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Reassessment of Liquefaction Potential and Estimation of Earthquake-Induced Settlements at Paducah Gaseous Diffusion Plant, Paducah, Kentucky

by David W. Sykora, Donald E. Yule

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Prepared for U.S. Department of Energy

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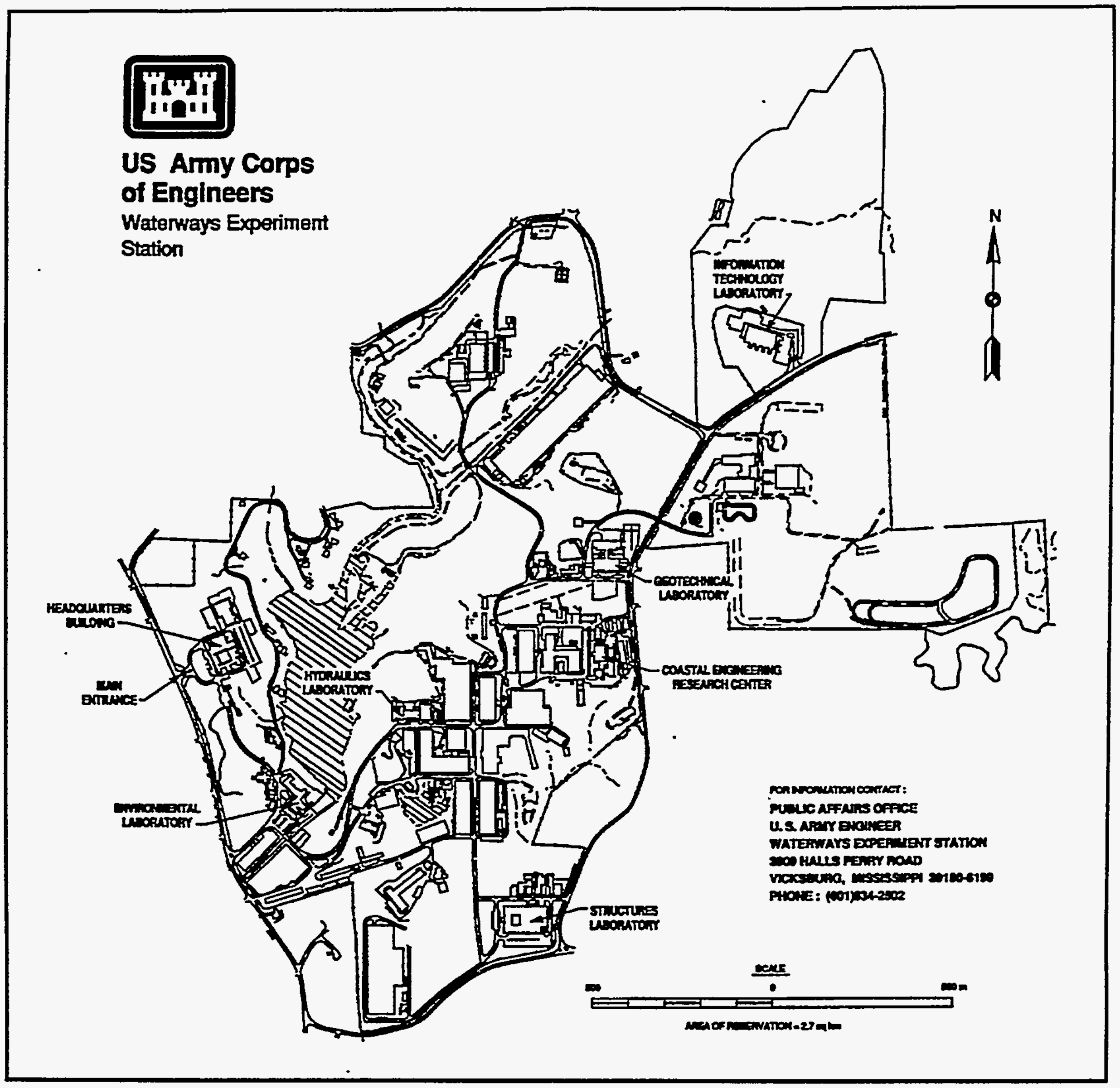

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PREFACE

This report documents a reassessment of liquefaction potential and estimation of earthquake-induced settlements for the U.S. Department of Energy (DOE), Paducah Gaseous Diffusion Plant (PGDP), located southwest of Paducah, KY. The U.S. Army Engineer Waterways Experiment Station (WES) was authorized to conduct this study from FY91 to FY94 by the DOE, Oak Ridge Operations (ORO), Oak Ridge, TN, through Inter-Agency Agreement (IAG) No. DE-AI05-910R21971. The study was conducted under the Gaseous Diffusion Plant Safety Analysis Report (GDP SAR) Program.

The IAG was managed for Martin Marietta Energy Systems, Inc., by Ms. Karen E. Shaffer, Uranium Enrichment, Martin Marietta Energy Systems, Inc., Oak Ridge, TN. Mr. William R. Brock, GDP SAR Engineering Manager, Technical Operations, and Mr. R. Joe Hunt, Center for Natural Phenomena Engineering, Technical Operations, provided technical requirements and oversight for the study. The overall program manager was Mr. Anthony Angelelli, Uranium Enrichment. A previous study of site response analysis has been reported under separate cover.

The WES Principal Investigator was Mr. David W. Sykora, Earthquake Engineering and Seismology Branch (EESB), Earthquake Engineering and Geosciences Division (EEGD), Geotechnical Laboratory (GL), WES. Mr. Donald E. Yule, EESB, assisted Mr. Sykora. Dr. Mary Ellen Hynes was Chief, EESB, during this study. Overall direction at WES was provided by Dr. A. G. Franklin, Chief, EEGD, and Dr. William F. Marcuson III, Director, GL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

CONVERSION FACTORS, NON-SI to SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	Abbreviation	By	To Obtain
acres	-	0.4047	square kilometers
feet	ft	0.3048	meters
inches	in.	2.540	centimeters
miles (US statute)	mis.	1.609	kilometers
pounds (mass) per cubic foot	pcf	16.018	kilograms per cubic meter
pounds (mass) per square foot	psf	4.882	kilograms per square meter
tons (mass) per square foot	tsf	9,765.	kilograms per square meter
pounds (force) per inch	psi	175.1	newtons per meter

EXECUTIVE SUMMARY

A reassessment of liquefaction potential and an estimation of earthquake-induced, free-field settlements were conducted for the Department of Energy (DOE), Paducah Gaseous Diffusion Plant (PGDP), located near Paducah, KY, to provide guidance for the seismic safety analysis and future design of structures and facilities there. This work follows an initial assessment of liquefaction potential (ERCE, Inc. 1990). Shear stresses calculated as part of the site response analysis at four sites by the U.S. Army Engineer Waterways Experiment Station (WES)(Sykora and Davis 1993) were made available, allowing a more accurate assessment of liquefaction potential.

Soils at PGDP generally consist of a thin veneer of loess over Pleistocene-age alluvium overlying Tertiary-age deposits and hard limestone. Two groundwater systems are reported to exist in the Pleistocene-age deposits—a shallow system and a regional gravel system. The regional gravel system was found at depths between 20 and 50 ft, whereas the shallow system exists at depths between 9 and 15 ft at a few locations.

The reassessment made for granular soils under the design basis earthquake indicates that liquefaction is not likely to occur at any of the four sites at PGDP. Some isolated zones of soil at depths on the order of 45 to 50 ft may experience a small buildup of excess pore water pressures. However, these pore water pressures are expected to quickly dissipate, given that underlying and overlying deposits are granular and dense (thereby not generating pore water pressures). Furthermore, the presence of buildings is not expected to increase the likelihood of liquefaction.

Earthquake-induced settlements at the ground surface are expected to be negligible. A most conservative estimate of settlements within a layer at a depth of 50 ft is 2/3 in. This settlement is not expected to be uniform in a horizontal direction and much of the differential movement within a suspect zone is expected to be compensated for through stress redistribution.

The methods used generally follow widely accepted and validated practices of the geotechnical earthquake engineering profession. The analyses are considered to be conservative because the methods used were developed from the performance of much younger deposits of soil during earthquakes.

REASSESSMENT OF LIQUEFACTION POTENTIAL AND ESTIMATION OF EARTHQUAKE-INDUCED SETTLEMENTS AT PADUCAH GASEOUS DIFFUSION PLANT, PADUCAH, KENTUCKY

PART I: INTRODUCTION

1. The Paducah Gaseous Diffusion Plant (PGDP), owned by the U.S. Department of Energy (DOE) and operated under contract by Martin Marietta Energy Systems, Inc., is located southwest of Paducah, Kentucky. An aerial photograph and an oblique sketch of the plant are shown in Figures 1 and 2, respectively. The fenced portion of the plant consists of 748 acres.* This plant was constructed in the 1950's and is one of only two gaseous diffusion plants in operation in the United States; the other is located near Portsmouth, Ohio.

2. The facilities at PGDP are currently being evaluated for safety in response to natural seismic hazards. Design and evaluation guidelines to evaluate the effects of earthquakes and other natural hazards on DOE facilities follow probabilistic hazard models that have been outlined by Kennedy et al. (1990). Criteria also established by Kennedy et al. (1990) classify diffusion plants as "moderate hazard" facilities.

3. The U.S. Army Engineer Waterways Experiment Station (WES) was tasked with reassessing the potential for liquefaction and estimating earthquake-induced settlements using the results of their site response analysis (Sykora and Davis 1993). The calculated shear stresses provide a more accurate means to assess liquefaction than using estimates of peak horizontal acceleration previously available. The current evaluation is based on hypothetical ground motions produced by a 1000-year event which represents the Design Basis Earthquake (DBE) for design and seismic evaluation studies at moderate hazard DOE facilities (Risk Engineering, Inc. 1993).

4. This report begins by describing the geologic and seismologic setting at PGDP, including descriptions of how soils at PGDP are believed to have performed during previous major earthquakes. Next, geotechnical engineering investigations pertinent to these analyses are summarized. Finally, the analyses, including the methodologies, are described.

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* A table of factors for converting US customary units of measurement to metric (SI) units is presented on page 6.

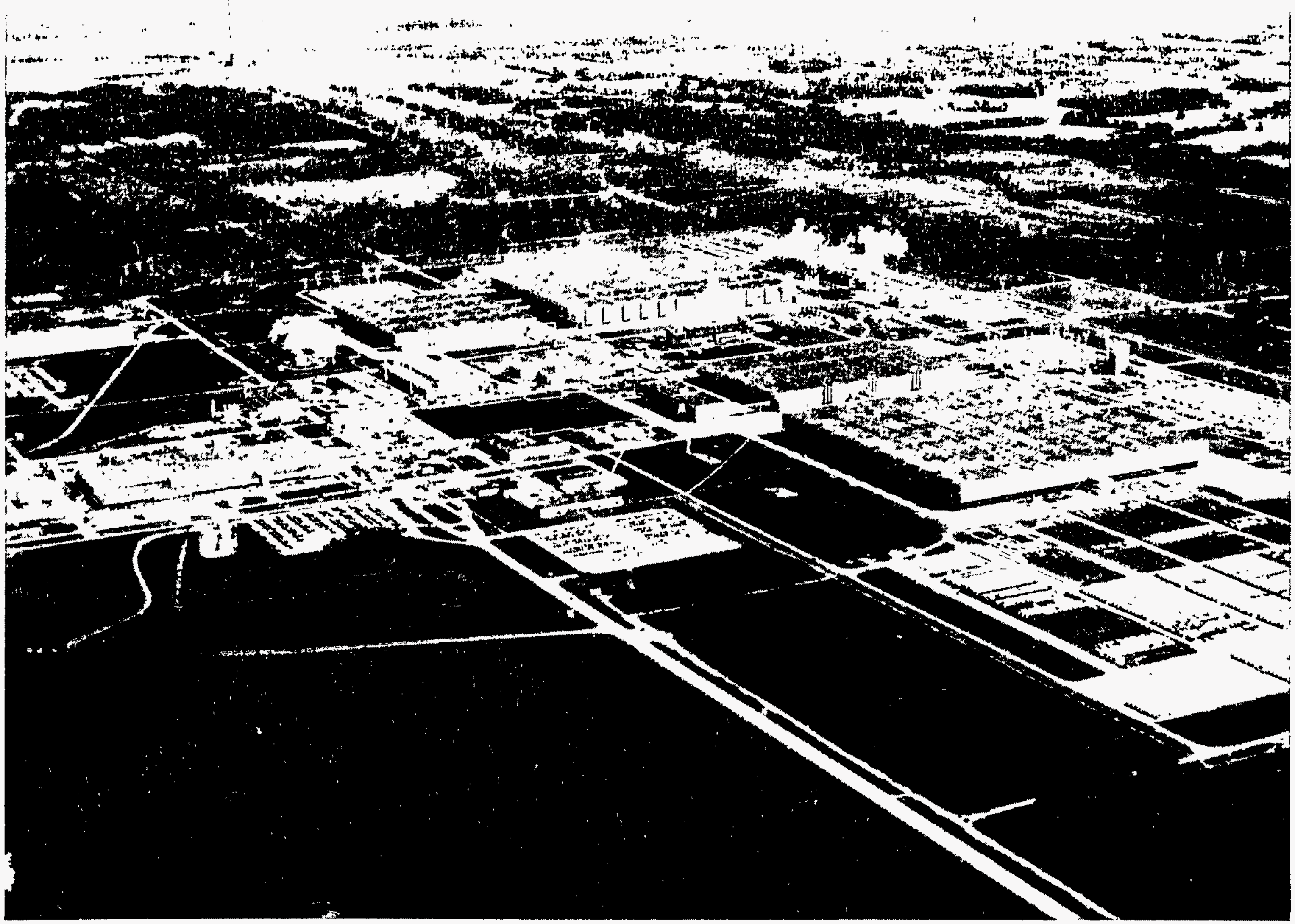


Figure 1. Aerial photograph of Paducah Gaseous Diffusion Plant looking northeast

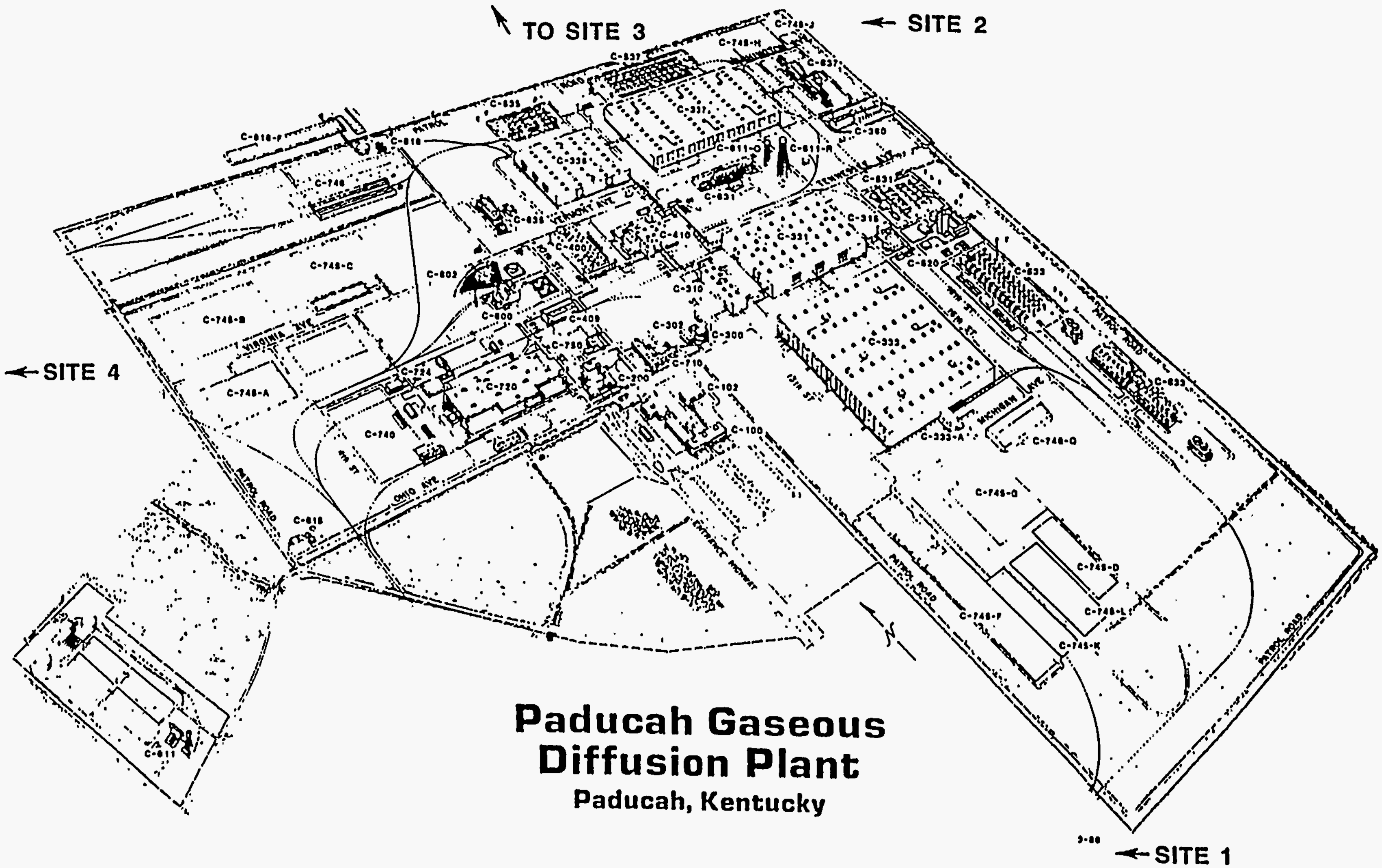


Figure 2. Oblique sketch of PGDP showing general locations of four sites used in the analysis

PART II: GEOLOGIC AND SEISMOLOGIC SETTING

5. PGDP is located about 10 miles west of Paducah, in McCracken County, Kentucky, about 4 miles south of the Ohio River and about 3 miles south of the Ohio River Valley. This area is at the northern boundary of the Coastal Plain Province and the plant is situated on an upland surface that was graded during construction in the early 1950's to between el 370 and 380 MSL.** (ERCE, Inc. 1990b) The region around the plant is relatively flat with some upland erosion from nearby streams.

Site Geology

6. Most soil deposits at PGDP are part of the Mississippi Embayment which consists of Cretaceous-age (pre-Tertiary) to Pleistocene-age deposits. The Mississippi Embayment has undergone several cycles of uplifting with consequent erosion and downwarping with consequent deposition. Tertiary-age deposits were placed in marine environments. Pleistocene-age continental deposits were deposited in fresh-water environments on erosional surfaces of Tertiary-age deposits. "These deposits may represent part of a large alluvial fan, and may consist partly of reworked glacial outwash" (ERCE 1990b). Based on the history of deposition and erosion, continental deposits are expected to be normally consolidated or possibly slightly overconsolidated.

7. Soil deposits can be generally described as consisting of a surficial veneer of loess, alluvial continental deposits that consist of gravel, sand, silt and clay overlying Tertiary-age deposits of predominantly clay interbedded with sands and silts, and occasionally a "rubble zone." Fill is expected at the ground surface in isolated locations. Hard limestone underlies the entire site. The soil deposits and limestone dip gently downward to the south (ERCE 1990b). An illustrated cross section showing the primary soil deposits along a line projected north-south through the plant area is shown in Figure 3. This figure is not to scale, but it generally shows the distribution of materials along the profile. Brief descriptions of the soil deposits and bedrock are presented in Part III.

** Mean Sea Level

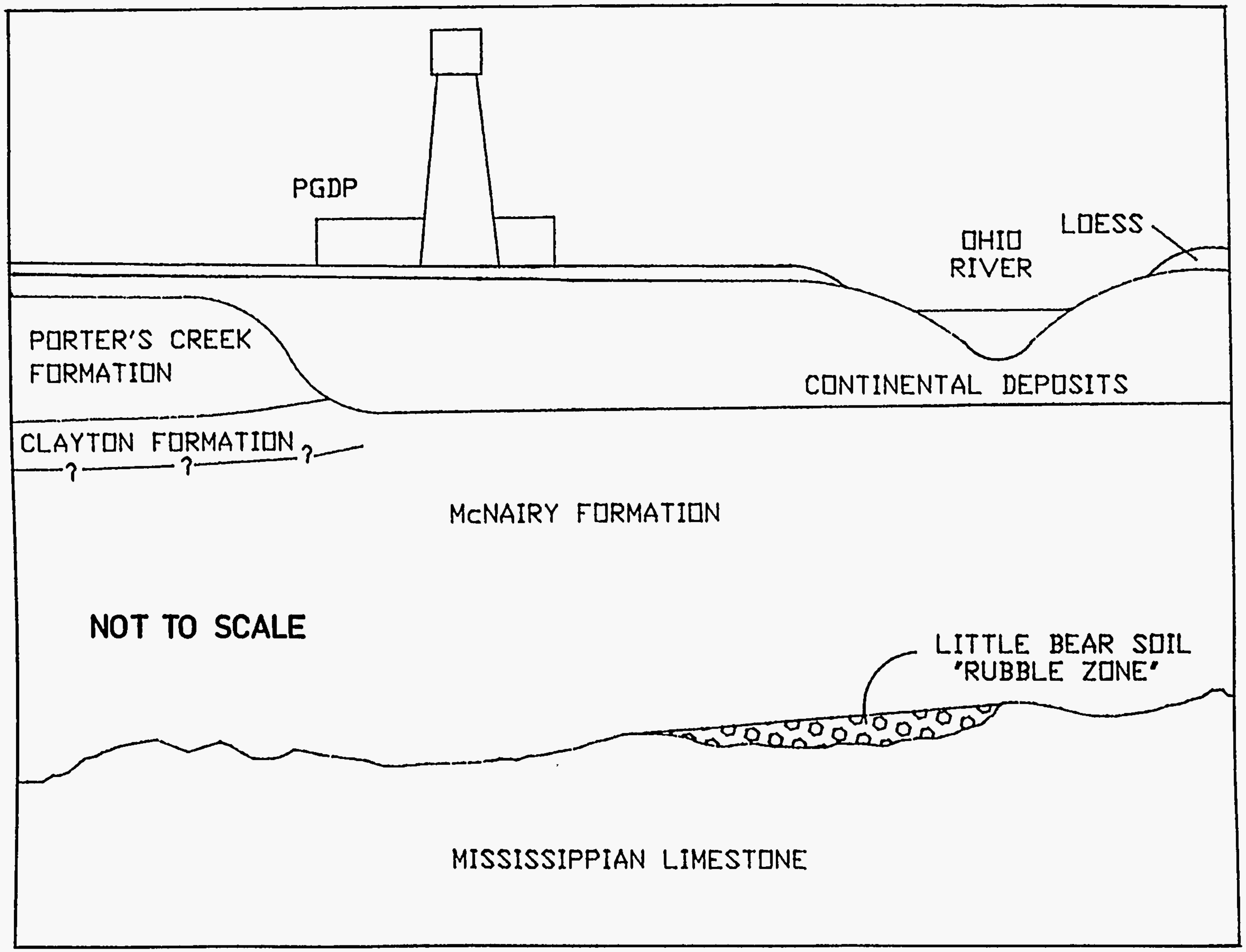


Figure 3. Illustrated cross section through PGDP and Ohio River Valley looking west (adapted from ERCE, Inc. 1990b)

New Madrid Seismic Zone

8. The New Madrid Seismic Zone (NMSZ) lies in the five-state area of Missouri, Arkansas, Tennessee, Kentucky, and Illinois. Four or five major earthquakes are believed to have occurred in the NMSZ in late 1811 and early 1812 (Nuttli 1987). Recent seismic activity in the NMSZ is shown in Figure 4. Much of the region, except for river towns, was uninhabited at the time of these cataclysmic events, so accounts of damage and details of near-field ground response and failure were few. Two studies after the turn of the century documented eyewitness accounts (Berry 1908) and remnants of ground failure (Fuller 1912). Street and Nuttli (1984) revisited these two early studies, adding more recent information. Obermeier (1984) focused on liquefaction and ground failures and several other studies have also been added (see Gori and Hays 1984).

9. All evidence suggests that no liquefaction or ground failure occurred in the upland surface at the present site of PGDP. Liquefaction is believed to have occurred in the Wabash Valley, further away from the NMSZ than PGDP, but in young alluvial deposits (Obermeier et al. 1991). The only reported failures in the upland surface during the 1811-1812 events were slope failures on the bluffs of the Mississippi River near Wickliffe, Kentucky (Jibson and Keefer 1984). These bluffs are within 15 km of at least one epicenter. The Pleistocene-age and older deposits at greater distances, like at PGDP, have performed well.

