AD-A142 692 NATIONAL DAM INSPECTION PROGRAM MIANUS RESERVOIR DAM (CT 00050) CONNECTIC..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV SEP 78 1/2

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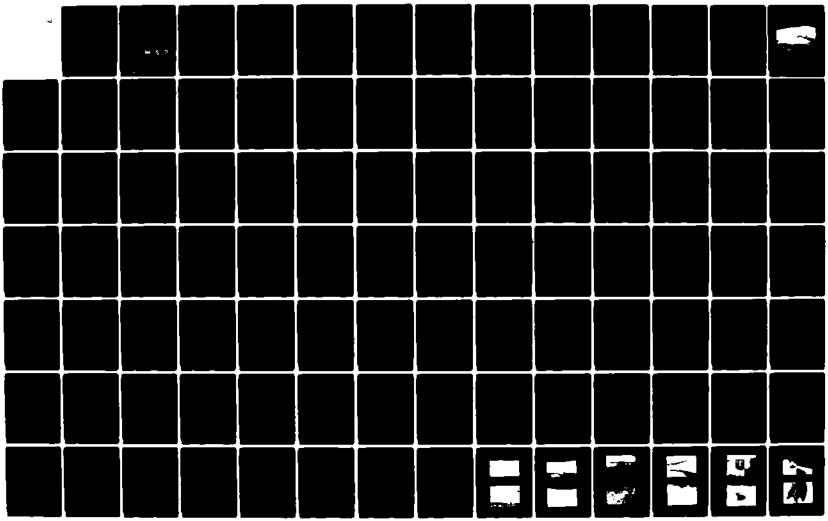

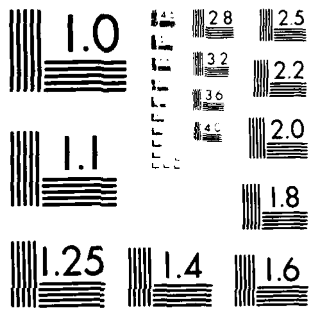

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AD-A142 692

CONNECTICUT WESTERN COASTAL AREA
STAMFORD, CONNECTICUT

MIANUS RESERVOIR DAM
CT 00050

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

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

SEPTEMBER 1978

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Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Conn. Western Coastal Area
Stamford, Conn.
Mianus Reservoir Dam

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

The Mianus Reservoir Dam consists of an earth embankment with a selected impervious fill core, drop inlet spillway, stilling basin, a wasteway having a fuse plug and p pumping facilities. The dam section is 950 ft. long with a maximum height of 75 ft. The top of the dam is 20 ft. wide The downstream side slopes of the earth embankment are 2 horizontal to 1 vertical. The upstream slope is 2-1/2 horizontal to 1 vertical. Riprap is in place on the upstream face. The morning glory-type spillway is 8 ft. in diameter.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:
NEDED

JUN 11 1979

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

I am forwarding to you a copy of the Mianus Reservoir Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, Connecticut-American Waterworks Company, Inc., 125 East Putnam Avenue, Greenwich, Connecticut 06830, ATTN: Mr. Joseph O. Yates, Jr., Manager.

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

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely yours,

JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

MIANUS RESERVOIR DAM

CT 00050

CONNECTICUT WESTERN COASTAL AREA

STAMFORD, CONNECTICUT

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

PHASE I REPORT

NATIONAL DAM SAFETY PROGRAM

Name of Dam: MIANUS RESERVOIR DAM

State Located: Connecticut

County Located: Fairfield County

Stream: Mianus River

Date of Inspection 3 AUGUST 1978

BRIEF ASSESSMENT

The Mianus Reservoir Dam consists of an earth embankment with a selected impervious fill core, drop inlet spillway, stilling basin, a wasteway having a fuse plug and pumping facilities. The dam section is 950 feet long with a maximum height of 75 feet. The top of the dam is 20 feet wide. The downstream side slopes of the earth embankment are 2 horizontal to 1 vertical. The upstream slope is 2-1/2 horizontal to 1 vertical. Riprap is in place on the upstream face. The morning glory-type spillway is 8 feet in diameter.

Based on the visual inspection, records available, and past operational performance, the dam is judged to be in good condition.

The project will not pass the test flood without overtopping the dam, and therefore, the total spillway capacity is inadequate. The spillway capacity is judged seriously inadequate since the project will not pass one-half the test flood without overtopping the dam (35 percent). The test flood will overtop the dam by approximately 2.9 feet.

It is recommended that the owner ensure that the materials contained in the fuse plug are excavated and replaced, without tamping. Additionally, the scour hole at the spillway should be repaired. The owner should control trespassing on the dam, particularly near the junction of the left abutment and the downstream slope where trespassing is extensive at the present time.

Round-the-clock surveillance should be provided during periods of high precipitation. The owner should develop a formal warning system. An operational procedure to follow in the event of an emergency should also be adopted.

Recommendations and remedial measures described should be implemented by the owner within 2 years after receipt of this Phase I Inspection Report.

S. Giavara

S. Giavara, P.E.
Principal

Registered, CT 7634

This Phase I Inspection Report on Mianus Reservoir Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Charles G. Tiersch

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

Fred J. Ravens Jr.

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

Saul Cooper

SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

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

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

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reaonably 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 aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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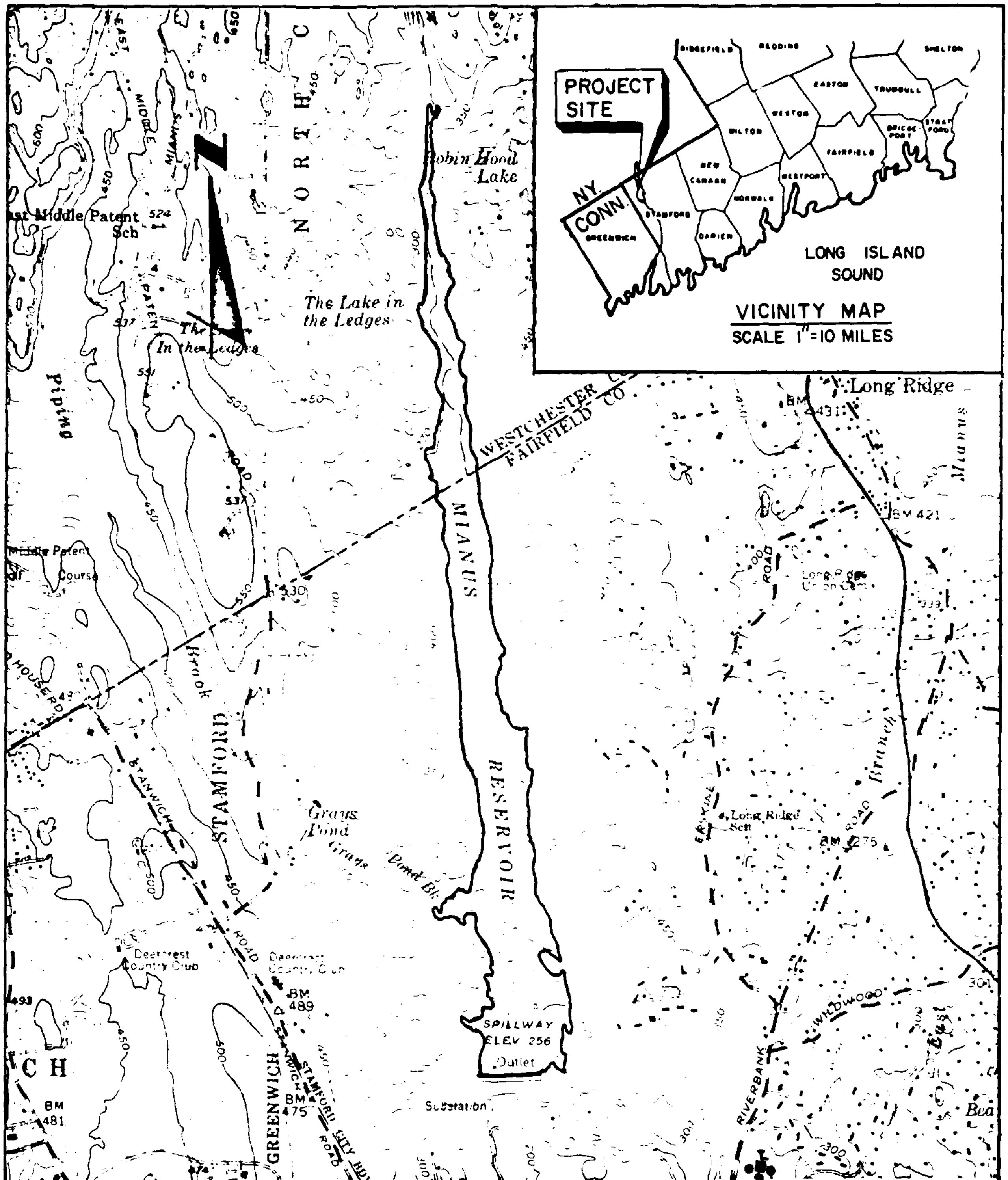
MIANUS RESERVOIR DAM



SCALE IN FEET

2000 0 1000 2000

LOCATION MAP

MIANUS RESERVOIR DAM

STAMFORD, CONNECTICUT

PHASE I INSPECTION REPORT
MIANUS RESERVOIR DAM CT 00050

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL:

a. Authority. Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection through the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Flaherty Giavara Associates, P.C. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed was issued to Flaherty Giavara Associates, P.C. under a letter of 25 April 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0309 has been assigned by the Corps of Engineers for this work.

b. Purpose.

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

2) Encourage and assist the States to initiate quickly effective dam safety programs for non-federal dams.

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

1.2 DESCRIPTION OF PROJECT:

a. Description of Dam and Appurtenances. This dam popularly known as the Samuel J. Bargh Reservoir Dam consists of an earth embankment with a selected impervious fill core, drop inlet spillway, stilling basin, a wasteway having a fuse plug and pumping facilities, built in 1955. The dam section is 950 feet long with a maximum height of 75 feet. The top of the dam is 20 feet wide. The downstream side slopes of the earth embankment are 2 horizontal to 1 vertical. The upstream slope is 2-1/2 horizontal to 1 vertical. Riprap is in place on the upstream face. The morning glory-type spillway is 8 feet in diameter.

b. Location. The dam is located approximately 2-1/2 miles north of the Merritt Parkway in the Town of Stamford on Mianus River within the Connecticut western coastal area.

c. Size Classification. The applicable guideline indicates that for an intermediate category the storage in acre-feet for the impoundment must be greater than or equal to 1,000 and less than 50,000. The size classification may be determined by either storage or height, whichever gives the larger size category. Based on the storage capacity of the dam, the size classification is intermediate. The top of dam storage for Mianus Reservoir Dam is 7,515 acre-feet.

d. Hazard Classification. The dam is classified as having a high hazard potential. This classification is based on the 10 or more houses situated along the narrow valley which would be affected by a dam failure flood wave.

e. Ownership. Mianus Reservoir Dam is owned by the Connecticut-American Waterworks Company, Inc. - Greenwich District.

f. Purpose of Dam. The dam was constructed to form an impounding reservoir. The reservoir forms part of the water company's supply and distribution system, providing potable water to the residents of Greenwich.

g. Design and Construction History. The dam was completed in 1955. The American Water Works Service Company, Inc. designed the dam and its appurtenances under the direction of Howard J. Carlock. The dam was constructed by Merritt-Chapman and Scott Corporation, New York, New York. Any subsequent modifications are unknown.

h. Normal Operating Procedures. A 20" lock joint pipe transports water from Brush Dam to the stilling basin at Mianus Reservoir from which it flows downstream to the Mianus Filter Plant. Just upstream of the stilling basin, pumping facilities withdraw a portion of the water from the 20" lock joint pipe and pumps water to Rockwood Lake.

1.3 PERTINENT DATA:

a. Drainage Area - 18.3 sq. miles

b. Discharge at Dam Site -

Maximum Known Flood	Unknown
Warm Water Outlet	Not Available
Div. Tunnel Low Pool Outlet	Not Available
Diversion Tunnel Outlet	Not Available
Gated Spillway	None
Ungated Spillway at Max. Pool	2,200 CFS @ 1 Ft. freeboard
Total Spillway Cap. at Max. Pool	3,000 CFS @ no freeboard
Emergency Wasteway without Fuse Plug	2,900 CFS @ 1 Ft. freeboard
Emergency Wasteway without Fuse Plug	3,900 CFS @ no freeboard

c. Elevation (above M.S.L.) -

Top of Dam	262
Max. Design Pool	Not Available
Full Flood Control Pool	Not Available
Recreation Pool	Not Available
Spillway Crest Ungated	252
Upstream Portal Invert. Div. Tunnel	188
Downstream Portal Invert. Div. Tunnel	188
Streambed at Centerline of Dam	183
Maximum Tailwater	Unknown

d. Reservoir -

Length of Max. Pool	13,500 feet
Length of Recreation Pool	Not Applicable
Length of Flood Control Pool	Not Applicable

e. Storage -

Recreation Pool	Not Applicable
Flood Control Pool	Not Applicable
Design Surcharge	Not Applicable
Top of Dam	7,515 Acre-Feet

f. Reservoir Surface (acres) -

Top of Dam	Not Available
Max. Pool	Not Available
Flood Control Pool	Not Applicable
Recreation Pool	Not Applicable
Spillway Crest	230

g. Dam -

Type:	Earth embankment, concrete core
Length:	950 feet
Height:	75 feet
Top width:	20 feet

Side Slopes:	Downstream:	1 vertical to 2 horizontal
	Upstream:	1 vertical to 2-1/2 horizontal
Zoning:	Selected impervious fill core	
Impervious core:	Selected fill	
Grout Curtain:	Unknown	

h. <u>Diversion and Regulating Tunnel</u> -

Type:	Cast iron pipe
Length:	500 feet
Diameter:	30 inches
Access:	From stilling basin
Regulation:	Butterfly valve

i. <u>Spillway</u> -

Type:	8' diameter conc. morning glory
Length of Weir:	25 feet
Crest Elevation:	252
Gates:	Ungated
Upstream Channel:	Reservoir
Downstream Channel:	Natural Streambed
Spillway is founded on:	Rock

j. <u>Regulating Outlets</u> -

Gates:	None
Conduits:	30" diameter cast iron pipe to stilling basin

SECTION 2 - ENGINEERING DATA

2.1 DESIGN:

The design of the Dam was made by American Waterworks Service Company, Inc., Philadelphia, Pennsylvania. The plans were prepared by Howard C. Carlock, P.E., Connecticut Registration Number 3286.

The principal engineering data available are:

a. Project 18 - "Contract Documents for the Construction of a Dam and Impounding Reservoir on the Mianus River for the Greenwich Water Company, Greenwich, Connecticut," January, 1956.

b. Project 18 - Contract Drawings Nos. 36-268-1 through 36-268-8 (see Appendix VI).

c. Greer and McClelland, "Investigation and Design of the Mianus River Dam, Stamford, Connecticut," November, 1954.

d. Haley and Aldrich, "Report on Stability and Seepage Analysis, Mianus River Dam, Stamford, Connecticut," April, 1959 (see Appendix V).

e. Woodward-Clyde-Sherard and Associates, "Field and Laboratory Investigation, S. J. Bargh Reservoir (Mianus River) Dam, Stamford, Connecticut," January 19, 1961.

f. Haley & Aldrich, "Letter Report - S. J. Bargh Reservoir (Mianus River) Dam," July 6, 1961.

g. Observation well readings for five wells installed along the downstream slope for the following dates: July 8, 1965, July 1, 1966, November 18, 1968, and August 3, 1978.

2.2 CONSTRUCTION:

No information is available concerning the foundation preparation or embankment construction.

2.3 OPERATION:

No formal operation records are available.

2.4 EVALUATION:

a. Availability. The information available concerning the embankment consists of a design cross section and the identification of the embankment materials as "impervious core wall" and "random fill." Limited engineering data is available concerning the properties of the random fill to a depth of 5 feet. Six test pits were excavated on October 26, 1960, and the field results are reported in Reference e, Subsection 2.1. No additional engineering data are available concerning the properties of other materials in the dam. No information is available about the foundation materials encountered during the construction of the embankment sections.

b. Adequacy. The available data are not sufficient to evaluate the soils in the core and shells and in the foundation of the dam. The evaluation must be based primarily on the results of the visual inspection. The information available is considered adequate for the purposes of a Phase I investigation.

No conflicts have been noted between the available data and the observations made during the inspection. In general, there is no reason to question the validity of the available data.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS:

a. General. The dam appears to be in good condition and well maintained. No seepage was observed. Vertical and horizontal alignment of the dam was good. No lateral movement or settlment of the crest was noted. Vehicular paths and footpaths exist at several locations on the downstream slope. Some local deterioration of the riprap has occurred.

b. Dam. The upstream slope is covered with riprap from below the water surface to the crest of the dam. The riprap was in good condition except for some surface erosion and deterioration and small windows exposed through the riprap.

The crushed stone which covers the majority of the crest is generally in good condition and well maintained.

Several bike and footpaths were located on the downstream slope from the toe to the crest of the dam. Vehicular traffic has worn a path at the junction between the left abutment and the downstream face of the dam. Thd downstream slope was covered with grass and was generally well maintained. No animal holes were located on the downstream slope at the time of inspection.

The groundwater elevations obtained on August 3, 1978 in the five observation wells in the downstream embankment, located as shown on Figure 4, Appendix B, are indicated below:

GROUNDWATER OBSERVATIONS

PIPE NO.	DEPTH TO WATER IN PIPE	ELEVATION OF WATER IN PIPE
1	46.0	216.6
2	34.1	204.1
3	44.7	219.3
4	29.6	207.9
5	14.0	194.6

The cover of the drainage manhole was removed and a reading obtained from the V notch weir. Water was observed to be flowing underneath the weir into the drainage pipe. The flowing water appeared clear and free of suspended solids. No seepage was observed at the toe or abutments of the embankment. In the vicinity of Station 6+50, approximately 82 feet south of the toe, a hole was located which was approximately 3 feet long, 12 inches wide, and 8 feet deep. There appeared to be 3 feet of water in the bottom of this feature.

To the east of the dam exists an emergency spillway channel which contains a "fuse plug." The fuse plug is approximately 90 feet long, 5 feet high, and 5 feet wide at the crest. The pavement appeared compact as a result of foot and vehicular traffic. The material excavated from a small test pit on the surface of the fuse plug was a gravelly silty coarse to fine sand.

c. Appurtenant Structures.

1) Inlet Spillway - The exterior was in excellent condition. The interior was in generally good condition with some spalling noted near the bottom. The base of the drop structure according to the plans is founded directly on bedrock. The inspection of this feature indicated that an irregular scour hole approximately 5 feet deep has developed. Water is apparently flowing from this area into the outlet pipe at a steady rate.

2) Spillway Conduit - The interior of this 8 foot diameter conduit was inspected and found to be in generally good condition. Some hairline cracking was noted at the crown, with efflorescence and moisture also observed. The two sections of pipe nearest the inlet structure were found to have little or no mortar at the joints. No indication of spalling or deterioration of the concrete surfaces were noted. Alignment of the joints was good, there is an apparent slight sag in the pipe profile, about where the pipe passes beneath the dam crest.

3) Stilling Basin - The concrete endwall and wingwalls are in excellent condition. The bottom of the apron was not clearly visible due to the depth of water. The contacts between the embankment and the outlet structures were in good condition.

4) Blow-Off - The valve for the 30-inch blow-off pipe was easily opened by one man. The flow from the blow-off was clear and free of debris. Upon closing the valve, there was no leakage observed. The end of the blow-off pipe was in good condition.

d. Reservoir Area. The reservoir perimeter has well vegetated banks at moderate to steep slopes. There was no evidence of slides or sloughing. The depth of sediment and rate of accumulation in the reservoir is unknown.

e. Downstream Channel.

1) Spillway Channel (Mianus River) - The open channel downstream from the stilling basin was in excellent condition. It has large stone riprap forming stable banks, and there were no deposits of debris or sediment. The bottom and banks were nearly free of vegetation.

2) Wasteway Channel - This channel, excavated into earth and a limited area of rock, is overgrown with weeds and underbrush, approximately four feet high. Where visible, the sides and bottom were of stable earth with no evidence of erosion.

3.2 EVALUATION:

Based on the visual inspection, the dam appears to be in good condition. Trespassing has resulted in a loss of vegetation at several locations on the downstream slope and at the intersection with the left abutment from the toe to the crest of the downstream slope. Trespassing should be controlled, since it could lead to unacceptable long-term erosion. Brush and vegetation are growing at many locations on the riprapped slopes which could lead to development of erosion channel. For this reason, any vegetation in the riprap should be removed. Trespassing and vehicular traffic has resulted in compaction of soils contained in the fuse plug. Trespassing should be controlled, since it could lead to an unacceptable condition where the fuse plug would not erode as designed under high water.

The scoured area beneath the spillway inlet could lead to settlement of the spillway and outlet conduit.

The wasteway channel (downstream of fuse plug) is overgrown, reducing flow-carrying capacity.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES:

The Mianus Reservoir has a 2,450 million gallon capacity when full. It supplies water to Rockwood Lake and downstream to the Mianus Filter Plant. At Bargh there is a fuse plug for protection. When Bargh is full and a storm is coming, the blow-off (butterfly valve) is opened as additional protection.

4.2 MAINTENANCE OF DAM:

The dam is well maintained with a regular program of grass mowing and general maintenance in effect.

4.3 MAINTENANCE OF OPERATING FACILITIES:

The regulating gates and valves were tested and appear to be in mechanically good operating condition and are completely functional.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT:

There was no warning system of any kind in effect at the time of the inspection. The Connecticut-American Waterworks Company is currently developing procedures which will provide for surveillance during peak flow conditions and a warning system.

4.5 EVALUATION:

The Mianus Reservoir Dam, which is over 20 years old, is very well operated and maintained. Although not designed for rapid drawdown, it should be noted that if the need should arise, drawdown could be effected only through the operational procedure of opening the 30-inch blow-off. Therefore, this valve should be periodically exercised to insure proper functioning.

SECTION 5 - HYDRAULICS/HYDROLOGY

5.1 EVALUATION OF FEATURES:

a. Design Data. There is no available information on the hydraulic design criteria for this dam and appurtenances. Under established criteria (OCE Guidelines) the recommended spillway design flood for the size (intermediate) and hazard potential (high) classification is the probable maximum flood (PMF). The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

As estimate of the magnitude of the PMF at the site is based on an analysis of several sets of regional flood frequency data as presented in Appendix II.

As a conservative approach to the investigation, the more critical flood hydrograph was used. A peak inflow rate of 28,000 CFS was used as the test flood for determining spillway adequacy.

A stage-discharge relationship was calculated for both the spillway and the emergency wasteway. The calculations for the wasteway assumes that the fuse plug has been fully eroded to Elevation 252.

STAGE - DISCHARGE RATING

Stage Elevation	Q CFS Spillway	Emergency Wasteway	Total
252	0	0	0
253	120	270	390
254	330	760	1,090
255	550	1,400	1,950
256	720	2,160	2,880
257	840	3,020	3,860
258	940	3,970	4,910
259	1,040	5,000	6,040
260	1,090	6,110	7,200
261	1,160	7,290	8,450
262	1,220	8,540	9,760

The maximum spillway capacity, with no freeboard, is less than the peak discharge rate of the test flood (compare 9,760 CFS with 28,000 CFS). In order to determine the effect of the reservoir storage capacity, a hydrograph of the test flood was routed through the reservoir.

The hydrograph was formed by assuming the test flood had a duration of 24 hours, with the peak of 28,000 CFS occurring at 8 hours from the beginning of runoff. The rising and falling limbs of the hydrograph were assumed to be changing at a constant rate, forming a triangle. The routing operation indicated that the peak rate of discharge would not be significantly reduced, even if the fuse plug was assumed to be eroded to elevation 252 prior to the storm. The test flood routed through the reservoir reached elevation 264.9, 2.9 feet above the crest of the earth dam.

b. Experience Data. Discussion with water company personnel indicates that since the mid-1950's the dam has not been overtopped. However, flow has reached the crest elevation of the fuse plug.

c. Visual Observations. The on-site inspection of the dam and record information provided the data for the hydraulic evaluation of the spillways.

d. Overtopping Potential. The maximum spillway capacity is equal to 35 per cent of the spillway test flood peak flows. The peak rate of discharge from the test flood will overtop the embankment by about 2.9 feet.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY:

a. Visual Observations. There are no signs of structural instability evident.

b. Design and Construction Data. The design data presented in the previously noted references (see Section 2) do not point to any sources or areas of structural instability.

c. Operating Records. Five observation wells were installed in the downstream embankment early in 1957, see Appendix V. The water level readings made subsequent to the installation of these wells which have been provided indicate that the water level in the downstream shell is below that assumed for the stability analyses performed in the previously cited Haley and Aldrich reference, Reference d, Subsection 2.1. The available records suggest the free water surface is relatively unaffected by 11 foot variations in pool elevation. Given the assumption made for the friction angle, unit weights, and effective stresses that exist in this dam, it would appear that the factors of safety for normal seepage conditions (1.45) are close to what is usually required (1.5). Given the lower water levels recorded, there is no reason for concern about the factor of safety.

d. Post-Construction Changes. The five observation wells noted above were installed subsequent to construction of the dam. It is reported that the fuse plug has previously been replaced. However, details are uncertain.

e. Seismic Stability. This dam is in Seismic Zone 1 and, in accordance with recommended Phase I guidelines, does not warrant seismic analysis.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT:

a. Condition. Based on the visual inspection, records available, and past operational performance, the dam is judged to be in good condition. However, there are some items which should be repaired to assure the continued safe performance and operation of the dam. The conditions that need to be remedied include trespassing on the downstream slope of the main dam and growth of brush on the riprapped upstream slope. In addition, trespassing on the crest of the fuse plug should be eliminated.

The project will not pass the test flood without overtopping the dam, and therefore, the total spillway capacity is inadequate. The spillway capacity is judged seriously inadequate since the project will not pass one-half the test flood without overtopping the dam (35 per cent).

b. Adequacy of Information. The information available was adequate for a Phase I inspection. In addition to visual inspection, design plans, specifications, and stability analyses were available. The evaluation of stability was limited to the strength data contained in the Woodward-Clyde-Sherard report. No data were available on the degree of density attained during construction.

c. Urgency. The recommendations and remedial measures should be implemented by the owner within 2 years after receipt of this Phase I Report.

d. Need for Additional Investigation. Additional investigation of the scour hole beneath the spillway base is necessary to determine methods of repair and should be undertaken by the owner.

7.2 RECOMMENDATIONS:

It is recommended that the following measures be undertaken by the owner:

1) The materials contained in the fuse plug should be excavated and replaced, without tamping.

2) The apparent leak underneath the weir, which is located within the drainage manhole, should be repaired. This would insure accurate measurement of the seepage from the toe drains by recording the volume of water being discharged.

3) Subsequent to studying feasible methods, the scour hole at the spillway should be repaired.

7.3 REMEDIAL MEASURES:

a. Alternatives. Not applicable.

b. Operation and Maintenance and Procedures.

1) The owner should control trespassing on the dam, particularly near the junction of the left abutment and the downstream slope where trespassing is extensive at the present time.

2) The brush and vegetation growth on the upstream riprapped surface should be controlled.

3) The hole, approximately 80 feet south of the toe of the dam in the vicinity of Station 6+50, should be properly backfilled.

4) The wasteway channel should be cleared of heavy brush and a dense stand of grass maintained.

5) Round the clock surveillance should be provided during periods of high precipitation.

6) The owner should develop a formal warning system. An operational procedure to follow in the event of an emergency should also be adopted.

7) The owner should provide continued periodic inspections at an annual frequency.

