NEW ENGLAND POWER COMPANY

HARRIMAN DAM

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SUMMARY OF STATIC AND SEISMIC STABILITY ANALYSES

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I. INTRODUCTION

New England Power Company retained Chas. T. Main, Inc. (MAIN) and Geotechnical Engineers, Inc. (GEI) to prepare a geotechnical report summarizing the results of recent exploratory borings, field instrumentation, laboratory soils testing, static stability analyses and seismic evaluations performed on Harriman Dam, Whitingham, Vermont. This report covers work performed during the period June 1978 to August 1981.

During this period, 47 borings were advanced into the embankment and abutments with 75 piezometer tips installed in the borings; six test pits were excavated into the embankment; and representative samplings of all zones of the embankment section with laboratory tests to identify soil types, strength parameters, and resistances to static and seismic loading conditions were performed. Measured and extrapolated pore water pressures and soil strengths measured on undisturbed samples were used to evaluate static and seismic stability and to formulate embankment improvements.

Initial field exploration and laboratory testing programs were conducted between June 1978 and August 1980. During the 1981 spring runoff, unusually high pore pressures were measured in shallow piezometers on the downstream slope. A program of close surveillance was implemented in February and extended into April 1981. Based on conservatively assumed soil strength parameters, the calculated stability during the 1981 spring runoff for shallow, noncritical failure surfaces appeared to be marginal and resulted in NEPCo imposing a voluntary reservoir operating restriction until further studies could be completed. Stability analyses for deep critical failure surfaces representing overall stablity of the dam yielded acceptable factors of safety throughout the study period.

During the spring and summer of 1981, exploration in the downstream and upstream shells was resumed to establish more realistic critical soil strength parameters applicable to the embankment. Based on this program, significantly higher soil strength parameters were found applicable to the embankment shells and the marginal embankment stability calculated in the spring of 1981 was therefore not as serious a problem as believed at that time. The embankment was stable under static conditions except for lower than desirable factors of safety for shallow potential failure surfaces. In June 1981, NEPCo requested GEI to perform an assessment of dynamic stability of the dam including a very conservative 0.2g earthquake loading condition. The results show that the existing dam will behave satisfactorily during and after a 0.2g earthquake.

Based on field observations, piezometric data and the static stability analyses, two performance problems remained: 1) potential high pore water pressures near the downstream surface of the embankment with minor surface seepage sometimes visible, and 2) factors of safety for potential shallow failure surfaces on the downstream shell less than 1.5 (a value required for new construction). NEPCo retained MAIN to formulate embankment improvements to eliminate surface seepage outbreak and the related piping potential while increasing the factor of safety for all potential sliding surfaces to a value exceeding 1.5.

Improvements should consist of a compacted processed gravel filter drain with a compacted glacial till overlay on the downstream slope. In July 1981, MAIN provided documents for construction. Construction of the proposed embankment improvements are scheduled to be completed between August and December 1981. Based on the completion of the embankment improvements, the voluntary reservoir operation restrictions imposed by NEPCo for the 1981 season can be removed and normal reservoir operations resumed in January 1982.

The results of the completed construction program will be presented in a companion report by Chas. T. Main, Inc. dated March 1982.

II. SUMMARY AND CONCLUSIONS

During the period of June 1978 through August 1981, extensive field, laboratory and analytical investi tions were performed on Harriman Dam. Undisturbed samples were obtained in the shell and core of the dam. Laboratory tests were performed on these samples to determine static and dynamic properties of the embankment soils. Pore water pressures within the dam and foundation were determined by piezometers installed in the boreholes.

Data from soil borings, piezometers, test pits, laboratory investigations and construction reports were used as input to the static and seismic stability analyses. Static analyses were performed for three critical cross sections of the dam: the maximum (highest) section (Station 10+00) and two sections with highest measured pore pressures near the slope surface, one near the left abutment (Station 12+00) and one on the right section (Station 7+00) of the dam. Seismic analyses were performed for the maximum section. Results from the stability analyses are summarized as follows:

- The static factor of safety for Harriman Dam, under maximum steady-state pool elevation (Elev. 1392.0), is greater than 1.22 for shallow slides and greater than 1.53 for deep slides; both on the downstream section of the dam. The minimum static factor of safety for the upstream slope under rapid drawdown conditions is 1.54.[1]
- Recognizing that the factors of safety on shallow surfaces are below 1.5 for maximum steady-state pool elevations, MAIN/GEI recommends that an overlay be placed on the downstream slope. The overlay should consist of a designed and processed filter blanket including associated

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[1] Rapid drawdown conditions are theoretical only; during actual operation, the maximum discharge through Harriman Power Station below elevation 1386.0 is only 1800 cfs which will lower the reservoir at a rate of about 1.7 feet per day.

perforated collection drains (to intercept any potential embankment seepage) and a compacted glacial till cover. The overlay mass would be sufficient to increase the factor of safety on all potential failure surfaces to greater than 1.5. A conceptual drawing of the compacted overlay is included in this report.

- The seismic stability analysis showed that the minimum factor of safety is 1.35 against a major flow slide or slope failure resulting from earthquake induced liquefication of embankment soils. This factor of safety is greater than the minium value of 1.1 required for new construction. Therefore, no massive flow slide could occur in the embankment.
- The deformations of the dam were calculated for 0.1 g and 0.2 g earthquakes, having estimated probabilities of exceedence on the order of 10^{-3} and 10^{-4} per year, respectively. Conservative estimates of the crest settlements are 0.6 feet and 1.2 feet for the 0.1 g and 0.2 g earthquakes. Settlements of these magnitudes would not affect the integrity of the embankment.
- Until the overlay is completed, MAIN/GEI recommend not raising the pool above elevation 1386.0 and the piezometers be read at least once a week. After the improvements have been made, no operating restrictions would be required on the pool. However, the recommended embankment surveillance program should be continued through the next FERC inspection period.

Once the recommended improvement program is complete, Harriman Dam will meet current standards for static and seismic stability for earthen dams.

III. DESCRIPTION OF DAM

a. Location, Size and Shape - Harriman Dam is owned and operated by New England Power Company (NEPCo). The dam is located in Whitingham, Vermont on the Deerfield River, as shown on Plate 1. Photos 1, 2, and 3 show the dam under normal operating conditions in 1979.

The dam was constructed during the period, June 1922 to December 1923, by the semi-hydraulic fill method using high lift dumped fill shells and a puddle core formed by sluicing the dumped shells with hydraulic monitors. Harriman Dam has a maximum height of 215 feet above the toe, is 1300 feet long and has a crest width of 12 feet with a normal freeboard of 29.5 feet above a Morning Glory spillway crest elevation 1386.0.[1] Historic reservoir operation has included six feet of stoplogs (flashboards) to elevation 1392.0. The right half of the dam is approximately 115 feet high.

The embankment section is of compound slopes (1.5H:1V to 3.5H:1V - downstream section and 1.5H:1V to 4H:1V - upstream section) with a puddle core and beach (washed) zone occupying approximately the central half of the embankment. Plate 2 shows the dam in plan view.

All elevations quoted in this report are related to NEPCo's local datum. The project zero datum is 105.66 feet above the United States Coast and Geodetic Survey mean sea level datum (NGVD).

b. Construction History and Procedures - In February 1922 construction began on Davis Bridge Dam (renamed Harriman Dam) by constructing the diversion tunnel. When the diversion tunnel excavation fell behind schedule (see Table 1 for chronology of construction events),a narrow, rock-filled timber crib wall was constructed at the upstream toe area and along the [illegible] centerline to divert

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1 The crest elevation of the dam is elevation 1415.5.

the river flow toward the left (east) bank. This permitted placement of a high lift dumped fill from the right (west) bank in the upstream third section of the dam to form the upstream cofferdam.

1. River Diversion - The crib wall ran diagonally across the majority of the channel, from left bank to right and then turned downstream, parallel to the left bank, for some 300 feet, see photo 4. Complete diversion of the river was accomplished by dumped fill operations, proceeding from both banks across the river in a single 50 foot lift once the diversion tunnel was completed.

Final closure of the upstream toe dike involved the following steps. At low flow, large rocks were rolled down the right bank to form a small rock dam towards the downstream end of the approximate 300 foot dumping face of the upstream toe dike. Approximately 200 feet upstream of this closure dike, the face of the right abutment (bank) was sluiced with a 750 gallons per minute pump to fill the voids in the rock dike and the bed of the river. It was noted that this operation gave a "tight bank and no leakage trouble", (Eaton, 1924-a). The upstream toe dike was then quickly raised to the 50 foot level for its length by side dumping from railroad cars and moving (jacking) track from both banks of the river.

2. Containment Toe Dikes - The two outer third sections of the dam formed containment dikes between which the core was placed by a hydraulic sluicing operation (see Plate 3). Construction of the upstream containment dike was as described above. The downstream containment dike was constructed similarly as the upstream dike, that is, benches were cut in the banks approximately 50 feet above the river bed on which dinkey trains with side dump cars were used to haul glacial till borrow materials to the site, see

Photo 4. Foundation clean-up under both dikes was minimal with the coarse river deposits, including boulders up to five feet in size, left in place.

The 1979 through 1981 exploration program indicates the fill in the downstream containment dike is primarily a brown glacial till consisting of silty sand with gravel and boulders. The soil is similar to tills found at other project sites in the area, such as Somerset and Sherman Dams and the Bear Swamp Pumped Storage Project. In boring B-5 of the investigation (see Plate 5), the fill was noticeably more sandy and pervious than other samples recovered from the containment dikes. Possibly, the coarse borrow materials were ear-marked for placement in this portion of the dam. In reviewing the literature, it was noted that the river bed and slope materials removed during the clean-up and cutoff trench excavations for the core foundation were dumped into the downstream containment dike (Eaton, 1924-b). After reviewing the many construction photographs available at NEPCo, it was observed that dumping fill material from high lifts tends to cause layers of cobbles and boulders to accumulate along the bottom of the lift. This results from the tendency of large rocks to roll down the slope of the lift. Photo 6 shows a construction view of the containment dikes.

3. <u>Foundation Preparation</u> - Minimal foundation preparation was done under the containment dike foundations. In these areas, foundation clean-up was limited to removal of trees and brush on abutments. The boulders, sand and gravel deposits in the river bed were apparently left in place as was the top soil on the abutments.

Preparation of the semi-hydraulic fill core foundation (the middle third of the base of the dam section), was more extensive. The layer of boulders in the river channel as well as the topsoil on abutment slopes were completely removed in the middle third of the dam. Following these clean-up operations, a cutoff trench with approximate dimensions 16 feet deep by 50 feet wide was constructed across the river channel and extended up the abutments. "Good hard bottom in the old river bed was found at about 4 feet", (New England Power Service Publication, 1922). From the "Progress Cross-Section" shown on Plate 3, it would appear that the cutoff trench, at least in the area of the river channel, was backfilled (in the dry) with glacial till similar to that material used in the construction of the containment dikes. Similarly, Plate 3 indicates the area in the middle third of the dam was backfilled with at least a few feet of glacial till over the full area stripped. In the literature, it is noted that "great care was taken in cleaning up the river bed and the middle third on either side of the old river channel", (Eaton, 1924-a).

In the literature, the foundation soils are referred to as "hard-pan with some clay" (Barrows, 1943) and as "very dense material" (Eaton, 1924-b). In borings B-5, B-103 and B-106 of the field investigations, the natural foundation soil was a dense, sandy silt till (lodgment till) which agrees with the literature description. Similar glacial till soil is visible in the cut face of the former 1964 borrow pit adjacent to and above the existing crest level on the right abutment.

4. Semi-Hydraulic Fill Placement of Core - After the containment dikes and the foundation were prepared, water was pumped into the middle third of the dam between the containment dikes and sluicing operations were begun to place the core of the dam. Two "750 gallon (per minute) pumps at 100 lbs. nozzle pressure directly connected to 100 hp motors" were mounted on individual rafts which were free to move about the puddle core pond

(Eaton, 1924-a). Monitors were directed at the fill being dumped along the dikes of the pond by the dinkey train side dump cars.

Photo 6 depicts clearly the main features of this opertion. An overflow pipe or weir was not used in this operation. Makeup water was supplied as required with a 750 gallon per minute pump located at the river channel below the dam. Above puddle core pool elevation 1310 feet, it was noted that "little makeup water was required as the elevation of the pool was raised largely by displacement due to the material washed in...with this indicating that the retaining dikes were very tight" (Eaton, 1924-a).

The effect of the sluicing operation was to wash a significant percentage of the clay and silt size fines along with considerable amounts of sand and gravel into temporary suspension until it reached the quieter waters of the puddle core. At this point, the coarser particles of sand and gravel would tend to settle out near the edge of the pool while the finer fractions would settle closer in towards the center of the pool. This hydraulic separation is shown on Plate 3, which presents gradation data on the core obtained during construction. In the literature, the "core" effective size for Harriman Dam is given as 0.01 mm (Barrows, 1943).

5. <u>Raising the Dam</u> - Two construction methods were used when raising the containment dikes above the initial 50 foot lift level. The first method involved constructing wooden trestles from which fill could be continuously side dumped until the trestle was capped off with fill. At that time, the tracks were moved (jacked) horizontally to obtain the design width of the berm. Twenty to thirty foot high trestles were used for this mode of construction. Plate 3 shows the different dike levels and where trestles were used.

A second construction method was developed during construction and found advantageous. It involved the use of a large dragline capable of placing fill to a height of 30 feet above its operating base. This method required regrading the dike slopes due to over-building the fill beyond the design slope limits. Over-building the berms provided the working space required for the dragline.

Borrow materials used for construction of the containment dikes came from the valley walls in the immediate vicinity of the dam site, with the majority coming from just upstream. Photo 5 illustrates the upstream borrow areas. Construction materials were called "glacial drift" (till) in the literature and are described as "an ideal mixture of boulders, stone, and rock dust from which to build a hydraulic fill dam," (Eaton, 1924-a). Intermittent deposits of relatively clean sands and gravels were found in the borrow areas. However, with the large dump lifts used to raise the dikes, any such materials would not have formed continous layers across the dike sections.

The puddle core was raised in more or less continuous fashion by the methods outlined in Subsection III.b.4 above.

c. Modifications (Pre-1981) - Harriman Dam was originally a 200 foot high, 1,900,000 cubic yard dam. In 1939, the dam crest was raised six feet to crest elevation 1406.0 feet. As part of the raising, a 23 foot steel sheet pile wall was installed along the centerline of the dam using a trench and backfill operation. Plate 4 shows the location of the sheet pile wall.

In 1964, the crest was again raised, this time by 9.5 feet to the present crest elevation 1415.5. The raising was accomplished by placement of an earthfill cap with an impervious core zone tied into the existing dam just behind the 1939 steel sheet pile wall. Plate 4 illustrates the earthwork details for the 1964 crest raising.

During 1979, precautionary remedial actions were taken to treat several springs along the abutments, two wet areas on the downstream embankment surface and a marsh area on the right abutment downstream toe. These seepage features had shown no significant change over the fifty-five years of project operation and existed for many years prior to the 1968 Federal Energy Regulatory Commission Safety inspection performed by MAIN. The first 1979 improvement was a primary and secondary ditch excavated to drain the marsh area and collect the seepage along the right abutment from Station 3+00 to Station 6+50. The second improvement was to remove all tree growth from the downstream slope that had developed below elevation 1320.0. The third improvement, was to install a gravel pack french drain to remove water from the left abutment downstream slope near elevation 1330.0. The maximum yield from this drain was one to two gallons per minute depending on rainfall and seemed to persist whenever the reservoir rose above elevation 1375.0.

d. Interpretive Cross-Sections - On the basis of construction history, soil borings and laboratory testing on recovered samples, interpretive cross-sections were developed for three critical sections of the dam as follows:

- Station 10+00 - The maximum section of the dam.
- Station 7+00 - Right embankment section at the "swamp" area, with high pore water pressures in the downstream shell.
- Station 12+00 - Left embankment section on bedrock with high pore water pressures in the downstream shell.

Interpretive cross-sections at Stations 7+00, 10+00 and 12+00 are presented on Plate 5. The embankment consists of high lift dumped fill shells on both upstream and downstream sections as shown on Plate 3. Adjacent to the dumped shell is a wash zone of residual shell material. This zone is made up of coarser sands, silts and gravels. The center portion of the dam is a semi-hydraulic fill core which represents the finer size fractions of the sluiced material (clays, silts and fine sands). These interpretive sections agree well with the construction sections shown in Plate 3. A complete description of the soils making up the dam is presented in Section V.b and in Table 2.

The foundation for the dam consists of bedrock on the left side of the Deerfield River channel, and glacial till on the right side of the channel. A thin layer of organic foundation soil (apparently the original topsoil) was encountered in a limited zone on the upper portion of the downstream left abutment, just over the bedrock. Detailed descriptions of the foundation materials are presented in Section V.b. and in Table 3.

e. Interpretive Longitudinal Profiles - Two longitudinal profiles have been developed based on construction history and soil borings information. Profiles presented are at the original centerline (Station 51+00) and downstream slope (Station 52+50) as shown on Plate 6.

IV. FIELD EXPLORATIONS AND DATA

a. Scope of Field Explorations - Five field exploration programs have been conducted at Harriman Dam since its construction: These programs included the following works:

- Forty-seven (47) borings in the dam, foundation soils, right abutment and downstream toe area, including undisturbed tube sampling in 16 borings and split-spoon sampling in 33 borings.
- Installation of 75 pneumatic piezometers and about 34 observation wells.
- Five in situ falling head or constant head permeability tests.
- Six test pits on the crest and downstream slope.

The locations of the borings and test pits, and the twelve observation wells still in service as of August 1981, are shown on Plate 7.

Details of the field explorations are presented in separate reports by MAIN (1979), GEI (1981-a) and GEI (1981-b).

b. Borings and Sampling - To evaluate embankment, foundation and abutment soil properties, piezometric levels, and stratigraphy, forty-seven (47) borings were performed. Borings were performed during two preliminary programs in 1958 (B-1,2) and 1979 (B-2 through B-5), and a comprehensive field program divided into three phases between December 1979 and June 1981 (GW-101, B-9 through B-14, B-101 through B-119 and B-201 through B-215).

Locations of borings are shown on Plate 7. Twenty-five borings were drilled along the three critical cross-sections investigated in this study: Stations 7+00, 10+00 and 12+00. Four of these borings were drilled on the upstream slope at Station 10+00, including one from a float in the reservoir. The remainder of the borings were distributed over the crest, downstream slope, downstream toe and right abutment. Twenty-three of the borings drilled through the dam were continued into the underlying foundation soil or bedrock.

Split-spoon sampling was performed in 33 of the 47 borings using both the standard 2-inch OD by 1-3/8-inch ID sampler and a larger 3-1/2-inch OD by 3-inch ID sampler. In 15 of the borings, split-spoons were taken nearly continuously. In the other 18 borings, split-spoons were generally taken at five foot intervals or in zones where undisturbed samples were not attempted.

Undisturbed tube samples were attempted in 16 of the 47 borings. In boring B-2 (1979 program) eight Shelby tubes were attempted in the shell and core, but no testable samples were obtained. During Phases II and III, 3-inch diameter undisturbed fixed piston tube samples were obtained as follows:

Material	Borings	Tube Attempts	Testable Samples
Dumped Shell	B-201, 202, 203, 204, 206, 208, 211	93	27
Hydraulic and Washed Core	B-106, 113, 207, 211	59	44
Foundation Soil at Station 12+00	B-210, 213, 214,	13	4

Careful measurements were made on the undisturbed tube samples to determine any changes in density during sampling, handling and transportation to the laboratory.

Detailed logs for each boring are presented in MAIN (1979), GEI (1981-a) and GEI (1981-b).

c. Corrected Blowcounts - Standard Penetration Test blowcounts were measured for each standard split-spoon sample. Blowcounts were also measured for the 3-1/2-inch OD split-spoon, which was driven using a 300-lb. weight falling 30 inches. In several borings, the 2-inch and 3-1/2-inch split-spoons were used alternately for continuous sampling and to permit comparison of blowcounts. Although the blowcount data varied erratically with depth, the comparison indicated that the two split-spoon sizes generally gave similar blowcount results.

For all of the split-spoon measurements recorded during the 100- and 200-series borings, blowcounts were measured for each inch of penetration so that the effects of gravel on the measured blowcount data could be evaluated. Data from blowcounts and samples from the dumped shell indicate less than five percent of the blowcounts were affected by gravel. Therefore, gravel had no significant effect on the measured blowcounts.

The measured blowcounts were corrected for the effect of overburden pressure using the empirical equation (Teng, 1962):

$$N' = \frac{50\ N}{\bar{\sigma}_v + 10}$$

where N = measured blowcount (blows/ft)

$\bar{\sigma}_v$ = overburden effective stress (psi)

The overburden (vertical) effective stress was computed using measured total unit weights and pore water pressures.

Corrected blowcounts for the borings at Stations 7+00, 10+00 and 12+00 are plotted in Plate 8.

d. Piezometers and Observation Wells - Following the construction of the dam, 33 shallow (15 to 30 feet deep) observation wells were installed at various locations on the dam. Weekly readings were taken weekly from approximately 1933 to 1979. During the 1964 raising of the dam crest, a number of these observation wells were abandoned. As of 1979, 11 of these wells were still operational. A review of the data obtained through 1979 led to the conclusion that these wells were responding mostly to rainwater infiltration rather than reservoir level, as discussed in MAIN (1979).

Seventy-five (75) pneumatic piezometers were installed in the dam and foundation soils during 1979-1981. The number of piezometers in each boring is indicated on Plate 7. Each piezometer was installed in a two to five foot long sanded zone in the borehole, with a bentonite or grout seal above and below the sand zone. Thus, each piezometer measures the pore water pressure at a particular point within the dam. The elevation of each piezometer is indicated on the boring logs and summarized in tables in MAIN (1979), GEI (1981-a) and GEI (1981-b). The piezometer data are presented and discussed in Section VI of this report.

One shallow observation well, GW-101, was installed to measure water levels in the pervious backfill placed in the "swamp" area at the downstream toe between Stations 4+00 and 6+50.

e. In-Situ Permeability Tests - Five in-situ falling head and constant head permeability tests were performed in B-9 and B-11 as described in GEI (1981-b). The permeabilities from four tests in the washed zone of the core ranged from 9.6×10^{-6} to 3.1×10^{-5} cm/sec. The permeability from one test in the ablation till foundation soil was 1.1×10^{-3} cm/sec.

f. Test Pits Six test pits (TP-101 through 106) were excavated on the downstream slope and crest to investigate the near-surface materials. Thirteen sand cone field density tests were performed, in TP-101 through TP-104. Bulk samples were taken in TP-101 through 104 for laboratory index testing and for preparation of compacted triaxial specimens. In TP-105 and 106, two block samples of naturally frozen shell material were recovered as a first attempt at obtaining "undisturbed" specimens of the shell for laboratory testing. Limited testing was performed on these samples since undisturbed tube samples were subsequently obtained in the shell.

V. SOIL PROPERTIES

a. Laboratory Testing Program - An extensive laboratory testing program was performed to measure index and engineering properties of the embankment and foundation soils. Index properties consisting of unit weight, water content, specific gravity, Atterberg limits, and grain size distribution were measured on undisturbed tube, split-spoon, frozen block and bag samples.

Engineering properties were measured primarily on undisturbed specimens. Static friction angles were measured using consolidated undrained ($\overline{R}$) and consolidated drained (S) triaxial tests. Resistance to cyclic loading was determined from isotropically and anisotropically consolidated cyclic triaxial triaxial ($C\overline{R}$) tests. Dynamic shear modulus and damping were determined using resonant column (RC) and small strain cyclic triaxial (E) tests.

The shape of the steady-state shear strength line for the dumped shell was determined from $\overline{R}$ tests on compacted specimens.

All undisturbed tube samples were X-rayed prior to testing and only specimens that did not contain large pieces of gravel, significant voids or other evidence of major disturbance were used for testing. Photographs were taken of each undisturbed specimen after testing and are contained in GEI (1981-a) and GEI (1981-b).

Undisturbed samples were obtained in the lower blowcount and less gravelly zones of the dumped shell and washed zones of the core. These zones appear to be the only materials which could be successfully sampled. In addition, care was taken to test samples from nonplastic zones of the hydraulic core, rather than the slightly plastic zones. Thus, the laboratory tests were performed on the more critical materials in the dam and may provide a slightly conservative representation of the strength properties of the embankment soils.

Details of the test procedures and plots of individual test results are presented in GEI (1981-a) and GEI (1981-b).

b. Description of Soils - Each split-spoon sample was carefully logged by a geotechnical engineer or engineering geologist in the field before it was removed from the sampler, with particular attention given to noting stratification or other features of the soil structure. For all Phase I, II, and III borings, split-spoon samples were reclassified in the laboratory where visual comparison could be made between samples. Each undisturbed sample was carefully described and photographed at the completion of testing.

Based on these visual examinations and the results of the index and gradation tests, general descriptions of each of the embankment and foundation soils were developed. These descriptions are presented in Tables 2 and 3.

c. Index Properties - Unit weights and natural water contents were measured on undisturbed tube samples from the core and shell and from field density tests on the rolled shell. The test results are summarized as follows:

Material	Number of Tests	Total Unit Weight (pcf)		Water Content (%)	
		Range	Average	Range	Average
Dumped Shell	25	131.1-146.3	137.6	11.9-20.7	16.3
Rolled Shell	3	143.7-151.8	148.5	8.2-8.5	8.3
Core	36	111.6-142.1	130.1	12.3-34.6	20.9

The specific gravity measured on 20 samples of core and shell ranged from 2.70 to 2.83, with an average value of 2.75.

Atterberg limits were performed on 14 samples of core and shell, using material passing the No. 40 sieve. The liquid limits ranged from 16 to 33. The plasticity index ranged from 0 to 14, with most values less than 7. These data indicate that the shell and core are generally nonplastic or low plasticity soils.

d. Gradations - Gradation analyses were performed on 146 samples, including all undisturbed test specimens and a number of split spoon samples. Both sieve and hydrometer analyses were performed for most specimens.

Gradation bands for the dumped shell, washed zone and hydraulic core of the dam are shown on Plate 9. These bands cover all the gradation tests performed on these materials except for a few samples which were considered not representative of the particular material type.

e. Static Strength - Effective stress friction angles were determined on undisturbed samples of shell and foundation soils using anisotropically consolidated $\overline{R}$ and S tests. Consolidation stresses for these tests were generally selected to be similar to in-situ effective stresses.

During transport, set-up, and consolidation, all undisturbed specimens densified somewhat. For the core and shell specimens, the as-tested dry density ranged from 0.1 to 9.7 pcf higher than the sample density in the tube when removed from the ground. Most specimens densified between two and seven pcf. Since stress-strain curves and stress paths are sensitive to density variations, the $\overline{R}$ and S test curves may not be representative of the in-situ soil. However, effective stress friction angles are less sensitive to density changes. Therefore, the measured friction angles are considered representative of the in-situ soil.

The range of friction angles at maximum stress ratio from 16 $\overline{R}$ tests on core samples was 33.0° to 37.5°, with an average value of 35.5°. In all but four tests, the friction angles were equal to or greater than 35°. Four S tests on core specimens indicated friction angles of 36.3° to 39.4°, with an average of 37.6°. Typical stress paths for the $\overline{R}$ tests on core samples are shown on Plate 10.

For the dumped shell, the range of friction angles for 12 $\overline{R}$ tests were 34.0° to 37.8° with an average of 36.1°. One S test and two constant-q tests [1] on shell specimens indicated friction angles of 36.2° to 38.9°, with an average of 37.3°. Typical stress paths for the $\overline{R}$ tests on dumped shell are shown in Plate 10.

Four $\overline{R}$ tests on the organic foundation soil at Station 12+00 had a range of friction angles from 37.8° to 44.1° with an average of 41.4°.

Three of the $\overline{R}$ tests on shell specimens and three on foundation specimens were subjected to cyclic loading prior to the monotonic $\overline{R}$ loading. The results of these tests indicated that cyclic loading had no effect on friction angle or pore water pressure behavior during the subsequent monotonic loading.

Five $\overline{R}$ tests on shell specimens from shallow depth (20 to 40 ft) and five on core specimens reached approximate steady-state conditions [2] during shear.

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[1] Drained triaxial test failed by maintaining constant deviator stress while reducing effective confining pressure by raising pore pressure in drained increments.

[2] Steady-state conditions are defined as a constant rate of deformation at constant volume with constant shear stress and pore pressure. The steady-state shear strength is the minimum strength which can occur at a given effective stress or void ratio. The shear strength is reduced to the steady-state value only after very large shear deformations. For further discussion, see GEI (1982).

The in-situ undrained steady-state shear strengths, estimated by correcting the $\overline{R}$ test strength to account for the effect of sample densification during sampling, handling and consolidation, are discussed in GEI (1982).

f. Cyclic Load Resistance - Thirteen $C\overline{R}$ tests were performed on undisturbed samples from the core. Seven of the tests were isotropically consolidated, and six were anisotropically consolidated ($K_c = \overline{\sigma}_1/\overline{\sigma}_3 = 2.0$). Thirteen $C\overline{R}$ tests were also performed on undisturbed shell samples, three of which were isotropically consolidated and 10 were anisotropically consolidated. Three $C\overline{R}$ tests were performed on anisotropically consolidated specimens of the foundation soil at Station 12+00.

The consolidation stresses were generally selected to be similar to the in-situ effective stresses. The range of cyclic stresses in the $C\overline{R}$ tests covered the range of seismic stresses induced in the dam by the 0.1g and 0.2g earthquakes used for analysis.

The results of the $C\overline{R}$ tests are summarized on Plate 11 for both embankment and foundation materials. The anisotropic data, which are more representative of in-situ consolidation conditions, indicates that the dumped shell and hydraulic fill core have similar cyclic load resistance. The washed zone of the core appears, on the basis of limited data, to be slightly less resistant to cyclic loading than the shell or the core.

Accumulated cyclic shear strain versus cyclic shear stress ratio for five cycles of loading were plotted based on anisotropic (K_c=2.0) $C\overline{R}$ test data from the dumped shell and hydraulic core samples, as shown on Plate 12. The cyclic stress ratio for each test was corrected for void ratio changes during handling and consolidation and for loss of "aging effects" during sampling, (GEI (1982)). The cyclic shear stress ratio is $\tau_{fy}/\overline{\sigma}_{fc}$, where τ_{fy} is the cyclic shear stress on the failure plane, $(45 + \emptyset/2)$, and $\overline{\sigma}_{fc}$ is the normal consolidation stress on the failure plane. This ratio is directly comparable to the earthquake stress ratio $\tau_{avg}/\bar{\sigma}_v$ described in Section VII.C. Most of the $C\bar{R}$ tests were performed at an effective confining stress ($\bar{\sigma}_{3c}$) equal to 2750 psf. From the $C\bar{R}$ test data, a curve of accumulated strain vs. stress ratio for $\bar{\sigma}_{3c}$ = 2750 psf was plotted. A curve with similar shape was plotted using results of two $C\bar{R}$ tests performed at $\bar{\sigma}_{3c}$ = 1250 psf. Extrapolation of the curves to higher values of $\bar{\sigma}_{3c}$ was based on published $C\bar{R}$ test data for several different sands and gravelly sands, as described in GEI (1982). Similar cyclic strain versus cyclic stress ratio curves for the washed zone were constructed using 75% of the cyclic resistance for the dumped shell and hydraulic core.

g. Dynamic Properties - Dynamic shear modulus, G, and damping ratio, D, were measured on undisturbed specimens from the dumped shell and hydraulic core. Two resonant column (RC) tests and one small strain cyclic triaxial (E) test were performed on each material.

Based on the test data, a plot of maximum shear modulus at low strain, G_{max}, versus mean effective confining pressure, $\bar{\sigma}_m$, was developed, see Plate 13. Plots of the decrease in shear modulus and the increase in damping with increasing shear strain level were also prepared, as shown in Plate 13. The shear modulus versus shear strain and damping versus shear strain curves for the shell are similar to the average curves for cohesionless soils presented by Seed and Idriss (1971). The curves for the hydraulic core are similar to data for low plasticity silty soils presented by Kim and Novak (1981).

VI. SEEPAGE CONDITIONS

a. General - Since the construction of Harriman Dam there have been a number of observed wet spots on the downstream face above about elevation 1325.0. Several efforts have been made to collect this seepage and remove it from the slope. In June 1928, Mr. A. C. Eaton of New England Power Construction Company dug a small ditch into the face of the dam for a distance of 10 to 15 feet and then back-filled it with cobbles to serve as a collection zone and drain. Between the period 1936 to 1964, 33 well points were installed in the embankment. Several of the early wells were installed horizontally and acted as relief drains. The remaining vertical or sloping wells were used to measure the distance to the phreatic surface. In August of 1979, NEPCo installed a French Drain between Stations 11+00 and 13+00 at elevation 1330± to collect seepage from the surficial wet areas and transfer it to the left abutment.

b. Pore Water Pressures - During the period July 1979 to June 1981, 75 piezometers were installed in the dam; 74 are currently functional. Through August 1981, these units were read weekly during relatively steady pool levels and at daily intervals during high water conditions. Reservoir levels normally reach elevation 1386.0 (top of spillway crest) each year with a historic high reading of 1392.4 (with flashboards) on December 31, 1948. Flashboards can be installed to elevation 1392.0 for temporary storage during the short period of high flow during spring runoff; however, they are not installed unless spring floods are unlikely and the boards are always removed in the fall. The maximum steady-state pool elevation is 1392.0; however, the pool is maintained only one or two days at this level. Data from each piezometer along with daily rainfall quantities and reservoir elevations, are logged by NEPCo. Plates 14, 15a, 15b and 16 illustrate a representative set of piezometer readings from Stations 7+00, 10+00 and 12+00 for the period of January 1980 through August 1981. Piezometric data from pre-1980 are presented in MAIN (1979).

From the piezometer data, it was observed that pore water pressure response in the embankment follows the reservoir with a one day to two week lag. Therefore, steady-state conditions are unlikely to be reached during the one to two day duration at a reservoir level of elevation 1392.0.

Upstream rapid drawdown for the reservoir is limited to a maximum of 1.7 feet per day below elevation 1386.0 when running 1800 cfs through Harriman Power System. On the basis of the preceeding discussion and the method of construction (timber crib under the upstream shell), it is believed that pore water pressures within the upstream shell during rapid drawdown follow and are only slightly higher than the reservoir level.

