Report on

SEISMIC STABILITY EVALUATION

HARRIMAN DAM

Submitted to

New England Power Company
Westborough, Massachusetts

Submitted by

Geotechnical Engineers Inc.
1017 Main Street
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Project 81858

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1. INTRODUCTION

1.1 Purpose and Scope of Report

The purpose of this report is to present the results of a seismic stability analysis of Harriman Dam performed by Geotechnical Engineers Inc. (GEI).

Preliminary results of this study were presented by Dr. Gonzalo Castro at a joint Federal Energy Regulatory Commission (FERC)/Nuclear Regulatory Commission (NRC) meeting held in Washington, D. C. on September 29, 1981. The information contained herein supplements and significantly expands the data contained in the handout distributed at that meeting, (GEI, 1981c).(1)

1.2 Sources of Data

The seismic stability analysis was based on dam geometry, cross sections, soil properties, and pore pressures determined from field and laboratory data presented in several reports which are available separately and referenced in Section 6., namely: Main (1979), GEI (1981a), GEI (1981b), and Main (1981). These reports contain details of the borings and the tests performed on undisturbed and remolded samples and information from construction records.

Earthquake acceleration spectra used for the seismic analysis were taken from a report by Yankee Atomic Electric Co. (1981) and a letter from the Nuclear Regulatory Commission (1981).

1.3 Technical Approach

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

1. Evaluate the factor of safety against a major flow slide or slope failure resulting from earthquake-induced liquefaction of the embankment soils.

2. Estimate the deformations of the core and shell during seismic loading using calculated seismic shear stresses and the results of cyclic triaxial ($\bar{CR}$) tests.

3. Compare Harriman Dam to another existing dam, Lower San Fernando Dam, which was subjected to an 0.55g earthquake and which experienced a major slide on the upstream slope.

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(1) See references in Section 6.

2. SUMMARY AND CONCLUSIONS

The analysis showed that a flow slide or major slope failure could not occur in Harriman Dam as a result of earthquake-induced liquefaction of the embankment soils. The minimum factor of safety against earthquake-induced liquefaction was 1.35 for a deep potential failure wedge on the downstream slope. This factor of safety was determined using steady-state shear strengths, which are the minimum undrained strengths of the soils. For stability analyses of this type, a minimum factor of safety of 1.1 is adequate.

No significant deformations of the dam will occur due to seismic loads from the Yankee composite earthquake spectrum with a peak ground acceleration of 0.1g. The Yankee composite earthquake spectrum with a peak ground acceleration of 0.1g was selected by GEI as the design earthquake, based on studies by Weston Geophysical Corp. and Yankee Atomic Electric Co. This spectrum has a 10^{-3} probability of being exceeded in any given year. Shear strains at three locations (crest, upstream slope, downstream slope) at the highest section of the dam were estimated from results of cyclic triaxial tests and shear stresses determined by a one-dimensional analysis using the computer program SHAKE. Five cycles of earthquake loading were used to determine best estimates of the shear strains and deformations in the dam. The estimated shear strains were equal to or less than 1% in all three sections, resulting in a best-estimate of crest settlement equal to 0.5 ft. A more conservative estimate of deformations was made for seven cycles of earthquake loading and indicated a crest settlement of 0.6 ft. Deformations of this magnitude would not affect the overall safety of the dam.

Harriman Dam is also expected to behave satisfactorily when subjected to the NRC recommended spectrum with a peak ground acceleration of 0.2g. The NRC spectrum recommended for the Yankee Rowe Nuclear plant is a very conservative earthquake with a 10^{-4} probability of being exceeded in any given year. Shear strains were computed for the same three sections noted above. The best-estimate of crest settlement for this earthquake is about 1.0 ft. The more conservative estimate of crest settlement (seven cycles of loading) is 1.2 ft. These deformations would not be expected to impair the overall integrity of the dam.

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

a. The freeboard in the dam is about 23 ft, even at maximum design reservoir operating level of El 1392.

b. Although transverse cracking of the dam is not expected as a result of the small estimated settlements, the widely graded dumped shell materials would be self-healing even if transverse cracking should occur.

Analytical approaches to evaluating the seismic stability and deformations of dams are subject to question until the analytical results can be compared with actual behavior. Therefore, Harriman Dam was compared with the Lower San Fernando Dam, which experienced a failure of the upstream slope due to an earthquake with 0.55g peak ground acceleration. First, it was determined that San Fernando Dam would have remained stable if the peak acceleration of the earthquake had been less than about 0.35g. Then, by comparing cyclic strengths of Harriman and San Fernando shell materials, we determined that if the San Fernando Dam had been constructed of Harriman Dam soils it could have withstood the San Fernando earthquake at 0.2g peak ground acceleration.

Since the analyses for Harriman Dam indicate that a flow slide cannot occur, that the deformations would be small in an 0.2g earthquake, since the freeboard is large, and the materials are "self-healing," and since the detailed comparison with a dam that failed during an 0.55g earthquake indicates it would not have failed at 0.2g if it was constructed of Harriman soils, we conclude that Harriman Dam would behave satisfactorily during an earthquake with a 0.2g peak ground acceleration.

3. LIQUEFACTION EVALUATION

3.1 General

The first step of 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 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 undrained steady-state shear strength. The slope of the soil mass after liquefaction is usually very flat because the undrained steady-state strength is very low.

The undrained steady-state shear strength is the minimum strength which can exist during undrained loading. The strength is reduced to this value only after large shear deformations.

The laboratory data for Harriman Dam 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 strength would be lower if the mass is converted to the undrained condition such as would occur during an earthquake. To evaluate whether flow slides could occur (i.e., whether the static driving stresses were greater than the steady-state strengths), stability analyses were performed.

3.2 Steady-State Shear Strengths

3.2.1 Dumped Shell

For the liquefaction evaluation, the dumped shell was divided into two zones, based on the corrected split-spoon blowcounts (see GEI, 1981c). Below a depth of about 60 ft, the corrected blowcounts increased noticeably.

Steady-state strength data for the shallow dumped shell were obtained from the five consolidated-undrained triaxial compression ($\bar{R}$) tests on undisturbed specimens which reached approximate steady-state conditions. Steady-state conditions are defined as a constant rate of deformation at constant volume with constant shear stress and pore pressure (Poulos, 1981). Since the steady-state strength is sensitive to void ratio changes, the measured strengths from the $\bar{R}$ tests were corrected for the effect of void ratio changes during consolidation in the laboratory to estimate the in-situ steady-state strengths.

The shape of the steady-state line for the dumped shell was determined from undrained triaxial ($\bar{R}$) tests on compacted specimens, as shown in Fig. 1. The steady-state line from the compacted specimens and the uncorrected and corrected undrained steady-state strength from the five undisturbed shell specimens are shown in Fig. 2. The average in-situ undrained steady-state shear strength for the dumped shell is about 2000 psf after correction for consolidation. The in-situ undrained steady-state shear strengths are, on average, about 80% of the drained steady-state strengths.

For the dumped shell below a depth of 60 ft, the higher corrected blowcounts indicate that the soils are dilative, which means that the undrained strength would be greater than the drained strengths below a depth of 60 ft. However, the negative pore pressures required to mobilize undrained strength greater than drained strength should not be relied upon for stability analyses. Therefore, an undrained strength equal to the drained strength was used for the deeper zones of the dumped shell. Drained steady-state strengths were calculated using the steady-state friction angle, $\phi_s = 30°$, determined from the $\bar{R}$ tests on compacted specimens (GEI, 1981a).

3.2.2 Hydraulic and Washed Core

In-situ undrained steady-state shear strengths for the hydraulic core and washed zone of core were determined using the same approach as described above for the shallow dumped shell. Steady-state strengths were measured on three undisturbed specimens of washed core and two undisturbed specimens of hydraulic core. Steady-state lines were not measured for either core material at Harriman Dam. However, a band of steady-state lines was determined for the hydraulic core at Sherman Dam (GEI, 1982). Since Sherman Dam was constructed using procedures and materials similar to Harriman, it was assumed that the steady-state lines of the Harriman core materials were parallel to the lines from Sherman Dam.

The steady-state lines and the uncorrected and corrected undrained steady-state strengths for the Harriman hydraulic core and washed core are shown in Fig. 3. The average in-situ steady-state strength for the three specimens of washed core was 2000 psf. For the hydraulic core, the steady-state strength was taken as the lower of the two test results, 3600 psf.

3.3 Stability Analysis

To compute the factor of safety against a slope failure or flow slide due to earthquake-induced liquefaction, static stability analyses were performed using the undrained steady-state shear strengths described above. These strengths are the minimum or "residual" strengths rather than the peak strengths. The analyses were performed for the downstream slope at the highest section of the dam, Sta 10+00, using the sliding-wedge method. A series of trial wedges intersecting the crest and upstream slope were considered to identify the critical failure surface.

The minimum factor of safety was 1.35 for the critical downstream failure surface shown on Fig. 4. This surface passes primarily through the washed zone of core and then along the base of the dam.

An undrained stability analysis of a trial wedge on the upstream slope indicated a significantly higher factor of safety for the upstream slope than for the downstream slope. This would be expected for full reservoir conditions. Therefore, further analyses were not performed for the upstream slope.

For a stability analysis of this type, a minimum factor of safety of 1.1 is adequate. As dissipation of excess porewater pressures occurs after the earthquake, the drained shear conditions existing prior to the earthquake will be re-established and the factors of safety for static load conditions will become applicable again.

Based on these analyses, a major slope failure or flow slide cannot occur at Harriman Dam as a result of earthquake-induced liquefaction.

4. DEFORMATIONS UNDER SEISMIC LOADING

4.1 General

Deformations of the core and the dumped shell due to the design earthquake were estimated based on seismic shear stresses calculated by the SHAKE computer model and cyclic triaxial stress-strain data from tests on undisturbed specimens.

