Tennessee Valley Authority Kingston Fossil Plant Harriman, Roane County, Tennessee	**Perimeter Containment** **North Dredge Cell (Dike C) Segment 1** **Sta. A161+50 to A179+50**

Document Control Number	RDP-0113-E
Calculation Package	FPGKIFFESCDX00030020100006

Stantec

Exhibit 10
Ground Response Analyses

Purpose:

- Using top-of-hard rock acceleration time histories, predict ground motions at various locations within the closed facility.
- Compute maximum induced cyclic stresses for use in liquefaction analyses.

Methods:

- 1D equivalent linear ground response analyses accomplished using ProSHAKE.

Results:

- Acceleration time histories to be used as input to the dynamic FLAC simulations.
- Cyclic stress profiles, for individual SPT boring logs and cone penetration test (CPT) logs, as needed for liquefaction analyses.

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Revisions:	

Ground Response Analyses

The purpose of the ground response analyses is to compute the maximum induced cyclic shear stresses under design earthquake events, which will be used in the liquefaction triggering analyses. Another objective is to calculate the ground motion time histories at the top of weathered shale based on the input motions at the top of hard rock. The obtained weathered shale ground motions will be utilized in the *FLAC* analyses.

1.1. *ProShake* Computer Code

One-dimensional, equivalent linear ground response analyses were performed to predict the ground motions (acceleration time histories) and induced cyclic shear stresses in the landfilled embankment, perimeter earthen berm, and foundation soils for the closure plan. As depicted schematically in Figure 1.1, the analyses involve the numerical propagation of the earthquake ground motion time histories at the top of rock through a one-dimensional soil profile.

The *ProShake* computer program (version 1.12, EduPro Civil Systems, Inc., Sammamish, Washington) was used to perform these analyses for selected soil profiles during the design earthquakes. *ProShake* is based on the theoretical principles used in earlier programs, such as *SHAKE* and *SHAKE91*. *ProShake* computes the response of a system of homogeneous, visco-elastic soil layers of infinite horizontal extent subjected to vertically traveling shear waves. The program is based on the continuous solution of the wave equation, adapted for use with transient motions through the *Fast Fourier Transform* algorithm. The non-linearity of the shear modulus (G) and damping ratio (ξ) is modeled with equivalent linear soil properties. An iterative procedure is used to obtain values of modulus and damping compatible with the effective strains in each layer. *ProShake* calculates the maximum shear stresses and accelerations, within each modeled deposit, that will be induced by the design earthquakes.

The following assumptions are made in the analyses:

- The soil/rock system extends infinitely in the horizontal direction.
- Each soil/rock deposit in the system is completely defined by its thickness, unit weight, shear modulus, and damping ratio.
- The strain dependence of modulus and damping is accounted for by an equivalent linear procedure based on an average effective strain level computed for each deposit.
- The responses in the system are caused by the upward propagation of shear waves from the underlying bedrock formation.
- The shear waves are given as acceleration values at equally spaced time intervals. Cyclic repetition of the acceleration time history is implied in the solution.

The program can model systems with variations in both moduli and damping ratios and accounts for the effects of an elastic half space. The motion used as a basis for the analysis (the object motion) can be defined in any one layer in the system and new motions can be computed in any other layers.

Input parameters required for the *ProShake* program include:

- Relationships between shear modulus and effective strains, and relationships between critical damping ratio and effective strains.
- Acceleration time history (top of hard rock).
- Depth to the phreatic surface (this information is not used in ground response analysis because *ProShake* performs total stress analysis; this information is required when vertical effective stress needs to be calculated).
- Soil/rock data, including number of deposits (i.e., layers), soil type, plasticity index (*PI*) for cohesive soils, layer thickness, shear modulus (or shear wave velocity), and unit weight.

The relationships between the shear moduli and effective strains are obtained from normalized shear modulus reduction curves (G/G_{max} vs. strain). A number of such relationships are available within *ProShake* for various soil types. Similarly, damping curves are also available within *ProShake* for relating the equivalent linear damping ratio to effective shear strain. *ProShake* can also accommodate user defined modulus and damping curves.

1.2. Profiles Analyzed for Kingston Closure Plan

ProShake analyses were carried out for three generalized soil/rock profiles for the closure plan. These profiles (Table 1.1) were developed to represent the subsurface conditions for groups of CPT and SPT data having similar proposed embankment heights. Soil layers with fairly equal thickness, rock layers representing the weathered shale (shear wave velocity (V_s) << 9,000 ft/sec), and a rock layer representing "hard rock" (i.e., V_s > 9,000 ft/sec) were defined in the analysis for each of the simplified profiles. Groundwater levels were assumed at El. 760 and El. 750, respectively, for boring locations inside and outside the proposed perimeter earthen berm. Material properties (unit weight, shear modulus or shear wave velocity, modulus reduction curve, and damping curve) used in the analyses are detailed in Exhibits 3 and 9. Thickness of the weathered shale was estimated based on the results obtained from the SASW testing performed at the site by the University of Texas (Exhibit 8). Design earthquakes and ground motion time histories were provided by TVA and prepared by AMEC Geomtrix, Inc. (AMEC) based on a site-specific seismic study (Exhibit 7). The results (maximum induced shear stresses) of the ground response predictions were then applied to the appropriate groups of exploration holes in the liquefaction triggering analyses.

Table 1.1. Generic soil profiles used in ground response analyses to represent groups of subsurface explorations.

Profile	Ground Surface Elevation, ft.	Subsurface Layers	CPT/SPT Holes Represented
1	El. 765	0' – 5': Earth Berm 5' – 35': Hydraulically Placed Ash 35' – 45': Lean Clay Foundation Soil 45' – 60': Sandy Silt to Silty Sand 60' – 730': Weathered Shale 730': Hard Rock	09-200 09-211 09-400 through 09-413
2	El. 750	0' – 20': Hydraulically Placed Ash 20' – 30': Lean Clay Foundation Soil 30' – 45': Sandy Silt to Silty Sand 45' – 715': Weathered Shale 715': Hard Rock	09-201 through 09-210
3	El. 790	0' – 30': Landfilled Ash 30' – 60': Hydraulically Placed Ash 60' – 70': Lean Clay Foundation Soil 70' – 85': Sandy Silt to Silty Sand 85' – 755': Weathered Shale 755': Hard Rock	09-100 through 09-110 09-301 through 09-304 09-500 through 09-503 09-600 through 09-605

1.3. Ground Response Analysis of Closure Plan

The site-specific seismic study performed by AMEC produced 18 time histories due to two potential earthquake scenarios (M_w=7.6 (1 Hz) New Madrid earthquake and M_w=6.0 (10 Hz) local earthquake). Three motions per scenario (actual recordings, spectrally matched, and scaled to the hazard) and three components of motion (two horizontal and one vertical) for each motion were generated. Because the results of the ground response analysis are utilized primarily in the liquefaction triggering analysis, selection of time histories for the *ProShake* analysis should be based on those motions that control the liquefaction response. In other words, both earthquake magnitude and associated maximum induced shear stresses for each generic profile should be considered. Since vertical accelerations are generally small compared to horizontal accelerations, they are ignored for liquefaction triggering calculations (Youd et al. 2001). The six horizontal components of motion for each design earthquake were applied to the top of hard rock and run through the *ProShake* analyses to estimate the maximum induced shear stresses at the midpoint of each defined subsurface layer. As an example, Generic Profile 1 is presented in Figure 1.2. The input acceleration time history at the top of hard rock and the maximum induced shear stress at the midpoint of each defined overlying soil layer are also shown for the case using scenario M_w=7.6. Note that the input motions were used as outcrop motions based on information provided by AMEC; and a deconvolution calculation was performed in *ProShake* to convert the motion to an in-layer motion.

The calculated induced shear stresses versus depth for each horizontal component of ground motion and each generic profile are plotted in Figures 1.3 through 1.5. It can be seen that the M_w=6.0 earthquake produces greater shear stresses than the M_w=7.6 earthquake.

Given the clustered results for each earthquake magnitude, a conservative approach was taken when selecting values of the maximum induced shear stress (τ_{max}) to utilize in liquefaction triggering calculations. The results of the six time histories for each magnitude were enveloped by selecting the highest shear stress value in each layer in the profile. While this does mean that a single time history was not used for liquefaction triggering analysis, each of the six enveloped time histories were derived from events of equal magnitude, thus maintaining consistency in the calculations. The selected τ_{max} versus depth plots for each profile are presented in Figures 1.6 through 1.8.

As detailed in the liquefaction triggering analysis (Exhibit 11), the factor of safety against liquefaction is linearly proportional to the scaled τ_{max} (τ_{max}/MSF, where *MSF* is the earthquake magnitude scaling factor). In order to evaluate which earthquake (M_w=6.0 or M_w=7.6) is more critical in terms of generating liquefaction at the Kingston site, scaled τ_{max} was then computed by dividing the enveloped τ_{max} (Figures 1.6 through 1.8) by *MSF*.

There have been many different suggestions for the relationship between *MSF* and earthquake magnitude. With recently developed new correction correlations for overburden stresses and static shear stresses in liquefaction resistance (detailed in the liquefaction analysis report), Idriss and Boulanger (2008) suggested using the following equation for *MSF*, which was actually the *MSF* published by Idriss (1999).

$$MSF = 6.9\exp(-0.25M_w) - 0.058 \leq 1.8 \qquad \textbf{Eqn. 1.1}$$

Based on the above equation, the New Madrid earthquake (M_w=7.6) results in an *MSF* = 0.97 and the local earthquake (M_w=6.0) results in an *MSF* = 1.48.

The scaled maximum induced shear stresses are presented in Figures 1.9 through 1.11. These plots indicate that in general, the local earthquake is slightly more critical than the New Madrid earthquake, particularly at shallower depths; however, at deeper depths, the New Madrid earthquake may become more critical. Hence, both earthquakes will be subjected to liquefaction evaluations.

