TABLE OF CONTENTS: APPENDICES

Appendix A

DESRA-MUSC COMPUTER PROGRAM DESCRIPTION AND ILLUSTRATIVE APPLICATION

NUMERICAL EVALUATION OF LIQUEFACTION INDUCED GROUND OSCILLATION AND LURCH

by

Geoffrey R. Martin(1) and Ping Qiu(2)

(1) University of Southern California, Los Angeles, California
(2) Woodward Clyde Consultants, Santa Ana, California

ABSTRACT

The occurrence of liquefaction in soil layers at depth during earthquake ground shaking, can lead to large ground oscillations and permanent ground displacement (ground lurch) of overlying surficial soil layers. Past earthquakes have demonstrated that such ground displacements and associated shear deformations in the liquefied soil layer, can lead to significant damage to piles and buried pipelines. In the case of buried pipelines, relative displacements or ground oscillations in surface layers generated by non-uniform site conditions can result in ground cracking and compression leading to pipeline damage.

In this paper, a modified version of the one dimensional non-linear effective stress site response program DESRA (DESRA-MUSC) is used to illustrate the sensitivity of ground oscillation and lurch amplitudes to site stratigraphy and input ground motion characteristics. The features of the new program DESRA-MUSC are also described and are shown to be compatible with the practical application of numerical evaluations of ground oscillation and lurch potential for design purposes.

INTRODUCTION

As described by Youd (National Research Council, 1985), earthquake induced liquefaction of soil strata at some depth in level ground sites, "commonly decouples overlying soil blocks, allowing them to jostle back and forth on the liquefied layer during an earthquake." This is illustrated schematically in Figure 1. Pease and O'Rourke (1997) note a number of case studies providing field evidence of ground oscillation damage, and investigated this mechanism as the cause of pipeline damage in the Marina District during the 1989 Loma Prieta earthquake. The role of liquefaction induced ground oscillations in causing pipeline breakage in the Northridge earthquake is described by Holzer et al. (1999). In the case of pipelines, Pease and O'Rourke (1997) attribute damage to ground strains arising from differential ground oscillations associated with varying depths of liquefied soil, as shown in Figure 2. In the case of pile foundations, large ground oscillations or permanent ground lurch generated by a large velocity pulse, may place excessive displacement ductility demand on the piles, resulting in damage. For bridge viaduct structures, differential transverse or longitudinal displacements between adjacent bents, may lead to deck collapse.

The magnitude of ground oscillations or permanent ground lurch will depend on a number of variables including the amplitude and frequency characteristics of the firm ground post liquefaction ground motions underlying the liquefied soil layer, and the stress-strain characteristics and thickness of the liquefied ground layer. For large magnitude near field earthquakes in particular, where post liquefaction damage potential to pipelines and structures arising from ground deformations could be significant, an analytical approach to evaluating site specific ground oscillation lurch magnitudes is desirable. Such an analysis requires a non-linear effective stress approach to evaluate both earthquake excess pore pressure response and post liquefaction ground deformations. In this paper, a modified version of the non-linear effective stress site response program DESRA (DESRA-MUSC) is used to illustrate the sensitivity of ground oscillation and lurch amplitudes to site stratigraphy and input ground motion characteristics.

THE DESRA-MUSC COMPUTER PROGRAM

Background to the DESRA program

They computer programs DESRA and DESRA -2 are described and published by Finn et al. (1978) and Lee and Finn (1978) respectively. The constitutive model for excess pore pressure generation during earthquake induced ground response is based on the model described by Martin el. al. (1975), where shear strain amplitude dependent volume changes during drained loading cycles are coupled to pore pressure increases under undrained loading cycles through elastic rebound parameters. Non-linear ground response is characterized by a hyperbolic shear stress-shear strain curve initially defined by drained maximum shear modulus G_m and shear strength τ_m parameters. These parameters are subsequently degraded (Figure 3) as a function of pore pressures increases to selected low limiting values when excess pore pressures approach the initial vertical effective stress. Hysteresis loops are defined during load and unload cycles using the Masing criterion. The degradation rules used in DESRA for G_m and τ_m are:

$$G_{mt} = G_{mo}(1\text{-}u/\sigma'_{vo})^{1/2} \qquad (1)$$

and

$$\tau_{mt} = \tau_{mo}\,(1\text{-}\,u/\sigma'_{vo}) \qquad (2)$$

where G_{mt} and τ_{mo} are initial low strain modulus and shear strength respectively and G_{mt} and τ_{mt} are degraded values at time *t*.

A value of the excess pore pressure u approximately 90-95% of the initial vertical effective stress σ'_{vo} is often used to set values of the minimum limiting residual values approaching liquefaction.

Based on experimental data such as that shown in Figure 4, Matasovic and Vucetic (1993) noted equation (2) overestimates the stress degradation that occurs as pore pressures increase, and modified the degradation rule as:

$$\tau_m = \tau_m\,[(1\text{-}\,u/\sigma'_{vo})^{v}] \qquad (3)$$

Values of v depend on soil type and were typically about 3.5. Comparisons between equations (2) and (3) are shown in Figure 5, together with a square root degradation rule used in analyses described in this paper.

Note that the degradation model does not take into account the post liquefaction undrained dilative behavior that can occur at large cyclic shear strains in the case of dense sands, such as illustrated in Figure 6, and the corresponding cyclic reductions in pore pressure. Such behavior has also been observed in the field (Zeghal and Elgemal, 1994) and in centrifuge tests. However, the post liquefaction low residual shear resistance (prior to dilation) will in most cases control ground oscillation amplitudes, particularly for looser sands and higher frequency input motions where cyclic shearing strains are less than those required to cause significant dilation. Note that values of residual resistance τ_{mr} of about 0.1 σ'_{vo} seem representative of experimental data irrespective relative density.

As with all non-linear dynamic response programs, the key to practical application is the simplicity of the constitutive model and the ease of parameter selection to match insitu soil conditions. In this respect, two studies are noteworthy in promoting the practical application. Martin et. al. (1981) describe parametric studies using DESRA-2 to establish an approach to backfit the 4 volume change and 3 rebound constants in the model to match a given field liquefaction strength curves, established from say SPT data. Byrne (1991) describes a procedure to reduce volume change constants from 4 to 2, and relates parameters to relative density to match liquefaction strength curves, assuming reasonable and constant values of the rebound parameters. Values of G_m and τ_m may also be related to SPT blowcount, and Byrne (1991) notes good agreement with the simplified constitutive model and laboratory test data.

The DESRA-MUSC Program

Over recent years the DESRA-2 program has been modified by research programs conducted at USC to form the program DESRA-MUSC. The modifications are described in detail by Qiu (1998), and are summarized below.

In relation to the previous discussion, the DESRA-MUSC program incorporates the option of using the 2 parameter Byrne (1991) volume change model used to match liquefaction strength curves, and the Matasovic and Vucetic (1993) residual resistance degradation model. Another key modification is the replacement of the hyperbolic backbone curve by the more versatile Iwan (1967) kinematic strain hardening model, which in a one-dimensional form, can be simulated numerically by a mechanistic array of elastic (k_i) and coulomb sliding resistance (R_i) elements as shown in Figure 7. Using this model, an initial loading or backbone curve may be built up as a piecewise linear approximation as shown in Figure 8. Usually 10-20 elements are sufficient to produce reasonable results. The model automatically simulates the Masing criteria resulting in hysteresis loop generation during cyclic loading. An additional bi-linear element has been added to simulate observed low strain damping.

From a practical standpoint, an initial backbone curve can be constructed using established relationships between G/G_m versus shear strain γ as shown in Figure 9, and knowledge of site shear wave velocities or G_{max} versus SPT blowout relationships. Unfortunately, G/G_m versus γ data is generally restructed to values of γ less than about 0.5%. Consequently, judgement is required to construct the balance of the backbone curve at larger strains to meet the assumed τ_m value, defined as residual lower strain resistance (approximately the drained strength for sands) as opposed to the large strain undrained strength for dilatant soils as shown in Figure 10. Clearly more research and test data is needed in the larger strain range, to better define the non-linear backbone curve shapes for response analyses.

As for the original DESRA-2 program, analyses may be performed in a totally undrained mode, or alternatively, allow dissipation and/or re-distribution of excess pore pressures incrementally in analysis time steps, by solving the one dimensional consolidation equation. The elastic rebound parameters define the coefficient of compressibility and permeability is used as an additional input parameter.

With the above program modifications, Qiu (1998) describes a number of earthquake site response analyses using the schematic approach shown in Figure 11. The DESRA-MUSC program includes a transmitting base boundary as illustrated. The following examples illustrate applications of the program to study liquefaction induced ground oscillation.

SINGLE LAYER SENSITIVITY STUDY

To illustrate the influence of input motion frequency and values of residual resistance on ground oscillation amplitudes, an idealized simple model was considered. The model comprises a 10 foot thick loose saturated sand layer (SPT blowcount N_{160} = 9) overlying soft rock and subjected to sinusoidal 0.8g amplitude ground (rock outcrop) input accelerations. Soil parameters used in the DESRA-MUSC analyses are listed in Table 1 below. And are consistent with a liquefaction strength curve for a sand with N_{160} = 9. Analyses were performed in an undrained mode simulating an impervious cap on top of the sand layer, with an assumed constant post residual resistance.

Table 1. Basic Material Properties in the Case Studies

Soil Properties	
SAND	
Unit Weight (pcf)	122
Friction Angle	33
Shear Wave Velocity (ft/s)	560
Residual Shear Resistance (psf)	200
BEDROCK	
Unit Weight (pcf)	130
Shear Wave Velocity (ft/s)	1800

Parameters for Pore Pressure Model	
k	0.0025
m	0.38
n	0.6
C_1	0.415
C_2	1.157
C_3	0.434
C_4	0.888

Effect of Input Frequency on Ground Oscillation

Figure 12 shows analysis results for 1, 2 and 5 hz input motions at a depth of 5 ft. Liquefaction is seen to occur after about 2 cycles when steady state accelerations drop to about 0.3g corresponding to an assumed post liquefaction residual resistance of 200 psf. For a l hz input motion, ground oscillation amplitudes are about ± 1 ft. (10% shear strain in 10 ft.) and show about a 1 ft. permanent ground lurch. As input frequencies increase to 2 and 5 hz, ground oscillation and permanent lurch amplitudes are seen to reduce significantly.

Effect of Assumed Residual Resistance

Figure 13 shows analysis results at a depth of 5 ft. for a 1 hz input frequency and for assumed residual resistance values of 80, 200 and 400 psf. Post liquefaction acceleration levels are reduced with reductions in residual resistance is would be expected. However, amplitudes of oscillation increase only slightly with decreased residual resistance although significant increases in permanent ground lurch occur.

CASE STUDY – MULTILAYER SITE PROFILE

The soil profile and physical soil properties used in the case study analyses are shown in Figure 14, and are representative of an actual site in Southern California. Field liquefaction strength curves for the three sand layers are shown in Figure 15 together with the matched analytical curves using the DESRA-MUSC parameters. The elastic rebound curves for the sands are also shown in Figure 15 together with the normalized shear strength-shear strain non-linear backbone curves. The latter curves were based on the ERPI (1993) G/G_{max} versus shear strain amplitude curves up to strains of about 1% and then extrapolated to estimated failure strains. Post liquefaction residual resistance values for the three sand layers were arbitrarily assumed equal to 0.15 times the average initial vertical effective stress in each layer namely: layer 3, 180 psf, layer 6, 360 psf, and layer 8 , 450 psf.

Total stress behaviour with no pore pressure increases was assumed for the clayey silts. For the clayey silts, the static undrained shear strengths were assumed for analyses with backbone shear stress-shear strain curves developed using the G/G_{max} versus shear strain amplitude curves given by Vucetic and Dobry (1991) for a PI = 50, with extrapolation to failure strains beyond 1%.

Two earthquake records were used as rock outcrop motions, namely the 1992 Landers earthquake Amboy record (M_w = 6.7 @ 74 km) scaled to 0.2g and the 1992 Cape Mendocino earthquake Petrolia record (M_w = 6.5 @ 9 km), scaled to 0.7g.

For the Amboy record, liquefaction occurred in the very loose sand layer 3 after about 29 seconds of shaking leading to ground oscillations of about 0.25 ft amplitudes as shown in Figure 16 and 17. No permanent ground lurch and only small pore pressure increases occurred in the denser sands of layers 6 and 8. The maximum shear strengths were not reached in the clayey silt layers. Note ground surface accelerations (Figure 16) were similar in amplitude and frequency to input accelerations, until liquefaction occurred in layer 3, at which time longer period acceleration pulses commenced.

For the Petrolia record, the shear strength of the layer 4 soft clayey silt was exceeded prior to liquefaction of the layer 3 sand. The soft clayey silt was hence the seat of the maximum ground oscillation amplitudes of about 0.5 ft as shown in Figures 18 and 19, with an associated permanent ground lurch also of about 0.5 ft. It is important to note (and often overlooked) that the strength of soft cohesive soils is often similar to the post liquefaction residual resistance of sands. Figure 20 show pore pressures increasing significantly in the denser sands of layers 6 and 8, but not contributing to ground displacements.

CONCLUSIONS

The occurrence of large ground oscillations and potential associated ground lurch resulting from underlying earthquake induced liquefaction at level ground sites, is clearly of design concern to both buried pipelines and bridges. Past earthquakes have graphically illustrated the damage potential of such ground displacements.

As the magnitude of ground oscillations or ground lurch depends strongly on the amplitude and frequency characteristics of earthquake ground motions, the depth of liquefied soil, and the relative density and residual resistance of liquefiable soils, a site specific approach to evaluate potential ground displacements is desirable.

In this paper, the practical application of the computer program DESRA-MUSC to evaluate ground oscillation and lurch displacements is described. Example illustrates the importance of input amplitude and frequency characteristics and the nature of site stratigraphy in controlling ground oscillations. The concept of post liquefaction small strain residual shear resistance as a major controlling factor in determining displacements in introduced. The need for research to better define and understand residual shear resistance as opposed to residual strength (of primary concern to flow slide potential) is strongly recommended.

ACKNOWLEDGEMENTS

The research described in this paper was funded jointly by an NSF grant to the National Center for Earthquake Engineering, Buffalo, and a Federal Highway Administration grant (FHWA Contract DTFH61-92-6-00106) to the National Center for Earthquake Engineering. This support is gratefully acknowledged.

REFERENCES

Byrne, Peter M., (1991). "A Cyclic Shear-volume Coupling and Pore Pressure Model for Sand", Proceedings: Second International Conference on Recent Advances in Geotechnical Engineering and Soil Dynamics, March 11-15, 1991, St. Louis, Missouri, Paper No. 1.24, pp. 47-55.

EPRI (1993). Guidelines for Determining Design Basis Ground Motions, Electric Power Research Institute, Palo Alto, California, Vol. 1, pp. 8-1 through 8-69.

Finn, W.D. Liam, Martin, Geoffrey R. and Lee, Michael K. W., (1978). "Comparison of Dynamic Analysis of Saturated Sands", Proceedings of the 1978 ASCE Conference on Earthquake Engineering and Soil Dynamics, Pasadena, California.

Holzer, Thomas L., Bennett, Michael J., Ponti, Daniel J., and Tinsley III, John C. (1999), "Liquefaction and Soil Failure During 1994 Northridge Earthquake," Journal of Geotehcnical and Geoenvironemtnal Engineering, Vol. 125, No. 6, pp. 438-452.

Ishihara, K., (1985). "Stability of Natural Deposit During Earthquake," Theme Lecture, Proceedings, 11th International Conference on soil Mechanics and Foundation Engineering, San Francisco, Vol. 2, pp. 321-376.

Iwan, W.D., (1967). "On a Class of Models for the Yielding Behaviors of Continuous and Composite System", Transactions of the ASME, Sept. 1967.

Matasovic, Neven and Vucetic, Mladen, (1993). "Cyclic Characterization of Liquefiable Sands," Journal of Geotechnical Engineering, ASCE, Vol. 119, No. 11, pp. 1805-1822.

Lee, M.K.W. and Finn, W.D.L. (1978). "DESRA-2, Dynamic Effective Stress Response Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential," Soil Mechanics Series No. 38, University of British Columbia, Vancouver.

Martin, G.R. Finn, W.D.L. and Seed, H.B. (1975). "Fundamentals of Liquefaction Under Cyclic Loading," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT5, pp. 423-438.

Martin, G.R., Lam, I.P., McCaskie, S.L., and Tsai, C.-F. (1981). "A Parametric Study of an Effective Stress Liquefaction Model," Proceedings, International Conference on Geotechnical Earthquake Engineering and Soil Dynamics, St. Louis, Missouri, Vol. 2, pp. 699-705.

Pease, J.W., and O'Rourke, T.D., (1997). "Seismic Response of Liquefaction Sites," Journal of Geotechnical and Geoenvironmental Engineering, Vol. 123, No. 1, pp. 37-45.

Qiu, Ping, Earthquake Induced Non-Linear Ground Deformation Analyses, (1998), Ph.D. Dissertation submitted to the Graduate School, University of Southern California.

Vucetic, M. and Dobry, R. (1991). "Effect of Soil Plasticity on Cyclic Response," Journal of Geotechnical Engineering, ASCE, Vol. 117, No. 1, pp. 89-107.

Youd, T.L. (1984). "Geologic Effects – Liquefaction and Associated Ground Failure," Proceedings, Geologic and Hydrologic Hazards Training Program, Open File Report 84-760, U.S. Geological Survey, Menlo Park, California, pp. 210-232.

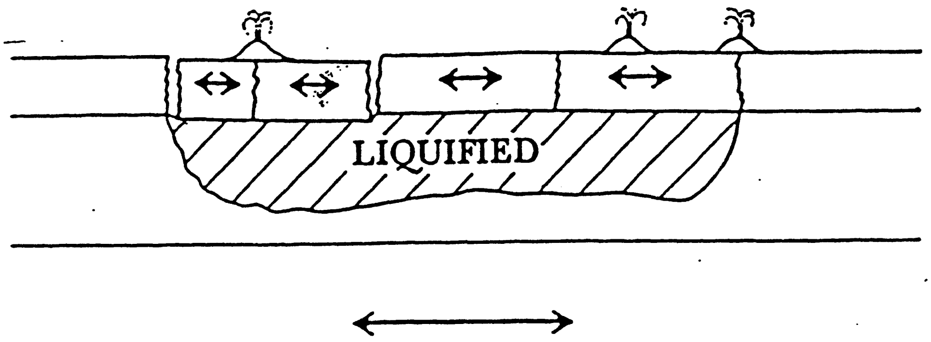


Figure 1 Mechanism of Ground Oscillation (after Youd, 1984)

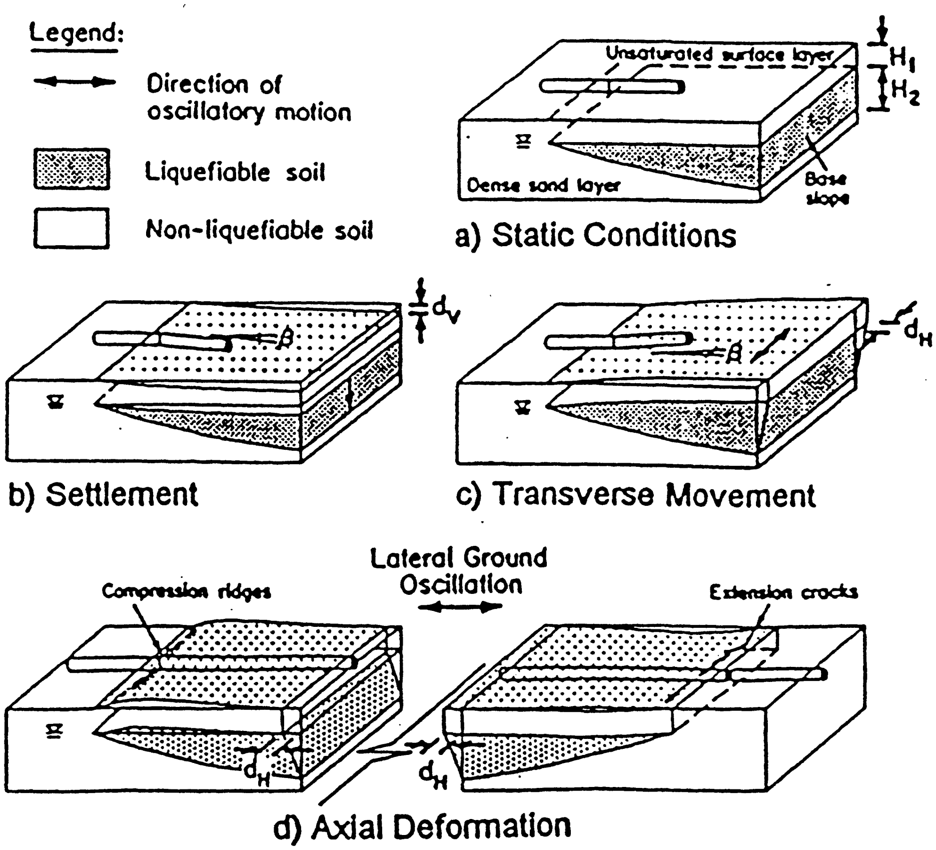


Figure 2 Schematic of Buried Pipeline Response to Transient Displacements at Liquefaction Sites (after Pease and O'Rourke, 1997)

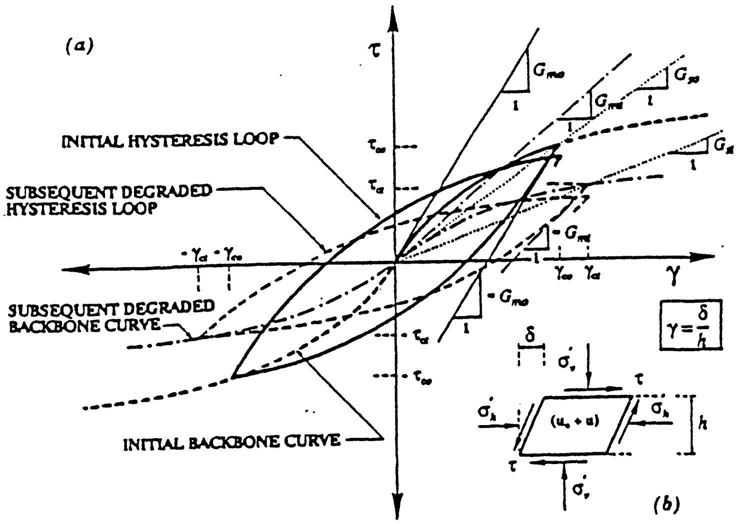


Figure 3 Degradation of Backbone Curves (after Matasovic and Vucetic; 1993)

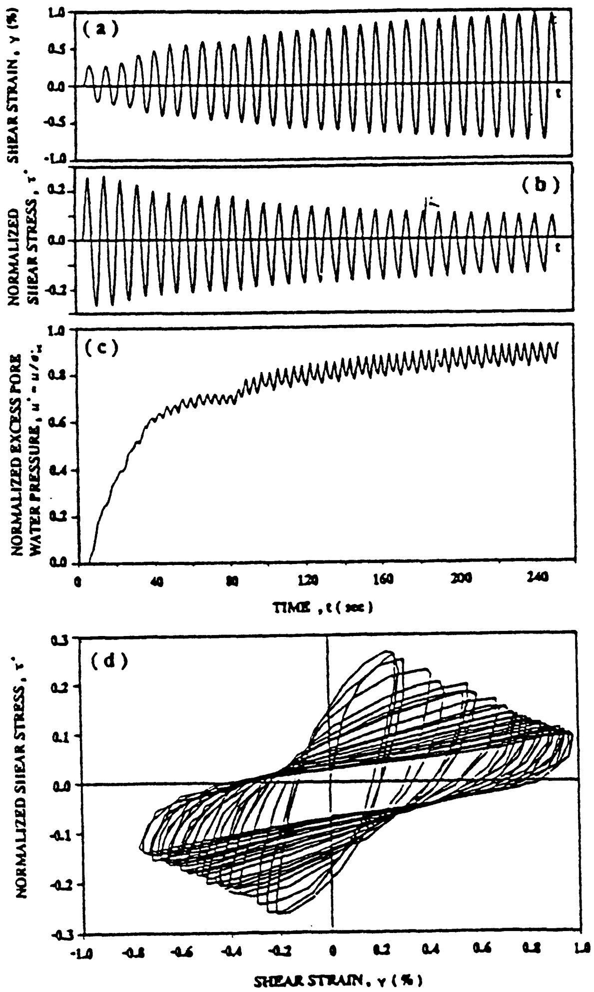
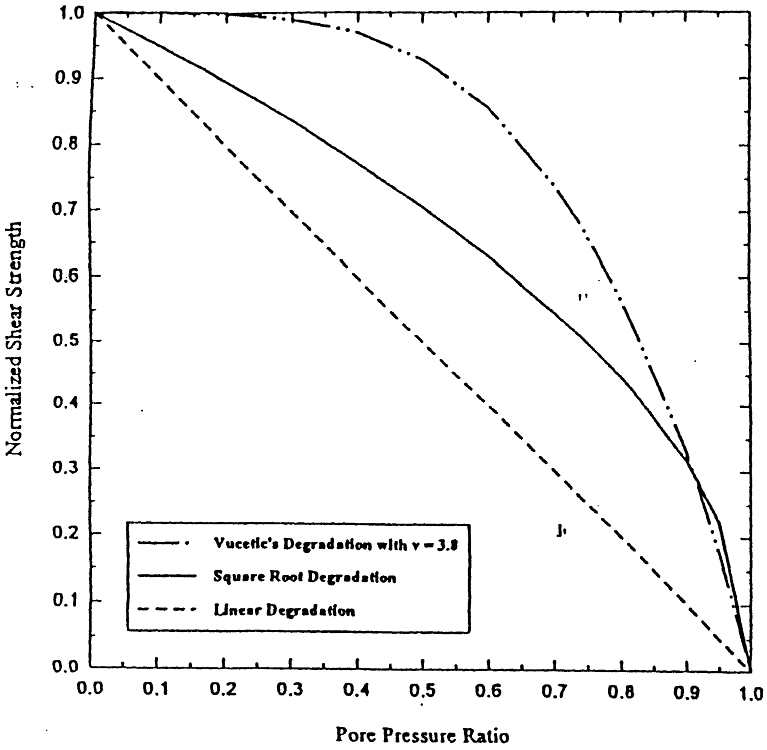


Figure 4 Results of Representative Cyclic Simple Shear Tests (after Matasovic and Vucetic, 1993)



Linear Degradation Law:

$$\tau_{ml} = \tau_{m0}\left(1-\frac{u}{\sigma'_{v0}}\right)$$

Square Root Degradation Law:

$$\tau_{ml}' = \tau_{m0}\sqrt{1-\frac{u}{\sigma'_{v0}}}$$

The Vucetic's Degradation Law:

$$\tau_{ml} = \tau_{m0}\left(1-\left[\frac{u}{\sigma'_{v0}}\right]^{v}\right)$$

Figure 5 Comparison of Stress Degration Laws (after Qiu, 1998)

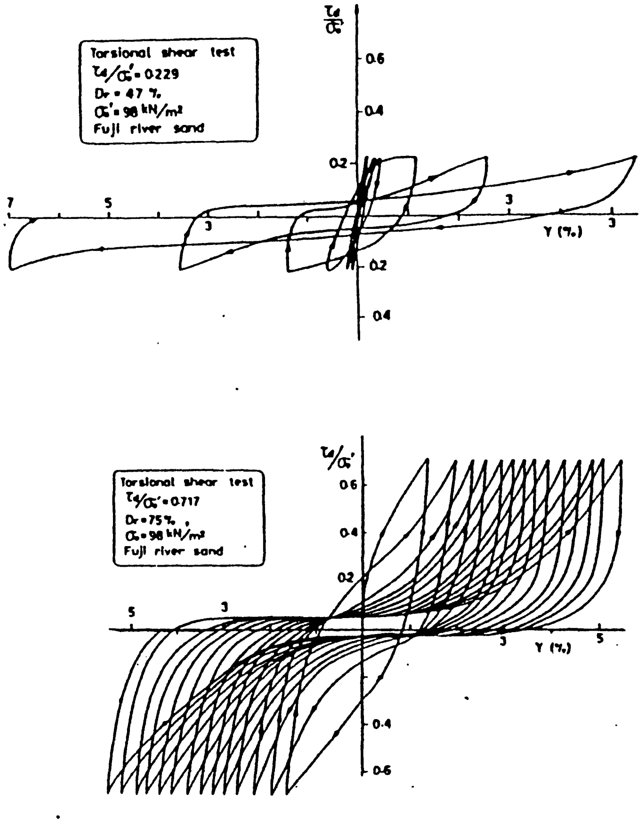


Figure 6 Cyclic Torsional Tests on Saturated Sands (after Ishihara, 1985)

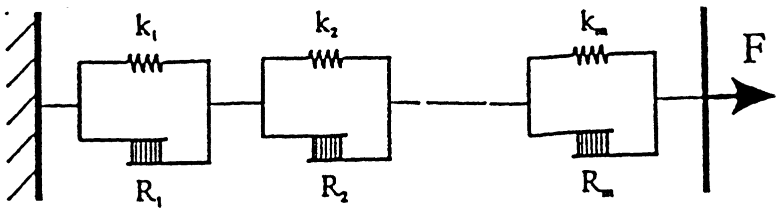


Figure 7 Mechanistic Iwan Model

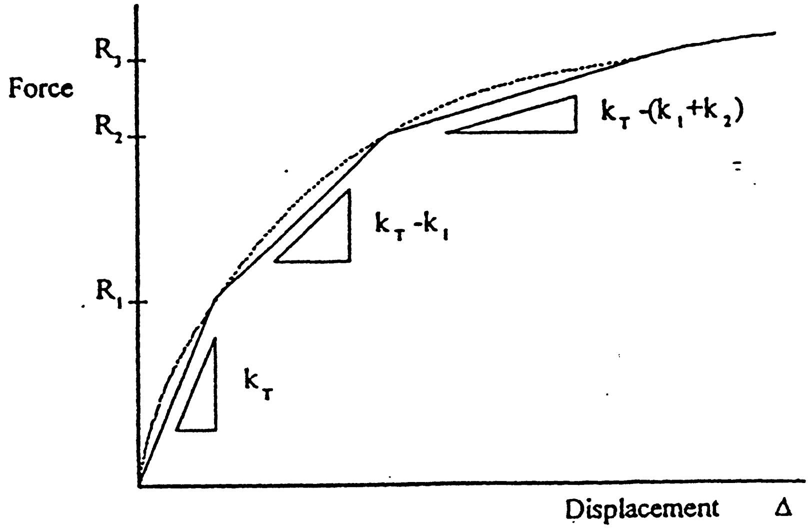


Figure 8 Piecewise Linear Approximation to Backbone Curve

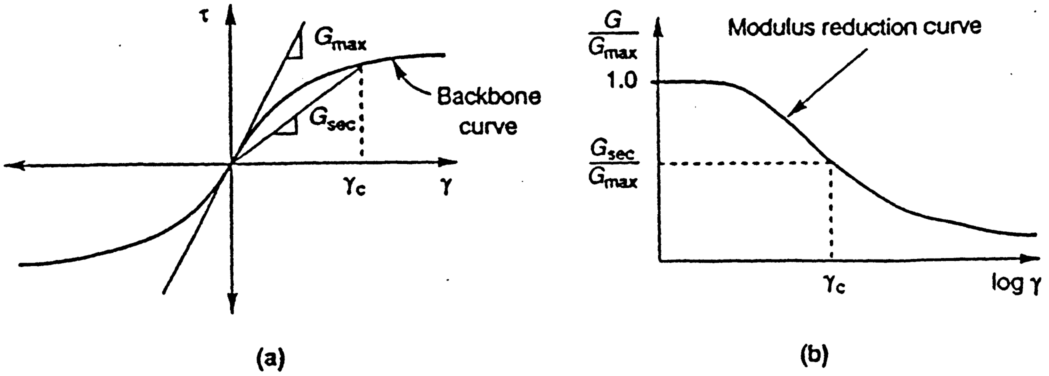


Figure 9 Backbone Curve Showing Variation of G_{sec} with Shear Strain

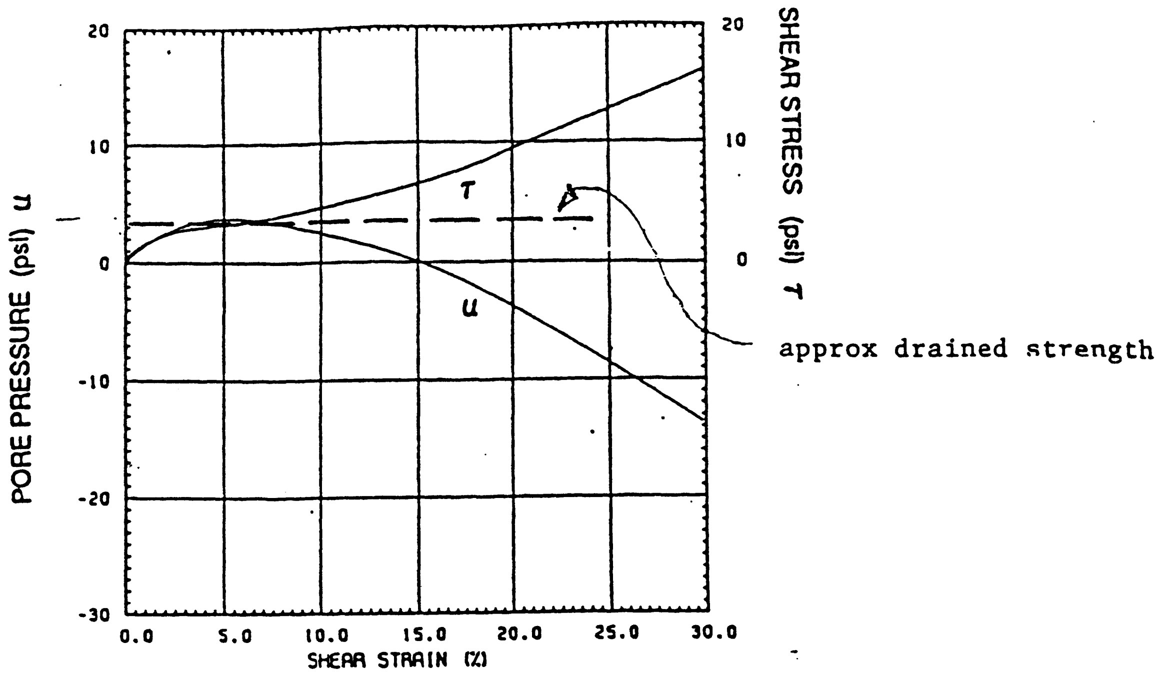


Figure 10 RapidMonotonic Undrained Simple Shear Test on a Medium Dense Silty Sand

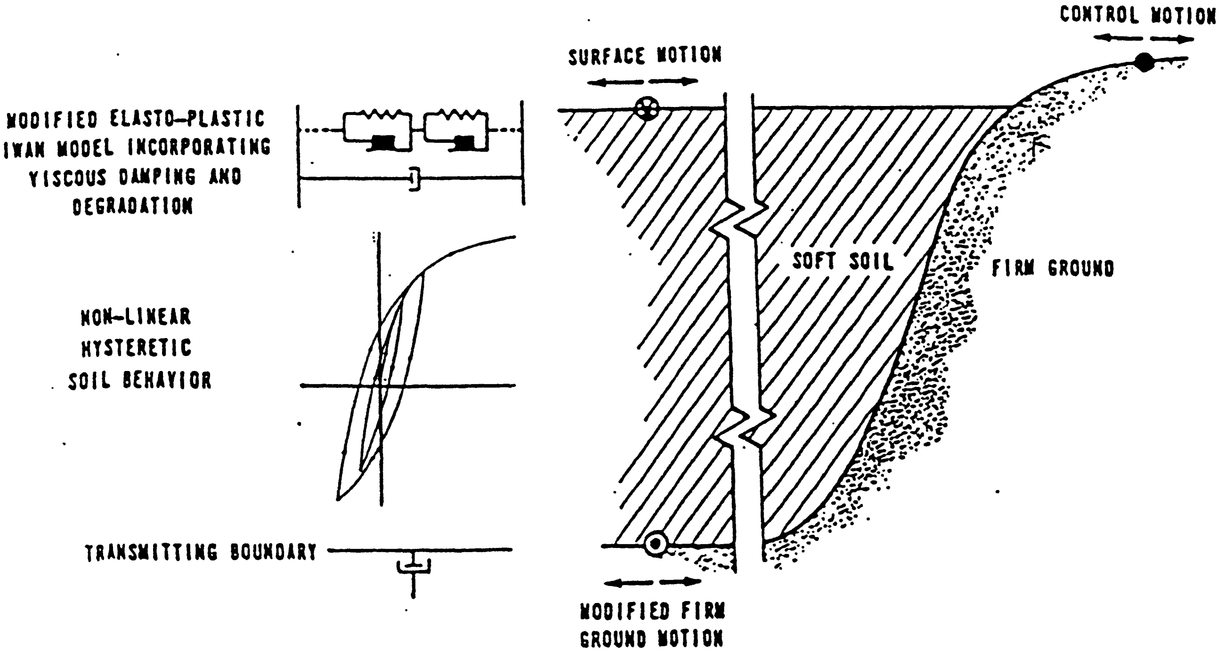


Figure 11 General Features of the DESRA-MUSC Program

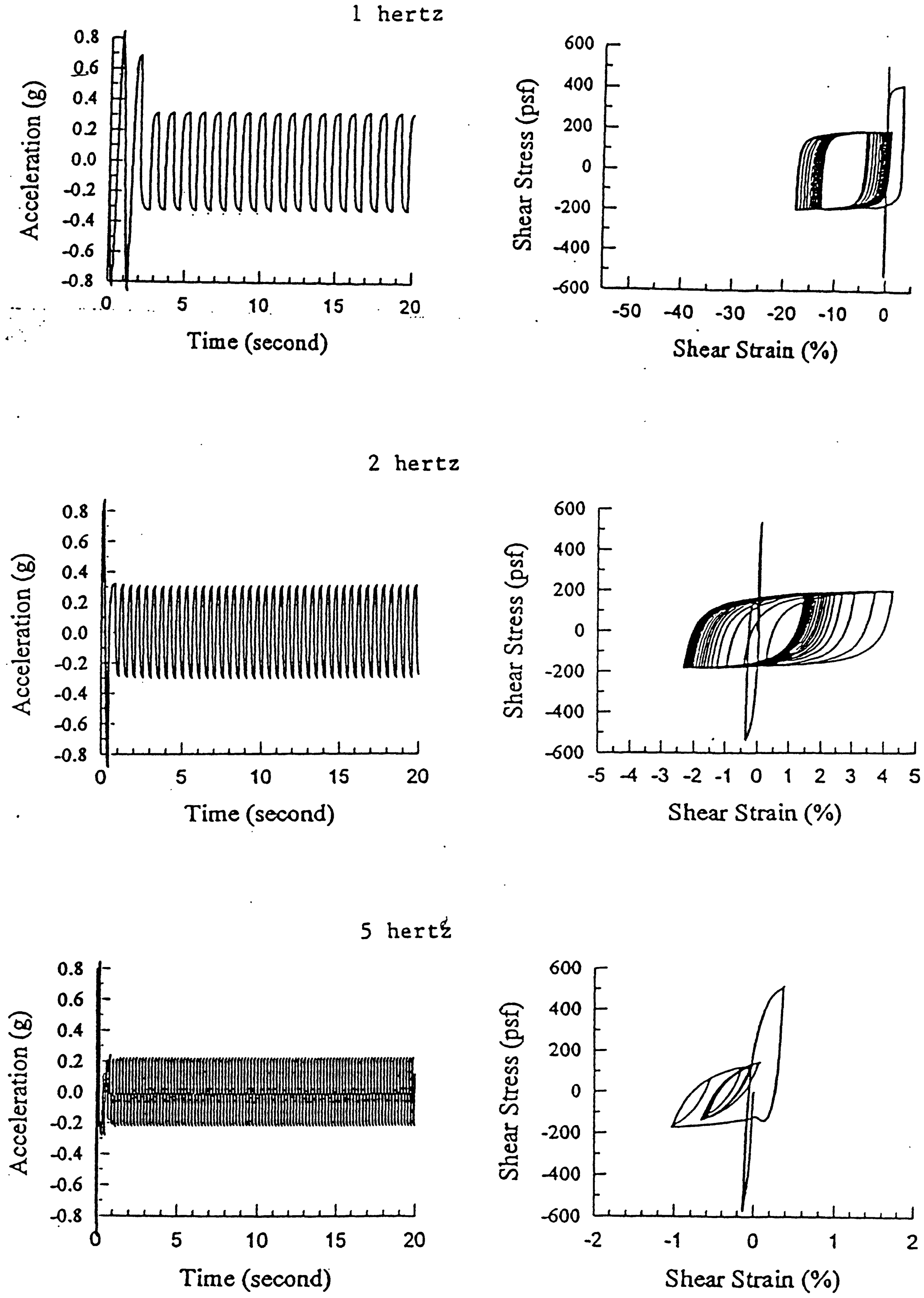


Figure 12 Effect of Input Frequency on Ground Oscillation

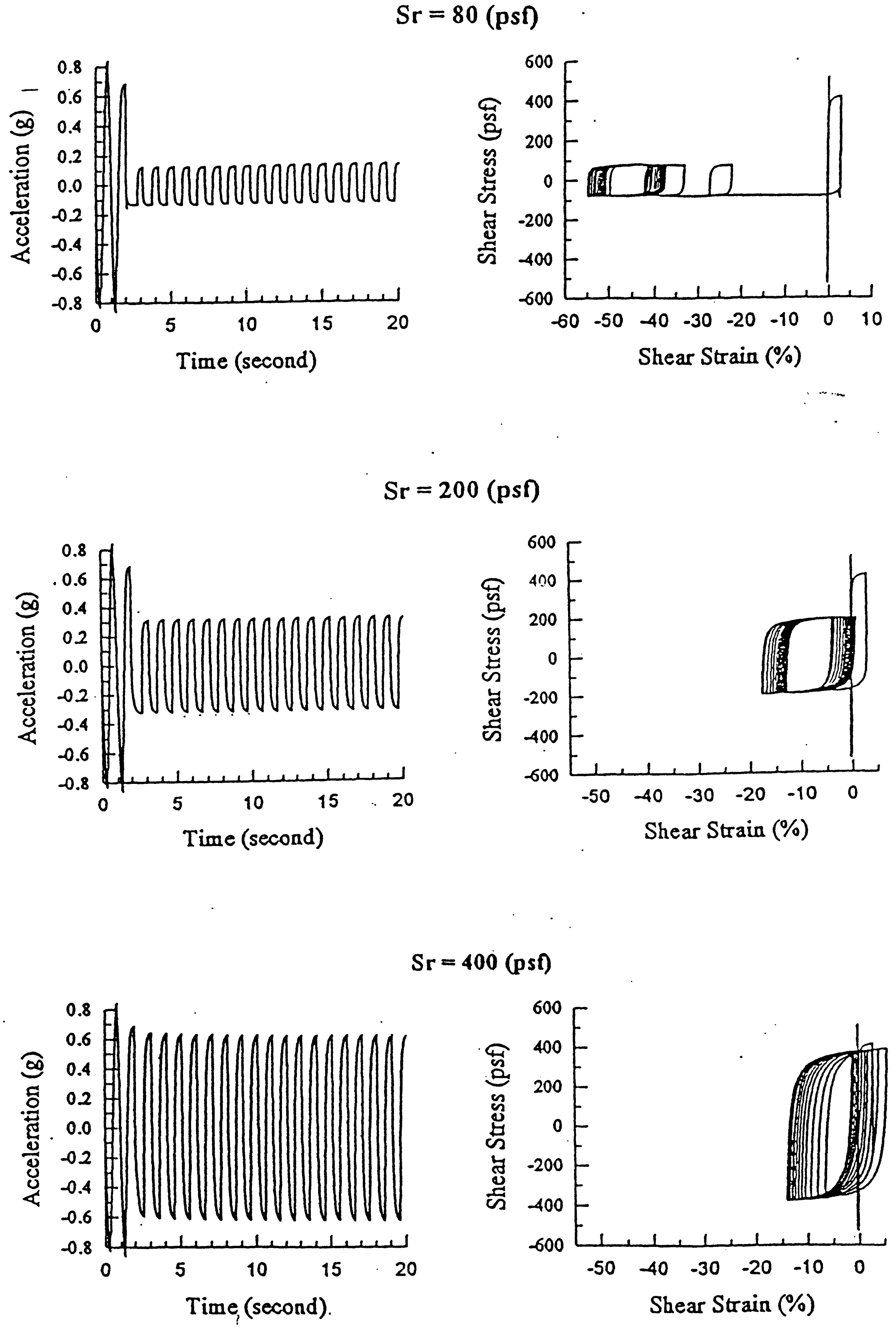
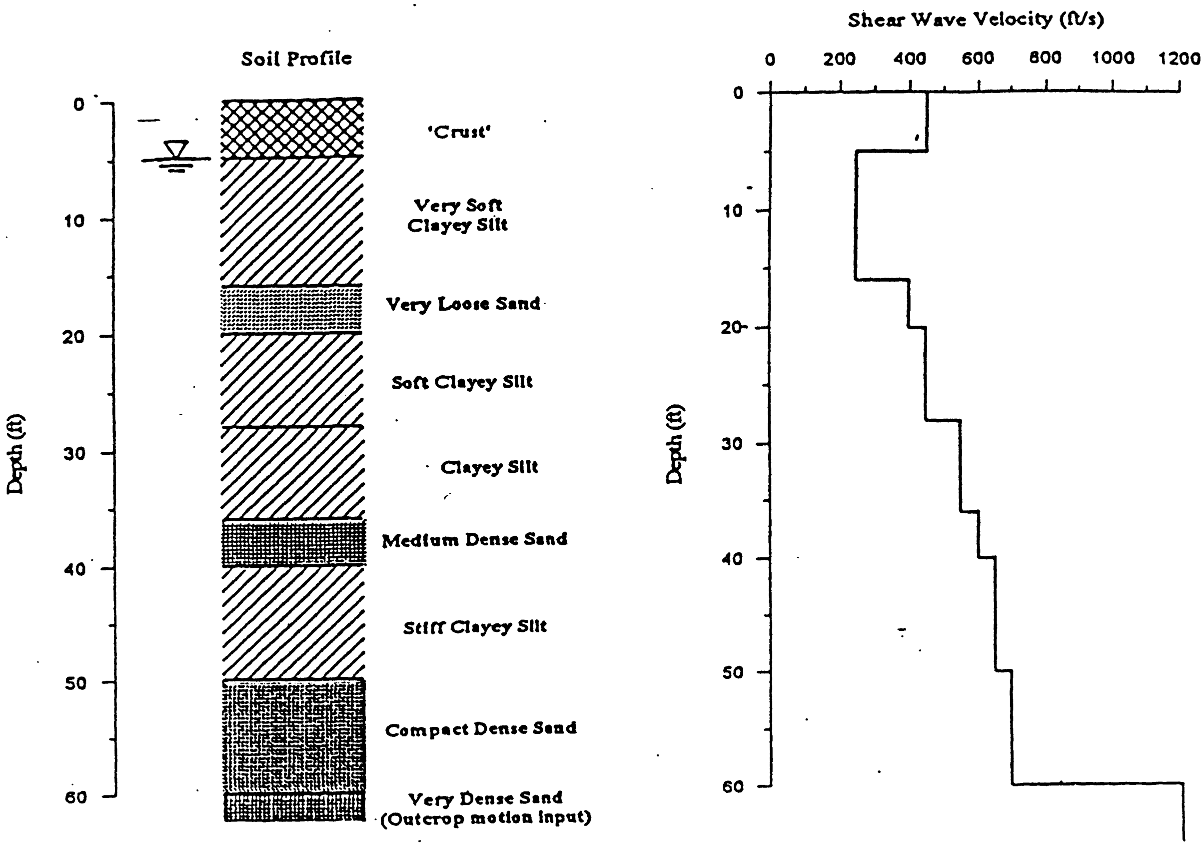


Figure 13 Effect of Residual Resistance on Ground Oscillation (after Qiu, 1998)



Soil Materials Used in the Case Studies

Layer No	Unit Weight (pcf)	Cohesion (psf)	Friction Angle	Shear Wave Velocity (ft/s)	SPT Blowcount
1	110	1200	N.A.	450	N.A.
2	110	400	N.A.	250	N.A.
3	110	N.A.	30	400	15
4	110	450	N.A.	450	N.A.
5	110	1500	N.A.	550	N.A.
6	110	N.A.	30	600	20
7	110	1800	N.A.	650	N.A.
8	110	N.A.	35	700	25
Bedrock	125	N.A.	N.A.	1200	N.A.

Figure 14 Soil Profile Used in Case Study Analyses (after Qiu, 1998)

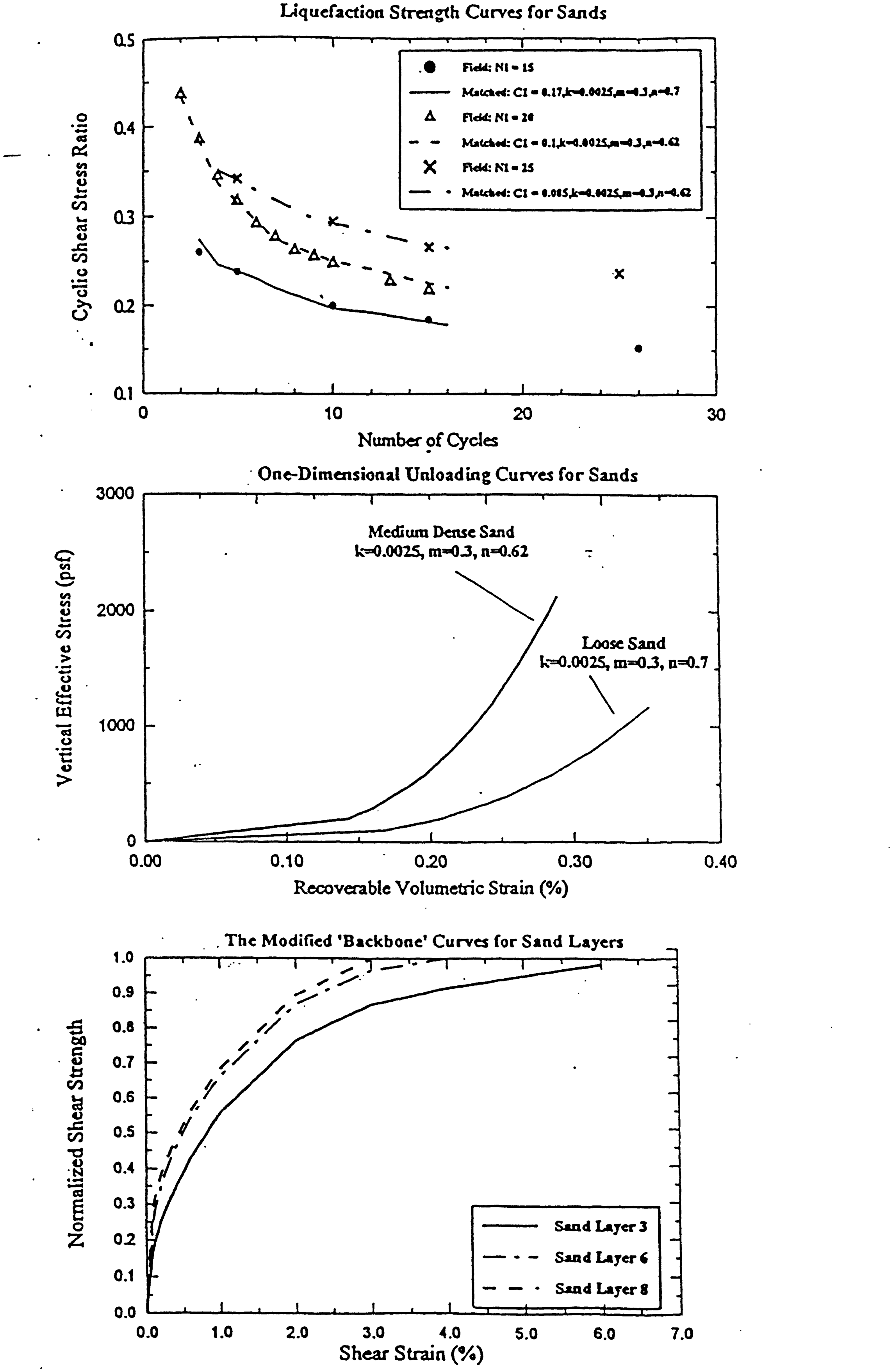


Figure 15 Liquefaction Strength, Elastic Rebound and Backbone Curves for Sand Strata Used in Case Study Analyses (after Qiu, 1998)

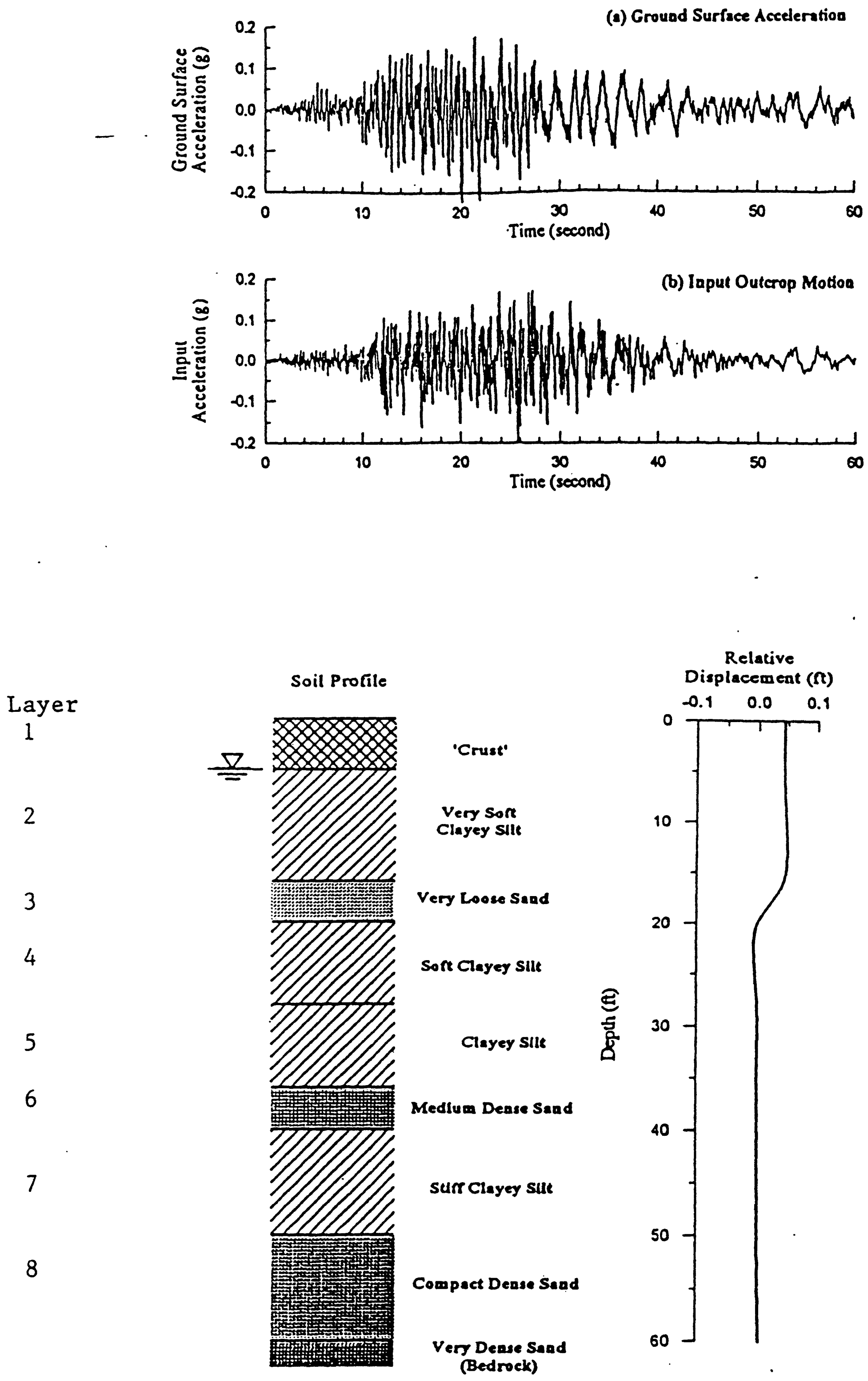
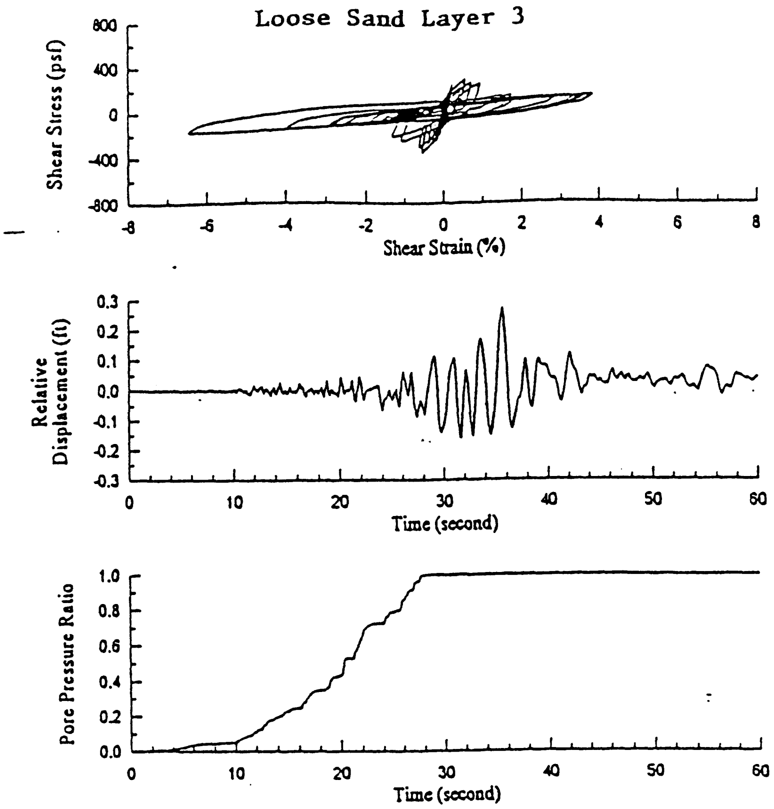


Figure 16 Acceleration and Ground Displacement Response: Amboy Record (after Qiu, 1998)



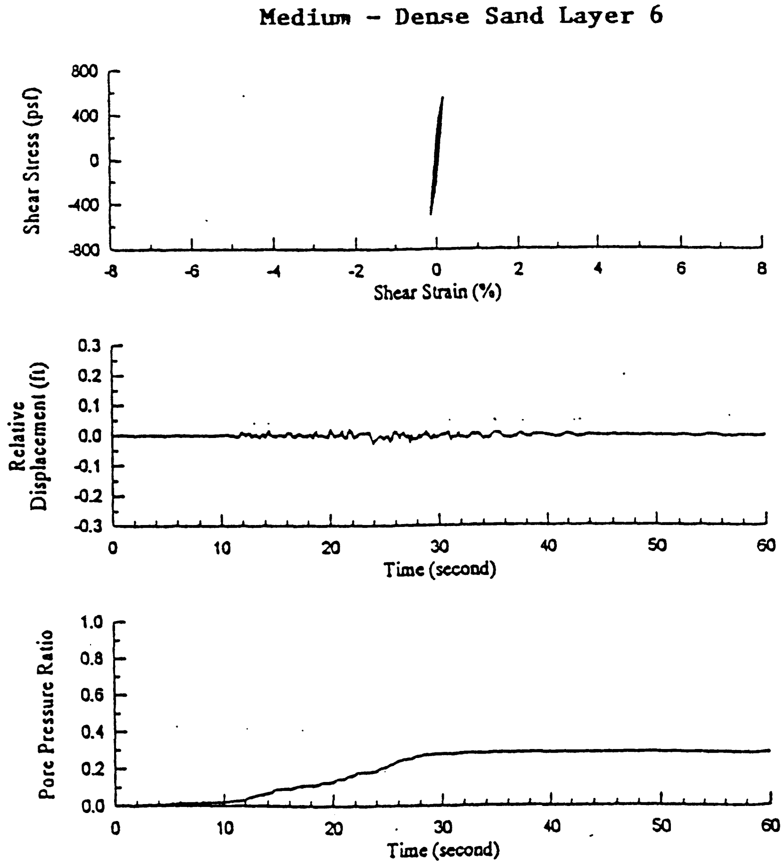
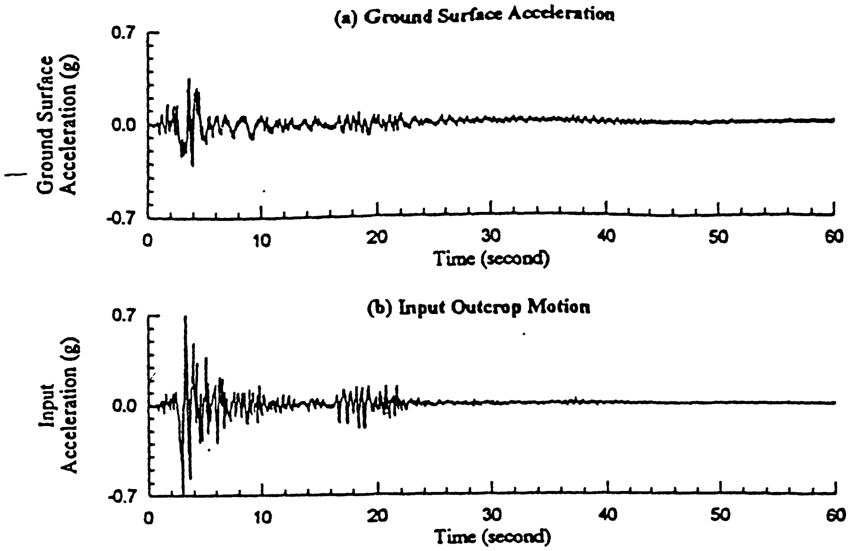


Figure 17 Pore Pressure, Dispalcement and Stress-Strain Response: Amboy Record (after Qiu, 1998)



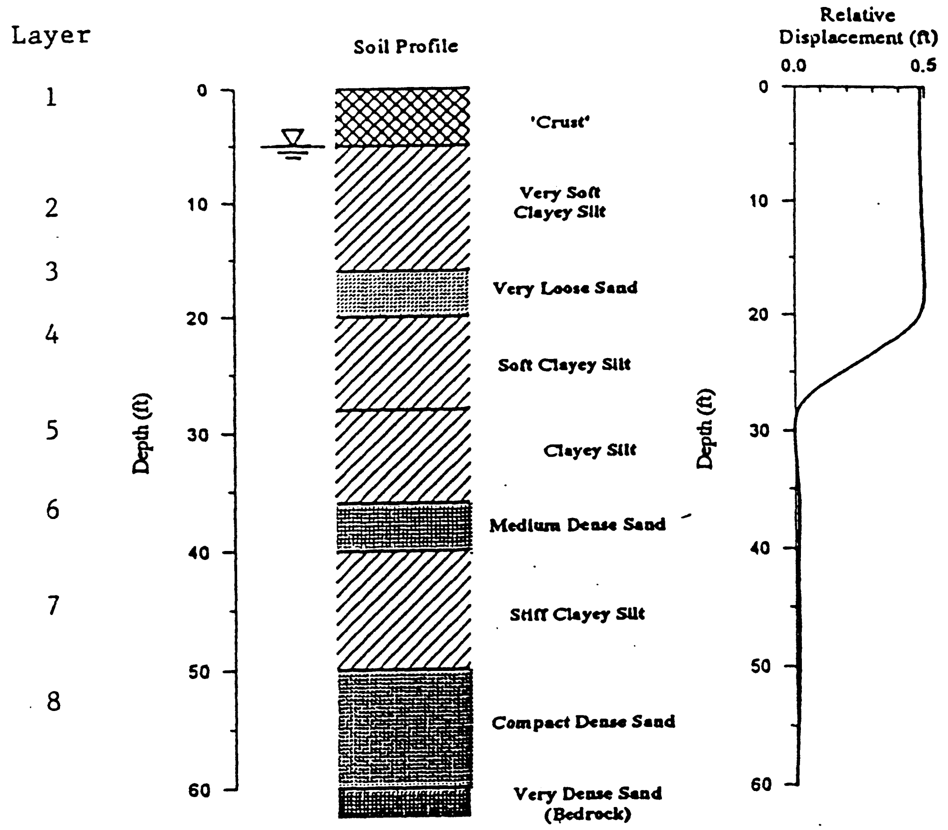
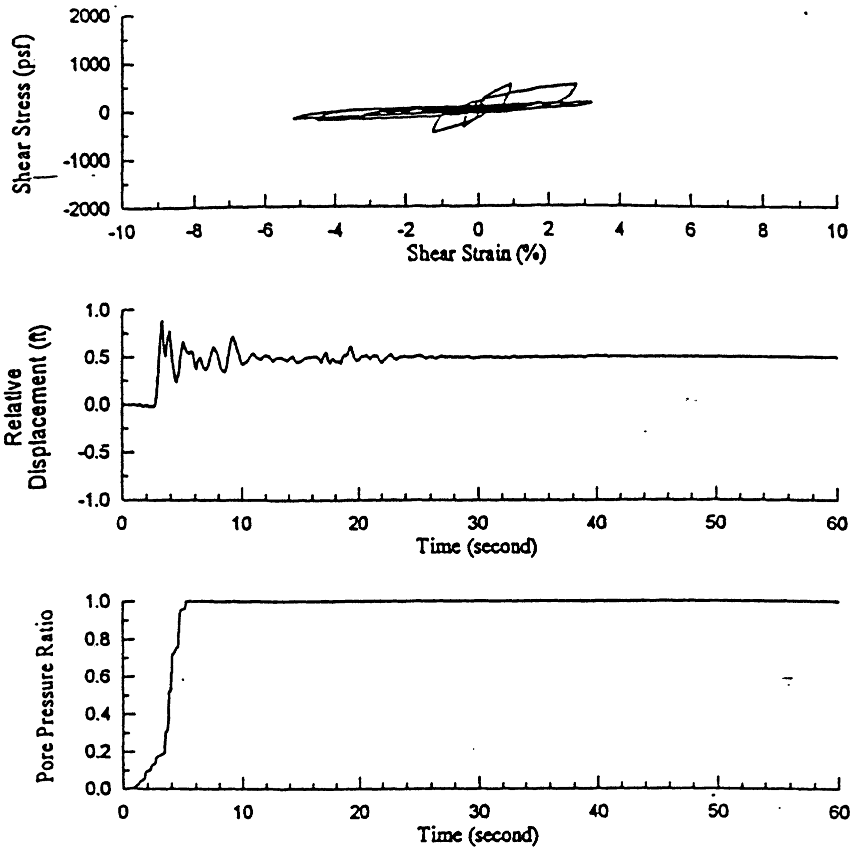


Figure 18 Acceleration and Ground Displacement Response: (after Qiu, 1998)

Loose Sand Layer 3



Soft Clayey Silt Layer 4

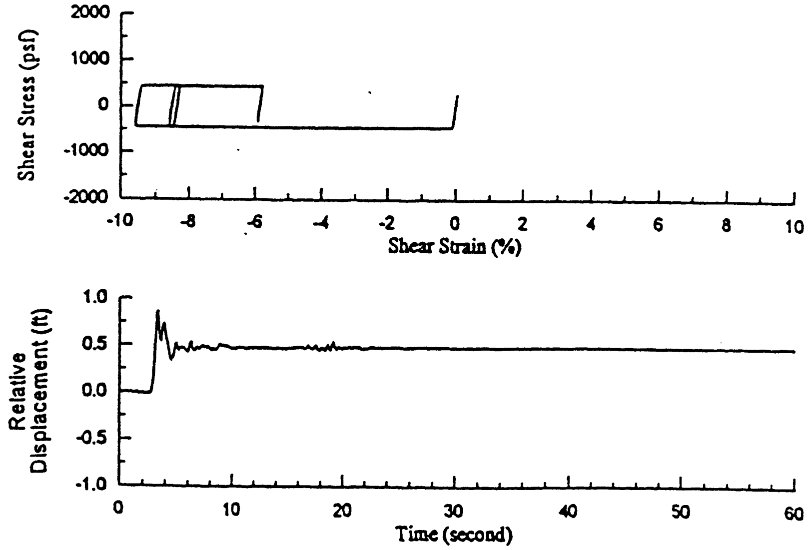
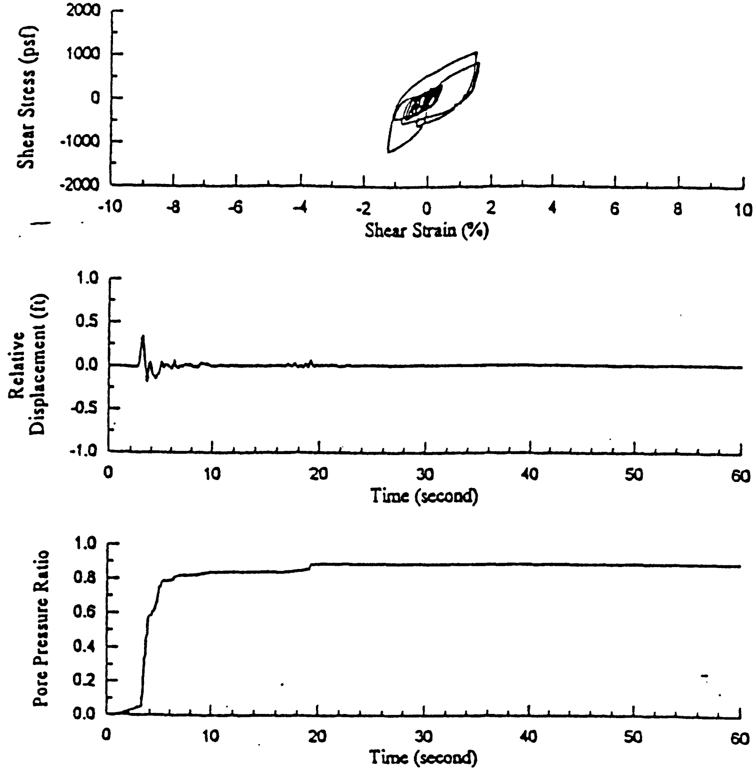


Figure 19 Pore Pressure, Displacement and Stress-Strain Response: Petrolia Record (after Qiu, 1998)

Medium - Dense Sand Layer 6



Medium - Dense Sand Layer 8

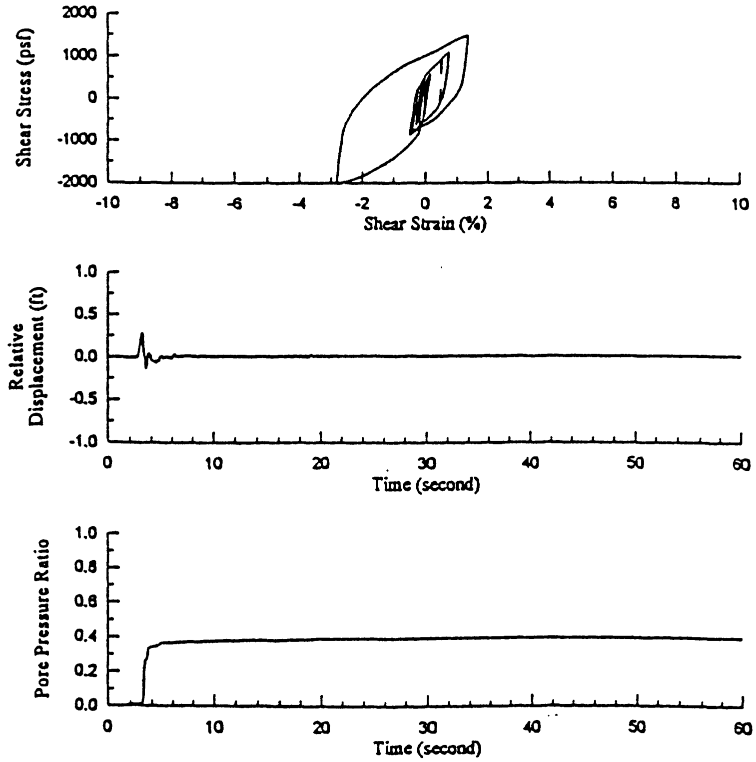


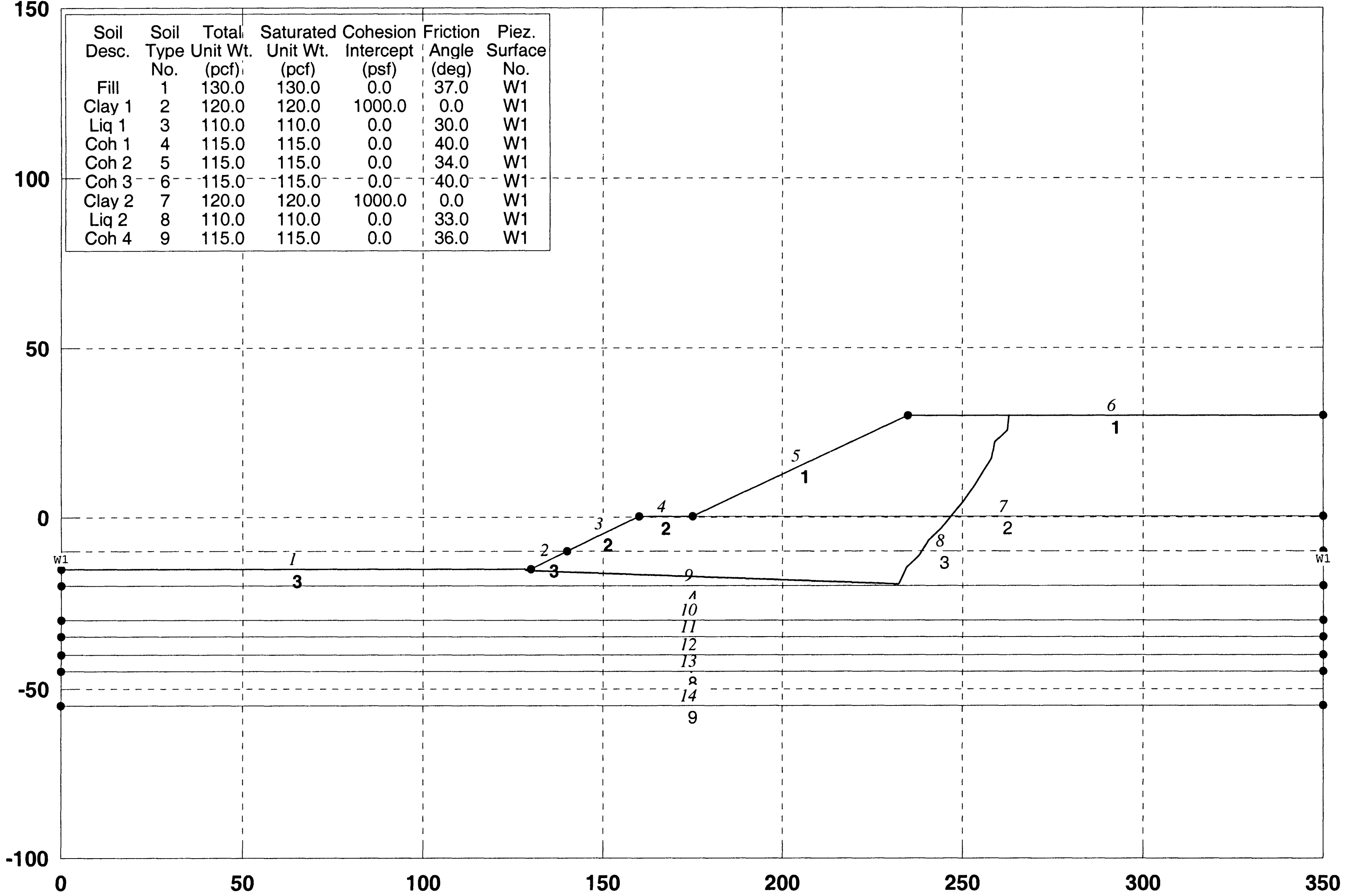
Figure 20 Pore Pressure, Displacement and Stress-Strain Response: Petrolia Record (after Qiu, 1998)

Appendix B
SIMPLIFIED SOIL ANALYSES — WESTERN SITE

B.1 Washington Stability Analyses — Preliquefaction Case

NCHRP 1249, Right Abut, RAWU0E wedge, PRELIQ, kh=0, Spencer

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\RIGHTA~1\PRAEW0.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 3:27PM

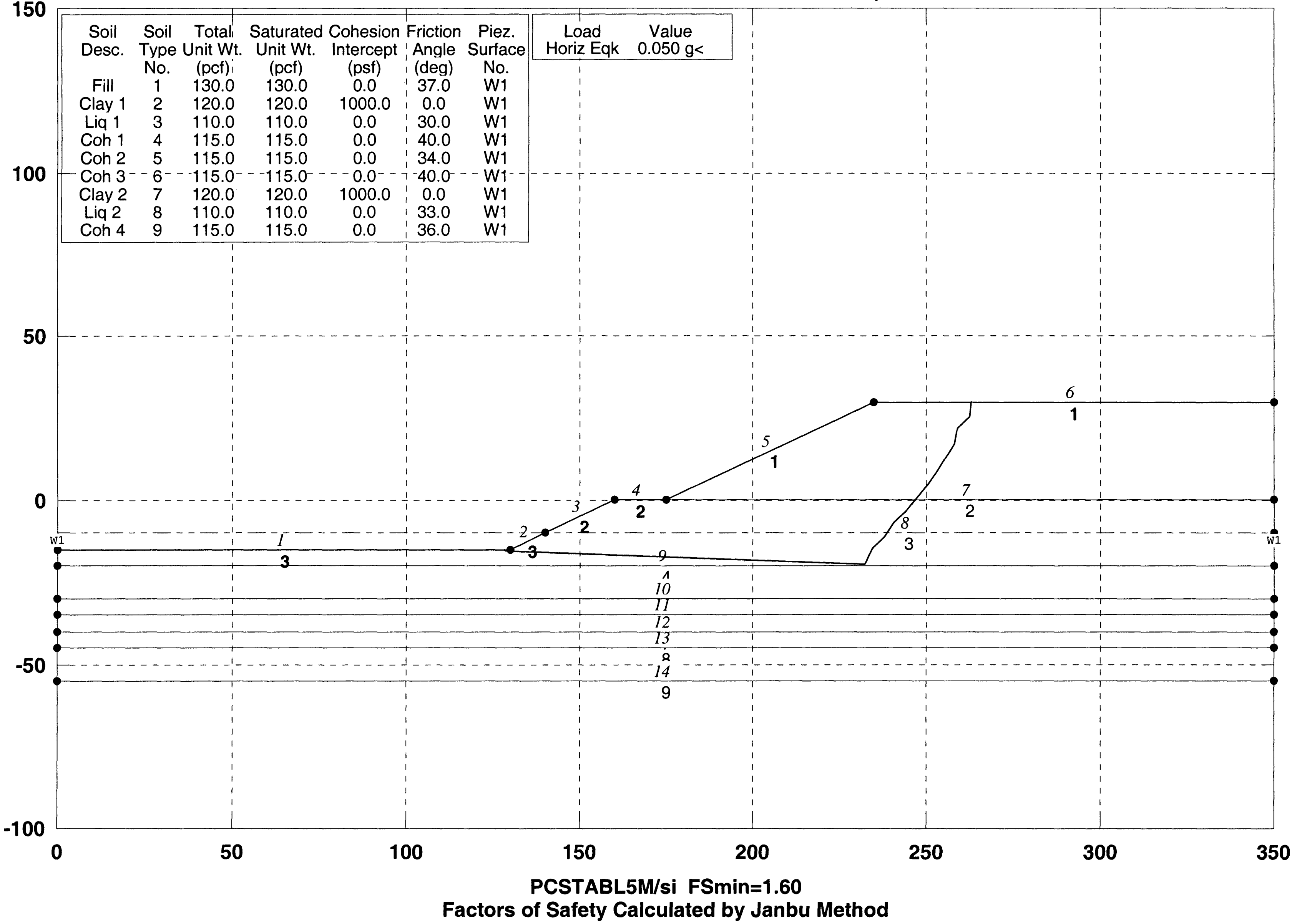


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

PCSTABL5M/si FSmin=1.87
Factors of Safety Calculated by Janbu Method

NCHRP 1249, Right Abut, RAWU0E wedge, PRELIQ, kh=0.05, Spencer

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\RIGHTA~1\PRAEW05.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 3:28PM

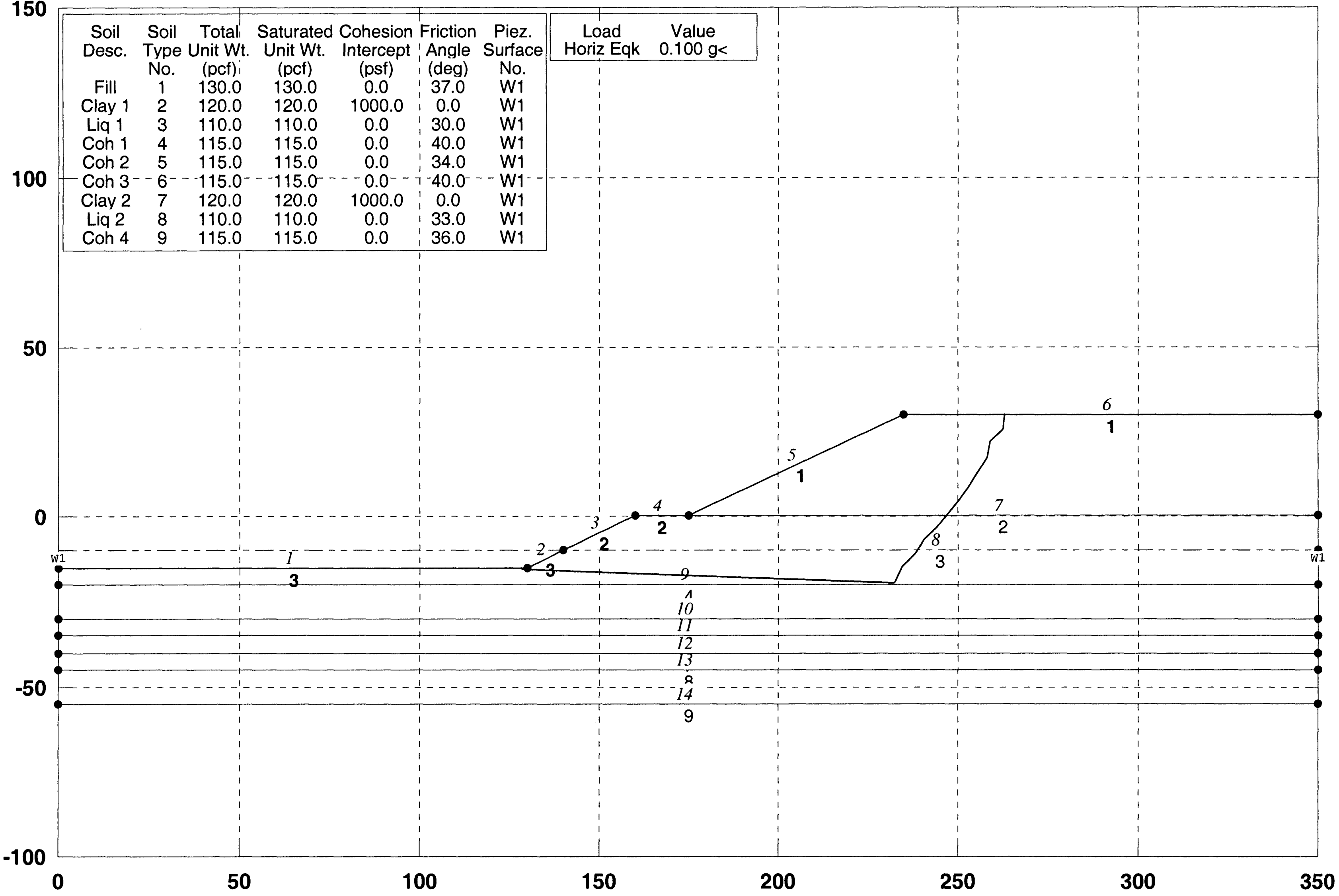


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.050 g<

NCHRP 1249, Right Abut, RAWU0E wedge, PRELIQ, kh=0.1, Spencer

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\RIGHTA~1\PRAEW1.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 3:29PM



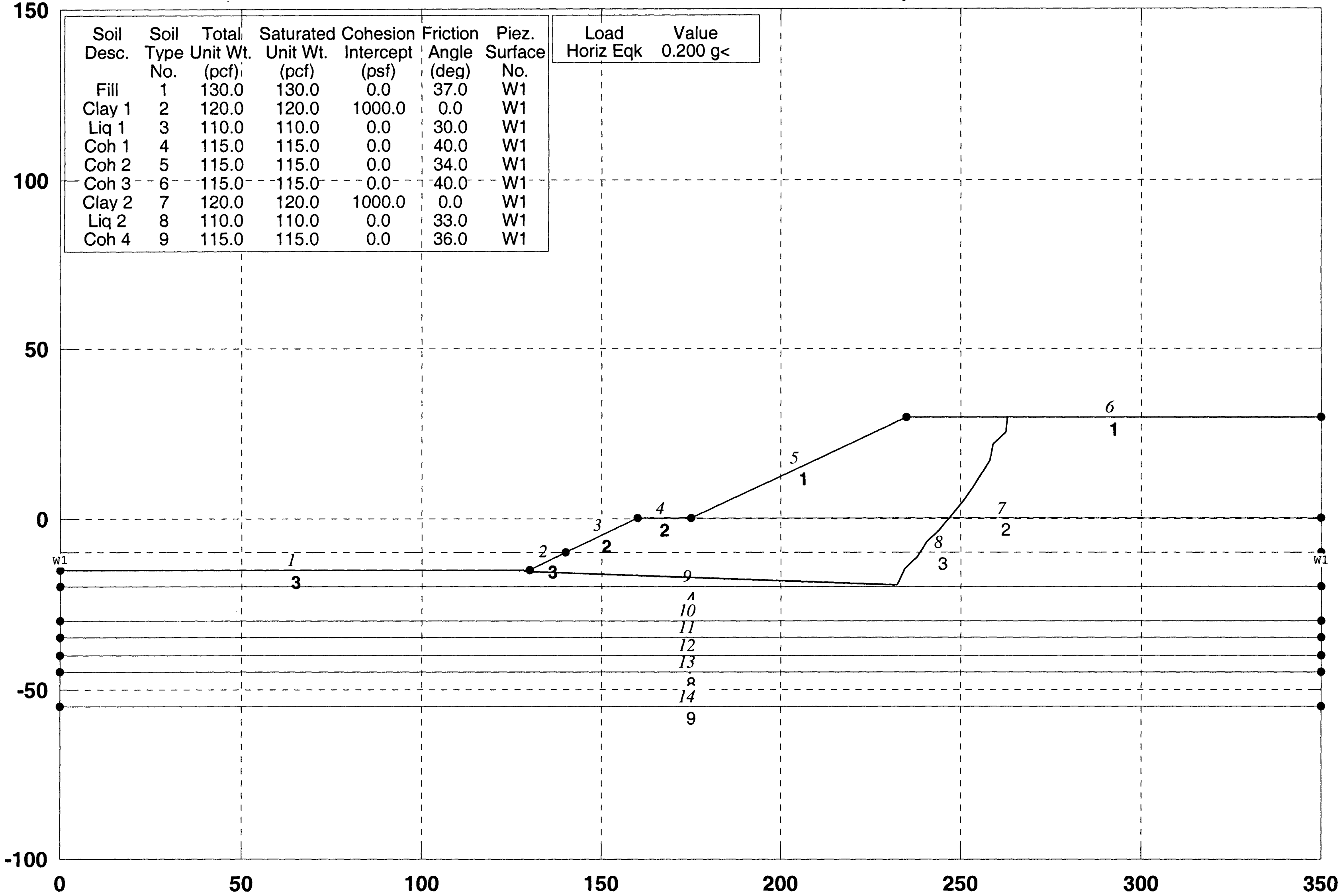
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=1.40
Factors of Safety Calculated by Janbu Method

NCHRP 1249, Right Abut, RAWU0E wedge, PRELIQ, kh=0.2, Spencer

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\RIGHTA~1\PRAEW2.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 3:30PM



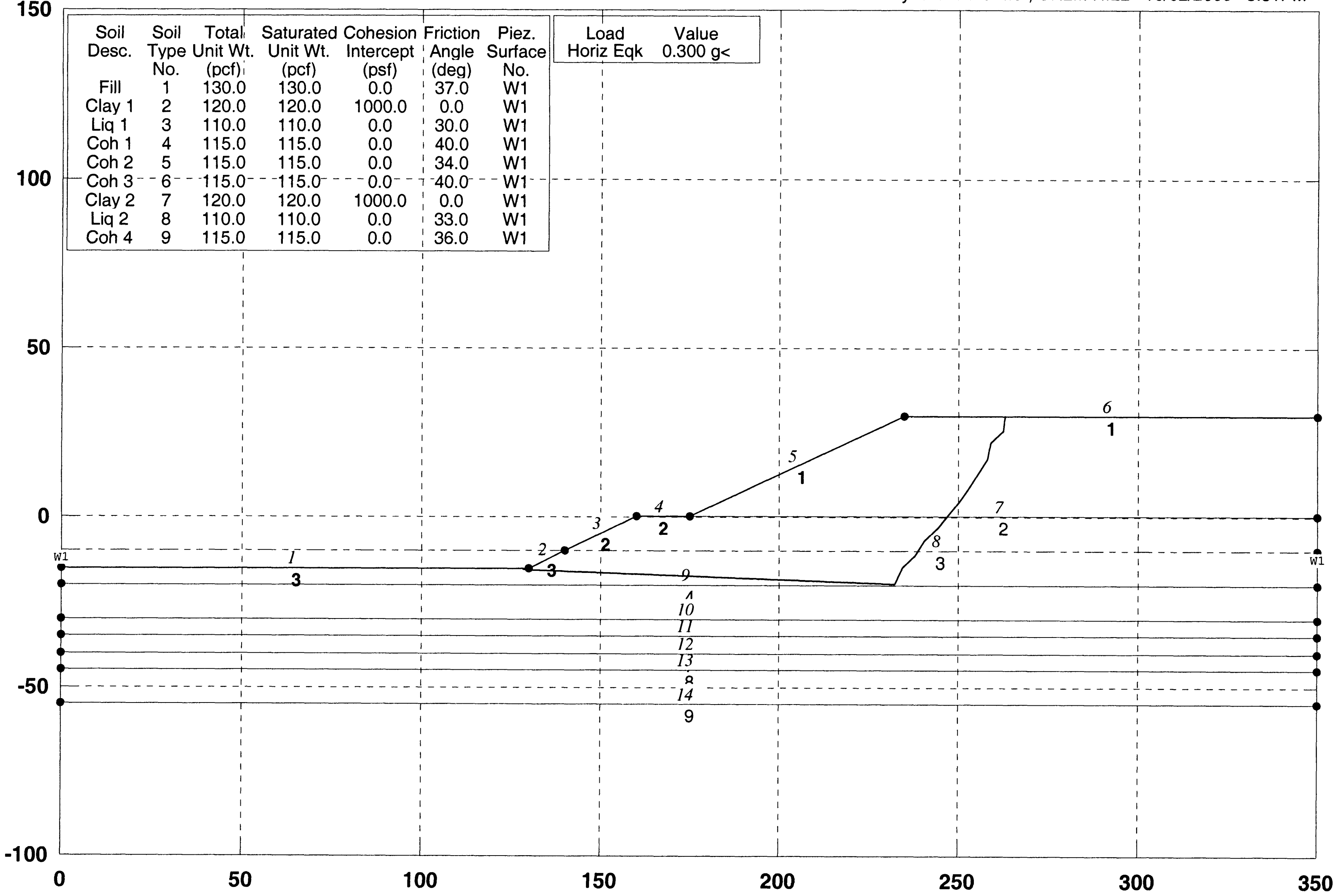
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.200 g<

PCSTABL5M/si FSmin=1.11
Factors of Safety Calculated by Janbu Method

NCHRP 1249, Right Abut, RAWU0E wedge, PRELIQ, kh=0.3, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\RIGHTA~1\PRAEW3.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 3:31PM



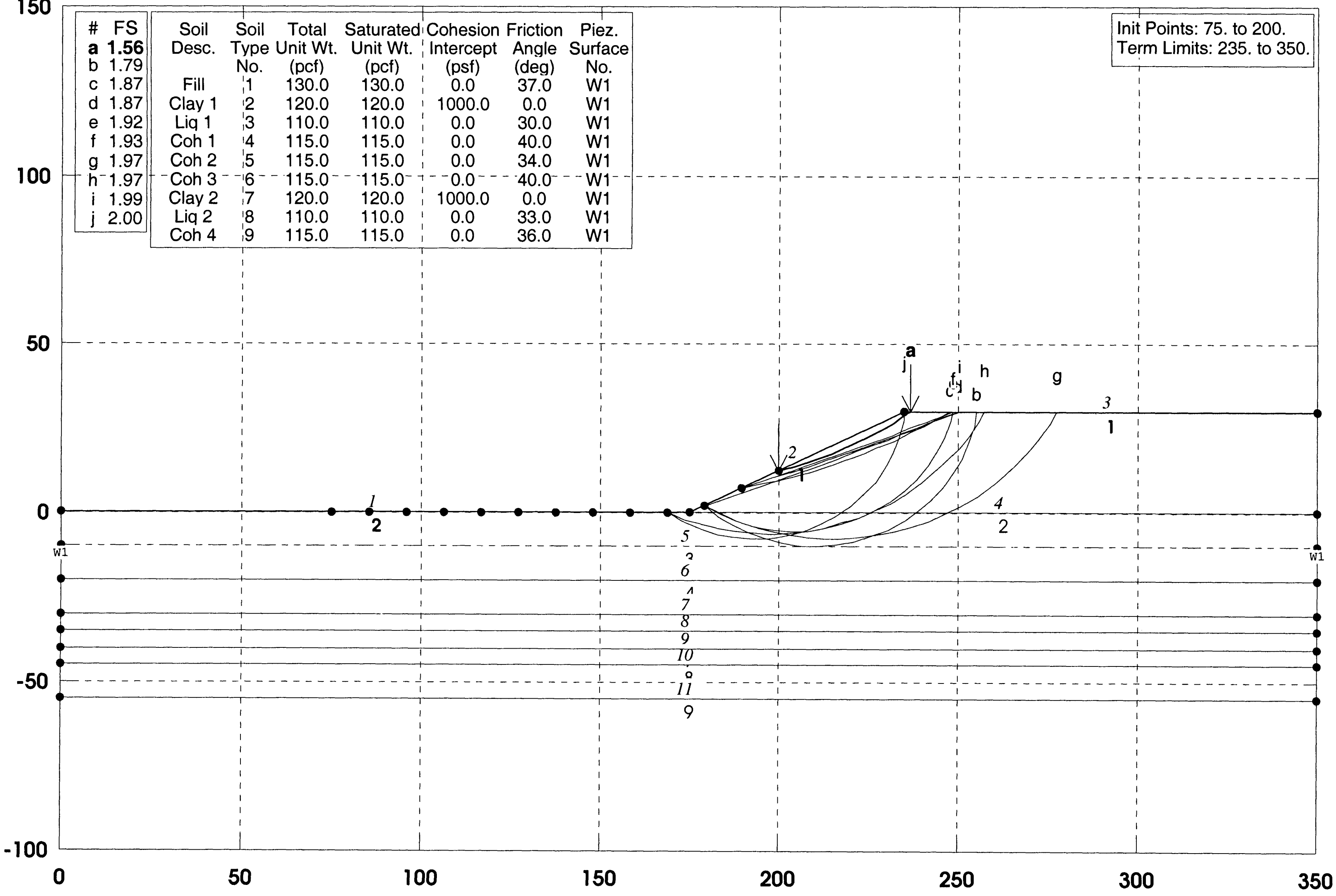
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.300 g<

PCSTABL5M/si FSmin=0.91
Factors of Safety Calculated by Janbu Method

NCHRP 1249, LEFT Abutment, Circle, kh=0, pre-liq strengths

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\LEFTAB~1\PLAC0.PL2 Run By: Jen Schaeffer 07/24/2000 9:51PM

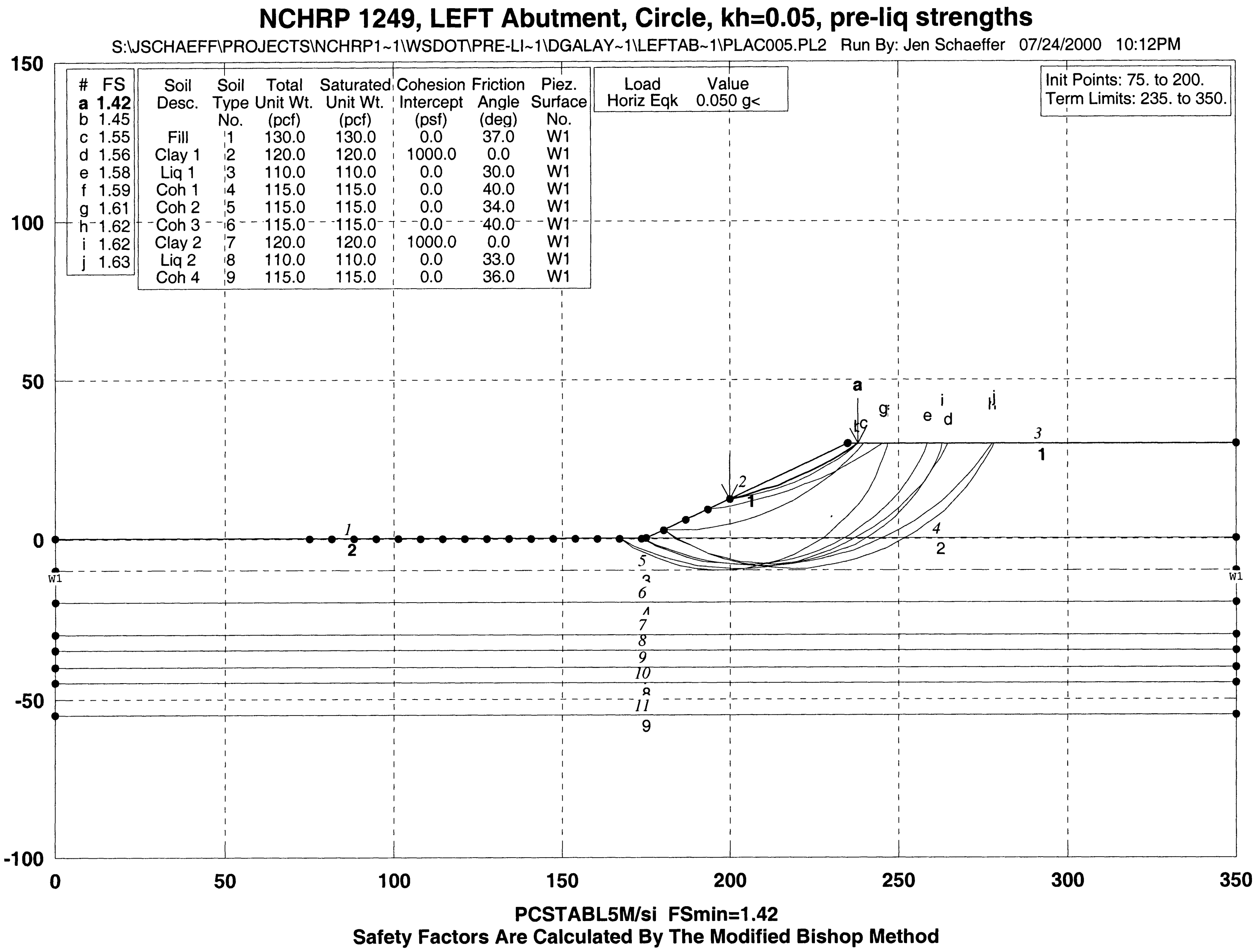


#	FS
a	**1.56**
b	1.79
c	1.87
d	1.87
e	1.92
f	1.93
g	1.97
h	1.97
i	1.99
j	2.00

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Init Points: 75. to 200.
Term Limits: 235. to 350.

PCSTABL5M/si FSmin=1.56
Safety Factors Are Calculated By The Modified Bishop Method



NCHRP 1249, LEFT Abutment, Circle, kh=0.05, pre-liq strengths

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\LEFTAB~1\PLAC005.PL2 Run By: Jen Schaeffer 07/24/2000 10:12PM

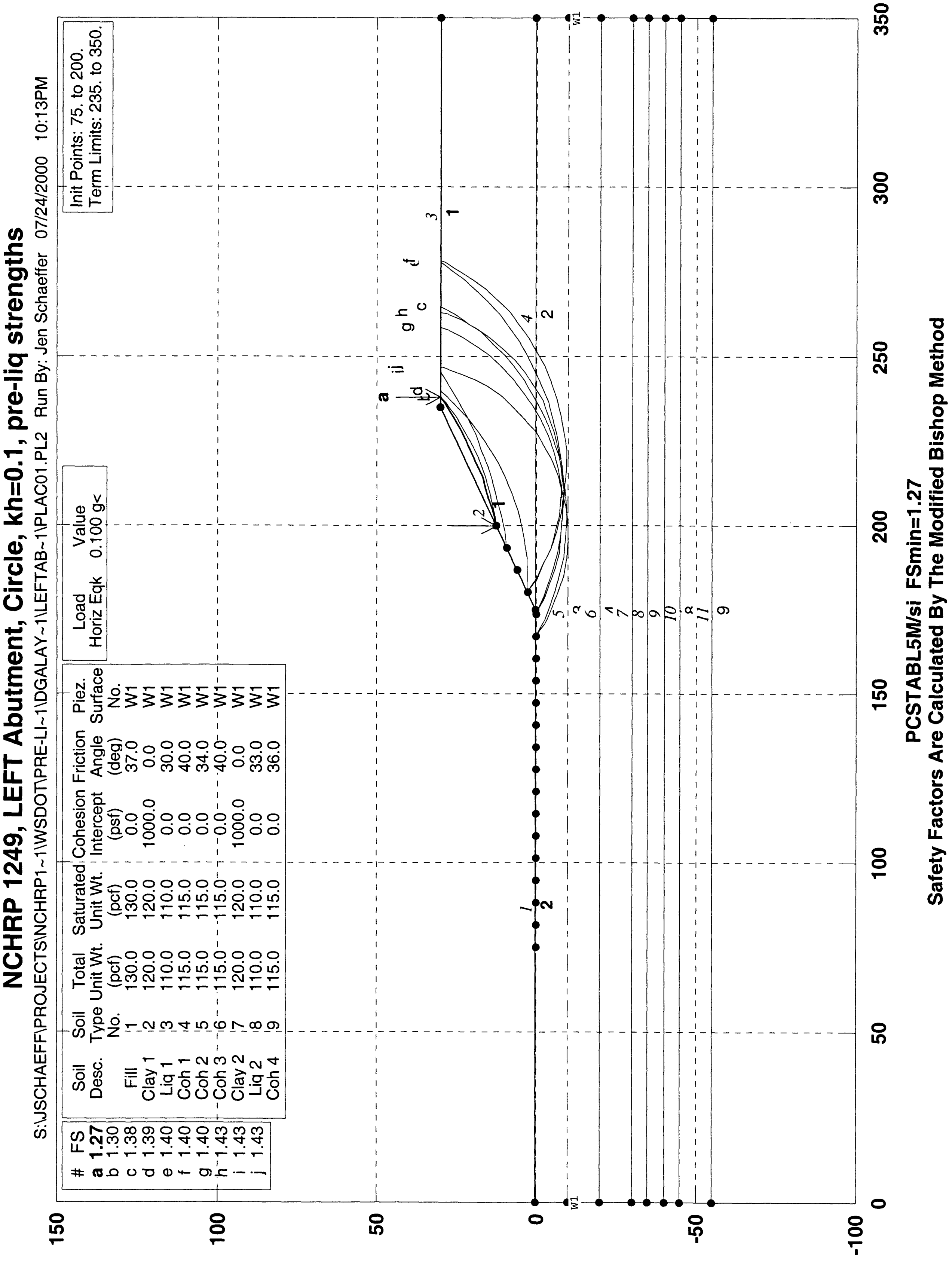
#	FS
a	**1.42**
b	1.45
c	1.55
d	1.56
e	1.58
f	1.59
g	1.61
h	1.62
i	1.62
j	1.63

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.050 g<

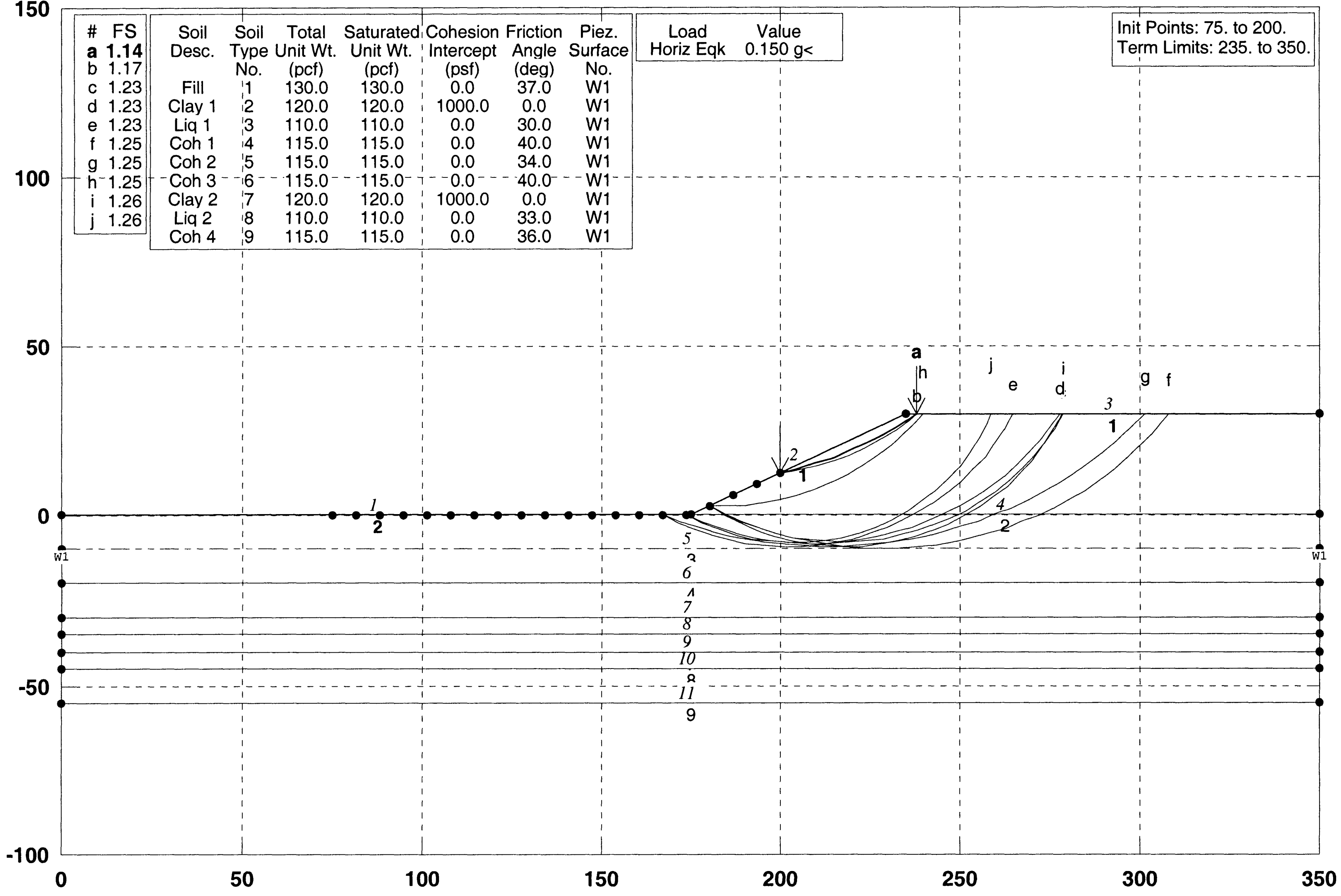
Init Points: 75. to 200.
Term Limits: 235. to 350.

PCSTABL5M/si FSmin=1.42
Safety Factors Are Calculated By The Modified Bishop Method


NCHRP 1249, LEFT Abutment, Circle, kh=0.1, pre-liq strengths
S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\LEFTAB~1\PLAC01.PL2 Run By: Jen Schaeffer 07/24/2000 10:13PM
FS
a 1.27
b 1.30
c 1.38
d 1.39
e 1.40
f 1.40
g 1.40
h 1.43
i 1.43
j 1.43
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Fill 1 130.0 130.0 0.0 37.0 W1
Clay 1 2 120.0 120.0 1000.0 0.0 W1
Liq 1 3 110.0 110.0 0.0 30.0 W1
Coh 1 4 115.0 115.0 0.0 40.0 W1
Coh 2 5 115.0 115.0 0.0 34.0 W1
Coh 3 6 115.0 115.0 0.0 40.0 W1
Clay 2 7 120.0 120.0 1000.0 0.0 W1
Liq 2 8 110.0 110.0 0.0 33.0 W1
Coh 4 9 115.0 115.0 0.0 36.0 W1
Load Value
Horiz Eqk 0.100 g<
Init Points: 75. to 200.
Term Limits: 235. to 350.
W1
a
ij
bd
g h
c
e f
1 2 3 4 5 6 7 8 9 10 11
1 2
150
100
50
0
-50
-100
0
50
100
150
200
250
300
350
PCSTABL5M/si FSmin=1.27
Safety Factors Are Calculated By The Modified Bishop Method

NCHRP 1249, LEFT Abutment, Circle, kh=0.15, pre-liq strengths

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\LEFTAB~1\PLAC015.PL2 Run By: Jen Schaeffer 07/24/2000 10:15PM



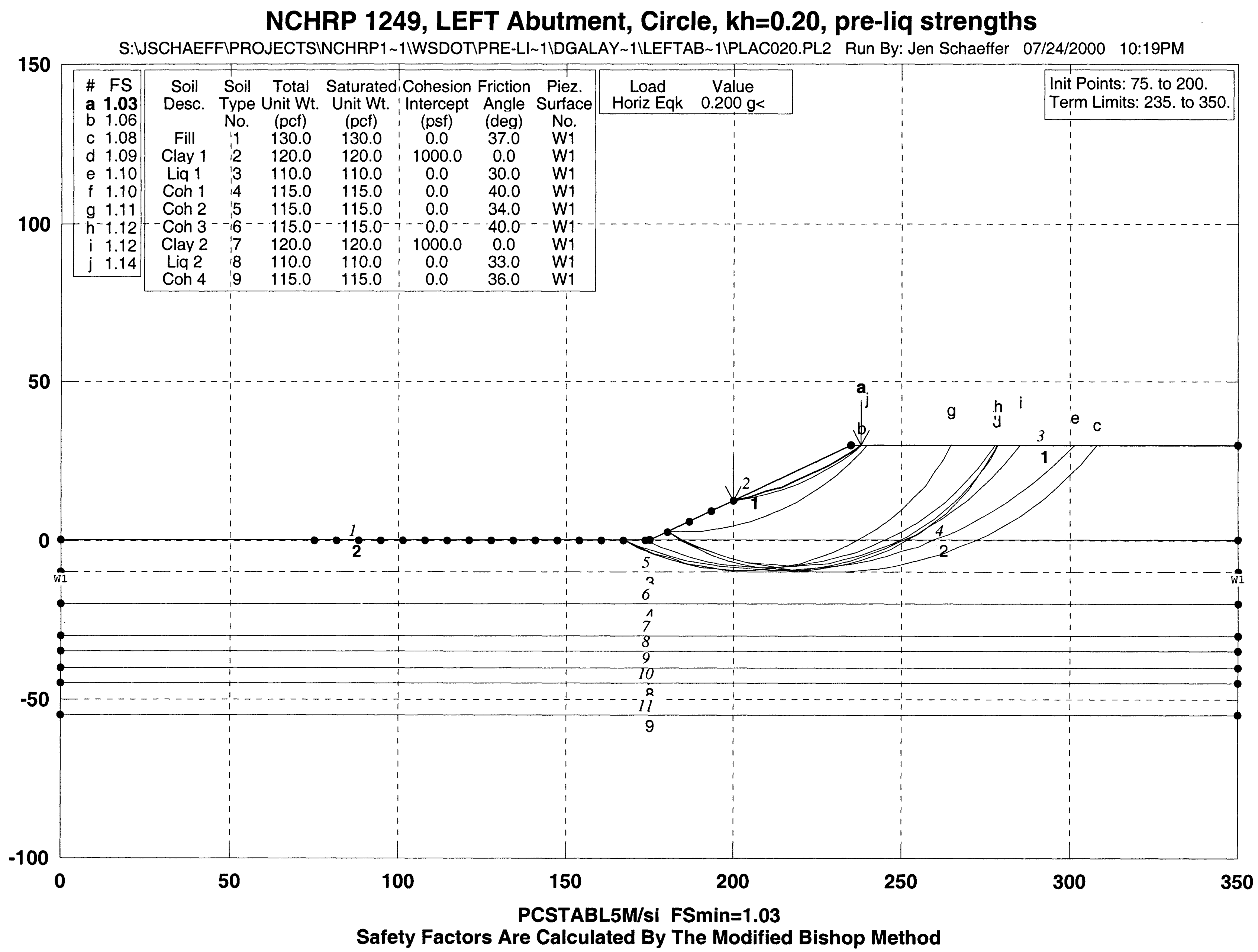
#	FS
a	**1.14**
b	1.17
c	1.23
d	1.23
e	1.23
f	1.25
g	1.25
h	1.25
i	1.26
j	1.26

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.150 g<

Init Points: 75. to 200.
Term Limits: 235. to 350.

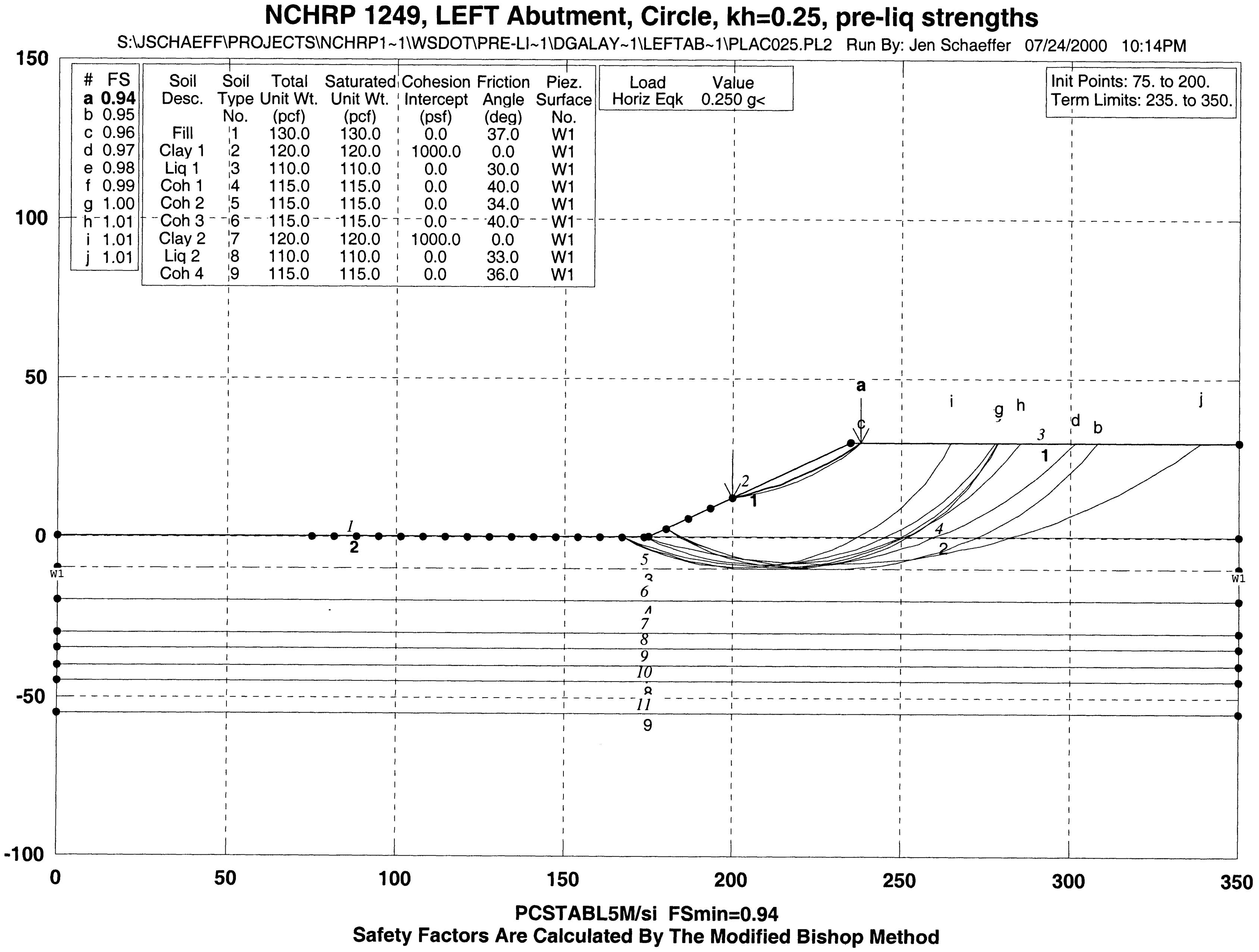
PCSTABL5M/si FSmin=1.14
Safety Factors Are Calculated By The Modified Bishop Method



#	FS
a	**1.03**
b	1.06
c	1.08
d	1.09
e	1.10
f	1.10
g	1.11
h	1.12
i	1.12
j	1.14

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.200 g<



NCHRP 1249, LEFT Abutment, Circle, kh=0.25, pre-liq strengths

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PRE-LI~1\DGALAY~1\LEFTAB~1\PLAC025.PL2 Run By: Jen Schaeffer 07/24/2000 10:14PM

#	FS
a	**0.94**
b	0.95
c	0.96
d	0.97
e	0.98
f	0.99
g	1.00
h	1.01
i	1.01
j	1.01

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	0.0	30.0	W1
Coh 1	4	115.0	115.0	0.0	40.0	W1
Coh 2	5	115.0	115.0	0.0	34.0	W1
Coh 3	6	115.0	115.0	0.0	40.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	0.0	33.0	W1
Coh 4	9	115.0	115.0	0.0	36.0	W1

Load	Value
Horiz Eqk	0.250 g<

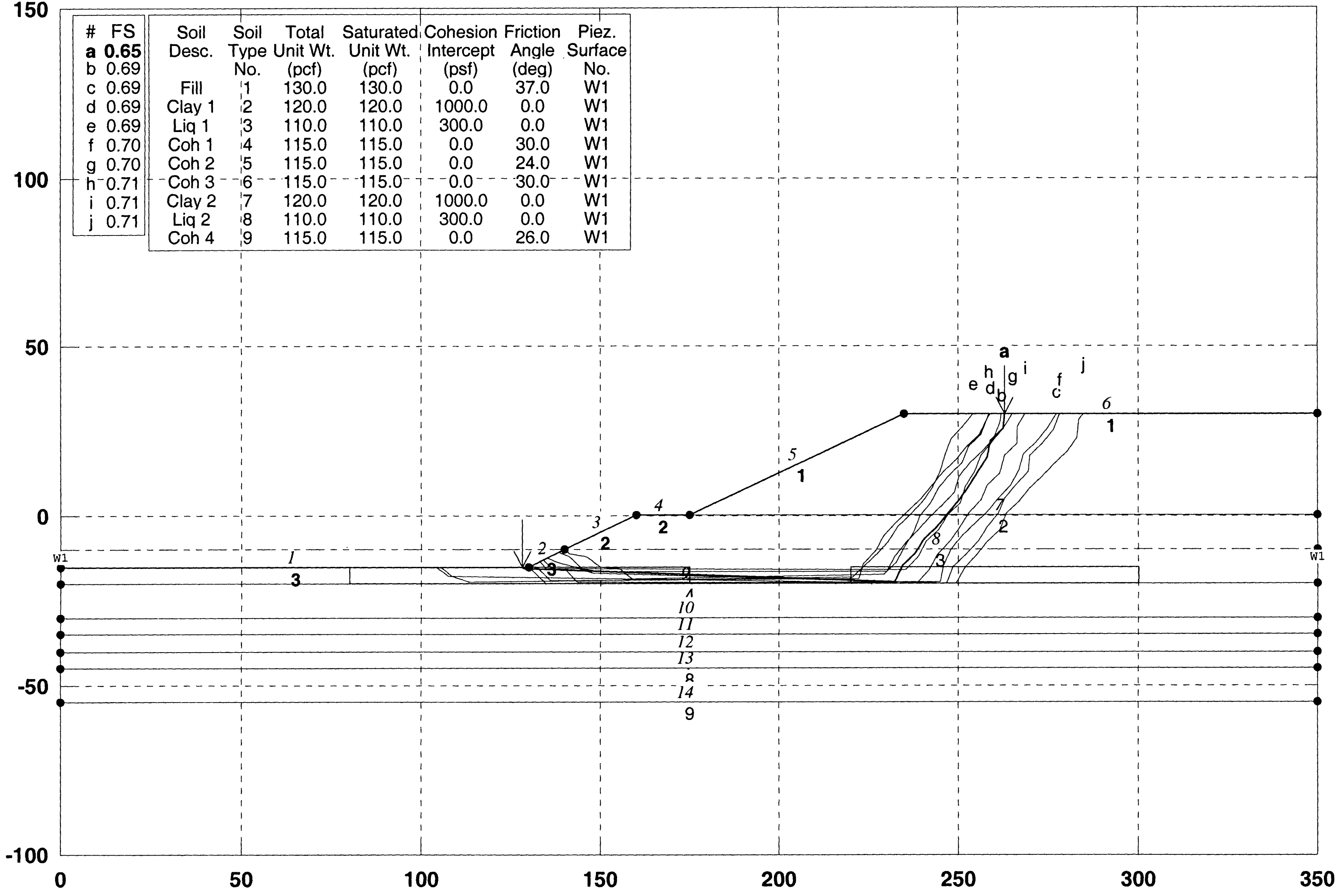
Init Points: 75. to 200.
Term Limits: 235. to 350.

PCSTABL5M/si FSmin=0.94
Safety Factors Are Calculated By The Modified Bishop Method

B.2 Washington Stability Analyses — Flow Failure Case

NCHRP 1249, RIGHT Abut., wedge, random, kh=0, liq=300, target up liq

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\POSTLI~1\RIGHTA~1\WEDGES\RAWU0E.PL2 Run By: Jen Schaeffer 08/28/2000 5:15PM



#	FS
a	**0.65**
b	0.69
c	0.69
d	0.69
e	0.69
f	0.70
g	0.70
h	0.71
i	0.71
j	0.71

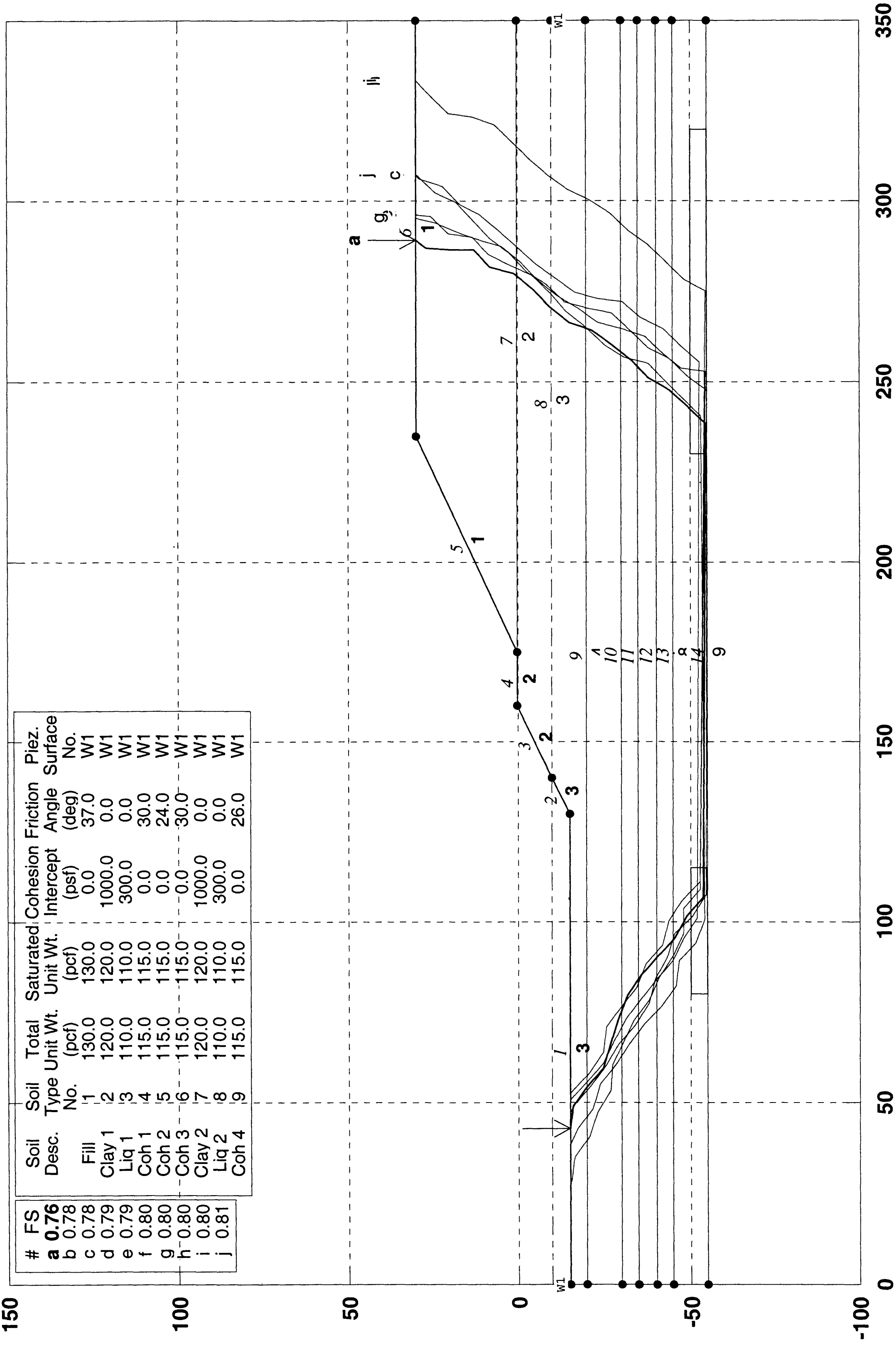
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=0.65

Safety Factors Are Calculated By The Modified Janbu Method

NCHRP 1249, WSDOT, R Abut, lower wedge pin, kh=0, search for crit surface J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PILES\SMEARE~1\PINNED~1\XRLWP1.PL2 Run By: Jen Schaeffer, CH2M HILL 10/11/00 10:50AM



#	FS
a	**0.76**
b	0.78
c	0.78
d	0.79
e	0.79
f	0.80
g	0.80
h	0.80
i	0.80
j	0.81

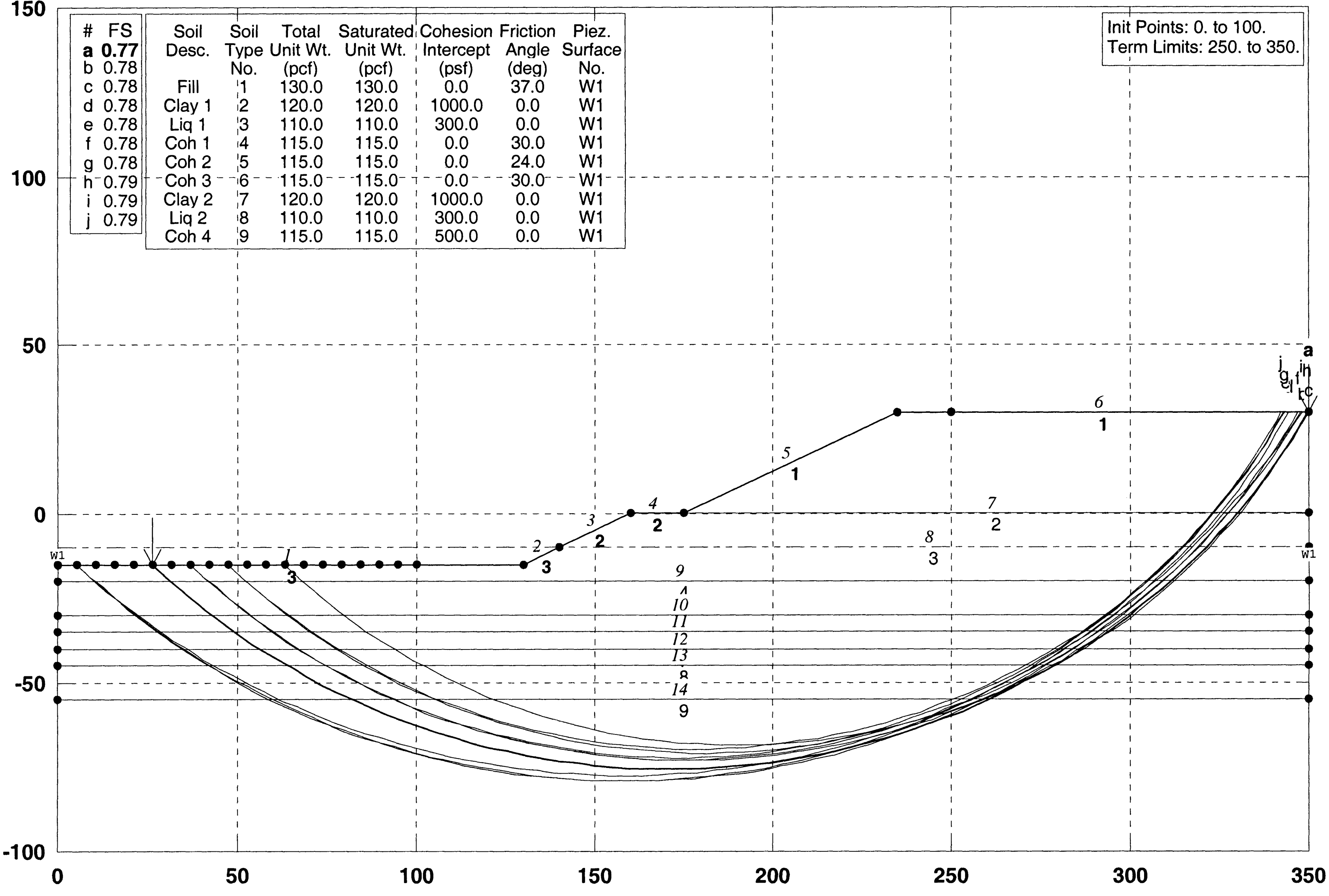
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=0.76

Safety Factors Are Calculated By The Modified Janbu Method

NCHRP 1249, RIGHT Abut., very deep liq, circle search, kh=0, #9 c=500

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\POSTLI~1\RIGHTA~1\VERYDE~1\VDC1.PL2 Run By: Jen Schaeffer, CH2M HILL 10/7/00 9:39PM



#	FS
a	**0.77**
b	0.78
c	0.78
d	0.78
e	0.78
f	0.78
g	0.78
h	0.79
i	0.79
j	0.79

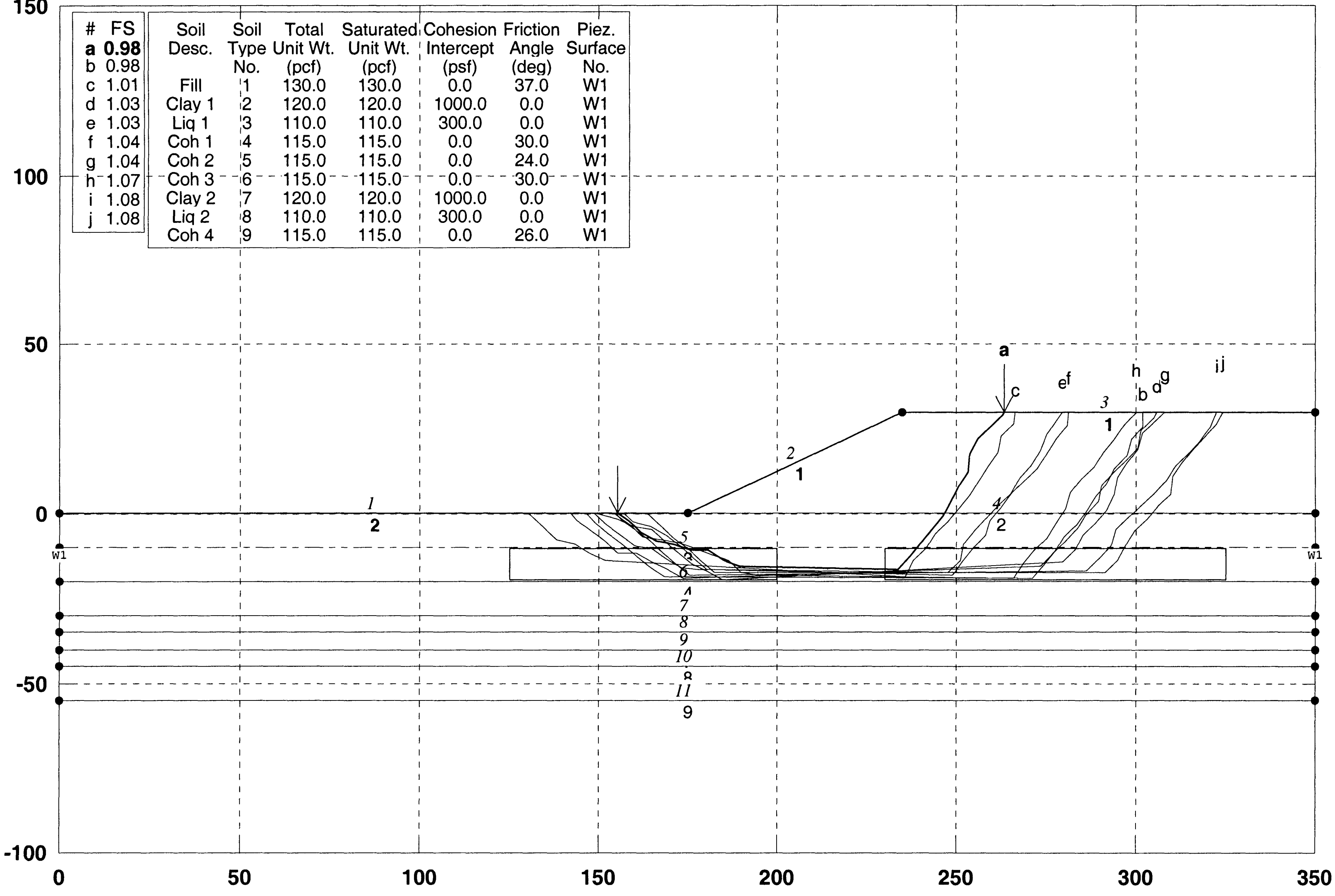
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	500.0	0.0	W1

Init Points: 0. to 100.
Term Limits: 250. to 350.

PCSTABL5M/si FSmin=0.77
Safety Factors Are Calculated By The Modified Bishop Method

NCHRP 1249 Project - LEFT Abutment, Wedge, upper liq layer, kh =0, Random

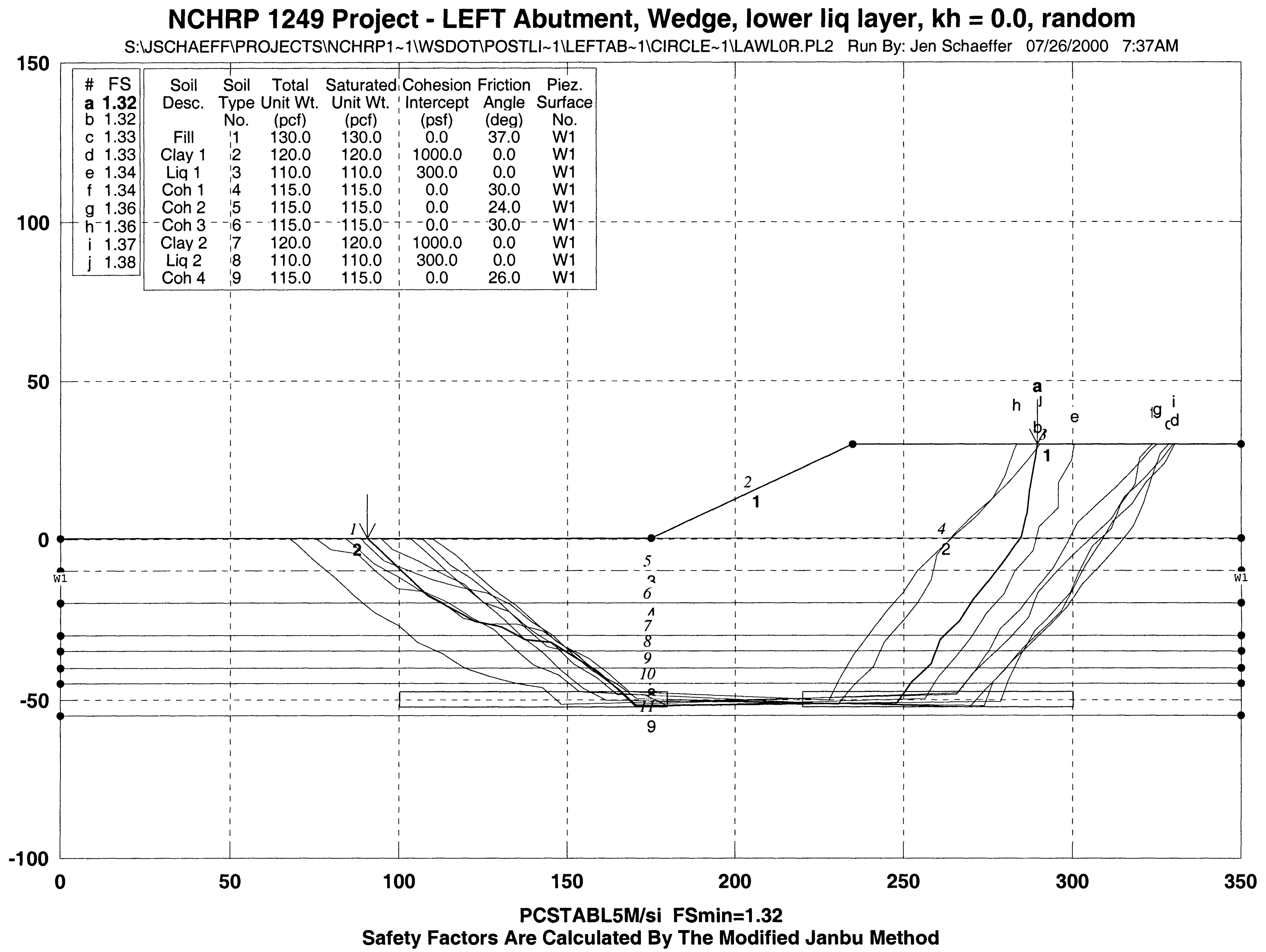
S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\POSTLI~1\LEFTAB~1\CIRCLE~1\LAWU0R.PL2 Run By: Jen Schaeffer 07/26/2000 7:31AM



#	FS
a	**0.98**
b	0.98
c	1.01
d	1.03
e	1.03
f	1.04
g	1.04
h	1.07
i	1.08
j	1.08

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=0.98
Safety Factors Are Calculated By The Modified Janbu Method



NCHRP 1249 Project - LEFT Abutment, Wedge, lower liq layer, kh = 0.0, random

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\POSTLI~1\LEFTAB~1\CIRCLE~1\LAWL0R.PL2 Run By: Jen Schaeffer 07/26/2000 7:37AM

#	FS
a	**1.32**
b	1.32
c	1.33
d	1.33
e	1.34
f	1.34
g	1.36
h	1.36
i	1.37
j	1.38

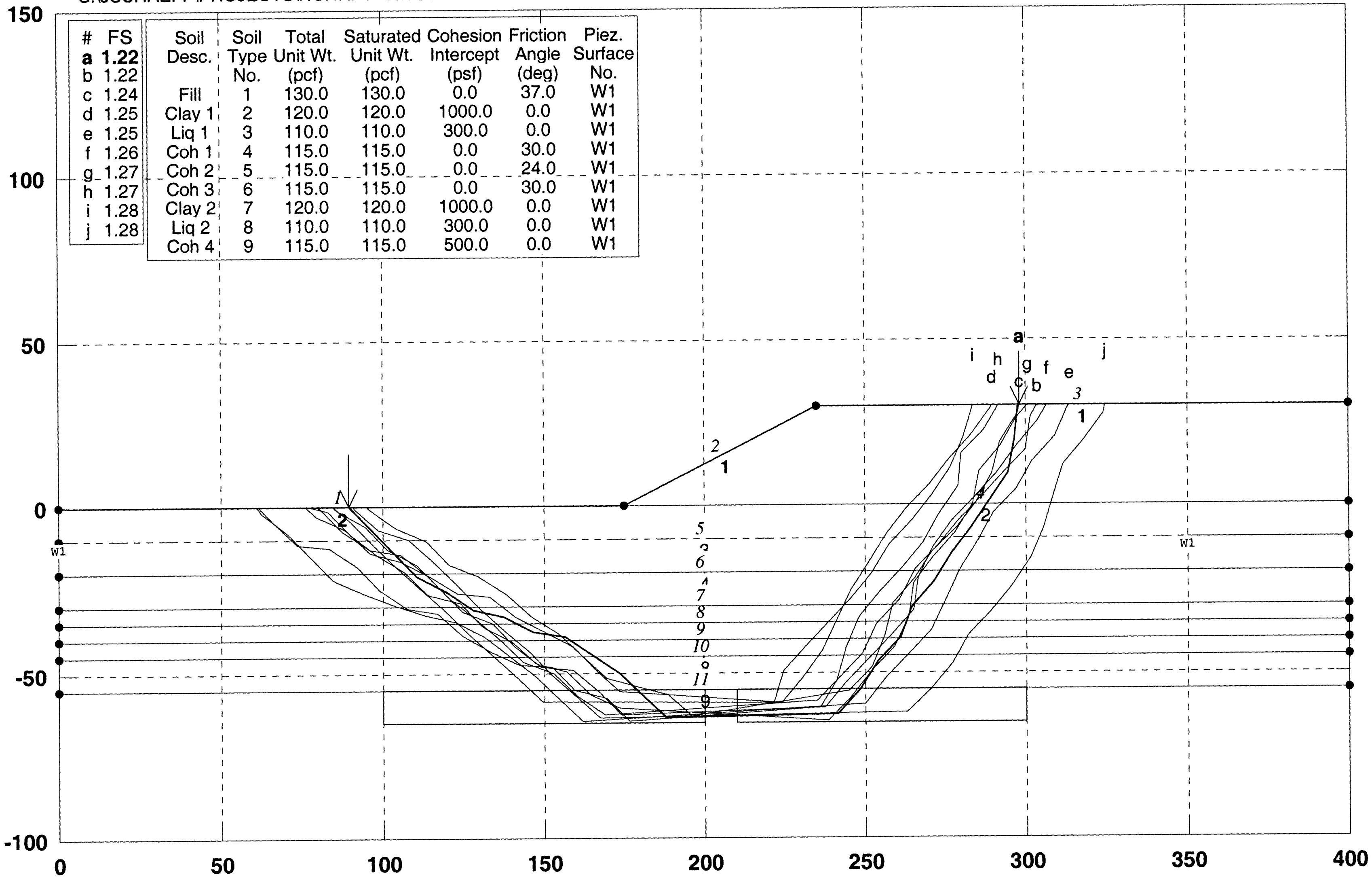
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=1.32

Safety Factors Are Calculated By The Modified Janbu Method

NCHRP 1249, Left Abut, Very Deep Liq, wedge search, kh = 0.0, #9 c=500 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\POSTLI~1\LEFTAB~1\VERYDE~1\LVDW1.PL2 Run By: Jen Schaeffer, CH2M HILL 10/12/2000 4:16PM



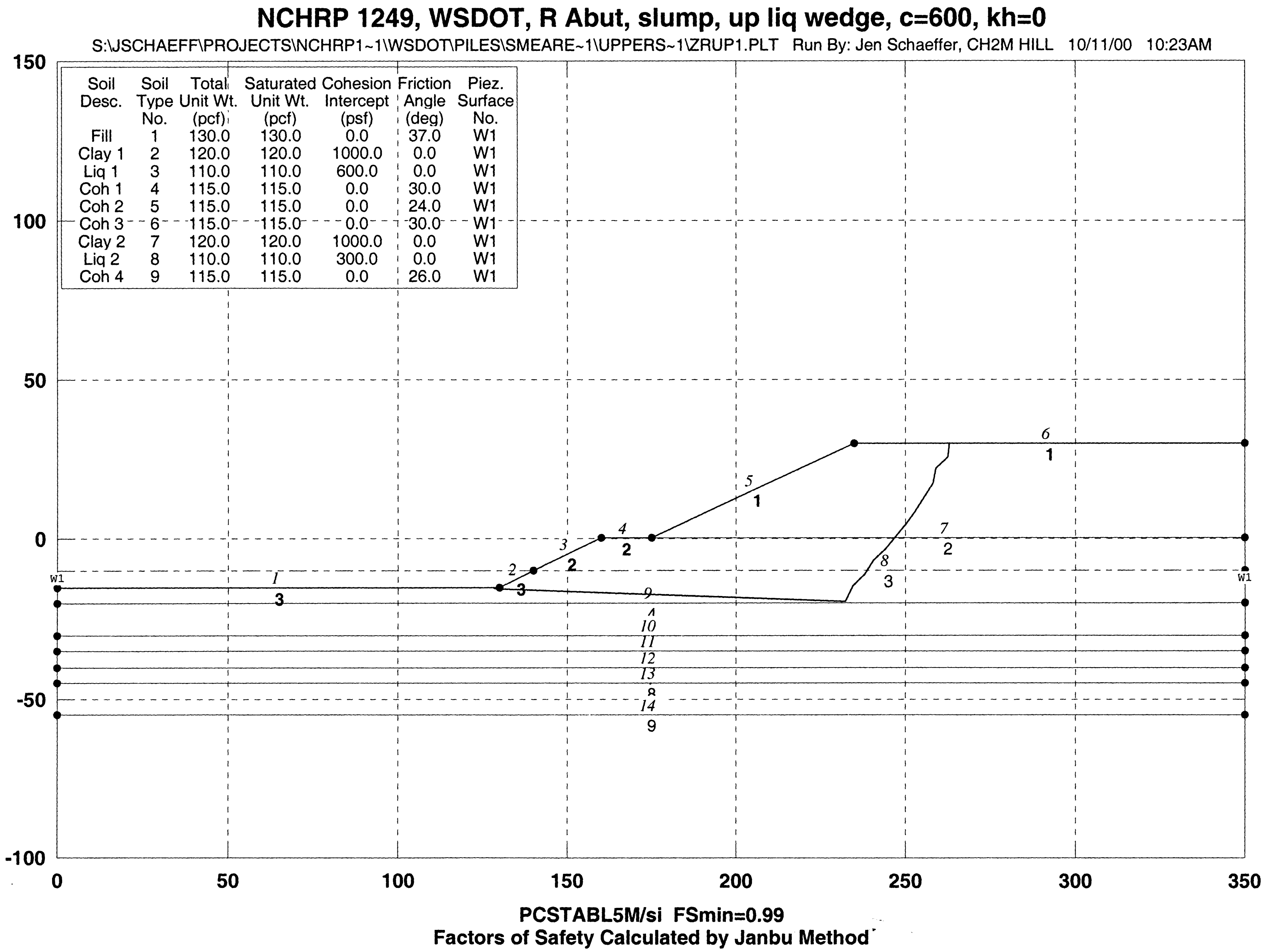
#	FS
a	**1.22**
b	1.22
c	1.24
d	1.25
e	1.25
f	1.26
g	1.27
h	1.27
i	1.28
j	1.28

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	500.0	0.0	W1

PCSTABL5M/si FSmin=1.22

Safety Factors Are Calculated By The Modified Janbu Method

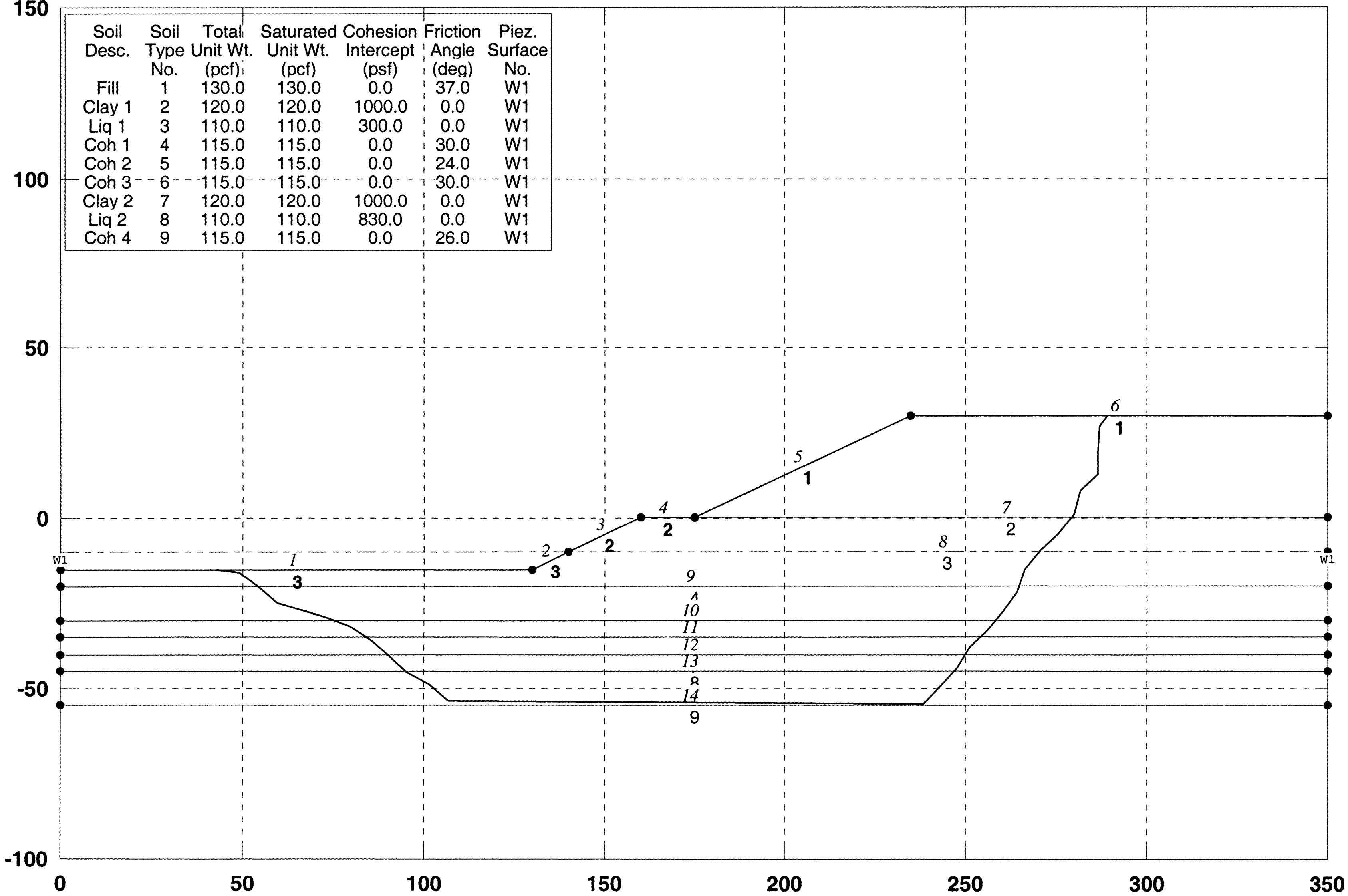
B.3 Washington Stability Analyses —Pile/Structure Pinning Effects



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	600.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

NCHRP 1249, WSDOT, R Abut, lower wedge pin, kh=0, J=0, #8 c=830 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PILES\SMEARE~1\PINNED~1\XRLWP2.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 11:03AM

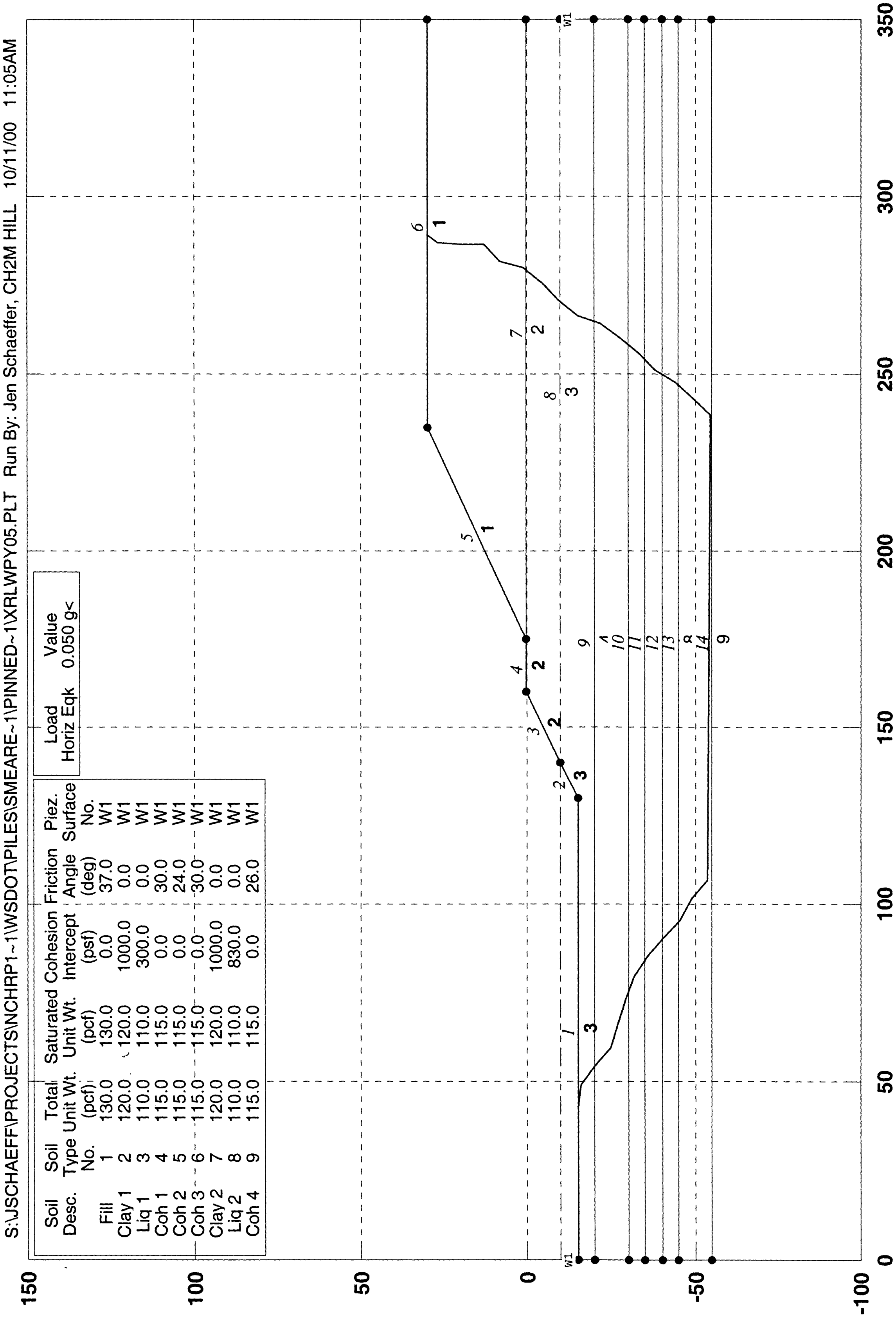


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	830.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=1.06
Factors of Safety Calculated by Janbu Method

NCHRP 1249, WSDOT, R Abut, lower wedge pin, kh=0.05, J=0, #8 c=830 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PILES\SMEARE~1\PINNED~1\XRLWPY05.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 11:05AM



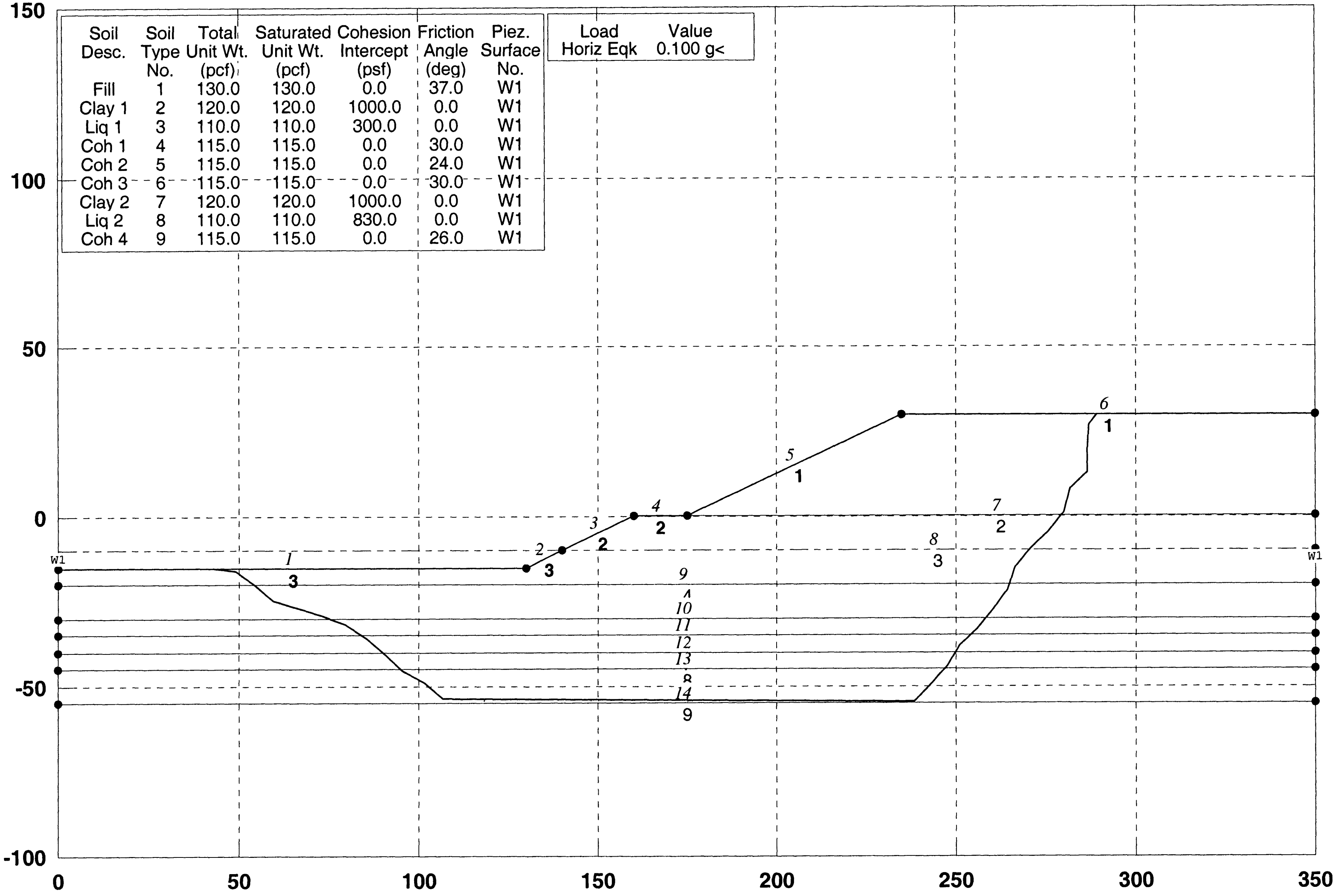
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	830.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.050 g<

PCSTABL5M/si FSmin=0.87
Factors of Safety Calculated by Janbu Method

NCHRP 1249, WSDOT, R Abut, lower wedge pin, kh=0.1, J=0, #8 c=830 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\PILES\SMEARE~1\PINNED~1\XRLWPY1.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 11:04AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	300.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	830.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

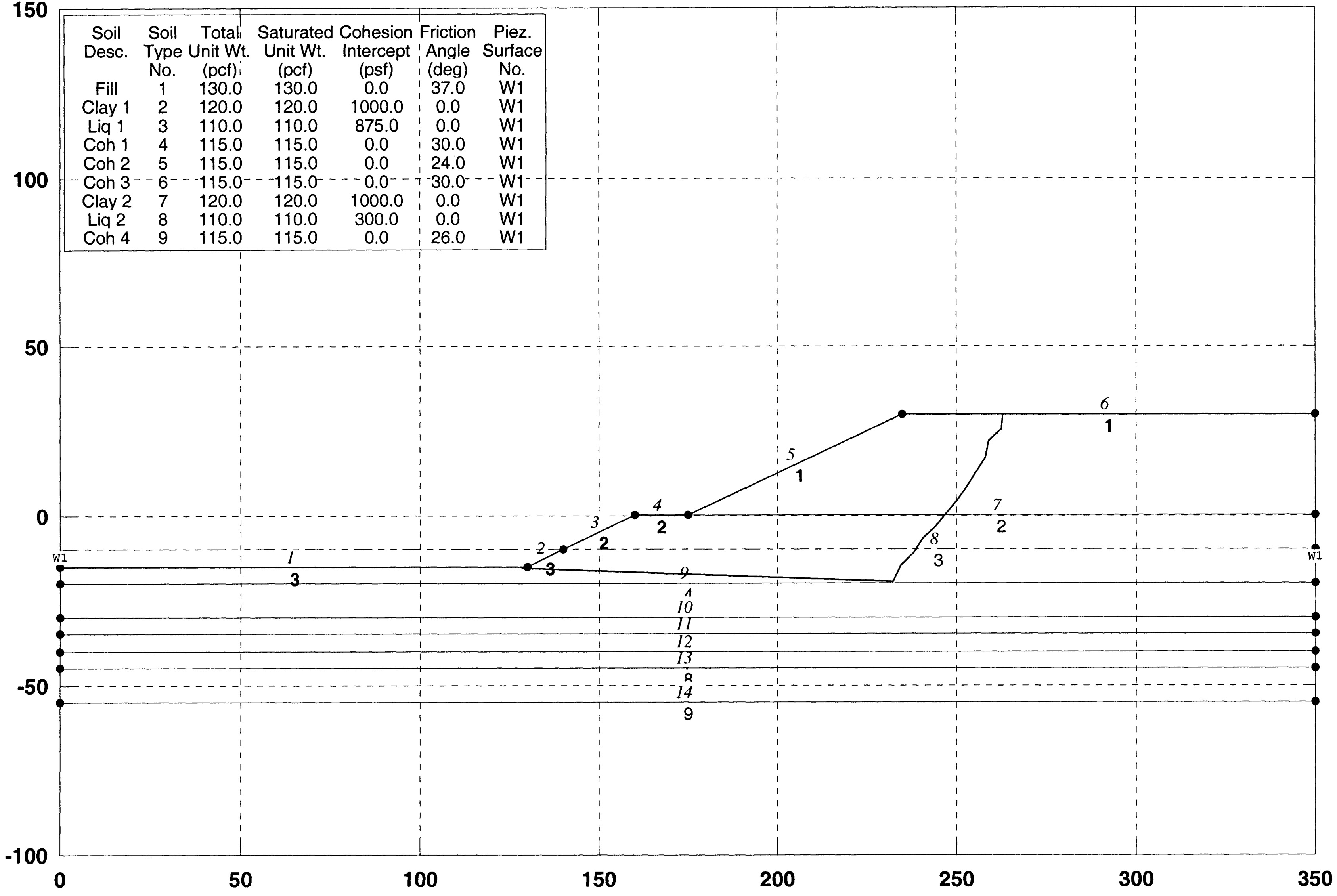
Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=0.74
Factors of Safety Calculated by Janbu Method

B.4 Washington Stability Analyses — Ground Improvement Effects

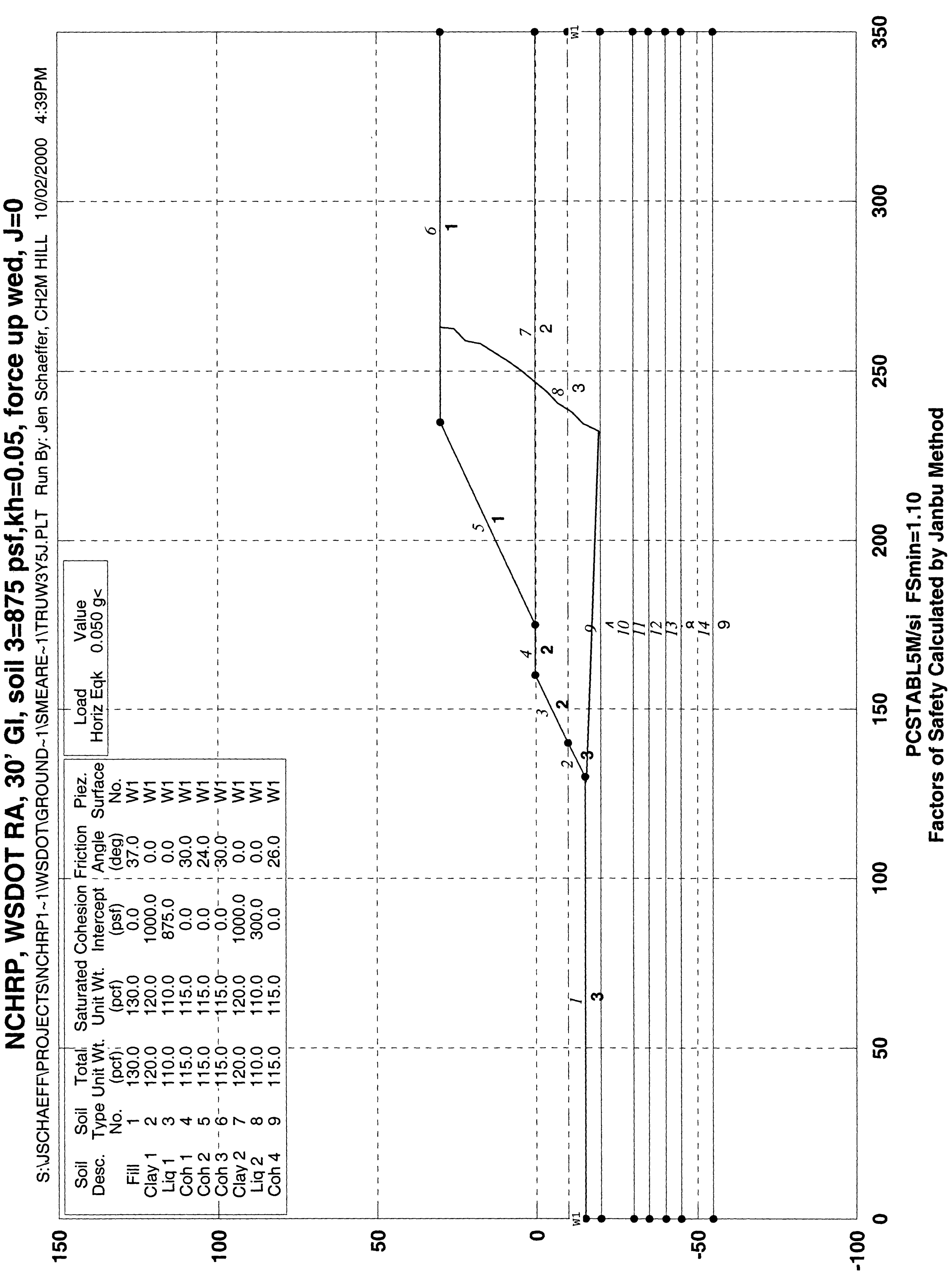
NCHRP, WSDOT RA, 30' GI, soil 3=875 psf,kh=0, force up wed, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW30J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:38PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	875.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=1.28
Factors of Safety Calculated by Janbu Method

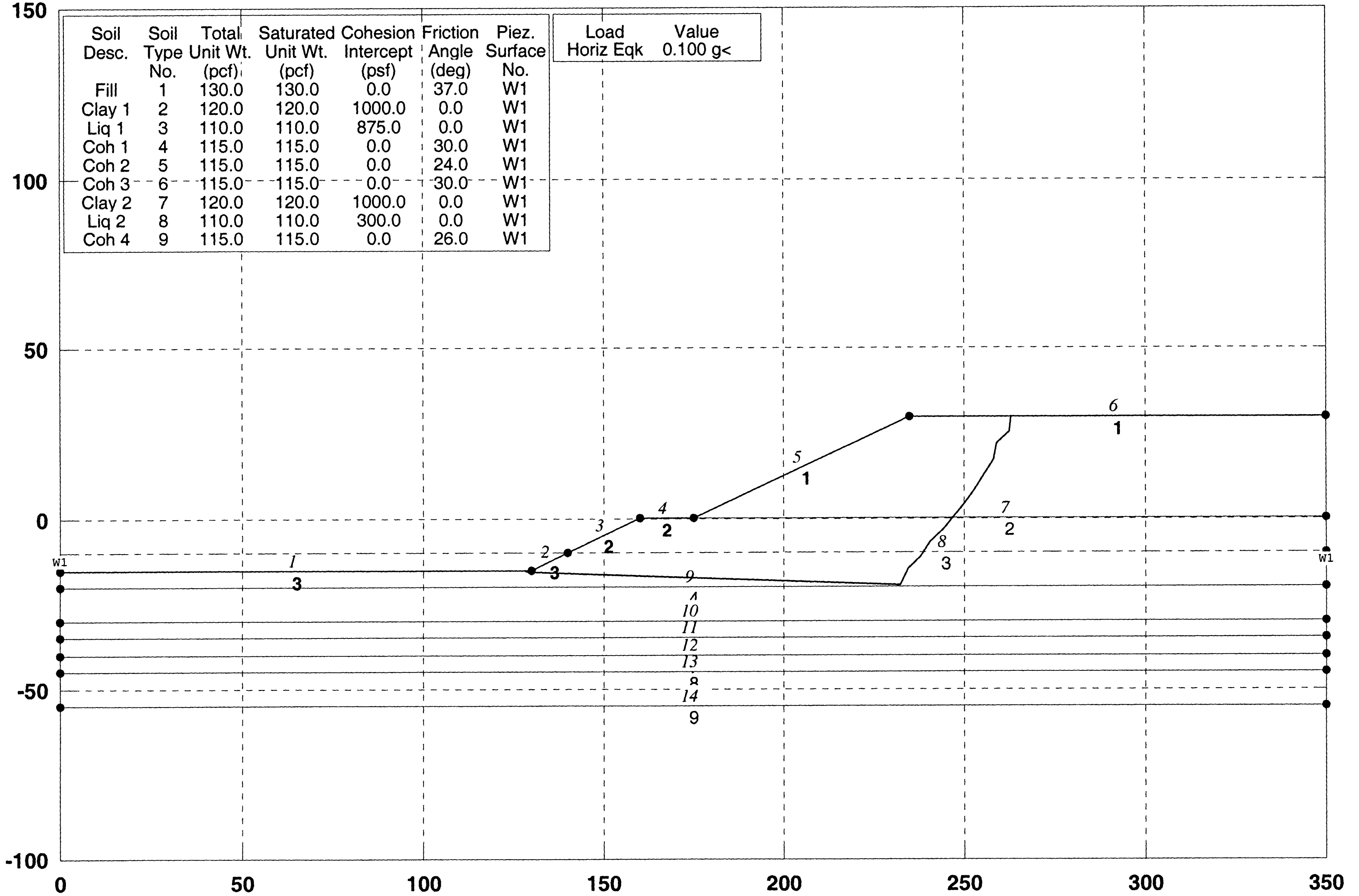


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	875.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.050 g<

NCHRP, WSDOT RA, 30' GI, soil 3=875 psf,kh=0.1, force up wed, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW3Y1J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:40PM



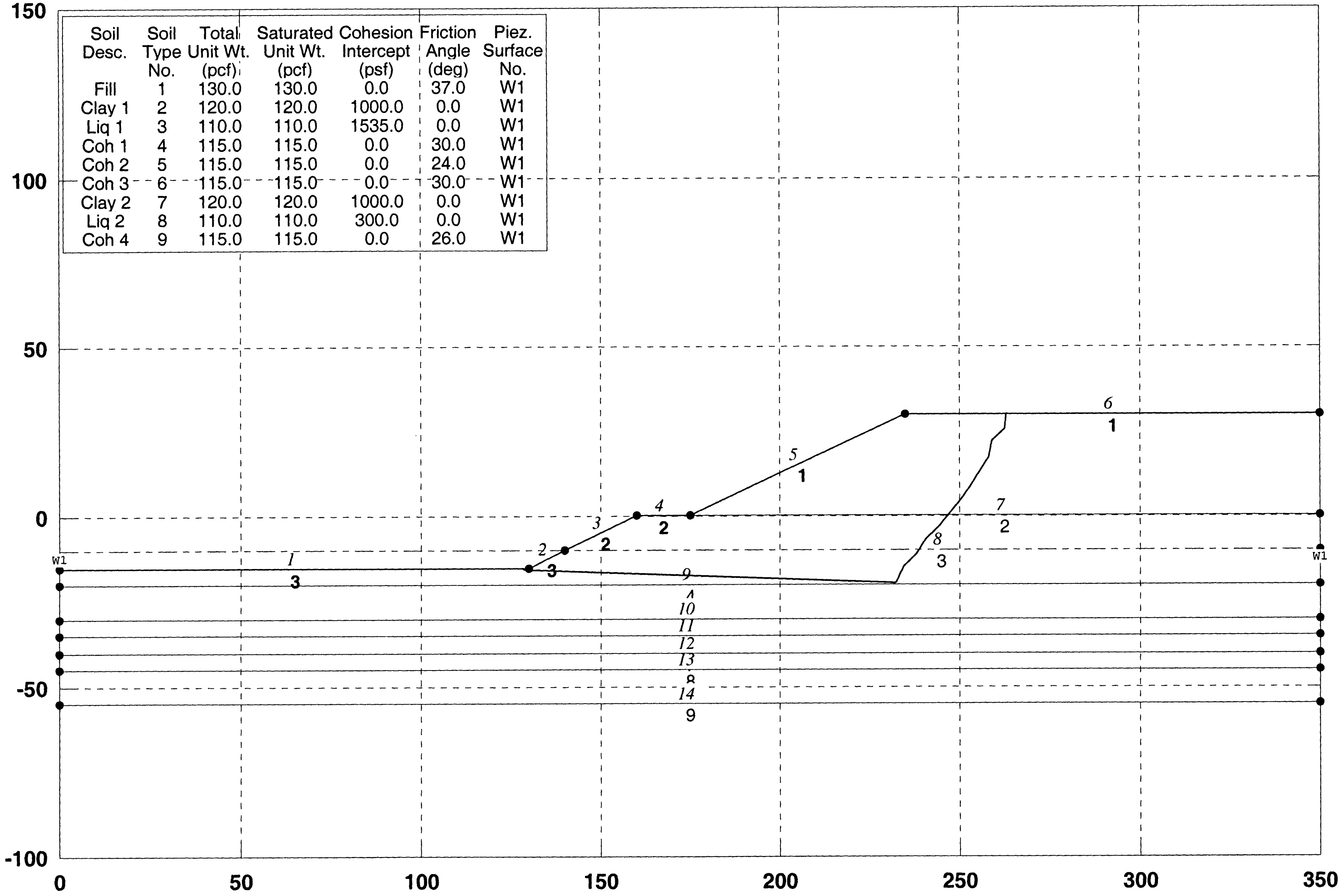
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	875.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=0.96
Factors of Safety Calculated by Janbu Method

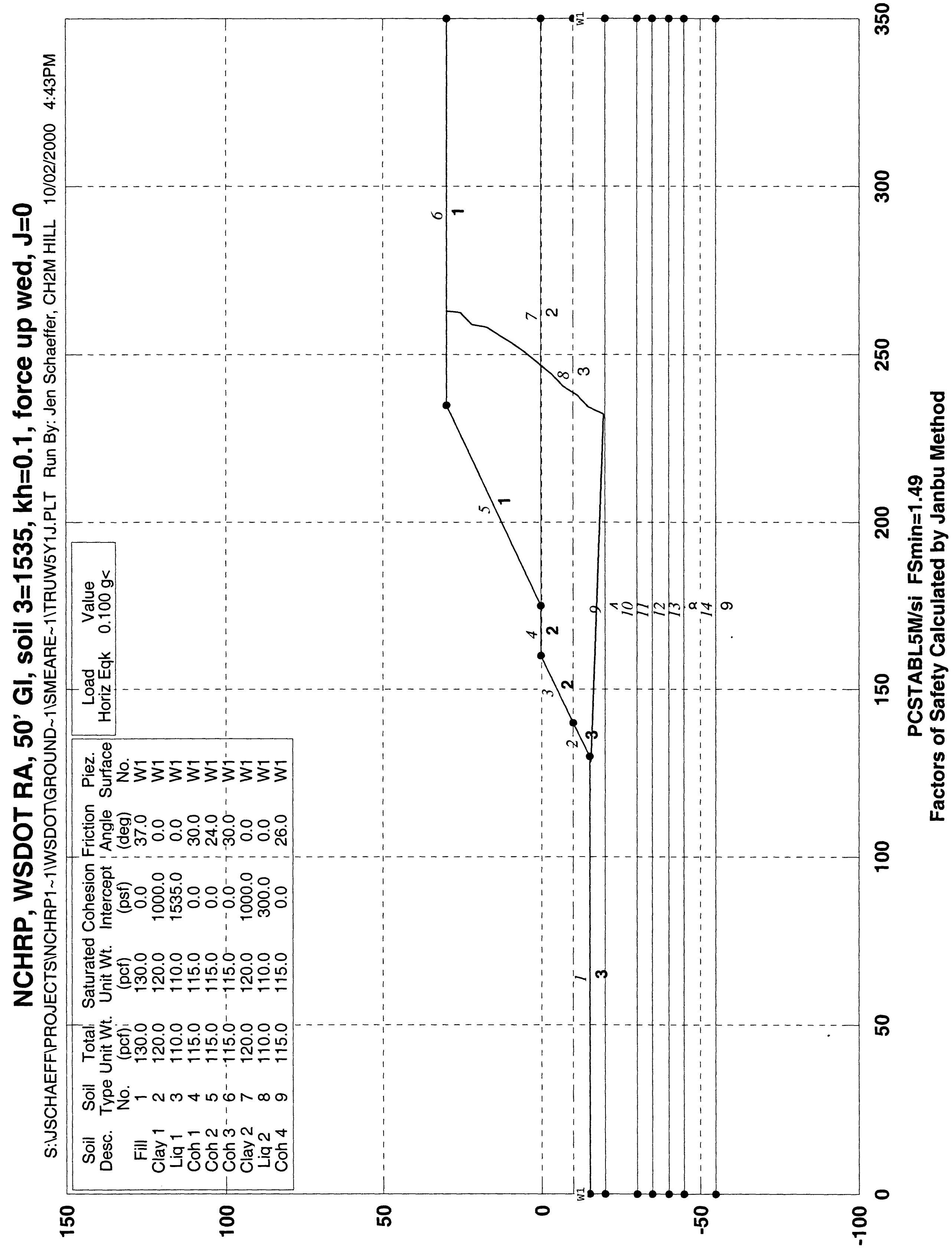
NCHRP, WSDOT RA, 50' GI, soil 3=1535, kh=0, force up wed, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW50J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:42PM

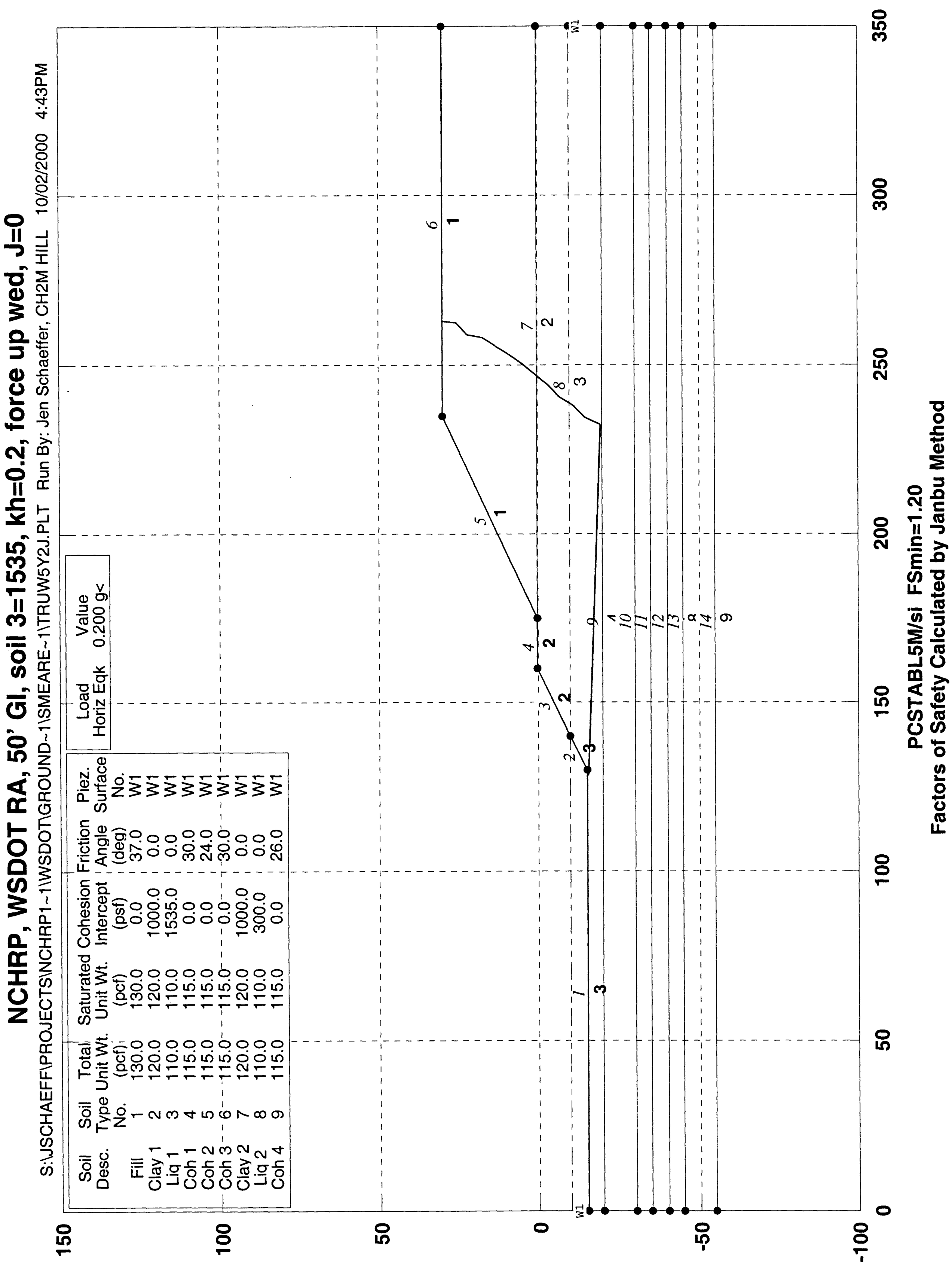


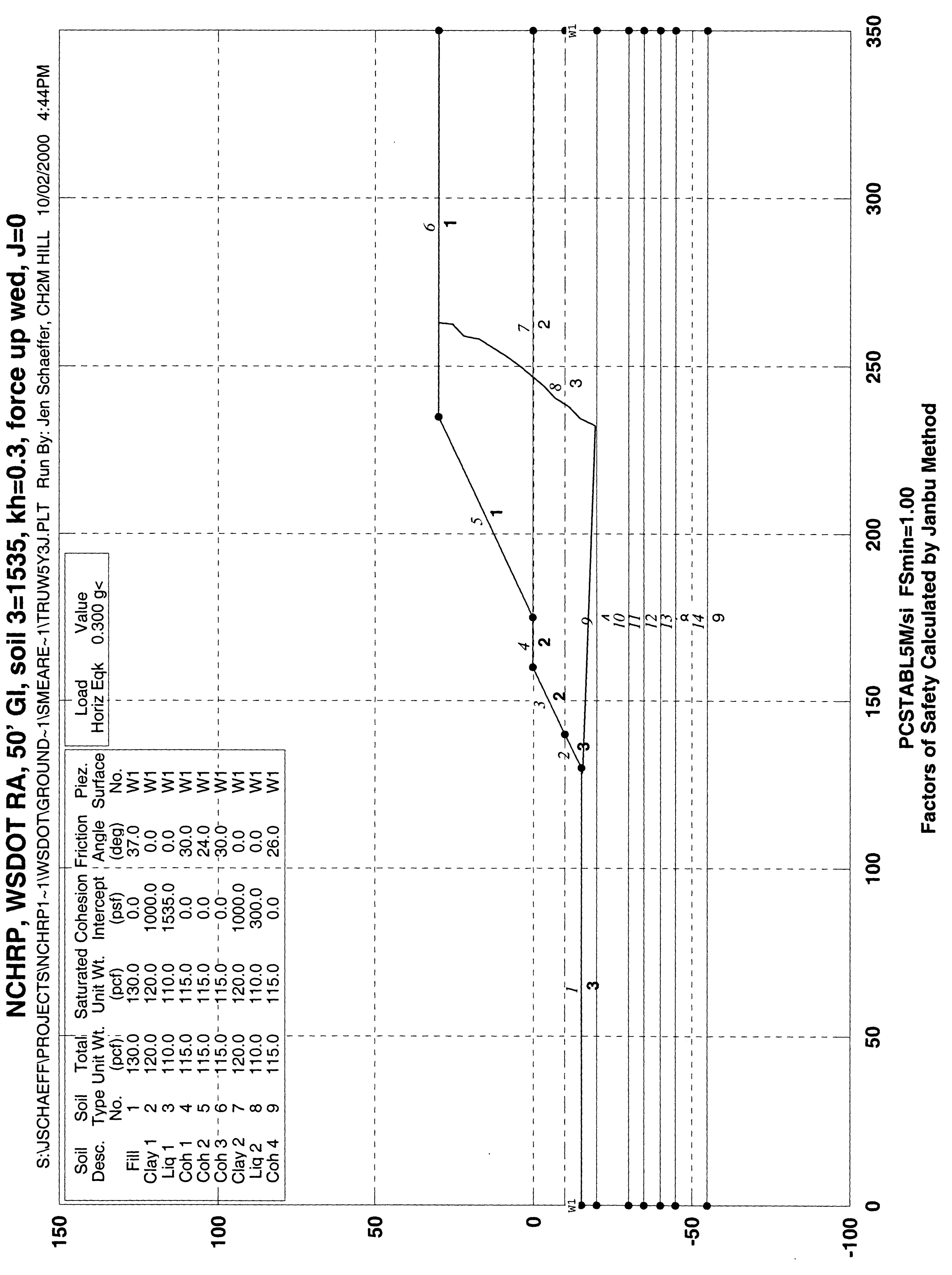
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	1535.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=1.94
Factors of Safety Calculated by Janbu Method



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	1535.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1


NCHRP, WSDOT RA, 50' GI, soil 3=1535, kh=0.2, force up wed, J=0
S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW5Y2J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:43PM
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Fill 1 130.0 130.0 0.0 37.0 W1
Clay 1 2 120.0 120.0 1000.0 0.0 W1
Liq 1 3 110.0 110.0 1535.0 0.0 W1
Coh 1 4 115.0 115.0 0.0 30.0 W1
Coh 2 5 115.0 115.0 0.0 24.0 W1
Coh 3 6 115.0 115.0 0.0 30.0 W1
Clay 2 7 120.0 120.0 1000.0 0.0 W1
Liq 2 8 110.0 110.0 300.0 0.0 W1
Coh 4 9 115.0 115.0 0.0 26.0 W1
Load Value
Horiz Eqk 0.200 g<
150
100
50
0
-50
-100
0
50
100
150
200
250
300
350
W1
1
2
3
4
5
6
7
8
9
10
11
12
13
14
PCSTABL5M/si FSmin=1.20
Factors of Safety Calculated by Janbu Method



NCHRP, WSDOT RA, 50' GI, soil 3=1535, kh=0.3, force up wed, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW5Y3J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:44PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	1535.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

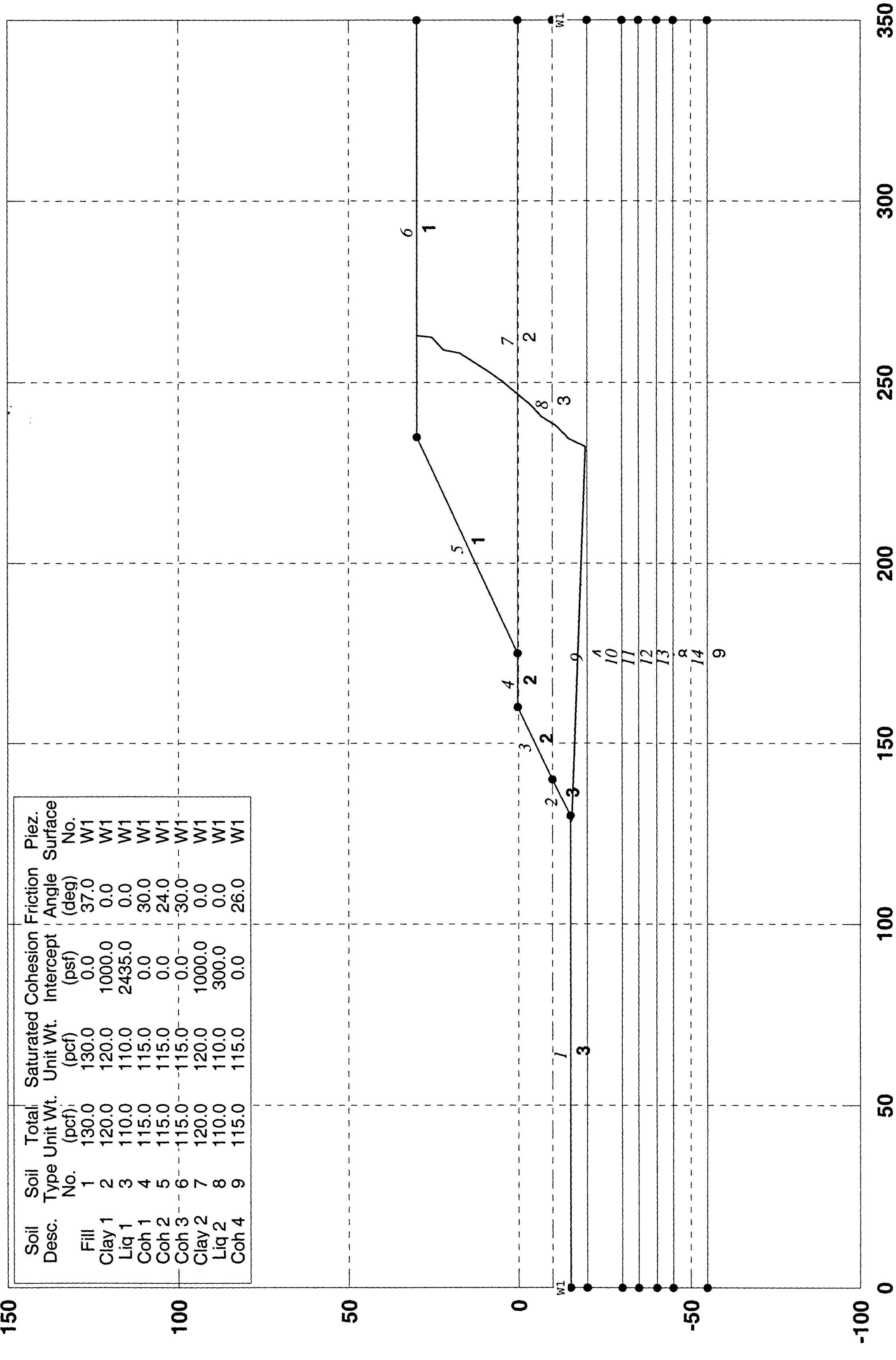
Load	Value
Horiz Eqk	0.300 g<

PCSTABL5M/si FSmin=1.00

Factors of Safety Calculated by Janbu Method

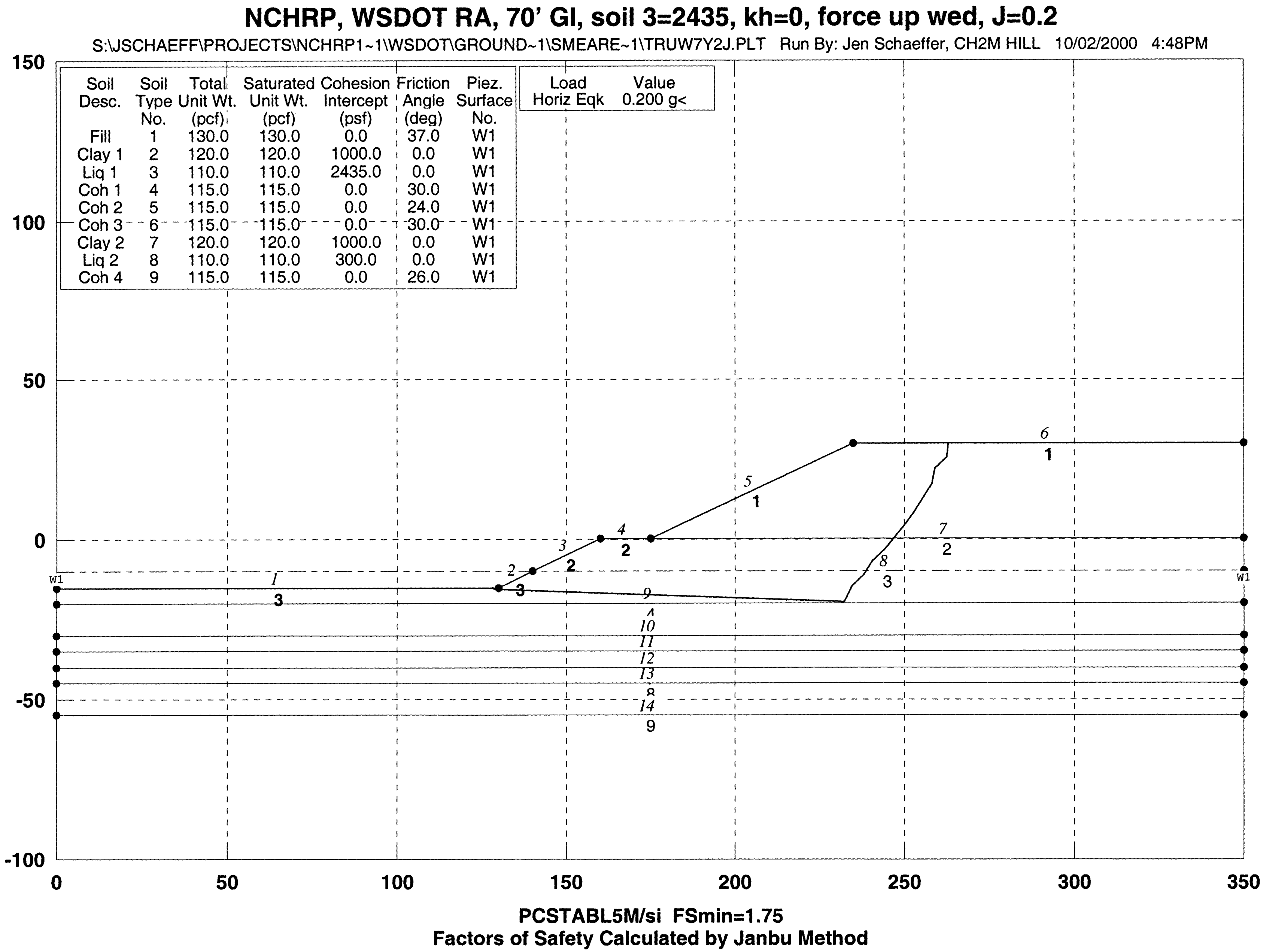
NCHRP, WSDOT RA, 70' Gl, soil 3=2435, kh=0, force up wed, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW70J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:47PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	2435.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

PCSTABL5M/si FSmin=2.81
Factors of Safety Calculated by Janbu Method

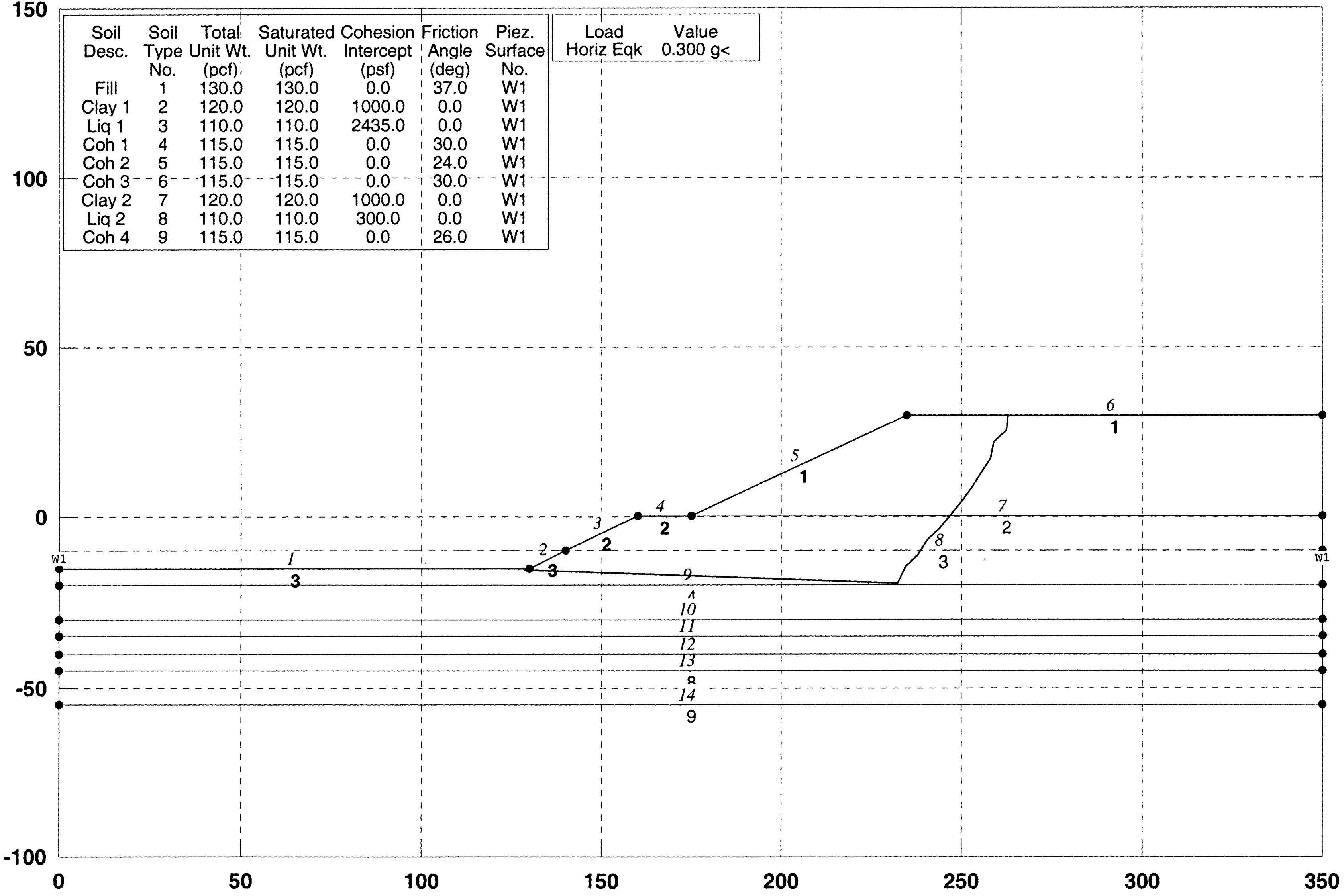


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	2435.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.200 g<

NCHRP, WSDOT RA, 70' GI, soil 3=2435, kh=0, force up wed, J=0.3

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW7Y3J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:49PM



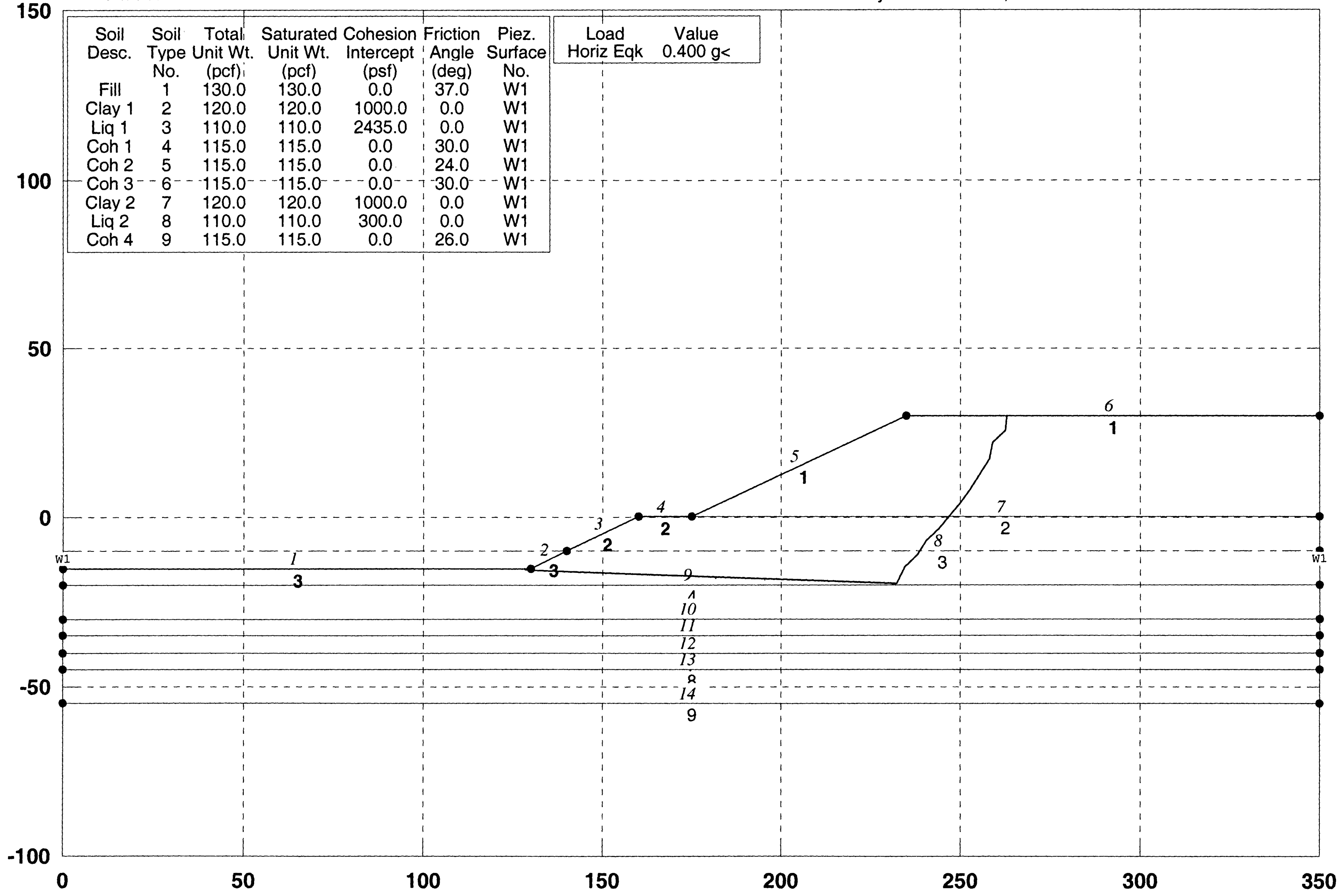
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	2435.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.300 g<

PCSTABL5M/si FSmin=1.47
Factors of Safety Calculated by Janbu Method

NCHRP, WSDOT RA, 70' GI, soil 3=2435, kh=0, force up wed, J=0.4

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW7Y4J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:49PM

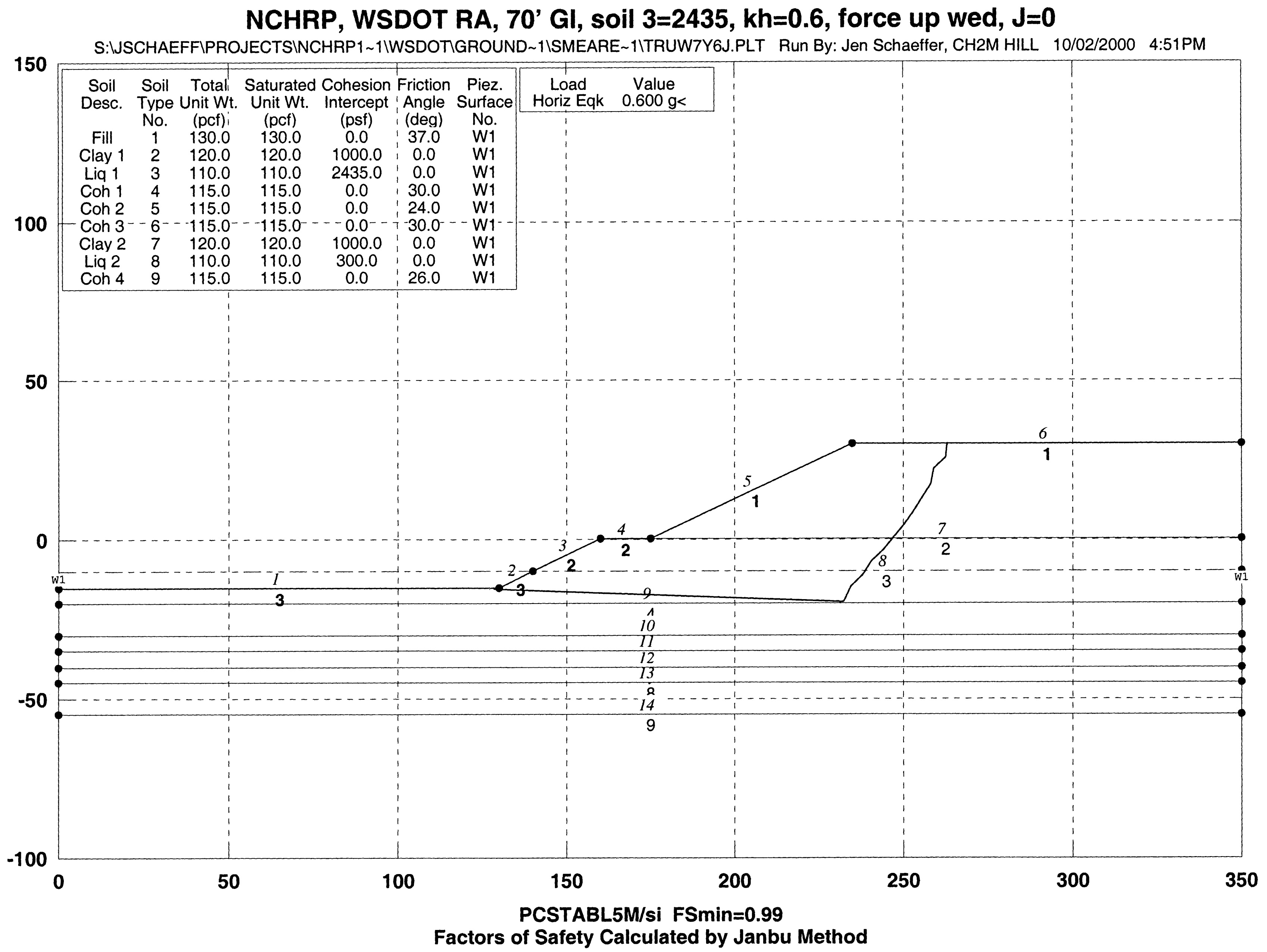


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	2435.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.400 g<

PCSTABL5M/si FSmin=1.27

Factors of Safety Calculated by Janbu Method



NCHRP, WSDOT RA, 70' Gl, soil 3=2435, kh=0.6, force up wed, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW7Y6J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:51PM

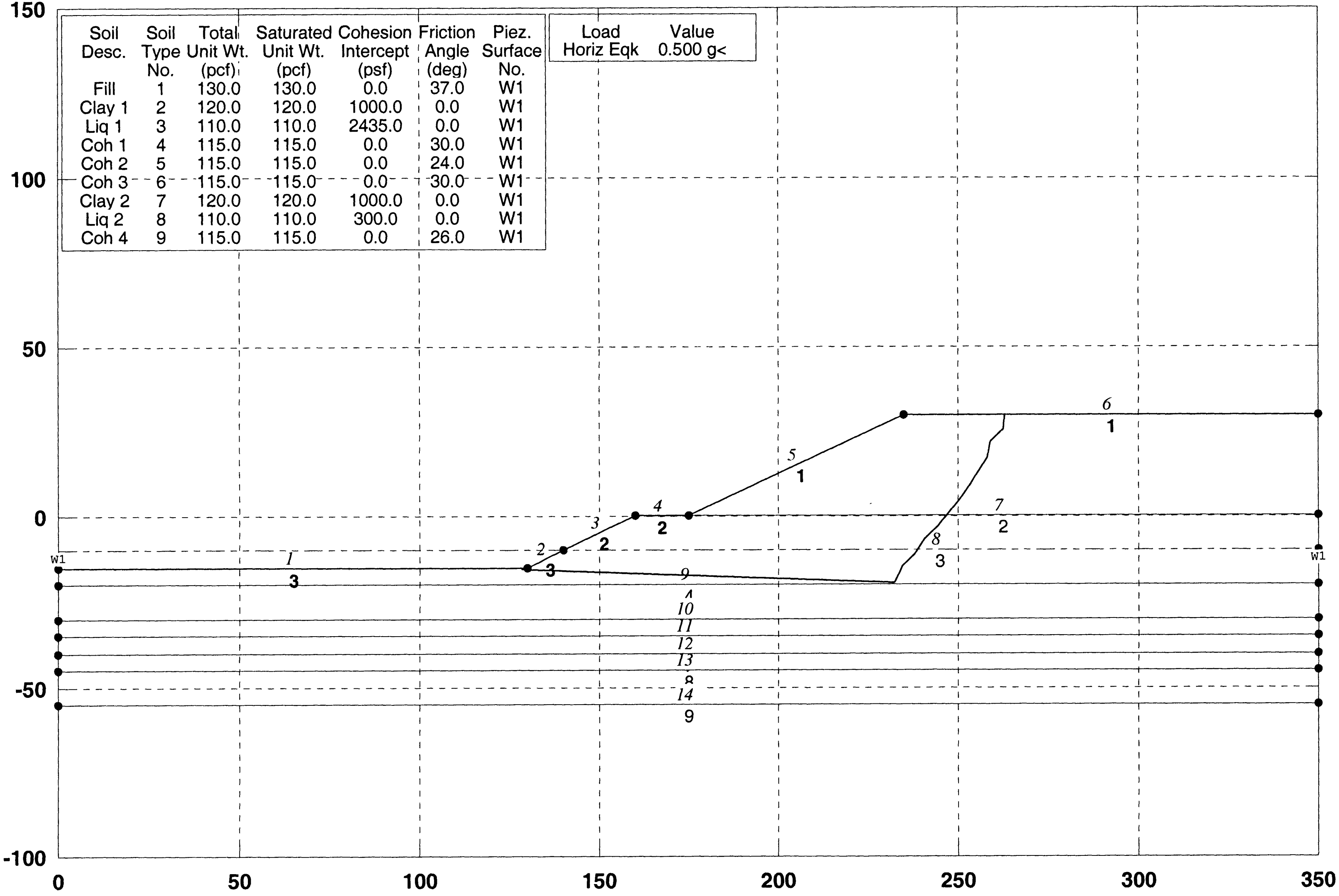
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	2435.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.600 g<

PCSTABL5M/si FSmin=0.99
Factors of Safety Calculated by Janbu Method

NCHRP, WSDOT RA, 70' GI, soil 3=2435, kh=0, force up wed, J=0.5

S:\JSCHAEFF\PROJECTS\NCHRP1~1\WSDOT\GROUND~1\SMEARE~1\TRUW7Y5J.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 4:50PM



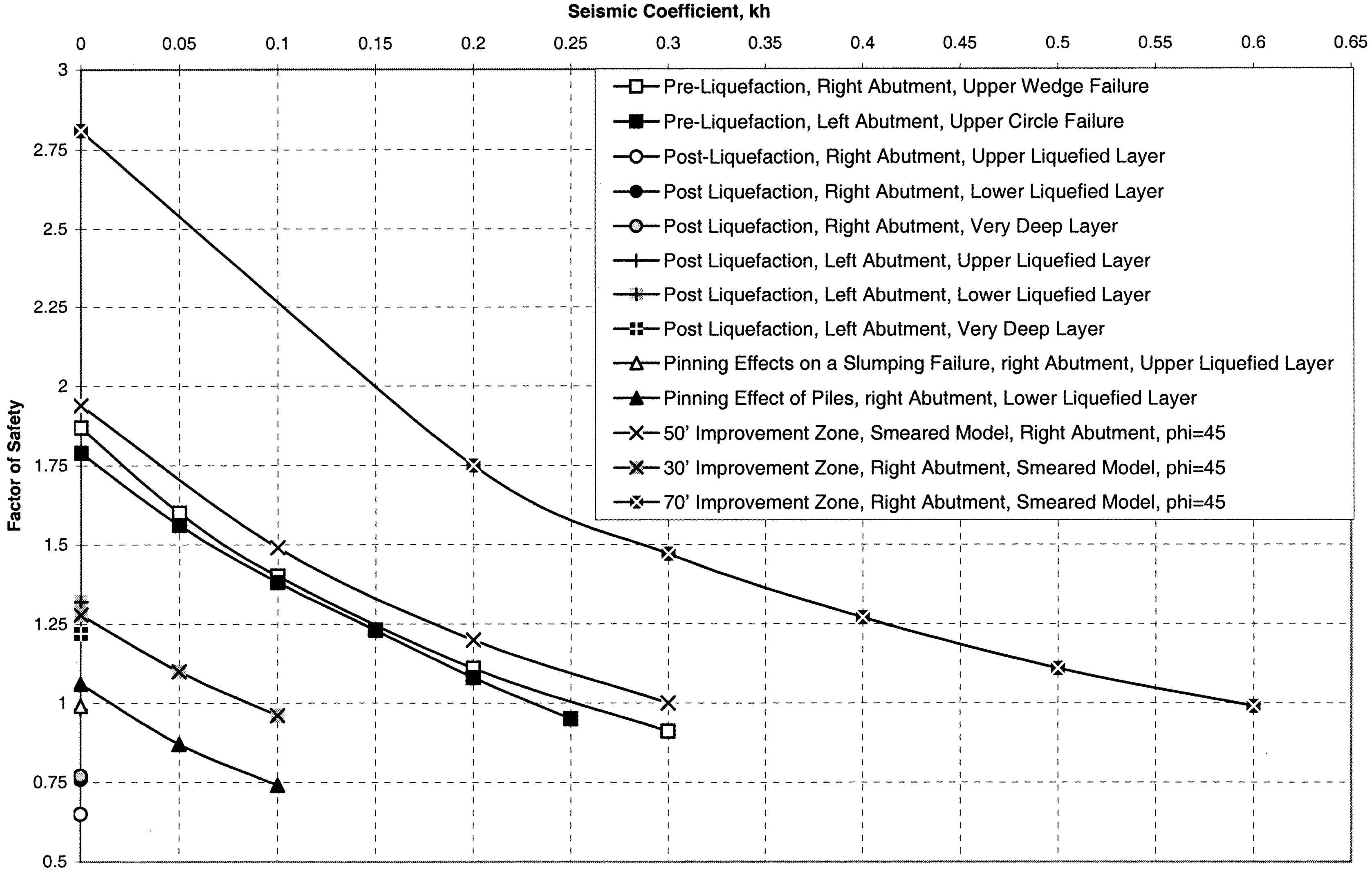
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay 1	2	120.0	120.0	1000.0	0.0	W1
Liq 1	3	110.0	110.0	2435.0	0.0	W1
Coh 1	4	115.0	115.0	0.0	30.0	W1
Coh 2	5	115.0	115.0	0.0	24.0	W1
Coh 3	6	115.0	115.0	0.0	30.0	W1
Clay 2	7	120.0	120.0	1000.0	0.0	W1
Liq 2	8	110.0	110.0	300.0	0.0	W1
Coh 4	9	115.0	115.0	0.0	26.0	W1

Load	Value
Horiz Eqk	0.500 g<

PCSTABL5M/si FSmin=1.11

Factors of Safety Calculated by Janbu Method

B.5 Washington Stability Analyses — Yield Acceleration Comparisons



Yield Acceleration -- WSDOT Location H-13
See Notes for More Information

WSDOT Location - Comparison of Analyses

NCHRP 12-49 Project

The data annotated here are shown in the "Yield Acceleration" Chart

General Notes:

1. Names shown in brackets, e.g. [RAWU0E] refer to the anlaysis file name for reference.
2. Analyses for the "upper" liquefaction layer were generally performed on the same wedge failure surface [RAWU0E]. Other failure surfaces were used for other cases (see analyses results).
3. All wedge analyses performed with Janbu Simplified method, i.e. Janbu Coefficient = 0. Note that this is generally more conservative than a Spencer analysis on the same failure surface.
   Circular analyses performed with Bishop Modified Method.
4. STABL Version 5 with STEDWIN pre/post processor used for all anlayses.