4.2 Earthquake Spectra

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 comparison purposes, analyses were also performed using the very 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.

The Yankee composite and NRC recommended spectra are shown on Fig. 5.

For the seismic analyses an artificial earthquake, referred to as the Housner earthquake, was used to model the Yankee composite and the NRC recommended spectra. As shown in Fig. 5, the Housner earthquake with a peak ground acceleration of 0.1g closely follows the Yankee composite while the Housner earthquake with a peak ground acceleration of 0.2g closely follows the NRC recommended spectrum.

4.3 Earthquake Shear Stresses

The computer program SHAKE (Schnabel et al., 1972) was used to evaluate the accelerations and seismic shear stresses within the dam when subjected to an earthquake. Dynamic soil properties for the dumped shell and core were based on the results of resonant column and small strain cyclic triaxial tests on undisturbed specimens reported in GEI (1981b). The SHAKE analyses were performed at Sta 10+00, the highest section of the dam. One analysis was made at the crest (transverse Sta 51+07), one in the middle of the upstream slope (transverse Sta 49+57), and one in the middle of the downstream slope (transverse Sta 52+30).

Plots of acceleration and seismic shear stresses for these three locations vs depth are shown in Figs. 6 and 7, respectively. The natural periods for each soil column used in the SHAKE analysis are shown in Fig. 6.

4.4 Seismic Deformations

The magnitude of the strains due to seismic loading were estimated from cyclic triaxial ($\overline{CR}$) test results and the seismic shear stresses computed by the SHAKE analysis.

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} 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) shown on Fig. 8, it can be seen that 4.5 cycles of loading correspond to the mean value for an M = 6.0 earthquake. Hence, analysis with five cycles of earthquake loading gives a best-estimate of strains and deformations. For comparison, deformations were also calculated for the more conservative case of seven cycles of uniform loading.

Cyclic test data from anisotropically consolidated undisturbed specimens with K_c = 2.0 were used to determine cyclic strains, since the in-situ static stress ratio calculated from circular arc stability analyses were about 2.0. The cyclic test results were plotted in terms of $\tau_{fy}/\bar{\sigma}_{fc}$ vs shear strain after five and seven cycles, where τ_{fy} is the earthquake-induced cyclic shear stress on the failure plane and $\bar{\sigma}_{fc}$ is the effective normal consolidation stress on the failure plane. The cyclic stress-strain curves for five cycles of loading for the dumped shell, based on eight $C\bar{R}$ tests on undisturbed specimens, are plotted in Fig. 9. Two tests on the hydraulic core indicated that the core had slightly higher cyclic resistance than the dumped shell. Therefore, for these analyses the curves for the dumped shell were also used for the hydraulic core. Two tests indicated that the washed core material had about 75% of the cyclic resistance of the dumped shell. Therefore, curves for the washed core were constructed parallel to the dumped shell curves, but at 75% of the cyclic stress level.

To estimate in-situ cyclic resistance, the laboratory $C\bar{R}$ test data were corrected to account for the decrease in void ratio between the sample when removed from the ground and the specimen as tested. Correction was also made for the gain in cyclic resistance with time experienced by the in-situ soils during the 60 years since the dam was constructed, which was lost during the sampling and reconsolidaton process, using the relationship for time effects shown in Seed (1976). The combined effect of these two corrections increases by about 10 to 15% the cyclic stress ratio $\tau_{fy}/\bar{\sigma}_{fc}$ causing any given level of shear strain in five or seven cycles; i.e., the cyclic shear resistance in situ is about 10 to 15% higher than indicated by the $\overline{CR}$ test data. Most of the $\overline{CR}$ tests were performed at a consolidation pressure of $\bar{\sigma}_{3c}$ = 2750 psf with two tests at $\bar{\sigma}_{3c}$ = 1250 psf. Extrapolation of the CR test results to consolidation stresses higher than used for the laboratory testing was based on relationships developed from data in Lee and Seed (1966).

The shear strain in the dam was computed from the normalized earthquake shear stress in the dam, $\tau_{avg}/\bar{\sigma}_v$. For example, in the dumped shell on the downstream slope at a depth of 57 ft, the cyclic shear stress ratio is $\tau_{avg}/\bar{\sigma}_v = 0.14$ for the 0.2g earthquake and the minor effective principal stress is about $\bar{\sigma}_{3c}$ = 5000 psf. The resulting cyclic shear strain, from Fig. 9, is 0.7% for five cycles of loading. Profiles of shear strain at the three locations analyzed are shown in Fig. 10.

Approximate crest settlements resulting from these shear strains were estimated using two methods. First, the horizontal strain profiles at the upstream and downstream sections were assumed to represent the outward movement of a volume of soil. This volume of outward movement was then assumed to cause an equal volume of settlement of the crest and slopes between these two profiles (i.e., between Sta 49+57 and 52+30). The resulting estimated crest settlements are:

	Best-Estimate (5 cycles loading)	Conservative Estimate (7 cycles loading)
0.1g earthquake	0.5 ft	0.6 ft
0.2g earthquake	1.0 ft	1.2 ft

In the second method, the strains for the centerline profile were assumed to represent maximum shear strains inclined at an angle of ± 45° from the vertical. (Use of any other orientation would result in smaller calculated crest settlements.) For an undrained plane strain condition with Poisson's ratio, ν = 0.5, the resulting crest settlements were estimated to be:

	Best-Estimate (5 cycles loading)	Conservative Estimate (7 cycles loading)
0.1 rthquake	0.4 ft	0.5 ft
0.2g earthquake	1.2 ft	1.7 ft

At 0.1g, the two methods give very similar estimates of crest settlement. For the 0.2g earthquake, the crest settlements are exaggerated somewhat in the second method due to the larger strains calculated for the washed zone of core. In the field, the strains in the washed core would be significantly reduced by the restraint of the surrounding shell. Therefore, the crest settlements from the first method are considerd more reasonable values for evaluating the seismic performance of the dam at 0.2g.

The magnitude of estimated horizontal deformations and crest settlements indicate that the deformations from either the 0.1g design earthquake or the very conservative 0.2g earthquake will not be sufficient to affect the integrity of the dam. Therefore, it is concluded that 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:

a. 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.

b. The earthquake spectra used for this analysis had a much higher energy content for the range of periods equal to the natural period of the dam, i.e., T = 0.8 to 1.6 sec than shown by records obtained at short epicentral distances for magnitude 6.0 earthquakes in California and in other seismically active areas of the world.

c. It is our experience that the strains developed in laboratory $C\bar{R}$ tests are exaggerated relative to field behavior due to cumulative test errors and to differences between stress paths in situ and in the laboratory.

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

a. The freeboard in the dam is about 23 ft, even at maximum design reservoir operating level of El 1392.

b. Although transverse cracking of the dam is not expected as a result of the small estimated settlements, the widely graded dumped shell materials would be self-healing even if transverse cracking should occur.

5. COMPARISON WITH 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, the 140-ft-high Lower San Fernando Dam, which has been subjected to a major earthquake. The San Fernando Dam experienced a massive upstream slope failure as a result of an 0.55g earthquake and has been extensively tested and analyzed. This comparison is based on the results of the seismic analyses performed by Seed et al. (1973).

The comparison was performed on a profile through the critical zone in the dam where failure apparently started after two major cycles of earthquake stress. The critical zone was just upstream from the core, in the shell composed of hydraulic fill sand ranging in gradation from silty fine and medium sand to clean widely graded sand. SHAKE analyses were performed using soil properties taken from Seed et al. (1973) and the Taft earthquake record scaled to various input accelerations. Shear stresses from the SHAKE analyses, matched to the value reported in Seed et al. (1973) for 0.55g, were used to generate a curve of average cyclic shear stress in the critical zone vs earthquake acceleration for the full duration of the earthquake (four major cycles of loading), as shown in Fig. 11. It can be seen that the average earthquake shear stress in the failure zone was slightly greater than the cyclic shear stress causing failure in two cycles in the CR tests.

Since the San Fernando Dam actually failed, the first step in the stability evaluation was to determine at what smaller earthquake San Fernando Dam would have remained stable. The available cyclic shear resistance at four cycles determined from cyclic triaxial tests performed on the San Fernando hydraulic fill was compared to the curve of shear stress vs peak acceleration, as shown in Fig. 11. It was concluded that the San Fernando Dam could have sustained a full duration (four cycles) Taft earthquake with a peak ground acceleration of 0.35g or less. A crest settlement of about 3 ft was calculated for the 0.35g earthquake, using the first method described in Subsection 4.4 above and the San Fernando cyclic triaxial test data. This estimate confirms that San Fernando Dam would have performed suitably for an earthquake up to 0.35g.

The cyclic shear resistance from tests on San Fernando hydraulic fill sand were then compared to the cyclic resistance of the Harriman dumped shell and hydraulic core. The Harriman soils had approximately 70% of the resistance available at San

Fernando. Entering Fig. 11 with the Harriman cyclic strength, it was determined that the corresponding peak acceleration would be 0.2g.

Thus, if San Fernando Dam had been composed of the Harriman dumped shell material, it could have sustained the San Fernando earthquake at 0.2g peak acceleration. This result is consistent with the results of the seismic analyses reported in Section 4.4 above.