For using the results of the ground response analysis in the liquefaction triggering calculations, a curve was fit through the enveloped maximum induced shear stresses (Figures 1.6 through 1.8) to define the induced shear stress at any point in a soil profile. Best-fit equations give a continuous relationship between maximum induced shear stress and depth. As shown in Figures 1.12 through 1.17, both third and fourth order polynomials were fit to the data, the curves were extrapolated to the greatest depth of any CPT and SPT data points that will utilize this fit, and the best of the two curve fits was selected for use in the liquefaction triggering calculations. The average induced Cyclic Stress Ratio (*CSR*) was then calculated at the elevation of each CPT and SPT data point as:

$$CSR = 0.65\frac{\tau_{\max}}{\sigma_v'} \qquad \textbf{Eqn. 1.2}$$

Where, σ_v' is the effective overburden stress

In addition to calculating the maximum induced shear stresses for liquefaction triggering analysis as detailed above, ground motion time histories at the top of weathered shale were exported for Generic Profile 3 (corresponding to the full height of the landfilled ash embankment). These time histories were then used as the input ground motions in *FLAC* analyses.

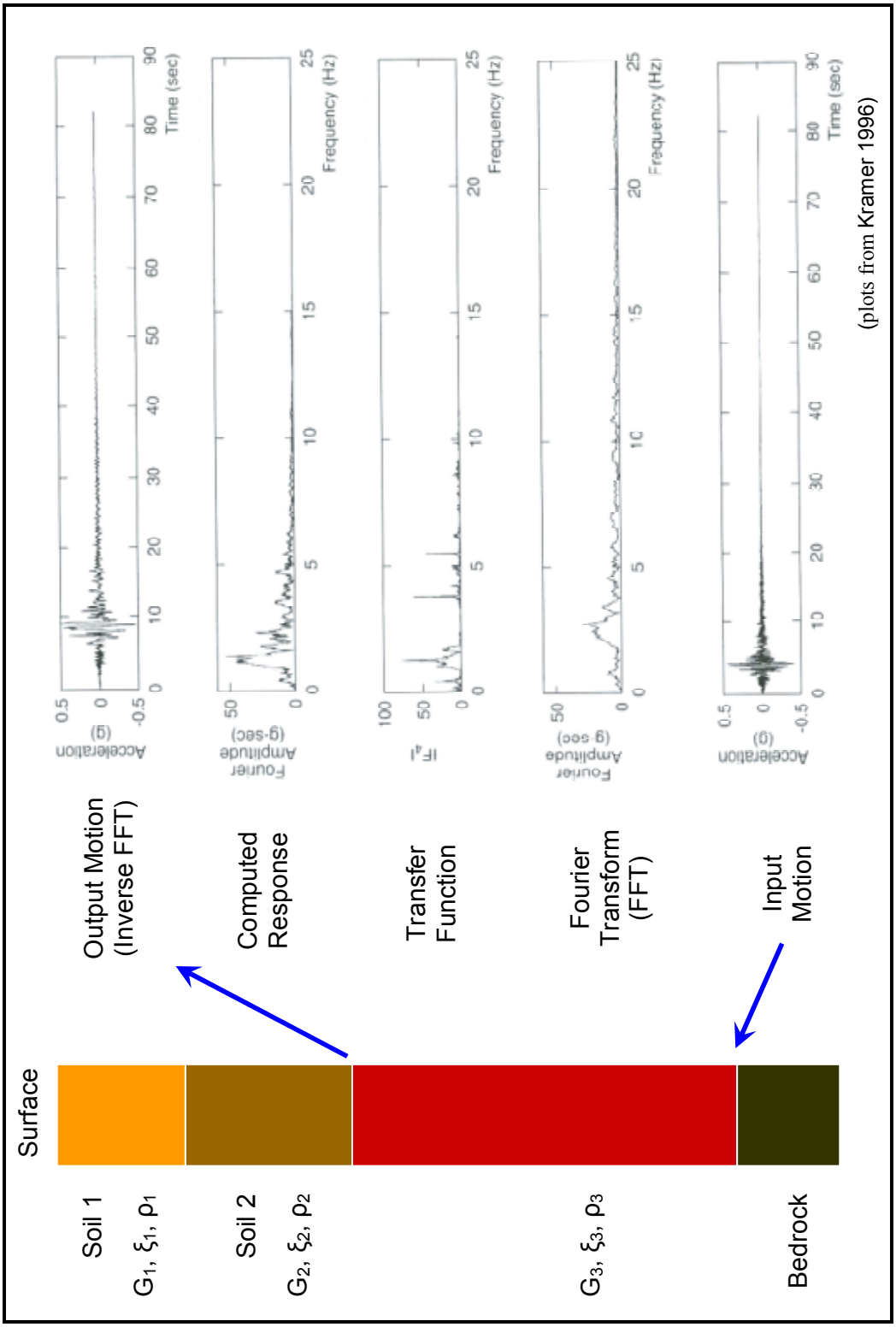

Figure 1.1 – Conceptual Sketch of 1-D Ground Response Analysis

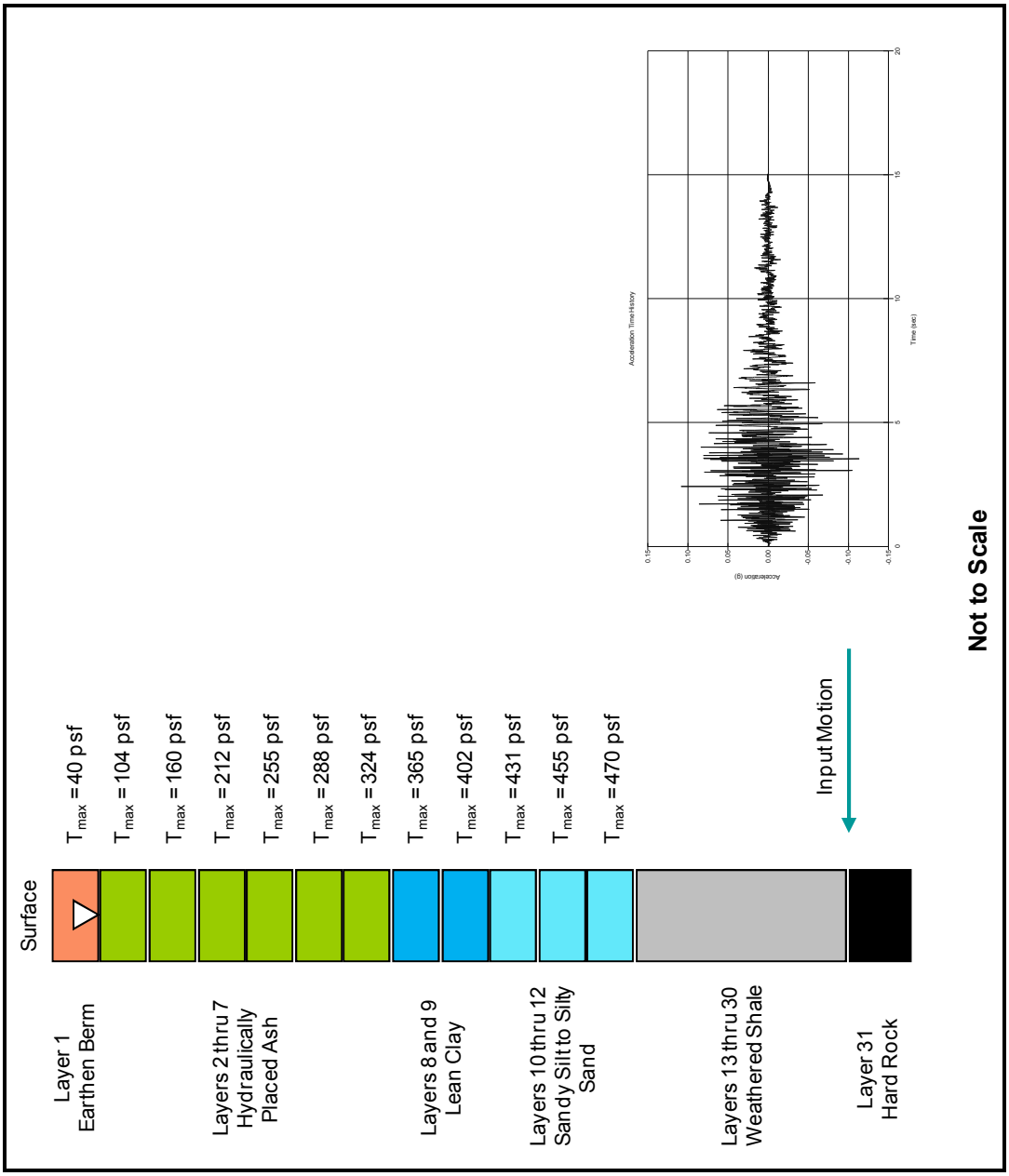

Figure 1.2 – Generic Example of Induced Maximum Shear Stresses in Each Defined Soil Layer (Profile 1, M_w=7.6)