APPENDIX A

VISUAL INSPECTION - CHECK LIST

PERIODIC INSPECTION CHECK LIST

ROJECT Mianus Reservoir Dam — DATE August 3, 1978

NSPECTOR Richard F. Murdock — DISCIPLINE Geotechnical

NSPECTOR Robert C. Smith — DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
DAM EMBANKMENT	
Crest Elevation	
Current Pool Elevation	
Maximum Impoundment to Date	
Surface Cracks	None observed
Pavement Condition	Crushed stone in good condition.
Movement or Settlement of Crest	None observed
Lateral Movement	None observed
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Good
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	Vehicular and foot paths at several locations on downstream slope.
Sloughing or Erosion of Slopes or Abutments	
Rock Slope Protection - Riprap Failures	Some local deterioration of riprap
Unusual Movement or Cracking at or near Toes	None observed
Unusual Embankment or Downstream Seepage	None observed

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam DATE August 3, 1978

INSPECTOR Richard F. Murdock DISCIPLINE Geotechnical

INSPECTOR Robert C. Smith DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
DAM EMBANKMENT - (continued)	
Piping or Boils	None observed
Foundation Drainage Features	Drainage manhole collection basin weir needs repair
Toe Drains	Toe drains observed leading into manhole
Instrumentation System	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam DATE August 3, 1978

INSPECTOR ______________ DISCIPLINE ______________

INSPECTOR ______________ DISCIPLINE ______________

AREA EVALUATED	CONDITION
OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL	Not Applicable
General Condition of Concrete	
Rust or Staining	
Spalling	
Erosion or Cavitation	
Visible Reinforcing	
Any Seepage or Efflorescence	
Condition at Joints	
Drain Holes	
Channel	
Loose Rock or Trees Overhanging Channel	
Condition or Discharge Channel	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam DATE August 3, 1978

INSPECTOR ______ DISCIPLINE ______

INSPECTOR ______ DISCIPLINE ______

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	None
a. Concrete and Structural	
General Condition	
Condition of Joints	
Spalling	
Visible Reinforcing	
Rusting or Staining of Concrete	
Any Seepage or Efflorescence	
Joint Alignment	
Unusual Seepage or Leaks in Gate Chamber	
Cracks	
Rusting or Corrosion of Steel	
b. Mechanical and Electrical	
Air Vents	
Float Wells	
Crane Hoist	
Elevator	
Hydraulic System	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam　　DATE August 3, 1978

INSPECTOR ____________　　DISCIPLINE ____________

INSPECTOR ____________　　DISCIPLINE ____________

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER (continued)	None
Service Gates	
Emergency Gates	
Lightning Protection System	
Emergency Power System	
Wiring and Lighting System In Gate Chamber	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR James MacBroom

DISCIPLINE Hydraulics/ Hydrology

INSPECTOR Robert C. Smith

DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	Wasteway
a. Approach Channel	
General Condition	Extensive vegetation along bottom of channel.
Loose Rock Overhanging Channel	
Trees Overhanging Channel	None
Floor of Approach Channel	See above
b. Weir and Training Walls	Upstream and downstream portions separated by fuse plug; surface has been compacted due to trespassing.
General Condition of Concrete	
Rust or Staining	
Spalling	
Any Visible Reinforcing	
Any Seepage or Efflorescence	
Drain Holes	
c. Discharge Channel	
General Condition	Poor
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	Trees adjacent to channel, extensive vegetation.
Floor of Channel	
Other Obstructions	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam	DATE August 3, 1978
INSPECTOR James MacBroom	DISCIPLINE Hydraulics/ Hydrology
INSPECTOR Robert Smith	DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
OUTLET WORKS - TRANSITION AND CONDUIT	Spillway
General Condition of Concrete	Good
Rust or Staining on Concrete	None
Spalling	
Erosion or Cavitation	Deep scour hole at base of spillway. Hairline cracking.
Cracking	
Alignment of Monoliths	
Alignment of Joints	Good
Numbering of Monoliths	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam DATE August 3, 1978

INSPECTOR ______ DISCIPLINE ______

INSPECTOR ______ DISCIPLINE ______

AREA EVALUATED	CONDITION
OUTLET WORKS - SERVICE BRIDGE	None
a. Super Structure	
Bearings	
Anchor Bolts	
Bridge Seat	
Longitudinal Members	
Under Side of Deck	
Secondary Bracing	
Deck	
Drainage System	
Railings	
Expansion Joints	
Paint	
b. Abutments & Piers	
General Condition of Concrete	
Alignment of Abutment	
Approach to Bridge	
Condition of Seat & Backwall	

APPENDIX B

ENGINEERING DATA

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE I

NAME OF DAM Mianus Reservoir

I.D. NO. CT 00050

ITEM	REMARKS
AS-BUILT DRAWINGS	None Exist
REGIONAL VICINITY MAP	Available From U.S.G.S.
CONSTRUCTION HISTORY	Unknown
TYPICAL SECTIONS OF DAM	Available From Plan
OUTLETS - Plan	From Plans
- Details	From Plans
- Constraints	Unknown
- Discharge Ratings	Unavailable
RAINFALL/RESERVOIR RECORDS	From Connecticut-American Waterworks Co.
DESIGN REPORTS	From Connecticut-American Waterworks Co.
GEOLOGY REPORTS	None
DESIGN COMPUTATIONS	None
HYDROLOGY & HYDRAULICS	None
DAM STABILITY	From Connecticut-American Waterworks Co.
SEEPAGE STUDIES	From State D.E.P. Records
MATERIALS INVESTIGATIONS	None
BORINGS RECORDS	On Plans
LABORATORY	From Reports/Studies
FIELD	None

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE I

NAME OF DAM Mianus Reservoir

I.D. NO. CT 00050

ITEM	REMARKS
POST-CONSTRUCTION SURVEYS OF DAM	None
BORROW SOURCES	Unknown
MONITORING SYSTEMS	Yes, Water Levels, Seepage Quantity
MODIFICATIONS	Monitoring System
HIGH POOL RECORDS	From Connecticut-American Waterworks Co.
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	From State D.E.P.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	None
MAINTENANCE OPERATION RECORDS	From Connecticut-American Waterworks Co.
SPILLWAY PLAN	
SECTIONS	Plans
DETAILS	Plans
OPERATING EQUIPMENT	Plans
PLANS & DETAILS	Plans

REPORT ON
STABILITY AND SEEPAGE ANALYSES

MIANUS RIVER DAM
STAMFORD, CONNECTICUT

APRIL 1959

for
Water Resources Commission
State of Connecticut

by
Haley & Aldrich
Consulting Soil Engineers
Cambridge, Mass.

58-211

HALEY & ALDRICH
CONSULTING SOIL ENGINEERS
238 MAIN STREET. CAMBRIDGE 42. MASS.
UNIVERSITY 4-2779

April 30, 1959

Mr. John J. Mozzochi
Consulting Engineer
265 Hebron Avenue
Glastonbury, Connecticut

Subject: Report on Stability and Seepage Analyses
Mianus River Dam, Stamford, Connecticut

Dear Mr. Mozzochi:

Submitted herewith is our report of stability and seepage investigations in connection with the Mianus River Dam. This study was made in accordance with the first phase of our contract, dated 2 January 1959, with the Water Resources Commission, State of Connecticut. The study was based on a review of pertinent correspondence, records, the design study report, contract plans and specifications.

The dam site was visited and information on as-built conditions were obtained in discussions with Mr. David M. Greer, Greer Engineering Associates and by review of daily reports of the soils control engineer.

After you and personnel of the Commission have had an opportunity to review the report we would be glad to discuss or furnish any further details desired. If you feel that additional assurance as to the stability of the dam is desirable, then the program of field observations and tests contained in the recommendations could be initiated.

We appreciate the opportunity of working with you on this project and will be pleased to be of further service.

Very truly yours,
HALEY & ALDRICH
Steve J Poulos
S. J. Poulos
J. F. Haley
J. F. Haley

Submitted: 3 copies
Water Resources Commission, 3 copies

TABLE OF CONTENTS

SYNOPSIS

The purpose of this report is to determine the estimated stability of the downstream slope of the Mianus River Dam and the cause of the high seepage losses which have been measured.

A flow net obtained graphically was modified in accordance with data measured in the field and used as a basis for seepage loss and seepage force computations. The estimated seepage losses came to approximately 275,000 gallons per day using what are believed to be reasonable permeability assumptions. The measured seepage loss, after subtracting surface runoff, is approximately 570,000 gallons per day. The agreement between the two figures is believed to be reasonable when it is considered that variations in permeability by factors of 10 and even greater are common in stream bed deposits.

Stability analyses which included the effect of seepage forces resulted in an estimated factor of safety of 1.45 for the case of zero cohesion and an angle of internal friction of 35 degrees in the embankment and foundation soils. This factor of safety is considered to be adequate for the steady seepage state with maximum reservoir level. Inclusion of horizontal earthquake forces equal to 0.1 times the acceleration of gravity produces a factor of safety of 1.05, which is considered satisfactory for the extreme assumptions made.

Assumptions that are in our opinion reasonably conservative for the soil properties would lead to the conclusion that the dam is safe as constructed. However, in order to arrive at a more definite indication of the margin of safety of the downstream slope of the dam it would be necessary to obtain additional information on soil properties and seepage conditions in the dam. It is therefore recommended if more positive assurance than was possible from this study is desired, that a program of field and laboratory investigations be undertaken to measure the required soil properties to confirm or refine the assumptions used in the analysis.

I. INTRODUCTION

1-01. GENERAL

This report of studies of seepage and stability of the Mianus River Dam, Stamford, Connecticut, is submitted in fulfillment of the first phase of the contract between the Water Resources Commission and Haley & Aldrich, entitled Consulting Engineer Services for Inspection and Reports on Dams, Dikes, Reservoirs and Similar Structures dated 2 January 1959. Measurements of flow through the dam have previously been made which indicate that approximately six times the original estimate of seepage loss has occurred. Standpipes installed in the downstream shell of the dam show that the piezometric level to be relatively high. Measurements of the surface elevation of the downstream slope at various locations along the dam show that the actual shape of the slope differs from the design. The possibility that movement has occurred since completion of construction was therefore postulated.

1-02. PURPOSE

The purpose of this study is to determine the indicated stability of the downstream slope of the Mianus River Dam and the effect on the stability of the measured piezometric level and the seepage losses. It is also the purpose of this report to formulate a suggested program of field and laboratory investigations so that a more thorough analysis of the stability of the dam may be made in the event that available data and analyses are found to be insufficient for the purposes of the Commission.

1-03. SCOPE

The scope of this report includes the review of past correspondence concerning the stability of the downstream slope of the Mianus River Dam and an analysis of its stability based upon the information contained therein. Data on previous field measurements of seepage loss through the dam, shape of the downstream slope and water surface elevations were made available to this office by the Water Resources Commission. No field studies were made directly in connection with this report. We have made no attempt to confirm the hydrologic study or to determine the stability of the upstream slope.

The dam was examined by Mr. James F. Haley of this firm on January 27, 1959 and again on January 30, 1959. On the latter date Mr. John J. Curry, Chief Engineer and Mr. John J. Mozzochi, Consultant for the Water Resources Commission were present during the inspection. On January 29, 1959 Mr. Haley visited the office of Greer Engineering Associates, Montclair, New Jersey, and discussed the design and construction of the dam with Mr. David M. Greer of that firm. Mr. Greer made available for review results of their studies and also the soil control reports made during construction. These reports contained the results of field density and permeability tests and grain size distribution of the materials used in the various portions of the construction. The shear strength and permeability characteristics used in the analyses in this report were based or assumed primarily from the review of the soil control reports.

II. SEEPAGE ANALYSES

2-01. SEEPAGE LOSS

The flow net for a section of the dam at Station 5+0 is shown in Figure 1. In sketching the net, it was assumed, based on field control reports, that the upstream shell and core were of the same permeability and that it was one-tenth that of the downstream shell. For the purpose of drawing the flow net the downstream shell and foundation were assumed to have equal permeabilities and the stratification ratio in all parts of the dam was assumed to be 9. Since the upstream portion of the embankment is relatively impermeable when compared with the foundation, seepage loss estimates may be made by considering the effects of the foundation and embankment individually without substantial error. This was done in all of the following analyses.

In comparing the measured manhole outflow with estimates from the flow net it is necessary to make proper assumptions for permeability and to discount flows due to rainfall and subsurface seepage that by-passes downstream drainage system. Available information shows that the core and upstream shell have a permeability between 100 and 500 feet per year. The foundation permeability varies between 3100 and 5400 feet per year. The permeability of the downstream shell is in the vicinity of 1800 feet per year. Permeabilities referred to are in a horizontal direction. The assumed values of permeability for the purpose of computing quantity of flow are:

Core and upstream shell . . . 180 ft/yr.
Downstream shell. . . . . . . 1800 ft/yr.
Foundation . . . . 3600 ft/yr.

As shown in the flow net, not all of the seepage loss is reflected at the weir which measures flow out of the manhole in the rock toe. Thus approximately 1.1 flow paths in the foundation must be discounted when comparing estimated and measured flows. Also, flow measured at the manhole reflects surface runoff after a rainfall. This fact is vividly demonstrated in the lower curves of Figure 3 which compare weekly rainfall with weekly measurements of manhole outflow. Curve (a) shown connecting the low points of the plot of manhole outflow is the best estimate of measured seepage loss through the dam. The remainder of the flow should be credited to the surface infiltration of rainfall.

Calculations based upon the values of permeability assumed above, the flow net in Figure 1 and a pool elevation of 253, with proper allowance for the location of the measuring weir, result in an estimated seepage loss of 275,000 gallons per day. From curve (a) in Figure 3 at a pool elevation of 253, a reasonable estimate for measured seepage loss is 570,000 gallons per day. Permeability variations from field or laboratory test results by factors of 10 or even greater in deposits, such as those considered herein, are common. We therefore feel that the agreement between the estimated and the measured losses is entirely reasonable. The quantity of flow in itself does not affect stability and need only be considered from the standpoint of function for which the dam was constructed. The various factors that do affect stability are discussed in Part III.

In the event that the pool elevation rises to 260 and remains there long enough to affect seepage loss, the flow measured at the manhole may be expected to increase by 100,000 gallons per day over the amount of flow measured with the pool at elevation 253, as shown in Figure 3.

To compute the estimated seepage losses it was necessary to use an "effective length" for the foundation and core. The "effective length" is that length of dam of constant cross section equal in geometry and scale to that shown in Figure 1 which allows seepage loss equal to the actual dam with its varying geometry and scale. The effective length of the foundation was obtained by means of a weighted average using the relationship between flow per unit head drop and the ratio of base width to foundation thickness for flow beneath impermeable dams. This relationship is plotted on page 212 of reference (1). An average value of flow per unit head loss over the length of the dam was computed by inserting the actual ratios of base width to foundation thickness into the above referenced graph and obtaining the corresponding flow per unit head loss. Dividing this average by the value for the section on which the flow net was drawn and multiplying by the length of the actual dam results in the new effective length of foundation. This method yields a value of 262 feet, which agrees with that used in the original design study. The equivalent length of the core was similarly obtained by use of the relation that seepage through the core is proportional to the quantity:

$$(d^2 + h_t^2)^{1/2} - d$$

where "d" is a horizontal distance which depends upon the cross section of the dam and "h_T" is the total head loss through the dam. This relationship, given on page 178 of reference (2), results in an effective length of core of 610 feet. Both of the above methods are approximate and based upon the assumption previously stated that flow beneath and through the upstream portion of the dam may be considered independently without causing serious error.

2-02. FACTORS AFFECTING SEEPAGE LOSS

Seepage loss through and beneath the dam is dependent upon permeability, boundary conditions and headwater elevation. The measured seepage loss is affected by the surface runoff and the location of the measuring weir as noted above.

a. Permeability: The sensitivity of seepage losses to the permeability of the soils is shown by the plot contained in Figure 1. The formula for computing flow which is also shown on Figure 1 indicates that flow is directly proportioned to permeability.

The ratios of permeabilities between the various portions of the embankment and the foundation also affect the flow since changing ratios changes the flow net. The ratio of the number of flow paths to the number of equipotentials would therefore change in the formula used for computing flows.

The stratification ratio also affects flow by changing the flow net. Higher stratification ratios result in higher rates of flow if the effective permeability

$$(k_v \times k_h)^{1/2}$$

remains constant.

Of the three effects of permeability mentioned above, it is the absolute values which most strongly affect flow. It is therefore necessary to obtain more accurate values before refinements in seepage computations can be made. The ratios of permeability do not strongly affect quantity of flow, however they do have substantial effect on seepage forces and stability as discussed in Part III.

b. Boundary Conditions: The boundary conditions which are variable and affect the rate of flow appreciably are the back pressures in the graded filter and rock toe. If the porous pipe in the filter is plugged, or if the filter does not operate properly, the pressure within it will rise and cause a decrease in the rate of flow through and beneath the dam. This also holds true for a rise in pressure in the rock toe. A more important effect of this increased pressure in the filters is its affect on stability as shown in Part 3-02. Rough computations of the capacity of the porous concrete drains to pass the seepage water through its walls indicate that no back pressure will result from the use of the pipe as shown in the plans. A filter constructed of the materials and in the manner as proposed in the design would, in our opinion, result in satisfactory filter operation.

Another boundary effect is the three dimensional variation in the foundation thickness and dam height. Consideration was given to this effect by computing the effective lengths mentioned above, however each cross section was analysed independently of the adjacent section. In reality, the three dimensional effect causes the flow through each section to be affected by its neighbor i. e., a component of the flow in a direction perpendicular to the main river direction does occur in the dam. These lateral velocities are likely to be small in the central portions of the dam and would tend to increase at the edges where the ground and bedrock slopes are steeper. Since the majority of the flow occurs through the central portions, the effect on flow rates of three dimensional lateral velocities should be small. The three dimensional effect also explains the fact that the measured free water surface elevation is higher near the abutments of the dam. Since the ground elevation and gravel filter are at a higher elevation near the abutments and the pool elevation is constant across the dam, the piezometric surface does not have to drop as rapidly to meet the tailwater elevation.

c. Headwater Elevation: Assuming the flows through the embankment and foundation to be independent of each other, the flow through the foundation is directly proportional to headwater elevation. The flow net in the embankment changes as headwater elevation changes. Therefore the values of n_f and n_d as well as the total head must be changed when computing the flow through the embankment. In computing the values given above for seepage losses with a pool elevation of 253, an estimate of the change in n_f was made. Since the change is small, little error is introduced. The change in n_d in this case is smaller and was neglected.

d. Surface Runoff: An indication of the quantity of surface runoff reaching the manhole can be obtained only by considering the duration and rate of precipitation in any storm, the drainage area and the runoff relationships. A rough calculation shows that peak flows of one to 1.5 million gallons per day can be expected from a storm in which 4" of rainfall occurs in 8 hours if the runoff is 60% of 70% of the precipitation. These are reasonable assumptions and the quantity checks with the peak flow of 2.02 million gallons per day noted in Figure 3. This latter value includes seepage losses through and beneath the dam on the order of 0.6 million gallons per day.

III. STABILITY ANALYSES

3-01. RESULTS

Stability analyses using the method shown in Figure 2 on the trial circles shown in Figure 6 result in a factor of safety of 1.45, assuming that there is no cohesion in the embankment or foundation and that the friction angle is constant at 35°. These assumptions of soil strength are considered conservative. Inclusion of earthquake forces equal to one-tenth the acceleration of gravity in the horizontal direction decreases the safety factor to 1.05. Earthquake forces are not generally considered in stability studies for earth dams in New England, however, we have included this possibility in our study because of the downstream occupancy. The low factor of safety for this extreme loading condition is considered satisfactory. In all stability calculations, the seepage forces caused by flow through the dam were included. To obtain realistic values of these forces, the free surface of flow was modified by field measurements as shown in the notes of Figure 1.

Based on these preliminary studies, the downstream slope thus appears to be safe. To obtain a more refined estimate of the safety of the downstream slope it would be necessary to make field and laboratory investigations to determine the soil properties more accurately to confirm the assumptions. The effect of each of the assumed soil properties on the stability is discussed in the following section.

3-02. FACTORS AFFECTING STABILITY

a. Friction Angle: As shown in the graph contained in Figure 2 increasing the friction angle markedly increases the factor of safety. The normal range of friction angles for soils of the type in the downstream portion of the embankment and the foundation is between 33 and 45 degrees, with the higher values being more prevalent.

b. Cohesion: Since the cohesive force resisting failure is the product of arc length and cohesion, the factor of safety increases with increasing cohesion. This relation is also evident from the graph in Figure 2. The rate of change of the factor of safety with the value of cohesion is greater for trial circles entirely in the embankment portion of the dam. This is true since a greater proportion of the total force resisting failure is in the cohesive strength for those circles. The foundation soils probably have low cohesion while the downstream materials would have only slight cohesion.

c. Seepage Forces: Seepage forces caused by the friction between soil and water while water is flowing through the dam result in lower safety factors than occur in identical dry embankments. Seepage forces are increased in the downstream slope and stability is therefore decreased when a) the ratio of the permeability of the downstream shell to the core or the foundation is decreased, b) the ratio of horizontal to vertical permeability is increased and c) back pressures build up in the graded filter or rock toe. All three results in a rise in the free water surface within the downstream portion of the dam.

The above factors indicate that accurate measurements of the permeability ratios involved and the pressure in the graded filter are required to adequately analyze the embankment. If field investigations indicate that the rock toe is clogged, it is also advisable to measure the pressure therein.

d. Unit Weight: The unit weight of soil in the embankment also affects the stability. Except in the case of zero cohesion, an increase in unit weight will cause a small decrease in safety factor if other variables are held constant. Unit weight has an even smaller effect on the safety factor when no cohesion exists.

e. Earthquakes: Greenwich, Connecticut is generally considered to be in a seismically active area although no earthquakes of consequence have occurred there in the recent past. For the design of earth dams earthquake forces acting horizontally on the failure mass with a magnitude of 0.1 times its weight are sometimes used in such seismic regions. Under these conditions a safety factor of just over one is deemed adequate since the possibility that maximum seepage and earthquake forces will act concurrently is rare. There are few if any well documented failures of earth dam embankments that have been credited to earthquake forces. On the other hand, several instances of earth dams that have withstood earthquakes may be found in ·he literature.

IV. CONCLUSIONS

Based on the stability and seepage analyses reported herein, made using what are believed to be conservative assumptions of shear strength and permeability of foundation and embankment soils, the conclusions are summarized as follows:

1. The indicated minimum factor of safety of the downstream portion of the dam with the reservoir at maximum level and a steady seepage state is 1.45. This factor of safety is lowered to 1.05 when earthquake forces are considered. Factors of safety of these magnitudes are considered adequate for the assumed conditions. The indicated stability of the dam would decrease if the free water surface in the downstream portion of the dam rises to a higher level than assumed in the analyses or if the foundation and embankment soils have less shear strength than assumed.
2. The stability study indicates that the factor of safety against a relatively shallow slide within the downstream slope is 1.61. Thus it is believed that the apparent sloughing at the downstream toe is due to erosion of the topsoil or material from the slope surface prior to establishment of turf cover and possibly shallow creep of the surface materials occurring during frost melting periods.
3. The measured quantity of seepage flowing through the embankment and foundation into the drainage system is approximately 0.88 cu. ft. per sec as compared with an estimated quantity from flow net analysis of 0.42 cu. ft. per sec. These figures are in reasonable agreement considering the possible variations in permeability of the soils and the assumptions of boundary conditions that are necessary in the flow net analyses.
4. In order to make more refined analyses of the stability it will be necessary to perform the field and laboratory test program outlined in Part V.

V. RECOMMENDATIONS

5-01. FIELD INVESTIGATIONS

A layout of the recommended field test program is shown in Figure 4, if it is desired to obtain a more definitive estimate of safety of the downstream portion of the dam.

a. Test Pits: Six test pits approximately three feet square and five feet deep should be dug where shown. Sufficient samples are to be taken to provide for the laboratory tests outlined in paragraph 5-02. Test Pit No. 5 is to penetrate through the soil cover and into the rock toe so that a visual indication of whether or not fines have migrated into the toe may be obtained. Pit No. 6 is to be excavated to determine the properties of the foundation since the critical failure circle shown in Figure 2 is partially in the foundation material. The other four pits are to be dug at varying elevations in the embankment in an attempt to reflect the properties of material from various borrow pits used during construction.

In the process of excavation or at adjacent locations, field densities of the material to be laboratory tested should be taken.

b. Water Observations: One wellpoint and two standpipes should be installed as shown. The standpipes will reveal the location of the free surface near the toe of the dam where its elevation is very sensitive to changes in permeability of the soil in the embankament and is difficult to establish analytically. These standpipes should be driven to the elevation shown or as governed by field conditions. A minimum depth, just sufficient to reach the free surface, should be drilled.

The wellpoint is to be installed in the graded filter in order to determine the pressure therein. As drilling proceeds, the change in material from the downstream shell to the filter should be evident from the washings and an elevation measurement. The wellpoint itself (see sketch in Figure 5) should be imbedded in the filter gravel.

c. Displacement Stakes: Lateral, or downward movement of the downstream slope may be detected from continual observation of the displacement stakes shown in Figure 4. A typical stake is shown in Figure 5.

d. Other: In addition to the above field measurements, a limited number of manhole outflow, rainfall and pool elevation records should be made to obtain a more complete correlation of data, and to determine if any changes in seepage quantities have occurred since the time of previous observations.

5-02. LABORATORY TESTS

Using samples obtained from the test pits, the following laboratory tests are recommended:

a. Triaxial tests to determine shear strength and permeability.

b. Grain size analyses.

c. Permeability tests separate from the triaxial test to obtain the variation of permeability with void ratio.

Standard classifications, water content, Atterberg limits (if applicable) and specific gravity measurements should also be made to supply the information required for analysis.

5-03. ANALYSIS

The primary object of the analysis is to obtain proper seepage forces and soil properties to evaluate stability. A flow net corrected on the basis of measured water surface elevations, filter pressures and permeabilities should first be drawn. If there is no appreciable change in seepage forces from those used herein, then it is only necessary to insert the new shear strength data into the stability trials already completed. Any substantial change in seepage forces will necessitate a stability analysis using the newly measured values of cohesion, frction, unit weight and seepage forces.

It is possible that the present variation of the downstream slope was caused by erosion prior to establishment of the turf cover. If this is true the grain size analyses of the upper strata of soils near the toe of the slope, will indicate that they are the finer components of the soils located higher on the slope which have been washed down.

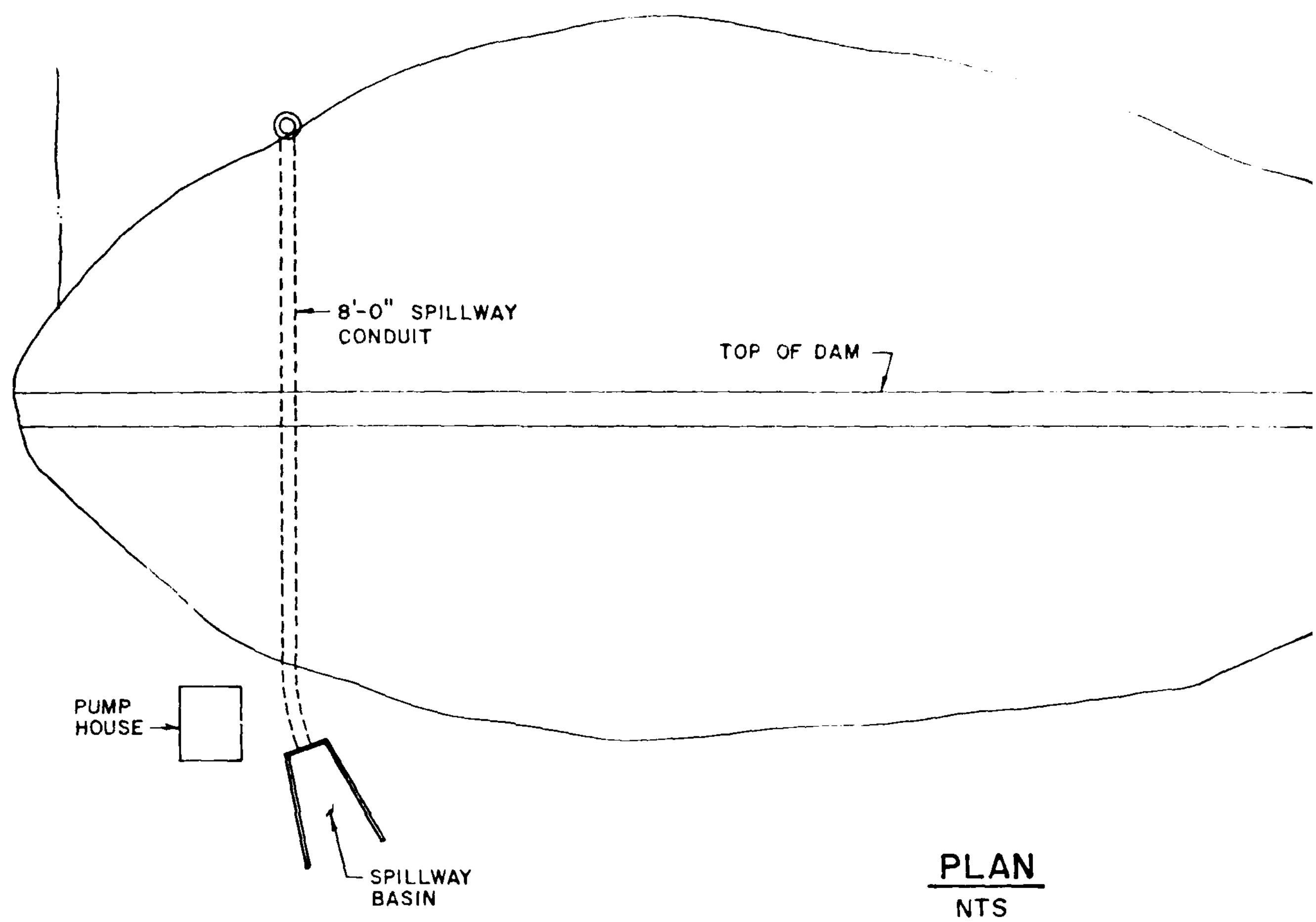
The above combined with the results of displacement stake observations will permit proper evaluation of the stability of the downstream slope. It should be pointed out that the above recommended tests may substantiate assumptions made herein and obviate the requirement for involved calculations over and above those completed to date.