PRE- AND POST LIQUEFACTION CASES

Pre-Liquefaction- Right Abutment

kh	FS
0	1.87
0.05	1.6
0.1	1.4
0.2	1.11
0.3	0.91

Notes

1. Layering is the same as all Post Liquefaction cases (i.e., all other analyses). Refered to as "D.A." layering in analyses.
2. Pre-liquefaction strength in liquefied layer is phi=30 for upper layer. Note there is also a lower liquefied layer, not examined in these analyses.
3. [PRAEW0, 05, 1, 2, 3]

Pre-Liquefaction- Left Abutment

kh	FS
0	1.79
0.05	1.56
0.1	1.38
0.15	1.23
0.2	1.08
0.25	0.95

Notes

1. Results reported here are for general circle failures (Bishop Modified Method). Because critical surfaces are shallow failures through the embankment, the most critical circle failure surface for each analyses was chosen for the yield acceleration plot.
2. Pre-liquefaction strength in liquefied layer is phi=30 for the upper layer. Note there is also a lower liquefied layer, not examined in these analyses.
3. [PLAC0, 005, 01, 015, 020, 025]

Post-Liquefaction

Right Abutment

Upper Liquefied Layer

kh	FS
0	0.65

Notes

1. Strength of liquefied layer is c=300 psf (upper layer, soil #3).
2. [RAWU0E]

Lower Liquefied Layer

kh	FS
0	0.76

Notes

1. Strength of liquefied layer is c=300 psf (lower layer, soil #8).
2. [XRLWP1]

Left Abutment

Upper Liquefied Layer

kh	FS
0	0.98

Notes

1. Strength of liquefied layer is c=300 psf (upper layer, soil #3).
2. [LAWU0R]

Lower Liquefied Layer

kh	FS
0	1.32

Notes

1. Strength of liquefied layer is c=300 psf (lower layer, soil #8).
2. [LAWL0R]

Very Deep Liquefaction

Right Abutment

kh	FS
0	0.77

Notes

1. Strength of liquefied layer is c=500 psf (lower layer, soil #9).
2. Results shown for a general Bishop Circle Serach [VDC1]

Left Abutment

kh	FS
0	1.22

Notes

1. Strength of liquefied layer is c=500 psf (lower layer, soil #9).
2. Results shown for a wedge in upper 10' of layer [LVDW1]

PILE CASES

"Pinned" Pile Models

Pinned pile models were analyzed by "smearing" the pile/strut loads over the failure surface.
The total structural load includes the pile shear capacities as well as any strut loading and/or passive resistance from the structure and . abutment soils.

Pinning Effects on a Slumping Failure - Upper Liquefied Layer

kh	FS	Notes
0	0.99	1. Strength determined by:
		a. Pile/Strut contribution = (31 kips/ft)/(105') = 0.3 ksf = 300 psf
		b. Soil contribution = 300 psf (liquefied strength).
		c. COMBINED strength, c = 600 psf for liquefied soil (upper layer).
		2. [ZRUP1]

Pinning Effect of Piles - Lower Liquefied Layer

kh	FS	Notes
0	1.06	1. Strength determined by:
0.05	0.87	a. Pile/Strut contribution = (3340 kips)/(48' x 132') = 0.530 ksf = 530 psf
0.1	0.74	b. Soil contribution = 300 psf (liquefied strength).
		c. COMBINED strength, c = 830 psf for liquefied soil (upper layer).
		2. [XRLWP2, PY05, PY1]

GROUND IMPROVEMENT CASES

Smeared Models

"Smeared" analyses model the strength of the ground improvement zone by smearing it over the failure surface.

50' Improvement Zone, Smeared Model, phi=45

kh	FS	Notes
0	1.94	1. Soil strength determined as a weighted average over the failure surface (105').
0.1	1.49	2. Strength determined by:
0.2	1.2	a. Liquefied soil strength x length in liquefied soil, 300 psf x 55'
0.3	1	b. Improved zone strength x length in improved zone, (sig'v x tan 45) x 50', where improved gamma is used
		c. Total strength = total of above/105' = 1535 psf.
		3. [TRUW50J, 5Y1J, 5Y2J, 5Y3J]

30' Improvement Zone, Smeared Model, phi=45

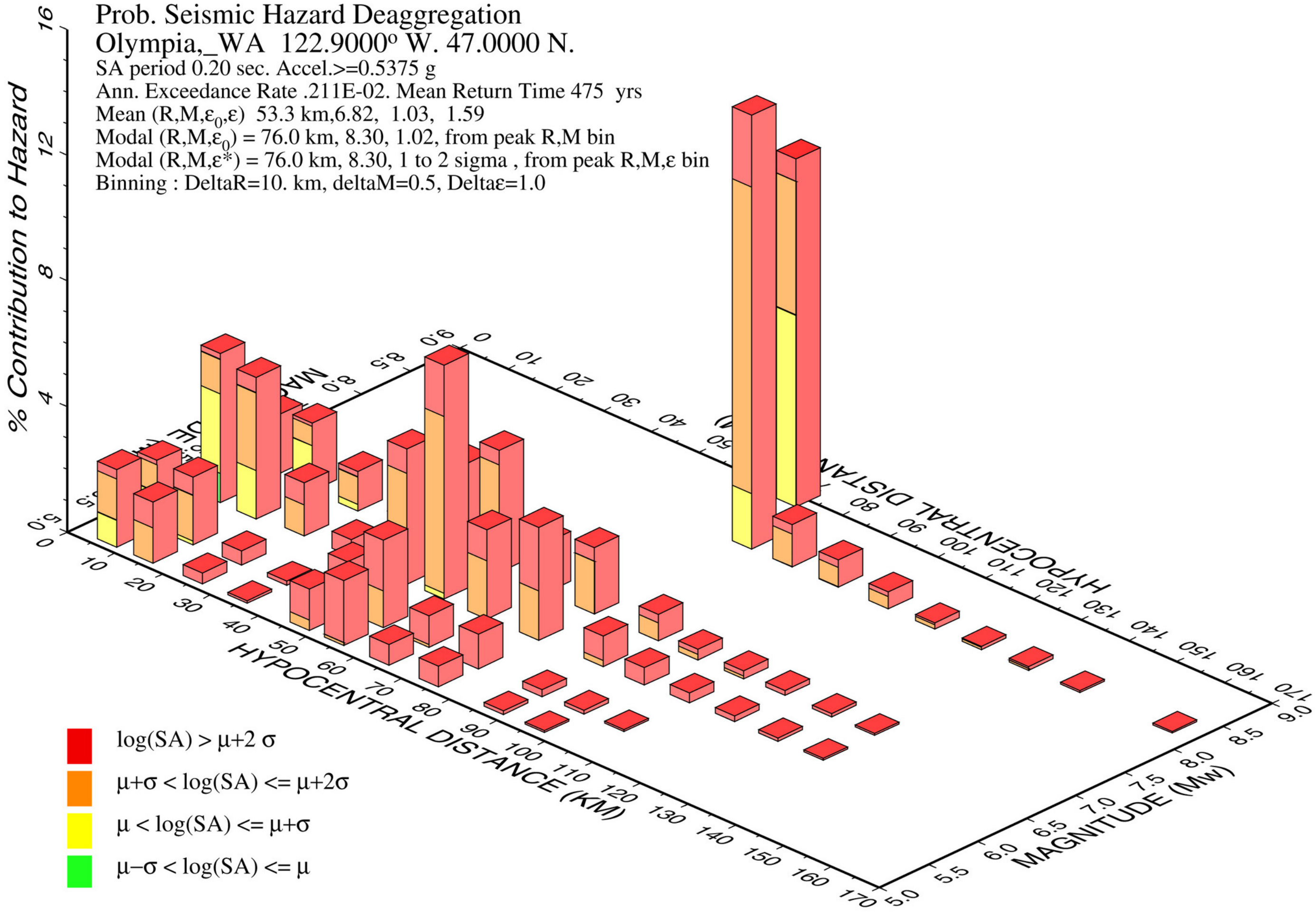
kh	FS	Notes
0	1.28	1. Strength determined by:
0.05	1.1	a. Liquefied soil strength x length in liquefied soil, 300 psf x 75'
0.1	0.96	b. Improved zone strength x length in improved zone, (sig'v x tan 45) x 30', where improved gamma is used
		c. Total strength = total of above/105' = 875 psf
		2. [TRUW30J, 3Y5J, 3Y1J]

70' Improvement Zone, Smeared Model, phi=45

kh	FS	Notes
0	2.81	1. Strength determined by:
0.2	1.75	a. Liquefied soil strength x length in liquefied soil, 300 psf x 35'
0.3	1.47	b. Improved zone strength x length in improved zone,
0.4	1.27	(sig'v x tan 45) x 70', where improved gamma is used
0.5	1.11	c. Total strength = total of above/105' = 2435 psf
0.6	0.99	2. [TRUW70J, 7Y2J, 7Y3J, 7Y4J, 7Y5J, 7Y6J]

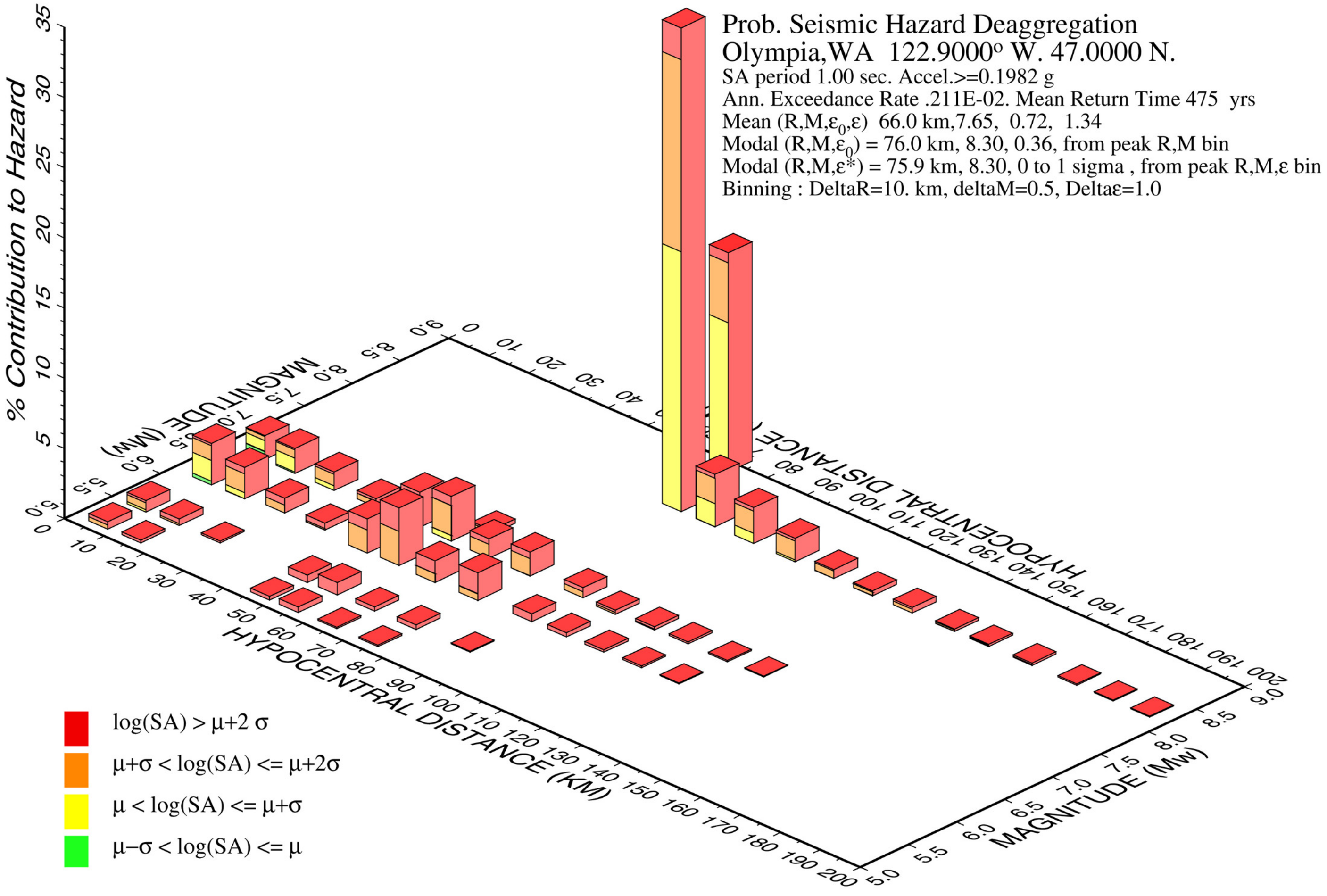
Appendix C

ADDITIONAL DEAGGREGATION PLOTS AND TIME HISTORIES FROM EARTHQUAKE HAZARDS STUDY FOR WASHINGTON SITE



GMT Jun 28 15:16 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

Figure C.1 Hazard Deaggregation, 475-Year Return Period, 0.2-Second Period Spectral Acceleration, Washington Site



GMT Jun 28 15:22 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

Figure C.2 Hazard Deaggregation, 475-Year Return Period, 1.0-Second Period Spectral Acceleration, Washington Site

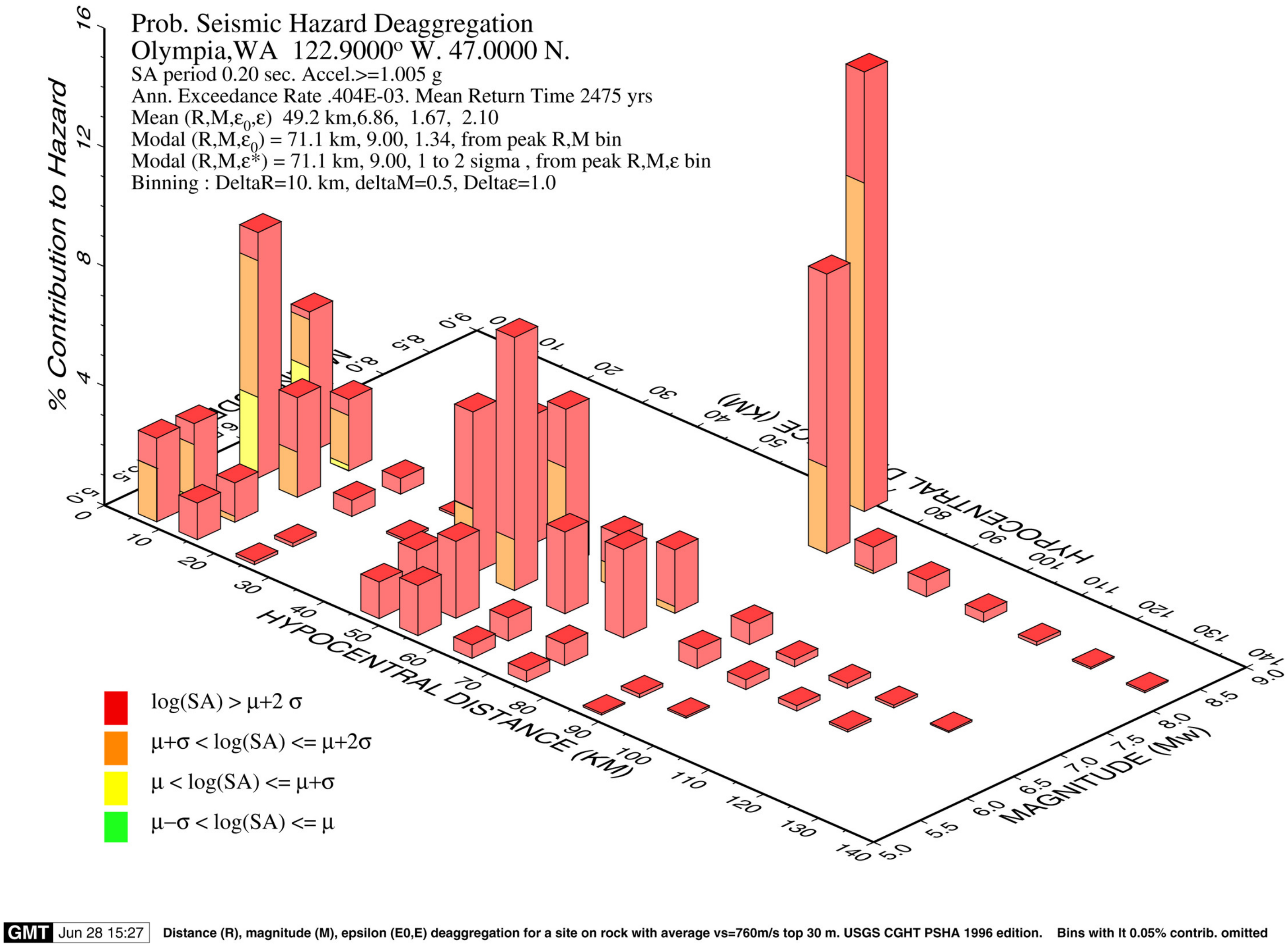
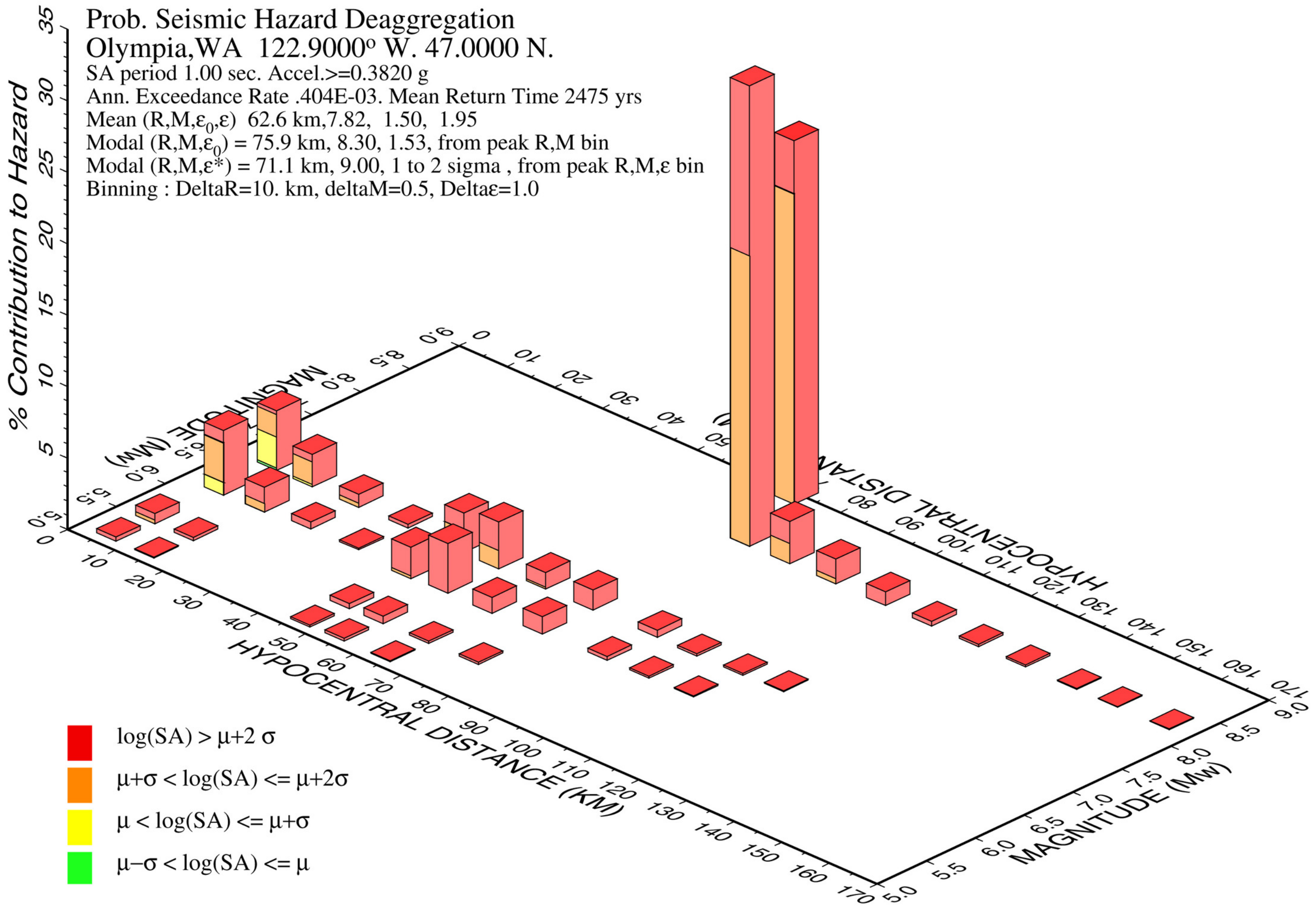


Figure C.3 **Hazard Deaggregation, 2,475-Year Return Period, 0.2-Second Period Spectral Acceleration, Washington Site**



GMT Jun 28 15:32 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

Figure C.4 Hazard Deaggregation, 2,475-Year Return Period, 1.0-Second Period Spectral Acceleration, Washington Site

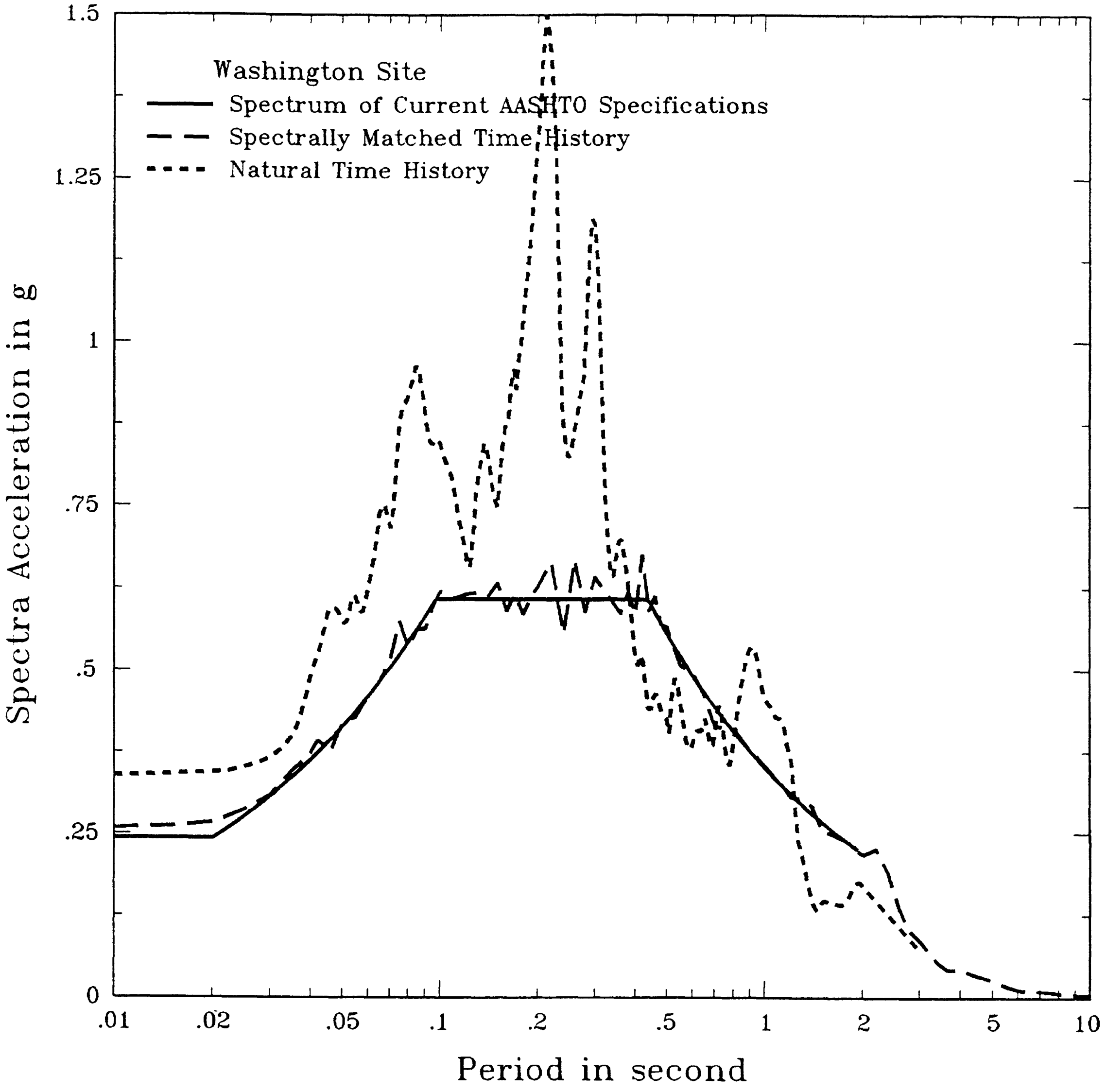


Figure C.5 Comparison Of Response Spectrum Of Scaled Recorded Time History From 1985 Chile Earthquake With Design Response Spectrum Based On Current AASHTO Specifications, Site Class II, Washington Site

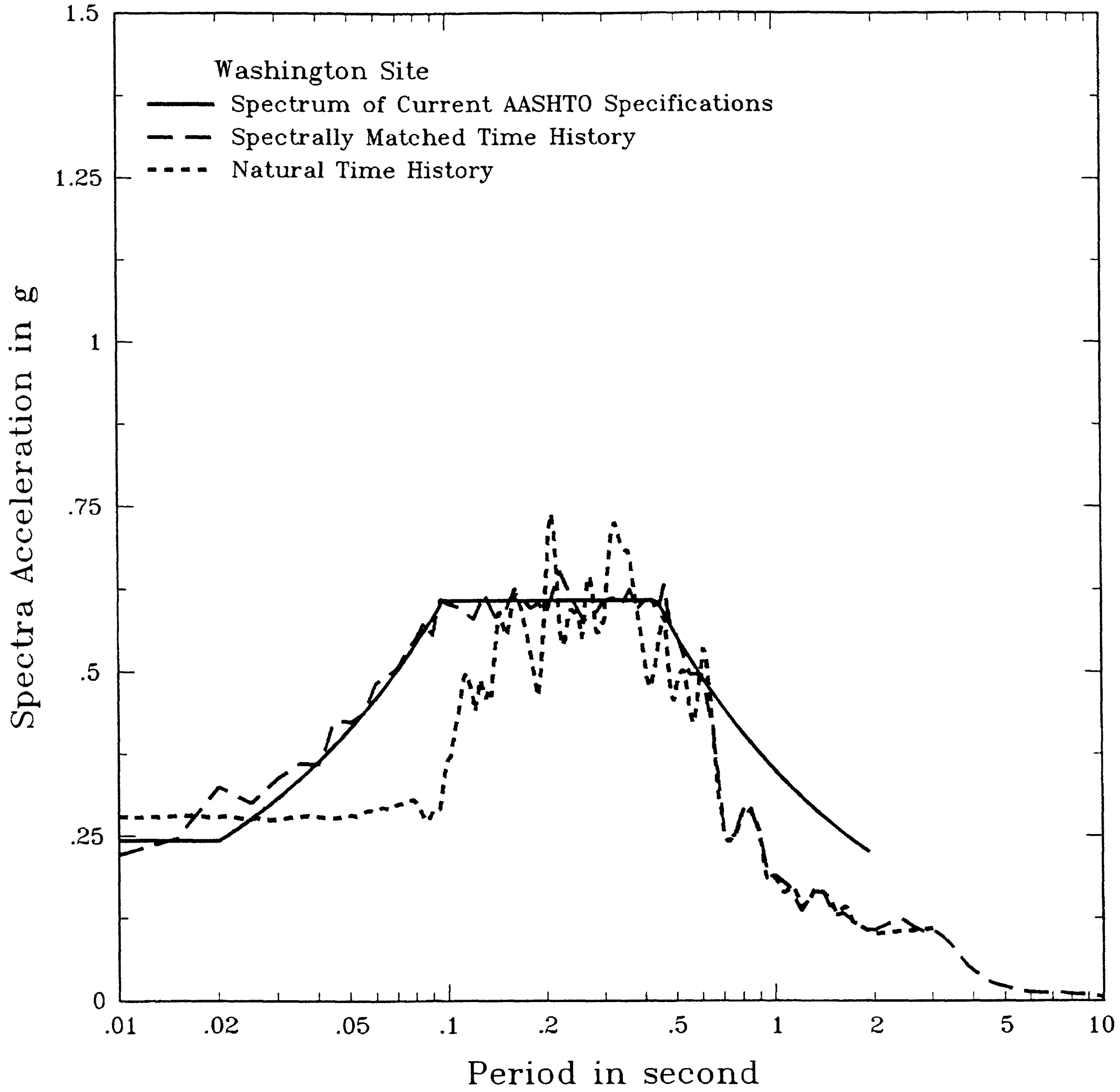


Figure C.6 Comparison Of Response Spectrum Of Scaled Recorded Time History From 1949 Western Washington Earthquake With Design Response Spectrum Based On Current AASHTO Specifications, Site Class II, Washington Site

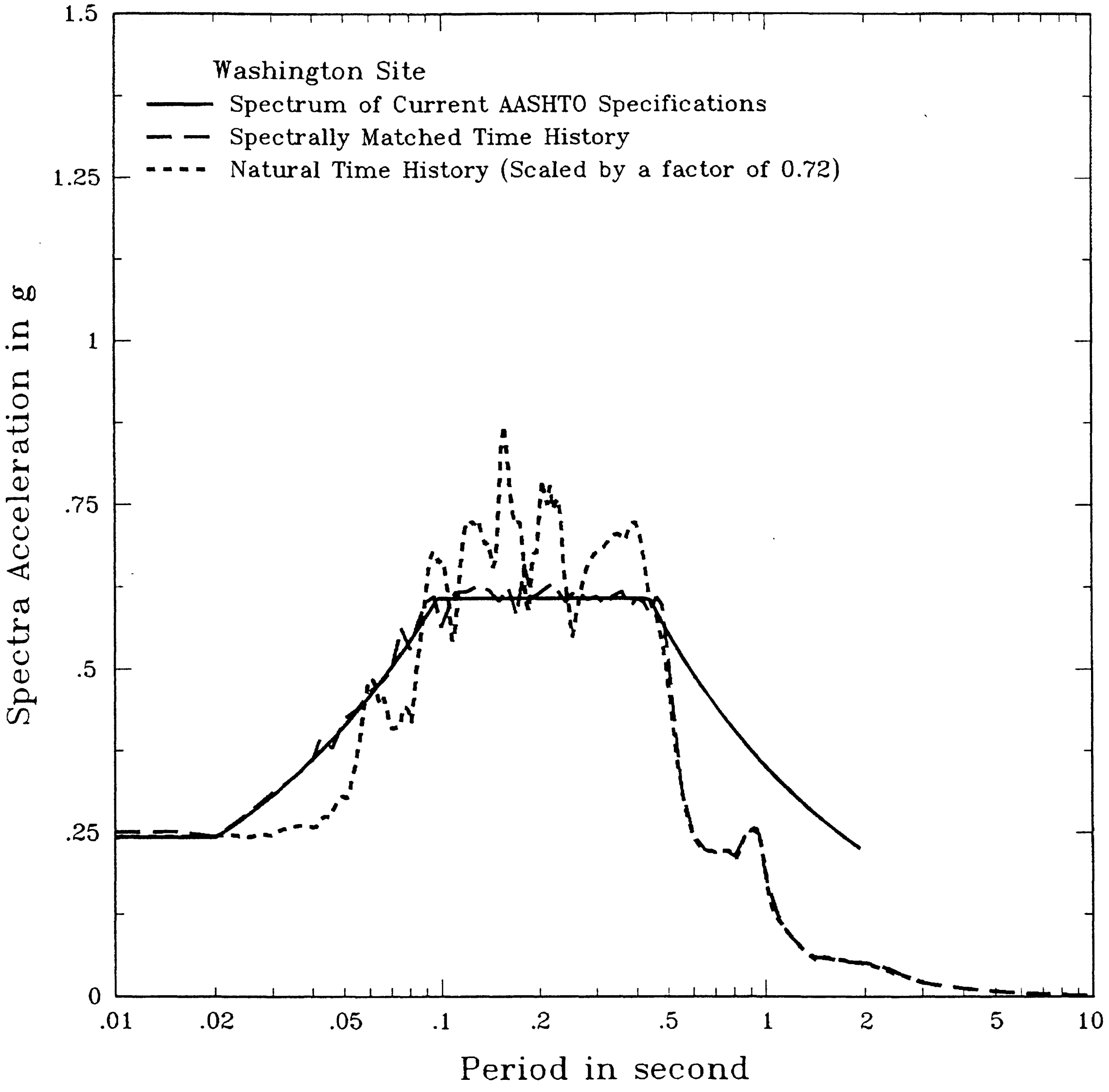


Figure C.7 Comparison Of Response Spectrum Of Scaled Recorded Time History From 1986 North Palm Springs Earthquake With Design Response Spectrum Based On Current AASHTO Specifications, Site Class II, Washington Site

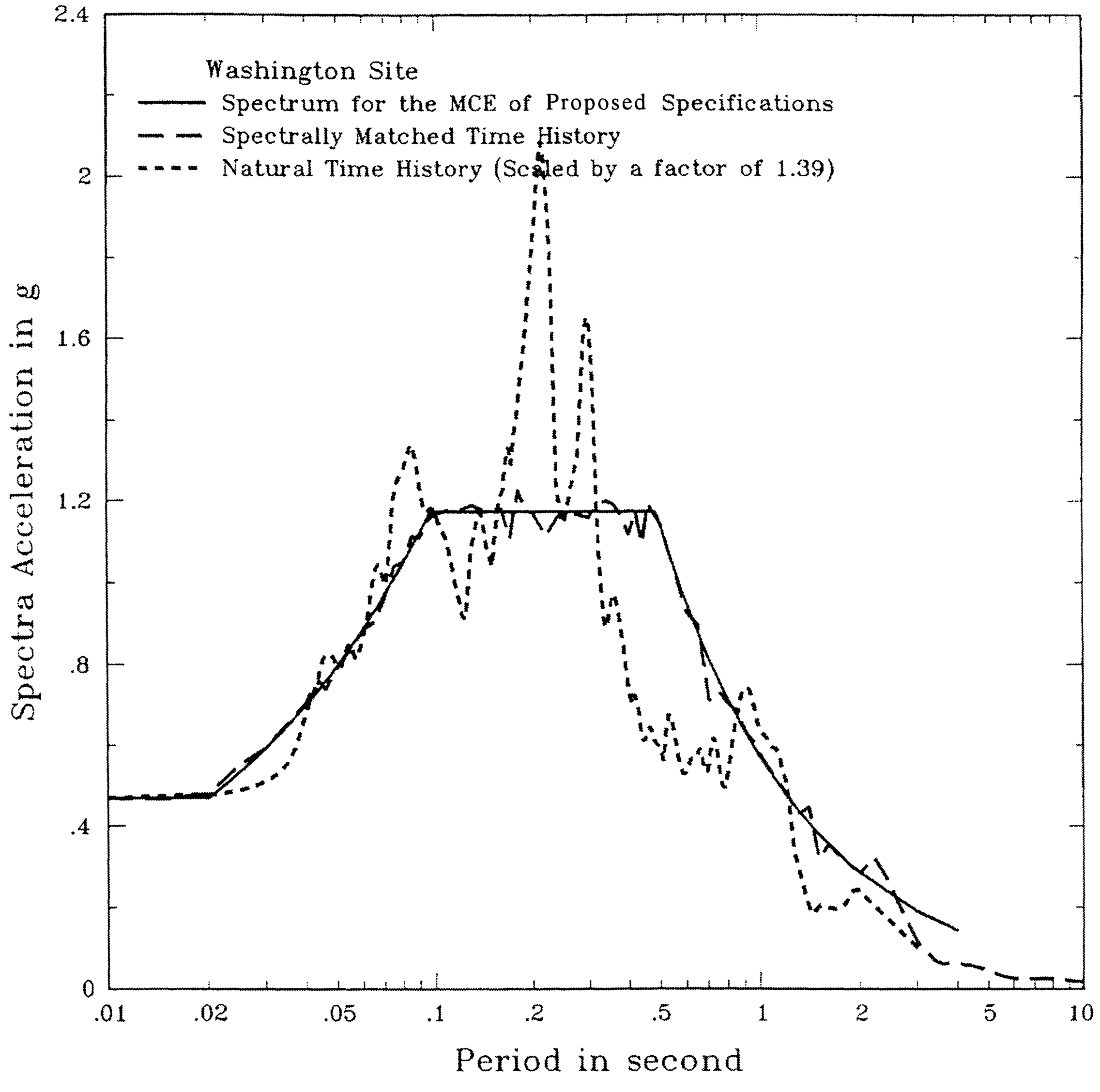


Figure C.8 Comparison Of Response Spectrum Of Scaled Recorded Time History From 1985 Chile Earthquake With MCE Design Response Spectrum Of Proposed LRFD Specifications, Site Class C, Washington Site

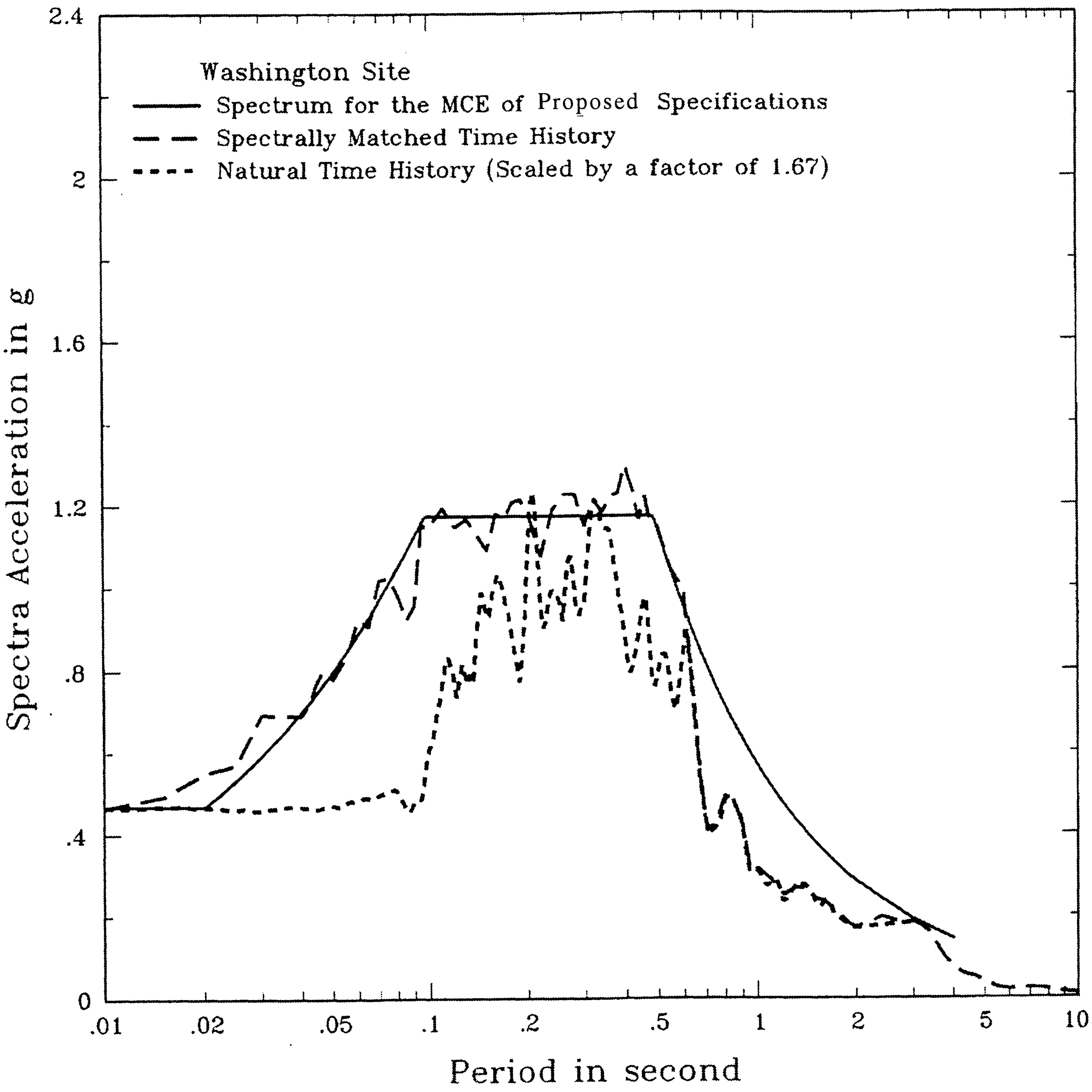


Figure C.9 Comparison Of Response Spectrum Of Scaled Recorded Time History From 1949 Western Washington Earthquake With MCE Design Response Spectrum Of Proposed LRFD Specifications, Site Class C, Washington Site

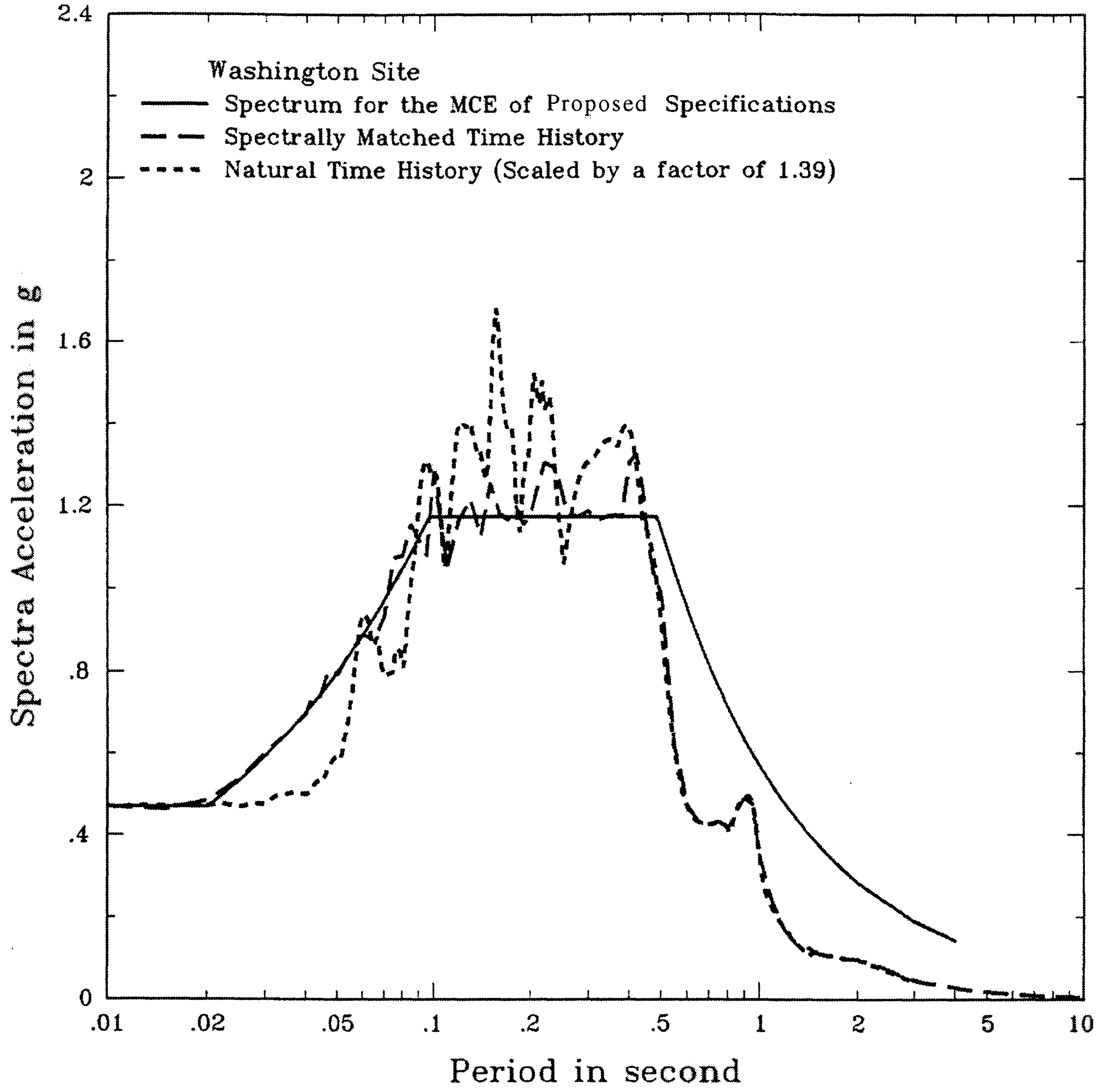


Figure C.10 Comparison Of Response Spectrum Of Scaled Recorded Time History From 1986 North Palm Springs Earthquake With MCE Design Response Spectrum Of Proposed LRFD Specifications, Site Class C, Washington Site

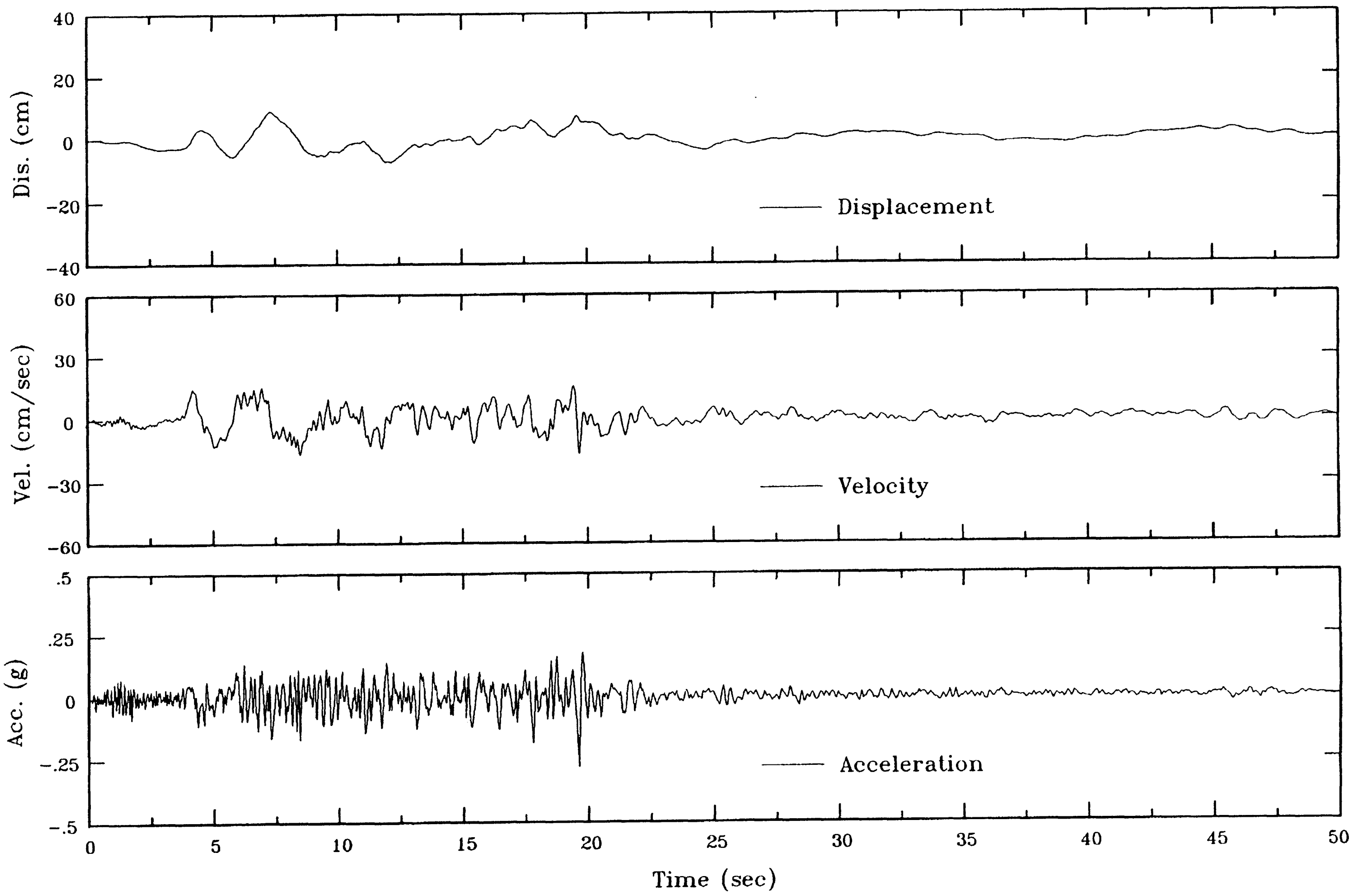


Figure C.11 Acceleration, Velocity, And Displacement Time Histories Of 1949 Western Washington Earthquake Recording

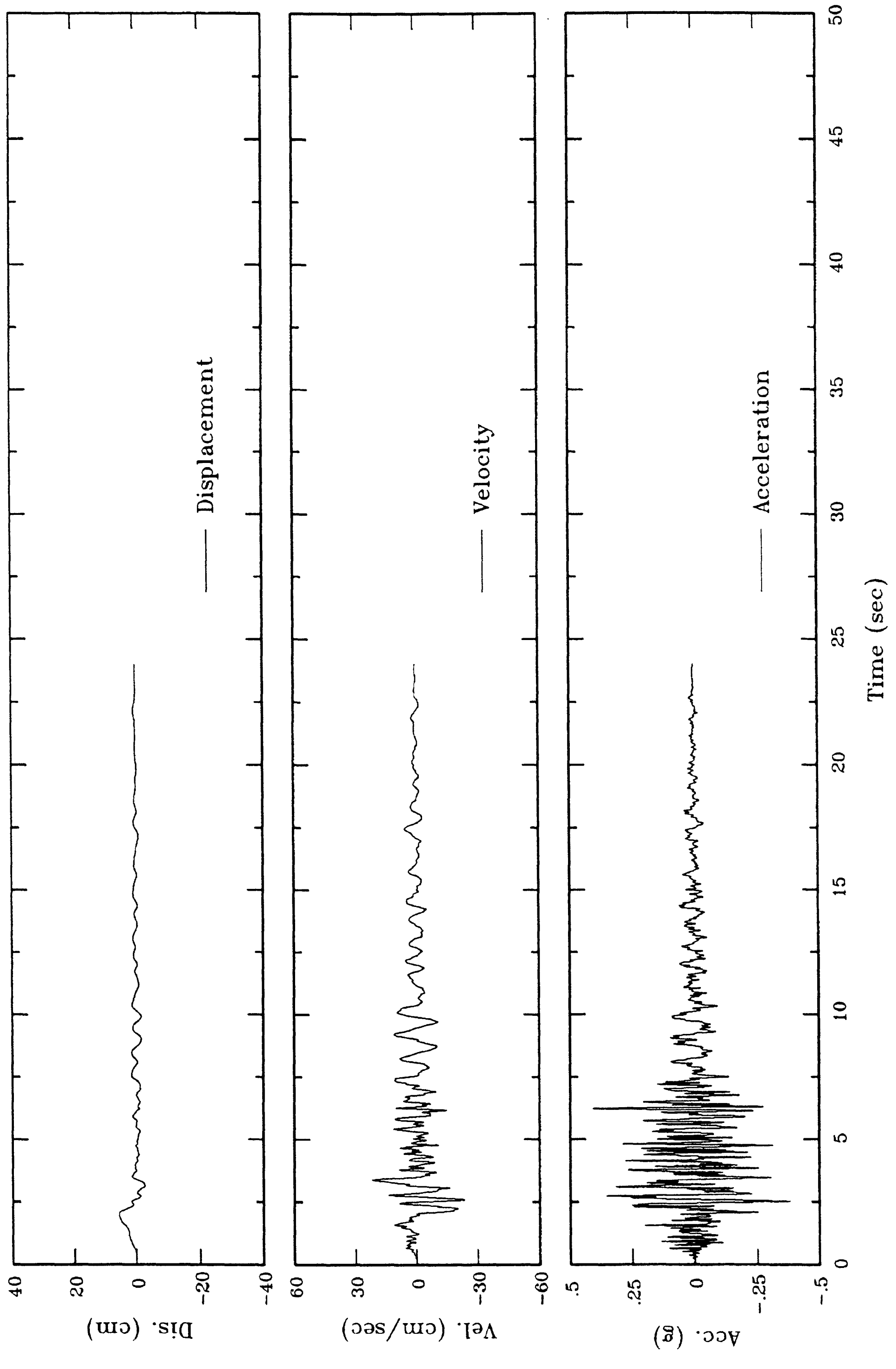


Figure C.12 Acceleration, Velocity, And Displacement Time Histories Of 1986 North Palm Springs Earthquake Recording

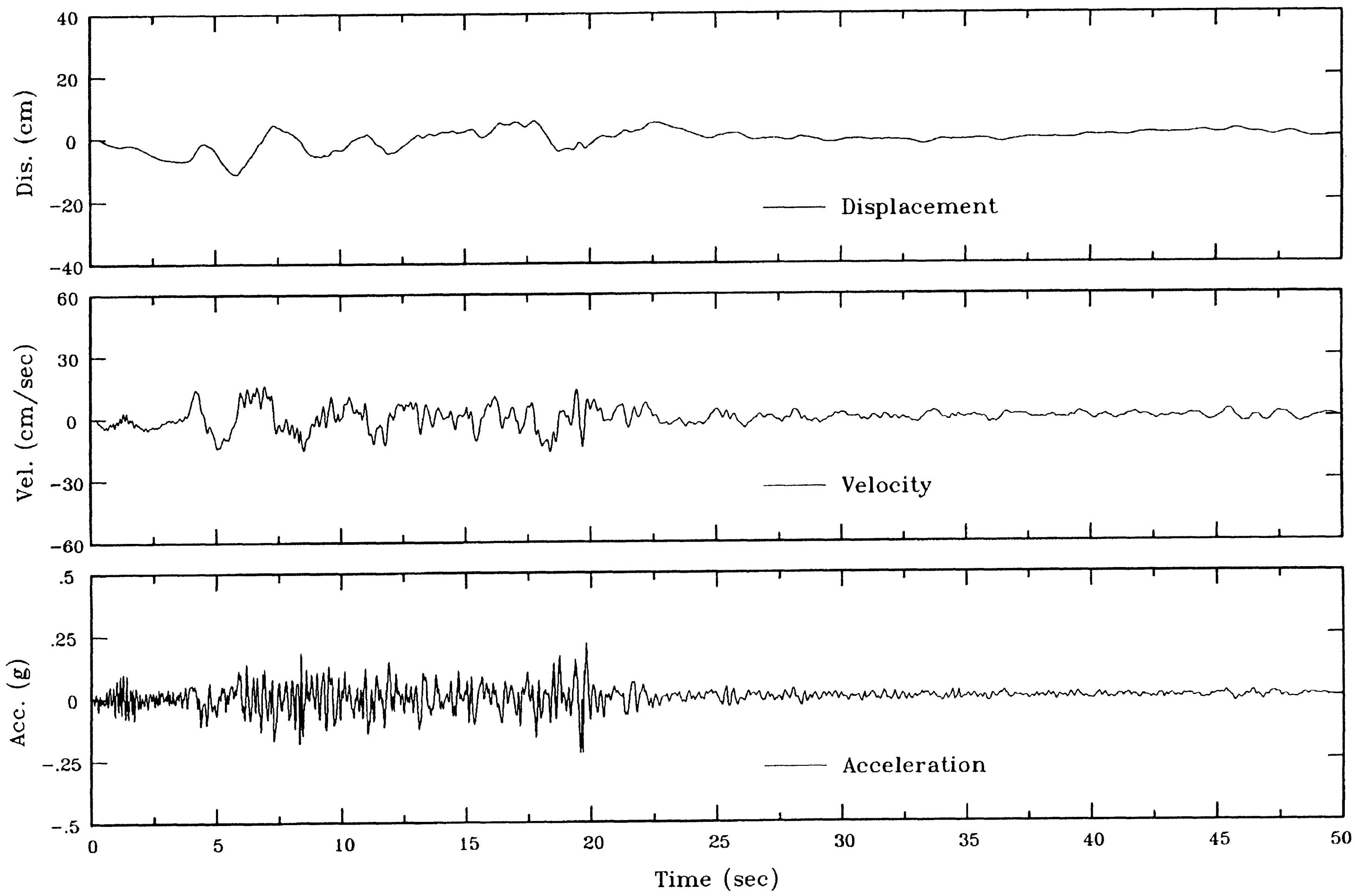


Figure C.13 Acceleration, Velocity, And Displacement Time Histories Of 1949 Western Washington Earthquake Recording, After Scaling And Spectrum Matching The Recording To The Design Response Spectrum Based On Current AASHTO Specifications, Site Class II, Washington Site

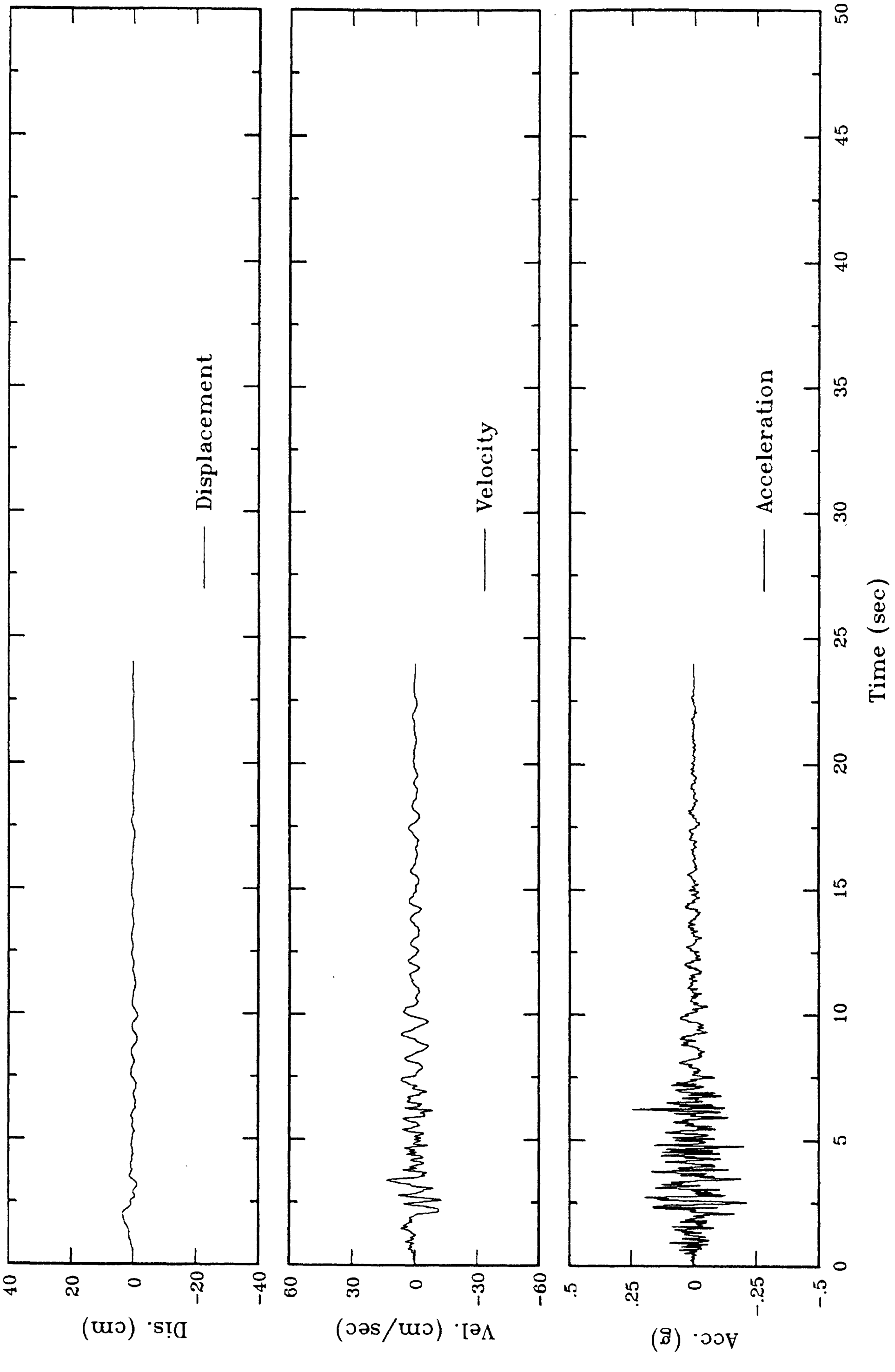


Figure C.14 Acceleration, Velocity, And Displacement Time Histories Of 1986 North Palm Springs Earthquake Recording, After Scaling And Spectrum Matching The Recording To The Design Response Spectrum Based On Current AASHTO Specifications, Site Class II, Washington Site

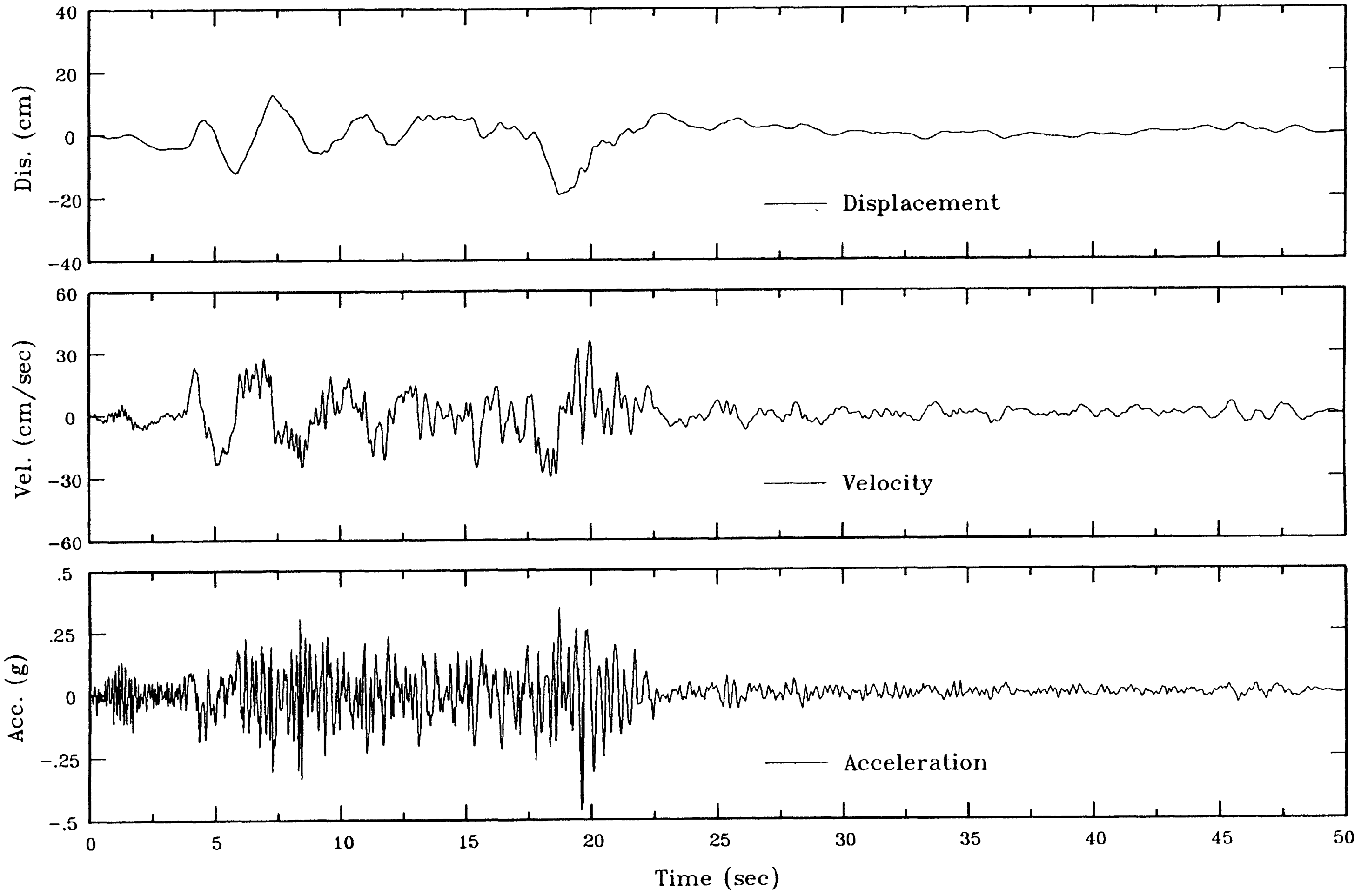


Figure C.15 Acceleration, Velocity, And Displacement Time Histories Of 1949 Western Washington Earthquake Recording, After Scaling And Spectrum Matching The Recording To The MCE Design Response Spectrum Based On The Proposed LRFD Specifications, Site Class C, Washington Site

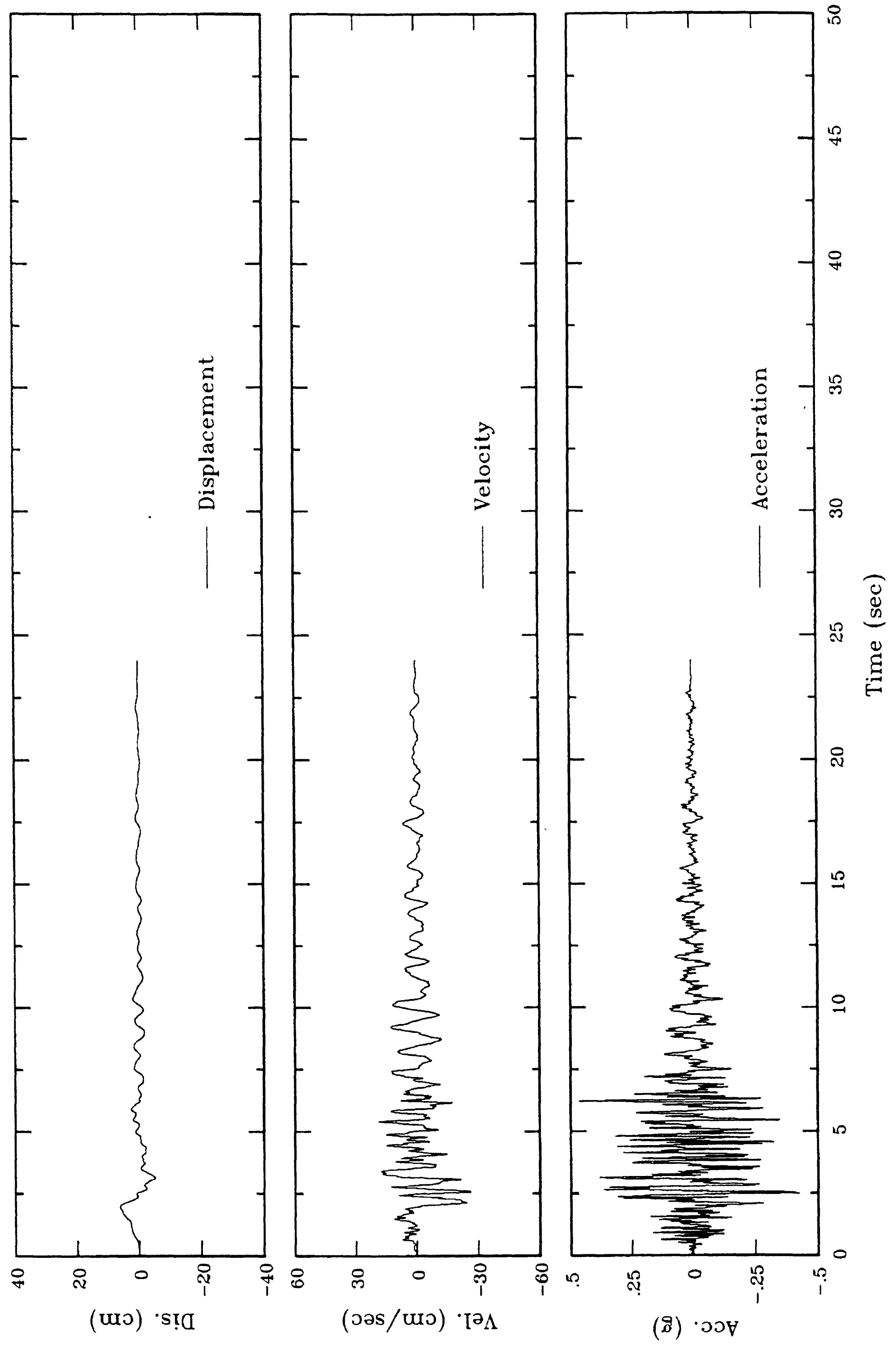


Figure C.16 Acceleration, Velocity, And Displacement Time Histories Of 1986 North Palm Springs Earthquake Recording, After Scaling And Spectrum Matching The Recording To The MCE Design Response Spectrum Based On The Proposed LRFD Specifications, Site Class C, Washington Site

Appendix D
SITE-SPECIFIC ANALYSES — WESTERN SITE

WASHDOT SITE

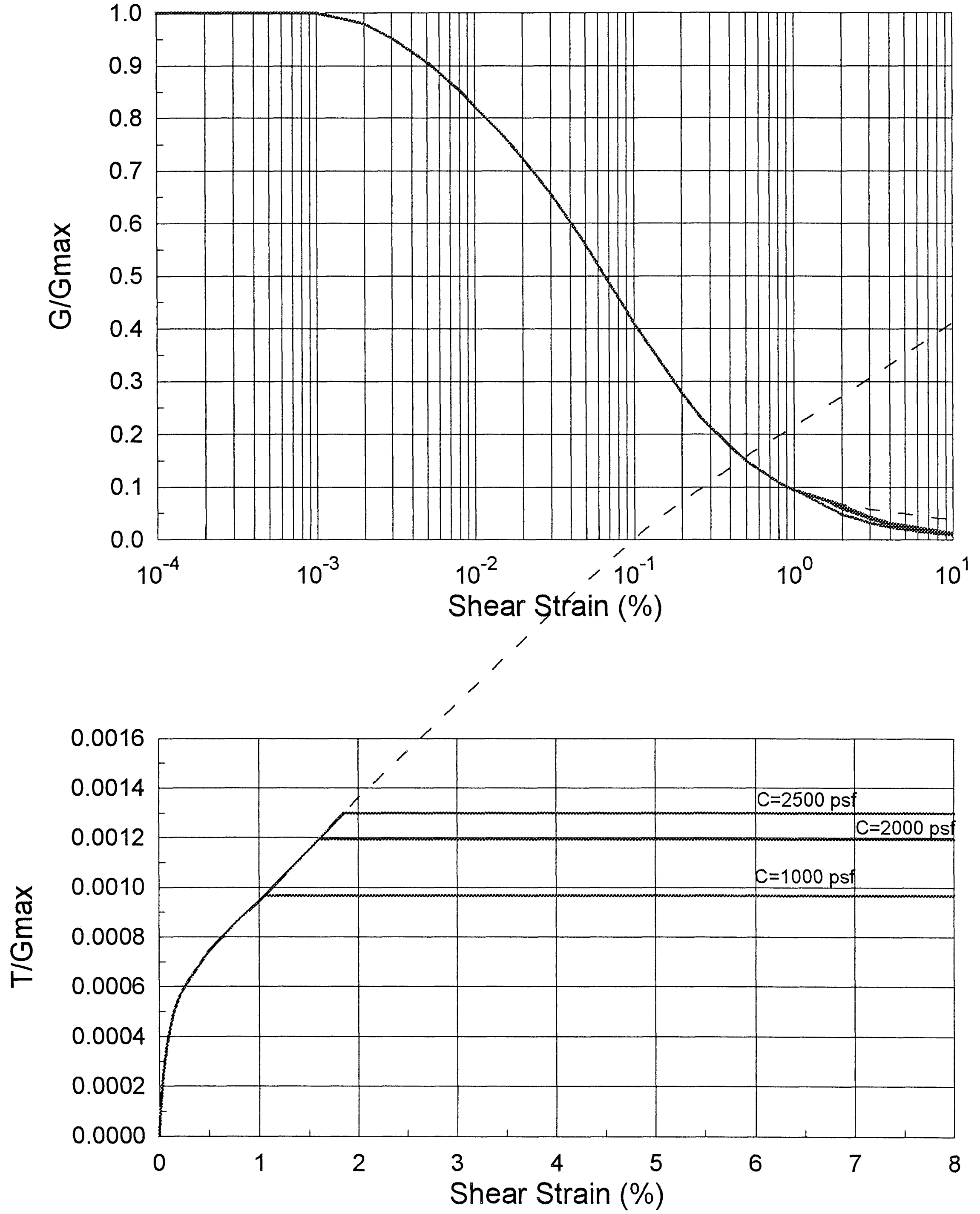
DESRA-MUSC ANALYSES

Graphs and Data Sheets Documenting:

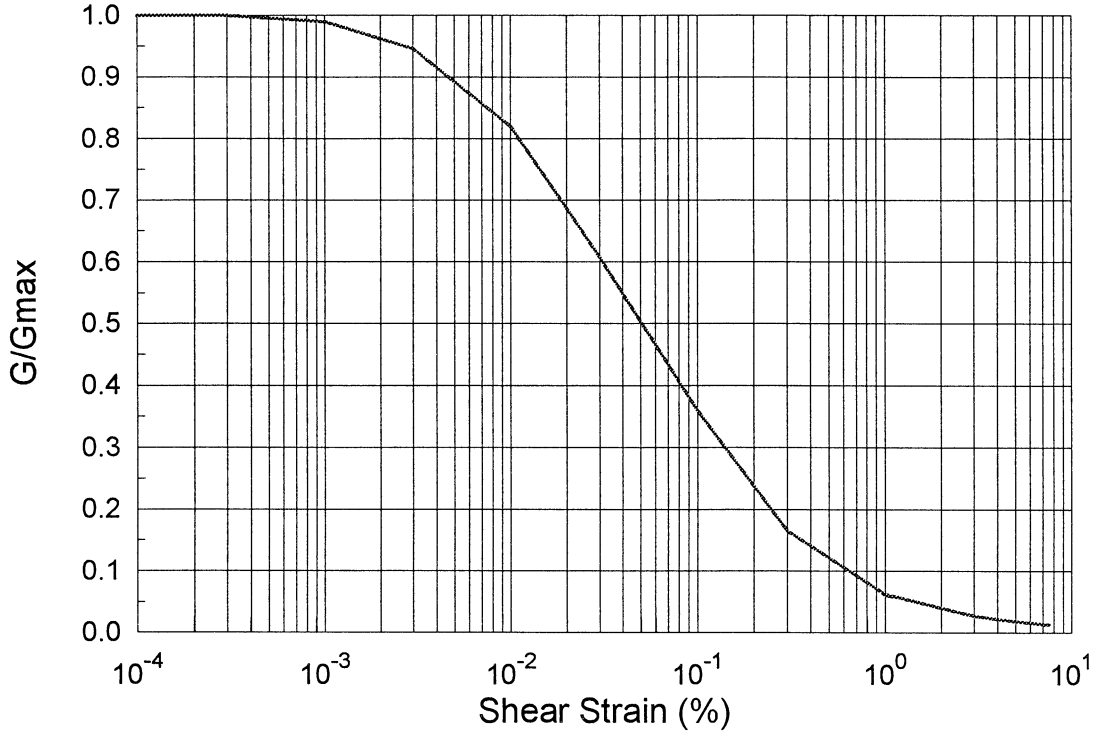
1. Input Data:
   - $G/G_{\max}$ and Backbone Curves
   - Liquefaction Strength Curves

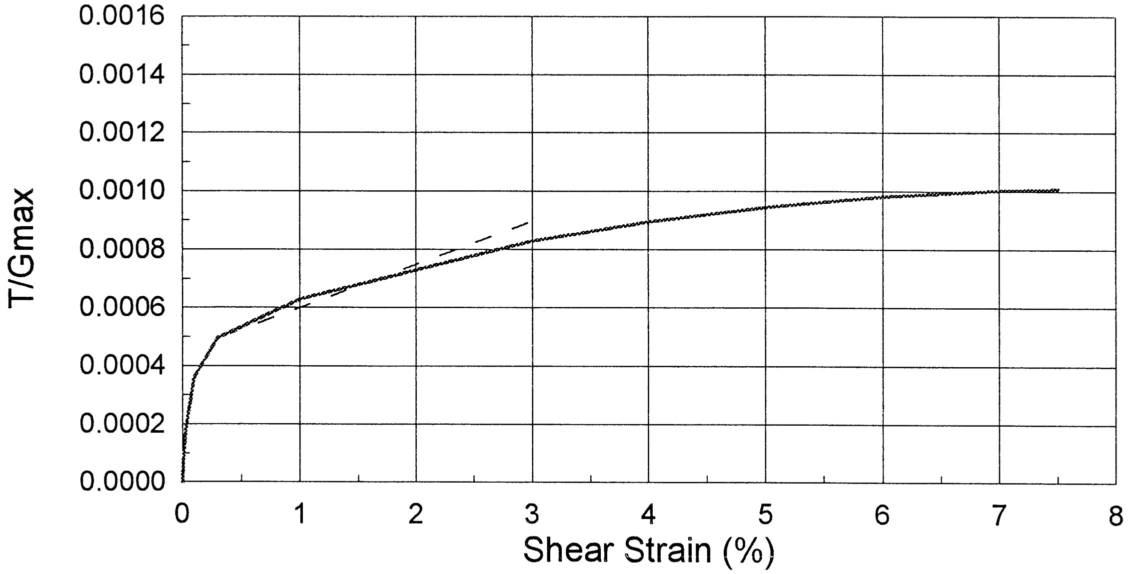
2. Results of Response Analyses

3. Tabular Summaries of Response Analyses

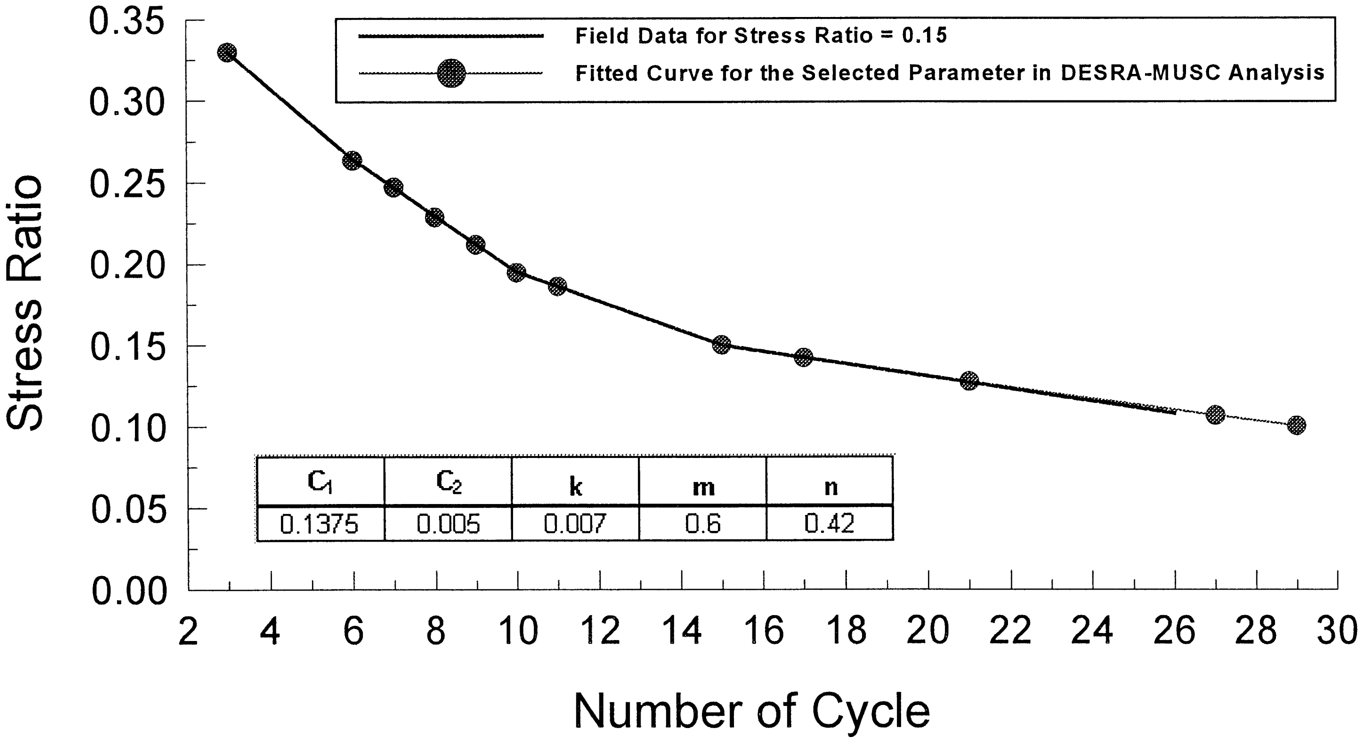


Matched G/Gmax and Backbones for P1 = 15



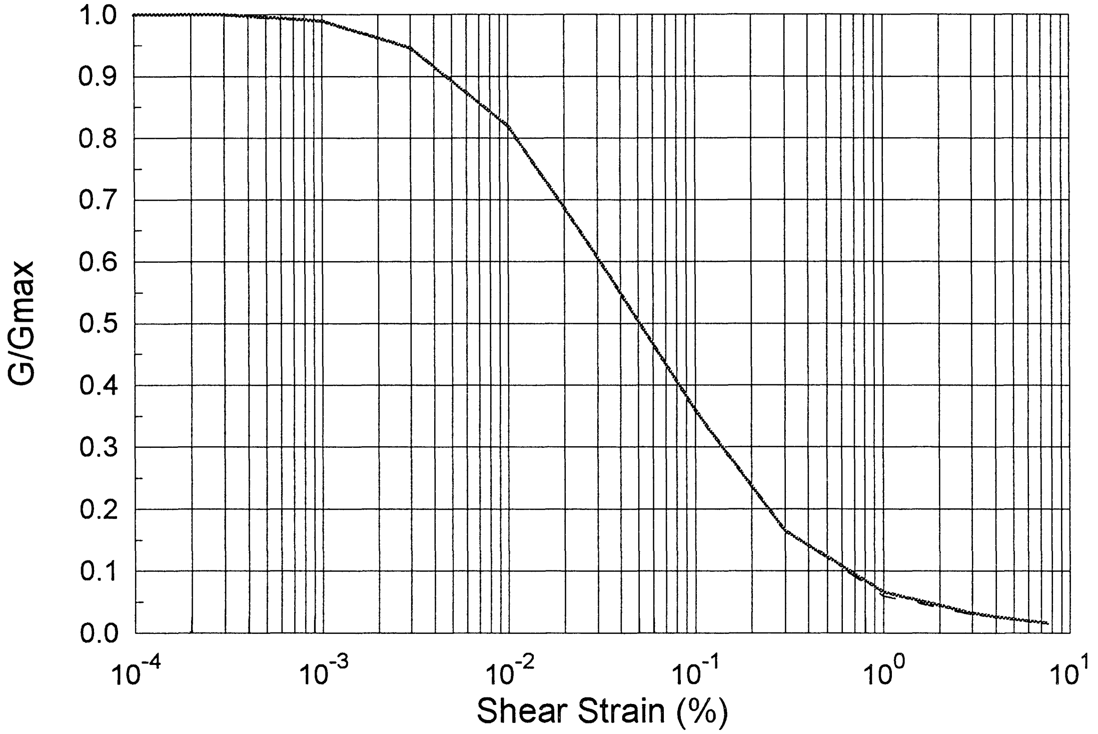


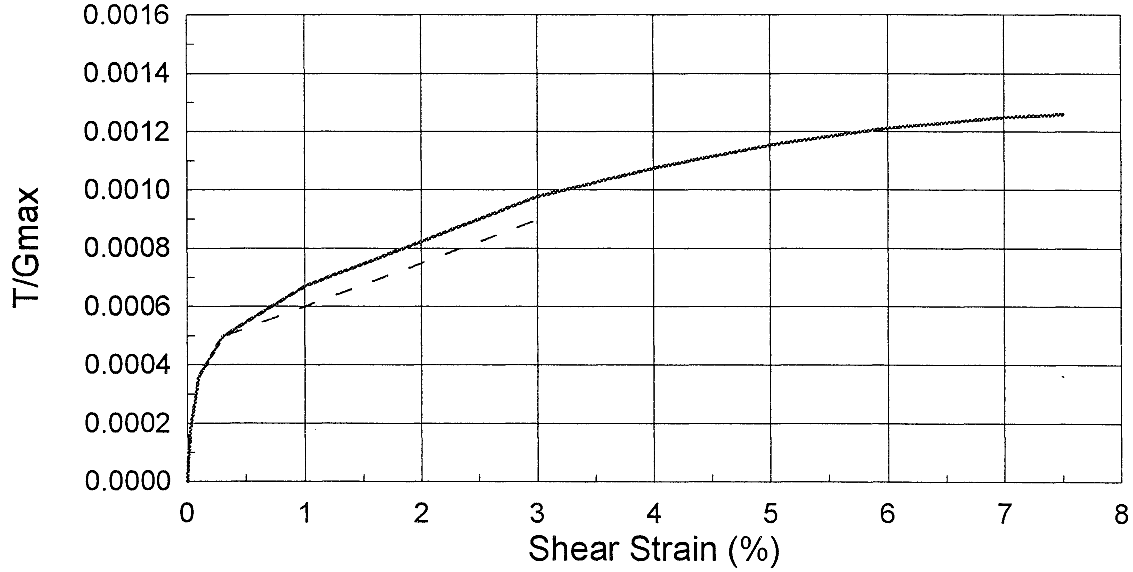
Upper 50', Stress Ratio = 0.15, Fric. Ang. = 32



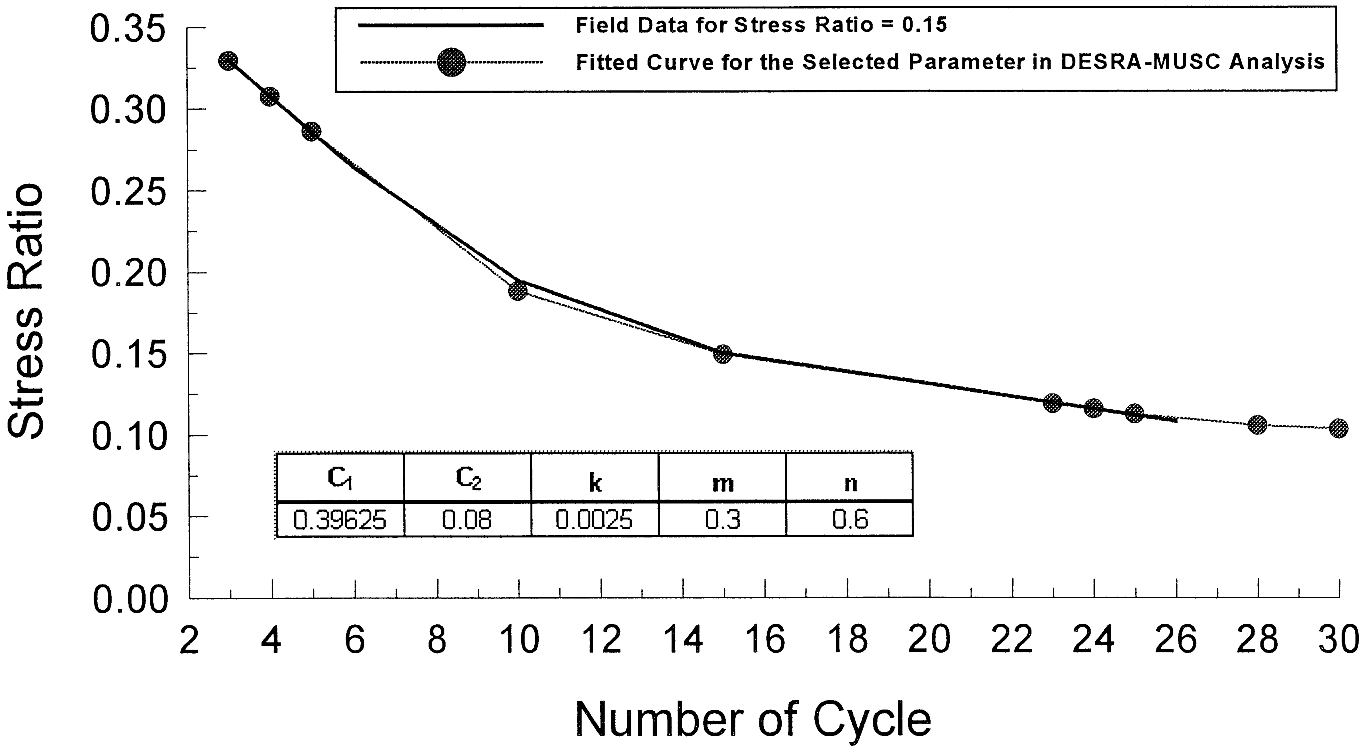
C_1	C_2	k	m	n
0.1375	0.005	0.007	0.6	0.42

Comparison of Liquefaction Curve for Stress Ratio of 0.15 (For Soil Layer at Depth Above 50 ft, Friction Angle = 32 Deg.)



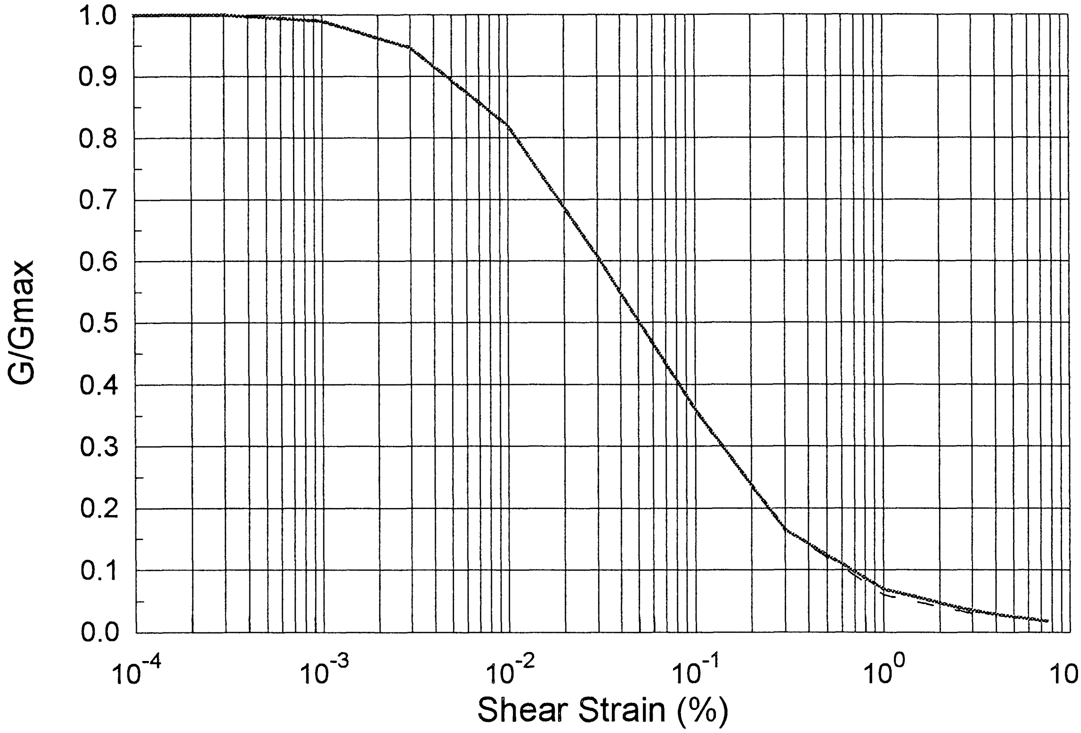


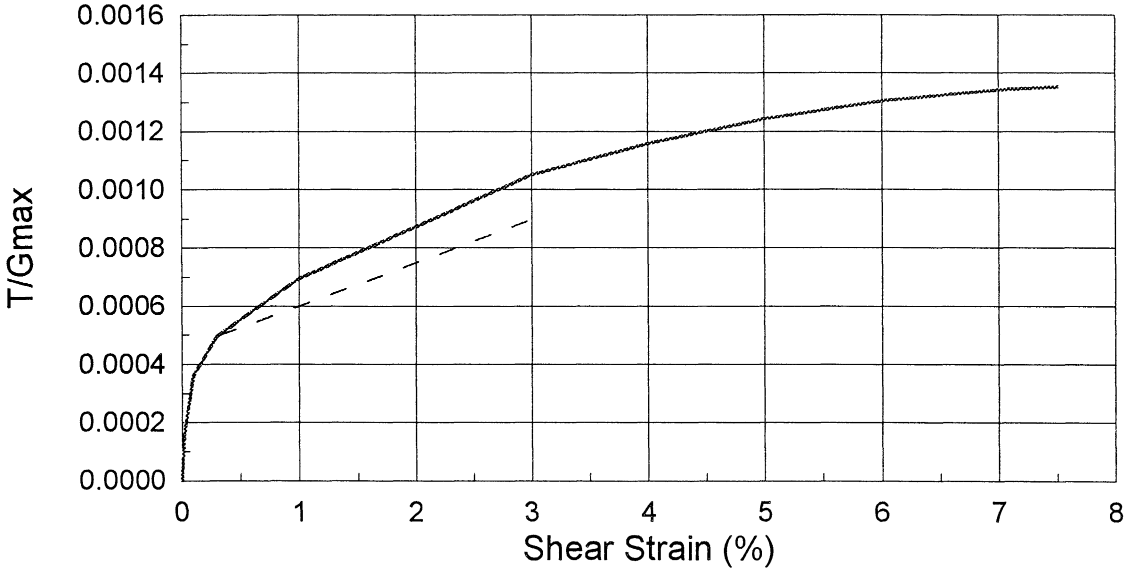
Upper 50', Stress Ratio = 0.15, Fric. Ang. = 38



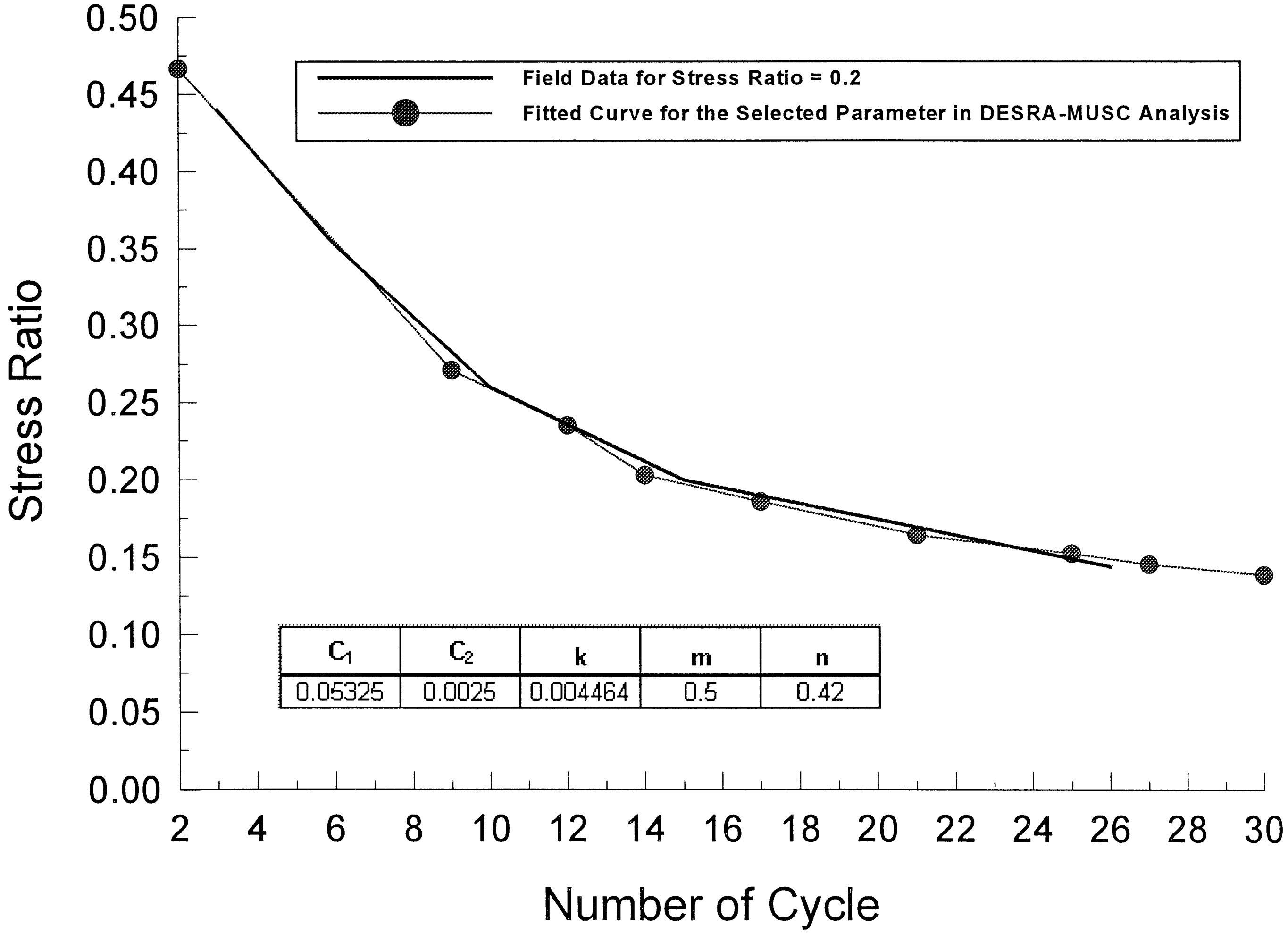
C_1	C_2	k	m	n
0.39625	0.08	0.0025	0.3	0.6

Comparison of Liquefaction Curve for Stress Ratio of 0.15 (For Soil Layer at Depth Above 50 ft, Friction Angle = 38 Deg.)



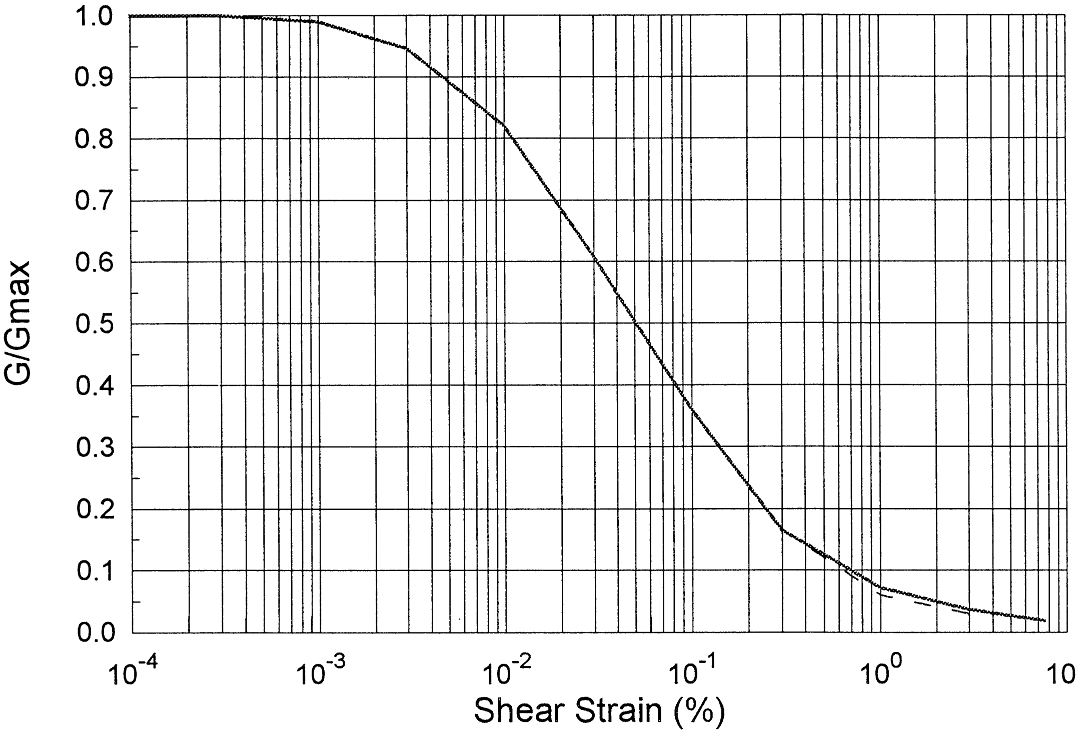


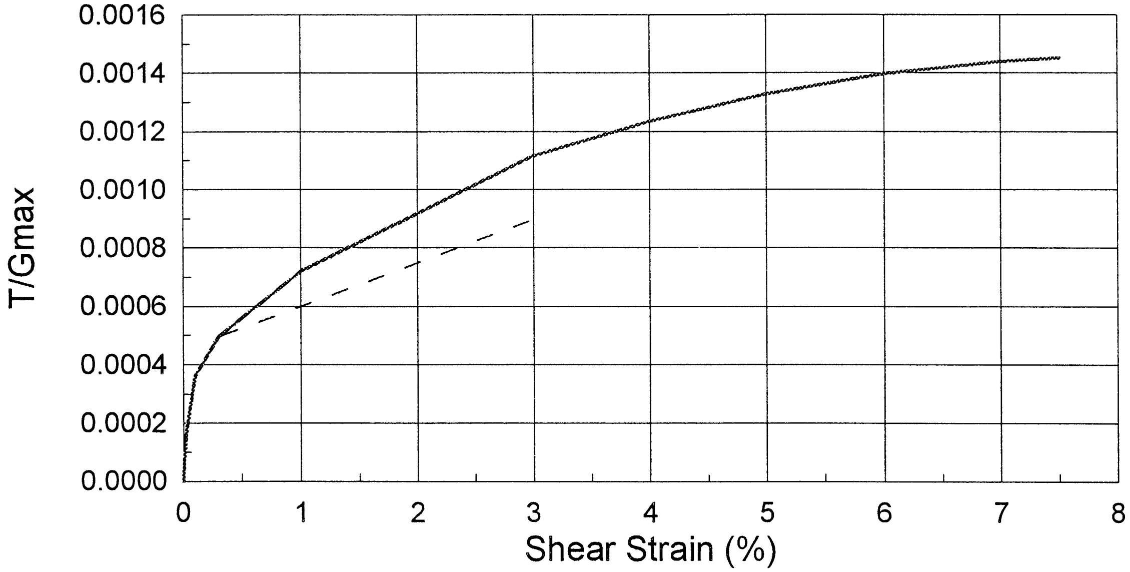
Upper 50', Stress Ratio = 0.20, Fric. Ang. = 40



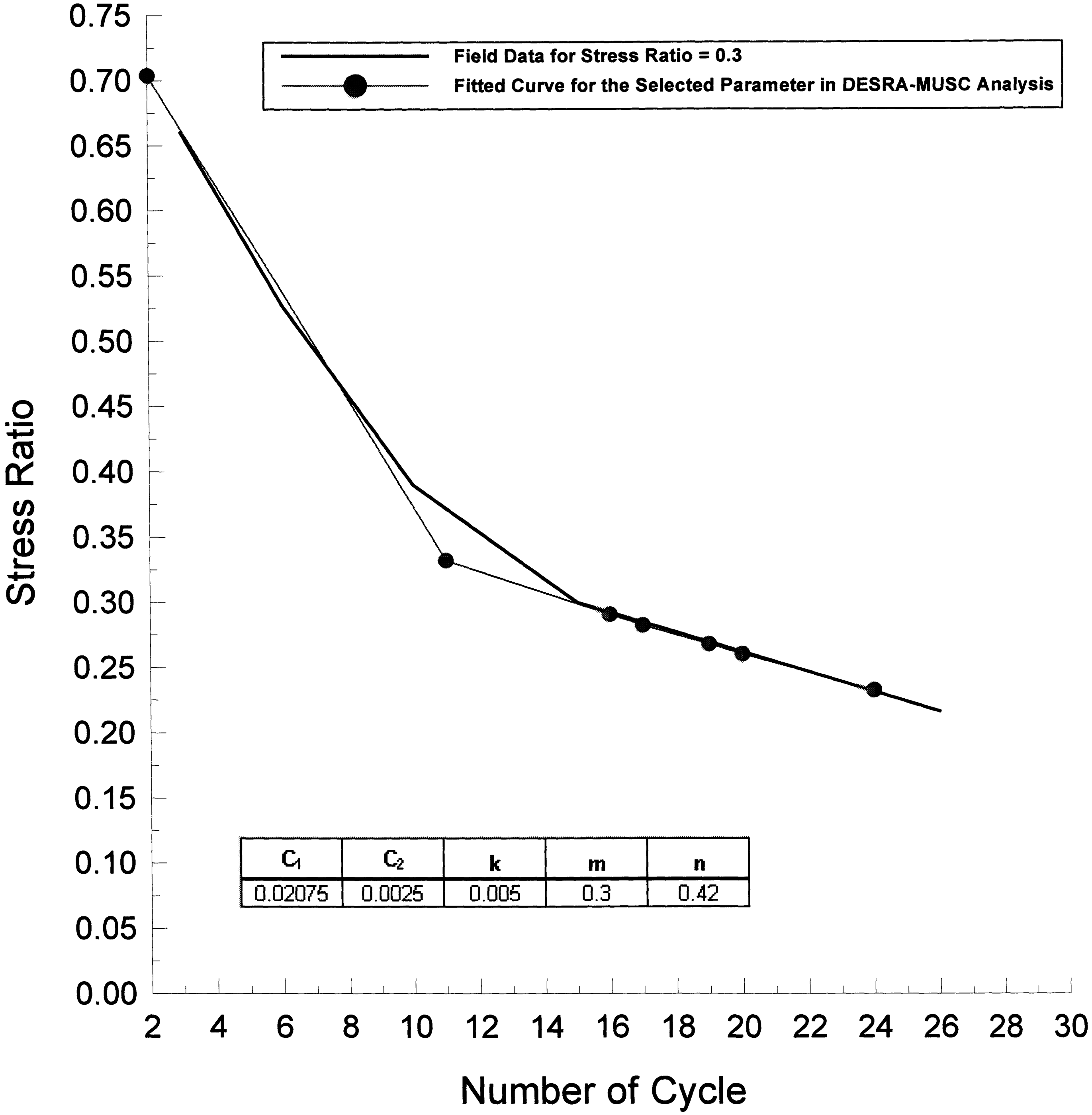
C_1	C_2	k	m	n
0.05325	0.0025	0.004464	0.5	0.42

Comparison of Liquefaction Curve for Stress Ratio of 0.20 (For Soil Layer at Depth Above 50 ft, Friction Angle = 40 Deg.)



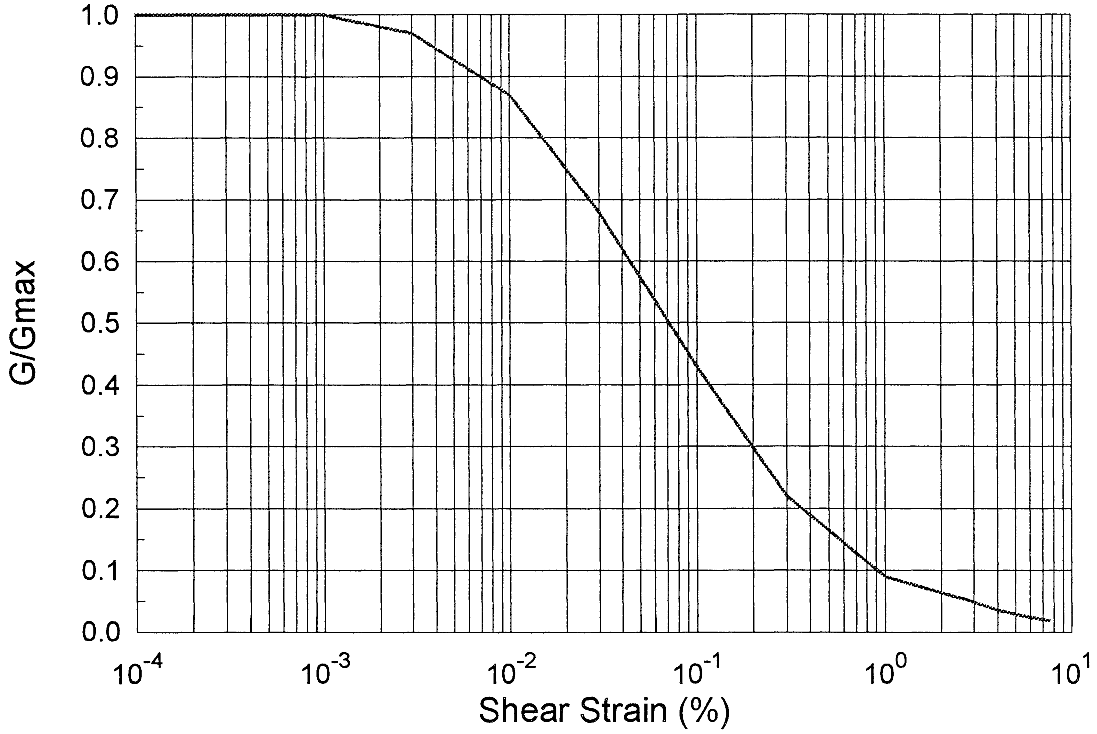


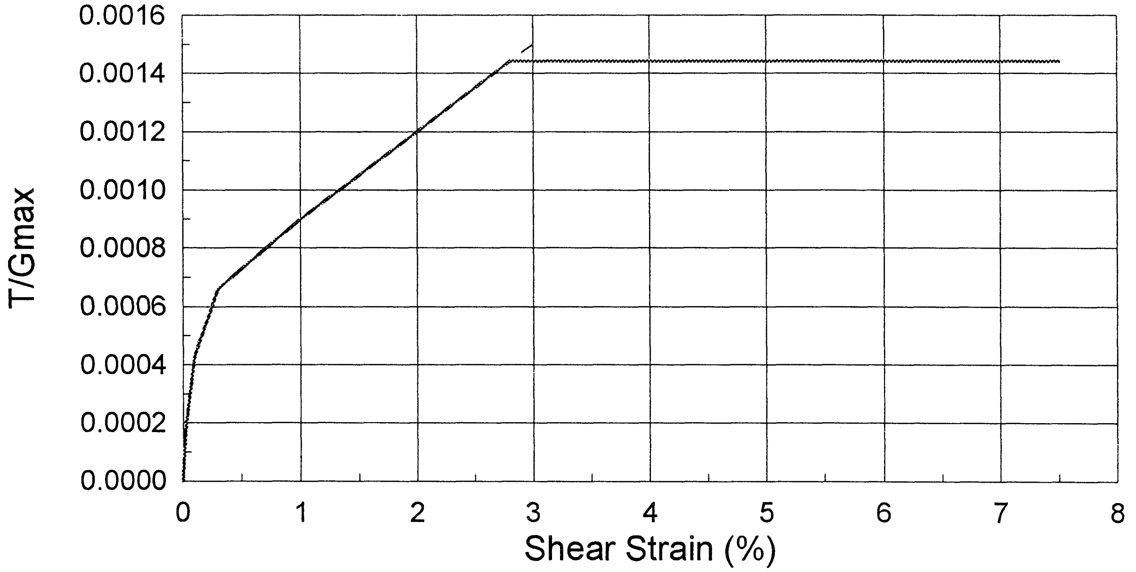
Upper 50', Stress Ratio = 0.30, Fric. Ang. = 42



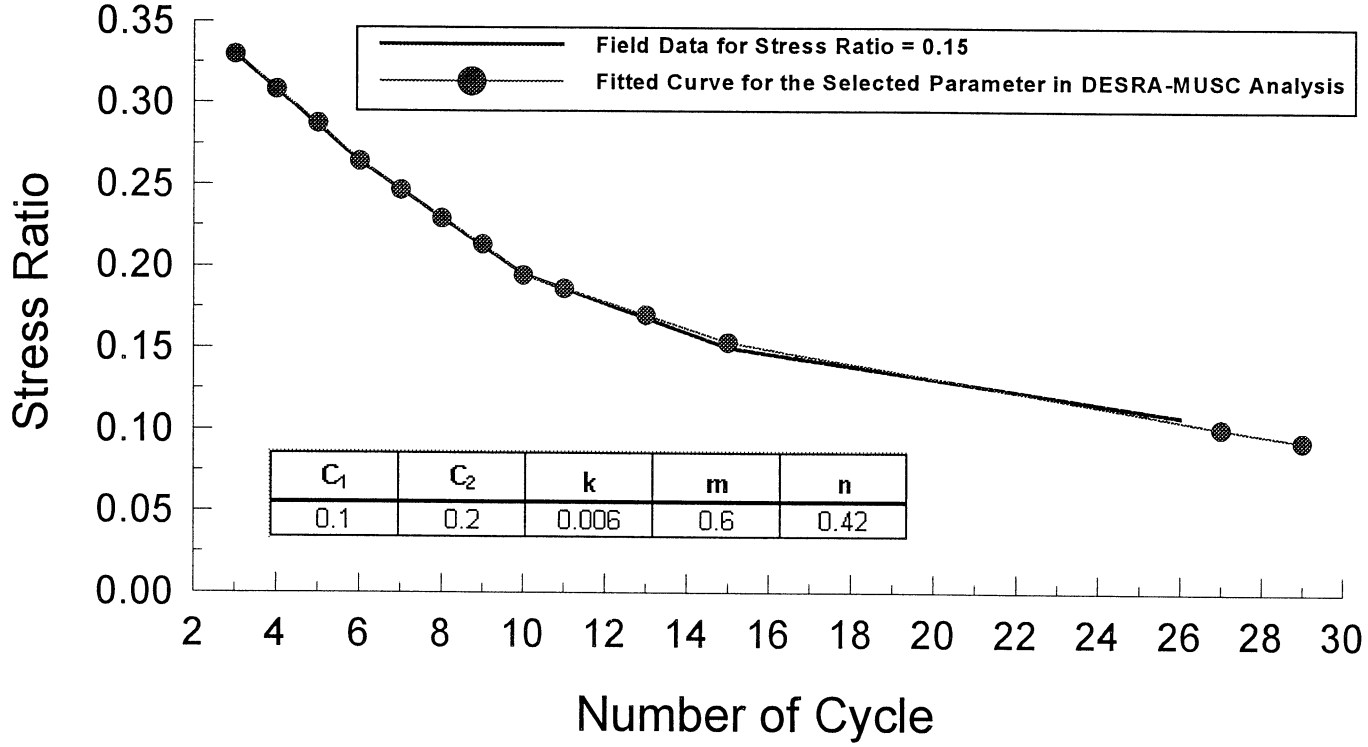
C_1	C_2	k	m	n
0.02075	0.0025	0.005	0.3	0.42

Comparison of Liquefaction Curve for Stress Ratio of 0.30
(For Soil Layer at Depth Above 50 ft, Friction Angle = 42 Deg.)





Lower 50', Stress Ratio = 0.15, Fric. Ang. = 32



C_1	C_2	k	m	n
0.1	0.2	0.006	0.6	0.42

Comparison of Liquefaction Curve for Stress Ratio of 0.15 (For Soil Layer at Depth Below 50 ft, Friction Angle = 32 Deg.

WASHDOT SITE

Case No	Descriptions
Case 1a	Effective Stress Analysis, Soil Profile without Fill, 1986 Desert Hot Spring EQ, 475-yr ARP
Case 1b	Effective Stress Analysis, Soil Profile without Fill, 1986 Desert Hot Spring EQ, 2475-yr ARP
Case 2a	Effective Stress Analysis, Soil Profile with Fill, 1986 Desert Hot Spring EQ, 475-yr ARP
Case 2b	Effective Stress Analysis, Soil Profile witht Fill, 1986 Desert Hot Spring EQ, 2475-yr ARP
Case 3a	Effective Stress Analysis, Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP
Case 3b	Effective Stress Analysis, Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP
Case 4a	Effective Stress Analysis, Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP
Case 4b	Effective Stress Analysis, Soil Profile with Fill, 1985 Chile EQ, 2475-yr ARP
Case 5a	Effective Stress Analysis, Soil Profile without Fill, 1949 Olympia EQ, 475-yr ARP
Case 5b	Effective Stress Analysis, Soil Profile without Fill, 1949 Olympia EQ, 2475-yr ARP
Case 6a	Effective Stress Analysis, Soil Profile with Fill, 1949 Olympia EQ, 475-yr ARP
Case 6b	Effective Stress Analysis, Soil Profile with Fill, 1949 Olympia EQ, 2475-yr ARP

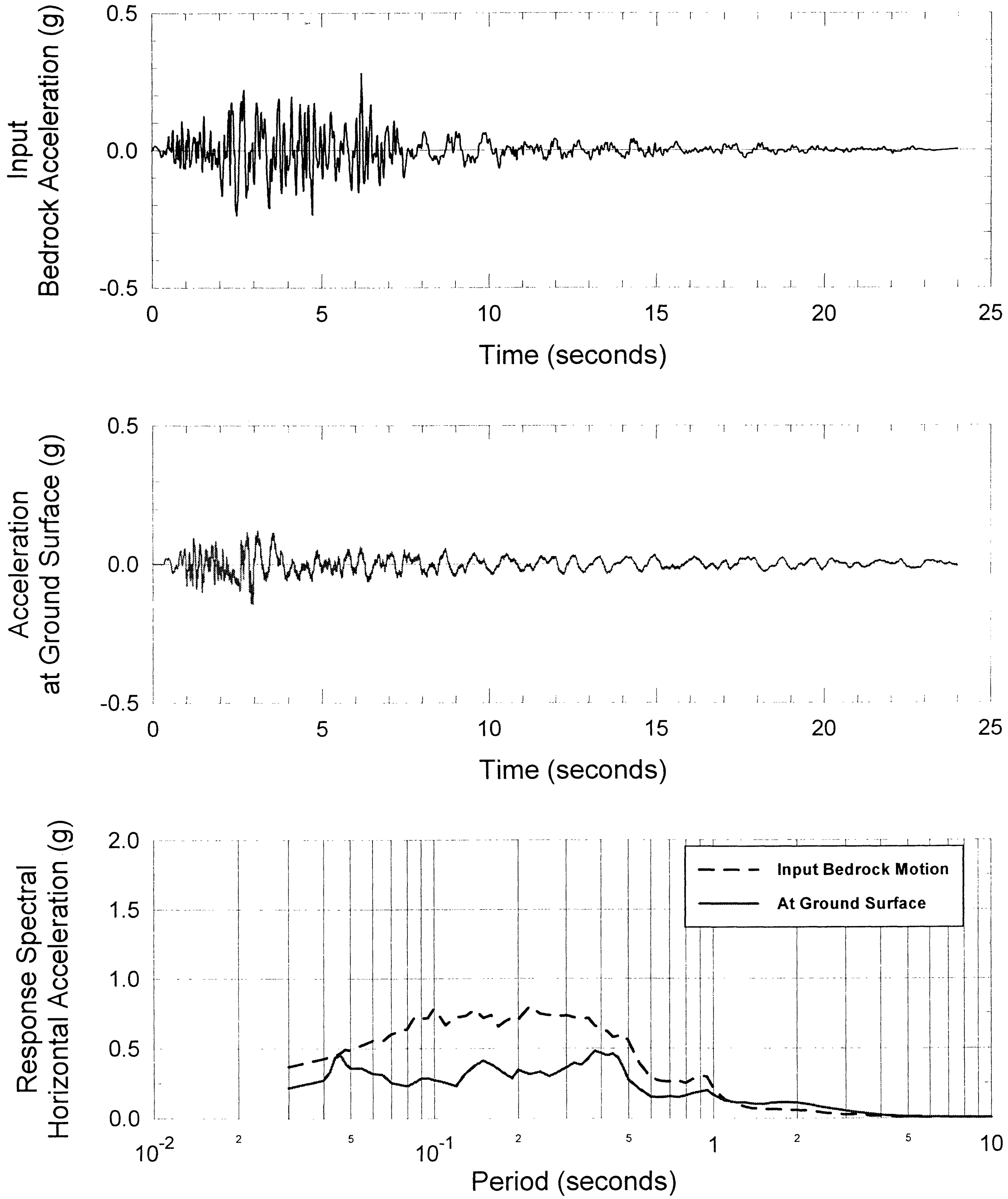


Figure 1a-1 Acceleration Time Histories and Response Spectra Case 1a: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 475-yr ARP

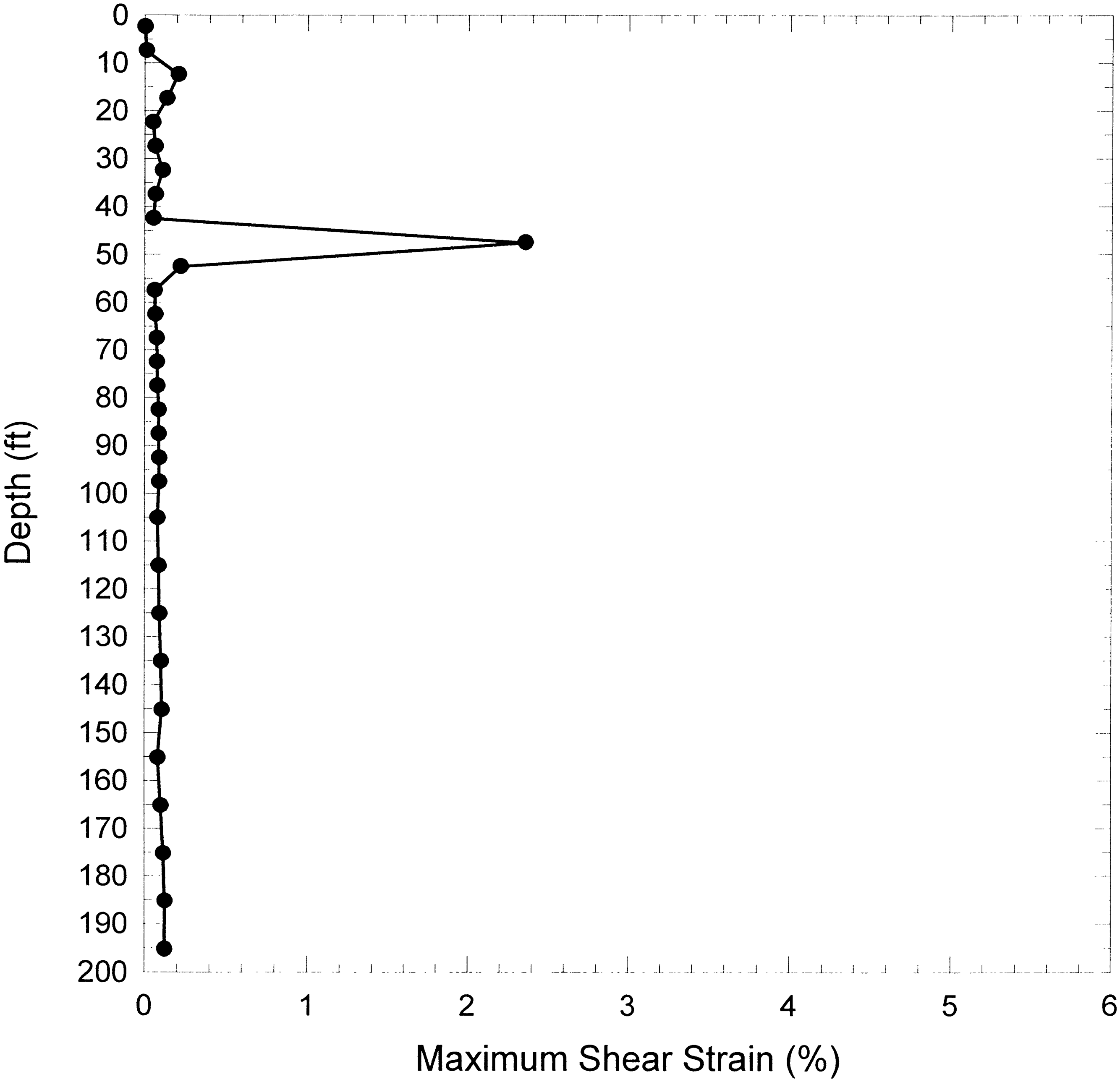


Figure 1a-2 Maximum Shear Strain Occurred During the Shaking Case 1a: Soil Profile without Fill 1986 Desert Hot Springs EQ, 475-yr ARP

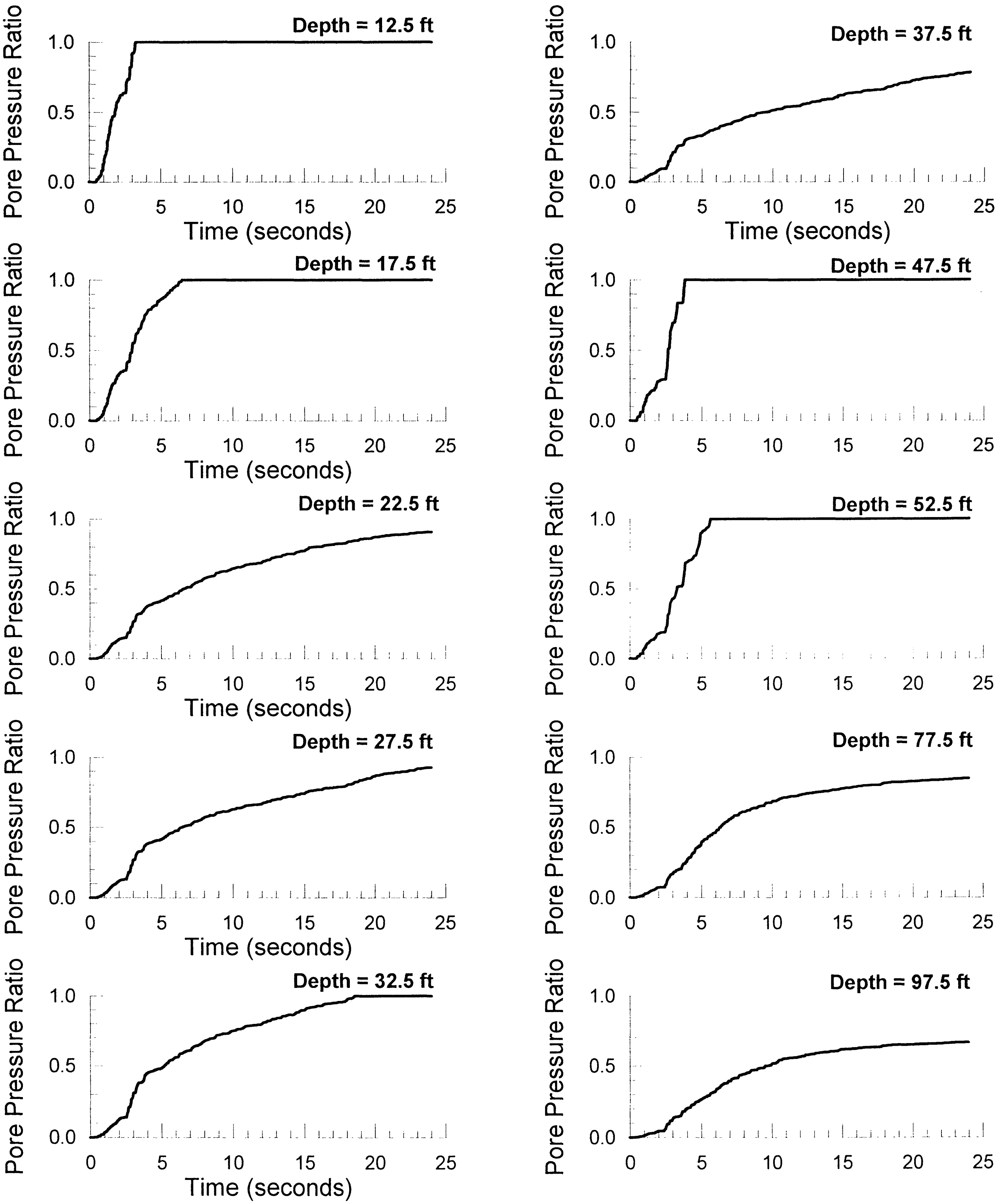


Figure 1a-3 Pore Pressure Generation
Case 1a: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 475-yr ARP

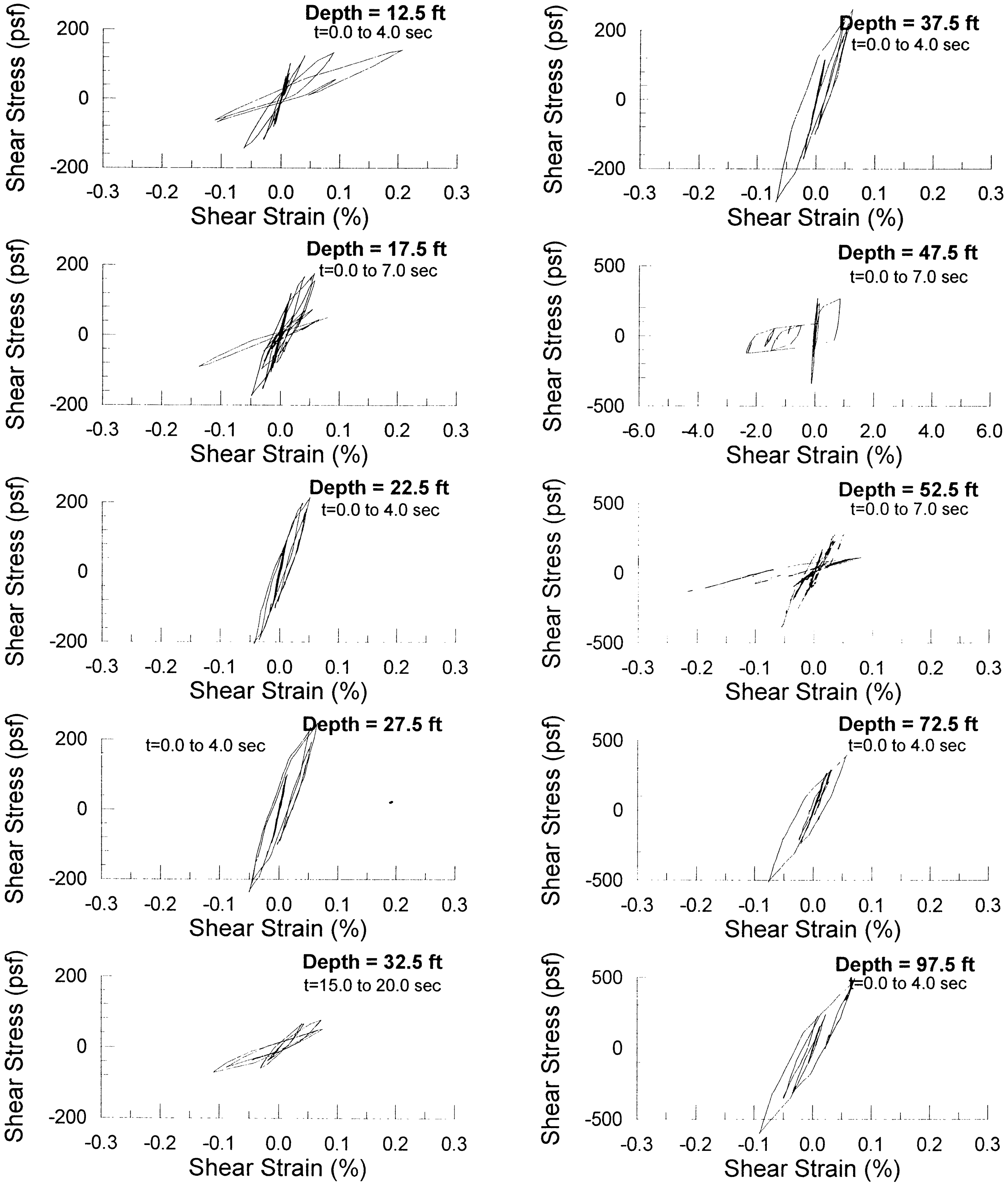


Figure 1a-4 Shear Stress-Shear Strain Loops
Case 1a: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 475-yr ARP

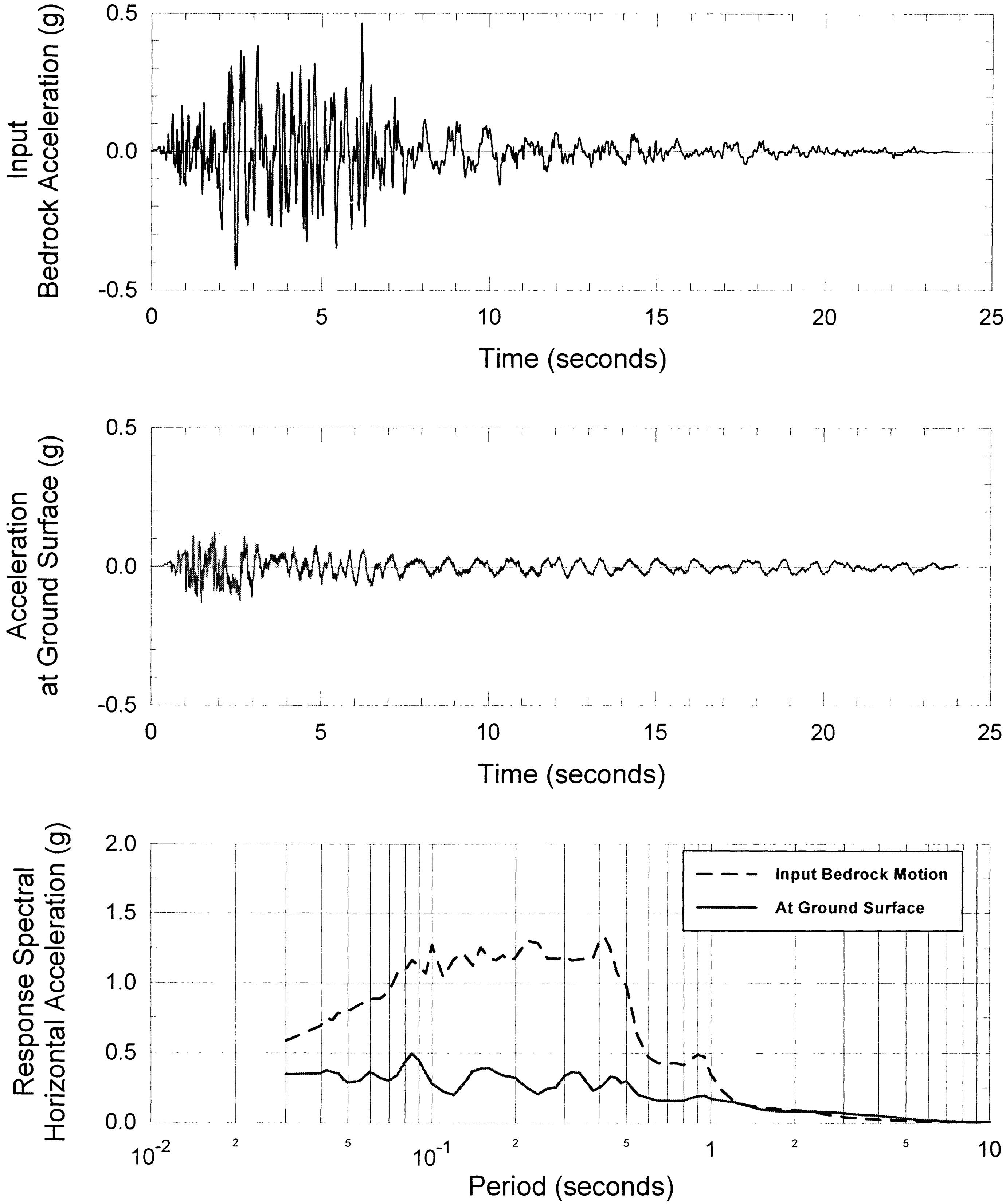


Figure 1b-1 Acceleration Time Histories and Response Spectra
Case 1b: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP

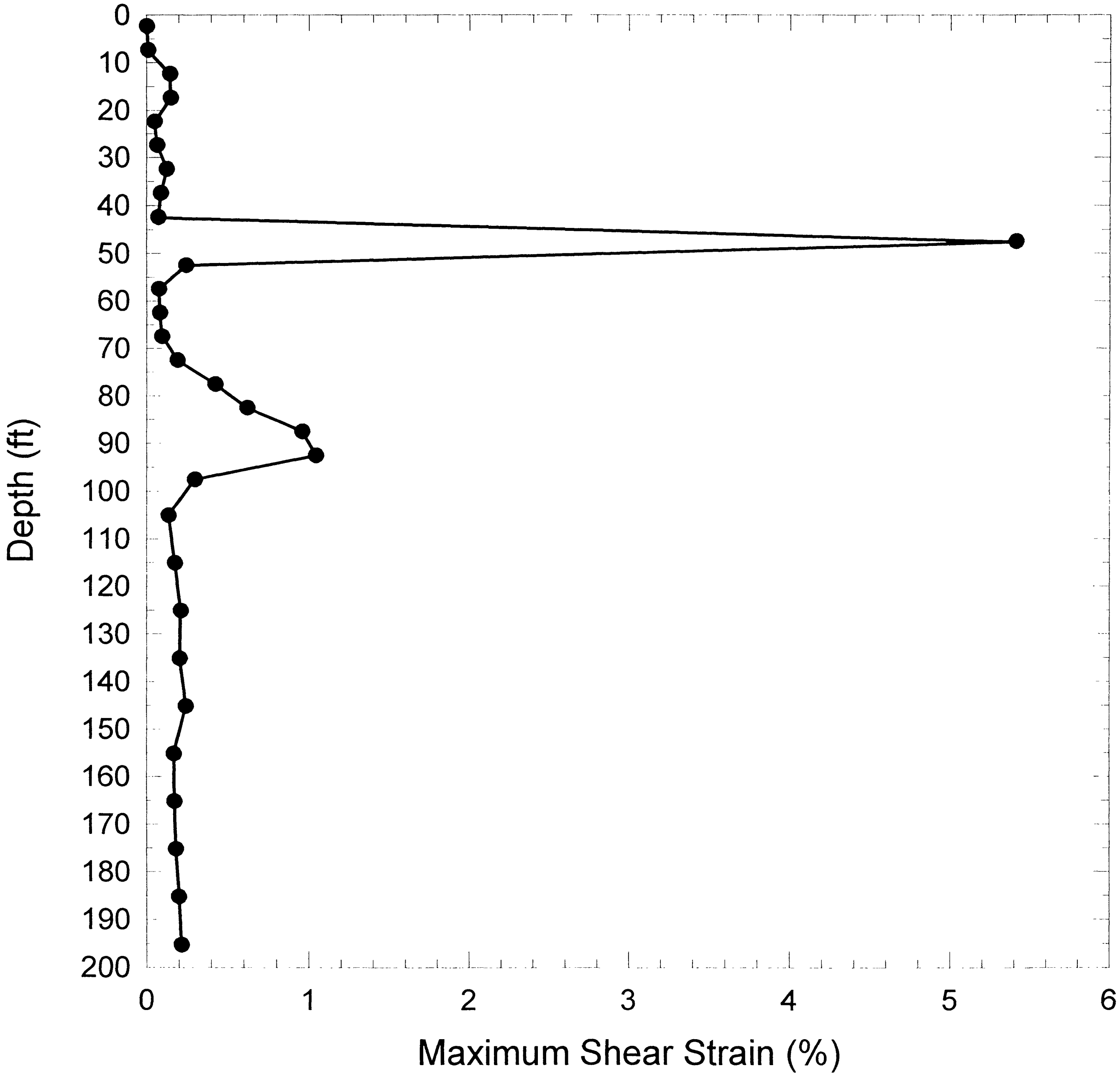


Figure 1b-2 Maximum Shear Strain Occurred During the Shaking Case 1b: Soil Profile without Fill 1986 Desert Hot Springs EQ, 2475-yr ARP

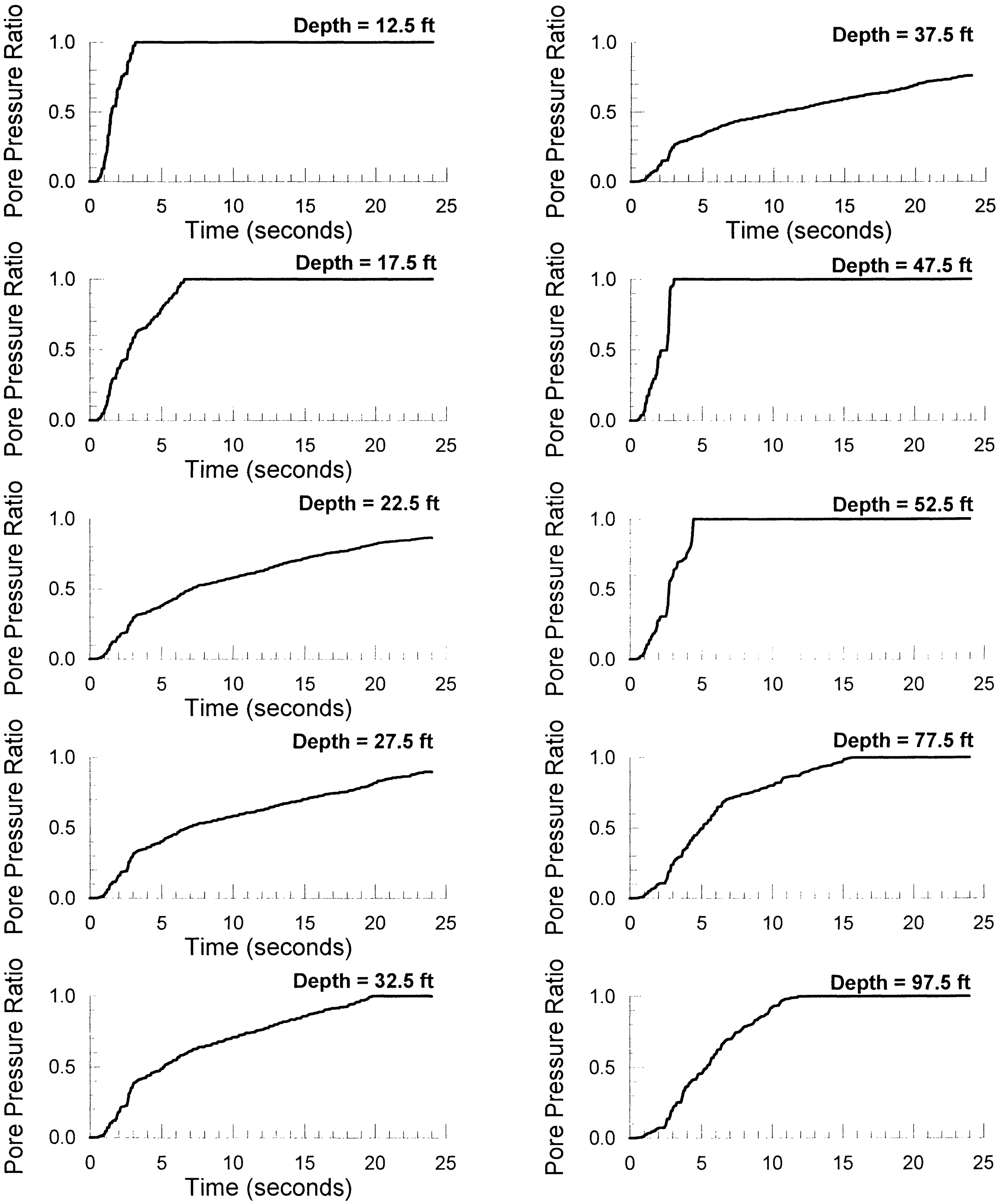


Figure 1b-3 Pore Pressure Generation
Case 1b: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP

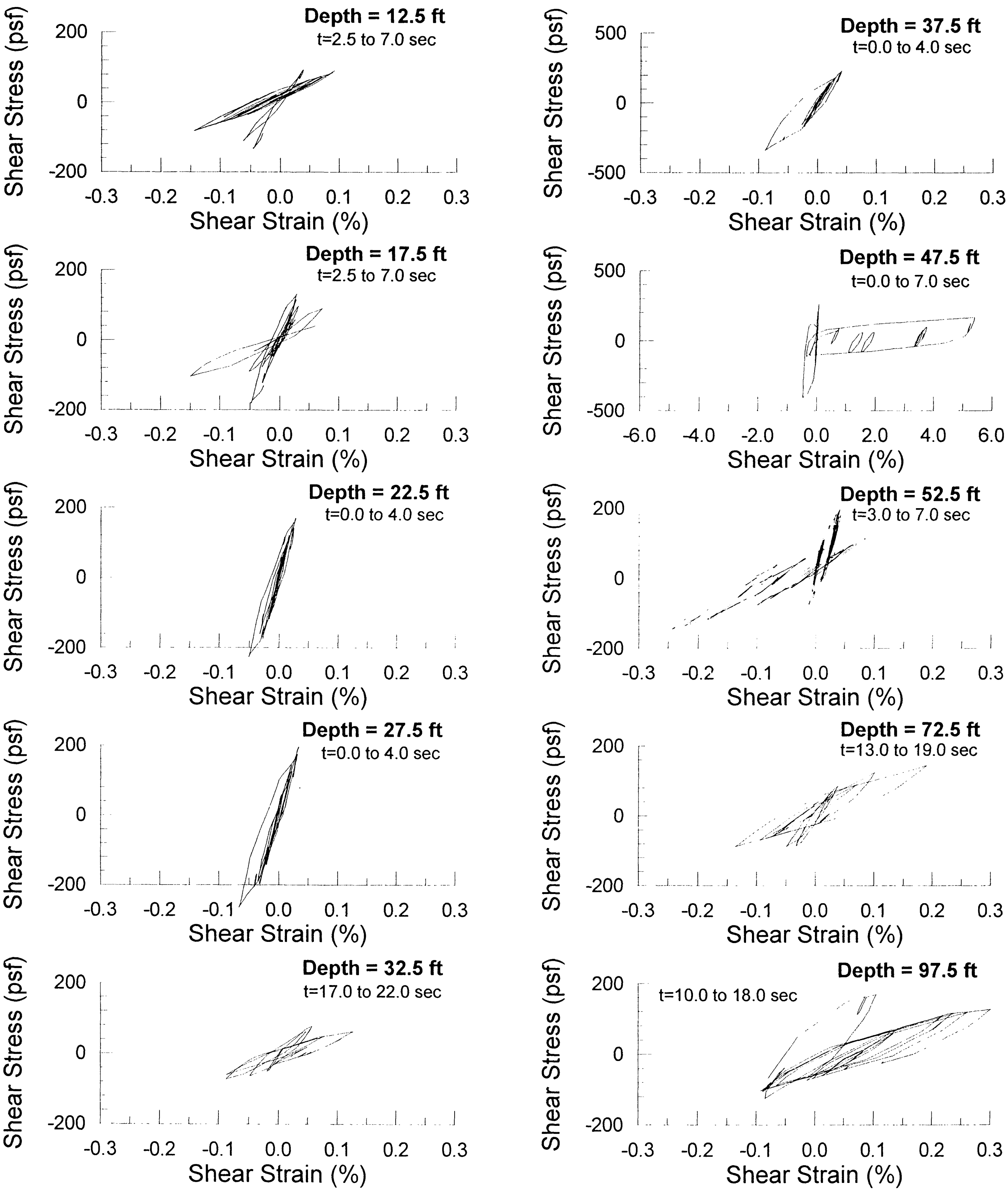


Figure 1b-4 Shear Stress-Shear Strain Loops
Case 1b: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP

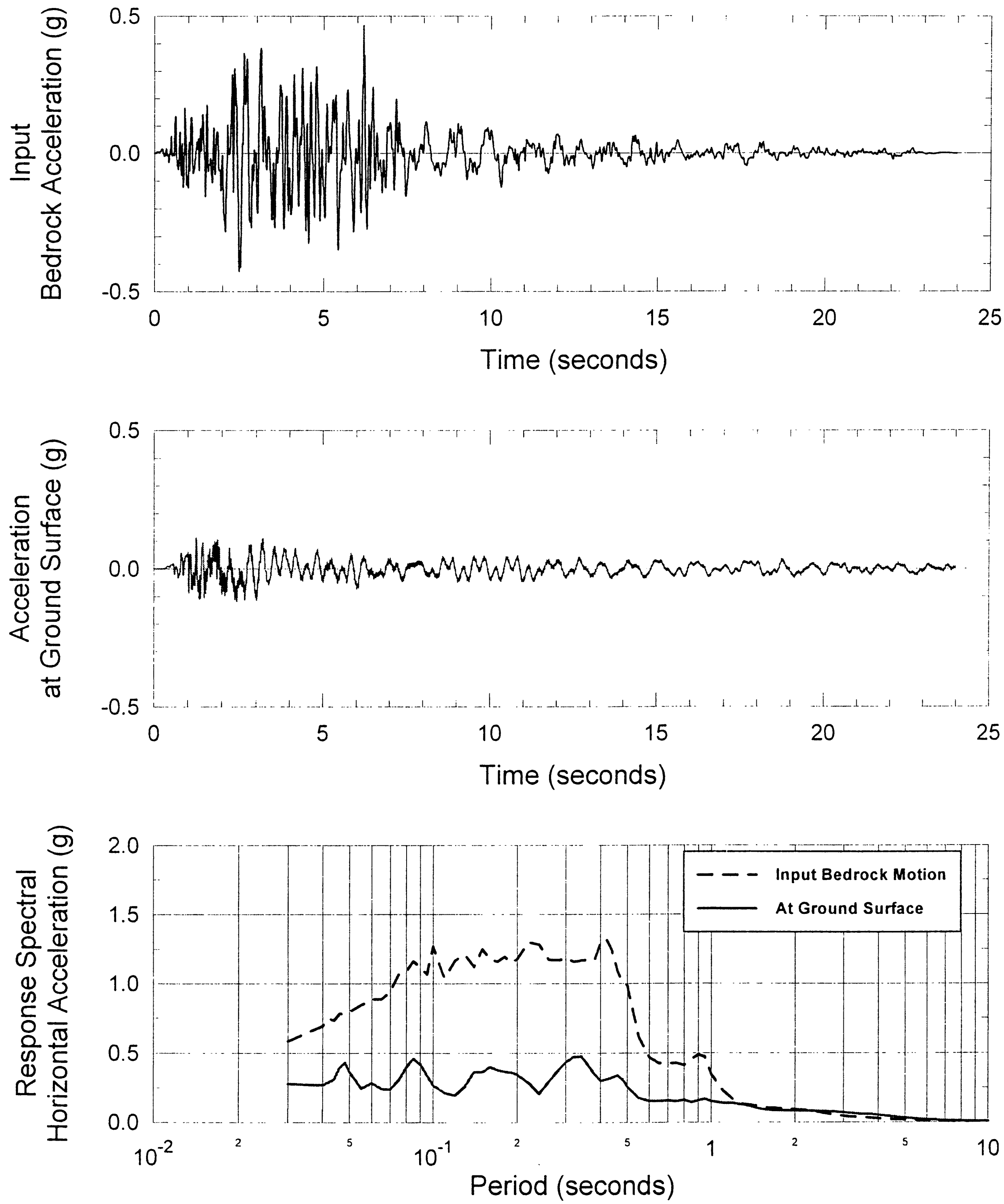


Figure 1c-1 Acceleration Time Histories and Response Spectra Case 1c: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP No Pore Pressure Redistribution

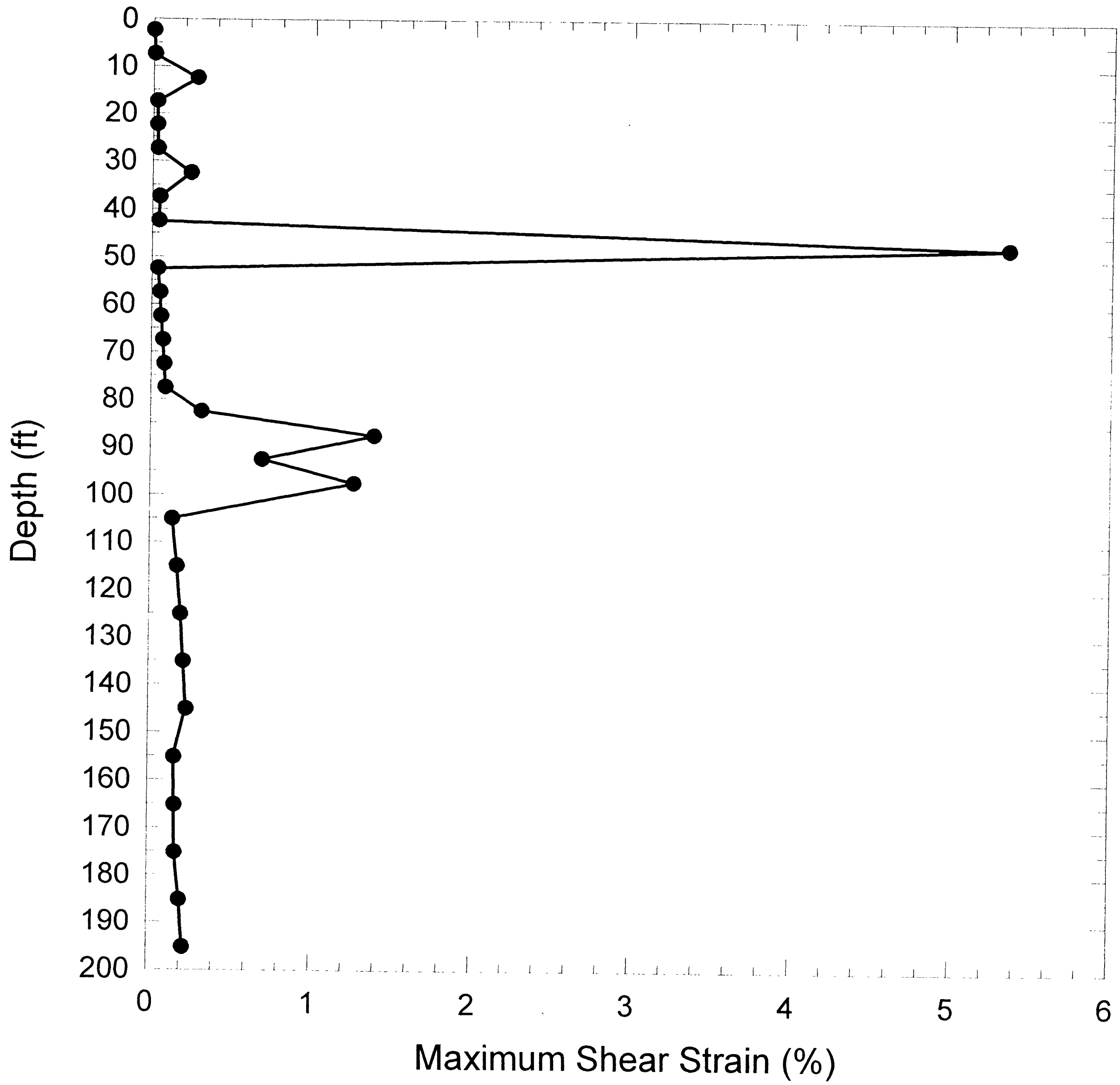


Figure 1c-2 Maximum Shear Strain Occurred During the Shaking
Case 1c: Soil Profile without Fill
1986 Desert Hot Springs EQ, 2475-yr ARP
No Pore Pressure Redistribution

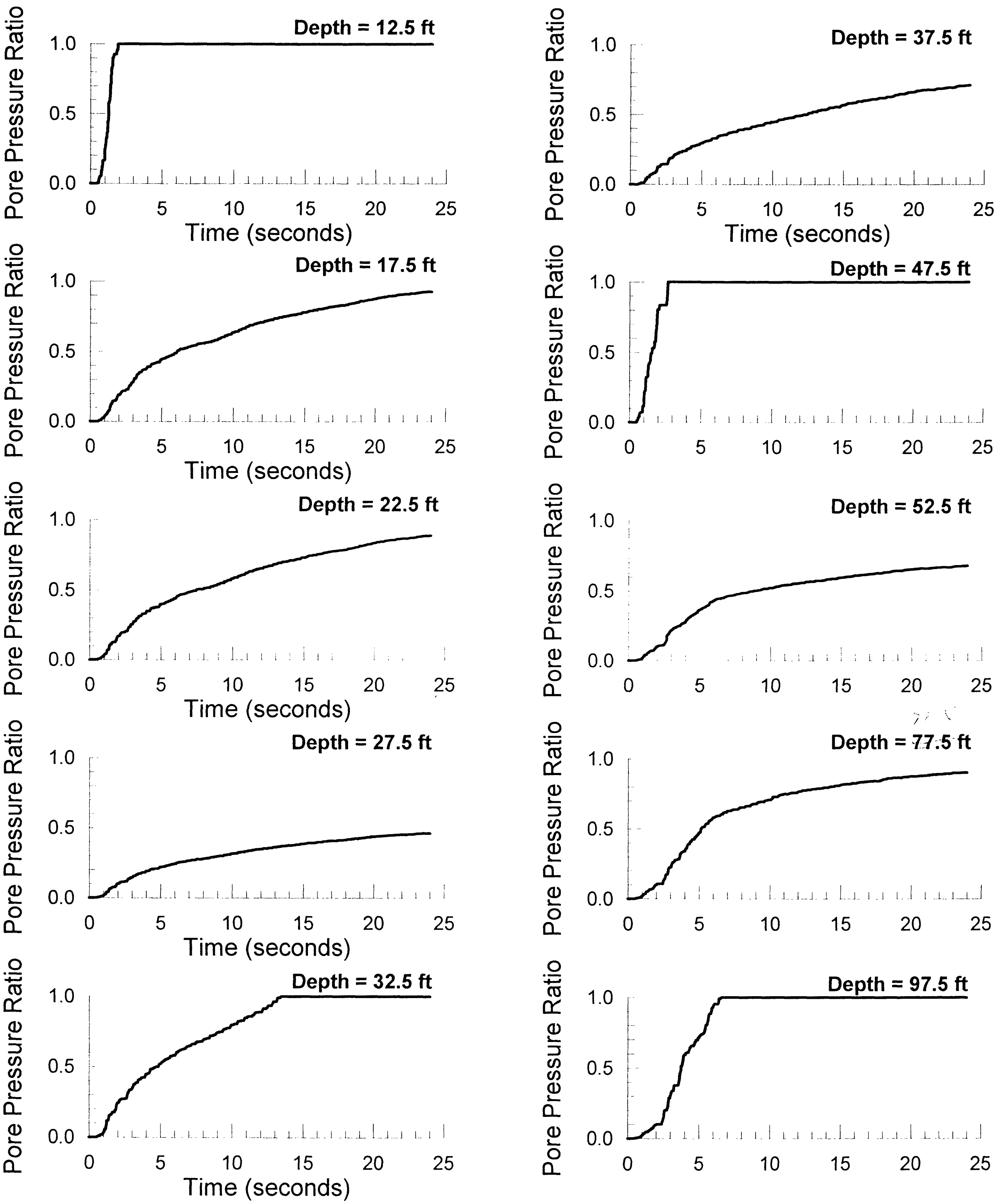


Figure 1c-3 Pore Pressure Generation
Case 1c: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP
No Pore Pressure Redistribution

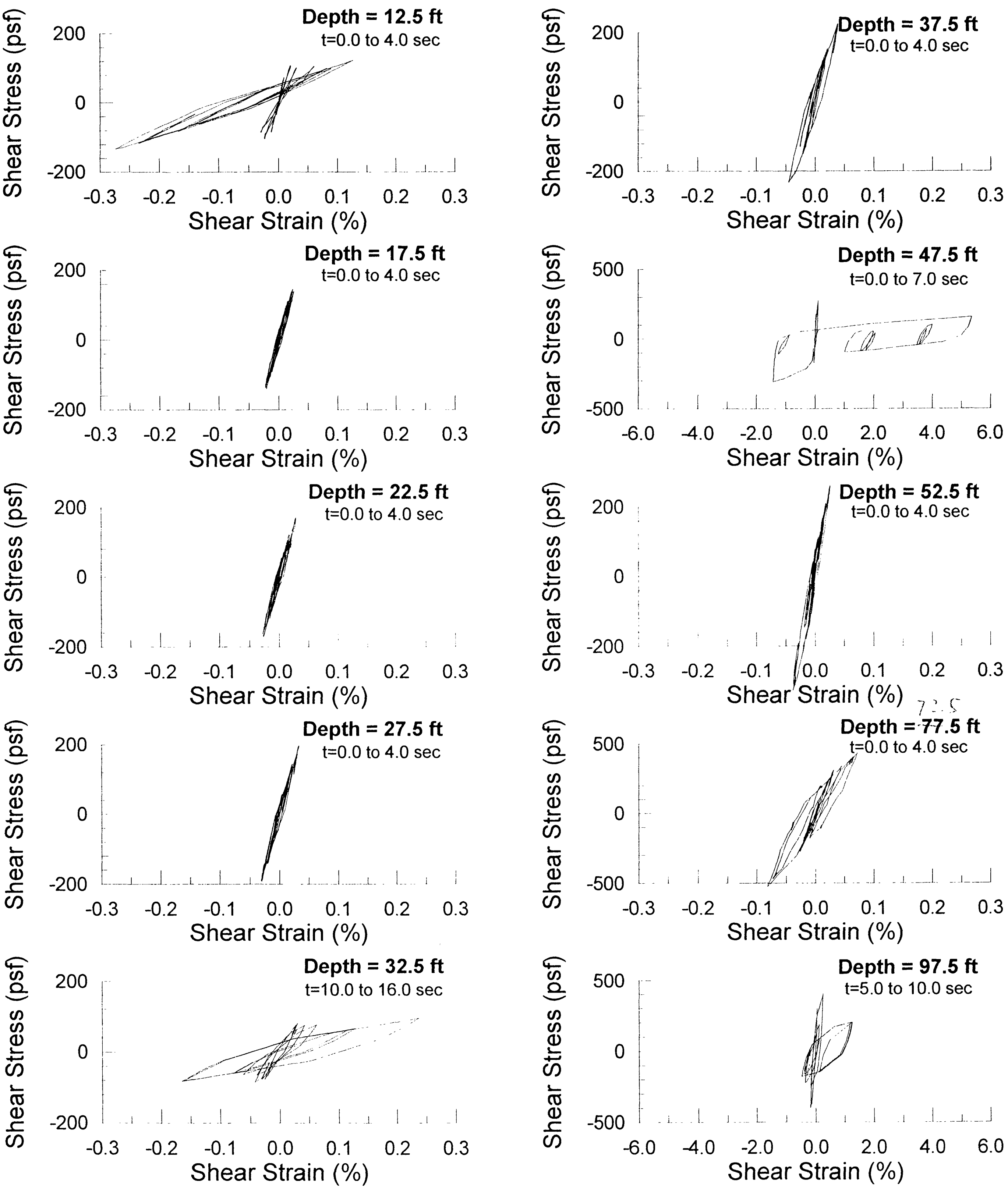


Figure 1c-4 Shear Stress-Shear Strain Loops
Case 1c: Soil Profile without Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP
No Pore Pressure Redistribution

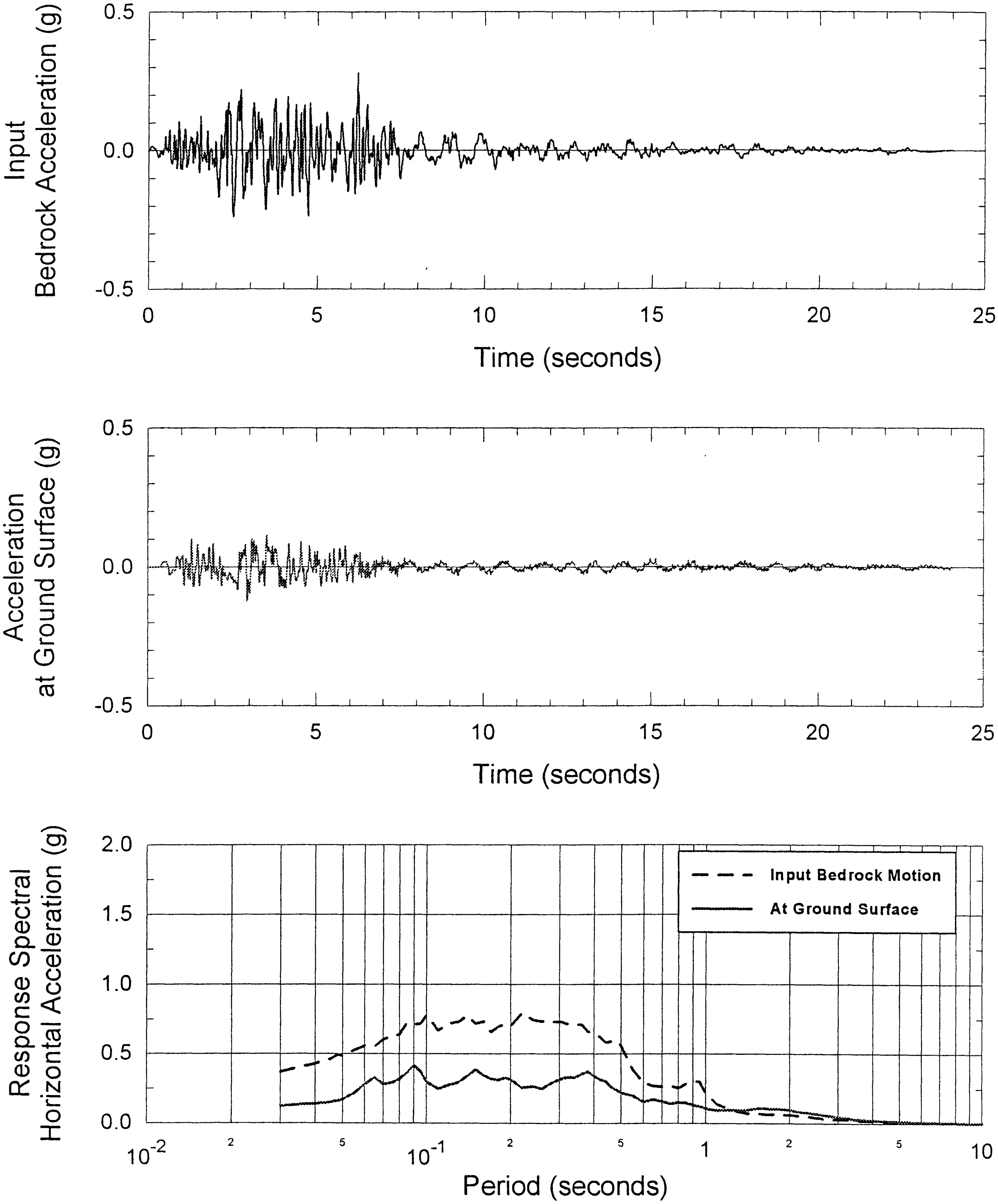


Figure 2a-1 Acceleration Time Histories and Response Spectra
Case 2a: Soil Profile with Fill, 1986 Desert Hot Springs EQ, 475-yr ARP

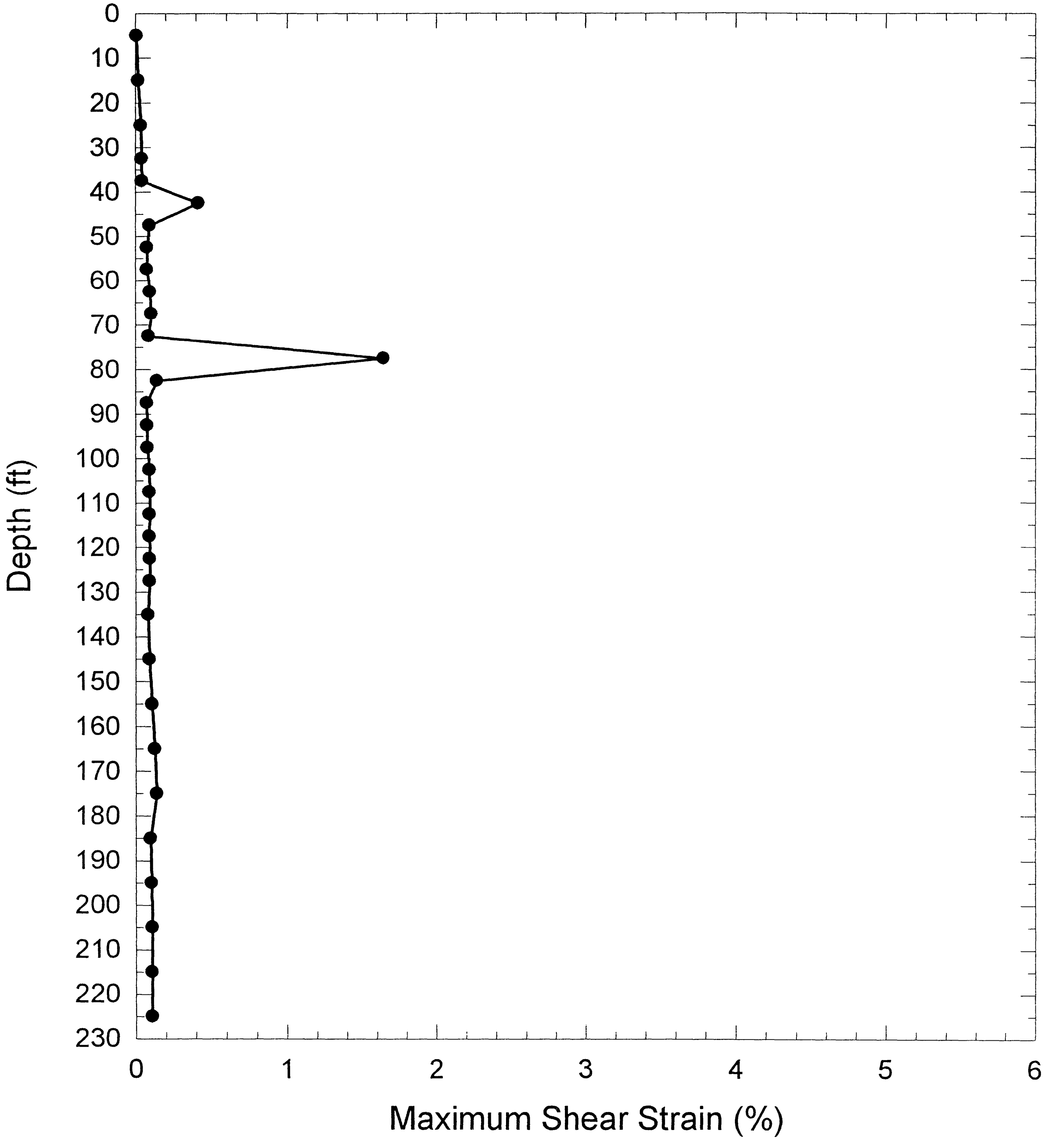


Figure 2a-2 Maximum Shear Strain Occurred During the Shaking
Case 2a: Soil Profile with Fill
1986 Desert Hot Springs EQ, 475-yr ARP

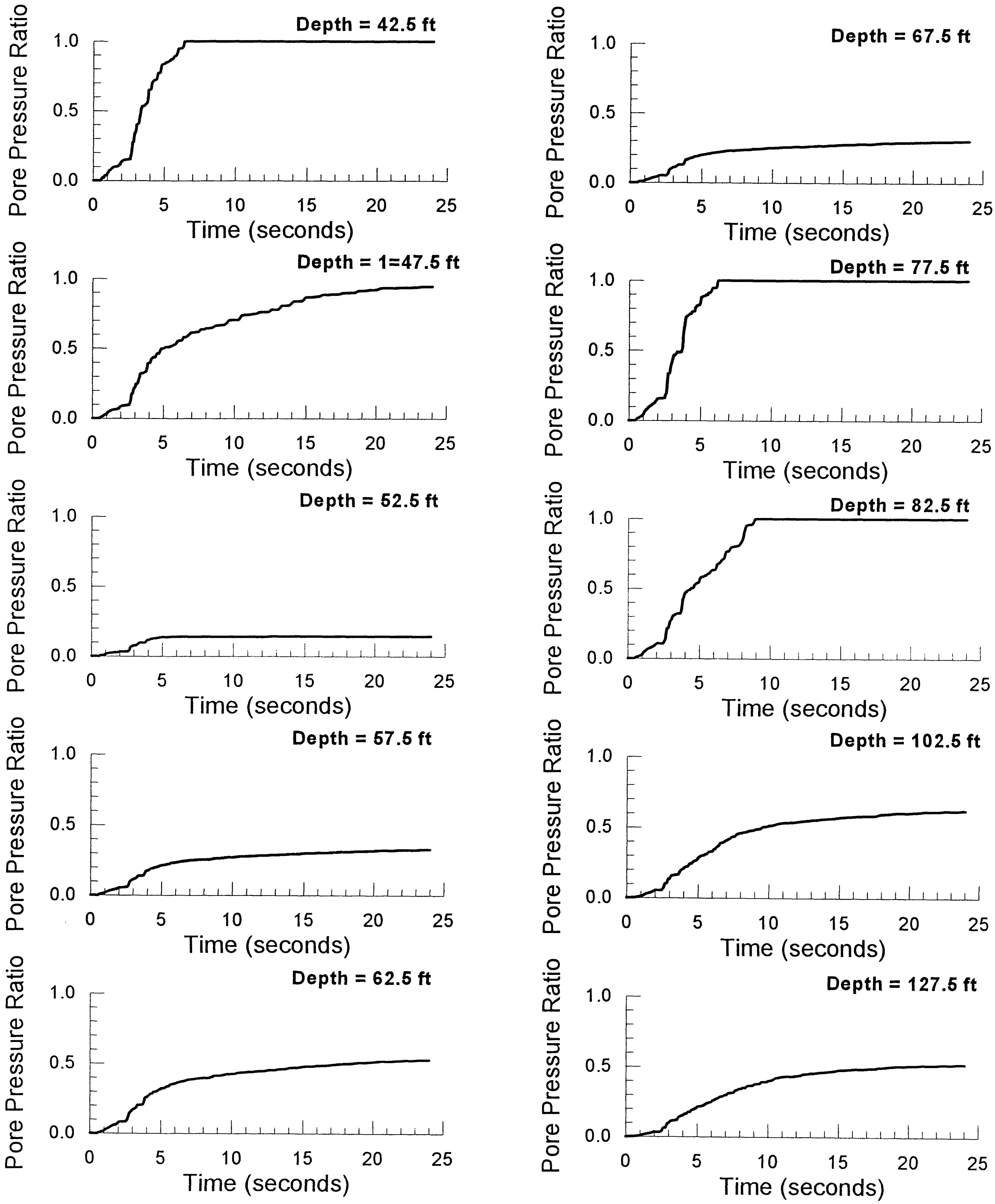


Figure 2a-3 Pore Pressure Generation
Case 2a: Soil Profile with Fill, 1986 Desert Hot Springs EQ, 475-yr ARP

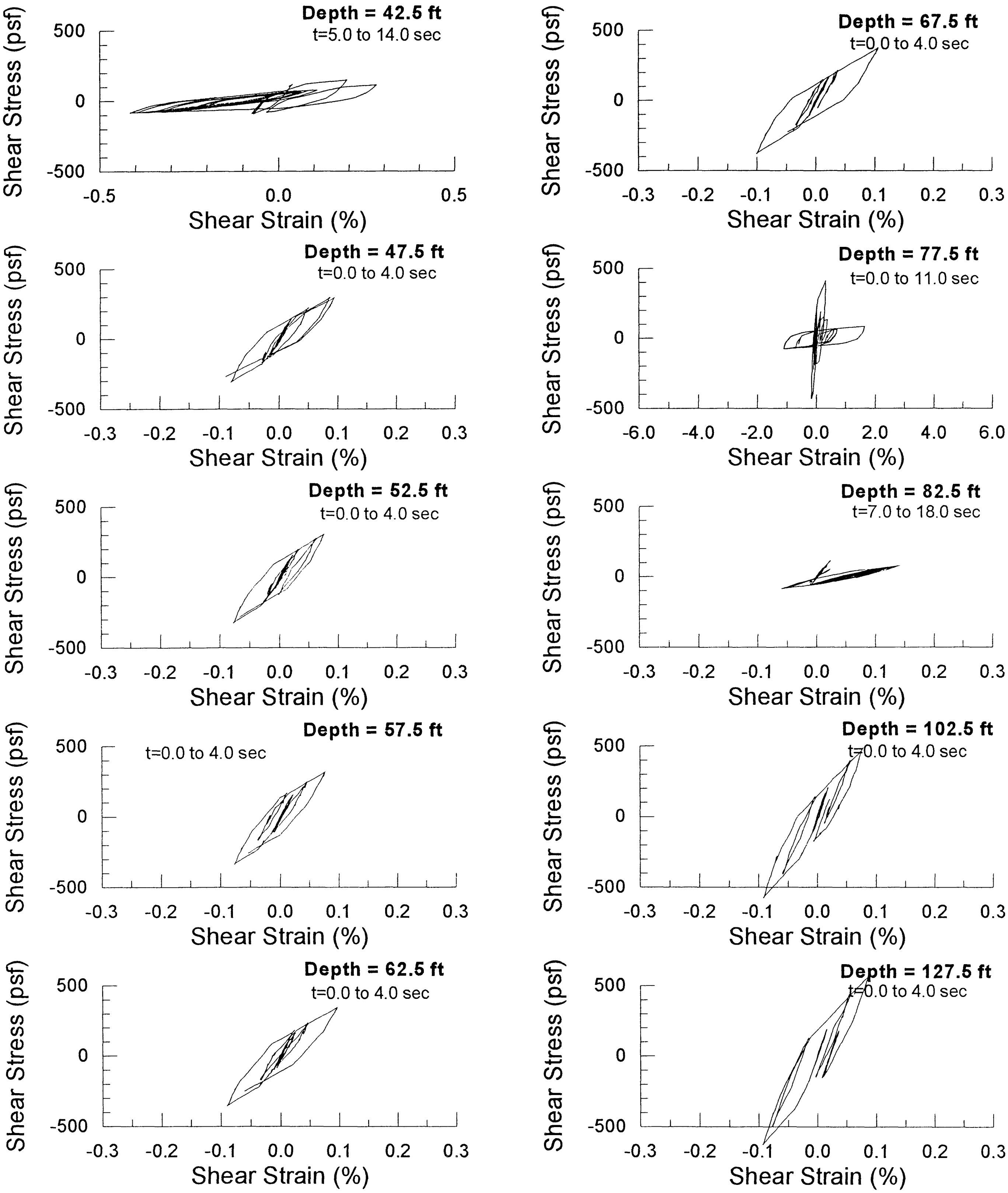


Figure 2a-4 Shear Stress-Shear Strain Loops
Case 2a: Soil Profile with Fill, 1986 Desert Hot Springs EQ, 475-yr ARP

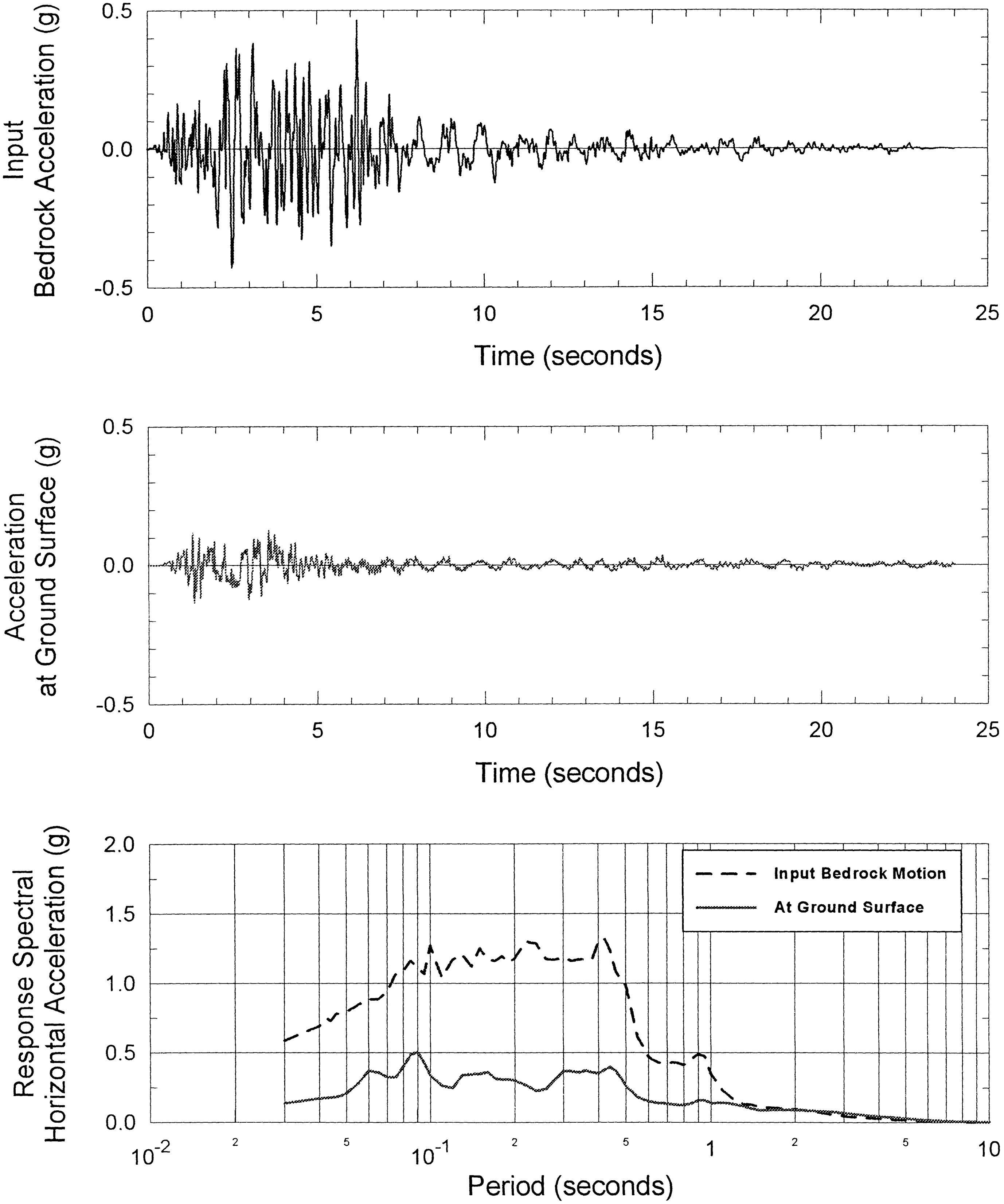


Figure 2b-1 Acceleration Time Histories and Response Spectra Case 2b: Soil Profile with Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP

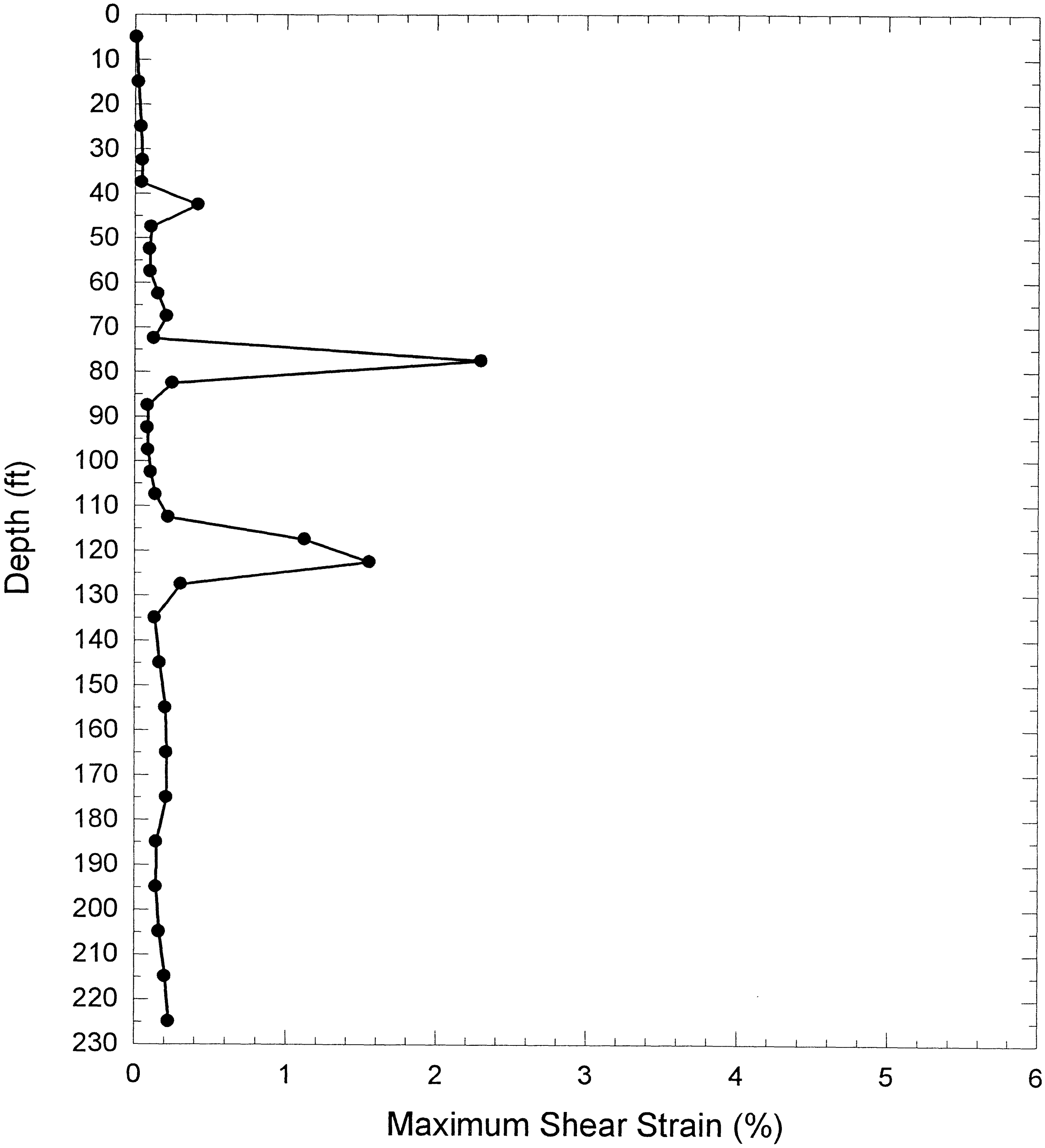


Figure 2b-2 Maximum Shear Strain Occurred During the Shaking
Case 2b: Soil Profile with Fill
1986 Desert Hot Springs EQ, 2475-yr ARP

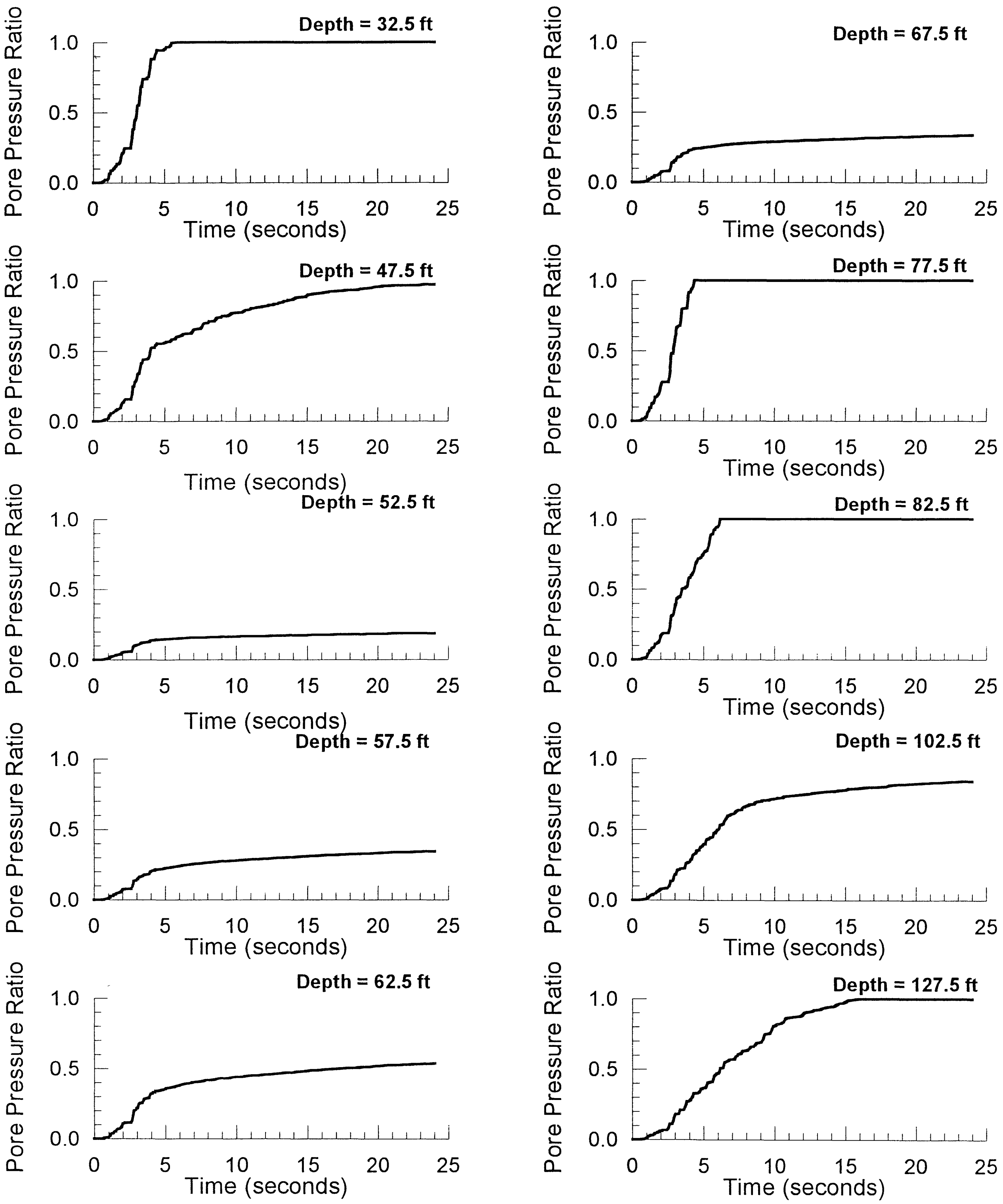


Figure 2b-3 Pore Pressure Generation
Case 2b: Soil Profile with Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP

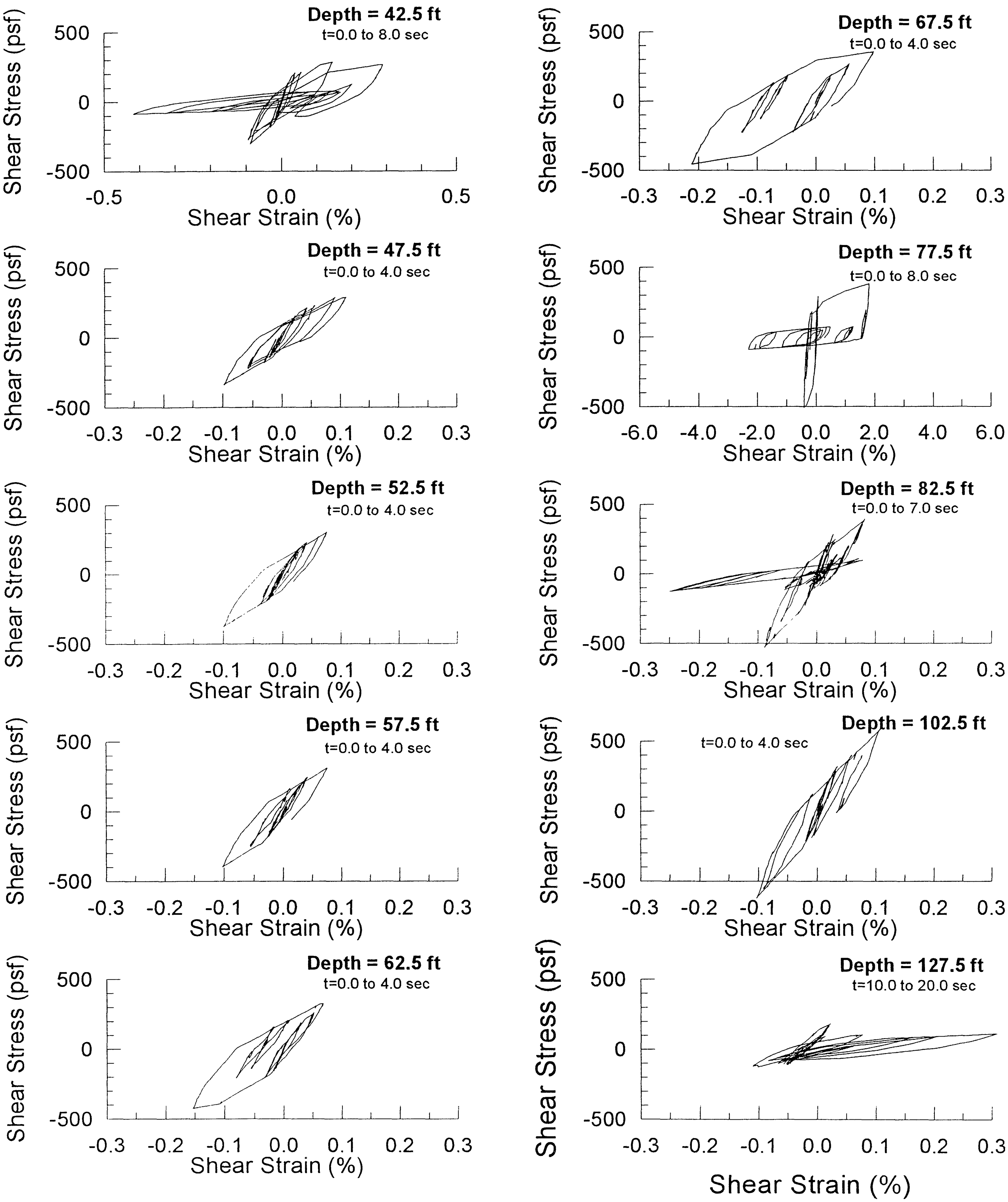


Figure 2b-4 Shear Stress-Shear Strain Loops
Case 2b: Soil Profile with Fill, 1986 Desert Hot Springs EQ, 2475-yr ARP

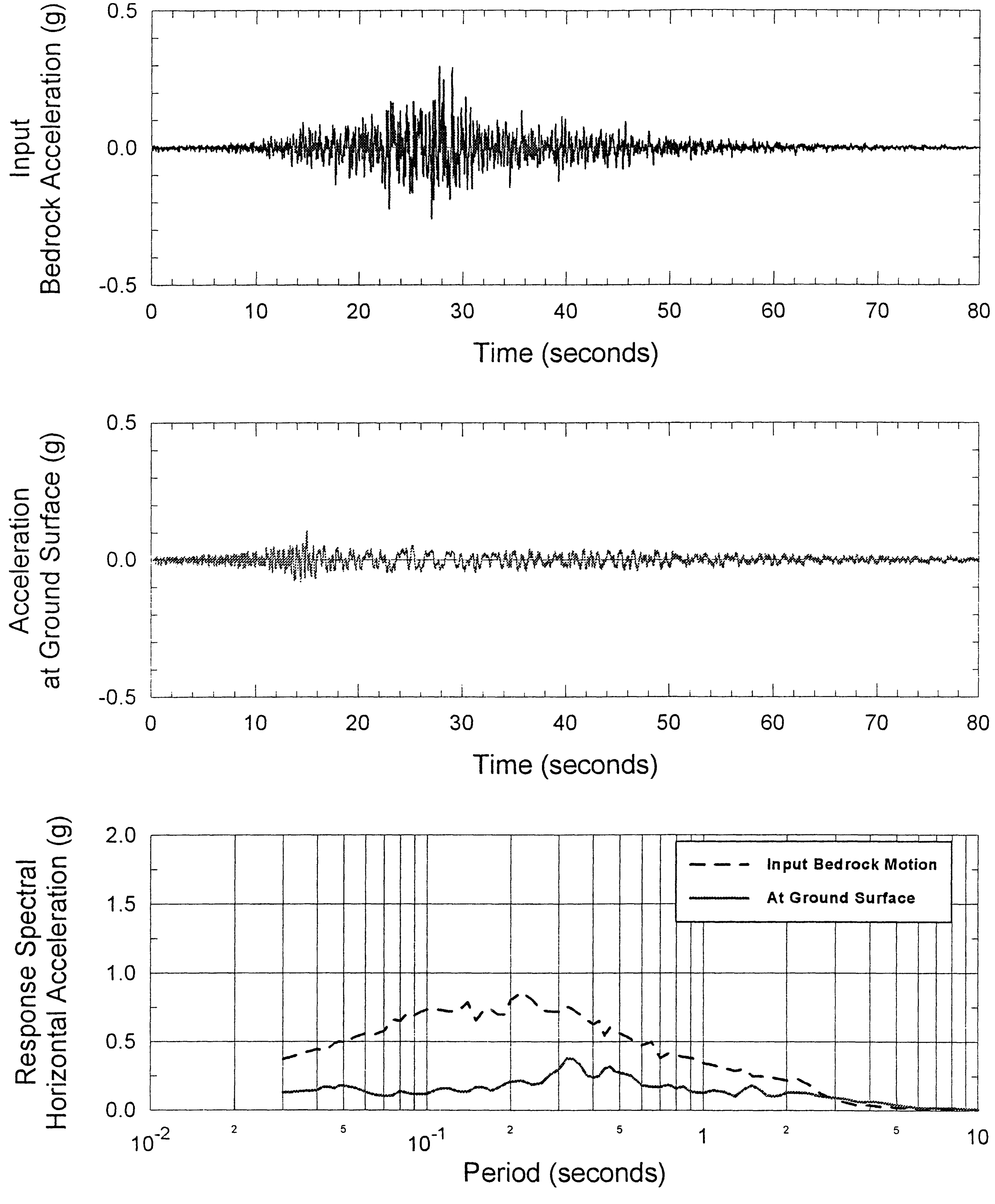


Figure 3a-1 Acceleration Time Histories and Response Spectra Case 3a: Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP

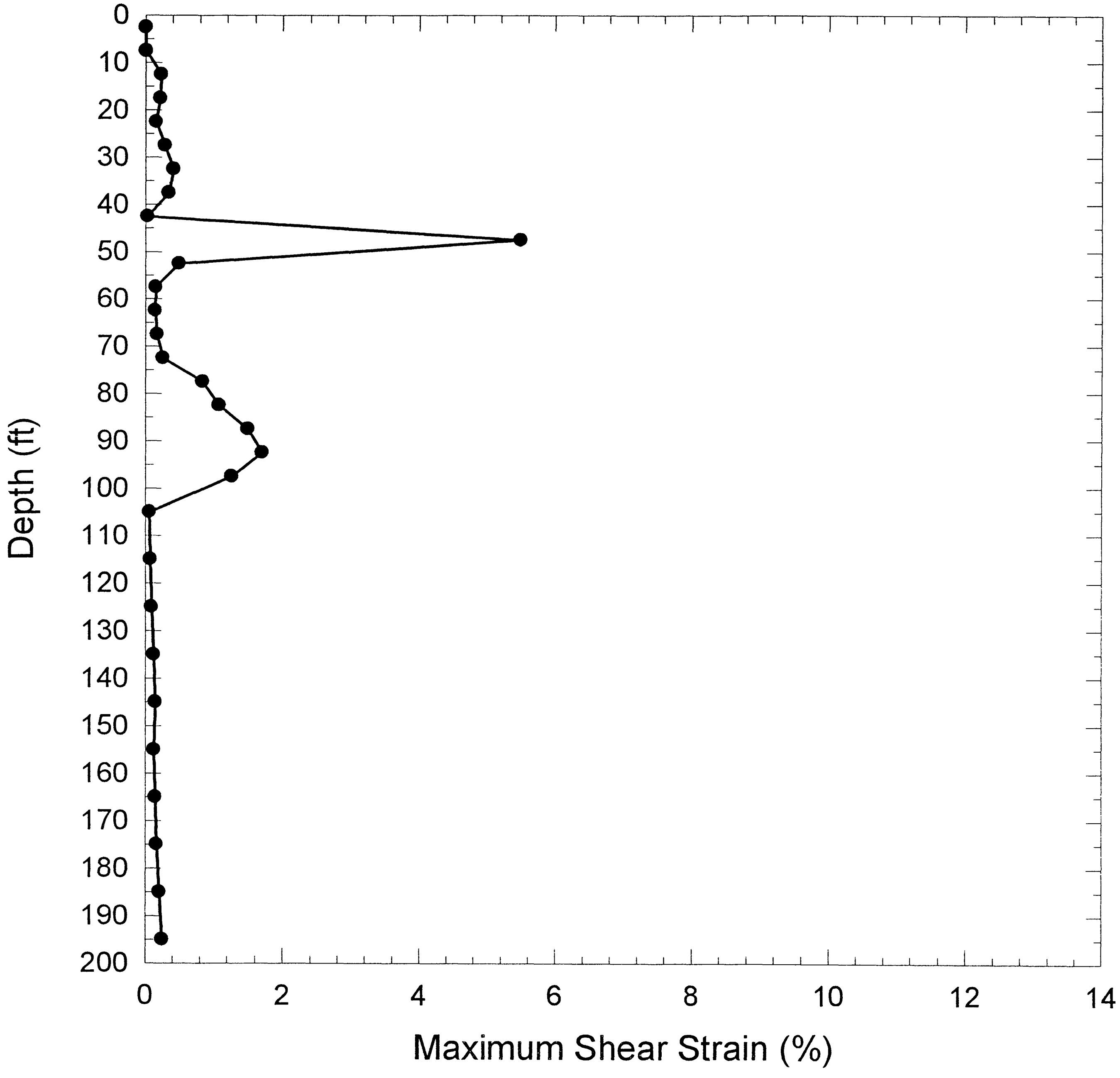


Figure 3a-2 Maximum Shear Strain Occurred During the Shaking
Case 3a: Soil Profile without Fill
1985 Chile EQ, 475-yr ARP

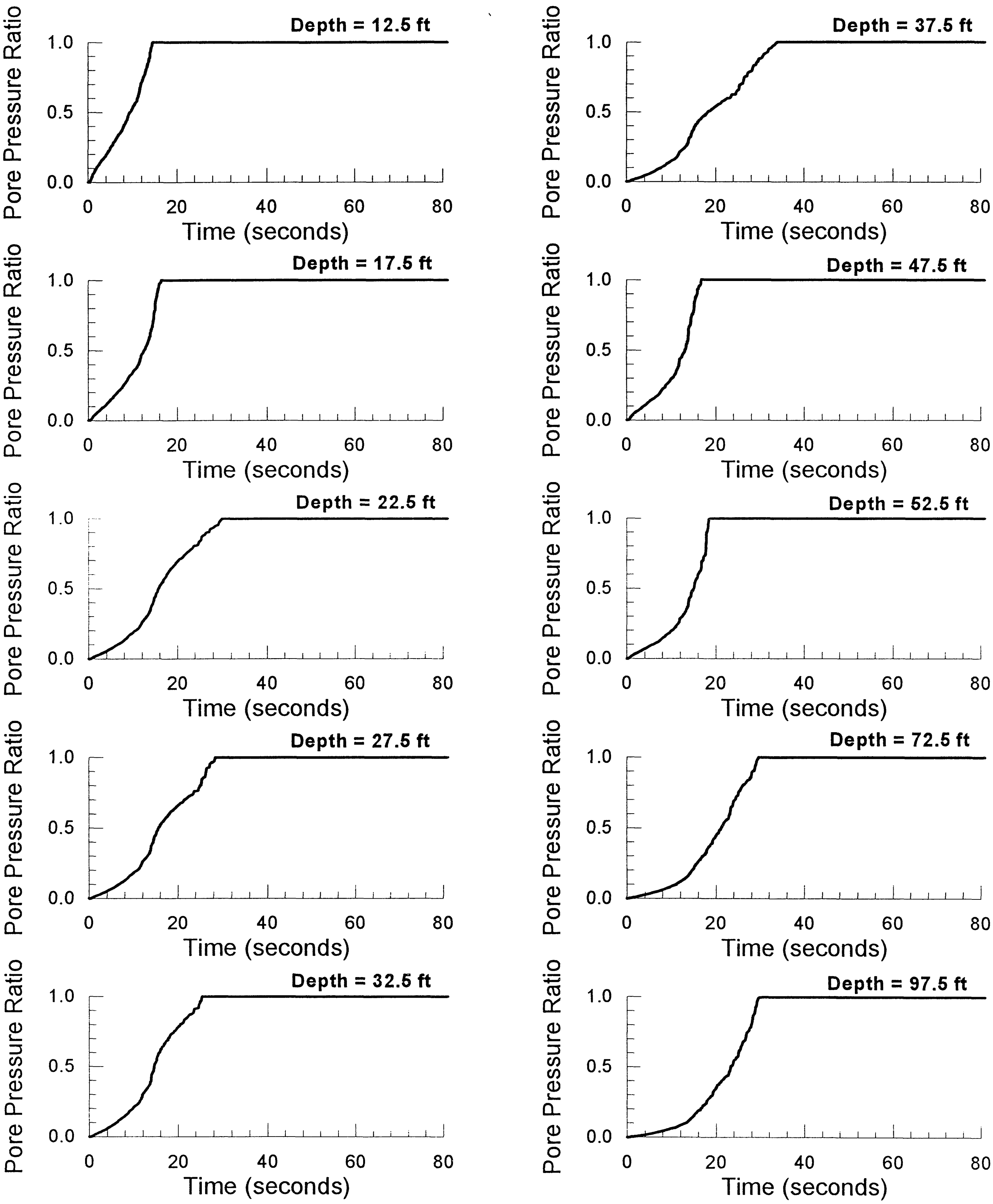


Figure 3a-3 Pore Pressure Generation
Case 3a: Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP

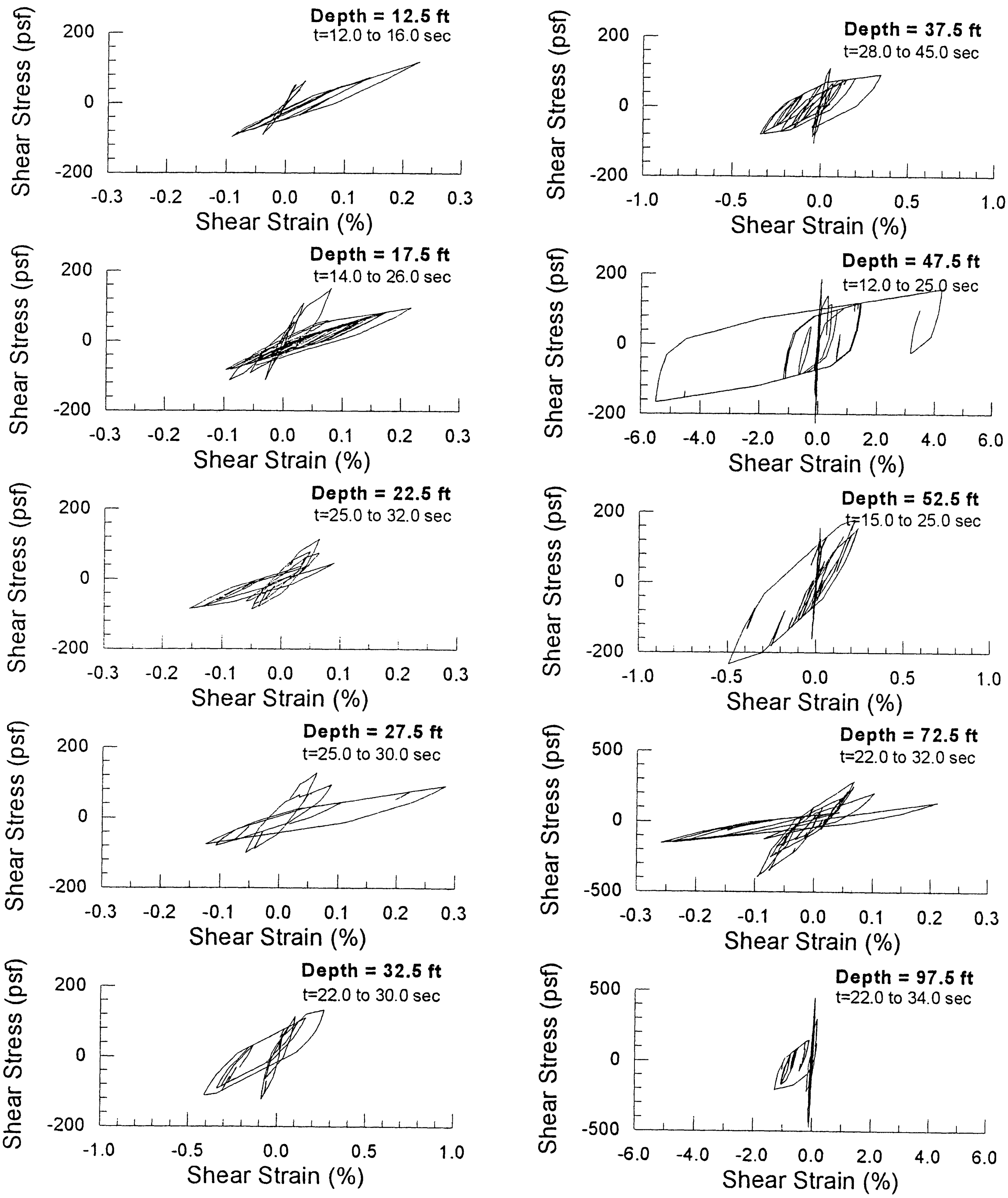


Figure 3a-4 Shear Stress-Shear Strain Loops
Case 3a: Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP

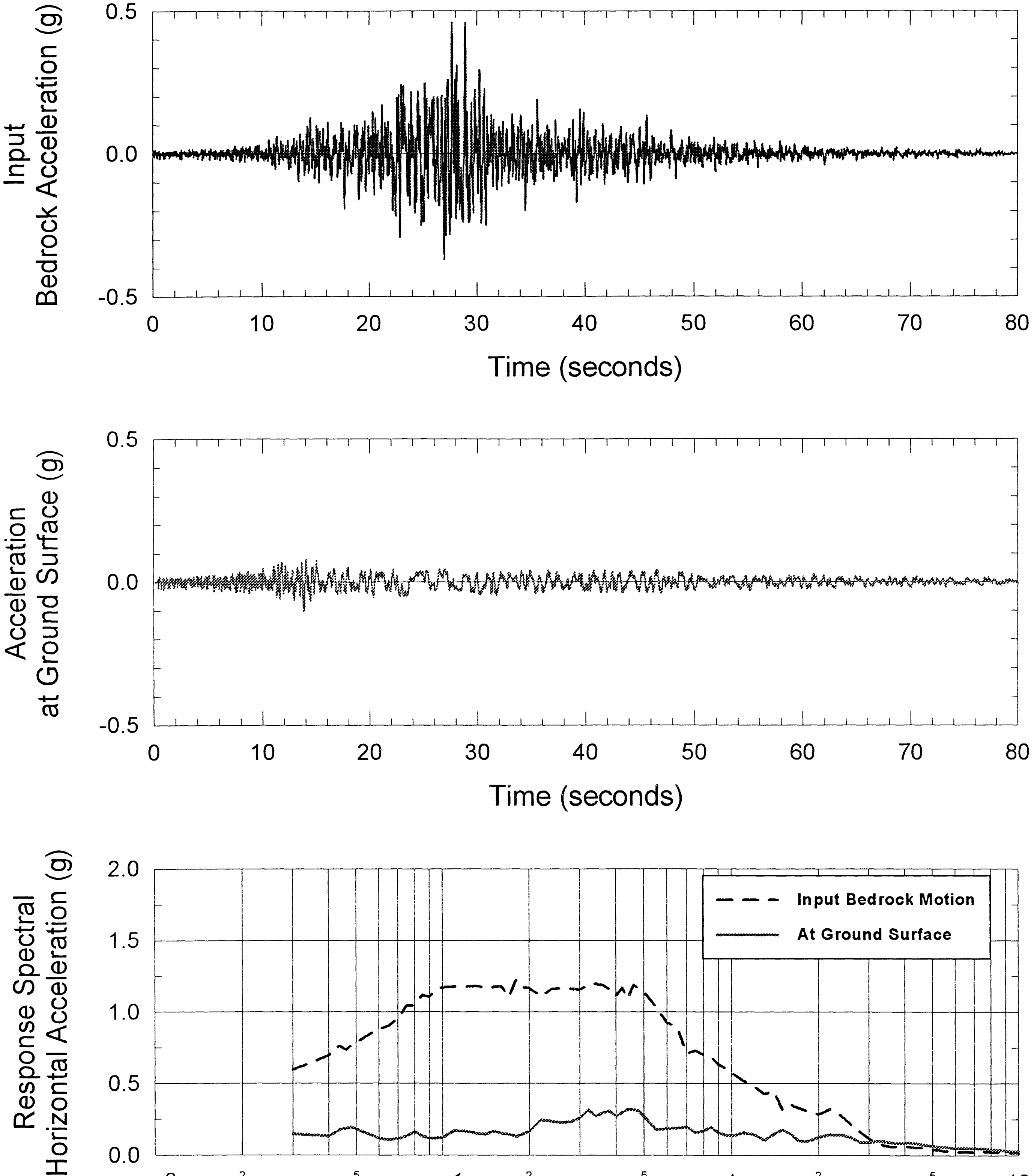


Figure 3b-1 Acceleration Time Histories and Response Spectra Case 3b: Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP

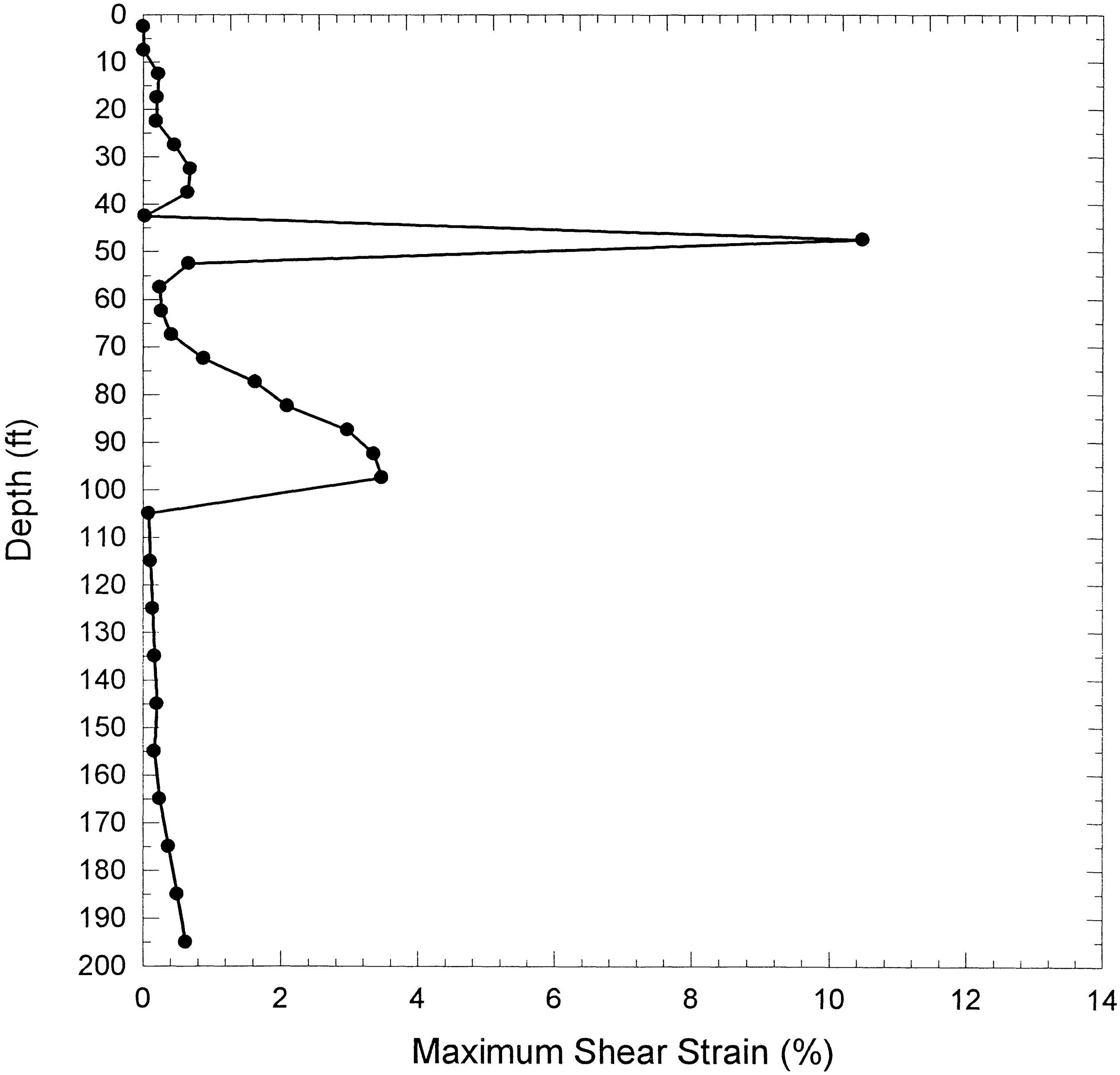


**Figure 3b-2 Maximum Shear Strain Occurred During the Shaking
Case 3b: Soil Profile without Fill
1985 Chile EQ, 2475-yr ARP**

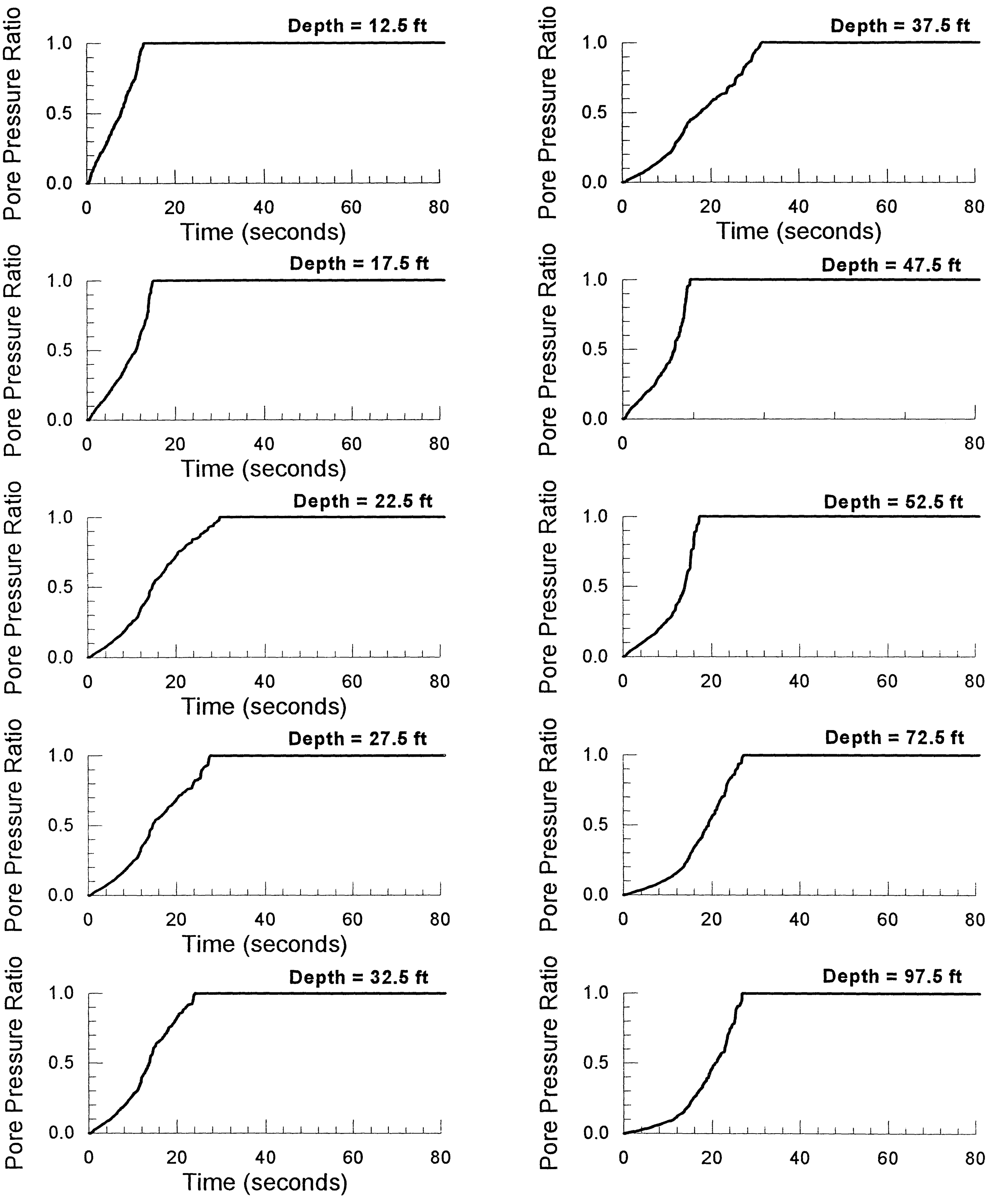


Figure 3b-3 Pore Pressure Generation
Case 3b: Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP

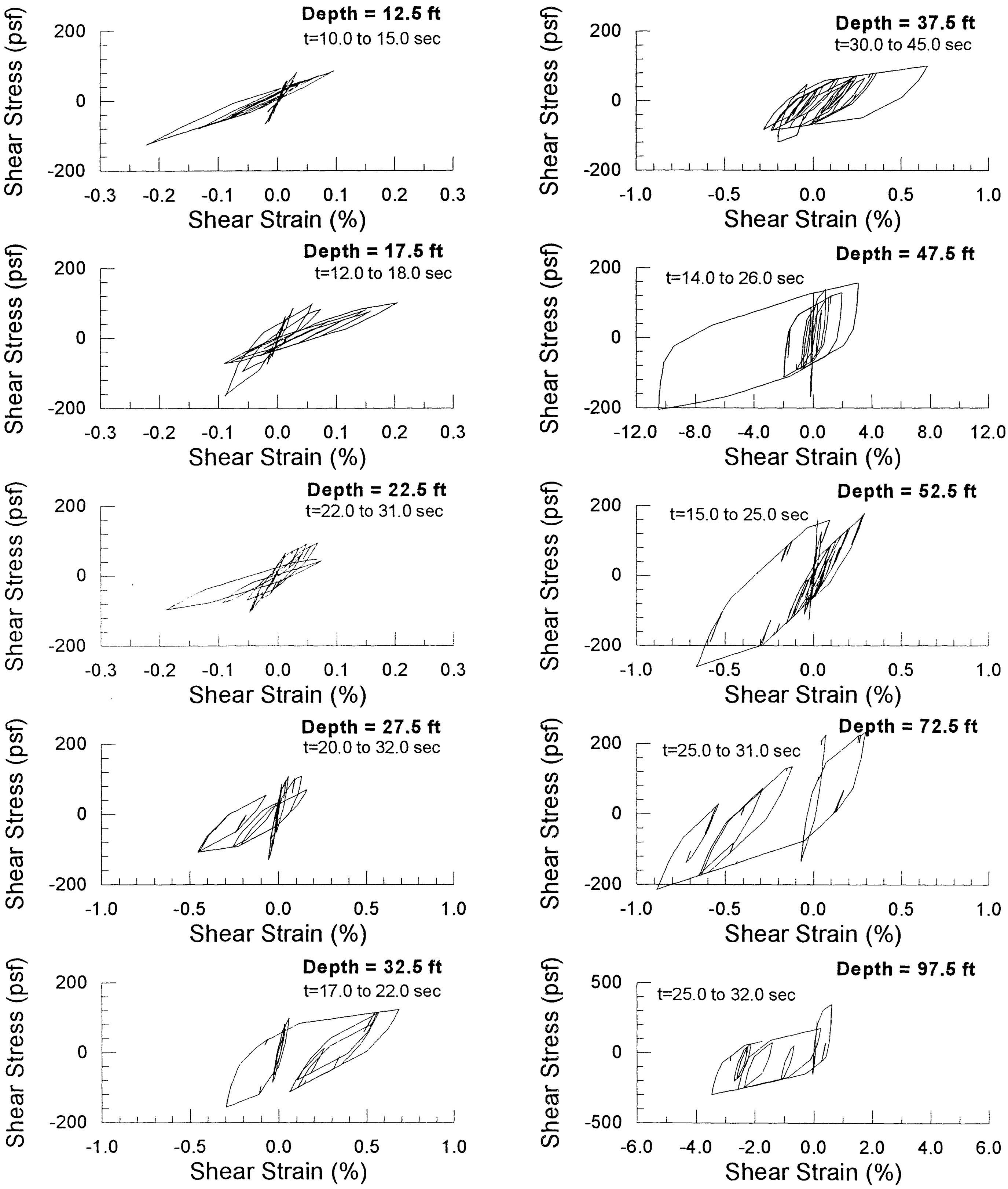


**Figure 3b-4 Shear Stress-Shear Strain Loops
Case 3b: Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP**

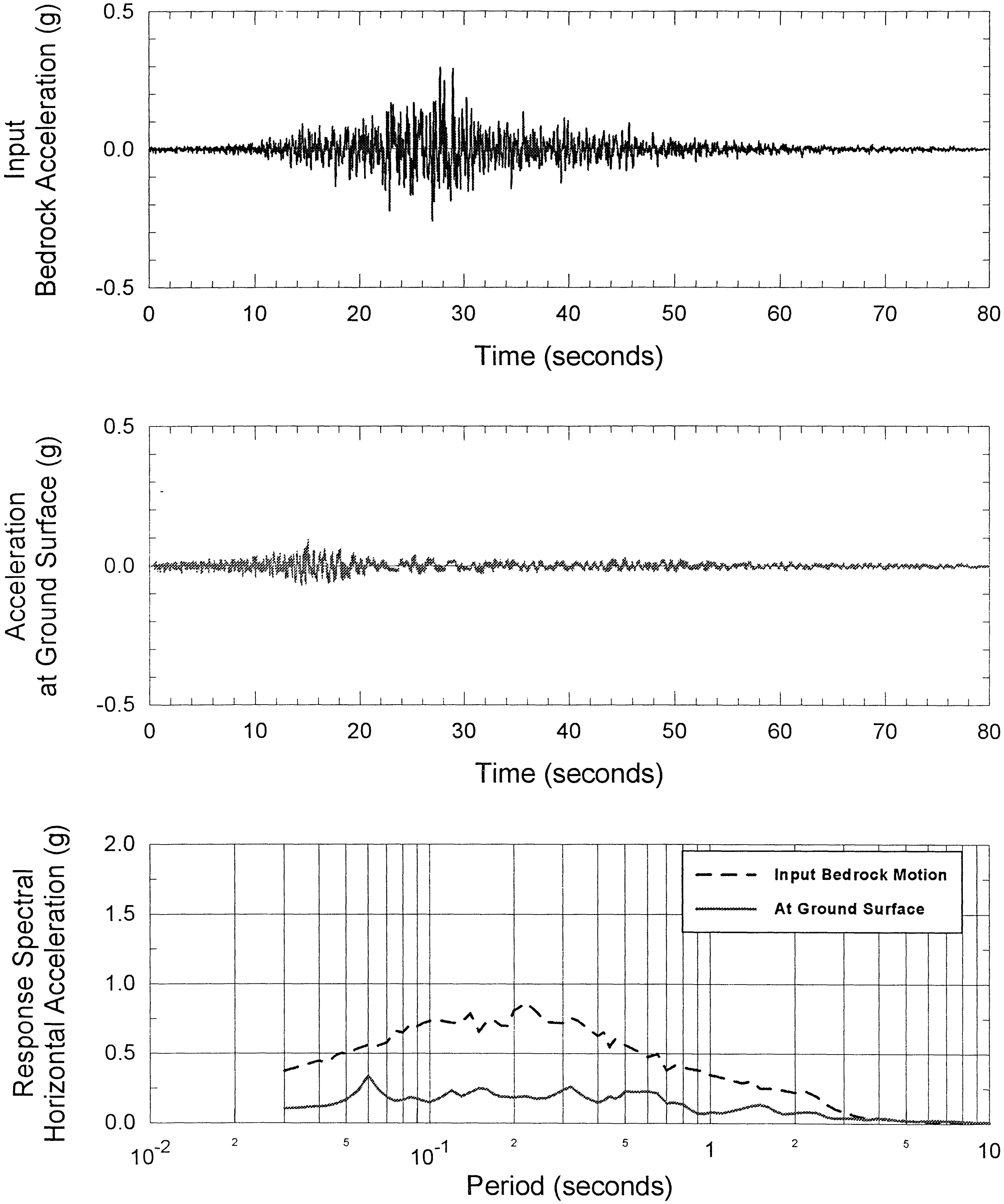


Figure 4a-1 Acceleration Time Histories and Response Spectra Case 4a: Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP

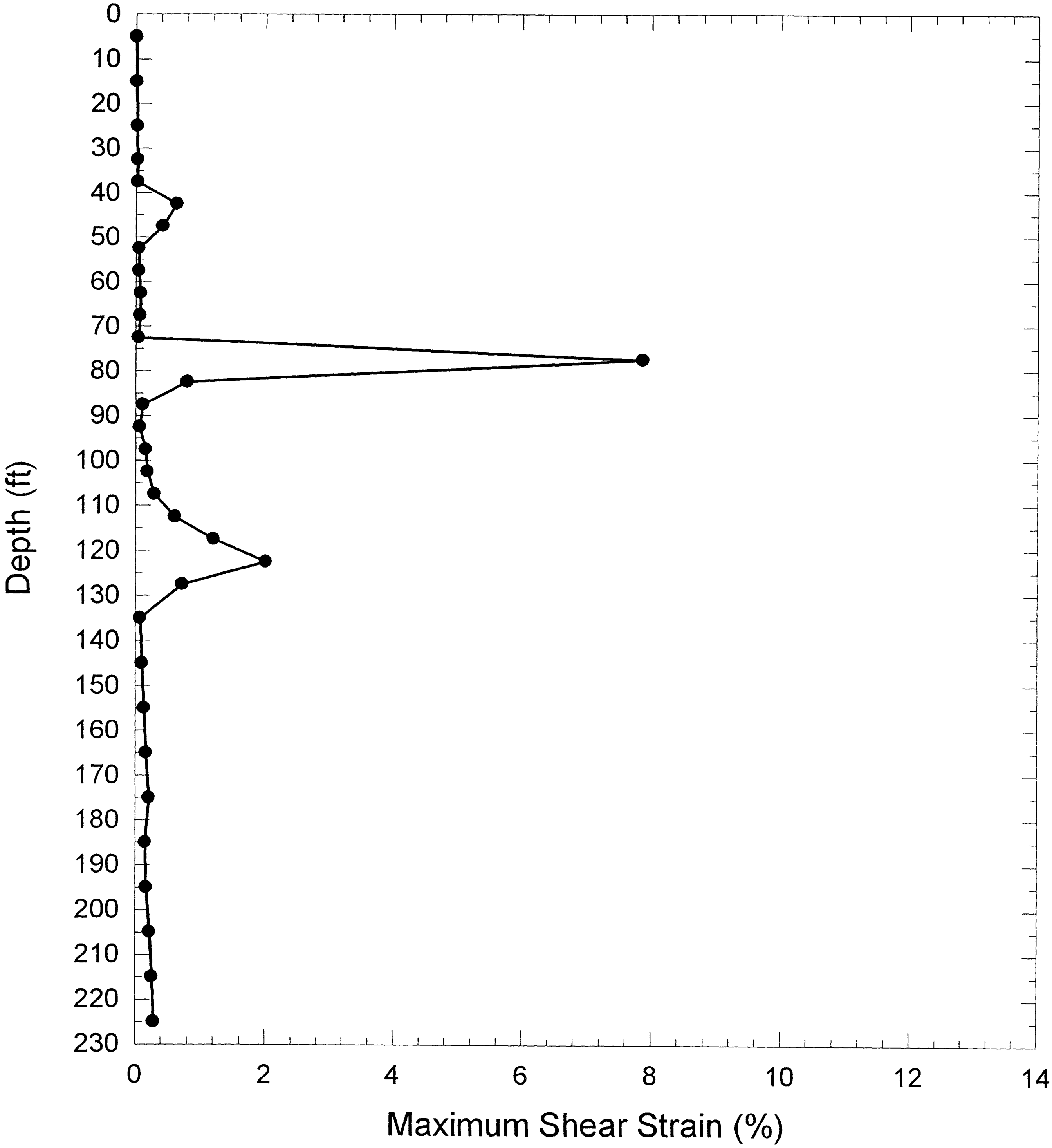


**Figure 4a-2 Maximum Shear Strain Occurred During the Shaking
Case 4a: Soil Profile with Fill
1985 Chile EQ, 475-yr ARP**

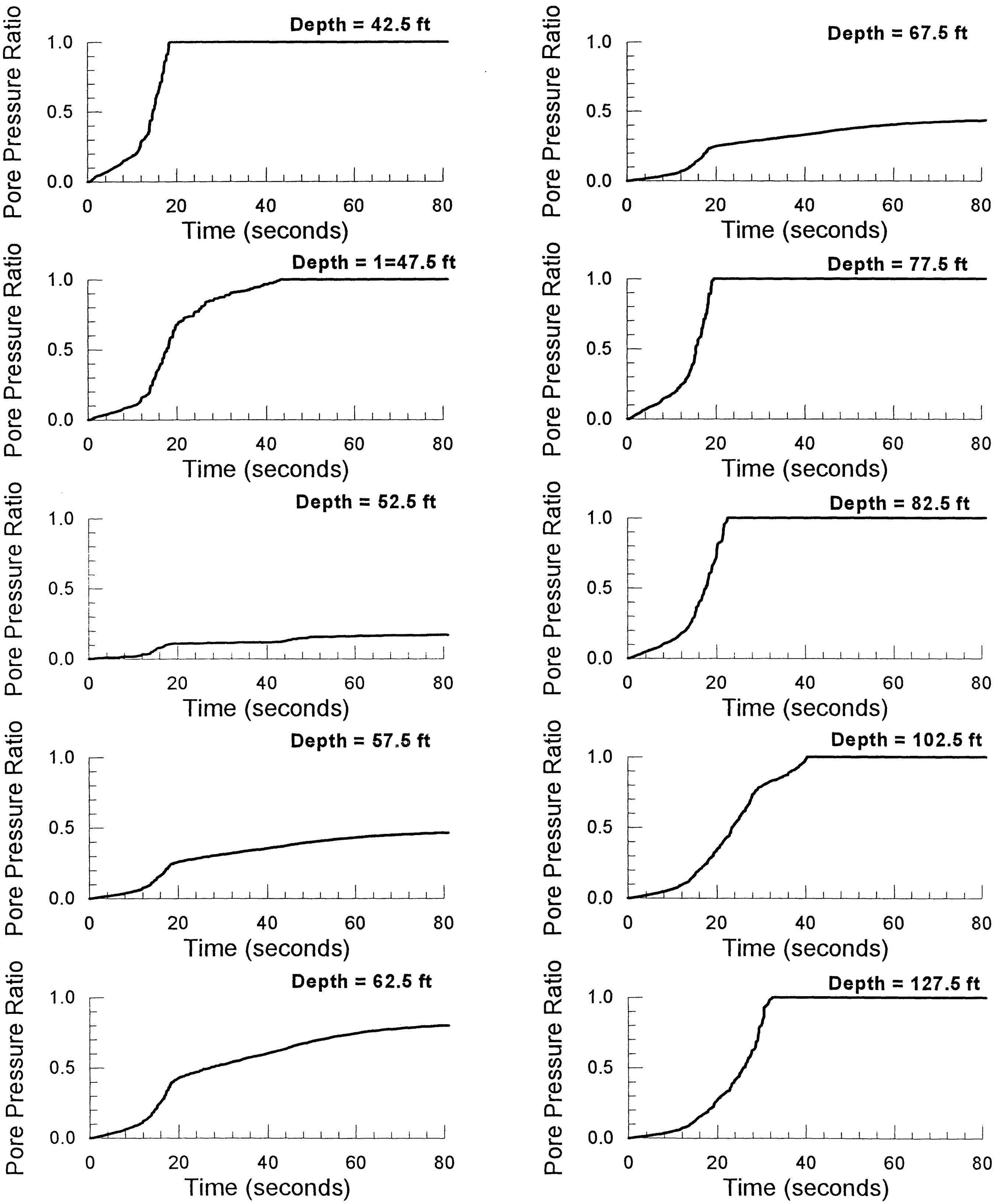


Figure 4a-3 Pore Pressure Generation
Case 4a: Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP

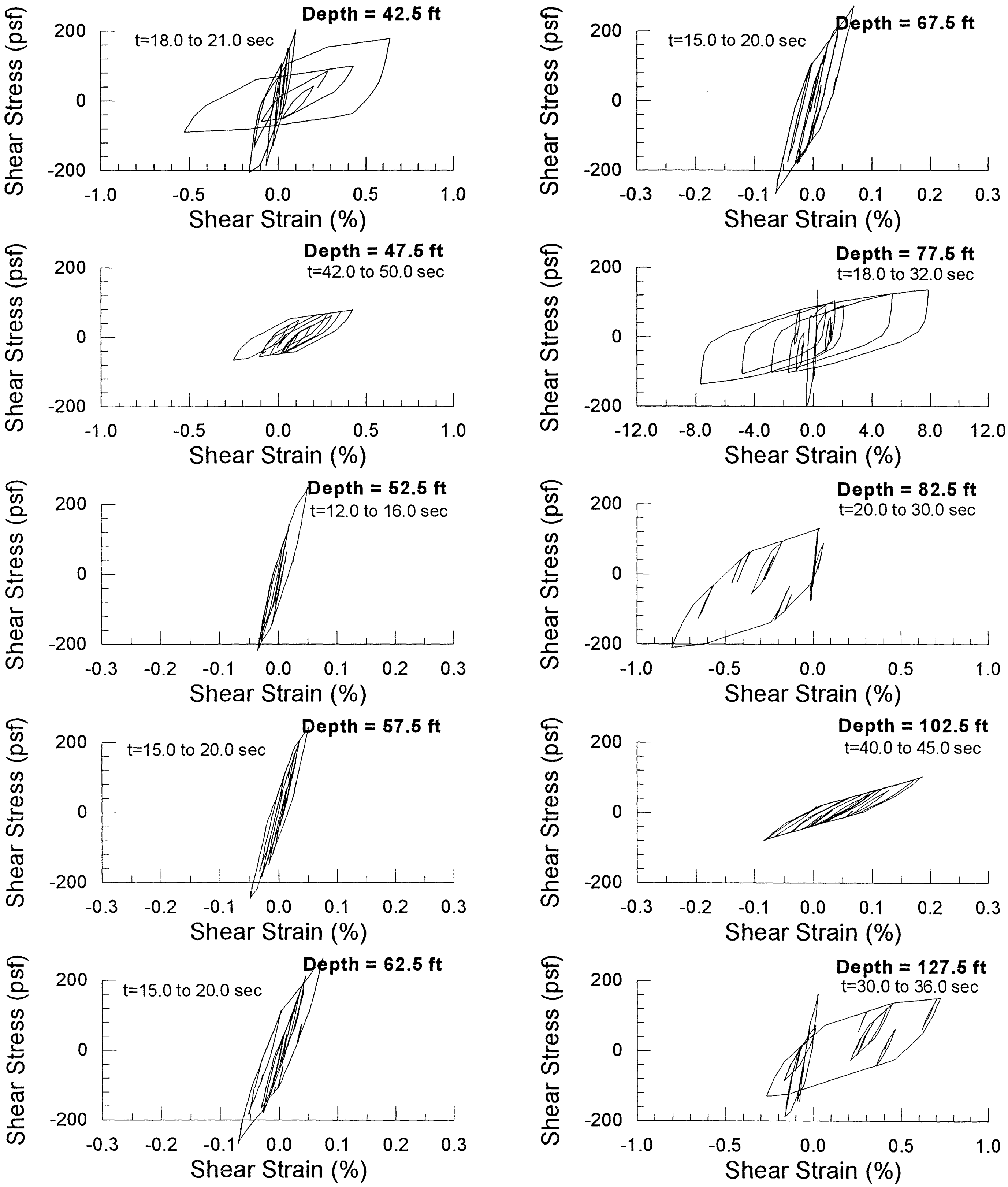


Figure 4a-4 Shear Stress-Shear Strain Loops
Case 4a: Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP

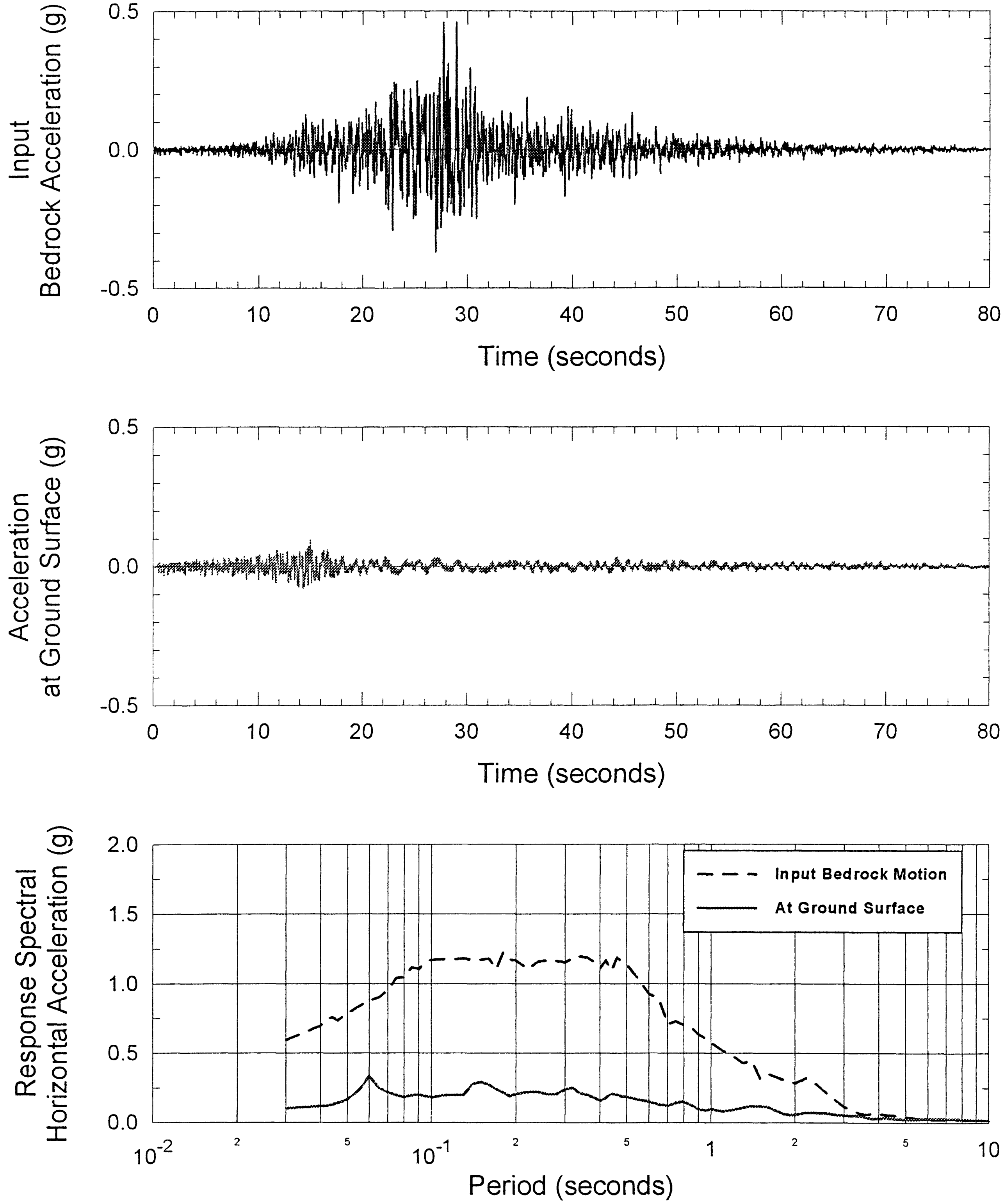


Figure 4b-1 Acceleration Time Histories and Response Spectra
Case 4b: Soil Profile with Fill, 1985 Chile EQ, 2475-yr ARP

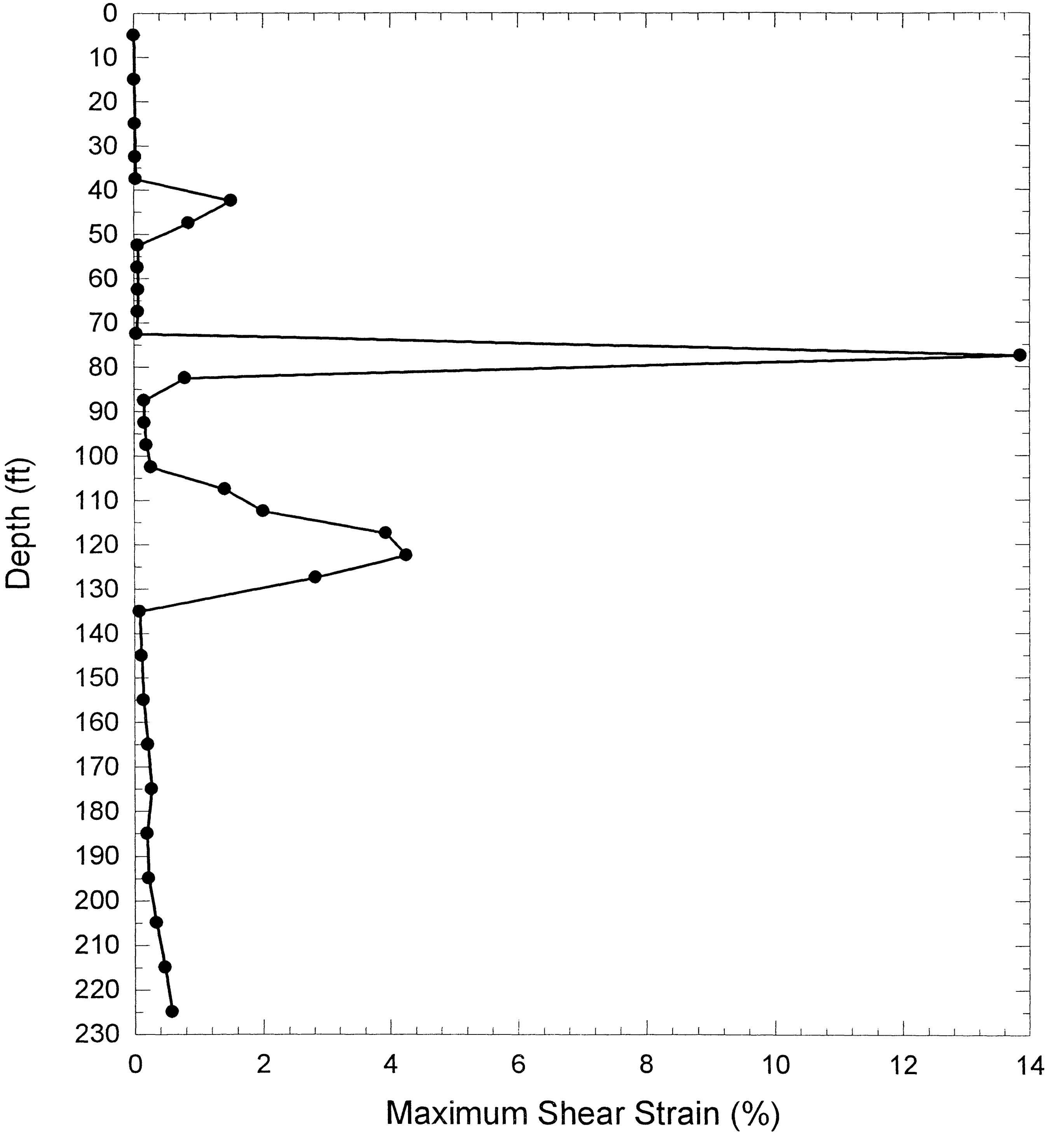


**Figure 4b-2 Maximum Shear Strain Occurred During the Shaking
Case 4b: Soil Profile with Fill
1985 Chile EQ, 2475-yr ARP**

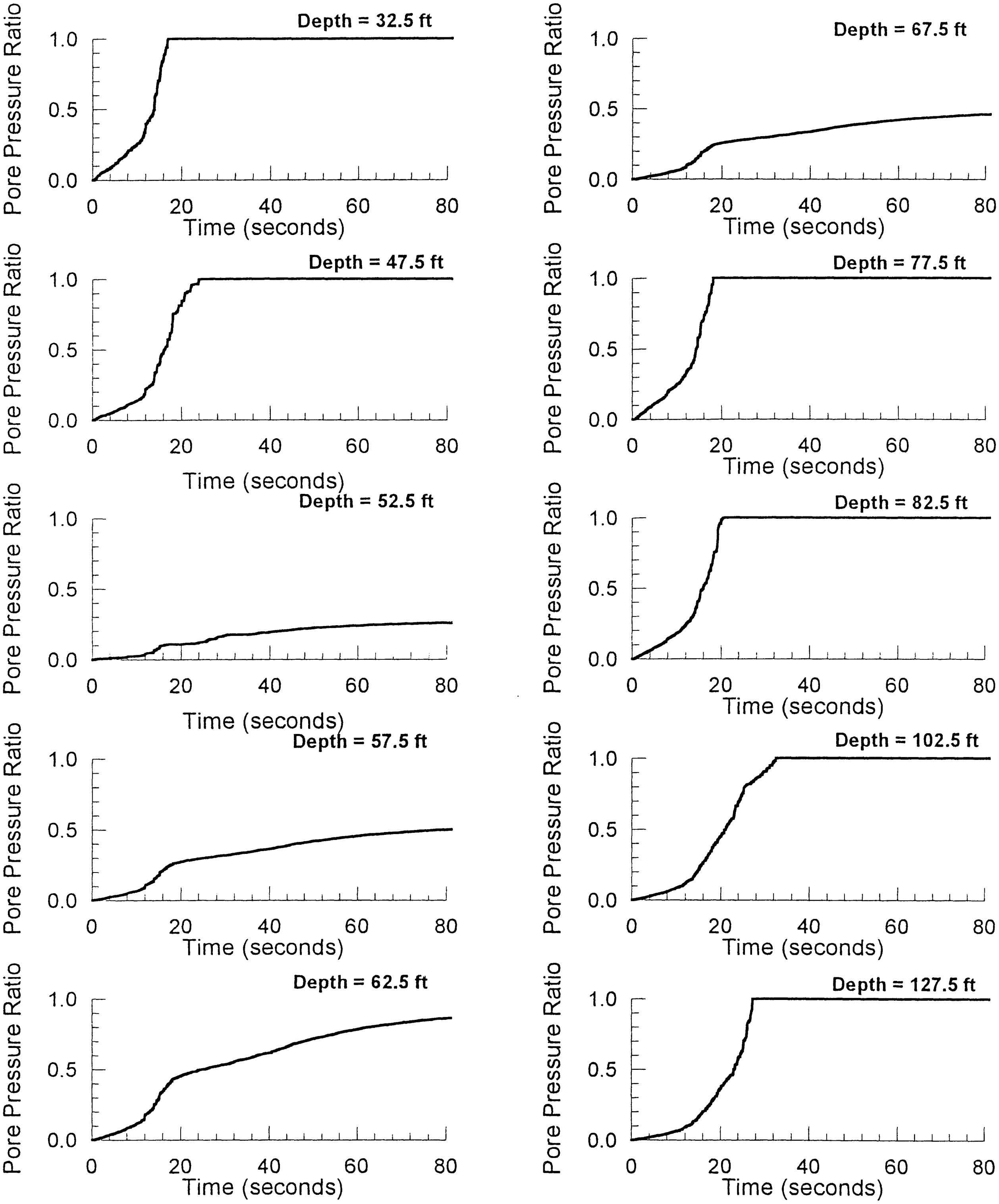


Figure 4b-3 Pore Pressure Generation
Case 4b: Soil Profile with Fill, 1985 Chile EQ, 2475-yr ARP

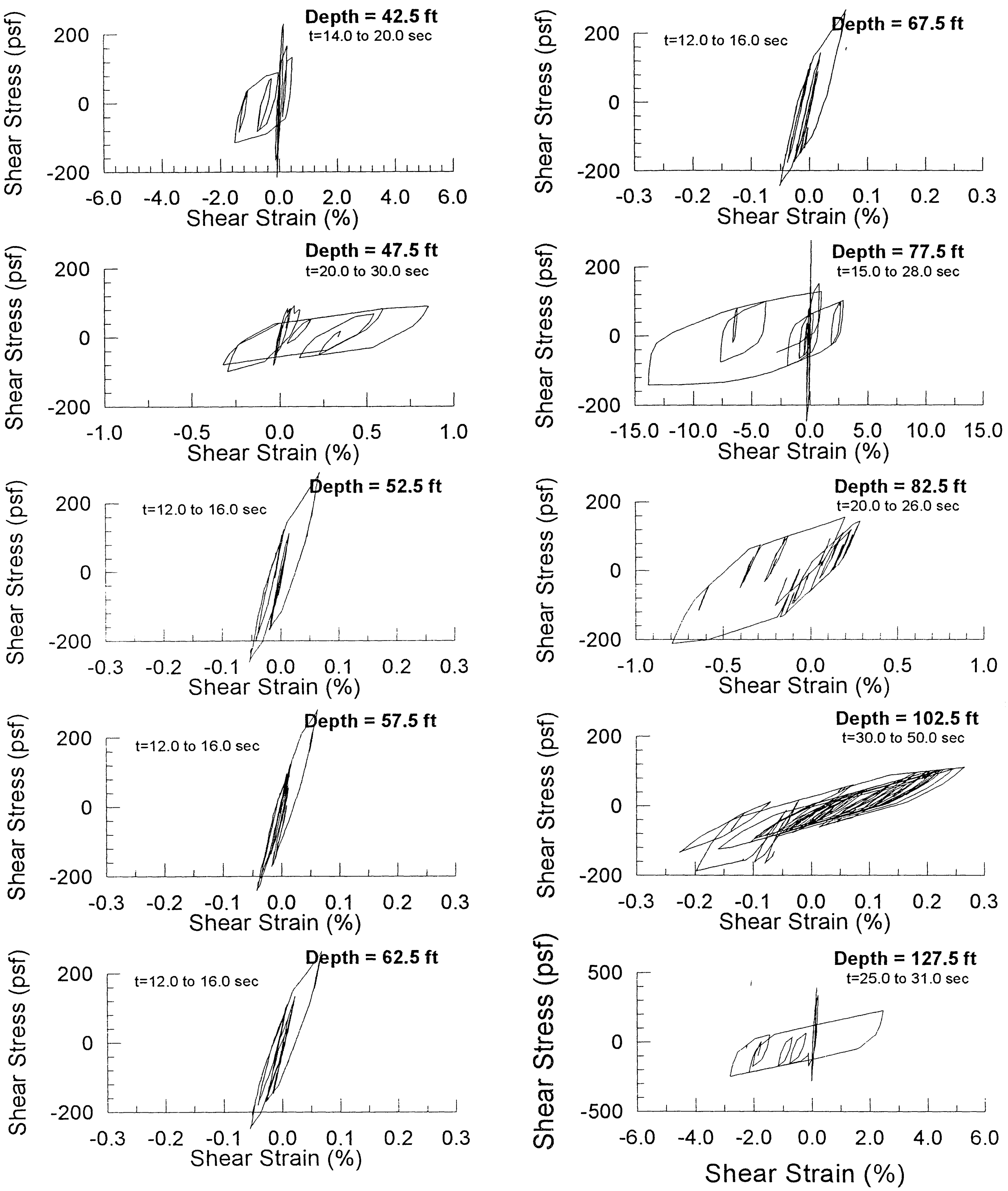


Figure 4b-4 Shear Stress-Shear Strain Loops
Case 4b: Soil Profile with Fill, 1985 Chile EQ, 2475-yr ARP

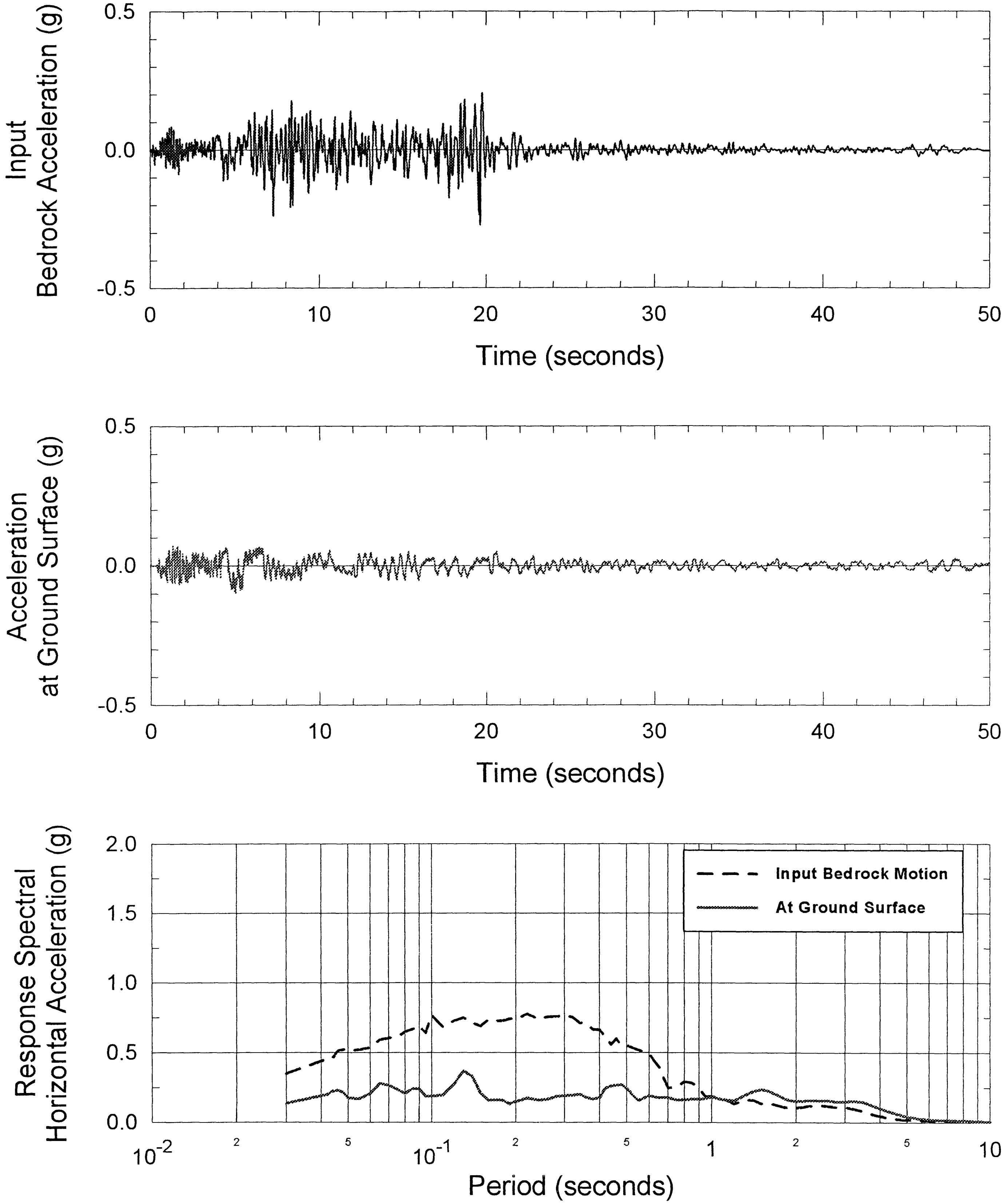


Figure 5a-1 Acceleration Time Histories and Response Spectra
Case 5a: Soil Profile without Fill, 1949 Olympia EQ, 475-yr ARP

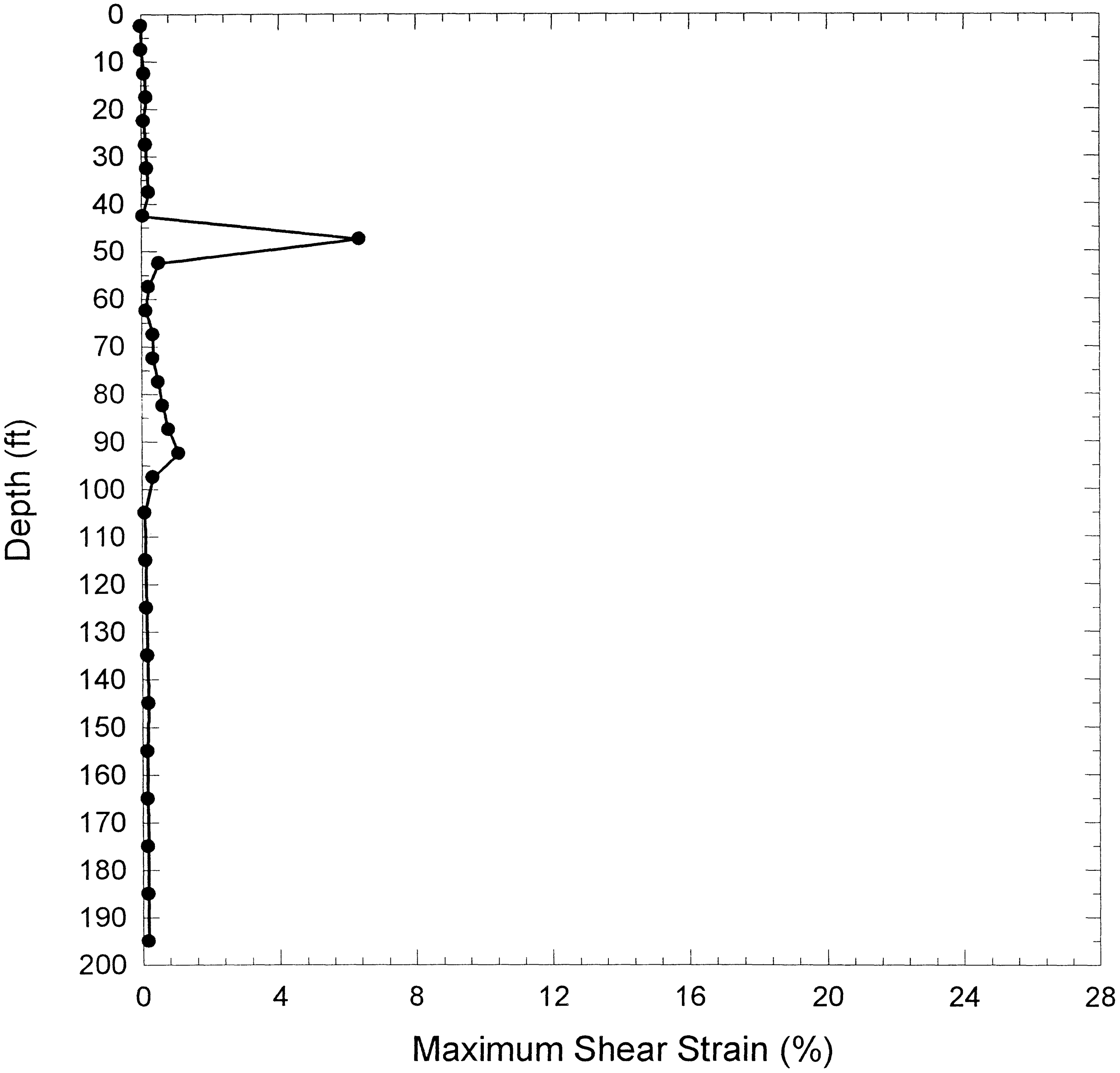


Figure 5a-2 Maximum Shear Strain Occurred During the Shaking Case 5a: Soil Profile without Fill 1949 Olympia EQ, 475-yr ARP

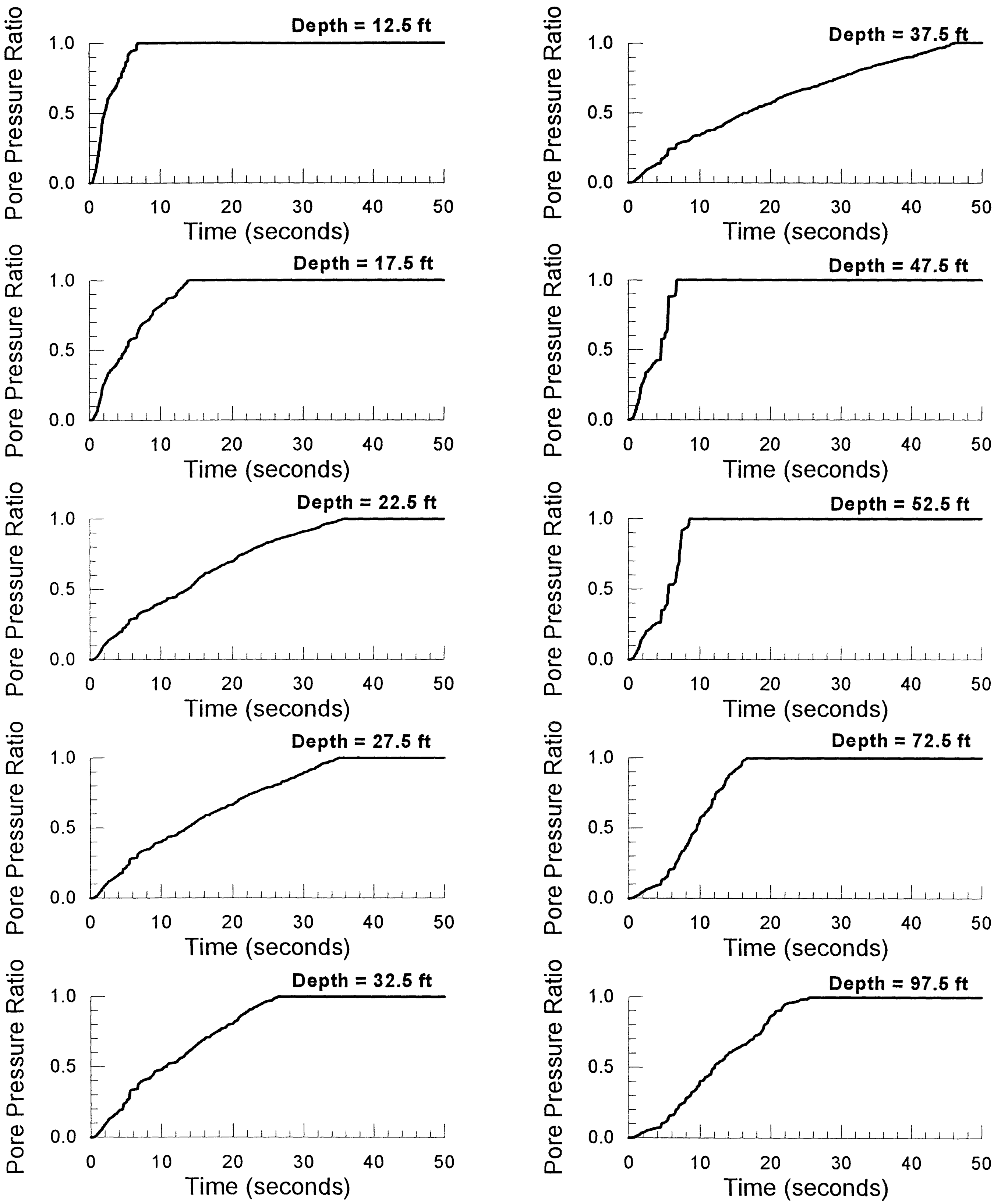


Figure 5a-3 Pore Pressure Generation
Case 5a: Soil Profile without Fill, 1949 Olympia EQ, 475-yr ARP

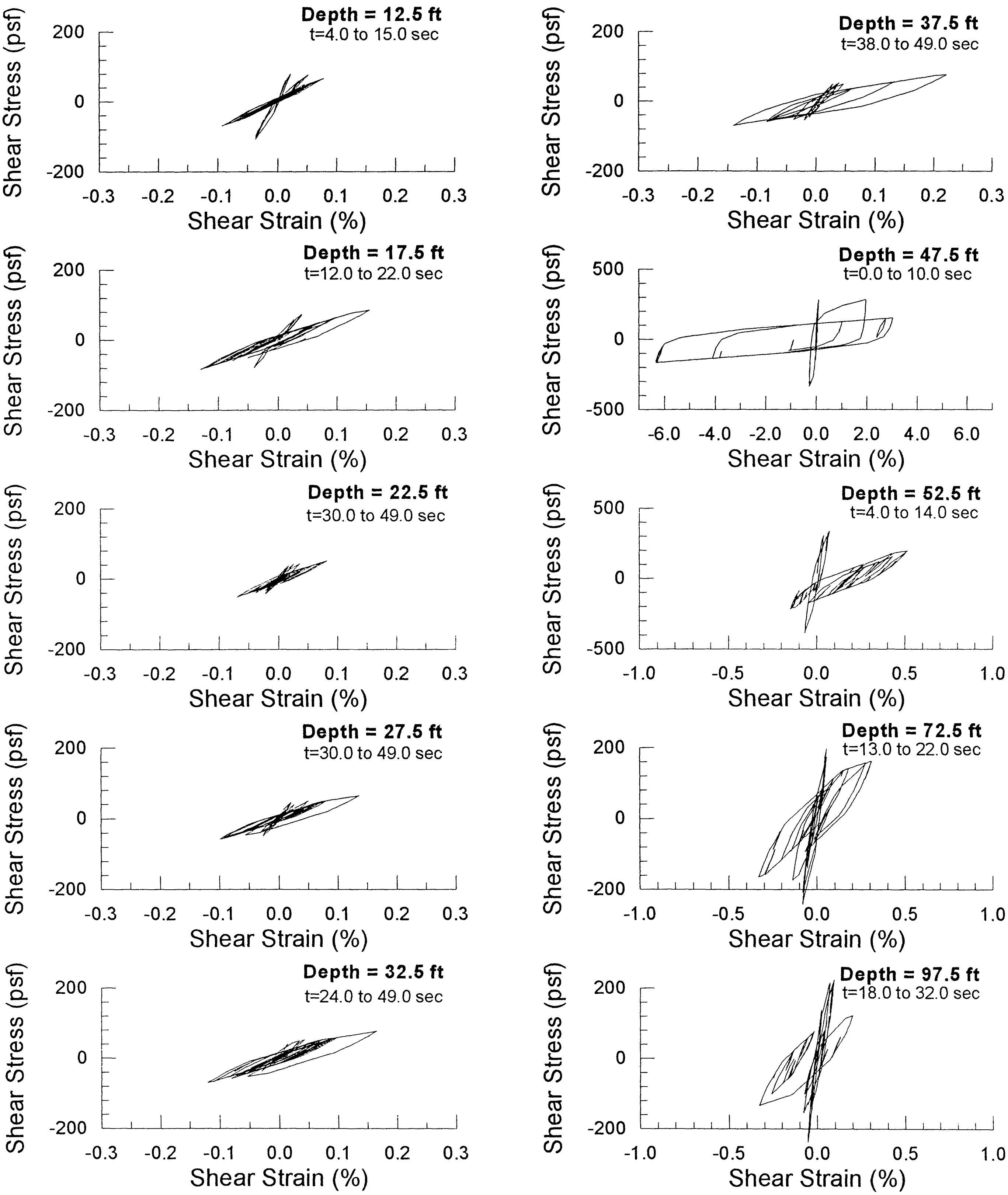


Figure 5a-4 Shear Stress-Shear Strain Loops
Case 5a: Soil Profile without Fill, 1949 Olympia Q, 475-yr ARP

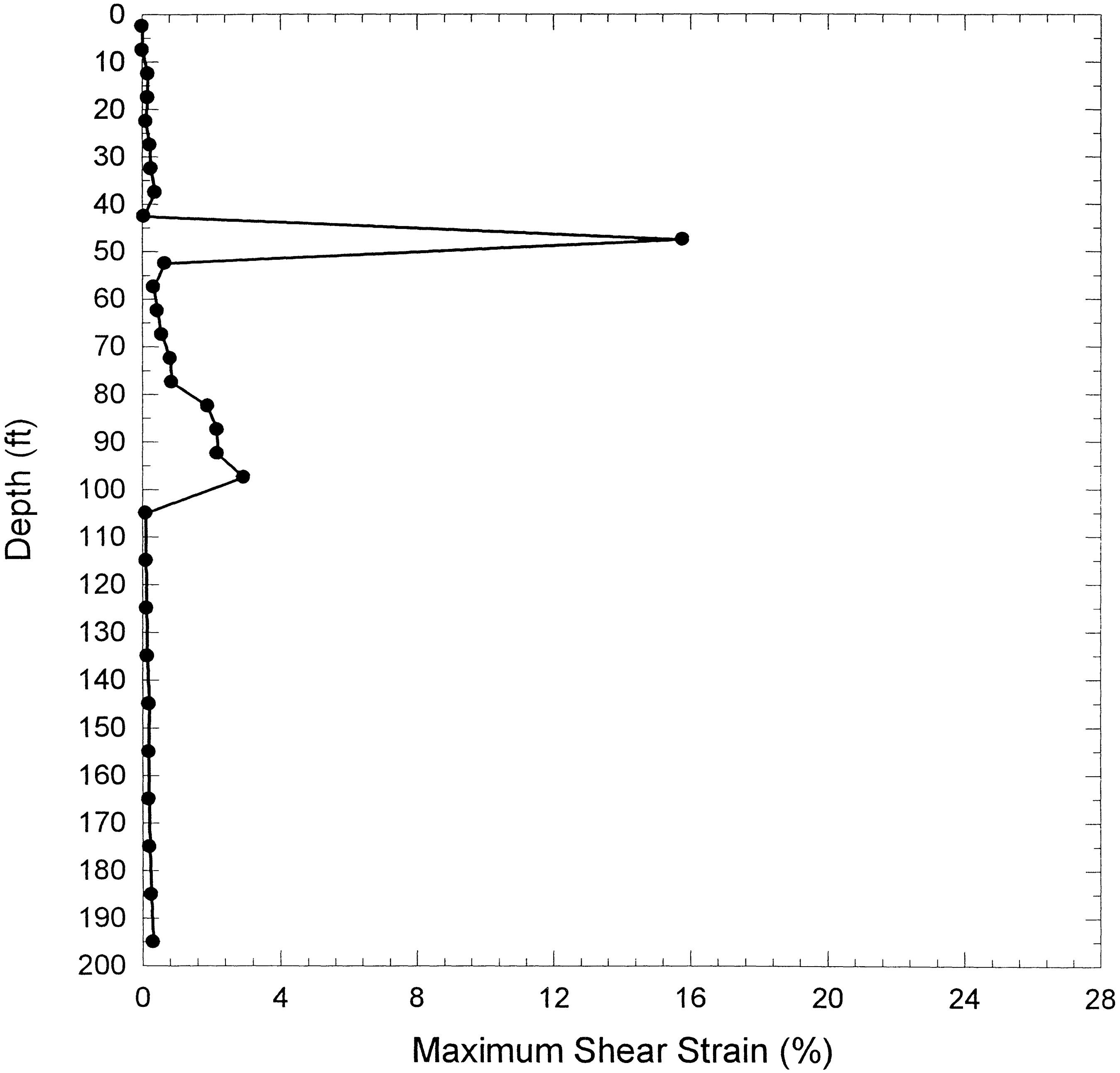


**Figure 5b-2 Maximum Shear Strain Occurred During the Shaking
Case 5b: Soil Profile without Fill
1949 Olympia EQ, 2475-yr ARP**

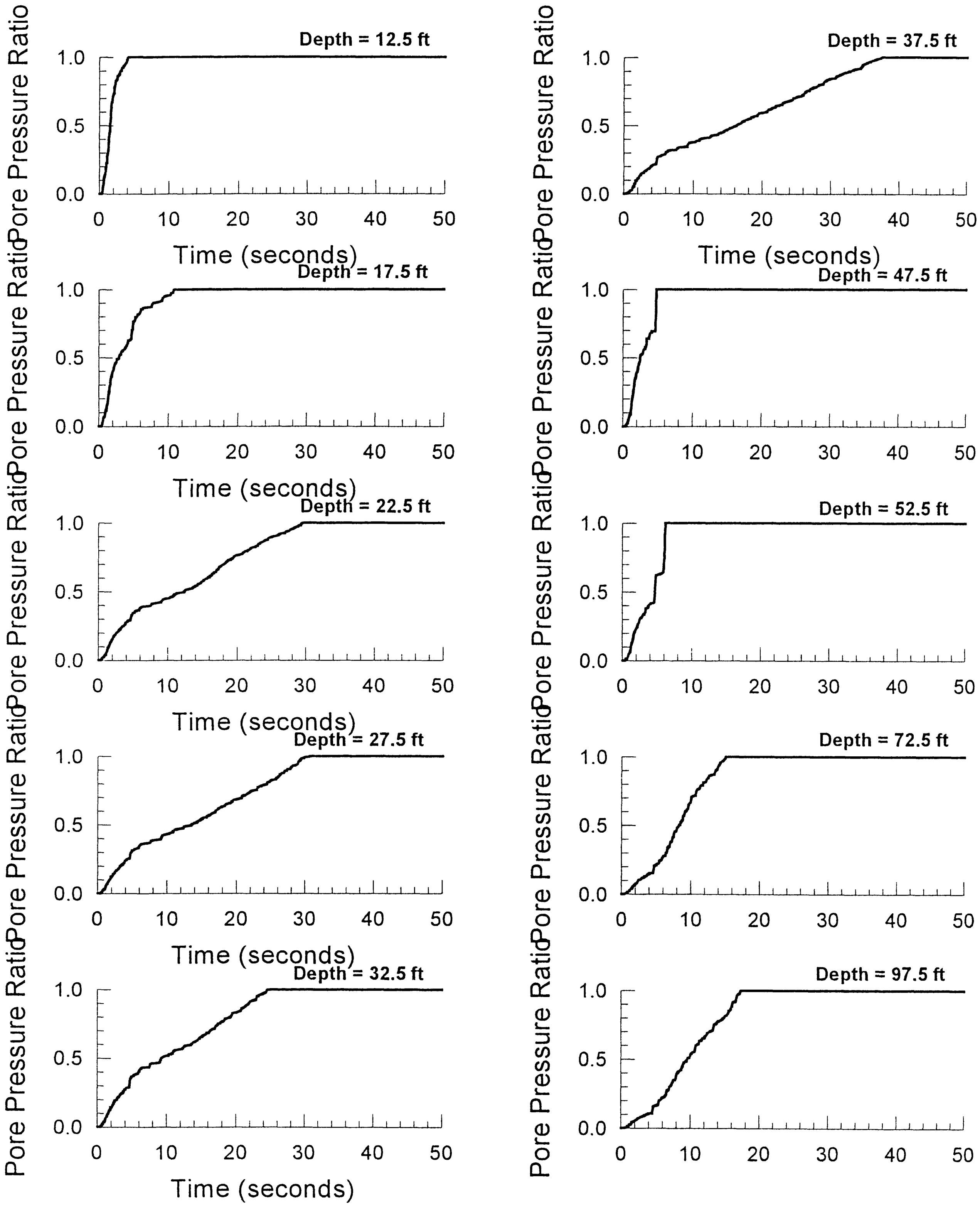


Figure 5b-3 Pore Pressure Generation
Case 5b: Soil Profile without Fill, 1949 Olympia EQ, 2475-yr ARP

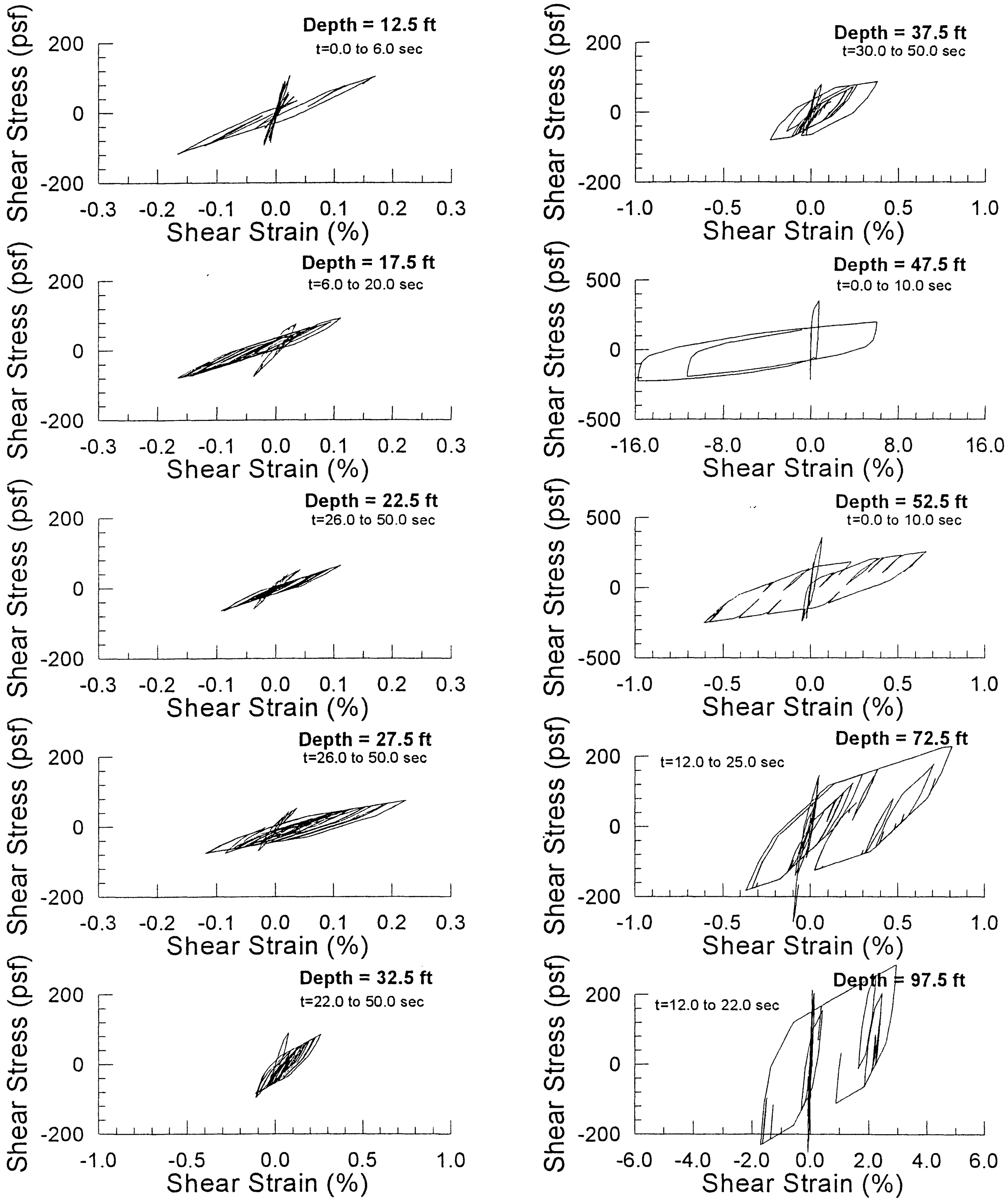


Figure 5b-4 Shear Stress-Shear Strain Loops
Case 5b: Soil Profile without Fill, 1949 Olympia EQ, 2475-yr ARP

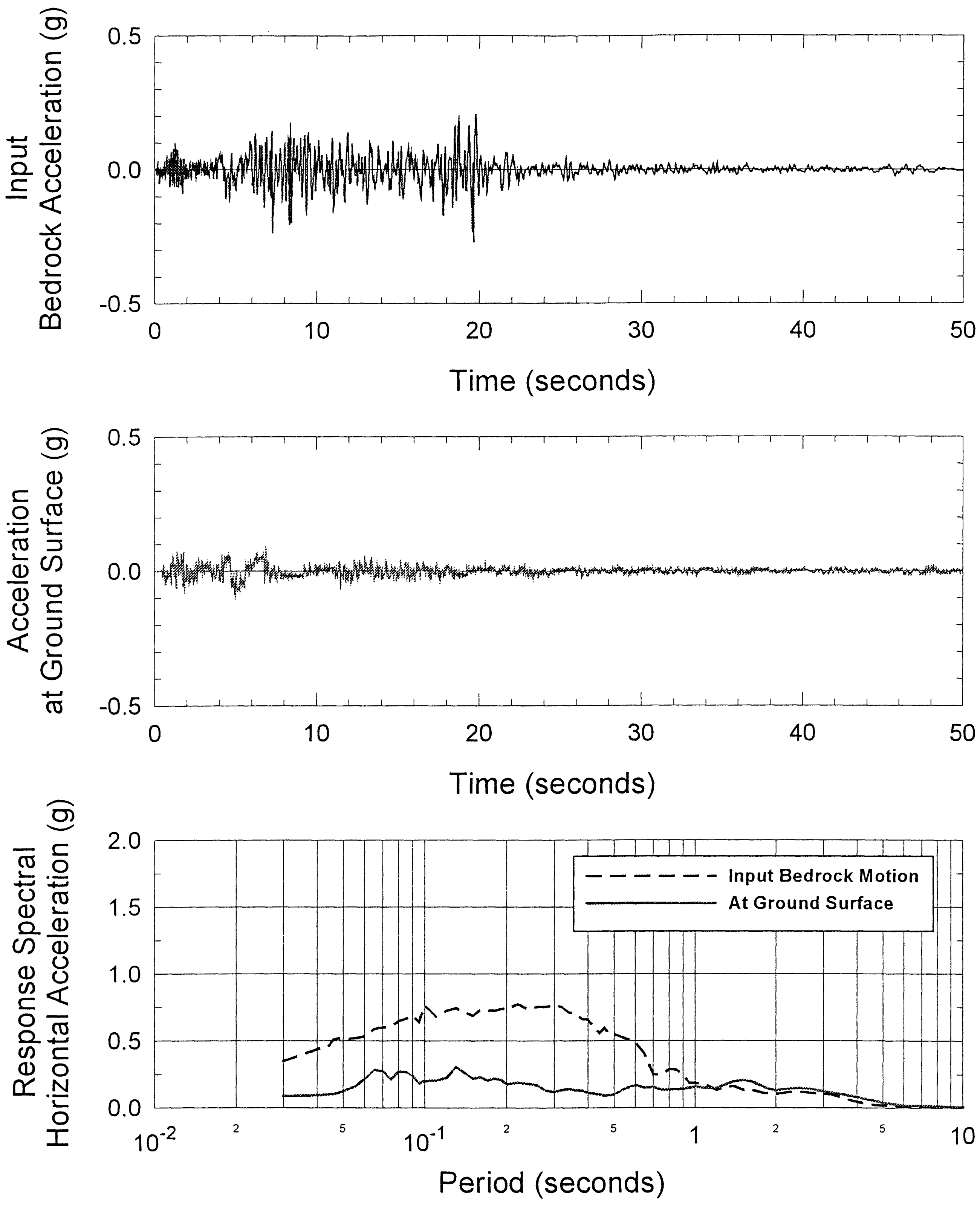


Figure 6a-1 Acceleration Time Histories and Response Spectra Case 6a: Soil Profile with Fill, 1949 Olympia EQ, 475-yr ARP

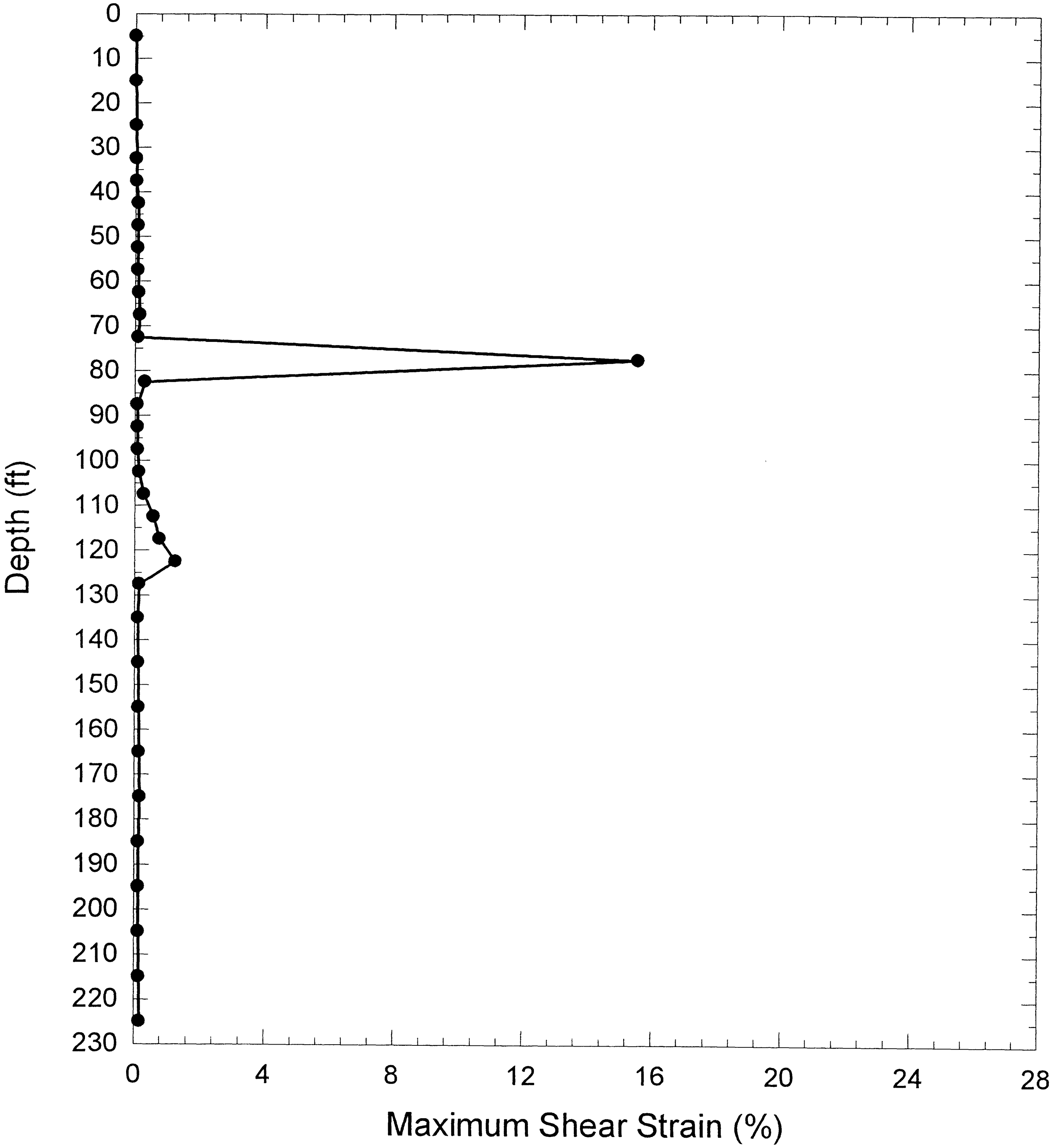


Figure 6a-2 Maximum Shear Strain Occurred During the Shaking
Case 6a: Soil Profile with Fill
1949 Olympia EQ, 475-yr ARP

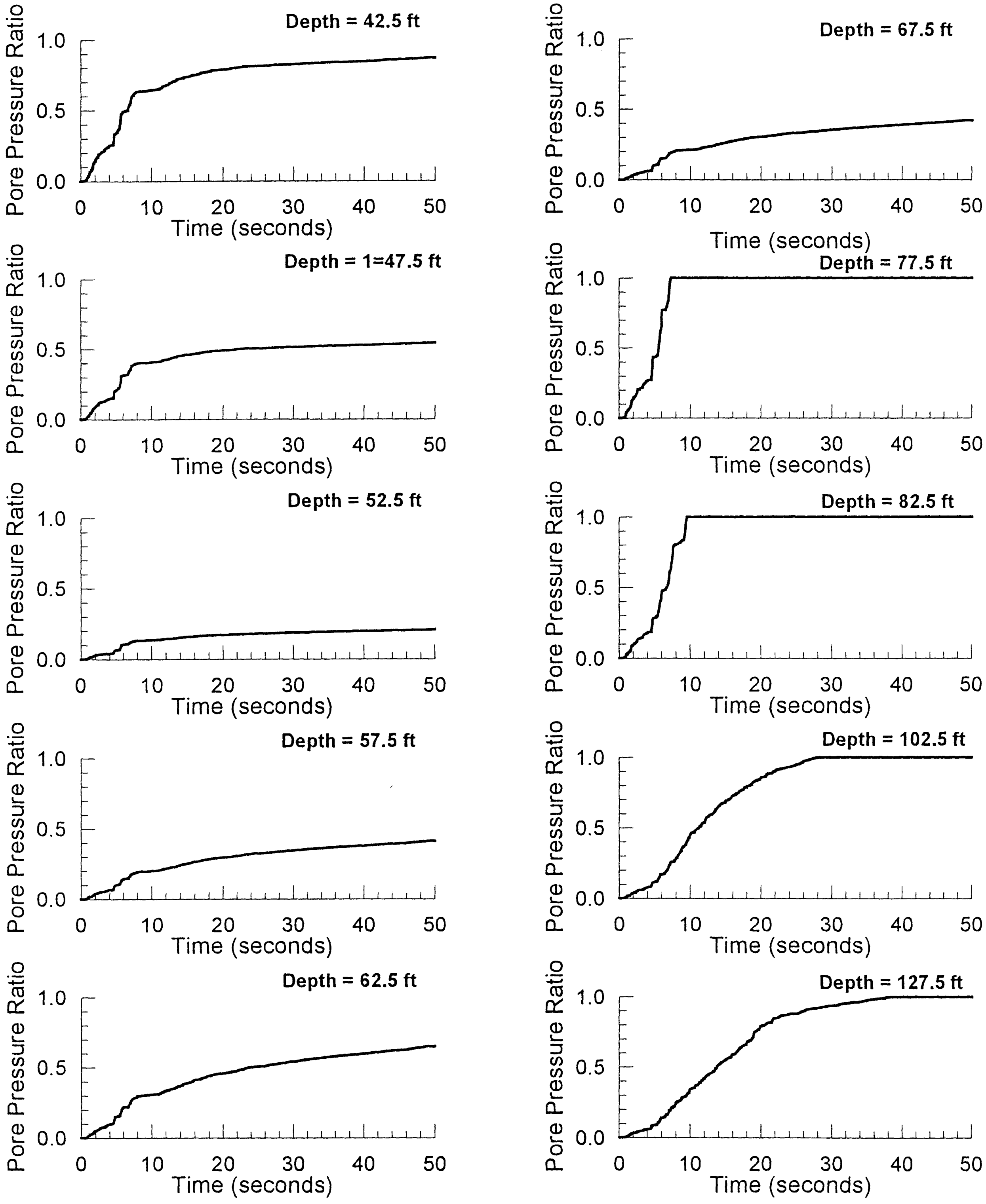


Figure 6a-3 Pore Pressure Generation
Case 6a: Soil Profile with Fill, 1949 Olympia EQ, 475-yr ARP

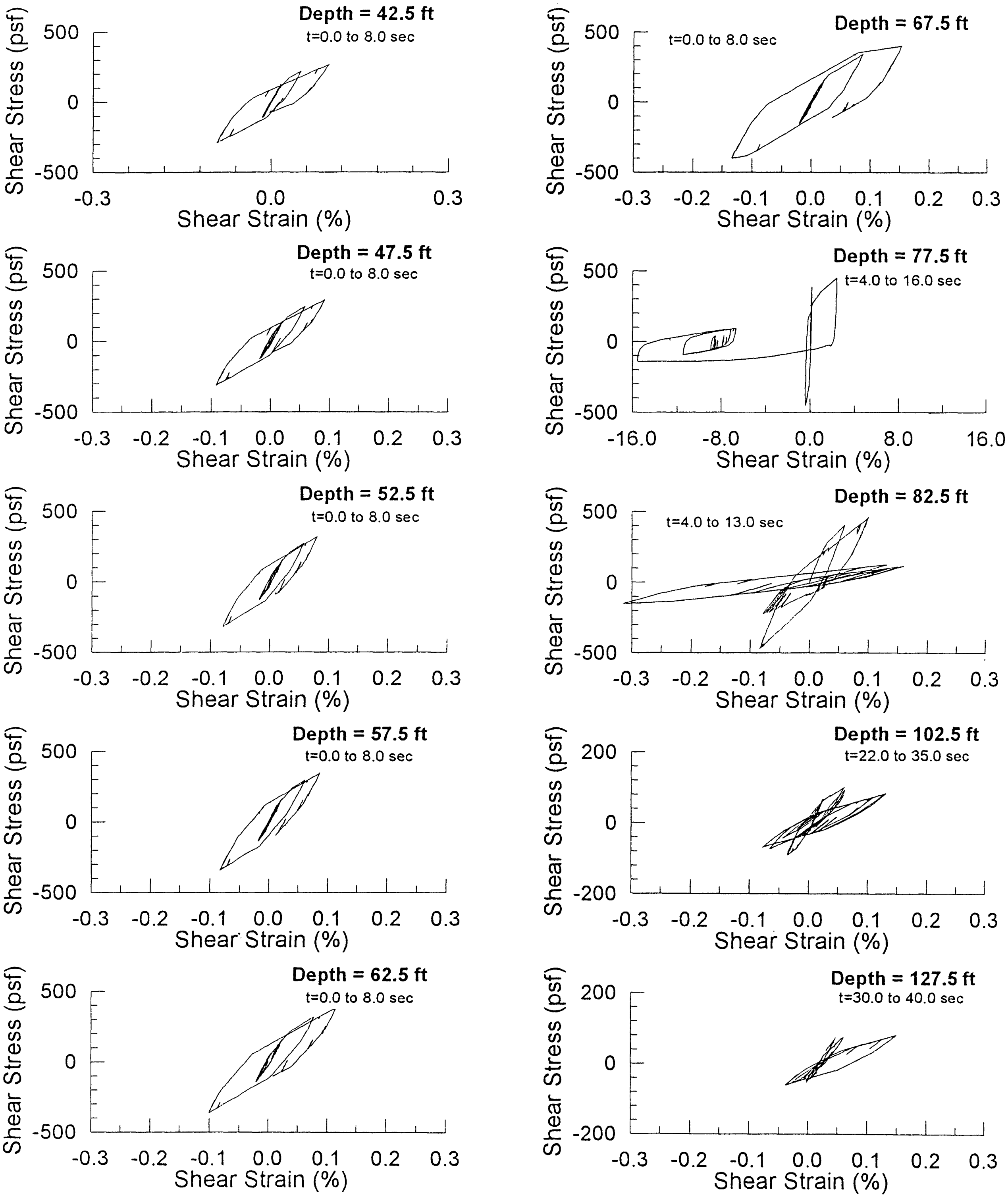


Figure 6a-4 Shear Stress-Shear Strain Loops
Case 6a: Soil Profile with Fill, 1949 Olympia EQ, 475-yr ARP

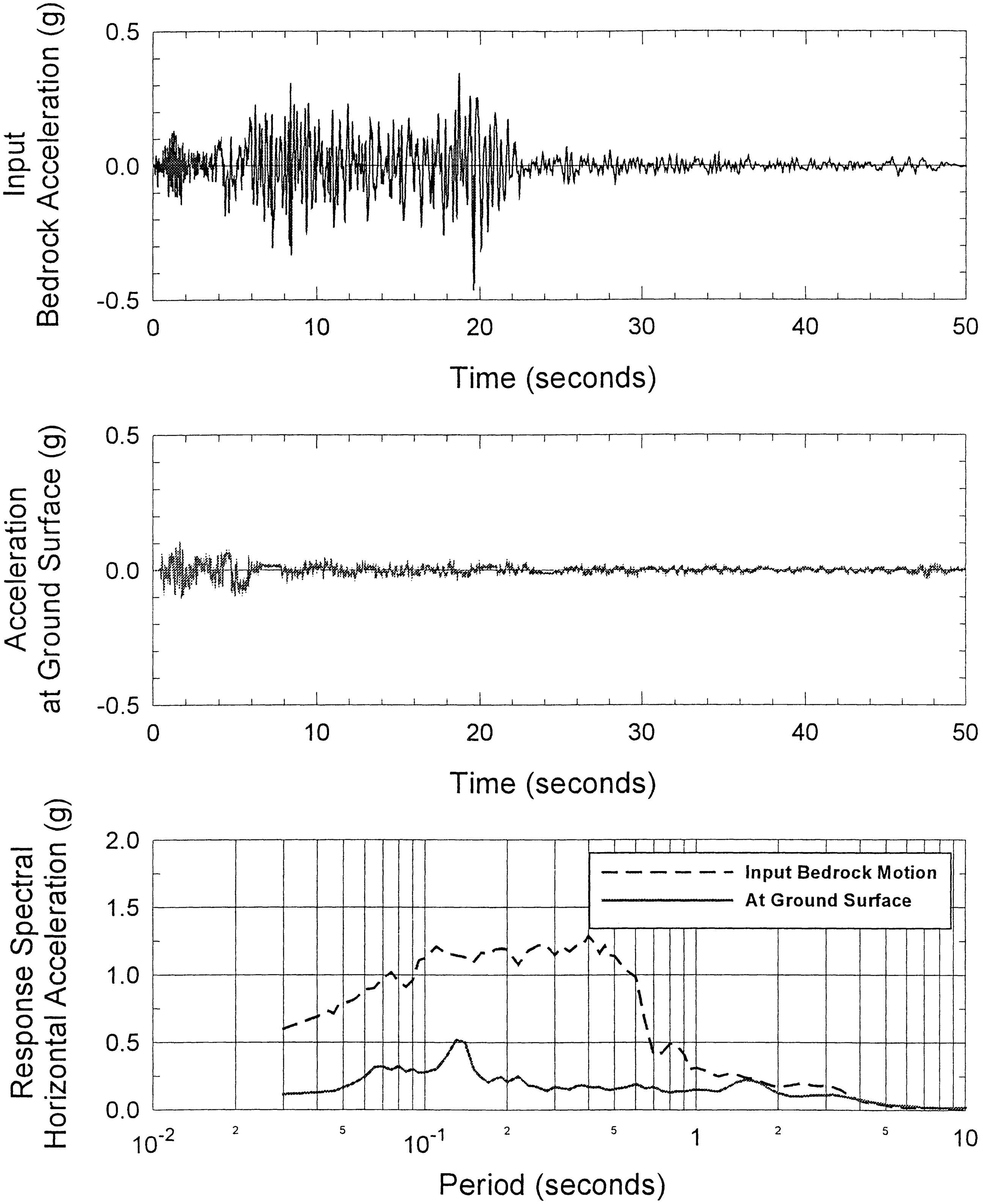


Figure 6b-1 Acceleration Time Histories and Response Spectra Case 6b: Soil Profile with Fill, 1949 Olympia EQ, 2475-yr ARP

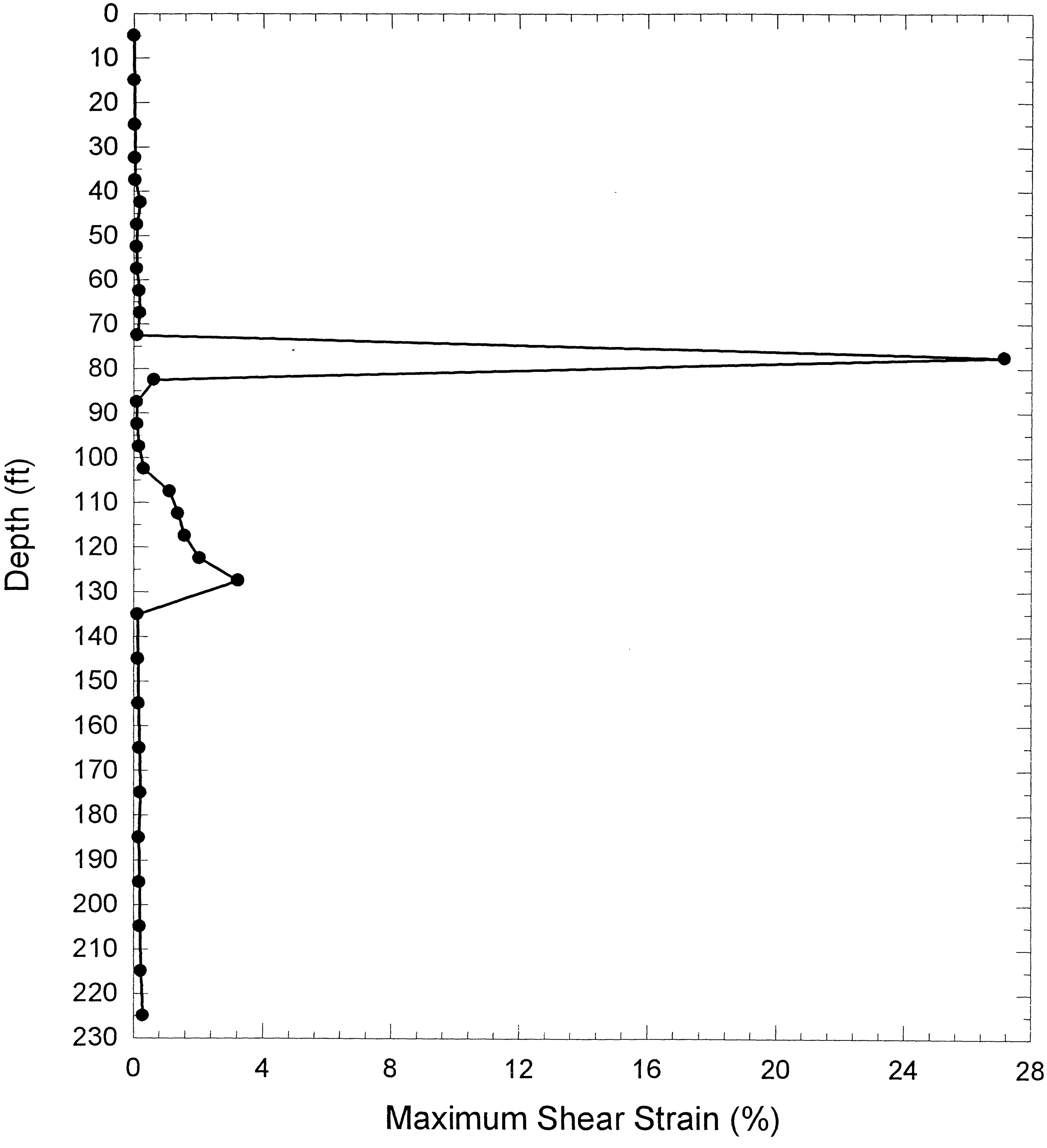


Figure 6b-2 Maximum Shear Strain Occurred During the Shaking
Case 6b: Soil Profile with Fill
1949 Olympia EQ, 2475-yr ARP

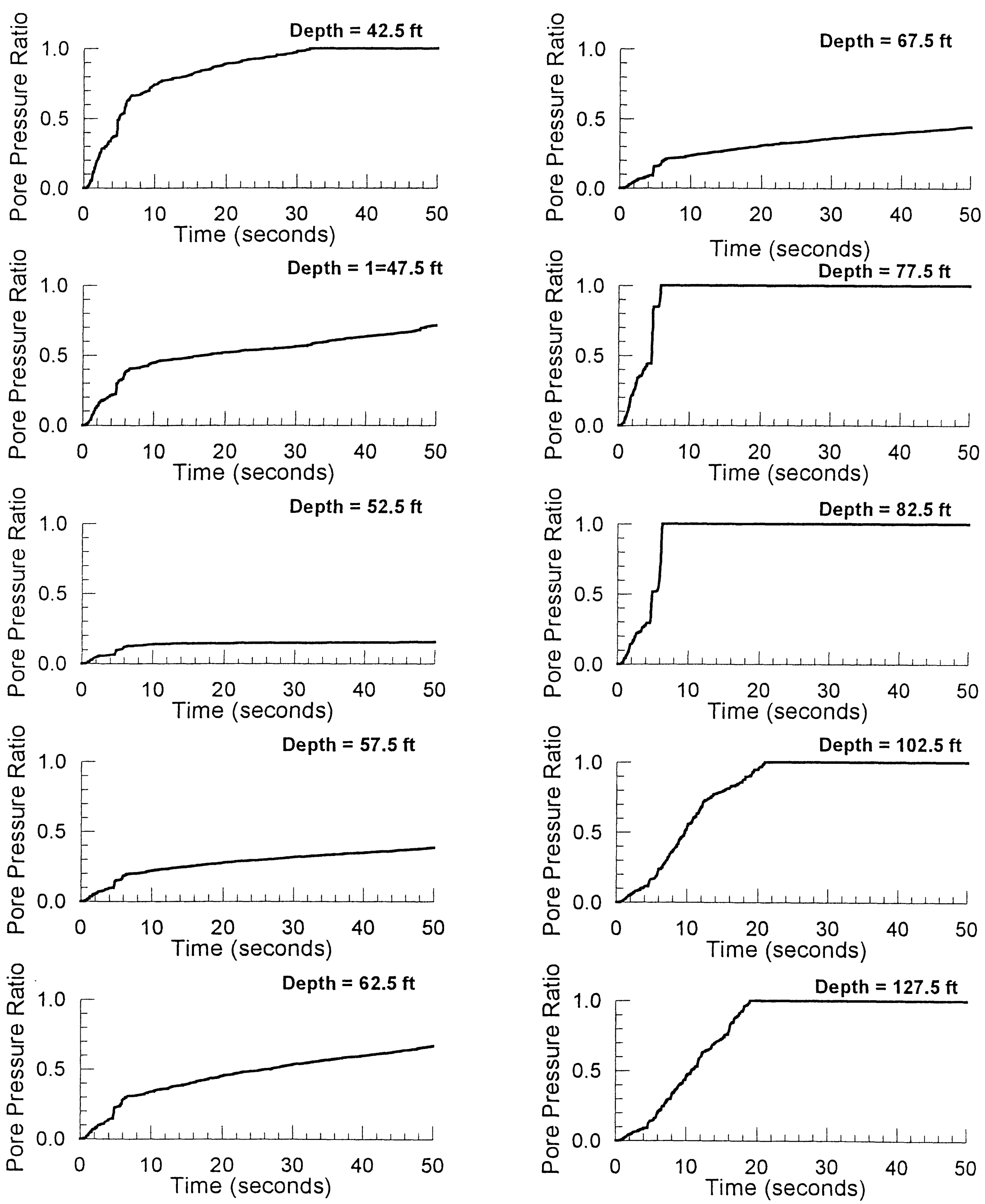


Figure 6b-3 Pore Pressure Generation
Case 6b: Soil Profile with Fill, 1949 Olympia EQ, 2475-yr ARP

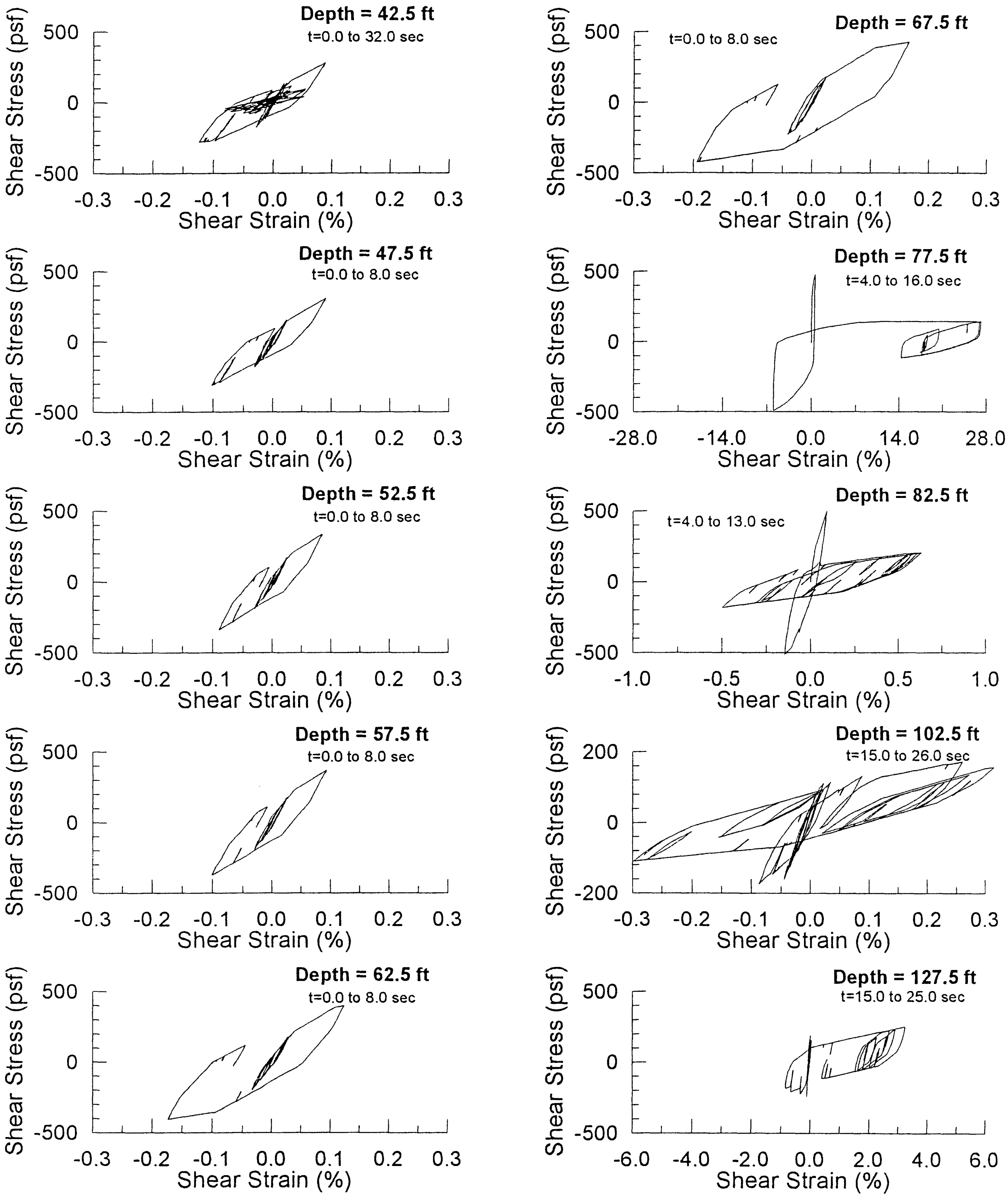


Figure 6b-4 Shear Stress-Shear Strain Loops
Case 6b: Soil Profile with Fill, 1949 Olympia EQ, 2475-yr ARP

Case 1a: Soil profile without fill, 1986 Desert Hot Springs, 475-yr ARP									
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	24.005	SEC		
Depth ft	LAYER NO.	TIME Max. Acc Occurred	ABS. Acc (ft/s/s)	TIME Max. Strain Occurred	STRAIN (%)	TIME Max. Stress Occurred	STRESS (psf)	TIME Max. Pore Pressure Built	PORE Pressure (psf)
2.5	1	2.97	4.64	2.92	0.003927	2.97	39.66	0	0
7.5	2	2.96	3.803	2.96	0.01134	2.96	98.35	0	0
12.5	3	2.94	4.509	3.56	0.2078	2.95	144.3	23.91	595
17.5	4	3.14	3.836	6.42	0.1373	3.14	177.2	24	833
22.5	5	2.55	2.946	3.13	0.05113	3.13	215.7	24	973.6
27.5	6	2.81	3.527	3.13	0.06504	3.13	249.1	23.98	1214
32.5	7	2.93	4.422	18.27	0.1108	2.53	271.2	24	1547
37.5	8	2.91	5.009	2.52	0.0677	2.52	296.3	24	1389
42.5	9	2.96	3.823	2.51	0.05458	2.51	315.8	0	0
47.5	10	2.89	3.745	4.62	2.355	2.5	342	23.94	2261
52.5	11	2.87	7.55	6.42	0.2217	2.49	386.9	23.98	2499
57.5	12	2.74	7.457	2.47	0.06122	2.47	422.1	23.99	2370
62.5	13	2.73	8.193	2.46	0.06538	2.47	461.8	24	2327
67.5	14	2.72	7.243	2.45	0.07483	2.45	490.1	23.98	2625
72.5	15	2.91	7.878	2.45	0.0767	2.45	514.1	23.97	2919
77.5	16	2.69	7.462	2.43	0.07902	2.44	536.6	23.94	3312
82.5	17	2.68	7.149	2.42	0.08895	2.43	564.1	24	3792
87.5	18	2.67	8.116	2.42	0.08751	2.42	573.5	23.74	4105
92.5	19	2.66	7.042	2.41	0.09011	2.41	581.4	23.89	4168
97.5	20	4.96	6.759	2.39	0.09096	2.4	600.8	24	3077
105	21	4.96	5.116	2.38	0.08041	2.38	628.7	0	0
115	22	2.97	5.954	2.37	0.08768	2.37	659.9	0	0
125	23	2.62	6.28	2.35	0.0929	2.36	679.6	0	0
135	24	2.93	6.223	2.33	0.1026	2.33	712	0	0
145	25	2.91	7.26	2.3	0.1083	2.31	728.7	0	0
155	26	2.89	7.109	3.47	0.08289	3.48	751.4	0	0
165	27	2.87	6.979	3.46	0.1001	3.46	821.4	0	0
175	28	2.85	6.875	3.44	0.116	3.44	875.7	0	0
185	29	2.83	6.718	3.42	0.1258	3.42	909.2	0	0
195	30	6.23	5.817	3.4	0.1242	3.4	903.2	0	0
Base	31	6.2	6.461	0	0	0	0	0	0
maximum	base	acc.=	9.02	at	6.2	sec			

Case 1b:	Soil profile without fill, 1986 Desert Hot Springs, 2475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	24.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	1.49	4.039	1.49	0.003435	1.49	34.52	0	0
7.5	2	1.25	3.574	1.25	0.01051	1.25	91.61	0	0
12.5	3	1.9	3.301	6.05	0.1454	2.62	133	3.2	595
17.5	4	1.85	5.259	6.49	0.1504	2.61	181.3	6.61	833
22.5	5	1.44	3.889	2.59	0.05095	2.59	226.9	24	928.7
27.5	6	1.43	4.346	2.58	0.06678	2.59	264.8	24	1176
32.5	7	1.42	3.924	20.44	0.1269	2.58	302.1	19.81	1547
37.5	8	1.17	4.349	2.55	0.08905	2.56	337.4	24	1357
42.5	9	1.33	4.074	2.54	0.07431	2.54	374.2	0	0
47.5	10	2.77	3.623	3.88	5.413	2.54	406	3.04	2261
52.5	11	2.76	8.879	6.63	0.2453	2.52	433.4	4.45	2499
57.5	12	2.78	10.27	2.44	0.07723	2.45	483.8	24	2623
62.5	13	2.79	10.89	2.45	0.08396	2.45	532.8	24	2613
67.5	14	2.72	9.711	2.44	0.09536	2.44	571	24	3196
72.5	15	2.71	8.095	17.81	0.1923	2.43	600.8	15.63	3451
77.5	16	2.7	8.902	10.57	0.426	2.48	620.4	10.57	3689
82.5	17	2.68	8.542	9.56	0.6238	2.47	665.1	8.31	3927
87.5	18	4.01	9.522	7.68	0.9596	2.46	681.6	7.31	4165
92.5	19	3.99	10.48	9.44	1.045	2.45	699.9	7.25	4403
97.5	20	6.47	10.39	17.62	0.301	2.43	728.5	11.86	4641
105	21	3.97	8.234	3.56	0.1372	3.57	814.5	0	0
115	22	4.78	7.717	3.55	0.1753	3.55	899.6	0	0
125	23	2.62	8.904	3.53	0.2129	3.53	964.2	0	0
135	24	2.6	9.096	3.51	0.2053	3.52	954.1	0	0
145	25	4.88	8.933	2.33	0.2449	2.34	1006	0	0
155	26	2.9	10.92	2.29	0.1695	2.3	1037	0	0
165	27	2.88	10.04	2.27	0.1734	2.27	1045	0	0
175	28	2.86	10.07	2.64	0.1821	2.64	1065	0	0
185	29	2.84	9.617	2.62	0.2011	2.63	1105	0	0
195	30	2.52	10.07	2.6	0.2182	2.6	1137	0	0
Base	31	2.49	10.79	0	0	0	0	0	0
maximum	base	acc.=	15	at	6.2	sec			

Case 1c:	Soil profile without fill, 1986 Desert Hot Springs, 2475-yr ARP, No Pore Pressure Redistribution								
MAXIMUM	VALUES	DURING	0	TO	24.005	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	2.44	3.778	2.44	0.003147	2.44	32.3	0	0
7.5	2	1.25	3.34	3.03	0.009463	1.25	84.56	0	0
12.5	3	2.42	3.389	3.01	0.2749	3.02	132.5	1.93	595
17.5	4	1.86	4.94	3.19	0.02545	1.86	144.6	24	772.9
22.5	5	1.44	4.566	1.8	0.02833	1.8	172.8	24	953.3
27.5	6	1.43	3.828	1.79	0.03284	1.79	198.3	23.99	607.2
32.5	7	1.42	4.076	15.07	0.2379	1.78	207.6	13.44	1547
37.5	8	1.17	4.403	2.58	0.04513	2.59	232.7	24	1270
42.5	9	1.16	3.865	2.6	0.04324	2.6	269.7	0	0
47.5	10	1.15	3.348	3.91	5.352	2.61	307.6	2.71	2261
52.5	11	2.79	8.846	2.5	0.03809	2.51	327.5	23.96	1695
57.5	12	2.77	9.162	2.43	0.05248	2.43	377.4	24	2032
62.5	13	2.92	8.2	2.43	0.05925	2.43	426.6	24	2369
67.5	14	2.81	8.72	2.44	0.0727	2.45	471.7	24	2829
72.5	15	2.83	9.684	2.46	0.08213	2.46	527.5	24	3114
77.5	16	2.81	7.787	2.82	0.09045	2.47	568.5	24	3544
82.5	17	4.02	8.915	11.27	0.3164	2.82	622.5	10.66	3927
87.5	18	4.01	10.15	6.42	1.387	2.45	643.6	6.14	4165
92.5	19	3.99	11.43	10.44	0.6923	3.55	681	10.09	4403
97.5	20	4.8	9.87	7.61	1.26	3.55	731.5	6.58	4641
105	21	5.6	9.119	3.52	0.1411	3.53	826.2	0	0
115	22	2.64	8.57	3.55	0.1724	3.55	893.8	0	0
125	23	2.62	8.836	3.53	0.1965	3.53	939.5	0	0
135	24	2.6	10	2.36	0.2159	2.36	968.4	0	0
145	25	2.63	8.812	2.33	0.235	2.34	993.5	0	0
155	26	2.9	10.33	2.29	0.1629	2.29	1023	0	0
165	27	2.88	9.326	2.27	0.1674	2.27	1032	0	0
175	28	2.86	9.544	2.26	0.1708	2.26	1040	0	0
185	29	6.22	9.571	2.62	0.2003	2.63	1104	0	0
195	30	2.82	10.04	2.6	0.2239	2.6	1146	0	0
Base	31	2.49	10.79	0	0	0	0	0	0
maximum	base	acc.=	15	at	6.2	sec			

Case 2a:	Soil profile with fill, 1986 Desert Hot Springs, 475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	24.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	2.94	3.961	2.93	0.007543	2.94	67.72	0	0
15	2	2.96	3.022	3.52	0.01905	3.53	142.4	0	0
25	3	2.98	3.729	2.98	0.03742	2.98	241	0	0
32.5	4	3.44	3.394	3.56	0.04248	3.57	267.1	0	0
37.5	5	3.17	3.806	3.57	0.04366	3.57	271.6	0	0
42.5	6	3.15	4.28	13.72	0.4158	2.6	289.5	6.4	3895
47.5	7	2.95	6.803	3.6	0.09495	2.58	309.8	24	3908
52.5	8	2.97	6.024	2.57	0.07787	2.57	324.5	24	637.9
57.5	9	2.81	6.013	2.55	0.07762	2.55	339.3	24	1506
62.5	10	2.79	7.055	3.65	0.09627	2.54	354.9	23.95	2562
67.5	11	2.77	6.414	3.66	0.1059	2.52	379.4	24	1532
72.5	12	3.11	5.825	2.51	0.08791	2.52	405.5	0	0
77.5	13	2.84	6.157	7.79	1.644	2.51	433	6.23	5561
82.5	14	5.05	7.89	17.7	0.1419	2.48	474.1	8.91	5799
87.5	15	3.7	8.58	2.47	0.07532	2.48	507	24	3343
92.5	16	3.72	5.598	2.46	0.07766	2.47	534.9	24	3411
97.5	17	3.72	6.552	2.46	0.08079	2.46	552.1	23.99	3808
102.5	18	3.73	7.194	2.44	0.09232	2.45	579.8	23.99	4185
107.5	19	2.77	5.366	2.43	0.09372	2.44	595	23.99	4552
112.5	20	2.68	6.14	2.42	0.09384	2.43	608.7	23.92	4897
117.5	21	2.67	5.407	2.41	0.09219	2.42	616.7	24	5204
122.5	22	3	6.058	2.41	0.09453	2.41	621.4	24	5414
127.5	23	2.99	6.037	2.39	0.09338	2.4	628.4	24	4129
135	24	2.98	5.531	2.38	0.08357	2.38	643.3	0	0
145	25	2.96	6.576	2.37	0.09108	2.38	672.6	0	0
155	26	2.94	6.632	3.52	0.1091	3.52	731.7	0	0
165	27	2.92	7.143	3.5	0.1268	3.5	783.6	0	0
175	28	2.9	6.981	3.49	0.139	3.49	819.8	0	0
185	29	2.88	7.021	3.48	0.09886	3.48	818.4	0	0
195	30	3.49	6.066	3.46	0.1047	3.46	837.5	0	0
205	31	3.51	6.257	3.44	0.1094	3.44	853.2	0	0
215	32	3.49	5.772	3.41	0.1083	3.41	850.3	0	0
225	33	3.46	6.057	3.39	0.1093	3.39	852.8	0	0
235	34	6.2	6.599	0	0	0	0	0	0
Base	maximum	base	acc.=	9.02	at	6.2	sec		

Case 2b:	Soil profile with fill, 1986 Desert Hot Springs, 2475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	24.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	1.37	4.34	1.37	0.00841	1.37	74.2	0	0
15	2	2.99	3.555	2.98	0.02246	2.98	160.1	0	0
25	3	3.01	3.526	3.01	0.0404	3.01	252.1	0	0
32.5	4	3.66	3.876	3	0.04979	3	300.7	0	0
37.5	5	1.29	4.654	2.99	0.04558	2.99	280	0	0
42.5	6	3.63	4.729	7.48	0.4182	2.63	306.2	5.71	3895
47.5	7	2.98	7.936	3.68	0.1099	2.63	340.1	24	4034
52.5	8	2.84	7.362	2.61	0.09976	2.61	376.3	24	824.9
57.5	9	2.85	7.737	2.6	0.1026	2.6	398.5	24	1587
62.5	10	2.8	8.403	2.59	0.1548	2.59	423.9	24	2600
67.5	11	2.82	6.708	2.56	0.2124	2.57	459.5	24	1713
72.5	12	2.77	7.066	2.55	0.1288	2.56	484.3	0	0
77.5	13	2.76	7.812	6.76	2.297	2.54	510.1	4.35	5561
82.5	14	2.75	9.848	6.63	0.2506	2.52	533.3	6.14	5799
87.5	15	2.73	9.081	2.51	0.08702	2.51	549.2	24	4130
92.5	16	3.97	7.042	2.5	0.08652	2.5	563.6	24	4170
97.5	17	3.75	8.2	2.49	0.0907	2.49	585.3	24	4779
102.5	18	2.7	8.613	3.64	0.1088	2.48	620.2	24	5681
107.5	19	2.69	9.346	3.64	0.1386	2.48	652.1	21.81	6989
112.5	20	6.51	8.194	17.76	0.2248	2.46	678	11.92	7227
117.5	21	2.67	8.356	10.51	1.123	2.45	716.1	9.26	7465
122.5	22	6.49	10.14	10.44	1.553	2.44	731.8	9.11	7703
127.5	23	6.47	10.1	17.63	0.3088	2.43	764.4	15.76	7941
135	24	4.29	7.99	2.41	0.1358	2.41	811	0	0
145	25	2.98	8.29	3.56	0.1686	3.56	887	0	0
155	26	3.94	8.816	3.54	0.2076	3.54	957.2	0	0
165	27	2.94	8.307	3.52	0.2145	3.52	966.5	0	0
175	28	2.92	8.849	2.33	0.2149	2.33	967	0	0
185	29	2.9	9.682	3.47	0.1479	3.47	984.6	0	0
195	30	2.88	9.829	2.65	0.1457	2.66	976.6	0	0
205	31	4.66	10.14	2.64	0.167	2.65	1032	0	0
215	32	2.65	9.229	2.63	0.205	2.63	1115	0	0
225	33	2.63	9.28	2.6	0.2281	2.61	1153	0	0
235	34	6.2	10.66	0	0	0	0	0	0
Base	maximum	base	acc.=	9.02	at	6.2	sec		

Case 3a:	Soil profile without fill, 1985 Chile EQ, 475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	81.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	14.96	3.396	14.96	0.00285	14.96	29.03	0	0
7.5	2	14.95	2.955	14.96	0.008448	14.96	75.91	0	0
12.5	3	14.94	3.093	14.95	0.2289	14.96	119.1	14.43	595
17.5	4	14.77	2.5	25.19	0.2173	14.97	150.9	16.32	833
22.5	5	15.07	2.371	31.23	0.1551	14.95	151.3	29.96	1071
27.5	6	14.73	2.529	29.1	0.2848	13.8	173.8	28.37	1309
32.5	7	14.82	2.612	26.13	0.4129	13.78	202.2	25.34	1547
37.5	8	14.59	2.865	44.25	0.3436	13.8	218.9	33.84	1785
42.5	9	15.02	2.498	13.77	0.03213	13.77	220.7	0	0
47.5	10	14.78	2.617	23.35	5.49	13.77	225	16.76	2261
52.5	11	23.34	5.459	23.24	0.495	23.24	233.4	18.82	2499
57.5	12	22.88	5.063	33.22	0.1515	23.18	295.6	32.76	2737
62.5	13	19.33	4.557	39.13	0.144	23.18	328.2	36.01	2975
67.5	14	23.42	4.342	33.18	0.1716	23.2	360.2	32.72	3213
72.5	15	23.4	4.565	30.18	0.259	23.21	396.2	29.6	3451
77.5	16	19.37	5.021	28.63	0.8379	23.19	417.3	28	3689
82.5	17	29.26	5.501	30.11	1.078	23.18	437.5	26.7	3927
87.5	18	27.95	7.033	30.1	1.497	23.16	429	26.54	4165
92.5	19	23.16	6.583	30.05	1.705	22.77	436.8	26.69	4403
97.5	20	27.86	7.147	29.97	1.265	22.75	475.4	30.02	4641
105	21	29.11	7.69	22.74	0.05943	22.74	525.7	0	0
115	22	29.13	7.727	25.33	0.07447	25.33	600.4	0	0
125	23	29.15	8.172	26.19	0.09535	26.19	688.4	0	0
135	24	28.22	6.36	26.16	0.1242	26.16	776.3	0	0
145	25	28.19	6.705	26.13	0.1493	26.17	850.2	0	0
155	26	27.76	7.525	26.15	0.1282	26.15	917.2	0	0
165	27	27.75	6.244	26.15	0.1462	26.15	978.2	0	0
175	28	27.73	9.011	26.14	0.1658	26.14	1029	0	0
185	29	27.72	9.893	28.85	0.2061	28.86	1116	0	0
195	30	27.69	9.519	28.83	0.2458	28.83	1179	0	0
Base	31	27.72	8.033	0	0	0	0	0	0
maximum	base	acc.=	9.58	at27.71	sec				

Case 3b: Soil profile without fill, 1985 Chile, 2475-yr ARP									
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	81.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	13.84	3.342	13.84	0.002815	13.84	28.56	0	0
7.5	2	13.85	3.146	13.84	0.008841	13.84	78.69	0	0
12.5	3	13.86	3.141	13.85	0.2219	13.85	125.2	12.94	595
17.5	4	17.92	2.485	15.06	0.2046	13.86	164.1	14.89	833
22.5	5	14.76	2.781	30.24	0.19	13.84	187.7	29.85	1071
27.5	6	14.77	2.513	30.32	0.4563	13.83	195.3	27.78	1309
32.5	7	41.76	2.485	27.08	0.6864	13.81	210.1	24.08	1547
37.5	8	14.85	3.267	44.31	0.649	13.81	198.2	31.61	1785
42.5	9	13.81	3.224	13.78	0.02912	13.78	204.6	0	0
47.5	10	14.82	2.982	23.43	10.48	13.77	220	15.23	2261
52.5	11	23.38	5.773	23.28	0.6656	23.29	262.5	17.29	2499
57.5	12	19.4	5.224	48.63	0.2462	23.26	290.1	30.38	2737
62.5	13	19.4	5.072	48.58	0.2662	23.25	307.2	31.72	2975
67.5	14	19.37	4.879	30.27	0.4161	23.24	338.5	29.66	3213
72.5	15	19.38	5.796	30.23	0.8863	23.23	381.9	27.23	3451
77.5	16	23.09	5.91	30.21	1.634	23.21	393	25.36	3689
82.5	17	23.08	7.413	26.68	2.094	22.87	440.4	24.52	3927
87.5	18	23.06	7.125	30.11	2.967	22.85	475.1	23.95	4165
92.5	19	23.05	9.41	26.6	3.354	22.81	529.9	23.81	4403
97.5	20	23.02	10.49	29.99	3.471	22.79	585.8	26.88	4641
105	21	27.89	9.523	22.77	0.08602	22.77	654.2	0	0
115	22	27.9	8.2	22.75	0.1119	22.75	740.3	0	0
125	23	27.92	8.775	22.73	0.1446	22.73	836.6	0	0
135	24	28.22	9.503	22.72	0.1685	22.72	887	0	0
145	25	28.2	8.734	22.69	0.2073	22.69	957	0	0
155	26	27.77	9.621	28.93	0.1673	28.93	1032	0	0
165	27	27.75	11.55	28.92	0.243	28.92	1175	0	0
175	28	27.73	12.36	28.9	0.3714	28.9	1324	0	0
185	29	27.72	13.81	28.87	0.5011	28.87	1462	0	0
195	30	28.93	13.28	28.84	0.6252	28.84	1563	0	0
Base	31	27.67	12.65	0	0	0	0	0	0
maximum	base	acc.=	14.8	at27.69	sec				

Case 4a:	Soil profile with fill, 1985 Chile, 475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	81.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	15.01	3.011	15.01	0.00554	15.01	51.48	0	0
15	2	14.98	2.275	16.59	0.0116	16.59	99.31	0	0
25	3	14.89	2.101	16.97	0.01955	16.97	148.2	0	0
32.5	4	17.05	2.262	14.97	0.02444	14.97	179.3	0	0
37.5	5	14.95	2.184	14.96	0.02891	14.96	203.3	0	0
42.5	6	15.64	2.266	18.06	0.6366	14.95	228.3	18.33	3895
47.5	7	18.03	2.788	48.92	0.4221	14.94	244.4	43.25	4133
52.5	8	18.05	2.814	14.95	0.04992	14.95	248.6	81	748.8
57.5	9	18.07	2.856	17.97	0.05129	16.95	250	81	2140
62.5	10	15.38	2.888	17.96	0.07519	16.94	268.1	81	3881
67.5	11	15.36	2.743	17.96	0.0695	17.97	274.1	81	2204
72.5	12	15.35	3.031	17.94	0.04689	17.94	286.1	0	0
77.5	13	15.34	2.906	29	7.867	17.93	290.3	19.5	5561
82.5	14	23.36	6.247	23.24	0.8042	17.9	281.9	22.51	5799
87.5	15	23.24	6.057	48.58	0.1126	17.9	299.7	49.01	6037
92.5	16	27.65	5.732	62.61	0.06647	17.88	324.3	58.03	6275
97.5	17	23.27	6.261	48.54	0.155	23.19	368.1	46.75	6513
102.5	18	25.23	5.107	44.99	0.1868	23.19	410.5	40.31	6751
107.5	19	23.3	5.557	39.77	0.2938	23.17	441.9	35.27	6989
112.5	20	27.91	4.867	31.35	0.6115	23.17	480.8	30.9	7227
117.5	21	27.9	5.764	30.05	1.208	26.31	478.4	29.29	7465
122.5	22	27.88	8.945	29.09	2.01	26.29	489.3	29.21	7703
127.5	23	27.85	9.808	34.63	0.7259	26.27	536.4	32.42	7941
135	24	27.87	7.538	26.23	0.07507	26.24	603.2	0	0
145	25	29.12	9.045	26.22	0.1036	26.22	715.3	0	0
155	26	28.22	6.413	26.19	0.1363	26.19	811.8	0	0
165	27	22.96	5.811	26.17	0.17	26.17	889.3	0	0
175	28	22.94	5.621	26.18	0.2135	26.18	965.1	0	0
185	29	25.29	7.321	26.18	0.1547	26.18	1005	0	0
195	30	25.26	6.597	28.92	0.1724	28.92	1044	0	0
205	31	27.72	6.776	28.9	0.2258	28.9	1149	0	0
215	32	25.23	7.895	28.89	0.2645	28.89	1202	0	0
225	33	25.2	7.183	28.85	0.2912	28.86	1233	0	0
235	34	26.97	7.688	0	0	0	0	0	0
Base	maximum	base	acc.=	9.58	at27.71	sec			

4

Case 5b:	Soil profile with fill, 1985 Chile, 2475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	81.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	15.02	3.032	15.02	0.005549	15.02	51.83	0	0
15	2	15.05	2.52	15.05	0.01245	15.05	106.1	0	0
25	3	14.9	2.301	13.88	0.02068	13.88	154	0	0
32.5	4	15.11	2.518	13.85	0.02677	13.85	191.8	0	0
37.5	5	14.94	2.857	13.84	0.03466	13.84	232.3	0	0
42.5	6	15.1	2.807	17.23	1.514	13.84	256.7	16.86	3895
47.5	7	15.6	3.423	27.25	0.8551	13.85	265.4	23.95	4133
52.5	8	14.63	3.179	14.97	0.06688	14.97	289.3	81	1127
57.5	9	13.86	3.135	14.96	0.06277	14.96	281.7	81	2308
62.5	10	13.82	3.357	14.94	0.06833	14.94	265	81	4188
67.5	11	15.38	4.09	14.93	0.06354	14.93	268.1	81	2352
72.5	12	15.36	3.974	15.07	0.04161	15.07	263.1	0	0
77.5	13	17.45	3.544	23.59	13.86	14.9	281.3	18.08	5561
82.5	14	17.93	5.81	23.27	0.7955	14.88	300.1	20.37	5799
87.5	15	23.13	5.976	48.61	0.1565	14.87	335.3	45.17	6037
92.5	16	23.56	6.089	48.58	0.1623	14.86	352.3	46.07	6275
97.5	17	17.97	6.855	39.77	0.1916	14.85	371.4	39.18	6513
102.5	18	23.09	5.885	48.6	0.266	23.18	386.3	32.45	6751
107.5	19	23.08	6.682	31.55	1.413	23.19	412.4	28.98	6989
112.5	20	23.07	7.018	31.47	2.005	14.82	428.8	26.97	7227
117.5	21	23.17	7.54	26.64	3.933	22.82	437.2	25.83	7465
122.5	22	25.04	8.09	26.67	4.253	22.8	482.5	25.34	7703
127.5	23	29.2	10.35	30	2.82	22.78	548.8	27.26	7941
135	24	27.89	8.981	22.76	0.0822	22.77	637.1	0	0
145	25	27.89	7.397	22.75	0.1137	22.75	745.3	0	0
155	26	22.98	8.079	22.73	0.1441	22.73	835.5	0	0
165	27	28.22	9.636	23.64	0.2091	23.64	959.3	0	0
175	28	24.02	8.522	23.63	0.2672	23.63	1036	0	0
185	29	27.77	9.663	23.59	0.1944	23.6	1091	0	0
195	30	27.75	10.55	28.91	0.2182	28.92	1137	0	0
205	31	27.73	12.72	28.9	0.3393	28.9	1287	0	0
215	32	27.72	13.41	28.87	0.4723	28.87	1438	0	0
225	33	28.93	12.91	28.84	0.5822	28.84	1528	0	0
235	34	27.67	12.59	0	0	0	0	0	0
Base	maximum	base	acc.=	14.8	at27.69	sec			

Case 5a:	Soil profile without fill, 1949 Olympia, 475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	50.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	5.04	3.151	5.04	0.002628	5.04	26.93	0	0
7.5	2	4.83	2.636	5.04	0.007468	5.04	68.37	0	0
12.5	3	5.01	2.919	14.14	0.09324	4.81	106.9	6.82	595
17.5	4	5.34	2.766	20.56	0.1552	4.81	148.8	13.84	833
22.5	5	5.36	2.777	48.03	0.08308	4.82	184.9	35.95	1071
27.5	6	5.37	3.011	48.08	0.1358	4.8	217.4	34.97	1309
32.5	7	4.92	2.936	48.02	0.1652	4.8	248.1	26.72	1547
37.5	8	4.91	3.024	47.97	0.2217	5.04	280.2	46.15	1785
42.5	9	4.86	2.818	5.04	0.05336	5.05	311	0	0
47.5	10	4.88	2.746	8.43	6.344	5.03	334.3	6.76	2261
52.5	11	8.7	6.895	9.81	0.5154	4.97	387.3	8.6	2499
57.5	12	7.14	6.156	23.13	0.208	4.96	431.7	20.74	2737
62.5	13	9.68	5.005	26.43	0.1352	4.96	473.9	21.38	2975
67.5	14	13.56	4.205	19.75	0.3391	4.95	510.2	18.89	3213
72.5	15	10.02	5.044	18.72	0.332	4.94	539.3	16.59	3451
77.5	16	20.08	5.961	19.85	0.4857	4.93	567.2	15.76	3689
82.5	17	19.85	6.045	20.33	0.6154	4.92	588.5	14.98	3927
87.5	18	9.64	5.579	18.4	0.7862	4.92	596.8	15.34	4165
92.5	19	8.53	5.62	18.6	1.081	4.94	615.5	16.62	4403
97.5	20	19.77	5.626	26.34	0.3283	4.89	632.5	25.4	4641
105	21	19.8	5.096	6.21	0.0884	6.21	663.3	0	0
115	22	19.72	5.237	6.19	0.1059	6.2	721.8	0	0
125	23	19.74	4.82	6.17	0.1303	6.17	794	0	0
135	24	19.75	4.952	6.14	0.1538	6.15	858.7	0	0
145	25	8.44	5.847	6.13	0.1838	6.13	915.5	0	0
155	26	8.43	5.881	6.09	0.1455	6.09	976.2	0	0
165	27	19.8	6.721	6.08	0.1496	6.09	989.5	0	0
175	28	19.77	7.667	6.06	0.1595	6.07	1015	0	0
185	29	19.75	7.546	6.03	0.1652	6.03	1027	0	0
195	30	19.77	6.797	6.01	0.171	6.01	1040	0	0
Base	31	19.65	6.696	0	0	0	0	0	0
maximum	maximum	base	acc.=	8.72	at19.65	sec			

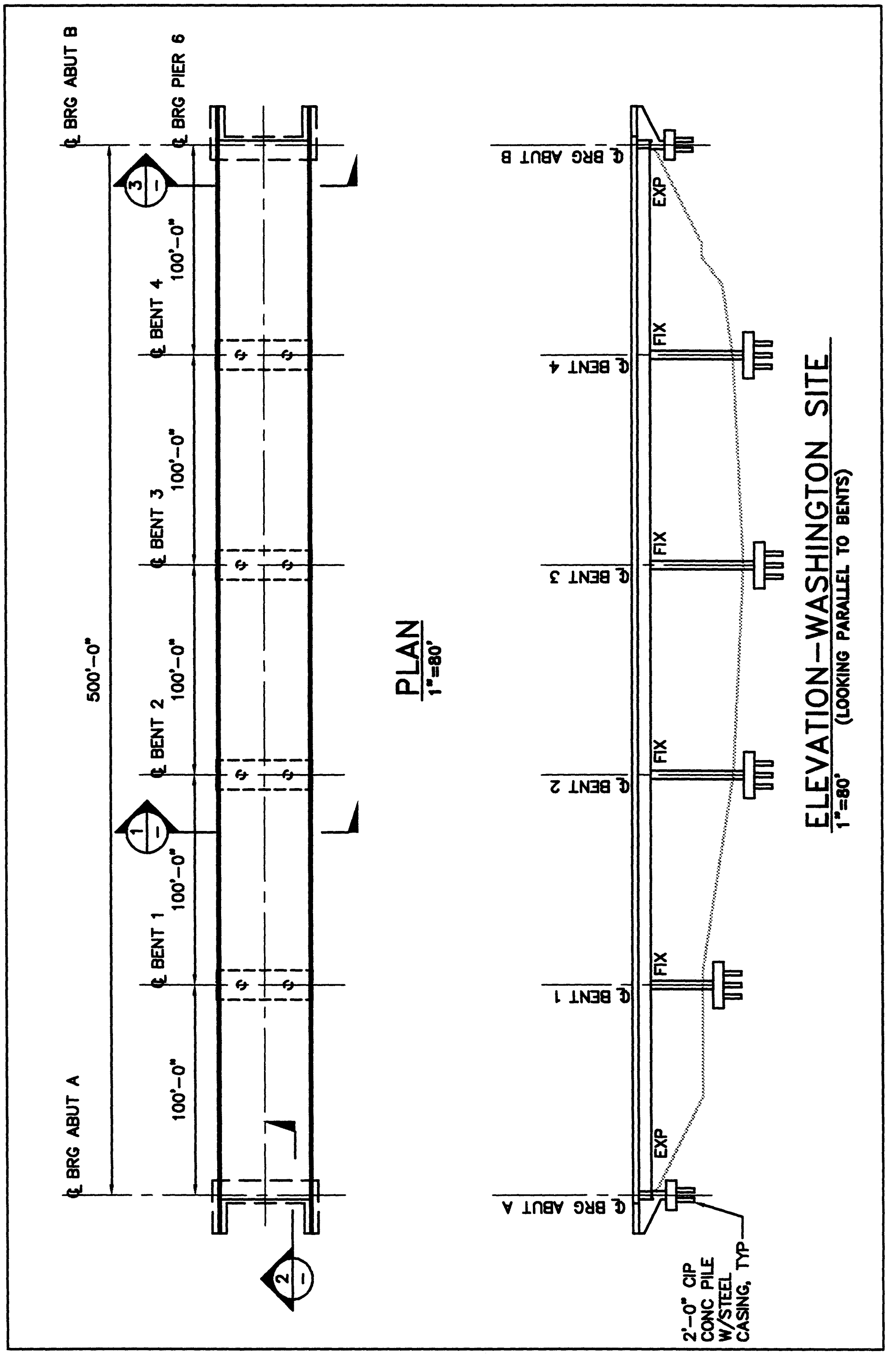
Case 5b:	Soil profile without fill, 1949 Olympia EQ, 2475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	50.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	1.59	3.556	1.59	0.002992	1.59	30.39	0	0
7.5	2	4.82	2.921	4.82	0.007947	4.82	71.98	0	0
12.5	3	1.36	3.265	4.42	0.1703	4.81	117.8	4.05	595
17.5	4	2.14	3.486	19	0.1668	4.46	136.8	10.84	833
22.5	5	1.42	3.188	48.01	0.1116	4.47	171.1	29.51	1071
27.5	6	1.28	3.728	48.07	0.2246	4.49	208	30.72	1309
32.5	7	1.62	3.302	48.02	0.2609	4.59	240.1	24.69	1547
37.5	8	1.67	3.467	48.2	0.3757	4.58	274.3	37.49	1785
42.5	9	4.24	3.376	4.57	0.05409	4.57	313.7	0	0
47.5	10	4.25	3.65	5.84	15.78	4.57	349.5	4.75	2261
52.5	11	9.67	6.836	7.23	0.6606	4.51	364.5	6.15	2499
57.5	12	8.65	6.742	20.43	0.3359	4.5	392.9	18.16	2737
62.5	13	8.66	6.564	20.49	0.439	4.44	430.6	20.1	2975
67.5	14	8.68	5.738	20.51	0.5657	4.45	466.6	17.73	3213
72.5	15	8.69	7.805	20.49	0.8142	4.46	496.2	14.99	3451
77.5	16	9.68	6.234	20.45	0.8577	4.47	527.7	12.51	3689
82.5	17	9.66	8.015	20.4	1.902	4.46	557.7	11.82	3927
87.5	18	9.65	8.592	20.34	2.172	4.45	591.9	11.82	4165
92.5	19	11.2	8.246	20.3	2.184	4.43	623.9	12.36	4403
97.5	20	19.82	8.585	20.24	2.942	4.43	651.7	17.34	4641
105	21	18.83	7.421	4.41	0.09417	4.42	684	0	0
115	22	18.85	6.899	4.4	0.1057	4.4	721.5	0	0
125	23	19.76	7.606	4.37	0.117	4.37	754.5	0	0
135	24	7.97	6.817	19.57	0.1438	19.57	834	0	0
145	25	18.75	6.999	20.11	0.1867	20.11	921.6	0	0
155	26	8.43	7.996	20.08	0.1808	20.09	1061	0	0
165	27	19.82	9.513	20.07	0.1847	20.07	1071	0	0
175	28	19.8	11.45	5.13	0.207	5.14	1118	0	0
185	29	19.77	11.23	18.13	0.2522	18.13	1188	0	0
195	30	19.74	9.924	18.11	0.306	18.11	1249	0	0
Base	31	19.66	11.5	0	0	0	0	0	0
maximum	maximum	base	acc.=	14.9	at19.59	sec			

Case 6a: Soil profile with fill, 1949 Olympia EQ, 475-yr ARP

MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	50.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	4.94	2.799	4.93	0.005106	4.94	47.85	0	0
15	2	4.91	2.461	5.15	0.01357	5.15	113.8	0	0
25	3	5.13	2.447	5.13	0.02754	5.13	189.7	0	0
32.5	4	6.79	2.977	5.13	0.03466	5.14	232.3	0	0
37.5	5	6.78	2.778	5.15	0.04146	5.15	262.3	0	0
42.5	6	5.03	3.09	6.66	0.09732	5.17	293.6	49.83	3392
47.5	7	5.05	3.082	6.64	0.09262	5.16	316.1	50	2246
52.5	8	6.74	3.449	6.63	0.08086	5.18	322.3	50	899.8
57.5	9	5.07	3.009	6.61	0.08679	6.61	344.6	50	1892
62.5	10	6.72	3.044	6.62	0.1145	6.62	373	49.81	3154
67.5	11	6.7	3.169	6.61	0.1533	6.61	402.5	50	2132
72.5	12	6.69	2.902	5.08	0.09934	5.08	430	0	0
77.5	13	2	3.048	9.05	15.57	5.09	454.9	7.26	5561
82.5	14	7.72	6.087	12.05	0.3163	5.08	475.3	9.51	5799
87.5	15	7.11	6.008	6.43	0.08044	4.96	499.9	50	5845
92.5	16	7.13	5.115	6.44	0.08659	4.95	526.4	50	5564
97.5	17	13.53	4.768	6.45	0.08943	6.45	545.9	46.84	6513
102.5	18	13.55	4.647	34.09	0.1314	6.47	569.8	27.97	6751
107.5	19	9.66	4.843	20.54	0.2831	6.47	592.5	20.75	6989
112.5	20	8.6	4.903	19.8	0.5877	6.45	613.7	19.43	7227
117.5	21	8.54	5.688	20.45	0.7732	5.36	630.1	18.78	7465
122.5	22	8.53	6.098	18.32	1.265	5.34	669.6	18.73	7703
127.5	23	20.01	6.738	37.51	0.1492	5.34	685.7	37.84	7941
135	24	19.81	6.081	6.39	0.1074	6.39	726.4	0	0
145	25	19.97	5.922	6.4	0.1228	6.4	771.7	0	0
155	26	18.92	5.005	5.28	0.1388	5.28	819.2	0	0
165	27	6.93	4.915	5.26	0.1534	5.26	858	0	0
175	28	8.44	5.964	6.14	0.1766	6.14	901.7	0	0
185	29	8.43	5.908	6.11	0.1351	6.11	940.5	0	0
195	30	19.8	5.695	6.09	0.1313	6.09	927.7	0	0
205	31	19.78	7.929	6.32	0.1391	6.32	954.3	0	0
215	32	19.8	7.833	6.3	0.1514	6.3	996.4	0	0
225	33	19.73	6.778	6.28	0.1743	6.28	1047	0	0
235	34	19.65	6.615	0	0	0	0	0	0
Base	maximum	base	acc.=	8.72	at19.65	sec			

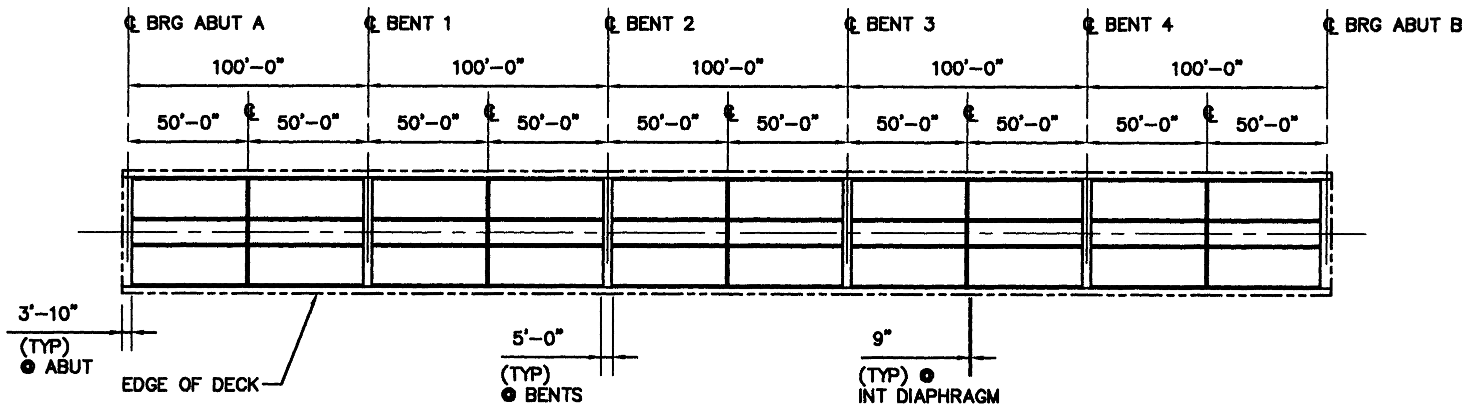
Case 6b:	Soil profile with fill, 1949 Olympia EQ, 2475-yr ARP								
MAXIMUM	MAXIMUM	VALUES	DURING	0	TO	50.005	SEC		
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	1.65	3.57	1.65	0.006727	1.65	61.03	0	0
15	2	4.94	2.725	4.94	0.01743	4.95	134.3	0	0
25	3	1.81	2.788	4.92	0.02952	4.92	200.1	0	0
32.5	4	1.74	3.517	4.9	0.03683	4.9	242	0	0
37.5	5	1.94	3.634	4.89	0.04277	4.89	267.9	0	0
42.5	6	4.86	4.18	47.6	0.2026	4.51	279.6	31.74	3895
47.5	7	1.84	3.095	5.63	0.1024	5.63	311.2	50	2953
52.5	8	1.29	3.366	5.61	0.09	5.61	342.1	50	673.6
57.5	9	1.53	3.172	5.59	0.1011	5.59	377.5	50	1771
62.5	10	1.52	3.852	5.6	0.1737	5.33	406.8	49.99	3243
67.5	11	1.26	3.585	5.59	0.1946	5.59	426.6	50	2264
72.5	12	2.02	3.596	5.55	0.1166	5.55	461.5	0	0
77.5	13	2.03	3.278	10.8	27.14	5.57	494.2	5.93	5561
82.5	14	9.68	7.157	7.22	0.6303	5.45	515.1	6.27	5799
87.5	15	8.61	7.78	45.95	0.09998	5.44	548.2	36.92	6037
92.5	16	6.39	6.624	45.91	0.1074	5.43	567.8	46.06	6275
97.5	17	8.66	6.284	29.55	0.1644	5.42	591.4	29.24	6513
102.5	18	8.68	6.406	21.45	0.3161	5.41	616.6	20.88	6751
107.5	19	8.69	7.41	20.46	1.123	5.4	637.6	17.15	6989
112.5	20	8.71	5.84	21.49	1.378	5.39	659.1	15.21	7227
117.5	21	9.65	7.81	15.85	1.588	5.38	695.8	13.82	7465
122.5	22	9.64	8.904	21.37	2.05	5.36	716.3	14.03	7703
127.5	23	13.44	8.154	20.23	3.24	5.34	728.1	18.92	7941
135	24	18.83	7.21	5.32	0.1216	5.32	768.4	0	0
145	25	12.01	7.307	5.3	0.1369	5.31	813.7	0	0
155	26	19.98	7.54	5.29	0.1548	5.29	860.8	0	0
165	27	6.95	7.077	5.26	0.1861	5.27	919.9	0	0
175	28	18.75	7.071	5.22	0.2111	5.23	962	0	0
185	29	18.77	7.41	20.08	0.167	20.08	1032	0	0
195	30	19.82	9.816	5.18	0.1841	5.18	1068	0	0
205	31	19.8	11.41	5.14	0.1987	5.15	1100	0	0
215	32	19.77	11.28	18.13	0.2323	18.14	1159	0	0
225	33	19.74	9.836	18.11	0.2912	18.12	1233	0	0
235	34	19.66	11.6	0	0	0	0	0	0
Base	maximum	base	acc.=	14.9	at19.59	sec			

Appendix E
STRUCTURAL DATA FOR WESTERN BRIDGE



PLAN
1"=80'

ELEVATION–WASHINGTON SITE
1"=80' (LOOKING PARALLEL TO BENTS)



FRAMING PLAN

1"=80'

E.1 SAP2000 Model Development: Models & Framing

NCHRP 12-49 LIQUEFACTION STUDY
Analysis & Design Cases

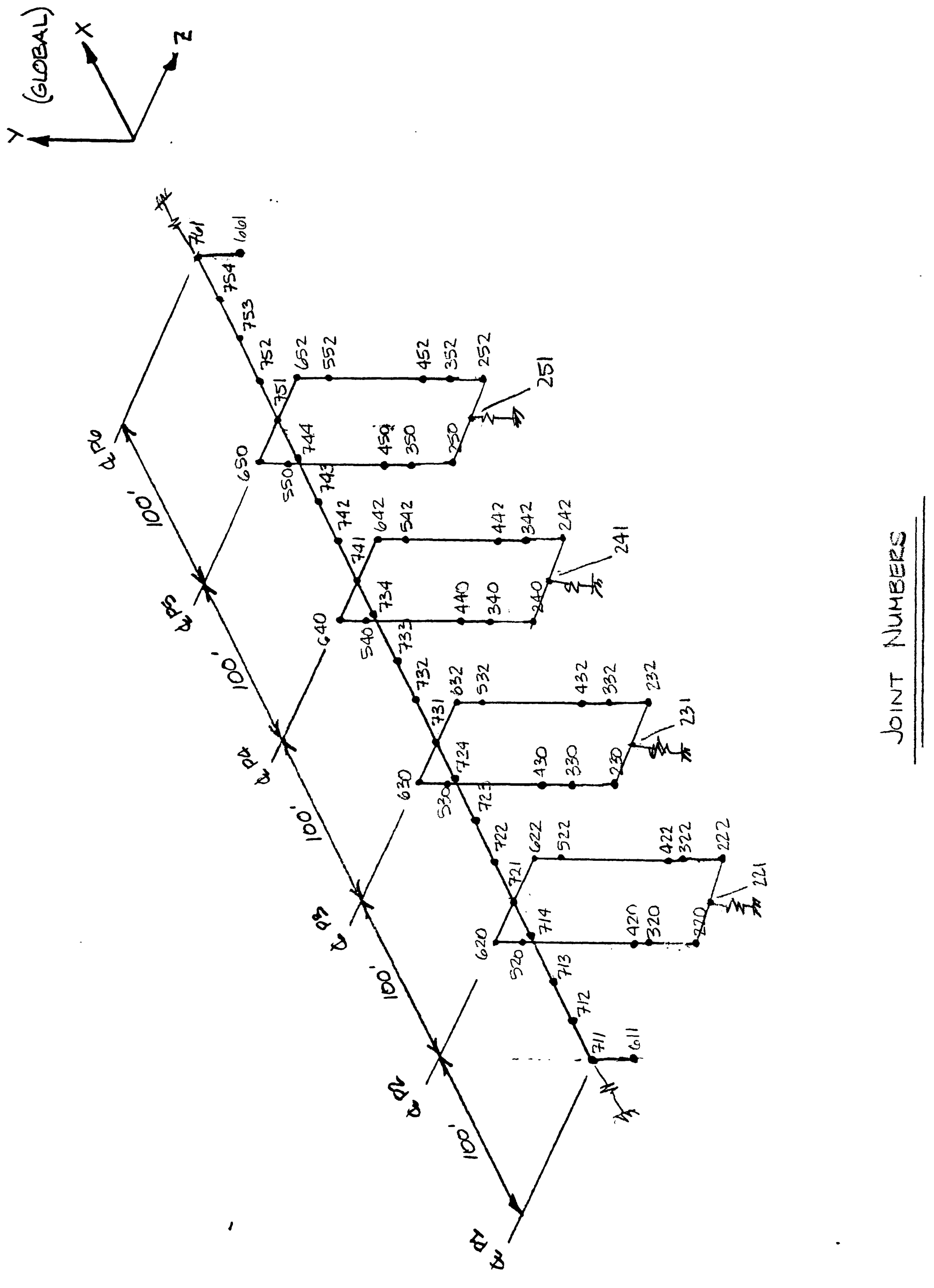
Earthquake Recurrence	**Soil Profile**	**Foundation Restraints & Springs**	**Washington Bridge**	**Missouri Bridge**
100-Year	Non-Liquefied	Springs at all piers	WA100N	MO100N
	Liquefied	Springs at all piers	- - -	- - -
500-Year	Non-Liquefied	Springs at all piers	WA500N	MO500N
	Liquefied	Springs at all piers	WA500L (Check Case)	MO500L (Check Case)
2500-Year	Non-Liquefied	Interior piers fixed, longitudinal spring & transverse fixed at abutment	WA2500NF (Check Case)	MO2500NF (Check Case)
		Springs at all piers	WA2500N	MO2500N
	Liquefied	Springs at all piers	WA2500L	MO2500L

NCHRP 12-49 LIQUEFACTION STUDY
Spectrum Development

Earthquake Recurrence	Seismic Code	Ground Motion Parameters	Washington Site		Missouri Site	
			Liquefied Case	Non-Liquefied Case	Liquefied Case	Non-Liquefied Case
100-YR	NCHRP 12-49	F_a, S_s, F_v, S_1	- - -	Site Class E (F_a=2.5, S_s=0.261, F_v=3.5, S_1=0.081)	- - -	Site Class D (F_a=1.0, S_s=1.331, F_v=1.4, S_1=0.411)
500-YR	AASHTO Division 1A	A, S	- - -	A = 0.25g Type III (S=1.5)	- - -	A = 0.15g Type III (S=1.5)
2500-YR	NCHRP 12-49	F_a, S_s, F_v, S_1	2/3*(Site Class E) or Site Specific	Site Class E (F_a=0.9, S_s=1.250, F_v=2.4, S_1=0.437)	2/3*(Site Class D) or Site Specific	Site Class D (F_a=1.60, S_s=0.091, F_v=2.40, S_1=0.0185)

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject WA SITE
SAP 2000 MODEL

Sheet ______ of ______
Job Number A99067-400
Designer MLT
Date 28-JUN-00



Project NCHRP 12-49

LIQUEFACTION STUDY

Subject WA SITE

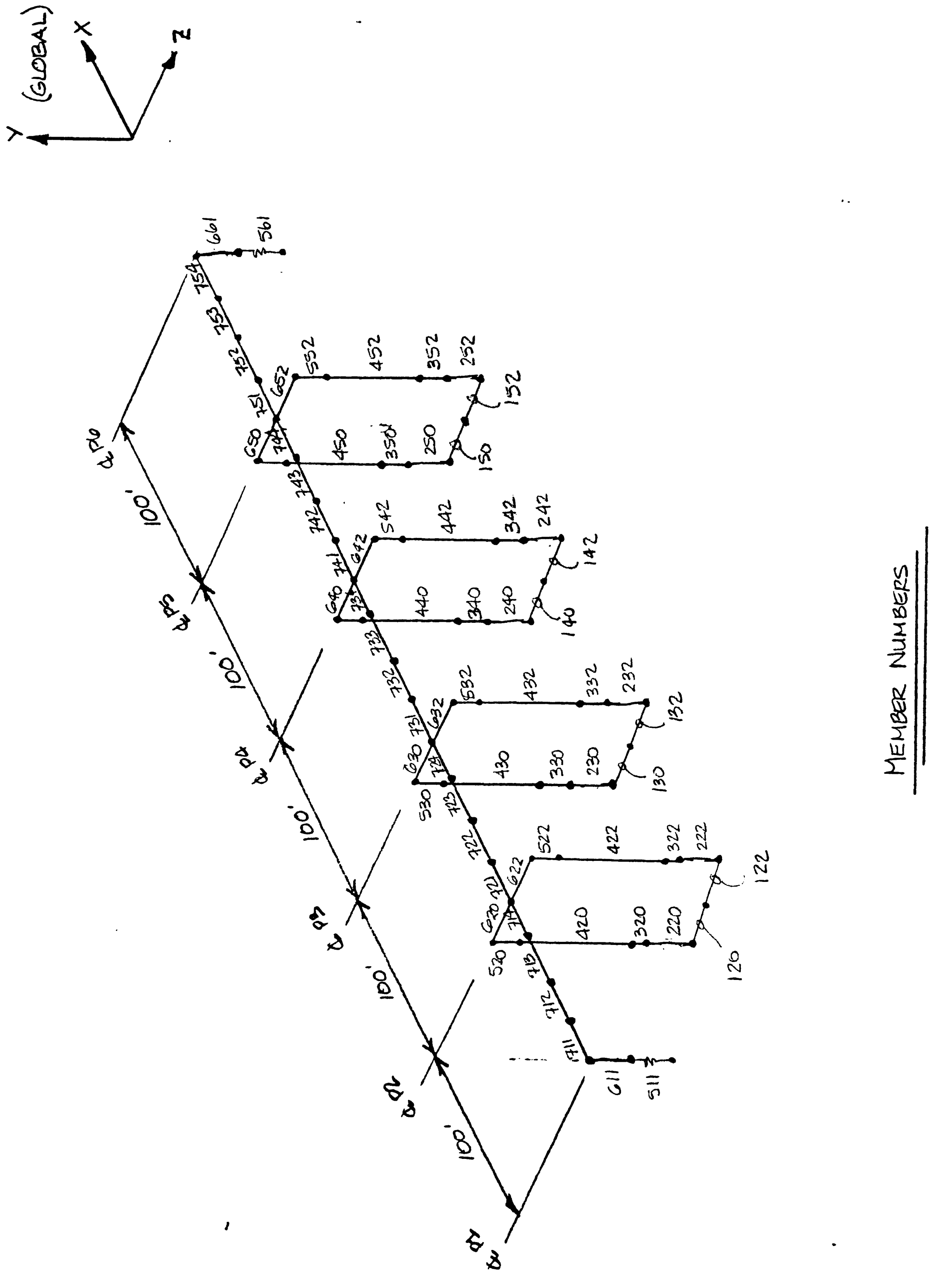
SAP 2000 MODEL

Sheet ______ of ______

Job Number A99067-400

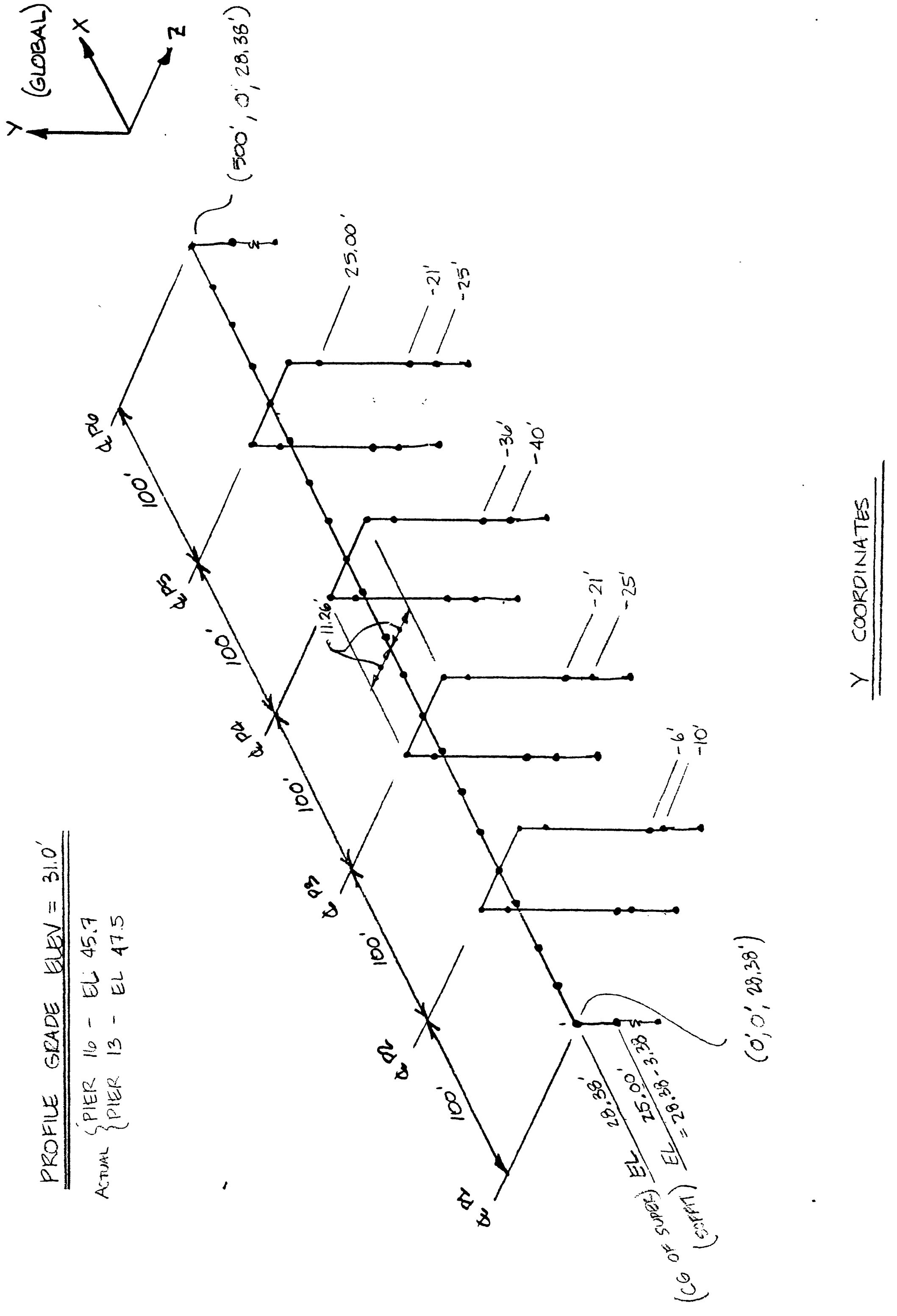
Designer MLT

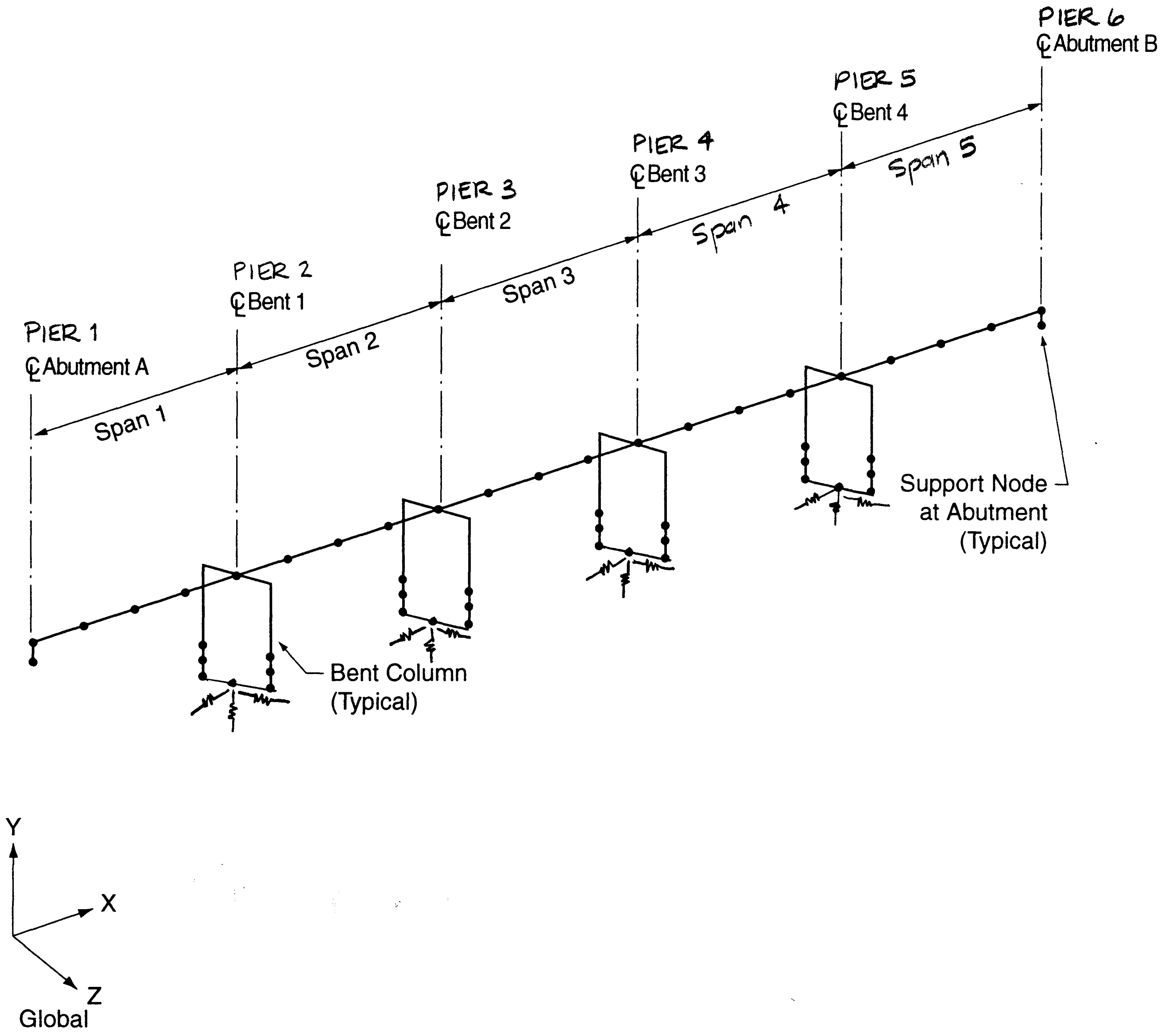
Date 28-JUN-00



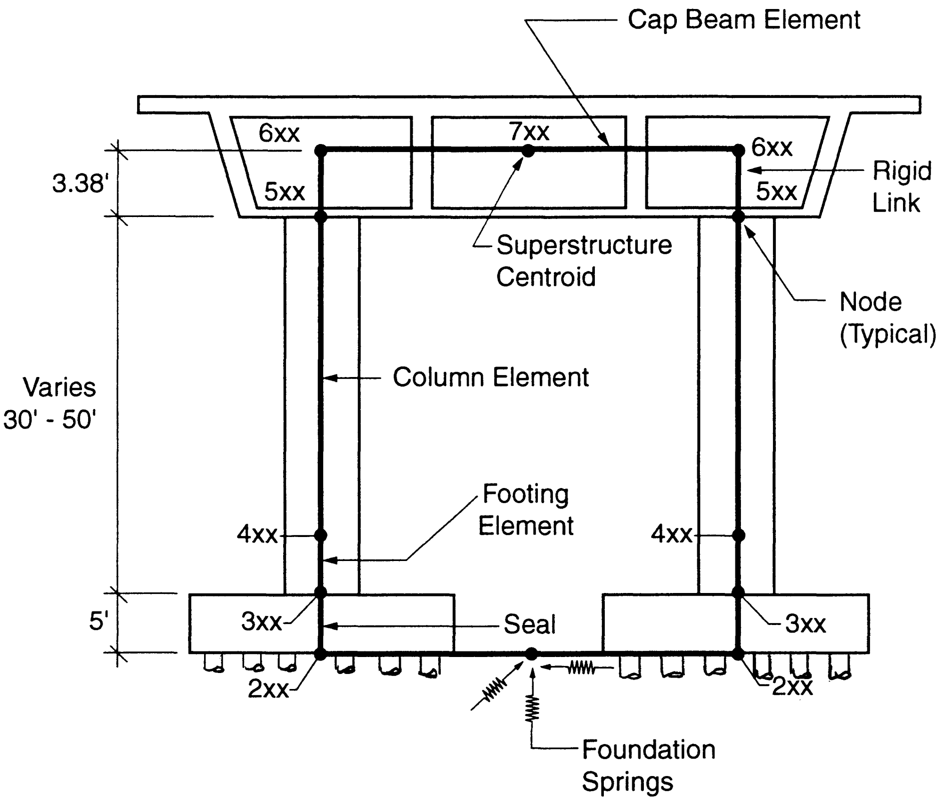
Project NCHRP 12-49
LIQUEFACTION STUDY
Subject WA SITE
SAP 2000 MODEL

Sheet ______ of ______
Job Number A99067-400
Designer MLT
Date 28-JUN-00



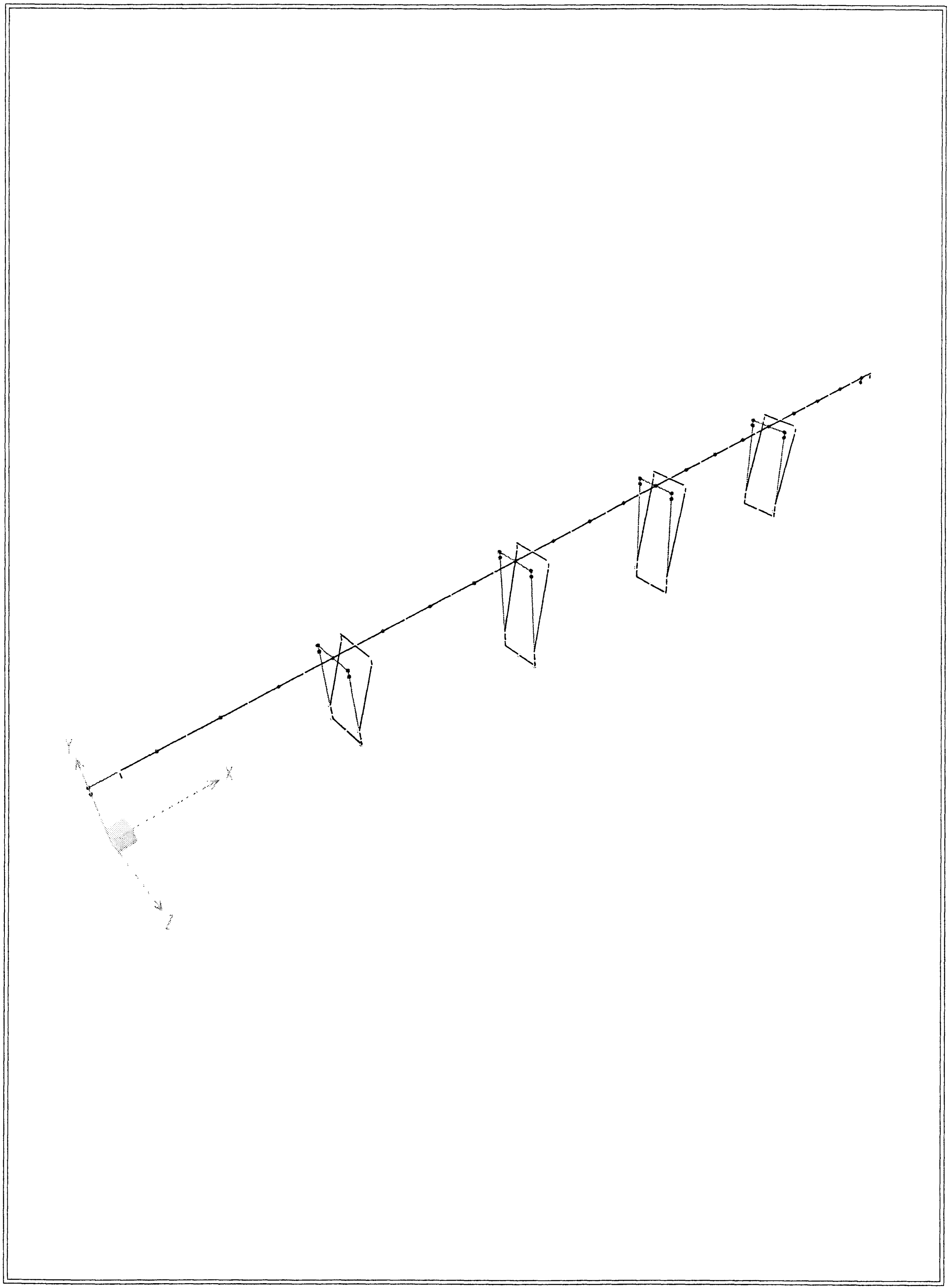


Structural Model of Bridge



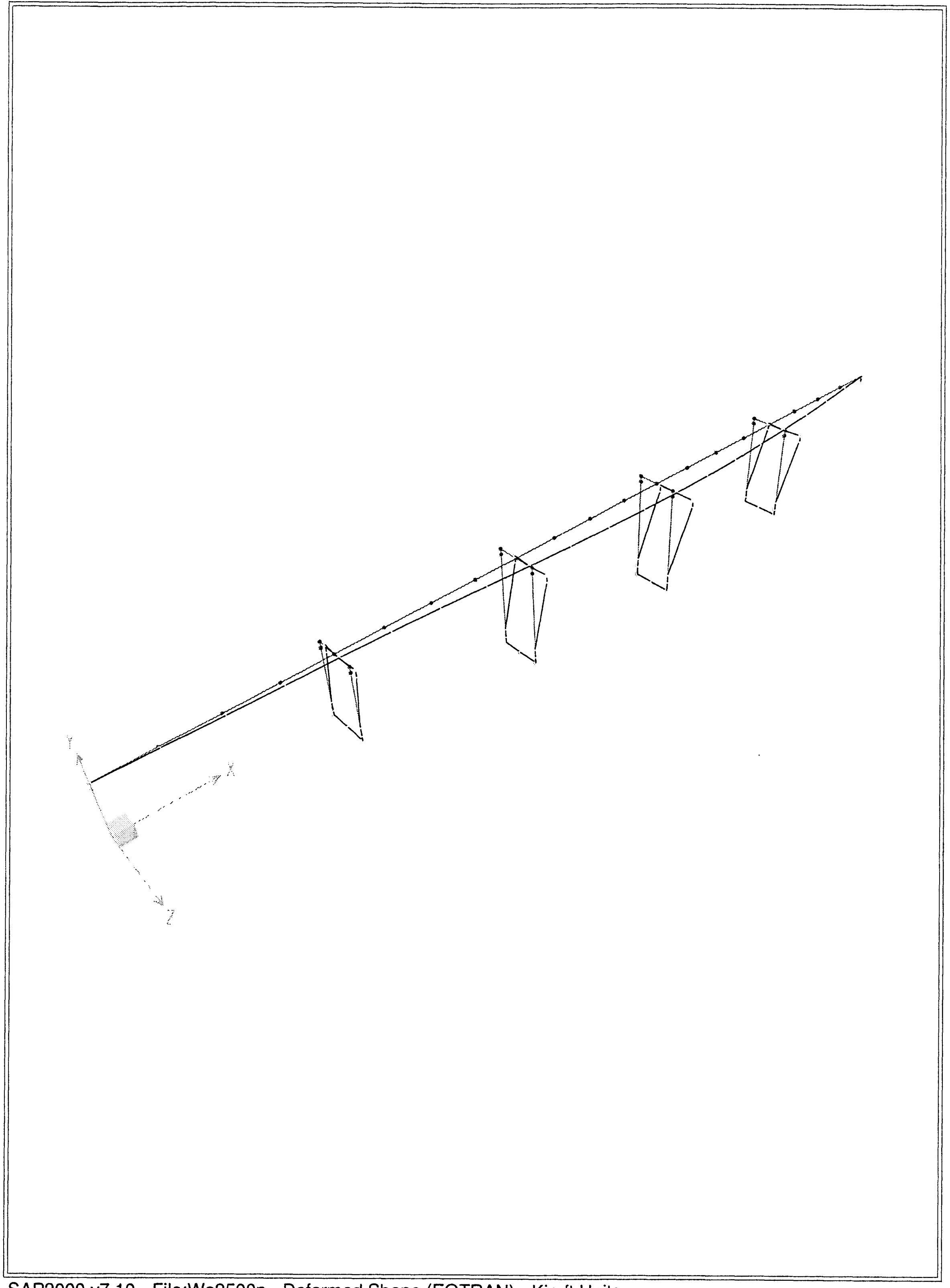
Details of Bent Elements

SAP2000 10/17/00 18:11:47



SAP2000 v7.10 - File:Wa2500n - Deformed Shape (EQLONG) - Kip-ft Units

SAP2000 10/17/00 18:11:55



SAP2000 v7.10 - File:Wa2500n - Deformed Shape (EQTRAN) - Kip-ft Units

E.2 SAP2000 Model Development: Section Properties and Loads

Design Step 6.1.2 (continued)

Table 1
Section Properties for Model

UNCRACKED SECTION PROPERTIES

Model Element

	CIP Box Superstructure	Bent Cap Beam	Bent Columns (Each)
Area (ft^2)	72.74	27.00	12.57
I x - Torsion (ft^4)	1,177	100,000 (1)	25.13
I y (ft^4)	9,697	100,000 (2)	12.57
I z (ft^4)	401	100,000 (3)	12.57
Density (lb / ft^3)	182 USE OK, EFFECTIVE	180 150	150

(1) This value has been increased for force distribution to bent columns. Actual value is I x = 139 ft^4.