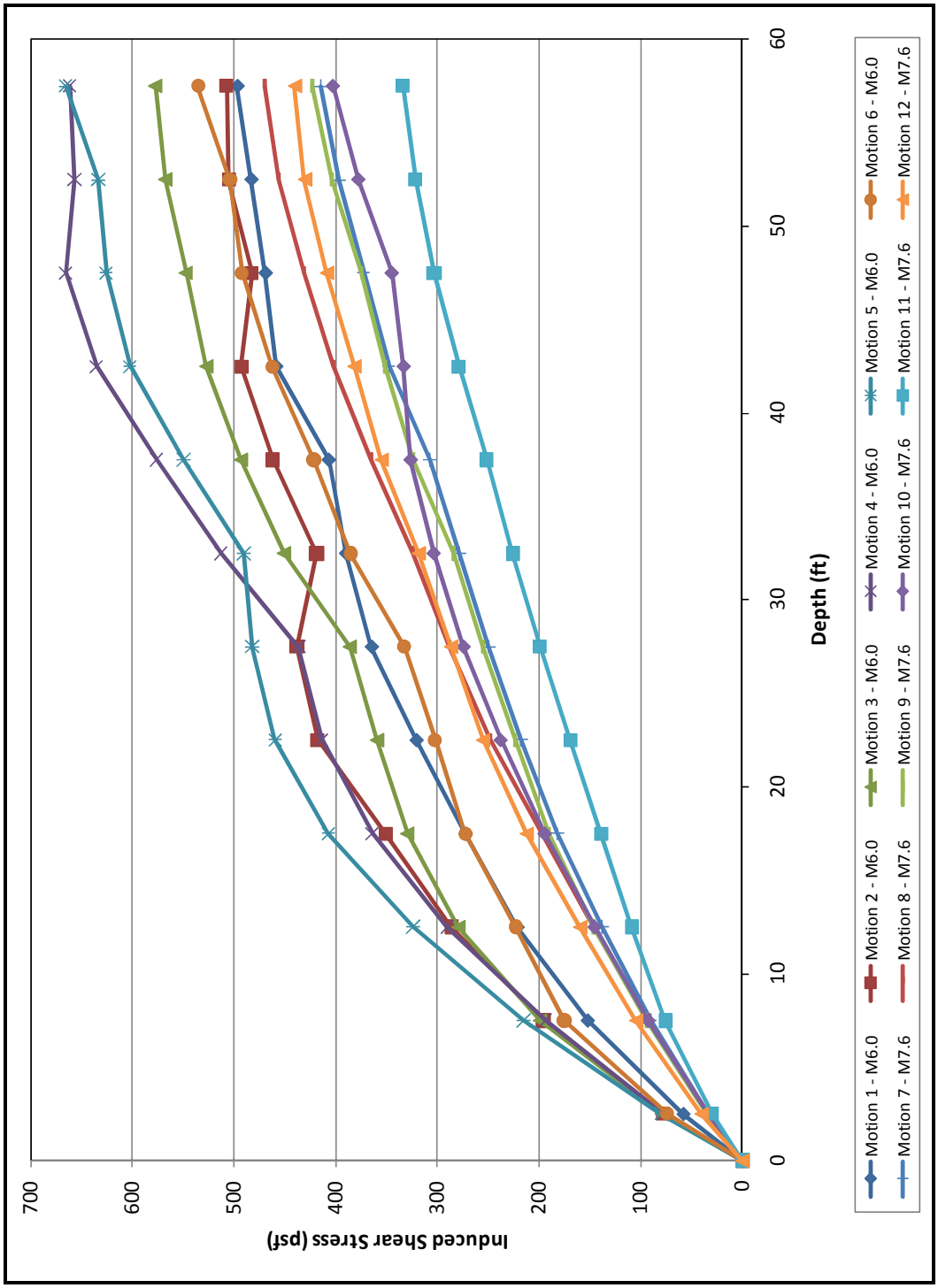

Figure 1.3 – Induced Shear Stresses (Profile 1)

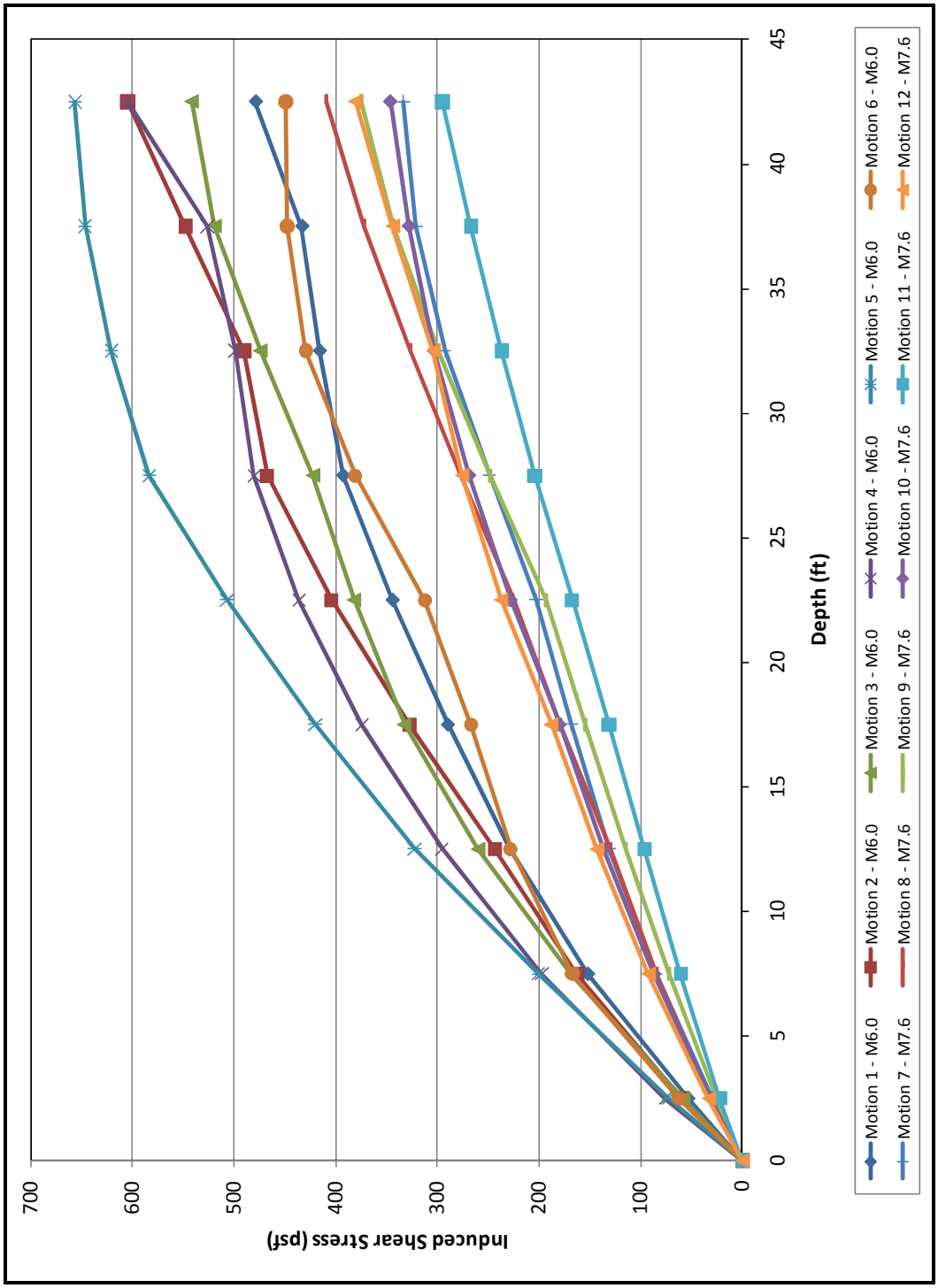

Figure 1.4 – Induced Shear Stresses (Profile 2)

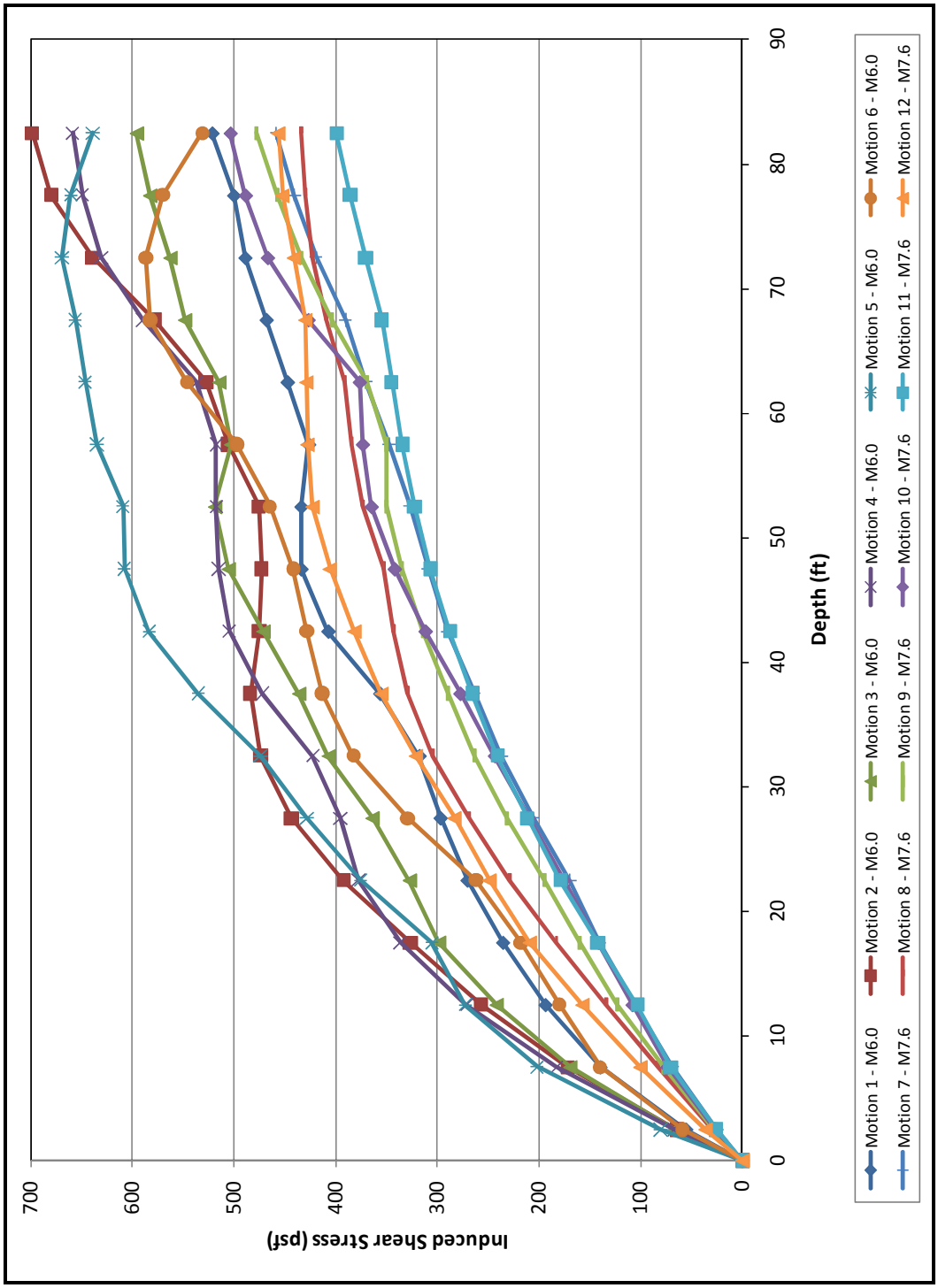

Figure 1.5 – Induced Shear Stresses (Profile 3)

