Phase 2 Geotechnical Engineering Services

Murray Morgan Bridge Rehabilitation
Tacoma, Washington

for
City of Tacoma

November 8, 2011

GeoEngineers
Earth Science + Technology

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File No. 0570-100-02

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INTRODUCTION

This report presents the results of our Phase 2 geotechnical engineering services in support of the Murray Morgan Bridge Rehabilitation project. We previously provided geotechnical engineering services for this project, the results of which are presented in our Phase 1 Geotechnical Engineering Services report dated July 21, 2011. Our Phase 2 services have been completed to further our Phase 1 analyses and provide recommendations for future construction, ground improvement and foundation mitigation at the project site. Our services have been provided in general accordance with the signed agreement between GeoEngineers, Inc. (GeoEngineers) and the City of Tacoma (COT) dated March 8, 2011, and the April 8, 2011, Amendment No. 1 authorized by COT on July 6, 2011.

The primary objectives of our Phase 2 geotechnical engineering services are to:

- Further assess the seismic performance of the City Approach and Port Approach structures using the numerical modeling program FLAC 2D;
- Refine our recommendations for mitigation (ground improvement) with our numerical model;
- Evaluate the seismic foundation performance of the Center Span structure; and
- Provide conceptual foundation replacement recommendations for the Center Span and approach structures.

PROJECT UNDERSTANDING AND BACKGROUND

General

The Murray Morgan Bridge project site is located in Tacoma, Washington, as shown in Figure 1. The bridge comprises three main sections: 1) City Approach, 2) Center Span and 3) Port Approach. The bridge was constructed in the early 1900s; the original Port Approach was replaced with the current configuration in 1957. Plan and section views of the bridge layout are provided in Figures 2 and 3, respectively. Figures 4 and 5 show an expanded cross section view.

Phase 1 of the Murray Morgan Bridge Rehabilitation project was completed as part of a design-build contract to make repairs to the superstructure required to reopen the bridge to traffic. This included a seismic evaluation of the approach structures under two design events: the American Association of State Highway and Transportation Officials (AASHTO) specified design level earthquake and an operation level earthquake (OLE).

The AASHTO-specified design level earthquake has a 7 percent probability of exceedance in 75 years (1,000-year recurrence interval return period) with the performance objective of collapse prevention to maintain life safety. In addition to the AASHTO design earthquake, COT elected to evaluate an OLE design earthquake with the performance objective of repairable damage and maintaining operation. The OLE design level earthquake has a 50 percent probability of exceedance in 75 years (108-year recurrence interval return period). As part of our Phase 1 services, GeoEngineers provided information to aid COT in selecting a design OLE event. For a detailed discussion of the selection process please refer to our Phase 1 Geotechnical Engineering Services report.

During our Phase 1 analyses it became clear that significant ground improvement and/or foundation replacement will be necessary to retrofit the bridge to withstand the AASHTO level event. As a result, we understand that during the Phase 1 Design Build contract COT will focus on retrofitting the bridge to the OLE design level. It is our understanding that COT plans to fully retrofit the bridge to AASHTO specifications in the future. This report provides analysis and design recommendations to support future evaluation of options to retrofit the bridge to AASHTO specifications and provides geotechnical design parameters for evaluation of the Center Span structure, including earthquake time histories and foundation spring values.

City Approach Structure (Bents 1 through 10)

The City Approach structure has 10 bents supported on shallow foundations that comprise shallow concrete pedestal foundations. Based on our review of the available as-built drawings, the foundations at Bents 2, 3 and 4 may have been modified during construction of I-705. For the purposes of this study, we have assumed that Bents 2 through 4 are supported on concrete pedestal foundations similar to the other foundations for the City Approach structure. The approximate location and bottom dimensions of the pedestal foundations are shown in Figure 4.

Based on conversations with Hardesty & Hanover, LLP (H&H), we understand that the City Approach foundations can tolerate an appreciable amount of settlement and maintain the collapse prevention performance objective. Specific vertical settlement and lateral displacement tolerances were not provided. Foundation loading information provided by H&H for the City Approach is provided in our Phase 1 Geotechnical Engineering Services report.

Center Span Bridge (Piers 1 through 4)

The Center Span is supported on four piers, each founded on a group of driven timber piles. Piers 2 and 3 are supported by 196 piles each, and Piers 1 and 4 are supported by 144 piles each. Pile spacing is typically between 2 feet 8 inches and 2 feet 10 inches on center. Average pile length below the pile cap is 28 feet for Pier 1, 52 feet for Pier 2 and 65 feet for Piers 3 and 4. The upper 12 feet of each pile group is embedded in a concrete pile cap that is approximately 30 feet thick.

Center Span vertical settlement and lateral displacement tolerances to maintain the collapse prevention performance objective were not provided. Foundation loading information provided by H&H for the Center Span is provided in Appendix A.

Port Approach Structure (Bents 11 through 18)

The Port Approach has eight bents supported on driven precast concrete piles. Each bent is supported by eight piles. The precast concrete piles consist of hollow tubes with a 36-inch outside diameter and a 26-inch inside diameter. Typical pile embedment is on the order of 25 to 30 feet. Average pile embedment at each bent is shown in Figure 5. The eastern end of the Port Approach comprises a retaining wall-faced earth embankment. We understand that the retaining wall footings are supported on shallow timber piles. Evaluation of the earth embankment is not within the scope of this project.

Based on conversations with Exeltech Consulting, Inc. (Exeltech), we understand that the Port Approach foundations can withstand an appreciable amount of settlement and maintain the collapse prevention performance objective. Specific vertical settlement tolerances were not provided. We understand that lateral deformations control the seismic design with a tolerance of about 3 inches or less at the ground surface. Foundation loading information provided by Exeltech for the Port Approach is provided in our Phase 1 Geotechnical Engineering Services report.

SCOPE OF GEOTECHNICAL SERVICES

Our analyses include completing numerical modeling to refine our design recommendations for the City Approach and Port Approach and evaluating the Center Span seismic foundation performance. Our scope is summarized below:

1. Numerical Modeling: For the selected mitigation options at the City Approach and Port Approach, we performed geotechnical analyses of the subsurface conditions using FLAC 2D, a finite difference program. We completed these analyses for two purposes: 1) to better define the expected performance (deformation and stresses) under loading conditions during the design earthquake events; and 2) to further evaluate the proposed ground improvement layouts. These analyses included the following:
   - Evaluating the seismic stability of the waterway slopes in their current condition during the AASHTO and OLE design seismic events.
   - Evaluating the seismic stability of the waterway slopes with the proposed ground improvement at the City Approach and Port Approach for both the AASHTO and OLE design seismic events.
   - Estimating soil/structure movements of the bridge foundations during the AASHTO and OLE design seismic events. These estimates were completed for the existing conditions and the improved ground conditions.
2. Seismic Study of Center Span Bridge Structure: We completed the following seismic design studies:
   - Providing seismic design criteria, including defining the expected lateral and vertical pile performance during the design earthquake events (AASHTO and OLE) and evaluating the liquefaction and lateral spread potential of site soils.
   - Evaluating the extent of slope instability and potential lateral spreading anticipated under the design seismic conditions.
   - Providing recommendations to reduce the effect of soil liquefaction and lateral spreading. This includes general ground improvement and foundation replacement recommendations.

SITE CONDITIONS

Geologic Setting

The project site is situated on the edge of the delta formed at the mouth of the Puyallup River as it enters Commencement Bay, straddling the contact between recent alluvial sediments and older glacially consolidated sediments (Troost and Booth, in review). The glacially consolidated deposits are present at the ground surface along the west end of the City Approach and are overlain by up to about 250 feet of alluvium at the east end of the Port Approach. The bedrock depth is estimated at about 1,600 feet (Hall and Othberg, 1974).

Subsurface Explorations

Site subsurface conditions were explored by drilling three borings and advancing six cone penetrometer test (CPT) soundings. Shear wave velocity data were collected in four of the CPT soundings. We also reviewed subsurface exploration logs from studies completed by others for the Murray Morgan Bridge and I-705. Additional information regarding our subsurface explorations and laboratory testing is provided in Appendix B.

Subsurface Conditions

General

Subsurface conditions at the site consist of four major soil units: 1) fill, 2) tidal deposits, 3) alluvium and 4) glacially consolidated deposits. Based on our assessment of the physical and engineering properties of the site soils, we further characterize the alluvium unit into four sub-units: upper alluvium 1, upper alluvium 2, lower alluvium 1 and lower alluvium 2. A more detailed discussion of subsurface conditions is provided in our Phase 1 Geotechnical Engineering Services report.

Figures 3 through 5 present our general interpretation of the subsurface conditions at the site, including depth intervals at which the soil units were encountered. Additional information regarding our subsurface explorations and laboratory testing is provided in Appendix B.

Groundwater

Groundwater was generally encountered between 7.5 and 10 feet below ground surface (bgs) in explorations completed on land. Groundwater elevations are expected to vary with season, tidal fluctuations and other factors. Based on our review and experience, we expect that groundwater levels near the Thea Foss Waterway will fluctuate with the tide.

SEISMIC DESIGN CONSIDERATIONS

Seismicity

The Puget Sound Lowland is located near the convergent tectonic plate boundary known as the Cascadia Subduction Zone (CSZ). The CSZ is an approximately 650-mile-long thrust fault that extends along the Pacific Coast from mid-Vancouver Island to Northern California at approximately 50 to 75 miles off the Washington coast, where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate. The interaction of these two plates results in two potential seismic source zones within the CSZ: 1) an intraplate source zone and 2) an interplate source zone. A third seismic source zone, referred to as the shallow crustal source zone, is associated with the north-south compression resulting from northerly movement of the Sierra Nevada block of the North American Plate. A more detailed discussion of the three identified source zones is provided in our Phase 1 Geotechnical Engineering Services report.

Representative Design Earthquake Types and Sources

We assessed the potential contribution of each of the regional earthquake source zones to the seismic hazard at the project site during our Phase 1 studies. Table 1 below presents a summary of the earthquake type, magnitude and associated contribution to the seismic hazard at the project site for both the AASHTO and OLE events.

TABLE 1. SUMMARY OF SEISMIC HAZARD DEAGGREGATION FOR PERIODS OF 0.5 AND 1.0 SECONDS

Earthquake Type	Characteristic Magnitude		Percent Contribution to Project Site Seismic Hazard	
	AASHTO	OLE	AASHTO	OLE
Intraplate	6.6	6.4	40 – 50 %	50 – 55 %
Interplate	9.0	8.5	25 – 40 %	15 – 20 %
Shallow Crustal	6.9	6.3	20 – 25 %	25 – 30 %

Site-Specific Response Spectra

We completed site-specific seismic response analyses for the AASHTO and OLE events in general accordance with AASHTO and American Society of Civil Engineers (ASCE) 7-05 guidelines. Our recommended AASHTO and OLE design response spectra for the City Approach, Center Span and Port Approach are presented in Figure 6 and Figure 7, respectively. Details of our analysis are presented in our Phase 1 Geotechnical Engineering Services report.

Liquefaction

We evaluated the liquefaction potential of the site soils by comparing the stresses in the ground caused by ground shaking (cyclic shear stress ratio) to the resisting strength of the soil (cyclic resistance ratio). The ratio of these parameters defines the factor of safety (FS) against liquefaction. If the FS is less than 1.0, the soil will likely liquefy, resulting in a significant reduction in shear strength. We evaluated liquefaction potential using three simplified methods: Youd et al. (1997), Seed et al. (2003) and Idriss and Boulanger (2004). A detailed discussion of our analysis methodology and the liquefaction analysis results for the City and Port Approaches are presented in our Phase 1 Geotechnical Engineering Services report.

We evaluated the liquefaction potential of the site soils at the Center Span for the AASHTO and OLE design events using the subsurface information obtained from our overwater explorations. These results were integrated with the results of our Phase 1 liquefaction analysis of the City Approach and Port Approach. Table 2 below presents the liquefaction susceptibility of the soil layers delineated at the Center Span.

TABLE 2. SUMMARY OF LIQUEFACTION ANALYSIS RESULTS, CENTER SPAN

Soil Unit	Liquefaction Potential[1]	
	AASHTO	OLE
Fill and Tide Flats	Not Present	Not Present
Upper Alluvium 1 and 2	Not Present	Not Present
Lower Alluvium 1	FL	PL to FL
Lower Alluvium 2	Not Present	Not Present
Glacially Consolidated Deposits	NL	NL

Note:

[1] NL: Non-Liquefiable; PL: Partially Liquefiable; FL: Fully Liquefiable

Liquefaction-Induced Settlement

The results of our liquefaction analysis for the Center Span are generally consistent with that of the City and Port Approaches, indicating that soils overlying Lower Alluvium 2 will likely liquefy and settle. Table 3 below presents the estimated liquefaction-induced ground settlement calculated for the Center Span and the City and Port Approaches.

TABLE 3. SUMMARY OF LIQUEFACTION-INDUCED SETTLEMENT

Bridge Structure	Liquefaction-Induced Settlement at the Ground Surface	
	AASHTO	OLE
Center Span (Piers 1 – 4)	16 to 31 inches	2 to 12 inches
City Approach (Bents 1 – 10)	4 to 15 inches	2 to 12 inches
Port Approach (Bents 11 – 18)	5 to 18 inches	3 to 7 inches

Lateral Spreading

Lateral spreading involves lateral displacements of large volumes of soil impacted by liquefaction. Lateral spreading can occur on near-level ground as blocks of near-surface soils are displaced relative to adjacent blocks. Lateral spreading also occurs as blocks of surface soils are displaced toward a nearby slope or free-face by movement of the underlying liquefied soil. The Thea Foss Waterway represents a free-face condition that will likely contribute to lateral spreading at both the City and Port Approaches. Figures 8 and 9 present the critical failure surfaces identified in our Phase 1 study at the City and Port Approaches under the OLE and AASHTO design events. A detailed discussion of our analysis methodology is presented in our Phase 1 Geotechnical Engineering Services report.

The potential soil movement resulting from failure along the planes identified during Phase 1 could impose significant loads on the Center Span foundations. To better evaluate the extent and potential effect of lateral spreading at the project site we completed engineering analyses using numerical modeling, as discussed in the following sections of this report.

SEISMIC PERFORMANCE EVALUATION OF CITY AND PORT APPROACH STRUCTURES

General

During our Phase 1 study we completed engineering analyses to evaluate the effect of the estimated liquefaction-induced ground settlement and potential lateral soil movement on the foundations supporting the City and Port Approaches. We evaluated the anticipated seismic lateral deformation at the City and Port Approaches using simplified limit equilibrium and Newmark analyses and developed ground improvement programs at the City and Port Approaches to reduce the lateral and vertical settlements. We also identified the potential need for foundation replacement at Bent 11 of the Port Approach structure. The results of our analyses and the ground improvement layouts for the AASHTO and OLE design events are presented in our Phase 1 Geotechnical Engineering Services report.

The methods of analysis employed in our Phase 1 study allowed us to identify the lateral spreading hazard at the project site. We completed numerical modeling to quantify the anticipated seismic displacements and account for the interaction of the soil and structural elements of the foundation systems as part of our Phase 2 engineering services. Our analyses included an evaluation of foundation performance considering the existing soil conditions and the improved soil conditions with the compaction grouting ground improvement layout recommended in our Phase 1 Geotechnical Engineering Services report.

Numerical Modeling Approach

We used the Fast Lagrangian Analysis of Continua (FLAC) 2D V 6.0 computer program (Itasca, 2008) to evaluate the performance of the City and Port Approach structures under the design earthquake events. FLAC 2D is a two-dimensional (2D) plane-strain, dynamic finite difference program that can analyze the nonlinear soil response and soil-structure interaction during a seismic event. In our FLAC analysis, the structural element model is coupled to the soil model, meaning that the analysis solves for equilibrium for both models concurrently. In this way, we were able to analyze the soil-structure interaction during ground shaking under the design earthquake events. We also coupled the dynamic model to the groundwater flow model in order to simulate pore pressure change and shear wave propagation through the soil simultaneously, thereby modeling soil liquefaction under the design earthquake events. In addition, the earthquake loading was modeled with acceleration time histories, providing more realistic loading conditions.

The general steps in our analyses were:

1. Develop a 2D mesh that represents the typical cross section through the City and Port Approach structures and soil profile.
2. Develop soil and structural properties for the expected loading conditions. For the seismic analyses, we applied a soil model which simulates liquefaction and strength loss during the design earthquake events.
3. Develop seismic loading conditions and incorporate them into our model. Seismic loads were modeled using the earthquake acceleration time histories obtained from our site-specific seismic site response analysis (completed during Phase 1).

4. Model the ground and structural movement considering ground shaking from both the AASHTO and OLE design earthquake events for existing and improved ground conditions.

Input Parameters for FLAC Analysis

FLAC 2D Mesh

The FLAC 2D mesh represents a 1-foot-thick vertical slice through the structure in the longitudinal direction. Because it is 2D, the program treats this section as if it extends infinitely into and out of the page. Each unit in the mesh is a 4- to 5-foot trapezoidal soil element with representative strength, modulus and damping properties. The soil elements are interconnected within the FLAC 2D model via a set of equations that model the continuity and balance of forces between each element.

City Approach. The FLAC 2D meshes developed for the City Approach considering existing soil conditions and improved soil conditions are shown on Figures 10 and 11, respectively. Based on discussions with COT we understand that when ground improvement is completed at the City Approach it will be to the AASHTO level identified in our Phase 1 Geotechnical Services report. Accordingly, we only evaluated the improved conditions recommended for the AASHTO event.

Port Approach. Figure 12 depicts the FLAC 2D mesh developed for the Port Approach considering existing soil conditions. Figures 13 and 14 depict the FLAC 2D mesh developed for improved soil conditions for the AASHTO and OLE design events, respectively.

FLAC Input Soil Parameters

Modeled soil units, each with the assigned static and dynamic soil properties, are shown in different colors on Figures 10 through 14. The soil units are delineated considering the liquefaction potential and soil strength under the design earthquake events.

We derived static soil strength (friction and cohesion) values based on averages of the corrected blow counts and CPT tip resistances from the subsurface explorations. We determined Young's modulus based on the shear wave velocity measurements completed for the project. Under seismic conditions, the nonlinear soil properties (modulus and damping) were modeled using the EPRI (1993) relationship.

Under seismic conditions, we delineated the soil units in terms of their liquefaction potential. For the soil units that are determined to be non-liquefiable and where the effect of the excess pore pressure is negligible, we used the conventional Mohr-Coulomb (M-C) soil model to simulate the dynamic response of the soil. For the potentially liquefiable soils, we employed an effective stress plasticity model, UBCSAND 904aR (UBCSAND) (Beaty and Byrne, 2011). We applied the UBCSAND model through the FLAC User Defined Model (UDM) feature to analyze the stress-strain behavior and time-dependent pore pressure change associated with liquefaction. For this project, soils below depth of 80 feet were modeled using the M-C model as recommended by Washington State Department of Transportation (WSDOT) Geotechnical Design Manual.

City Approach. A summary of the soil input parameters used in our FLAC 2D analyses for the City Approach are presented in Table 4.

TABLE 4. INPUT SOIL PARAMETERS USED IN FLAC 2D MODEL (CITY APPROACH)

Soil Unit	Fill (Above Water Table)	Upper Alluvium 1	Upper Alluvium 1 with Compaction Grout Columns	Upper Alluvium 2	Upper Alluvium 2 with Compaction Grout Columns	Lower Alluvium 1	Lower Alluvium 1 with Compaction Grout Columns	Glacial Till
Average (N1)60 Field Measurement	10-33	4-25	N/A	8-33	N/A	3-35	N/A	>30
Liquefaction Potential	Non-Liquefiable	Liquefiable	Non-Liquefiable	Liquefiable	Non-Liquefiable	Liquefiable	Non-Liquefiable	Non-Liquefiable
Soil Model	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	Mohr-Coulomb
Saturated Unit Weight (pcf)	120	120	122	120	122	120	122	135
Young's Modulus (ksf)	1589	See Table 5[1]	4441	See Table 5[1]	4864	See Table 5[1]	6112	23236
Poisson's Ratio	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Friction Angle (^{0})	34	N/A	0	N/A	0	N/A	0	40
Cohesion (psf)	0	N/A	7200	N/A	7200	N/A	7200	400
Modulus Degradation and Damping	EPRI (1993)[2]							

Notes:

[1] In the UBCSAND Model Young's modulus is nonlinear and calculated as a function of Shear modulus.

[2] EPRI: Electric Power Research Institute.

Additional input parameters for the UBCSAND model at the City Approach are summarized in Table 5.

TABLE 5. UBCSAND MODEL INPUT PARAMETERS (CITY APPROACH)

Soil Unit	Upper Alluvium 1	Upper Alluvium 2	Lower Alluvium 1 (Upper Denser Section)	Lower Alluvium 1 (Lower Looser Section)
K_G^e, Shear modulus number: G_{max}/P_{atm} at $\sigma'_m = P_{atm}$	902.1	1157.0	1157.0	992.8
n^e, Stress dependence of G_{max}	0.5	0.5	0.5	0.5
K_b, Bulk modulus number: B/P_{atm} at $\sigma'_m = P_{atm}$	631.5	809.9	809.9	695.0
m^e, Stress dependence of bulk modulus	0.5	0.5	0.5	0.5
K_G^P, Plastic shear modulus number	319.2	1353.0	1353.0	528.9
n^p, stress dependence of plastic shear modulus	0.5	0.5	0.5	0.5
ϕ_f, Maximum friction angle at $\sigma'_m = P_{atm}$ (degrees)	32.9	36.7	36.7	35.2
ϕ_{cv}, Constant volume friction angle (degrees)	32	34	34	34
R_f, Fitting constant for hyperbolic stress-strain curve	0.7911	0.7073	0.7073	0.7577

Port Approach. A summary of the soil input parameters used in our FLAC 2D analyses for the Port Approach are presented in Table 6.

TABLE 6. INPUT SOIL PARAMETERS USED IN FLAC 2D MODEL (PORT APPROACH)

Soil Unit	Fill (Above Water Table)	Tidal Deposits	Tidal Deposits with Compaction Grout Columns	Upper Alluvium 1	Upper Alluvium 1 with Compaction Grout Columns	Upper Alluvium 2	Upper Alluvium 2 with Compaction Grout Columns	Lower Alluvium 1	Lower Alluvium 1 with Compaction Grout Columns	Glacial Till
Average $(N_1)_{60}$ Field Measurement	4-37	5-23	N/A	5-25	N/A	10-40	N/A	2-50	N/A	>30
Liquefaction Potential	Non-Liquefiable	Liquefiable	Non-Liquefiable	Liquefiable	Non-Liquefiable	Liquefiable	Non-Liquefiable	Liquefiable	Non-Liquefiable	Non-Liquefiable
Soil Model	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	UBCSAND 904aR	Mohr-Coulomb	Mohr-Coulomb
Saturated Unit Weight (pcf)	120	118	120	120	122	120	122	120	122	135
Young's Modulus (ksf)	1589	See Table 7[1]	4002	See Table 7[1]	4441	See Table 7[1]	4864	See Table 7[1]	6112	23236
Poisson's Ratio	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Friction Angle (^{0})	34	N/A	0	N/A	0	N/A	0	N/A	0	40
Cohesion (psf)	0	N/A	7200	N/A	7200	N/A	7200	N/A	7200	400
Modulus Degradation and Damping	EPRI (1993)[2]									

Notes:

[1] In the UBCSAND Model Young's modulus is nonlinear and calculated as a function of Shear modulus.

[2] EPRI: Electric Power Research Institute.

The additional input parameters for the UBCSAND model at the Port Approach are summarized in Table 7.

TABLE 7. UBCSAND MODEL INPUT PARAMETERS (PORT APPROACH)

Soil Unit	Tidal Deposits	Upper Alluvium 1	Upper Alluvium 2	Lower Alluvium 1 (Upper Denser Section)	Lower Alluvium 1 (Lower Looser Section)
K_G^e, Shear modulus number: G_{max}/P_{atm} at $\sigma'_m = P_{atm}$	1136.3	992.8	1267.7	1157.0	992.8
n^e, Stress dependence of G_{max}	0.5	0.5	0.5	0.5	0.5
K_b, Bulk modulus number: B/P_{atm} at $\sigma'm = P_{atm}$	795.4	695.0	887.4	809.9	695.0
m^e, Stress dependence of bulk modulus	0.5	0.5	0.5	0.5	0.5
K_G^P, Plastic shear modulus number	1204.5	528.9	2476.9	1353.0	528.9
n^p, stress dependence of plastic shear modulus	0.5	0.5	0.5	0.5	0.5
ϕ_f, Maximum friction angle at $\sigma'_m = P_{atm}$ (degrees)	32.4	33.2	38.5	36.7	35.2
ϕ_{cv}, Constant volume friction angle (degrees)	30.0	32	34	34	34
R_f, Fitting constant for hyperbolic stress-strain curve	0.7130	0.7577	0.6787	0.7073	0.7577

Improved Soil Parameters. During Phase 1, we performed iterative slope stability and Newmark analyses to evaluate the compaction grout strength and layout required to limit foundation deformation. Our selected layout has a 10 percent grout replacement ratio, which is achieved by installing 42-inch-diameter compaction grout columns on 10-foot square grid spacing. The required minimum grout unconfined compressive strength is 1,000 pounds per square inch (psi). Table 8 presents the input parameters developed and used to model the improved soil.

TABLE 8. IMPROVED SOIL SHEAR STRENGTH PARAMETERS

Soil Unit	Total Unit Weight	Effective Unit Weight	Soil Shear Strength Properties	
			Friction Angle	Cohesion
	(psf)	(psf)	(deg)	(psf)
Improved Fill	120	120	0	7,200
Improved Tide Deposits	118	54	0	7,200
Improved Upper Alluvium 1	120	56	0	7,200
Improved Upper Alluvium 2	125	61	0	7,200
Improved Lower Alluvium 1	120	56	0	7,200

FLAC Structural Input Parameters

City Approach Structure. We did not model structural elements for the City Approach. We used the seismic performance of the ground to evaluate the seismic performance of the foundations because the shallow concrete pedestals are expected to move the same amount as the soil during ground shaking.

Port Approach Structure. We simulated the Port Approach structure with beam elements and pile elements. The bending and normal stiffness of the structural elements depends on the modulus of elasticity of the material, E, the moment of inertia of the structural elements, I, and the cross sectional area of the structural elements, A. We calculated the bending and normal stiffness of the individual structural element, and then scaled the properties using the horizontal spacing of the structural element to determine an equivalent stiffness. For the bridge deck, we used a continuous beam in our analysis. Structural properties were provided by Exeltech. The FLAC input parameters for the structural elements are summarized in Table 9.

TABLE 9. STRUCTURAL INPUT PARAMETERS USED IN FLAC 2D MODEL

Structural Element	Density (slug/ft^3)	Modulus of Elasticity, E (psf)	Cross Section Area, A (ft^2)	Moment of Inertia, I (ft^4)	Perimeter (ft)
Pile	4.658	6.358E8	3.384	2.896	9.425
Beam (Bridge Deck)	0.137	5.184E8	84.028	5.488E3	n/a

Dynamic Loading Conditions

Earthquake loading is simulated in the FLAC 2D model by applying acceleration time histories to the base of the mesh, which we modeled at Elevation -160 feet. We selected two acceleration time histories to represent the design earthquake events. The first is a recorded acceleration time history at Orion 8244 station during the 1971 San Fernando earthquake. The second is a recorded acceleration time history at La Union station during the 1985 Michoacán earthquake. We obtained the acceleration time histories from our site specific seismic site response analysis (completed during Phase 1). Prior to propagating the motions through our model, the recorded acceleration time histories were scaled to generally match the anticipated AASHTO and OLE level shaking intensity at the project site.

For the AASHTO event, we used both the 1971 San Fernando earthquake and 1985 Michoacán earthquake acceleration time history. To evaluate the performance of the waterway slope and the approach structures considering the existing conditions we used the San Fernando acceleration time history. For the improved conditions, we used the 1985 Michoacán acceleration time history. Figure 15 presents the scaled input acceleration time histories applied in our FLAC2D model for the AASHTO design event.

For the OLE event, we used the 1971 San Fernando earthquake acceleration time history. This acceleration time history was used to evaluate the performance of the waterway slopes considering both the existing conditions and improved conditions. Figure 16 presents the scaled input acceleration time history applied in our FLAC 2D model for the OLE design event.

FLAC Results

The results of our numerical modeling analyses at the City Approach and Port Approach are presented in the following sections. The failure surfaces presented in our figures and discussed below are the failure surfaces critical to the seismic foundation performance of the approach structures. We identified these failure surfaces based on the horizontal displacement computed in our numerical model and the location of the approach structure foundations.

City Approach

Existing Conditions. The horizontal and vertical deformations computed at the City Approach after the AASHTO design event considering existing conditions are presented in Figures 17 and 18, respectively. The horizontal movement of the failure surface is estimated to be more than 2 feet. The vertical ground deformation is estimated to be about 2 to 8 feet. The vertical ground deformation calculated by FLAC 2D is based on shear deformation of the slope resulting from lateral spreading. It does not take into account post-liquefaction reconsolidation settlement. The total vertical ground deformation along the slope at the City Approach, including the liquefaction-induced settlement, is estimated to be approximately 2 to 9 feet after the AASHTO design earthquake event.

Figures 19 and 20 present the horizontal and vertical deformation computed after the OLE design event considering existing conditions at the City Approach. The horizontal movement of the failure surface is estimated to be less than about 2 inches. The vertical ground deformation resulting from this lateral movement is estimated to be about 1 to 2 inches. The total vertical ground deformation along the slope at the City Approach, including the liquefaction-induced settlement, is estimated to be approximately 3 to 13 inches after the OLE design earthquake event.

The FLAC 2D results for existing conditions show appreciable horizontal and vertical soil movement at the City Approach after the AASHTO and OLE design earthquake events. These results support our recommendation for ground improvement to stabilize the slope and foundation soil, as provided in our Phase 1 Geotechnical Engineering Services report.

Improved Conditions. The horizontal and vertical deformations computed after the AASHTO design earthquake event at the City Approach with our recommended ground improvement scheme implemented are presented in Figures 21 and 22, respectively. The horizontal movement of the failure surface is estimated to range from about 3 to 6 inches. The vertical ground deformation along the slope is estimated to be about 3 inches. Because foundation soils near the failure surface will be improved to mitigate soil liquefaction, ground settlement from post-liquefaction reconsolidation settlement is negligible.

Figures 23 and 24 show the horizontal and vertical deformations computed after the OLE earthquake event considering improved conditions at the City Approach. The maximum horizontal movement of the failure surface was estimated to be less than about 1 inch. The vertical ground deformation along the slope is estimated to range from about 1 to 2 inches. Because foundation soils near the failure surface will be improved to mitigate soil liquefaction, additional ground settlement from soil liquefaction will be negligible.

The results of our FLAC 2D analysis considering improved soil conditions at the City Approach show that our proposed ground improvement scheme reduces the expected horizontal and vertical deformations. The anticipated soil deformations should be reviewed by the structural engineer to confirm that they are within project tolerances for the City Approach bridge structure and meet the collapse prevention design objective.

Port Approach

Existing Conditions. The horizontal and vertical deformations computed at the Port Approach after the AASHTO design event considering existing conditions are presented in Figures 25 and 26, respectively. We estimate that the anticipated horizontal movement along the failure surface to be greater than 2 feet. The vertical ground deformation is estimated to be about 1 to 3 feet. As noted previously, the vertical ground deformation calculated by FLAC 2D is based on shear deformation of the slope resulting from lateral spreading and it does not take into account post-liquefaction reconsolidation settlement. The total vertical ground deformation along the slope at the Port Approach, including the liquefaction-induced settlement, is estimated to be approximately 1½ to 4½ feet after the AASHTO design earthquake event.

Figures 27 and 28 present the horizontal and vertical deformation computed after the OLE design event considering existing conditions at the Port Approach. The horizontal movement of the failure surface is estimated to be less than about 2 inches. The vertical ground deformation resulting from this lateral movement is estimated to be less than about 6 inches. The total vertical ground deformation along the slope at the Port Approach, including the liquefaction-induced settlement, is estimated to be approximately 9 to 13 inches after the OLE design earthquake event.

The FLAC 2D results show horizontal and vertical soil movement after the AASHTO design event that will likely cause instability in the piles supporting the Port Approach bridge structure. While the deformations computed after the OLE design event are smaller in magnitude, we believe that the piles may still become unstable. These results support our recommendation for ground improvement to stabilize the slope and foundation soil, as provided in our Phase 1 Geotechnical Engineering Services report.

Improved Conditions. The horizontal and vertical deformations computed at the Port Approach considering the AASHTO design earthquake event and our recommended ground improvement scheme are presented in Figures 29 and 30, respectively. The horizontal movement along the failure surface is estimated to be less than 1 inch. The vertical ground deformation resulting from this lateral movement is also estimated to be less than 1 inch. Because foundation soils near the critical failure surface will be improved to mitigate soil liquefaction, the additional ground settlement from soil liquefaction is negligible.

Figures 31 and 32 show the horizontal and vertical deformations computed after the OLE earthquake event considering improved conditions at the Port Approach. We could not clearly define a failure surface for the improved conditions; therefore, deformations are presented relative to the Port Approach foundation footprint. The horizontal movement within the foundation footprint is estimated to be less than 1 inch. The vertical ground deformation within the foundation footprint resulting from this lateral movement is also estimated to be less than 1 inch. Again, because foundation soils near the critical failure surface will be improved to mitigate soil liquefaction, the additional ground settlement from soil liquefaction is negligible.

The results of our FLAC 2D analysis show that the recommended ground improvement scheme is effective in stabilizing the slope and reduces the slope movement and the impact of the slope movement to the pile foundations at the Port Approach. The anticipated soil deformations should be reviewed by the structural engineer to confirm that they are within project tolerances for the Port Approach bridge structure and meet the collapse prevention design objective.

SEISMIC PERFORMANCE EVALUATION OF CENTER SPAN BRIDGE STRUCTURE

Foundation Analysis

Center Span Vertical Pile Capacity

The settlements computed in our simplified liquefaction analysis are an estimation of the settlement of the soil and do not take into account the soil-pile interaction effect. The piles for the Center Span structure are embedded in partially and fully liquefiable soil. Liquefaction and the associated settlement can cause downward friction (downdrag) on piles. In order to assess the axial capacity of each pier, we completed axial pile capacity analysis that includes the effect of downdrag force induced by soil liquefaction.

Figures 33 through 36 show the soil profiles used to evaluate the axial pile capacity for Piers 1 through 4 considering the AASHTO design earthquake event. Figures 37 through 40 show the soil profiles used to evaluate the axial pile capacity for Piers 1 through 4 considering the OLE design earthquake event. Axial loading information was provided by H&H and is presented in Appendix A.

For the AASHTO design earthquake event, the results of our analysis indicate that foundation failure or "plunging" will likely occur at Pier 3 where most of the soils supporting the piles liquefy. Piles supporting Piers 1, 2 and 4 were found to have adequate axial capacity to support the dead load of the bridge with a FS of at least 1.5. The results of our axial capacity analysis under the OLE design earthquake event indicate that piles supporting Piers 1 through 4 will have adequate axial capacity to support the dead load of the bridge, with a FS of at least 2.0.

