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Determination of Modulus Reduction and Damping Curves for Bonneville Soils

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Technical Abstract
This work verifies earlier work that found that Bonneville silty clays have significantly higher elastic threshold strains than most soils with the same plasticity. This means, that Bonneville soils exhibit to elastic behavior, and low damping, to higher strain levels, than other similar soils. A modulus reduction and damping model for Bonneville soils was developed based upon resonant column and torsional shear testing of 15 Bonneville silty clays. Site response analyses were performed using this new model along with two commonly used generic models. These analyses did not show that the Bonneville soil model to consistently predict higher shaking levels. However, the new model does more accurately characterize the unique behavior of Bonneville silty clays.

Nontechnical Abstract
Earlier work by the authors suggested that Lake Bonneville soil deposits that underlie much of the urban Wasatch Front might behave differently than other similar soils during earthquake shaking. It was inferred that these differences might lead to higher ground shaking levels than previously predicted. This work verified the unique behavior of Bonneville soils, and provides a model to more accurately predict the behavior of these soils. However, in modeling ground shaking levels using the new model, and more general models there was not a consistent trend predicting higher shaking levels using the new model.

Introduction

In 2004, Bay and Sasanakul performed resonant column and torsional shear tests on seven samples of Bonneville silty clays as part of USGS NEHRP award number 04HQGR0055. This work showed that Bonneville silty clay exhibit elastic behavior at much higher strains than would be predicted by any commonly used soil models. During wave propagation due to earthquake loading, soils exhibiting elastic behavior have very low attenuation due to material damping. Therefore, it was surmised that the ground shaking levels at sites underlain by Bonneville silty clays might larger than would be predicted using conventional soil models. This work was funded in 2009 to test additional soils, to develop modulus reduction and damping predictions for Bonneville soils, and to evaluate the effect of the Bonneville soil behavior on ground shaking levels.

Eight additional soil samples were obtained from sites on the Wasatch Front, and resonant column and torsional shear tests were performed on those specimens. These test were very consistent with the 2004 tests, showing elastic behavior occurring at relatively high strains. The results of all of the laboratory tests were then used to develop predictive models for modulus reduction and damping of Bonneville soils. These models were then compared to two generic predictive models, particularly Darendeli (2001) and Vucetic and Dobry (1991). It was observed that elastic threshold for Bonneville soils occurred at higher strains than the generic models predicted, and that damping in the Bonneville soils remained at low levels at strains beyond the elastic threshold. However, above the elastic threshold, damping increased more rapidly with increasing strain than the generic models predicted, and reached levels similar to or higher than the generic model predictions at high strains.

In order to evaluate the effects of the unique behavior of Bonneville soils on ground shading levels, an equivalent linear site response analysis was performed using the soil profile at the Salt Lake City and County Building. The earthquake motion was a single event, with fairly uniform frequency content, scaled from very low to high levels of shaking. The modulus reduction and damping in the upper 200 ft of the soil profile was modeled using the predictive relationships developed for Bonneville soils, Darendeli's (2001) predictive relationships, and Vucetic and Dobry's (1991) predictive relationships. Over all levels of shaking, and at all spectral accelerations the three predictive models produced similar ground motions, typically within ±10% of each other. Further, ground shaking levels using the Bonneville predictive relationships were not consistently higher than those obtained using the generic relationships. Therefore, it is concluded, that the unique elastic behavior of Bonneville silty clays does not significantly increase the seismic hazard at sites underlain by those soils.

This work does provide researchers and engineers with modulus reduction and damping predictions specific to Bonneville silty clays. These relationships will provide somewhat better predictions of ground shaking in site response analyses than would be obtained using generic predictive relationships.

The first section of this report characterizes the properties of the soils used to develop the modulus reduction and damping relationships for Bonneville silty clays. The second section compares the elastic threshold strain of Bonneville soils to conventional predictions, showing the unique behavior of Bonneville soils. The third section shows the modulus reduction and damping predictive relationships developed for Bonneville soils. The forth section compares the Bonneville predictive relationships to generic

predictive relationships. The fifth section presents the site response analyses using the Bonneville and generic predictive relationships. And the final sections contains the conclusions and recommendations. The results of resonant column and torsional shear tests from this study are presented in the appendix. The results of resonant column and torsional shear tests from the 2004 study are presented in that report.

Properties of Bonneville Soils Used in Investigation

Table 1 provides a summary of the soils used in this investigation. Soils were tested at their in situ mean effective stress, and at higher consolidation pressures. Because in this study, we are interested in predicting the behavior of soils at their in situ conditions, only the testing at the in situ effective stresses were used in this investigation.

Figure 1 is a plasticity chart showing the Atterberg Limits of the all of the soils. All of the soils, except two are classified as CL or ML. The two exceptions are soil from Logan and Nibley that classified as CH. These two soils are not typical of Bonneville silty clays, and later it will be shown that they have elastic threshold strains more like generic soils than Bonneville soils, therefore these two soils were not included in the data used to develop predictive models for Bonneville soils.

The soils that were used in developing predictive models classify as ML or CL, have Liquid Limits (LL) between 27 and 50, and Plasticity Indexes (PI) that plot between 3.3 above the A-line and 5.4 below the A-line. In the authors' experience, this range of plasticity is typical of that encountered for Bonneville soils. The average PI of all soils included investigation is 16.5. Again would be typical average value for all Bonneville soils.

Figure 2 shows the mean effective stresses and overconsolidation ratios of the soils used in this investigation. As is expected for soils that are slightly overconsolidated due to aging, overconsolidation ratios (OCR) are higher for shallow soils (at low mean effective stresses), and lower for deeper soils. The error bars represent the uncertainty in values of overconsolidation ratio. The soils used in this investigation have in situ mean effective stresses less than 2.0 atm, and OCR's between 1.3 and 2.7.

Elastic Threshold Strain of Bonneville and Generic Soils

Figure 3 shows a typical shear modulus reduction curve for soils. Three important strain levels are shown on the plot (Santamarina, 2001, Richart, Hall, and Woods, 1970). The first important threshold strain is the linear threshold, γ^{th}_{linear}. This threshold strain is the strain where $G/G_{max} = 0.99$. At strains below the linear threshold all deformations occur at the contacts between soil particles, with no rotation of soil particles, nor sliding between particles. Very little energy is dissipated due to straining below the linear threshold. The next important threshold is the elastic threshold, $\gamma^{th}_{elastic}$. The elastic threshold for most soils occurs at $G/G_{max} = 0.8$. Between the linear and elastic thresholds all deformation occurs due to deformation at soil contacts and rotation of soil particles, with no sliding between particles. Below the elastic threshold, soils exhibit no contractive nor dilative behavior, typical of elastic materials. Again, little energy is dissipated due to straining below the elastic threshold. Above the elastic threshold, particle sliding occurs, along with contraction or dilation of the soil skeleton. Large amounts of energy are dissipated due to sliding and dilation. The last strain identified on Figure 3 is the reference strain, γ_{ref}. The reference strain occurs at G/G_{max} =

Table 1 Properties of soils tests for investigation of modulus reduction and damping behavior of Bonneville silty clays

Site	Project Year	Sample Depth, ft	Liquid Limit, LL	Plasticity Index, PI	Unified Soil Class.	In Situ Mean Effective Stress, σ'_0, atm	Estimated OCR
Geneva Rd Bridge over Provo River, Provo, UT	2004	37	43	20	CL	0.84	1.4-1.8
Geneva Rd Bridge over Provo River, Provo, UT	2004	67	33	11	CL	1.41	1.3-1.5
Geneva Rd Bridge over Provo River, Provo, UT	2004	93	41	10	ML	1.90	1.2-1.3
2835 S. 1000 W, Nibley, UT	2004	24	63	34	CH	0.50	1.8-2.3
USU Drainage Farm Site, Logan,UT	2004	14	65	34	CH	0.42	1.9-2.6
BYU Site West of Airport, SLC, UT	2004	14	42	17	CL	0.53	1.7-2.2
BYU Site West of Airport, SLC, UT	2004	35	27	2	ML	0.90	1.4-1.7
500 W.500 S., SLC UT	2009	21	45	22	CL	0.33	2.1-3.0
500 W.500 S., SLC UT	2009	43	30	9	CL	0.60	1.6-2.1
900 S. 50 W., SLC, UT	2009	25	38	15	CL	0.50	1.8-2.3
900 S. 50 W., SLC, UT	2009	22	43	19	CL	0.47	1.8-2.4
700 W. Antelope Dr., Layton, UT	2009	19	41	18	CL	0.44	1.9-2.5
700 W. Antelope Dr., Layton, UT	2009	19	42	19	CL	0.41	1.9-2.6
200 E. 1600 N., Logan, UT	2009	22	43	21	CL	0.41	1.9-2.6
200 E. 1600 N., Logan, UT	2009	25	48	24	CL	0.44	1.9-2.5

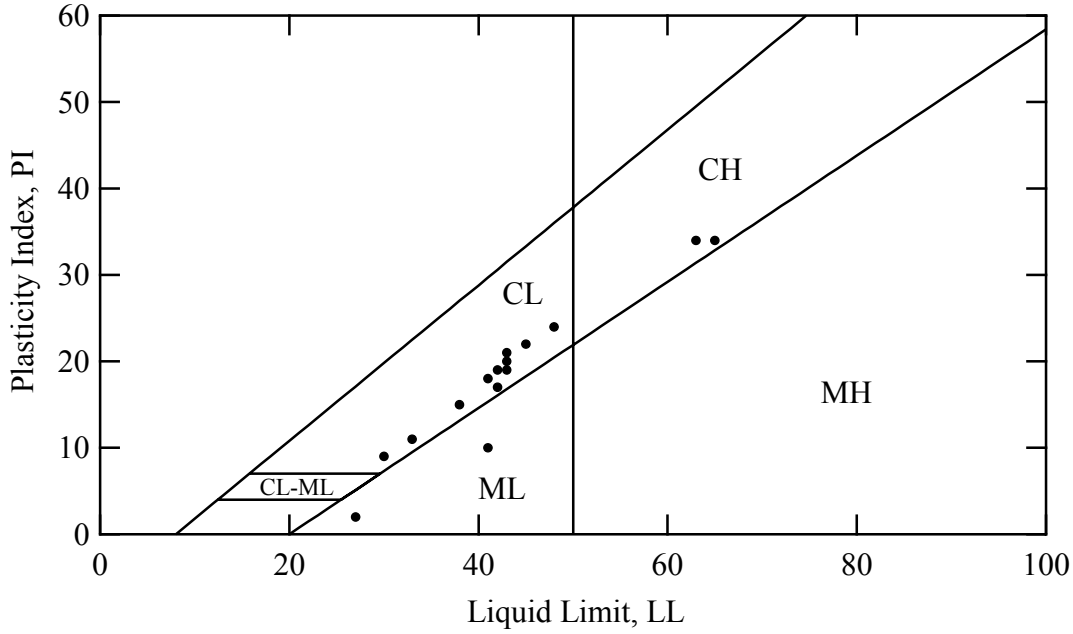

Figure 1 Plasticity Chart Showing Atterberg Limits of Soils used in Investigation of Bonneville Silty Clays

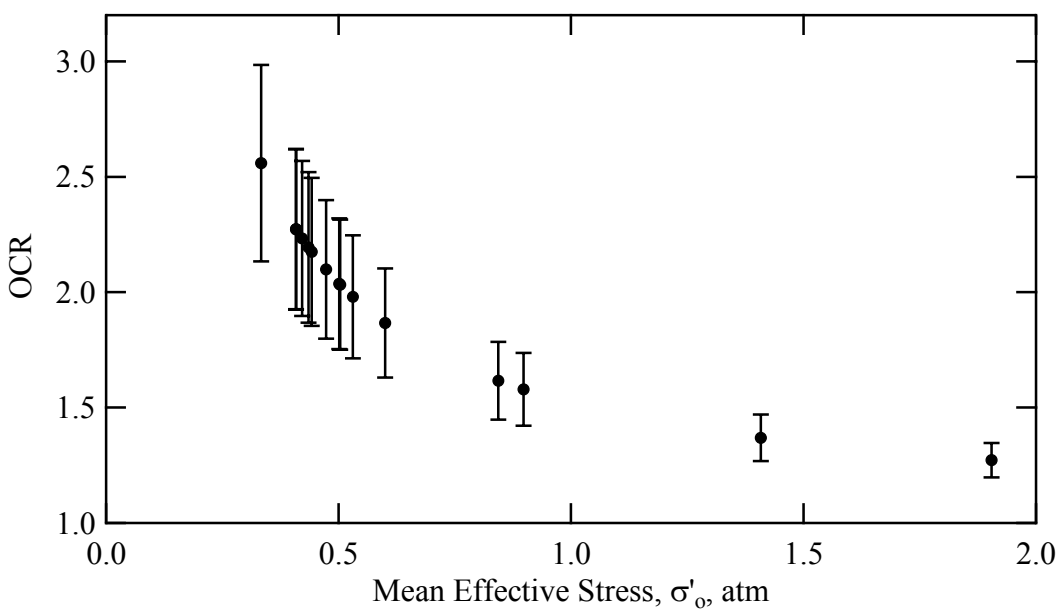

Figure 2 Mean Effective Stresses and Overconsolidation Ratios of Soils used in Investigation of Bonneville Silty Clays

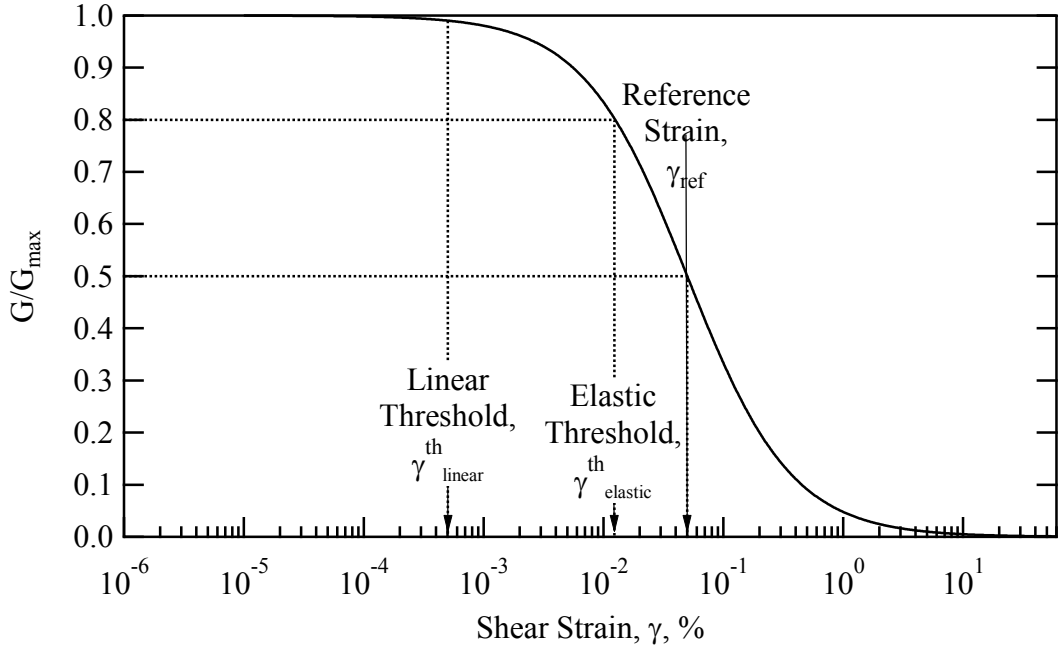

Figure 3 Typical Shear Modulus Reduction Curve for Soils

0.5, and is a parameter in the 2 and 3 parameter hyperbolic models that will be discussed in the following section.

