Woodward-Clyde Consultants

SEISMIC MARGIN ASSESSMENT
ISSUES RELATED TO SOILS AND EARTHQUAKE GROUND MOTIONS
GEORGIA POWER COMPANY'S
EDWIN I. HATCH NUCLEAR POWER PLANT
APPLING COUNTY, GEORGIA

By

I. M. Idriss

Y. Moriwaki

M. G. Smith

Woodward-Clyde Consultants

203 N. Golden Circle Drive

Santa Ana, California 92705

For

Southern Company Services

P.O. Box 2625

Birmingham, Alabama 35202

WCC Project No. 8743076A

28 April 1988

TABLE OF CONTENTS

LIST OF FIGURES

SEISMIC MARGIN ASSESSMENT
ISSUES RELATED TO SOILS AND EARTHQUAKE GROUND MOTIONS
GEORGIA POWER COMPANY'S
EDWIN I. HATCH NUCLEAR POWER PLANT
APPLING COUNTY, GEORGIA

1.0 INTRODUCTION

This report covers a Seismic Margin Assessment (SMA) of issues related to soils and earthquake ground motions for Georgia Power Company's Edwin I. Hatch Nuclear Power Plant in Appling County, Georgia. The SMA incorporated the following guidelines:

1. The Seismic Margin Earthquake (SME) is conservatively specified.

2. The response of earth structures (eg, soil profile, slope, etc.) to the SME is median centered.

3. The capacity (eg, shear stress required to cause liquefaction) assessment for a given response is selected conservatively.

The following elements of work were completed as part of this SMA for the Edwin I. Hatch Nuclear Power Plant (HNP):

1. Review of available subsurface information and development of generalized soil profiles in the plant area and in the water intake area.

2. Development of a site-specific response spectrum and a synthetic accelerogram to represent the SME.

3. Evaluation of the liquefaction potential in the plant area due to the occurrence of the selected SME.

4. Evaluation of the stability of the slope in the water intake area under the SME shaking conditions.

5. Estimation of dynamic soil properties in the plant area for use in soil-structure interaction analyses.

These elements of work are described in more detail in the remaining pages of this report.

2.0 SUBSURFACE CONDITIONS

2.1 Plant Area

2.1.1 Generalized Subsurface Conditions

The subsurface conditions in the plant area were examined based on the logs of borings for 54 borings performed between 1967 and 1970 as part of the preparation of the PSAR and the FSAR for the HNP. The location of these borings are shown in Fig. 1. Based on these borings, the following generalized soil profile was established:

a. The ground surface in the plant area is generally at elevation +130 feet (MSL datum).

b. The upper 55 feet (ie, from elevation 130 to elevation 75) consist of either cemented sand or engineered, compaction-controlled fill.

c. The soils below depth of 55 feet (ie, below elevation 75) consist mainly of silty or clayey fine sand to a depth of about 130 feet (elevation 0). These soils are generally dense to very dense with occasional lower density lenses as reflected by the standard penetration (SPT) blow count.

d. The soils below a depth of about 130 feet (elevation 0) consist mostly of very dense fine sand and hard clay to a depth of about 230 feet (elevation -100) where rock or rock-like material is encountered.

This generalized soil profile is illustrated in Fig. 2. Figure 3 illustrates the extent of fill adjacent to the Reactor and Turbine Buildings.

2.1.2 Water Level

As described in the FSAR, the normal water level is at elevation +72 (ie, depth of 58 feet below the ground surface). The 10-year flood is estimated to raise the water level to about elevation +85 (ie, depth of 45 feet below the ground surface). Measurements of the water level in the plant area over the past 18 years show that the highest water level was at elevation +79 (ie, depth of 51 feet) in this time period.

For the purposes of this SMA, the water level was considered to be at elevation +85, or at a depth of 45 feet below the ground surface, as shown in Fig. 2.

2.1.3 Shear Wave Velocity Profiles

Two shear wave velocity profiles were constructed for the plant area based on the following information:

a. The generalized soil profile shown in Fig. 2;

b. The geophysical data provided by Georgia Power Company; and

c. The initial shear wave velocity estimates also provided by Georgia Power Company.

These two shear wave velocity profiles are shown in Fig. 4 and summarized below:

		Shear Wave Velocity	
Depth	Elevation	Profile I	Profile II
0 to 55 feet	130 to 75 feet	Varying linearly from 580 fps at Elevation 130 to 800 fps at Elevation 75.	1,200 pfs
55 to 130	75 to 0	Varying linearly from 800 fps at Elevation 75 to 1,120 fps at Elevation 0.	Same as Profile I
130 to 230	0 to -100	Varying linearly from 1,600 fps at Elevation 0 to 2,500 fps at Elevation -100.	Same as Profile I
Below 230	Below -100	2,500	2,500

Note that the two shear wave velocity profiles are identical below elevation 75 (depth of 55 feet). The reason for distinguishing the two velocity profiles in the upper 55 feet is to reflect the difference in subsurface conditions as shown in Figs. 2 and 3. Velocity Profile I reflects the fact that the upper 55 feet had been excavated and replaced by engineered fill around the Reactor Building and to approximately 30 feet adjacent to the Turbine Building. Velocity Profile II is intended to reflect the presence of cemented sand. These two velocity profiles appear to be equally present in the Plant Area. Accordingly, when both velocity profiles are used in the analysis, the results of velocity Profile I are given in weight of 1/2 and those of velocity Profile II a weight of 1/2.

Using either shear wave velocity profiles, the site conditions in the plant area would be described as stiff soil site conditions in accordance with the generalized subsurface classification system originally proposed by Seed et al (1976).

2.2 Water Intake Area

The subsurface conditions in the water intake area were examined based on the logs of 45 borings performed between 1967 and 1969. The two cross-sections depicted in Fig. 5 were also examined. Cross-section B-B' was judged to be the critical section and its stability was examined in more detail as summarized in Section 5.0 of this report. The locations of borings used to construct Section B-B' are shown in Fig. 5.

The generalized subsurface conditions in cross-section B-B' are depicted in Fig. 6 and are summarized below:

Layer No.	Elevation	Pre-Earthquake Strength Parameters
1	112 to 104 feet	c = 0; ϕ = 35°
2	104 to 70	c = 0; ϕ = 50°
3	70 to 60	c = 0; ϕ = 30°
4	60 to 50	c = 0; ϕ = 30°
5	Below 50	critical potential slip surfaces do not extend below Elevation 50

Note that while layers No. 3 and 4 were assigned the same pre-earthquake strength, layer No. 4 is the most critical, as described in Section 5.0. It consists mostly of clayey and silty fine sand. Layer No. 3, on the other hand, consists mostly of cohesive soils with generally higher SPT blow counts. Also, as noted in Section 5.0, Layer No. 4 is considered liquefiable and its residual shear strength is used in evaluating the post-earthquake stability of cross-section B-B'.

The water level was also considered to be at elevation 85 in the water intake area as shown in Fig. 6.

3.0 EARTHQUAKE GROUND MOTIONS

3.1 General

The HNP was designed for an OBE having a peak zero-period acceleration (ZPA) of 0.08g and an SSE having a peak ZPA = 0.15g. The SME was selected to have a peak ZPA of about 0.3g, which is significantly greater than that used for the SSE. Based on seismicity considerations described in the FSAR, the SME is considered to have a magnitude, m = 6-1/4, and to occur within 25 km of the site. Thus, the Seismic Margin Earthquake has been selected conservatively. Note that the ZPA for the SME was initially selected to be equal to 0.3g. However, as noted later in this report, to obtain the minimum required factors of safety against liquefaction in the Plant Area, the finally recommended ZPA is 0.28g. Nevertheless, the remaining discussion of earthquake ground motions and site response incorporate the use of 0.3g as the ZPA for the postulated SME.

3.2 Site-Specific Response Spectrum

A site-specific response spectrum for this evaluation was selected based on examination of the procedure outlined in NUREG/CR-0098 (Newmark and Hall, 1978). The resulting smooth spectral shape was also compared to that obtained using the average spectral shape developed by Seed et al (1976) and modified for magnitude effects using the procedure proposed by Idriss (1985).

For the purpose of using NUREG/CR-0098, values of a = ZPA and the ratio v/a and ad/v^2 are required. As noted above, a = ZPA = 0.3g was selected initially for this SME. The value of v/a = 100 cm/sec/g or 39.4 in sec/g and ad/v^2 = 5 were selected by the values included in NUREG/CR-0098, those suggested by Seed and Idriss (1982) and the values calculated for the recordings from the 1979 Imperial Valley earthquake and from the 1971 San Fernando earthquake.

The spectral shape, presented in terms of spectral velocity versus period, for ZPA = 1 g using v/a = 100 cm/sec/g, ad/v^2 =5 and the median amplification factors given in NUREG/CR-0098, is shown in Fig. 7. Also shown in this figure is the spectral shape obtained for a stiff soil site using the average spectral shape developed by Seed et al (1976) and adjusted for m = 6-1/4. The two spectral shapes are reasonably close. The spectral shape based on NUREG/CR-0098 was then used to construct the smooth response spectrum for the selected SME, which is shown in Fig. 8.

A synthetic accelerogram, having a ZPA = 0.3g and spectral ordinates that provide a reasonable fit to the smooth response spectrum, was developed using the program RASCAL (Silva and Lee, 1987). This synthetic accelerogram is shown in Fig. 9 and its spectrum is compared to the smooth response spectrum in Fig. 10. The calculated time histories of velocity and displacement for this synthetic accelerogram are presented in Figs. 11 and 12, respectively. The results shown in Figs. 9 through 12 indicate the following:

a. The spectrum for the synthetic accelerogram provides a reasonable fit to the target smooth response spectrum as illustrated in Fig. 10.

b. The total duration and the duration of strong shaking of the synthetic accelerogram are of the order of 2 to 3 times what they should be for an earthquake with m = 6-1/4. However, in as much as this accelerogram is intended mainly for use in "elastic" soil-structure interaction analyses, this longer than necessary duration, should create no difficulty nor any serious additional conservatism.

c. The peak velocity calculated for this synthetic accelerogram is about 26.9 cm/sec as shown in Fig. 11. Thus the ratio v/a for this synthetic accelerogram is about 90 cm/sec/g, which is slightly lower than the value used for constructing the smooth spectrum shown in Fig. 8. However, the spectrum for this synthetic accelerogram provides a reasonable fit to the target smooth spectrum as shown in Fig. 10. Therefore, obtaining a value of v/a about 10% lower than what was used to construct the smooth spectrum has little impact on the final results.

d. The peak displacement calculated for the synthetic time history is 13.5 cm as shown in Fig. 12. The resulting ad/v^2 for this synthetic accelerogram is thus equal to about 5.5, which is about [illegible]% greater than that used in constructing the smooth spectrum. The comments made above regarding v/a apply equally to the ratio ad/v^2.

4.0 LIQUEFACTION POTENTIAL IN PLANT AREA

4.1 General Procedure

The procedure used herein to evaluate the liquefaction potential due to the SME in the plant area consisted of the following steps:

1. The shear stresses induced by the SME were calculated using a ground response analysis procedure as outlined in Section 4.2.

2. The shear stresses required to cause liquefaction were estimated using charts, based on field case histories, and relating such stresses to the standard penetration resistance (SPT). The SPT values obtained in borings in the plant area (Section 4.3) were used in these charts to estimate these stresses for the plant area.

3. The stresses induced by the SME were then compared to those required to cause liquefaction at various depths in the plant area. This comparison was made in terms of the ratio of shear stress required to cause liquefaction divided by the shear stress induced by the SME at the same depth in the soil profile. This ratio represents the margin available to resist liquefaction at a site due to the postulated earthquake ground motion. This ratio is often referred to as a factor of safety against liquefaction.

Because of the variability of the SPT data within the plant area, statistics were conducted on these data to obtain mean and other percentile values. Thus, the stresses required to cause liquefaction (and hence the stress ratio, or margin, described in Step 3 above), will depend on the selected percentile value of the SPT data. For this purpose, the following minimum values of the ratio of shear stress required to cause liquefaction divided by the shear stress induced by the SME, were selected for the SMA for this project.

SPT Values Used	Minimum Required Margin
Mean	1.50
Mean - 1/2 σ	1.30
Mean - σ	1.05

Note that margin is used herein to represent the stress ratio described above, mean is the arithmetic average of the data and σ is the standard deviation.

The (mean - 1/2 σ) and the (mean - σ) values correspond approximately to the 30-percentile and the 16-percentile, respectively.

4.2 Shear Stresses Induced by the SME

The shear stresses induced by the SME were calculated using a ground response analysis procedure. The program SHAKE (Schnabel et al, 1972) was used for this purpose. The synthetic accelerogram shown in Fig. 9 was applied at the ground surface in the Plant area and the calculations were made for the two shear wave velocity profiles shown in Fig. 4. The variations of modulus and damping with shear strain were based on the average values published for sands (Seed and Idriss, 1970).

The results of these calculations are presented in Fig. 13. In addition to the maximum shear stresses induced by the SME, the peak horizontal accelerations calculated at various depths in the plant area are shown in this figure.

The results presented in Fig. 13 indicate that the peak horizontal acceleration decreases somewhat with depth and is equal to about 0.25g at a depth of 55 feet, which is the embedment depth of both Unit 1 and Unit 2 reactor buildings. Below a depth of 55 feet, the peak horizontal acceleration increases somewhat. The calculated peak horizontal accelerations do not appear to be significantly affected by the shear wave velocity profile used.

The maximum shear stresses calculated using shear wave velocity Profile II are greater than those calculated using shear wave velocity Profile I. As shown in Fig. 3, however, Profile I represents a significant portion of the plant area. As discussed in Section 2.1.3, the maximum shear stresses induced by the SME were considered to be the average of those calculated using shear wave velocity Profile I and those using Profile II. The resulting maximum shear stresses considered for the SME in the plant area are shown in Fig. 14. (Note that the results shown in both Figs. 13 and 14 are for a ZPA = 0.3g, which was initially assigned to the SME.)

The maximum shear stresses (open circles) shown in Fig. 14 were multiplied by 0.65 to convert them to equivalent uniform shear stresses (filled in circles in Fig. 14). (The factor of 0.65 was originally recommended by Seed and Idriss (1971) and is commonly used for this purpose in liquefaction studies.) The smooth curve shown in Fig. 14 was then used in the liquefaction evaluation in the Plant area.

4.3 Shear Stresses Required to Cause Liquefaction

4.3.1 Charts Relating Shear Stress Required to Cause Liquefaction With SPT Blow Count

Using field performance data, plots relating to the ratio $\tau/\sigma'c$ to $(N_1)_{60}$ were developed by Seed et al (1985) as shown in Fig. 15. The ratio $\tau/\sigma'o$ represents the shear stress required to cause liquefaction divided by the vertical effective stress. The parameter (N1)60 is the modified SPT blow count adjusted for vertical effective confining pressure to obtain N_1 (ie, the blow count for a vertical effective confining pressure of 1 tsf or 2 ksf) and considering that the system used in obtaining SPT blow count delivers 60% hammer energy. Three curves are presented in Fig. 15 to reflect the influence of fines content on susceptibility to liquefaction. Note that the curves in Fig. 15 are for use with earthquake magnitude m = 7-1/2 and $\sigma'o$ = 1 tsf or 2 ksf.

4.3.2 Adjustment for Earthquake Magnitude

For other earthquake magnitudes, the stress ordinates may be increased by the ratios suggested by Seed and Idriss (1982). For m = 6-1/4, this ratio is approximately 1.264. The curve, with percent fines $\leq$ 5%, thus adjusted is shown in Fig. 16.

4.3.3 Adjustment for Vertical Effective Stress

For vertical effective stresses different from 1 tsf or 2 ksf, an additional adjustment is required. The adjustment factor K_σ is obtained as follows:

$$K_\sigma = \frac{(\tau/\sigma'o)\ \ for\ \ \sigma'o = \sigma'o}{(\tau/\sigma'o)\ \ for\ \ \sigma'o = 1\,tsf}$$

The range of K_σ obtained for cohesionless soils and the curve adopted for this project are presented in Fig. 17.

4.3.4 Stresses Required to Cause Liquefaction

The SPT blow count is a key parameter in evaluating the stress required to cause liquefaction. The raw blow count for each SPT sample in each boring is adjusted to obtain the corresponding $(N_1)_{60}$ for that sample. For the SPT data in the Plant area, the following adjustments were carried out to obtain $(N_1)_{60}$:

1. The raw blow count was multiplied by a factor of 0.75 because a donut hammer had been used to drive the sampler.

2. The resulting N-value was then converted to N1 (ie, blow count for σ'o = 1 tsf) using the equation:

$$N_1 = C_N \cdot N$$

The value of C_N was obtained from the curve shown in Fig. 18 for Dr = 60 to 80%.

3. Except for those samples that were clearly identified in the borings as essentially clean sands, the N1 obtained in Step 2 was increased by $\Delta N_1 = 5$ to obtain an "equivalent clean sand" value of N_1. The available grain-size distribution data for 25 samples of the silty/clayey sands in the Plant area indicate an average percent fines of about 20% for these samples. A $\Delta N_1 = 5$ thus appears reasonable based on the curves shown in Fig. 15. The resulting N_1 is then considered representative of $(N_1)_{60}$.

The N1-values in the Plant area obtained from Step 3 are presented in Fig. 19. Based on the data shown in Fig. 18, the following values are obtained:

Elevation	Mean $(N_1)_{60}$	Standard Deviation, σ
65 to 80	17.3	5.6
45 to 65	15.3	4.7
40 to 45	20.3	5.2
30 to 40	23.5	7.0
20 to 30	23.9	6.4
0 to 20	25.5	6.3

Note that $(N_1)_{60}$ greater than 40 were excluded in the above calculations below elevation 45; $(N_1)_{60}$ greater than 30 were excluded above elevation 65; and between elevation 45 and 65, $(N_1)_{60}$ greater than 25 were excluded.