Center Span Lateral Pile Capacity

We analyzed the lateral pile capacity of the Center Span piers using the computer program GROUP developed by Ensoft, Inc. GROUP is a three-dimensional (3D) computer program used to analyze the behavior of piles in a group subjected to axial, transverse, longitudinal and moment loading. The program computes the distribution of loads (vertical and lateral) and overturning moment in up to three orthogonal axes. We completed our analyses using the soil properties presented in Table 10.

TABLE 10. GROUP SOIL PROPERTIES

Material Property		Lower Alluvium 1	Liquefied Lower Alluvium 1	Reduced Strength Lower Alluvium 1	Glacial Till
γ'	(psi)	0.0323	0.0323	0.0323	0.0410
c	(psi)	0	0	0	0
ϕ	(deg)	34	10	23	36
ψ	(deg)	0	0	0	0
E_{50}	%	0	0	0	0
K		60	20	40	90
Soil Model		Sand	Liquefied Sand	Sand	Sand

We completed lateral pile capacity analysis for both the AASHTO and OLE design events. Structural loading information was provided by H&H for the AASHTO event. For the OLE design event, we scaled the AASHTO loads by a factor of 0.43.

Based on our review of the available drawings and information of the Center Span structure, we developed two pile group models for our lateral pile capacity evaluation, one for Piers 1 and 4 and the other representative of Piers 2 and 3. The pile group layout for Pier 1 and Pier 4 is presented in Figure 41. The pile group layout for Pier 2 and Pier 3 is presented in Figure 42. The lateral displacements, shear force and bending moment in the transverse and longitudinal direction were computed for each pile. For brevity, we selected a set of piles at each pier that best captured the response of the pile group. These piles are highlighted in Figures 41 and 42.

The AASHTO design event results for Piers 2, 3 and 4 could not be computed because the analysis software, GROUP, computed loads and displacements that exceeded the capabilities of the software (that is, the program could not reach convergence). The results indicate that the piles supporting Piers 2, 3 and 4 are unstable under the AASHTO design earthquake event. The results of our analysis for Pier 1 show that up to 50 inches of lateral cap deformation is expected. The displacement, shear force and bending moment at Pier 1 under AASHTO design loads in both the longitudinal and transverse direction are presented in Appendix C, Figures C-1 through C-16.

The displacement, shear force and bending moment at Piers 1 through 4 under the OLE design loads in both the longitudinal and transverse direction are also presented in Appendix C. Pier 1 results are shown in Figures C-17 though C-34, Pier 2 results are shown in Figures C-35 through C-52, Pier 3 results are shown in Figures C-53 through C-70 and Pier 4 results are shown in Figures C-71 through C-86.

For the OLE design earthquake event, our results show only a small amount of lateral pile cap deformation at Piers 1, 3 and 4. Because of the deeper liquefied soil zone, we calculated pile cap lateral deformations of up to approximately 14 inches at Pier 2. In order to maintain strain compatibility between bridge piers, the lack of lateral support at Pier 2 may cause the lateral load resisted by other piers to increase as a result of load sharing.

Foundation Soil Springs

At the request of H&H, we developed preliminary soil springs for use in the structural analysis of the Center Span to evaluate the seismic performance of the structure. Based on our analysis, the Center Span foundations will likely be unstable under the AASHTO design event due to liquefaction of the foundation soils. Accordingly, we are not able to provide foundation soil springs for the AASHTO design event at this time because the spring values are highly dependent on the selected foundation mitigation option.

Using the results of our axial and lateral pile analyses, we developed foundation soil springs for the OLE design event for use in the structural analysis of the Center Span. Our pile foundation analyses indicate that appreciable lateral deformation is anticipated at Pier 2, resulting in a lower lateral soil spring. Table 11 presents our recommended soil springs for the OLE event.

TABLE 11. CENTER SPAN FOUNDATION SOIL SPRINGS, OLE DESIGN EVENT

Center Span Pier No.	Vertical Spring	Longitudinal Shear Spring	Transverse Shear Spring
	(kip/in)	(kip/in)	(kip/in)
Pier 1	29,350	18,980	41,300
Pier 2	44,350	90	110
Pier 3	44,020	14,000	22,000
Pier 4	27,810	15,870	28,350

Note:

kip/in: kips per inch

Acceleration Time Histories

The structural design team requested that we provide two orthogonal sets of earthquake acceleration time histories for both the AASHTO and OLE design events. We considered the same earthquake time histories for the Center Span as were considered for the City Approach and Port Approach in our Phase 1 Geotechnical Engineering Services report. For a detailed discussion of our selection and scaling process please refer to our Phase 1 Geotechnical Engineering Services report.

Our recommended scaled acceleration time histories for the Center Span AASHTO design event are shown in Figures 43 and 44. Our recommended scaled acceleration time histories for the Center Span OLE design event are shown in Figures 45 and 46.

CONCLUSIONS AND RECOMMENDATIONS

We completed analyses using numerical modeling methods to evaluate the impacts of design level ground shaking and the resulting soil liquefaction and lateral spreading on the performance of the City and Port approach foundations and structures and to evaluate the effects of our recommended ground improvement schemes. We also completed pile capacity analysis to evaluate the foundation performance of the Center Span structure when subjected to the design earthquake events. The following is a summary of our conclusions from these analyses:

City Approach

- Considering existing soil conditions, AASHTO level ground shaking is expected to induce liquefaction, lateral spreading and slope movement that will likely cause foundation and structure instability. Without ground improvement the slope and structure near the waterway are expected to experience more than 2 feet of horizontal displacement and up to 9 feet of total vertical ground deformation. With the proposed ground improvement, these displacements are expected to reduce to 3 to 6 inches of horizontal displacement and less than 3 inches of total vertical deformation.
- When the existing soils are subjected to the OLE design earthquake event the slope and structure near the waterway are expected experience less than 2 inches of horizontal displacement and approximately 3 to 13 inches of total vertical ground deformation. With the proposed ground improvement these displacements are reduced to less than 1 inch of horizontal displacement and 1 to 2 inches of total vertical deformation.

Port Approach

- When subjected to the AASHTO design earthquake event the slope and structure near the waterway are expected to experience more than 2 feet of horizontal displacement and up to 4½ feet of total vertical ground deformation. This magnitude of soil movement will likely result pile foundation instability and instability of the Port Approach bridge structure. With the proposed ground improvement, these displacements are reduced to less than 1 inch of horizontal displacement and less than 1 inch of total vertical deformation.
- When subjected to the OLE design earthquake event and without ground improvement the slope and structure near the waterway is expected to experience less than 2 inches of horizontal displacement and less than 6 inches of total vertical ground deformation. With ground improvement these displacements are reduced to less than 1 inch of horizontal displacement and less than 1 inch of total vertical deformation.
- The proposed ground improvement schemes for the AASHTO and OLE design events reduce the horizontal deformation at Bent 11, but do not address the vertical post-liquefaction ground deformation. The vertical deformations may be mitigated by constructing a new foundation system at Bent 11. Alternatively, if a new foundation system is constructed at Pier 4 of the Center Span structure, the bridge may be able to span the vertical deformations at Bent 11. We recommend that the structural engineer evaluate the seismic performance of Bent 11 in conjunction with the seismic upgrades planned for the Center Span structure.

Ground Improvement

- Results of our analyses indicate that our recommended ground improvement schemes for the City and Port Approaches are effective in reducing the slope movement and ground deformation under both the AASHTO and OLE design earthquake events. These ground improvement schemes may be further refined once a better understanding of the collapse prevention and repairable damage deformation tolerances have been defined.
- We recommend that the project structural engineer review the results of our analyses for the City and Port Approaches to determine if the horizontal and vertical ground deformations for our AASHTO and OLE ground improvement schemes meet the collapse prevention and repairable damage design criteria.

Center Span

- Under the AASHTO design earthquake event the axial pile capacity at Pier 3 is exceeded and the foundation will likely loose bearing capacity and plunge. The axial pile capacity at Piers 1, 2 and 4 is not exceeded. All four piers are expected to lose lateral pile support due to soil liquefaction, resulting in lateral deformation in excess of 4 feet. Accordingly, we recommend that a new foundation system or ground improvement around the existing foundations be designed and implemented.
- When subjected to the OLE design earthquake event the axial pile capacity is not exceeded. The lateral pile displacements at Piers 1, 3 and 4 are expected to be less than 1 inch. Piles supporting Pier 2 are expected to lose some lateral support, resulting in up to 14 inches of lateral deformation. We recommend that the effect of the loss of support at Pier 2 be evaluated by the project structural engineer for conformance with collapse prevention criteria.

Construction considerations to support our conclusions and recommendations are provided in the following section.

CONSTRUCTION CONSIDERATIONS

Based on our analyses, we conclude that ground improvement or new foundations are needed to provide foundation stability to the City Approach structure, the Port Approach structure and the Center Span structure. The following sections provide a general description of the construction considerations for new foundation systems and ground improvement methods that may be considered for this project.

Foundation Replacement

The timber pile supported piers at the Center Span require a combination of vertical and lateral support to withstand the AASHTO level design event. It is our opinion that either driven steel piles or drilled shafts constructed on both sides of the bridge and adjacent to the existing piers may be considered to provide additional foundation support to the existing bridge structure. Construction considerations for both drilled shafts and driven steel pipe piles are presented below.

Drilled Shaft Foundations

Drilled shaft foundations are constructed by drilling a shaft of specified minimum diameter to a specified tip elevation. In over-water construction, the shafts are advanced from a barge or through a temporary cofferdam. Shafts may be constructed using either: 1) a single flight auger or 2) an oscillator/rotator.

Auger-drilled shafts are constructed by advancing a flight auger with cutting teeth on the leading edge. The drill rotates the auger into the ground until it fills with soil, then draws the auger out and spins it around to remove the drill cuttings. This process is repeated until the desired shaft depth is reached. Upon completion of the shaft excavation, steel reinforcement is placed in the shaft and the shaft is filled with concrete. Because of the presence of water and caving soils, we recommend that temporary casing and drilling mud/slurry be used to stabilize the shaft walls. Because the Thea Foss Waterway is an environmentally sensitive area, a proper spoils and slurry management plan will be required to reduce the likelihood of slurry or spoils migrating into the waterway.

Oscillated drilled shafts are constructed by pushing a thick-walled casing into the ground, and then excavating soils from within the casing using a clam bucket. The casing is advanced until the desired shaft tip elevation is achieved. Upon completion of the shaft excavation, steel reinforcement is placed in the shaft, and the shaft is filled with concrete. Because the hole is fully cased, this method of installation mitigates heave, the sloughing of soils and the potential for slurry migrating into the Thea Foss Waterway.

The key construction considerations for drilled shafts are:

- Over-water work
- Environmental impact
- Height clearance
- Shaft cleanout

Over-water construction of drilled shafts in the Thea Foss Waterway will likely require significant planning and permitting. Wet drilled shaft construction will require the use of slurry, which may present an environmental hazard if not properly contained. Because the drilled shafts are to be installed adjacent to the existing structure, height clearance should not be a concern.

Driven Steel Piles

Driven steel piles are constructed by advancing a prefabricated steel pile into the ground using a vibratory, hydraulic or diesel hammer. We anticipate that an impact hammer will be required to drive the piles to the intended tip depth that will provide the required pile capacity. Pile driving can be completed on a barge and may be more cost effective compared to the drilled shaft option.

The key construction considerations for driven piles are:

- Over-water work
- Pile driving impacts to fish

- Vibration monitoring
- Driveability

Over-water construction of driven piles in the Thea Foss Waterway will likely require significant planning and permitting. During installation, there may be significant noise and vibration when driving the piles to the appropriate embedment in the glacially consolidated materials. Because of the close proximity of downtown Tacoma, we recommend that a vibration monitoring plan be implemented during construction.

The piles should be installed using an appropriately sized pile-driving hammer. The pile hammers should be of sufficient size to drive the piling to attain the recommended embedments without damaging the pile. Selection of an appropriate pile hammer can reduce pile damage during driving. We recommend that a pile driveability analysis be completed after the final choice of pile size and hammer has been made. The driveability analysis should be based on the pile installation conditions, the pile hammer configuration and the maximum delivered energy from the potential hammer. The analysis is necessary to evaluate the induced stresses in the pile. The pile hammer, yield stress and wall thickness of the steel must be chosen such that the induced stresses in the pile do not exceed allowable levels.

Ground Improvement

The objective of the ground improvement is to mitigate soil liquefaction and lateral spreading at the City Approach and Port Approach structures. Based on our evaluation, we concluded that compaction grouting is the most appropriate ground improvement method for this project. Construction considerations for compaction grouting are presented below. For a detailed discussion of the other ground improvement methods considered for this project please refer to our Phase 1 Geotechnical Engineering Services report.

Compaction Grouting

Compaction grouting is a displacement-based ground improvement technique that increases the shear strength of the soil by increasing the friction angle (a result of soil densification) and by adding a cohesive strength component to the soil (the grout column).

The key construction considerations for compaction grout columns are:

- Required improved soil shear strength
- Environmental considerations
- Low overhead clearance and existing utilities
- Construction-related settlement issues

Compaction grouting can accommodate the overhead clearance requirements at the approach structures. Compaction grouting can also be completed at a batter to target areas under existing foundations and to aid in avoiding utility conflicts. Compaction grouting construction does not generate significant amounts of excess spoils. Lastly, low slump grout reduces the risk of grout contamination in the waterway. However, potential for surface blowouts and hydro-fracturing (a process where grout finds an underground preferential pathway and grout pressure and volume are lost) makes this option environmentally prohibitive for over-water ground improvement.

LIMITATIONS

We have prepared this report for use by the City of Tacoma for the Murray Morgan Bridge Rehabilitation Project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty or other conditions express or implied should be understood.

Please refer to Appendix D “Report Limitations and Guidelines for Use” for additional information pertaining to use of this report.

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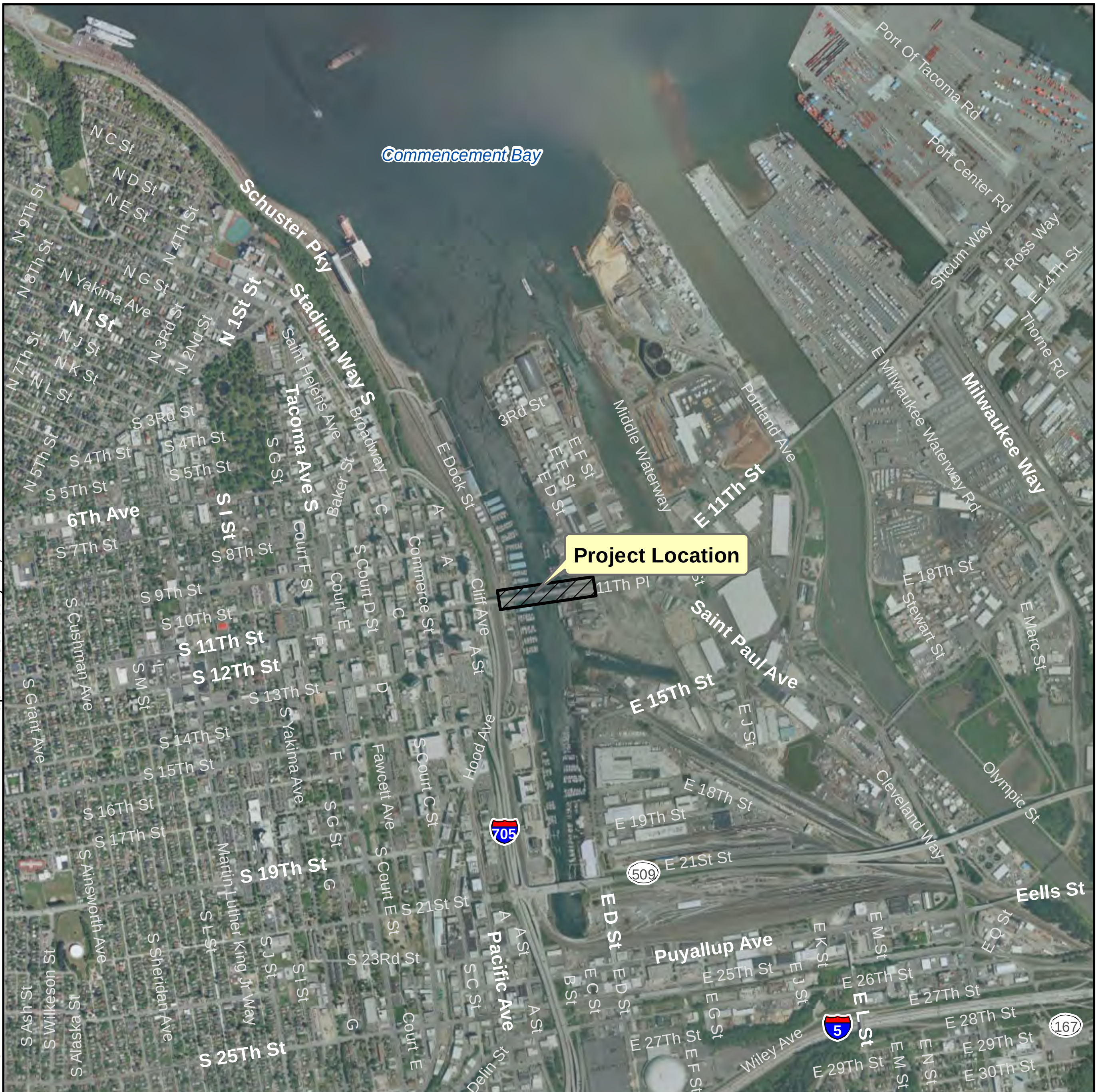

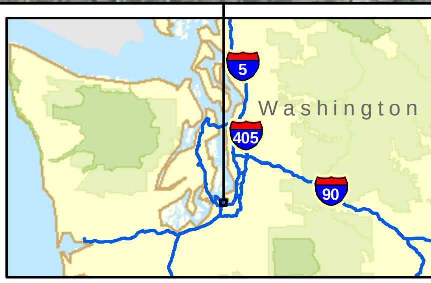

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Feet

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Data Sources: ESRI Data & Maps, Street Maps 2005

Transverse Mercator, Zone 10 N North, North American Datum 1983
North arrow oriented to grid north

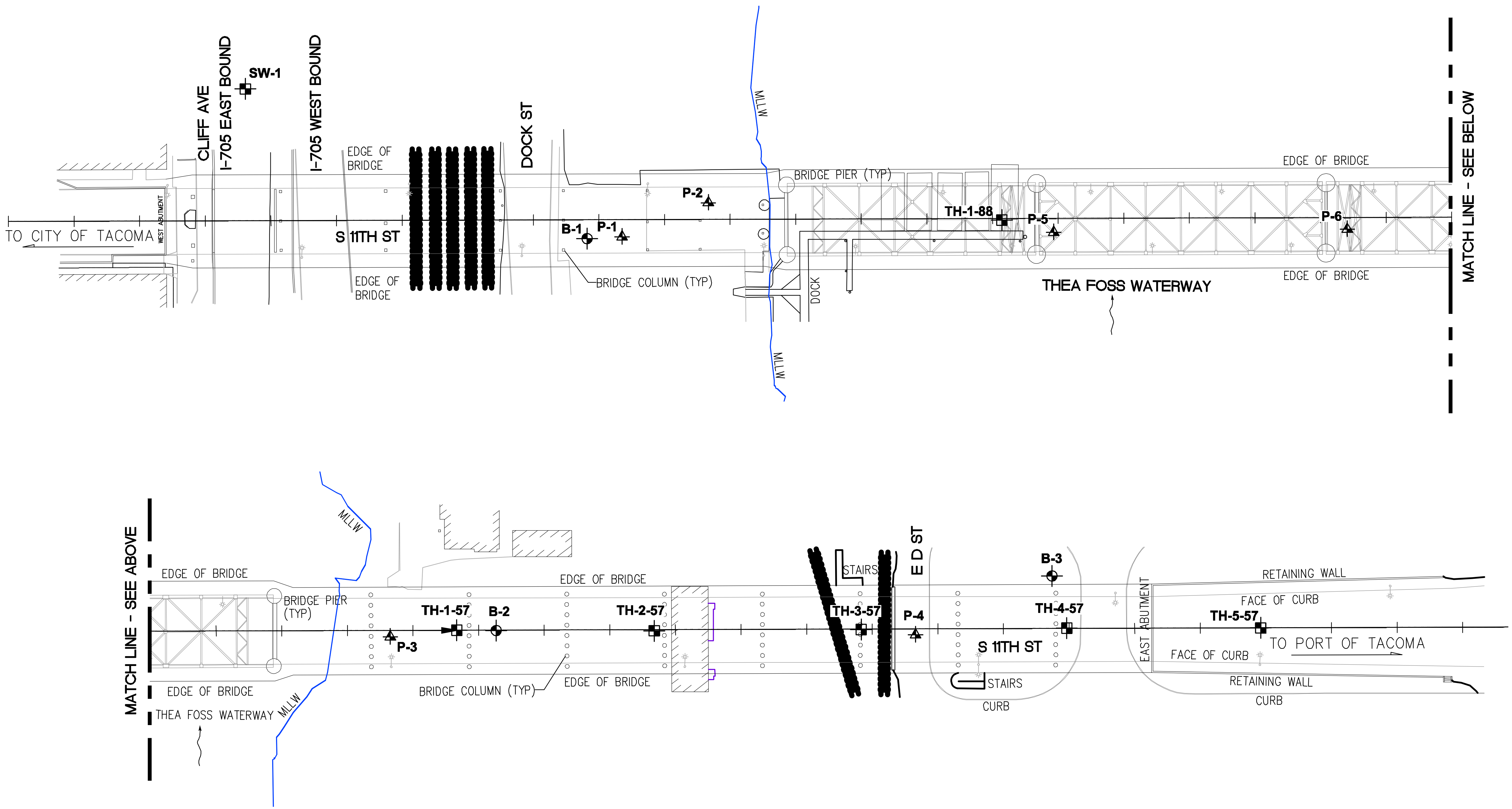
Vicinity Map

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 1

Office: TAC Path: P:\0\0570100\GIS\057010001_F1_VM.mxd Map Revised: July 14, 2011 AMM



\\TAC\PROJECTS\0\0570100\02\CAD\0057010001_FIG_2 SITE PLAN.DWG\TAB:F2 MODIFIED BY CVANSLYKE ON NOV 08, 2011 - 9:09

Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. This original drawing is in color. Black and white reproduction may result in difficulty differentiating between symbols.

Reference: Base drawings provided by David Evans and Associates, RFP drawings dated 6/23/2010.

Legend

B-1	Boring by GeoEngineers
P-1	Cone Penetration test or Seismic Cone Penetration test by GeoEngineers
TH-1-88	Boring by others

N W E S

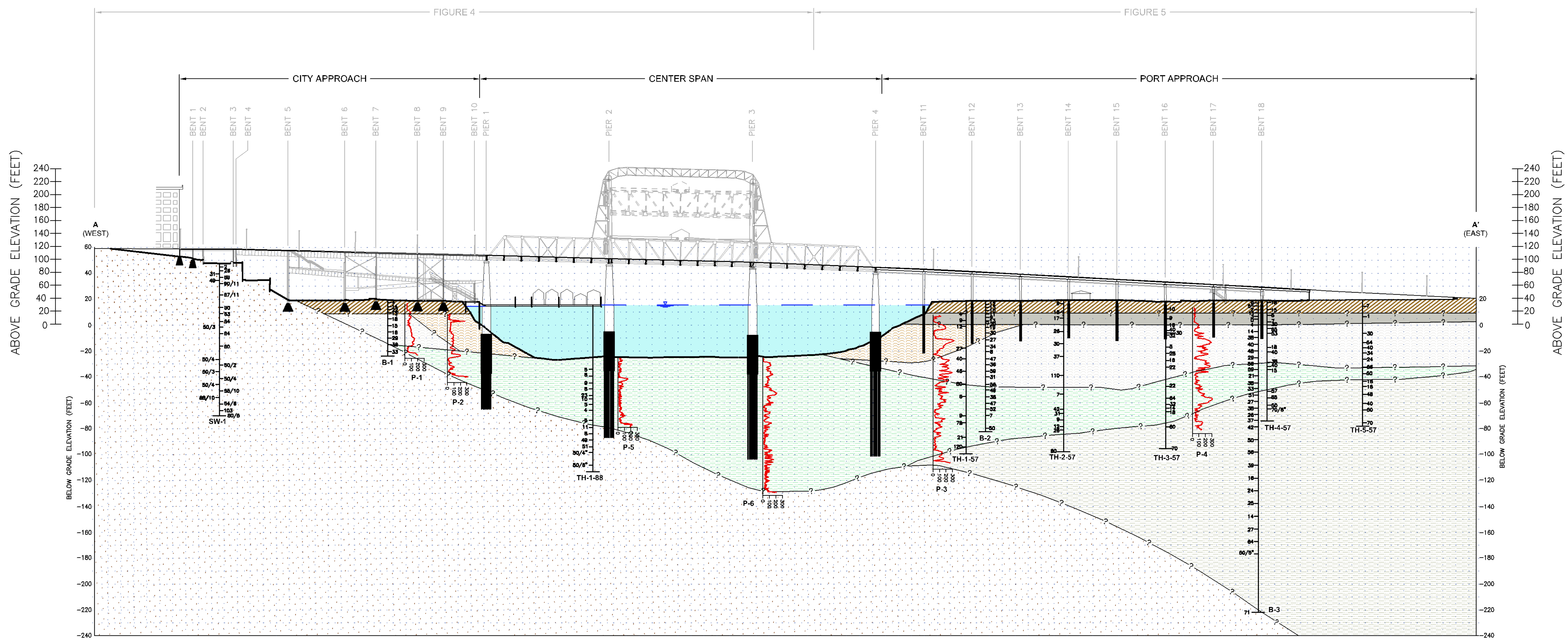
80 0 80
FEET

Site Plan

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 2



HORIZONTAL SCALE: 1" = 160'
VERTICAL SCALE: AS SHOWN
BELOW GRADE VERTICAL EXAGGERATION: 2X

160 0 160
FEET

Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. This original drawing is in color. Black and white reproduction may result in difficulty differentiating between symbols.

Reference: Base drawings provided by David Evans and Associates, RFP drawings dated 6/23/2010.

Legend

CPT Tip Resistance (TSF)

0 100 200 300

P-1 CPT Number and Approximate Location

Inferred soil contact

25 SPT N-value

B-1 Boring Number and Approximate Location

Fill

Tidal deposits - predominantly silt with organics

Upper Alluvium 1 - predominantly sand with layers of sandy silt

Upper Alluvium 2 - predominantly sand with layers of sandy silt

Lower Alluvium 1 - Interbedded sand and silt

Lower Alluvium 2 - predominantly sand with occasional layers of silt

Glacially consolidated soils

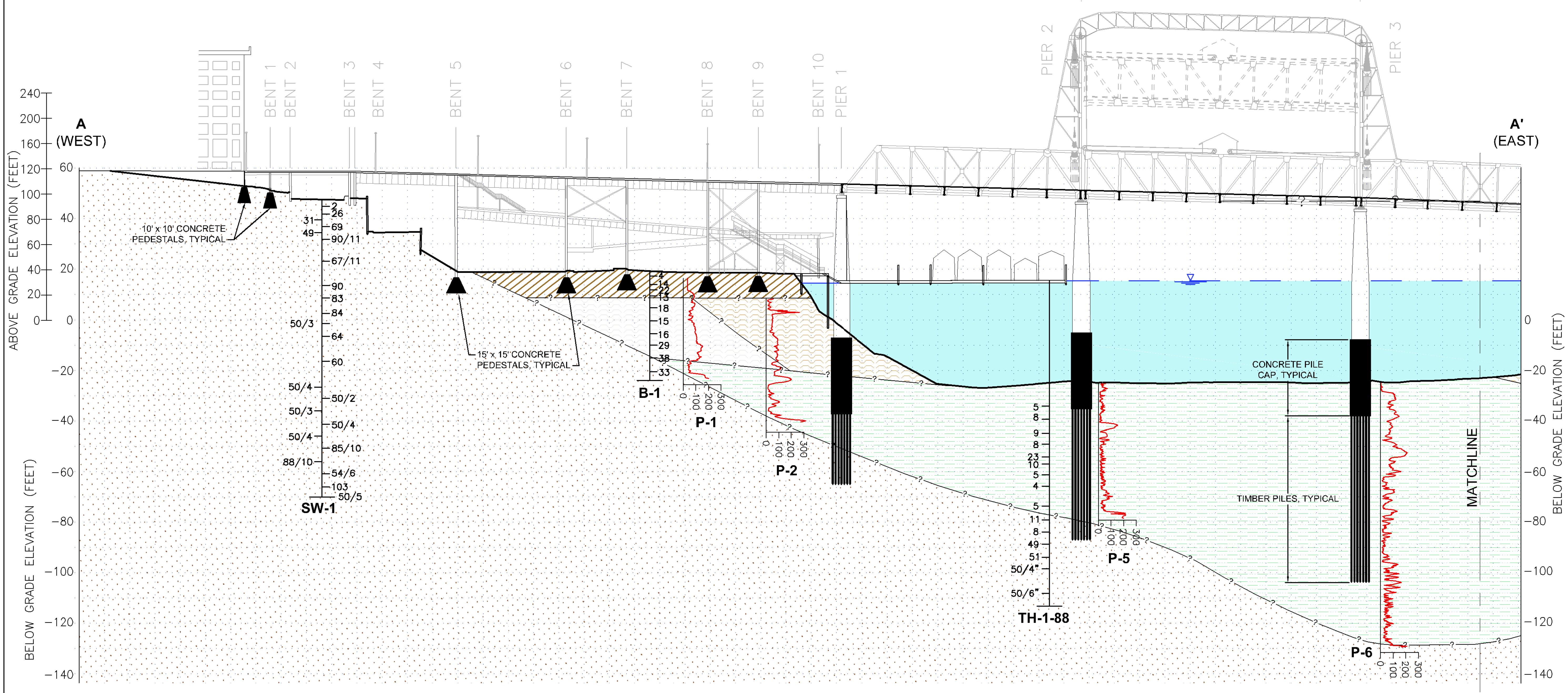
Generalized Cross Section

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 3

\\TAC\PROJECTS\0\0570100\02\CAD\0057010001_FIG_3 X-SECTION.DWG\TAB:FIG 3 MODIFIED BY CVANSLYKE ON NOV 04, 2011 - 9:02



HORIZONTAL SCALE: 1"= 80'
VERTICAL SCALE: AS SHOWN
BELOW GRADE VERTICAL EXAGGERATION: 2X

0 1"

Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. This original drawing is in color. Black an d white reproduction may result in difficulty differentiating between symbols.

Reference: Base drawings provided by Davi d Evans and Associates, RFP drawings dated 6/23/2010.

Legend

CPT Tip Resistance (TSF)

0 100 200 300

P-1 CPT Number and Approximate Location

? Inferred soil contact

25 SPT N-value

B-1 Boring Number and Approximate Location

Fill

Tidal deposits - predominantly silt with organics

Upper Alluvium 1 - predominantly sand with layers of sandy silt

Upper Alluvium 2 - predominantly sand with layers of sandy silt

Lower Alluvium 1 - Interbedded sand and silt

Lower Alluvium 2 - predominantly sand with occasional layers of silt

Glacially consolidated soils

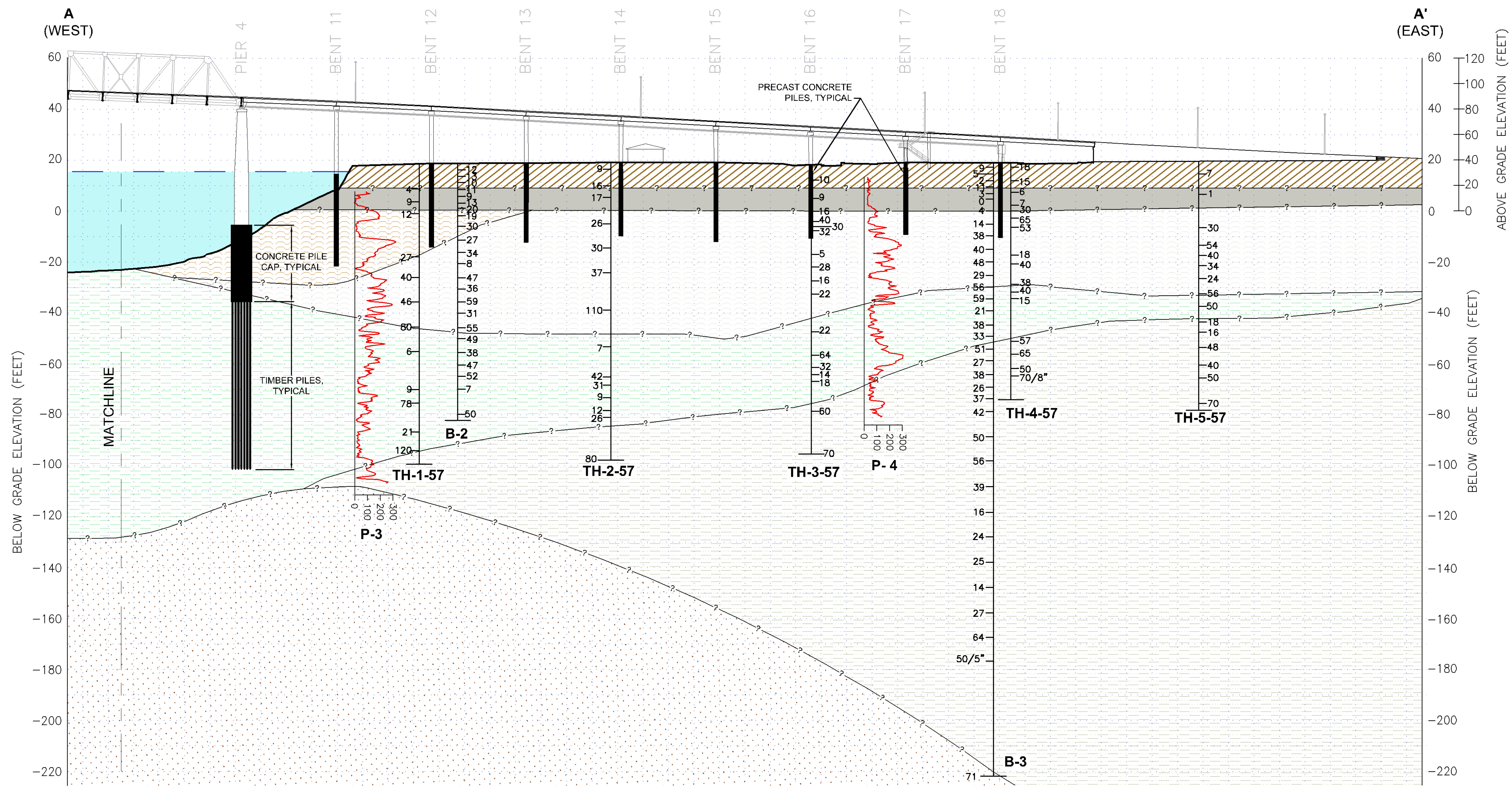
Detailed Cross Section - City Approach

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 4

P:\0\0570100\102\CAD\0057010001_Fig_4-5.dwg\TAB:Fig 4 modified by cvanslyke on Jul 19, 2011 - 10:46



Notes

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. can not guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.
3. This original drawing is in color. Black and white reproduction may result in difficulty differentiating between symbols.

Reference: Base drawings provided by David Evans and Associates, RFP drawings dated 6/23/2010.