Because the elastic threshold is dividing line between low and high energy dissipation, it is a good parameter for comparing the elastic behavior of soils. Figure 4 shows the elastic threshold strain versus PI for all soils included in this study. It also shows the elastic threshold strains predicted by Darendeli (2001), and Vucetic and Dobry (1991). In the Figure 4 it can be seen that nearly all of the Bonneville soils have significantly higher elastic threshold strains than predicted using the generic model. The exceptions are the two CH soils (from Logan and Nibley) which Vucetic and Dobry prediction, and one soil from Layton that has a lower value than either generic prediction. It is also evident in the Figure 4 that elastic threshold strains vary widely in Bonneville soils, and that a small portion of that variation can be attributed to PI.

Observed elastic threshold strains led to the hypothesis that earthquake ground shaking levels at sites underlain by Bonneville soils might be larger than would be predicted using generic modulus reduction and damping predictions. This hypothesis did not prove to be true, as will be shown in later sections.

Modulus Reduction and Damping Curves for Bonneville Silty Clays

The modulus reduction and damping curves presented in this section are based upon torsional shear tests performed in 2004 and 2009. In both sets of testing, only 5 cycles of loading were applied to the soil to minimize sample disturbance in subsequent tests. The 2004 tests were performed at a frequency of 0.333 Hz, and the 2009 tests at 0.1 Hz. Both frequencies are sufficiently low that dynamic affects were negligible. Only tests performed at the in situ mean effective stress were used in developing these predictions. Complete test results the 2004 report and in the appendix to this report.

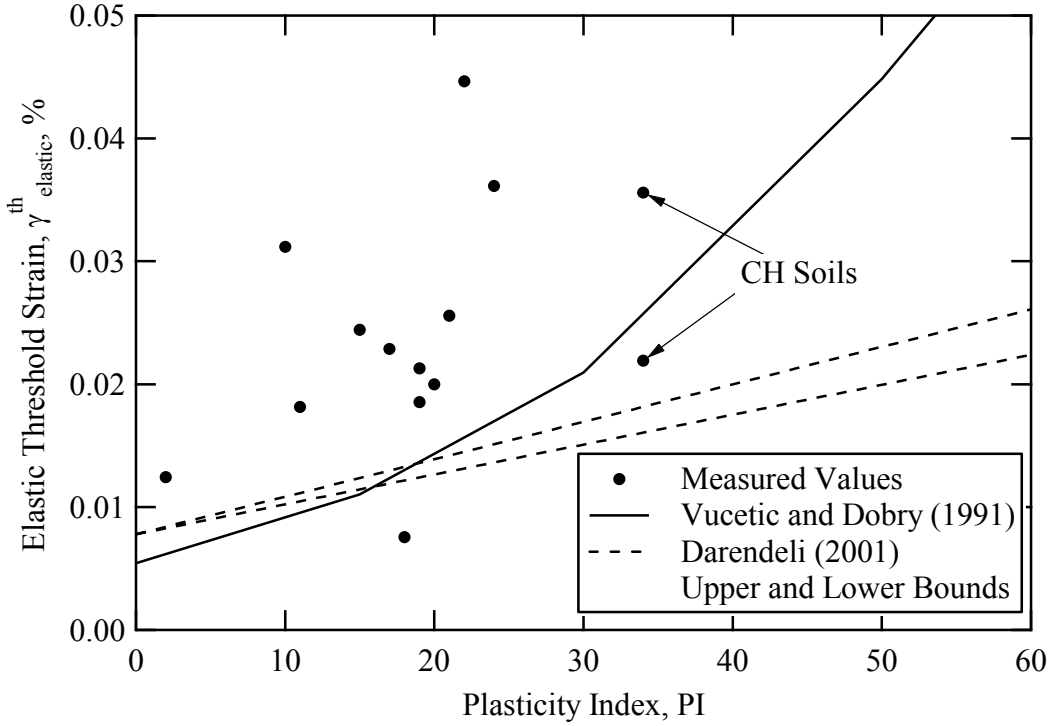

Figure 4 Measured Elastic Threshold Strains in Bonneville Soils Along with Predicted Values from Vucetic and Dobry (1991) and Darendeli (2001)

Modulus Reduction Curves

Two basic models are commonly used to describe stress strain behavior in soils. The first is the two parameter hyperbolic model. The equation describing modulus reduction is:

$$\frac{G}{G_{max}} = \frac{1}{1 + \frac{\gamma}{\gamma_{ref}}} \tag{1}$$

where: G is shear modulus,
G_{max} is the maximum low strain shear modulus,
γ_{ref} is the reference shear strain shown in Figure 3, and
γ is shear strain.

The two parameters required to describe the shear modulus of a soil at any strain are G_{max} and γ_{ref}.

The second is the three parameter hyperbolic model. This is the model used by Darendeli (2001). The equation describing modulus reduction is:

$$\frac{G}{G_{max}} = \frac{1}{1 + \left(\frac{\gamma}{\gamma_{ref}}\right)^{a}} \tag{2}$$

where: a is the coefficient of curvature.

The three parameters required to describe the shear modulus of a soil at any strain are G_{max} γ_{ref}, and a.

The torsional shear data was analyzed using the stress integration approach (Sasanakul and Bay, 2008). This approach involves assuming a stress-strain model, and uses curve fitting to match the measured torque-rotation from torsional shear or resonant column tests. In the analysis we used the two parameter hyperbolic model for all tests, and the three parameter hyperbolic model for some of the tests. Table A.1 shows the curve fitting parameters from all tests. A very good match to the experimental data was obtained using both models.

The mean coefficient of curvature from all tests is a = 0.993, with a standard deviation of 0.134. By comparison, Darendeli (2001) found a mean value of a = 0.919 for all soils. Because the mean value of a for Bonneville soils is so close to 1.0, which makes the two and three parameter hyperbolic models identical, and because the two parameter hyperbolic model consistently provide a very good match to the experimental data, the two parameter hyperbolic model is used to characterize Bonneville soils.

The first required parameter is maximum shear modulus, G_{max}. Figure 5 shows all of the measured values of G_{max} versus in situ mean effective stress. These laboratory values will almost certainly underpredict the field values of G_{max} because G_{max} is very sensitive to small amounts of sample disturbance. Site specific seismic analyses should use in situ measurements to determine values of G_{max}.

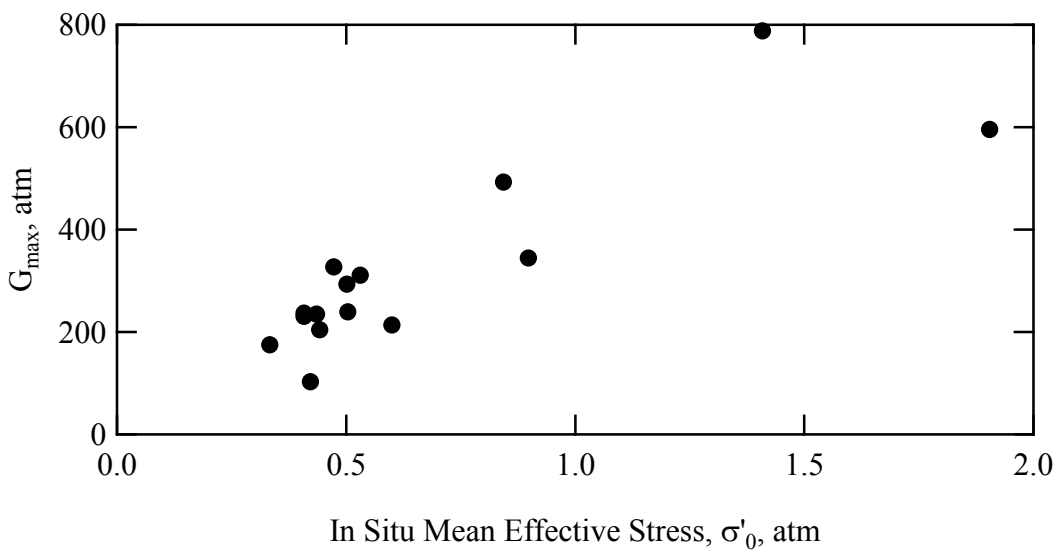

Figure 5 Laboratory Measurements of G_{max} in Bonneville Soils

The second required parameter is reference strain, γ_{ref}. All researchers have found that threshold strain in soils tends to increase with increasing PI. Darendeli (2001) also found smaller effects with threshold strain increasing with mean effective stress, σ'_0, and also increasing with OCR. To evaluate the correlations between reference strain and those parameters the measured threshold strains are plotted relative to PI, in situ mean effective stress, and OCR in Figures 6,7, and 8, respectively.

Figure 6 shows an increasing trend in reference strain with increasing PI, as is expected. However, there is a significant amount of variance in reference strain that cannot be attributed to PI. A best fit line has the equation:

$$\gamma_{ref} = 0.06974 + 0.001561 \times \mathrm{PI} \,. \qquad (3)$$

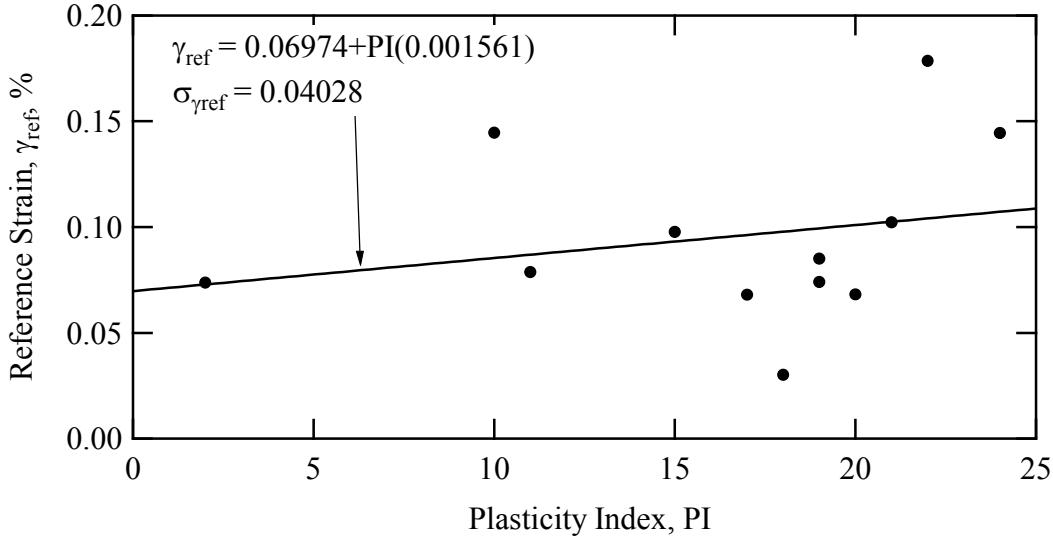

Figure 6 Laboratory Measurements of γ_{max} relative to PI in Bonneville Soils

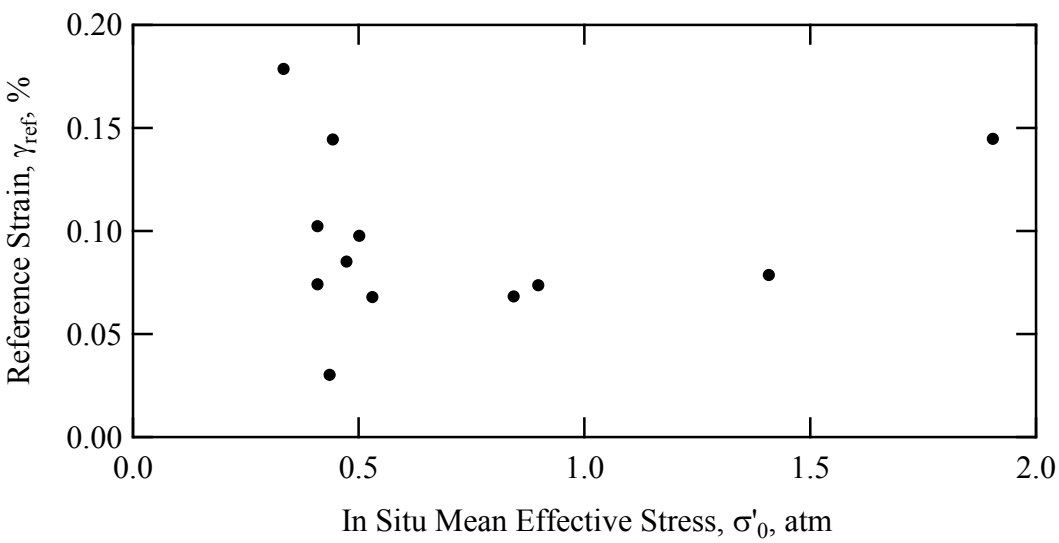

Figure 7 Laboratory Measurements of γ_{max} relative to In Situ Mean Effective Stress in Bonneville Soils

The standard deviation of the reference strain relative to the predicted value is:

$$\sigma_{\gamma_{ref}} = 0.04028. \quad (4)$$

Figure 7 shows the reference strain relative to in situ mean effective stress. There is not a clear trend of reference strain increasing with in situ mean effective stress. However, it should be noted that the range of stresses in the sample of soils included in this study and in actual Bonneville soils in the field is small, as Bonneville soils are in shallow near-surface deposits.

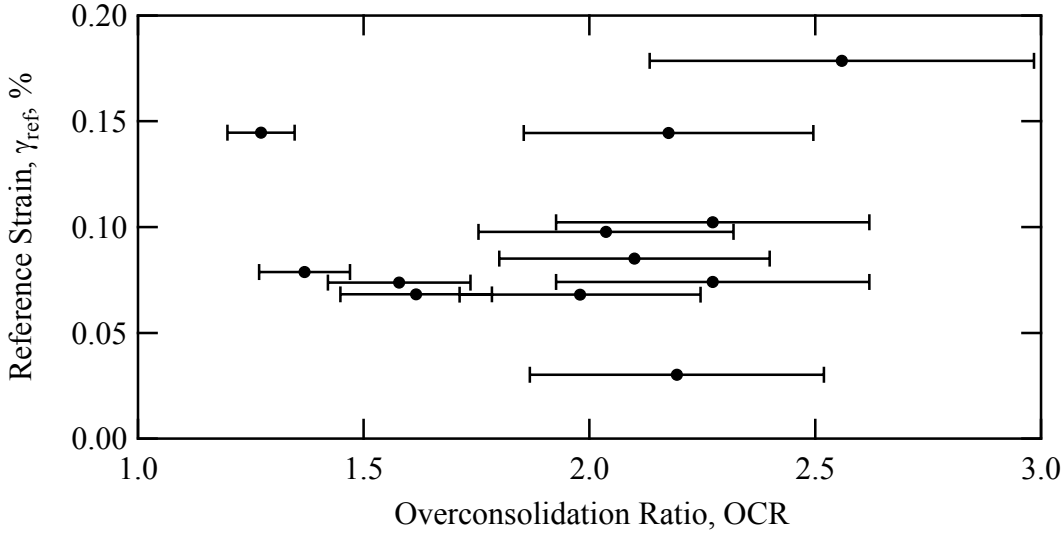

Figure 8 Laboratory Measurements of γ_{max} relative to OCR in Bonneville Soils

Figure 8 shows the reference strain relative to OCR. Again, there is not a clear trend of reference strain increasing with OCR. And again, it should be noted that the range of OCR of soils included in this study and in actual Bonneville soils in the field is also small.