Since the piezometers have been in operation, no data had been obtained for a pool level above elevation 1381.4 (as of August 1981). To perform slope stability analyses, it was necessary to extrapolate instrument records to maximum reservoir levels (elevation 1392.0). Data from two periods in 1980, using both rising and falling reservoir data trends, were used to extrapolate pore water pressures for a 1392.0 steady-state pool elevation. Details on how the extrapolations were performed are presented in Section VII. In general, both rising and falling reservoir projections yielded similar results which were used to establish the estimated flownet for the static stability analyses.

c. Equipotential Lines (Lines of Constant Head) - Minor visible seepage and measured pore water pressures close to the embankment surface showed that the phreatic line is near the ground surface on the upper third of the downstream slope. During late February 1981, the combination of high pool elevation, a 55 year record monthly rainfall and unseasonably warm temperatures while the frost was still in the ground caused unusually high pore water pressures in the shallow downstream slopes. Pore water pressures recorded during February - March 1981 were used to construct lines of equipotential pressure (flownets) for Stations 7+00 and 12+00. Plates 17 and 18 illustrate pore water pressure levels for Stations 7+00 and 12+00, respectively.

One likely explanation for high pore water pressure close to the embankment surface is the presence of the seven downstream shell construction berms shown on Plate 3. These seven berms could easily influence seepage patterns on the downstream slope by causing local perched groundwater tables. Existence of such berms suggests alternating layers of relatively impermeable and permeable material due to construction traffic compaction. Dumped soil near the surface of the construction berms would be compacted by the dinkey-trains, large heavy drag lines and other construction traffic. This would cause the surface of the berms to be less pervious than the main body of the berm.

Piezometer data to date indicate that seepage through the embankment intersects the construction berms, see Plates 17 and 18. It appears that the wet spots located at Station 12+00 at elevations 1360.0 and 1330.0 coincide with the construction berms at elevations 1355.0 and 1323.0. The wet spots at elevation 1330.0 extend from Station 10+00 to the left abutment. The occurence of surface seepage water at these berms supports the notion that the top of these established berms are impervious, which causes seepage water to be perched. The impervious nature of the berm surfaces will extend the phreatic surface (line of zero pore water pressure) further toward the downstream slope. In addition, material dumped from the berm levels was washed toward the center of the dam, resulting in alternating layers of fine and coarse grained material. The slope of these layers is generally shallow. As shown on Plates 17 and 18, the slope of the equipotential lines closely parallel washed slopes in the sluiced zone.

Thus, the construction methods associated with the berms may explain why the equipotential lines tend to be almost horizontal.

The shallow slope of the equipotential lines means that pore water pressures in the downstream shell are significantly less than hydrostatic based on the phreatic surface. Therefore, actual pore water pressures at a point rather than hydrostatic pressures below the phreatic surface should be used for stability analyses.

VII. STATIC STABILITY

a. General - Since original construction in 1922-3, no significant displacement of the crest or slopes has been observed. There have been no large slides on either the downstream or upstream slopes of Harriman Dam. The crest has been regraded to compensate for nominal crest settlements (1939, 1964 and 1981). Several shallow frost related slides have occurred on the downstream slope. During the spring of 1928, "a couple of surface slides on the downstream slope of the dam within the upper third of the elevation... no great depth, possibly a foot or 18 inches at a maximum... at the time the frost is coming out in the spring." (Inspection Report, A. C. Eaton, June 8, 1928). A second slide occurred on March 28, 1936. This slide occurred near Station 8+00 at approximately elevation 1330. "This slide moved down about 30 feet and is probably 150 feet wide. The maximum depth of the slide is about 5 feet... Undoubtedly, this is a frost slide and was caused by the recent rains and warm weather." (NEPCO Office Memo., H. L. Hurd, April 2, 1936). Both slides were due to frost action combined with spring snowmelt and heavy warm rain. NEPCO repaired all of the minor slides immediately after occurrence. Since 1936, no evidence of distress has been observed at the dam. No frost-related slides were observed during the period of sudden thaw and high pore pressures in February 1981.

b. Soil Properties - Physical properties for the embankment and foundation materials were determined from field and laboratory data presented in Sections IV and V. Table 4 highlights the soil properties used for the static stability analysis. Properties for the dumped shell, rolled shell and core were based on a substantial number of laboratory tests on undisturbed samples. In general, the properties measured for the embankment soils were from samples representative of the less dense and more easily sampled zones of the dam, therefore, values selected are conservative. In addition, the friction angles selected for the dumped shell and core are slightly below the average. The combination of a large data base and adopting lesser than average soil strengths will provide conservative stability results. Physical properties for the abutment soils and bedrock were assumed from typical values reported in the literature. The thin layer of organic foundation soil encountered at Station 12+00 on the left downstream abutment was not included in the stability analyses. Investigation revealed that this soil exists only in a limited zone on the upper portion of the left abutment (see Plate 6, Profile at Station 52+50).

c. Pore Water Pressures - Actual pore water pressures in the dam were determined for various pool elevations from piezometric records presented in Section VI. Recorded piezometer readings for pool elevations (1349.0-1381.4) were evaluated and flownets were constructed for steady-state pool elevations 1370.0, 1377.5 and 1381.0. The number of piezometers installed at stations 7+00, 10+00 and 12+00 were sufficient to define reasonably accurate flownets.

To assess downstream slope stability at maximum pool, the measured pore water pressures were extrapolated to the maximum steady-state pool elevation of 1392.0. The extrapolation was performed using a best-fit line on an arithmetic plot of the measured pore water pressures during June to September 1980 when the pool dropped steadily from elevation 1383.0 to 1372.0. A similar extrapolation was performed for a rising pool condition in October-November 1980. Both extrapolations yielded similar estimates of pore water pressures corresponding to elevation 1392.0 pool level. The extrapolated pore water pressures provided data for graphical construction of a phreatic surface and flownets (lines of equipotential pressure) for critical cross-sections within the embankment under the maximum pool condition.

d. Method of Analysis - Using information available on construction methods, soil properties and pore water pressures, slope stability analyses were performed on the three critical crosssections (stations 7+00, 10+00 and 12+00) for various reservoir elevations. The simplified Bishop method of slices for analyzing circular arc failure surfaces was used. The simplified procedure assumes the orientation of inter-slice forces as constant and then sums to zero. Extensive computational experience demonstrates this is a conservative assumption. The procedure adopted by MAIN is outlined in Bishop (1955) and Whitman and Bailey (1967).

MAIN's computer program SLØPE which incorporates the Simplified Bishop Method of Analysis, was used to locate critical failure surfaces. The program automatically searches for the critical failure surface by changing the center coordinates and radius of an initial trial failure surface by five-foot increments converging toward the center and radius defining the critical failure surface. The program stops when the failure arc's value of factor of safety against sliding (FS) becomes a minimum.

In SLØPE, pore water pressures on the failure surface are computed as the vertical distance from the phreatic surface to the failure arc multiplied by the unit weight of water (62.4 pcf). This method is correct for horizontal water surfaces and highly conservative for steeply sloping phreatic surfaces.

Flownets constructed at stations 7+00 and 12+00 on the dam, as shown on Plates 17 and 18 demonstrate shallow sloping equipotential lines. For such flownets, the SLØPE program will substantially overestimate pore water uplift pressures on the trial failure surface resulting in too low a factor of safety. However, the computer can be used to define potential critical surfaces. Although the location of the critical failure surface is a function of the uplift pressure distribution, experience has shown that the computer is sufficiently accurate.

Once the computer had identified the critical surface, the factor of safety was recomputed by hand for any critical surface with a factor of safety less than 1.5 using the actual pore water pressures from the flownets to achieve a more accurate solution.

e. Stability Results - August 1981 Conditions - Plate 19 summarizes the results of the Static Stability analysis of Stations 7+00, 10+00 and 12+00 analyzed under maximum (pool elevation 1392.0) steady-state pool conditions. The SLØPE stability analysis at the maximum downstream section Station 10+00 yielded a factor of safety of 1.81 for the critical deep failure surface.

Shallow and deep surface slope stability analyses were performed at Stations 7+00 and 12+00 using the computer program SLØPE and hand computations. For all failure surfaces the hand calculated Bishop analysis is more accurate than SLØPE since it more accurately interprets the actual uplift pore water pressures. Therefore, only hand Bishop results will be reported for downstream failure surfaces at Stations 7+00 and 12+00. The critical shallow surface at Station 7+00 had a factor of safety of 1.24. Shallow surfaces at Station 7+00 exhibited FS less than 1.5 for a steady-state 1392.0 pool; therefore, additional Bishop analyses were performed on failure surface 7A, providing a relationship between FS and steady-state pool elevation. Plate 19 illustrates FS versus pool elevations for surface 7A in the upper right-hand corner of the drawing.

During late February 1981, the reservoir pool rose to elevation 1381.4; the resulting FS equaled 1.39 under these conditions. The factor of safety (FS) for deep surface 7B equals 1.55.

At Station 12+00, FS is characterized by shallow failure surface 12A and deep surface 12B. The factors of safety were 1.22 and 1.53 for shallow and deep failure surfaces. The relationship between FS and steady-state pool elevation for failure surface 12A is shown on Plate 19 in the upper right-hand corner. The factor of safety for surface 12A during late February 1981 was 1.38.

Stability of the upstream slope during rapid drawdown was analyzed and the results are presented on Plate 19. Due to limited release capability below elevation 1386.0, the pool cannot be lowered at a rate greater than 1.7 feet per day. The design rapid drawdown condition was actually experienced during Spring 1981 from elevation 1381.4 to 1351.0 with no visible effect to the dam. The calculated FS for rapid drawdown to elevation 1351 is 1.79 for failure surface 10B. The maximum possible drawdown would be to elevation 1300, illustrated by failure surface 10C resulting in a FS of 1.54. Thus the dam is adequately stable under all rapid drawdown conditions.

Static stability was evaluated under Probable Maximum Flood (PMF) conditions. PMF calculations assume an initial steady-state pool of 1386.0; and a maximum PMF pool elevation of 1415.5 (zero freeboard) with pore water pressures, internal to the dam, equal to a 1392.0 steady-state pool. Review of the PMF data, indicates that the duration period for the PMF event will be less than 36 hours; therefore, embankment pore water pressures will not have time to respond. Resulting sulting factors of safety equal 1.74 for failure surface 10A* (deep surface stability) at Station 10+00 and 1.41 for intermediate failure surface 12C at Station 12+00. Plate 19 illustrates the results.

f. Proposed Modification - On the basis of visual observations and readings from piezometers installed on the downstream slope, high pore water pressures were identified at several locations along the downstream face of the dam. These high pore water pressures led to factors of safety less than 1.5, on shallow failure surfaces at Stations 7+00 and 12+00, for a maximum pool elevation of 1392.0.

It was determined that a downstream overlay fill/drain would raise the factor of safety above 1.5 for maximum estimated steady-state seepage at pool elevation 1392.0. The overlay would eliminate the potential for seepage breakout by providing a filter drain blanket extending from the downstream toe up to a minimum elevation of 1374.0. (Note: Seepage breakout itself is not a failure mechanism, however, it provides an opportunity for "piping" to begin). Sufficient compacted fill would be placed over the drainage layer to raise the static factor of safety on all potential failure surfaces to greater than 1.5.

The proposed improvement is shown on Plate 24. A drawing similar to this was submitted to the FERC in June 1981 as part of the license application by NEPCo required to permit construction of the proposed modifications in 1981. The improvement includes: 1) stripping the downstream face, 2) installing a compacted filter drain blanket and placing a compacted fill on top of the filter which will provide mass to increase the static factors of safety, 3) installing collection drains to monitor seepage water, 4) constructing a service road across the top of the overlay, 5) installing several new piezometers and 6) extending existing piezometer leads in trenches below the overlay to monitoring stations adjacent to the dam.

Stability analyses were performed on the proposed overlay using the computer program SLØPE and hand Bishop analysis techniques. Piezometric pressures were estimated for a steady-state pool elevation of 1392.0 as previously discussed. Stations 7+00, 10+00 and 12+00 were analyzed and Plate 20 illustrates the results. Station 12+00 represents the section with the minimum Factor of Safety. Factor of Safety for intermediate surface 0-12B equals 1.73 using material properties noted earlier. The location of failure surface 7A and 7B on Plate 20 are identical with surfaces 0-7A and 0-7B on Plate 20 to permit direct comparison.

In addition, overlay stability was evaluated for PMF conditions (zero freeboard). The factor of safety for deep failure surface O-10A* equals 1.81. FS for intermediate surface O-12D equals 1.67 at Station 12+00. For all test cases, the factor of safety is greater than 1.5.

VIII. SEISMIC STABILITY

a. Method of Analysis - The seismic stability analyses for Harriman Dam were performed on the maximum section of the embankment including the proposed overlay, at Station 10+00.

The technical approach used to evaluate the behavior of the dam during and after an earthquake follows:

- Evaluate the factor of safety against a major flow slide or slope failure resulting from earthquake-induced liquefaction of the embankment soils.
- Estimate deformations in the core and shell during seismic loading from the computed seismic shear stresses and the results of cyclic triaxial ($C\overline{R}$) tests.
- Compare Harriman Dam to another existing dam, Lower San Fernando Dam, which was subjected to an 0.55g earthquake and experienced a major slide on the upstream slope.

b. Seismic Stability Analysis - The first step in the seismic analysis was to determine the factor of safety against a major flow slide or slope failure resulting from liquefaction of the embankment soils induced by earthquake loading.

Liquefaction is a phenomenon in which a mass of soil loses a significant percentage of its shear strength when subjected to undrained static, cyclic or shock loading. The loading converts the soil mass from a drained condition, in which it can sustain the in-situ shear stresses, to an essentially undrained condition in which the shear resistance is less than the imposed shear stresses. The soil then deforms until the shear stresses acting on the mass are as low as the reduced shear strength, termed the steady-state shear strength.

The laboratory data indicated that the in-situ undrained steady-state shear strengths for the shallow dumped shell (less than 60 feet depth) and for the core were generally less than the drained strengths. Therefore, the potential for liquefaction of these materials required further evaluation. To evaluate whether flow slides or slope failures could occur (i.e., whether the static driving stresses were greater than the steady-state strengths), stability analyses were performed using steady-state shear strengths.

The in-situ steady-state shear strengths for the shallow dumped shell and core material were determined from the laboratory $\overline{R}$ tests, with corrections for the change in sample void ratio during handling and consolidation as described in GEI (1982). Below a depth of 60 feet in the dumped shell, the corrected blowcounts increased noticeably, indicating a denser soil. Even a small density increase would be sufficient to increase the undrained steady-state strength to values above the drained steady-state strength. Therefore, an undrained strength equal to the drained strength was used for the deeper zones of dumped shell. Drained steady-state strengths were calculated using the steady-state friction angle determined from the R tests on compacted specimens.

Stability analyses were performed using the sliding-wedge method. A series of trial wedges intersecting the crest and upstream slope were analyzed to identify the critical surface. The minimum factor of safety was 1.35 for a downstream wedge along the base of the dam, extending up through the washed zone of core to the upstream slope at about El. 1400.0. For seismic stability, a factor of safety of 1.1 or greater is adequate. Therefore, the dam is safe against a major flow slide or slope failure caused by liquefaction. Details of the seismic stability evaluation are contained in GEI (1982).

c. Deformations Induced by Seismic Loading - Even though the dam is stable against an earthquake-induced flow slide, the dam may undergo deformation as a result of the earthquake. Calculated deformations were based on estimated seismic shear stresses and on cyclic stress-strain data from laboratory cyclic triaxial tests on undisturbed specimens.

For this study, the Yankee composite earthquake spectrum (Yankee 1981) with a peak ground acceleration of 0.1g was selected as a conservative design input earthquake. This earthquake spectrum was developed for the Yankee Rowe Nuclear plant six miles downstream of the dam, as described in Weston (1979, 1980) and Yankee (1981). The Yankee composite spectrum has a 10^{-3} probability of being exceeded in any given year.

For comparative purposes, analyses were also performed using the more conservative NRC recommended response spectrum (NRC 1981) for the Yankee Rowe plant, which has a peak ground acceleration of 0.2g. The probability of the NRC spectrum being exceeded is about 10^{-4} in any given year.

An artificial earthquake record, referred to as the "Housner" earthquake, was used to model the Yankee composite and the NRC recommended spectra in these analyses. The Yankee composite and NRC recommended spectra and the scaled "Housner" records are shown on Plate 21.

The one-dimensional program SHAKE (Schnabel et al., 1972) was used to evaluate the accelerations and seismic shear stresses in the dam during an earthquake. Dynamic soil properties for the dumped shell and core used in the analysis were based on the results of resonant column and small strain cyclic triaxial tests on undisturbed specimens. The dynamic soil properties are described in Section V.g. and shown on Plate 13 of this report. Three SHAKE analyses were performed at Station 10+00: one at the crest (transverse Station 51+07), one in the middle of the upstream slope (transverse Station 49+57), and one in the middle of the downstream slope (transverse Station 52+30). Plots of maximum seismic shear stresses versus depth for these three locations are shown in Plate 22.

For evaluation of shear strains in the dam, the irregular earthquake stress history with maximum shear stress, τ_{max}, from SHAKE, was equated to five cycles of equivalent uniform cyclic stress, τ_{avg}, was using the relationship of $\tau_{avg} = 0.65\tau_{max}$. Selection of five cycles was based on the maximum expected earthquake magnitude, M = 6.0, in the geologic province containing the site or in adjacent provinces (Yankee, 1981). Using the relationship between earthquake magnitude and equivalent number of cycles from Seed (1976), five cycles lie on the mean plus one standard deviation line for an M = 6.0 earthquake and is, therefore, an appropriately conservative value for design. For a comparison, seismic strains were also analyzed for the more conservative equivalent loading of seven cycles.

The shear strains in the dam were computed from the normalized earthquake shear stress $\tau_{avg}/\bar{\sigma}_v$, using the laboratory $\overline{CR}$ test results (described in Section V.f and shown on Plate 11). Profiles of shear strain versus depth for the three locations analyzed are shown on Plate 23.

Crest settlements resulting from the earthquake strains were calculated assuming that the horizontal strain profiles on the upstream and downstream slopes represented outward movement of a volume of soil. This outward soil movement was assumed to cause an equal volume of crest and slope settlement between these two profiles. The resulting settlements were:

	5 cycles loading	7 cycles loading
0.1g earthquake	0.5 ft	0.6 ft
0.2g earthquake	1.0 ft	1.2 ft

The magnitude of estimated horizontal deformations and crest settlements indicate that the deformations from either the 0.1g or the 0.2g earthquake will not be sufficient to affect the integrity of the dam. Therefore, the dam has adequate seismic resistance for either earthquake.

The strains and deformations computed using the methods described above are considered conservative for the following reasons:

- The dumped shell specimens used for laboratory testing generally represent the less gravelly and lower blowcount soils of the dam, since these were the only dumped shell materials which could be sampled successfully.
- The design spectra for these analyses had a much higher energy content for the range of periods equal to the natural period of the dam, i.e., T = 0.5 to 1.5 sec, than shown by actual earthquake records obtained at short epicentral distances for magnitude 6.0 earthquakes in California and in other seismically active areas of world.
- Strains developed in laboratory $\overline{CR}$ tests are somewhat exaggerated relative to field behavior due to cumulative test errors and differences between stress paths in-situ and in the laboratory.

The effects of the crest settlements resulting from either the the 0.1g or the 0.2g earthquake would be minor due to the following factors:

- Freeboard for the dam is about 23.5 ft, even at the maximum reservoir operating level of elevation 1392.0.
- Transverse cracking of the dam is not expected as a result of the small estimated settlements; however, the widely graded dumped shell materials would be self-healing even if transverse cracking should occur.

d. Comparison to Lower San Fernando Dam - As a separate indication of the seismic stability of Harriman Dam, it is useful to compare the calculated seismic stability of Harriman Dam to the actual performance of another hydraulic fill dam, such as the Lower San Fernando Dam, which has been subjected to a major earthquake. The Lower San Fernando Dam experienced a massive upstream slope failure as a result of an 0.55g earthquake having four major cycles of loading. The dam was composed of hydraulic fill sand shells with a hydraulic fill clay core. The failure was initiated in the lower portions of the upstream shell. The properties of the San Fernando Dam soils, the seismic stability of the dam, and the details of the slope failure have been extensively investigated and analyzed (Seed _et al._, 1973).

Since the San Fernando Dam actually failed, the first step of the comparison was to estimate at what smaller earthquake acceleration the dam would have remained stable. Analysis performed for this study indicated that the San Fernando Dam could have sustained the San Fernando earthquake with a peak ground acceleration of up to 0.35g.[1] Details of the analysis are contained in GEI (1982).

The cyclic resistance of the Harriman Dam soils is about 70 per cent of the resistance of the San Fernando materials, based on a comparison of $C\overline{R}$ test data from the two dams. If the San Fernando Dam had been composed of Harriman Dam dumped shell and hydraulic core materials, it could have sustained the San Fernando earthquake at 0.2g acceleration. This result lends support to the conclusion in Section VIII.c above, that Harriman Dam would adequately resist an 0.2g earthquake.

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[1] The estimated crest settlement of Lower San Fernando Dam due to the 0.35g earthquake was about three feet.

IX. SURVEILLANCE PROGRAM

MAIN/GEI recommend that nineteen piezometers and one observation well be read and recorded on a regular basis during construction of the recommended improvements and through the 1982 spring filling. The following list of 19 Critical Piezometers and one well should be monitored:

B-102B	B-109C	B-114	B-201B
B-103C	B-109D	B-116	B-202B
B-104B	B-110D	B-117	B-202C
B-104C	B-112C	B-118	B-208C
B-108B	B-113D	B-119	W-26

These instruments should be read on the following schedule:

September 1 to December 31, 1981 - once a week

January 1 to May 31, 1982 - twice a week

All data should be plotted and reviewed by competent engineering personnel as soon as available. Following the 1982 spring runoff season, the reading frequency schedule should be reviewed and adjusted.

Table 5 lists the piezometer pore water pressures used to develop the pool elevation 1392.0 estimated phreatic surfaces used in the analyses previously reported. If readings in any one of these piezometers exceed the listed values after construction of the proposed overlay, the project engineers should be immediately advised so that necessary monitoring, analysis or corrective action can be implemented if required.

Prior to May 31, 1982, if a significant storm causes a rapid increase in reservoir levels, reading frequency should return to twice a week until the piezometers stabilize under the new reservoir conditions. In addition, all piezometers at Harriman Dam should be read during the first week of every month to insure proper operation and maintain nitrogen in the piezometer leads. The reservoir elevation should be recorded each time the piezometers are read. To insure that pore water pressures do not build up too high during the construction period, MAIN/GEI recommends that the pool not exceed elevation 1386.0 for any extended period during the construction phase.

During construction of the proposed improvements and associated piezometer lead relocation, extreme care should be taken to prevent any contaminants from entering the leads. Instrument readings should be taken immediately before and after splicing the leads. The leads shall be properly bedded in collection trenches extending to monitoring stations located off the embankment. Extreme care shall be taken to prevent kinking or other damage to the tubing. A number of observation wells will be covered and abandoned by the proposed overlay.[1] MAIN/GEI recommends that wells 21, 25 and 26 be saved and instrumented with piezometers.

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[1] Before covering these wells, the wells should be grouted in place.

LIST OF REFERENCES

Barrows, H. K. (1943), "Water Power Engineering," McGraw Hill Book Company, 1943.

Bishop, A. W. (1967), "The Use of the Slip Circle in the Stability Analysis of Slopes", Geotechnique, Vol. 5, 1955.

Eaton, A. C. (1924 a), "Davis Bridge Power Project to Develop 60,000 HP," Engineering News-Record, Vol. 92, No. 4, January 24, 1924.

Eaton, A. C. (1924 b), "Building the Highest Hydraulic Fill Dam," Engineering News-Record, Vol. 92, No. 6, February 7, 1924.

Geotechnical Engineers, Inc. (1981 a), "Harriman Dam, Summary of Field Investigations and Laboratory Tests for Phases I and II," Project 79646, April 10, 1981.

Geotechnical Engineers, Inc. (1981 b), "Harriman Dam, Summary of Field Investigations and Laboratory Tests for Phase III," Project 81807, August 13, 1981.

Geotechnical Engineers, Inc. (1982), "Seismic Stability Evaluation, Harriman Dam," Project 81858, June 1982.

Kim. T. C. and Novak, M. (1981), "Dynamic Properties of Some Cohesive Soils of Ontario," Canadian Geotechnical Journal, V. 18, pp. 371-389.

C. T. Main, Inc. (1979), "Harriman Dam Stability Analysis," December 1979.

C. T. Main, Inc. (1981), "New England Power Company, Harriman Dam," September 29, 1981.

C. T. Main, Inc. (1982), "New England Power Company, Harriman Dam Improvement Project, Final Report," March 1982.

New England Power Company (1922-1924), "Photographic Records of Davis Bridge Dam Construction," Five Volume Series, 1922-1924. Located at New England Power Offices in Westborough, MA.

New England Power Company (1928), "Davis Bridge Dam (Harriman Dam) Inspection Report," prepared by A. C. Eaton, June 8, 1928.

New England Power Company (1936), "Office Memorandum from H. L. Hurd to C. R. Bliss, RE: Frost Slide on Downstream Face of Harriman Dam," April 2, 1936.

New England Power Service Company, "Inspection Report of the New England Power Company Deerfield River Project, Federal Energy Regulatory Commission Project No. 2323;" 1968, 1973 and 1978 Reports.

New England Power System (1922), "The Davis Bridge Development," Contact (New England Power System Employee's Monthly Magazine), Vol. 3, No. 11, November 1922.

Nuclear Regulatory Commission (1981), "Site Specific Ground Response Spectra for SEP Plants Located in Eastern United States," NRC letter to all SEP plant owners, from Mr. Dennis M. Crutchfield, Chief, Operating Reactors Branch, Division of Licensing, June 8, 1981.

Schnabel, P. B.; Lysmer, J.; and Seed, H. B. (1972), "SHAKE, A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites." Report No. EERC 72-12, College of Engineering, University of California at Berkeley, December 1972.

Seed, H. B. (1976), "Evaluation of Soil Liquefaction Effects on Level Ground During Earthquakes," from Liquefaction Problems in Geotechnical Engineering, Specialty Session at ASCE Annual Convention, Philadelphia, September 27-October 1, 1976.

Seed, H. B. and Idriss, I. M. (1971), "Simplified Procedure for Evaluating Soil Liquefaction Potential," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 97, No. SM9, September 1971.

Seed, H. B.; Lee, K. L.; Idriss, I. M.; and Makdisi, F. (1973), "Analysis of the Slides in the San Fernando Dams during the Earthquake of February 9, 1971," Report No. EERC 73-2, Earthquake Engineering Research Center, University of California, Berkeley, June 1973.

Teng, W. C. (1964), Foundation Design, Prentice Hall, Inc., 1964.

Weston Geophysical Corp. (1979), "Eastern United States Tetonic Structures and Provinces Significant to the Selection of a Safe Shutdown Earthquake," August 1979.

Weston Geophysical Corp. (1980), "Site Dependent Response Spectra, Yankee Rowe," February 1981.

Whitman, R. V. and Bailey, A. M. (1967), "Use of Computers for Slope Stability Analysis," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM4, April 1967.

Yankee Atomic Electric Co. (1981), "Seismic Response Spectra for the Yankee Rowe Nuclear Power Station, Rowe, Massachusetts," June 1981.

PHOTOGRAPHS

P1 — HARRIMAN DAM EXISTING CONDITIONS

PHOTO 1

Crest of dam. View from left (East) abutment, Fall 1979.

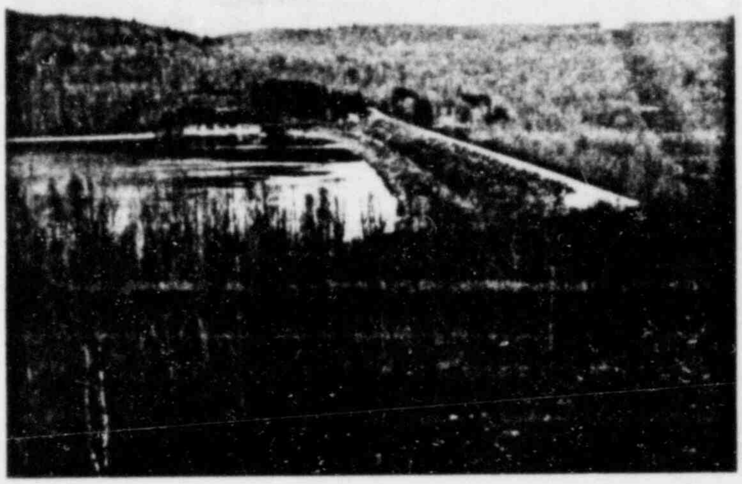


PHOTO 2

Upstream slope of dam and Morning Glory Spillway. View from right abutment, Fall 1979.

PHOTO 3

Downstream slope of dam. View from downstream toe of dam, Fall 1979.

P2 — HARRIMAN DAM CONSTRUCTION (COURTESY OF NEPCO)

PHOTO 4

Initial phase of dam construction. View looking upstream from left abutment, July 1922.

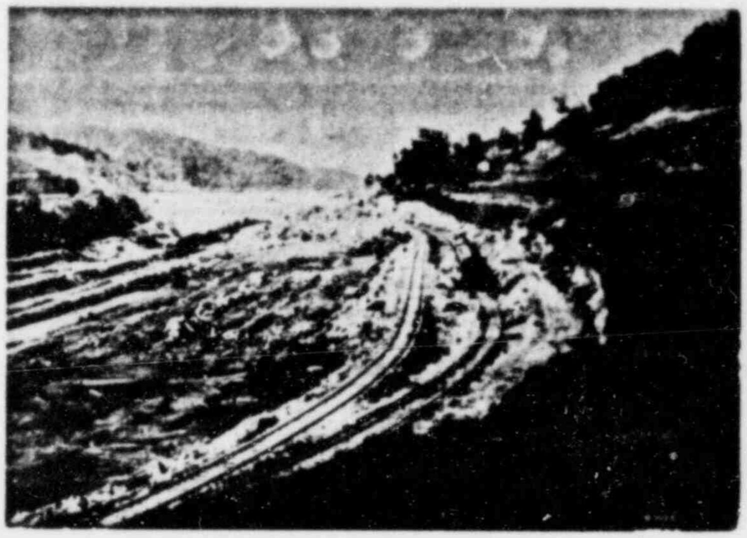


PHOTO 5

Upstream borrow area. View downstream towards dam, July 1923.

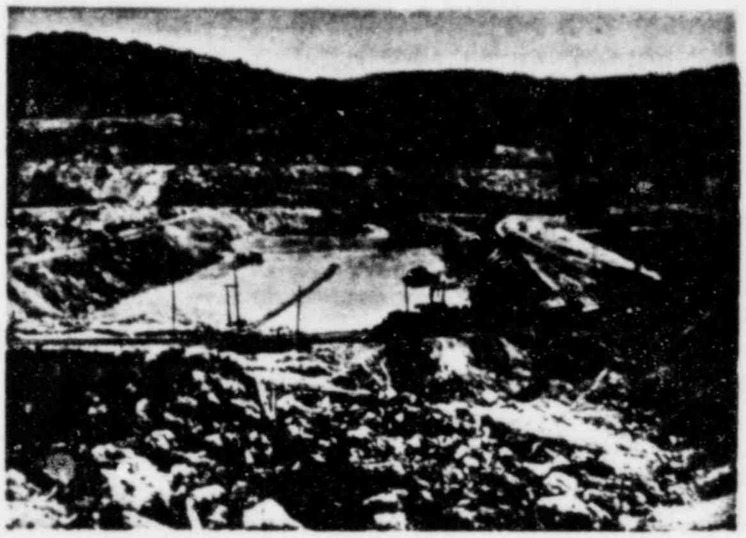


PHOTO 6

Hydraulic placement of core. View from left abutment, July 1923.

TABLES

TABLE 1

CHRONOLOGY OF DAM CONSTRUCTION EVENTS

DATE	EVENT
2 - 1 - 1922	Began diversion tunnel construction
6 - 1922	Timber crib wall for partial river diversion started
6 - 19 - 1922	First fill placement (at upstream toe dike)
9 - 15 - 1922	Diversion tunnel completed
9 - 17 - 1922	River fully diverted 200,000 CY fill placed to date 33,000 CY/Week average fill placement following river diversion
9 - 1922	Began cleanup of central third of dam with boulder removal
10 - 1 - 1922	Began excavation of cutoff trench
11 - 15 - 1922	500,000 CY fill placed to date
11 - 20 - 1922	Cleanup of central third of dam with cutoff trench
11 - 1922	First pumping of water into core area
12 - 1922	Began clearing reservoir area
1 - 1923	Shutdown by cold weather
4 - 15 - 1923	Began placing fill again
4 - 1923	Began placement of core by sluicing dumped fill
6,7 & 8 - 1923	Placed 750,000 CY of fill
11 - 1923	Completed placement of core by sluicing dumped fill
12 - 1923	Dam topped out at elevation 1400.0 Ft.
12 - 1923	Completed reservoir clearing.

TABLE 1 (Cont.)

CHRONOLOGY OF DAM CONSTRUCTION EVENTS

DATE	EVENT
3 - 1924	Diversion tunnel closed
5 - 1924	First power
1930's	Installation of observation wells on downstream slope of dam
3 - 1936	Minor surface slide on downstream face of dam due to severe frost action, warm weather & rain. Approximately at centroid of face area.
1939	Crest of dam raised 6.0 Ft. to elevation 1406.0 Ft.
1958	2 borings at maximum section (Station 10+00)
1964	Crest of dam raised 9.5 Ft. to elevation 1415.5 Ft.
Fall 1979	Installed French Drain near left abutment and placed filter blanket in right abutment swamp area

TABLE 2

DESCRIPTIONS OF TYPICAL SOILS IN HARRIMAN DAM

DUMPED SHELL

GRAVELLY SILTY SAND (SM). Predominantly medium and fine sand with occasional coarse sand. Contains about 10-20% coarse to fine gravel. Generally contains about 30-45% slightly plastic fines. Color varies with increasing percent fines from brown to olive-brown. This was the predominant material encountered in the dumped shell by Phase I, II and III borings.