REFERENCES

(1) Muskat, M., "The Flow of Homogeneous Fluids through Porous Media", New York, McGraw Hills, 1937.

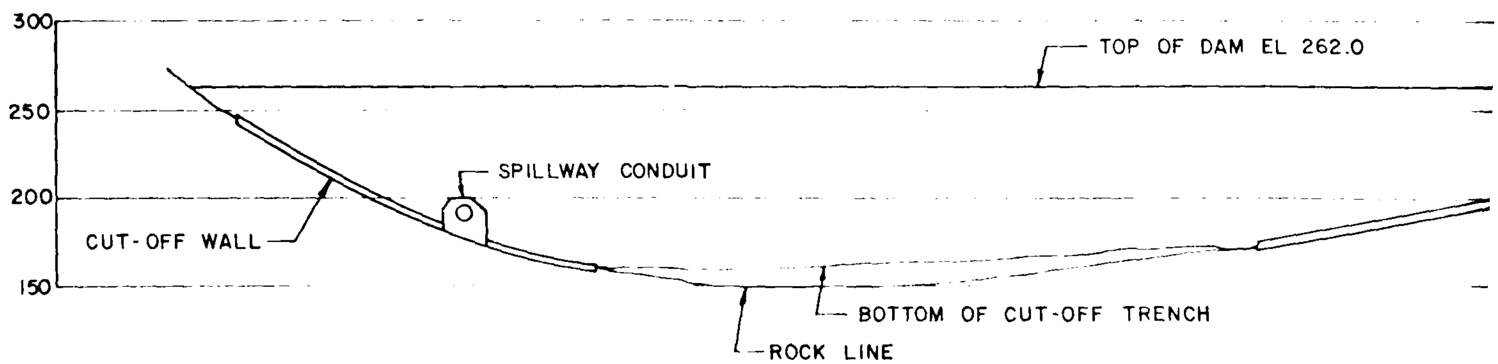
(2) Taylor, D. W., "Fundamentals of Soil Mechanics," New York, Wiley 1948

(3) Greer and McClelland, "Investigation and Design of the Mianus River Dam, Stamford, Connecticut", November 1954.



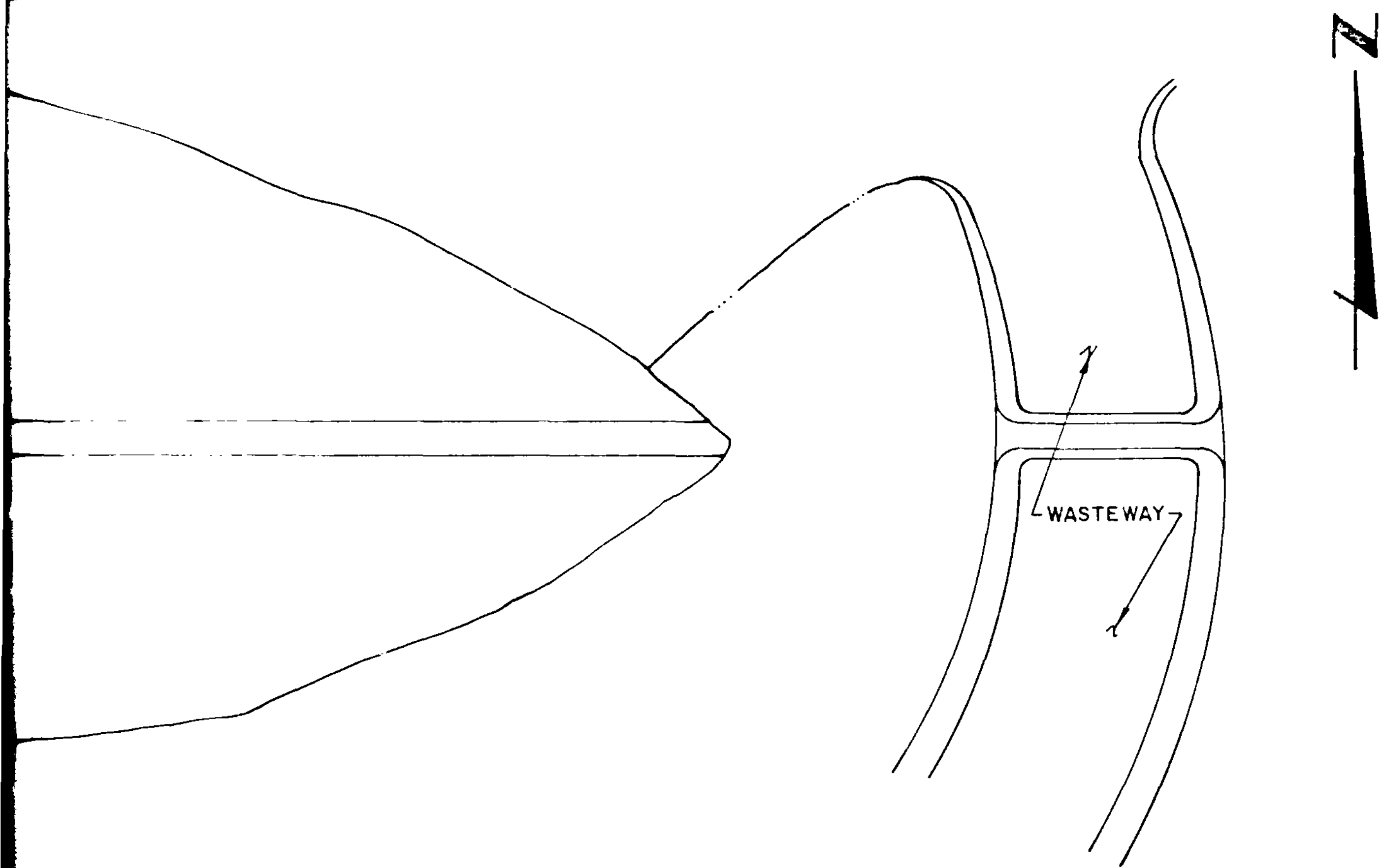
PLAN

NTS

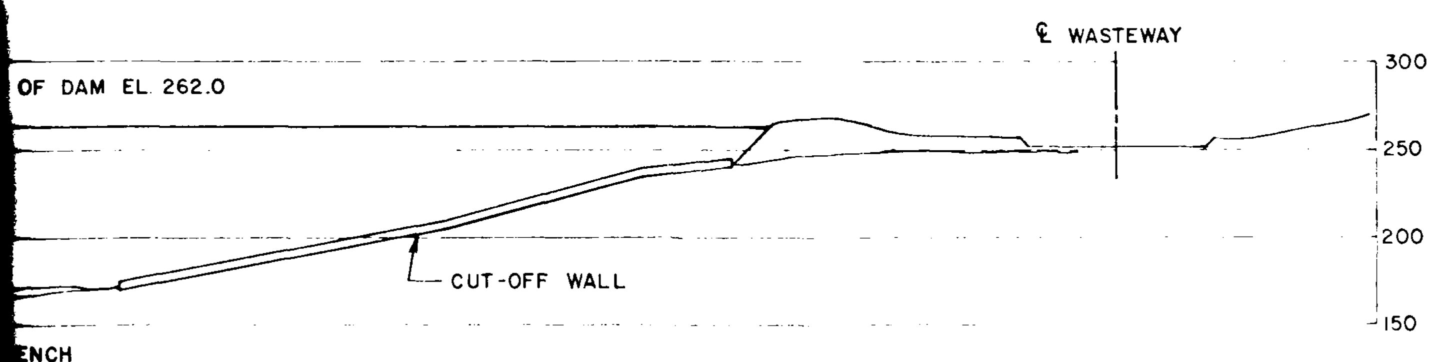


DOWNSTREAM SECTION OF DAM

NTS



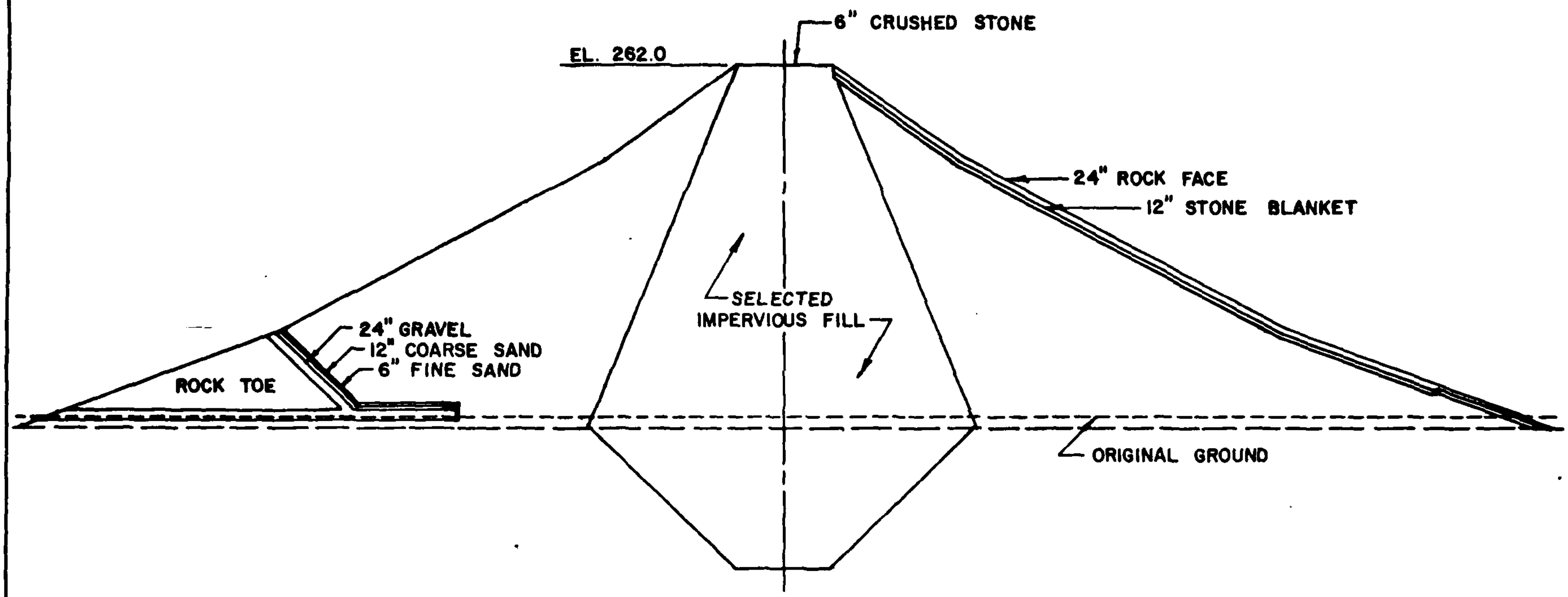
LAN
TS



SECTION OF DAM
NTS

MIANUS RESERVOIR DAM

MIANUS RIVER

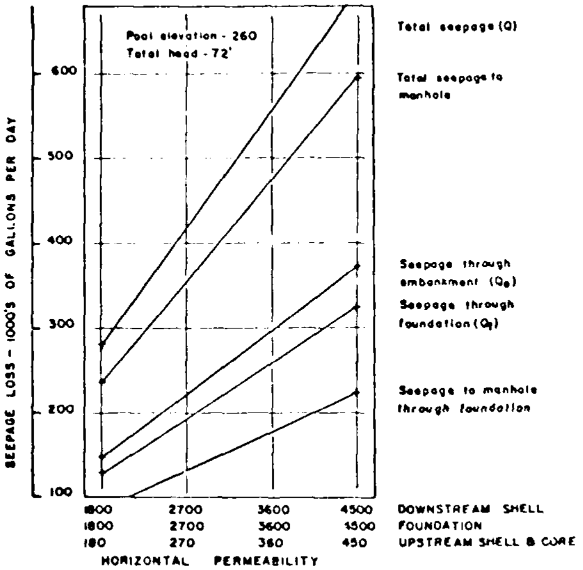


SECTION THRU DAM

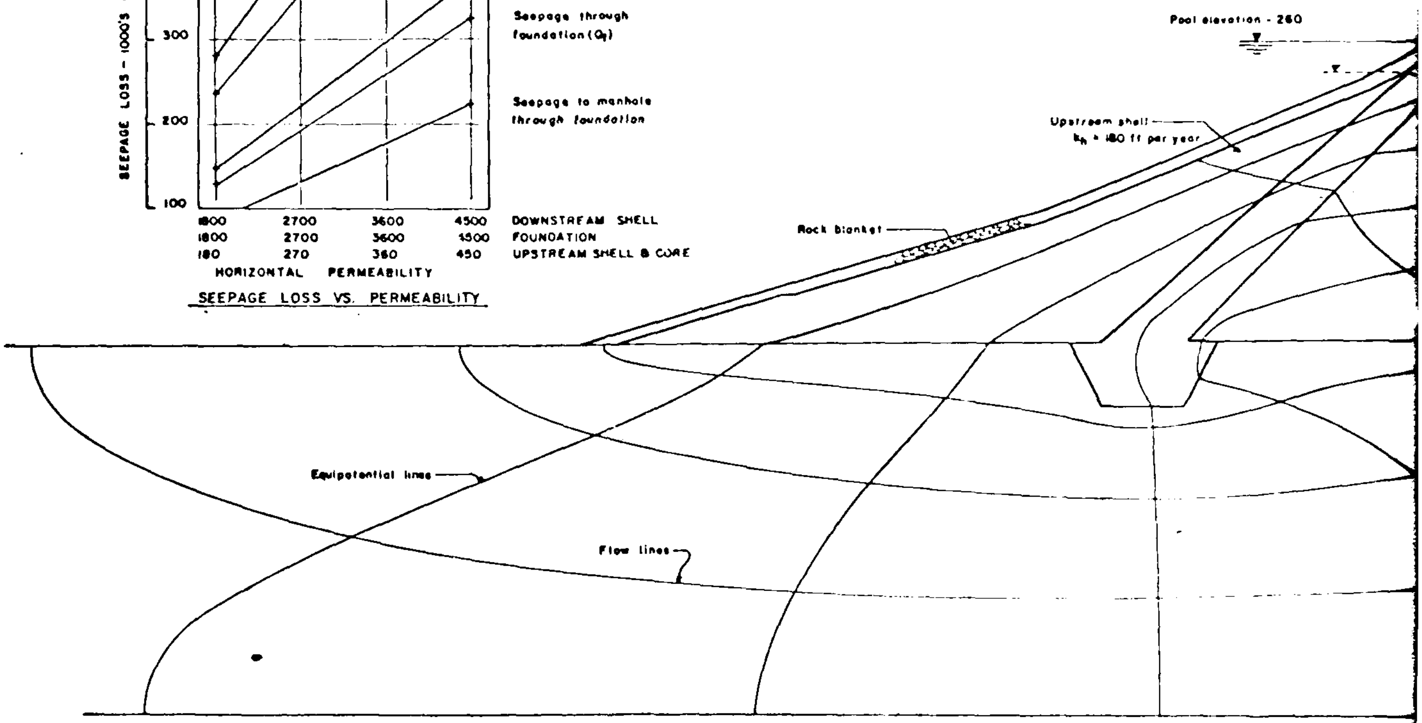
NTS

MIANUS RESERVOIR DAM

MIANUS RIVER



SEEPAGE LOSS VS. PERMEABILITY



FLOW NET AT CROS

NOTATION

k_v = Vertical coefficient of permeability (ft./yr.)

k_h = Horizontal coefficient of permeability (ft./yr.)

n_d = Number of equipotential drops

n_f = Number of flow paths

H = Total head (ft.)

Q = Rate of seepage loss through dam (gal/day)

L_e = Effective length of embankment (ft.)

L_f = Effective length of foundation (ft.)

k_h/k_v = Stratification ratio

$$Q = \frac{7.5}{365} \times L \times \frac{n_f}{n_d} \times H \times \sqrt{k_h k_v}$$

SAMPLE SEEPAGE COMPUTATIONS

a) Embankment (pool elevation - 260)

$k_h = 9k_v = 1800$ ft/yr $\quad Q_e = 610 \times \frac{1.65}{6}\ 72 \times \sqrt{1800 \times 200} \times \frac{7.5}{365}$

$H = 72$ ft.

$n_f = 1.65$ $\quad Q_e =$ 149,000 gal per day (plotted above)

$n_d = 6$

$L_e = 610$ ft

b) Foundation (pool elevation - 260)

$k_h = 9k_v = 1800$ ft/yr.

$H = 72$ ft. $\quad Q_f = 262\ \frac{3.35}{6} \times 72 \times \sqrt{1800 \times 200} \times \frac{7.5}{365}$

$n_f = 3.35$

$n_d = 6$ $\quad Q_f =$ 130,000 gal per day (plotted above)

$L_f = 262$ ft.

$Q = Q_f + Q_e =$ 279,000 gal per day (plotted above)

NOTES

1. ⊙ The
measur
values

Dotted
opposed
by flow

2. To obta
for use
a)

b)

Seepage
from
in the
increa

58-211

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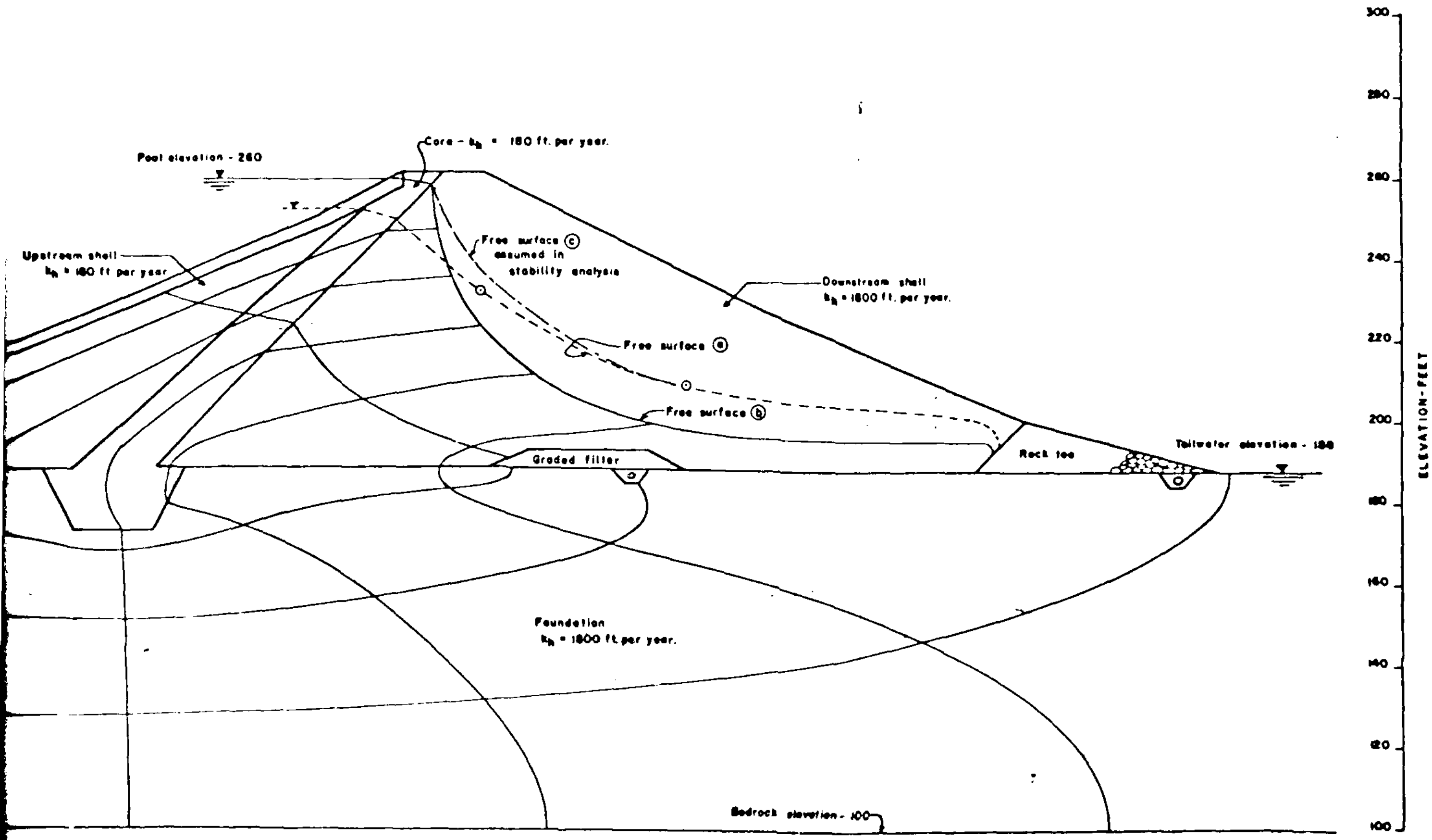
CONSULTING SOIL ENGINEERS



FLOW NET AT CROSS SECTION 5+0

80)

$= \frac{165}{6} 72 \times \sqrt{1800 \times 200} \times \frac{7.5}{365}$

169,000 gal per day (plotted above)

$\frac{335}{6} \times 72 \times \sqrt{1800 \times 200} \times \frac{7.5}{365}$

000 gal per day (plotted above)

Q_e = 279,000 gal per day (plotted above)

NOTES

1. ⊙ These points shown above are at locations of standpipes where field measurements of free surface were made. Points shown are the maximum values from Figure 3 with a pool elevation of 253.

   Dotted line (a) indicates probable location of actual free surface as opposed to solid line (b) which is location of free surface obtained by flow net sketching using the noted permeability assumptions.

2. To obtain free surface (c) and seepage forces that reflect field data for use in stability analysis, the following procedures were used.
   a) Raise elevation of free surface (a) by approximately 7 feet at the pool and to conform with pool elevation of 260
   b) Start equipotentials at this new surface from the elevation at which they intersect line (b) and extend them to the left (upstream) parallel to their original direction

   Seepage forces from the altered flow net are greater than those from the original net. However, only the shallow trial circles in the embankment are materially affected by these increased forces (See location of trial circles in Figure 6)

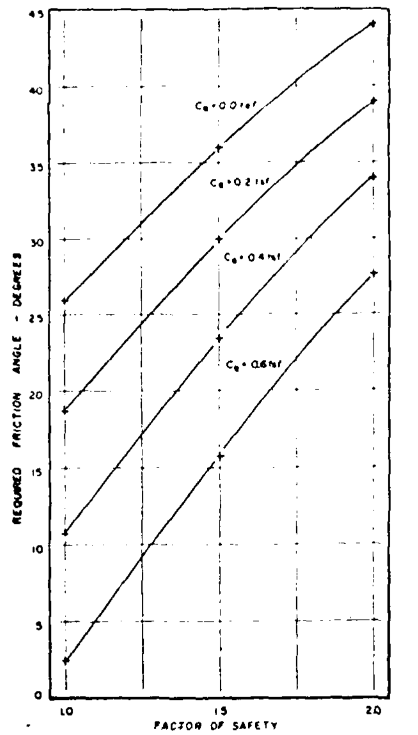
MIANUS RIVER DAM
STAMFORD, CONNECTICUT

FLOW NET STUDY

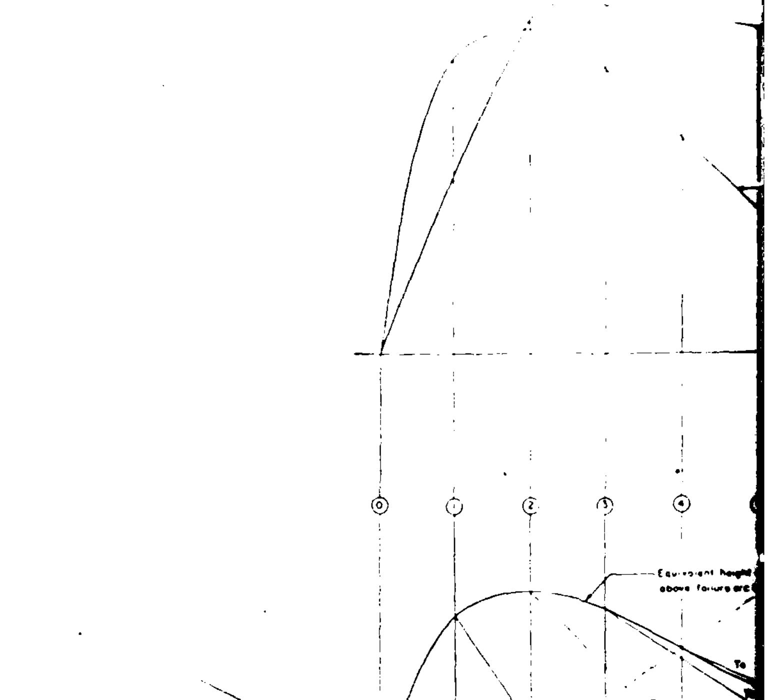
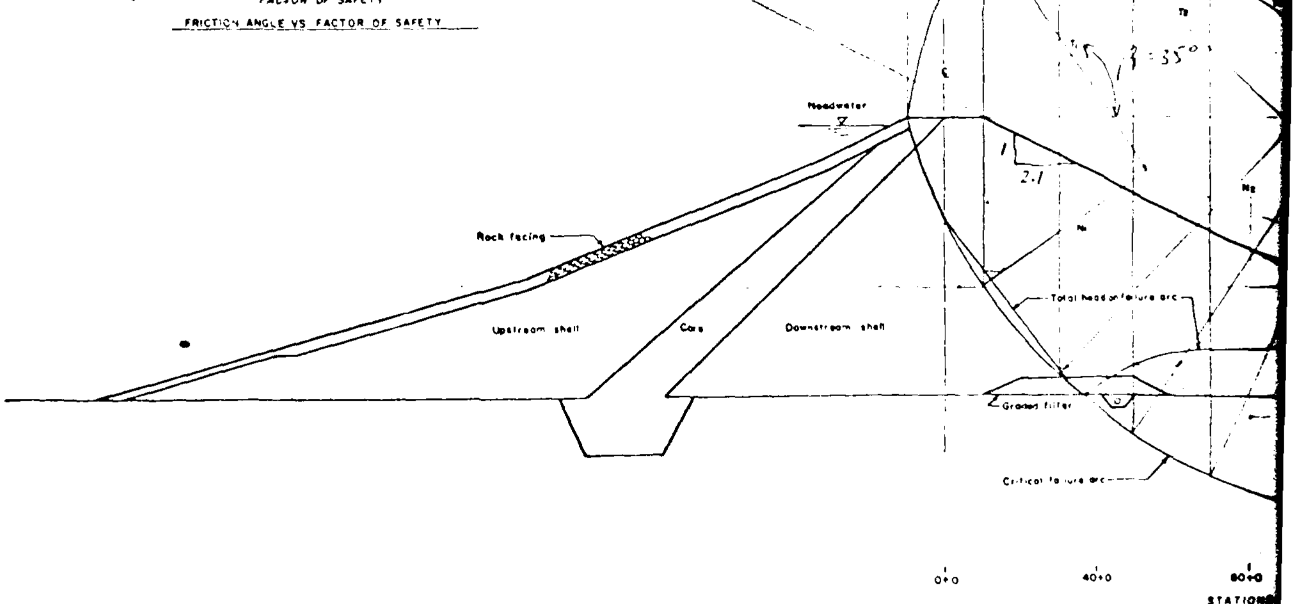
STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
STATE OFFICE BUILDING HARTFORD, CONN.