Figure 9 shows all of the modulus reduction curves compared to the model prediction. Shear stain values for each measurement are normalized with respect to the predicted reference strain from equation 4 for each soil. The dashed show the predicted variance of ± 1 σ.

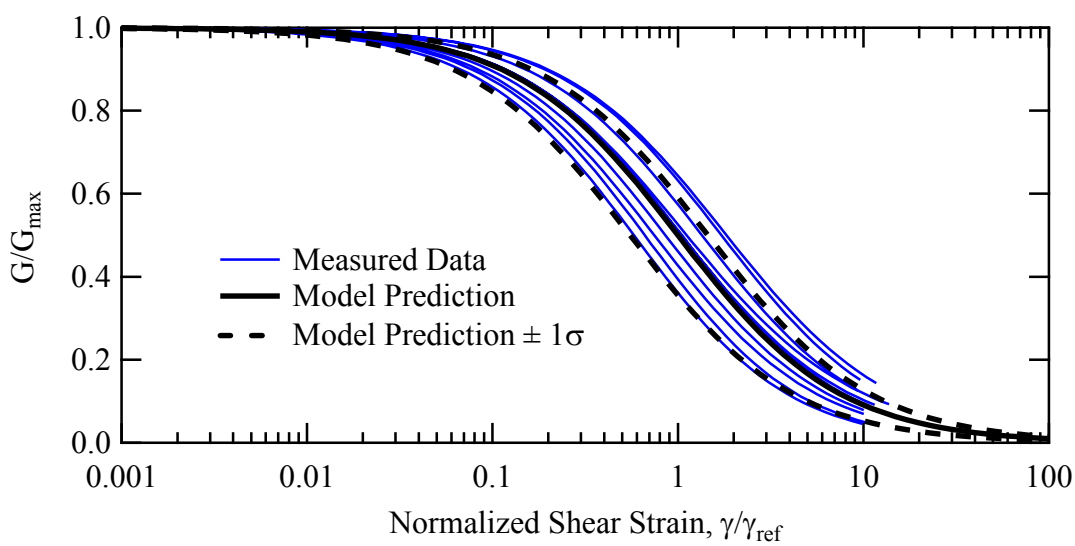

Figure 9 Model Prediction and Measured Values of Modulus Reduction for Bonneville Soils

Damping Curves

A general model for damping is soils is:

$$D = D_{min} + D_{hys}(\gamma), \tag{5}$$

where: D is the total damping,

D_{min} is low strain damping , and

$D_{hys}(\gamma)$ is the hysteretic damping that increases with shear strain.

For the two parameter hyperbolic model, if the soil exactly behaved according to Masing rules for cyclic loading, then the hysteretic damping would be (Ishihara, 1996):

$$D_{hys}(\gamma) = 100\% \times \left[\frac{4}{\pi}\left(1+\frac{1}{\frac{\gamma}{\gamma_{ref}}}\right)\left(1-\frac{\ln\left(1+\frac{\gamma}{\gamma_{ref}}\right)}{\frac{\gamma}{\gamma_{ref}}}\right)-\frac{2}{\pi}\right]. \tag{6}$$

Real soil does not exactly following Masings rules for cyclic loading. The hysteresis loops in real soils are smaller than Masing would predict. Interestingly, for Bonneville soils, the hysteretic damping remains low to strains much higher than predicted by equation 6.

Figure 10 shows damping measurements from all of the tests conducted at high enough strains to measure hysteretic damping (about half of the tests). The strains were once again normalized with respect to the reference shear strain predicted using equation 3. The average minimum damping ratio is D_{min} = 1.126%, with a standard deviation of σ = 0.147. Equation 6 was modified to obtain a best fit to the measured damping value as follows:

for $\gamma/\gamma_{ref} \leq 1.5$,

$$D_{hys}(\gamma)=125\% \times \left(1-0.00894\ln(N)\right)\times \left[\frac{4}{\pi}\left(1+\frac{1}{\frac{\gamma}{1.5\times\gamma_{ref}}}\right)\left(1-\frac{\ln\left(1+\frac{\gamma}{1.5\times\gamma_{ref}}\right)}{\frac{\gamma}{1.5\times\gamma_{ref}}}\right)-\frac{2}{\pi}\right]^{1.128},$$

and for $1.5 < \gamma/\gamma_{ref} \leq 10$

$$D_1=125\% \times \left[\frac{4}{\pi}(2)(1-\ln(2))-\frac{2}{\pi}\right]^{1.128}, \text{ and,}$$

$$D_{hys}(\gamma)=\left(1-0.00894\ln(N)\right)\times \left[D_1+\left(21-D_1\right)\times\left(\frac{\ln\left(\frac{\gamma}{\gamma_{ref}}\right)}{2.996}\right)^{0.5}\right], \tag{7}$$

where N is the number loading cycles.

All of the damping values presented are from the first cyclic loading cycle. The terms in equation 7 accounting for effect of number of loading cycles was taken from Darendeli

(2001). Also, the upper limit on damping ratio, was also taken from a combination of measured values and Darendeli's predictions for similar soils.

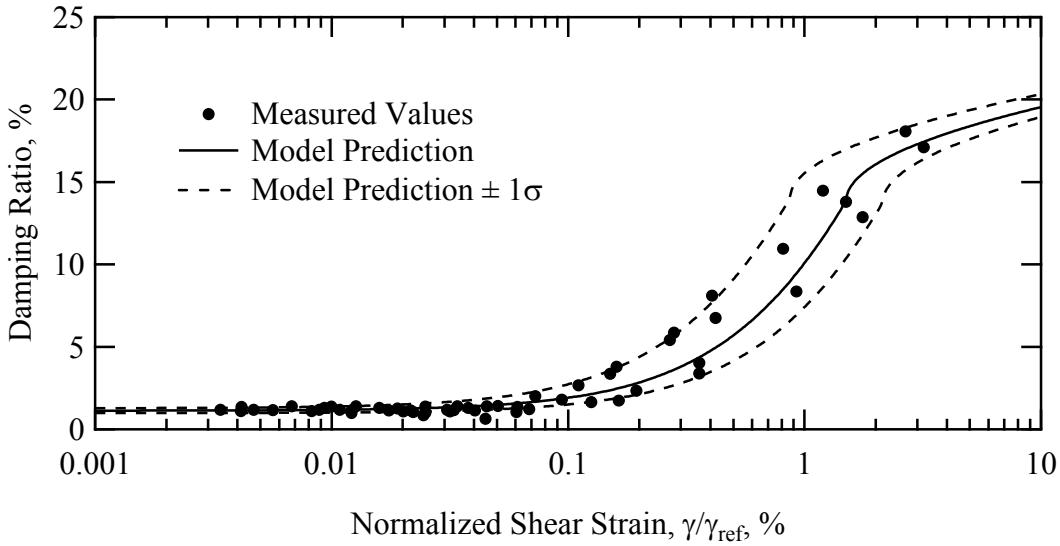

Figure 10 Model Prediction and Measured Values of Damping Ratio for Bonneville Soils

The comparison between model predictions and measured damping values shown in Figure 10 indicate that equation 7 provides a good prediction of damping ratios in Bonneville soils.

Comparison Between Generic and Bonneville Soil Predictive Models

Comparison with Darendeli (2001) Model

In order to compare the Darendeli (2001) and the Bonneville model we need to determine soil properties representing lower, average and upper bounds for the soils tested. Table 2 shows the soil properties used in this comparison.

Table 2 Soil parameters used to compare Darendeli (2001) and Bonneville predictive models

	Lower Bound	Average	Upper Bound
Bonneville Model	PI = 9 γ_{ref} = mean - 1σ D_{min} = mean + 1σ N = 1	PI = 16.5 γ_{ref} = mean D_{min} = mean N = 1	PI = 24 γ_{ref} = mean + 1σ D_{min} = mean - 1σ N = 1
Darendeli (2001) Model	PI = 9 σ'_o = 0.333 OCR = 1.3 N = 1 Freq = 0.333 Hz	PI = 16.5 σ'_o = 1.1 OCR = 2.0 N = 1 Freq = 0.333 Hz	PI = 24 σ'_o = 1.9 OCR = 2.6 N = 1 Freq = 0.333 Hz

Figures 11 and 12 show comparisons between the Bonneville soil model and Darendeli (2001) model for modulus reduction and damping, respectively. Measured values are also shown in each figure. The Bonneville soil model predicts more linear behavior than Darendeli, with lower damping values below shear strains of about 0.12%.

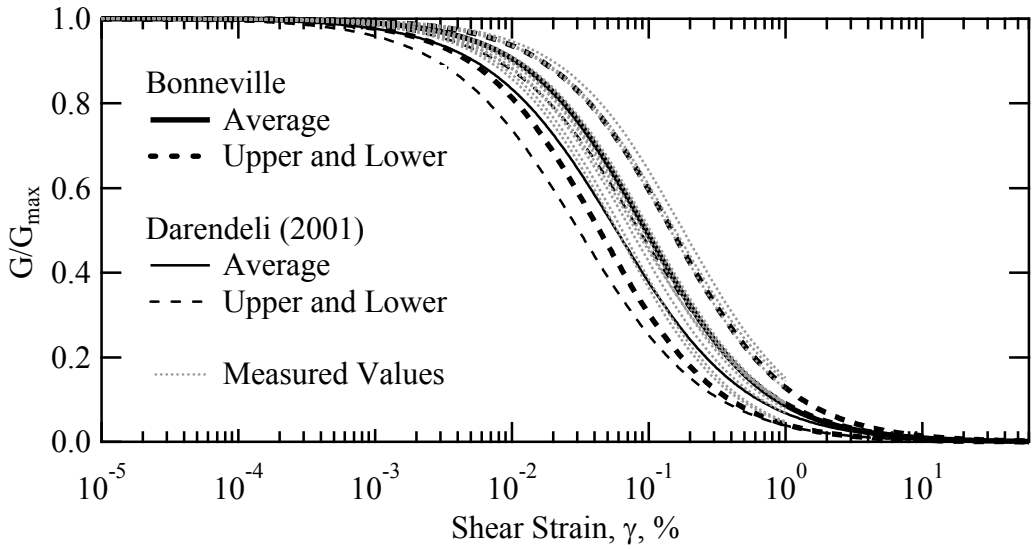

Figure 11 Comparison of Modulus Reduction Predicted by the Bonneville Soil Model and Darendeli (2001)

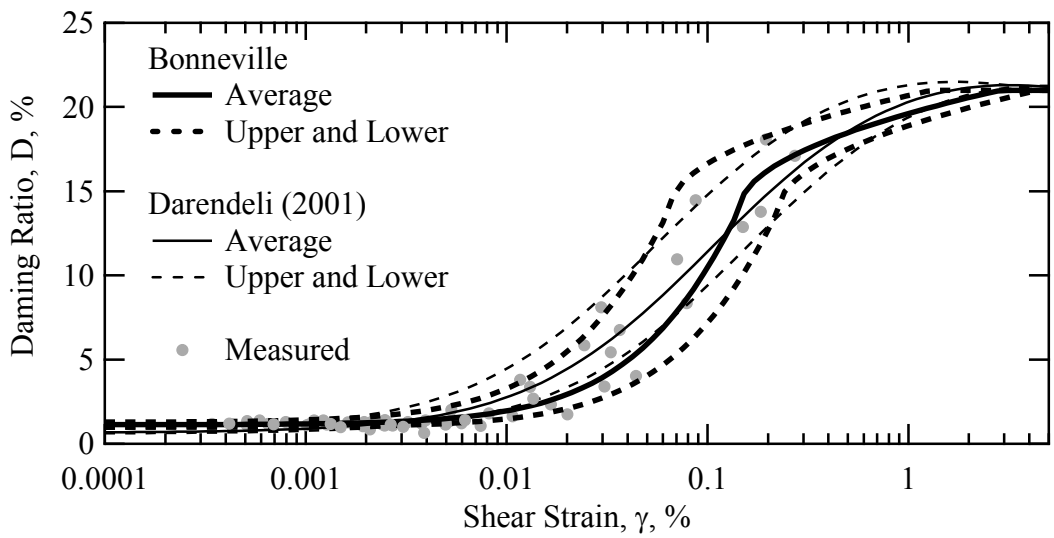

Figure 12 Comparison of Damping Ratios Predicted by the Bonneville Soil Model and Darendeli (2001)

Comparison with Vucetic and Dobry (1991) Model

In order to compare the Vucetic and Dobry (1991) and the Bonneville model we need to determine soil properties representing lower, average and upper bounds for the soils tested. Table 3 shows the soil properties used in this comparison. Vucetic and Dobry only use PI to predict modulus reduction and damping.

Figures 13 and 14 show comparisons between the Bonneville soil model and Vucetic and Dobry (1991) model for modulus reduction and damping, respectively. Measured values are also shown in each figure. The Bonneville soil model predicts more linear behavior than Vucetic and Dobry, with much lower damping values below shear strains of about 0.1%.