(2) This value has been increased for force distribution to bent columns. Actual value is I y = 90 ft^4.

(3) This value has been increased for force distribution to bent columns. Actual value is I z = 63 ft^4.

Design Step 6.1.3

Substructure

The bents and abutments are skewed 30 degrees; therefore, the properties of the bent elements are rotated in the model to properly account for the skew. (There are no elements to model the abutments, only support nodes as shown in Figure 4). The bents are modeled with 3-D frame elements that represent the cap beam and individual columns. Figure 5 shows the relationship between the actual bent and the "stick" model of 3-D frame elements. A single element was used for each column between the top of footing and the soffit of the box girder superstructure. The connection of the column top at the soffit of the box girder to the center of gravity of the cap (at the superstructure centroid) beam is made with rigid link elements. The node at the top of the footing (4xx) is released for rotation in both plan directions to model the pinned column base. Foundation springs are connected to the node (3xx) at

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject WA SITE
SAP 2000 MODEL

Sheet ______ of ______
Job Number A99067 - 400
Designer MLT
Date 5 JULY 2000

CHECK SECTION PROPERTIES FROM DESIGN EXAMPLE NO. 4

CIP BOX SUPERSTRUCTURE

$$A = (43')(0.75') + (32')(0.667') + 2\left[(1')(4.583') + \overset{?}{(1')}\left(\frac{4.583'}{\sin 74.05^\circ}\right)\right]$$

$$= 32.25 + 21.33 + 9.167 + 9.533 \text{ ft}^2 \Rightarrow \underline{\underline{A = 72.3 \text{ ft}^2}}$$

(OK, CLOSE TO 72.74 ft²)

Should have been 72.18 ft² but use DE 4 I value.

$$y_b = \frac{(32.25 \text{ ft}^2)(5.625') + (9.167 + 9.533)(2.958') + (21.33 \text{ ft}^2)(0.333')}{72.3 \text{ ft}^2} \Rightarrow \underline{\underline{y_b = 3.37'}}$$

$$I_z = \frac{1}{12}\left[(43')(0.75')^3 + (32')(0.667')^3 + 4(1')(4.6')^3\right] + (32.25)(5.625' - 3.37')^2$$

$$+ (9.167 + 9.533)(2.958 - 3.37)^2 + (21.33)(0.333 - 3.37) \Rightarrow \underline{\underline{I_z = 398.2 \text{ ft}^4}}$$

(OK, CLOSE TO 401 ft⁴)

PIER COLUMNS [4' ø]

$$A = \frac{\pi}{4}(4')^2 \Rightarrow \underline{\underline{A = 12.57 \text{ ft}^2}} \text{ OK}$$

$$I_z = I_y = \frac{\pi}{4}(2')^4 \Rightarrow \underline{\underline{I_z = I_y = 12.57 \text{ ft}^4}} \text{ OK}$$

$$I_x = \frac{\pi}{2}(2')^4 \Rightarrow \underline{\underline{I_x = 25.13 \text{ ft}^4}} \text{ OK}$$

CRACKED PROP.
$$\begin{cases} I_{eff} = 0.4(12.57) = 5.0 \text{ ft}^4 \\ I_{x\,eff} = 0.4(25.13) = 10.0 \text{ ft}^4 \end{cases}$$

MODULUS OF ELASTICITY

$$E = 33000(0.150 \text{ kcf})^{1.5}\sqrt{4 \text{ ksi}}$$

$$= 3834 \text{ ksi} \times \frac{144 \text{ in}^2}{1 \text{ ft}^2} \Rightarrow \underline{\underline{E_{CONC} = 552000 \text{ k/ft}^2}}$$

$$E_{SEAL} = 33000(0.140 \text{ kcf})^{1.5}\sqrt{3 \text{ ksi}}$$

$$= 2994 \text{ ksi} \times 144 \Rightarrow E_{seal} = 431000$$

Project NCHRP 12-49

Sheet ______ of ______

Job Number A99067-400

Subject WA SITE

SAP 2000 MODEL

Designer MLT

Date 24 JULY 2000

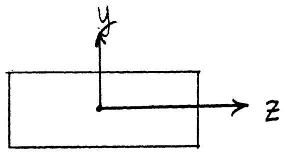
PILE CAP AT INTERIOR PIERS

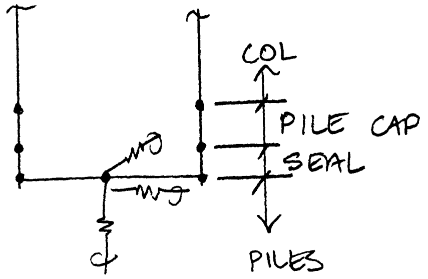
$A_{FTG} = (22')(46')$ $\Rightarrow A_{FTG} = 1012\ ft^2$

$I_z = \frac{1}{12}(46)(22)^3$ $I_z = 40817\ ft^4$

$I_y = \frac{1}{12}(22)(46)^3$ $I_y = 178449\ ft^4$

$J = \frac{1}{12}(1012 ft^2)(22^2 + 46^2)$ $\Rightarrow J = 219267\ ft^4$





~ ASSUME 1/2 OF PROPERTIES ABOVE PER COLUMN LINE

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject WA SITE
SAP 2000 MODEL

Sheet ______ of ______
Job Number A99067 - 400
Designer MLT
Date 5 JULY 2000

CHECK LOADS

☐ SUPERIMPOSED DEAD LOAD

BARRIER: $w_b = 2(2.48\ ft^2)(0.150\ \frac{k}{ft^3}) \Rightarrow \underline{w_b = 0.744\ k/ft}$

OVERLAY: (3" ACP) $w_{OL} = (3''/12''/') (43' - 2(0.875'))(0.140\ kcf)$

$\Rightarrow \underline{w_{OL} = 1.444\ k/ft}$

$\Sigma = w_{SDL} = 2.19\ k/ft$

☐ INT. DIAPHRAGM LOAD

9" MIDSPAN DIAPHRAGM: (APPROX)

$$(9''/12''/')\Big[\underbrace{\big[(43')(9''/12) + \tfrac{1}{2}(32' + 35')(\tfrac{72''-9''}{12''/'})\big]}_{\text{WEBS + FLANGES + VOIDS}} - \underbrace{[72.18\ ft^2]}_{\text{WEBS + FLANGES}}\Big](0.150\ kcf)$$

$$= (0.75)[\underbrace{32.25 + 175.875}_{208.125} - 72.18](0.150) = 15.29^k$$

$\Rightarrow \boxed{P_{DIAPH} = 15.3^k}$

☐ CROSS BEAM LOAD (5' WIDE)

$(5')(208.125 - 72.18\ ft^2)(0.150) = 101.96^k$

$\Rightarrow \boxed{P_{XBEAM} = 102.0^k}$

☐ END DIAPHRAGM LOAD (4' WIDE)

$$(0.150)\Big[(4')\underbrace{(208.125')}_{\text{Solid box}} + \underbrace{(4'')(9'')(32')}_{\text{STUB FOR DRT}} - \underbrace{(2')(72.18ft^2)}_{\text{L × Hollow box (already in frame model (SAP))}}\Big] \Rightarrow \boxed{P_{E.DIAPH} = 117.6^k}$$

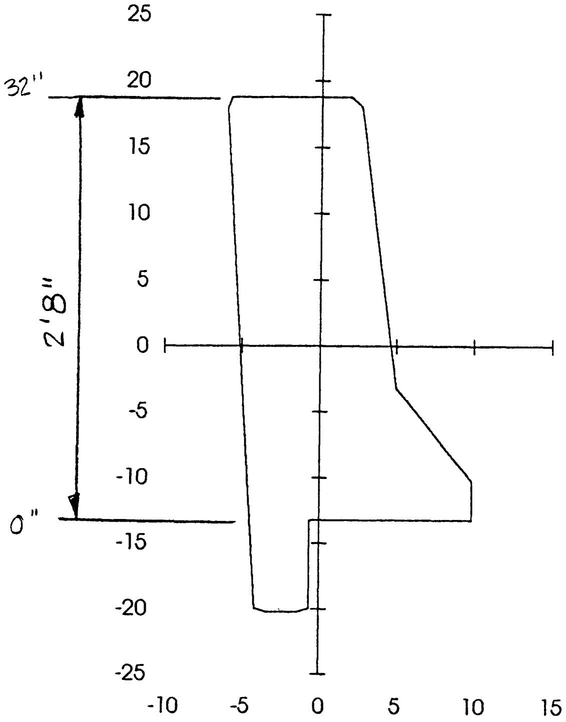
*

SECTION PROPERTIES WORKSHEET

PROJECT: WASHINGTON SITE ~~NISQUALLY BNSF RR OVERXING~~
SUBJECT: **BARRIER PROPERTIES**
DESIGNER: MLT
DATE: 10-Jul-00

SECTION COORDINATES (CLOCKWISE):

X	Y
0.0000	0.0000
-10.5000	0.0000
-10.5000	-6.7500
-11.2500	-7.0000
-13.2500	-7.0000
-14.0000	-6.7500
-15.8750	31.2500
-15.6250	32.0000
-8.0000	32.0000
-7.2500	31.2500
-4.8750	10.0000
0.0000	3.0000
0.0000	0.0000
END	



HOLE COORDINATES (COUNTERCLOCKWISE):

X	Y

CIRCULAR HOLES:

Xo	Yo	D

RECTANGULAR HOLES:

Xo	Yo	B	H

SECTION PROPERTIES:
L = inches

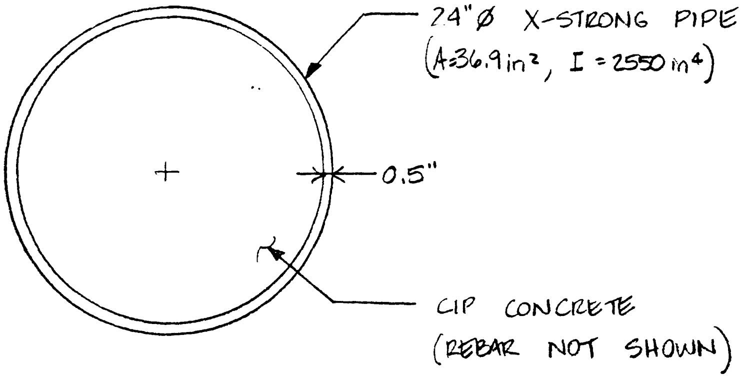
Area:	357.6406	L^2
Xbar:	-9.8683	L
Ybar:	13.2225	L
Ix:	37144.7703	L^4
Iy:	4213.8878	L^4
S top:	1978.1579	L^3
S bot:	1836.8000	L^3

$A = 2.48\ ft^2$ per barrier

E.3 SAP2000 Model Development: Lateral Pile (LPILE) Analyses

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject WA SITE
PILES
Sheet ____ of ____
Job Number A99067-400
Designer MLT
Date 25 JULY 2000

PILE SECTION PROPERTIES (NON-COMPOSITE)



AREA:

$$A_c = \frac{\pi D^2}{4} = \frac{\pi (23")^2}{4} \Rightarrow \underline{\underline{A_c = 415.5\ in^2}}$$

$$n = \frac{E_s}{E_c} = \frac{29000\ ksi}{33000(0.15)^{1.5}\sqrt{4}} = 7.6 \Rightarrow \underline{\underline{n = 8}}$$

$$A_{s_t} = (n)\left(\frac{\pi}{4}\right)(D_o^2 - D_i^2) = 8\left(\frac{\pi}{4}\right)(23.875^2 - 23^2) \Rightarrow \underline{\underline{A_{s_t} = 257.7\ in^2}}$$

Neglect 1/16" for corrosion.

(transformed)

$$A_{p_t} = A_c + A_{s_t} = 415.5 + 257.7 \Rightarrow \boxed{A_{p_t} = 673.2\ in^2}$$

MOMENT OF INERTIA:

$$I_c = \frac{\pi}{64} D^4 = \frac{\pi}{64}(23")^4 \Rightarrow \underline{\underline{I_c = 13737\ in^4}}$$

$$I_s = \frac{\pi}{64}(D_o^4 - D_i^4) = \frac{\pi}{64}(D_o^2 - D_i^2)(D_o^2 + D_i^2) = \frac{1}{16}\left[\left(\frac{\pi}{4}\right)(D_o^2 - D_i^2)\right](D_o^2 + D_i^2)$$

$$= \frac{1}{16} A_s(D_o^2 + D_i^2) \simeq \frac{1}{16} A_s(D_{\text{℄}}^2)$$

very close approximation

$$\therefore I_{s_t} = \frac{1}{16} A_{s_t}(D_o^2 + D_i^2) = \frac{1}{16}(257.7)(23.875^2 + 23^2) \Rightarrow \underline{\underline{I_{s_t} = 17702\ in^4}}$$

$$I_{p_t} = 13737\ in^4 + 17702\ in^4 \Rightarrow \boxed{I_{p_t} = 31439\ in^4}$$

USE $E_c = 552000$ ksf
$= 3830$ ksi

Project

Sheet of

Job Number A99067 - 400

Subject

Designer MLT

Date 31 JULY 2000

FOR STEEL CASING:

$$Z = \frac{1}{6}(D_o^3 - D_i^3) = \frac{1}{6}\left[(23.875'')^3 - (23'')^3\right]$$

$$\Rightarrow \underline{\underline{Z = 240.4 \text{ in}^3}}$$

$$\cong 0.139 \text{ ft}^3$$

CRACKED PILE:

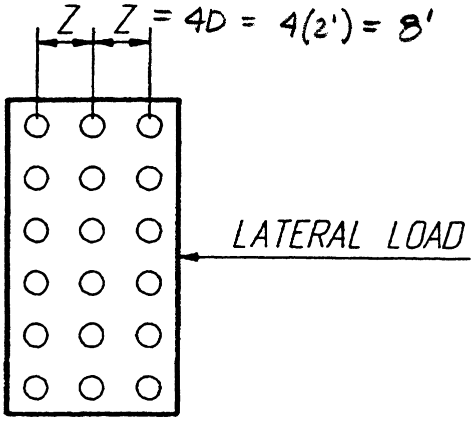
$$I_{cr} = I_{st} + 0.40\, I_c = 17702 \text{ in}^4 + (0.40)(13737 \text{ in}^4)$$

$$\Rightarrow \underline{\underline{I_{cr} = 23197 \text{ in}^4}}$$

ALL PIERS

Z	EFFICIENCY (REDUCTION) FACTOR
8D	1.0
6D	0.8
4D	0.5
3D	0.4

EFFICIENCY FACTOR



GROUP PILE PLAN

Efficiency Factor
Table 4.4.3-1

For driven piles, the following factors apply:

INPUT INTO LPILE 0.5K

Contact the Olympia Service Center Materials Lab to verify any assumptions.

The LPILE1 computer program will generate P-Y curves, or the user can input them. To obtain generated curves, input a modulus of subgrade reaction (K) and a soil shear strength (C) which are the values taken from the soils report multiplied by the efficiency factor. To figure P-Y curves for input, multiply the P-Y values from the soils report by the efficiency factor.

$0.5C_u$

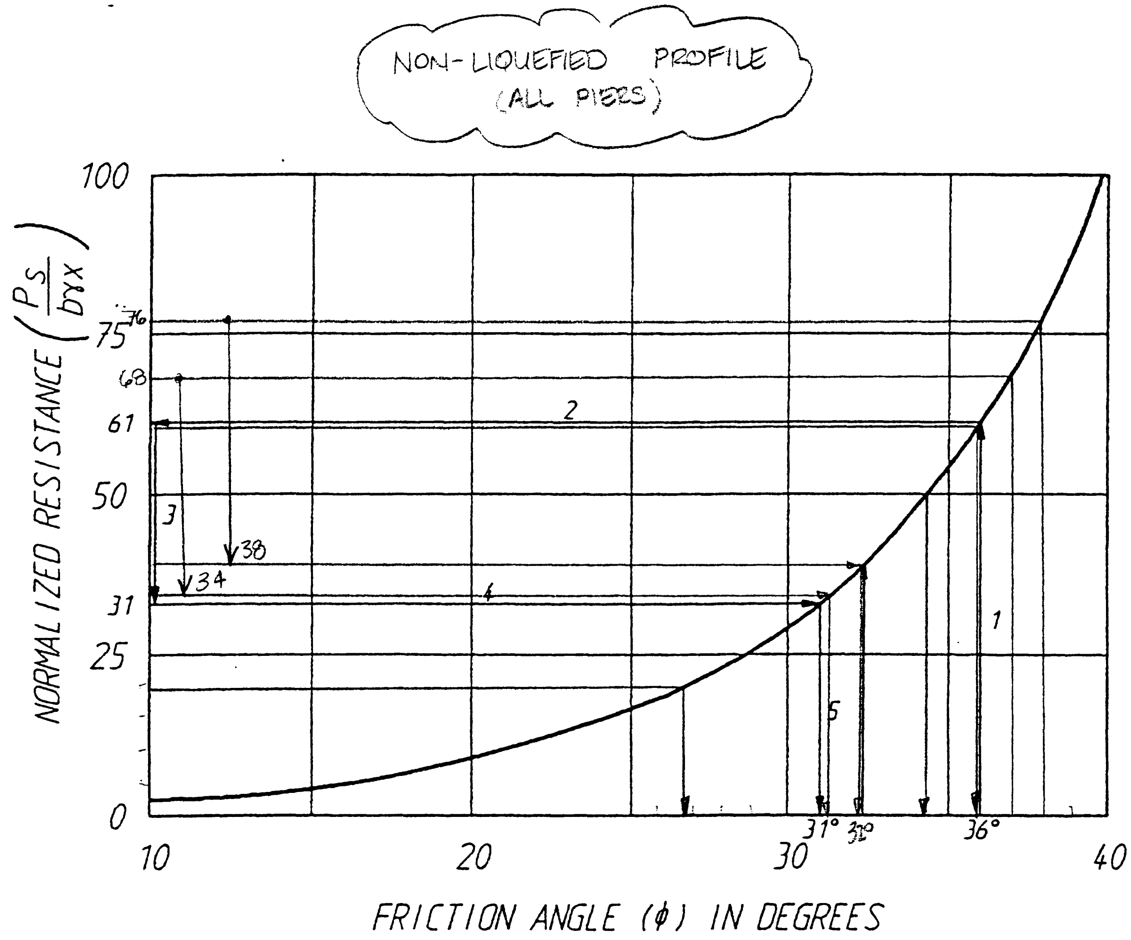
For a typical soil, the relationship between its normalized resistance value and friction angle is defined by the curve in Figure 4.4.3-1. The friction angle could be adjusted for efficiency and input to LPILE1 by following these steps:

$\phi \Rightarrow$

1. Begin at the coordinate of the natural friction angle (36°).
2. Read across to the normalized resistance (61).
3. Multiply the resistance by the efficiency reduction factor, i.e., 61 (0.5) = 31.
4. Read across from the reduced value to obtain the adjusted friction angle (31°).
5. Input the φ value to LPILE1.

$z = 8'$ $D = 2'$

$\frac{z}{D} = 4 \Rightarrow 4D$



EFFICIENCY FACTOR = 0.5

LAYER	$Ø_1$	NORM. RESIS.^{2}	N.RESIST. x0.5	$Ø_{LPILE}$
	-			-
②	37°	68	34	31°
③⑨	38°	76	38	32°
④⑦	40°	100	50	34°
⑤	42°	SAY 120	60	36°
⑥⑩	32°	38	19	28°

Friction Angle (φ)

$$\frac{PS}{b\gamma x} = K_a\,(\tan^8 B - 1) + K_o \tan\phi \tan^4 B$$

P_s = Soil Resistance on Pile Element

b = Pile Width

g = Soil Unit Weight

X = Depth to Pile Element

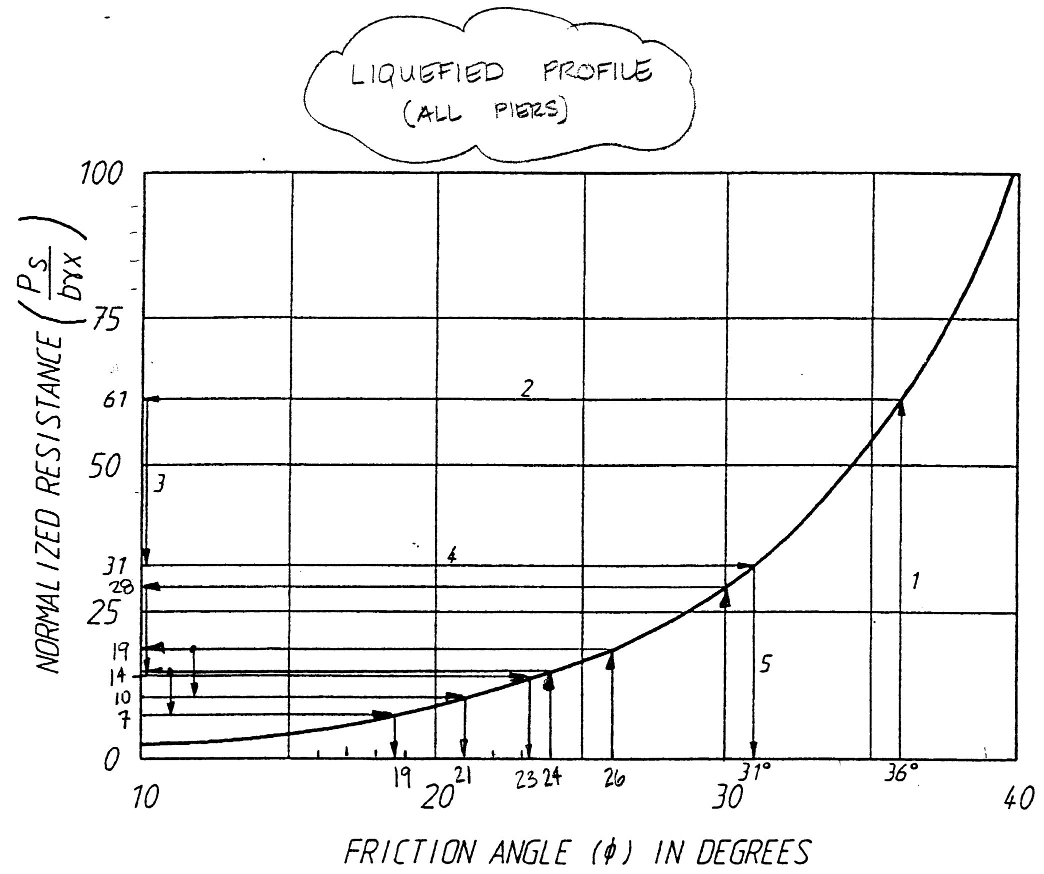
N = Step in Example

B = 45° + φ/2

K_a = tan2(45° – φ/2)

K_o = 1 – Sin φ

Figure 4.4.3-1



Friction Angle (φ)

$$\frac{PS}{b\gamma x} = K_a (\tan^8 B - 1) + K_o \tan \phi \tan^4 B$$

P_S = Soil Resistance on Pile Element

b = Pile Width

g = Soil Unit Weight

X = Depth to Pile Element

N = Step in Example

B = 45° + φ/2

K_a = tan2(45° – φ/2)

K_o = , 1 – Sin φ

Figure 4.4.3-1

EFFICIENCY FACTOR = 0.5

LAYER	Φ_1	NORM. RESIS.^{2}	N. RESIST. × 0.5	Ø_{LPILE}
④	30°	28	14	23
⑤	24°	14	7	19
⑥	30°	28	14	23
⑨	26°	19	10	21

1 - FROM GEOFF MARTIN

2 - FROM CHART

INPUT INTO LPILE

LPILE COORDINATES

WASHINGTON SITE

NON-LIQUEFIED SOIL PROFILE

			Pier					
			1	2	3	4	5	6
Layer No.	Soil Type	Top of Soil Layer Elevations	Pile Head Elevation					
			13	-13	-29	-36	-29	13
1	Sand (Fill)	23	-120					-120
2	Soft Clay	0	156	-156				156
3	Sand (Liquefiable)	-10	276	-36				276
4	Sand	-15	336	24	-168	-192	-168	336
5	Sand	-25	456	144	-48	-132	-48	456
6	Sand	-30	516	204	12	-72	12	516
7	Sand	-35	576	264	72	-12	72	576
8	Soft Clay	-40	636	324	132	48	132	636
9	Sand (Liquefiable)	-45	696	384	192	108	192	696
10	Sand	-50	756	444	252	168	252	756
	Pile Tip Elevation	-180	2316	2004	1812	1728	1812	2316

LIQUEFIED SOIL PROFILE

			Pier					
			1	2	3	4	5	6
Layer No.	Soil Type	Top of Soil Layer Elevations	Pile Head Elevation					
			13	-13	-29	-36	-29	13
1	Sand (Fill)	23	-120					-120
2	Soft Clay	0	156	-96				156
3	Soft Clay	-10	276	-36	-168		-168	276
4	Sand	-20	396	84	-108	-132	-108	396
5	Sand	-30	516	204	12	-72	12	516
6	Sand	-35	576	264	72	-12	72	576
7	Soft Clay	-40	636	324	132	48	132	636
8	Soft Clay	-45	696	384	192	108	192	696
9	Sand	-55	816	504	312	228	312	816
10	Stiff Clay w/o Free Water	-100	1356	1044	852	768	852	1356
	Pile Tip Elevation	-180	2316	2004	1812	1728	1812	2316

LPILE COORDINATES

WASHINGTON SITE

NON-LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations
1	Sand (Fill)	23
2	Soft Clay	0
3	Sand (Liquefiable)	-10
4	Sand	-15
5	Sand	-25
6	Sand	-30
7	Sand	-35
8	Soft Clay	-40
9	Sand (Liquefiable)	-45
10	Sand	-50
	Pile Tip Elevation	-180

P-Y Curve Depths at Mid-Depth of Each Soil Layer

Pier					
1	2	3	4	5	6
18					18
216	-96				216
306	-6				306
396	84	-108	-162	-108	396
486	174	-18	-102	-18	486
546	234	42	-42	42	546
606	294	102	18	102	606
666	354	162	78	162	666
726	414	222	138	222	726
1536	1224	1032	948	1032	1536

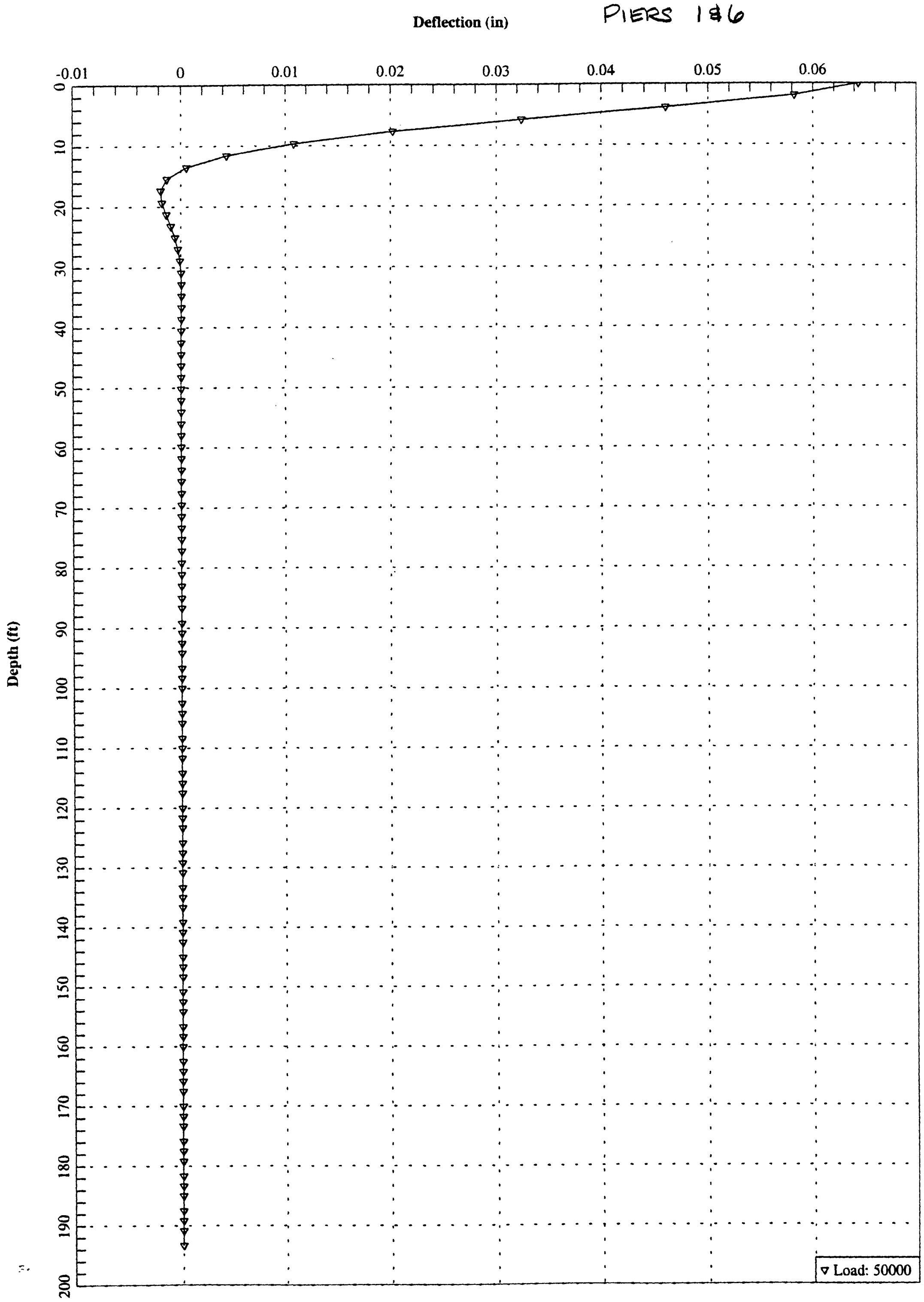
LIQUEFIED SOIL PROFILE

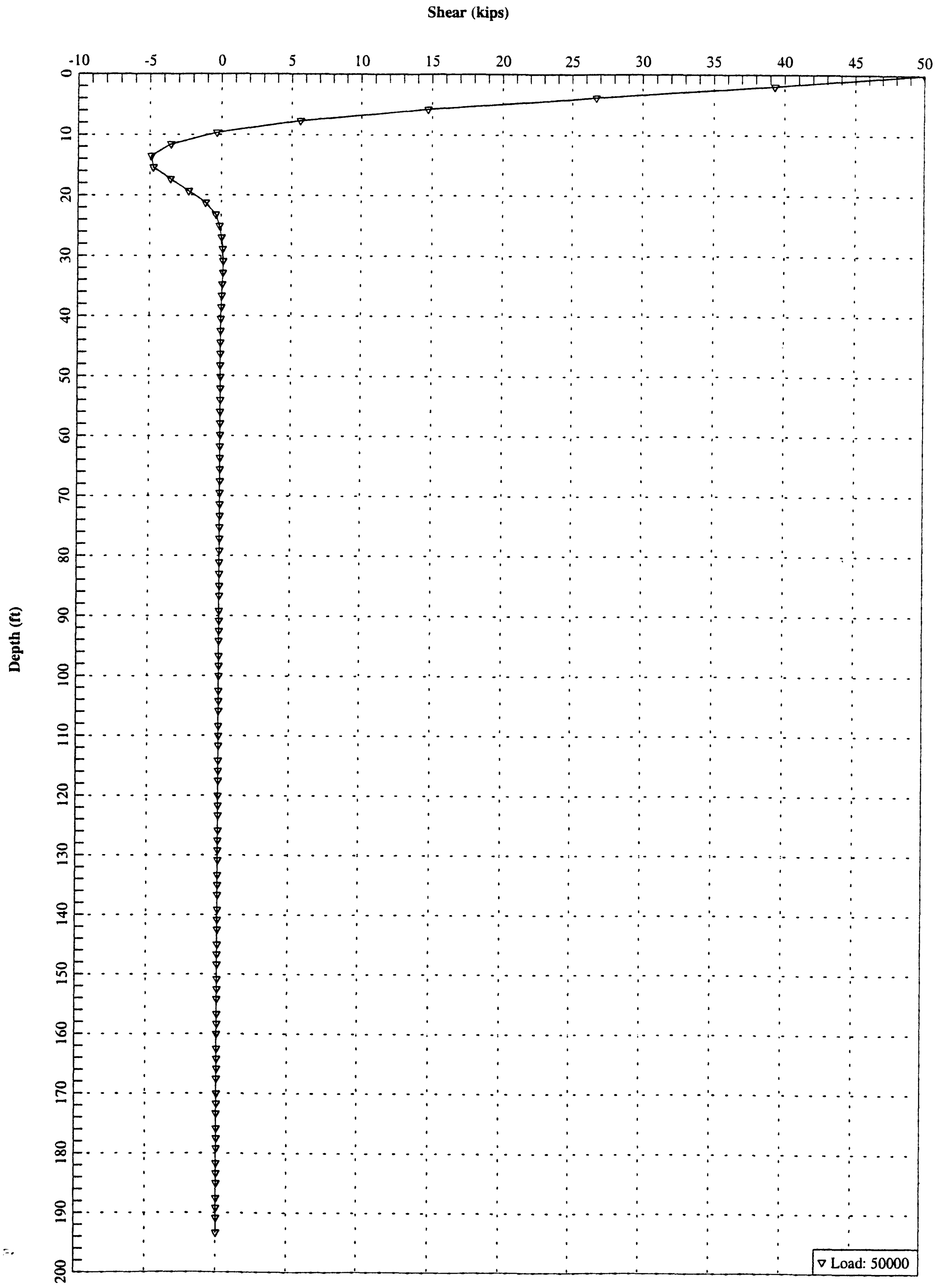
Layer No.	Soil Type	Top of Soil Layer Elevations
1	Sand (Fill)	23
2	Soft Clay	0
3	Soft Clay	-10
4	Sand	-20
5	Sand	-30
6	Sand	-35
7	Soft Clay	-40
8	Soft Clay	-45
9	Sand	-55
10	Stiff Clay w/o Free Water	-100
	Pile Tip Elevation	-180

P-Y Curve Depths at Mid-Depth of Each Soil Layer

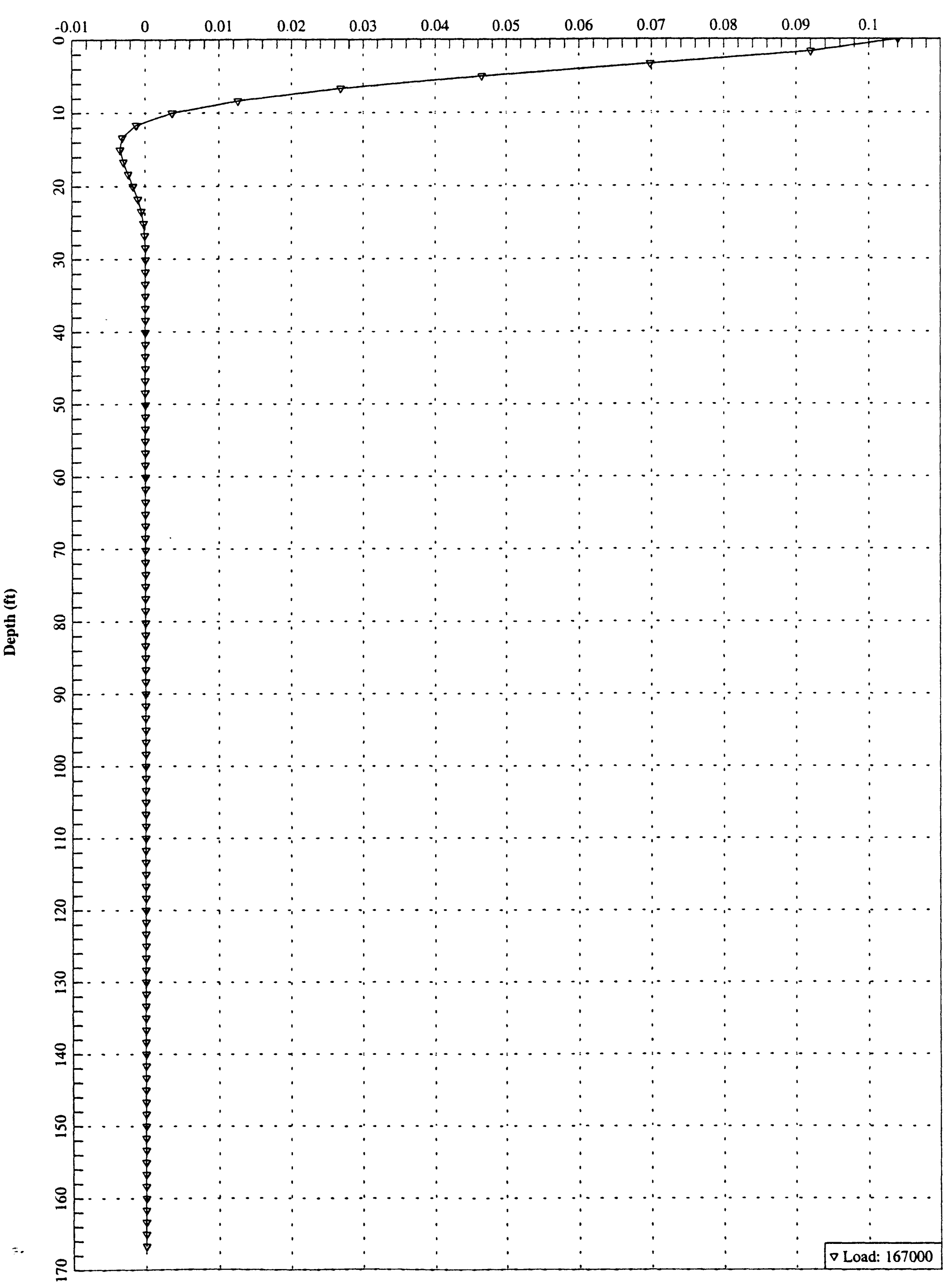
Pier					
1	2	3	4	5	6
18					18
216	-66				216
336	24	-138		-138	336
456	144	-48	-102	-48	456
546	234	42	-42	42	546
606	294	102	18	102	606
666	354	162	78	162	666
756	444	252	168	252	756
1086	774	582	498	582	1086
1836	1524	1332	1248	1332	1836

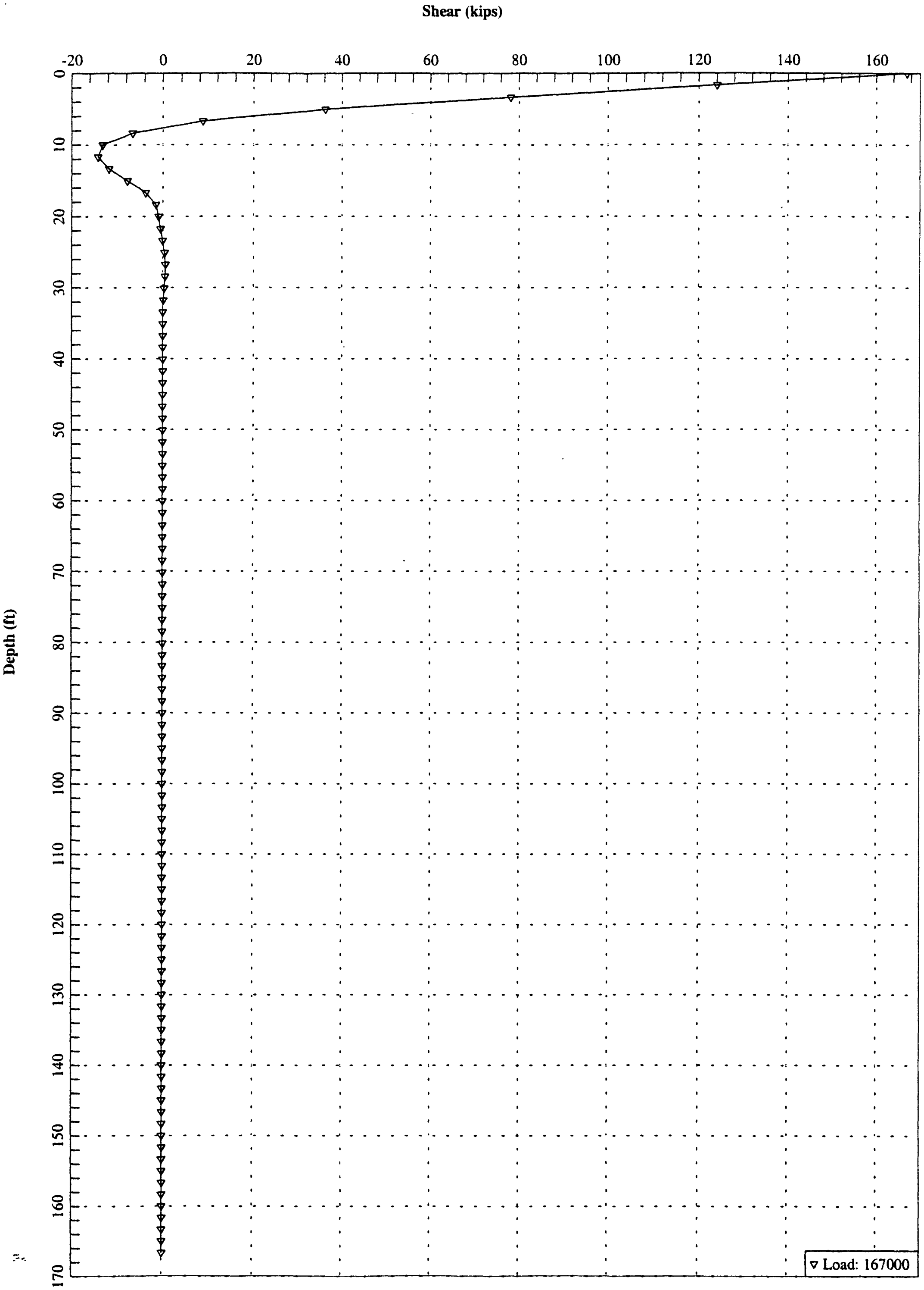
NON-LIQUEFIED
PIERS 1 & 6



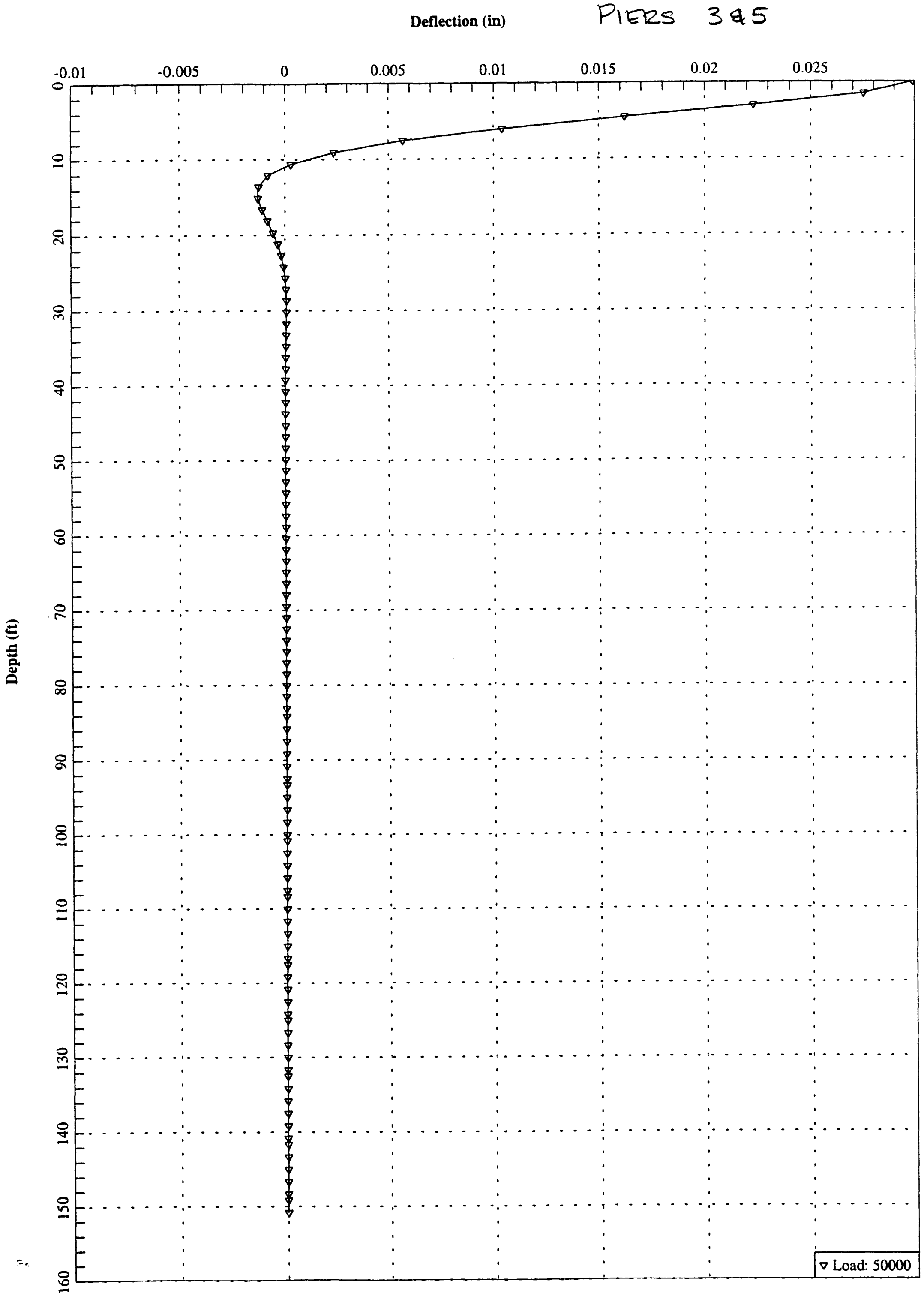

Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
▿ Load: 50000

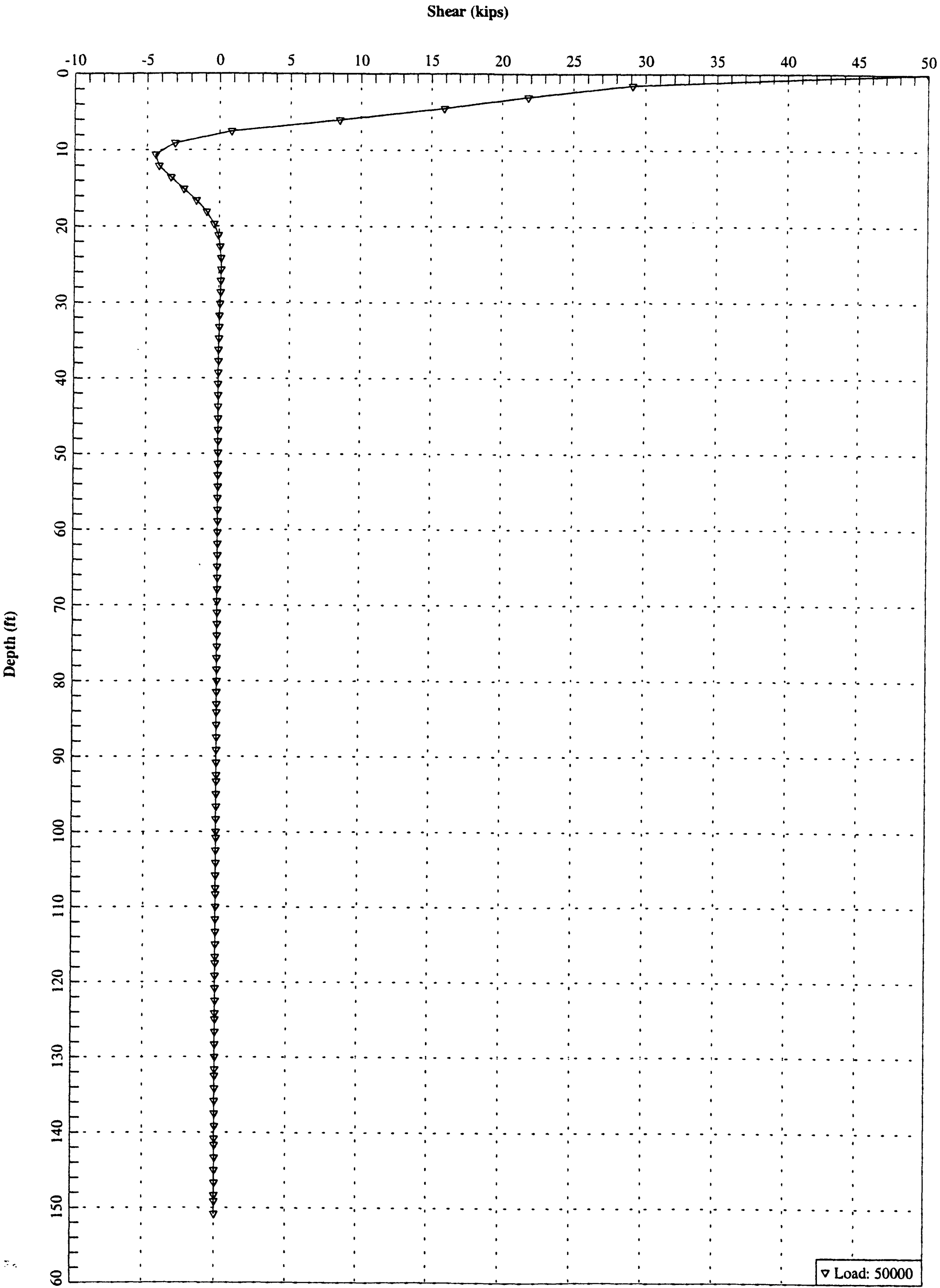
NON- LIQUEFIED
PIER 2



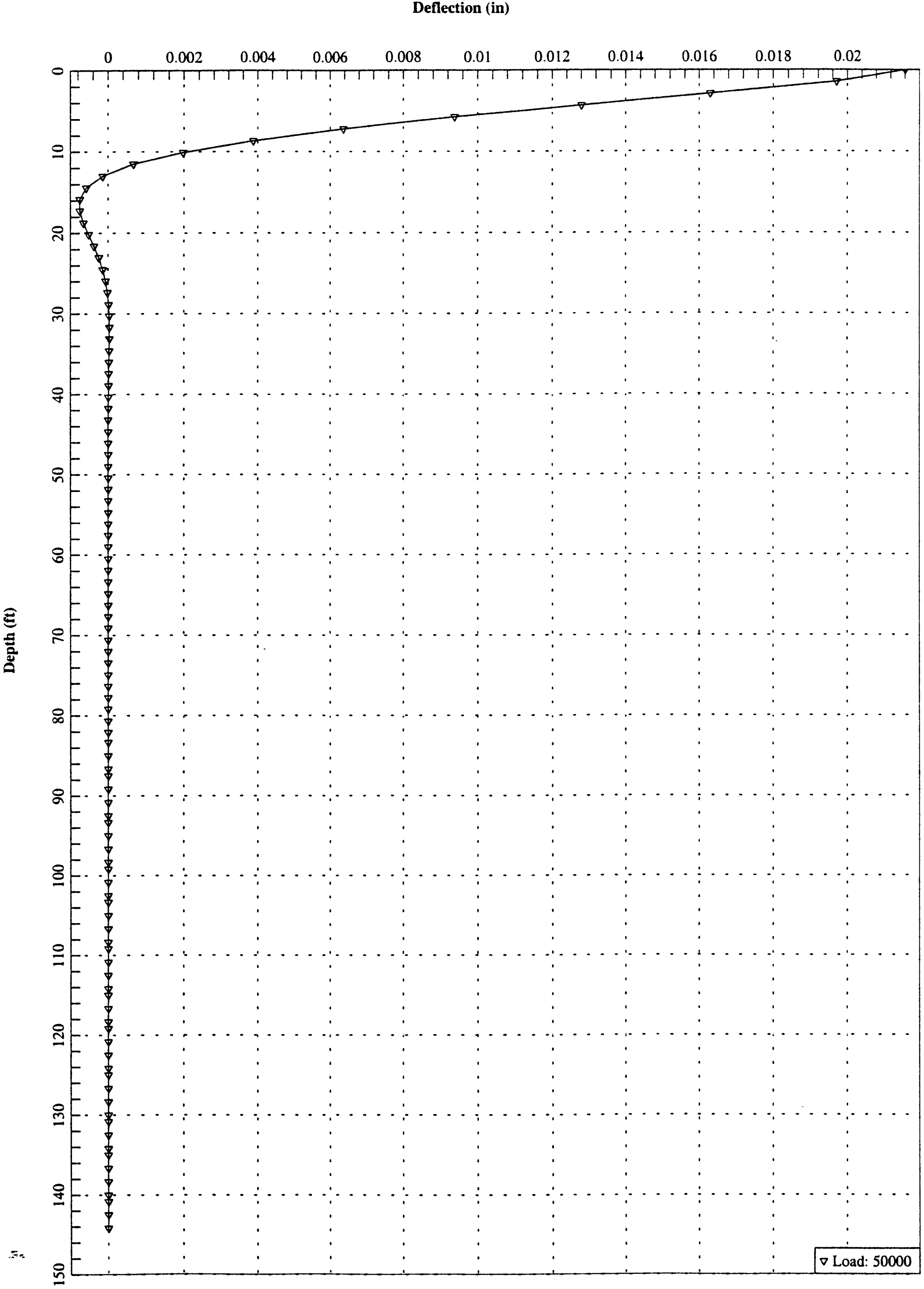

Shear (kips)
-20
0
20
40
60
80
100
120
140
160
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
▿ Load: 167000

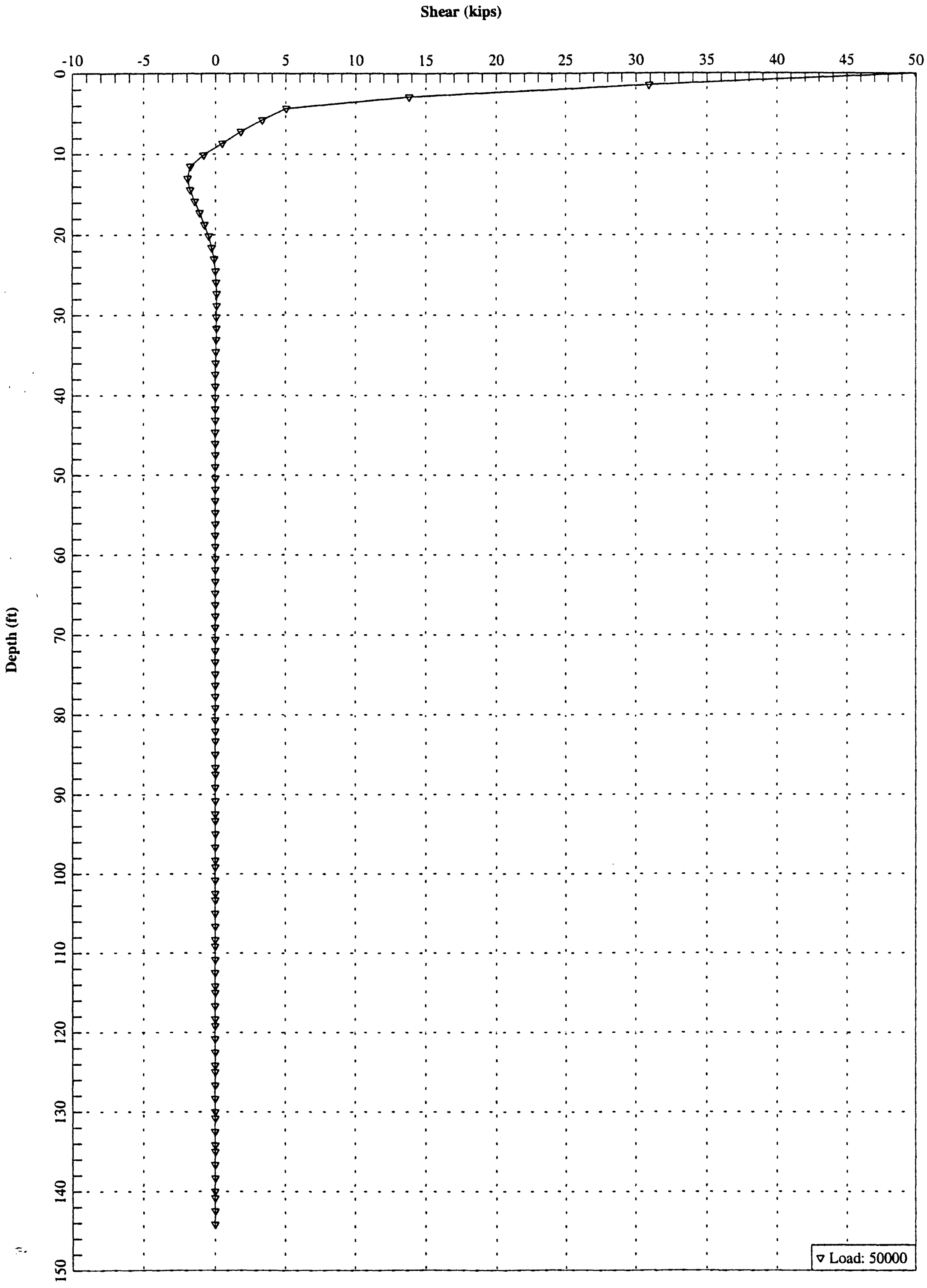
NON- LIQUEFIED
PIERS 3 & 5



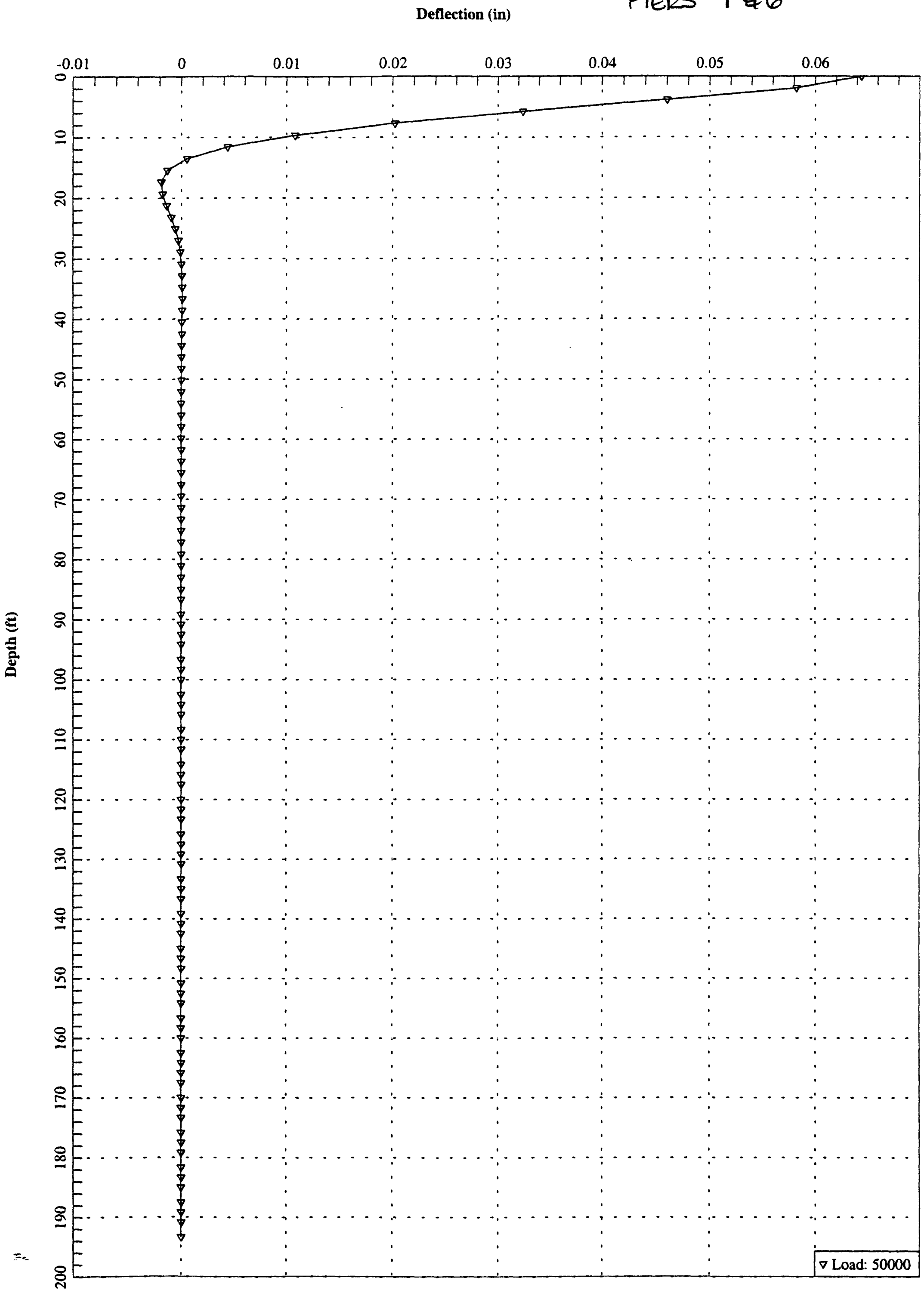

Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
▿ Load: 50000

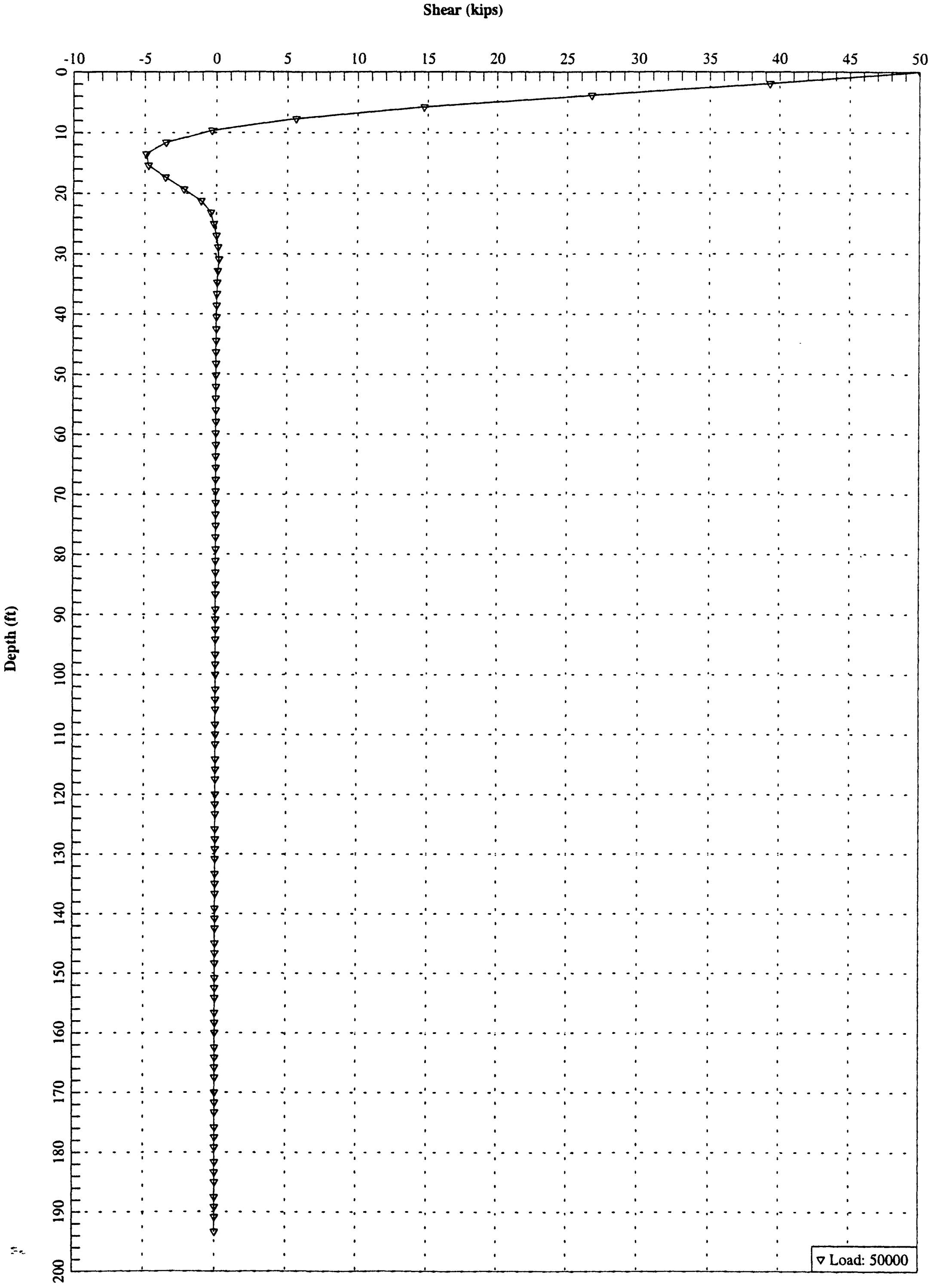
NON- LIQUEFIED
PIER 4



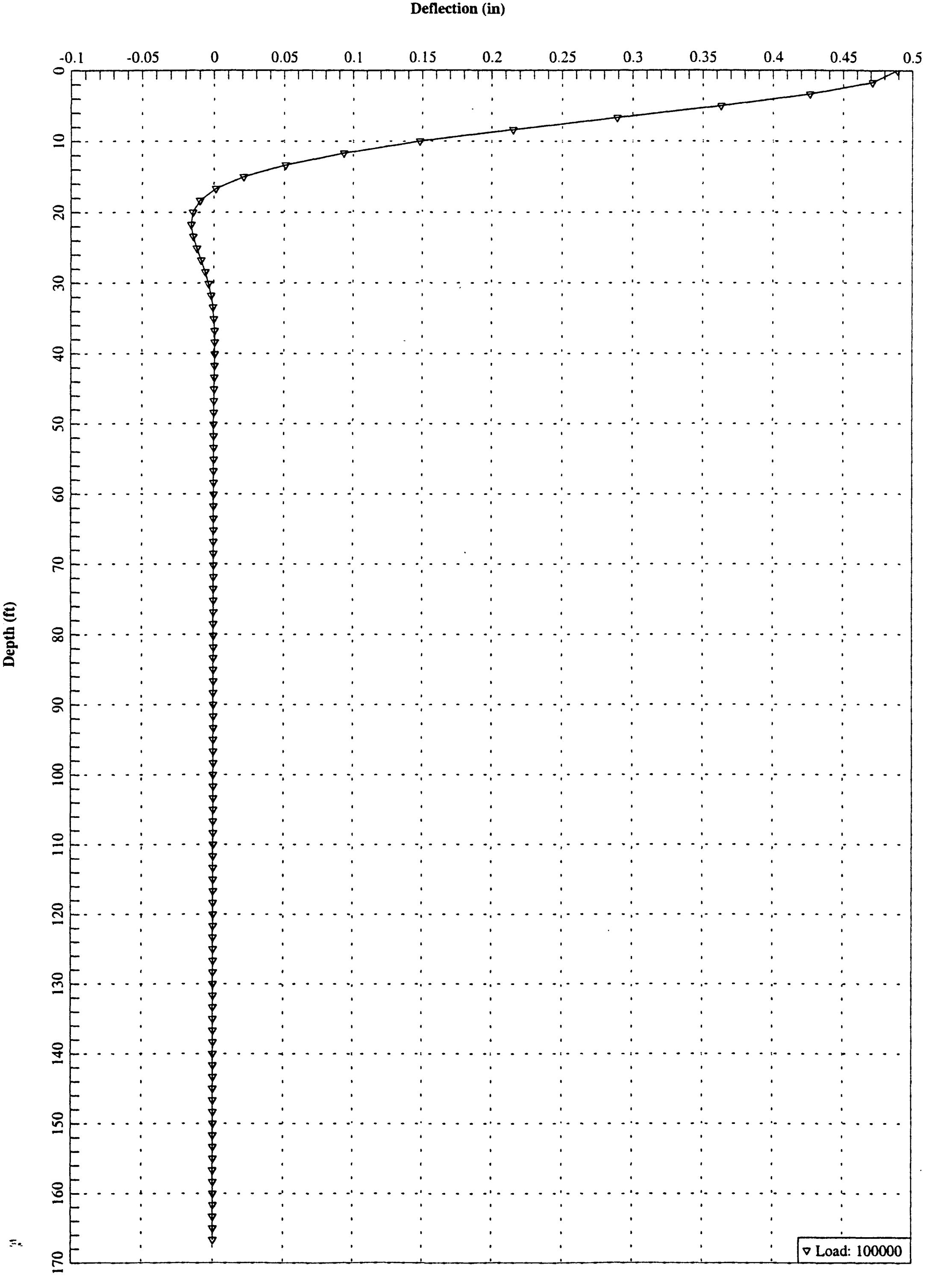

Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
▿ Load: 50000

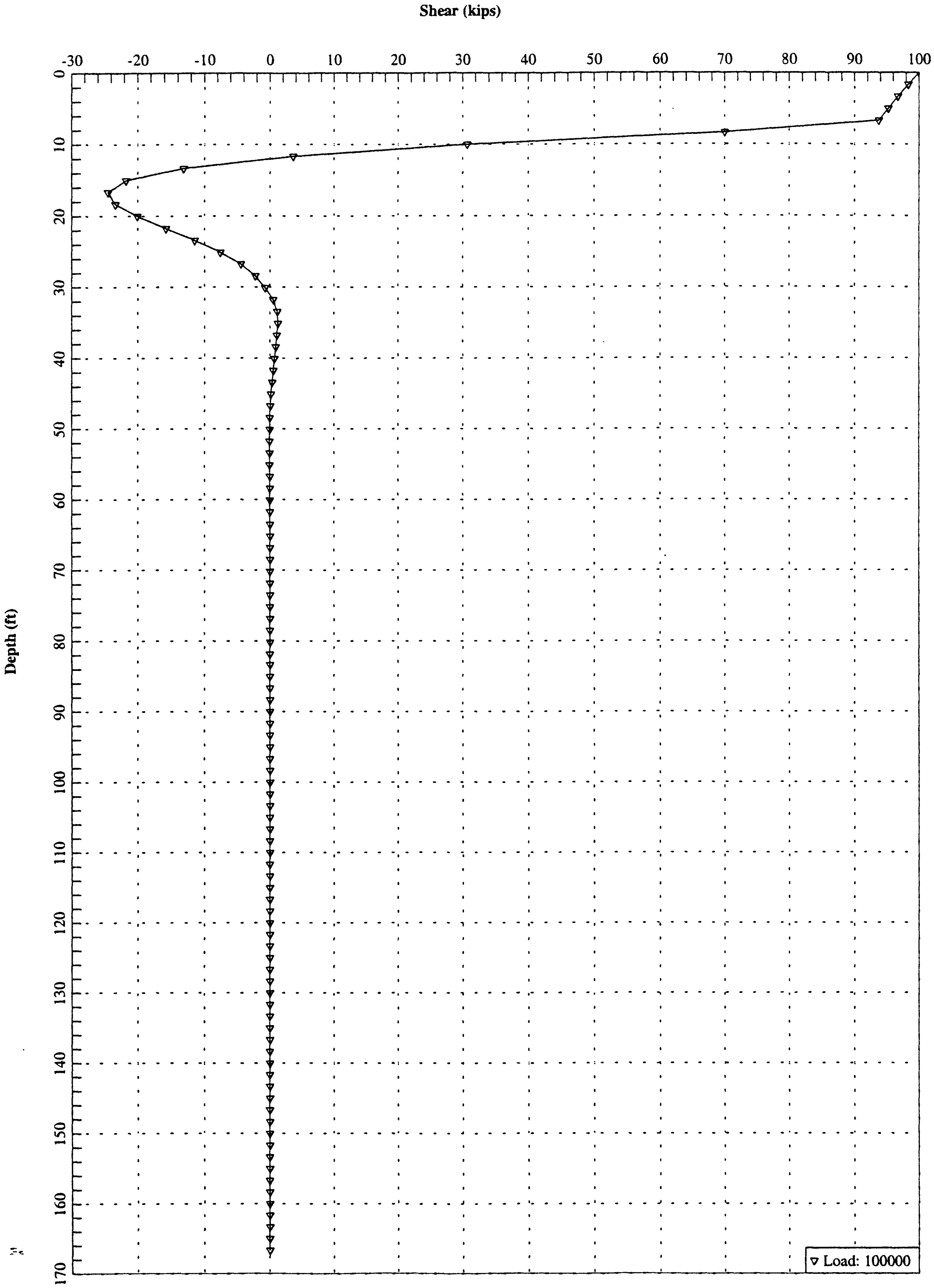
LIQUEFIED
PIERS 1 & 6



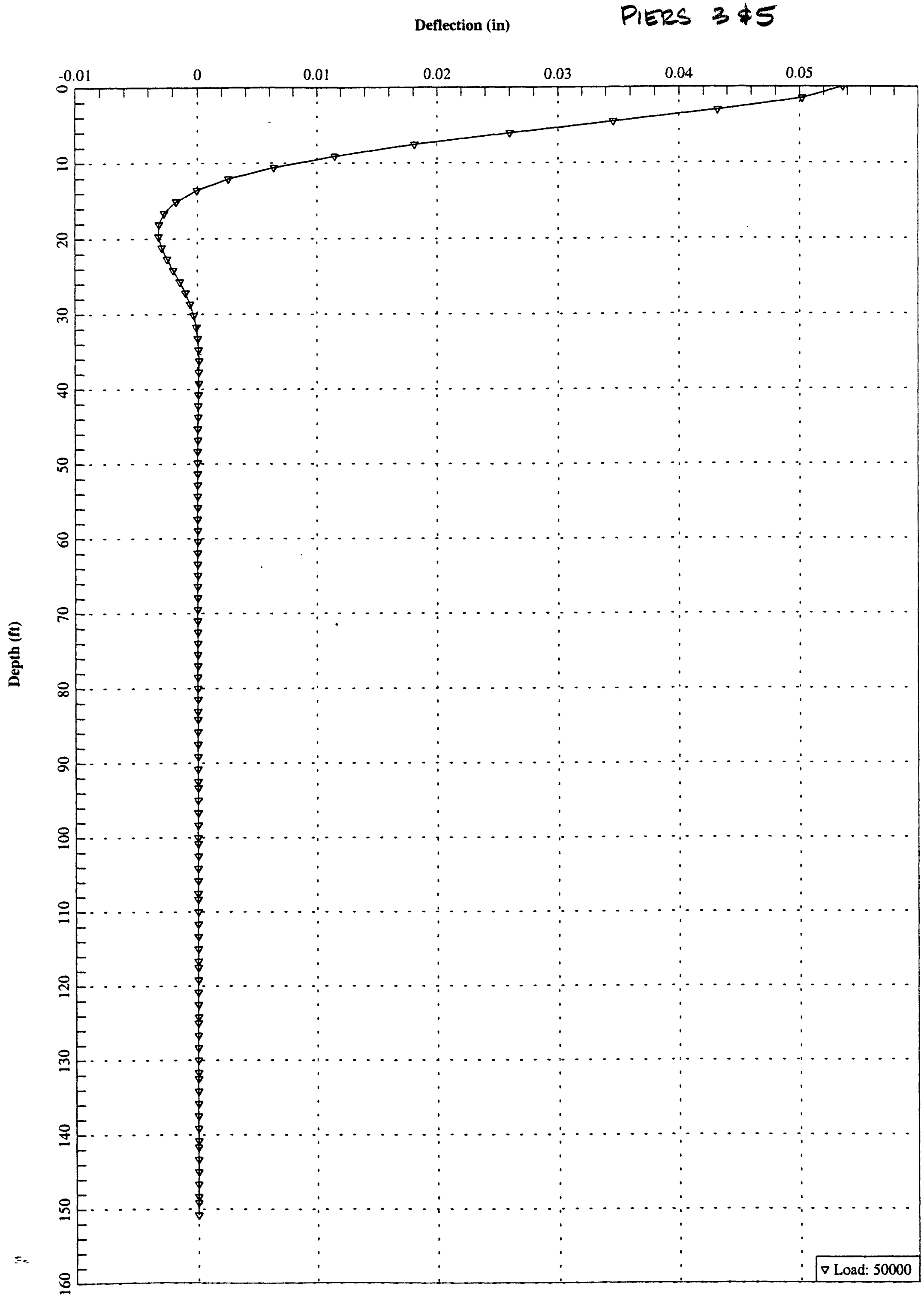

Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180
190
200
▽ Load: 50000

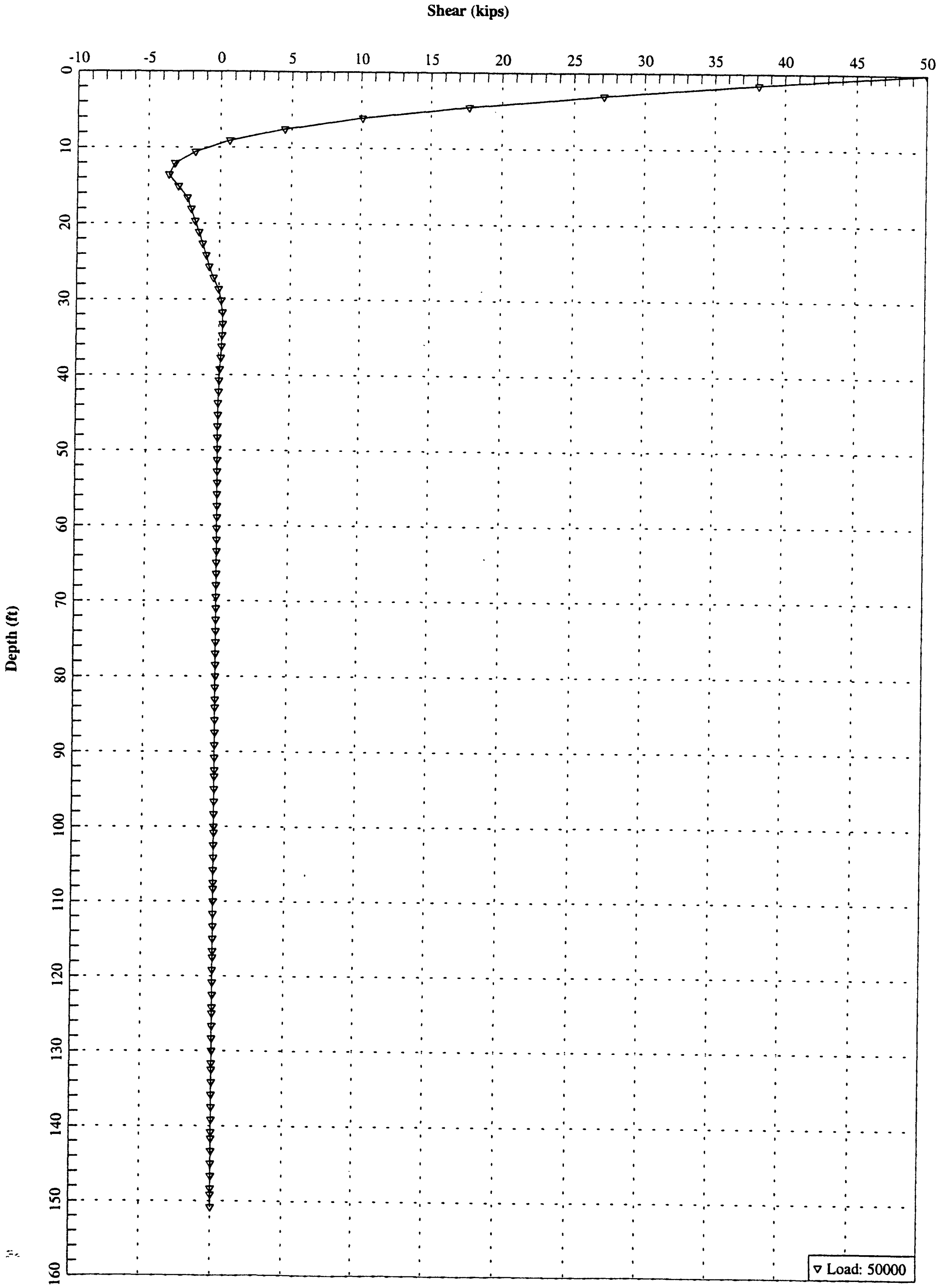
LIQUEFIED
PIER 2



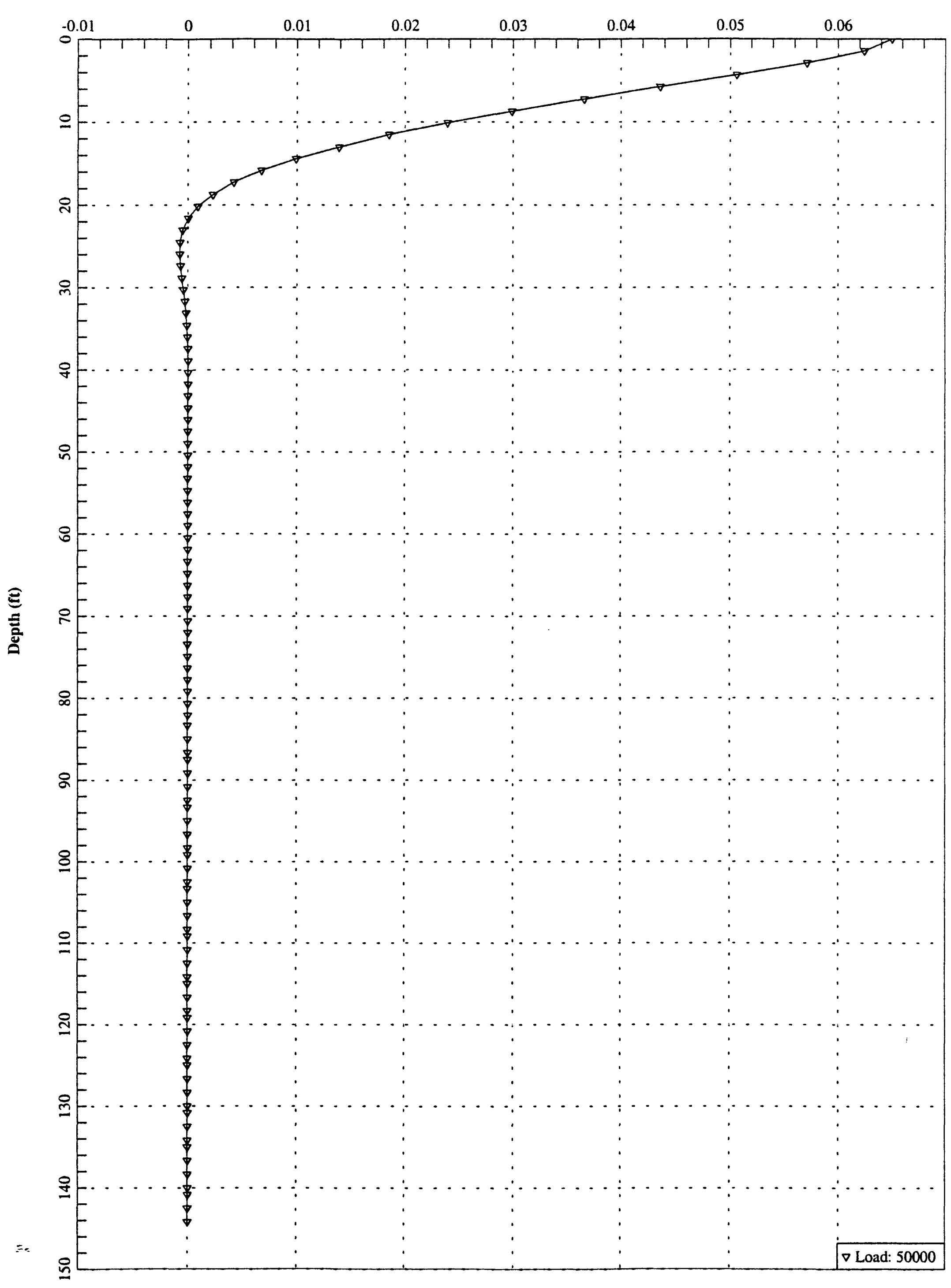

Shear (kips)
-30
-20
-10
0
10
20
30
40
50
60
70
80
90
100
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
▿ Load: 100000

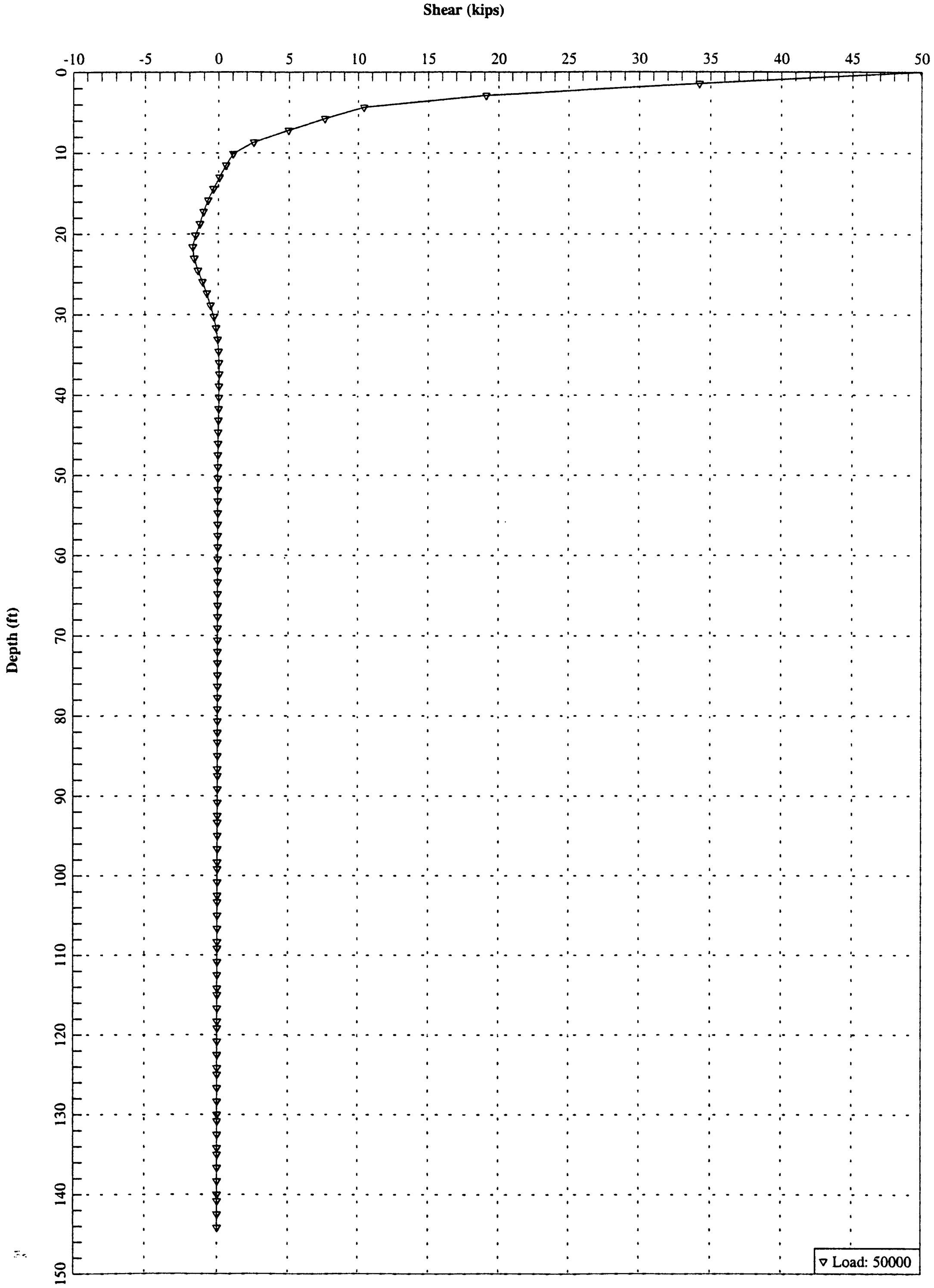
LIQUEFIED
PIERS 3 & 5

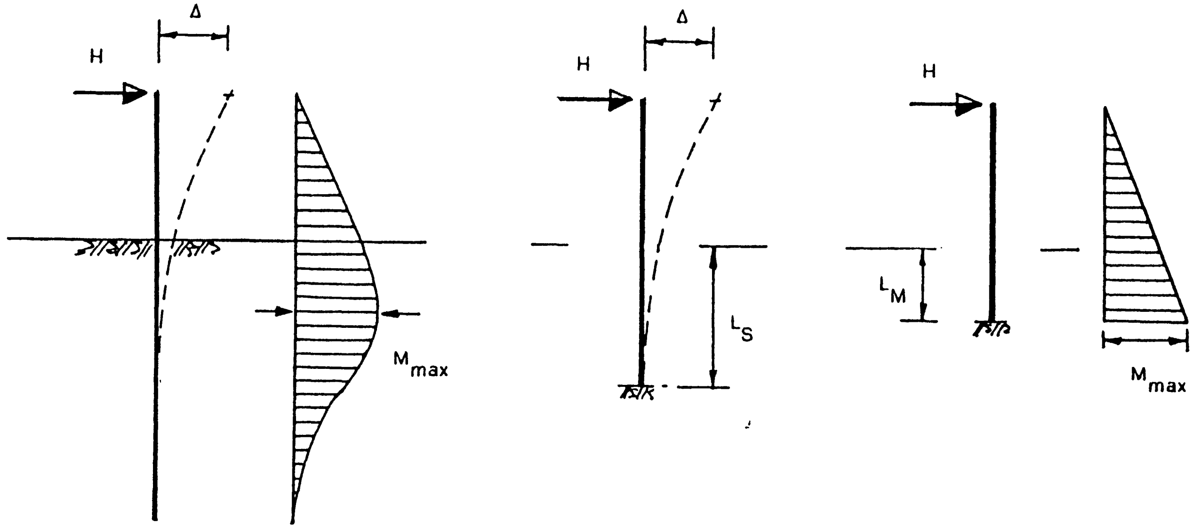



Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
▿ Load: 50000

LIQUEFIED
PIER 4




Shear (kips)
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
▿ Load: 50000



	L_S	L_M
Cohesive Soil Constant K_h	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless Soil Constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

Figure 69: Equivalent Cantilever Method using Relative Stiffness Factors (after References 29 and 36)

"T" DISTANCE

WASHINGTON SITE

$E_c =$ 3830 ksi

$I_{cr} =$ 23197 in.^{4}

$I =$ 31439 in.^{4}

NON-LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Subgrade Modulus K_h or n_h (pci)	$L_M = 2T$ feet	$L_S = T$ feet	$L_M = 2T$ feet	$L_S = T$ feet
1	Sand (Fill)	75	4.2	9.8	4.5	10.4
2	Soft Clay	5	13.4	42.6	14.4	46.0
3	Sand (Liquefiable)	50	4.6	10.6	4.9	11.3
4	Sand	75	4.2	9.8	4.5	10.4
5	Sand	100	4.0	9.2	4.3	9.8
6	Sand	25	5.3	12.2	5.6	13.0
7	Sand	75	4.2	9.8	4.5	10.4
8	Soft Clay	5	13.4	42.6	14.4	46.0
9	Sand (Liquefiable)	50	4.6	10.6	4.9	11.3
10	Sand	30	5.1	11.8	5.4	12.5

LIQUEFIED SOIL PROFILE

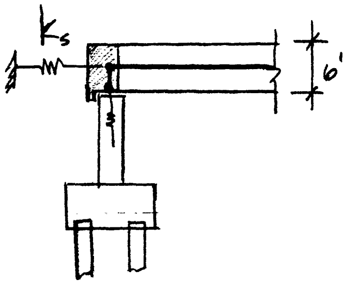
Layer No.	Soil Type	Subgrade Modulus K_h or n_h (pci)	$L_M = 2T$ feet	$L_S = T$ feet	$L_M = 2T$ feet	$L_S = T$ feet
1	Sand (Fill)	75	4.2	9.8	4.5	10.4
2	Soft Clay	5	13.4	42.6	14.4	46.0
3	Soft Clay	1.5	18.1	57.6	19.5	62.1
4	Sand	20	5.5	12.8	5.9	13.6
5	Sand	20	5.5	12.8	5.9	13.6
6	Sand	20	5.5	12.8	5.9	13.6
7	Soft Clay	5	13.4	42.6	14.4	46.0
8	Soft Clay	1.5	18.1	57.6	19.5	62.1
9	Sand	25	5.3	12.2	5.6	13.0
10	Stiff Clay w/o Free Water	100	6.3	20.1	6.8	21.7

E.4 SAP2000 Model Development: Foundation Springs

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject SOIL SPRINGS

Sheet ______ of ______
Job Number A99067-400
Designer MLT
Date 25 JULY 2000

LONGITUDINAL SPRINGS AT SUPERSTRUCTURE



$K_s = \frac{F_p}{\Delta}$, F_p = force due to passive pressure

$$F_p = p_p A$$

NCHRP 11.6.5.1.16

$$p_p = \frac{2}{3}H = \frac{2}{3}(6') = 4 \text{ ksf}$$

$$A = A_{\text{solid box}} - A_{\text{flanges}} = 208.125 \text{ ft}^2 - 2(4')(0.75') = 202.1 \text{ ft}^2$$

$\Delta = 0.02H$ to mobilize soil $= 0.02(6') = 0.12$ ft

$$\therefore k_s = \frac{p_p A}{\Delta} = \frac{(4 \text{ ksf})(202.1 \text{ ft}^2)}{0.12 \text{ ft}} \quad \Rightarrow \quad \underline{\underline{k_s = 6738 \text{ klf}}}$$

$= 808^k = F_p$

FOR SAP MODEL, PLACE ½ k_s AT EACH END
(PIERS 1 & 6)

$$\therefore k_{11} = \frac{1}{2}(6738 \text{ ksf}) \quad \Rightarrow \boxed{k_{11} = 3369 \text{ klf}}$$

AT SUPER. ENDS

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject
Sheet ______ of ______
Job Number A99067 - 400
Designer MLT
Date 27 SEPT 2000

☐ LONGITUDINAL SPRING FOR PASSIVE PRESSURE ON PILE CAP
& TRANSVERSE

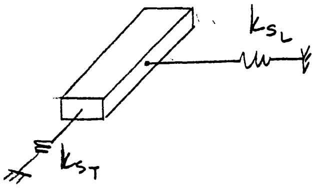
ADD TO SOIL SPRINGS — CAN SUPERIMPOS B/C IN PARALLE

AT ABUTMENTS

PILE CAP IS 46' W x 14' L x 5' D

$A_L = (46')(5') = 230 \text{ ft}^2$

$A_T = (14')(5') = 70 \text{ ft}^2$



$k_s = \frac{F_P}{\Delta}$, $F_P = P_P A$

$P_P = 10 \text{ ksf}$

$\Delta = 0.02H = 0.02\,(5')$ $\Rightarrow$ $\underline{\underline{\Delta = 0.10'}}$

$k_{s_L} = \frac{P_P A_L}{\Delta} = \frac{(10 \text{ ksf})(230 \text{ ft}^2)}{0.10 \text{ ft}}$ $\Rightarrow$ $\boxed{k_{s_L} = 23000 \text{ klf}}$

$k_{s_T} = \frac{P_P A_T}{\Delta} = \frac{(10 \text{ ksf})(70 \text{ ft}^2)}{0.10 \text{ ft}}$ $\Rightarrow$ $\boxed{k_{s_T} = 7000 \text{ klf}}$

AT ABUTMENTS

AT INTERIOR PIERS

☐ k_{s_L} is same as for abutments since A_L, H are same.

☐ $k_{s_T} = \frac{P_P A_T}{\Delta} = \frac{(1.0 \text{ ksf})\overbrace{(22')(5')}^{110 \text{ ft}^2}}{0.10 \text{ ft}}$ $\Rightarrow$ $\boxed{k_{s_T} = 11000 \text{ klf}}$

Project NCHRP 12-49

LIQUEFACTION STUDY

Subject SOIL SPRINGS

Sheet ____ of ____

Job Number A99067 - 400

Designer MLT

Date 25 JULY 2000

AXIAL STIFFNESS FOR PILES

PIERS 1 & 6

(NCHRP 10.7.4.2a)

$$k_{11} = k_{AXIAL} N_P \quad , \quad N_P = 12$$

$$k_{AXIAL} = \underbrace{(1}_{\text{end bearing}} \text{ to } \underbrace{2)}_{\text{skin friction}} \frac{AE}{L}$$

$$k_{AXIAL_1} = \frac{A_{P_t} E_c}{L_p} = \frac{(710.8 in^2)(3830 ksi)}{|\underbrace{105'}_{\text{NOW } 13'} - 180'|} \Rightarrow \underline{\underline{k_{AXIAL_1} = 16014 \, k/ft}}$$

$$k_{AXIAL_{1.5}} = \frac{1.5 AE}{L} = 1.5 \, k_{AXIAL_1} \Rightarrow \underline{\underline{k_{AXIAL_{1.5}} = 24021 \, k/ft}}$$

Similarly, $\Rightarrow \underline{\underline{k_{AXIAL_2} = 32028 \, k/ft}}$

$$\therefore \; k_{11_1} = N_P \, k_{AXIAL_1} = 12(16014 \, k/ft) \Rightarrow \boxed{k_{11_1} = 193{,}167 \, k/ft}$$

$$\boxed{k_{11_{1.5}} = 288{,}250 \, k/ft}$$

$$\boxed{k_{11_2} = 384{,}334 \, k/ft}$$

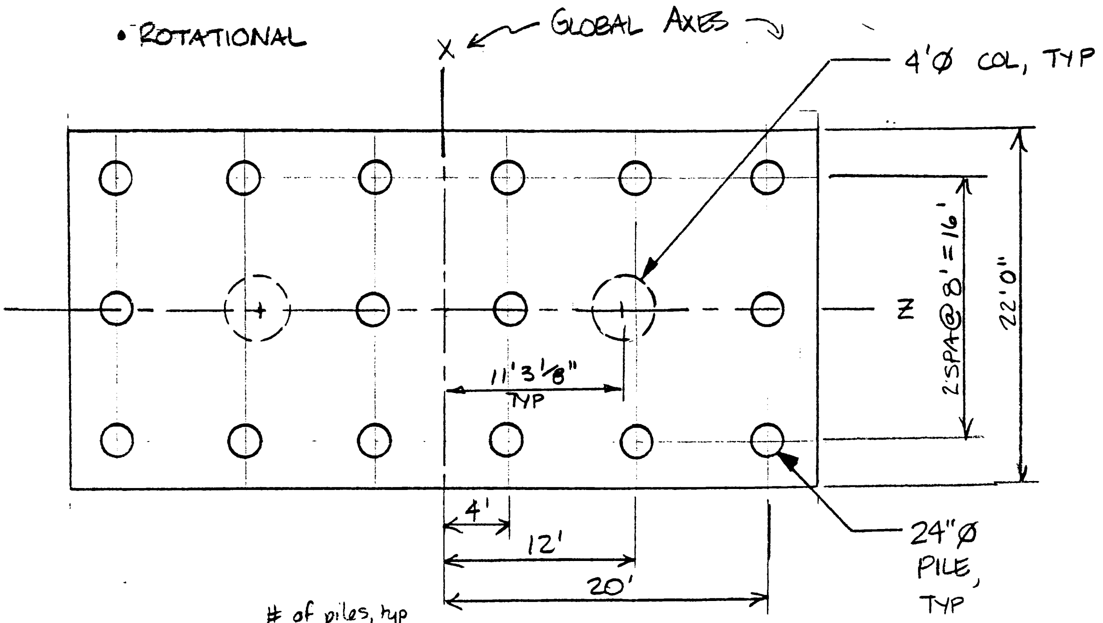
CHANGED FOR FTG EL=13' (NEXT PAGE)

{See Table on next 2 pages for k_{11} calc for all piers.}

Project NCHRP 12-49
Sheet ______ of ______
Job Number A99067 - 400
Subject SOIL SPRINGS
Designer MLT
Date 26 JULY 2000

INTERMEDIATE PIERS

- ROTATIONAL



# of piles, typ

M_x: $k_{\theta_x} = k_{AXIAL}\left(6(4')^2 + 4(12')^2 + 6(20')^2\right) \Rightarrow \underline{\underline{k_{\theta_x} = (3072\ ft^2)\,k_{axial}}}$

M_z: $k_{\theta_z} = k_{AXIAL}\left(12(8')^2\right) + 16\overset{\text{ASSUME 0}}{\cancel{k_\theta}} \Rightarrow \underline{\underline{k_{\theta_z} = (768\ ft^2)\,k_{axial}}}$

FOR PIERS 1 & 6 (ABUTMENTS)

$k_{\theta_x} = (3072\ ft^2)(195619\ k/ft) \Rightarrow \underline{\underline{k_{\theta_x} = 6.009 \times 10^8\ \frac{k\text{-}ft}{rad}}}$

$k_{\theta_z} = (768\ ft^2)(195619\ k/ft) \Rightarrow \underline{\underline{k_{\theta_z} = 1.502 \times 10^8\ \frac{k\text{-}ft}{rad}}}$

(SEE TABLE ON NEXT PAGE FOR ALL PIERS)

Project

Sheet of

Job Number A99067 - 400

Subject SOIL SPRINGS

Designer MLT

Date 26 JULY 2000

INTERMEDIATE PIERS

- TRANSLATIONAL (PIER 2)

APPLY 100^k SHEAR TO FIXED HEAD PILE

IN LPILE $\Rightarrow$ $\Delta_{HEAD} = 0.555''$ (APPLIED TO PIER 2 PILE)

TRIAL ① $F = 100^k$, $\Delta = 0.56''$

$$\therefore k_{\Delta x} = k_{\Delta z} = N_p\left(\frac{F}{\Delta}\right) = (16)\underbrace{\left(\frac{100^k}{0.555''}\right)\left(12''/ft\right)}_{2162\ k/ft}$$

$$k_{\Delta x} = k_{\Delta z} = 34595\ k/ft$$

close to Po Lam $\rightarrow$ $k = 27001\ k/ft$
(25% DIFF.)

TRIAL ② $F = 30^k$, $\Delta = 0.153''$

$$k_{\Delta x} = k_{\Delta z} = (16)\left(\frac{30^k}{0.153''}\right)\left(12''/1\right) = 37650\ k/ft$$

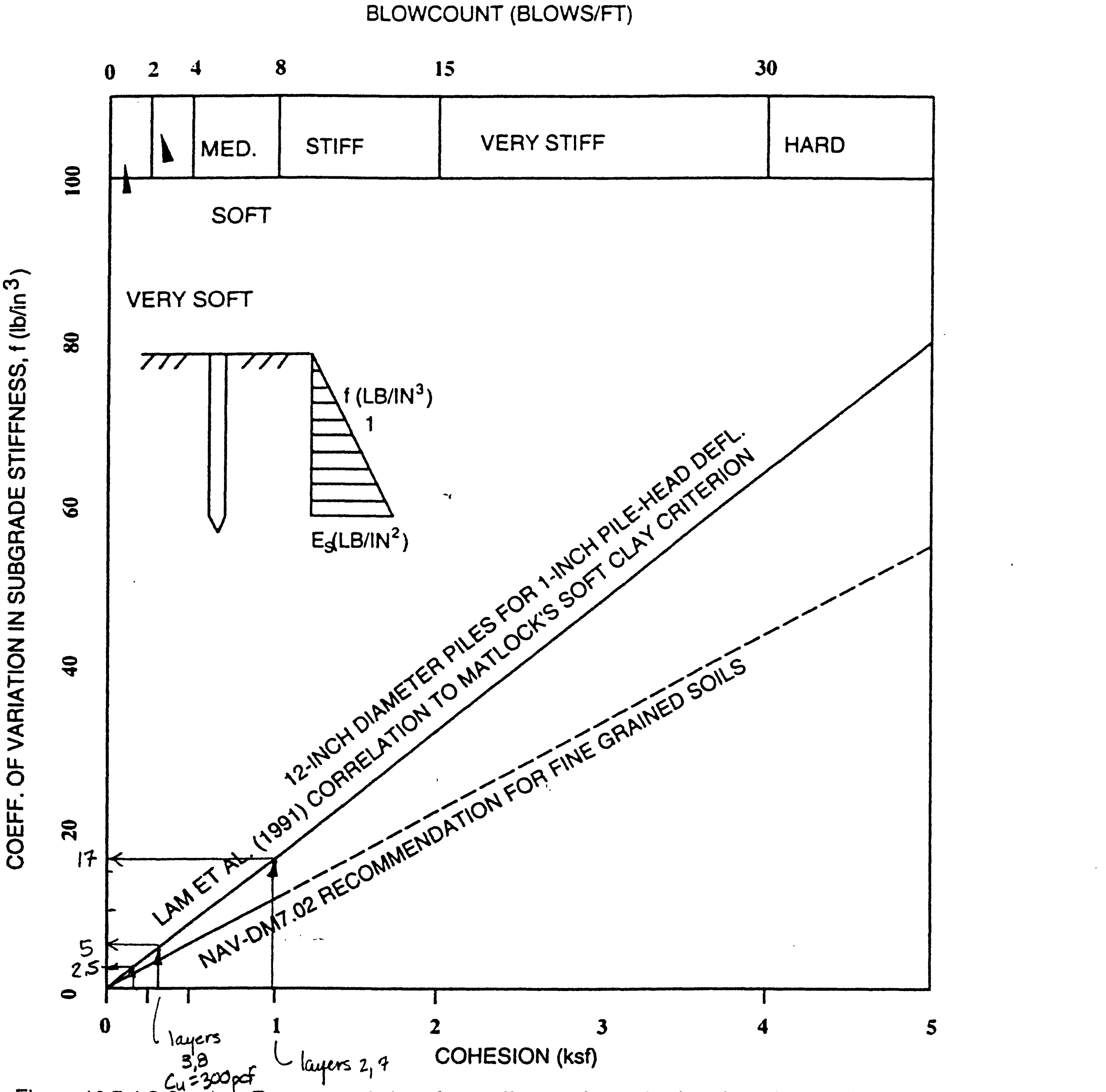
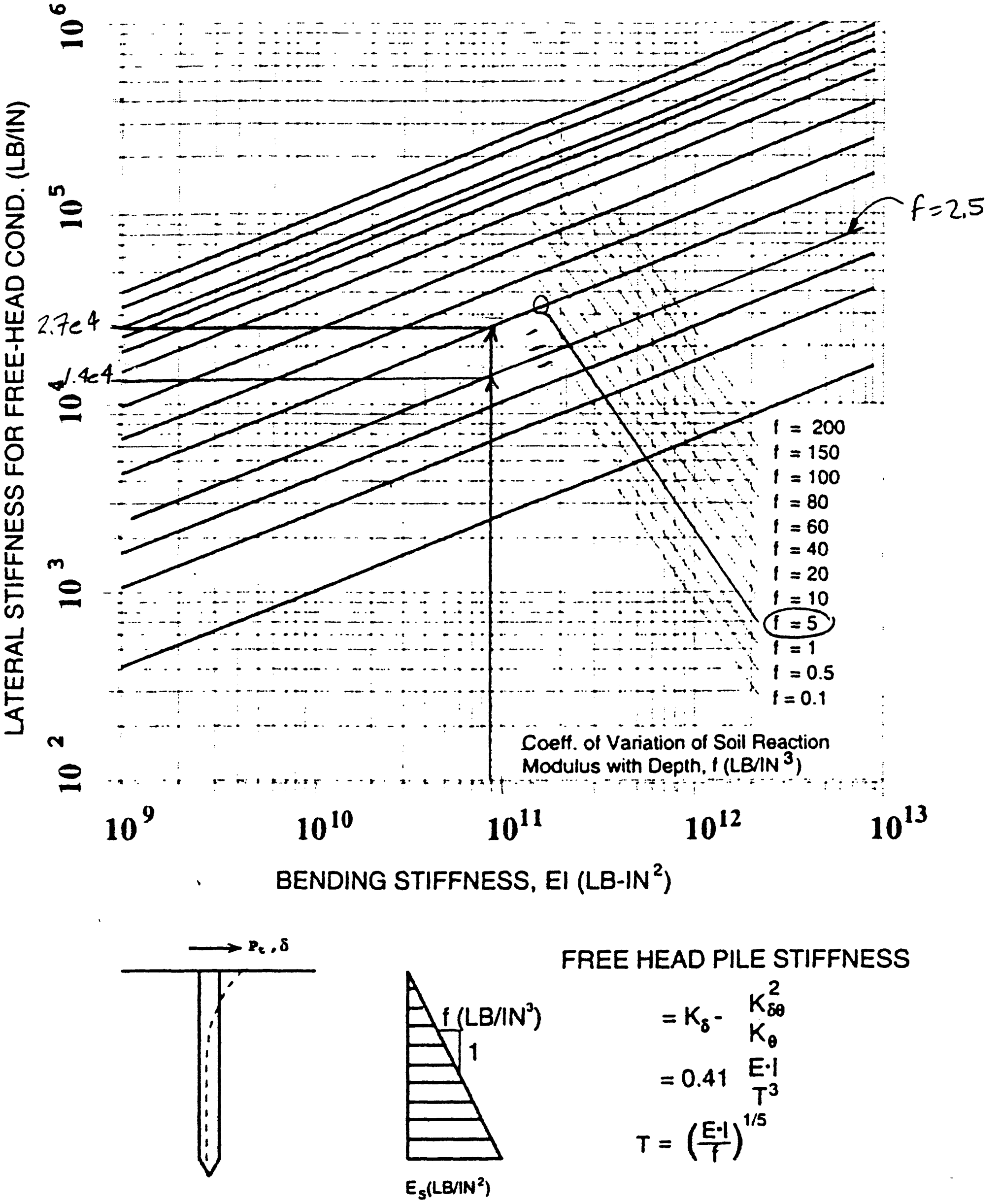


Figure 10.7.4.2-2 Recommendations for coefficient of variation in subgrade modulus with depth for clay

or 150pcf for group effect

FOR THE SOFTEST CLAY LAYER, f ≈ 5 pci

f = 2.5 pci (INCL. GROUP EFFECT)



FREE HEAD PILE STIFFNESS

$$= K_\delta - \frac{K_{\delta\theta}^2}{K_\theta}$$

$$= 0.41\ \frac{E\cdot I}{T^3}$$

$$T = \left(\frac{E\cdot I}{f}\right)^{1/5}$$

Figure 10.7.4.2-3 Coefficient for lateral pile head stiffness (free head pile lateral stiffness).

$(EI)_{pile} = (3830\ ksi)(23197 in^4)(1000 \#/k) = 8.9 \times 10^{10}$

FROM CHART $\Rightarrow K_t = 2.7 \times 10^4$ lb/in $\Rightarrow K_t = 27$ k/in

× 16 piles

$K_t = 432$ k/in

= 5200 k/ft

w/GROUP EFFECT $K = (14 k/in)(12)(16 piles) = 2700$ k/

Project ________ Sheet ____ of ____

Job Number A99067 - 400

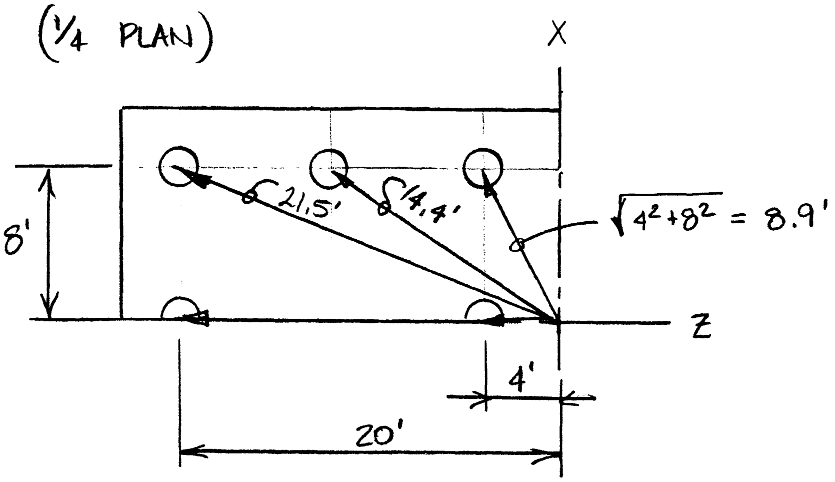
Subject ________ Designer MLT

Date 26 JULY 2000

INTERMEDIATE PIERS

- TORSIONAL

(1/4 PLAN)



$$\overset{\text{or } k_{\Delta z}}{} \quad \underset{\text{GLOBAL}}{k_{\theta y}} = k_{\Delta x}\left[2(4^2+20^2)+4(8.9^2+14.4^2+21.5^2)\right]$$

$$\Rightarrow \underline{\underline{k_{\theta y} = (3840\ ft^2)\ k_{\Delta x}}}$$

FOR PIERS 2-5

$$k_{\theta y} = (3840\ ft^2)(34595\ ^{k}/_{ft}) \Rightarrow \boxed{k_{\theta y} = 1.328\times10^8\ \frac{k\text{-}ft}{rad}}$$

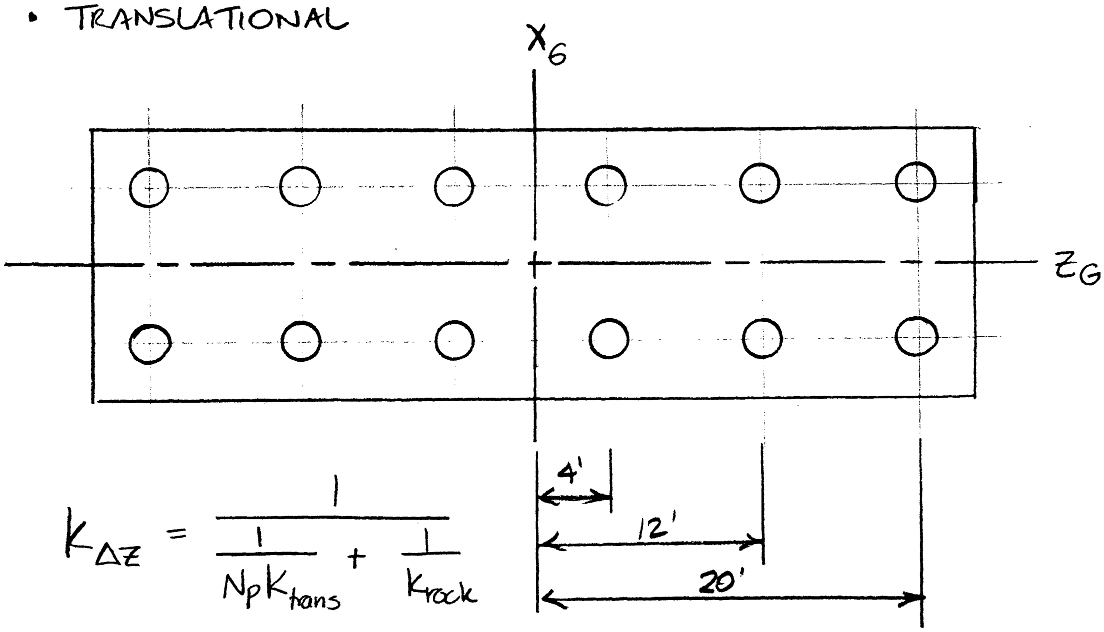
TORSIONAL

TRIAL ② $k_{\theta y} = (3840)(37650) \Rightarrow k_{\theta y} = 1.446\times10^8$

Project NCHRP 12-49
Sheet ______ of ______
Job Number A99067-400
Subject SOIL SPRINGS
Designer MLT
Date 26 JULY 2000

ABUTMENTS

- TRANSLATIONAL



$$K_{\Delta Z} = \frac{1}{\frac{1}{N_p K_{trans}} + \frac{1}{K_{rock}}}$$

$$K_{rock} = \frac{K_{axial}}{(h_{abut})^2}\left[4(4^2 + 12^2 + 20^2)\right] = \frac{\overset{13355}{\cancel{195619}}\ ^{k}/_{ft}}{(25' - 13')^2}(2240\ ft^2)$$

$$\Rightarrow K_{rock} = \overset{2.08\times10^5\ ^{k}/_{ft}}{\cancel{3.043\times10^6}}\ ^{k}/_{ft}$$

$$K_{\Delta x} = K_{trans_{pile}} = N_p\left(\frac{F}{\Delta}\right) = (12)\underbrace{\left(\overset{\frac{50^k}{0.064''}}{\cancel{\frac{100^k}{0.165''}}}\right)(12''/ft)}_{7273\ ^{k}/_{ft}} \Rightarrow \boxed{K_{\Delta x} = \underset{112500}{\cancel{87273}}\ ^{k}/_{ft}}$$

$$\therefore K_{\Delta Z} = \frac{1}{\frac{1}{\underset{112500}{\cancel{87273}}} + \frac{1}{\underset{2.08\times10^5}{\cancel{3.043e6}}}} = \frac{1}{(\underset{8.89e-6}{\cancel{1.146e-5}}) + (\underset{4.81e-6}{\cancel{3.286e-6}})} = \frac{1}{\underset{1.37e-5}{\cancel{1.179e-5}}}$$

$$\Rightarrow \boxed{K_{\Delta Z} = \overset{72980}{\cancel{84840}}\ ^{k}/_{ft}}$$

@ P1 & P6

Project

Sheet ____ of ____

Job Number A99067 - 400

Subject SOIL SPRINGS

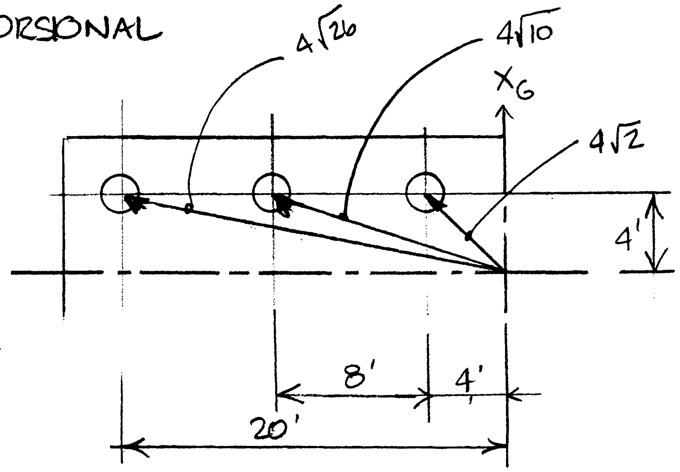
Designer MLT

Date 26 JULY 2000

ABUTMENTS

- TORSIONAL

(1/4 PLAN)



$$k_{\theta y} = k_{\Delta x}\left[4\left((4\sqrt{2})^2 + (4\sqrt{10})^2 + (4\sqrt{26})^2\right]$$

or $k_{\Delta z}$

Excluding passive pressure on pile cap.

$$= k_{\Delta x}\left[4\left(32 + 160 + 416\right)\right] = 2432\, k_{\Delta x} \text{ or } 2432\, k_{\Delta z}$$

$\hookrightarrow 0$

$$k_{\theta y} = (2432)(\quad 0 \quad) \Rightarrow \boxed{k_{\theta y} = 0 \quad \text{k-ft/rad}}$$

PIERS 1 & 6

Project ______________________

Sheet ______ of ______

Job Number A99067 - 400

Designer MLT

Subject ______________________

Date ______

LATERAL (TRANSVERSE) ROTATIONAL STIFFNESS

- PIERS 1 & 6

per pile

$$K_{RX} = K_{UY} \Sigma d_i^2$$

$$K_{RX} = (13355 \text{ k/ft})(\underbrace{4(4^2 + 12^2 + 20^2)}_{2240 \text{ ft}^2}) \Rightarrow \boxed{K_{RX} = 3.0 \times 10^7 \ \frac{\text{k-ft}}{\text{rad}}}$$

WASHINGTON BRIDGE FOUNDATION SPRINGS

Pile Properties

Apile =	673	in^2
Ec =	3830	ksi

Translational Springs for Pile Cap Passive Pressures

	Longitudinal k/ft	Lateral k/ft
Abutments	0	0
Int. Piers	23000	11000

TRIAL 1

NON-LIQUEFIED CASE

						Axial	Longitudinal	Lateral		Axial		Lateral		Longitudinal
Pier	No. of Piles	Pile Head Elev	Pile Tip Elev	Pile Length	k_{axial} = AE/L	K_{UY}	K_{UX}	K_{UZ}	K_{RY} Group	K_{RY}	K_{RX} Group	K_{RX}	K_{RZ} Group	K_{RZ}
		ft	ft	ft	k/ft	k/ft	k/ft	k/ft	ft^2	k/ft	ft^2	k/ft	ft^2	k/ft
1	12	13	-180	193.0	13355	1.60E+05	0	7.30E+04	0	0.00E+00	2240	2.99E+07	0	0.00E+00
2	16	-13	-180	167.0	15435	2.47E+05	3.33E+05	3.21E+05	3840	1.19E+09	3072	7.59E+08	768	1.90E+08
3	16	-29	-180	151.0	17070	2.73E+05	3.43E+05	3.31E+05	3840	1.23E+09	3072	8.39E+08	768	2.10E+08
4	16	-36	-180	144.0	17900	2.86E+05	4.59E+05	4.47E+05	3840	1.68E+09	3072	8.80E+08	768	2.20E+08
5	16	-29	-180	151.0	17070	2.73E+05	3.43E+05	3.31E+05	3840	1.23E+09	3072	8.39E+08	768	2.10E+08
6	12	13	-180	193.0	13355	1.60E+05	0	7.30E+04	0	0.00E+00	2240	2.99E+07	0	0.00E+00

Applied Shear Force
50 kips

Pile Head Displ.	Spring
in.	k/ft
0.064	9375
0.031	19355
0.030	20000
0.022	27273
0.030	20000
0.064	9375

LIQUEFIED CASE

						Axial	Longitudinal	Lateral		Axial		Lateral		Longitudinal
Pier	No. of Piles	Pile Head Elev	Pile Tip Elev	Pile Length	k_{axial} = AE/L	K_{UY}	K_{UX}	K_{UZ}	K_{RY} Group	K_{RY}	K_{RX} Group	K_{RX}	K_{RZ} Group	K_{RZ}
		ft	ft	ft	k/ft	k/ft	k/ft	k/ft	ft^2	k/ft	ft^2	k/ft	ft^2	k/ft
1	12	13	-180	193.0	13355	1.60E+05	0	1.13E+05	0	0.00E+00	2240	2.99E+07	0	0.00E+00
2	16	-13	-180	167.0	15435	2.47E+05	6.37E+04	5.17E+04	3840	1.56E+08	3072	7.59E+08	768	1.90E+08
3	16	-29	-180	151.0	17070	2.73E+05	2.01E+05	1.89E+05	3840	6.83E+08	3072	8.39E+08	768	2.10E+08
4	16	-36	-180	144.0	17900	2.86E+05	1.71E+05	1.59E+05	3840	5.67E+08	3072	8.80E+08	768	2.20E+08
5	16	-29	-180	151.0	17070	2.73E+05	2.01E+05	1.89E+05	3840	6.83E+08	3072	8.39E+08	768	2.10E+08
6	12	13	-180	193.0	13355	1.60E+05	0	1.13E+05	0	0.00E+00	2240	2.99E+07	0	0.00E+00

Applied Shear Force
50 kips

Pile Head Displ.	Spring
in.	k/ft
0.064	9375
0.236	2542
0.054	11111
0.065	9231
0.054	11111
0.064	9375

WASHINGTON BRIDGE FOUNDATION SPRINGS

NON-LIQUEFIED CASE

	Axial	Longitudinal	Lateral	Axial	Longitudinal	Lateral
Global	**UY**	**UX**	**UZ**	**RY**	**RX**	**RZ**
Pier	K_{11} k/ft	K_{22} k/ft	K_{33} k/ft	K_{44} k-ft/rad	K_{55} k-ft/rad	K_{66} k-ft/rad
1	1.60E+05	0.00E+00	7.30E+04	0.00E+00	2.99E+07	0.00E+00
2	2.47E+05	3.33E+05	3.21E+05	1.19E+09	7.59E+08	1.90E+08
3	2.73E+05	3.43E+05	3.31E+05	1.23E+09	8.39E+08	2.10E+08
4	2.86E+05	4.59E+05	4.47E+05	1.68E+09	8.80E+08	2.20E+08
5	2.73E+05	3.43E+05	3.31E+05	1.23E+09	8.39E+08	2.10E+08
6	1.60E+05	0.00E+00	7.30E+04	0.00E+00	2.99E+07	0.00E+00

LIQUEFIED CASE

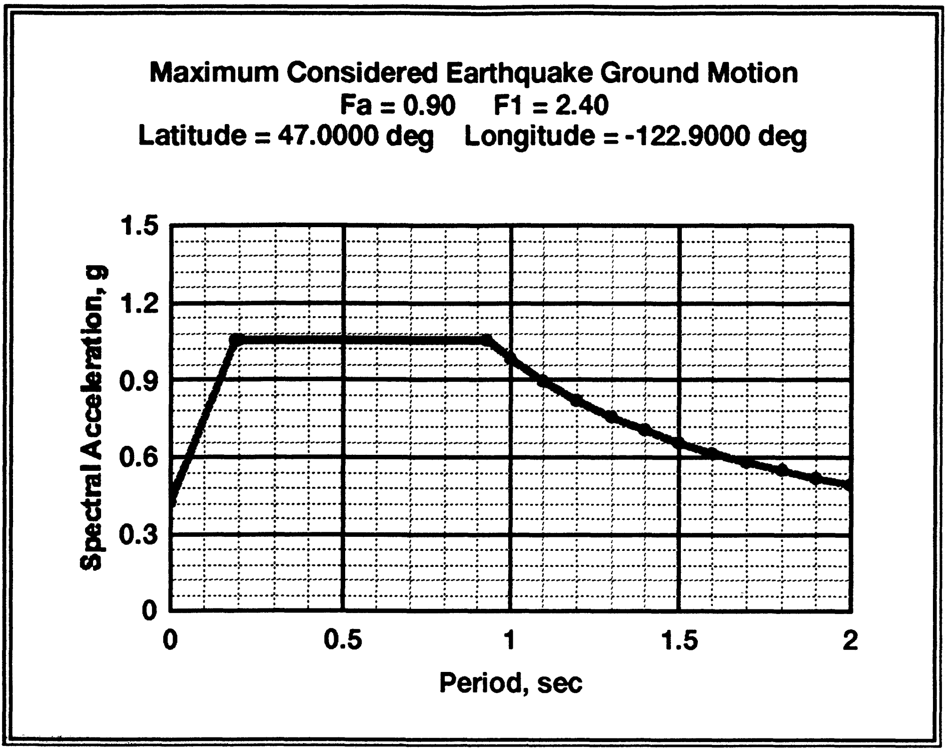
	Axial	Longitudinal	Lateral	Axial	Longitudinal	Lateral
Global	**UY**	**UX**	**UZ**	**RY**	**RX**	**RZ**
Pier	K_{11} k/ft	K_{22} k/ft	K_{33} k/ft	K_{44} k-ft/rad	K_{55} k-ft/rad	K_{66} k-ft/rad
1	1.60E+05	0.00E+00	1.13E+05	0.00E+00	2.99E+07	0.00E+00
2	2.47E+05	6.37E+04	5.17E+04	1.56E+08	7.59E+08	1.90E+08
3	2.73E+05	2.01E+05	1.89E+05	6.83E+08	8.39E+08	2.10E+08
4	2.86E+05	1.71E+05	1.59E+05	5.67E+08	8.80E+08	2.20E+08
5	2.73E+05	2.01E+05	1.89E+05	6.83E+08	8.39E+08	2.10E+08
6	1.60E+05	0.00E+00	1.13E+05	0.00E+00	2.99E+07	0.00E+00

E.5 SAP2000 Model Development: Response Spectra

```
Washington Site - 2500yr EQ
 Date and Time: 9/20/00 5:21:40 PM

 MCE Parameters - Conterminous 48 States
 Latitude = 47.0000,  Longitude = -122.9000
Data are based on the 0.10 deg grid set
 Period        SA
  (sec)        (%g)
   0.2         117.5   Map Value, Soil Factor of 1.0
   1.0         041.1   Map Value, Soil Factor of 1.0
  MCE Parameters x Specified Soil Factors
   0.2         105.8   Soil Factor of 0.9
   1.0         098.6   Soil Factor of 2.4

 MCE Parameters - Conterminous 48 States
 Latitude = 47.0000,  Longitude = -122.9000
Data are based on the 0.10 deg grid set
 Period        SA
  (sec)        (%g)
   0.2         117.5   Map Value, Soil Factor of 1.0
   1.0         041.1   Map Value, Soil Factor of 1.0
  MCE SPECTRUM x SOIL FACTORS
   Fa = 0.9
   Fv = 2.4
   Period        SA
    (sec)        (%g)
    0.000        042.3    0.4FaSs
    0.187        105.7    To
    0.200        105.7    T=0.2, FaSs
    0.933        105.7    Ts
    1.000        098.6    T=1.0, FvS1
    1.100        089.7
    1.200        082.2
    1.300        075.9
    1.400        070.5
    1.500        065.8
    1.600        061.6
    1.700        058.0
    1.800        054.8
    1.900        051.9
    2.000        049.3
```



eriod, se	Sa, g
0.00	0.423
0.19	1.057
0.20	1.057
0.93	1.057
1.00	0.986
1.10	0.897
1.20	0.822
1.30	0.759
1.40	0.705
1.50	0.658
1.60	0.616
1.70	0.580
1.80	0.548
1.90	0.519
2.00	0.493

2500yr EQ

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject WA SITE
100-YR EQ

Sheet ______ of ______
Job Number A99067 - 400
Designer MLT
Date 9/20/00

LINEARLY INTERPOLATE FOR F_a, F_v - 100YR EQ

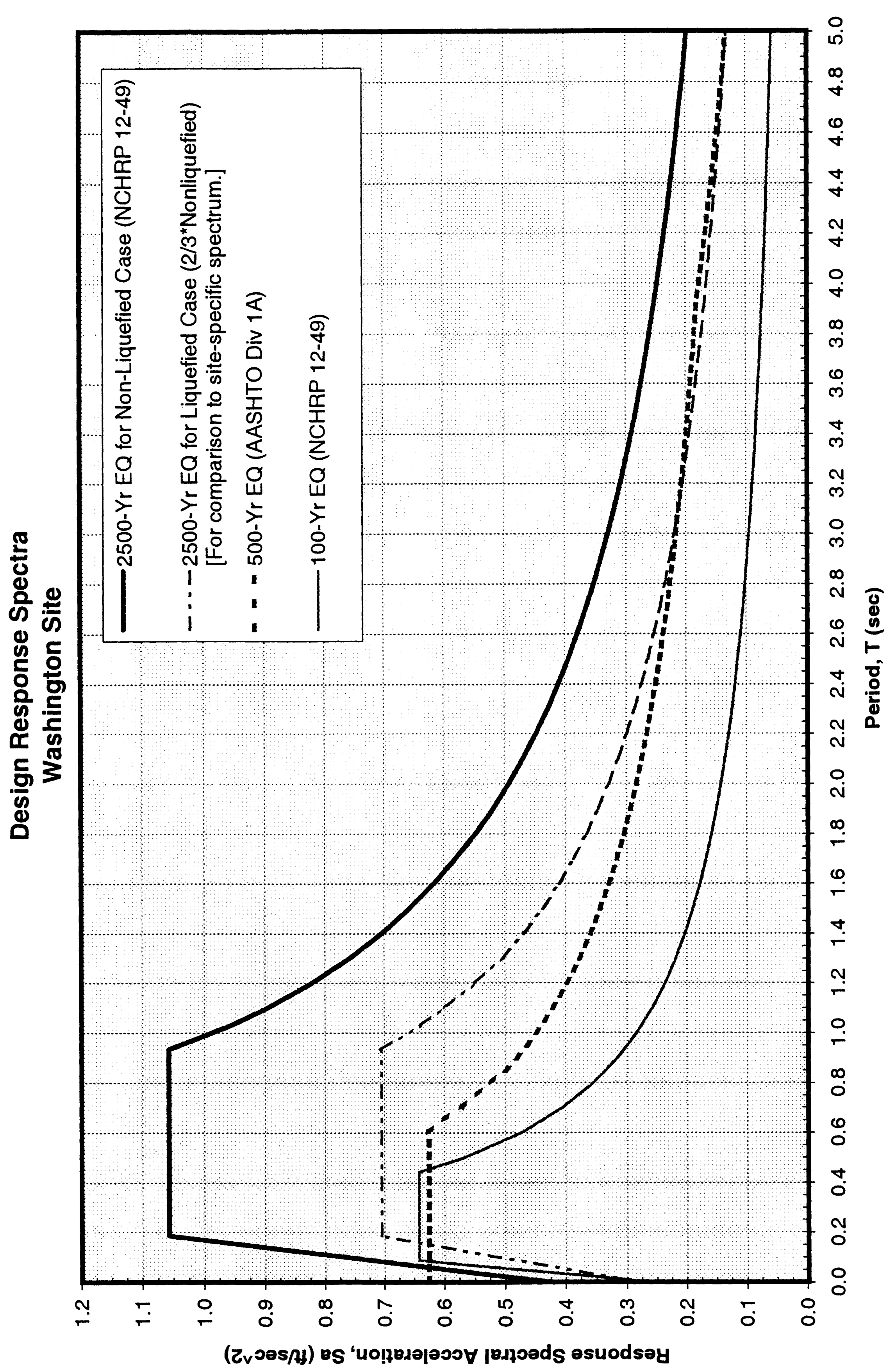
FROM GEOTECH. $\begin{cases} S_S = 0.261 & F_a = ? \\ S_1 = 0.081 & F_v = ? \end{cases}$ $\Rightarrow$ $\underline{\underline{F_v = 3.5}}$

USING VALUES IN TABLES 3.10.2.2.3-1, -2 (p. 3-19)

$$F_a: \quad \frac{F_a - 2.5}{0.261 - 0.25} = \frac{1.7 - 2.5}{0.50 - 0.25}$$

$$\Rightarrow \quad 0.25\,(F_a - 2.5) = -0.0088$$

$$\Rightarrow \quad F_a = 2.5 - 0.0352 \quad \Rightarrow \quad \underline{\underline{F_a = 2.46}}$$



Design Response Spectra
Washington Site

NCHRP 12-49 SOIL LIQUEFACTION STUDY
WASHINGTON SITE - 2500Yr EQ, Non-Liquefied & Liquefied Case (NCHRP)

DESIGN RESPONSE SPECTRUM
Using Two-Point Construction Method

	Short Periods	1-Sec Periods
Site Coefficients:	F_a = 0.9	F_v = 2.4
Spectral Acceleration on Class B Rock:	S_s = 1.175	S_1 = 0.411
Spectral Response Acceleration:	**$S_{DS} = F_aS_s$ = 1.058**	**$S_{D1} = F_vS_1$ = 0.986**

Key Points on Response Spectrum:

$T_s = S_{D1}/S_{DS}$ = 0.933
$T_0 = 0.2T_s$ = 0.187
Y-Intercept = $0.4S_{DS}$ = 0.423

Non-Liquefied

T sec	Sa ft/sec^2
0.000	0.423
0.187	1.058
0.933	1.058
1.030	0.958
1.100	0.897
1.20	0.822
1.30	0.759
1.40	0.705
1.50	0.658
1.60	0.617
1.70	0.580
1.80	0.548
1.90	0.519
2.00	0.493
2.10	0.470
2.20	0.448
2.30	0.429
2.40	0.411
2.50	0.395
2.60	0.379
2.70	0.365
2.80	0.352
2.90	0.340
3.00	0.329
3.10	0.318
3.20	0.308
3.30	0.299
3.40	0.290
3.50	0.282
3.60	0.274
3.70	0.267
3.80	0.260
3.90	0.253
4.00	0.247
4.10	0.241
4.20	0.235
4.30	0.229
4.40	0.224
4.50	0.219
4.60	0.214
4.70	0.210
4.80	0.206
4.90	0.201
5.00	0.197
5.10	0.193
5.20	0.190
5.30	0.186
5.40	0.183
5.50	0.179
5.60	0.176
5.70	0.173
5.80	0.170
5.90	0.167

Liquefied

T sec	2/3*Sa ft/sec^2
0.000	0.282
0.187	0.705
0.933	0.705
1.030	0.638
1.100	0.598
1.200	0.548
1.300	0.506
1.400	0.470
1.500	0.438
1.600	0.411
1.700	0.387
1.800	0.365
1.900	0.346
2.000	0.329
2.100	0.313
2.200	0.299
2.300	0.286
2.400	0.274
2.500	0.263
2.600	0.253
2.700	0.244
2.800	0.235
2.900	0.227
3.000	0.219
3.100	0.212
3.200	0.206
3.300	0.199
3.400	0.193
3.500	0.188
3.600	0.183
3.700	0.178
3.800	0.173
3.900	0.169
4.000	0.164
4.100	0.160
4.200	0.157
4.300	0.153
4.400	0.149
4.500	0.146
4.600	0.143
4.700	0.140
4.800	0.137
4.900	0.134
5.000	0.132
5.100	0.129
5.200	0.126
5.300	0.124
5.400	0.122
5.500	0.120
5.600	0.117
5.700	0.115
5.800	0.113
5.900	0.111

NCHRP 12-49 SOIL LIQUEFACTION STUDY
WASHINGTON SITE - 2500Yr EQ, Non-Liquefied & Liquefied Case (NCHRP)

DESIGN RESPONSE SPECTRUM
Using Two-Point Construction Method

	Short Periods	1-Sec Periods
Site Coefficients:	F_a = 0.9	F_v = 2.4
Spectral Acceleration on Class B Rock:	S_s = 1.175	S_1 = 0.411
Spectral Response Acceleration:	**$S_{DS} = F_aS_s$ = 1.058**	**$S_{D1} = F_vS_1$ = 0.986**

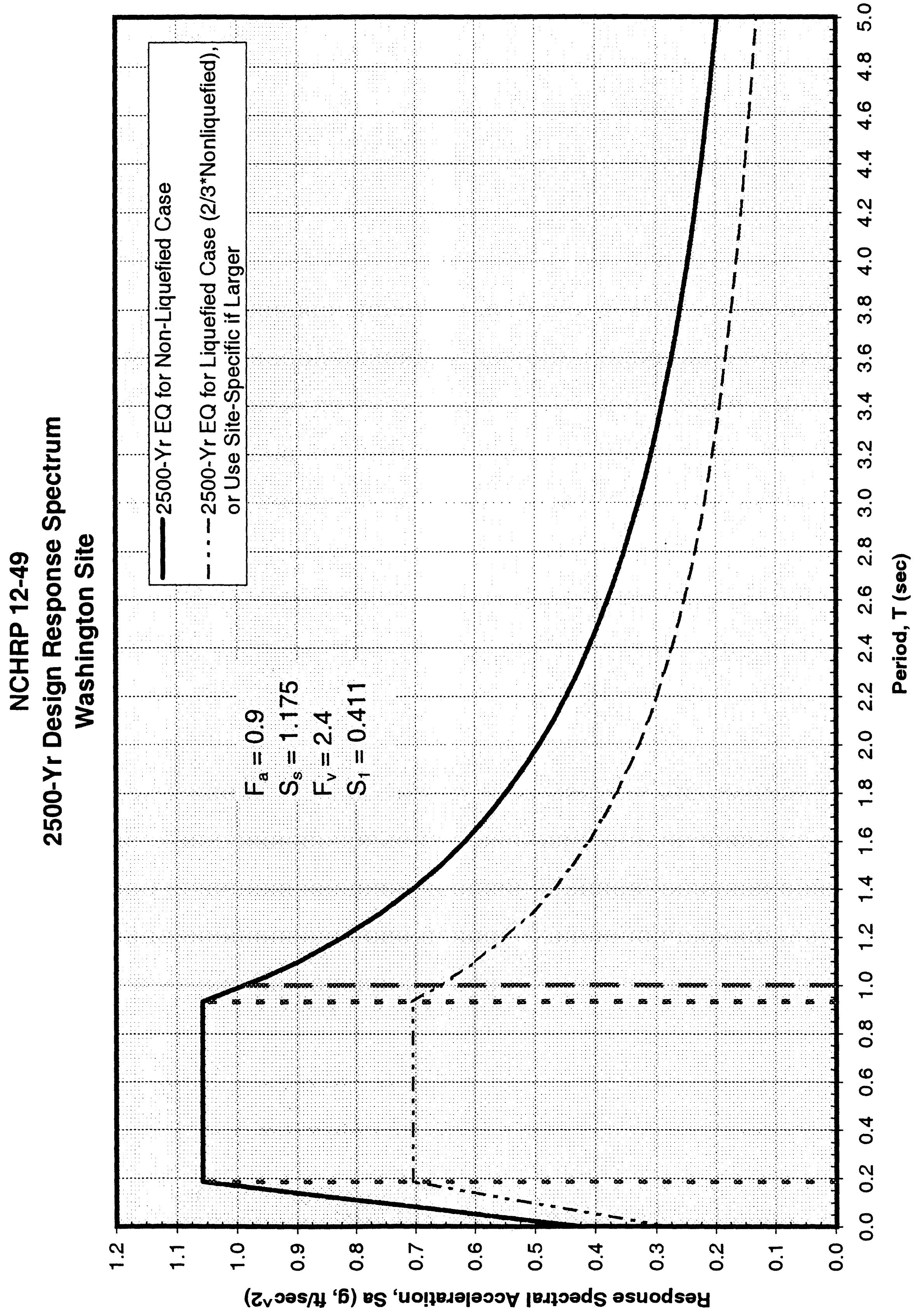
Key Points on Response Spectrum:
$T_s = S_{D1}/S_{DS}$ = 0.933
$T_0 = 0.2T_s$ = 0.187
Y-Intercept = $0.4S_{DS}$ = 0.423

Non-Liquefied

T sec	Sa ft/sec^2
6.00	0.164
6.10	0.162
6.20	0.159
6.30	0.157
6.40	0.154
6.50	0.152
6.60	0.149
6.70	0.147
6.80	0.145
6.90	0.143
7.00	0.141
7.10	0.139
7.20	0.137
7.30	0.135
7.40	0.133
7.50	0.132
7.60	0.130
7.70	0.128
7.80	0.126
7.90	0.125
8.00	0.123
8.10	0.122
8.20	0.120
8.30	0.119
8.40	0.117
8.50	0.116
8.60	0.115
8.70	0.113
8.80	0.112
8.90	0.111
9.00	0.110
10.00	0.099

Liquefied

T sec	2/3*Sa ft/sec^2
6.000	0.110
6.100	0.108
6.200	0.106
6.300	0.104
6.400	0.103
6.500	0.101
6.600	0.100
6.700	0.098
6.800	0.097
6.900	0.095
7.000	0.094
7.100	0.093
7.200	0.091
7.300	0.090
7.400	0.089
7.500	0.088
7.600	0.087
7.700	0.085
7.800	0.084
7.900	0.083
8.000	0.082
8.100	0.081
8.200	0.080
8.300	0.079
8.400	0.078
8.500	0.077
8.600	0.076
8.700	0.076
8.800	0.075
8.900	0.074
9.000	0.073
10.000	0.066


NCHRP 12-49
2500-Yr Design Response Spectrum
Washington Site
2500-Yr EQ for Non-Liquefied Case
2500-Yr EQ for Liquefied Case (2/3*Nonliquefied), or Use Site-Specific if Larger
$F_a = 0.9$
$S_s = 1.175$
$F_v = 2.4$
$S_1 = 0.411$
Response Spectral Acceleration, Sa (g, ft/sec^2)
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
Period, T (sec)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
4.4
4.6
4.8
5.0

NCHRP 12-49 LIQUEFACTION STUDY
WASHINGTON SITE - 500Yr (Div 1A)

ELASTIC SEISMIC RESPONSE FOR MULTIMODE ANALYSIS

$$C_{sm} = \frac{1.2 \cdot A \cdot S}{T_m^{\frac{2}{3}}} \leq 2.5 \cdot A$$

(AASHTO DIV. 1A Eqn. 3-2)

Design parameters :	A (g) =	0.25	g
(Type III Soil)	S =	1.5	

Basic Spectrum

T_m	C_{sm}
0.00	0.625
0.05	0.625
0.10	0.625
0.15	0.625
0.20	0.625
0.25	0.625
0.30	0.625
0.35	0.625
0.40	0.625
0.45	0.625
0.50	0.625
0.55	0.625
0.60	0.625
0.65	0.600
0.70	0.571
0.75	0.545
0.80	0.522
0.85	0.501
0.90	0.483
0.95	0.466
1.00	0.450
1.05	0.436
1.10	0.422
1.15	0.410
1.20	0.398
1.25	0.388
1.30	0.378
1.35	0.368
1.40	0.360
1.45	0.351
1.50	0.343
1.55	0.336
1.60	0.329
1.65	0.322
1.70	0.316
1.75	0.310
1.80	0.304
1.85	0.299
1.90	0.293
1.95	0.288
2.00	0.283
2.05	0.279
2.10	0.274
2.15	0.270
2.20	0.266
2.25	0.262
2.30	0.258
2.35	0.255
2.40	0.251
2.45	0.248
2.50	0.244
2.55	0.241
2.60	0.238
2.65	0.235
2.70	0.232
2.75	0.229
2.80	0.227
2.85	0.224
2.90	0.221

NCHRP 12-49 LIQUEFACTION STUDY
WASHINGTON SITE - 500Yr (Div 1A)

ELASTIC SEISMIC RESPONSE FOR MULTIMODE ANALYSIS

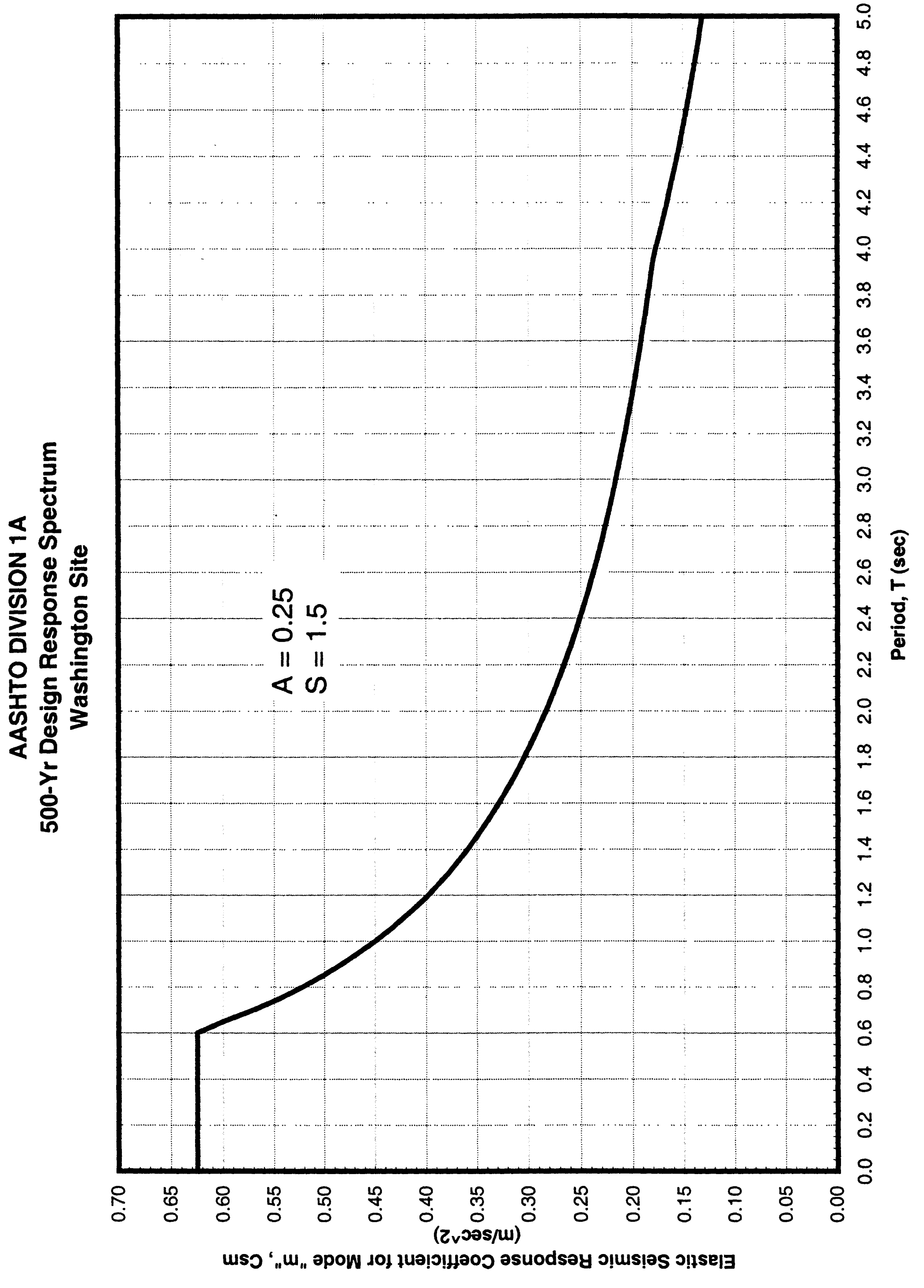
$$C_{sm} = \frac{1.2 \cdot A \cdot S}{T_m^{\frac{2}{3}}} \leq 2.5 \cdot A$$

(AASHTO DIV. 1A Eqn. 3-2)

Design parameters : A (g) = 0.25 g
(Type III Soil) S = 1.5

Basic Spectrum

T_m	C_{sm}
2.95	0.219
3.00	0.216
3.05	0.214
3.10	0.212
3.15	0.209
3.20	0.207
3.25	0.205
3.30	0.203
3.35	0.201
3.40	0.199
3.45	0.197
3.50	0.195
3.55	0.193
3.60	0.192
3.65	0.190
3.70	0.188
3.75	0.186
3.80	0.185
3.85	0.183
3.90	0.182
3.95	0.180
4.00	0.177
4.05	0.174
4.10	0.171
4.15	0.169
4.20	0.166
4.25	0.163
4.30	0.161
4.35	0.158
4.40	0.156
4.45	0.154
4.50	0.151
4.55	0.149
4.60	0.147
4.65	0.145
4.70	0.143
4.75	0.141
4.80	0.139
4.85	0.137
4.90	0.135
4.95	0.133
5.00	0.132


AASHTO DIVISION 1A
500-Yr Design Response Spectrum
Washington Site
A = 0.25
S = 1.5
Elastic Seismic Response Coefficient for Mode "m", Csm
(m/sec^2)
0.70
0.65
0.60
0.55
0.50
0.45
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Period, T (sec)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
4.4
4.6
4.8
5.0

NCHRP 12-49 SOIL LIQUEFACTION STUDY
WASHINGTON SITE - 100Yr EQ (NCHRP)

DESIGN RESPONSE SPECTRUM

Using Two-Point Construction Method

	Short Periods	1-Sec Periods
Site Coefficients:	F_a = 2.46	F_v = 3.5
Spectral Acceleration on Class B Rock:	S_s = 0.261	S_1 = 0.081
Spectral Response Acceleration:	**$S_{DS} = F_aS_s$ = 0.642**	**$S_{D1} = F_vS_1$ = 0.284**

Key Points on Response Spectrum:

$T_s = S_{D1}/S_{DS}$ = 0.442
$T_0 = 0.2T_s$ = 0.088
Y-Intercept = $0.4S_{DS}$ = 0.257

T sec	Sa ft/sec^2
0.00	0.257
0.088	0.642
0.442	0.642
0.490	0.579
0.50	0.567
0.60	0.473
0.70	0.405
0.80	0.354
0.90	0.315
1.00	0.284
1.10	0.258
1.20	0.236
1.30	0.218
1.40	0.203
1.50	0.189
1.60	0.177
1.70	0.167
1.80	0.158
1.90	0.149
2.00	0.142
2.10	0.135
2.20	0.129
2.30	0.123
2.40	0.118
2.50	0.113
2.60	0.109
2.70	0.105
2.80	0.101
2.90	0.098
3.00	0.095
3.10	0.091
3.20	0.089
3.30	0.086
3.40	0.083
3.50	0.081
3.60	0.079
3.70	0.077
3.80	0.075
3.90	0.073
4.00	0.071
4.10	0.069
4.20	0.068
4.30	0.066
4.40	0.064
4.50	0.063
4.60	0.062
4.70	0.060
4.80	0.059
4.90	0.058
5.00	0.057
5.10	0.056
5.20	0.055
5.30	0.053

NCHRP 12-49 SOIL LIQUEFACTION STUDY
WASHINGTON SITE - 100Yr EQ (NCHRP)

DESIGN RESPONSE SPECTRUM

Using Two-Point Construction Method

	Short Periods	1-Sec Periods
Site Coefficients:	F_a = 2.46	F_v = 3.5
Spectral Acceleration on Class B Rock:	S_s = 0.261	S_1 = 0.081
Spectral Response Acceleration:	$S_{DS} = F_aS_s$ = **0.642**	$S_{D1} = F_vS_1$ = **0.284**

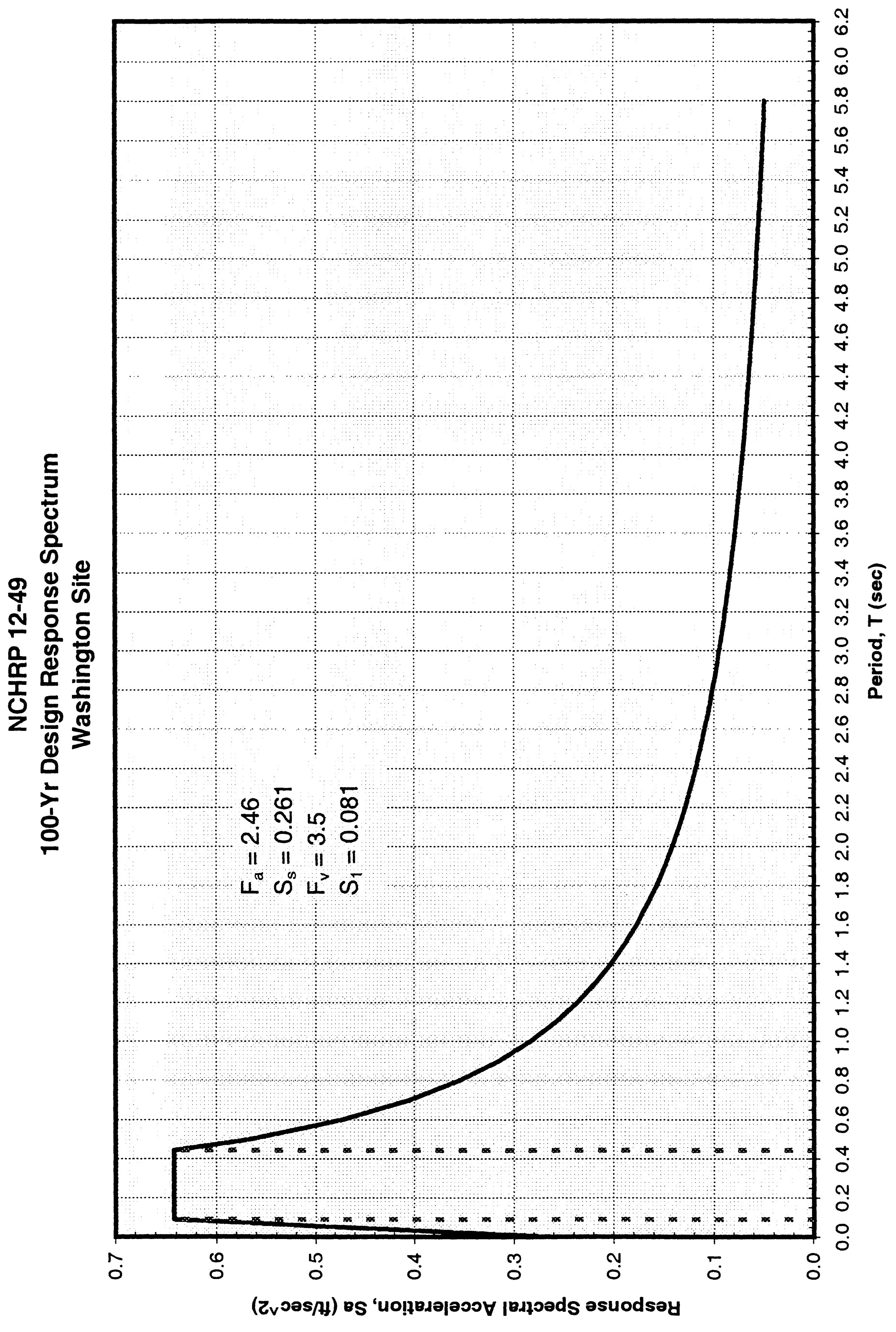
Key Points on Response Spectrum:

$T_s = S_{D1}/S_{DS}$ = 0.442

$T_0 = 0.2T_s$ = 0.088

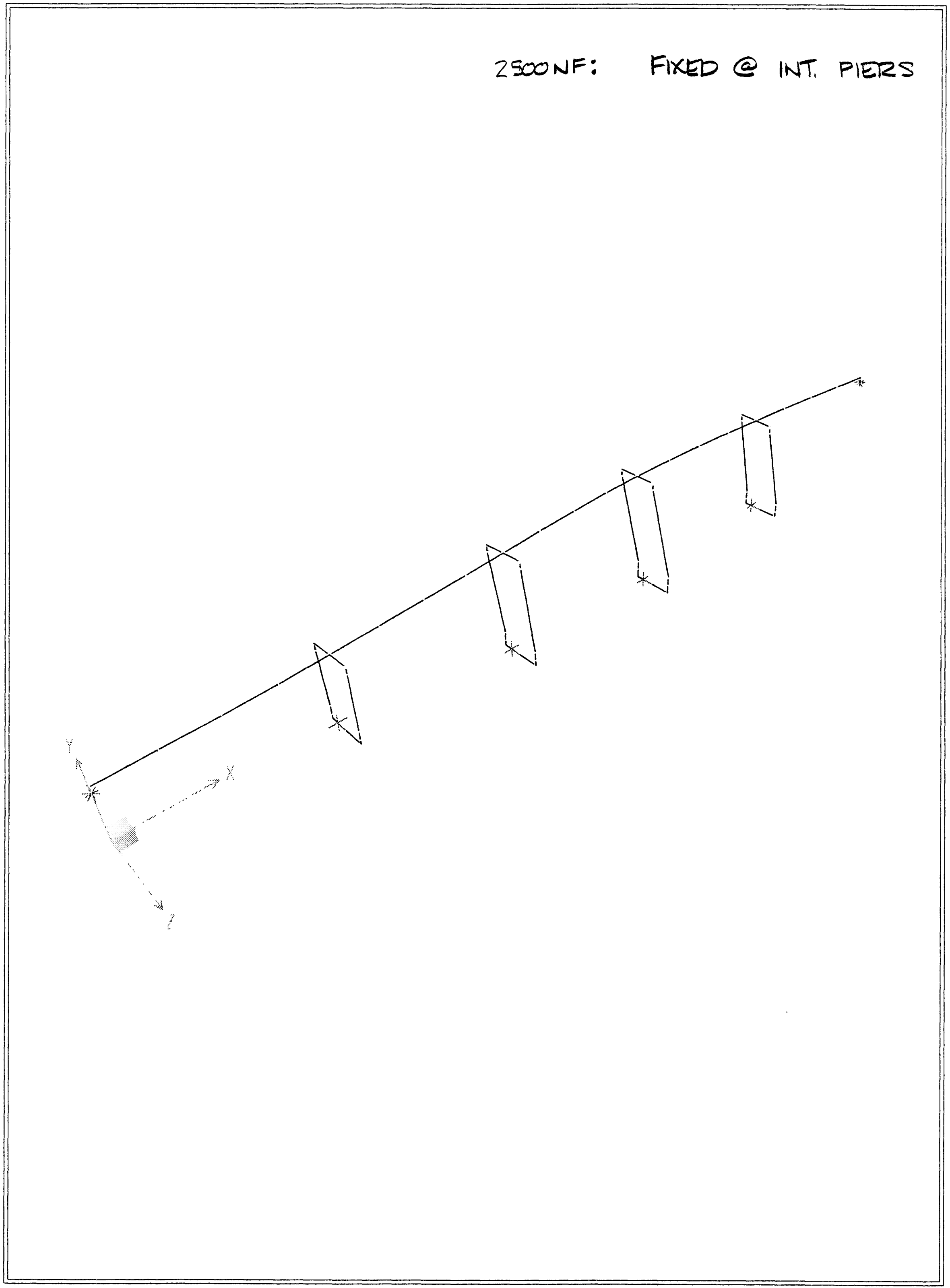
Y-Intercept = $0.4S_{DS}$ = 0.257

T sec	Sa ft/sec^2
5.40	0.053
5.50	0.052
5.60	0.051
5.70	0.050
5.80	0.049
5.90	0.048
6.00	0.047
6.10	0.046
6.20	0.046
6.30	0.045
6.40	0.044
6.50	0.044
6.60	0.043
6.70	0.042
6.80	0.042
6.90	0.041
7.00	0.041
7.10	0.040
7.20	0.039
7.30	0.039
7.40	0.038
7.50	0.038
7.60	0.037
7.70	0.037
7.80	0.036
7.90	0.036
8.00	0.035
8.10	0.035
8.20	0.035
8.30	0.034
8.40	0.034
10.00	0.028


NCHRP 12-49
100-Yr Design Response Spectrum
Washington Site
$F_a = 2.46$
$S_s = 0.261$
$F_v = 3.5$
$S_1 = 0.081$
Response Spectral Acceleration, Sa (ft/sec^2)
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
Period, T (sec)
0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0 3.2 3.4 3.6 3.8 4.0 4.2 4.4 4.6 4.8 5.0 5.2 5.4 5.6 5.8 6.0 6.2

E.6 SAP2000 Model Development: Period Check

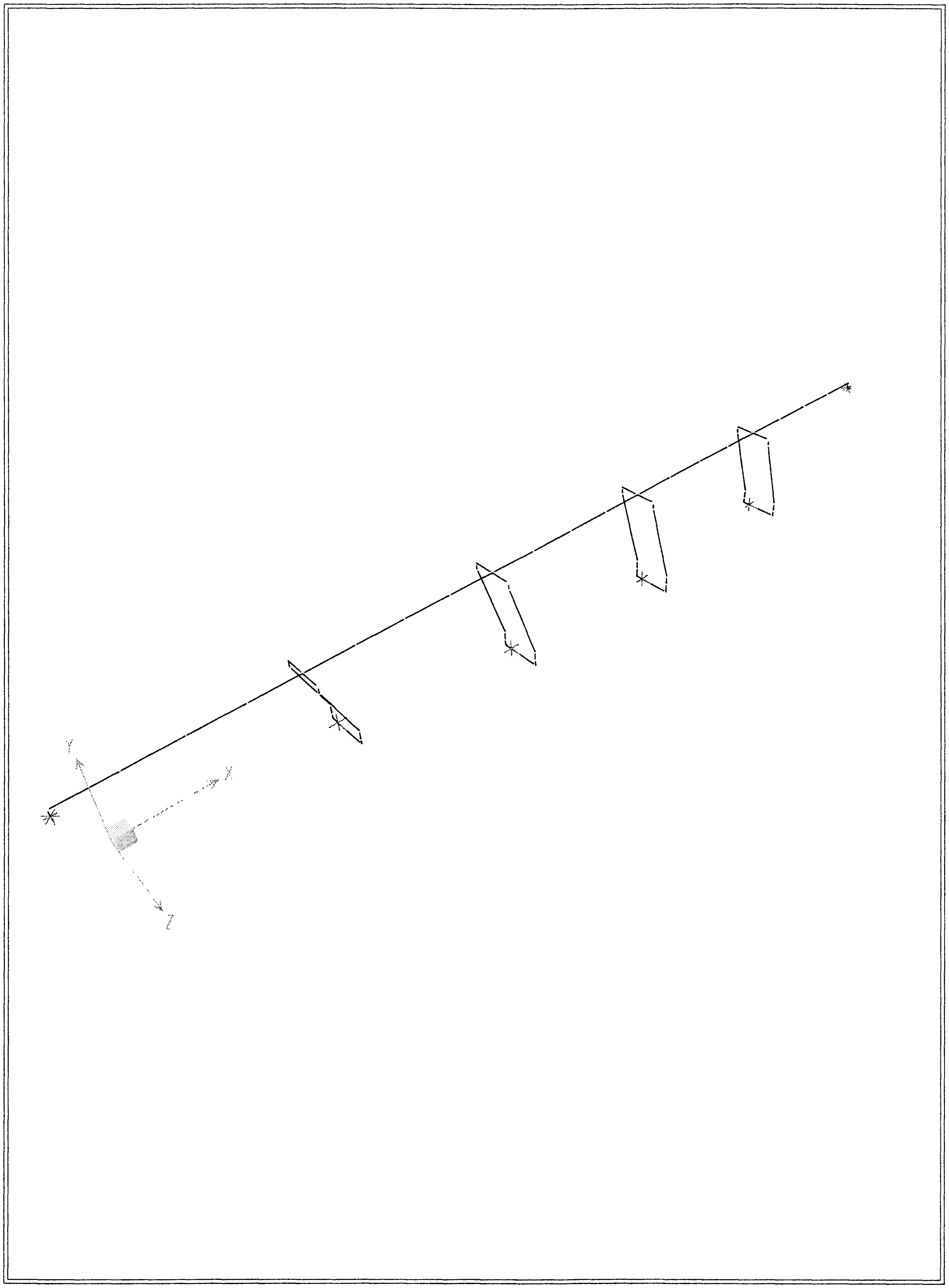
SAP2000 10/17/00 18:05:37



2500NF: FIXED @ INT. PIERS

SAP2000 v7.10 - File:Wa2500nf - Mode 1 Period 1.6189 seconds - Kip-ft Units

SAP2000 10/17/00 18:05:45



SAP2000 v7.10 - File:Wa2500nf - Mode 2 Period 1.3713 seconds - Kip-ft Units

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject PERIOD HAND CHECK
Sheet ____ of ____
Job Number A99067-400
Designer MLT
Date 24 JULY 2000

HAND CHECK FUNDAMENTAL PERIOD IN TRANSVERSE DIRECTION

STRUCTURE TOTAL WEIGHT

		W
CIP BOX GIRDER	$(72.18 ft^2)(0.150 kcf) = 10.83^{k}/ft \times 500'$	5415^k
SDL	$2.19^{k}/ft \times 500'$	1095^k
INT DIAPHRAGM	5×15.3^k	76.5^k
CROSS BEAMS	4×102^k	408^k
END DIAPHRAGM	2×117.6^k	235.2^k
COLUMN	$(12.57 ft^2)(0.15) = (1.89^{k}/ft)(30' + 45'(2) + 50')(2)$	$642.6^k \times 1/2$
~~INT. PIER PILE CAP~~	~~$4 \times (1012 ft^2)(5')(0.15)$~~	~~3036^k~~
~~SEALS~~	~~$(1012 ft^2)(3' + 4' + 6' + 4')(0.15)$~~	~~2580.6^k~~

$W_{TOTAL} = \cancel{13489^k}$

7551^k

SINGLE COLUMN STIFFNESS (CRACKED)

$$K_{col} = \frac{12EI_{cr}}{L^3} = \frac{12(552000\ ksf)(5\ ft^4)}{(42.5')^3} \Rightarrow \underline{\underline{K_{col} = 431\ k/ft}}$$

$$L_{avg} = \frac{30' + 45' + 50' + 45'}{4}$$

$$K_{bent} = 2K_{col} = 2(431\ k/ft) \Rightarrow \underline{\underline{K_{bent} = 863\ k/ft}}$$

$$T_T = 2\pi\sqrt{\frac{W_{TOTAL}/\cancel{\#}SPANS}{K_{bent}\,g}} = 2\pi\sqrt{\frac{17551^k(1/5\ spans)}{(863^{k}/ft)(32.2\ ft/sec^2)}} \Rightarrow \boxed{T_T = 1.46\ sec}$$

FOR INT. PIERS w/o SPRINGS, SAP $\Rightarrow T_T = 1.62 sec$

$$\left(\frac{1.46}{1.62} = 90\%\right)$$

Project NCHRP 12-49

Sheet ______ of ______

Job Number A99067-400

Subject

Designer MLT

Date 24 JULY 2000

HAND CHECK FUNDAMENTAL PERIOD IN LONGITUDINAL DIRECTION

$$W_{TOTAL} = 7551^{k}$$

$$k_{col} = \frac{12EI_{cr}}{L^3} = 431 \, ^{k}/_{ft}$$

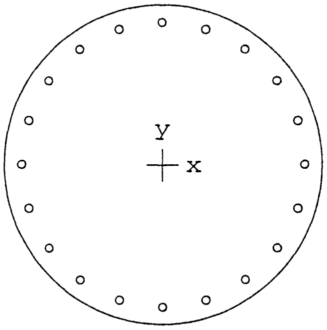
$$k_L = 8 \, k_{col} = 3448 \, ^{k}/_{ft}$$

$$T_L = 2\pi \sqrt{\frac{7551^{k}}{(3448)(32.2)}} \Rightarrow \boxed{T_L = 1.64 \text{ sec}}$$

FOR INT. PIERS w/o SPRINGS, SAP $\Rightarrow T_L = 1.37$

$$\left(\frac{1.64}{1.37} = 120\%\right)$$

E.7 Column Design



48 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

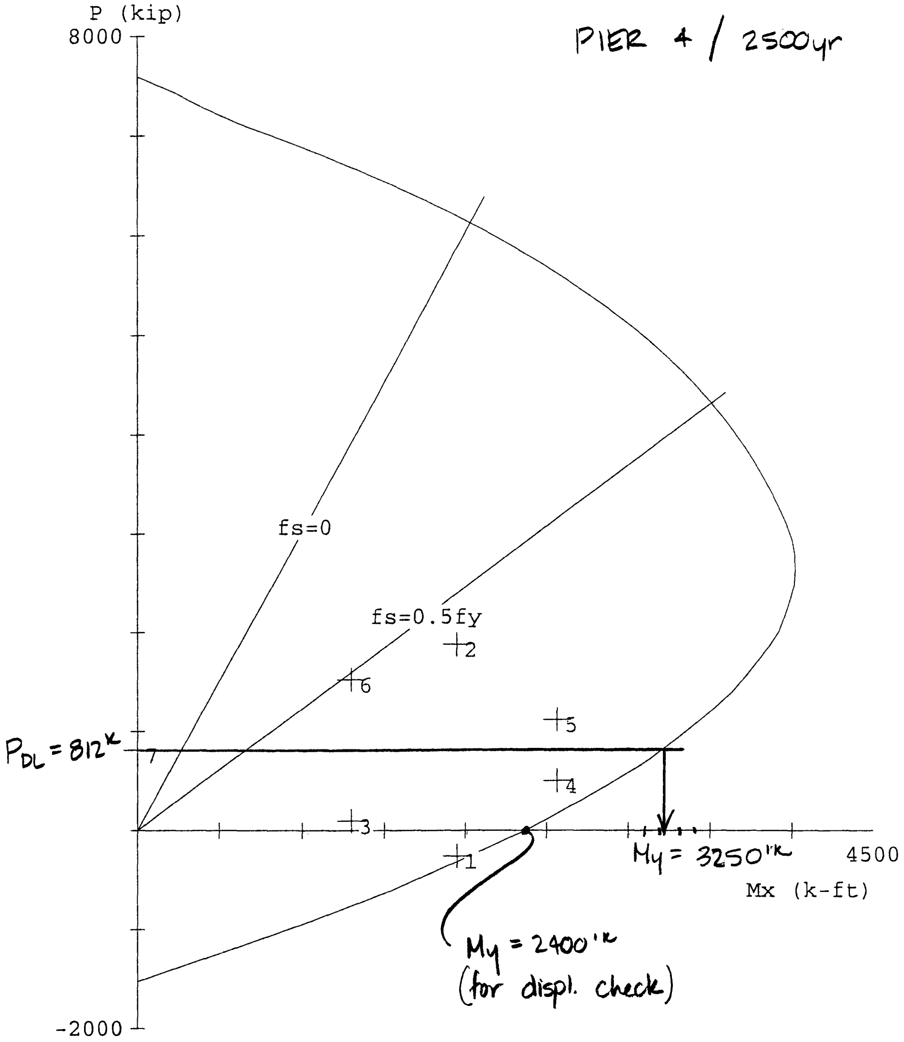
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 12:27:07



PIER 4 / 2500yr

$P_{DL} = 812^k$

$M_y = 3250'^k$

$M_y = 2400'^k$
(for displ. check)

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\PIER4.COL

Project:

Column: Engineer:

f'c = 4 ksi fy = 60 ksi Ag = 1809.56 in^2 20 #10 bars

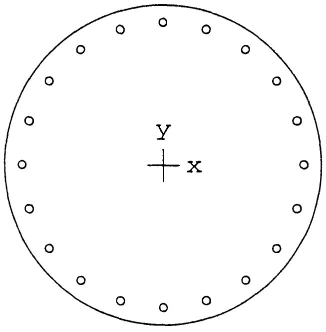
Ec = 3605 ksi Es = 29000 ksi As = 25.40 in^2 Rho = 1.40%

fc = 3.4 ksi e_rup = Infinity Xo = 0.00 in Ix = 260576 in^4

e_u = 0.003 in/in Yo = 0.00 in Iy = 260576 in^4

Beta1 = 0.85 Clear spacing = 5.41 in Clear cover = 2.00 in

Confinement: Other phi(a) = 1, phi(b) = 1, phi(c) = 1



48 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

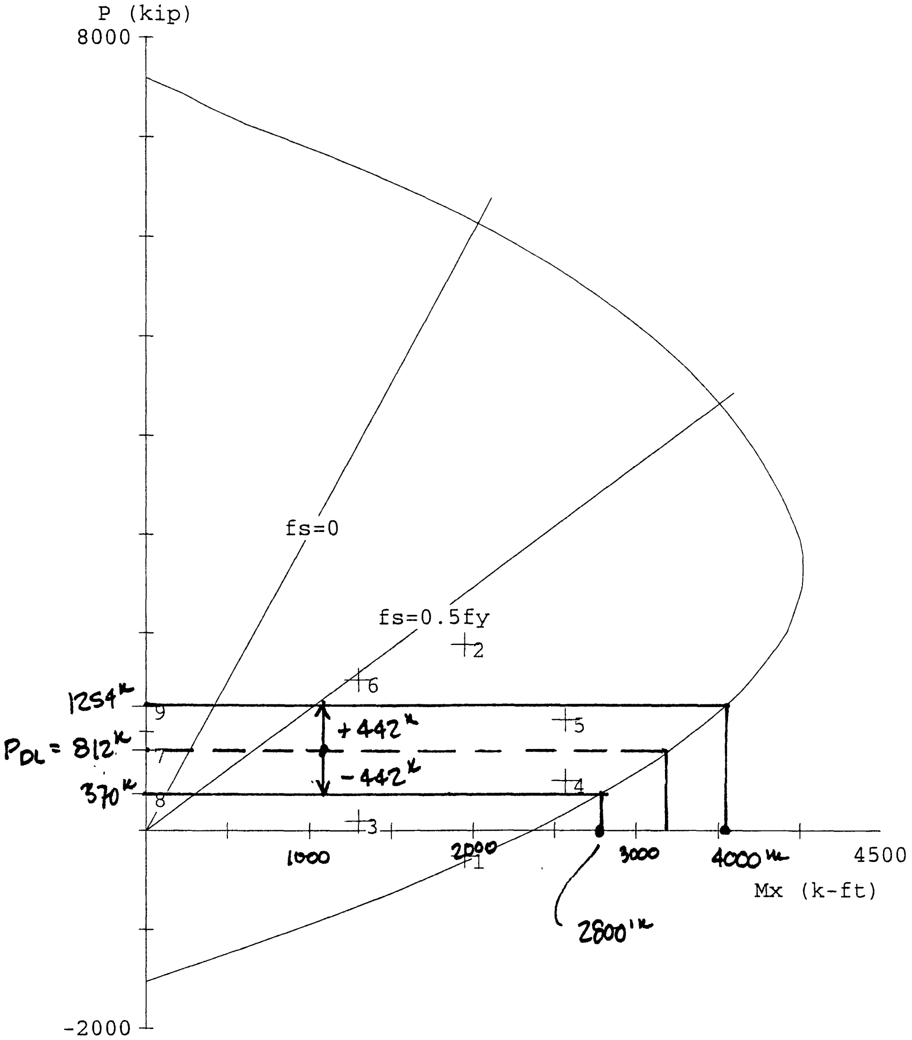
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/12/00

Time: 15:06:53



PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\PIER4.COL

Project:

Column:

Engineer:

f'c = 4 ksi

fy = 60 ksi

Ag = 1809.56 in^2

20 #10 bars

Ec = 3605 ksi

Es = 29000 ksi

As = 25.40 in^2

Rho = 1.40%

fc = 3.4 ksi

e_rup = Infinity

Xo = 0.00 in

Ix = 260576 in^4

e_u = 0.003 in/in

Yo = 0.00 in

Iy = 260576 in^4

Beta1 = 0.85

Clear spacing = 5.41 in

Clear cover = 2.00 in

Confinement: Other

phi(a) = 1, phi(b) = 1, phi(c) = 1

NCHRP 12-49 LIQUEFACTION STUDY

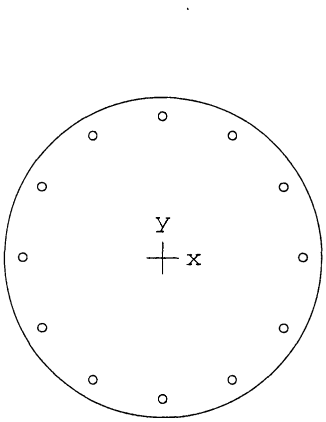
Washington Bridge - Pier 4 (2500yr)

h = 53.38 ft	V = 1.5*Mntop + 1.5*Mnbot / H	Eq. (1)
H = 50.00 ft	ΔP = [S(V)*h - S(1.5*Mnbot)] / (a*10/9)	Eq. (2)
a = 22.52 ft	Adjusted P = P +/- ΔP	Eq. (3)

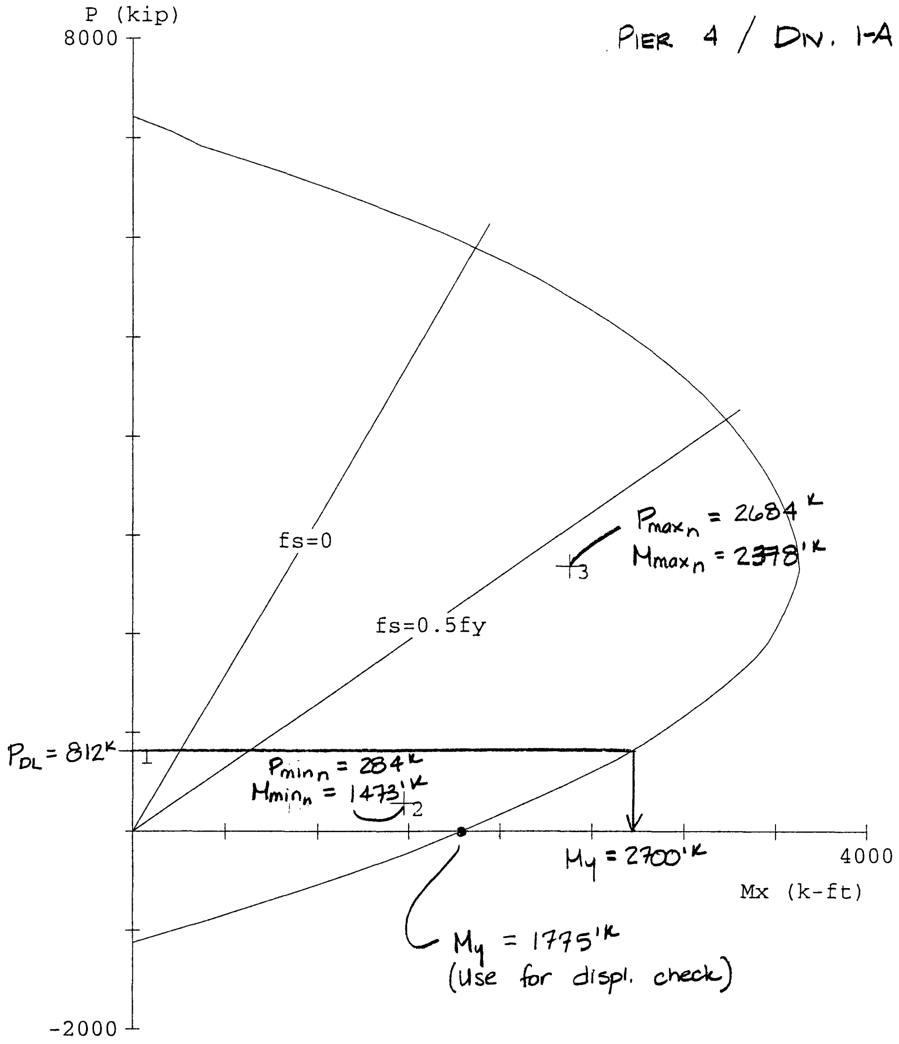
Step	Column Moments (kips-ft)					Column Shears (kips)				Column Axial Forces	
	M	Left		Right		Left	Right	Total	ΔP	P (kips)	
		Top	Bottom	Top	Bottom				(kips)	Left	Right
Given										812	812
1a	Mn	3,250	3,250	3,250	3,250						
1b	1.5*Mn	4,875	4,875	4,875	4,875						
2						195	195	**390**			
3a									442		
3b										370	1254
4a	Mn	2,800	2,800	4,000	4,000						
4b	1.5*Mn	4,200	4,200	6,000	6,000						
4c						168	240	**408**			
4d									463		
4e										349	1275
4f		**Difference in Total Column Plastic Shears (%) =** (from Steps 2 and 4c)						**-4.6% <10%**		**OK**	

The forces in the individual columns in the plane of the bent associated with plastic hinging of the bent columns are

	a)	Axial Load	P min_p =	349 kips
Minimum Axial Load:	b)	Moment	Mminp = 1.5*Mminn =	4200 ft-kips
	c)	Shear	V min_p =	168 kips
	a)	Axial Load	P max_p =	1275 kips
Maximum Axial Load :	b)	Moment	Mmaxp = 1.5*Mmaxn =	6000 ft-kips
	c)	Shear	V max_p =	240 kips



48 in diam.



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Design

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 11:28:44

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\PIER4500.COL

Project:

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 1809.56 in^2	12 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 18.72 in^2	Rho = 1.03%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 260576 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 260576 in^4
Beta1 = 0.85		Clear spacing = 9.55 in	Clear cover = 2.13 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		

COLUMN DESIGN - Modify PCACOL loads to introduce ϕ-factors:

(Use ϕ=1 in PCACOL load combinations.)

$\mathrm{ksi} := 1000 \cdot \frac{\mathrm{lb}}{\mathrm{in}^2}$

$\mathrm{kips} := 1000 \cdot \mathrm{lb}$

PIER 4

Input:

$Pmin_u := 237 \cdot \mathrm{kips}$

$Pmax_u := 1387 \cdot \mathrm{kips}$

$M_u := 1229 \cdot \mathrm{ft} \cdot \mathrm{kips}$

ϕ Factors

Concrete strength: $f'_c := 4 \cdot \mathrm{ksi}$ $0.2 \cdot f'_c = 0.800 \cdot \mathrm{ksi}$

Column radius: $r := \frac{48 \cdot \mathrm{in}}{2}$ $r = 24 \cdot \mathrm{in}$

$A_g := \pi \cdot r^2$ $A_g = 12.566\ \mathrm{ft}^2$

$\sigma_{min} := \frac{Pmin_u}{A_g}$ $\sigma_{min} = 0.131 \cdot \mathrm{ksi}$

$\phi_{min} := 0.9 - \frac{\sigma_{min}}{0.2 \cdot f'_c} \cdot (0.9 - 0.5)$ $\phi_{min} = 0.835$

$\sigma_{max} := \frac{Pmax_u}{A_g}$ $\sigma_{max} = 0.766 \cdot \mathrm{ksi}$

$\phi_{max} := 0.9 - \frac{\sigma_{max}}{0.2 \cdot f'_c} \cdot (0.9 - 0.5)$ $\phi_{max} = 0.517$

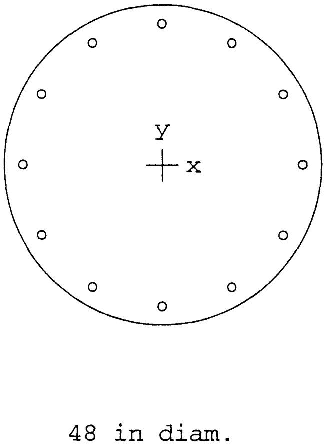
Axial Load and Moment

$Pmin_n := \frac{Pmin_u}{\phi_{min}}$ $Pmin_n = 284 \cdot \mathrm{kips}$

$Mmin_n := \frac{M_u}{\phi_{min}}$ $Mmin_n = 1472.7 \cdot \mathrm{ft} \cdot \mathrm{kips}$

$Pmax_n := \frac{Pmax_u}{\phi_{max}}$ $Pmax_n = 2684 \cdot \mathrm{kips}$

$Mmax_n := \frac{M_u}{\phi_{max}}$ $Mmax_n = 2378.3 \cdot \mathrm{ft} \cdot \mathrm{kips}$



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

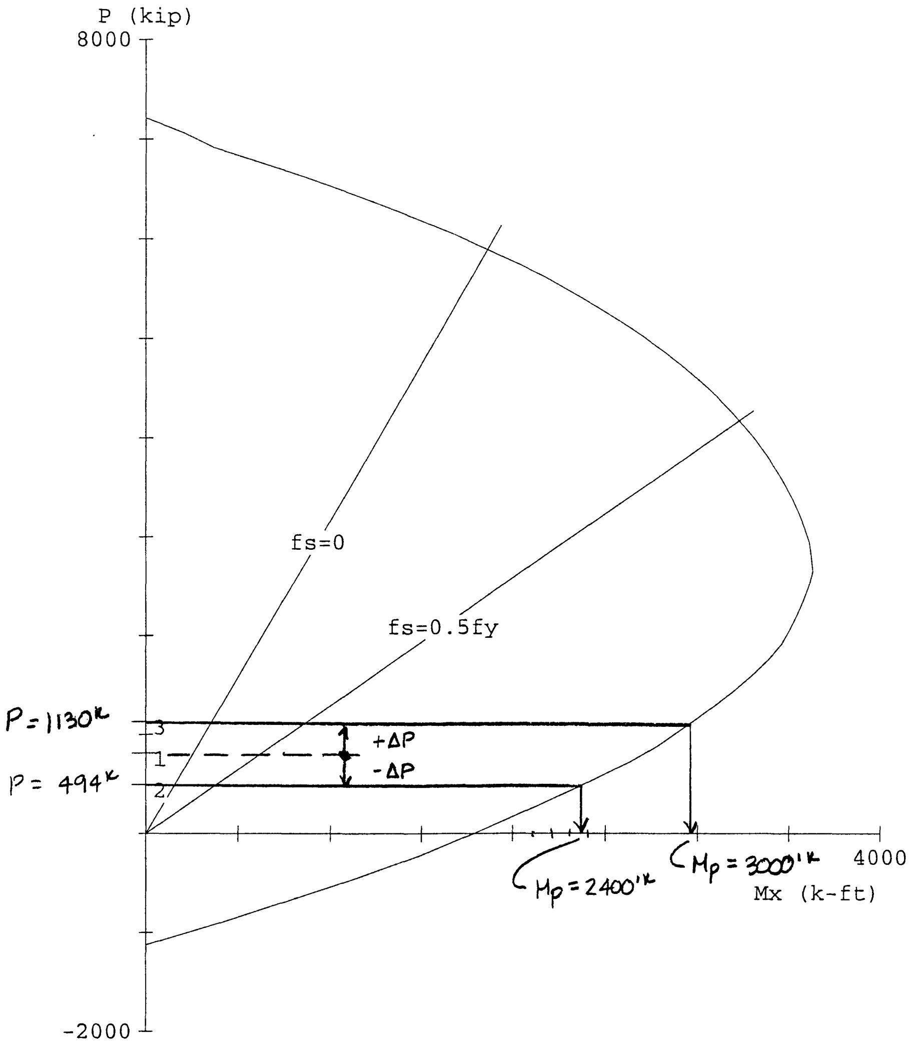
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 11:54:00



PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\P4500-H.COL

Project:

Column: Engineer:

f'c = 4 ksi fy = 60 ksi Ag = 1809.56 in^2 12 #11 bars

Ec = 3605 ksi Es = 29000 ksi As = 18.72 in^2 Rho = 1.03%

fc = 3.4 ksi e_rup = Infinity Xo = 0.00 in Ix = 260576 in^4

e_u = 0.003 in/in Yo = 0.00 in Iy = 260576 in^4

Beta1 = 0.85 Clear spacing = 9.55 in Clear cover = 2.13 in

Confinement: Other phi(a) = 1, phi(b) = 1, phi(c) = 1

NCHRP 12-49 LIQUEFACTION STUDY

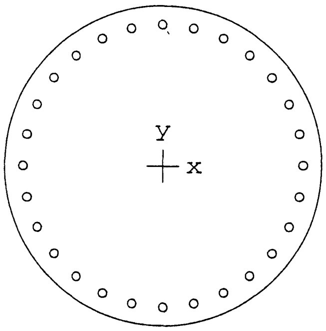
Washington Bridge - Pier 4 (500yr)

h =	53.38 ft	V = 1.3*Mntop + 1.3*Mnbot / H	Eq. (1)
H =	50.00 ft	ΔP = [S(V)*h - S(1.3*Mnbot)] / (a*10/9)	Eq. (2)
a =	22.52 ft	Adjusted P = P +/- ΔP	Eq. (3)

Step	Column Moments (kips-ft)					Column Shears (kips)				Column Axial Forces	
	M	Left		Right		Left	Right	Total	ΔP	P (kips)	
		Top	Bottom	Top	Bottom				(kips)	Left	Right
Given										812	812
1a	Mn	2,700	2,700	2,700	2,700						
1b	1.3*Mn	3,510	3,510	3,510	3,510						
2						140	140	**281**			
3a									318		
3b										494	1130
4a	Mn	2,400	2,400	3,000	3,000						
4b	1.3*Mn	3,120	3,120	3,900	3,900						
4c						125	156	**281**			
4d									318		
4e										494	1130
4f		**Difference in Total Column Plastic Shears (%) =** (from Steps 2 and 4c)						**0.0% <10%**		**OK**	

The forces in the individual columns in the plane of the bent associated with plastic hinging of the bent columns are

	a)	Axial Load	P min_p =	494 kips
Minimum Axial Load:	b)	Moment	$Mmin_p$ = 1.3*$Mmin_n$	3120 ft-kips
	c)	Shear	V min_p =	125 kips
	a)	Axial Load	P max_p =	1130 kips
Maximum Axial Load :	b)	Moment	$Mmax_p$ = 1.3*Mmax	3900 ft-kips
	c)	Shear	V max_p =	156 kips



48 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

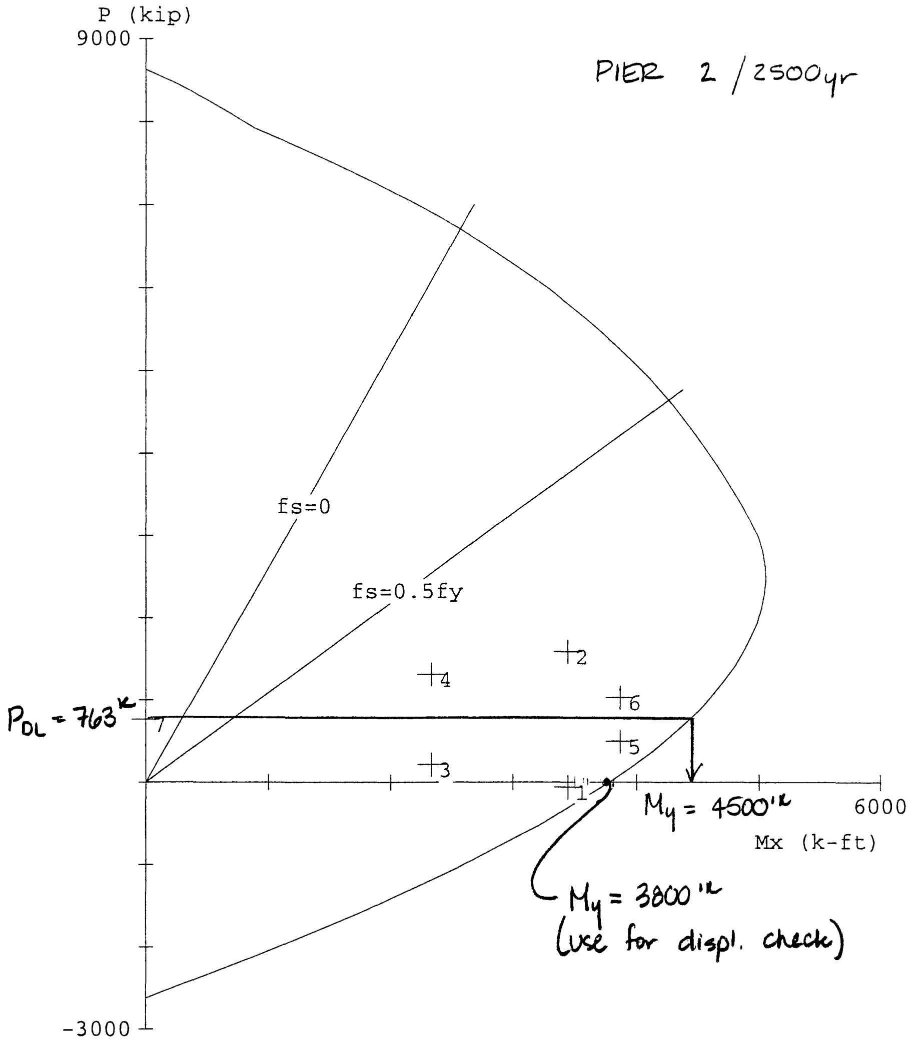
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 12:42:07

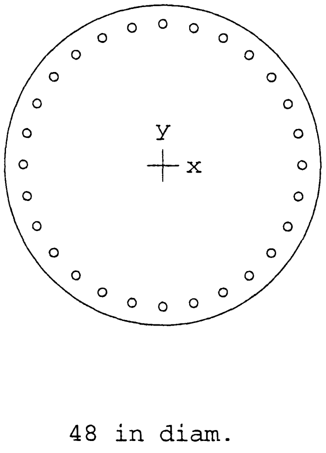


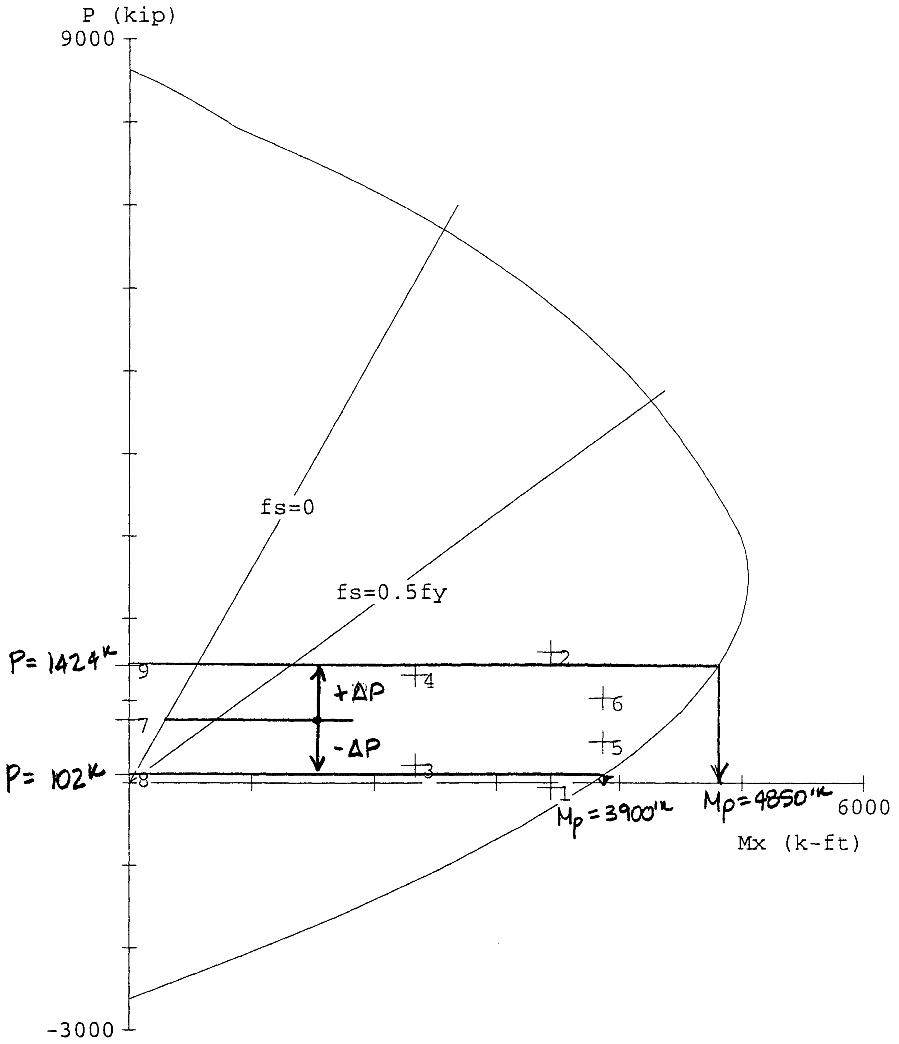
PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\PIER2.COL

Project:

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 1809.56 in^2	28 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 43.68 in^2	Rho = 2.41%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 260576 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 260576 in^4
Beta1 = 0.85		Clear spacing = 3.33 in	Clear cover = 2.13 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		





Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 12:45:45

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\PIER2.COL

Project:

Column: Engineer:

f'c = 4 ksi	fy = 60 ksi	Ag = 1809.56 in^2	28 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 43.68 in^2	Rho = 2.41%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 260576 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 260576 in^4
Beta1 = 0.85		Clear spacing = 3.33 in	Clear cover = 2.13 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		

NCHRP 12-49 LIQUEFACTION STUDY

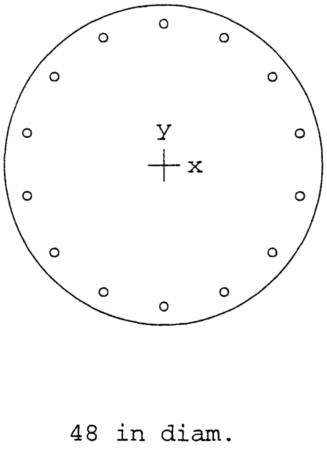
Washington Bridge - Pier 2 (2500yr)

h = 33.38 ft V = 1.5*Mntop + 1.5*Mnbot / H Eq. (1)
H = 30.00 ft ΔP = [S(V)*h - S(1.5*Mnbot)] / (a*10/9) Eq. (2)
a = 22.52 ft Adjusted P = P +/- ΔP Eq. (3)

Step	Column Moments (kips-ft)					Column Shears (kips)				Column Axial Forces	
	M	Left		Right		Left	Right	Total	ΔP	P (kips)	
		Top	Bottom	Top	Bottom				(kips)	Left	Right
Given										763	763
1a	Mn	4,500	4,500	4,500	4,500						
1b	1.5*Mn	6,750	6,750	6,750	6,750						
2						450	450	**900**			
3a									661		
3b										102	1424
4a	Mn	3,900	3,900	4,850	4,850						
4b	1.5*Mn	5,850	5,850	7,275	7,275						
4c						390	485	**875**			
4d									643		
4e										120	1406
4f		**Difference in Total Column Plastic Shears (%) =** (from Steps 2 and 4c)						**2.8% <10%**		**OK**	

The forces in the individual columns in the plane of the bent associated with plastic hinging of the bent columns are

	a)	Axial Load	P min_p =	120 kips
<u>Minimum Axial Load:</u>	b)	Moment	Mminp = 1.5*Mminn =	5850 ft-kips
	c)	Shear	V min_p =	390 kips
	a)	Axial Load	P max_p =	1406 kips
<u>Maximum Axial Load :</u>	b)	Moment	Mmaxp = 1.5*Mmaxn =	7275 ft-kips
	c)	Shear	V max_p =	485 kips



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Design

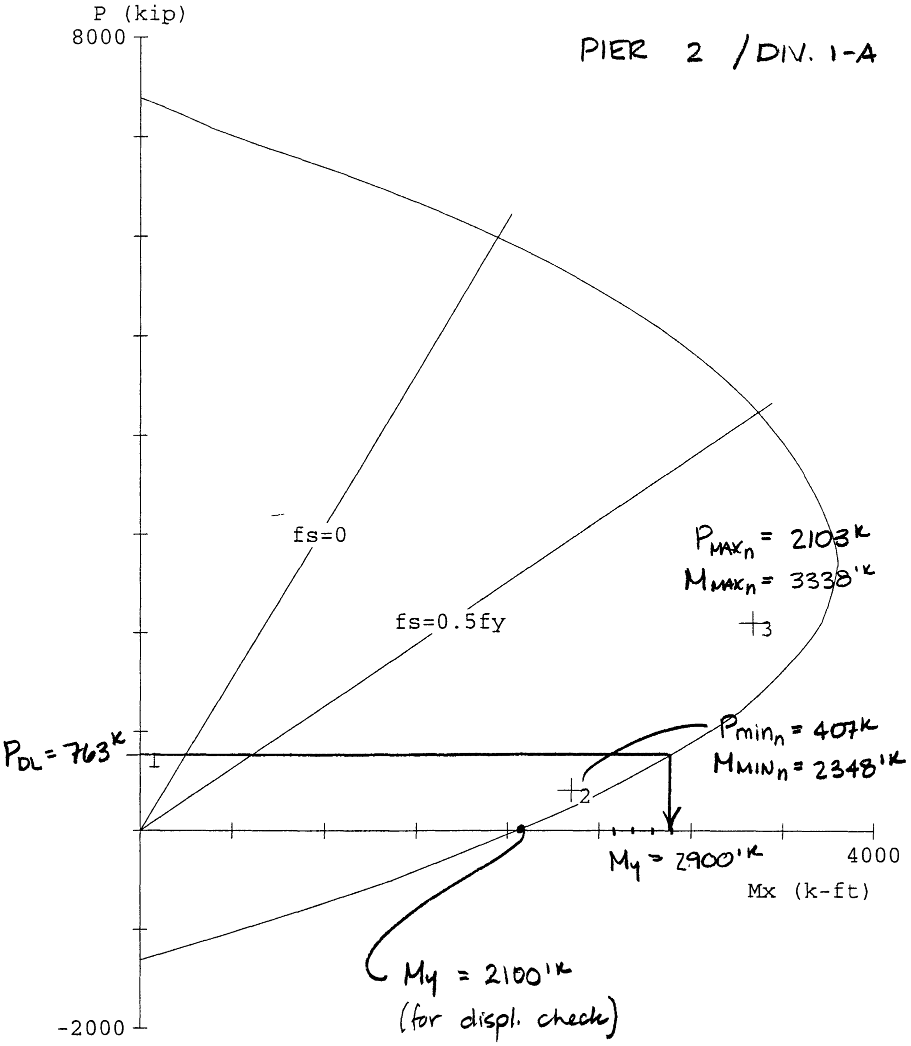
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 12:12:17



PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\PIER2500.COL

Project:

Column: Engineer:

f'c = 4 ksi fy = 60 ksi Ag = 1809.56 in^2 14 #11 bars

Ec = 3605 ksi Es = 29000 ksi As = 21.84 in^2 Rho = 1.21%

fc = 3.4 ksi e_rup = Infinity Xo = 0.00 in Ix = 260576 in^4

e_u = 0.003 in/in Yo = 0.00 in Iy = 260576 in^4

Beta1 = 0.85 Clear spacing = 8.01 in Clear cover = 2.13 in

Confinement: Other phi(a) = 1, phi(b) = 1, phi(c) = 1

COLUMN DESIGN - Modify PCACOL loads to introduce ϕ-factors:

(Use ϕ=1 in PCACOL load combinations.)

