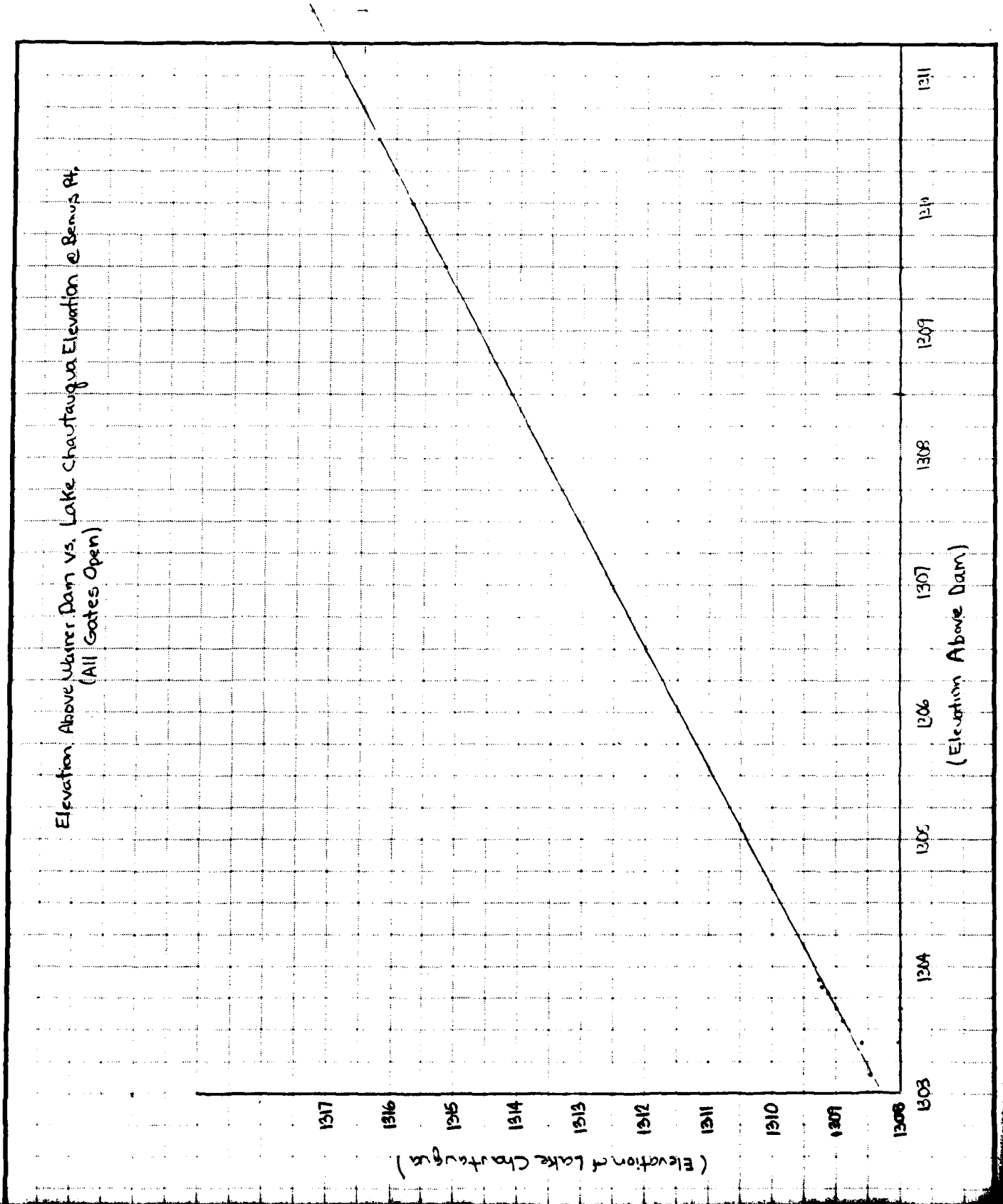
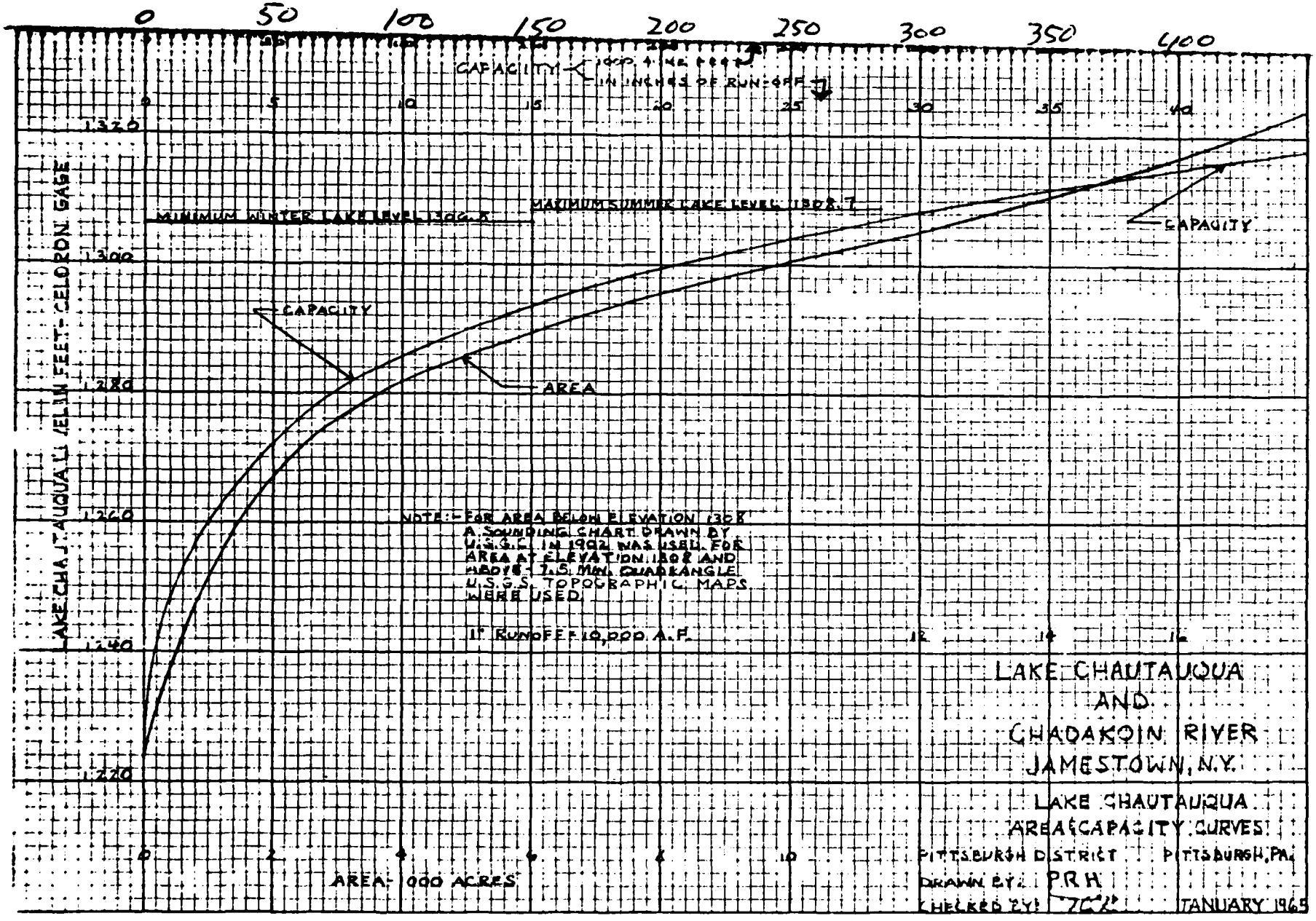
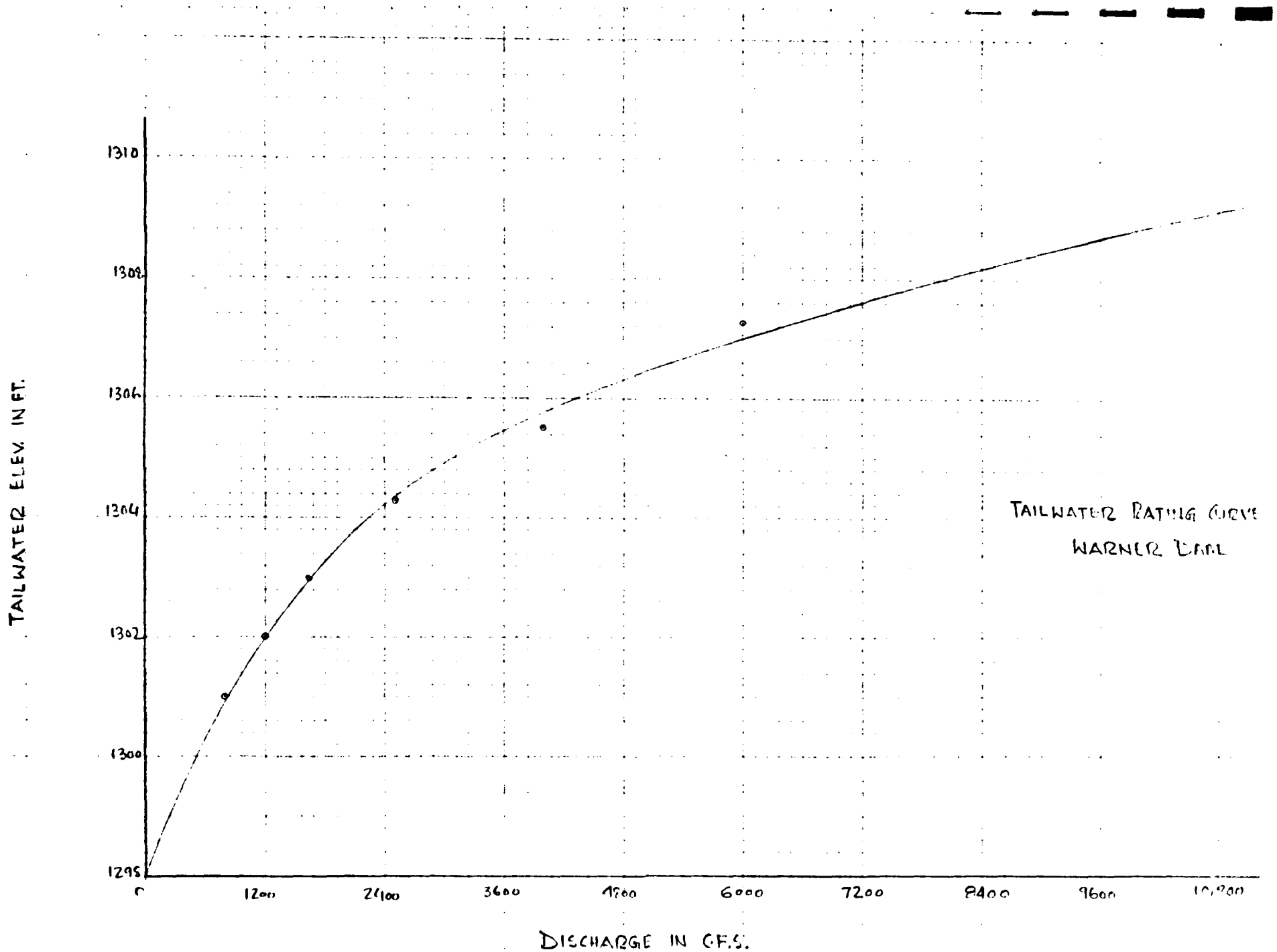
171 Front Street
BINGHAMTON, NEW YORK 13905

JOB Warner Dam Hydroelectric Study
SHEET NO _____ OF _____
CALCULATED BY R. Weidt DATE 7/21/80
CHECKED BY _____ DATE _____
SCALE _____

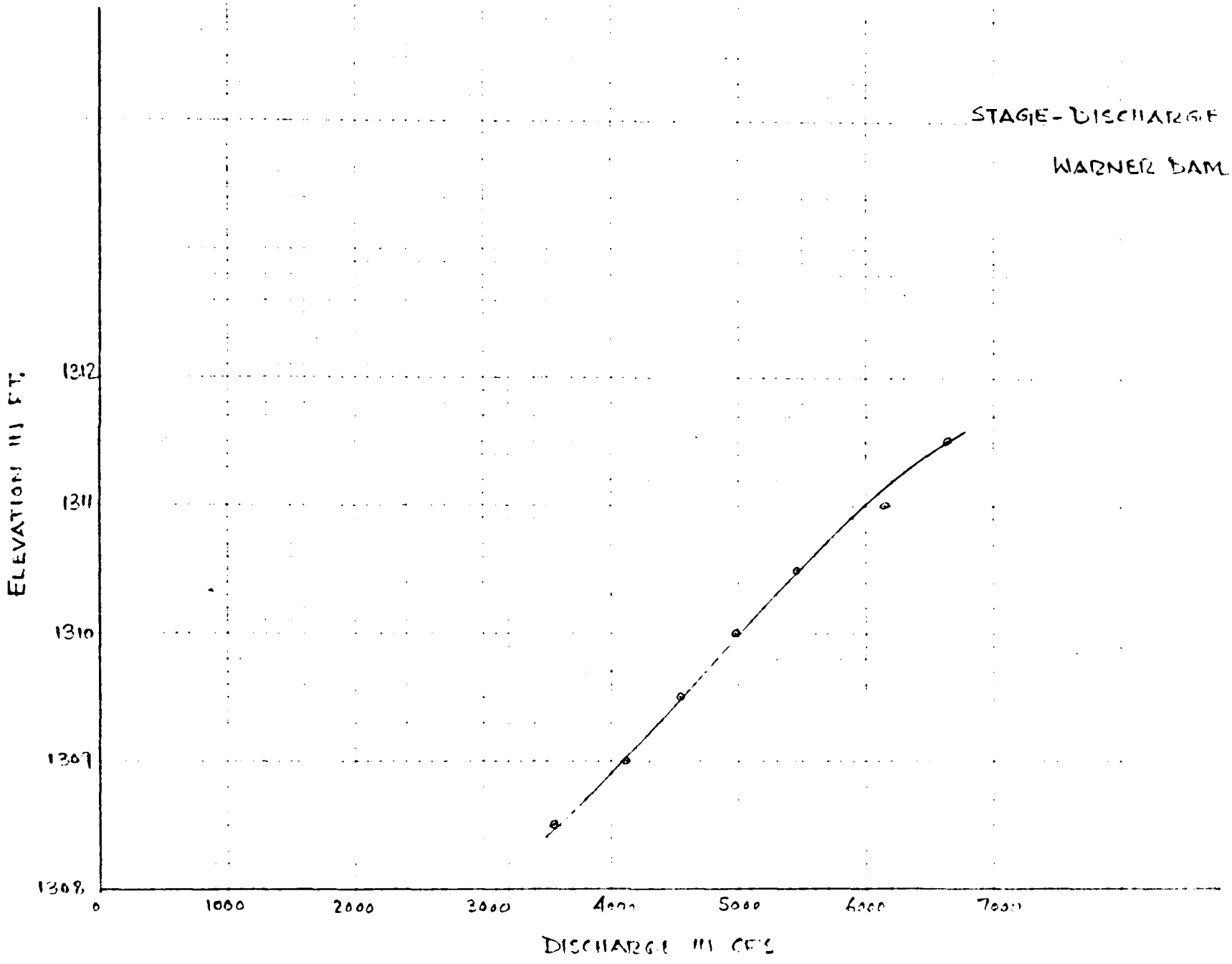


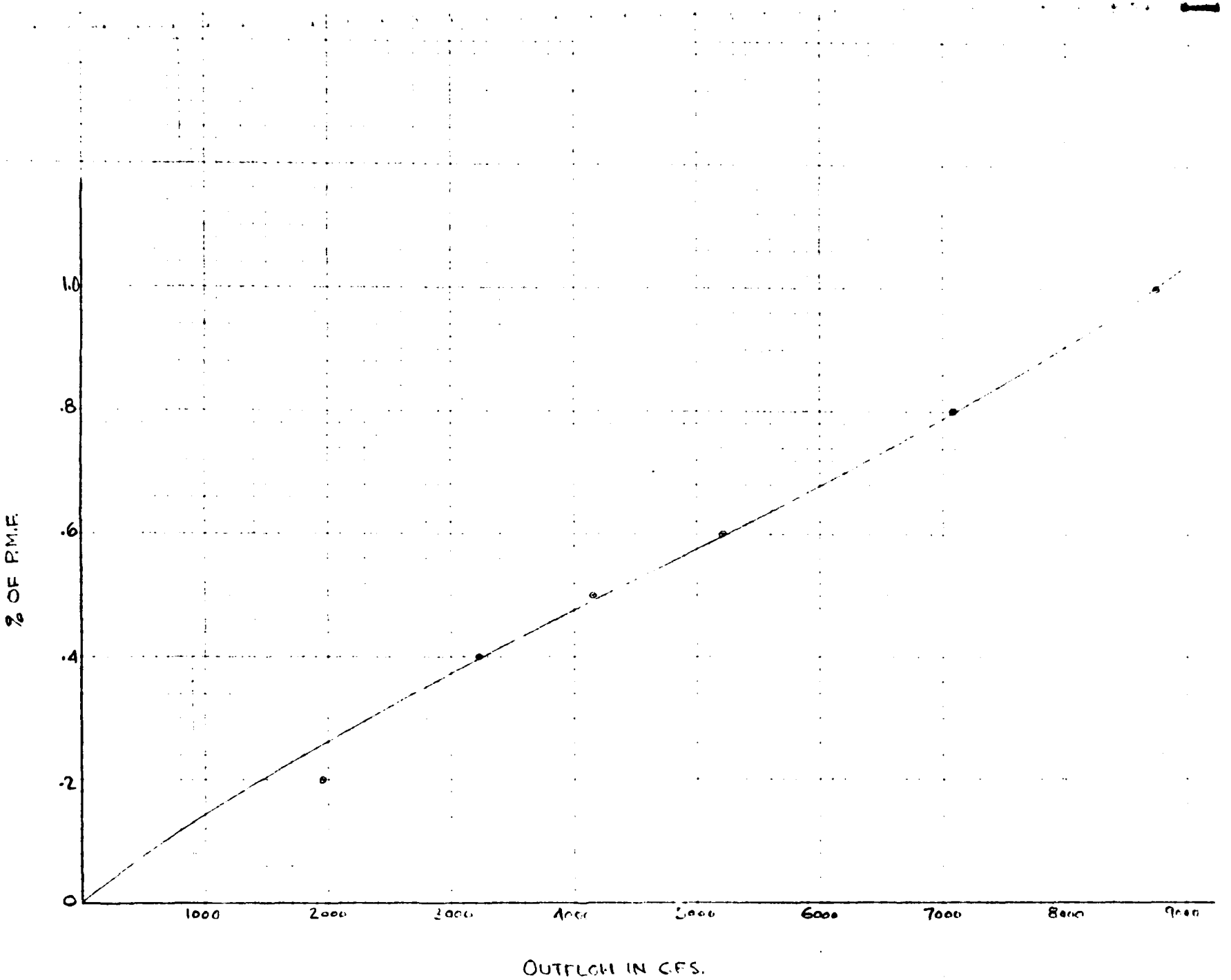

0 50 100 150 200 250 300 350 400
CAPACITY (1000 ACRE FEET)
IN INCHES OF RUN-OFF
0 5 10 15 20 25 30 35 40
1320
1300
1280
1260
1240
1220
LAKE CHAUTAUQUA (ELEVATION IN FEET - CELORON GAGE)
MINIMUM WINTER LAKE LEVEL 1306.5
MAXIMUM SUMMER LAKE LEVEL 1308.5
CAPACITY
CAPACITY
AREA
NOTE:- FOR AREA BELOW ELEVATION 1308
A SOUNDING CHART DRAWN BY
U.S.G.S. IN 1902 WAS USED. FOR
AREA AT ELEVATION 1308 AND
ABOVE - U.S. MAN. QUADRANGLE
U.S.G.S. TOPOGRAPHIC MAPS
WERE USED.
1" RUNOFF 10,000 A.F.
0 2 4 6 8 10 12 14 16
AREA - 000 ACRES
LAKE CHAUTAUQUA
AND
CHADAKOIN RIVER
JAMESTOWN, N.Y.
LAKE CHAUTAUQUA
AREA-CAPACITY CURVES
PITTSBURGH DISTRICT PITTSBURGH, PA.
DRAWN BY: PRH
CHECKED BY: JCC
JANUARY 1965


TAILWATER ELEV. IN FT.
1310
1308
1306
1304
1302
1300
1298
0
1200
2400
3600
4800
6000
7200
8400
9600
10,800
DISCHARGE IN C.F.S.
TAILWATER RATING CURVE
WARNER DAM

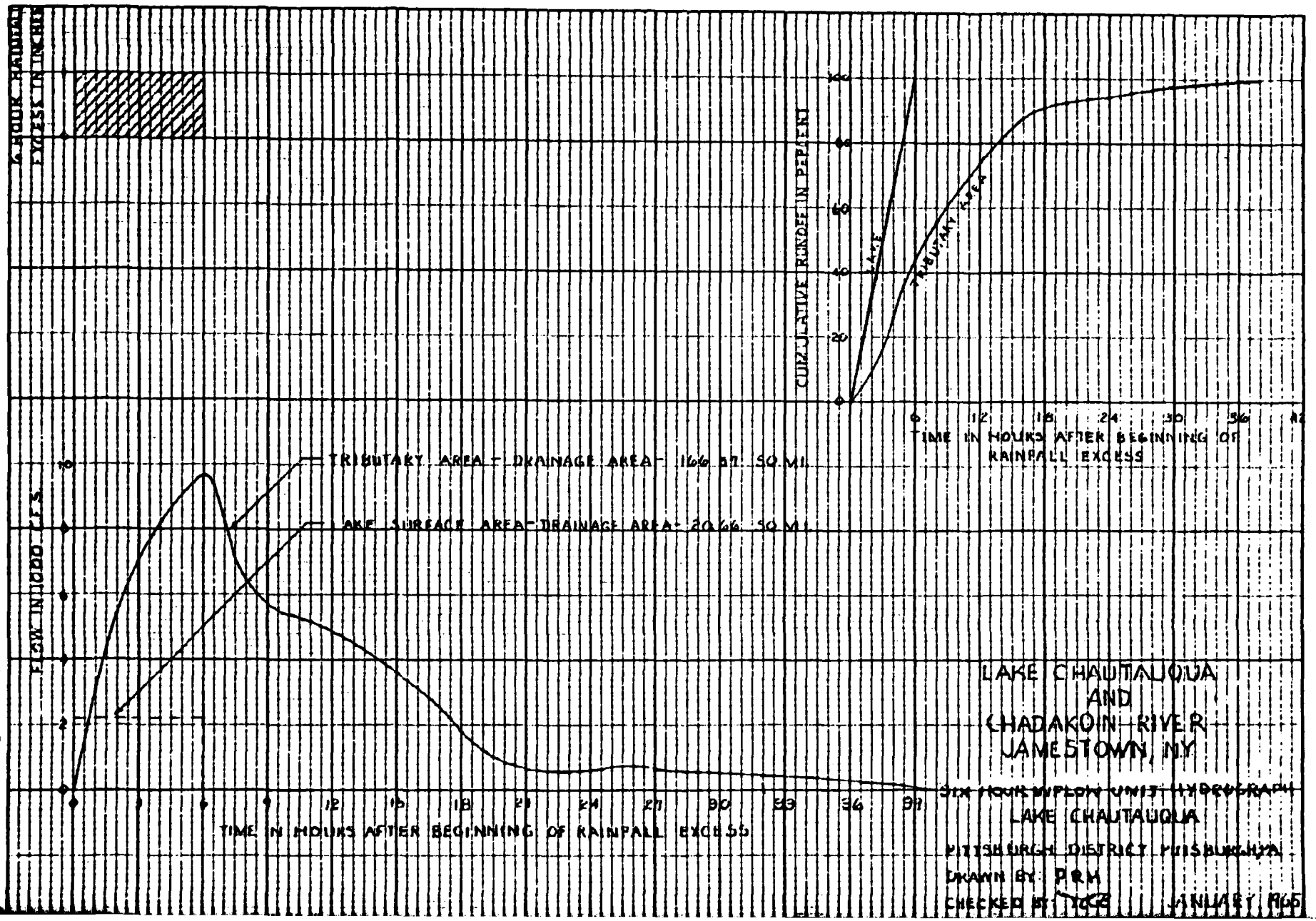
STAGE-DISCHARGE CURV

WARNER DAM




1.0
.8
.6
.4
.2
0
% OF P.M.F.
1000
2000
6000
7000
OUTFLOW IN C.F.S.

K&E 12 x 20 TO THE INCH 46 1973
7 X 10 INCHES
KEUFFEL & ESSER CO.



LAKE CHAUTAUQUA
AND
CHADAKOIN RIVER
JAMESTOWN, NY

SIX HOUR SYNTHETIC UNIT HYDROGRAPH
LAKE CHAUTAUQUA

PITTSBURGH DISTRICT PITTSBURGH, PA.
DRAWN BY: PRM
CHECKED BY: [illegible] JANUARY 1976

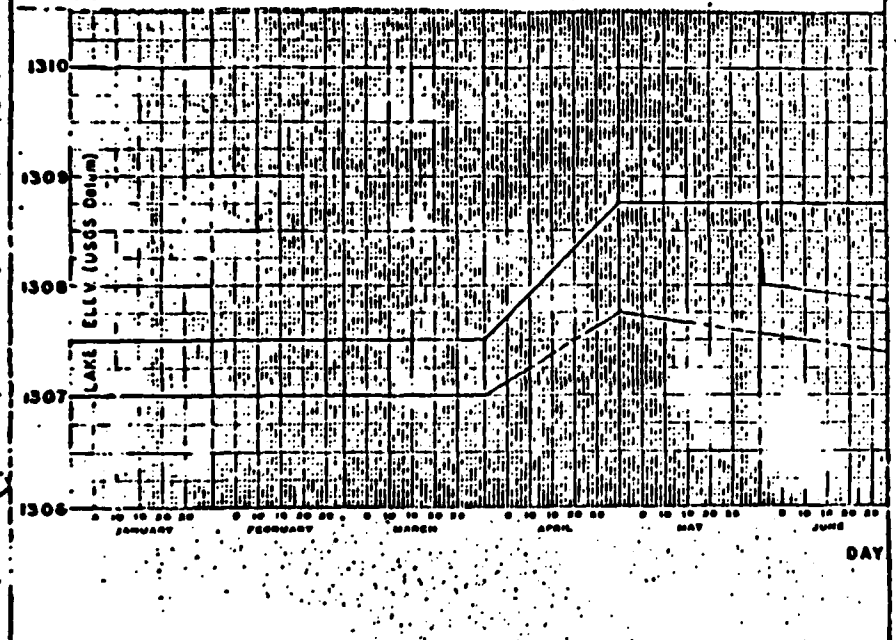
APPENDIX I PLATE 12

EXHIBIT B
Enclosure 1

LAKE OUTFLOW OPERATIONAL SCHEDULE

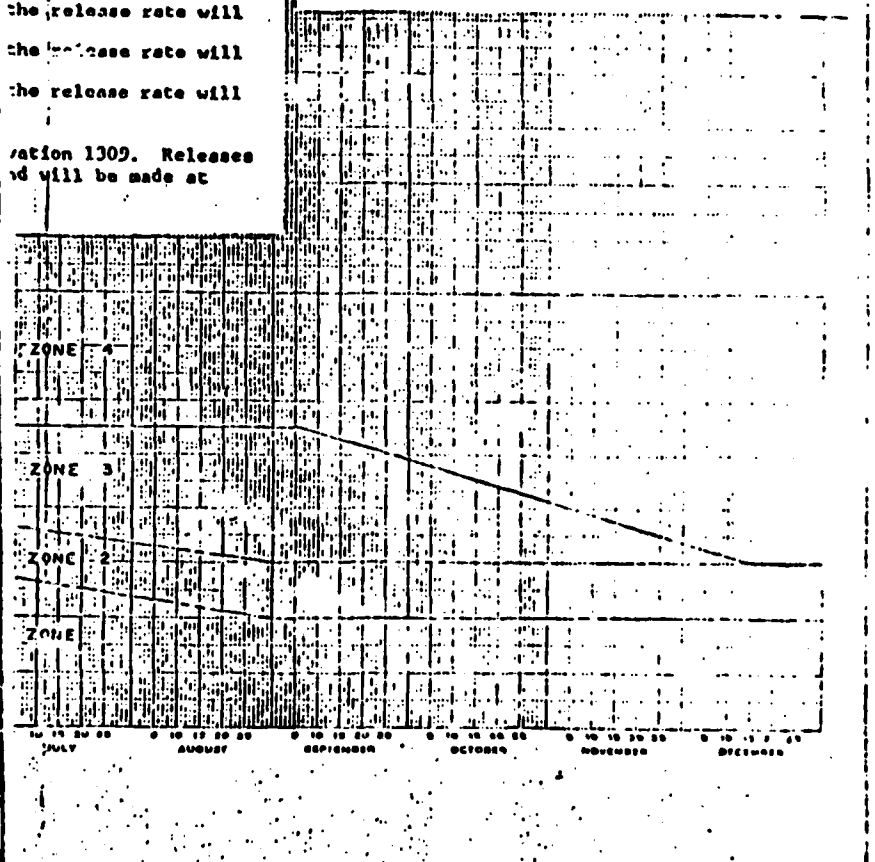
Zone	Purpose	Release Rate in cfs	Operations
4	Flood Control	60 to 1270 (1)	Releases up to 1270 cfs will be ma to evacuate lake storage as rapidly as possible. W n lake stages exceed elevation 1310, releases in excess f 1270 cfs should be made.
3	Conservation	60 (2)	When the lake level is in this zon the release rate will be maintained at 60 cfs.
2	Conservation	40 (2)	When the lake level is in this zon the release rate will be maintained at 40 cfs.
1	Conservation	20 (2)	When the lake level is in this zon the release rate will be maintained at 20 cfs.

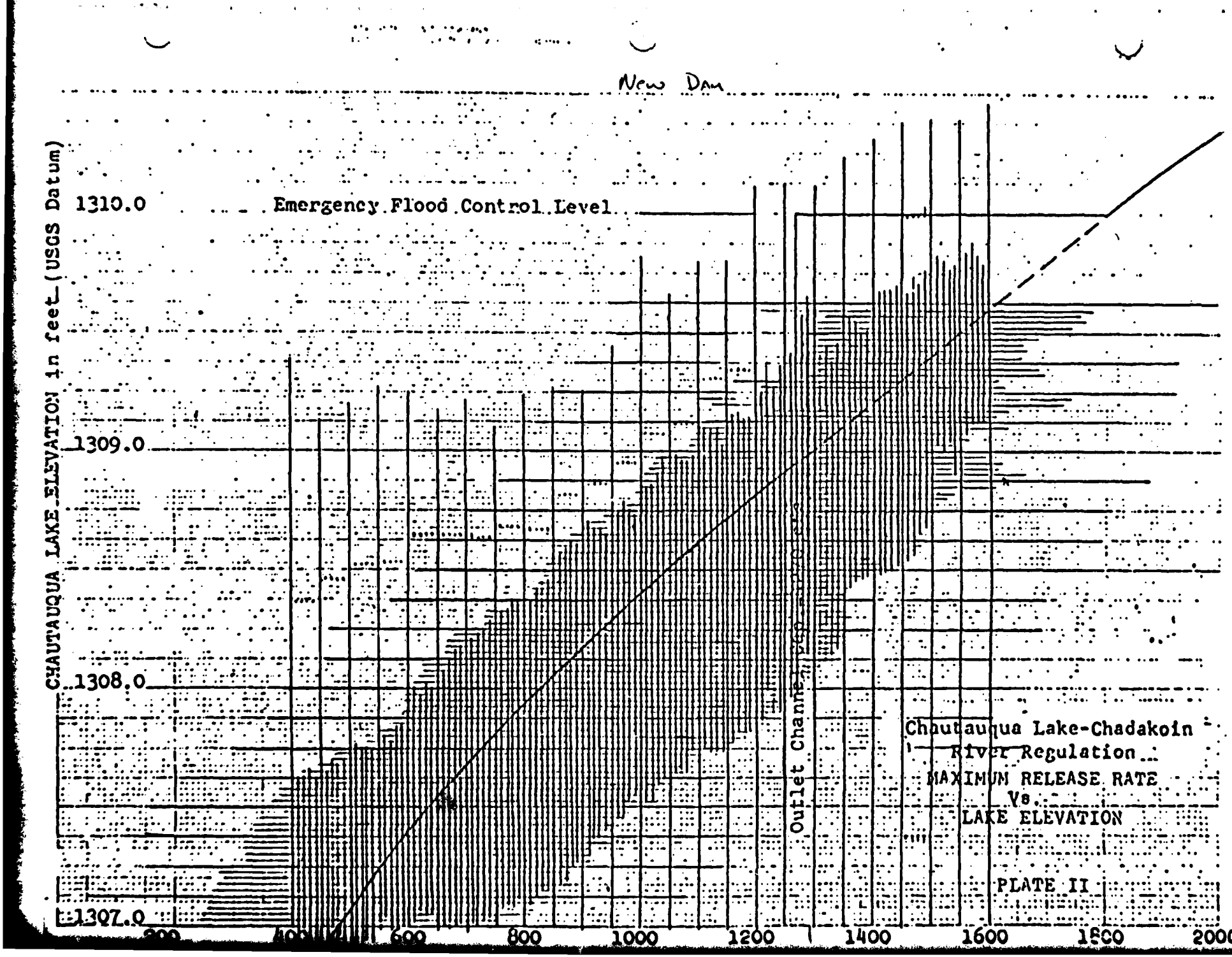
(1) Releases up to 1270 cfs are not possible until lake stage exceeds e vation 1309. Releases in this zone are affected by channel capacity of the outlet channel nd will be made at the maximum capacity possible.
(2) Releases are both a minimum and maximum.

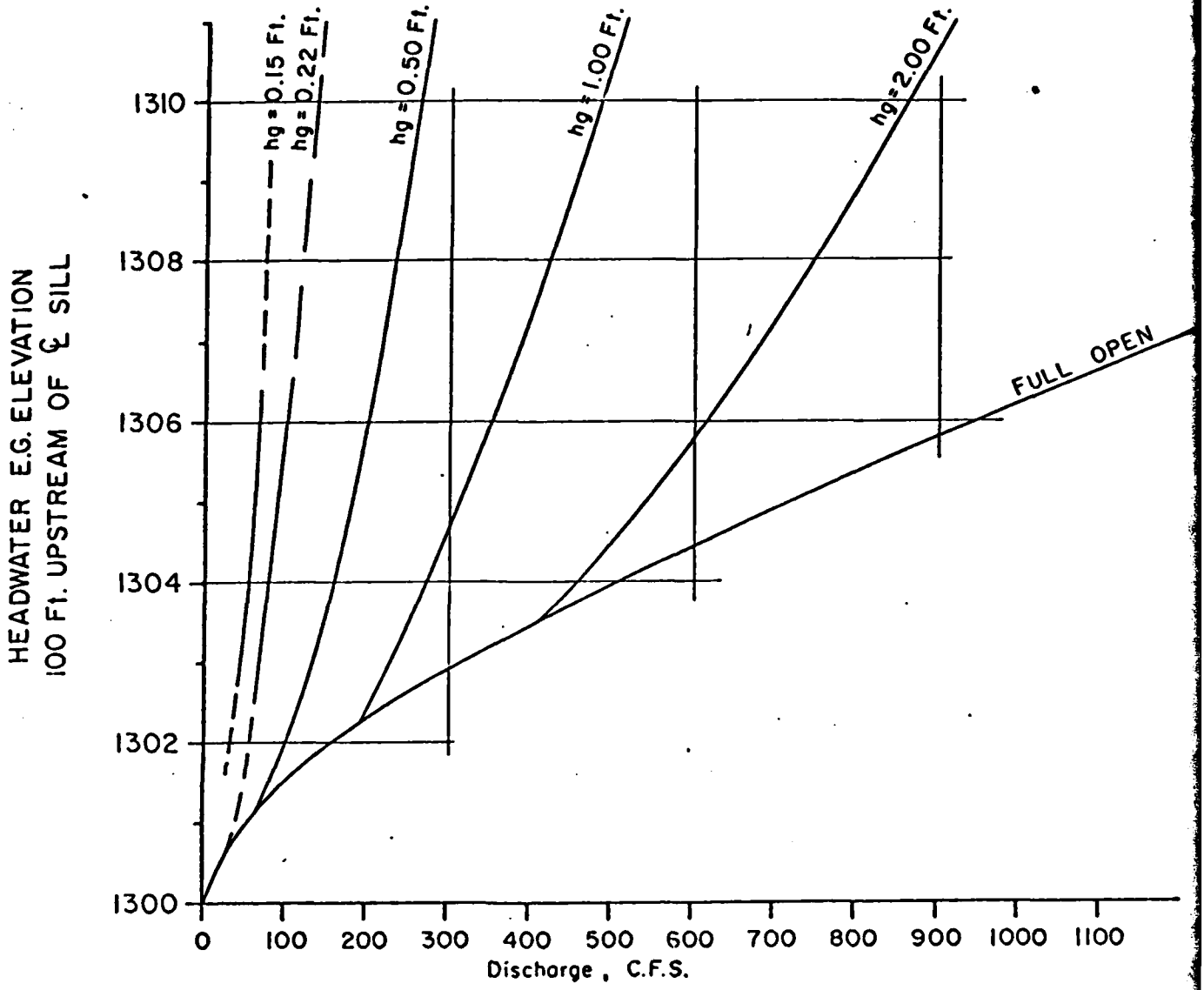


ALLEGHENY RIVER BASIN
CHAUTAUQUA LAKE SUB-BASIN

LAKE LEVEL REGULATION PLAN
20-40-60-1270 WS III




New Dam
Emergency Flood Control Level
1310.0
1309.0
1308.0
1307.0
CHAUTAUQUA LAKE ELEVATION in feet (USGS Datum)
Outlet Channel
200
400
600
800
1000
1200
1400
1600
1800
2000
Chautauqua Lake-Chadakoin
River Regulation
MAXIMUM RELEASE RATE
Vs.
LAKE ELEVATION
PLATE II



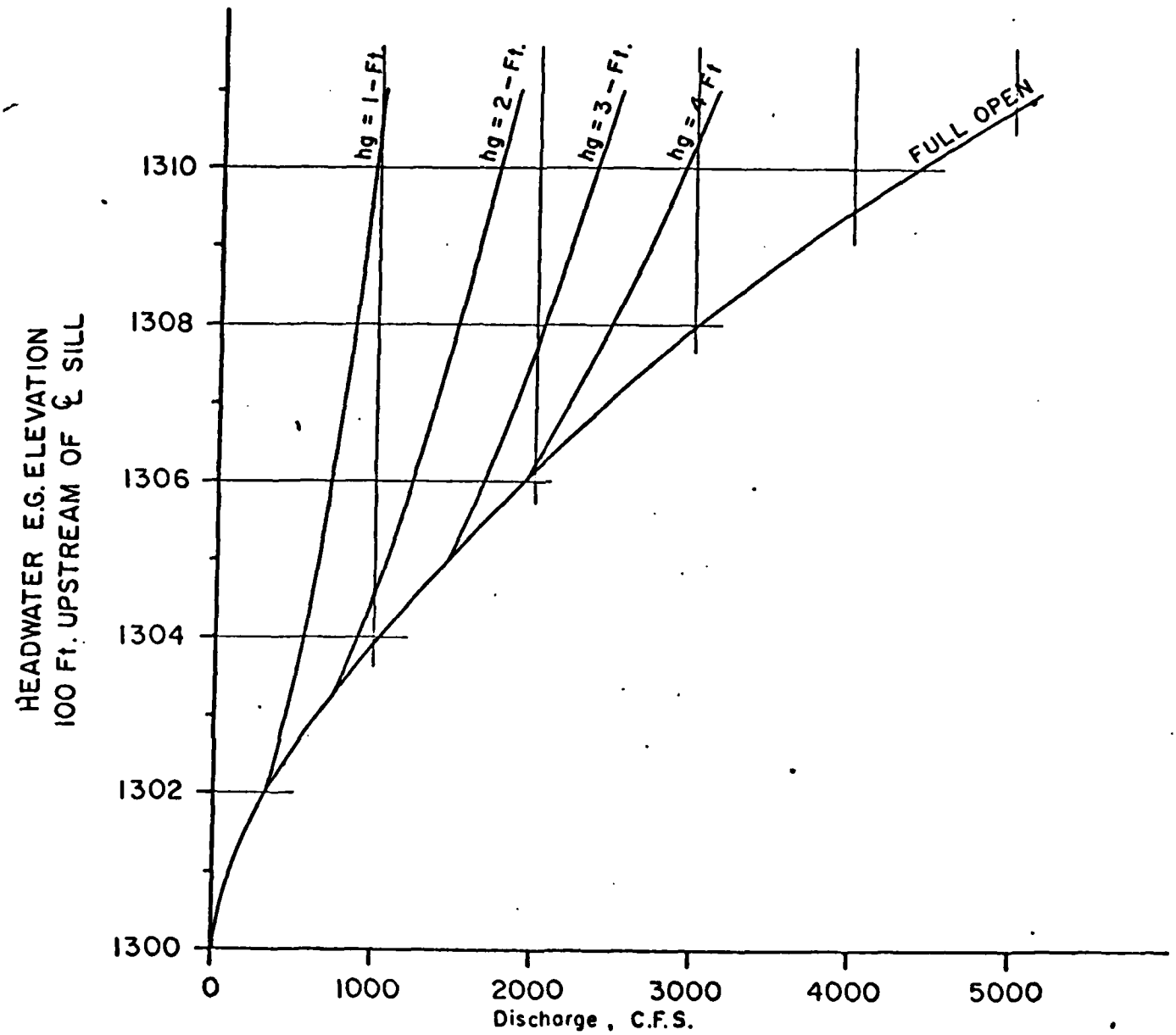
RATING CURVE
FOR
ONE GATE
WARNER DAM

EDWAN ANTHONY ASSOCIATES

4/9/80 A J.D.