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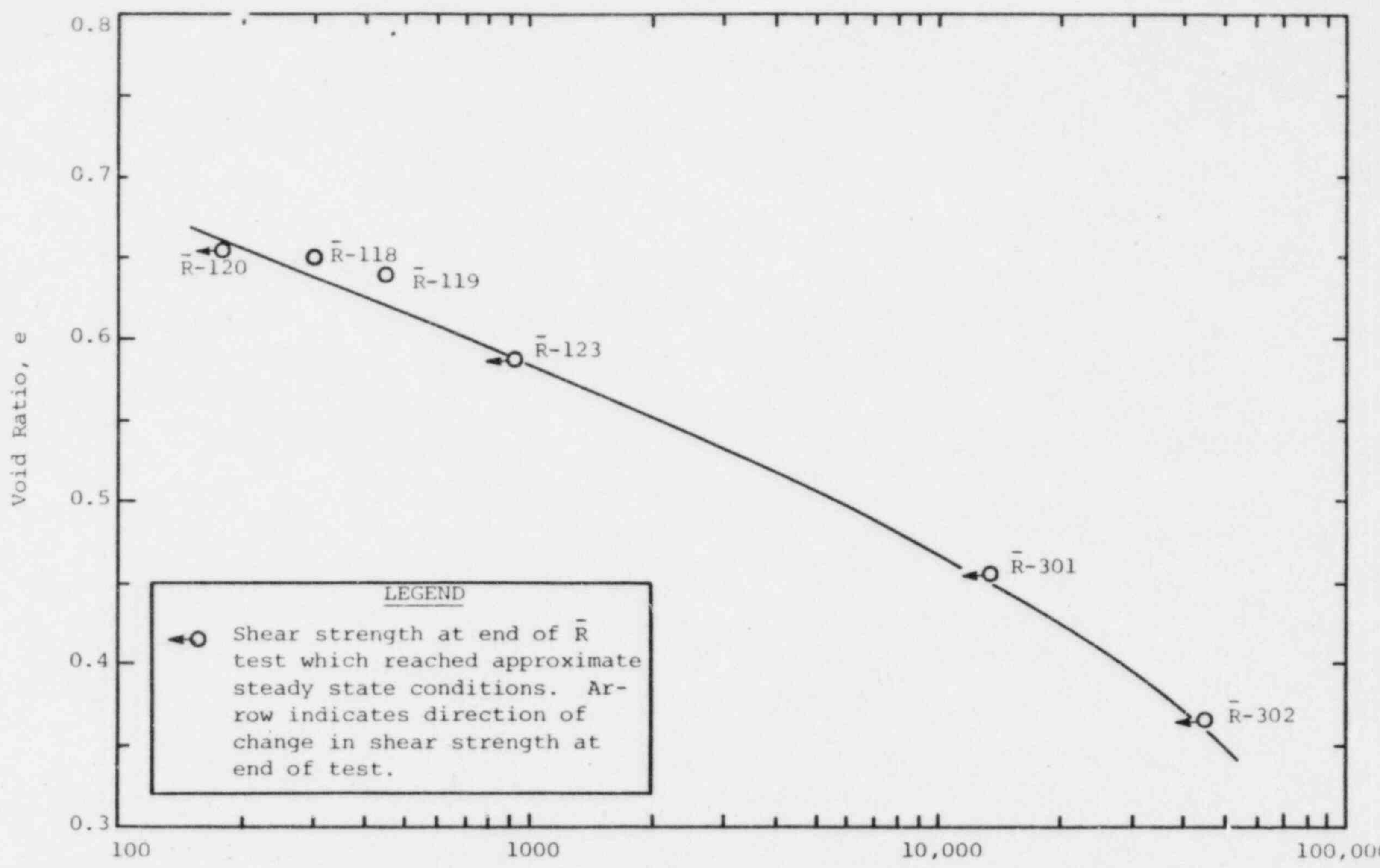
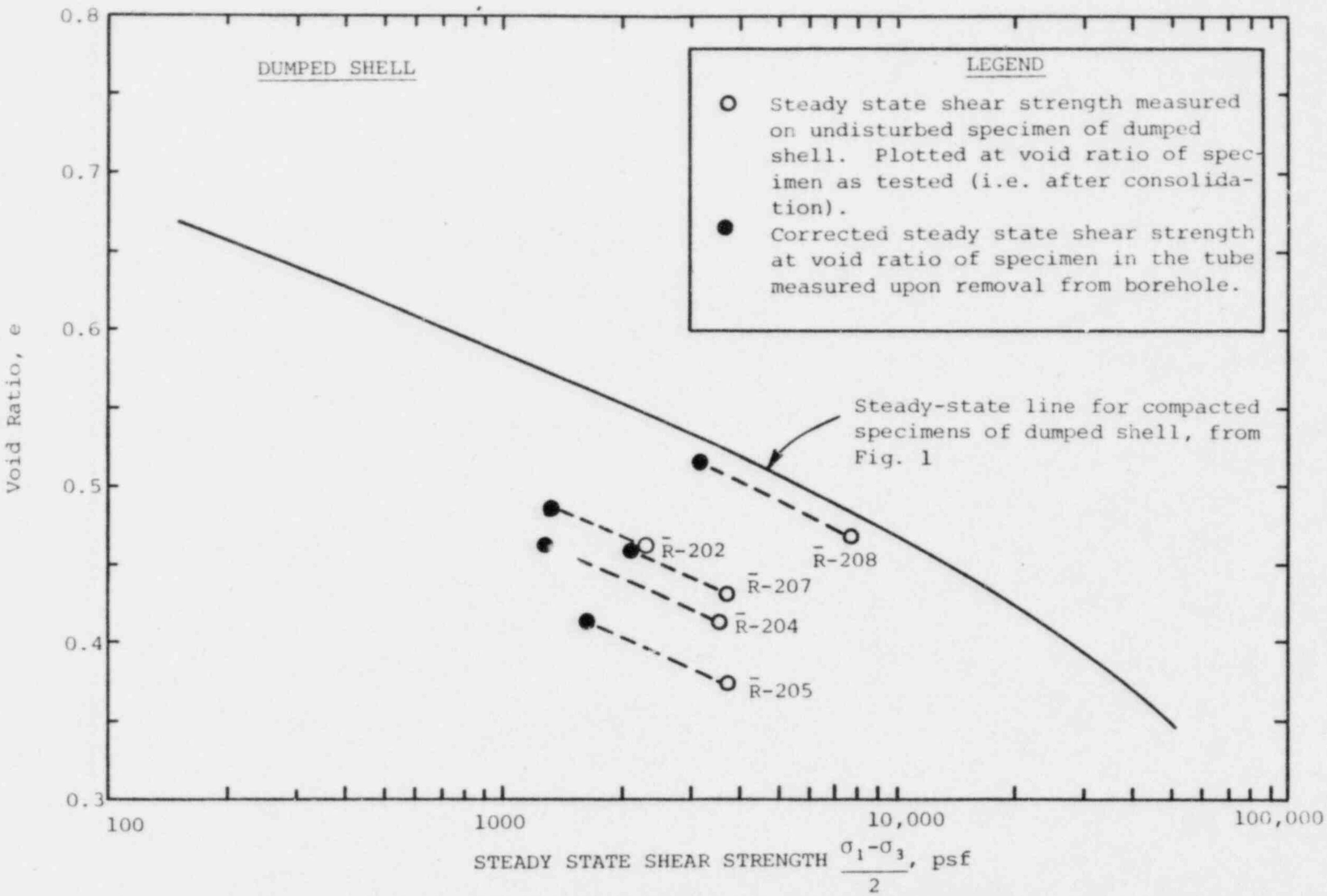
FIGURES

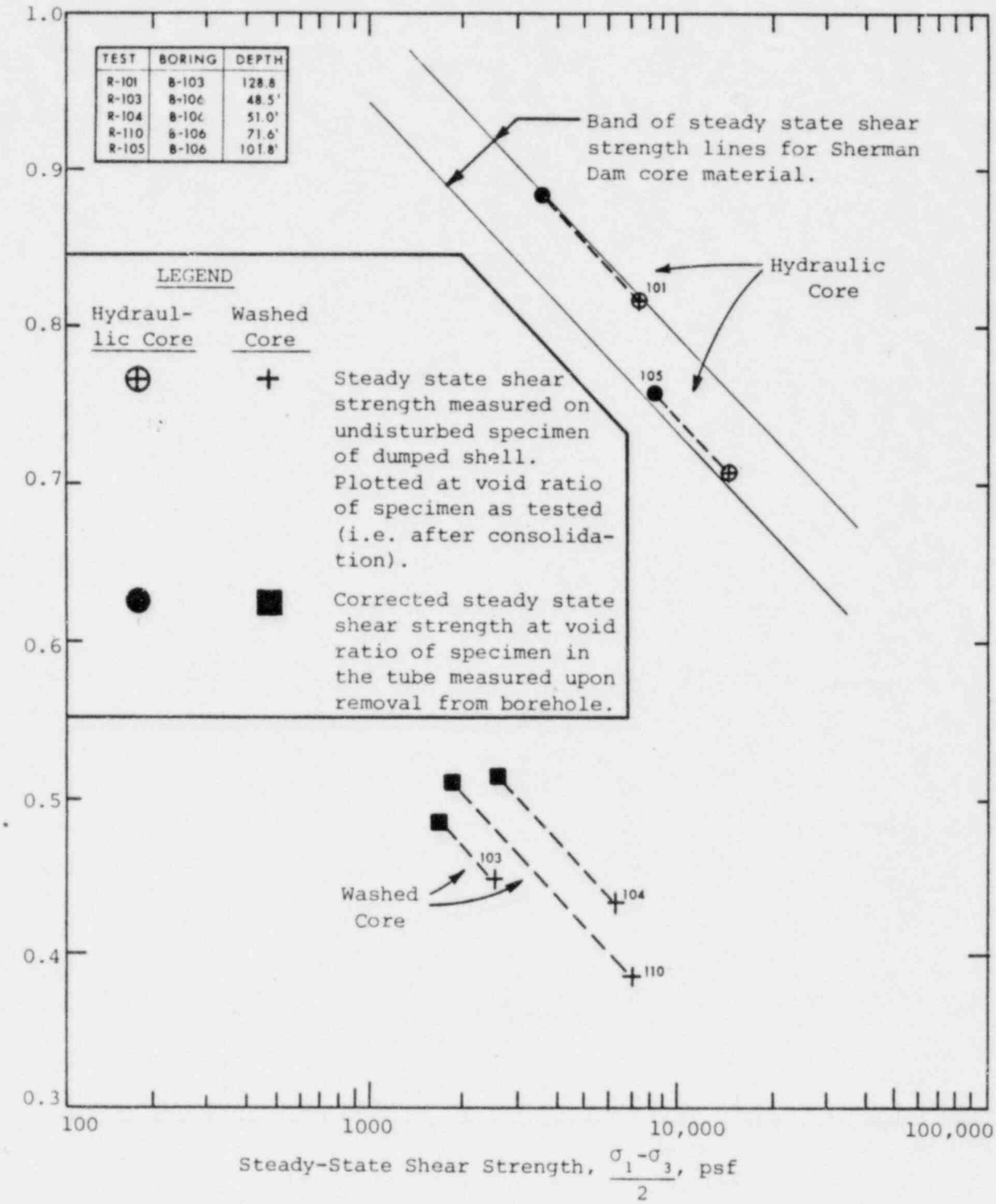


NOTE: All tests performed on compacted specimens of dumped shell from TP-102, samples adjacent to FD-105 and FD-106.