The mean, (mean - 1/2 σ) and (mean - σ) values of $(N_1)_{60}$ were then used in Fig. 16 to obtain $\tau/\sigma'o$ for $\sigma'o$ = 2 ksf and adjusted by the parameter K_σ (Fig. 17) to obtain the cyclic shear stress required to cause liquefaction for the applicable $\sigma'o$.

4.4 Liquefaction Potential

The equivalent uniform shear stresses induced by the SME are compared in Fig. 20 (again, for ZPA = 0.3g) to those required to cause liquefaction in the Plant area. The latter stresses are presented in Fig. 20 for two cases: one case is based on the use of the mean values of $(N_1)_{60}$ and the other case is based on the use of the (mean - 1/2 σ) values. As can be seen in Fig. 20, the stresses required to cause liquefaction for both cases are well above those induced by the SME. The values of the calculated margin to resist liquefaction (ie, the ratio of the stress required to cause liquefaction divided by the stress induced by the SME) in the Plant area based on the mean values of $(N_1)_{60}$ are the following:

Elevation	Mean $(N_1)_{60}$	Range of Calculated Margin	Minimum Required Margin
65 to 85	17.3	1.60 to 1.71	1.5
45 to 65	15.3	1.42 to 1.48	1.5
40 to 45	20.3	1.97 to 2.00	1.5
Below 40	>23	>2.3	1.5

Those based on (mean - 1/2 σ) are the following:

Elevation	(Mean - 1/2 σ) $(N_1)_{60}$	Range of Calculated Margin	Minimum Required Margin
65 to 85	14.5	1.34 to 1.43	1.3
45 to 65	13.0	1.20 to 1.26	1.3
40 to 45	17.7	1.72 to 1.74	1.3
Below 40	>20	>2.0	1.3

And those based on (mean - σ) values of $(N_1)_{60}$ are the following:

Elevation	(Mean - σ) $(N_1)_{60}$	Range of Calculated Margin	Minimum Required Margin
65 to 85	11.7	1.08 to 1.16	1.05
45 to 65	10.6	0.98 to 1.03	1.05
40 to 45	15.1	1.46 to 1.49	1.05
Below 40	>16	>1.6	1.05

Thus, for an SME having a peak horizontal ground surface acceleration (ZPA) equal to 0.3g, the minimum required margin is equalled or is exceeded within the soil profile in the Plant area except in the elevation range of about 45 to 65. Within this elevation range, the minimum calculated margin is 1.42, 1.20, and 0.98 based on the use of the mean (mean - 1/2 σ) and (mean - σ) values of $(N_1)_{60}$, respectively.

To meet the minimum margins of 1.5, 1.3, and 1.05, the peak horizontal ground surface acceleration should be reduced to 0.28g. Therefore, an SME with ZPA = 0.28g appears appropriate for the Hatch NP.

5.0 SLOPE STABILITY IN WATER INTAKE AREA

5.1 General Procedure

The procedure used herein to evaluate the slope stability in the water intake area consisted of the following steps:

1. The available borings in the water intake area were examined to assess the general subsurface conditions in this area.

2. The available topographic map (Fig. 5) was used to construct cross-sections and to examine the stability of slopes in this area. Cross-section B-B' (Fig. 6) was judged to be the critical section for this purpose.

3. Layer No. 4, which extends from about elevation 60 to elevation 50 as shown in Fig. 6, was considered liquefiable. Thus, its strength was assumed to be reduced to the residual strength.

4. The available blow count in borings located along or reasonably close to, Section B-B' were examined and converted to $(N_1)_{60}$ using the steps outlined in Section 4.3.4. The results are summarized in Fig. 21. Note that values of $(N_1)_{60}$ shown in Fig. 21 do not include adjustment for fines content.

5. Using the (mean - 1/2 σ) value of $(N_1)_{60}$ (with appropriate adjustment for fines content) for Layer No. 4, the residual shear strength was estimated. The post-earthquake slope stability of Section B-B' was then evaluated using this residual shear strength in Layer No. 4. The four potential slip surfaces used for this evaluation are shown in Fig. 22.

6. The amounts of lateral movement along Section B-B' due to the occurrence of the SME were then estimated.

5.2 Overall Stability

Using the soil properties of layers Nos. 1 through 4, summarized in Section 2.2, the calculated pre-earthquake minimum factors of safety against sliding of the four potential slide surfaces shown in Fig. 22 are well over 3. The minimum post-earthquake factors of safety were computed assuming that the SME causes the shear strength in Layer No. 4 to decrease to the residual strength (as noted in Step 3 above).

Figure 23 was used to estimate the residual strength in layer No. 4. This figure relates the residual strength to the "equivalent clean sand" $(N_1)_{60}$. The values of $(N_1)_{60}$ in Layer 4 are shown in Fig. 21 and range from 3 to 15 before adjustment for fines content is included. Using the guidelines given by Seed (1987), a ΔN_1 = 1.5 is justified for this purpose. Increasing each $(N_1)_{60}$ in Layer No. 4 by 1.5, the following values are obtained.

mean = 10.0

Standard deviation, σ = 3.8

Thus, for (mean - 1/2 σ) = 8.1, the residual strength would range from about 100 to 500 psf, with an average value of 300 psf. The value of 300 psf was therefore used as the residual strength in Layer No. 4.

The minimum post-earthquake factor of safety against sliding was calculated for potential slip surface A and was equal to 1.5. The post-earthquake factors of safety against sliding for potential slide surfaces B, C, and D were greater than 1.5.

Thus, it is our judgement that the slopes in the water intake area are unlikely to experience serious instability due to the occurrence of the postulated SME. Limited amounts of lateral movement are likely; these amounts are discussed in the following section.

5.3 Lateral Movement

The procedure used to estimate lateral movements of the potential slide surfaces A, B, C, and D shown in Fig. 22, was based on Newmark's (1965) approach as augmented by Goodman and Seed (1966) and by Makdisi and Seed (1978).

The procedure considers that a maximum acceleration k_{max} is applied and that the slope has a yield acceleration k_y. If, during shaking, k_{max} exceeds k_y, an amount of permanent lateral movement is imparted to the slope. As shaking continues and if k_{max} again exceeds k_y, additional permanent lateral movement takes place. At the end of shaking, the total permanent lateral movement is then the sum of the individual movements caused whenever k_{max} exceeded k_y. Makdisi and Seed (1978) provided charts that related the amounts of movement to the ratio of k_y divided by k_{max} for various earthquake magnitudes. For ease of application for calculating deformation along Section B-B', the curves published by Makdisi and Seed for earthquake magnitude, m = 6-1/2, were divided by 9 (the number of cycles estimated for m = 6-1/2) to represent the estimated lateral displacement per cycle as shown in Fig. 24. This figure then provides a reasonable approximation for m = 6-1/4 on a per cycle basis.

The number of cycles for m = 6-1/4 is estimated to be 7. Considering that the residual strength in Layer No. 4 is reached at the end of shaking, then the strength in Layer No. 4 within potential slip surfaces A, B, C, and D would vary with the number of cycles approximately as shown in Fig. 25. The values of k_y are dependent on shear strength and are equal to those shown in Fig. 26. The value of k_{max} is assumed equal to 0.3g for potential slide surface A and decreasing linearly to 0.2g for surface D. These assumptions produce the curve shown in Fig. 27 and labeled best estimate. The range shown in Fig. 27 reflects considering k_{max} = 0.3g for all potential slide surfaces to the case where the initial shear strength is applicable for the first 6 cycles and then drops to the residual strength.

The best estimate curve indicates the values summarized above the slope in Fig. 27 and presented below:

Potential Slip Surface	Best Estimate Lateral Movement
A	less than 2-1/2 inches
B	less than 2
C	less than 1
D	less than 1/2

6.0 SUMMARY AND CONCLUSIONS

The results of a seismic margin assessment (SMA) of issues related to soils and earthquake ground motions for Georgia Power Company's Edwin I. Hatch Nuclear Power Plant are included in this report. The issues related to soils addressed in this report pertain to liquefaction in the Plant area and to slope stability in the Water Intake area.

Minimum required margins to resist liquefaction in the Plant area were obtained considering that the postulated seismic margin earthquake (SME) has a zero-period acceleration (ZPA) equal to 0.28g.

The slopes in the Water Intake area are unlikely to experience serious instability due to the occurrence of the postulated SME. Limited amounts of lateral movement are likely. The movements estimated are of the order of 2-1/2 inches near the top of the slope decreasing to less than about 1/2 inch about 200 feet behind the top of the slope as illustrated in Fig. 27.

7.0 REFERENCES

DeAlba, P., Seed, H. B., Retmala, E., and Seed, R. B. (1987) "Residual Strength of Sand from Dam Failures in the Chilean Earthquake of March 3, 1985", Report No. UCB/EERC-87/11, Earthquake Engineering Research Institute, University of California, Berkeley, September.

Goodman, R. E. and Seed, H. B. (1966) "Earthquake-Induced Displacements in Sand Embankments", ASCE Journal of the Soil Mechanics and Foundation Engineering Division, v. 92, no. SM2, 125-146.

Harder, L. F. (1986) Personal Communication.

Idriss, I. M. (1985) "Theme Lecture No. 6: Evaluating Seismic Risk in Engineering Practice", Proceedings, XI International Conference on Soil Mechanics and Foundation Engineering, v. I, 255-320, San Francisco, California, August.

Makdisi, F. I. and Seed, H. B. (1978) "Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations", ASCE Journal of the Geotechnical Engineering Division, v. 104, no. GT7, 849-867.

Marcuson, W. F., III and Bieganousky, W. A. (1977) "Laboratory Standard Penetration Tests on Fine Sands", ASCE Journal of the Geotechnical Engineering Division, v. 103, no. GT6, 565-588.

Newmark, N. M. (1965) "Effects of Earthquakes on Dams and Embankments: 5th Rankine Lecture", Geotechnique, v. 15, no. 2.

Newmark, N. M. and Hall, W. J. (1978) "Development of Criteria for Seismic Review of Selected Nuclear Power Plants", prepared for U.S. Nuclear Regulatory Commission, No. NUREG/CR-0098, May.

Schnabel, P. B., Lysmer, J., and Seed, H. B. (1972) "SHAKE", Report No. EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, December.

Seed, H. B. (1987) "Design Problems in Soil Liquefaction", ASCE Journal of the Geotechnical Engineering Division, v. 113, no. 8, August, 827-845.

Seed, H. B. and Idriss, I. M. (1970) "Soil Moduli and Damping Factors for Dynamic Response Analyses", Earthquake Engineering Research Center, Report No. EERC 70-10, University of California, Berkeley.

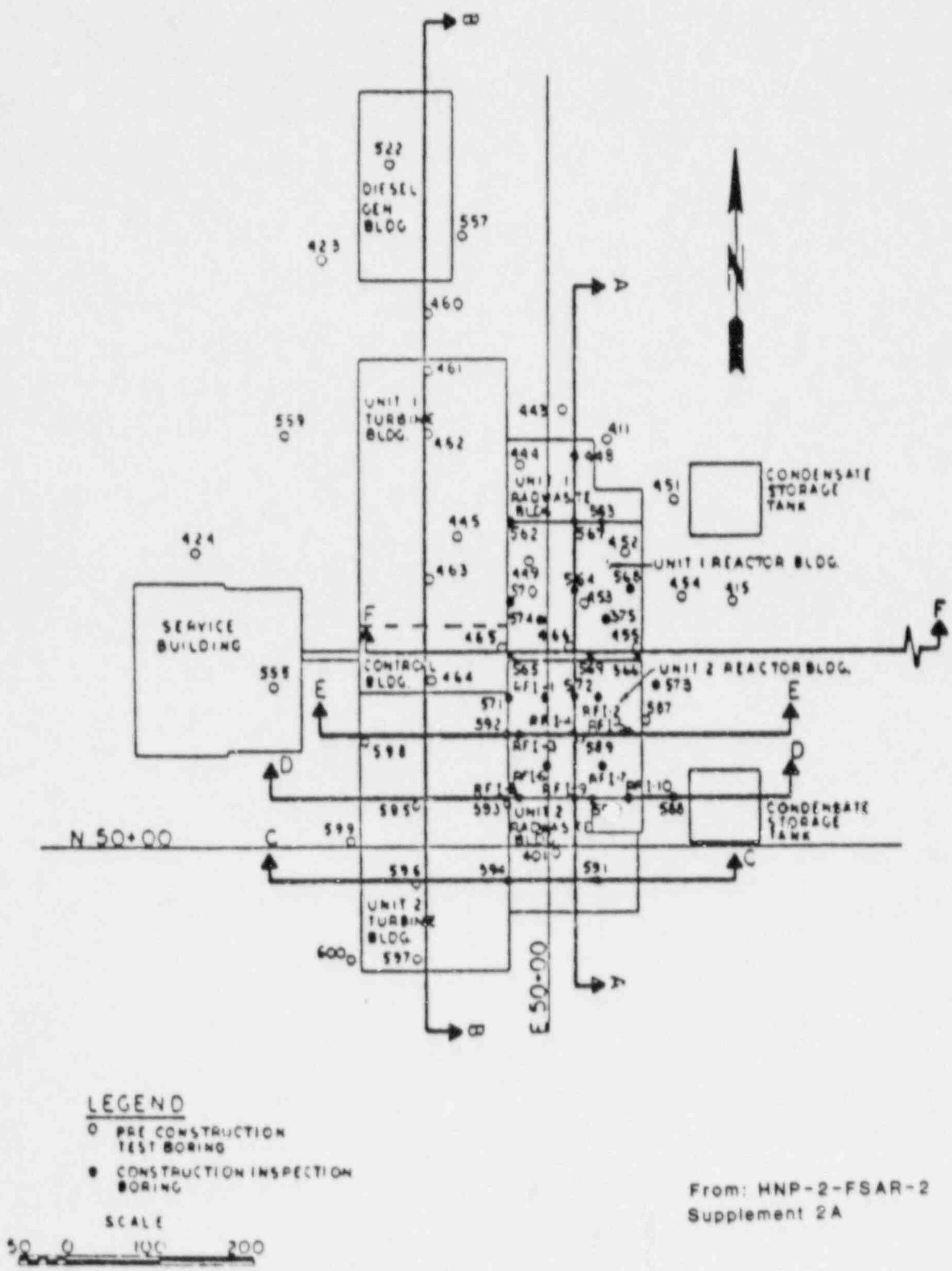
Seed, H. B. and Idriss, I. M. (1971) "Simplified Procedure for Evaluating Soil Liquefaction Potential", ASCE Journal of the Soil Mechanics and Foundation Division, v. 97, no. SM9, 1249-1273.

Seed, H. B. and Idriss, I. M. (1982) "Ground Motions and Soil Liquefaction During Earthquakes", Monograph No. 5, Monograph Series, Earthquake Engineering Research Institute, Berkeley, California, 134 p.

Seed, H. B., Ugas, C., and Lysmer, J. R. (1976) "Site-Dependent Spectra for Earthquake Resistant Design", Bulletin of the Seismological Society of America, v. 66, no. 1, 221-243.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M. (1985) "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations", ASCE Journal of the Geotechnical Engineering Division, v. 111, no. 12, December, 1425-1445.

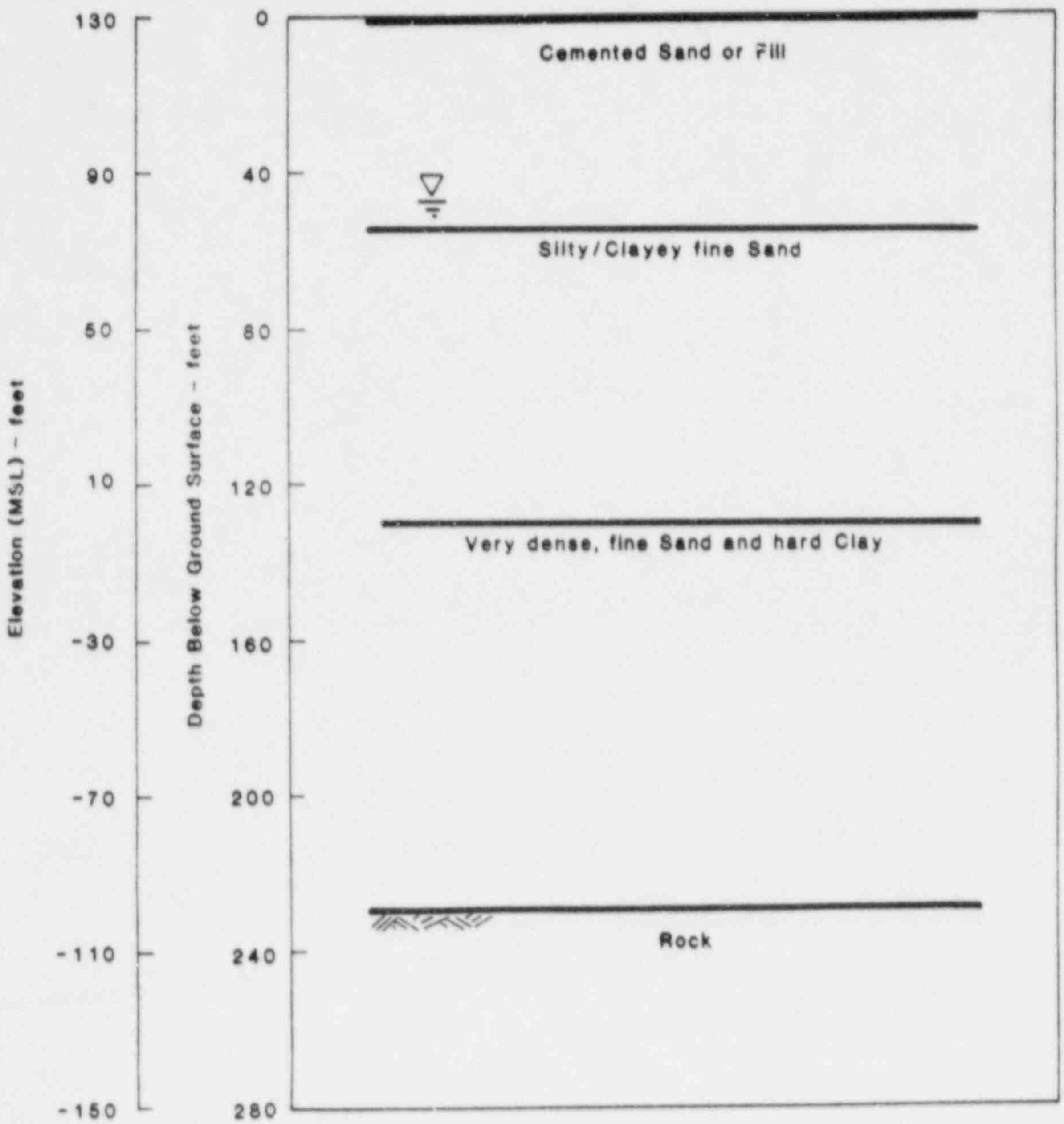
Silva, W. J. and Lee, K. (1987) "State-of-the-Art for Assessing Earthquake Hazards in the United States", Report prepared for U.S. Army Corps of Engineers, Miscellaneous Paper S-73-1, May.