WIDTH OF GATE: 26'-8"
ELEVATION OF SILL: 1300.0
E.G. = ENERGY GRADIENT
hg = VERTICAL OPENING OF ONE GATE
(OTHER GATES FULLY CLOSED)

EXHIBIT 5a

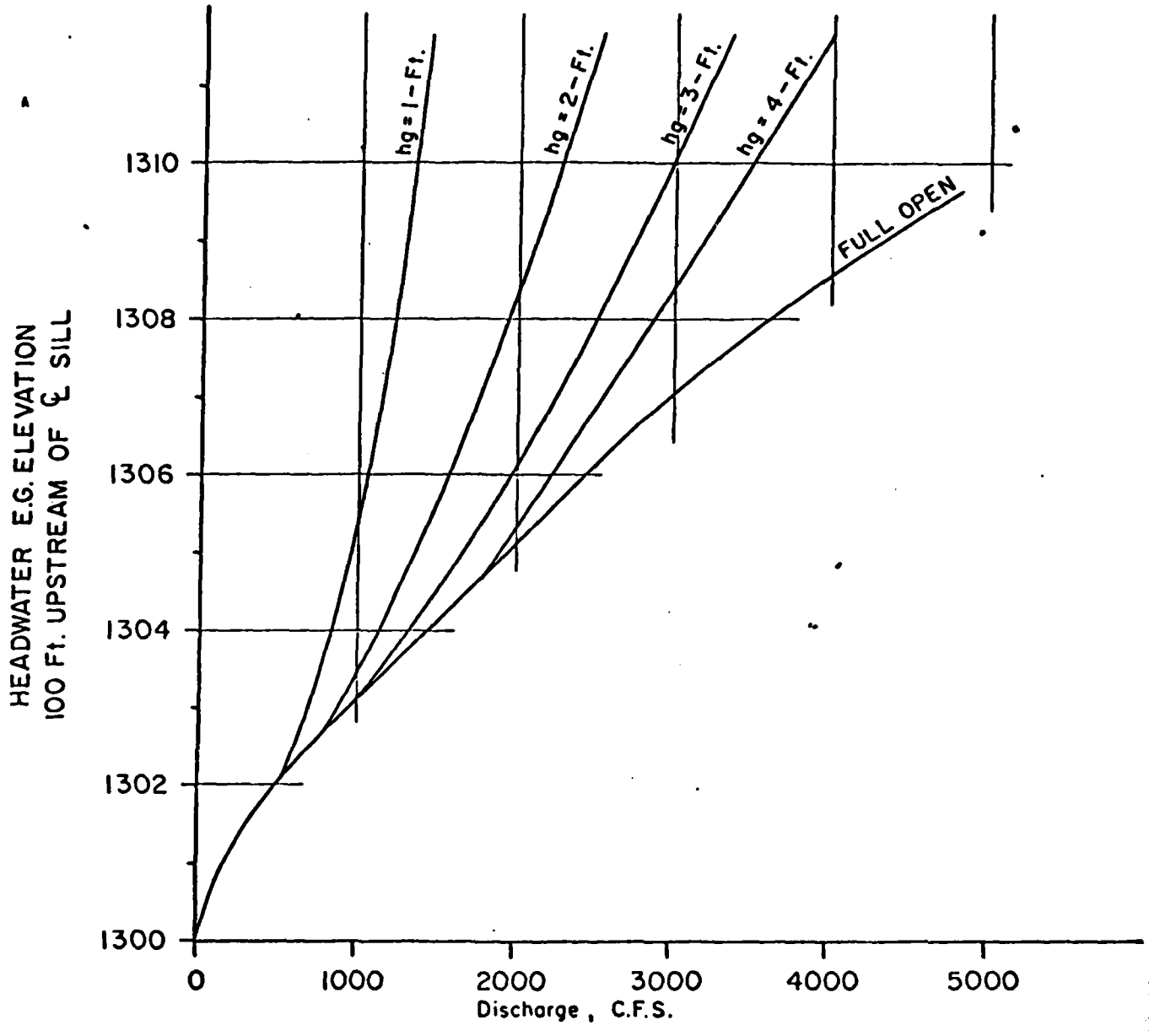


RATING CURVE
FOR
TWO GATES
WARNER DAM

WIDTH OF GATE: 26' - 8"
ELEVATION OF SILL: 1300.0
E.G. = ENERGY GRADIENT
hg = VERTICAL OPENING OF EACH GATE
(OTHER GATE FULLY CLOSED)

EXHIBIT 5b

EDMAN ANTHONY ASSOCIATES
4/3/80 A.J.D.



RATING CURVE
FOR
THREE GATES
WARNER DAM

WIDTH OF GATE: 26' - 8"
ELEVATION OF SILL: 1300.0
E.G. = ENERGY GRADIENT
hg = VERTICAL OPENING OF EACH GATE

EXHIBIT 5c

EDMAN ANTHONY ASSOCIATES
4/9/80 A J.D.

```
*********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  26 FEB 79
*********************************
    1                 A1                    ANALYSIS OF DAM OVERTO
    2                 A2                   HYDROLOGIC-HYDRAULIC AN
    3                 A3                   RATIOS OF PMF ROUTED TH
    4                 B      40        0        0        0        0
    5                 B1      5
    6                 J       1        0        1
    7                 J1     .2       .4       .5       .6       .8
    8                 K       0        1        0        0        0
    9                 K1       COMPUTATION OF INFLOW HYDROGRAPH
   10                 M       1       -1      187        0      187
   11                 P       0     22.8       80       90      100
   12                 T
   13                 U       6
   14                 U1  11800     4850     1800      600      420
   15                 X      -2      -.1        2
   16                 K       1        2        0        0        0
   17                 K1        ROUTING OF INFLOW HYDROGRAPH
   18                 Y       0        0        0        1        1
   19                 Y1      1        0        0        0        0
   20                 Y2305000   390000   397000   405000   412000
   21                 Y3    950     3570     4112     4539     4988
   22                 K      99
```

```
LYSIS OF DAM OVERTOPPING USING RATIOS OF PMF
DLOGIC-HYDRAULIC ANALYSIS OF SAFETY OF NY750 DAM
OS OF PMF ROUTED THROUGH LAKE CHAUTAUQUA
 0        0        0        0        0        0        0        0

 1
 5       .6       .8        1
 0        0        0        0        1        0        0        0
 INFLOW HYDROGRAPH FROM GIVEN UNIT HYDROGRAPH
 7        0      187        0        0        0        1        0
 0       90      100      110
                                    1       .1

 0      600      420       20
 2
 0        0        0        0        1        0        0        0
 FLOW HYDROGRAPH
 0        1        1
 0        0        0        0   305000
 0   405000   412000   420000   430000   441000
 2     4539     4986     5459     6152     6629
```

PREVIEW OF SEQUENCE OF STREAM NETWORK CALCULATIONS

RUNOFF HYDROGRAPH AT	1
ROUTE HYDROGRAPH TO	2
END OF NETWORK	

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

          TIME OF EXECUTION       6-SEP-79    14:41:02

                                    ANALYSIS OF DAM OVERTOPPING USING RATIOS
                                    HYDROLOGIC-HYDRAULIC ANALYSIS OF SAFETY OF
                                    RATIOS OF PMF ROUTED THROUGH LAKE CHAUTAUQU

                                              JOB SPECIFICATION
                        NO      NWAR      NMAIN      IDAY       IHR      IMIN     METRC      IPL
                        40         6          0         0         0         0         0
                                                    JOPER       NNT    LROPT     TRACE
                                                         5         0         0         0

                                                MULTI-PLAN ANALYSES TO BE PERFORMED
                                                    NPLAN= 1 NRTIO= 6 LRTIO= 1
                                RTIOS=    0.20    0.40    0.50    0.60    0.80    1.00

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

                                                SUB-AREA RUNOFF COMPUTATION

                                        COMPUTATION OF INFLOW HYDROGRAPH FROM GIVEN UNIT HYDRO

                                                ISTAG    ICOMP    IECON    ITAPE     JPLT     JPRT
                                                    1        0        0        0        0        0

                                                          HYDROGRAPH DATA
                                    IHYDG     IUHG    TAREA     SNAP    TRSDA    TRSPC    RATIO     IS
                                        1       -1   167.00     0.00   167.00     0.00    0.000

                                                          PRECIP DATA
                                              SPFE      PMS       R6       R12      R24      R48       R
                                              0.00    22.80    80.00    90.00   100.00   110.00     0.
TRSPC COMPUTED BY THE PROGRAM IS 0.861

                                                          LOSS DATA
                                    LROPT    STRKR    DLTKR    RTIOL    ERAIN    STRKS    RTIOK    STRTL
                                        0     0.00     0.00     1.00     0.00     0.00     1.00     1.00

                                                    GIVEN UNIT GRAPH, NUHGO=    6
                                    11800.     4650.     1800.      600.      420.       20.
                                                UNIT GRAPH TOTALS  19490. CFS OR 0.97 INCHES OVER

                                                          RECESSION DATA
                                                    STRTQ=    -2.00    QRCSN=    -0.10    RTIOR=

                                    0                                 END-OF-PERIOD FLOW
                                  MO.DA   HR.MN   PERIOD     RAIN     EXCS     LOSS     COMP Q      MO.DA   HR.MN
                                   1.01    6.00        1     0.06     0.00     0.06        349.      1.00    6.00
                                   1.01   12.00        2     0.20     0.00     0.20        326.      1.00   12.00
                                   1.01   18.00        3     1.61     0.56     1.05        [illegible]  1.00   18.00

                                                                McFARLAND-JOHNSON ENGINEERS, INC.
```

14:41:02

LYSIS OF DAM OVERTOPPING USING RATIOS OF PMF
OLOGIC-HYDRAULIC ANALYSIS OF SAFETY OF NY750 DAM
OS OF PMF ROUTED THROUGH LAKE CHAUTAUQUA

JOB SPECIFICATION

IDAY	IHR	IMIN	METRC	IPLT	IPRT	NSTAN
0	0	0	0	0	0	0

JOPER	NMI	DROPI	TRACE
5	0	0	0

MULTI-PLAN ANALYSES TO BE PERFORMED
NPLAN= 1 NRTIO= 6 LRTIO= 1
.40 0.50 0.60 0.80 1.00

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

SUB-AREA RUNOFF COMPUTATION

INFLOW HYDROGRAPH FROM GIVEN UNIT HYDROGRAPH

ICOMP	IECON	ITAPE	JPLT	JPRT	INAME	ISTAGE	IAUTO
0	0	0	0	0	1	0	0

HYDROGRAPH DATA

A	SNAP	TRSDA	TRSPC	RATIO	ISNOW	ISAME	LOCAL
0	0.00	167.00	0.00	0.000	0	1	0

PRECIP DATA

	R6	R12	R24	R48	R72	R96
0	80.00	90.00	100.00	110.00	0.00	0.00

LOSS DATA

RTIOL	ERAIN	STRKS	RTIOK	STRTL	CNSTL	ALSMX	RTIMP
1.00	0.00	0.00	1.00	1.00	0.10	0.00	0.00

GIVEN UNIT GRAPH, NUHGO= 6
600. 420. 20.
TOTALS 19490. CFS OR 0.97 INCHES OVER THE AREA

RECESSION DATA
= -2.00 QRCSN= -0.10 RTIOR= 2.00

END-OF-PERIOD FLOW

LOSS	COMP Q	MO.DA	HR.MN	PERIOD	RAIN	EXCS	LOSS	COMP Q
0.08	349.	1.00	6.00	21	0.00	0.00	0.00	8547.
0.20	326.	1.00	12.00	22	0.00	0.00	0.00	8068.
0.20	304.	1.00	18.00	23	0.00	0.00	0.00	7528.

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1.02	0.00	4	0.12	0.00	0.12	2962.	1.07	0.00
1.02	6.00	5	0.80	0.20	0.00	3000.	1.07	6.00
1.02	12.00	6	2.01	1.41	0.60	10168.	1.07	12.00
1.02	18.00	7	16.07	15.47	0.60	190189.	1.07	18.00
1.03	0.00	8	1.21	0.01	0.00	85040.	1.08	0.00
1.03	6.00	9	0.00	0.00	0.00	31909.	1.08	6.00
1.03	12.00	10	0.00	0.00	0.00	18530.	1.08	12.00
1.03	18.00	11	0.00	0.00	0.00	17295.	1.08	18.00
1.04	0.00	12	0.00	0.00	0.00	10137.	1.09	0.00
1.04	6.00	13	0.00	0.00	0.00	15056.	1.09	6.00
1.04	12.00	14	0.00	0.00	0.00	14048.	1.09	12.00
1.04	18.00	15	0.00	0.00	0.00	13107.	1.09	18.00
1.05	0.00	16	0.00	0.00	0.00	12229.	1.10	0.00
1.05	6.00	17	0.00	0.00	0.00	11410.	1.10	6.00
1.05	12.00	18	0.00	0.00	0.00	10646.	1.10	12.00
1.05	18.00	19	0.00	0.00	0.00	9933.	1.10	18.00
1.06	0.00	20	0.00	0.00	0.00	9268.	1.11	0.00

	PEAK	6-HOUR	24-HOUR	72-HOUR	TO
CFS	190169.	137617.	81376.	30615.	
CMS	5380.	3897.	2304.	1037.	
INCHES		6.85	16.19	21.86	
MM		173.88	411.28	555.17	
AC-FT		68240.	161408.	217874.	
THOUS CU M		84173.	199094.	268744.	

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 1

70.	65.	1374.	590.	733.	3038.	38038.
3459.	3227.	3011.	2810.	2621.	2446.	2282.
1729.	1614.	1506.	1405.	1311.	1223.	1141.
865.	807.	753.	702.	655.	611.	571.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TO
CFS	38036.	27523.	16275.	7323.	
CMS	1077.	779.	461.	207.	
INCHES		1.37	3.24	4.37	
MM		34.76	82.20	111.03	
AC-FT		13648.	32282.	43575.	
THOUS CU M		16835.	39819.	53749.	

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 2

140.	130.	2748.	1193.	1467.	7275.	76076.
6918.	6455.	6022.	5619.	5243.	4892.	4564.
3459.	3227.	3011.	2810.	2621.	2446.	2282.
1729.	1614.	1506.	1405.	1311.	1223.	1141.

	PEAK	6-HOUR	24-HOUR	72-HOUR	T
CFS	76076.	55047.	32551.	14646.	
CMS	2154.	1559.	922.	415.	
INCHES		2.74	6.48	8.74	
MM		69.55	164.51	222.07	
AC-FT		27296.	64503.	87150.	
THOUS CU M		33669.	79637.	107	

0.12	2962.	1.07	0.00	24	0.00	0.00	0.00	7024.
0.00	3000.	1.07	6.00	25	0.00	0.00	0.00	6554.
0.00	16168.	1.07	12.00	26	0.00	0.00	0.00	6115.
0.00	190189.	1.07	18.00	27	0.00	0.00	0.00	5705.
0.00	85046.	1.08	0.00	28	0.00	0.00	0.00	5323.
0.00	31909.	1.08	6.00	29	0.00	0.00	0.00	4967.
0.00	18536.	1.08	12.00	30	0.00	0.00	0.00	4634.
0.00	17295.	1.08	18.00	31	0.00	0.00	0.00	4324.
0.00	10137.	1.09	0.00	32	0.00	0.00	0.00	4034.
0.00	15056.	1.09	6.00	33	0.00	0.00	0.00	3764.
0.00	14046.	1.09	12.00	34	0.00	0.00	0.00	3512.
0.00	13107.	1.09	18.00	35	0.00	0.00	0.00	3277.
0.00	12229.	1.10	0.00	36	0.00	0.00	0.00	3057.
0.00	11410.	1.10	6.00	37	0.00	0.00	0.00	2853.
0.00	10646.	1.10	12.00	38	0.00	0.00	0.00	2662.
0.00	9933.	1.10	18.00	39	0.00	0.00	0.00	2483.
0.00	9268.	1.11	0.00	40	0.00	0.00	0.00	2317.
				SUM	22.09	18.24	3.85	584038.
					(561.)	(463.)	(98.)	(16538.11)

K	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
.	137617.	81376.	30615.	582706.
.	3897.	2304.	1037.	16500.
	6.85	16.19	21.86	26.99
	173.88	411.28	555.17	736.26
	68240.	161408.	217874.	288945.
	84173.	199094.	268744.	356408.

RAPH AT STA 1 FOR PLAN 1, RTIO 1

590.	733.	3038.	38038.	17009.	6382.	3707.
2810.	2021.	2446.	2282.	2129.	1987.	1854.
1405.	1311.	1223.	1141.	1065.	993.	927.
702.	655.	611.	571.	532.	497.	463.

	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
	27523.	16275.	7323.	116541.
	779.	461.	207.	3300.
	1.37	3.24	4.37	5.80
	34.78	82.20	111.03	147.25
	13046.	32282.	43575.	57769.
	16835.	39819.	53749.	71282.

APH AT STA 1 FOR PLAN 1, RTIO 2

1193.	1467.	7275.	76076.	34018.	12764.	7414.
5619.	5243.	4892.	4564.	4259.	3973.	3707.
2810.	2021.	2446.	2282.	2129.	1987.	1854.
1405.	1311.	1223.	1141.	1065.	993.	927.

	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
	55047.	32551.	14646.	233062.
	1559.	922.	415.	6600.
	2.74	6.48	8.74	11.59
	69.55	164.51	222.07	294.50
	27296.	64563.	87150.	115578.
	33609.	79437.	107	142563.

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 3

174.	163.	3435.	1491.	1633.	9094.	95094.
8647.	8066.	7526.	7024.	6554.	6115.	5705.
4324.	4034.	3764.	3512.	3277.	3057.	2853.
2702.	2017.	1882.	1756.	1638.	1529.	1426.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL
CFS	95094.	68809.	40686.	18307.	
CMS	2693.	1948.	1152.	518.	
INCHES		3.42	8.10	10.93	
MM		86.94	205.64	277.58	
AC-FT		34120.	80704.	108937.	
THOUS CU M		42066.	99547.	134372.	

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 4

209.	195.	4122.	1789.	2200.	10913.	114113.
10377.	9682.	9034.	8429.	7864.	7338.	6846.
5188.	4841.	4517.	4214.	3932.	3669.	3423.
2594.	2421.	2258.	2107.	1966.	1834.	1712.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL
CFS	114113.	82570.	48826.	21969.	
CMS	3231.	2338.	1383.	622.	
INCHES		4.11	9.72	13.11	
MM		104.33	246.77	333.10	
AC-FT		40944.	96845.	130724.	
THOUS CU M		50504.	119456.	161246.	

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 5

279.	260.	5490.	2386.	2933.	14550.	152151.
13830.	12909.	12045.	11238.	10486.	9783.	9128.
6916.	6455.	6022.	5619.	5243.	4892.	4564.
3459.	3227.	3011.	2810.	2621.	2446.	2282.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL
CFS	152151.	110094.	65101.	29292.	
CMS	4308.	3116.	1843.	829.	
INCHES		5.48	12.95	17.49	
MM		139.11	329.03	444.13	
AC-FT		54592.	129126.	174299.	
THOUS CU M		67338.	159275.	214995.	

HYDROGRAPH AT STA 1 FOR PLAN 1, RTIO 6

349.	326.	6870.	2982.	3666.	18188.	190189.
17295.	16137.	15056.	14048.	13107.	12229.	11410.
8647.	8066.	7526.	7024.	6554.	6115.	5705.
4324.	4034.	3764.	3512.	3277.	3057.	2853.

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL
CFS	190189.	137617.	81376.	36615.	
CMS	5386.	3897.	2304.	1037.	
INCHES		6.85	16.19	2	
MM					

YDROGRAPH AT STA 1 FOR PLAN 1, RTIO 3

9.	1491.	1833.	9094.	95094.	42523.	15955.	9268.
9.	7024.	6554.	6115.	5705.	5323.	4967.	4634.
4.	3512.	3277.	3057.	2853.	2662.	2483.	2317.
2.	1756.	1638.	1529.	1426.	1331.	1242.	1159.

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
95094.	68809.	40686.	18307.	291353.
2693.	1948.	1152.	518.	8250.
	3.42	8.10	10.93	14.49
	86.94	205.64	277.58	366.13
	34120.	80704.	108937.	144472.
	42066.	99547.	134372.	178204.

YDROGRAPH AT STA 1 FOR PLAN 1, RTIO 4

3.	1789.	2200.	10913.	114113.	51027.	19145.	11122.
3.	8429.	7864.	7338.	6846.	6386.	5960.	5561.
3.	4214.	3932.	3669.	3423.	3194.	2980.	2780.
3.	2107.	1966.	1834.	1712.	1597.	1490.	1390.

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
14113.	82570.	48826.	21969.	349623.
3231.	2338.	1383.	622.	9900.
	4.11	9.72	13.11	17.39
	104.33	246.77	333.10	441.76
	40944.	96845.	130724.	173367.
	50504.	119456.	161246.	213645.

YDROGRAPH AT STA 1 FOR PLAN 1, RTIO 5

3.	2386.	2933.	14550.	152151.	68037.	25527.	14829.
3.	11236.	10486.	9783.	9128.	8517.	7947.	7414.
3.	5619.	5243.	4892.	4564.	4259.	3973.	3707.
3.	2810.	2621.	2446.	2282.	2129.	1987.	1854.

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
52151.	110094.	65101.	29292.	466164.
4306.	3116.	1843.	829.	13200.
	5.48	12.95	17.49	23.19
	139.11	329.03	444.13	589.01
	54592.	129126.	174299.	231156.
	67338.	159275.	214995.	285126.

DROGRAPH AT STA 1 FOR PLAN 1, RTIO 6

	2982.	3666.	18188.	190189.	85046.	31909.	18536.
	14048.	13107.	12229.	11410.	10646.	9933.	9268.
	7024.	6554.	6115.	5705.	5323.	4967.	4634.
	3512.	3277.	3057.	2853.	2662.	2483.	2317.

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
0189.	137617.	81376.	36615.	582700.
5386.	3897.	2304.	1037.	16500.
	6.85	16.19	2	28.99

AC-FT	66240.	161406.	217874.	
INCHES CU M	64173.	199091.	266744.	

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

HYDROGRAPH ROUTING

ROUTING OF INFLOW HYDROGRAPH

ISTAG	ICOMP	IECON	ITAPE	JPLT	JPRT
2	61	0	0	0	0

ROUTING DATA

QLOSS	CLOSS	AVG	IRES	ISAME	IOPT	IPMP
0.0	0.000	0.00	1	1	0	0

NSTPS	NSTDL	LAG	AMSKK	X	ISK
1	0	0	0.000	0.000	0.000

STORAGE	385000.00	390000.00	397000.00	405000.00	412000.00	420000.0
OUTFLOW	950.00	3570.00	4112.00	4539.00	4968.00	5459.0

STATION 2, PLAN 1, RTIO 1

OUTFLOW

937.	923.	920.	921.	917.	937.	1239.
1660.	1808.	1907.	1922.	1934.	1943.	1949.
1952.	1946.	1942.	1934.	1920.	1910.	1905.
1851.	1835.	1819.	1803.	1786.	1768.	1750.

STOR

304507.	304139.	304039.	304071.	303944.	304568.	314362.
334706.	335434.	336039.	336533.	336924.	337219.	337426.
337506.	337306.	337177.	336938.	336654.	336330.	335969.
334220.	333720.	333201.	332664.	332111.	331544.	330964.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	IO
CFS	1955.	1955.	1953.	1942.	
CMS	55.	55.	55.	55.	
INCHES		0.10	0.39	1.16	
MM		2.47	9.87	29.45	
AC-FT		969.	3874.	11559.	
INCUS CU M		1190.	4779.	14257.	

MAXIMUM STORAGE = 337004.

STATION 2, PLAN 1, R

McFARLAND-JOHNSON ENGINEERS, INC.

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      66240.   101400.   217874.        288945.
      64173.   199091.   206744.        356406.



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

         HYDROGRAPH ROUTING

OW HYDROGRAPH

 ICOMP    IECON    ITAPE     JPLT     JPRT    INAME  ISTAGE    IAUTO
    01        0        0        0        0        1        0        0
            ROUTING DATA
  AVG     IRES    ISAME     IOPT     IPMP             LSTR
 0.00        1        1        0        0                0

 NSTOL      LAG    AMSKK        X      ISK    STORA  ISPRAT
     0        0    0.000    0.000    0.000 305000.        0

0.00   405000.00   412000.00   420000.00   430000.00   441000.00

2.00     4539.00     4966.00     5459.00     6152.00     6629.00

  STATION       2, PLAN 1, RTIO 1

              OUTFLOW
   921.      917.      937.     1239.     1637.     1790.     1839.
  1922.     1934.     1943.     1949.     1953.     1955.     1954.
  1934.     1926.     1916.     1905.     1892.     1879.     1865.
  1603.     1786.     1768.     1750.     1732.     1714.     1695.

               STOR
304071.   303944.   304568.   314362.   327297.   332247.   333848.
336533.   336924.   337219.   337426.   337552.   337604.   337586.
336938.   336654.   336330.   335969.   335574.   335149.   334697.
332064.   332111.   331544.   330964.   330374.   329775.   329168.

              STAGE
   0.0       0.0       0.0       0.0       0.0       0.0       0.0
   0.0       0.0       0.0       0.0       0.0       0.0       0.0
   0.0       0.0       0.0       0.0       0.0       0.0       0.0
   0.0       0.0       0.0       0.0       0.0       0.0       0.0

AK     6-HOUR    24-HOUR    72-HOUR   TOTAL VOLUME
5.       1955.     1953.     1942.        66929.
5.         55.       55.       55.         1695.
          0.10      0.39      1.16         3.33
          2.47      9.87     29.45        84.57
           969.     3874.    11559.       33158.
          1196.     4779.    14257.       40930.


 MAXIMUM STORAGE =   337604.


 STATION       2, PLAN 1, R
```

OUTFLOW

936.	926.	933.	949.	955.	1007.	1024.
2926.	2965.	3035.	3077.	3113.	3142.	3166.
3215.	3217.	3215.	3211.	3203.	3193.	3180.
3110.	3066.	3003.	3040.	3015.	2986.	2961.

STOR

304601.	304206.	304459.	304969.	305157.	306838.	326851.
369183.	371032.	372033.	374004.	375162.	376124.	376905.
378475.	378536.	378490.	378340.	378096.	377767.	377359.
375071.	374363.	373611.	372819.	371991.	371131.	370242.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL
CFS	3217.	3216.	3214.	3196.	
CMS	91.	91.	91.	90.	
INCHES		0.10	0.64	1.91	
MM		4.06	16.24	48.46	
AC-FT		1595.	6375.	19017.	
THOUS CU M		1967.	7864.	23457.	

MAXIMUM STORAGE = 378536.

STATION 2, PLAN 1, RTIO 3

OUTFLOW

938.	927.	940.	903.	974.	1042.	1816.
3460.	3534.	3641.	3778.	3891.	3984.	4056.
4164.	4164.	4158.	4144.	4124.	4094.	4051.
3826.	3762.	3694.	3623.	3562.	3532.	3501.

STOR

304618.	304240.	304069.	305418.	305703.	307972.	333096.
386420.	386831.	390919.	392067.	394152.	395341.	396278.
397975.	397983.	397853.	397598.	397232.	396765.	396210.
393329.	392464.	391502.	390689.	389749.	388770.	387764.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL
CFS	4164.	4164.	4159.	4113.	
CMS	118.	118.	118.	116.	
INCHES		0.21	0.83	2.46	
MM		5.20	21.02	62.36	
AC-FT		2065.	8249.	24472.	
THOUS CU M		2547.	10175.	30166.	

OUTFLOW

3.	949.	955.	1007.	1624.	2454.	2752.	2863.
5.	3077.	3113.	3142.	3166.	3185.	3199.	3209.
5.	3211.	3203.	3193.	3180.	3166.	3149.	3130.
5.	3040.	3015.	2986.	2961.	2953.	2904.	2874.

STOR

9.	304969.	305157.	306838.	326851.	353141.	363454.	367065.
3.	374004.	375162.	376124.	376905.	377517.	377975.	378291.
0.	378340.	378096.	377767.	377359.	376879.	376334.	375730.
1.	372819.	371991.	371131.	370242.	369327.	368390.	367434.

STAGE

.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
3217.	3216.	3214.	3196.	106370.
91.	91.	91.	90.	3012.
	0.16	0.64	1.91	5.29
	4.06	16.24	48.46	134.40
	1595.	6375.	19017.	52745.
	1967.	7864.	23457.	65061.

MAXIMUM STORAGE = 378538.

STATION 2, PLAN 1, RTIO 3

OUTFLOW

0.	903.	974.	1042.	1816.	2832.	3233.	3375.
1.	3776.	3891.	3984.	4056.	4111.	4138.	4156.
8.	4144.	4124.	4094.	4051.	4002.	3948.	3890.
8.	3823.	3562.	3532.	3501.	3469.	3436.	3402.

STOR

0.	305418.	305703.	307972.	333096.	366063.	379058.	383673.
0.	392687.	394152.	395341.	396278.	396987.	397493.	397817.
0.	397598.	397232.	396765.	396210.	395581.	394885.	394132.
0.	390689.	389749.	388776.	387764.	386720.	385646.	384546.

STAGE

0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
4164.	4164.	4159.	4113.	130166.
116.	118.	118.	116.	3686.
	0.21	0.83	2.46	6.48
	5.20	21.02	62.36	164.47
	2065.	8249.	24472.	64545.
	2547.	10175.	30186.	79615.

MAXIMUM STORAGE = 397983.

STATION 2, PLAN 1, RTIO 4

OUTFLOW

939.	926.	946.	977.	992.	1077.	2008.
4445.	4602.	4751.	4875.	4978.	5054.	5113.
5207.	5201.	5180.	5163.	5131.	5093.	5048.
4800.	4734.	4659.	4582.	4508.	4440.	4371.

STOR

304630.	304273.	304879.	305808.	306309.	309107.	339340.
403247.	405977.	408299.	410242.	411838.	413120.	414116.
415715.	415621.	415360.	414965.	414432.	413782.	413026.
409169.	408047.	406878.	405609.	404425.	403149.	401844.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTA
CFS	5207.	5204.	5199.	5143.	
CMS	147.	147.	147.	146.	
INCHES		0.26	1.03	3.07	
MM		6.58	26.28	77.97	
AC-FT		2581.	10312.	30600.	
THOUS CU M		3183.	12719.	37745.	

MAXIMUM STORAGE = 415715.

STATION 2, PLAN 1, RTIO 5

OUTFLOW

940.	930.	959.	1004.	1030.	1147.	2393.
6426.	6573.	6699.	6804.	6891.	6960.	7013.
7091.	7082.	7064.	7038.	7004.	6962.	6915.
6673.	6602.	6528.	6451.	6372.	6290.	6206.

STOR

304670.	304340.	305299.	306766.	307581.	311376.	351829.
436311.	439719.	442615.	445040.	447031.	448622.	449847.
451048.	451450.	451036.	450426.	449638.	448688.	447592.
442014.	440381.	438672.	436697.	435064.	433182.	431255.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOT
CFS	7091.	7090.	7083.	7026.	
CMS	201.	201.	201.	199.	
INCHES		0.35	1.41		
MM					

MAXIMUM STORAGE = 397963.

STATION 2, PLAN 1, RTIO 4

OUTFLOW

6.	977.	992.	1077.	2008.	3230.	3927.	4276.
1.	4875.	4978.	5054.	5113.	5156.	5185.	5202.
6.	5163.	5131.	5093.	5048.	4998.	4939.	4875.
9.	4562.	4506.	4440.	4371.	4300.	4228.	4155.

STOR

7.	305668.	306369.	309137.	339340.	378985.	394609.	400079.
9.	410242.	411838.	413120.	414116.	414852.	415349.	415630.
6.	414965.	414432.	413782.	413026.	412176.	411242.	410237.
3.	405669.	404425.	403149.	401844.	400514.	399166.	397802.

STAGE

0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
5207.	5204.	5199.	5143.	161739.
147.	147.	147.	146.	4580.
	0.26	1.03	3.07	8.05
	6.56	26.28	77.97	204.36
	2581.	10312.	30600.	80201.
	3183.	12719.	37745.	98926.

MAXIMUM STORAGE = 415715.

STATION 2, PLAN 1, RTIO 5

OUTFLOW

.	1004.	1030.	1147.	2393.	4523.	5829.	6254.
.	6804.	6891.	6960.	7013.	7051.	7076.	7089.
.	7036.	7004.	6962.	6915.	6862.	6803.	6740.
.	6451.	6372.	6290.	6206.	6103.	5967.	5830.

STOR

.	306760.	307581.	311376.	351829.	404706.	425337.	432347.
.	445040.	447031.	448622.	449847.	450735.	451314.	451610.
.	450426.	449638.	448688.	447592.	446303.	445016.	443563.
.	436697.	435064.	433182.	431255.	429297.	427325.	425352.

STAGE

	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	0.0	0.0	0.0	0.0	0.0	0.0	0.0

PEAK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
7091.	7090.	7083.	7026.	222768.
201.	201.	201.	199.	6309.
	0.35	1.41		11.08
				281.50

AC-FT	3516.	14049.	41616.
THOUS CU M	4337.	17329.	51581.

MAXIMUM STORAGE = 451648.

STATION 2, PLAN 1, RTIO 6

OUTFLOW

941.	932.	972.	1032.	1067.	1216.	2776.
7679.	8061.	8226.	8302.	8473.	8502.	8632.
8740.	8732.	8712.	8682.	8642.	8592.	8535.
8242.	8150.	8065.	7971.	7873.	7773.	7671.

STOR

304704.	304407.	305719.	307065.	308793.	313645.	364318.
409637.	474172.	477366.	480968.	483527.	485585.	487183.
489690.	489502.	489044.	488339.	487411.	486279.	484963.
478196.	470203.	474115.	471943.	469698.	467389.	465025.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

	PEAK	6-HOUR	24-HOUR	72-HOUR	TOTA
CFS	8740.	8738.	8731.	8501.	
CMS	247.	247.	247.	245.	
INCHES		0.43	1.74	5.17	
MM		11.04	44.13	131.32	
AC-FT		4333.	17317.	51538.	
THOUS CU M		5345.	21360.	63571.	

MAXIMUM STORAGE = 489690.

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3516.	14049.	41616.	110473.
4337.	17329.	51581.	136207.

MAXIMUM STORAGE = 451648.

STATION 2, PLAN 1, RTIO 6

OUTFLOW

1032.	1067.	1216.	2776.	6167.	7280.	7661.
8302.	8473.	8502.	8632.	8683.	8717.	8736.
8662.	8642.	8592.	8535.	8471.	8400.	8324.
7971.	7873.	7773.	7671.	7566.	7460.	7352.

STOR

307065.	308793.	313645.	364316.	430340.	456004.	464806.
480968.	483527.	485585.	487183.	488359.	489147.	489581.
488339.	487411.	486279.	484963.	483481.	481849.	480083.
471943.	469698.	467389.	465025.	462614.	460164.	457682.

STAGE

0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0	0.0	0.0

AK	6-HOUR	24-HOUR	72-HOUR	TOTAL VOLUME
0.	8738.	8731.	8501.	274200.
7.	247.	247.	245.	7764.
	0.43	1.74	5.17	13.64
	11.04	44.13	131.32	346.46
	4333.	17317.	51538.	135967.
	5345.	21360.	63571.	167713.

MAXIMUM STORAGE = 489690.