Design Basis Earthquake

10. Probabilistic methods of hazard analysis were used to derive parameters defining the DBE and to develop corresponding synthetic records. This procedure was based on current DOE guidelines and the moderate hazard classification assigned to PGDP. The probabilistic assessment of seismic hazard was conducted by Risk Engineering, Inc. (1993) using an extended-source seismic hazard model for site-specific evaluations at PGDP. The extended-source model of the NMSZ is a system of parallel faults running in a north-northeasterly direction.

11. Two synthetic earthquake records representing rock outcrop motions for the DBE were developed corresponding to median levels of hazard for a

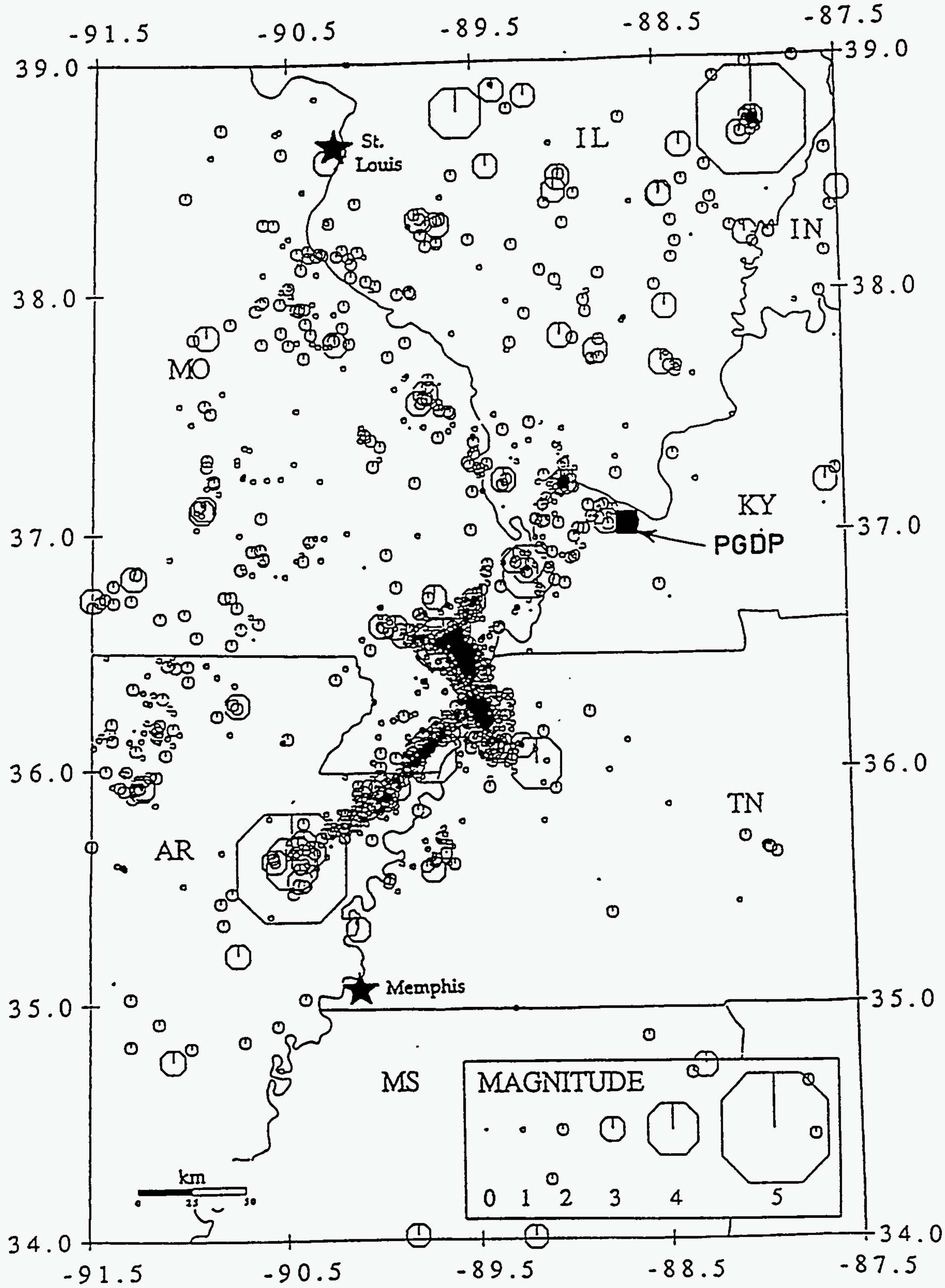


Figure 4. Seismic activity in central U.S. during a 189-month period between 1974 and 1990 (courtesy of Saint Louis University)

1000-year event by Risk Engineering, Inc. (1992) to completely envelop the uniform hazard spectra. The two horizontal components of the DBE at rock outcrop are shown in Figure 5 and correspond to a dominant magnitude of 7.3 at an epicentral distance of 52 km. Characteristics of the event are summarized in Table 1. The peak horizontal ground accelerations are 0.26 and 0.27 g for the Horizontal 1 and Horizontal 2 components, respectively, and the durations of strong motion (accelerations $\geq$ 0.05 g) are 15 and 17 sec.

Table 1

Characteristics of DBE Outcrop Motions

Component	Peak Acceleration (cm/sec^2)	Peak Velocity (cm/sec)	Peak Displacement (cm)	Duration Strong Motion (sec)
Horizontal 1	258	18.1	13.7	15
Horizontal 2	265	14.7	11.1	17

12. Records of the variation of acceleration, velocity, and displacement with time and absolute acceleration response spectra are presented in Figure 6. The acceleration and velocity records were integrated to allow inspection of the variations of velocity and displacement, respectively, with time. The integration was exact since the acceleration records consist of discrete values generated at equal time steps. The Horizontal 1 component has slightly larger peak values of velocity and displacement. The variations of displacement for each component are slightly skewed to one direction or the other.

13. The absolute acceleration response spectra at six levels of system damping for the 1000-year event are shown for both components of rock outcrop motion in Figure 7. The spectra corresponding to 5 percent damping are similar with peak spectral accelerations of 0.68 and 0.77 g. Predominant periods for the two components are again 0.042 and 0.035 sec, and the Horizontal 2 component has a consistently greater response at periods less than 0.04 sec.

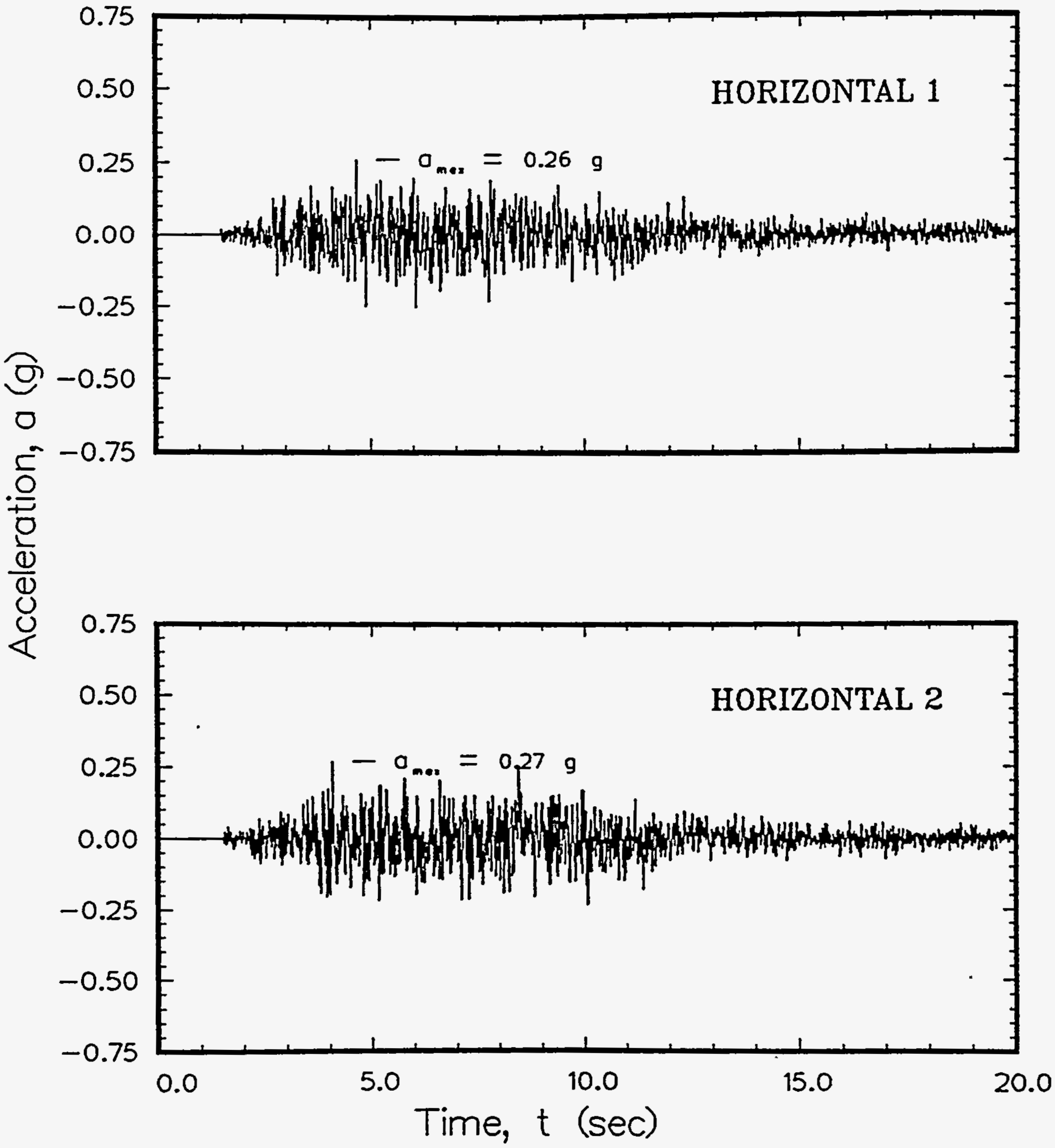
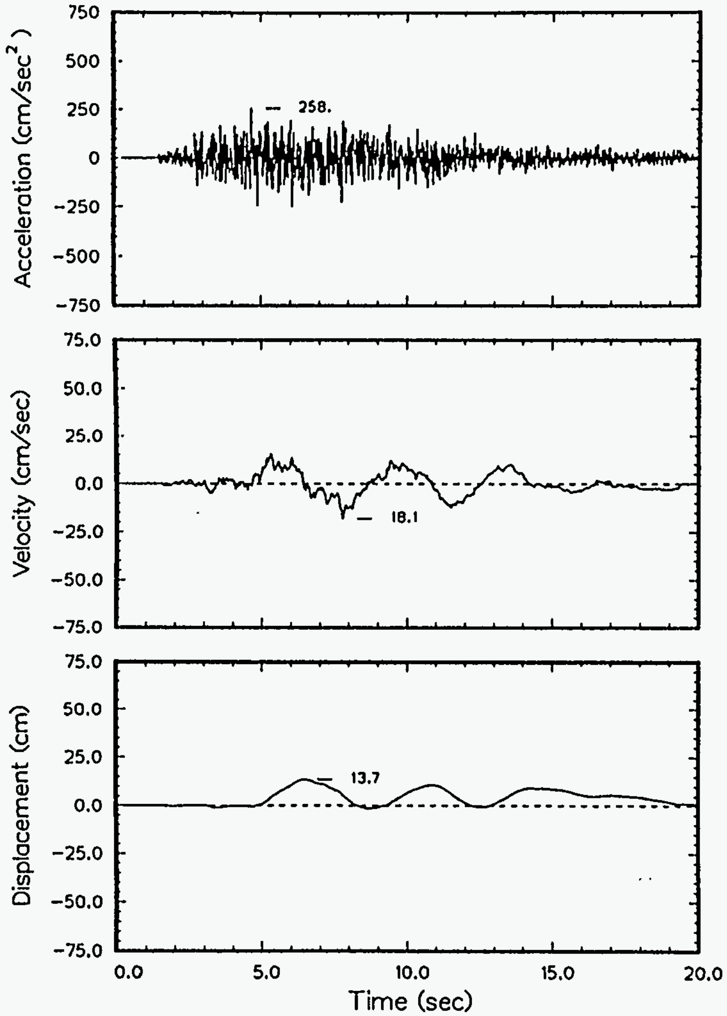
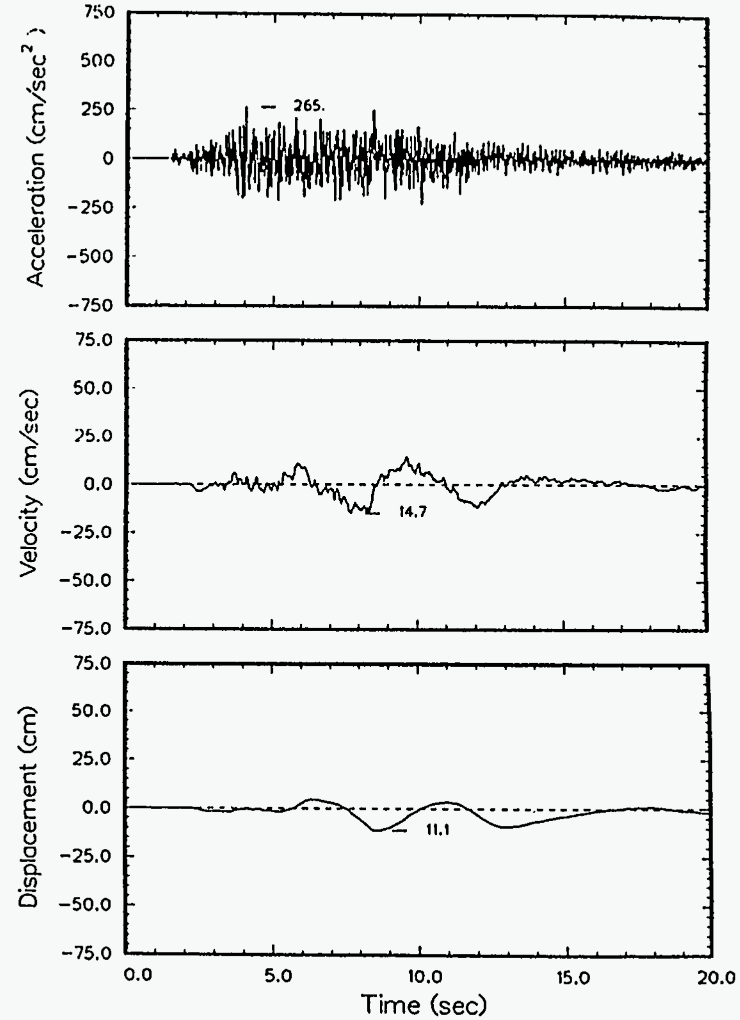


Figure 5. Horizontal components of acceleration versus time for the 1000-year earthquake (Risk Engineering, Inc. 1992)



a. Horizontal 1 component



b. Horizontal 2 component

Figure 6. Variations of acceleration, velocity, and displacement for 1000-year earthquake

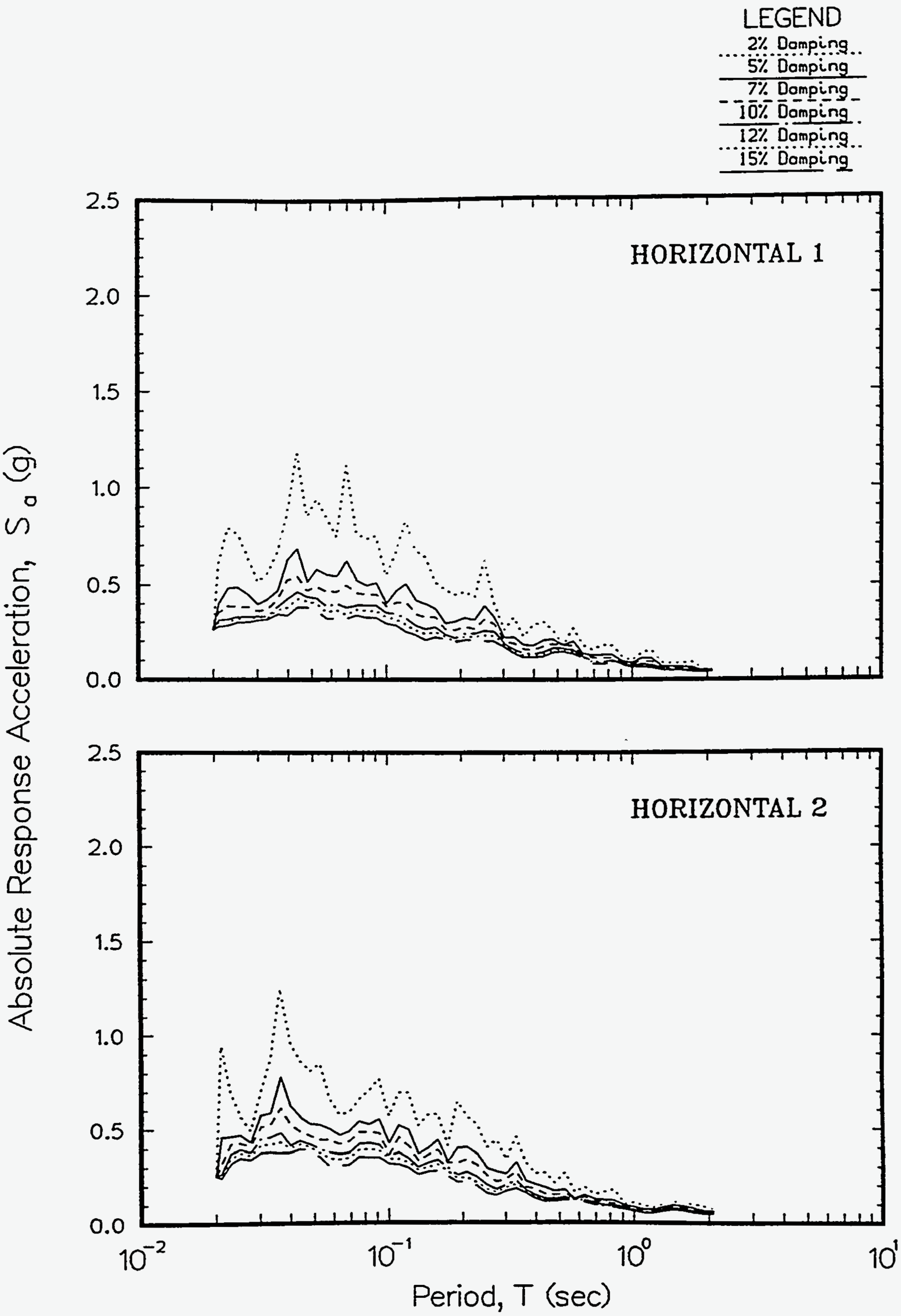


Figure 7. Acceleration (outcrop) response spectra for the 1000-year design earthquake event

PART III: PREVIOUS GEOTECHNICAL ENGINEERING STUDIES

14. Certain information about subsurface and groundwater conditions from previous geotechnical engineering studies is important for these analyses. An attempt was made to obtain information from all prior geotechnical investigations at PGDP, particularly those conducted by the U.S. Army Corps of Engineers (USACE) in the 1950's for original construction of the plant. Despite considerable effort, this information was not found. Therefore, in general, only recently-obtained information was involved.

Groundwater Levels

15. Saturation of soils increases the potential for liquefaction. Therefore, the groundwater conditions are an important aspect of evaluating liquefaction potential. Regional and local groundwater modeling studies for PGDP have been conducted in the past by CH2M Hill and are presently underway by Martin Marietta Energy Systems, Inc. A general hydrogeologic description of the site and summary tables of groundwater monitoring wells and elevation measurements were made by Martin Marietta Energy Systems, Inc. (1992) from which the following was extracted.

16. Three hydrogeologic formations have been identified at PGDP: a shallow groundwater system, a regional gravel aquifer, and a deep groundwater system. The shallow system exists at the southern and western margins of the plant area and corresponds with deposits of loess and the upper continental deposits. The gradient of this system is to the north and northeast. The regional gravel system is in the lower continental deposits and has a gradient toward the north. The deep system is in Tertiary Deposits and, therefore, is not of further interest with regard to liquefaction potential.

17. Thirty-six groundwater monitoring wells comprising 13 well clusters are distributed around the plant as shown in Figure 8. The coordinates, elevations, and screened depths of these wells are provided in Table A1 of Appendix A. Fluctuations of phreatic surfaces over a 25-month time beginning in January 1989 are summarized in Table A2 of Appendix A. Note that the number of readings ranges from only 1 to 3. A synopsis of information pertinent to this study at wells screened in the surface and regional gravel systems is presented in Table 2.

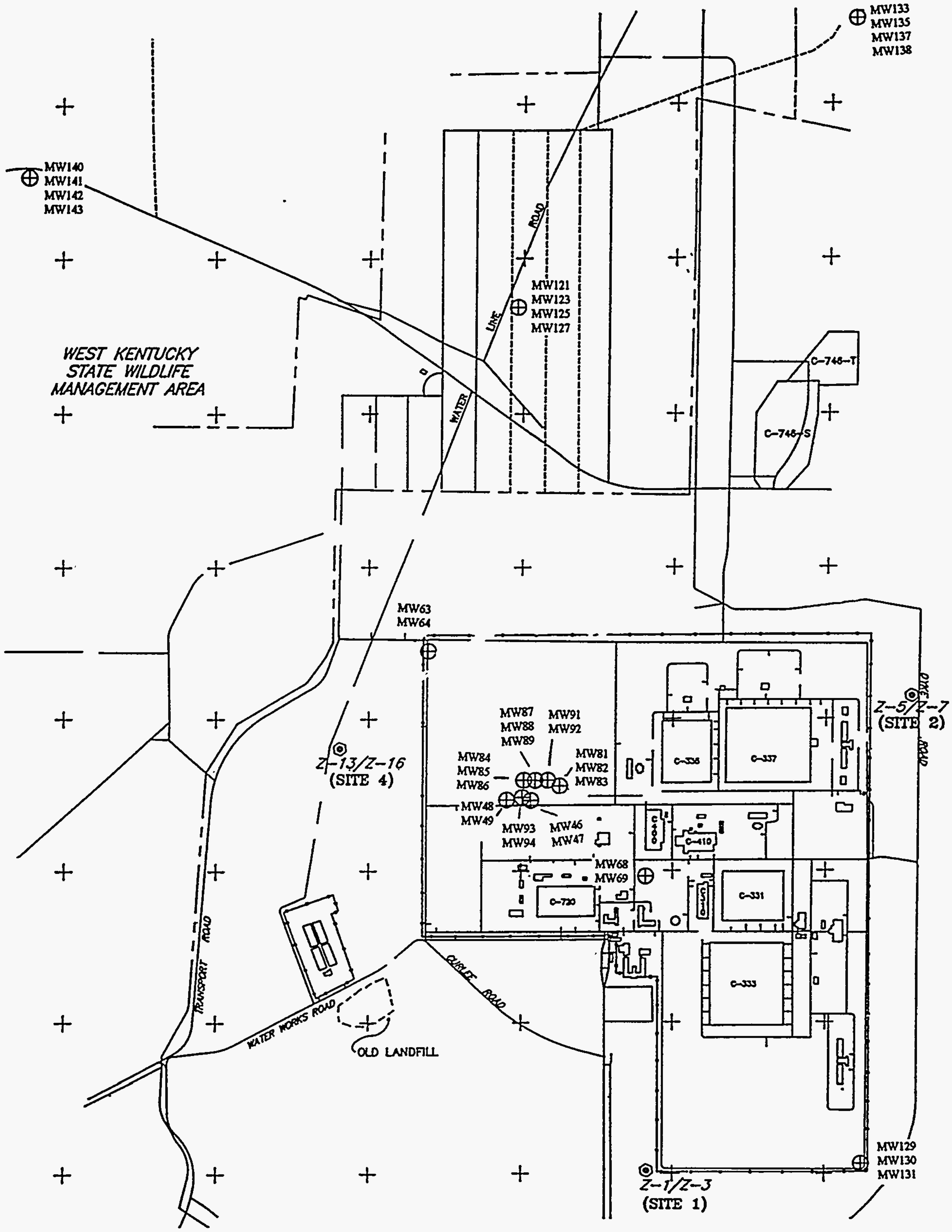


Figure 8. Site map showing locations of groundwater monitoring wells and well clusters

Table 2

Depths to Groundwater at Select Wells

Site	Groundwater System	Nearby Wells	Min. Depth to Groundwater (ft)	Max. Depth to Groundwater (ft)	Depth Used for Analysis (ft)
1	Surface	MW120	unknown	unknown	9
2	Surface	MW69	36	37	
		MW128	33	33	
	Regional Gravel	MW68	52	55	20
		MW124	36	36	
		MW126	33	36	
3	Regional Gravel	MW135	12	12	20
		MW137	12	12	
		MW152	36	36	
4	Surface	MW47	15	15	
		MW49	21	21	
		MW64	8	9	
		MW82	13	19	15
		MW83	12	16	
		MW85	15	15	
		MW88	14	15	
	Regional Gravel	MW46	48	51	
		MW48	51	51	
		MW63	44	48	
		MW81	49	51	
		MW84	48	50	50
		MW86	49	52	
		MW87	48	50	
		MW89	48	50	
		MW92	45	49	

18. The uppermost groundwater surface at Sites 1, 2, and 3 is within the regional gravel system. The depths to the phreatic surface at these sites are 9, 54, and 20 ft, respectively. The interpretation for a profile of effective stress is simple in these cases. At Site 4, on the other hand, two phreatic surfaces are believed to exist within the range of depths important for a liquefaction assessment—both within the shallow system and the regional gravel aquifer. The depths to these surfaces are 15 and 50 ft, respectively. The profile of effective stress at Site 4 was calculated assuming that buoyancy effects for the upper aquifer ended at a depth of 40 ft. Therefore,

effective stresses were calculated at depths greater than 50 ft assuming that total unit weights existed above.