GRAVELLY SAND AND SANDY GRAVEL (SW, GW). Contains about 30-50% coarse to fine gravel. Generally widely graded sand with less than 15% nonplastic fines. Brown.

This material was encountered in the dumped shell below El. 1280 in borings 110, 202 and 212 and was associated with an increased frequency of cobbles and boulders. Uncased borings in this material experienced significant loss of drill fluid.

WASHED ZONE OF CORE

SANDY SILT AND SILTY SAND (ML, SM). Contains about 40-60% slightly plastic fines. Sand is predominantly medium to fine grained with trace or no coarse sand. Generally contains less than 10% coarse to fine gravel. Olive.

This material contains heterogeneous pockets of gravelly silty sand in a matrix of silt and fine sand and becomes finer grained with less gravel and silty sand near the hydraulic core.

In-situ permeability of this material measured in boring B-9 ranged from 10^{-5} to 10^{-6} cm/sec.

HYDRAULIC CORE

SILT AND SILTY FINE SAND (ML, SP). Predominantly fine silty sand or fine sandy silt with nonplastic fines. Contains no gravel or coarse sand and occasional medium sand. Soil is stratified with layers of narrowly graded fine sand generally 2 to 20 mm thick. Olive.

TABLE 3

DESCRIPTIONS OF FOUNDATION SOILS BENEATH HARRIMAN DAM

ABLATION TILL

SILTY GRAVELLY SAND (SW-SM). Widely graded sand. Contains about 25-40% severely weathered coarse to fine gravel. About 15% nonplastic fines. Olive-brown.

This material extends from ground surface to a depth of 5 to 15 feet on a terrace (ground surface elevation 1300 ft) located on the right abutment of the dam. This terrace extends from about 500 ft upstream from the dam to about 500 ft downstream from the dam and occurs beneath the dam between Stations 4+00 and 7+50.

Ablation till was encountered beneath the dam in borings 104, 105, 107 and 201, but was not encountered in borings 103 and 106, indicating that the ablation till has been removed beneath the core of the dam prior to its construction.

Uncased borings in the ablation till experienced significant loss of drill fluid. An in-situ permeability value of about 1×10^{-3} cm/sec was obtained in boring 11.

LODGEMENT TILL

GRAVELLY SANDY SILT (SM-ML). Predominantly consists of slightly plastic fines. Generally contains about 30-40% fine sand and medium sand. Contains about 10-20% fine gravel. Olive.

This material extends beneath the dam on the right side of the Deerfield River Channel (about Station 10+00). The permeability of this material measured in laboratory tests is on the order of 10^{-8} cm/sec. This material directly underlies the ablation till on the right abutment of the dam.

FOUNDATION SOIL, LEFT DOWNSTREAM ABUTMENT

ORGANIC SILTY SAND (SM). Contains about 30-40% slightly plastic fines with numerous root fibers and organic silt. Consists predominantly of fine and medium sand with occasional coarse sand. Contains about 5-15% coarse to fine gravel. Olive to dark brown.

This material was encountered beneath the dam in borings 109, 112, 202, 210, 213, 214 and 215, but was not encountered in boring 113, indicating that this material has been removed beneath the core of the dam prior to its construction. On the basis of information obtained from the borings, this material forms a layer 2 to 7 ft thick in a limited area beneath the downstream dumped shell from about Station 11+50 to the left abutment and Station 52+00 to at least Station 53+00. It is likely that this material represents topsoil overlying shallow bedrock on the left side of the Deerfield River Valley that was not removed prior to construction of the dam.

BEDROCK

GNEISS. Foliation about 10-30 degrees from horizontal. Generally hard and intact except for moderate weathering along joints from 0 to 5 ft below bedrock surface. Joints spaced from 4 to 20 in. apart.

TABLE 4

MATERIAL PROPERTIES

MATERIAL	γ_T (pcf)	Ø'	C'(psf)
Proposed Overlay Fill	140	36°	0
1964 Rolled Shell	145	37°	0
Dumped Shell	135	35°	0
Core	130	35°	0
Glacial Till	135	35°	0
Gneiss	170	63°	2000

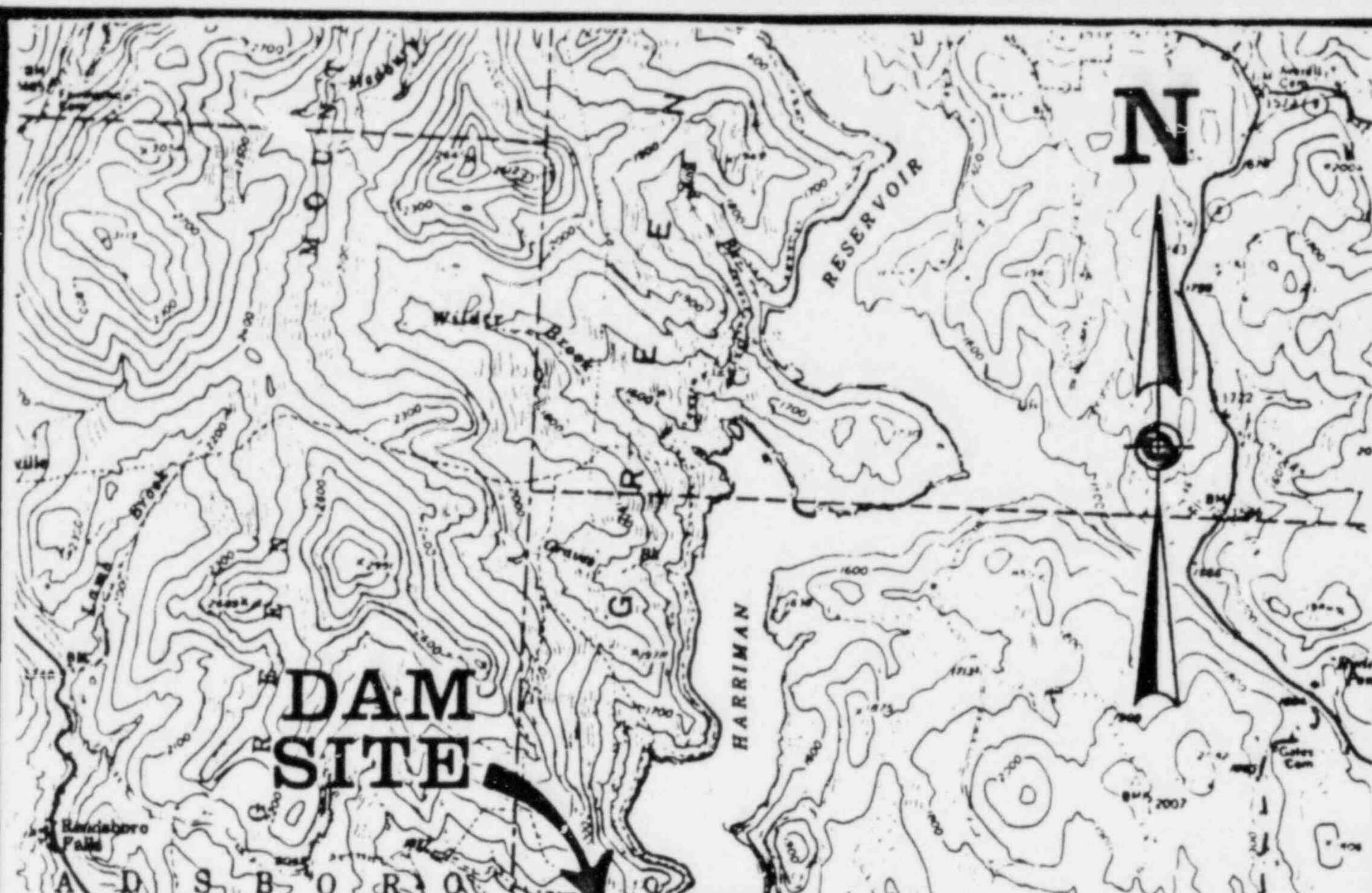
Note: γ_T = Field wet bulk density

TABLE 5

CRITICAL PIEZOMETER PORE WATER PRESSURES

Piezometer Tip	Critical Elevation Head
B-103C	1380.1
B-104B	1337.3
B-104C	1352.0
B-109C	1342.0
B-109D	1363.0
B-110D	1335.0
B-112C	1316.0
B-113D	1383.0
B-114	1375.0
B-116	1345.0
B-118	1360.0
W-26	1374.0

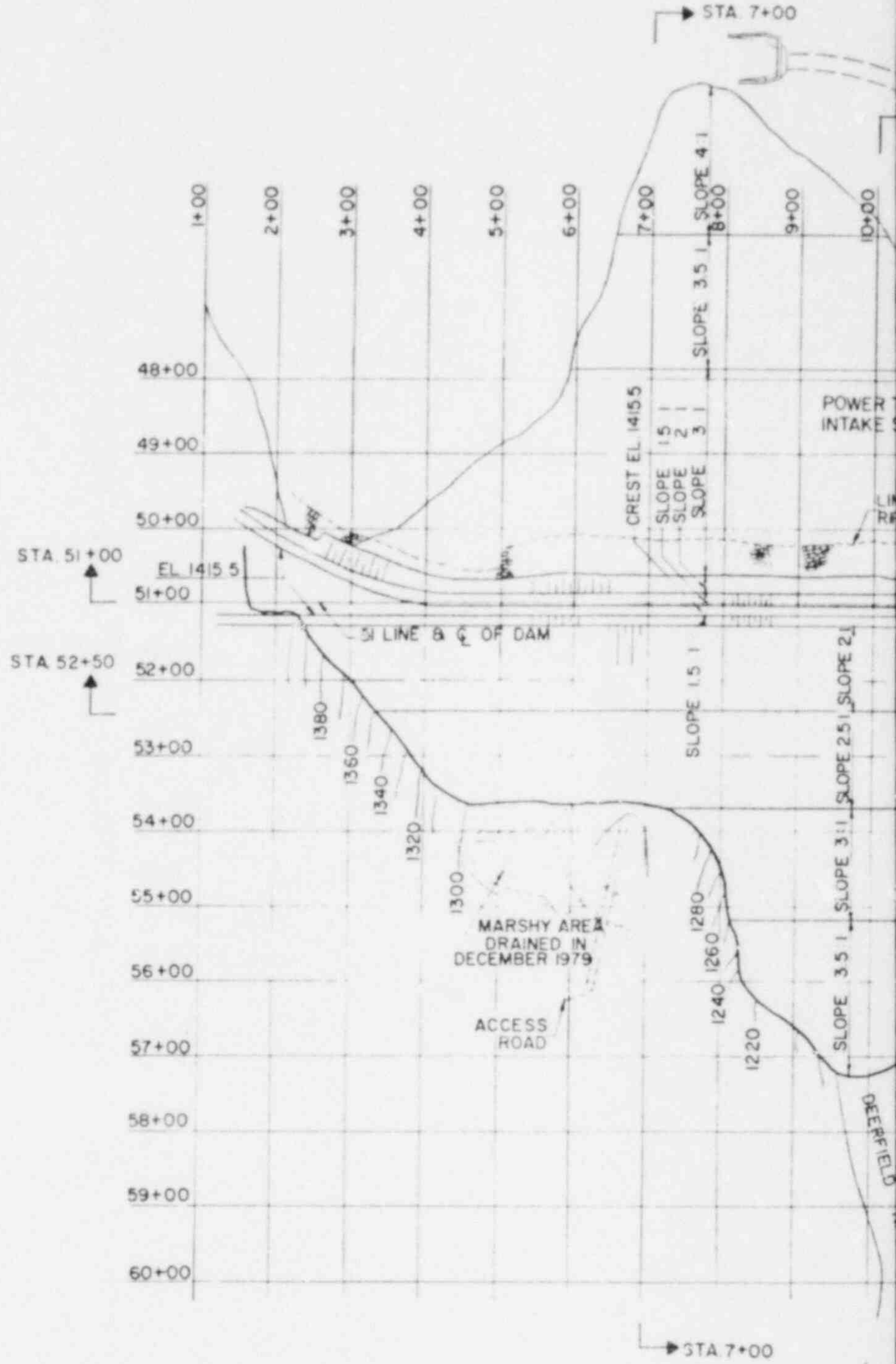
PLATES

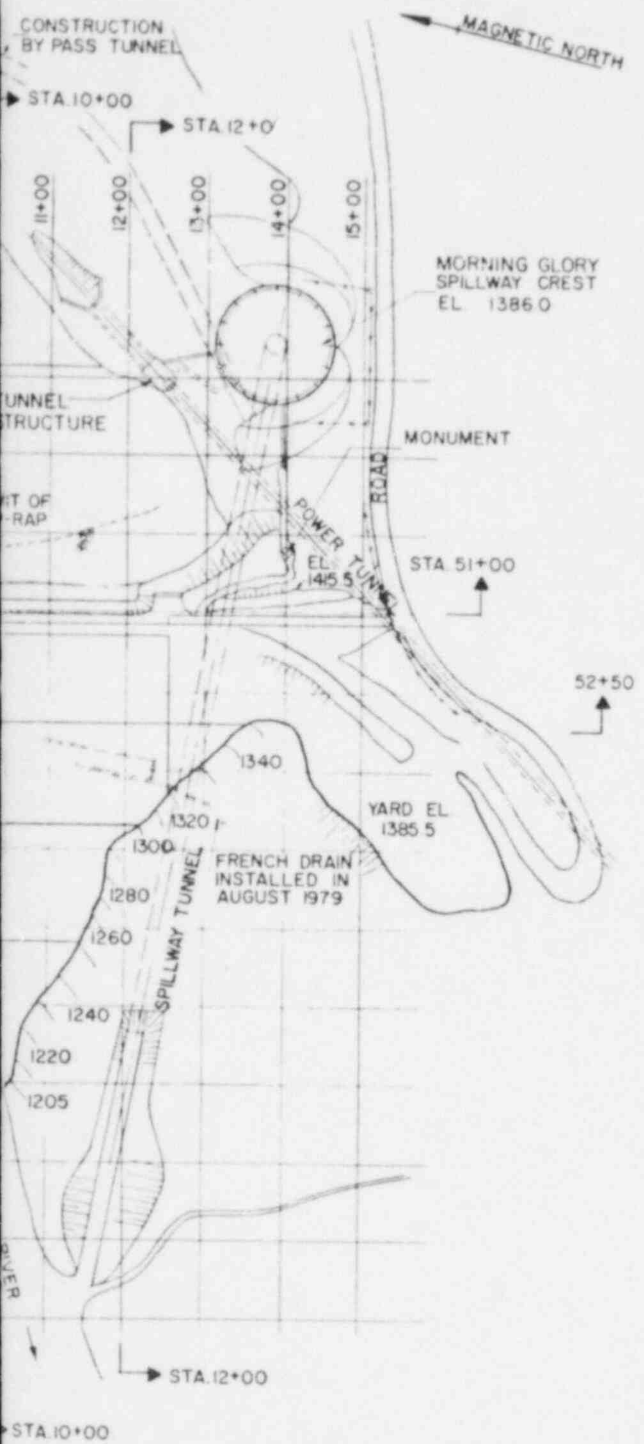


LOCATION MAP

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN	DATE MAY, 1982
GEOTECHNICAL ENGINEERS INC	PLATE 1270-76 1


STA. 7+00
1+00
2+00
3+00
4+00
5+00
6+00
7+00
SLOPE 4:1
8+00
9+00
10+00
SLOPE 3.5:1
48+00
49+00
50+00
51+00
52+00
53+00
54+00
55+00
56+00
57+00
58+00
59+00
60+00
CREST EL. 1415.5
SLOPE 1.5:1
SLOPE 2:1
SLOPE 3:1
POWER T
INTAKE S
STA. 51+00
EL. 1415.5
STA. 52+50
51 LINE & ℄ OF DAM
SLOPE 1.5:1
SLOPE 2:1
SLOPE 2.5:1
SLOPE 3:1
SLOPE 3.5:1
1380
1360
1340
1320
1300
1280
1260
1240
1220
MARSHY AREA
DRAINED IN
DECEMBER 1979
ACCESS
ROAD
DEERFIELD
STA. 7+00
PLAN



NOTE

ALL ELEVATIONS SHOWN ARE NEPCO DATUM
(NEPCO ELEV + 105.66 = NGVD ELEV.)

100 0 100 200

Scale 1" = 100'

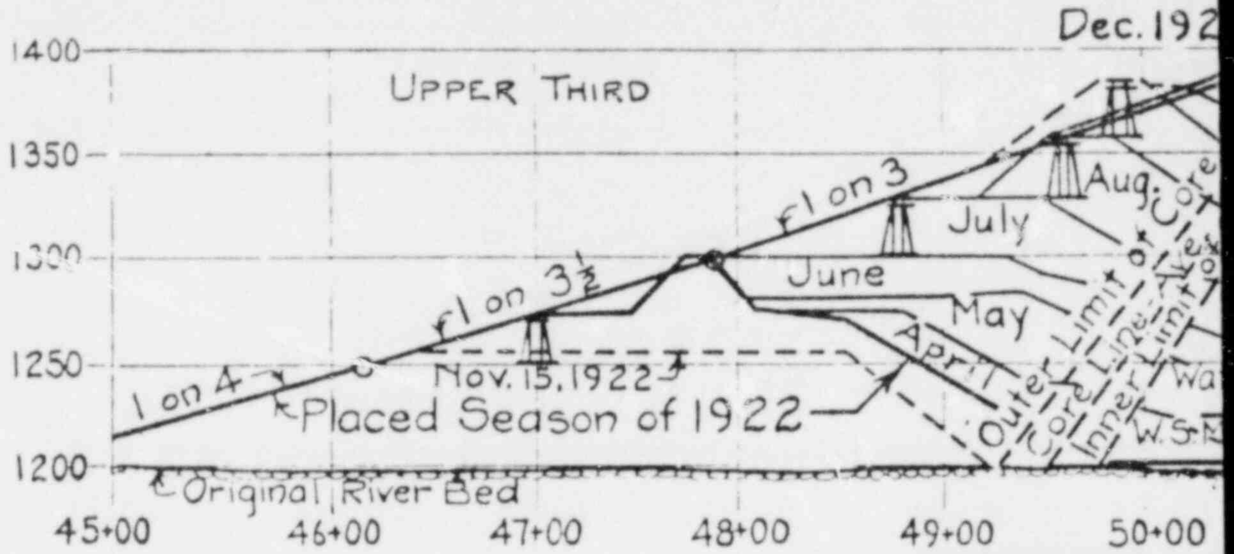
SITE PLAN

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

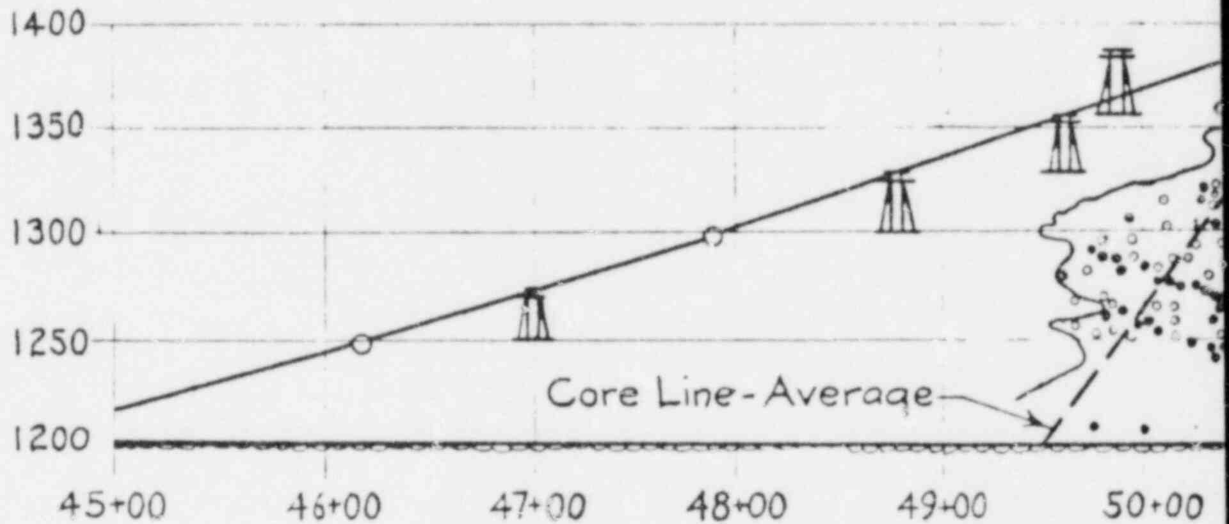
MAIN

GEOTECHNICAL ENGINEERS INC

DATE	MAY, 1982
PLATE	1270-76 2



PROGRES



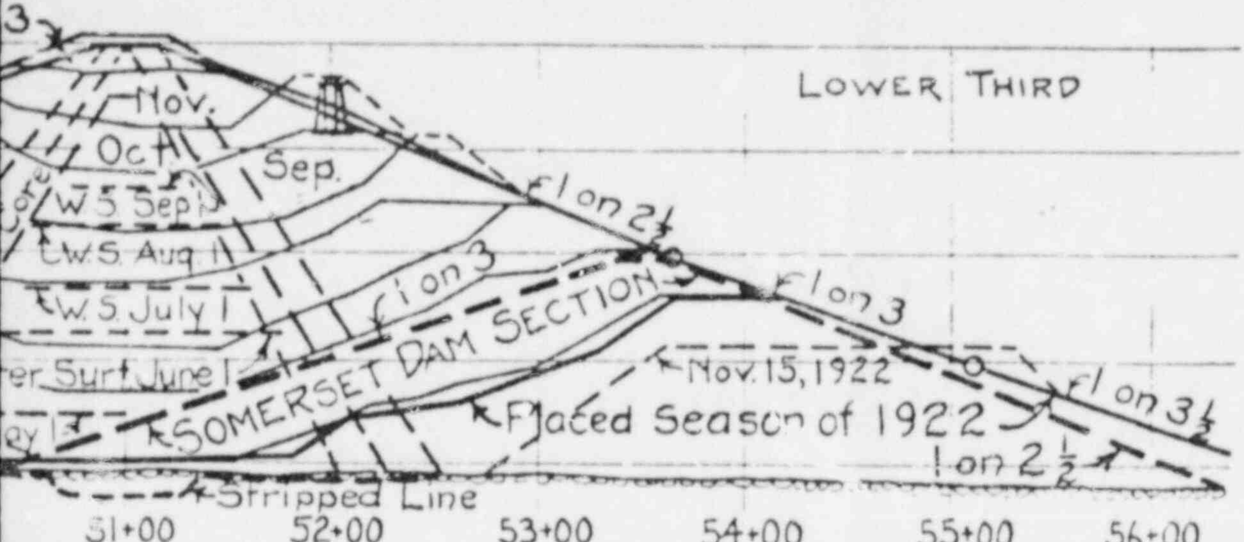
BEACH LINE

NOTE:-

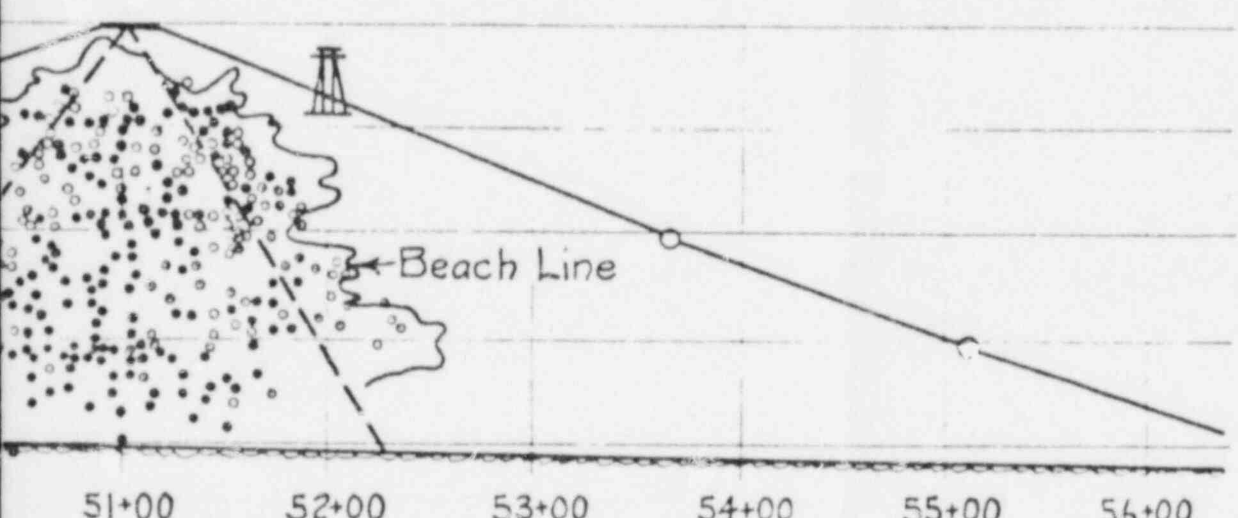
Black Dots represent samples of which 50 % o passed 100 Mesh Sieve.

Circles represent samples of which less tha passed 100 Mesh Sieve.

This Drawing Reproduced From NEPCO DWG. File No. E1812



CROSS-SECTION

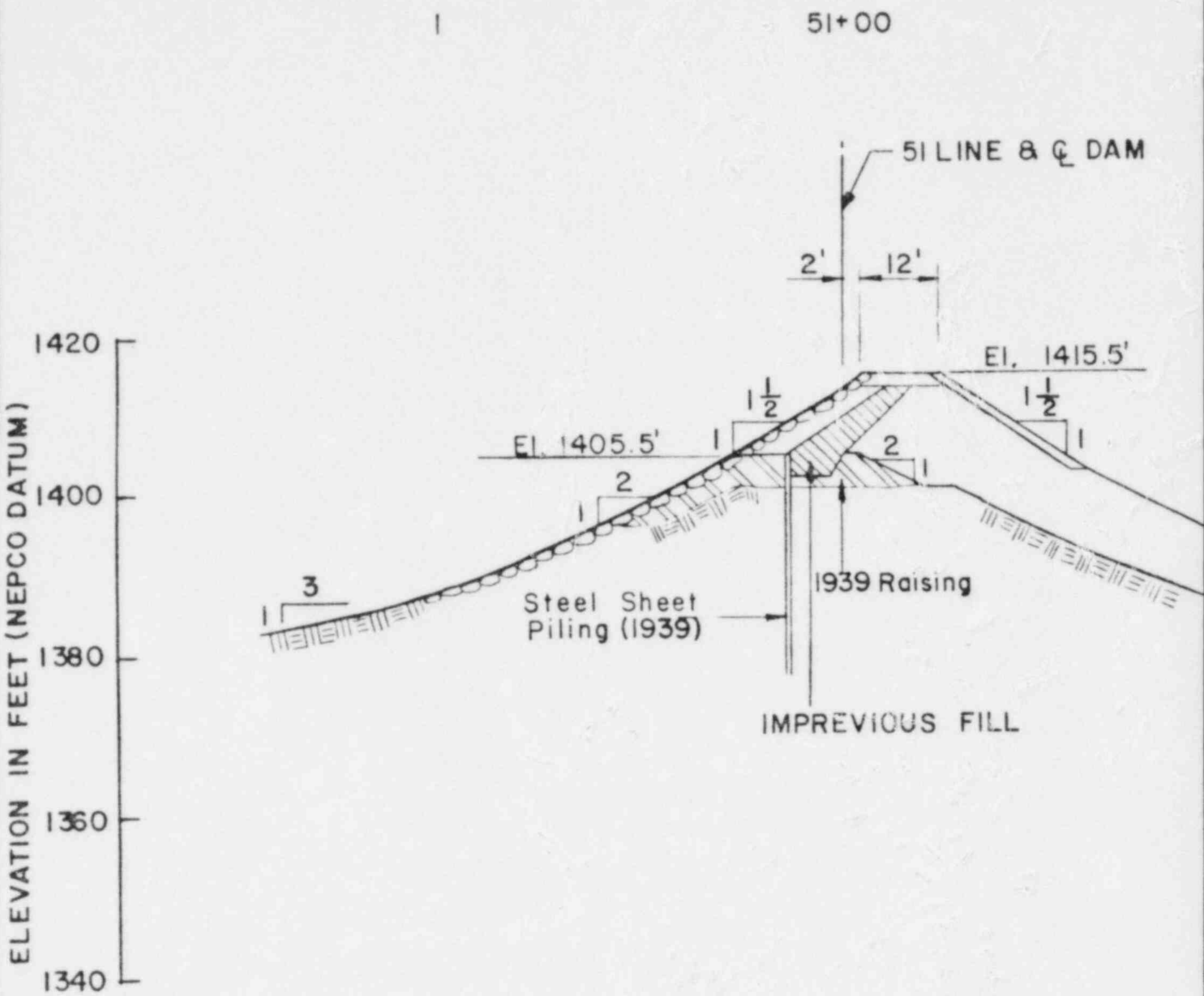


AND SAMPLE LOCATION

r more

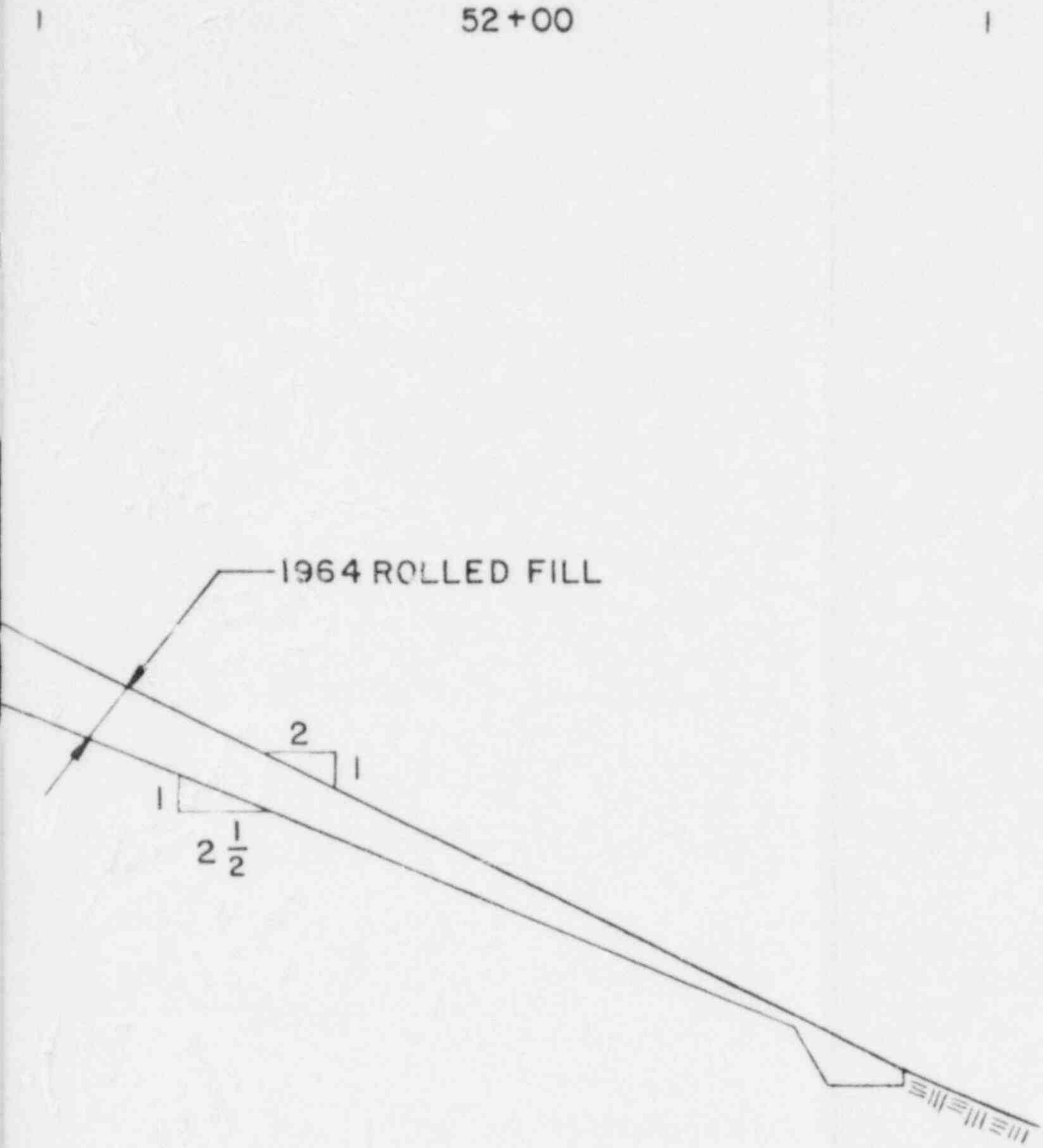
n 50 %

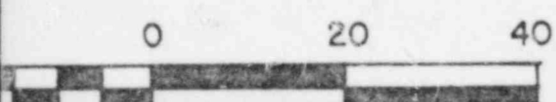
CONSTRUCTION SECTIONS AT STATION 10+00	
NEW ENGLAND POWER COMPANY HARRIMAN DAM	
MAIN GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982
	PLATE 1270-76 3



SECTION AT STA. 8+00

20





SCALE: 1" = 20'

1939 RAISING AND
1964 OVERLAY AND RAISING

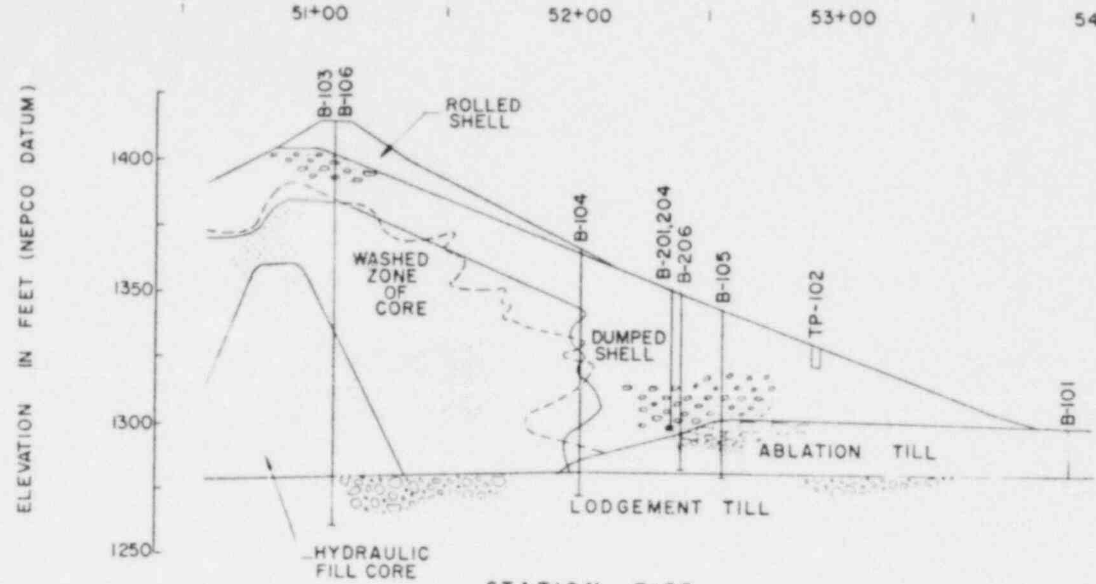
NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN

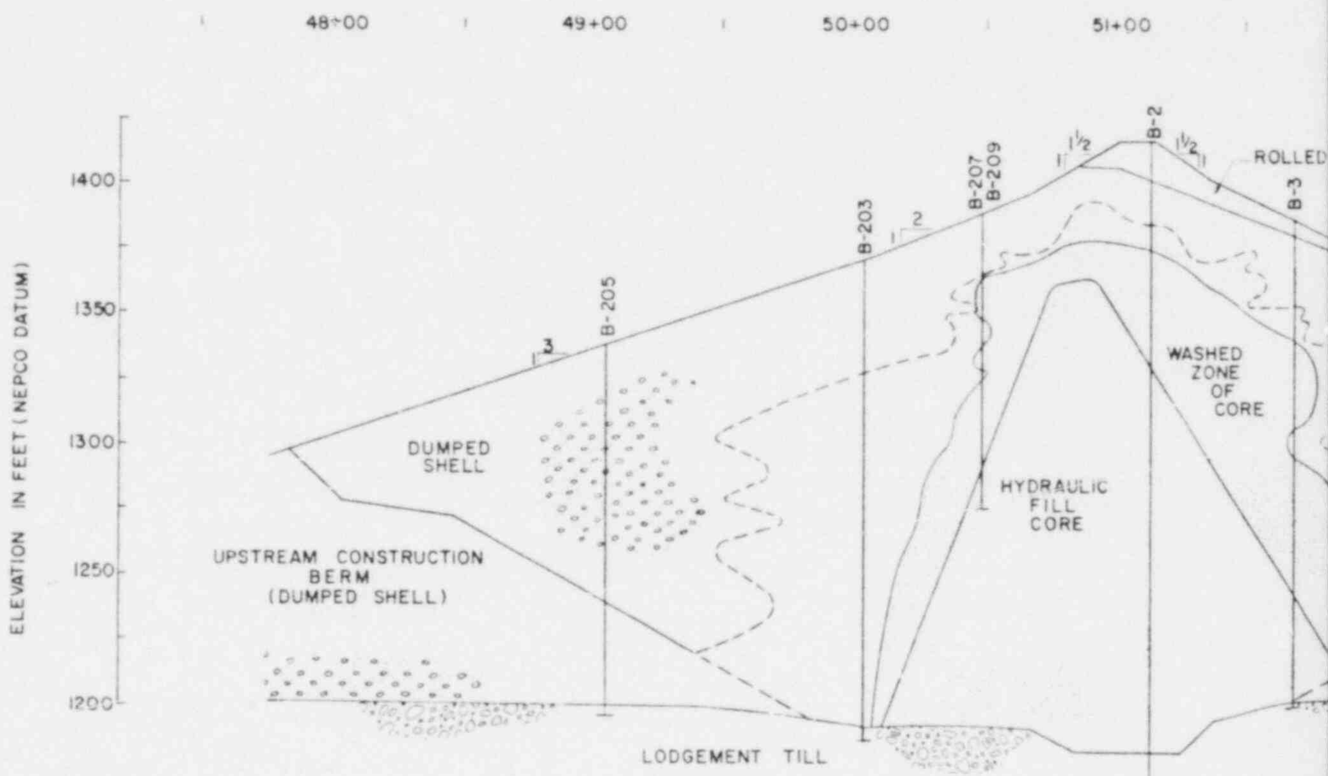
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

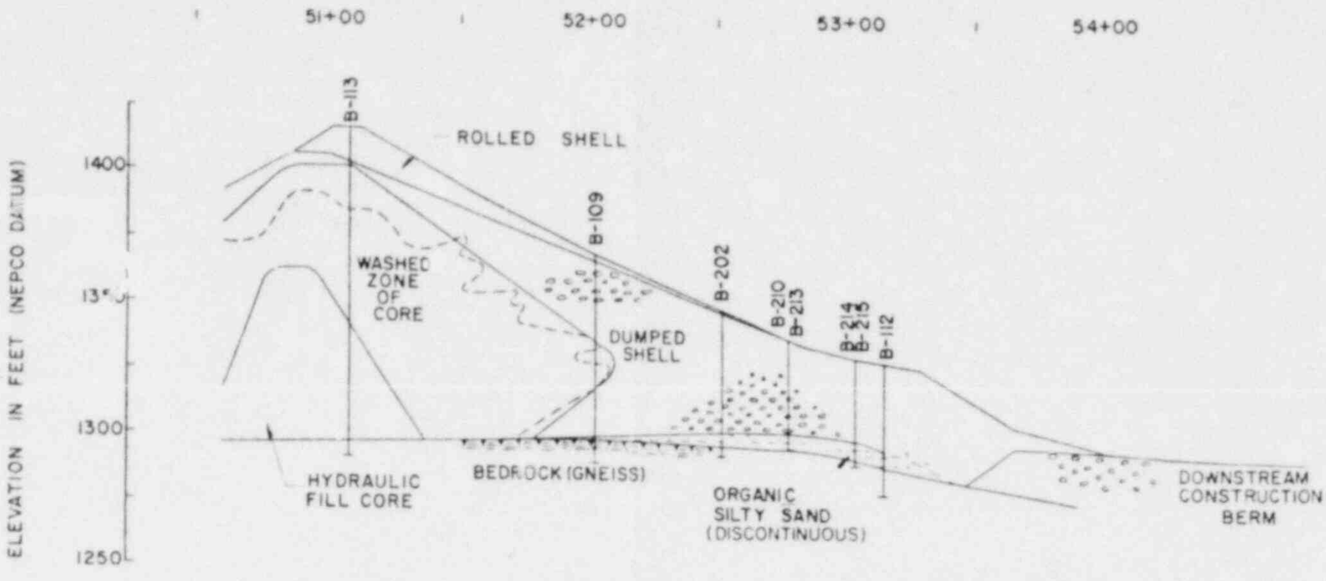
PLATE
1270-76 4



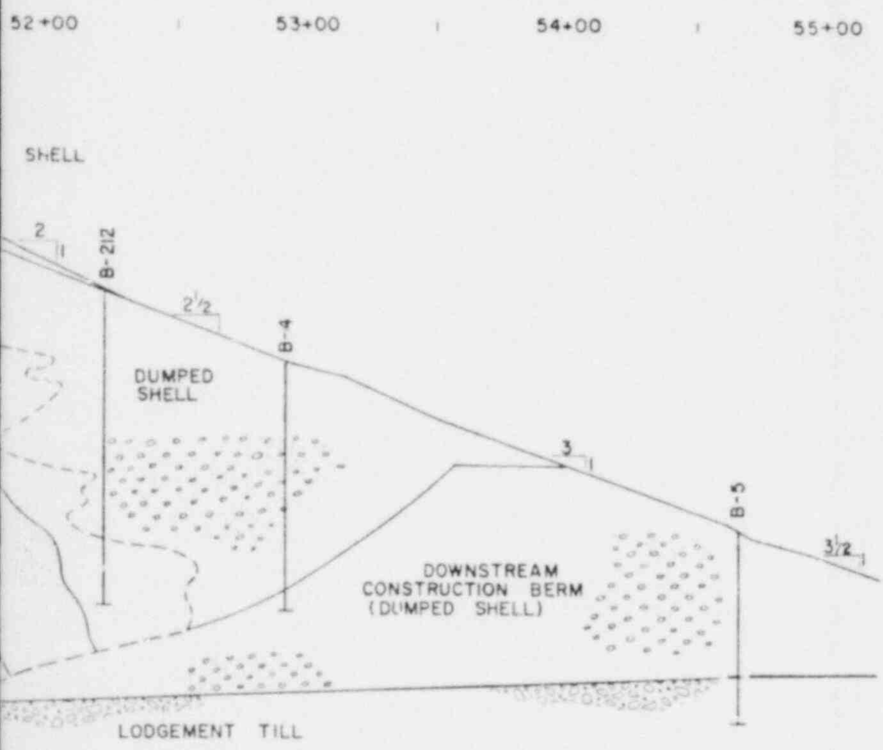
STATION 7+00
INTERPRETIVE CROSS SECTION



STATION 10+00
INTERPRETIVE CROSS SECTION



STATION 12+00
INTERPRETIVE CROSS SECTION



LEGEND

— — Beach Line From Sta 10+00 See Plate 1270-76-3

—— Internal Zones Estimated From Borings

NOTES:

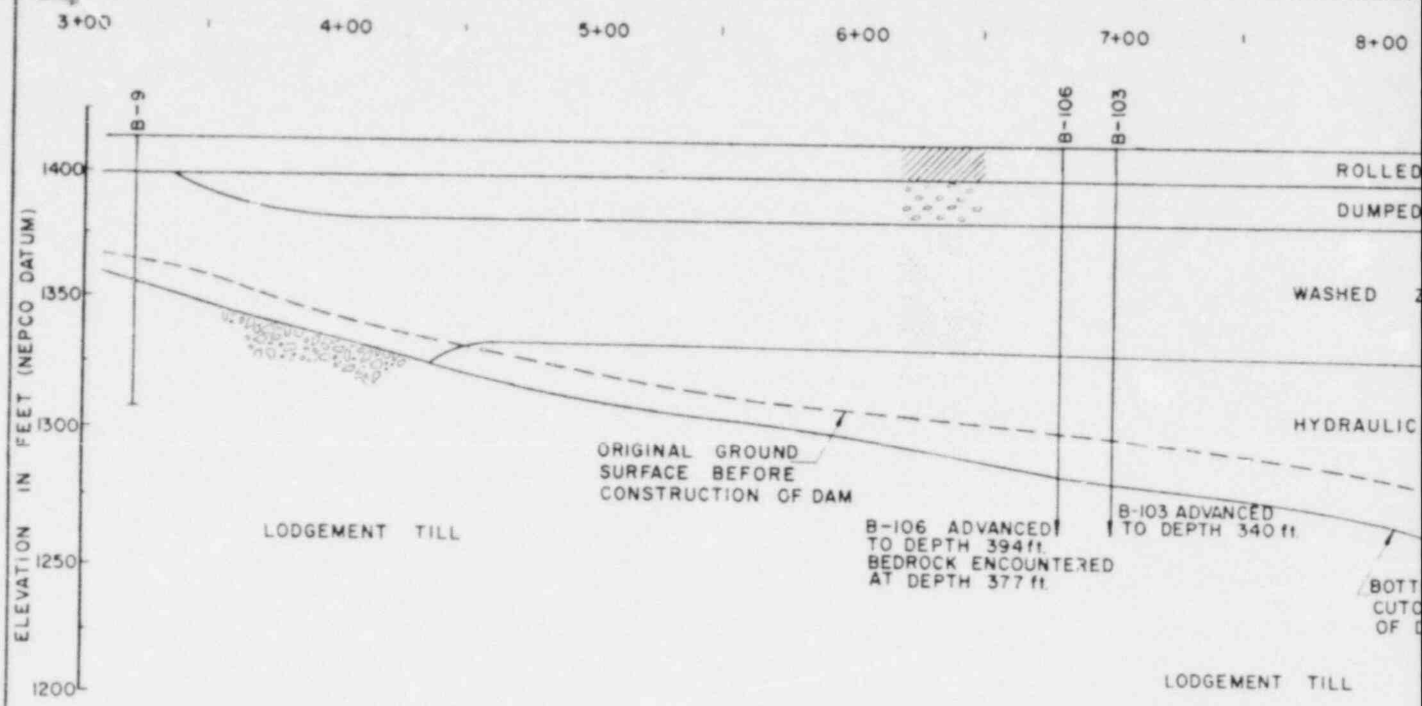
1) All Stations And Elevations Are In Feet (NEPCO Datum And Coordinates)

2) For Boring And Cross Section Locations, See PLATE 2.

30 0 30 60

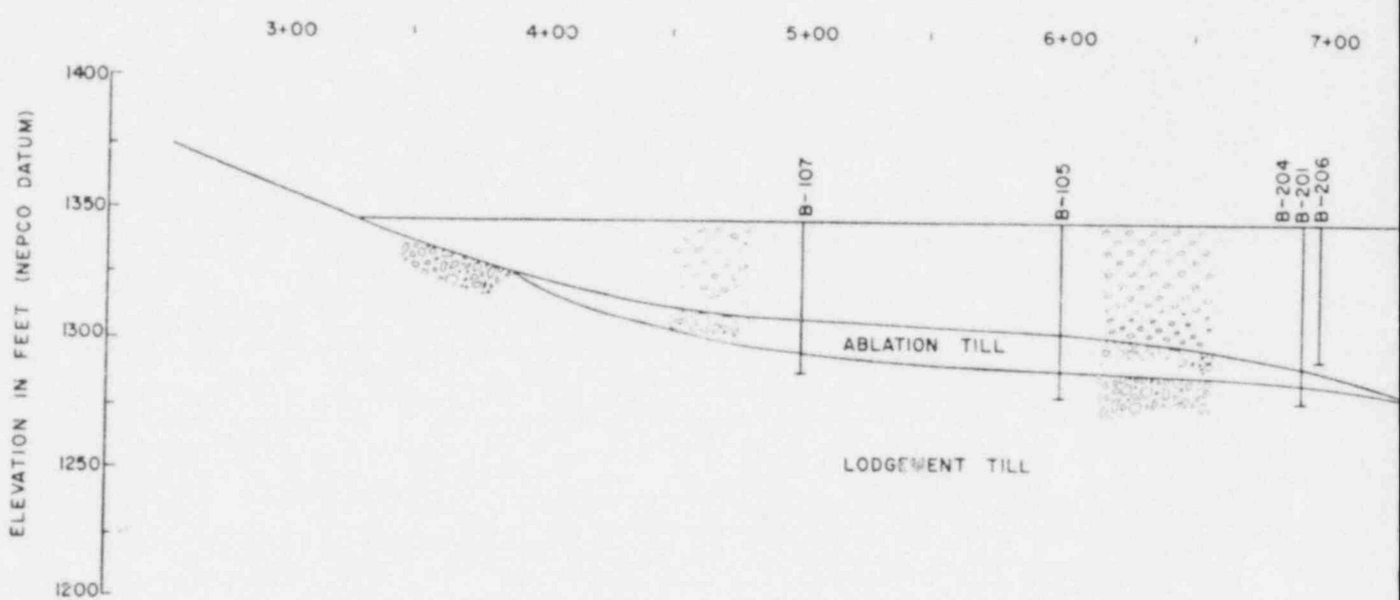
Scale 1" = 30'

INTERPRETIVE CROSS SECTIONS AT STATIONS 7+00, 10+00 AND 12+00	
NEW ENGLAND POWER COMPANY HARRIMAN DAM	
MAIN GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982 PLATE 1270-76 5



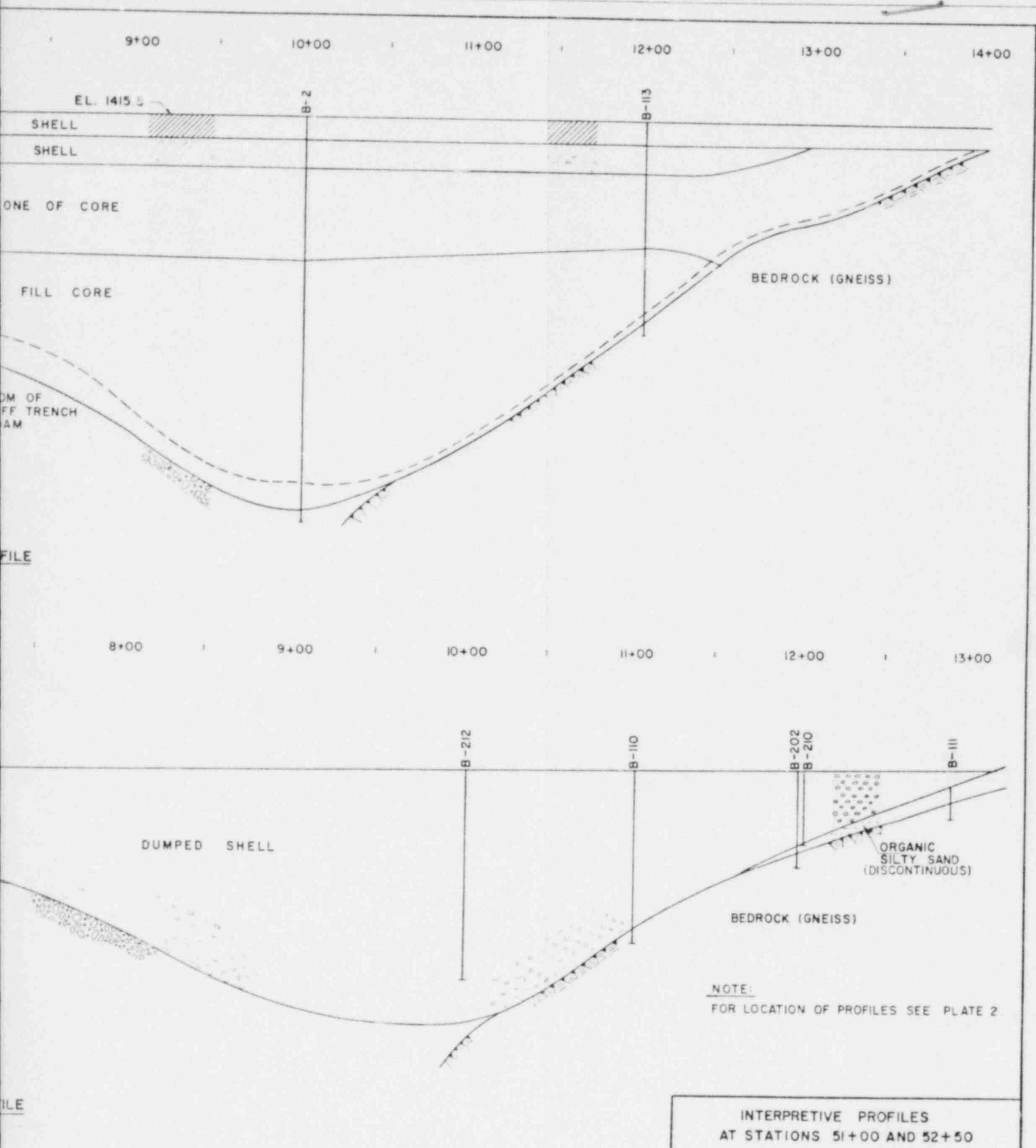
INTERPRETIVE PRO

STATION 51+00



INTERPRETIVE PROF

STATION 52+50



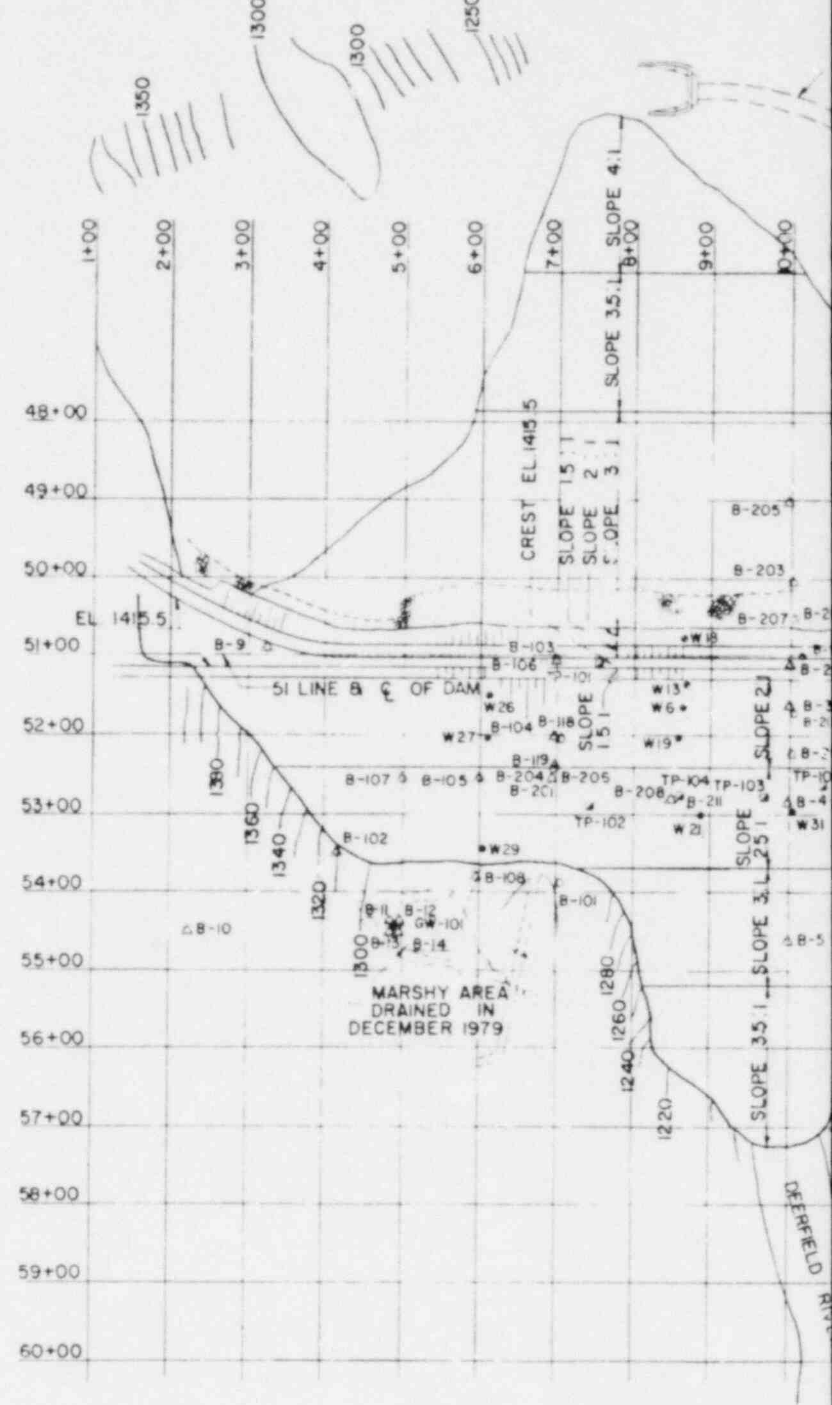
INTERPRETIVE PROFILES
AT STATIONS 51+00 AND 52+50

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC

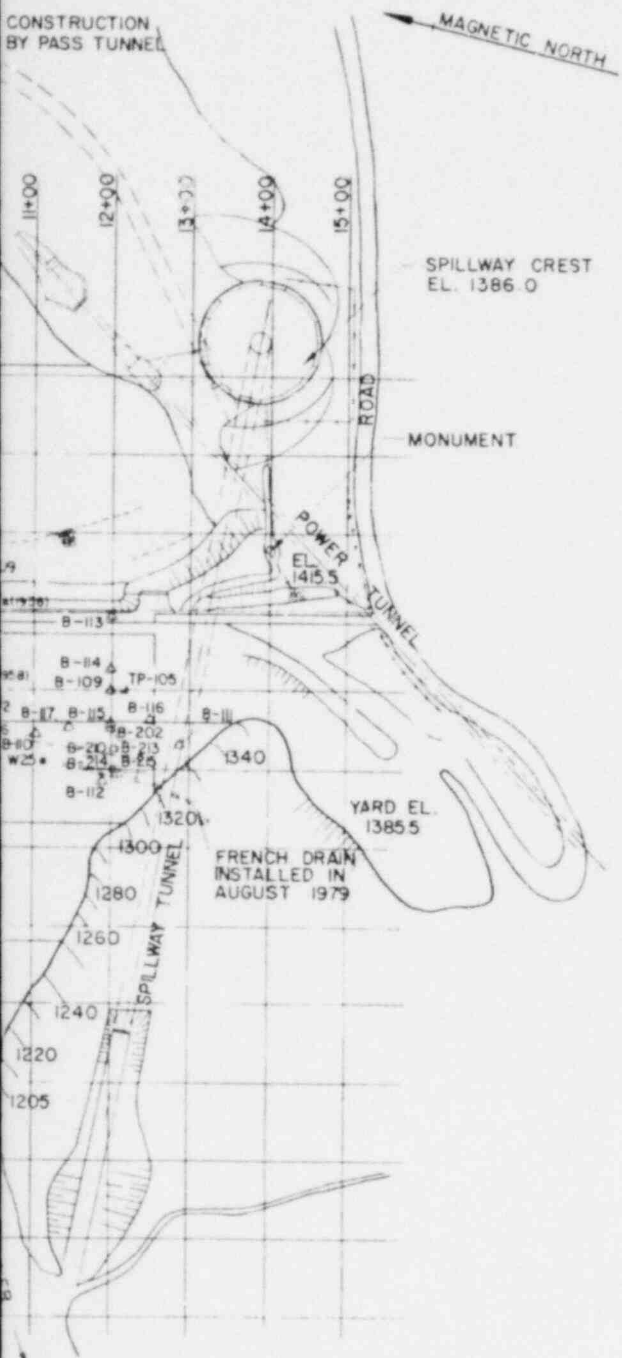
DATE MAY, 1982

PLATE 1270-76 6



1350
1300
1300
1250
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
10+00
SLOPE 4:1
SLOPE 3.5:1
CREST EL. 1415.5
SLOPE 1.5:1
SLOPE 2:1
SLOPE 3:1
48+00
49+00
50+00
51+00
52+00
53+00
54+00
55+00
56+00
57+00
58+00
59+00
60+00
EL. 1415.5
51 LINE & ℄ OF DAM
MARSHY AREA
DRAINED IN
DECEMBER 1979
SLOPE 2:1
SLOPE 2.5:1
SLOPE 3:1
SLOPE 3.5:1
DEERFIELD RIV

PLAN



PIEZOMETER DATA	
BORING No	No of TIPS
B 2	2
B 3	2
B 4	2
B 5	1
B 9	3
B 10	3
B 11	1
B 12	1
B 13	1
B 14	1
B 101	1
B 102	2
B 103	3
B 104	3
B 105	2
B 107	2
B 108	2
B 109	4
B 110	4
B 111	2
B 112	3
B 113	4
B 114	1
B 115	1
B 116	1
B 117	1
B 118	1
B 119	1
B 201	3
B 202	3
B 203	4
B 205	4
B 208	3
B 212	3

LEGEND

- • WELL POINTS INSTALLED AFTER CONSTRUCTION OF DAM, STILL IN SERVICE AS OF 1979-1981 (OTHER WELL POINTS HAVE BEEN REMOVED FROM SERVICE BETWEEN 1937, 1964 CREST RAISE)
- ○ NON-INSTRUMENTED BORINGS (1958-1981)
- △ BORING WITH PNEUMATIC PIEZOMETERS (1979-1981)
- ▪ TEST PITS (1980-1981)

100 0 100 200

Scale 1" = 100'

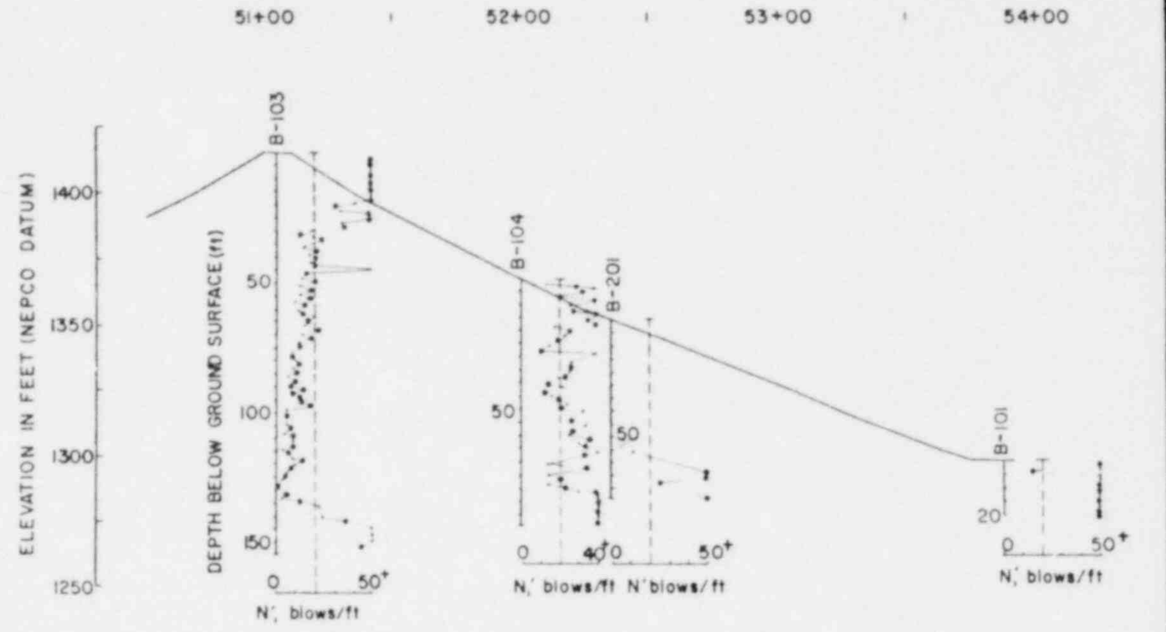
BORING, PIEZOMETER AND OBSERVATION WELL LOCATION PLAN

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

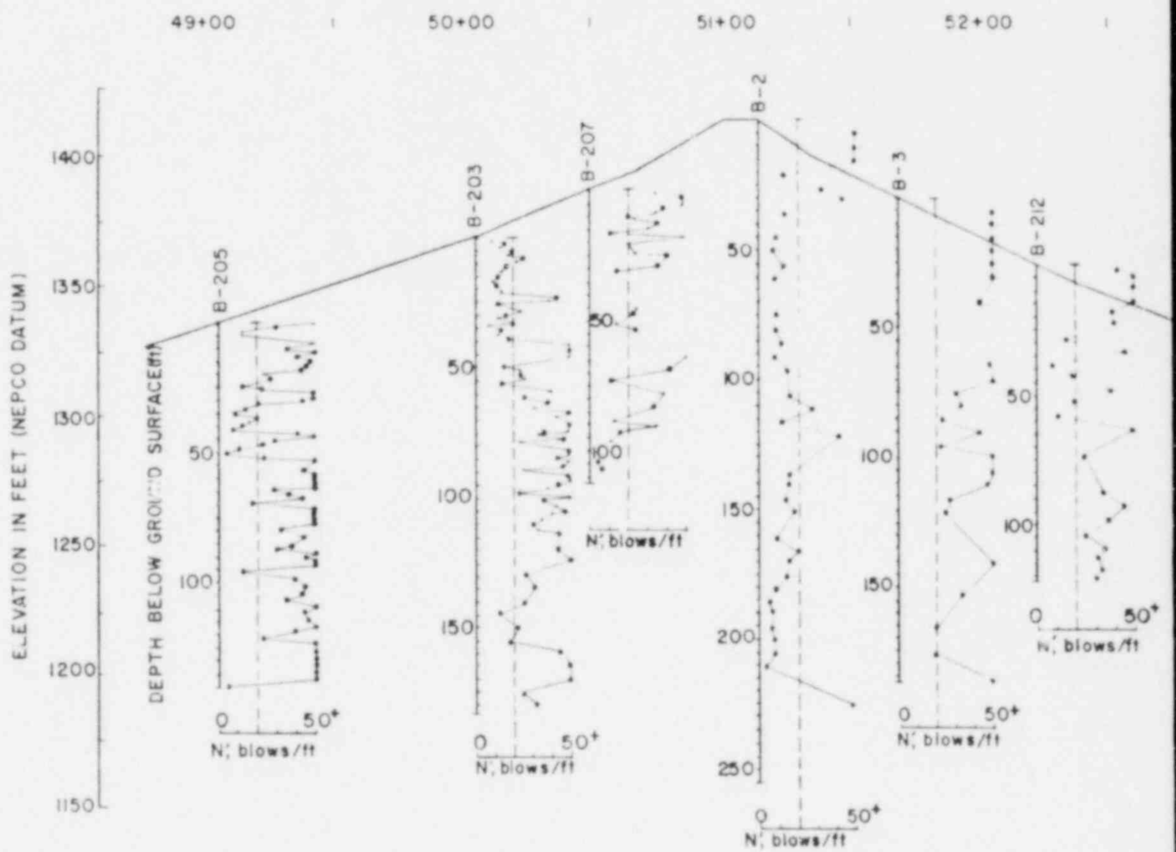
MAIN
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