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PEAK FLOW AND STORAGE (END OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RAT
FLOWS IN CUBIC FEET PER SECOND (CUBIC METERS PE
AREA IN SQUARE MILES (SQUARE KILOMETERS)

OPERATION	STATION	AREA	PLAN	RATIO 1	RATIO 2	RATIOS APPLIED TO F RATIO 3	RATIO 4
				0.20	0.40	0.50	0.60
HYDROGRAPH AT	1	187.00	1	38036.	76076.	95094.	114113.
	(	484.33)	(	1077.11)(	2154.22)(	2692.77)(	3231.33)(
ROUTED TO	2	187.00	1	1955.	3217.	4164.	5207.
	(	484.33)	(	55.36)(	91.09)(	117.92)(	147.44)(

ND OF PERIOD) SUMMARY FOR MULTIPLE PLAN-RATIO ECONOMIC COMPUTATIONS
S IN CUBIC FEET PER SECOND (CUBIC METERS PER SECOND)
AREA IN SQUARE MILES (SQUARE KILOMETERS)

AN	RATIO 1	RATIO 2	RATIO 3	RATIO 4	RATIO 5	RATIO 6
	0.20	0.40	0.50	0.60	0.80	1.00
1	38036.	76076.	95094.	114113.	152151.	190189.
	(1077.11)(	2154.22)(	2692.77)(	3231.33)(	4308.44)(	5385.55)(
1	1955.	3217.	4164.	5207.	7091.	8740.
	(55.36)(	91.09)(	117.92)(	147.44)(	200.79)(	247.50)(

APPENDIX D

STRUCTURAL STABILITY ANALYSIS

A. Erdman, Anthony, Associates Analysis

B. Review by Thomsen Associates

C. Thomsen Associates Analysis

MEMORANDUM

From: K. Ketchek

To: E. C. Tonias

Date: 15 September 1980

Re: DESIGN WARNER DAM FOUNDATION

The foundation design against water percolation through the subsoil of the dam was performed on the basis of Art. 59 (pages 502-505, 1954) "Soil Mechanics in Engineering Practice" by K. Terzaghi and R. Peck. The weighted creep ratio C_w is expressed by the equation (59.2)

$$C_w = \frac{1/3\ B + t}{h}$$

Four borings (B-1, B-2, No. 3, and No. 4) reveal, that below the elevation of the dam bottom (1291.5) the soil consists predominately of dense sand and gravel, with traces of clay and silt.

Table 27 of the above mentioned reference recommends a value of C_w, for coarse sand and fine gravel, between 4 and 5 (used vg 4.5). In our calculation 1/3 B + t was estimated as 68.0' (page 3 of Structural Calculations). The maximum head h = 14', which yields

$$\text{design } C_w = \frac{68}{14} = 4.85 \qquad 4.50$$

The actual value of (1/3) B + t, after some modifications during the final design, equals 49/3 + 58'-6" = 74.83' which gives

$$\text{actual } C_w = \frac{74.83}{14} \approx 5.35$$

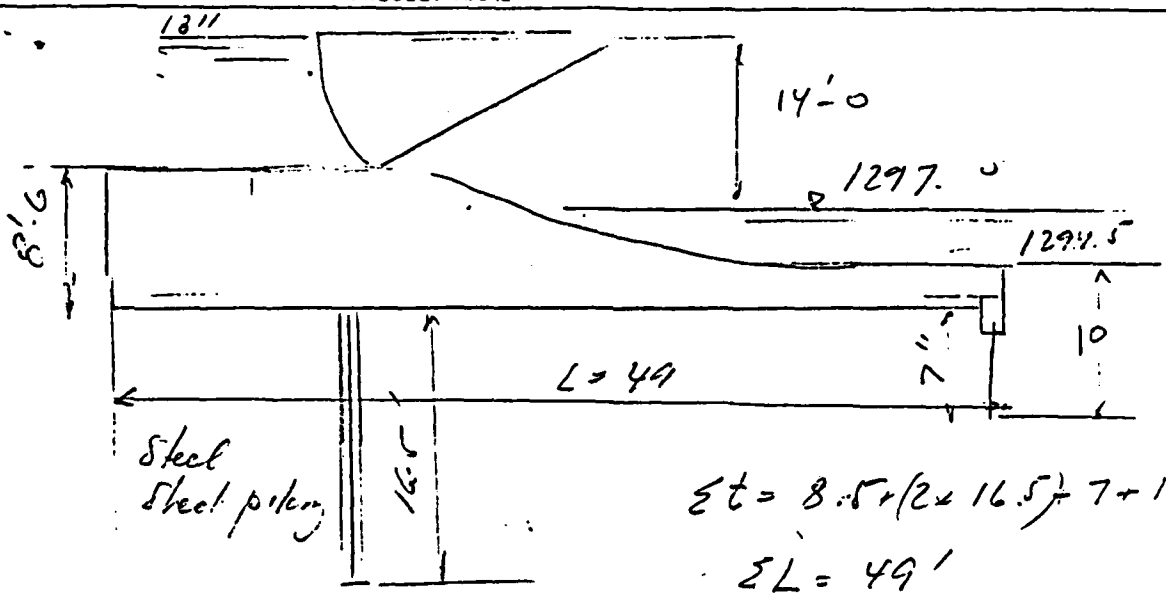
Additionally, the old dam foundation was taken into consideration. The old dam, located just 60' upstream with the same head, had one timber sheet piling with unspecified length and composed of three 2" planks. Due to the denseness of the soils, length could not be more than 15'. Accordingly, B/3 + t of the old dam equals $\frac{43}{3}$ + (2X15 + 7.5 + 4 + 2) = 58'; $C_w = \frac{58}{14}$ = 4.14. This dam was built in 1913 and was in operation until 1978 (65 years).

On the basis of this comparison, using a more conservative approach, we increased our design of C_w to 4.85 which, with the further final design modification, became 5.35. We consider this value safe against damage due to water percolation.

BY DATE **ERDMAN, ANTHONY, ASSOCIATES** SHEET OF

CKD DATE SUBJECT SUB-SHEET NO.

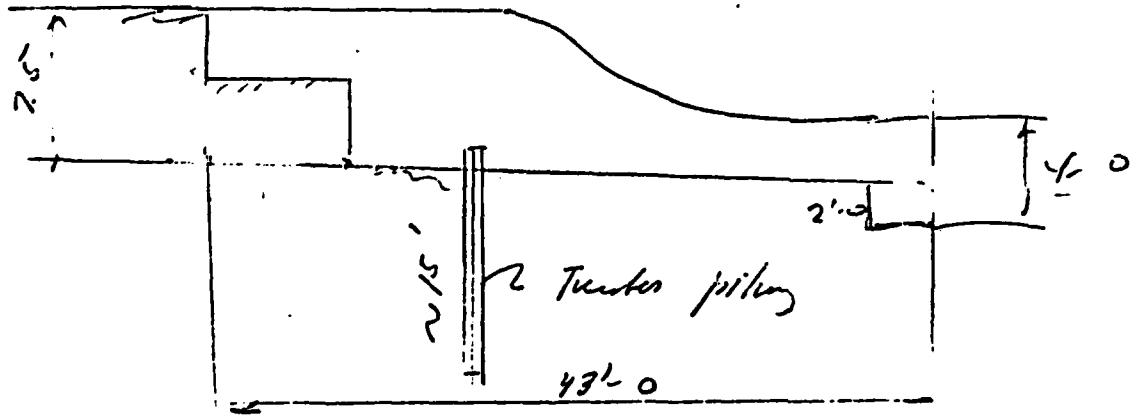
OWNER PROJECT NAME



$\Sigma t = 8.5 + (2 \times 16.5) + 7 + 10 = 49.0$

$\Sigma L = 49'$

New dam



$\Sigma t = 7.50 + (2 \times 15) + 3 + 4.0$

$= 49.5$

$\Sigma H = 43$

Old dam

D-2

DATE _____ ERDMAN, ANTHONY, ASSOCIATES SHEET 10 OF _____

DATE SUBJECT SUB-SHEET NO.

PROJECT NAME WARNER DAM

STABILITY

BY [illegible] DATE [illegible] ERDMAN, ANTHONY, ASSOCIATES SHEET [illegible] OF

CKD MD DATE 2/15/74 SUBJECT SUB-SHEET NO. 1

OWNER PROJECT NAME Warner dam

Calculation of structure stability and foundation pressure

General concepts

The Warner Dam stability calculation are performed with the following criteria:

1. The dam is founded on sand gravel strates. *)

   Weighted creep value (See Terzaghi & Peck, Soil Mechanics in Engineering Practice, Second Edit. page 517) is selected as for mixture of sand in gravel (see table 63.1) as average between 4 & 5. $\frac{4+5}{2} = 4.50$ (min)

2. Safety sliding factor ≥ 1.50

3) Friction factor for sliding 0.40.

4) Safety factor for overturning ≥ 2.0

5) Max foundation pressure.

For all computations the portion of the dam with the attached pier, is considered as unite. The abutments are not included in the computation. However in detailing on final plans, the abutment are designed integrated with the dam, which considerably increases dam stability.

The assumed dam dimensions - see sub-sheet 2

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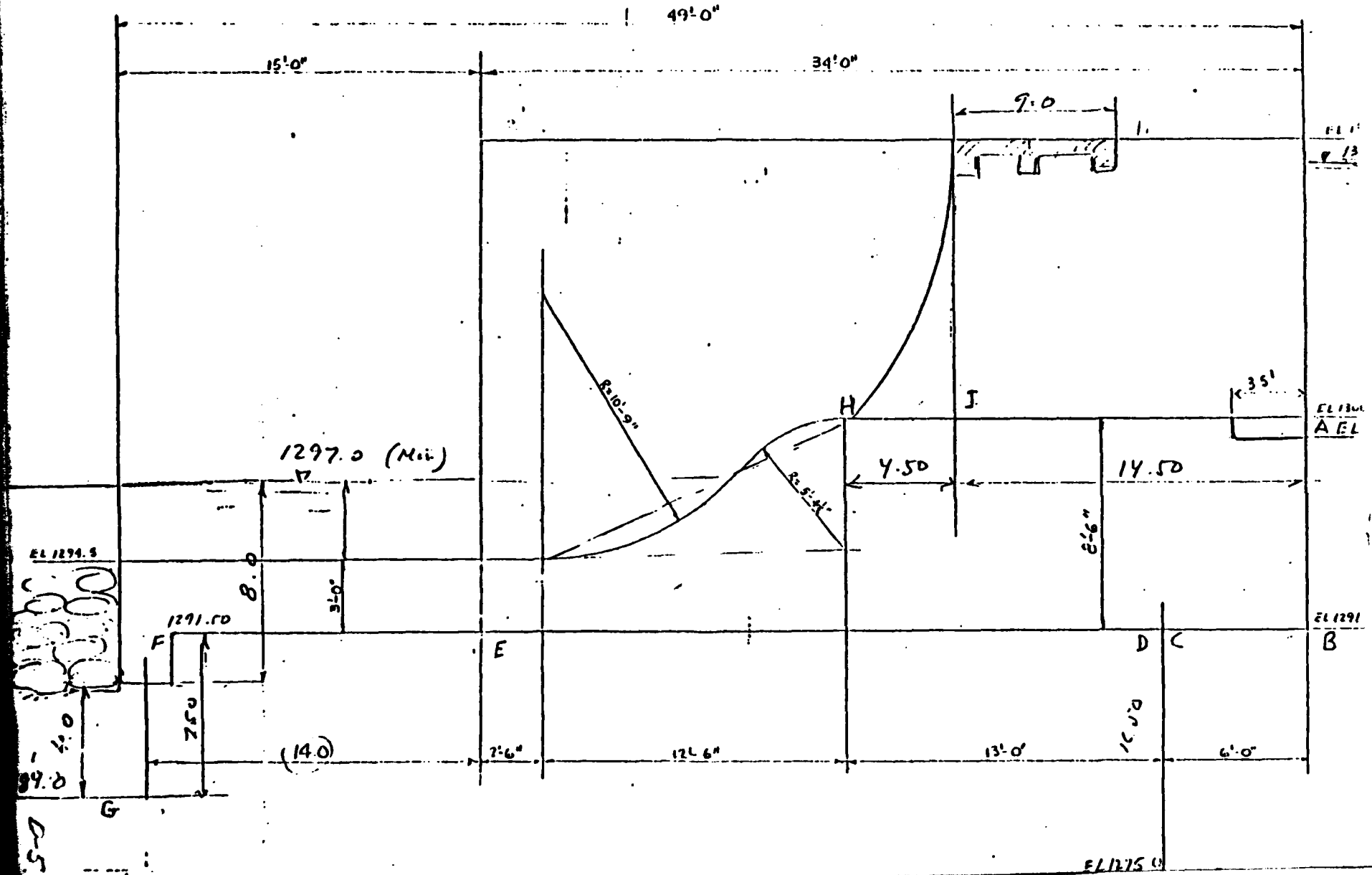
*) See borings data in separate cover

Warner dam profile

Note: They are minor changes in the final plans



CKD. ICK DATE 2/.17 5 SUBJECT SUB-SHEET NO. 3

OWNER PROJECT NAME WARNER DAM 1.6115-00

WEIGHTED LENGTH OF PATH

$7.5 + 6.0/3 + 16.5 + 16.5 + 42.0/3 + 7.5 + 40 = 68.0'$ ✓

HEAD

$1311.0 - 1297.0 = 14.0'$ ✓

GRADIENT

$14.0/68.0 = .206$ ✓

Creep factor $68/14 = 4.85 > 4.5$ (selected min)

PRESSURE

@ A $12.0(.0624) = .749$ KSF ✓

@ B $(19.5 - 7.5(.206))(.0624) = 1.120$ KSF ✓

@ C $(17.955 - 6.0/3(.206))(.0624) = 1.095$ KSF ✓

@ D $(17.543 - 33.0(.206))(.0624) = .670$ KSF ✓

@ E $(10.745 - 28.0/3(.206))(.0624) = .551$ KSF ✓

@ F $(8.822 - 14.0/3(.206))(.0624) = .491$ KSF ✓

@ G $(7.861 + 3.5 - 11.5(.206))(.0624) = .561$ KSF

VERTICAL

FORCE	CALCULATION	V	ARM ↗ ABOUT TOE OF DAM		M
WATER					
B-C	$(1.120+1.095)/2\,(6.0)\,(31.667)$	−210.40 ✓	31.0 ✓		−6523.237
D-E	$(.670+.551)/2\,(28.0)\,(31.667)$	−541.30 ✓	14.454 ✓		−7824.181
H-I	$\sim(1/3)(11.0)(4.5)(.0624)(31.667)$	32.604 ✓	18.0 ✓		586.872 ✓
I-A	$\sim[(11.0)(14.5)+(3.5)(1.0)](.0624)(31.667)$	322.50 ✓	26.86 ✓		8651.602
CONCRETE					
PIER	$(20.0)(34.0)(5.0)(.15)$	510.0 ✓	17.0	+	8670.00
DAM (1)	$[(19.0)(8.5)-(3.5)(1.0)](26.667)(.15)$	632.0 ✓	23.30		14725.80
DAM (2)	$(15.0)(3.0)(26.667)(.15)$	180.00 ✓	7.5 ✓		1350.00
DAM (3)	$(5.5)(12.5)(1/2)(26.667)(.15)$	137.50 ✓	10.833		1489.559
		1062.50	Total		+35474.0
			(Rounded)		−14347.0
					21127.0

HORIZONTAL

WATER				
A-B	$(.749+1.120)/2\,(7.5)\,(31.667)$	221.946 ✓	3.437	−762.828
DOWNSTREAM	$8.^2/2\,(.0624)(31.667)$	−63.233 ✓	disregarded for overturning	
GATE+STEP	$(12.0)^2/2\,(.0624)(31.667)$	142.273 ✓	11.5	−1636.140
		300.985	D-6	−2398.968

For stability on sliding entire structure including portion EF and weight of concrete bridge is considered

Additional forces which shall be accounted:

Vertical:

Water pressure on EF $-\frac{0.551 + 0.491}{2} \times 14 \times 31.667 = -231^{K}$

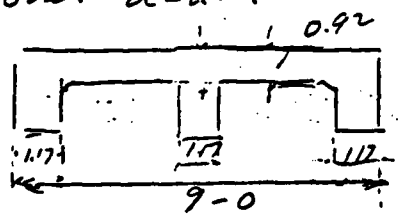
Weight of concrete $15 \times 3.0 \times 0.15 \times 31.667 = +214$

Weight of water above a) $17.50 \times 2.50 \times 0.0624 \times 26.667 = 73^{K}$

b) $\frac{2}{3} \times 2.50 \times 6.0 \times 0.0624 \times 26.667 = 78^{K}$

$+ 72^{K}$

Weight of concrete bridge over dam:



$A = (9.0 \times 0.92) + (1.50 \times 0.50) = 9.0\ Ft^2$

$W = 9.0 \times 0.15 \times 31.667 = 43.0^{K}$

Addit. vert. forces $V = 72 + 43 = 115^{K}$

Total vertical forces on one section (31.667') of dam are. $\Sigma V = 1062.50 + 115 = 1177.50^{K}$

$\Sigma H = 301^{K}$

for friction factor between concrete and footing use 0.40

Sliding S.F. $= \frac{1177.50 \times 0.40}{301} = 1.56 > 1.50$ OK.

Actual sliding S.F. is higher, because of additional weight of abutments, gates and hoisting equipment.

Additional stability moment due to weight of concrete bridge

$$M_v = 43 \times 24 = 1032^{1K}$$

Total: Overturning moment = 14347 + 2400 = 16747 ^{1K}

Stabilizing moment = 35474 + 1032 = 36506 ^{1K}

Overturning S.F = $\frac{36506}{16747} = 2.18 > 2.0$ (OK)

Total ΣM on the portion of dam to the pt. E

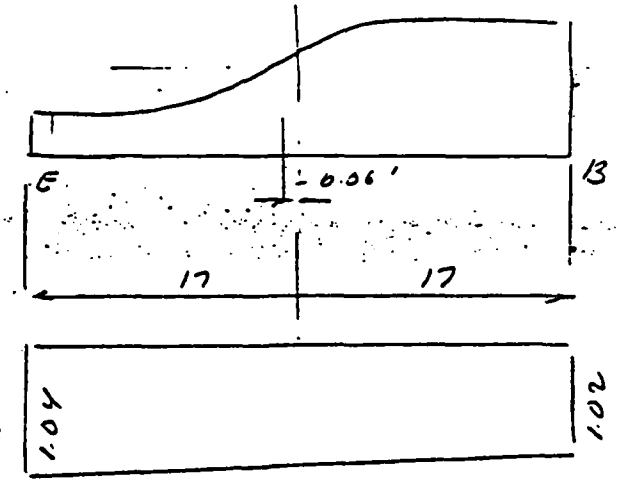
$\Sigma M = 21127 - 2400 = 18727^{1K}$

$\Sigma P = 1062.50 + 43 = 1105.5^{K}$

Eccentricity

$\eta = \frac{18727}{1105.5} = 16.94$

$e = 17 - 16.94 = 0.06$



Foundation pressure:

$$\sigma = \frac{1105.50}{34 \times 31.667}\left(1 \pm \frac{6 \times 0.06}{34}\right)$$

$= 1.027 (1 \pm 0.01)$

$P_{max} = 1.04$ KSF

$P_{min} = 1.02$ KSF

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CKD KK DATE 2/2/78 SUBJECT SUB-SHEET NO. 6
OWNER PROJECT NAME WARNER DAM 1.6115

Considering water pressure, total pressure on the concrete at different points is:

Pt. B = 1.02 + 1.12 = 2.14 KSF

Pt C = 1.025 + 1.095 = 2.12 KSF

Pt D = 1.025 + 0.67 = 1.70 KSF

Pt E = 1.04 + 0.551 ≅ 1.60 KSF

Check for seismic stability

ADDITIONAL INERTIA FORCE DUE TO SEISMIC LOADING EQ = CD

WEIGHT OF DAM (CONCRETE) = (510.0 + 632.0 + 180.0 + 137.5) = 1459.5
concrete bridge = 43.0
1502.5 K

C = .04 (FROM AASHTO)

EQ = 1502.5 (.04) = 60.0 K

ΣF_H = 301 + 60 = 360 K

SLIDING S.F. = $\frac{1177.50 \times 0.40}{302 \cdot 360}$ = 1.30 > 1.0

addit. Horiz. force
FOR OVERTURNING

		ARM	MOMENT	
PIER	510.0	10.0	5100.0	✓
DAM (1)	632.0	4.25	2686.0	✓
DAM (2)	180.002	1.5	270.0	✓
DAM (3)	137.502	4.833	664.5	
			8720.5	(.04) = 348.82

OVERTURNING S.F. $\frac{1.36506}{(16.743 + 3.49)}$ = 2.13

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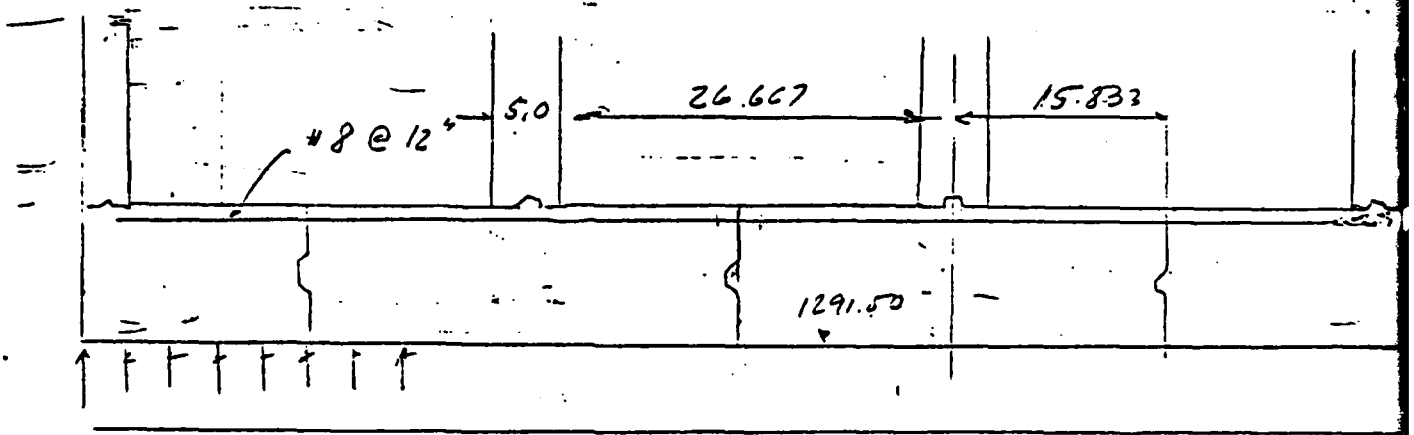
OWNER PROJECT NAME

Stresses in the body of the dam



Upstream end

Max. footing pressure	-2.14	*) No water on the top
Weight of concrete 85#.0.15	+1.27	
Total	-0.87	
Considering 11' of water	-0.87	Water at elev. 1311.
11 x 0.0624	+0.69	
	+0.18 K/L.F.	

Downstream end

Max footing pressure	-1.60	
Weight of concrete 3.0 · 0.15	+0.45	— Min. thickness
3' of water 0.0624·3 =	+0.18	at the end
	0.97 K/SF.	

* This condition can exist by abrupt drop of upstream water level. (No water on the slab, pressure at bottom is not relieved)

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CKD HP DATE 2/15/78 SUBJECT SUB-SHEET NO. 8
OWNER PROJECT NAME

Upstream section, Thickness of concrete 8.5'

Max possible moments: (per 1 Lf)

Over supports considering partial restriction over piers - use $M_{max} = wl^2/10$

$$M = \frac{0.87 \times 26.67^2}{10} = 62 \text{ ik/Lf.}$$

Check stresses for plain concrete

Slab thickness = 8.5'

Stresses: $f = \frac{M}{S}$; $S = \frac{bh^2}{6} = \frac{1.0 \times 8.5^2}{6} = 12 \text{ ft}^3$

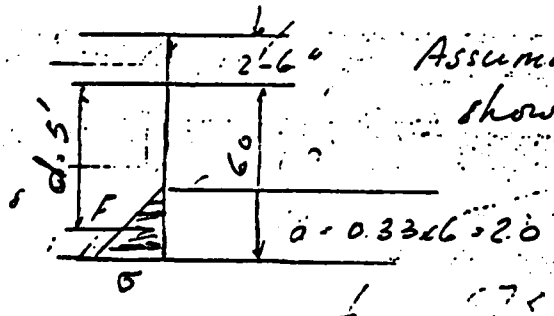
$$f = \frac{62}{12} = 5.2 \text{ k/sf} \times 6.95 = 36 \text{ psi (ok)}$$

Tensil stress is very low, no reinforcement required.

At mid-span joint

At the joint no tensil stresses can be taken by concrete. Calculate required reinforcement.

Using the same moment $M = 62$ ik/ft



Assuming stress distribution as shown on the sketch

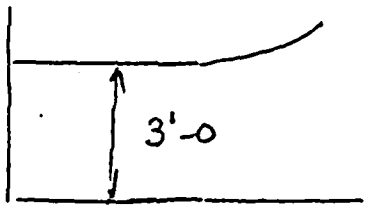
$$A_s = \frac{62}{5 \times 24} = 0.52 \text{ in}^2$$

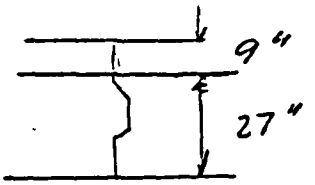
Use #8 @ 12" $A_s = 0.78$

Concrete stress $F = 62/5 = 12.4$ k/ft $\frac{\sigma \times 24 \times 12}{2} = 12400$

$$\sigma = \frac{12400 \times 2}{[illegible]} = 86 \text{ psi}$$

Downstream end.





Max down pressure 0.97 K/S.F

$M_{max} = \frac{0.97 \times (26.67)^2}{12} =$ (consider as slab restricted at both ends)

$= 57.50$ 'K

$M_u = \frac{1.50}{0.90} * 57.50 = 95.80$ 'K

Slab Thickness = 36" $d = 36" - 9 = 27$

$F = \frac{bd^2}{12000} = \frac{12 \times 27^2}{12000} = 0.729$

$K_u = \frac{M}{F} = \frac{95.80}{0.729} = 131.41$

Use $p = 0.0025$ (From tables for $f_c = 3000$
use $p = 0.0030$ $f_s = 60$ ksi $K_u = 156$)

$A_s = 0.003 \times 27 \times 12 = 0.972$

Use #9 @ 12" $A_s = 1.0$ in^2

AD-A090 937 NEW YORK STATE DEPT OF ENVIRONMENTAL CONSERVATION ALBANY F/G 13/13
NATIONAL DAM SAFETY PROGRAM. WARNER DAM, CHAUTAUGUA LAKE OUTLET--ETC(U)
SEP 80 B L THOMSEN, G L WOOD DACW51-79-C-0001

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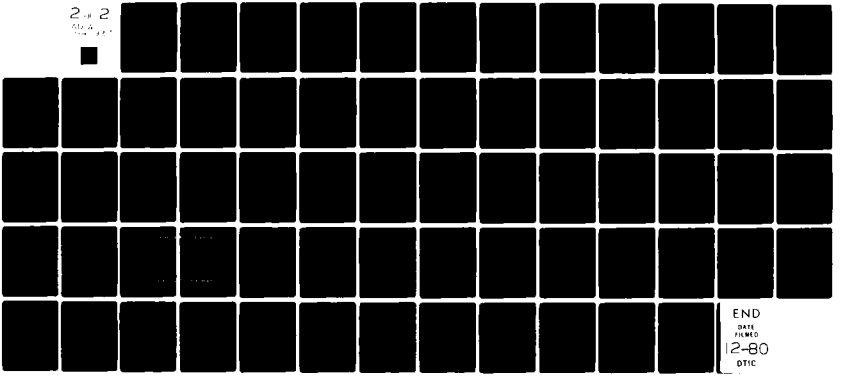
2 # 2



END
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12-80
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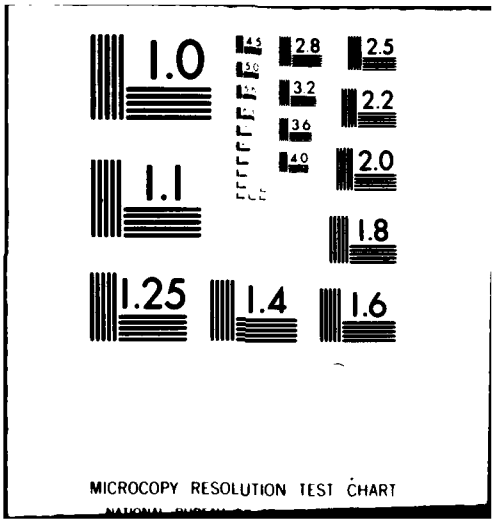
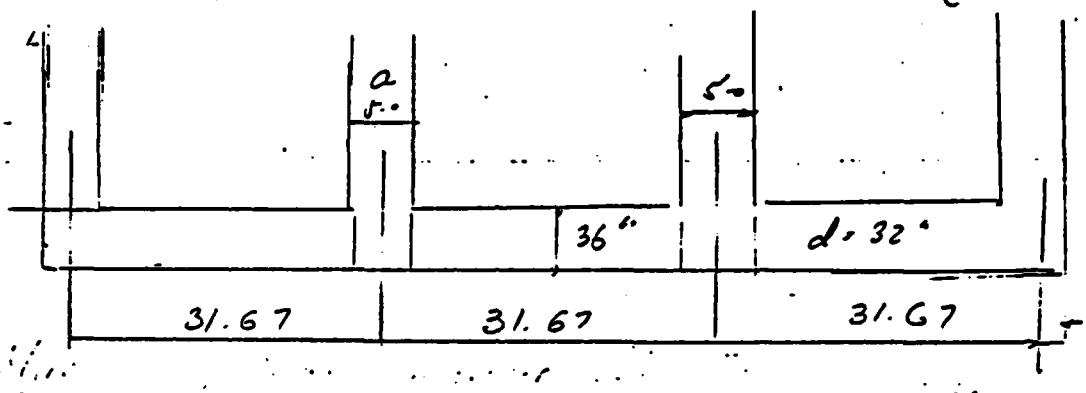

1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0
MICROCOPY RESOLUTION TEST CHART

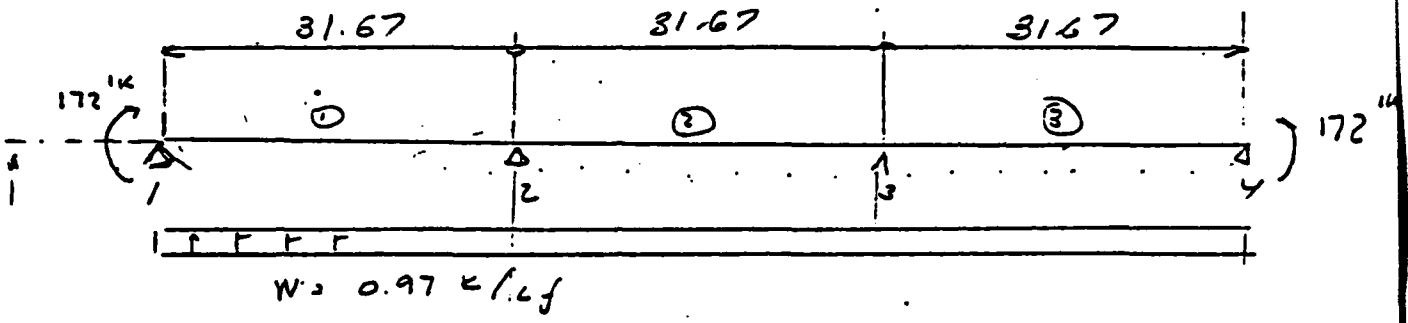
Reinforcement of bottom slab (Continued)



Max bottom pressure reduced to the weight of slab + 3'-0 of water ... (see page 4 of stability calculation) = 0.97 K/S.F

This pressure shall be considered as conservative, for the case of abrupt drop of water downstream usually level of water downstream is higher than 3'-0. The footing of dam is built integrated with the piers and abutments. The preliminary computations (see dam's stability) were performed without considerating of moments at the ends of footing due to horizontal forces. Using abutment design computation, we apply moments of 172 K at both ends, and by

The scheme of forces & moments applied to the footing is shown below



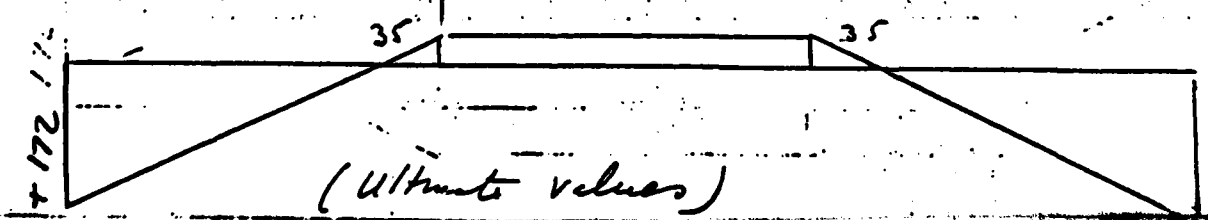
Distribution factors $K_1 = K_3 = 3$ $K_2 = 4$

D.F @ pt 1 = 0 DF(1-2) = 3/7 = 0.43
DF @ pt 4 = 0 DF(2-1) = 4/7 = 0.57

FEM due to $W = 0.97$ K/L.f $= \frac{0.97 \times 31.67^2}{12} = 81$ "K

Moment distribution for applied moment ±172 "K

+172 (1.0)	0.43	0.57	— — —	0.43	1.0 (0) -172 (2)
+172	—				-172
	+86			-86	1st C.O
	-37	-49	+49	+37	1st Dist
		+24.5	-24.5		C.O
	-10.5	-14.0	+14	+10.5	Dist
		+7	-7		C.O
	-3	-4	-4	+3	Dist
		+2	-2		C.O
	-1	-1	+1	+1	Dist
+172	+35	-35	+35	-35	-172 Total



D-14

Moment distribution for uniform load 0.97 k/ℓf

	0.43	0.57	0.87	0.43	Dist. F.
+81	-81	+81	-81	+81	-81	FGM
-81	0	0	0	0	+81	Dist
	-40.5			+46.50		C.O
	+20.25	+20.25	-20.25	+20.25		Dist
		-10.0	+10			C.O
	+5	+5	-5	-5.0		Dist
		-2.5	+2.5			C.O
	+1.0	+1.5	-1.5	-1.0		Dist
	-95	+95	+95	-95		Total

$R_1 = 0.97 \times 31.67/2 - 95/31.67 = 15.36 - 3.0 = 12.36$

$M_1 = 12.36 \times 3.167 - 0.97 \times 3.167^2/2 = 39.14 - 9.86 = 34.28$

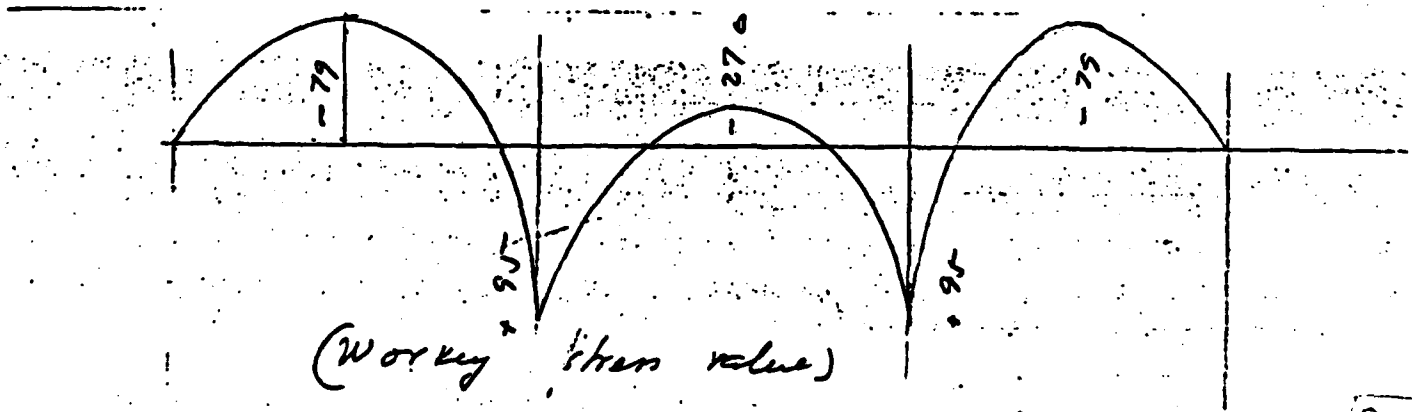
$M_2 = 2 \times 39.14 - 4.86 \times 4 = 78.28 - 19.44 = = 58.84$

$M_3 = 3 \times 39.14 - 4.86 \times 9 = 117.42 - 43.74 = = 73.68$

$M_4 = 4 \times 39.14 - 4.86 \times 16 = 156.56 - 77.76 = 78.8$

$M_5 = 5 \times 39.14 - 4.86 \times 25 = 195.7 - 121.50 \rightarrow 74.2$

$M_{10} = 10 \times 39.14 - 4.86 \times 100 = 391.40 - 486 = \sim -95.00$



(Working stress values)

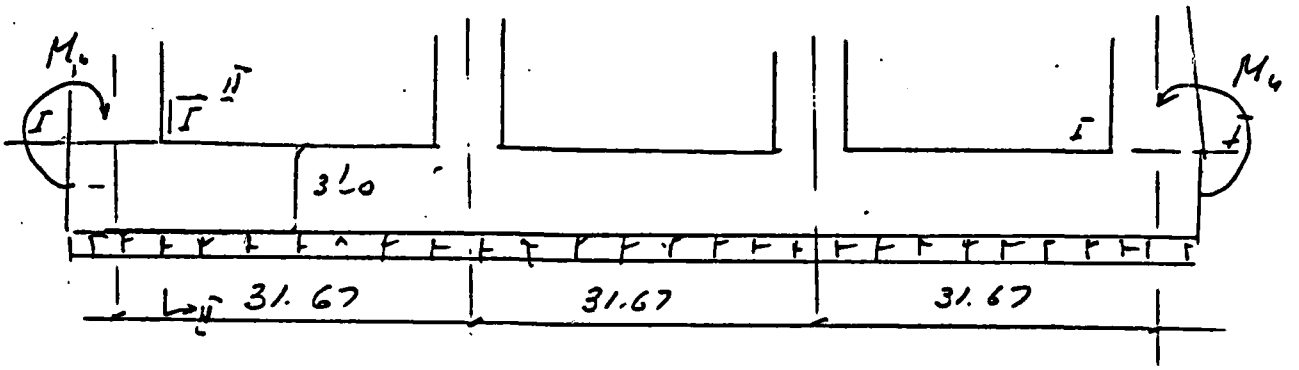
Mid of cent typ $M = \frac{0.97 \times 31.67^2}{8} - 95 = 26.60$

(These moments shall be multiplied by the factor 1.5, (for M_c)

D-15

BY KK DATE 6/23/78 ERDMAN, ANTHONY, ASSOCIATES SHEET 19 OF
CKD DATE SUBJECT SUB-SHEET NO. 4
OWNER PROJECT NAME

From the vertical forces acting on the footing and from abutments design horizontal pressure produces at sect I-I, bending moment $M_u = 172$ '^k, use



Consider the following Ult. Moments to be considered.

1) Max positive moments at sect. II-II — $M_1 = 172$ '^k

2) Max negative moment at span $1.50 \times 79 \cong 119$ '^k

3) Max positive moment under piers $-1.50 \times 95 = 143$ '^k

4) Max negative moment - central span $35 + 1.50 \times 27 = 75.5$ '^k

Reinforcement: (Use ACI spec. publ. #17 — Table 1.1 & 5

Sect. II-II. $d = 32''$ $b = 12''$ $F = bd^2/12000 = 1.02$

$f_s = 60000$ psi $f_c = 3000$ psi

$K_u = \frac{M_u}{F} = \frac{172}{1.02} = 169$ $p = 0.0030 + \frac{13}{25} \times 0.0005 = 0.00326$

$A_s = 0.00326 \times 12 \times 32 = 1.25$ in^2

Used #7 & #8 @ 12" ckt.