$ksi := 1000 \cdot \frac{lb}{in^2}$

$kips := 1000 \cdot lb$

PIER 2

Input: $Pmin_u := 329 \cdot kips$

$M_u := 1900 \cdot ft \cdot kips$

$Pmax_u := 1197 \cdot kips$

ϕ Factors

Concrete strength: $f'_c := 4 \cdot ksi$ $\quad 0.2 \cdot f'_c = 0.800 \cdot ksi$

Column radius: $r := \frac{48 \cdot in}{2}$ $\quad r = 24 \cdot in$

$A_g := \pi \cdot r^2$ $\quad A_g = 12.566\ ft^2$

$\sigma_{min} := \frac{Pmin_u}{A_g}$ $\quad \sigma_{min} = 0.182 \cdot ksi$

$\phi_{min} := 0.9 - \frac{\sigma_{min}}{0.2 \cdot f'_c} \cdot (0.9 - 0.5)$ $\quad \phi_{min} = 0.809$

$\sigma_{max} := \frac{Pmax_u}{A_g}$ $\quad \sigma_{max} = 0.661 \cdot ksi$

$\phi_{max} := 0.9 - \frac{\sigma_{max}}{0.2 \cdot f'_c} \cdot (0.9 - 0.5)$ $\quad \phi_{max} = 0.569$

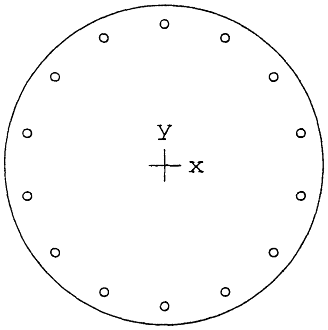
Axial Load and Moment

$Pmin_n := \frac{Pmin_u}{\phi_{min}}$ $\quad Pmin_n = 407 \cdot kips$

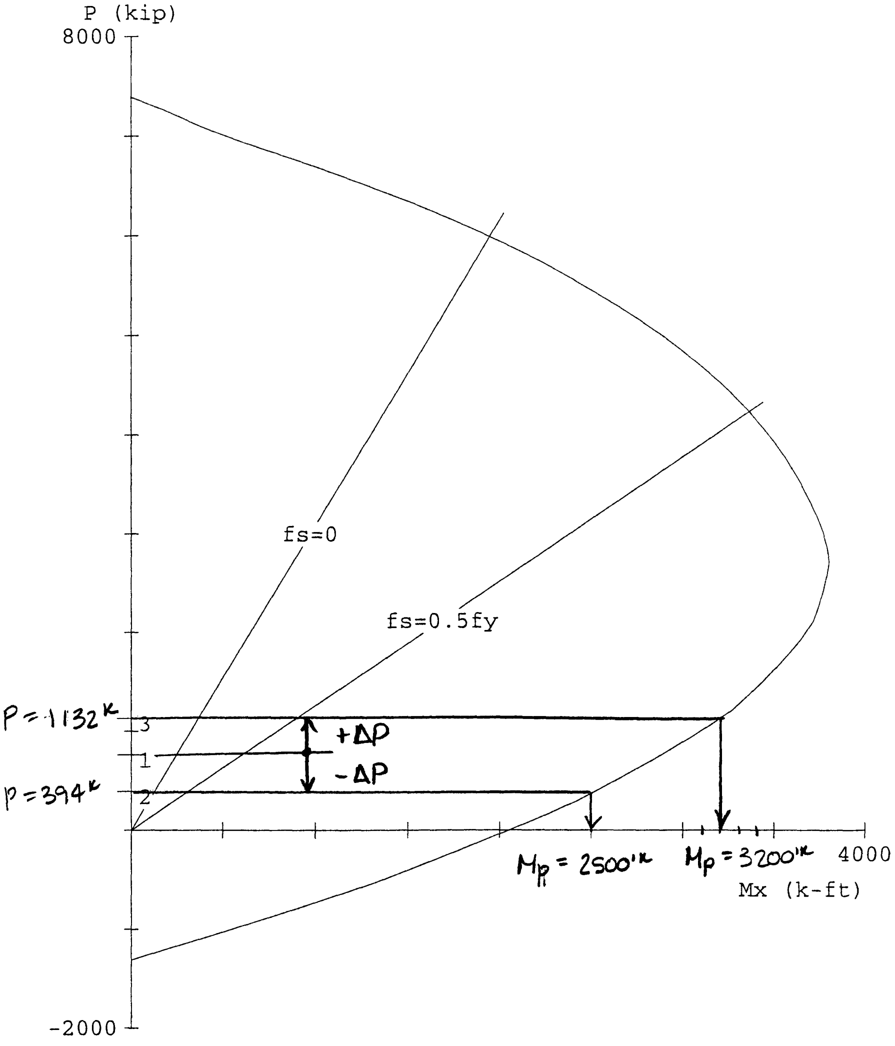
$Mmin_n := \frac{M_u}{\phi_{min}}$ $\quad Mmin_n = 2348.3 \cdot ft \cdot kips$

$Pmax_n := \frac{Pmax_u}{\phi_{max}}$ $\quad Pmax_n = 2103 \cdot kips$

$Mmax_n := \frac{M_u}{\phi_{max}}$ $\quad Mmax_n = 3337.7 \cdot ft \cdot kips$



48 in diam.



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 12:17:49

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\DESIGN\P2500-H.COL

Project:

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 1809.56 in^2	14 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 21.84 in^2	Rho = 1.21%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 260576 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 260576 in^4
Beta1 = 0.85		Clear spacing = 8.01 in	Clear cover = 2.13 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		

NCHRP 12-49 LIQUEFACTION STUDY

Washington Bridge - Pier 2 (500yr)

h =	33.38 ft	V = 1.3*Mntop + 1.3*Mnbot / H	Eq. (1)
H =	30.00 ft	ΔP = [S(V)*h - S(1.3*Mnbot)] / (a*10/9)	Eq. (2)
a =	22.52 ft	Adjusted P = P +/- ΔP	Eq. (3)

Step	Column Moments (kips-ft)					Column Shears (kips)				Column Axial Forces	
	M	Left		Right		Left	Right	Total	ΔP	P (kips)	
		Top	Bottom	Top	Bottom				(kips)	Left	Right
Given										763	763
1a	Mn	2,900	2,900	2,900	2,900						
1b	1.3*Mn	3,770	3,770	3,770	3,770						
2						251	251	**503**			
3a									369		
3b										394	1132
4a	Mn	2,500	2,500	3,200	3,200						
4b	1.3*Mn	3,250	3,250	4,160	4,160						
4c						217	277	**494**			
4d									363		
4e										400	1126
4f	**Difference in Total Column Plastic Shears (%) =** (from Steps 2 and 4c)							**1.7% <10%**	**OK**		

The forces in the individual columns in the plane of the bent associated with plastic hinging of the bent columns are

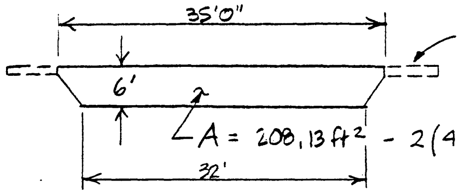
	a)	Axial Load	P min_p =	400 kips
Minimum Axial Load:	b)	Moment	$Mmin_p$ = 1.3*$Mmin_n$	3250 ft-kips
	c)	Shear	V min_p =	217 kips
	a)	Axial Load	P max_p =	1126 kips
Maximum Axial Load :	b)	Moment	$Mmax_p$ = 1.3*Mmax	4160 ft-kips
	c)	Shear	V max_p =	277 kips

E.8 Passive Soil Forces

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject FLOW FORCES
ON ABUTMENT

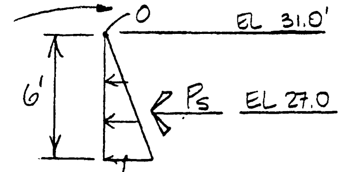
Sheet ______ of ______
Job Number A99067
Designer MLT
Date 24 AUG 2000

SUPERSTRUCTURE / END DIAPHRAGM - LAYER ①



$A = 208.13 ft^2 - 2(4')(0.75') = 208.13 - 6 \Rightarrow \underline{A_{super} = 202.13 ft^2}$

ASSUME NO SURCHARGE



$\gamma z K_p = (.110 kcf)(6')(4.0)$
$p_p = 2.64 ksf$

$(6')(\frac{208.13}{6'})$

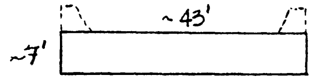
$P_S = \frac{1}{2} p_p A_{super} = \frac{1}{2}(2.64 ksf)(202.13 ft^2)$

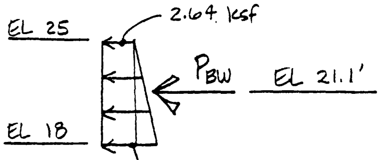
OR ASSUME RECT.
$p_p = (.11)(3')(4)$
$= 1.32 ksf$
$(1.32 ksf)(A) =$

$\boxed{P_S = 267^k}$
@ EL 27.0

ABUTMENT BACK WALL - LAYER ①

$y = \frac{(2.64)(3.5') + \frac{1}{2}(3.08)(2.33)}{\frac{1}{2}(2.64 + 5.72)} = 3.07'$





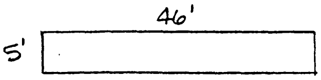
$(0.11 kcf)(13')(4.0) = 5.72 ksf$

$P_{BW} = \frac{1}{2}(2.64 + 5.72 ksf)(43')(7') \longrightarrow \boxed{P_{BW} = 1258^k}$
@ EL 21.5'

OR RECT $P_{BW} = (4.18 ksk)(43)(7) = 1258$ @ EL 21.5'

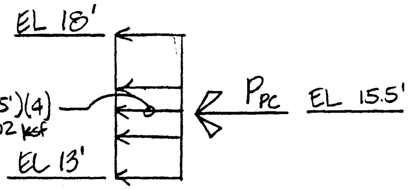
JUST USE RECT. DISTRIBUTI

ABUTMENT PILE CAP - LAYER ①



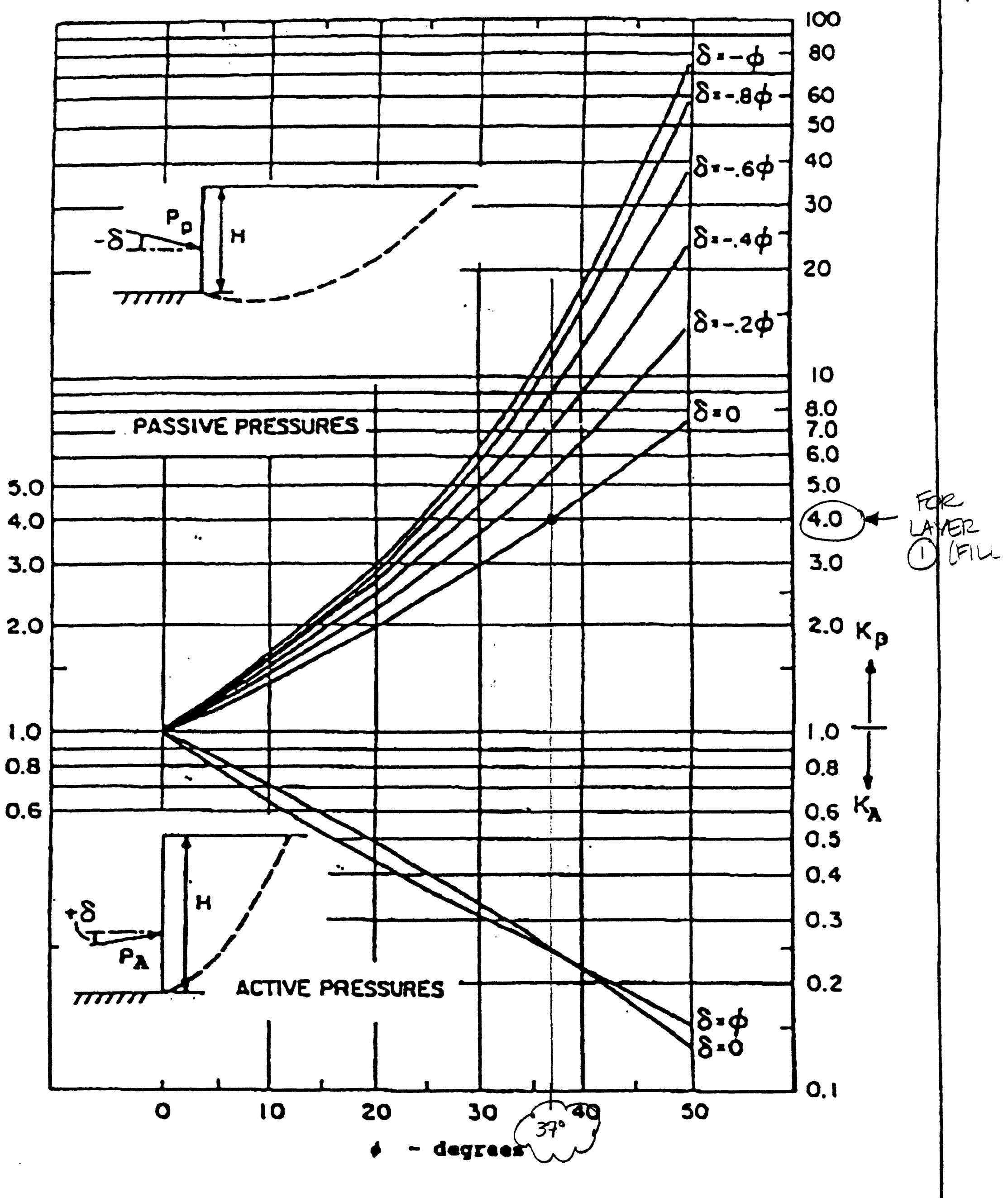
$P_{PC} = p_P A = (6.82 ksf)(46')(5')$

$\boxed{P_{PC} = 1569^k}$
@ EL 15.5'



$p_P = (0.11)(15.5')(4)$
$p_p = 6.82 ksf$

12/13



ACTIVE AND PASSIVE PRESSURE COEFFICIENTS FOR VERTICAL WALL AND HORIZONTAL BACKFILL - BASED ON LOG SPIRAL FAILURE SURFACES (AFTER CAQUOT AND KERISEL, 1948)

Seismic Design of Highway Bridge Foundations - Training Course

(FROM GEOFF MARTIN 9/12/00 FAX)

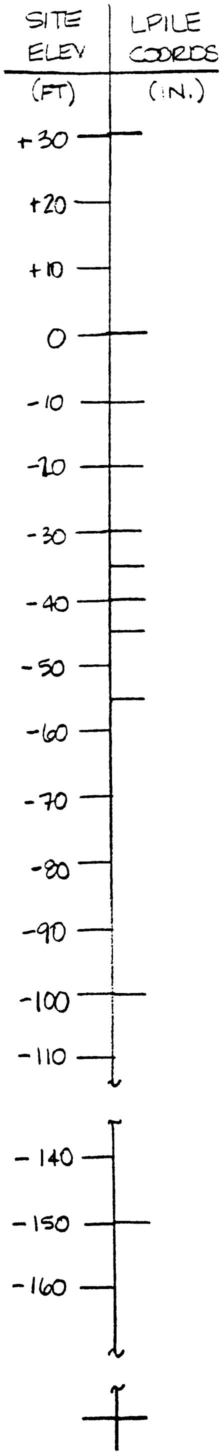
7.19

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject SOIL PROFILE
FOR LIQUEFIED STATE

Sheet ______ of ______
Job Number A99067 - 400
Designer HLT
Date 13 JULY 2000

PIER PILE

LIQUEFIED SOIL PROFILE



① SAND (FILL) γ = 110 pcf = 0.064 pci
k = 150 pci → 75 pci for Group Effect (GE)
Ø = 37° → 31° (GE)

② SOFT CLAY γ = 120 pcf = 0.069 pci K = 10 pci → 5 pci (GE)
C_u = 1000 psf = 6.94 psi → 3.47 psi (GE), typ
ε_{50} = 0.010

③ SOFT CLAY γ = 60 pcf = 0.035 pci k = 3 pci → 1.5 pci (GE
C_u = 300 psf = 2.08 psi → 1.04 (GE) ε_{50} = 0.020

④ SAND γ = 60 pcf Ø = 30°
k = 40 pci → 20 pci (GE)

⑤ SAND γ = 60 pcf k = 40 pci Ø = 24°

⑥ SAND γ = 60 pcf k = 40 pci Ø = 30°

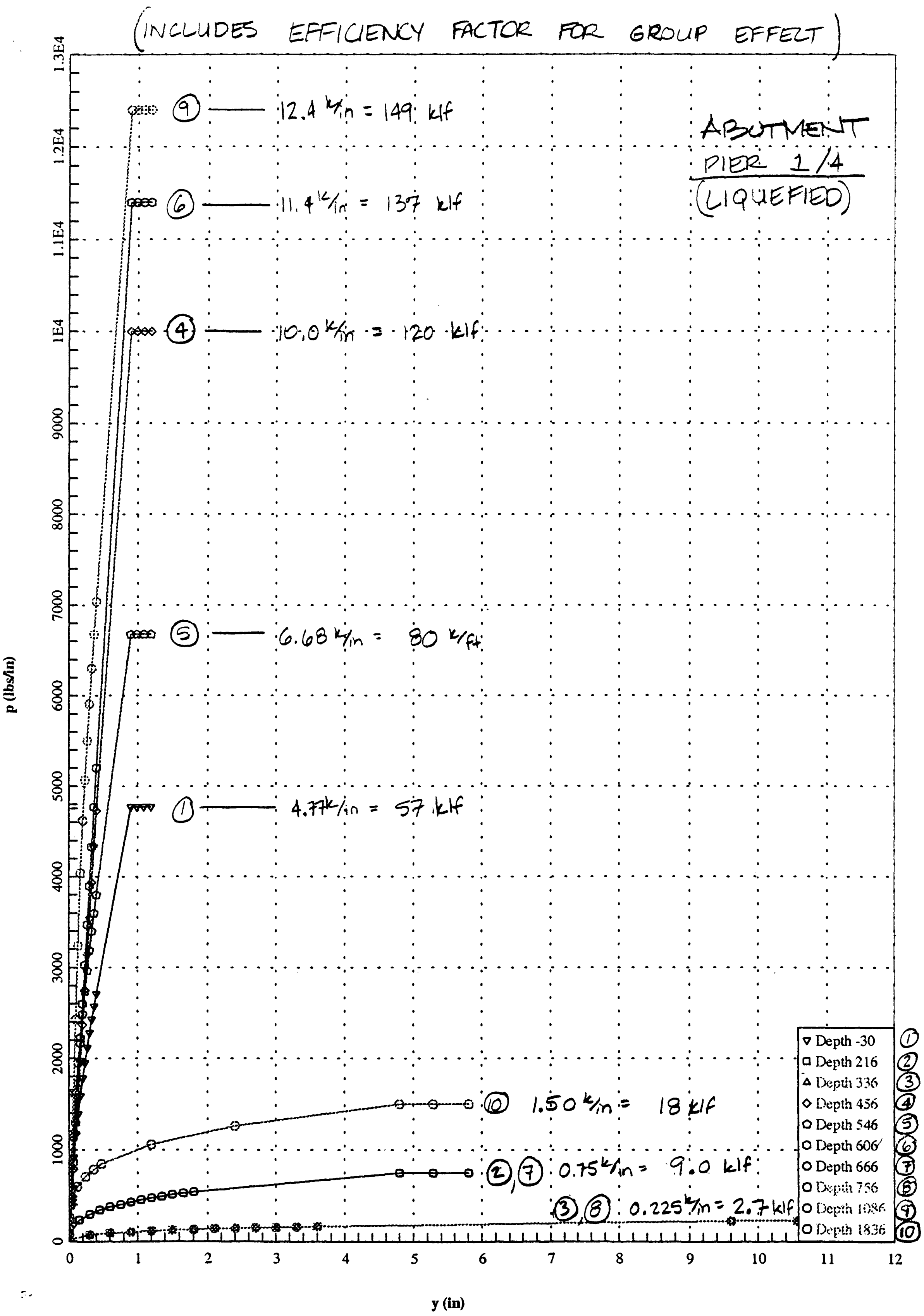
⑦ SOFT CLAY γ = 60 pcf k = 10 pci C_u = 1000 psf ε_{50} = 0.01

⑧ SOFT CLAY γ = 60 pcf k = 3 pci
C_u = 300 psf ε_{50} = 0.020

⑨ SAND γ = 60 pcf
k = 50 pci → 25 pci (GE)
Ø = 26° → 21° (GE)

⑩ STIFF CLAY W/O FREE WATER γ = 60 pcf
k = 200 pci → 100 pci (GE)
C_u = 2000 psf = 13.89 psi → 6.95 psi (G
ε_{50} = 0.005

~~STIFF CLAY W/O FREE WATER γ = 60 pcf~~
~~k = 200 pci~~
~~C_u = 2500 psf = 17.36 pci~~
~~ε_{50} = 0.005~~

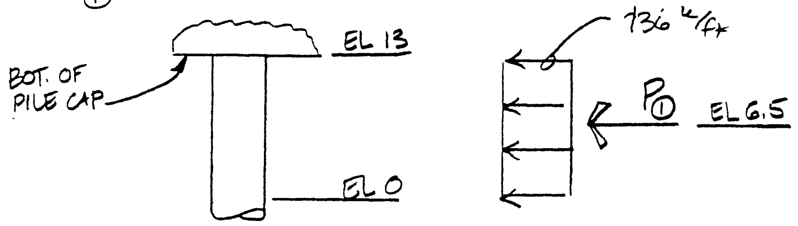

(INCLUDES EFFICIENCY FACTOR FOR GROUP EFFECT)
ABUTMENT
PIER 1/4
(LIQUEFIED)
⑨ —— 12.4 k/in = 149 klf
⑥ —— 11.4 k/in = 137 klf
④ —— 10.0 k/in = 120 klf
⑤ —— 6.68 k/in = 80 k/ft
① —— 4.77k/in = 57 klf
⑩ 1.50 k/in = 18 klf
②,⑦ 0.75k/in = 9.0 klf
③,⑧ 0.225k/in = 2.7 klf
▽ Depth -30 ①
□ Depth 216 ②
△ Depth 336 ③
◇ Depth 456 ④
○ Depth 546 ⑤
○ Depth 606 ⑥
○ Depth 666 ⑦
○ Depth 756 ⑧
○ Depth 1086 ⑨
○ Depth 1836 ⑩
p (lbs/in)
0
1000
2000
3000
4000
5000
6000
7000
8000
9000
1E4
1.1E4
1.2E4
1.3E4
y (in)
0
1
2
3
4
5
6
7
8
9
10
11
12

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject FLOW FORCES
ON ABUTMENT PILES
Sheet ____ of ____
Job Number A99067 - 400
Designer MLT
Date 24 AUG 2000

PILES - LAYER ①

2 x 6 PILE LAYOUT - 24"∅ PILES

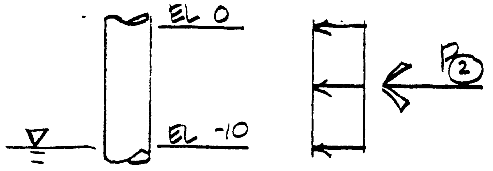
$P_{①}$ ON 12 PILES



$P_{①} = (.57 \frac{k}{ft})(13')(12 \text{ piles})$ ⇒ $\boxed{P_{①} = 8892^{k}}$

$P_{①} = [\underbrace{(0.11 \text{ kcf})(31'-6.5')(4)}_{10.78 \text{ ksf}}](13')(12 \text{ piles})(2')$ ⇒ $P_{①} = 3363^{k}$
@ EL 6.5'

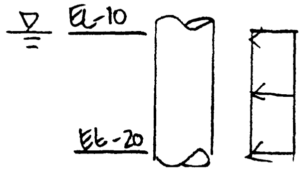
PILES - LAYER ②



$P_{②} = (9 \text{ k/ft})(10')(12 \text{ piles})$ ⇒ $\boxed{P_{②} = 1080^{k}}$
@ EL -5'

PILES - LAYER ③

$P_{③} = (2.7 \text{ k/ft})(10')(12 \text{ piles})$ → $\boxed{P_{③} = 324^{k}}$
@ EL -15
@ EL -15'

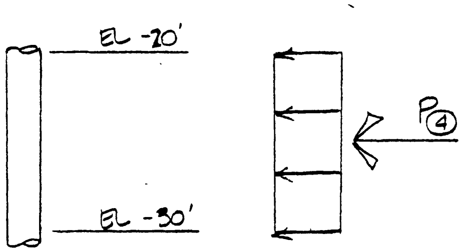


Project NCHRP 12-49
LIQUEFACTION STUDY
Subject FLOW FORCES
ON ABUTMENT PILES

Sheet ______ of ______
Job Number A99067-400
Designer MLT
Date 24 AUG 2000

PILES - LAYER ④

(12 PILES)



$P_{④} = (120\ ^k/_{ft})(10')(12)$

$\boxed{P_{④} = 14400^k}$

@ EL -25'

PILES - LAYER ⑤

EL -30' to -35'

$P_{⑤} = (80\ ^k/_{ft})(5')(12)$

$\boxed{P_{⑤} = 4800^k}$

@ EL -32.5'

PILES - LAYER ⑥

EL -35' to -40'

$P_{⑥} = (137\ k/ft)(5')(12)$

$\boxed{P_{⑥} = 8220^k}$

@ EL -37.5'

PILES - LAYER ⑦

EL -40 to -45

$P_{⑦} = (9\ ^k/_{ft})(5')(12)$

$\boxed{P_{⑦} = 540^k}$

@ EL -42.5'

PILES - LAYER ⑧

EL -45 to -55

$P_{⑧} = (2.7\ ^k/_{ft})(10')(12)$

$\boxed{P_{⑧} = 324^k}$

@ EL-50'

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject FLOW FORCES
ON ABUTMENT PILES

Sheet ______ of ______
Job Number A99067
Designer MLT
Date 25 AUG 2000

CASE

Ⓐ ASSUME LOWEST LIQUEFIED LAYER IS LAYER ③,

THEN PILE IS FIXED 0.78T BELOW EL -20

OR [-20' - 5.5' =] EL -25.5'

ΣM @ EL -25.5 = 0

$1258^k\,(21.5-(-25.5)) = 1258\,(47') = 59126'^k$

$1569^k\,(15.5+25.5) = 1569\,(41') = 64329'^k$

$8892^k\,(6.5+25.5) = 8892\,(32') = 284544'^k$

$1080^k\,(-5+25.5) = 1080\,(20.5) = 22140'^k$

$324^k\,(-15+25.5) = 324\,(10.5) = 3402'^k$

~~$14400^k\,(-25+25.5) = 14400\,(0.5) = 7200'^k$~~

$V =$ ~~27523^k~~ 13123^k

$M =$ ~~$440741'^k$~~ $433{,}541'^k$

$$\frac{V}{12 \text{ piles}} = \frac{13123}{12} = 1094^k \text{ per pile}$$

$$\frac{M}{12 \text{ piles}} = \frac{433541}{12} = 36128'^k \text{ per pile}$$

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject FLOW FORCES
ON ABUTMENT PILES
Sheet ____ of ____
Job Number A99067
Designer MLT
Date 25 AUG 2000

CASE Ⓑ ASSUME LOWEST LIQUEFIED LAYER IS ⑧,

$\therefore$ PILE IS FIXED AT $(-55' - 5.5' =)$ EL $-60.5'$

ΣM @ EL $-60.5' = 0$

$1258^k\ (21.5' - (-60.5')) = 1258^k\ (82') = 103156'^k$

$1569^k\ (15.5 + 60.5) = 1569\ (76') = 119244'^k$

$8892^k\ (6.5 + 60.5) = 8892\ (67') = 595764'^k$

$1080^k\ (-5 + 60.5) = 1080\ (55.5') = 59940$

$324^k\ (-15 + 60.5) = 324\ (45.5) = 14742$

$14400^k\ (-25 + 60.5) = 14400\ (35.5) = 511200$

$4800^k\ (-32.5 + 60.5) = 4800\ (28') = 134400$

$8220^k\ (-37.5 + 60.5) = 8220\ (23') = 189060$

$540^k\ (-42.5 + 60.5) = 540\ (18') = 9720$

$\underline{324^k}\ (-50 + 60.5) = 324\ (15.5) = \underline{5022}$

$V = 41407^k$ $M = 1742248'^k$

$$\frac{V}{12\ \text{piles}} = \frac{41407}{12} = 3451^k \text{ per pile}$$

$$\frac{M}{12} = \frac{1742248}{12} = 145187'^k \text{ per pile}$$

CHECK MOMENT APPLIED w/ PLASTIC HINGING MOMENT OF PILE. $\Longrightarrow \underline{\underline{M_{APPLIED} >> M_p}}$

(SEE PCACOL OUTPUT)

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject FLOW FORCES
ON ABUTMENT PILES

Sheet ______ of ______
Job Number A99067
Designer MLT
Date 25 AUG 2000

(UPPER & LOWER CASES)

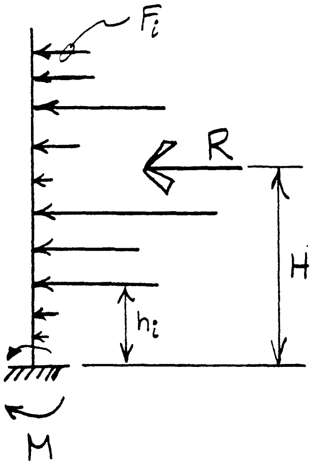
CASE B (LAYER 8 LIQUEFIES)

- DETERMINE RESULTANT FORCE ON PILE/ABUTMENT FRAME:

$$\Sigma M = 0 \Rightarrow M = 145187^{\prime k}/\text{PILE}$$

$$\Sigma F = V = 2371^{k}/\text{PILE} = R$$

$$M = \Sigma(F_i h_i) = RH \Rightarrow H = \frac{M}{R} = \frac{M}{V}$$



$$\therefore H = \frac{145187^{\prime k}}{3451^{k}} \Rightarrow \underline{\underline{H = 42.1'}}$$

$$\therefore EL = 42.1' + (-60.5')$$

$$\underline{\underline{EL = -18.4'}}$$

CASE A (LAYER 3 LIQUEFIES)

$$\Sigma M = 0 \Rightarrow M = 36728^{\prime k}/\text{PILE}$$

$$\Sigma F = V = 2294^{k}/\text{PILE} = R$$

$$M = RH \Rightarrow H = \frac{M}{R} = \frac{36728}{1094} \Rightarrow \underline{\underline{H = 33.0'}}$$

$$\therefore EL = 33' + (-25.5')$$

$$\underline{\underline{EL = +7.5'}}$$

E.9 Pinning Calculations

Project NCHRP 12-49
Sheet ______ of ______
Job Number ______
Subject WA Pinning Calcs Summary
Designer LM
Date 10/10/00

CASE 1 - Pier 5 & 6 Resist Movement

Pier 5 - 91k/pile (16 piles) = 1440k

- 210k/col (2 cols) = 420k

Pier 6 - 91k/pile (12 piles) = 1080k

- Abutment, 1/2 Prescriptive = 400k

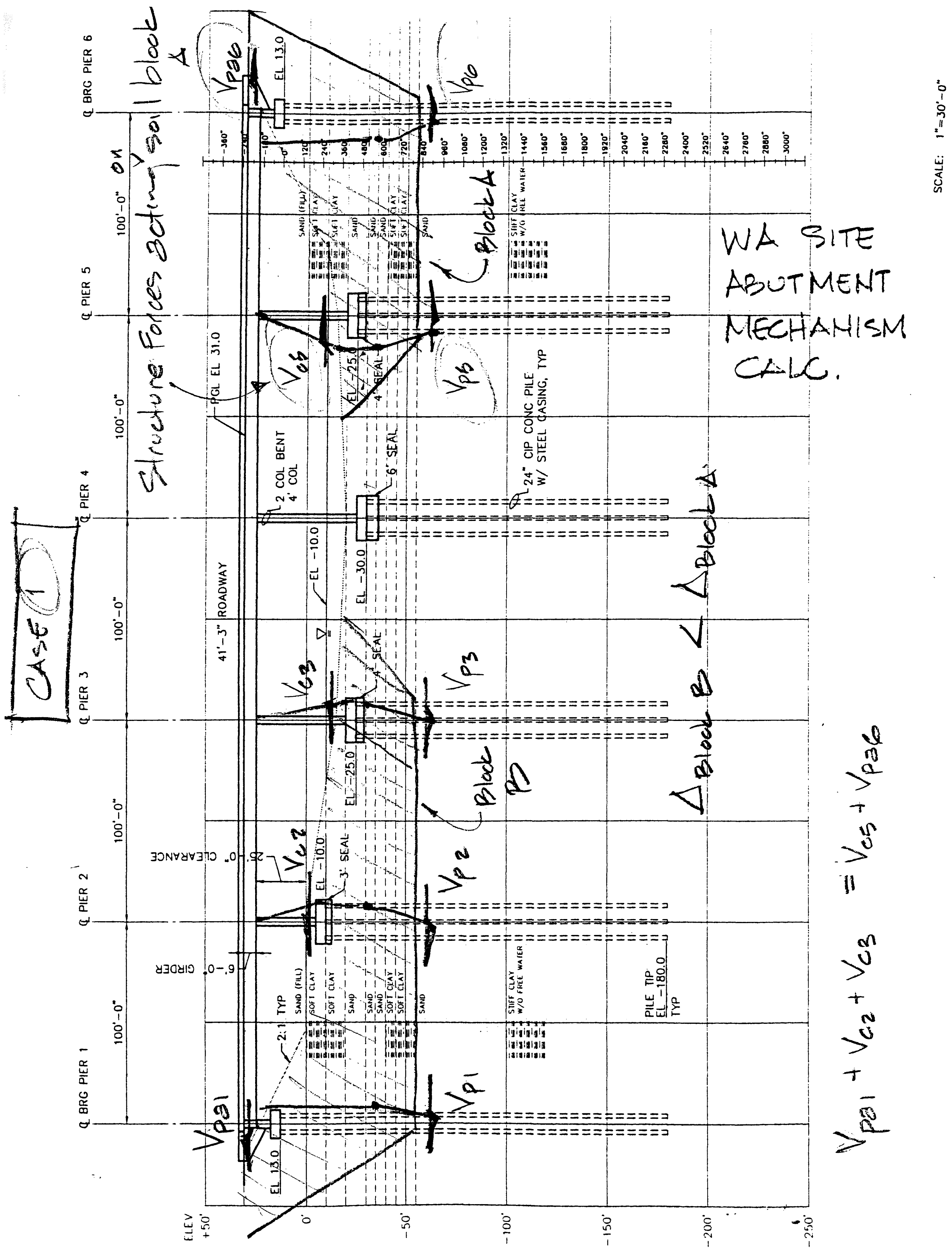
total - 3340k

CASE 2 - Pier 6 Resists Soil Movement

Pier 6 - 91k/pile (12 piles) = 1080k

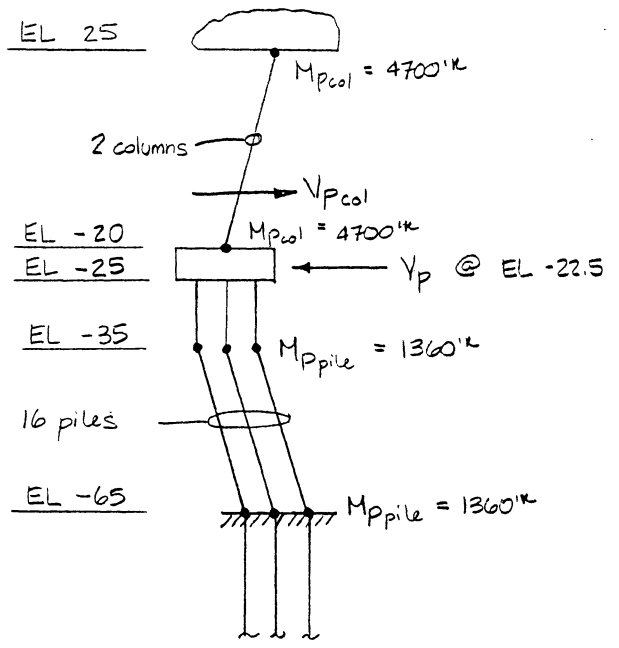
- Abutment, 1/2 prescriptive = 400k

total - 1480k


WA SITE
ABUTMENT
MECHANISM
CALC.
Case 1
Structure Forces acting on soil block A
Block A
Block B
Block B < Block A
$V_{pa1} + V_{c2} + V_{c3} = V_{c5} + V_{pa6}$
V_{pa6}
V_{p6}
V_{c5}
V_{p5}
V_{c3}
V_{p3}
V_{c2}
V_{p2}
V_{pa1}
V_{p1}
℄ BRG PIER 6
℄ PIER 5
℄ PIER 4
℄ PIER 3
℄ PIER 2
℄ BRG PIER 1
100'-0"
100'-0"
100'-0"
100'-0"
100'-0"
41'-3" ROADWAY
PGL EL 31.0
2 COL BENT
4' COL
6' SEAL
4' SEAL
4' SEAL
3' SEAL
EL 13.0
EL 13.0
EL -10.0
EL -10.0
EL -25.0
EL -25.0
EL -30.0
25'-0" CLEARANCE
6'-0" GIRDER
2:1 TYP
24" CIP CONC PILE
W/ STEEL CASING, TYP
PILE TIP
EL -180.0
TYP
SAND (FILL)
SOFT CLAY
SOFT CLAY
SAND
SAND
SAND
SOFT CLAY
SOFT CLAY
SAND
STIFF CLAY
W/O FREE WATER
-360
-240
-180
0
120
240
360
480
600
720
840
960
1080
1200
1320
1440
1560
1680
1800
1920
2040
2160
2280
2400
2520
2640
2760
2880
3000
ELEV
+50'
0'
-50'
-100'
-150'
-200'
-250'
SCALE: 1"=30'-0"

Project NCHRP 12-49
LIQUEFACTION STUDY
Subject
Sheet ____ of ____
Job Number A99067 - 400
Designer MLT
Date 9 OCT 2000

CASE 1 - PIER 5 DEEP LIQUEFACTION



TRANSVERSE ELEV

FBD OF COL (2 cols.)

$$(2)\left[(45')V_{Pcol}\right] - 4M_{Pcol} = 0$$

$$\Rightarrow \boxed{V_{Pcol} = \frac{M_{Pcol}}{22.5'}}$$

For M_{Pcol} = 4700'k,

V_{Pcol} = 209 k
per column

FBD OF PILE (1 pile)

$$(30')V_{Ppile} - 2M_{Ppile} = 0$$

$$\boxed{V_{Ppile} = \frac{M_{Ppile}}{15'}}$$

For M_{Ppile} = 1360'k,

V_{Ppile} = 91 k
per pile

$$\Delta_{ycol} = \frac{M_pH^2}{6EI_{cr}} = \frac{(4700'^k)(45')^2}{6(552000)(15ft^4)}(12'') = 6.9''$$

FBD OF PILE CAP

$$\Sigma F_H = 0$$

$$16V_{Ppile} + 2V_{Pcol} - V_p = 0$$

$$V_p = \underset{1456^k}{16(91^k)} + \underset{418^k}{2(209^k)} \Rightarrow V_p = 1874^k \text{ / Pier 5}$$

$$\underset{(pile)}{\Delta_y} = \frac{M_pH^2}{6EI_{cr}} = \frac{(1360'^k)(30')^2}{6(552000\ ksf)(1.12ft^4)} = 0.33' = 4'' = \Delta_y$$

$$I_{cr} = I_{st} + 0.4I_c$$
$$= 17702\ in^4 + (0.4)(13737\ in^4) = 23197$$
transformed

HM 10/13/00

Case ① Pier 5 Permissible Deflections

$\Delta_{y\,col} \approx 6.9''$

$\Delta_{y\,piles} \approx 4''$ (same as for abutment piles)

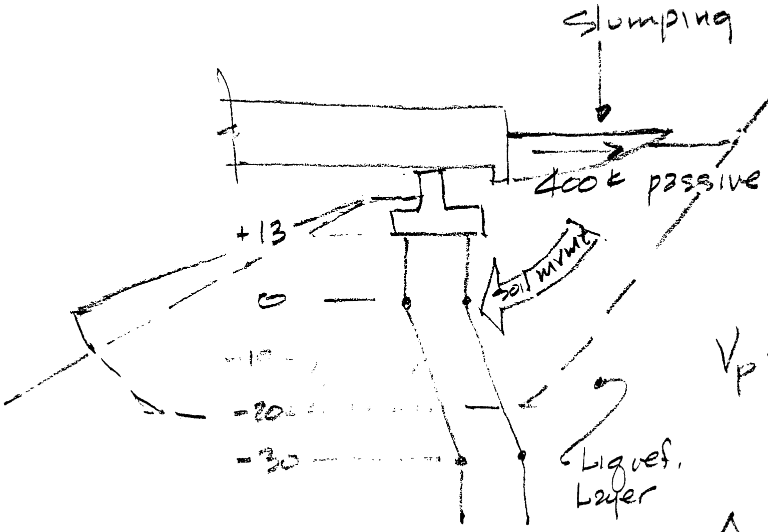
$\Delta_{p\,col} \approx 0.05(45) = 2.25'$ $(27'')$

$\Delta_{p\,piles} \approx 0.05(30) = 1.5'$ $(18'')$

$\Delta_{t\,col} = \Delta_{y\,col} + \Delta_{p\,col} \sim 7'' + 27'' = 34''$

$\Delta_{t\,piles} = 4'' + 18'' = 22''$ ⟵ controls

Project NCHRP 12-49
Sheet ________ of ________
Job Number ________
Subject WA Abutment Shallow Liquef. Case (2)
Designer LM
Date 10/10/00



$$V_p = \frac{2(1360^{k'})}{30} = \boxed{91^k} \text{ per pile}$$

$$\Delta_y \sim \frac{1360(30)^2}{6(552{,}000)(1.12)}(12) = 4''$$

Passive force @ abut. $\boxed{400^k}$
(assume 1/2 due to slumping)

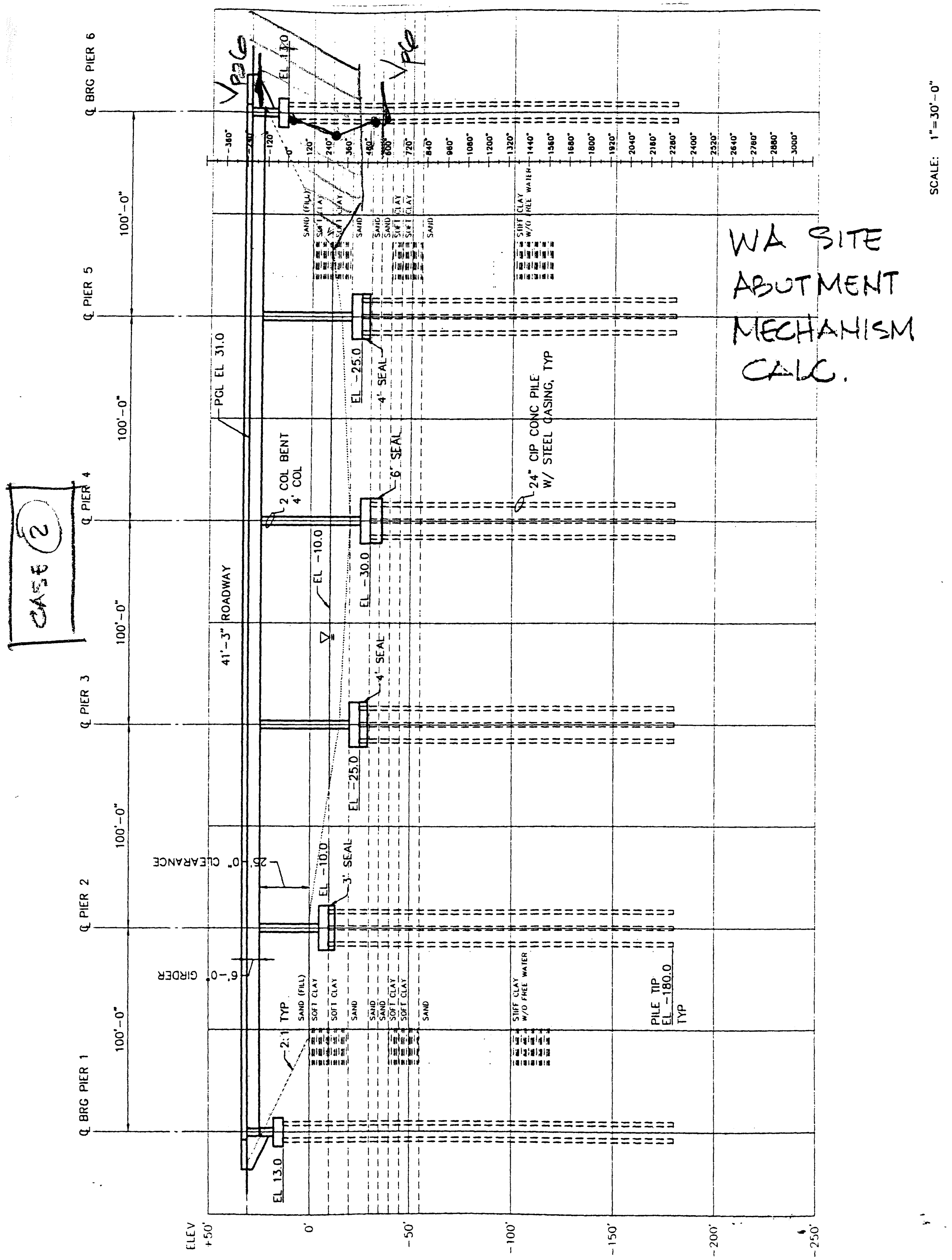
$$\frac{\text{Force}}{\text{width}} = \frac{91(12) + 400}{40'} = \boxed{31^{k}/\text{ft of width}}$$

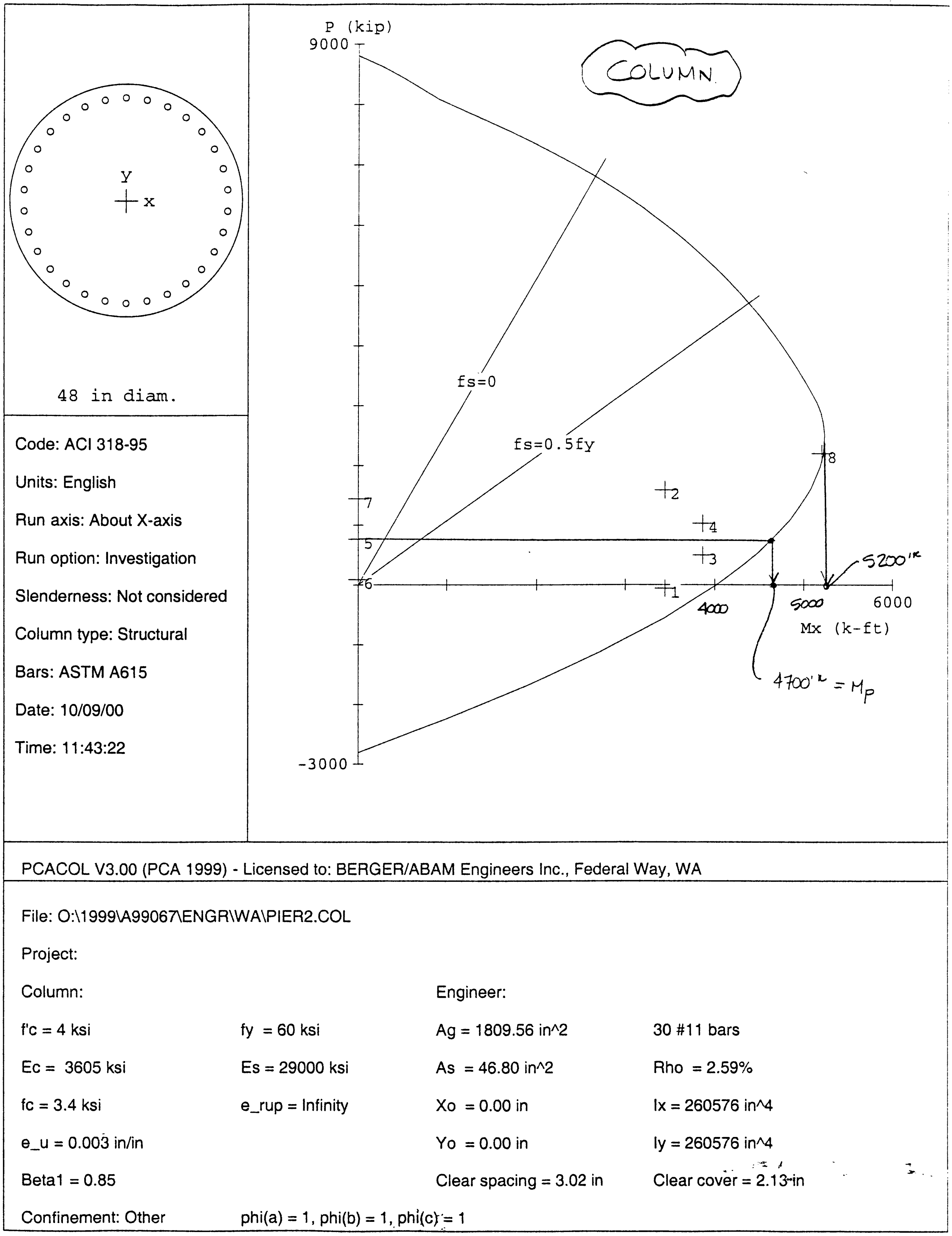
Permissible Deflections

$$\Delta_y \sim 4''$$

$$\Delta_p \sim \theta_p(H) = 0.05(30) = 1.5'$$

$$\Delta_{total} = 4'' + 18'' = \underline{\underline{22''}}$$


℄ BRG PIER 6
℄ PIER 5
℄ PIER 4
℄ PIER 3
℄ PIER 2
℄ BRG PIER 1
100'-0"
100'-0"
100'-0"
100'-0"
100'-0"
PGL EL 31.0
41'-3" ROADWAY
2 COL BENT
4' COL
6'-0" GIRDER
25'-0" CLEARANCE
2:1 TYP
EL 13.0
EL 13.0
EL -10.0
3' SEAL
EL -25.0
4' SEAL
EL -10.0
EL -30.0
6' SEAL
EL -25.0
4' SEAL
24" CIP CONC PILE
W/ STEEL CASING, TYP
PILE TIP
EL -180.0
TYP
SAND (FILL)
SOFT CLAY
SOFT CLAY
SAND
SAND
SAND
SOFT CLAY
SOFT CLAY
SAND
STIFF CLAY
W/O FREE WATER
ELEV
+50'
0'
-50'
-100'
-150'
-200'
-250'
-360"
-240"
-120"
0"
120"
240"
360"
480"
600"
720"
840"
960"
1080"
1200"
1320"
1440"
1560"
1680"
1800"
1920"
2040"
2160"
2280"
2400"
2520"
2640"
2760"
2880"
3000"
SCALE: 1"=30'-0"
V_{P3G}
V_{PG}
CASE ②
WA SITE
ABUTMENT
MECHANISM
CALC.



48 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/09/00

Time: 11:43:22

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\PIER2.COL

Project:

Column:

Engineer:

f'c = 4 ksi

fy = 60 ksi

Ag = 1809.56 in^2

30 #11 bars

Ec = 3605 ksi

Es = 29000 ksi

As = 46.80 in^2

Rho = 2.59%

fc = 3.4 ksi

e_rup = Infinity

Xo = 0.00 in

Ix = 260576 in^4

e_u = 0.003 in/in

Yo = 0.00 in

Iy = 260576 in^4

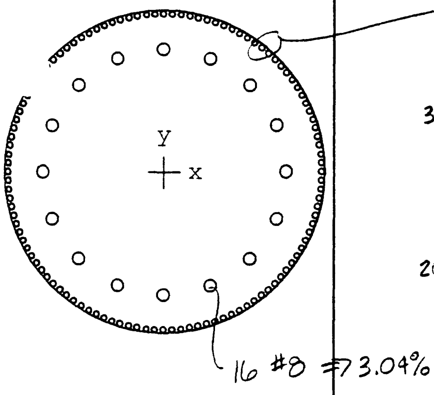
Beta1 = 0.85

Clear spacing = 3.02 in

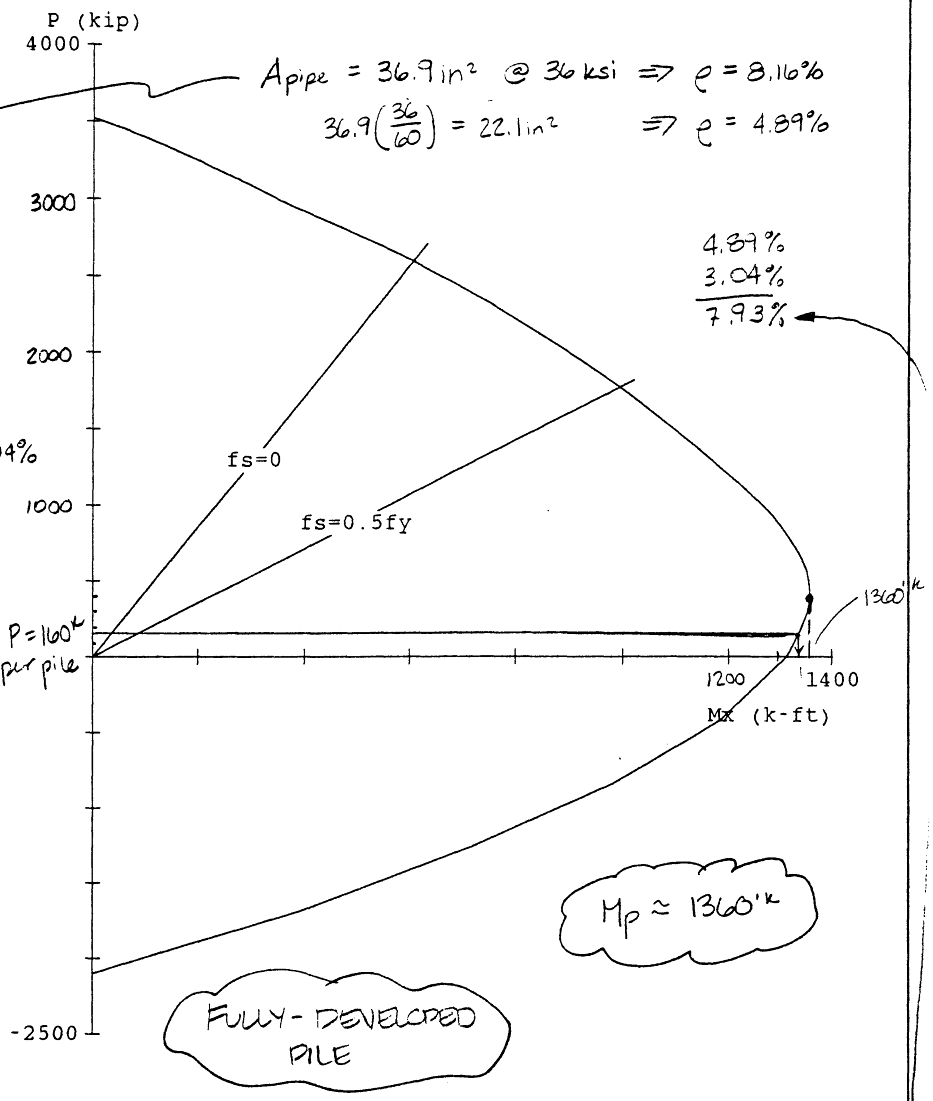
Clear cover = 2.13 in

Confinement: Other

phi(a) = 1, phi(b) = 1, phi(c) = 1



24 in diam.



Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

umn type: Structural

Bars: ASTM A615

Date: 08/25/00

Time: 15:05:33

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WASHIN~1\LPILE\PILEPIPE.COL

Project:

Column: Engineer:

f'c = 4 ksi   fy = 60 ksi   Ag = 452.389 in^2   128 bars

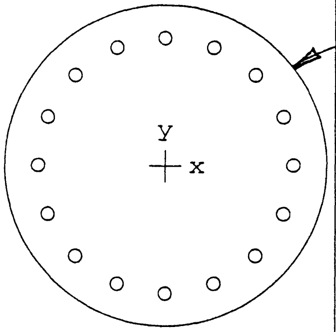
Ec = 3605 ksi   Es = 29000 ksi   As = 35.04 in^2   Rho = 7.75% ✓ OK CLOSE ENOUGH

fc = 3.4 ksi   e_rup = Infinity   Xo = 0.00 in   Ix = 16286 in^4

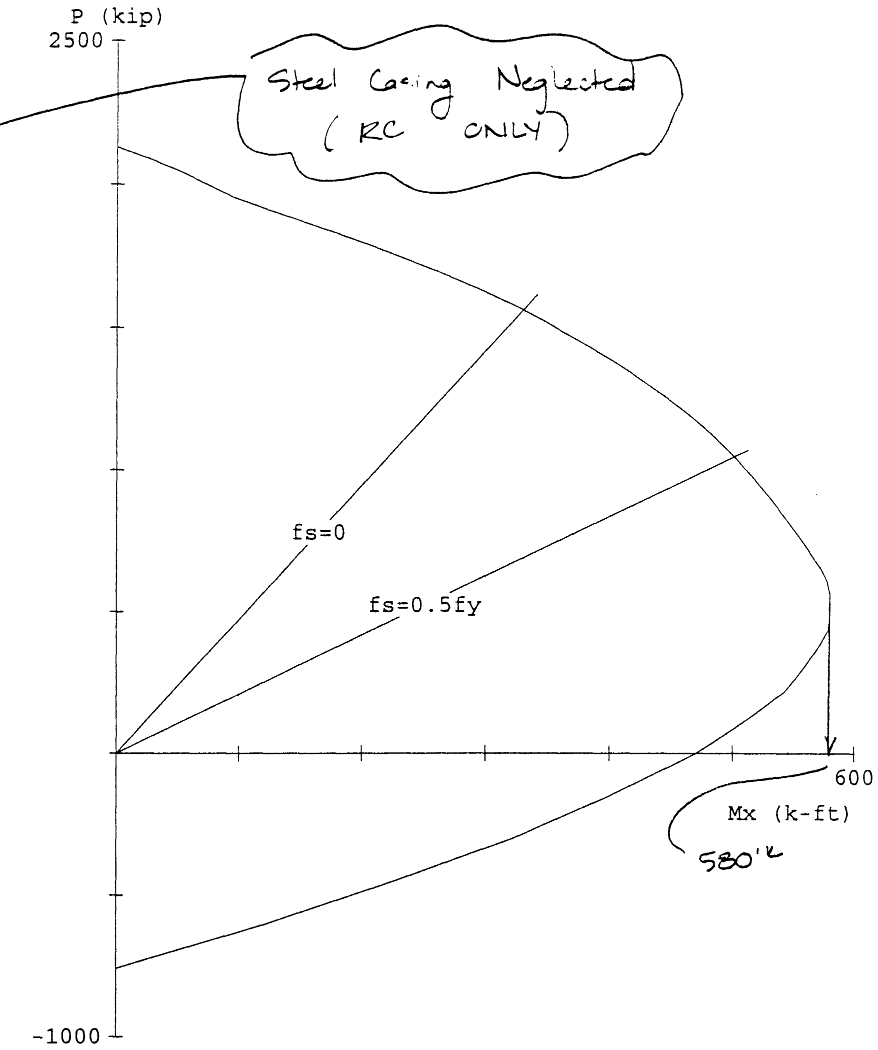
_u = 0.003 in/in   Yo = 0.00 in   Iy = 16286 in^4

Beta1 = 0.85   Clear spacing = 0.15 in   Clear cover = -0.25 in

Confinement: Other   phi(a) = 1, phi(b) = 1, phi(c) = 1



23 in diam.



Steel Casing Neglected
(RC ONLY)

580'k

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/09/00

Time: 12:34:36

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\WA\LPILE\OLD\PILE24IN.COL

Project:

Column:
Engineer:

f'c = 4 ksi
fy = 60 ksi
Ag = 415.476 in^2
16 #8 bars

Ec = 3605 ksi
Es = 29000 ksi
As = 12.64 in^2
Rho = 3.04%

fc = 3.4 ksi
e_rup = Infinity
Xo = 0.00 in
Ix = 13736.7 in^4

e_u = 0.003 in/in
Yo = 0.00 in
Iy = 13736.7 in^4

Beta1 = 0.85
Clear spacing = 2.56 in
Clear cover = 1.88 in

Confinement: Other
phi(a) = 1, phi(b) = 1, phi(c) = 1

E.10 Cost Analysis

PILE AND BRIDGE COST DATA

Washington Pile Cost

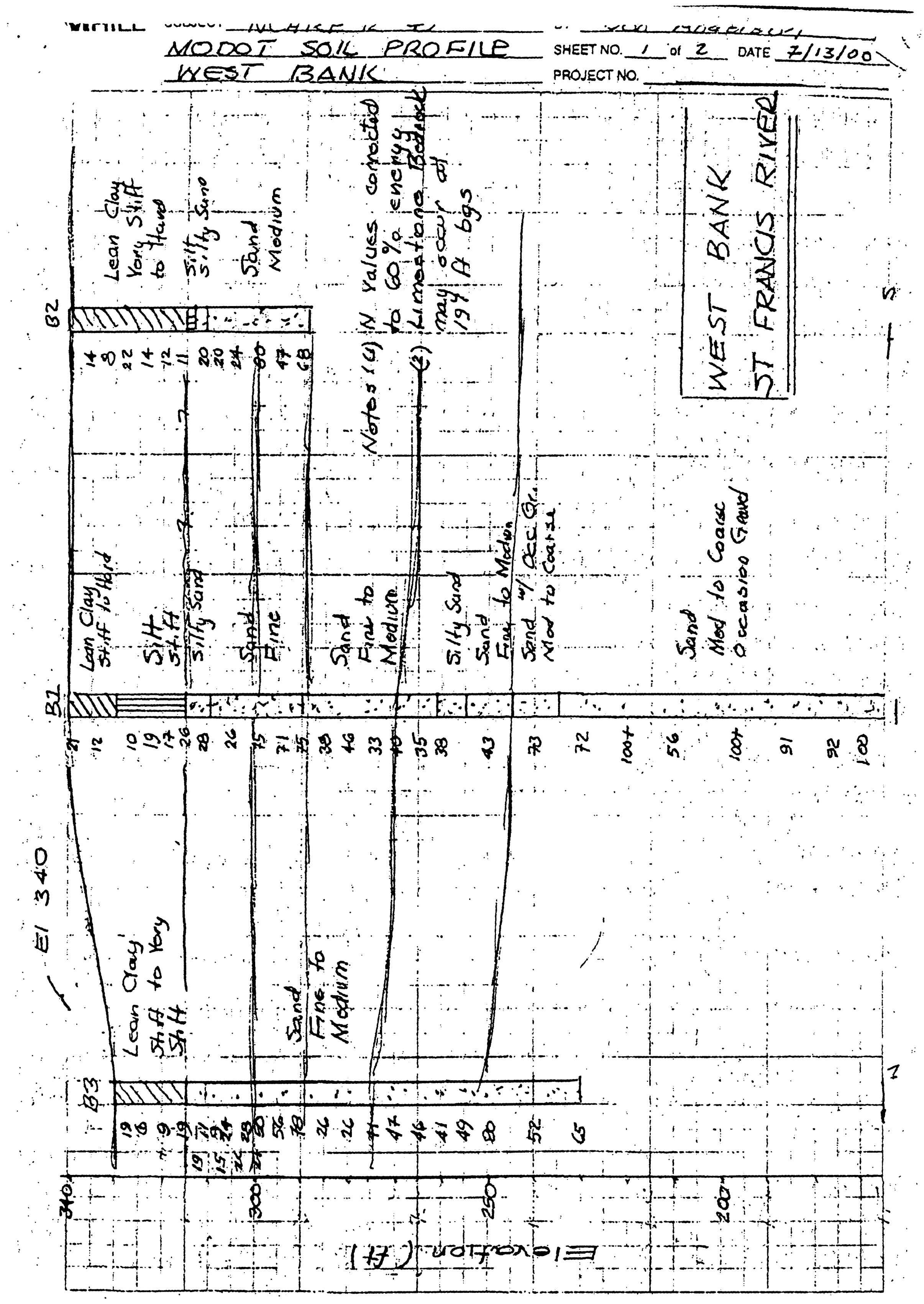
Bid Item	Unit Quantity	Length Along Pile	Total Quantity	Unit Cost	Cost per Pile
24"φ X-Strong Pipe ($t_{wall} = {}^1/_{2"}$)	126 lbs/ft of pipe	160 ft	20160 lbs	Steel Pipe Piles $ 0.35 / lbs	$ 7,056
Concrete In Place	0.11 cy./ft of pile	160 ft	17 cy	Concrete In Place $ 150 / yd	$ 2,562
Reinforcing Steel (8 #11 bars)	46.6 lbs/ft of pile	80 ft	3731 lbs	Reinforcing Steel $ 0.50 / lbs	$ 1,866
Pipe Pile Splice			1 splice	Pile Splices $ 500 / splice	$ 500
Pile Driving			1 pile	Pile Driving $ 600 / pile	$ 600
					$ 12,600

Washington Bridge Cost

		Unit Cost	Cost
Overall Width =	43 ft		
x Total Length =	500 ft	$ 100 / sq ft	**$ 2.2 Million**
Bridge Deck Area =	21500 sq ft	$ 150 / sq ft	**$ 3.2 Million**

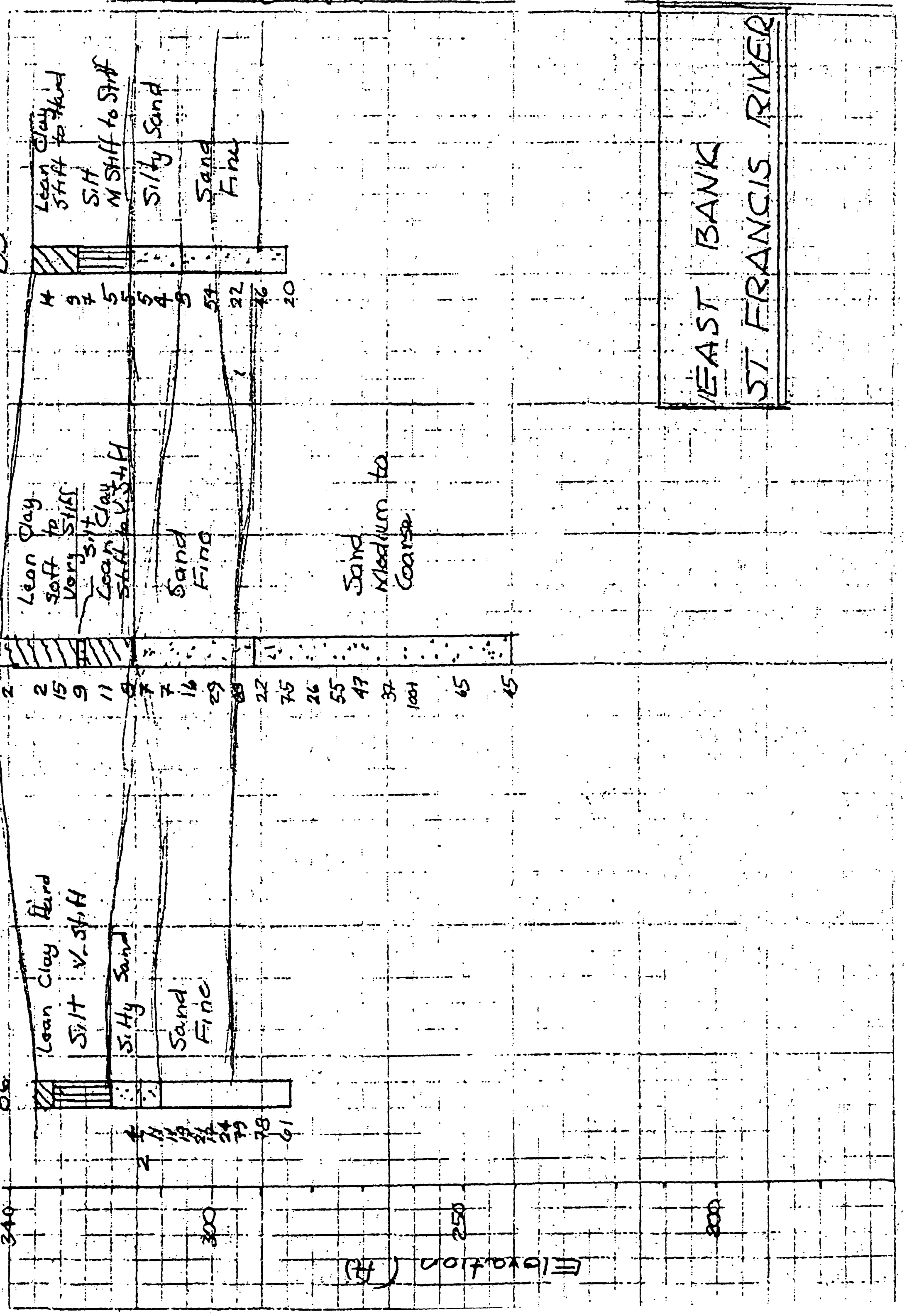
Appendix F
SIMPLIFIED SOIL ANALYSES – CENTRAL U.S. SITE

F.1 Missouri Site Information


MODOT SOIL PROFILE
WEST BANK
SHEET NO. 1 of 2
DATE 7/13/00
PROJECT NO.
WEST BANK
ST FRANCIS RIVER
B2
B1
B3
El 340
Elevation (ft)
340
300
250
200
Lean Clay Very Stiff to Hard
Silt
Silty Sand
Sand Medium
Notes (1) N Values corrected to 60% energy
(2) Limestone Bedrock may occur at 197 ft bgs
Lean Clay Stiff to Hard
Silt Stiff
Silty Sand
Sand Fine
Sand Fine to Medium
Silty Sand
Sand Fine to Medium
Sand w/ Occ Gr. Med to Coarse
Sand Med to Coarse Occasional Gravel
Lean Clay Stiff to Very Stiff
Sand Fine to Medium
14 8 22 14 12 11 20 20 24 30 47 68
21 12 10 19 24 26 28 26 75 71 75 38 46 33 40 35 38 43 73 72 100+ 56 100+ 91 92 100
19 8 9 19 12 24 28 80 56 78 26 26 41 47 46 41 49 80 52 65
19 15 24 24

SUBJECT NCHRP 12-49
MDOT SOIL PROFILE
EAST BANK

BY Von Anderson
SHEET NO. 2 of 2 DATE 7/13/00
PROJECT NO.

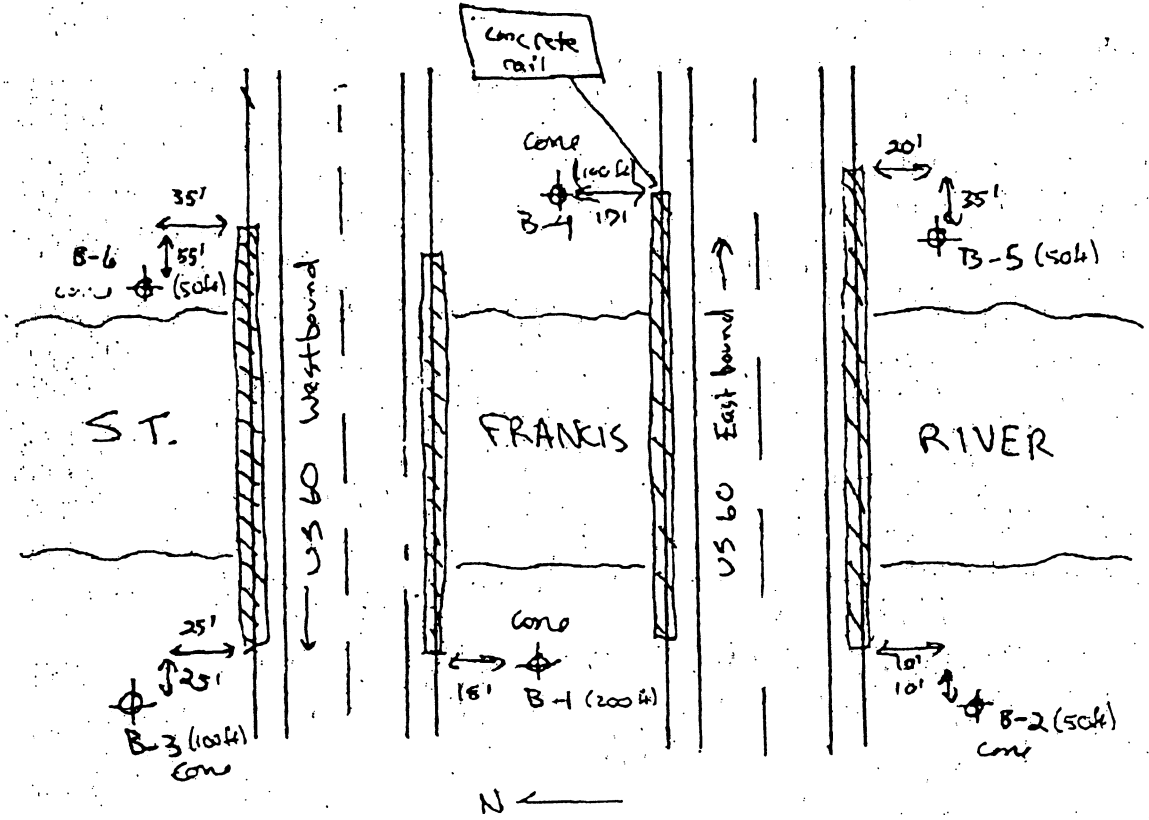


PMS
5/25/99

St. Francis River Boring & Test Pit Locations

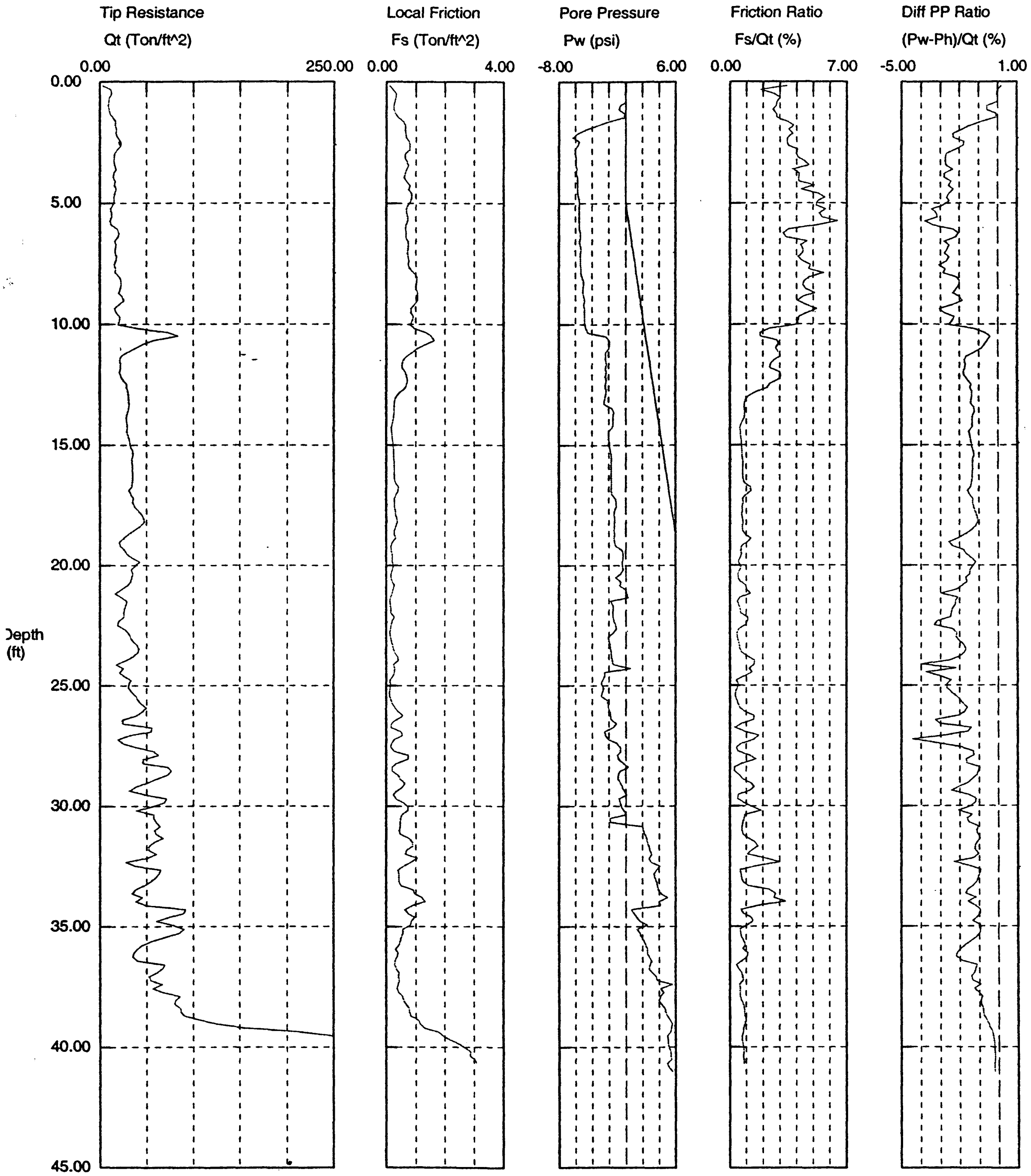
Note:

1. locations marked from end of concrete rail on bridges
2. boring, ~~and~~ test pits, & cone penetrometer holes will be near each other and in general vicinity of stake
3. boring locations are given a "B-1, B-2, etc." designation
4. ~~test pit locations are given a "TP-1, TP-2, etc." designation~~
5. each boring location is marked in the field with a wooden stake painted orange



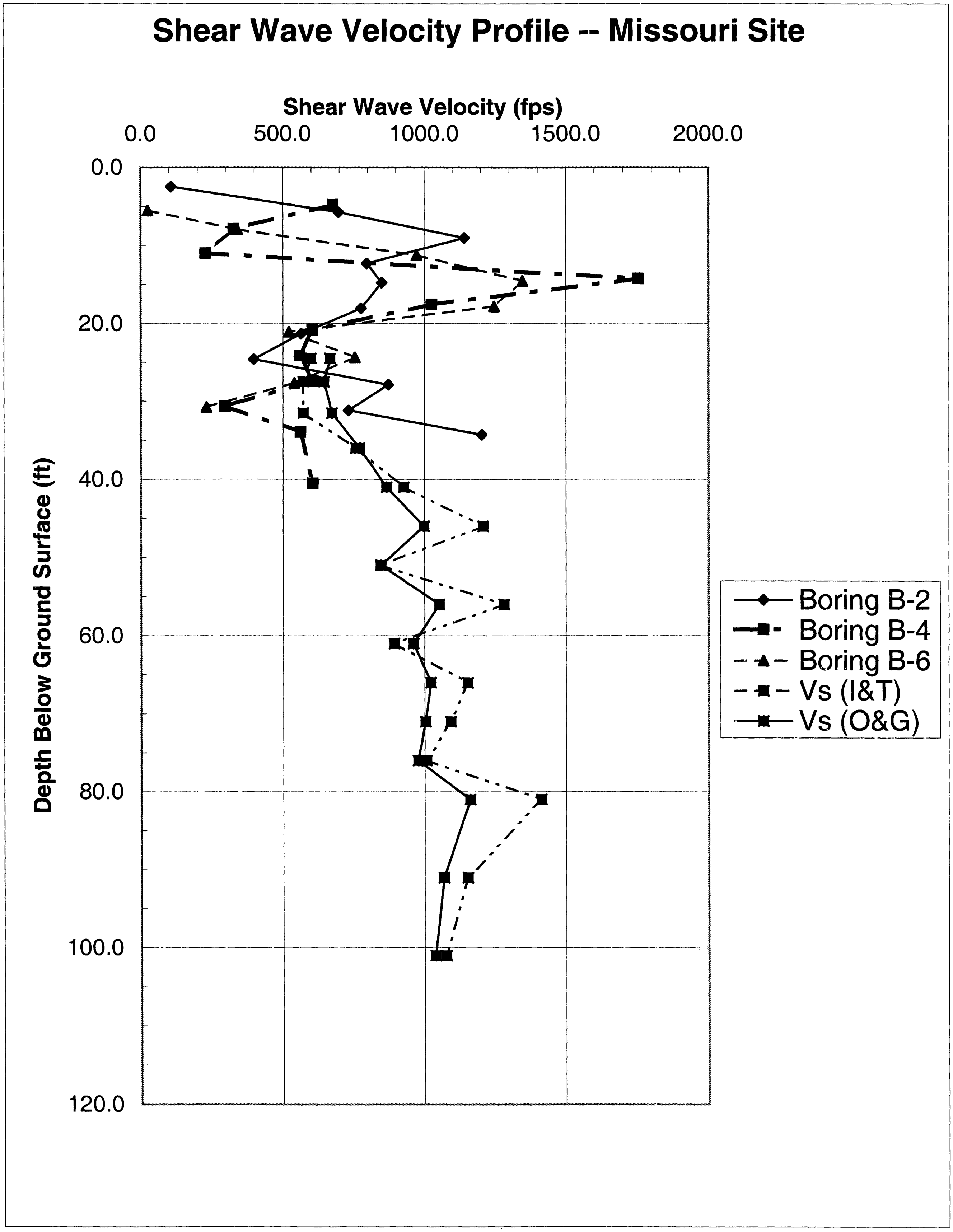
Operator: KEVIN
Sounding: r60b41
Cone Used: 680tc

CPT Date: 06-09-99 08:43
Location: route 60
Job No.: spr 98

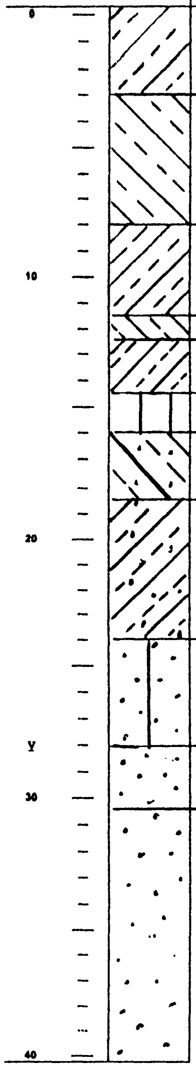


Maximum Depth = 41.01 feet

Depth Increment = 0.16 feet


Shear Wave Velocity Profile -- Missouri Site
Shear Wave Velocity (fps)
0.0
500.0
1000.0
1500.0
2000.0
Depth Below Ground Surface (ft)
0.0
20.0
40.0
60.0
80.0
100.0
120.0
Boring B-2
Boring B-4
Boring B-6
Vs (I&T)
Vs (O&G)

Sheet ___ of 1



DEPTH AND DESCRIPTION Elevation – 342.0'			P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γ_d pcf	Qu/2 psf	Particle Size Analysis <4.75mm %	<2.00mm %	<0.425mm %	<0.075mm %
0.0-3.0'	Brown lean clay, with sand and gravel, hard, dry	@ 1.3'	4.50	0.86				12.4						
	- SPT 2.5-4.0', N_{60} = 2													
3.0-8.0'	Brown mottled lean clay, soft to medium stiff, moist - trace gravel 3.0-6.0'													
		@ 5.3'	0.75	0.43				27.1						
	- SPT 6.5-8.0', N_{60} = 2													
		@ 7.3'						37.9						
8.0-11.5	Brown mottled lean clay, very stiff, moist													
		@ 9.3'	3.00	0.79	46	26	CL	23.6						
	- SPT 10.5-12.0', N_{60} = 16	@ 11.0'	2.75	—				23.5						
11.5-12.5'	Grayish brown lean clay, very stiff, moist													
12.5-14.5'	Light brown lean clay, very stiff, moist	@ 13.5'	2.50	0.59	36	15	CL	24.2						
14.5-16.0'	Light brown silt, stiff to very stiff, moist - SPT 14.5-16.0', N_{60} = 9	@ 15.3'	2.25	—	23	4	CL-ML	17.2						
16.0-18.5'	Light brown sandy lean clay, stiff, moist	@ 17.3'	1.30	0.22				19.6						
18.5-24.0'	Brown sandy lean clay, medium stiff to stiff, moist - SPT 18.5-20.0', N_{60} = 11	@ 19.3'	1.50	—	19	2	CL							
		@ 21.3'	1.00	0.60				12.6						
	- SPT 22.5-24.0', N_{60} = 6	@ 23.3'	0.50	—										
24.0-28.0'	Brown and gray silty fine sand, medium stiff, moist	@ 25.3'			NP	NP		23.0			100	100	100	36
	- SPT 26.5-28.0', N_{60} = 7	@ 27.3'						23.8						
28.0-30.5'	Brown and gray fine sand, loose, moist	@ 29.3'						26.3						
30.5-48.3'	Gray fine sand, loose to very dense, wet - SPT 30.5-32.0', N_{60} = 7	@ 31.3'									100	100	99	23
	- SPT 35.0-36.5', N_{60} = 16													

JOB NO.:	RI96-043
COUNTY:	Stoddard
ROUTE:	US 60
COORDINATE (83 DATUM):	907812.85, 348869.34
GENERAL LOCATION:	B-4, St. Francis River

Key:
∇ Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane"

FIGURE 4 of 6

Sheet _ of 3

	DEPTH AND DESCRIPTION Elevation – 342.0'		P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γ_d pcf	Qu/2 psf	Particle Size Analysis			
											<4.75mm %	<2.00mm %	<0.425mm %	<0.075mm %
40		Gray fine sand, loose to very dense, wet - SPT 40.0-41.5', N_{60} = 29												
		- SPT 45.0-46.5', N_{60} = 63												
	48.3-78.3'	Gray medium sand, medium dense to very dense, wet												
50		- SPT 50.0-51.5', N_{60} = 22												
		- SPT 55.0-56.5', N_{60} = 75												
60		- SPT 60.0-61.5', N_{60} = 26												
		- SPT 65.0-66.5', N_{60} = 66												
70		- SPT 70.0-71.5', N_{60} = 47												
		- SPT 75.0-76.5', N_{60} = 37												
	78.3-88.8'	Gray fine to medium sand, very dense, wet												
80														

JOB NO.:	RI98-043
COUNTY:	Stoddard
ROUTE:	US 60
COORDINATE (83 DATUM):	907812.83, 348569.34
GENERAL LOCATION:	B-4, St. Francis River

Key:

$\underline{\nabla}$	Water Table
P.P.	"Pocket Penetrometer"
Tv.	"Torvane"

FIGURE 4 of 6

Sheet 3 of 3

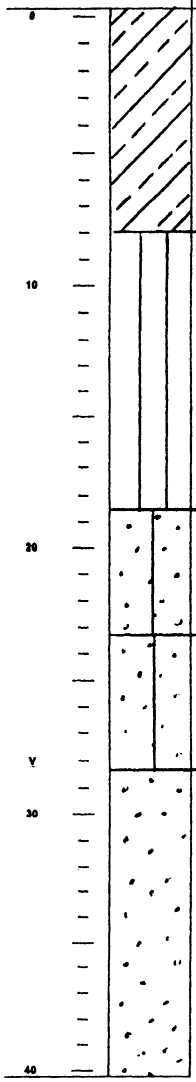
	DEPTH AND DESCRIPTION Elevation – 342.0'		P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γ_d pcf	Qu/2 psf	Particle Size Anal.			
											<4.75mm %	<2.00mm %	<0.425mm %	<0.075mm %
80		Gray fine to medium sand, very dense, wet - SPT 80.0-81.5', N_{60} = 100+ - gravel layer 82.2-83.2												
	85.8-101.5'	Gray medium to coarse sand, dense to very dense, wet												
90		- SPT 90.0-91.5', N_{60} = 55												
100		- SPT 100.0-101.5', N_{60} = 45												
		Termination of boring @ 101.5'												
110														
120														

JOB NO.: RI98-043
COUNTY: Stoddard
ROUTE: US 60
COORDINATE (83 DATUM): 907812.83, 348569.34
GENERAL LOCATION: B-4, St. Francis River

Key:		
	$\underline{\nabla}$	Water Table
	P.P.	"Pocket Penetrometer"
	Tv.	"Torvane"

FIGURE 4 of 6

Sheet _ _ of 2



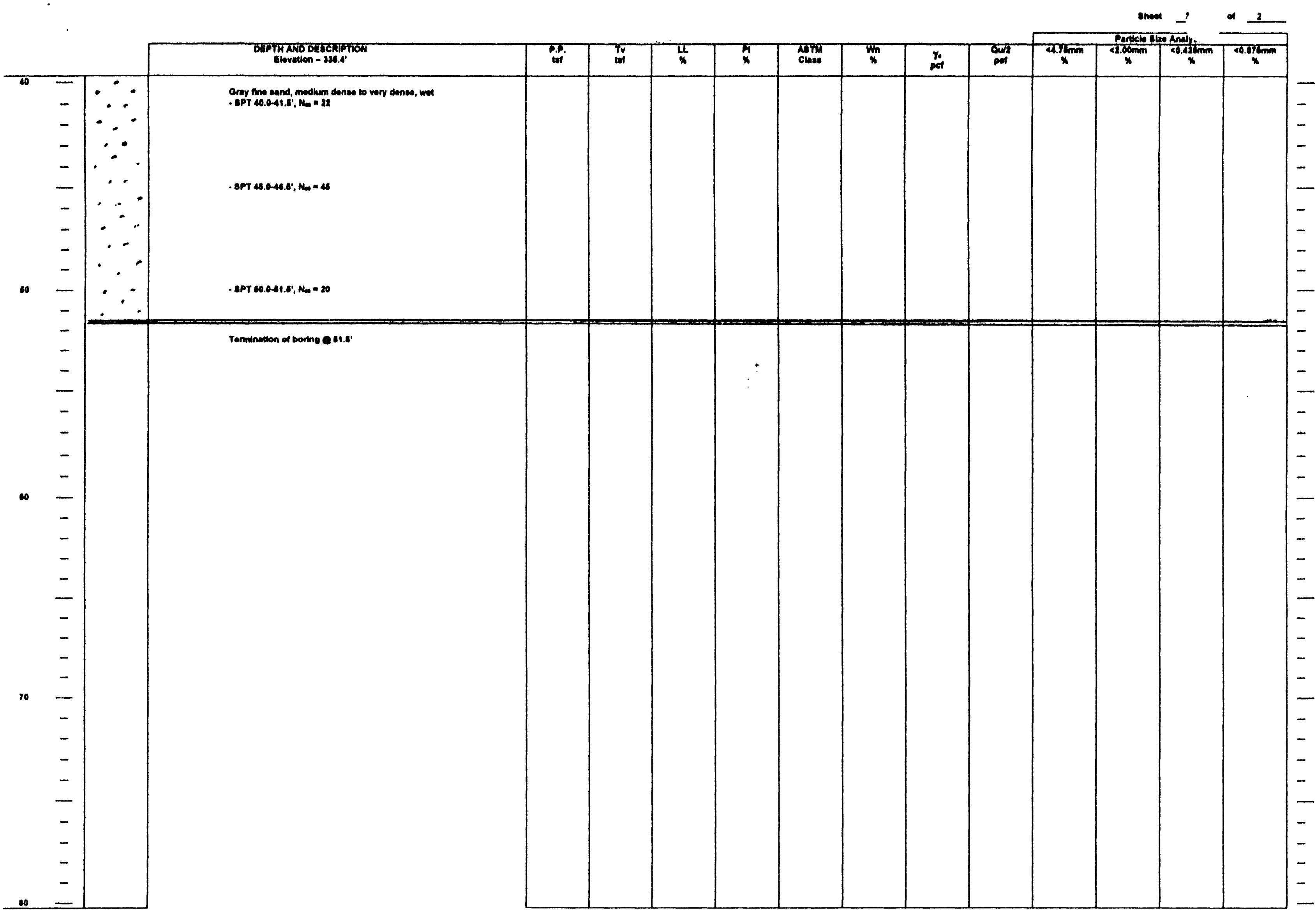
DEPTH AND DESCRIPTION Elevation – 335.4'			P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γd pcf	Qu/2 psf	Particle Size An. <4.75mm %	<2.00mm %	<0.425mm %	<0.075mm %
0.0-8.0'	Brown lean clay, stiff to hard, dry													
	- SPT 2.5-4.0', N_{60} = 14													
		@ 3.3'	1.50	—				13.4						
		@ 5.3'	4.50	0.75				19.2						
	- SPT 6.5-8.0', N_{60} = 9													
		@ 7.3'	1.00	—				20.6						
8.0-18.5'	Brown silt, medium stiff to stiff, moist													
		@ 9.3'	1.25	0.20	27	3	ML	21.2/23.2	94.6	3783	100	100	100	91
	- SPT 10.5-12.0', N_{60} = 7													
		@ 11.3'	0.75	—				18.9						
		@ 13.3'	0.50	0.40				18.8						
	- SPT 14.5-16.0', N_{60} = 5													
		@ 15.3'	0.50	—				19.1						
		@ 17.3'	0.75	0.20				18.3						
18.5-23.3'	Brown silty fine sand, loose, wet - SPT 18.5-20.0', N_{60} = 5 - SPT 20.0-21.5', N_{60} = 5	@ 19.3'			NP	NP					100	100	100	37
23.3-33.3'	Gray silty fine sand, loose, wet													
	- SPT 25.0-26.5', N_{60} = 4													
	- SPT 30.0-31.5'', N_{60} = 3													
		@ 30.8'									100	100	99	36
33.3-51.5'	Gray fine sand, medium dense to very dense, wet													
	- SPT 35.0-36.5', N_{60} = 64													

JOB NO.:	RI98-043
COUNTY:	Stoddard
ROUTE:	US 60
COORDINATE (83 DATUM):	907781.28, 348478.11
GENERAL LOCATION:	B-4, St. Francis River

Key:
▽ Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane"

FIGURE 5 of 6

Sheet 2 of 2



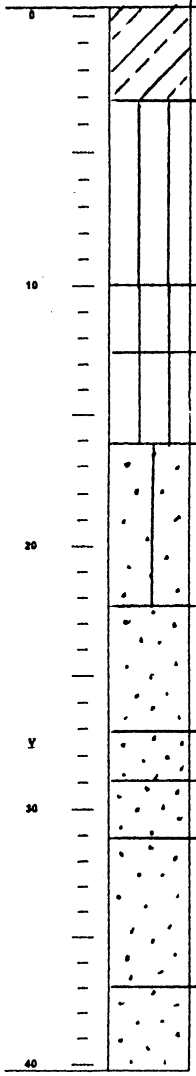
	DEPTH AND DESCRIPTION Elevation – 336.4'	P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γ_d pcf	Qu/2 psf	Particle Size Analy. <4.75mm %	<2.00mm %	<0.425mm %	<0.075mm %
40	Gray fine sand, medium dense to very dense, wet - SPT 40.0-41.5', N_{60} = 22												
	- SPT 45.0-46.5', N_{60} = 45												
50	- SPT 50.0-51.5', N_{60} = 20												
	Termination of boring @ 51.5'												
60													
70													
80													

JOB NO.: RI98-043
COUNTY: Stoddard
ROUTE: US 60
COORDINATE (88 DATUM): 907781.28, 348478.11
GENERAL LOCATION: B-5, St. Francis River

Key:
$\underline{\nabla}$ Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane"

FIGURE 5 of 6

Sheet of 2



DEPTH AND DESCRIPTION Elevation – 336.1			P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γ_d pcf	Qu/2 psf	Particle Size Analy <4.75mm %	<2.00mm %	<0.425mm %	<0.075mm %
0.0-3.1'	Brown lean clay, with sand, hard, dry	@ 1.2'	4.50+	0.95+				10.3						
	- gravel layer 2.8-3.1'													
3.1-10.0'	Brown silt, very stiff, moist													
		@ 6.3'	2.75	0.45				23.0						
		@ 8.8'			28	5	ML				97	96	96	86
10.0-12.5'	Brown silt, stiff, moist													
		@ 11.3'	1.50	0.75				22.4						
12.5-16.3'	Brown silt with sand, stiff, moist													
		@ 13.8'	1.25	0.45	24	2	ML	21.4/23.8	100.2	—	100	100	100	79
		@ 15.7'	1.00	0.38				22.2						
16.3-22.3'	Brown silty fine sand, trace lean clay, very loose to loose, moist	@ 16.9'						21.2						
		@ 18.8'			NP	NP	ML				100	100	100	60
	- SPT 20.0-21.5', N_{60} = 4													
		@ 20.8'												
	- SPT 21.5-23.0', N_{60} = 2													
		@ 21.9'												
22.3-27.1'	Grayish brown fine sand, loose to medium dense, wet - occasional silt and lean clay seams and partings 22.3-27.1'	@ 23.8'									100	100	100	28
	- SPT 23.0-24.5', N_{60} = 11 - SPT 24.5-26.0', N_{60} = 11													
	- SPT 26.0-27.5', N_{60} = 11													
27.1-29.0'	Gray fine sand, medium dense, wet - SPT 27.5-29.0', N_{60} = 19													
29.0-31.0'	Grayish brown fine to medium sand, medium dense, wet - SPT 29.0-30.5', N_{60} = 21 - SPT 30.5-32.0', N_{60} = 17													
31.0-37.0'	Gray fine sand, medium dense, wet													
	- SPT 35.0-36.5', N_{60} = 24													
37.0-48.3'	Gray fine sand, very dense, wet													

JOB NO.:	RI98-043
COUNTY:	Stoddard
ROUTE:	US 60
COORDINATE (88 DATUM):	907710.69, 348678.49
GENERAL LOCATION:	B-6, St. Francis River

Key:

Y	Water Table
P.P.	"Pocket Penetrometer"
Tv.	"Torvane"

FIGURE 6 of 6

Sheet _* of 2

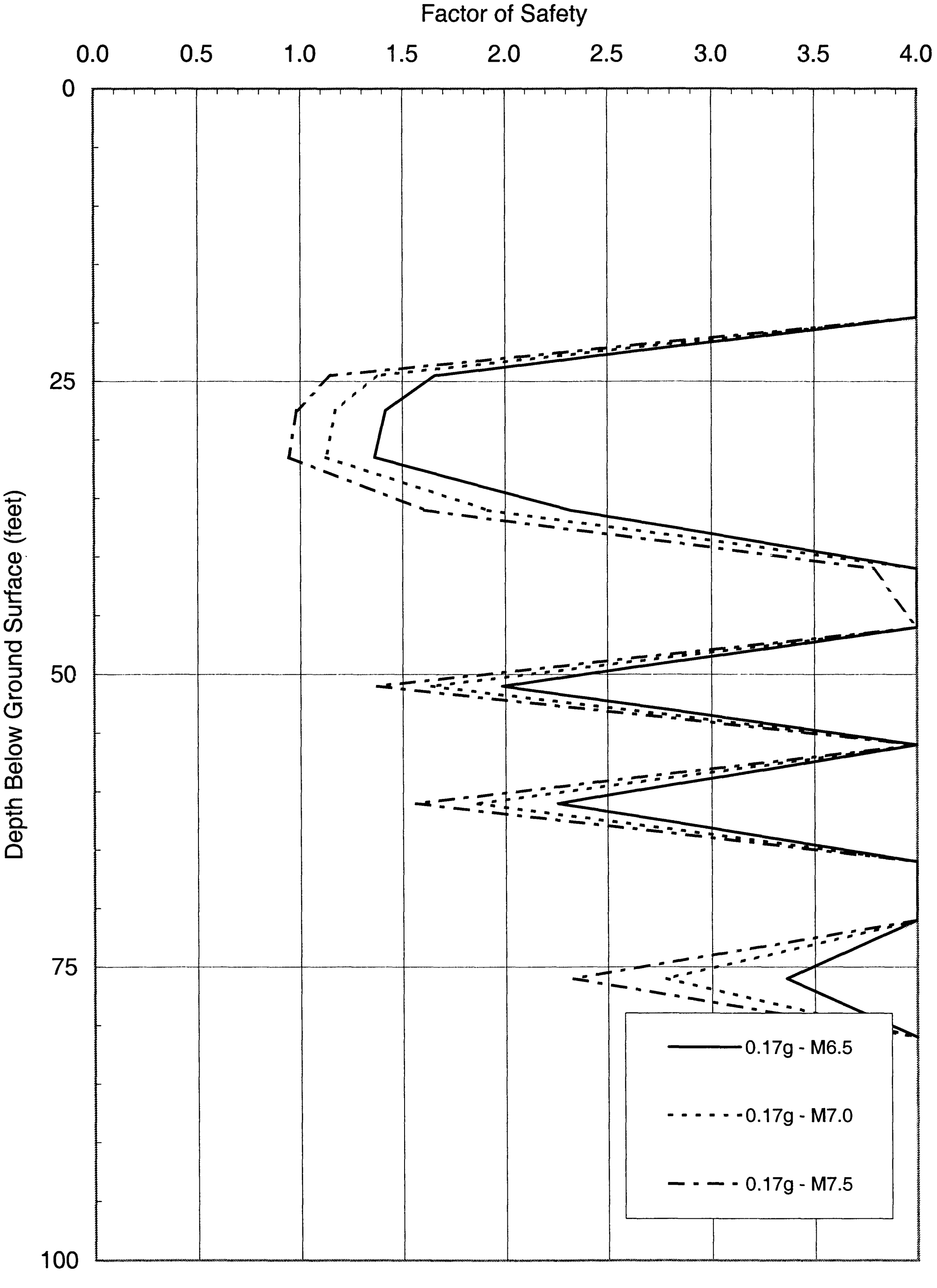
	DEPTH AND DESCRIPTION Elevation – 336.1	P.P. tsf	Tv tsf	LL %	PI %	ASTM Class	Wn %	γ_d pcf	Qu/2 psf	Particle Size Ana <4.75mm %	 <2.00mm %	 <0.425mm %	 <0.075mm %
40	Gray fine sand, very dense, wet - SPT 40.0-41.5', N_{60} = 79 - SPT 45.0-46.5', N_{60} = 78												
50	48.3-51.5' Grayish brown fine sand, very dense, wet - SPT 50.0-51.5', N_{60} = 61												
	Termination of boring @ 51.5'												
60													
70													
80													

JOB NO.: RI98-043
COUNTY: Stoddard
ROUTE: US 60
COORDINATE (88 DATUM): 907710.69, 348878.49
GENERAL LOCATION: B-4, St. Francis River

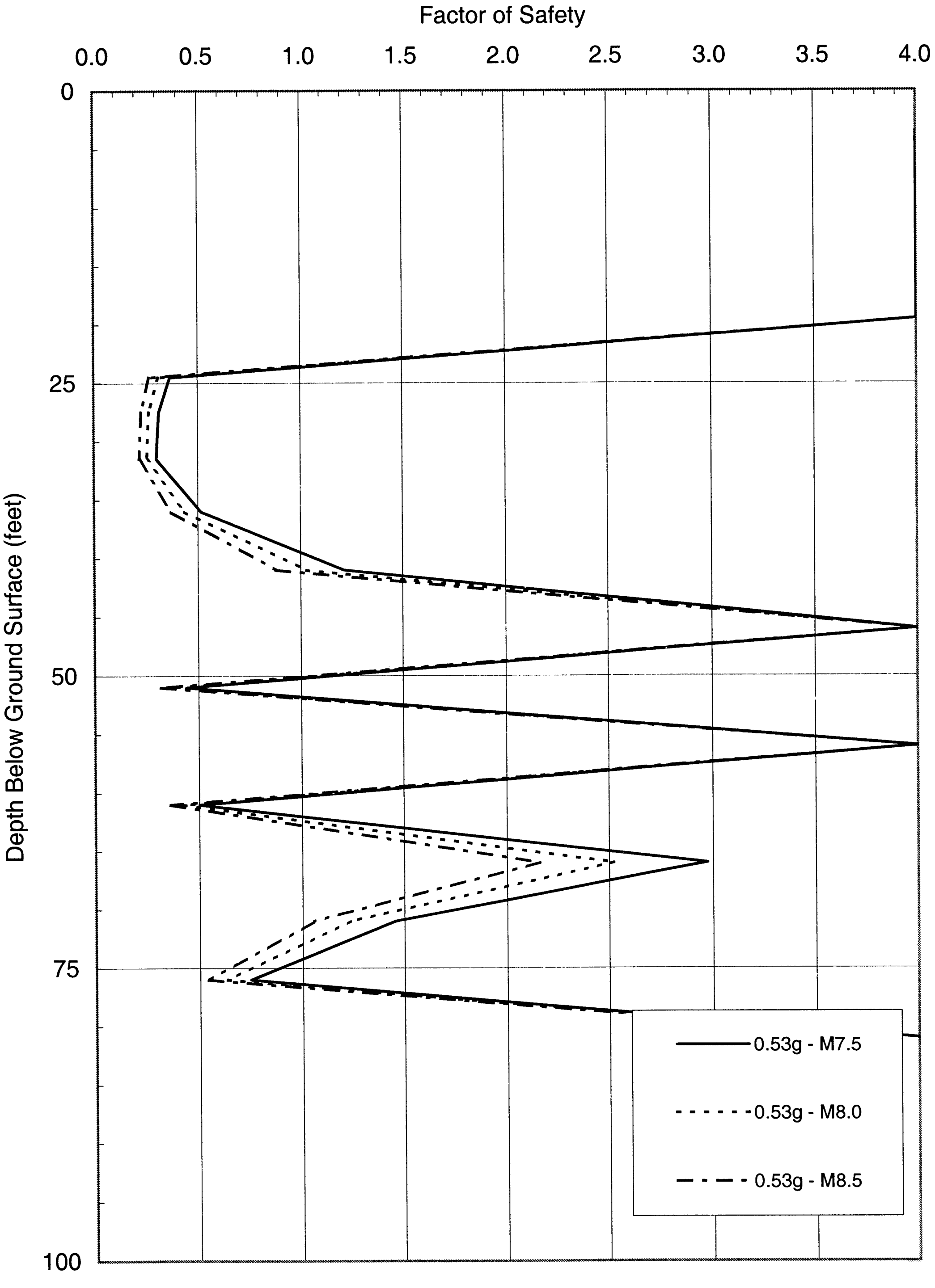
Key:
Ⴤ Water Table
P.P. "Pocket Penetrometer"
Tv. "Torvane"

FIGURE 6 of 6

F.2 Missouri Liquefaction Analyses: SPT Simplified Method

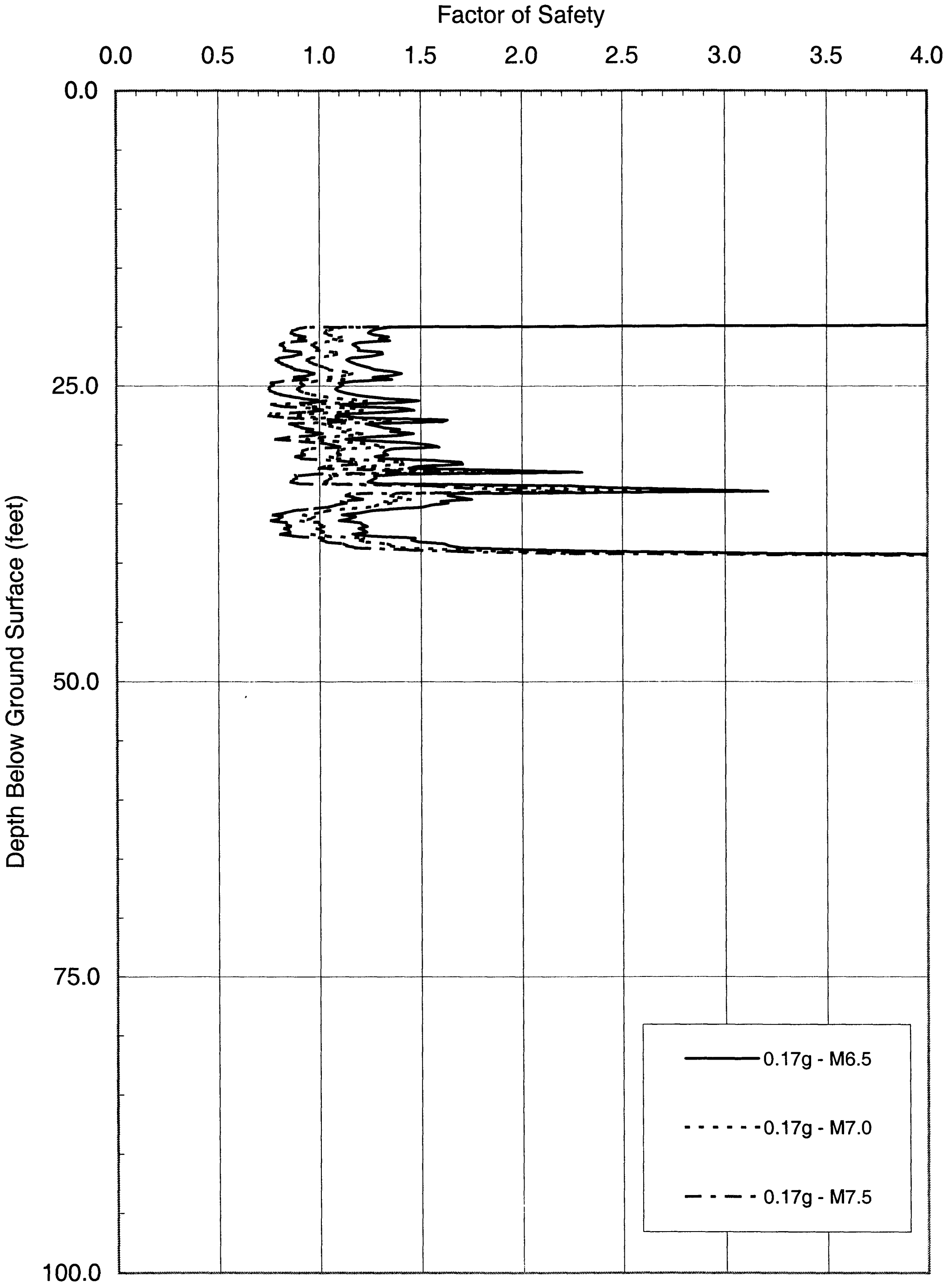


SPT Liquefaction Potential -- Missouri Site
475-Year Return Period

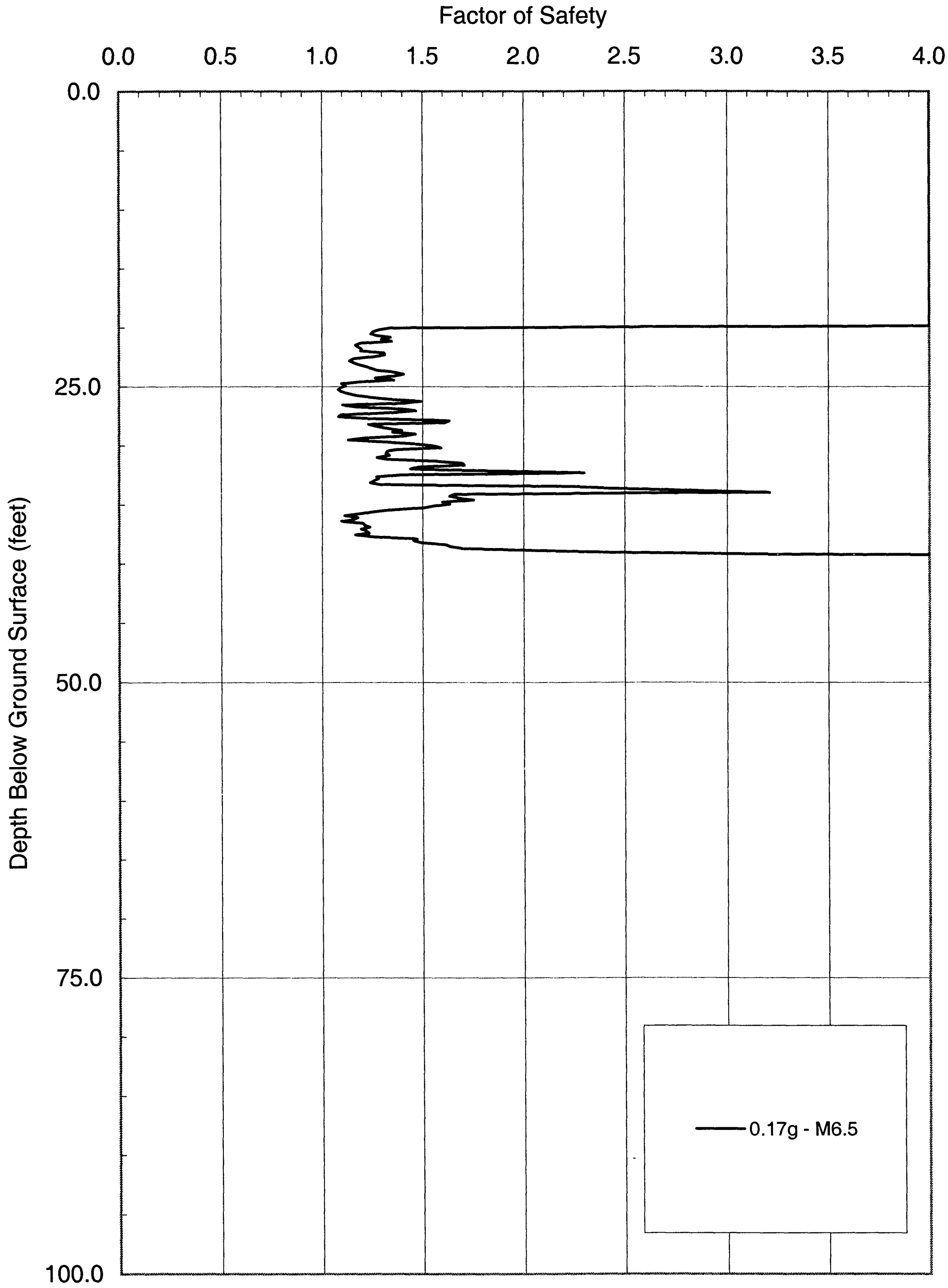


SPT Liquefaction Potential -- Missouri Site
2475-Year Return Period

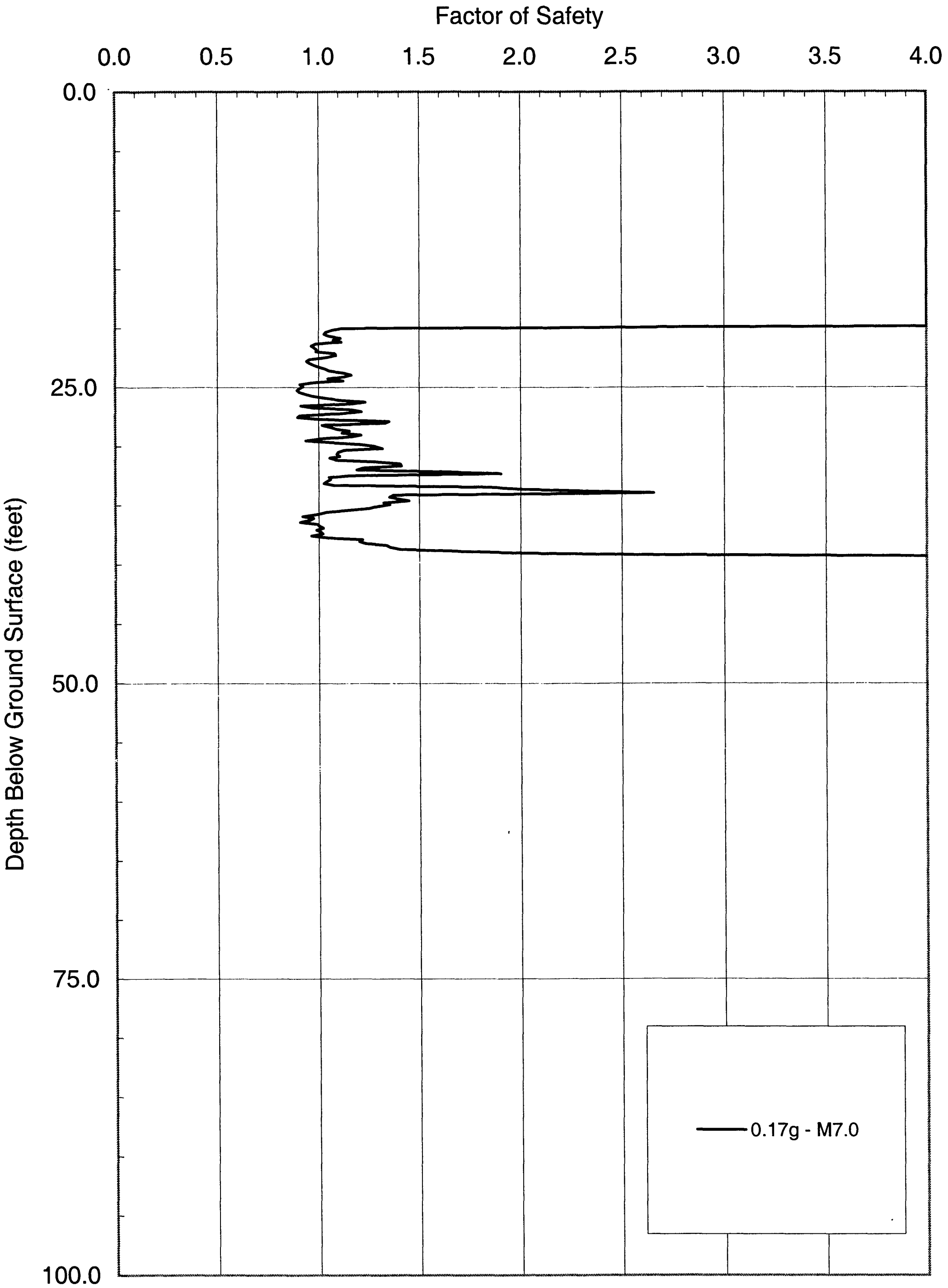
F.3 Missouri Liquefaction Analyses: CPT Simplified Method



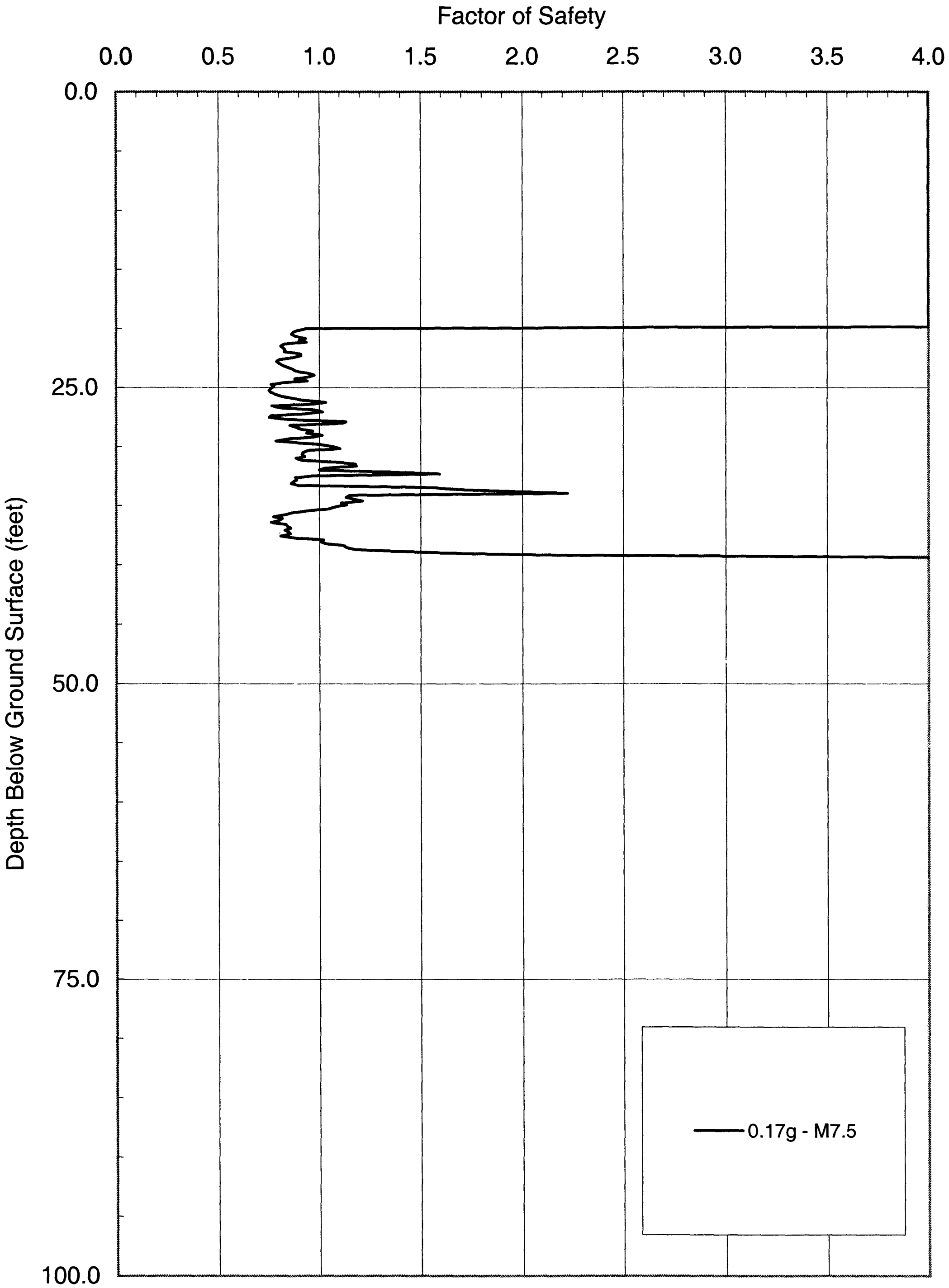
CPT Liquefaction Potential -- Missouri Site
475-Year Return Period



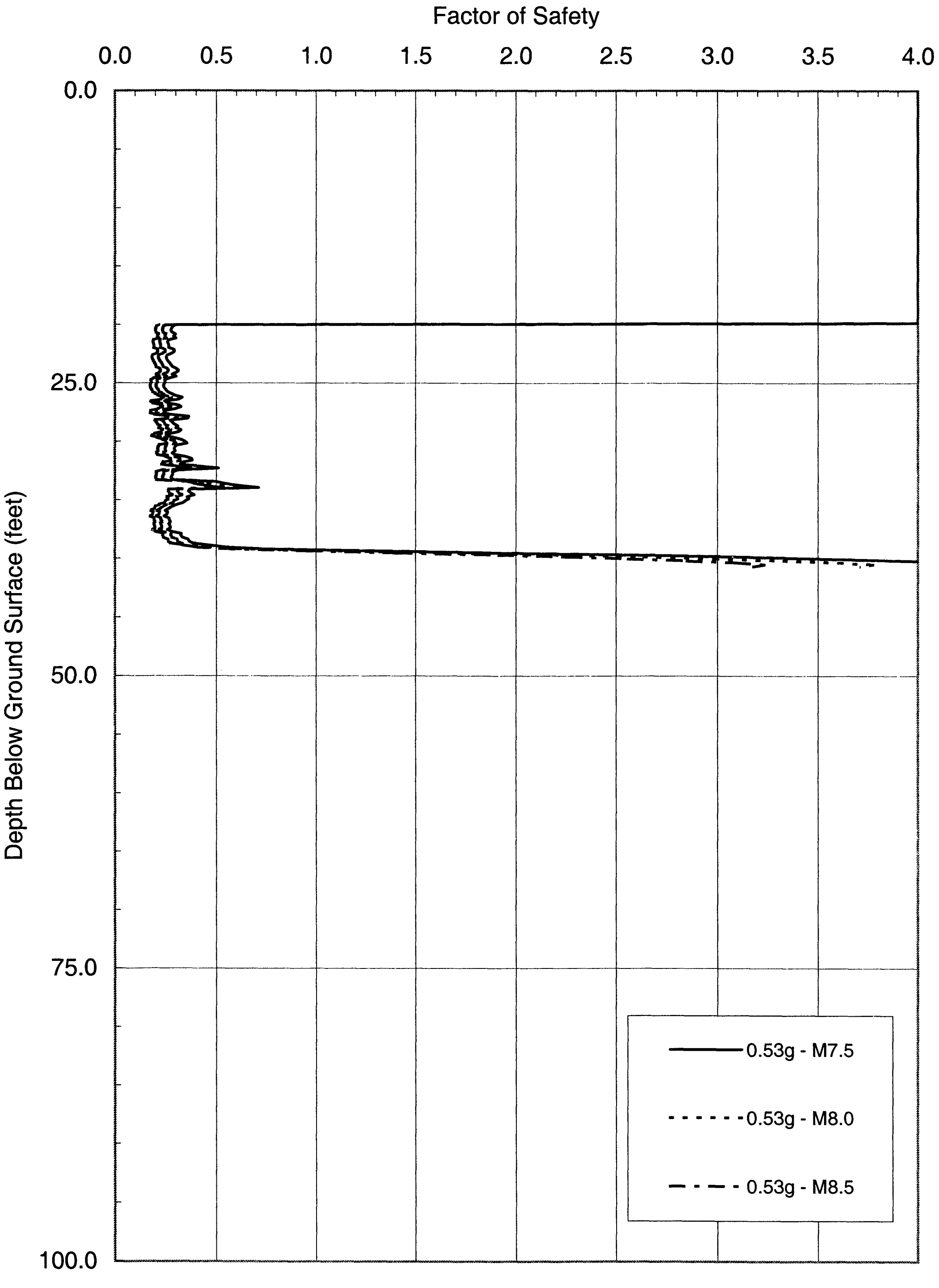
**CPT Liquefaction Potential -- Missouri Site
475-Year Return Period**



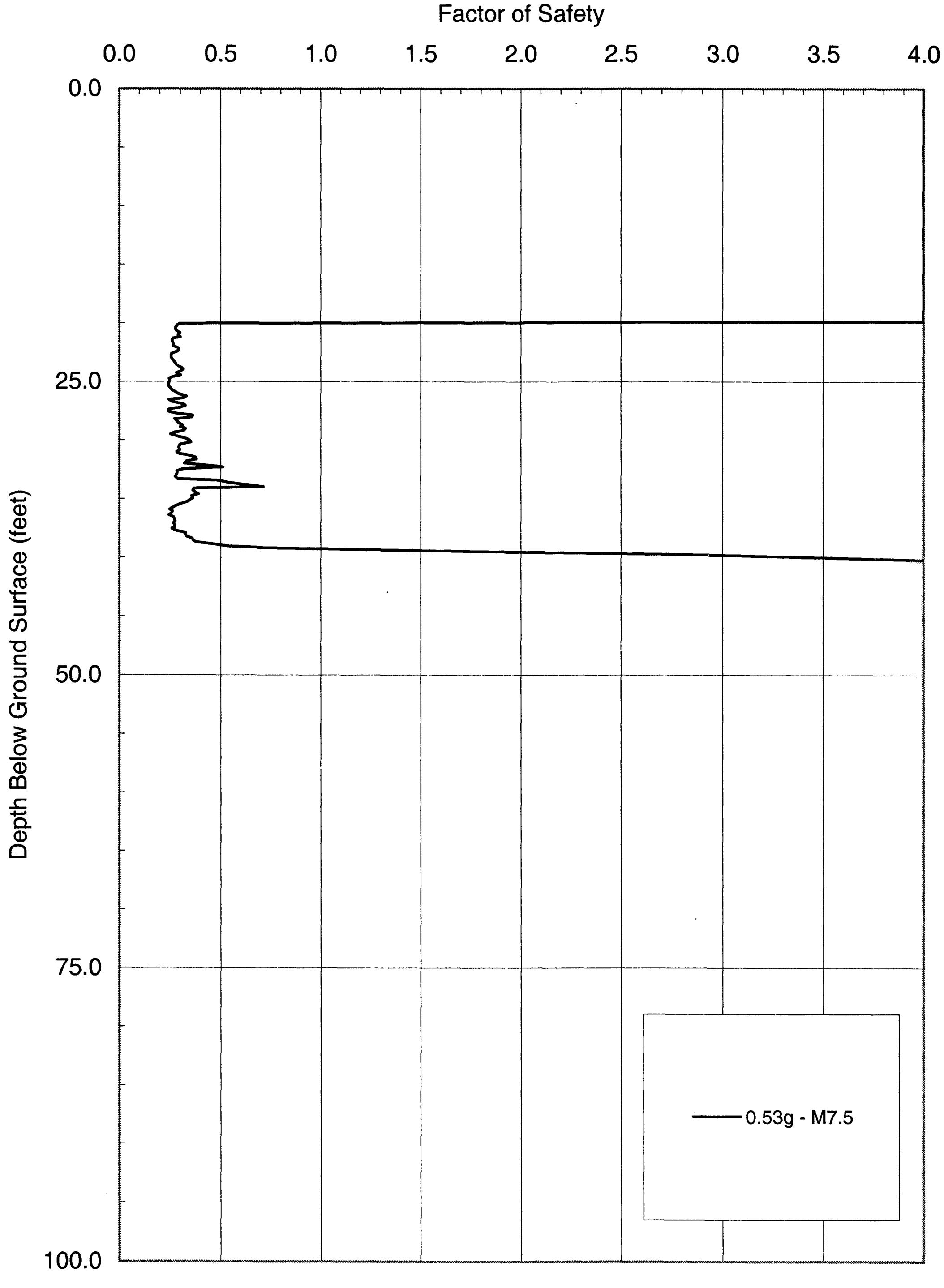
CPT Liquefaction Potential -- Missouri Site
475-Year Return Period



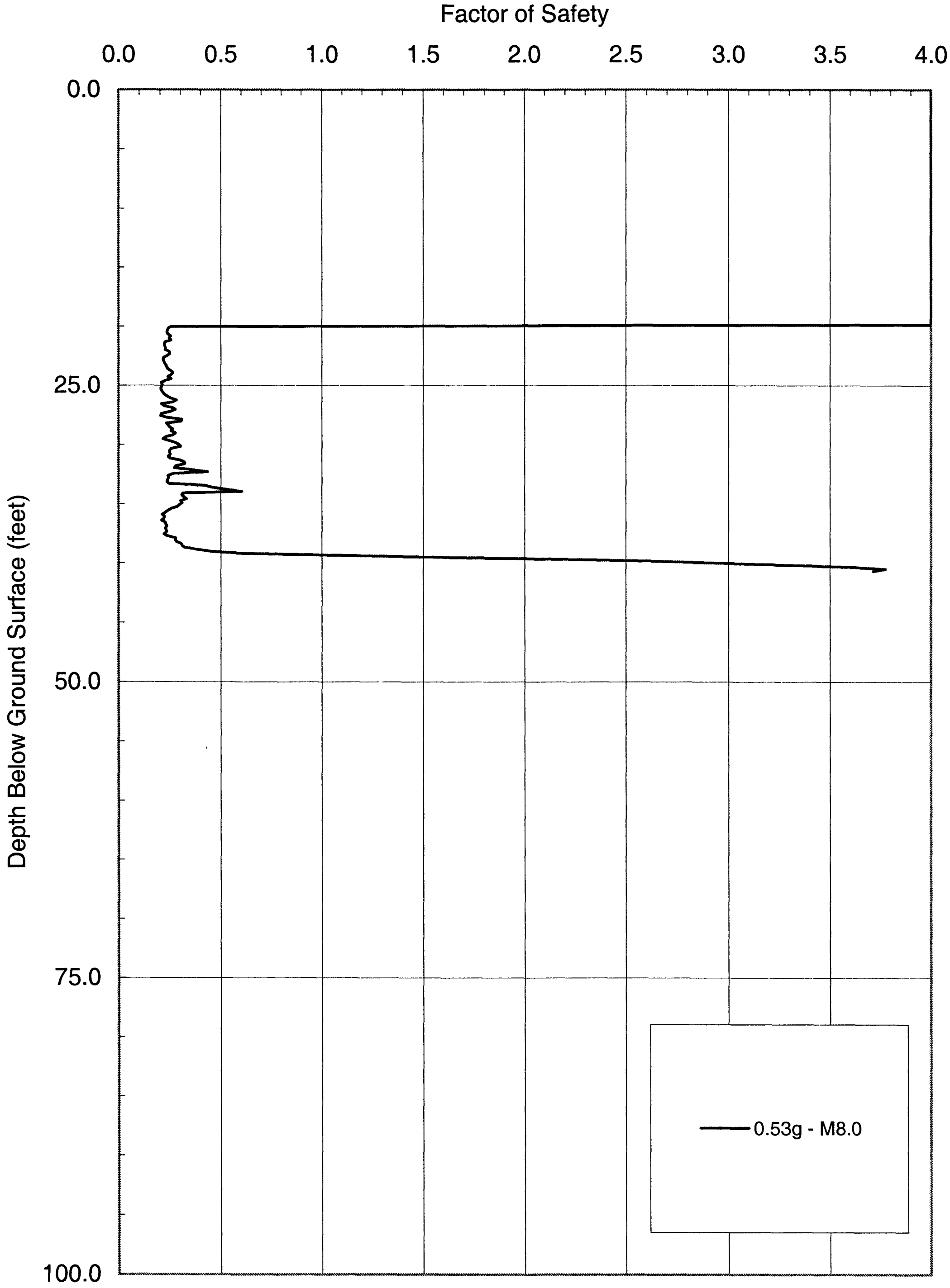
CPT Liquefaction Potential -- Missouri Site
475-Year Return Period



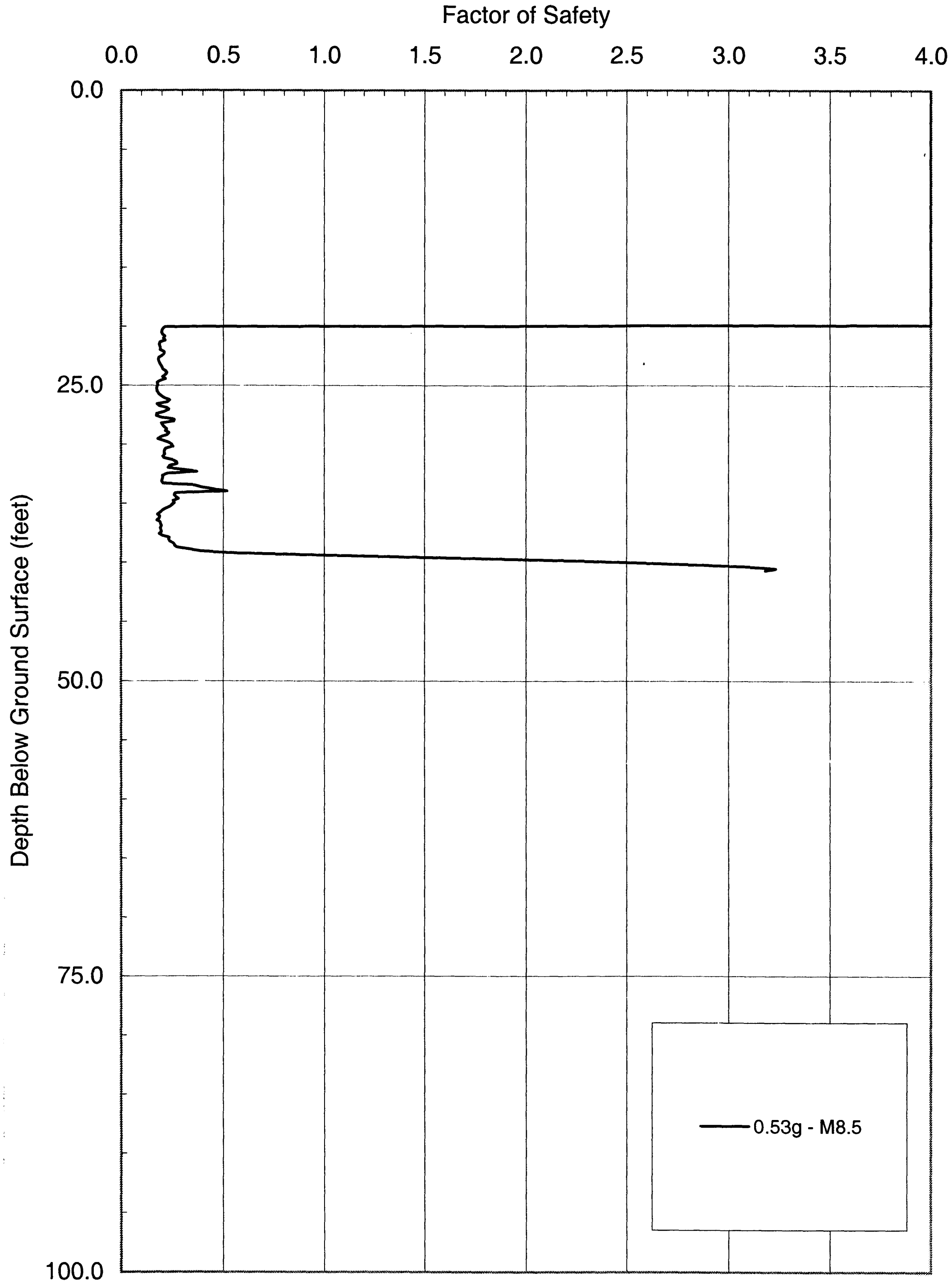
CPT Liquefaction Potential -- Missouri Site
2475-Year Return Period



CPT Liquefaction Potential -- Missouri Site
2475-Year Return Period



CPT Liquefaction Potential -- Missouri Site
2475-Year Return Period



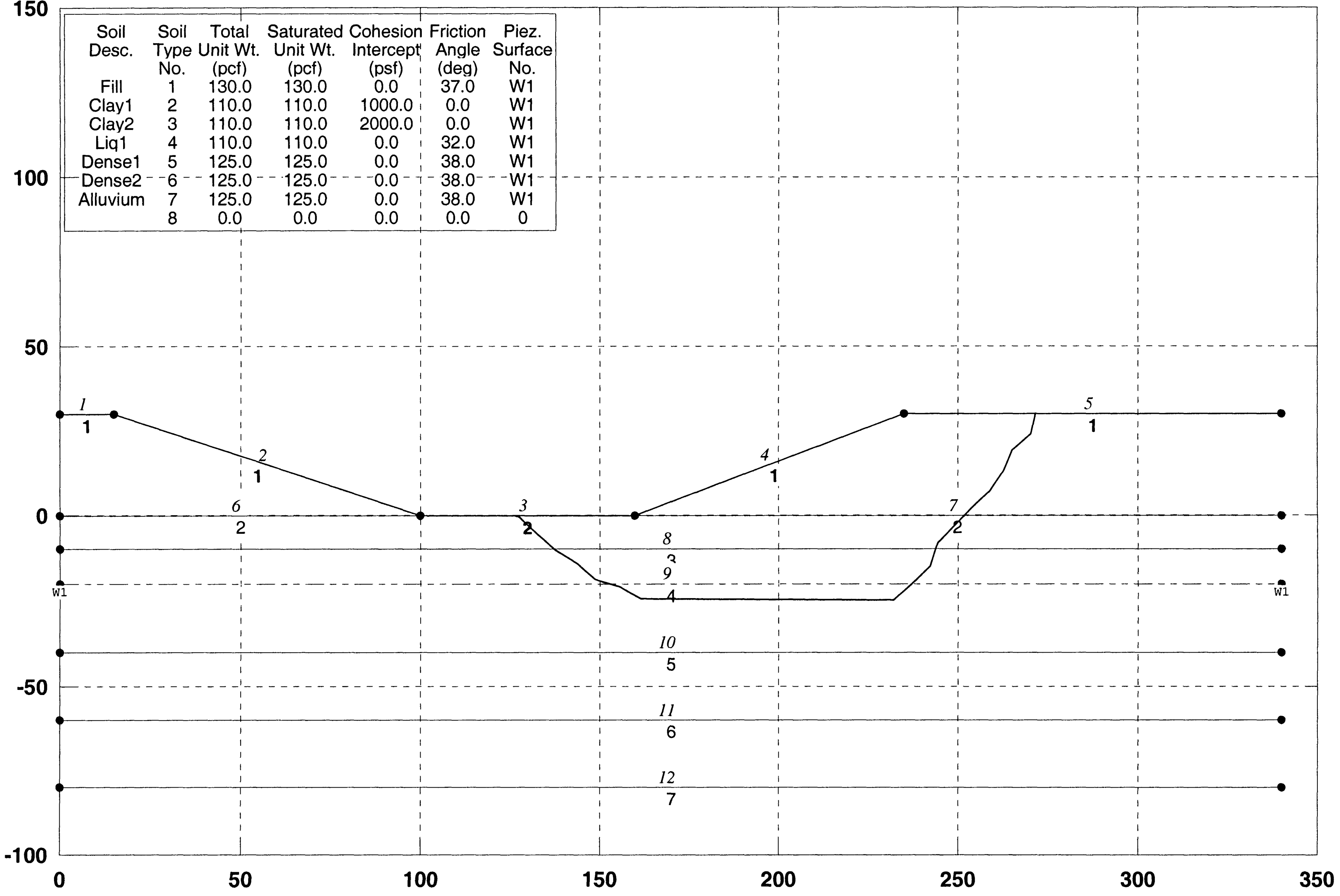
CPT Liquefaction Potential -- Missouri Site
2475-Year Return Period

F.4 Missouri Stability Analyses

F.4.1 Missouri Stability Analyses: Preliquefaction Case

NCHRP MISS, PRELIQ, con. front wedge, kh=0, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPFW0.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:40AM

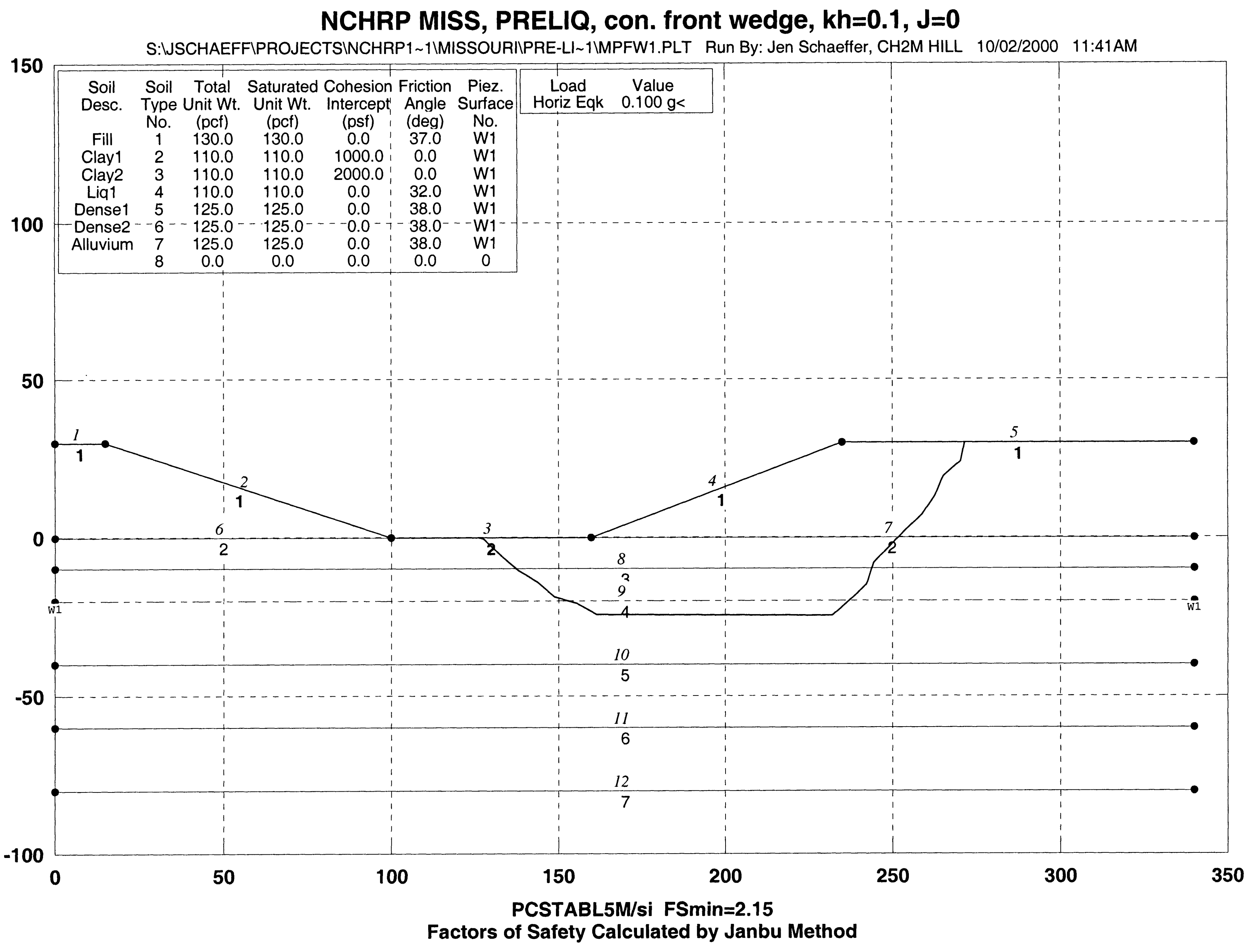


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

PCSTABL5M/si FSmin=2.93
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. front wedge, kh=0.1, J=0

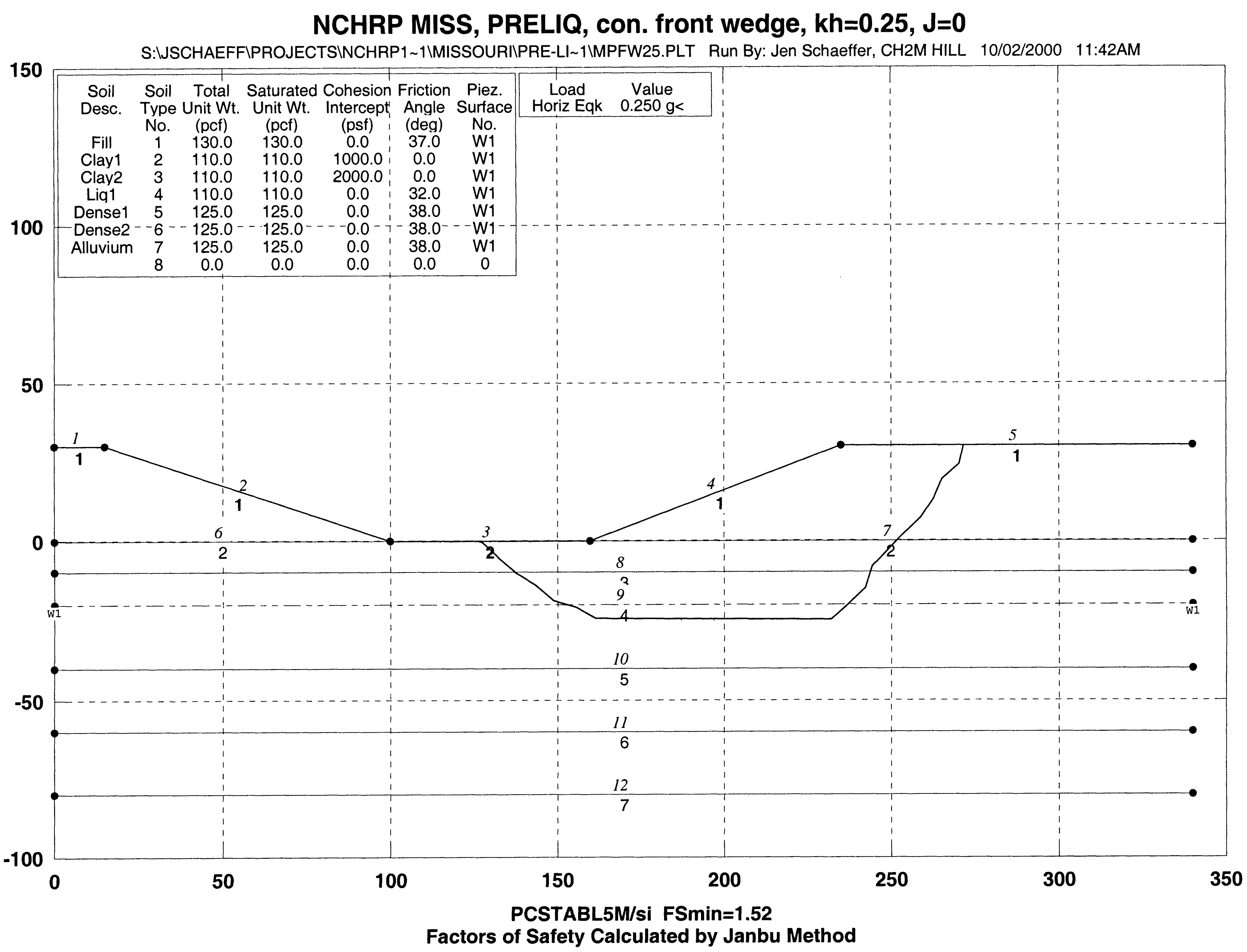
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPFW1.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:41AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=2.15
Factors of Safety Calculated by Janbu Method



NCHRP MISS, PRELIQ, con. front wedge, kh=0.25, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPFW25.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:42AM

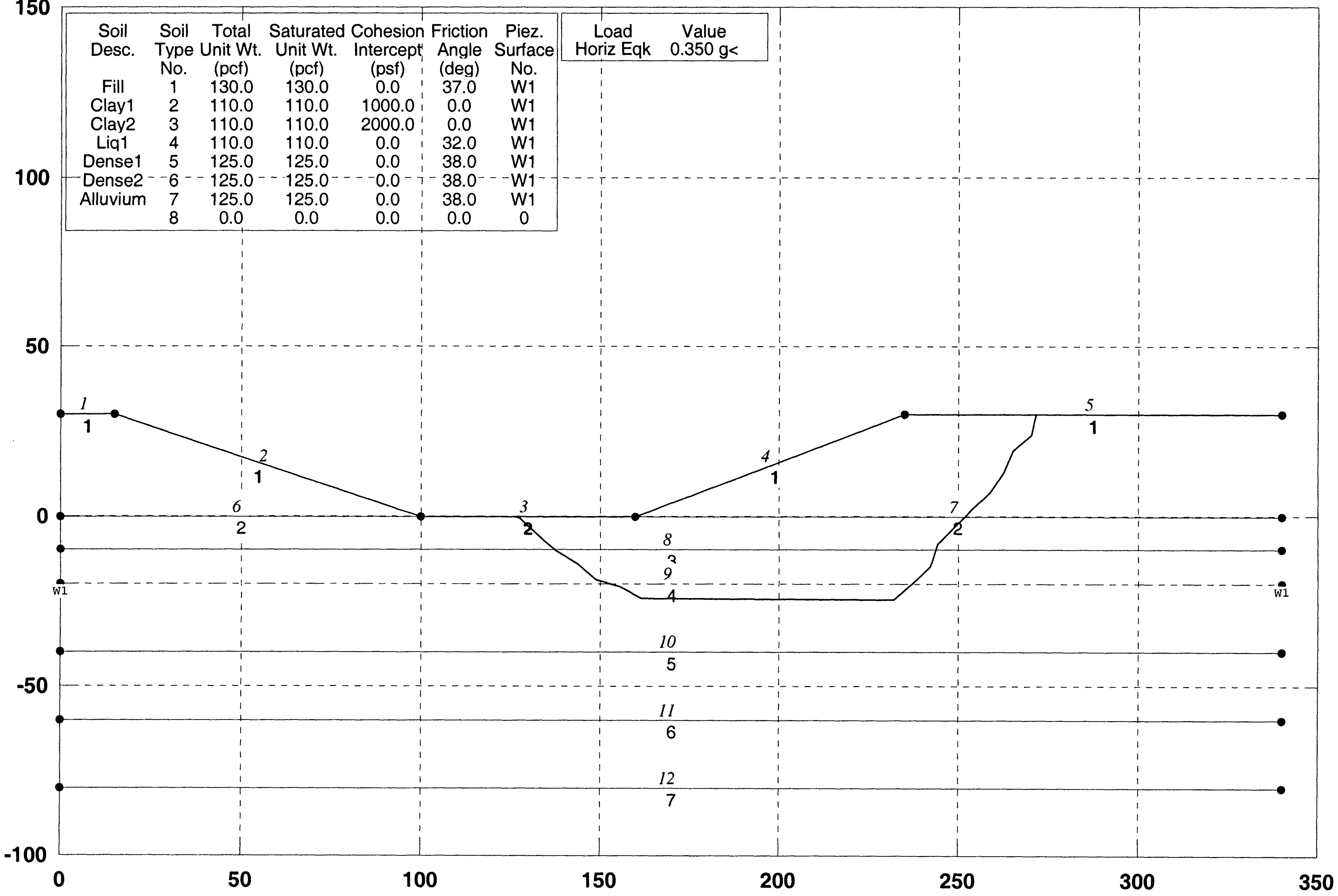
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.250 g<

PCSTABL5M/si FSmin=1.52
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. front wedge, kh=0.35, J=0

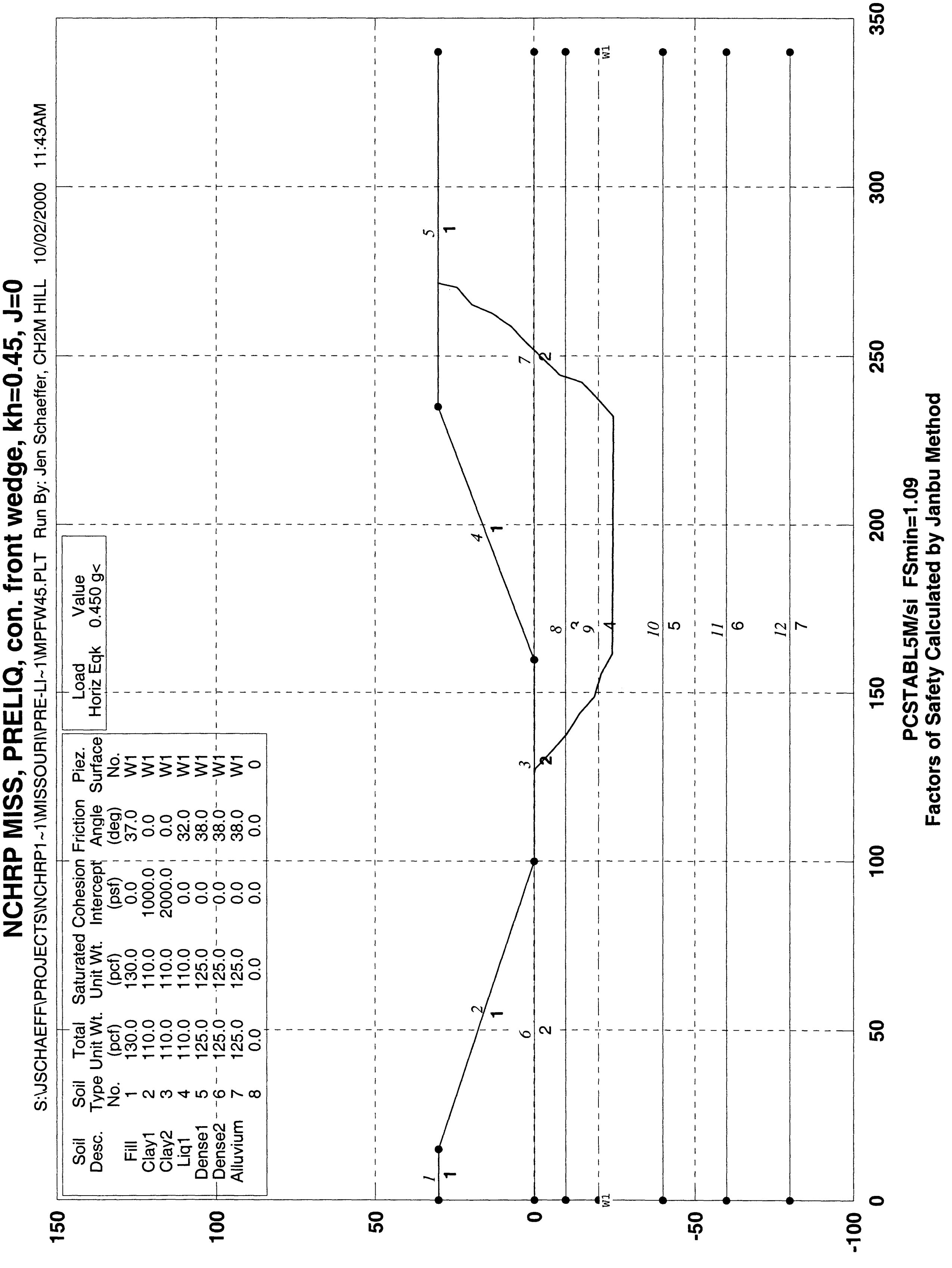
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPFW35.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:42AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

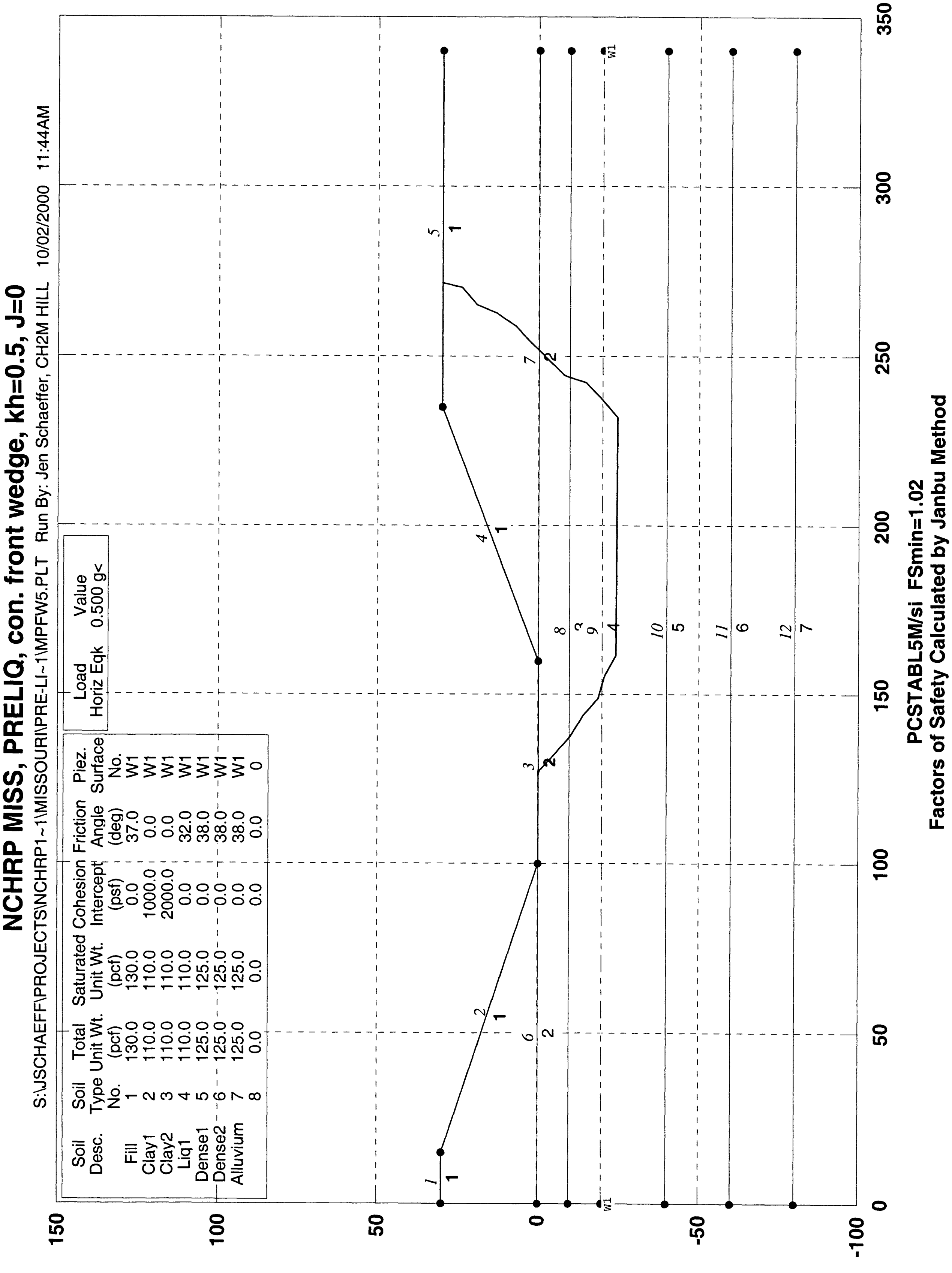
Load	Value
Horiz Eqk	0.350 g<

PCSTABL5M/si FSmin=1.27
Factors of Safety Calculated by Janbu Method



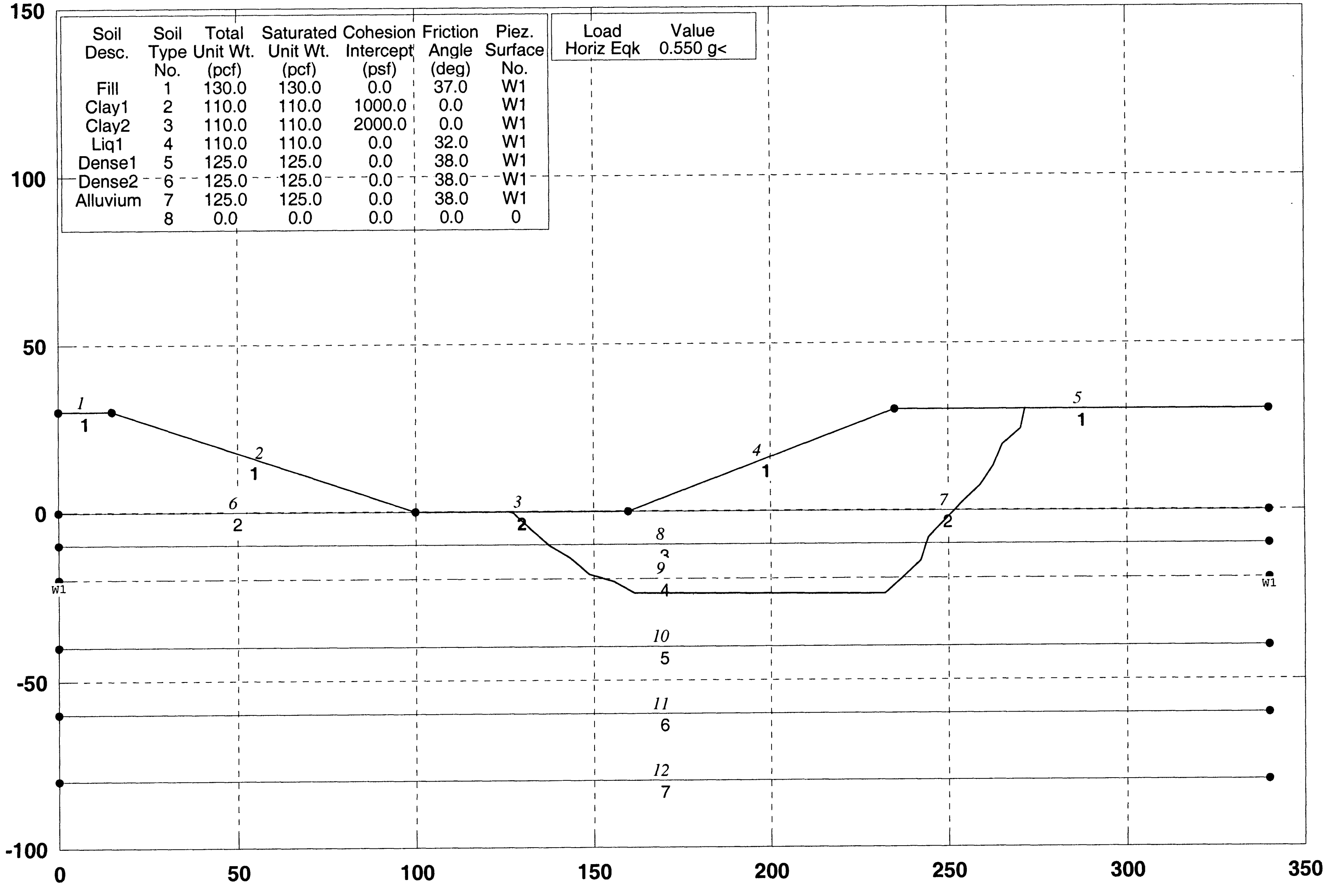
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.450 g<


NCHRP MISS, PRELIQ, con. front wedge, kh=0.5, J=0
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPFW5.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:44AM
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Fill 1 130.0 130.0 0.0 37.0 W1
Clay1 2 110.0 110.0 1000.0 0.0 W1
Clay2 3 110.0 110.0 2000.0 0.0 W1
Liq1 4 110.0 110.0 0.0 32.0 W1
Dense1 5 125.0 125.0 0.0 38.0 W1
Dense2 6 125.0 125.0 0.0 38.0 W1
Alluvium 7 125.0 125.0 0.0 38.0 W1
8 0.0 0.0 0.0 0.0 0
Load Value
Horiz Eqk 0.500 g<
W1
150
100
50
0
-50
-100
0
50
100
150
200
250
300
350
PCSTABL5M/si FSmin=1.02
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. front wedge, kh=0.55, J=0

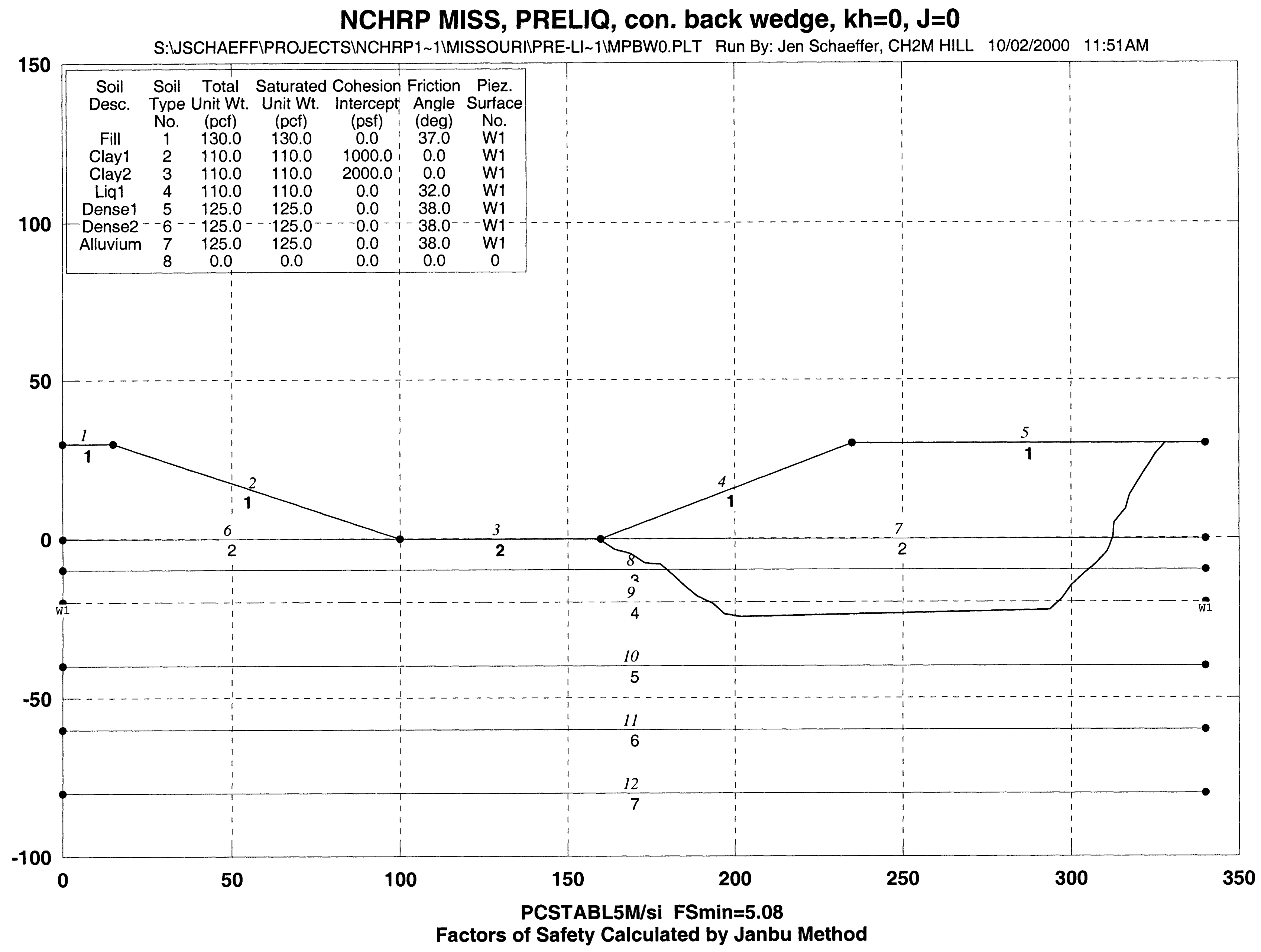
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPFW55.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:45AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.550 g<

PCSTABL5M/si FSmin=0.95
Factors of Safety Calculated by Janbu Method



NCHRP MISS, PRELIQ, con. back wedge, kh=0, J=0

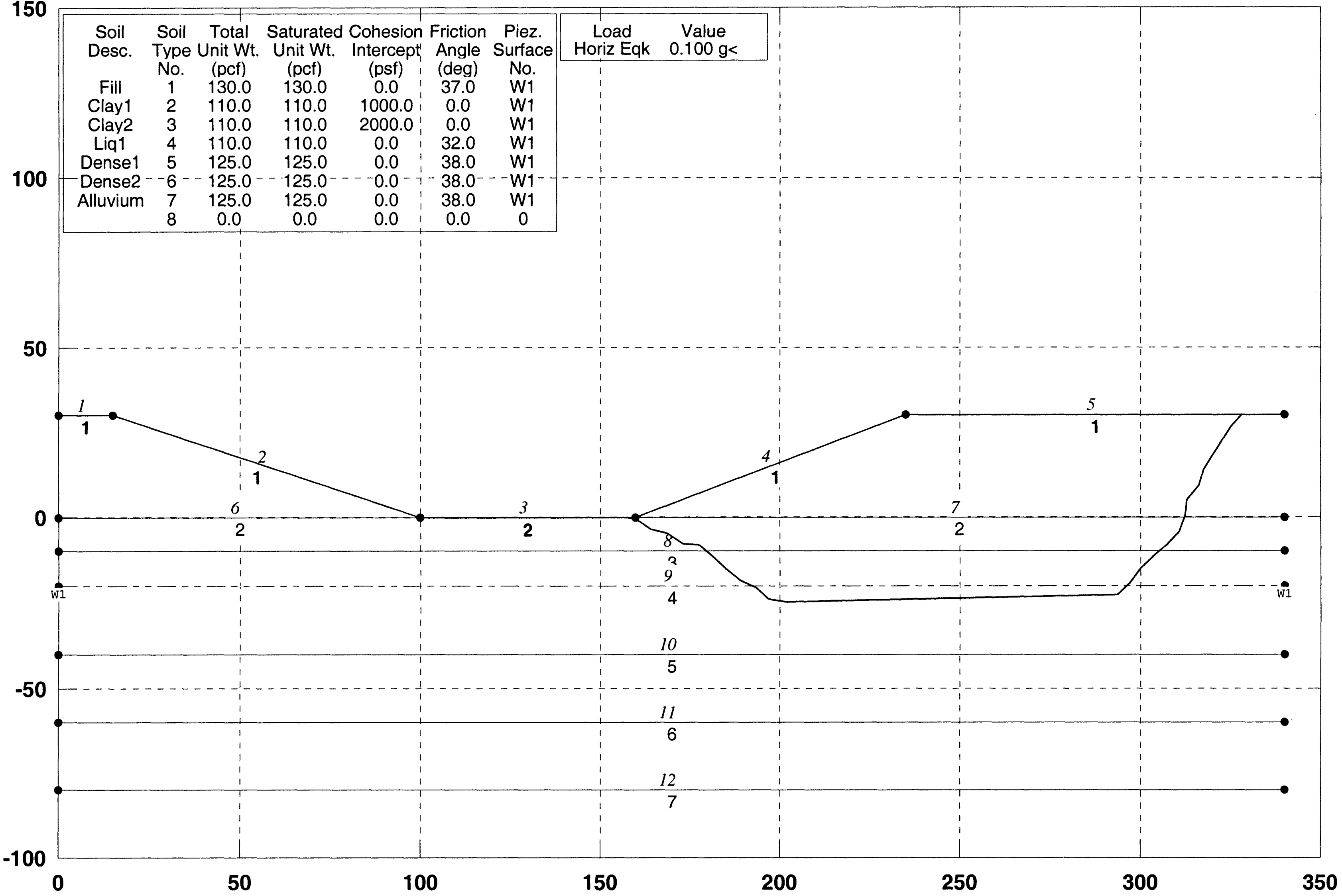
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW0.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:51AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

PCSTABL5M/si FSmin=5.08
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. back wedge, kh=0.1, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW1.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:52AM



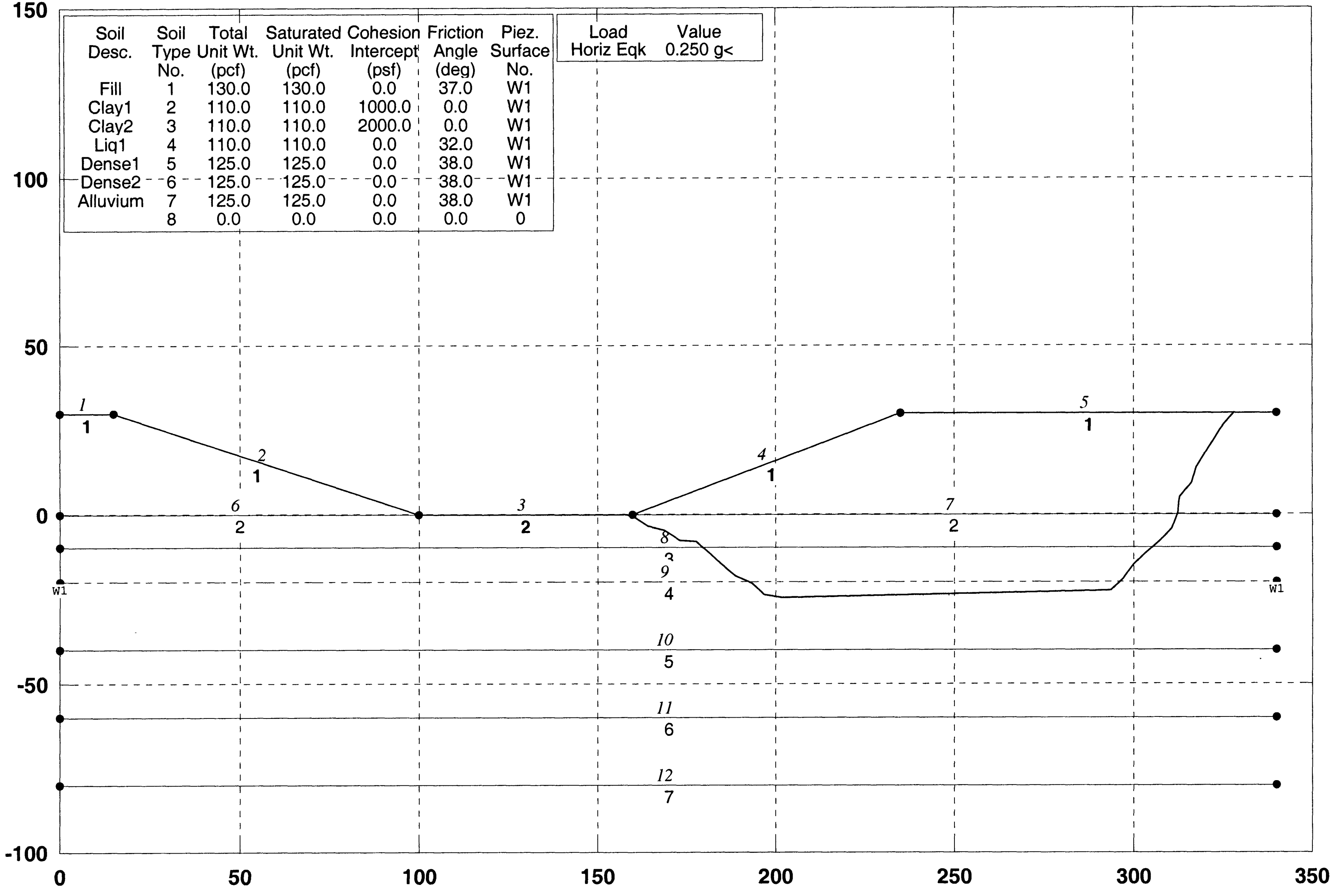
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=3.02
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. back wedge, kh=0.25, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW25.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:53AM



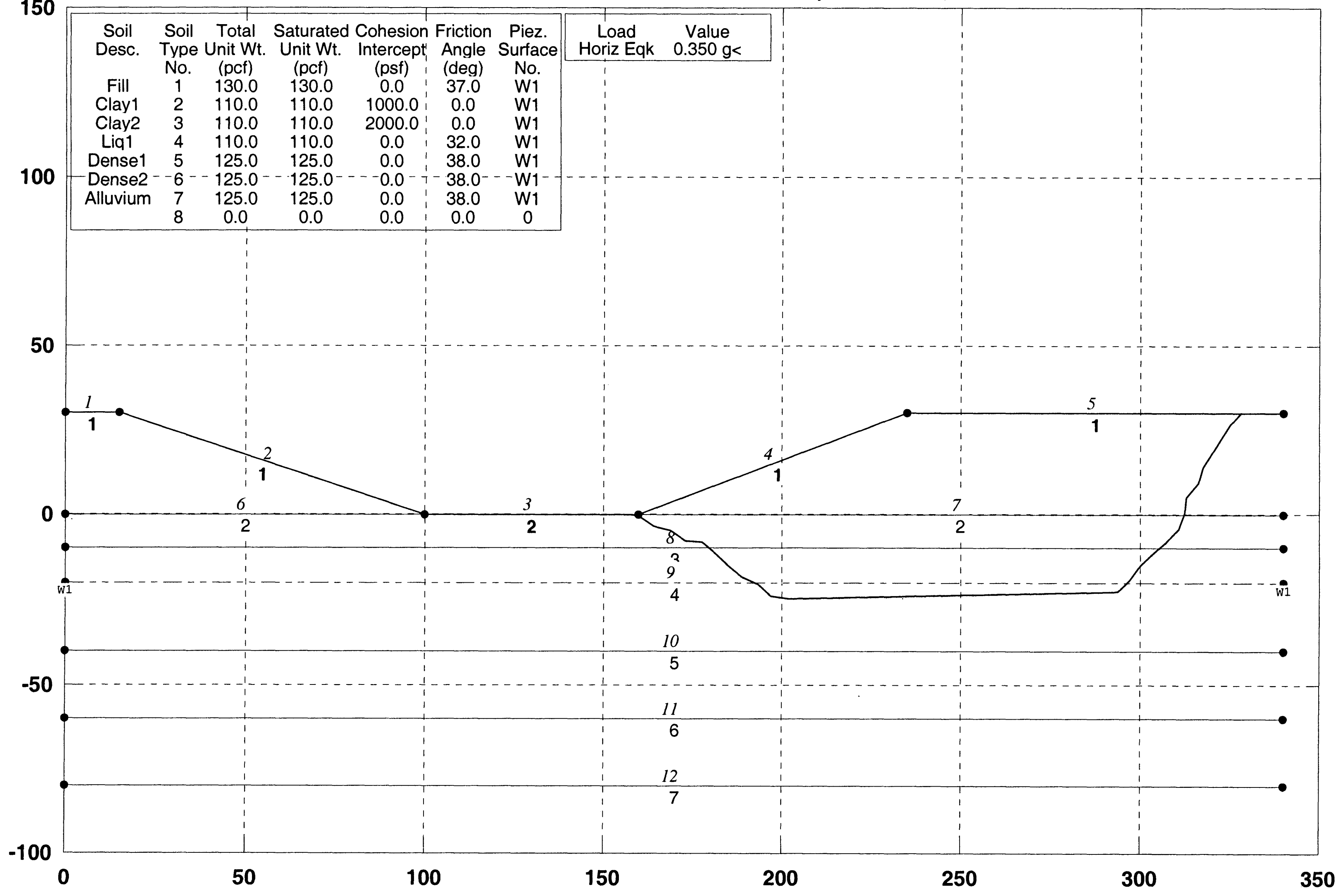
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.250 g<

PCSTABL5M/si FSmin=1.86
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. back wedge, kh=0.35, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW35.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:53AM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

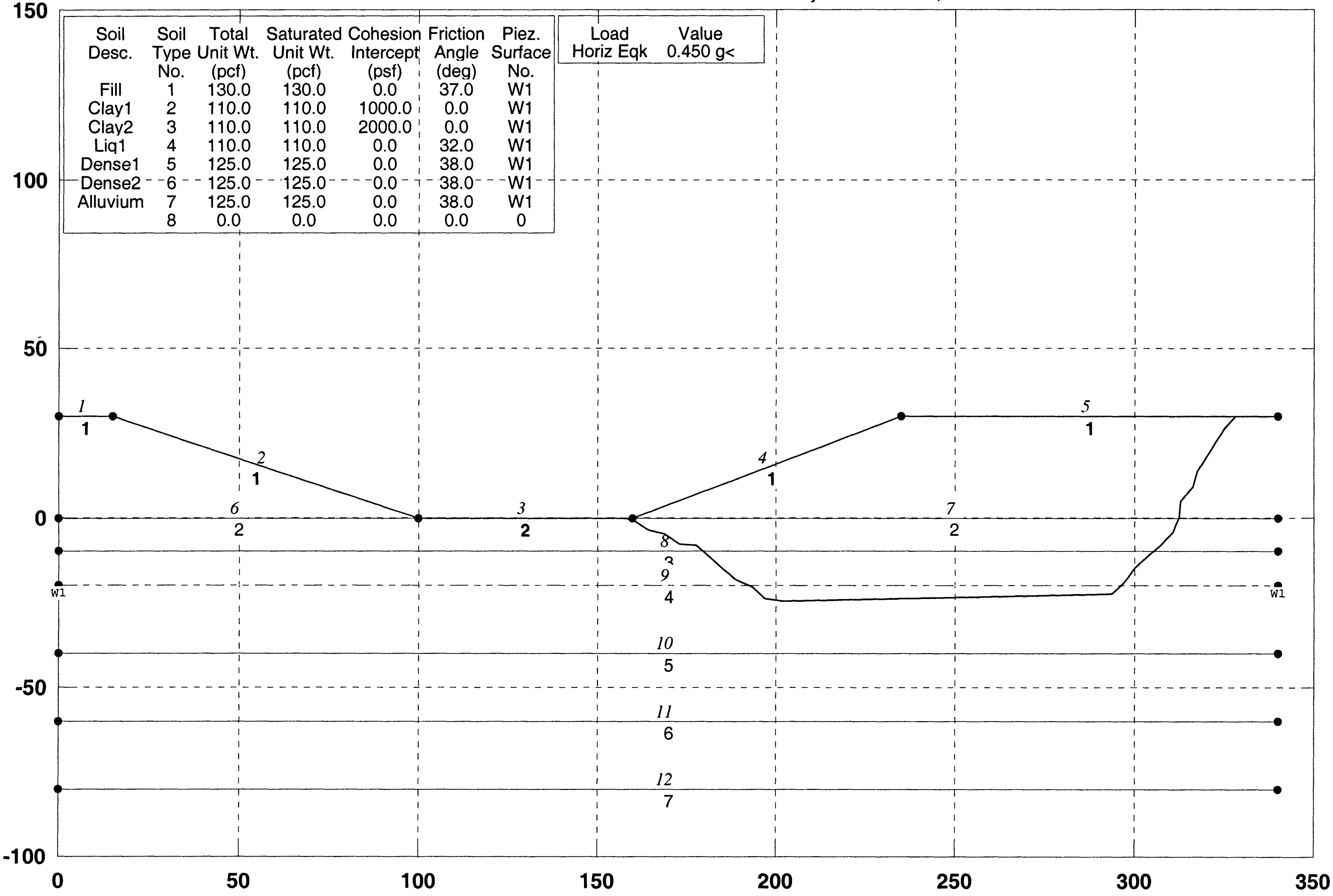
Load	Value
Horiz Eqk	0.350 g<

PCSTABL5M/si FSmin=1.48

Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. back wedge, kh=0.45, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW45.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 11:54AM



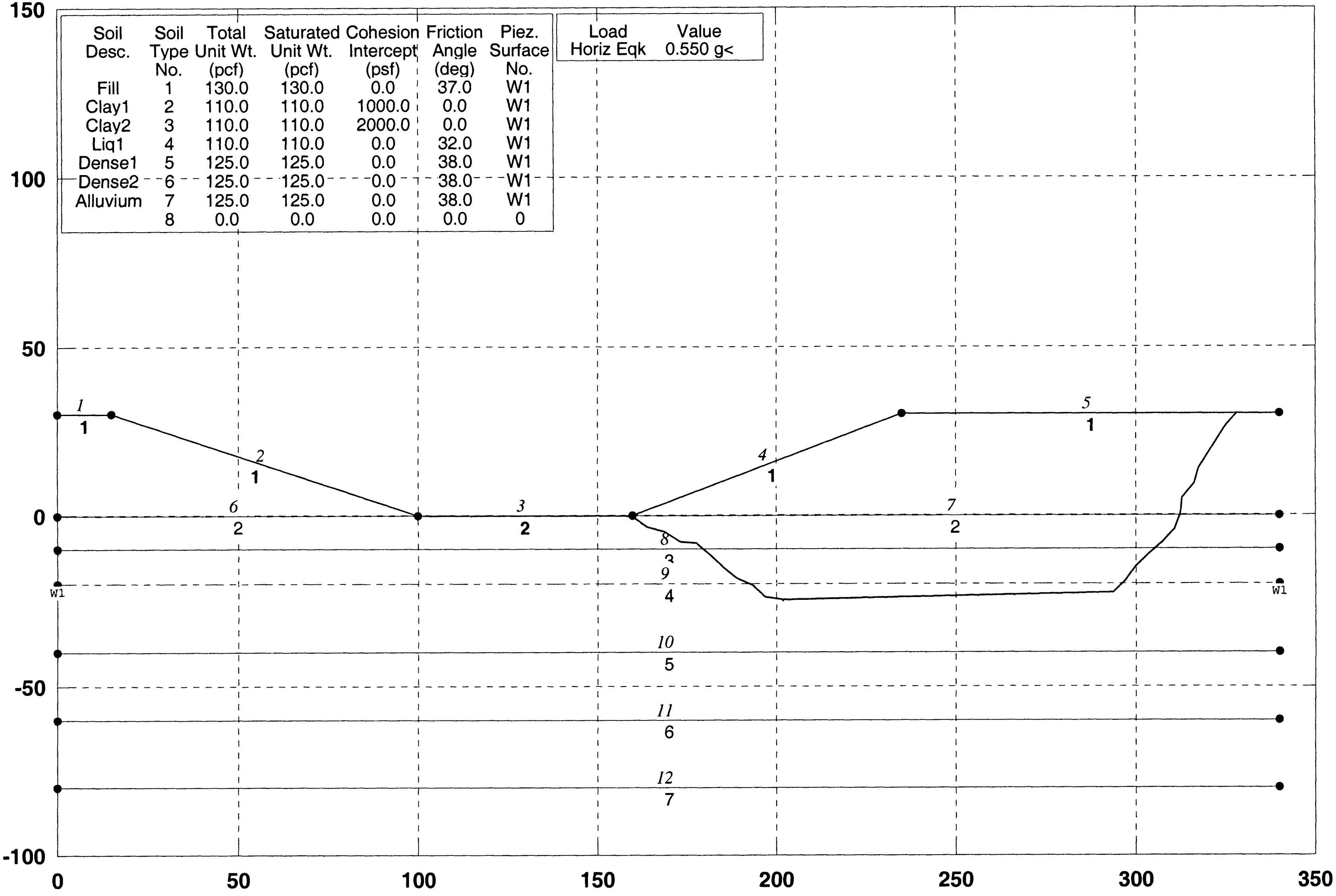
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.450 g<

PCSTABL5M/si FSmin=1.23
Factors of Safety Calculated by Janbu Method

NCHRP MISS, PRELIQ, con. back wedge, kh=0.55, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW55.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 12:01PM



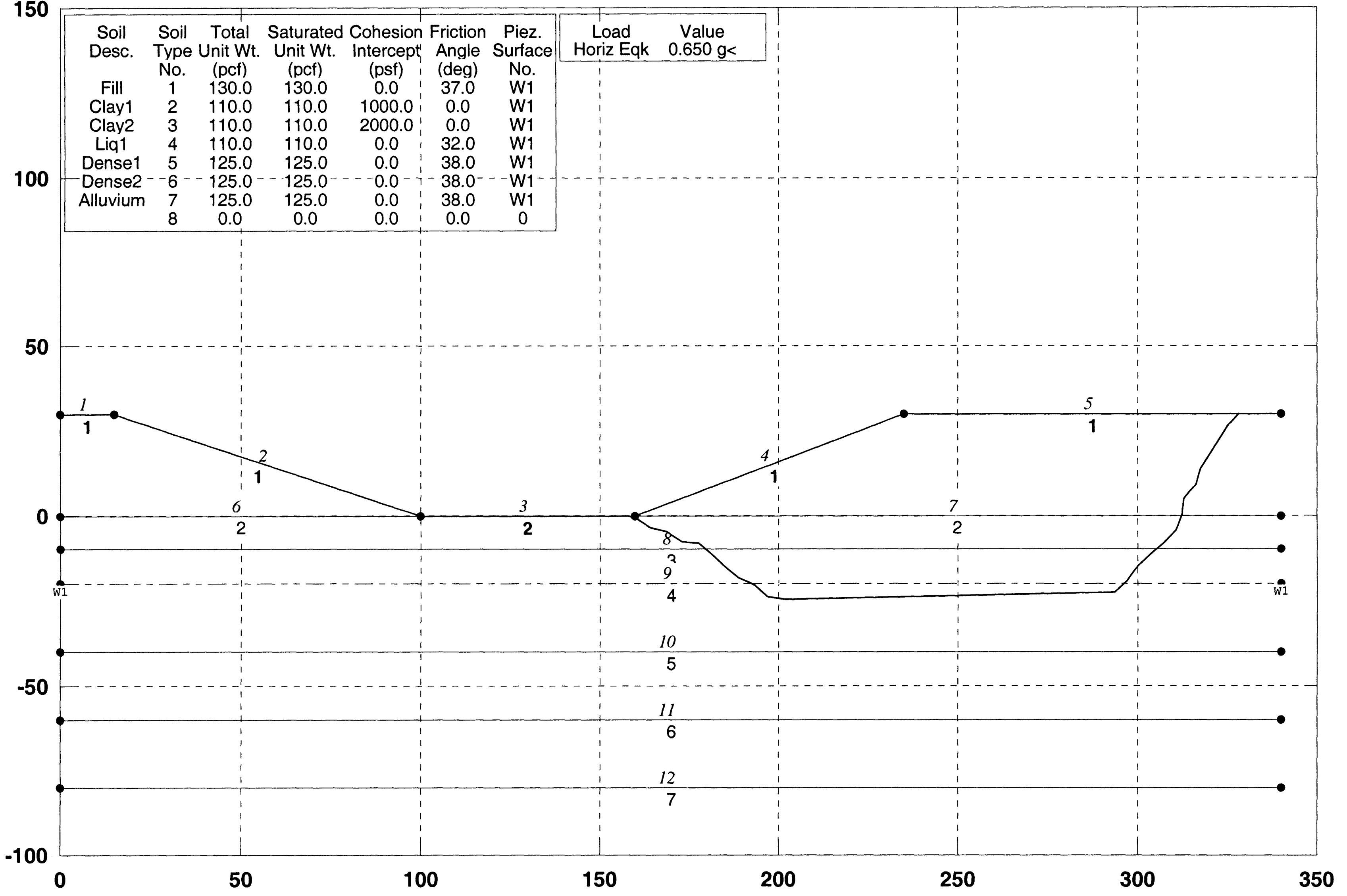
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	0.0	32.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1
	8	0.0	0.0	0.0	0.0	0

Load	Value
Horiz Eqk	0.550 g<

PCSTABL5M/si FSmin=1.06
Factors of Safety Calculated by Janbu Method

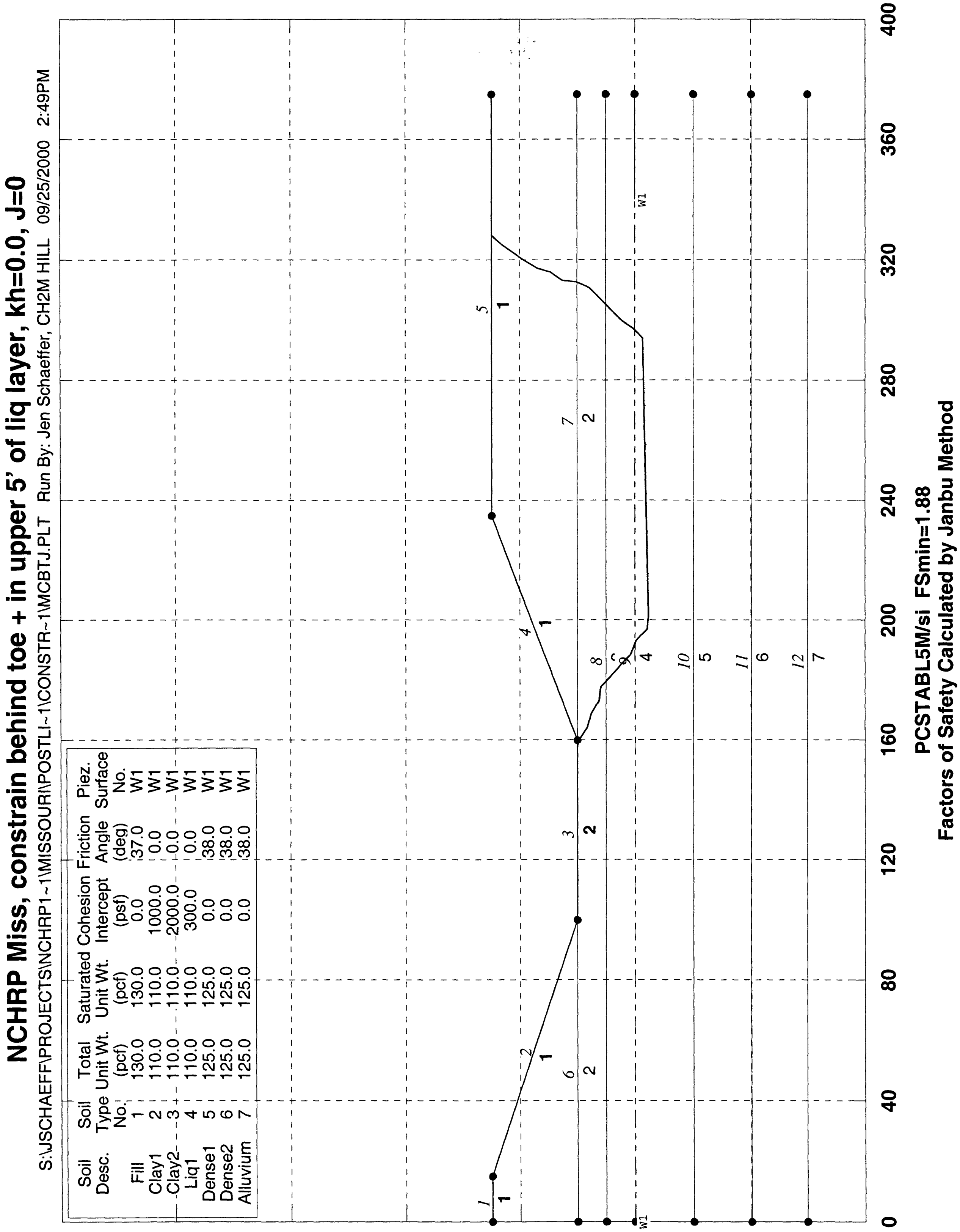
NCHRP MISS, PRELIQ, con. back wedge, kh=0.65, J=0

