NEW ENGLAND POWER COMPANY

SHERMAN DAM

EXECUTIVE SUMMARY REPORT

GEOTECHNICAL INVESTIGATION

MAY, 1982

Prepared for

New England Power Service Company

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MAIN

EXECUTIVE SUMMARY REPORT

GEOTECHNICAL INVESTIGATION - SHERMAN DAM

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INTRODUCTION

New England Power Service Company contracted with Chas. T. Main, Inc. (MAIN) and Geotechnical Engineers, Inc. (GEI) to conduct a geotechnical investigation of Sherman Dam, which is owned by New England Power Company and is situated in Rowe and Monroe, Massachusetts. The Yankee Rowe nuclear plant is located just east of the dam's left abutment.

GEI was responsible for conducting the exploratory drilling program, obtaining embankment and abutment samples, and conducting the laboratory testing program. The field work took place during the period of 17 June to 16 September 1981, while the laboratory portion began shortly after June and ended on 15 January 1982. GEI also conducted the seismic stability analysis of the embankment.

MAIN was responsible for the overall geotechnical investigation of the dam, and using the developed properties conducted the static stability analyses for various loading conditions.

Sherman Dam is a part of the Deerfield River Project - Federal Energy Regulatory Commission (FERC) Licensed Project No. 2323 and has undergone the regular FERC five year safety inspections in 1968, 1973 and 1978. MAIN conducted a stability analysis of the dam for the 1973 report.

DAM GEOMETRY

Sherman Dam was constructed in 1926 by the semi-hydraulic fill method, which consisted of developing a central puddle core by washing material from the inside slopes of the upstream and downstream dumped shells. This process also created a transition zone of coarse washed material between the core and shell.

Remedial work in 1964 consisted of raising the crest elevation 10 feet by adding a layer of compacted glacial till to the downstream slope of the dam. At the same time, the rock channel downstream of the spillway was deepened for greater capacity.

The embankment is about 100 feet high with a crest length of 560 feet at an elevation of 1129.66 NGVD.

Figure 1 shows the general location of the dam near Rowe, Massachusetts. Figures 2 and 3 show a detailed plan of the dam and cross section at Station 16+00, the approximate center and maximum section of the embankment.

FIELD WORK - FALL 1981

The field work conducted by GEI for this investigation took place during the period from 17 June to 16 September, 1981. Eight borings (B-101 through B-107), totaling 627 feet in length, investigated the core and downstream shell of the dam, and the nature of a seepage breakout zone on the downstream left abutment. Boring B-103 required redrilling (B-103A) after a steel drive shoe loss at a depth of 33 feet. Nineteen pneumatic type piezometers were installed in the eight drillholes to provide information about seepage characteristics and existing pore pressures in the dam.

A single falling-head permeability test was conducted in boring B-103A (at a depth of 64 feet in shell material) which resulted in a calculated in-situ permeability ranging between 3.9 and 7.4 x 10^{-4} cm./sec., depending on assumptions regarding soil anisotropy. The final boring, B-107, utilized a special procedure to avoid hydrofracture in the deeper portions of the boring.

Both split-spoon and undisturbed fixed-piston tube samples provided material for soil classification and laboratory testing. Because of the dense, gravelly nature of the shell material, undisturbed samples in this zone could not be recovered. Figures 2 and 3 show the plan and elevations of borings and piezometers, respectively. Table 1 lists elevations of the top and bottom of the borings and the elevations of the piezometer diaphragms.

LABORATORY INVESTIGATION

GEI conducted laboratory tests on the samples to classify them and to obtain appropriate engineering parameters for use in stability analyses. Table 2 lists the type and number of tests performed.

Based on visual examination and index test results, the following soil descriptions result:

Shell - Silty Gravelly Sand. Predominantly widely graded sand with weathered to unweathered gravel and non- to slightly plastic fines. Percentage of gravel ranges from 5-40%. Percentage of fines ranges from 5-40%. Color varies from brown, olive-brown, gray-brown, to gray. The Unified Soil Classification symbols are SM, SW, and GM for various samples of the shell.

Core - Sand and Silt. Predominantly stratified fine sand and silt; percentage of fines ranges from 25-75%. Fines are predominantly non-plastic, with a trace of fine gravel and fine root fragments; olive-brown, gray and brown. Occasional 1/16- to 1-inch thick layers of clean fine sand or slightly to moderately plastic clayey silt were observed. The Unified Soil Classification symbols for various samples are SM, ML, and SP.

Vertical permeability tests of undisturbed specimens of core material, tests R-4 and R-5, gave values of 1.1 and 10.3 x 10^{-6} cm./sec., respectively.

Figures 4 and 5 show grain size curves for the shell and core materials, respectively. Table 3 summarizes the results of tests to determine engineering properties.

Undisturbed sampling of the shell proved impossible. However, since the material, construction procedure, grain size distributions and range of blow counts at Sherman Dam appear similar to those found at Harriman Dam, MAIN used 35 degrees for the effective stress friction angle of the shell, based on test results from Harriman Dam.

Sample selection and preparation for the undisturbed tests involved great care, including examination of x-ray photographs. Direct extrusion from the tube to the membrane helped to minimize disturbance. Tests on reconstituted specimens involved elaborate preparations to produce test samples witn the proper grain size, void ratio and density.

Seven $C\bar{R}$ triaxial tests, performed on 3-inch diameter thin-wall tube samples of the core material (both isotropic and anisotropic consolidation) permitted the development of parameters applicable to Sherman Dam and for comparison to data in the literature for dams that have experienced earthquakes.

STABILITY ANALYSES

Static Conditions

To conduct a stability evaluation for an earth dam, it is necessary to obtain and evaluate the following information:

1) dam geom.:try, foundation geometry and geology
2) soil properties for dam and foundation
3) seepage conditions
4) seismic coefficients
5) appropriate analytical methods
6) appropriate loading conditions

Careful visual examination of the site, along with data from soil borings, laboratory tests, field tests, and construction records have permitted MAIN to construct a reasonable picture of embankment geometry and to determine engineering properties of the existing materials.

The complex zoning of the dam creates seepage anomalies not completely explained by the piezometer data. Sufficient pore pressure data does exist, howeve-, to construct an estimated phreatic surface as shown in Figure 3. After studying the field and laboratory data, MAIN selected the following parameters for use in the analysis:

Parameter	Core	Shell	Foundation
effective stress friction angle, ϕ'	35°	35°	36°
cohesion, c', TSF	0	0	0
unit weight, pcf	125	135	140

The Simplified Bishop procedure analyzes circular arc failure surfaces by the method of slices. The simplified procedure assumes the orientation of inter-slice forces is constant and sums to zero. Extensive computational experience indicates this is generally a conservative assumption.