(Both). $A_s = 0.79 + 0.60 = 1.39$ in^2

Sect. 2. $M = -119$. (all other dimension the same as above)

$K_u = \frac{119}{1.02} = 117$ use $p = 0.0023$

$A_s = 0.0023 \times 12 \times 32 = 0.88\ in^2$

Used #8@12 $A_s = 0.79$.

Comparison with the computation on the page 9 it can be seen, that negative moment 119 ^{ik} is too conservative, disregarding deduction due to thickness of support. Considering deduction recommended by Portl. Cement Inst.

$M = 119 - \frac{V C_c}{6}$; where $V = 0.5 \times 0.97 \times 31.67 = 15.35$

$M = 119 - \frac{15.35 \times 5}{6} = 106^{ik}$ $K = \frac{106}{1.02} = 104^{ik}$

$A_s = 0.002 \times 12 \times 32 = 0.77$ (used 0.79) OK.

Sect 3 $M_u = 143$. $K = \frac{143}{1.02} = 140$

use $0.0030 \times 12 \times 32 = 1.15$ used 2#8 $A_s = 1.58$

Sec 4 Positive moment considering deduction

$M_u = \frac{75^{ik}}{2} - 15.35 \times 5/6 = 63^{ik}$

$K = 63/1.02 = 62$ use $K = 80$ $p = 0.0015$

$A_s = 0.0015 \times 12 \times 32 = 0.58$

Used #8@12 = 0.79 in^2

C-12

THOMSEN ASSOCIATES
CONSULTING GEOTECHNICAL ENGINEERS & GEOLOGISTS

Review of Structural Stability Analysis
Performed by Erdman, Anthony, Associates

The case analyzed by the designer is a high water condition with the upstream pool at Elevation 1311.0 and the tailwater at Elevation 1297.0. The gates are assumed closed and the sheet piling is assumed to be 100 percent effective in preventing underseepage and reducing hydrostatic uplift pressure.

The method used in evaluating underseepage control is based on empirical rules discussed in Terzaghi & Peck, Soil Mechanics in Engineering Practice, Article 63, pp. 615-618, 2nd Edition.

The fundamental assumption that the sheet piles are 100 percent effective results in an increased cross-sectional length of the dam thereby decreasing the hydraulic gradient and uplift pressure along the base of the dam.

From a design standpoint the above fundamental assumption is a reasonable approach. The following errors were noted in the stability analysis:

A. Sheet 10, Subsheet 4.

The weight of the concrete (214 kips) used in the additional vertical force computed for sliding stability is already included in the calculations from Sheet 9, Subsheet 3. (See Weight of Dam(2) near bottom of Subsheet 3). The correct additional vertical forces should read -99 kips rather than +115 kips, therefore, the resulting sliding S.F. should be 1.28 not 1.56.

B. Sheet 11, Subsheet 5

The computation of the total sum of moments acting on the dam between pt. B and pt. E is incorrect. This moment included the weight of the concrete between pt. E and point G (See Weight of Dam (2) and resulting moment near bottom of Subsheet 3). The correct total sum of moments should be reduced by 1350.0 kip-ft. This correction effects the calculated eccentricity, foundation pressure and internal stress.

We further note the Check for Seismic Stability on Sheet 12, Subsheet 6 used a seismic coefficient of 0.04. The Corps of Engineer Guidelines requires a seismic coefficient of 0.1 for structures in Seismic Zone 3. In addition, no hydrodynamic force and resulting moment were included in the computations for sliding and overturning safety factors, respectively. The difference in the seismic coefficient used and that recommended as well as the omission of the hydrodynamic force results in a significantly lower sliding safety factor.

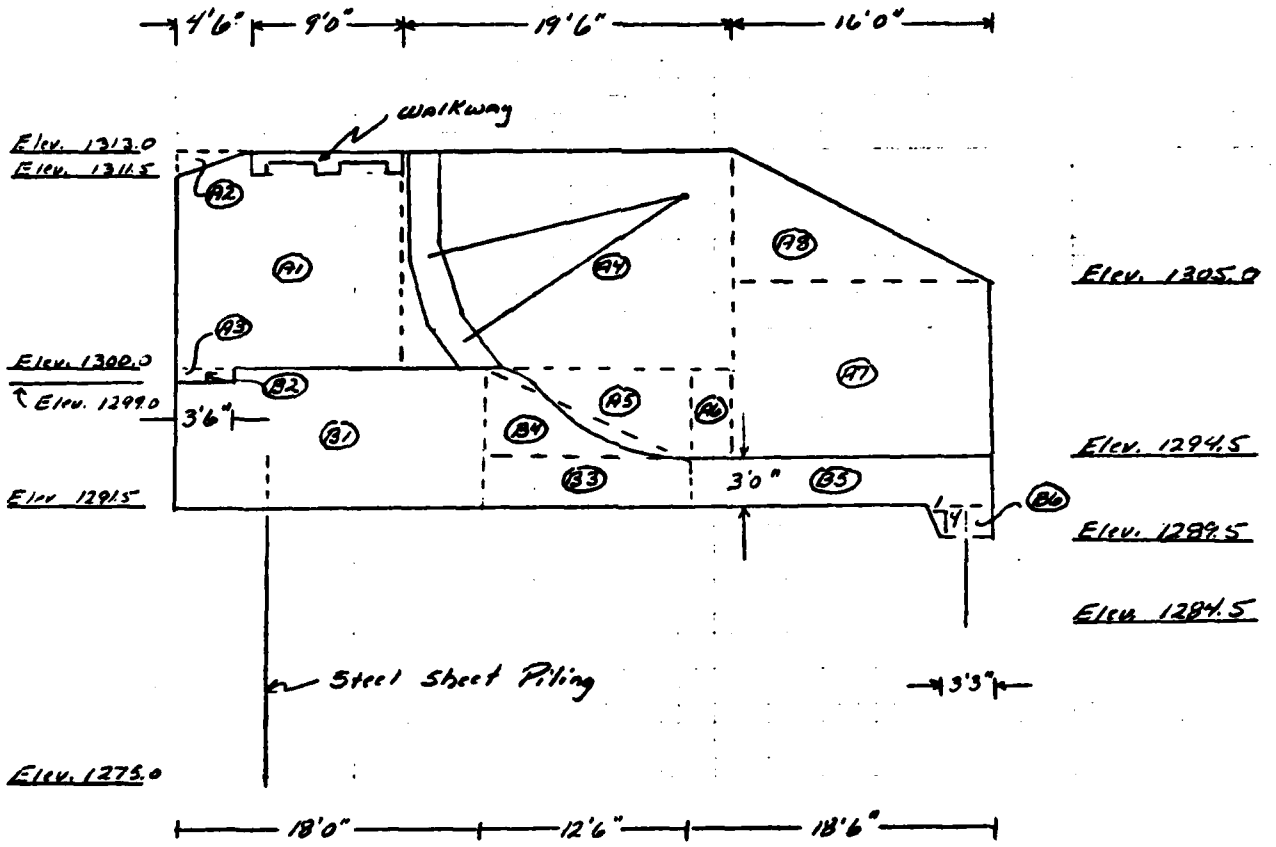
The selected friction factor for sliding (0.40) is conservative and the analysis is based on the conventional 2 dimensional appproach. The influence of additional sliding resistance between the abutments and soil was not considered as indicated on the bottom of subsheet 1.

Since the stability analysis was limited in scope, assumed completely effective sheet piling, and contained errors/omissions; a structural stability analysis was performed as part of the Phase I inspection. This following structural stability analysis is based on what we believe is the critical case--that being maximum ice loading with and without seismic loads and includes the effects of sidewall shear friction (i.e. 3 dimensional analysis).

WARNER DAM

The following sections and dimensions shown are based on the as-built drawings prepared by the designer (Erdalen, Anthony, Associates)

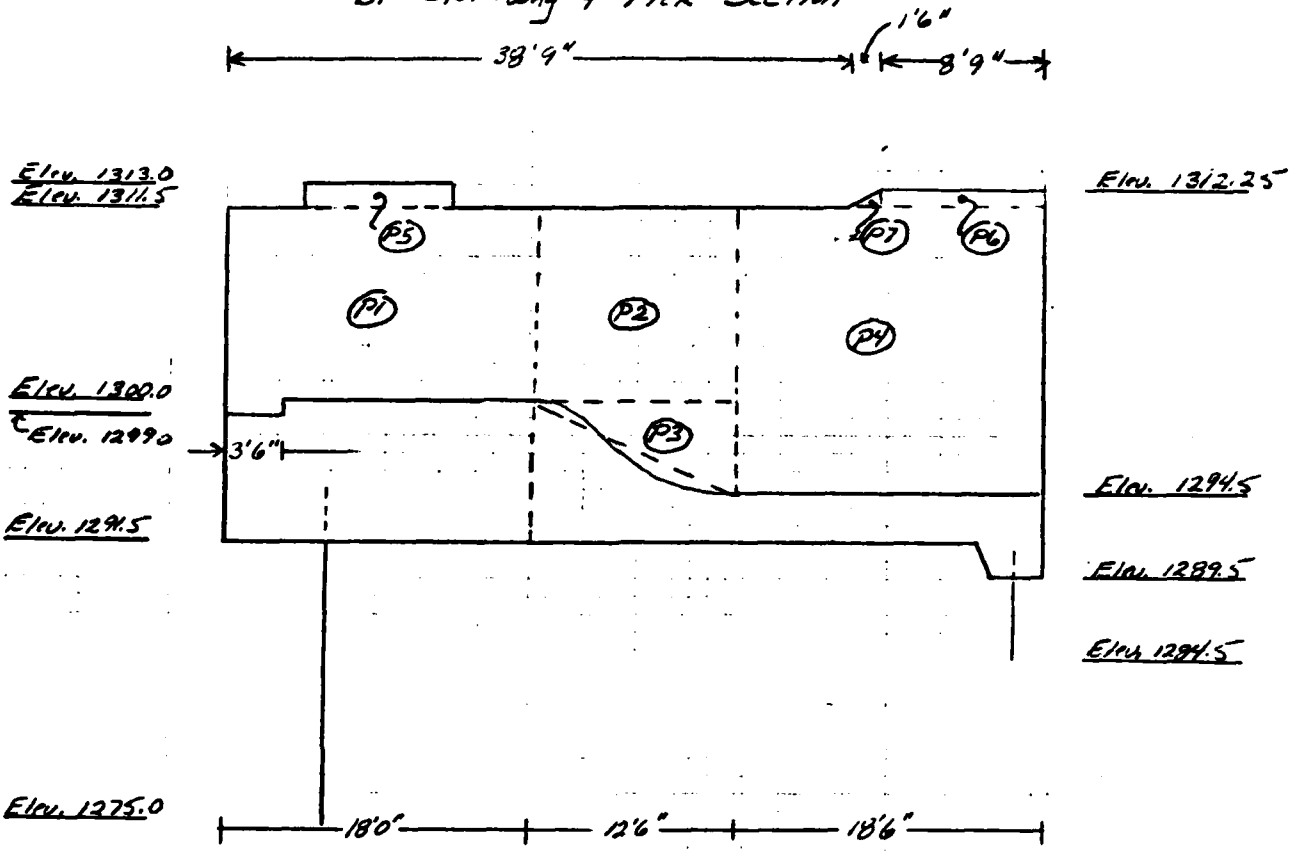
A. Spillway & Abutment Section



(A1) - (A8) Abutment SubAreas

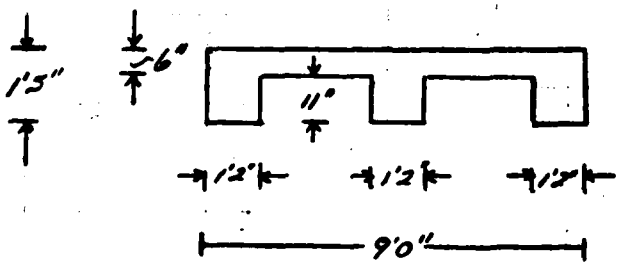
(B1) - (B6) Base SubAreas

B. Spillway & Pier Section



P1 - P7 Pier Sub Areas

C. Walkway Section



Assumptions

1) Tainter Gates Closed
2) Tailwater at Elev. 1294.5 (ie. no discharge)
3) Maximum Ice Load Applied during Winter Pool @ Elev. 1307.0
4) Steel Sheet Piling Ineffective & Full Hydrostatic Uplift Applied until proven otherwise by actual piezometric data.
5) "At Rest" Lateral Earth Pressures Applied on Upstream and Downstream Embedded Sections as well as Abutments
6) Upstream Streambed at Elev. 1300.0
7) Downstream Streambed @ Elev. 1294.5
8) Soil Behind Abutment @ Elev. equal to top of Adjacent Abutment
9) Backfill against Abutments, Upstream & Downstream Embedded Sections and Foundation Soil as the following properties:
   i) $\phi = 35°$
   ii) $c = 0$
   iii) $\gamma_{sat} = 135$ pcf
   iv) $\gamma' = 73$ pcf
10) No Silt Load
11) Seismic Loading for Zone 3 Seismicity
12) Sheet Piling Provides No Shear Resistance
    (Available Resistance is not mobilized until the dam moves thus deforming sheet piling and beginning mobilization of soil resistance. However, if dam moves, this is considered to be failure.)

1) Determine Weight of Concrete (Wc)

a) Base

Area = (18.0)(8.5) - (3.5)(1.0) + (12.5)(3.0) + ½ (12.5)(5.5) + (18.5)(3.0) + (3.25)(2.0)

= 283.38 ft²

Weight = (283.38 ft²)(.150 kcf) = 42.5 kips / lineal foot

Resultant acts 29.4 ft. from toe

b) Abutment

Area = (13.5)(13.0) - ½ (4.5)(1.5) + (3.5)(1.0) + (19.5)(13.0) + ½ (12.5)(5.5)

+ (2.5)(5.5) + (16.0)(10.5) + ½ (16.0)(8.0) = 712.62 ft²

Weight = (709.25 ft²)(5.5 ft thick)(.150 kcf) = 585 kips / abutment

Resultant acts 24.2 ft. from toe

c) Pier

Area = (18.0)(11.5) + (12.5)(11.5) + ½ (12.5)(5.5) + (18.5)(17.0) = 699.63 ft²

Weight = (699.63 ft²)(5.0 ft. thick)(.150 kcf) = 525 kips / pier

i) Add Weight where pier is adjacent to Walkway @ Elevation 1313.0

Volume = (3.0)(1.5)(9.0) = 40.5 ft³

Weight = (40.5 ft³)(.150 kcf) = 6 kips / pier

ii) Add Weight where top of pier RISES to Elev. 1312.25

Area = (8.75)(0.75) + ½ (1.5)(0.75) = 7.1 ft²

Weight = (7.1 ft²)(5.0 ft thick)(.150 k.f) = 5.3 kips / pier

TOTAL WEIGHT = 536.3 kips / pier

Resultant acts 22.3 ft from toe

d) WALKWAY

Area = (0.5')(9.0) + (3)(0.92')(1.17') = 7.71 ft²

Weight = (7.71 ft²)(3)(286.7' long)(.150 kcf) = 99.45 kips total

Resultant acts 40.0 feet from toe

TOTAL WEIGHT OF CONCRETE

$= (42.5 \text{ kips/ln ft})(101 \text{ ft}) + 2(585 \text{ kips}) + 2(536.3 \text{ kips}) + 99.5 \text{ kips}$

$= \underline{6634.6 \text{ kips}}$

Resultant Acts 27.5 ft. from toe using Moment-Weight Method

2) Determine Weight of Water (Ww) on Base Upstream of Gate

Water Level @ Elevation 1307.0

$\text{Area} = (1307 - 1299)(3.5) + (1307 - 1300)(10.0) = 98 \text{ ft}^2$

$\text{Weight} = (98 \text{ ft}^2)(26.67 \text{ ft})(3)(.0624) = \underline{489 \text{ kips}}$

Resultant acts 42.4 feet from toe

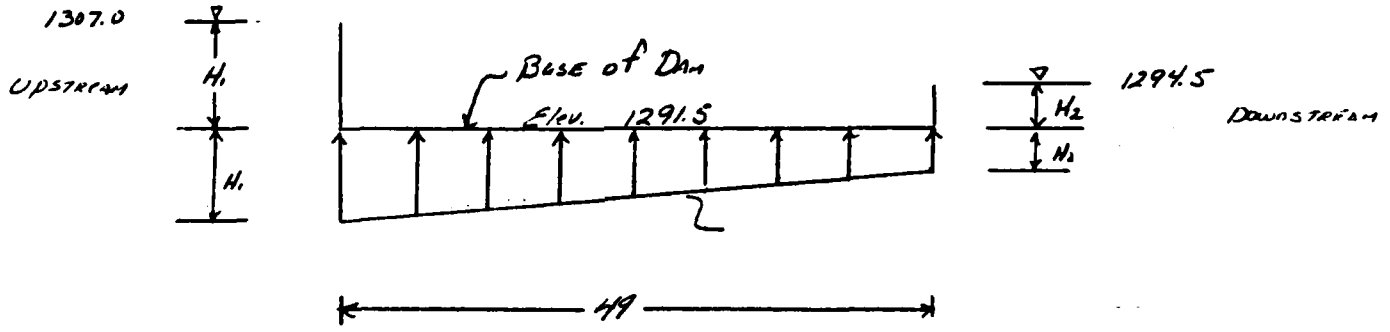
3) Determine Water Force (Pw) on Upstream Face

$$Pw = \frac{(1307 - 1291.5)^2 (.0624)}{2} = 7.5 \text{ kips/ln ft.}$$

$\text{Total Force} = (7.5 \text{ kips/ln ft.})[(26.67)(3) + (5.0)(2)] = \underline{675 \text{ kips}}$

Resultant Acts 5.17 feet above BASE

4) Determine Hydrostatic Uplift Force (Pu) on the Base



$Pu = (H_2)(\gamma_w)(49) + (H_1 - H_2)(\gamma_w)(1/2)(49)$

$= (3)(.0624)(49) + (15.5 - 3.0)(0.0624)(1/2)(49)$

$= 28.3 \text{ kips/ln ft.}$

$\text{Total } Pu = (28.3 \text{ kips/ln ft})(101 \text{ ft}) = 2857 \text{ kips}$

Resultant acts 30 ft from toe

E) Determine "At Rest" Lateral Earth Force (P_o)

a) Upstream Face

$$P_{oU} = \frac{(1300.0 - 1291.5)^2 (0.75)(0.073\ k.f)}{2} = 2.0\ k/ln\,ft.$$

Total $P_{oU} = (2.0\ k/ln\,ft)(101') = \underline{202\ kips}$

Resultant Acts 2.83 ft above base

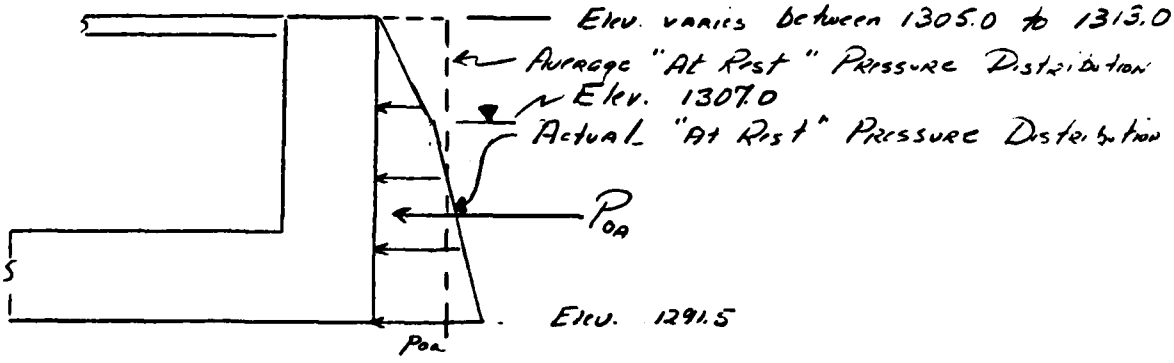
b) Downstream Face

$$P_{oD} = \frac{(1294.5 - 1291.5)^2 (0.75)(0.073\ k.f)}{2} = 0.25\ k/ln\,ft.$$

Total $P_{oD} = (0.25\ k/ln\,ft.)(101') = \underline{25\ kips}$

Resultant Acts 1.0 ft above base

c) Abutment Face



i) For Maximum Section (ie top of abutment 1313.0)

Max. Pressure $P_{oA} = (1313 - 1307)(0.75)(.135\ kcf) + (1307 - 1291.5)(0.75)(0.073)$

$= 1.46\ KSF$

Force $P_{oA} = \left[\frac{(1313 - 1307)^2 (0.75)(.135\ kcf)}{2} + (1313 - 1307)(.75)(.135)(1307 - 1291.5) + \frac{(1307 - 1291.5)^2 (.75)(.073)}{2}\right] \times 33\ ft. = 587.4\ kips$ per abutment

Resultant acts 7.93 ft above base

ii) For Section where top of Abutment varies between Elev 1305 and 1313.

$$\text{Average Elevation} = \left[\frac{1313-1305}{2}\right] + 1305 = 1309$$

MAX. PRESSURE: $P_{OA} = (1309-1307)(0.75)(.135\ kcf) + (1307-1291.5)(0.75)(.073)$

$= 1.05\ ksf$

FORCE $P_{OA} = \left[\frac{(1309-1307)^2(0.75)(.135)}{2} + (1309-1307)(0.75)(.135)(1307-1291.5) + \frac{(1307-1291.5)^2(.75)(.073)}{2}\right] \times 16\ ft = 158.7$ Kips per Abutment

Resultant Acts 7.72 feet above base

TOTAL FORCE PER ABUTMENT = 587.4 + 158.7 = 746 Kips

6) Determine Ice Load (P_I)

Assume $P_{I\,max}$ = 10 kips/ln ft.

Total P_I = (10 k/ln ft)(90 feet) = 900 kips

Resultant Acts 14'5 feet above Base

7) Determine Hydrodynamic Force (V_e) and Moment (M_e) (Zangar Method)

a) Hydrodynamic pressure @ Base

$P_e = C \lambda \gamma_w h$ = 0.071 ksf/ln.ft.

where C = 0.73

λ = 0.1

γ_w = .0624 kcf

h = 1307 - 1291.5 = 15.5'

b) V_e = 0.726 P_e h = (0.726)(0.071)(15.5) = 0.80 k/ln ft

Total V_e = (0.80 k/ln ft)(90 ft) = 72 kips

c) M_e = 0.299 P_e h^2 = (0.299)(0.071)(15.5)^2 = 5.10 k-ft/ln ft

Total M_e = (5.10 k-ft/ln ft)(90 ft) = 459 k-ft

Resultant Act 6.375 ft. above Base

8) Determine Inertial Force (P_c) due to Seismic Loading

$P_c = \lambda W_c$

= (0.1)(6612 kips) = 661.2 kips

Resultant Acts 9.5' above Base

Structural Stability

CASE 1) ICE LOADING

A. Overturning Stability

Overturning Force	Magnitude (Kips)	Moment Arm (ft)	Overturning Moment (Kip-ft)
P_w	675	5.17	3,490
P_u	2857	30.0	85,710
P_{o_u}	202	2.83	572
P_I	900	14.5	13,050

ΣM_o = 102,822 Kip-ft.

Resisting Force	Magnitude (Kips)	Moment Arm (ft.)	Resisting Moment (Kip-ft)
W_c	66346	27.5	182,451
W_w	489	42.25	20,660
P_{o_p}	25	1.0	25

ΣM_R = 203,209 Kip-ft

$$S.F. = \frac{\Sigma M_R}{\Sigma M_o} = \frac{203,209}{102,822} = 1.98$$

$$\bar{X} = \frac{\Sigma M_R - \Sigma M_o}{\Sigma F_v} = \frac{203,209 - 102,822}{66346 + 489 - 2857} = 23.5 \text{ ft.}$$

$$e = B/2 - \bar{X} = 49/2 - 23.5 = 1.0$$

$$B/6 = 49/6 = 8.17 > 1.0$$

OK, Resultant in Middle 1/3

B. Sliding Stability

$$S.F. = \frac{(W_c + W_w - P_w)\tan\phi + 2P_a(\tan\phi)}{\Sigma \text{ Driving Forces}}$$

$$S.F. = \frac{(6634.6 + 489 - 2857)(\tan 35°) + (2)(746)(\tan 35°)}{675 + 202 + 900 - 25} = 2.30$$

No Good $2.30 < 3.0$ Recommended Min.

(CASE 2) SEISMIC Loading

A. Overturning Stability

Overturning Force	Magnitude (kips)	Moment Arm (ft)	Moment k-ft
P_w	675	5.17	3,490
P_w	2857	30.0	85,710
P_{ae}	202	2.83	572
V_c	72	6.38	459
P_E	661	9.50	6,279

ΣM_o = 96,507 k-ft

$$S.F. = \frac{203,209}{96,507} = 2.11. \quad OK > \text{CASE I}$$

B. Sliding Stability

$$S.F. = \frac{4031.02}{675 + 202 + 72 + 661 - 25} = 2.54 \quad OK > 1.5$$

CASE 3, ICE AND SEISMIC

A. Overturning

$\Sigma M_o = 102,822 + 459 + 6,279 = 109,560$

$\Sigma M_R = 203,209$ k-ft

$S.F. = \frac{203,209}{109,560} = 1.86$

$\bar{x} = \frac{214,573 - 109,560}{6640 + 489 - 2857} = 21.95$ ft

$e = B/2 - \bar{x} = 49/2 - 21.95 = 2.55$

OK Resultant in Middle 1/3

B. Sliding

$S.F. = \frac{403102}{675 + 202 + 900 + 72 + 661 - 25} = 1.62$ OK > 1.5

APPENDIX E

Operation Plans

Figure No. 1
Initial Operation Plan

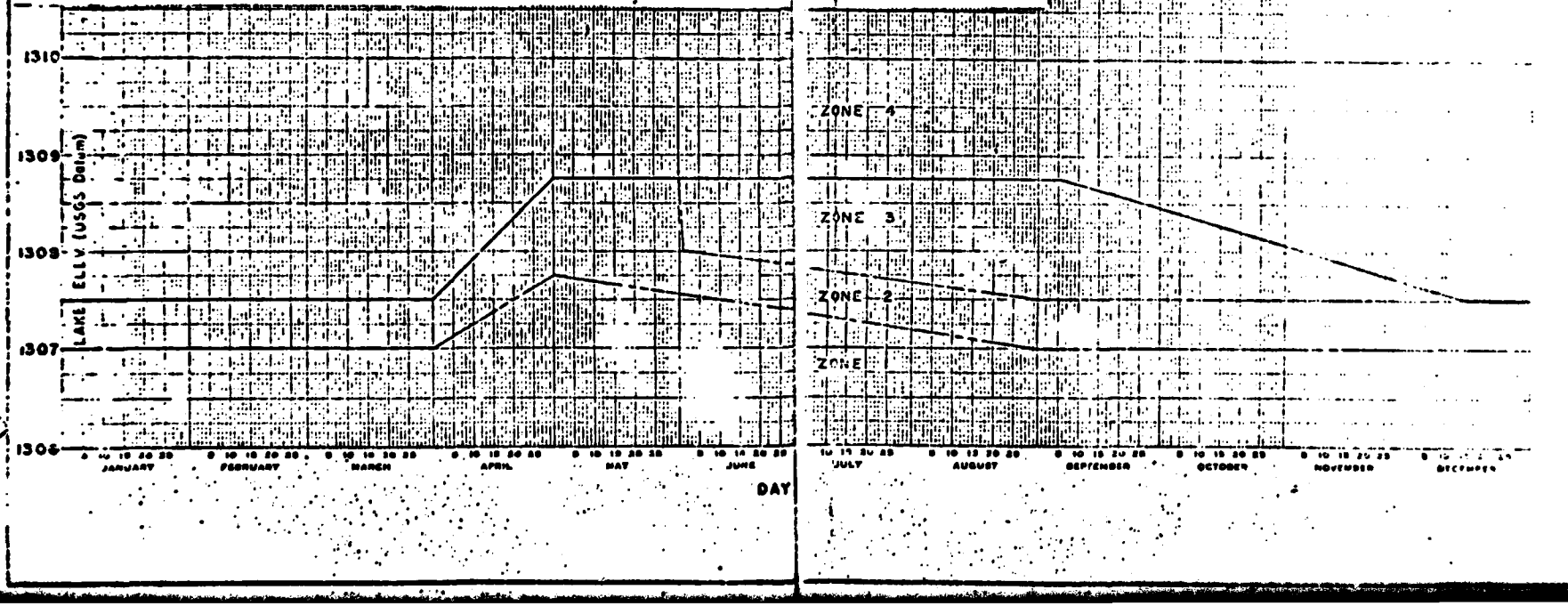
EXHIBIT B
Enclosure 1

LAKE OUTFLOW OPERATIONAL SCHEDULE

Zone	Purpose	Release Rate in cfs	Operations
4	Flood Control	60 to 1270 (1)	Releases up to 1270 cfs will be ma to evacuate lake storage as rapidly as possible. W lake stages exceed elevation 1310, releases in excess f 1270 cfs should be made.
3	Conservation	60 (2)	When the lake level is in this zon the release rate will be maintained at 60 cfs.
2	Conservation	40 (2)	When the lake level is in this zon the release rate will be maintained at 40 cfs.
1	Conservation	20 (2)	When the lake level is in this zon the release rate will be maintained at 20 cfs.

(1) Releases up to 1270 cfs are not possible until lake stage exceeds e vation 1309. Releases in this zone are affected by channel capacity of the outlet channel d will be made at the maximum capacity possible.
(2) Releases are both a minimum and maximum.

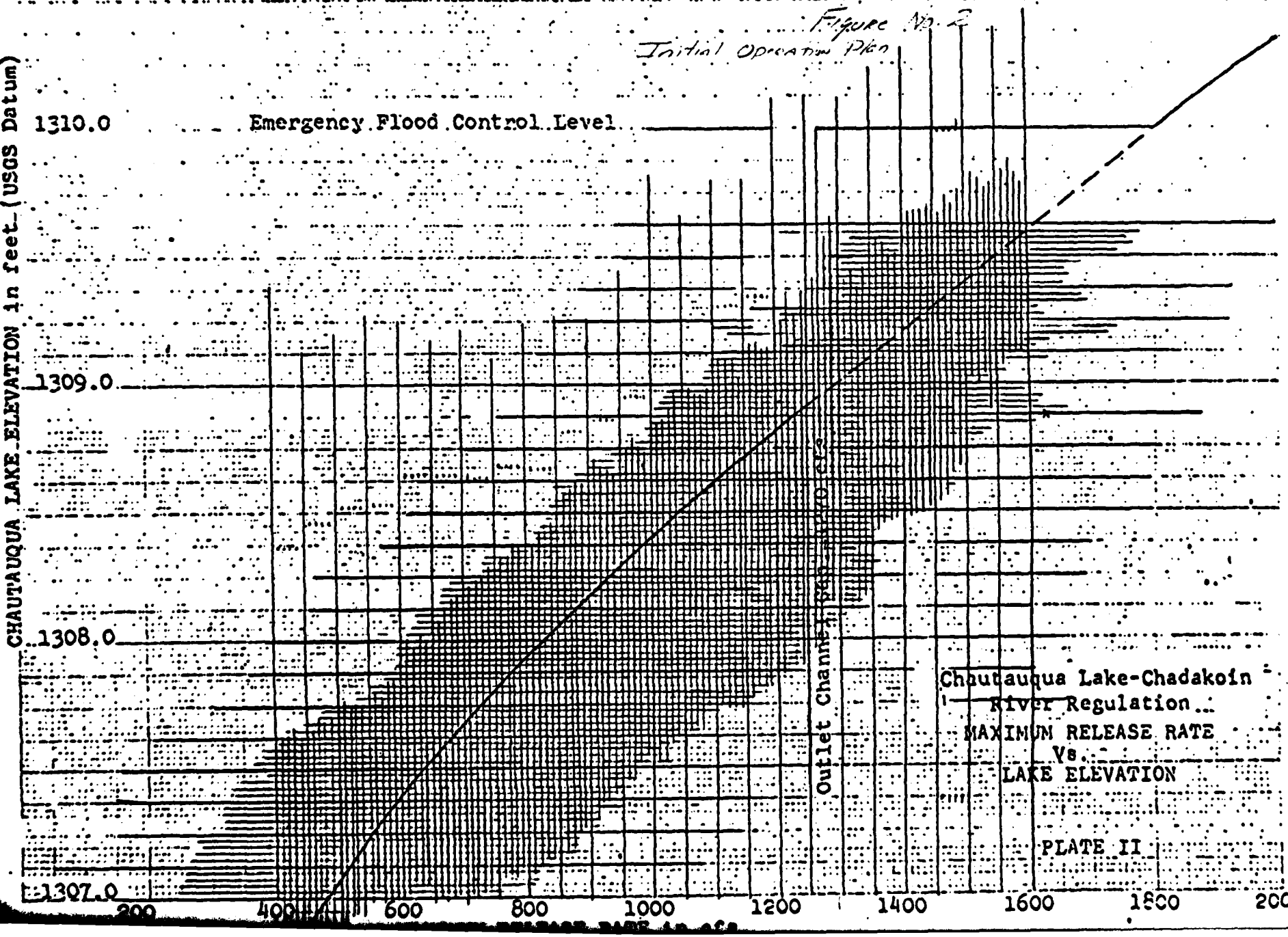
ALLEGHENY RIVER BASIN
CHAUTAUQUA LAKE SUB-BASIN
LAKE LEVEL REGULATION PLAN
20-40-60-1270 W.S.E.



New Dam

Figure No. 2

Initial Operation Plan



Chautauqua Lake-Chadakoin River Regulation

MAXIMUM RELEASE RATE Vs. LAKE ELEVATION

PLATE II

NO. 340-TIS DIETZGEN GRAPH PAPER
ONE YEAR BY DAYS — CALENDAR YEAR
366 DAYS X 250 (10 PER UNIT) DIVISIONS

EUGENE DIETZGEN CO.
MADE IN U. S. A.

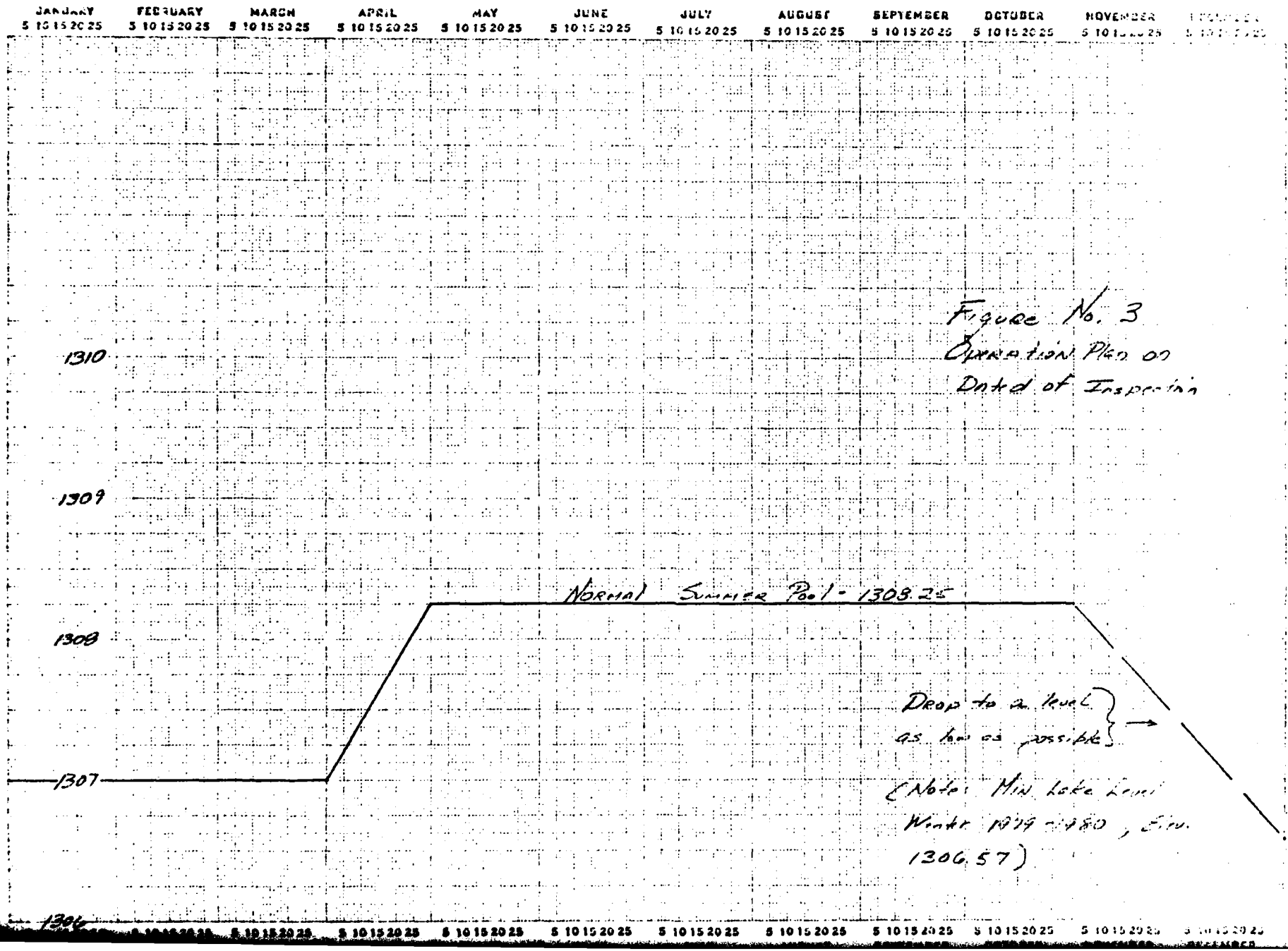


Figure No. 3
Operation Plan on Date of Inspection

APPENDIX F

Available Documents

October 30, 1969

Dam No. 164 (Warner Dam)
City of Jamestown
County of Chautauqua

Mr. Richard Burke
Blauvelt Engineering Company
79 Madison Avenue
New York, New York 10016

Dear Sir:

As per your telephone request, a search was made of our records concerning the above referenced dam.

Please be advised that the data and information in our files pertaining to the structure are antiquated, dating back to the years 1912 through 1918. However, some of the information as listed below may be of help to you, to wit:

1. The original structure was built in or about the year 1840. It was extensively repaired or reconstructed during the subsequent years since then.

2. The ownership of the dam during the year 1912 was vested in an organization known as the "Warner Dam Association" which consisted of six or seven concerns that owned water rights in Chadakoin River.

3. During the year 1913, the Superintendent of Public Works was authorized by an act of the Legislature (under Chapter 758, of the Laws of 1913) to "....cause Chadakoin River, known also as Chautauqua Lake Outlet, to be dredged or otherwise improved between such points at or near the City of Jamestown as the Superintendent may determine in such manner as will relieve the high water conditions at and near such city, subject to the approval of the Canal Board. Such work shall be accomplished pursuant to plans and specifications to be furnished by the State Engineer and Surveyor;....."

4. A memorandum in the record states that "Warner Dam was replaced by the State with a new Tainter Gate Dam, under Chapter 758 of the Laws of 1913."