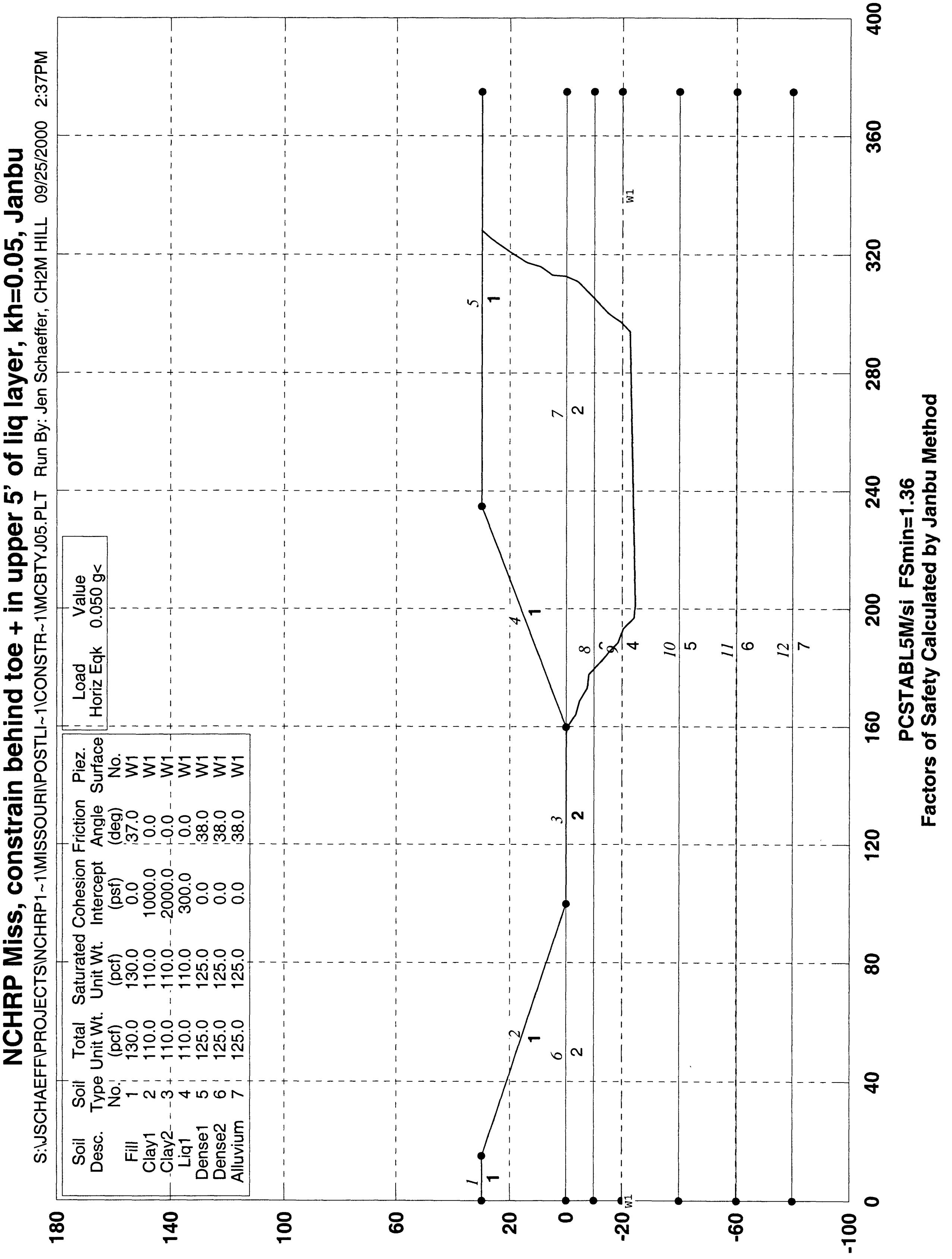
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PRE-LI~1\MPBW65.PLT Run By: Jen Schaeffer, CH2M HILL 10/02/2000 12:02PM



PCSTABL5M/si FSmin=0.94
Factors of Safety Calculated by Janbu Method

F.4.2 Missouri Stability Analyses: Flow Failure Case


NCHRP Miss, constrain behind toe + in upper 5' of liq layer, kh=0.0, J=0
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\POSTLI~1\CONSTR~1\MCBTJ.PLT Run By: Jen Schaeffer, CH2M HILL 09/25/2000 2:49PM
Soil Desc. | Soil Type No. | Total Unit Wt. (pcf) | Saturated Unit Wt. (pcf) | Cohesion Intercept (psf) | Friction Angle (deg) | Piez. Surface No.
Fill 1 130.0 130.0 0.0 37.0 W1
Clay1 2 110.0 110.0 1000.0 0.0 W1
Clay2 3 110.0 110.0 2000.0 0.0 W1
Liq1 4 110.0 110.0 300.0 0.0 W1
Dense1 5 125.0 125.0 0.0 38.0 W1
Dense2 6 125.0 125.0 0.0 38.0 W1
Alluvium 7 125.0 125.0 0.0 38.0 W1
W1
PCSTABL5M/si FSmin=1.88
Factors of Safety Calculated by Janbu Method
180
140
100
60
20
0
-20
-60
-100
0
40
80
120
160
200
240
280
320
360
400

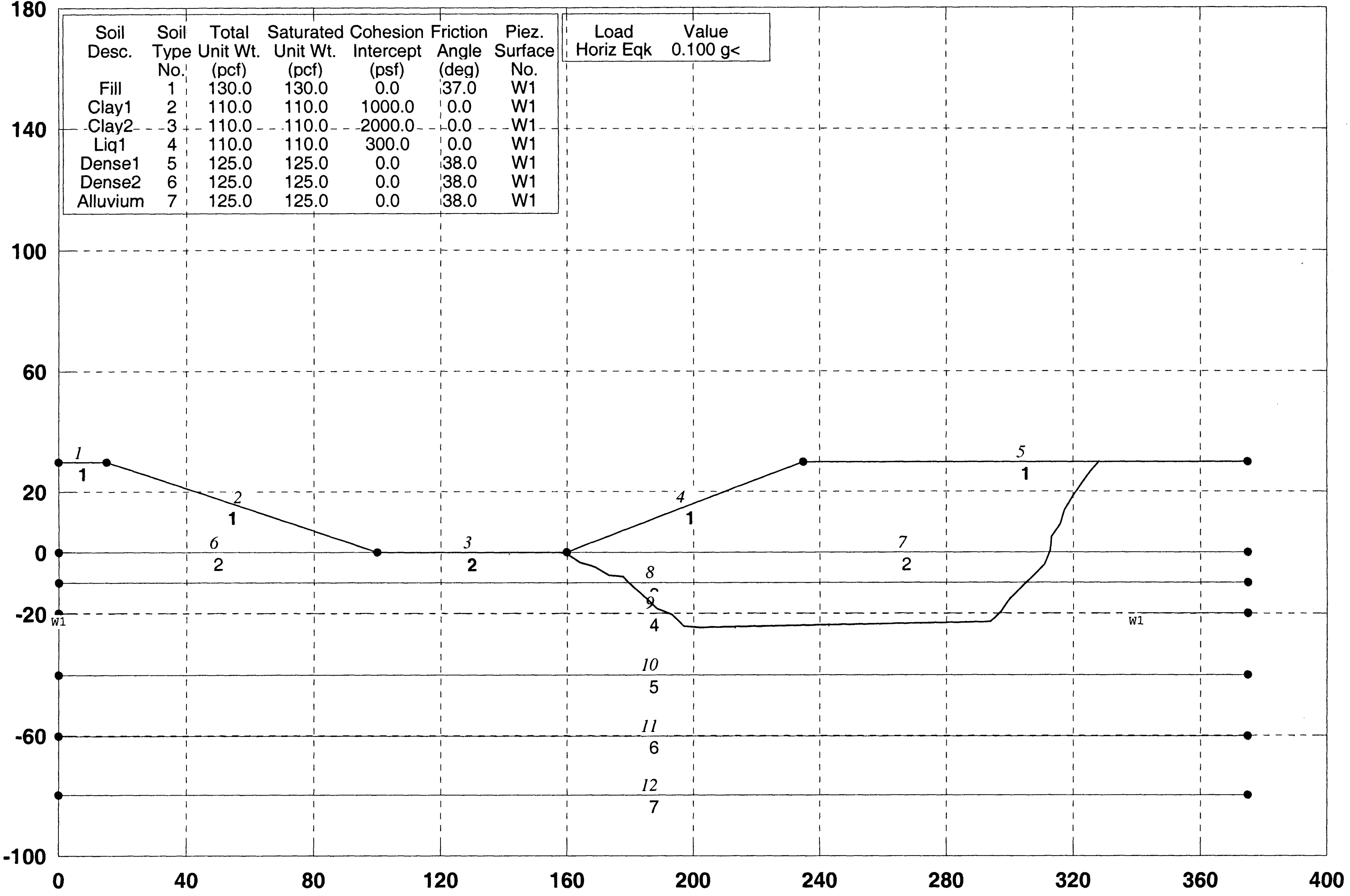


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.050 g<

NCHRP Miss, constrain behind toe + in upper 5' of liq layer, kh=0.1, Janbu

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\POSTLI~1\CONSTR~1\MCBTYJ1.PLT Run By: Jen Schaeffer, CH2M HILL 09/25/2000 2:36PM

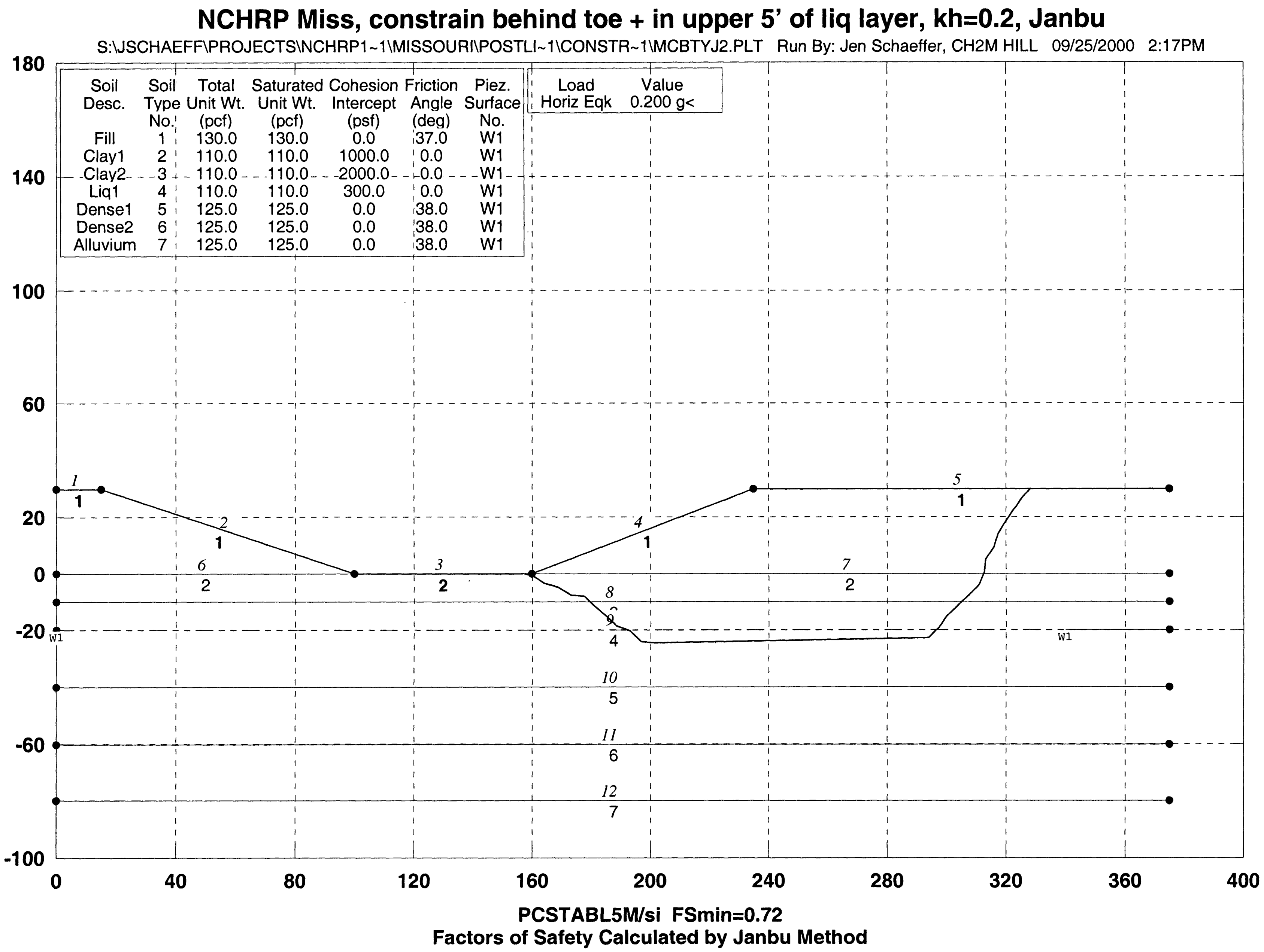


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=1.06

Factors of Safety Calculated by Janbu Method

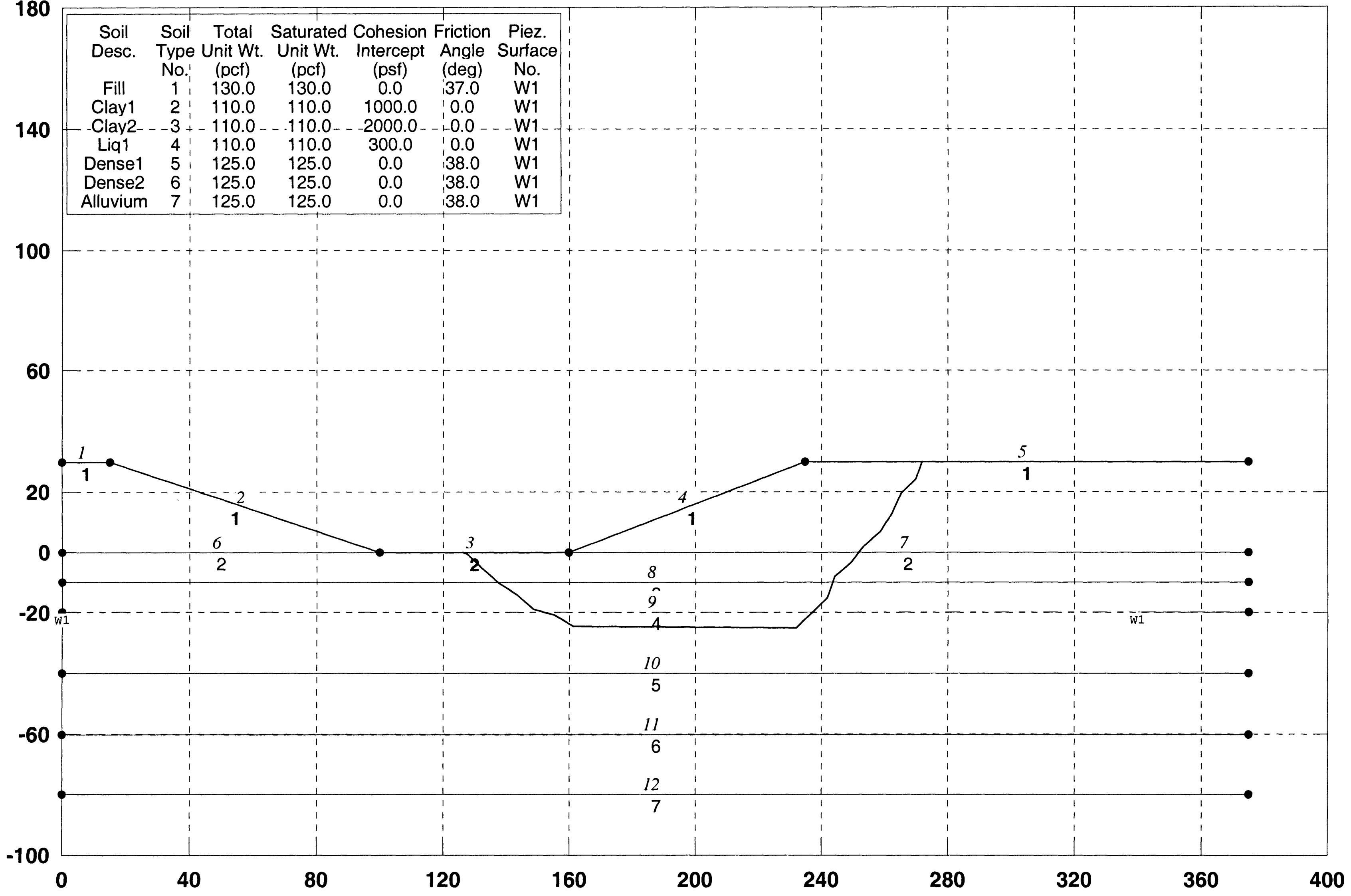


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.200 g<

NCHRP 1249 MISS, constrain in upper 5' allow past toe, kh=0.0, Janbu

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\POSTLI~1\CONSTR~1\MCFTJ.PLT Run By: Jen Schaeffer, CH2M HILL 09/25/2000 2:51PM



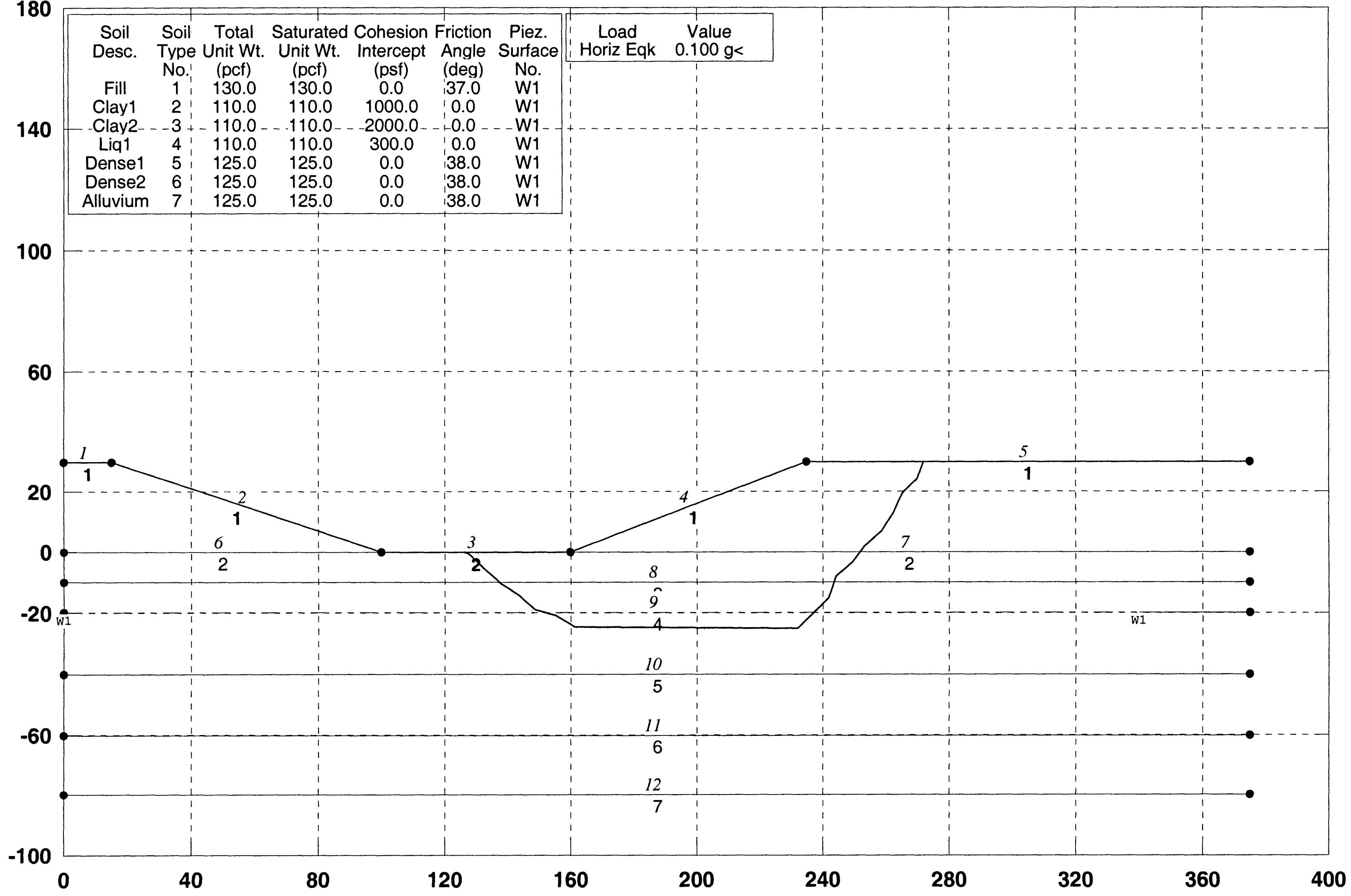
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=1.44

Factors of Safety Calculated by Janbu Method

NCHRP 1249 MISS, constrain in upper 5' allow past toe, kh=0.1, Janbu

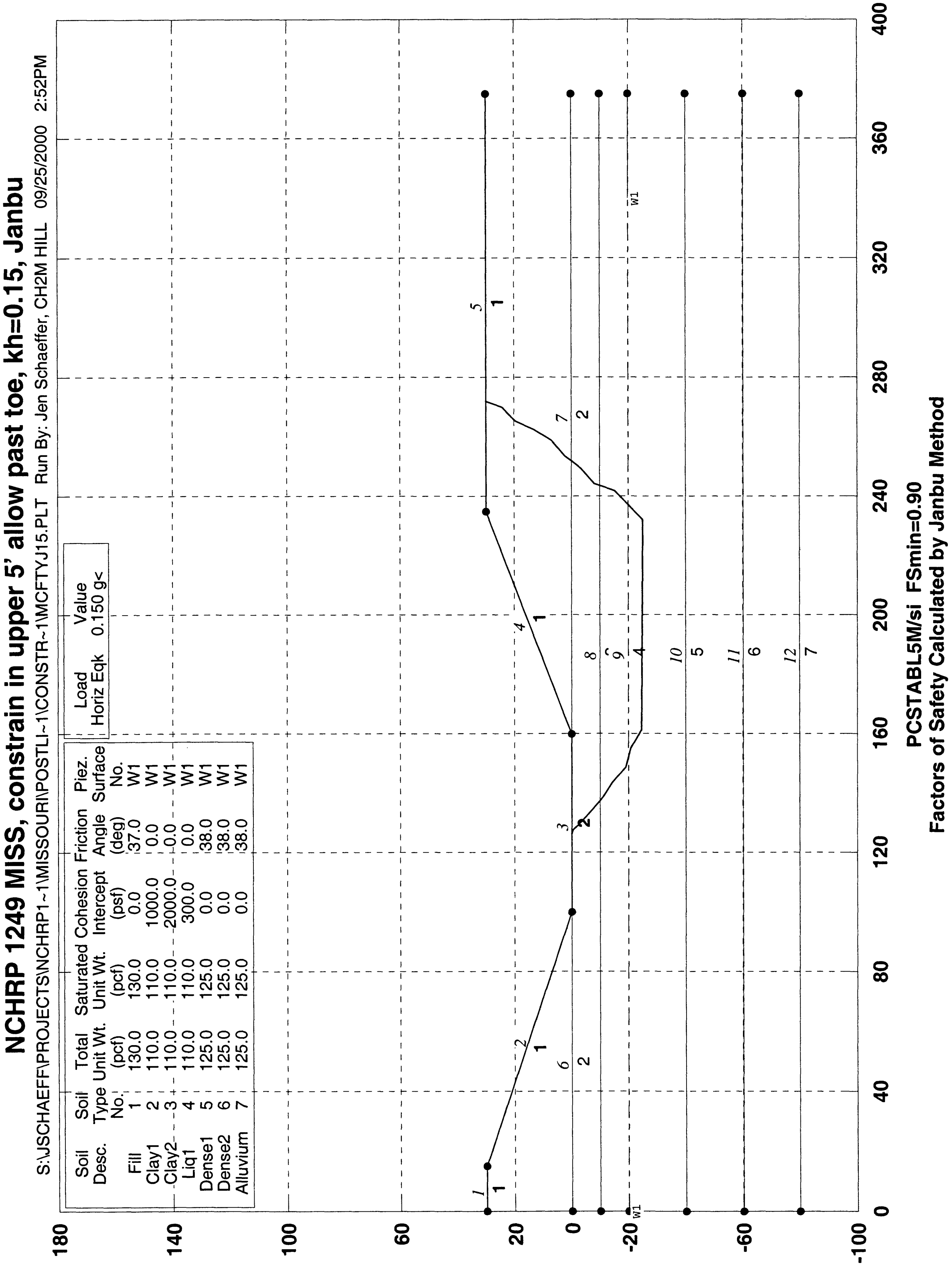
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\POSTLI~1\CONSTR~1\MCFTYJ1.PLT Run By: Jen Schaeffer, CH2M HILL 09/25/2000 2:48PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=1.04
Factors of Safety Calculated by Janbu Method



NCHRP 1249 MISS, constrain in upper 5' allow past toe, kh=0.15, Janbu

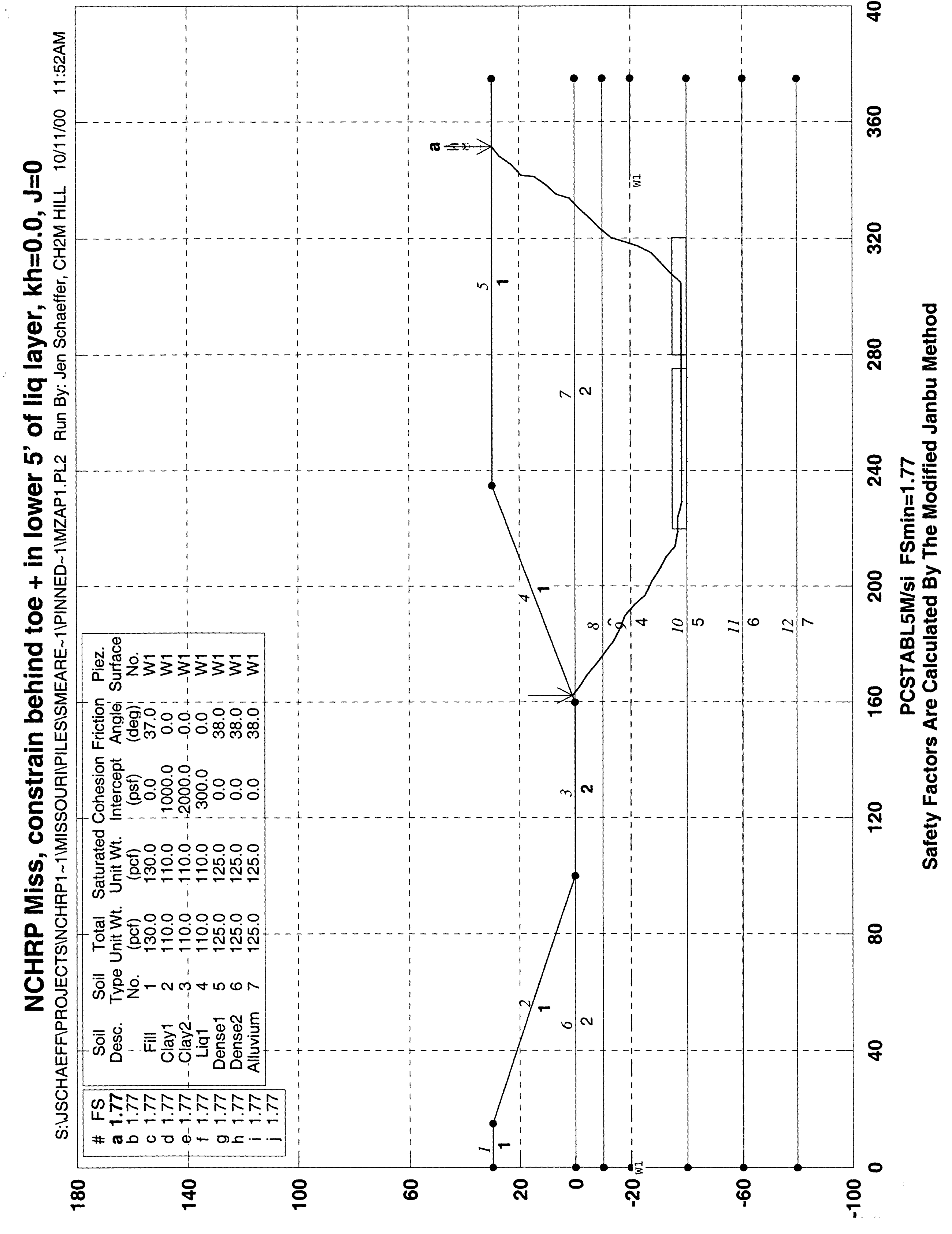
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\POSTLI~1\CONSTR~1\MCFTYJ15.PLT Run By: Jen Schaeffer, CH2M HILL 09/25/2000 2:52PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.150 g<

PCSTABL5M/si FSmin=0.90

Factors of Safety Calculated by Janbu Method



NCHRP Miss, constrain behind toe + in lower 5' of liq layer, kh=0.0, J=0

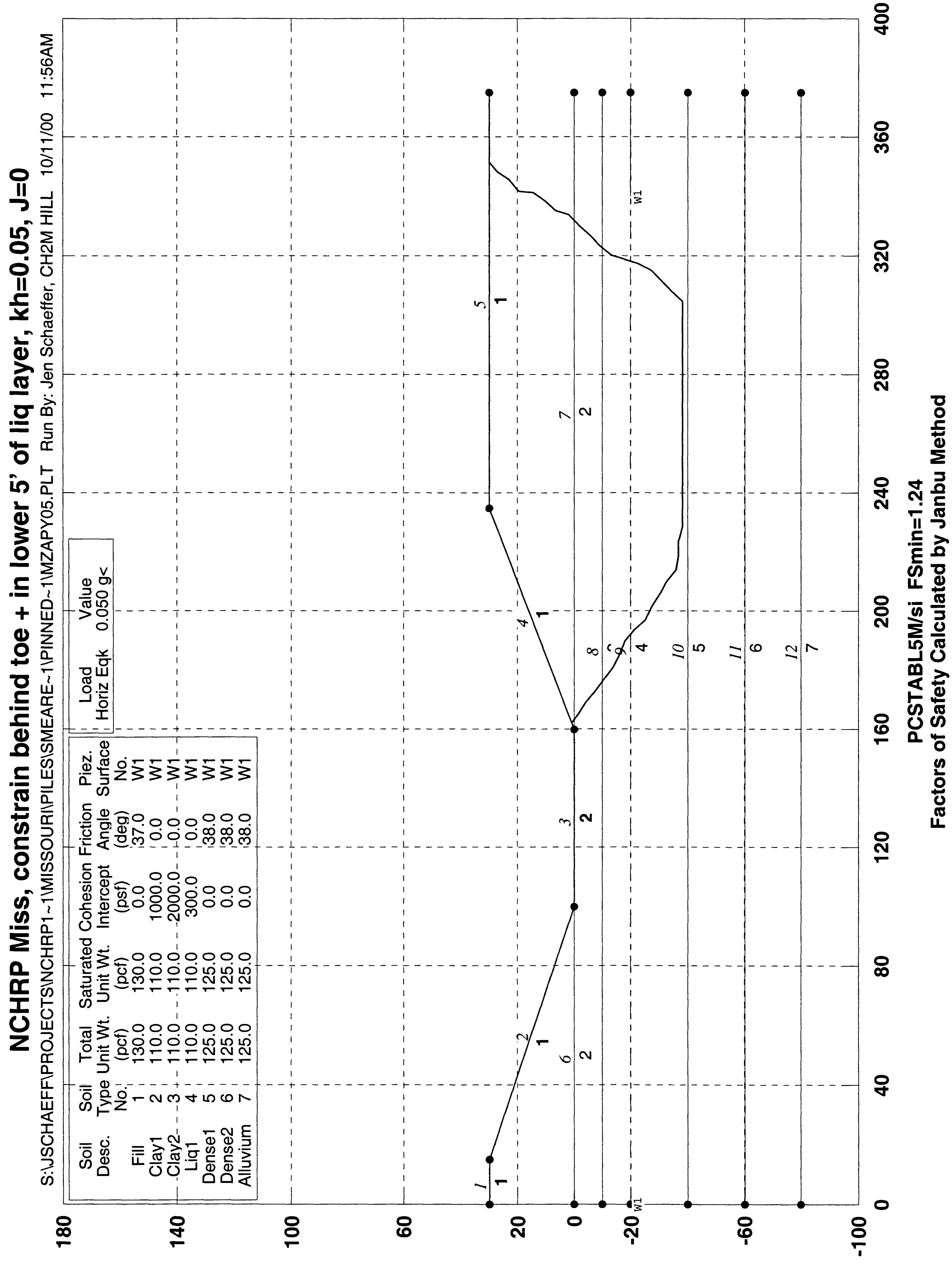
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAP1.PL2 Run By: Jen Schaeffer, CH2M HILL 10/11/00 11:52AM

#	FS
a	1.77
b	1.77
c	1.77
d	1.77
e	1.77
f	1.77
g	1.77
h	1.77
i	1.77
j	1.77

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=1.77

Safety Factors Are Calculated By The Modified Janbu Method



NCHRP Miss, constrain behind toe + in lower 5' of liq layer, kh=0.05, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAPY05.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 11:56AM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

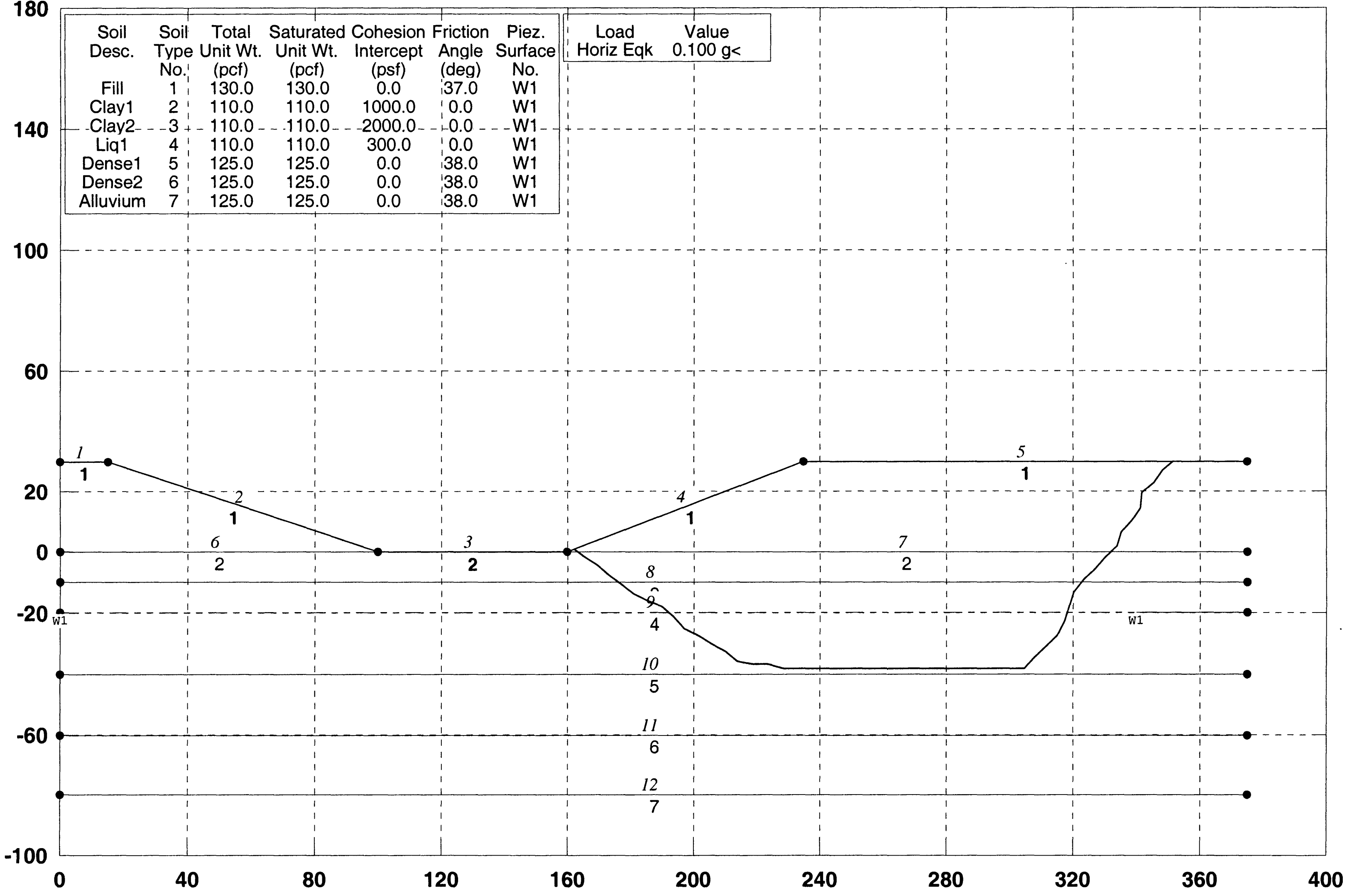
Load	Value
Horiz Eqk	0.050 g<

PCSTABL5M/si FSmin=1.24

Factors of Safety Calculated by Janbu Method

NCHRP Miss, constrain behind toe + in lower 5' of liq layer, kh=0.1, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAPY1.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 11:55AM



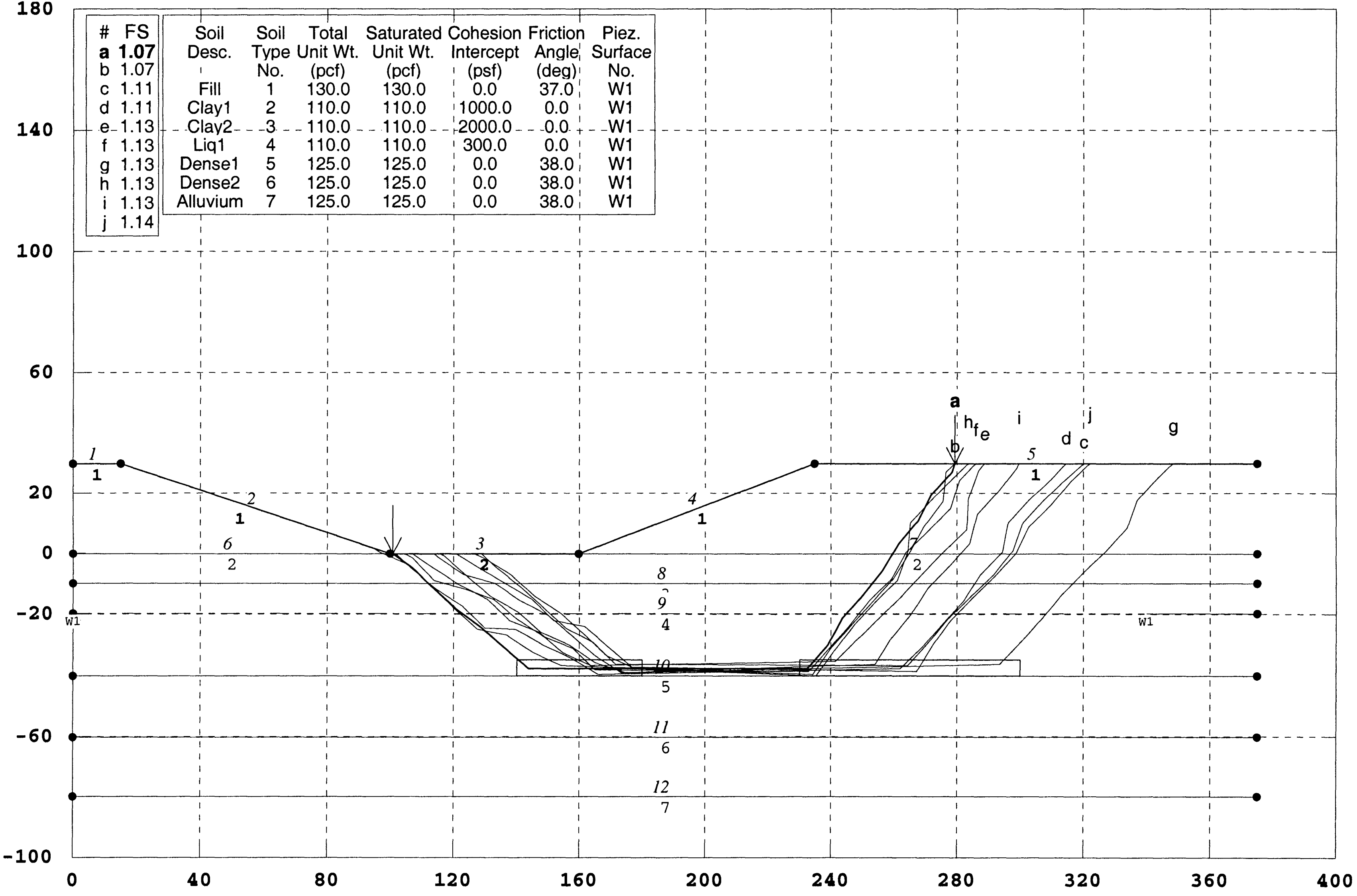
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=0.94
Factors of Safety Calculated by Janbu Method

NCHRP Miss, front toe + lower 5' of liq layer, kh=0.0, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MHAPP1.PL2 Run By: Jen Schaeffer, CH2M HILL 10/16/2000 7:10PM



#	FS
a	**1.07**
b	1.07
c	1.11
d	1.11
e	1.13
f	1.13
g	1.13
h	1.13
i	1.13
j	1.14

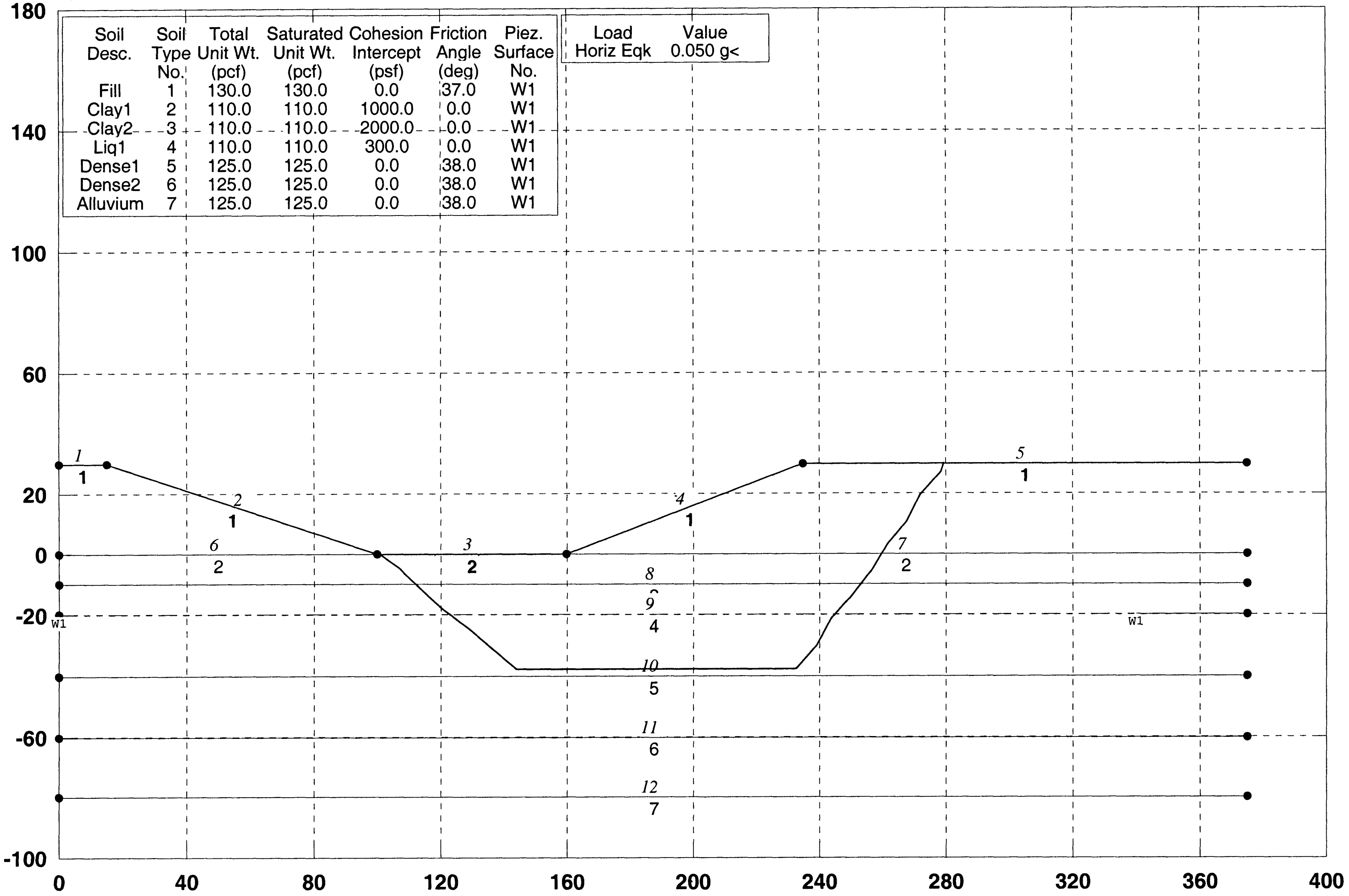
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=1.07

Safety Factors Are Calculated By The Modified Janbu Method

NCHRP Miss, front toe + lower 5' of liq layer, kh=0.05, J=0

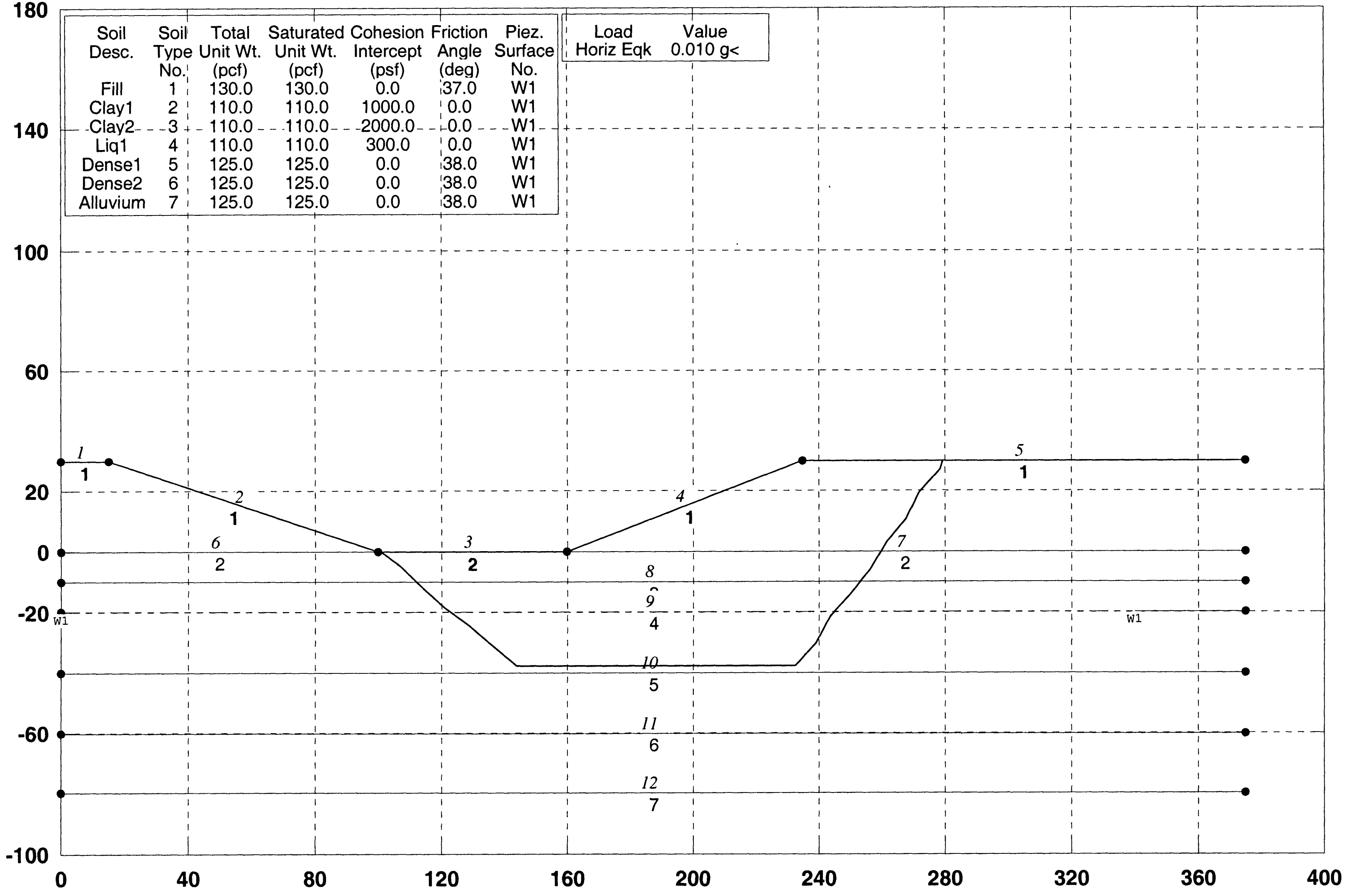
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MHAPP1Y5.PLT Run By: Jen Schaeffer, CH2M HILL 10/16/2000 7:09PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.050 g<

PCSTABL5M/si FSmin=0.88
Factors of Safety Calculated by Janbu Method



NCHRP Miss, front toe + lower 5' of liq layer, kh=0.01, J=0

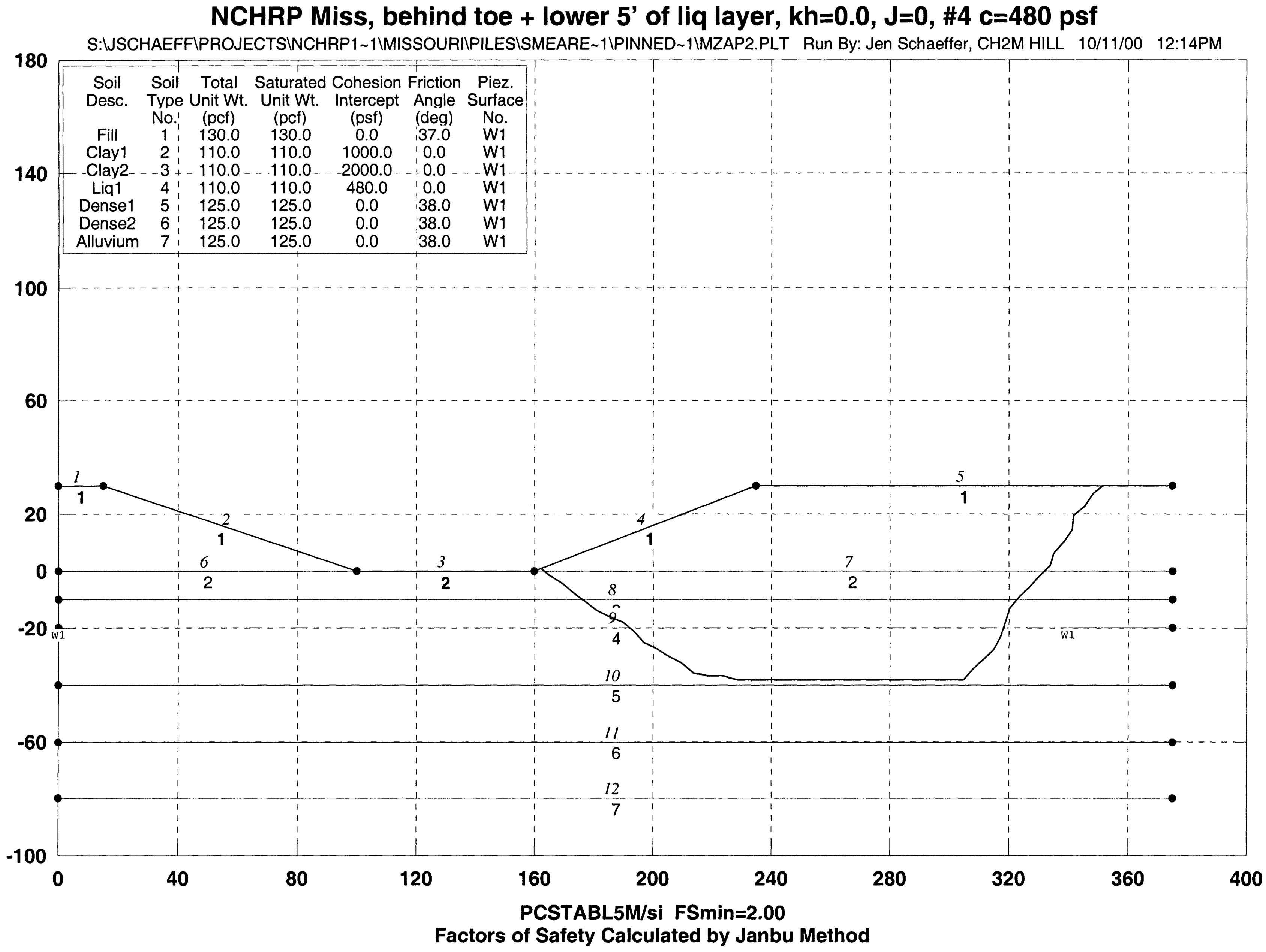
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MHAPP1Y1.PLT Run By: Jen Schaeffer, CH2M HILL 10/16/2000 7:09PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	300.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.010 g<

PCSTABL5M/si FSmin=1.03
Factors of Safety Calculated by Janbu Method

F.4.3 Missouri Stability Analyses: Pile/Structure Pinning Effects



NCHRP Miss, behind toe + lower 5' of liq layer, kh=0.0, J=0, #4 c=480 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAP2.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 12:14PM

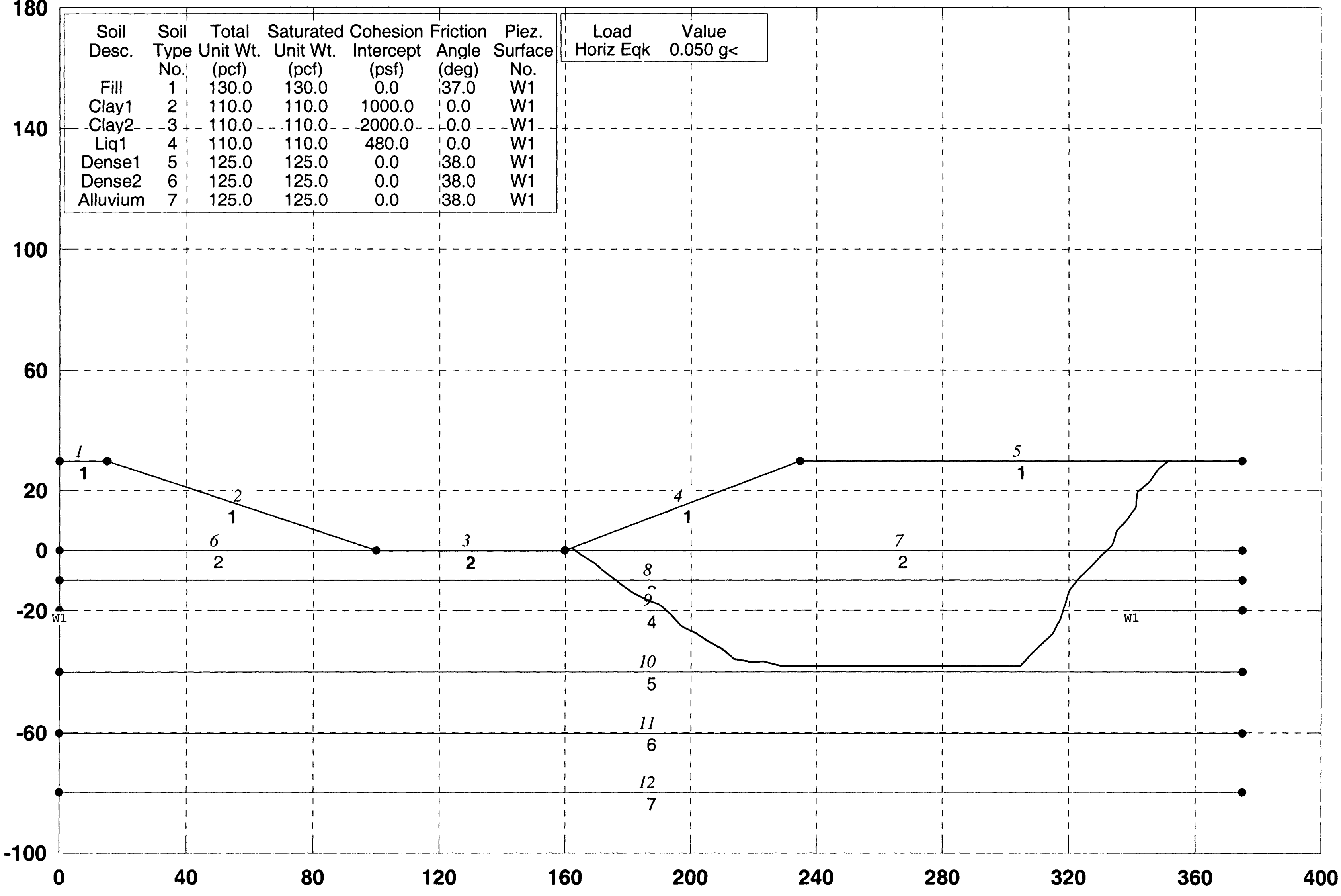
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	480.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=2.00

Factors of Safety Calculated by Janbu Method

NCHRP Miss, behind toe + lower 5' of liq layer, kh=0.05, J=0, #4 c=480 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAP2Y05.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 12:14PM



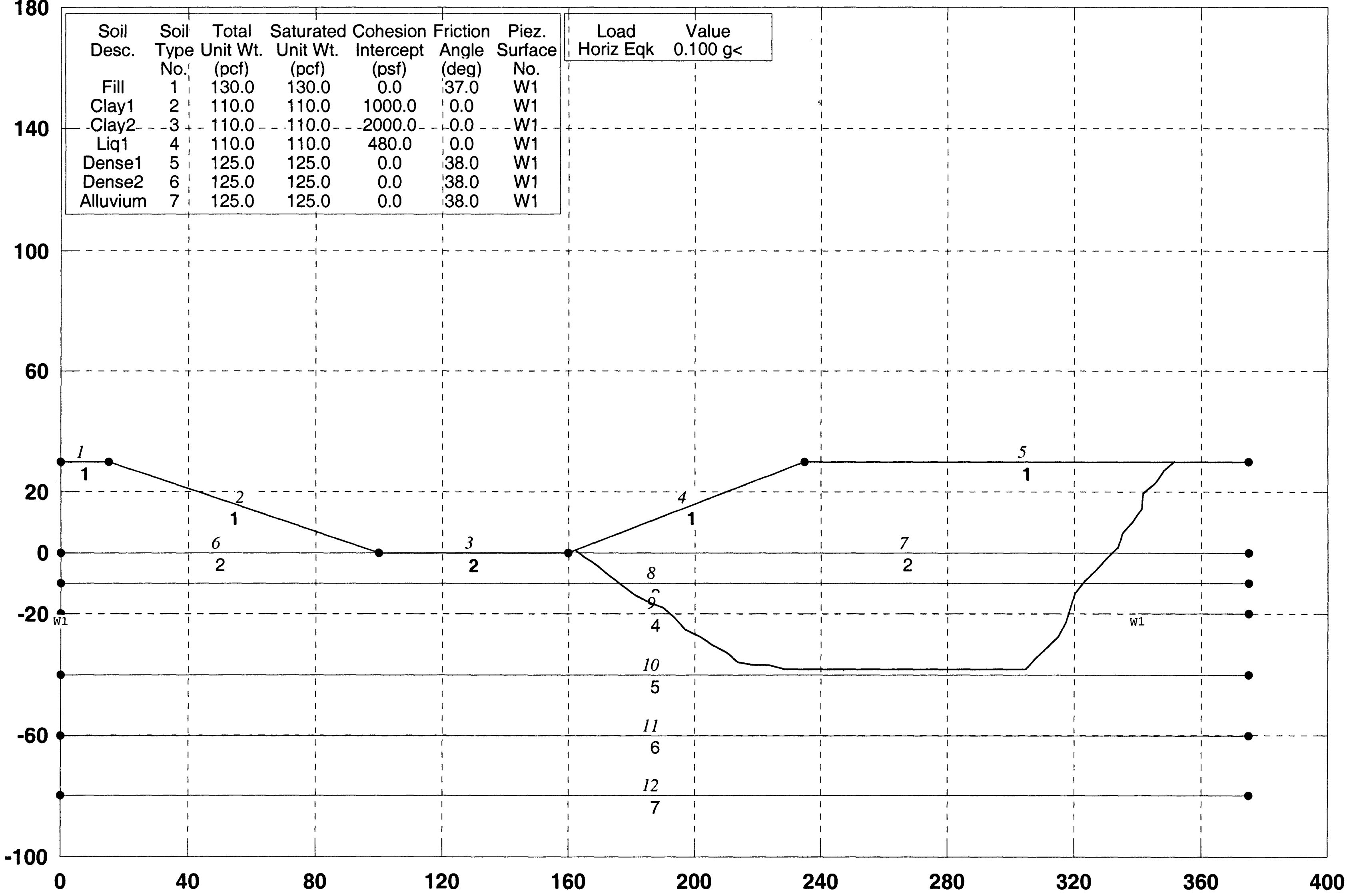
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	480.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.050 g<

PCSTABL5M/si FSmin=1.41
Factors of Safety Calculated by Janbu Method

NCHRP Miss, behind toe + lower 5' of liq layer, kh=0.1, J=0, #4 c=480 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAP2Y1.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 12:16PM



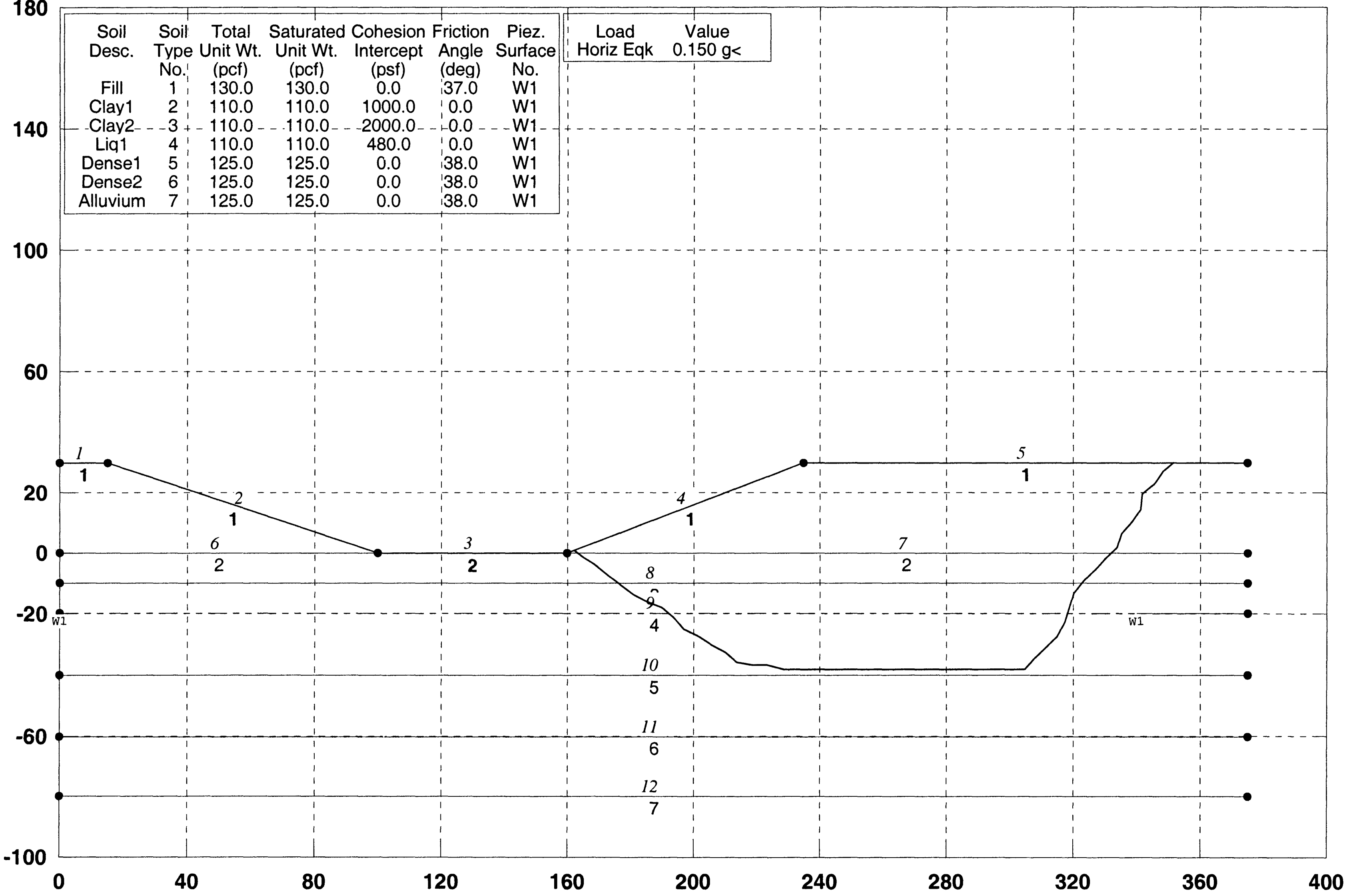
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	480.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=1.08
Factors of Safety Calculated by Janbu Method

NCHRP Miss, behind toe + lower 5' of liq layer, kh=0.15, J=0, #4 c=480 psf

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MZAP2Y15.PLT Run By: Jen Schaeffer, CH2M HILL 10/11/00 12:17PM



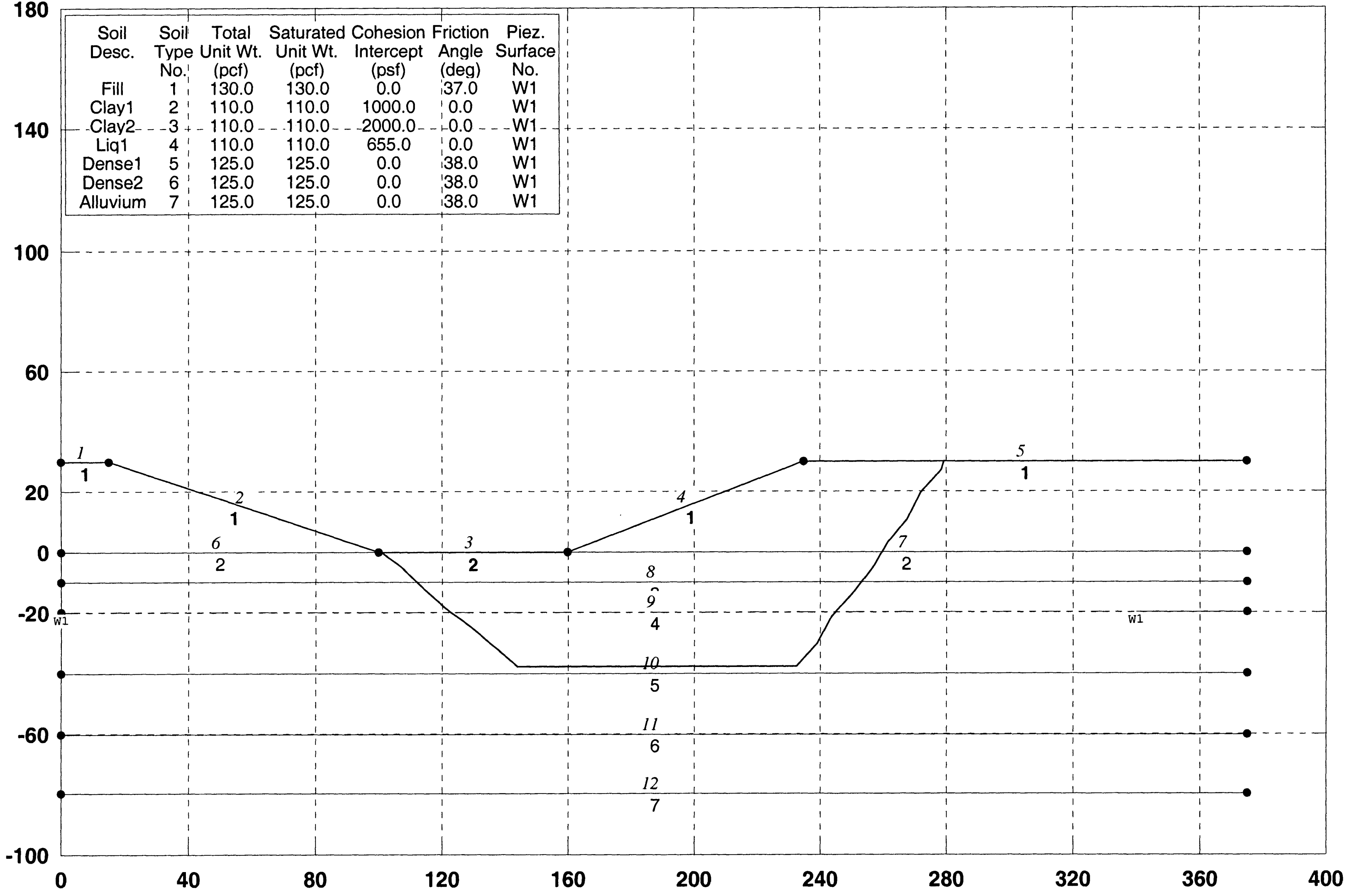
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	480.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.150 g<

PCSTABL5M/si FSmin=0.87
Factors of Safety Calculated by Janbu Method

NCHRP Miss, front toe + lower 5' of liq layer, kh=0.0, J=0, a&p pin, #4c=655

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MHAPP2.PLT Run By: Jen Schaeffer, CH2M HILL 10/16/2000 7:13PM



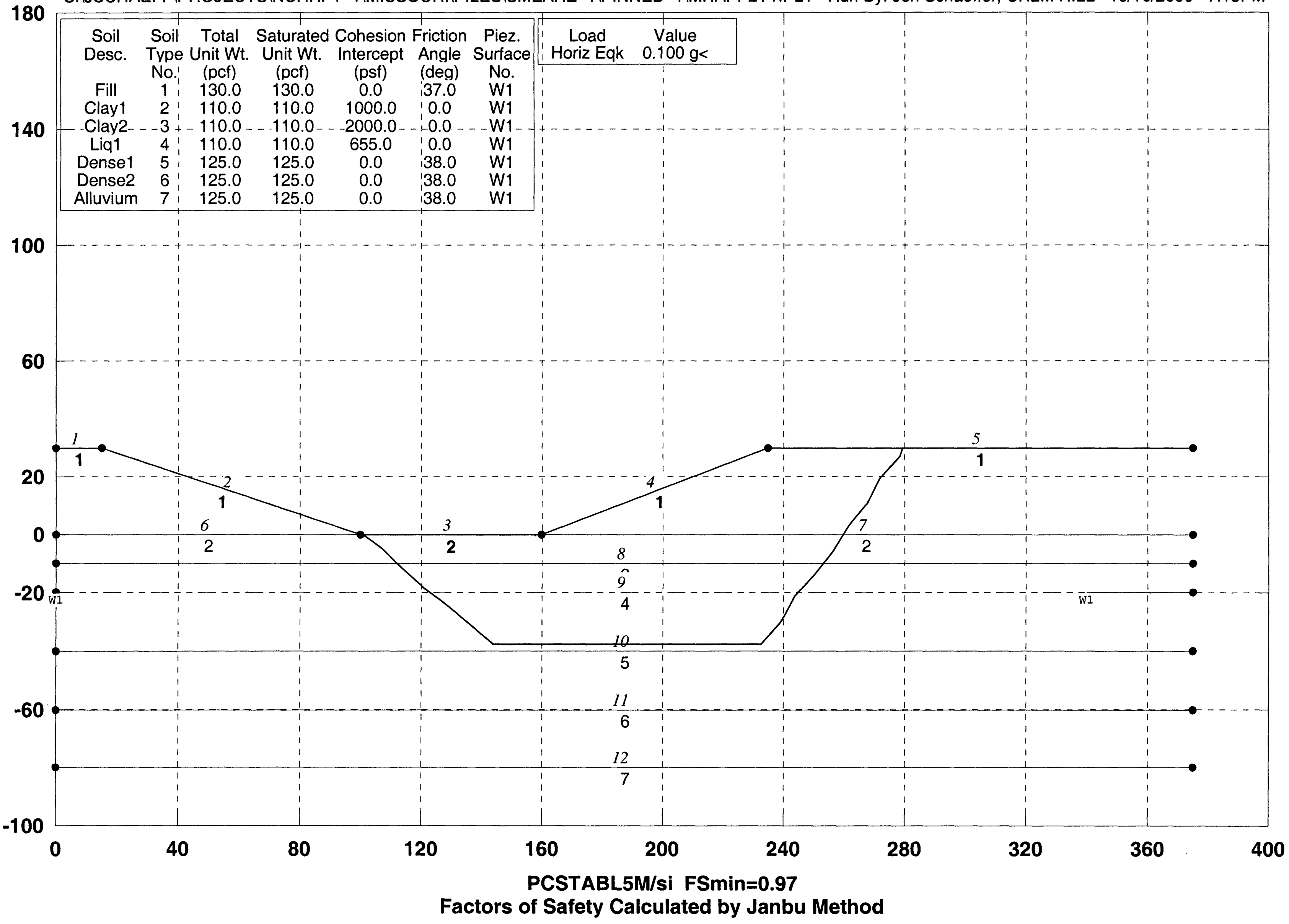
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	655.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=1.39

Factors of Safety Calculated by Janbu Method

NCHRP Miss, front toe + lower 5' of liq layer, kh=0.1, J=0, a&p pin, #4c=655

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MHAPP2Y1.PLT Run By: Jen Schaeffer, CH2M HILL 10/16/2000 7:13PM

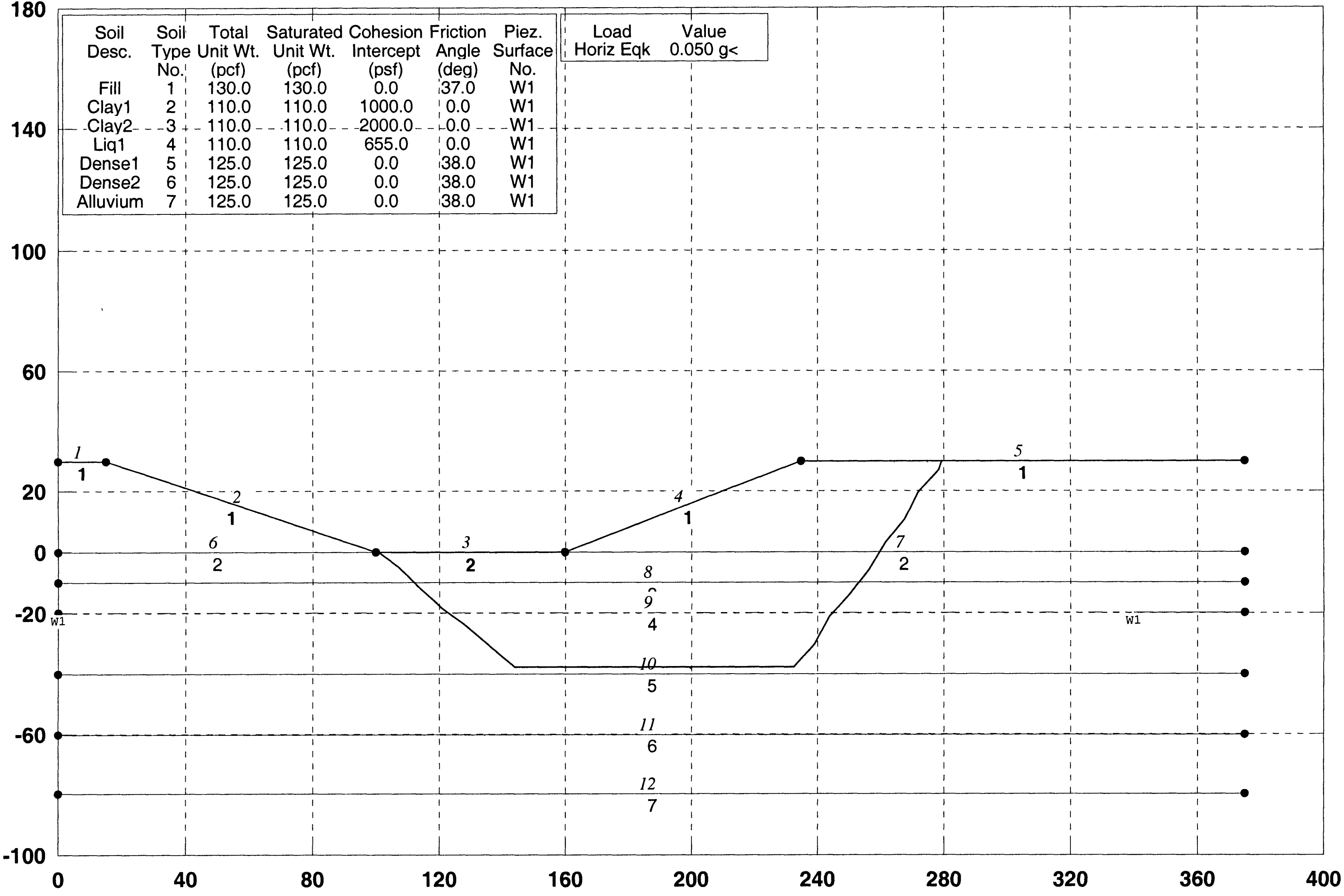


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	655.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

NCHRP Miss, front toe + lower 5' of liq layer, kh=0.05, J=0, a&p pin, #4c=655

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\PILES\SMEARE~1\PINNED~1\MHAPP2Y5.PLT Run By: Jen Schaeffer, CH2M HILL 10/16/2000 7:14PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	655.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

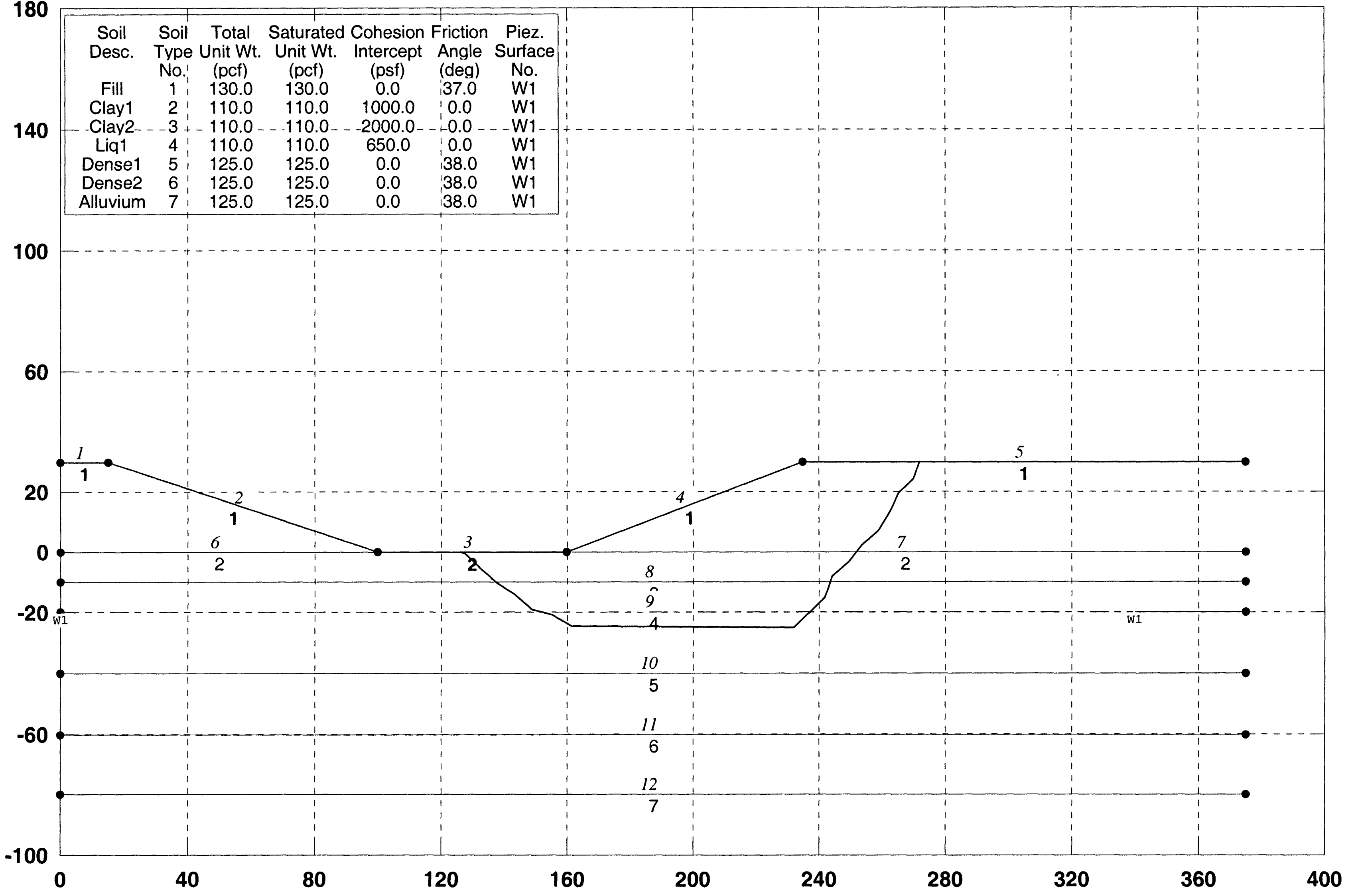
Load	Value
Horiz Eqk	0.050 g<

PCSTABL5M/si FSmin=1.14
Factors of Safety Calculated by Janbu Method

F.4.4 Missouri Stability Analyses: Ground Improvement Effects

NCHRP MISS, constrained wedge, 10' GI, comp=650 psf, kh=0.0, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW10R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:41PM

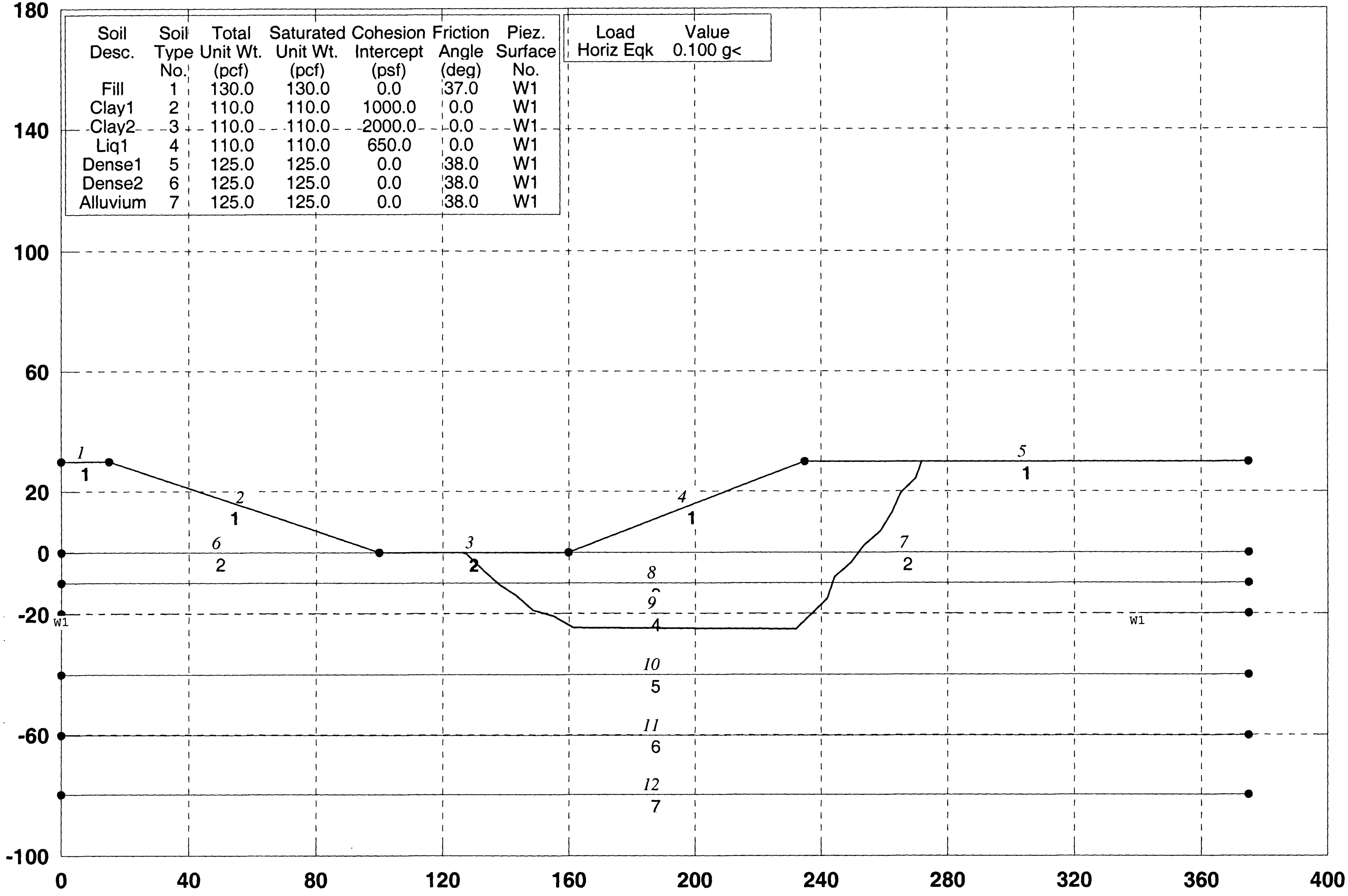


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	650.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=1.67
Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 10' GI, comp=650 psf, kh=0.1, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW1Y1R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:41PM



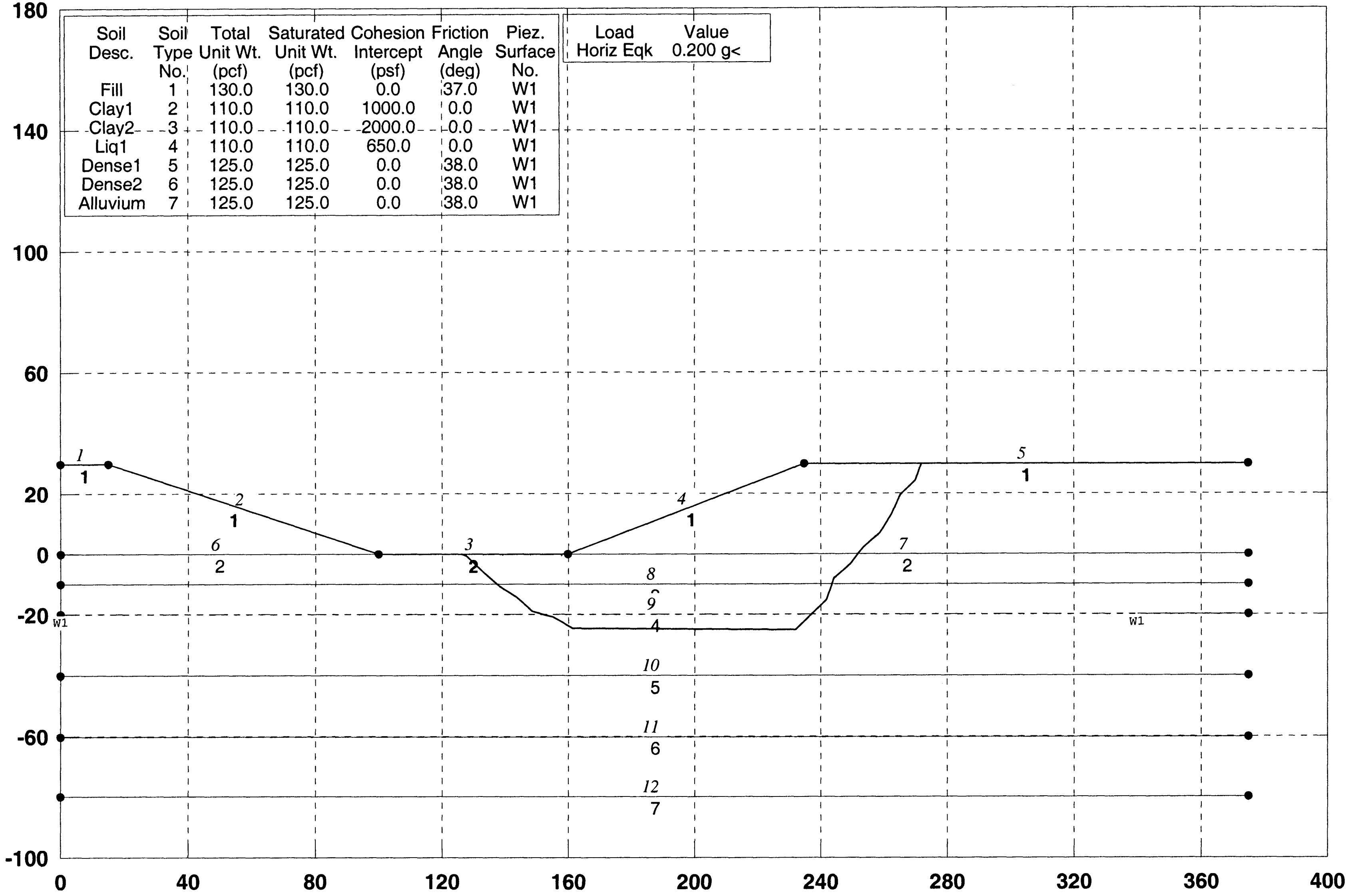
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	650.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=1.21
Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 10' GI, comp=650 psf, kh=0.2, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW1Y2R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:42PM



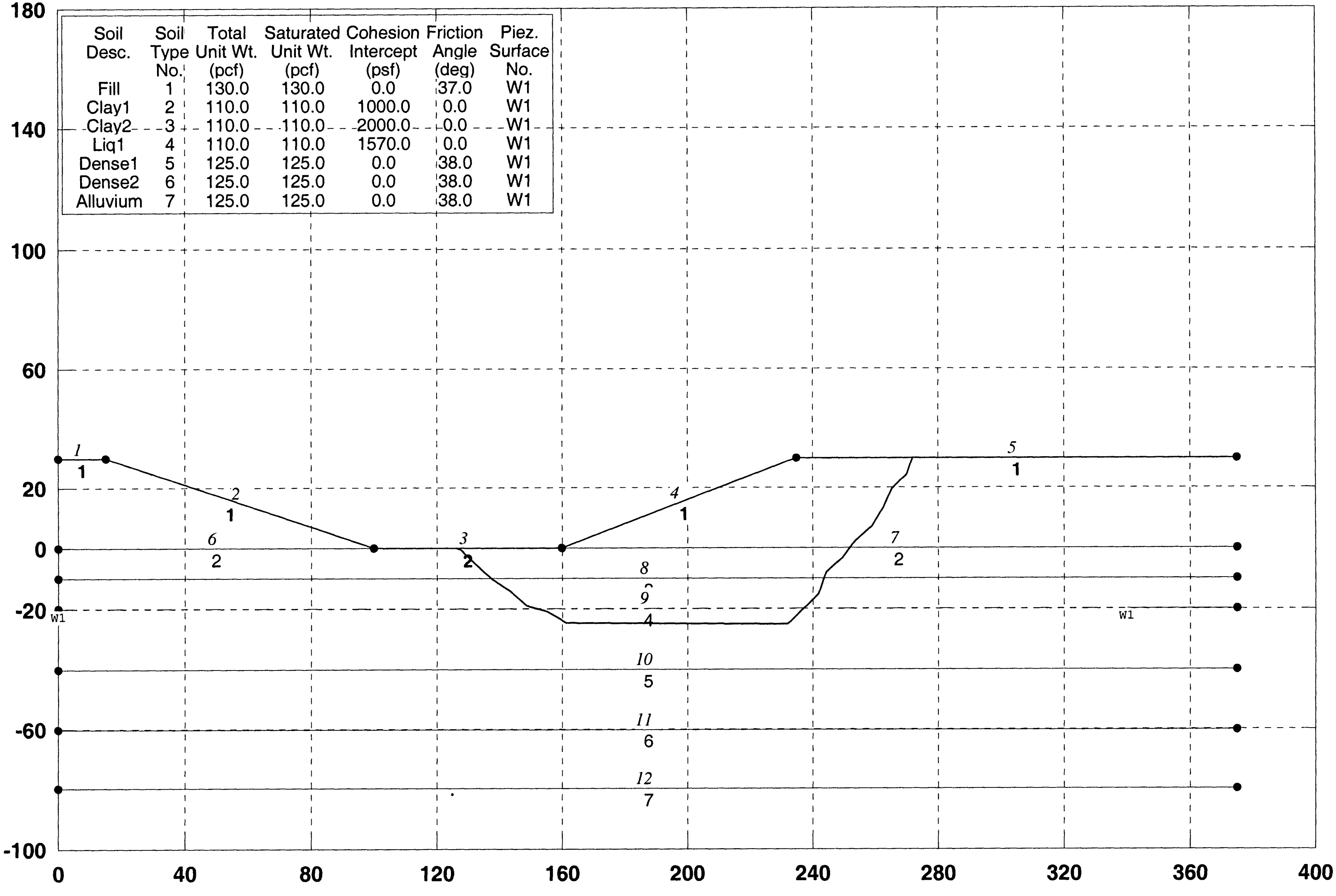
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	650.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.200 g<

PCSTABL5M/si FSmin=0.94
Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 30' GI, comp=1570 psf, kh=0.0, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW30R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:24PM

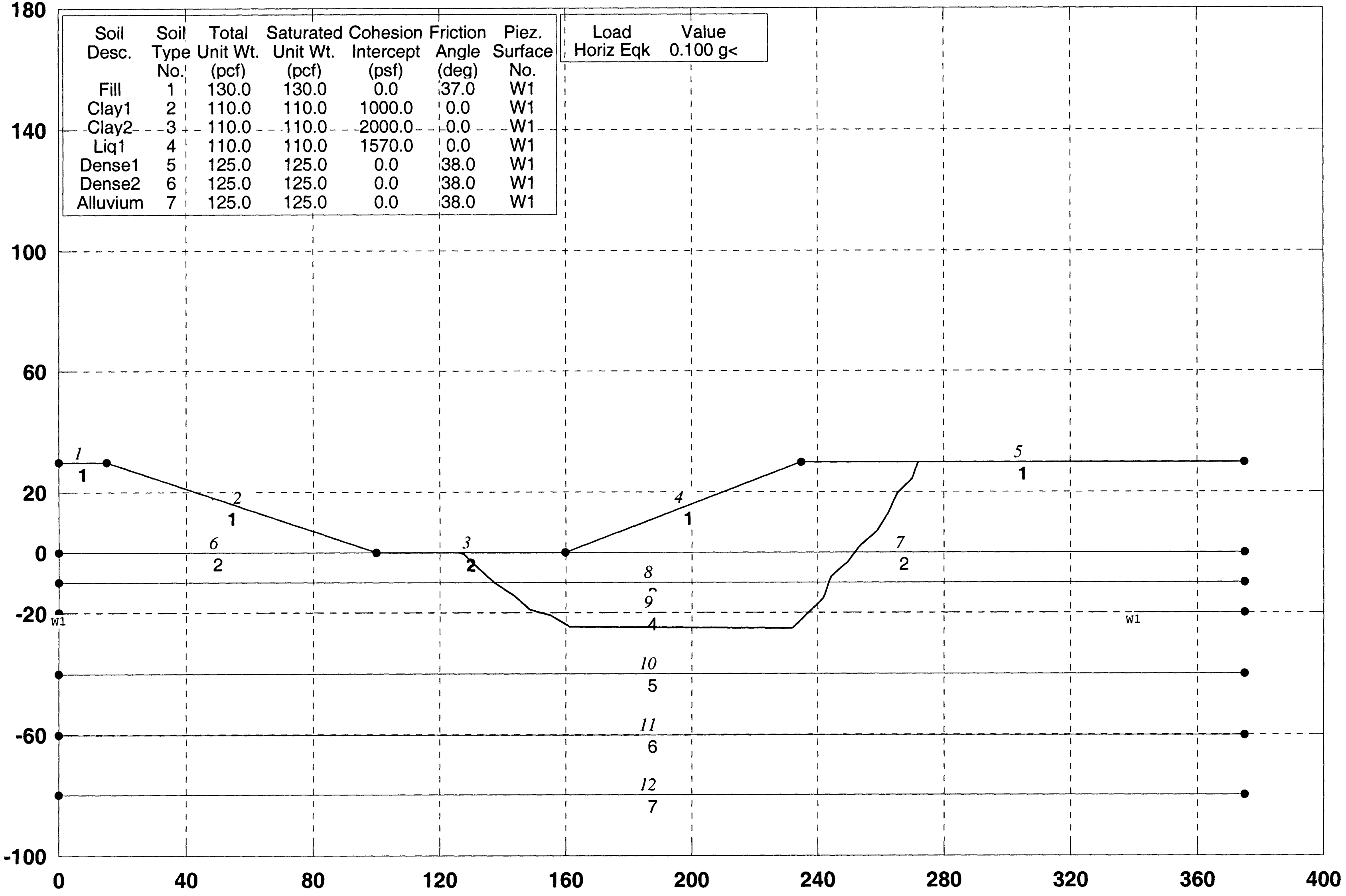


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	1570.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=2.27
Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 30' GI, comp=1570 psf, kh=0.1, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW3Y1R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:26PM

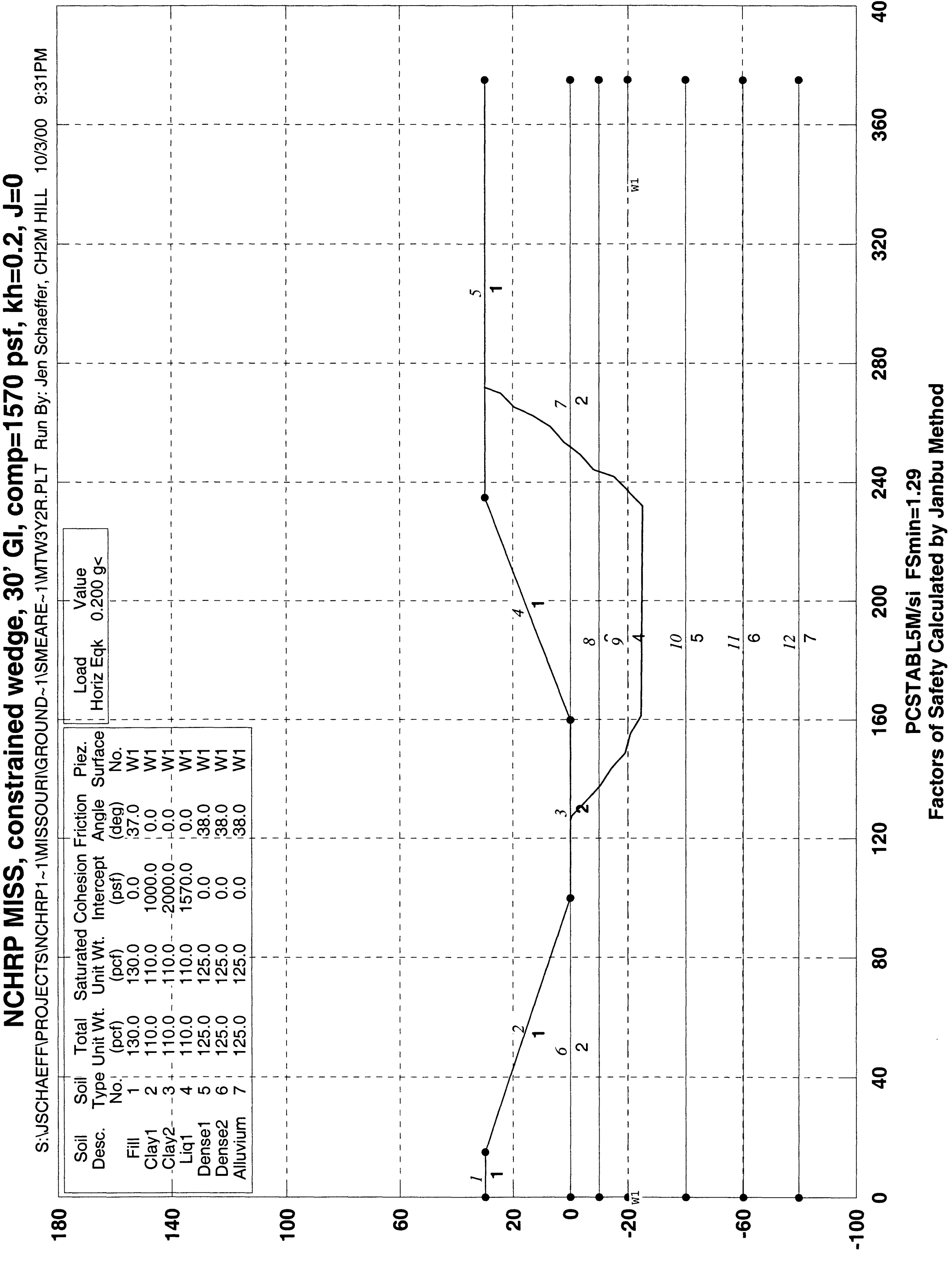


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	1570.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.100 g<

PCSTABL5M/si FSmin=1.65

Factors of Safety Calculated by Janbu Method



NCHRP MISS, constrained wedge, 30' GI, comp=1570 psf, kh=0.2, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW3Y2R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:31PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	1570.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

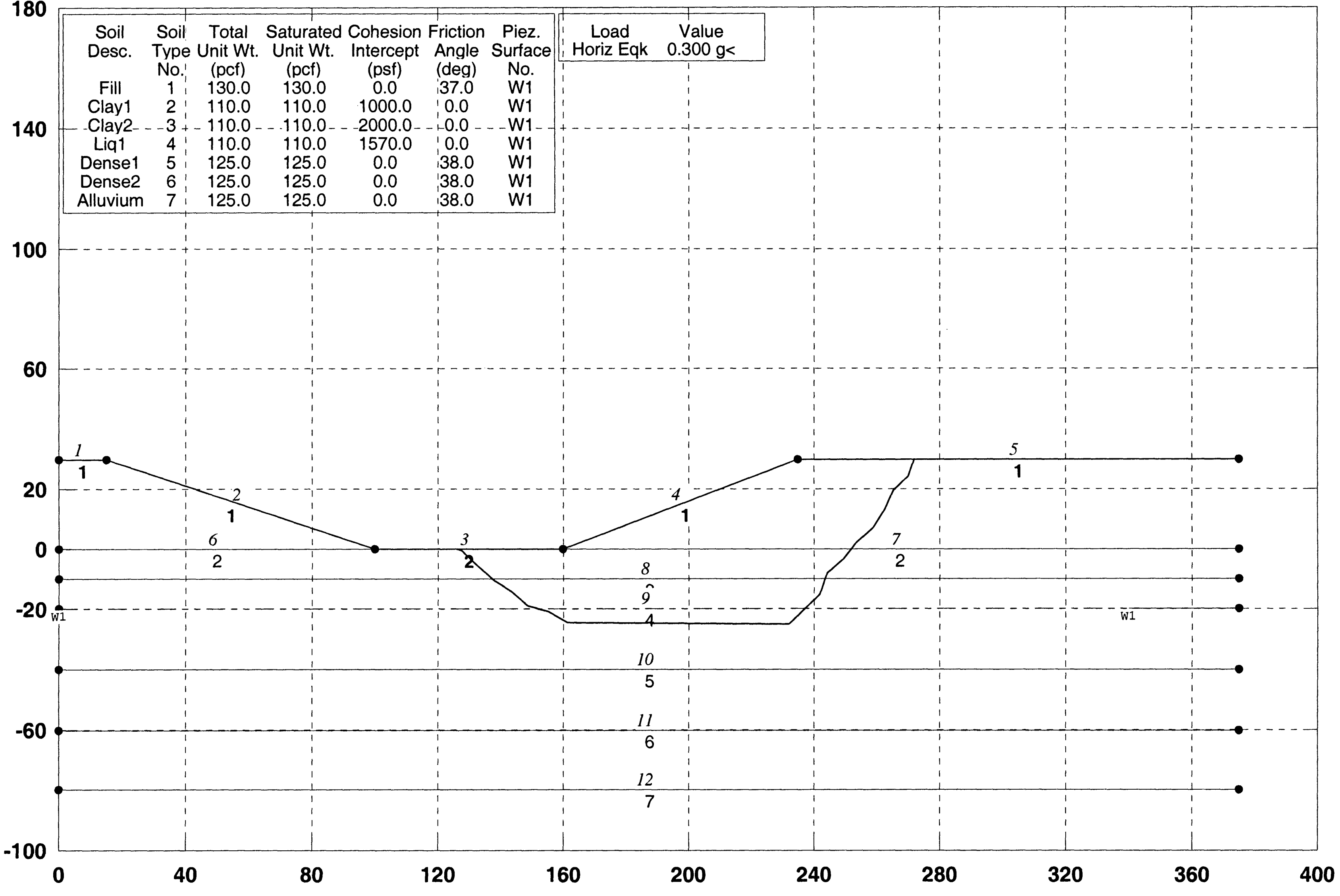
Load	Value
Horiz Eqk	0.200 g<

PCSTABL5M/si FSmin=1.29

Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 30' GI, comp=1570 psf, kh=0.3, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW3Y3R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:32PM

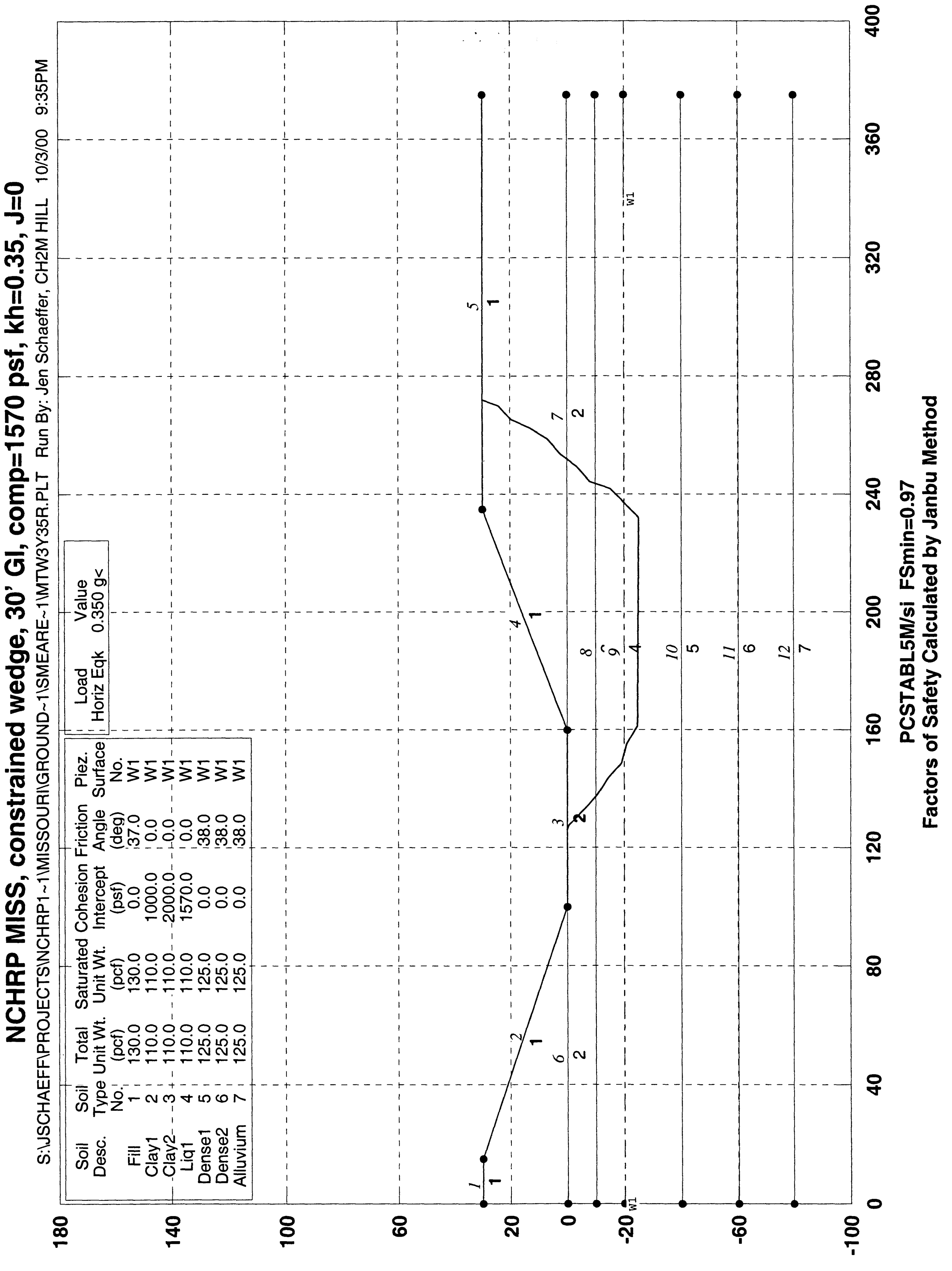


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	1570.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.300 g<

PCSTABL5M/si FSmin=1.06

Factors of Safety Calculated by Janbu Method



NCHRP MISS, constrained wedge, 30' GI, comp=1570 psf, kh=0.35, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW3Y35R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:35PM

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	1570.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

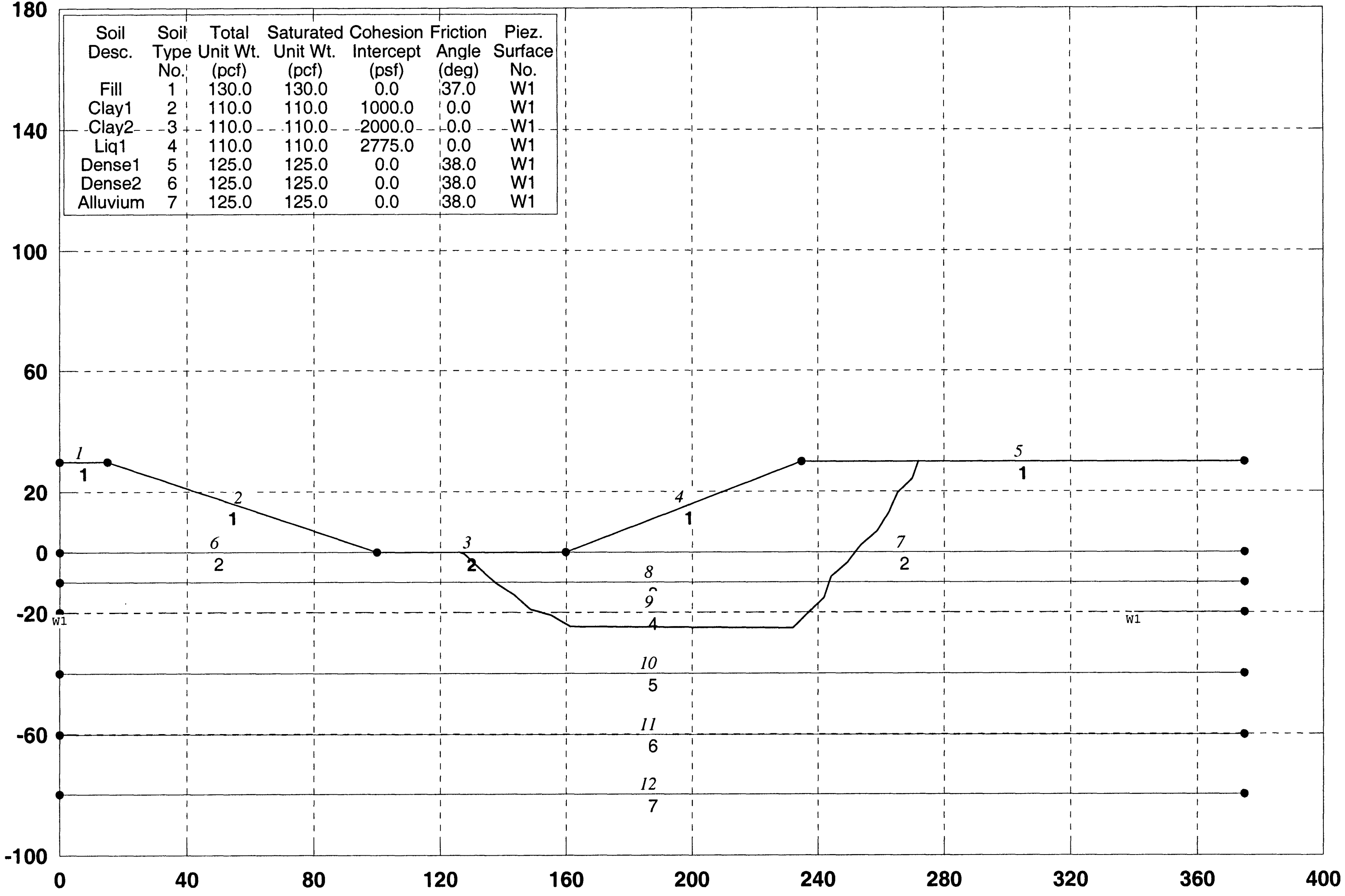
Load	Value
Horiz Eqk	0.350 g<

PCSTABL5M/si FSmin=0.97

Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 50' GI, comp=2775 psf, kh=0.0, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW50R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:44PM

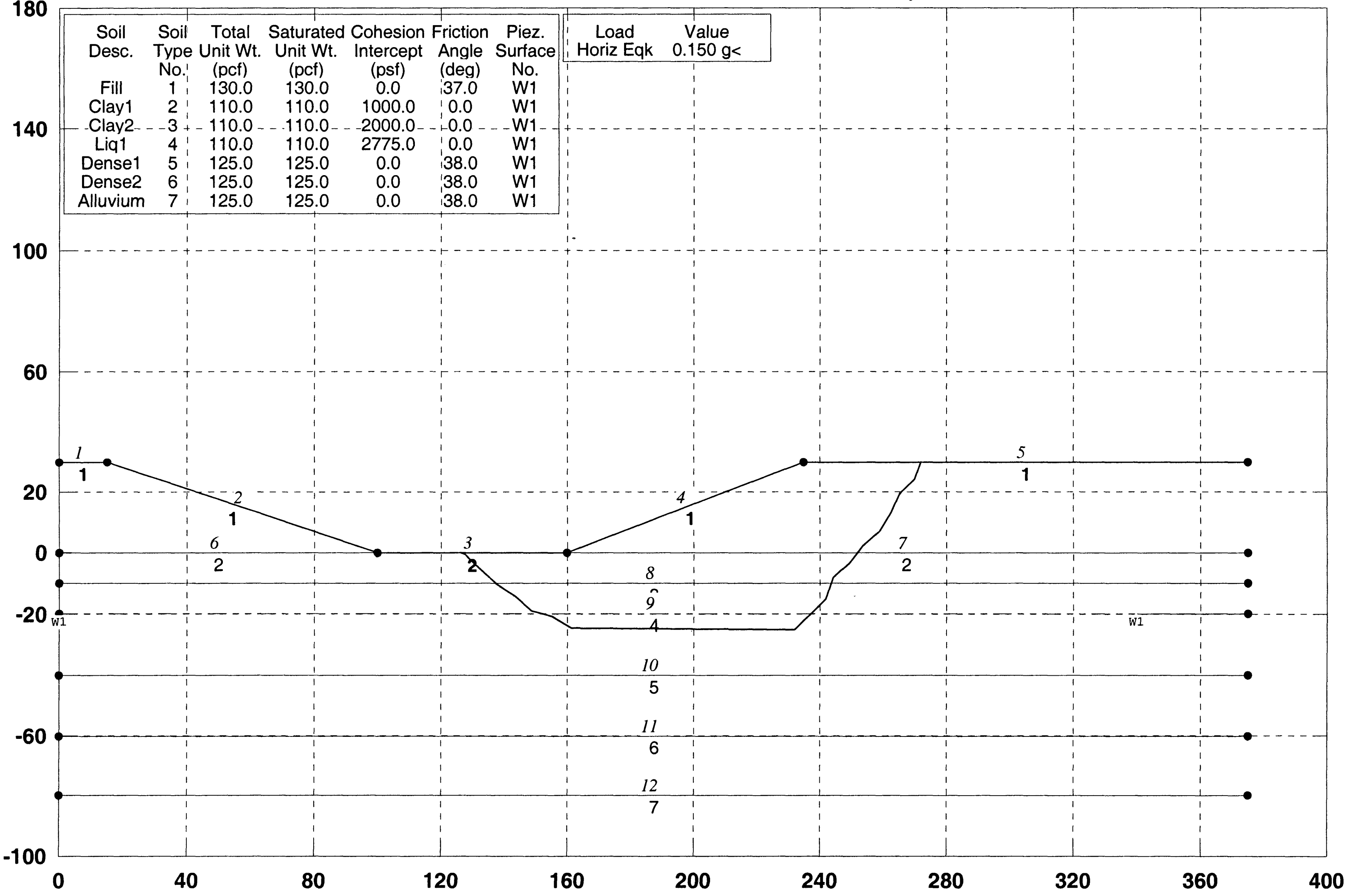


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	2775.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

PCSTABL5M/si FSmin=3.03
Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 50' GI, comp=2775 psf, kh=0.15, J=0

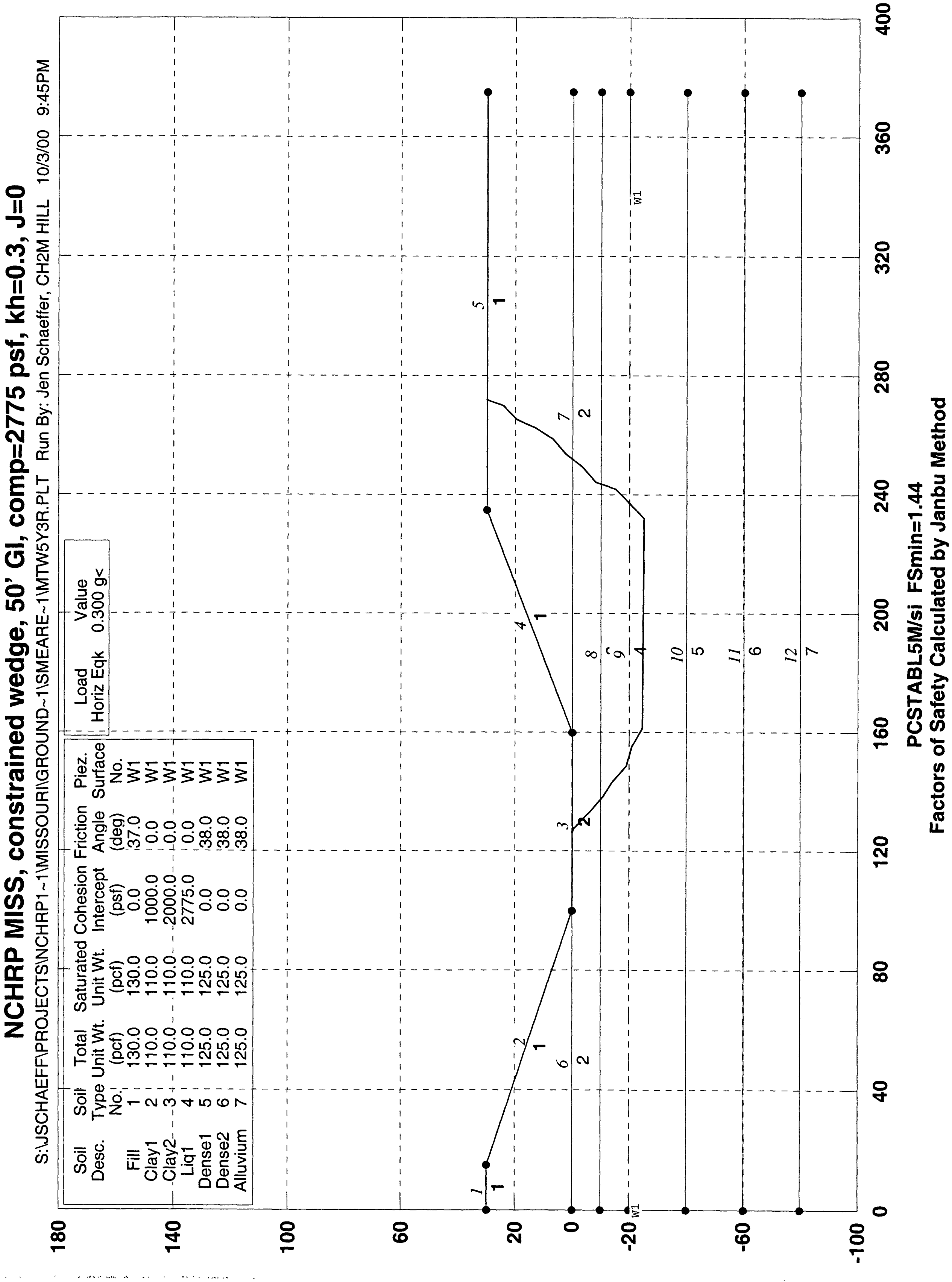
S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW5Y15R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:45PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	2775.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.150 g<

PCSTABL5M/si FSmin=1.96
Factors of Safety Calculated by Janbu Method

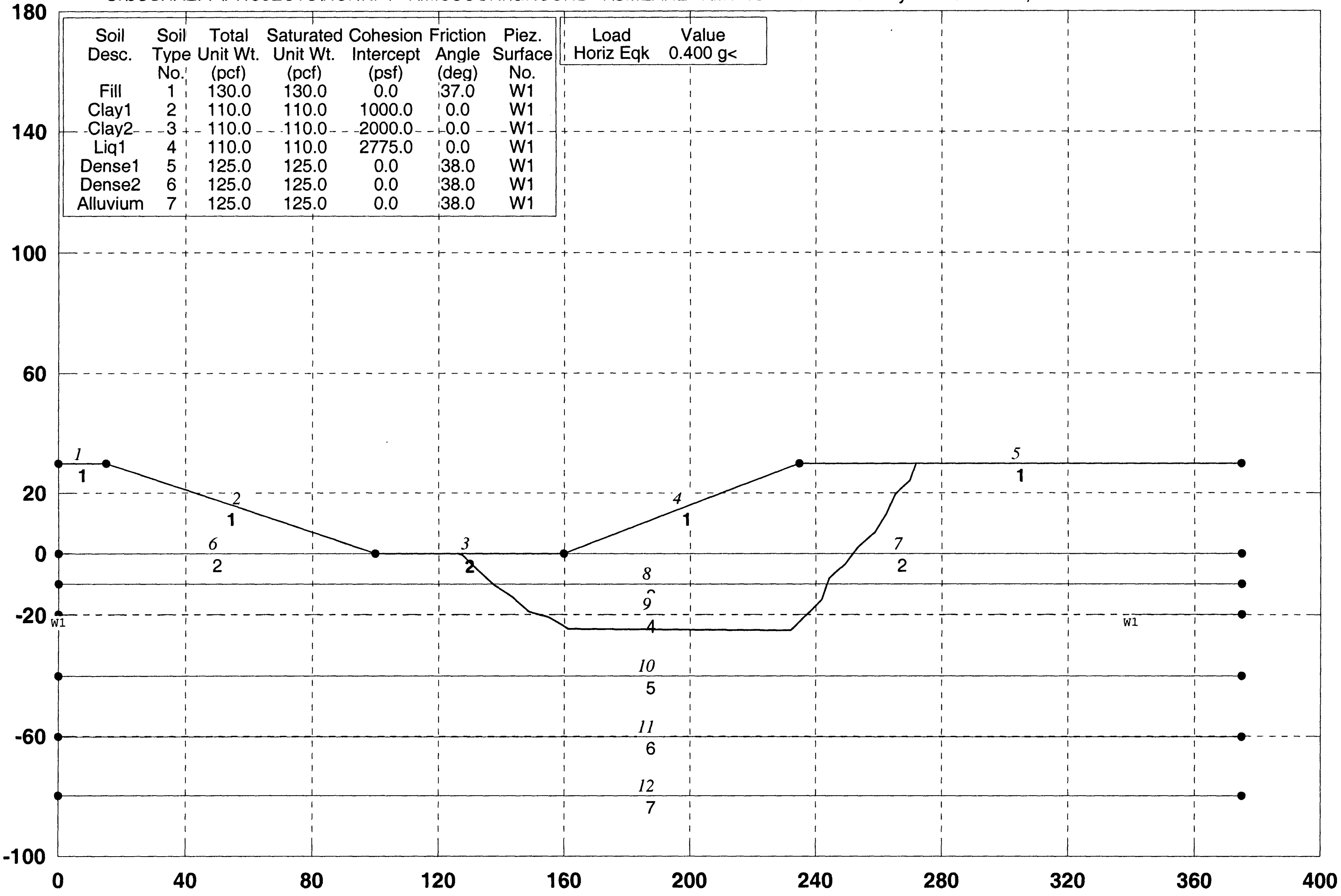


Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	2775.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.300 g<

NCHRP MISS, constrained wedge, 50' GI, comp=2775 psf, kh=0.4, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW5Y4R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:46PM



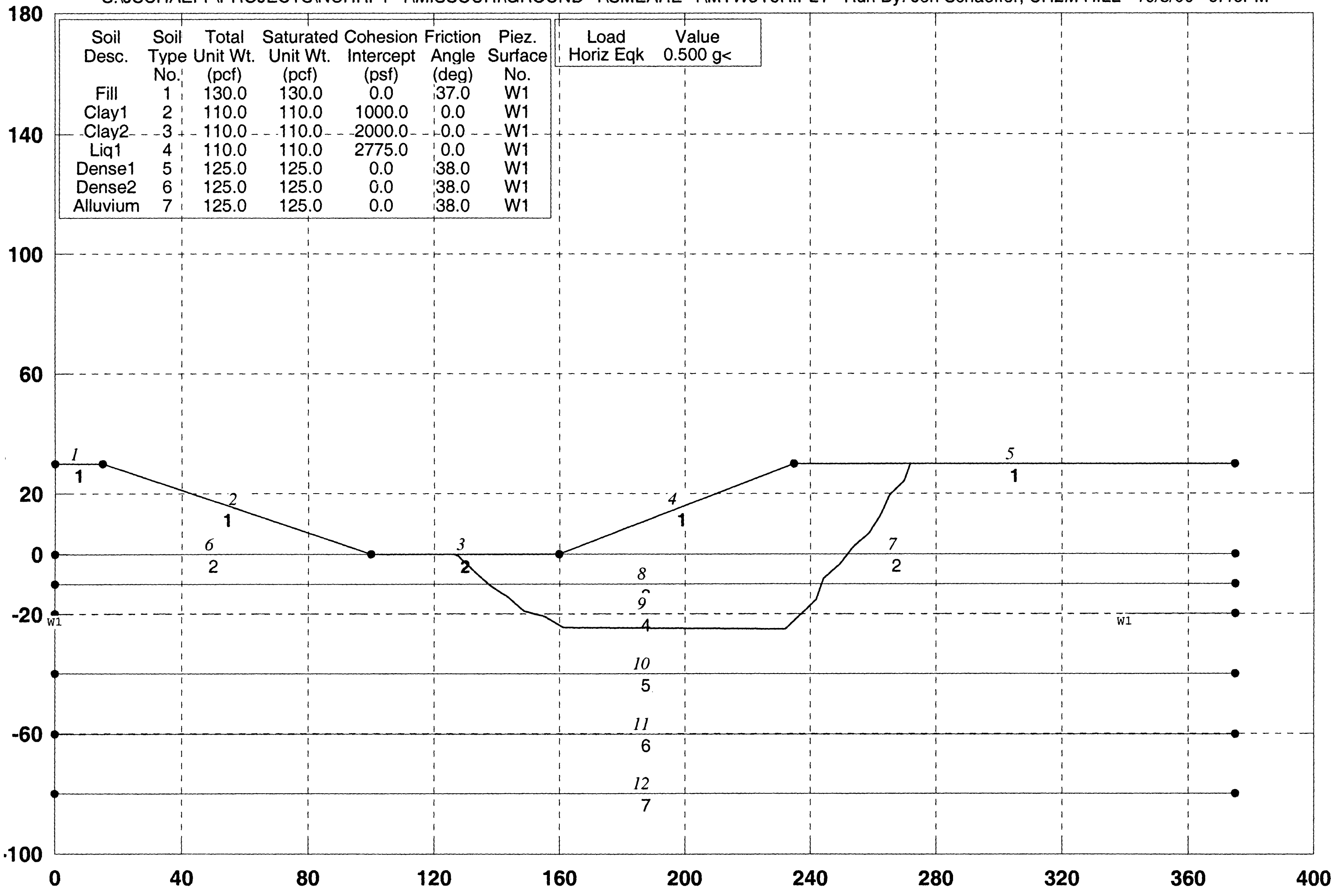
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	2775.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.400 g<

PCSTABL5M/si FSmin=1.22
Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 50' GI, comp=2775 psf, kh=0.5, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW5Y5R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:46PM



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	2775.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

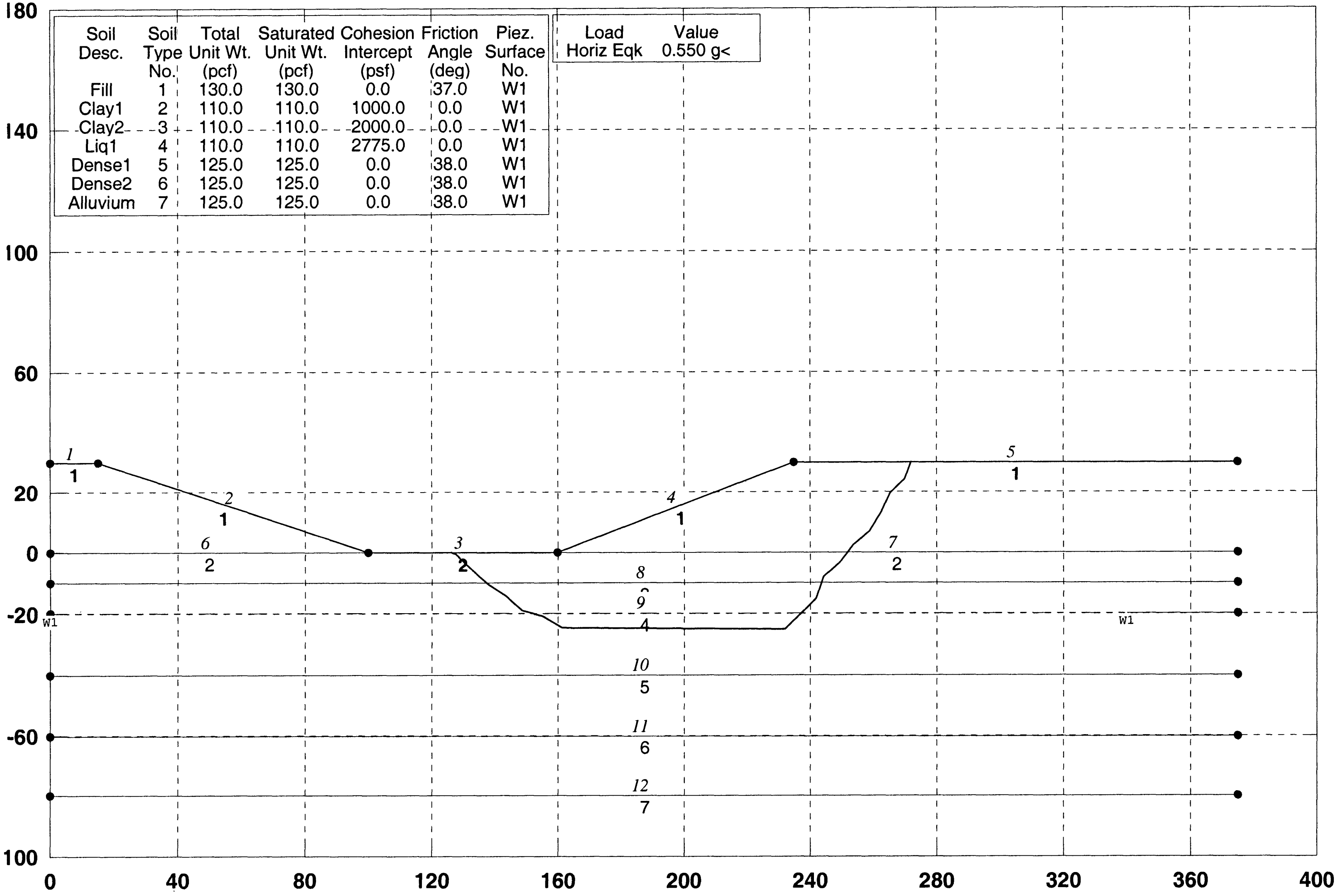
Load	Value
Horiz Eqk	0.500 g<

PCSTABL5M/si FSmin=1.06

Factors of Safety Calculated by Janbu Method

NCHRP MISS, constrained wedge, 50' GI, comp=2775 psf, kh=0.55, J=0

S:\JSCHAEFF\PROJECTS\NCHRP1~1\MISSOURI\GROUND~1\SMEARE~1\MTW5Y55R.PLT Run By: Jen Schaeffer, CH2M HILL 10/3/00 9:47PM



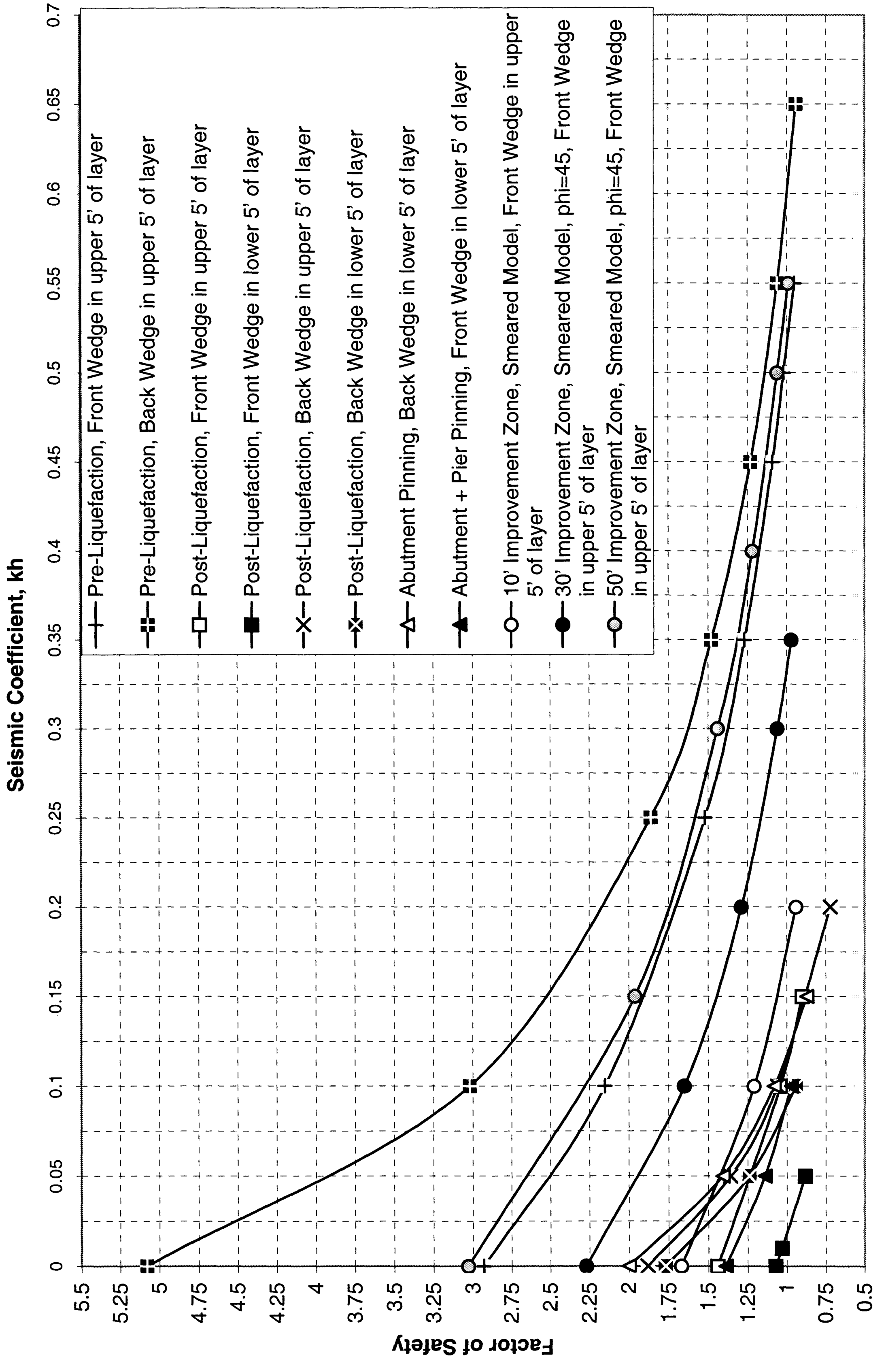
Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Fill	1	130.0	130.0	0.0	37.0	W1
Clay1	2	110.0	110.0	1000.0	0.0	W1
Clay2	3	110.0	110.0	2000.0	0.0	W1
Liq1	4	110.0	110.0	2775.0	0.0	W1
Dense1	5	125.0	125.0	0.0	38.0	W1
Dense2	6	125.0	125.0	0.0	38.0	W1
Alluvium	7	125.0	125.0	0.0	38.0	W1

Load	Value
Horiz Eqk	0.550 g<

PCSTABL5M/si FSmin=0.99

Factors of Safety Calculated by Janbu Method

F.4.5 Missouri Stability Analyses: Yield Acceleration Comparisons



Yield Acceleration - Missouri Location

MISSOURI Location - Comparison of Analyses

NCHRP 12-49 Project

The data annotated here are shown in the "Yield Acceleration" Chart

General Notes:

1. Names shown in brackets [MCFT] refer to the anlaysis file name for reference.
2. Wedge Designations:
   "Front" refers to a wedge that is allowed to daylight beyond the embankment toe [MCFT for upper 5', MHAPP1 for lower 5'].
   "Back" refers to a wedge that is constrained behind or just at the embankment toe [MCBT, MZAP1 for lower 5'].
3. All analyses performed with Janbu Simplified method, i.e. Janbu Coefficient = 0. Note that this is generally more conservative than a Spencer analysis on the same failure surface.

PRE- AND POST LIQUEFACTION CASES

Pre-Liquefaction, Front and Back Wedges

kh	Front FS	Back
0	2.93	5.08
0.1	2.15	3.02
0.25	1.52	1.86
0.35	1.27	1.48
0.45	1.09	1.23
0.5	1.02	xx
0.55	0.95	1.06
0.65	xx	0.94

Notes

1. Layering is the same as all Post Liquefaction cases (i.e., all other analyses). [MPFWO, FW1, FW25, etc., and MPBWO, BW1, BW25, etc.]
2. Pre-liquefaction strength in liquefied layer is phi=32.

Post-Liquefaction

Post-Liquefaction, Front and Back Wedges in Upper 5' of Layer

kh	Front FS	Back FS
0	1.44	1.88
0.05	xx	1.36
0.1	1.04	1.06
0.15	0.9	xx
0.2	xx	0.72

Notes

1. Strength of liquefied layer is c=300 psf.
2. [MCFTJ, MCFTYJ1, MCFTYJ15; MCBTJ, MCBTYJ05, MCBTYJ1, MCBTYJ2]
3. Wedges in upper 5' of liquefied layer.

Post-Liquefaction, Back Wedge in Lower 5' of Layer

kh	FS
0	1.77
0.05	1.24
0.1	0.94

Notes

1. Strength of liquefied layer is c=300 psf.
2. [MZAP1, PY05, PY1]
3. Wedges in lower 5' of liquefied layer.

Post-Liquefaction, Front Wedge in Lower 5' of Layer

kh	FS
0	1.07
0.01	1.03
0.05	0.88

Notes

1. Strength of liquefied layer is c=300 psf.
2. [MHAPP1, 1Y5, 1Y1]
3. Wedges in lower 5' of liquefied layer.

PILE CASES

Pinned Cases

"Pinned" analyses model the strength of the pile and any structural strut loads or embankment resistanct by smearing it over the failure surface. Pile analyses were run on two different cases: with abutment pinning only and for abutment and pier pinning.

Abutment Pinning, Back Wedge in Lower 5' of Layer

kh	FS
0	2
0.05	1.41
0.1	1.08
0.15	0.87

Notes

1. Failure surface length taken as 90'.
2. Strength determined by:
   a. Structural contribution = (piles/strut/reaction loads)/(failure surface length x pile group "width") (16k/ft)/(90') = 0.180 ksf = 180 psf
   b. Soil contribution = 300 psf (liquefied strength).
   c. COMBINED strength, c = 480 psf for liquefied soil.
3. [MZAP2, MZAP2Y05, Y1, Y15]

Abutment + Pier Pinning, Front Wedge in Lower 5' of Layer

kh	FS
0	1.39
0.05	1.14
0.1	0.97

Notes

1. Failure surface length taken as 100'.
2. Strength determined by:
   a. Structural contribution = (piles/strut/reaction loads)/(failure surface length x pile group "width") (32k/ft)/(90') = 0.355 ksf = 355 psf
   b. Soil contribution = 300 psf (liquefied strength).
   c. COMBINED strength, c = 655 psf for liquefied soil (upper layer).
3. [MHAPP2, 2Y5, 2Y1]

GROUND IMPROVEMENT CASES

All ground improvement cases run with "FRONT" wedge.

Smeared Models

"Smeared" analyses model the strength of the ground improvement zone by smearing it over the failure surface.

50' Improvement Zone, Smeared Model, phi=45, Front Wedge in Upper 5' of Layer

kh	FS
0	3.03
0.15	1.96
0.3	1.44
0.4	1.22
0.5	1.06
0.55	0.99

Notes

1. Soil strength determined as a weighted average over the failure surface (100').
2. Strength determined by:
   a. Liquefied soil strength x length in liquefied soil, 300 psf x 50'
   b. Improved zone strength x length in improved zone,
   (sig'v x tan 45) x 50', where improved gamma is used
   c. Total strength = total of above/100' = 2775 psf.
3. [MTW50R, 5Y15R, 5Y2R, 5Y3R, etc.]

30' Improvement Zone, Smeared Model, phi=45, Front Wedge in Upper 5' of Layer

kh	FS
0	2.27
0.1	1.65
0.2	1.29
0.3	1.06
0.35	0.97

Notes

1. Strength determined by:
   a. Liquefied soil strength x length in liquefied soil, 300 psf x 70'
   b. Improved zone strength x length in improved zone,
   (sig'v x tan 45) x 30', where improved gamma is used
   c. Total strength = total of above/100' = 1570 psf
2. [MTW30R, 3Y1R, 3Y2R, 3Y3R, etc.]

10' Improvement Zone, Smeared Model, phi=45, Front Wedge in Upper 5' of Layer

kh	FS
0	1.67
0.1	1.21
0.2	0.94

Notes

1. Strength determined by:
   a. Liquefied soil strength x length in liquefied soil, 300 psf x 35'
   b. Improved zone strength x length in improved zone,
   (sig'v x tan 45) x 70', where improved gamma is used
   c. Total strength = total of above/100' = 650 psf
2. [MTW10R, 1Y1R, 1Y2R]

Appendix G
ADDITIONAL DEAGGREGATION PLOTS AND TIME HISTORIES FROM EARTHQUAKE HAZARDS STUDY FOR MISSOURI SITE

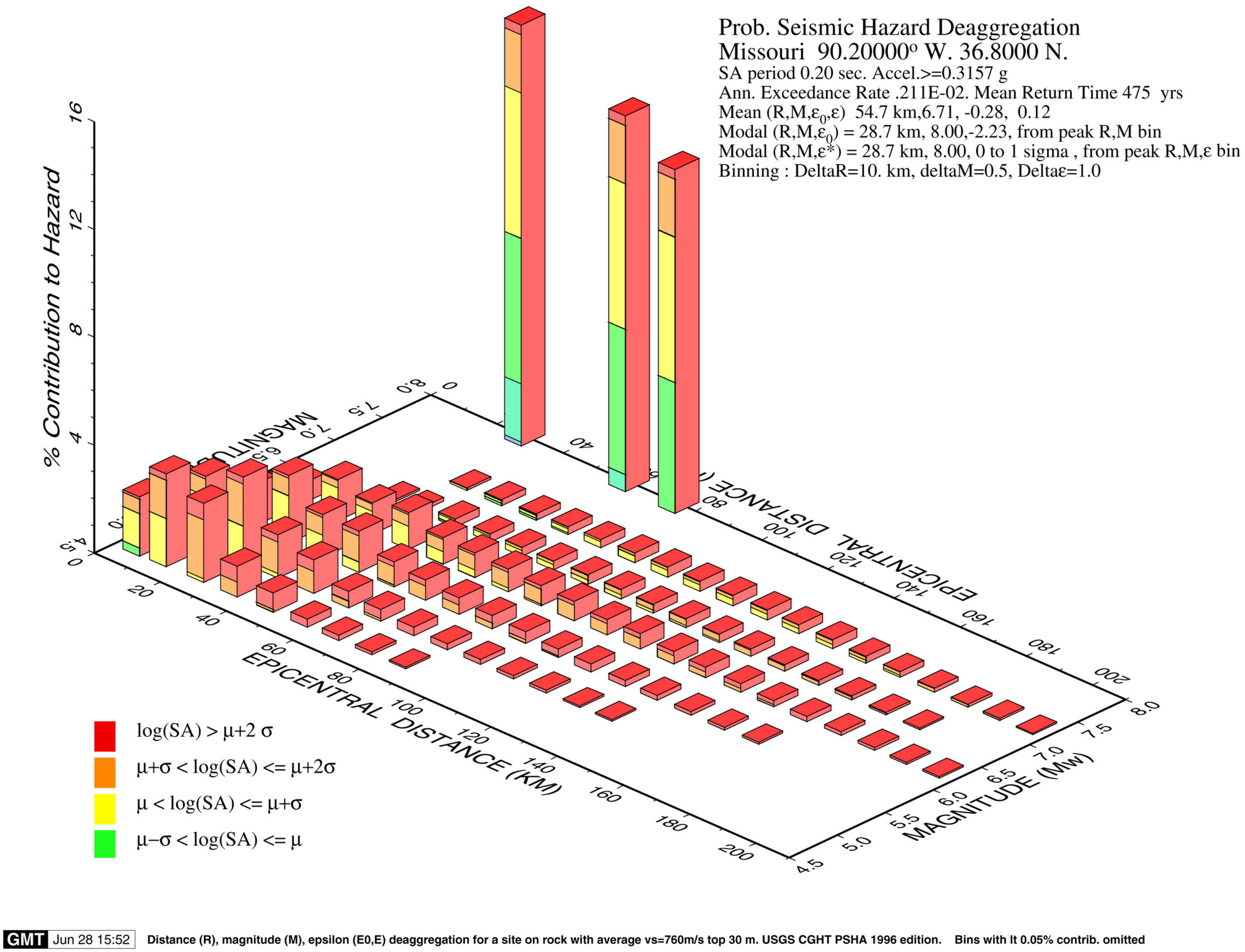
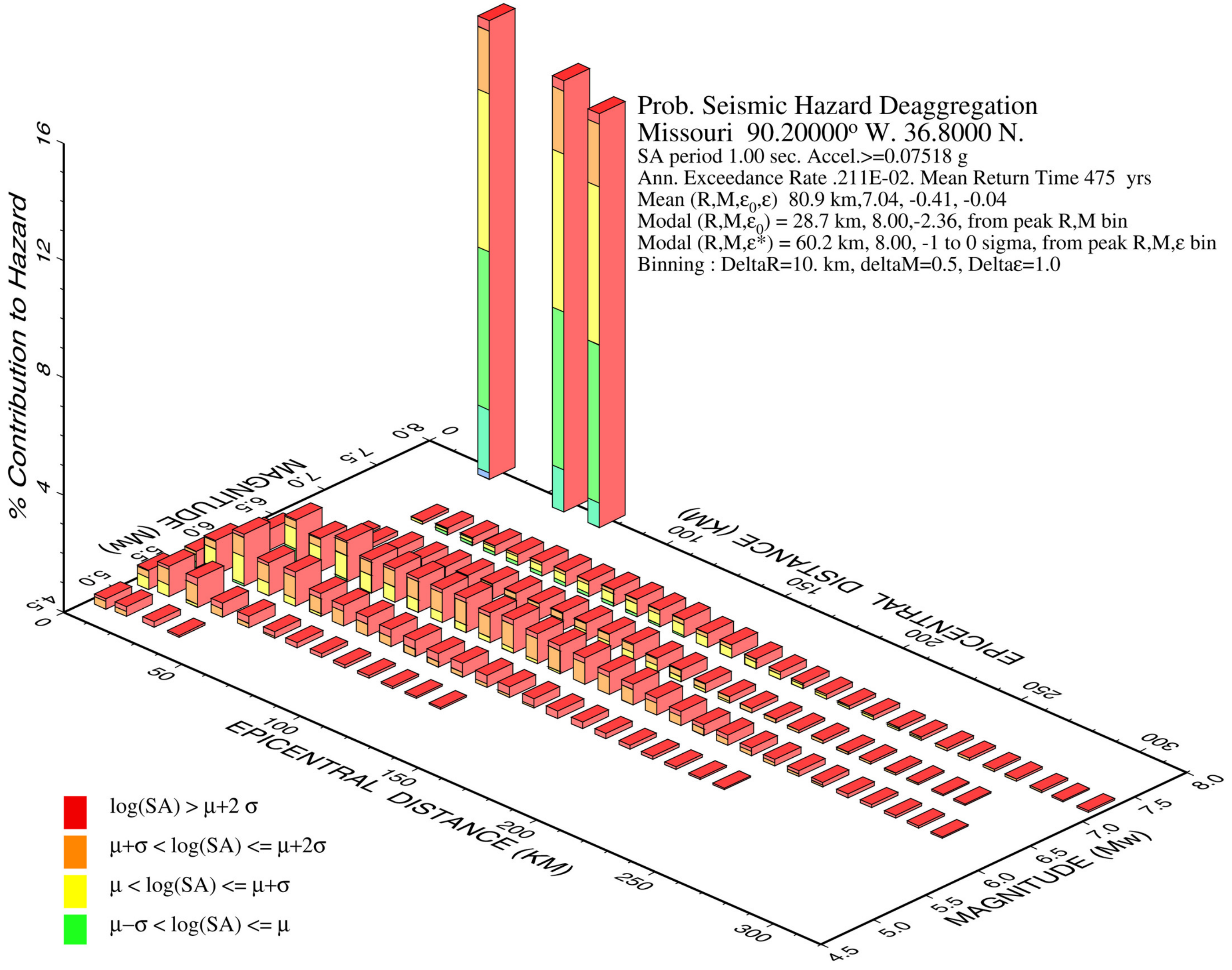
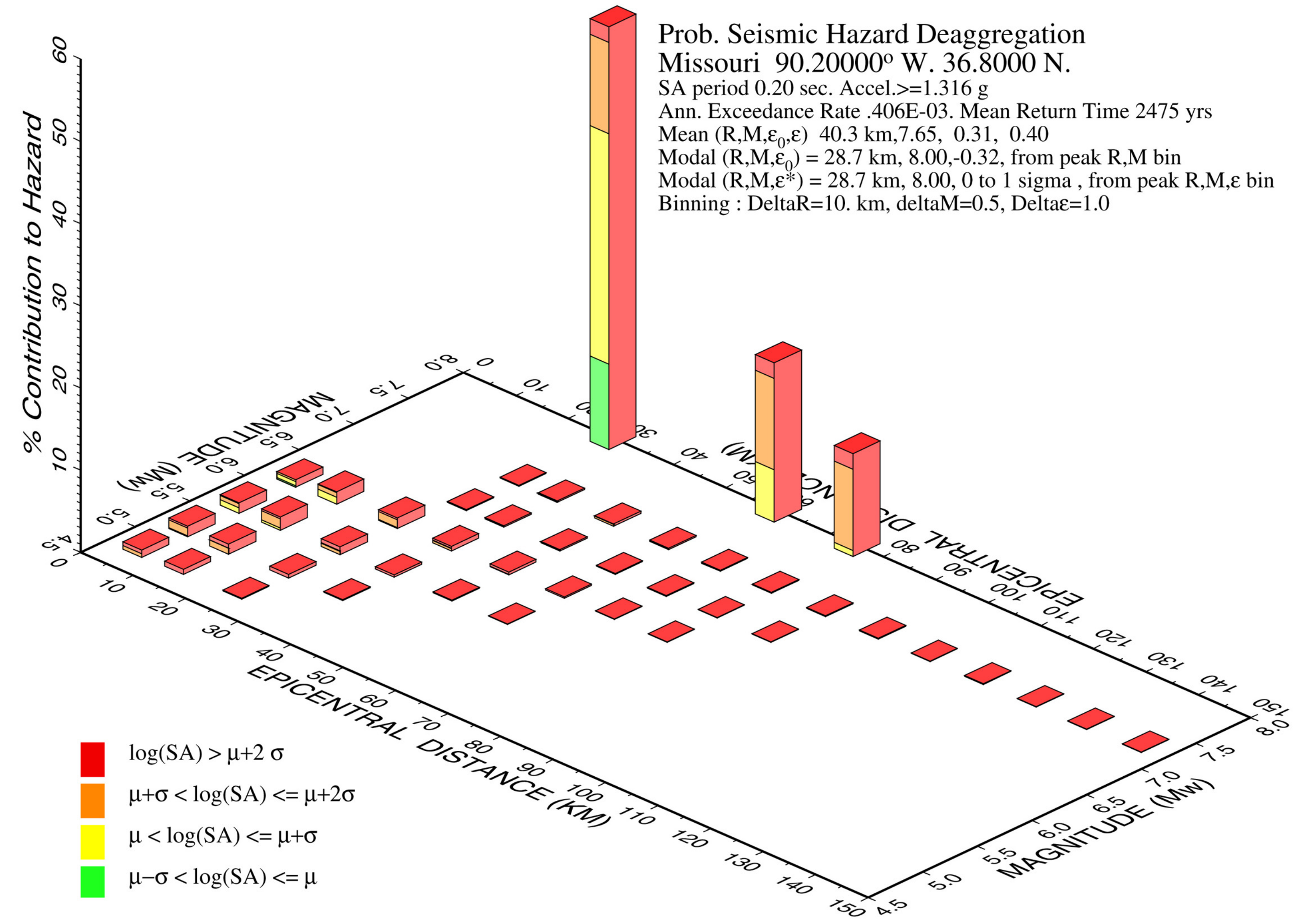


Figure G.1 Hazard Deaggregation, 475-Year Return Period, 0.2-Second Period Spectral Acceleration, Missouri Site



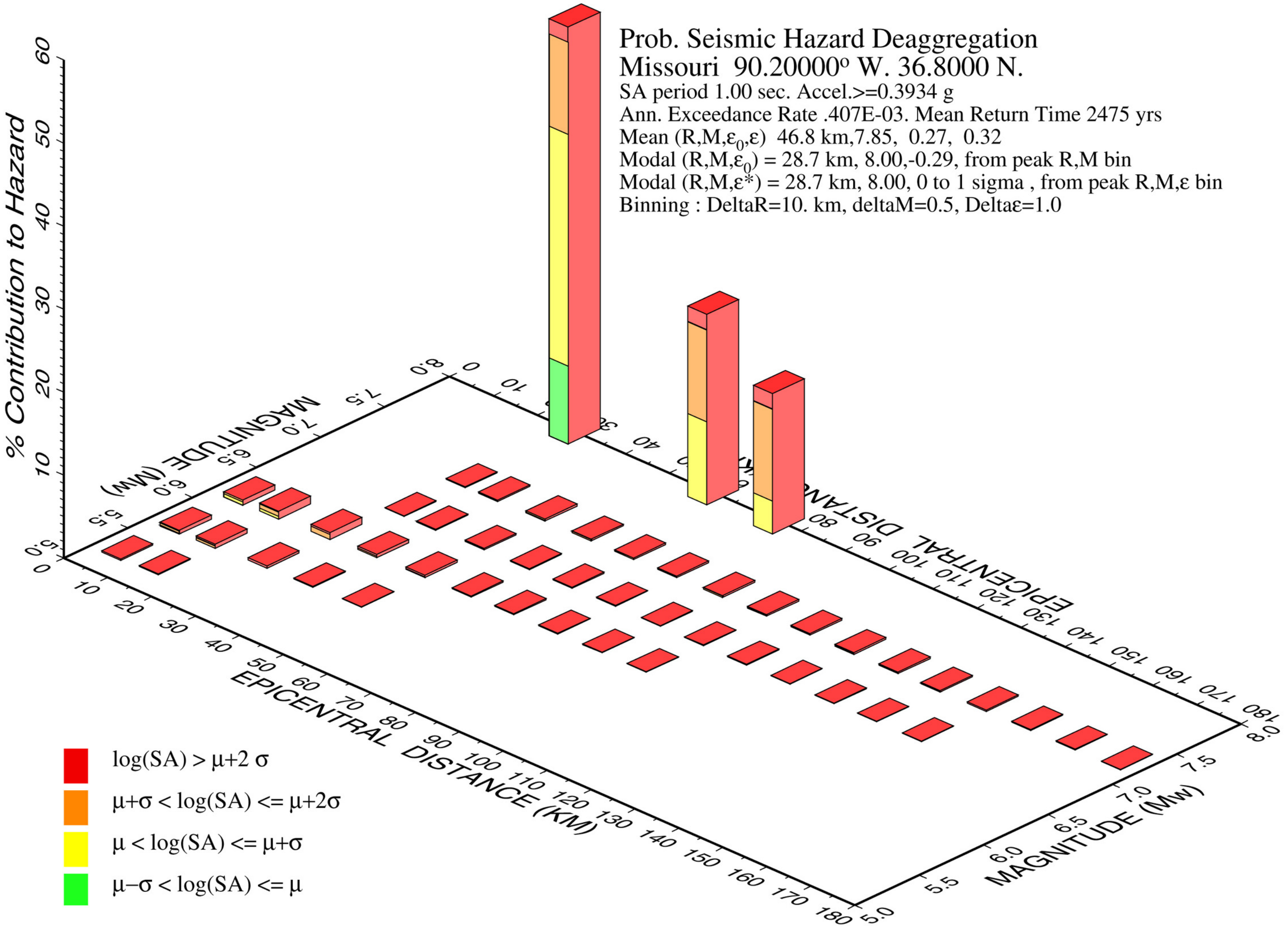
GMT Jun 28 15:55 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

Figure G.2 Hazard Deaggregation, 475-Year Return Period, 1.0-Second Period Spectral Acceleration, Missouri Site



GMT Jun 28 16:01 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

Figure G.3 **Hazard Deaggregation, 2,475-Year Return Period, 0.2-Second Period Spectral Acceleration, Missouri Site**



GMT Jun 28 16:04 Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs=760m/s top 30 m. USGS CGHT PSHA 1996 edition. Bins with lt 0.05% contrib. omitted

Figure G.4 Hazard Deaggregation, 2,475-Year Return Period, 1.0-Second Period Spectral Acceleration, Missouri Site

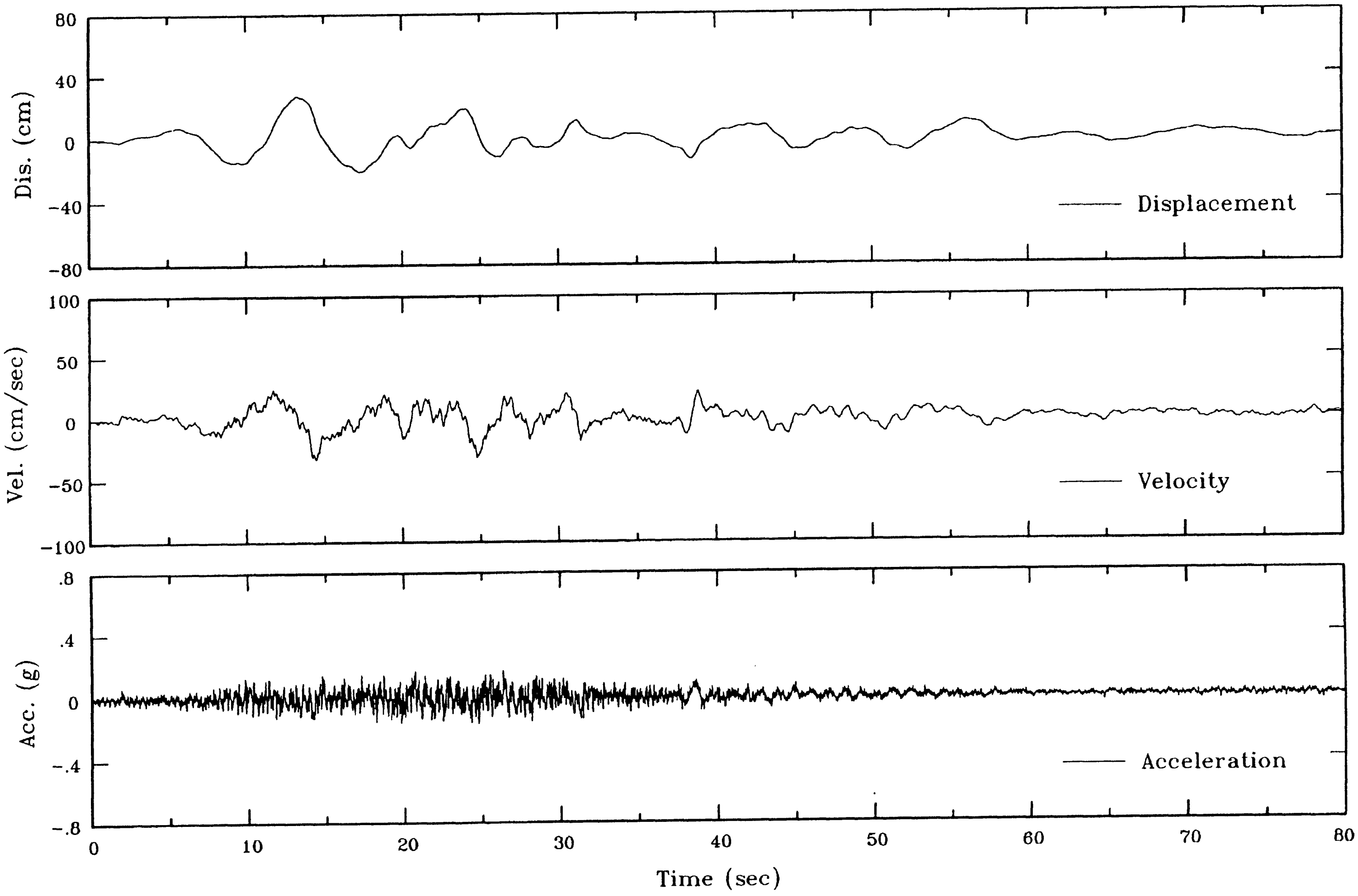


Figure G.5 Acceleration, Velocity, And Displacement Time Histories Of The Synthetic Ground Motion For The Current AASHTO Specifications. The Phase Spectrum Of The 1985 Michoacan Earthquake Recording At La Union Was Used.

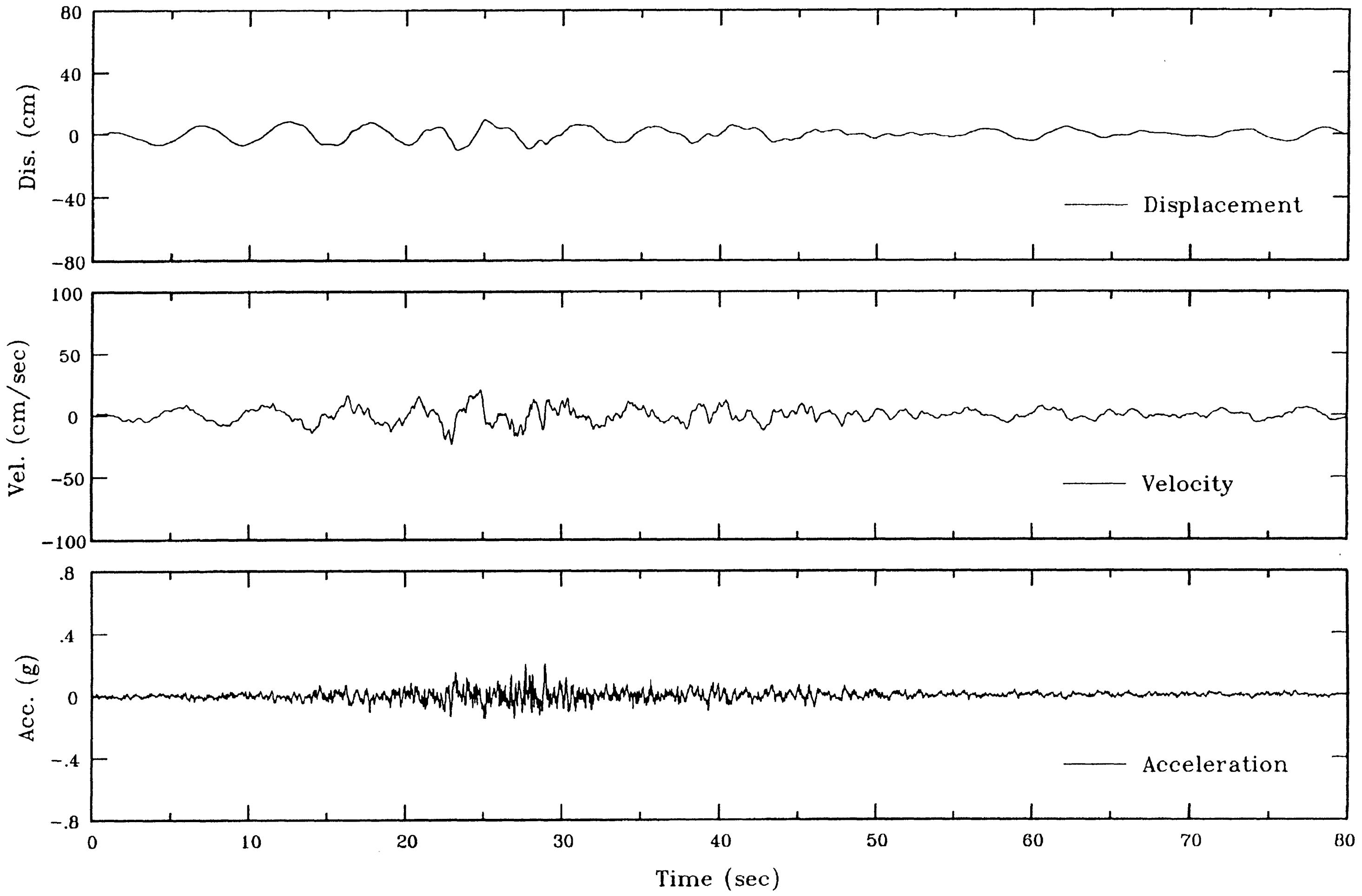


Figure G.6 Acceleration, Velocity, And Displacement Time Histories Of The Synthetic Ground Motion For The Current AASHTO Specifications. The Phase Spectrum Of The 1985 Chile Earthquake Recording At San Fernando Was Used.

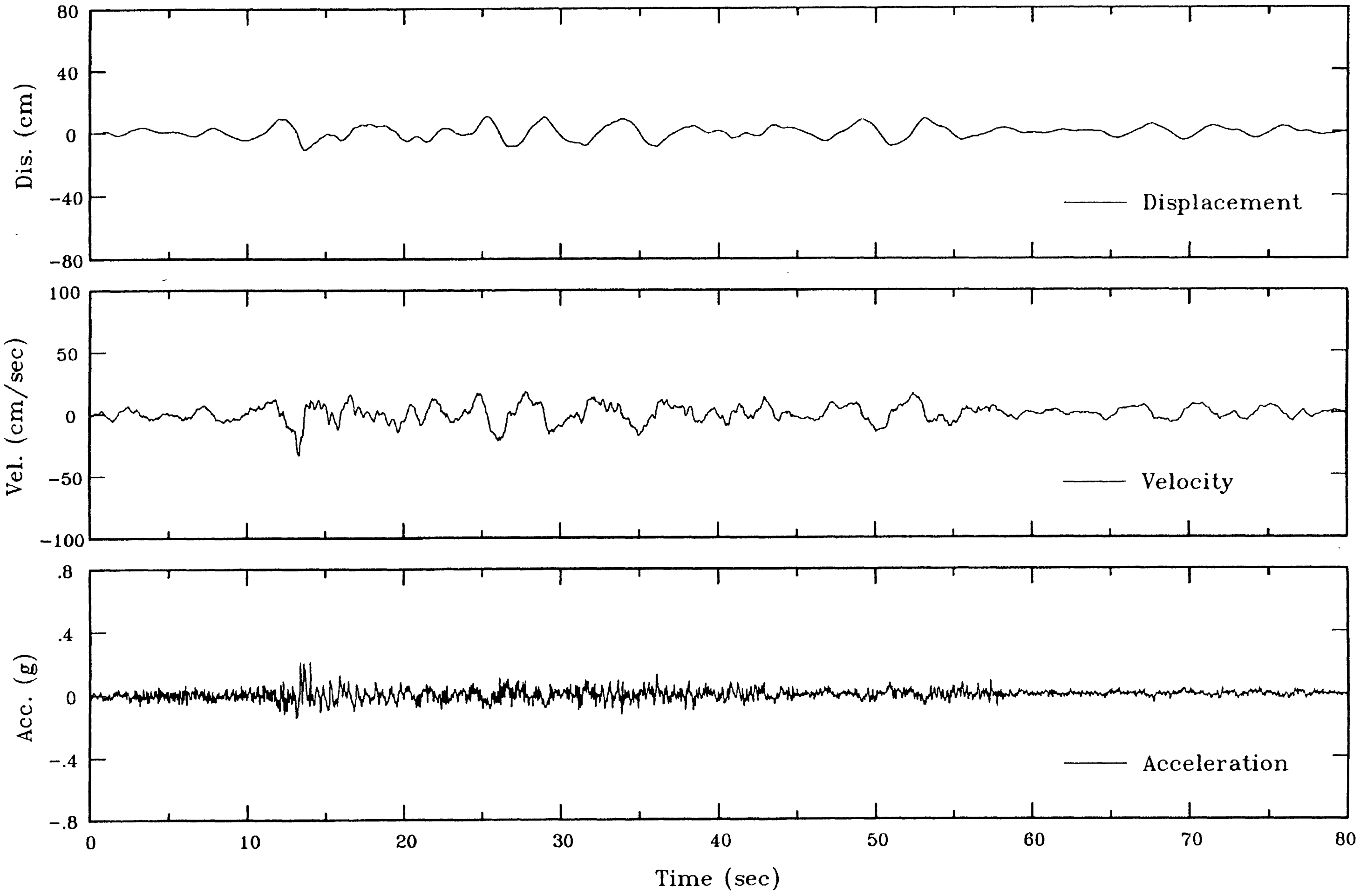


Figure G.7 Acceleration, Velocity, And Displacement Time Histories Of The Synthetic Ground Motion For The Current AASHTO Specifications. The Phase Spectrum Of The 1968 Tokachi-Oki Earthquake Recording At Hachinohe Was Used.

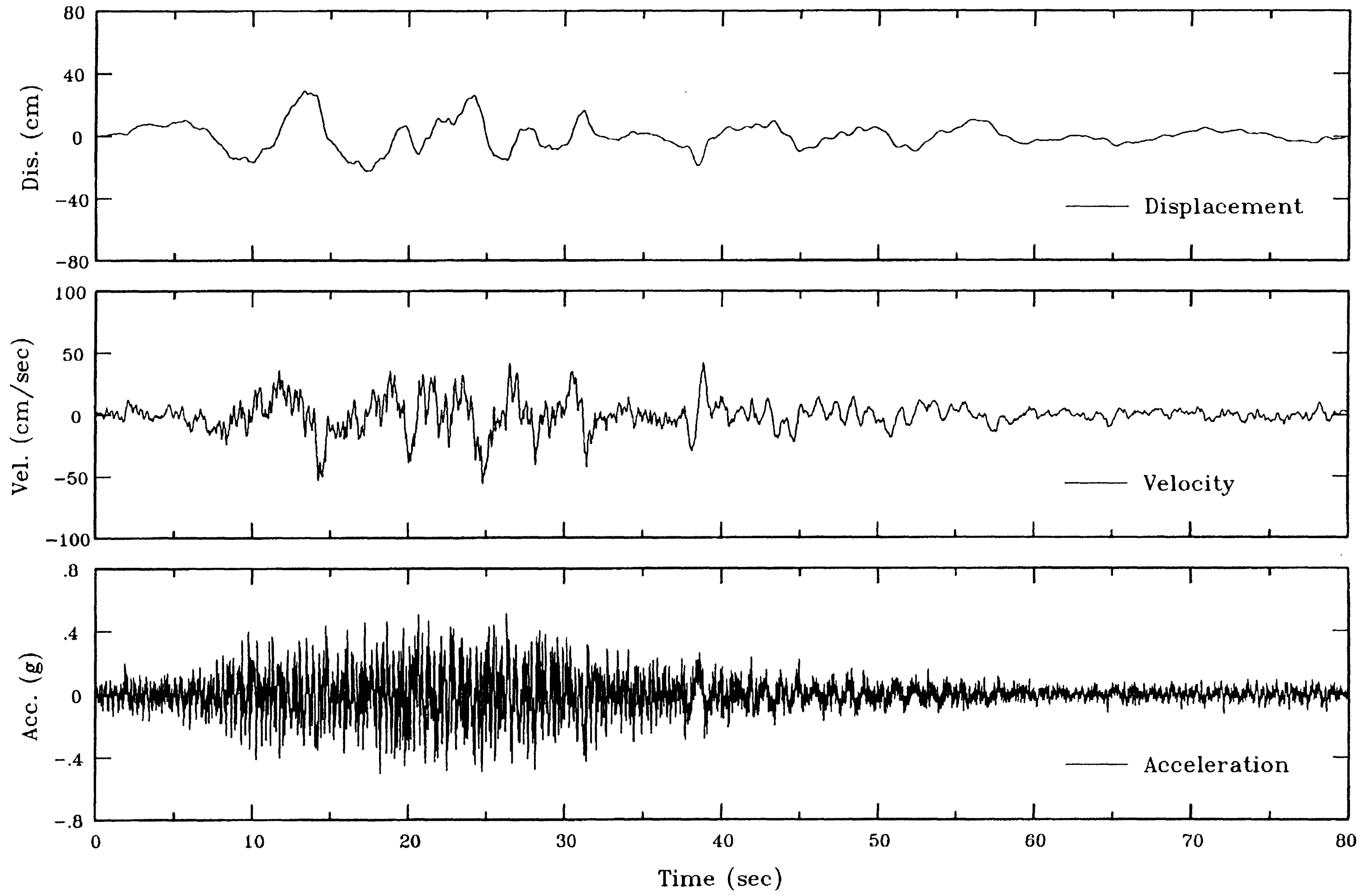


Figure G.8 Acceleration, Velocity, And Displacement Time Histories Of The Synthetic Ground Motion For The MCE Of The Proposed Specifications. The Phase Spectrum Of The 1985 Michoacan Earthquake Recording At La Union Was Used.

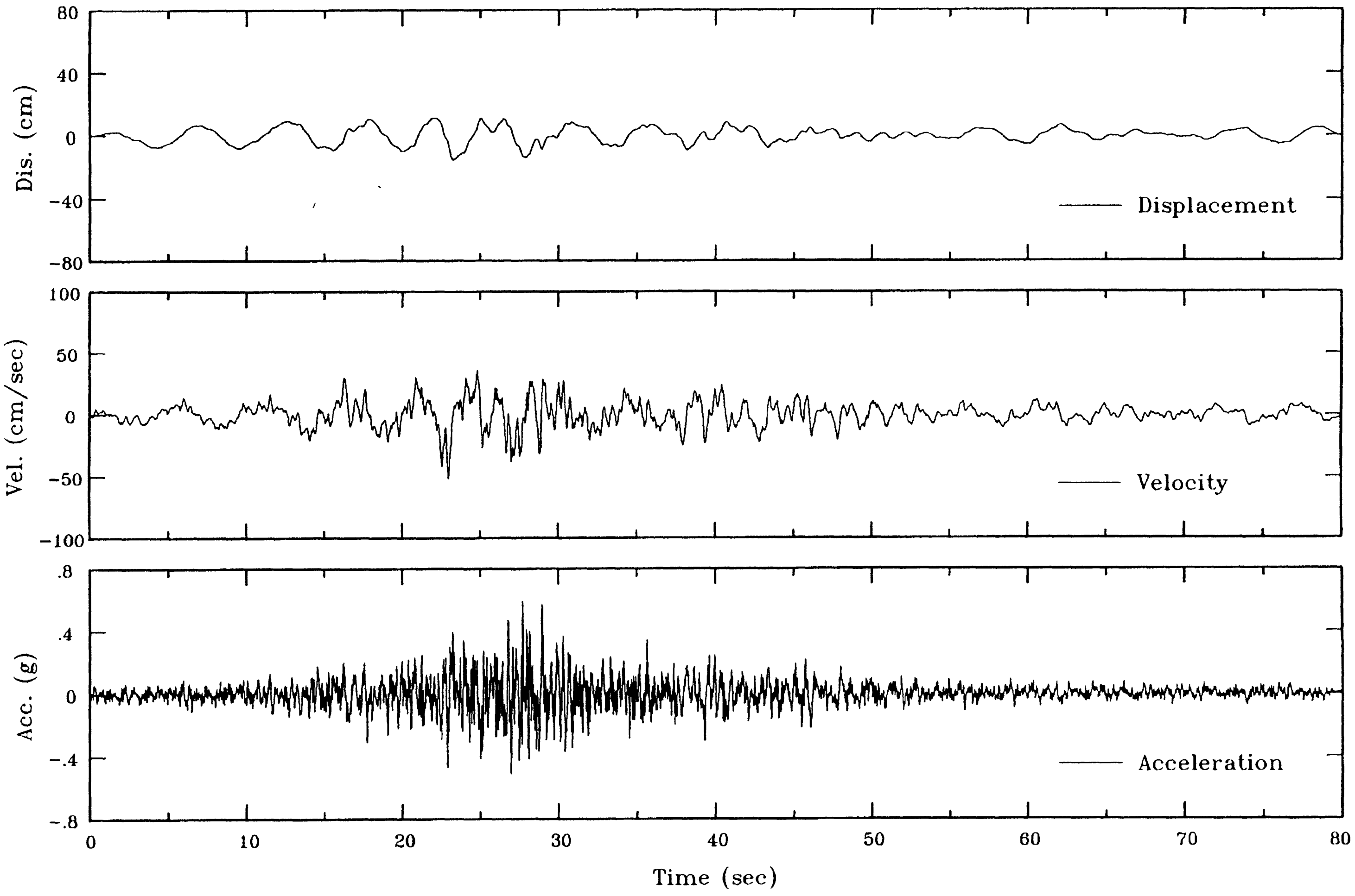


Figure G.9 Acceleration, Velocity, And Displacement Time Histories Of The Synthetic Ground Motion For The MCE Of The Proposed Specifications. The Phase Spectrum Of The 1985 Chile Earthquake Recording At San Fernando Was Used.

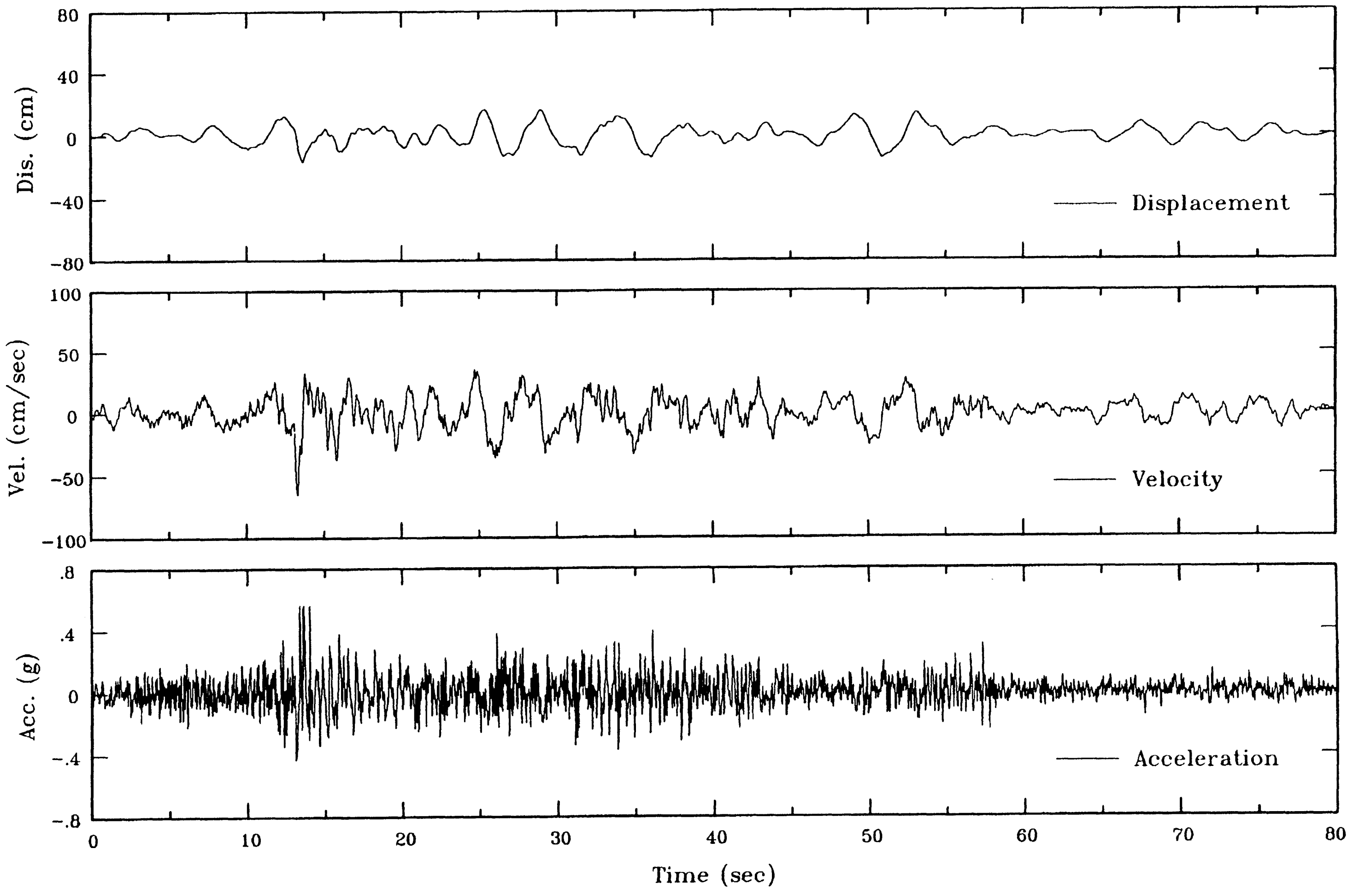


Figure G.10 Acceleration, Velocity, and Displacement Time Histories Of The Synthetic Ground Motion For The MCE Of The Proposed Specifications. The Phase Spectrum Of The 1968 Tokachi-Oki Earthquake Recording At Hachinohe Was Used.

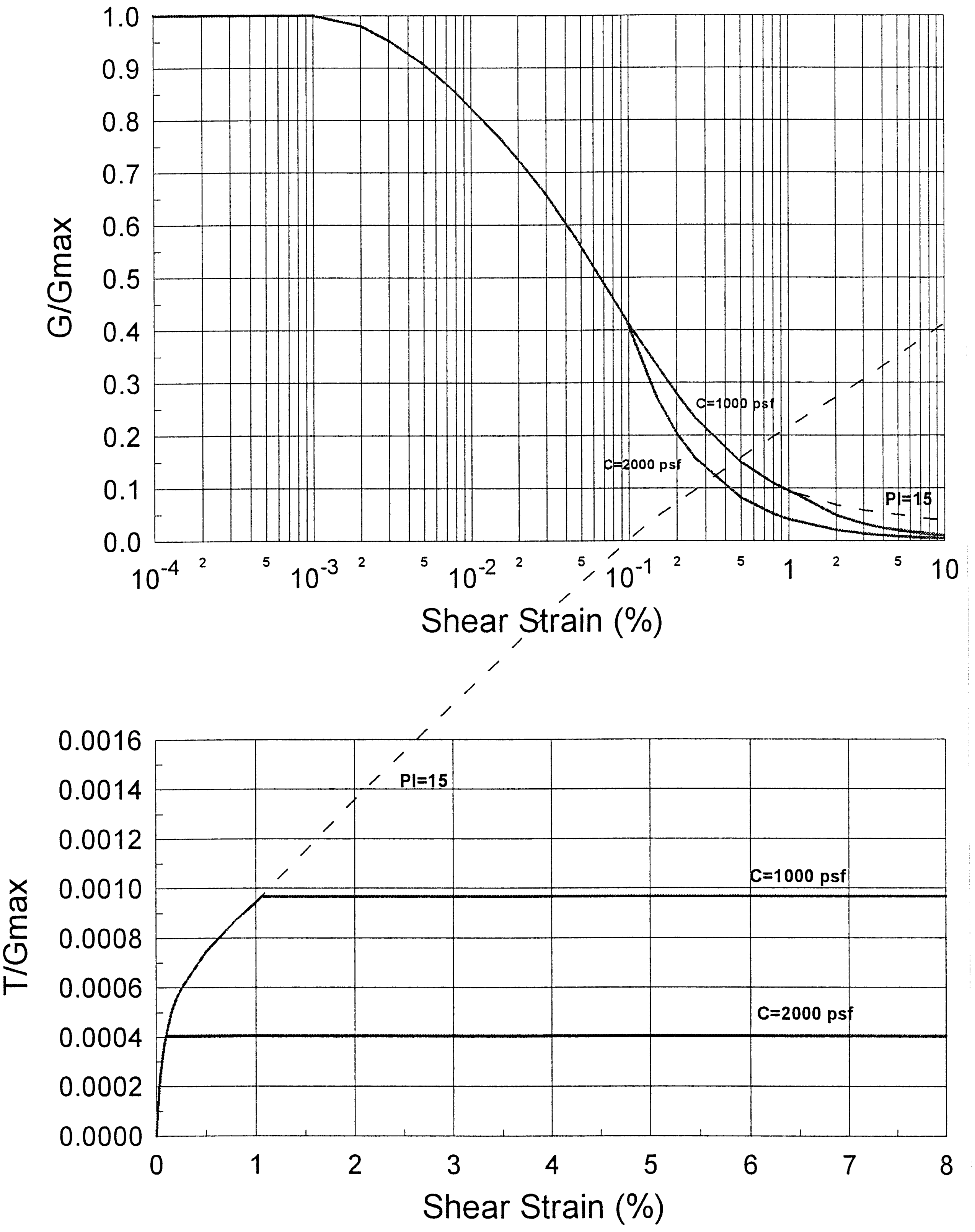
Appendix H
SITE-SPECIFIC ANALYSES – CENTRAL U.S. SITE

MISSOURI SITE

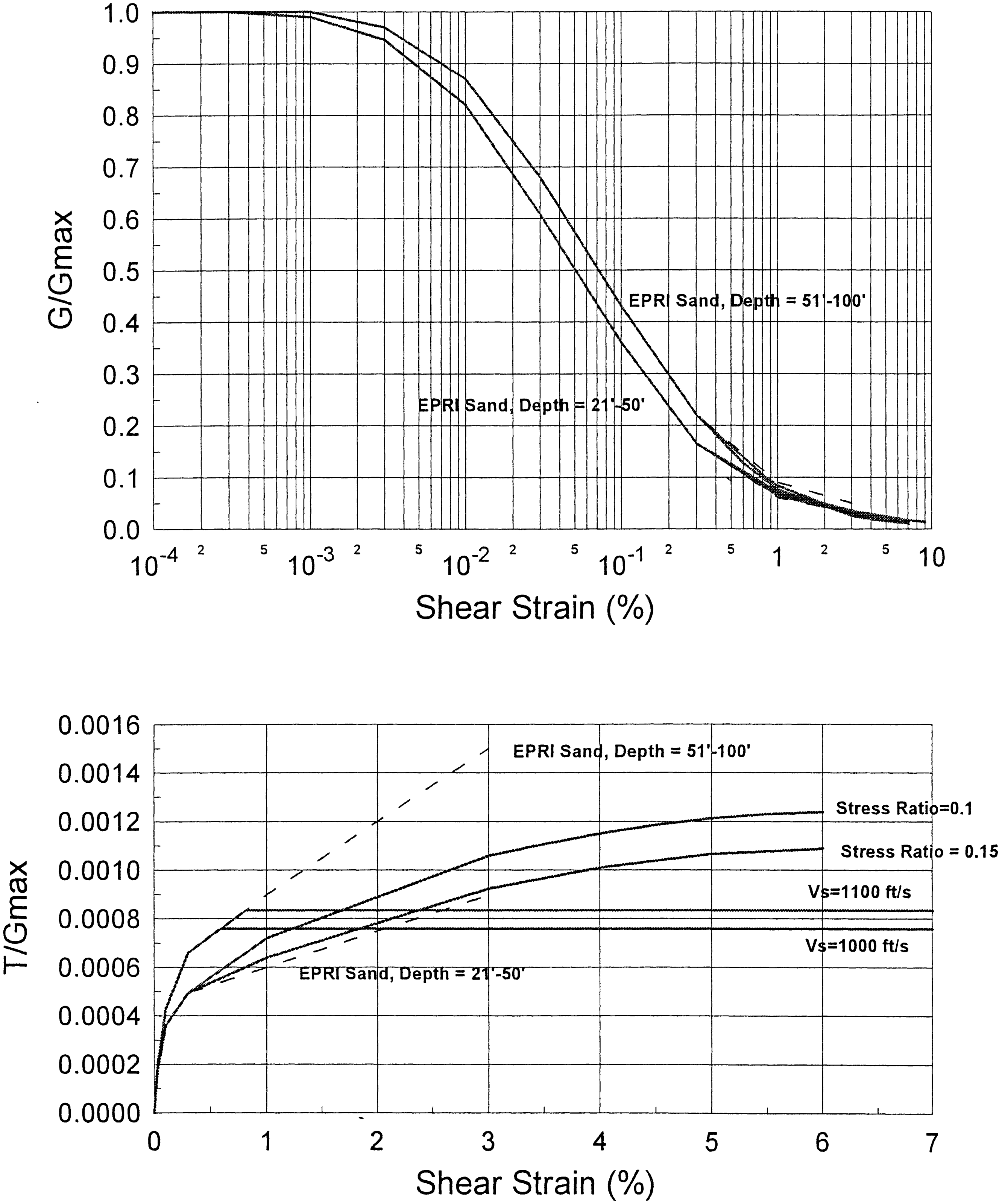
DESRA-MUSC ANALYSES

Graphs and Data Sheets Documenting:

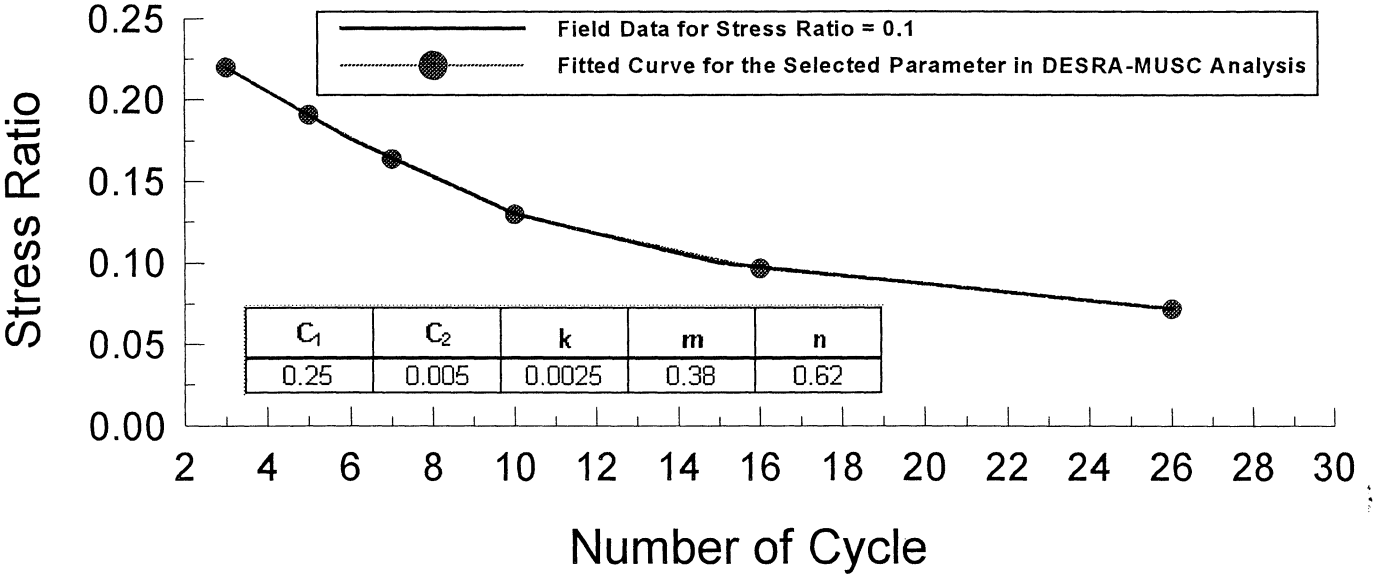
1. Input Data:
   - G/G_{max} and Backbone Curves
   - Liquefaction Strength Curves
2. Results of Response Analyses
4. Tabular Summaries of Response Analyses



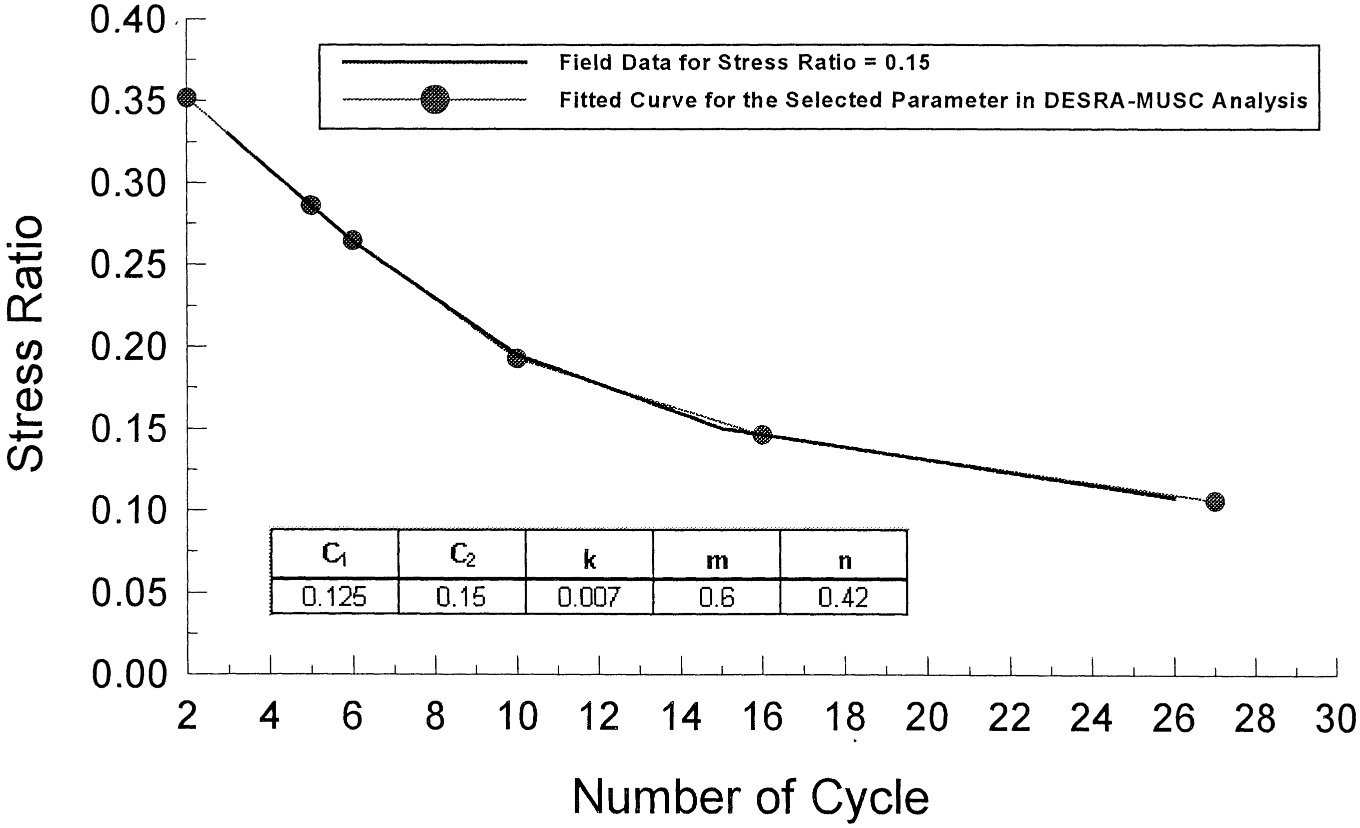
Matched G/Gmax and Backbones for P1 = 15, Missouri Site



Missouri Site, Matches of the G/Gmax vs. Shear Strain Curves for Sand Layers



C_1	C_2	k	m	n
0.25	0.005	0.0025	0.38	0.62



C_1	C_2	k	m	n
0.125	0.15	0.007	0.6	0.42

Comparison of Liquefaction Curves
Missouri Site, Friction Angle = 32 Deg.