Scale: - 1" = 20'

FEBRUARY 1959



FRICTION ANGLE VS FACTOR OF SAFETY

CROSS SECTION AT STA 5+0

Scale 1"=20' 1sq in = 400 sq ft

58-211

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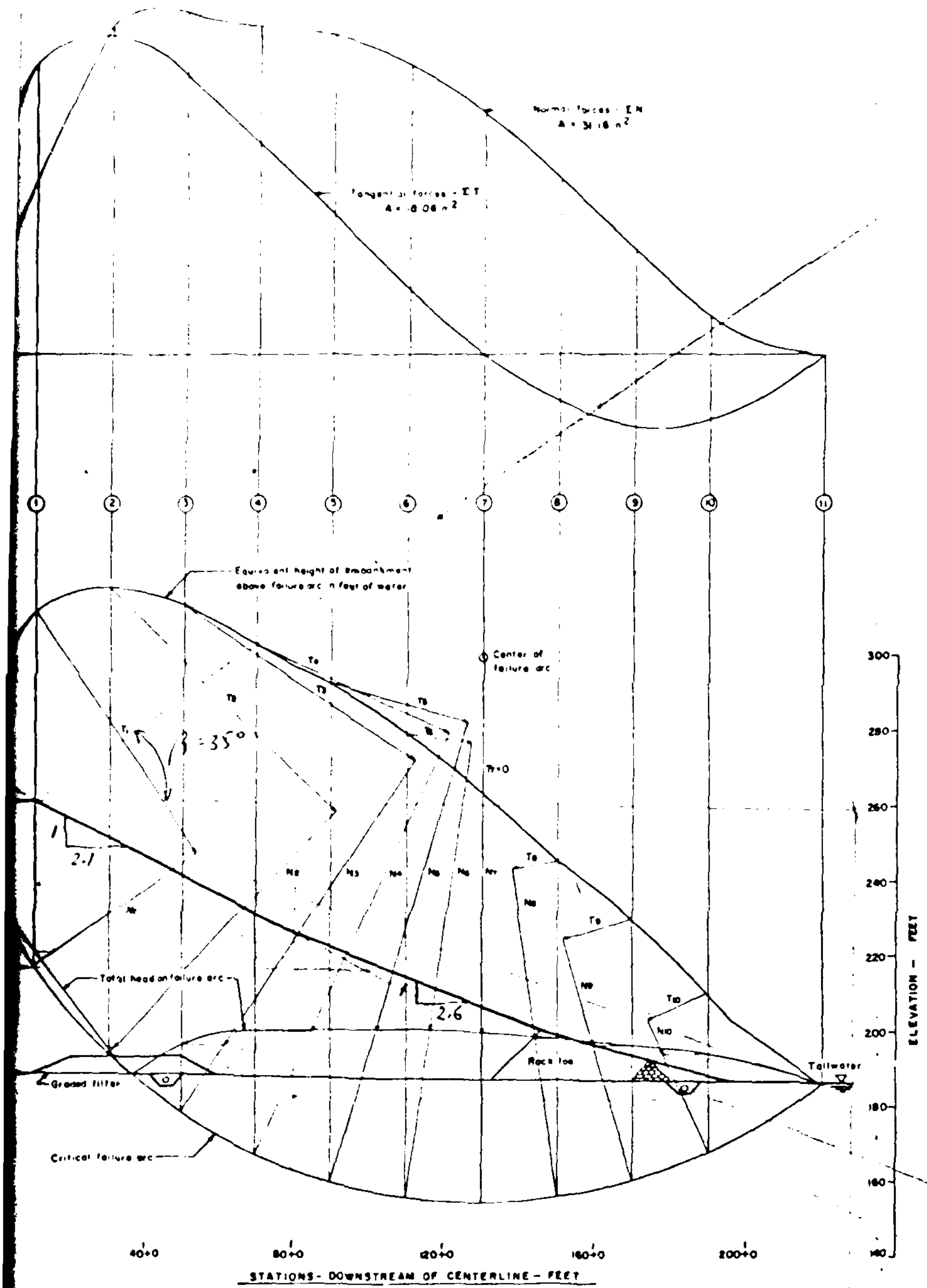
CONSULTING SOIL ENGINEERS



AT STA 5+0

40

400 sq ft

ASSUMPTIONS

Embankment	135 lbs per cu foot
Foundation	128 lbs per cu foot
Cohesion	Constant along failure arc
Friction angle	Constant along failure arc
Flow net	See Figure 1

PROPERTIES OF CRITICAL CIRCLE

Radius	145 feet
Central angle	113°
Arc length	286 feet
Location of center	Sta 130' Rt, Elev 300

COMPUTATION OF FACTOR OF SAFETY

$$F_s = \frac{\Sigma N \tan \phi_e + C_e L_a}{\Sigma T}$$ (Factor of safety with respect to strength)

ΣT = Sum of tangential driving forces.
ΣN = Sum of normal intergranular forces.
ϕ_e = Required effective friction angle
C_e = Required effective cohesion
L_a = Length of failure arc

$$\Sigma T = 15.06 \times \frac{62.4}{2000} \times 400 = 187.7 \text{ tons}$$

$$\Sigma N = 31.16 \times \frac{62.4}{2000} \times 400 = 387.7 \text{ tons}$$

F_s	C_e	$\tan \phi_e$	ϕ_e
1.0	0.0	0.485	25.8
	0.2	0.337	18.6
	0.4	0.189	10.7
	0.6	0.0414	2.4
1.5	0.0	0.726	36.0
	0.2	0.580	30.1
	0.4	0.432	23.4
	0.6	0.284	15.8
2.0	0.0	0.969	44.0
	0.2	0.823	39.4
	0.4	0.674	34.0
	0.6	0.524	27.7

NOTES

1. See Figure 6 for tabulated data on all trial failure circles and their location in the embankment.
2. See Figure 1 for flow net used in obtaining total head on failure arc.

MIANUS RIVER DAM

STAMFORD, CONNECTICUT

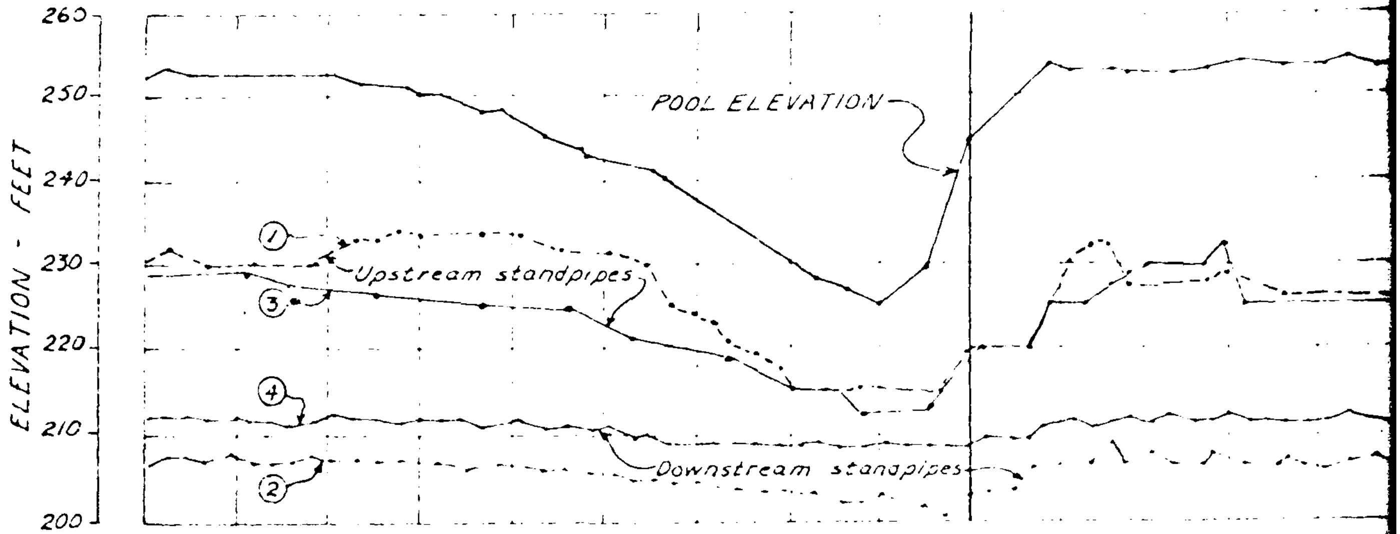
STABILITY ANALYSIS

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
STATE OFFICE BUILDING HARTFORD, CONNECTICUT

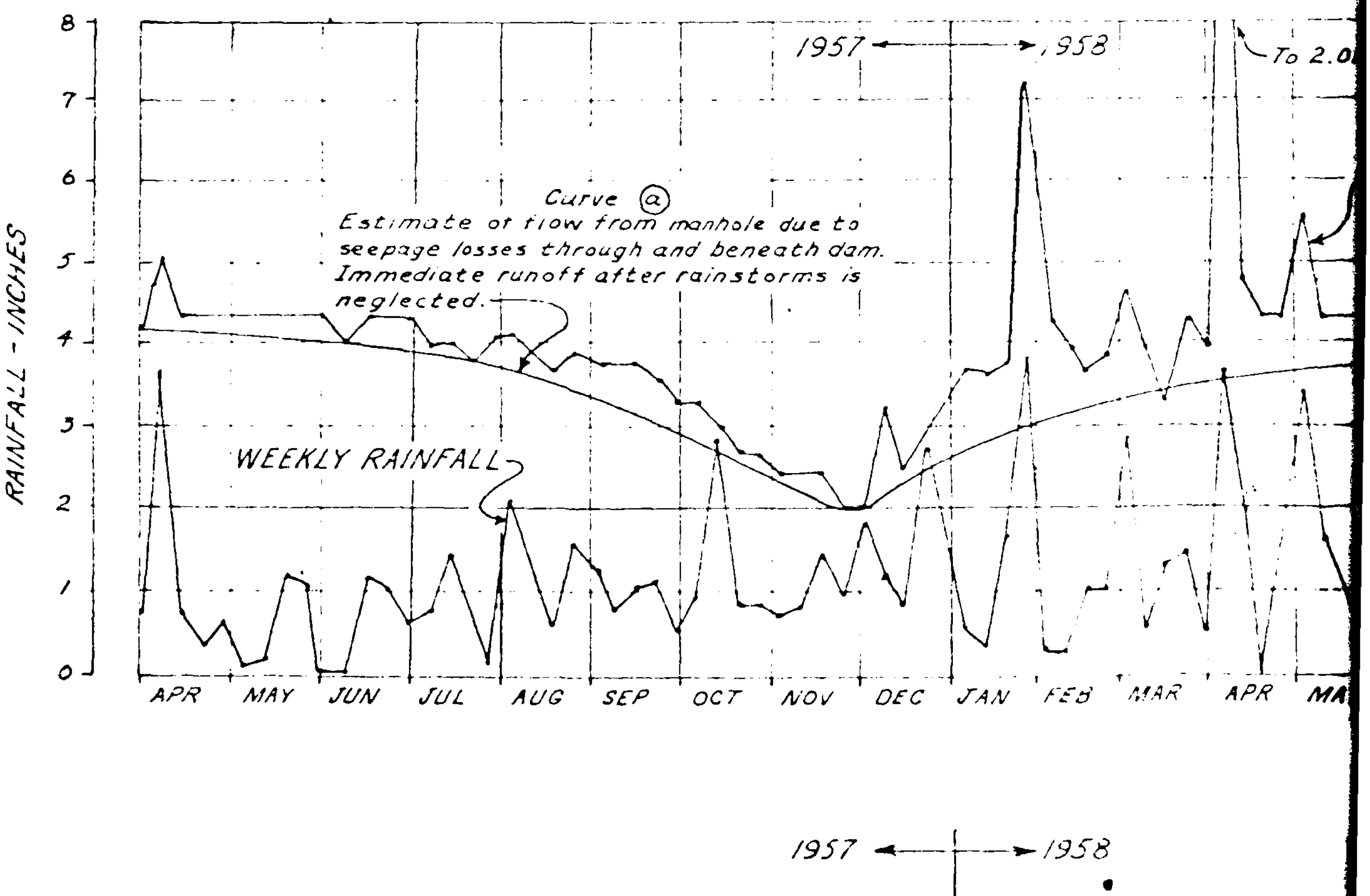
SCALE AS SHOWN

FEBRUARY 1959

FIGURE 2



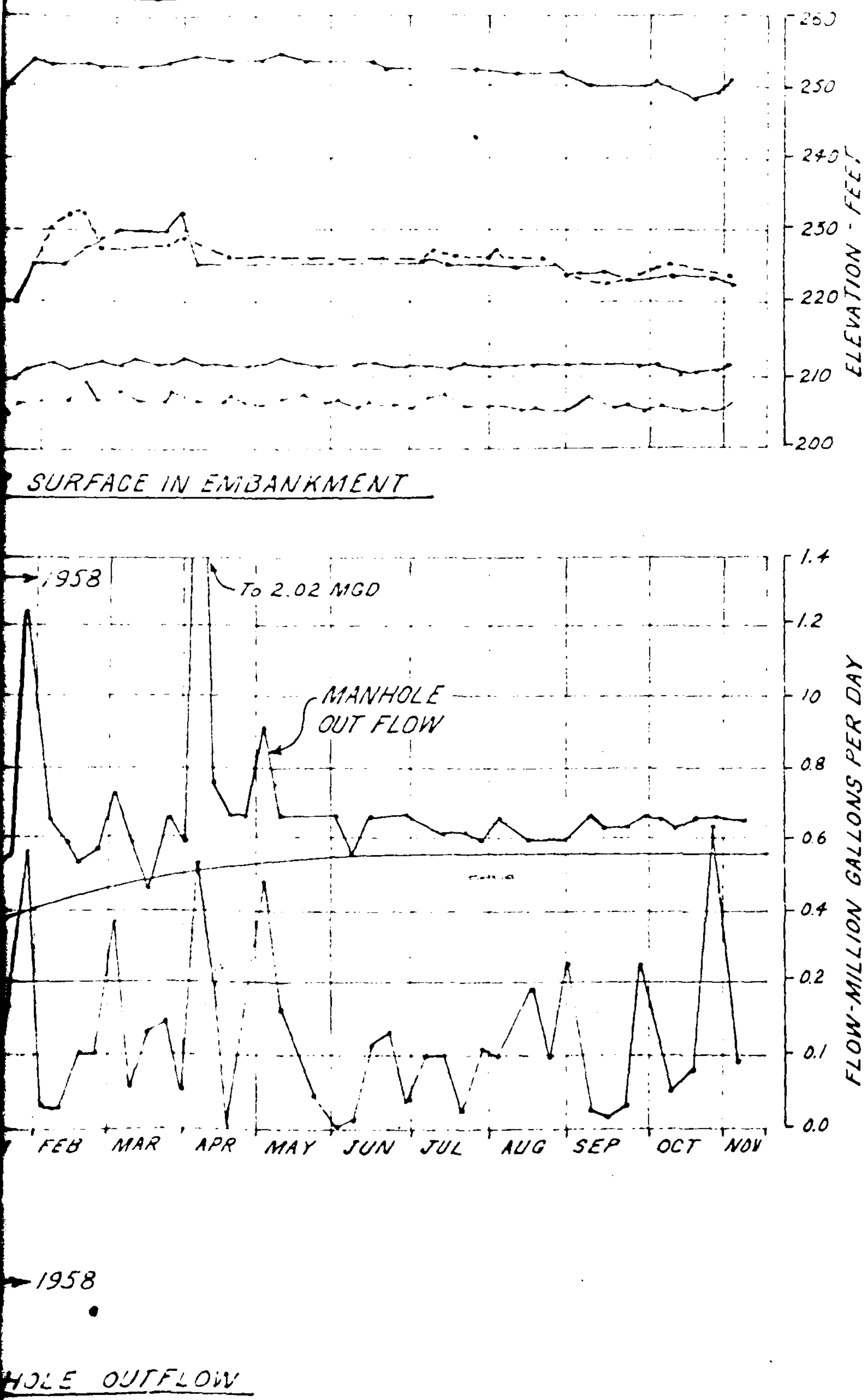
OBSERVATIONS OF FREE WATER SURFACE IN EMBAN



RAINFALL AND MANHOLE OUTFLOW

58-211

HALEY & ALDRICH



NOTES:

1. Data plotted hereon is obtained from field data sheets made available by the Water Resources Commission of Connecticut. Data was taken by the Greenwich Water Company

2. See Figure 4 for location of standpipes.

MIANUS RIVER DAM
STAMFORD, CONNECTICUT

PLOT OF PREVIOUS FIELD MEASUREMENTS

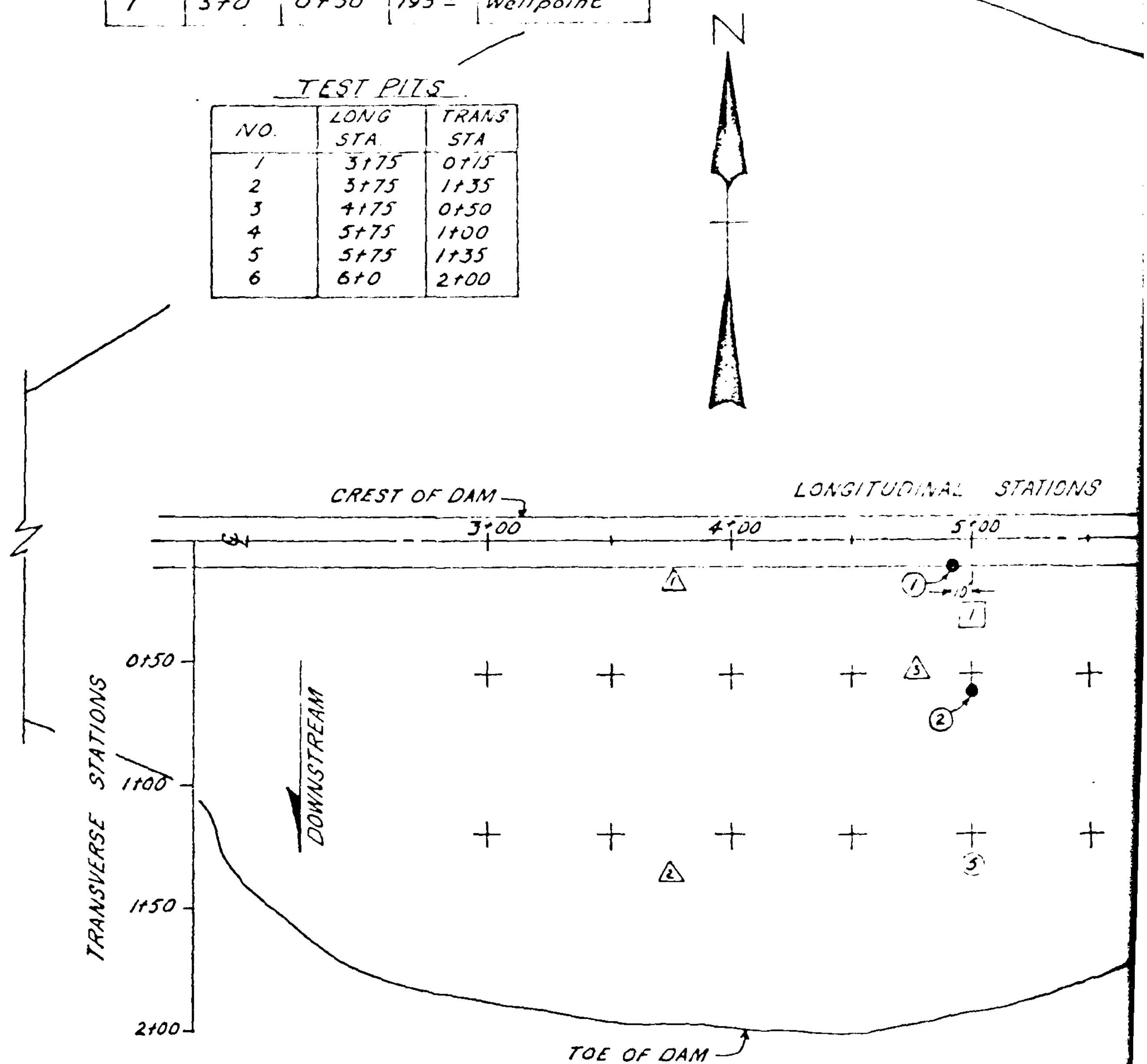
FEBRUARY 1959

WATER OBSERVATIONS

NO.	LONG. STA.	TRANS STA.	BOT ELEV.	REMARKS
5	5+0	1+30	195	Standpipe
6	6+0	1+10	203	Standpipe
1	5+0	0+30	193 ±	Wellpoint

TEST PITS

NO.	LONG STA.	TRANS STA
1	3+75	0+15
2	3+75	1+35
3	4+75	0+50
4	5+75	1+00
5	5+75	1+35
6	6+0	2+00

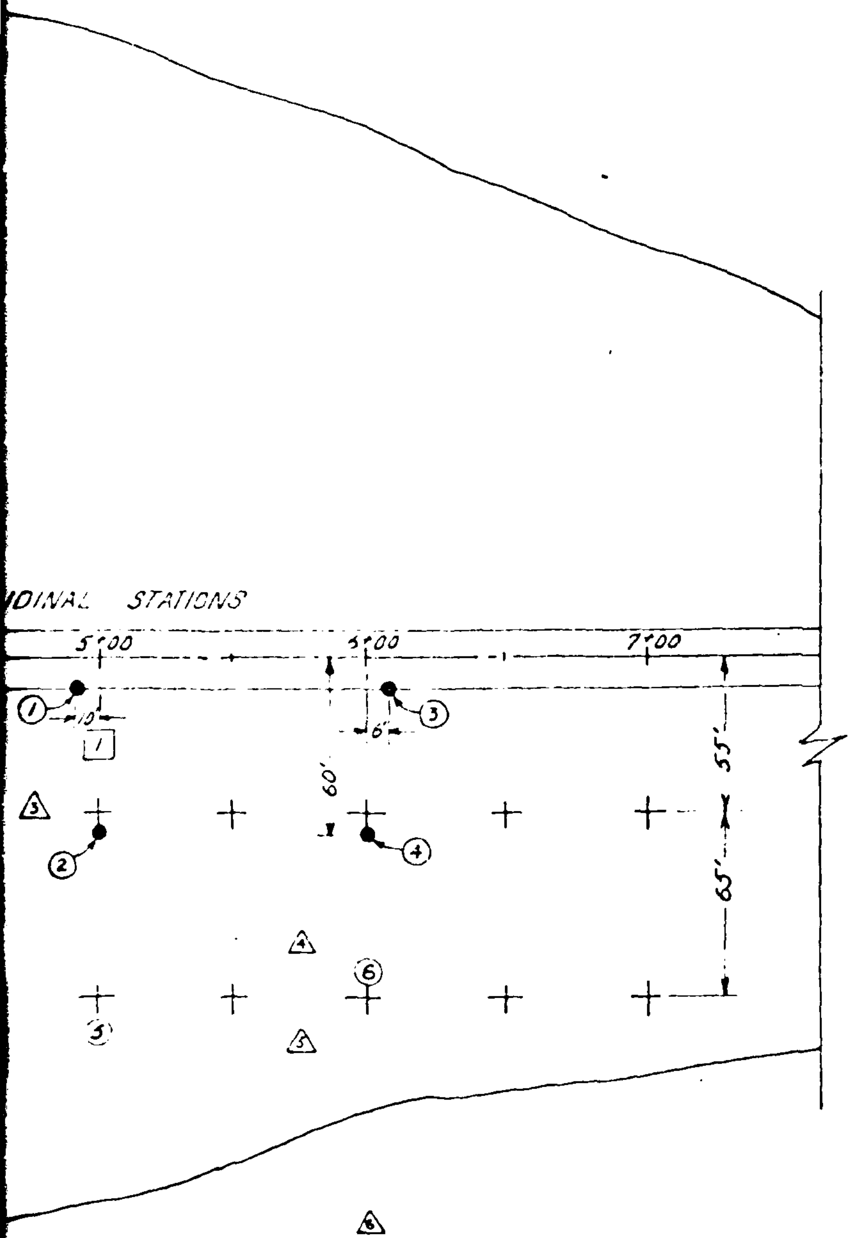


PLAN VIEW OF DAM

Scale: 1" = 60'

58-211

HALEY & ALDRICH



IEW OF DAM
1" = 60'

NOTES:

1. See Figure 5 for details of wellpoint and displacement stakes.
2. Wellpoint is to be installed in graded filter.
3. See text for description of test pits
4. Test pit No 5 is to be driven into rock toe so that rock is exsposed.
5. Test pit No 6 may be relocated as necessary to obtain representative samples of the foundation material.
6. Displacement stakes in each row are spaced 50' on center.

LEGEND:

- Wellpoint
- Test Pits
- New Standpipes
- + Displacement stakes
- ● Existing Standpipes

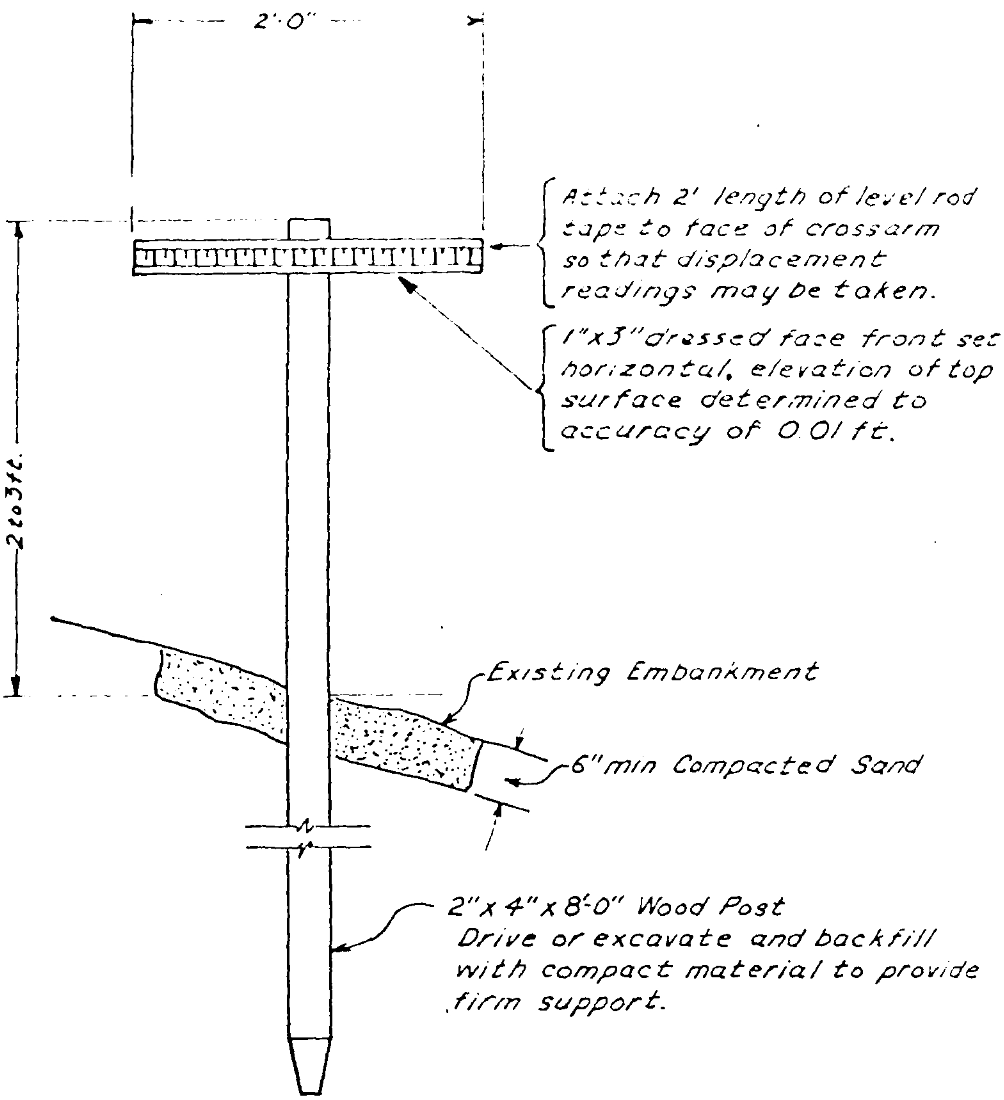
MIANUS RIVER DAM

STAMFORD, CONN.