5. Subsequent correspondence concerning the Warner Dam from Frank H. Williams, State Engineer, to George E. Pratt, Commissioner, Conservation Commission, dated January 14, 1918, states that "....this dam is being reconstructed and other improvements made by the State of New York under Chapter 181 of the Laws of 1917, along Chadakoin River." Upon research of the said chapter, the only mention is in the Section for the General Fund, wherein it is stated that an appropriation was made to the Department of Public Works for the dredging of the Chadakoin River.

In view of the foregoing, it is evident that the State of New York was involved in the Warner Dam.

Since our records are incomplete concerning the structure, we suggest that you communicate with Mr. Norman W. Krapf, Regional Director of Transportation, New York State Department of Transportation, 13 Wayside Court, Buffalo, New York 14226, (Telephone 1-507-842-4432), who may be able to furnish you with additional and more recent data which we do not have.

Very truly yours,

E. V. HOURIGAN
Acting Deputy Chief Engineer

By: _____

A. W. Moon
Asst. Deputy Chief Engineer

RCK/JEP/lo
cc: B. Heller
N. W. Krapf
T. P. Curran

164

Form 1W71. 6-18-12 2000 (16-15288)

(NOTICE: After filling out one of these forms as completely as possible for each dam in your district, return it at once to the Conservation Commission, Albany.)

STATE OF NEW YORK

CONSERVATION COMMISSION

ALBANY

DAM REPORT

Jamestown, N.Y., Dec. 18, 1912
(Date)

CONSERVATION COMMISSION,

DIVISION OF INLAND WATERS.

GENTLEMEN:

I have the honor to make the following report in relation to the structure known as the Warner Dam.

This dam is situated upon the Outlet of Chautauqua Lake (Give name of stream) in the ~~Town~~ City of City of Jamestown Chautauqua County, about _____ (State distance) from the Village or City of _____.

The distance down (Up or down) stream from the dam, to the Main Street Bridge (Give name of nearest important stream or of a bridge), is about 630' (State distance).

The dam is now owned by the Warner Dam Association (consisting of seven Firms and Corporations) that (Give name in full) Quaternwater Rights and was built in or about the year 70 to 80 years ago, and was extensively repaired or reconstructed during the year 1907.

As it now stands, the spillway portion of this dam is built of Timber (State whether of masonry, concrete or timber) and the other portions are built of South Abutment is of Concrete (State whether of masonry, concrete, earth or timber with or without rock fill).

As nearly as I can learn, the character of the foundation bed under the spillway portion of the dam is of Piling and under the remaining portions such foundation bed is Gravel.

The total length of this dam is 78 feet. The spillway or waste-weir portion, is about 70 9/12 feet long, and the crest of the spillway is about 6' 9/12 feet below the top of the dam.

The number, size and location of discharge pipes, waste pipes or gates which may be used for drawing off the water from behind the dam, are as follows:

State briefly, in the space below, whether, in your judgment, this dam is in good condition, or bad condition, describing particularly any leaks or cracks which you may have observed.)

The dam was recently repaired. The first plan was to rebuild, but lateness of season and high water determined the repairs. It is now proposed to rebuild the dam next season. The owners believe the dam is in safe and serviceable

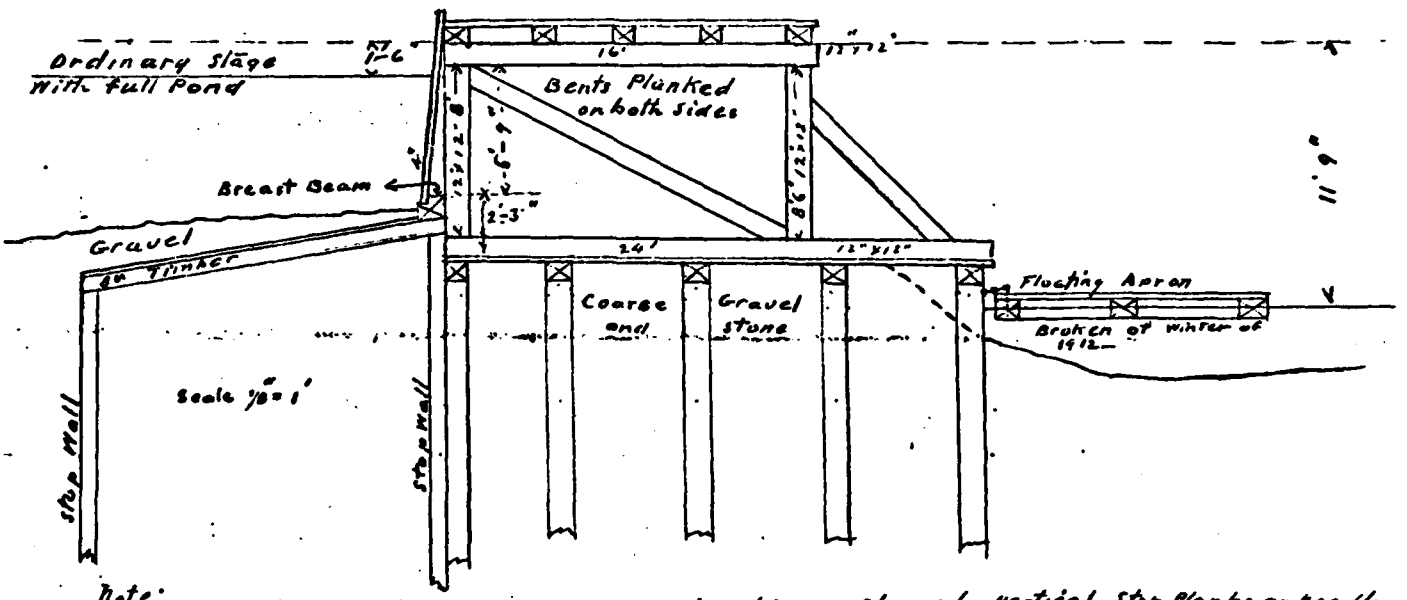
Reported by H. W. Eyde Agent
(Signature)
for Warner and Marvin

201 Main street
(Address—Street and number, P. O. Box or R. F. D. route)

Jamestown, N.Y.
(Name of place)

(SEE OTHER SIDE)

(In the space below, make one sketch showing the form and dimensions of a cross section through the spillway or waste-weir of this dam, and a second sketch showing the same information for a cross section through the other portion of the dam. Show particularly the greatest height of the dam above the stream bed, its thickness at the top, and thickness at the bottom, as nearly as you can learn.)

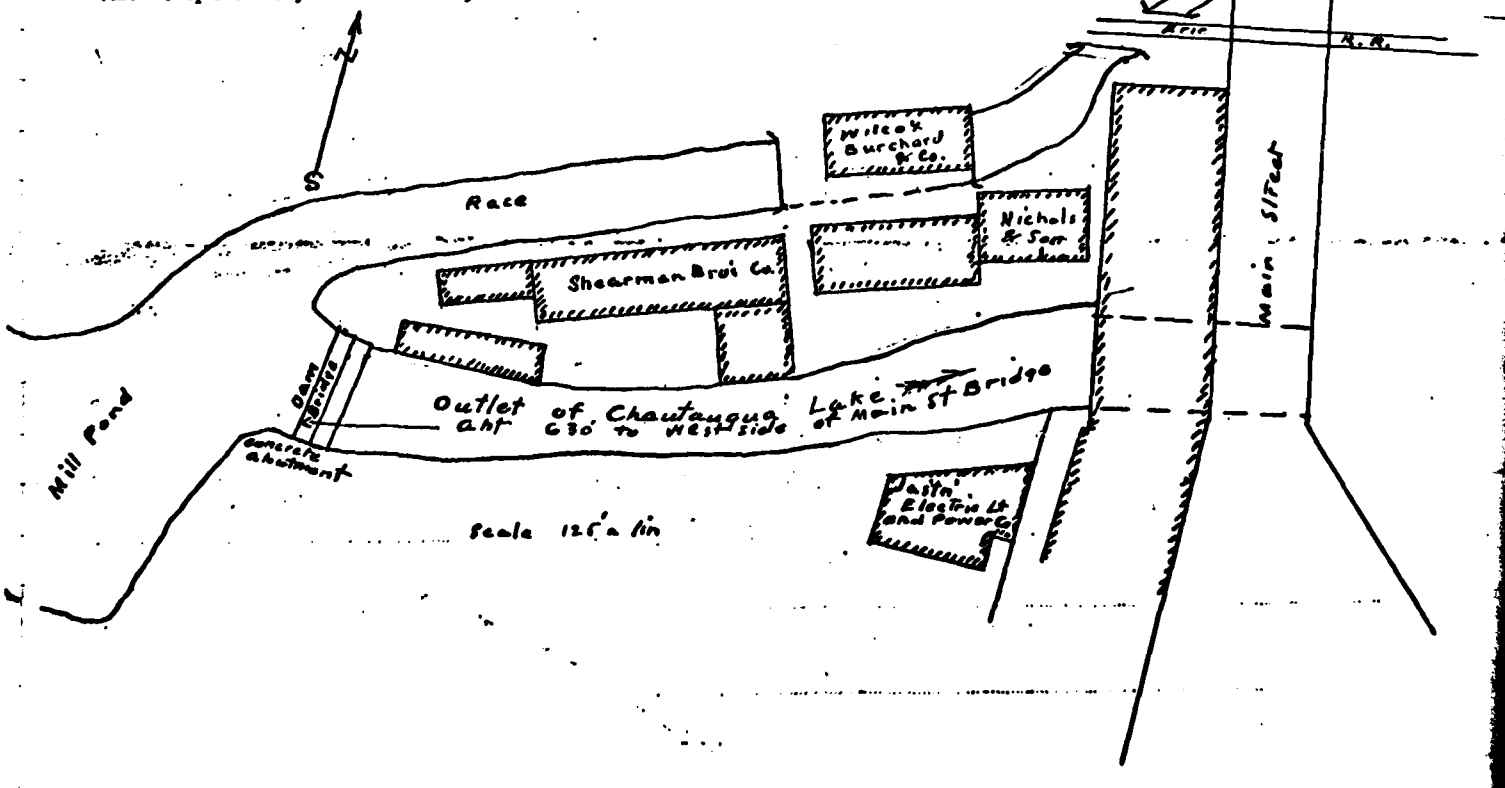


Note:

Bay 1	18'	north
" 2	11' 5"	
" 3	13' 9"	
" 4	13' 6"	
" 5	13' 9"	south
	70' 5" = Total spillway	

Bays 1 and 2 are closed by vertical Stop Planks or Needles
" 3-4+5 Have Horizontal Flash Boards all are removed when necessary beginning with No 2.

(In the space below, make a third sketch showing the general plan of the dam, and its approximate position in relation to buildings or other conspicuous objects in the vicinity.



ENVIRONMENTAL ASSESSMENT

WARNER DAM REPLACEMENT

CITY OF JAMESTOWN, NEW YORK

ERDMAN, ANTHONY, ASSOCIATES

April 1978

I. PROJECT DESCRIPTION

1. Introduction

The purpose of an environmental assessment is to identify the effects of a proposed action on the physical, economic, social, and ecological elements of the study area. In this instance, the proposed action consists of constructing a replacement for the existing Warner Dam. Through the joint efforts of local, State, and Federal officials, the need for evaluation of the existing dam was established and a report entitled "Inspection and Analysis, Warner Dam, Jamestown, N.Y." was prepared by Konski Engineers, dated December 27, 1976. Based on the findings of that report, a decision was made that replacement of the existing dam would be in the best interests of the general public. The firm of Erdman, Anthony, Associates has been retained by the New York State Department of Environmental Conservation to furnish professional services for the detailed design and construction supervision of the project. This environmental assessment is part of the overall scope of services to be provided.

This assessment has been prepared concurrently with the preliminary design efforts for the project. The extent and nature of the proposed facilities has been essentially established. In recognition of this fact, the emphasis of the assessment is directed towards specific impacts, rather than generalities. In this manner, any mitigation measures to minimize adverse effects can be realistically evaluated.

The organization of this environmental assessment is that which would normally be followed for an Environmental Impact Statement. Section I provides the background information concerning the existing facilities and proposed action. In Section II, the general setting of the project is described as it exists today. Section III defines the impacts of the proposed action. Alternatives to the proposed action are enumerated in Section IV. Irreversible or irretrievable commitments of resources required by the proposed action are covered in Section V. Supporting materials are included in the Appendix.

2. Study Area

The site of the proposed action is located in Chautauqua County, within the City of Jamestown, New York. The existing dam is located on the Chadakoin River, approximately three miles downstream from Chautauqua Lake. A general location map and site map have been included in the Appendix of this assessment as Plates I and II respectively.

3. Existing Warner Dam Facilities

The existing structure is a concrete gravity dam which was constructed in 1919. The dam consists of four separate bays, each controlled by a 20 foot span counterweighted tainter gate. Each tainter gate is raised and lowered by manual cranking of a handwheel. An open steel decking walkway allows access to the gate operators.

Due to its age and physical deterioration, the existing facility is approaching the end of its useful lifespan. The construction of the Washington Street Bridge directly above the existing dam in the early 1960's has to some extent aided in the deterioration process. The drainage and roadway salts from the overhead bridge fall directly onto the dam . The corrosion of exposed steel members has progressed to such an extent that holes are present through the tainter gate skin and sections of structural members are entirely gone. Extensive deterioration has also occurred to the exposed concrete surfaces, resulting in areas of exposed reinforcement and deep failure cracking.

Downstream of the structure, the continuous flows over the dam have created scour holes which go as deep as 4 feet below the bottom of the existing dam apron. From information obtained by underwater inspection during the Konski study, there are no evidences of stone riprap downstream of the dam as originally specified on the construction plans of the existing dam. Uncontrolled scour of this magnitude is considered to be a potential threat to the safety of the dam.

Operation of the dam is currently under the direction of the City of Jamestown, Board of Public Utilities. The Attorney Generals Office has ruled that the State of New York must be considered as the current owner of the dam.

4. Functions of the Existing Dam

The primary function of the existing dam is to maintain the level of Chautauqua Lake for recreational and fishery resources during low flow periods of the year, while ensuring that minimum release objectives are met for the Chadakoin River.

The magnitude of minimum flows for the Chadakoin River to satisfy water quality management objectives, as determined by the Chautauqua County Comprehensive Sewerage Study and subsequent studies by the NYSDEC, is 60 CFS for the summer-fall period and 40 CFS for the winter-spring season. These flows were based on meeting State water quality standards in the Chadakoin River, considering the effects of treated waste water discharges from the South Chautauqua Lake Sewer District plant and the City of Jamestown plant.

Reference is made to Chapter V of the Comprehensive Water Resources Plan for the Allegheny River Basin, as prepared by the NYSDEC for the Allegheny River Basin Regional Water Resource Planning Board. This document contains details on the development of the Chautauqua Lake-Chadakoin River Regulation Plan, which is the current basis for operation of Warner Dam. The Plan bases the regulation of Chadakoin River flows on water quality management, recreation, power generation, and flood control considerations. It was determined in the regulation plan that the water needs for the Jamestown municipal power plant could be satisfied by maintenance of 60 CFS flow during the summer months.

According to the regulation plan, flood damage reduction benefits of the controls on Chautauqua Lake have been reduced in recent years due to shoreline encroachments.

5. The Proposed Action

The proposed action includes construction of a new dam approximately 95 feet downstream of the existing structure. The construction is intended to be a "replacement in kind" type of project. The new structure will be a concrete gravity dam having the same structural elevations and hydraulic

capacity as the existing dam. For economic reasons, the number of tainter gates have been reduced from four to three, but the total clear span of the gates remains at 80 feet (26'-8" per gate x 3 gates = 80'). The gates will be operated by a system of cable drum hoists and electric motors.

Control of seepage under the dam and scour downstream of the dam will be provided by steel sheet piling. In addition, steel sheeting will be used upstream of the dam to provide for maintenance working areas.

If additional details of the proposed action are desired, the reader is referenced to copies of the contract plans which are on file at the City of Jamestown, Department of Public Works; Chautauqua County Department of Public Works; and Chautauqua County Planning office.

The new dam has been located such that the Contractor may divert flows to the south of the cofferdam area during construction. It is anticipated that this will be the method of flow maintenance selected by the Contractor. The diversion would consist of an excavated channel which would carry all low flows plus storm discharges. A crossing of the diversion channel by means of pipes or temporary bridging may be necessary, depending on the contrators scheme for delivery of materials to the dam site.

After completion of the new dam, the existing dam will be removed as described under Section III-1A.

II. ENVIRONMENTAL SETTING WITHOUT THE PROJECT

1. Physical Elements

A. Geology and Soil

The Jamestown area lies in an uplands plateau region which is characterized by the presence of deep glacial deposits. Bedrock under the soils consists of nearly level-bedded shale, siltstone, and fine grained sandstone of the Devonian Age (about 280 million years old). Most of the soils on the plateau have developed from glacial till, defined as the remains of rocks ground up by action of the glaciers and deposited during the glacial retreat.

From a review of existing boring logs from projects immediately adjacent to the dam site, bedrock is approximately 140 feet below ground surface. Borings taken at the location of the proposed dam found the first 15 feet below ground to be composed of miscellaneous fill materials. Below the fill, a very dense strata of sands and gravels was encountered to a depth of 50 feet. Drilling logs from these test holes have been included in the Appendix. The extreme density of this lower strata is reflected by the blow count of the sampler required for penetration, and is indicative of the tremendous pressures exerted by the glaciers.

From field observations, the bed of the existing river channel consists of relatively clean sands and gravels, without evidence of fine grained river bottom sediments.

B. Topography

The uplands plateau area in the vicinity of Jamestown features rolling hills with rounded slopes. The river valley areas, such as that along the Chadakoin River, are relatively flat and sloping in the direction of the river flow.

C. Water Resources

(1) Groundwater Resources

Groundwater is used extensively in the upper plateau region of Chautauqua County as a source of water supply. There are three basic types of deposits which can be identified as follows:

a. Stream deposited beds of sands and gravels, with well yields up to 700 GPM.

b. Till and lake deposits, with yield of 1-10 GPM.

c. Bedrock, with yields up to 100 GPM.

The density and relative imperviousness of the strata found in the 15-50 foot depth range would identify the local subsoils to be of the till and lake deposits nature.

The general quality of the groundwater is good, as attested by the extensive use of wells for water supply in the Jamestown area.

(2) Surface Waters

The current classifications which have been assigned to surface waters tributary to the project area are as follows.

Chautauqua Lake	Class "A"
Chadakoin River, from Chautauqua Lake to Eighteenth Street	Class "C"
Chadakoin River, Downstream of Eighteenth Street	Class "D"

The site of the proposed construction would be within the reach having a Class "D" rating. The best uses of class "A", "C", and "D" waters as defined in the New York State Codes, Rules, and Regulations, Part 6, are as follows:

Class "A" Waters

Source of water supply for drinking, culinary or food processing purposes and any other usages.

Class "C" Waters

Suitable for fishing and all other uses except as a source of water supply for drinking, culinary or food processing purposes and primary contact recreation.

Class "D" Waters

"These waters are suitable for secondary contact recreation, but due to such natural conditions as intermittency of flow, water conditions not conducive to propagation of game fishery or stream bed conditions, the waters will not support the propagation of fish."

The drainage area of the river at the site of the dam is approximately 190 square miles. From records of the USGS gauging station at Falconer, N.Y., from 1935 to 1976, the following data is available on the Chadakoin River flows:

Average Discharge (1935-1976)	= 343 CFS
Minimum Daily Recorded Flow, Flow, Nov. 20, 1960	= 3.0 CFS
Minimum Daily Flow 1973	= 30 CFS
Minimum Daily Flow 1974	= 30 CFS
Minimum Daily Flow 1975	= 40 CFS
Minimum Daily Flow 1976	= 47 CFS
Maximum Peak Discharge, March 5, 1976	= 2070 CFS

The above records indicate that the outflows for extreme flooding occurrences such as the March 1976 flood are quite low for a basin as large as the Chautauqua Lake Basin. This reflects the effect of Chautauqua Lake storage and the lack of adequate conveyance capacity in the Chadakoin River.

2. ECONOMIC ELEMENTS

A. Land Use

The extent of land adjacent to the proposed dam which should be viewed from a land use compatability standpoint would be bounded by the old Erie Railway on the north, Main Street on the east, Harrison Street on the south, and the Washington Street Bridge on the west. Within this area, the land use north of the river consists of a municipal parking lot and a formica countertop factory. Facilities south of the river include a restaurant, sporting goods store, and a lumber yard.

The localized area described above is situated in a general corridor of industrial, commercial, and municipal land uses which parallel the Chadakoin River between Main Street and Third Street. Major features in this larger corridor include the railway, the Board of Public Utilites power plant, and old commercial-industrial type buildings being removed by the City of Jamestown Urban Renewal agency.

B. Employment

The existing Warner Dam provides employment in the sense that personnel from the Municipal Power Plant are responsible for the control and maintenance of the facility.

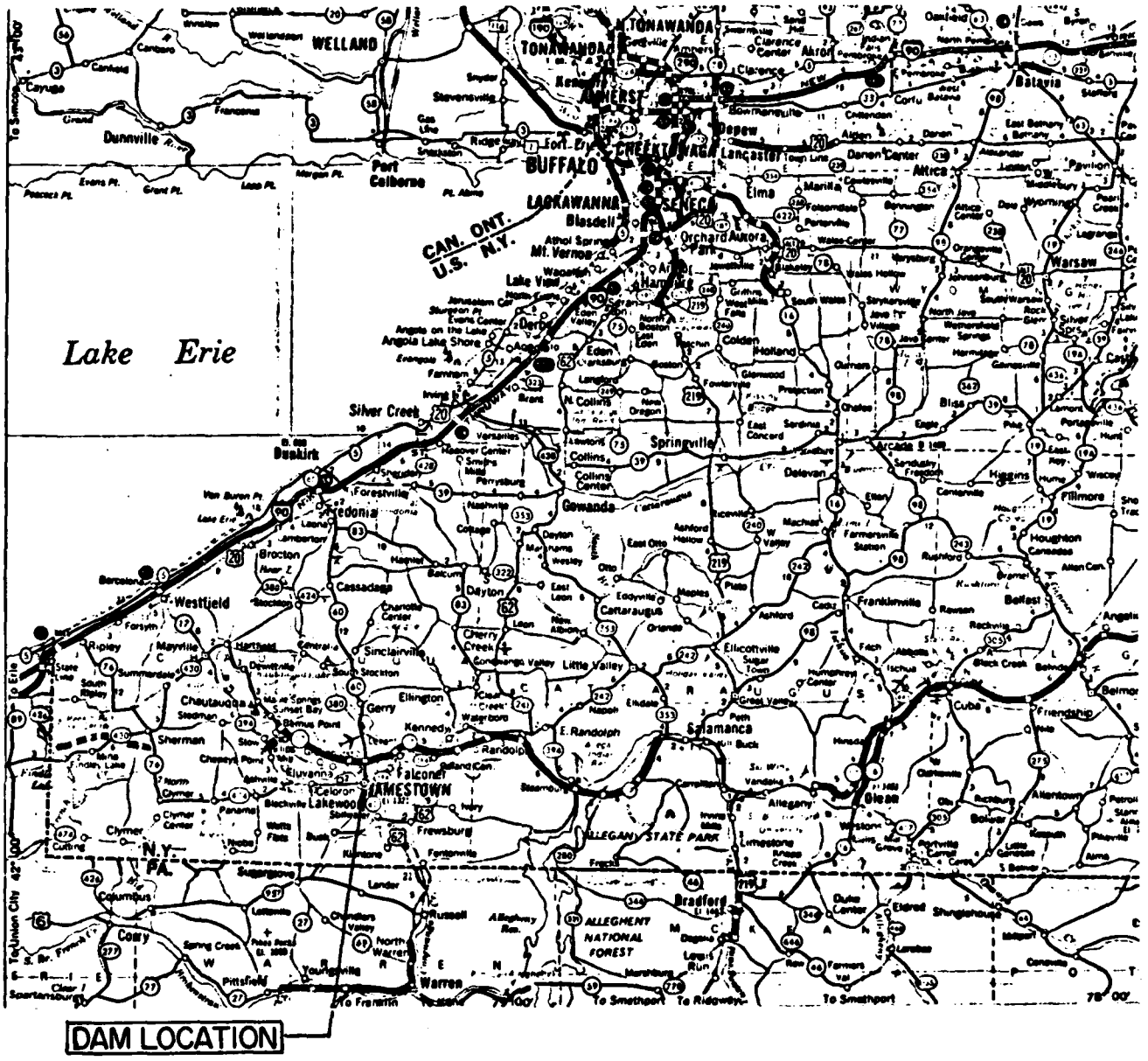
3. SOCIAL ELEMENTS

A. Recreational

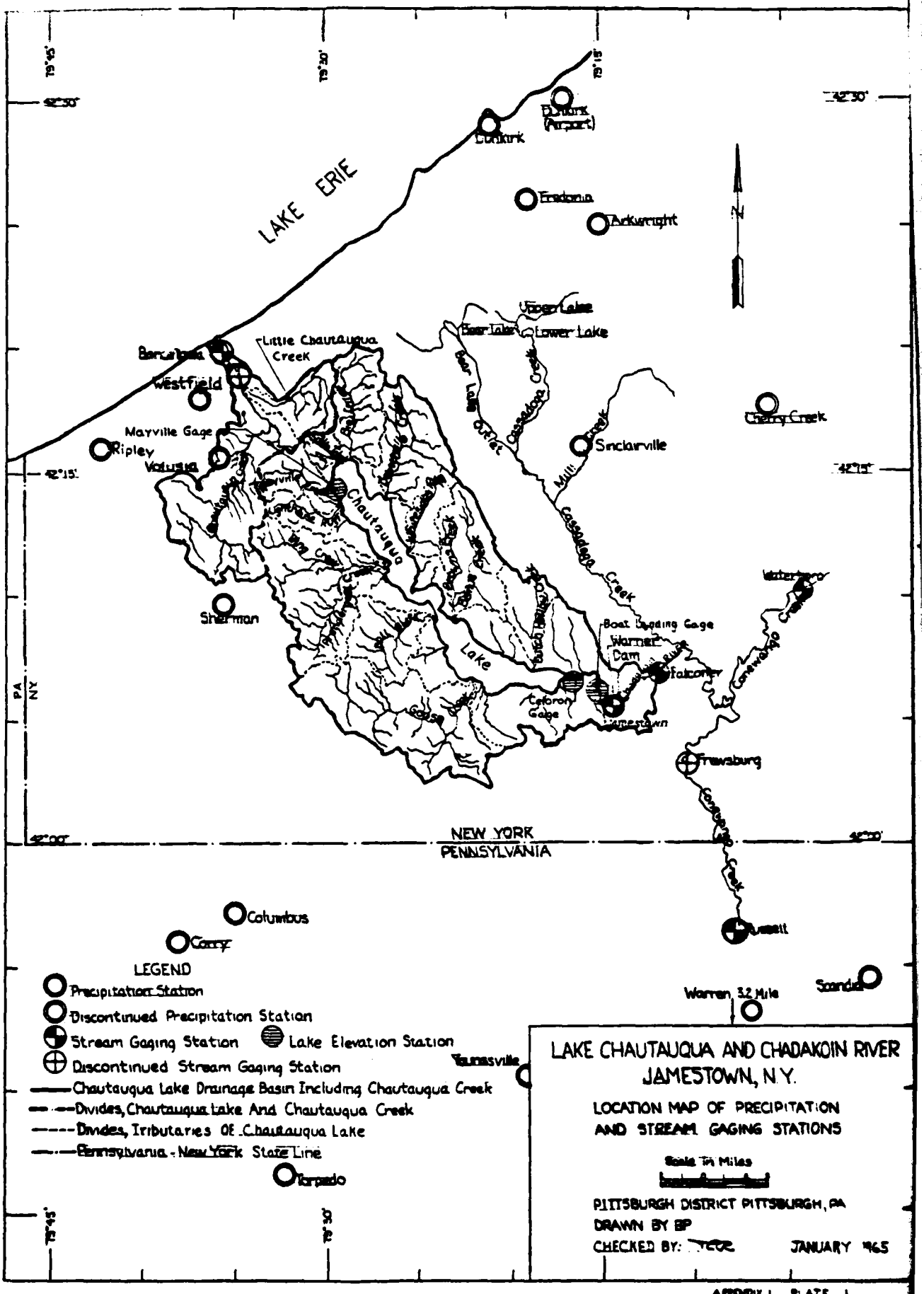
There are no recreational sites or activities currently in the vicinity

APPENDIX G

DRAWINGS



VICINITY MAP
WARNER DAM
CHAUTAUQUA LAKE OUTLET
ALLEGHENY RIVER BASIN
I.D. NO. N.Y. 750



LEGEND

- Precipitation Station
- Discontinued Precipitation Station
- Stream Gaging Station
- Lake Elevation Station
- Discontinued Stream Gaging Station
- Chautauqua Lake Drainage Basin Including Chautauqua Creek
- Divides, Chautauqua Lake And Chautauqua Creek
- Divides, Tributaries Of Chautauqua Lake
- Pennsylvania - New York State Line

LAKE CHAUTAUQUA AND CHADAKOIN RIVER JAMESTOWN, N.Y.

LOCATION MAP OF PRECIPITATION AND STREAM GAGING STATIONS

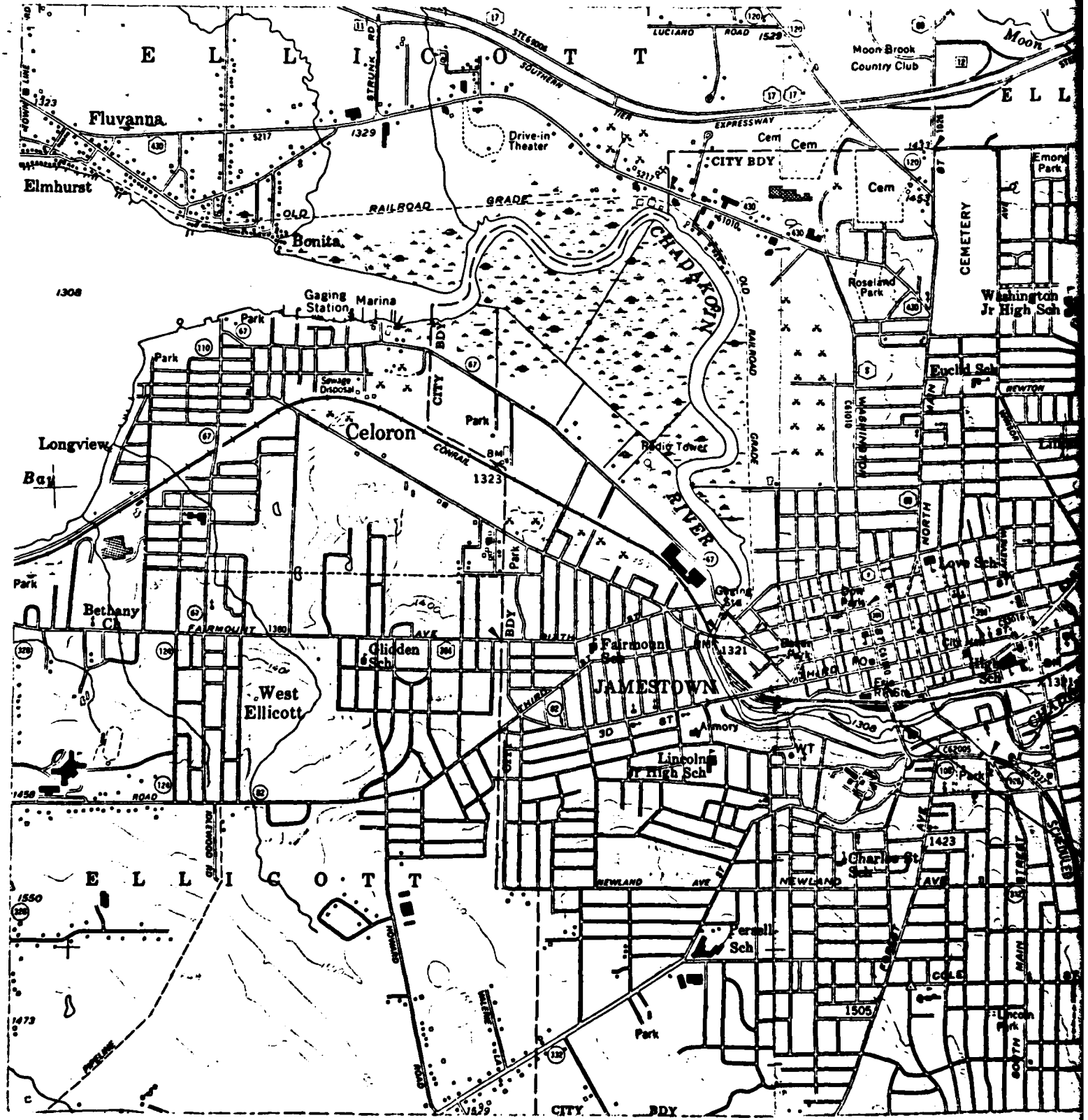
Scale In Miles

PITTSBURGH DISTRICT PITTSBURGH, PA

DRAWN BY BP

CHECKED BY: [illegible] JANUARY 1965

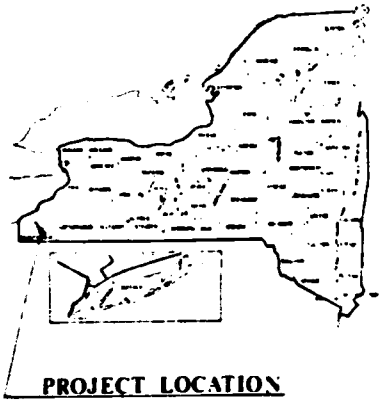
APPENDIX I PLATE 1

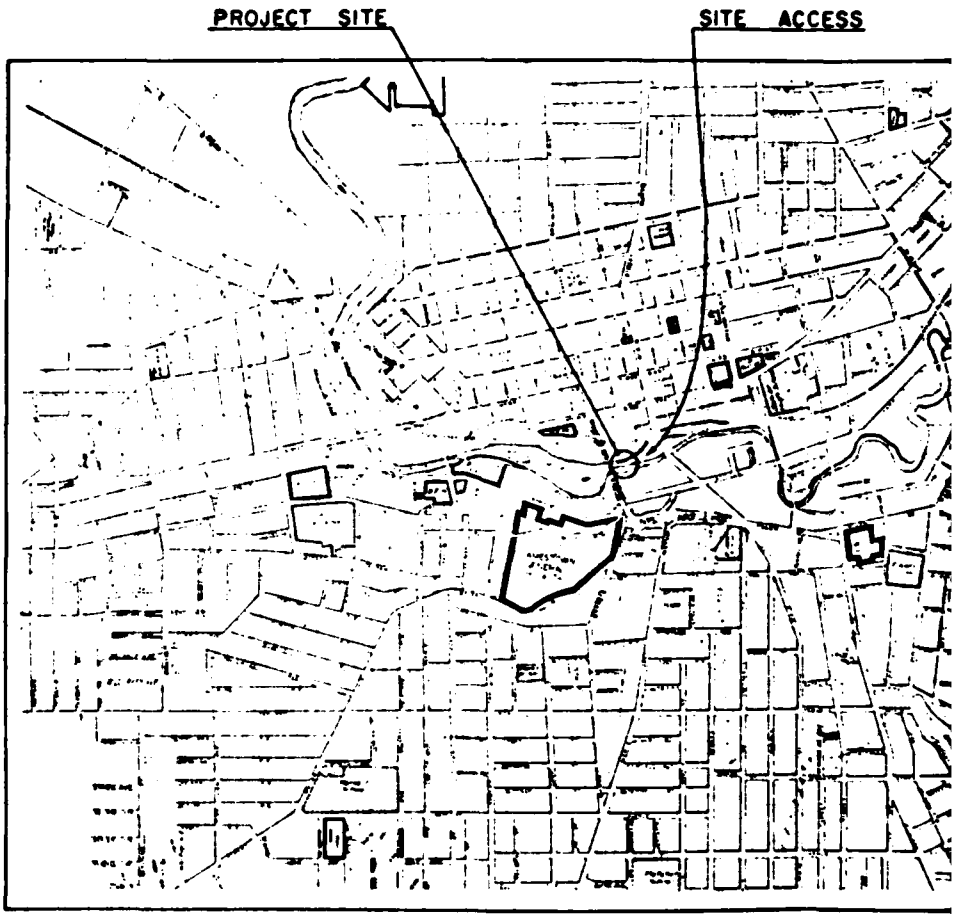


E L L I C O T T
Fluvanna
Elmhurst
Bonita
Drive-in Theater
Moon Brook Country Club
CITY BDY
Cem
CEMETERY
Emory Park
Roseland Park
Washington Jr High Sch
Euclid Sch
Gaging Station
Marina
Park
Sewage Disposal
Celoron
Longview
Bay
Radio Tower
CHADAKOIN
RIVER
Bethany Ch
Glidden Sch
West Ellicott
Fairmount Sch
JAMESTOWN
Armory
Lincoln Jr High Sch
Charles St Sch
Persell Sch
Lincoln Park
Love Sch
E L L I C O T T
CITY BDY

TOPOGRAPHIC MAP
WARNER DAM
CHAUTAUQUA LAKE OUTLET
ALLEGHENY RIVER BASIN
I.D. NO. N.Y. 750

WARNER DAM RE





JAMESTOWN, N

DATE .19

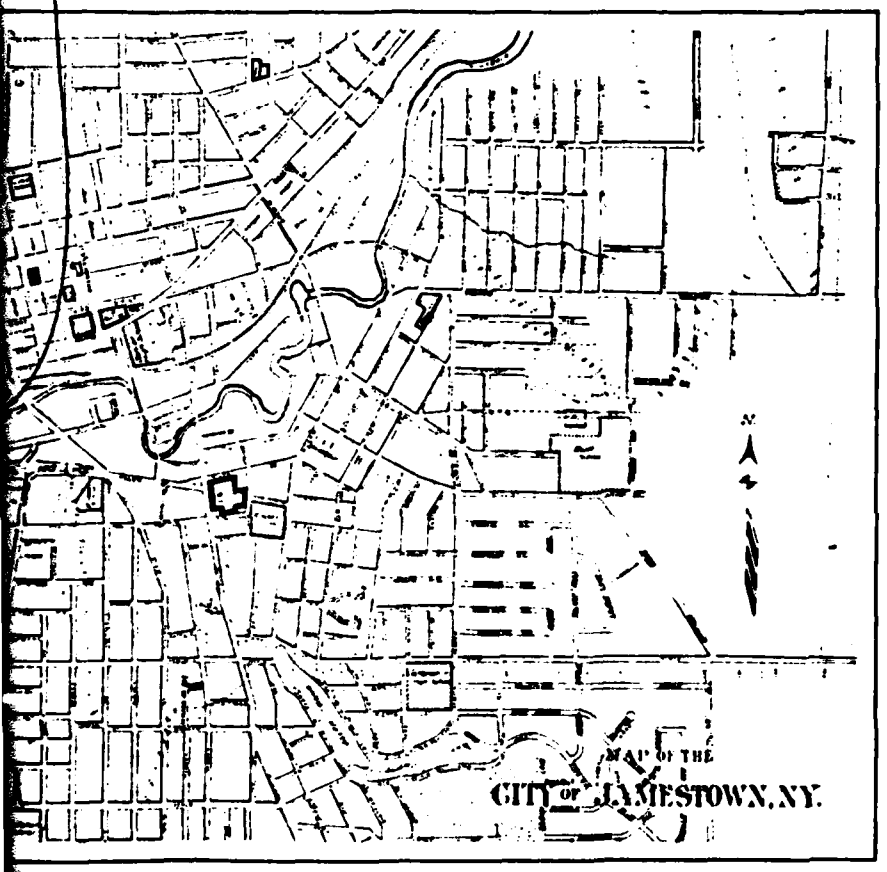
DATE .19

STATE OF NEW YORK

DEPARTMENT OF ENVIRONMENTAL CON

CONTRACT Nº D125679

M REPLACEMENT



RECORD PLANS

FEB 14, 1980

N, NEW YORK

TE OF NEW YORK
ENVIRONMENTAL CONSERVATION
STRACT Nº D125679

ERDMAN, ANTHONY, ASSOCIATES
CONSULTING ENGINEERS & PLANNERS
242 ANDREWS ST
ROCHESTER NY
415 FALLOWFIELD RD
CAMP HILL PENNA

APPROVAL RECOMMENDED _____ DATE _____ 19__

2

TABLE OF BENCHMARKS

	ELEVATION	DESCRIPTION
B.M. 1-B	1312.632	Chiseled "X" in the west cap bolt of a fire hydrant, located approximately 25 feet east of the north side of the existing harbor Dam and approximately 31 feet east of the east side of the A Street Bridge.