MISSOURI SITE

Case No	Descriptions
Case 1a	Effective Stress Analysis, Soil Profile without Fill, 1985 Michocan EQ, 475-yr ARP
Case 1b	Effective Stress Analysis, Soil Profile without Fill, 1985 MichocanEQ, 2475-yr ARP
Case 2a	Effective Stress Analysis, Soil Profile with Fill, 1985 Michocan EQ, 475-yr ARP
Case 2b	Effective Stress Analysis, Soil Profile with Fill, 1985 Michocan EQ, 2475-yr ARP
Case 3a	Effective Stress Analysis, Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP
Case 3b	Effective Stress Analysis, Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP
Case 4a	Effective Stress Analysis, Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP
Case 4b	Effective Stress Analysis, Soil Profile with Fill, 1985 Chile EQ, 2475-yr ARP
Case 5a	Effective Stress Analysis, Soil Profile without Fill, 1968 Tokachi Oki EQ, 475-yr ARP
Case 5b	Effective Stress Analysis, Soil Profile without Fill, 1968 Tokachi Oki EQ, 2475-yr ARP
Case 6a	Effective Stress Analysis, Soil Profile with Fill, 1968 Tokachi Oki EQ, 475-yr ARP
Case 6b	Effective Stress Analysis, Soil Profile with Fill, 1968 Tokachi Oki EQ, 2475-yr ARP

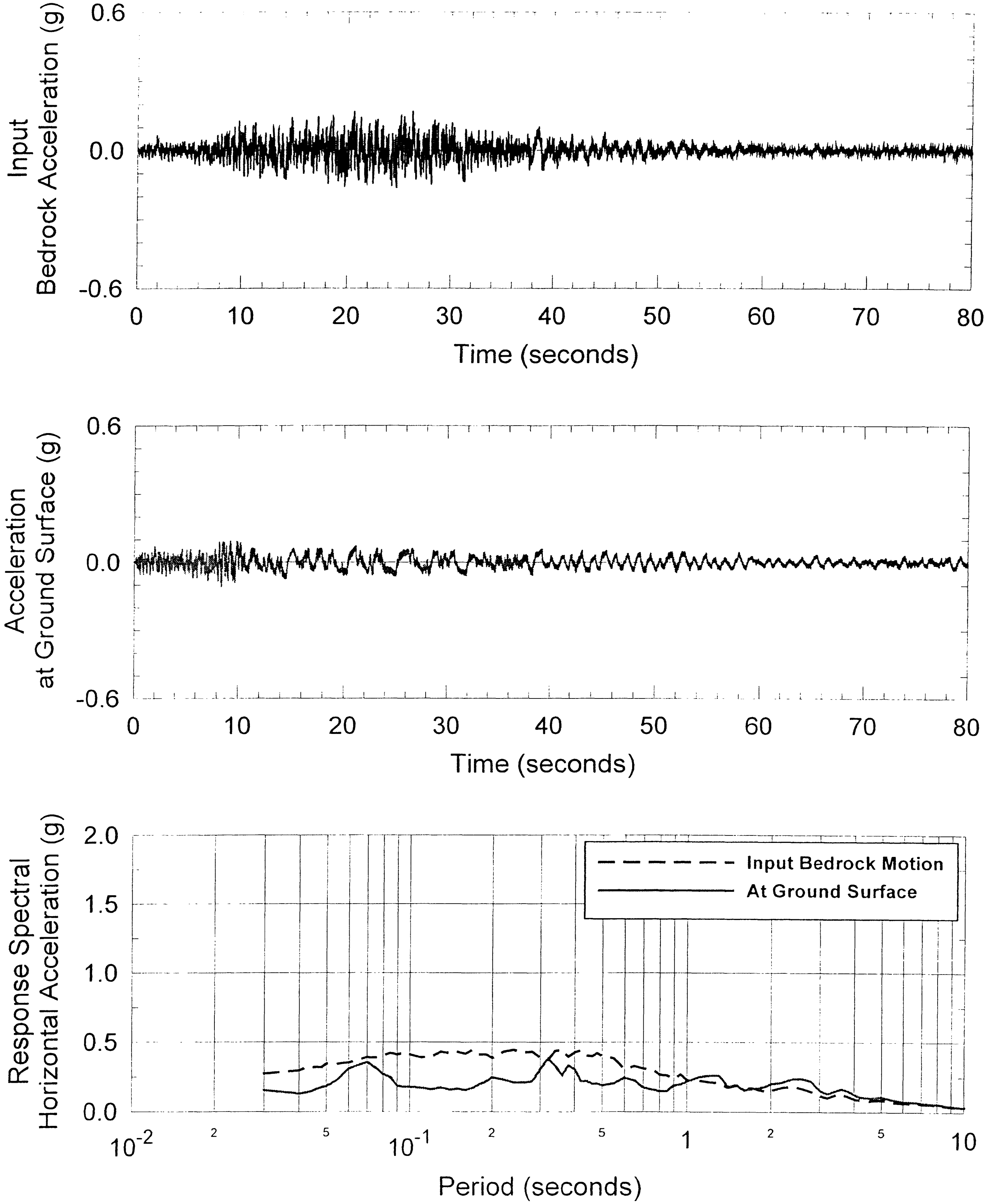


Figure 1a-1 Acceleration Time Histories and Response Spectra
Case 1a: Soil Profile without Fill, 1985 Michoacan EQ, 475-yr ARP

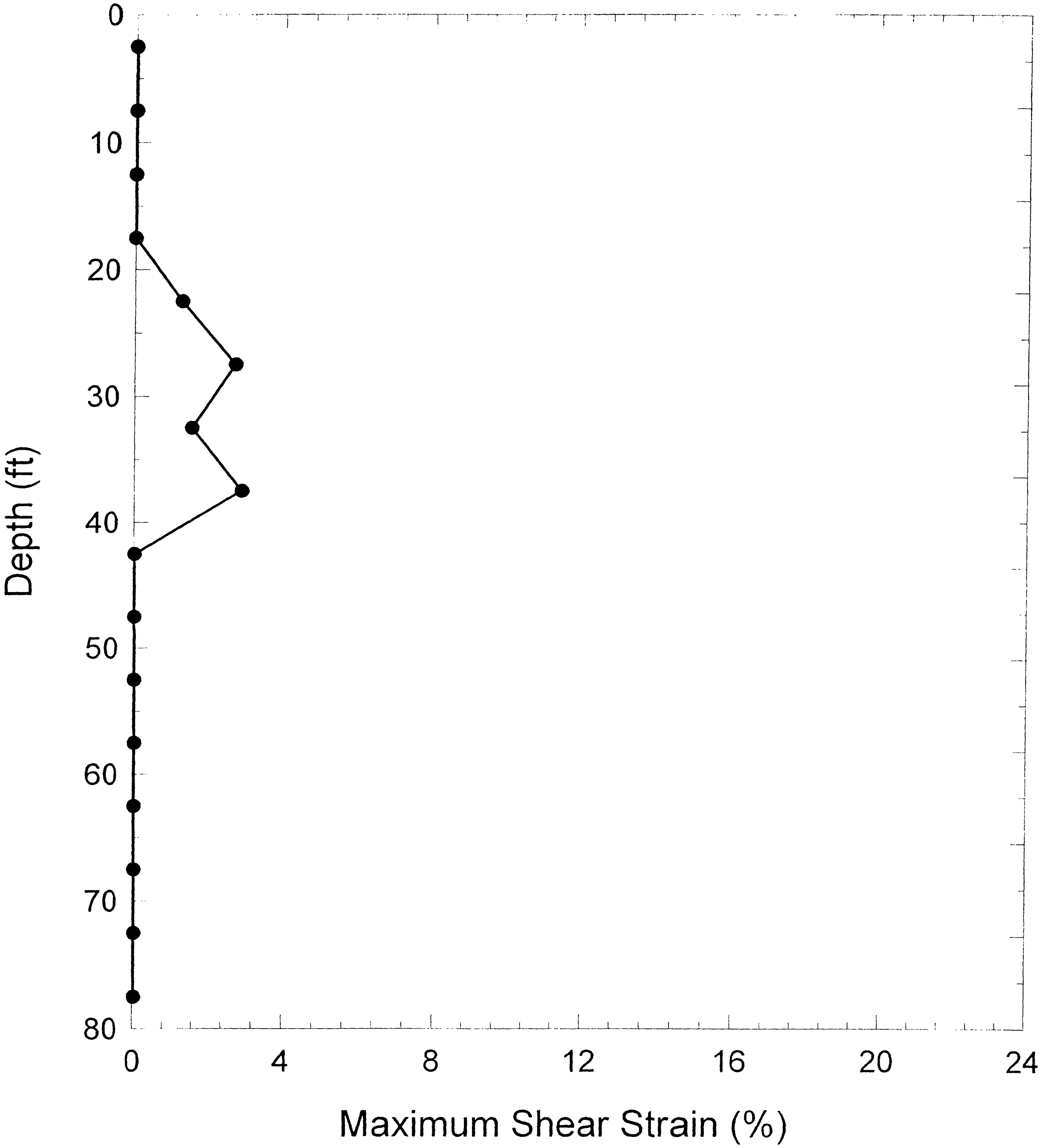


Figure 1a-2 Maximum Shear Strain Occurred During the Shaking
Case 1a: Soil Profile without Fill
1985 Michoacan EQ, 475-yr ARP

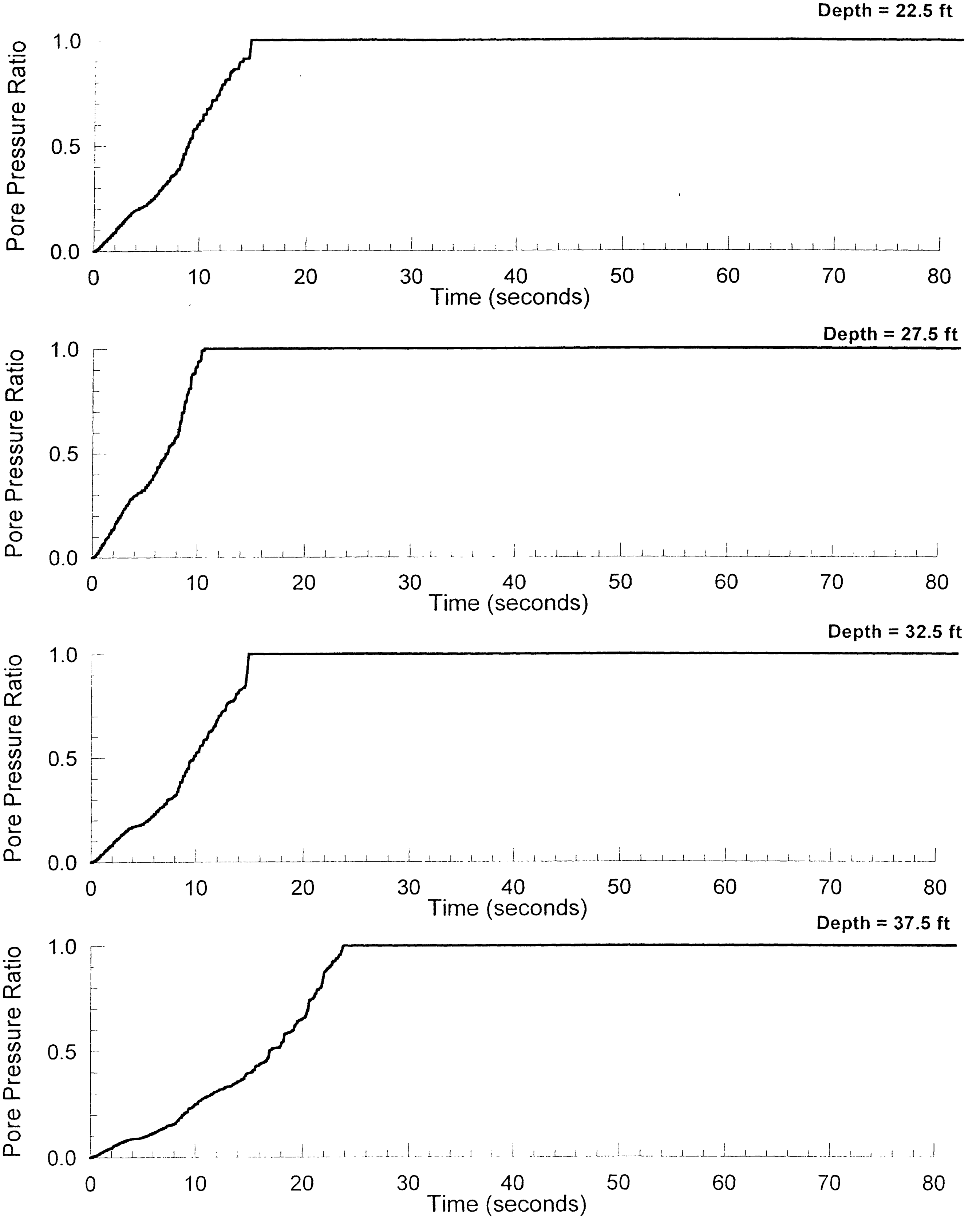


Figure 1a-3 Pore Pressure Generation
Case 1a: Soil Profile without Fill, 1985 Michoacan EQ, 475-yr ARP

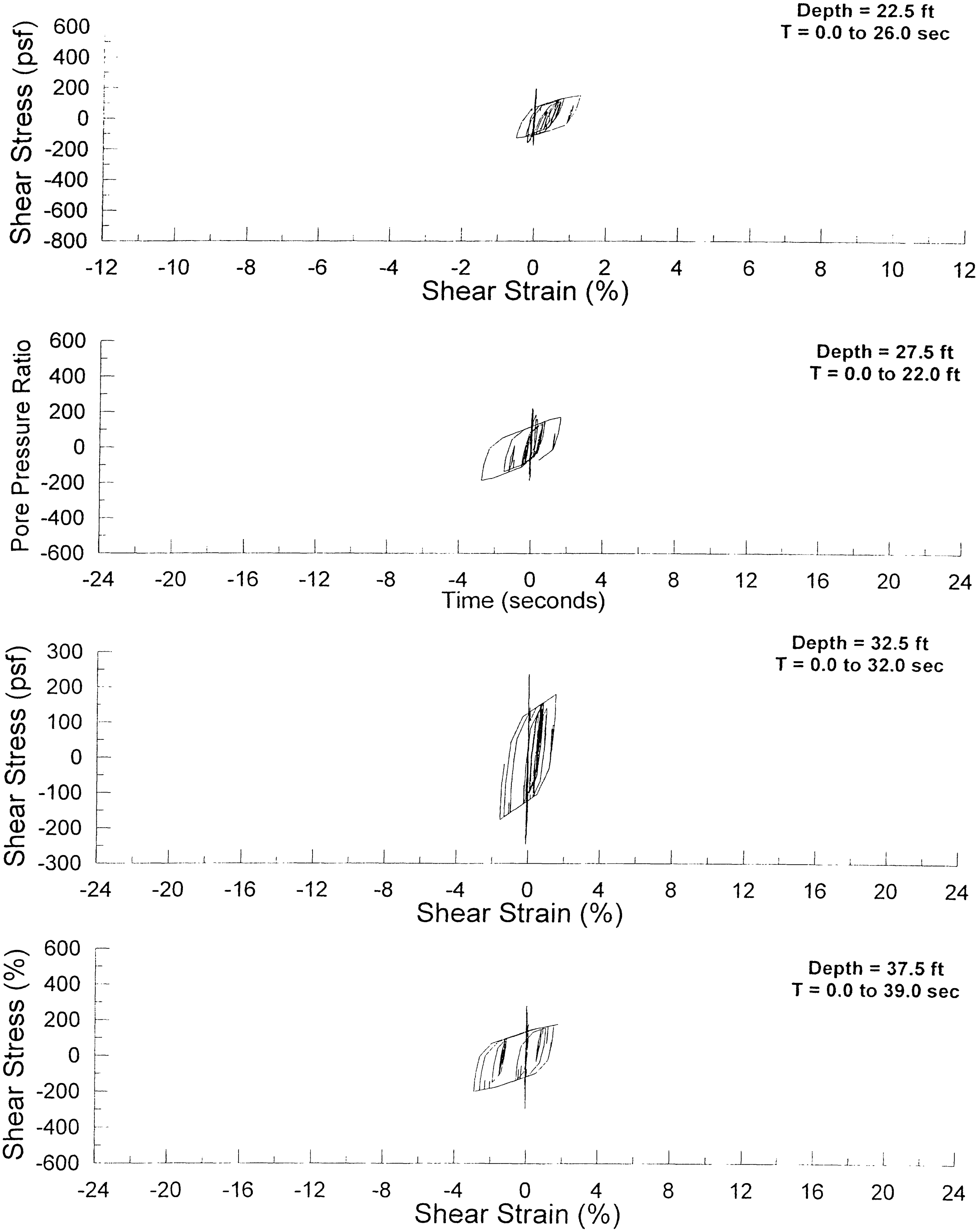


Figure 1a-4 Shear Stress - Shear Strain Loops
Case 1a: Soil Profile without Fill, 1985 Michoacan EQ, 475-yr ARP

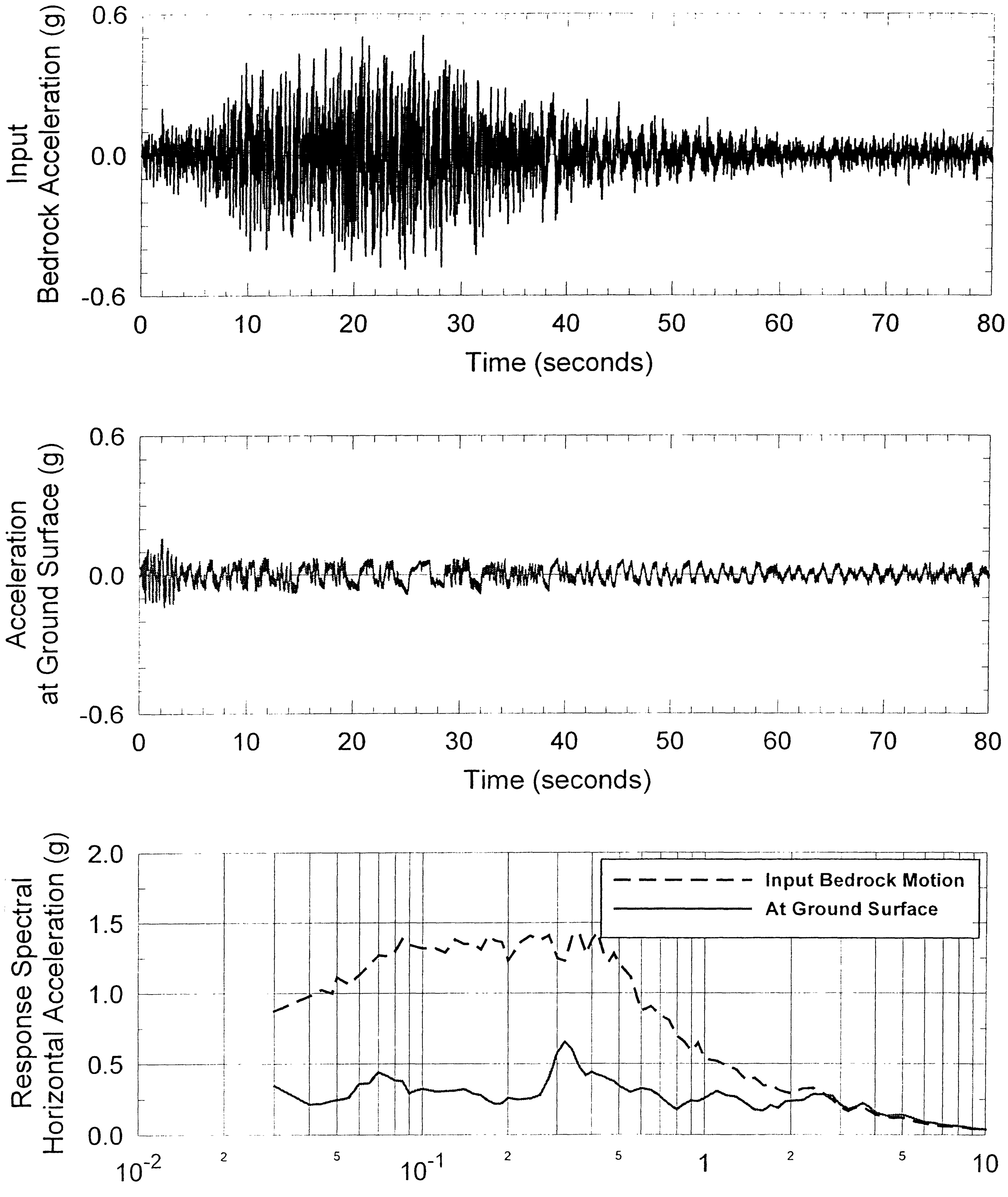


Figure 1b-1 Acceleration Time Histories and Response Spectra Case 1b: Soil Profile without Fill, 1985 Michoacan EQ, 2475-yr ARP

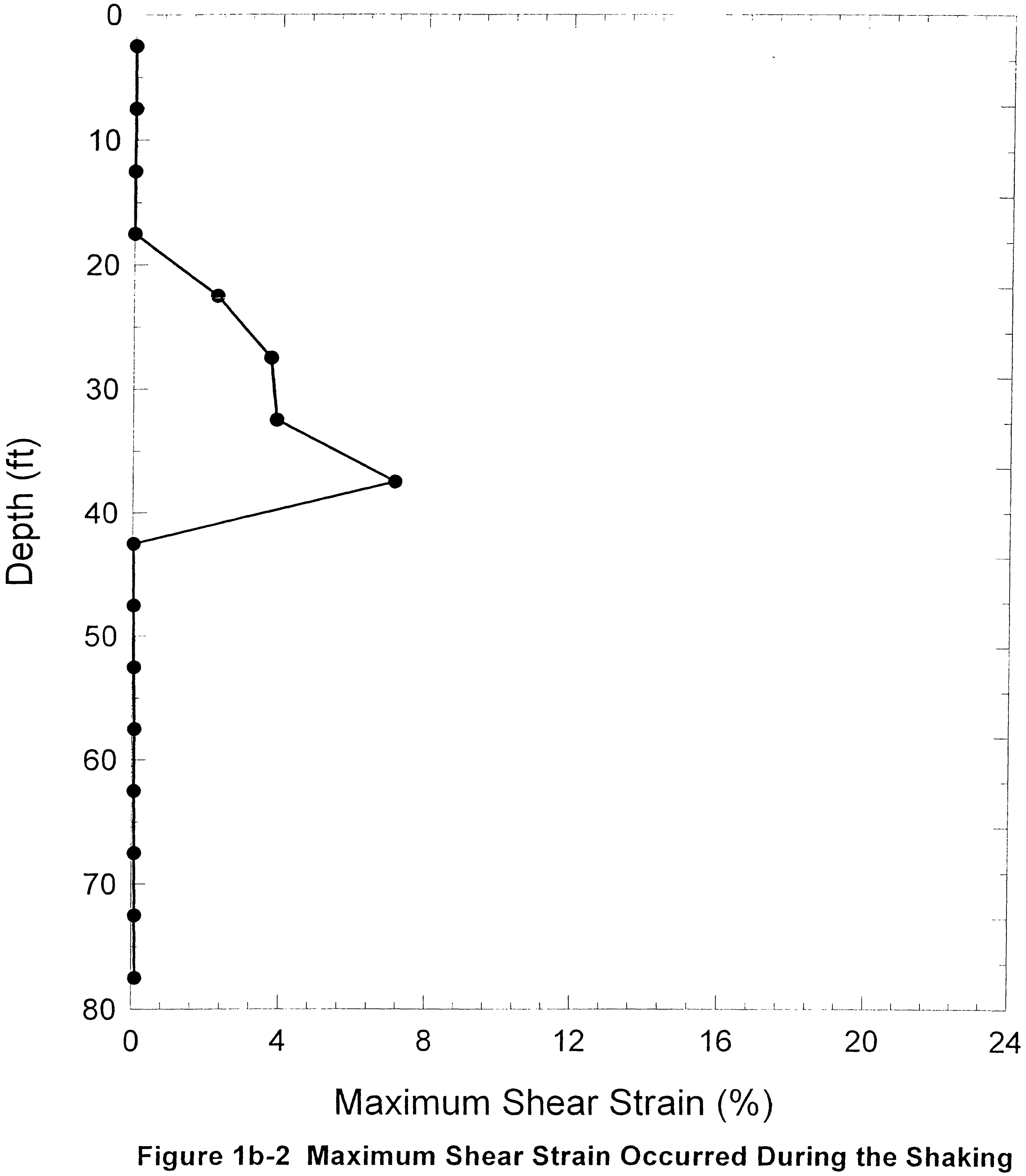


Figure 1b-2 Maximum Shear Strain Occurred During the Shaking
Case 1b: Soil Profile without Fill
1985 Michoacan EQ, 2475-yr ARP

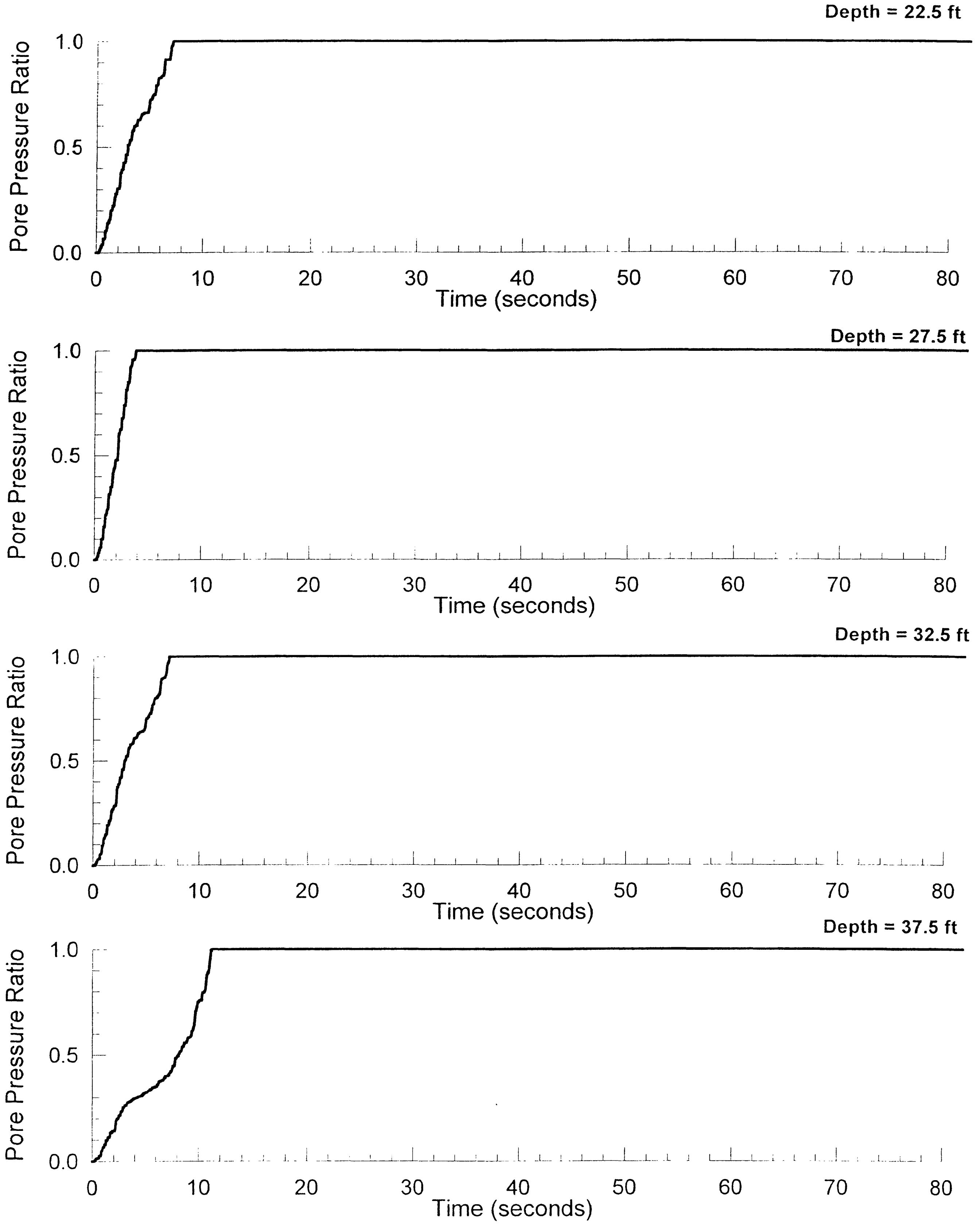


Figure 1b-3 Pore Pressure Generation
Case 1b: Soil Profile without Fill, 1985 Michoacan EQ, 2475-yr ARP

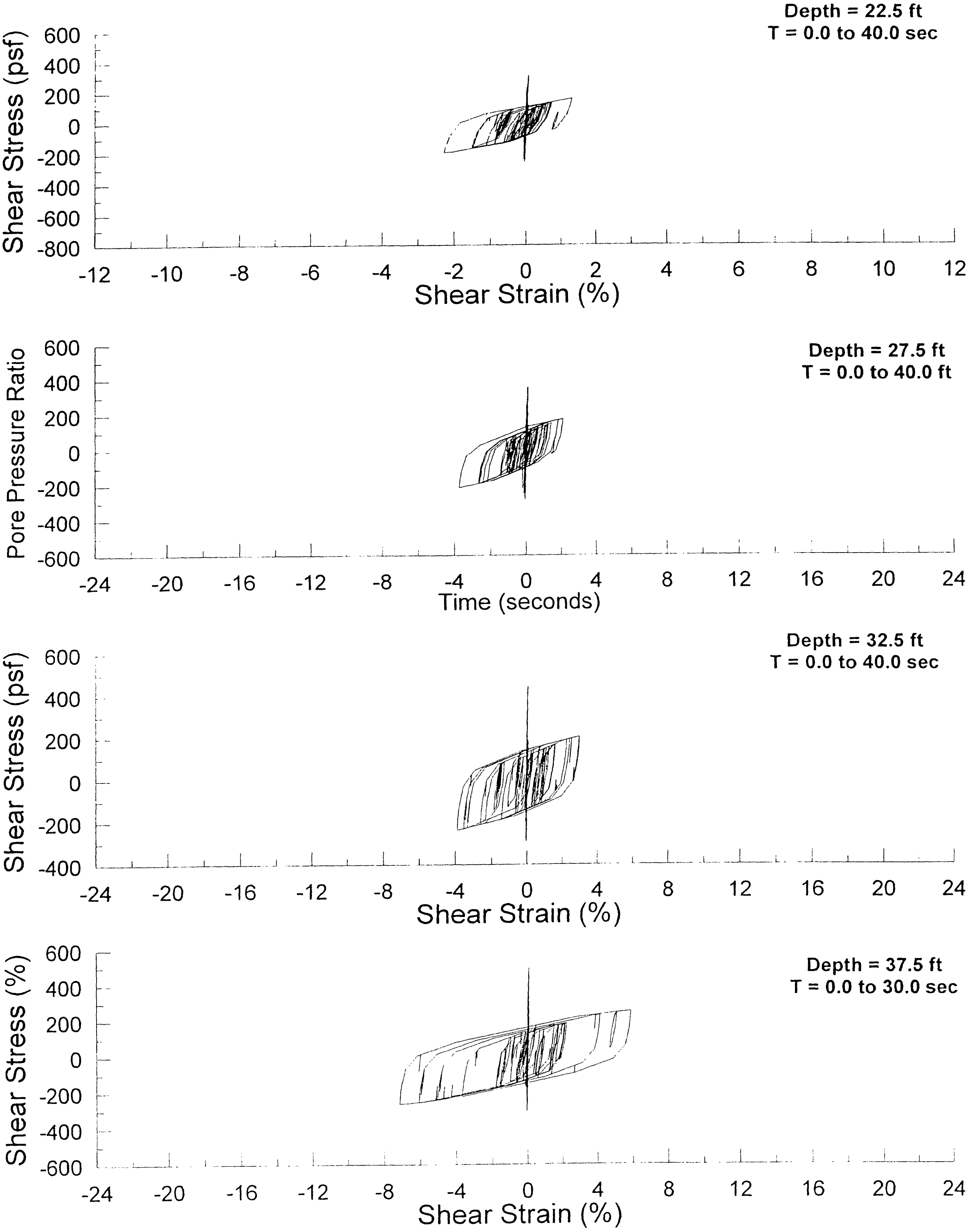


Figure 1b-4 Shear Stress - Shear Strain Loops
Case 1b: Soil Profile without Fill, 1985 Michoacan EQ, 2475-yr ARP

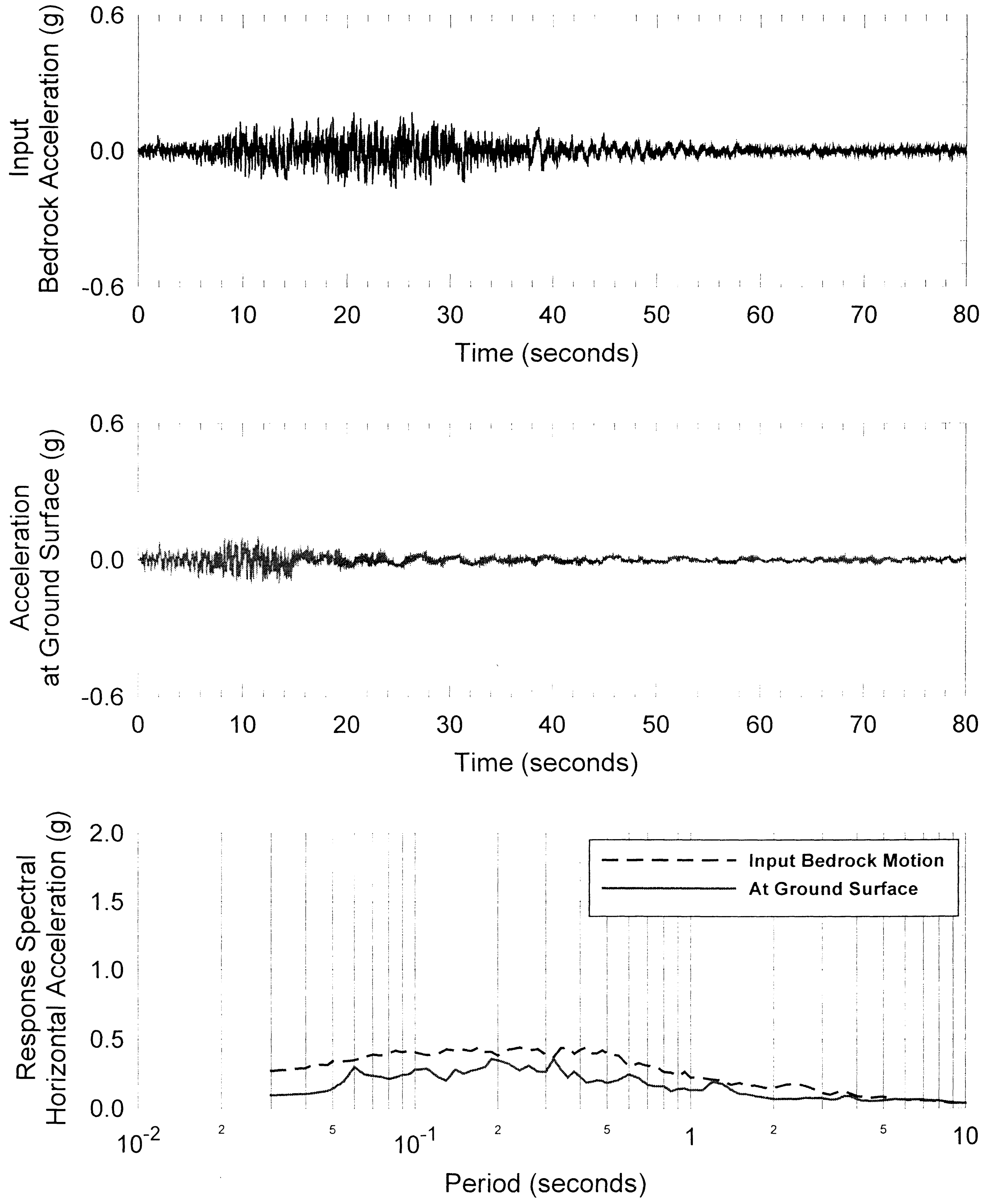


Figure 2a-1 Acceleration Time Histories and Response Spectra Case 2a: Soil Profile with Fill, 1985 Michoacan EQ, 475-yr ARP

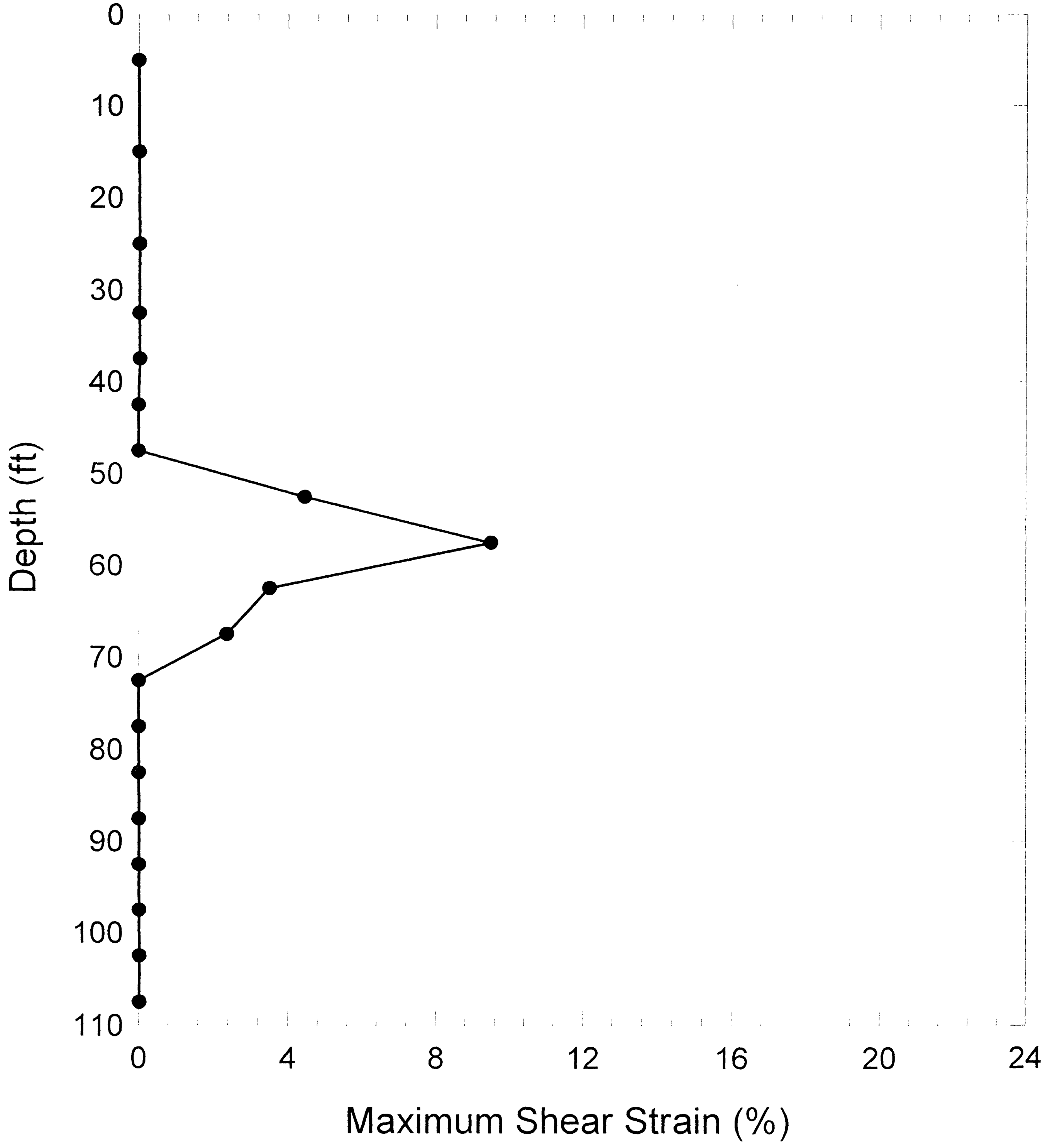


Figure 2a-2 Maximum Shear Strain Occurred During the Shaking
Case 2a: Soil Profile with Fill
1985 Michoacan EQ, 475-yr ARP

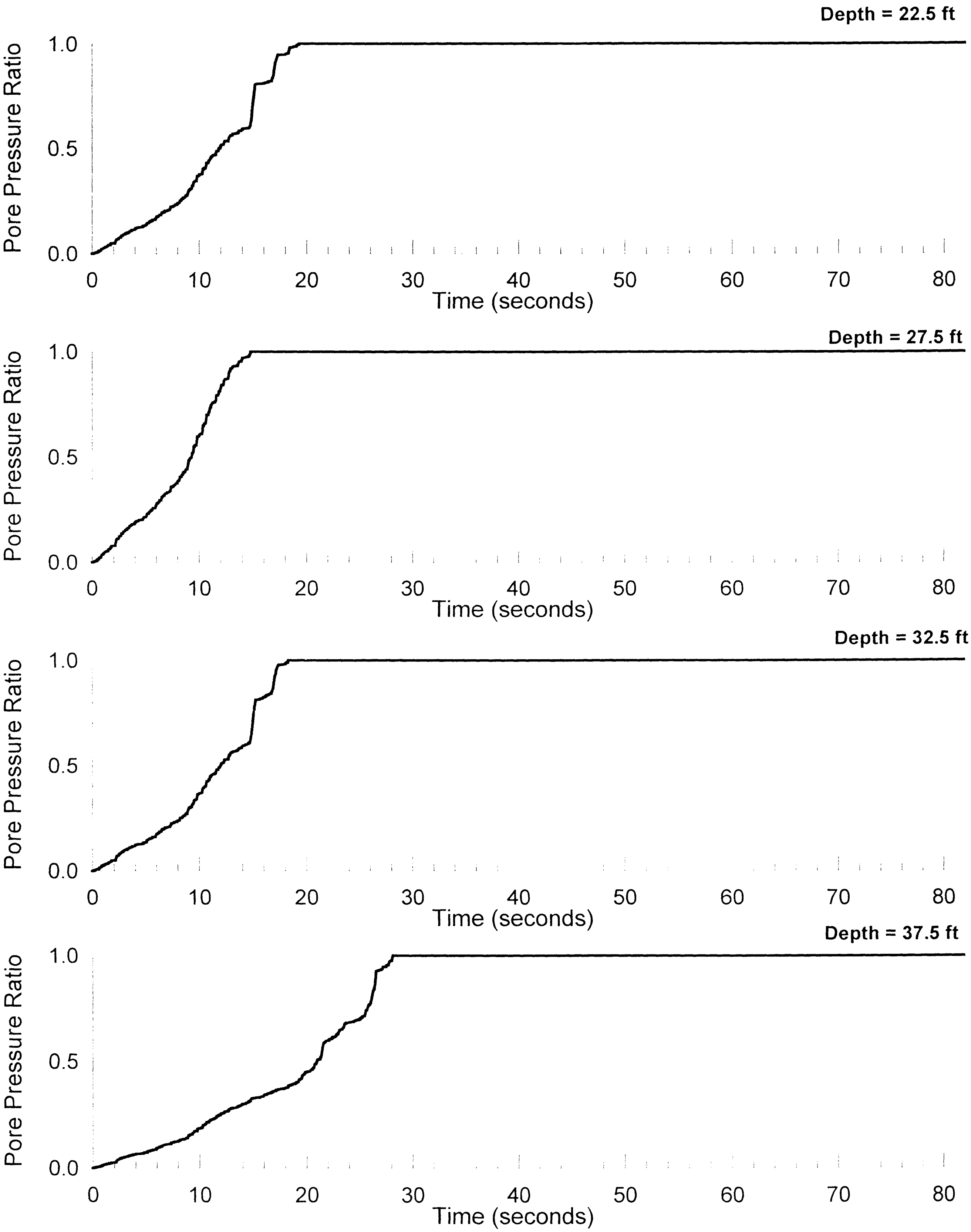


Figure 2a-3 Pore Pressure Generation
Case 2a: Soil Profile with Fill, 1985 Michoacan EQ, 475-yr ARP

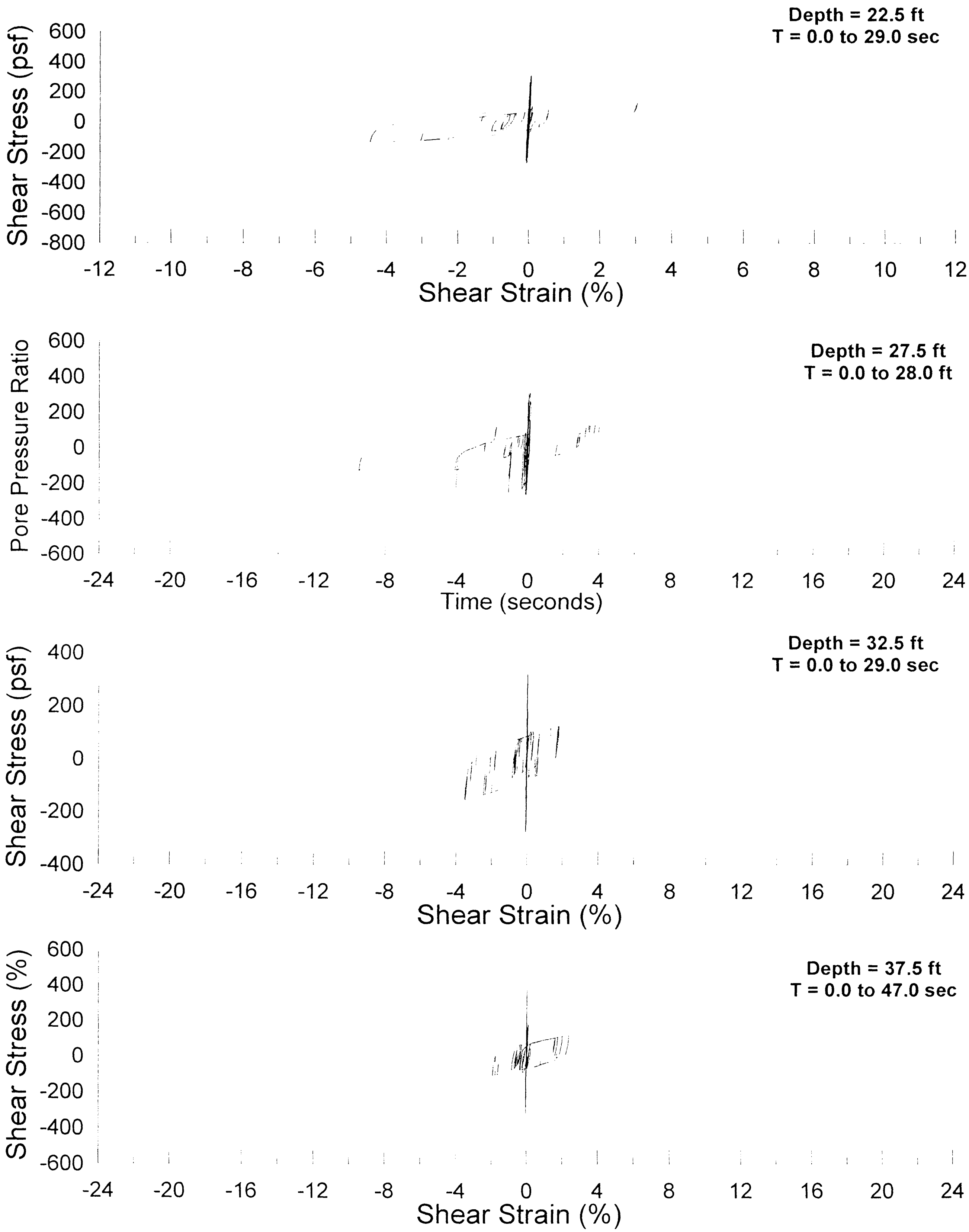


Figure 2a-4 Shear Stress - Shear Strain Loops
Case 2a: Soil Profile with Fill, 1985 Michoacan EQ, 475-yr ARP

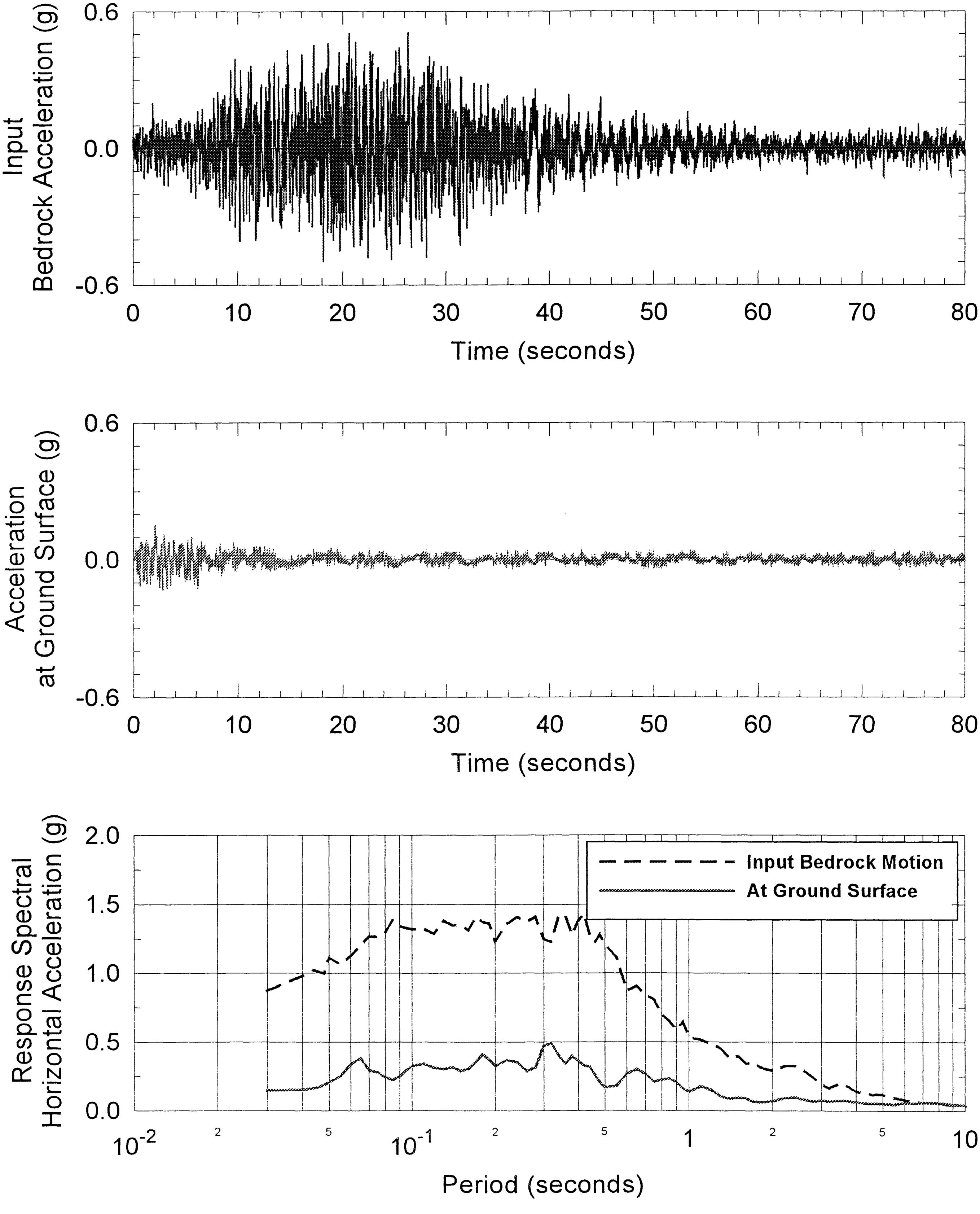


Figure 2b-1 Acceleration Time Histories and Response Spectra Case 2b: Soil Profile with Fill, 1985 Michoacan EQ, 2475-yr ARP

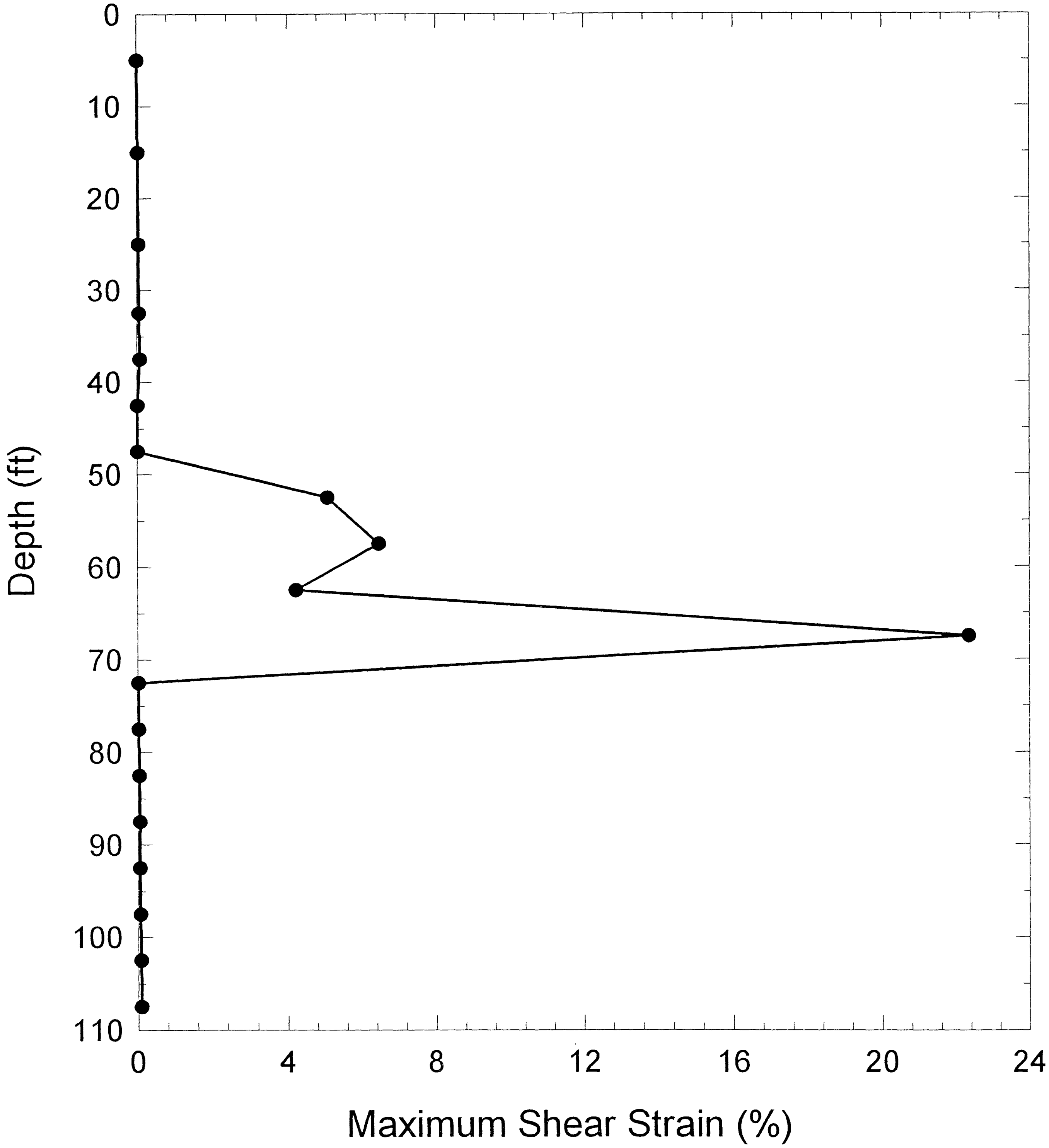


Figure 2b-2 Maximum Shear Strain Occurred During the Shaking Case 2b: Soil Profile with Fill 1985 Michoacan EQ, 2475-yr ARP

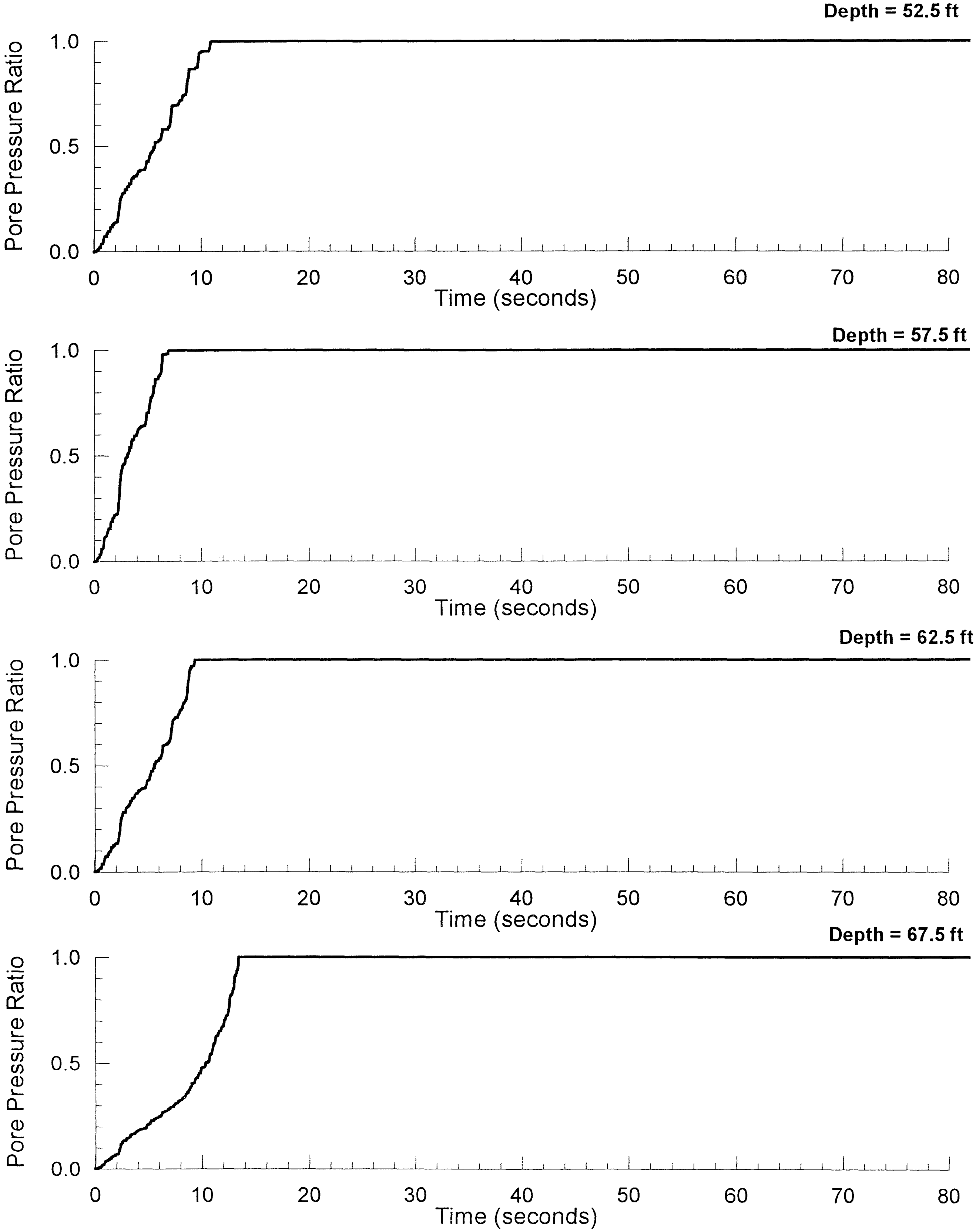


Figure 2b-3 Pore Pressure Generation
Case 2b: Soil Profile with Fill, 1985 Michoacan EQ, 2475-yr ARP

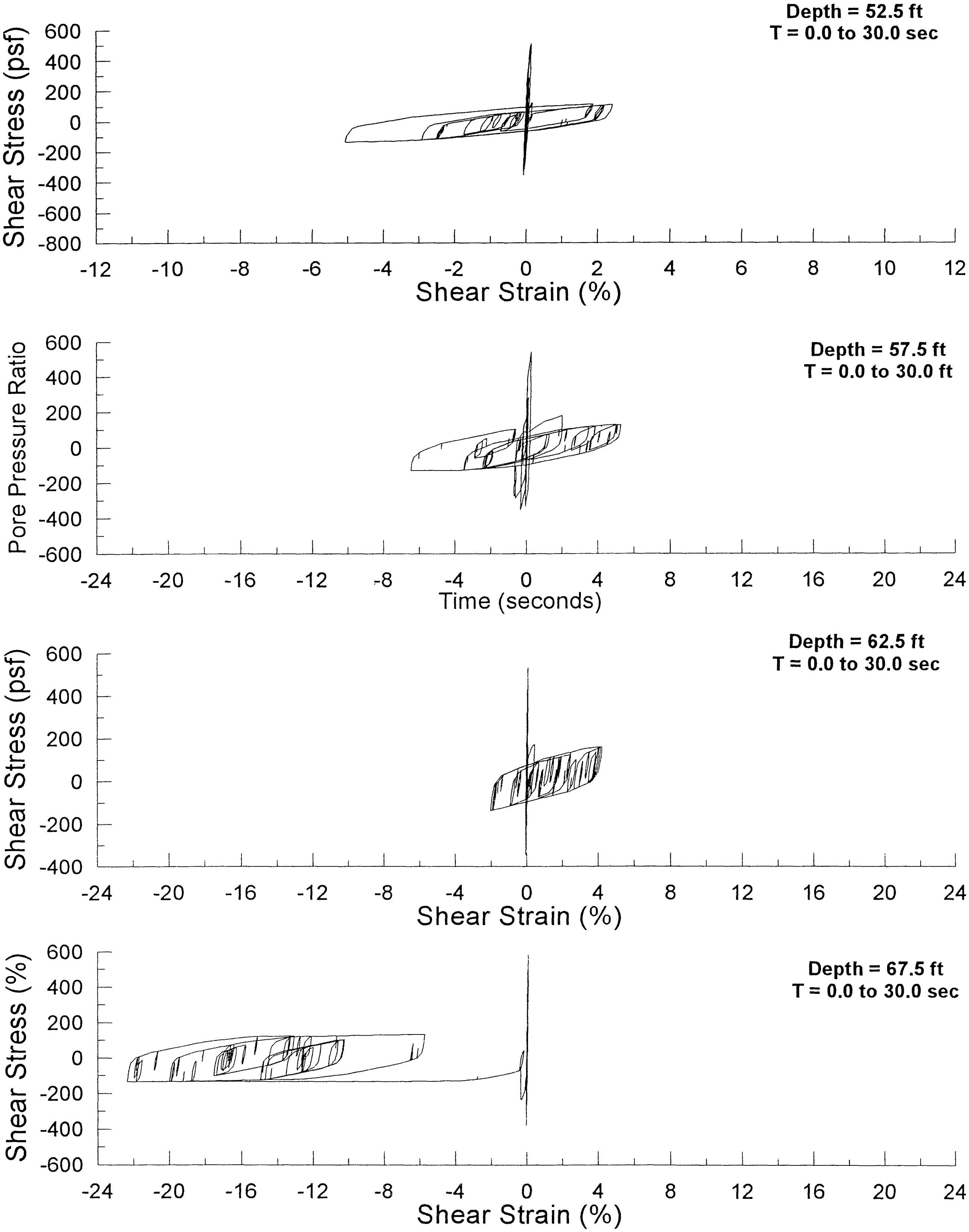


Figure 2b-4 Shear Stress - Shear Strain Loops
Case 2b: Soil Profile with Fill, 1985 Michoacan EQ, 2475-yr ARP

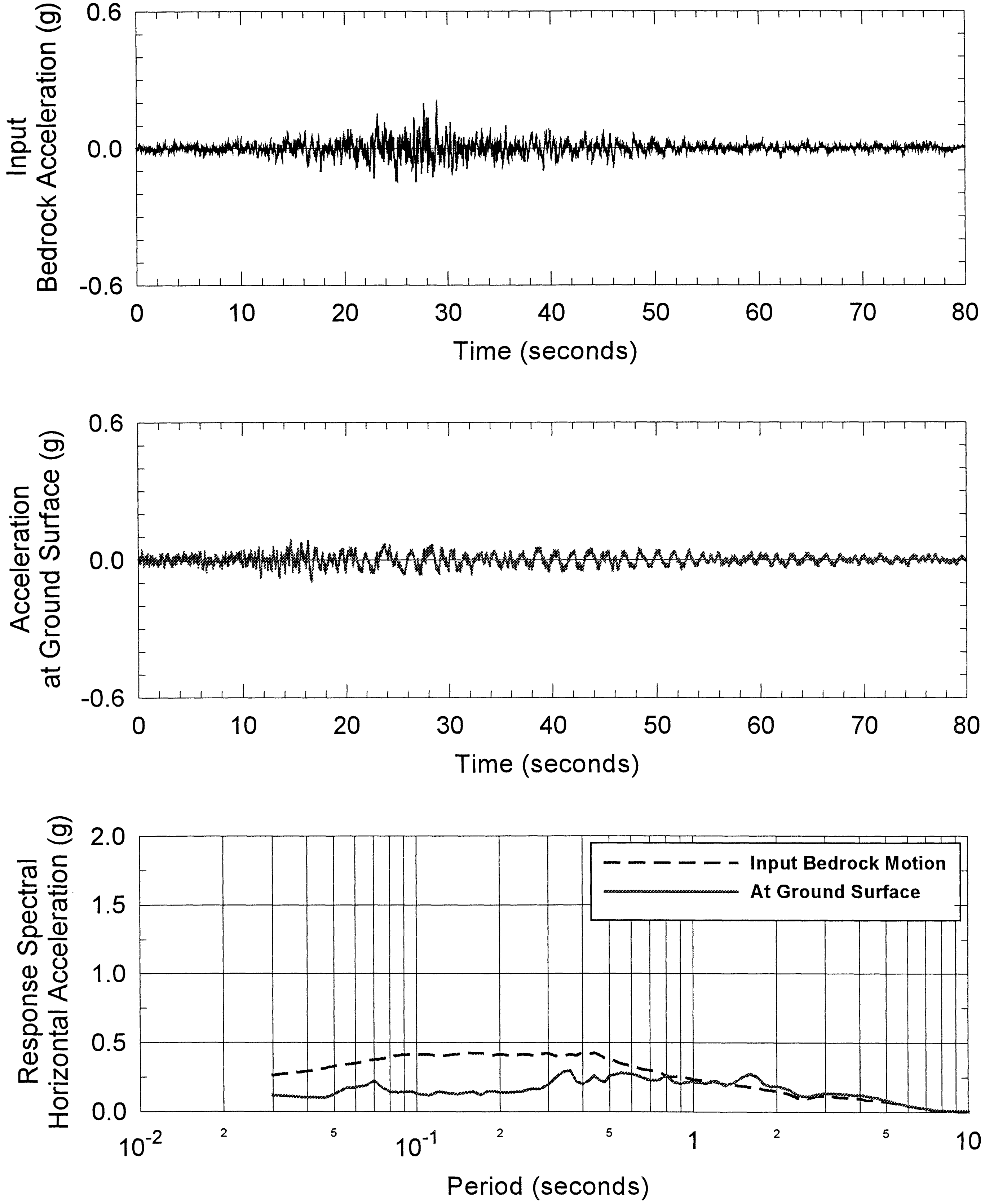


Figure 3a-1 Acceleration Time Histories and Response Spectra Case 3a: Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP

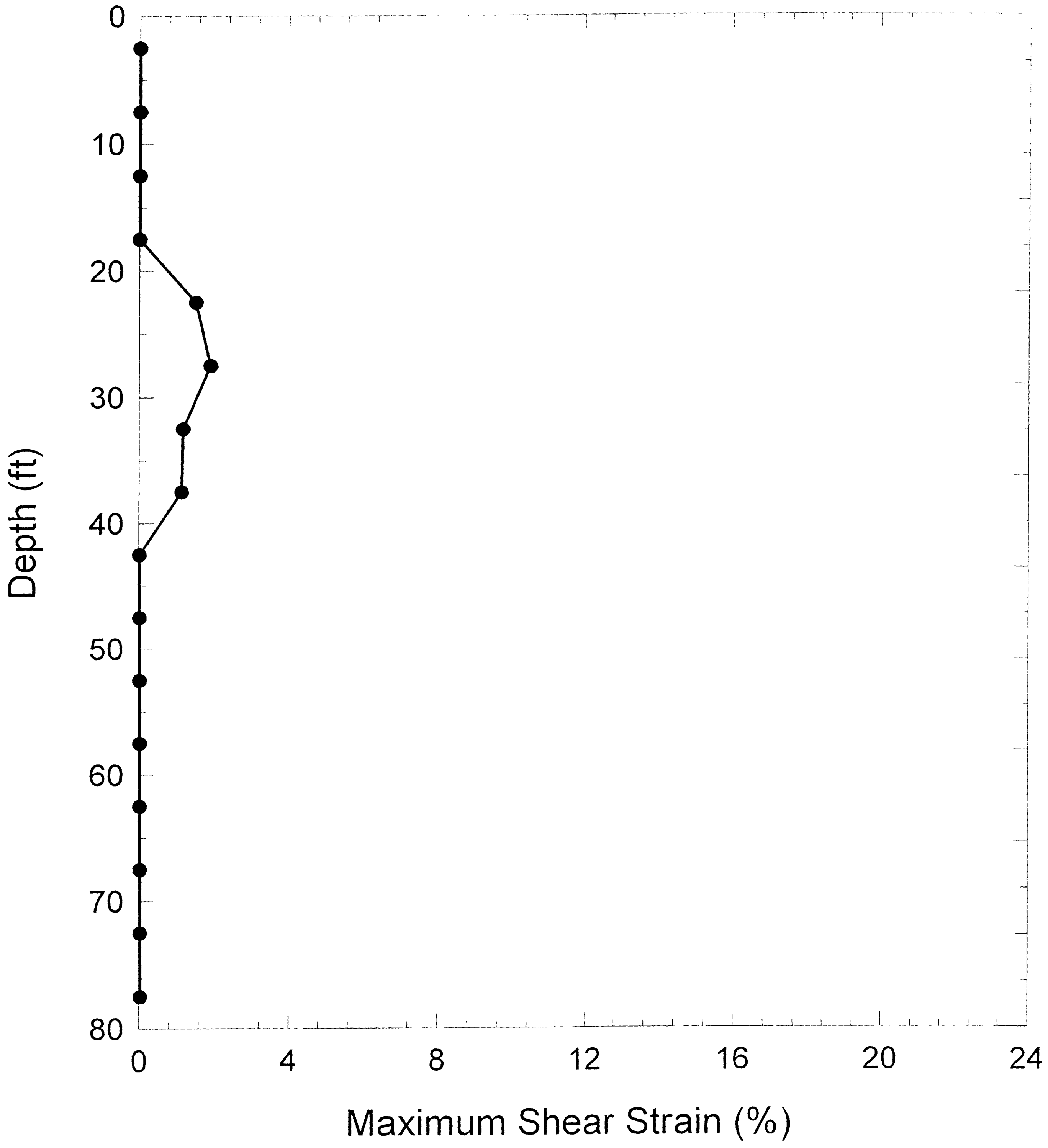


Figure 3a-2 Maximum Shear Strain Occurred During the Shaking Case 3a: Soil Profile without Fill 1985 Chile EQ, 475-yr ARP

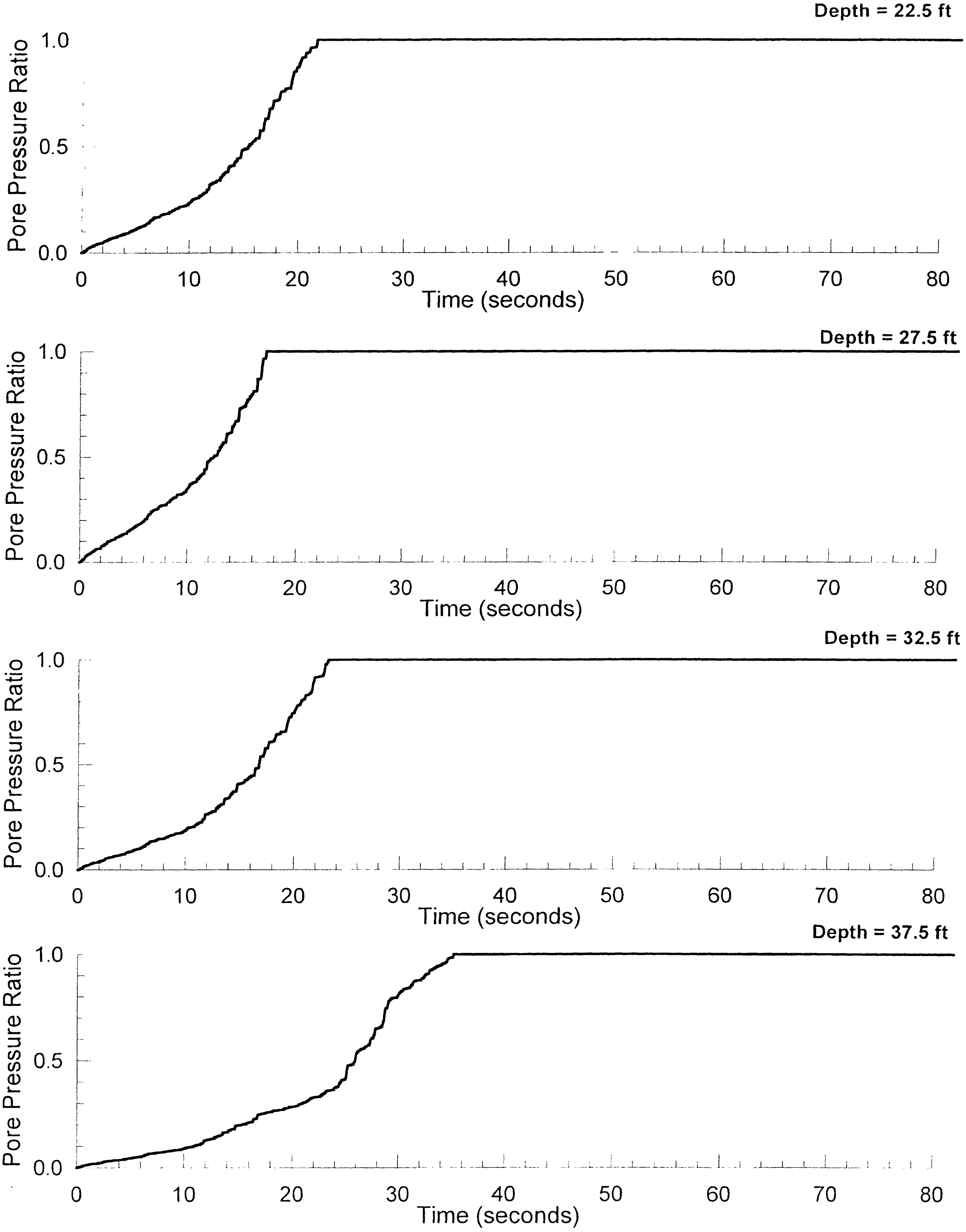


Figure 3a-3 Pore Pressure Generation
Case 3a: Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP

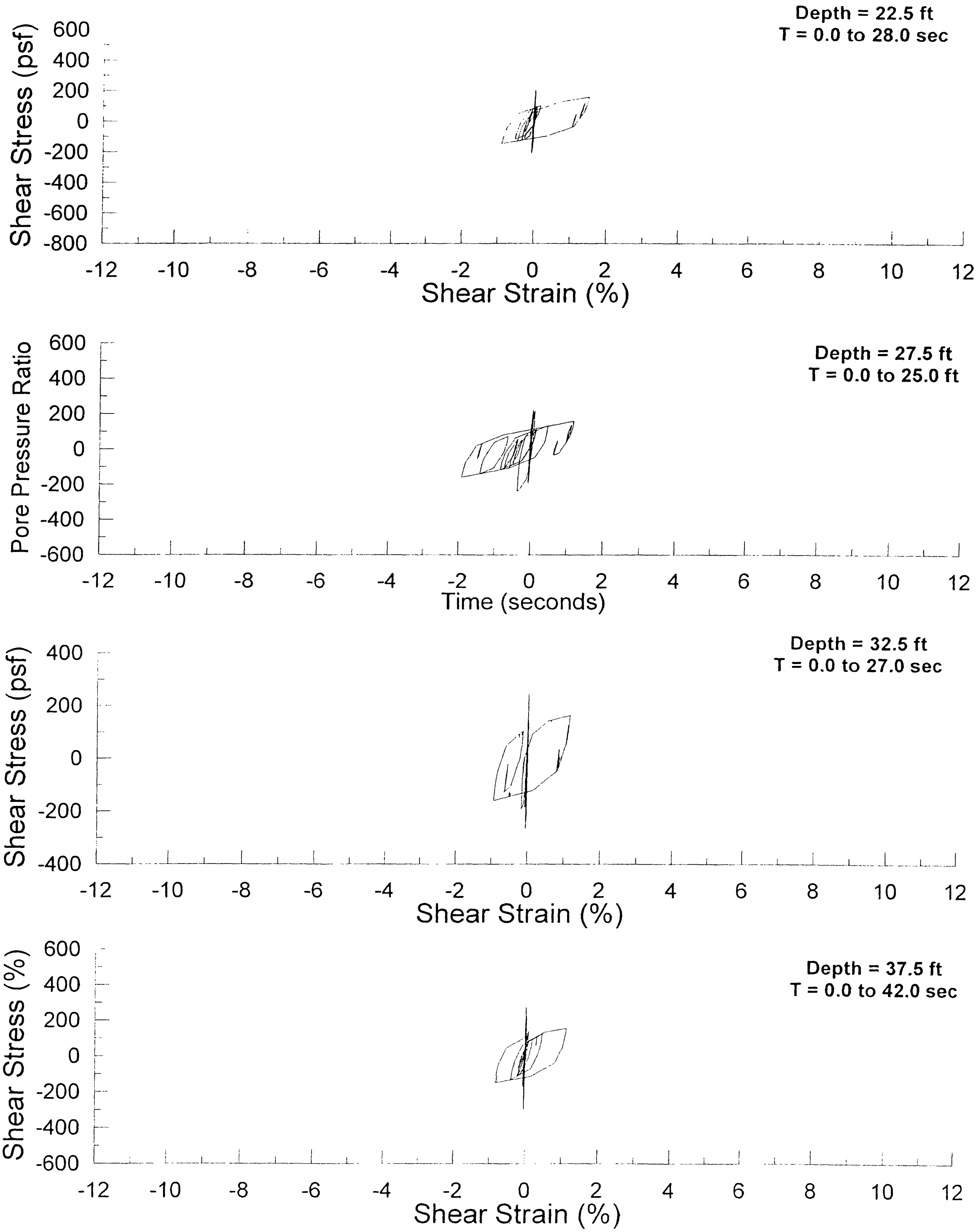


Figure 3a-4 Shear Stress - Shear Strain Loops
Case 3a: Soil Profile without Fill, 1985 Chile EQ, 475-yr ARP

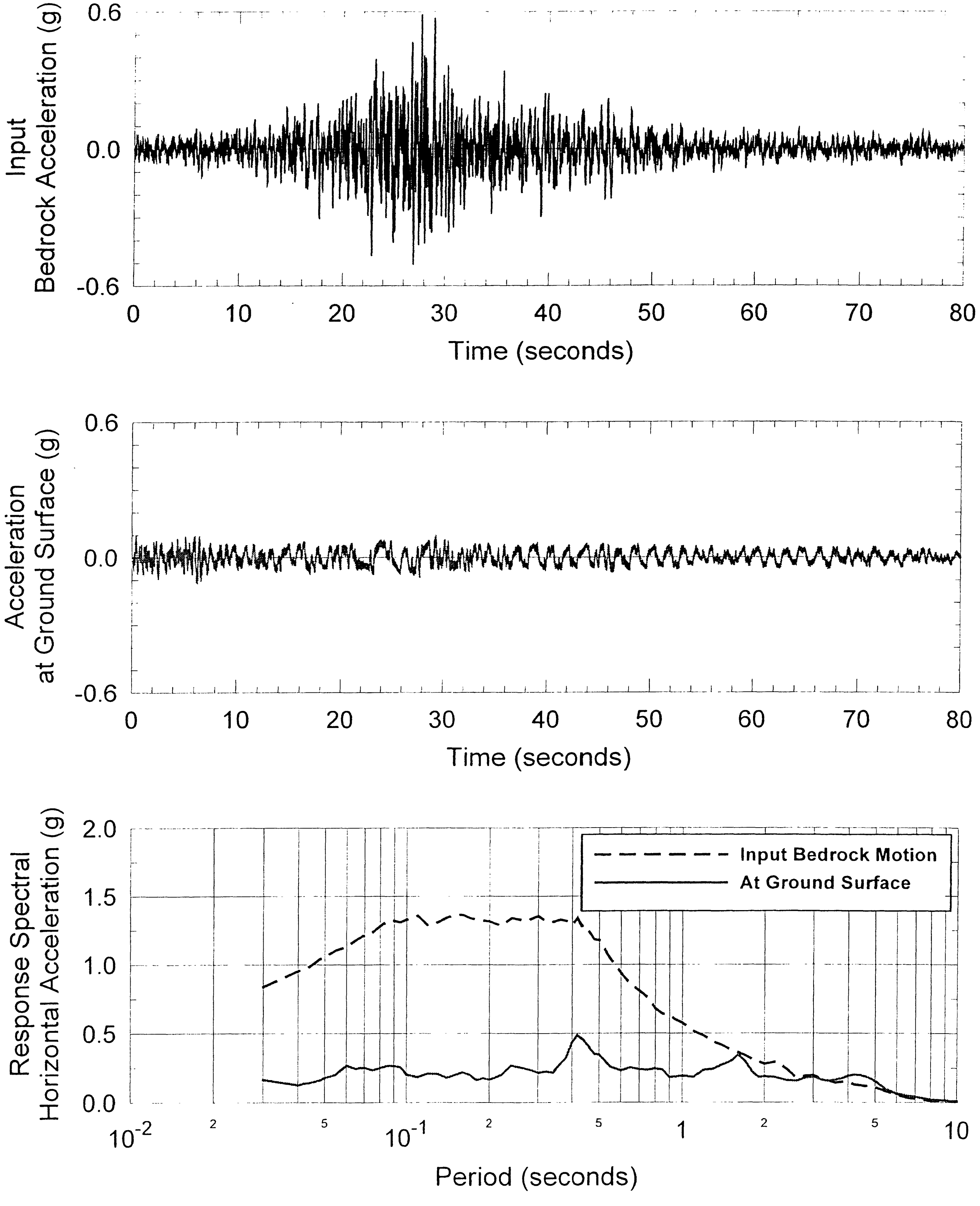


Figure 3b-1 Acceleration Time Histories and Response Spectra Case 3b: Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP

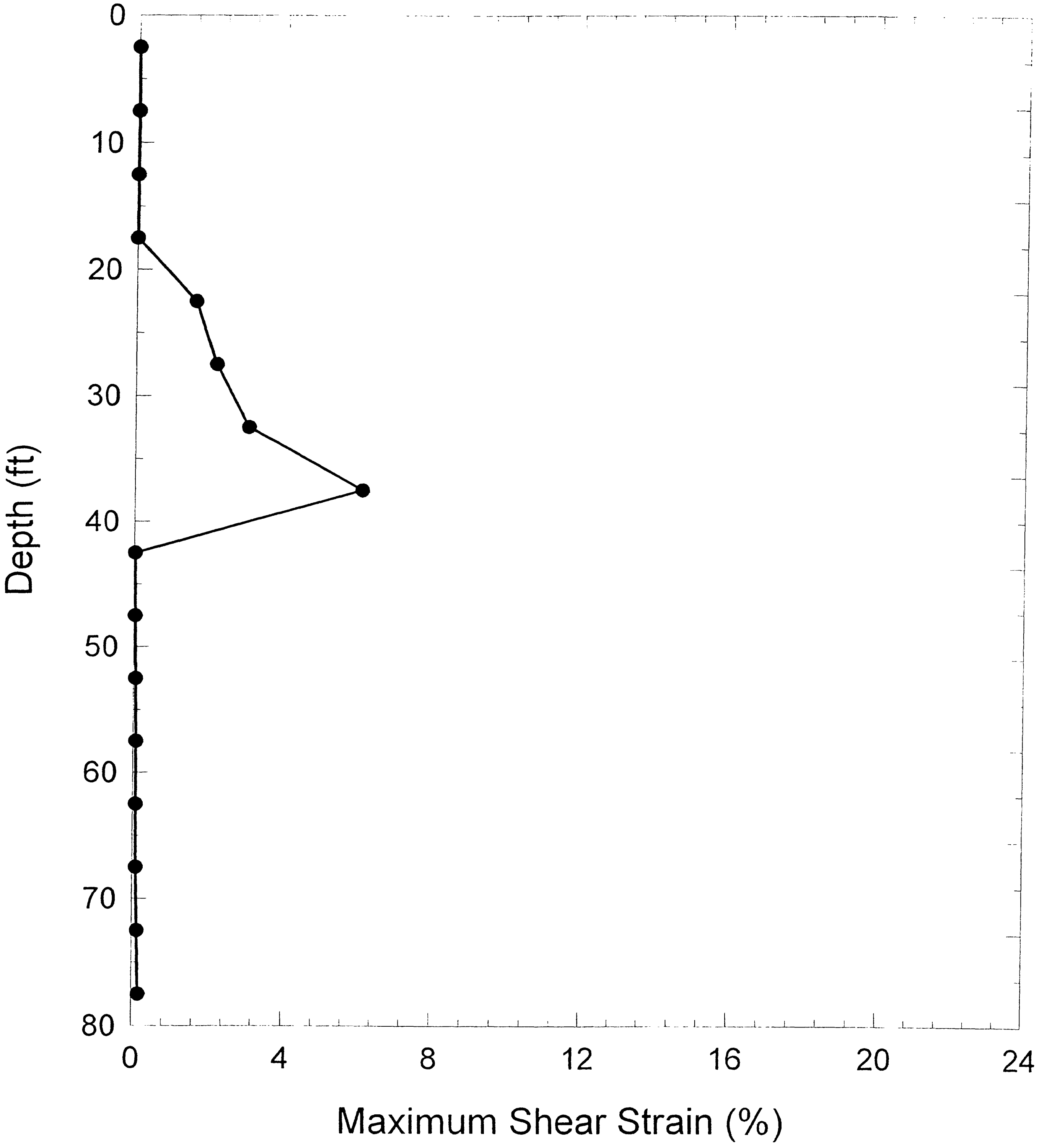


Figure 3b-2 Maximum Shear Strain Occurred During the Shaking Case 3b: Soil Profile without Fill 1985 Chile EQ, 2475-yr ARP

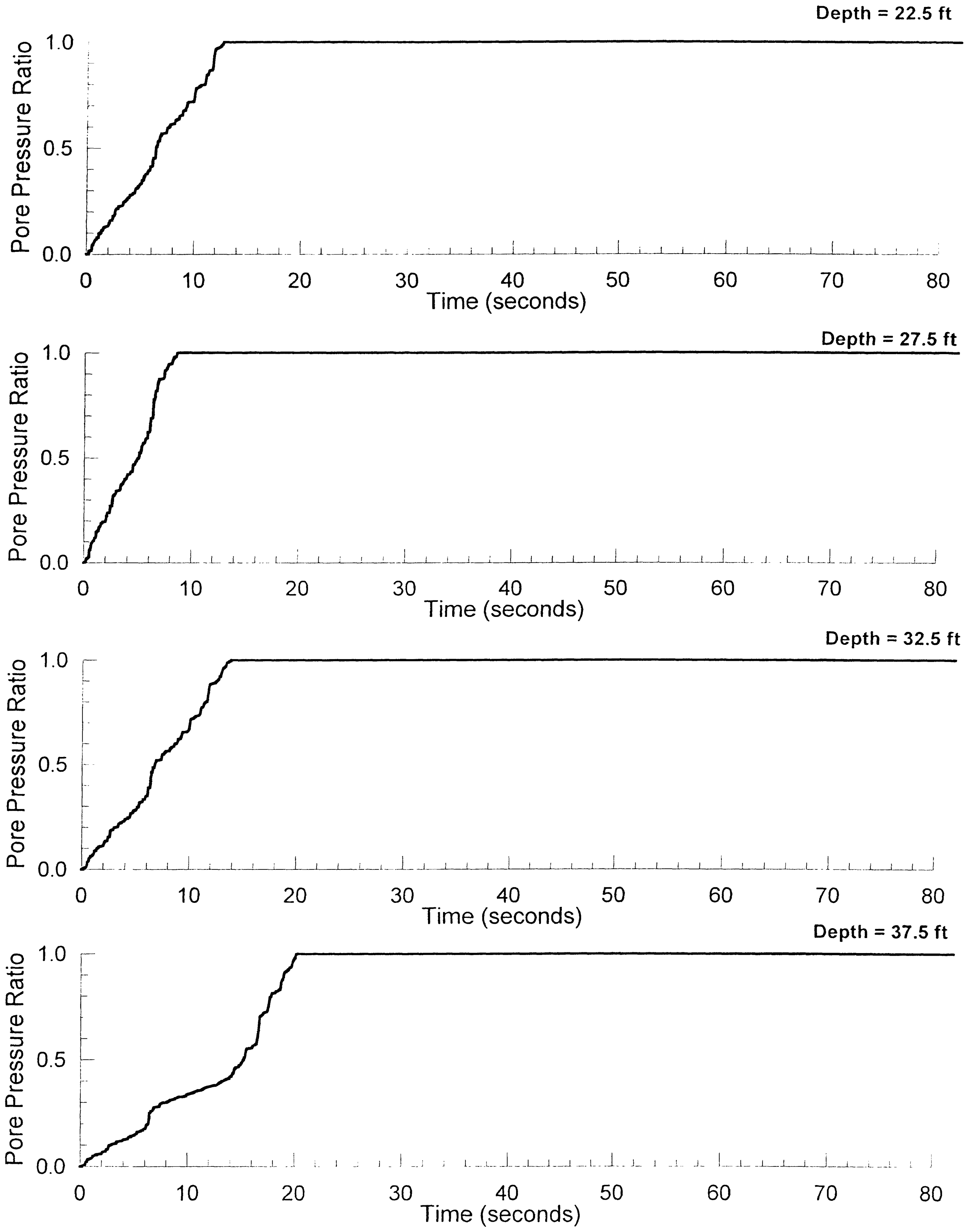


Figure 3b-3 Pore Pressure Generation
Case 3b: Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP

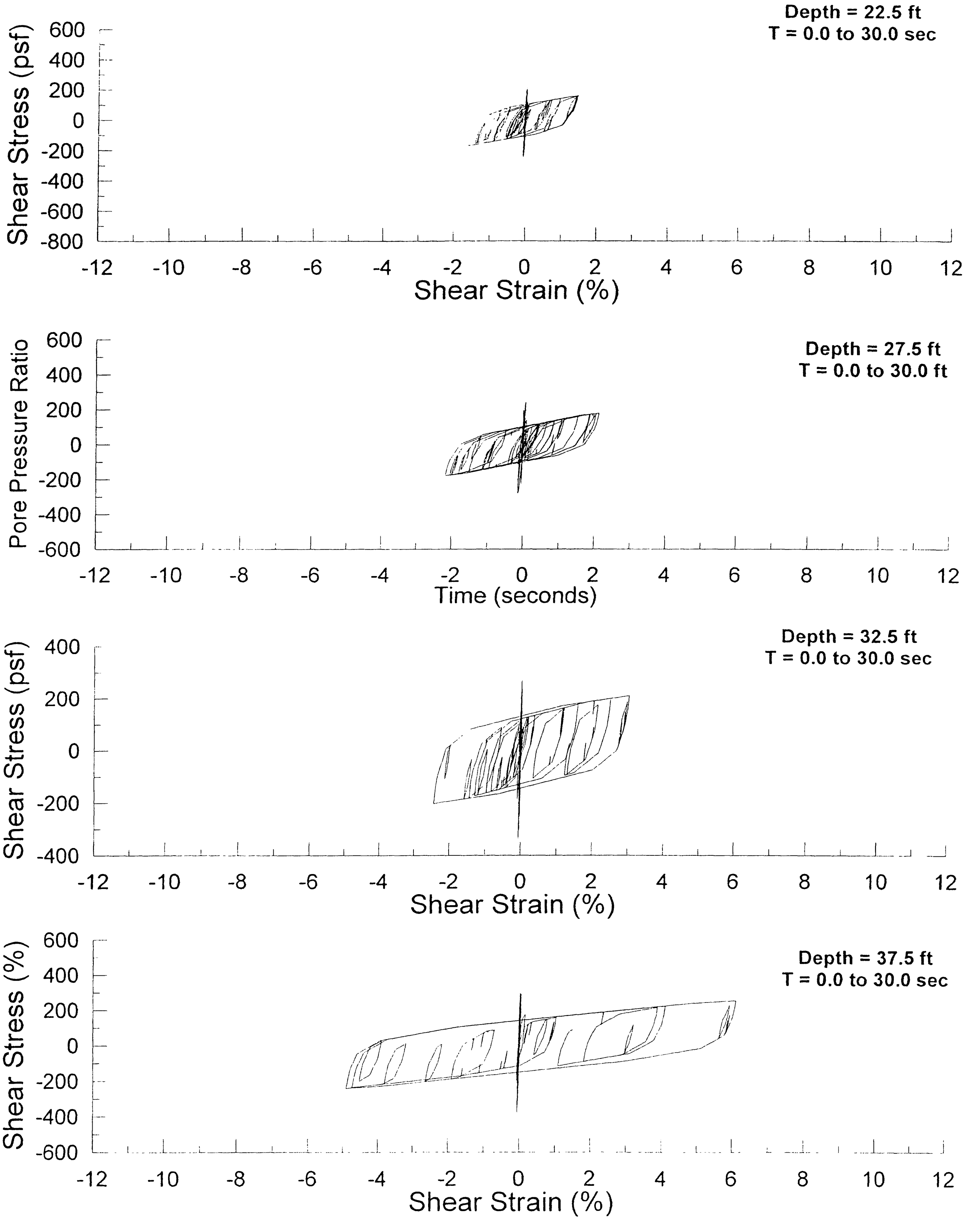


Figure 3b-4 Shear Stress - Shear Strain Loops
Case 3b: Soil Profile without Fill, 1985 Chile EQ, 2475-yr ARP

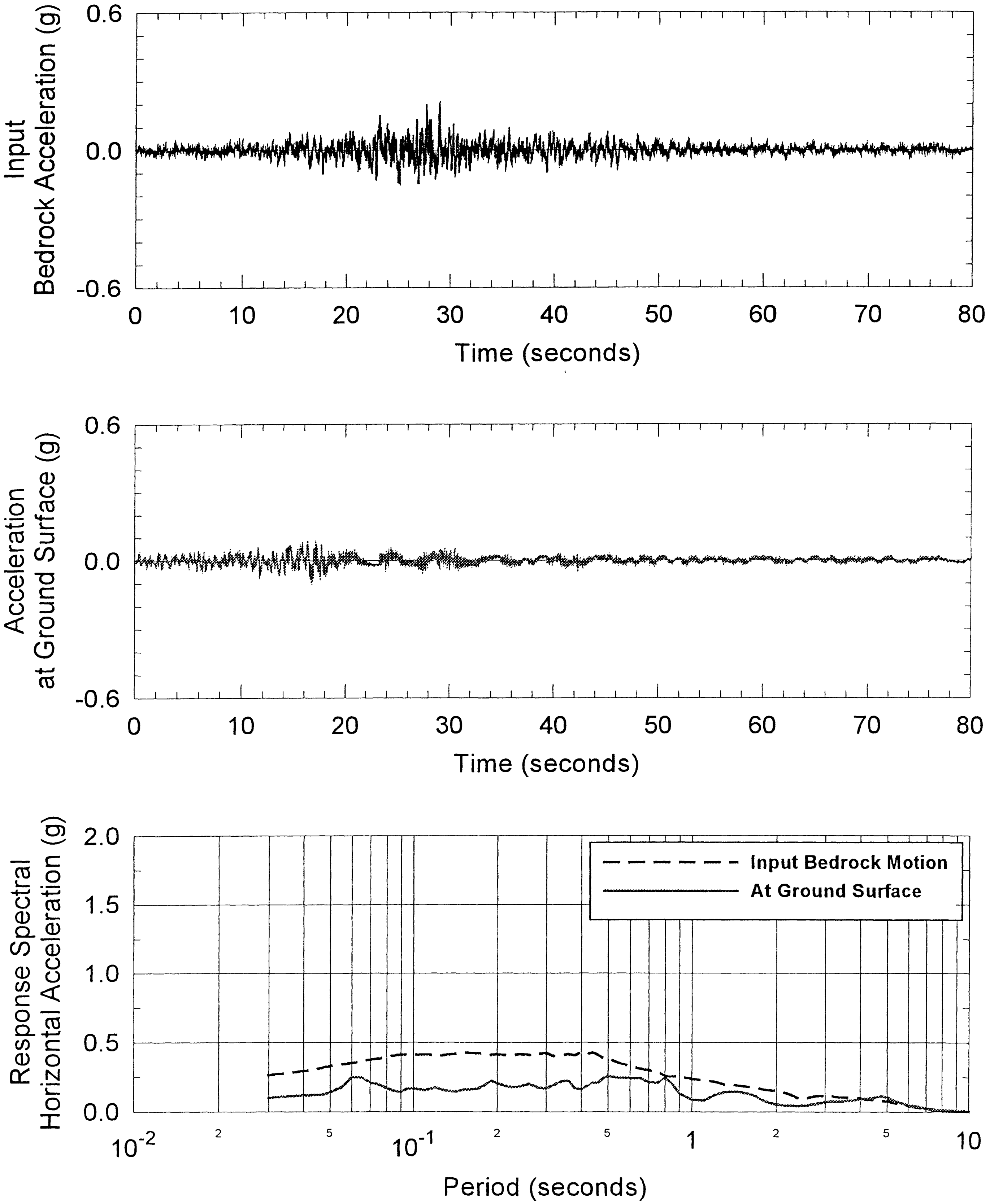


Figure 4a-1 Acceleration Time Histories and Response Spectra Case 4a: Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP

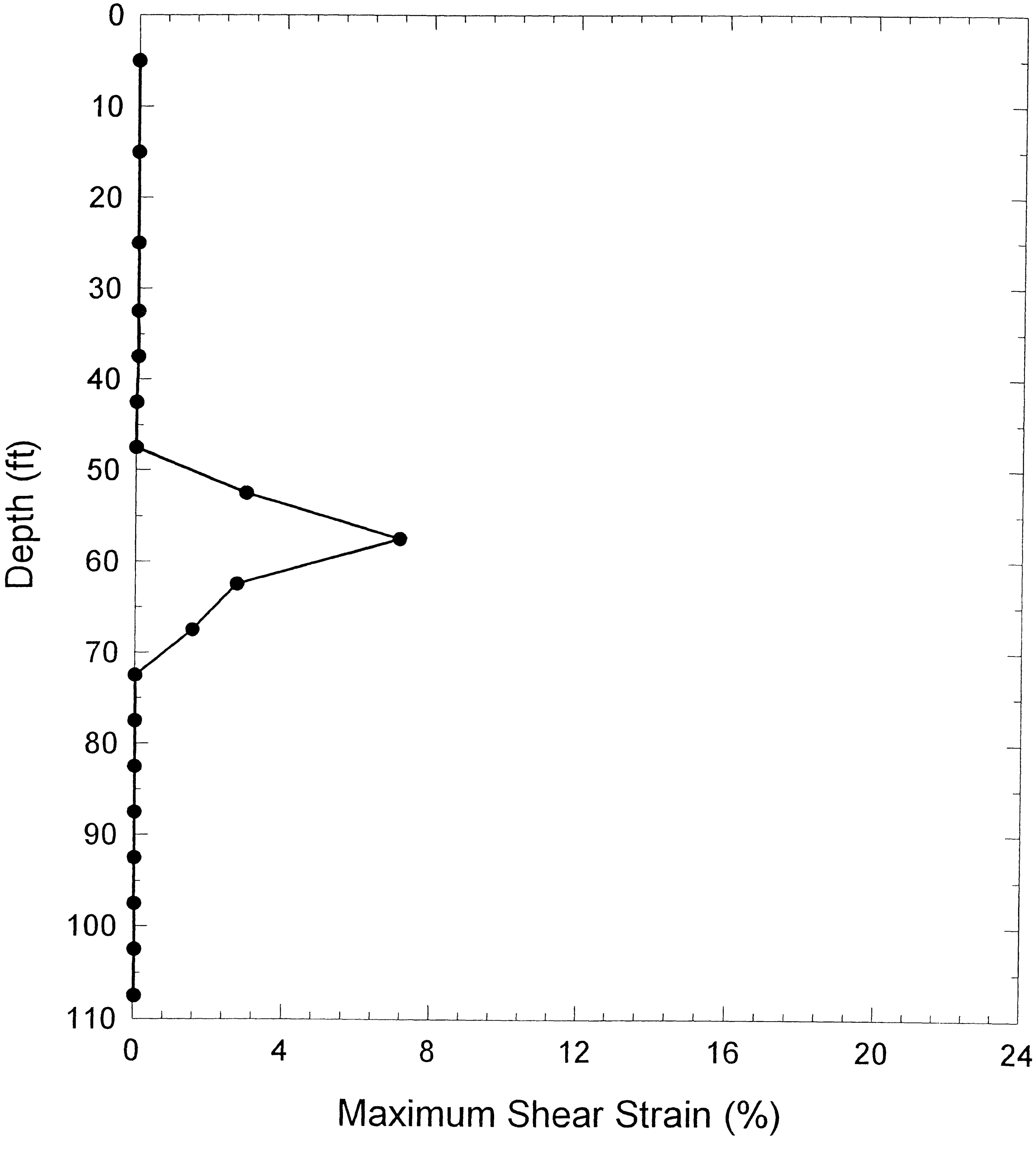


Figure 4a-2 Maximum Shear Strain Occurred During the Shaking
Case 4a: Soil Profile with Fill
1985 Chile EQ, 475-yr ARP

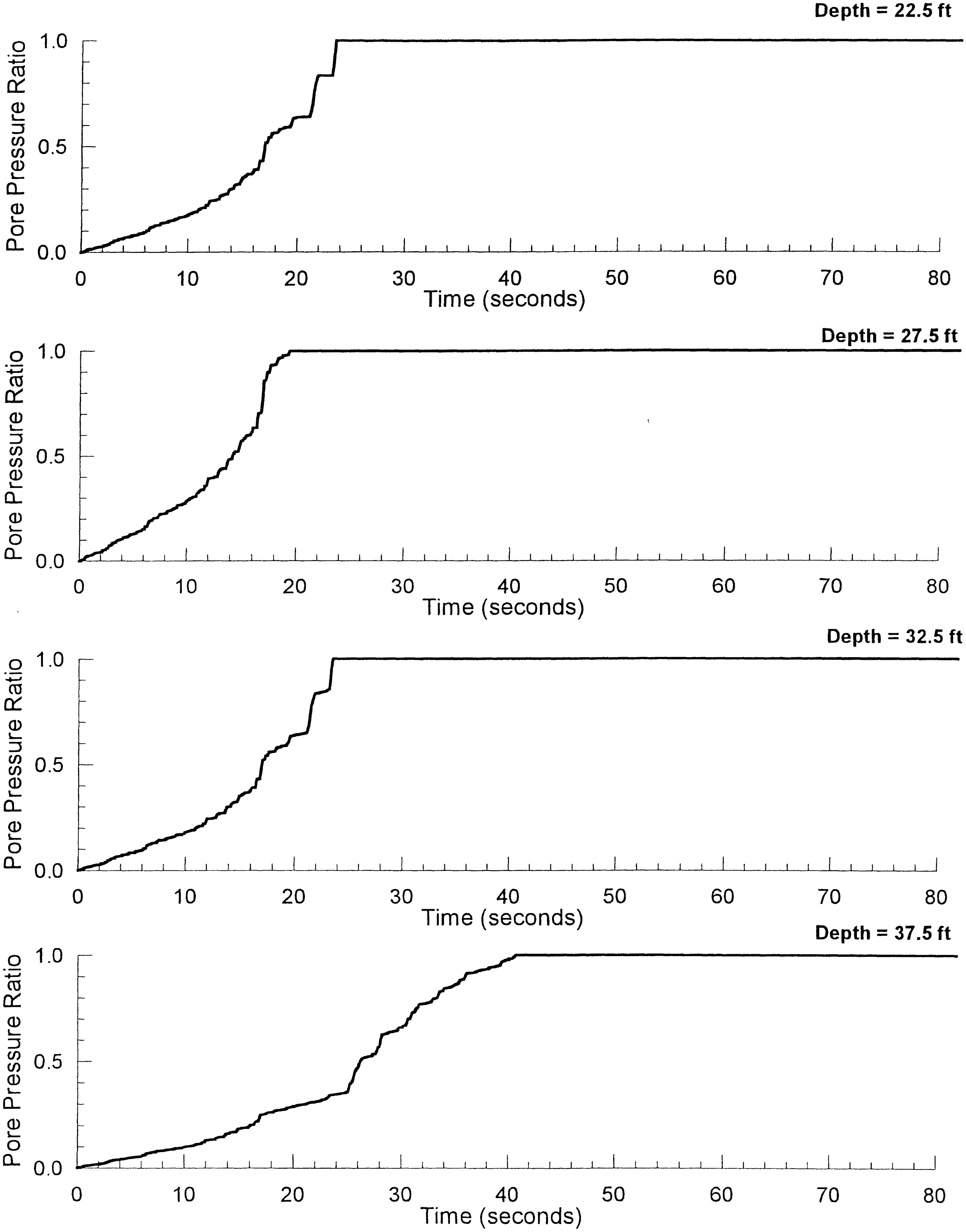


Figure 4a-3 Pore Pressure Generation
Case 4a: Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP

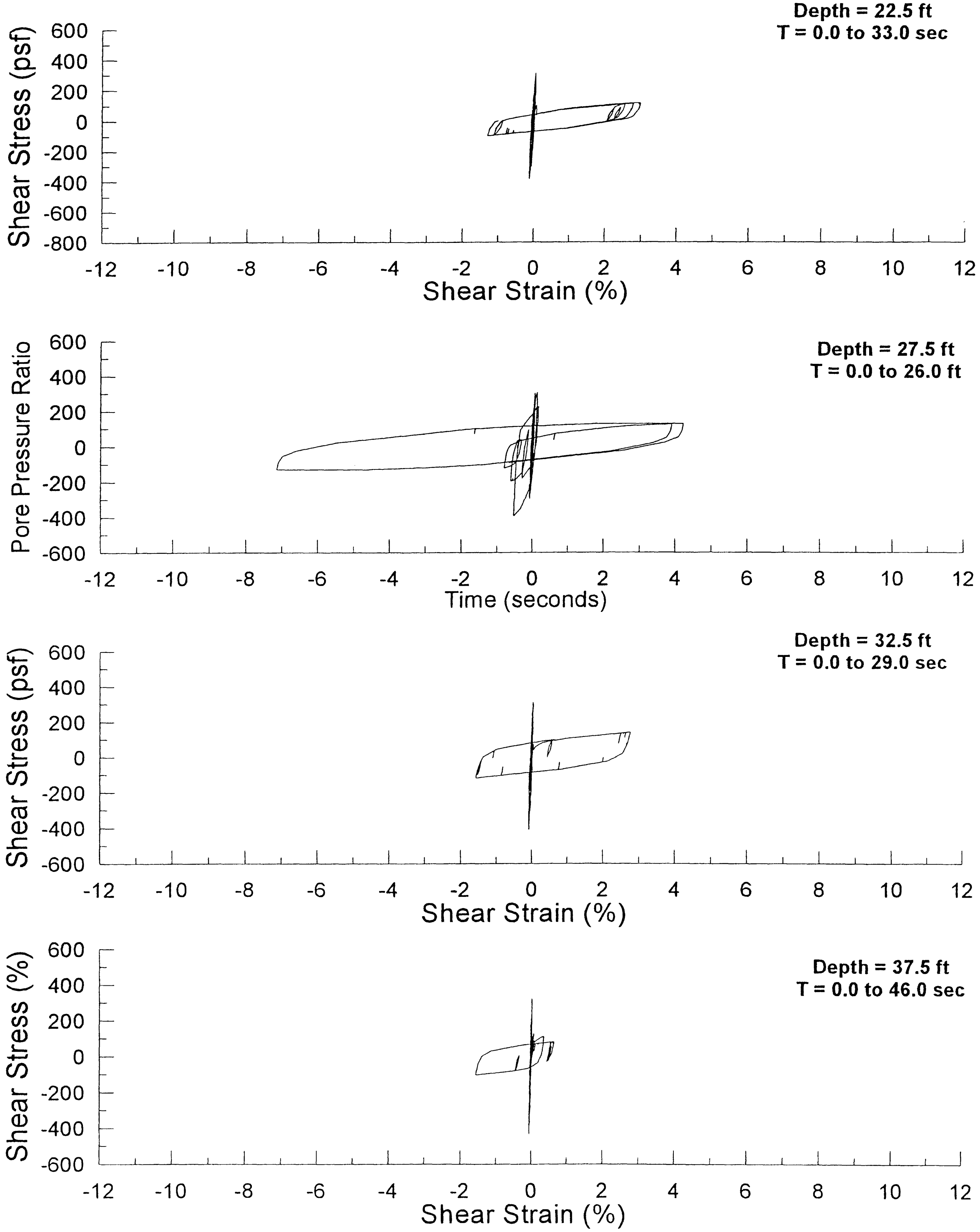


Figure 4a-4 Shear Stress - Shear Strain Loops
Case 4a: Soil Profile with Fill, 1985 Chile EQ, 475-yr ARP

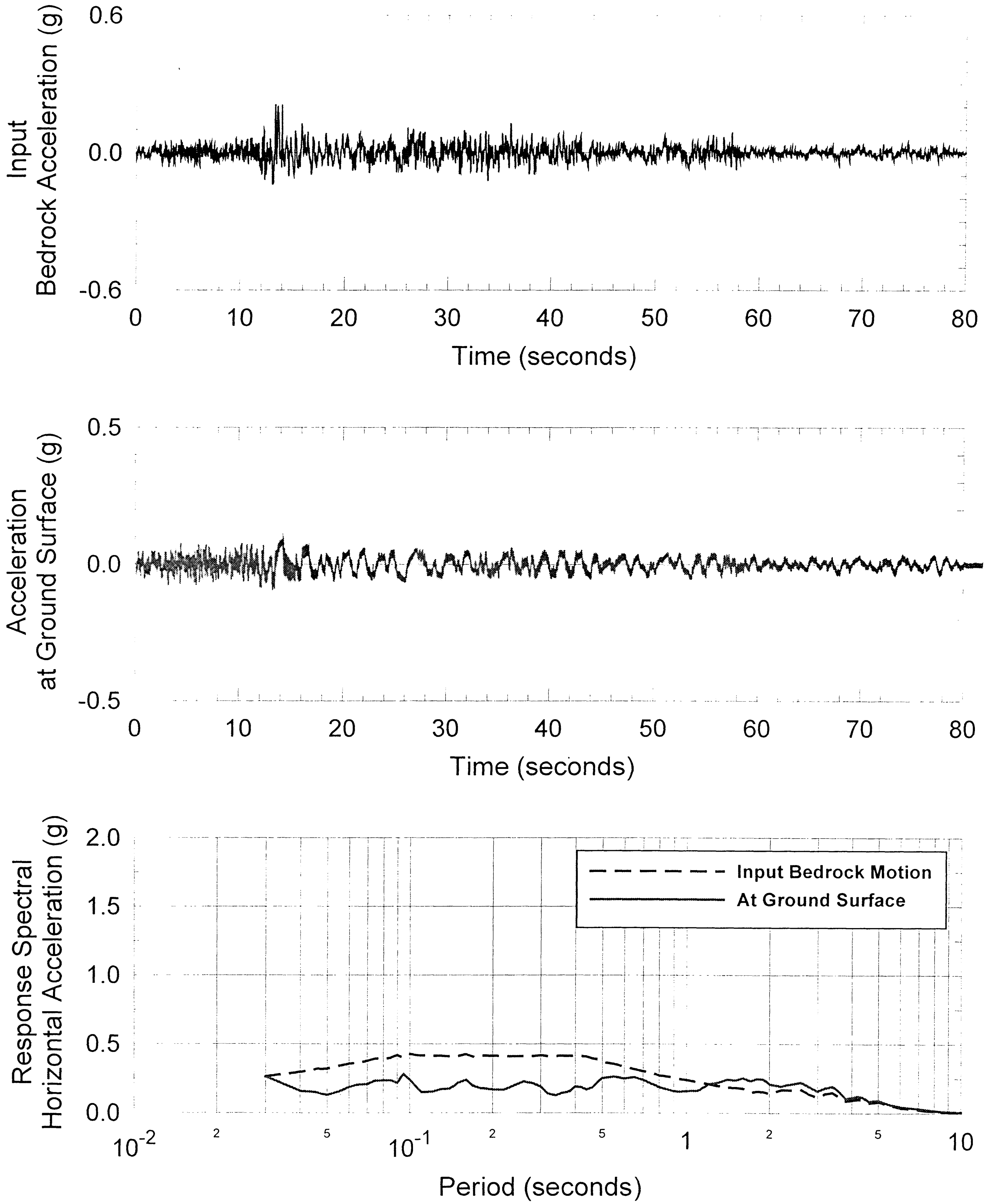


Figure 5a-1 Acceleration Time Histories and Response Spectra Case 5a: Soil Profile without Fill, 1968 Tokachi-Oki EQ, 475-yr ARP

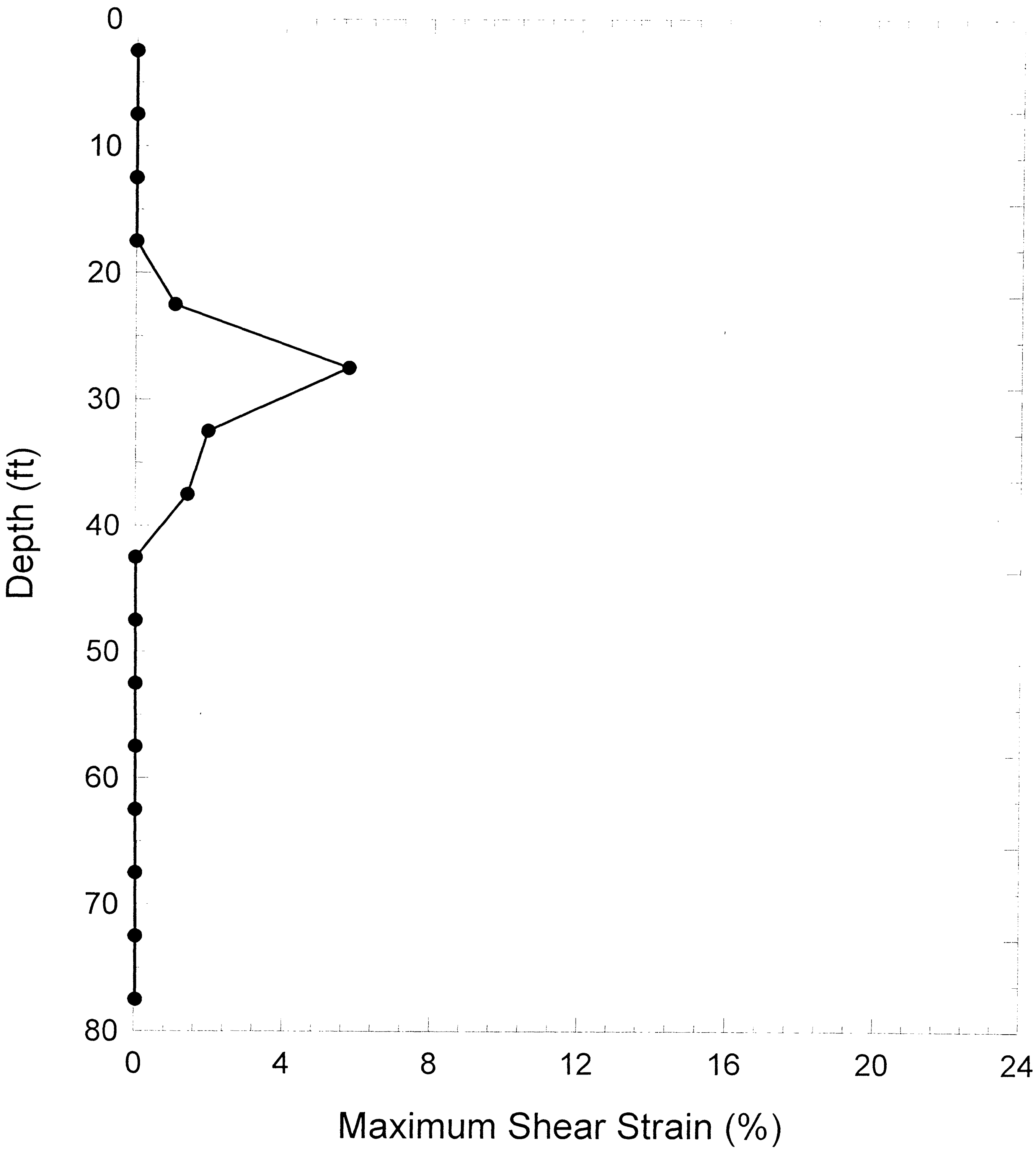


Figure 5a-2 Maximum Shear Strain Occurred During the Shaking
Case 5a: Soil Profile without Fill
1968 Tokachi-Oki EQ, 475-yr ARP

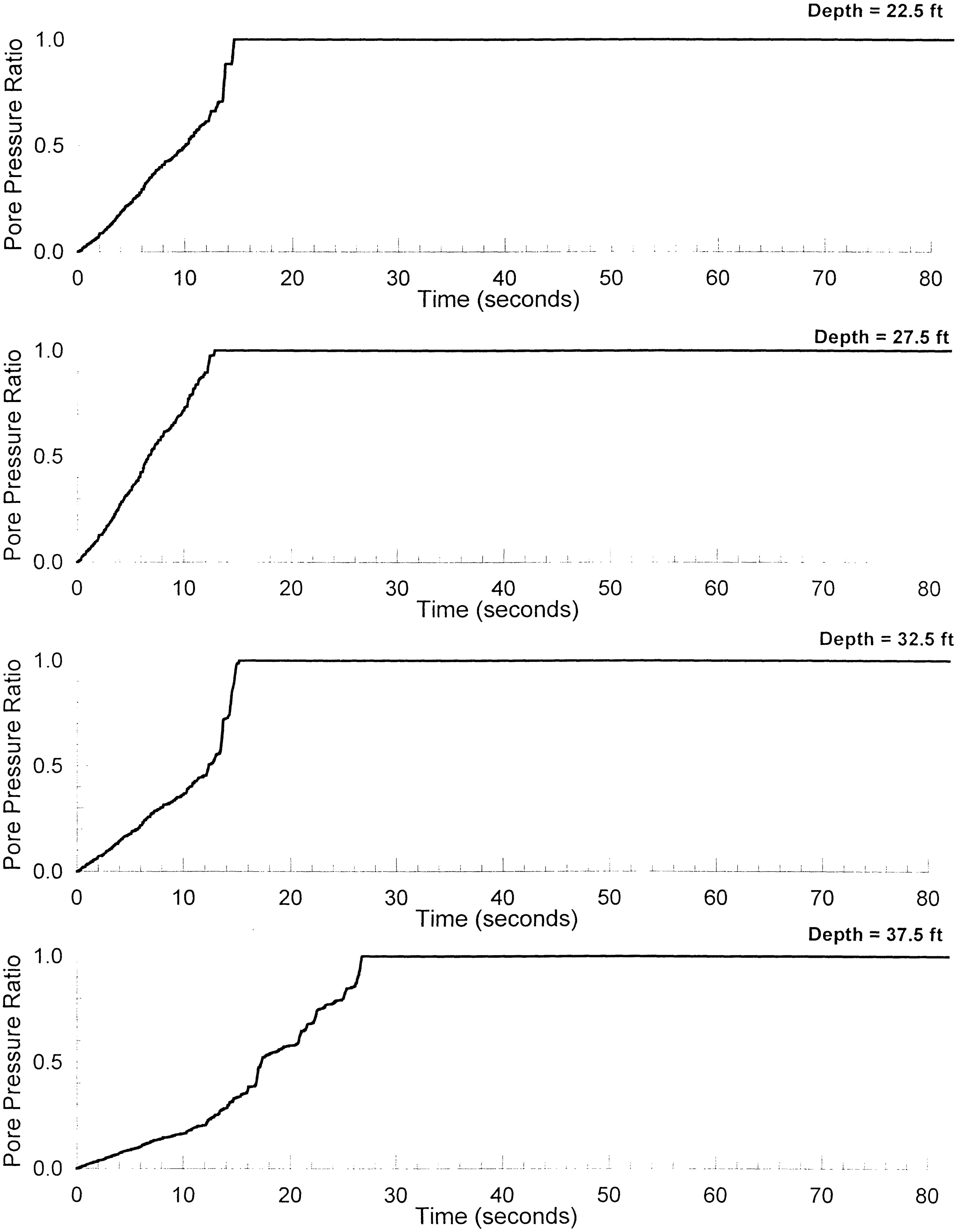


Figure 5a-3 Pore Pressure Generation
Case 5a: Soil Profile without Fill, 1968 Tokachi-Oki EQ, 475-yr ARP

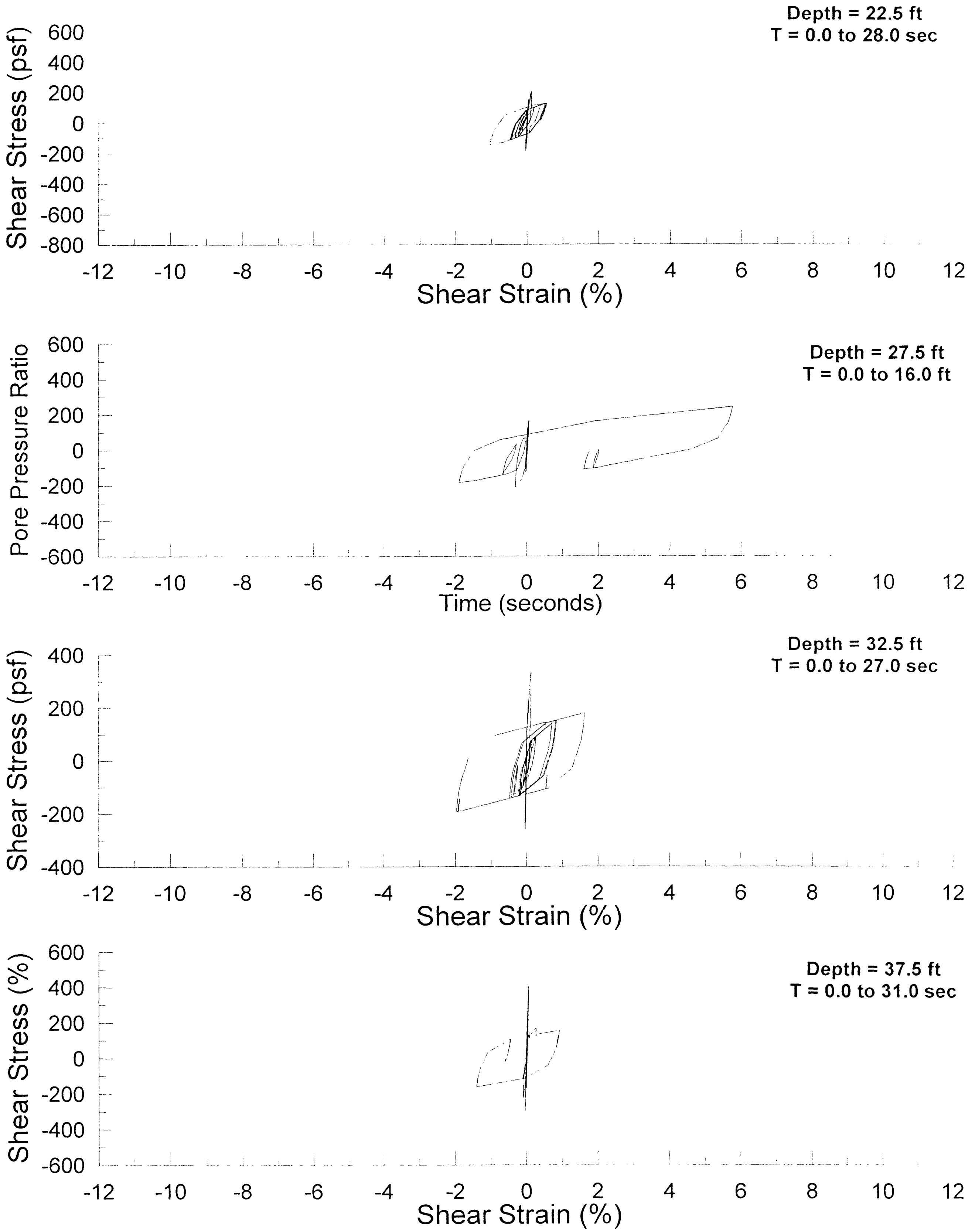


Figure 5a-4 Shear Stress - Shear Strain Loops
Case 5a: Soil Profile without Fill, 1968 Tokachi-Oki EQ, 475-yr ARP

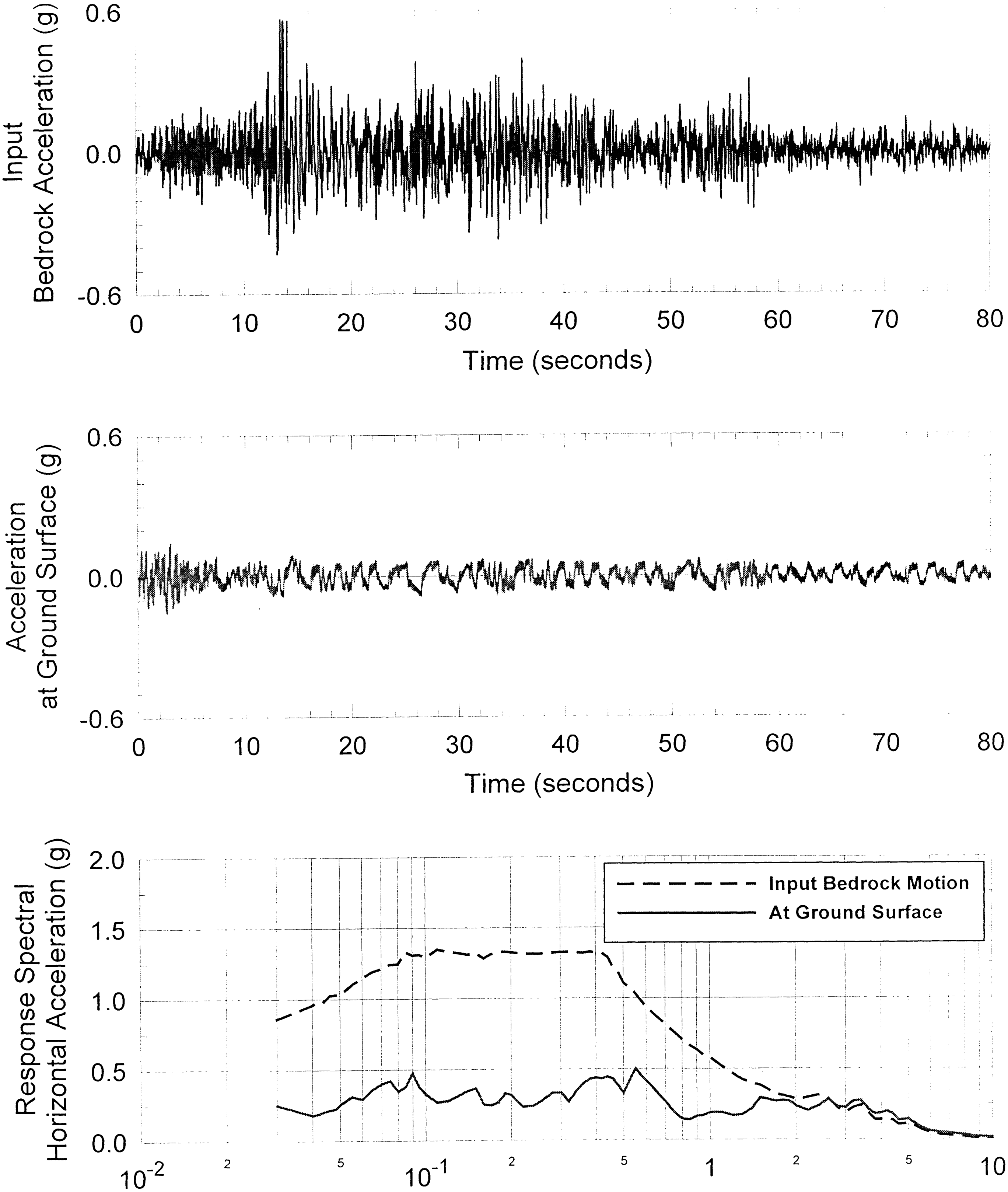


Figure 5b-1 Acceleration Time Histories and Response Spectra Case 5b: Soil Profile without Fill, 1968 Tokachi-Oki EQ, 2475-yr ARP

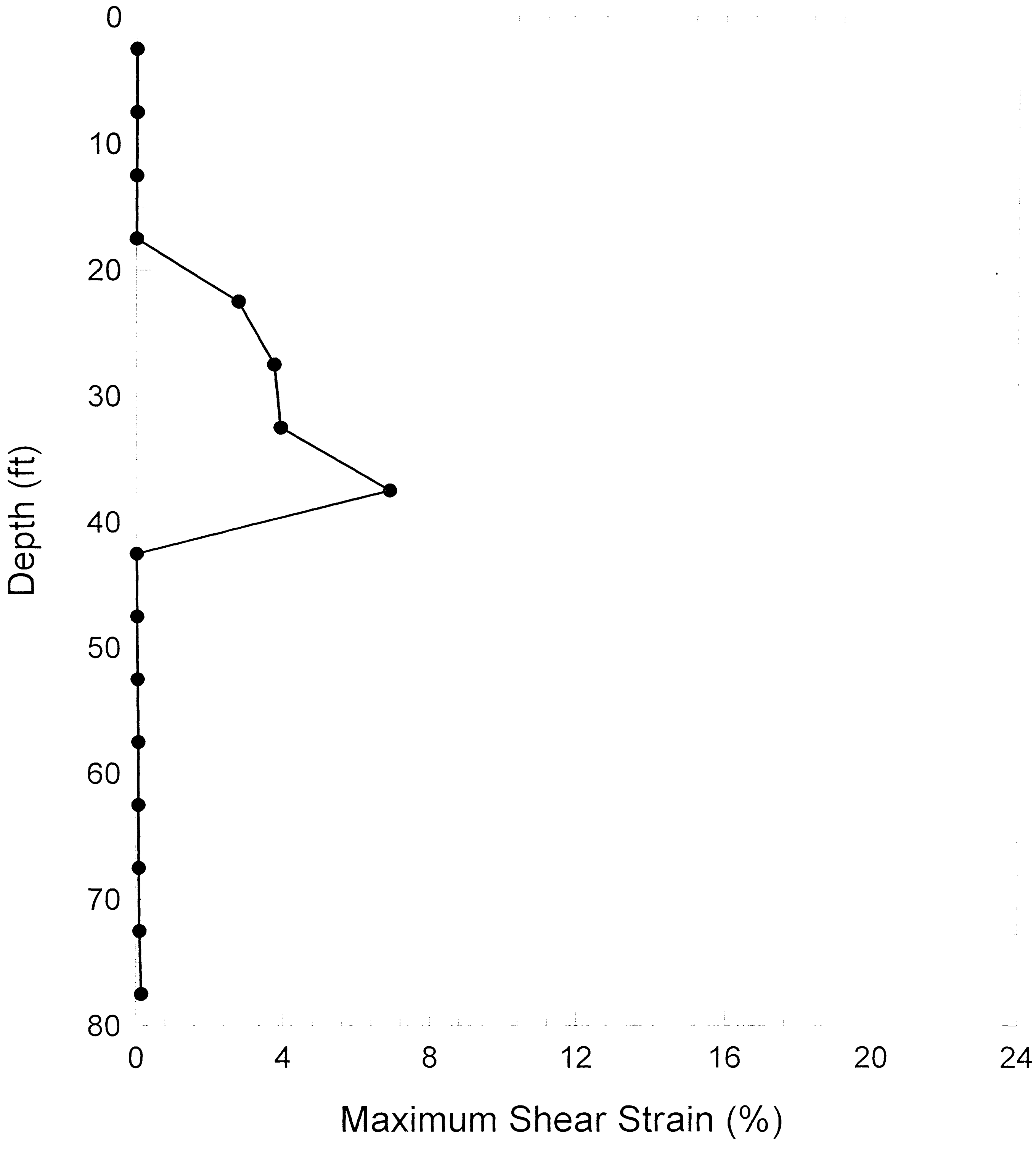


Figure 5b-2 Maximum Shear Strain Occurred During the Shaking
Case 5b: Soil Profile without Fill
1968 Tokachi-Oki EQ, 2475-yr ARP

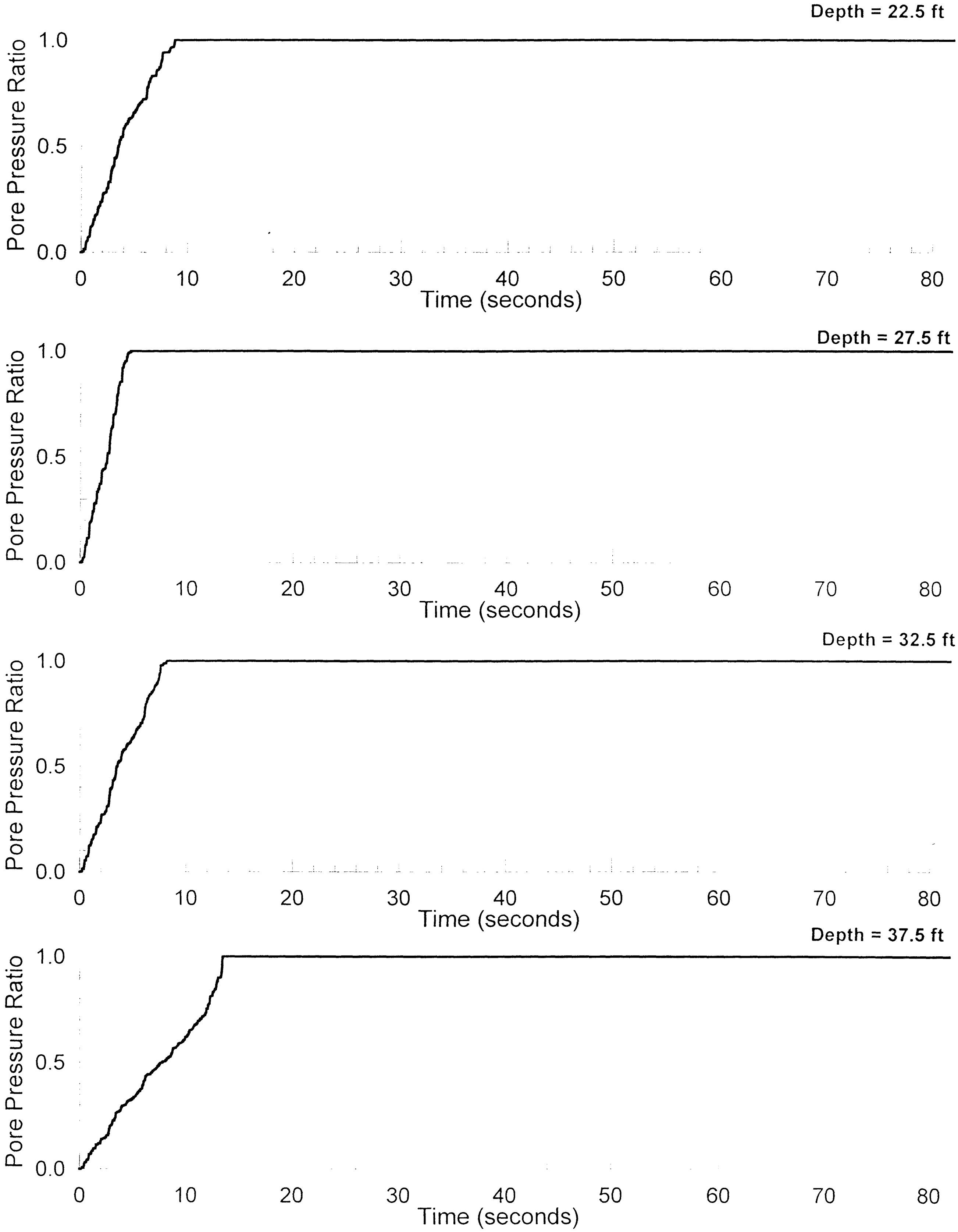


Figure 5b-3 Pore Pressure Generation
Case 5b: Soil Profile without Fill, 1968 Tokachi-Oki EQ, 2475-yr ARP

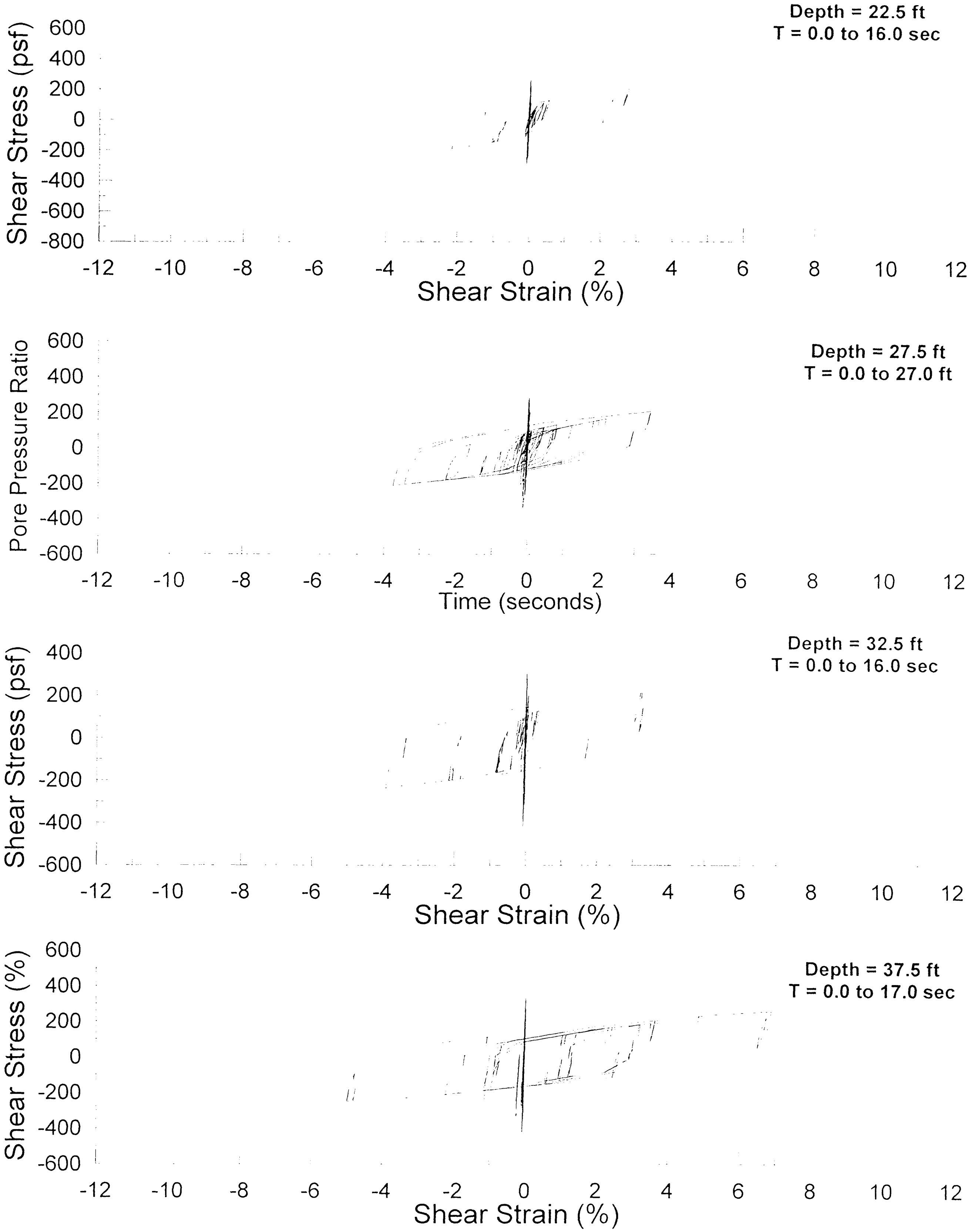


Figure 5b-4 Shear Stress - Shear Strain Loops
Case 5b: Soil Profile without Fill, 1968 Tokachi-Oki EQ, 2475-yr ARP

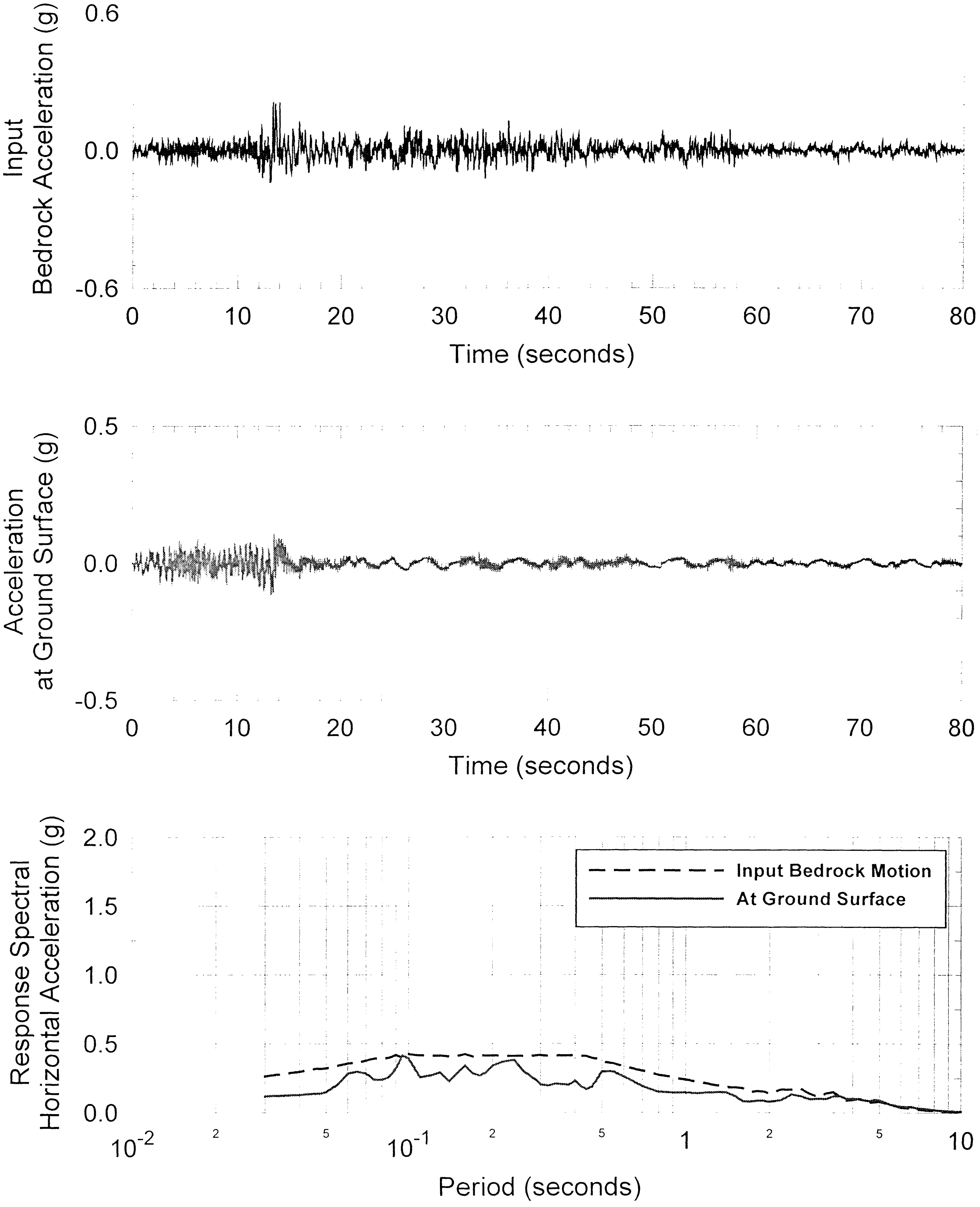


Figure 6a-1 Acceleration Time Histories and Response Spectra Case 6a: Soil Profile with Fill, 1968 Tokachi-Oki EQ, 475-yr ARP

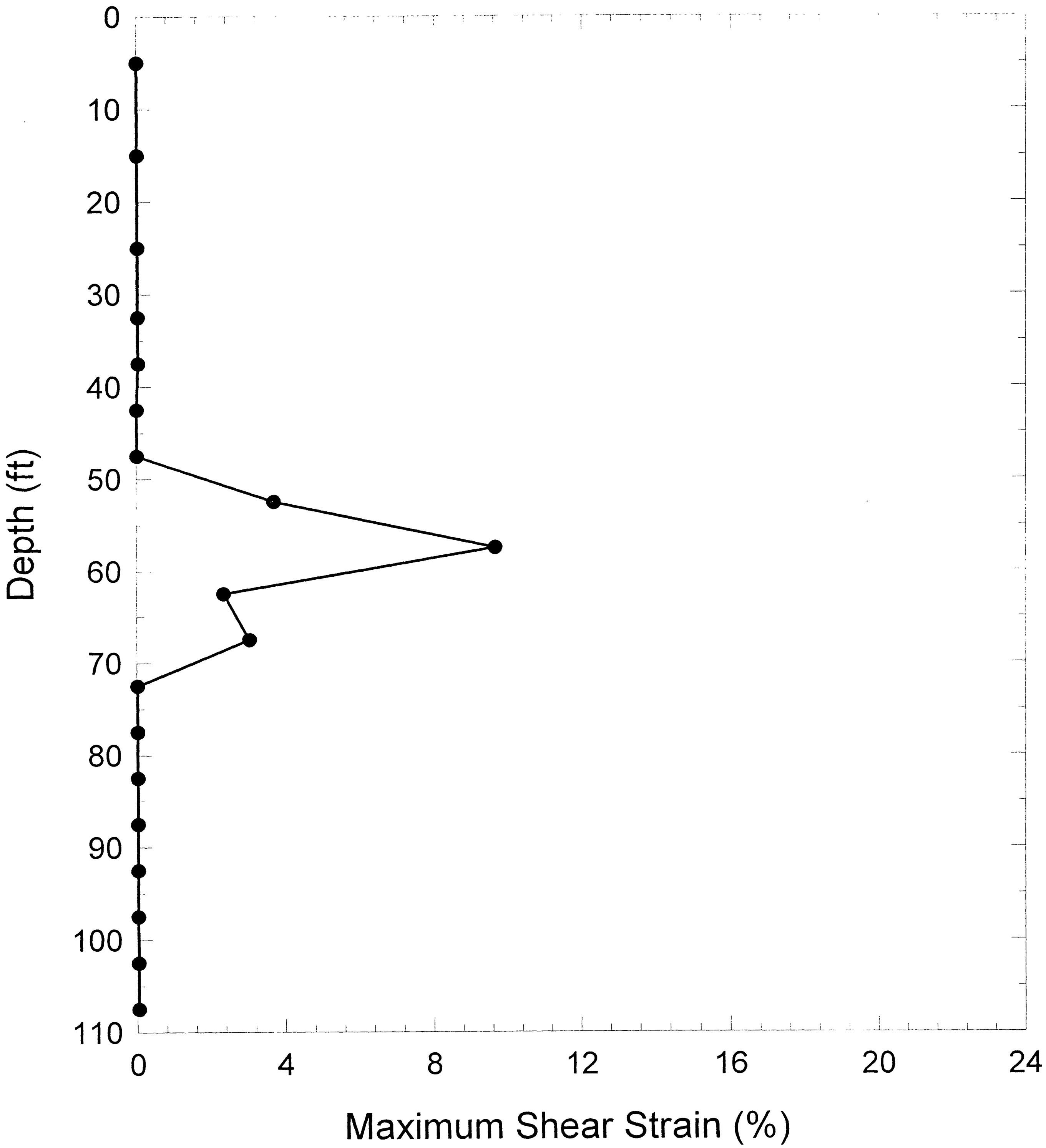


Figure 6a-2 Maximum Shear Strain Occurred During the Shaking
Case 6a: Soil Profile with Fill
1968 Tokachi-Oki EQ, 475-yr ARP

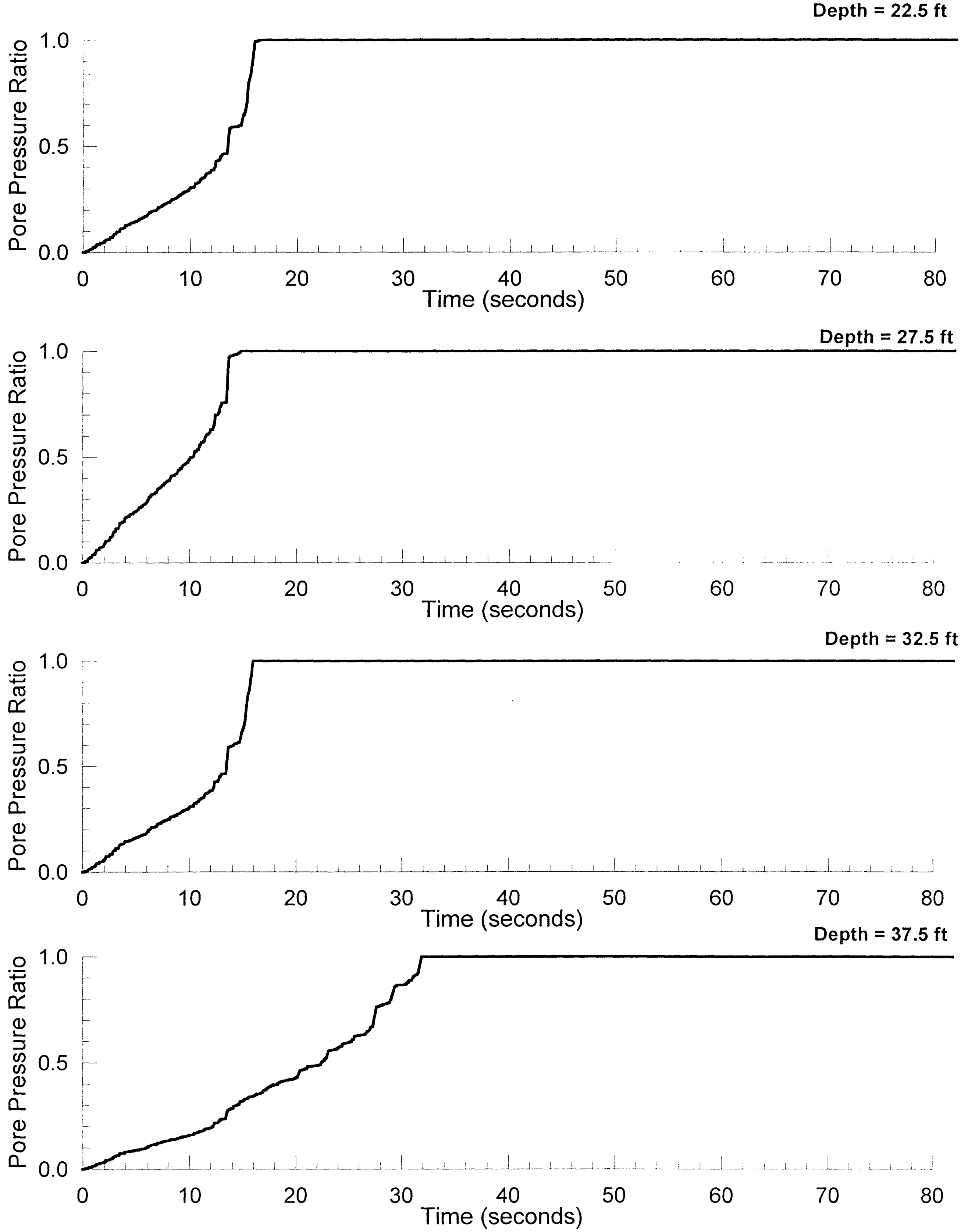


Figure 6a-3 Pore Pressure Generation
Case 6a: Soil Profile with Fill, 1968 Tokachi-Oki EQ, 475-yr ARP

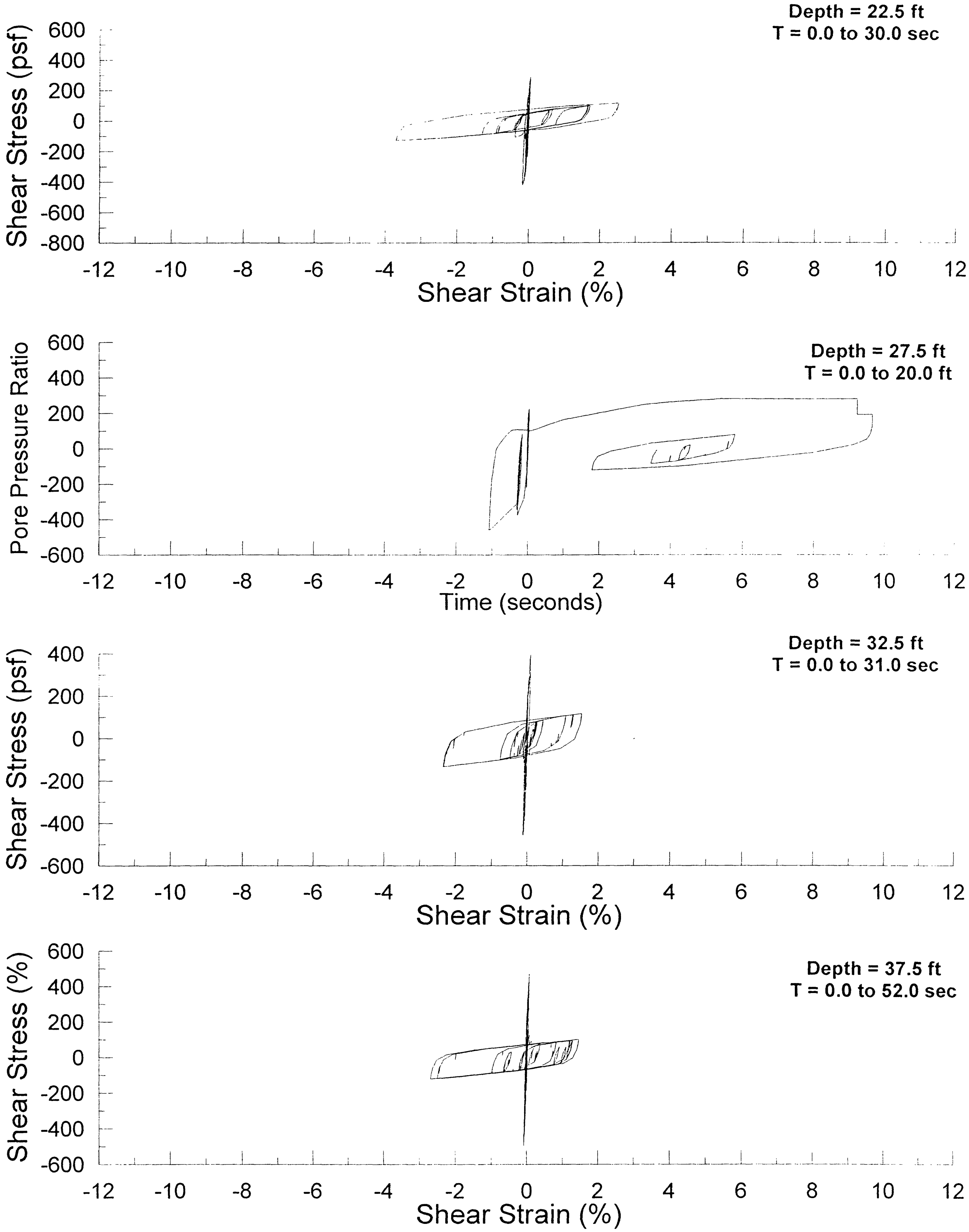


Figure 6a-4 Shear Stress - Shear Strain Loops
Case 6a: Soil Profile with Fill, 1968 Tokachi-Oki EQ, 475-yr ARP

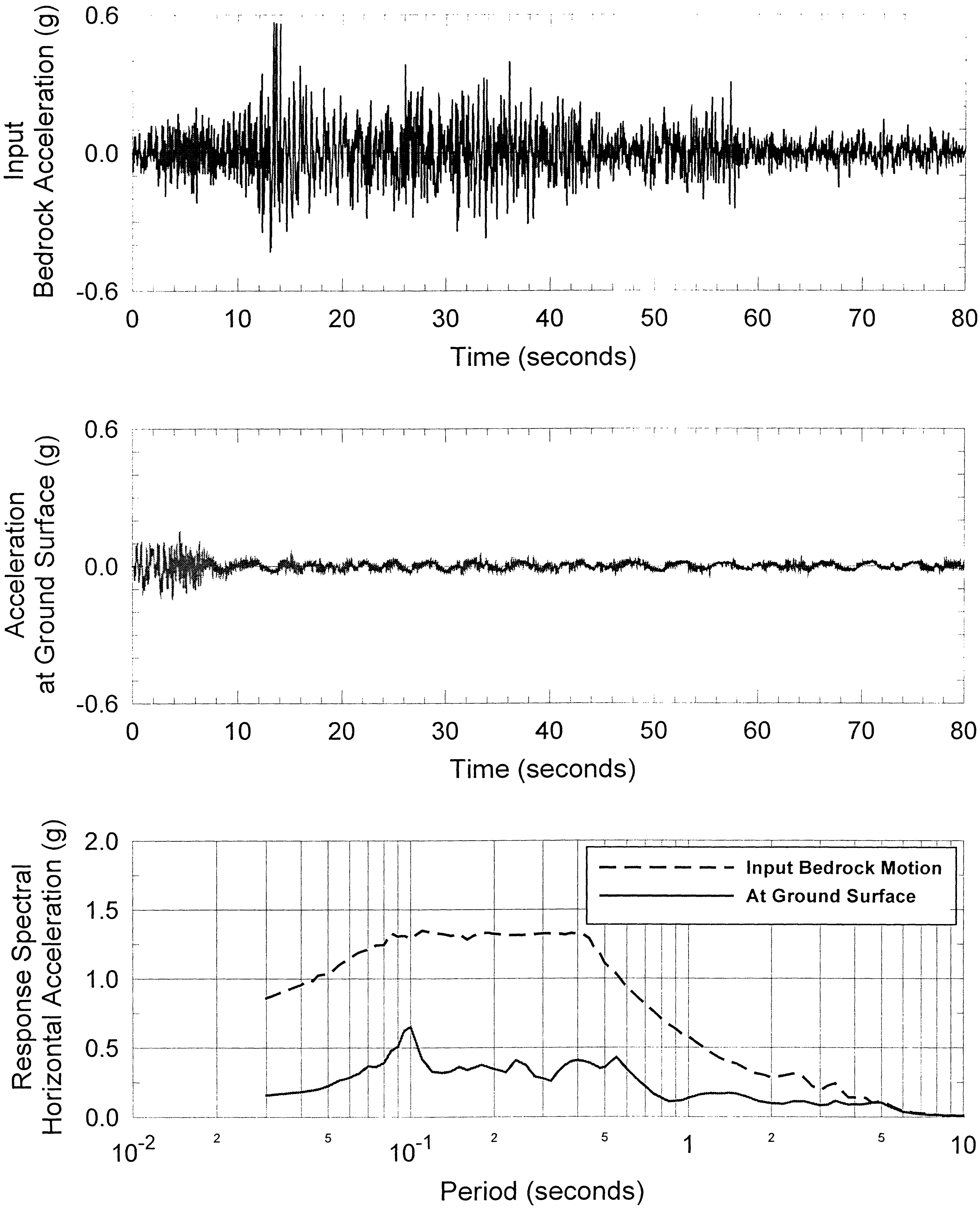


Figure 6b-1 Acceleration Time Histories and Response Spectra Case 6b: Soil Profile with Fill, 1968 Tokachi-Oki EQ, 2475-yr ARP

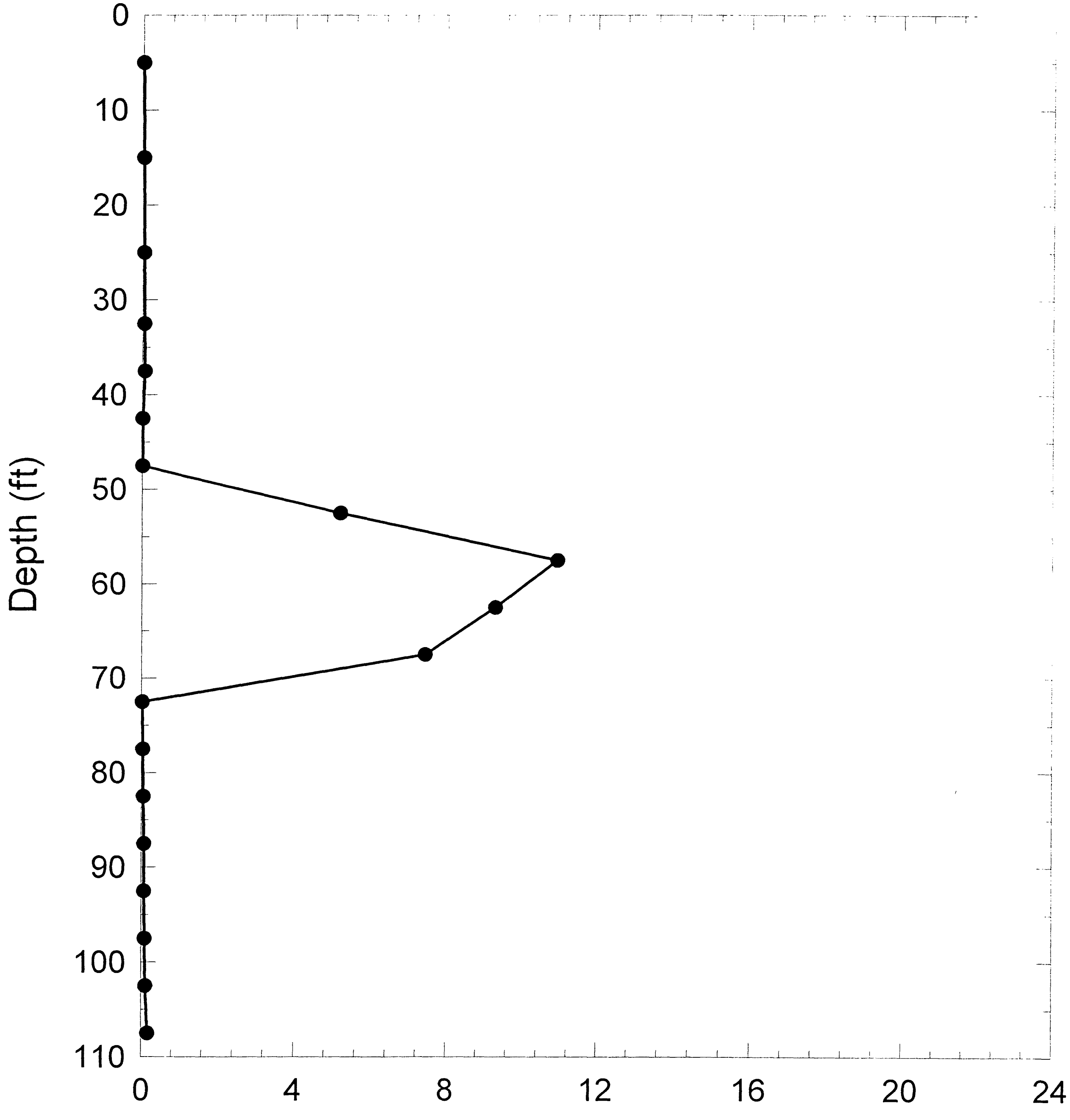


Figure 6b-2 Maximum Shear Strain Occurred During the Shaking Case 6b: Soil Profile with Fill 1968 Tokachi-Oki EQ, 2475-yr ARP

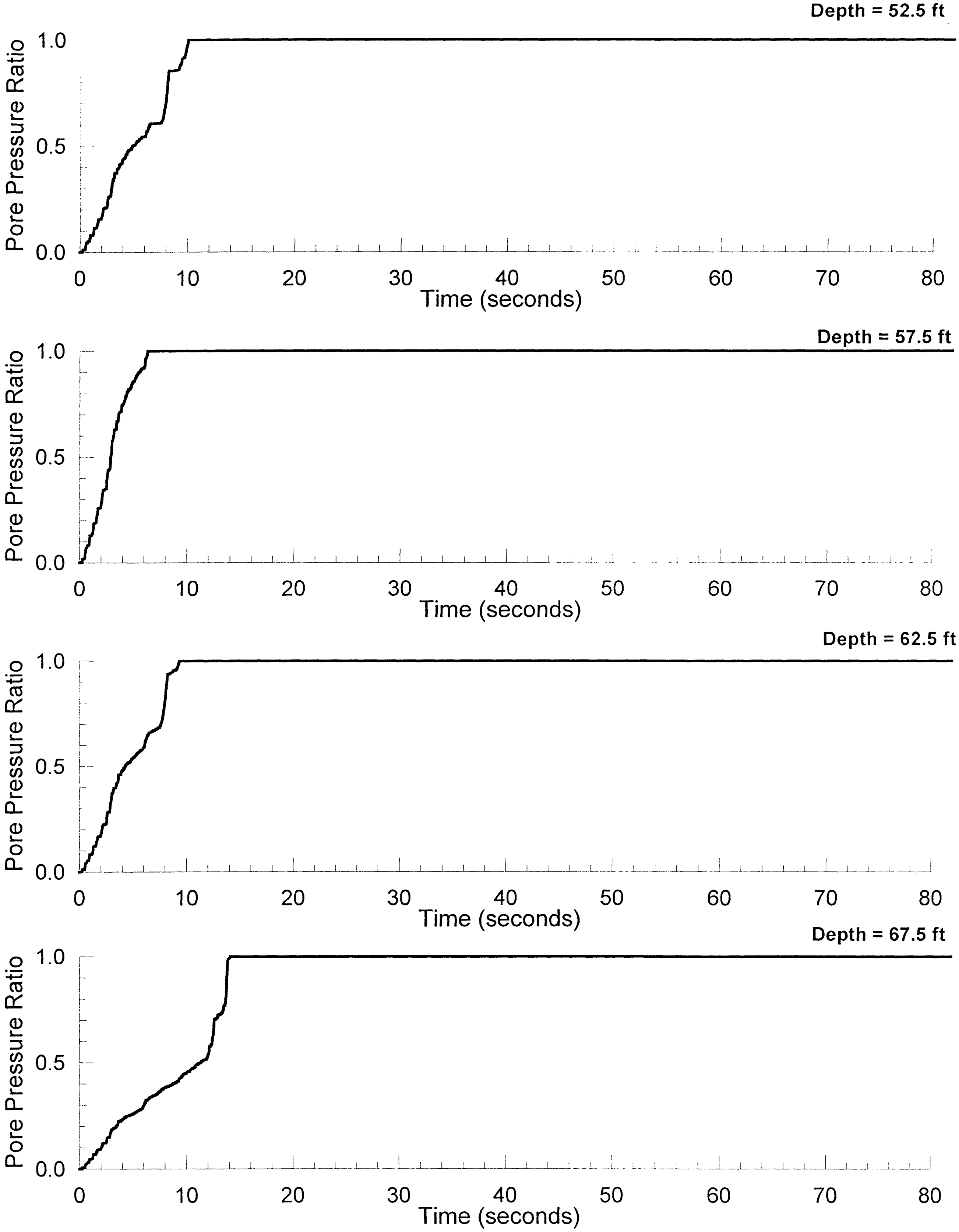


Figure 6b-3 Pore Pressure Generation
Case 6b: Soil Profile with Fill, 1968 Tokachi-Oki EQ, 2475-yr ARP

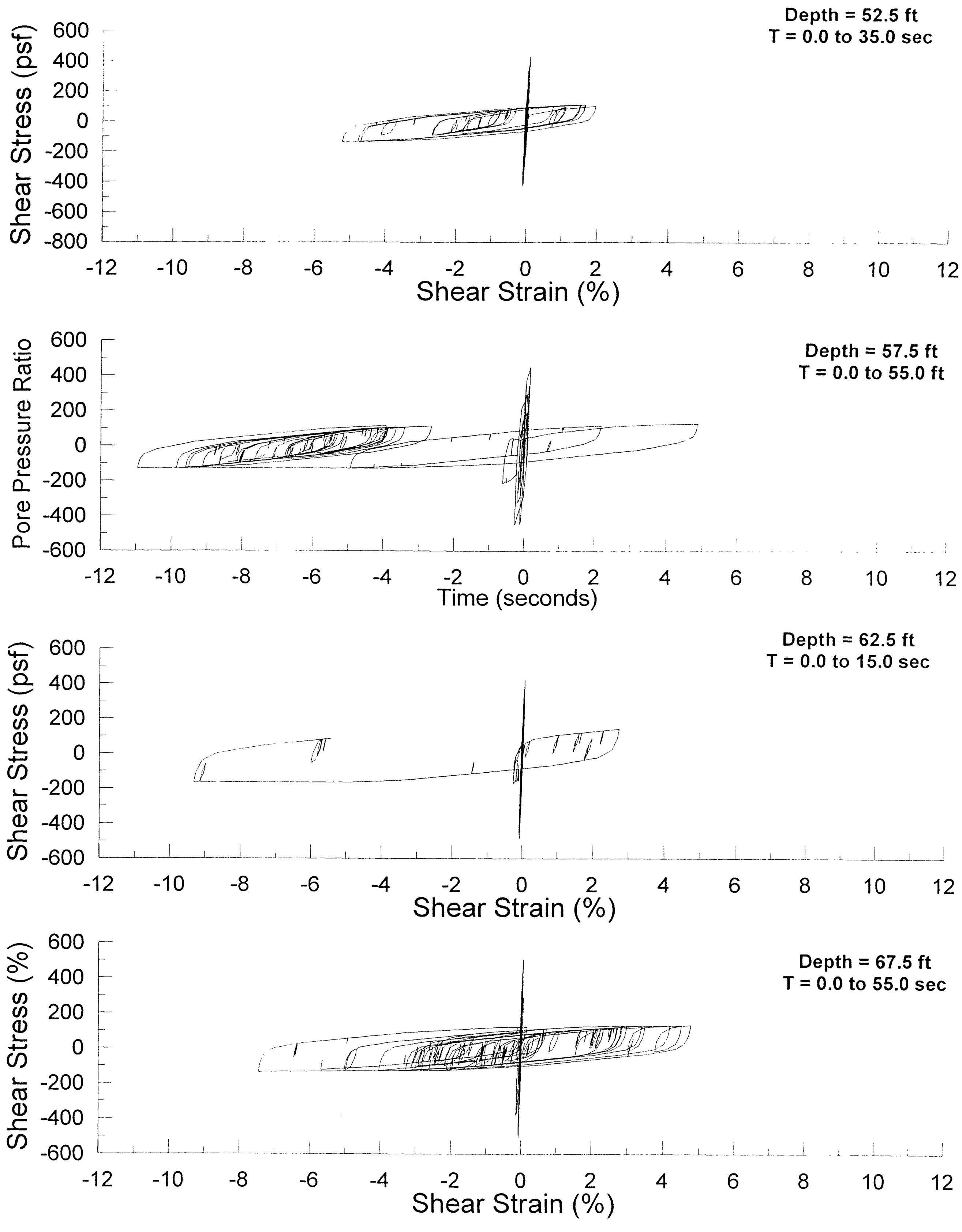


Figure 6b-4 Shear Stress - Shear Strain Loops
Case 6b: Soil Profile with Fill, 1968 Tokachi-Oki EQ, 2475-yr ARP

Case 1a:	Soil profile without fill, 1985 Michoacan EQ, 475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	8.47	3.54	8.46	0.002773	8.47	30.26	0	0
7.5	2	8.26	4.055	8.46	0.008544	8.46	92.24	0	0
12.5	3	0.21	22.97	9.27	0.002792	0.16	365.6	0	0
17.5	4	0.16	42.41	8.68	0.003899	9.3	383.8	0	0
22.5	5	0.16	23.43	21.14	1.281	8.69	198.9	14.75	2319
27.5	6	23.52	4.109	14.51	2.739	8.68	221.2	10.65	2557
32.5	7	10.37	5.59	31.64	1.561	14.45	246.3	14.87	2795
37.5	8	21.33	6.15	31.55	2.915	14.25	298.4	23.86	3033
42.5	9	0.1	8.542	14.24	0.01108	14.24	407.6	0	0
47.5	10	0.09	16.05	14.23	0.01519	14.24	518.5	0	0
52.5	11	0.14	23.1	20.68	0.01856	14.23	637.2	0	0
57.5	12	0.14	30.92	20.68	0.02247	14.23	774.3	0	0
62.5	13	0.09	44.15	20.68	0.02183	14.22	986.2	0	0
67.5	14	0.09	56.96	14.22	0.02536	14.22	1100	0	0
72.5	15	0.09	58.36	14.21	0.02821	14.22	1115	0	0
77.5	16	0.09	46.1	14.2	0.02862	14.2	1028	0	0
Base	17	26.38	16.89	0	0	0	0	0	0
maximum	base	acc.=	5.51	at26.29	sec				

Case 1b:	Soil profile without fill, 1985 Michoacan EQ, 2475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	2.12	5.184	2.12	0.004472	2.12	44.31	0	0
7.5	2	2.13	5.973	2.13	0.01576	2.13	139.1	0	0
12.5	3	0.16	23.34	2.12	0.005042	2.12	434.9	0	0
17.5	4	0.16	39.85	2.12	0.006728	2.12	491.7	0	0
22.5	5	0.16	23.44	25.14	2.273	2.05	315.7	7.13	2319
27.5	6	9.48	7.172	25.17	3.734	2.11	358.2	3.82	2557
32.5	7	10.35	11.09	25.1	3.894	2.08	444.6	7.17	2795
37.5	8	10.3	12.41	24.97	7.141	2.09	496.6	11.24	3033
42.5	9	20.7	17.85	2.08	0.019	2.09	581.2	0	0
47.5	10	18.74	20.9	24.8	0.0289	24.81	779.3	0	0
52.5	11	25.05	26.18	24.8	0.04557	24.8	1034	0	0
57.5	12	17.3	34.67	24.78	0.06451	24.79	1258	0	0
62.5	13	23.14	47.84	24.77	0.05963	24.78	1427	0	0
67.5	14	31.4	60.4	24.77	0.07103	24.77	1634	0	0
72.5	15	11.24	60.27	24.77	0.08121	24.78	1784	0	0
77.5	16	12.09	50.47	24.77	0.09245	24.77	1923	0	0
Base	17	26.43	23.38	0	0	0	0	0	0
maximum	base	acc.=	16.4	at26.29	sec				

Case 2a:	Soil profile with fill, 1985 Michoacan EQ, 475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	11.44	3.067	11.44	0.00569	11.44	52.43	0	0
15	2	9.94	2.841	9.94	0.01899	9.94	142.1	0	0
25	3	10.79	2.722	9.93	0.03468	9.93	227.4	0	0
32.5	4	9.46	3.039	11.48	0.03474	11.48	233.3	0	0
37.5	5	9.44	4.333	11.47	0.04239	11.47	274	0	0
42.5	6	0.22	23.22	11.47	0.007384	11.48	497.1	0	0
47.5	7	0.16	42.42	10.24	0.007772	10.26	515.6	0	0
52.5	8	0.16	23.62	25.16	4.453	10.26	316.5	19.27	5619
57.5	9	10.38	4.373	25.48	9.5	10.26	314.6	14.76	5857
62.5	10	17.3	6.06	25.42	3.515	10.21	336.5	18.26	6095
67.5	11	26.36	6.33	46.32	2.381	10.19	372.7	28.1	6333
72.5	12	0.15	7.972	10.19	0.01281	10.2	445	0	0
77.5	13	0.14	15.41	14.23	0.01515	14.24	533.3	0	0
82.5	14	0.14	21.39	14.2	0.0187	14.23	650.9	0	0
87.5	15	0.14	28.78	14.2	0.0227	14.22	789.9	0	0
92.5	16	0.09	42.55	14.21	0.02268	14.22	999.1	0	0
97.5	17	0.09	55.51	14.22	0.02556	14.22	1113	0	0
102.5	18	0.09	56.44	14.21	0.03091	14.22	1127	0	0
107.5	19	0.09	43.46	14.2	0.02816	14.21	1051	0	0
Base	20	14.83	15.39	0	0	0	0	0	0
maximum	base	acc.=	5.51	at26.29	sec				

Case 2b:	Soil profile with fill, 1985 Michoacan EQ, 2475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	2.1	4.676	2.09	0.009166	2.1	79.93	0	0
15	2	2.08	3.677	2.09	0.03062	2.09	202.6	0	0
25	3	2.13	3.915	2.12	0.049	2.12	274.8	0	0
32.5	4	2.15	4.095	2.14	0.05904	2.14	329.7	0	0
37.5	5	2.16	5.754	2.15	0.07961	2.15	389.9	0	0
42.5	6	0.16	22.62	2.15	0.01216	2.17	628.8	0	0
47.5	7	0.16	39.91	2.17	0.01365	2.17	673.4	0	0
52.5	8	0.16	23.01	25.39	5.074	2.18	512.4	10.88	5619
57.5	9	27.33	4.931	25.48	6.459	2.16	544.1	6.98	5857
62.5	10	8.72	10.98	10.63	4.221	2.14	539.3	9.36	6095
67.5	11	11.79	15.64	25.48	22.34	2.1	583.8	13.43	6333
72.5	12	23.1	17.91	2.1	0.02122	2.1	644.7	0	0
77.5	13	13.96	22.07	24.8	0.02396	2.1	698.7	0	0
82.5	14	14.84	27.52	24.8	0.03559	24.8	916.3	0	0
87.5	15	26.71	33.86	24.78	0.05495	24.78	1169	0	0
92.5	16	26.72	48.34	24.78	0.05249	24.79	1365	0	0
97.5	17	32.8	57.5	24.78	0.06378	24.78	1557	0	0
102.5	18	11.14	59.18	24.77	0.07971	24.78	1748	0	0
107.5	19	33.44	46.7	24.77	0.08653	24.77	1873	0	0
Base	20	24.27	22.05	0	0	0	0	0	0
maximum	base	acc.=	16.4	at26.29	sec				

Case 3a:	Soil profile without fill, 1985 Chile EQ, 475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	16.66	3.129	16.65	0.002447	16.66	26.75	0	0
7.5	2	16.65	4.087	16.65	0.008608	16.65	92.15	0	0
12.5	3	0.16	22.99	16.65	0.00302	0.17	361.3	0	0
17.5	4	0.16	42.8	16.65	0.004219	16.67	398	0	0
22.5	5	0.16	23.57	23.76	1.52	16.68	211.5	21.69	2319
27.5	6	26.97	5.417	22.67	1.913	16.68	238.3	17.25	2557
32.5	7	28.99	6.047	23.85	1.176	16.63	267.7	23.18	2795
37.5	8	27.74	7.479	38.81	1.139	16.64	296.5	35.21	3033
42.5	9	0.15	8.48	16.6	0.009588	16.61	377.6	0	0
47.5	10	0.15	16.98	27.75	0.0132	27.02	461.6	0	0
52.5	11	0.14	23.9	27.75	0.01745	27.02	585.8	0	0
57.5	12	0.14	31.64	27.74	0.02126	27.76	733.3	0	0
62.5	13	0.09	44.31	27.74	0.02101	29.01	943.1	0	0
67.5	14	0.09	57.71	29	0.02373	29.01	1079	0	0
72.5	15	0.09	60.01	29	0.02844	29	1119	0	0
77.5	16	0.09	48.13	28.98	0.031	29	1058	0	0
Base	17	0.1	17.22	0	0	0	0	0	0
maximum	base	acc.=	6.44	at28.90	sec				

Case 3b:	Soil profile without fill, 1985 Chile EQ, 2475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	6.31	3.779	6.3	0.003029	6.31	32.3	0	0
7.5	2	6.32	4.595	6.31	0.01052	6.32	105.5	0	0
12.5	3	0.17	23.51	6.32	0.003633	0.41	386.5	0	0
17.5	4	0.17	41.19	6.32	0.005094	0.42	426.6	0	0
22.5	5	0.17	23.99	25.88	1.588	6.32	249	12.72	2319
27.5	6	27.03	6.553	27.31	2.165	6.31	280.3	8.57	2557
32.5	7	17.8	11.1	24.36	3.05	6.29	337.1	13.86	2795
37.5	8	28.99	11.55	24.3	6.113	6.28	380.9	20.16	3033
42.5	9	28.99	20.84	27.03	0.01654	27.02	507.2	0	0
47.5	10	29.04	23.07	28.99	0.03107	28.99	834.4	0	0
52.5	11	27.57	25.3	28.99	0.05506	28.99	1145	0	0
57.5	12	27.57	33.54	29	0.07719	29	1420	0	0
62.5	13	28.03	48.88	29.01	0.07826	29.01	1713	0	0
67.5	14	26.11	57.99	29	0.09236	29.01	1962	0	0
72.5	15	27.67	59.3	29	0.1315	29	2211	0	0
77.5	16	24.64	49.98	28.99	0.168	28.99	2406	0	0
Base	17	27.67	26.06	0	0	0	0	0	0
maximum	base	acc.=	19	at27.67	sec				

Case 4a:	Soil profile with fill, 1985 Chile EQ, 475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	16.75	3.17	16.74	0.005904	16.75	54.2	0	0
15	2	16.78	2.951	16.77	0.01675	16.77	130.2	0	0
25	3	16.81	2.67	16.8	0.02959	16.8	200.4	0	0
32.5	4	16.68	2.498	16.81	0.03944	16.81	253.6	0	0
37.5	5	16.67	3.523	16.81	0.04807	16.81	300.1	0	0
42.5	6	0.16	23.08	16.8	0.008796	16.8	550.1	0	0
47.5	7	0.16	42.78	16.79	0.009688	16.8	576.2	0	0
52.5	8	0.16	23.74	25.01	2.997	16.78	383	23.57	5619
57.5	9	23.81	3.575	23.17	7.149	16.77	394.1	19.4	5857
62.5	10	22.95	5.081	24.86	2.757	16.73	410.7	23.53	6095
67.5	11	29	7.426	42.93	1.56	16.72	433.6	40.67	6333
72.5	12	29.01	9.395	16.7	0.01445	16.72	485	0	0
77.5	13	0.15	15.98	16.7	0.01516	16.72	532.9	0	0
82.5	14	0.15	22.19	28.99	0.02043	29	662.5	0	0
87.5	15	0.15	29.66	28.98	0.02618	29	826.5	0	0
92.5	16	0.09	43.44	28.98	0.02648	29	1037	0	0
97.5	17	0.09	56.7	28.97	0.03	29.02	1171	0	0
102.5	18	0.09	57.5	29	0.03748	29.01	1211	0	0
107.5	19	0.09	44.62	28.99	0.03781	29	1177	0	0
Base	20	0.1	15.73	0	0	0	0	0	0
maximum	base	acc.=	6.44	at28.90	sec				

Case 4b:	Soil profile with fill, 1985 Chile EQ, 2475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	6.39	3.109	6.39	0.005802	6.39	53.15	0	0
15	2	7.26	2.356	7.27	0.01585	7.27	125.8	0	0
25	3	2.6	2.567	2.6	0.02738	2.61	189.1	0	0
32.5	4	6.33	2.723	2.63	0.03393	2.63	229.4	0	0
37.5	5	6.07	3.62	2.63	0.04307	2.63	277.2	0	0
42.5	6	0.17	22.61	6.34	0.007984	6.35	528.5	0	0
47.5	7	0.22	40.92	6.36	0.008949	6.34	549.1	0	0
52.5	8	0.17	23.25	21.84	3.576	6.37	354.9	16.76	5619
57.5	9	17.84	4.696	21.41	7.747	6.36	358.9	11.34	5857
62.5	10	15.39	7.815	21.32	4.331	6.33	334.9	16.48	6095
67.5	11	17.74	10.58	27.74	10.38	9.84	331.8	22.52	6333
72.5	12	28.99	23.82	28.99	0.01543	28.99	478.1	0	0
77.5	13	29	20.98	28.99	0.03185	28.99	843	0	0
82.5	14	22.96	24.97	28.99	0.05291	28.99	1121	0	0
87.5	15	22.97	33.01	29	0.07518	29	1427	0	0
92.5	16	22.97	46.61	28.99	0.07499	29	1706	0	0
97.5	17	28.65	57.81	29	0.09053	29	1964	0	0
102.5	18	30.18	59.78	29	0.1158	29	2206	0	0
107.5	19	27.67	50.4	28.99	0.1694	28.99	2445	0	0
Base	20	27.67	26.34	0	0	0	0	0	0
maximum	base	acc.=	19	at27.67	sec				

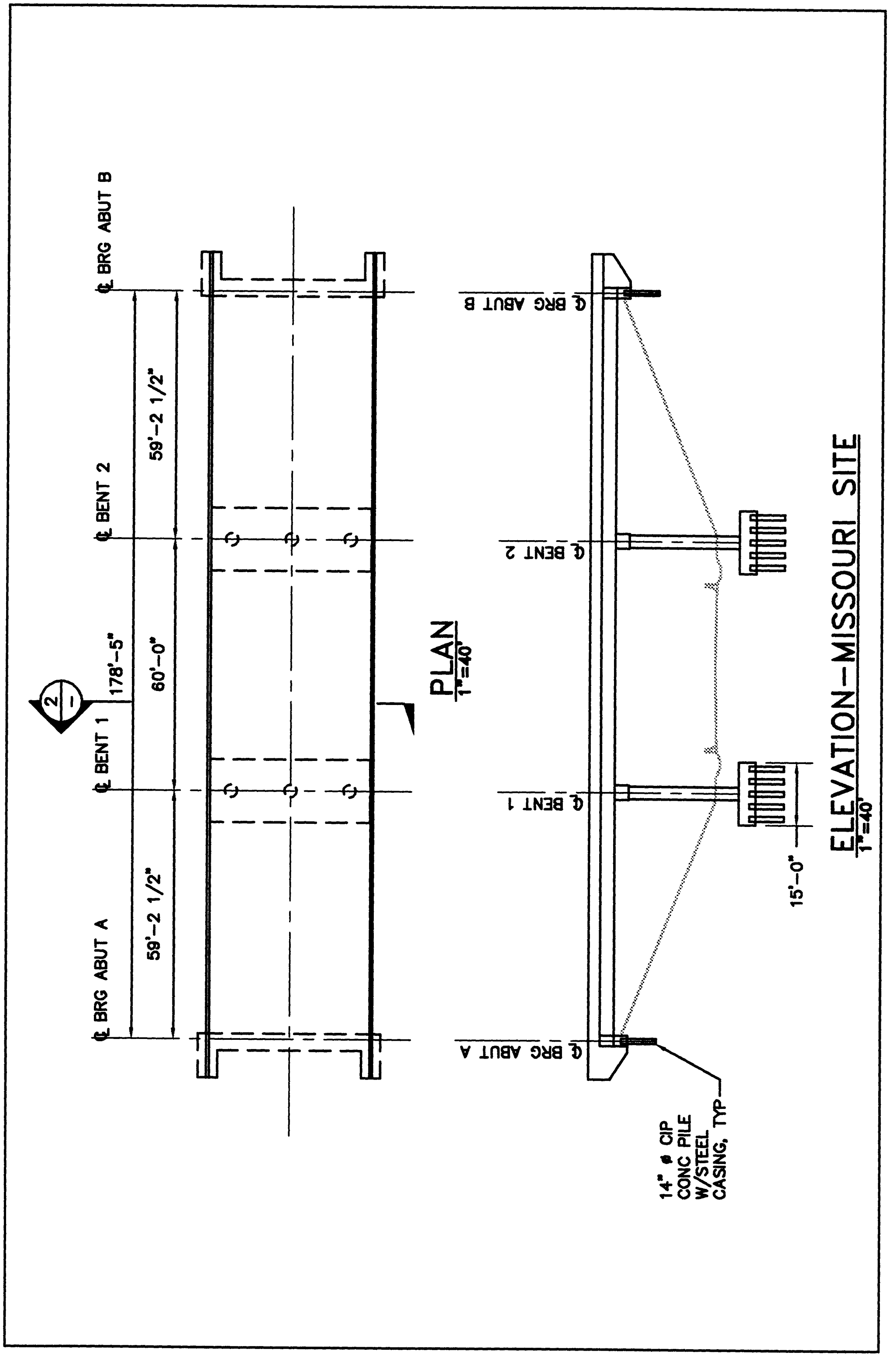
Case 5a:	Soil profile without fill, 1968 Tokachi-Oki EQ, 475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	13.26	3.053	14.27	0.002443	13.26	26.09	0	0
7.5	2	14.28	4.079	14.19	0.0082	14.19	88.45	0	0
12.5	3	0.17	23.67	14.19	0.002965	0.17	374.5	0	0
17.5	4	0.17	42.45	14.19	0.00421	14.22	385.4	0	0
22.5	5	0.17	24.31	26.07	1.055	14.22	209.1	14.46	2319
27.5	6	14.23	3.41	14.2	5.757	14.2	244.3	12.83	2557
32.5	7	13.64	8.152	26	1.968	14.08	336.1	15.15	2795
37.5	8	13.65	7.38	29.44	1.41	14.06	403.8	26.72	3033
42.5	9	13.43	9.092	14.06	0.01418	14.06	480.7	0	0
47.5	10	0.16	16.53	14.04	0.01761	13.69	562.4	0	0
52.5	11	0.16	24.7	13.66	0.02199	13.7	697.6	0	0
57.5	12	0.16	32.87	13.67	0.02752	13.68	849.8	0	0
62.5	13	0.16	45.29	13.67	0.02882	13.71	1086	0	0
67.5	14	0.1	56.59	13.68	0.03398	13.72	1226	0	0
72.5	15	0.1	58.51	13.68	0.03876	13.71	1262	0	0
77.5	16	0.1	47.62	13.68	0.04111	13.69	1205	0	0
Base	17	0.1	16.98	0	0	0	0	0	0
maximum	base	acc.=	6.8	at13.34	sec				

Case 5b:	Soil profile without fill, 1968 Tokachi-Oki EQ, 2475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
2.5	1	2.71	4.958	2.71	0.004103	2.71	42.38	0	0
7.5	2	2.65	5.492	2.71	0.01393	2.72	128.7	0	0
12.5	3	0.19	23.74	2.71	0.004637	2.71	418.1	0	0
17.5	4	0.19	40.29	2.71	0.006381	2.71	475.6	0	0
22.5	5	0.19	23.96	14.41	2.786	2.71	294.5	8.76	2319
27.5	6	12.46	7.566	26.42	3.746	2.7	339.4	4.77	2557
32.5	7	13.48	10.7	13.41	3.921	2.68	418	8.18	2795
37.5	8	13.42	15.58	14.67	6.899	2.66	452.1	13.48	3033
42.5	9	13.43	20.89	13.22	0.01895	13.22	561.5	0	0
47.5	10	13.69	22.94	13.22	0.03102	13.22	828.4	0	0
52.5	11	13.72	26.39	13.21	0.04806	13.69	1054	0	0
57.5	12	13.72	36.85	13.68	0.06721	13.68	1292	0	0
62.5	13	13.72	53.02	13.68	0.06811	13.68	1560	0	0
67.5	14	13.56	61.48	13.67	0.08028	13.68	1777	0	0
72.5	15	13.72	60.29	13.66	0.09625	13.67	2009	0	0
77.5	16	31.29	47.8	13.66	0.1463	13.66	2289	0	0
Base	17	36.16	21.26	0	0	0	0	0	0
maximum	base	acc.=	18.3	at13.35	sec				

Case 6a:	Soil profile with fill, 1968 Tokachi-Oki EQ, 475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth ft	LAYER NO.	TIME Max. Acc Occurred	ABS. Acc (ft/s/s)	TIME Max. Strain Occurred	STRAIN (%)	TIME Max. Stress Occurred	STRESS (psf)	TIME Max. Pore Pressure Built	PORE Pressure (psf)
5	1	13.24	3.721	13.24	0.007137	13.24	63.61	0	0
15	2	13.27	3.413	13.26	0.0229	13.27	162.1	0	0
25	3	13.57	3.294	13.28	0.0356	13.28	231.8	0	0
32.5	4	13.38	3.2	13.42	0.04328	13.42	271	0	0
37.5	5	13.39	4.316	13.4	0.0529	13.42	316.4	0	0
42.5	6	0.17	22.91	13.4	0.009691	13.41	567.7	0	0
47.5	7	0.17	42.07	13.39	0.0108	13.4	599	0	0
52.5	8	0.17	23.56	26.53	3.695	13.39	420.6	16.38	5619
57.5	9	13.4	5.294	14.68	9.653	13.38	458.8	14.7	5857
62.5	10	13.65	8.756	30.3	2.343	13.36	457.6	15.91	6095
67.5	11	13.63	7.518	52.6	3.039	13.25	497.3	31.89	6333
72.5	12	13.72	8.223	13.25	0.01915	13.26	588.8	0	0
77.5	13	0.11	14.87	13.24	0.02183	13.25	679.6	0	0
82.5	14	0.16	21.35	13.24	0.02446	13.24	774.8	0	0
87.5	15	0.1	28.82	13.22	0.02755	13.24	886	0	0
92.5	16	0.1	42.87	13.67	0.02623	13.26	1066	0	0
97.5	17	0.1	54.53	13.67	0.02829	13.25	1150	0	0
102.5	18	0.1	55.4	13.67	0.0338	13.72	1145	0	0
107.5	19	0.1	43.78	13.67	0.03744	13.67	1125	0	0
Base	20	0.1	15.46	0	0	0	0	0	0
maximum	base	acc.=	6.8	at13.34	sec				

Case 6b:	Soil profile with fill, 1968 Tokachi-Oki EQ, 2475-yr ARP								
MAXIMUM	VALUES	DURING	0	TO	81.92	SEC			
Depth	LAYER	TIME	ABS. Acc	TIME	STRAIN	TIME	STRESS	TIME	PORE Pressure
ft	NO.	Max. Acc Occurred	(ft/s/s)	Max. Strain Occurred	(%)	Max. Stress Occurred	(psf)	Max. Pore Pressure Built	(psf)
5	1	4.53	4.959	4.52	0.009681	4.53	84.77	0	0
15	2	2.67	3.413	2.68	0.02454	2.68	170.7	0	0
25	3	3.86	4.008	2.69	0.0418	2.69	255.8	0	0
32.5	4	3.74	4.043	2.7	0.05145	2.71	304.4	0	0
37.5	5	2.8	4.641	2.72	0.06545	2.73	353	0	0
42.5	6	0.17	20.69	2.72	0.01002	1.16	581.9	0	0
47.5	7	0.17	38.04	1.97	0.01087	1.97	622.3	0	0
52.5	8	0.17	21.62	30.26	5.213	1.97	433.5	10.05	5619
57.5	9	3.78	4.854	50.95	10.96	1.95	450.7	6.36	5857
62.5	10	6.18	9.987	13.6	9.316	1.12	485.5	9.32	6095
67.5	11	13.45	21.34	50.88	7.455	1.12	514.2	14.05	6333
72.5	12	13.42	21.1	13.21	0.02237	13.21	638.2	0	0
77.5	13	13.43	21.98	13.22	0.03686	13.22	902	0	0
82.5	14	13.72	28.04	13.43	0.05295	13.43	1129	0	0
87.5	15	13.73	33.69	13.42	0.07207	13.43	1390	0	0
92.5	16	38.47	47.54	13.68	0.07174	13.72	1676	0	0
97.5	17	27.8	57.75	13.68	0.08802	13.69	1934	0	0
102.5	18	38.47	58.05	13.68	0.1082	13.68	2164	0	0
107.5	19	16.53	44.87	13.67	0.1653	13.67	2423	0	0
Base	20	26.58	20.38	0	0	0	0	0	0
maximum	base	acc.=	18.3	at13.35	sec				

Appendix I
STRUCTURAL DATA FOR CENTRAL U.S. BRIDGE


℄ BRG ABUT A
℄ BENT 1
℄ BENT 2
℄ BRG ABUT B
178'-5"
59'-2 1/2"
60'-0"
59'-2 1/2"
2
1
PLAN
1"=40'
℄ BRG ABUT A
℄ BENT 1
℄ BENT 2
℄ BRG ABUT B
15'-0"
14" ⌀ CIP CONC PILE W/STEEL CASING, TYP
ELEVATION-MISSOURI SITE
1"=40'

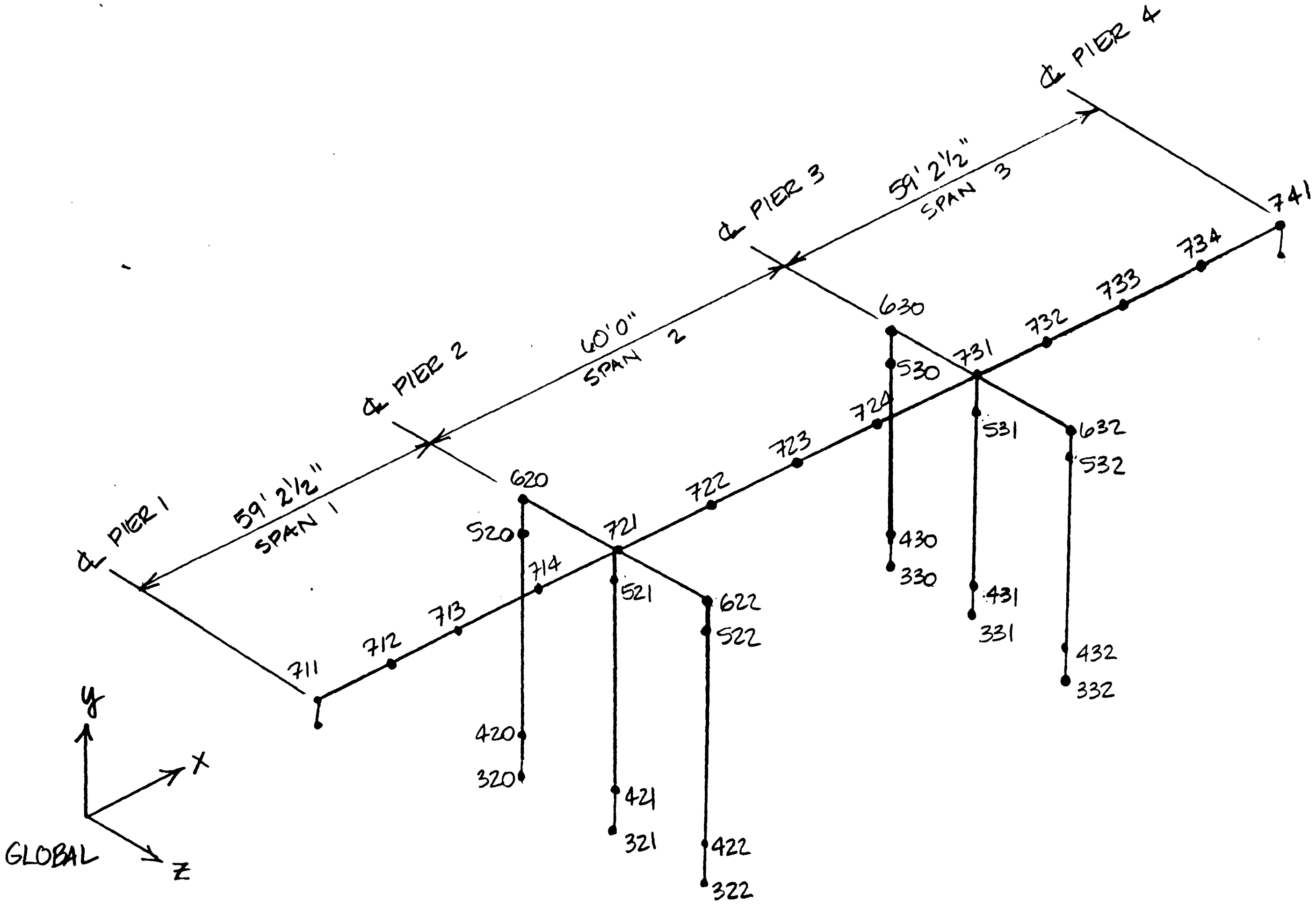
I.1 SAP2000 Model Development: Models and Framing

NCHRP 12-49 LIQUEFACTION STUDY
Analysis & Design Cases

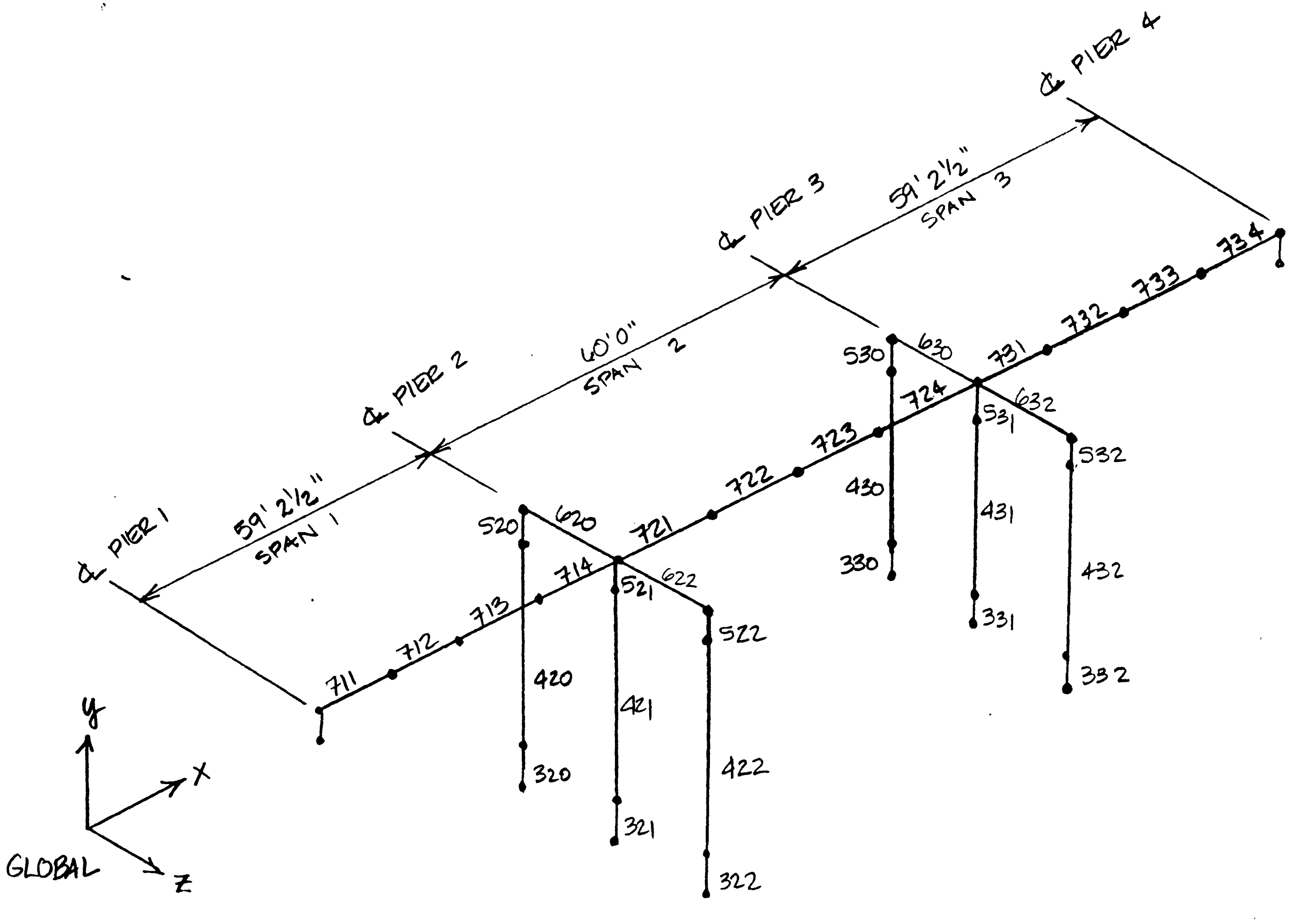
Earthquake Recurrence	Soil Profile	Foundation Restraints & Springs	Washington Bridge	Missouri Bridge
100-Year	Non-Liquefied	Springs at all piers	WA100N	MO100N
	Liquefied	Springs at all piers	---	---
500-Year	Non-Liquefied	Springs at all piers	WA500N	MO500N
	Liquefied	Springs at all piers	WA500L (Check Case)	MO500L (Check Case)
2500-Year	Non-Liquefied	Interior piers fixed, longitudinal spring & transverse fixed at abutment	WA2500NF (Check Case)	MO2500NF (Check Case)
		Springs at all piers	WA2500N	MO2500N
	Liquefied	Springs at all piers	WA2500L	MO2500L