Site Explorations

19. Four sites were explored during recent drilling and geotechnical engineering investigations (ERCE, Inc. 1990c) and geophysical measurements (Automated Science Group, Inc. 1991). All four of the sites are located outside the fenced boundary of PGDP and are separated by great distances. The locations of three of the sites are shown in Figure 2. Site 2 is the closest to a large, important building, about 1,500 ft from building C-337. Site 1 is 2,000 ft from building C-333 and Site 3 is 4,000 ft from building C-720. Site 3 is located near the edge of the upland surface where the cooling water pipes emerge and is about 11,700 ft from buildings C-335 and C-337. Coordinates and surface elevations for all borings drilled at each site are presented in Appendix B.

20. Three or four boreholes were drilled at each of the four sites. At Sites 1 and 2, three boreholes were drilled to depths ranging from 70 to 125 ft. Boreholes were not extended to limestone bedrock at either of these sites. At Sites 3 and 4, four boreholes were drilled, three to depths of about 125 ft and the last terminated in bedrock (encountered at depths of 364 and 322 ft, respectively).

21. Both CME 550 and a CME 75 drill rigs were used to advance shallow holes with hollow-stem augers, "A" rods, and drilling mud. Standard Penetration Tests (SPT's) were made using a safety hammer operated with a rope and cathead. Deep holes were made using a Schramm T-450 drill rig with a rotary bit, NX rods and drilling mud; SPT's were made with a Mobile safety hammer operated with a hydraulic winch. SPT's were specified to follow ASTM standard D1586 although an option to use 10-inch hollow stem augers was allowed in the scope of work. It is not known if the drill bit was made to deflect drilling fluid upward, if the split spoon samplers had a constant inner diameter as recommended by Seed et al. (1985),+ or what is the energy ratio for each hammer system. The energy ratio for the CME 75 has been

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+ A contract dispute has made obtaining this information impossible.

measured by Kovacs, Salomone, and Yokel (1983) and the ratio for the CME 550 is believed to be similar. The Schramm was not used in alluvium.

22. A summary of geotechnical tests and shear wave velocity measurements made at each of the four sites are shown in Figures 9 through 12. Seismic velocities were assimilated by Staub, Wang, and Selfridge (1991). One of the three "shallow" holes was used as the primary source of geotechnical data at each site. Geotechnical data was also obtained from the two "deep" holes at Site 3 and Site 4. SPT's were generally performed at 2.5-ft depth intervals in the upper strata and 5-ft intervals in continental deposits.

23. Geotechnical engineering data measured by ERCE, Inc. (1990a) for the assessment of liquefaction included gradation, plasticity index (PI), unit weights, and specific gravity. A number of these tests were performed at each site, particularly in the loess and continental deposits. The gradation and Atterberg limit values were used to classify the soil to determine the appropriate number of layers and the thickness of each layer. Values of unit weight were measured in the laboratory and are summarized in Appendix C. All but one unit weight was measured in the laboratory on samples of loess and continental deposits at depths less than 55 ft.

Fill

24. Fill was encountered in the upper five feet at Site 2. The fill material is essentially a silty clay with limestone fragments (ERCE, Inc. 1990b). For this analysis, this material was generally lumped together with loess.

Loess deposits

25. Wind-blown loess deposits cover nearly the entire fenced area of PGDP. These deposits are of Pleistocene age and vary in thickness from 15 to 40 ft (ERCE, Inc. 1990b). At the four sites used for site response analysis, the thickness ranged from 10 to 20 ft. The loess generally classifies as a silty clay (CL) with some CL-ML material. The liquid limits and plasticity indices range from 22 to 35 and 4 to 14, respectively; moist unit weights range from 120 to 124 pcf. The range in SPT N-values is 5 to 26 with an average of 11 blows per foot indicating a firm to very stiff consistency.

Continental deposits

26. Continental deposits appear to underlie the entire area around PGDP. These alluvial deposits are of Pleistocene age (possibly pre-Pleistocene); they vary in thickness from 20 ft at Site 1 to 93 ft at Sites 3

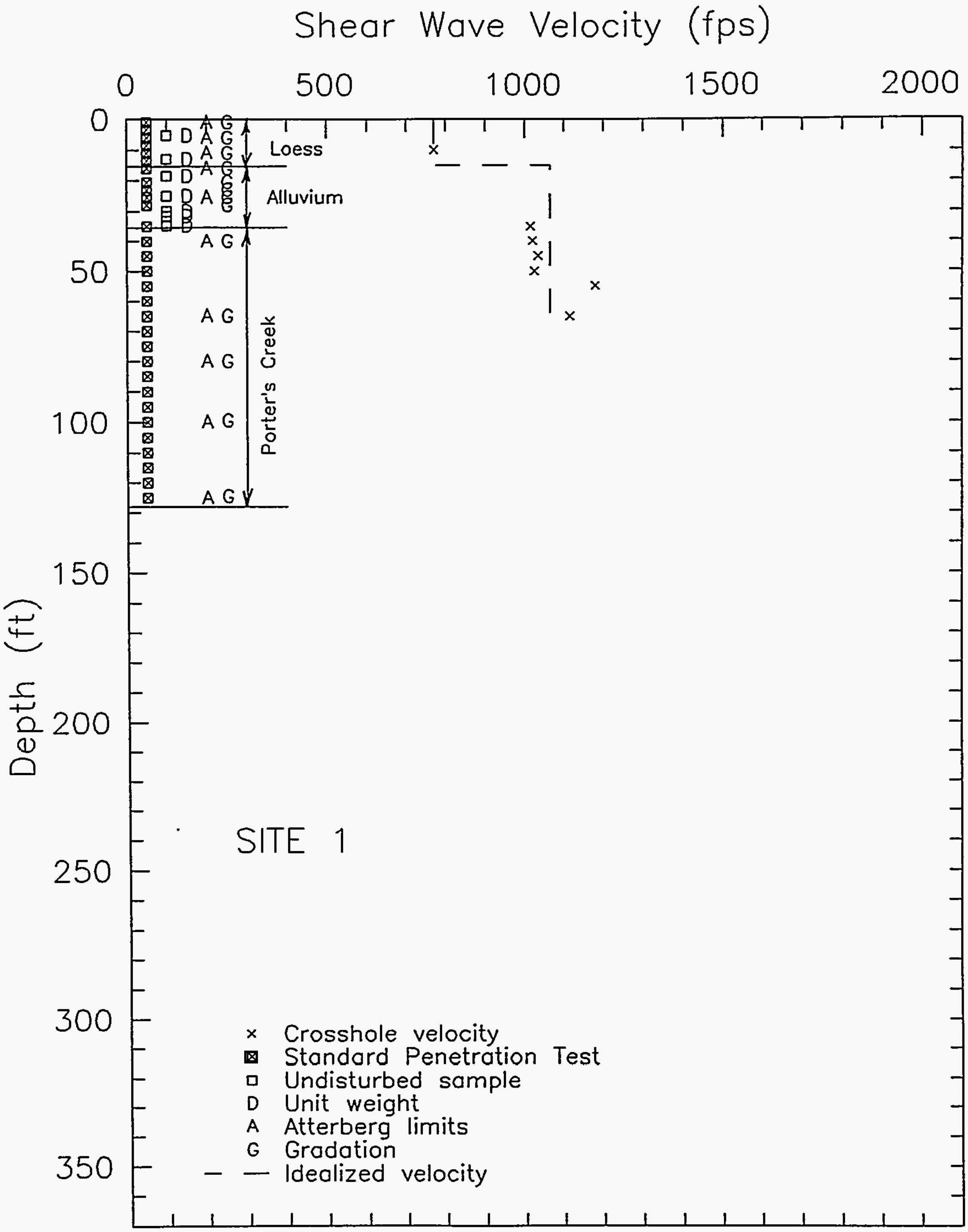

Figure 9. Depths of geotechnical testing and results of shear wave velocity measurements at Site 1

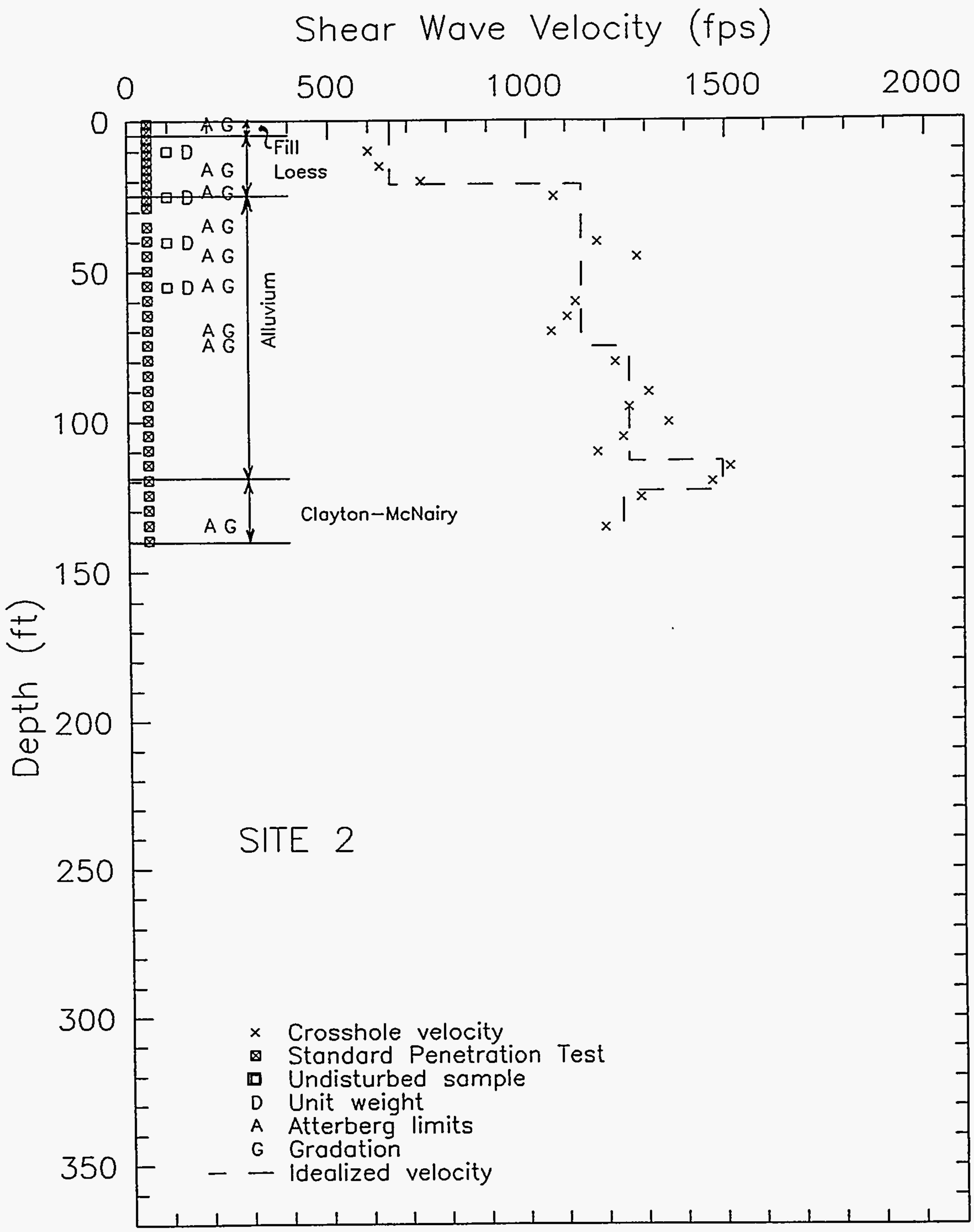

Figure 10. Depths of geotechnical testing and results of shear wave velocity measurements at Site 2

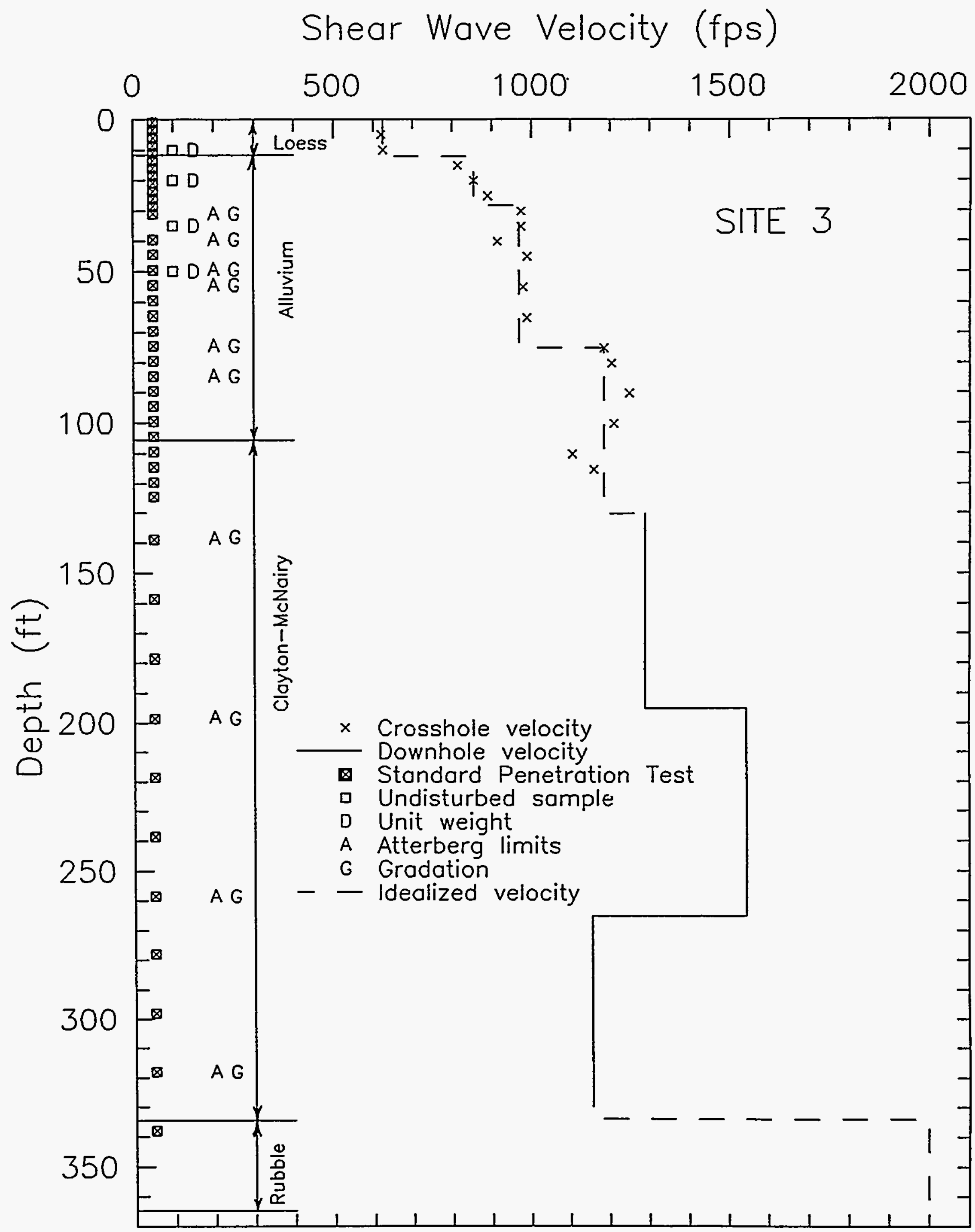

Figure 11. Depths of geotechnical testing and results of shear wave velocity measurements at Site 3

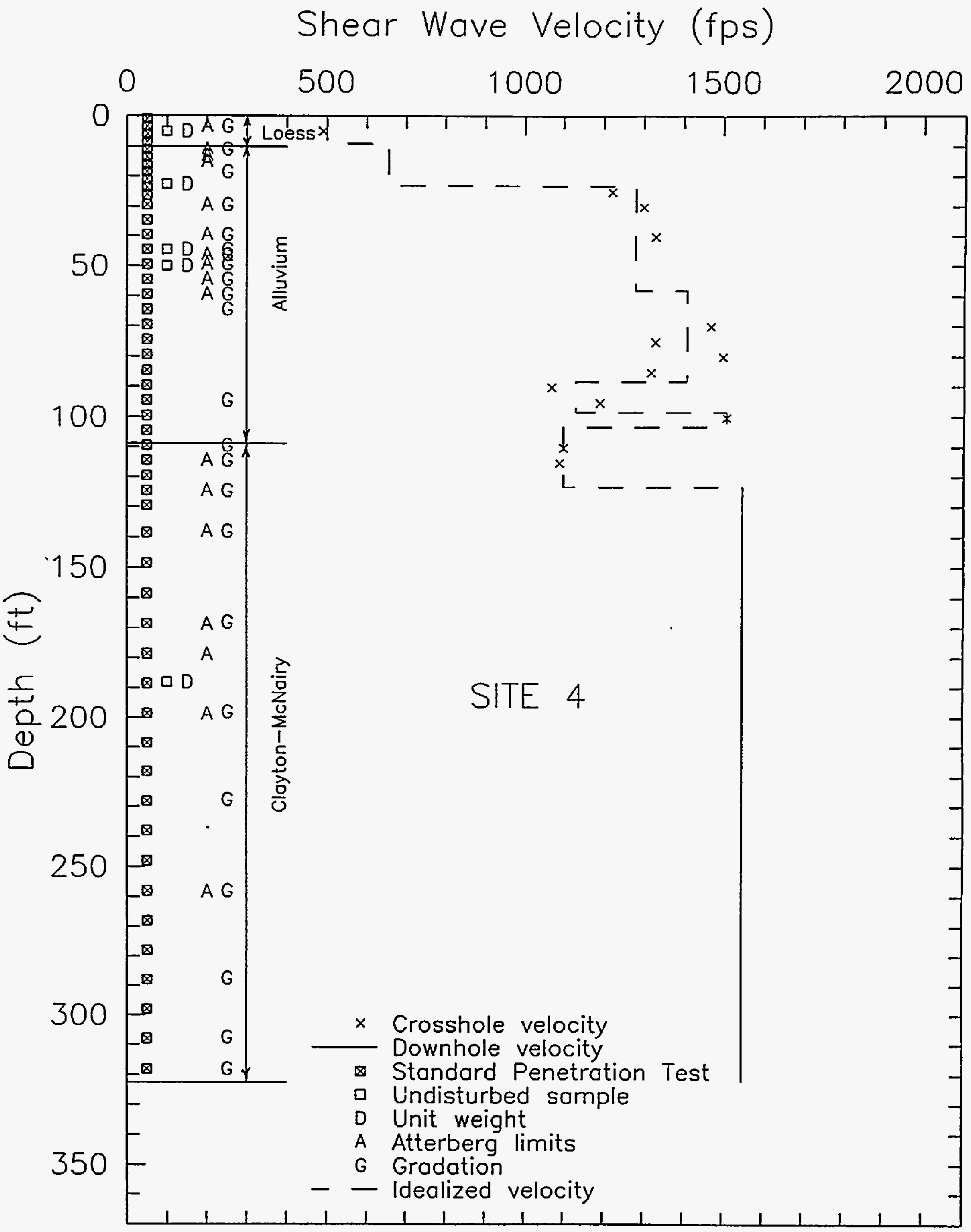

Figure 12. Depths of geotechnical testing and results of shear wave velocity measurements at Site 4

and 4 and 95 ft at Site 2, and consist of low plasticity clays and silts, silty and clayey sands, and gravels. The liquid limits and plasticity indices range from 14 to 40 and non-plastic to 20, respectively; moist unit weights range from 97 to 136 pcf. The range in SPT N-values is from 4 blows per foot to refusal with an average of about 45 blows per foot; this range confirms that there is a wide variation in material densities and consistencies.

Tertiary-age deposits

27. Three primary Tertiary-age formations exist in the area of the PGDP: the Clayton, McNairy, and Porter's Creek. The Clayton and McNairy Formations are combined for engineering purposes of this study because the materials are very similar. The Porter's Creek and Clayton-McNairy Formations are described separately below. Depths to Tertiary-age deposits range from 35 ft at Site 1 to 118 ft at Site 2.

28. Porter's Creek Formation. The Porter's Creek Formation was encountered at Site 1. The thickness of the deposits within this formation is 84 ft. These materials are micaceous silts and clays with intervals of fine sand, in part glauconitic (ERCE, Inc. 1990b). The plasticity of these deposits is high and the Atterberg Limits plot well below the "A-line." The liquid limits and plasticity indices range from 88 to 106 and 11 to 25, respectively; moist unit weights were not measured. The range in SPT N-values is 43 to 170 blows per foot with an average of 92 blows, indicating a hard to very hard soil consistency.

29. Clayton-McNairy Formation. The Clayton-McNairy Formation was encountered at all four sites beneath continental deposits (at Sites 2, 3, and 4) or Porter's Creek Clay (at Site 1). These materials consist of interbedded clay, silt, and fine sand. The thickness of these deposits ranged from 210 to 225 ft at the four sites. The liquid limits and plasticity indices of these materials range from 22 to 43 and non-plastic to 18, respectively; moist unit weights were not measured. The range in SPT N-values is 45 blows per foot to refusal with more than half of the N-values being greater than 100, indicating a hard to very hard soil consistency.

Little Bear Soil

30. Little Bear Soil (rubble zone) was apparently encountered at Site 3 at depths between 334 and 364 ft but not at any of the other three sites. This deposit is believed to consist of silty clay with chert fragments and limonite nodules (ERCE, Inc. 1990b). This material is described from the

drilling log as "Probably siliceous limestone and chert fragments (rubble zone)." An SPT sampler could not penetrate material in this zone.

Bedrock

31. Bedrock beneath the plant area at PGDP generally consists of limestone of Mississippian Age, presumably of the Warsaw Formation. The limestone tends to be moderately hard to hard. Two borings for this study fully penetrated the soils (at Sites 3 and 4) and were extended 5 to 35 ft, respectively, into limestone using a roller bit.

Site Response Analysis

32. The computer program *SHAKE*, as modified by Sykora, Wahl, and Wallace (1992), was used to calculate site-specific ground motions caused by the synthetic earthquakes. *SHAKE* was developed at the University of California at Berkeley (Schnabel, Lysmer, and Seed 1972) to calculate the horizontal ground motions caused by an earthquake at any depth of a soil profile. The methodology and algorithms incorporated in the program are fairly simple and straightforward and quite adequate for the purpose intended. This is supported by the prolific publication of results and favorable comparisons with measured response (e.g., Seed et al. 1987 and Seed, Dickenson, and Idriss 1991). The simplicity associated with *SHAKE* is attributed to some basic assumptions regarding the cyclic behavior of materials and the one-dimensional idealization.

33. The predominant site period is in the range of 0.9 to 1.2 sec. Secondary response peaks occurred at periods around 0.2 and 0.4 sec. The peak horizontal acceleration at (free field) ground surface was calculated to be 0.27 g. The peak spectral velocity is 26 in./sec at 5 percent damping at a period of 1.1 sec and peak spectral acceleration is 1.1 g at a period of 0.2 sec. The combined pseudo-velocity spectra are shown in Figure 13.