LIST OF ABBREVIATIONS

Sta.	Station
	Centerline
	Gasoline
	Pavilion
	At
Max.	Maximum
Min.	Minimum
Stel.	Standard
Std.	Steel
	Diameter
A.S.T.M.	American Society for Testing Materials
	Length
No.	Number
Az.	Azimuth
	Angle
PSF	Pounds per Square Foot
	Reinforced Concrete Pipe
	Manhole
	Inside Diameter
	Invert
	Reinforced Concrete
	Cubic Yard
	Square Yard
Tr.	Trench
Sty	Storm
	Right
	Abandoned
	Property Line
	Permanent Benchmark
	Point of Vertical Intersection
	Elevation
	Point of Curve
	Point of Tangent
	Point of Intersection
	Point of Reverse Curve
	Point Back
	Point Ahead
	Right of Way
	Radius
	Tangent
	Chord
	Degree of Curve
	Central Angle
	Length of Curve
	Tangent Distance
	External Distance
	Middle Ordinate
	Long Chord
	Point of Compound Curve
	Point of Reverse Curve

SURVEY CO

HORIZONTAL CONTROL

The existing control in the vicinity was considered for selecting the closing traverse is part of a traverse established and Geodetic Survey. Station "J" Azimuth Mark were used for a sta The accuracy of these stations is & G.S. as second order. The coor are based on the 1927 North Amer dinates are computed on the Trans Plane Coordinate System, West Zo

VERTICAL CONTROL

The bench mark is a loop originate & G.S. No. 1 (City of Jamestown) road, 500 feet east of the statio walk on the south side of Cherry of a water column, 70 feet south eastbound track, at the top of a and about 1 foot lower than the of Jamestown bronze disk.

SURVEY BA

NUMBER	STATION
P.I. 1	9+77.87
P.O.C. 1A	12+54.16
P. B	.5+78.7o

GENERAL

CENTERLINE GEOMETRY

POINT	STATION	COORDINATES	CURVE DATA	AZIMUTH
PI	A 10+00.00	N 763426.54 E 321034.69		
PC	A 10+39.97	N 763410.61 E 321071.34	Δ= 51°-17'10" D= 95° 29' 35"	113° 29'19"
PI	A 10+70.07	N 763398.62 E 321098.95	R= 60.00' T= 30.10 L= 55.80 E= 7.13	113° 29' 19"
PT	A 10+95.77	N 763413.57 E 321125.07		60°12' 09"
PI	A 11+39.37	N 763435.20 E 321162.84		60° 12' 09"
PI	A 12+55.57	N 763465.62 E 321275.14		74° 52' 28"

POINT	STATION	COORDINATES	CURVE DATA	AZIMUTH
PI	W 10+00.00	N 763430.77 E 321114.59		
				170°-50'-60"
PI	W 11+37.78	N 763294.82 E 321136.50		
				241°-50'-60"
PC	W 12+42.96	N 763265.15 E 321063.70	Δ= 83°-00'-00" D= 114° 35'-30"	241°-50'40"
PI	W 12+67.20	N 763224.28 E 321004.70	R= 50.00' T= 44.24 L= 72.43 E= 16.76	158° 50' 40"
PT	W 13+15.40	N 763183.03 E 321020.66		158° 50' 40"
PI	W 14+42.20	N 763018.14 E 321084.47		157° 50' 4
PI	W 15+91.10	N 762926.54 E 321121.75		

TABLE OF C

SHEET	TITLE
1.	Cover
2.	Index and Legend
3.	Site Plan
4.	Site Plan
5.	Typical Sections, Earthwork and
6.	Water Main Relocation Details
7.	Standard Structure Sheet
8.	General Structural Plan
9.	General Structural Elevations
10.	Deck Plans and Railway Details
11.	Abutment Details
12.	Intermediate Pier Details
13.	Concrete wall Repair Details
14.	Pile Schedule
15.	Steel Sheet Piling Details
16.	Fencing and Railing Details
	[illegible]
	[illegible]
	[illegible]
	[illegible]
	[illegible]

SURVEY CONTROL

AL CONTROL

ting control in or near the project area which was ed for starting and closing the primary traverse of a traverse established by United States Coast etic Survey. Station "Jamestown 1935" and Jamestown Mark were used for a starting and closing azimuth. racy of these stations is published by the U.S.C. a second order. The position of these stations d on the 1927 North American Datum and plane coor- are computed on the Traverse Mercator Projection, ordinate System, West Zone.

CONTROL

h run is a loop originating and closing at U.S.C. o. 1 (City of Jamestown) located on the Erie Rail- 0 feet east of the station, in line with the side- the south side of Cherry Street, 120 feet southeast r column, 70 feet south of the south rail of the track, in the top of a concrete retaining wall 1 foot lower than the track. Station is a City own bronze disk.

SURVEY BASELINE

STATION	AZIMUTH	COORDINATES
9+77.87	90°-54'-47"	N 763,432.81 E 320,986.48
12+54.16	59°-19'-50"	N 763,428.41 E 321,262.66
15+78.76		N 763,593.97 E 321,541.84

GENERAL NOTES

TABLE OF CONTENTS

TITLE	Legend
d Legend	T-1
n	PL-1
n	PL-2
Sections, Earthwork and Restoration	TS-1
in Relocation Details	W-1
Structure Sheet	S-1
Structural Plan	S-2
Structural Elevations	S-3
and Walkway Details	S-4
Details	S-5
rate Pier Details	S-6
wall Repair Details	S-7
Hole	S-8
et Piling Details	S-9
nd Railing Details	S-10
Gate, [illegible]	S-11
Gate, [illegible]	S-12
[illegible]	S-13
[illegible]	[illegible]

LEGEND

PROPOSED

- FEE TAKING
- EASEMENTS, P.E. or T.E.
- SIDESLOPE IN CUT OR FILL
- DITCH or CHANNEL
- STONE FILL
- STRUCTURE
- CABLE GUIDE RAILING
- SPOT ELEVATIONS
- R.O.W. MARKERS, CONCRETE TYPE L (low) Item 625.04
- BORING LOCATION
- SUBBASE COURSE
- ELECTRIC PULLBOX, TELEPHONE PULLBOX
- STREET LIGHT, STD LUMINAIRE
- ELECTRIC
- TELEPHONE

EXISTING

- BORING LOCATION
- TEST CORE
- BENCHMARK
- ℄ P.I., ℄ P.I.
- GAS MAIN
- WATER MAIN
- SANITARY SEWER
- STORM SEWER
- MANHOLE
- OVERHEAD — ELECTRIC LINE, OVERHEAD
- TREE
- TREES or BRUSH
- STREAM
- UTILITY POLE
- FENCE
- PROPERTY LINE
- PE — EXISTING EASEMENT
- 1310.0 — SPOT ELEVATIONS
- HYDRANT
- R.O.W. MARKER

ERDMAN ANTHONY ASSOCIATES

CONSULTING ENGINEERS & PLANNERS

ROCHESTER, N.Y. CAMP HILL, P

DATE DATE

NOTE:

UNAUTHORIZED ALTERATION OR ADDITION TO THIS DRAWING IS A VIOLATION OF THE NEW YORK STATE EDUCATION LAW ARTICLE 145, SECTION 7209.

REVISIONS

NO.	DATE	DESCRIPTION	REV.	CHK

No As Built Revisions

PROJECT NAME

WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO. D125679

CLIENT

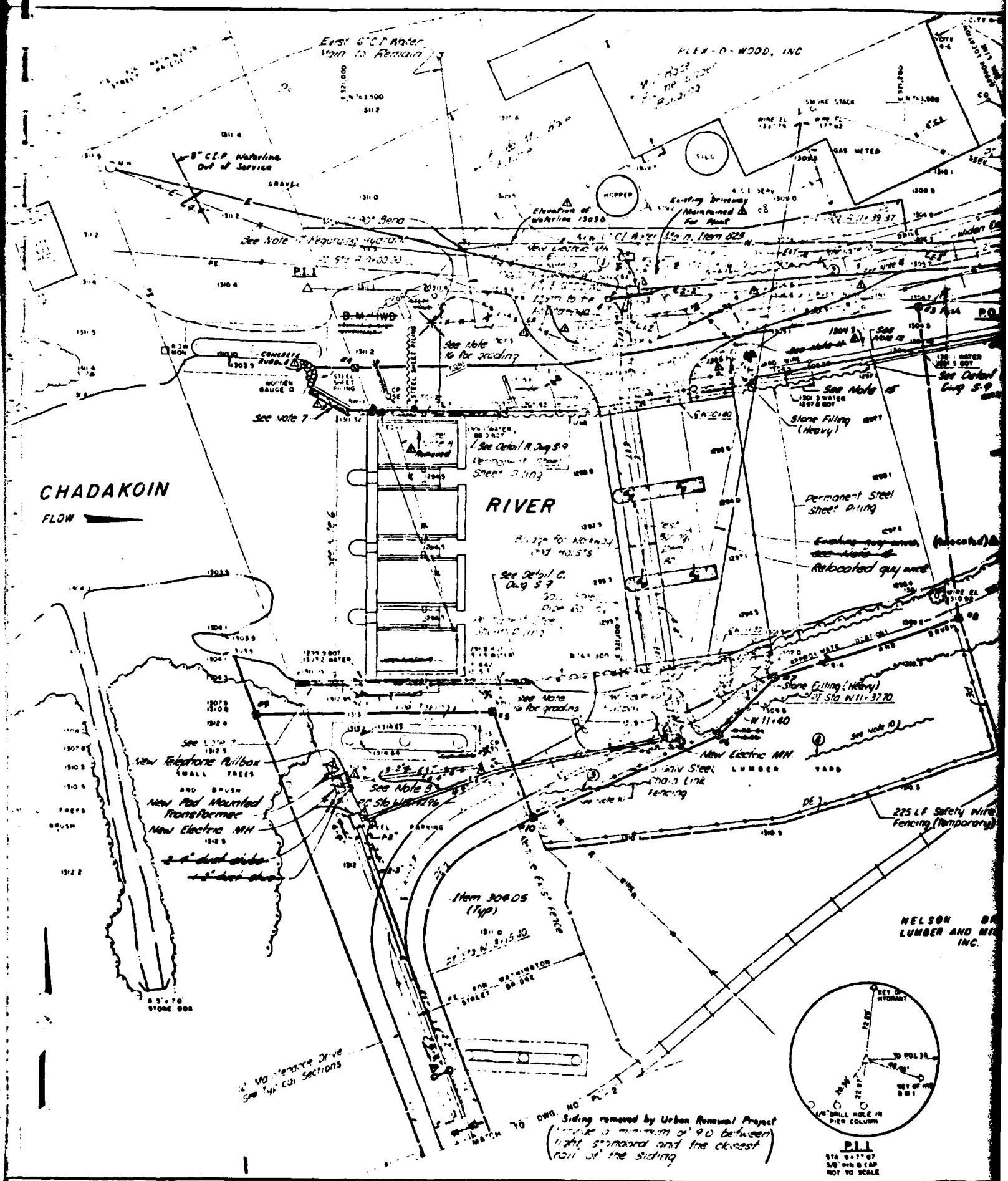
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE

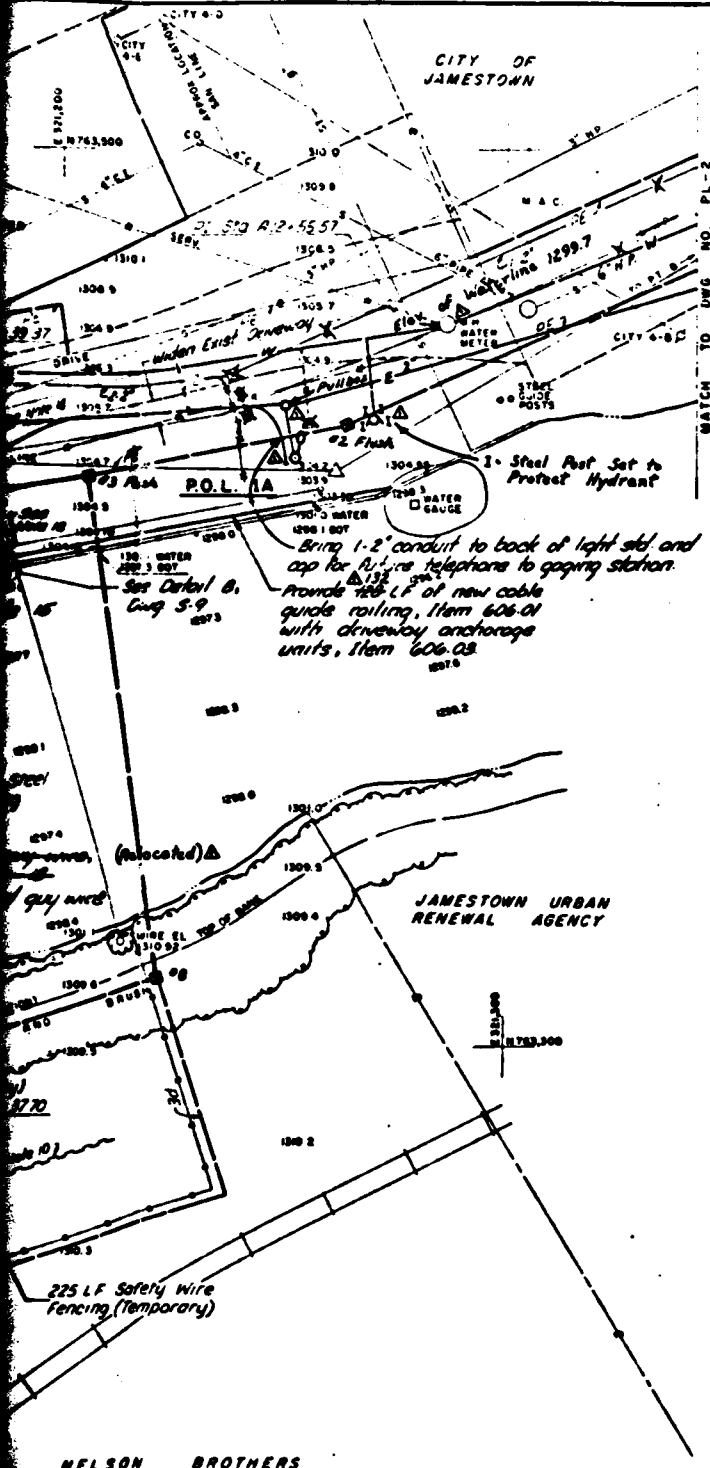
INDEX AND LEGEND

NONE — KPA — SP — ECI — 6/30/78 — 2

2

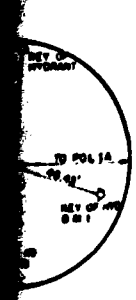


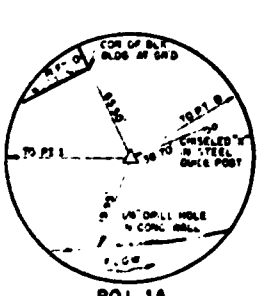
PLEX-O-WOOD, INC
CHADAKOIN
FLOW
RIVER
Permanent Steel Sheet Piling
Relocated guy wire
Stone Filling (Heavy)
New Telephone Pullbox
New Pad Mounted Transformer
New Electric MH
Item 304.05 (Typ)
225 LF Safety Wire Fencing (Temporary)
NELSON BR
LUMBER AND MI
INC.
Siding removed by Urban Renewal Project
P.I.1

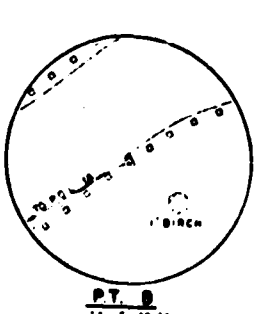


NOTES

1. Superstructure of Washington Street not shown.
2. Water surface elevations shown were at time of survey. Water levels vary. See specifications or Contract Documents for Hydrology Data.
3. Location of existing borings are approximate only.
4. Elevations shown on bottom of river are approximate only.
5. Details of new dam such as gate operators, etc. not shown on 1"=20' scale plan. See structural plans for details.
6. All features of the existing dam between the abutment walls to be removed to elevation 1300.0 under Item 202.12. Item 202.2 also includes removal of fencing on abutment walls.
7. Existing abutment walls to be recapped. See Details, Dwg S-7.
   ⚠ NW Wingwall-5' cap on side to elev. 1306.5
8. Existing light pole and luminaire to be removed and salvaged for Board of Public Utilities, cost included in Item 201.0601.
9. Existing ROW Marker to be reset after all work items in this area are completed. Cost to be included in Item 625.06.
10. Proposed steel sheeting deadman T-rods not shown. See structural plans. Numbers shown on sheeting deadmen refer to Table "A" on Dwg S-9.
11. ~~Provide 22 LF of 18" CSP, 16 ga., with end sections. Culvert to be located at E 811.06. Set invert to be a minimum of 12.2' higher than south invert.~~ Not Needed. ⚠
12. Existing cable guide railing to be removed under Item 201.0601.
13. Profile of "A" line shall be El 1311.5 from "A" 10+00 to "A" 10+91, 10% slope from "A" 10+91 to "A" 11+40.
14. Existing Gas Main and Service to be relocated if necessary by National Fuel Gas Co., cost to be paid by State of New York.
15. Row of existing timber piles observed during low water period extending approximately 85' to west from this point along base of existing wall.
16. Area to be filled and graded to drain west, refer to Dwg TS-1.
17. Existing Hydrant to be relocated to a point on the new 6" water main near the meter pit. Exact location of hydrant to be set by Plex-O-Wood Inc. and fire insurance agent.
18. Contractor to provide adequate temporary support of existing guy wire until new anchor on abutment wall is completed.







2

ERDMAN ANTHONY ASSOCIATES

CONSULTING ENGINEERS & PLANNERS
ROCHESTER, N.Y. CAMP HILL, PA.

DATE 6/29/78

NOTE.
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REVISIONS

NO	DATE	DESCRIPTION	
1.	2/19/80	As Built	JFM

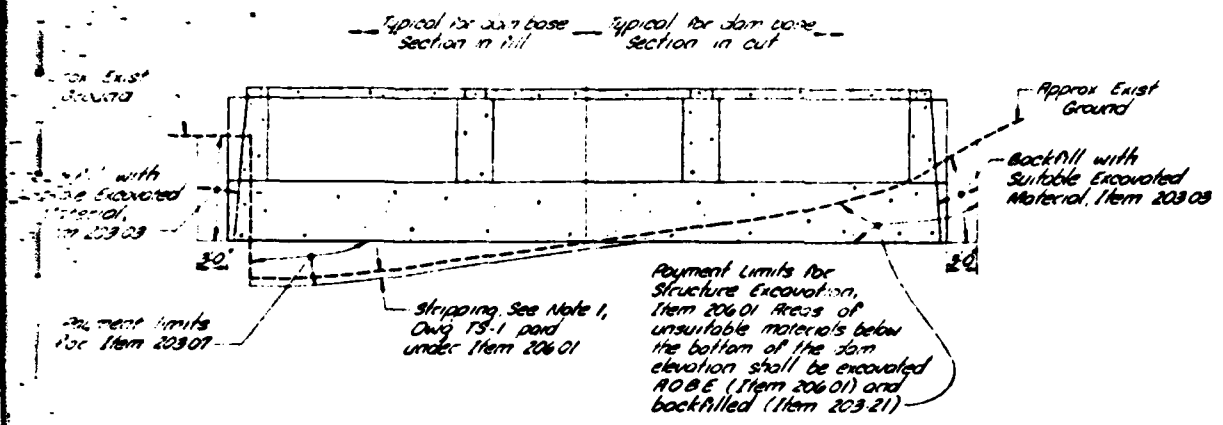
PROJECT NAME
WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO. D125679

CLIENT
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

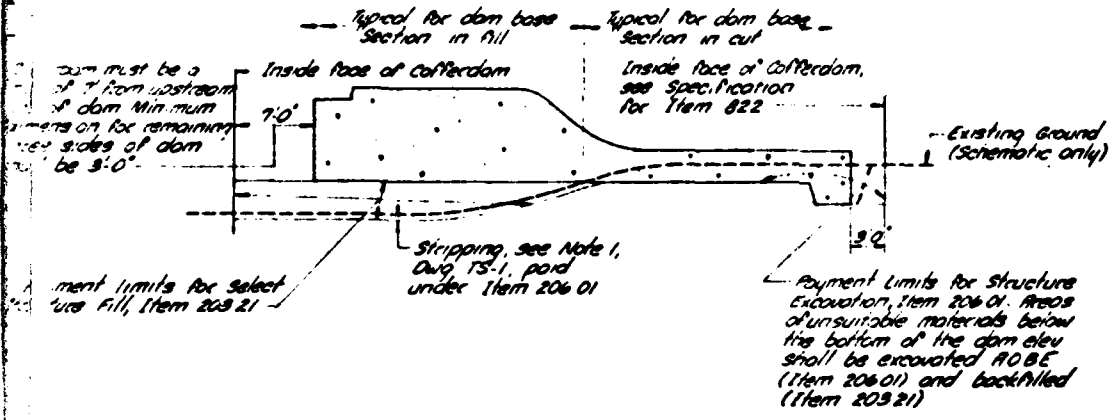
DRAWING TITLE

SITE PLAN

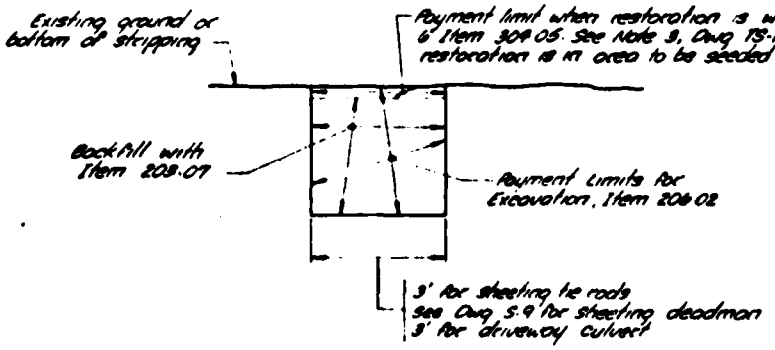
1" = 20'	APA SP ECT	PL
6/30/78		3



LONGITUDINAL SECTION THRU DAM



TRANSVERSE SECTION THRU DAM



MINOR STRUCTURE EXCAVATION

EXCAVATION & BACKFILL DETAILS

No Scale

STANDARD STRUCTURE NOTES

1. Material and Construction Specifications for Specifications of the New York State Department of Transportation dated January 3, 1978, with current Addenda.

2. Concrete for Structures shall be Class "A", Item 555.01 or Class "B", Item 555.02 as noted on the Plans. Structures have been designed for 28 day concrete compressive strength of 3000 psi.

3. All bar reinforcement for concrete shall conform to Item 556.0201.

4. Details of reinforcement not specifically shown shall be in conformance with A.C.I. Standard 318-71.

5. All exposed edges of concrete shall be chamfered 1".

6. The following minimum cover dimensions shall be required for all bar reinforcement in structural concrete:

   Dam walkway and cap of existing wall 2"
   All other concrete pours 3"

7. The cost of all joint material will be included in the price bid for the various items of the Contract, except as otherwise specified.

8. Bituminous material, Item 558.01 shall be applied to the backs of all walls above top of footings where fill is in contact with the walls.

9. All embankments shall be compacted to not less than 95% of Standard Proctor Maximum Density.

10. For design purposes the foundation pressure does not exceed 1.5 tons per sq. foot.

11. All anchor bolts, nuts, and washers, except stainless steel hardware, shall conform to ASTM A 307, grade B, and shall be galvanized in accordance with ASTM A 153.

ERDMAN ANTHONY ASSOCIATES

CONSULTING ENGINEERS & PLANNERS

ROCHESTER N.Y. CAMP HILL PA

DATE 6/27/78 DATE

NOTE:

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REVISIONS

NO	DATE	DESCRIPTION	[illegible]

No As Built Revisions

PROJECT NAME

WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO D125679

CLIENT

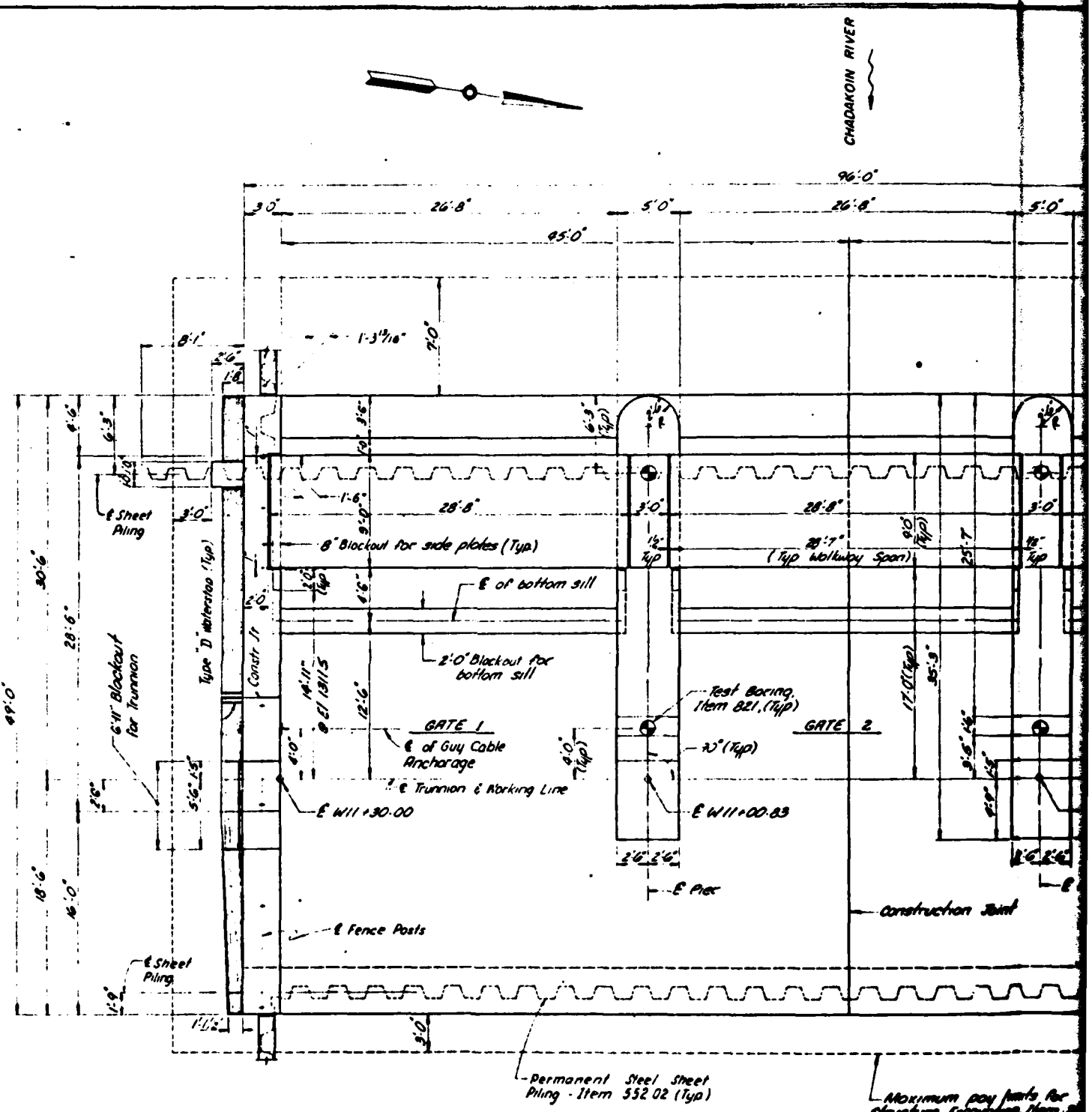
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE

STANDARD STRUCTURE SHEET

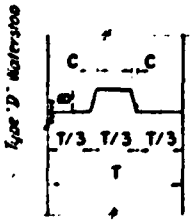
SCALE NO SCALE	DRAWN BY KPA / CHECKED BY RJ	DRAWING NO. S-1
DATE ISSUED 6/30/78	CKD BY E.C.T. / DATE	SHEET 7 OF 21

2



NOTES:

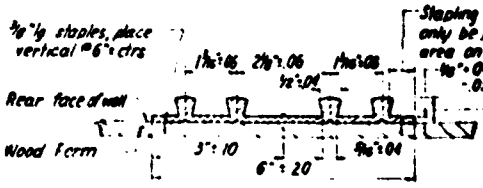
1 Gates and Operators not shown on this Dwg
2 For Fence Post Spacing See Dwg No S-3



C	B	T/3
1/4"	1 1/2"	to 6"
1/2"	3 1/2"	6" to 10"
3/4"	5 1/2"	over 10"

Note All Keys should begin and end 1'-0" ± from the edge of pour as indicated above

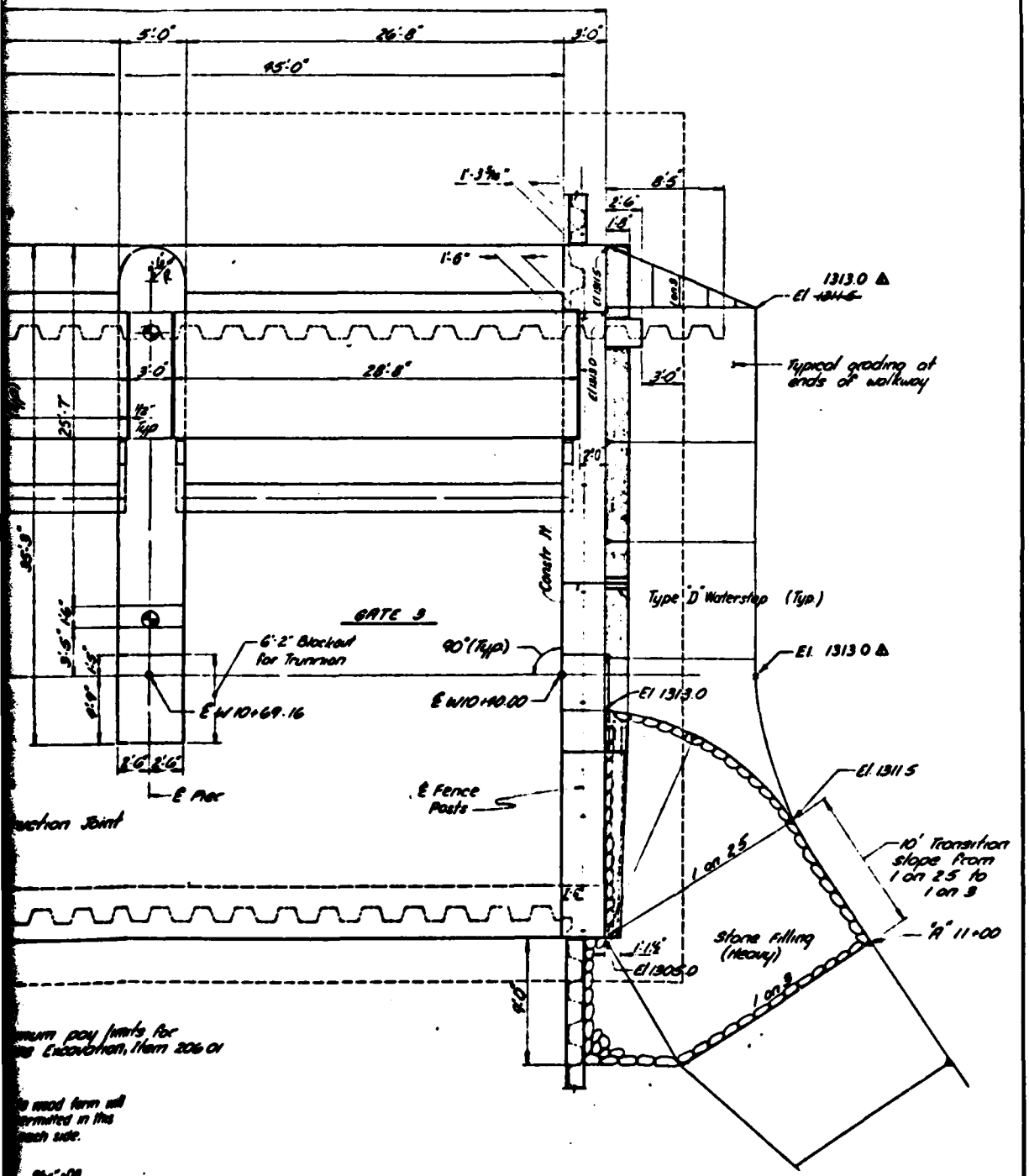
CONSTRUCTION JOINT DETAIL



TYPE D WATERSTOP

No scale

Note: Waterstops shall ... of Section 555.3.07 ... Holes must not be ... for any purpose ... stapling to forms ... Type D Waterstop ... in color. The cost of ... waterstops shall ... price bid for the ...



Note: Waterstops shall conform to the requirements of Section 205.07 of the Specifications. Holes must not be made in waterstop for any purpose except as required for stapling to forms. Type "D" Waterstop shall be light gray in color. The cost of furnishing and placing waterstops shall be included in the unit price bid for the concrete items.

Note: Test borings shall be performed by the Contractor as located hereon or as ordered by the Engineer in Accordance with Item 821 of the Specifications.

ERDMAN ANTHONY ASSOCIATES
CONSULTING ENGINEERS & PLANNERS
ROCHESTER, N.Y. CAMP HILL, PA.