Table 3 Soil parameters used to compare Darendeli (2001) and Bonneville predictive models

	Lower Bound	Average	Upper Bound
Bonneville Model	PI = 9 γ_{ref} = mean - 1σ D_{min} = mean + 1σ N = 1	PI = 16.5 γ_{ref} = mean D_{min} = mean N = 1	PI = 24 γ_{ref} = mean + 1σ D_{min} = mean - 1σ N = 1
Vucetic and Dobry (1991) Model	PI = 9	PI = 16.5	PI = 24

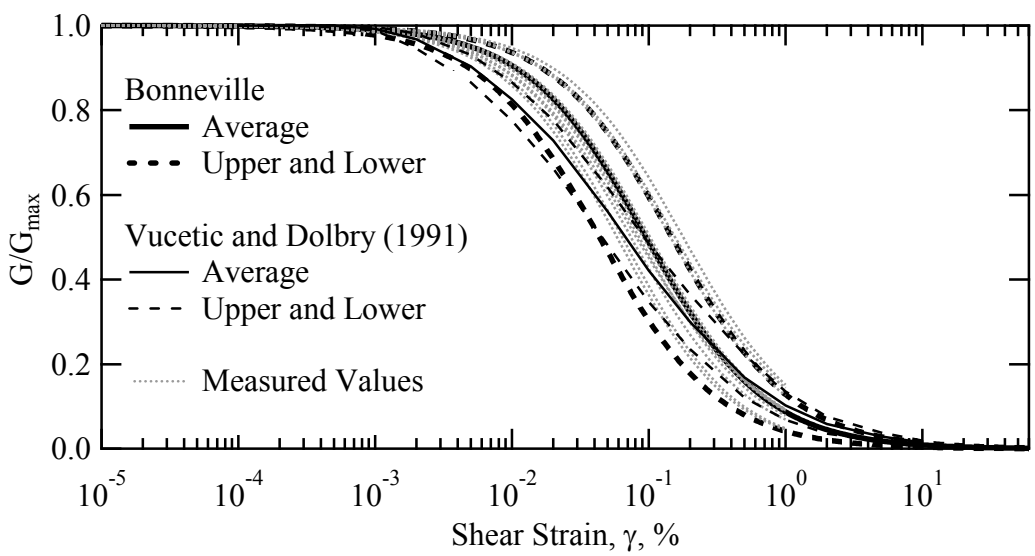

Figure 13 Comparison of Modulus Reduction Predicted by the Bonneville Soil Model and Vucetic and Dobry (1991)

Summary of Model Comparisons

The differences between the Bonneville soils and the generic soil model predictions shown in Figures 11-14 are striking. The Bonneville soils exhibit less modulus reduction and significantly lower damping between strains of 0.001% and 0.1%. Above that strain level, damping is somewhat higher than the generic soil models prediction.

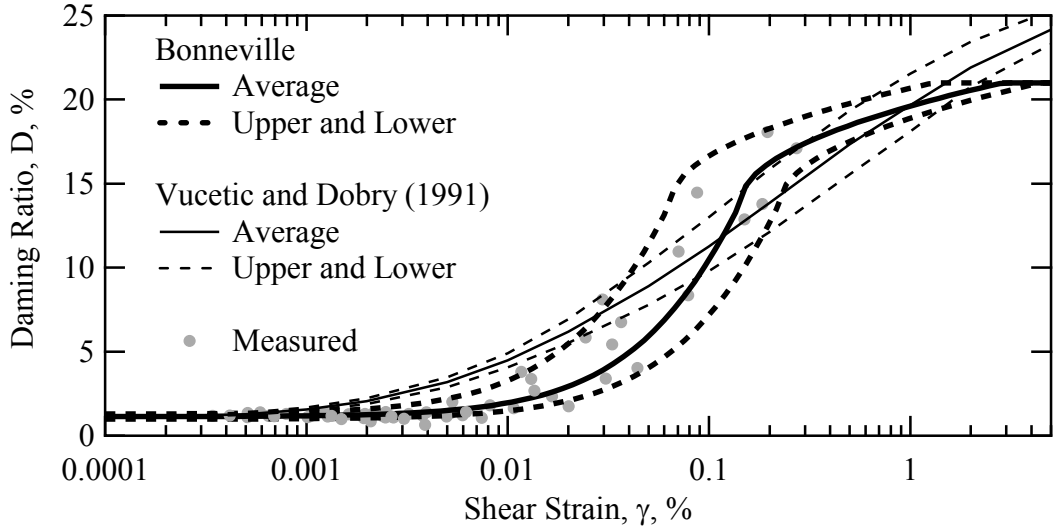

Figure 14 Comparison of Damping Ratios Predicted by the Bonneville Soil Model and Vucetic and Dobry (1991)

Site Response Analyses using the Bonneville and Generic Soil Models

In order to evaluate the effect of increased linearity of Bonneville soils, a site response analysis was performed on a typical Bonneville soil site. The measured shear wave velocity profile in the upper 100 ft from the Salt Lake City and County Building was used for the upper soil profile. A reasonable assumed shear wave velocity profile was used for deeper layers. Figure 15 shows the shear wave velocity profile that was used, along with the soil models used. The upper 200 ft was modeled using the Bonneville soil model, developed in this work, the Darendeli (2001) model, and the Vucetic and Dobry (1991) model. Bedrock was assumed to be at a depth of 1225 ft, and soil below 200 ft was modeled using the Vucetic and Dobry (1991) model with a PI of 30 to represent increased linearity in deep soils.

The earthquake record selected for the analysis is the Rio Dell Overpass record from the $M_w = 7.1$, Cape Mendocino 1992 earthquake. The site is USGS Site Class B. This record was selected because it has somewhat uniform magnitude spectra. In order to evaluate the effect of shaking amplitude, the record was scaled using factors of 0.125, 0.25, 0.5, 0.75, 1.0, 1.25, and 1.5.

The program ProShake was used to perform equivalent linear seismic site response analyses for the site. Figure 16 shows a plot of the effective shear strain ($0.65 \times \gamma_{max}$) profile for each analysis using the Bonneville soil model. The effective shear strain controls the values shear modulus and damping used in each analysis. In Figure 16, it can be seen that only two smallest records have peak effective shear strain levels less than 0.1%. And it is at strains less than about 0.1% that Bonneville soils exhibit low damping levels. All of the higher shaking levels have peak effective shear strain levels in the range that Bonneville soils behave similarly to generic soils.

Response spectra (5% damped, absolute) are shown for the all of the analysis in Figures 17, 18, and 19, for the Bonneville model, the Darendeli (2001) model, and the Vucetic and Dobry (1991) model, respectively.

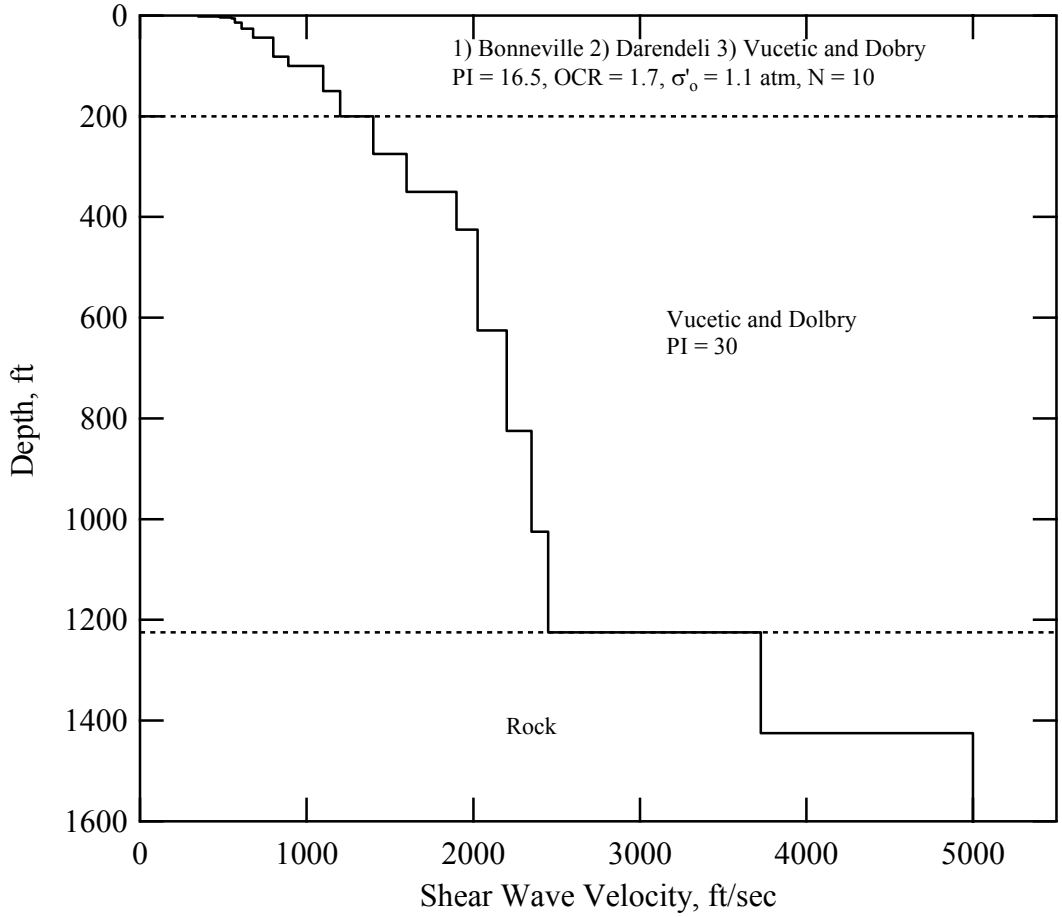

Figure 15 Typical Bonneville Soil Site Shear Wave Velocity Profile used in Site Response Analysis

To provide a comparison between the spectral accelerations, the response spectra were smoothed, and ratios of the spectral accelerations were calculated. Figure 20 shows the ratio of Darendeli/Bonneville, and Figure 21 shows the ratio of Vucetic and Dobry/Bonneville.

In Figure 20, it can be seen that for high shaking levels, and periods less than 2 sec, Darendeli (2001) under-predict spectral accelerations by 10%-20%. For low and moderate levels of shaking at periods less than 1 sec, Darendeli (2001) under-predict spectral accelerations by 10% or less. At periods between 1 and 2 sec Darendeli over-predict spectral accelerations by 10%-20%. At periods greater than 2 sec, Darendeli over-predict spectral accelerations by 5%-15%.

In Figure 21, it can be seen that for high shaking levels, and periods less than 1 sec, Vucetic and Dolbry over-predict spectral accelerations about 10%. For low and moderate levels of shaking at periods less than 1 sec, Vucetic and Dolby under-predict spectral accelerations by 10% or less. At periods between 1 and 2 sec Vucetic and Dobry under-predicts spectral accelerations by 10%, or less for high shaking levels, and over-predict the spectral accelerations by 5% or less for low to moderate shaking levels. At

periods greater than 2 sec, Vucetic and Dobry slightly under-predicts spectral accelerations for high shaking levels, and slightly over-predict for low shaking levels.

Overall, the differences between the Bonneville model and generic models appear to be similar to the differences between the various generic models. Further, the Bonneville model does not consistently predict higher levels of shaking than the generic models. Ground shaking predictions made using the Bonneville soil model should be more accurate because the model better characterizes the unique modulus reduction and damping behavior in Bonneville soils.

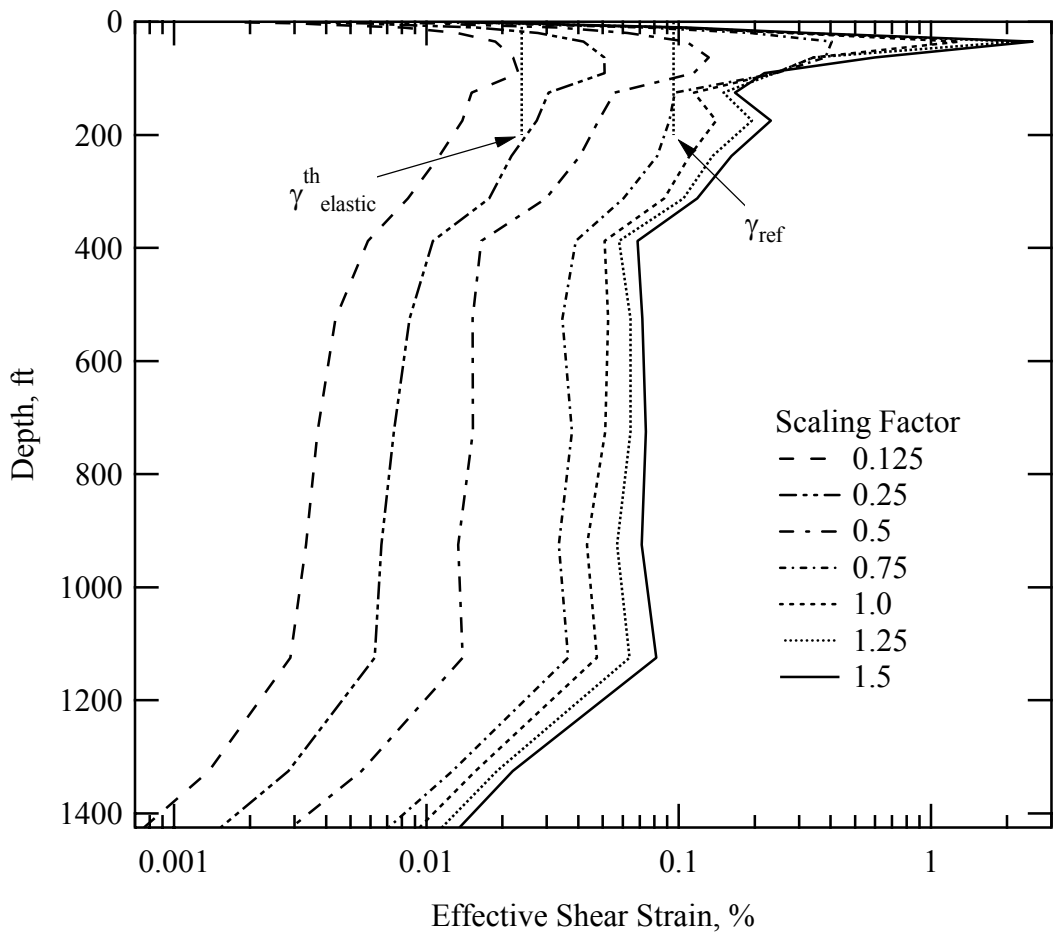

Figure 16 Effective Shear Strain Profiles for All Analyses using the Bonneville Soil Model