The Simplified Bishop method of analysis has been used to develop MAIN's in-house computer program SLØPE which was used for these analyses. The program handles any section that can be defined by up to one hundred lines and up to 10 concentrated loads. An initial trial failure arc is specified by center coordinates and radius to begin the solution. The program then automatically searches for the critical failure surface by changing the initial center coordinates and radius by 5-foot increments (other increments can be specified) converging toward the center and radius defining the critical failure surface. The program stops searching when the least FS_s value is determined.

Two methods are provided in the program to define pore water pressure conditions within the section. The method used in this analysis was to input the phreatic and free water surfaces as part of the straight lines defining the section analyzed. The program then computes pore water uplift pressures as the vertical distance from the phreatic surface to the failure arc times the unit weight of water. This method is correct for horizontal water surfaces and highly conservative for steeply dipping phreatic surfaces.

The program SLØPE can perform a pseudostatic analysis with the use of input earthquake coefficients. While this procedure is not ideal for evaluation of semi-hydraulic fill embankments, it does given an estimate of the structures' resistance to liquefication failure. Analyses for the following cases was made:

Case	Slope	Earthquake Coefficient
Steady-state seepage	downstream	0, 0.1g., 0.15g., 0.2g.
Rapid drawdown	upstream	0, 0.1g., 0.15g., 0.2g.

Note: Separate searches were made for shallow and deep failure surfaces.

Figure 6 shows the location of typical critical failure surfaces and the resulting safety factors for an earthquake coefficient of 0.1g. For static conditions, the minimum computed safety factor for the downstream slope is about 1.4, and 2.0 for the upstream slope. The safety factors remain above 1.0 for earthquake coefficients up to 0.1g. The analyses utilized a composite of construction data, soil boring data, laboratory test results and field piezometer data.

Seismic Conditions

A preliminary seismic stability analysis has been completed for Sherman Dam by GEI. Data used in this analysis came from field and laboratory studies for Sherman Dam, construction records for Sherman Dam, and field and laboratory studies for Harriman Dam. Since no undisturbed samples were recovered of the Sherman Dam shell material, correlations were made between Harriman Dam samples and the Sherman Dam conditions. Analyses were made for the maximum section occurring near the center of the dam at approximately Station 16+00.

The core at Sherman Dam is somewhat less dense than the core at Harriman Dam and would likely liquefy during the design earthquake; however, the shell material would retain most of its shear strength. Strength parameters for the flow slide stability analysis are shown in Table 4. Analyses were made using input earthquake motions from both the Nuclear Regulatory Commission Spectrum and the Yankee Atomic Electric Spectrum, previously used to analyze Harriman Dam.

The calculated minimum Factor of Safety against a flow slide failure triggered by an earthquake loading was 1.3, occurring on the downstream slope (see Table 4 for soil properties used). Crest settlements and horizontal displacements in all cases were less than 1.5 feet. With a minimum freeboard at Sherman Dam of 24 feet, deformations of these magnitudes would not impair the overall integrity of the dam.

DISCUSSION OF CURRENT FIELD DATA

Nineteen (19) piezometer sensors have been installed in Sherman Dam (installation data seen in Table 1). MAIN has reviewed data for these units through April 5, 1982. Irregular fluctuations in these data seen in November and December have ceased and current data appear reasonable. All readings show total heads below ground surface. The data show that the core, though small, does reduce flow through the dam and helps to maintain relatively low pore pressures in the downstream portion of the dam. On the basis of an initial evaluation, MAIN has selected the following critical piezometers to be read every two weeks for the next two years.

102D	104B
103C	104C
103D	104D

The data to date confirms visual observations of the dam that no adverse seepage problems appear to exist. Piezometers should be read more frequently after exceptionally heavy rain or significant increases in reservoir level. Any sudden change in a piezometer reading (up or down) would require an investigation.

Piezometer data and field observations confirmed that the seepage breakout on the downstream left abutment has no relation to headwater at Sherman Dam, nor to the safety and stability of the dam. The seepage is flow from the left abutment natural slopes.

SUMMARY AND CONCLUSIONS

This report reviews the results of a geotechnical investigation carried out at Sherman Dam that included: soil borings with sampling, laboratory testing, and static and seismic stability analyses. Data resulting from this investigation have given a reasonably good picture of the cross-section of the dam and fluid flow through and below the dam. The data have given a satisfactory understanding of the engineering properties of the dam and foundation. The study permits the following three primary conclusions:

1) no adverse geotechnical conditions now exist at Sherman Dam,

2) static and seismic stability factors of safety remain above minimum acceptable limits, and

3) deformations resulting from the design earthquake would be less than the available freeboard of the dam.

RECOMMENDATIONS

The preceding discussion and conclusions lead to the following recommendations:

1) no physical modifications to the dam are required at this time,

2) no changes in the operation of the dam are required at this time, and

3) NEPCo staff should carry out piezometer readings on selected units as discussed in the main portion of this report.

TABLES

TABLE 1

LIST OF BORINGS AND PIEZOMETERS

SHERMAN DAM - NEPCo

Boring No.	Surface Elevation, Feet	Bottom Elevation, Feet	Piezometer Diaphragm Elevation, Feet
B-101	983.4	966.8	970.2
B-102	1019.5	890.8	(A) 895.3
			(B) 925.5
			(C) 950.3
			(D) 980.7
B-103	1011.4	978.6	-
B-103A	1011.4	902.4	(AA) 905.4
			(AB) 922.9
			(AC) 951.4
			(AD) 971.4
B-104	984.6	893.3	(A) 894.7
			(B) 919.9
			(C) 938.2
			(D) 953.9
B-105	1010.8	899.3	(A) 902.1
			(B) 937.1
			(C) 967.2
			(D) 981.9
B-106	980.5	957.0	960.0
B-107	1024.0	910.8	951.5

Elevations are relative to NEPCo datum. For NGVD add 105.7 feet.

TABLE 2

LIST OF LABORATORY TESTS

SHERMAN DAM - NEPCo

Test Type	No. of Tests
Vertical Permeability - Core	2
Grain Size	40
Specific Gravity	10
Unit Weight	19
Water Content	20
Triaxial $\overline{CU}$*	
Undisturbed Specimens	9
Reconstituted Specimens	5
Cyclic Triaxial ($\overline{CR}$)*	7

* All triaxial tests on core material.