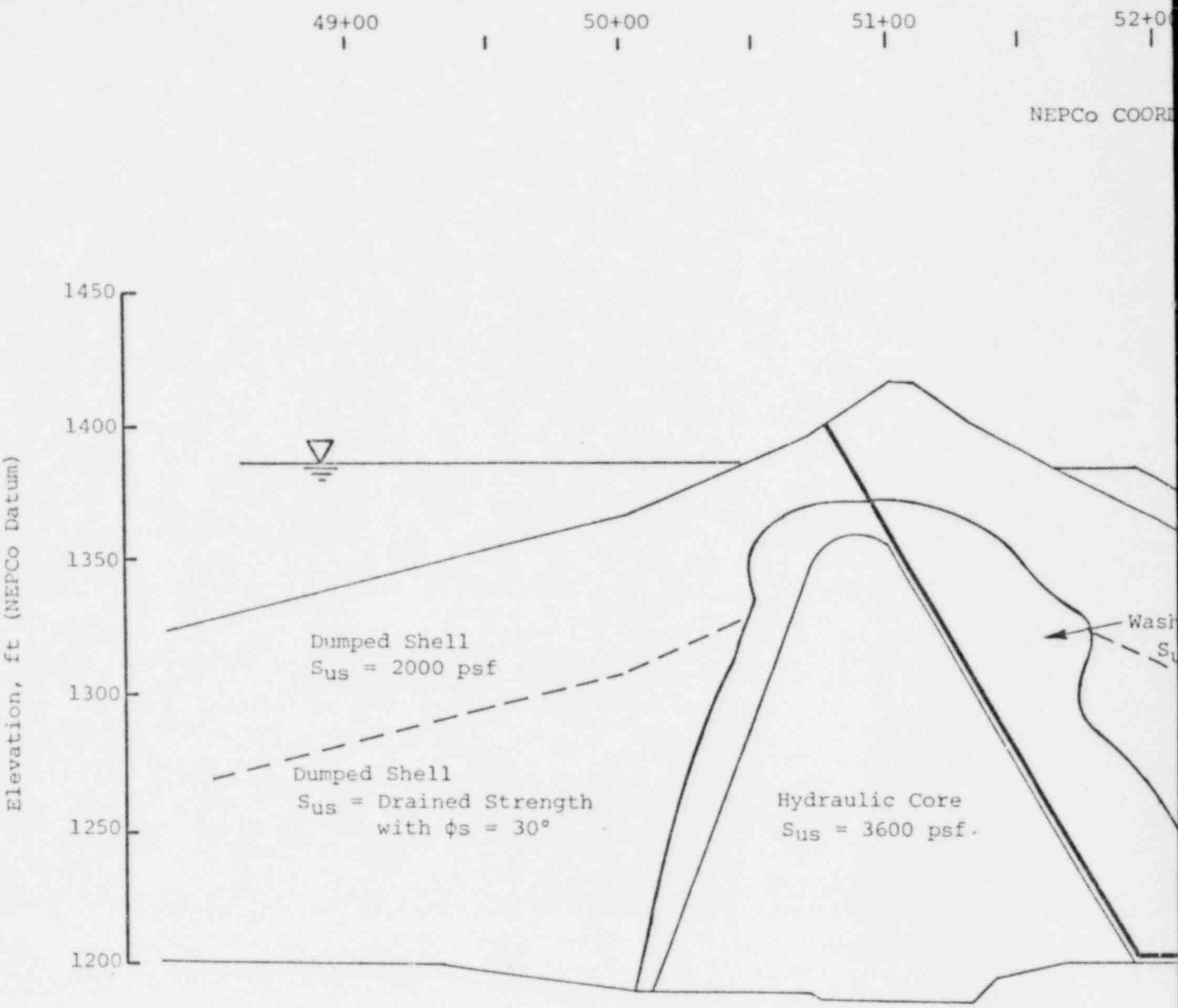
New England Power Company Westborough, Massachusetts	Harriman Dam	STEADY STATE LINE FROM COMPACTED SPECIMENS OF DUMPED SHELL
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	June 15, 1982 Fig. 1



New England Power Company Westborough, Massachusetts	Harriman Dam	STEADY STATE SHEAR STRENGTH FOR DUMPED SHELL
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	June 7, 1982 Fig. 2



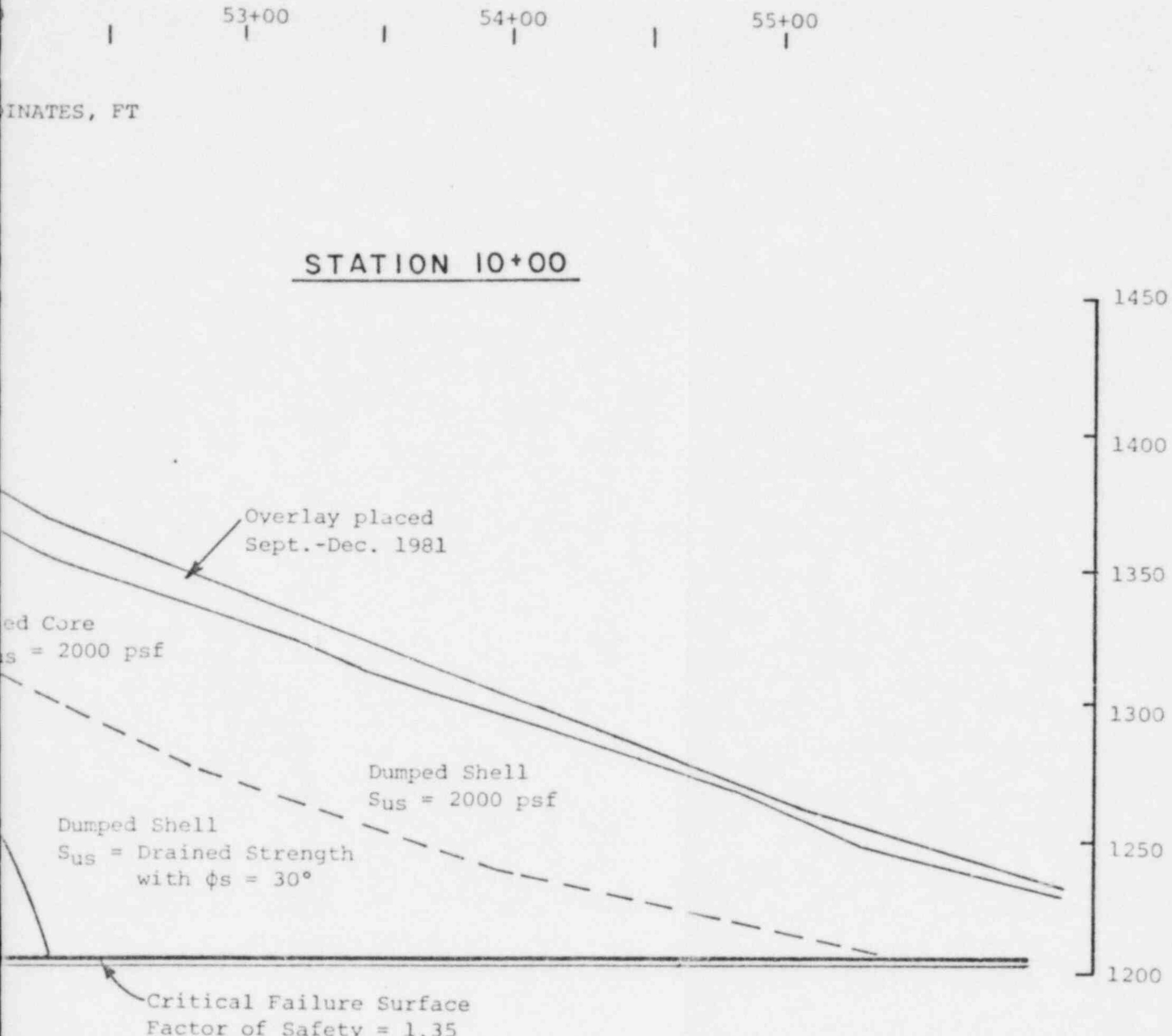
New England Power Company Westborough, Massachusetts	Harriman Dam	STEADY STATE SHEAR STRENGTHS FOR CORE MATERIALS
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	June 15, 1982 Fig. 3