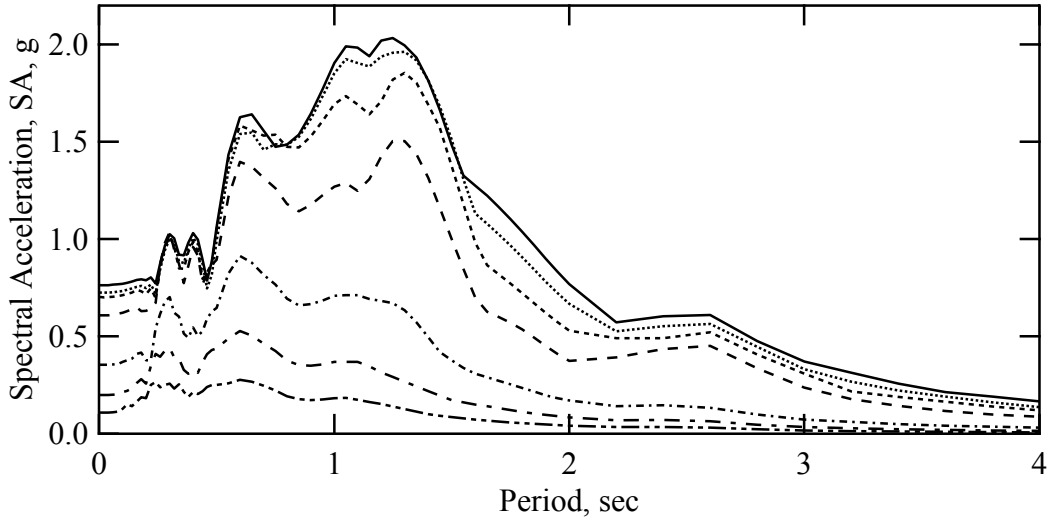

Figure 17 Response Spectra for All Analyses using the Bonneville Soil Model

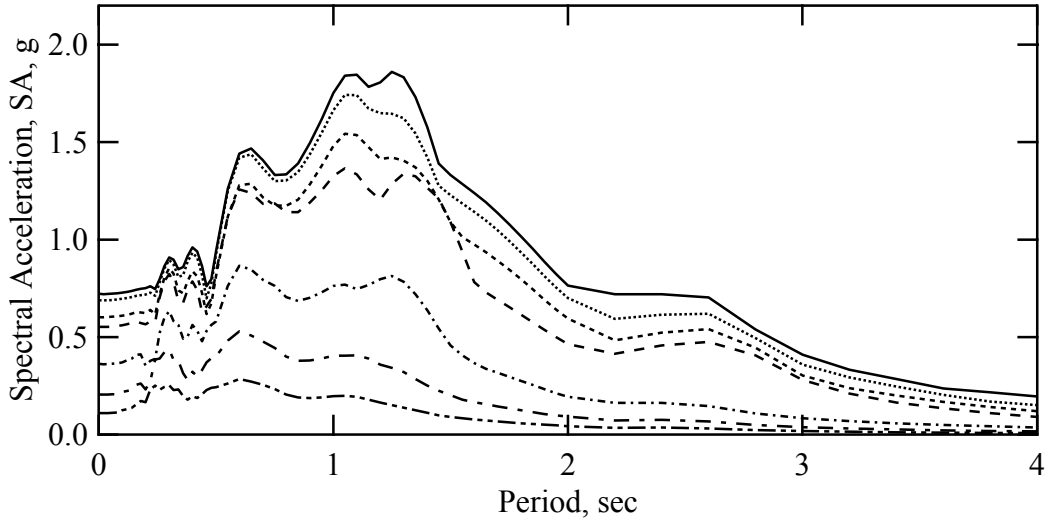

Figure 18 Response Spectra for All Analyses using the Darendeli (2001) Soil Model

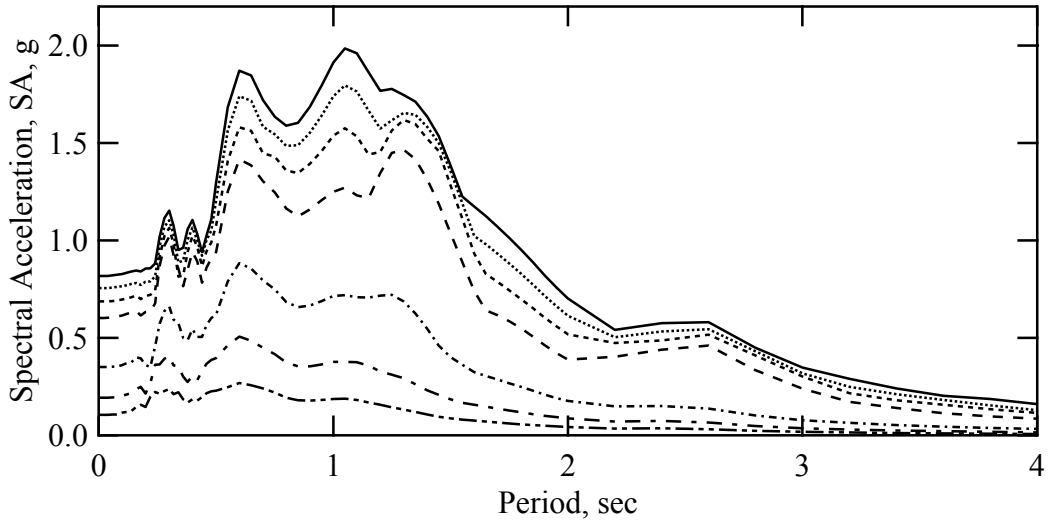

Figure 19 Response Spectra for All Analyses using the Vucetic and Dobry (1991) Soil Model

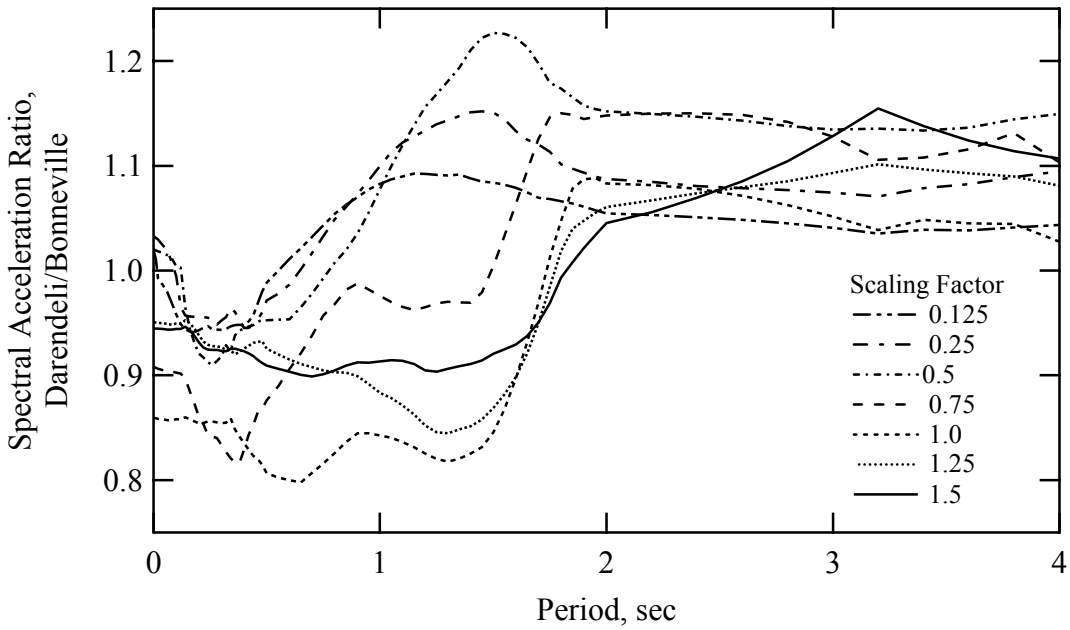

Figure 20 Ratio of Spectral Accelerations, Darendeli/Bonneville

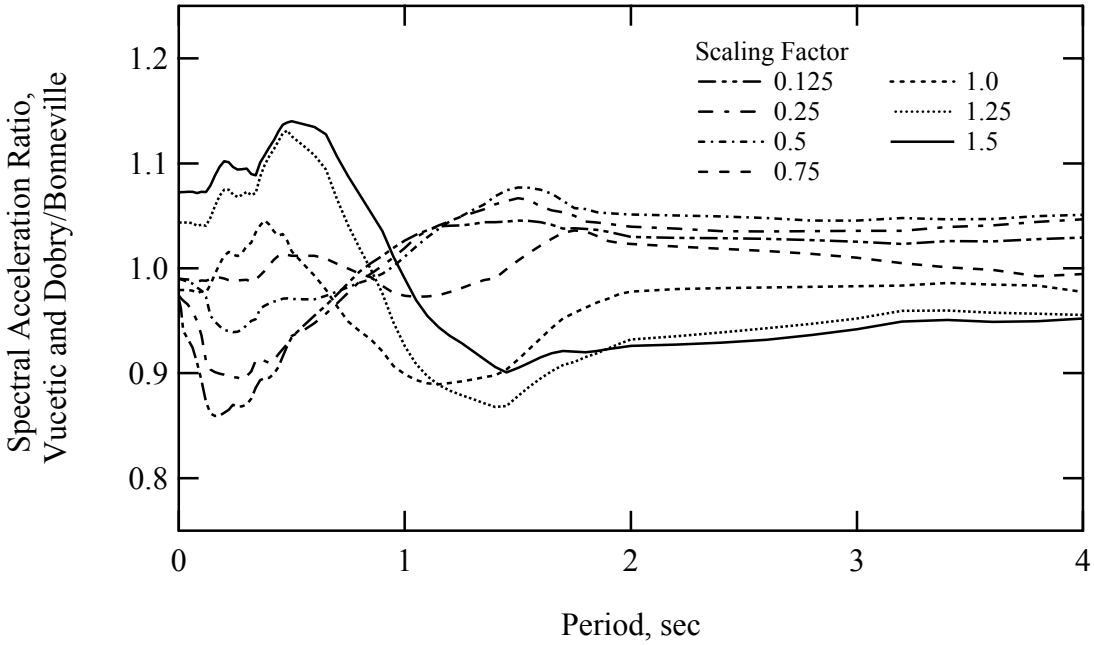

Figure 21 Ratio of Spectral Accelerations, Vucetic and Dobry/Bonneville

Conclusions

The following conclusions can be made based upon this work:

1) Bonneville soils have unique modulus reduction and damping behavior, with high elastic threshold strains, and exhibit more linear behavior and lower damping at low to moderate strains than most soils with similar plasticity.
2) The modulus reduction and damping models presented in this study provide more accurate characterization of the dynamic behavior of Bonneville soils than generic models.
3) A small portion of the variance of reference strain in the Bonneville soil model can be attributed to plasticity, and there does not appear to be a strong correlation with either effective stress or overconsolidation ratio.
4) Based upon a site response analyses, using the Bonneville soil model for near surface soils does not consistently result in higher ground shaking levels than those obtained from generic models.

Recommendations

The following recommendations can be made based upon this work:

1) The Bonneville soil model presented in this work should be incorporated into site response analyses of sites underlain by Bonneville silty clays.

2) More testing should be performed to determine the factors that contribute to the large variance in threshold strain in Bonneville soils.
3) Deeper, pre-Bonneville soils should be tested to determine if they exhibit behavior similar to Bonneville soils.
4) Other young, silty clays should be investigated to determine if other soils exhibit behavior similar to the Bonneville soils.

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Appendix

Table A1 Two and Three Parameter Hyperbolic Model Curve Fitting Parameters Obtained from Stress Integration Analysis (Sasanakul and Bay, 2008) of Torsional Shear Tests on Samples Consolidated to the In Situ Mean Effective Stress

Site	In Situ Mean Effective Stress, σ'_0, atm	Two Parameter Hyperbolic Model			Three Parameter Hyperbolic Model			
		G_{max}, atm	γ_{ref}, %	Chi-Square	G_{max}, atm	γ_{ref}, %	a	Chi-Square
Geneva Rd Bridge over Provo River, Provo, UT	0.84	493	0.0682	1.125e-5	493	0.0686	1.1247	1.125e-5
Geneva Rd Bridge over Provo River, Provo, UT	1.41	788	0.0787	4.777e-6	788	0.0813	0.9247	4.777e-6
Geneva Rd Bridge over Provo River, Provo, UT	1.90	596	0.1447	2.171e-6	596	0.1397	0.9245	2.171e-6
2835 S. 1000 W, Nibley, UT*	0.50	240	0.1454	1.334e-7	240	0.1903	0.8268	1.334e-7
USU Drainage Farm Site, Logan,UT*	0.42	103	0.1148	1.491e-7	103	0.1206	0.813	1.491e-7
BYU Site West of Airport, SLC, UT	0.53	311	0.0680	2.297e-5	311	0.0767	1.145	1.852e-6
BYU Site West of Airport, SLC, UT	0.90	345	0.0737	1.233e-4	345	0.0640	0.8463	1.19e-5
500 W.500 S., SLC UT	0.33	175	0.1786	4.59e-08	NA	NA	NA	NA
500 W.500 S., SLC UT	0.60	214	0.3236	6.6e-10	NA	NA	NA	NA
900 S. 50 W., SLC, UT	0.50	293	0.0977	1.42e-08	NA	NA	NA	NA
900 S. 50 W., SLC, UT	0.47	327	0.0851	4.71e-10	NA	NA	NA	NA
700 W. Antelope Dr., Layton, UT	0.44	235	0.0302	9.21e-09	NA	NA	NA	NA
700 W. Antelope Dr., Layton, UT	0.41	230	0.0741	1.18e-08	NA	NA	NA	NA
200 E. 1600 N., Logan, UT	0.41	237	0.1023	4.26e-10	NA	NA	NA	NA
200 E. 1600 N., Logan, UT	0.44	204	0.1445	2.31e-8	NA	NA	NA	NA

*Soils classified as CH, and were not used to develop Bonneville Soil model

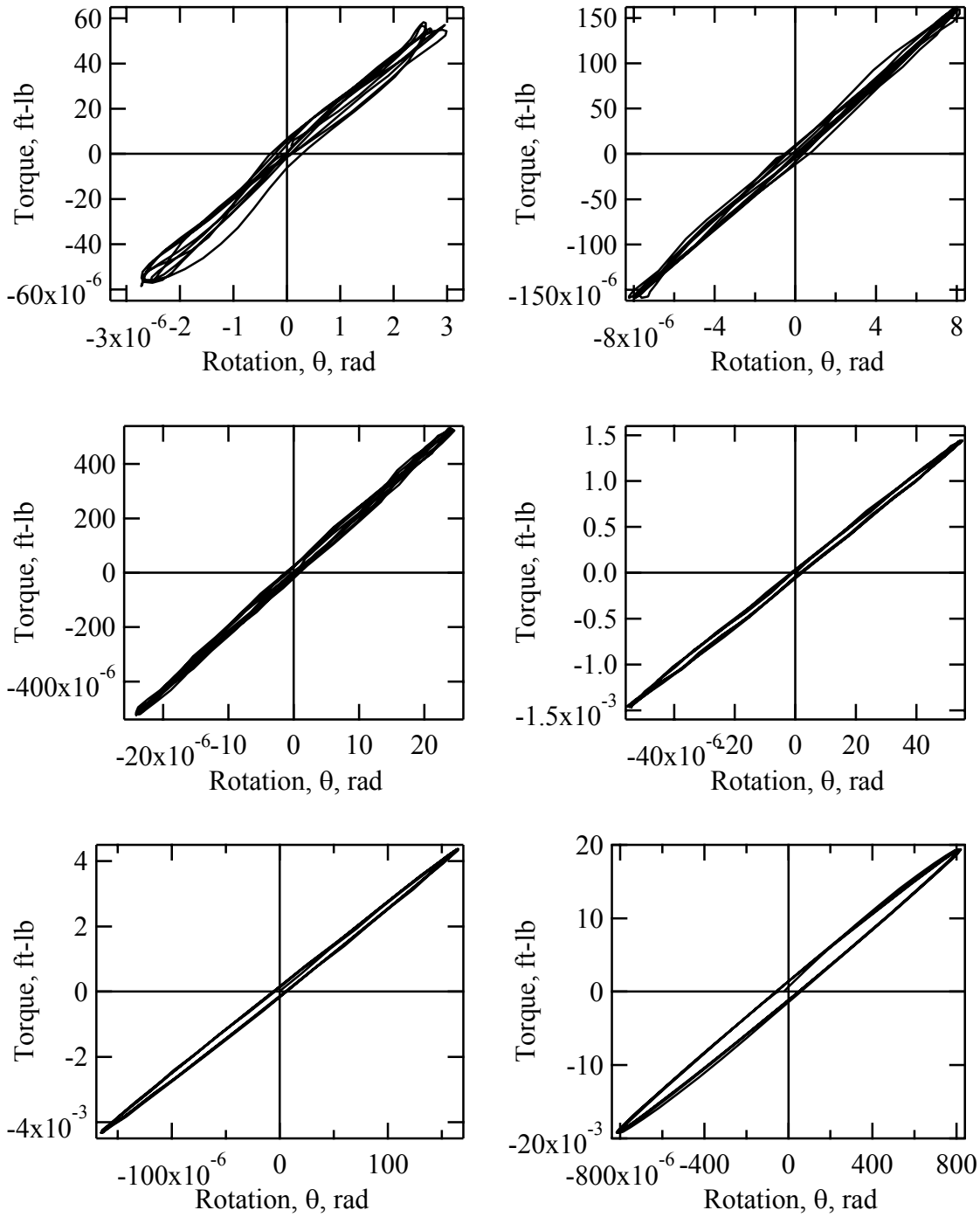

Figure A1 Torque Rotation plots from Tosional Shear Testing of Sample From 500 W.500 S., SLC UT, $\sigma'_o = 0.33$ atm