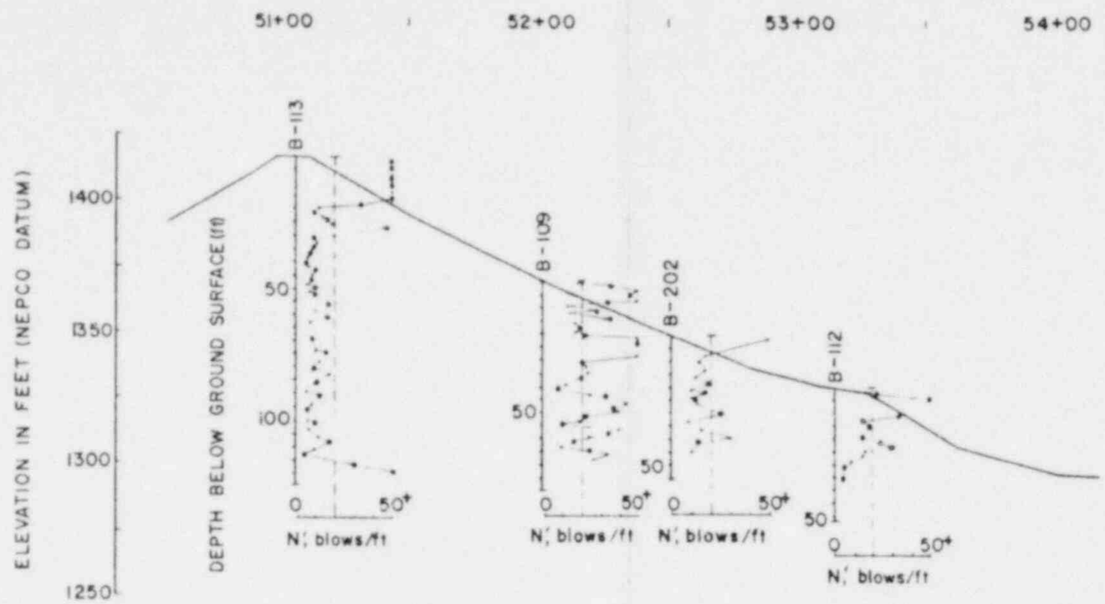
PLATE 1270-76 7



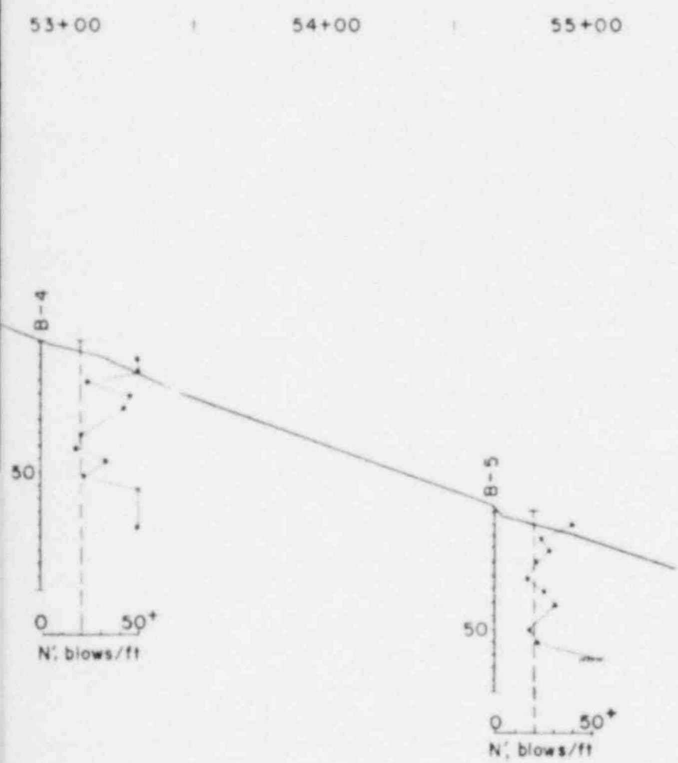
STATION 7+00
CORRECTED BLOWCOUNTS VS DEPTH



STATION 10+00
CORRECTED BLOWCOUNTS VS DEPTH



STATION 12+00
CORRECTED BLOWCOUNTS VS. DEPTH



LEGEND

• - 2" OD Split-spoon Sampler

· - 3½" OD Split-spoon Sampler

N' = Blowcount Corrected To 40 Psi Vertical Effective Stress $= \frac{50\ N}{(\bar{\sigma}_v + 10)}$, $\bar{\sigma}_v$ In Psi, N = Measured Blowcount (blows/ft)

NOTES

1. All Stations And Elevations Are In Feet (NEPCO Datum)
2. B103 Was Advanced To 340.1 Ft., No Additional Blowcounts Taken

30 0 30 60

Scale 1" = 30'

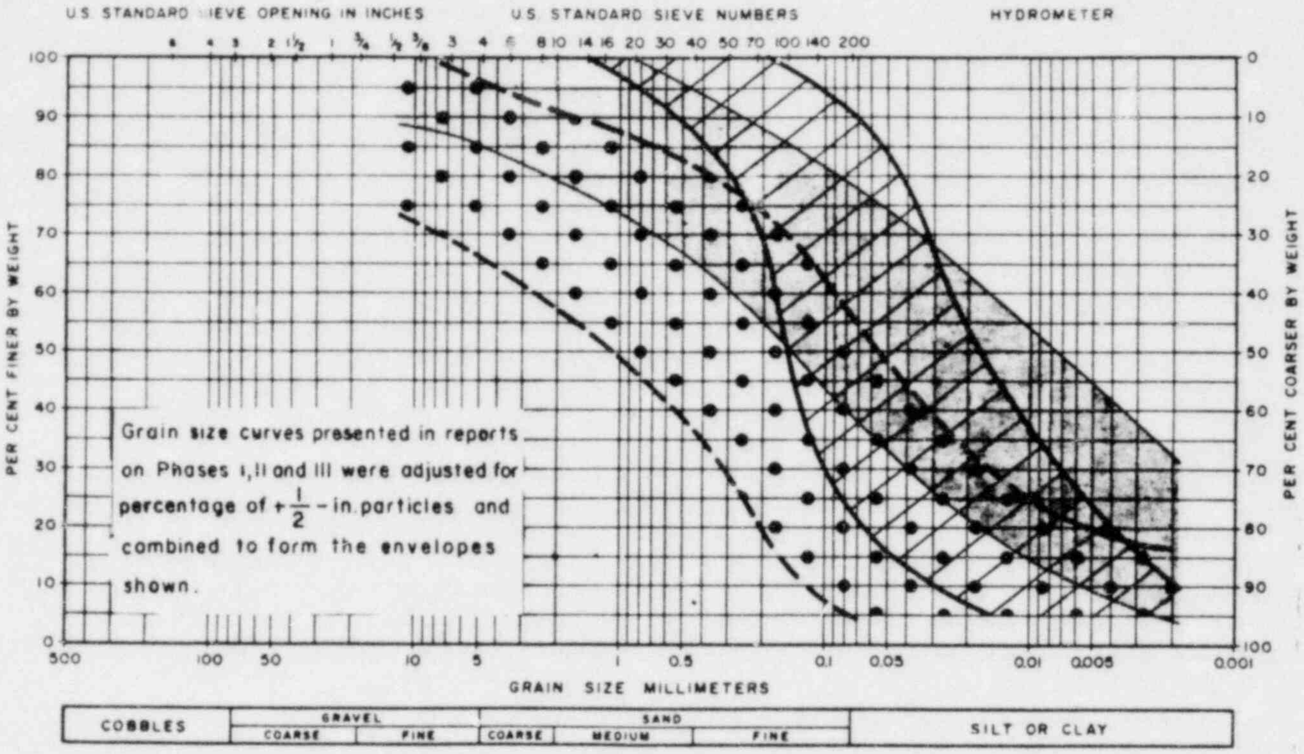
CORRECTED BLOWCOUNTS VS. DEPTH AT STATIONS 7+00, 10+00 AND 12+00

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

PLATE 1270-76 8



KEY

Dumped Shell

Washed Zone

Hydraulic Core

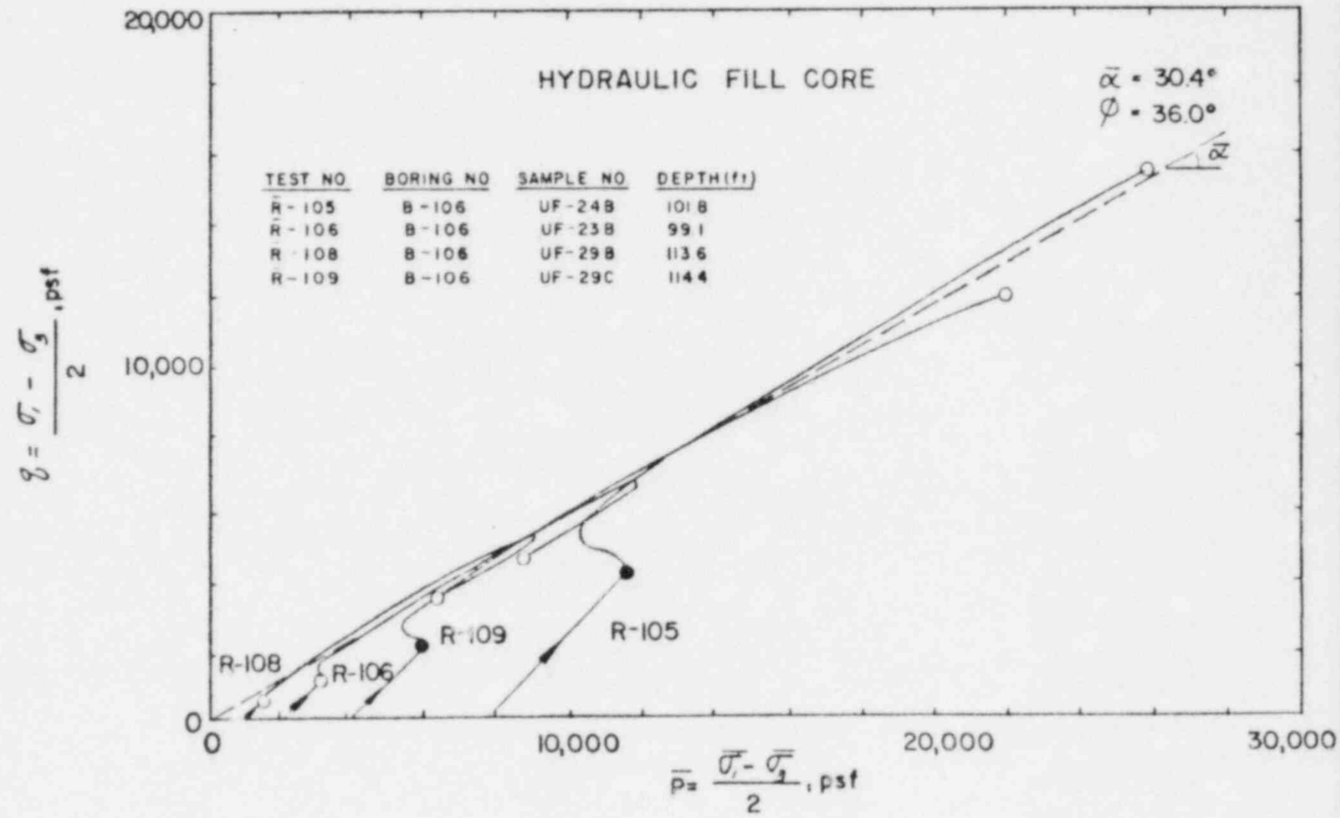
ENVELOPES OF GRAIN SIZE CURVES

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

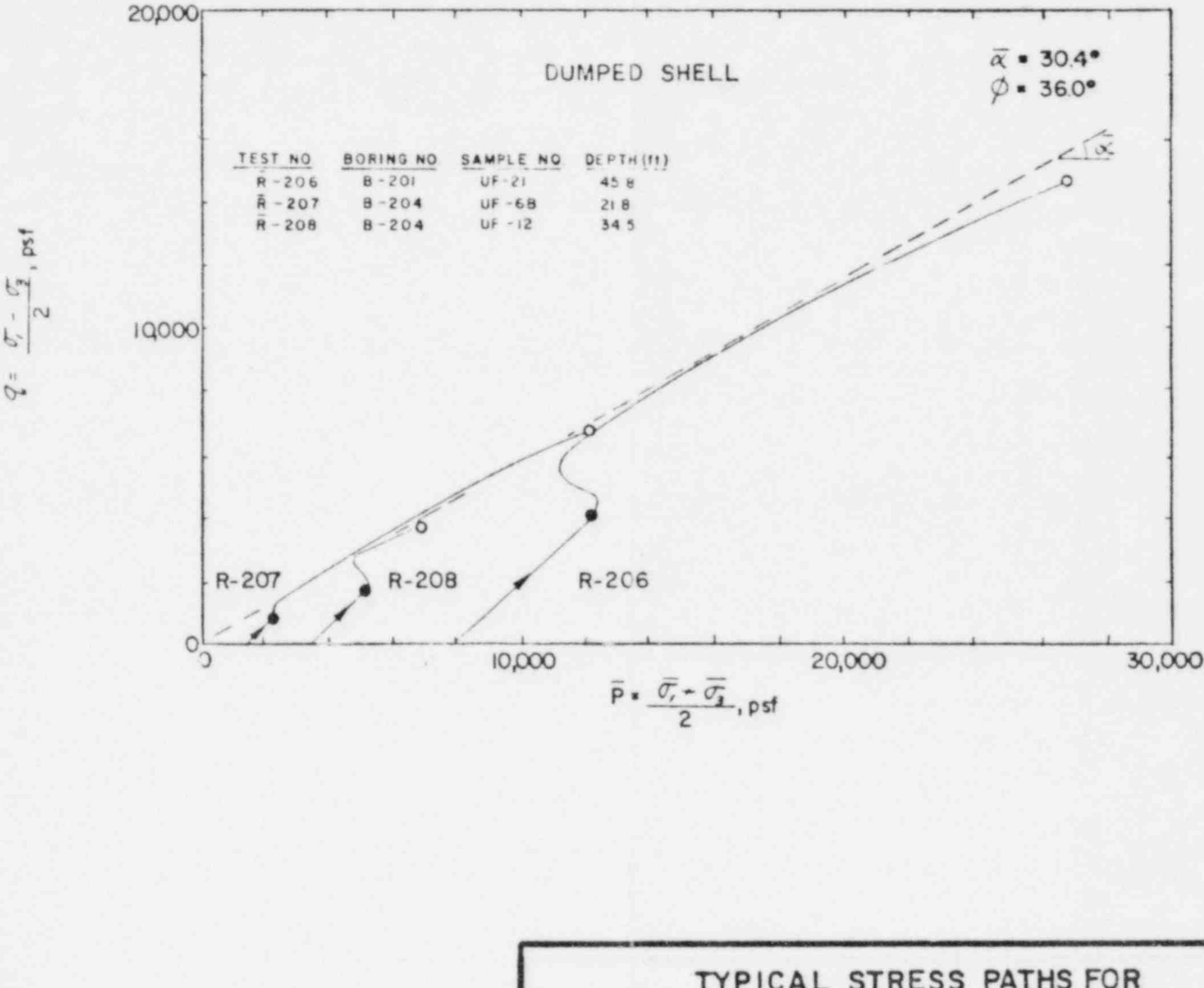
MAIN
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

PLATE
1270 — 76 9



HYDRAULIC FILL CORE
$\bar{\alpha}$ = 30.4°
$\bar{\phi}$ = 36.0°
$\bar{\alpha}$
TEST NO | BORING NO | SAMPLE NO | DEPTH(ft)
$\bar{R}$ - 105 | B - 106 | UF - 24B | 101.8
$\bar{R}$ - 106 | B - 106 | UF - 23B | 99.1
R - 108 | B - 106 | UF - 29B | 113.6
R - 109 | B - 106 | UF - 29C | 114.4
R-108
R-106
R-109
R-105
20,000
10,000
0
0
10,000
20,000
30,000
$q = \frac{\sigma_1 - \sigma_3}{2}$, psf
$\bar{p} = \frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$, psf



TYPICAL STRESS PATHS FOR
$\bar{R}$ TESTS ON EMBANKMENT MATERIALS

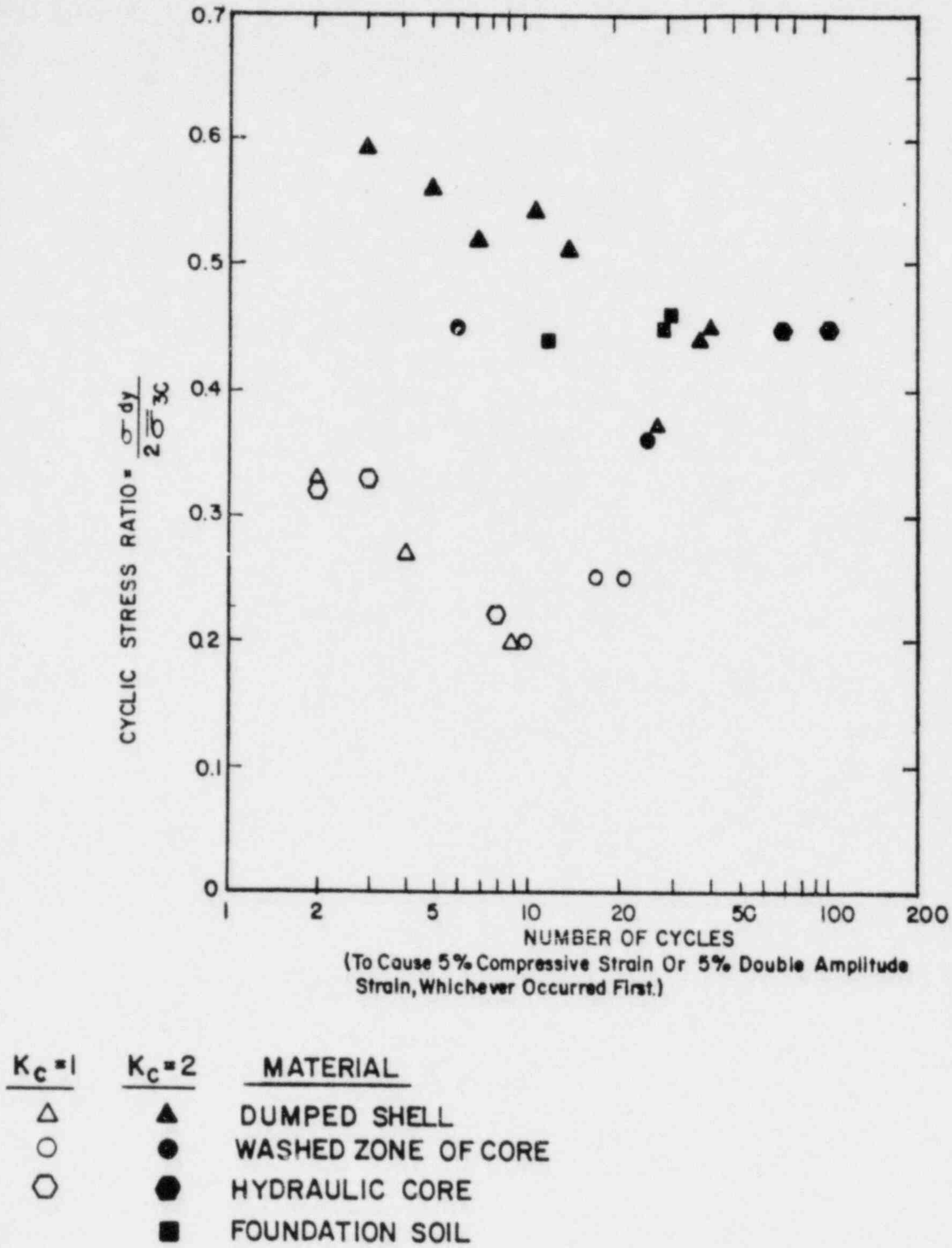
NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN

GEOTECHNICAL ENGINEERS INC

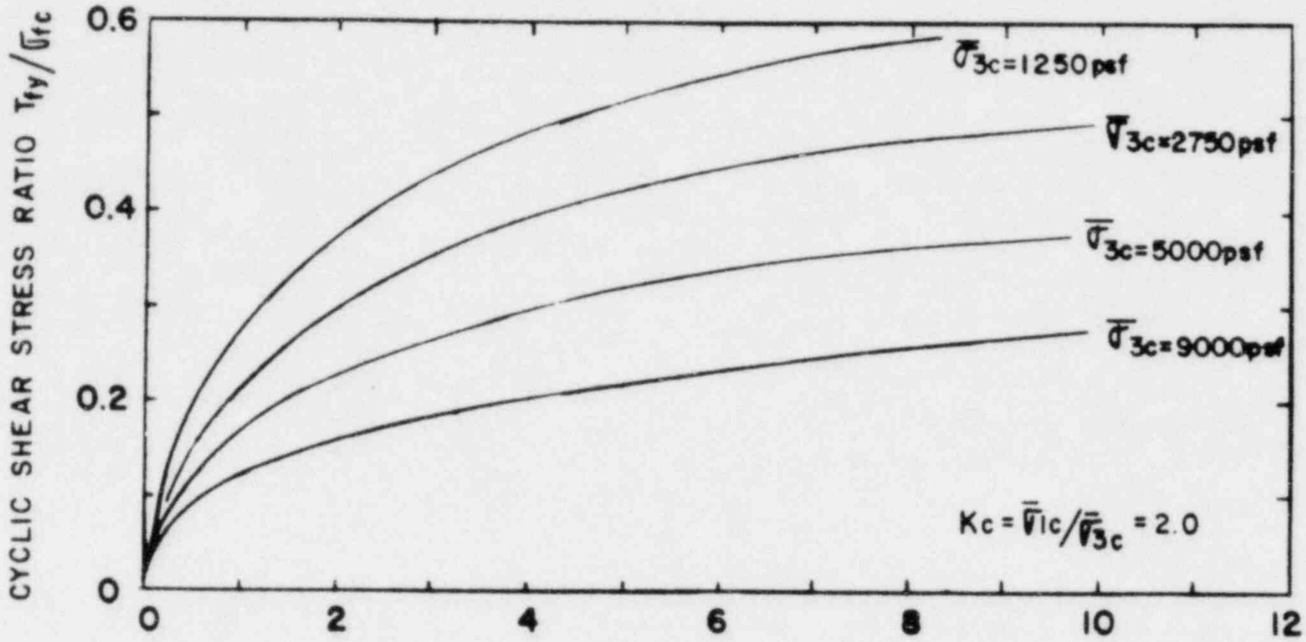
DATE MAY, 1982

PLATE 1270-76 10



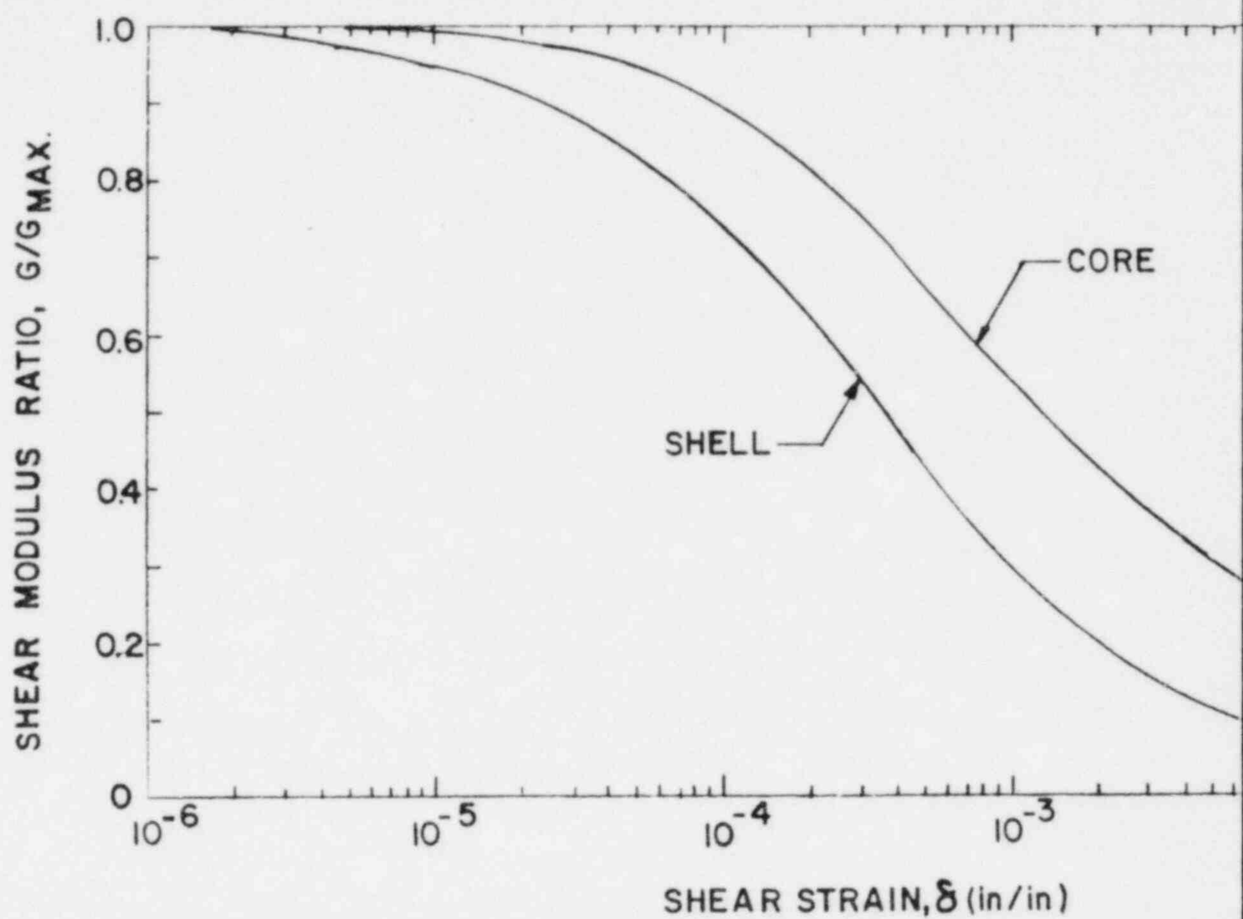
SUMMARY OF CYCLIC TRIAXIAL TEST RESULTS		
NEW ENGLAND POWER COMPANY HARRIMAN DAM		
MAIN GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982	
	PLATE 1270-76	11

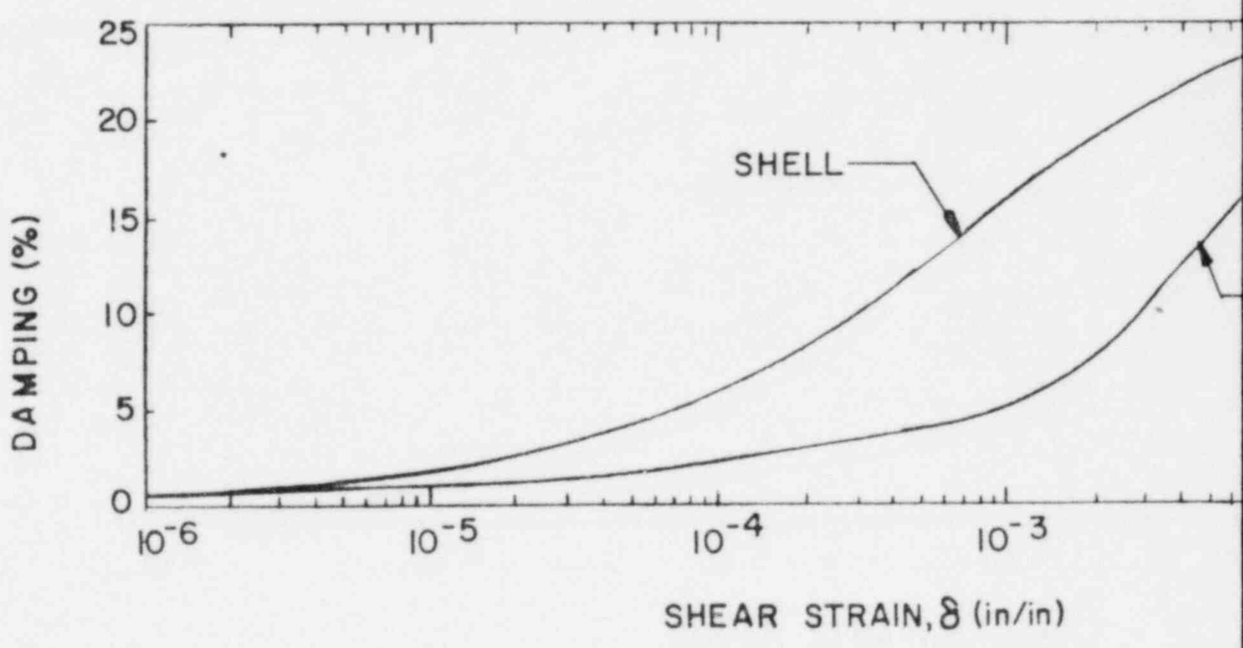
DUMPED SHELL AND HYDRAULIC CORE

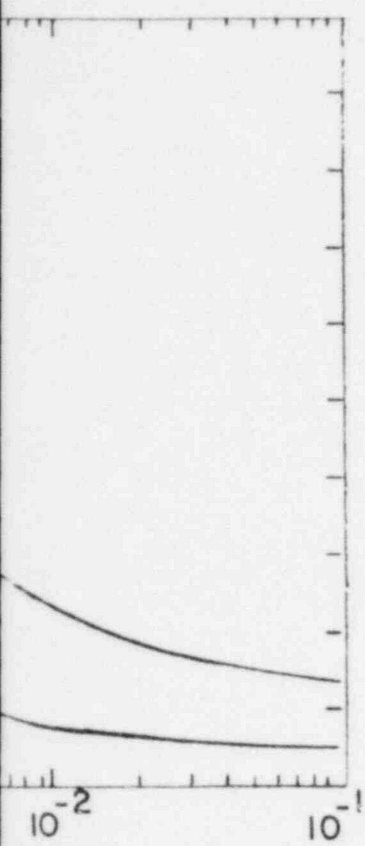


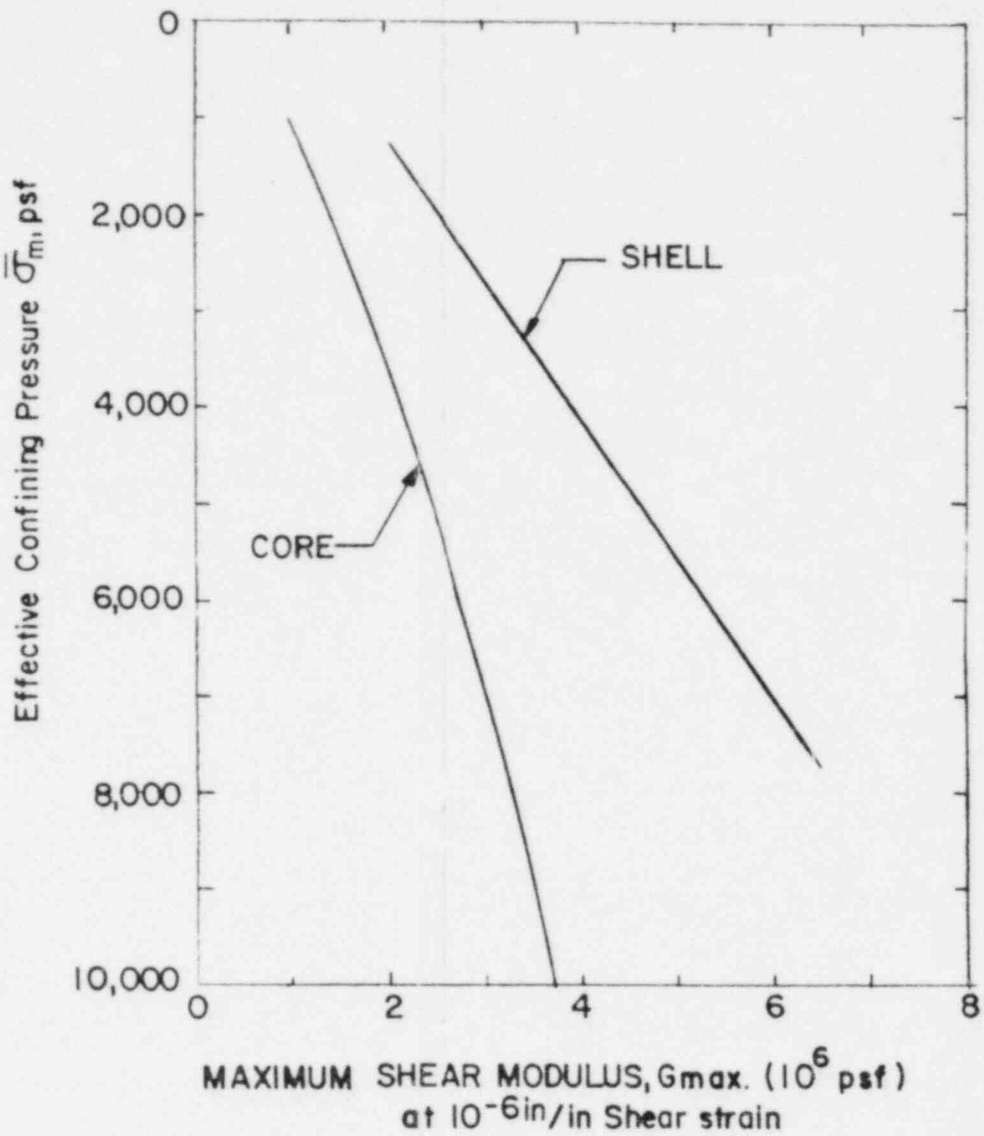
SHEAR STRAIN, δ (%)
(AFTER 5 CYCLES OF LOADING)

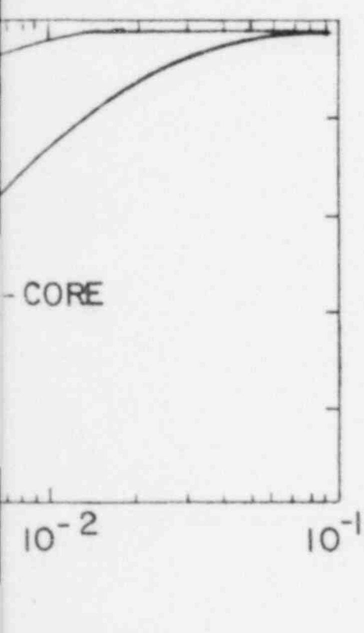
CYCLIC SHEAR STRESS RATIO VS. SHEAR STRAIN	
NEW ENGLAND POWER COMPANY HARRIMAN DAM	
MAIN GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982
	PLATE 1270-76 12


1.0
0.8
0.6
0.4
0.2
0
SHEAR MODULUS RATIO, G/G MAX.
CORE
SHELL
10^{-6}
10^{-5}
10^{-4}
10^{-3}
SHEAR STRAIN, δ (in/in)


25
20
15
10
5
0
DAMPING (%)
SHELL
10^{-6}
10^{-5}
10^{-4}
10^{-3}
SHEAR STRAIN, δ (in/in)