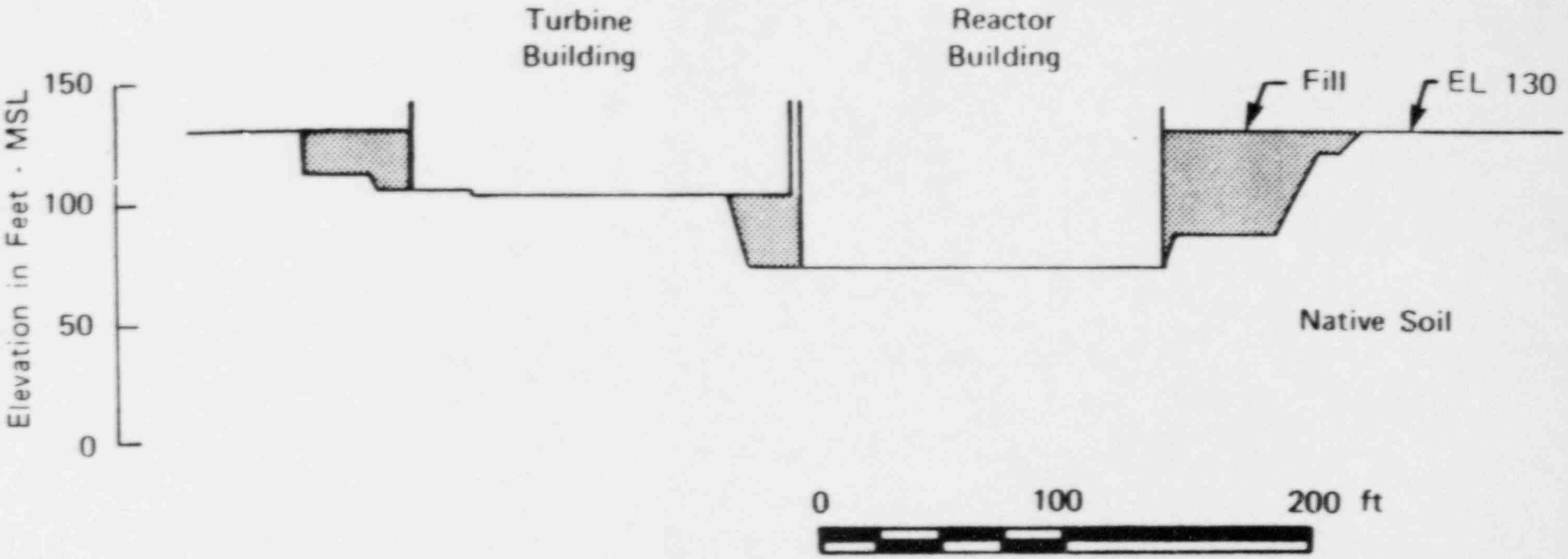
From: HNP-2-FSAR-2
Supplement 2A

Project	HATCH NP	BORING LOCATIONS - PLANT AREA	Fig 1
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS



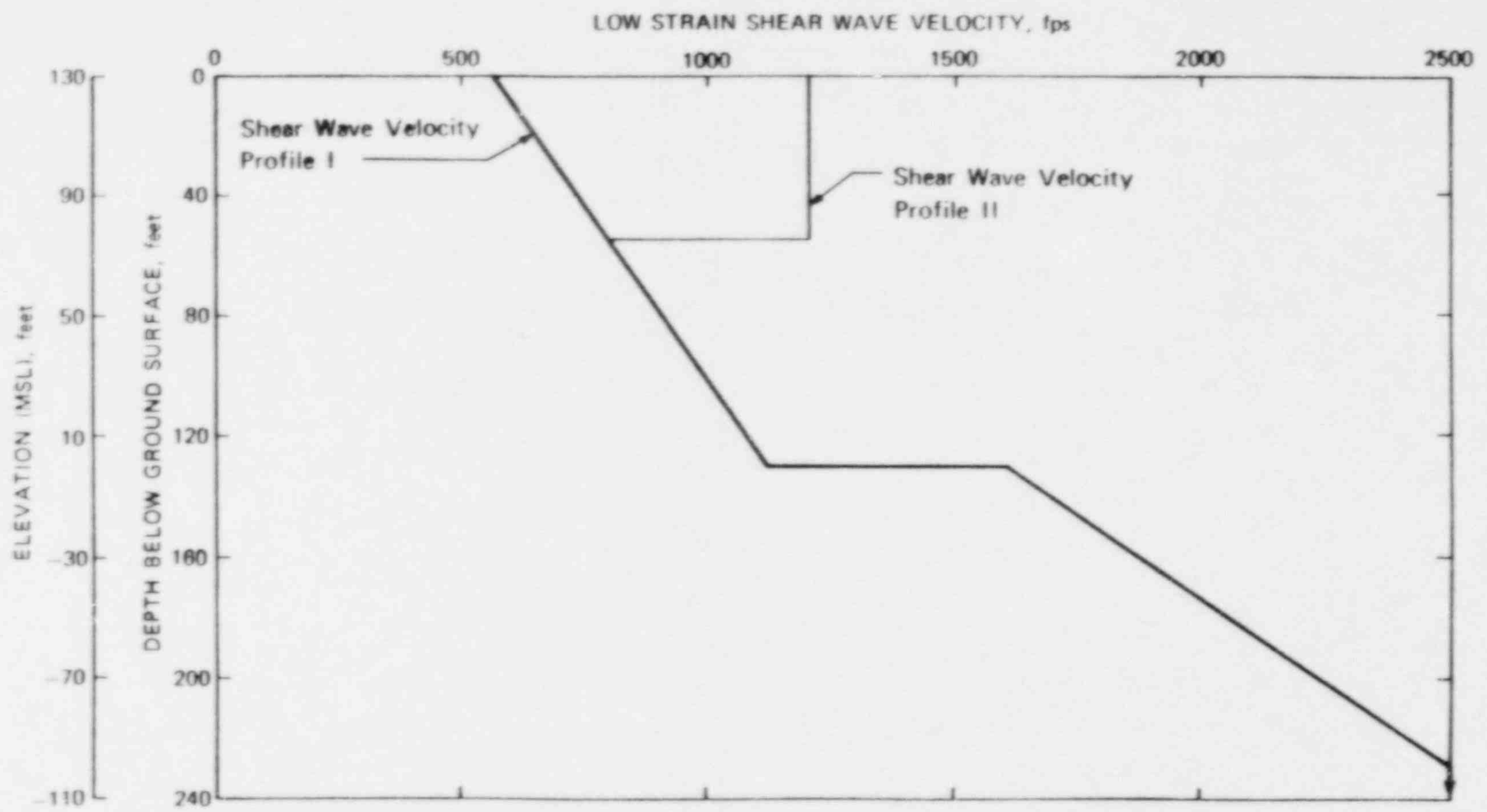
Project	HATCH NP	GENERALIZED SOIL PROFILE - PLANT AREA	Fig
Project No	8743076A		2



Section corresponds approximately with section E - E' shown in Fig. 1.
Information regarding extent of fill from Hatch Nuclear Plant 2-FSAR-2 Figure 2A-3.

Project:	HATCH NP	APPROXIMATE EXTENT OF FILL IN PLANT AREA	Fig. 3
Project No.	8743076A		

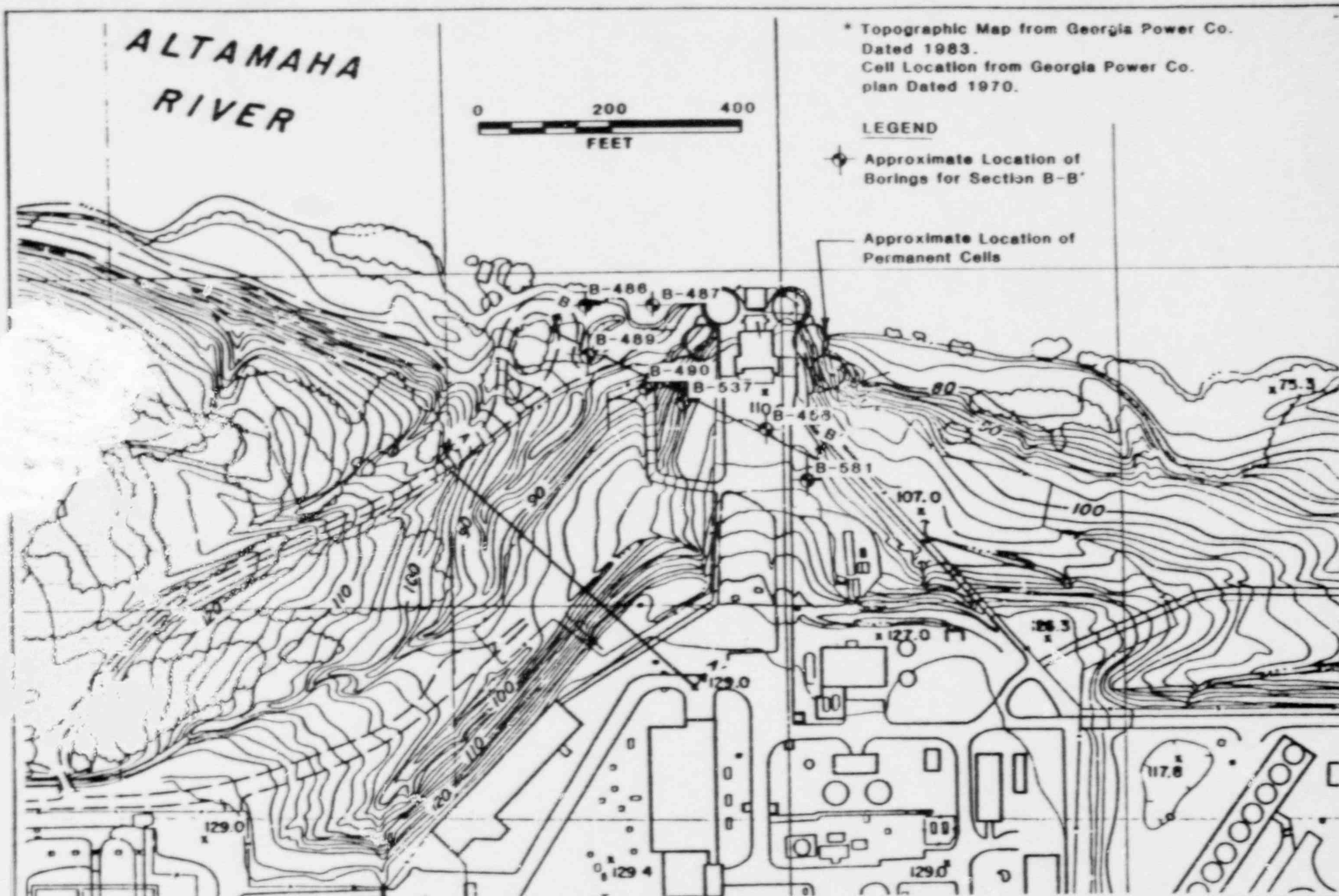
WOODWARD-CLYDE CONSULTANTS



BEST-ESTIMATE SHEAR WAVE VELOCITIES-PLANT AREA

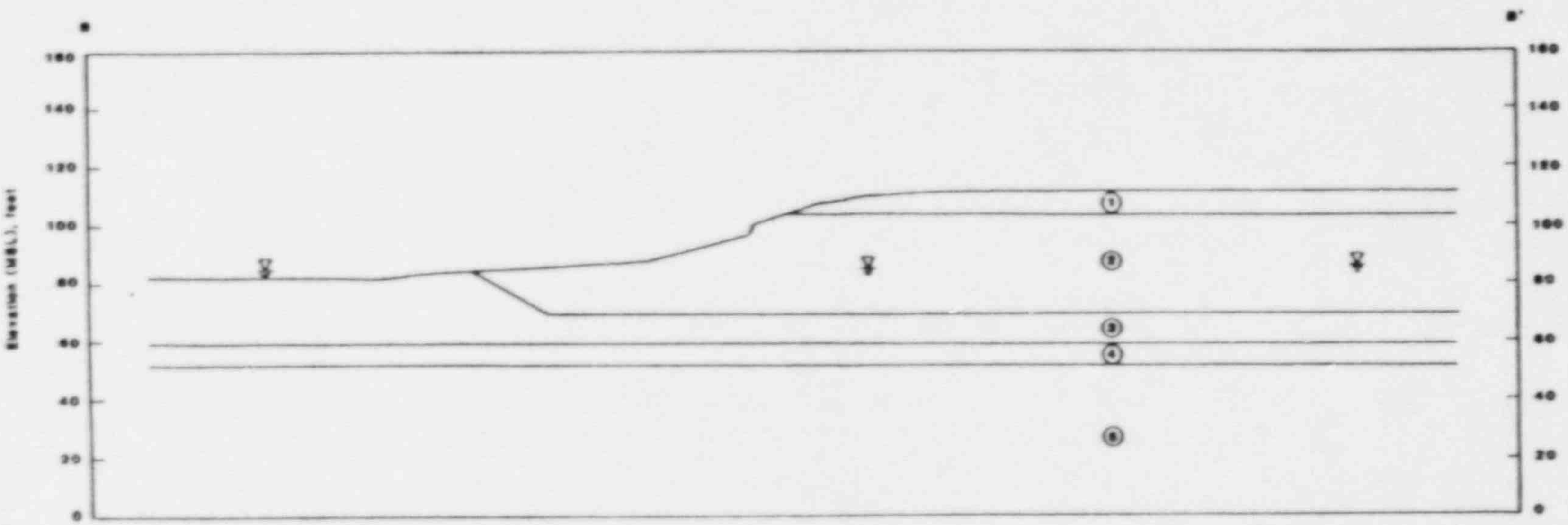
Project No.: 8743076A		Project: HATCH NP	Fig. 4

Woodward-Clyde Consultants



Project:	HATCH NP	BORING LOCATIONS - WATER INTAKE AREA	Fig
Project No	8743076A		5