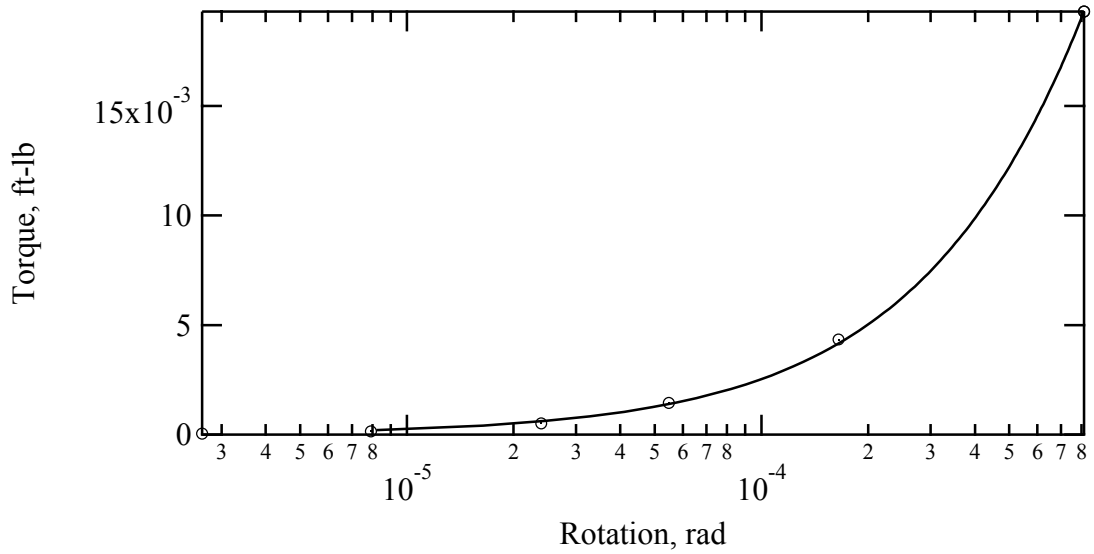

Figure A2 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample From 500 W.500 S., SLC UT, $\sigma'_o = 0.33$ atm

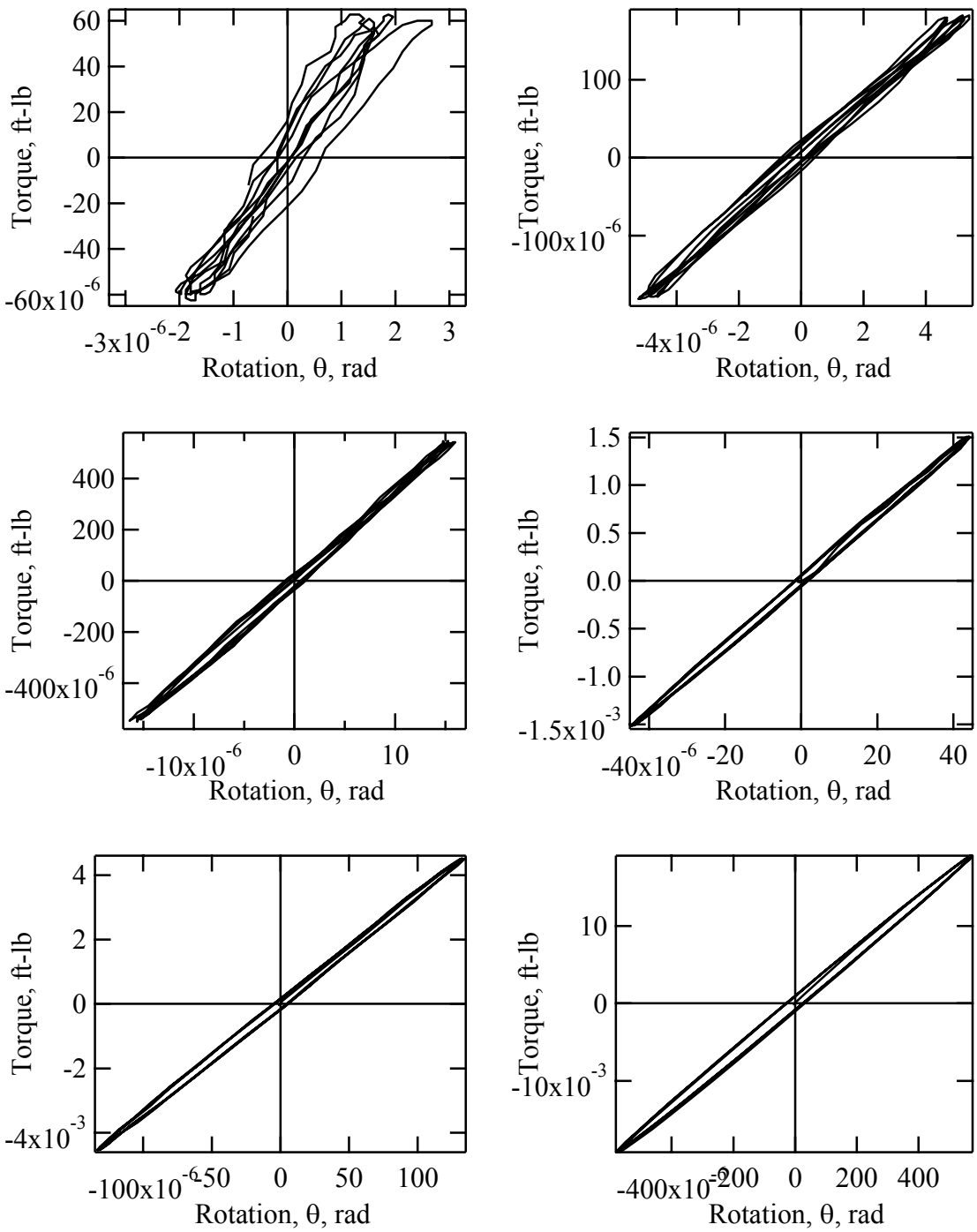

Figure A3 Torque Rotation plots from Tosional Shear Testing of Sample From 500 W.500 S., SLC UT, $\sigma'_o = 0.60$ atm

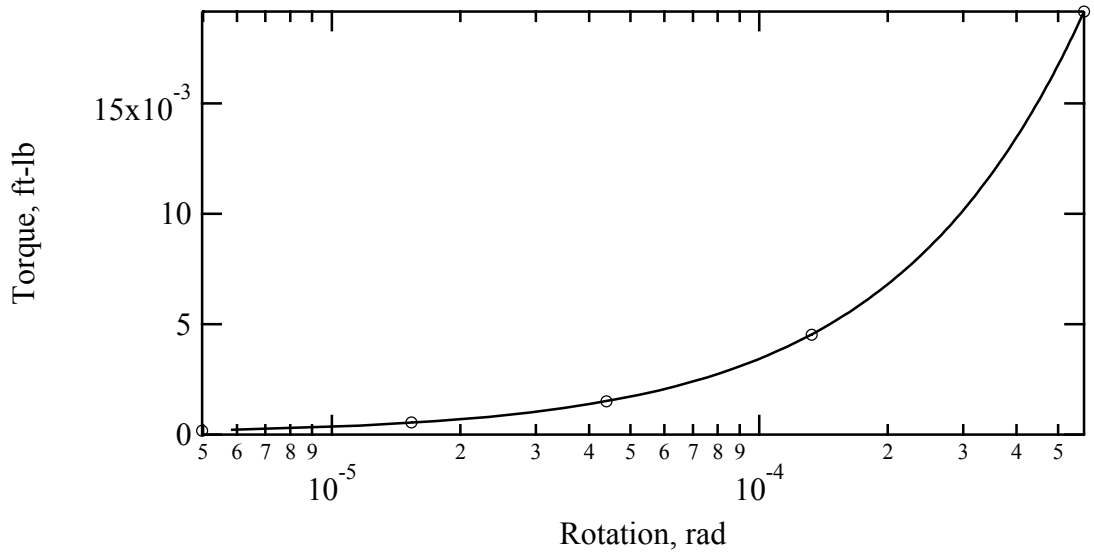

Figure A4 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample From 500 W.500 S., SLC UT, $\sigma'_o = 0.60$ atm

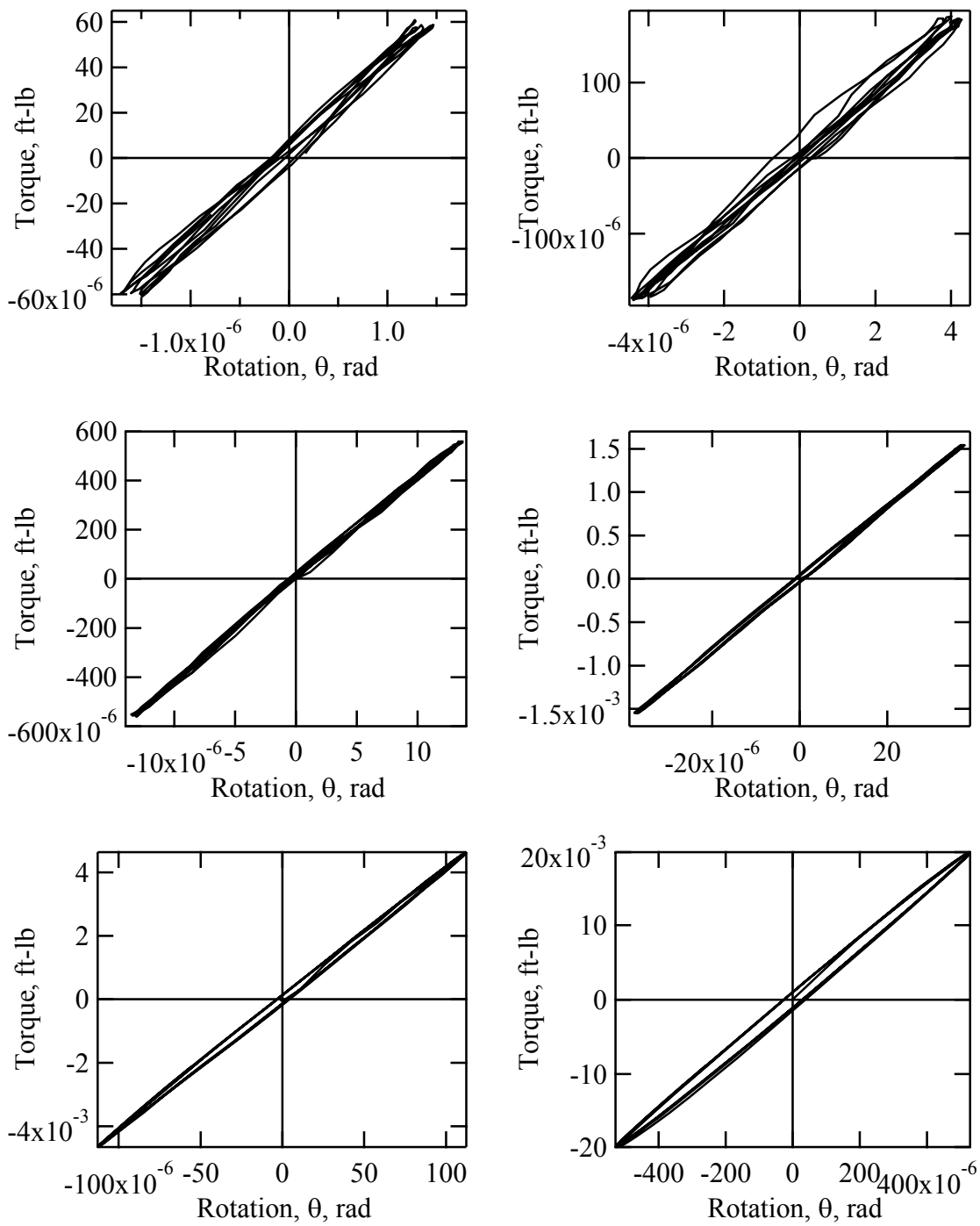

Figure A5 Torque Rotation plots from Tosional Shear Testing of Sample From 900 S. 50 W., SLC, UT, $\sigma'_o = 0.50$ atm

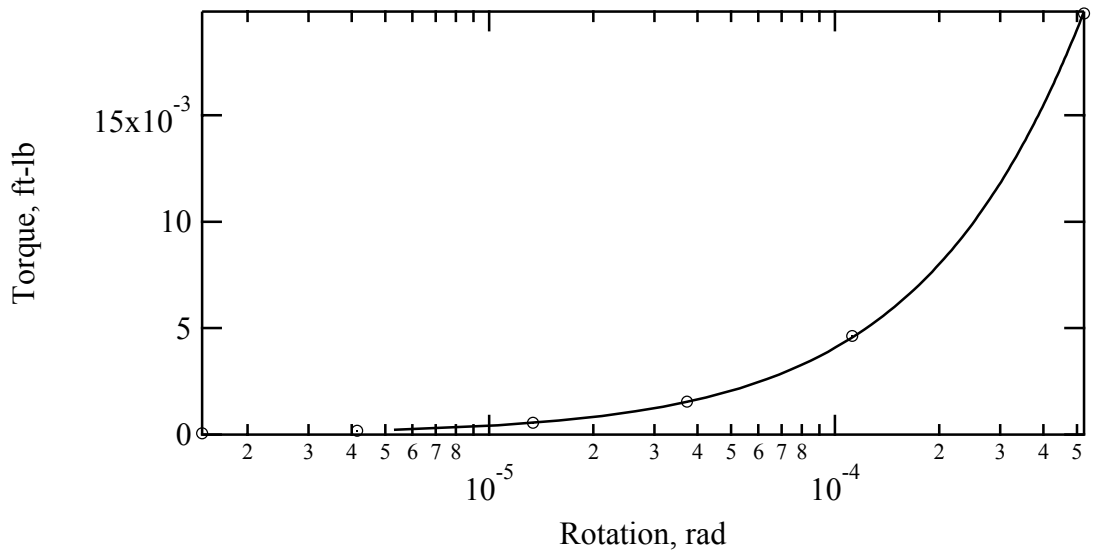

Figure A6 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample from 900 S. 50 W., SLC, UT, $\sigma'_o = 0.50$ atm