Legend

CPT Tip Resistance (TSF)

0 100 200 300

P-1 CPT Number and Approximate Location

Inferred soil contact

25 SPT N-value

B-1 Boring Number and Approximate Location

Fill

Tidal deposits - predominantly silt with organics

Upper Alluvium 1 - predominantly sand with layers of sandy silt

Upper Alluvium 2 - predominantly sand with layers of sandy silt

Lower Alluvium 1 - Interbedded sand and silt

Lower Alluvium 2 - predominantly sand with occasional layers of silt

Glacially consolidated soils

0 1"

HORIZONTAL SCALE: 1"= 80'
VERTICAL SCALE: AS SHOWN
BELOW GRADE VERTICAL EXAGGERATION: 2X

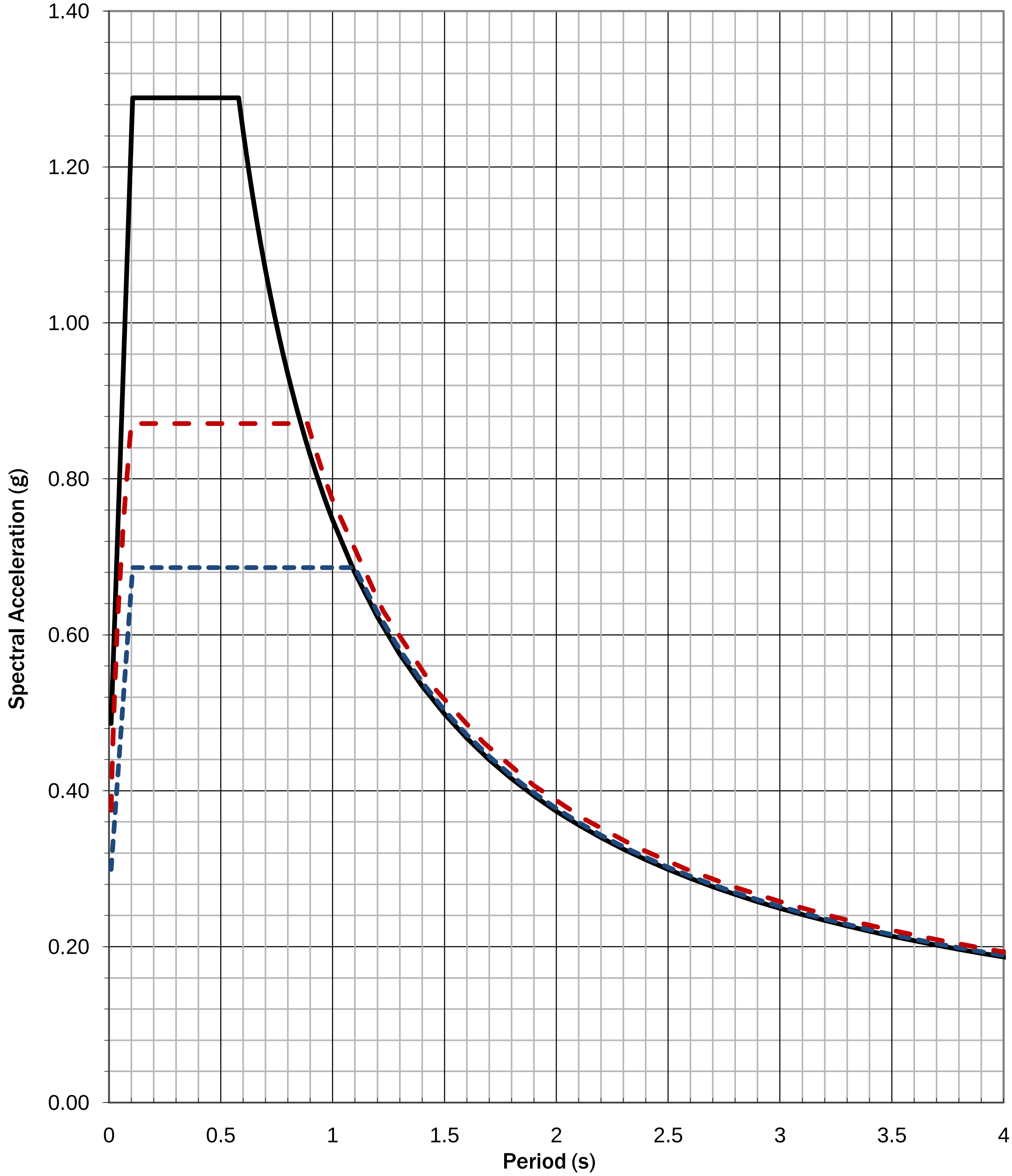
Detailed Cross Section - Port Approach

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 5

P:\0\0570100\02\CAD\005701000_Fig_4-5.DWG\TAB:Fig 5 MODIFIED BY CVANSLYKE ON JUL 19, 2011 - 10:46



City Approach Recommended Site Specific Design Response Spectrum for 1000-YR Event

Center Span Recommended Site Specific Design Response Spectrum for 1000-YR Event

Port Approach Recommended Site Specific Design Response Spectrum for 1000-YR Event

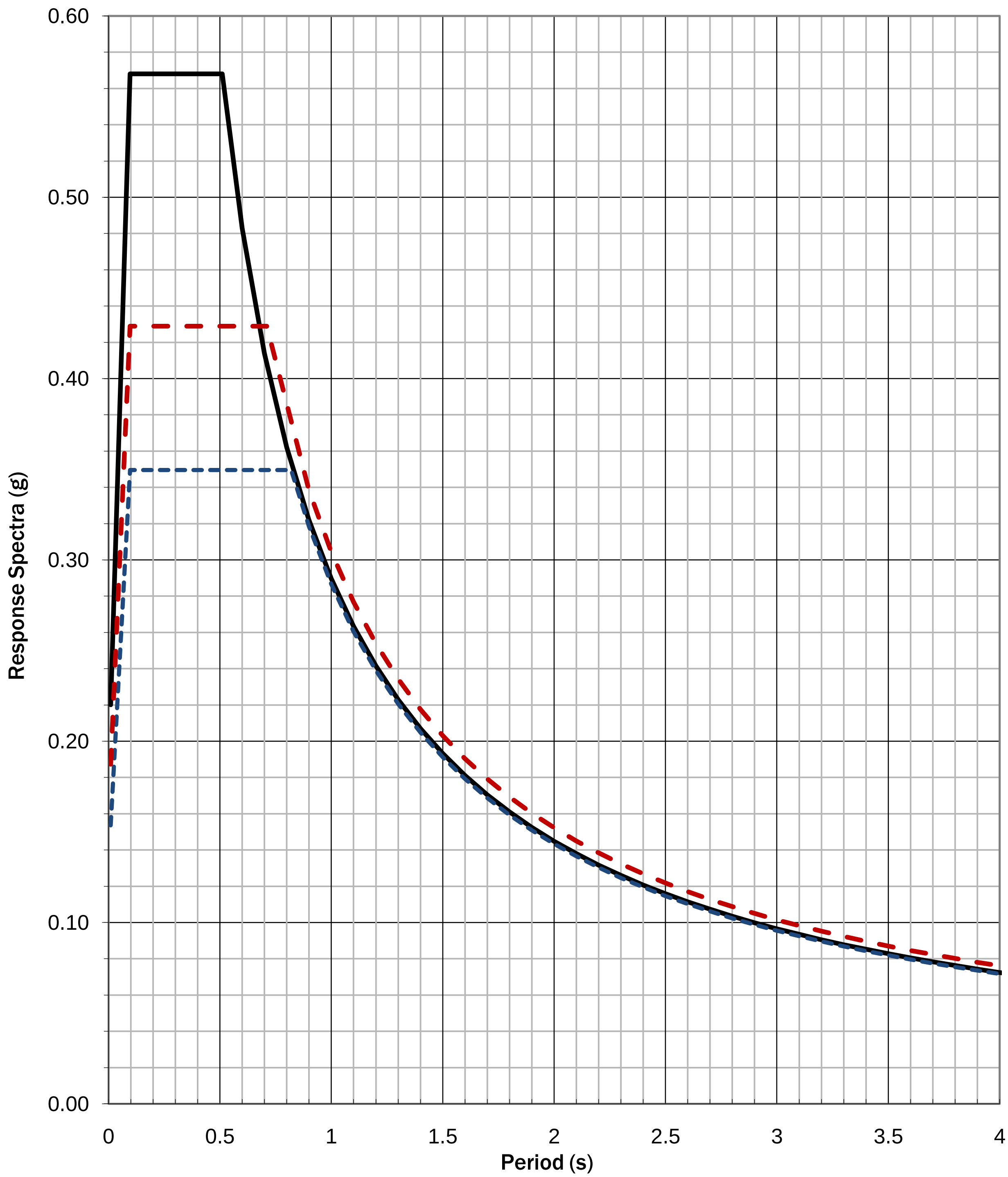
5% Damped Design Response Spectra AASHTO (1000-YR) Event

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 6

SharePoint: 0057010001\Technical Analysis\Site Response Analysis\Murray Morgan Bridge_Preliminary Design Response Spectra_1000.xlsx WLT:khc 7/18/11



City Approach Recommended Site Specific Design Response Spectrum for 108-YR Event

Center Span Recommended Site Specific Design Response Spectrum for 108-YR Event

Port Approach Recommended Site Specific Design Response Spectrum for 108-YR Event

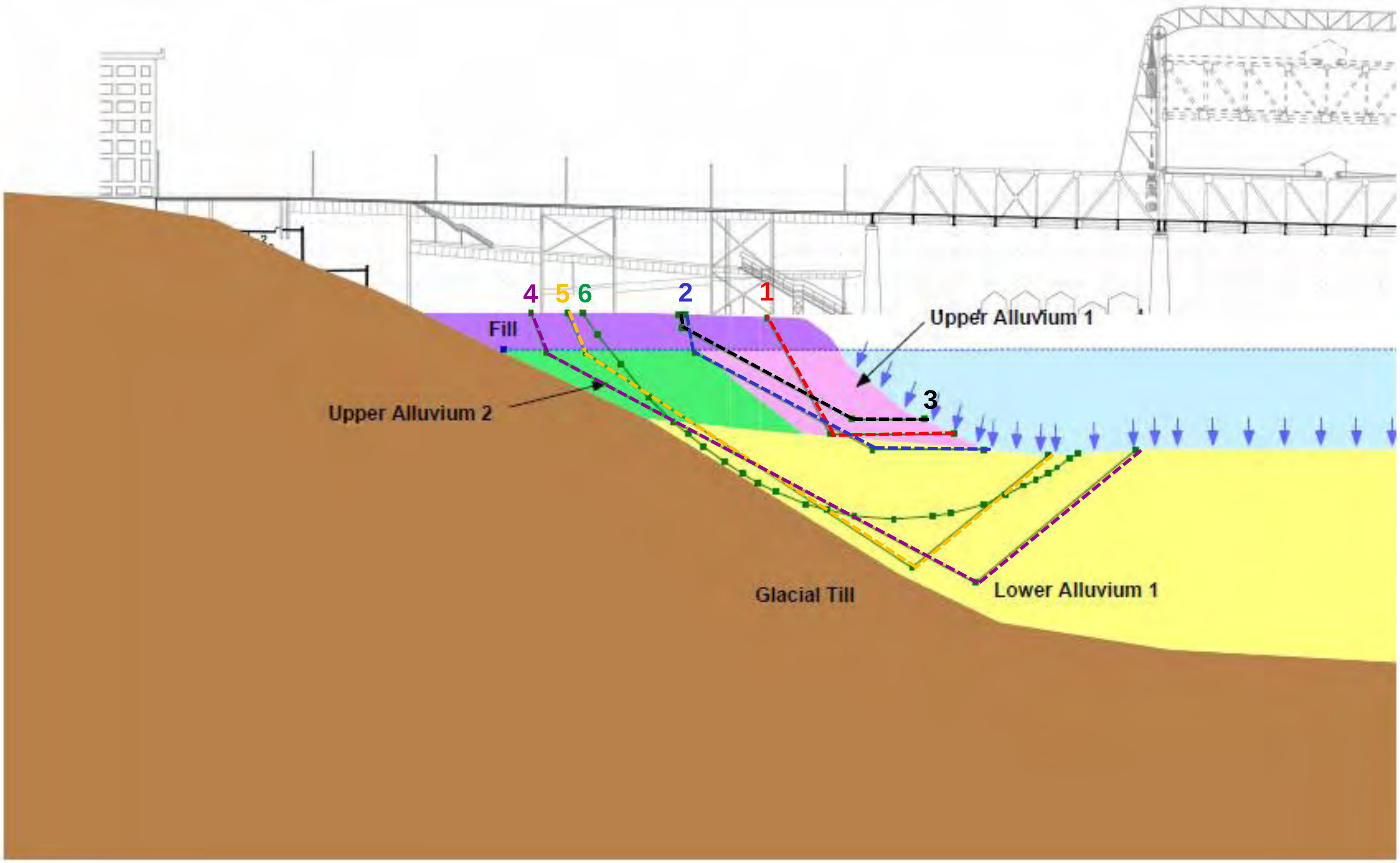
5% Damped Design Response Spectra
OLE (108-YR) Event

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 7

SharePoint: 0057010001\Technical Analysis\Site Response Analysis\Murray Morgan Bridge_Preliminary Design Response Spectra_108.xlsx WLT:khc 7/18/11



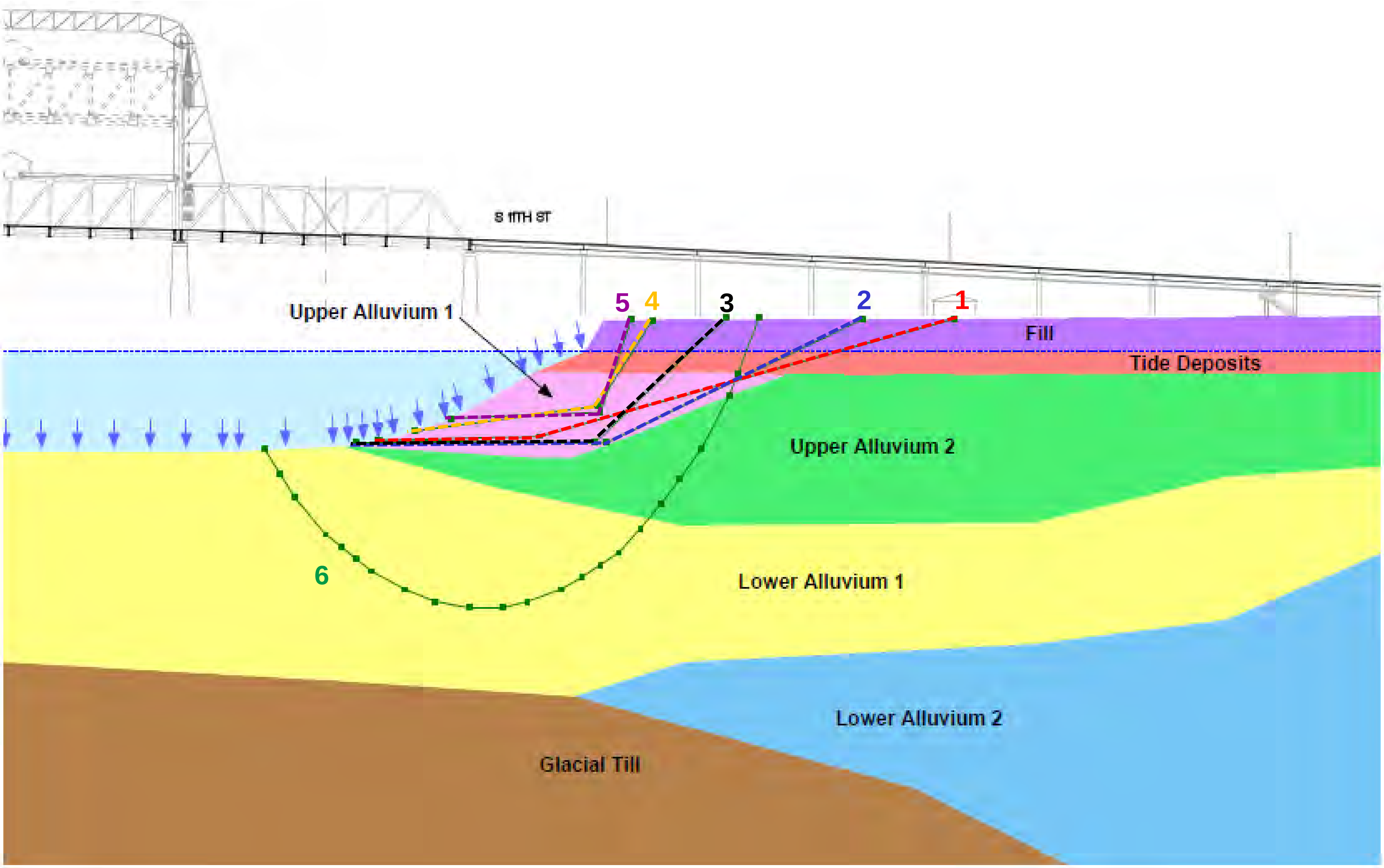
Legend

--- Failure Surface

Note:

1. Drawing not to scale

City Approach Failure Surfaces	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 8



Legend

--- Failure Surface

Note:

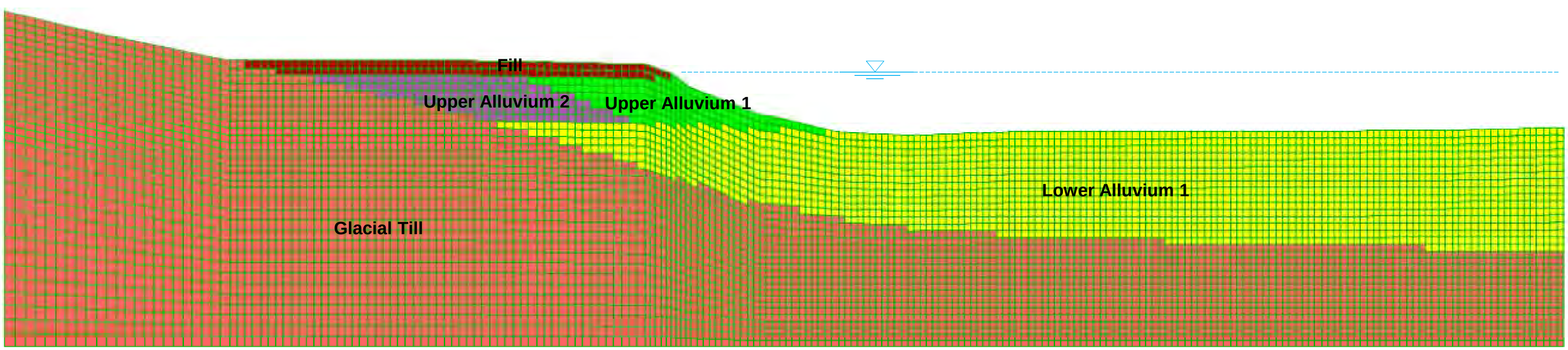
1. Drawing not to scale

Port Approach Failure Surfaces

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 9

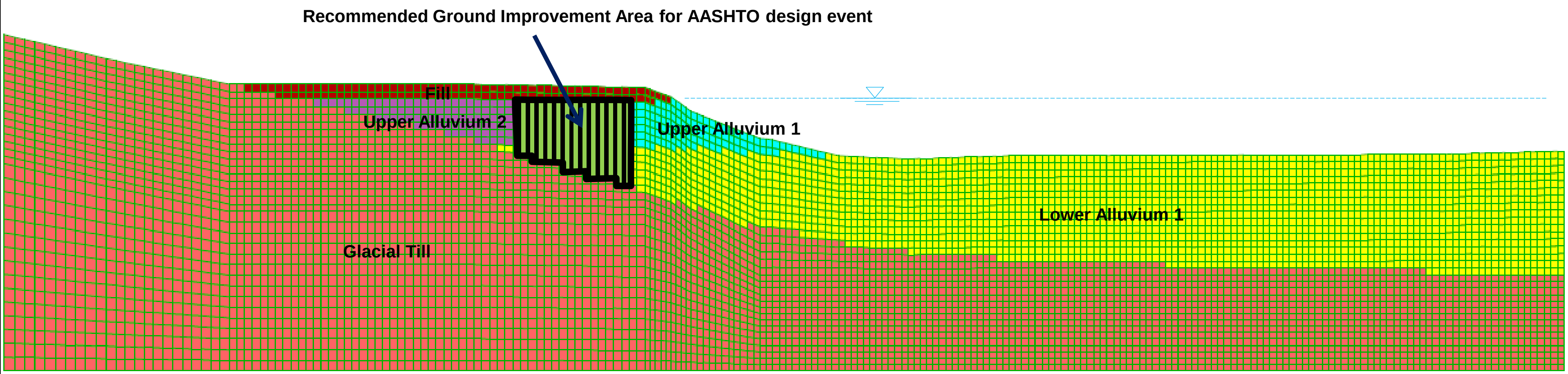


City Approach
FLAC 2D Mesh; Profile for Existing Conditions

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 10



City Approach

FLAC 2D Mesh; Profile with AASHTO Ground Improvement

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 11

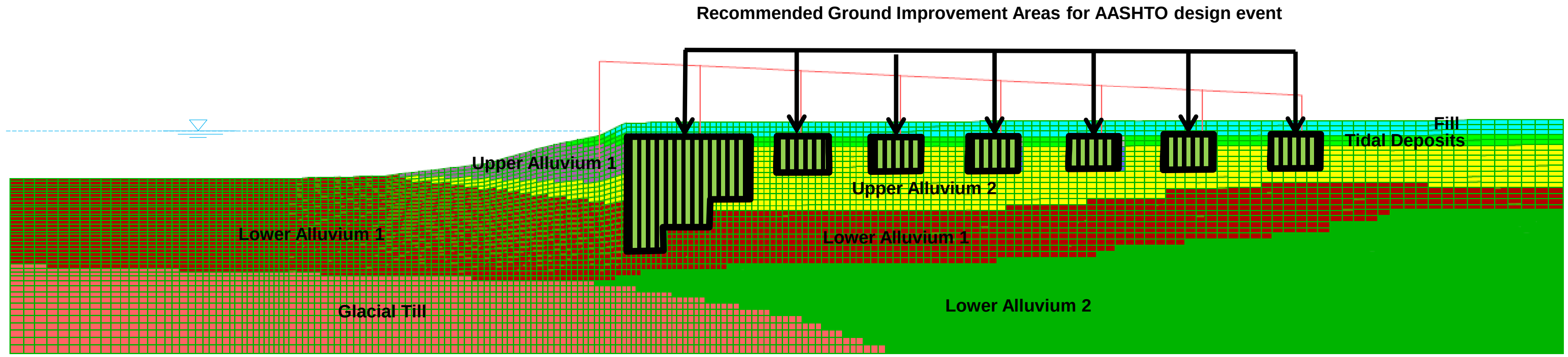
Port Approach

FLAC 2D Mesh; Profile for Existing Conditions

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 12



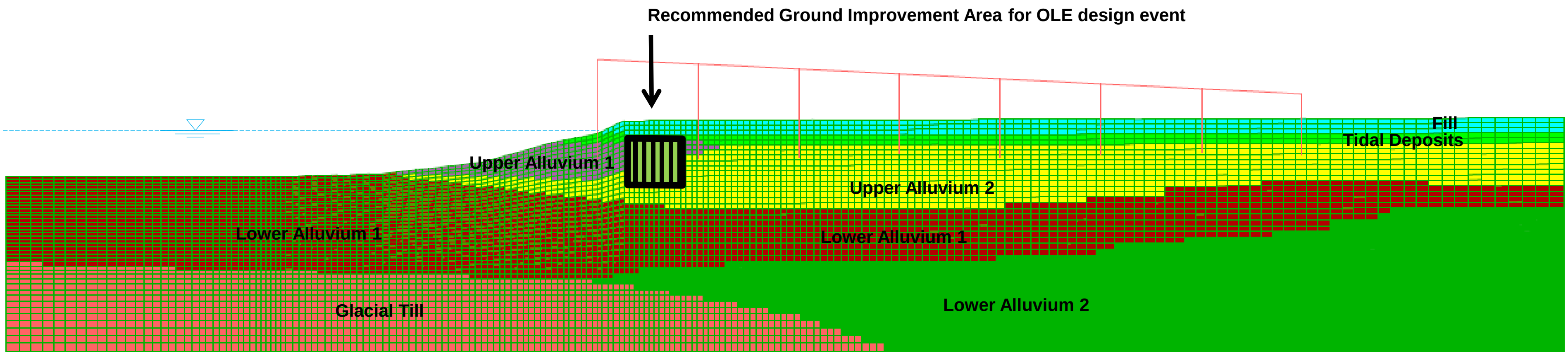
Port Approach

FLAC 2D Mesh; Profile with AASHTO Ground Improvement

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 13



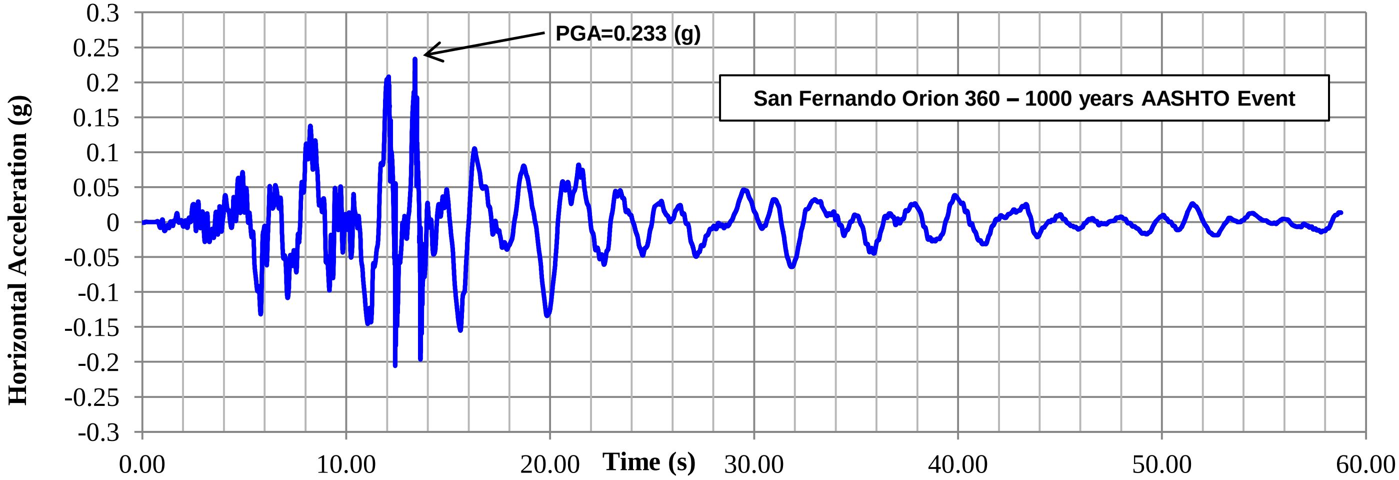
Port Approach

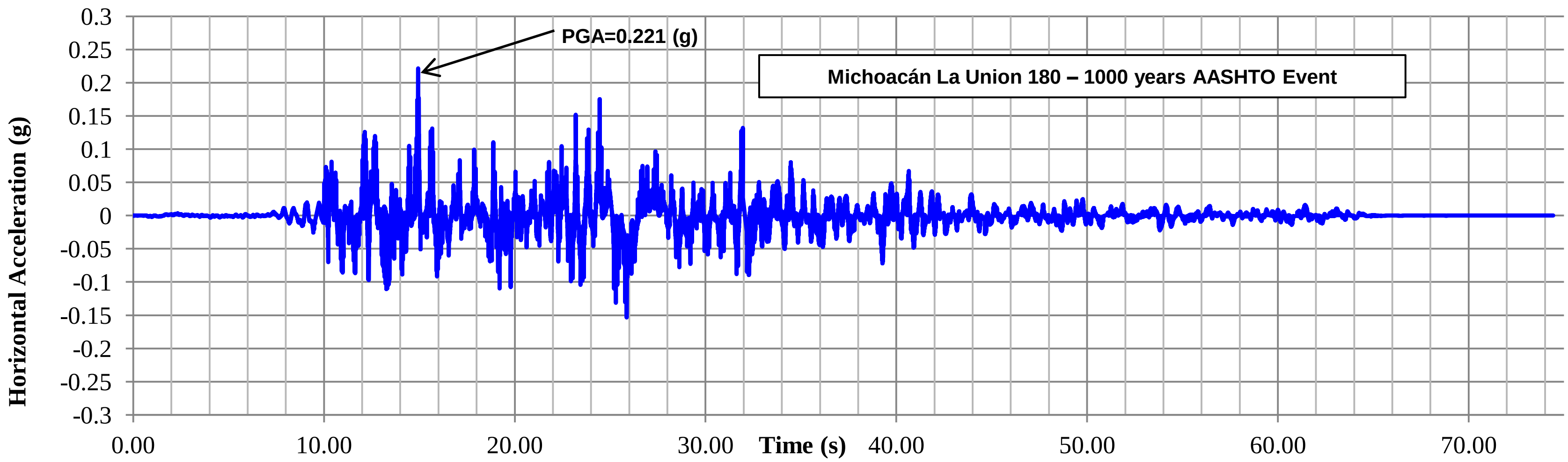
FLAC 2D Mesh; Profile with OLE Ground Improvement

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

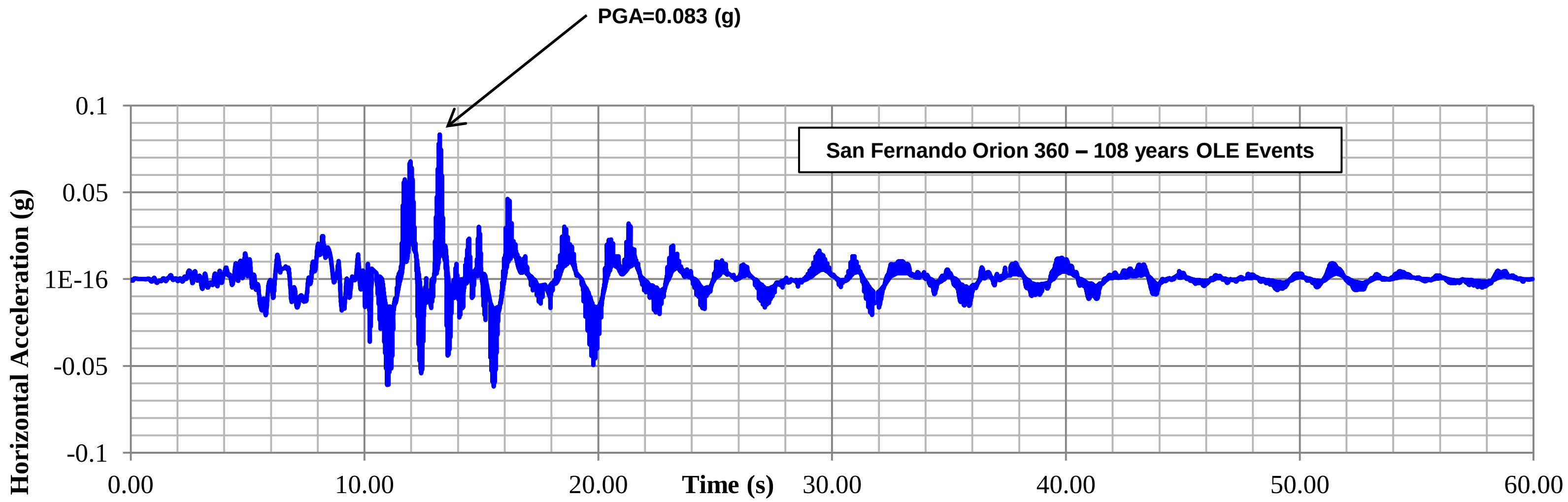
GeoEngineers

Figure 14





Time Histories of San Fernando Orion and Michoacán La Union, AASHTO Events at FLAC Model Base	
Murray Morgan Bridge Rehabilitation Project **Tacoma, Washington**	
GeoEngineers	Figure 15

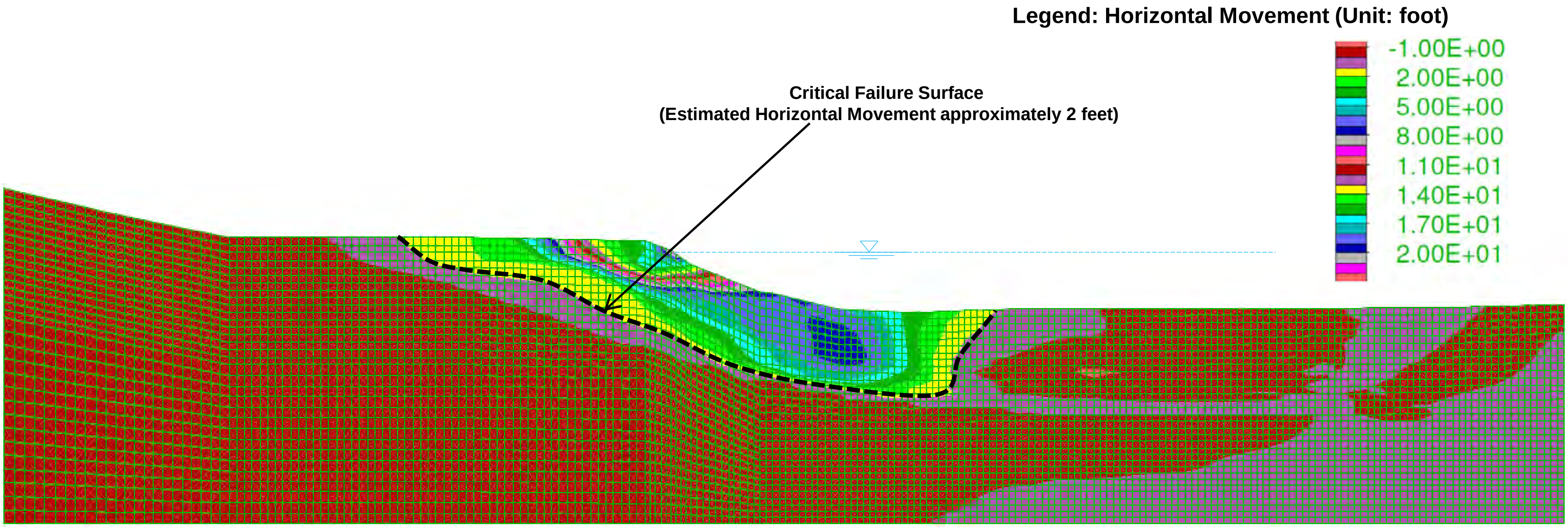


Time History of San Fernando Orion, OLE Event at FLAC Model Base

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 16



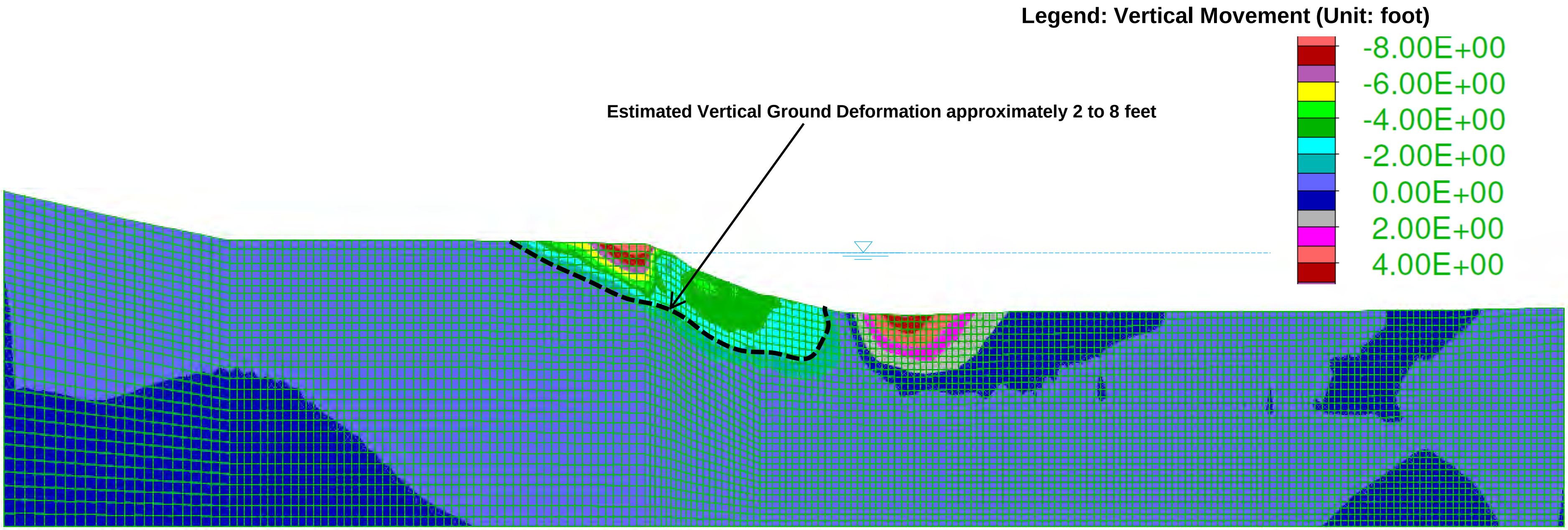
City Approach

Horizontal Ground Deformation Contour
(Existing Ground Conditions, San Fernando Orion, AASHTO Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