LOCATION PLAN

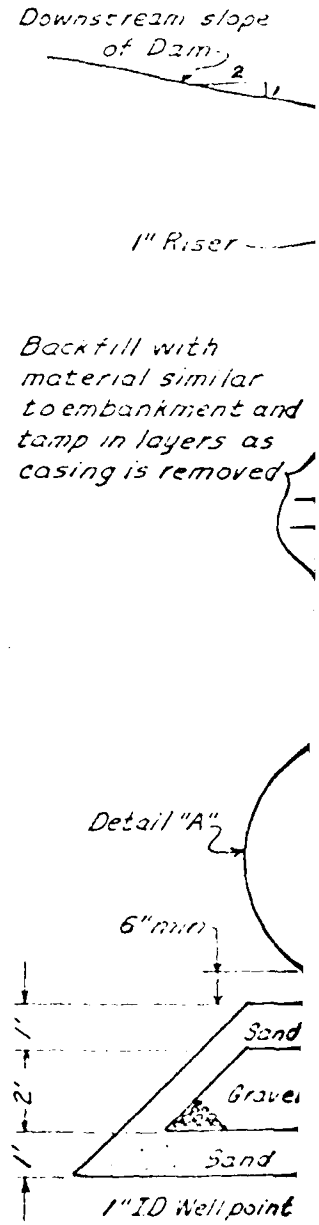
FIELD INVESTIGATIONS

FEBRUARY 1959



TYPICAL DISPLACEMENT STAKE

(Not to Scale)

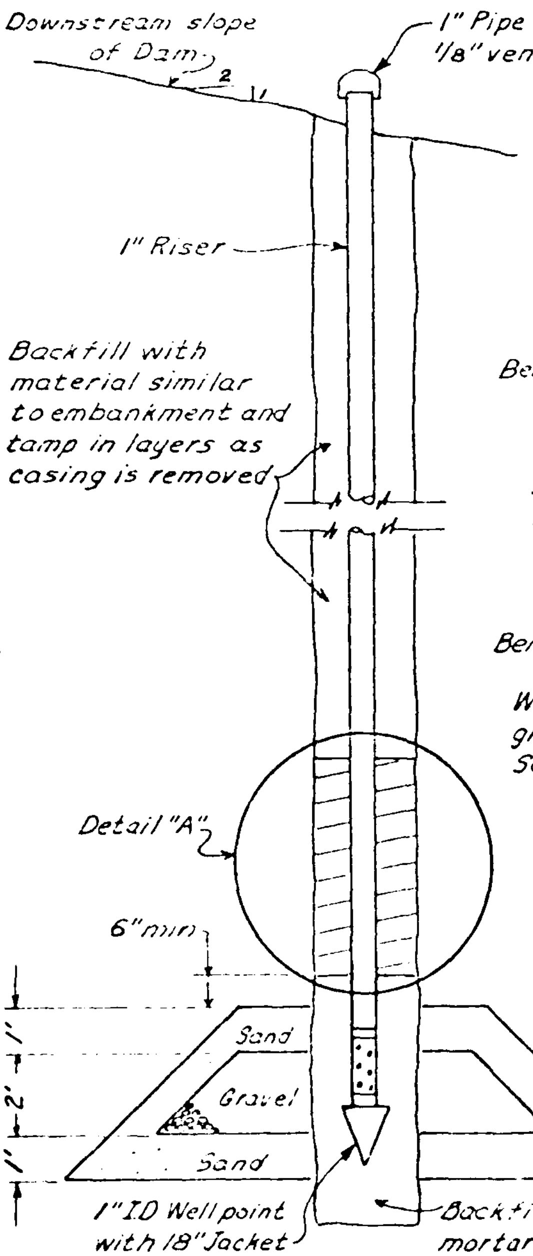


WELL

58-211

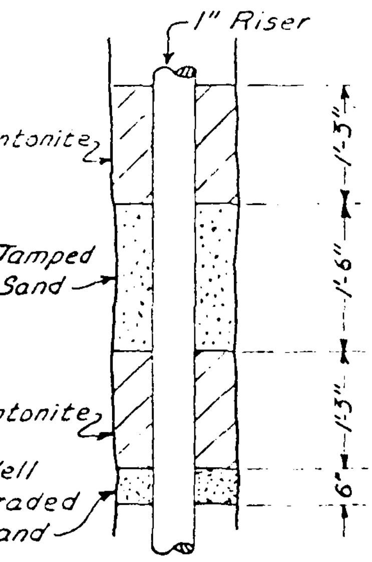
NOTES:

1. Bentonite compacted in 3" layers with 3/4" layer of pea stone on top of each layer to prevent Bentonite from sticking to tamping head. Bentonite formed in 3/8" balls near plastic limit and dropped into drive casing. Bentonite seal placed as casing is withdrawn.
2. Drive wellpoint into graded filter so that gravel surrounds jacket.



WELLPOINT INSTALLATION

(Not to Scale)



BENTONITE SEAL

Detail "A"

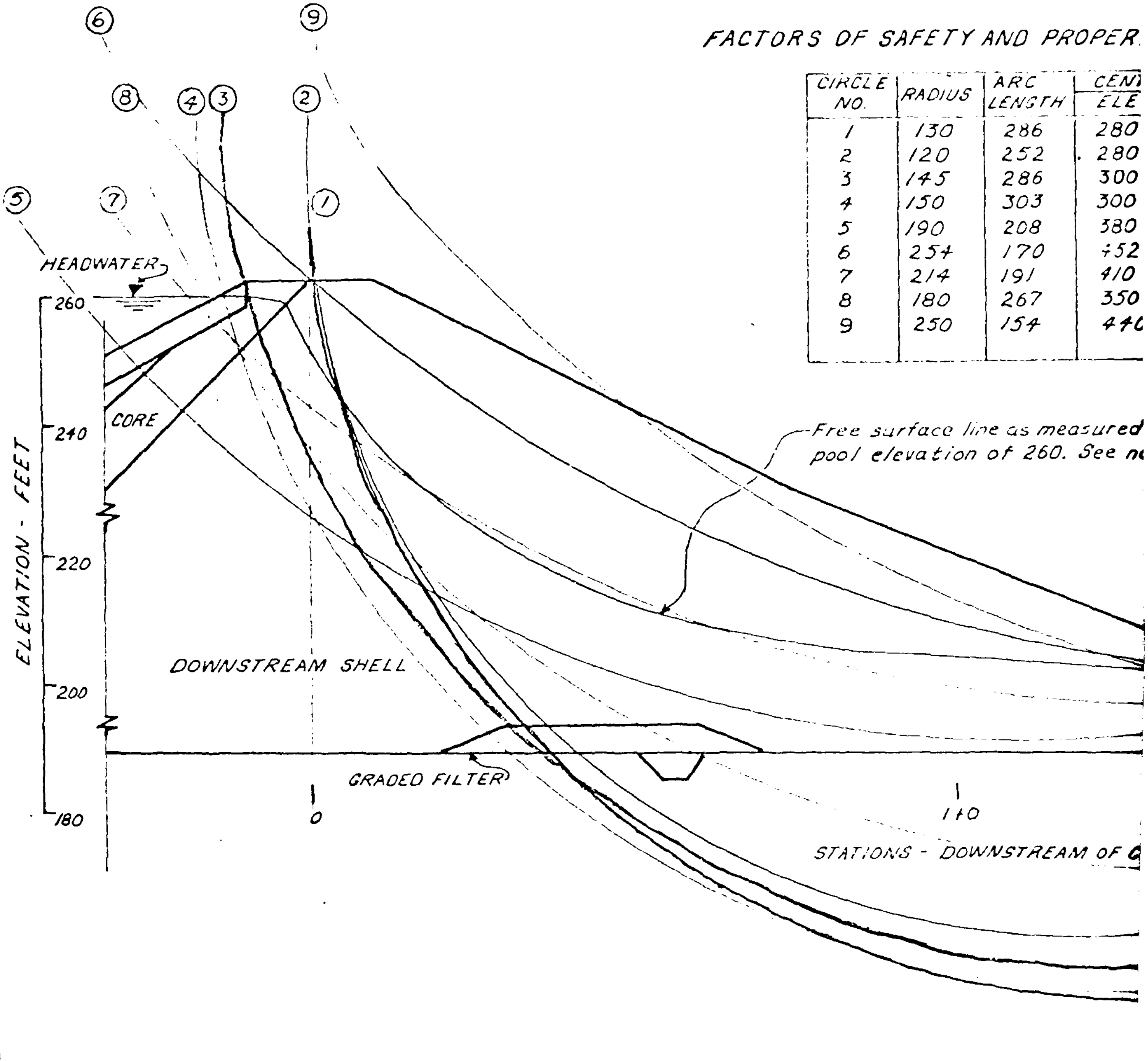
MIANUS RIVER DAM
STAMFORD, CONN.

DISPLACEMENT STAKES AND WELLPOINT INSTALLATION

FEBRARY 1959

FACTORS OF SAFETY AND PROPER

CIRCLE NO.	RADIUS	ARC LENGTH	CEN ELE
1	130	286	280
2	120	252	280
3	145	286	300
4	150	303	300
5	190	208	380
6	254	170	452
7	214	191	410
8	180	267	350
9	250	154	44[illegible]



TRIAL CIRCLES

CROSS SECTION AT STA. 5+0

SCALE: 1" = 20'

58-211

HALEY & ALDRICH

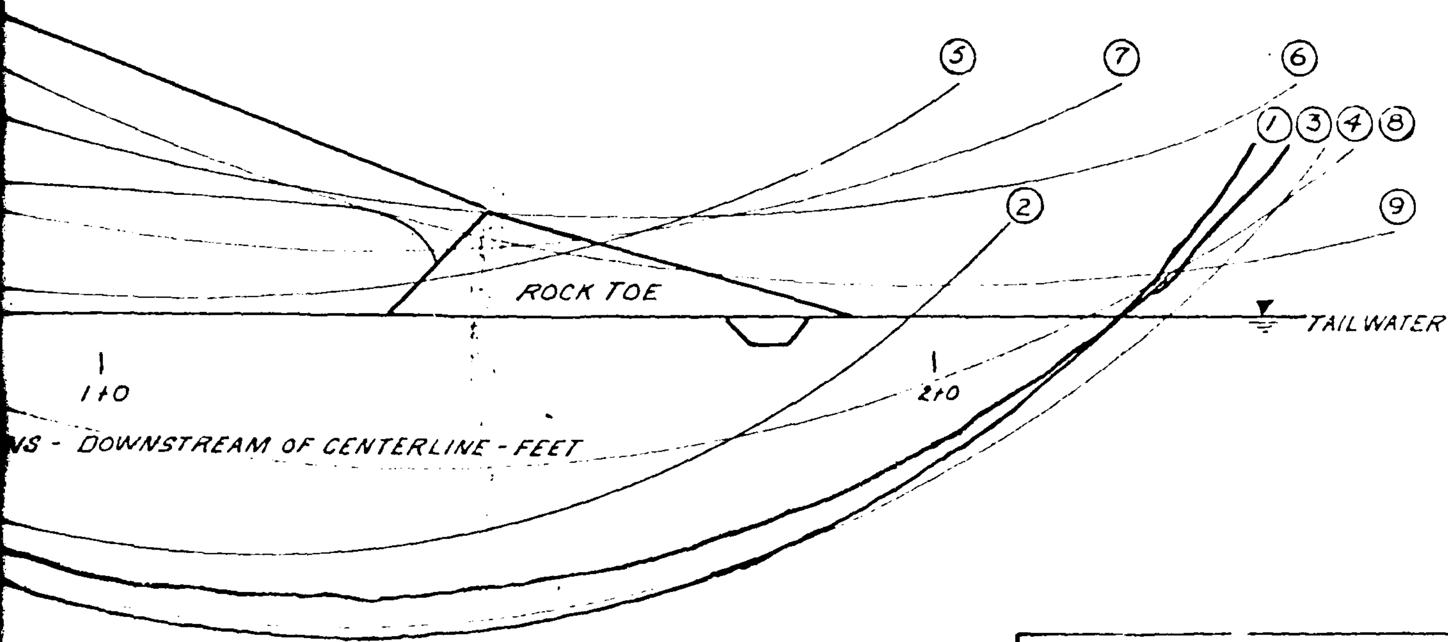
SAFETY AND PROPERTIES OF TRIAL CIRCLES

RADIUS	ARC LENGTH	CENTER LOCATION ELEV.	CENTER LOCATION STA.	FACTOR OF SAFETY
130	286	280	1+30	1.45
120	252	280.	1+20	1.57
145	286	300	1+30	1.45.
150	303	300	1+30	1.49.
190	208	380	1+10	1.94
254	170	452	1+66	1.61
214	191	410	1+34	1.68
180	267	350	1+40	1.55
250	154	440	2+00	1.82

NOTES:

1. See Figure 2 for stability analysis of the critical circle, No 3.
2. See Figure 1 for flow net used in all trials
3. Factors of safety are computed for case of zero cohesion and friction angle of 35°
4. Stations for locating centers of trial circle are measured downstream of dam centerline.

urface line as measured and modified for
levation of 260. See notes of Figure 1.



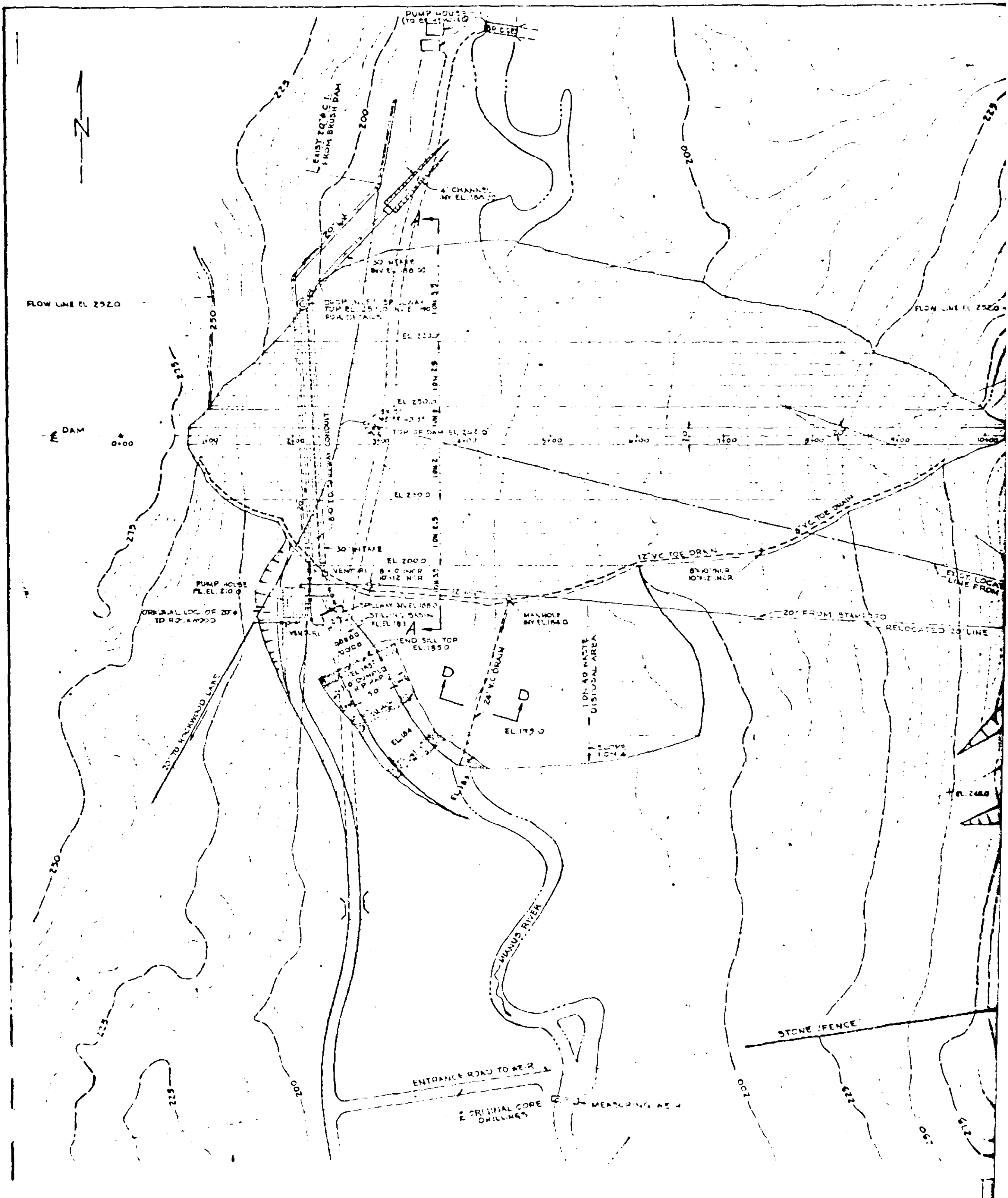
IAL CIRCLES
ON AT STA. 5+0
: 1" = 20'

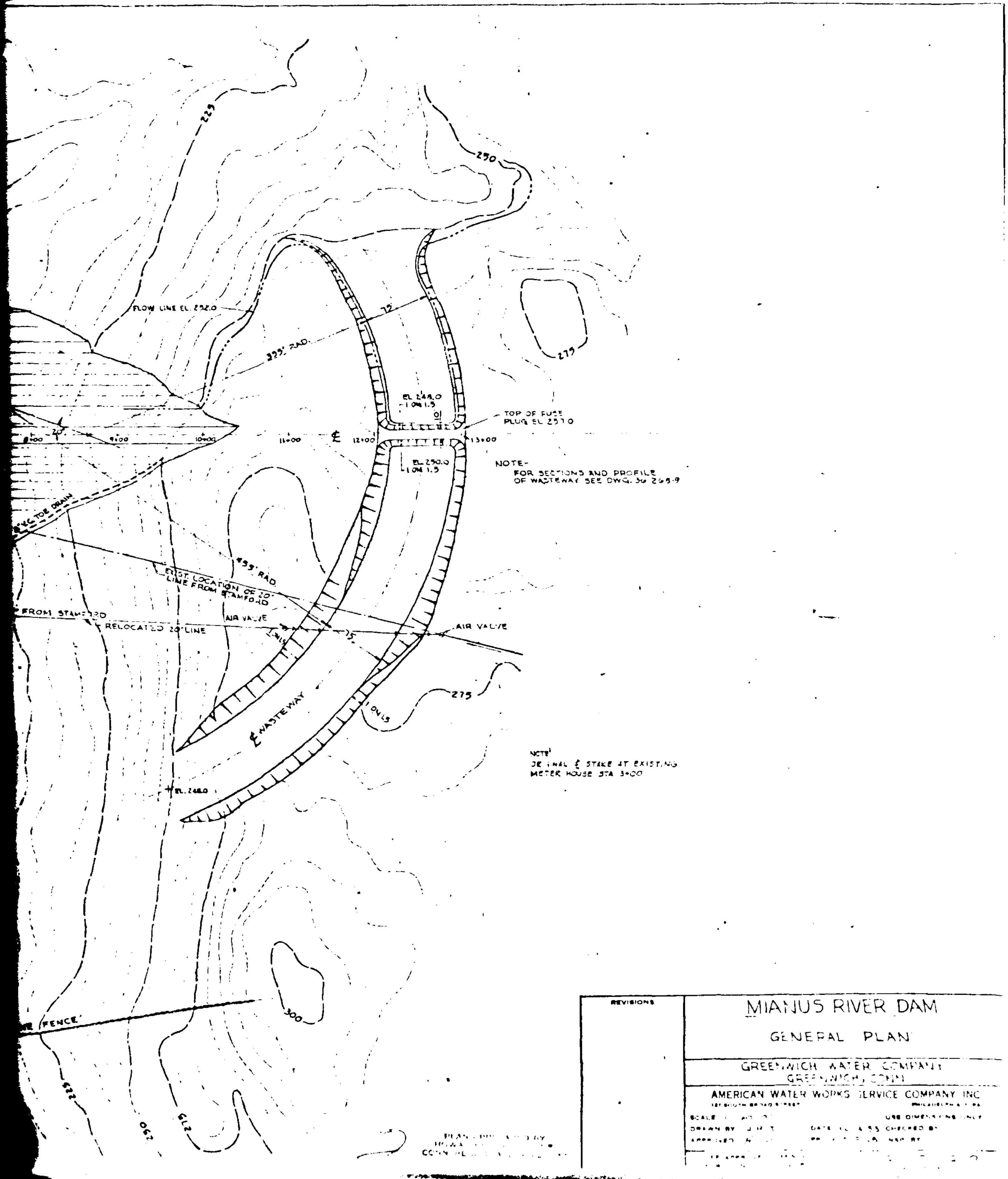
MIANUS RIVER DAM
STAMFORD, CONNECTICUT

TRIAL CIRCLES
FOR
STABILITY ANALYSIS

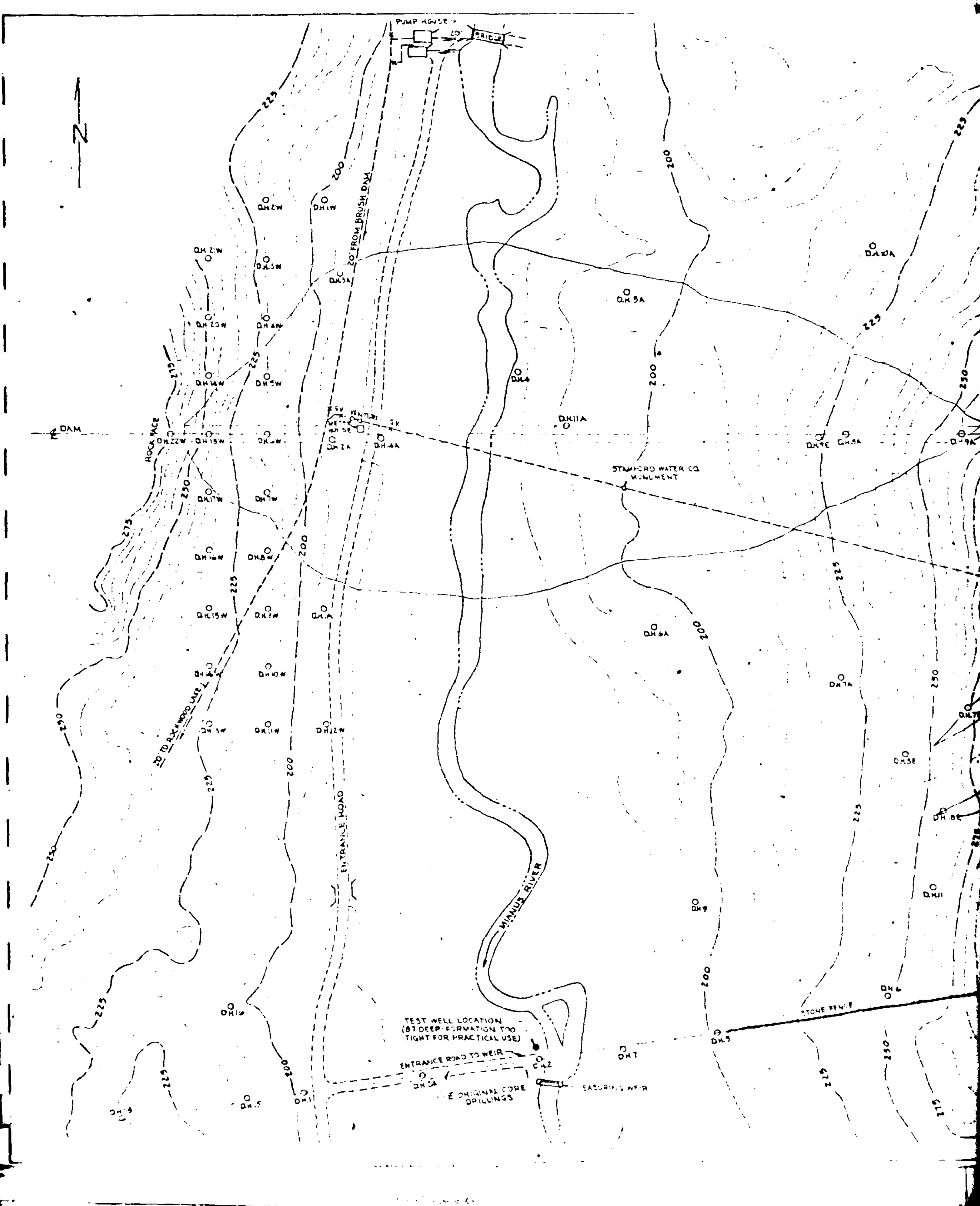
FEBRUARY 1959

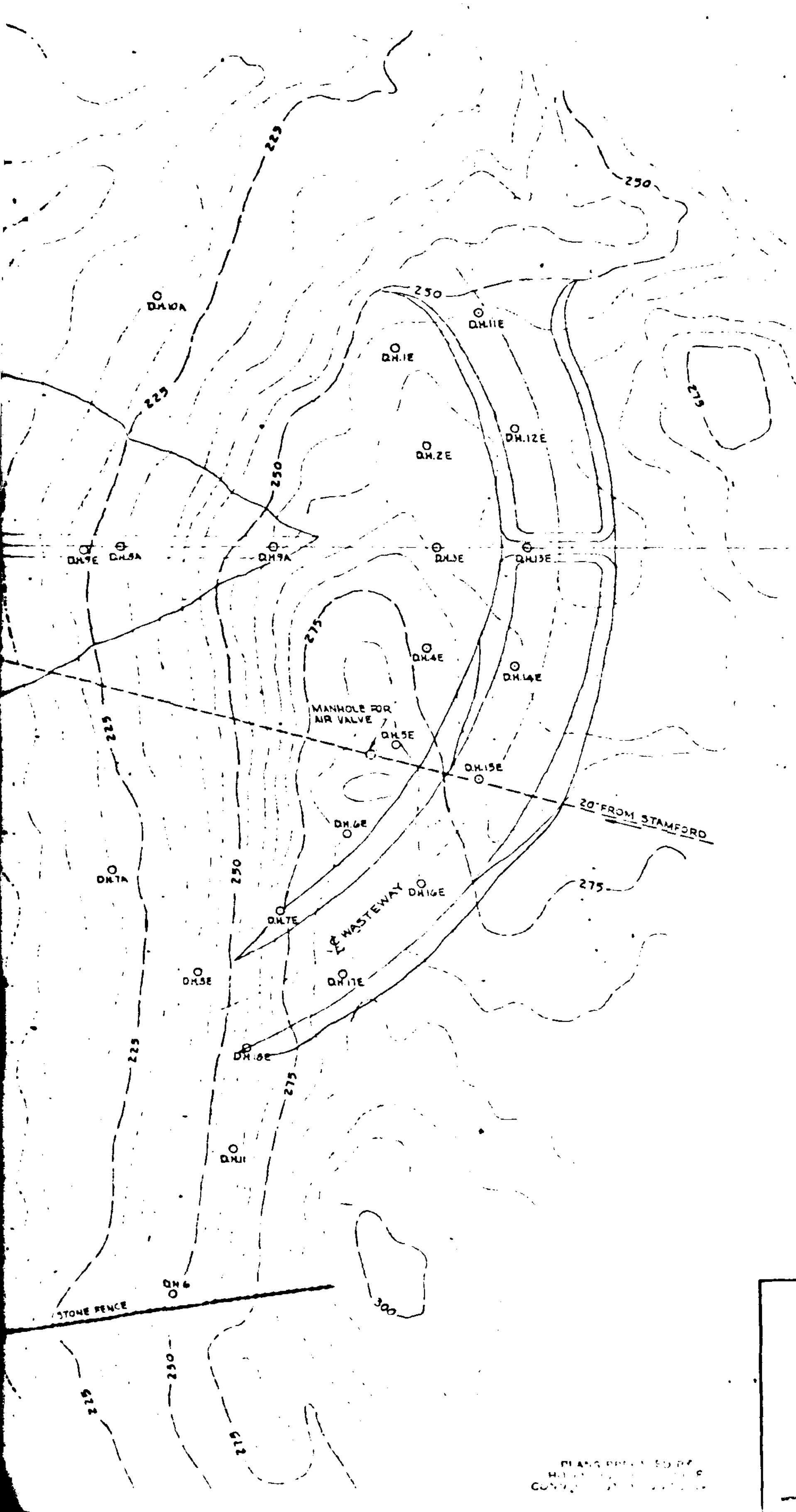
FIGURE


PUMP HOUSE
FLOW LINE EL. 252.0
DAM
0+00
EL. 250.0
EL. 230.0
30" INTAKE
EL. 200.0
VENTURI
PUMP HOUSE
FL. EL. 210.0
STILLING BASIN
12" V.C. TOE DRAIN
8" V.C. TOE DRAIN
MANHOLE
INV. EL. 184.0
20" FROM STAMFORD
RELOCATED 20" LINE
24" V.C. DRAIN
EL. 199.0
30" TO ROCKWOOD LAKE
MIANUS RIVER
STONE FENCE
ENTRANCE ROAD TO WEIR
MEASURING WEIR