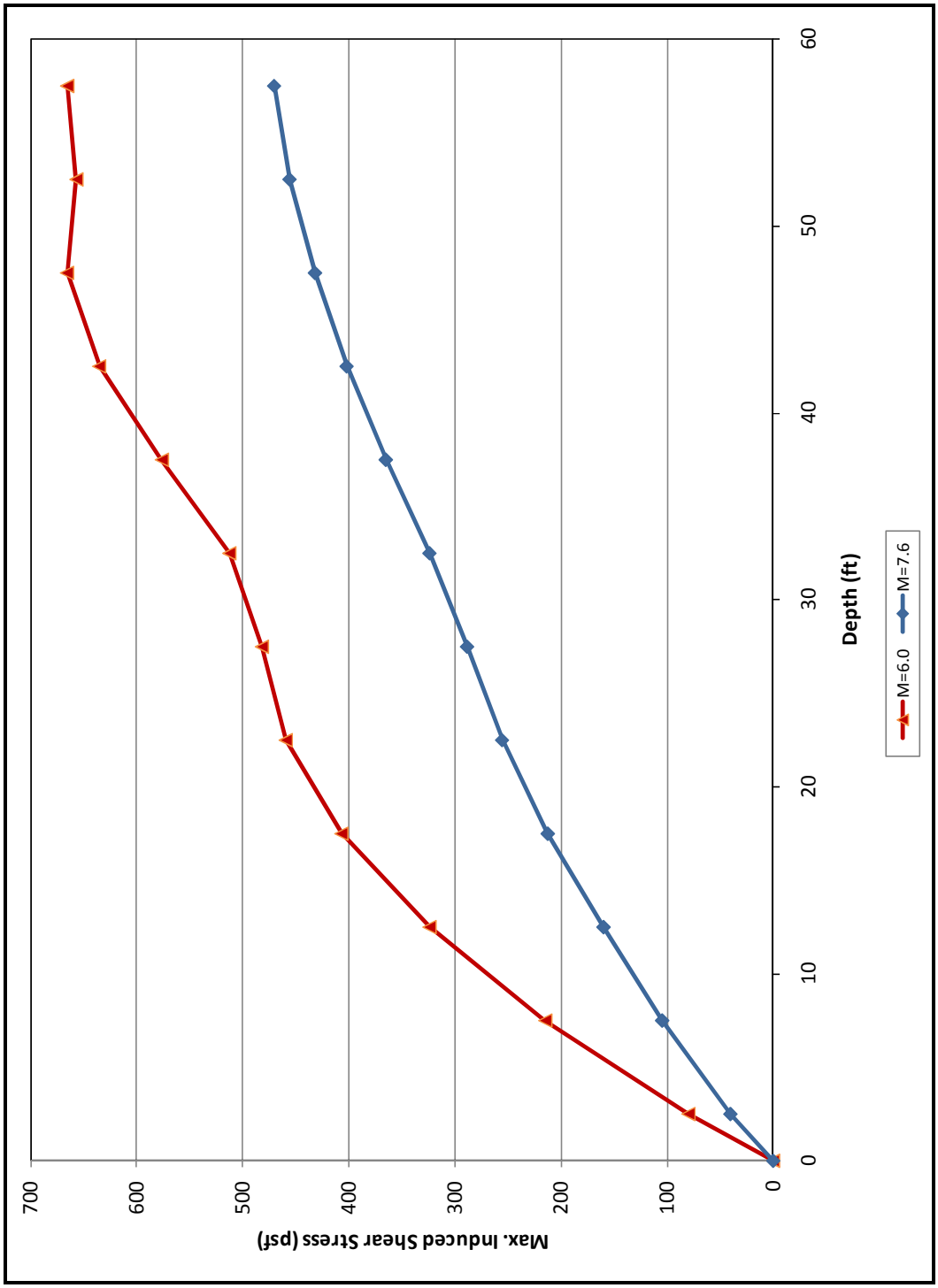

Figure 1.6 – Maximum Induced Shear Stresses (Profile 1)

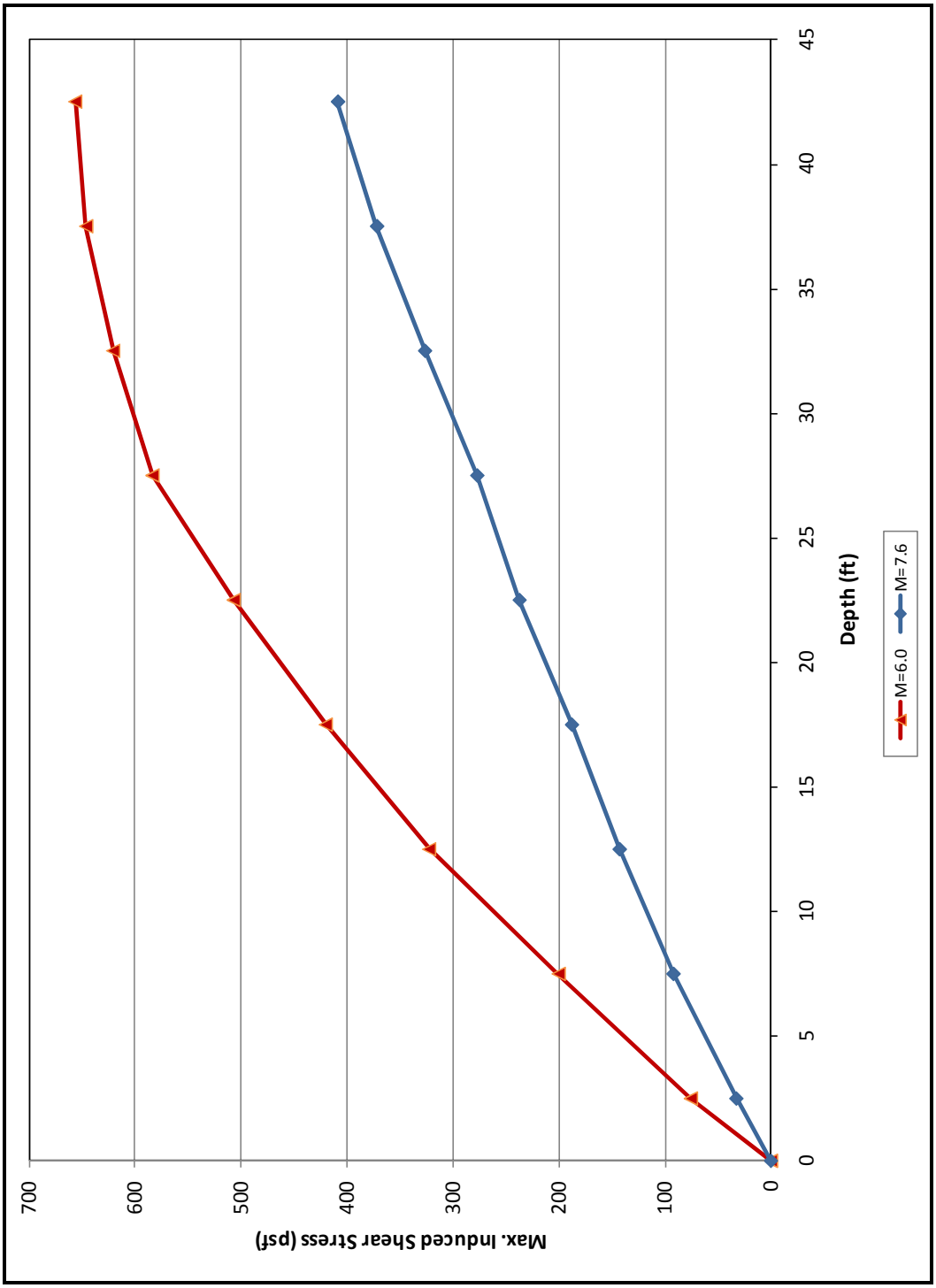

Figure 1.7 – Maximum Induced Shear Stresses (Profile 2)