(Tests performed by Geotechnical Engineers, Inc.)

TABLE 3

SUMMARY OF SOIL PROPERTIES

SHERMAN DAM - NEPCo

Parameter	Average Values	
	Core	Shell
Specific Gravity	2.75 (4)*	2.77 (6)
Dry Unit Weight,[1] pcf	85.6 (11)	-
Water Content,[2] %	34.8 (16)	-
Effective Stress Friction Angle,[3] Undisturbed (degrees) - ϕ_u'	37.4 (9)	35[4]
Effective Stress Friction Angle,[3] Reconstituted, (degrees) ϕ'	32.8 (5)	-

NOTES:

1. Corrected for possible densifying during sampling and transportation.
2. For static triaxial tests only.
3. From consolidated undrained $\overline{CU}$ triaxial tests.
4. From correlation studies with Harriman Dam testing.

* Indicates No. of Tests.

TABLE 4

STRENGTH PARAMETERS FOR

FLOW SLIDE STABILITY ANALYSIS

Hydraulic Core

Undrained Steady-State Shear Strength

S_{us}=700 psf

Dumped Shell (No samples. Data from correlation with similar samples from Harriman Dam)

S_{us}=2,000 psf*

Rolled Shell

ϕ'=45°

c-0

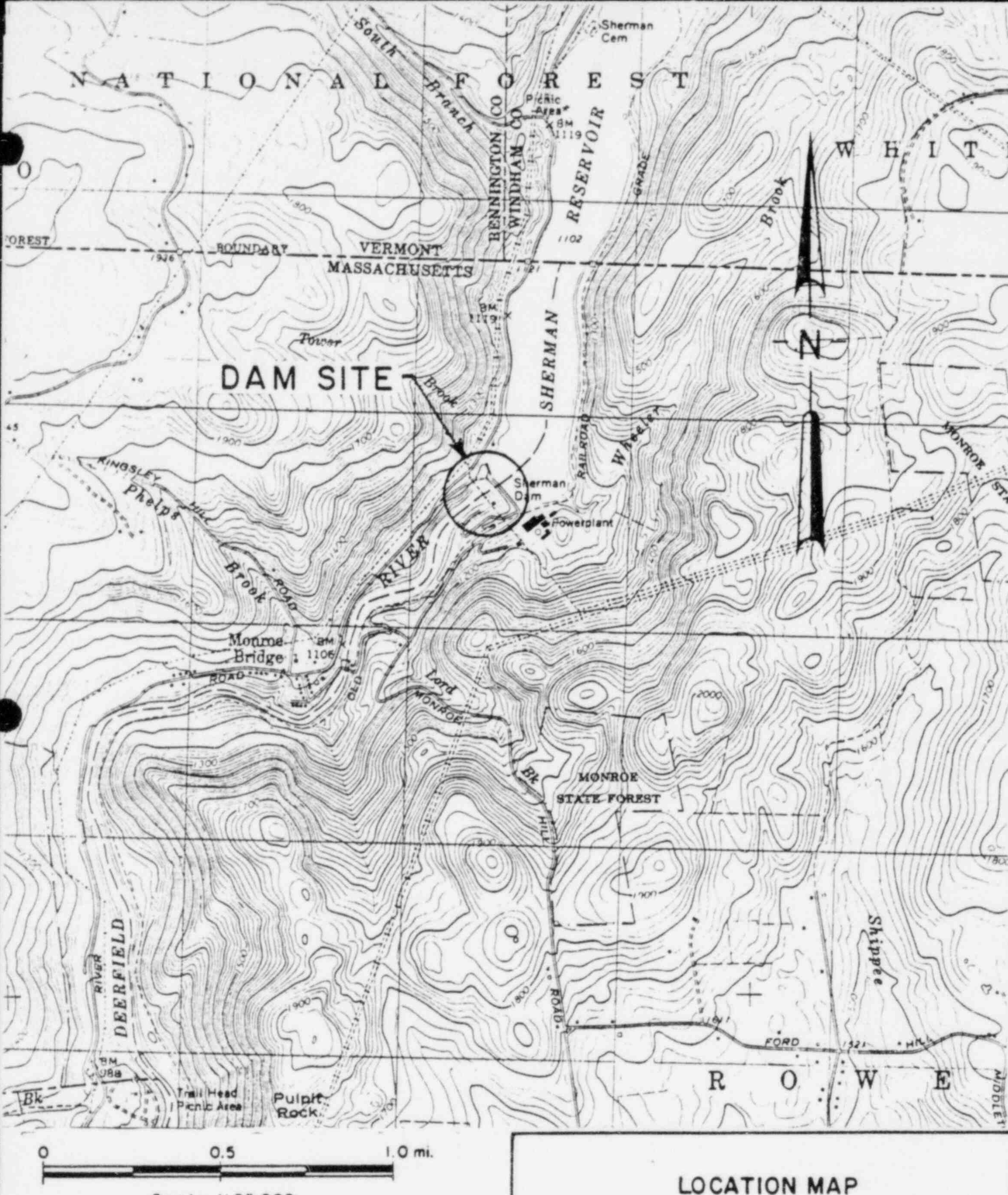
Glacial Till Foundation

ϕ'=40°

c=0

*Undrained residual strength after partial liquefaction of core material.