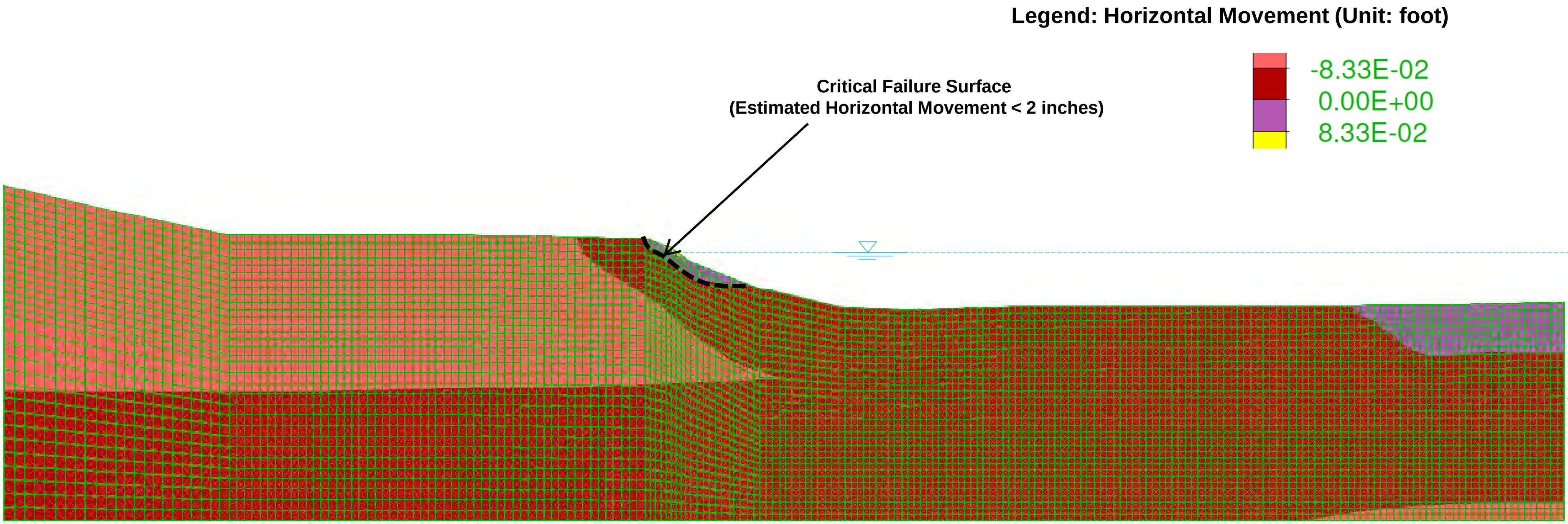
GEOENGINEERS

Figure 17



Note: Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

City Approach Vertical Ground Deformation Contour (Existing Ground Conditions, San Fernando Orion, AASHTO Event)	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 18

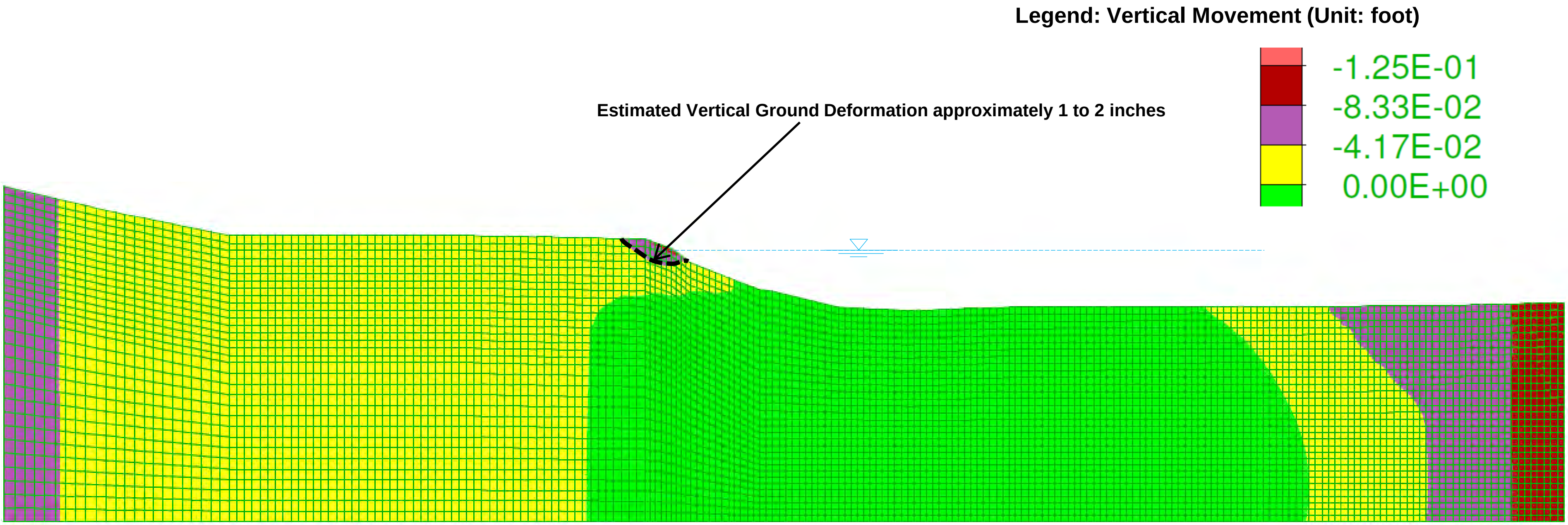


City Approach
Horizontal Ground Deformation Contour
(Existing Ground Conditions, San Fernando Orion, OLE Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

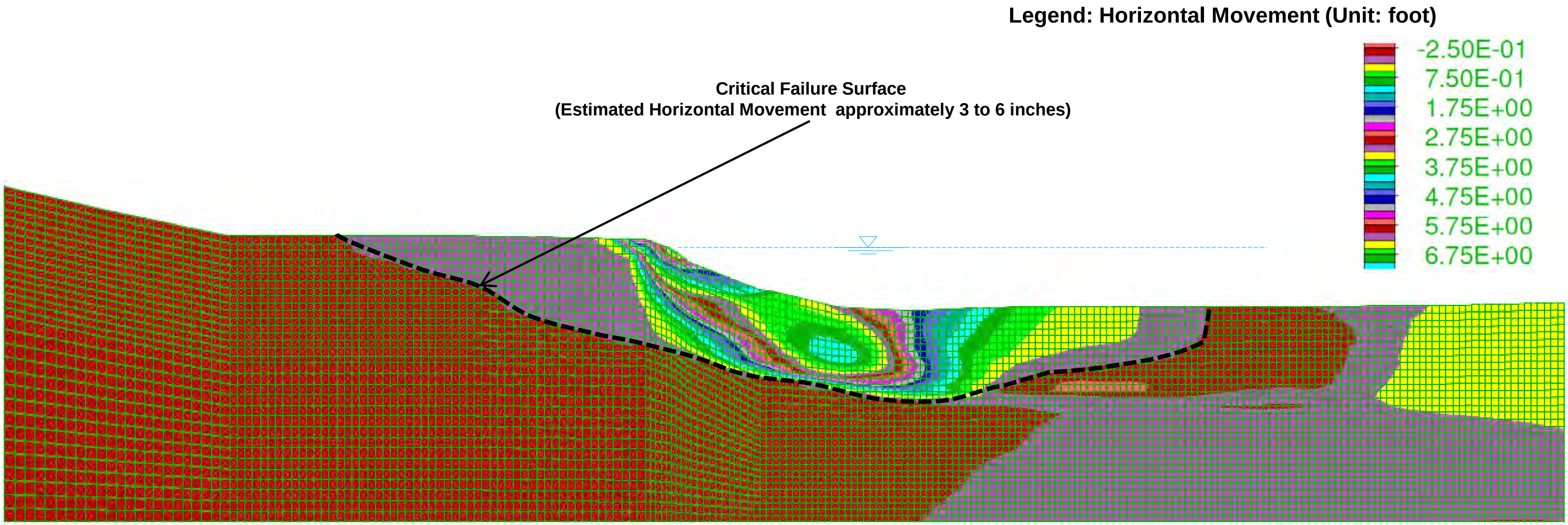
GEOENGINEERS

Figure 19



Note: Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

City Approach Vertical Ground Deformation Contour (Existing Ground Conditions, San Fernando Orion, OLE Event)	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 20

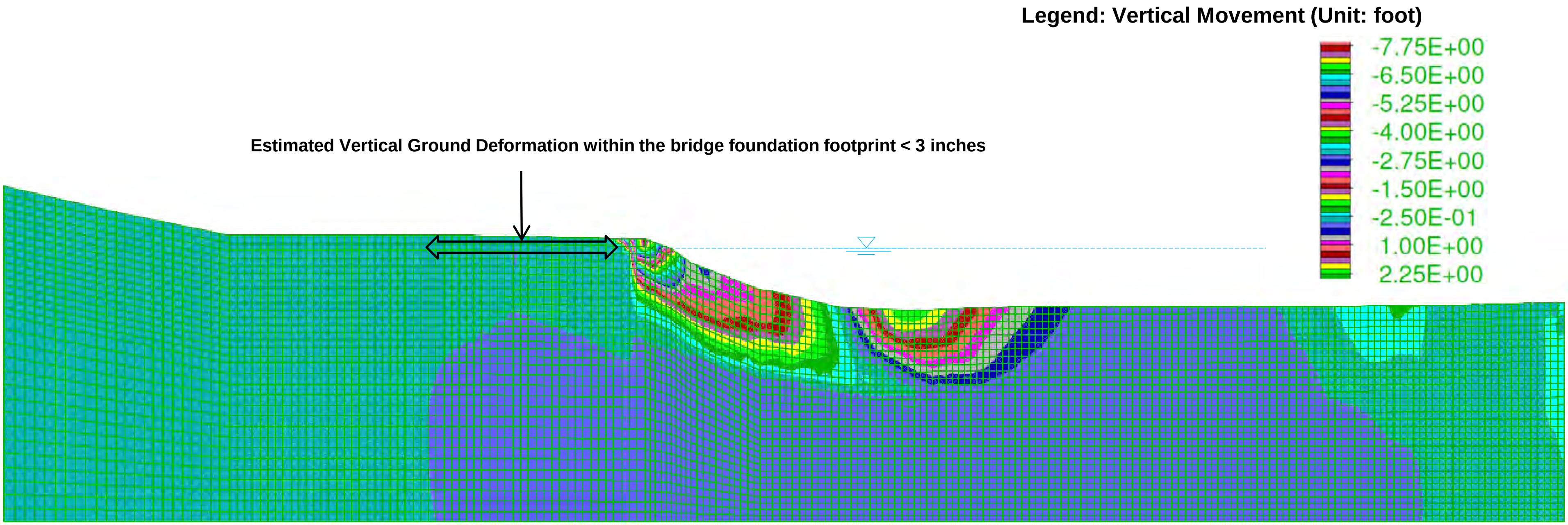


City Approach
Horizontal Ground Deformation Contour
(Improved Ground Conditions, Michoacan La Union, AASHTO Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 21



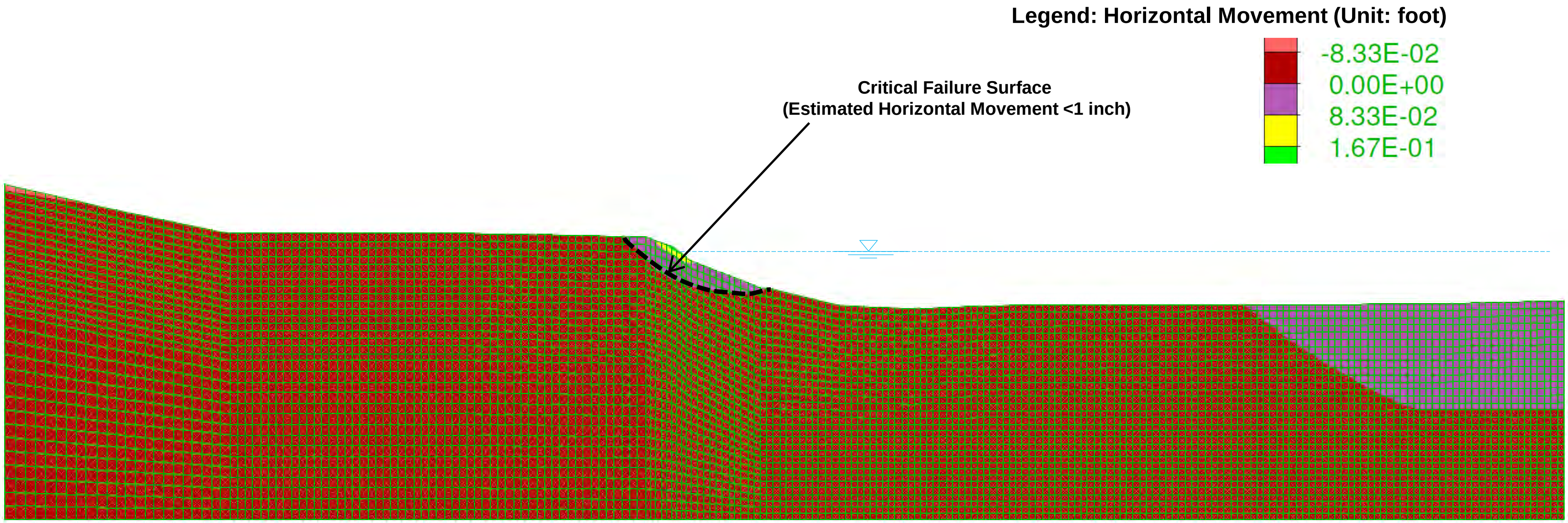
Note: Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

City Approach
Vertical Ground Deformation Contour
(Improved Ground Conditions, Michoacan La Union, AASHTO Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 22



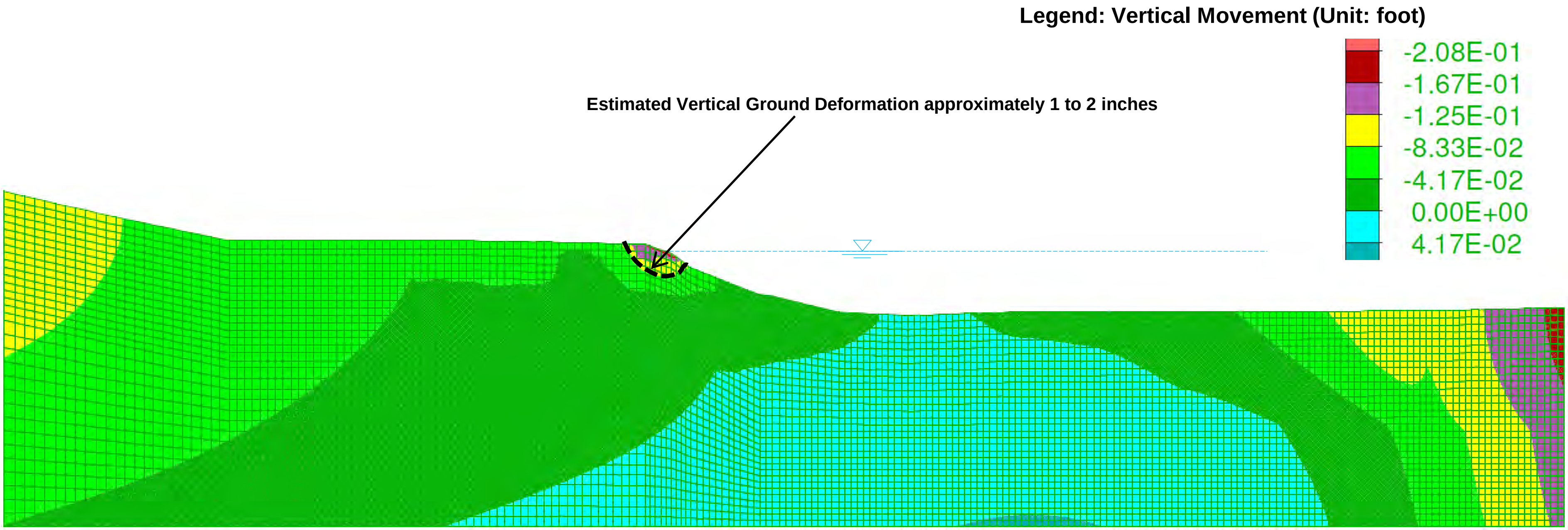
City Approach

Horizontal Ground Deformation Contour
(Improved Ground Conditions, San Fernando Orion, OLE Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure 23



Note: Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

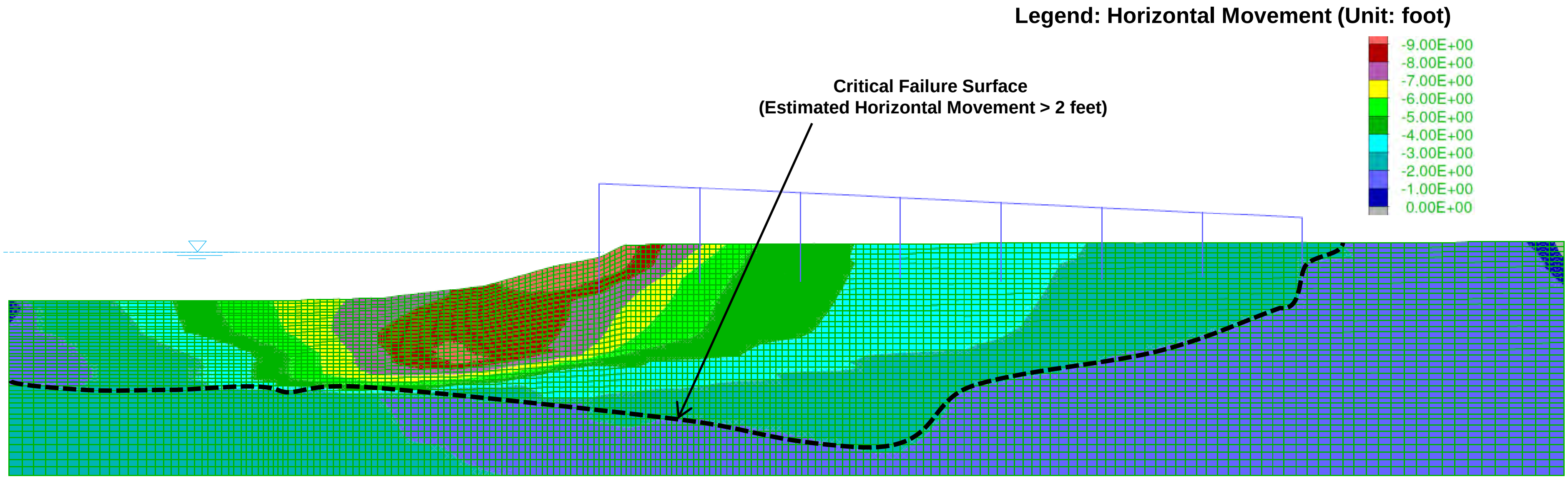
City Approach

Vertical Ground Deformation Contour (Improved Ground Conditions, San Fernando Orion, OLE Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

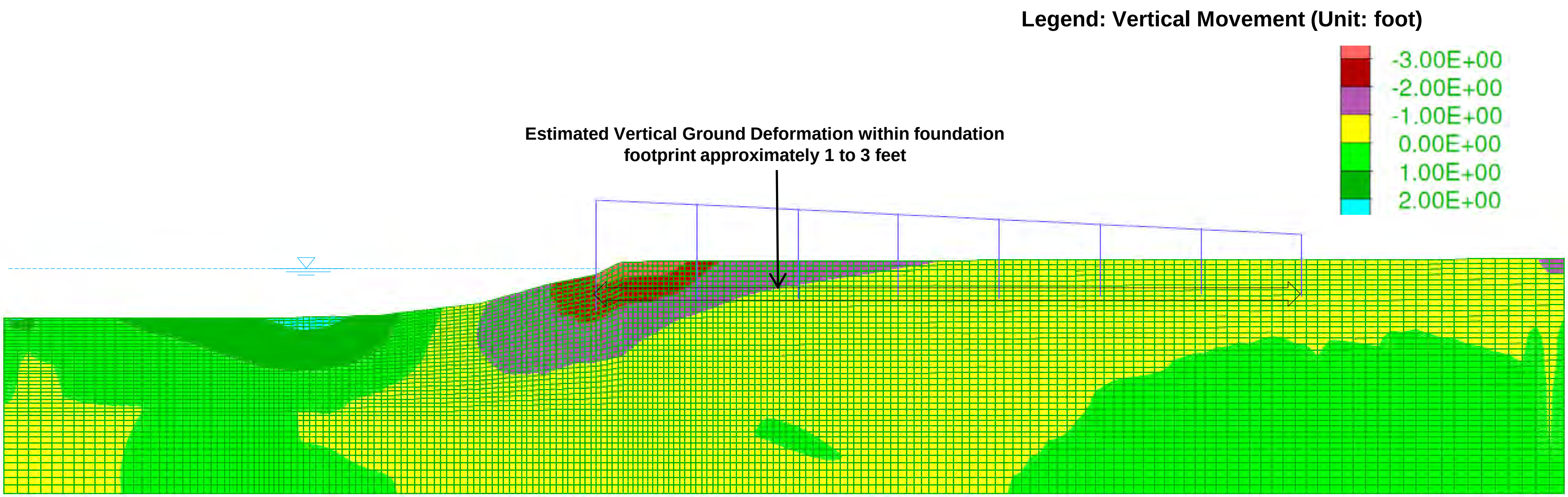
GeoEngineers

Figure 24



Note: Structural Elements were NOT included in the existing soil conditions analysis. The presence of structural elements are for illustration purpose only.

Port Approach Horizontal Ground Deformation Contour (Existing Ground Condition, San Fernando Orion, AASHTO Event)	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 25



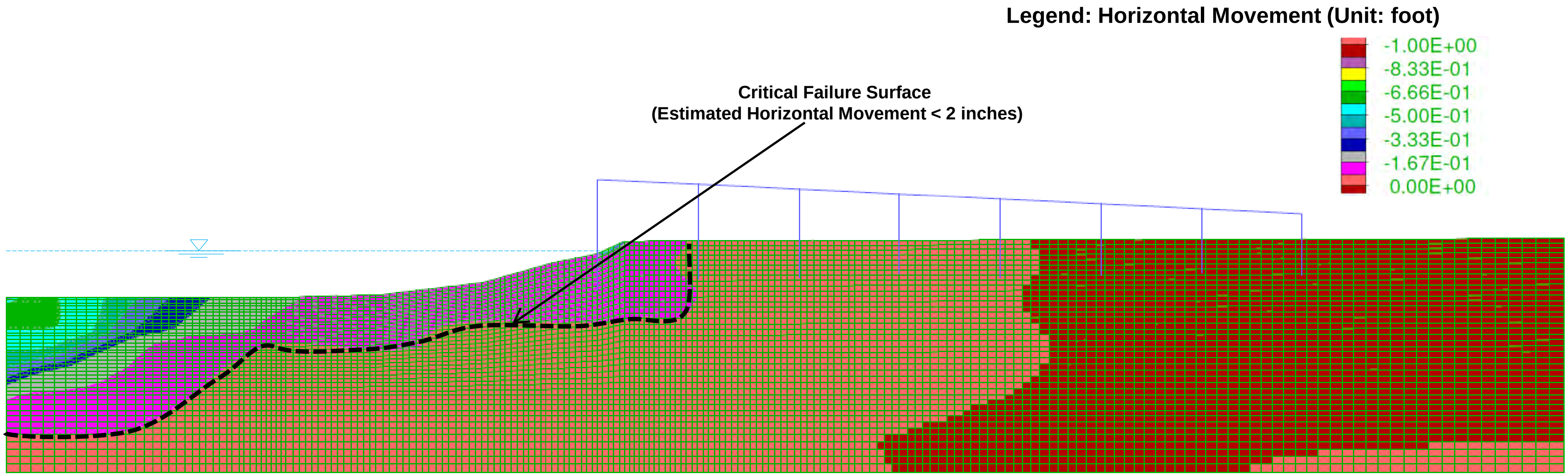
Notes:
1. Structural Elements were NOT included in the existing soil conditions analysis. The presence of structural elements are for illustration purpose only.
2. Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

Port Approach
Vertical Ground Deformation Contour
(Existing Ground Conditions, San Fernando Orion, AASHTO Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 26



Note: Structural Elements were NOT included in the existing soil conditions analysis. The presence of structural elements are for illustration purpose only.

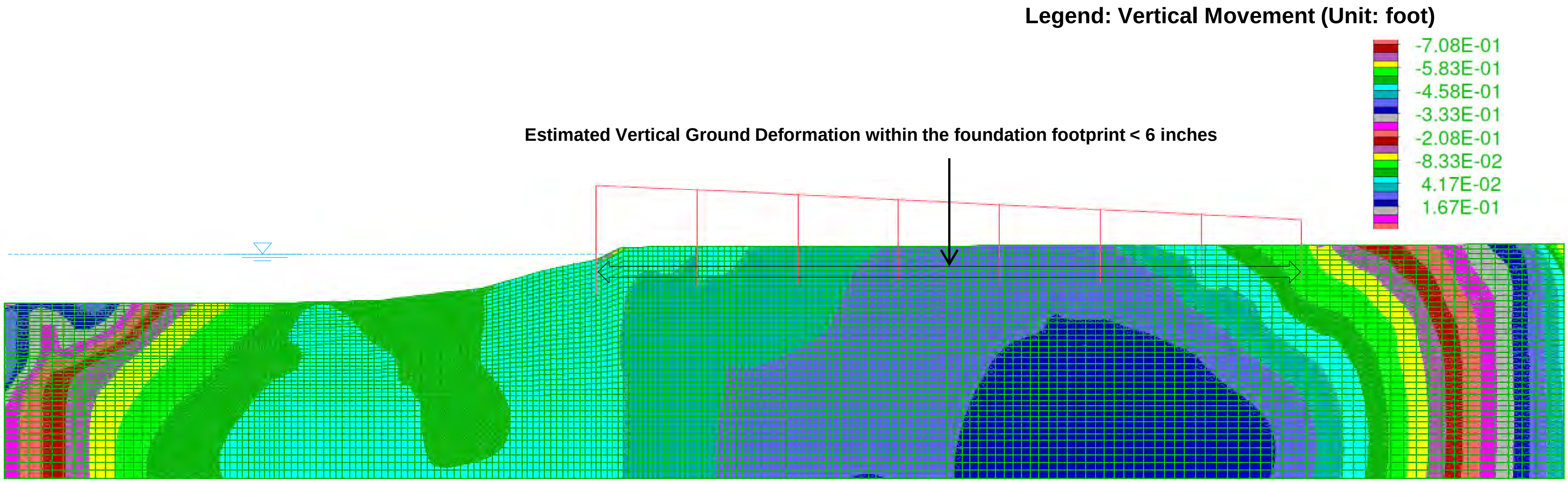
Port Approach

Ground Horizontal Deformation Contour
(Existing Ground Conditions, San Fernando Orion, OLE Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

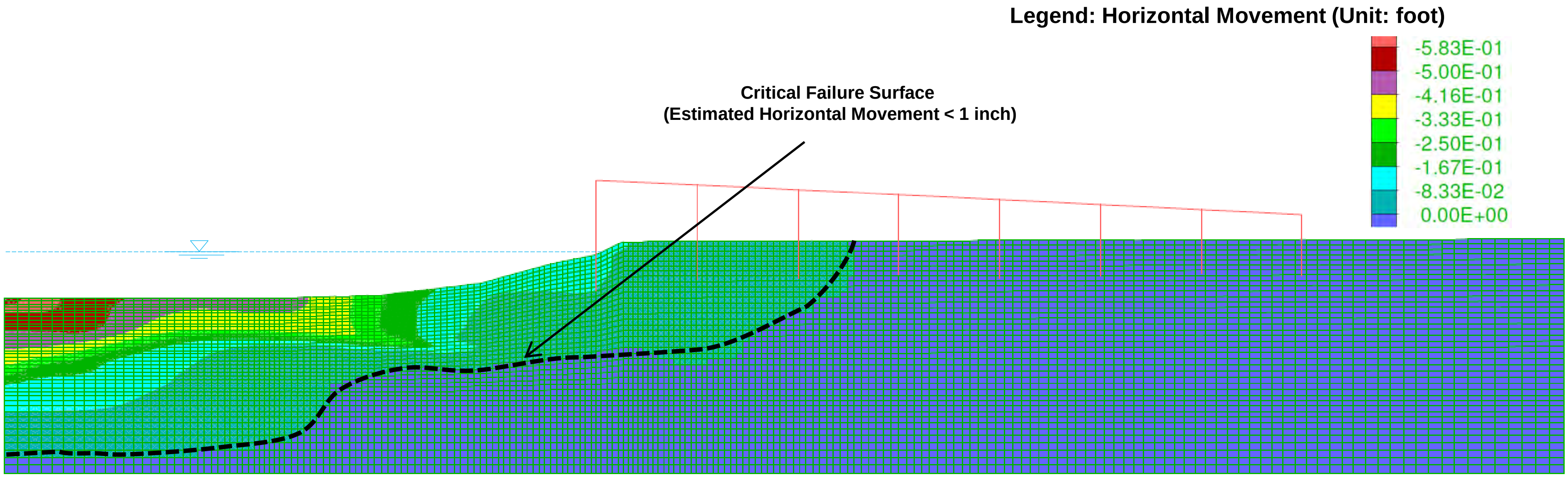
GeoEngineers

Figure 27



Notes:
1. Structural Elements were NOT included in the existing soil conditions analysis. The presence of structural elements are for illustration purpose only.
2. Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

Port Approach Ground Vertical Deformation Contour (Existing Ground Conditions, San Fernando Orion, OLE Event)	
Murray Morgan Bridge Rehabilitation Project **Tacoma, Washington**	
GeoEngineers	Figure 28

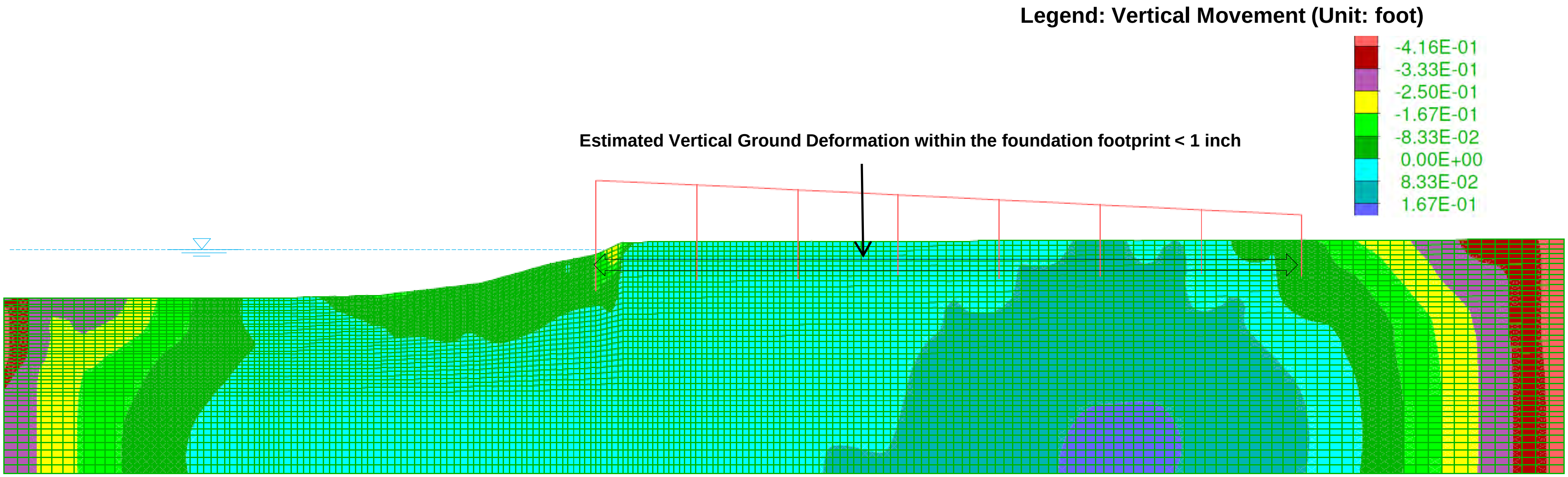


Port Approach
Horizontal Ground Deformation Contour
(Improved Ground Conditions, Michoacan La Union, AASHTO Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 29



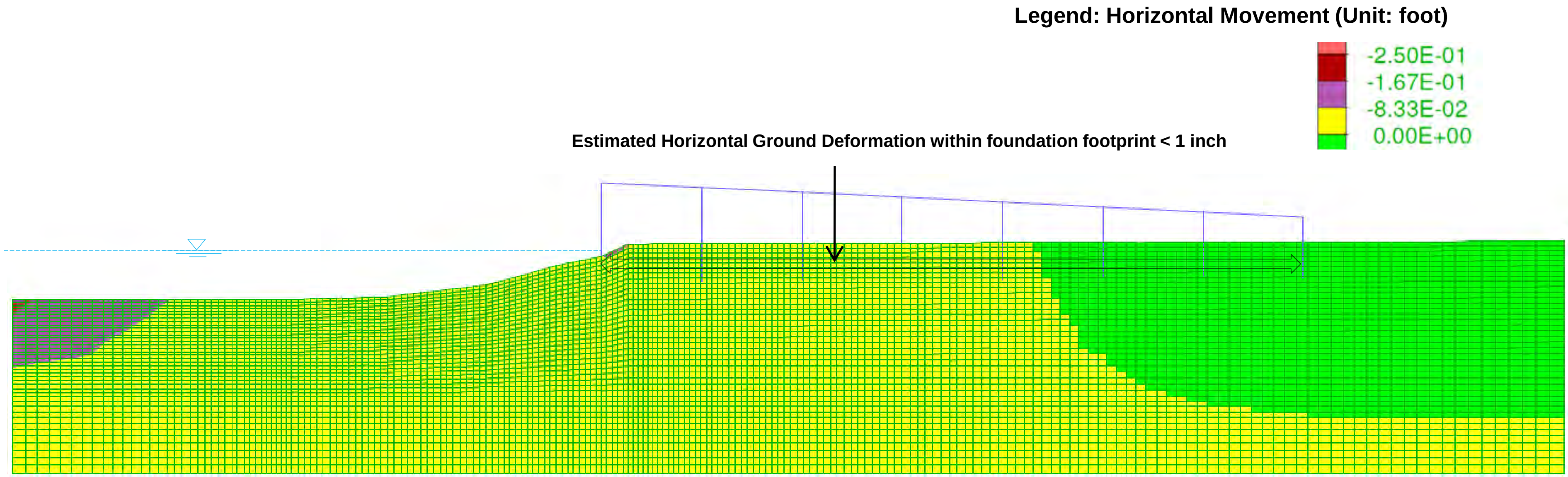
Note: Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

Port Approach
Vertical Ground Deformation Contour
(Improved Ground Conditions, Michoacan La Union, AASHTO Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 30



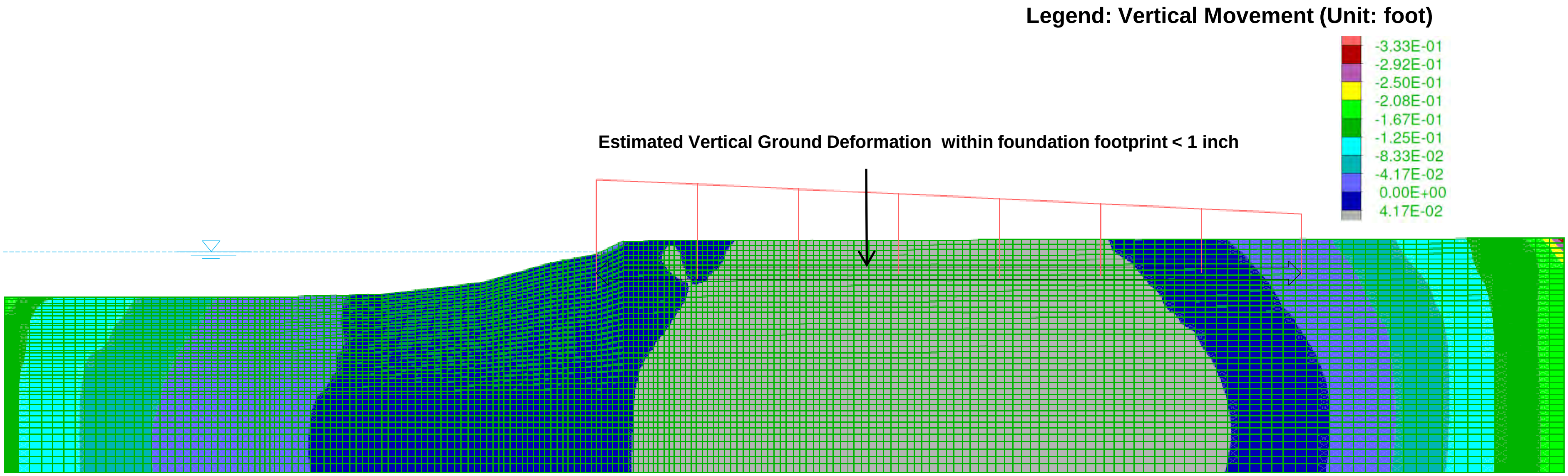
Port Approach

Horizontal Ground Deformation Contour (Improved Ground Conditions, San Fernando Orion, OLE Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 31



Note: Vertical ground deformation calculated by FLAC does not include the ground settlement induced by soil liquefaction.