FLOW LINE EL. 252.0
225
250
275
329' RAD.
EL. 248.0
1 ON 1.5
TOP OF FUSE PLUG EL. 257.0
EL. 250.0
1 ON 1.5
11+00
12+00
13+00
9+00
10+00
NOTE-
FOR SECTIONS AND PROFILE OF WASTEWAY SEE DWG. 36 265-9
TOE DRAIN
453' RAD.
EXIST. LOCATION OF 20" LINE FROM STAMFORD
FROM STAMFORD
AIR VALVE
RELOCATED 20" LINE
AIR VALVE
1 ON 1.5
℄ WASTEWAY
275
EL. 248.0
300
FENCE
REVISIONS
MIANUS RIVER DAM
GENERAL PLAN
GREENWICH WATER COMPANY
GREENWICH, CONN.
AMERICAN WATER WORKS SERVICE COMPANY INC


PUMP HOUSE
20"
BRIDGE
N
225
200
20" FROM BRUSH DAM
D.H.2W
D.H.1W
D.H.21W
D.H.3W
D.H.3A
D.H.20W
D.H.4W
D.H.14W
D.H.5W
275
DAM
ROCK FACE
D.H.22W
D.H.19W
D.H.6W
VENTURI
METER HOUSE
D.H.2A
D.H.4A
250
D.H.17W
D.H.7W
D.H.16W
D.H.8W
D.H.15W
D.H.9W
D.H.1A
D.H.10W
20" TO ROCKWOOD LAKE
D.H.13W
D.H.11W
D.H.12W
ENTRANCE ROAD
MIANUS RIVER
D.H.5A
D.H.4
D.H.11A
STAMFORD WATER CO. MONUMENT
D.H.10A
D.H.9E
D.H.8A
D.H.9A
D.H.6A
D.H.7A
D.H.5E
D.H.8E
D.H.11
D.H.9
D.H.6
STONE FENCE
TEST WELL LOCATION
(87' DEEP FORMATION TOO
TIGHT FOR PRACTICAL USE)
ENTRANCE ROAD TO WEIR
D.H.3
D.H.7
D.H.2
D.H.1
D.H.5
ORIGINAL CORE
DRILLINGS



TEST HOLE DATA

NO	GROUND ELEV	DEPTH TO ROCK	ROCK ELEV
1A	192.25	28	164.25
2A	192.12	23	169.12
3A	193.50	NOT	DRILLED
4A	185.12	30	155.72
5A	197.78	46	151.78
6A	198.08	45	153.08
7A	220.42	18	202.42
8A	229.50	20	209.50
9A	257.10	15	242.10
10A	216.31	16	200.31
11A	196.11	51	145.11
4	190.8	73	117.8

NO.	GROUND ELEV.	DEPTH TO ROCK	ROCK ELEV.
1	195.9'	29'	166.9
2	186.9'	93'	93.9'
5	202.1'	5.3'	196.8
6	250.9	4'	246.9
7	186.5'	81'	105.5'
9	197.7'	13'	NOT TO ROCK
11	263.9	10	253.9
15	216.7'	16	200.7
16	209.7'	11'	198.7'
18	223.5'	11	212.5
24	188.0	57.5	130.5

NO.	GROUND ELEV.	DEPTH TO ROCK	ROCK ELEV
1W	195.80'	32'	166.8'
2W	210.54'	13	197.54'
3W	216.24'	12'	204.24
4W	218.69'	11'	207.69'
5W	221.29'	14'	207.29'
6W	212.90'	16	196.9'
7W	220.39'	10'	210.34'
8W	221.81'	15'	206.81'
9W	221.95'	25	196.95'
10W	217.95'	27	190.95'
11W	215.44'	30'	185.44'
12W	193.30'	46	147.30'
13W	224.68'	13	211.68'
14W	226.18	9'	217.18'
15W	227.58'	20'	207.58'
16W	229.66'	15	214.66'
17W	234.61'	21	213.61'
18W	244.55'	NONE	
19W	247.35'	7	240.35
20W	246.90	7	239.90
21W	241.25'	13	228.25'
22W	273.20	0'	ON ROCK

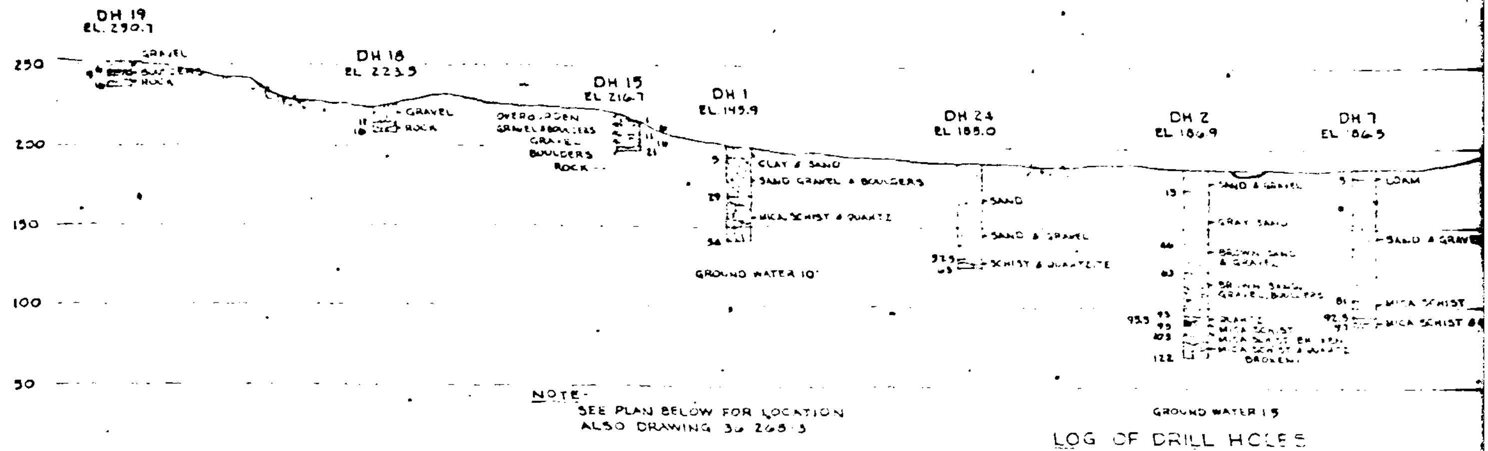
NO.	GROUND ELEV	DEPTH TO ROCK	ROCK ELEV
1E	257.25	13.5'	243.75'
2E	263.10'	13'	250.10
3E	264.07	14	250.07
4E	272.10'	14	258.10
5E	282.20	12	270.20
6E	281.58	16	265.58
7E	273.50'	15	258.50
8E	242.34	14	228.34
9E	225.00	16	209.00
11E	255.28	7.5	247.78
12E	258.35	[illegible]	247.65
13E	260.40	9'	251.40
14E	265.67	14.5	251.17
15E	271.10	17	254.10'
16E	279.55	11	268.55
17E	281.53	9'	272.53
18E	261.34'	9'	252.34'

REVISIONS

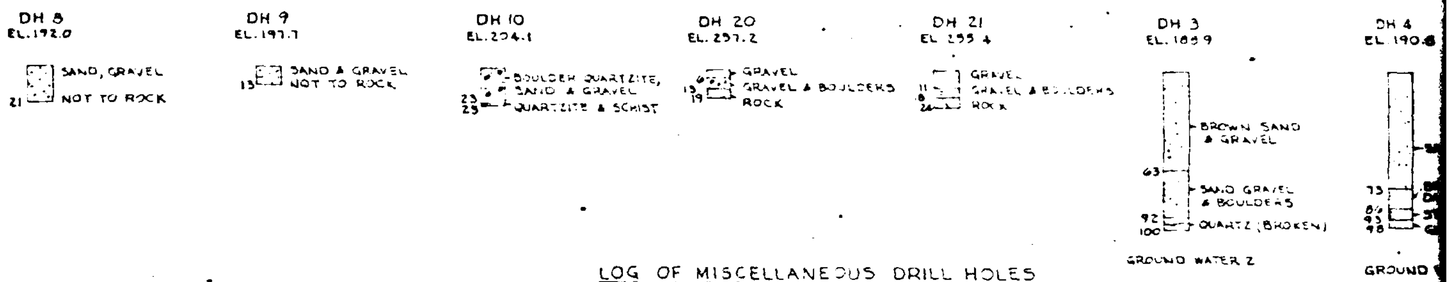
MIANUS RIVER DAM
TEST HOLE DATA AND
EXISTING FACILITIES

GREENWICH WATER COMPANY
GREENWICH, CONN.

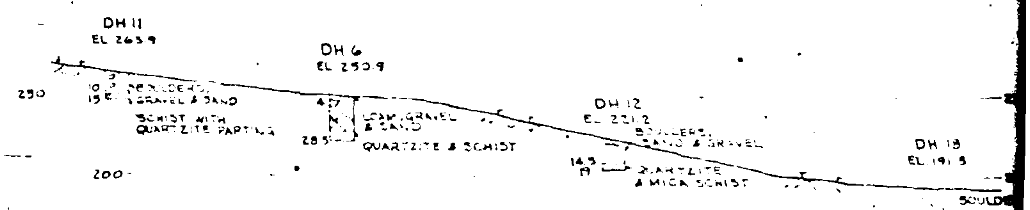
AMERICAN WATER WORKS SERVICE COMPANY INC.
SCALE
DRAWN BY
APPROVED
DATE
CHECKED BY
INSP. BY



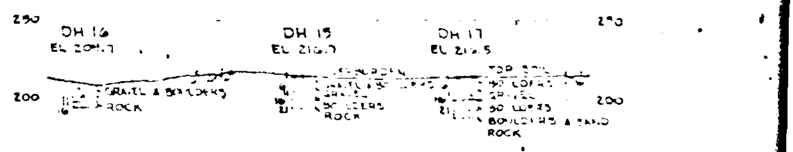
LOG OF DRILL HOLES
ALONG ORIGINAL CENTER LINE
SCALE: 1"=50'



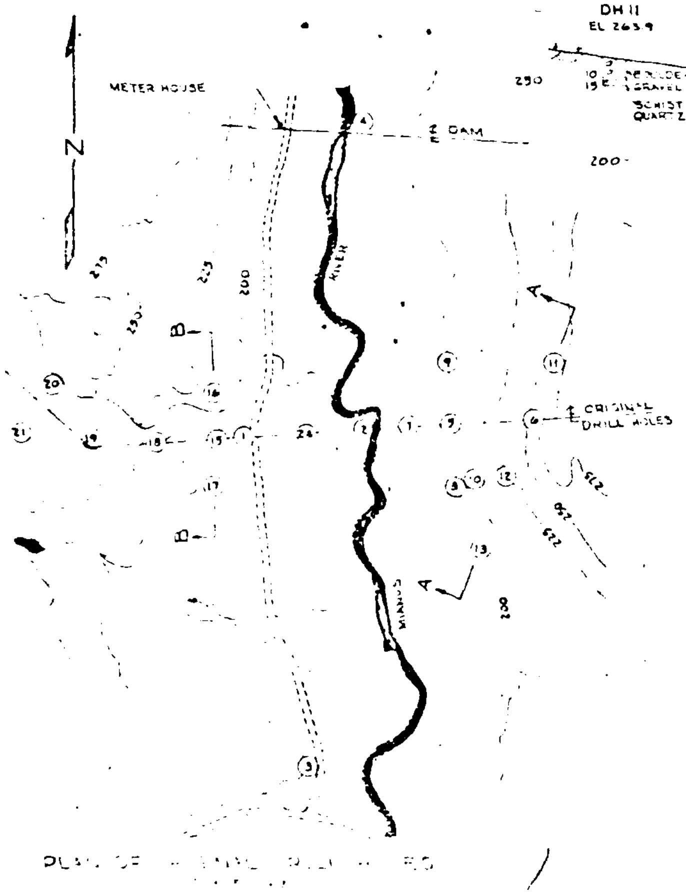
LOG OF MISCELLANEOUS DRILL HOLES
SCALE: 1"=50'



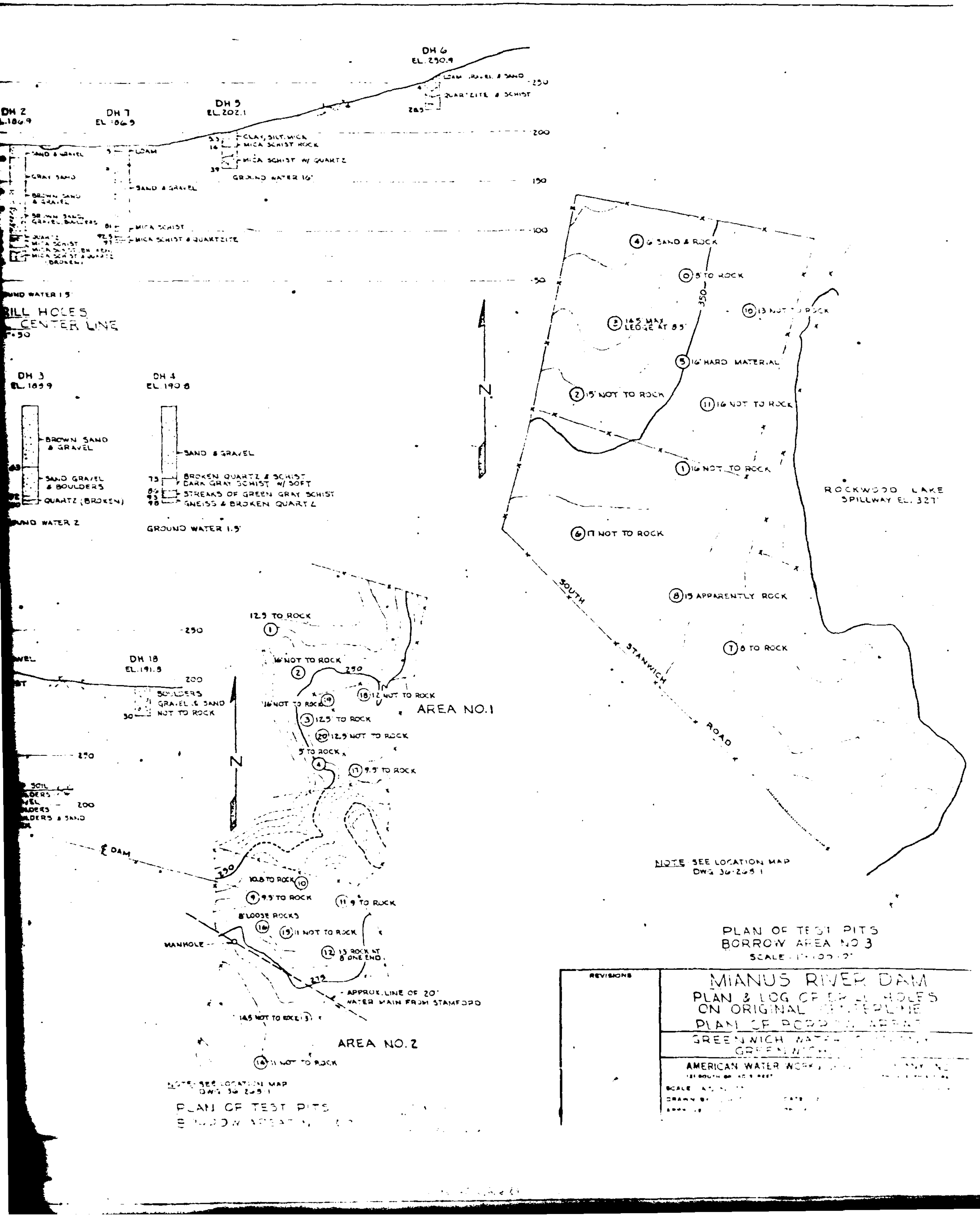
SECTION A-A
SCALE: 1"=50'

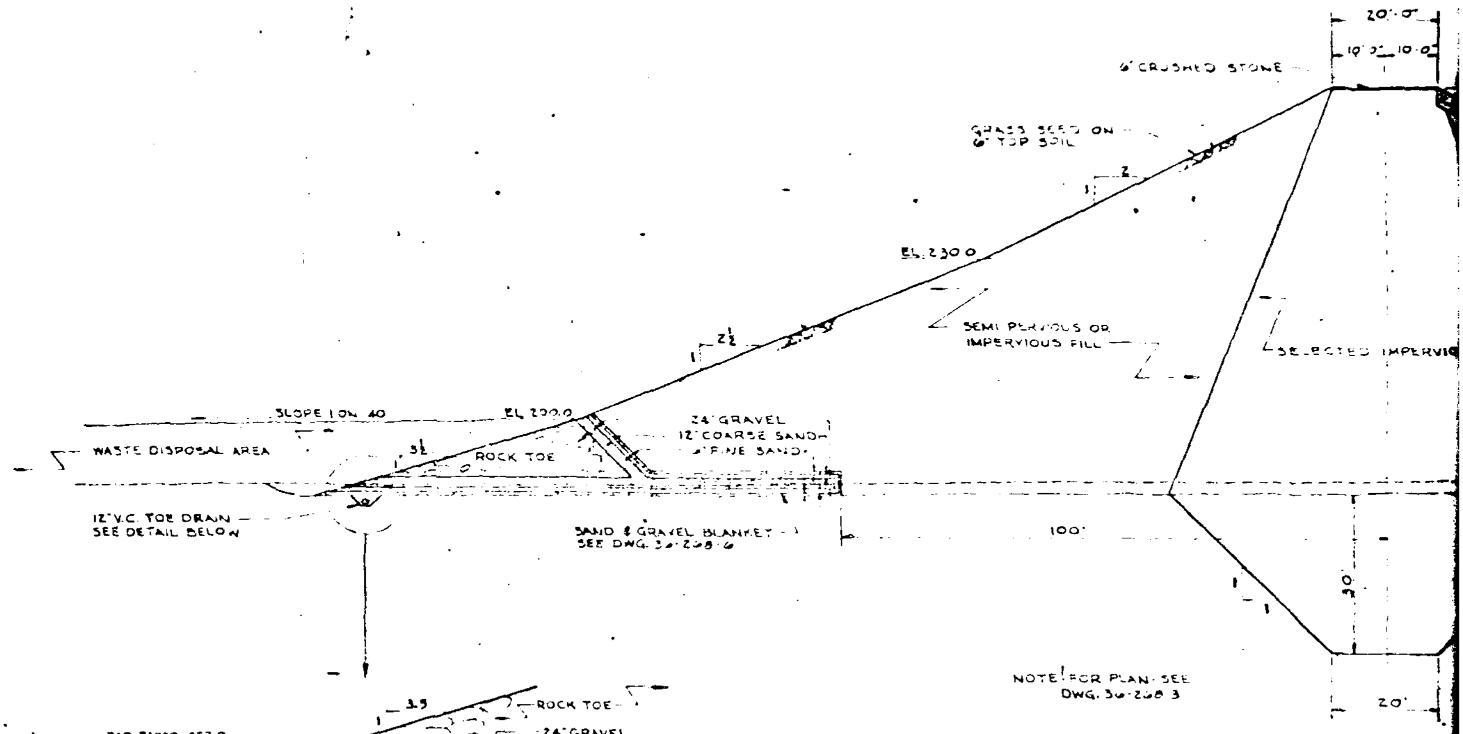


SECTION B-B
SCALE: 1"=50'



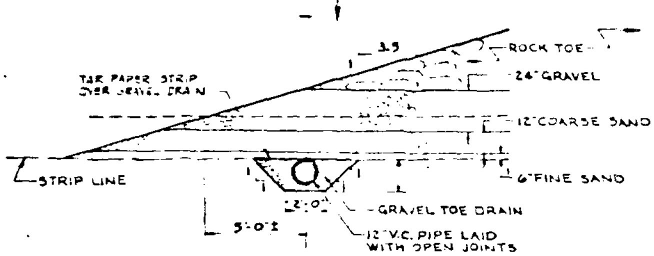
PLAN OF [illegible] DRILL HOLES


DH 6
EL. 250.9
LOAM, GRAVEL & SAND
QUARTZITE & SCHIST
DH 2
EL. 186.9
DH 7
EL. 186.9
DH 5
EL. 202.1
CLAY, SILT, MICA
MICA SCHIST ROCK
MICA SCHIST W/ QUARTZ
GROUND WATER 16'
SAND & GRAVEL
GRAY SAND
BROWN SAND & GRAVEL
LOAM
SAND & GRAVEL
MICA SCHIST
MICA SCHIST & QUARTZITE
DRILL HOLES
CENTER LINE
DH 3
EL. 185.9
BROWN SAND & GRAVEL
SAND GRAVEL & BOULDERS
QUARTZ (BROKEN)
DH 4
EL. 190.8
SAND & GRAVEL
BROKEN QUARTZ & SCHIST
DARK GRAY SCHIST W/ SOFT
STREAKS OF GREEN GRAY SCHIST
GNEISS & BROKEN QUARTZ
GROUND WATER 1.5'
DH 18
EL. 191.5
BOULDERS
GRAVEL & SAND
NOT TO ROCK
N
12.5 TO ROCK
16' NOT TO ROCK
16' NOT TO ROCK
12 NOT TO ROCK
12.5' TO ROCK
12.5 NOT TO ROCK
5' TO ROCK
9.5' TO ROCK
AREA NO. 1
℄ DAM
10.8 TO ROCK
9.5' TO ROCK
9' TO ROCK
8' LOOSE ROCKS
11 NOT TO ROCK
13 ROCK AT 8' ONE END
MANHOLE
APPROX. LINE OF 20" WATER MAIN FROM STAMFORD
14.5 NOT TO ROCK
AREA NO. 2
11 NOT TO ROCK
NOTE: SEE LOCATION MAP DWG 36-265-1
PLAN OF TEST PITS
6' SAND & ROCK
5' TO ROCK
14.5' MAX. LEDGE AT 8.5'
13 NOT TO ROCK
16' HARD MATERIAL
15' NOT TO ROCK
16 NOT TO ROCK
16' NOT TO ROCK
17 NOT TO ROCK
15 APPARENTLY ROCK
8 TO ROCK
SOUTH STANWICH ROAD
ROCKWOOD LAKE
SPILLWAY EL. 327'
NOTE: SEE LOCATION MAP DWG 36-265-1
PLAN OF TEST PITS
BORROW AREA NO 3
REVISIONS
MIANUS RIVER DAM
PLAN & LOG OF DRILL HOLES
ON ORIGINAL CENTERLINE
PLAN OF BORROW AREAS
GREENWICH
AMERICAN WATER WORKS
SCALE
DRAWN BY
DATE

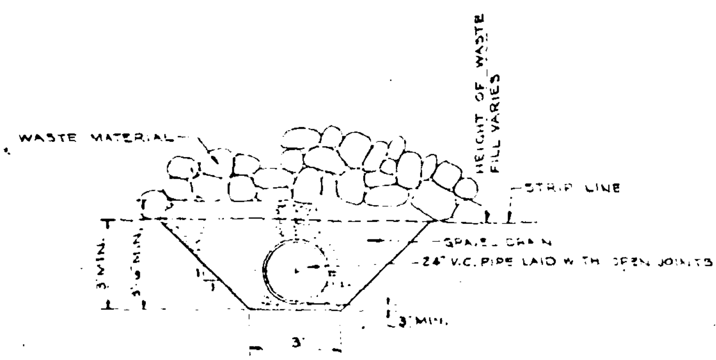


SCALE-1"=15'-0"

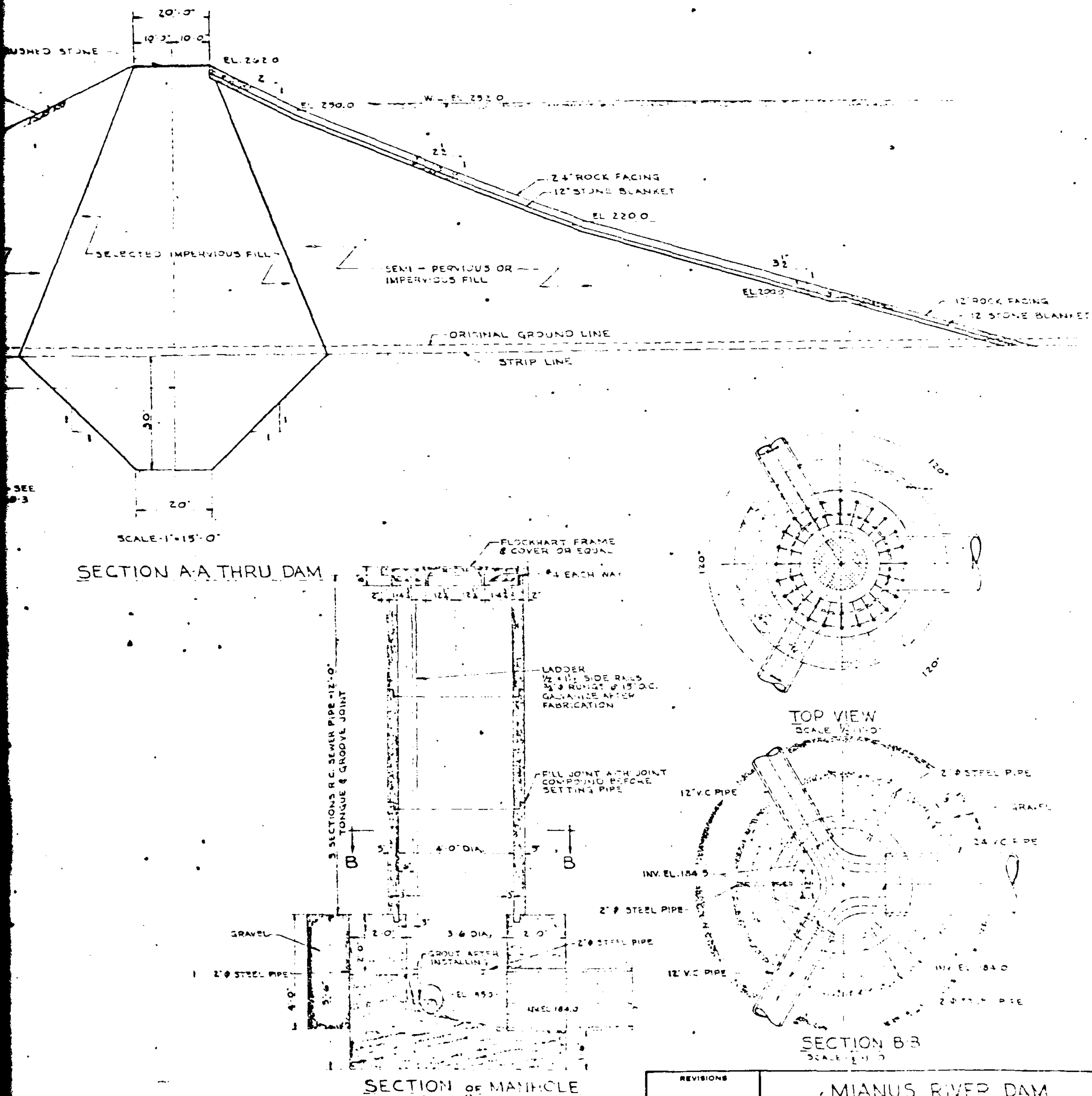
SECTION A-A



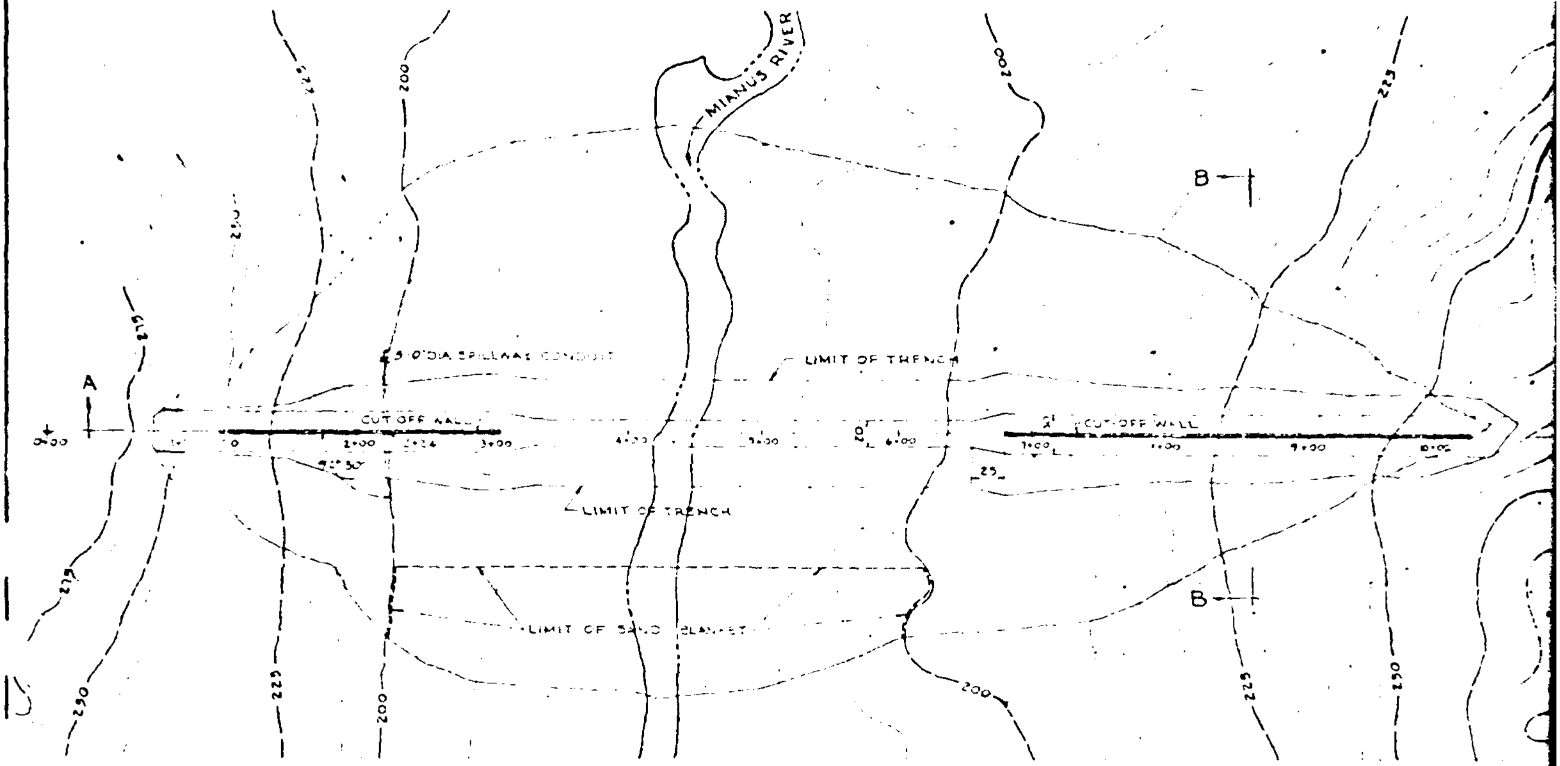
DETAIL OF TOE DRAIN
SCALE-1"=4'-0"