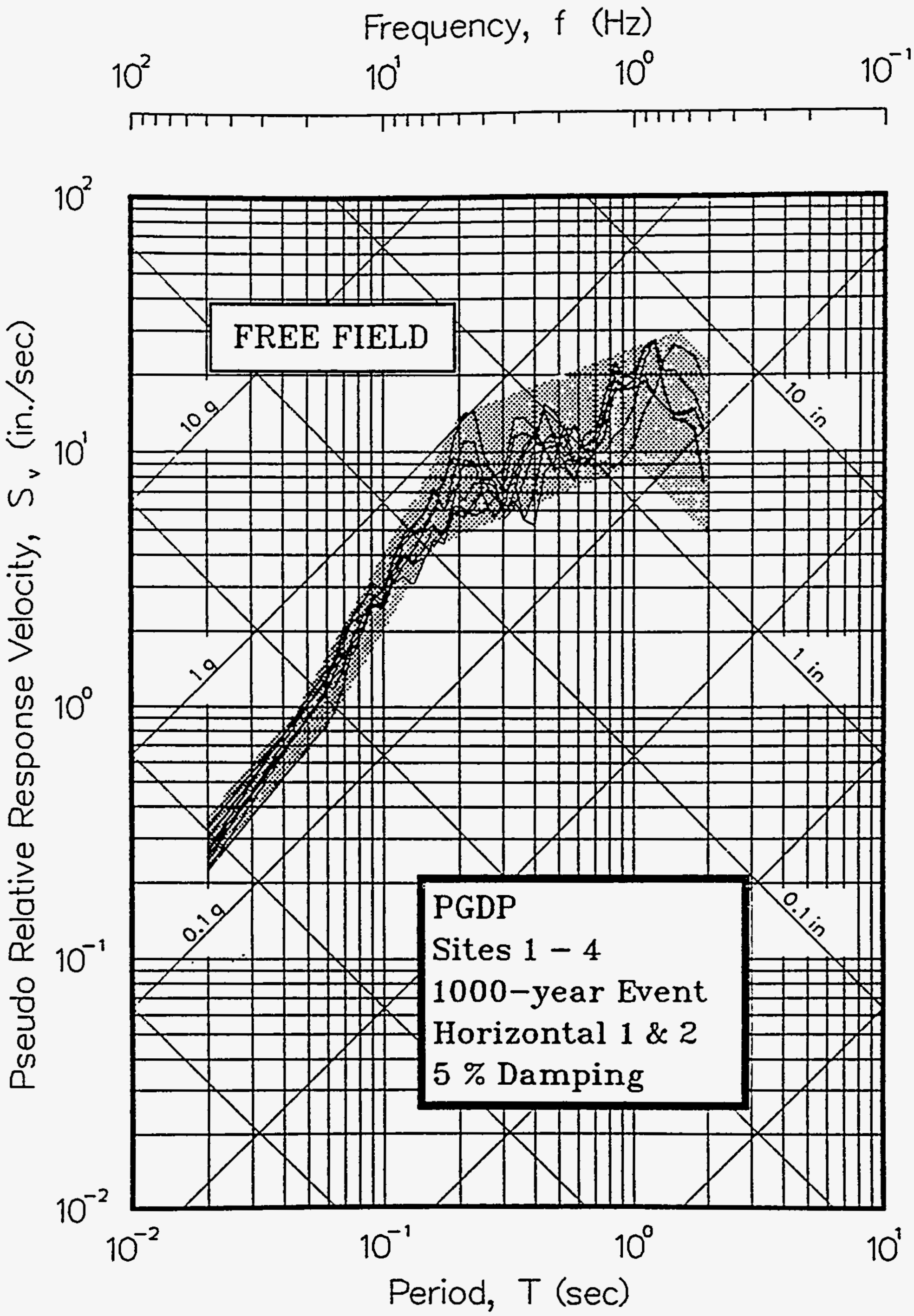

Figure 13. Tripartite representation of free-field response at 5 percent damping for collection of four sites for both components of DBE

PART IV: REEVALUATION OF LIQUEFACTION POTENTIAL

34. Many factors influence the ability of soil to withstand liquefaction and different methods are available to assess liquefaction resistance (National Research Council 1985). The most popular method was derived by Prof. Harry Seed and his colleagues at the University of California at Berkeley using an empirical correlation of the performance of soils during previous earthquakes with the SPT. It will be described in detail later.

35. The geologic age of soil deposits and the type of depositional environment can be used to categorize soils in terms of liquefaction susceptibility. The results of three research studies that examined such categorizations are summarized in Table 3. Clearly, the potential for liquefaction decreases significantly as the age of the deposit increases. However, procedures used for the reassessment of liquefaction potential and evaluation of earthquake-induced settlements at PGDP were developed using Holocene-age, recently-deposited, or laboratory-manufactured samples of soil. None of these represent older, inherently more resistant soils. Therefore, analysis performed and summarized in this report have an un-quantifiable level of conservatism.

Table 3

General Categories of Liquefaction Susceptibility

Deposit	Susceptibility to Liquefaction*		
	Youd & Perkins (1978)	Tinsley et al. (1985)	Iwasaki et al. (1982)
Holocene-age alluvium	Low to High**	Moderate	Likely
Pleistocene-age loess	High		
Pleistocene-age alluvium	Low	Low to Very Low	Not likely
Tertiary-age deposits	Very low		

* Assumes that soils are saturated.

** High corresponds to river channel, tephra, and loess.

36. Case studies of liquefaction and ground failure have been used to derive a methodology to evaluate the potential for liquefaction. The procedure involves determining the resistance of saturated, granular soils to shearing via a standard geotechnical tool—the SPT—and comparing these strengths with those from soils that have and have not liquefied during earthquakes of similar magnitude. A number of factors must be evaluated to approach a useful correspondence between the soils of interest and soils in the liquefaction data base. In addition, Rollins and Seed (1990) provide a threshold in terms of spectral and maximum accelerations to determine if the presence of a buildings influences liquefaction potential. Each of these aspects is described separately below before project-specific comparisons are made. The effect of applying these methods to gravelly soils is unknown because of the uncertainties surrounding SPT measurements in gravel.

Shear stresses

37. Earthquake-induced shear stresses are generally obtained via calculation with wave propagation computer codes or estimation using empirical distributions derived from computer codes (e.g., the simplified procedure by Seed and Idriss 1971).

38. Site-specific wave propagation. Computer codes are available to calculate the propagation of seismic waves in soils. The maximum shear stress produced by the vertical propagation of shear waves, $(\tau_{eq})_{max}$, can be calculated at any depth in the idealized profile although one value per soil layer is common. Once the distribution of maximum values is obtained, the average shear stress, $(\tau_{eq})_{av}$, is calculated from:

$$(\tau_{eq})_{av} = 0.65\ (\tau_{eq})_{max} \qquad (1)$$

39. Simplified procedure. Seed and Idriss (1971) developed a means to estimate the range of shear stresses at any depth during an earthquake. The equation for $(\tau_{eq})_{av}$ is:

$$(\tau_{eq})_{av} = 0.65\ \frac{a_{max}}{g}\ \sigma\ r_d \qquad (2)$$

where

a_{max} = peak horizontal acceleration at ground surface
σ = total overburden stress
r_d = stress reduction factor

A range for r_d was developed using wave propagation computer codes with assumptions of rigid body response, a wide variety of earthquake motions, and soil deposits with sand in the upper 50 ft. This range is shown in Fig. 14.

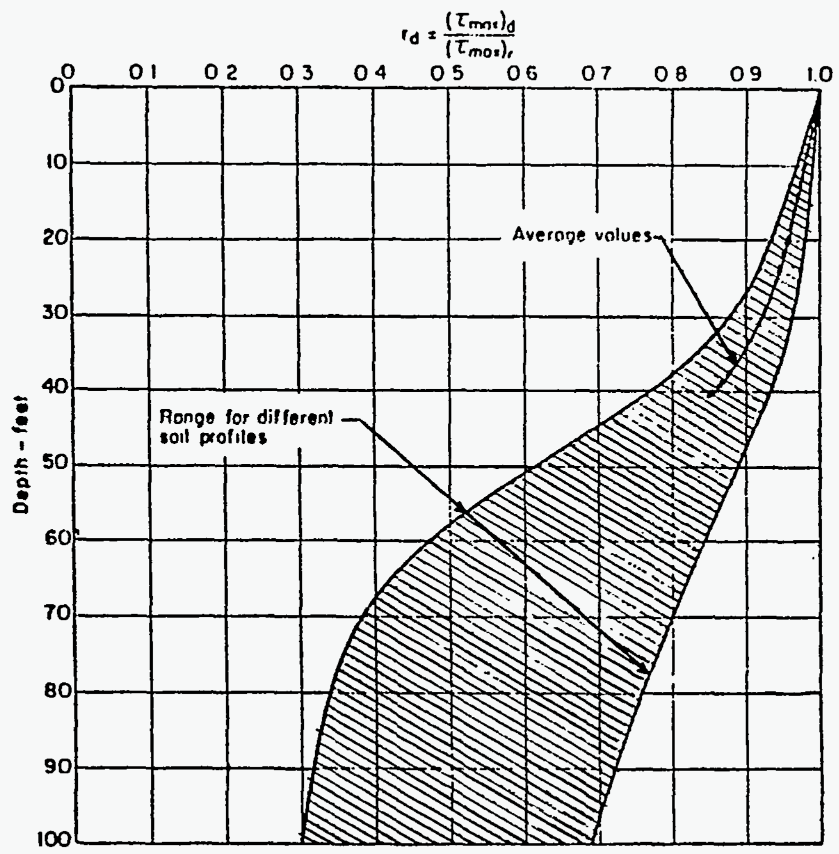

Figure 14. Stress reduction factor (Seed and Idriss 1971)

Shear strengths

40. The SPT represents the best known means to determine the shear strength against liquefaction in sandy soils. The SPT has been carefully scrutinized because of its importance in liquefaction evaluations and other applications in geotechnical engineering. Seed et al. (1985) recommended certain procedures for conducting the SPT for use in liquefaction correlations. These guidelines are summarized in Table 4. As reported in Part III, many of these guidelines were followed for SPT's recently performed.

Table 4

Guidelines for SPT for Liquefaction Assessments

(Seed et al. 1985)

A. Borehole:	4 to 5-in. diameter rotary borehole with bentonite drilling mud for borehole stability.
B. Drill Bit:	Upward deflection of drilling mud (tricone or baffled drag bit)
C. Sampler:	O.D. = 2.00 in. I.D. = 1.38 in. (constant; i.e., no room for liners in barrel)
D. Drill Rods:	A or AW for depths less than 50 ft N or NW for greater depths
E. Energy to Sampler:	2520 in.-lbs (60% of theoretical maximum)
F. Blowcount Rate:	30 to 40 blows per minute
G. Penetration Resistance Count:	Measures over range of 6 to 18 in. of penetration into the ground

41. The most critical factor, the energy efficiency of the hammer system, is expected to be about 67 percent for the CME drill rigs as described previously. Therefore, the SPT N-values corresponding to 60 percent of the theoretical maximum, N_{60}, is calculated by:

$$N_{60} = \frac{67}{60} N_{67} = 1.12\, N_{67} \tag{3}$$

Equivalent SPT N-values corresponding to a vertical effective stress of 1.0 tsf are then determined using the correction factor C_N:

$$(N_1)_{60} = C_N \cdot N_{60} \tag{4}$$

Relationships defining C_N for two different ranges in relative density measured by Marcuson and Bieganousky (1977) are shown in Figure 15 along with a correlation by Tokimatsu and Seed (1987) indicating that $(N_1)_{60} = 15$ corresponds to a relative density of 60 percent.

42. Once the values of $(N_1)_{60}$ are obtained, the cyclic stress ratio expecting to cause liquefaction, CSR, defined by:

$$CSR = \left(\frac{(\tau_{soil})_{av}}{\sigma'_v} \right) \qquad (5)$$

can be determined using empirical relationships defined by Seed et al. (1985) as shown in Figure 16. Data in Figure 16 correspond to soils from investigated sites that have and have not liquefied during magnitude 7.5 earthquakes as evidenced by surface manifestations. The data base consists primarily of observations from Holocene-age alluvial deposits from around the world. Three different relationships exist corresponding to different percentages of fines. The CSR corresponding to a fines content of 5 percent, $(N_1)_{60}{}^{cs}$, can also be used.

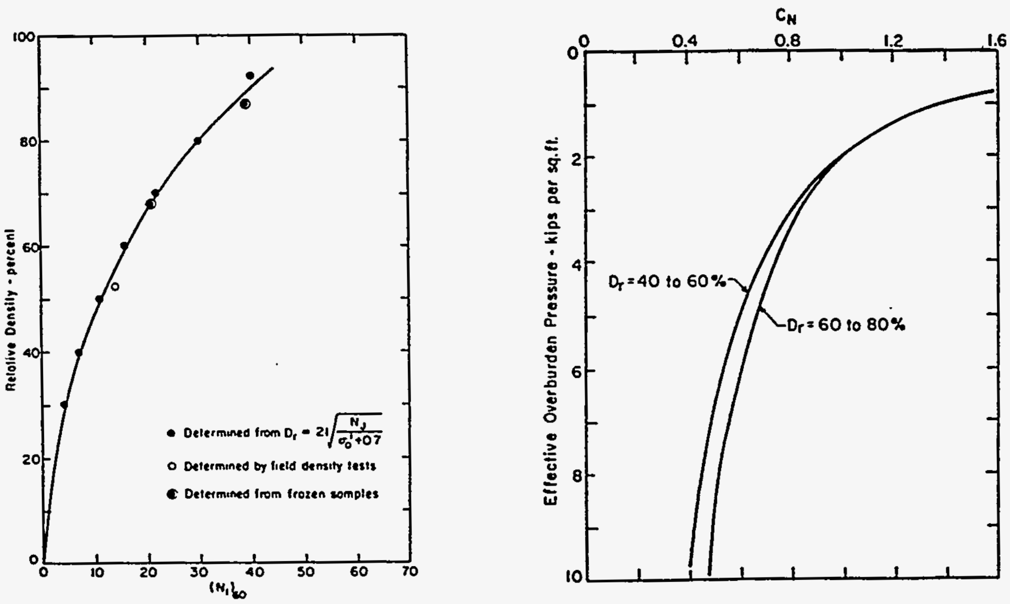

Figure 15. Empirical relations used to determine relative density (Tokimatsu and Seed 1987) and effective-overburden-pressure correction factors for SPT N-values (Seed et al. 1985; after Marcuson and Bieganousky 1977)

43. Finally, the value of $(\tau_{soil})_{av}$ is obtained:

$$(\tau_{soil})_{av} = CSR \cdot K_M \cdot K_o \cdot \sigma'_v \qquad (6)$$

where K_M is an adjustment factor to account for the difference in earthquake magnitude (Seed et al. 1985) which can be estimated by:

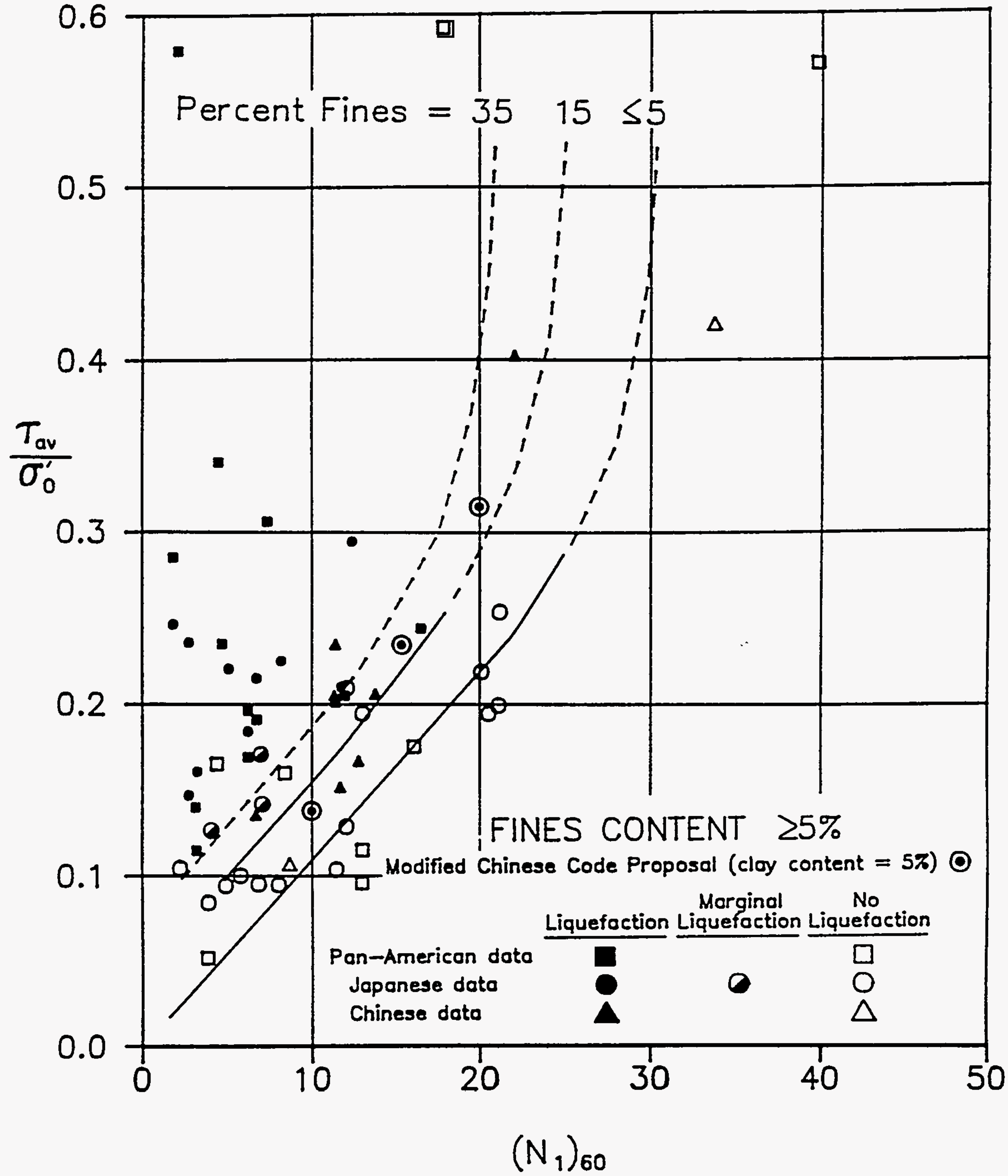

Figure 16. Relationships between stress ratios causing liquefaction and values of $(N_1)_{60}$ for silty sands and M = 7-1/2 earthquakes (Seed et al. 1985)

$$K_M = 9.53 \, M^{-1.12} \tag{7}$$

and K_σ is an adjustment used to account for additional effects of effective confining stress as proposed by Harder (1988) and shown in Figure 17.

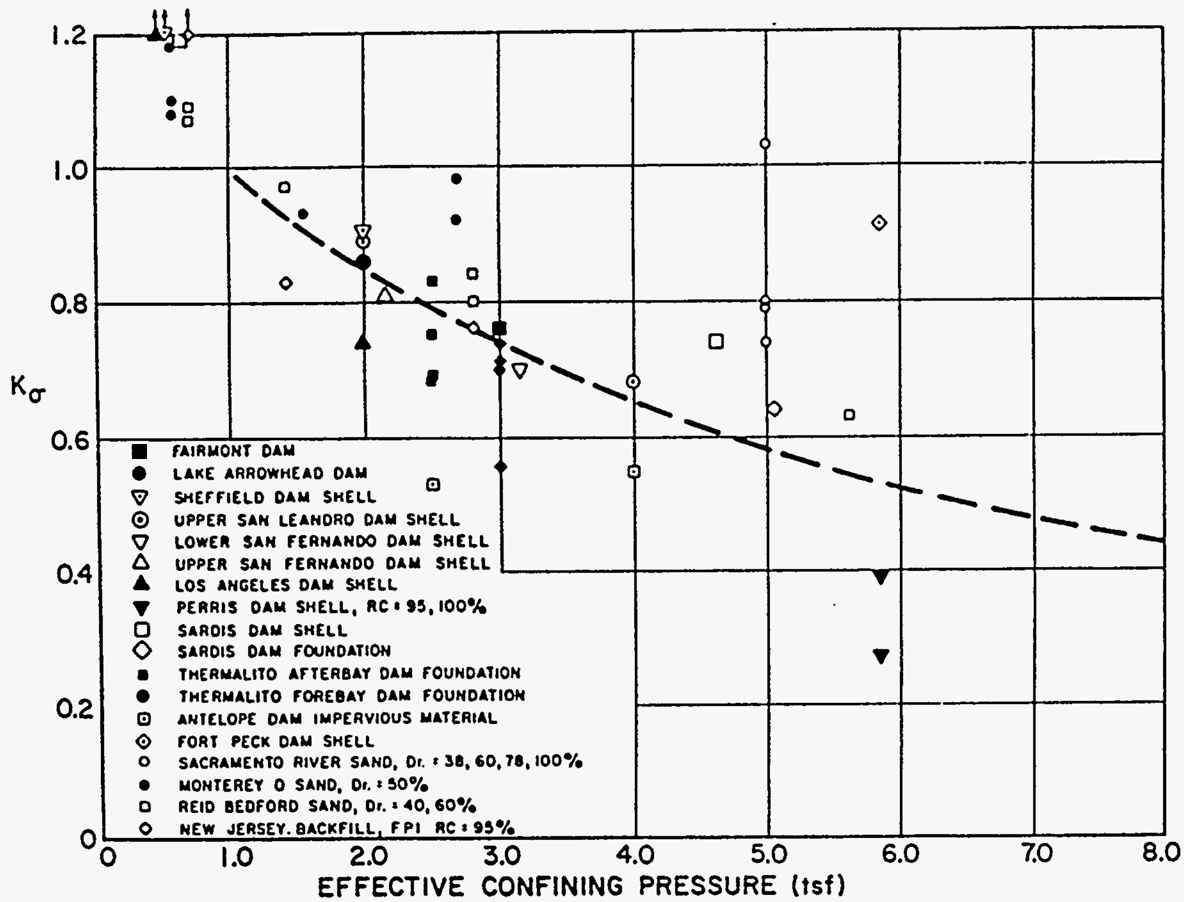

Figure 17. Adjustment to cyclic stress ratio to account for influence of vertical effective stress (Harder 1988)

Factor of safety against liquefaction

44. The factor of safety against liquefaction, FS_L, can be defined as:

$$FS_L = \frac{(\tau_{soil})_{av}}{(\tau_{eq})_{av}} \tag{8}$$

The FS_L is calculated at depths of interest and normally a profile of $(\tau_{eq})_{av}$ vs $(\tau_{soil})_{av}$ with contours of FS_L is used to evaluate the overall response at a specific site. A $FS_L \leq 1$ corresponds to impending liquefaction and a

$FS_L \leq 1.5$ generally corresponds to the generation of excess pore water pressures (Tokimatsu and Yoshimi 1983, Evans 1987, and Hynes 1988).

ERCE, Inc. 1990 Study

45. ERCE, Inc. (1990a) used the simplified procedure to estimate earthquake-induced shear stresses. The upper bound of r_d (refer to Figure 14) was used along with 300 samples of gravels, sands, and silts (evaluated as a granular material) from 84 boreholes (including H, S, and Z-series). Two different values of a_{max} at the ground surface—0.20 and 0.45 g—were used corresponding to a Magnitude 7.5 event. ERCE, Inc. (1990a) also used criteria proposed by Seed, Idriss, and Arango (1983) to evaluate the potential of clayey soils to liquefy.

46. ERCE, Inc. (1990a) reported that about 135 samples of gravel, sand, or silt met the liquefaction criteria at peak ground accelerations of 0.45 g and 37 samples are expected to liquefy at peak ground accelerations of 0.20 g. Only one sample of clay met the criteria developed for clayey soils. The criterion set for the grain size corresponding to 15 percent passing was found to be the most restrictive for soils at PGDP.

Results Using Calculated Stresses

47. The potential for liquefaction to occur at the four sites shown in Figure 3 was evaluated using calculated shear stresses from *SHAKE*. This evaluation could not be applied to other locations where N-values are available because of the specificity of stresses to the soil column representing the site.