FIGURES



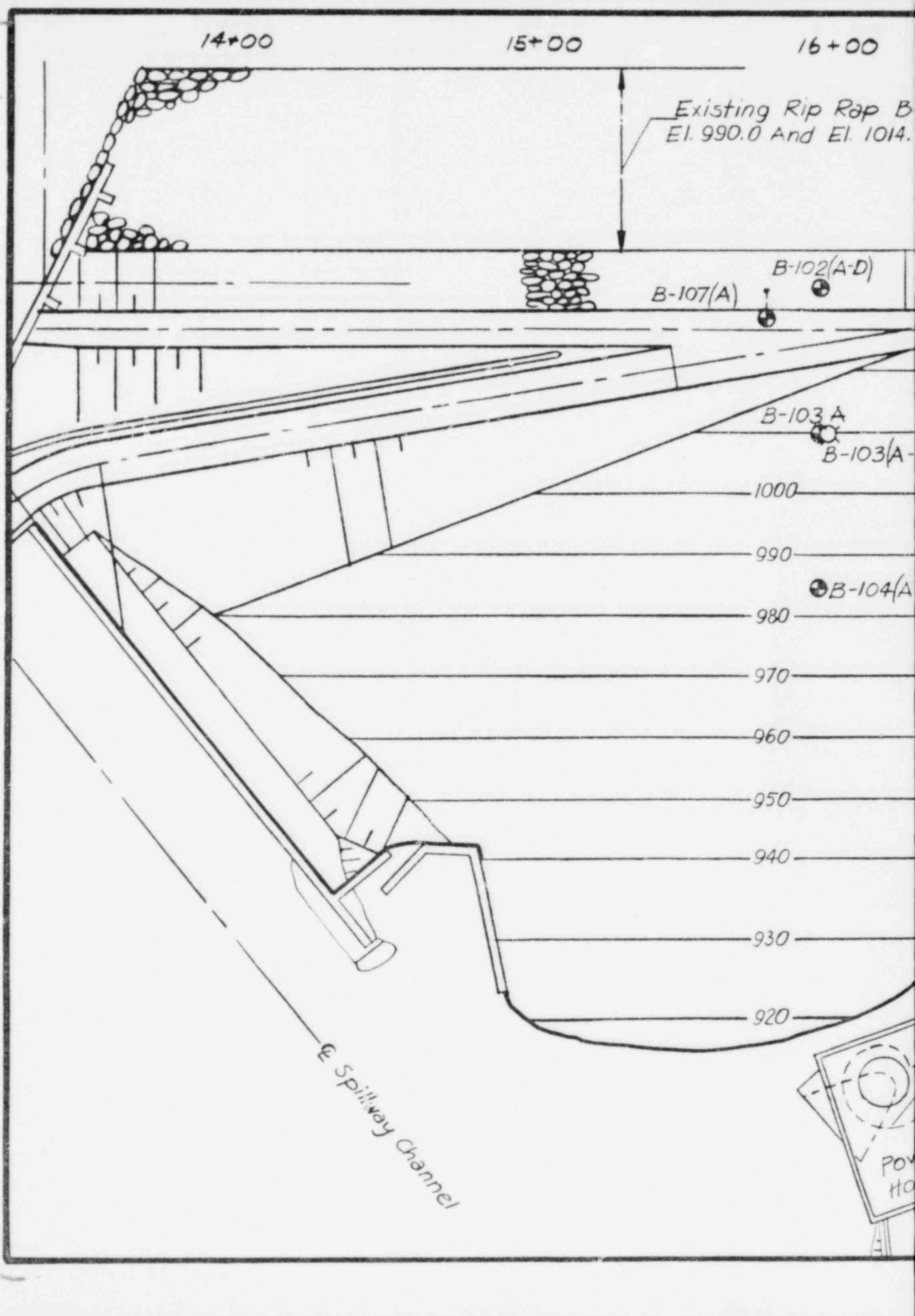
QUADRANGLE LOCATION

LOCATION MAP

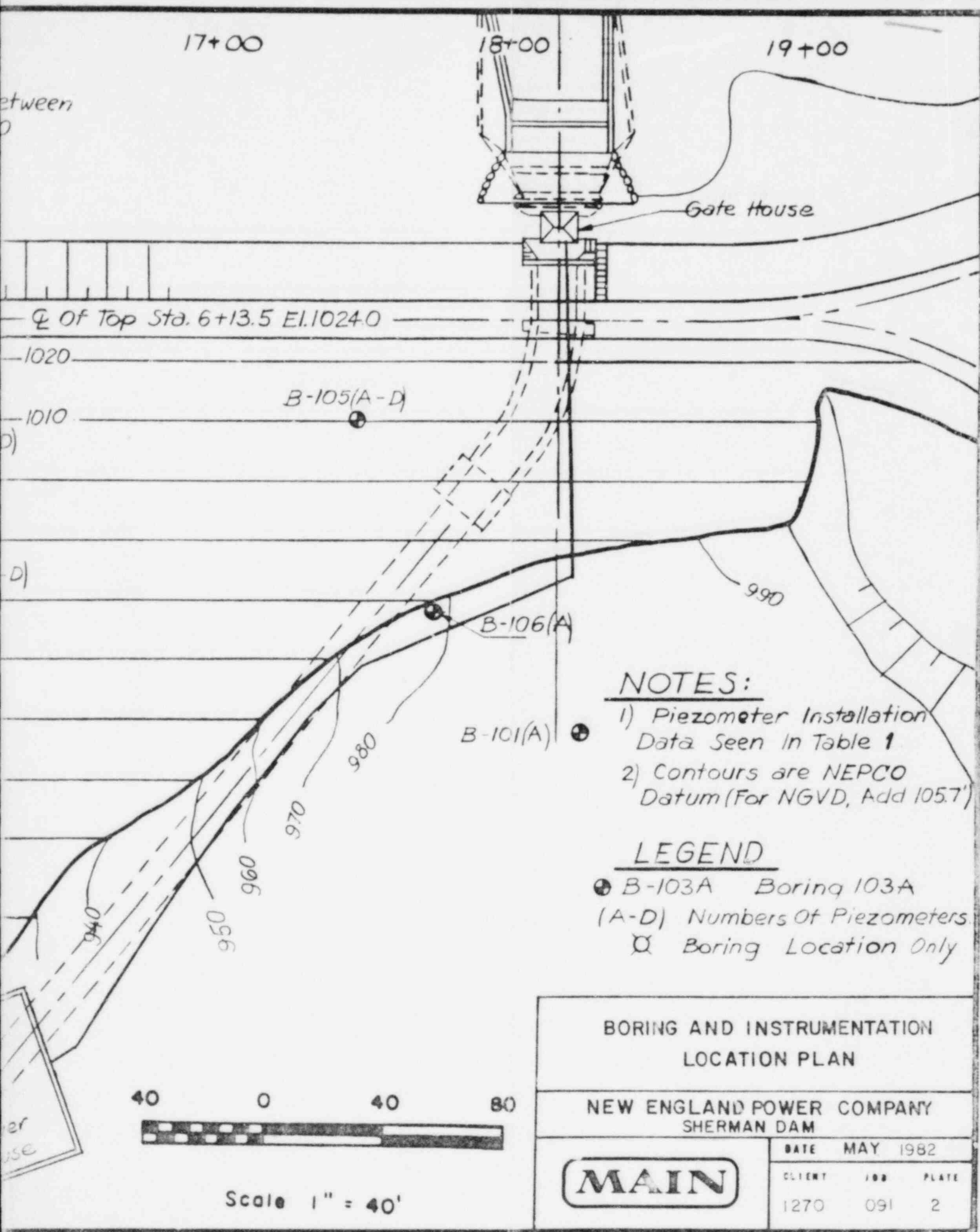
NEW ENGLAND POWER COMPANY
SHERMAN DAM

MAIN

DATE	MAY, 1982	
CLIENT	JOB	PLATE
1270	091	1



14+00
15+00
16+00
Existing Rip Rap B
El. 990.0 And El. 1014.
B-102(A-D)
B-107(A)
B-103 A
B-103(A
1000
990
B-104(A
980
970
960
950
940
930
920
℄ Spillway Channel
POV
HO