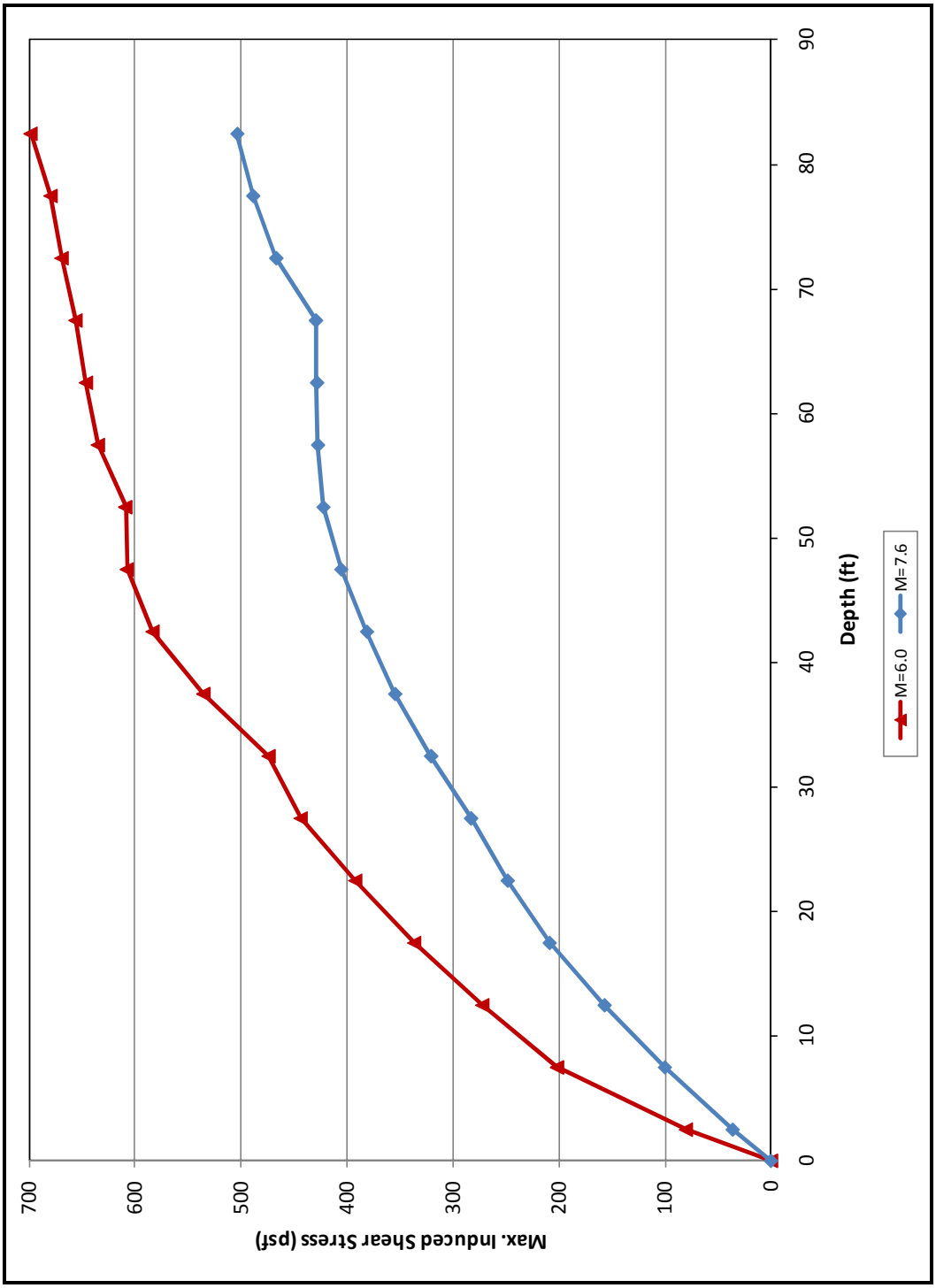

Figure 1.8 – Maximum Induced Shear Stresses (Profile 3)

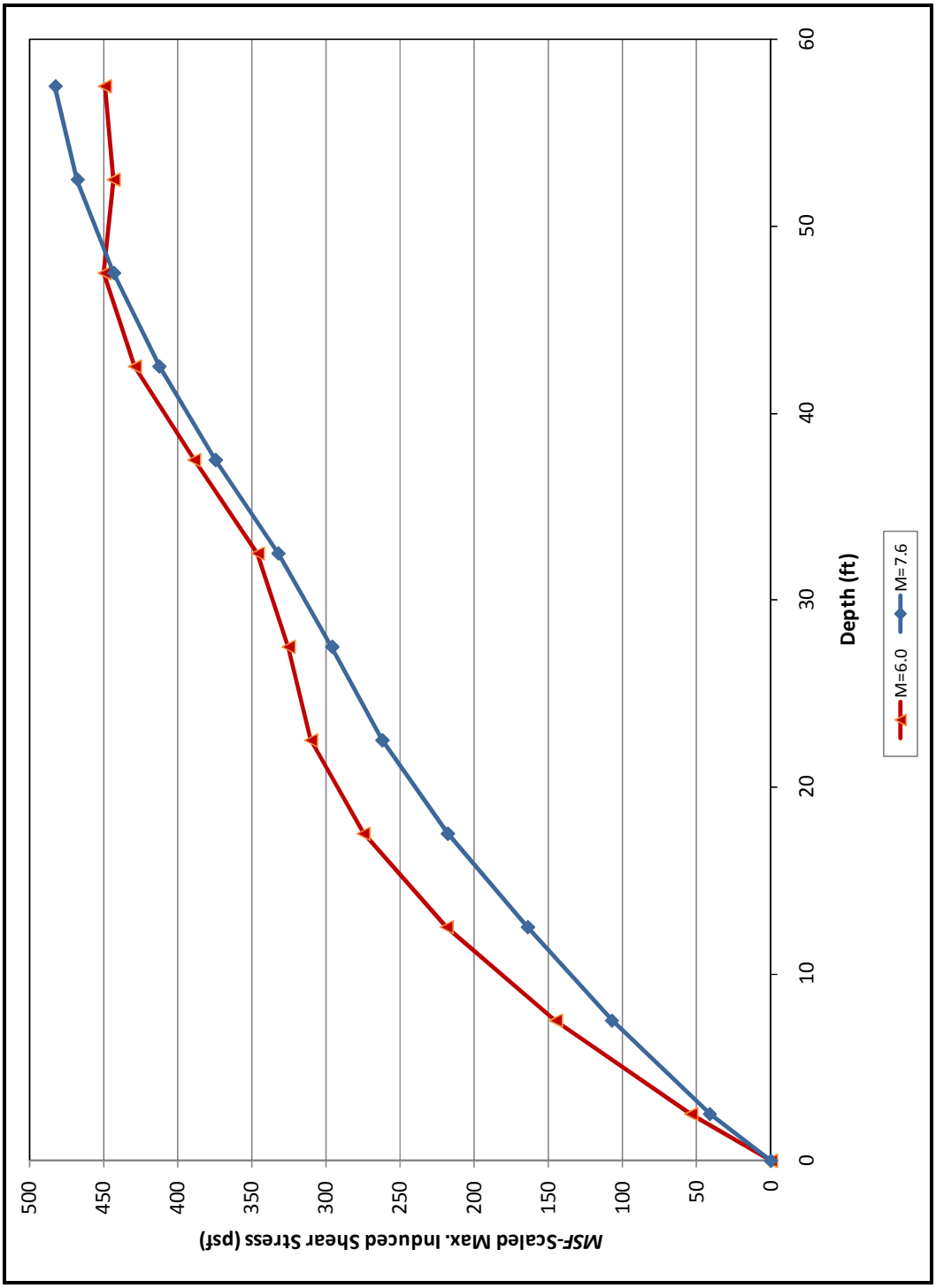

Figure 1.9 – Scaled Maximum Induced Shear Stresses (Profile 1)