NOTE:
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REVISIONS

NO.	DATE	DESCRIPTION	BY
1	2/14/80	As Built	JEMM

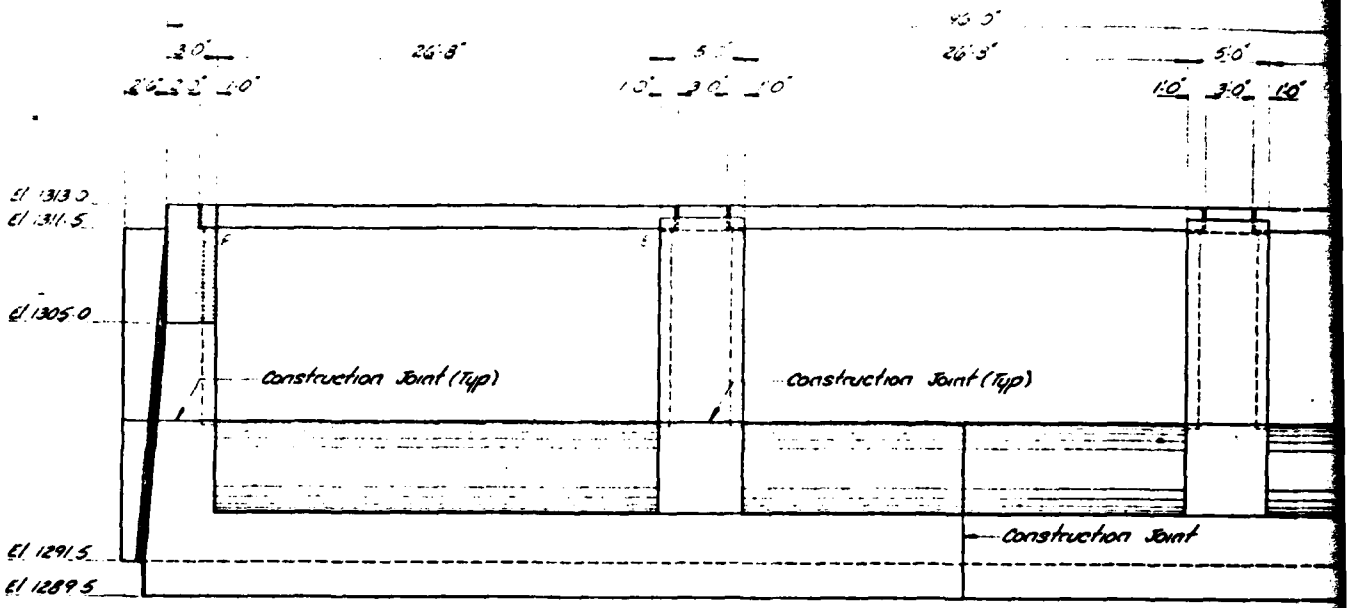
PROJECT NAME
WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO. D 125679

CLIENT
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

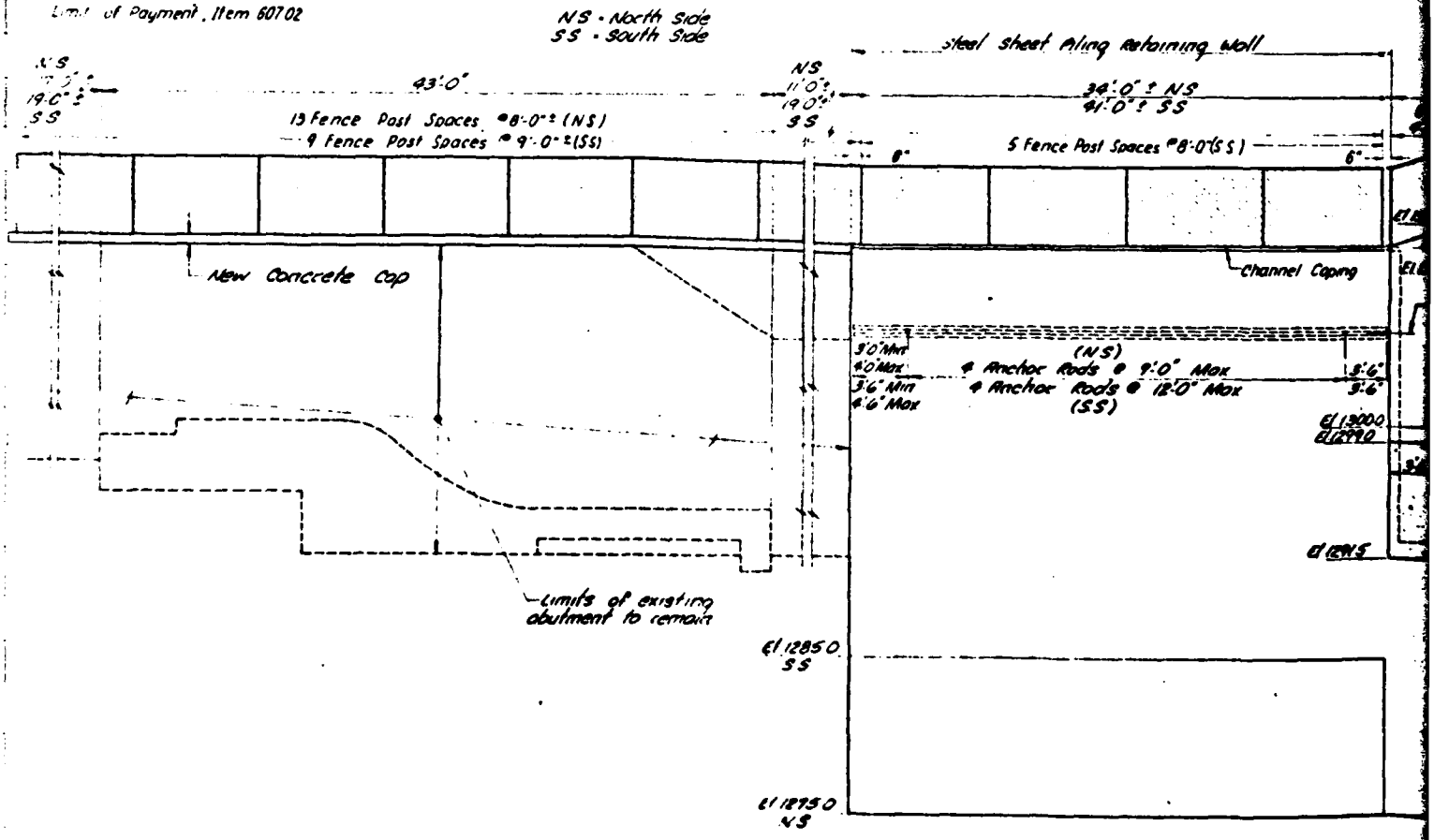
DRAWING TITLE
GENERAL STRUCTURAL PLAN

3/16" = 1'-0"	MD RJ	S-2
APP'D	CAD BY KK	SHEET NO.
6/30/78		

2

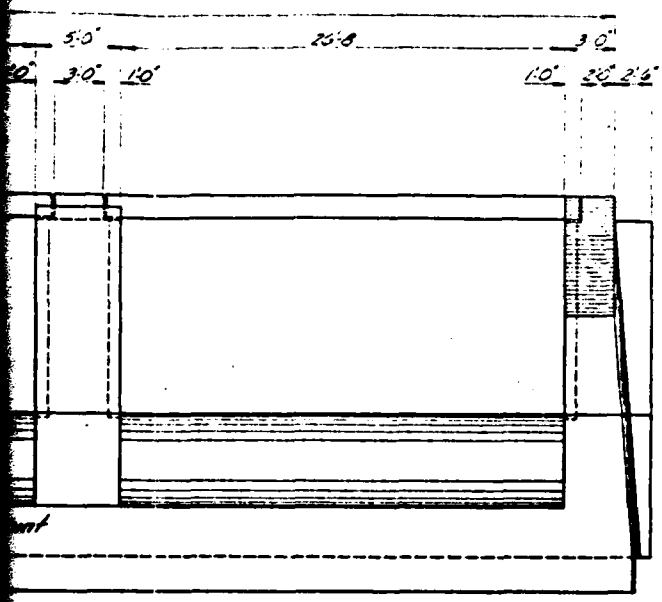


END ELEVATION - FACING SPILLWAY



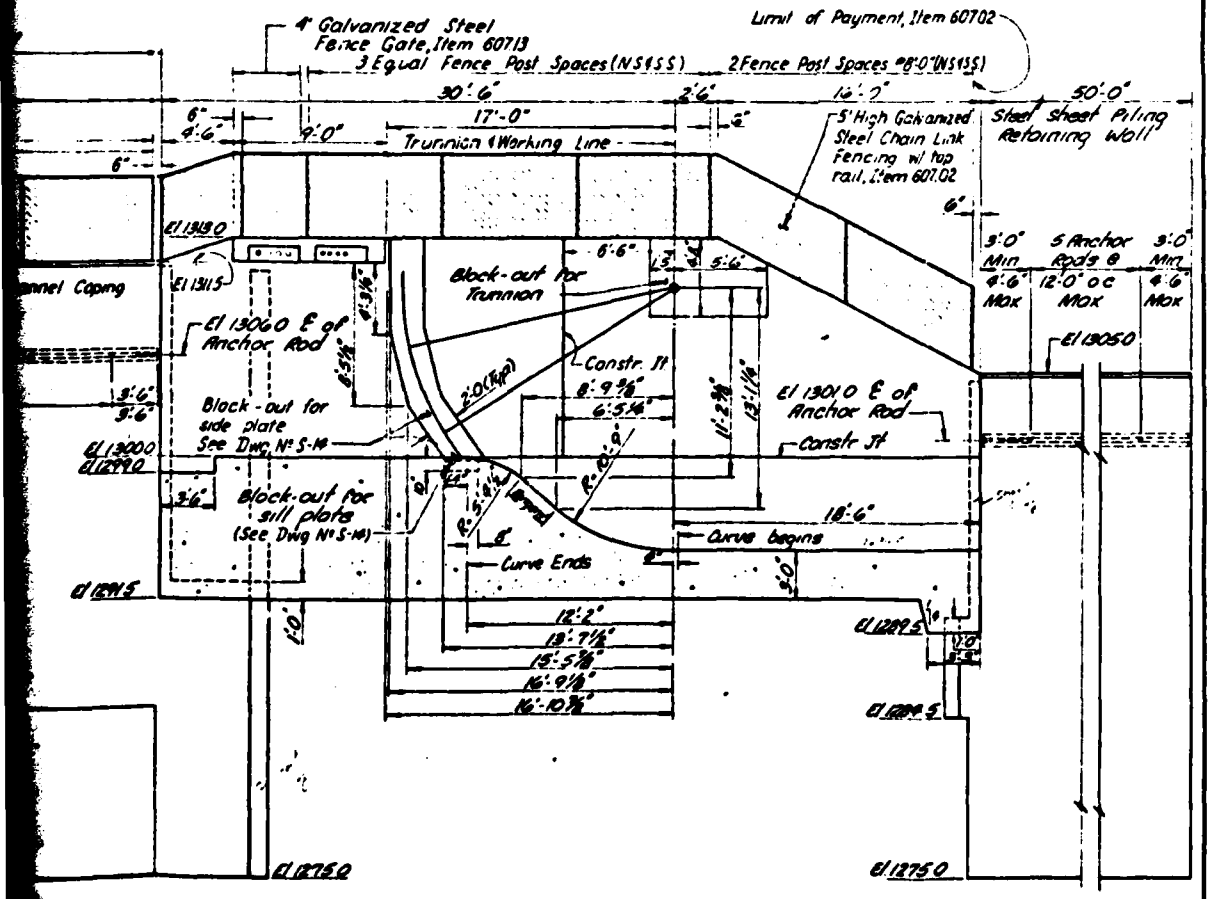
INTERIOR ELEVATION

NORTH SIDE SHOWN - SOUTH SIDE OPPOSITE HAND



Notes:

1) Gates and Hoisting equipment not shown on End Elevation Hoisting equipment not shown on Interior Elevation
2) For Concrete Cap Details See Dwg N° S-7
3) For Fence Post Detail See Dwg N°S-10



OPPOSITE HAND

ERDMAN ANTHONY ASSOCIATES

CONSULTING ENGINEERS & PLANNERS

ROCHESTER, N.Y. CAMP HILL, PA.

DATE DATE

NOTE:

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REVISIONS

NO.	DATE	DESCRIPTION

No As Built Revisions

PROJECT NAME

WARNER DAM REPLACEMENT

JAMESTOWN, NEW YORK

CONTRACT NO D 125679

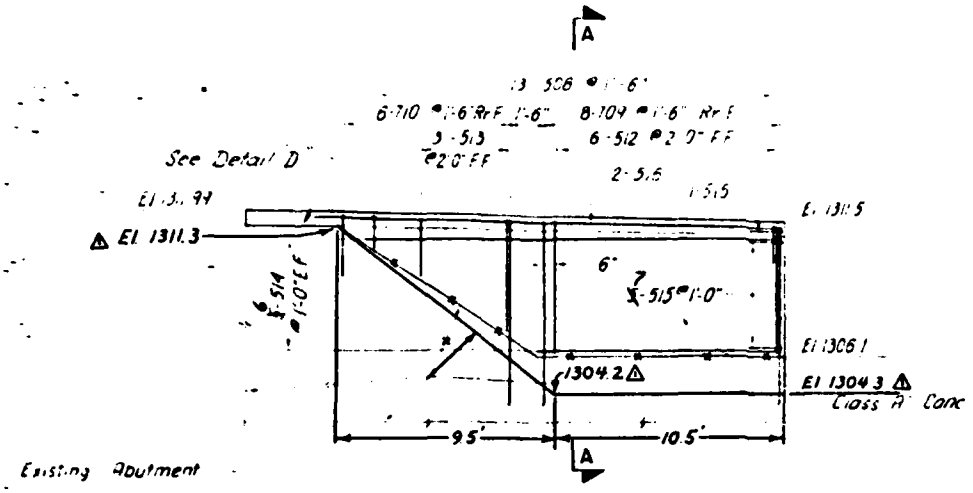
CLIENT

STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

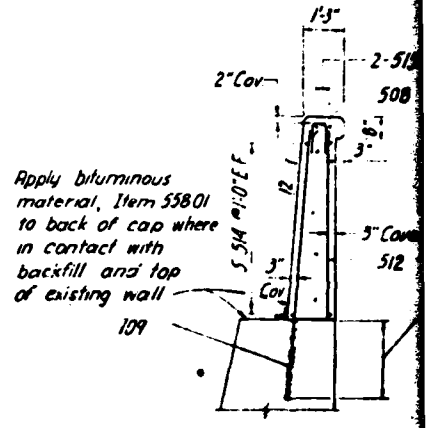
DRAWING TITLE

GENERAL STRUCTURAL ELEVATION

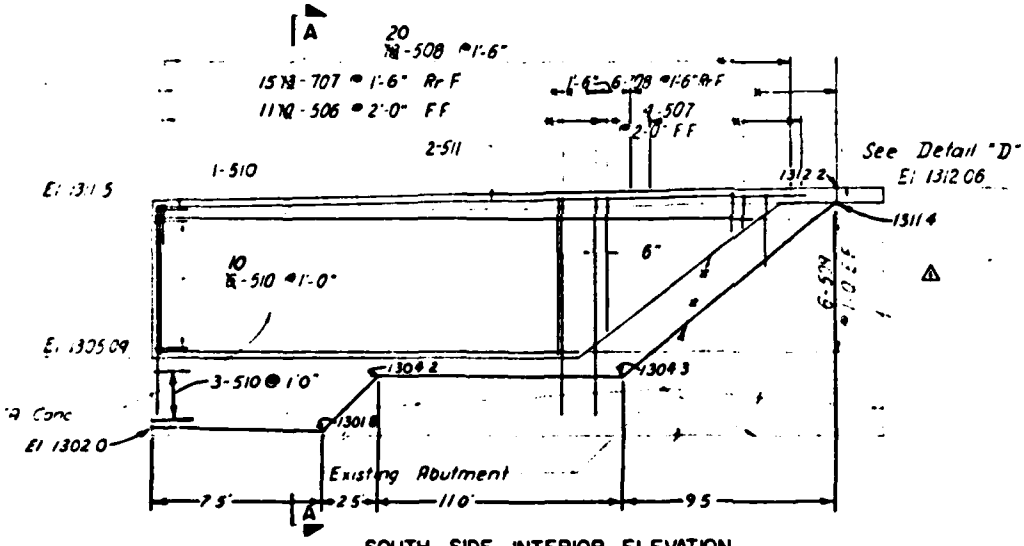
3/16" = 1'-0"	MD RJ	S-3
DATE TRACED	CHKD: KK	SHEET NO.



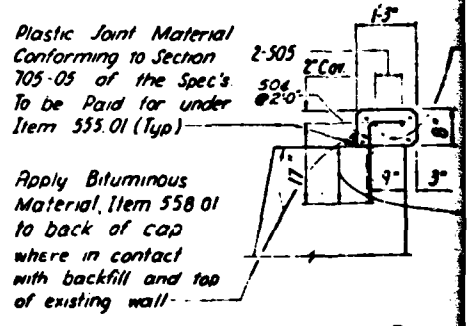
NORTH SIDE INTERIOR ELEVATION



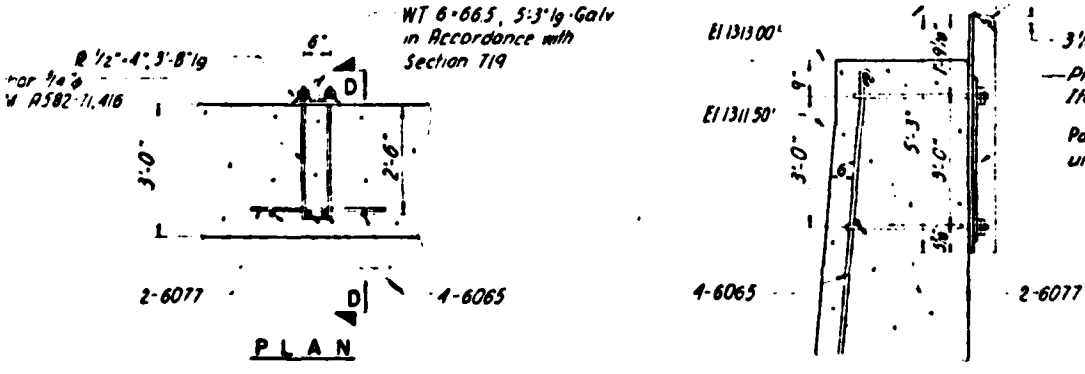
SECTION A-A



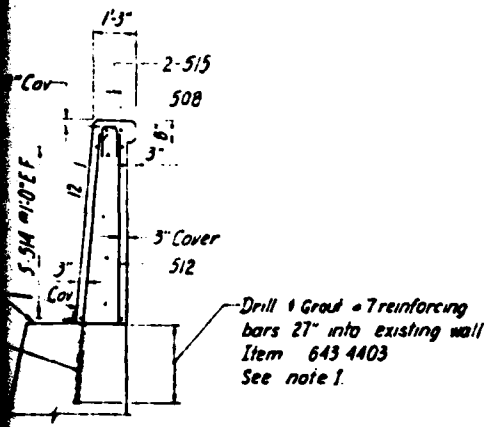
SOUTH SIDE INTERIOR ELEVATION



DETAIL D
CAP ON EXISTING WAL



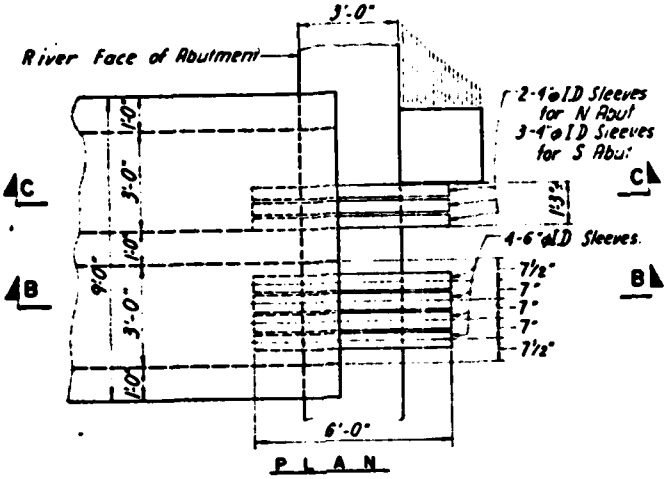
GUY CABLE ANCHORAGE DETAIL
Scale 1/2"=1'-0"



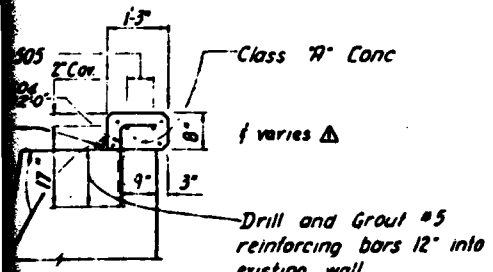
SECTION A-A

Notes

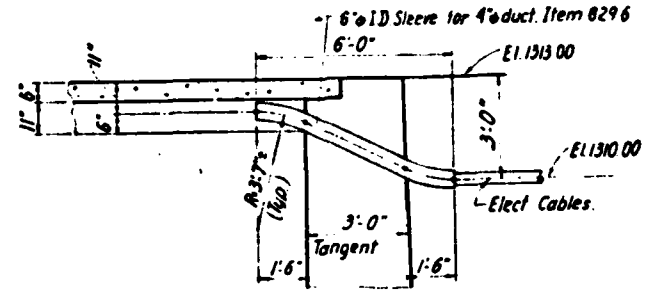
1. Contractor shall remove unsound concrete from top of existing wall before adding cap.
2. Dimensions & elevations of existing structure shall be verified in the field by the contractor.



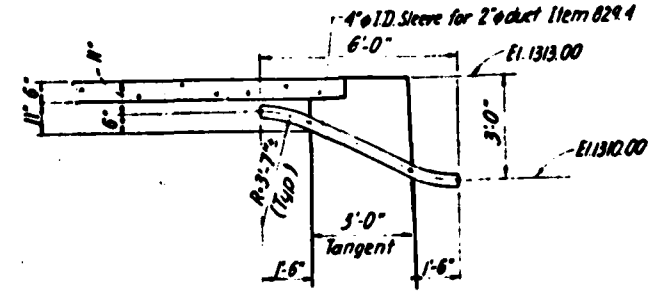
PLAN



DETAIL D
EXISTING WALL



SECTION B-B



SECTION C-C

DETAILS OF SLEEVES THROUGH ABUTMENT WALLS

South Abutment Shown
North Abutment Similar Except as Noted
Scale ¼"=1'-0"

2

ERDMAN ANTHONY ASSOCIATES
CONSULTING ENGINEERS & PLANNERS
ROCHESTER, N.Y. CAMP HILL, PA

DATE DATE

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REVISIONS

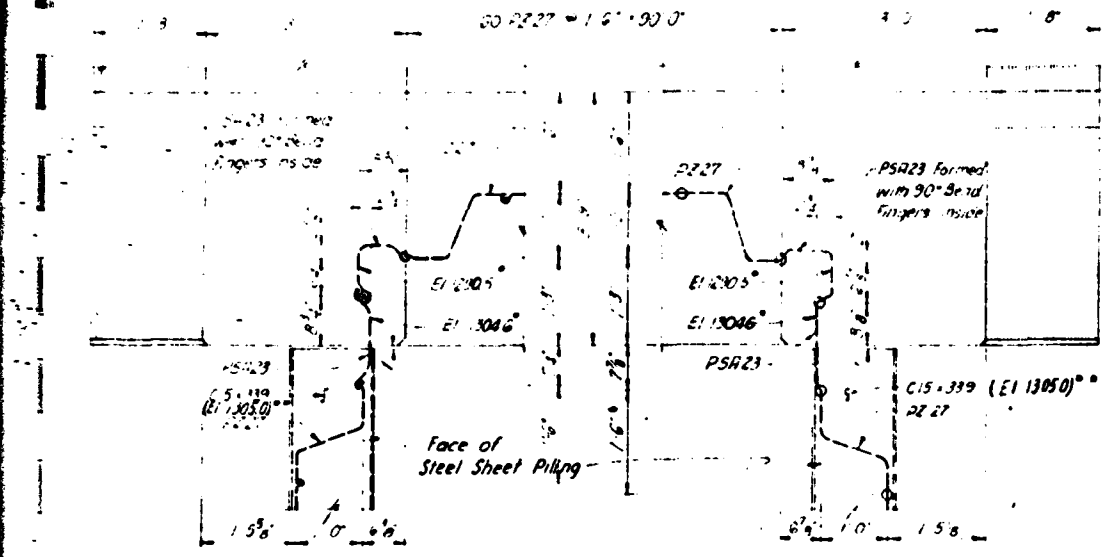
NO	DATE	DESCRIPTION	
1	2/14/80	As Built	JFM

PROJECT NAME
WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO D 125679

CLIENT
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE
CONCRETE WALL
REPAIR DETAILS

SCALE	DES. BY MD	DRAWING NO
NO SCALE	DWN. BY FZ	S-7
DATE ISSUED	CHD. BY KK	SHEET NO
6/30/78	DATE	13

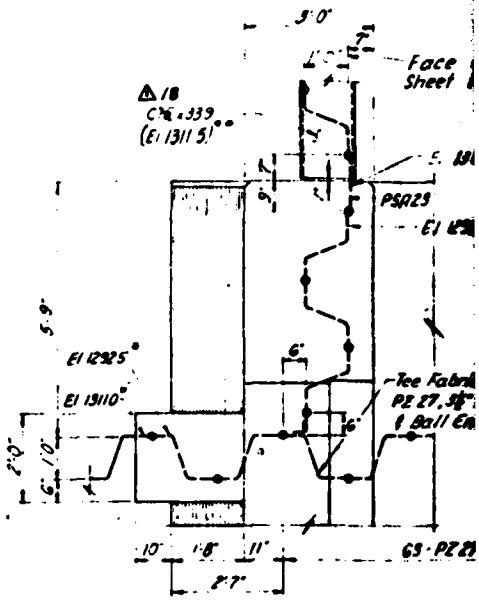


SOUTHEAST CORNER

NORTHEAST CORNER

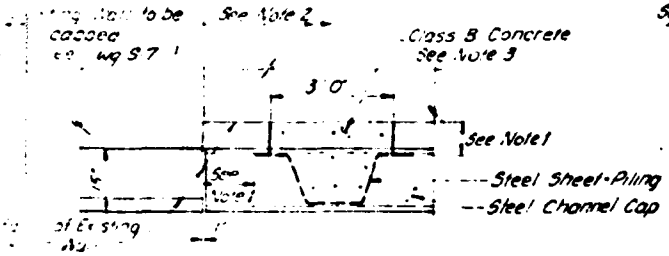
SHEET PILING PLAN

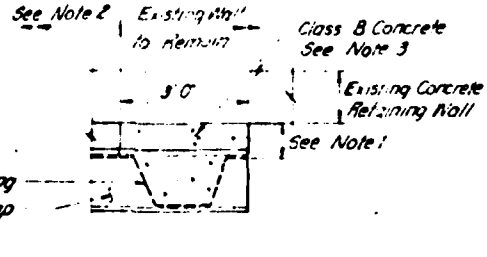
Note
* Top of Piling elevation
** Top of Cap elevation

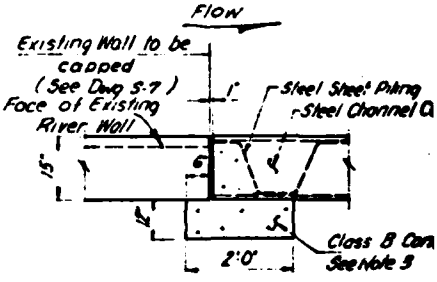


SOUTHWEST CORNER

SHEET PI







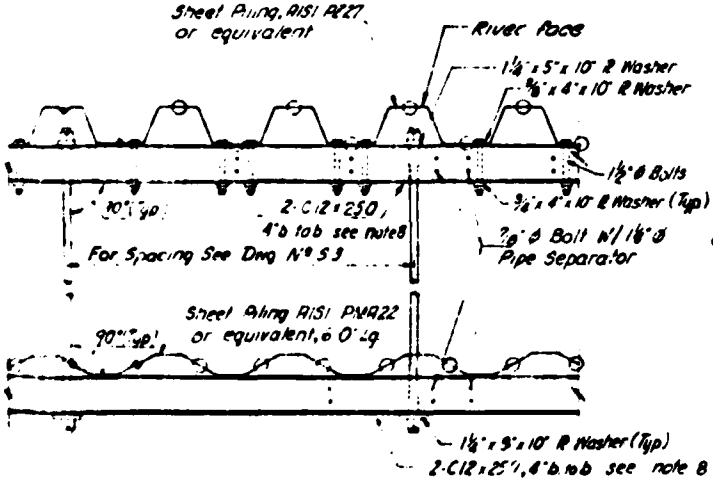
Flow

DETAIL A
NORTH UPSTREAM TIE-IN

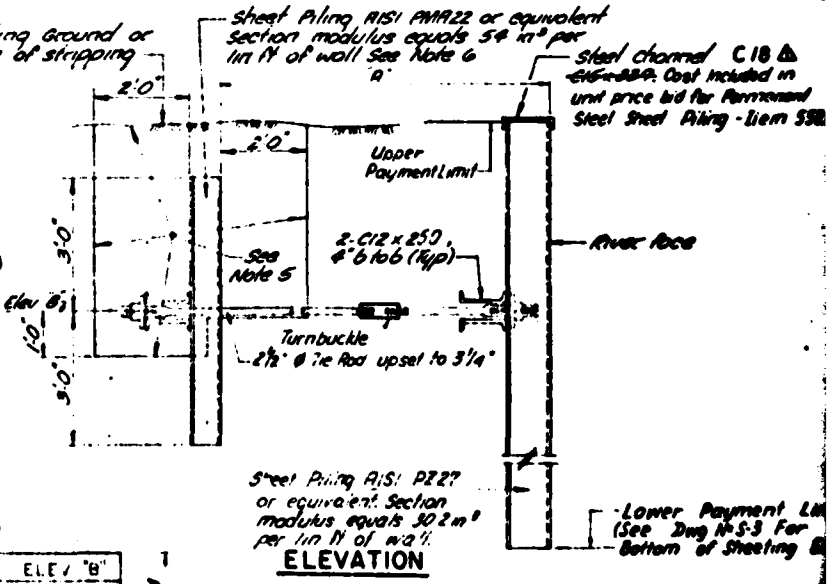
Flow

DETAIL B
NORTH DOWNSTREAM TIE-IN

DETAIL C
SOUTH UPSTREAM TIE-IN



PLAN

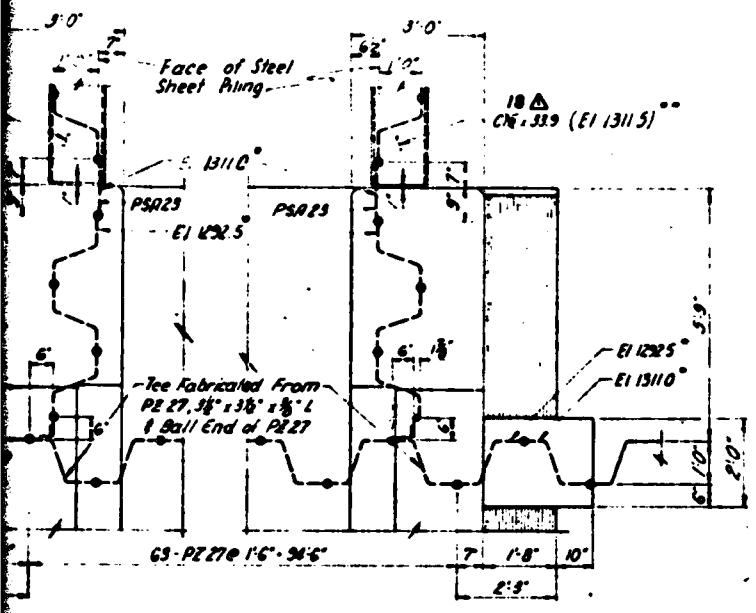


ELEVATION

TABLE "A"	LOCATION	A"	ELEV "B"
	1	52.0"	1301.0
	2	41.0	1300.0
	3	41.0"	1303.0
	4	41.0"	1303.0

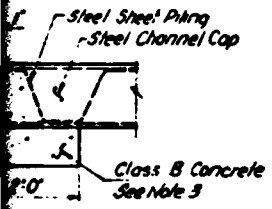
see note 4

For Locations See Dwg N° P1-1



CORNER NORTHWEST CORNER

SHEET PILING PLAN



L C
REAM TIE-IN

/ channel C18 △
Cost included in
nce bid for Permanent
Steel Piling - Item 552.02

er face

ower Payment Limit
e Dwg N° S-3 For
ttom of Sheeting Elevation)

NOTES:

1. Dimensions to be determined for Steel Sheeting to be driven as close as practical to existing retaining walls, subject to approval of Engineer.
2. Existing wall to be lowered as necessary to a point 4" below finished grade See Typical Section N° 8, Dwg TS-1.
3. Concrete to be paid under Item 555.02 Bottom of bulkhead pour to be at existing bottom of river. Top of pour to be level with adjacent existing concrete wall or concrete wall as lowered to clear grading.
4. Excavation for sheeting deadman shall be done in the presence of the engineer If bearing capacity of soil is not sufficient sheeting deadman embedment shall be AOB.E
5. Payment limits for sheeting deadman excavation, Item 206.02 Backfill with Item 203.07. Restore with 6" Item 304.05 or topsoil and seeding as appropriate
6. Cost of Sheeting deadmen, waler-, tie rods Turnbuckles, and accessories to be included in unit price bid for Permanent Steel Sheet Piling, Item 552.02.
7. See Dwg. PL-1 for location of Details A thru D.
8. Any splices in walers shall develop full moment capacity of 2-C12 x 25.0

ERDMAN ANTHONY ASSOCIATES
CONSULTING ENGINEERS & PLANNERS
ROCHESTER, N.Y. CAMP HILL, PA

DATE DATE

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REVISIONS

NO.	DATE	DESCRIPTION	BY
1	2/14/80	As Built	JFMc

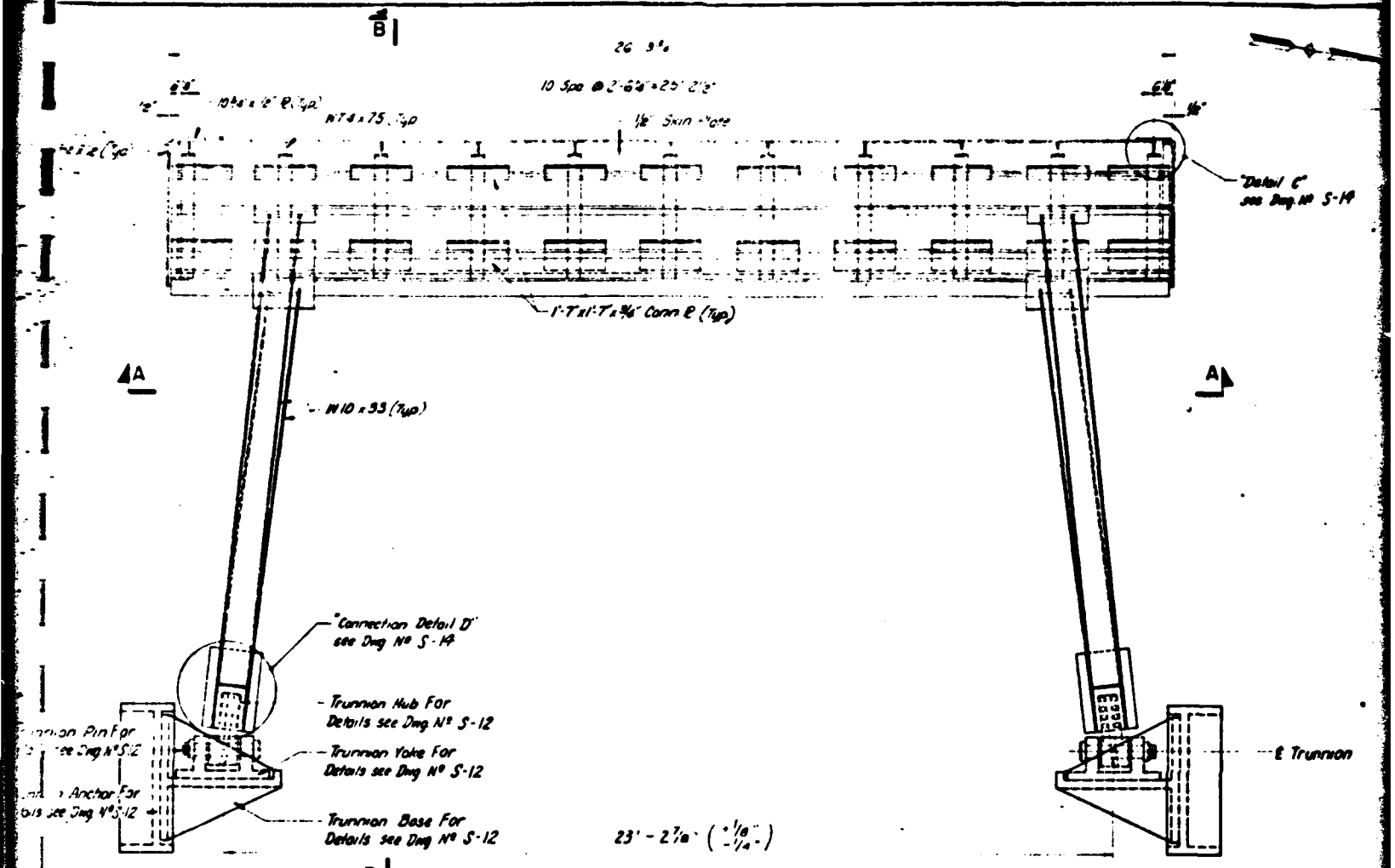
PROJECT NAME
WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO D 125679

CLIENT:
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

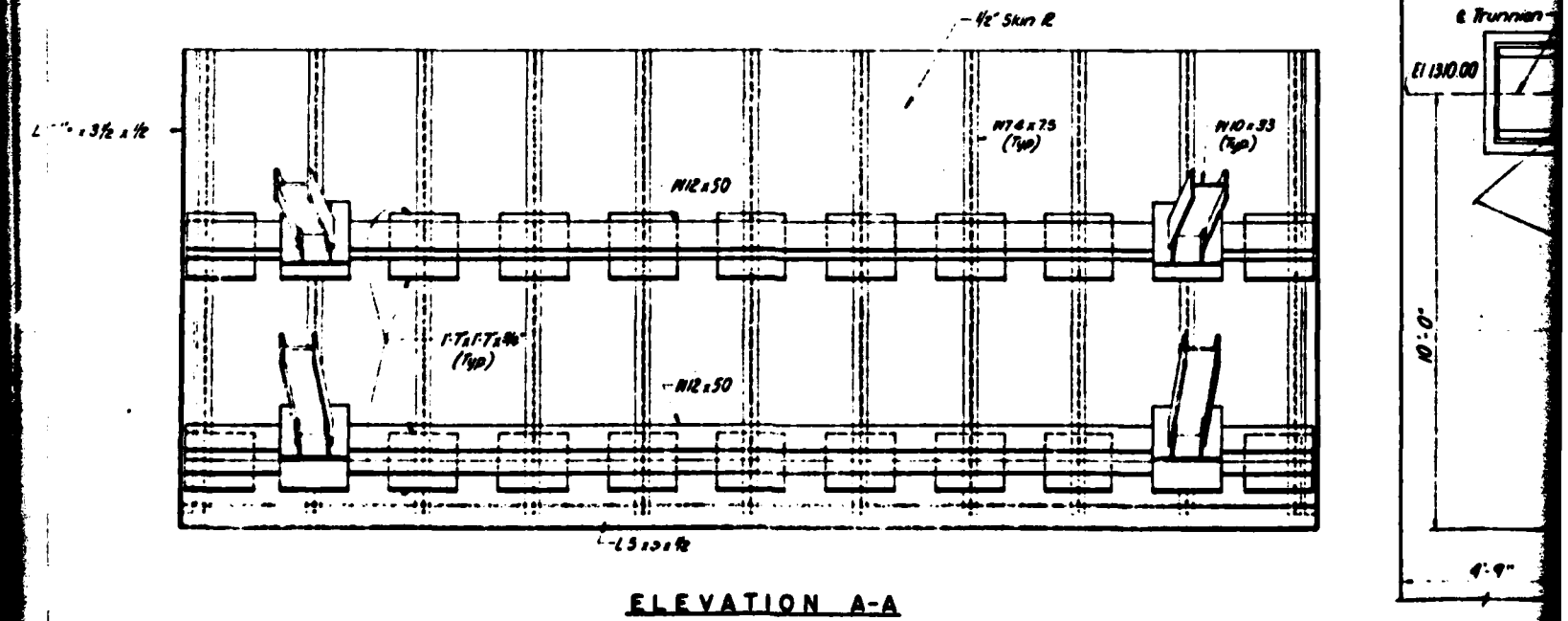
DRAWING TITLE
STEEL SHEET PILING DETAILS

NO SCALE	MD GP	S-9
DATE ISSUED 6/30/78	KK	

2



PLAN



ELEVATION A-A

N

"Detail C"
see Dwg. No. S-19

NOTE:

1) Payment for Tainter Gate included in Item 814

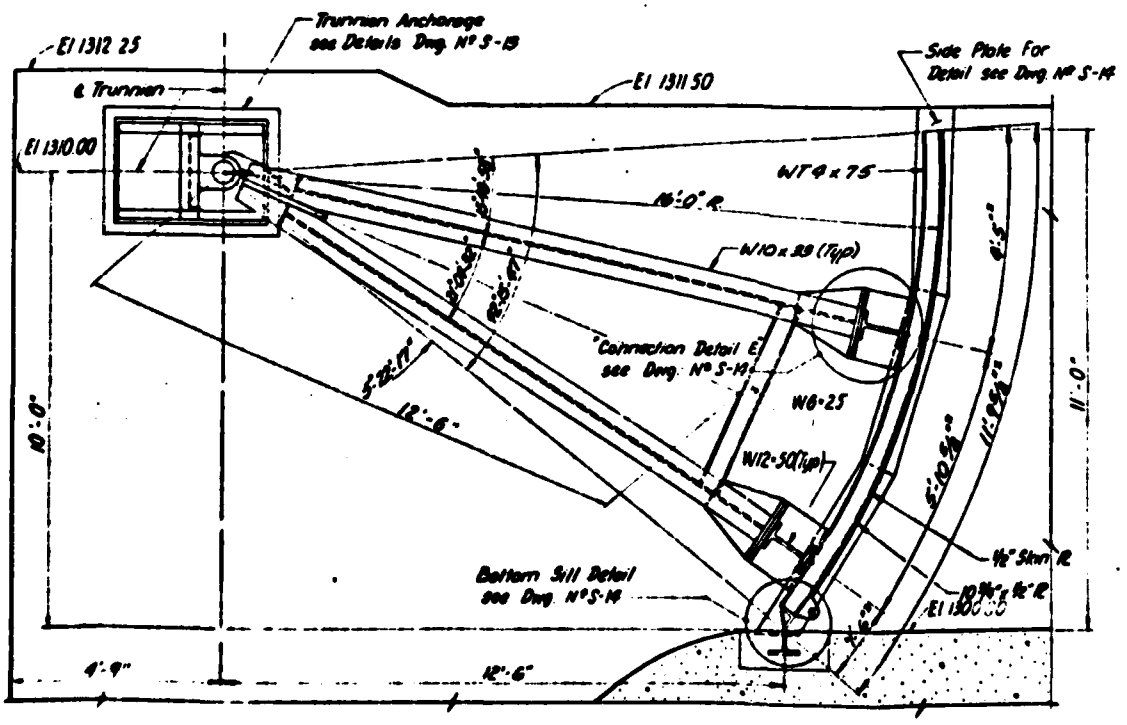
2) All Connections to be welded unless otherwise shown

3) Work this sheet with Drawings S-12 & S-13 & S-14

4) Dimensions suffixed by * are given along outer surface of skin plate @ R: 16'-0"

5) Under no circumstances should bottom edge of gate be raised above El. 1311.4.