Port Approach

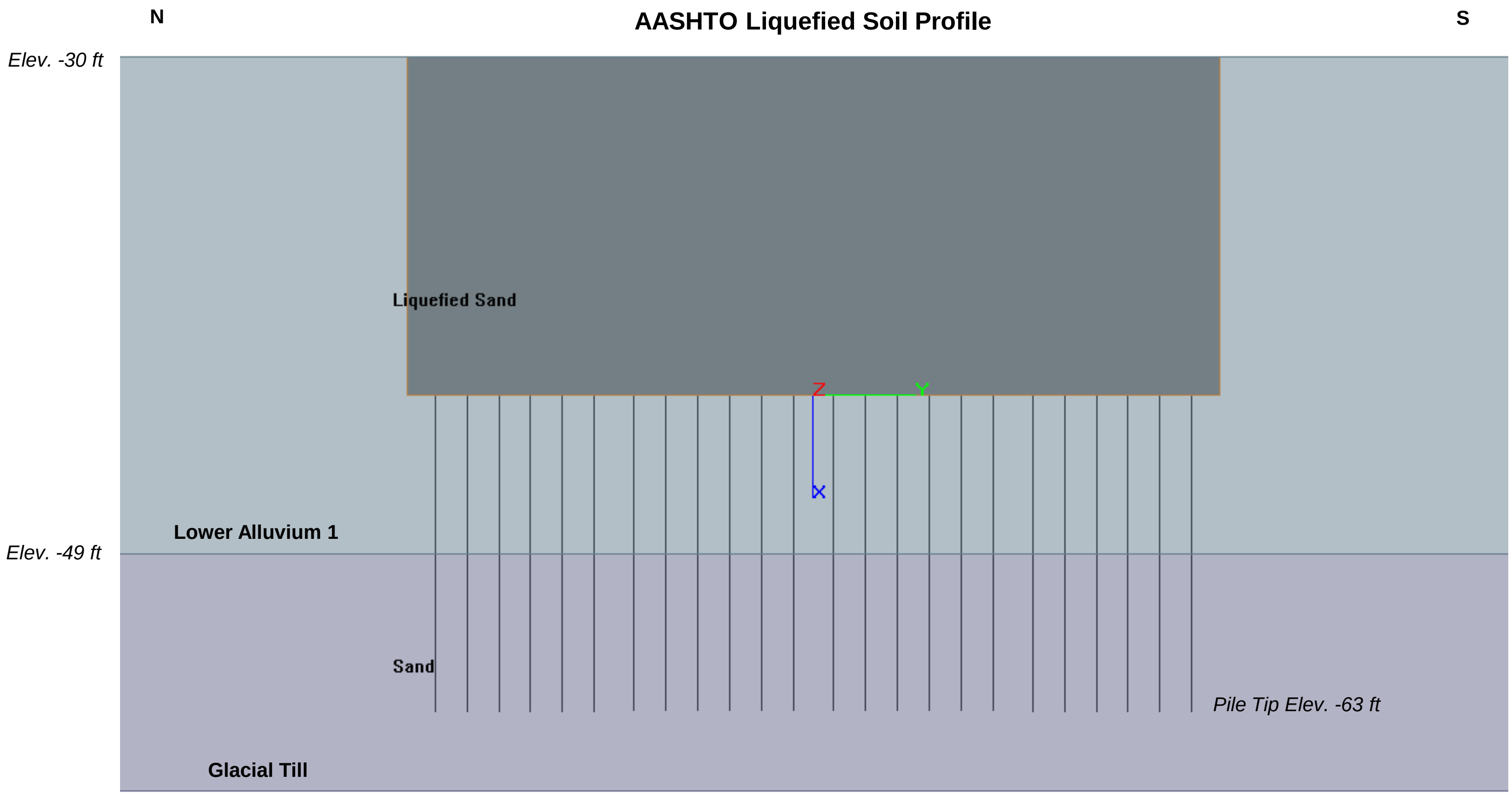
Vertical Ground Deformation Contour (Improved Ground Conditions, San Fernando Orion, OLE Event)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 32

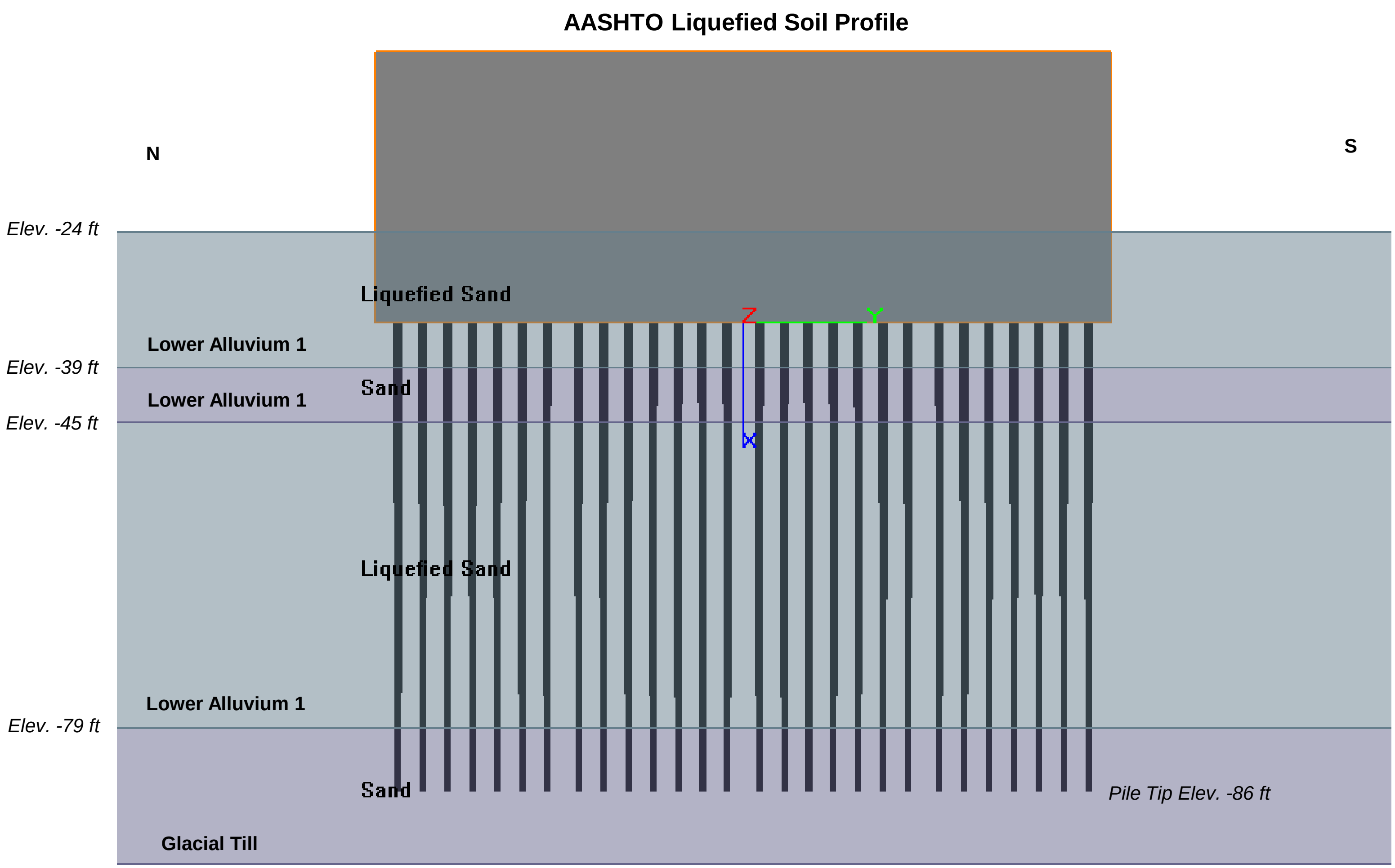
Pier 1



Reference: GROUP 7, Ensoft Inc.

AASHTO Liquefied Soil Profile, Pier 1	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 33

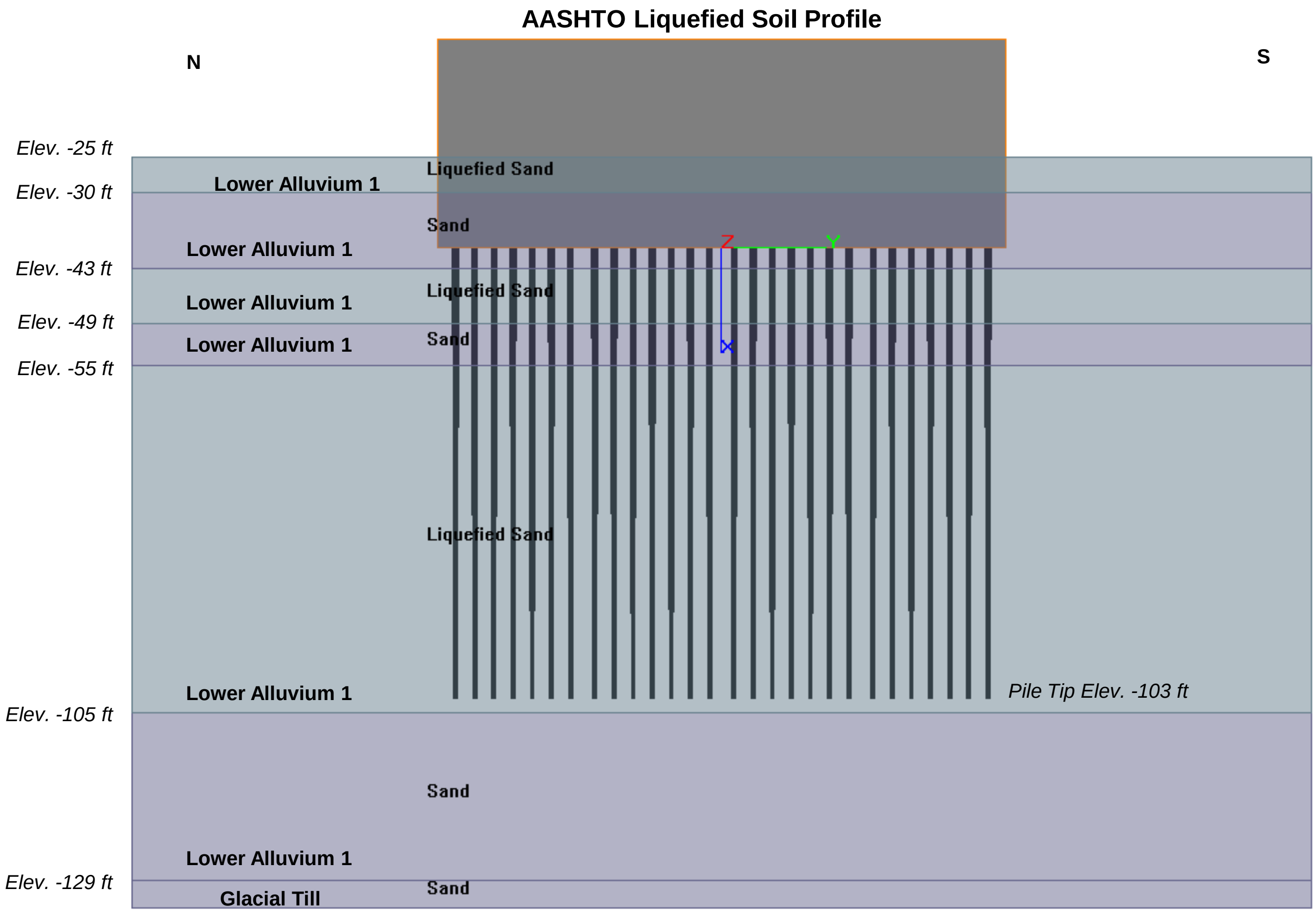
Pier 2



AASHTO Liquefied Soil Profile, Pier 2	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GEOENGINEERS	Figure 34

Reference: GROUP 7, Ensoft Inc.

Pier 3

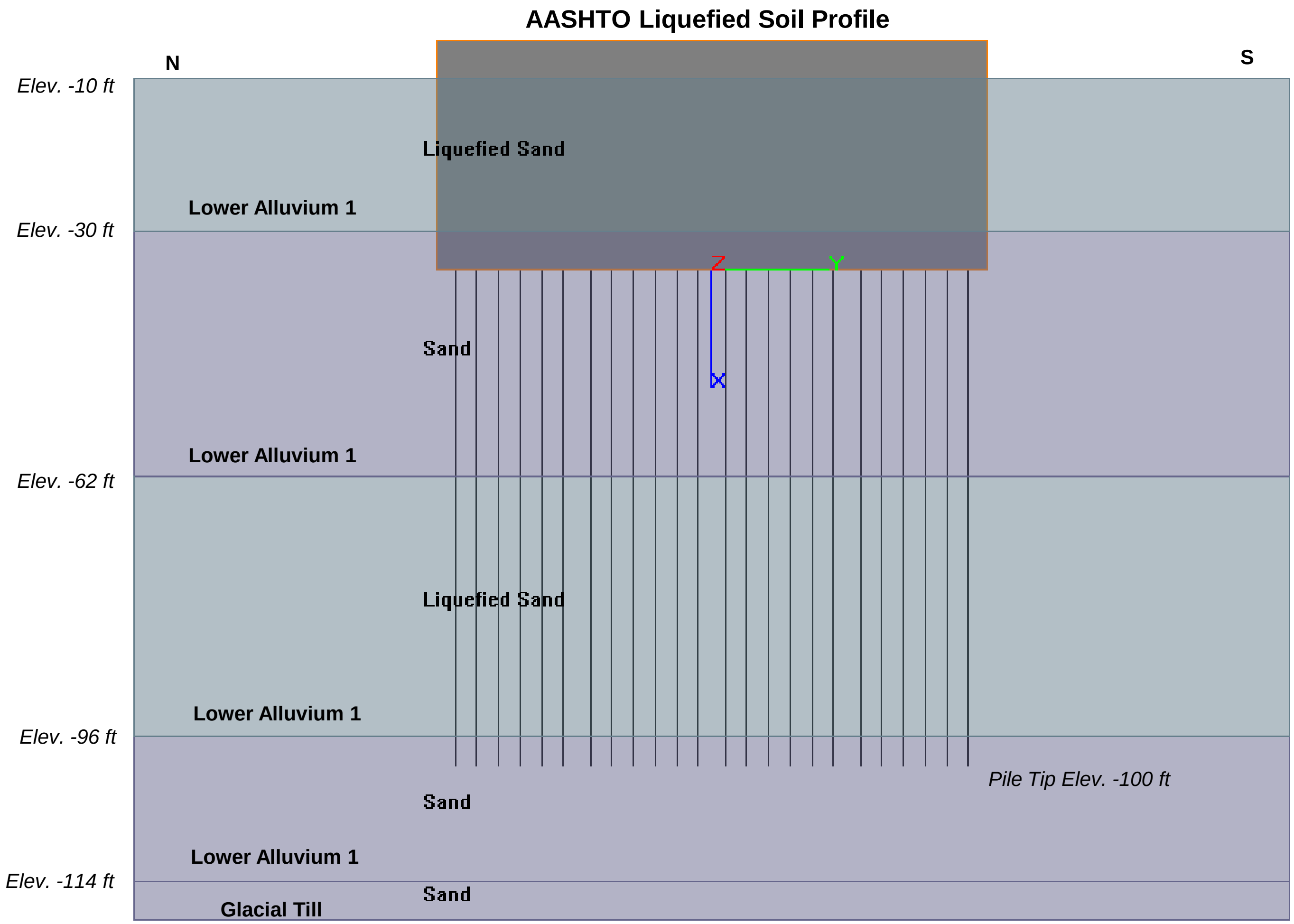


Reference: GROUP 7, Ensoft Inc.

AASHTO Liquefied Soil Profile, Pier 3	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 35

057010002_Figures 33_40.pptx WLT:khc 102611

Pier 4



AASHTO Liquefied Soil Profile, Pier 4

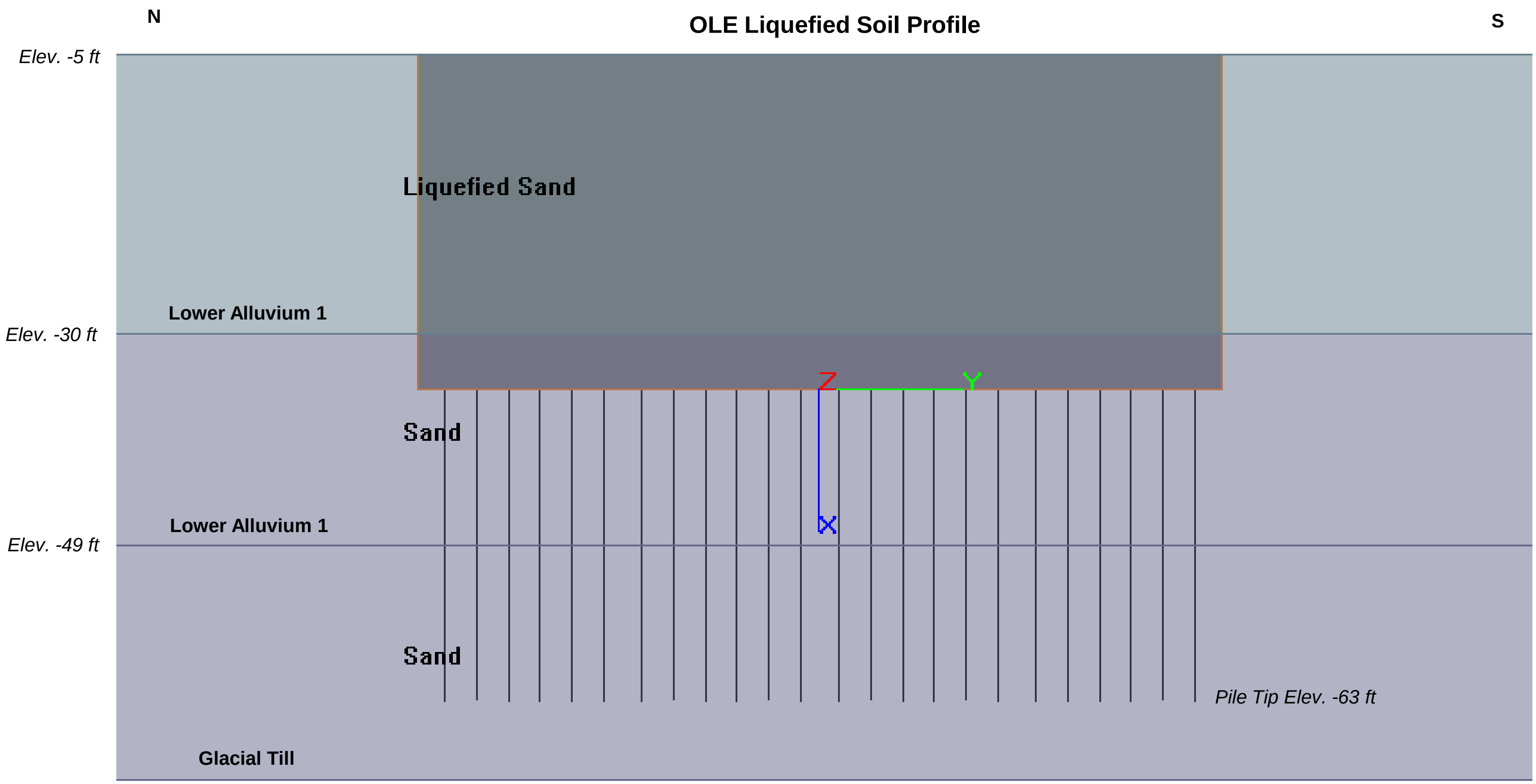
Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure 36

Reference: GROUP 7, Ensoft Inc.

Pier 1

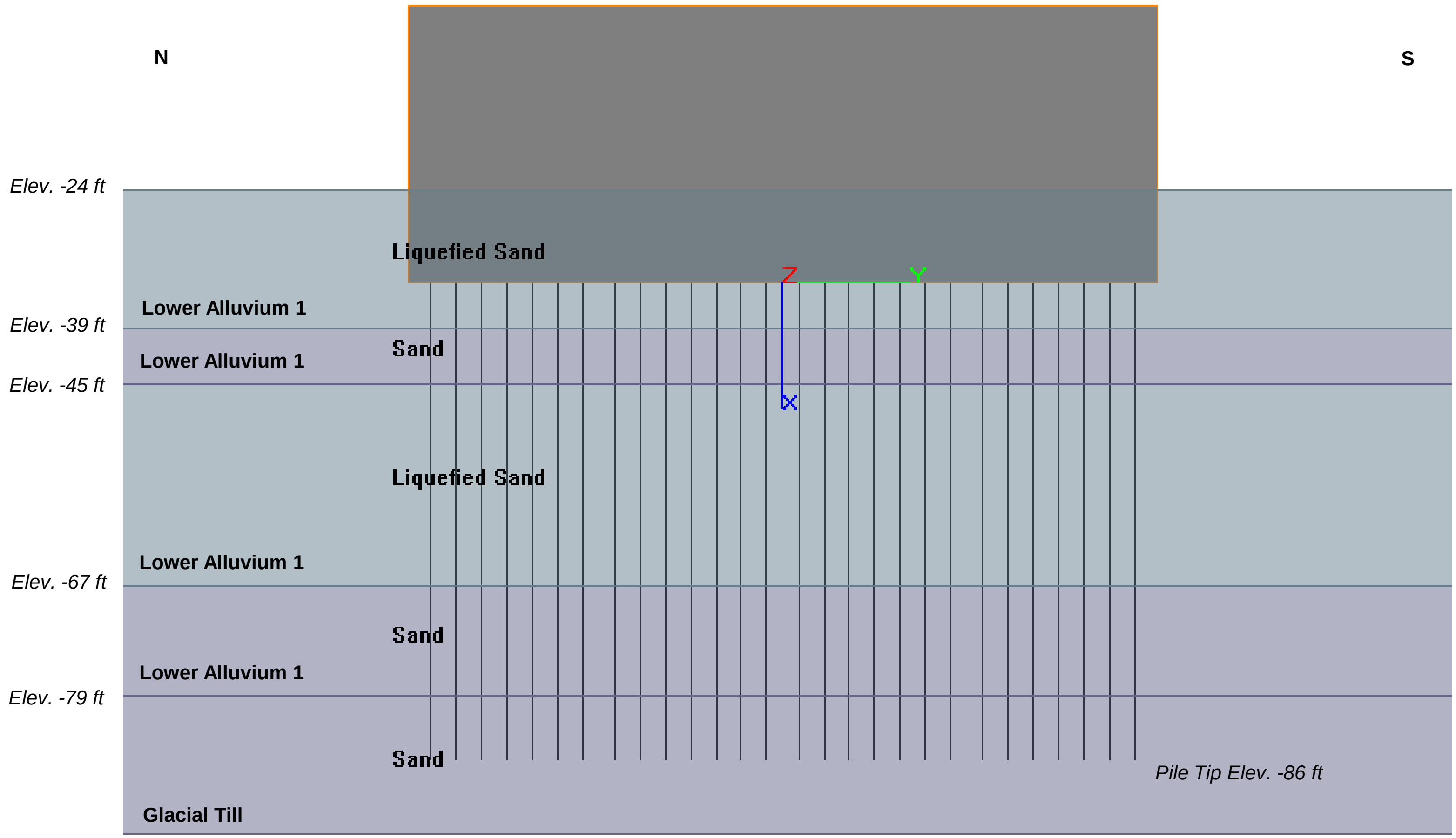


Reference: GROUP 7, Ensoft Inc.

OLE Liquefied Soil Profile, Pier 1	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 37

Pier 2

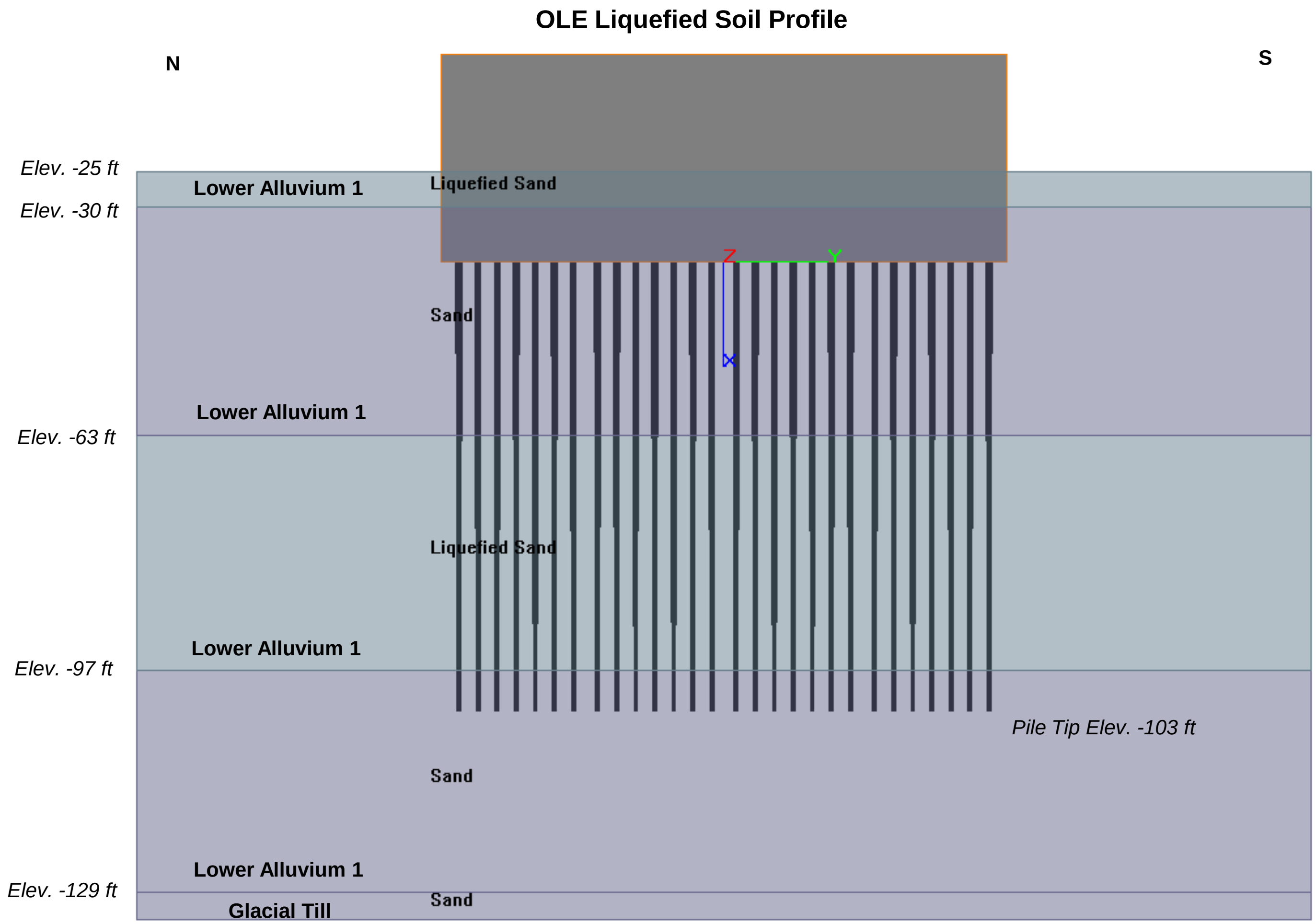
OLE Liquefied Soil Profile



Reference: GROUP 7, Ensoft Inc.

OLE Liquefied Soil Profile, Pier 2	
Murray Morgan Bridge Rehabilitation Project **Tacoma, Washington**	
GeoEngineers	Figure 38

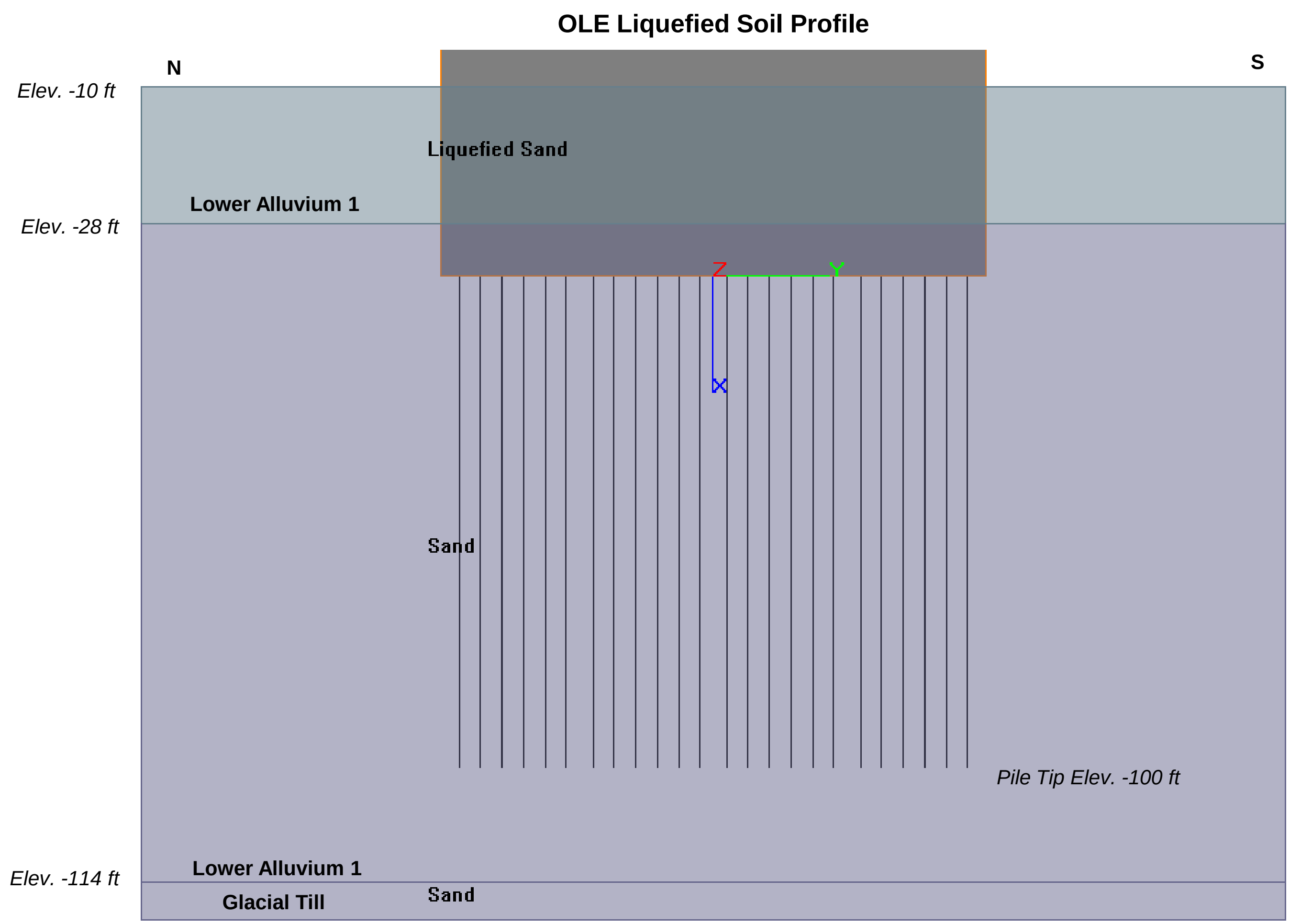
Pier 3



Reference: GROUP 7, Ensoft Inc.