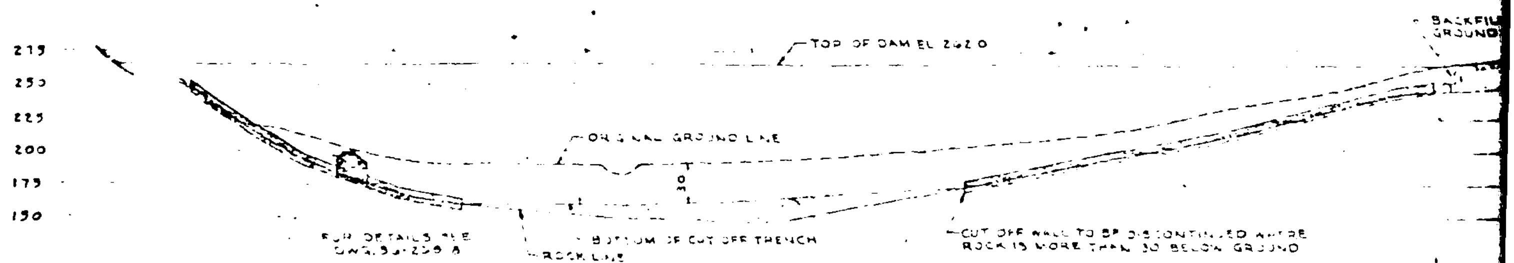
SECTION D-D
SCALE-1/8"=1'-0"



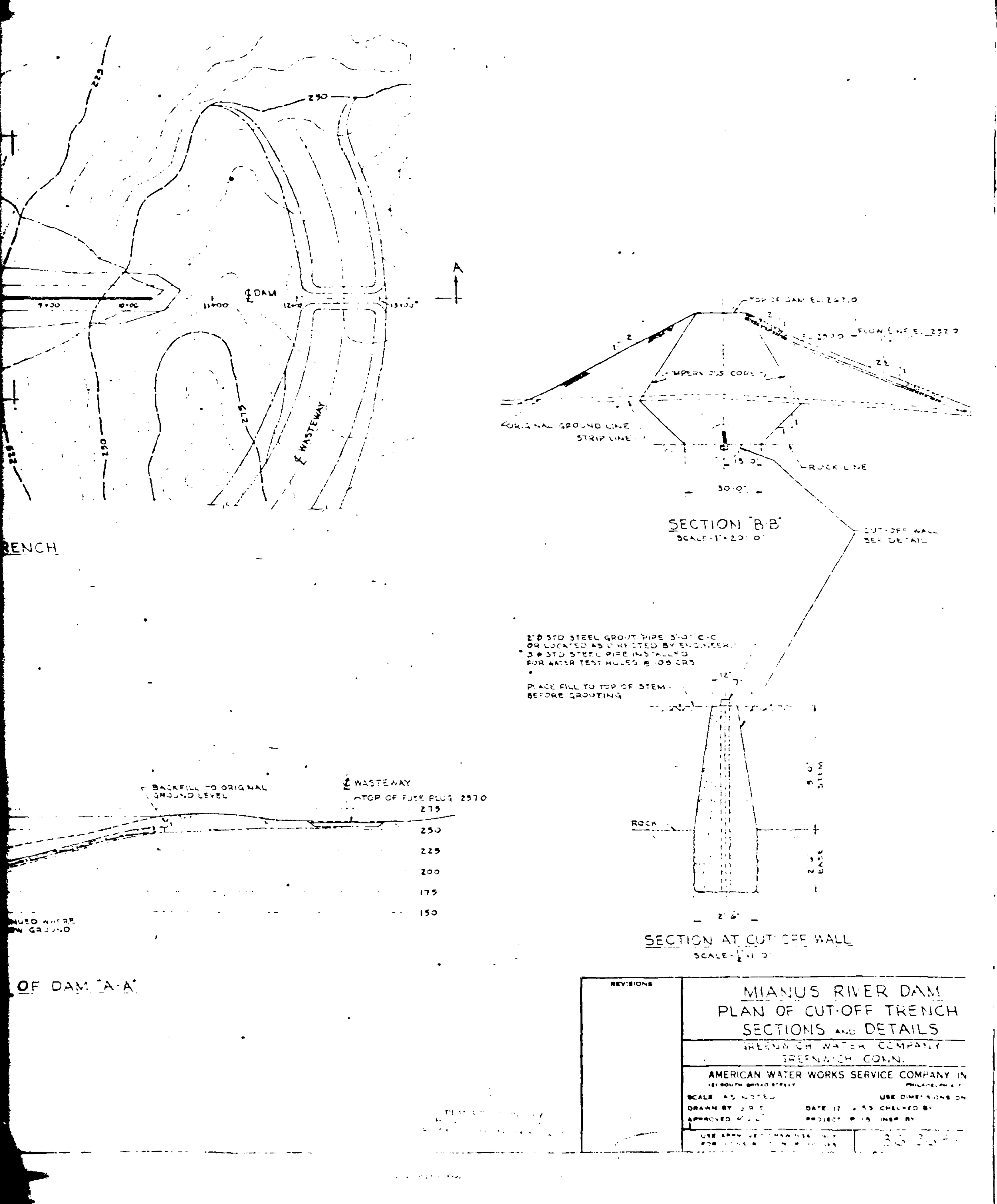
REVISIONS	MIANUS RIVER DAM EMBANKMENT SECTIONS DETAILS
	GREENWICH WATER COMPANY
	AMERICAN WATER WORKS SERVICE COMPANY, INC. PHILADELPHIA, PA.
	SCALE: AS NOTED
	DRAWN BY
	APPROVED



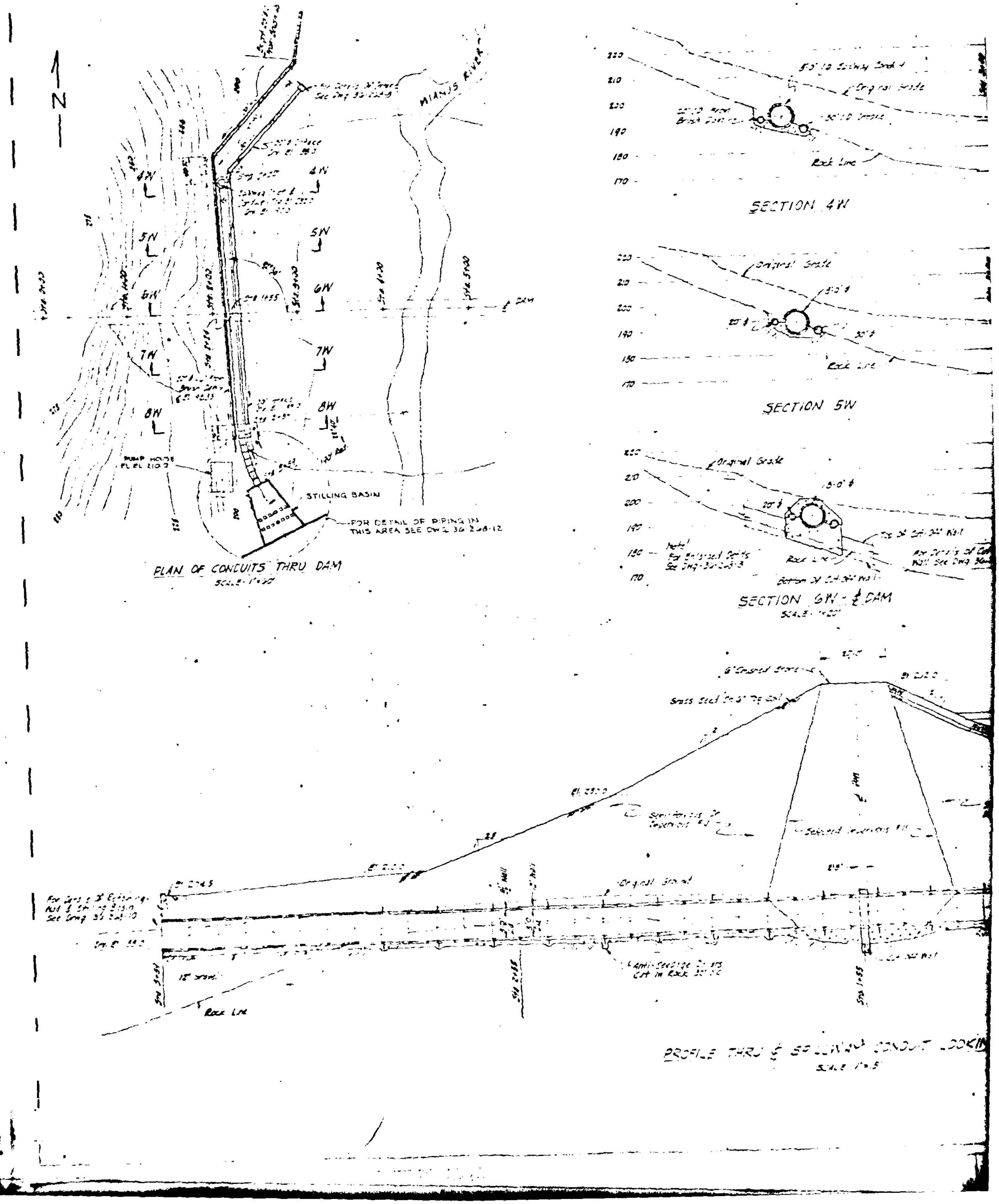
PLAN OF CUT-OFF WALL & TRENCH
SCALE 1"=60'-0"

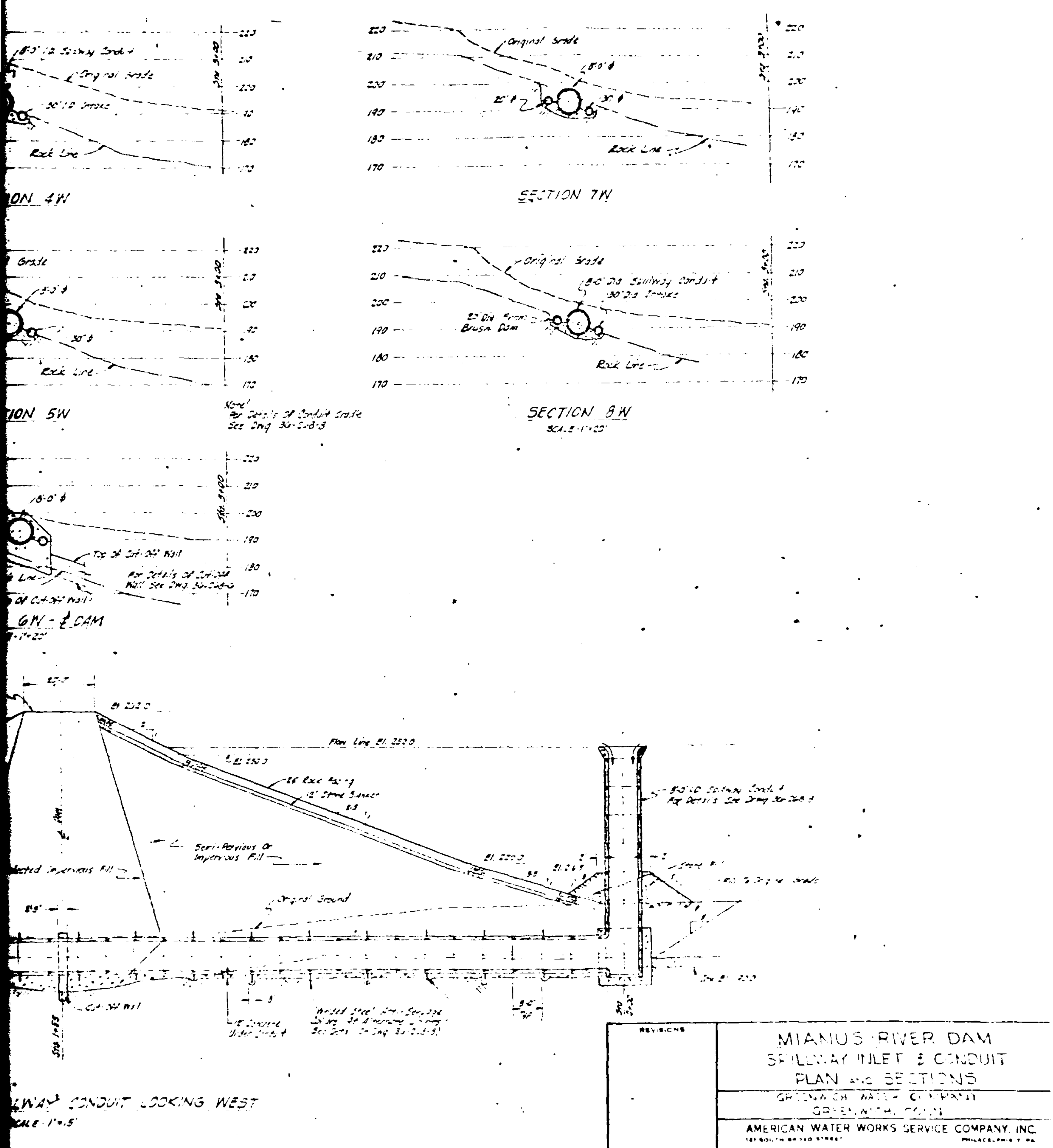


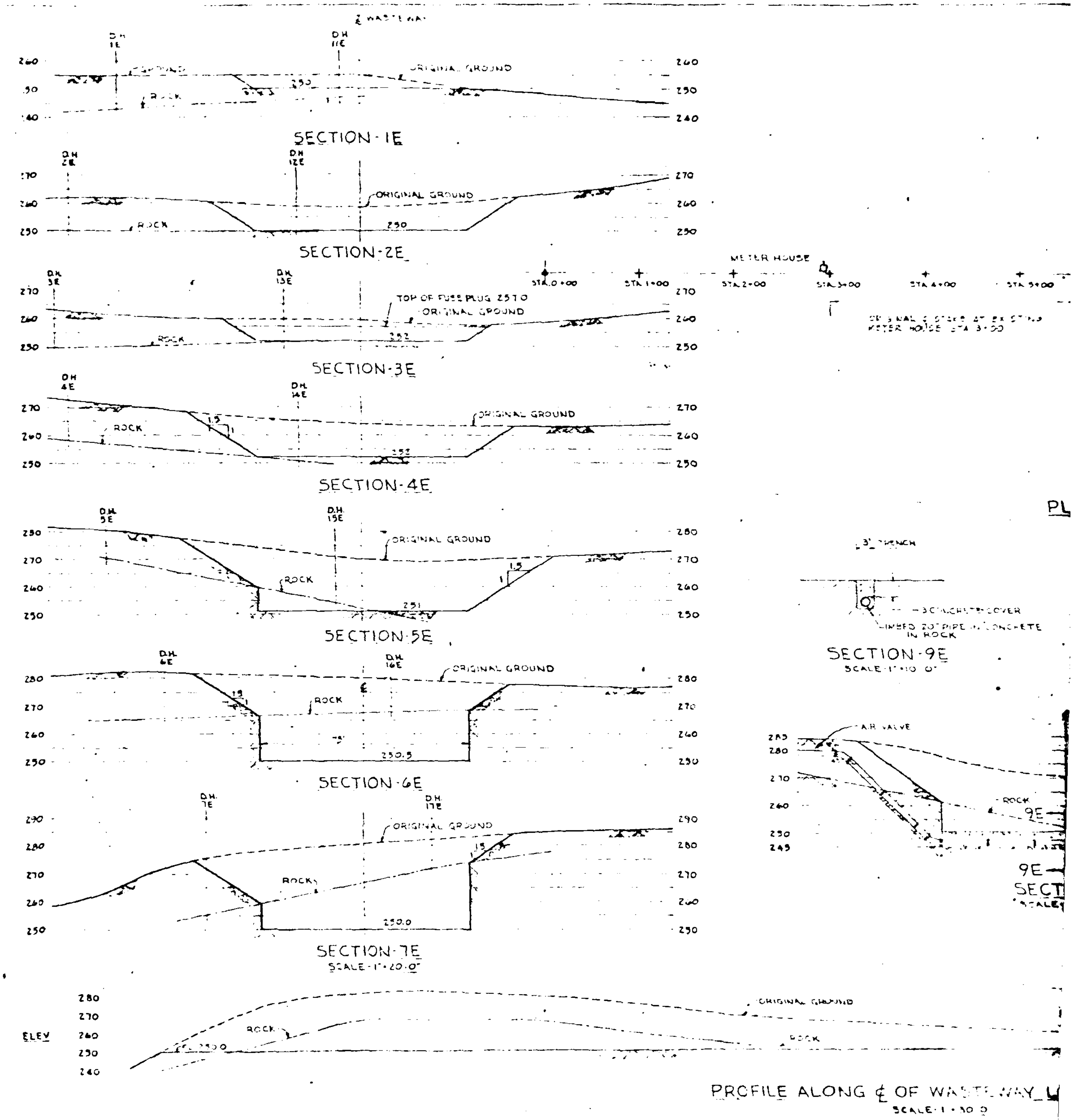
SECTION THRU CUT-OFF WALL AND ℄ OF DAM "A-A"
SCALE 1"=60'-0"



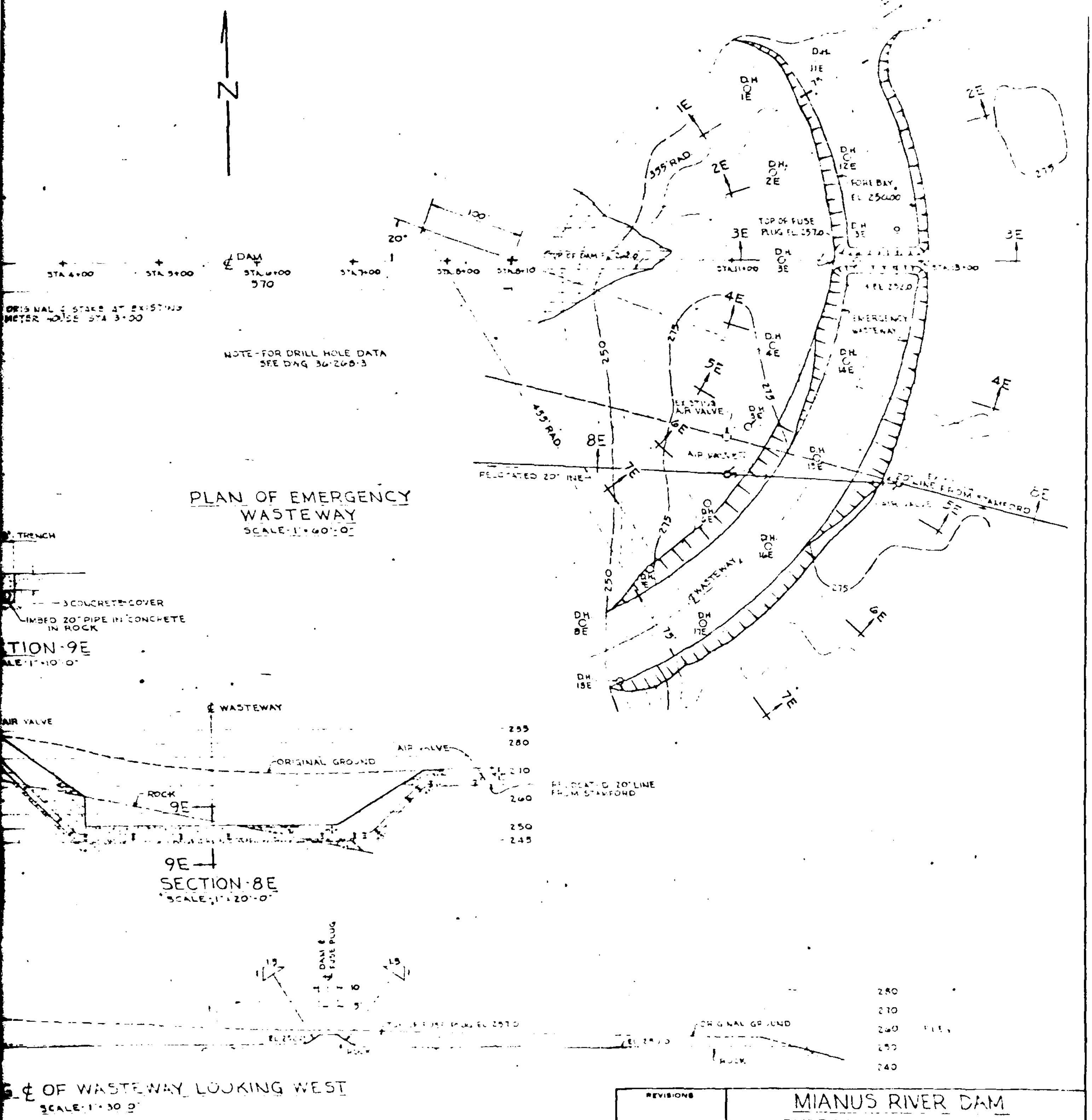
225
250
℄ DAM
9+00
10+00
11+00
12+00
13+00
A
275
℄ WASTEWAY
RENCH
TOP OF DAM EL. 262.0
FLOW LINE EL. 252.0
IMPERVIOUS CORE
ORIGINAL GROUND LINE
STRIP LINE
ROCK LINE
15'-0"
30'-0"
SECTION "B-B"
SCALE - 1" = 20'-0"
CUT-OFF WALL SEE DETAIL
2" ⌀ STD. STEEL GROUT PIPE 5'-0" C-C OR LOCATED AS DIRECTED BY ENGINEER
3" ⌀ STD. STEEL PIPE INSTALLED FOR WATER TEST HOLES @ 100 CRS.
PLACE FILL TO TOP OF STEM BEFORE GROUTING
12"
STEM
ROCK
BASE
2'-6"
SECTION AT CUT-OFF WALL
BACKFILL TO ORIGINAL GROUND LEVEL
℄ WASTEWAY
TOP OF FUSE PLUG 257.0
275
250
225
200
175
150
OF DAM "A-A"
REVISIONS
MIANUS RIVER DAM
PLAN OF CUT-OFF TRENCH
SECTIONS AND DETAILS
GREENWICH WATER COMPANY
GREENWICH, CONN.
AMERICAN WATER WORKS SERVICE COMPANY IN
SCALE AS NOTED
DRAWN BY
APPROVED
CHECKED BY


MIANUS RIVER
STILLING BASIN
PUMP HOUSE
FOR DETAIL OF PIPING IN THIS AREA SEE DWG 36 248-12
PLAN OF CONDUITS THRU DAM
Original Grade
Rock Line
SECTION 4W
SECTION 5W
SECTION 6W - ℄ DAM
Original Ground
Rock Line
Anti-Seepage Collars Cut in Rock
Selected Impervious Fill
PROFILE THRU ℄ SPILLWAY CONDUIT LOOKIN


SECTION 7W
SECTION 8W
SCALE 1"=20'
Original Grade
18'0" ϕ
30" ϕ
20" ϕ
Rock Line
18'0" Dia Spillway Conduit
30" Dia Intake
20" Dia From Brush Dam
Note:
For Details Of Conduit Grade See Dwg 30-208-3
18'0" ϕ Spillway Conduit
30" ϕ Intake
Top Of Cut-Off Wall
For Details Of Cut-Off Wall See Dwg
6W - ℄ DAM
Flow Line El. 230.0
El. 232.0
26" Rock Facing
12" Stone Blanket
Semi-Pervious Or Impervious Fill
Selected Impervious Fill
Original Ground
El. 220.0
Cut-Off Wall
Spillway Conduit
For Details See Dwg
Welded Steel
CONDUIT LOOKING WEST
SCALE 1"=5'
REVISIONS
MIANUS RIVER DAM
SPILLWAY INLET & CONDUIT
PLAN AND SECTIONS
GREENWICH WATER COMPANY
GREENWICH, CONN.
AMERICAN WATER WORKS SERVICE COMPANY, INC.
PHILADELPHIA, PA.
SCALE
USE DIMENSIONS ONLY
DRAWN BY
DATE
CHECKED BY
APPROVED