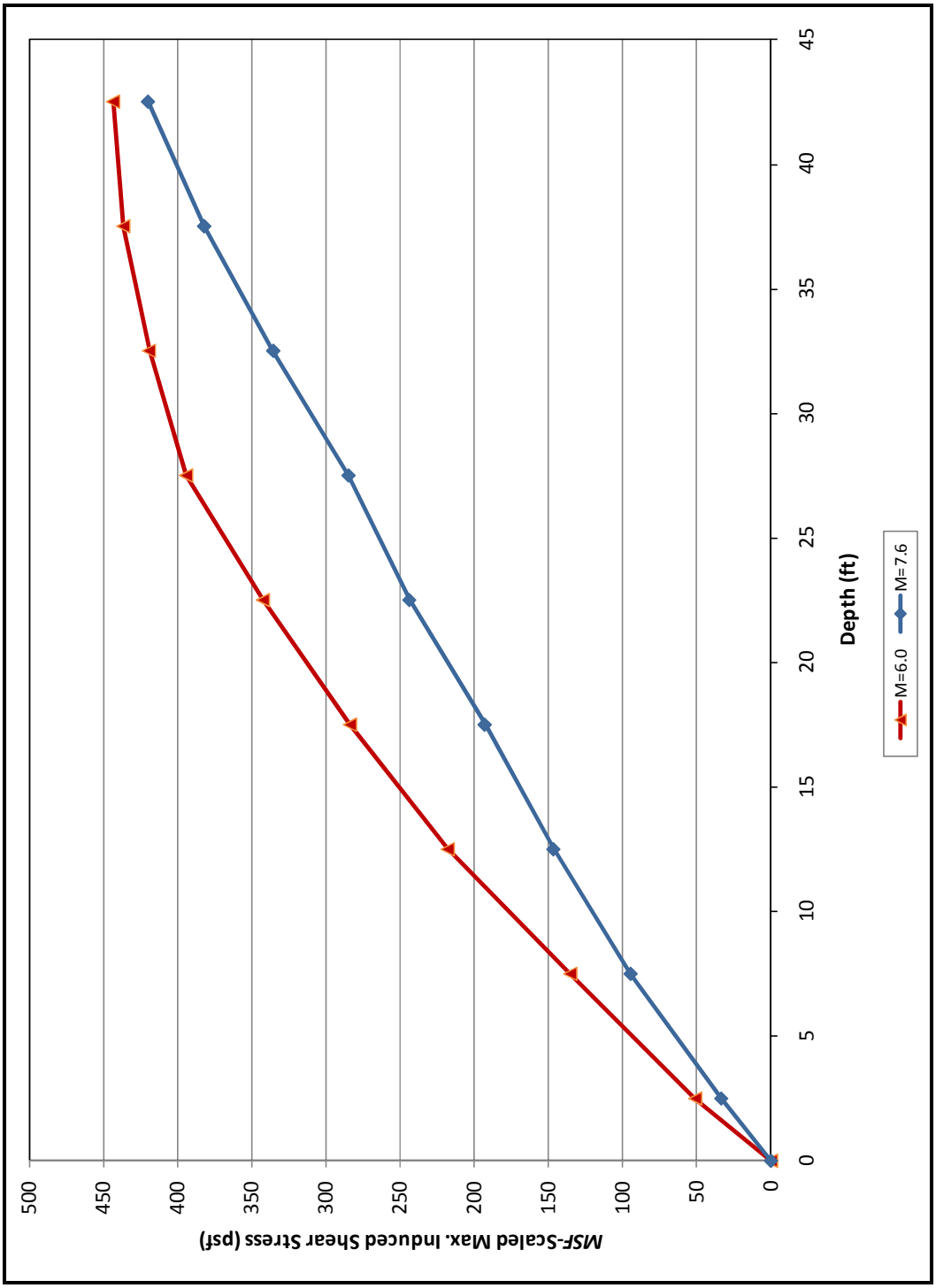

Figure 1.10 – Scaled Maximum Induced Shear Stresses (Profile 2)

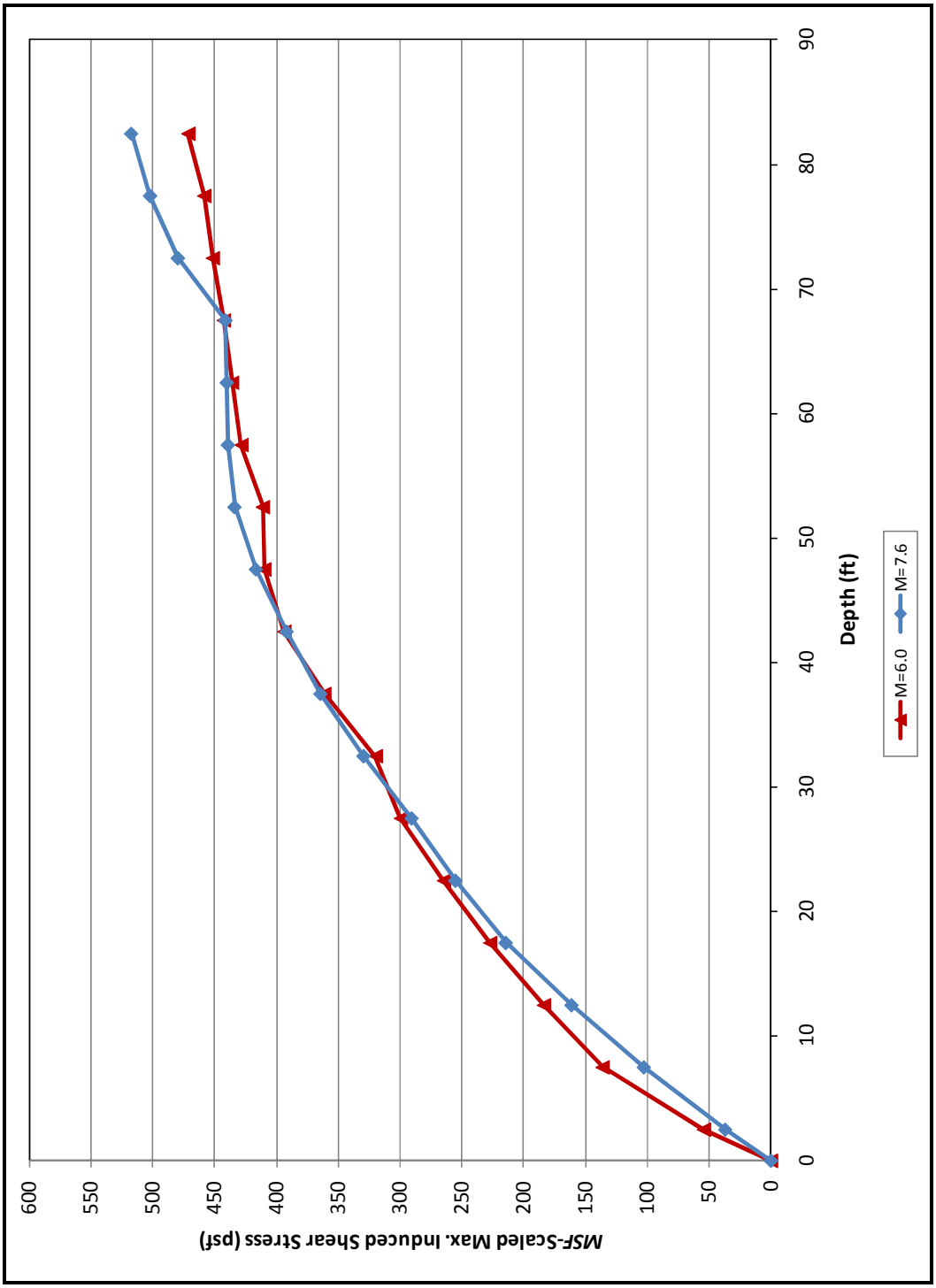

Figure 1.11 – Scaled Maximum Induced Shear Stresses (Profile 3)

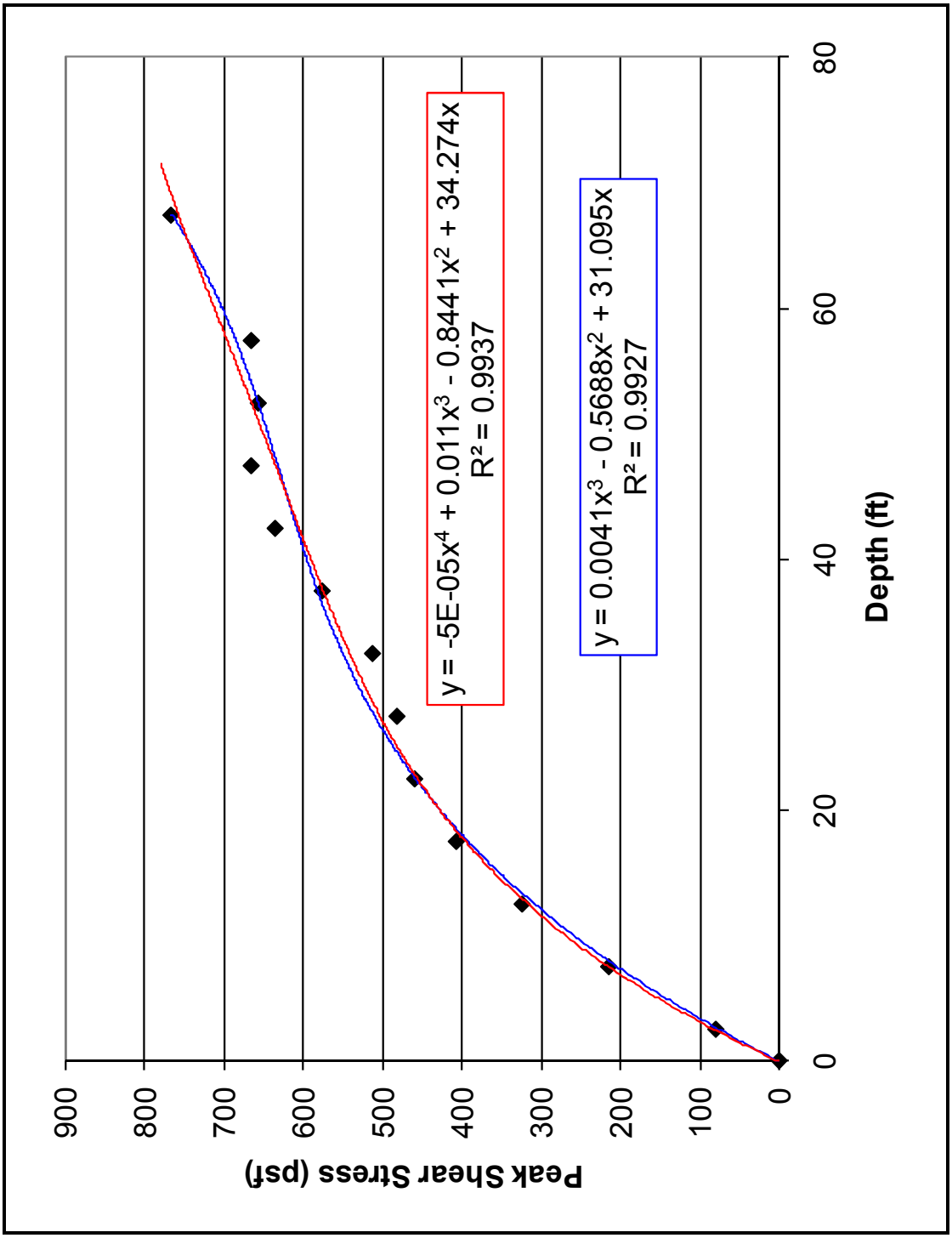

Figure 1.12 – Curve Fit to Define Maximum Induced Shear Stress vs. Depth (Profile 1, M_w=6.0)