Soil samples

48. A summary of cohesionless soil samples obtained at each site with the SPT sampler are listed in Tables 5 through 8. These samples are the basis for the evaluation of liquefaction potential. In many cases, the percentage of fines (passing #200 sieve) was not measured. Since this parameter is important in determining liquefaction resistance, it was inferred from the distribution of measured values from several borings made during the past five years.

Table 5

Samples for Evaluation at Site 1 (Z-1)

Sample	Depths (ft)	Description	Liquid Limit	Plasticity Index	Fines (%)	USCS
SS-7	15.0-16.5	SAND, silty	14	2	47	SM
SS-9	22.0-23.5	SAND, silty, clayey			47	SM-SC
SS-10	24.5-26.0	SAND, clayey	32	15	48	SC

Table 6

Samples for Evaluation at Site 2 (Z-5)

Sample	Depths (ft)	Description	Liquid Limit	Plasticity Index	Fines* (%)	USCS
SS-16	48.5-50.0**	SAND, silty, clayey	14	2	47	SM
SS-20	68.5-70.0	SAND, slt silty	15	NP	37	SM
SS-21	73.5-75.0	"	27	NP	13	SM
SS-22	78.5-80.0	GRAVEL, slt sandy			(5)	
SS-23	83.5-85.0	GRAVEL, sandy			(5)	
SS-24	88.5-90.0	"			(5)	
SS-25	93.5-95.0	"			(5)	
SS-26	98.5-100.0	"			(5)	
SS-27	103.5-105.0	"			(5)	
SS-28	108.5-110.0	"			(5)	
SS-29	113.5-115.0	GRAVEL with cobbles			(5)	
SS-30	118.5-120.0	SAND, gravelly, silty			(15)	

* ( ) = Assumed value

** Assumed saturated

Table 7

Samples for Evaluation at Site 3 (Z-9)

Sample	Depths (ft)	Description	Liquid Limit	Plasticity Index	Fines* (%)	USCS
SS-9	20.0-21.5	SAND, silty			(15)	
SS-13	30.0-31.5	SAND, clayey, silty	19	7	41	SC-SM
SS-16	48.5-50.0	SAND, silty	19	NP	18	SM
SS-19	63.5-65.0	SAND, slt silty			(15)	
SS-20	68.5-70.0	SAND, silty			(15)	
SS-22	78.5-80.0	GRAVEL, w/ sand, silty			(5)	
SS-23	83.5-85.0	SAND, silty w/ gravel	31	7	7	SW-SM
SS-24	88.5-90.0	"			(15)	
SS-25	94.5-95.0	SAND, w/ gravel, silty			(15)	
SS-26	98.5-100.0	"			(15)	
SS-27	103.5-105.0	"			(15)	

* ( ) = Assumed value

Table 8

Samples for Evaluation at Site 4 (Z-14)

Sample	Depths (ft)	Description	Liquid Limit	Plasticity Index	Fines* (%)	USCS
SS-8	17.5-19.0	SAND, silty, clayey			12	SM-SC
SS-14	38.5-40.0	SAND, clayey	28	10	31	SC
SS-16	48.5-50.0	SAND, silty, clayey	32	8	47	SM
SS-17	53.5-55.0	SAND, silty	27	4	35	SM
SS-18	58.5-60.0	"	21	NP	42	SM
SS-19	63.5-65.0	SAND, silty, clayey			27	SM-SC
SS-20	68.5-70.0	"			(15)	
SS-21	73.5-75.0	"			(15)	
SS-22	78.5-80.0	"			(15)	
SS-23	83.5-85.0	SAND, w/ gravel, silty			(15)	
SS-24	88.5-90.0	"			(15)	
SS-25	93.5-95.0	SAND, silty, clayey			44	SM-SC

* ( ) = Assumed value

49. The distribution of measured fines is shown in Figure 18. In general, the percentage of fines decreases rapidly with depth. Only five samples had 15 percent fines or less. As the underestimation of the percentage of fines increases, the evaluation of liquefaction becomes more conservative. Therefore, sands with an unknown percentage of fines were assumed to have 15 percent fines and gravels with an unknown percentage of fines were assumed to have 5 percent fines.

Shear stresses

50. Peak values of $(\tau_{eq})_{av}$ calculated for both horizontal components of the DBE and at all four sites are tabulated in Tables 9 and 10. The larger value produced from the two components is shown in bold print. The variation of shear stress with time at select depths for the four sites and both components of horizontal motion are presented in Appendix D. (Note that these plots correspond to depths at layer interfaces and, therefore, peak values may differ slightly from those in Tables 9 and 10 which correspond to the mid-height of layers.)

51. The earthquake-induced shear stress increases with depth, generally at a faster rate near the ground surface. These stresses are dependent not only on soil type but also on the earthquake motion as evidenced by having peak shear stresses at various depths produced from both horizontal components at Sites 2 through 4. A peak "composite" profile was used at each site for the assessment of liquefaction.

Shear strengths

52. The shear strengths calculated for each sample are listed in Tables 11 through 14. Samples are separated with respect to whether the fines content is known. Depths correspond to the mid-height of the N-value measurement (1.0 ft into each 1.5-ft SPT sample). The quantity of each correction or adjustment described previously is provided. Values of $(N_1)_{60}^{cs}$ were used along the curve in Figure 16 representing 5 percent fines. The DBE represents a magnitude 7.3 event which corresponds to constant value of $K_M = 1.03$.

53. In general, the soils at the four sites are highly resistant to liquefaction. Shear strengths are generally large and in most cases can not even be computed (indicated by the > sign and the superscript ^{c}). Also, the low to moderate values of shear strength that do exist are generally flanked above and below by much stronger materials.

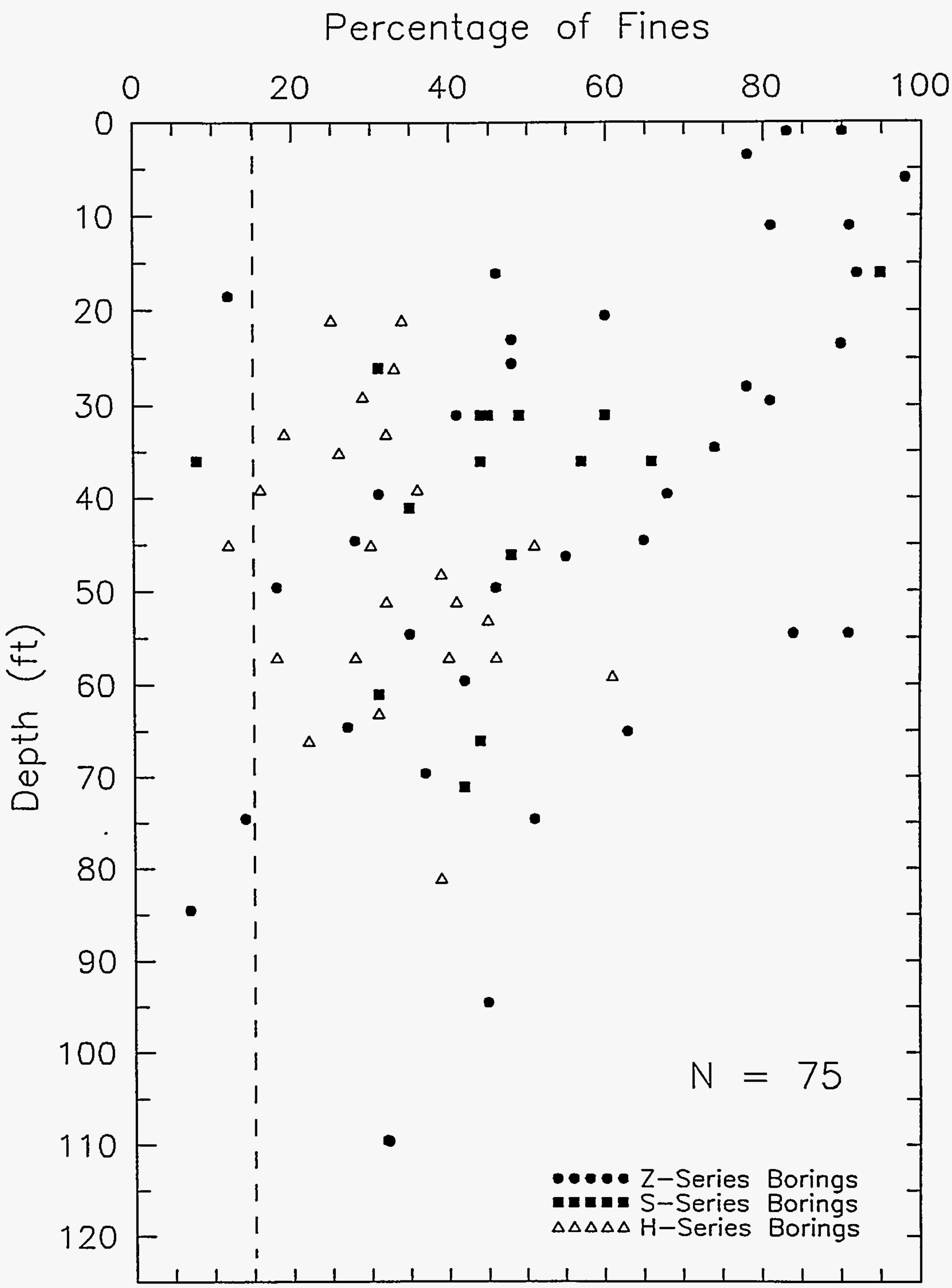

Figure 18. Distribution of measured fines

Table 9

Average Earthquake-Induced Shear Stresses for Sites 1 and 2

Site 1			Site 2		
Depth (ft)	$(\tau_{eq})_{av}$, in tsf		Depth (ft)	$(\tau_{eq})_{av}$, in tsf	
	Horizontal 1	Horizontal 2		Horizontal 1	Horizontal 2
7.5	0.06	**0.06**	5.5	0.04	**0.05**
16.5	0.11	**0.12**	16.0	0.11	**0.13**
23.0	0.15	**0.15**	25.5	0.16	**0.17**
48.0	0.24	**0.25**	35.0	0.20	**0.20**
95.5	0.35	**0.36**	49.0	0.25	**0.27**
			66.5	0.31	**0.33**
			94.0	**0.40**	0.36
			116.5	**0.43**	0.43

Table 10

Average Earthquake-Induced Shear Stresses for Sites 3 and 4

Site 3			Site 4		
Depth (ft)	$(\tau_{eq})_{av}$, in tsf		Depth (ft)	$(\tau_{eq})_{av}$, in tsf	
	Horizontal 1	Horizontal 2		Horizontal 1	Horizontal 2
6.0	**0.05**	0.04	4.5	0.04	**0.05**
20.0	**0.15**	0.12	16.0	0.12	**0.14**
48.5	**0.29**	0.28	30.5	0.19	**0.19**
72.0	**0.39**	0.36	48.0	0.25	**0.27**
87.5	**0.41**	0.40	73.0	**0.34**	0.34
115.0	0.43	**0.47**	93.0	**0.43**	0.38
			100.5	**0.45**	0.39

Table 11

Cyclic Shear Strengths at Site 1 (Z-1) Using Measured Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}^{cs**}$	CSR	$(\tau_{soil})_{av}$
SS-7	16.0	47	0.77	1.08	1.14	24	31	> 31[c]	> 0.50	> 0.43
SS-9	23.0	47	0.92	1.02	1.05	48	55	> 55[c]	> 0.50	> 0.50
SS-10	25.5	48	0.97	1.00	1.02	22	25	> 25[c]	> 0.50	> 0.50

* No estimates required.
** [c] = no clean sand equivalent

Table 12

Cyclic Shear Strengths at Site 2 (Z-5) Using Measured Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}^{cs**}$	CSR	$(\tau_{soil})_{av}$
SS-16+	49.5	47	2.99	0.77	0.62	24	17	26	0.30	0.71
SS-20+	69.5	37	3.67	0.71	0.55	33	20	29	0.40	> 1.00
SS-21+	74.5	13	3.79	0.70	0.47++	27	14	18	0.20	0.55
SS-22	79.5		3.93	0.68	0.53	NFP#	-	-^{c}	> 0.50	> 1.00
SS-23+	84.5		4.08	0.67	0.52	40	23	≥ 23	≥ 0.26	≥ 0.73
SS-24	89.5		4.23	0.66	0.50	83	46	> 46^c	> 0.50	> 1.00
SS-25	94.5		4.38	0.65	0.49	56	31	> 31^c	> 0.50	> 1.00
SS-26+	99.5		4.53	0.64	0.48	35	19	≥ 19	≥ 0.21	≥ 0.63
SS-27+	104.5		4.67	0.63	0.47	43	23	≥ 23	≥ 0.26	≥ 0.79
SS-28+	109.5		4.82	0.62	0.46	38	20	≥ 20	≥ 0.22	≥ 0.68
SS-29	114.5		4.97	0.60	0.45	54	27	≥ 27	≥ 0.33	> 1.00
SS-30	119.5		5.12	0.59	0.45	70	35	> 35^c	> 0.50	> 1.00

* Estimates of percentage of fines in Table 15
** ^{c} = no clean sand equivalent
\+ Determined to meet liquefaction criteria in previous study by ERCE (1990a)
\+\+ Density estimated to be in range of 40-60 percent
\# NFP = not full penetration

Table 13

Cyclic Shear Strengths at Site 3 (Z-9) Using Measured Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}^{cs}$**	CSR	$(\tau_{soil})_{av}$
SS-9	21.0		1.23	0.94	0.91	33	34	> 34^c	> 0.50	> 0.60
SS-13	31.0	41	1.52	0.91	0.84	28	26	> 30^c	> 0.50	> 0.71
SS-16+	49.5	18	2.04	0.86	0.63++	18	13	18	0.20	0.36
SS-19	64.5		2.52	0.81	0.68	56	43	> 43^c	> 0.50	> 1.00
SS-20	69.5		2.64	0.80	0.66	53	39	> 39^c	> 0.50	> 1.00
SS-22	79.5		2.94	0.77	0.62	61	42	> 42^c	> 0.50	> 1.00
SS-23	84.5	7	3.09	0.76	0.61	138	94	> 94^c	> 0.50	> 1.00
SS-24	89.5		3.23	0.74	0.59	64	42	> 42^c	> 0.50	> 1.00
SS-25	94.5		3.38	0.73	0.58	62	40	> 40^c	> 0.50	> 1.00
SS-26	99.5		3.53	0.72	0.56	67	42	> 42^c	> 0.50	> 1.00
SS-27	104.5		3.67	0.71	0.55	113	69	> 69^c	> 0.50	> 1.00

* Estimates of percentage of fines in Table 16
** ^{c} = no clean sand equivalent
+ Determined to meet liquefaction criteria in previous study by ERCE (1990a)
++ Density estimated to be in range of 40-60 percent

Table 14

Cyclic Shear Strengths at Site 4 (Z-14) Using Measured Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}^{cs}$**	CSR	$(\tau_{soil})_{av}$
SS-8	18.5	12	0.99	1.00	1.00	177	198	$>198^c$	> 0.50	> 0.51
SS-14+	39.5	31	2.40	0.82	0.61++	18	12	19	0.21	0.43
SS-16+	49.5	47	2.93	0.77	0.55++	8	5	11	0.12	0.28
SS-17+	54.5	35	3.08	0.76	0.53++	22	13	21	0.23	0.55
SS-18	59.5	42	3.23	0.74	0.59	34	22	$> 30^c$	> 0.50	> 1.00
SS-19	64.5	27	3.38	0.73	0.58	189	122	$>122^c$	> 0.50	> 1.00
SS-20	69.5		3.52	0.72	0.56	118	74	$> 74^c$	> 0.50	> 1.00
SS-21	74.5		3.67	0.71	0.55	84	52	$> 52^c$	> 0.50	> 1.00
SS-22	79.5		3.82	0.69	0.54	78	47	$> 47^c$	> 0.50	> 1.00
SS-23	84.5		3.97	0.68	0.53	60	36	$> 36^c$	> 0.50	> 1.00
SS-24	89.5		4.11	0.67	0.51	48	27	≥ 27	≥ 0.33	≥ 0.94
SS-25+	94.5	44	4.26	0.66	0.44++	26	13	21	0.23	0.67

* Estimates of percentage of fines in Table 17

** ^{c} = no clean sand equivalent

\+ Determined to meet liquefaction criteria in previous study by ERCE (1990a)

++ Density estimated to be in range of 40-60 percent

54. The shear strengths using assumed values of fines where necessary are summarized in Tables 15 through 17 for Sites 2 through 4 (no assumptions necessary at Site 1). In general, there is no impact by assuming fines content since most of these samples with lower shear strengths are gravel and assuming clean gravel (5 percent fines) does not change the results.

Factor of safety against liquefaction

55. Comparisons between shear stresses and inferred shear strengths for the delineated cohesionless samples are made in Figures 19 through 22. Only one of the 38 samples considered has $FS_L \leq 1.15$ (Sample SS-16 taken between depths of 48.5 and 50.0 ft at Site 4). Furthermore, the samples directly above and below Sample SS-16 are expected to have a strong resistance to the generation of excess pore water pressure. A less-likely chance for the build up of excess pore water pressures exists at Site 3 at a depth of 50.0 ft ($FS_L = 1.22$). Again, the samples above and below this point are much more dense.

Effect of buildings

56. Rollins and Seed (1990) showed that shear stresses imposed by buildings can affect the potential for liquefaction. The ratio of spectral accelerations to peak ground acceleration for the four sites and both components of the DBE is shown in Figure 24. Also shown in this figure is a line segment representing a ratio threshold of 2.4 suggested by Rollins and Seed (1990) to define which periods of motion are amplified by the presence of the building. The eight spectra fall below the threshold except at periods of motion less than 0.5 sec (frequencies greater than 2 Hz). Only three spectra break the threshold at periods greater than 0.28 sec.

57. Engineering Decision Analysis Company, Inc. (1981) reported the natural modes of horizontal vibration for the main process buildings (331 and 333). The first mode for the 333 structures and the first and second modes for the 331 structures are at periods greater than 0.5 sec. The second mode for the 333 structures and the third mode for the 331 structures are greater than 0.27 sec. At these natural periods the presence of the buildings is not expected to increase the liquefaction potential, and therefore building settlements.

Table 15

Cyclic Shear Strengths at Site 2 (Z-5) Using Estimated Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}^{cs**}$	CSR	$(\tau_{soil})_{av}$
SS-16+	49.5	47	2.99	0.77	0.62	24	17	26	0.30	0.71
SS-20+	69.5	37	3.67	0.71	0.55	33	20	29	0.40	> 1.00
SS-21+	74.5	13	3.79	0.70	0.47++	27	14	18	0.20	0.55
SS-22	79.5	(5)	3.93	0.68	0.53	NFP#	-	-^{c}	> 0.50	> 1.00
SS-23+	84.5	(5)	4.08	0.67	0.52	40	23	23	0.26	0.73
SS-24	89.5	(5)	4.23	0.66	0.50	83	46	> 46^c	> 0.50	> 1.00
SS-25	94.5	(5)	4.38	0.65	0.49	56	31	> 31^c	> 0.50	> 1.00
SS-26+	99.5	(5)	4.53	0.64	0.48	35	19	19	0.21	0.63
SS-27+	104.5	(5)	4.67	0.63	0.47	43	23	23	0.26	0.79
SS-28+	109.5	(5)	4.82	0.62	0.46	38	20	20	0.22	0.68
SS-29	114.5	(5)	4.97	0.60	0.45	54	27	27	0.33	> 1.00
SS-30	119.5	(15)	5.12	0.59	0.45	70	35	> 35^c	> 0.50	> 1.00

* ( ) = Estimated percentage of fines
** ^{c} = no clean sand equivalent
+ Determined to meet liquefaction criteria in previous study by ERCE (1990a)
++ Density estimated to be in range of 40-60 percent
\# NFP = not full penetration

Table 16

Cyclic Shear Strengths at Site 3 (Z-9) Using Estimated Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}$ cs**	CSR	$(\tau_{soil})_{av}$
SS-9	21.0	(15)	1.23	0.94	0.91	33	34	> 34[c]	> 0.50	> 0.60
SS-13	31.0	41	1.52	0.91	0.84	28	26	> 30[c]	> 0.50	> 0.71
SS-16+	49.5	18	2.04	0.86	0.63++	18	13	18	0.20	0.36
SS-19	64.5	(15)	2.52	0.81	0.68	56	43	> 43[c]	> 0.50	> 1.00
SS-20	69.5	(15)	2.64	0.80	0.66	53	39	> 39[c]	> 0.50	> 1.00
SS-21	74.5	51	2.79	0.78	0.64	32	23	30	> 0.50	> 1.00
SS-23	84.5	7	3.09	0.76	0.61	138	94	> 94[c]	> 0.50	> 1.00
SS-24	89.5	(15)	3.23	0.74	0.59	64	42	> 42[c]	> 0.50	> 1.00
SS-25	94.5	(15)	3.38	0.73	0.58	62	40	> 40[c]	> 0.50	> 1.00
SS-26	99.5	(15)	3.53	0.72	0.56	67	42	> 42[c]	> 0.50	> 1.00
SS-27	104.5	(15)	3.67	0.71	0.55	113	69	> 69[c]	> 0.50	> 1.00

* ( ) = Estimated percentage of fines
** [c] = no clean sand equivalent
\+ Determined to meet liquefaction criteria in previous study by ERCE (1990a)
\++ Density estimated to be in range of 40-60 percent

Table 17

Cyclic Shear Strengths at Site 4 (Z-14) Using Estimated Percentages of Fines

Sample	Depth (ft)	Fines* (%)	σ_v'	K_σ	C_N	N	$(N_1)_{60}$	$(N_1)_{60}{}^{cs**}$	CSR	$(\tau_{soil})_{av}$
SS-8	18.5	12	0.99	1.00	1.00	177	198	>198^c	> 0.50	> 0.51
SS-14$^+$	39.5	31	2.40	0.82	0.61^{++}	18	12	19	0.21	0.43
SS-16$^+$	49.5	47	2.93	0.77	0.55^{++}	8	5	11	0.12	0.28
SS-17$^+$	54.5	35	3.08	0.76	0.53^{++}	22	13	21	0.23	0.55
SS-18	59.5	42	3.23	0.74	0.59	34	22	> 30^c	> 0.50	> 1.00
SS-19	64.5	27	3.38	0.73	0.58	189	122	>122^c	> 0.50	> 1.00
SS-20	69.5	(15)	3.52	0.72	0.56	118	74	> 74^c	> 0.50	> 1.00
SS-21	74.5	(15)	3.67	0.71	0.55	84	52	> 52^c	> 0.50	> 1.00
SS-22	79.5	(15)	3.82	0.69	0.54	78	47	> 47^c	> 0.50	> 1.00
SS-23	84.5	(15)	3.97	0.68	0.53	60	36	> 36^c	> 0.50	> 1.00
SS-24	89.5	(15)	4.11	0.67	0.51	48	27	> 27^c	> 0.50	> 1.00
SS-25$^+$	94.5	44	4.26	0.66	0.44^{++}	26	13	21	0.23	0.67

* ( ) = Estimated percentage of fines

** ^{c} = no clean sand equivalent

$^+$ Determined to meet liquefaction criteria in previous study by ERCE (1990a)