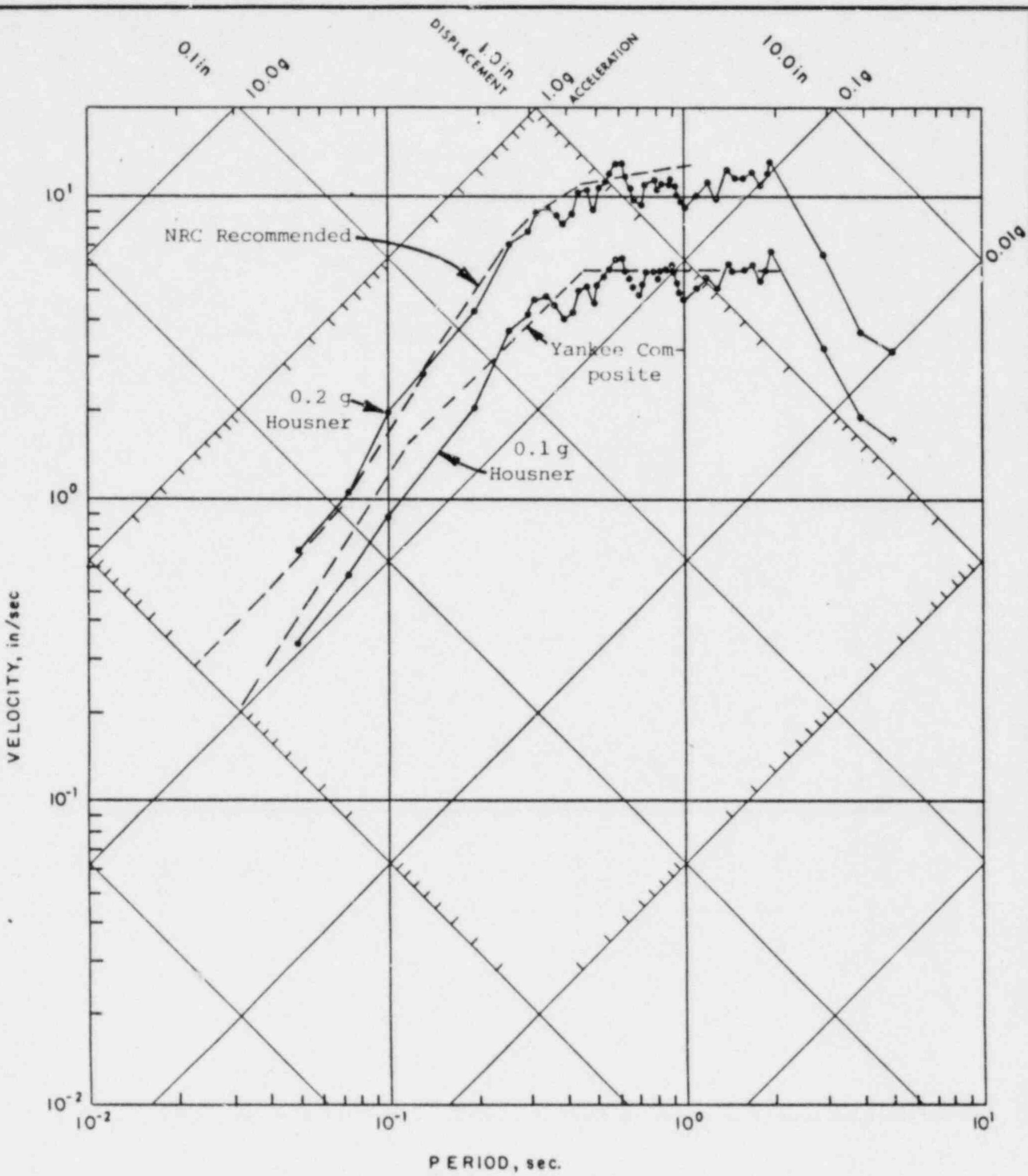
NOTES:

1) See text for discussion of undrained steady-state shear strengths used for each embankment soil.

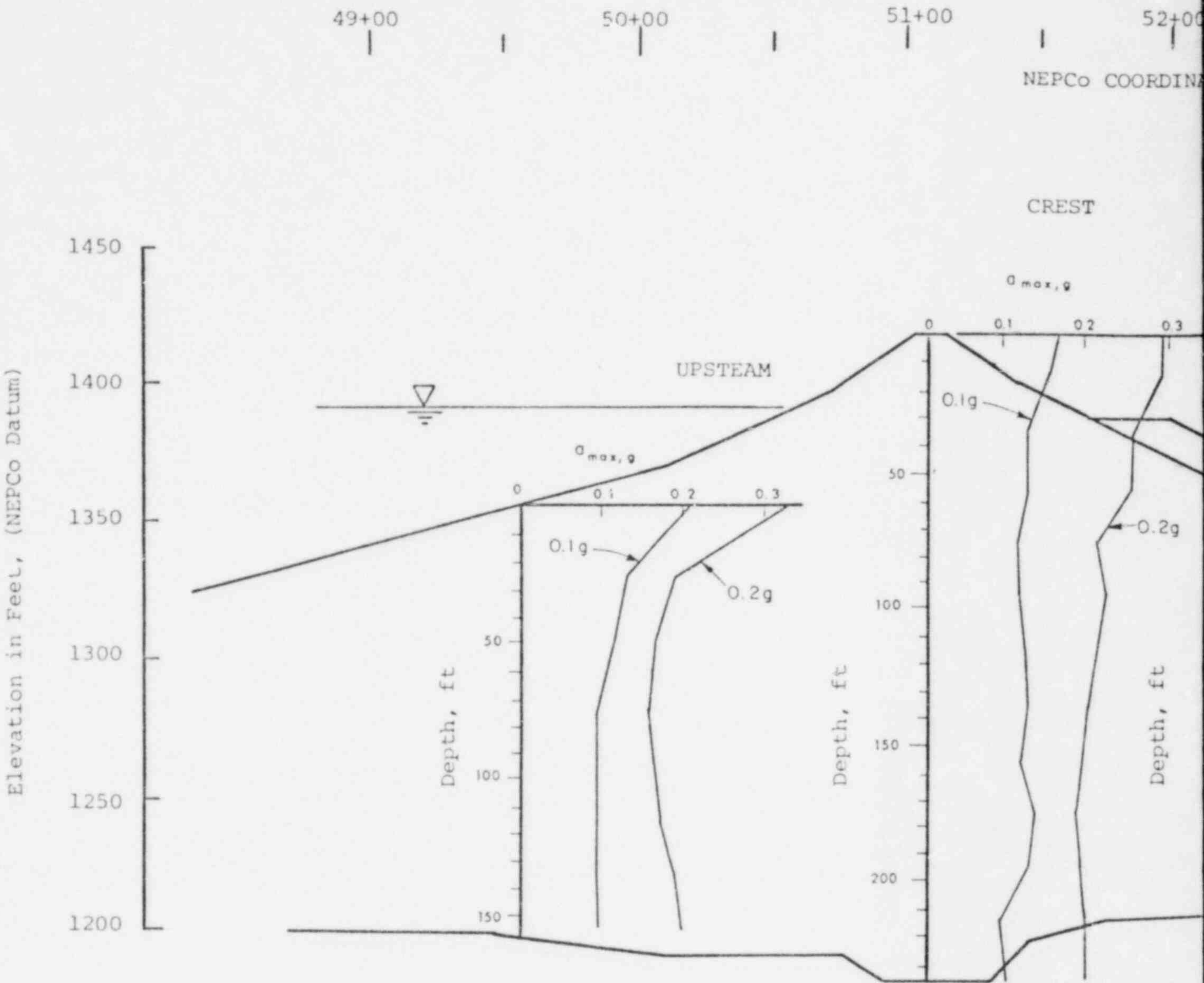
2) Factors of Safety for potential upstream slope failure surfaces are significantly higher than for downstream slope.