NOTES:

1) Piezometer Installation Data Seen In Table 1
2) Contours are NEPCO Datum (For NGVD, Add 105.7')

LEGEND

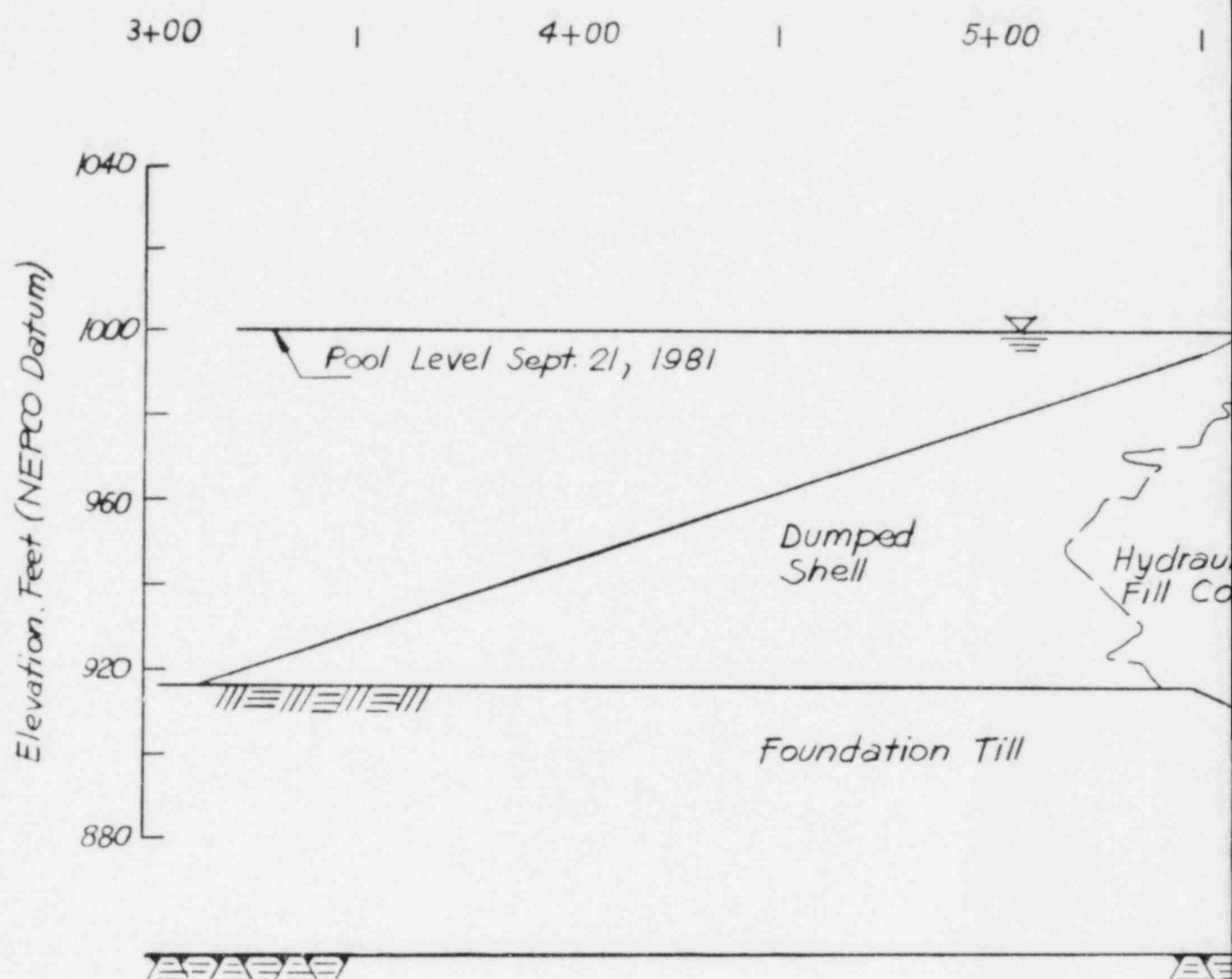
B-103A Boring 103A
(A-D) Numbers Of Piezometers
Boring Location Only