$^{++}$ Density estimated to be in range of 40-60 percent

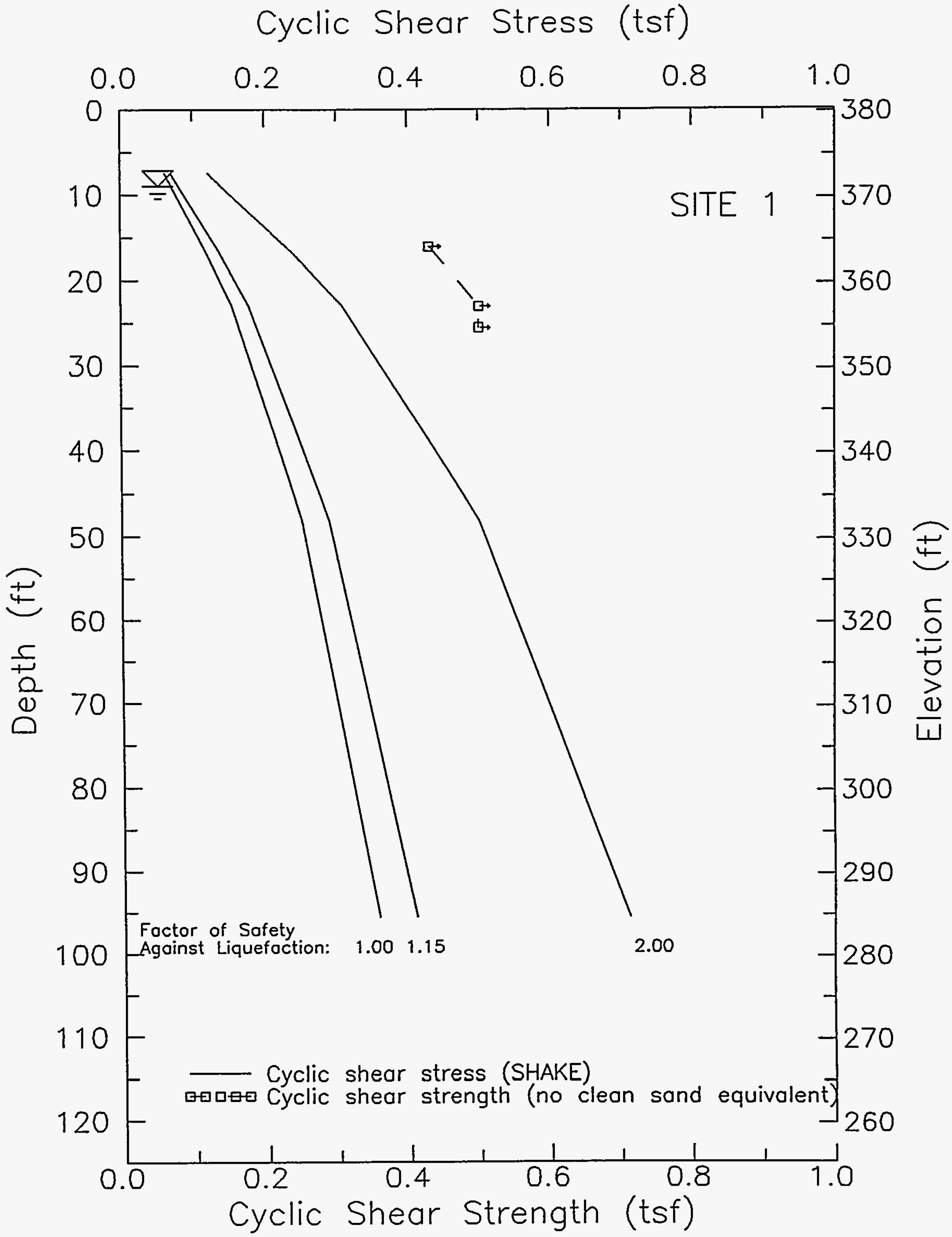

Figure 19. Comparison of shear stresses and shear strengths for Site 1 along with factors of safety against liquefaction

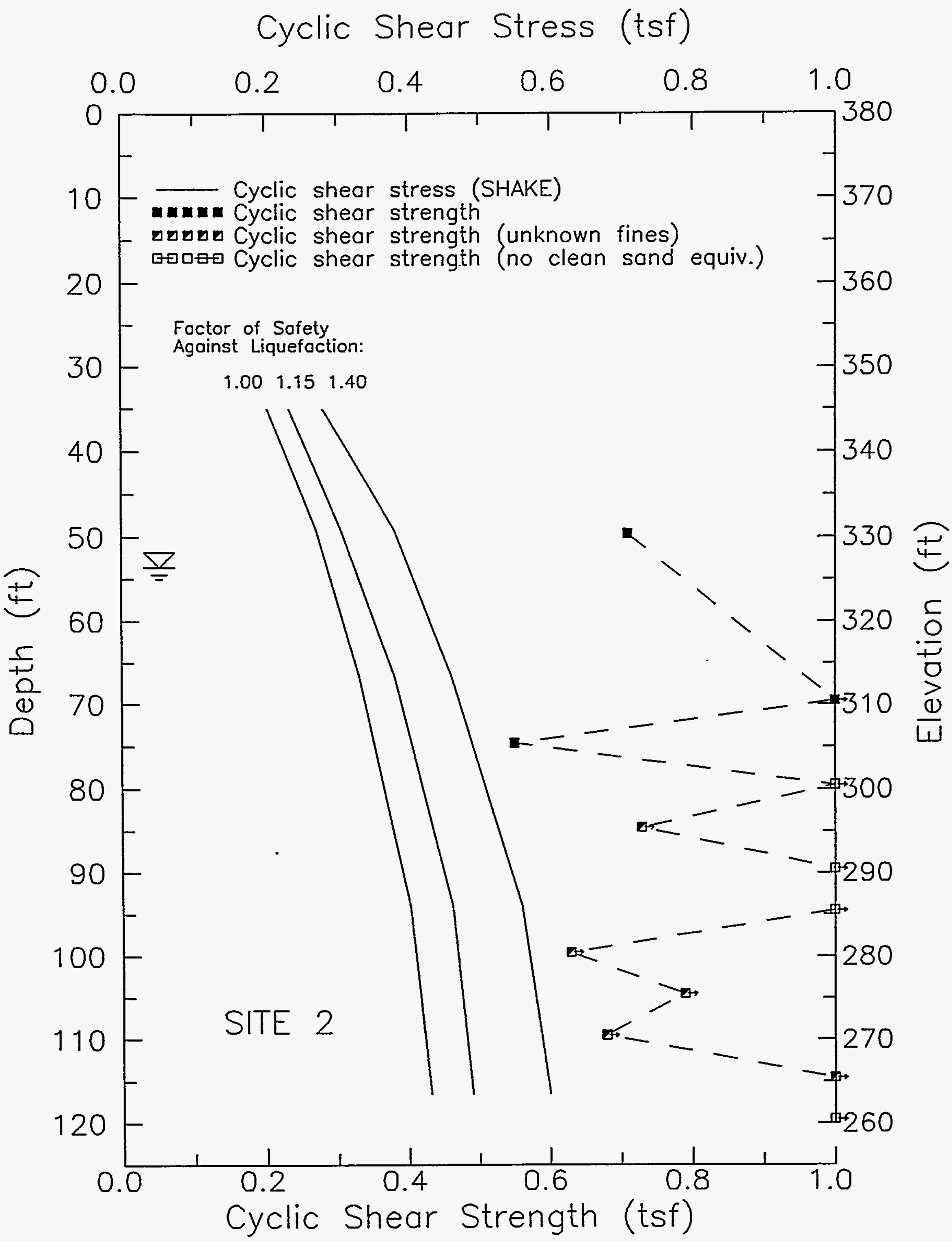

Figure 20. Comparison of shear stresses and shear strengths for Site 2 along with factors of safety against liquefaction

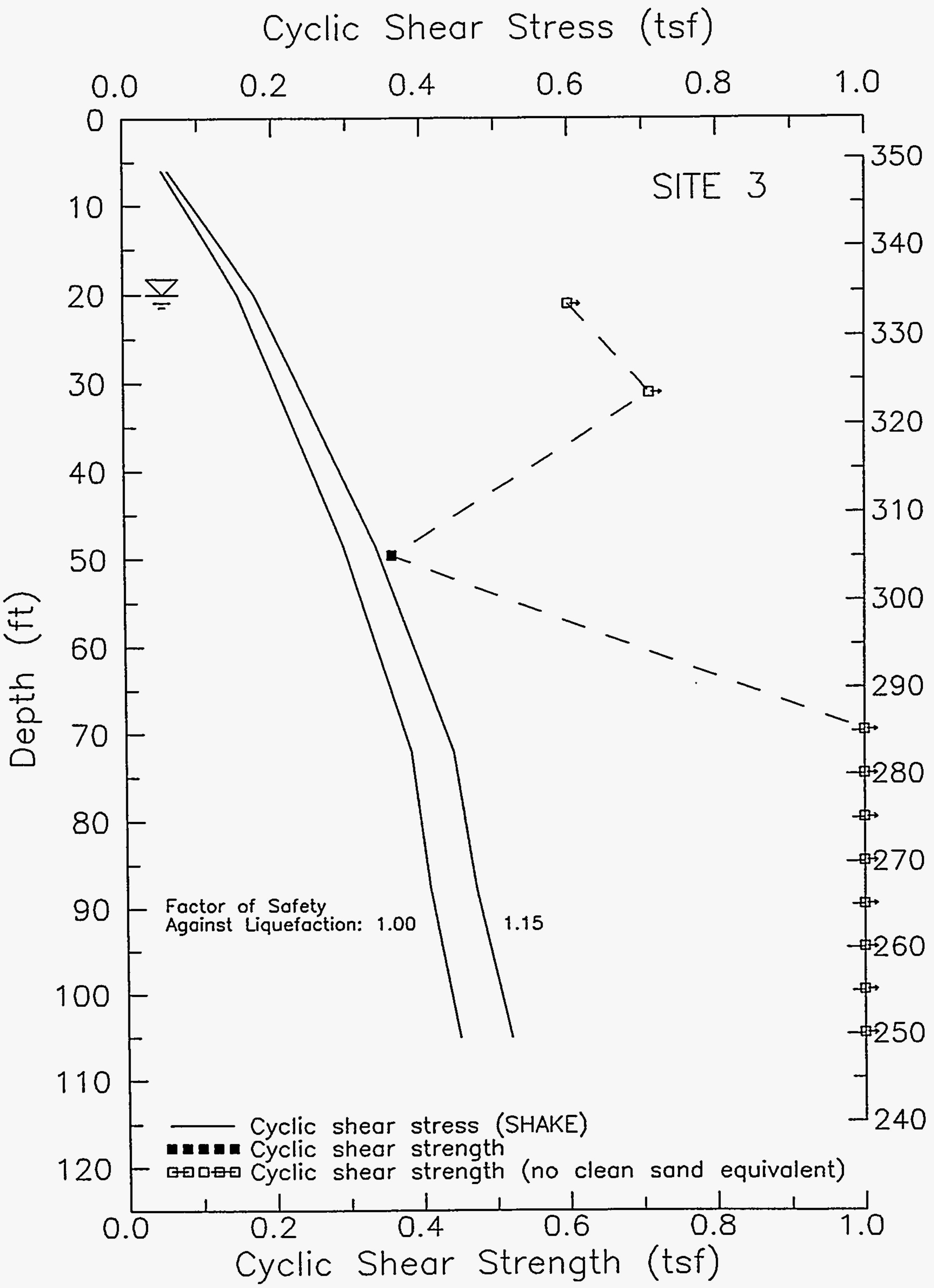

Figure 21. Comparison of shear stresses and shear strengths for Site 3 along with factors of safety against liquefaction

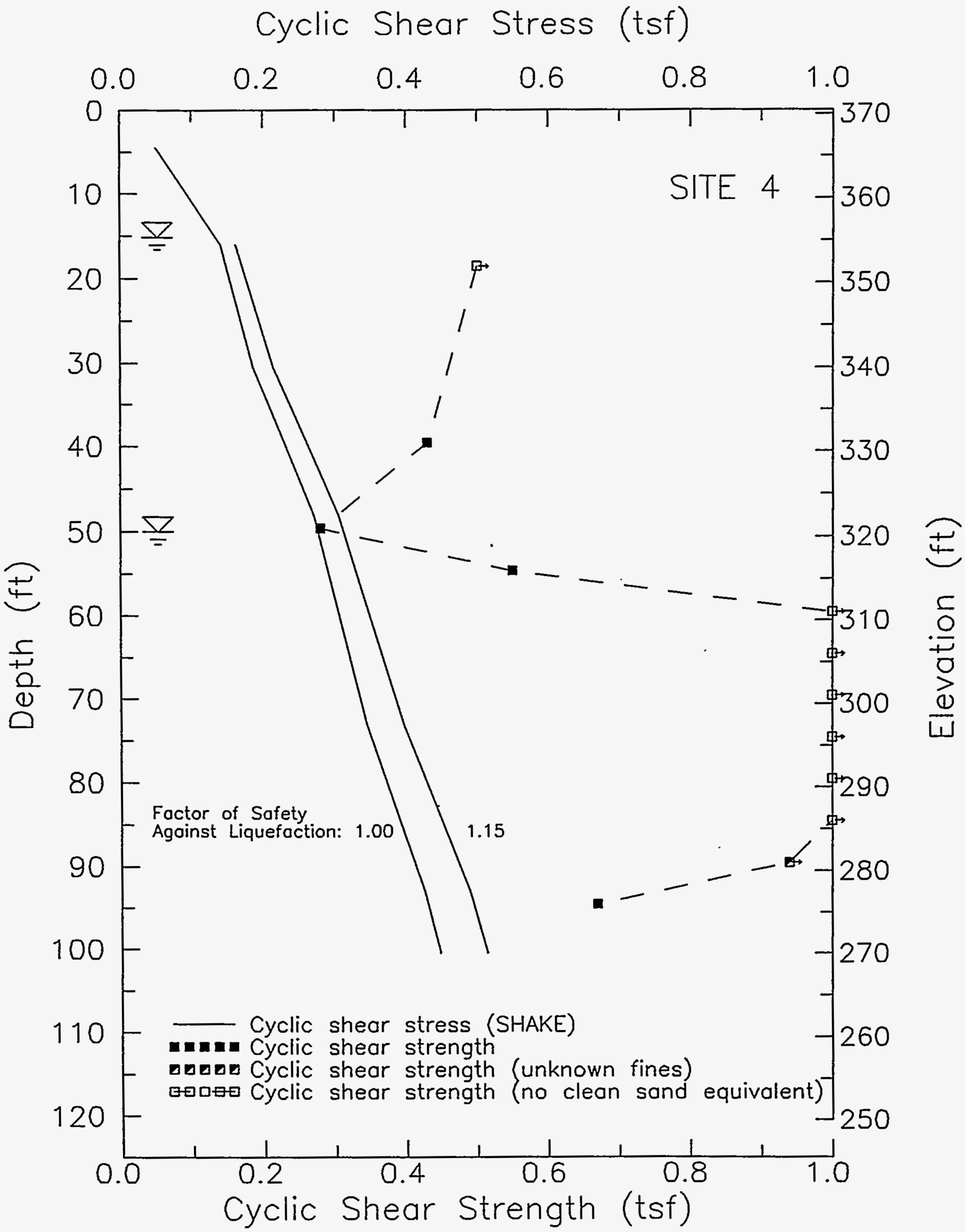

Figure 22. Comparison of shear stresses and shear strengths for Site 4 along with factors of safety against liquefaction

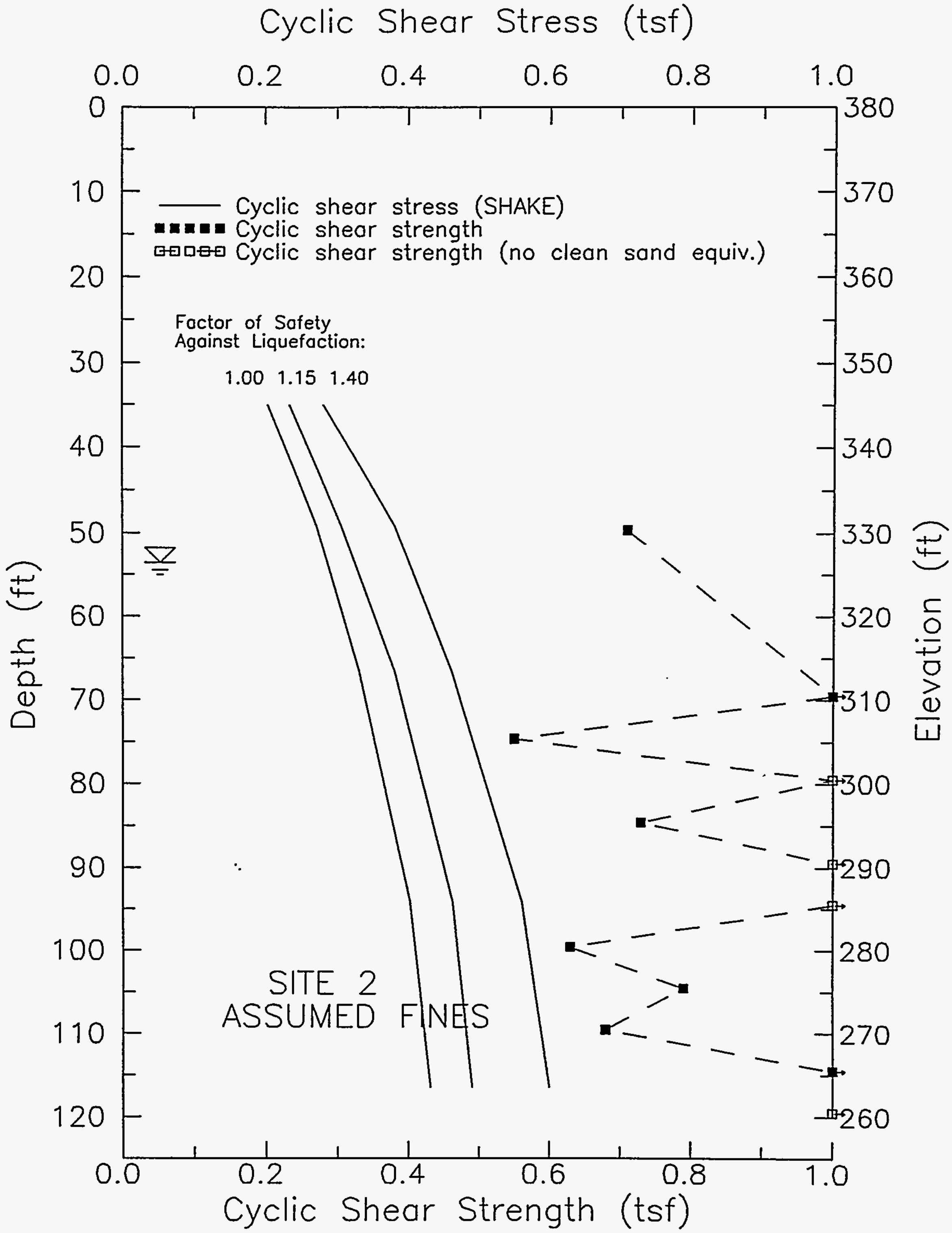

Figure 23. Comparison of shear stresses and shear strengths for Site 2 using some inferred values of fines along with factors of safety against liquefaction

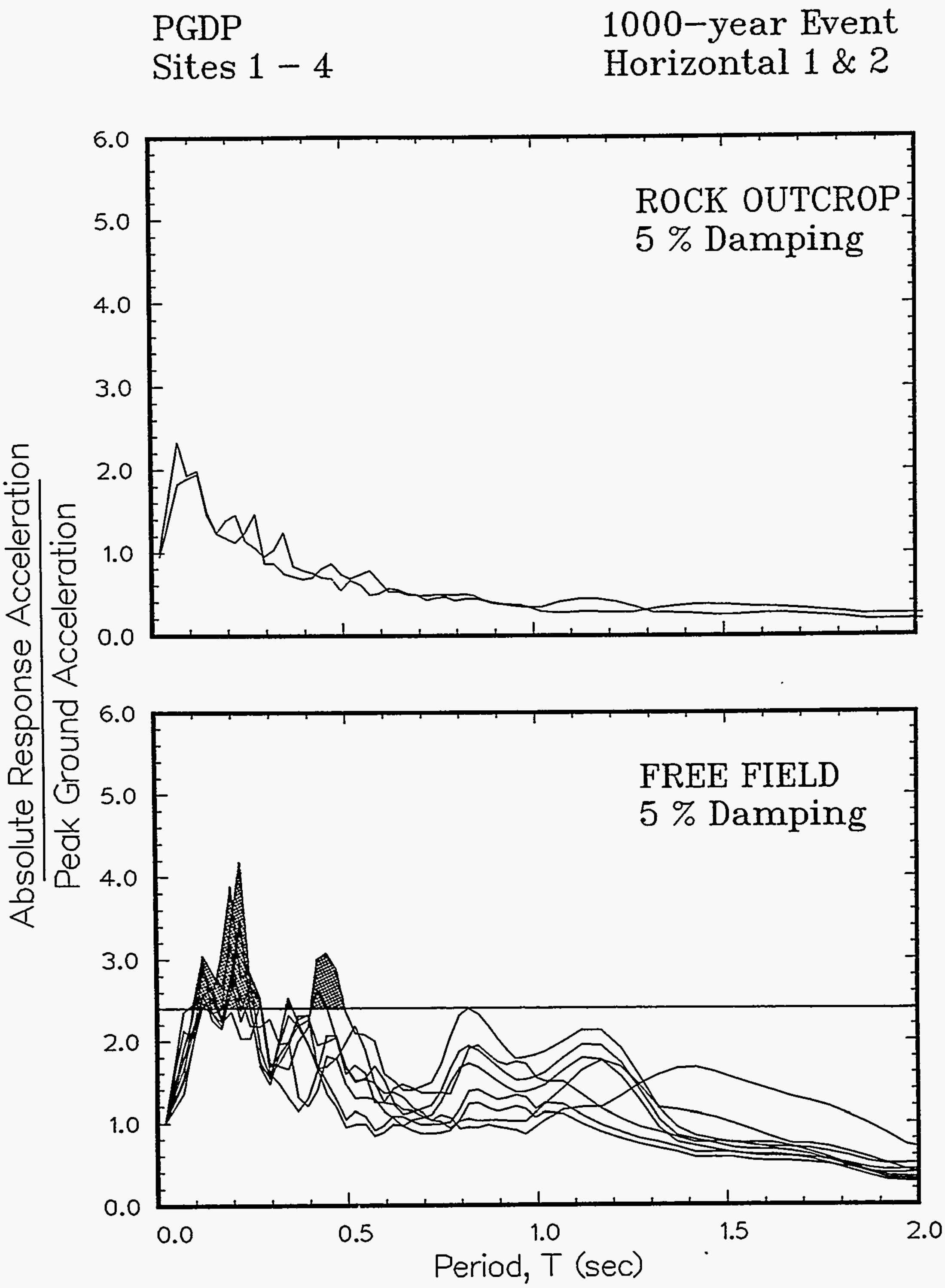

Figure 24. Comparison of ratio of spectral acceleration to peak ground acceleration with threshold proposed by Rollins and Seed (1990)

Conclusions

58. Liquefaction is not expected to occur at PGDP based on empirical methods of determining soil resistance and numerical methods of determining the vertical distribution of shear stresses. None of the 38 samples examined(including 12 samples expected to liquefy based on previous evaluations by others) as part of this study is expected to liquefy. Only one sample had $FS_L \leq 1.15$ indicating that some pore pressure development is expected and one other sample had $FS_L \leq 1.4$. Excess pore water pressures may cause earthquake-induced settlements which are analyzed in Part V.

PART V: ESTIMATION OF EARTHQUAKE-INDUCED SETTLEMENTS

59. The ability to estimate earthquake-induced settlements within reasonable bounds has developed only within the past decade. The two prominent methods presently available were proposed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) for free-field, level-ground conditions. These methods were derived for young deposits of sand (Holocene or recent). The application of these methods to older deposits provides some inherent conservatism (similar to conservatism associated with older deposits in liquefaction assessment presented in Part IV). The effect of applying these methods to gravelly soils is unknown because of the uncertainties surrounding SPT measurements in gravel.

60. These methods generally are only accurate to within a factor of two. More accurate means to determine settlements have not been sufficiently verified. Numerical methods may be used, however, to evaluate the effect of such factors as spatial variability, depth to the liquefied zone, and the presence of a clay cap overlying the liquefiable layer on surface settlements and the effect of variable surface settlements on stresses and moments in the superstructure. The method proposed by Ishihara and Yoshimine (1992) was used for this assessment.

Methodology

61. Ishihara and Yoshimine (1992) developed a family of curves used to derive volumetric strain from simple shear tests on clean sands performed in the laboratory. The independent variables needed are relative density (or penetration resistance) and FS_L. Their method was validated with measurements made following the 1964 Niigata earthquake in Japan.

62. The basis for this method is a family of curves relating FS_L and post-liquefaction volumetric strain as shown in Figure 25. Each curve represents a different relative density (which can be correlated with N-value or cone penetration test tip resistance) and are specific to clean sands. N-values are specific to Japanese SPT equipment and their standard energy ratio of 55 percent.