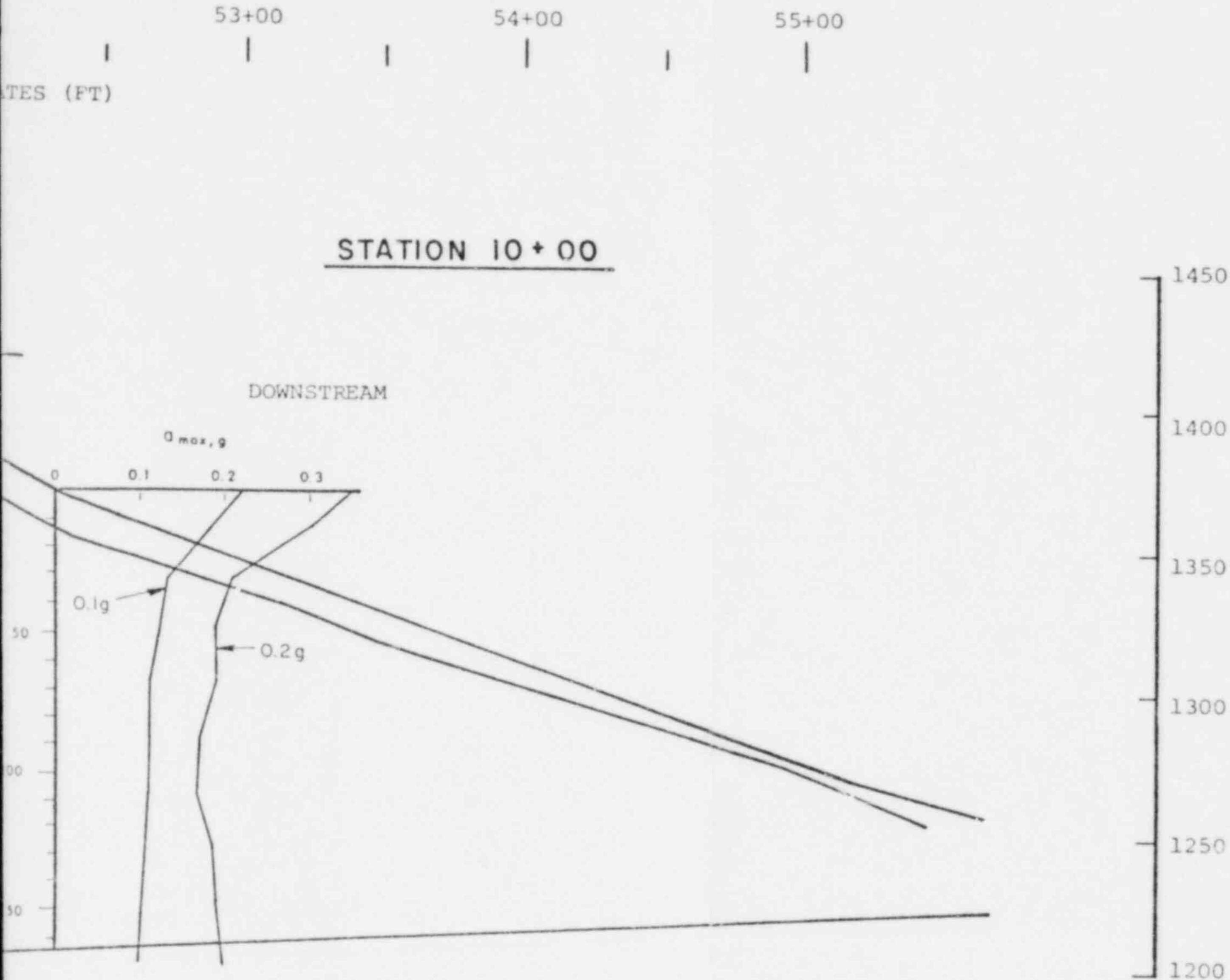
New England Power Company Westborough, Massachusetts	Harriman Dam	SEISMIC STABILITY ANALYSIS DOWNSTREAM SLOPE
	Project 81858	June 7, 1982 Fig. 4



New England Power Company Westborough, Massachusetts	Harriman Dam	EARTHQUAKE RESPONSE SPECTRA
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	Dec. 15, 1981 Fig. 5

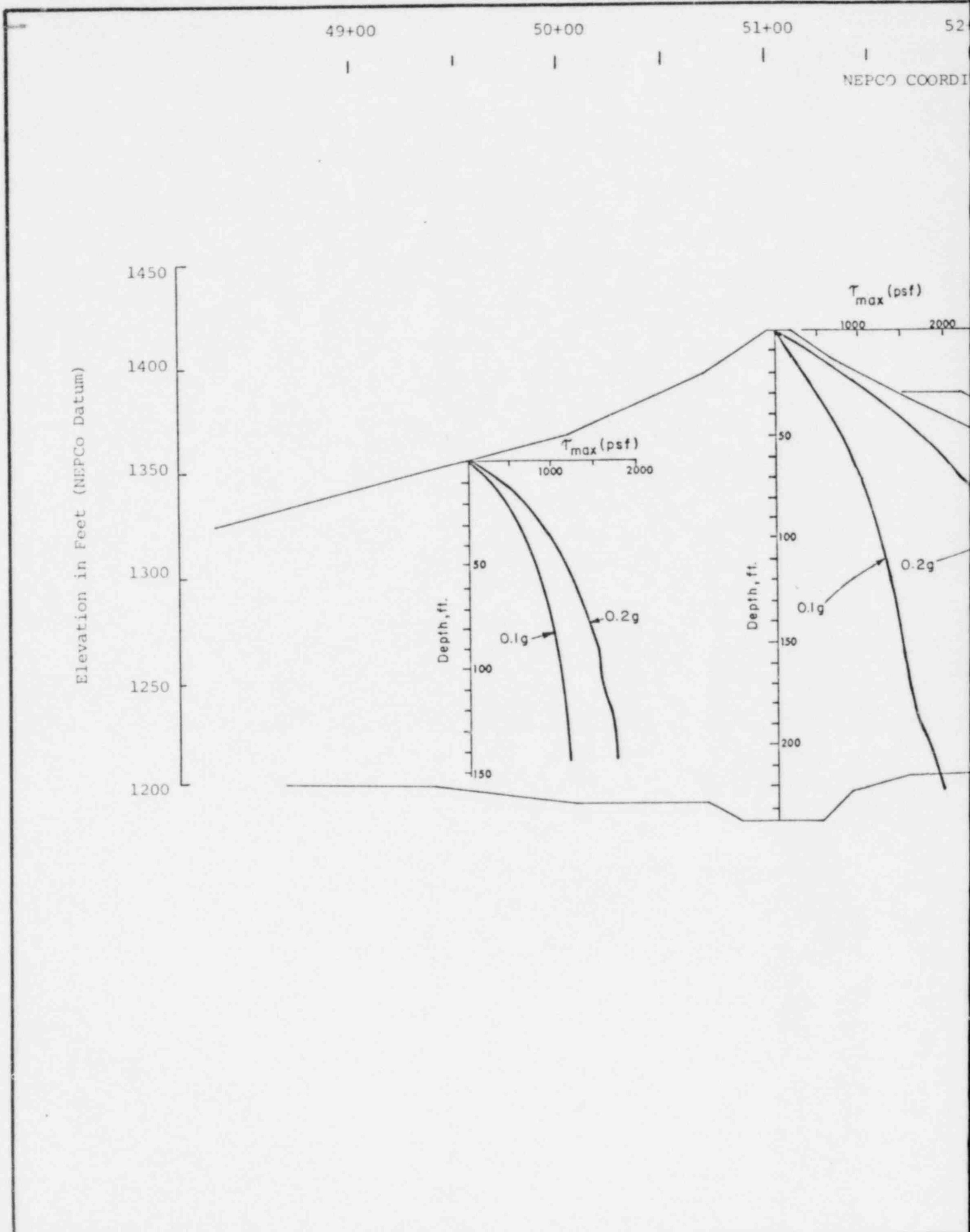


Base Acceleration a	Natural Period for Upstream Soil Column (sec)	Natural Period for Crest Soil Column (sec)
0.1 g	0.92	1.39
0.2 g	1.04	1.56