BORING AND INSTRUMENTATION LOCATION PLAN

NEW ENGLAND POWER COMPANY
SHERMAN DAM

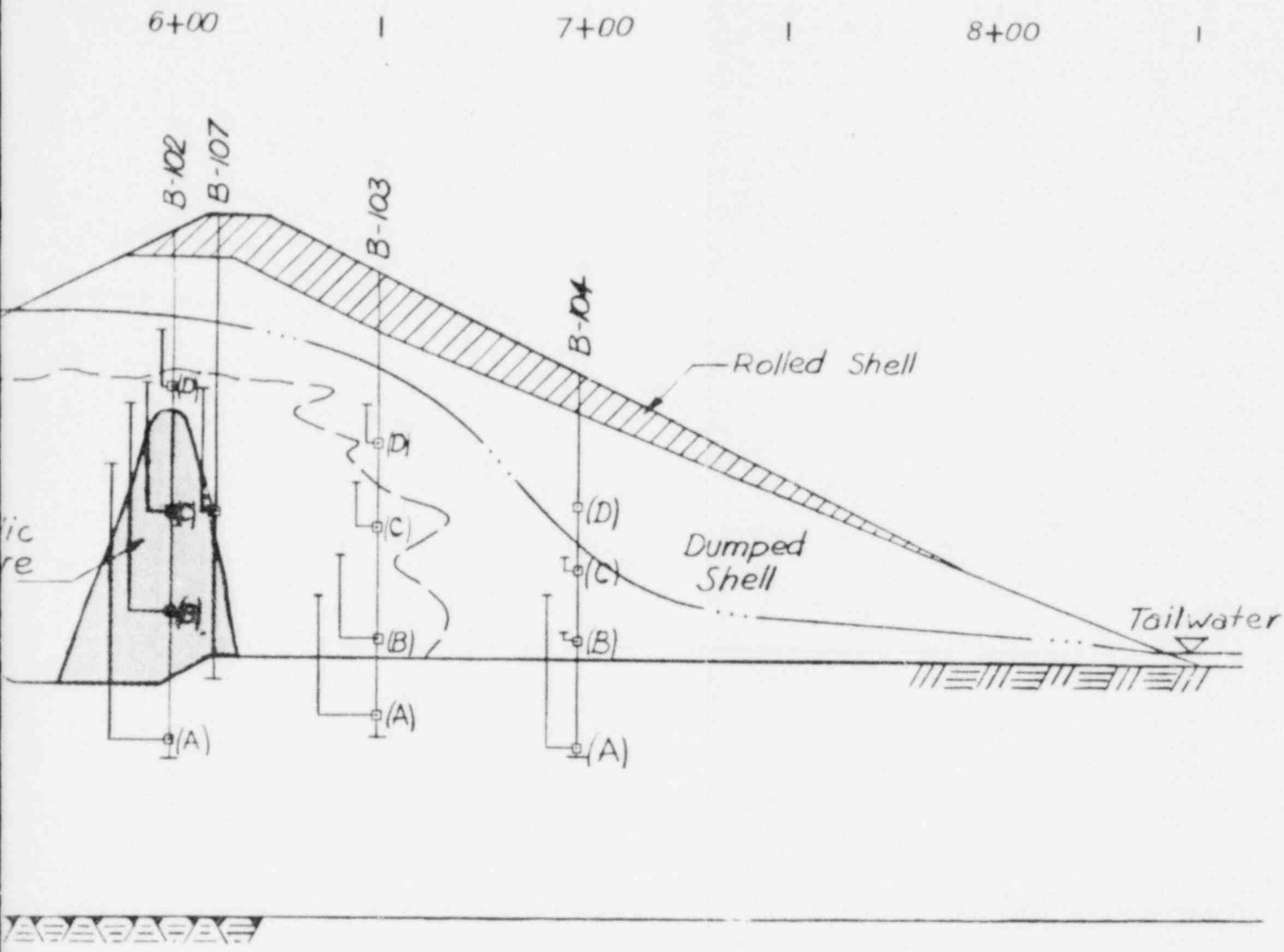
MAIN

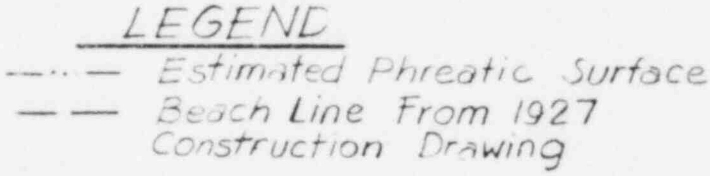
DATE	MAY 1982	
CLIENT	JOB	PLATE
1270	091	2

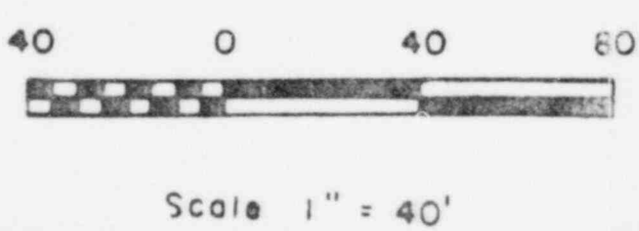


NOTES

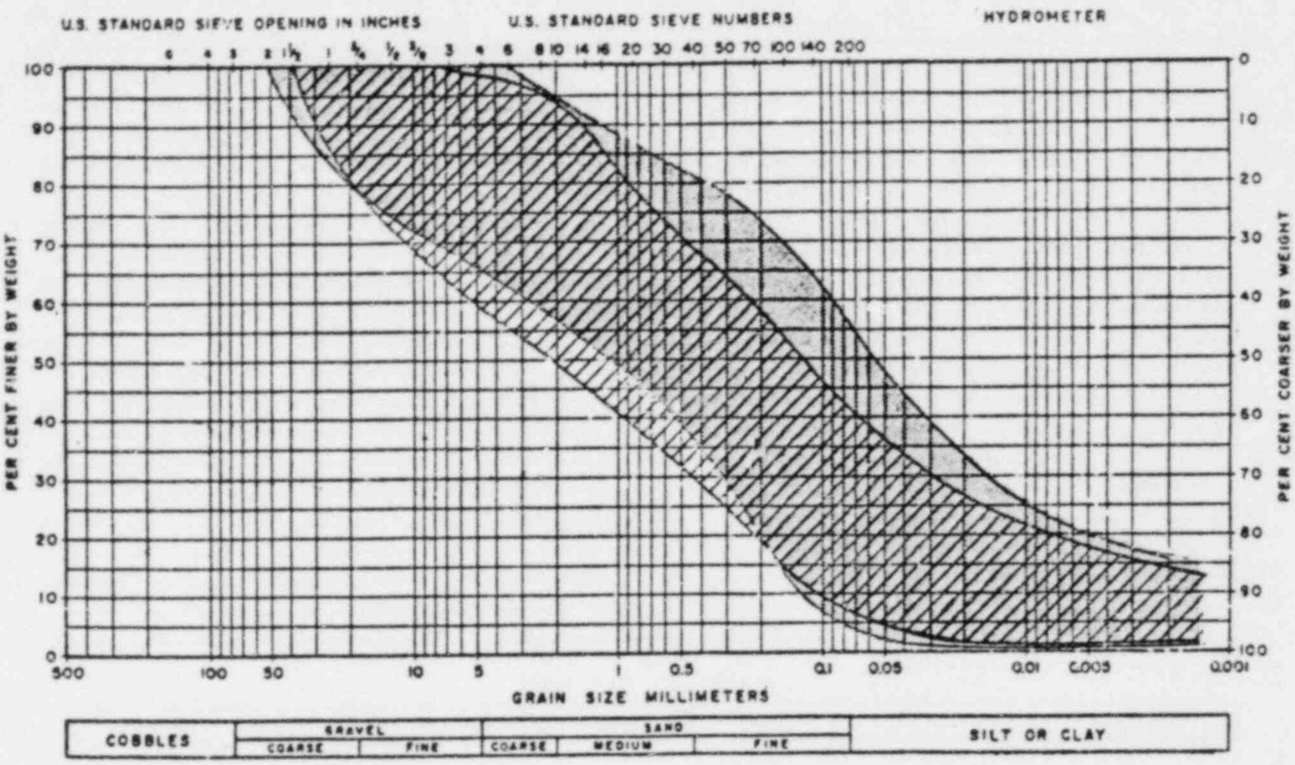
1. Piezometer Readings Correspond to Sept. 21, 1981 As Read By GEI Personnel
2. Boundaries Between Soil Types Within The Dam Are Based On Interpretation Of Soil Information From The Borings. Actual Boundaries May Be Different Than Those Shown.
3. Phreatic Surface Shown Is An Estimate Based On Piezometer Readings As Of Sept. 21, 1981. Actual Phreatic Surface May Be Different Than That Shown, And Will Vary With Fluctuations In Upstream Pool Level.
4. For NGVD, Add 105.7' To NEPCO Datum