OLE Liquefied Soil Profile, Pier 3	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GEOENGINEERS	Figure 39

Pier 4

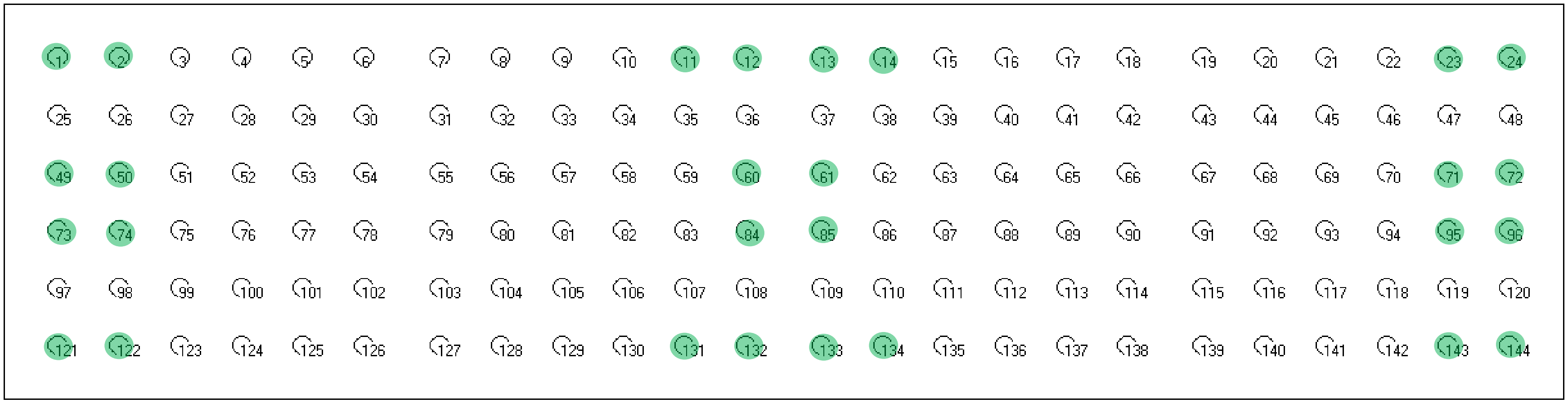


Reference: GROUP 7, Ensoft Inc.

OLE Liquefied Soil Profile, Pier 4	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 40

Pile Group Layout

Pier 1 and Pier 4



Legend

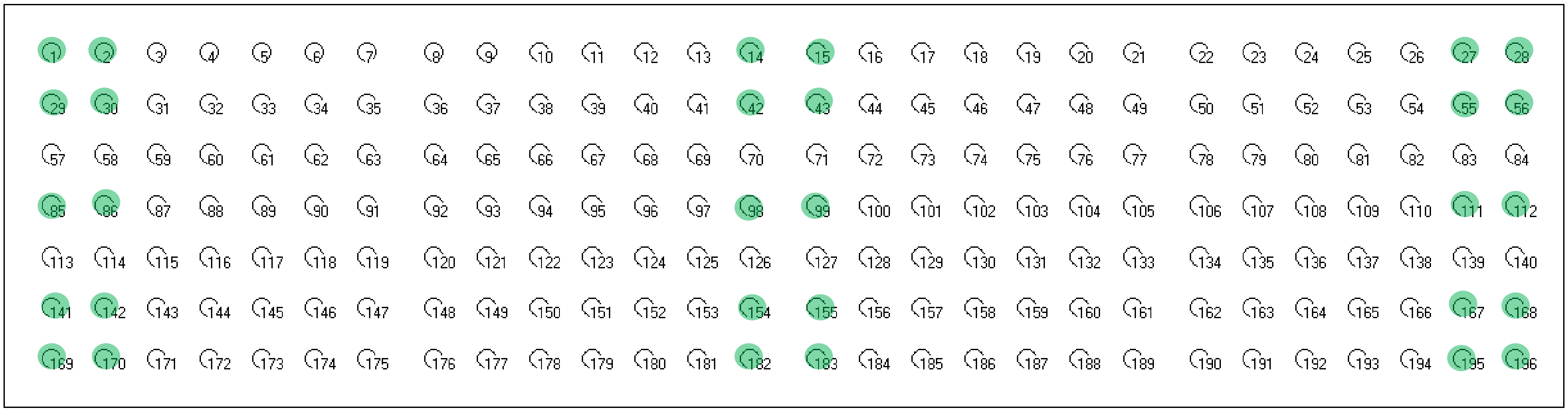
Pile displacement, shear, and moment data presented in Figures C-1 through C-86

Reference: GROUP 7, Ensoft Inc.

Pile Group Layout, Piers 1 and 4	
Murray Morgan Bridge Rehabilitation Project Tacoma, Washington	
GeoEngineers	Figure 41

Pile Group Layout

Pier 2 and Pier 3



Legend

Pile displacement, shear, and moment data presented in Figures C-1 through C-86

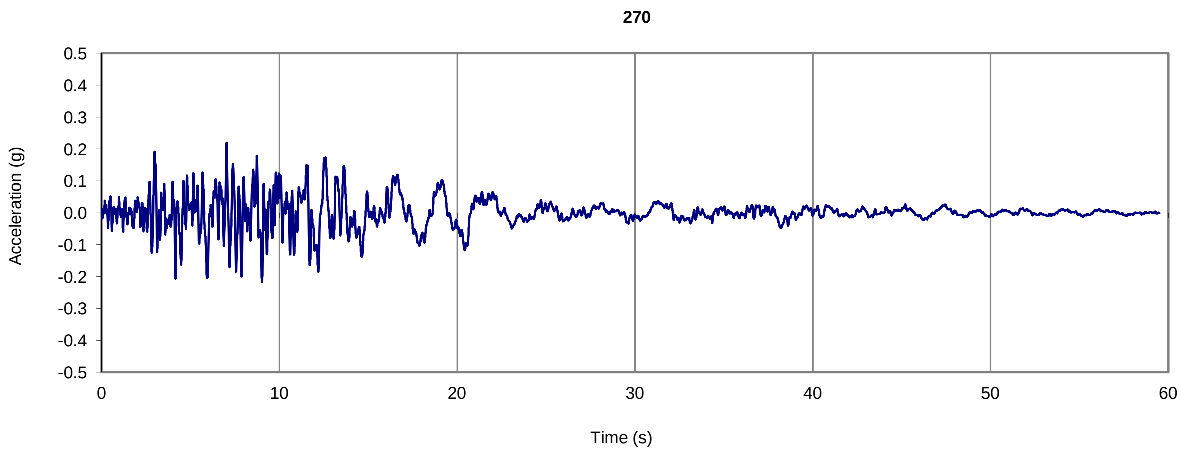
Reference: GROUP 7, Ensoft Inc.

Pile Group Layout, Piers 2 and 3	
Murray Morgan Bridge Rehabilitation Project **Tacoma, Washington**	
GeoEngineers	Figure 42

<u>Acceleration Time Histories for 1000-YR Event</u>

Linear Single Degree of Freedom System

<u>Ground Acceleration Time Histories</u>



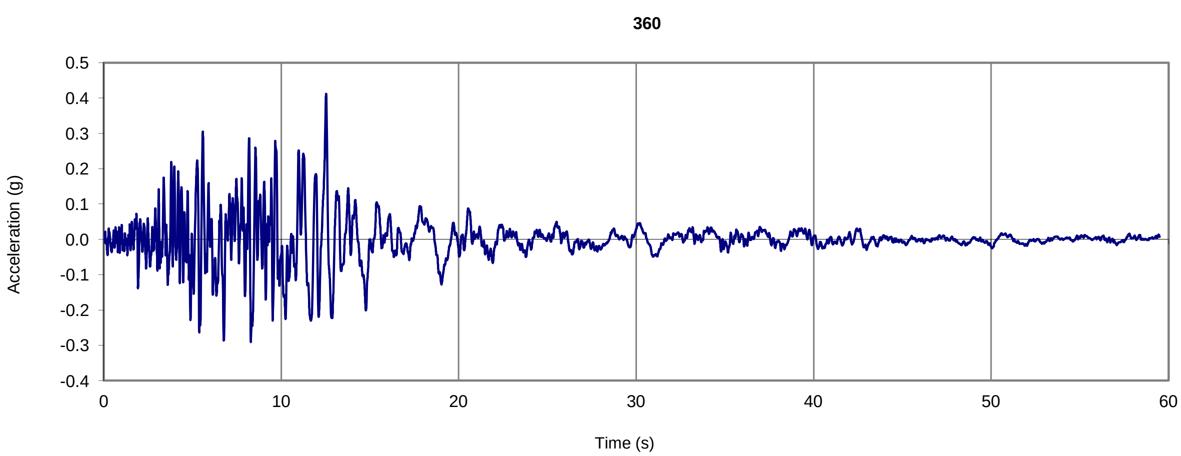


Figure 43.xlsx WLT:khc 11/4/11

057010002

Acceleration Time Histories for 1000-YR Event
San Fernando 1971 - Orion

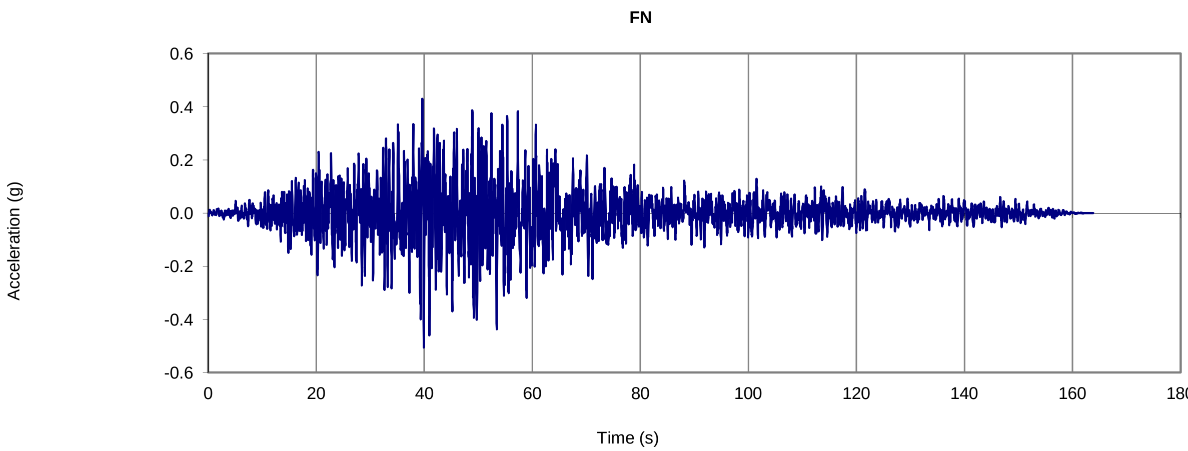
Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

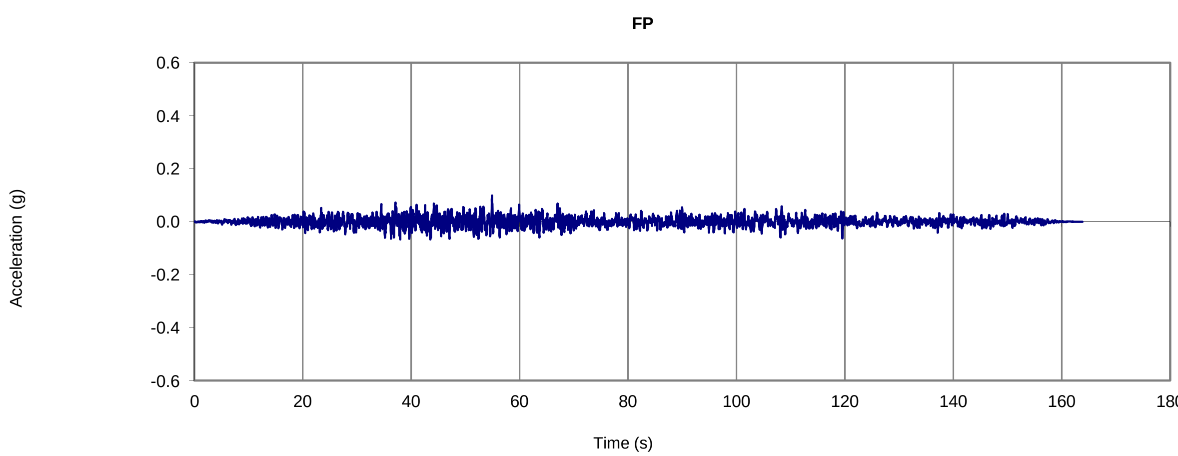
GEOENGINEERS

Figure 43

Acceleration Time Histories and Acceleration Response Time-Histories for 1000-YR Event
Linear Single Degree of Freedom System

Ground Acceleration Time Histories





Acceleration and Response Time Histories for 1000-YR Event Synthetic Cascadia

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

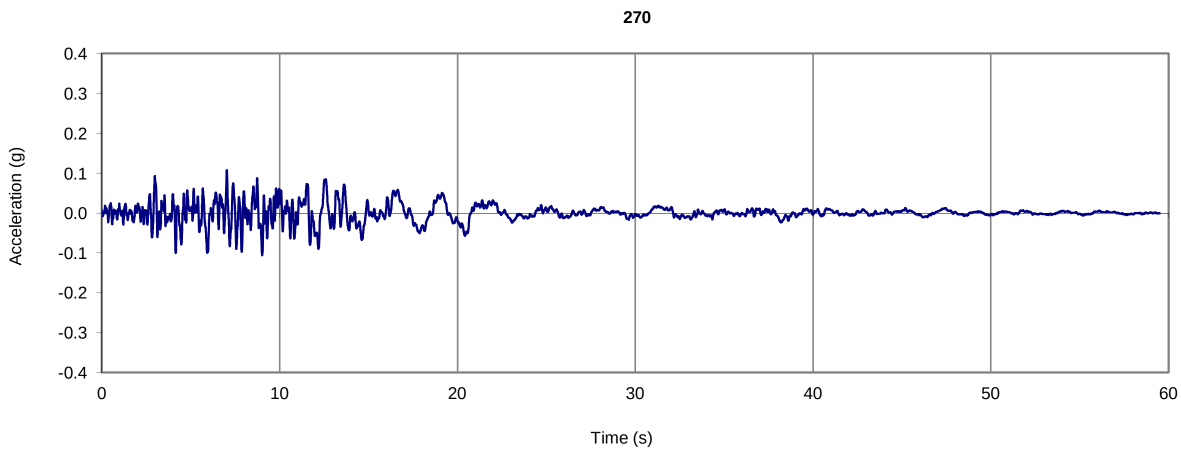
Figure 44

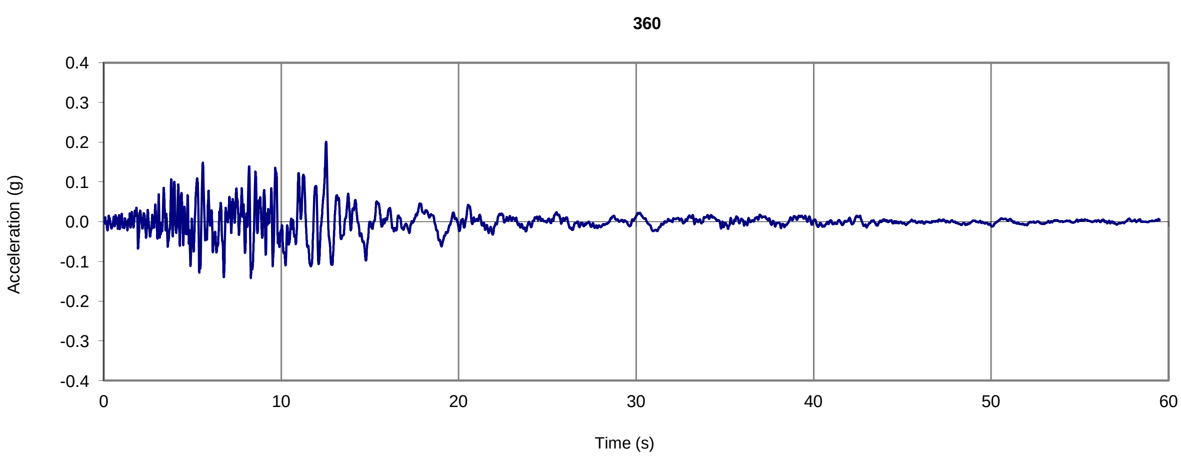
057010002 Figure 44.xlsx WLT:khc 11\7\11

<u>Acceleration Time Histories for 108-YR Event</u>

Linear Single Degree of Freedom System

Ground Acceleration Time Histories





057010002 Figure 45.xlsx WLT:khc 11/7/11

Acceleration Time Histories for 108-YR Event
San Fernando 1971 - Orion

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

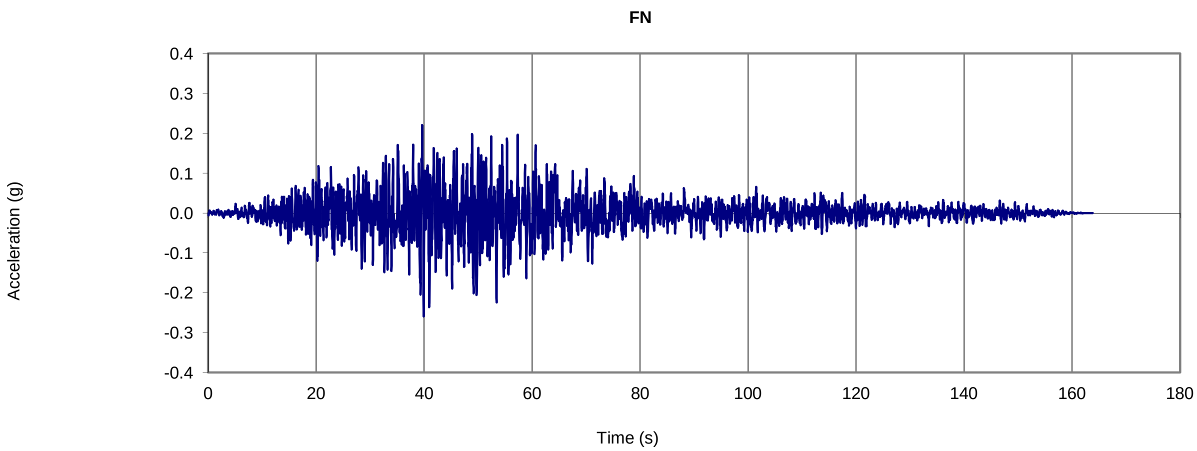
GeoEngineers

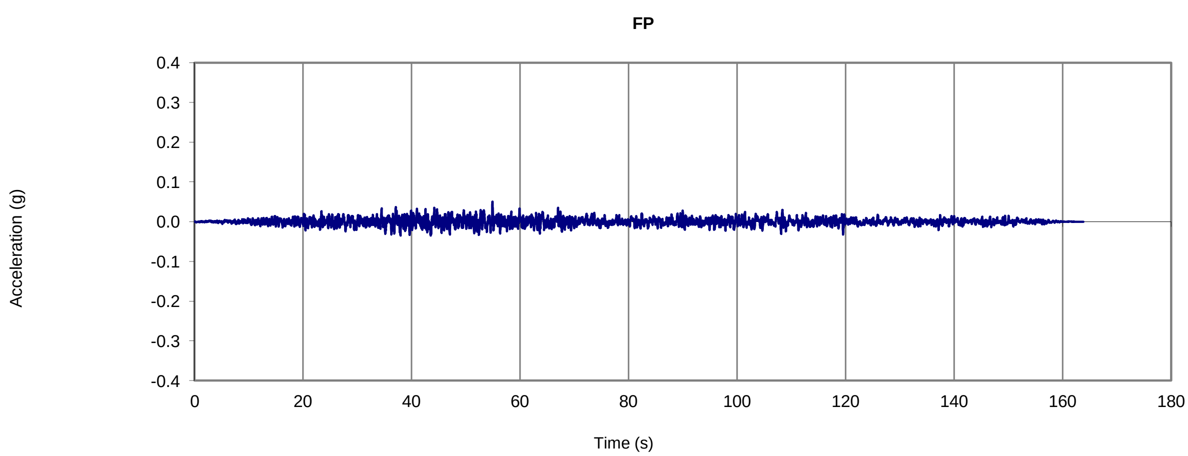
Figure 45

Acceleration Time Histories and Acceleration Response Time-Histories for 108-YR Event

Linear Single Degree of Freedom System

Ground Acceleration Time Histories





057010002 Figure 46.xlsx WLT:khc 11/7/11

Acceleration and Response Time Histories for 108-YR Event Synthetic Cascadia

Murray Morgan Bridge Rehabilitation Project Tacoma, Washington

GEOENGINEERS

Figure 46

APPENDIX A
Structural Loading Information Provided

Computations For Murray Morgan Seismic | Made By DMM | Date 06-28-2011 | Job No. 2658
Checked By ### | Date ## | Sec. No.
Back Checked By | Date | Sheet No. 1

MMB_01

Center Truss Spans - Foundation Forces — **Load Cases**

Forces (Values in Kips, ft) — **DL & 1000 Year RSA**

Results at bottom of pile cap — Fixed Foundations

Dead Load

Link	OutputCase	P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1	DEAD	-6,854	-5	0	-3	-615	0	-6.9E-02	-5.0E-05	-6.1E-08	3.7E-08	-2.7E-07	-6.2E-05
Pier 2	DEAD	-10,421	57	0	2	942	1	-1.0E-01	5.7E-04	5.2E-08	7.3E-08	2.2E-07	9.4E-05
Pier 3	DEAD	-10,362	-59	0	4	-855	-1	-1.0E-01	-5.9E-04	7.3E-08	-8.0E-08	4.2E-07	-8.5E-05
Pier 4	DEAD	-6,621	7	0	-2	693	0	-6.6E-02	6.7E-05	-6.4E-08	-4.3E-08	-2.1E-07	6.9E-05

RSA - Longitudinal

Link	OutputCase	P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1	RSA_Long	476	2,207	3	8	90,756	1	4.8E-03	2.2E-02	3.2E-05	7.5E-08	7.9E-07	9.1E-03
Pier 2	RSA_Long	660	2,810	4	9	120,836	7	6.6E-03	2.8E-02	3.8E-05	7.2E-07	9.3E-07	1.2E-02
Pier 3	RSA_Long	638	2,935	3	9	118,926	6	6.4E-03	2.9E-02	3.4E-05	6.1E-07	8.7E-07	1.2E-02
Pier 4	RSA_Long	326	2,214	4	11	90,152	1	3.3E-03	2.2E-02	4.0E-05	5.8E-08	1.1E-06	9.0E-03

RSA - Transverse

Link	OutputCase	P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1	RSA_Trans	18	2	2,207	81,020	13	3,397	1.8E-04	1.6E-05	2.2E-02	3.4E-04	8.1E-03	1.3E-06
Pier 2	RSA_Trans	14	7	2,507	108,432	39	19,857	1.4E-04	7.0E-05	2.5E-02	2.0E-03	1.1E-02	3.9E-06
Pier 3	RSA_Trans	12	6	2,559	99,660	43	18,390	1.2E-04	6.3E-05	2.6E-02	1.8E-03	1.0E-02	4.3E-06
Pier 4	RSA_Trans	13	3	1,976	63,849	21	3,106	1.3E-04	2.9E-05	2.0E-02	3.1E-04	6.4E-03	2.1E-06

Coordinate System

1 - Vertical

2 - Longitudinal

3 - Transverse

Computations For Murray Morgan Made By DMM Date 06-28-2011 Job No. 2658
Seismic Checked By ### Date ## Sec. No.
Back Checked By Date Sheet No. 1

MMB_01

Center Truss Spans - Foundation Forces

Forces (Values in Kips, ft)
Results at bottom of pile cap

Load Case Summary

DL & 1000 Year RSA
Fixed Foundations

Dead Load

Link		P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1		6,854	5	0	3	615	0	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
Pier 2		10,421	57	0	2	942	1	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
Pier 3		10,362	59	0	4	855	1	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00
Pier 4		6,621	7	0	2	693	0	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00	0.0E+00

RSA Case 1

Longitudinal 100%
Transverse 40%

Link		P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1		484	2,207	886	32,416	90,762	1,360	4.8E-03	2.2E-02	8.9E-03	1.4E-04	3.2E-03	9.1E-03
Pier 2		666	2,813	1,006	43,382	120,851	7,950	6.7E-03	2.8E-02	1.0E-02	7.9E-04	4.3E-03	1.2E-02
Pier 3		643	2,938	1,027	39,873	118,943	7,362	6.4E-03	2.9E-02	1.0E-02	7.4E-04	4.0E-03	1.2E-02
Pier 4		331	2,215	795	25,550	90,160	1,243	3.3E-03	2.2E-02	7.9E-03	1.2E-04	2.6E-03	9.0E-03

RSA Case 2

Longitudinal 40%
Transverse 100%

Link		P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1		209	884	2,209	81,023	36,316	3,398	2.1E-03	8.8E-03	2.2E-02	3.4E-04	8.1E-03	3.6E-03
Pier 2		278	1,131	2,508	108,436	48,373	19,860	2.8E-03	1.1E-02	2.5E-02	2.0E-03	1.1E-02	4.8E-03
Pier 3		267	1,180	2,560	99,663	47,613	18,392	2.7E-03	1.2E-02	2.6E-02	1.8E-03	1.0E-02	4.8E-03
Pier 4		143	888	1,978	63,853	36,082	3,107	1.4E-03	8.9E-03	2.0E-02	3.1E-04	6.4E-03	3.6E-03

Max RSA + DEAD LOAD

Link		P	V2	V3	M2	M3	T	U1	U2	U3	R1	R2	R3
Pier 1		7,337	2,207	2,209	81,023	90,762	3,398	4.8E-03	2.2E-02	2.2E-02	3.4E-04	8.1E-03	9.1E-03
Pier 2		11,087	2,813	2,508	108,436	120,851	19,860	6.7E-03	2.8E-02	2.5E-02	2.0E-03	1.1E-02	1.2E-02
Pier 3		11,005	2,938	2,560	99,663	118,943	18,392	6.4E-03	2.9E-02	2.6E-02	1.8E-03	1.0E-02	1.2E-02
Pier 4		6,952	2,215	1,978	63,853	90,160	3,107	3.3E-03	2.2E-02	2.0E-02	3.1E-04	6.4E-03	9.0E-03

Note: DL Shear and Moment taken as Zero

Coordinate System

1 - Vertical
2 - Longitudinal
3 - Transverse

APPENDIX B
Subsurface Explorations and Laboratory Testing

APPENDIX B
SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

Field Exploration

We explored subsurface conditions at the site by advancing three borings and six cone penetrometer test (CPT) soundings. Borings were completed between March 1 and 4 and on March 7, 2011. CPT soundings were completed between March 1 and 3 and on June 2 and 3, 2011. Our representative located the explorations in the field using measurements from existing site features. Elevation information provided on the subsurface logs is based on the datum provided on the RFP drawings (NAD83 Washington State Plane, South Zone). Because the explorations were located by field measurement, the locations and elevations should be considered approximate.

A key to the symbols used on the boring logs is included as Figure B-1. The boring logs are included as Figures B-2 through B-4. The CPT logs are included as Figures B-5 through B-10.

Soil Borings

The borings were advanced by Holocene Drilling using a truck-mounted drill rig under subcontract to GeoEngineers. The soil borings were advanced to depths ranging from 41½ to 249 feet below ground surface (bgs). Hollow-stem auger and mud rotary drilling methods were used to advance the borings, as indicated on the boring logs.

Disturbed soil samples were obtained from the borings using a 1.375-inch inside-diameter split-spoon standard penetration test (SPT) sampler driven into the soil using a 140-pound hammer free-falling a distance of 30 inches. The number of blows (N) required to drive the sampler the last 12 inches or until refusal was met (N>50) is recorded on the logs as the blow count. In some cases, additional sample recovery attempts were made using a 2.4-inch-inside-diameter split-spoon sampler.

Our representative continuously monitored the borings, maintained a log of the subsurface conditions and observed sample attempts, generally at 2.5- to 5-foot depth intervals. The soils encountered were visually classified in general accordance with the system described in Figure B-1, ASTM International (ASTM) D 2488.

CPT Soundings

CPT soundings were advanced by Subsurface Technologies, Inc. and In-Situ Engineering, Inc. using truck- and track-mounted hydraulically operated cone penetrometer equipment under subcontract to GeoEngineers, Inc. The CPT soundings were advanced to depths ranging from about 42 to 125 feet bgs. The CPT sounding process involves pushing an instrumented probe into the ground and recording soil friction, tip resistance and dynamic pore pressure using electronic methods. Soil samples are not obtained during CPT soundings. In some CPTs, seismic shear wave velocity measurements were also made at approximate 1- or 2-meter intervals. Soil types are interpreted based on empirical relationships between measured CPT parameters described above. Because it provides a continuous interpretation of subsurface data, the CPT method generally provides more detail regarding soil layering than conventional drilling and sampling methods.

Laboratory Testing

General

Soil samples obtained from the borings were returned to our laboratory for further examination and review. Representative soil samples were selected for laboratory tests to evaluate their pertinent geotechnical engineering characteristics and to confirm or modify our field classifications. The following paragraphs provide a description of the tests performed.

Moisture Content

The moisture content of selected samples was determined in general accordance with ASTM Test Method D 2216. The test results are used to aid in soil classification and correlation with other pertinent engineering soil properties. The results of these tests are presented on the exploration logs at the respective sample depths.

Percent Passing U.S. No. 200 Sieve (%F)

Selected samples were “washed” through the U.S. No. 200 mesh sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. The tests were conducted in general accordance with ASTM D 1140, and the results are shown on the boring logs at the respective sample depths.

Atterberg Limits

Selected samples were tested to determine the Liquid Limit and Plastic Limit. The tests were performed in general accordance with ASTM D 4318 (wet preparation method). The test results were used to classify the soils and to aid in evaluating index properties and consolidation characteristics of the fine-grained soil deposits. The results of the tests are shown in Figures B-11 and B-12.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	CC	Cement Concrete
	AC	Asphalt Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Measured groundwater level in exploration, well, or piezometer

Groundwater observed at time of exploration

Perched water observed at time of exploration

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Sonic Core

Bulk or grab

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE B-1

Drilled	Start	End	Total Depth (ft)	41.5	Logged By	WLT	Driller	Holocene	Drilling Method	Mud Rotary
	3/7/2011	3/7/2011			Checked By	CAM				

Surface Elevation (ft) Vertical Datum	19.2	Hammer Data	Autohammer 140 lb / 30" drop	Drilling Equipment	BK 81 Drill Rig
Easting (X) Northing (Y)		System Datum		Groundwater	

Groundwater Date Measured	Depth to Water (ft)	Elevation (ft)
3/7/2011	10.0	9.2

Notes: SPT

Elevation (feet)	Depth (feet)	FIELD DATA: Recovered (in)	FIELD DATA: Blows/foot	FIELD DATA: Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					CC	7 inches of Portland Cement Concrete			
						SP-SM	Black fine to medium sand with silt and shell fragments (very loose, moist) (fill)			
		13	4	1						
15										
	5	12	14	2			Grades to medium dense			
		11	22	3			With gravel			
10										
	10	12	13	4 %F	∇		Grades to wet	23		%F=6
						SP-SM	Black fine to medium sand with silt and shell fragments (medium dense, moist) (alluvium)			
5										
	15	8	18	5 %F				25		%F=11
							Grades to include silt seams and occasional gravel encountered			
0										
	20	9	15	6 %F				28		%F=5
-5										
	25	7	16	7 %F				24		%F=6
						GM	Silty fine to coarse gravel with sand (medium dense, wet) (glacially consolidated deposits)			
-10										
	30									

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-1

GeoEngineers

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	30		7	29		8							
									GP-GM	Gray fine to coarse gravel with sand and silt (dense, wet)			
-15													
	35		3	38		9							
									GM	Black-gray silty fine to coarse gravel with sand (dense, wet)			
-20													
	40		6	33		10							

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-1 (continued)

GEOENGINEERS

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Figure B-2
Sheet 2 of 2

Drilled	Start 3/1/2011	End 3/2/2011	Total Depth (ft) 101.5	Logged By WLT / Checked By CAM	Driller Holocene	Drilling Method Mud Rotary

Surface Elevation (ft) / Vertical Datum	18.7	Hammer Data	Autohammer 140 lb / 30" drop	Drilling Equipment	BK 81 Drill Rig
Easting (X) / Northing (Y)		System Datum		Groundwater	
Notes: SPT/Shelby				Date Measured / Depth to Water (ft) / Elevation (ft)	9.0 / 9.7

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Water Level	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0					SM	Brown-gray silty fine to coarse sand with gravel (medium dense, moist) (fill)			
15		12	12	1						Gravel in tip of sampler
						GM	Gray silty fine gravel with sand (medium dense, wet)			
	5	2	13	2						
						SM	Gray silty fine to medium sand with gravel and brick fragments (medium dense, moist)			
		12	10	3						
10					▽		Grades to wet			
	10	2	11	4		ML	Brown silt with wood fragments and trace gravel (medium dense, wet) (tidal deposits)			
		0	9	5			Grades to medium stiff			No recovery
5						SP-SM	Black fine to medium sand with silt, occasional gravel and wood fragments (medium dense, wet)			
	15	14	13	6						
						SP-SM	Gray fine to coarse sand with silt and gravel (medium dense, wet) (alluvium)			
0		2	20	7						
	20	4	19	8 %F			Grades to black	23		%F=6
-5						SP-SM	Black fine to medium sand with silt and shell fragments (dense, moist)			
	25	12	30	9						
-10						SM	Black silty fine to medium sand, occasional shell fragments (medium dense, wet)			
	30									

Note: See Figure B-1 for explanation of symbols.