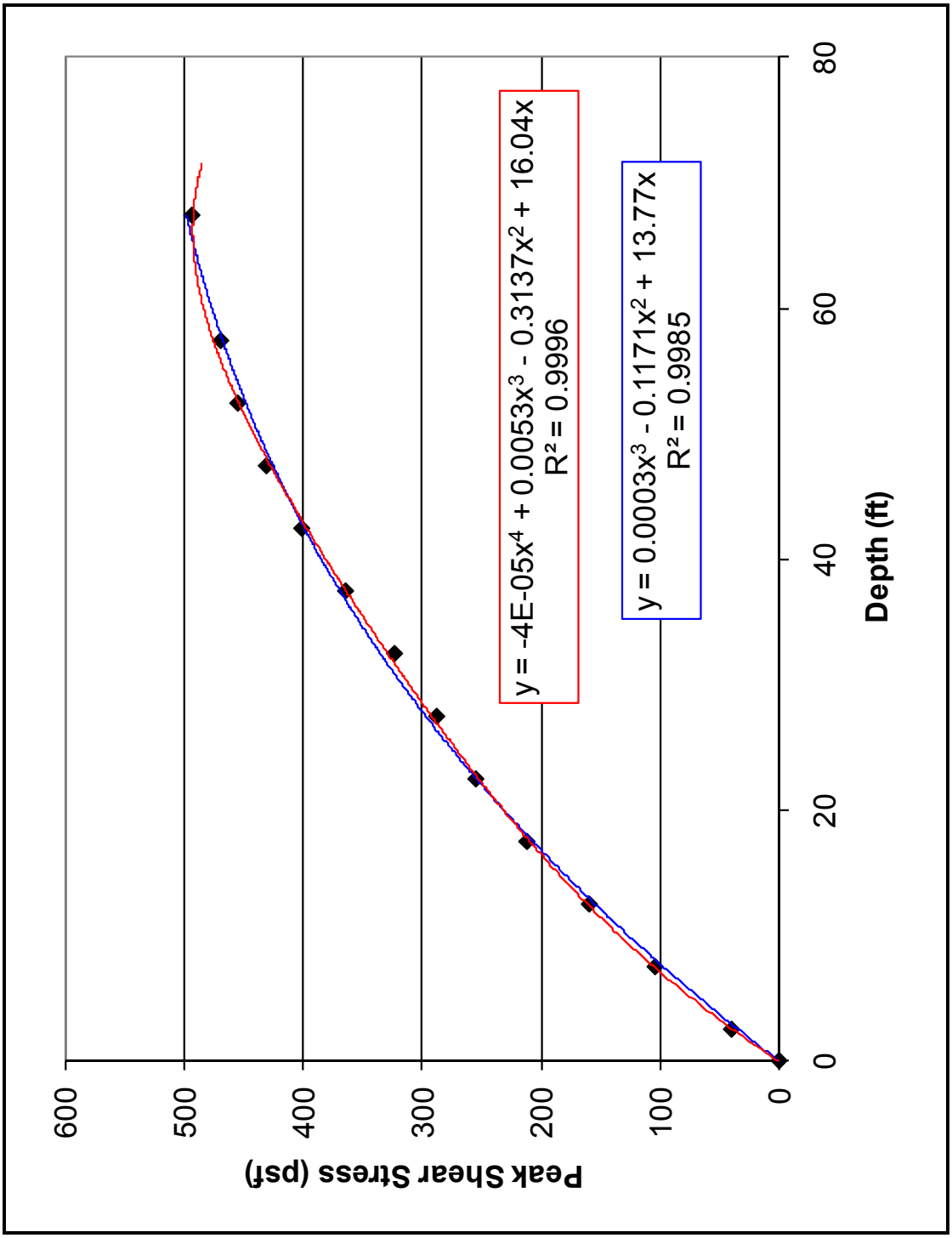

Figure 1.13 – Curve Fit to Define Maximum Induced Shear Stress vs. Depth (Profile 1, M_w=7.6)

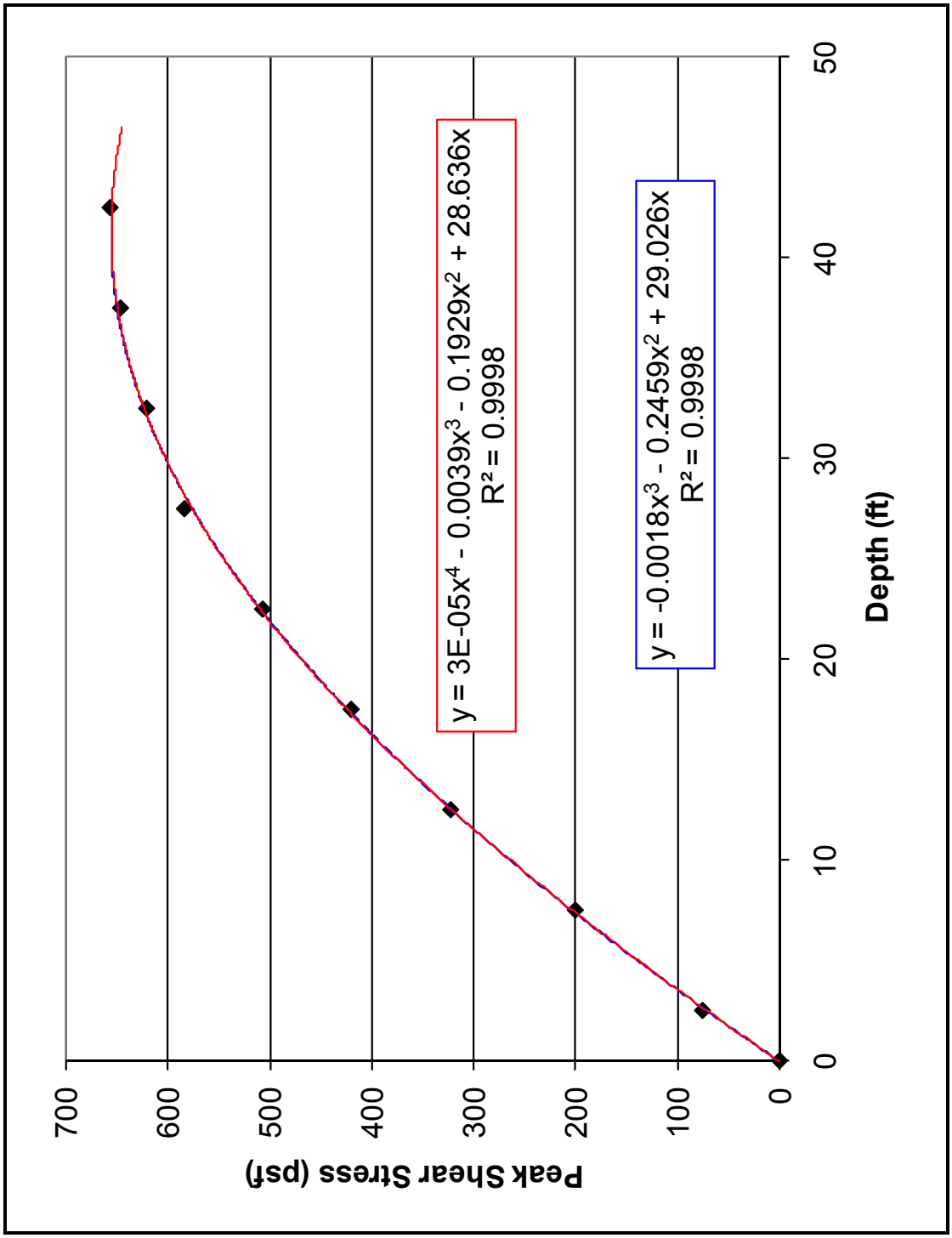

Figure 1.14 – Curve Fit to Define Maximum Induced Shear Stress vs. Depth (Profile 2, M_w=6.0)