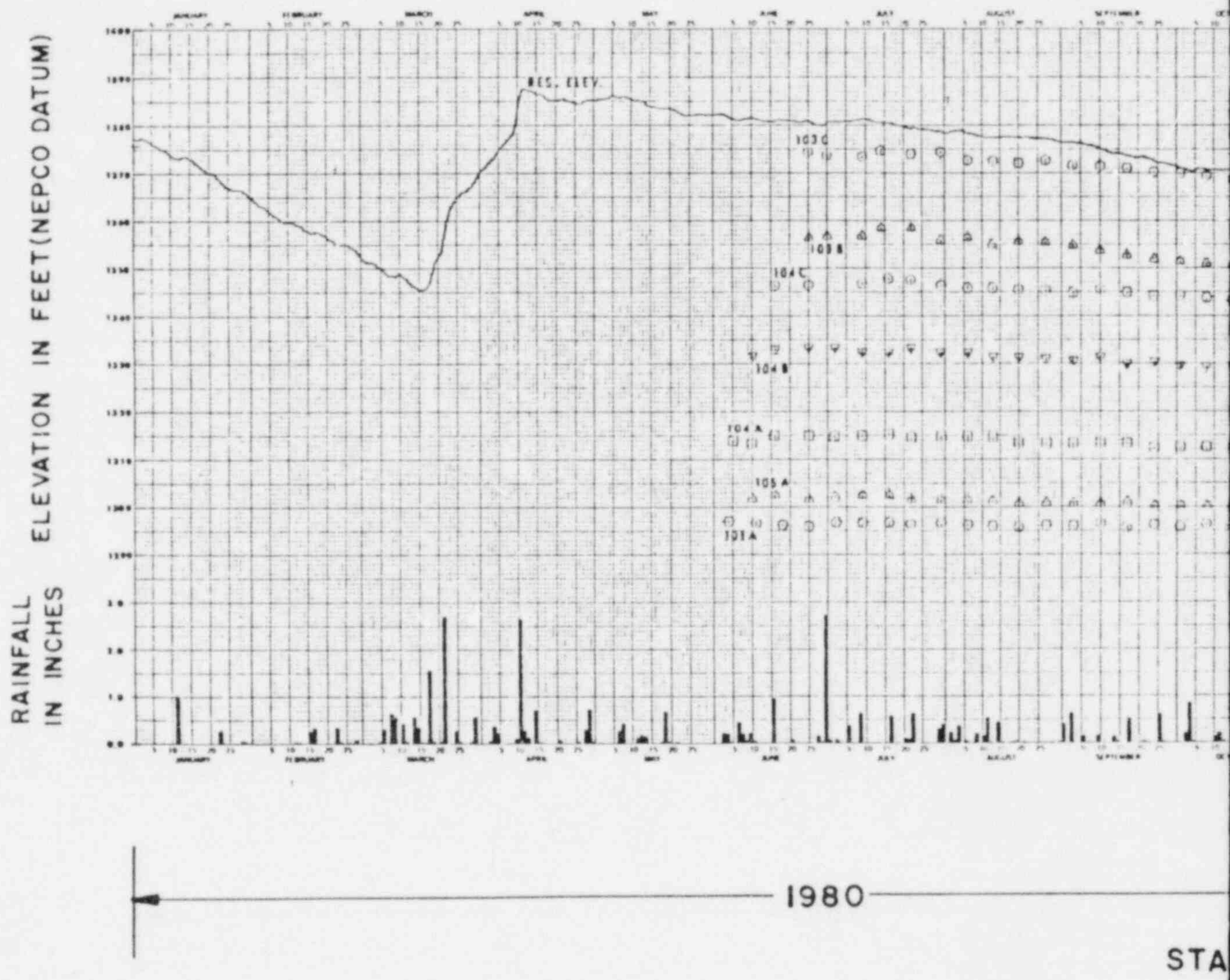
SUMMARY OF SEISMIC ANALYSIS PARAMETERS

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC

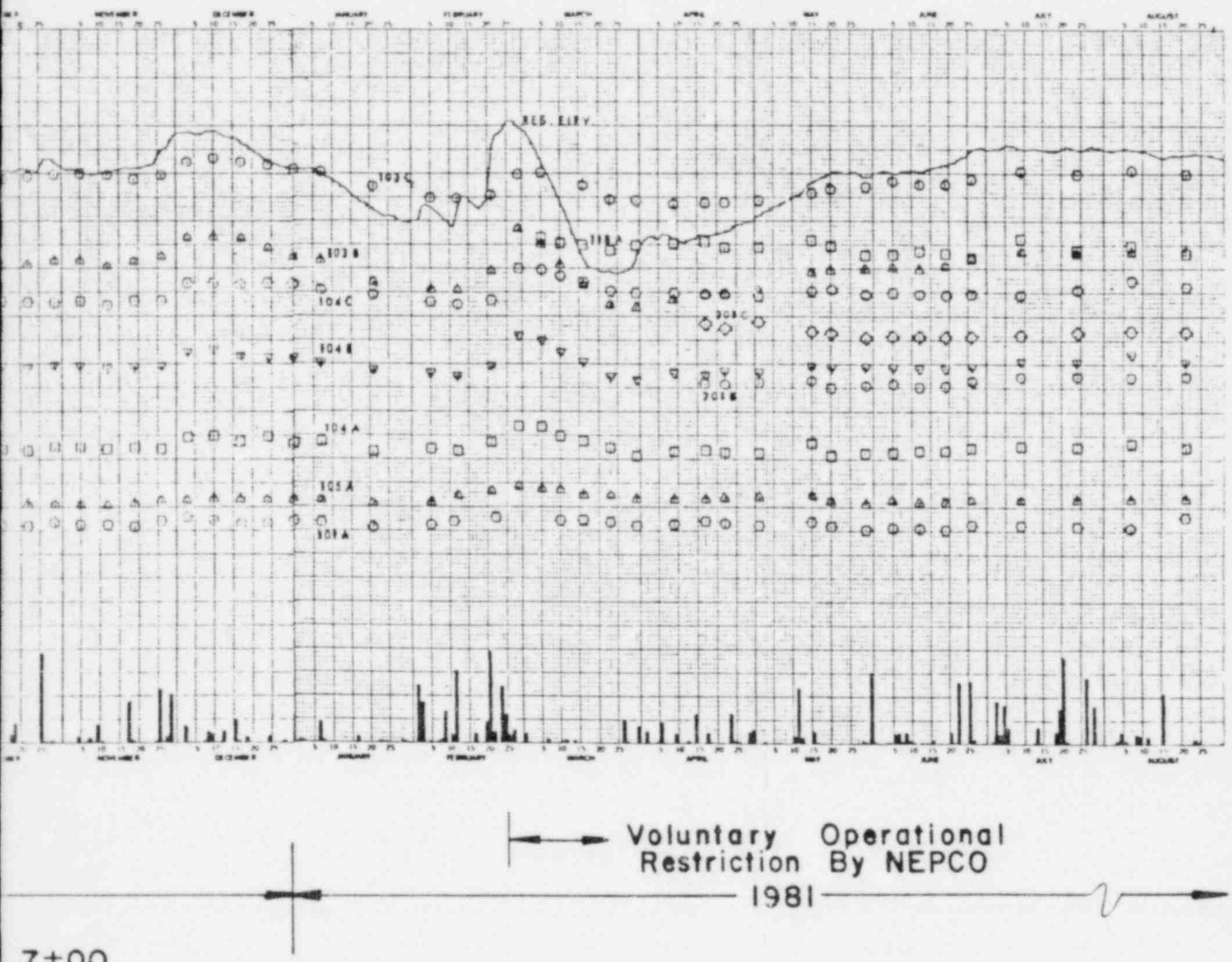
DATE MAY, 1982

PLATE 1270-76 13



ELEVATION IN FEET(NEPCO DATUM)
RAINFALL IN INCHES
RES. ELEV.
103 C
103 B
104 C
104 B
104 A
105 A
101 A
1980

STA

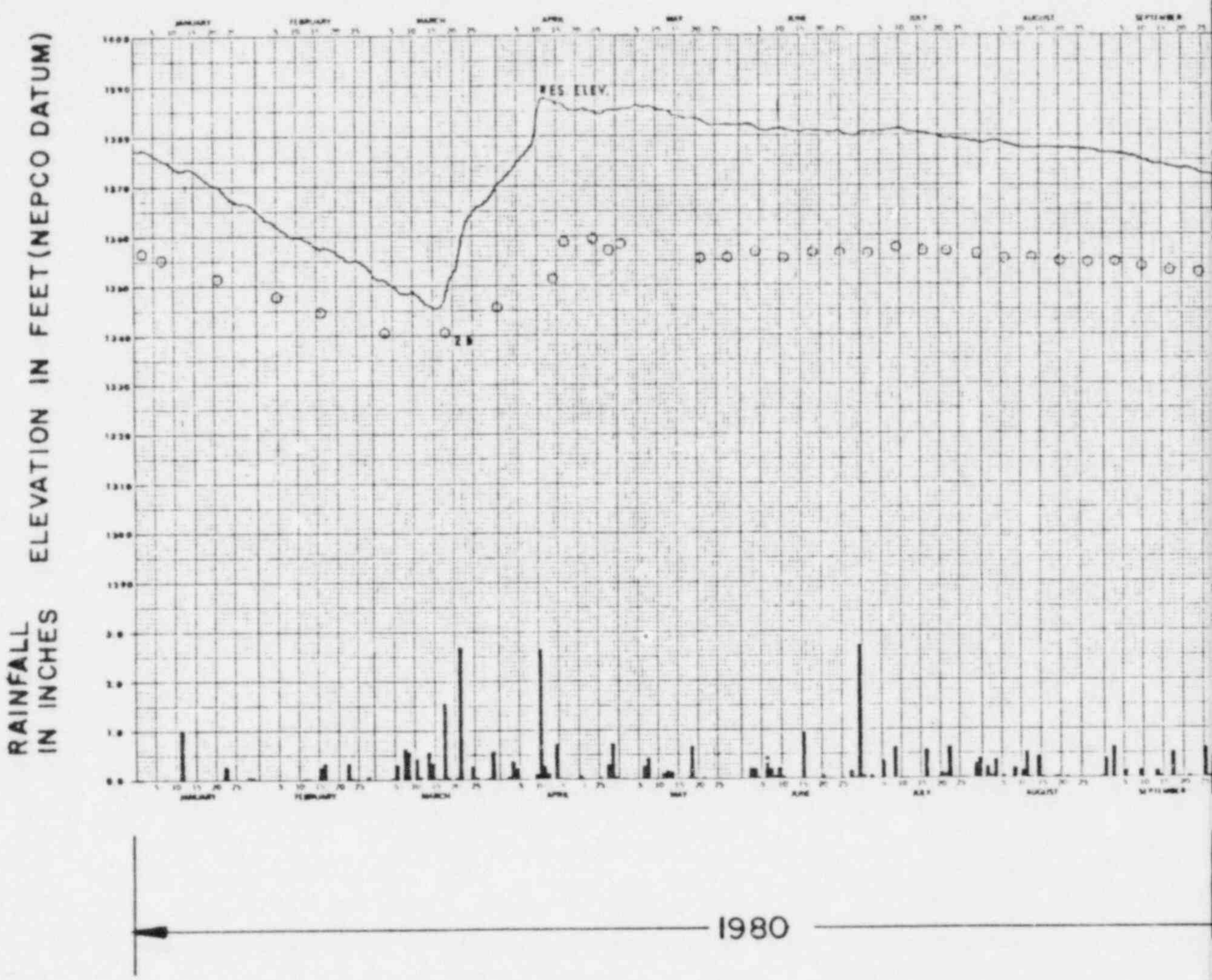


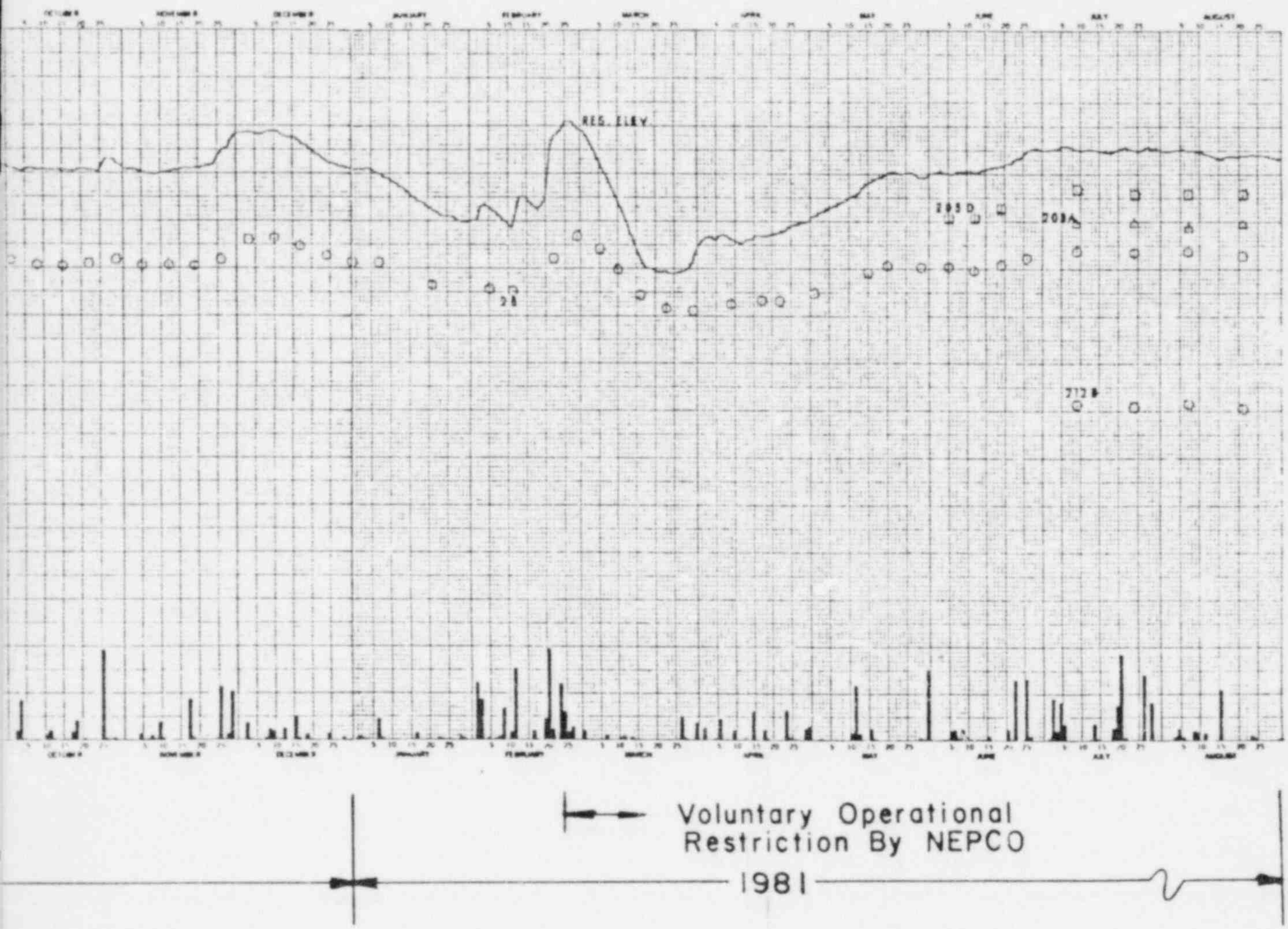
7+00

PIEZOMETER DATA AT STA. 7+00

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982
	PLATE 1270-76 14


ELEVATION IN FEET (NEPCO DATUM)
RAINFALL IN INCHES
RES. ELEV.
1980



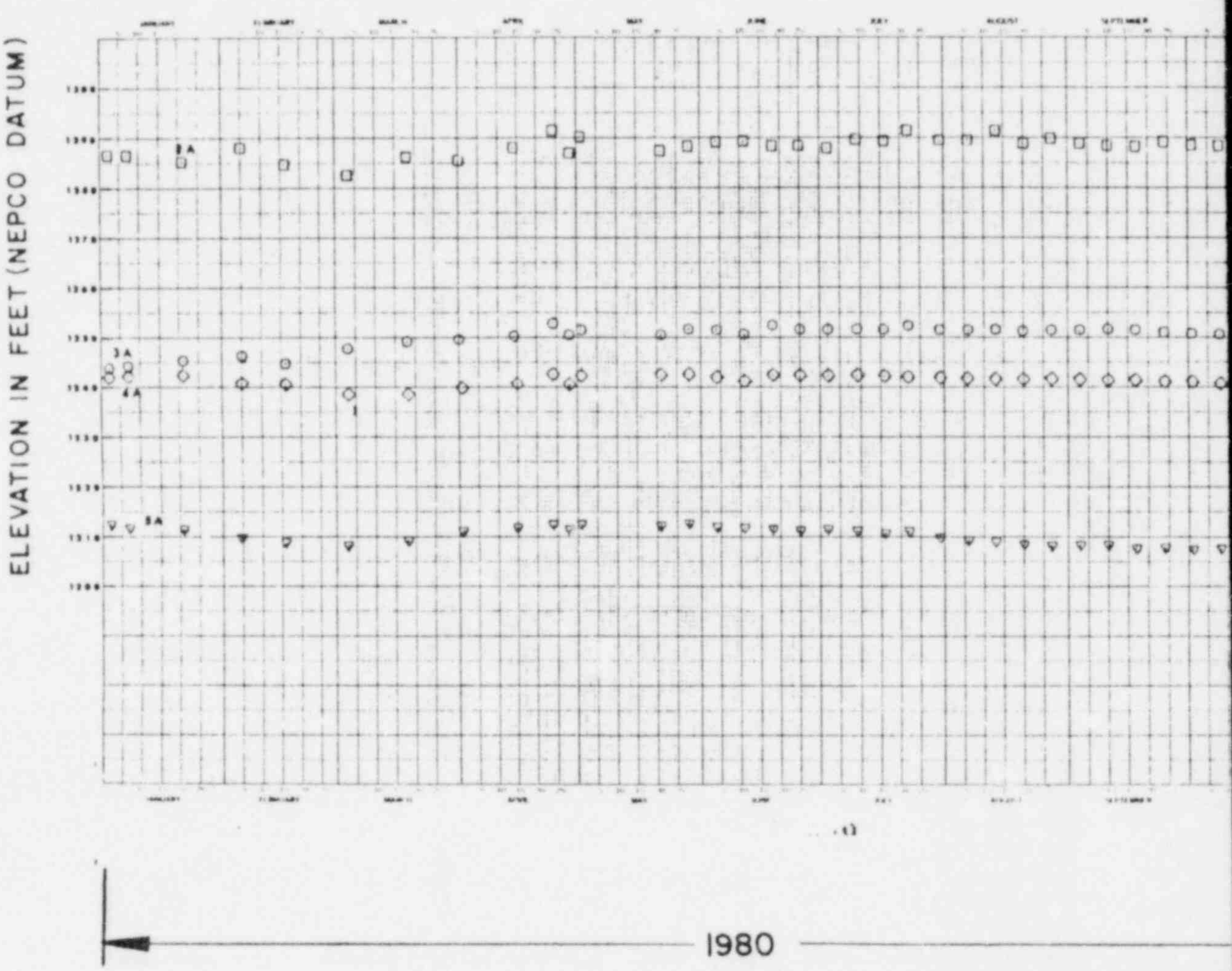
STA. 10+00
(ABOVE 1300)

PIEZOMETER DATA AT STA. 10+00
(ABOVE ELEV. 1300)

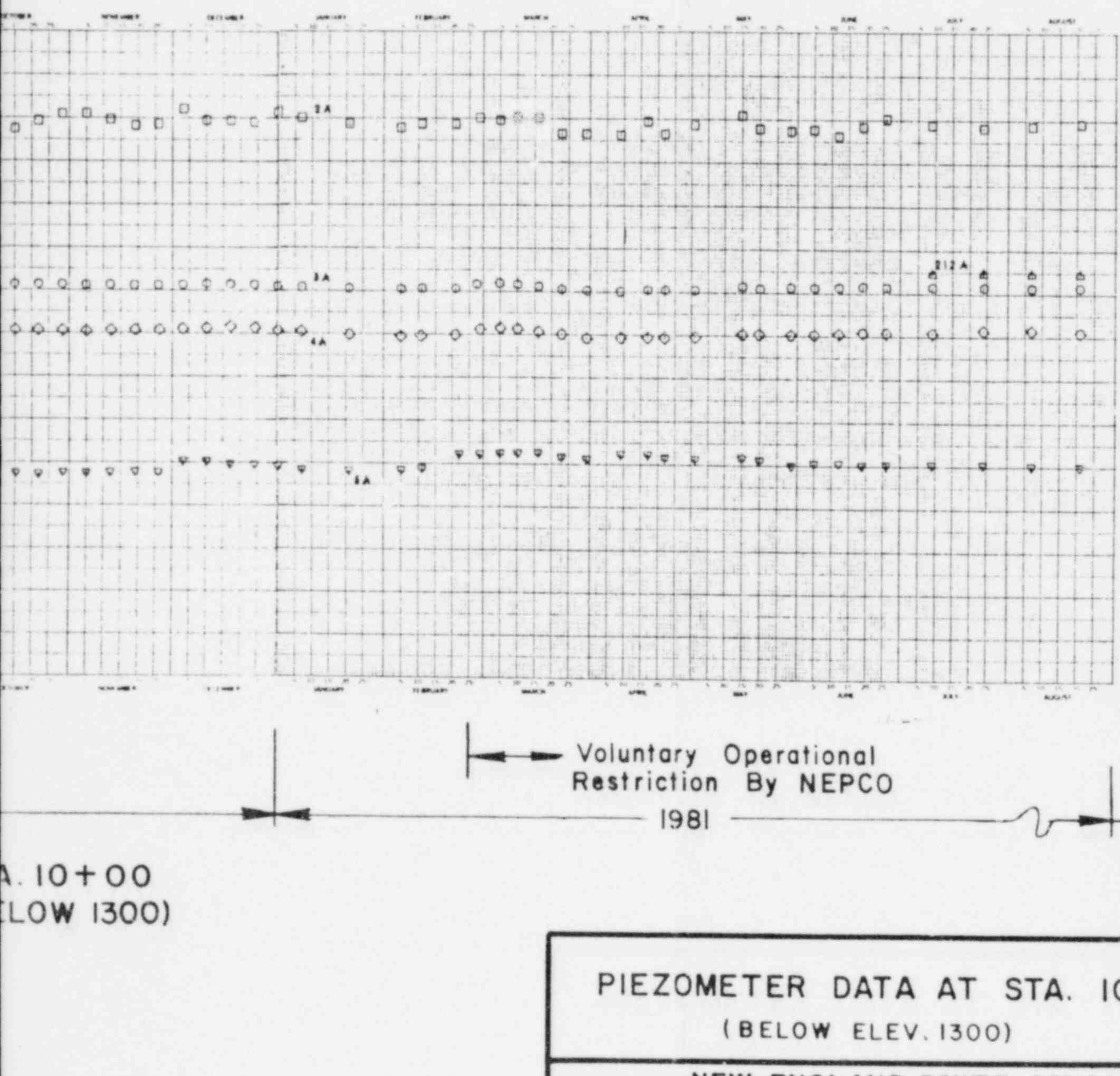
NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC.

DATE	MAY, 1982
PLATE 1270-76	15a



STA
(BE



A. 10+00
LOW 1300)

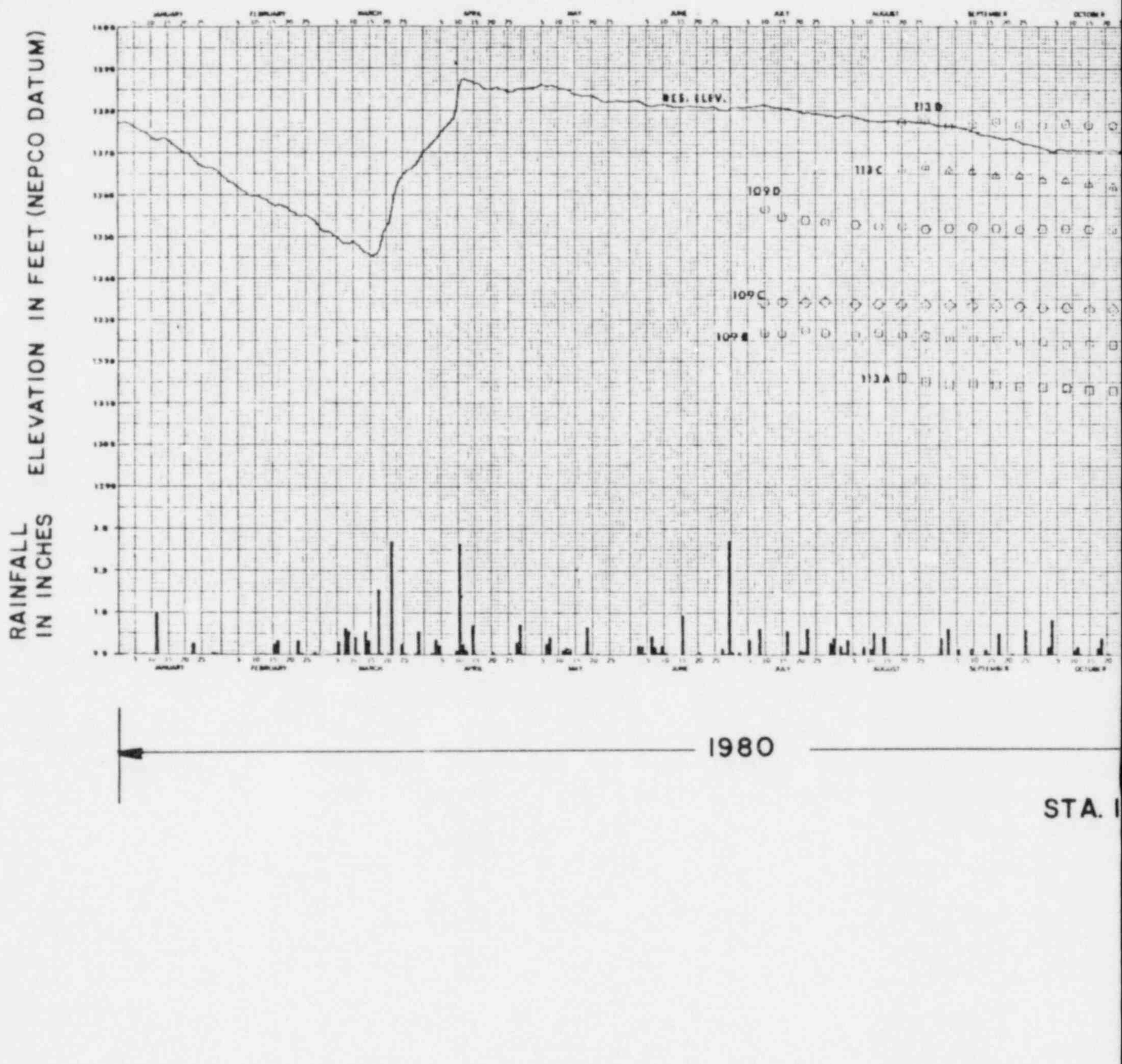
PIEZOMETER DATA AT STA. 10+00
(BELOW ELEV. 1300)

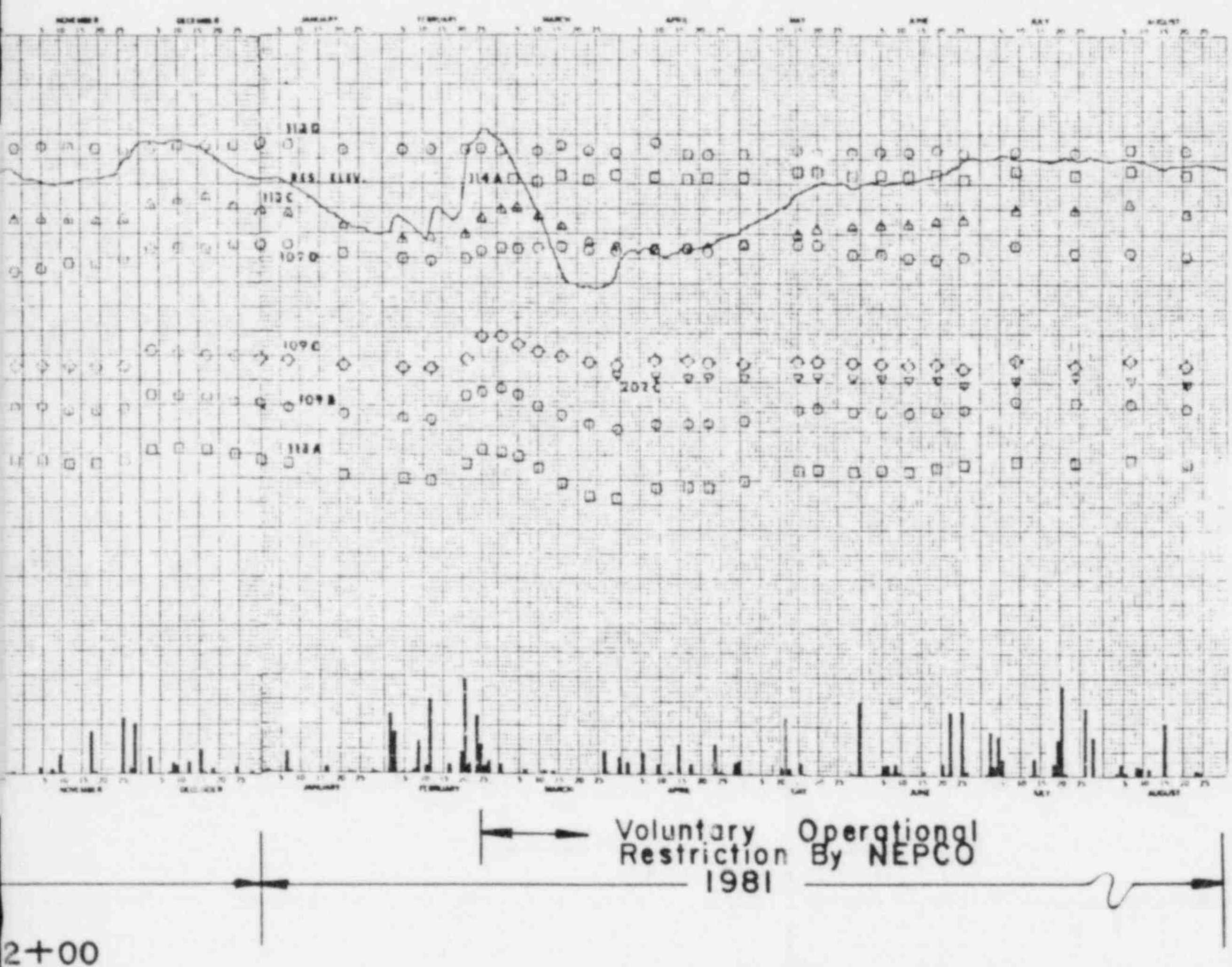
NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