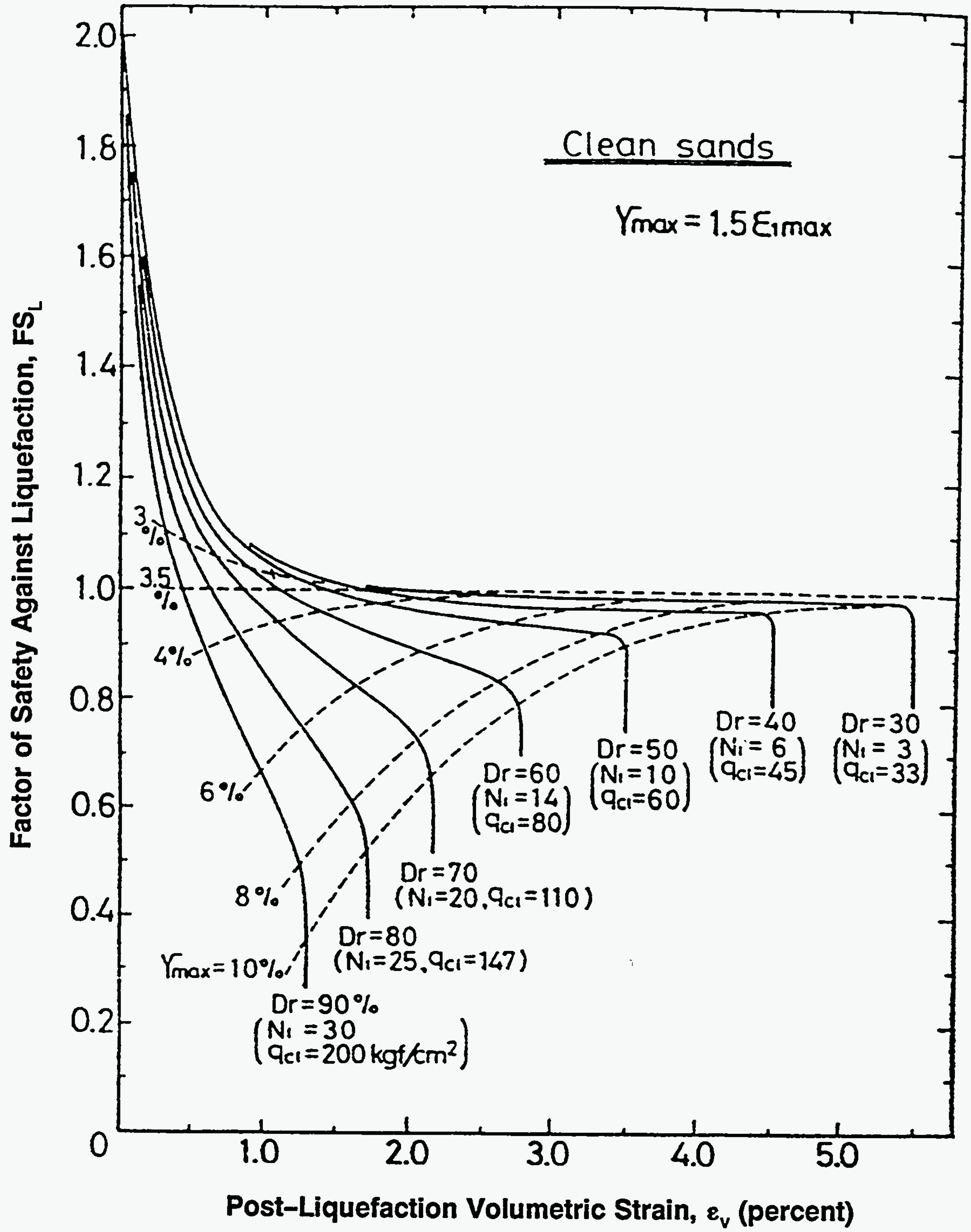

Figure 25. Chart showing relationship between density and vertical strain in soils induced by earthquakes (Ishihara and Yoshimine 1992)

63. As expected, the volumetric strain is highly dependent on FS_L and relative density. Only small volumetric strains (< 1 percent) tend to occur when FS_L < 1.1. At large relative densities ($D_r \geq 90$ percent corresponding to $(N_1)_{55}^{cs} \geq 30$), the volumetric strain remains relatively small even at very low values of FS_L. Most soils at the four sites fall into one of these two groups indicating that earthquake-induced settlements are not generally considered to be a significant hazard.

Evaluation for PGDP

64. The first step in the evaluation was the conversion of N-values to Japanese equivalents. Values of $(N_1)_{60}^{cs}$ were used in the analysis to have equivalent clean sand values and converted using Ishihara's (Letter: Oct 19,1994) recommendations:

$$(N_1)_j^{cs} = 0.833\ (N_1)_{60}^{cs} \tag{9}$$

65. The most critical zone, located at Site 4 at depths between 47.5 and 52.5 ft, was considered to estimate the maximum magnitude of earthquake-induced settlements. The equivalent $(N_1)_j^{cs}$ for the sample in this zone is 9. With a FS_L = 1.02, the volumetric strain is 1.4 percent as shown in Figure 25. Assuming one-dimensional straining, that translates into a settlement of less than 1.0 inch (0.014 in./in. x 60 in.).

66. The calculated settlement is considered to be conservative because the methodology was developed for and using young soils, not older soils present at PGDP, and because much denser soils typically lie above and below the weaker zones that will likely allow instantaneous dissipation of excess pore water pressures. If one inch settlement were to occur in a layer at a depth of about 50 ft, it is likely that less settlement would occur at the ground surface because significant stress redistribution would likely occur.

PART VI: SUMMARY AND CONCLUSIONS

67. A reassessment of liquefaction potential and an estimation of earthquake-induced settlements were conducted at four independent locations at the Paducah Gaseous Diffusion Plant. The design basis earthquake for this study was a 1000-year earthquake with a Magnitude 7.3. These analyses used the results of previous site response calculations (Sykora and Davis 1993) using the computer program *SHAKE* and the results of field investigations of material properties by ERCE, Inc. (1990b) to update previous assessments by ERCE, Inc. (1990a).

68. The PGDP is situated on a upland surface above the alluvial valley of the Ohio River. The soil deposits consist of loess overlying older alluvium (continental deposits) and then Tertiary-age deposits and hard rock. The younger loess is typically dry and the saturated granular materials are typically Pleistocene-age or older. Two groundwater systems affect alluvial deposits at PGDP—a shallow and a regional gravel system; both systems were considered to calculate effective stresses.

69. Liquefaction is not expected to occur at the four sites during the DBE. The smallest factor of safety against liquefaction was 1.02 indicating that liquefaction is not expected to occur. However, some generation of excess pore water pressure could occur at isolated locations in Pleistocene-aged sands and gravels (continental deposits). Excess pore water pressure is expected to dissipate rapidly in more dense layers above and below the isolated zones.

70. Earthquake-induced settlements could occur at isolated locations in the granular continental deposits where large excess pore water pressures are generated. If these pore water pressures do not dissipate rapidly, the maximum settlement is estimated to be less than 1.0 inch within the sand layer at a depth of about 50 ft. The amount of settlement at the ground surface is expected to be less than that occurring within the sand layer because of stress redistribution and the presence of a predominantly clayey cap in the continental deposits.

71. These analyses are considered to be conservative because the general methods available assume that the soils are young, Holocene-age or recent deposits. The older soils at PGDP are expected to produce much less excess pore water pressures and earthquake-induced settlements.

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APPENDIX A: GROUNDWATER MONITORING WELLS AND FLUCTUATIONS

CONTENTS

Table A1

Description of Groundwater Monitoring Wells

Well ID	West Coord.		North Coord.		Elevation (Ground)		Screen Interval	StratZone Interval	Zone ID
MW46	5881.88	W	-1069.49	N	374.1		68.6-78.6	66-TD	RGS
MW47	5872.27	W	-1070.53	N	a		31.7-33.7	28-37	SGS
MW48	6197.73	W	-1061.44	N	373.6		68-78	66-TD	RGS
MW49	6190.85	W	-1062.36	N	a		34.6-36.6	28-37	SGS
MW63	7234.94	W	896.09	N	369.93		58.5-63.5	59-TD	RGS
MW64	7234.63	W	881.04	N	369.93		27.8-32.8	21.5-33	SGS
MW68	4358.53	W	-2073.57	N	377.33		97.4-102.4	65-TD	RGS
MW69	4343.85	W	-2073.77	N	377.33		33.3-38.3	22-37	SGS
MW81	5499.92	W	-880.84	N	373.74		65.9-85.9	69-86	RGS
MW82	5510.2	W	-846.01	N	373.71		30.8-40.8	22-38	SGS
MW83	5540.13	W	-846.22	N	373.81		30.4-40.4	22-38	SGS
MW84	5975.07	W	-803.71	N	372.62		65.5-75.9	66-TD	RGS
MW85	5960.3	W	-804.3	N	372.81		29.7-40.1	28-37	SGS
MW86	5945.24	W	-804.9	N	374.35		75.2-85.6	66-TD	RGS
MW87	5825.09	W	-804.98	N	372.96		63.9-74.3	59-TD	RGS
MW88	5809.7	W	-804.68	N	373.09		29.1-39.7	17-34	SGS
MW89	5795.14	W	-804.13	N	372.84		77.1-88.1	59-TD	RGS
MW91	5660.09	W	-804.36	N	371.63		28.6-39		b
MW92	5645	W	-805.26	N	371.8		78.9-89.3	54-TD	RGS
MW93	5995.09	W	-1028.68	N	374.96		69.5-79.9	66-TD	RGS
MW94	5979.82	W	-1028.77	N	375.04		28.7-39.1	28-37	SGS
MW122	-2069	W	738	N	362.9		148-158	91-TD	MCN
MW124	-2067	W	743	N	362.65	c	83-93	54-91	RGS
MW126	-2067	W	744	N	362.57	c	55-65	54-91	RGS
MW128	-2067	W	764	N	362.58	c	33-43	36-40	SGS
MW121	5677.65	W	6161.53	N	372.41	c	200-210	88-211.5	MCN
MW123	5661.33	W	6125.6	N	372.74	c	63-73	57-88	RGS
MW125	5662.81	W	6139.28	N	372.67	c	78-88	57-88	RGS
MW127	5664.11	W	6161.23	N	372.43	c	29-39		b
MW129	1527	W	-5867	N	383.8	c	44.5-49.5	18-54	EOC
MW130	1515	W	-5867	N	383.86	c	34.5-39.5	18-54	EOC
MW131	1500	W	-5867	N	383.64	c	20-30	18-54	EOC
MW133	1700	W	9150	N	334.7		80-90	52-TD	MCN
MW135	1720.9	W	9137.28	N	333.51		41-51	31-52	RGS
MW137	1726.75	W	9150.86	N	333.2		34-39	31-52	RGS
MW138	1734.38	W	9163.18	N	333.21		10-20		b
MW140	12450	W	7075	N	341.83		136-146	78-TD	MCN
MW141	12173	W	6544.69	N	342.51		68-78	29-78	RGS
MW142	12162.5	W	6529.75	N	342.72		42-52	29-78	RGS
MW143	12156.1	W	6513.64	N	342.85		10-20	6-16	SGS

(continued)

Table A1 (cont.)

Well ID	West Coord.	North Coord.	Elevation (Ground)	Screen Interval	StratZone Interval	Zone ID
MW148	-3228.82 W	5754.34 N	371.08	60-90	45-TD	RGS
MW149	-3239.67 W	5755.06 N	371.3	50-60	45-TD	RGS
MW152	692.64 W	13136.67 N	351.61	45-75	40-TD	RGS
MW153	695.33 W	13122.54 N	351.43	25-35		b

EOC = Eocene Sands
MCN = Clayton/McNairy Sands and Silts
RGS = Regional Gravel Aquifer
SGS = Shallow Gravel and Sand Aquifer
a = Data not available
b = SGS not present,
screened zone is in fine grained continental deposits
c = Elevation of concrete pad (about 0.5 feet above ground surface)

Table A2

Groundwater Elevation Data for Period 1/89 to 2/91

Well ID	Maximum (feet msl)	Minimum (feet msl)	Fluctuation (feet)	Avg (feet msl)	STD Dev. (feet)	N OBS.
MW46	326.1	322.86	3.24	324.48	1.62	2
MW47	359.16	359.16	0	359.16	0	1
MW48	322.9	322.9	0	322.9	0	1
MW49	352.26	352.26	0	352.26	0	1
MW63	326.26	322.25	4.01	324.27	1.64	3
MW64	362.24	360.89	1.35	361.62	0.56	3
MW68	325.13	322.32	2.81	323.42	1.22	3
MW69	341.01	340.39	0.62	340.7	0.31	2
MW81	324.97	322.6	2.37	323.79	1.19	2
MW82	361.03	355.21	5.82	358.8	2.56	3
MW83	362.17	357.31	4.86	360.28	2.12	3
MW84	325.02	322.57	2.45	323.8	1.23	2
MW85	358.15	357.85	0.3	358	0.12	3
MW86	325.04	322.57	2.47	323.81	1.24	2
MW87	325.02	322.61	2.41	323.82	1.2	2
MW88	358.72	358.36	0.36	358.52	0.15	3
MW89	324.99	322.59	2.4	323.79	1.2	2
MW91	361.37	359.77	1.6	360.69	0.68	3
MW92	327.06	322.58	4.48	324.88	1.83	3
MW93	327.09	322.74	4.35	325	1.78	3
MW94	357.88	357.07	0.81	357.51	0.34	3
MW122	325.45	325.45	0	325.45	0	1
MW124	326.49	326.49	0	326.49	0	1
MW126	326.5	326.5	0	326.5	0	1
MW128	329.06	329.06	0	329.06	0	1
MW121	321.02	321.02	0	321.02	0	1
MW123	323.94	323.94	0	323.94	0	1
MW125	323.96	323.96	0	323.96	0	1
MW127	348.45	348.45	0	348.45	0	1
MW129	371.41	371.41	0	371.41	0	1
MW130	371.51	371.51	0	371.51	0	1
MW131	371.5	371.5	0	371.5	0	1
MW133	321.49	321.49	0	321.49	0	1
MW135	321.09	321.09	0	321.09	0	1
MW137	321.09	321.09	0	321.09	0	1
MW138	324.57	324.57	0	324.57	0	1

(continued)

Table A2 (cont.)

Well ID	Maximum (feet msl)	Minimum (feet msl)	Fluctuation (feet)	Avg (feet msl)	STD Dev. (feet)	N OBS.
MW140	321.85	321.85	0	321.85	0	1
MW141	324.16	324.16	0	324.16	0	1
MW142	324.92	324.92	0	324.92	0	1
MW143	332.66	332.66	0	332.66	0	1
MW148	324.06	324.06	0	324.06	0	1
MW149	324.06	324.06	0	324.06	0	1
MW152	315.54	315.54	0	315.54	0	1
MW153	ND*	ND	ND	ND	ND	ND

*ND indicates no data collected during the monitoring period.

APPENDIX B: LOCATIONS OF BORINGS USED FOR SOIL COLUMNS

Site	Boring No.	Coordinates (ft)		Elevation* (ft)	
		Northing	Easting	Top of Hole	Bottom of Hole
1	Z-1	S 5955.57	W 4327.82	380.3	251.3
	Z-2	S 5956.08	W 4312.61	380.4	311.9
	Z-3	S 5955.68	W 4342.33	380.1	311.6
2	Z-5	N 297.88	W 891.52	379.9	239.9
	Z-6	N 297.66	W 876.46	380.1	241.1
	Z-7	N 297.72	W 861.40	380.0	241.0
3	Z-9	N 12075.30	W 2930.84	354.6	229.6
	Z-10	N 12059.93	W 2930.41	353.7	229.7
	Z-11	N 12045.08	W 2930.60	354.2	230.2
	Z-12	N 12044.52	W 2980.58	351.1	-17.9
4	Z-13	S 385.11	W 8396.49	371.6	247.6
	Z-14	S 385.28	W 8381.33	371.5	238.0
	Z-15	S 385.15	W 8366.03	371.2	246.9
	Z-16	S 385.15	W 8436.66	370.9	14.3

* MSL

APPENDIX C: MEASURED UNIT WEIGHTS

Table C1

Measured Unit Weights ERCE, Inc.(1990b)

Boring	Sample	Range in Depths (ft)	Moist Unit Weight (pcf)	Moisture Content (%)	Dry Unit Weight (pcf)
Loess:					
Z-2	ST-1	4.3 - 6.3	123.5	24.5	99.2
Z-3	ST-1	12.0 - 14.0	125.7	16.6	107.8
Z-6	ST-1	9.0 - 11.0	120.9	21.5	99.5
Z-6	ST-2	24.0 - 26.0	127.3	16.9	108.9
Z-10	ST-1	9.0 - 11.0	124.1	25.6	98.8
Z-15	ST-1	4.0 - 6.0	124.6	21.7	102.4
Averages:			124.4		102.8
Continental Deposits:					
Z-1	ST-1	17.5 - 19.5	136.1	16.1	117.2
"	ST-2	29.5 - 30.6	100.7	63.2	61.7
Z-2	ST-2	24.1 - 25.8	97.4	23.7	78.7
"	ST-3	34.1 - 35.3	93.3	33.3	69.9
Z-3	ST-2	31.0 - 31.9	95.8	35.5	70.7
Averages (Site 1):			104.7		79.6
Z-6	ST-3	39.0 - 41.0	127.4	19.6	106.5
"	ST-4	54.0 - 56.0	123.6	23.7	99.9
Z-10	ST-2	19.0 - 21.0	124.1	17.8	105.4
"	ST-3	34.0 - 36.0	132.1	16.9	112.9
"	ST-4	49.0 - 51.0	116.7	23.3	94.6
Z-13	ST-1	49.0 - 51.0	123.2	23.1	100.1
Z-14	ST-1	21.5 - 23.5	119.6	18.7	100.7
"	ST-2	43.5 - 45.2	131.3	18.8	110.5
Averages (Sites 2-4):			117.0		94.5

APPENDIX D: VARIATION OF SHEAR STRESSES IN SOIL PROFILES

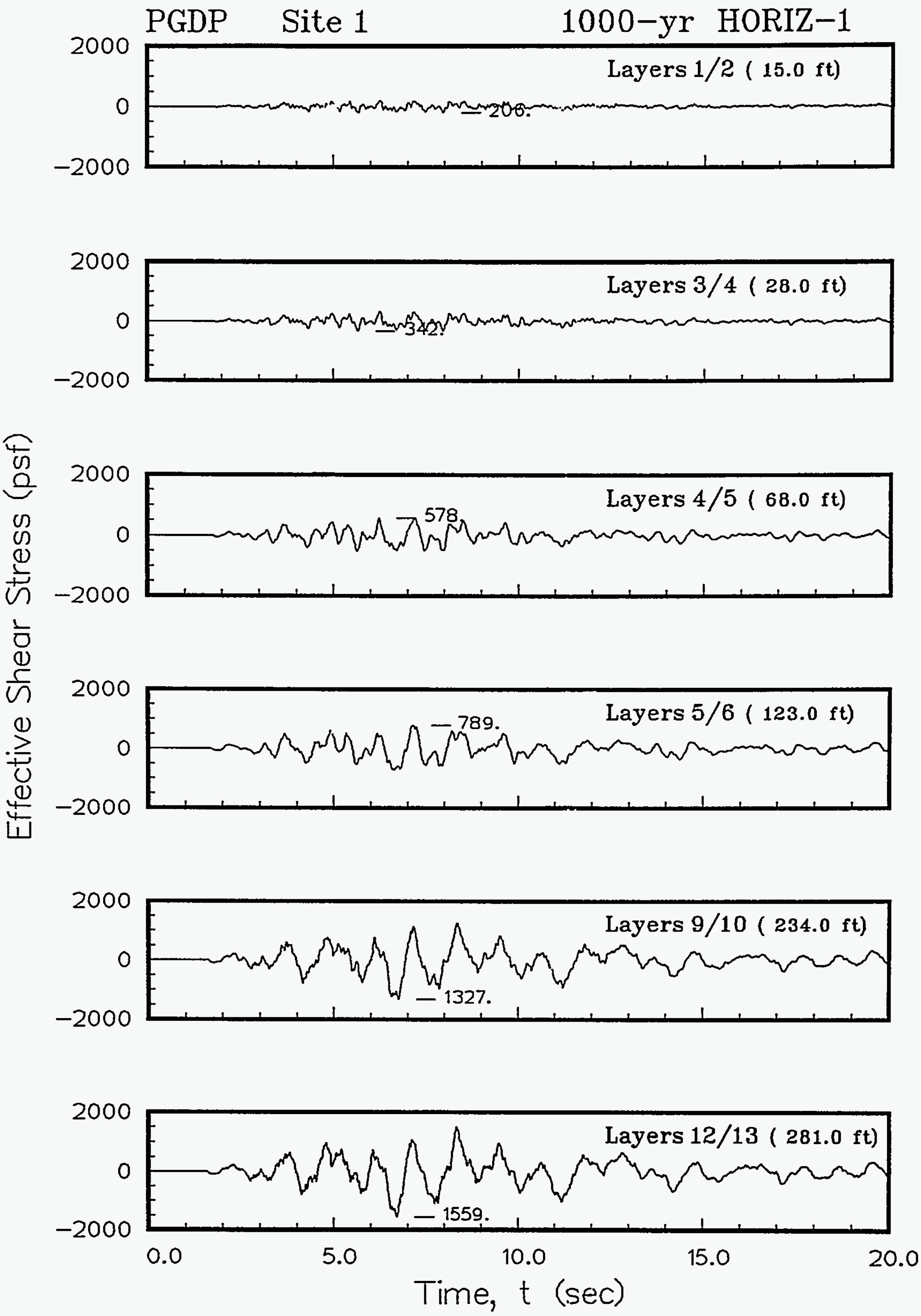

Figure D1. Variation of shear stresses at Site 1 for Horizontal 1 component of DBE

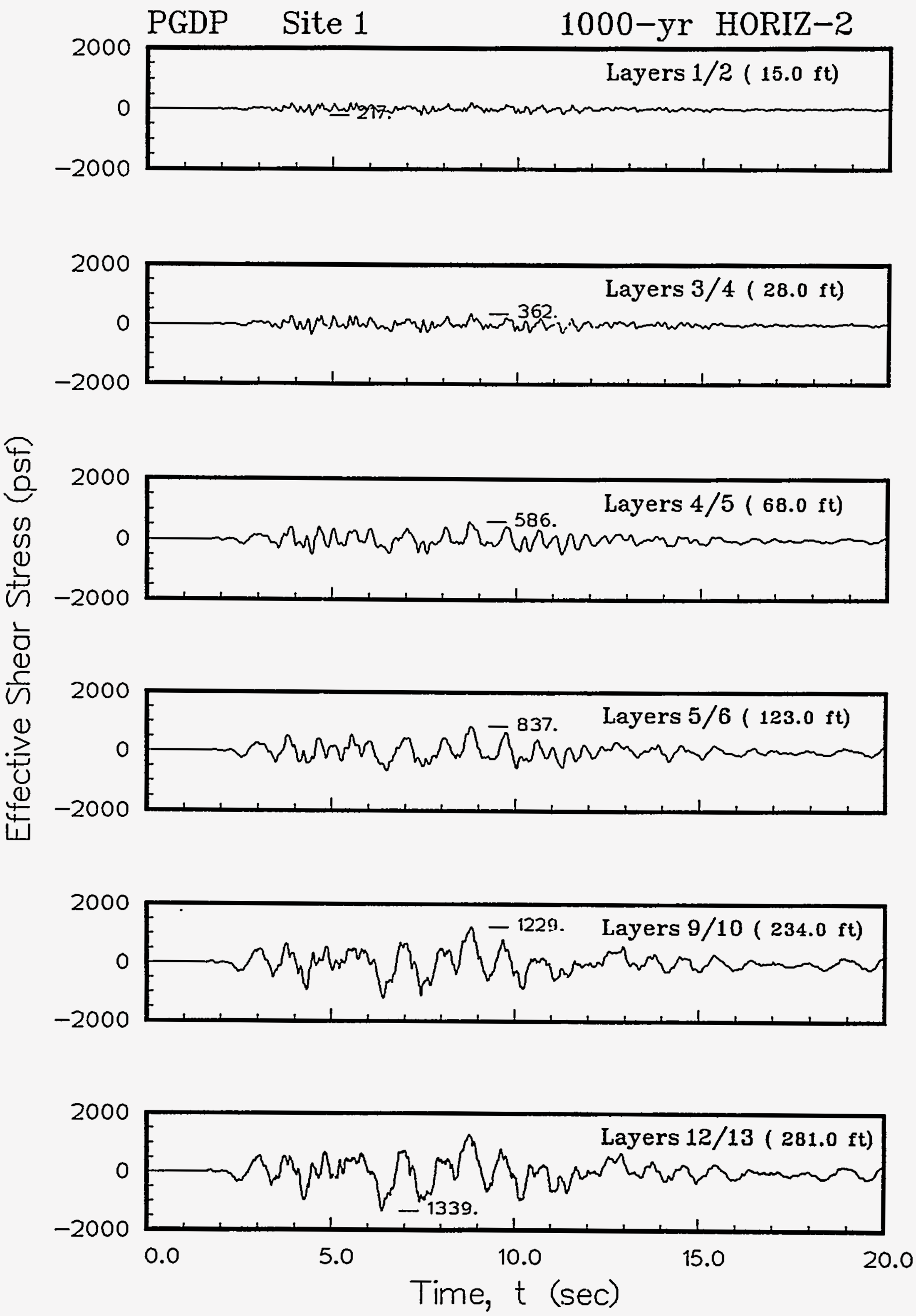

Figure D2. Variation of shear stresses at Site 1 for Horizontal 2 component of DBE