Tacoma: Date:10/25/11 Path:P:\0\0570100\GINT\0570-100-01 BORING LOGS.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring B-2

GEOENGINEERS

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	30		12	27		10 / %F					22		%F=16
-15													
	35		12	34		11				Grades to dense			
									SM	Gray silty fine to medium sand with shell fragments, includes silt seams (loose, wet)			
-20													
	40		14	8		12 / %F					29		%F=42
									SP-SM	Black fine to medium sand with silt and shell fragments (dense, wet)			
-25													
	45		18	47		13							
-30													
	50		18	36		14							
-35													
	55		18	59		15				Grades to very dense			4-inch heave
-40													
	60		18	31		16				Grades to dense	36		%F=12
									ML	Black sandy silt (very stiff, wet)			
-45													
									SP-SM	Black fine to medium sand with silt and occasional organics (very dense, wet)			
	65		14	55		17							

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-2 (continued)

GEOENGINEERS

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Figure B-3
Sheet 2 of 3

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
-50													
	70		17	49		18				Grades to dense			
-55									SP-SM/ML	Black fine to medium sand with silt, includes silt seams (dense, wet)			
	75		18	38		19							
-60									SP-SM	Black fine to medium sand with silt and occasional shell fragments (dense, wet)			
	80		16	47		20							
-65									SP-SM/SM	Black fine to medium sand with silt, includes silty fine sand lenses (very dense, wet)			
	85		12	52		21							
-70									ML	Gray silt with sand (medium stiff, wet)			
	90		18	7		22 / AL					32		
-75													
	95		24	P		23			SP-SM	Black fine to medium sand with silt (very dense, wet)			
-80													
	100		13	50		24							

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-2 (continued)

GeoEngineers

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	3/3/2011	3/4/2011	249	WLT	CAM	Holocene	Mud Rotary

Surface Elevation (ft) / Vertical Datum	18.9	Hammer Data	Autohammer 140 lb / 30" drop	Drilling Equipment	BK 81 Drill Rig
Easting (X) / Northing (Y)		System / Datum		Groundwater	
Notes: SPT/Shelby					

Groundwater: Date Measured	Depth to Water (ft)	Elevation (ft)
3/3/2011	7.5	11.4

FIELD DATA

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	0								SM	Gray silty fine to coarse sand with gravel and wood fragments (loose, moist) (fill)			
15			14	9		1							
	5		6	5		2				Grades to brown with brick fragments (very loose, moist)			
			2	2		3	∇						
									SP	Black-brown fine to medium sand with wood debris and fine gravel (very loose, wet)			
10									ML	Gray sandy silt with gravel and occasional wood fragments (stiff, moist to wet)			
	10		6	11		4							
									ML	Black sandy silt with trace wood fragments (soft, wet) (tidal deposits)			
			15	3		5							
5													
	15		18	0		6 / AL			ML	Black silt with sand (very soft, wet)	37		
0									SP-SM	Black fine to medium sand with silt (loose, wet)			
	20		12	4		7 / %F					46		%F=92
									ML	Gray sandy silt (medium stiff, wet)			
-5									SM	Black silty fine to medium sand with shell fragments, occasional wood fragments and gravel (medium dense, wet) (alluvium)			
	25		14	14		8 / %F					31		%F=18
-10										Grades to include silty fine sand seams (dense, wet)			
	30												

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-3

GeoEngineers

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	30		14	38		9				Grades to dense			
-15									SP-SM	Black fine to medium sand with silt, gravel and occasional shell fragments (dense, wet)			
	35		12	40		10							
-20													
	40		16	48		11							
-25									ML	Gray silt with sand, occasional shell fragments, wood fragments and gravel (very stiff, wet)			
	45		14	29		12 / AL					33		
									SP-SM	Black fine to medium sand with silt (medium dense, wet)			
-30													
	50		14	56		13				Grades to very dense			
-35													
	55		13	59		14							
-40													
	60		15	21		15 / %F			SP-SM/ML	Black fine to medium sand with silt, includes sandy silt seams and wood fragments (medium dense, wet)	42		%F=43
-45									ML	Gray silt with sand and occasional shell fragments (medium stiff, wet)			
	65		18	38		16 / AL					33		

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-3 (continued)

GeoEngineers

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
									SP-SM	Black fine to medium sand with silt (dense, wet)			
-50									SM	Dark gray silty fine to medium sand (dense, wet)			
	70		13	33		17							
-55													
	75		13	51		18				Grades to very dense, includes occasional wood fragments			
									SP-SM	Black fine to medium sand with silt (very dense, wet)			
-60													
	80		18	27		19							
									SM/ML	Gray silty fine sand and gray silt (medium dense, wet)			
			20	P		20							
-65									SP-SM	Black fine to medium sand with silt (dense, wet)			
	85		15	38		21							
-70									SM	Black silty fine to medium sand with silt seams (medium dense, wet)			
	90		12	26		22 %F					29		%F=44
			23	P		23							
-75													
	95		13	37		24				Grades to dense			
-80									SP-SM	Black fine to medium sand with silt (dense, wet)			
	100		12	42		25							

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-3 (continued)

Elevation (feet)	Depth (feet)	Interval Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
-90	110	14	50		26				Grades to very dense			
-100	120	16	56		27			SM	Black silty fine to medium sand (very dense, wet)			
-105	125							SP-SM	Black fine to medium sand with silt (dense, wet)			
-110	130	13	39		28							

Depth scale: 105, 110, 115, 120, 125, 130, 135. Elevation scale: -85, -90, -95, -100, -105, -110, -115.

Note: See Figure B-1 for explanation of symbols.

Tacoma: Date:10/25/11 Path:P:\0\0570100\GINT\0570-100-01 BORING LOGS.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring B-3 (continued)

GEOENGINEERS

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
-120													
	140		18	16		29 / AL				Grades to include silt seams	30		
									CL	Gray lean clay with sand seams (very stiff, wet)			
-125	145												
									SM	Black silty fine to medium sand, includes silt seams (medium dense, wet)			
-130	150		14	24		30							
-135	155												
-140	160		2	25		31							
-145	165								SP-SM	Black fine to medium sand with silt (medium dense, wet)			
-150	170		6	14		32							

Note: See Figure B-1 for explanation of symbols.

Tacoma: Date:10/25/11 Path:P:\0\0570100\GINT\0570-100-01 BORING LOGS.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring B-3 (continued)

GEOENGINEERS

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
-155													
	175												
-160													
	180		12	27		33							
-165													
	185												
-170													
	190		11	64		34			SP-SM	Grades to very dense			
-175													
	195												
-180													
	200		9	50/5"		35							
-185													
	205												
-190													

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-3 (continued)

GEOENGINEERS

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Figure B-4

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	210								SP-SM	Black fine to medium sand with silt (medium dense, wet)			
-195													
	215												
-200													
	220												
-205													
	225												
-210													
	230												
-215													
	235												Organic matter observed in cuttings
-220													
	240												
-225													

Note: See Figure B-1 for explanation of symbols.

Tacoma: Date:10/25/11 Path:P:\0\0570100\GINT\0570-100-01 BORING LOGS.GPJ DBTemplate/LibTemplate:GEOENGINEERS8.GDT/GEI8_GEOTECH_STANDARD

Log of Boring B-3 (continued)

GeoEngineers

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Figure B-4

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content, %	Dry Density, (pcf)	REMARKS
	245												
-230			6	71		36			GM	Gray silty fine to coarse gravel with sand (very dense, wet) (glacially consolidated deposits)			

Note: See Figure B-1 for explanation of symbols.

Log of Boring B-3 (continued)

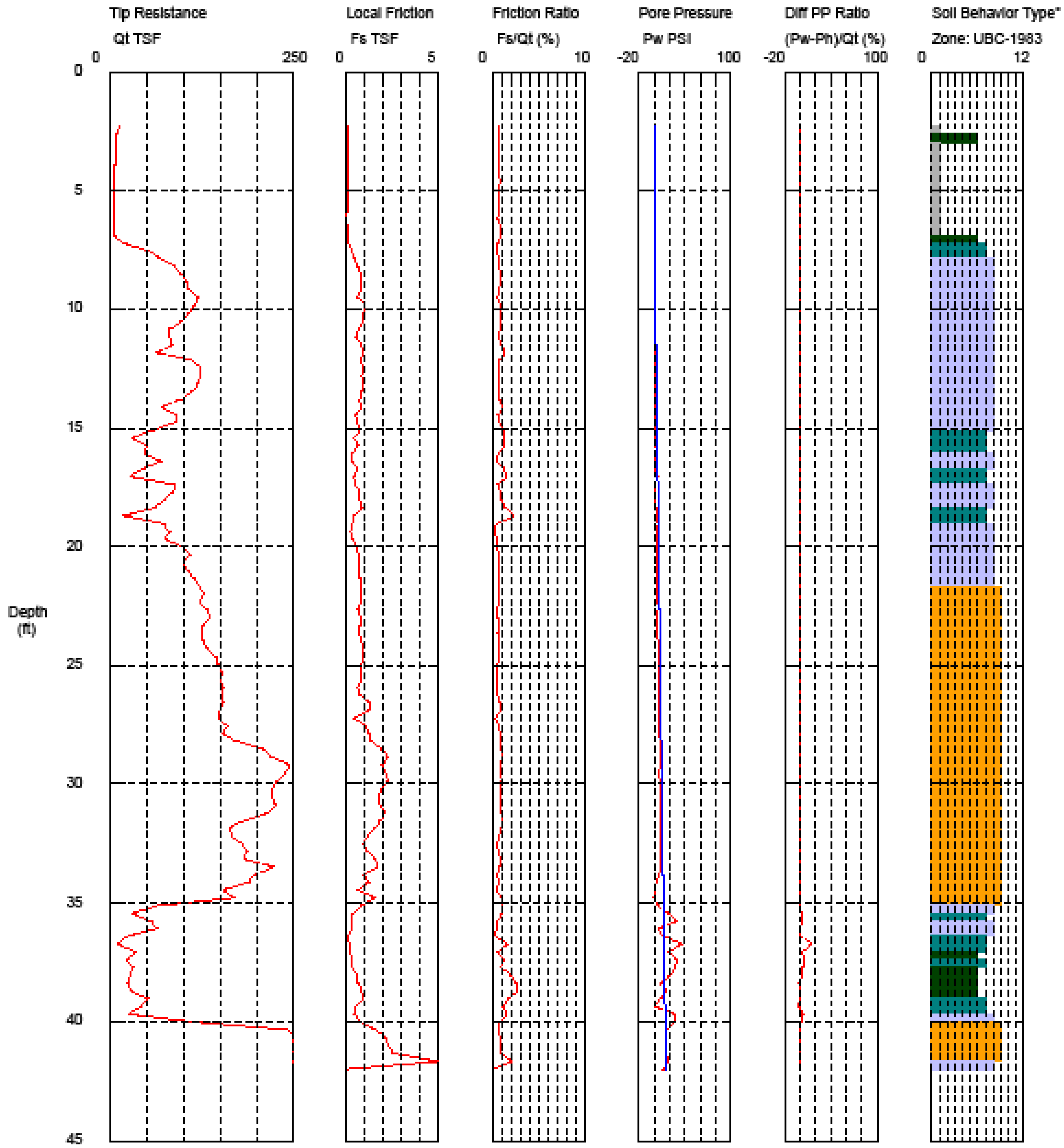
GeoEngineers

Project: Murray Morgan Bridge Rehabilitation Project
Project Location: Tacoma, Washington
Project Number: 0570-100-01

Subsurface Technologies

Operator: SAM
Sounding: P-1
Cone Used: DSG1021

CPT Date/Time: 3/3/2011 9:36:35 AM
Location: TACOMA
Job Number: 0570-100-01



Maximum Depth = 41.99 feet

Depth Increment = 0.328 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-1 (Page 1 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

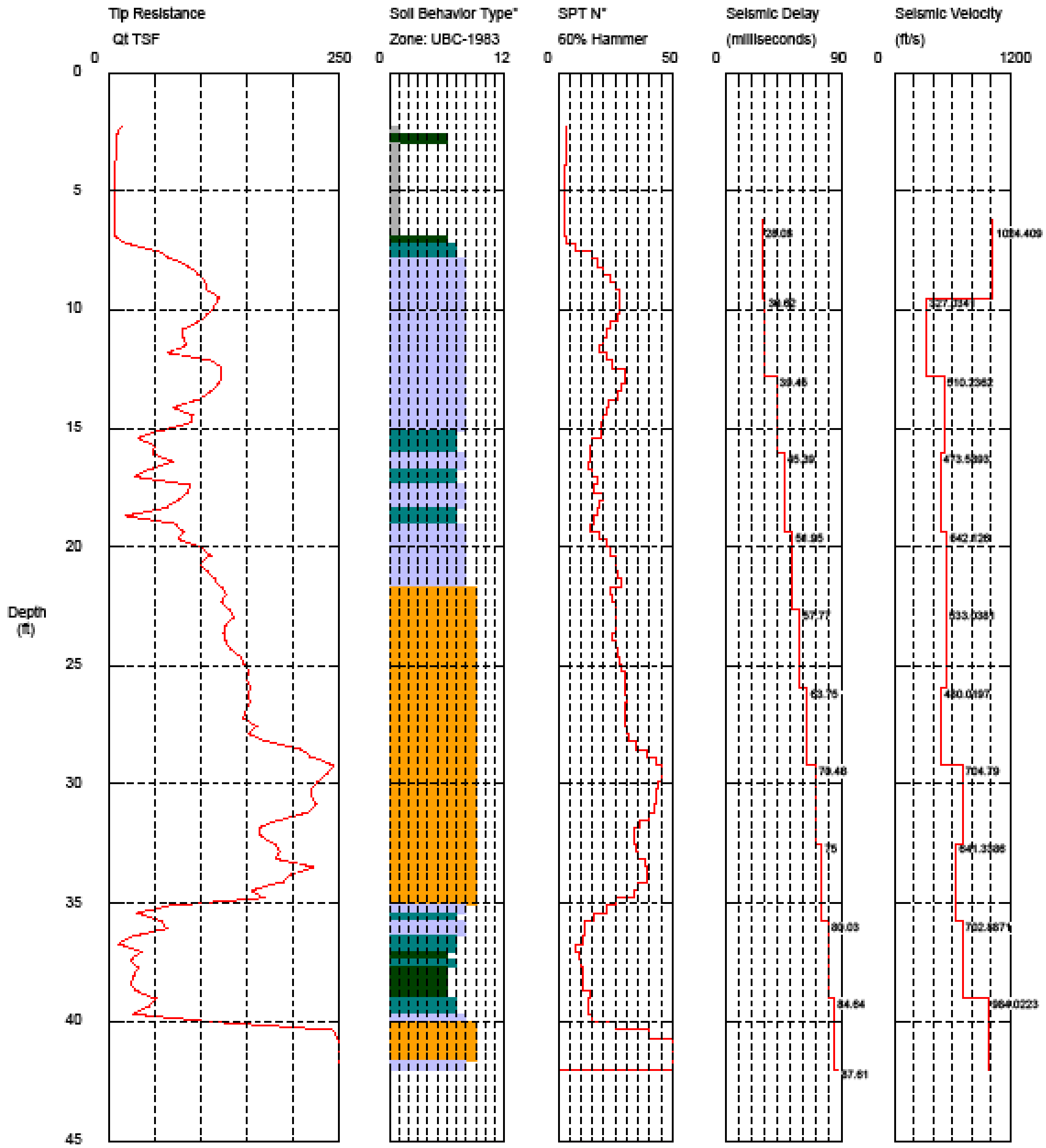
GeoEngineers

Figure B-5

Subsurface Technologies

Operator: SAM
Sounding: P-1
Cone Used: DSG1021

CPT Date/Time: 3/3/2011 9:36:35 AM
Location: TACOMA
Job Number: 0570-100-01



Maximum Depth = 41.99 feet

Depth Increment = 0.328 feet

1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-1 (Page 2 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure B-5

057010001 Figures B5_B10_CPTs.pptx

Subsurface Technologies

Operator: SAM
Sounding: P-2
Cone Used: DSG0457

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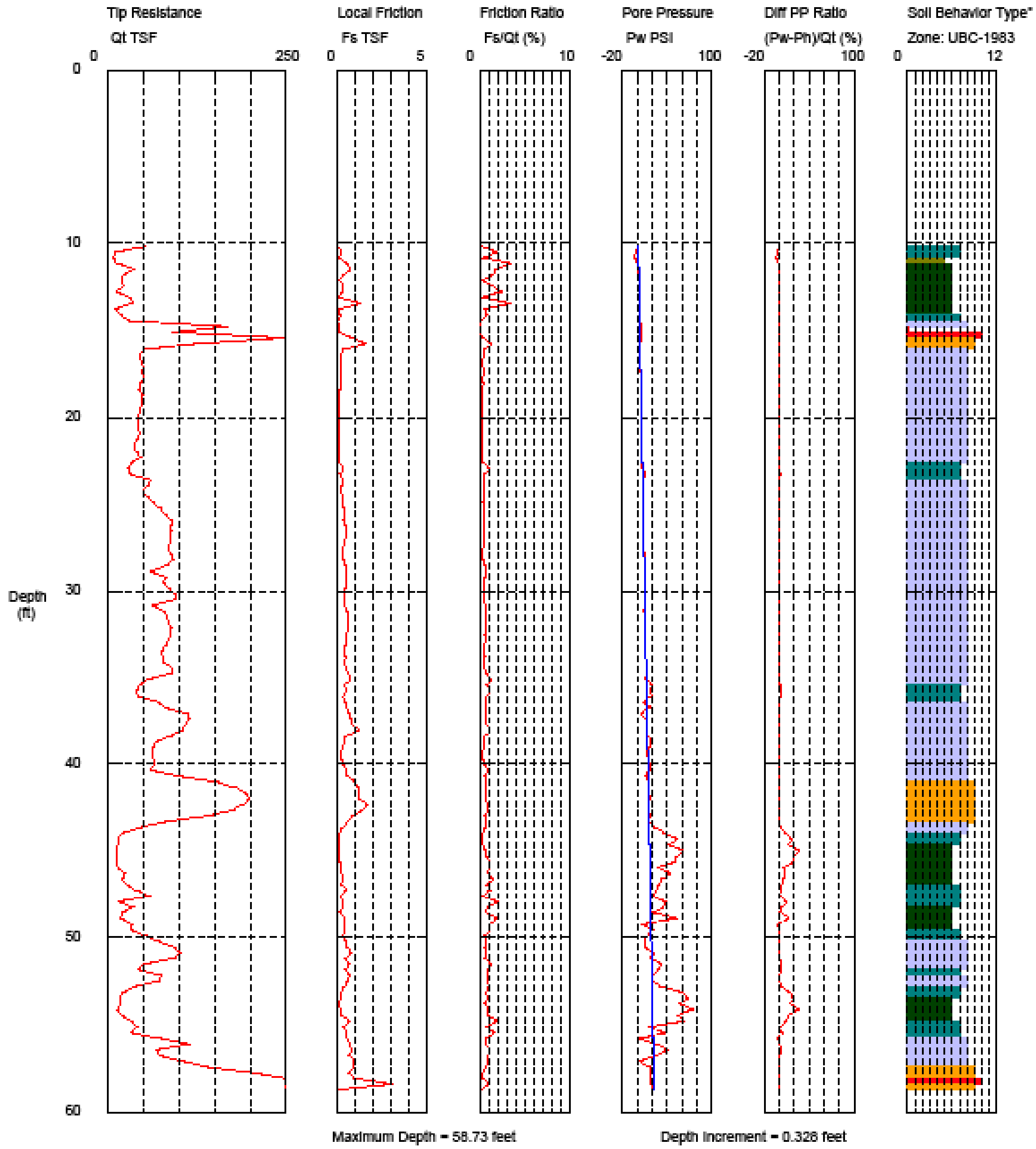
Location: TACOMA
Job Number: 0570-100-01



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-2 (Page 1 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

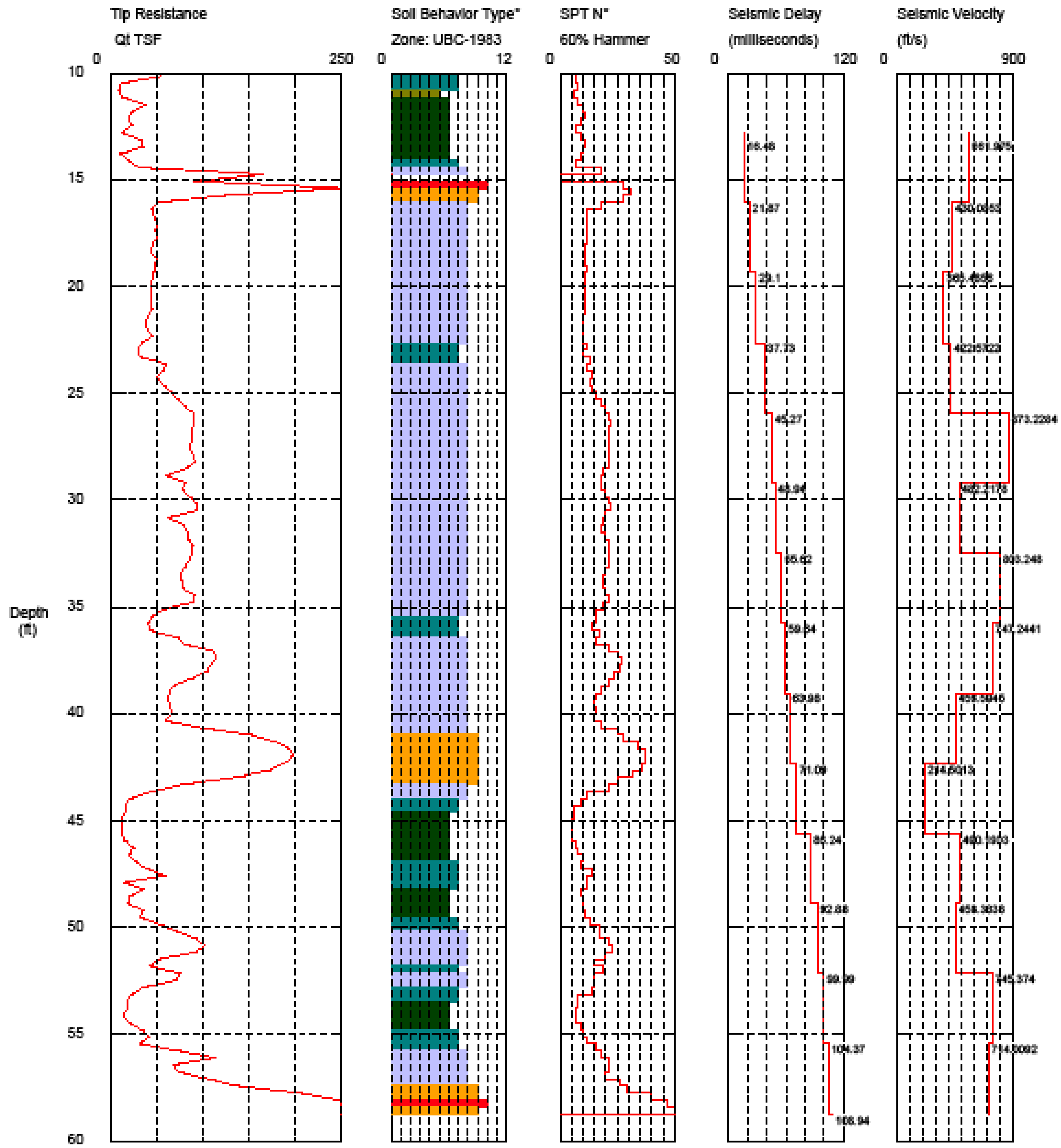
Figure B-6

057010001 Figures B5_B10_CPTs.pptx

Subsurface Technologies

Operator: SAM
Sounding: P-2
Cone Used: DSG0457

CPT Date/Time: 3/2/2011 2:39:20 PM
Location: TACOMA
Job Number: 0570-100-01



Maximum Depth = 58.73 feet
Depth Increment = 0.328 feet

1	sensitive fine grained
2	organic material
3	clay
4	silty clay to clay
5	clayey silt to silty clay
6	sandy silt to clayey silt
7	silty sand to sandy silt
8	sand to silty sand
9	sand
10	gravelly sand to sand
11	very stiff fine grained (*)
12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-2 (Page 2 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure B-6

057010001 Figures B5_B10_CPTs.pptx

Subsurface Technologies

Operator: SAM
Sounding: P-3
Cone Used: DSG0457

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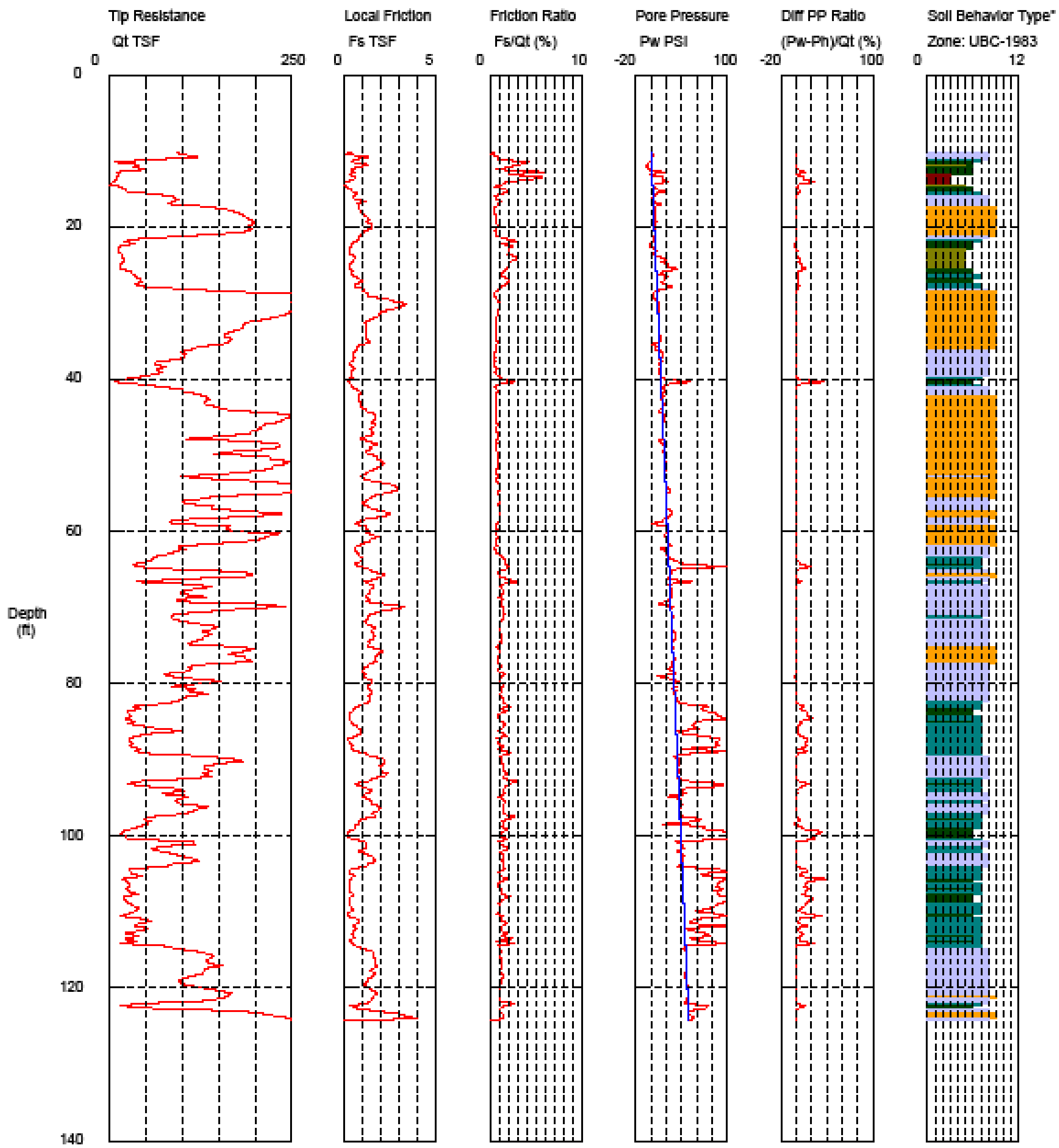
Location: TACOMA
Job Number: 0570-100-01



Maximum Depth = 124.34 feet

Depth Increment = 0.328 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-3 (Page 1 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure B-7

057010001 Figures B5_B10_CPTs.pptx

Subsurface Technologies

Operator: SAM
Sounding: P-3
Cone Used: DSG0457

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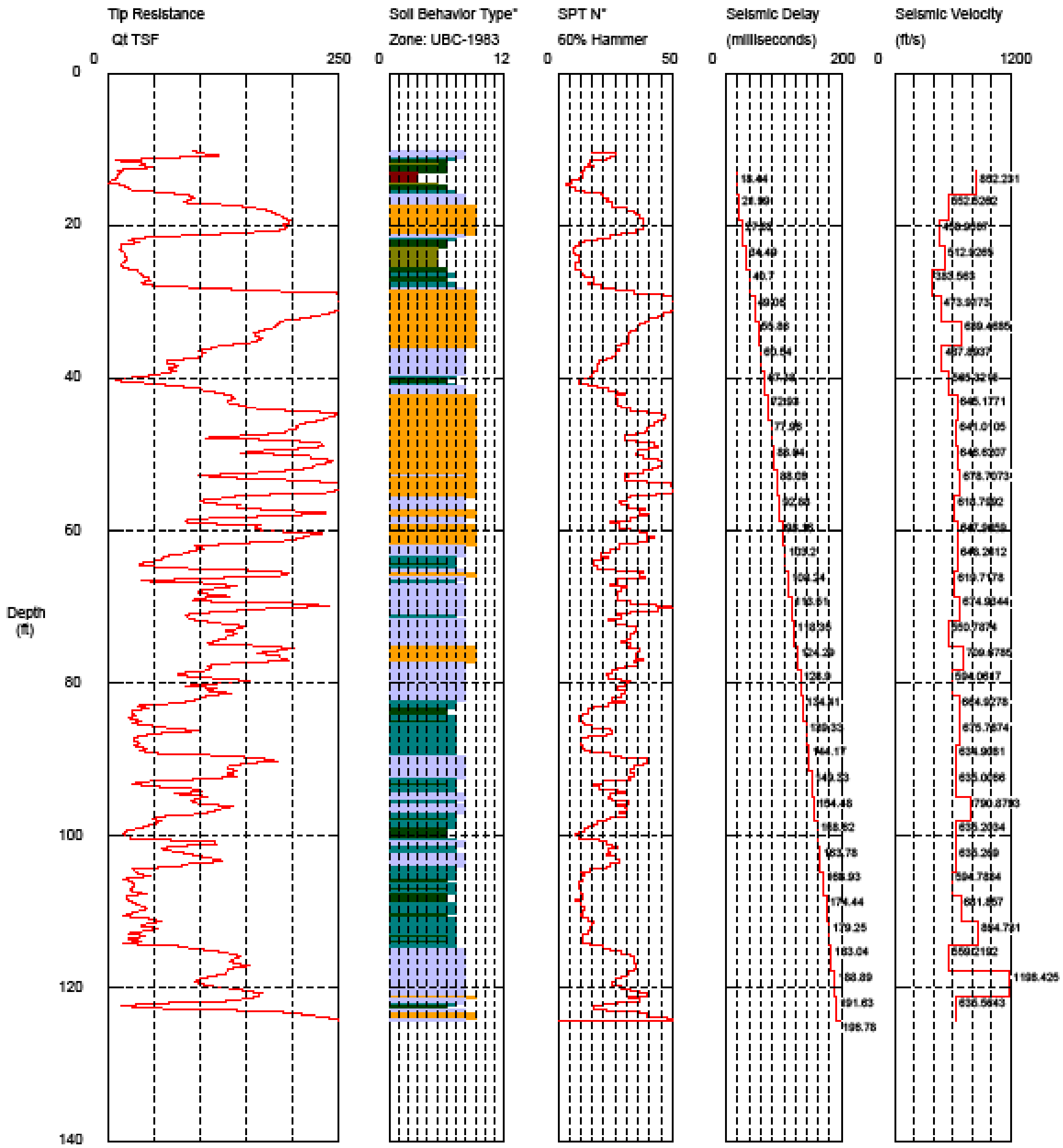
Location: TACOMA
Job Number: 0570-100-01



Maximum Depth = 124.34 feet

Depth Increment = 0.328 feet

1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-3 (Page 2 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

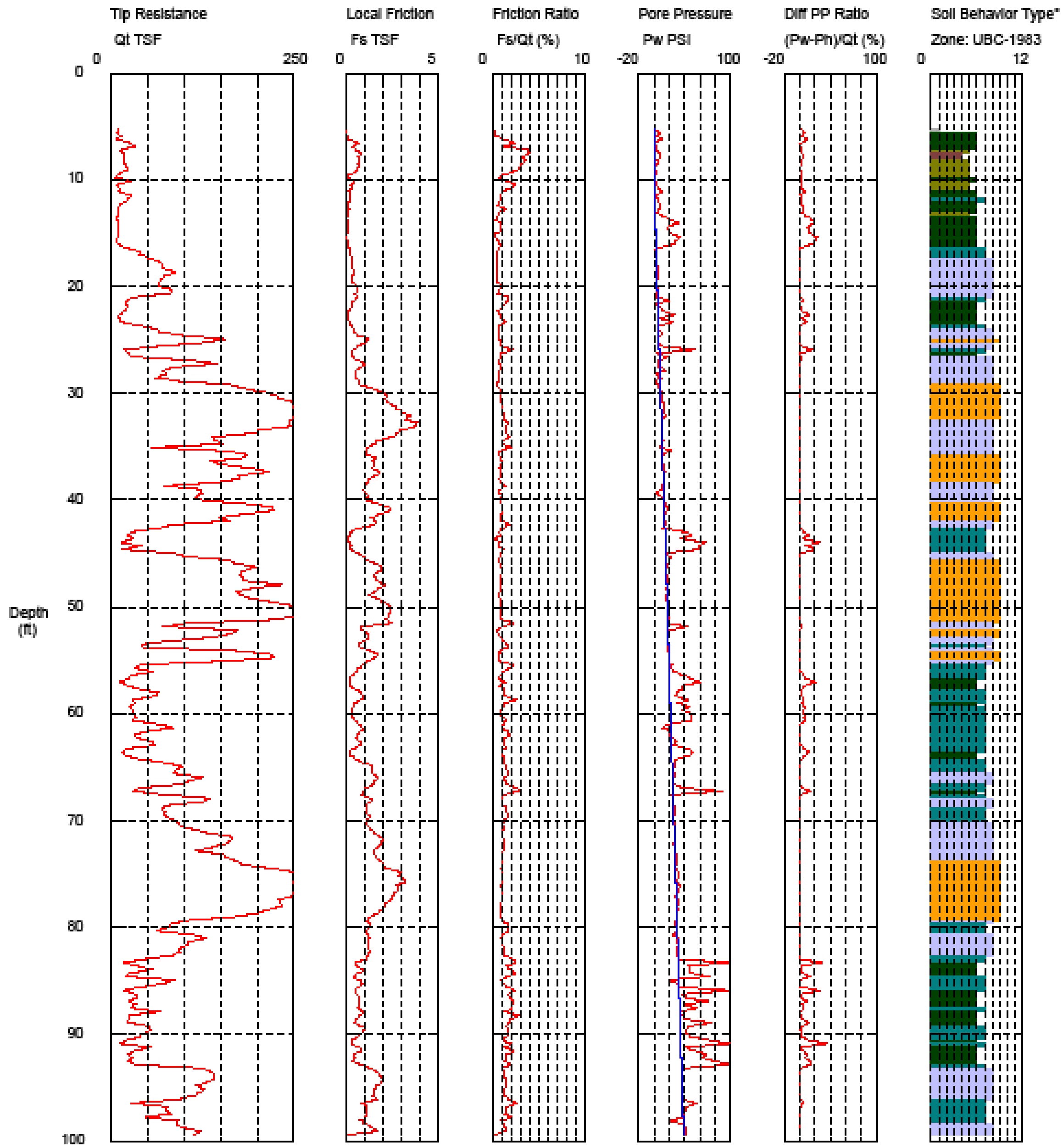
Figure B-7

057010001 Figures B5_B10_CPTs.pptx

Subsurface Technologies

Operator: SAM
Sounding: P-4
Cone Used: DSG0457

CPT Date/Time: 3/2/2011 9:37:13 AM
Location: TACOMA
Job Number: 0570-100-01



Maximum Depth = 99.41 feet
Depth Increment = 0.328 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-4 (Page 1 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