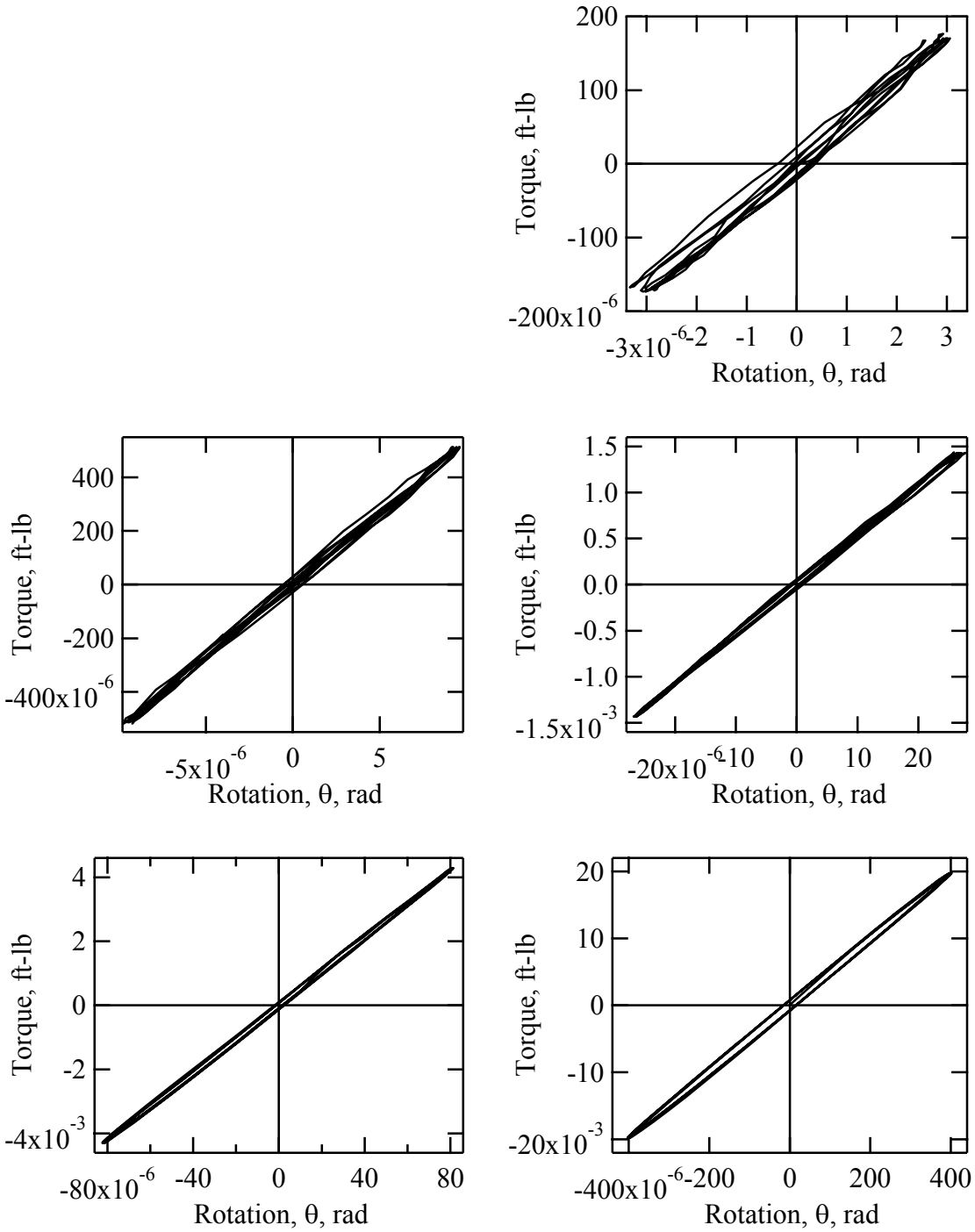

Figure A7 Torque Rotation plots from Tosional Shear Testing of Sample From 900 S. 50 W., SLC, UT, σ'_o = 0.47 atm

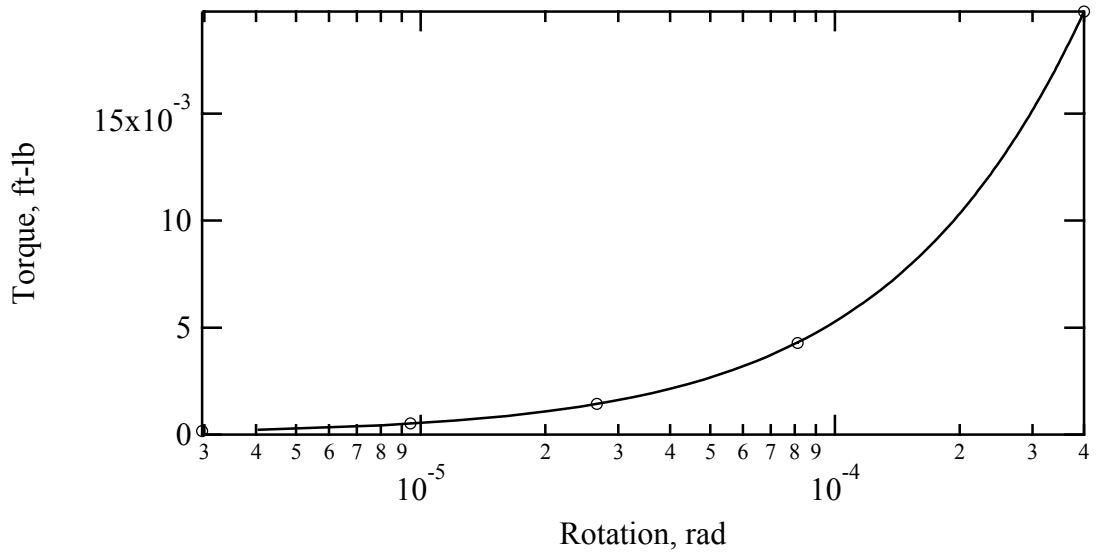

Figure A8 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample from 900 S. 50 W., SLC, UT, $\sigma'_o = 0.47$ atm

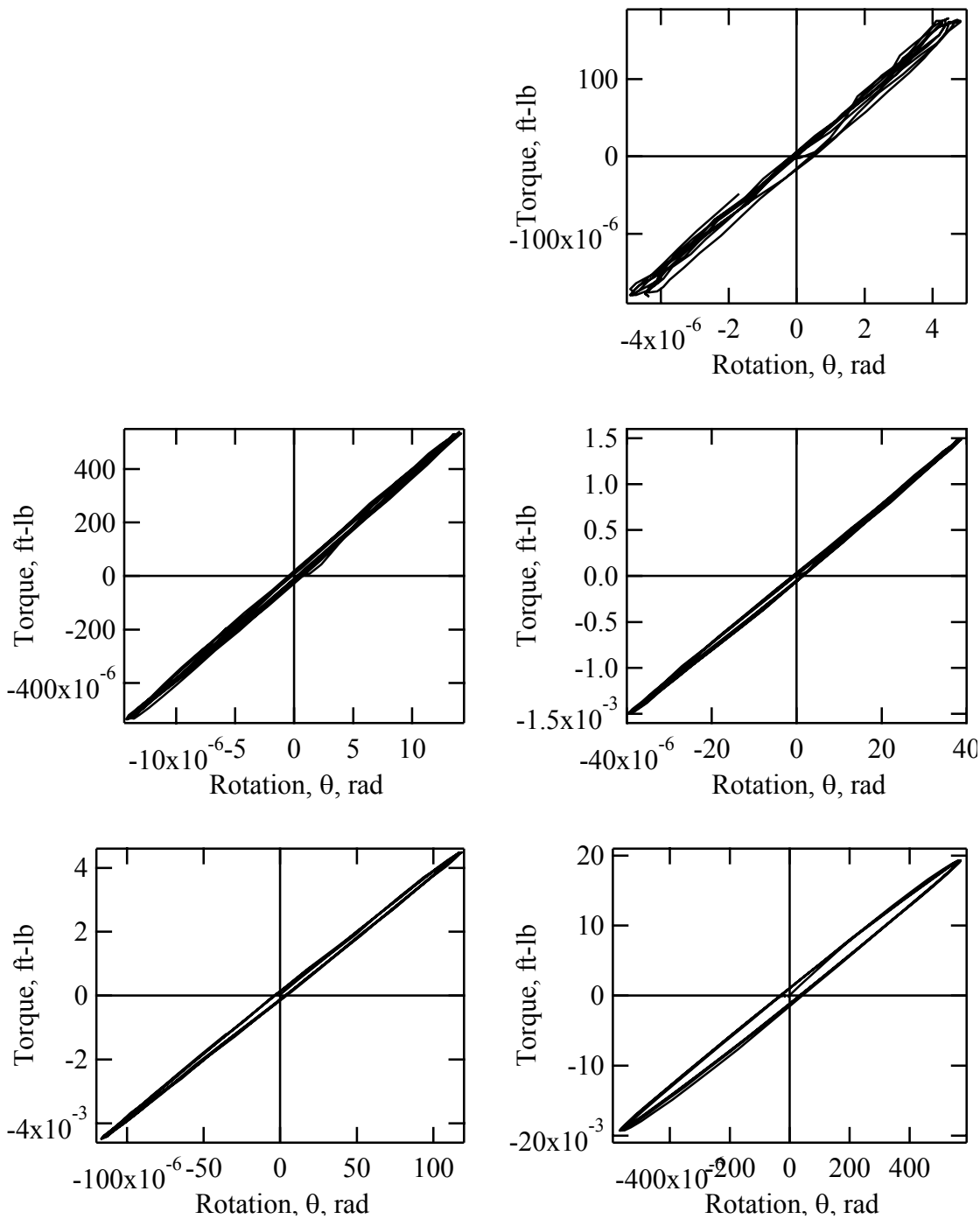

Figure A9 Torque Rotation plots from Tosional Shear Testing of Sample from 700 W. Antelope Dr., Layton, UT, $\sigma'_o = 0.44$ atm

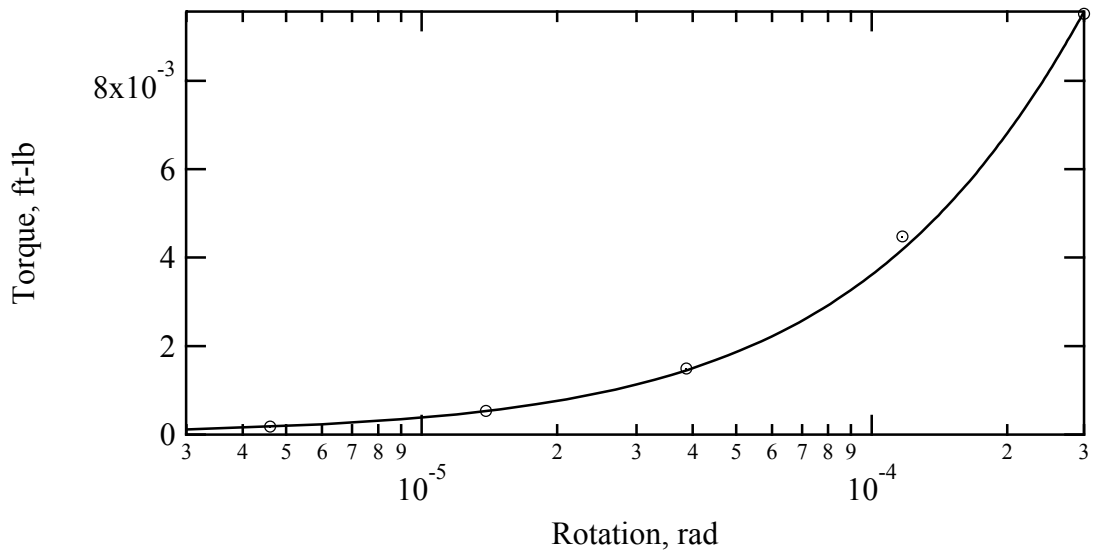

Figure A10 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample from 700 W. Antelope Dr., Layton, UT, σ'_o = 0.44 atm

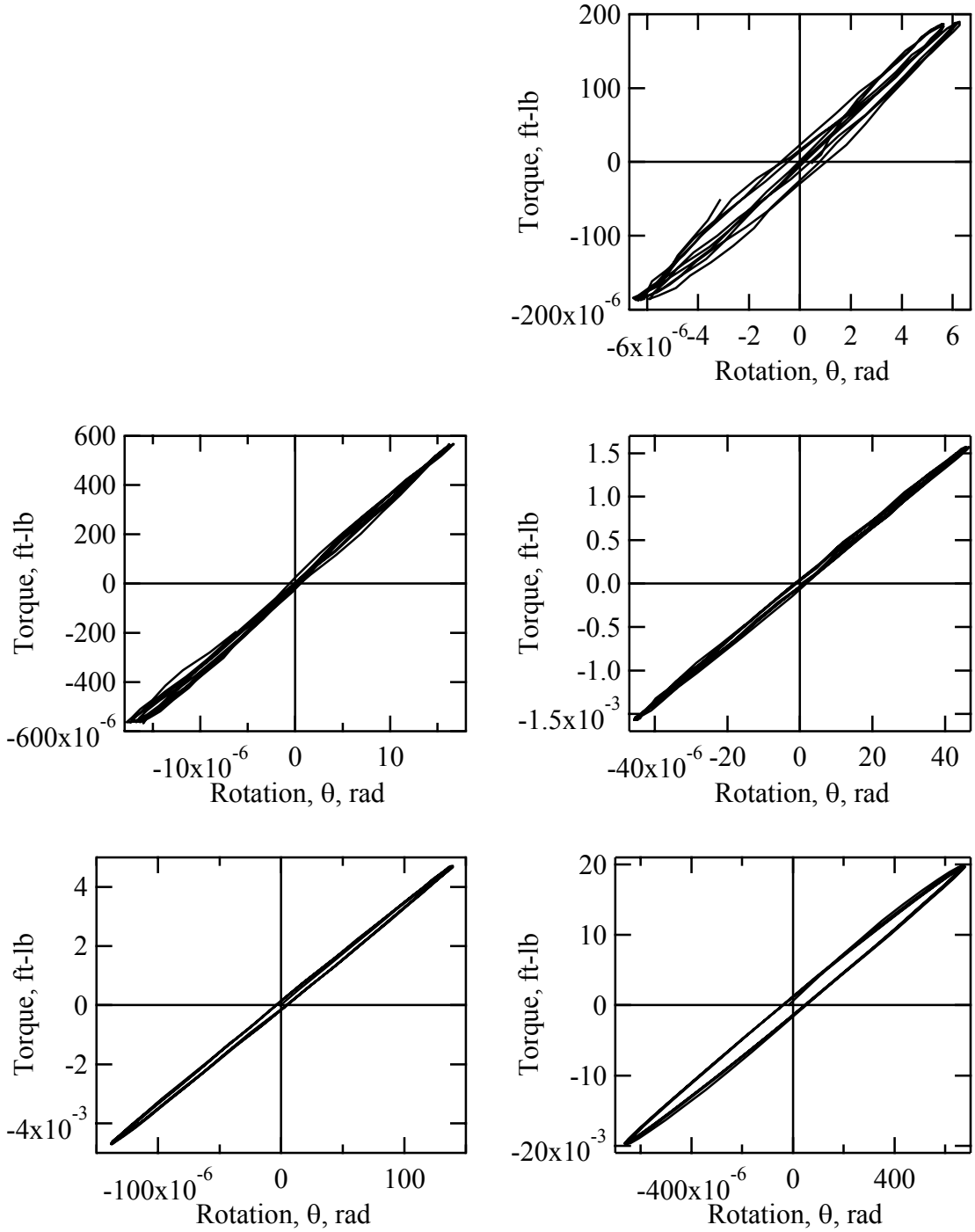

Figure A11 Torque Rotation plots from Tosional Shear Testing of Sample from 700 W. Antelope Dr., Layton, UT, $\sigma'_o = 0.41$ atm