NCHRP 12-49 LIQUEFACTION STUDY
Spectrum Development

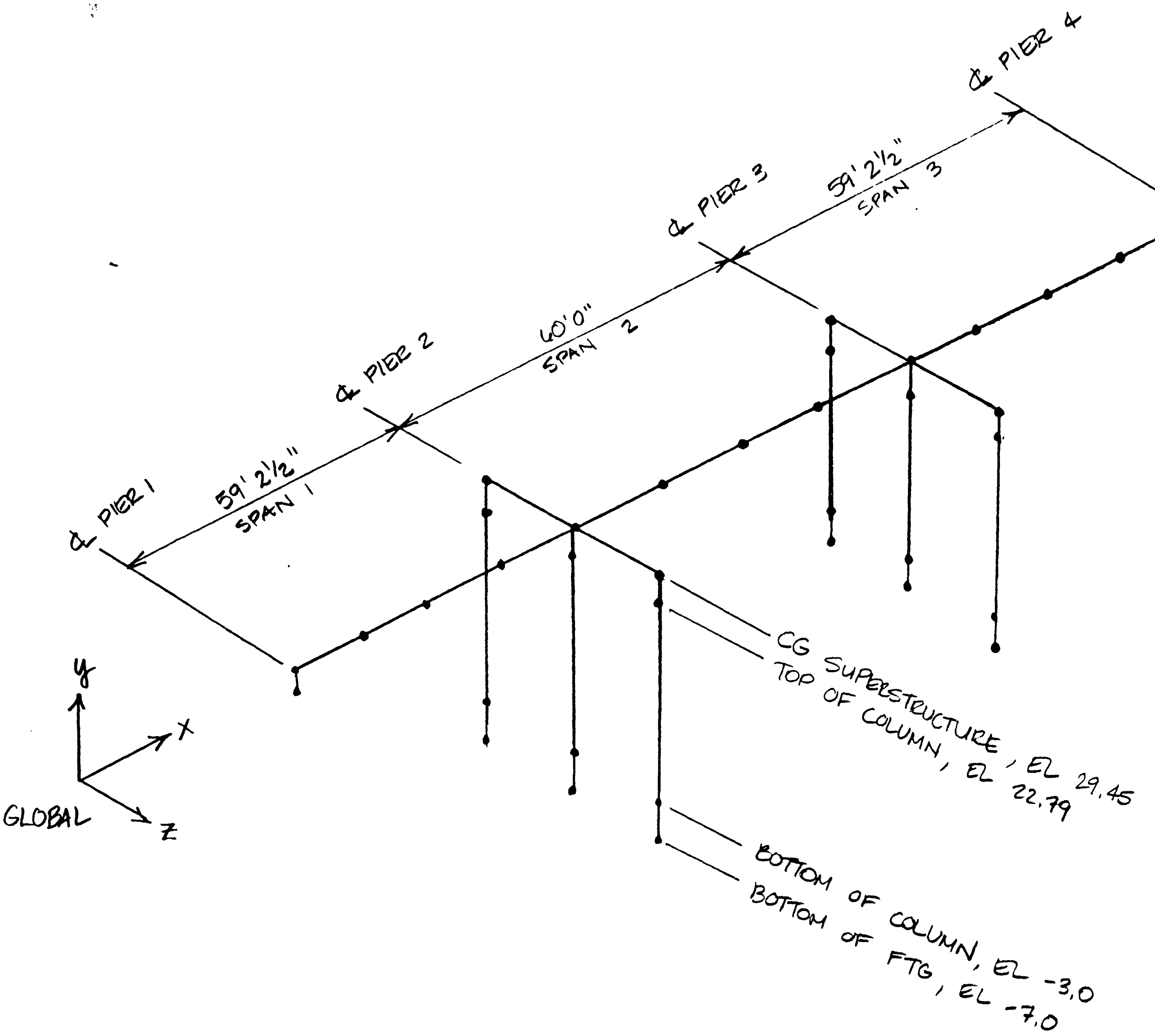
Earthquake Recurrence	Seismic Code	Ground Motion Parameters	Washington Site		Missouri Site	
			Liquefied Case	Non-Liquefied Case	Liquefied Case	Non-Liquefied Case
100-YR	NCHRP 12-49	F_a, S_s, F_v, S_1	- - -	Site Class E (F_a=2.5, S_s=0.261, F_v=3.5, S_1=0.081)	- - -	Site Class D (F_a=1.0, S_s=1.331, F_v=1.4, S_1=0.411)
500-YR	AASHTO Division 1A	A, S	- - -	A = 0.25g Type III (S=1.5)	- - -	A = 0.15g Type III (S=1.5)
2500-YR	NCHRP 12-49	F_a, S_s, F_v, S_1	2/3*(Site Class E) or Site Specific	Site Class E (F_a=0.9, S_s=1.250, F_v=2.4, S_1=0.437)	2/3*(Site Class D) or Site Specific	Site Class D (F_a=1.60, S_s=0.091, F_v=2.40, S_1=0.0185)



JOINT NUMBERS



MEMBER NUMBERS


℄ PIER 1
℄ PIER 2
℄ PIER 3
℄ PIER 4
59' 2½"
SPAN 1
60' 0"
SPAN 2
59' 2½"
SPAN 3
CG SUPERSTRUCTURE, EL 29.45
TOP OF COLUMN, EL 22.79
BOTTOM OF COLUMN, EL -3.0
BOTTOM OF FTG, EL -7.0
y
x
z
GLOBAL

DEVELOP MODEL COORDINATES

Superstructure Spine

	Spans 1 & 3	Span 2
L =	59.208	60.000
L/4 =	14.802	15.000

Joint Number	Joint Location	Member Length	X Coord
		ft	ft
711	CL Pier 1	14.802	0
712		14.802	14.802
713		14.802	29.604
714		14.802	44.406
721	CL Pier 2	15.000	59.208
722		15.000	74.208
723		15.000	89.208
724		15.000	104.208
731	CL Pier 3	14.802	119.208
732		14.802	134.010
733		14.802	148.813
734		14.802	163.615
741	CL Pier 4		178.417
		178.417	

PGL Elevation = 30.000 ft

Superstructure Depth = 3.958 ft 1" min. fillet negle

CG of Super from Girder Soffit = 3.412 ft 1" min. fillet negle

For entire spine of model:

Y Coord = 29.454 ft

Z Coord = 0.000 ft

29.454' - 3.412' = 26.042'

Piers 2 & 3

Crossbeam Depth from Girder Soffit = 3.250 ft

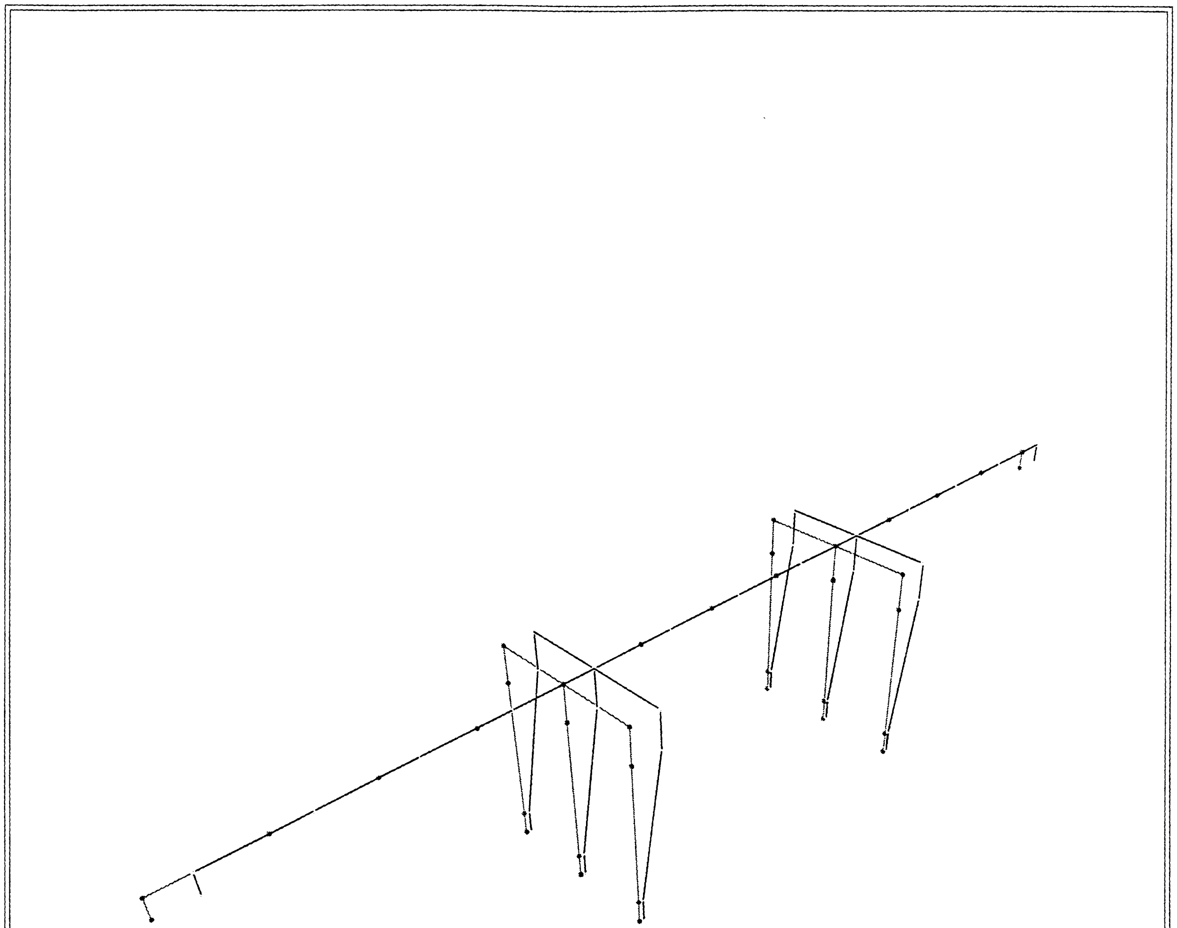
Footing Depth = 4.000 ft

Bottom of Footing Elevation = -7.000 ft

Joint Number	Joint Location	Y Coord
		ft
7xx	CG of Super	29.454
6xx	Top of Column	22.792
5xx	Bottom of Column	-3.000
4xx	Bottom of Footing	-7.000

Column Length = 25.792 ft

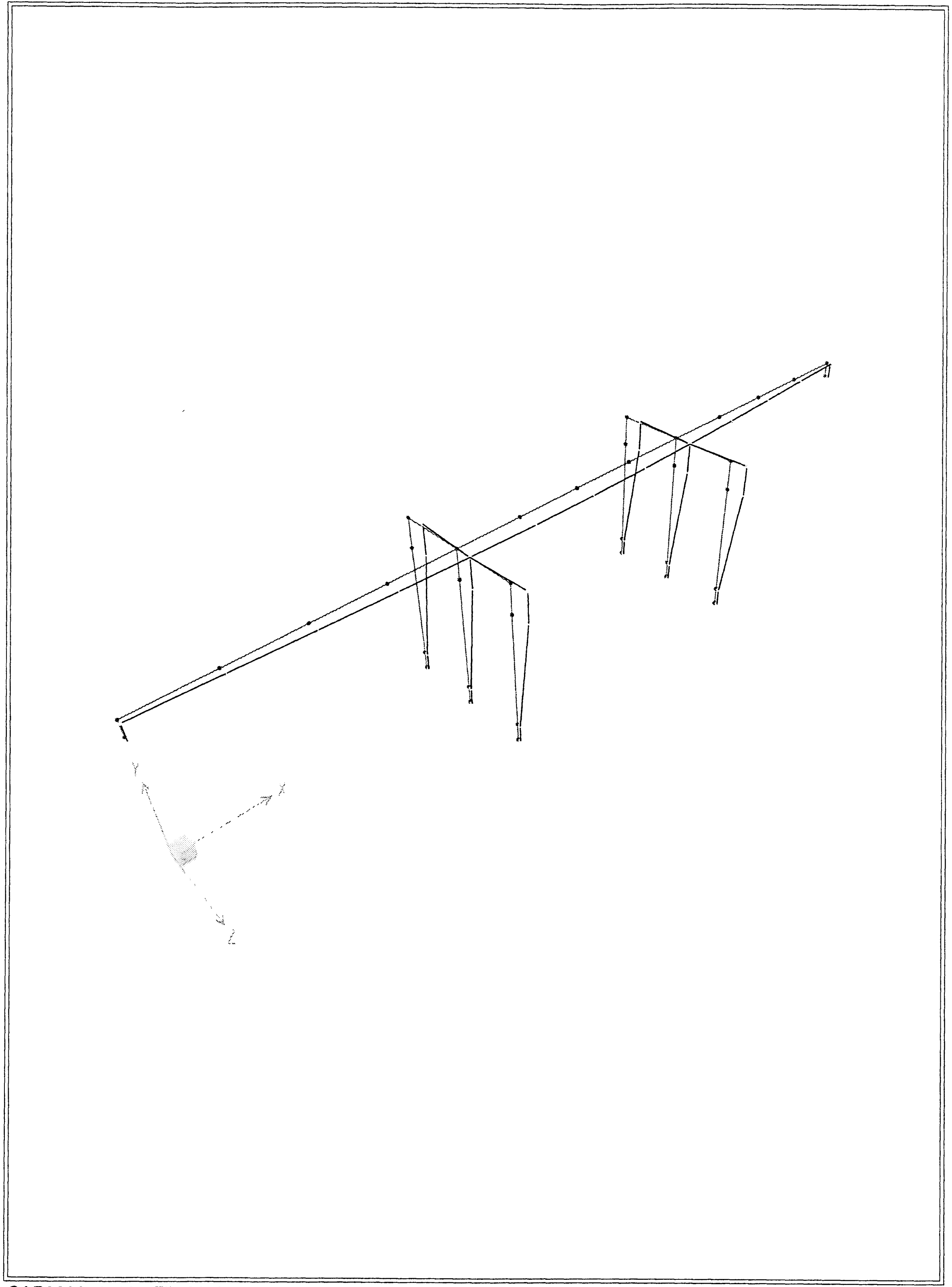
SAP2000 10/17/00 18:13:24



SAP2000 v7.10 - File:Mo2500n - Deformed Shape (EQLONG) - Kip-ft Units

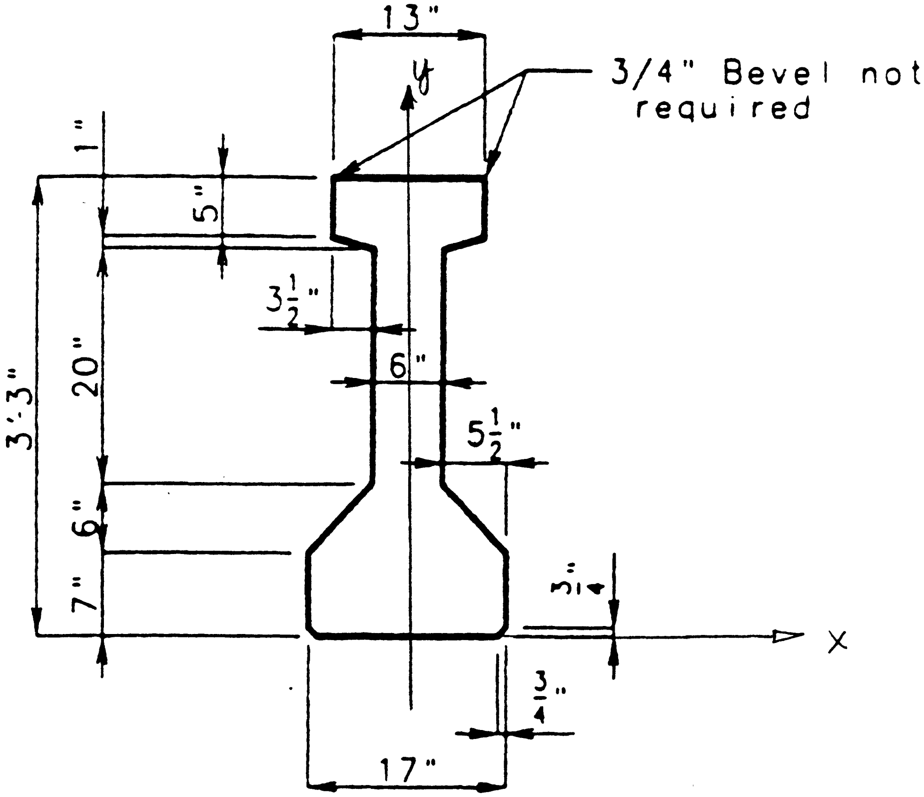
SAP2000

10/17/00 18:13:30



SAP2000 v7.10 - File:Mo2500n - Deformed Shape (EQTRAN) - Kip-ft Units

I.2 SAP2000 Model Development: Section Properties and Loads



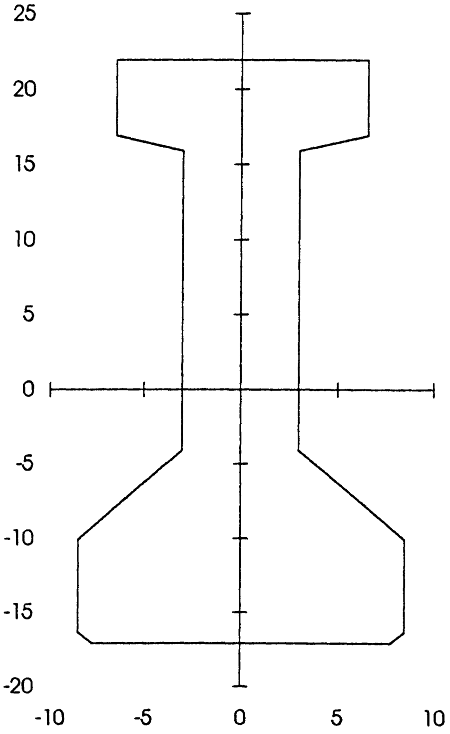
GIRDER DIMENSIONS

SECTION PROPERTIES WORKSHEET

PROJECT:	NCHRP MISSOURI BRIDGE
SUBJECT:	**BARE AASHTO GIRDER**
DESIGNER:	MLT
DATE:	2-Aug-00

SECTION COORDINATES (CLOCKWISE):

X	Y
0.0000	0.0000
-7.7500	0.0000
-8.5000	0.7500
-8.5000	7.0000
-3.0000	13.0000
-3.0000	33.0000
-6.5000	34.0000
-6.5000	39.0000
0.0000	39.0000
0.0000	39.0000
6.5000	39.0000
6.5000	34.0000
3.0000	33.0000
3.0000	13.0000
8.5000	7.0000
8.5000	0.7500
7.7500	0.0000
0.0000	0.0000
END	



HOLE COORDINATES (COUNTERCLOCKWISE):

X	Y

CIRCULAR HOLES:

Xo	Yo	D

RECTANGULAR HOLES:

Xo	Yo	B	H

SECTION PROPERTIES:

L = inches

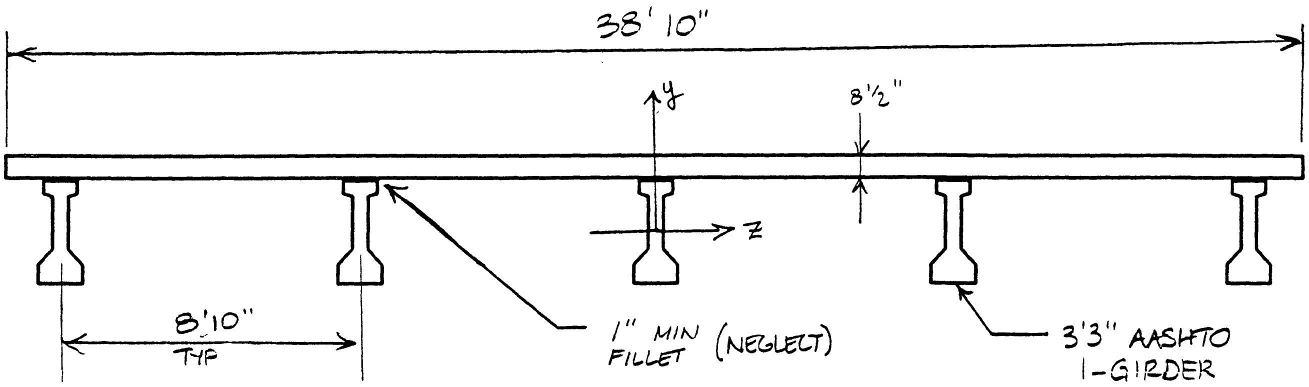
Area:	381.9375	L^2
Xbar:	0.0000	L
Ybar:	17.0831	L
Ix:	61841.2758	L^4
Iy:	5118.5514	L^4
S top:	2821.6306	L^3
S bot:	3620.0176	L^3

$A = 2.652$ ft^2

$I_x = 2.982$ ft^4

$I_y = 0.247'$ ft^4

COMPUTE SUPERSTRUCTURE PROPS



GIRDER: $y_{bot} = 17.08"$ $I_z = 2.982\ ft^4$ $A = 2.652\ ft^2$ $I_y = 0.247\ ft^4$

SLAB · $y_{bot} = 4.25"$

$$I_z = \frac{1}{12}(38.833')\left[\frac{8.5}{12}\right]^3 = 1.150\ ft^4$$

$$A = (38.833')\left(\frac{8.5}{12}\right) = 27.507\ ft^2$$

$$I_y = \frac{1}{12}\left(\frac{8.5}{12}\right)(38.833)^3 = 3456.8\ ft^4$$

SUPER: $$y_{bot} = \frac{5(2.652)(17.08/12) + (27.507)(39 + 4.25)/12}{(5)(2.652) + 27.507} \Longrightarrow \underline{\underline{y_{bot} = 3.412'}}$$

$$I_z = (2.982 \overset{(5)}{+} 1.150) + \left[\overset{(5)\,2.894}{(2.652)\left(3.412 - \frac{17.08}{12}\right)^2} + (27.507)\left(\frac{43.25}{12} - \overset{2.894}{3.412}\right)^2\right]$$

$$\underline{\underline{I_z = 15.636\ ft^4}}$$

$$I_y = 3456.8 + (0.247)(5) + 2\left[(2.652)(8.83^2 + 17.67^2)\right]$$

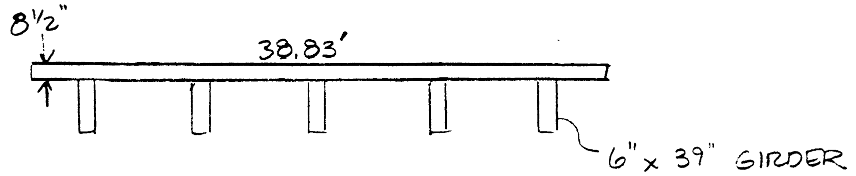
$$\underline{\underline{I_y = 5527.3\ ft^4}}$$

$$A = (5)(2.652) + 27.507 \qquad \underline{\underline{A = 40.77\ ft^2}}$$

- FOR TORSIONAL SUPERSTRUCTURE MOMENT OF INERTIA:

$$J = \Sigma \left(\text{deck} + \text{slab}\right)$$

SIMPLIFY SUPER AS:



$$J = \left(\frac{bh^3}{3}\right)\left(1 - 0.63\frac{h}{b}\right)$$, b = smaller dimension

$$J_{slab} = \frac{(38.83')\left(\frac{8.5}{12}\right)^3}{3}\left(1 - 0.63\,\frac{8.5/12}{38.83}\right) = 4.55\ ft^4$$

$$J_{girder} = \frac{(3.25)(0.5)^3}{3}\left(1 - 0.63\frac{0.5}{3.25}\right) = 0.122\ ft^4$$

$$J_{super} = 4.55 + 5(0.122) \Rightarrow \underline{\underline{J_{super} = 5.16\ ft^4}}$$

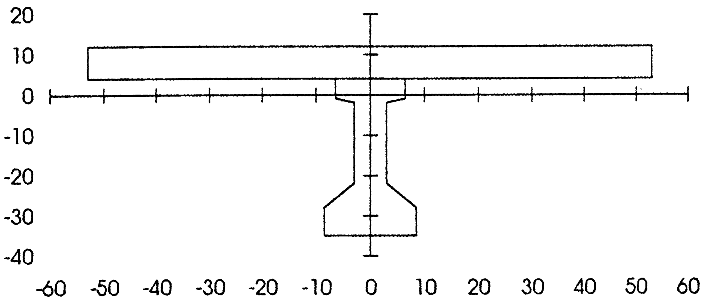
$$\left(\text{Compare to } J = 2J_{slab} = 2(4.55\ ft^4) = 9.1\ ft^4\right)$$

SECTION PROPERTIES WORKSHEET

PROJECT: NCHRP MISSOURI BRIDGE **FOR SECTION PROPERTY CALCULATIONS ONLY**
SUBJECT: **COMPOSITE GIRDER (For Design Only)**
DESIGNER: MLT
DATE: 4-Aug-00

SECTION COORDINATES (CLOCKWISE):

X	Y
0.0000	0.0000
-8.5000	0.0000
-8.5000	7.0000
-3.0000	13.0000
-3.0000	33.0000
-6.5000	34.0000
-6.5000	39.0000
0.0000	39.0000
-53.0000	39.0000
-53.0000	47.0000
53.0000	47.0000
53.0000	39.0000
0.0000	39.0000
6.5000	39.0000
6.5000	34.0000
3.0000	33.0000
3.0000	13.0000
8.5000	7.0000
8.5000	0.0000
0.0000	0.0000
END	



CIRCULAR HOLES:

Xo	Yo	D

RECTANGULAR HOLES:

Xo	Yo	B	H

HOLE COORDINATES (COUNTERCLOCKWISE):

X	Y

Slab Width
106
Slab Thickness
8 (minus 1/2" wear)

SECTION PROPERTIES:
L = inches

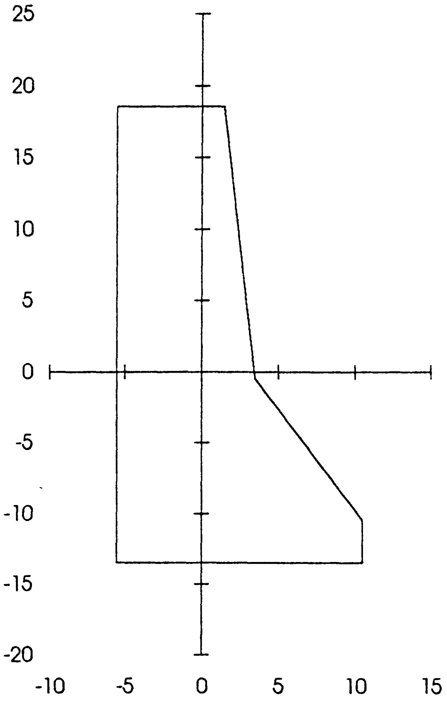
Area:	1230.5000	L^2
Xbar:	0.0000	L
Ybar:	34.9361	L
Ix:	243917.3875	L^4
Iy:	799167.5208	L^4
S top:	20218.7326	L^3
S bot:	6981.8211	L^3

SECTION PROPERTIES WORKSHEET

PROJECT: NCHRP MISSOURI BRIDGE
SUBJECT: **BARRIER PROPERTIES**
DESIGNER: MLT
DATE: 4-Aug-00

SECTION COORDINATES (CLOCKWISE):

X	Y
0.0000	0.0000
0.0000	32.0000
7.0000	32.0000
9.0000	13.0000
16.0000	3.0000
16.0000	0.0000
0.0000	0.0000
END	



HOLE COORDINATES (COUNTERCLOCKWISE):

X	Y

CIRCULAR HOLES:

Xo	Yo	D

RECTANGULAR HOLES:

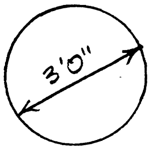
Xo	Yo	B	H

$W_b = 2\left(\frac{325 in^2}{144}\right)(0.150 kcf) = 0.68$ k/ft

SECTION PROPERTIES:
L = inches

Area:	325.0000	L^2
Xbar:	5.5287	L
Ybar:	13.4569	L
Ix:	28213.6469	L^4
Iy:	4475.9820	L^4
S top:	1521.5192	L^3
S bot:	2096.5897	L^3

COLUMN



$$A = \frac{\pi}{4}D^2 = \frac{\pi}{4}(3')^2 = 7.07\ ft^2$$

$$I_z = I_y = \frac{\pi}{64}D^4 = \frac{\pi}{64}(3')^4 = 3.97\ ft^4$$

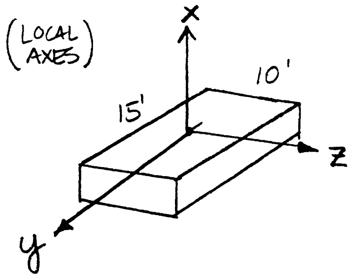
$$J = \frac{\pi}{32}D^4 = \frac{\pi}{32}(3')^4 = 7.95\ ft^4$$

ASSUME $I_{cr} = 0.4 I_g$

$$I_{z_{cr}} = I_{y_{cr}} = 0.4(3.97) = 1.6\ ft^4$$

$$J_{cr} = (0.4)(7.95) = 3.2\ ft^4$$

PILE CAP



$$A = (10')(15') = 150\ ft^2$$

$$I_y = \frac{1}{12}(15')(10')^3 = 1250\ ft^4$$

$$I_z = \frac{1}{12}(10')(15')^3 = 2812.5\ ft^4$$

$$J = \frac{1}{12}(150\ ft^2)(10^2 + 15^2) = 4062.5\ ft^4$$

LOADS

☐ SUPERIMPOSED DEAD LOAD

BARRIERS: $w_b = 0.68 \; ^k/_{ft}$

OVERLAY: $w_{o} = (0.25')(38.33' - 2(0.875))(0.140 \text{ kcf})$

$= 1.3 \; ^k/_{ft}$

$w_{SDL} = 0.68 + 1.3 \implies \underline{\underline{w_{SDL} = 1.98 \; ^k/_{ft}}}$

☐ CROSS BEAM

$$P_{XBEAM} \approx \underset{W \;\times\; H \;\times\; L \;\times\; \gamma}{(3.5')(3.25')(39.67')(0.150 \text{ kcf})} \implies \underline{\underline{P_{XBEAM} = 67.7^k}}$$

☐ INT. DIAPHRAGM (C15 x 33.9 w/ L4x6x½x16")

$$P_{I.D.} = \left[(8.83' - 0.5')(33.9 \text{ plf}) + 2\left(\frac{4.75 \text{ in}^2}{144}\right)(1.33')(490 \text{ pcf})\right](4)/1000 = 1.3^k$$

$$\implies \underline{\underline{P_{I.D.} = 1.5^k}}$$

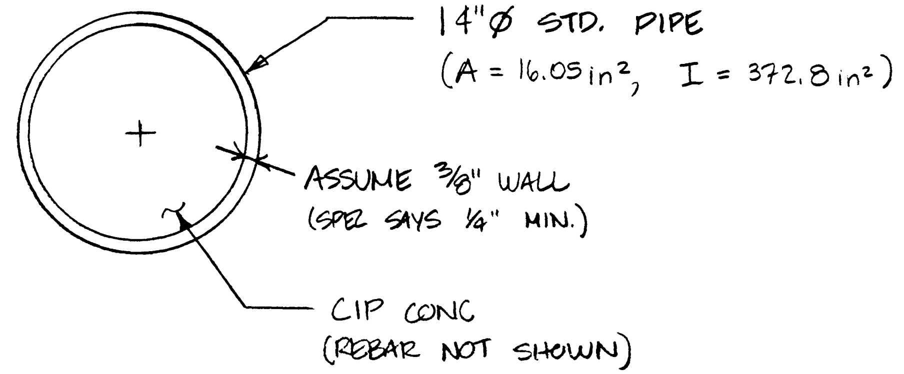
☐ END DIAPHRAGM (APPROX.)

$$P_{E.D.} = \Big(\underset{(7.125')}{\frac{8.5'' + 39'' + \overset{\text{MAX FILLET}}{2''}}{12}} + 3'\Big)(41.5')(2.5')(0.150 \text{ kcf})$$

$$\implies \underline{\underline{P_{ED} = 110.9^k}}$$

I.3 SAP2000 Model Development: Lateral Pile (LPILE) Analyses

PILE SECTION PROPERTIES



AREA:

$$A_c = \frac{\pi}{4}D^2 = \frac{\pi}{4}(13.25")^2 \Rightarrow \underline{\underline{A_c = 137.9\,\text{in}^2}}$$

$$n = \frac{E_s}{E_c} = \frac{29000\text{ ksi}}{3800\text{ ksi}} = 7.6 \approx 8$$

$$A_{st} = n\left(\frac{\pi}{4}\right)(D_o^2 - D_i^2) = (8)\left(\frac{\pi}{4}\right)(\underbrace{13.875^2}_{\text{NEGLECT } 1/16" \text{ FOR CORROSION}} - 13.25^2) \Rightarrow \underline{\underline{A_{st} = 106.5\,\text{in}^2}}$$

(transformed)

$$A_{pt} = A_c + A_{st} = 137.9 + 106.5 \Rightarrow \boxed{A_{pt} = 244.4\,\text{in}^2}$$

MOMENT OF INERTIA:

$$I_c = \frac{\pi}{64}D^4 = \frac{\pi}{64}(13.25")^4 \Rightarrow \underline{\underline{I_c = 1513.0\,\text{in}^4}}$$

$$I_{st} = \frac{1}{16}A_{st}(D_o^2 + D_i^2) = \frac{1}{16}(106.5\text{in}^2)(13.875^2 + 13.25^2)$$

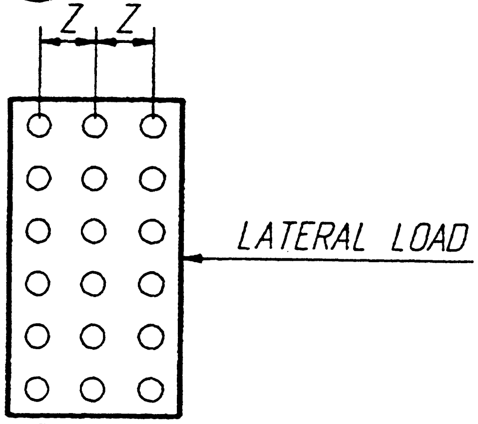
$$\Rightarrow \underline{\underline{I_{st} = 2450.0\,\text{in}^4}}$$

$$I_{pt} = I_c + I_{st} = 1513.0 + 2450.0 \Rightarrow \boxed{I_{pt} = 3963.0\,\text{in}^4}$$

PIERS 1 & 4

Z	EFFICIENCY (REDUCTION) FACTOR
8D	1.0
6D	0.8
4D	0.5
3D	0.4

EFFICIENCY FACTOR



GROUP PILE PLAN

Efficiency Factor
Table 4.4.3-1

For driven piles, the following factors apply:

Contact the Olympia Service Center Materials Lab to verify any assumptions.

0.5K

The LPILE1 computer program will generate P-Y curves, or the user can input them. To obtain generated curves, input a modulus of subgrade reaction (K), and a soil shear strength (C) which are the values taken from the soils report multiplied by the efficiency factor. To figure P-Y curves for input, multiply the P-Y values from the soils report by the efficiency factor.

0.5Cu

For a typical soil, the relationship between its normalized resistance value and friction angle is defined by the curve in Figure 4.4.3-1. The friction angle could be adjusted for efficiency and input to LPILE1 by following these steps:

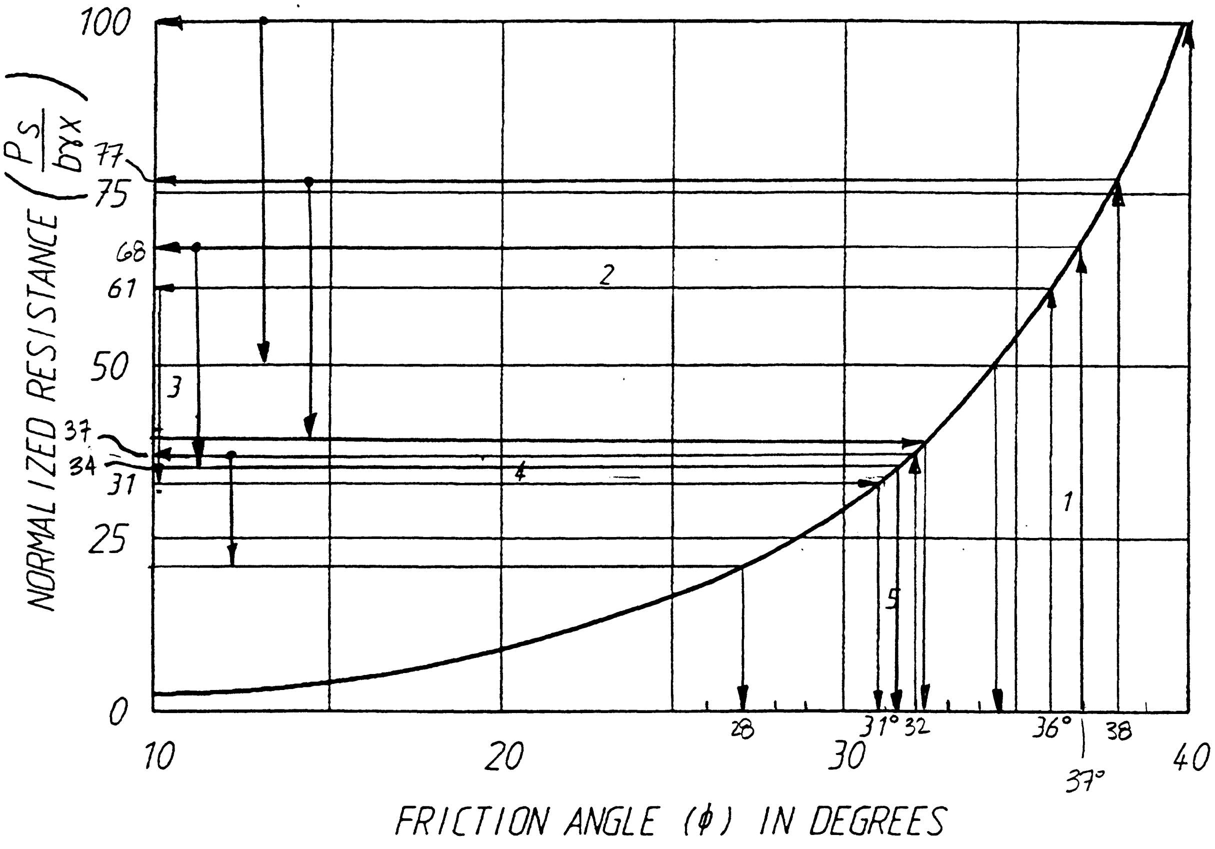
$\phi \Rightarrow$ 1. Begin at the coordinate of the natural friction angle (36°).

2. Read across to the normalized resistance (61).
3. Multiply the resistance by the efficiency reduction factor, i.e., 61 (0.5) = 31.
4. Read across from the reduced value to obtain the adjusted friction angle (31°).
5. Input the ϕ value to LPILE1.

$Z = 4.75'$ $D = 14'' = 1.167'$

$\frac{Z}{D} = 4.1 \Rightarrow \boxed{4D}$

PIERS 1 & 4



EFFICIENCY FACTOR = 0.5

Friction Angle (φ)

$$\frac{PS}{b\gamma x} = K_a\,(\tan^8 B-1) + K_o \tan\phi \tan{}^4 B$$

P_S = Soil Resistance on Pile Element

b = Pile Width

g = Soil Unit Weight

X = Depth to Pile Element

N = Step in Example

B = 45° + φ/2

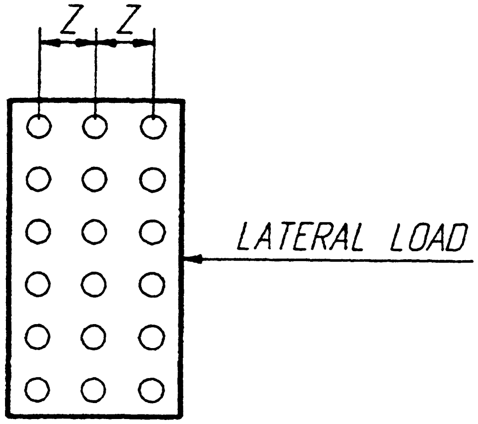
K_a = tan2(45° – φ/2)

K_o = , 1 – Sin φ

LAYER	φ	NORM. RESIS	N. RESIS ×0.5	φ LPILE
①	37°	68	34	31°
④	32°	37	19	28°
⑤	32°	37	19	28°
⑥	38°	77	39	32°
⑦	38°	77	39	32°
⑧	40°	100	50	34°

Z	EFFICIENCY (REDUCTION) FACTOR
8D	1.0
6D	0.8
4D	0.5
3D	0.4

EFFICIENCY FACTOR



GROUP PILE PLAN

Efficiency Factor
Table 4.4.3-1

For driven piles, the following factors apply:

Contact the Olympia Service Center Materials Lab to verify any assumptions.

0.4K

The LPILE1 computer program will generate P-Y curves, or the user can input them. To obtain generated curves, input a modulus of subgrade reaction (K), and a soil shear strength (C) which are the values taken from the soils report multiplied by the efficiency factor. To figure P-Y curves for input, multiply the P-Y values from the soils report by the efficiency factor.

0.4Cu

For a typical soil, the relationship between its normalized resistance value and friction angle is defined by the curve in Figure 4.4.3-1. The friction angle could be adjusted for efficiency and input to LPILE1 by following these steps:

$\phi \Rightarrow$

1. Begin at the coordinate of the natural friction angle (36°).
2. Read across to the normalized resistance (61).
3. Multiply the resistance by the efficiency reduction factor, i.e., 61 (0.5) = 31.
4. Read across from the reduced value to obtain the adjusted friction angle (31°).
5. Input the ϕ value to LPILE1.

TRANSVERSE SPACING

$Z = 3.0'$ $D = 14'' = 1.167'$

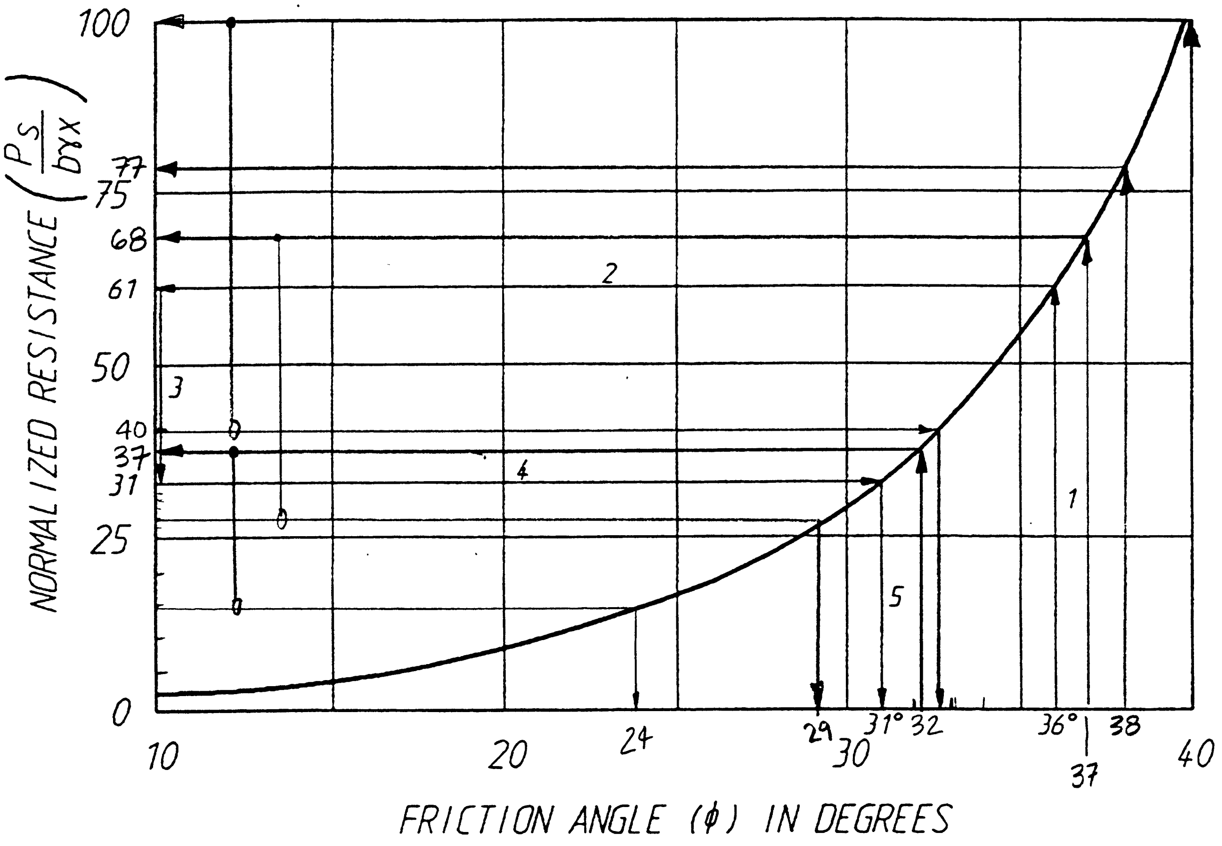
$\frac{Z}{D} = 2.6 \longrightarrow \textcircled{3D}$

LONGITUDINAL SPACING

$Z = 3.5'$ $D = 1.167'$

$\frac{Z}{D} = 3.0 \longrightarrow \textcircled{3D}$

PIERS 2 & 3



EFFICIENCY FACTOR = 0.4

Friction Angle (φ)

$$\frac{PS}{b\gamma x} = K_a (\tan^8 B - 1) + K_o \tan \phi \tan^4 B$$

P_s = Soil Resistance on Pile Element

b = Pile Width

g = Soil Unit Weight

X = Depth to Pile Element

N = Step in Example

B = 45° + φ/2

K_a = tan2(45° – φ/2)

K_o = , 1 – Sin φ

LAYER	Ø	NORM. RESIS.	RESIS. x0.4	Ø LPILE
①	37°	68	27	29°
④	32°	37	15	24°
⑤	32°	37	15	24°
⑥	38°	77	31	31°
⑦	38°	77	31	31°
⑧	40°	100	40	33°

LPILE COORDINATES

MISSOURI SITE

NON-LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations	Pier			
			1	2	3	4
			Pile Head Elevation			
			23.5	-7	-7	23.5
1	Sand (Fill)	30	-78	-120	-120	-78
2	Soft Clay	0	282	-84	-84	282
3	Soft Clay	-10	402	36	36	402
4	Sand (Liquefiable)	-20	522	156	156	522
5	Sand (Liquefiable)	-30	642	276	276	642
6	Dense Sand	-40	762	396	396	762
7	Dense Sand	-60	1002	636	636	1002
8	Dense Alluvium (Sand)	-40	762	396	396	762
	Pile Tip Elevation	-70	1122	756	756	1122

LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations	Pier			
			1	2	3	4
			Pile Head Elevation			
			23.5	-7	-7	23.5
1	Sand (Fill)	30	-78	-120	-120	-78
2	Soft Clay	0	282	-84	-84	282
3	Soft Clay	-10	402	36	36	402
4	Liquefied Sand	-20	522	156	156	522
5	Liquefied Sand	-30	642	276	276	642
6	Dense Sand	-40	762	396	396	762
7	Dense Sand	-60	1002	636	636	1002
8	Dense Alluvium (Sand)	-40	762	396	396	762
	Pile Tip Elevation	-70	1122	756	756	1122

LPILE COORDINATES

MISSOURI SITE

NON-LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations
1	Sand (Fill)	30
2	Soft Clay	0
3	Soft Clay	-10
4	Sand (Liquefiable)	-20
5	Sand (Liquefiable)	-30
6	Dense Sand	-40
7	Dense Sand	-60
8	Dense Alluvium (Sand)	-40
	Pile Tip Elevation	-70

P-Y Curve Depths at Mid-Depth of Each Soil Layer

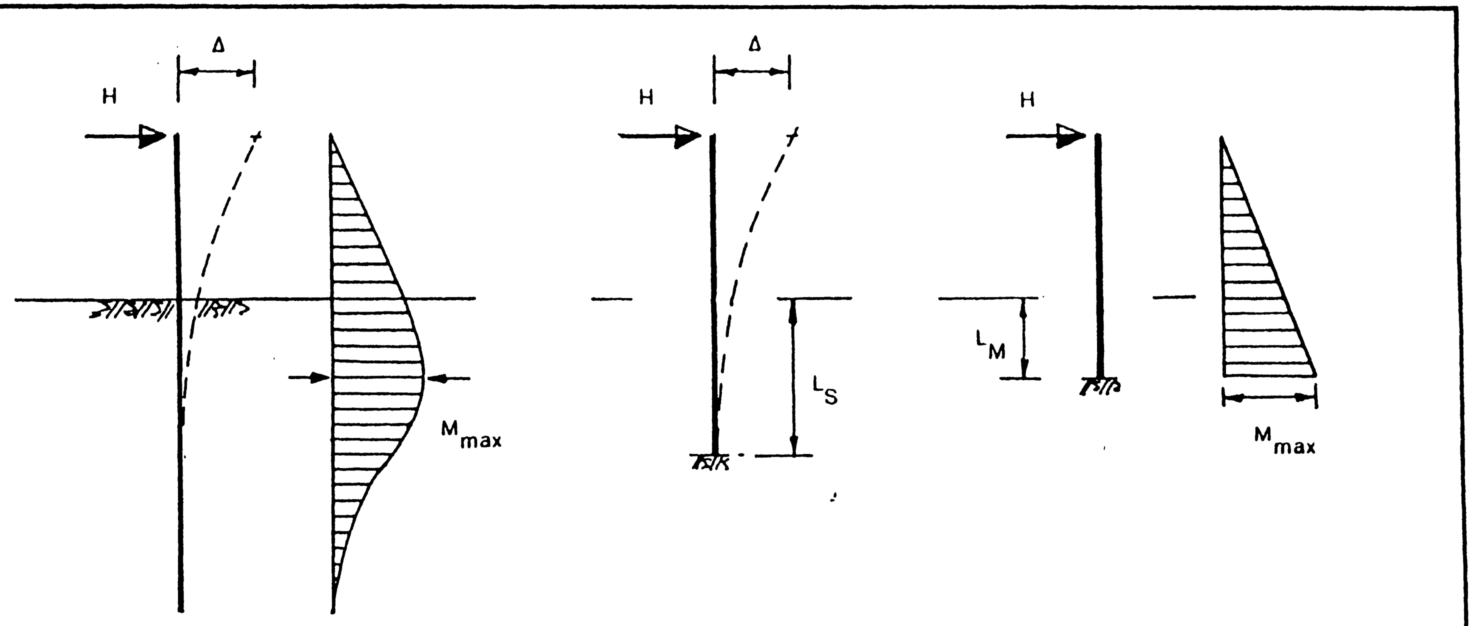
Pier			
1	**2**	**3**	**4**
102	-102	-102	102
342	-24	-24	342
462	96	96	462
582	216	216	582
702	336	336	702
882	516	516	882
882	516	516	882
762	396	396	762

LIQUEFIED SOIL PROFILE

Layer No.	Soil Type	Top of Soil Layer Elevations
1	Sand (Fill)	30
2	Soft Clay	0
3	Soft Clay	-10
4	Liquefied Sand	-20
5	Liquefied Sand	-30
6	Dense Sand	-40
7	Dense Sand	-60
8	Dense Alluvium (Sand)	-40
	Pile Tip Elevation	-70

P-Y Curve Depths at Mid-Depth of Each Soil Layer

Pier			
1	**2**	**3**	**4**
102	-102	-102	102
342	-24	-24	342
462	96	96	462
582	216	216	582
702	336	336	702
882	516	516	882
882	516	516	882
762	396	396	762



	L_S	L_M
Cohesive Soil Constant K_h	$1.4\sqrt[4]{\frac{EI}{K_h}}$	$0.44\sqrt[4]{\frac{EI}{K_h}}$
Cohesionless Soil Constant n_h	$1.8\sqrt[5]{\frac{EI}{n_h}}$	$0.78\sqrt[5]{\frac{EI}{n_h}}$

Figure 69: Equivalent Cantilever Method using Relative Stiffness Factors (after References 29 and 36)

"T" DISTANCE

MISSOURI SITE

$E_c =$ 3830 ksi
$I_{cr} =$ 2450 in.^{4}
$I =$ 3963 in.^{4}

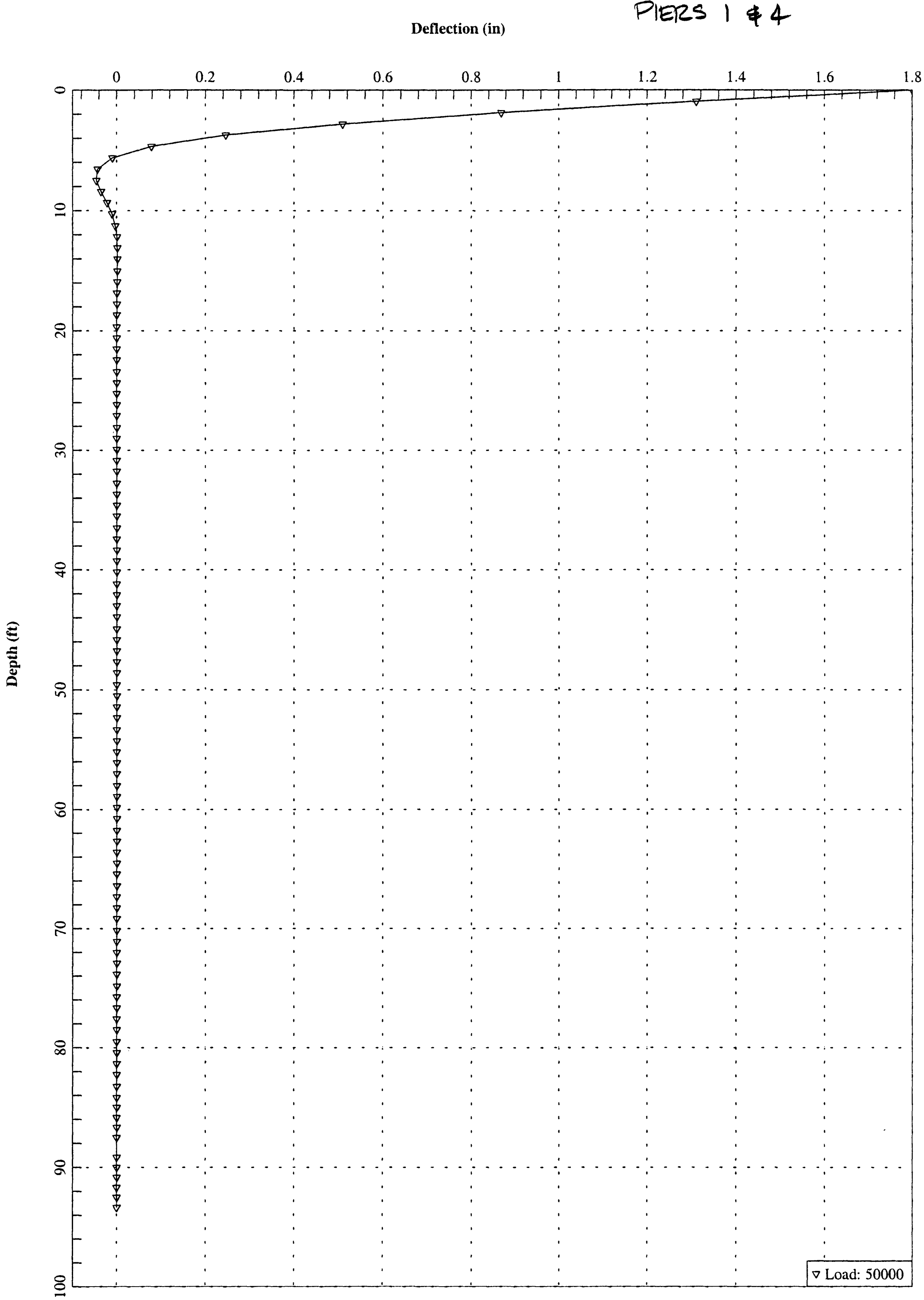
NON-LIQUEFIED SOIL PROFILE

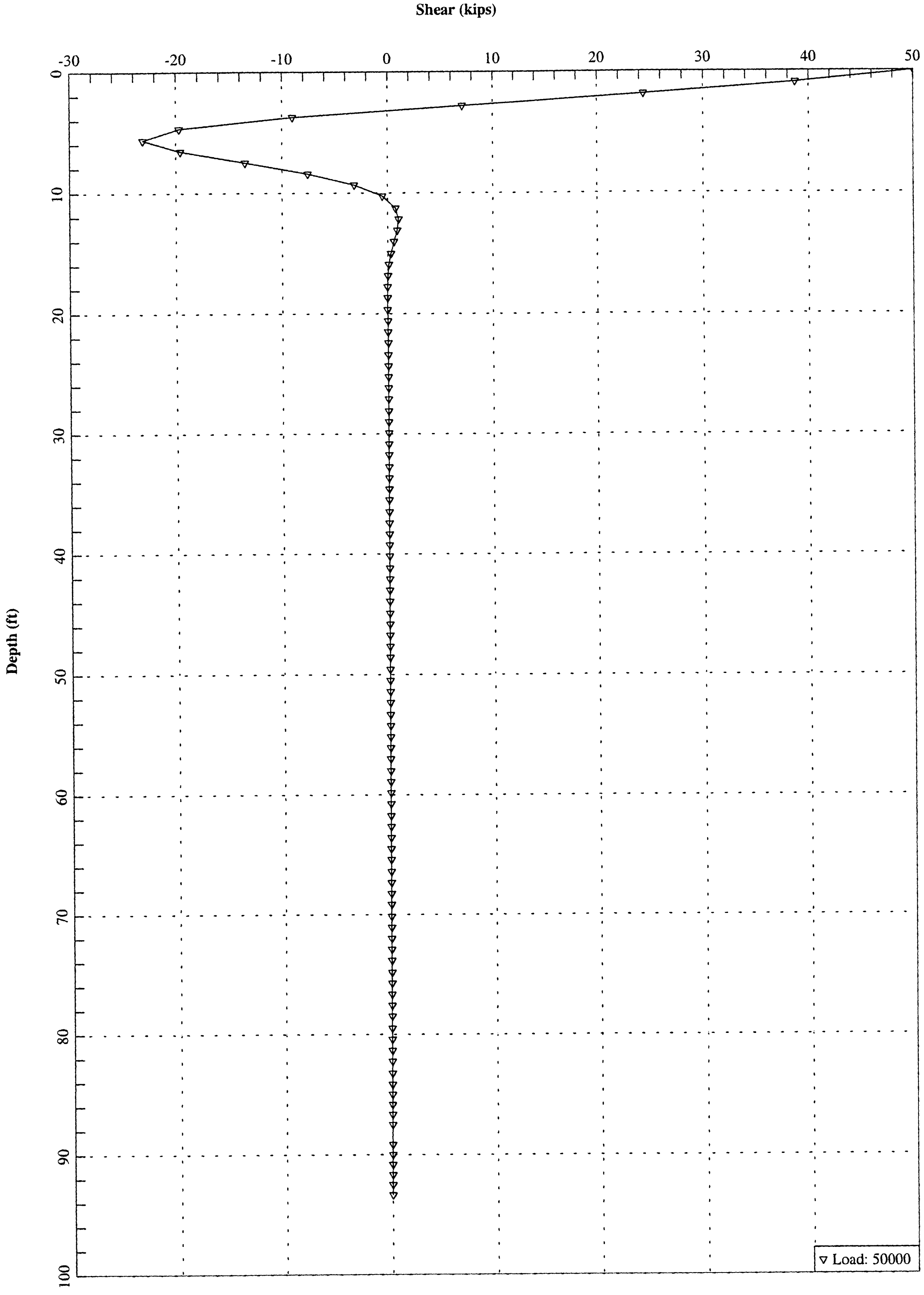
Layer No.	Soil Type	Subgrade Modulus K_h or n_h (pci)	Using Uncracked I $L_M = 2T$ feet	 $L_S = T$ feet	Using Cracked I $L_M = 2T$ feet	 $L_S = T$ feet
1	Sand (Fill)	150	2.4	5.4	2.6	6.0
2	Soft Clay	10	6.4	20.4	7.2	23.0
3	Soft Clay	20	3.5	8.1	3.9	9.0
4	Sand (Liquefiable)	40	3.1	7.1	3.4	7.8
5	Sand (Liquefiable)	40	3.1	7.1	3.4	7.8
6	Dense Sand	100	2.6	5.9	2.8	6.5
7	Dense Sand	100	2.6	5.9	2.8	6.5
8	Dense Alluvium (Sand)	200	3.0	9.7	3.4	10.9

LIQUEFIED SOIL PROFILE

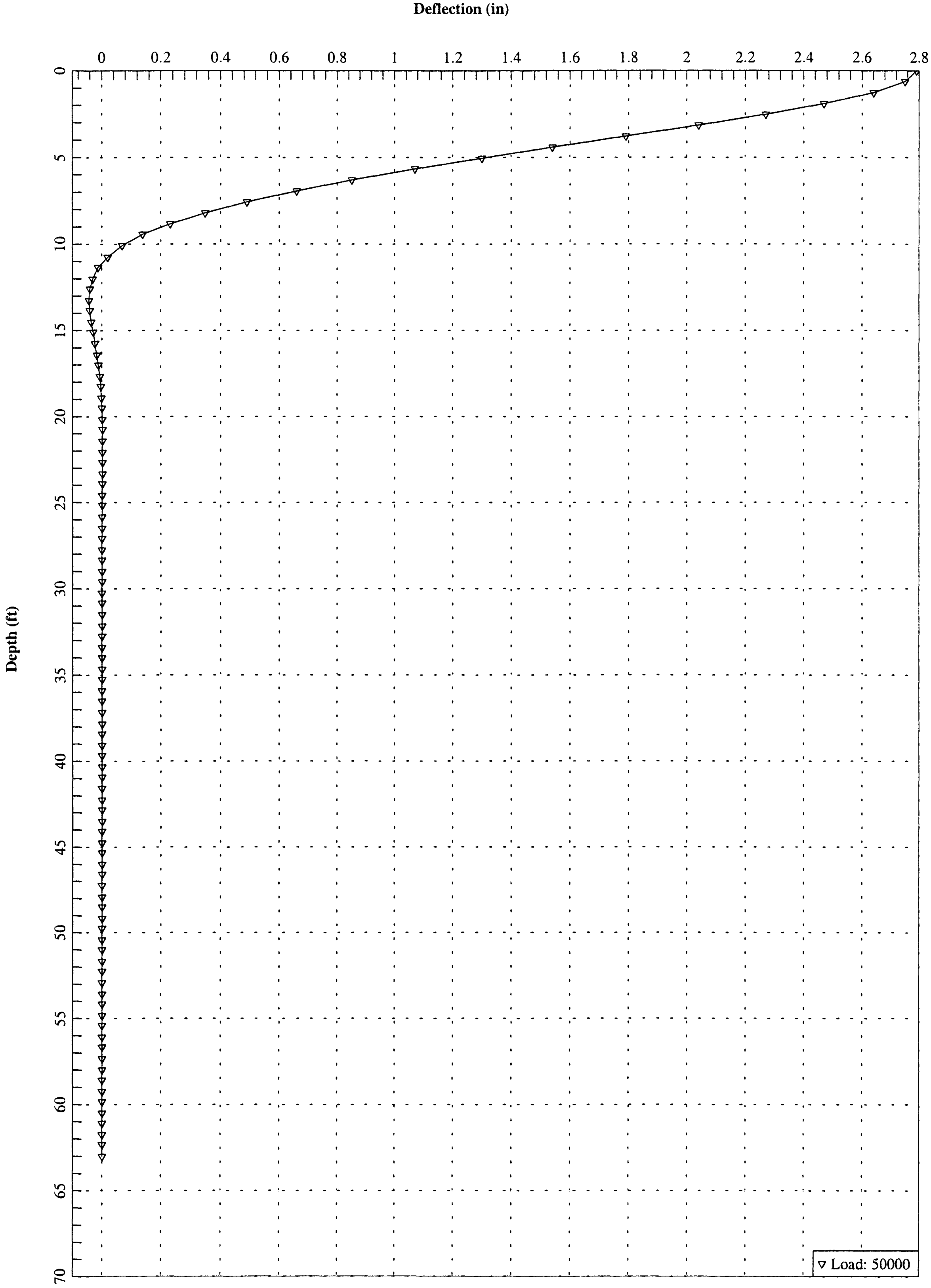
Layer No.	Soil Type	Subgrade Modulus K_h or n_h (pci)	$L_M = 2T$ feet	$L_S = T$ feet	$L_M = 2T$ feet	$L_S = T$ feet
1	Sand (Fill)	150	2.4	5.4	2.6	6.0
2	Soft Clay	10	6.4	20.4	7.2	23.0
3	Soft Clay	20	5.4	17.2	6.1	19.4
4	Liquefied Sand	3	5.2	11.9	5.7	13.1
5	Liquefied Sand	3	5.2	11.9	5.7	13.1
6	Dense Sand	100	2.6	5.9	2.8	6.5
7	Dense Sand	100	3.6	11.5	4.1	12.9
8	Dense Alluvium (Sand)	200	3.0	9.7	3.4	10.9

NON- LIQUEFIED
PIERS 1 & 4

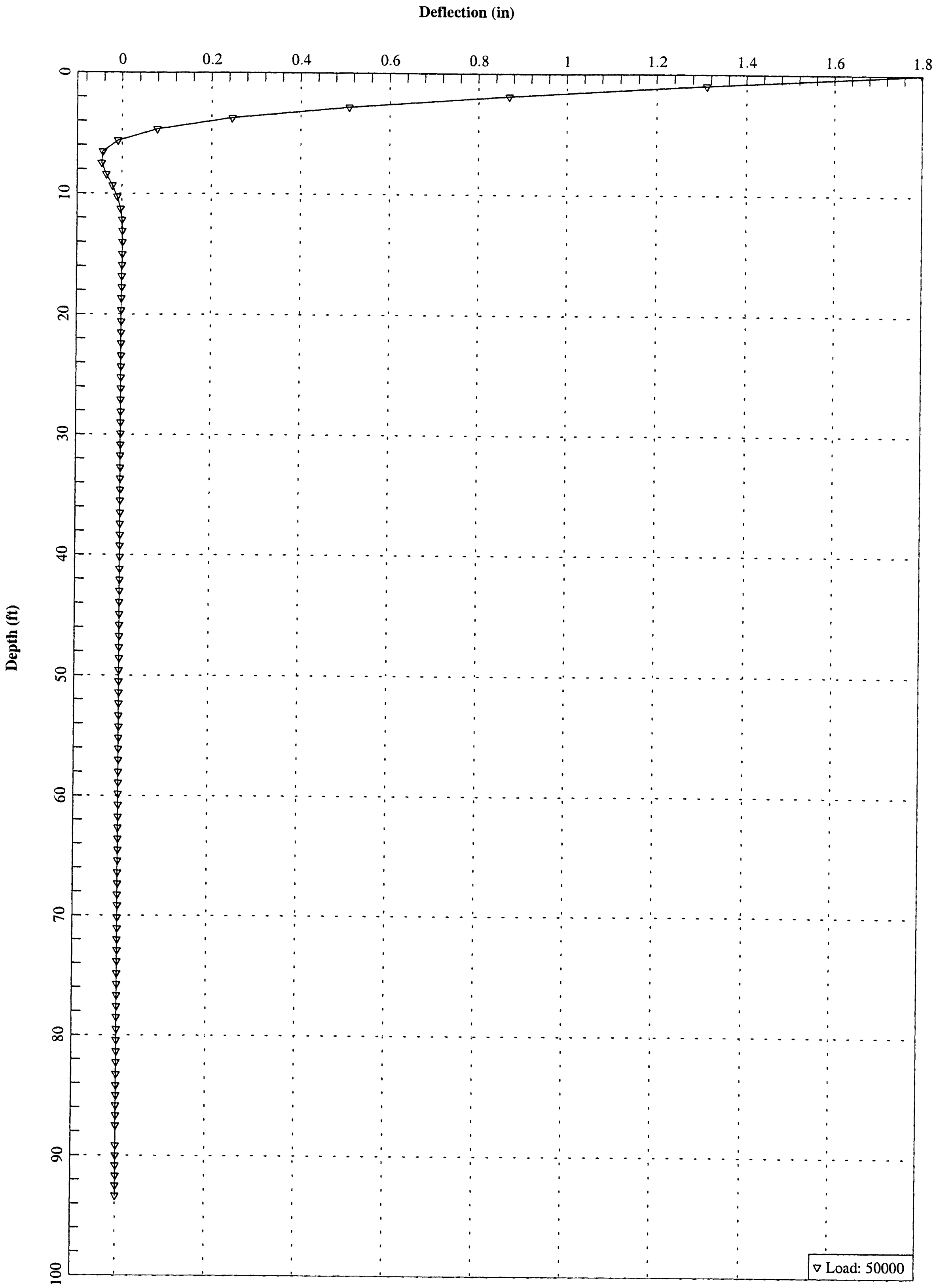


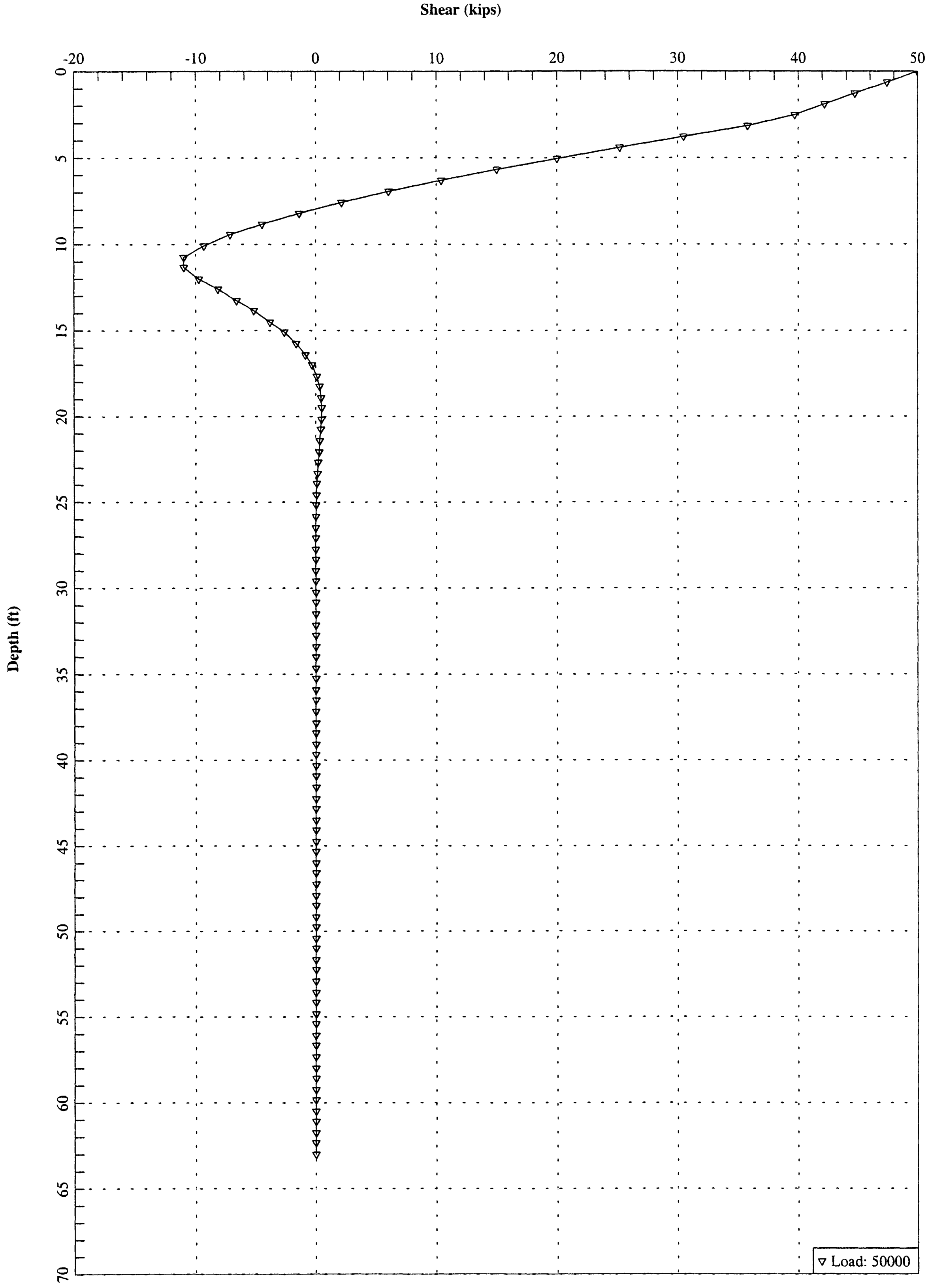

Shear (kips)
-30
-20
-10
0
10
20
30
40
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
▽ Load: 50000

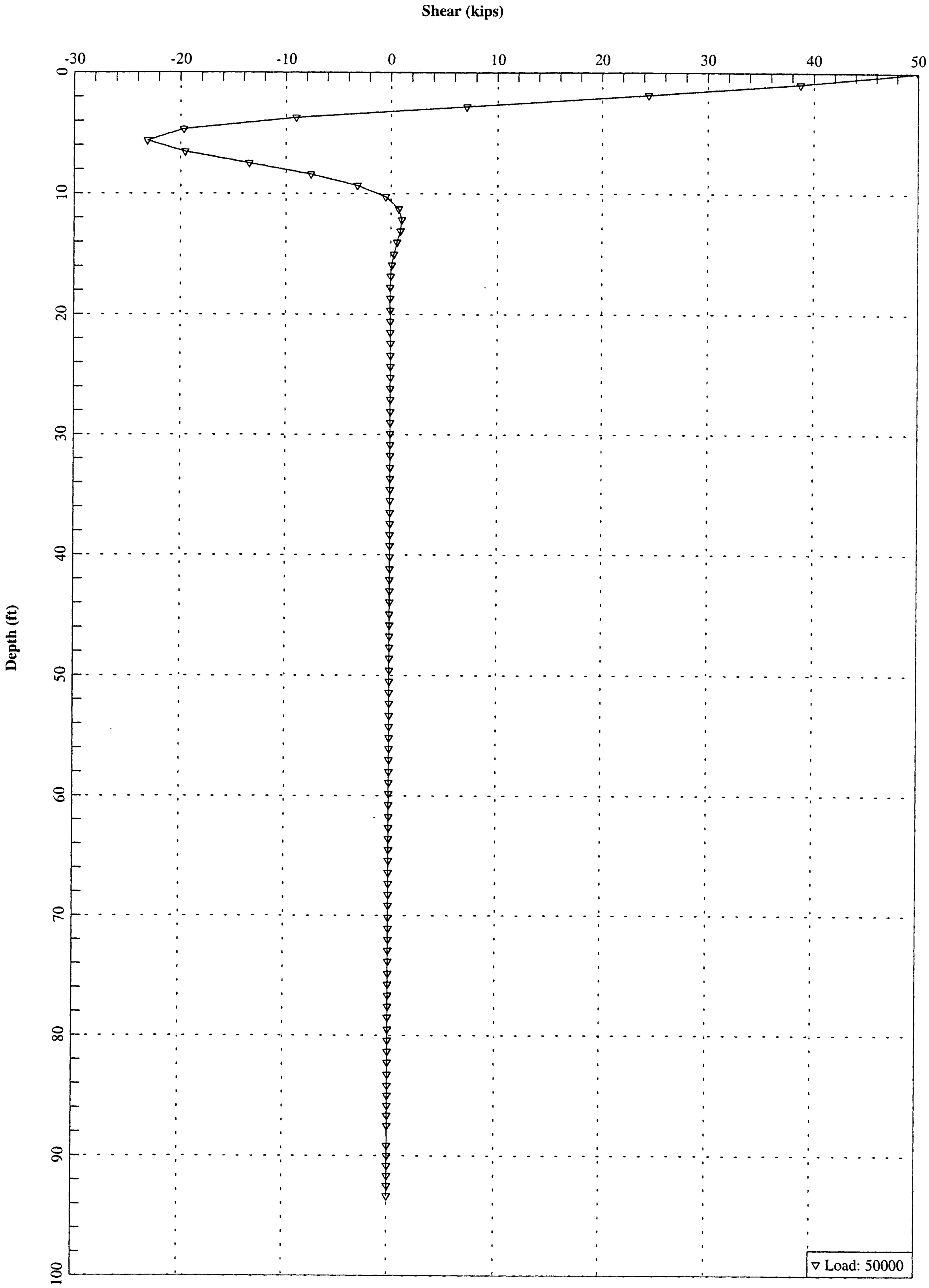
NON - LIQUEFIED
PIERS 2 & 3



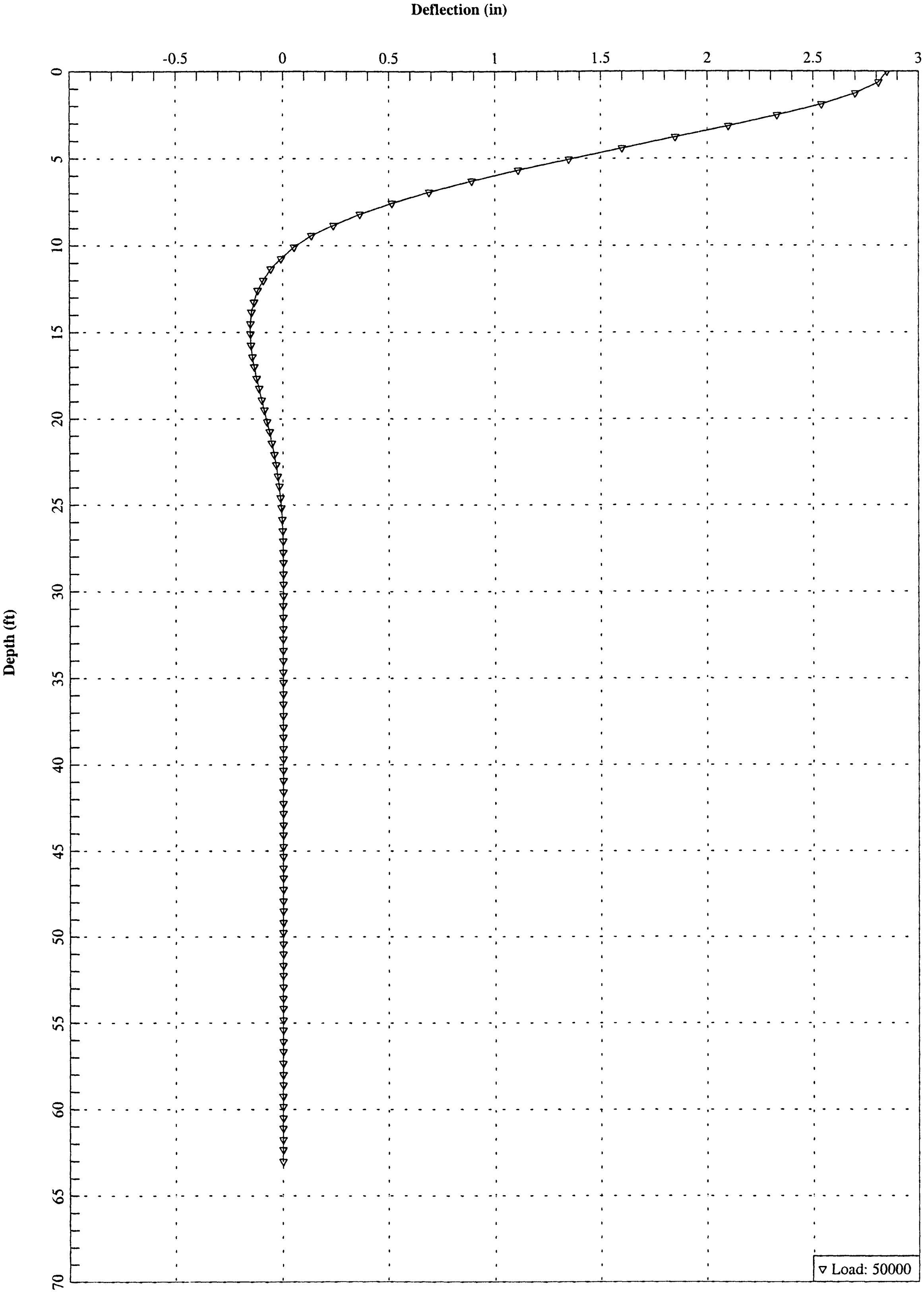
LIQUEFIED
PIERS 1 & 4

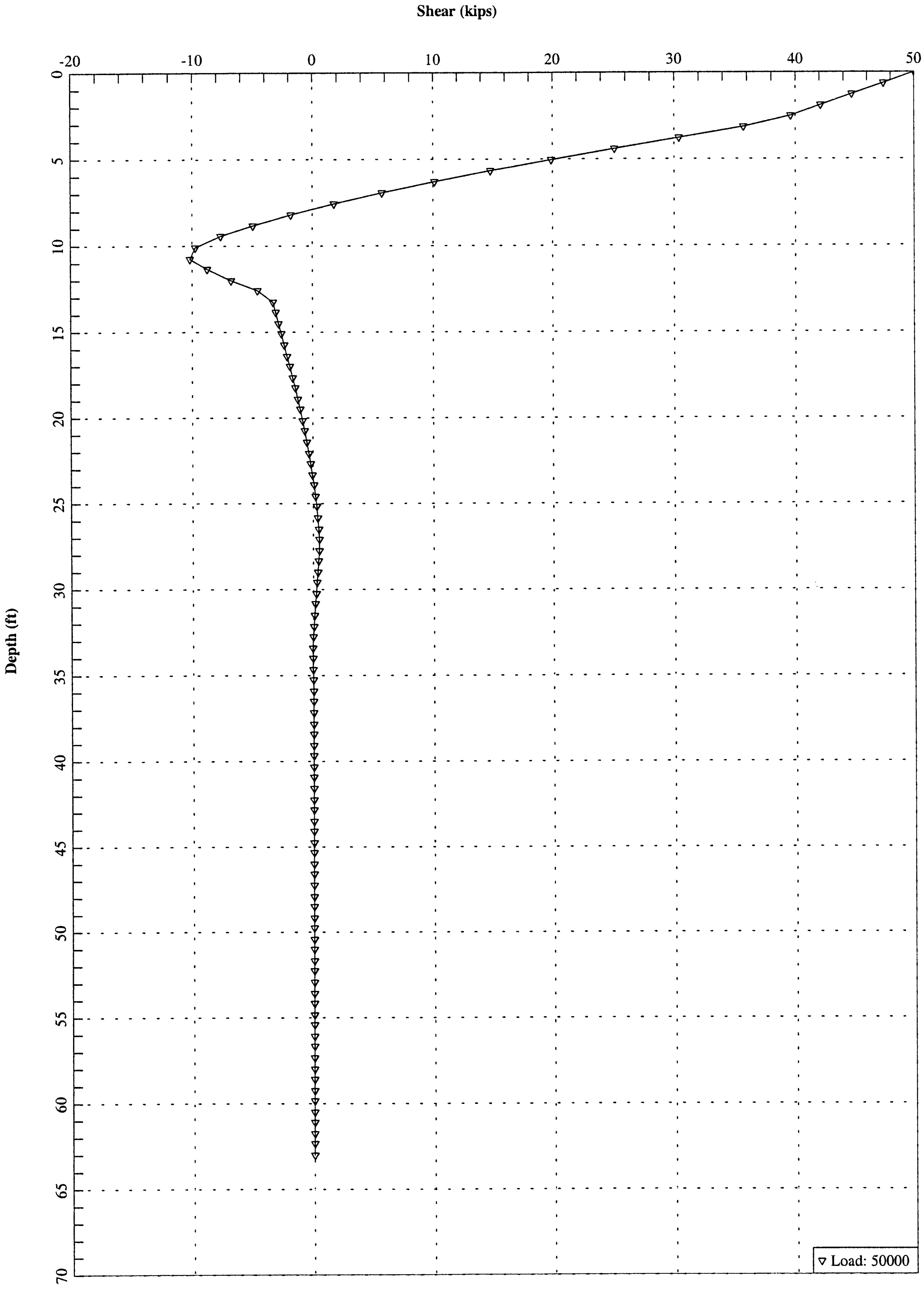



Shear (kips)
-20
-10
0
10
20
30
40
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
▽ Load: 50000


Shear (kips)
-30
-20
-10
0
10
20
30
40
50
Depth (ft)
0
10
20
30
40
50
60
70
80
90
100
▽ Load: 50000

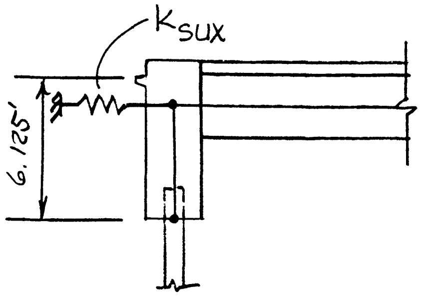
LIQUEFIED
PIERS 2 & 3




Shear (kips)
-20
-10
0
10
20
30
40
50
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
▽ Load: 50000

I.4 SAP2000 Model Development: Foundation Springs

ABUTMENT BACKWALL (LONGITUDINAL) STIFFNESS



$$K_{SUX} = \frac{F_P}{\Delta}$$

$$F_P = \underset{\uparrow}{P_P}A$$

NCHRP 11.6.5.1.1b

$$P_P = \frac{2}{3}H = \frac{2}{3}(6.125') = 4.1 \text{ ksf}$$

$$A = (6.125')(41.5') = 254.2 \text{ ft}^2$$

$$\Delta = 0.02H \text{ to mobilize soil} = 0.02(6.125') = 0.12'$$

$$\therefore K_{SUX} = \frac{P_P A}{\Delta} = \frac{(4.1 \text{ ksf})(254.2 \text{ ft}^2)}{0.12'} = \frac{1038}{0.12} \Rightarrow \boxed{K_{SUX} = 8473 \text{ k/ft}}$$

APPLY $1/2\, K_{SUX}$ @ EACH ABUTMENT NODE IN SAP $\Rightarrow$ $\underline{\underline{K_{SUX}' = 4236 \text{ k/ft}}}$

$(F_{1/2} = 519^k)$

$$\boxed{\text{AXIAL STIFFNESS}}$$

- PIERS 1 & 4 (ABUTMENTS)

$$K_{UY} = N_P K_{AXIAL} \quad , \quad N_P = 9$$

$$K_{AXIAL} = \frac{AE}{L} = \frac{(244.4 in^2)(3830 ksi)}{(93.5')} \Rightarrow \underline{\underline{K_{AXIAL} = 10011 \,^{k}/_{ft}}}$$

$$\therefore K_{UY} = (9)(10011 \,^{k}/_{ft}) \Rightarrow \boxed{K_{UY} = 90101 \,^{k}/_{ft}}$$

(PER ABUTMENT)

- PIERS 2 & 3 (INT. PIERS)

$$N_P = 14 \text{ (PER PILE CAP)}$$

$$K_{UY} = (14) \frac{(244.4 in^2)(3830 ksi)}{(63')} \Rightarrow \boxed{K_{UY} = 208012 \,^{k}/_{ft}}$$

(PER COLUMN)

LONGITUDINAL TRANSLATIONAL STIFFNESS

- PIERS 1 & 4 $(N_P = 9)$

APPLY SHEAR FORCE TO PILE HEAD IN LPILE

(PILE HEAD IS TO ROTATE, ∴ PINNED HEAD)

$$k_{UX} = \left(\frac{V}{\Delta_{HEAD}}\right) N_P$$

	V (kips)	Δ_{HEAD} (ft)	k_{UX} (k/ft)	(SAP)	
NON-LIQUEFIED CASE	50^k	1.29/12	4186		
LIQUEFIED CASE	50^k	1.29/12	4186		

- PIERS 2 & 3 $(N_P = 14)$ - FIXED HEAD

NON-LIQUEFIED CASE	50^k	2.04/12	4118		
LIQUEFIED CASE	50^k	2.10/12	4000		

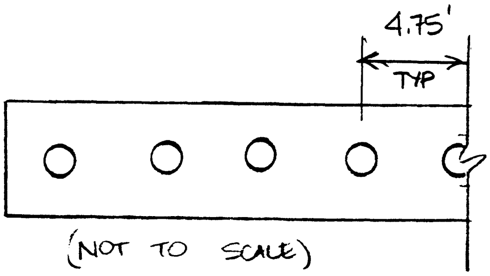
TRANSVERSE TRANSLATIONAL STIFFNESS

- **ALL PIERS**

$K_{UZ} = K_{UX}$ (SEE LONGITUDINAL TRANS. STIFF.)

TORSIONAL STIFFNESS

• PIERS 1 & 4



$$K_{RY} = K_{ux}\Sigma d_i^2 = K_{uz}\Sigma d_i^2$$

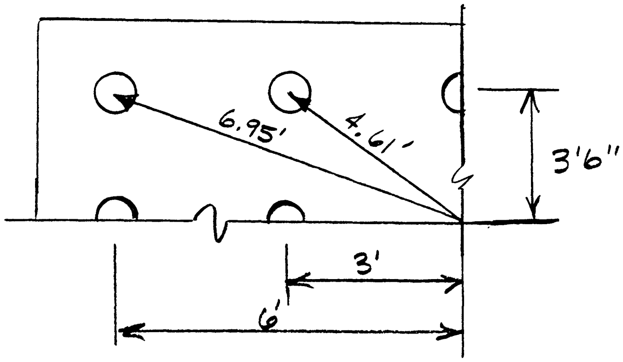
$$K_{RY} = (4186\ ^{k}/_{ft})(4.75^2 + 9.5^2 + 14.25^2 + 19^2)(2)$$

$$= (4186)(1354\ ft^2) \qquad \Rightarrow \boxed{K_{RY} = 5.668\times10^6\ \frac{k\text{-}ft}{rad}}$$

(NON-LIQUEFIED & LIQUEFIED)

• PIERS 2 & 3

$$K_{RY} = k_{ux}\Sigma d_i^2 = K_{uz}\Sigma d_i^2$$



NON-LIQUEFIED

$$K_{RY} = (4118\ ^{k}/_{ft})\underbrace{\left[2(3.5^2 + 3^2 + 6^2) + 4(4.61^2 + 6.95^2)\right]}_{392.5 ft^2}$$

$$NL \Rightarrow \boxed{K_{RY} = 1.616\times10^6\ \frac{k\text{-}ft}{rad}}$$

LIQUEFIED

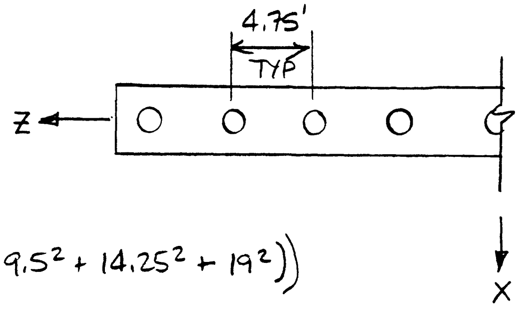
$$K_{RY} = \frac{4000}{4118}(1.616\times10^6) \qquad L \Rightarrow \boxed{K_{RY} = 1.570\times10^6\ \frac{k\text{-}ft}{rad}}$$

LATERAL / TRANSVERSE

$$\boxed{\text{~~LONGITUDINAL~~ ROTATIONAL STIFFNESS}}$$

- PIERS 1 & 4

$$k_{RX} = K_{uy}\ \Sigma d_i^2$$

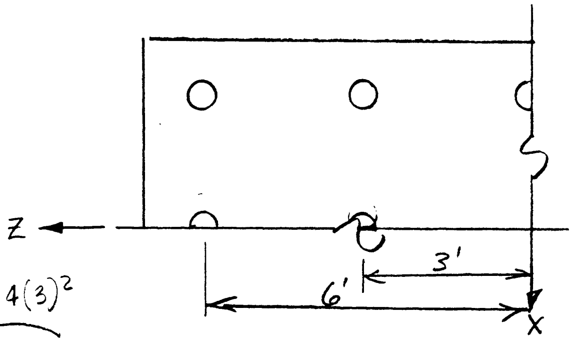


$$k_{RX} = (90101\ ^{k}/_{ft})(2(4.75^2 + 9.5^2 + 14.25^2 + 19^2))$$

$$= (90101)(1354\ ft^2)$$

$$\Rightarrow \boxed{k_{RX} = 1.220 \times 10^8\ \frac{k\text{-}ft}{rad}}$$

- PIERS 2 & 3



$$k_{RX} = (208012\ ^{k}/_{ft})\Big(\overbrace{6(\cancel{3^2+}\ 6^2)}^{6(6)^2 + 4(3)^2}\Big)$$

$$= (208012)(\underset{252}{\cancel{270}}\ ft^2)$$

$$\Rightarrow \boxed{k_{RX} = 5.616 \times 10^7\ \frac{k\text{-}ft}{rad}}$$

5.242×10^7

LONGITUDINAL

$\boxed{\text{~~TRANSVERSE~~ ROTATIONAL STIFFNESS}}$

- PIERS 1 & 4

$$K_{RZ} = K_{UY} \underbrace{\Sigma d_i^2}_{\to \sim 0}$$

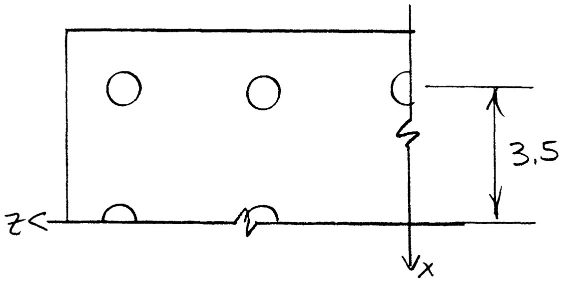
ASSUME $\boxed{K_{RZ} = 0}$

- PIERS 2 & 3

$$K_{RZ} = K_{UY} \Sigma d_i^2 + N_p \cancel{k}^{\,0}$$

$$= (208012 \, k/ft)(10 \cdot 3.5^2)$$

$$= (208012)(122.5)$$



$$\Longrightarrow \boxed{K_{RZ} = 2.548 \times 10^{7} \ \frac{k\text{-}ft}{rad}}$$

MISSOURI BRIDGE FOUNDATION SPRINGS

NON-LIQUEFIED CASE

	Axial	Longitudinal	Lateral	Axial	Longitudinal	Lateral
Global	**UY**	**UX**	**UZ**	**RY**	**RX**	**RZ**
Pier	K_{11} k/ft	K_{22} k/ft	K_{33} k/ft	K_{44} k-ft/rad	K_{55} k-ft/rad	K_{66} k-ft/rad
1	9.01E+04	4.19E+03	4.19E+03	5.67E+06	1.22E+08	0.00E+00
2	2.08E+05	4.12E+03	4.12E+03	1.62E+06	5.62E+07	2.55E+07
3	2.08E+05	4.12E+03	4.12E+03	1.62E+06	5.62E+07	2.55E+07
4	9.01E+04	4.19E+03	4.19E+03	5.67E+06	1.22E+08	0.00E+00

LIQUEFIED CASE

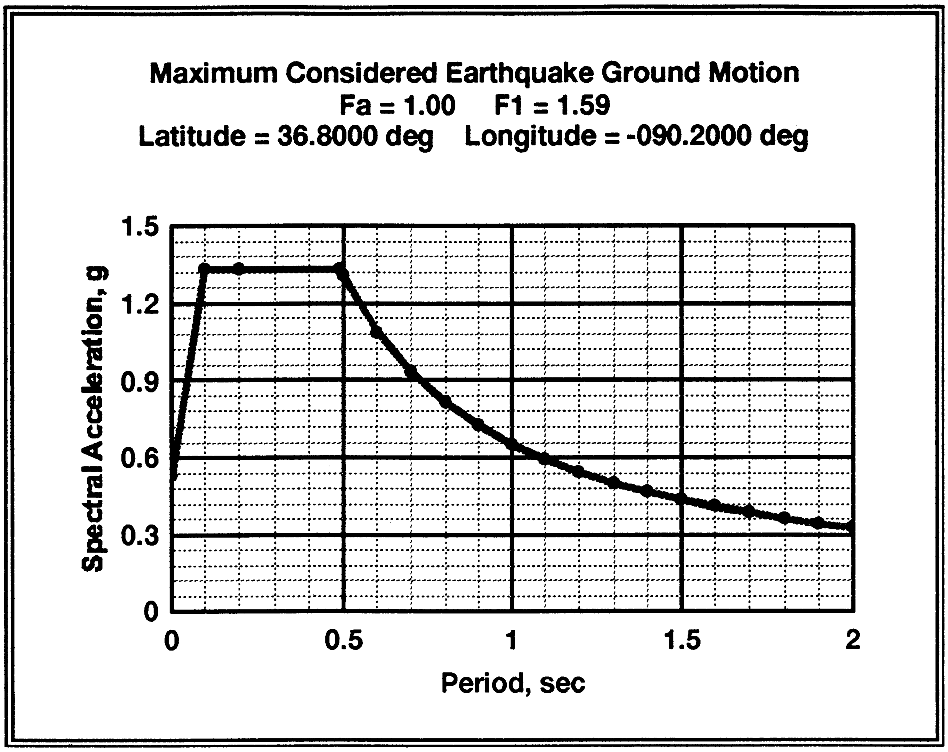
	Axial	Longitudinal	Lateral	Axial	Longitudinal	Lateral
Global	**UY**	**UX**	**UZ**	**RY**	**RX**	**RZ**
Pier	K_{11} k/ft	K_{22} k/ft	K_{33} k/ft	K_{44} k-ft/rad	K_{55} k-ft/rad	K_{66} k-ft/rad
1	9.01E+04	4.19E+03	4.19E+03	5.67E+06	1.22E+08	0.00E+00
2	2.08E+05	4.00E+03	4.00E+03	1.57E+06	5.62E+07	2.55E+07
3	2.08E+05	4.00E+03	4.00E+03	1.57E+06	5.62E+07	2.55E+07
4	9.01E+04	4.19E+03	4.19E+03	5.67E+06	1.22E+08	0.00E+00

I.5 SAP2000 Model Development: Response Spectra

2500-YR EQ

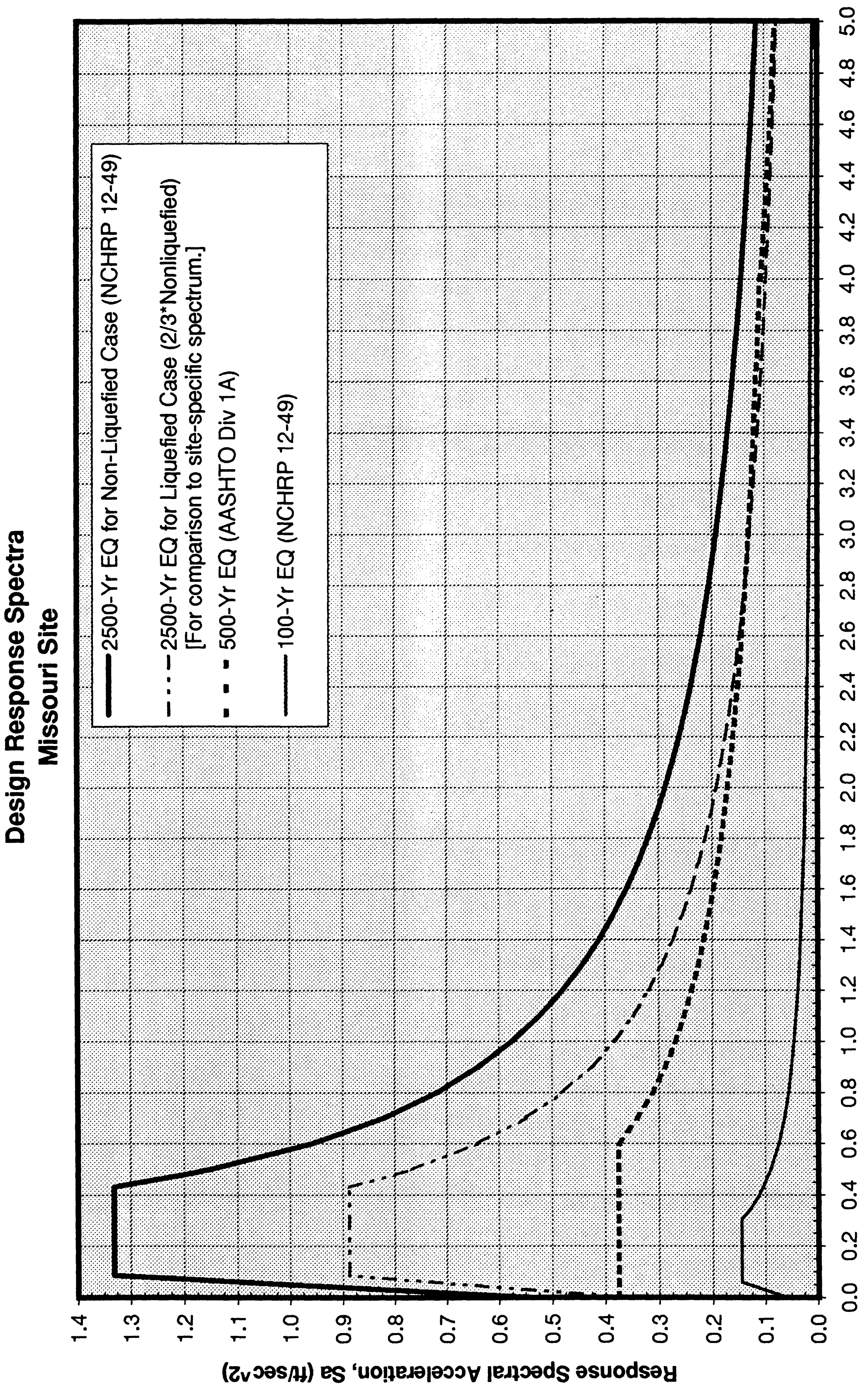
```
  MCE Parameters - Conterminous 48 States
  Latitude = 36.8000,  Longitude = -090.2000
Data are based on the 0.10 deg grid set
  Period       SA
   (sec)       (%g)
    0.2        133.1    Map Value, Soil Factor of 1.0
    1.0        041.1    Map Value, Soil Factor of 1.0
   MCE Parameters x Specified Soil Factors
    0.2        133.1    Soil Factor of 1.00
    1.0        065.3    Soil Factor of 1.59

  MCE Parameters - Conterminous 48 States
  Latitude = 36.8000,  Longitude = -090.2000
Data are based on the 0.10 deg grid set
  Period       SA
   (sec)       (%g)
    0.2        133.1    Map Value, Soil Factor of 1.0
    1.0        041.1    Map Value, Soil Factor of 1.0
   MCE SPECTRUM x SOIL FACTORS
    Fa = 1.00
    Fv = 1.59
    Period        SA
    (sec)         (%g)
    0.000         053.2    0.4FaSs
    0.098         133.1    To
    0.200         133.1    T=0.2, FaSs
    0.490         133.1    Ts
    0.500         130.6
    0.600         108.8
    0.700         093.3
    0.800         081.6
    0.900         072.5
    1.000         065.3    T=1.0, FvS1
    1.100         059.4
    1.200         054.4
    1.300         050.2
    1.400         046.6
    1.500         043.5
    1.600         040.8
    1.700         038.4
    1.800         036.3
    1.900         034.4
    2.000         032.6
```



'eriod, se	Sa, g
0.00	0.532
0.10	1.331
0.20	1.331
0.49	1.331
0.50	1.306
0.60	1.088
0.70	0.933
0.80	0.816
0.90	0.725
1.00	0.653
1.10	0.594
1.20	0.544
1.30	0.502
1.40	0.466
1.50	0.435
1.60	0.408
1.70	0.384
1.80	0.363
1.90	0.344
2.00	0.326

2500-YR EQ


Design Response Spectra
Missouri Site
2500-Yr EQ for Non-Liquefied Case (NCHRP 12-49)
2500-Yr EQ for Liquefied Case (2/3*Nonliquefied)
[For comparison to site-specific spectrum.]
500-Yr EQ (AASHTO Div 1A)
100-Yr EQ (NCHRP 12-49)
Response Spectral Acceleration, Sa (ft/sec^2)
1.4
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
Period, T (sec)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
4.4
4.6
4.8
5.0

NCHRP 12-49 SOIL LIQUEFACTION STUDY
MISSOURI SITE - 2500Yr EQ, Non-Liquefied & Liquefied Case (NCHRP)

DESIGN RESPONSE SPECTRUM
Using Two-Point Construction Method

	Short Periods	1-Sec Periods
Site Coefficients:	F_a = 1.0	F_v = 1.4
Spectral Acceleration on Class B Rock:	S_s = 1.331	S_1 = 0.411
Spectral Response Acceleration:	**$S_{DS} = F_aS_s$ = 1.331**	**$S_{D1} = F_vS_1$ = 0.575**

Key Points on Response Spectrum:
$T_s = S_{D1}/S_{DS}$ = 0.432
$T_0 = 0.2T_s$ = 0.086
Y-Intercept = $0.4S_{DS}$ = 0.532

Non-Liquefied

T sec	Sa ft/sec^2
0.000	0.532
0.086	1.331
0.432	1.331
0.480	1.199
0.500	1.151
0.60	0.959
0.70	0.822
0.80	0.719
0.90	0.639
1.00	0.575
1.10	0.523
1.20	0.480
1.30	0.443
1.40	0.411
1.50	0.384
1.60	0.360
1.70	0.338
1.80	0.320
1.90	0.303
2.00	0.288
2.10	0.274
2.20	0.262
2.30	0.250
2.40	0.240
2.50	0.230
2.60	0.221
2.70	0.213
2.80	0.206
2.90	0.198
3.00	0.192
3.10	0.186
3.20	0.180
3.30	0.174
3.40	0.169
3.50	0.164
3.60	0.160
3.70	0.156
3.80	0.151
3.90	0.148
4.00	0.144
4.10	0.140
4.20	0.137
4.30	0.134
4.40	0.131
4.50	0.128
4.60	0.125
4.70	0.122
4.80	0.120
4.90	0.117
5.00	0.115
5.10	0.113
5.20	0.111
5.30	0.109

Liquefied

T sec	2/3*Sa ft/sec^2
0.000	0.355
0.086	0.887
0.432	0.887
0.480	0.799
0.500	0.767
0.600	0.639
0.700	0.548
0.800	0.480
0.900	0.426
1.000	0.384
1.100	0.349
1.200	0.320
1.300	0.295
1.400	0.274
1.500	0.256
1.600	0.240
1.700	0.226
1.800	0.213
1.900	0.202
2.000	0.192
2.100	0.183
2.200	0.174
2.300	0.167
2.400	0.160
2.500	0.153
2.600	0.148
2.700	0.142
2.800	0.137
2.900	0.132
3.000	0.128
3.100	0.124
3.200	0.120
3.300	0.116
3.400	0.113
3.500	0.110
3.600	0.107
3.700	0.104
3.800	0.101
3.900	0.098
4.000	0.096
4.100	0.094
4.200	0.091
4.300	0.089
4.400	0.087
4.500	0.085
4.600	0.083
4.700	0.082
4.800	0.080
4.900	0.078
5.000	0.077
5.100	0.075
5.200	0.074
5.300	0.072

NCHRP 12-49 SOIL LIQUEFACTION STUDY
MISSOURI SITE - 2500Yr EQ, Non-Liquefied & Liquefied Case (NCHRP)

DESIGN RESPONSE SPECTRUM
Using Two-Point Construction Method

	Short Periods	1-Sec Periods
Site Coefficients:	F_a = 1.0	F_v = 1.4
Spectral Acceleration on Class B Rock:	S_s = 1.331	S_1 = 0.411
Spectral Response Acceleration:	**$S_{DS} = F_aS_s =$ 1.331**	**$S_{D1} = F_vS_1 =$ 0.575**

Key Points on Response Spectrum:

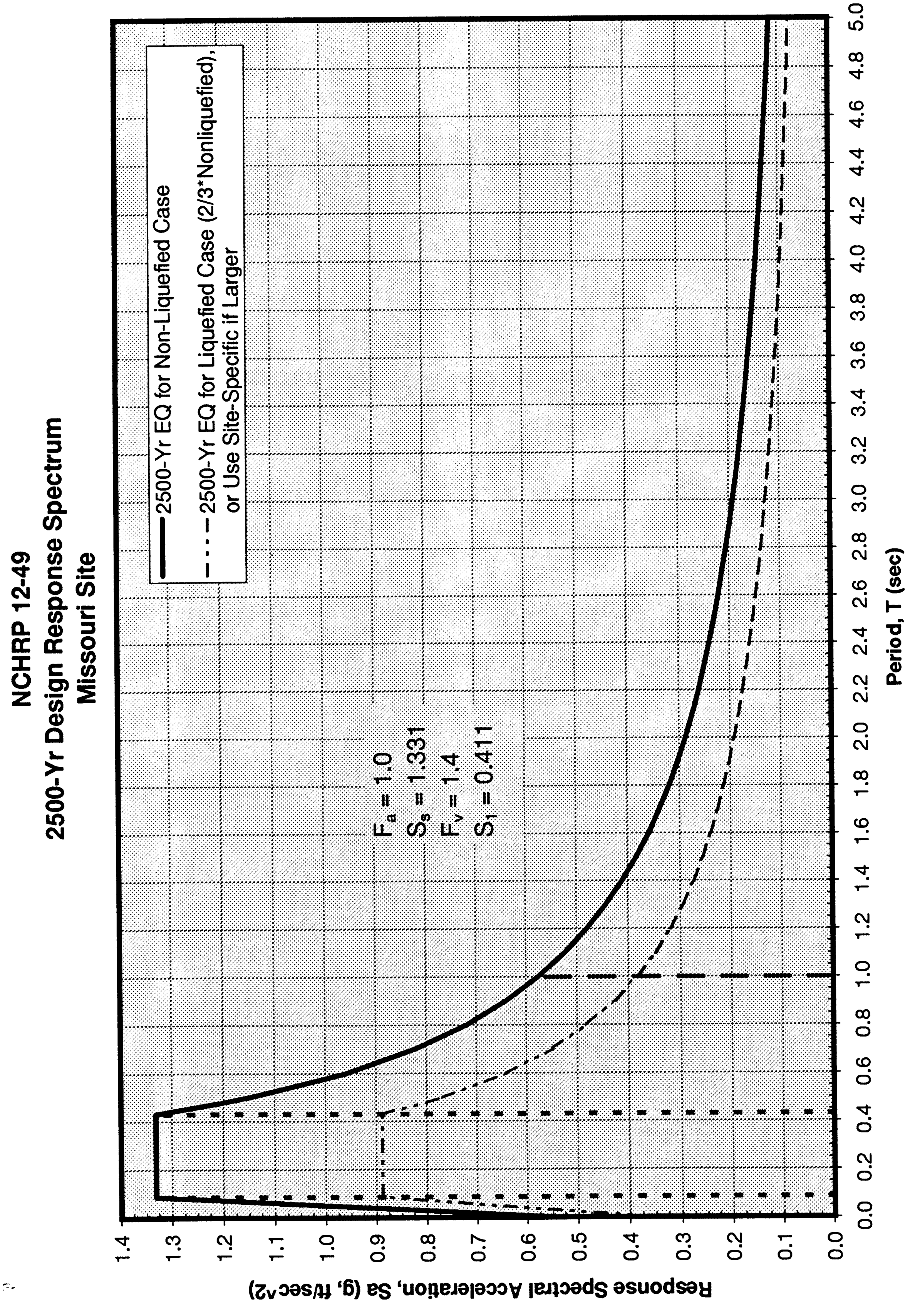
$T_s = S_{D1}/S_{DS} =$ 0.432
$T_0 = 0.2T_s =$ 0.086
Y-Intercept = $0.4S_{DS} =$ 0.532

Non-Liquefied

T sec	Sa ft/sec^2
5.40	0.107
5.50	0.105
5.60	0.103
5.70	0.101
5.80	0.099
5.90	0.098
6.00	0.096
6.10	0.094
6.20	0.093
6.30	0.091
6.40	0.090
6.50	0.089
6.60	0.087
6.70	0.086
6.80	0.085
6.90	0.083
7.00	0.082
7.10	0.081
7.20	0.080
7.30	0.079
7.40	0.078
7.50	0.077
7.60	0.076
7.70	0.075
7.80	0.074
7.90	0.073
8.00	0.072
8.10	0.071
8.20	0.070
8.30	0.069
8.40	0.069
10.00	0.058

Liquefied

T sec	2/3*Sa ft/sec^2
5.400	0.071
5.500	0.070
5.600	0.069
5.700	0.067
5.800	0.066
5.900	0.065
6.000	0.064
6.100	0.063
6.200	0.062
6.300	0.061
6.400	0.060
6.500	0.059
6.600	0.058
6.700	0.057
6.800	0.056
6.900	0.056
7.000	0.055
7.100	0.054
7.200	0.053
7.300	0.053
7.400	0.052
7.500	0.051
7.600	0.050
7.700	0.050
7.800	0.049
7.900	0.049
8.000	0.048
8.100	0.047
8.200	0.047
8.300	0.046
8.400	0.046
10.000	0.038



NCHRP 12-49
2500-Yr Design Response Spectrum
Missouri Site

NCHRP 12-49 LIQUEFACTION STUDY
MISSOURI SITE - 500Yr (Div 1A)

ELASTIC SEISMIC RESPONSE FOR MULTIMODE ANALYSIS

$$C_{sm} = \frac{1.2 \cdot A \cdot S}{T_m^{\frac{2}{3}}} \le 2.5 \cdot A$$

(AASHTO DIV. 1A Eqn. 3-2)

Design parameters : A (g) = 0.15 g
(Type III Soil) S = 1.5

Basic Spectrum

T_m	C_{sm}
0.00	0.375
0.05	0.375
0.10	0.375
0.15	0.375
0.20	0.375
0.25	0.375
0.30	0.375
0.35	0.375
0.40	0.375
0.45	0.375
0.50	0.375
0.55	0.375
0.60	0.375
0.65	0.360
0.70	0.342
0.75	0.327
0.80	0.313
0.85	0.301
0.90	0.290
0.95	0.279
1.00	0.270
1.05	0.261
1.10	0.253
1.15	0.246
1.20	0.239
1.25	0.233
1.30	0.227
1.35	0.221
1.40	0.216
1.45	0.211
1.50	0.206
1.55	0.202
1.60	0.197
1.65	0.193
1.70	0.190
1.75	0.186
1.80	0.182
1.85	0.179
1.90	0.176
1.95	0.173
2.00	0.170
2.05	0.167
2.10	0.165
2.15	0.162
2.20	0.160
2.25	0.157
2.30	0.155
2.35	0.153
2.40	0.151
2.45	0.149
2.50	0.147
2.55	0.145
2.60	0.143
2.65	0.141
2.70	0.139
2.75	0.138
2.80	0.136
2.85	0.134
2.90	0.133

NCHRP 12-49 LIQUEFACTION STUDY
MISSOURI SITE - 500Yr (Div 1A)

ELASTIC SEISMIC RESPONSE FOR MULTIMODE ANALYSIS

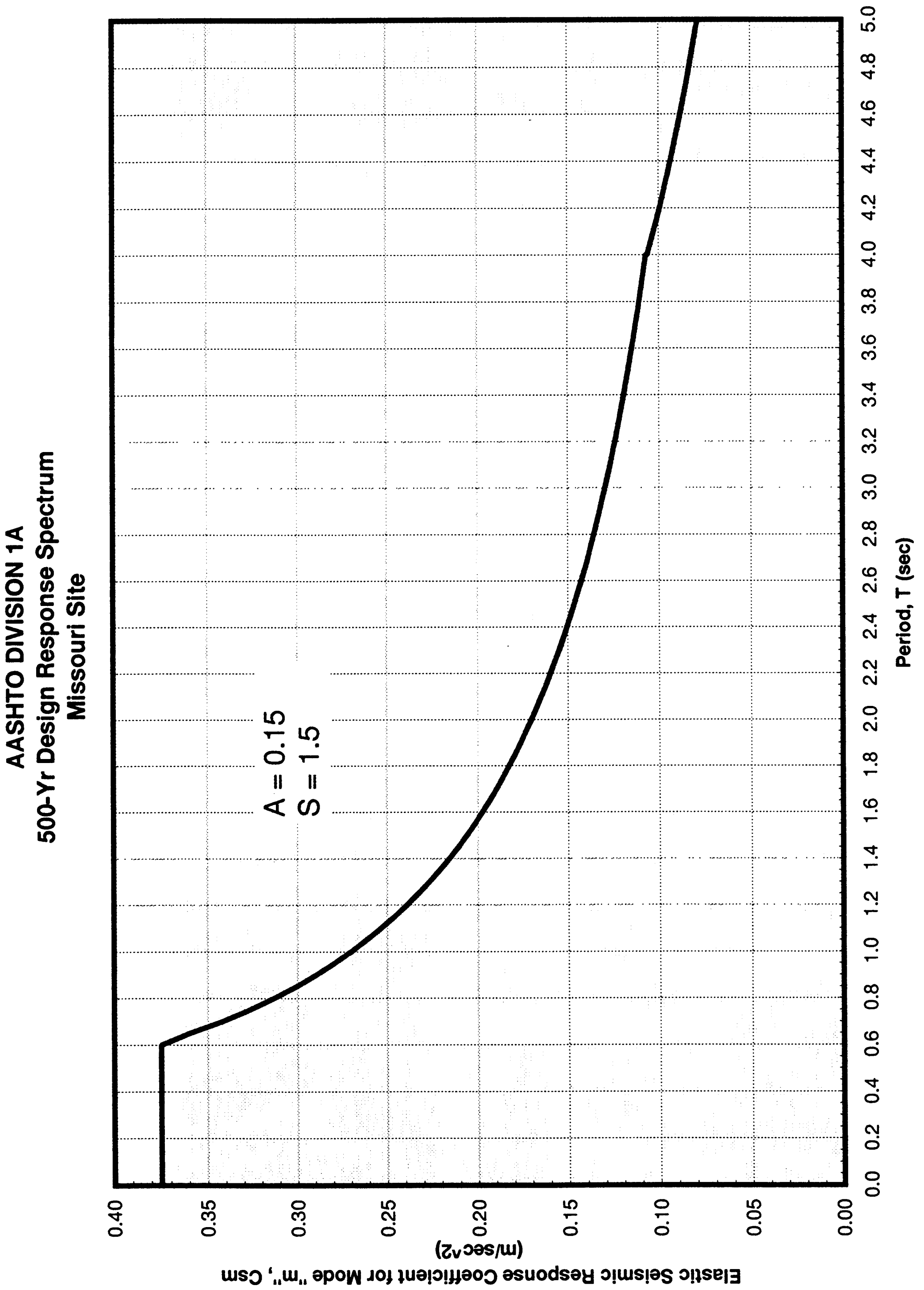
$$C_{sm} = \frac{1.2 \cdot A \cdot S}{T_m^{\frac{2}{3}}} \le 2.5 \cdot A$$

(AASHTO DIV. 1A Eqn. 3-2)

Design parameters : A (g) = 0.15 g
(Type III Soil) S = 1.5

Basic Spectrum

T_m	C_{sm}
2.95	0.131
3.00	0.130
3.05	0.128
3.10	0.127
3.15	0.126
3.20	0.124
3.25	0.123
3.30	0.122
3.35	0.121
3.40	0.119
3.45	0.118
3.50	0.117
3.55	0.116
3.60	0.115
3.65	0.114
3.70	0.113
3.75	0.112
3.80	0.111
3.85	0.110
3.90	0.109
3.95	0.108
4.00	0.107
4.00	0.106
4.05	0.105
4.10	0.103
4.15	0.101
4.20	0.100
4.25	0.098
4.30	0.097
4.35	0.095
4.40	0.094
4.45	0.092
4.50	0.091
4.55	0.090
4.60	0.088
4.65	0.087
4.70	0.086
4.75	0.085
4.80	0.083
4.85	0.082
4.90	0.081
4.95	0.080
5.00	0.079


AASHTO DIVISION 1A
500-Yr Design Response Spectrum
Missouri Site
A = 0.15
S = 1.5
Elastic Seismic Response Coefficient for Mode "m", Csm
(m/sec^2)
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0.00
Period, T (sec)
0.0
0.2
0.4
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
4.2
4.4
4.6
4.8
5.0

NCHRP 12-49 SOIL LIQUEFACTION STUDY
MISSOURI SITE - 100Yr EQ (NCHRP)

DESIGN RESPONSE SPECTRUM

Using Two-Point Construction Method

	Short Periods		1-Sec Periods	
Site Coefficients:	F_a =	1.6	F_v =	2.4
Spectral Acceleration on Class B Rock:	S_s =	0.091	S_1 =	0.0185
Spectral Response Acceleration:	$S_{DS} = F_aS_s$ =	**0.146**	$S_{D1} = F_vS_1$ =	**0.044**

Key Points on Response Spectrum:

$T_s = S_{D1}/S_{DS}$ = 0.305
$T_0 = 0.2T_s$ = 0.061
Y-Intercept = $0.4S_{DS}$ = 0.058

T sec	Sa ft/sec^2
0.00	0.058
0.061	0.146
0.305	0.146
0.350	0.127
0.40	0.111
0.50	0.089
0.60	0.074
0.70	0.063
0.80	0.056
0.90	0.049
1.00	0.044
1.10	0.040
1.20	0.037
1.30	0.034
1.40	0.032
1.50	0.030
1.60	0.028
1.70	0.026
1.80	0.025
1.90	0.023
2.00	0.022
2.10	0.021
2.20	0.020
2.30	0.019
2.40	0.019
2.50	0.018
2.60	0.017
2.70	0.016
2.80	0.016
2.90	0.015
3.00	0.015
3.10	0.014
3.20	0.014
3.30	0.013
3.40	0.013
3.50	0.013
3.60	0.012
3.70	0.012
3.80	0.012
3.90	0.011
4.00	0.011
4.10	0.011
4.20	0.011
4.30	0.010
4.40	0.010
4.50	0.010
4.60	0.010
4.70	0.009
4.80	0.009
4.90	0.009
5.00	0.009
5.10	0.009
5.20	0.009

NCHRP 12-49 SOIL LIQUEFACTION STUDY
MISSOURI SITE - 100Yr EQ (NCHRP)

DESIGN RESPONSE SPECTRUM
Using Two-Point Construction Method

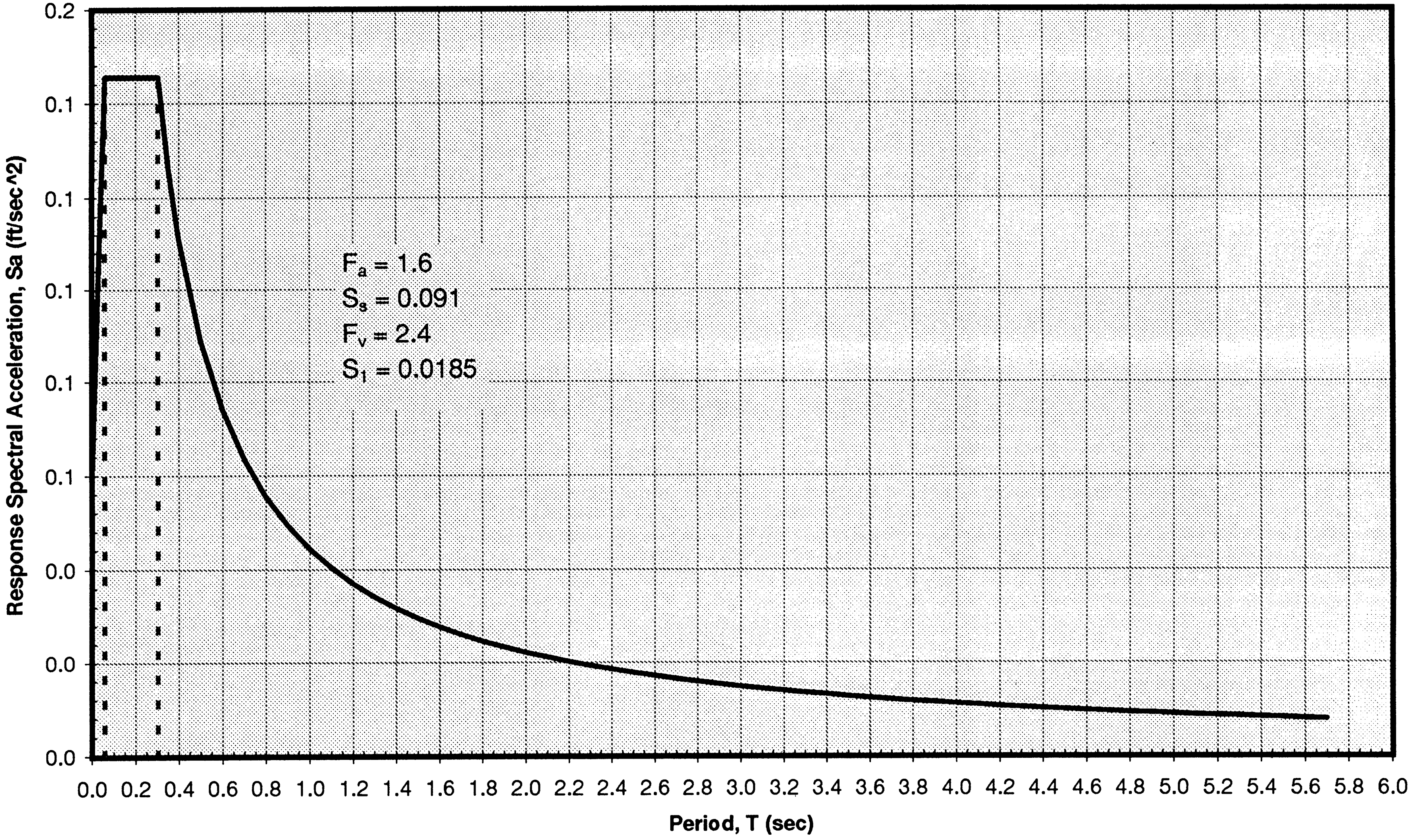
	Short Periods		1-Sec Periods	
Site Coefficients:	$F_a =$	1.6	$F_v =$	2.4
Spectral Acceleration on Class B Rock:	$S_s =$	0.091	$S_1 =$	0.0185
Spectral Response Acceleration:	$S_{DS} = F_a S_s =$	**0.146**	$S_{D1} = F_v S_1 =$	**0.044**

Key Points on Response Spectrum:

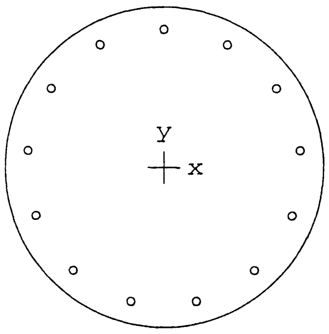
$T_s = S_{D1}/S_{DS} =$ 0.305
$T_0 = 0.2T_s =$ 0.061
Y-Intercept $= 0.4S_{DS} =$ 0.058

T sec	Sa ft/sec^2
5.30	0.008
5.40	0.008
5.50	0.008
5.60	0.008
5.70	0.008
5.80	0.008
5.90	0.008
6.00	0.007
6.10	0.007
6.20	0.007
6.30	0.007
6.40	0.007
6.50	0.007
6.60	0.007
6.70	0.007
6.80	0.007
6.90	0.006
7.00	0.006
7.10	0.006
7.20	0.006
7.30	0.006
7.40	0.006
7.50	0.006
7.60	0.006
7.70	0.006
7.80	0.006
7.90	0.006
8.00	0.006
8.10	0.005
8.20	0.005
8.30	0.005
10.00	0.004

NCHRP 12-49
100-Yr Design Response Spectrum
Missouri Site



I.6 Column Design



36 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

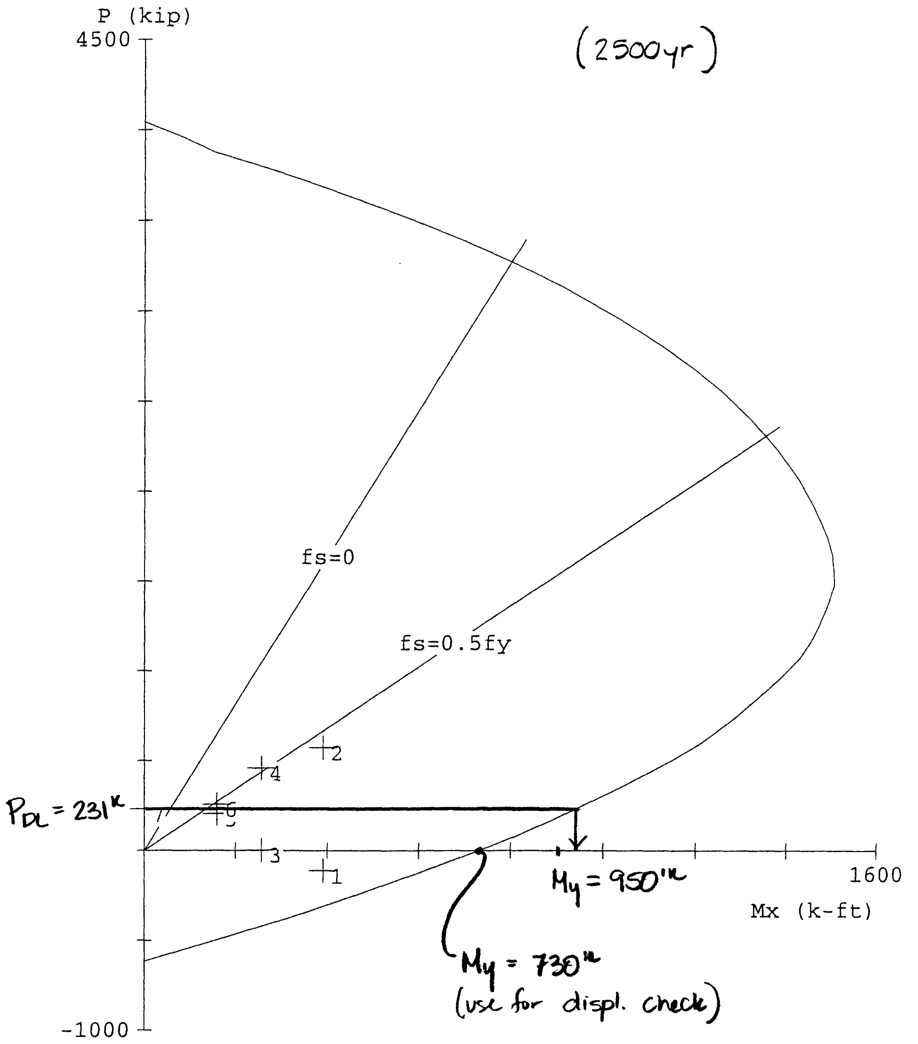
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 13:22:28



(2500yr)

$P_{DL} = 231^k$

$M_y = 950'^k$

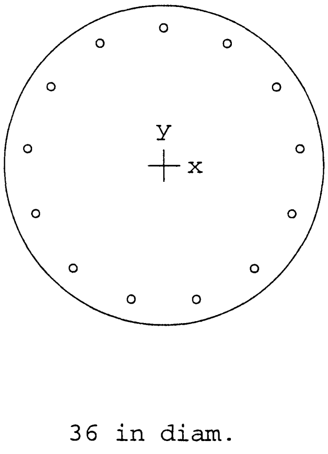
$M_y = 730'^k$
(use for displ. check)

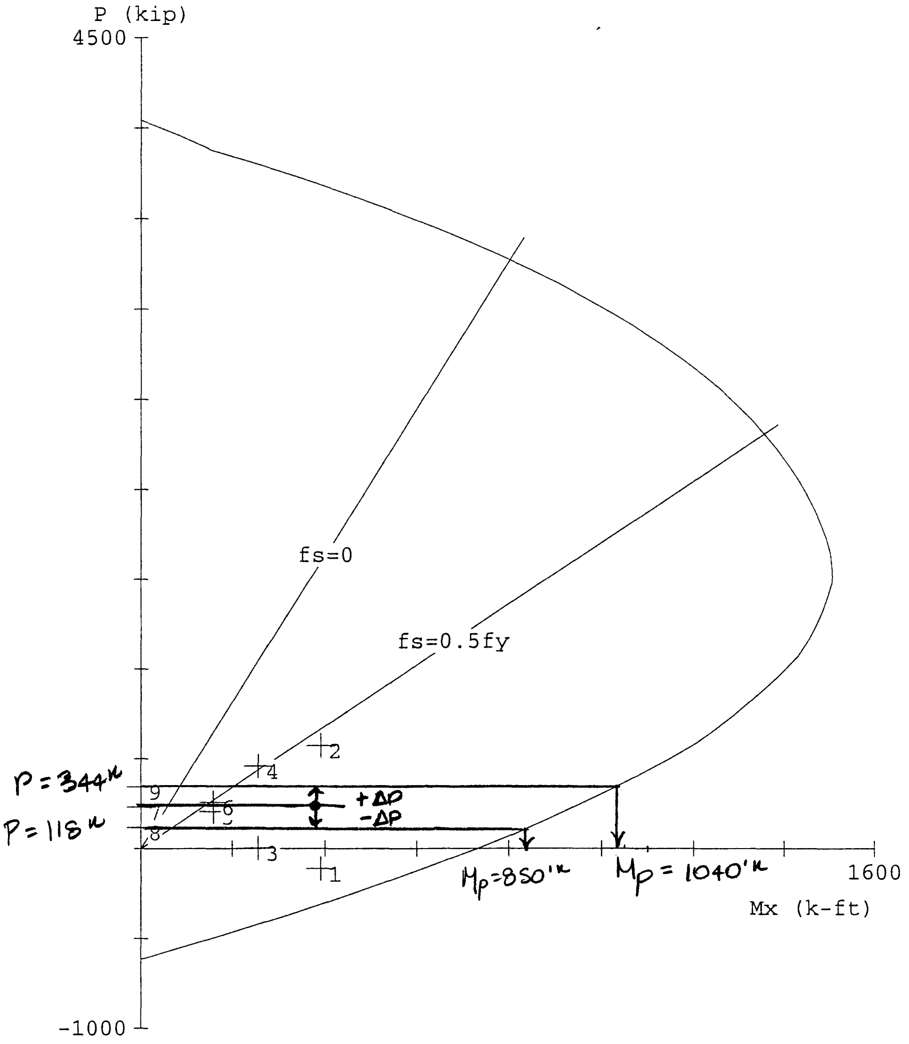
PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\MO\DESIGN\COLUMN.COL

Project: NCHRP 12-49 Missouri Bridge

Column: Piers 2 & 3		Engineer: MLT	
f'c = 4 ksi	fy = 60 ksi	Ag = 1017.88 in^2	13 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 10.27 in^2	Rho = 1.01%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 82448 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 82448 in^4
Beta1 = 0.85		Clear spacing = 6.42 in	Clear cover = 2.00 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		





Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 13:15:18

PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\MO\DESIGN\COLUMN.COL

Project: NCHRP 12-49 Missouri Bridge

Column: Piers 2 & 3		Engineer: MLT	
f'c = 4 ksi	fy = 60 ksi	Ag = 1017.88 in^2	13 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 10.27 in^2	Rho = 1.01%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 82448 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 82448 in^4
Beta1 = 0.85		Clear spacing = 6.42 in	Clear cover = 2.00 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		

NCHRP 12-49 LIQUEFACTION STUDY

Missouri Bridge - Piers 2 & 3 (2500yr)

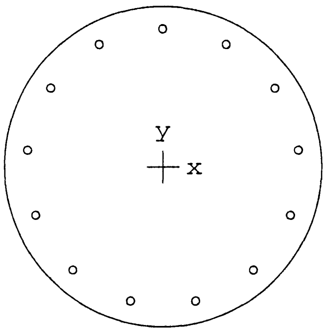
h = 29.41 ft V = 1.5*Mntop + 1.5*Mnbot / H Eq. (1)
H = 26.00 ft ΔP = [S(V)*h - S(1.5*Mnbot)] / (a*10/9) Eq. (2)
a = 28.66 ft Adjusted P = P +/- DP Eq. (3)

Step	Column Moments (kips-ft)							Column Shears (kips)				ΔP	Column Axial Forces			
	M	Left		Center		Right		Left	Center	Right	Total		P (kips)			
		Top	Bottom	Top	Bottom	Top	Bottom					(kips)	Left	Left Center	Right Center	Right
Given													231			231
1a	Mn	950	950			950	950									
1b	1.5*Mn	1,425	1,425	0	0	1,425	1,425									
2								110	0	110	**219**					
3a												113				
3b													118			344
4a	Mn	850	850			1,040	1,040									
4b	1.5*Mn	1,275	1,275	0	0	1,560	1,560									
4c								98	0	120	**218**					
4d												112				
4e													119			343
4f		**Difference in Total Column Plastic** (from Steps 2 and 4c)				**0.5% <10%**		**OK**								

The forces in the individual columns in the plane of the bent associated with plastic hinging of the bent columns are

<u>Minimum Axial Load:</u>
a) Axial Load P min_p = 119 kips
b) Moment $Mmin_p = 1.5*Mmin_n$ = 1275 ft-kips
c) Shear V min_p = 98 kips

<u>Maximum Axial Load :</u>
a) Axial Load P max_p = 343 kips
b) Moment $Mmax_p = 1.5*Mmax_n$ = 1560 ft-kips
c) Shear V max_p = 120 kips



36 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

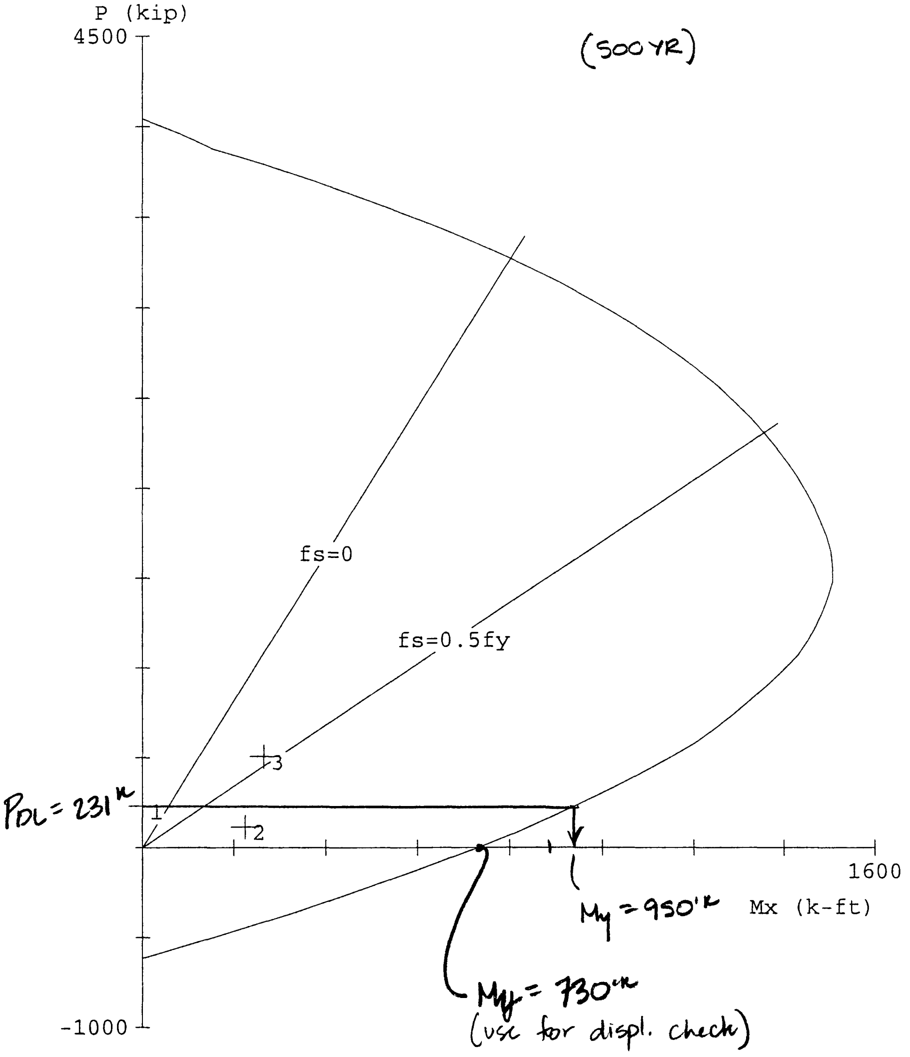
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 14:08:52



PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\MO\DESIGN\COL-500.COL

Project: NCHRP 12-49 Missouri Bridge

Column: Piers 2 & 3

Engineer: MLT

f'c = 4 ksi

fy = 60 ksi

Ag = 1017.88 in^2

13 #8 bars

Ec = 3605 ksi

Es = 29000 ksi

As = 10.27 in^2

Rho = 1.01%

fc = 3.4 ksi

e_rup = Infinity

Xo = 0.00 in

Ix = 82448 in^4

e_u = 0.003 in/in

Yo = 0.00 in

Iy = 82448 in^4

Beta1 = 0.85

Clear spacing = 6.42 in

Clear cover = 2.00 in

Confinement: Other

phi(a) = 1, phi(b) = 1, phi(c) = 1

COLUMN DESIGN - Modify PCACOL loads to introduce ϕ-factors:

(Use ϕ=1 in PCACOL load combinations.)

$\mathrm{ksi} := 1000 \cdot \frac{\mathrm{lb}}{\mathrm{in}^2}$

$\mathrm{kips} := 1000 \cdot \mathrm{lb}$

PIERS 2 & 3

Input:

$\mathrm{Pmin}_u := 97 \cdot \mathrm{kips}$

$M_u := 192 \cdot \mathrm{ft} \cdot \mathrm{kips}$

$\mathrm{Pmax}_u := 365 \cdot \mathrm{kips}$

ϕ Factors

Concrete strength: $f'_c := 4 \cdot \mathrm{ksi}$ $0.2 \cdot f'_c = 0.800 \cdot \mathrm{ksi}$

Column radius: $r := \frac{36 \cdot \mathrm{in}}{2}$ $r = 18 \cdot \mathrm{in}$

$A_g := \pi \cdot r^2$ $A_g = 7.069\ \mathrm{ft}^2$

$\sigma_{min} := \frac{\mathrm{Pmin}_u}{A_g}$ $\sigma_{min} = 0.095 \cdot \mathrm{ksi}$

$\phi_{min} := 0.9 - \frac{\sigma_{min}}{0.2 \cdot f'_c} \cdot (0.9 - 0.5)$ $\phi_{min} = 0.852$

$\sigma_{max} := \frac{\mathrm{Pmax}_u}{A_g}$ $\sigma_{max} = 0.359 \cdot \mathrm{ksi}$

$\phi_{max} := 0.9 - \frac{\sigma_{max}}{0.2 \cdot f'_c} \cdot (0.9 - 0.5)$ $\phi_{max} = 0.721$

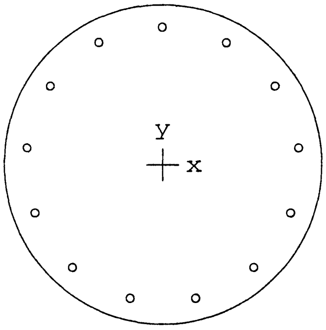
Axial Load and Moment

$\mathrm{Pmin}_n := \frac{\mathrm{Pmin}_u}{\phi_{min}}$ $\mathrm{Pmin}_n = 114 \cdot \mathrm{kips}$

$\mathrm{Mmin}_n := \frac{M_u}{\phi_{min}}$ $\mathrm{Mmin}_n = 225.3 \cdot \mathrm{ft} \cdot \mathrm{kips}$

$\mathrm{Pmax}_n := \frac{\mathrm{Pmax}_u}{\phi_{max}}$ $\mathrm{Pmax}_n = 506 \cdot \mathrm{kips}$

$\mathrm{Mmax}_n := \frac{M_u}{\phi_{max}}$ $\mathrm{Mmax}_n = 266.4 \cdot \mathrm{ft} \cdot \mathrm{kips}$



36 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

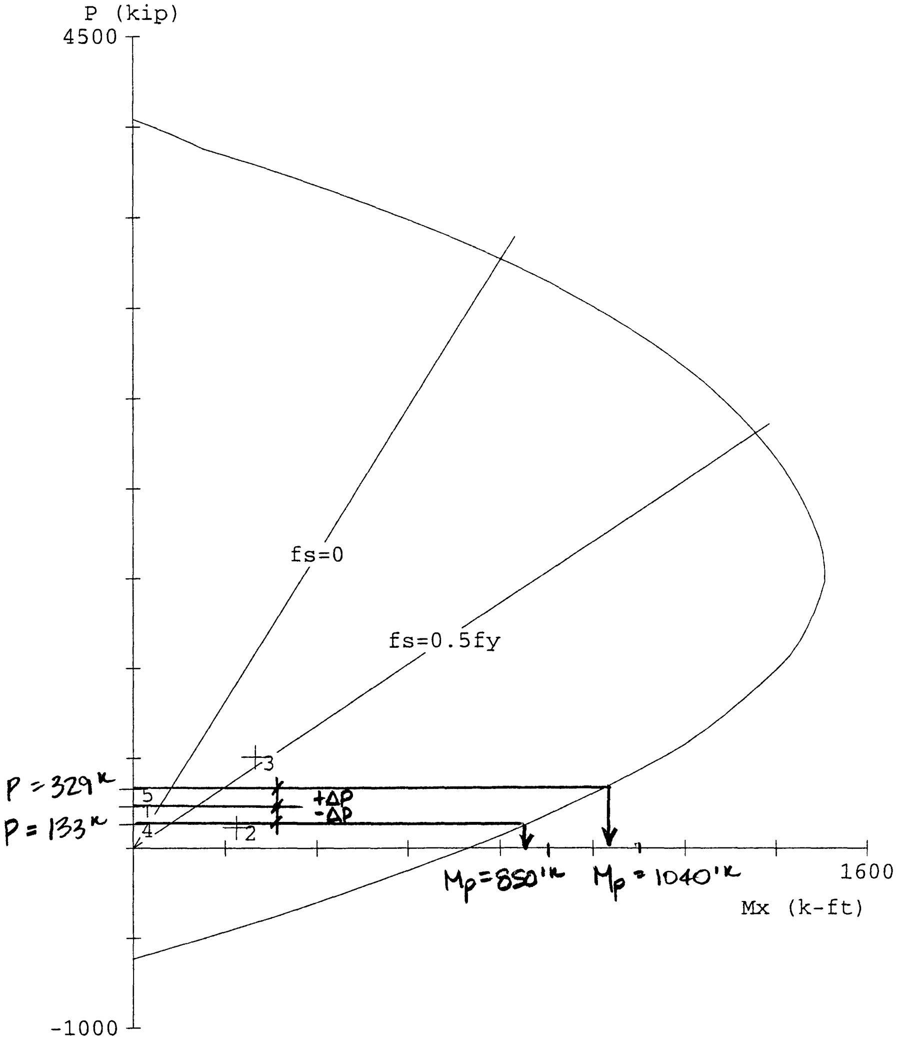
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/17/00

Time: 14:15:27



PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\MO\DESIGN\COL-500H.COL

Project: NCHRP 12-49 Missouri Bridge

Column: Piers 2 & 3		Engineer: MLT	
f'c = 4 ksi	fy = 60 ksi	Ag = 1017.88 in^2	13 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 10.27 in^2	Rho = 1.01%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 82448 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 82448 in^4
Beta1 = 0.85		Clear spacing = 6.42 in	Clear cover = 2.00 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		

NCHRP 12-49 LIQUEFACTION STUDY

Missouri Bridge - Piers 2 & 3 (500yr)

h = 29.41 ft $V = 1.3*Mntop + 1.3*Mnbot / H$ Eq. (1)

H = 26.00 ft $\Delta P = [\ S(\ V\ )*h - S(\ 1.3*Mnbot\ )]\ /\ (a*10/9)$ Eq. (2)

a = 28.66 ft Adjusted $P = P +/- \Delta P$ Eq. (3)

Step	Column Moments (kips-ft)							Column Shears (kips)					Column Axial Forces			
	M	Left		Center		Right		Left	Center	Right	Total	ΔP	P (kips)			
		Top	Bottom	Top	Bottom	Top	Bottom					(kips)	Left	Left Center	Right Center	Right
Given													231			231
1a	Mn	950	950			950	950									
1b	1.3*Mn	1,235	1,235	0	0	1,235	1,235									
2								95	0	95	**190**					
3a												98				
3b													133			329
4a	Mn	850	850			1,040	1,040									
4b	1.3*Mn	1,105	1,105	0	0	1,352	1,352									
4c								85	0	104	**189**					
4d												97				
4e													134			328
4f		**Difference in Total Column Plastic** (from Steps 2 and 4c)				**0.5% <10%**		**OK**								

The forces in the individual columns in the plane of the bent associated with plastic hinging of the bent columns are

<u>Minimum Axial Load:</u>

a) Axial Load $P\ min_p =$ 134 kips

b) Moment $Mminp = 1.3*Mminn =$ 1105 ft-kips

c) Shear $V\ min_p =$ 85 kips

<u>Maximum Axial Load :</u>

a) Axial Load $P\ max_p =$ 328 kips

b) Moment $Mmaxp = 1.3*Mmaxn =$ 1352 ft-kips

c) Shear $V\ max_p =$ 104 kips

I.7 Abutment Foundation Check

MISSOURI BRIDGE (DESIGN FOR LIFE SAFETY)

$$\left.\begin{array}{l} T_L = 0.41 \\ T_T = 0.62 \end{array}\right\} \text{2500 Non-Liquefied}$$

∴ Governed by $F_a S_s$

$$F_a S_s = (1.0)(1.331) = 1.331$$

⇒ SEISMIC HAZARD LEVEL IV (TABLE 3.10.3-1b)

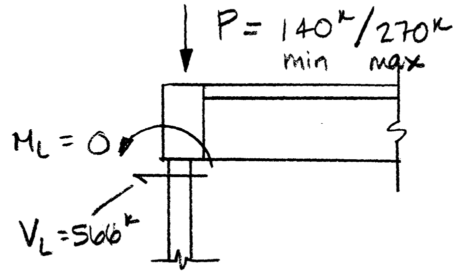
⇒ SDAP C/D/E (TABLE 3.10.3.2) ?

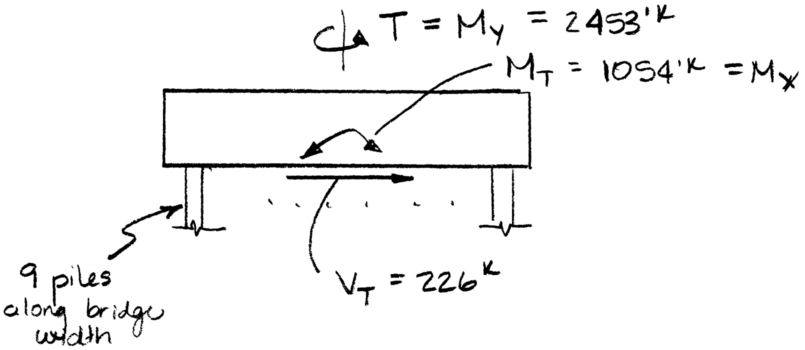
⇒ SDR 5 (TABLE 3.10.3-3)

SDR 5 $\Rightarrow$ $F_{OS} = 1.5$ CONC) NOT @ ABUTMENTS
$= 1.4$ STEEL)

$\Rightarrow$ SECTION 3.10.3.8 APPLIES (CAPACITY DESIGN)

$\Rightarrow$ NOTES 2, 3, 4 APPLY (p. 3-31)





CONSIDER 2 EXTERIOR PILES, WHICH GET MAX P DUE TO M_T. SHEAR ASSUMED DISTRIBUTED EQUALLY.

$$V_T = \frac{226^k}{9} = 25.1^k/\text{pile}$$

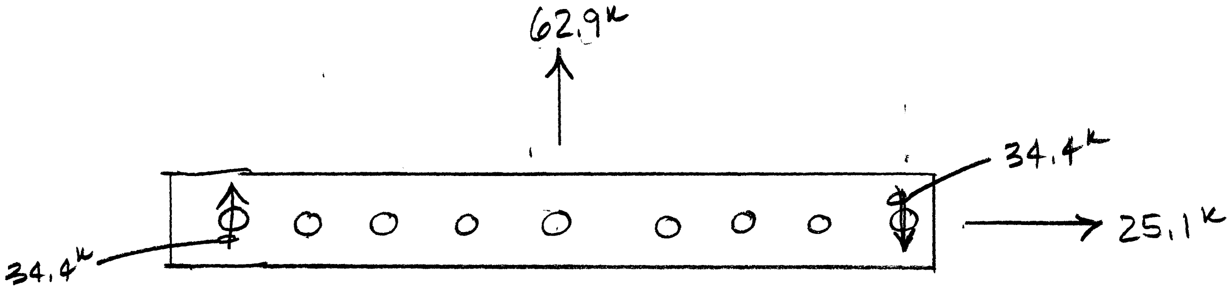
$$V_L = \frac{566^k}{9} = 62.9^k/\text{pile}$$

DETERMINE SHEAR IN PILES DUE TO M_Y (FROM TRANSVERSE EQ): $K_{RY} = 4.51 \times 10^5$ 'k/rad

$$\theta = \frac{M_Y}{k_{RY}} = \frac{2453'^k}{4.51 \times 10^5} \Rightarrow \underline{\underline{\theta = 0.00544 \text{ rad}}}$$

OUTER PILES } $\Delta_{①} = -\Delta_{⑨} = \theta \ell = (0.00544)\left(\frac{38'}{2}\right) \Rightarrow \underline{\underline{\Delta_{①} = 0.1033 \text{ ft}}}$

$$V_{LT} = \Delta_{①} k_{①} = (0.1033')(333\ ^k/_{ft}) \Rightarrow \underline{\underline{V_{LT} = 34.4^k}}$$



$$V = \sqrt{\underbrace{(62.9^k+34.4^k)^2}_{97.3^2} + 25.1^k} \Rightarrow \boxed{V = 100.5^k}$$

APPLY TO LPILE (PINNED HEAD) ⇒ $M_{MAX} = 3570''^k$

$\underline{\underline{M_{MAX} = 297.5'^k}}$

FOR 14"Ø w/ 3/8" WALL (NEGLECT OUTER 1/16") FOR PROPERTIES

$I = 306.3\,in^4 \Rightarrow S = \frac{I}{y} = \frac{306.3\,in^4}{7''} = 43.8\,in^3$

$A = 13.31\,in^2$ ↖ or 6.9375"

$S'f_y = (43.8\,in^3)(35\,ksi) = 1531.5''^k < 3570''^k$ YIELDING

↑ 252 Gr 2

Try Gr. 3 $(43.8\,in^3)(45\,ksi) = 1971''^k$ YIELDING

$$Z = \frac{d_o^3}{6} - \frac{d_i^3}{6} = \frac{13.875''^3}{6} - \frac{13.25''^3}{6} = 57.5\,in^3$$

Gr. 2 $Zf_u = (57.5\,in^3)(60\,ksi) = 3450''k$ $\boxed{\text{OK}}$

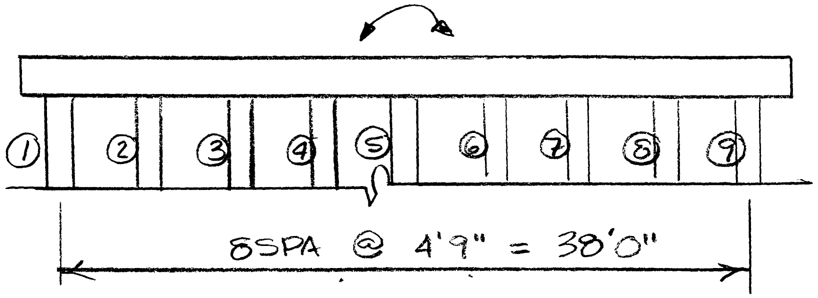
WORKS IF USE 10 #6 w/ CASING 3.2% RC

JUST BARELY

<u>CONSIDER M_T ON PILES:</u>

$$M_x = M_T = 1054'^k$$

PILES ①&⑨
CARRY HIGHEST
PORTION OF
M_x



$$k_{RX} = 1.36\times10^{7}\ \frac{k\text{-}ft}{rad}\ ,\quad \theta = \frac{M_x}{k_{RX}} = \frac{1054'^k}{1.36\times10^{7\,'k}} = 7.75\times10^{-5}\ rad$$

$$\Delta_{\textcircled{1}} = -\Delta_{\textcircled{9}} = \theta\ell = (7.75\times10^{-5} rad)\left(\frac{38'}{2}\right) \Rightarrow \Delta_{\textcircled{1}} = .00147\ ft$$

$$\therefore P_{M_x} = \underset{\nwarrow \text{1 pile}}{k_{AXIAL}}\,\Delta_{\textcircled{1}} = (10011\ k/ft)(0.00147') \Rightarrow \underline{\underline{P_{M_x} = 14.7^k}}$$

$$P_{DL} = 207^k/9 \qquad \Rightarrow \qquad \underline{\underline{P_{DL} - 23^k}}$$

$$P_{EQ} = 67^k/9 \qquad \Rightarrow \qquad \underline{\underline{P_{EQ} = 7.4^k}}$$

$$P_{MIN} = P_{DL} - P_M - P_{EQ} = 23^k - 14.7^k - 7.4^k$$

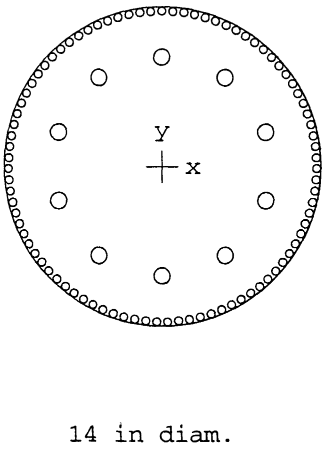
$$\Rightarrow \boxed{P_{MIN} = -0.9^k}$$

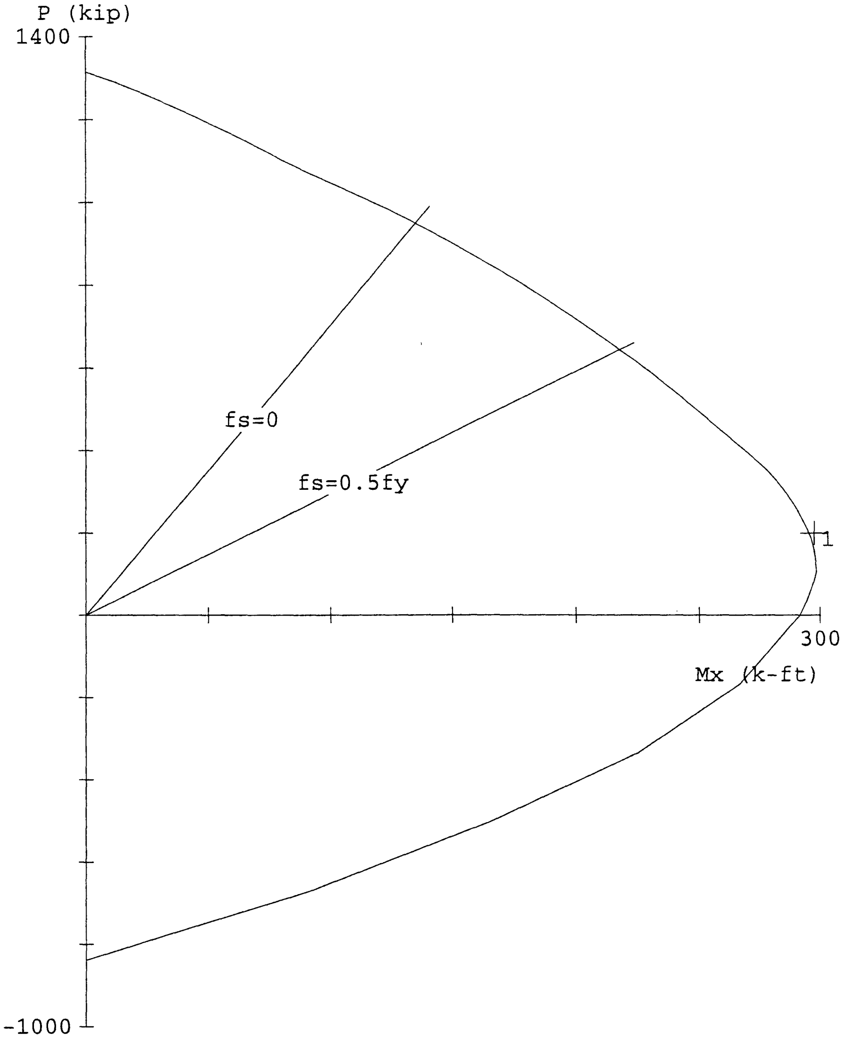
per pile

$$P_{MAX} = P_{DL} + P_M + P_{EQ} = 23^k + 14.7^k + 7.4^k$$

$$\Rightarrow \boxed{P_{MAX} = 45.1^k}$$

per pile





Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 10/11/00

Time: 16:36:24

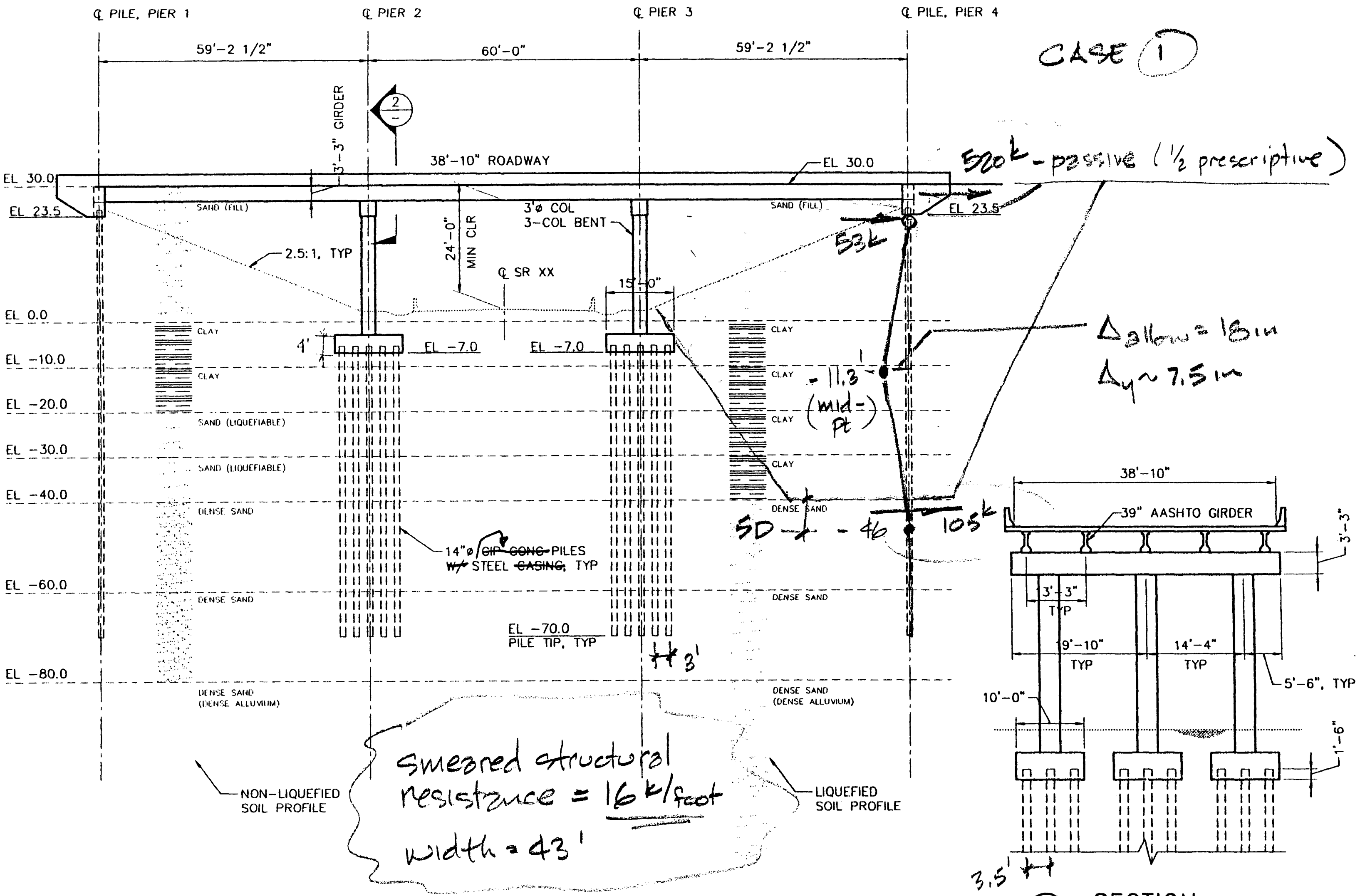
PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\MO\DESIGN\RC-PILE.COL

Project: NCHRP 12-49 Missouri Bridge

Column: Piers 2 & 3		Engineer: MLT	
f'c = 4 ksi	fy = 60 ksi	Ag = 153.938 in^2	97 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 13.97 in^2	Rho = 9.08%
fc = 3.4 ksi	e_rup = Infinity	Xo = 0.00 in	Ix = 1885.74 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 1885.74 in^4
Beta1 = 0.85		Clear spacing = 0.12 in	Clear cover = -0.19 in
Confinement: Other	phi(a) = 1, phi(b) = 1, phi(c) = 1		

I.8 Pinning Calculations



1 / – ELEVATION – MISSOURI BRIDGE
SCALE: 1" = 15'-0"

2 / – SECTION
SCALE: 1" = 20'-0"

Fig. 4.3

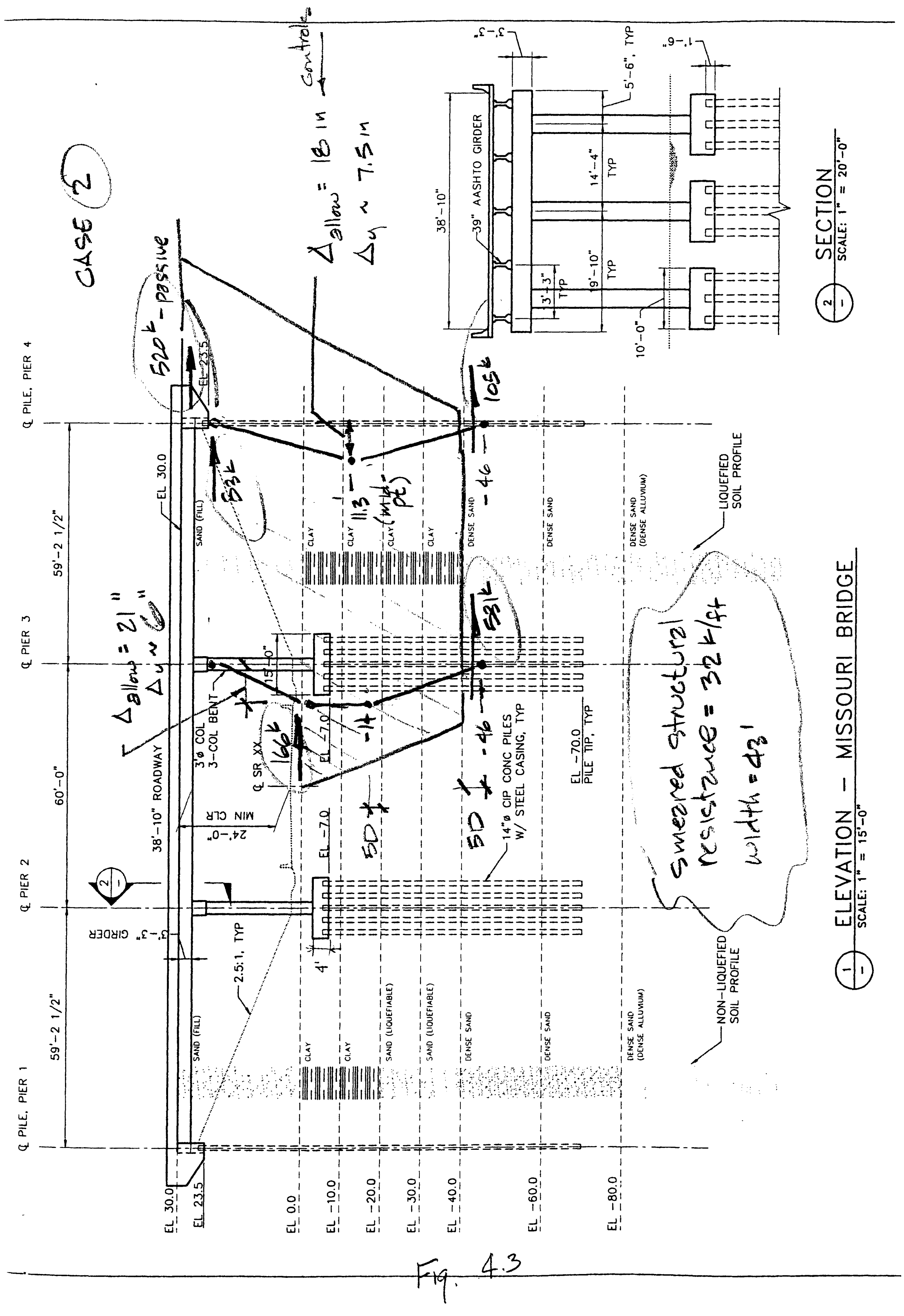

CASE 2
Δallow = 18 in controls
Δy ~ 7.5 in
520k - passive
53k
11.3 (mid-pt)
-46
105k
88k
Δallow = 21"
Δy ~ 6"
166k
-14
50
-46
50
Smeared structural resistance = 32 k/ft
width = 43'
℄ PILE, PIER 4
℄ PIER 3
℄ PIER 2
℄ PILE, PIER 1
59'-2 1/2"
60'-0"
59'-2 1/2"
EL 30.0
EL 23.5
EL 0.0
EL -10.0
EL -20.0
EL -30.0
EL -40.0
EL -60.0
EL -80.0
EL -7.0
EL -70.0
PILE TIP, TYP
SAND (FILL)
CLAY
SAND (LIQUEFIABLE)
DENSE SAND
DENSE SAND (DENSE ALLUVIUM)
38'-10" ROADWAY
24'-0" MIN CLR
3'-3" GIRDER
2.5:1, TYP
3'ø COL 3-COL BENT
15'-0"
℄ SR XX
14"ø CIP CONC PILES W/ STEEL CASING, TYP
4'
LIQUEFIED SOIL PROFILE
NON-LIQUEFIED SOIL PROFILE
1
ELEVATION – MISSOURI BRIDGE
SCALE: 1" = 15'-0"
38'-10"
39" AASHTO GIRDER
3'-3"
5'-6", TYP
1'-6"
14'-4" TYP
19'-10" TYP
3'-3" TYP
10'-0"
2
SECTION
SCALE: 1" = 20'-0"

Fig. 4.3

Case ①

14" pipe alone (see attached comparison)

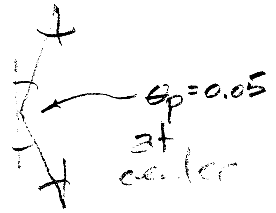
$$\Delta_y \sim \frac{2437\,(69.5(12))^2}{24\,(3830)\,2450} = 7.5 \text{ in} \qquad \frac{69.5}{2} = 34.8'$$

Cracked pipe xfrm to conc.

$$\Delta_p \sim 34.8\left(\frac{0.05}{2}\right)12 = 10.4 \text{ in}$$

using 0.05 rad θ_p limit at all hinges

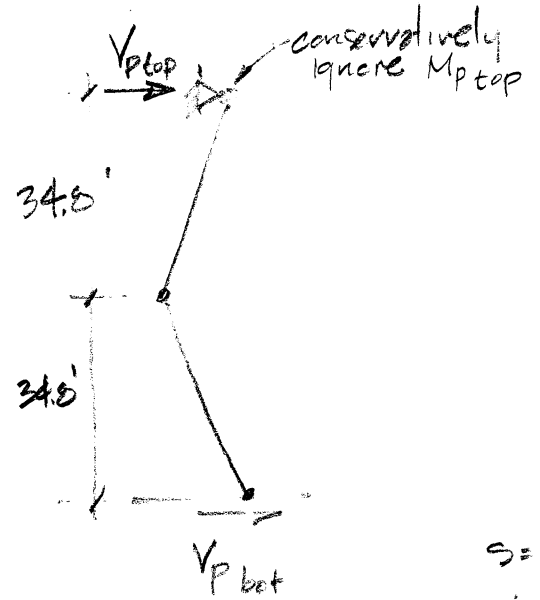
$$\Delta_t \sim \quad \sim (17.9 \text{ in})$$



Abutment Shears:

Case ①

$$V_p = \frac{1028}{2} = \boxed{520^k} \quad (\text{half max passive})$$



$$V_{p\,top} = \frac{2437}{12}\,\frac{1}{34.8} = 5.8^k \times 9 \text{ piles} = \boxed{53^k}$$

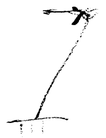
$$V_{p\,bot} = 2V_{p\,top} = 11.7^k \times 9 \text{ piles} = \boxed{105^k}$$

s = 4.75' 14"ϕ pipe spacing at abutment

width = 43' = 9(4.75')

$$\text{smeared force} = \frac{520 + 53 + 105}{43'} = \boxed{16 \text{ k/foot}}$$

Case 2



$$\Delta_{y\ col} \sim \frac{1650(29.8)^2}{3(3605)\frac{82448}{2\ 12^2}\cancel{12^2}} = 0.48' \text{ or } 5.8''$$

$$\Delta_{y\ piles} \sim \frac{202(32)^2}{6(3605)\frac{2450}{12^2}} = 0.57' \text{ or } 6.8''$$

$$\Delta_{p_{col}} \sim 0.05(29.8) = 1.49'$$

Δ_t = — = 1.97' total allowable Δ
(23.6'') as limited by column

$$\Delta_{p\ pile} = 0.05(32) = 1.60'$$

$\Delta_t = \Delta_{y\,pile} + \Delta_{pile} = 2.17' > 1.97'$ ∴ column controls
(26.0'')

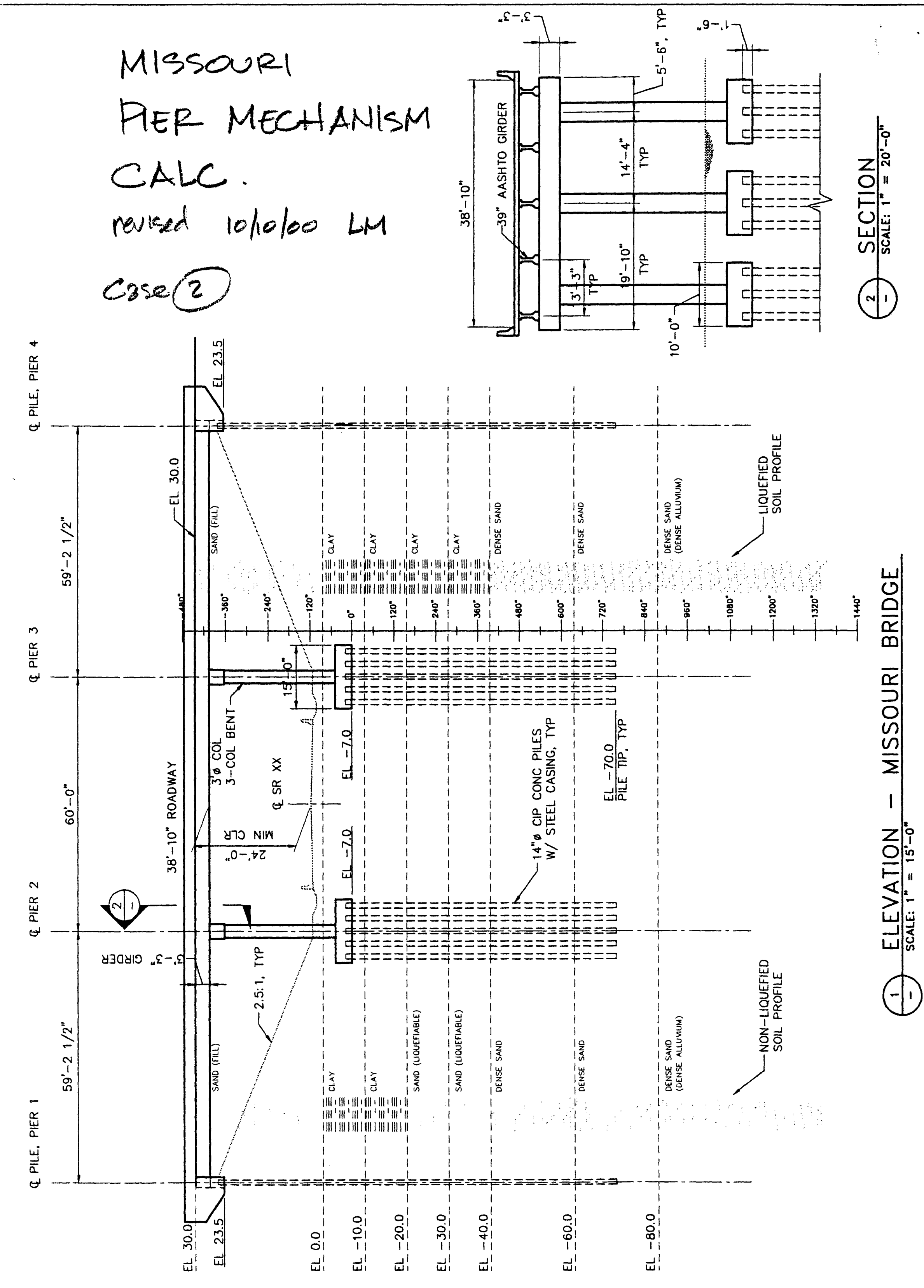
Overall case 1 abutment pile limits Δ's @ 18in
∴ 18'' is limiting drift of founding soil

CASE ②

smeared resistance =

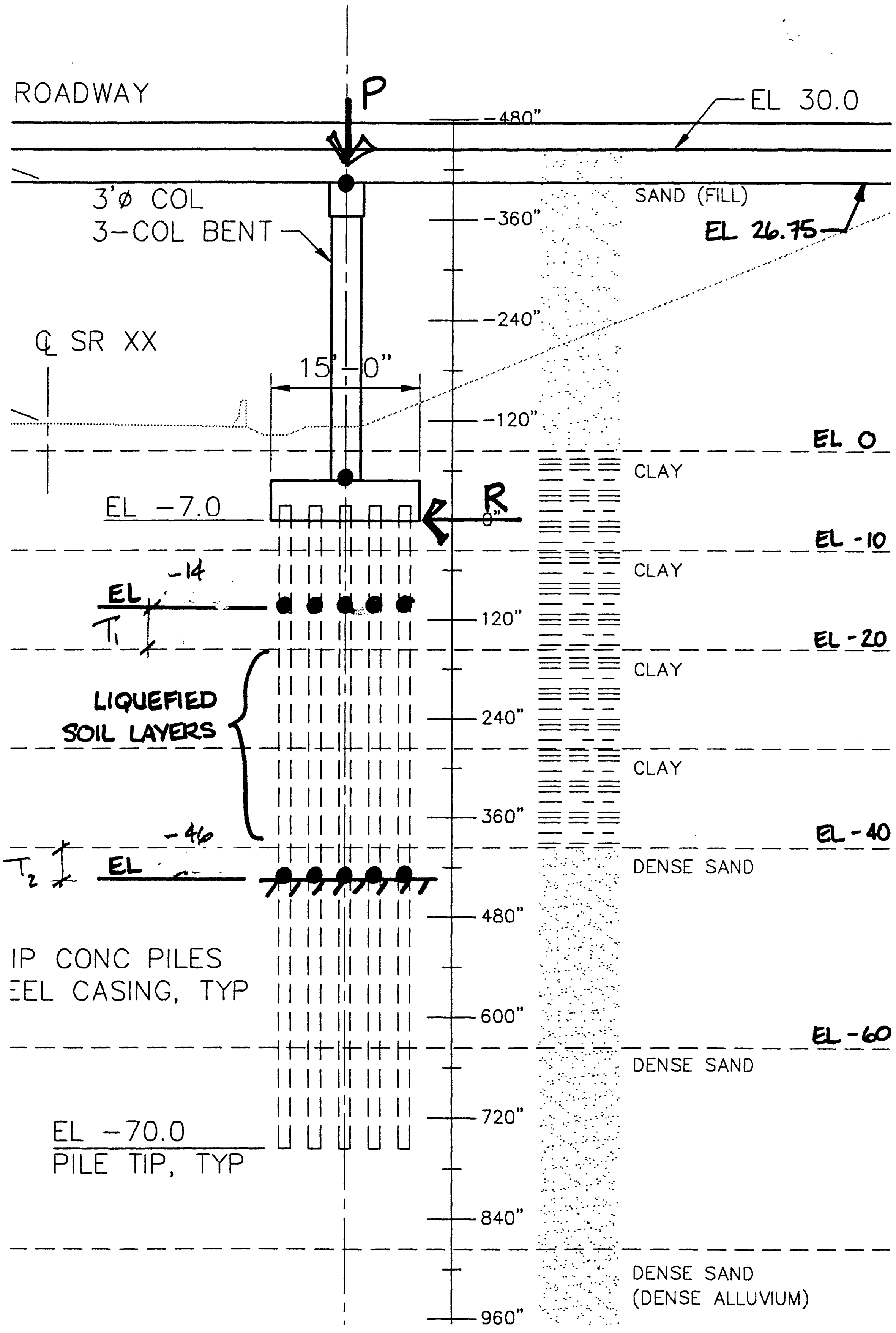
$$\frac{531 + 166 + 520 + 53 + 105}{93'} = 32\ ^{k}/_{ft}$$

MISSOURI
PIER MECHANISM
CALC.
revised 10/10/00 LM
Case ②

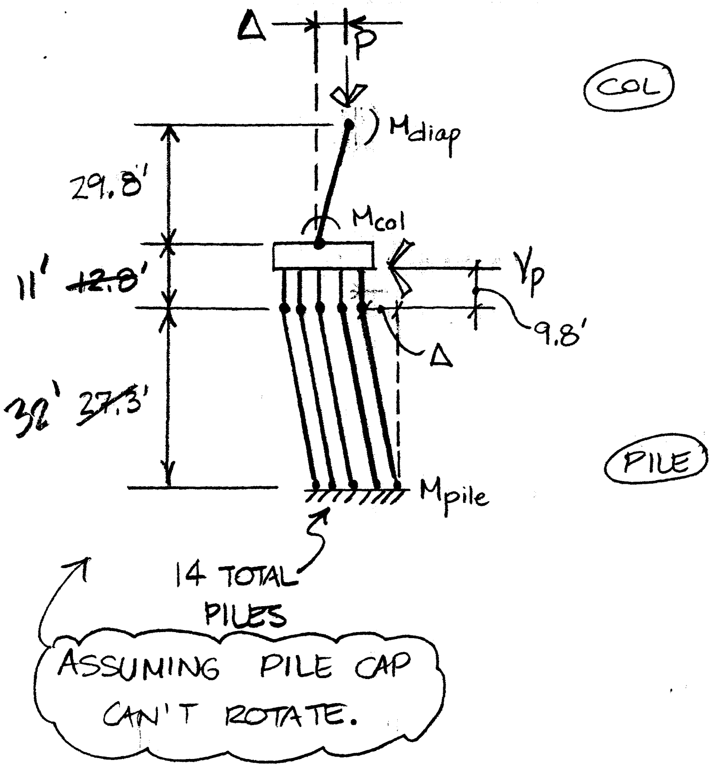


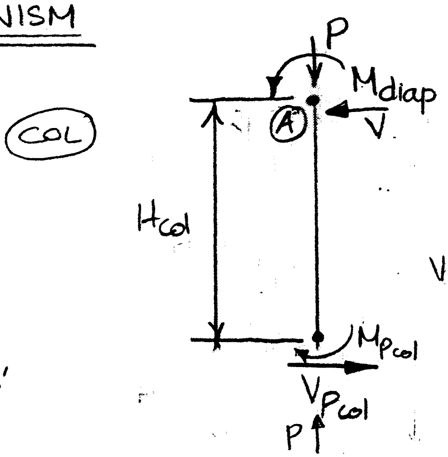
② SECTION
1
SCALE: 1" = 20'-0"

① ELEVATION – MISSOURI BRIDGE
1
SCALE: 1" = 15'-0"


ROADWAY
P
EL 30.0
-480"
SAND (FILL)
3'ø COL
3-COL BENT
-360"
EL 26.75
-240"
℄ SR XX
15'-0"
-120"
EL 0
CLAY
EL -7.0
R
0"
EL -10
-14
EL
T1
CLAY
120"
EL -20
CLAY
LIQUEFIED SOIL LAYERS
240"
CLAY
-46
360"
EL -40
T2
EL
DENSE SAND
480"
IP CONC PILES
EEL CASING, TYP
600"
EL -60
DENSE SAND
EL -70.0
PILE TIP, TYP
720"
840"
DENSE SAND
(DENSE ALLUVIUM)
960"

PLASTIC HINGING MECHANISM

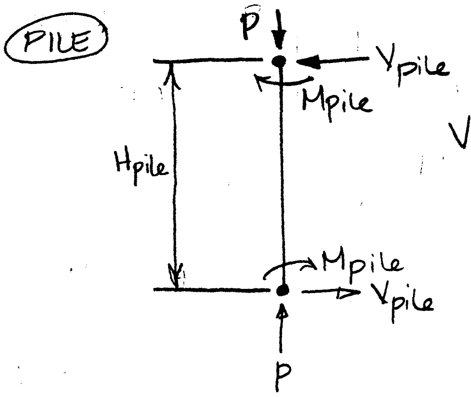




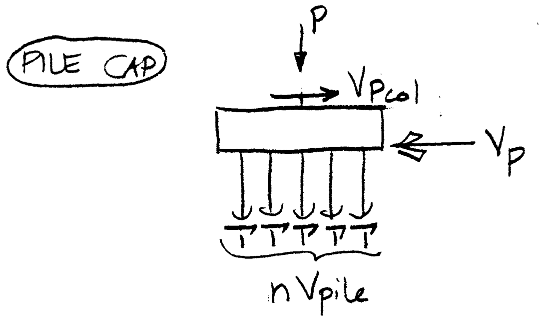
$$\Sigma M @ Ⓐ = 0$$

$$V_{Pcol} H_{col} - M_{Pcol} = 0$$

$$V_{P_{col}} = \frac{M_{Pcol}}{H_{col}}$$



$$V_{pile} = \frac{2 M_{pile}}{H_{pile}}$$



$$\Sigma F_H = 0$$

$$V_p = V_{Pcol} + n V_{pile}$$

$$\boxed{V_p = \frac{M_{Pcol}}{H_{col}} + \frac{2n M_{pile}}{H_{pile}}}$$

$$\therefore V_p = \frac{1650'^k}{29.8'} + \frac{2(14)(138'^k)}{\cancel{27.3'}\ 32'}$$

$= 55.4^k + ~~141.5^k~~$ ⇒ ~~$V_p = 197^k$~~ (1/4" wall)

[166 ^{k}/pier]

[3/8" WALL : $\frac{2(14)(203'^k)}{\cancel{27.3}\ 32'}$ = ~~208~~^{k} 177 ^{k}/col ⇒ V_p = ~~264^k~~] 3/8" wall

[531 ^{k}/pier]

233^k total/col.

1699^k total / pier

$$T = 0.78 \sqrt[5]{\frac{EI}{n_h}} = L_M$$

$E = 3830$ ksi

$I_t = 3963 \text{ in}^4$ → USE $I_{pipe} = 2450 \text{ in}^4$ B/C CONC CRACKED, NO REBAR

FOR UPPER 'DENSE SAND' LAYER, $n_h = 40$ pci
(INCL. GROUP EFFECT)

$$T_1 = 0.78 \sqrt[5]{\frac{(3830 \frac{k}{in^2})(2450 \text{ in}^4)}{0.040 \frac{k}{in^3}}} = 37'' \Rightarrow \underline{\underline{\cancel{T_1 = 3.1'}}}$$

FOR CLAY LAYER @ EL -10 TO EL -20, $n_h = 8$ pci
(W/ GROUP EFFECT

$$T_2 = 0.78 \sqrt[5]{\frac{(3830 \text{ ksi})(2450 \text{ in}^4)}{0.008 \text{ kci}}} = 51'' \Rightarrow \underline{\underline{\cancel{T_2 = 4.2'}}}$$

USE ~ 5D per Geoff Martin

$$5\left(\frac{14}{12}\right) \sim 6'$$

SUPERSTRUCTURE DEAD LOAD

$$A_{super} = 40.77\ ft^2$$

$$A_{barrier} = 4.52\ ft^2$$

$$\Sigma = 45.28\ ft^2$$

$$\omega_{super} = (150\ pcf)(45.28\ ft^2)/_{1000\#} \Rightarrow \underline{\underline{W_{super} = 6.79\ klf}}$$

$$P = \underbrace{\tfrac{1}{2}(60' + 59.21')}_{59.6'}(6.79\ klf) \Rightarrow \boxed{P = 405^k}$$

PLASTIC MOMENT OF PILE

NEGLECT CONC → CRACKED

→ NO REBAR

$$M_p = Z f_y$$

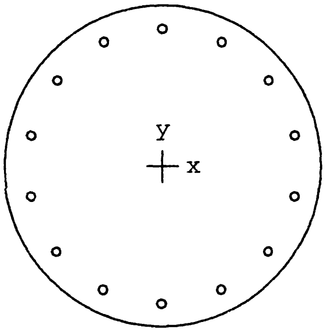
$$Z = \frac{d_o^3}{6} - \frac{d_i^3}{6} = \frac{(14'')^3}{6} - \frac{(13.5'')^3}{6} \Rightarrow \underline{\underline{Z = 47.3 in^3}}$$

$$M_p = (47.3 in^3)(\underset{\llcorner A252,\ Gr\ 2}{35\ ksi}) = 1654''^k \Rightarrow \boxed{M_p = 138'^k}$$

ASSUME $\frac{3}{8}''$ WALL

$$Z = \frac{(14'')^3}{6} - \frac{(13.25'')^3}{6} = 69.6 in^3$$

$$M_p = (69.6 in^3)(35 ksi) = 2437''^k = 203'^k$$



36 in diam.

Code: ACI 318-95

Units: English

Run axis: About X-axis

Run option: Investigation

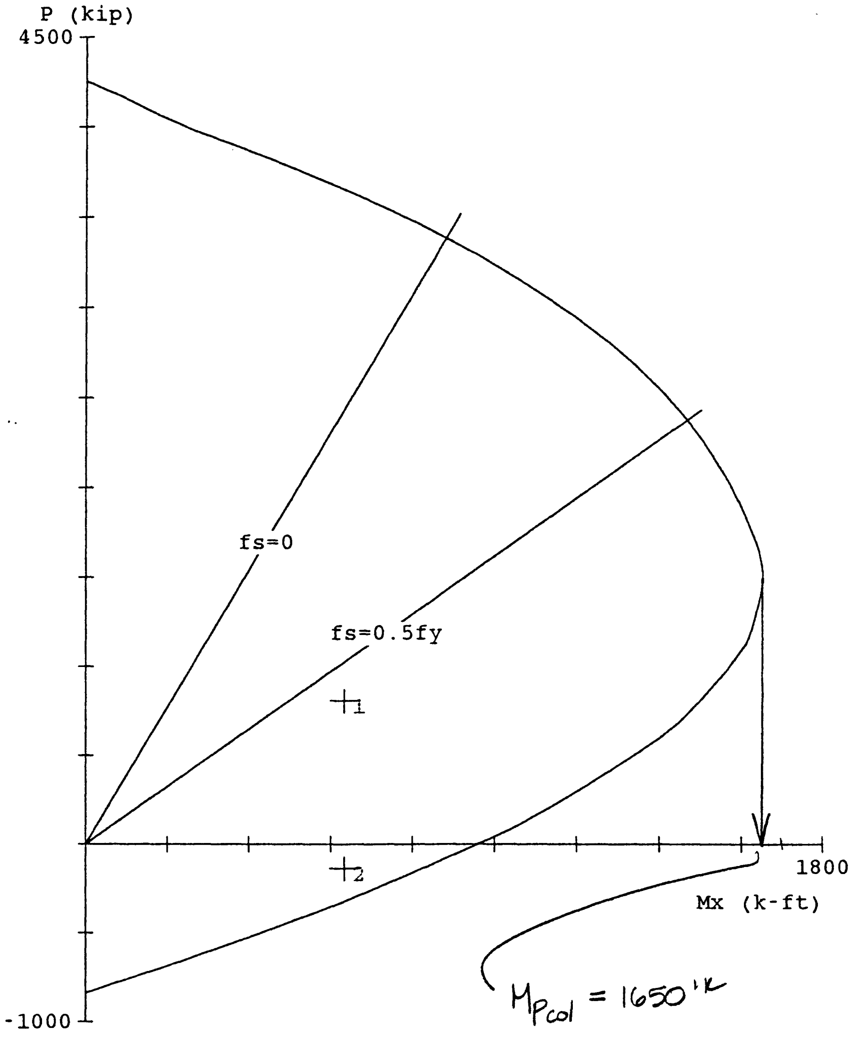
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 09/06/00

Time: 09:50:15



PCACOL V3.00 (PCA 1999) - Licensed to: BERGER/ABAM Engineers Inc., Federal Way, WA

File: O:\1999\A99067\ENGR\MISSOU~1\SAP2000\TRIAL2.COL

Project: NCHRP 12-49 Missouri Bridge

Column: Piers 2 & 3

Engineer: MLT

f'c = 4 ksi

fy = 60 ksi

Ag = 1017.88 in^2

14 #9 bars

Ec = 3605 ksi

Es = 29000 ksi

As = 14.00 in^2

Rho = 1.38%

2500 yr w/ R=6

fc = 3.4 ksi

e_rup = Infinity

Xo = 0.00 in

Ix = 82448 in^4

e_u = 0.003 in/in

Yo = 0.00 in

Iy = 82448 in^4

Beta1 = 0.85

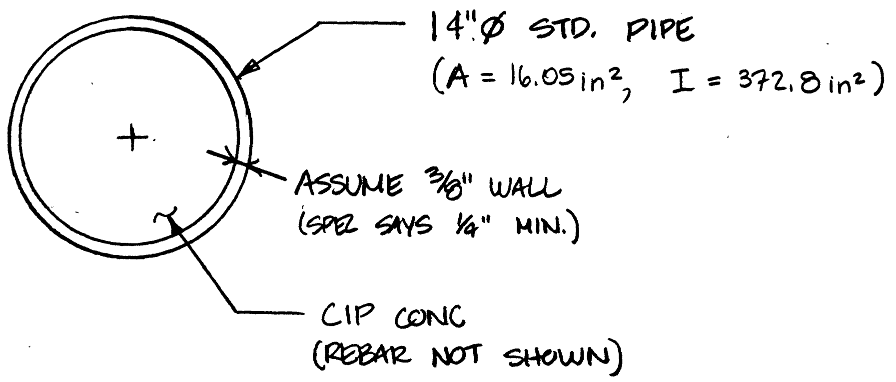
Clear spacing = 5.74 in

Clear cover = 2.00 in

Confinement: Other

phi(a) = 1, phi(b) = 1, phi(c) = 1

PILE SECTION PROPERTIES



AREA:

$$A_c = \frac{\pi}{4} D^2 = \frac{\pi}{4}(13.25'')^2 \Rightarrow \underline{\underline{A_c = 137.9\,in^2}}$$

$$n = \frac{E_s}{E_c} = \frac{29000\text{ ksi}}{3800\text{ ksi}} = 7.6 \approx 8$$

$$A_{st} = n\left(\frac{\pi}{4}\right)(D_o^2 - D_i^2) = (8)\left(\frac{\pi}{4}\right)(13.875^2 - 13.25^2) \Rightarrow \underline{\underline{A_{st} = 106.5\,in^2}}$$

NEGLECT 1/16" FOR CORROSION

(transformed)

$$A_{Pt} = A_c + A_{st} = 137.9 + 106.5 \Rightarrow \boxed{A_{Pt} = 244.4\,in^2}$$

MOMENT OF INERTIA:

$$I_c = \frac{\pi}{64} D^4 = \frac{\pi}{64}(13.25'')^4 \Rightarrow \underline{\underline{I_c = 1513.0\,in^4}}$$

$$I_{st} = \frac{1}{16} A_{st}(D_o^2 + D_i^2) = \frac{1}{16}(106.5\,in^2)(13.875^2 + 13.25^2)$$

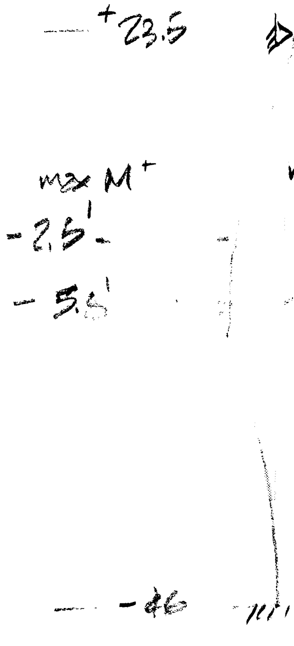
$$\Rightarrow \underline{\underline{I_{st} = 2450.0\,in^4}}$$

$$I_{Pt} = I_c + I_{st} = 1513.0 + 2450.0 \Rightarrow \boxed{I_{Pt} = 3963.0\,in^4}$$

$$I_{cr} = I_{st} = 2450\,in^4$$ (NO REBAR, ∴ NO CONC CONTRIBUTION)

transformed

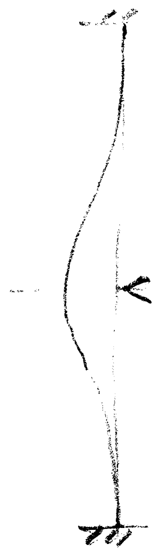
Δy — Propped Cantilever w/ Uniform Load



$$\max M^- = \frac{wl^2}{8} \rightarrow wl^2 = 8M$$

$$\Delta = \frac{wl^4}{185EI} \rightarrow \Delta_y = \frac{8M_y l^2}{185EI} = \frac{M_y l^2}{23EI}$$

Δy - Fixed-Fixed w/ Conc. Load



$$\Delta_y = \frac{M_P(l/2)^2}{6EI} \rightarrow \frac{M_P l^2}{24EI}$$

or $M = \frac{Pl}{8} \rightarrow P = \frac{8M}{l}$

$$\Delta = \frac{Pl^3}{192EI} \rightarrow \Delta_y = \frac{8M_y l^2}{192EI} = \frac{M_y l^2}{24EI}$$

I.9 Cost Analysis

PILE AND BRIDGE COST DATA

Missouri Pile Cost

Bid Item	Unit Quantity	Length Along Pile	Total Quantity	Unit Cost	Cost per Pile
24"φ X-Strong Pipe ($t_{wall} = {}^1/_2$")	55 lbs/ft of pipe	70 ft	3850 lbs	Steel Pipe Piles $ 0.35 / lbs	$ 1,348
Concrete In Place	0.04 cy./ft of pile	70 ft	2 cy	Concrete In Place $ 150 / yd	$ 373
Reinforcing Steel (8 #11 bars)	46.6 lbs/ft of pile	20 ft	933 lbs	Reinforcing Steel $ 0.50 / lbs	$ 466
Pipe Pile Splice			0 splice	Pile Splices / splice	$ -
Pile Driving			1 pile	Pile Driving $ 300 / pile	$ 300
					$ 2,500

Missouri Bridge Cost

		Unit Cost	Cost
Overall Width =	40 ft		
x Total Length =	180 ft	$ 80 / sq ft	**$ 0.6 Million**
Bridge Deck Area =	7200 sq ft	$ 100 / sq ft	**$ 0.7 Million**