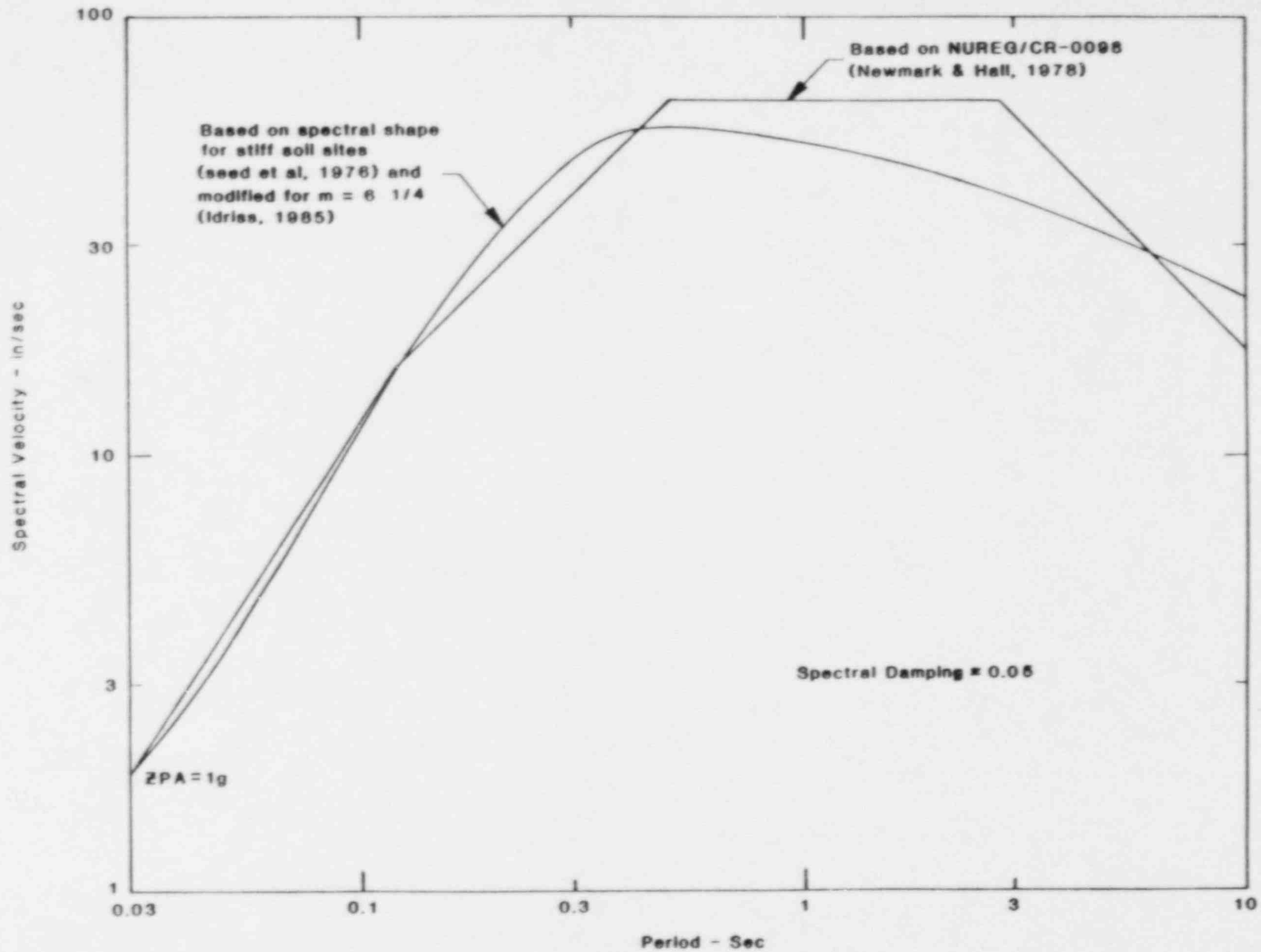
WOODWARD-CLYDE CONSULTANTS



LEGEND

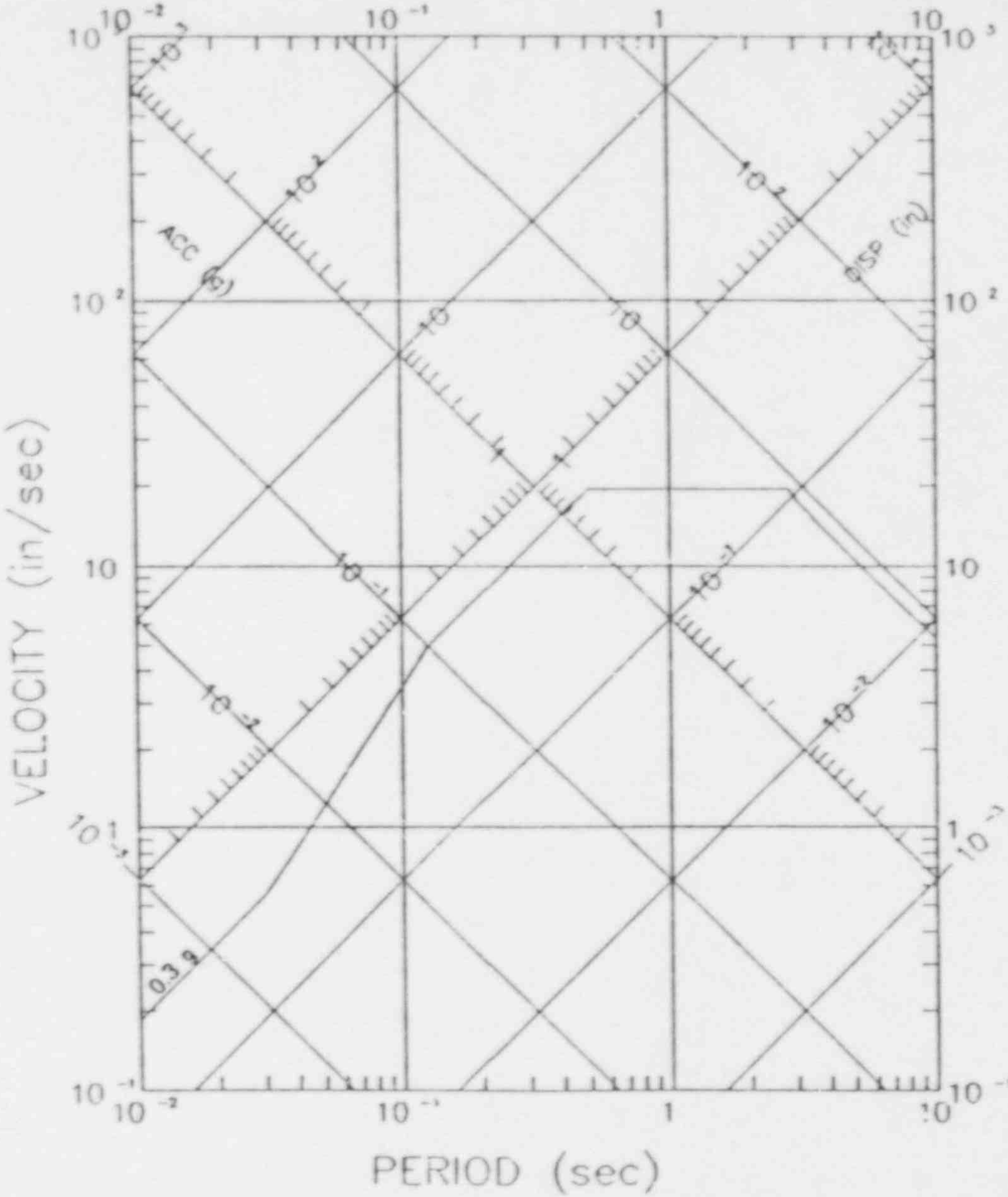
1. Silty/clayey fine Sand
2. Cemented Sand
3. Clay
4. Silty/clayey fine Sand
5. Very dense silty/clayey fine Sand and hard Clay

Woodward-Clyde Consultants	
GENERALIZED SOIL LAYERING - SECTION B-B' WATER INTAKE AREA	
Project No. 8743078A	Fig. 6
HATCH NP	



Project	HATCH NP	SPECTRAL SHAPES	Fig.
Project No	8743076A		7

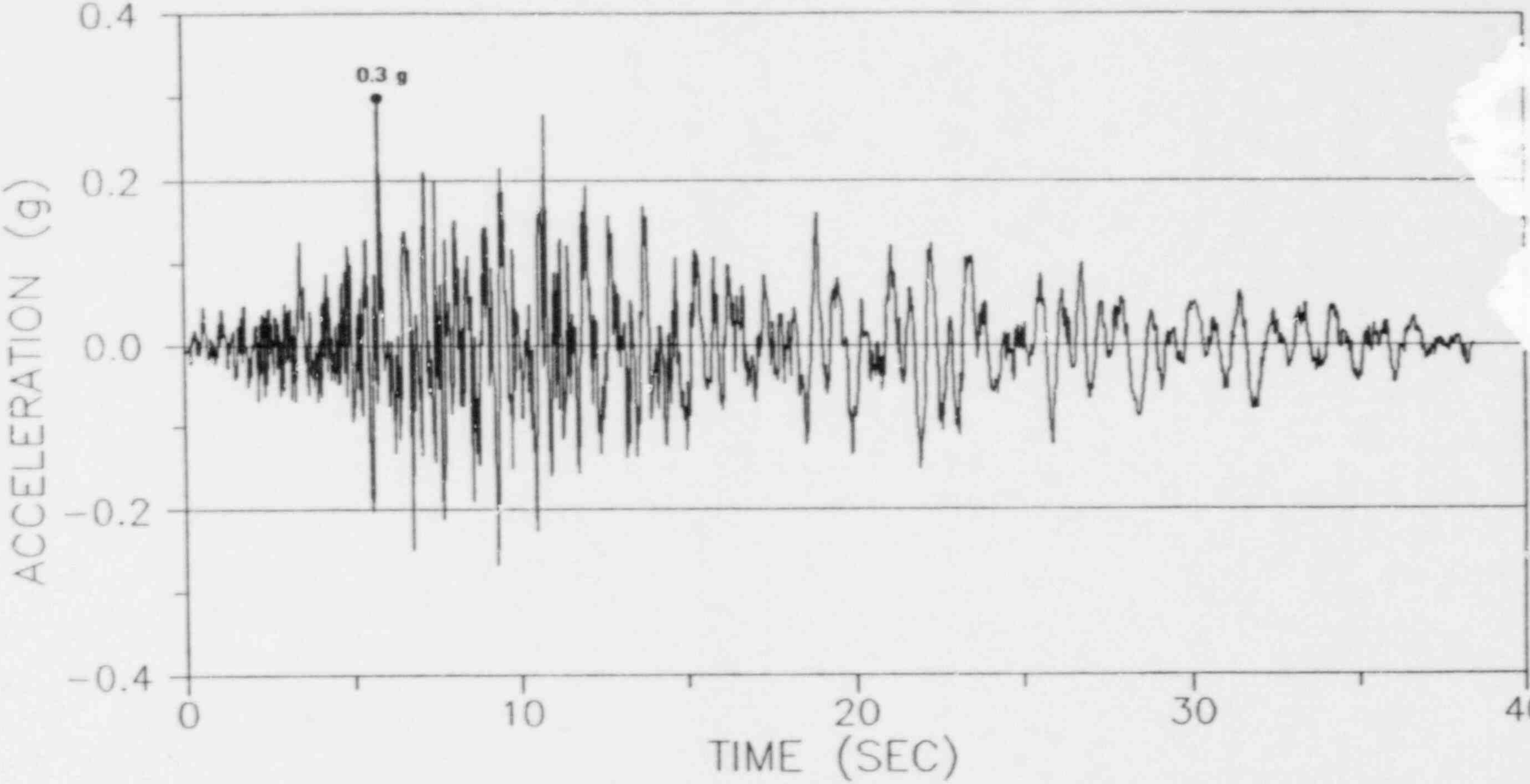
WOODWARD-CLYDE CONSULTANTS



Based on NUREG 0098
Spectral Damping = 0.05

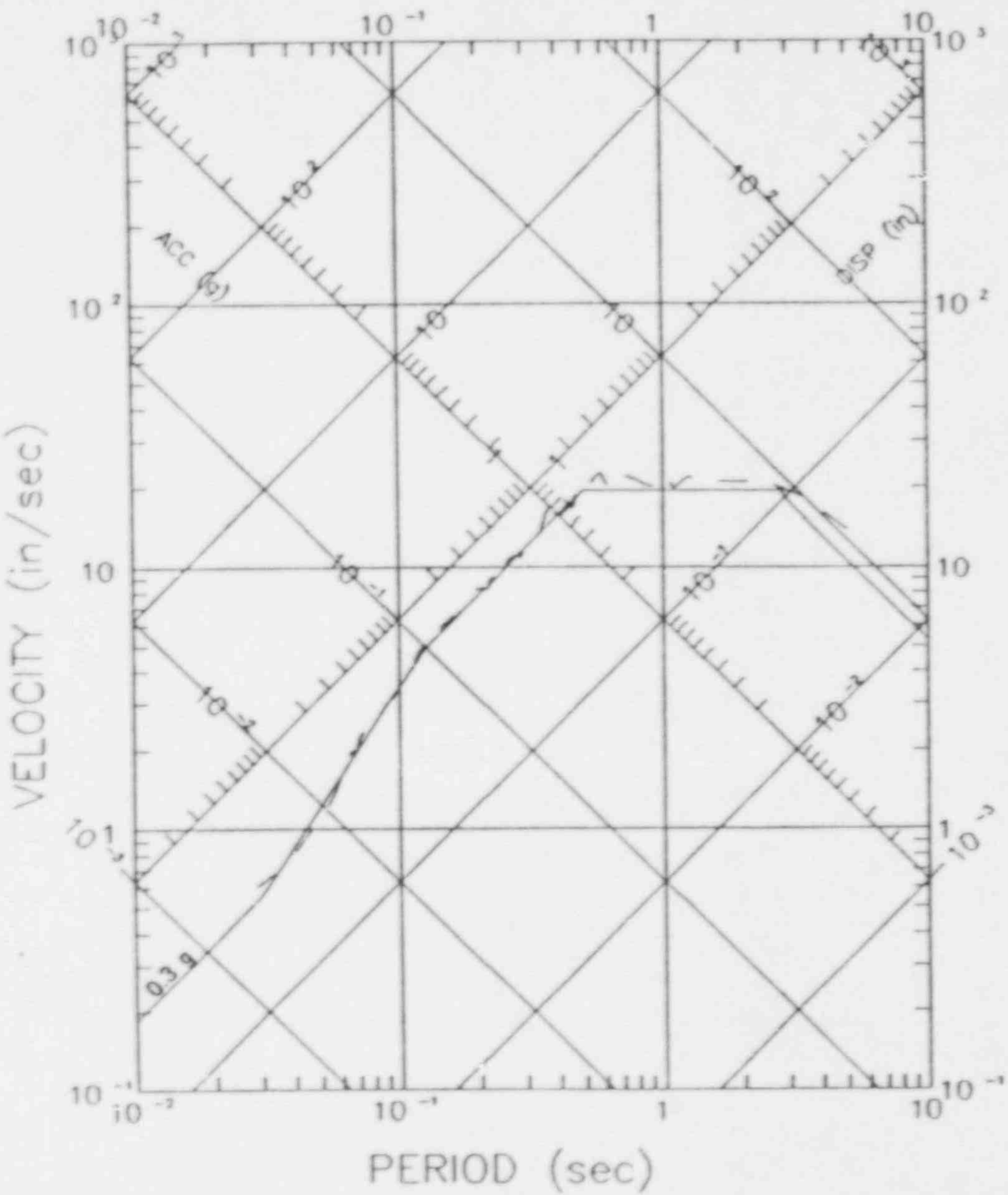
Project	HATCH NP	SMOOTH RESPONSE SPECTRUM FOR SELECTED SME	Fig
Project No	8743076A		8

WOODWARD-CLYDE CONSULTANTS



Project	HATCH NP	SYNTHENIC ACCELEROGRAM FOR SELECTED SME	Fig
Project No	8743076A		9