ESTIMATED PHREATIC SURFACE AND PIEZOMETER LOCATIONS AT STATION 16+00			
NEW ENGLAND POWER COMPANY SHERMAN DAM			
MAIN	DATE	MAY 1982	
	CLIENT	JOB	PLATE
	1270	091	3



LEGEND

SHERMAN DAM

HARRIMAN DAM

ENVELOPES OF GRAIN SIZE CURVES DUMPED SHELL

NEW ENGLAND POWER COMPANY
SHERMAN DAM

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	CLIENT	JOB	PLATE
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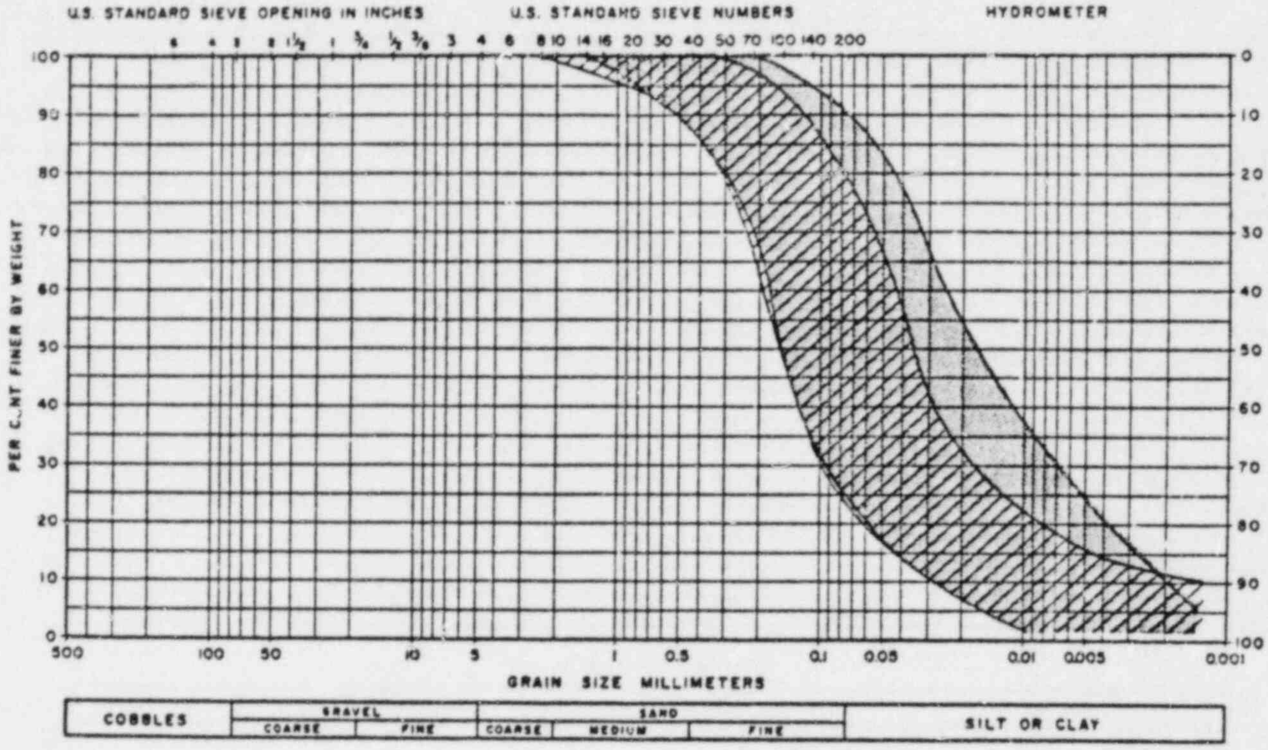