PLATE 1270-76 15b

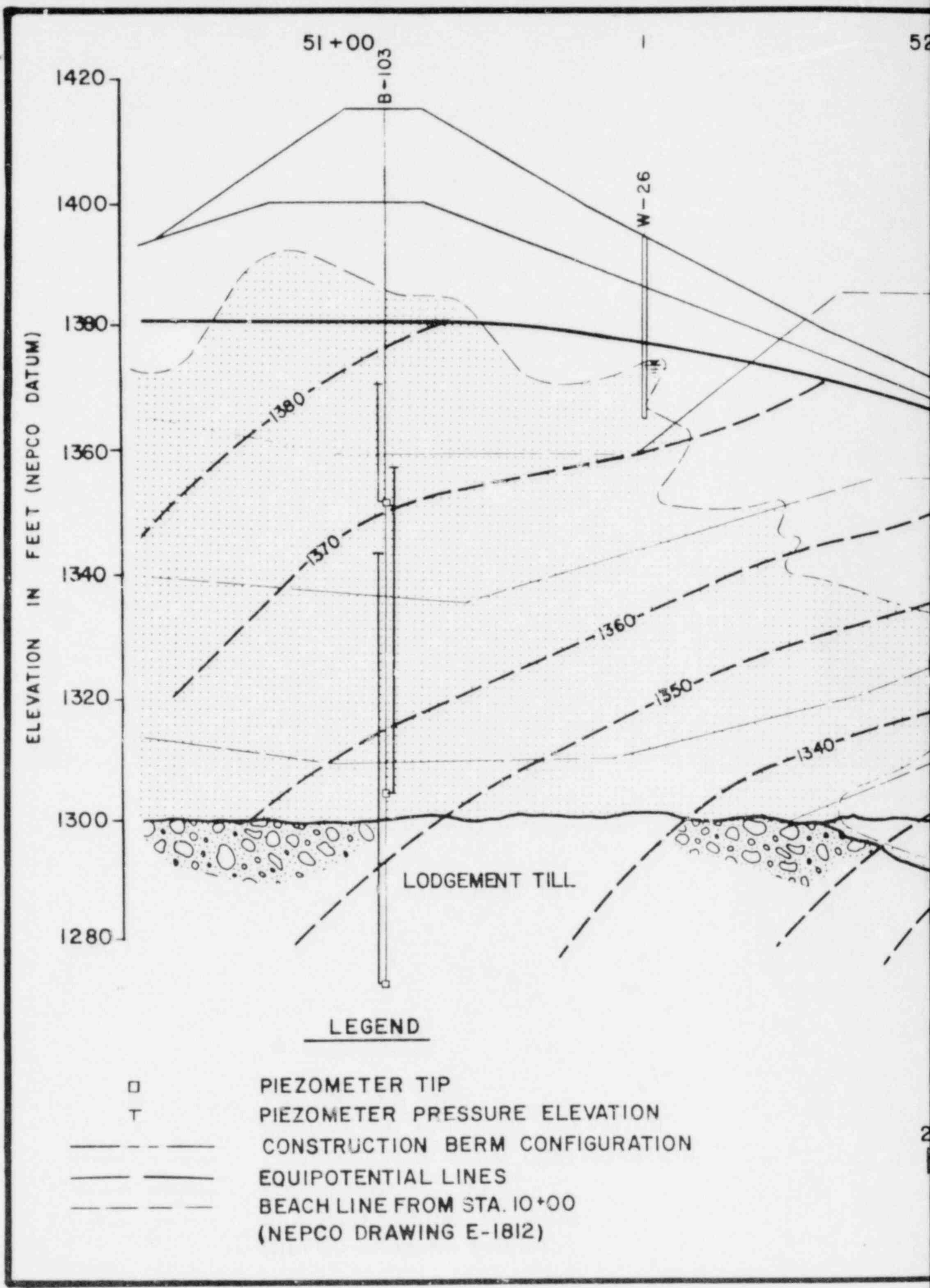

RAINFALL IN INCHES
ELEVATION IN FEET (NEPCO DATUM)
RES. ELEV.
113 D
113 C
109 D
109 C
109 B
113 A
1980
STA. I



PIEZOMETER DATA AT STA. 12+00

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

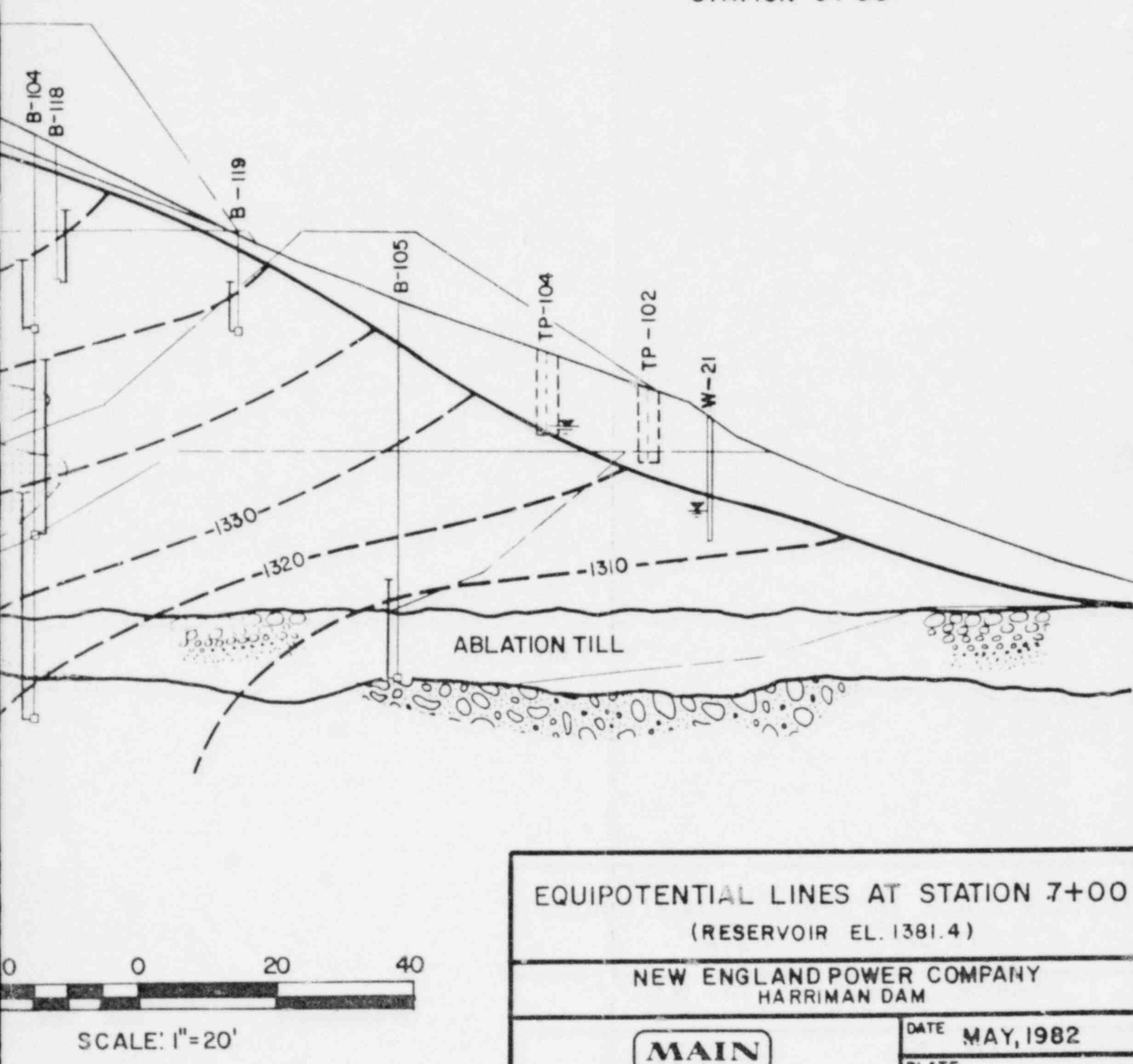
MAIN GEOTECHNICAL ENGINEERS INC.	DATE MAY, 1982
	PLATE 1270-76 16


51+00
B-103
W-26
1420
1400
1380
1360
1340
1320
1300
1280
ELEVATION IN FEET (NEPCO DATUM)
1380
1370
1360
1350
1340
LODGEMENT TILL
LEGEND
PIEZOMETER TIP
PIEZOMETER PRESSURE ELEVATION
CONSTRUCTION BERM CONFIGURATION
EQUIPOTENTIAL LINES
BEACH LINE FROM STA. 10+00
(NEPCO DRAWING E-1812)

+00 | 53+00

NOTES:

1) PIEZOMETER DATA AND EQUIPOTENTIAL LINES ARE FOR RESERVOIR AT EL. 1381.4
2) TP-104 AND W-21 ARE AT STATION 9+00



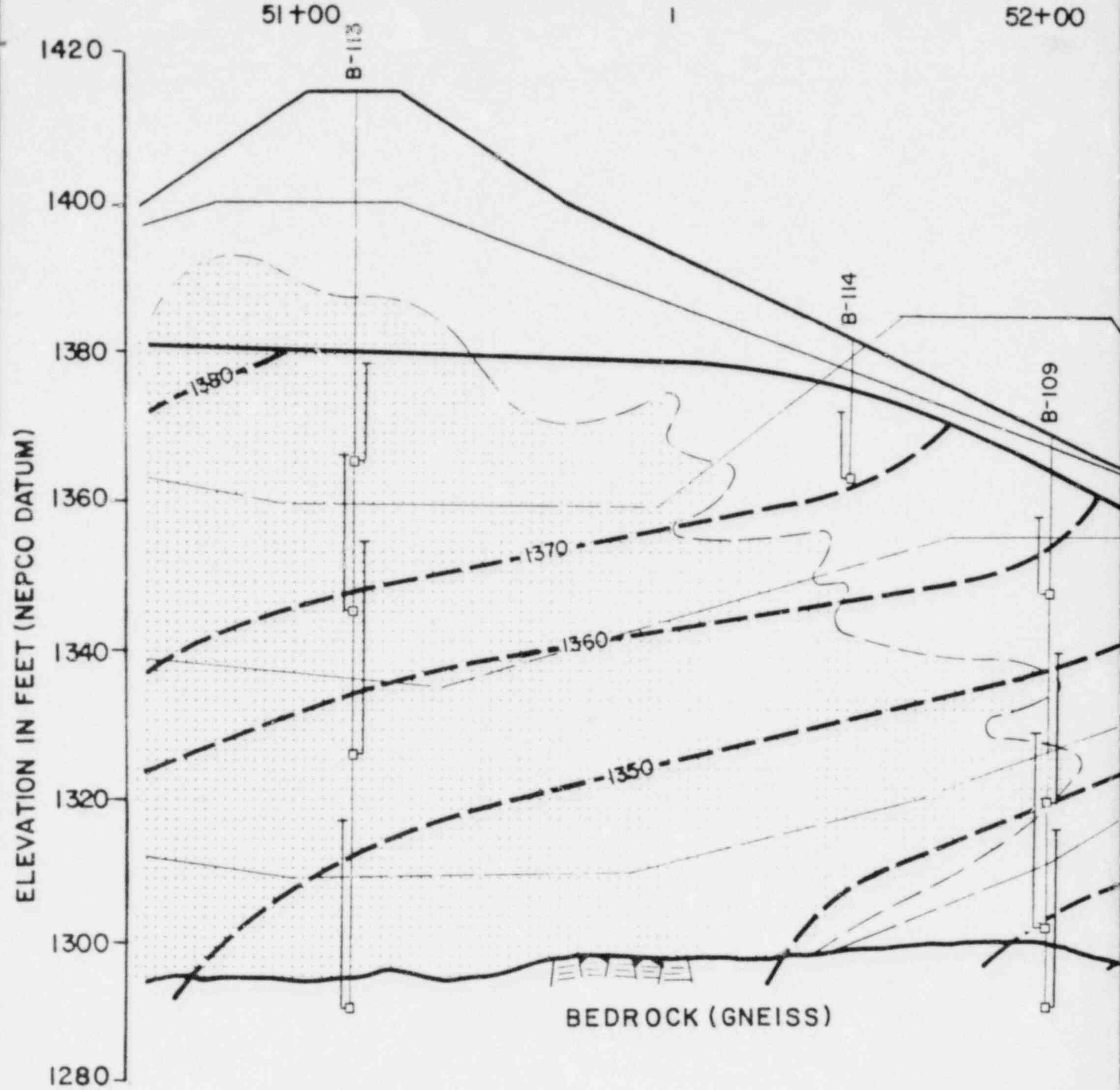
EQUIPOTENTIAL LINES AT STATION 7+00

(RESERVOIR EL. 1381.4)

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC.

DATE	MAY, 1982
PLATE	1270-76 17



LEGEND

□ PIEZOMETER TIP

T PIEZOMETER PRESSURE ELEVATION

CONSTRUCTION BERM CONFIGURATION

EQUIPOTENTIAL LINES

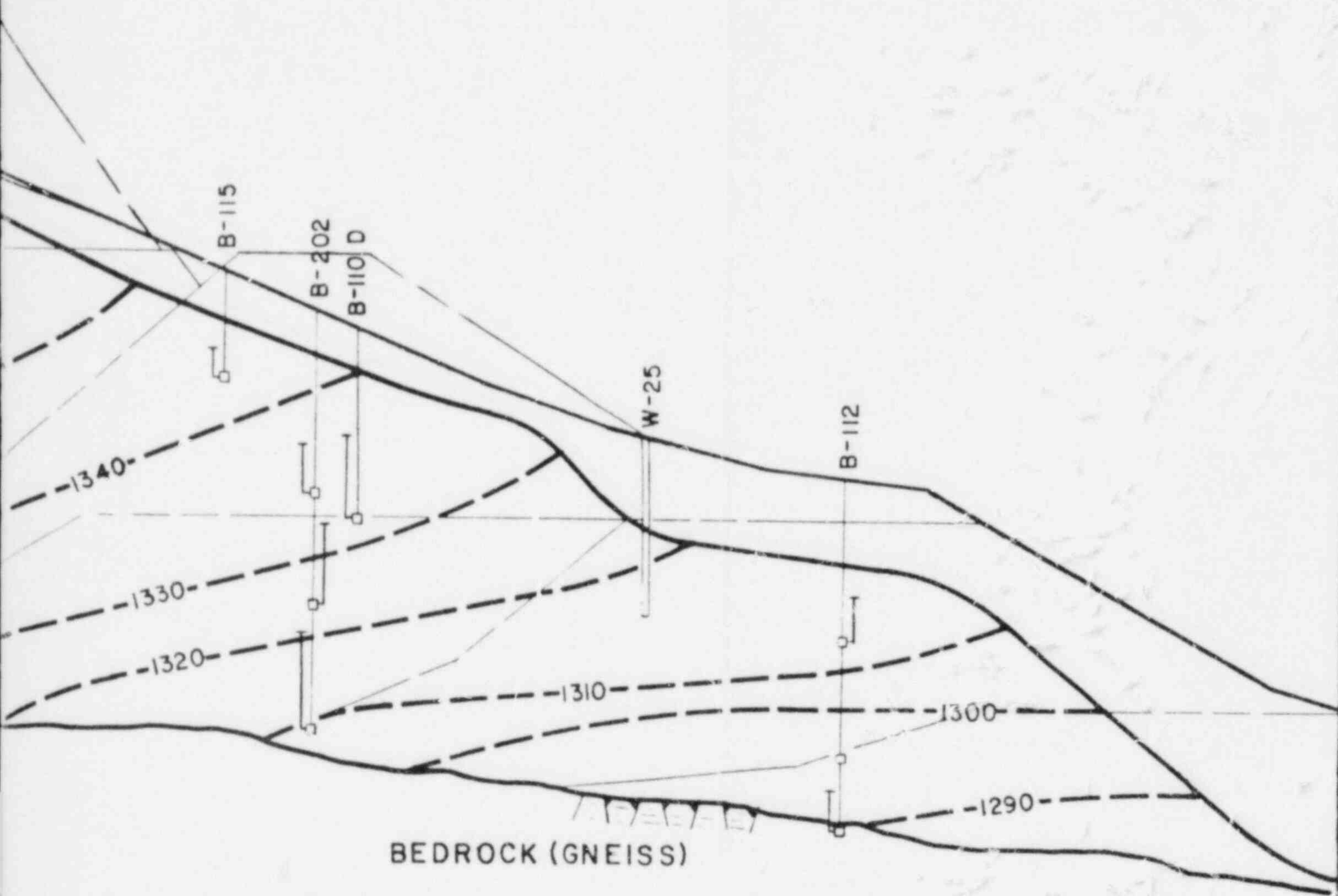
BEACH LINE FROM STA. 10+00 (NEPCO DRAWING E-1812)

1

53+00

NOTE:

PIEZOMETER DATA AND EQUIPOTENTIAL LINES ARE FOR RESERVOIR AT EL. 1381.4



0 20 40

SCALE: 1" = 20'

EQUIPOTENTIAL LINES AT STATION 12+00

(RESERVOIR EL. 1381.4)

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN

GEOTECHNICAL ENGINEERS INC

DATE	MAY, 1982
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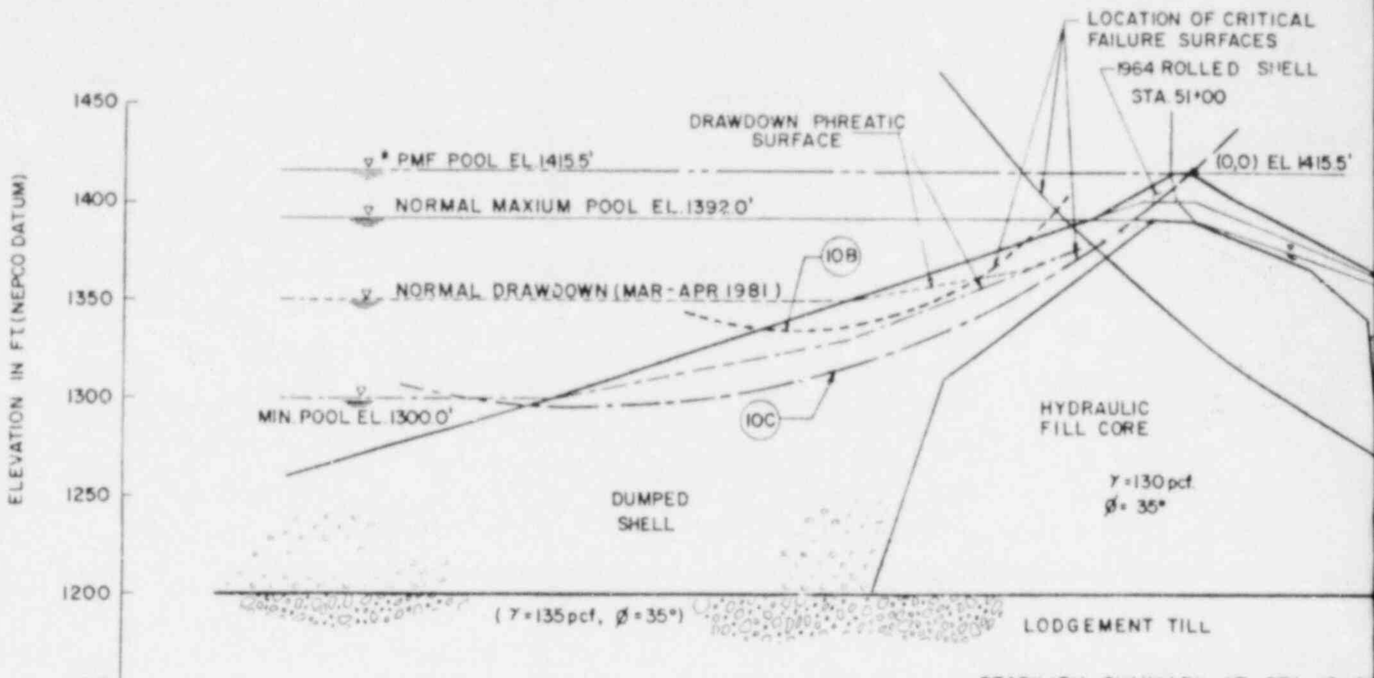
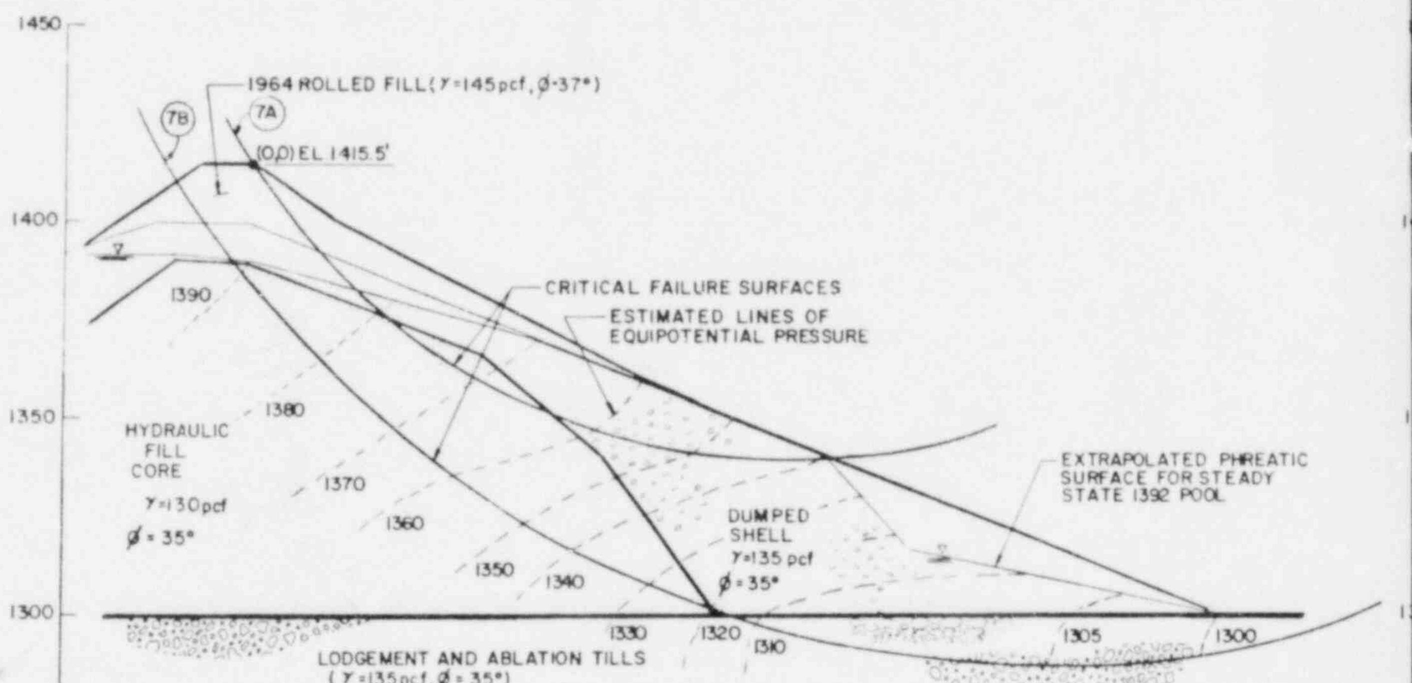


STABILITY SUMMARY AT STA. 10+00
SCALE 1" = 40'



STABILITY SUMMARY AT STA. 7+00
SCALE 1" = 20'

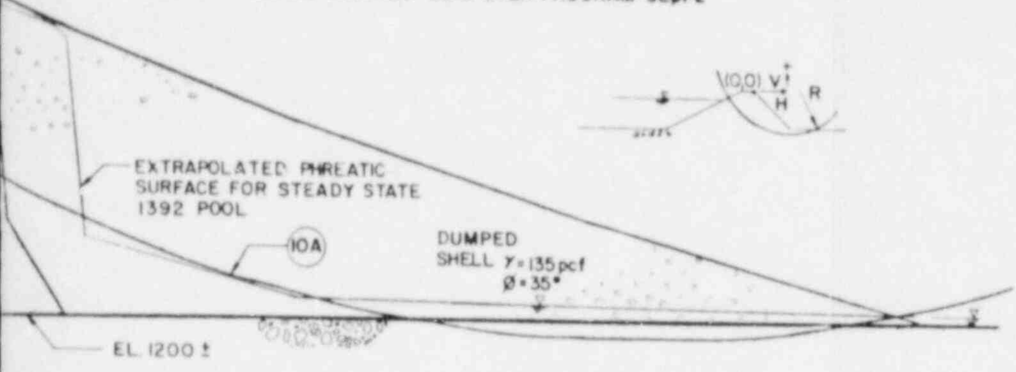
1250

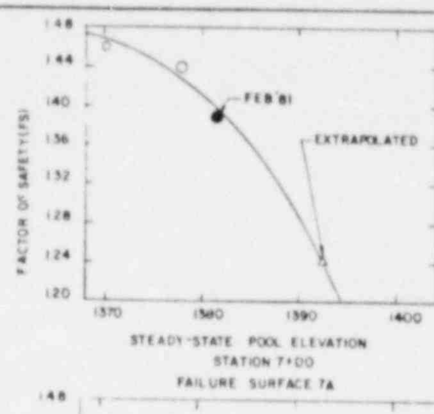
ANALYSIS METHOD	CASE NO.	H	V	R	FS	NOTES
COMPUTER SLØPE	7A	137.5	86.0	162.1	1.18	MAX. STEADY STATE POOL - SHALLOW SURFACE
	7B	205.0	135.0	265.1	1.21	MAX. STEADY STATE POOL - DEEP SURFACE
HAND BISHOP	7A	137.5	86.0	162.1	1.24	MAX. STEADY STATE POOL - SHALLOW SURFACE
	7B	205.0	135.0	265.1	1.55	MAX. STEADY STATE POOL - DEEP SURFACE

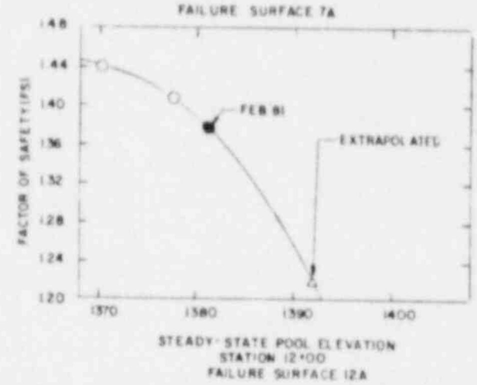
NOTE: HAND BISHOP ANALYSIS IS MORE ACCURATE THAN THE COMPUTER (SLOPE) ANALYSIS, DUE TO LOW ANGLE EQUIPOTENTIAL LINES

CASE NO.	H	V	R	FS	NOTES
10A	425.0	455.0	690.1	1.81	MAX STEADY STATE POOL - D/S STABILITY
10B	-207.5	90.0	170.1	1.79	RAPID DRAW TO 1351 - U/S STABILITY
10C	-317.5	325.0	465.1	1.54	DRAW TO 1300 U/S STABILITY
10A*	425.0	455.0	690.1	1.74	PMF ZERO - FREEBOARD - D/S STABILITY

NOTE: ANALYSIS METHOD - COMPUTER PROGRAM SLØPE

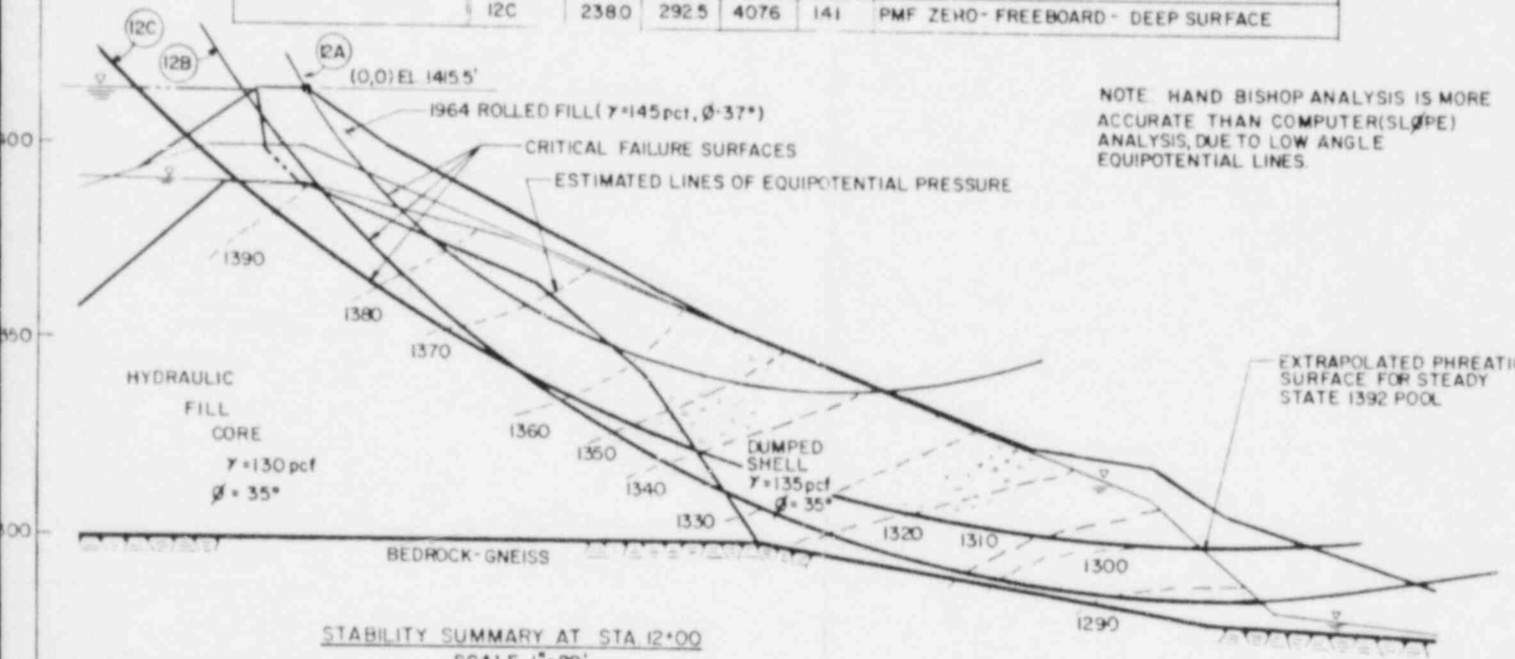






ANALYSIS METHOD	CASE NO.	H	V	R	FS	NOTES
COMPUTER SLØPE	12A	137.5	86.0	162.1	1.22	MAX STEADY-STATE POOL - SHALLOW SURFACE
	12B	235.0	180.0	307.6	1.30	MAX STEADY-STATE POOL - DEEP SURFACE
HAND BISHOP	12A	137.5	86.0	162.1	1.22	MAX STEADY-STATE POOL - SHALLOW SURFACE
	12B	235.0	180.0	307.6	1.53	MAX STEADY-STATE POOL - DEEP SURFACE
	12C	238.0	292.5	407.6	1.41	PMF ZERO - FREEBOARD - DEEP SURFACE

NOTE: HAND BISHOP ANALYSIS IS MORE ACCURATE THAN COMPUTER (SLØPE) ANALYSIS, DUE TO LOW ANGLE EQUIPOTENTIAL LINES.



STABILITY SUMMARY AT STA. 12+00

SCALE 1" = 20'

20 0 20 40

SCALE: 1" = 20'

40 0 40 80

SCALE: 1" = 40'

STATIC STABILITY SUMMARY AT STATIONS 7+00, 10+00 AND 12+00
AND
SHALLOW SLOPE STABILITY VERSUS POOL ELEVATION

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

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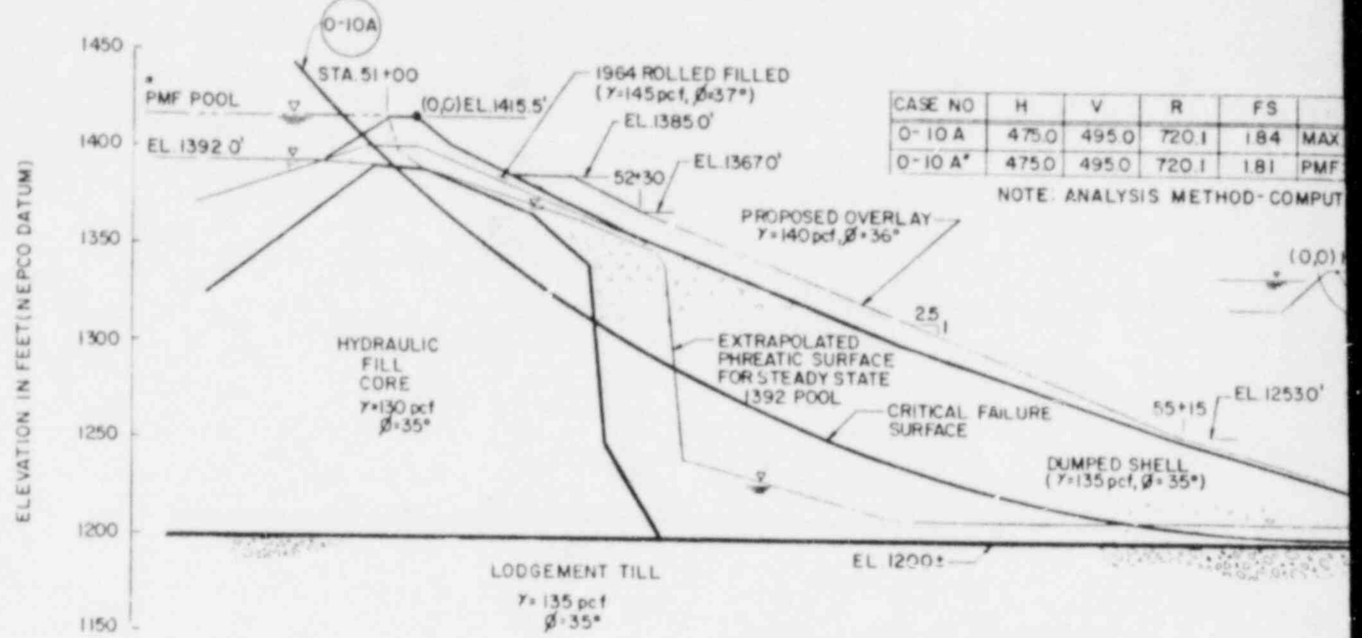
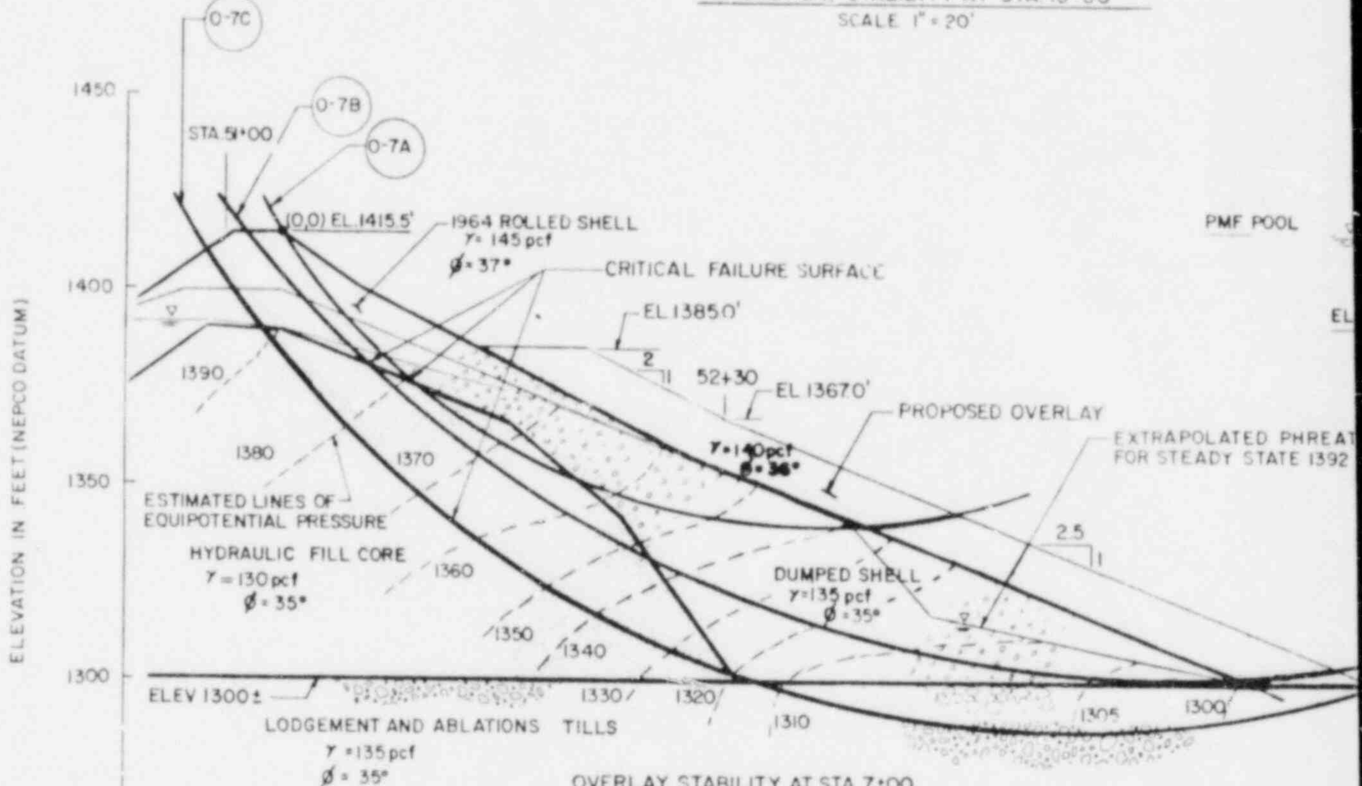


CASE NO	H	V	R	FS	
0-10 A	475.0	495.0	720.1	1.84	MAX
0-10 A*	475.0	495.0	720.1	1.81	PMF

NOTE: ANALYSIS METHOD-COMPUT

OVERLAY STABILITY AT STA. 10+00

SCALE 1"=20'