WOODWARD-CLYDE CONSULTANTS



LEGEND

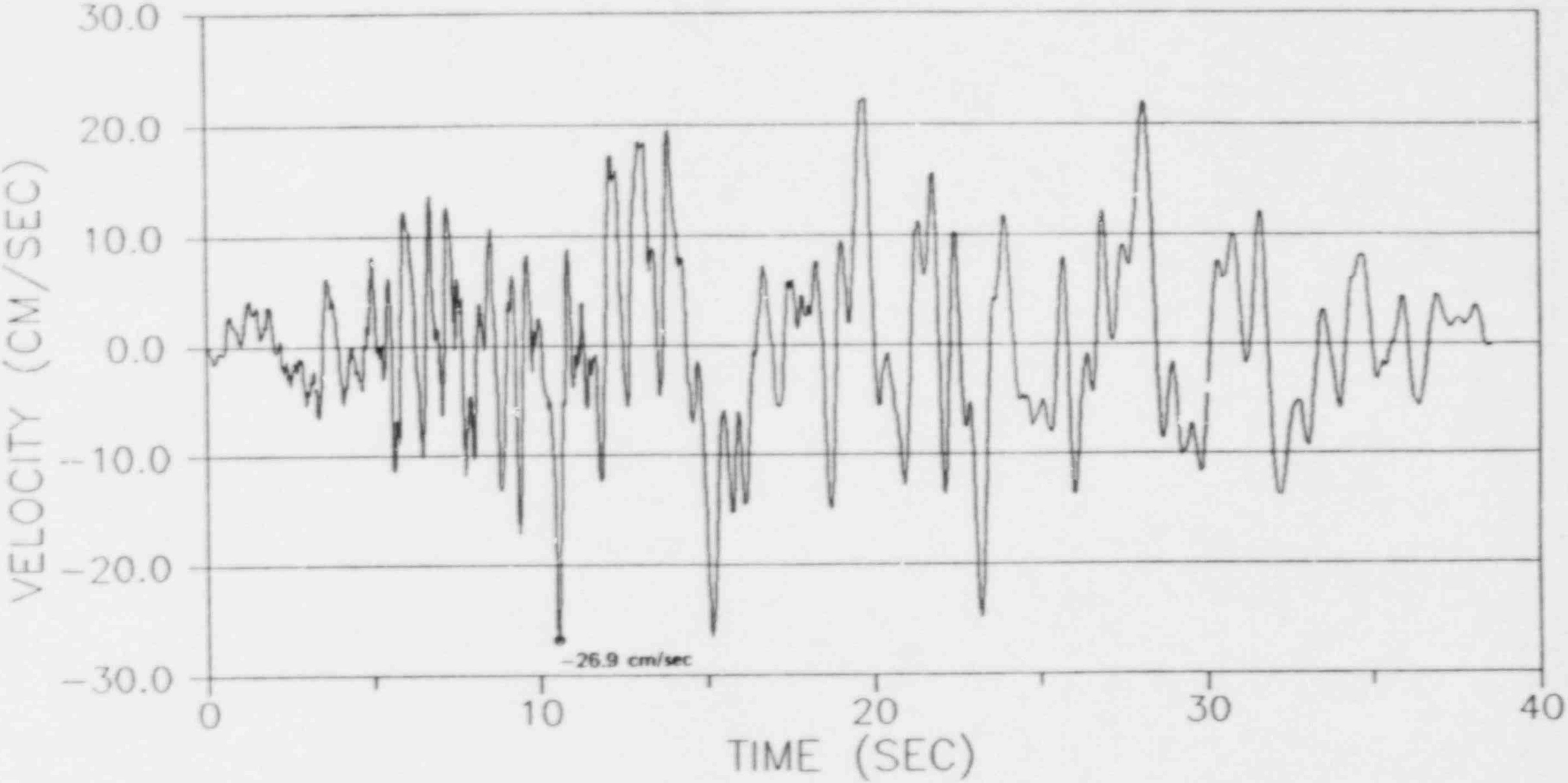
—— Smooth Spectrum for SME

- - - - Spectrum for Synthetic Accelerogram

Spectral Damping = 0.05

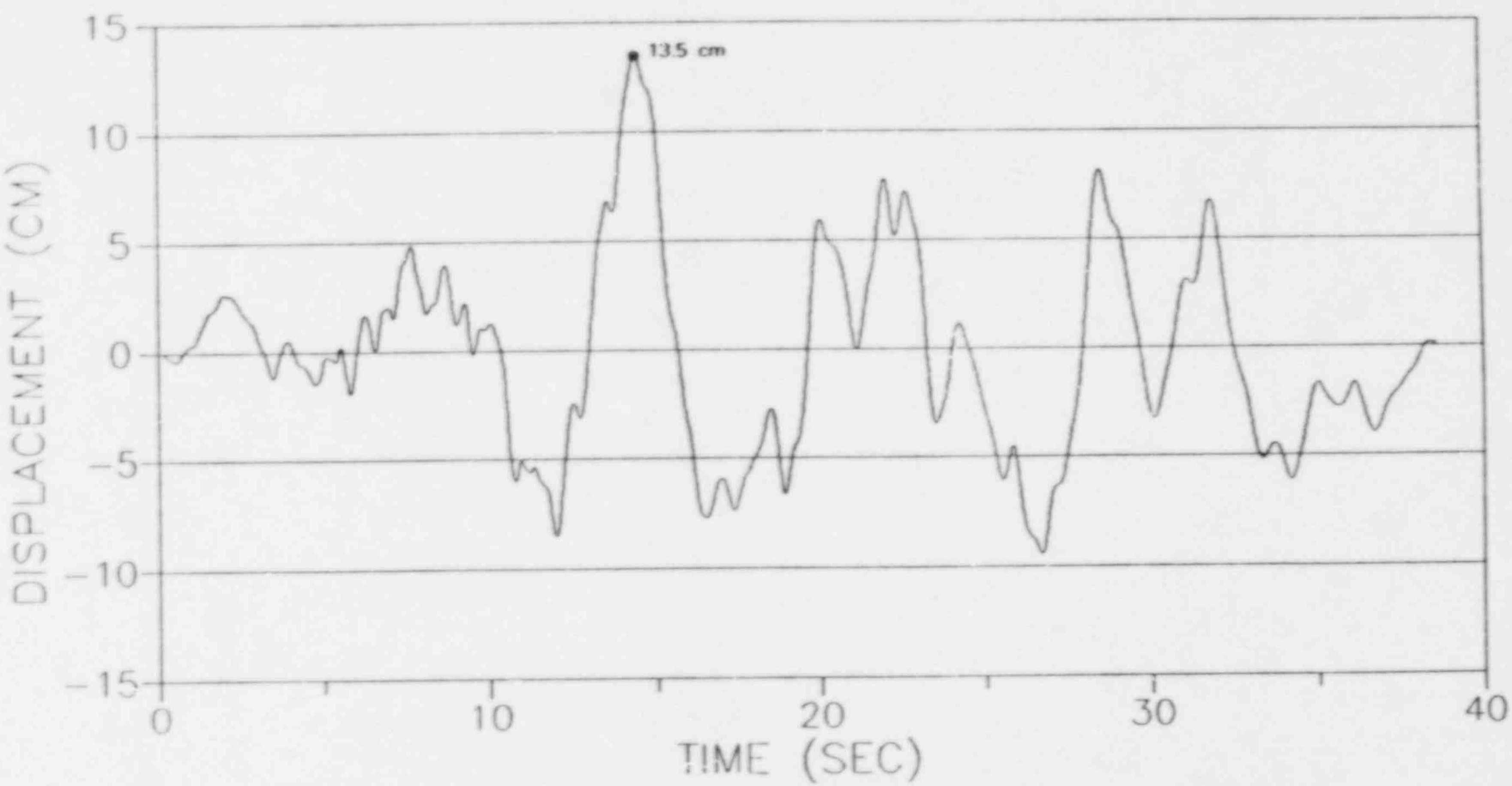
Project	HATCH NP	COMPARISON OF SMOOTH SPECTRUM WITH SPECTRUM FOR SYNTHETIC ACCELEROGRAM	Fig
Project No	8743076A		10

WOODWARD-CLYDE CONSULTANTS



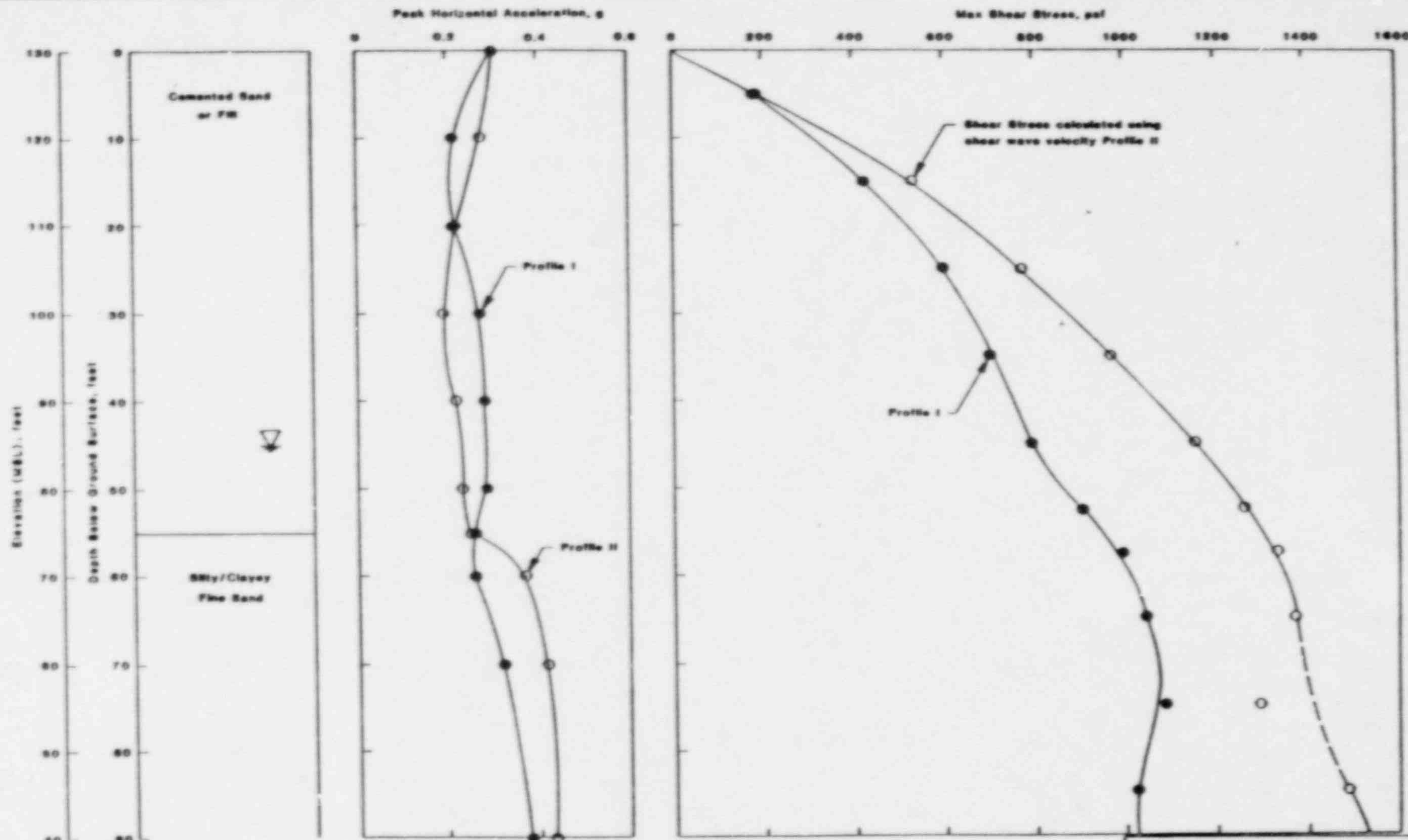
Project	HATCH NP	CALCULATED VELOCITY FOR SYNTHETIC ACCELEROGRAM	Fig. 11
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS



Project	HATCH NP	CALCULATED DISPLACEMENT FOR SYNTHETIC ACCELEROGRAM	Fig. 12
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS



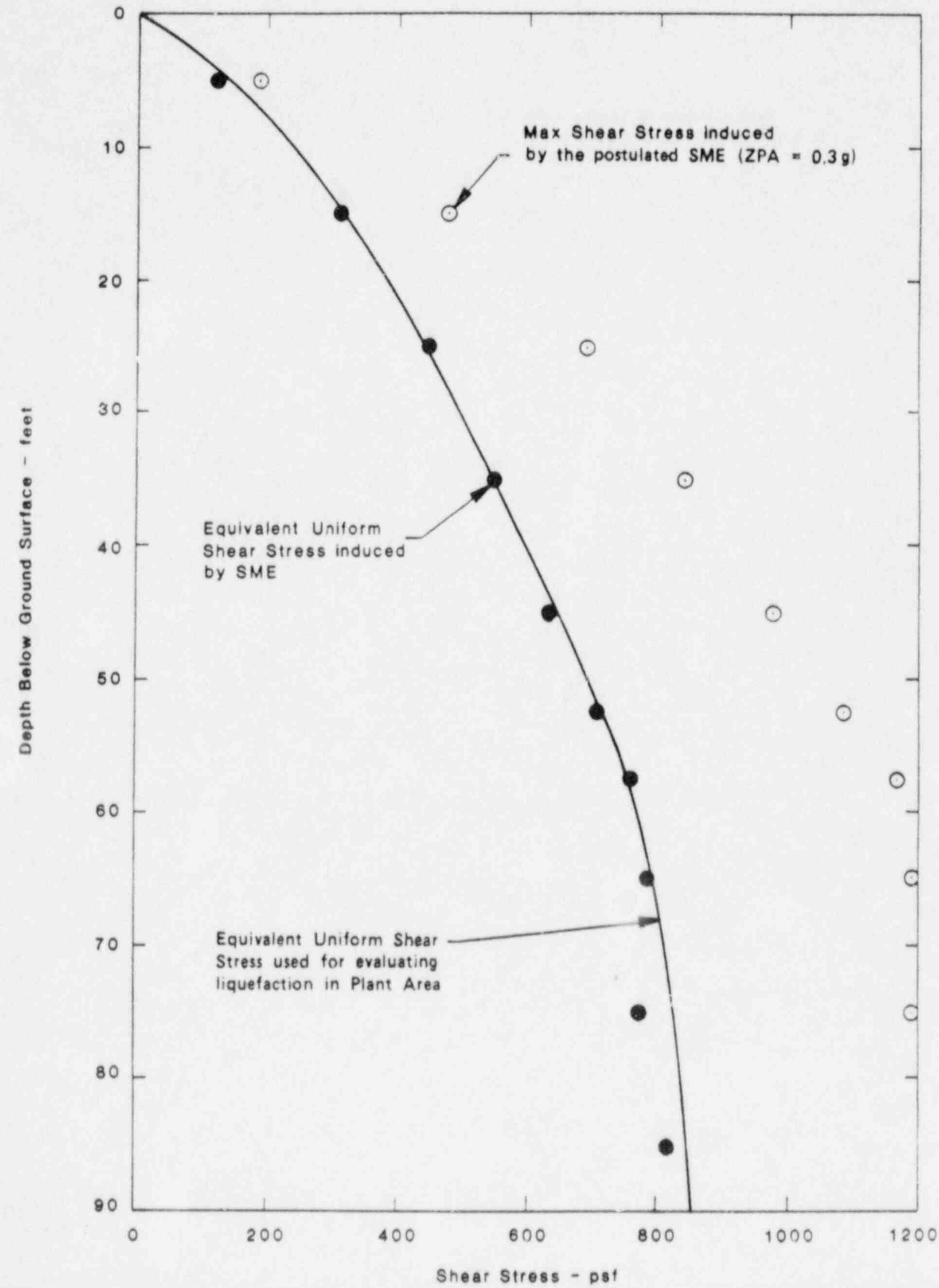
Woodward-Clyde Consultants

RANGE OF CALCULATED ACCELERATIONS AND SHEAR STRESSES - PLANT AREA

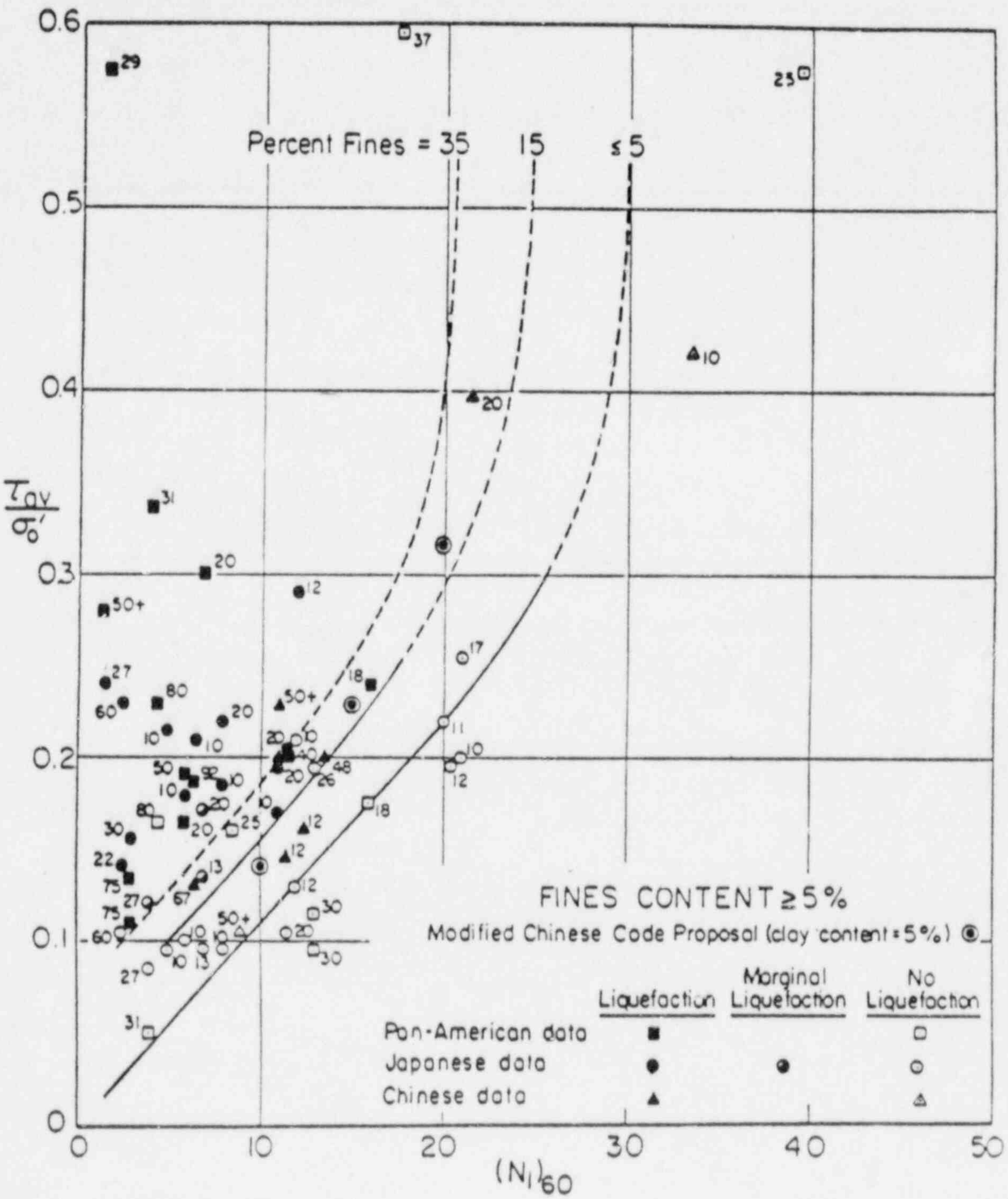
Project No. 8743076A

HATCH NP

Fig. 13

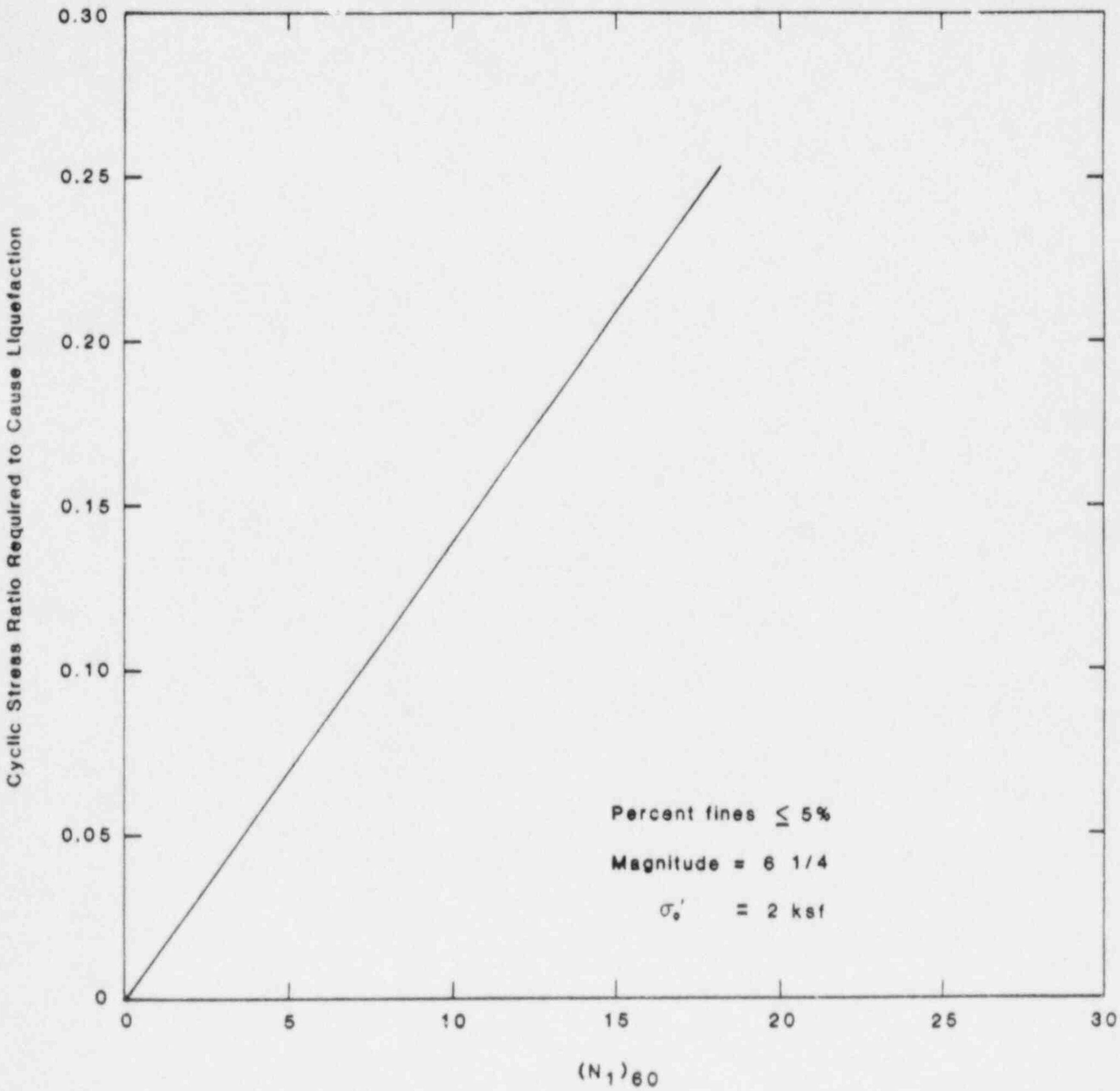


Project	HATCH NP	MAX AND EQUIVALENT UNIFORM STRESSES USED IN LIQUEFACTION EVALUATION-PLANT AREA	Fig
Project No	8743076A		14