℄ Trunnion



SECTION B-B

ERDMAN ANTHONY ASSOCIATES

CONSULTING ENGINEERS & PLANNERS

ROCHESTER, N.Y. CAMP HILL, PA

DATE DATE

NOTE:

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REVISIONS

NO.	DATE	DESCRIPTION	APP'D

No As Built Revisions

PROJECT NAME

WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO. D125679

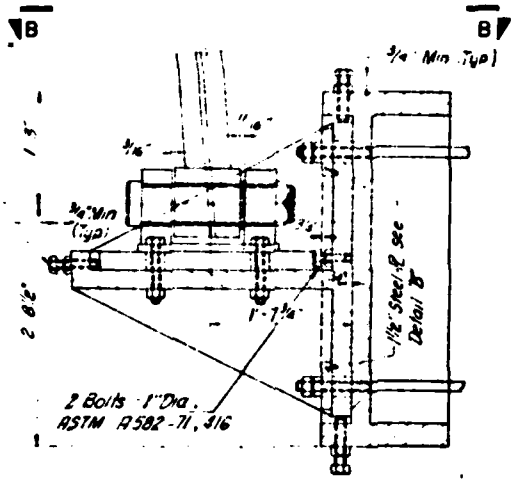
CLIENT

STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE

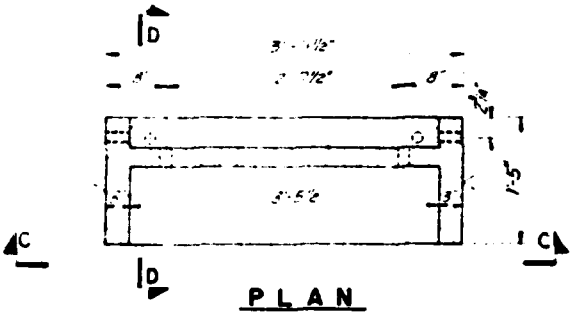
TAINTER GATE
GATE DETAILS

SCALE 1/2" = 1'-0"	DES'D AZ	DRAWING NO. S-11
DATE ISSUED 6/30/78	DRAWN RJ CHK'D KK	SHEET NO. 17 23



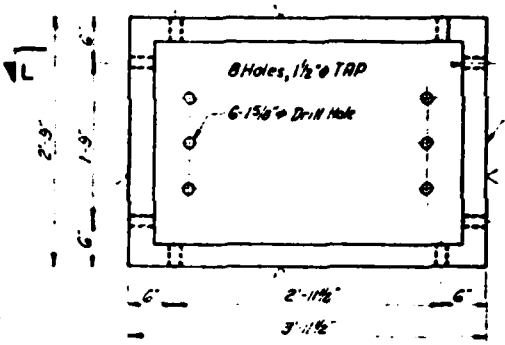
PLAN

SECTION K-K

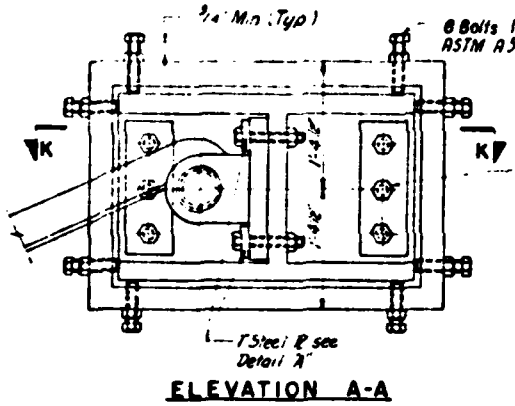


PLAN

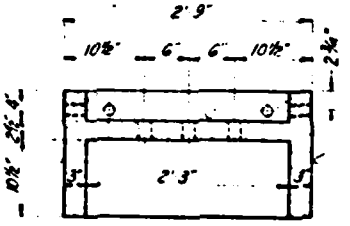
SECTION L-L



ELEVATION C-C



ELEVATION A-A

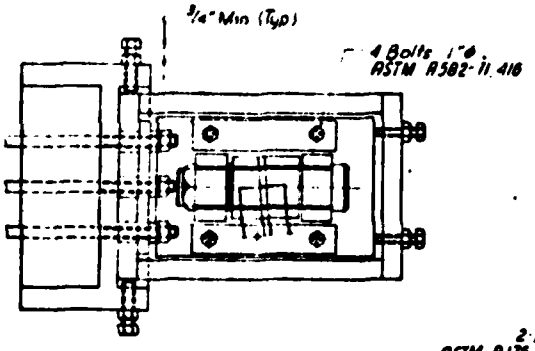


SECTION D-D

TRUNNION ANCHOR DETAILS

Scale 1"=1'-0"

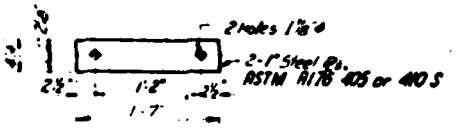
⚠ See Shop Drawings for ch... from cast steel to galva...



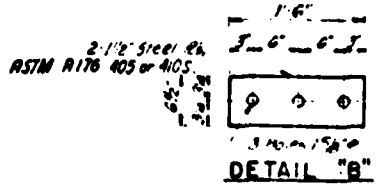
ELEVATION B-B

TRUNNION ASSEMBLY DETAILS

Scale 1"=1'-0"



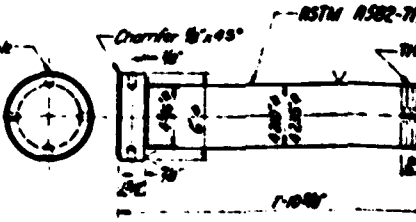
DETAIL "A"



DETAIL "B"

SHIM PLATES DETAILS

Scale 1"=1'-0"



TRUNNION PIN DETAIL

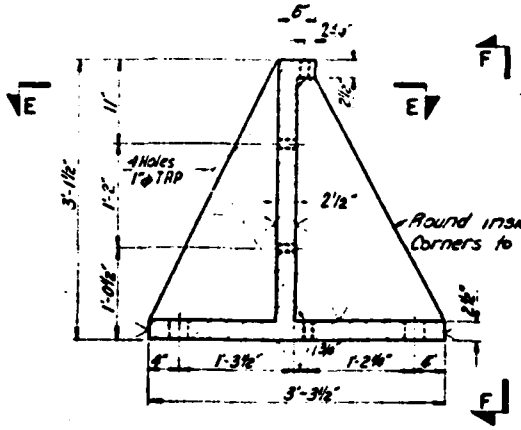
Not to Scale

Round Corn... to 2" Radius

TRU...

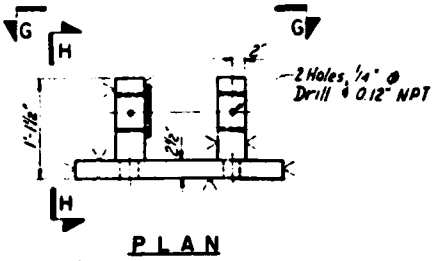
NOTE:

1. Payment for Trunnion Assembly included in Item 8.14.
2. For Notes this Sheet see Dwg N° S-14.

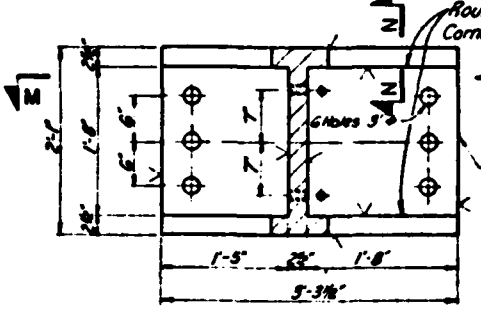


PLAN

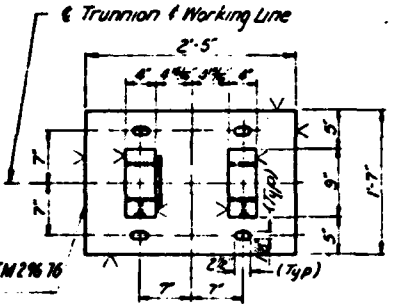
SECTION M M



PLAN

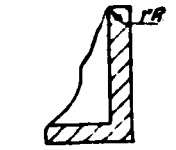


ELEVATION E-E

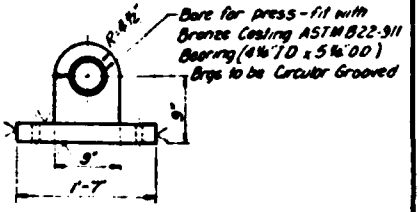


ELEVATION G-G

Shop Drawings for changes in trunnion assembly cast steel to galvanized steel



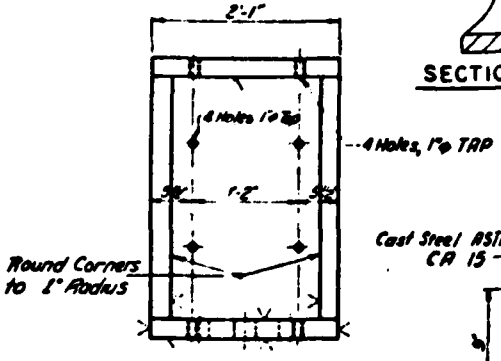
SECTION N-N



ELEVATION H-H

TRUNNION YOKE DETAILS

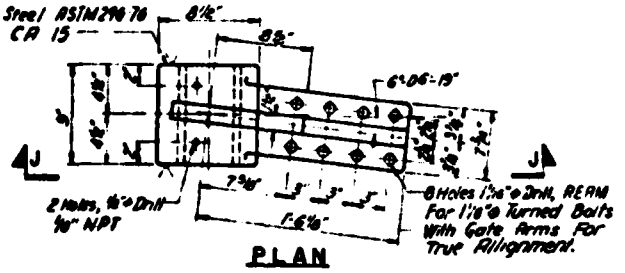
Scale: 1"=1'-0"



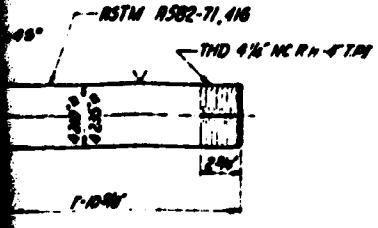
ELEVATION F-F

TRUNNION BASE DETAILS

Scale 1"=1'-0"

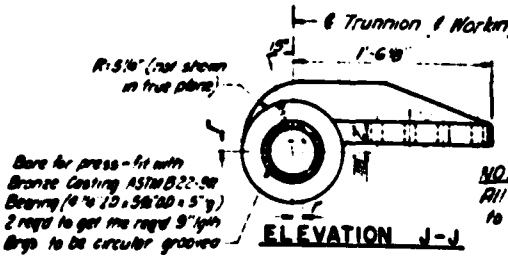


PLAN



PIN DETAIL

Not to Scale



ELEVATION J-J

NOTE:
All exposed edges are to have ⅛" Radius

TRUNNION HUB DETAILS

Scale 1½" = 1'-0"

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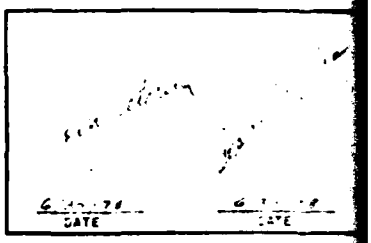
CONSULTING ENGINEERS & PLANNERS

ROCHESTER, N.Y. CAMP HILL, PA.



DATE DATE

NOTE:

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REVISIONS

NO.	DATE	DESCRIPTION	BY
1	2/14/80	As Built	JFM

PROJECT NAME

WARNER DAM REPLACEMENT

JAMESTOWN, NEW YORK

CONTRACT NO. D125679

CLIENT

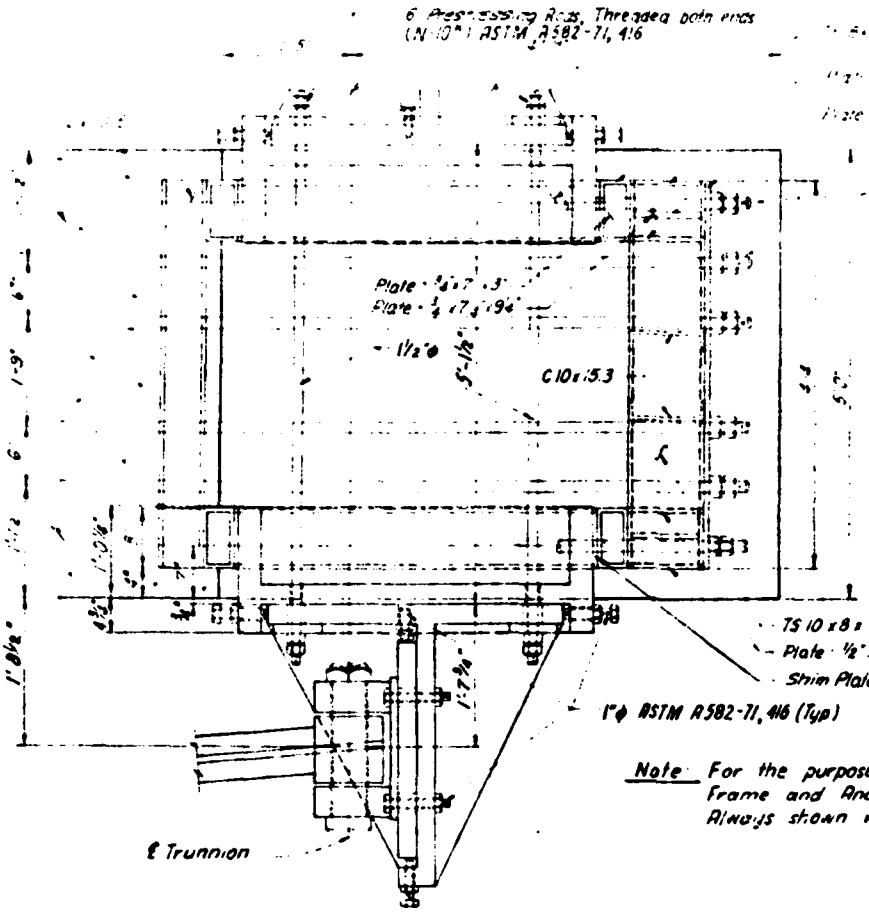
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE

TAINTER GATE

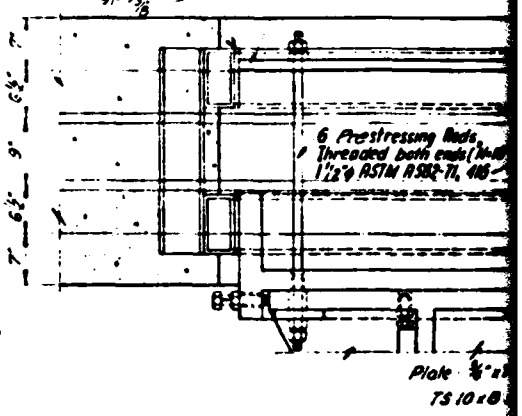
TRUNNION ASSEMBLY DETA

AS SHOWN	A.Z. V.R.	S-12
8/30/79	R.R.	



SECTION A-A - PIER

Scale 1"=1'0"

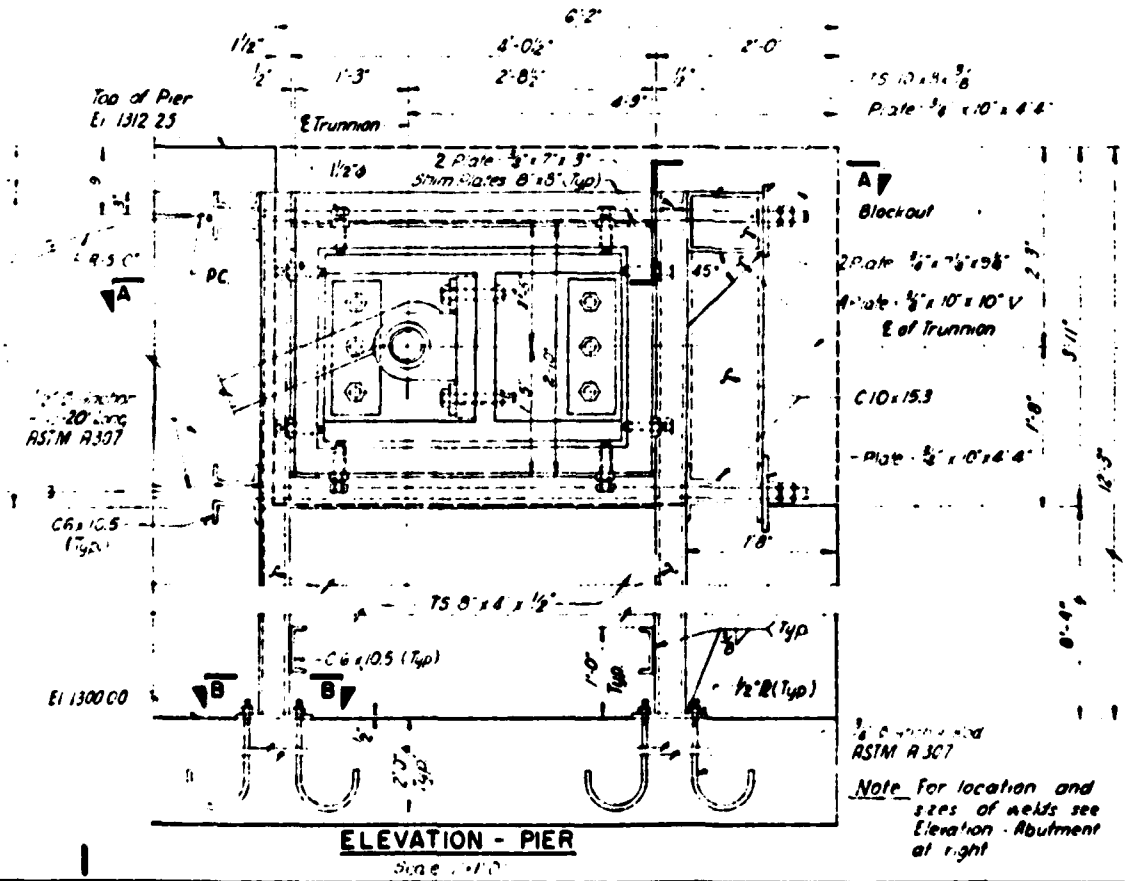


SECTION C-C - AB

Scale 1"=1'-0"

Note For the purpose of clarity the Frame and Anchor Rods are Always shown with Solid lines.

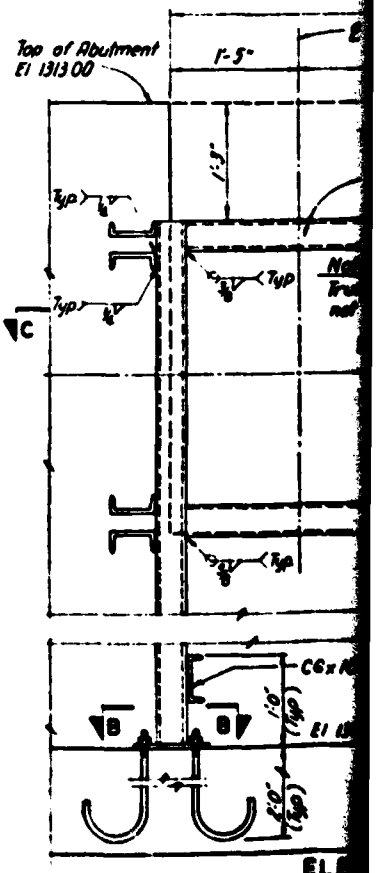
Δ See Shop Drawings for change f from cast steel to galvanized

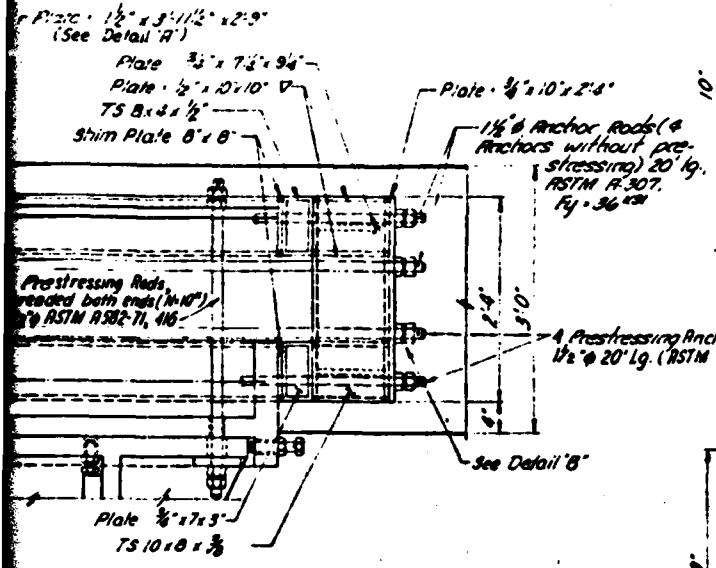


ELEVATION - PIER

Scale 1"=1'0"

Note For location and sizes of welds see Elevation - Abutment at right

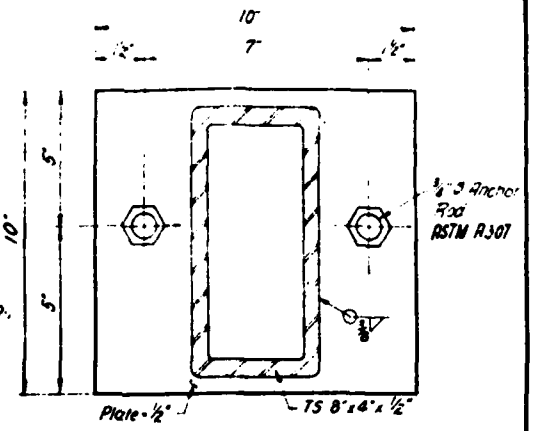




N C-C - ABUTMENT

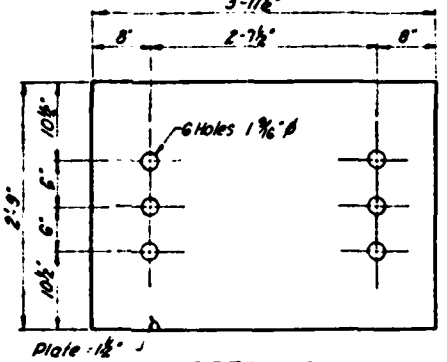
Scale 1"-1'-0"

...ings for change in trunnion assembly shall be galvanized steel.



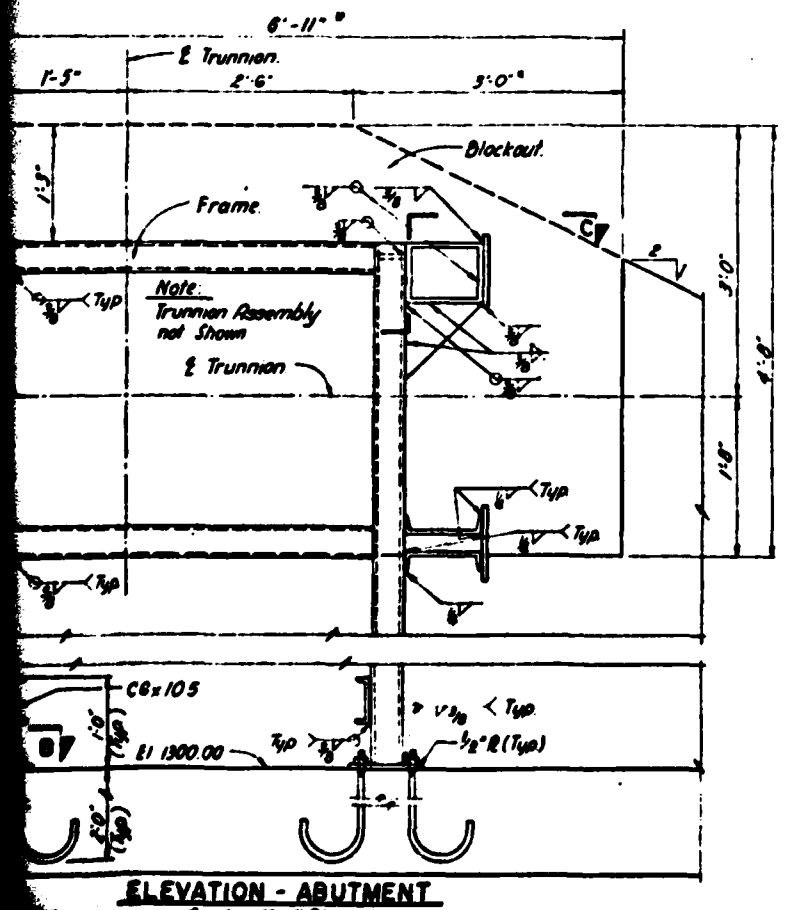
SECTION B-B

Scale ⅜"-1"



DETAIL A

Scale 1"-1'-0"



ELEVATION - ABUTMENT

Scale 1"-1'-0"

NOTE:

1) Payment for Trunnion Anchorage to be included under Item 656.01
2) All Structural Steel on this sheet shall be ASTM A-36 and paid for under Item 656.01
3) All Stainless Steel, Cast Steel and Anchor Bolts to be ASTM A307 and paid for under Item 814
4) All Connections are to be welded, except those shown as bolted
5) Frame to be prefabricated and completely assembled in shop before shipping to Construction Site for installation
6) During the installation of Trunion System into the frame (Before the concrete backfilling of the Blockout) place accurately by dimensions shown. Clearance space between frame and Trunnion to be filled by means of shim plates and field welded all together. Install 6 prestressing rods and reinforcing bars as shown on Pier and Abutment Drawings. Place Nuts on Prestress Rods and Prestress Manually. After recesses have been backfilled with concrete & reached 50% of f'c prestress each rod of both ends to 10 kips
7) Dimensions suffixed by * to be determined depending on prestressing jacks.

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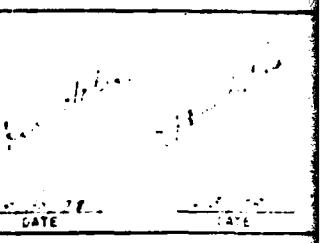
CONSULTING ENGINEERS & PLANNERS
ROCHESTER, N.Y. CAMP HILL, PA.



NOTE:
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REVISIONS

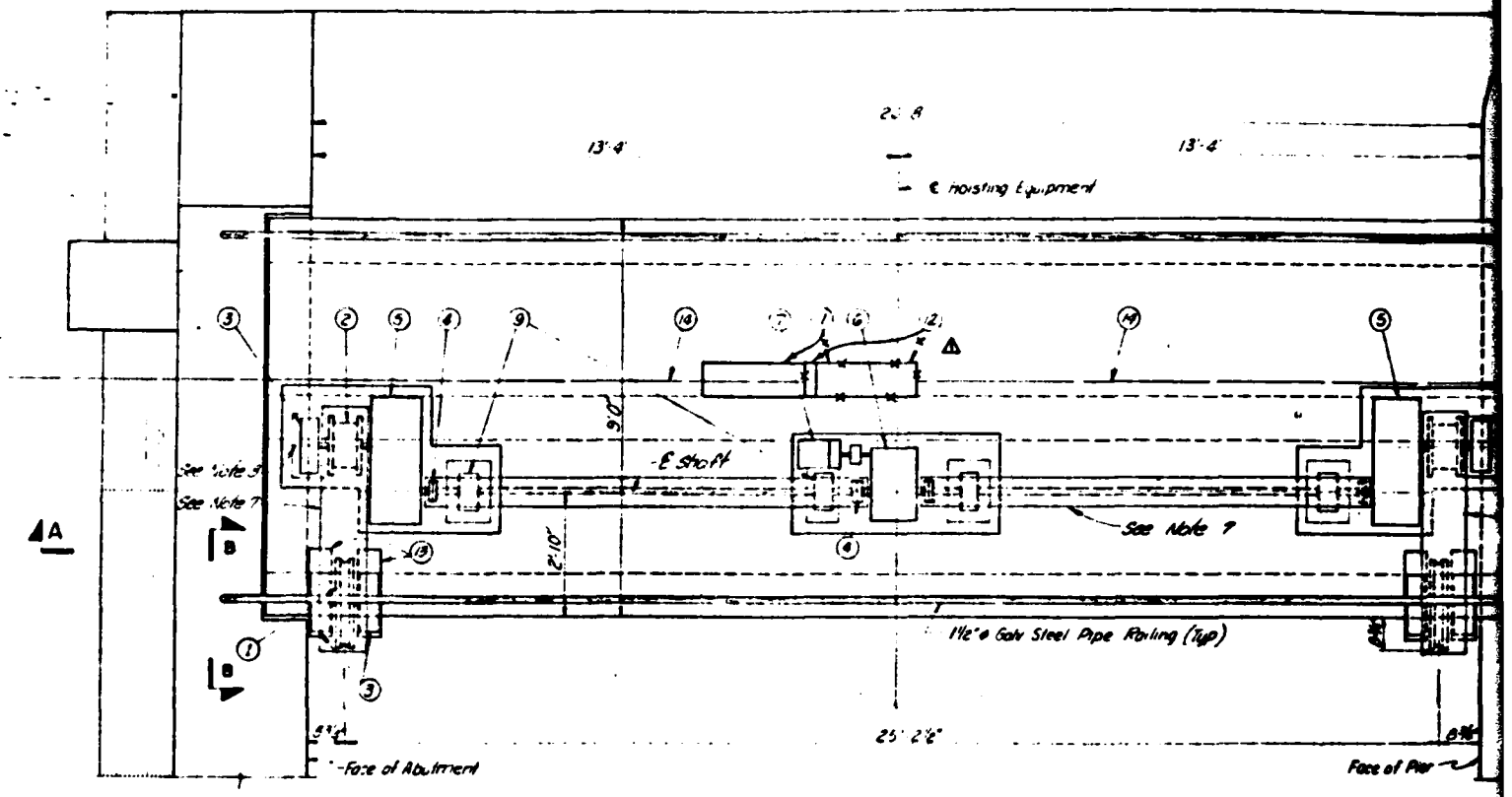
NO.	DATE	DESCRIPTION	BY
1	2/14/80	As Built	J.F.M.

PROJECT NAME
WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO. D125679

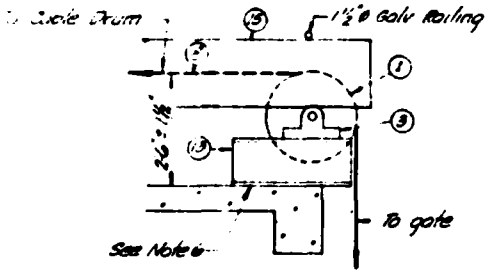
CLIENT
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE
TAINTER GATE
TRUNNION ANCHORAGE DETAIL

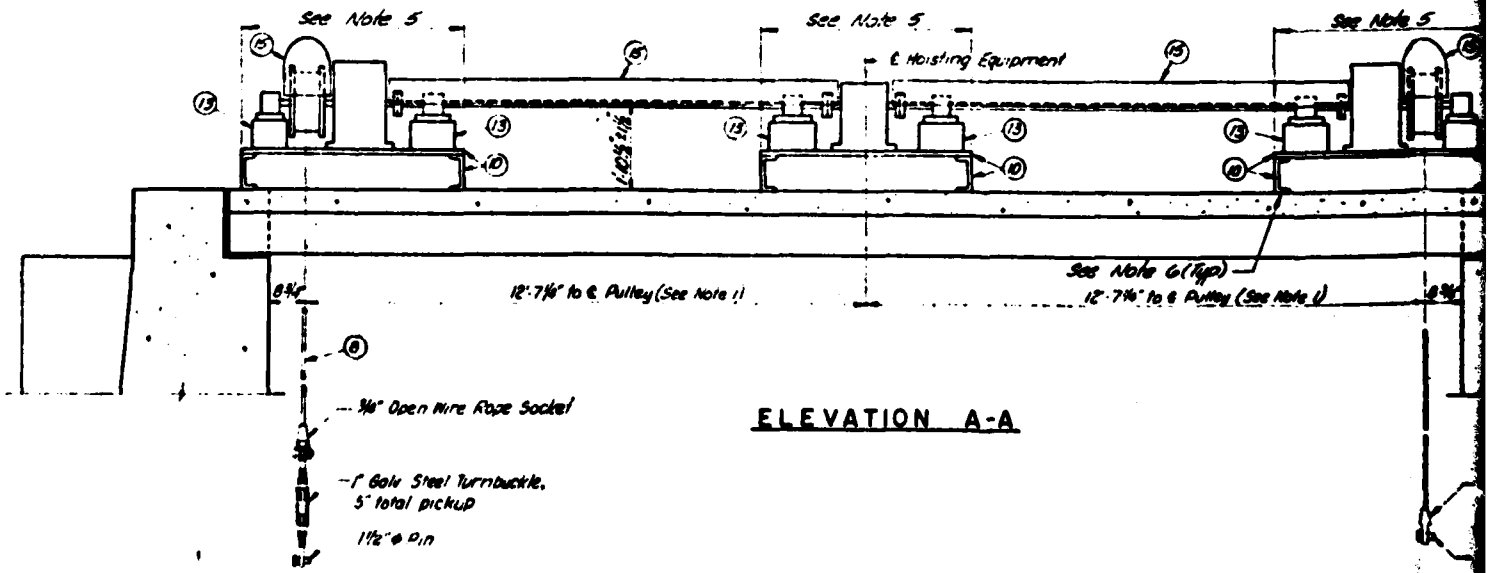
SCALE	DES. BY A.Z.	DRAWING NO.
AS NOTED	DRN. BY G.S.P.	S-13
DATE ISSUED	CHD. BY K.K.	SHEET NO.
6/30/78		



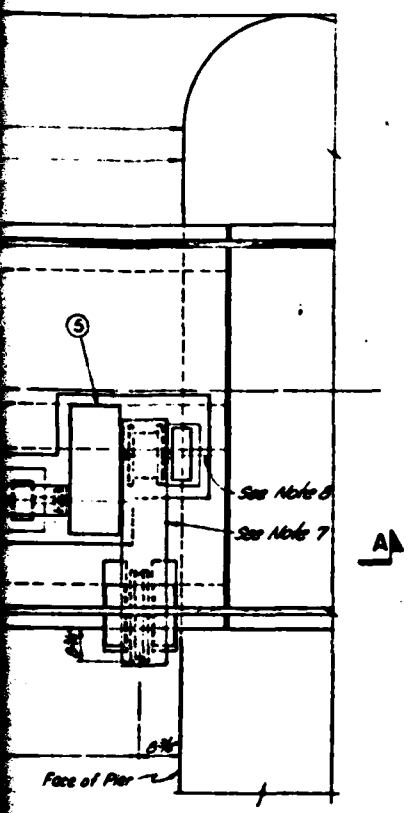
PLAN - GATE NO. 1



SECTION B-B
No Scale



ELEVATION A-A



NOTES:

1) Geometry of pulley and drum shall be such that wire rope makes angle of 90° to axis of pulley with gate raised half of the total cable movement required to the fully open position

2) The details for Gates 2 and 3 are the same as Gate 1, except as noted otherwise.

3) Refer to special specification, Item 815, for description and payment for Hoisting Equipment.

4) Refer to special specification, Item 818, for description and payment for Electric Equipment

5) Pillow blocks and reducer to be shop mounted and aligned on common base plate

6) Provide connection to deck with heavy duty Deco Adjustable Anchors as manufactured by Decatur Engineering Co Inc, Decatur, Illinois, or equivalent

7) Areas to be covered by ice shield

8) End of shaft to be drilled and tapped for standard ⅜"ø bolt to depth of 2".

⚠ See Shop Drawings for Revisions

EQUIPMENT LIST

① 24" Pitch dia Sheave - 4" dia shaft, ⅝" SS Cable. Sheaves to be provided with safety loop for cable.

② 15" Pitch dia Grooved Drum, 6 Wraps, 6" width

③ Pillow Block, 4 Bolt, Spherical Double Roller Brg 15⅝" x 5⅝" W

④ Flexible Coupling 3" x 6" dia, 2½" Shaft

⑤ Secondary Helical Gear Single Reduction Gear Reducer 7.56:1 Ratio, Parallel Shaft, Link Belt Series HS1200 67975 in Lb. Rated Output Capacity

⑥ Primary Gear Reducer, Double Worm Gear 2000:1 Ratio, Link Belt Model DNB 600, Rated 22975 In Lb Output

⑦ ¾ HP, T.E.F.C., 1750 RPM, 230V, 3 PH Drive Motor w/ Electric BC Brake Mechanism, Foot MTD

⑧ ¾" dia 6 x 37 Type 302 Stainless Steel Cable 44,800 Lb Rated Ultimate Strength

⑨ Pillow Block, Ball Brg, 2½" Shaft

⑩ Base plates and base plate supports to be galvanized ASTM A36 steel or ASTM A296 cast steel, size and height dependent on equipment selected

⑪ Main Control Panel (Gate 1)

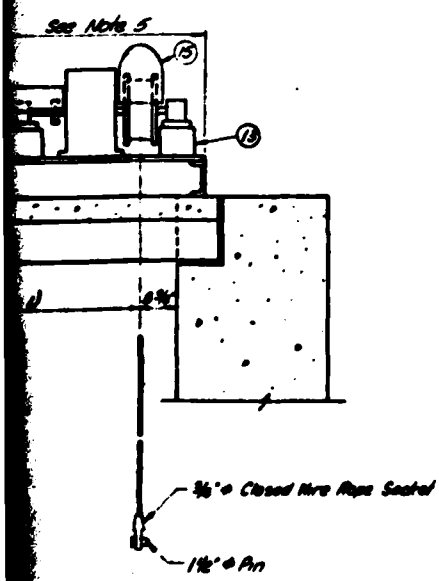
⑫ Gate Control Panel (Gates 2 & 3)

⑬ Pillow Block Supports, galv ASTM A36 Steel or ASTM A296 Cast Steel

⑭ See Electrical Detail sheets for conduit and wiring arrangement.

⑮ Ice shield, ¼ ga galv steel with galv anchoring hardware. Shields to be mounted a minimum of 2" from any moving part

Note: Sheave and Railing not shown on this view



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CONSULTING ENGINEERS & PLANNERS
ROCHESTER, N.Y. CAMP HILL, PA

DATE

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REVISIONS

NO	DATE	DESCRIPTION
1	2/14/80	As Built

PROJECT NAME
WARNER DAM REPLACEMENT
JAMESTOWN, NEW YORK
CONTRACT NO D125679

CLIENT
STATE OF NEW YORK
DEPARTMENT OF
ENVIRONMENTAL CONSERVATION
ALBANY, NEW YORK

DRAWING TITLE
HOISTING EQUIPMENT

1/2"=1'-0"	KA RJ TK	S-15
6/30/78		2