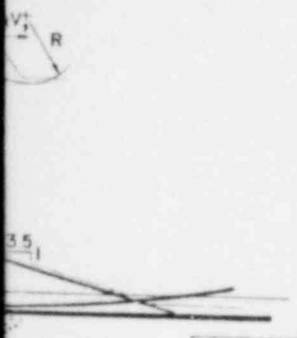
OVERLAY STABILITY AT STA. 7+00

SCALE 1"=20'

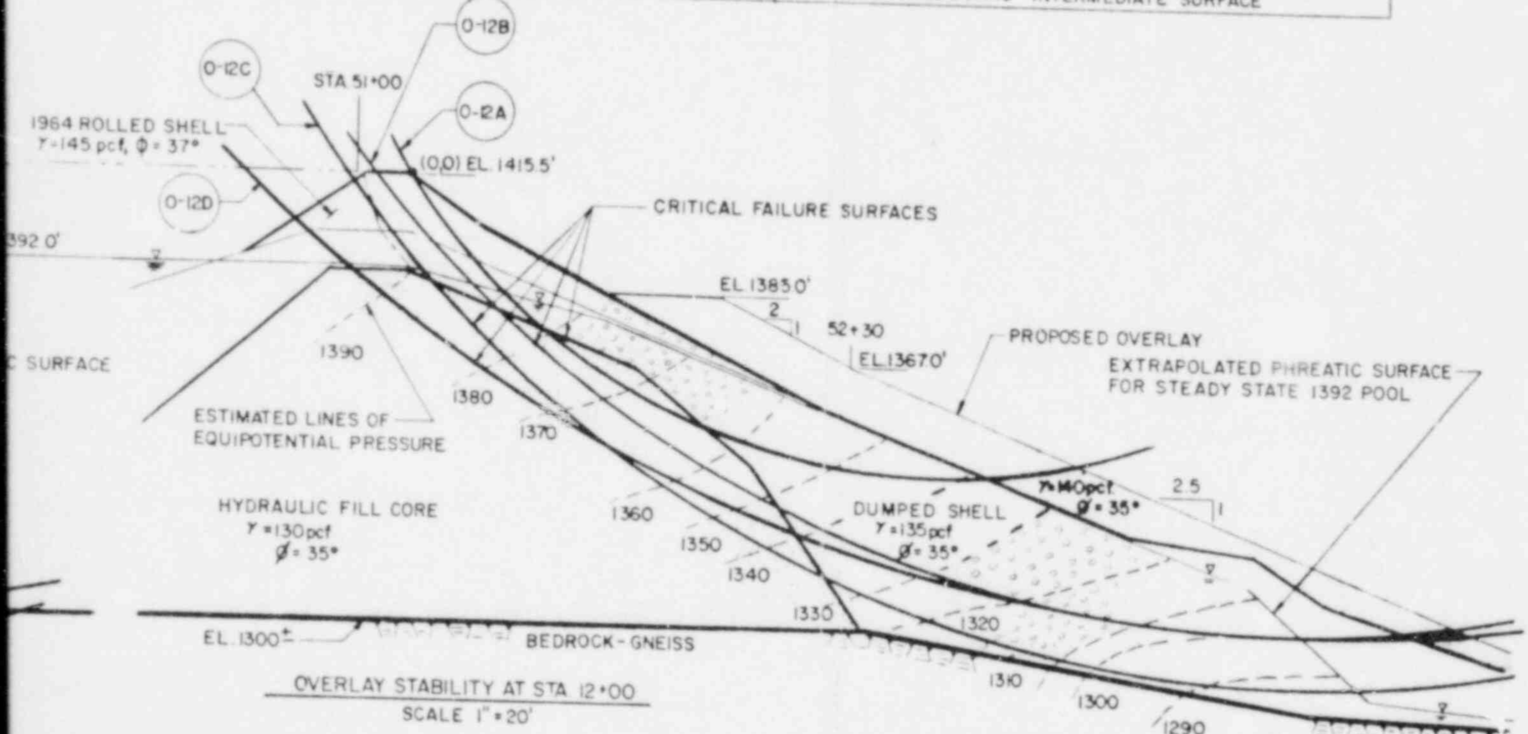
ANALYSIS METHOD	CASE NO.	H	V	R	FS	NOTES
HAND BISHOP	0-7A	137.5	86.0	162.1	1.91	MAX. STEADY-STATE POOL - SHALLOW SURFACE
	0-7B	230.0	190.0	305.1	1.81	MAX. STEADY-STATE POOL - INTERMEDIATE SURFACE
	0-7C	205.0	135.0	265.1	1.95	MAX. STEADY-STATE POOL - DEEP SURFACE

NOTES
STEADY STATE POOL D/S STABILITY
ZERO FREEBOARD - D/S STABILITY

ER PROGRAM SLØPE



ANALYSIS METHOD	CASE NO	H	V	R	FS	NOTES
HAND BISHOP	0-12A	137.5	86.0	162.1	1.89	MAX STEADY-STATE POOL - SHALLOW SURFACE
	0-12B	230.0	190.0	305.1	1.73	MAX STEADY-STATE POOL - INTERMEDIATE SURFACE
	0-12C	235.0	180.0	307.6	1.76	MAX STEADY-STATE POOL - DEEP SURFACE
	0-12D	238.0	292.5	407.6	1.67	PMF ZERO FREEBOARD - INTERMEDIATE SURFACE



OVERLAY STABILITY AT STA 12+00

SCALE 1" = 20'

20 0 20 40

SCALE: 1" = 20'

40 0 40 80

SCALE: 1" = 40'

PROPOSED OVERLAY STABILITY SUMMARY AT STATIONS 7+00, 10+00 AND 12+00

NEW ENGLAND POWER COMPANY

HARRIMAN DAM

MAIN

GEOTECHNICAL ENGINEERS INC

DATE	MAY, 1982
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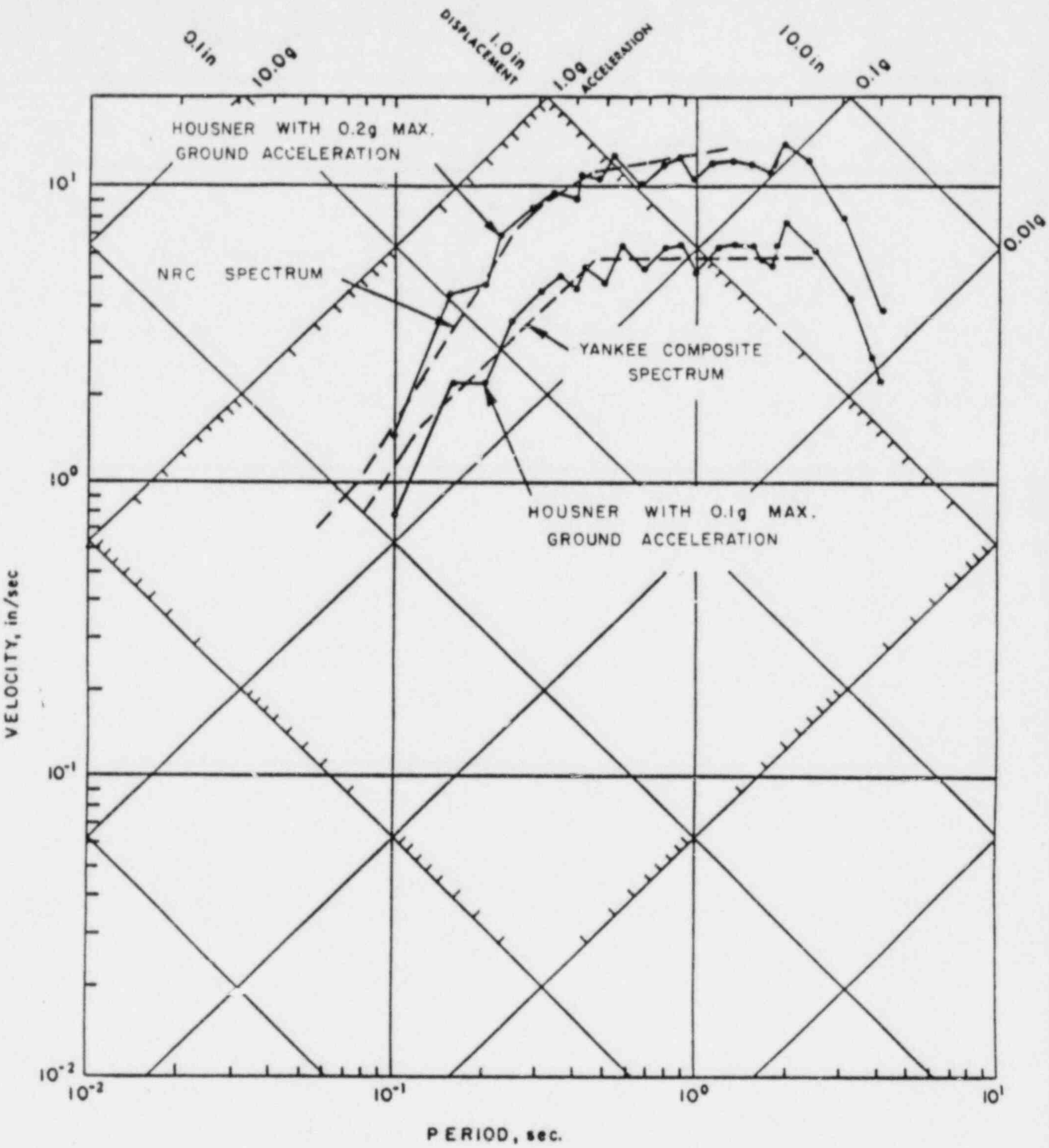
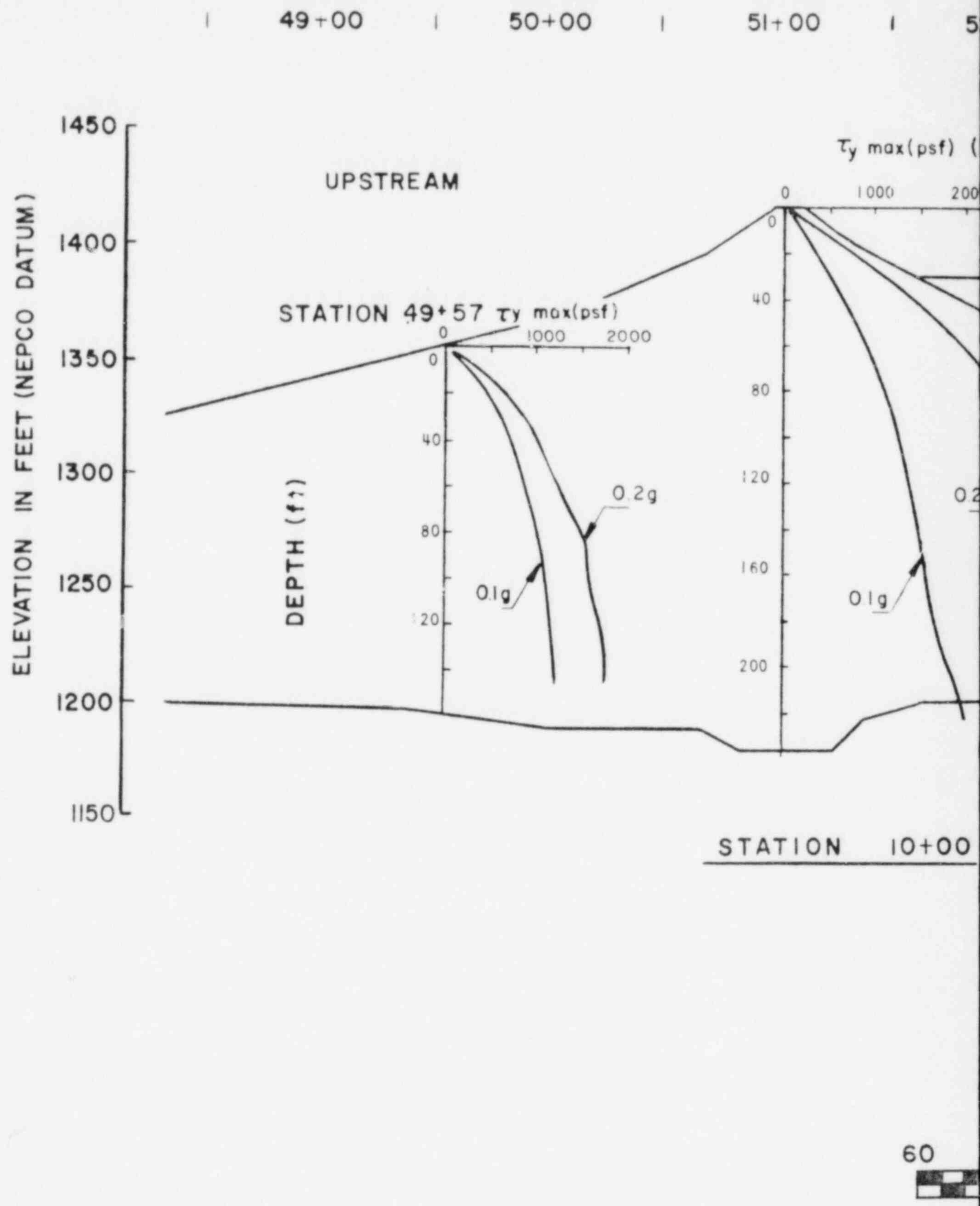
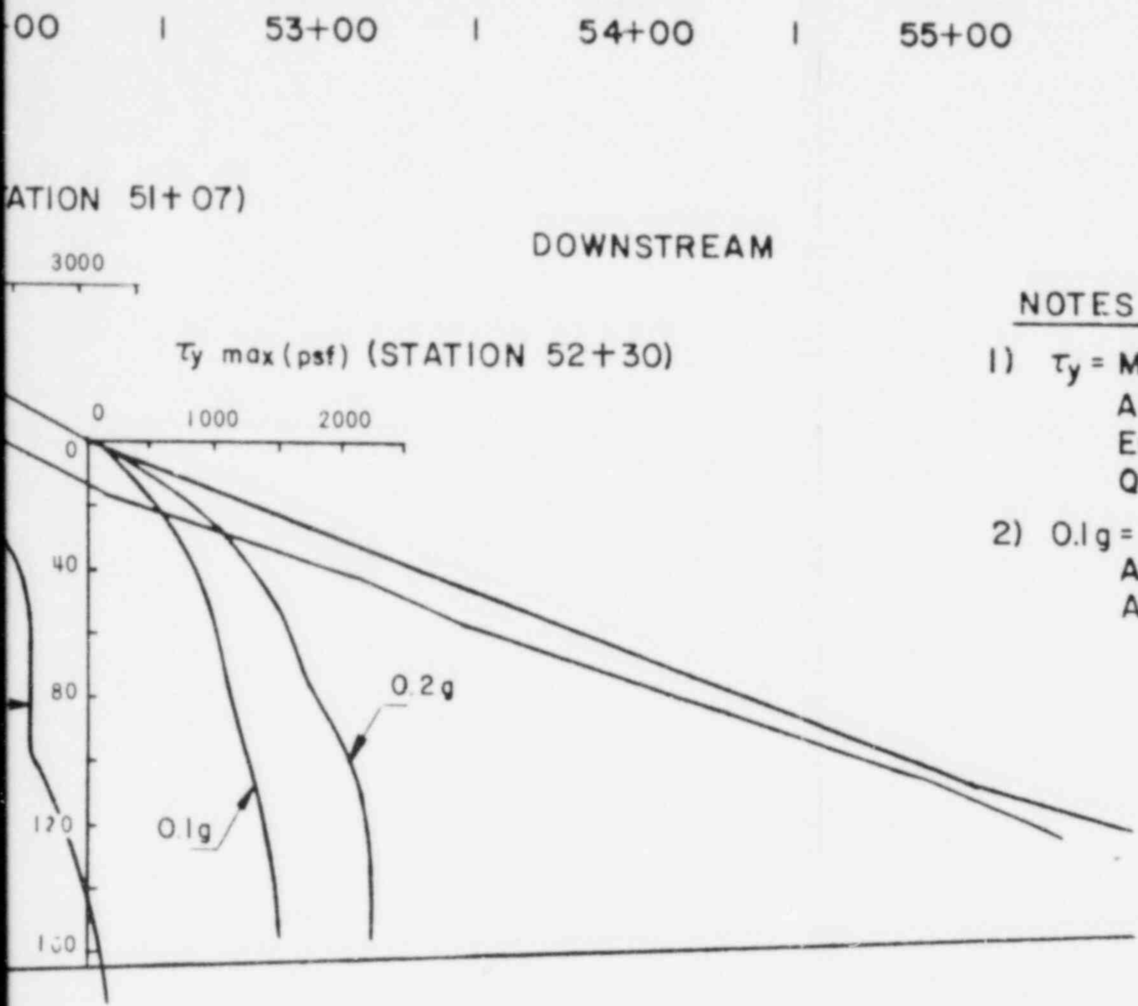


EARTHQUAKE RESPONSE SPECTRA	
NEW ENGLAND POWER COMPANY HARRIMAN DAM	
MAIN GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982
	PLATE 1270-76 21



49+00
50+00
51+00
1450
1400
1350
1300
1250
1200
1150
ELEVATION IN FEET (NEPCO DATUM)
UPSTREAM
STATION 49+57 τy max(psf)
0
1000
2000
0
40
80
120
DEPTH (ft)
0.1g
0.2g
τy max(psf)
0
1000
0
40
80
120
160
200
0.1g
STATION 10+00
60



NOTES

1) τ_y = MAX. SHEAR STRESS AFTER 5 CYCLES OF EQUIVALENT EARTHQUAKE LOADING.
2) 0.1g = PEAK EARTHQUAKE ACCELERATION INPUT AT BASE OF DAM.

0 60 120

Scale 1" = 60'

SEISMIC SHEAR STRESS VS. DEPTH AT STATION 10+00

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC.

DATE MAY, 1982

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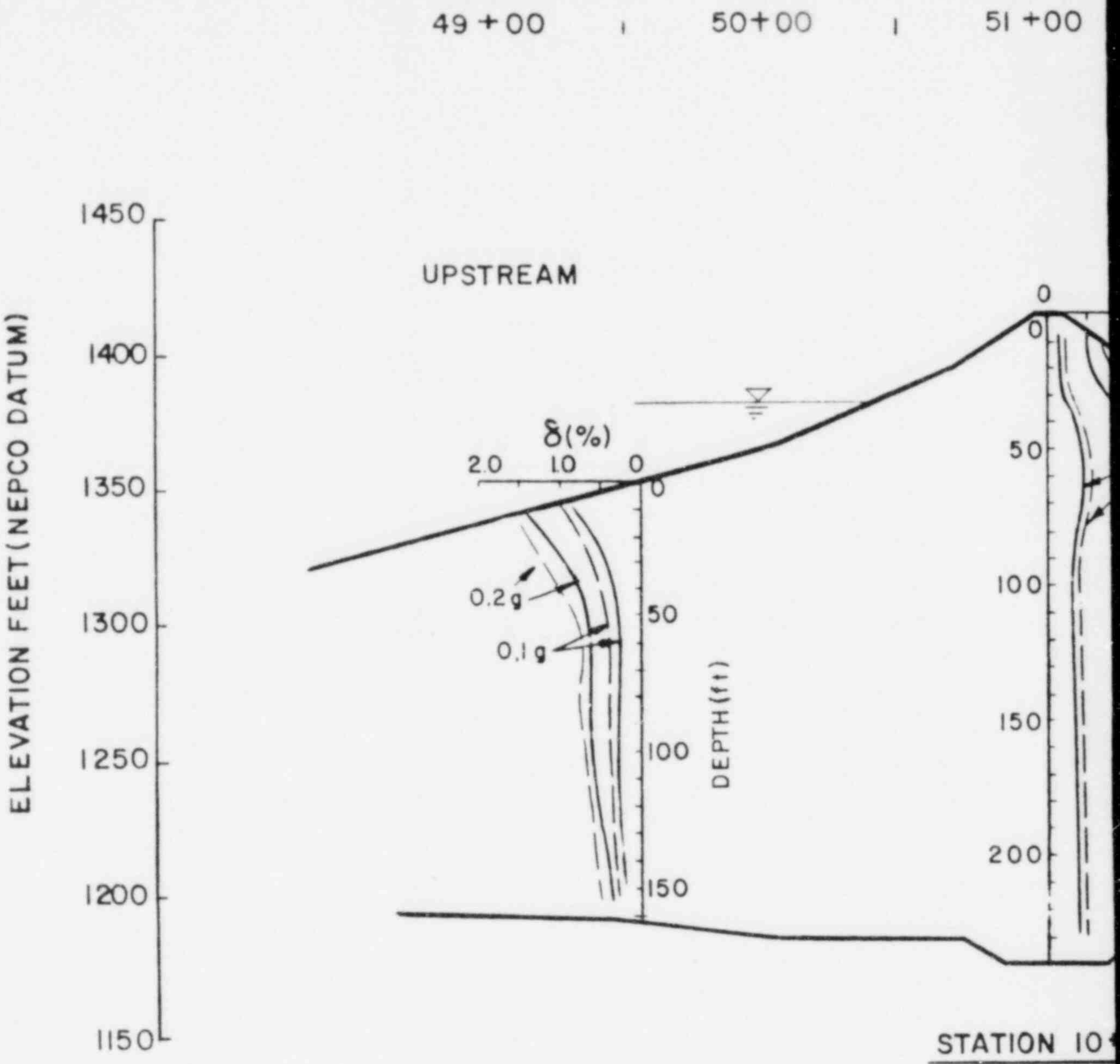
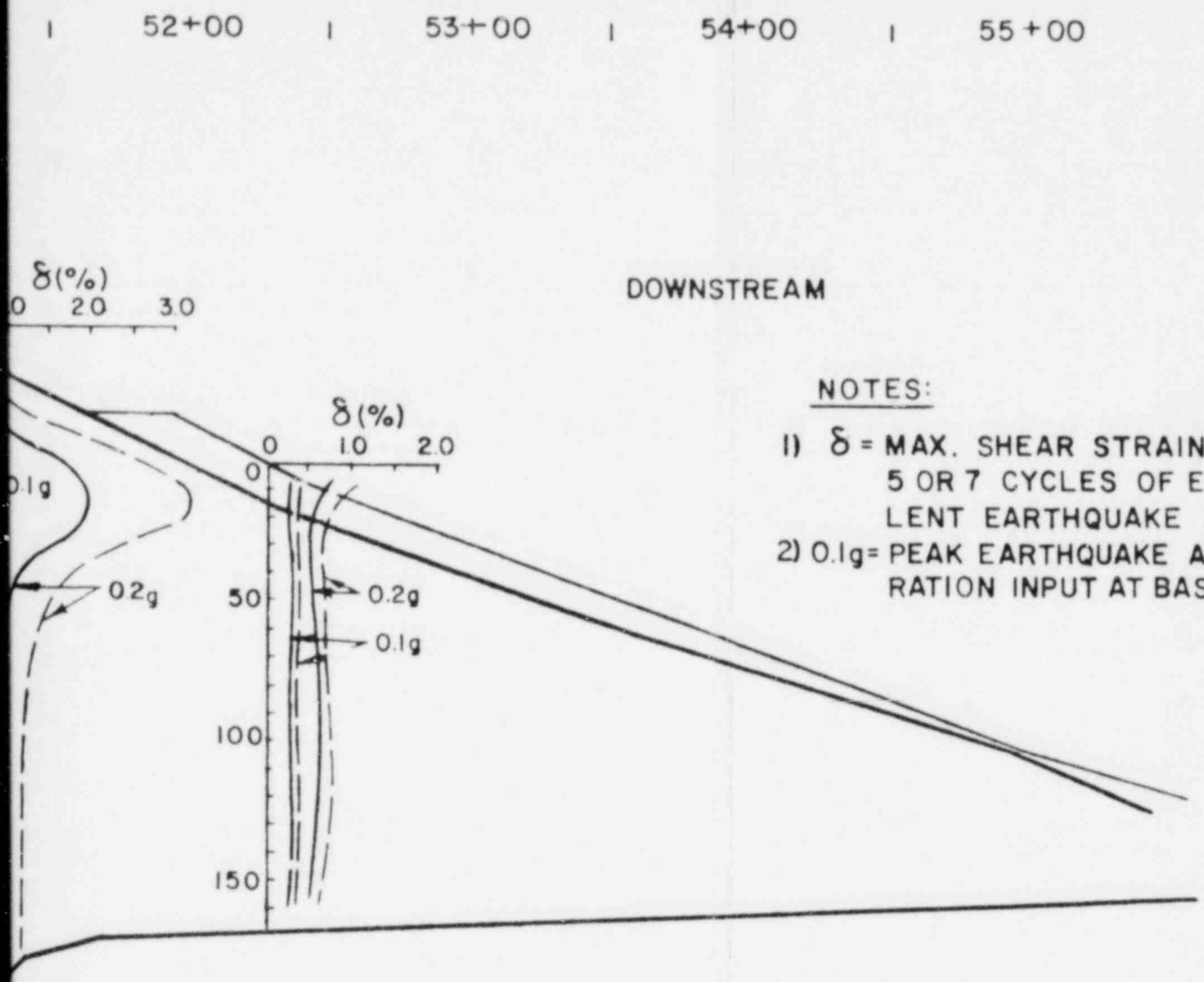


LEGEND

——— BEST ESTIMATE OF SHEAR STRAIN (5 CYCLES OF EARTHQUAKE LOA

– – – – CONSERVATIVE ESTIMATE OF SHEAR STRAIN (7 CYCLES OF EARTHQUA



NOTES:

1) δ = MAX. SHEAR STRAIN AFTER 5 OR 7 CYCLES OF EQUIVALENT EARTHQUAKE LOADING.
2) 0.1g = PEAK EARTHQUAKE ACCELERATION INPUT AT BASE OF DAM

00

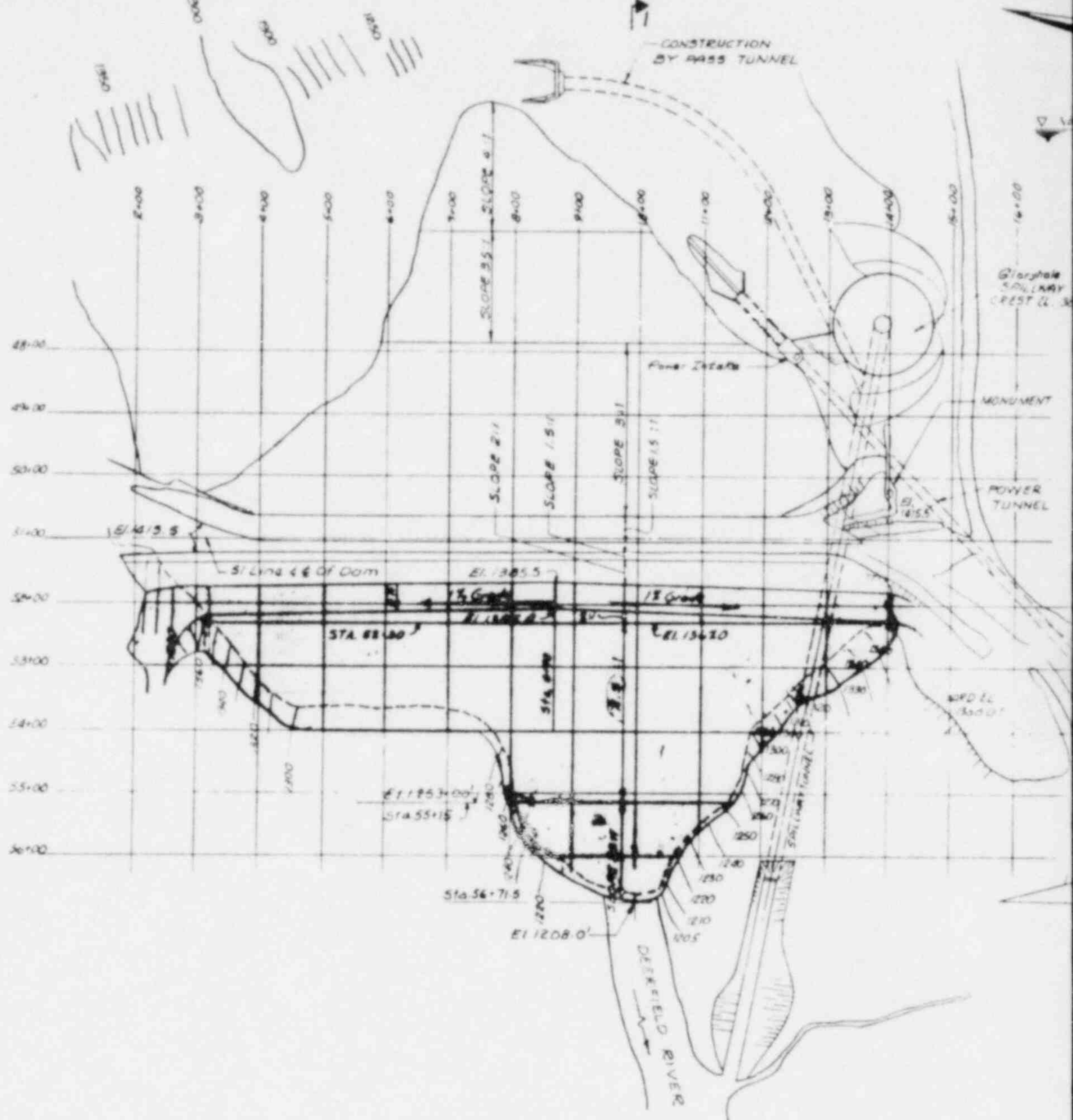
NG)

E LOADING)

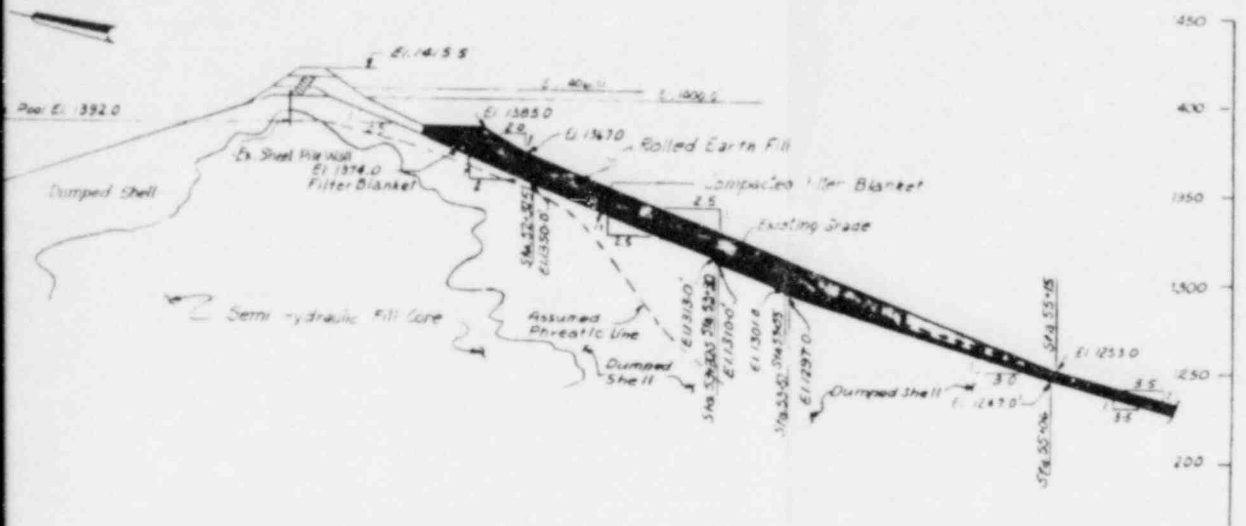
0 60 120

SCALE 1" = 60'

SEISMIC SHEAR STRAIN VS. DEPTH AT STATION 10+00	
NEW ENGLAND POWER COMPANY HARRIMAN DAM	
MAIN Φ GEOTECHNICAL ENGINEERS INC	DATE MAY, 1982 PLATE 1270-76 23

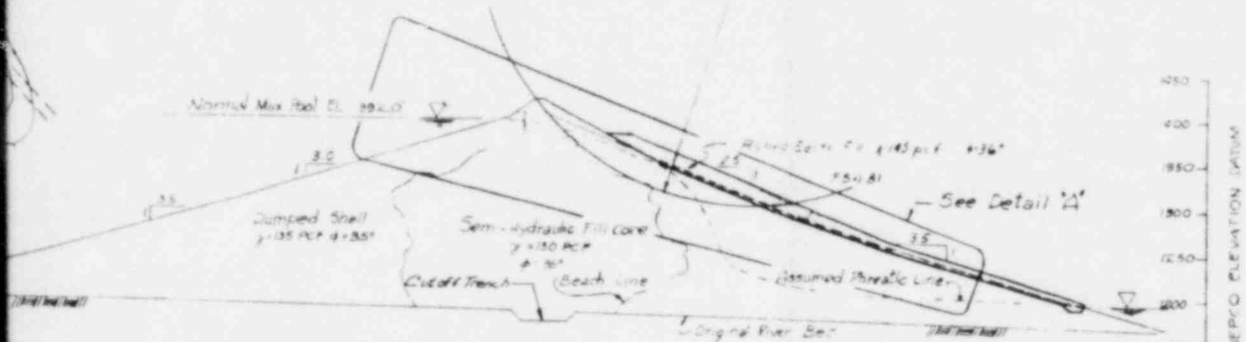


PLAN
Scale: 1"=100'



DETAIL-'A'

Station 0+00
Scale 1"=40'

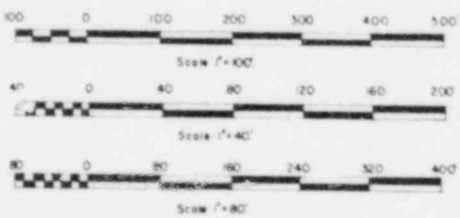


SECTION 1-1

Station 10+00
Scale 1"=80'

LEGEND

Proposed 1981 Downstream Overlay



DOWNSTREAM OVERLAY
PROPOSED EXHIBIT "L"

NEW ENGLAND POWER COMPANY
HARRIMAN DAM

MAIN
GEOTECHNICAL ENGINEERS INC

DATE MAY, 1982

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