LEGEND

SHERMAN DAM

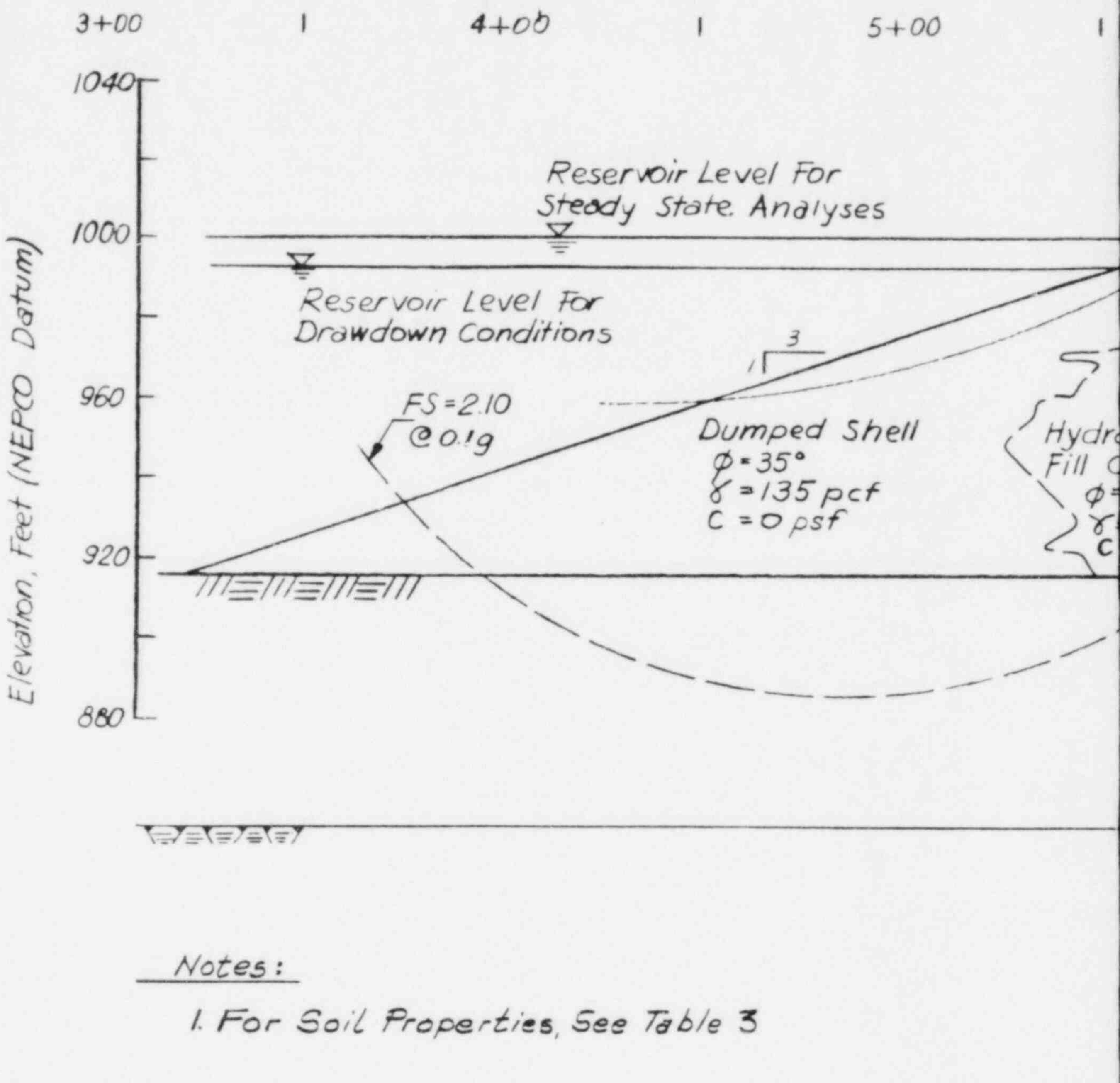
HARRIMAN DAM

ENVELOPES OF GRAIN SIZE CURVES CORE MATERIAL

NEW ENGLAND POWER COMPANY
SHERMAN DAM

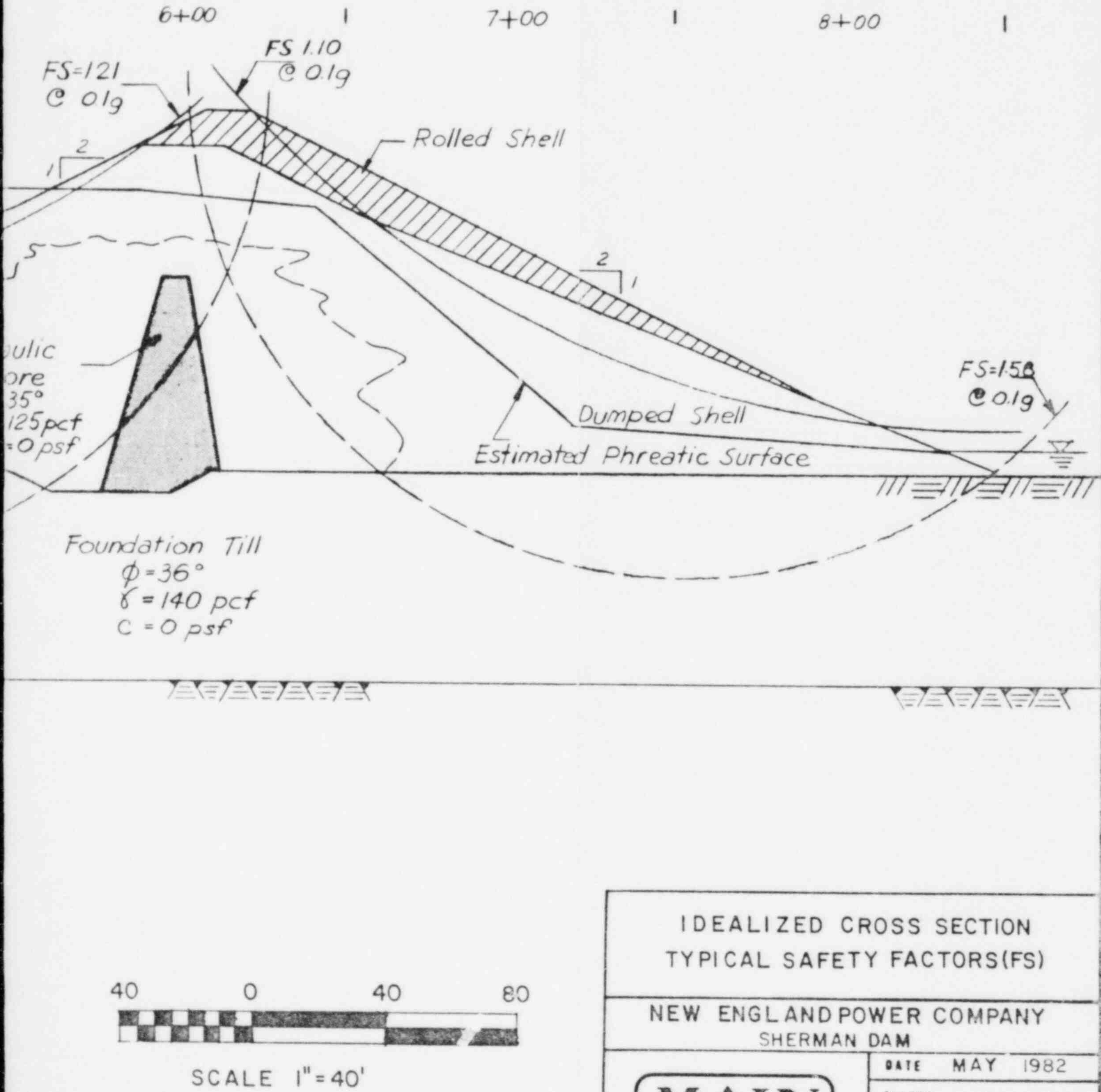
MAIN

DATE	MAY 1982	
CLIENT	JOB	PLATE
1270	091	5



Notes:

1. For Soil Properties, See Table 3



IDEALIZED CROSS SECTION
TYPICAL SAFETY FACTORS (FS)

NEW ENGLAND POWER COMPANY
SHERMAN DAM

DATE	MAY 1982
CLIENT	1270
JOB	091
PLATE	6

MAIN