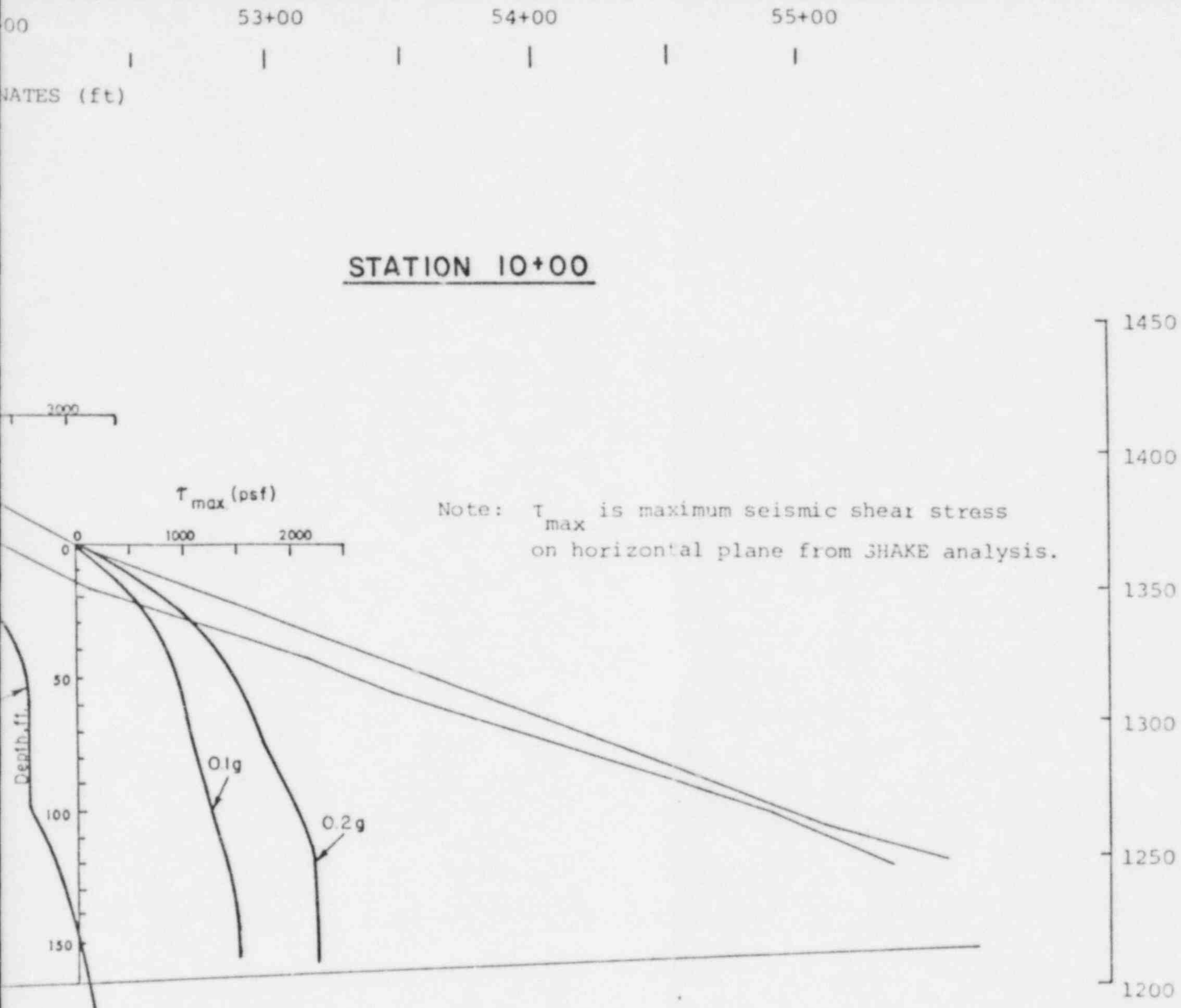


Natural Period for Downstream Soil Column (sec)
0.80
0.92

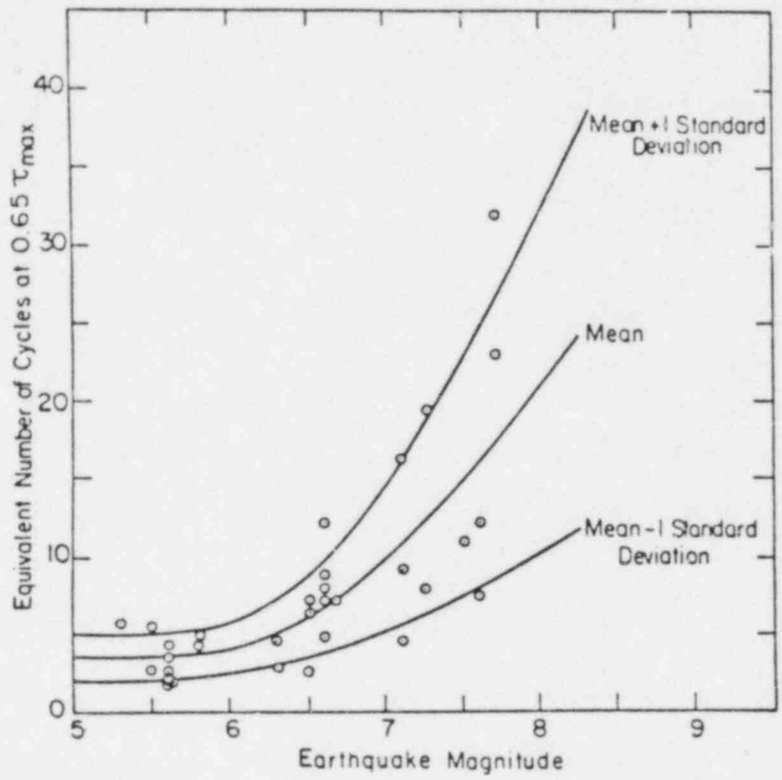
New England Power Company Westborough, Massachusetts	Harriman Dam	EARTHQUAKE ACCELERATION VS DEPTH
	Project 81858	June 7, 1982 Fig. 6



49+00
50+00
51+00
52+
NEPCO COORDI
Elevation in Feet (NEPCo Datum)
1450
1400
1350
1300
1250
1200
τ_{max} (psf)
1000
2000
Depth, ft.
50
100
150
0.1g
0.2g
τ_{max} (psf)
1000
2000
Depth, ft.
50
100
150
200
0.1g
0.2g

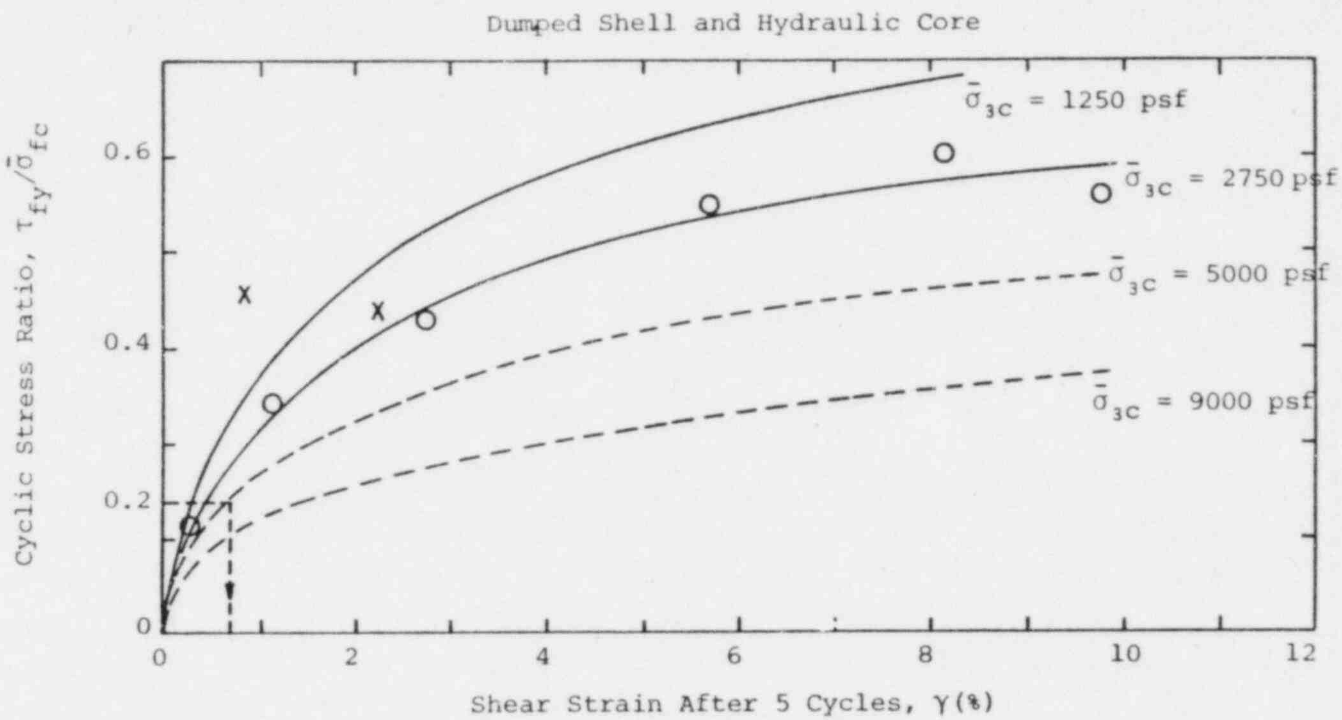


New England Power Company Westborough, Massachusetts	Harriman Dam	EARTHQUAKE SHEAR STRESSES VS. DEPTH AT STA 10+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	Dec. 15, 1981 Fig. 7



NOTE: Figure adapted from Seed, 1976

New England Power Company Westborough, Massachusetts	Harriman Dam	EQUIVALENT NUMBER OF UNIFORM STRESS CYCLES
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	June 21, 1982 Fig. 8



NOTES:

1) All test results corrected for change in void ratio during specimen handling and consolidation, and for loss of "aging effects" during sampling. See text for details.
2) Dashed curves extrapolated from measured data.
3) All tests performed on specimens with consolidation ratio K_c = 2.0.
4) Cyclic stress ratio causing a given level of strain in the washed core is 75% of the stress ratio for for dumped shell and hydraulic core.

LEGEND:

× $C\bar{R}$ test result for undisturbed specimen, dumped shell, $\bar{\sigma}_{3c}$ = 1250 psf

○ $C\bar{R}$ test result for undisturbed specimen, dumped shell, $\bar{\sigma}_{3c}$ = 2750 psf

New England Power Company Westborough, Massachusetts	Seismic Stability Evaluation Harriman Dam	CYCLIC STRESS VS STRAIN FOR 5 CYCLES OF LOADING
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	June 7, 1982 Fig. 9