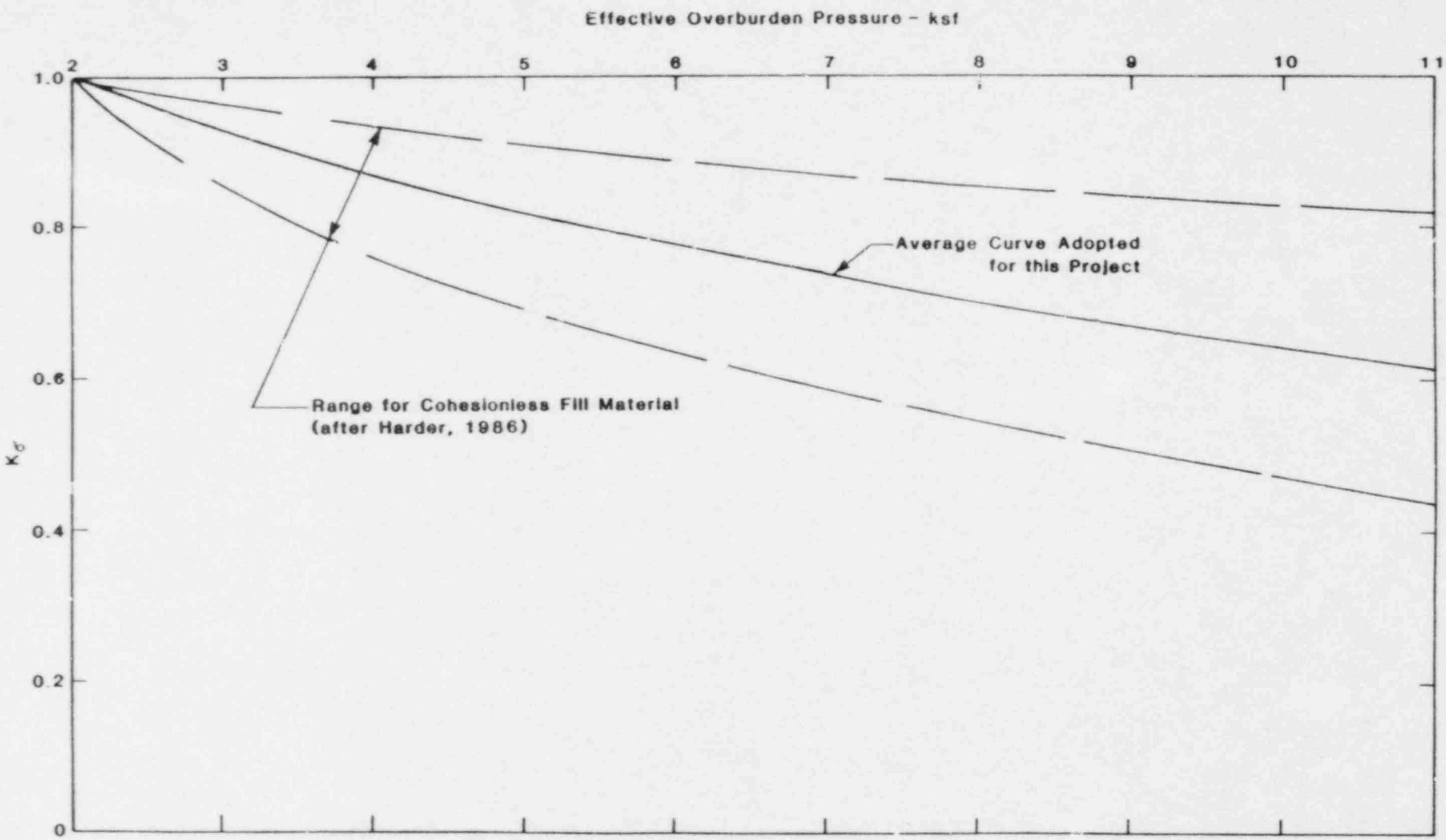


From Seed et al (1984)

Project	HATCH MP	CYCLIC STRESS RATIO VERSUS $(N_1)_{60}$	Fig
Project No	8743076A	M = 7 1/2	15

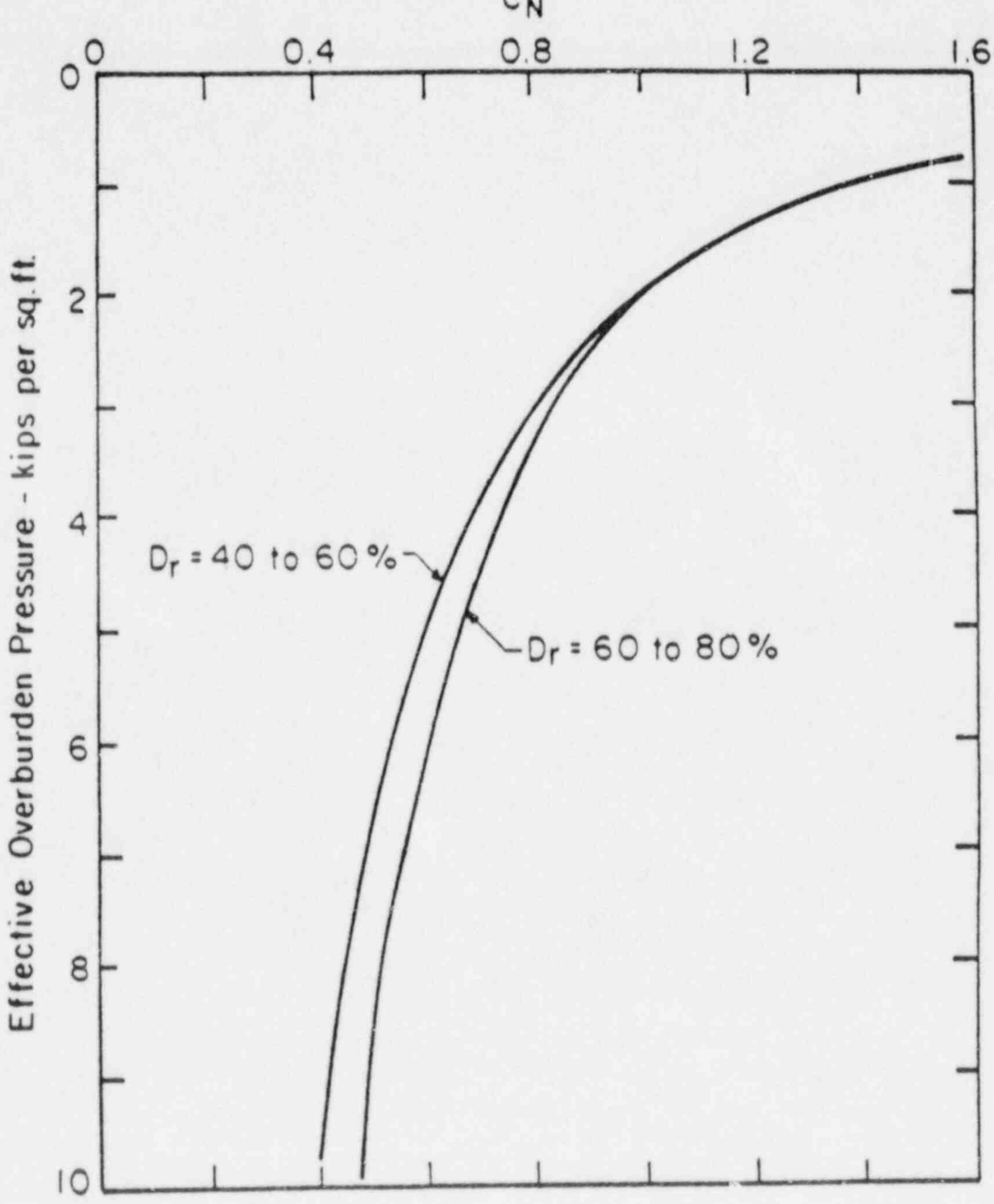


Project	HATCH NP	CYCLIC STRESS RATIO FOR m = 6 1/4	Fig. 16
Project No.	3743076A		



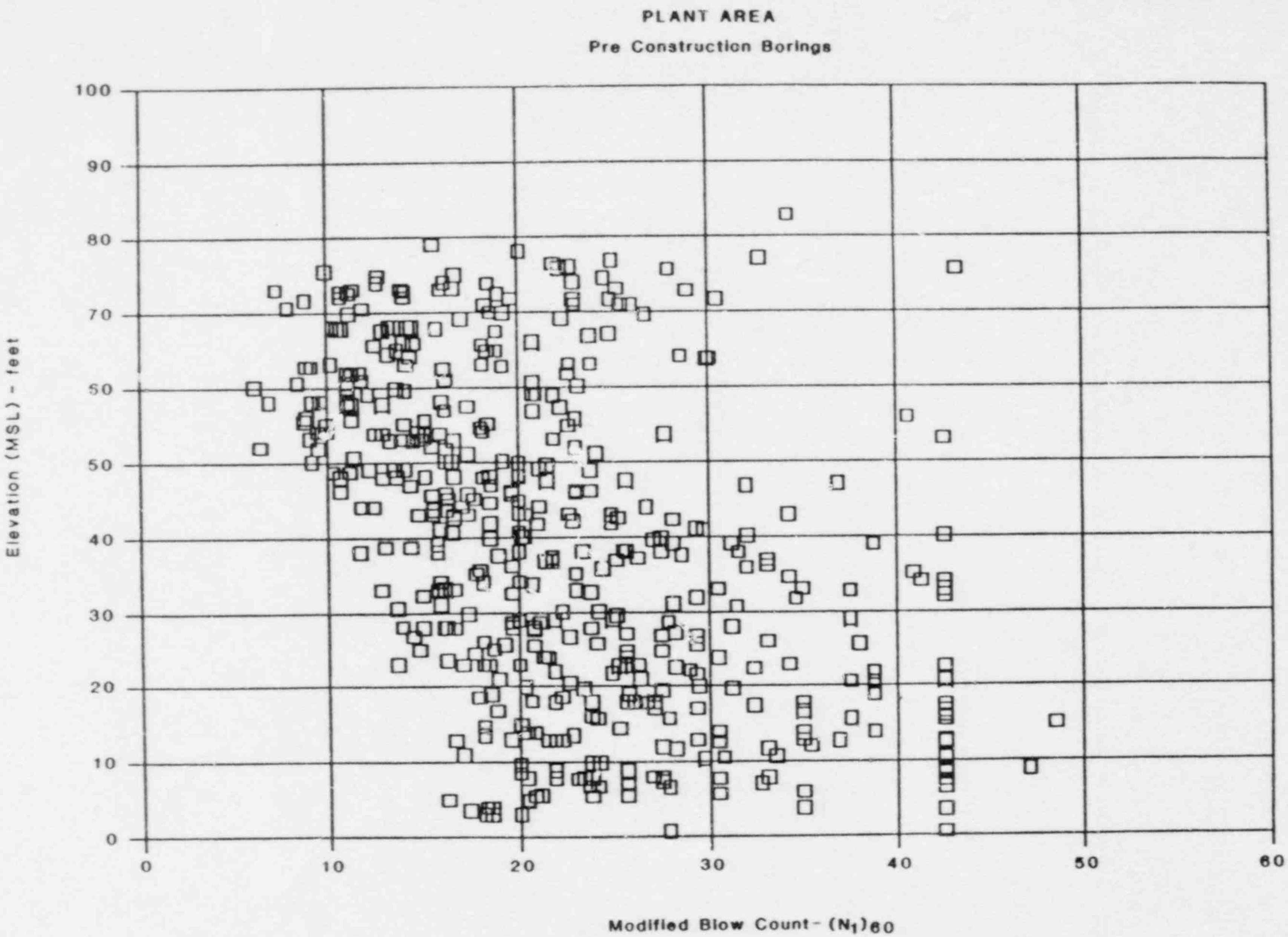
Project:	HATCH NP	EFFECTS OF INITIAL CONFINING PRESSURE ON CYCLIC STRESS RATIO	Fig.
Project No.	8743076A		17

WOODWARD-CLYDE CONSULTANTS



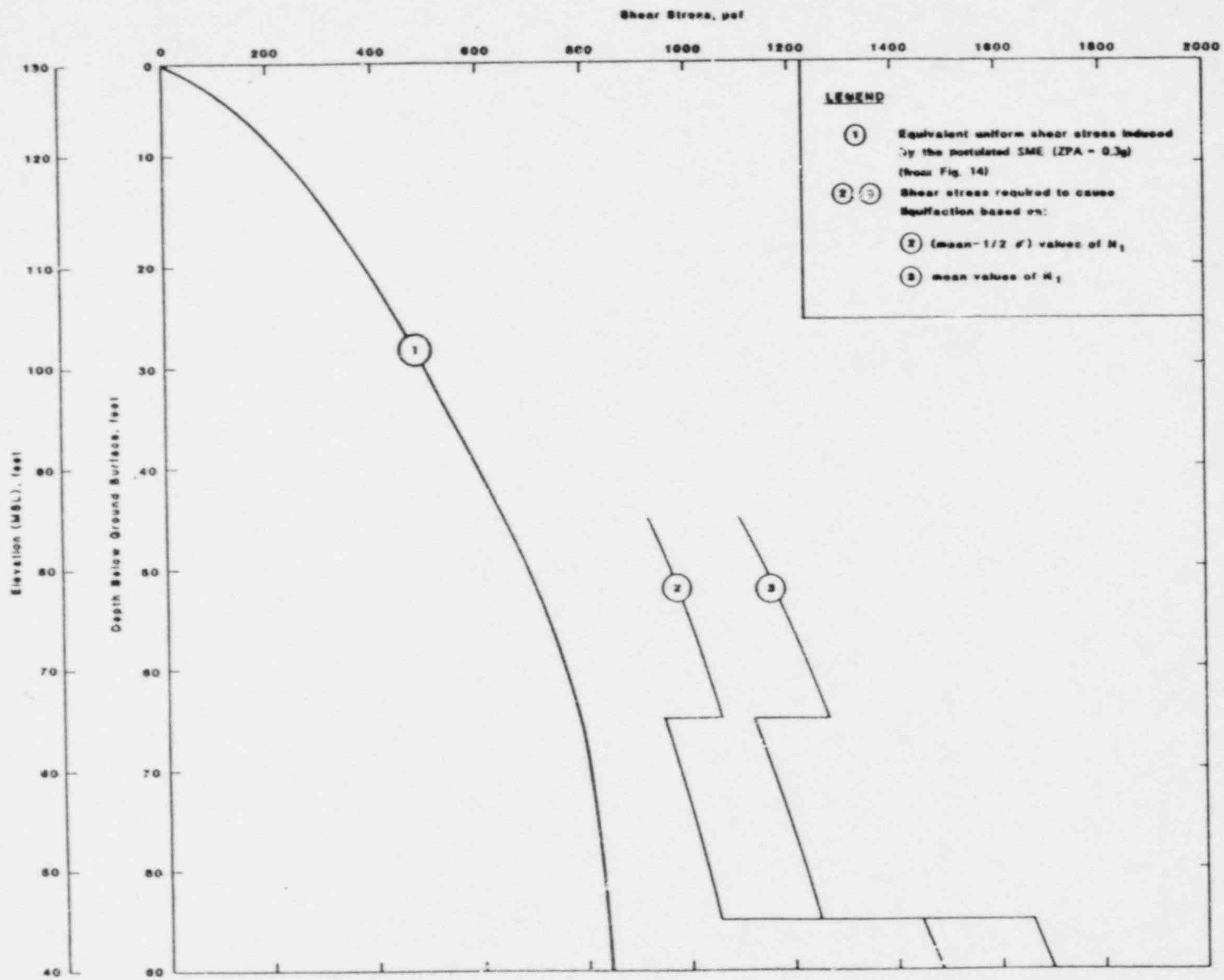
(After Marcuson and Bieganousky, 1977)

Project	HATCH NP	CHART FOR VALUES OF C_N	Fig
Project No	8743076A		18



Project:	HATCH NP	SUMMARY OF SPT BLOW COUNT - PLANT AREA	Fig. 19
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS



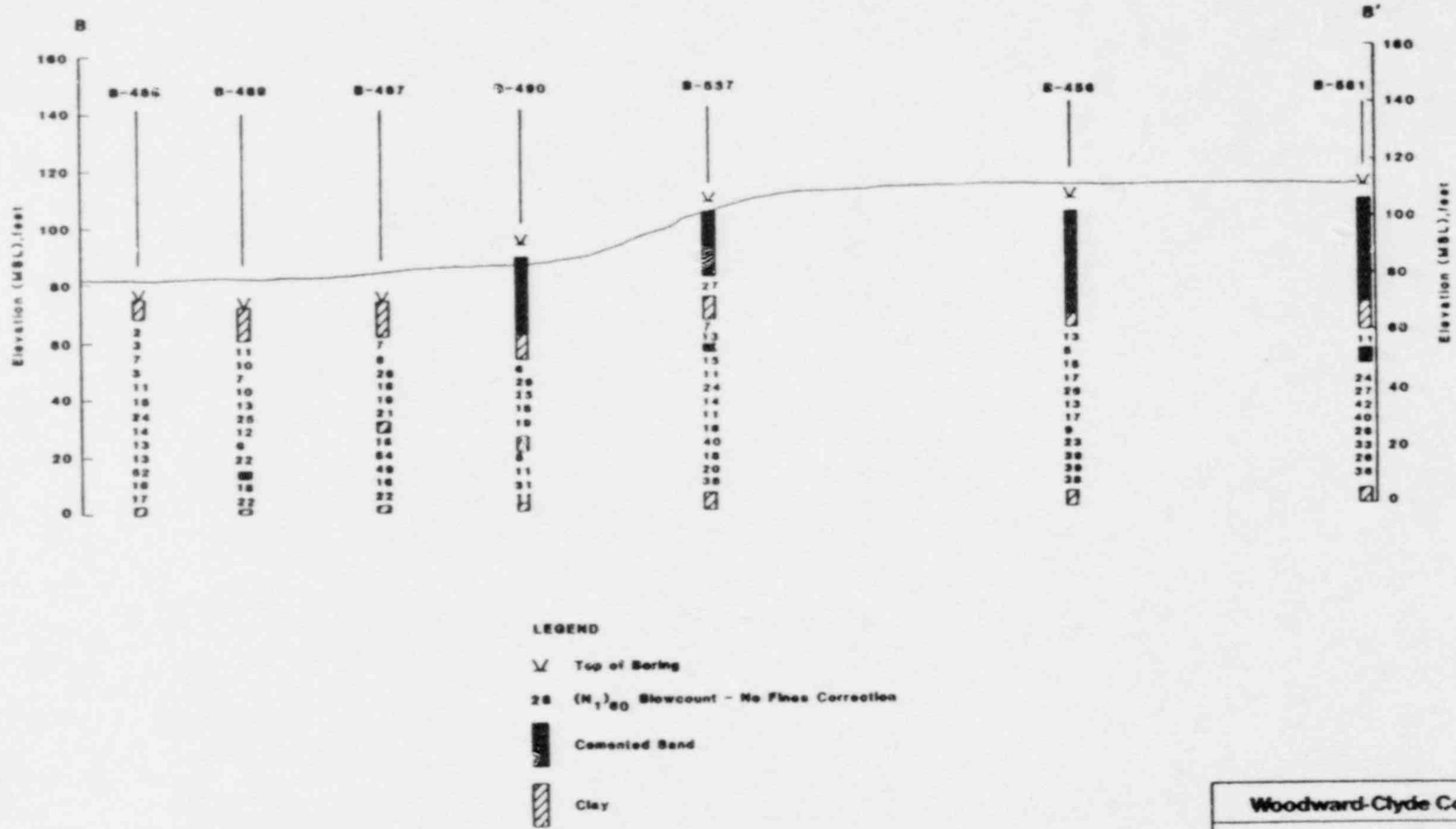
Woodward-Clyde Consultants

COMPARISOM OF INDUCED STRESSES WITH STRESSES REQUIRED TO CAUSE LIQUEFACTION - PLANT AREA

Project No. 8743076A

HATCH NP

Fig. 20



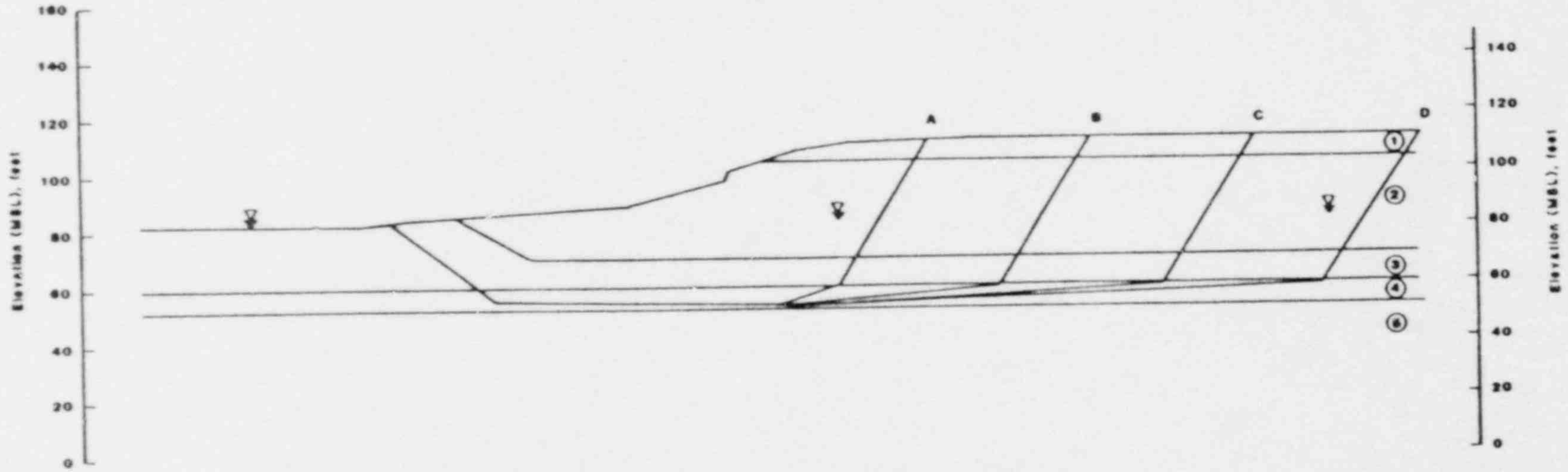
Woodward-Clyde Consultants

SPT BLOW COUNT - SECTION B-B'

Project No. 8743076A

HATCH NP

Fig. 21

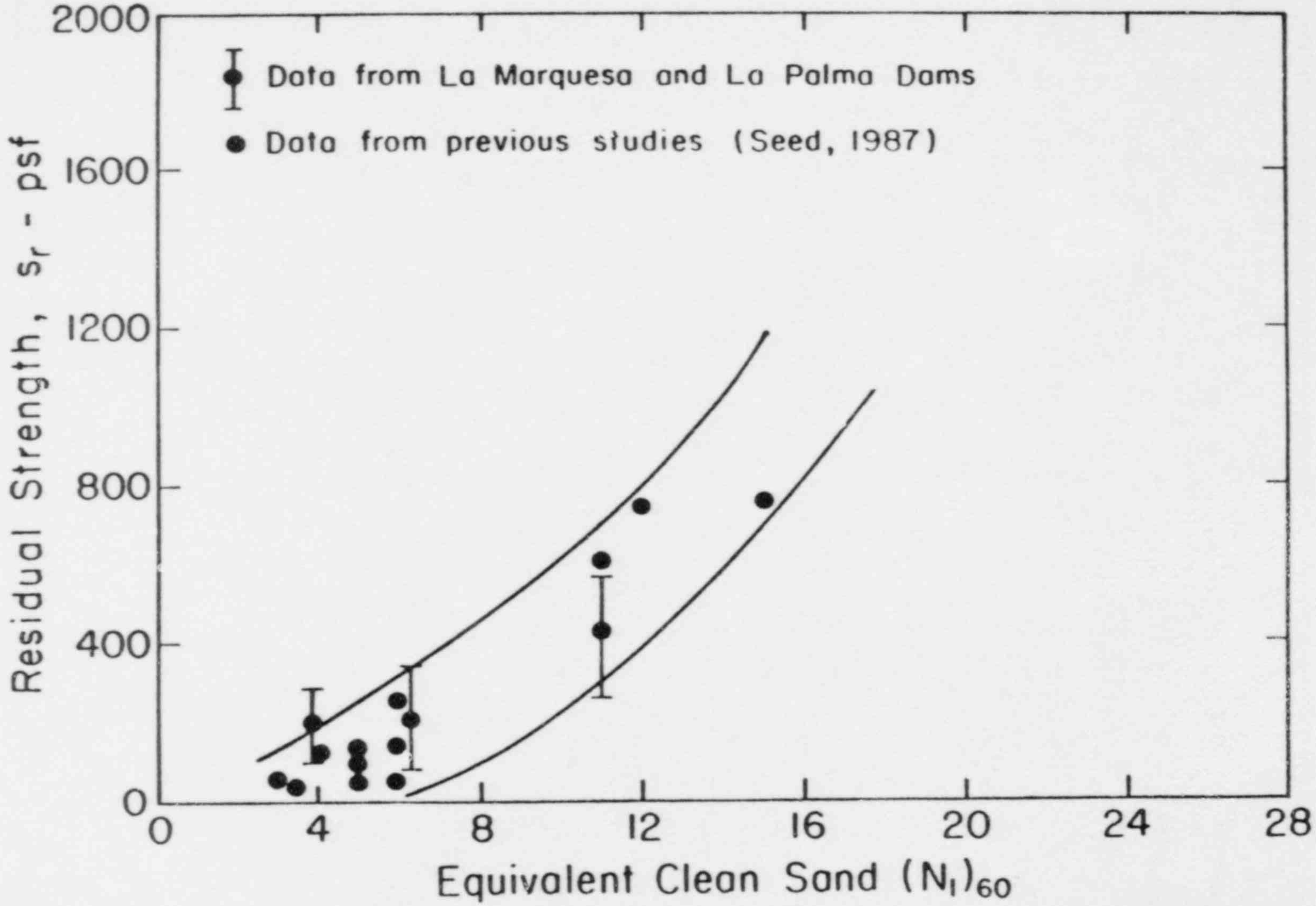


Woodward-Clyde Consultants

POTENTIAL SLIP SURFACES A, B, C and D
SECTION B-B'

Project No. 8743076A
HATCH NP

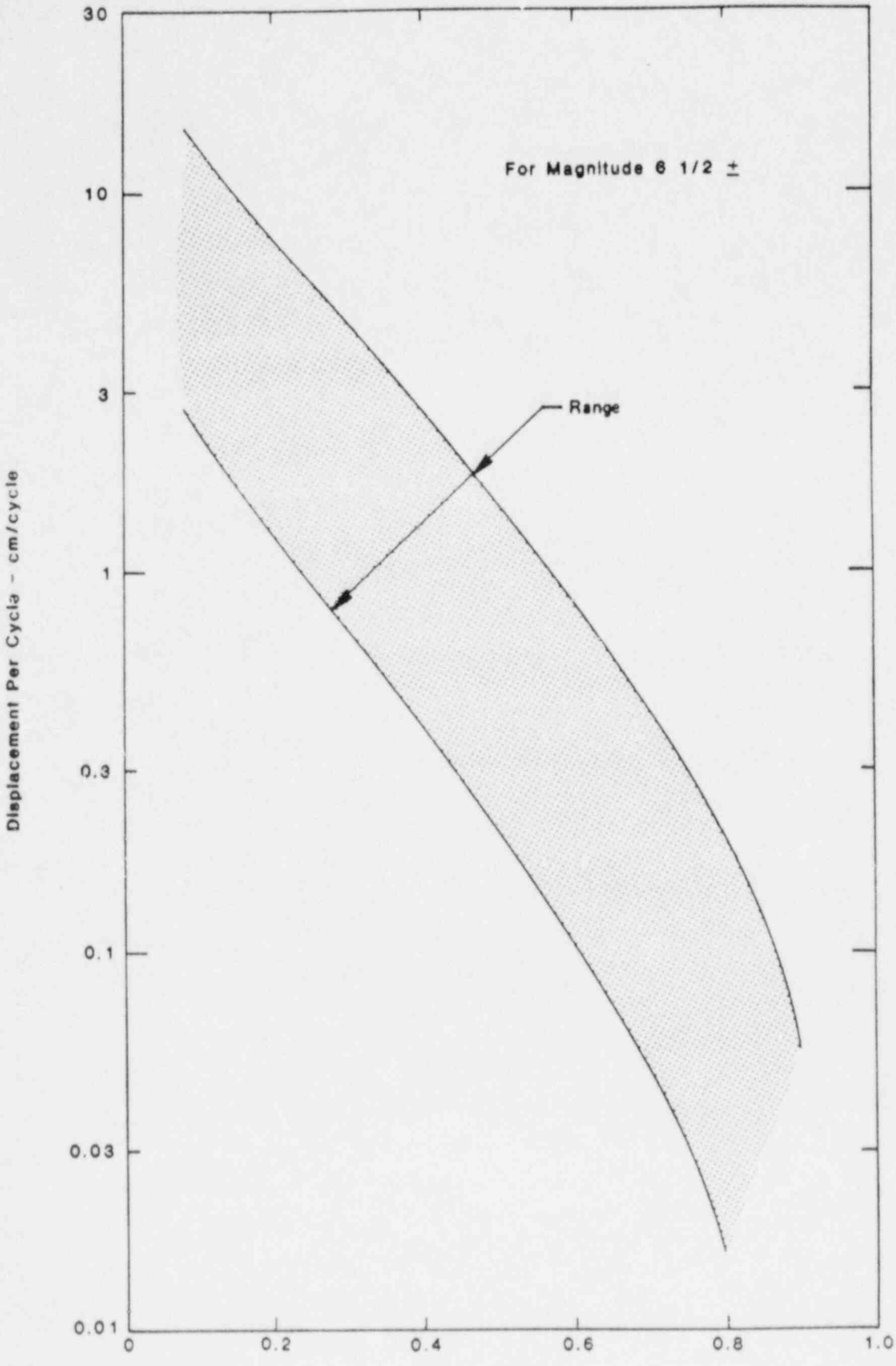
Fig. 22



(From Seed et al, 1987)

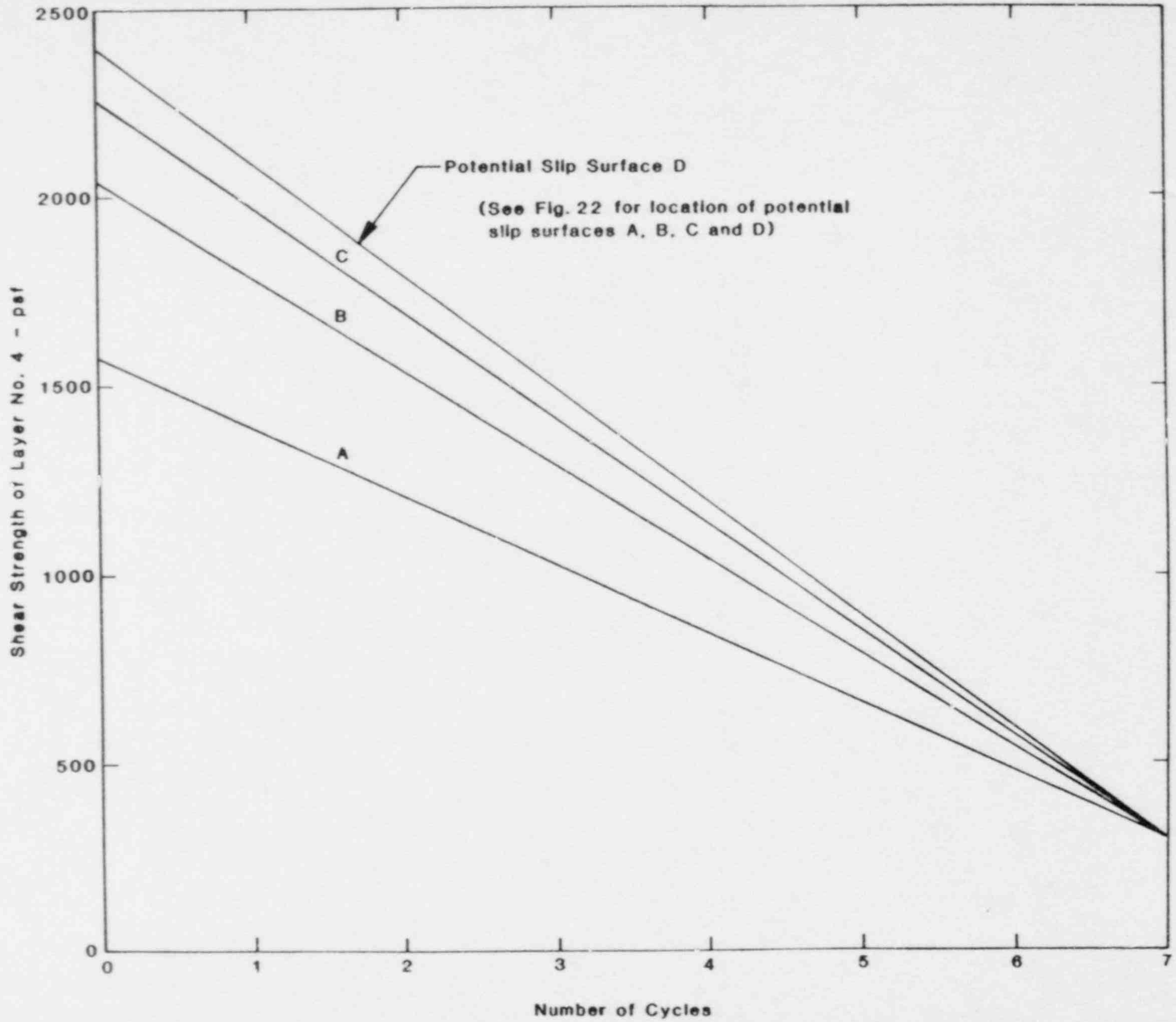
Project:	HATCH NP	RELATIONSHIP BETWEEN RESIDUAL STRENGTH AND EQUIVALENT CLEAN SAND VALUE OF $(N_1)_{60}$	Fig. 23
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS



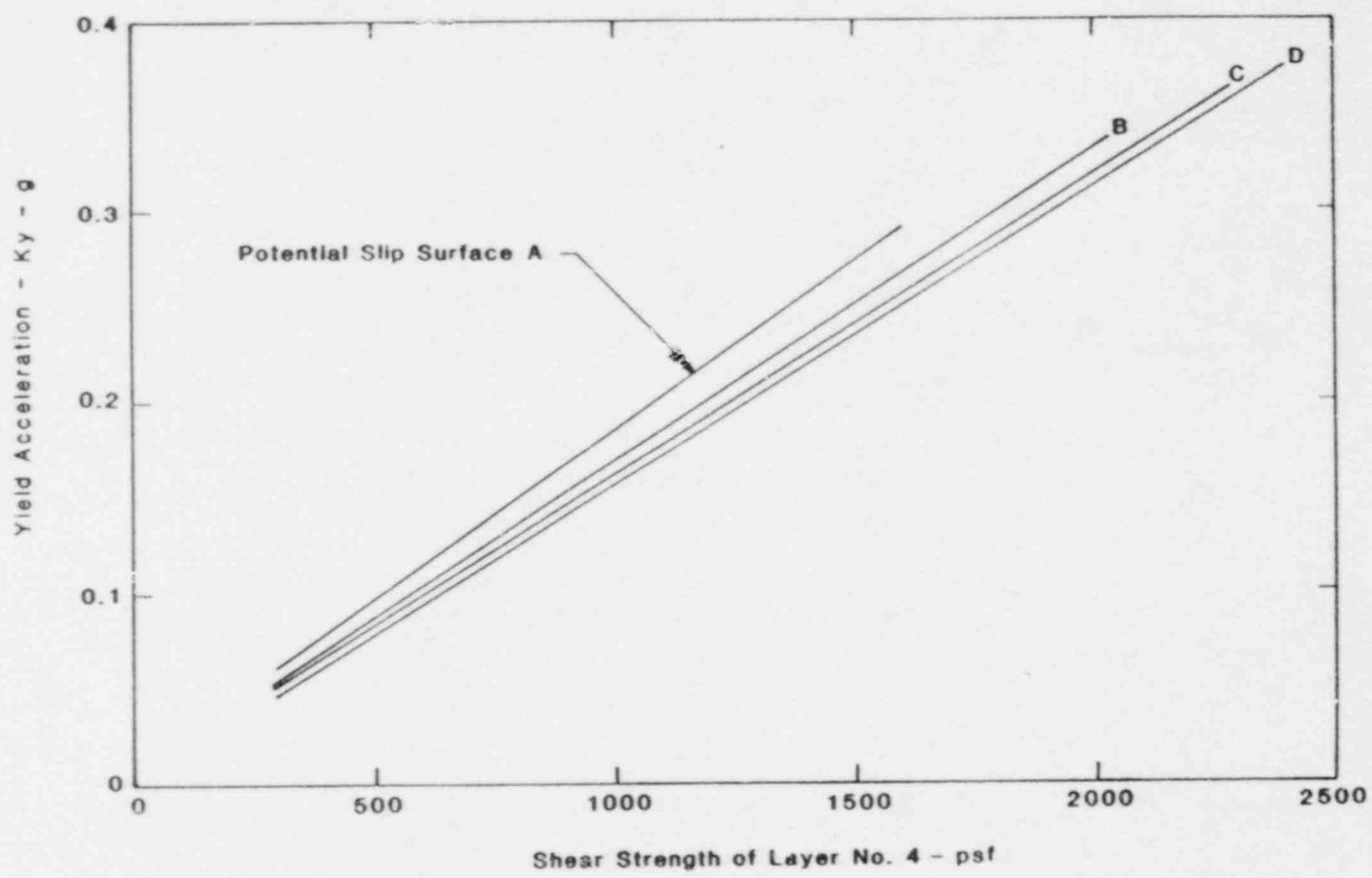
(After Makdisi and Seed, 1978)

Project	HATCH NP	SEISMICALLY-INDUCED DISPLACEMENT PER CYCLE	Fig 24
Project No	8743076A		



Project	HATCH NP	VARIATION OF SHEAR STRENGTH WITH NUMBER OF CYCLES IN LAYER NO. 4 - SECTION B-B'	Fig. 25
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS



Project	HATCH NP	YIELD ACCELERATION VS. SHEAR STRENGTH IN LAYER NO. 4 - SECTION B-B'	Fig. 26
Project No	8743076A		

WOODWARD-CLYDE CONSULTANTS

APPENDIX A

DYNAMIC SOIL PROPERTIES FOR SSI ANALYSES

APPENDIX A
DYNAMIC SOIL PROPERTIES FOR
SSI ANALYSES

A.1 INTRODUCTION

This appendix presents the dynamic soil properties for use in Soil-Structure Interaction (SSI) analyses in the Plant Area and in the Water Intake Area.

A.2 PLANT AREA

The strain-compatible soil dynamic properties obtained from the response analyses are listed in Tables A-1 and A-2. Table A-1 provides listings of these soil properties considering shear wave velocity Profile I (see Fig. 3). The results obtained using the average damping curve for sands are presented in this table. Corresponding results are presented in Table A-2 considering shear wave velocity Profile II. It may be noted that lower soil damping values were also used based on the lower range damping curve for sands. Almost identical shear moduli (and shear stresses) to those listed in Table A-1 for Profile I and in Table A-2 for Profile II were obtained with the lower soil damping values.

It is our understanding that SSI analyses to be conducted by EQE will be able to handle variations in soil properties both in the vertical and in the horizontal directions. Accordingly, it is recommended that the following modulus values be used in the SSI analyses:

Reactor Building - From the ground surface to a depth of 55 feet, it is recommended that modulus values corresponding to Profile I (Table A-1) be used adjacent to the reactor building and to a horizontal distance of about 75 feet on either side of the reactor building. Modulus values corresponding to Profile II (Table A-2) should be used within this depth and beyond the distance of 75 feet. Below a depth of 55 feet, it is recommended that the average of the modulus values listed in Tables A-1 and A-2 be used.

The above values represent best estimates at this time. It is recommended that analyses be conducted using these best estimates values and additional analyses be conducted using 1.5 times these values and 0.75 times these values.

Control Building - This building is founded essentially at the ground surface on the native cemented soils. Therefore, the modulus values listed in Table A-2 should be used for this building as best estimate values. Additional analyses using 1.5 times the best estimate values and then using 0.60 times the best estimate values should be conducted.

Diesel Generator Building - This building is founded essentially at the ground surface; however, there is no readily available information regarding the extent of fill, if any, beneath this building. Therefore, as a best estimate, it is recommended that the modulus values given for Profile I (Table A-1) be used in the analysis representing the best estimate values. Additional analyses using 2.5 times the best estimate values and those using 0.8 times the best estimate values should be conducted.

A total unit weight of 125 pcf, a Poisson's ratio of 0.35 are recommended for both Profiles I and II. It may be noted that either the damping values listed in Tables A-1 and A-2 can be used directly in the SSI analyses or an average constant damping value may be selected.

A.3 **WATER INTAKE AREA**

The total unit weight and Poisson's ratio given for the Plant Area are also recommended for the Water Intake Area. The best estimated shear moduli and damping ratio are listed below:

Depth Range (Below El. 110)	Shear Modulus	Damping Ratio
Upper 40 feet	G = 3,600 ksf	7%
From 40 to 80 feet	G = 900 ksf	15
From 80 to 120 feet	G = 1,600 ksf	13
Below 120 feet	G = 7,000 ksf	8

Again, it is recommended that variations of +50% (ie, 1.5 times the moduli listed above) and -25% (ie, 0.75 times) in these modulus values be used in additional SSI analyses.

The above values were estimated taking into account the variations of subsurface conditions in the vicinity of the water-intake area (see Fig. 21) and the fact that part of the soils adjacent to the structure were replaced with K-Krete fill to depths ranging from 20 to about 50 feet below grade. The available information on this K-Krete fill suggests that its modulus values may be reasonably represented by that of the cemented soils in the Plant Area.

TABLE A-1

Shear Wave Velocity Profile I
Strain-Compatible Modulus & Damping Parameters -- Plant Area

Depth From	Depth To	Shear Wave Vel	Shear Modulus	Damping Ratio
0	10	483	903	7.0
10	20	444	763	10.6
20	30	444	763	12.3
30	40	467	844	12.6
40	50	497	955	12.5
50	55	509	1,002	12.9
55	60	509	1,003	13.3
60	70	529	1,082	13.4
70	80	562	1,224	13.1
80	90	617	1,474	12.0
90	105	683	1,806	10.7
105	120	724	2,029	10.9
120	130	764	2,261	10.8
130	140	1,322	6,764	7.1
140	160	1,360	7,162	7.3
160	180	1,409	7,685	7.5
180	200	1,476	8,427	7.5
200	230	1,570	9,537	7.3

Notes:
1. Depth in feet below the ground surface.
2. Shear Wave Velocity in feet per sec.
3. Shear Modulus in kips per square foot.
4. Damping in percent of critical damping (using average damping curve for sands).

TABLE A-2

Shear Wave Velocity Profile II
Strain-Compatible Modulus & Damping Parameters -- Plant Area

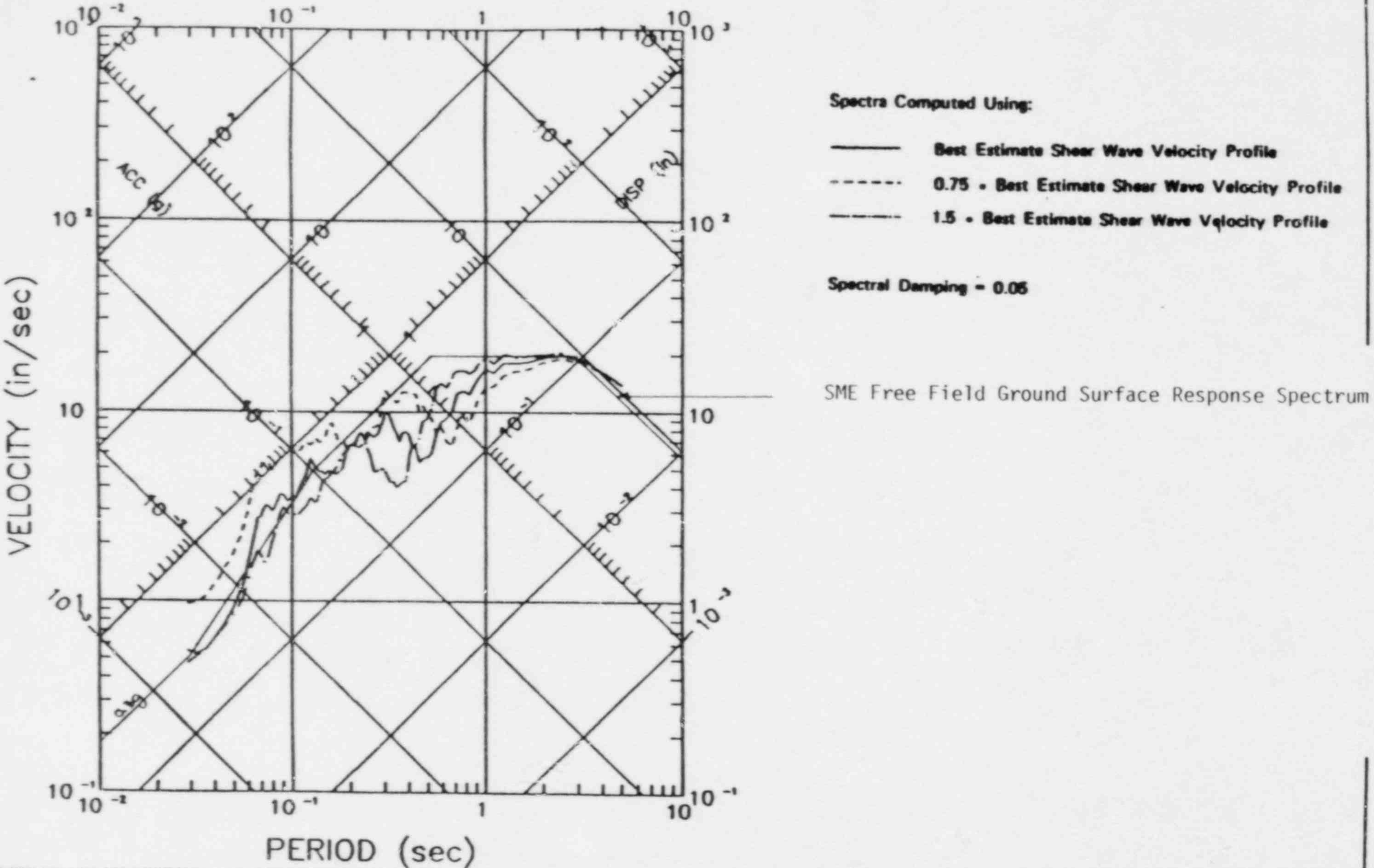
Depth From	Depth To	Shear Wave Vel	Shear Modulus	Damping Ratio
0	10	1,125	4,894	2.9
10	20	1,034	4,137	5.6
20	30	982	3,735	7.0
30	40	936	3,392	8.2
40	50	900	3,137	9.0
50	55	882	3,009	9.5
55	60	433	727	16.5
60	70	461	822	16.1
70	80	520	1,046	14.7
80	90	543	1,139	14.8
90	105	582	1,312	14.5
105	120	655	1,658	13.4
120	130	703	1,912	12.9
130	140	1,264	6,186	8.1
140	160	1,278	6,318	8.7
160	180	1,317	6,708	9.1
180	200	1,374	7,305	9.1
200	230	1,453	8,175	9.1

Notes:

1. Depth in feet below the ground surface.
2. Shear Wave Velocity in feet per sec.
3. Shear Modulus in kips per square foot.
4. Damping in percent of critical damping (using average damping curve for sands).

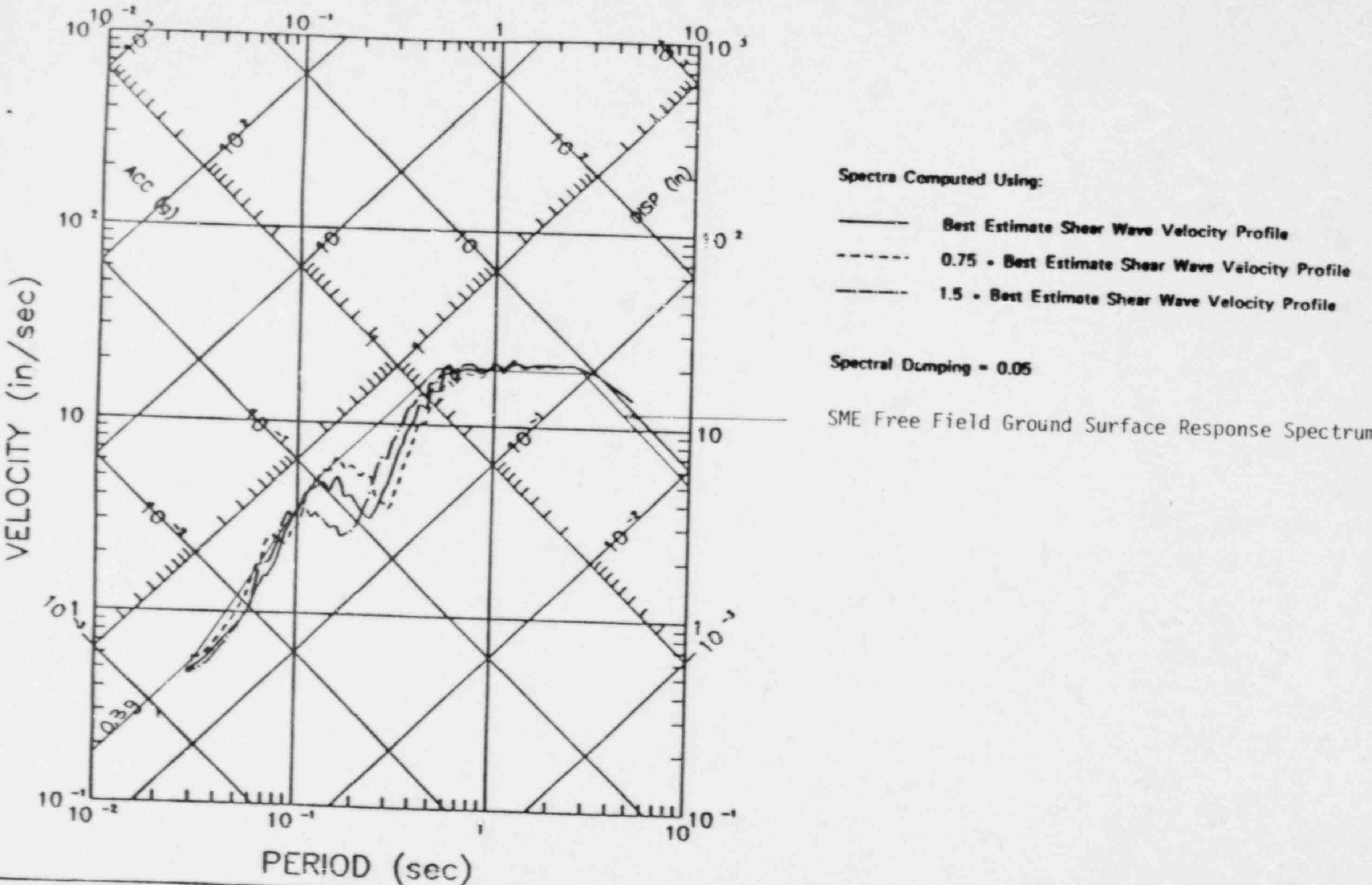
ENCLOSURE 2

COMPUTED RESPONSE SPECTRA FOR SOIL PROFILES



Project: HATCH NP SEISMIC MARGIN		COMPUTED RESPONSE SPECTRA AT FOUNDATION LEVEL – CASE I - Profile I	Fig.
Project No. 8743076A			

WOODWARD-CLYDE CONSULTANTS



Project: HATCH NP SEISMIC MARGIN Project No. 8743076A	COMPUTED RESPONSE SPECTRA AT FOUNDATION LEVEL – CASE II - Profile II	Fig.

WOODWARD-CLYDE CONSULTANTS

ENCLOSURE 3

DOCUMENT ENTITLED "E. I. HATCH NUCLEAR PLANT - UNIT 1
SEISMIC MARGIN ASSESSMENT (SMA) SOIL -
STRUCTURE INTERACTION ANALYSIS"