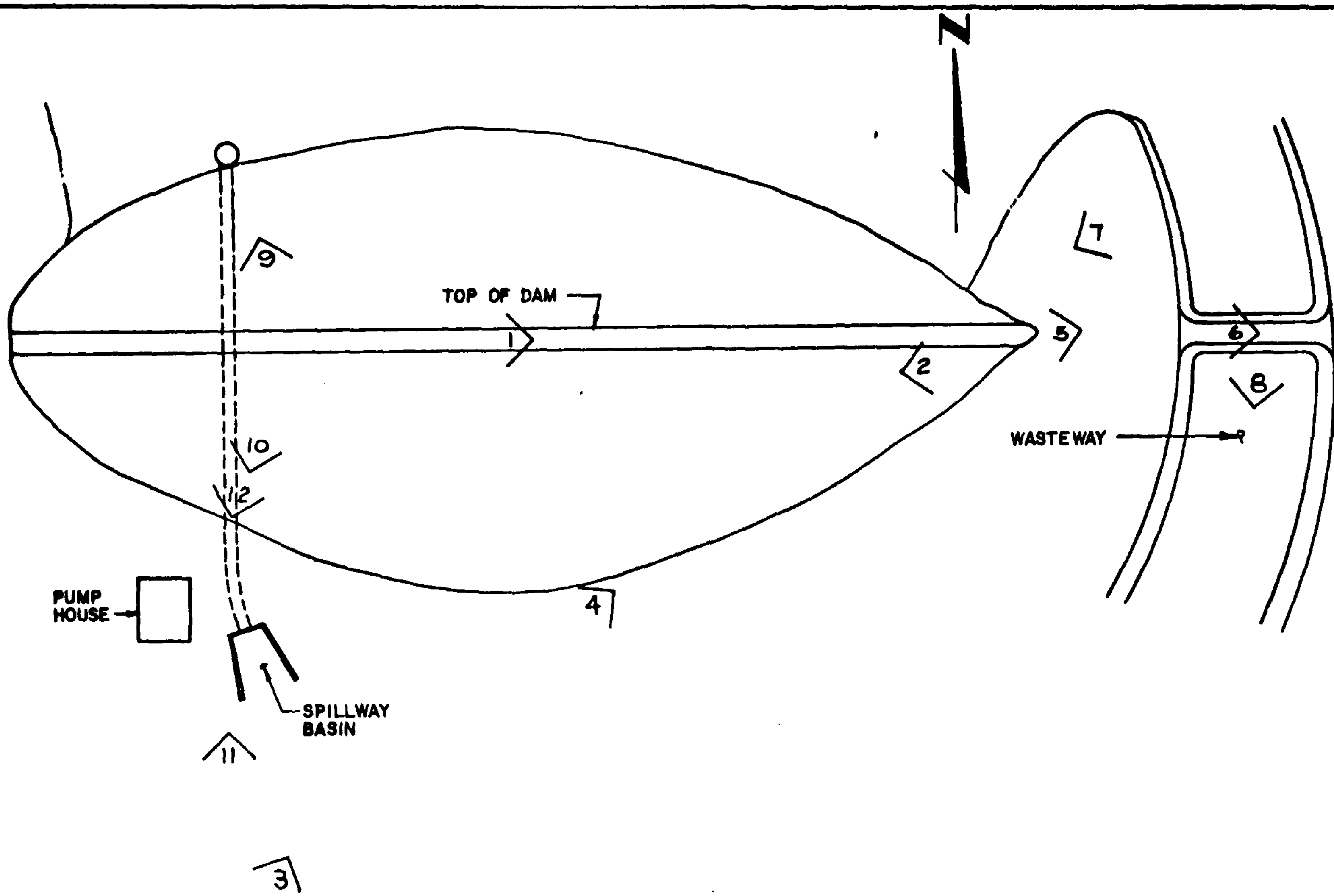
WASTEWAY
ORIGINAL GROUND
ROCK
SECTION-1E
SECTION-2E
TOP OF FUSE PLUG 251.0
SECTION-3E
METER HOUSE
STA.0+00
STA.1+00
STA.2+00
STA.3+00
STA.4+00
STA.5+00
SECTION-4E
SECTION-5E
3' TRENCH
SECTION-9E
SECTION-6E
AIR VALVE
SECTION-7E
SCALE-1"=20'-0"
ELEV
PROFILE ALONG ¢ OF WASTEWAY



REVISIONS	MIANUS RIVER DAM EMERGENCY WASTEWAY PLAN & SECTIONS
	GREENWICH WATER COMPANY GREENWICH, CONN.
	AMERICAN WATER WORKS SERVICE COMPANY INC. 131 SOUTH BROAD STREET PHILADELPHIA 7, PA.
	SCALE AS NOTED — USE DIMENSIONS ONLY
	DRAWN BY J.R.T. — DATE 12 11 53 — CHECKED BY
	APPROVED — PROJECT P 18 — INSP. BY
	USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES — 36-265-9

APPENDIX C

PHOTOGRAPHS



MIANUS RESERVOIR DAM

LOCATION OF PHOTOGRAPHS

PHOTO #1: Downstream Face of Embankment, looking east.

PHOTO #2: Downstream Face of Embankment, looking west.

PHOTO #3: Front View of Embankment.

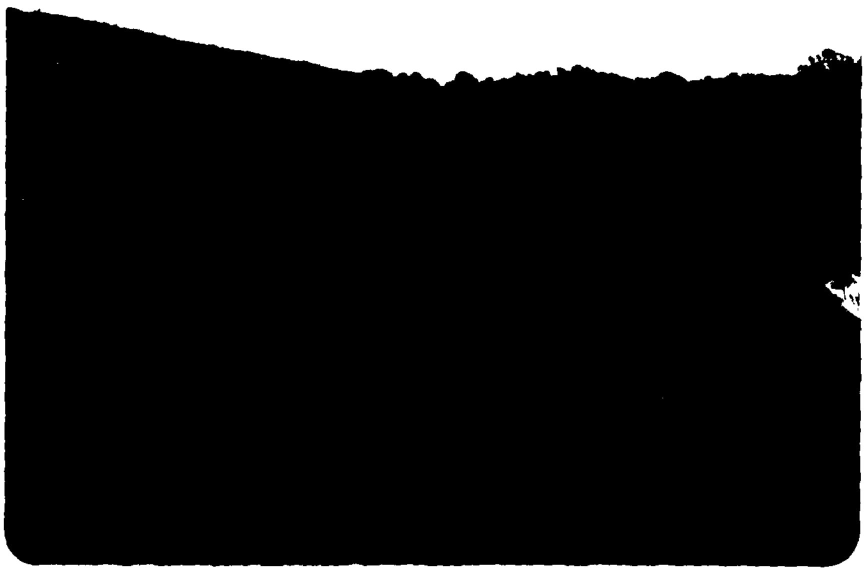


PHOTO #4: Toe of Slope, looking east.

PHOTO #5: Looking east toward fuse plug.

PHOTO #6: Small test pit in center of fuse plug.

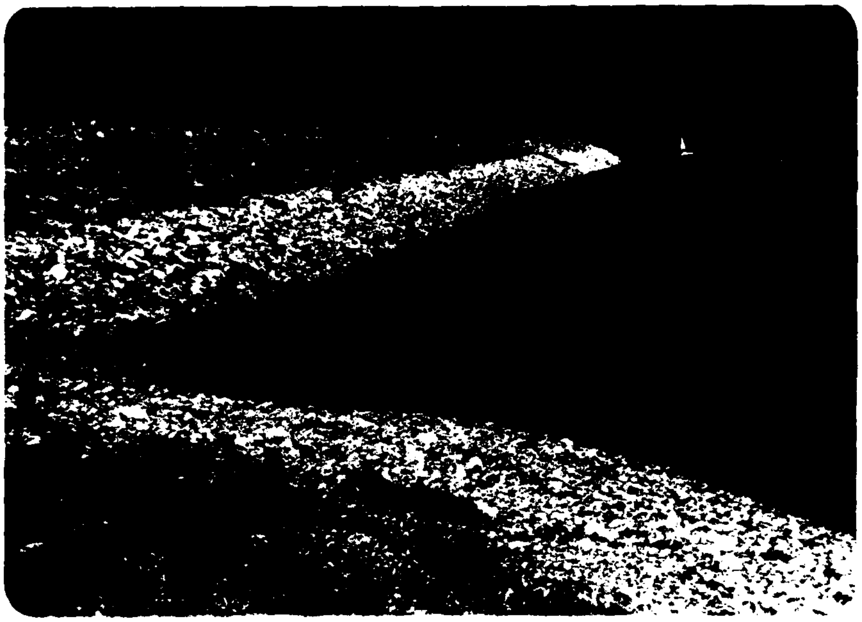


PHOTO #7: Upstream Face of Dam, looking west.

PHOTO #8: Emergency spillway (fuse plug) discharge channel, looking downstream.

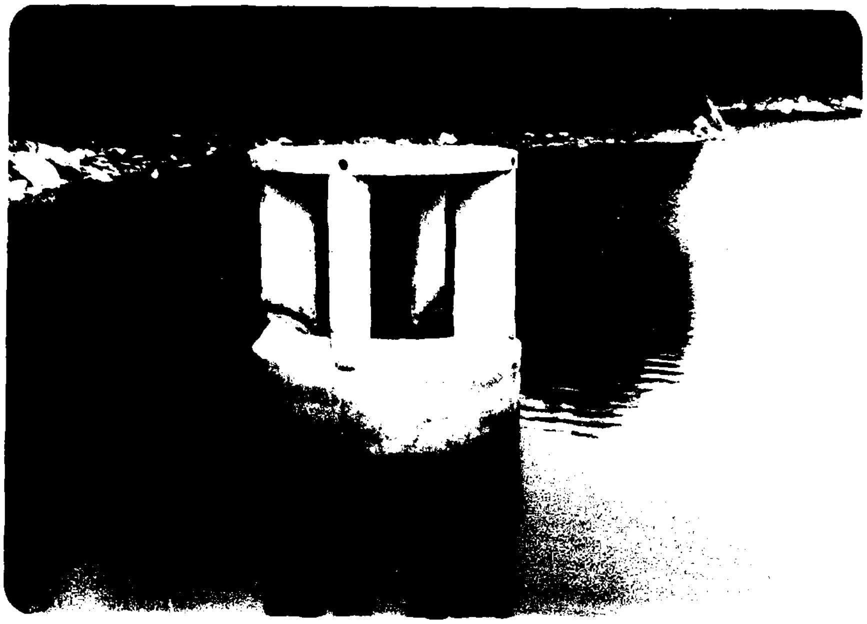


PHOTO #9: Service spillway - Top of the drop inlet shaft.

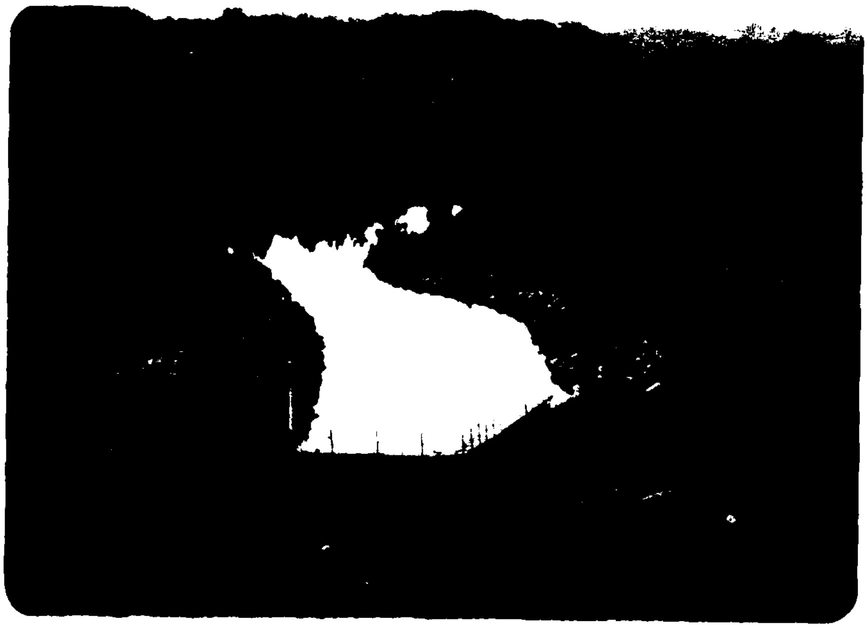


PHOTO #10: Service spillway discharge channel, looking downstream.

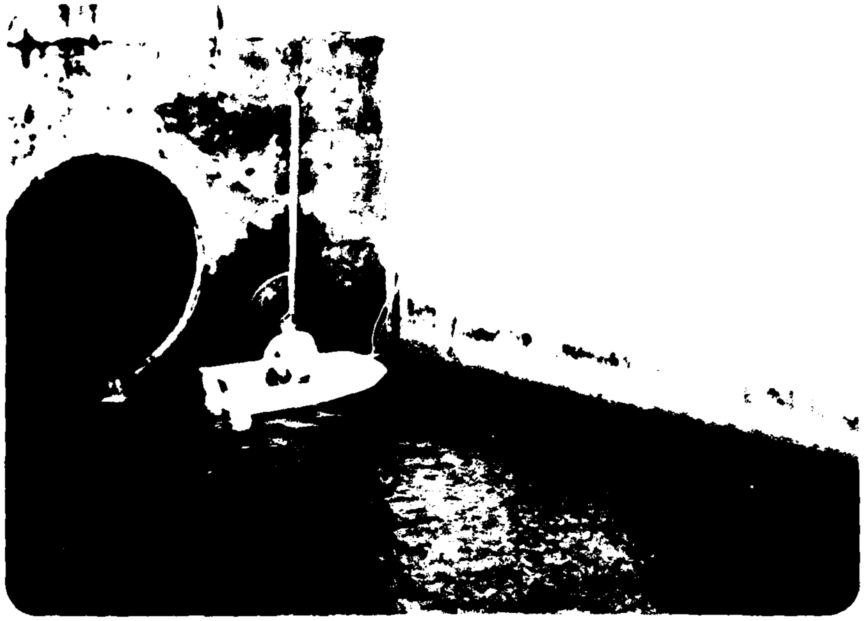


PHOTO #11: Outlet of the service spillway discharge conduit, and adjacent blow off.

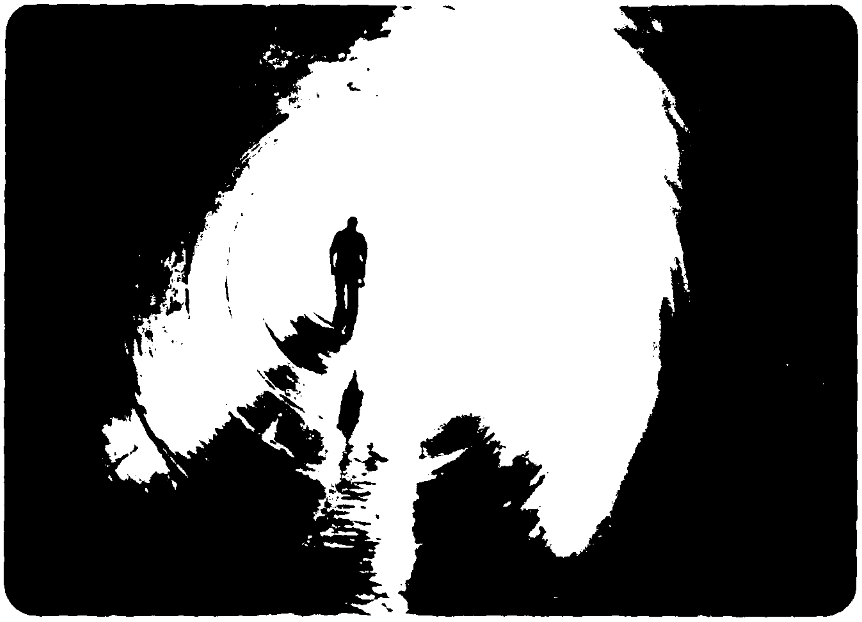


PHOTO #12: Interior view of the service spillway discharge conduit.

AD-A142 692 NATIONAL DAM INSPECTION PROGRAM MIANUS RESERVOIR DAM (CT 00050) CONNECTIC..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV SEP 78 2/2

UNCLASSIFIED F/G 13/13 NL

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DATE
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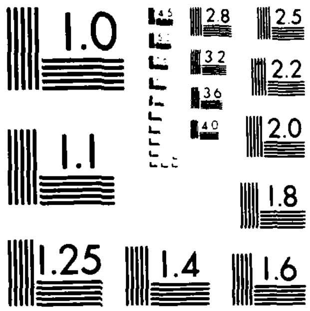
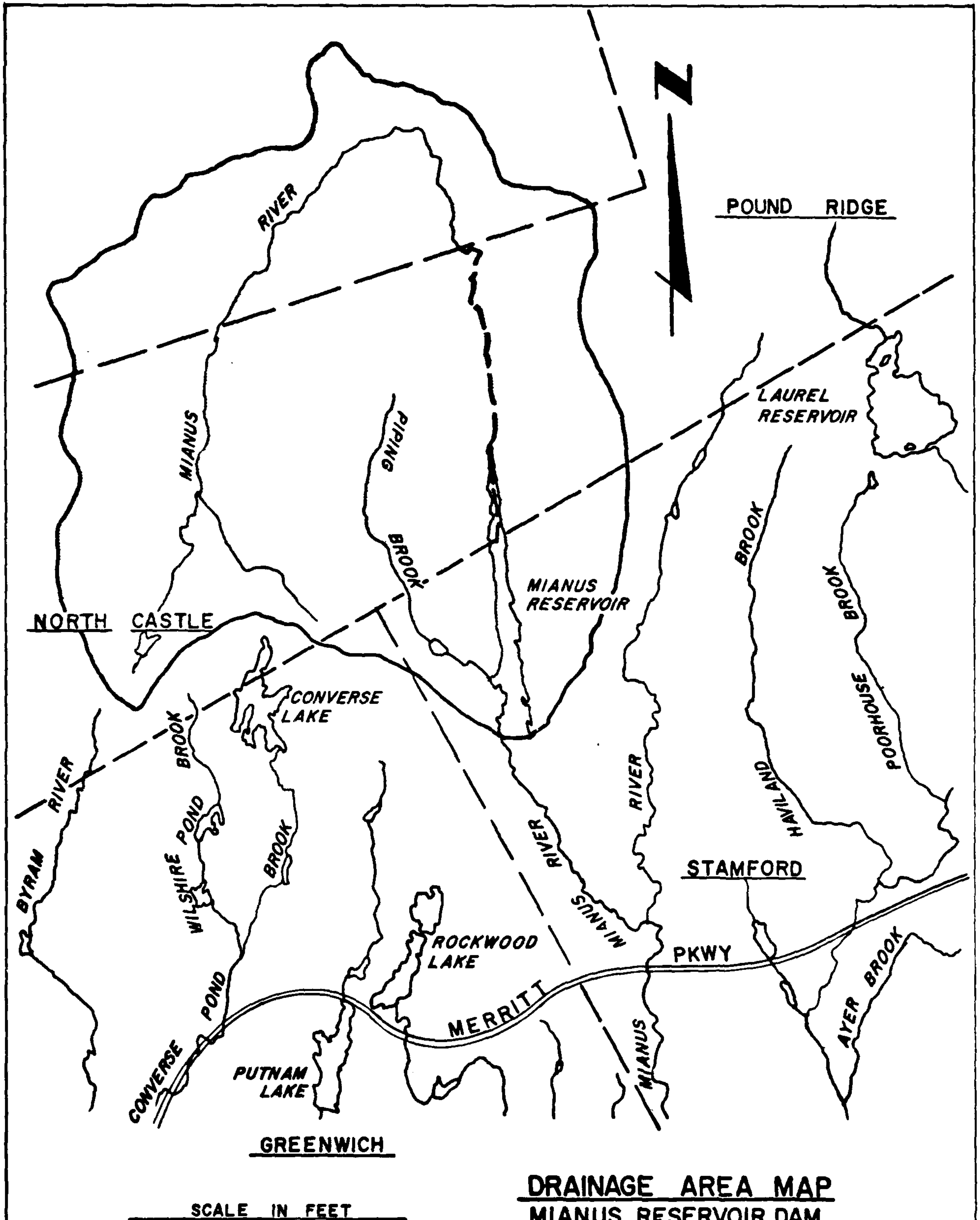
8-84
DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

APPENDIX D

HYDROLOGIC COMPUTATIONS



DRAINAGE AREA MAP
MIANUS RESERVOIR DAM
STAMFORD, CONNECTICUT

FLAHERTY-GIAVARA ASSOCIATES
ENVIRONMENTAL DESIGN CONSULTANTS
ONE COLUMBUS PLAZA, NEW HAVEN, CONN 06510/203/789-1260

P.M.F. PEAK FLOW ESTIMATE

WATERSHED AREA IS 18.3 SQ MILES

METHOD #1

REFER TO "PRELIMINARY GUIDANCE FOR ESTIMATING PMF DISCHARGES" BY NEW ENGLAND DIVISION, CORPS OF ENGINEERS

UNIT FLOW = 1550 CFS/MI^2 (ROLLING Curve)
PMF ≈ 18.3 mi^2 × (1550 CFS/mi^2) = 28,365 CFS
SAY ≈ 28,000 CFS

METHOD #2

REFER TO "CONN WATER RESOURCE BULLETIN #17, PART 4, BY U.S.G.S.

MEAN ANNUAL FLOOD = 700 CFS (FIG. 13)
Q_{100} = 5 × MAF = 5 × 700 CFS = 3500 CFS
PMF ≈ 5 × Q_{100} (APPROXIMATE)
PMF ≈ 5 × 3500 CFS ≈ 17,500

METHOD #3

REFER TO FAIRFIELD, CT., F.I.A. FLOOD INSURANCE STUDY "FREQUENCY, DISCHARGE, DRAINAGE AREA CURVES"

Q_{100} = 4500 CFS (FIG. 2)
PMF ≈ 5 × 4500 CFS = 22,500 CFS

FOR SPILLWAY TEST FLOOD, USE 28,000

FLAHERTY-GIAVARA ASSOCIATES
ENVIRONMENTAL DESIGN CONSULTANTS
ONE COLUMBUS PLAZA, NEW HAVEN, CONN. 06510/203/789-1260

FORMATION OF INFLOW HYDROGRAPH

1) TEST FLOOD = 28,000 CFS

2) FORM A TRIANGULAR HYDROGRAPH, WITH 24 HOUR DURATION, PEAK AT 8 HOURS

TIME HOURS	UNIT FLOW RATE	FLOW RATE CFS
0	0.00	0
2	0.25	7,000
4	0.50	14,000
6	0.75	21,000
8	1.00	28,000
10	0.875	24,500
12	0.75	21,000
16	0.50	14,000
20	0.25	7,000
24	0.00	0

SERVICE SPILLWAY

"MORNING GLORY" TYPE WITH 12 FT DIA WEIR, 8 FT DIA DROP SHAFT AND CONDUIT, $R_S = 6$ FT

$L =$ PERIMETER − PIER WIDTH

$L = 12'(\pi) - 4(1.5') = 31.7$ FT

H_o	H_o/R_S	C_o	Q (CFS)	STAGE
0	0		0	252
1	0.16	3.90	124	253
2	0.33	3.70	332	254
3	0.50	3.35	552	255
4	0.67	2.82	715	256
5	0.83	2.37	840	257
6	1.00	2.02	941	258
7	1.16	1.77	1039	259
8	1.33	1.52	1090	260
9	1.50	1.36	1164	261
10	1.67	1.22	1223	262

THE ABOVE ESTIMATED SERVICE SPILLWAY DISCHARGE RATES ARE BASED ON DATA FROM CHAP. IX, SECTION F, 212 OF "DESIGN OF SMALL DAMS", U.S. DEPT. OF THE INTERIOR, 1973.

WHERE:

$$Q = C_o L H^{3/2}$$

78-36-10
MIANUS

FLAHERTY-GIAVARA ASSOCIATES
ENVIRONMENTAL DESIGN CONSULTANTS
ONE COLUMBUS PLAZA, NEW HAVEN, CONN 06510/203/789-1260

SHEET NO. 4 OF
BY JGM DATE 8/10/78
CHK'D. BY EJ DATE 8/29/78

AUXILLARY SPILLWAY

CREST WILL ACT AS A BROAD CRESTED WEIR, COEFFICIENT "C" = 3.0
EFFECTIVE LENGTH IS 90 FEET ±.
ANALIZE WITH AND WITHOUT FUSE PLUG IN PLACE.

STAGE ELEVATION	H_o	W/FUSE PLUG Q CFS	W/O FUSE PLUG H_o	W/O FUSE PLUG Q CFS
252			0	0
253			1	270
254			2	764
255			3	1403
256			4	2160
257	0	0	5	3019
258	1	270	6	3968
259	2	764	7	5000
260	3	1403	8	6109
261	4	2160	9	7290
262	5	3019	10	8538

MIANUS DAM 78-36-10 FLOOD ROUTING JGM 9/19/78

INPUT DATA:	UNSUBMERGED WEIR				
SEGMENT 1	DISCHARGE COEFFICIENT = 2		LENGTH OF WEIR = 31.7		ELEVATION OF WEIR = 252
SEGMENT 2	DISCHARGE COEFFICIENT = 3		LENGTH OF WEIR = 90		ELEVATION OF WEIR = 252
SEGMENT 3	DISCHARGE COEFFICIENT = 3		LENGTH OF WEIR = 860		ELEVATION OF WEIR = 262
IE=252 IV=	0.0 E=252 A=230.00	E=260 A=240.00	E=270 A=240.00		

HOUR	INFLOW	MASS INFLOW	WATER EL.	TAIL WATER	OUTFLOW	MASS OUTFLOW	STORAGE(R)	STORAGE(A)
0.00	0CFS	0.00AC-F	252.00FT	0.00FT	0CFS	0.00AC-F	0.00AC-F	0.00AC-F
2.00	7,000CFS	578.51AC-F	254.13FT	0.00FT	1,036CFS	85.67AC-F	492.83AC-F	492.83AC-F
4.00	14,000CFS	2,314.04AC-F	258.97FT	0.00FT	6,143CFS	679.07AC-F	1,634.97AC-F	1,634.97AC-F
6.00	21,000CFS	5,206.51AC-F	263.27FT	0.00FT	16,366CFS	2,539.41AC-F	2,667.19AC-F	2,667.19AC-F
8.00	28,000CFS	9,256.19AC-F	264.87FT	0.00FT	27,995CFS	6,205.73AC-F	3,050.46AC-F	3,050.46AC-F
10.00	24,500CFS	13,595.04AC-F	264.56FT	0.00FT	25,420CFS	10,620.27AC-F	2,974.76AC-F	2,974.76AC-F
12.00	21,000CFS	17,355.37AC-F	264.05FT	0.00FT	21,551CFS	14,502.27AC-F	2,853.09AC-F	2,853.09AC-F
16.00	14,000CFS	23,140.49AC-F	263.03FT	0.00FT	14,930CFS	20,532.42AC-F	2,608.07AC-F	2,608.07AC-F
20.00	7,000CFS	26,611.57AC-F	261.00FT	0.00FT	9,012CFS	24,489.90AC-F	2,121.66AC-F	2,121.66AC-F
24.00	0CFS	27,768.59AC-F	257.01FT	0.00FT	3,746CFS	26,598.86AC-F	1,169.73AC-F	1,169.73AC-F
30.00	0CFS	27,768.59AC-F	252.79FT	0.00FT	234CFS	27,586.10AC-F	182.49AC-F	182.49AC-F
36.00	0CFS	27,768.59AC-F	252.43FT	0.00FT	96CFS	27,668.14AC-F	100.44AC-F	100.44AC-F

APPENDIX E

INFORMATION - NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR DIST.	STATE	COUNTY	CONGR DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	50	NED	CT	001	04				MIANUS RESERVOIR DAM	4107.8	7336.7	08SEP78

POPULAR NAME	NAME OF IMPOUNDMENT
SAMUEL J BARGH RESERVOIR DAM	MIANUS RESERVOIR

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI)	POPULATION
01	07	MIANUS RIVER	RIVERBANK	2	1000

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT)	HYDRAULIC HEIGHT (FT)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
REPG	1955	S	75	75	7515		NED	N	N	N	N	28AUG78

REMARKS

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT)	MAXIMUM DISCHARGE (FT)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NO	NAVIGATION LOCKS LENGTH (FT)	WIDTH (FT)	LENGTH (FT)	WIDTH (FT)	LENGTH (FT)	WIDTH (FT)	LENGTH (FT)	WIDTH (FT)
1	950	U	90	5000												

OWNER	ENGINEERING BY	CONSTRUCTION BY
CONN-AMERICAN WATERWORKS	AMERICAN WATERWORKS SERV	MERRITT CHAPMAN SCOTT

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
FLAHERTY GIAVARA ASSOC,PC	03AUG78	PL 92-367

REMARKS