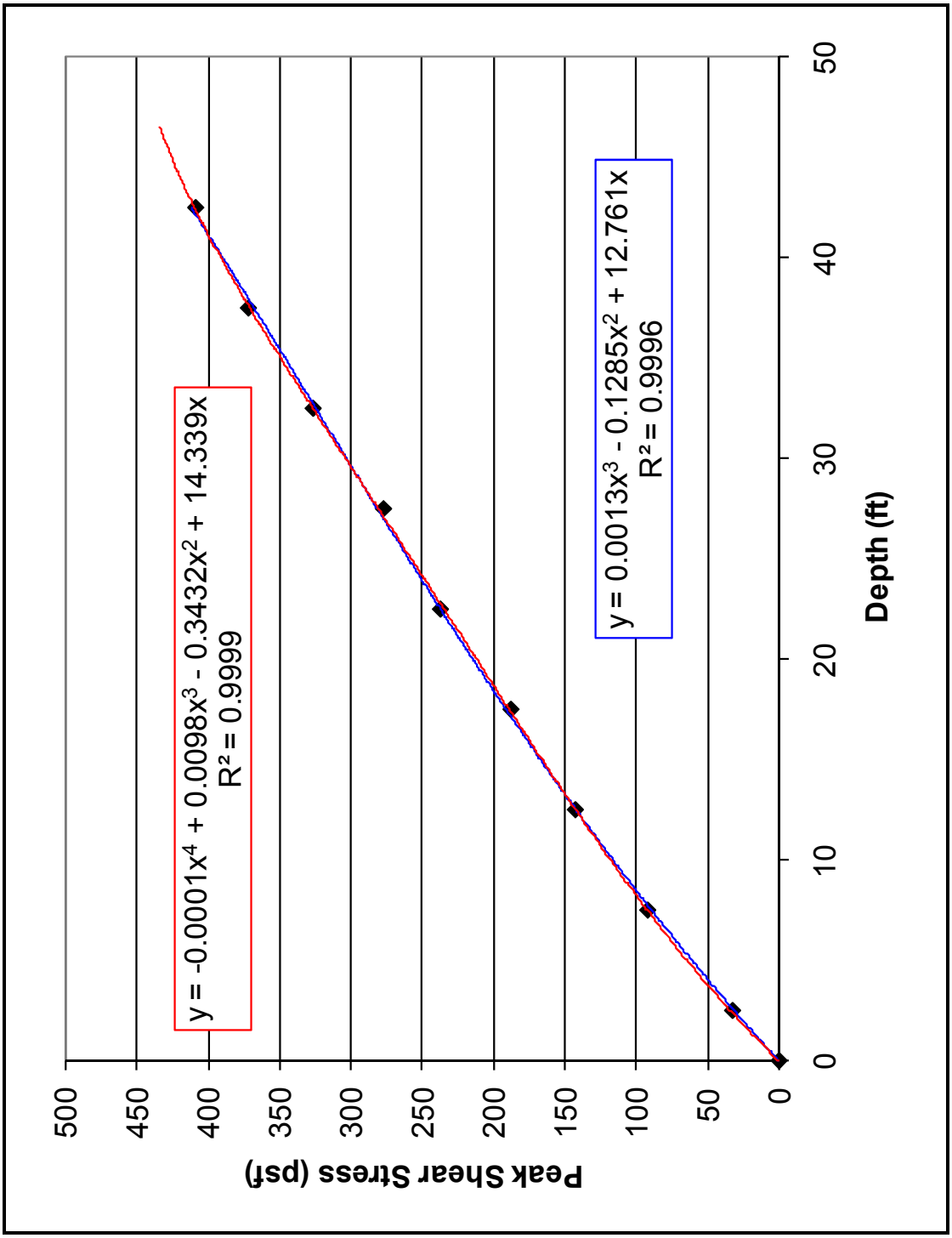

Figure 1.15 – Curve Fit to Define Maximum Induced Shear Stress vs. Depth (Profile 2, M_w=7.6)

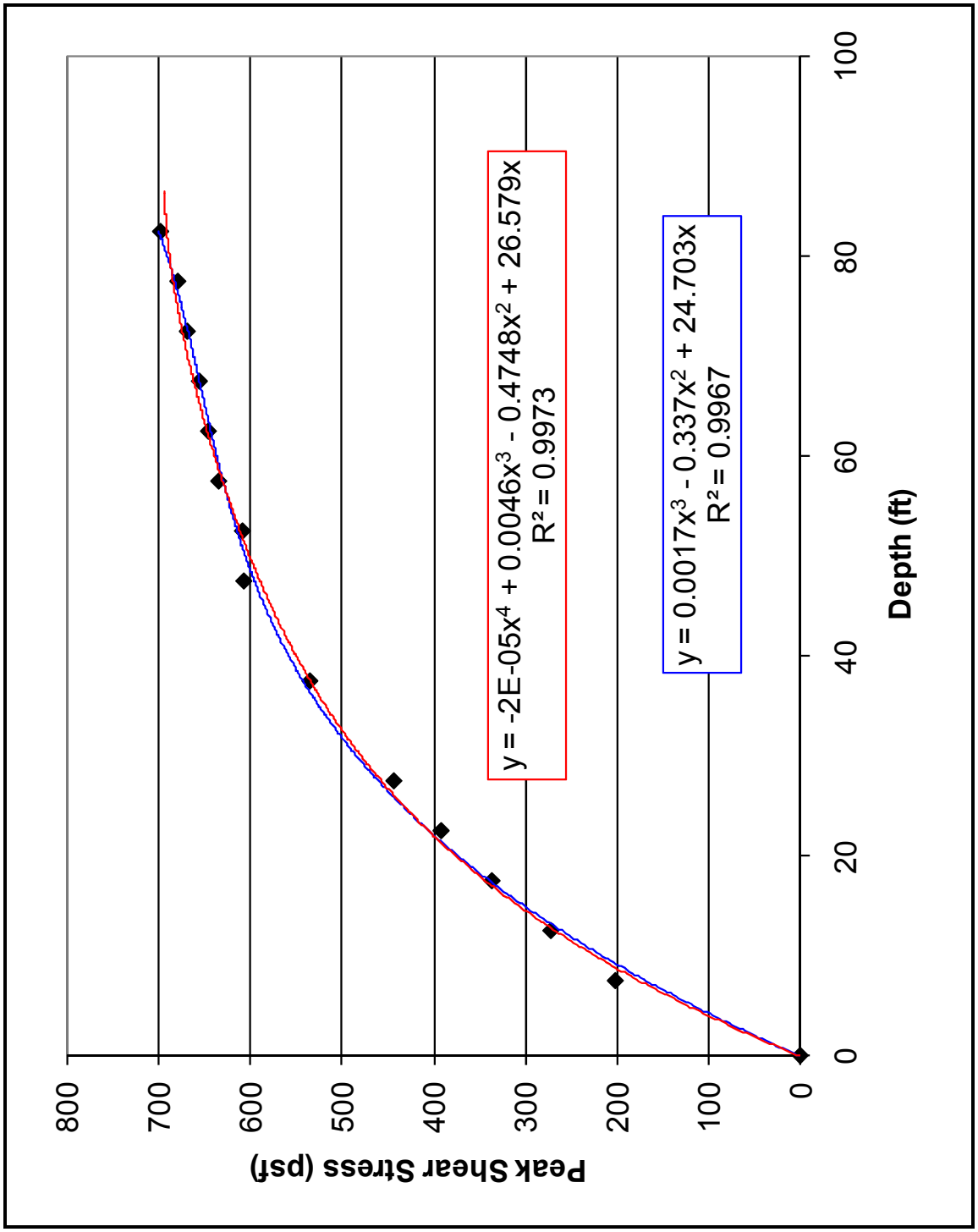

Figure 1.16 – Curve Fit to Define Maximum Induced Shear Stress vs. Depth (Profile 3, M_w=6.0)

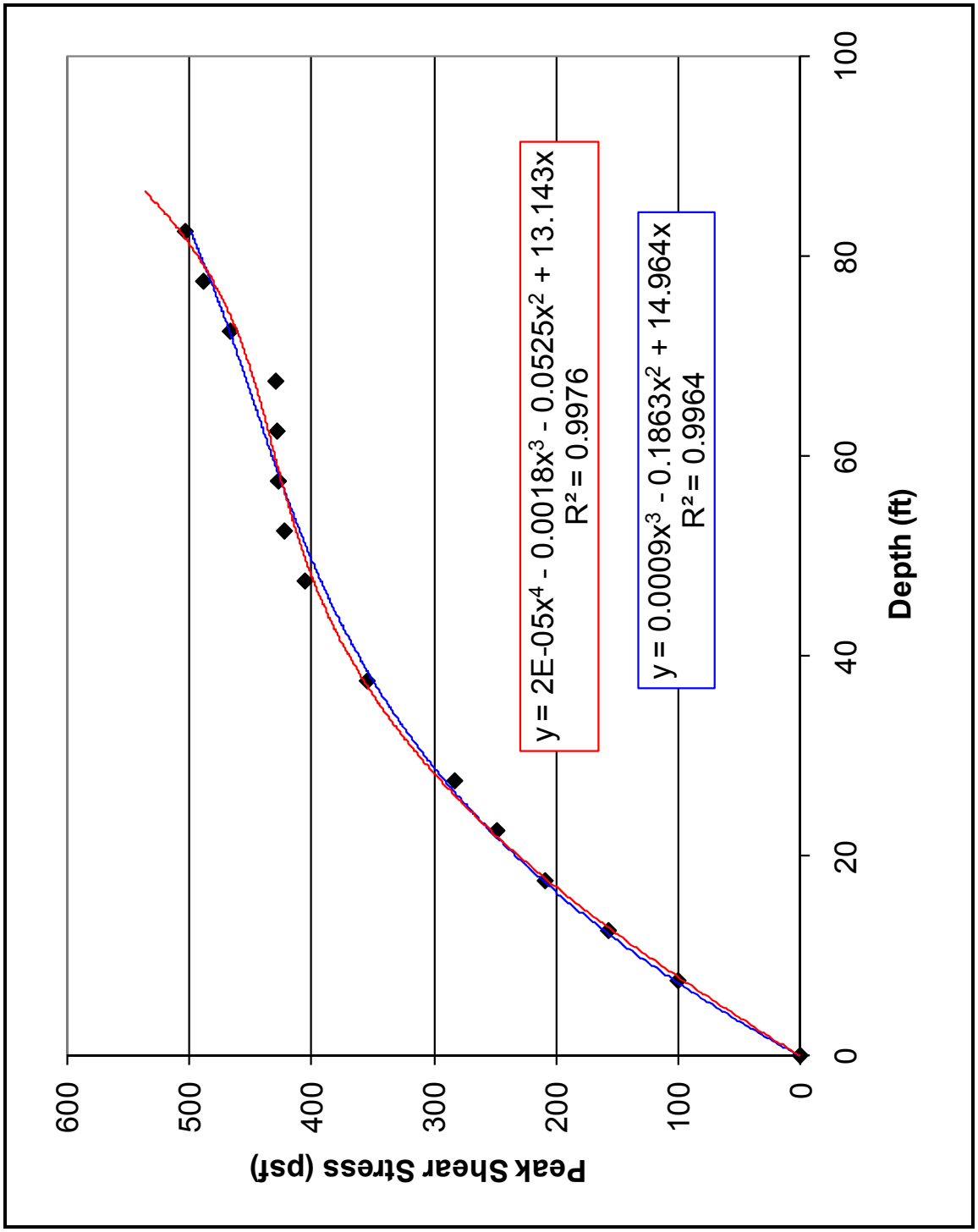

Figure 1.17 – Curve Fit to Define Maximum Induced Shear Stress vs. Depth (Profile 3, M_w=7.6)

References

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Idriss, I. M. and Boulanger, R. W. (2008), *Soil Liquefaction During Earthquakes*. Publication No. MNO-12, Earthquake Engineering Research Institute, Oakland, California.

Kramer, S. L. (1996), *Geotechnical Earthquake Engineering*. Prentice Hall, Upper Saddle River, New Jersey.

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