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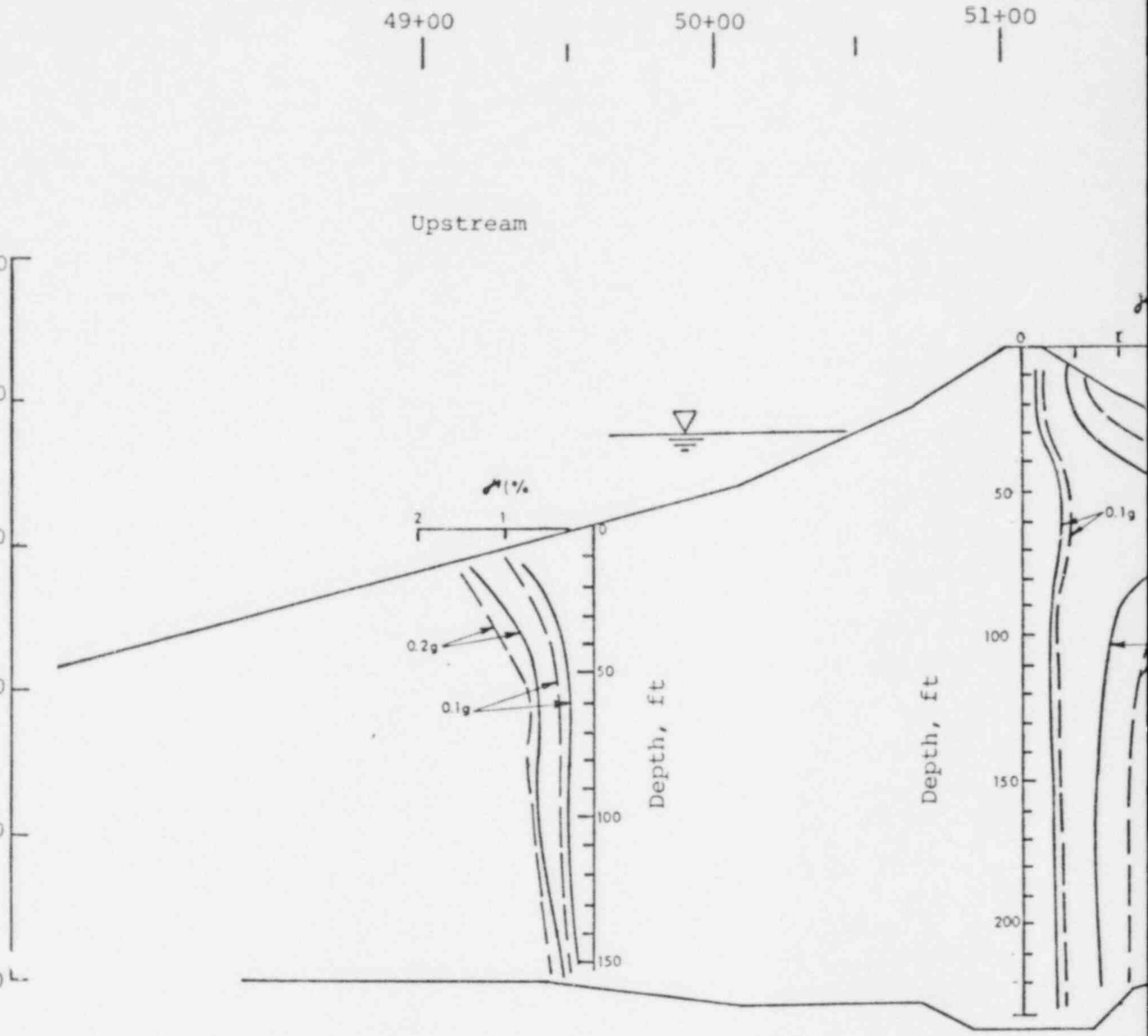
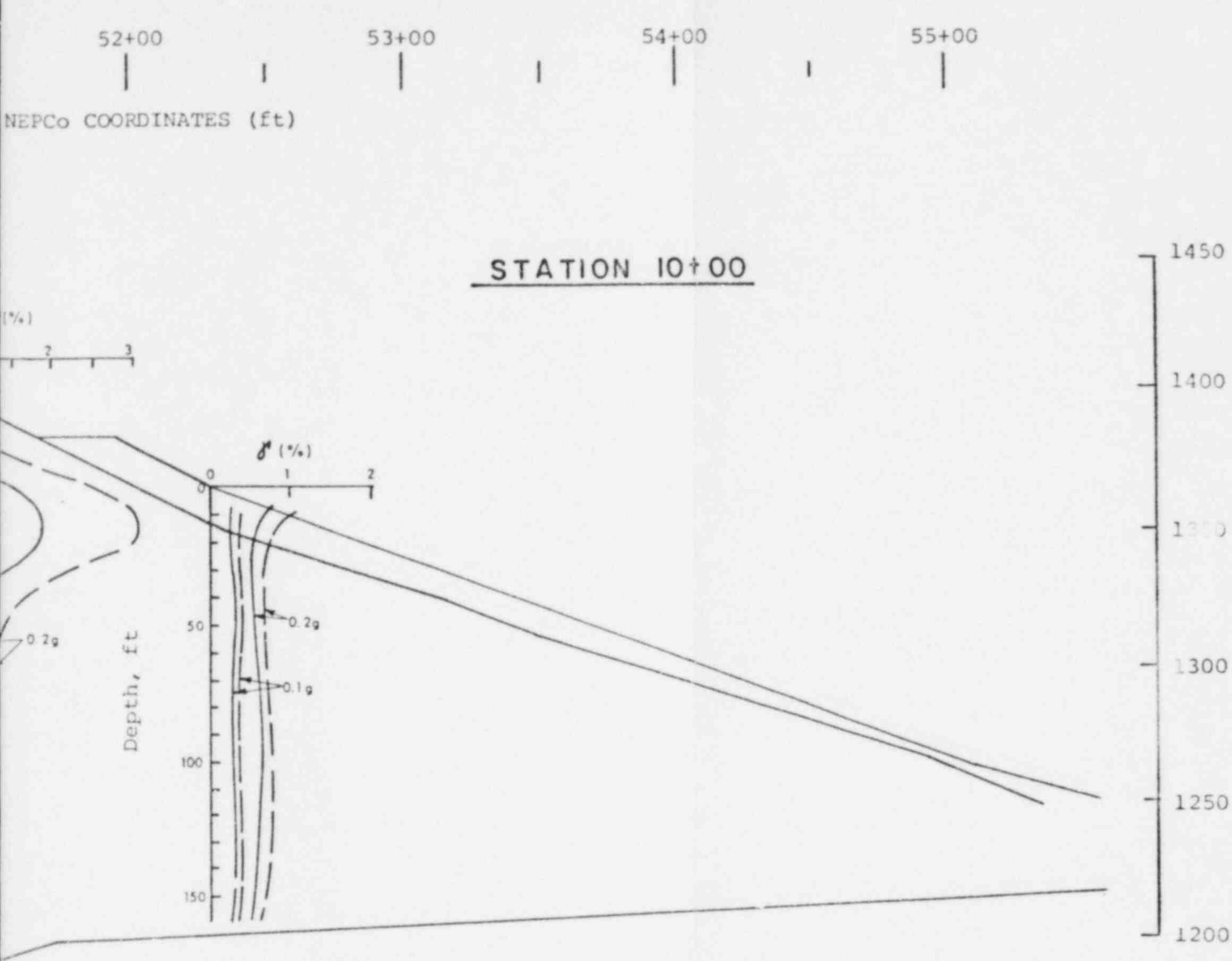
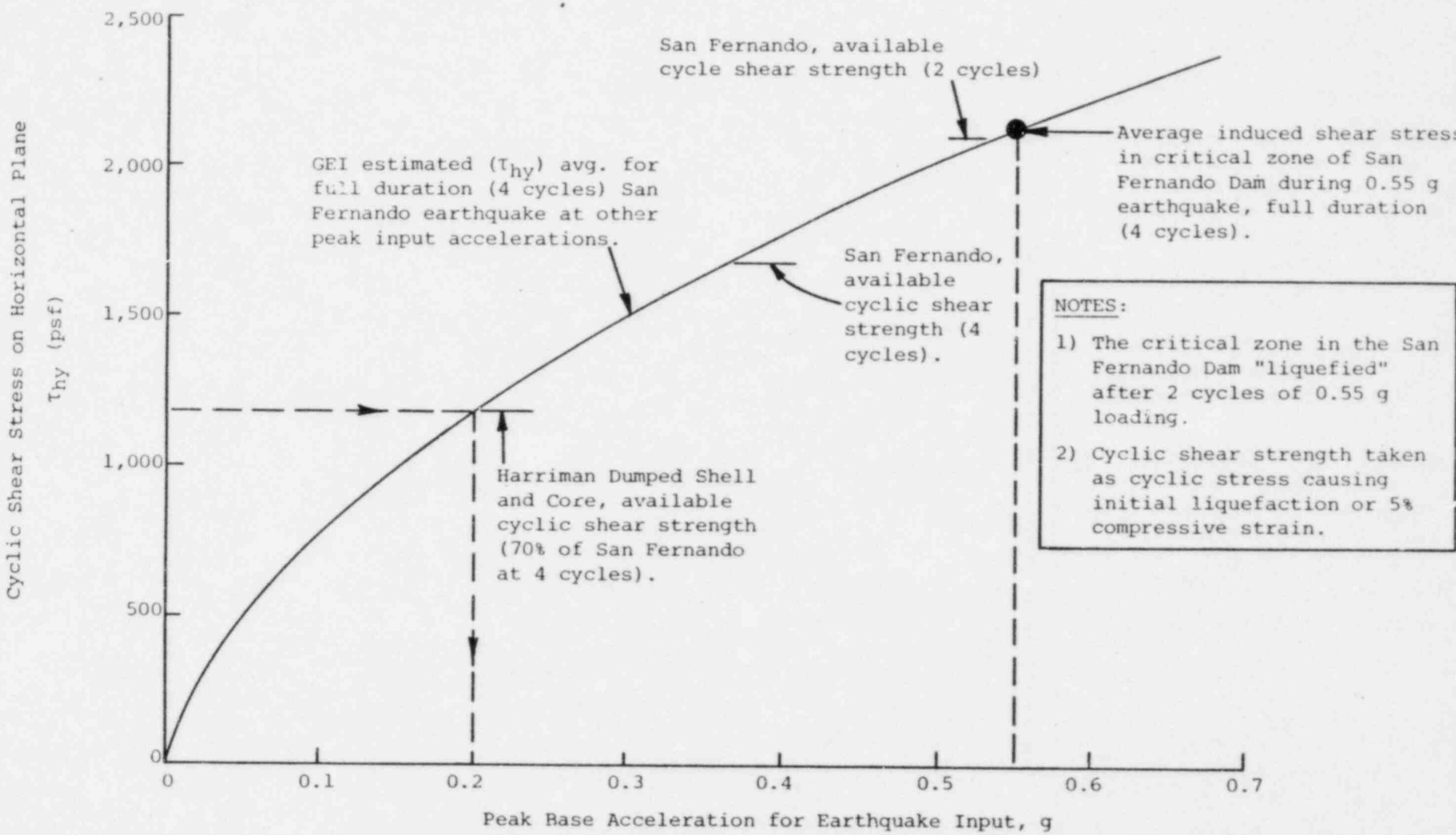


LEGEND

———	Best estimate of shear strain (5 cycles of earthquake loading)
– – –	Conservative estimate of shear strain (7 cycles of earthquake loading)
γ	Maximum accumulated shear strain after 5 or 7 cycles
0.1 g	Peak earthquake acceleration input at base of dam



New England Power Company Westborough, Massachusetts	Harriman Dam	SHEAR STRAIN VS DEPTH AT STA 10+00
	Project 81858	June 7, 1982 Fig. 10



New England Power Company Westborough, Massachusetts	Harriman Dam	EMPIRICAL COMPARISON WITH LOWER SAN FERNANDO DAM
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 81858	June 7, 1982 Fig. 11