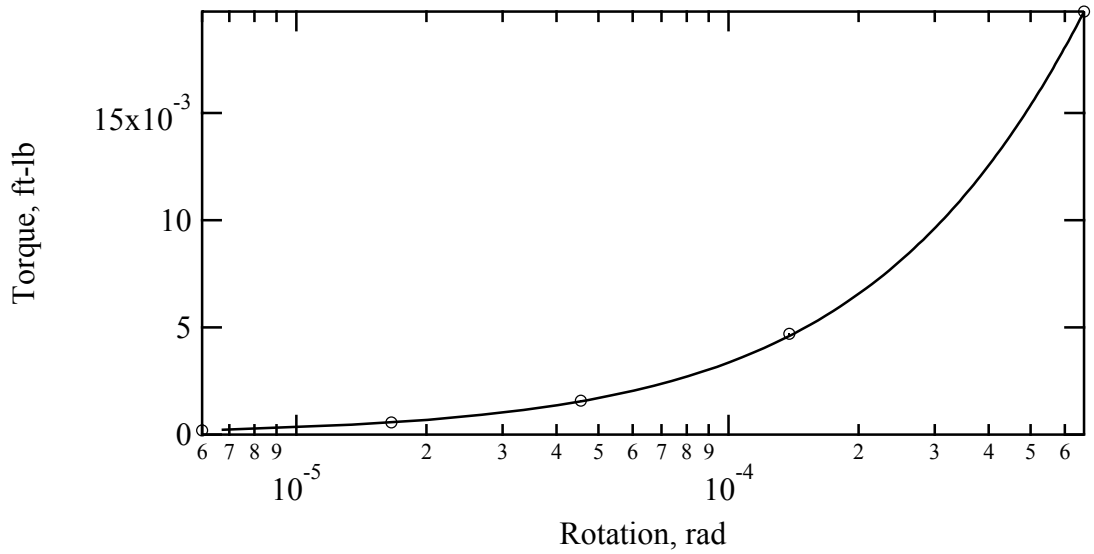

Figure A12 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample from 700 W. Antelope Dr., Layton, UT, $\sigma'_o = 0.41$ atm

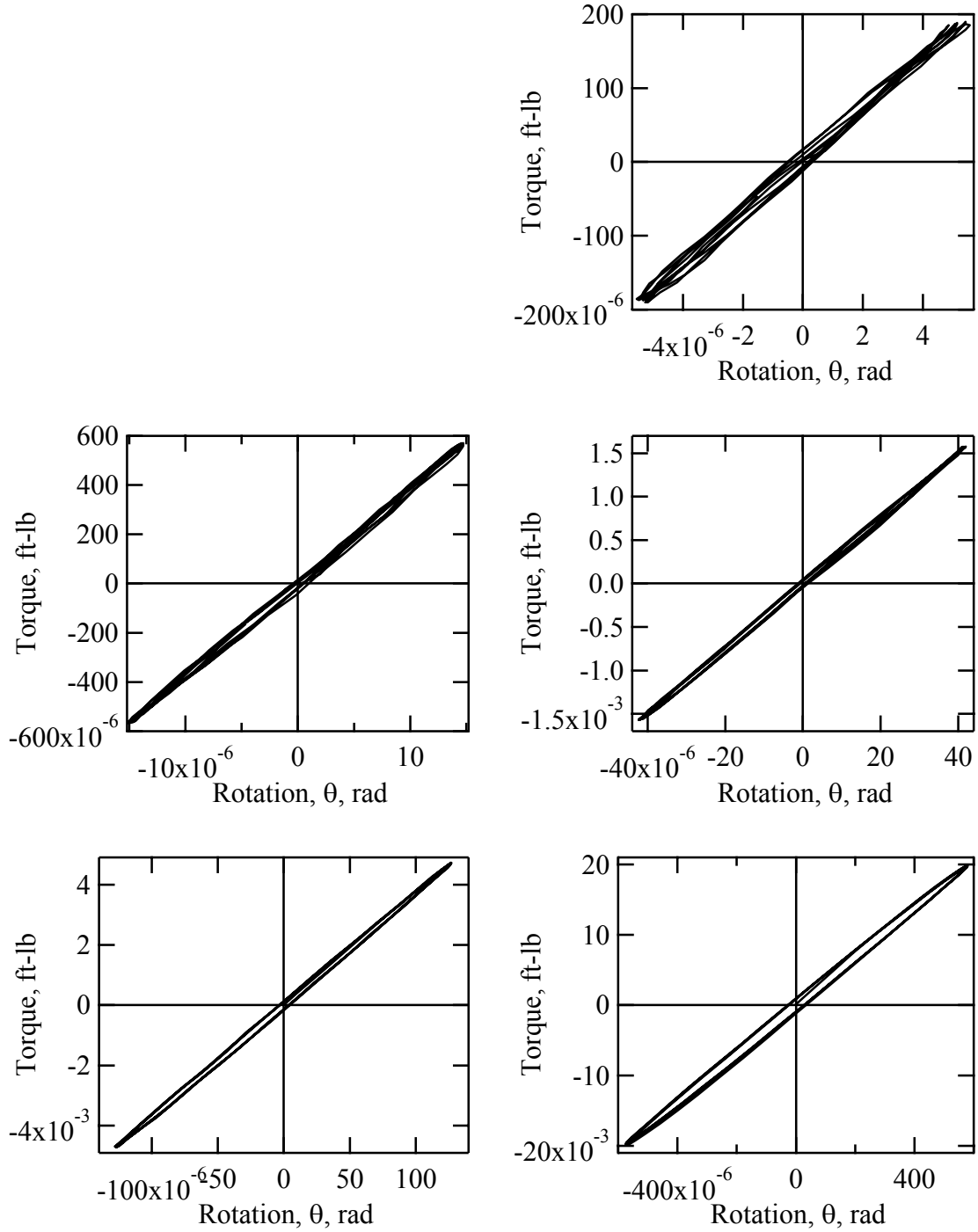

Figure A13 Torque Rotation plots from Tosional Shear Testing of Sample from 200 E. 1600 N., Logan, UT, $\sigma'_o = 0.41$ atm

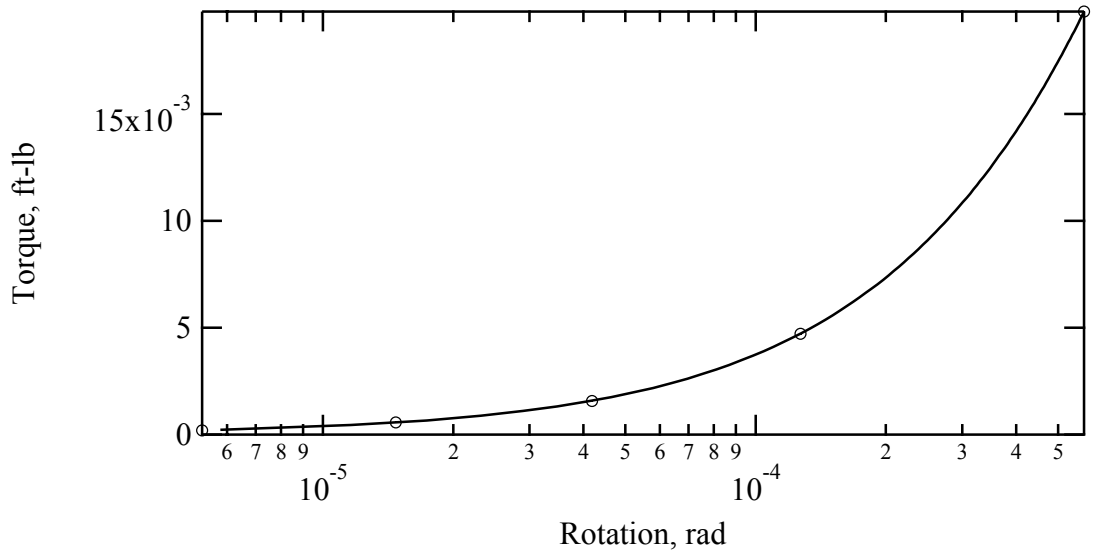

Figure A14 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample from 200 E. 1600 N., Logan, UT, σ'_o = 0.41 atm

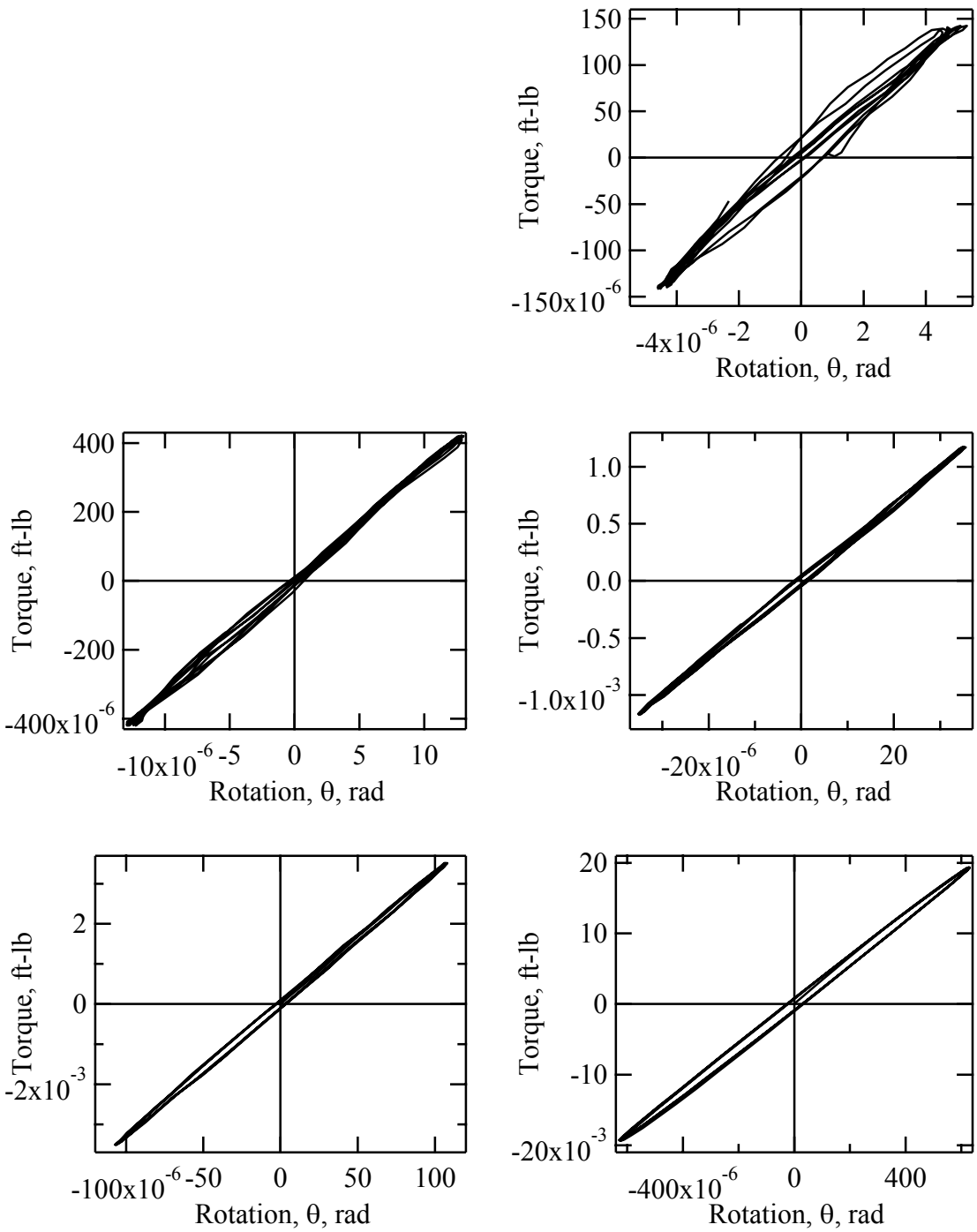

Figure A15 Torque Rotation plots from Tosional Shear Testing of Sample from 200 E. 1600 N., Logan, UT, $\sigma'_o = 0.44$ atm

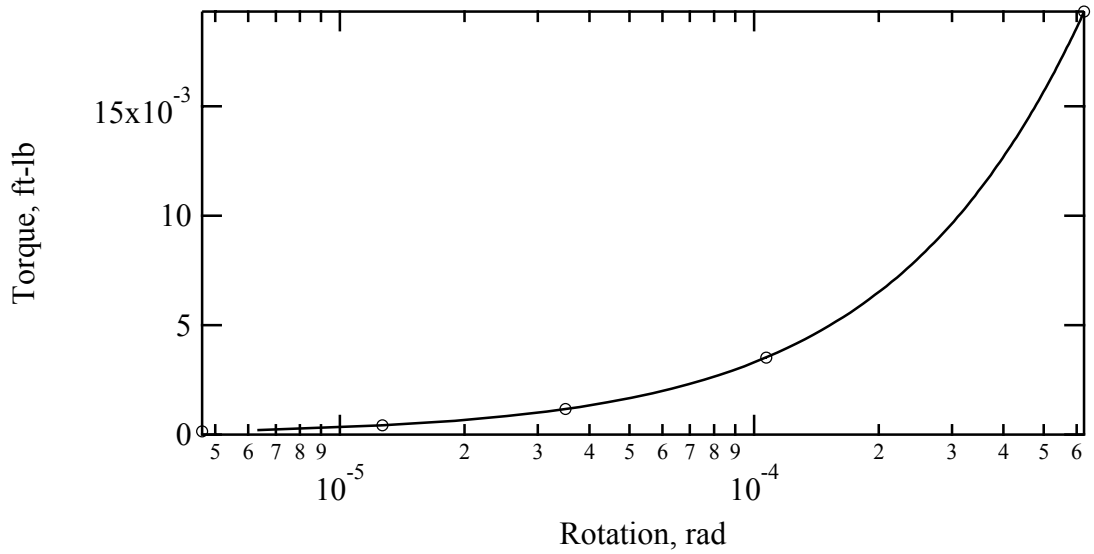

Figure A16 Curve Fit used to Determine Two Parameter Hyperbolic Model Parameters for Tosional Shear Testing of Sample from 200 E. 1600 N., Logan, UT, σ'_o = 0.44 atm