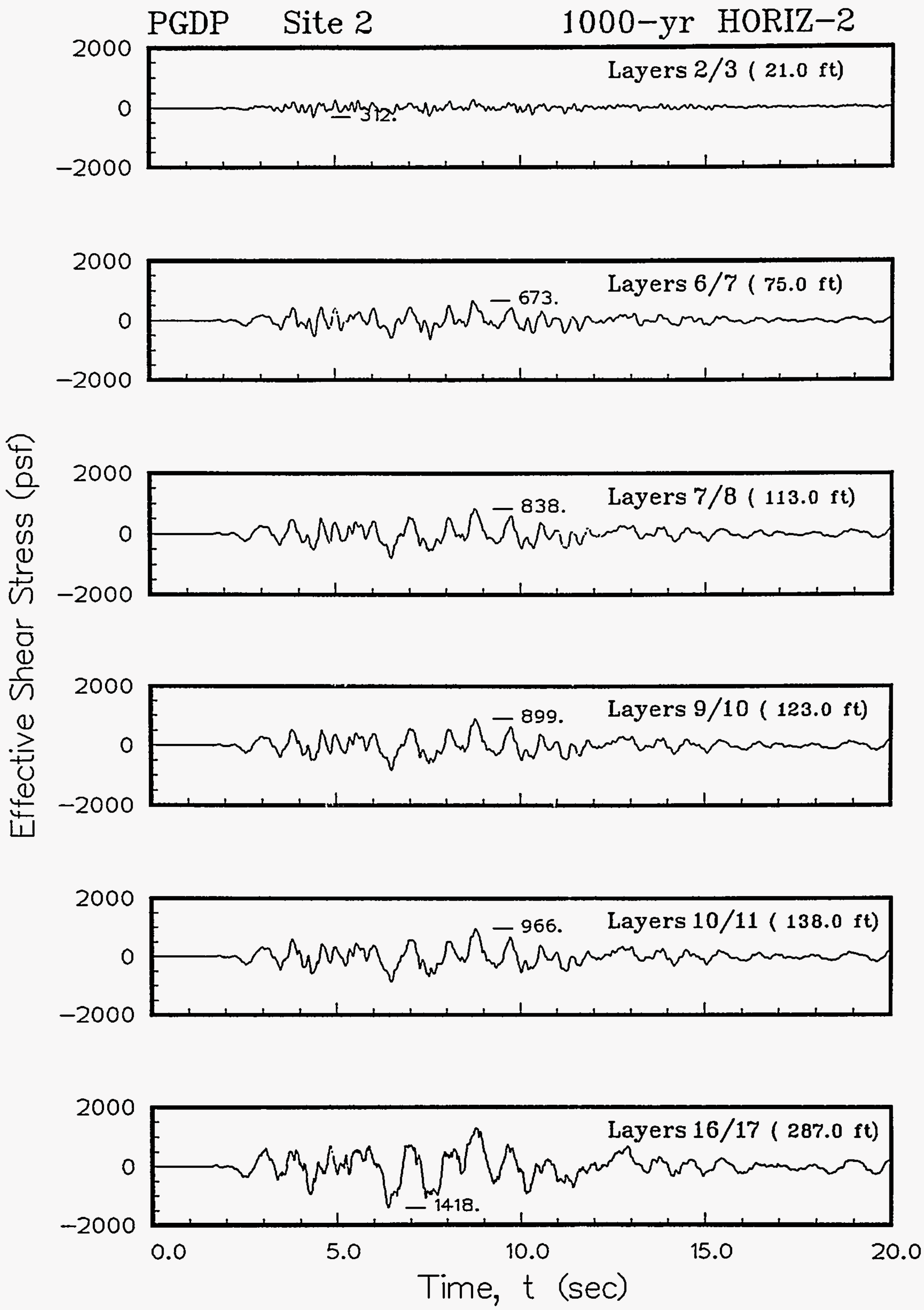

Figure D3. Variation of shear stresses at Site 2 for Horizontal 1 component of DBE

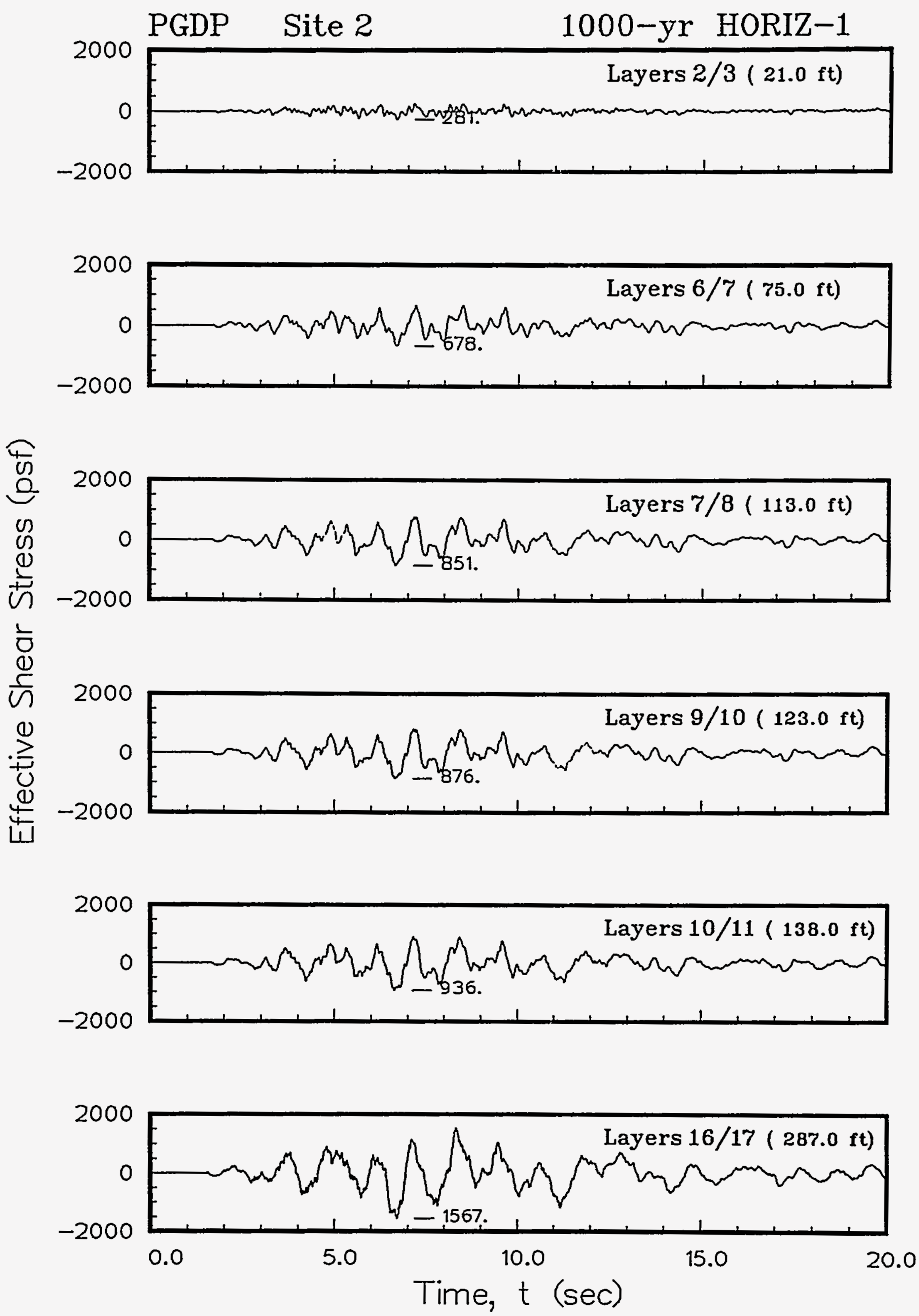

Figure D4. Variation of shear stresses at Site 2 for Horizontal 2 component of DBE

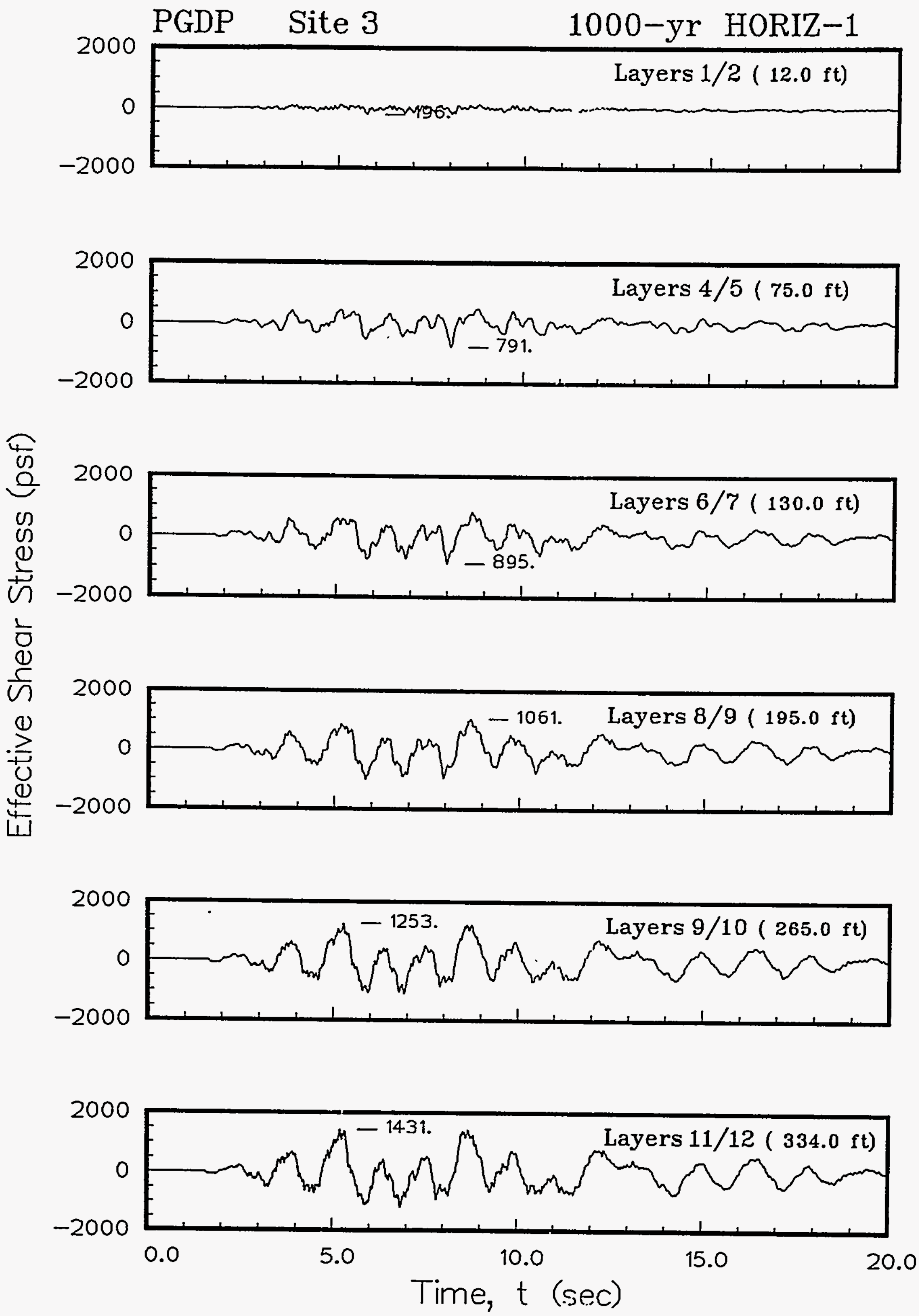

Figure D5. Variation of shear stresses at Site 3 for Horizontal 1 component of DBE

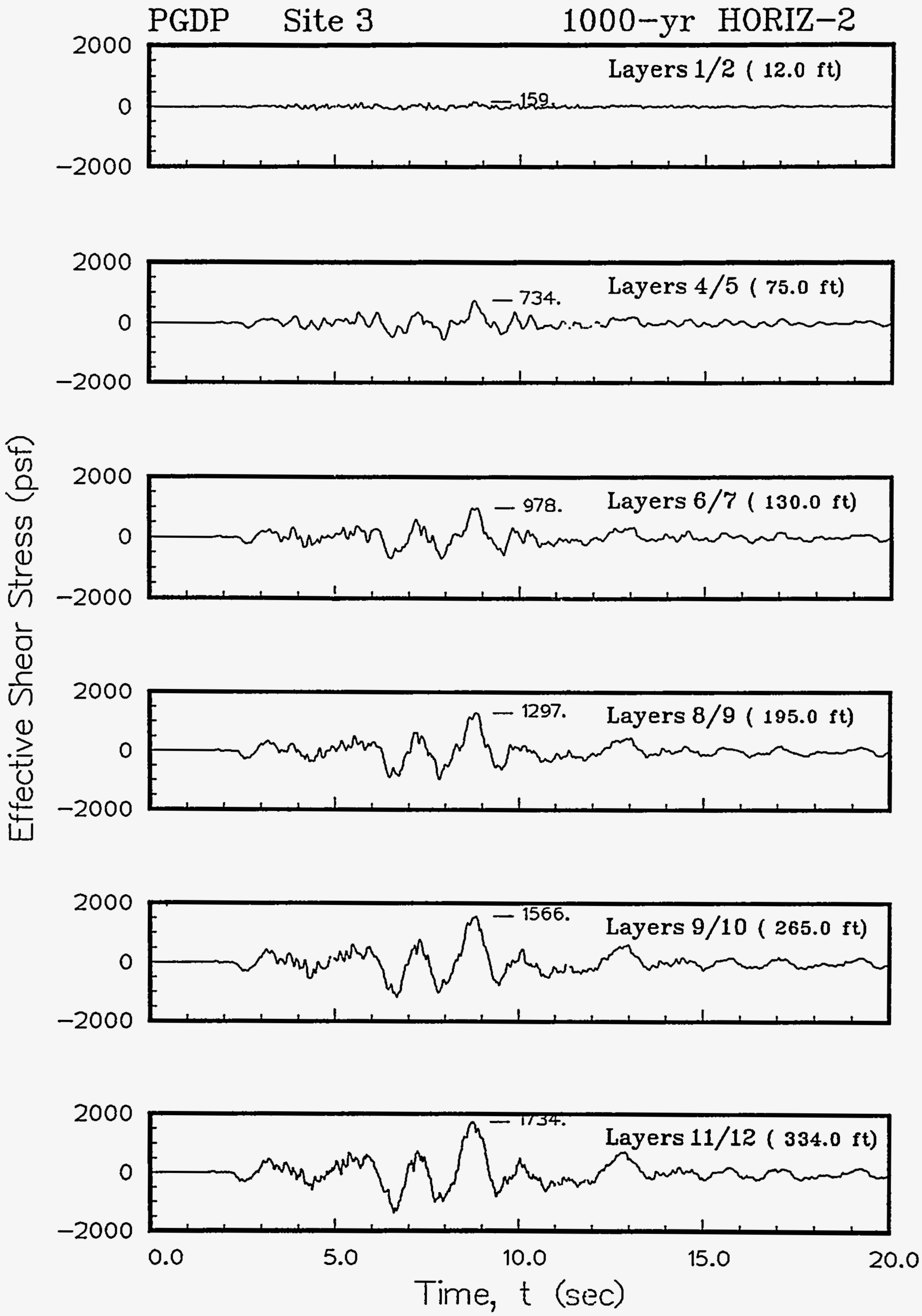

Figure D6. Variation of shear stresses at Site 3 for Horizontal 2 component of DBE

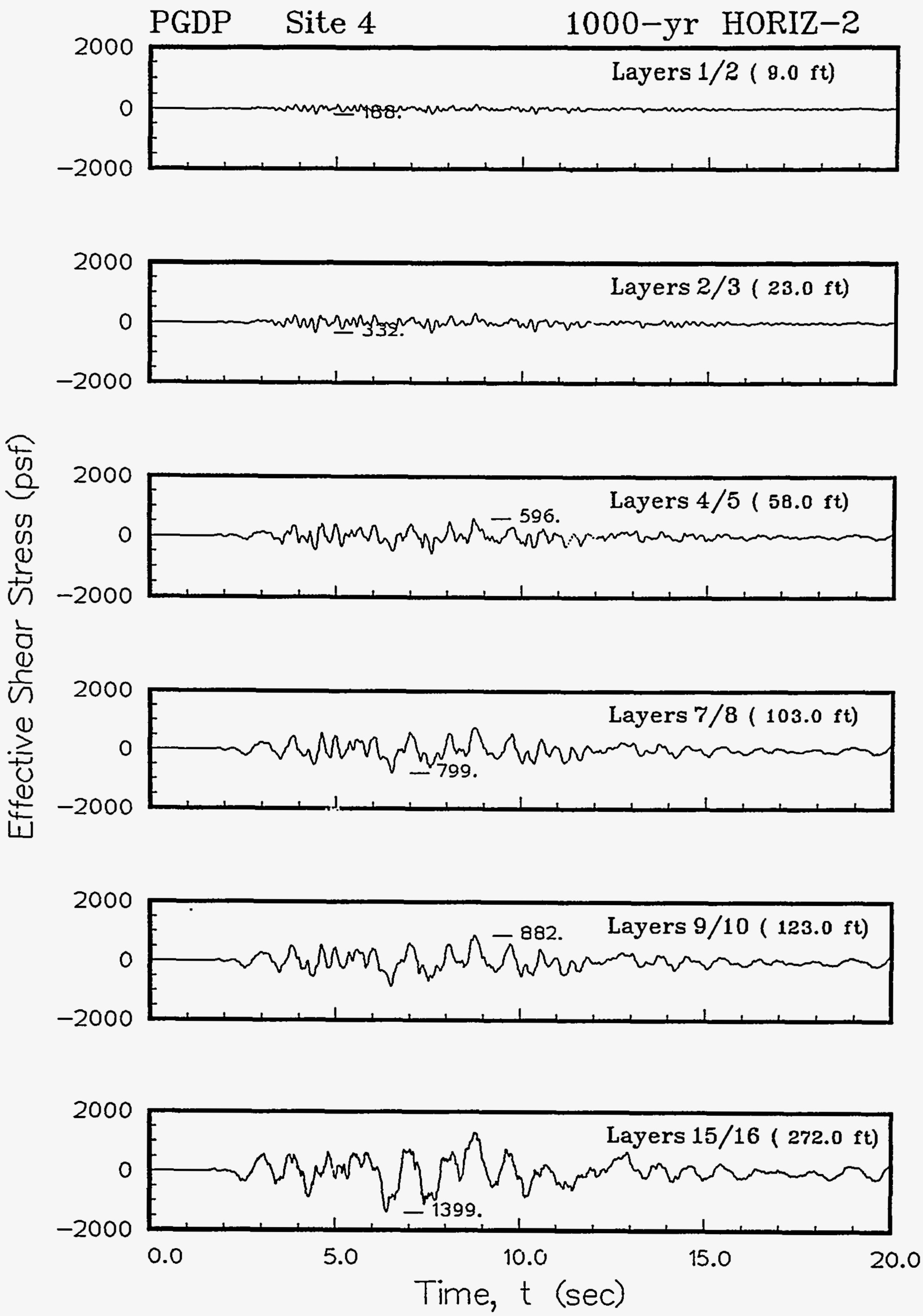

Figure D7. Variation of shear stresses at Site 4 for Horizontal 1 component of DBE

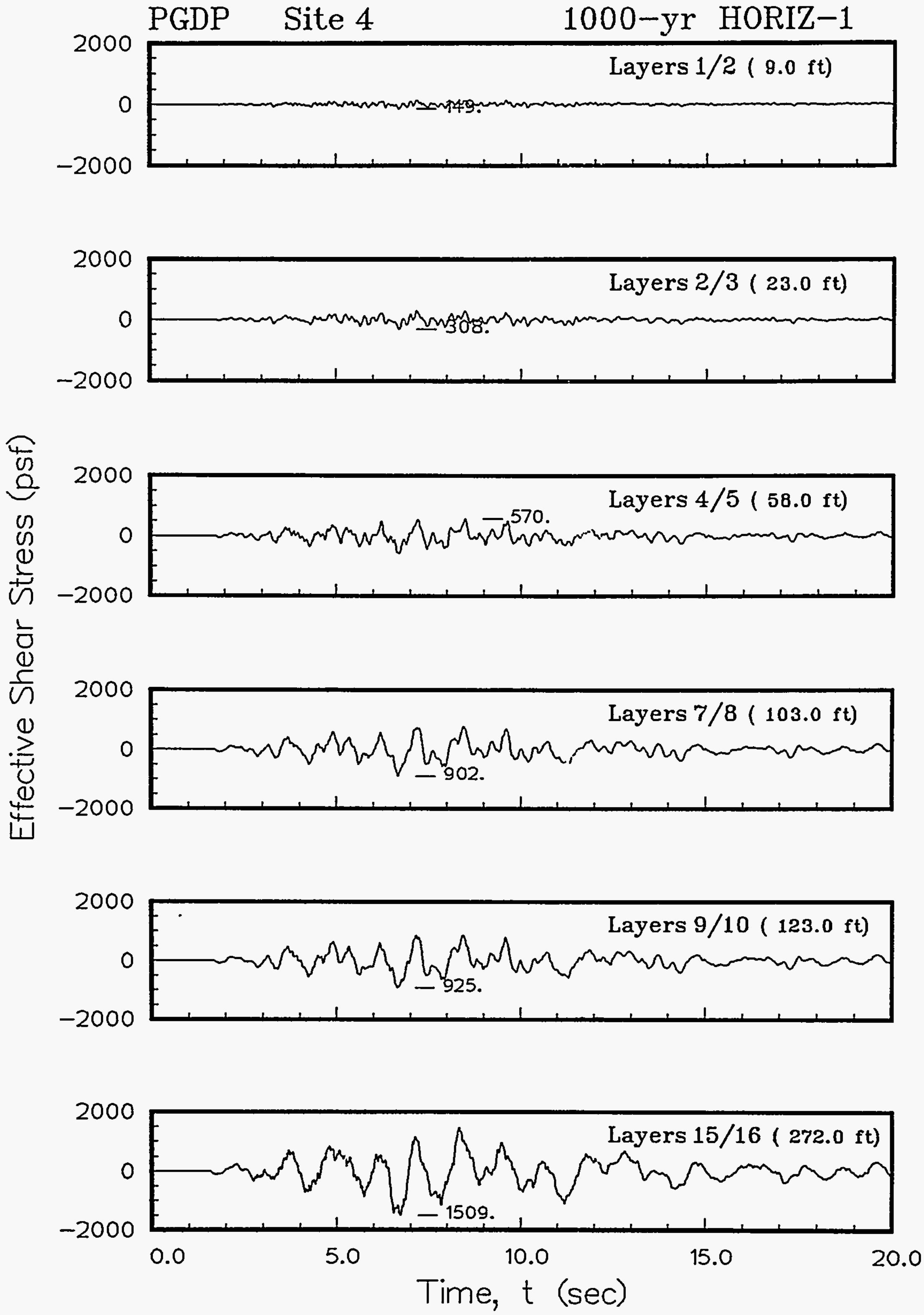

Figure D8. Variation of shear stresses at Site 4 for Horizontal 2 component of DBE

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David W. Sykora, Donald E. Yule

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U.S. Army Engineer Waterways Experiment Station
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A reassessment of liquefaction potential and an estimate of earthquake-induced, free-field settlements have been conducted for four sites at the DOE Paducah Gaseous Diffusion Plant located southwest of Paducah, KY. This work follows an initial effort completed in 1990 but uses induced shear stresses calculated in an earthquake site response study in 1993. This allowed a more accurate assessment of liquefaction potential and earthquake induced settlements. The soils and rock at the site consist of a thin veneer of loess over Pleistocene-age alluvium overlying Tertiary-age deposits and hard limestone. The reassessment indicates that liquefaction of the granular soils is not likely at the four sites studied. Earthquake-induced settlements at the ground surface are expected to be negligible.

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