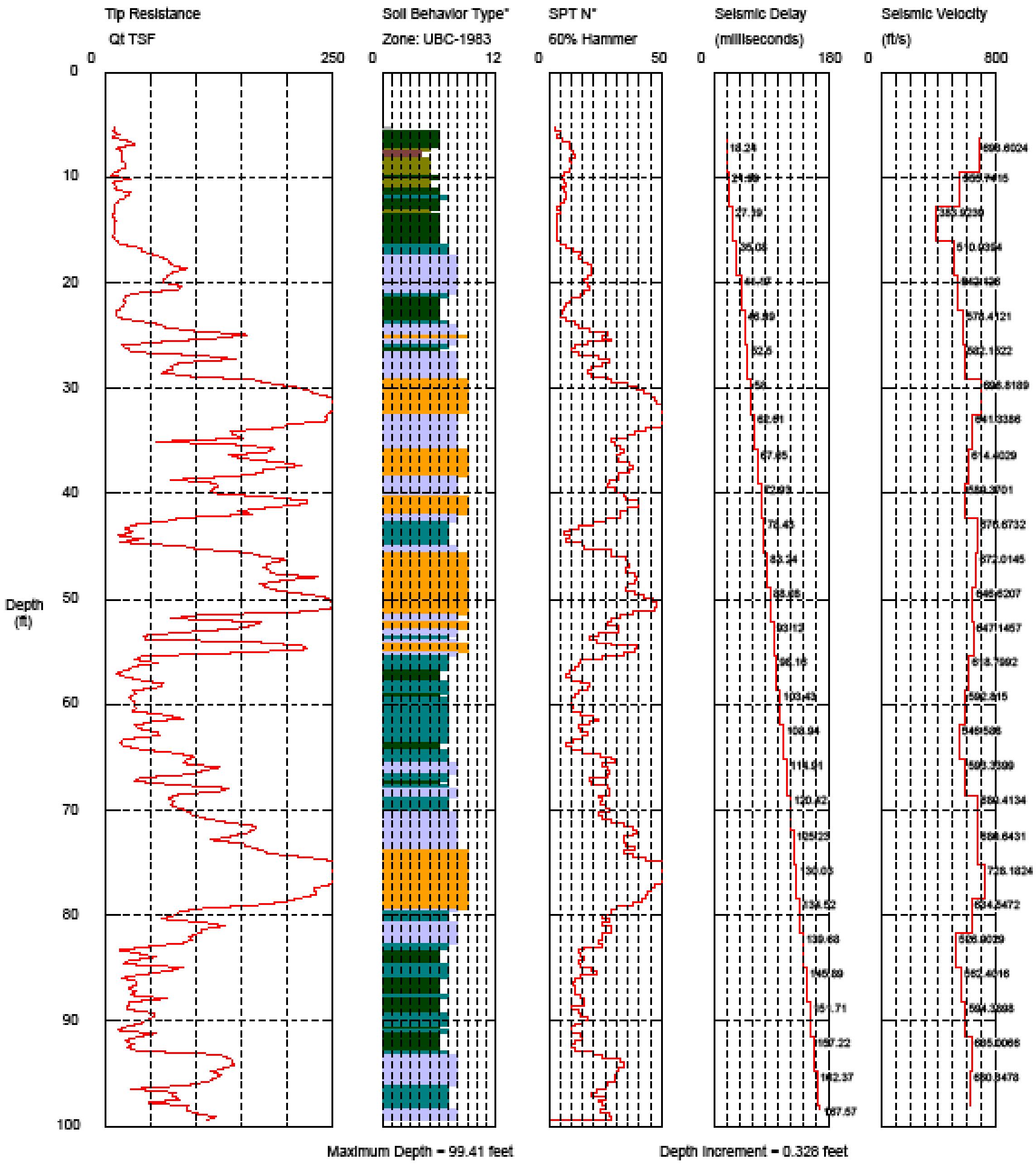
Figure B-8

057010001 Figures B5_B10_CPTs.pptx

Subsurface Technologies

Operator: SAM
Sounding: P-4
Cone Used: DSG0457

CPT Date/Time: 3/2/2011 9:37:13 AM
Location: TACOMA
Job Number: 0570-100-01



1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

*Soil behavior type and SPT based on data from UBC-1983

CPT P-4 (Page 2 of 2)

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

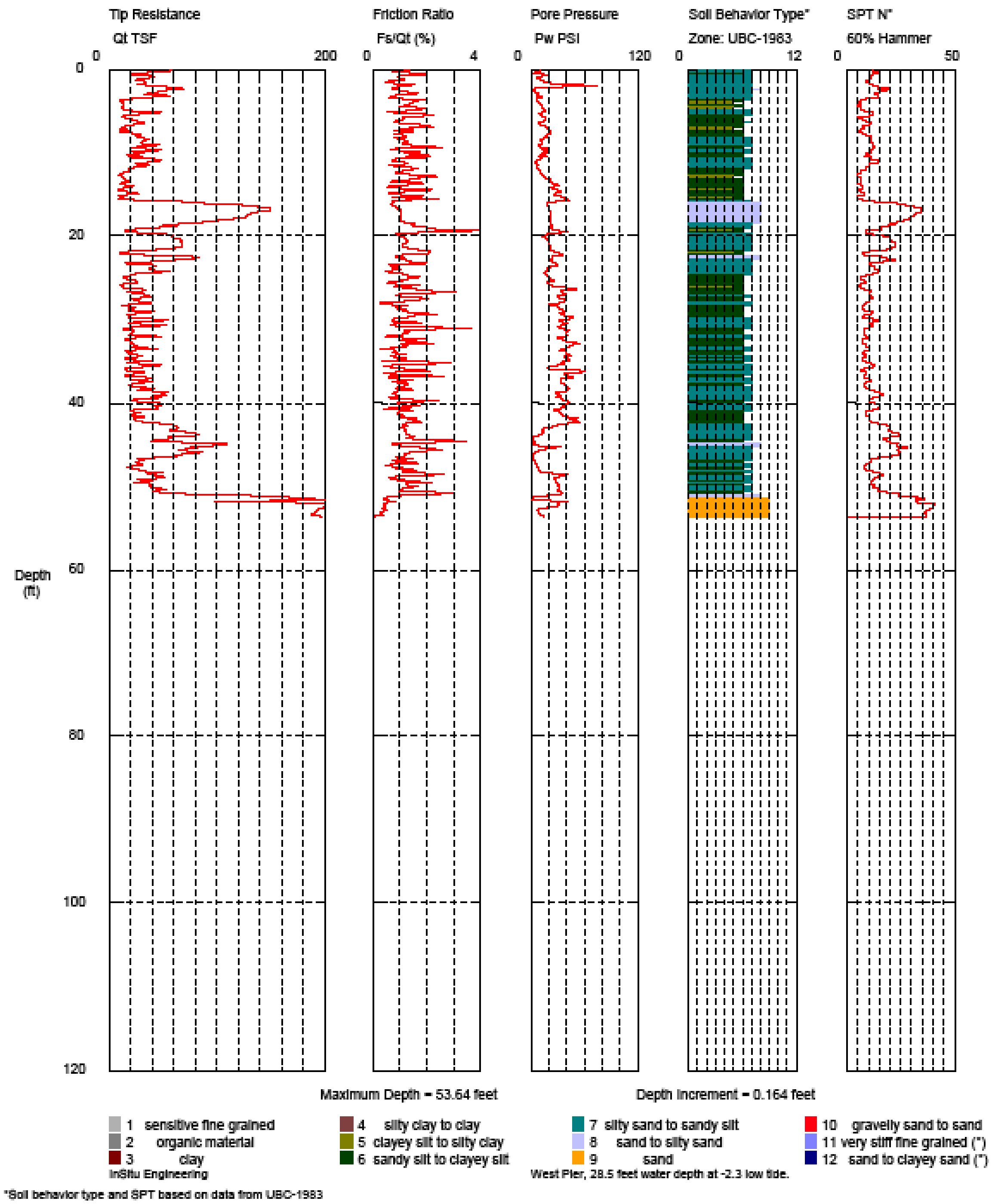
GEOENGINEERS

Figure B-8

GeoEngineers

Operator: Brown
Sounding: CPT-02
Cone Used: DSG1079

CPT Date/Time: 6/2/2011 12:38:49 PM
Location: 11th Street Bridge Tacoma
Job Number:



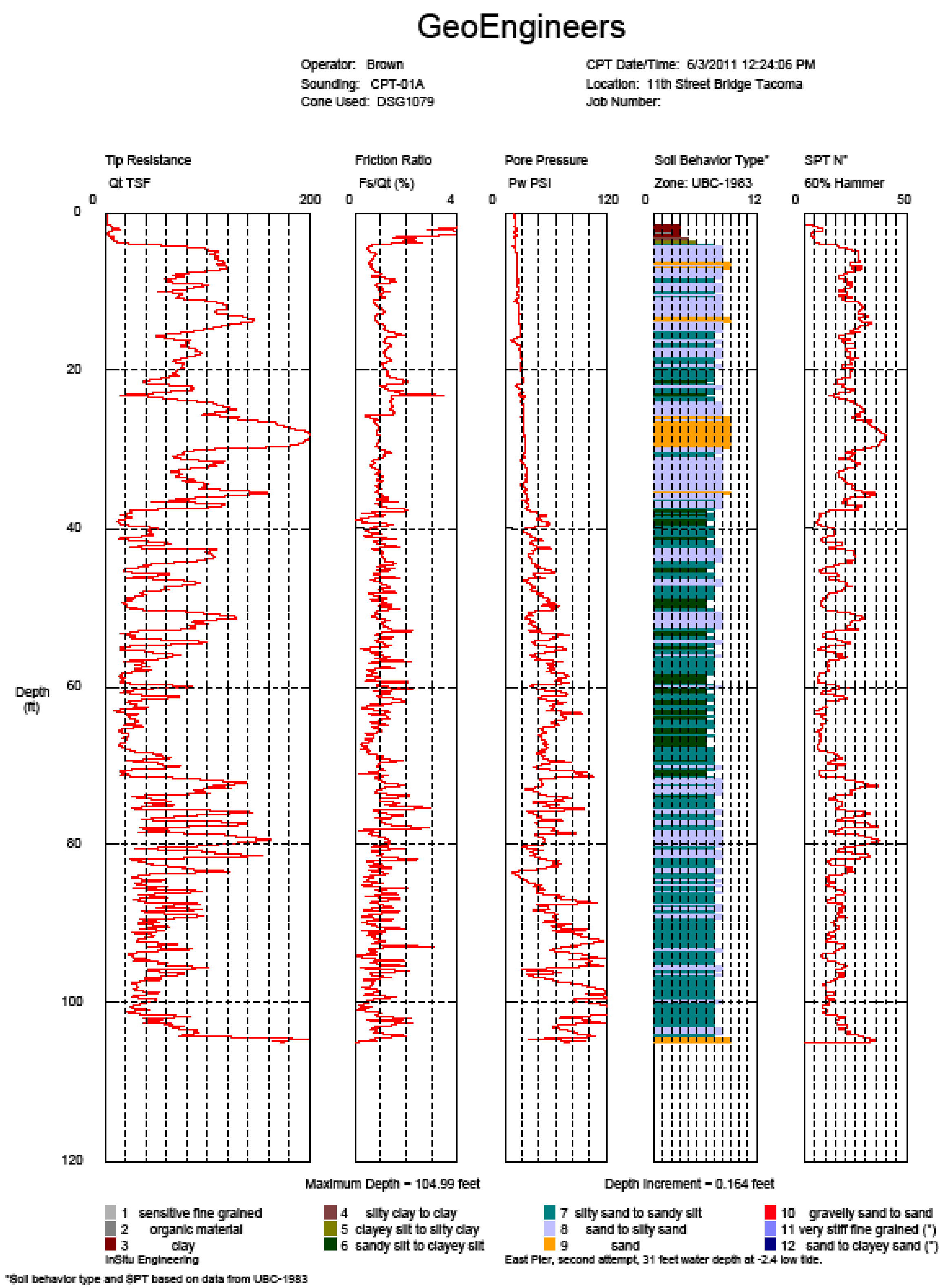
CPT P-5

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure B-9

057010001 Figures B5_B10_CPTs.pptx



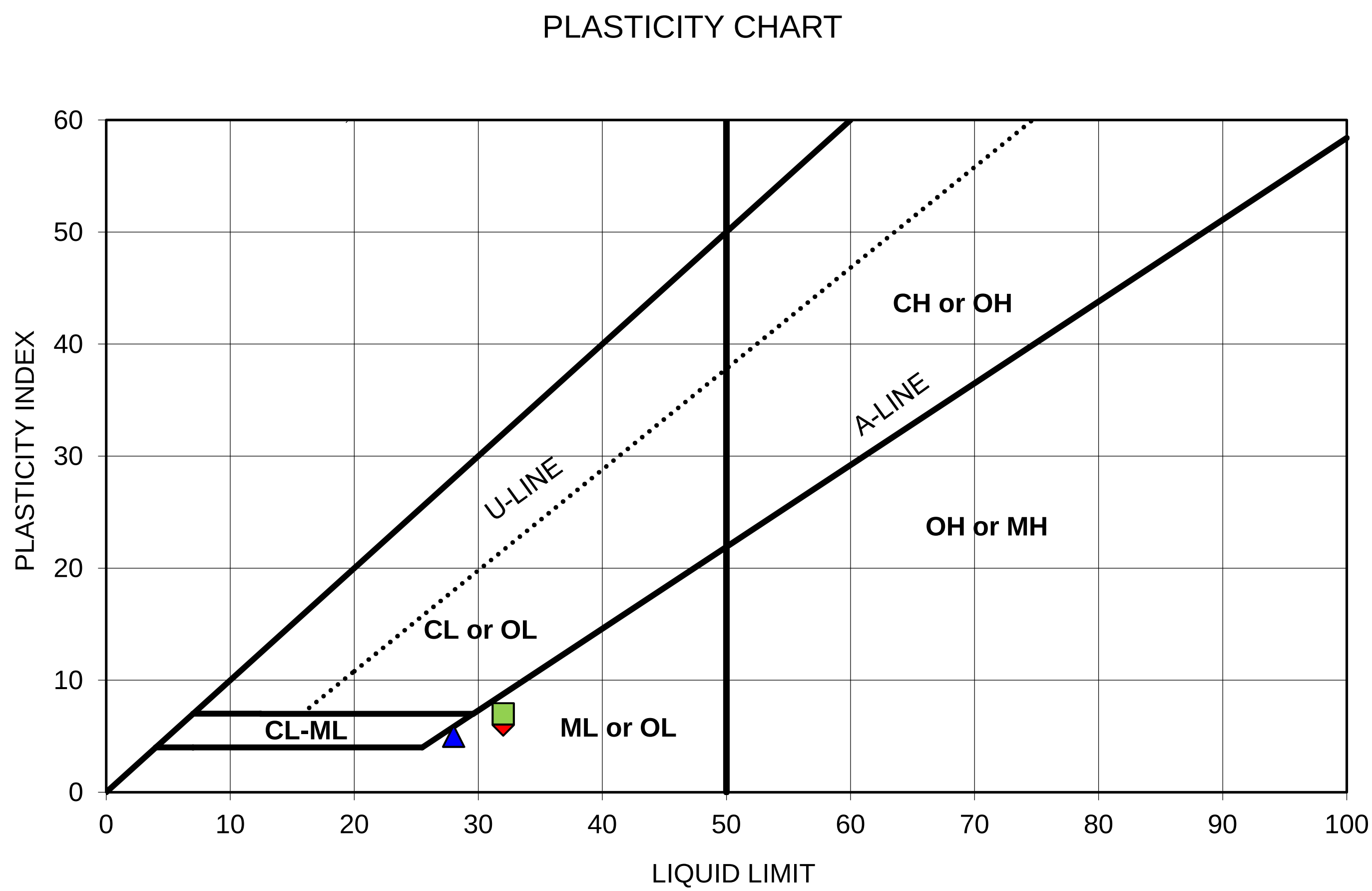
CPT P-6

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure B-10

057010001 Figures B5_B10_CPTs.pptx



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆ (red diamond)	B-2	90	32	32	6	Silt with sand (ML)
■ (green square)	B-3	15	37	32	7	Sandy silt (ML)
▲ (blue triangle)	B-3	45	33	28	5	Silt with sand (ML)

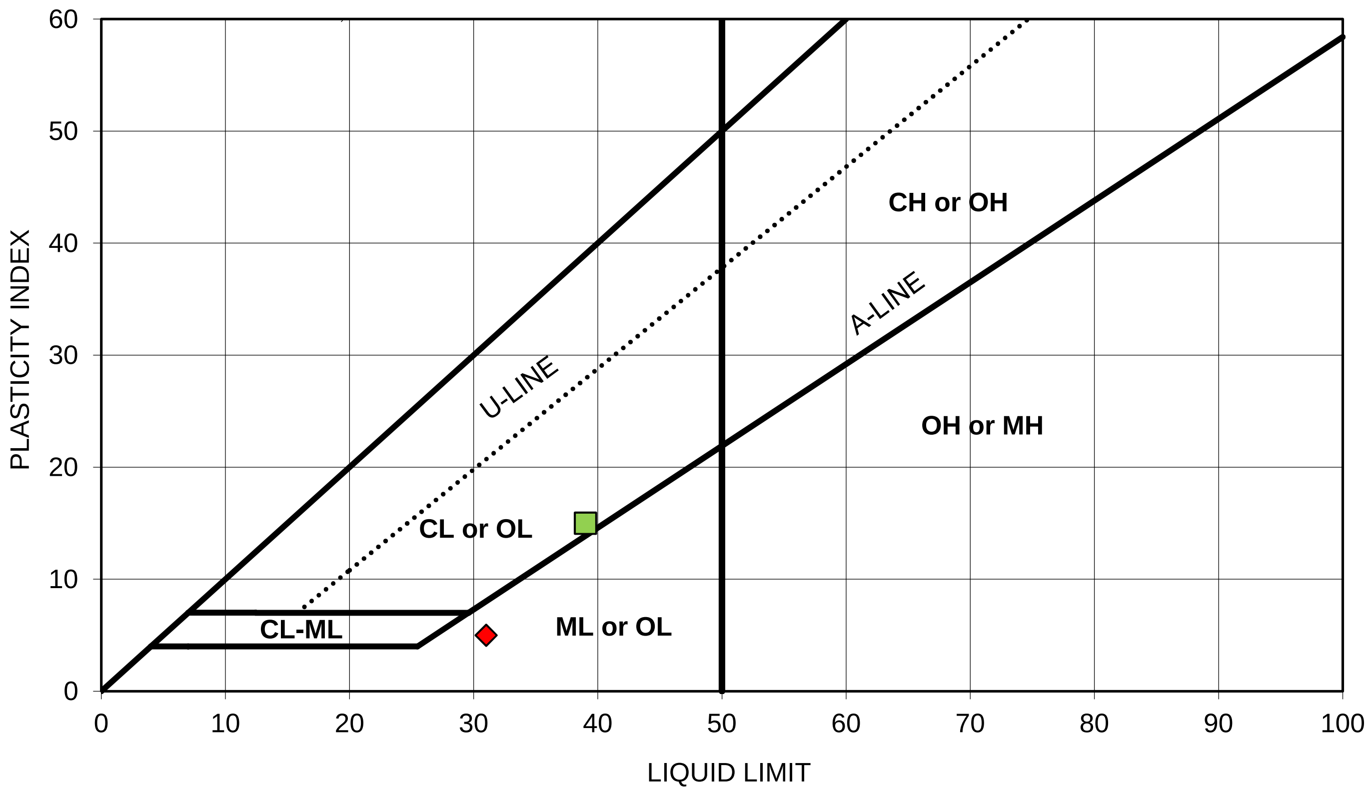
GeoEngineers

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

ATTERBERG LIMITS TEST RESULTS

FIGURE B-11

PLASTICITY CHART



SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH (ft)	MOISTURE CONTENT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	SOIL DESCRIPTION
◆	B-3	65	33	31	5	Silt with sand (ML)
■	B-3	140	30	39	15	Lean clay (CL)

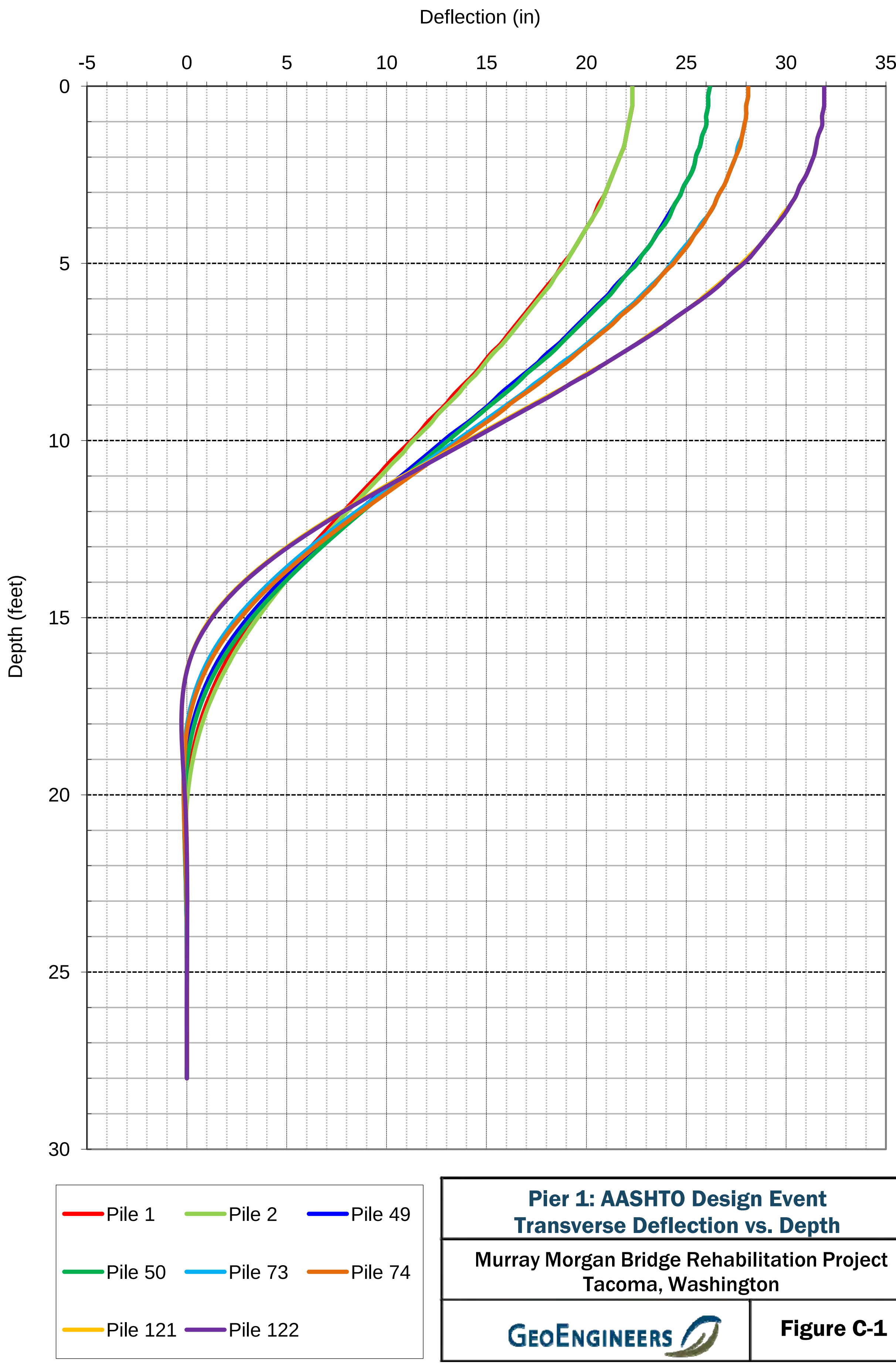
GeoEngineers

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

ATTERBERG LIMITS TEST RESULTS

FIGURE B-12

APPENDIX C
Displacement, Shear Force and Bending Moment at the Center Span Foundations (Piers 1 through 4)



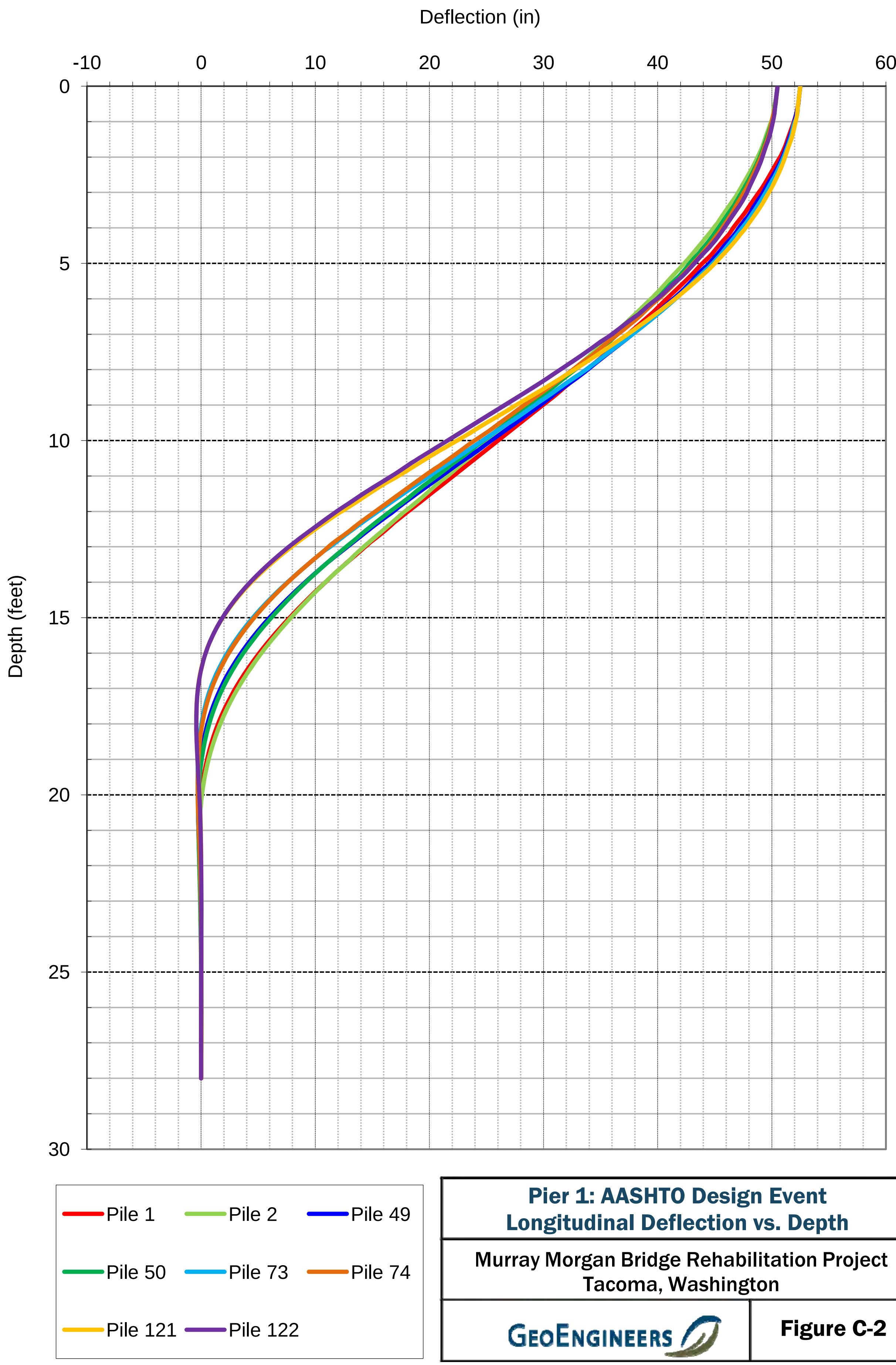
Pier 1: AASHTO Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-1

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



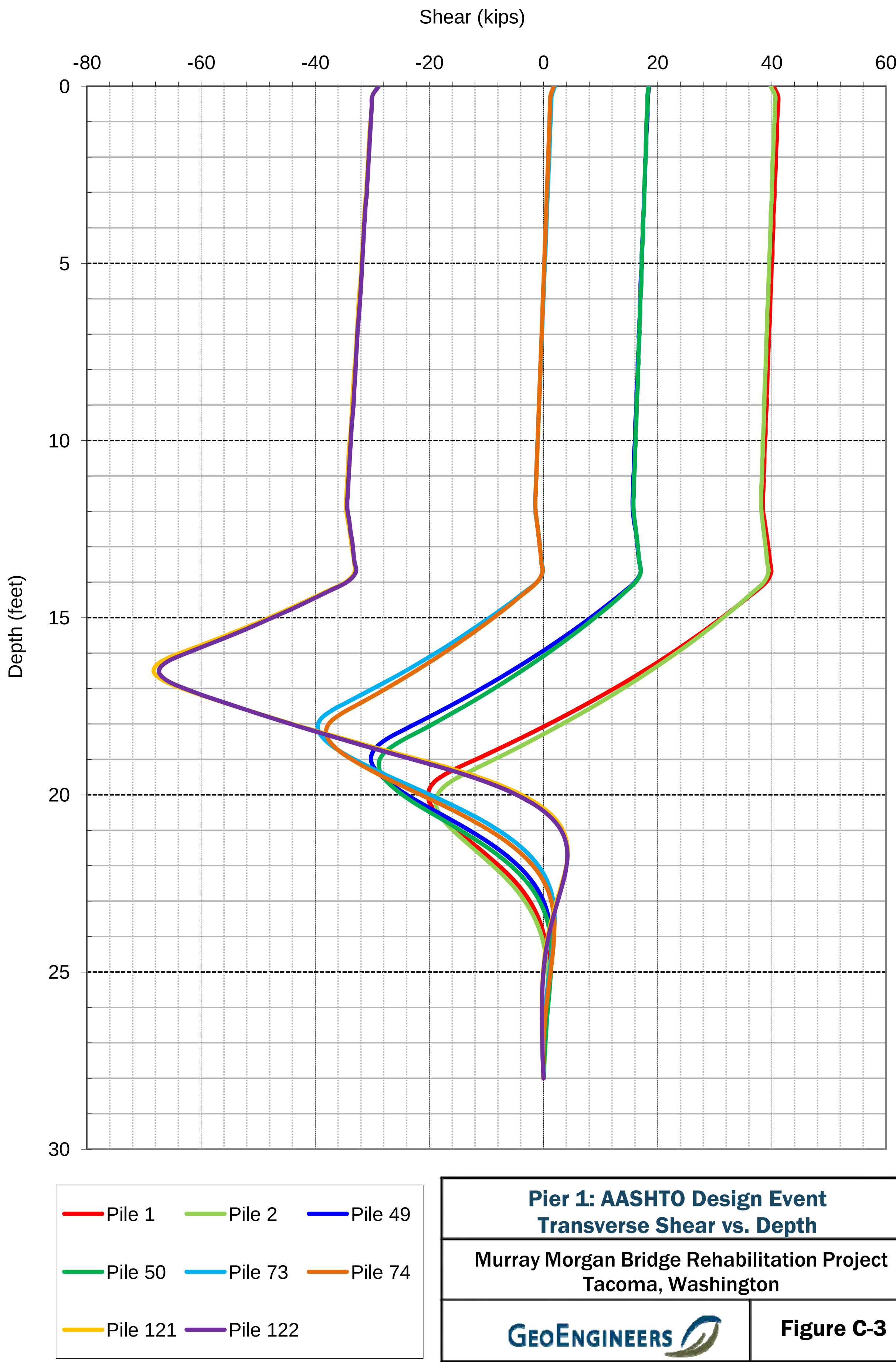
Pier 1: AASHTO Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-2

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



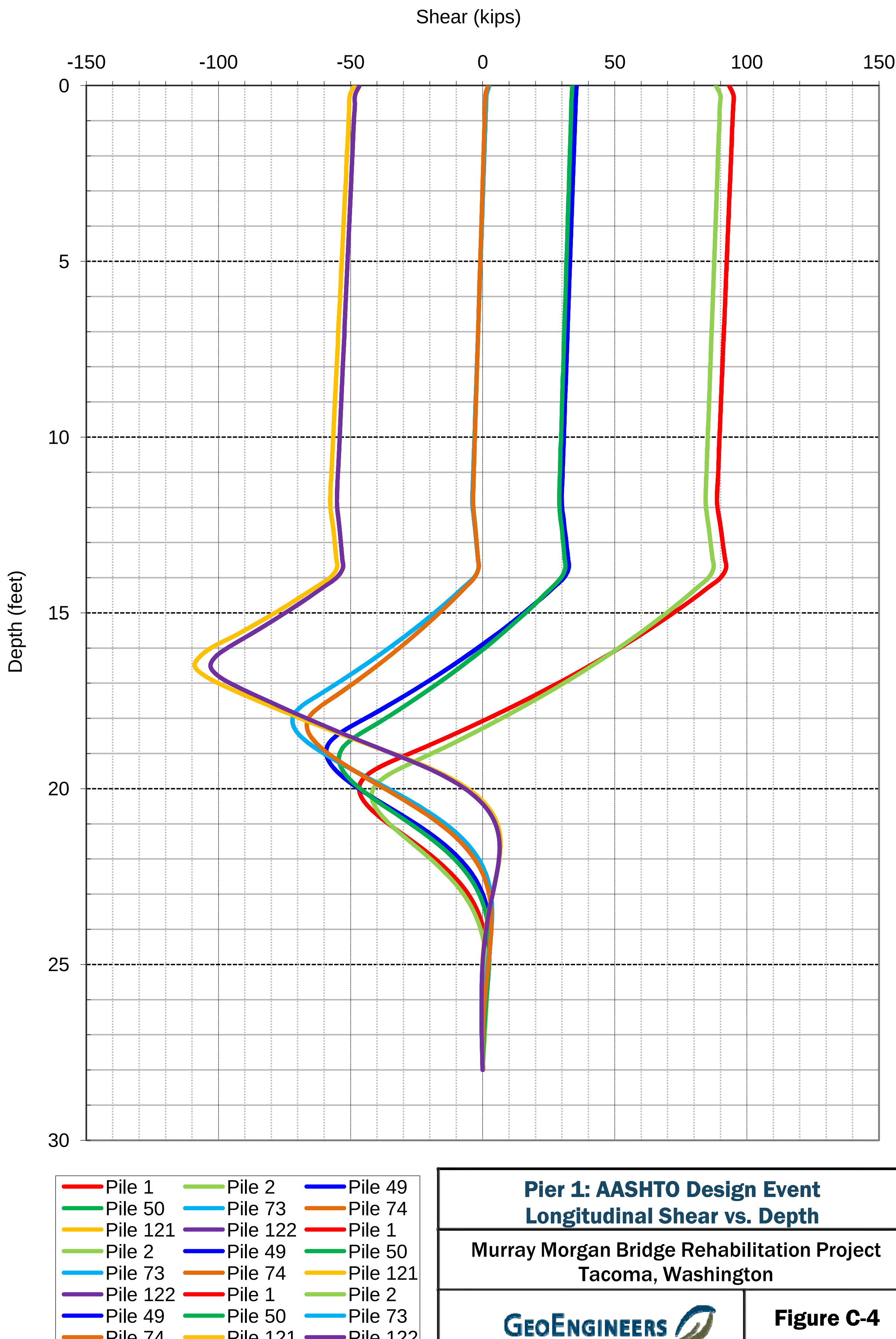
Pier 1: AASHTO Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-3

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



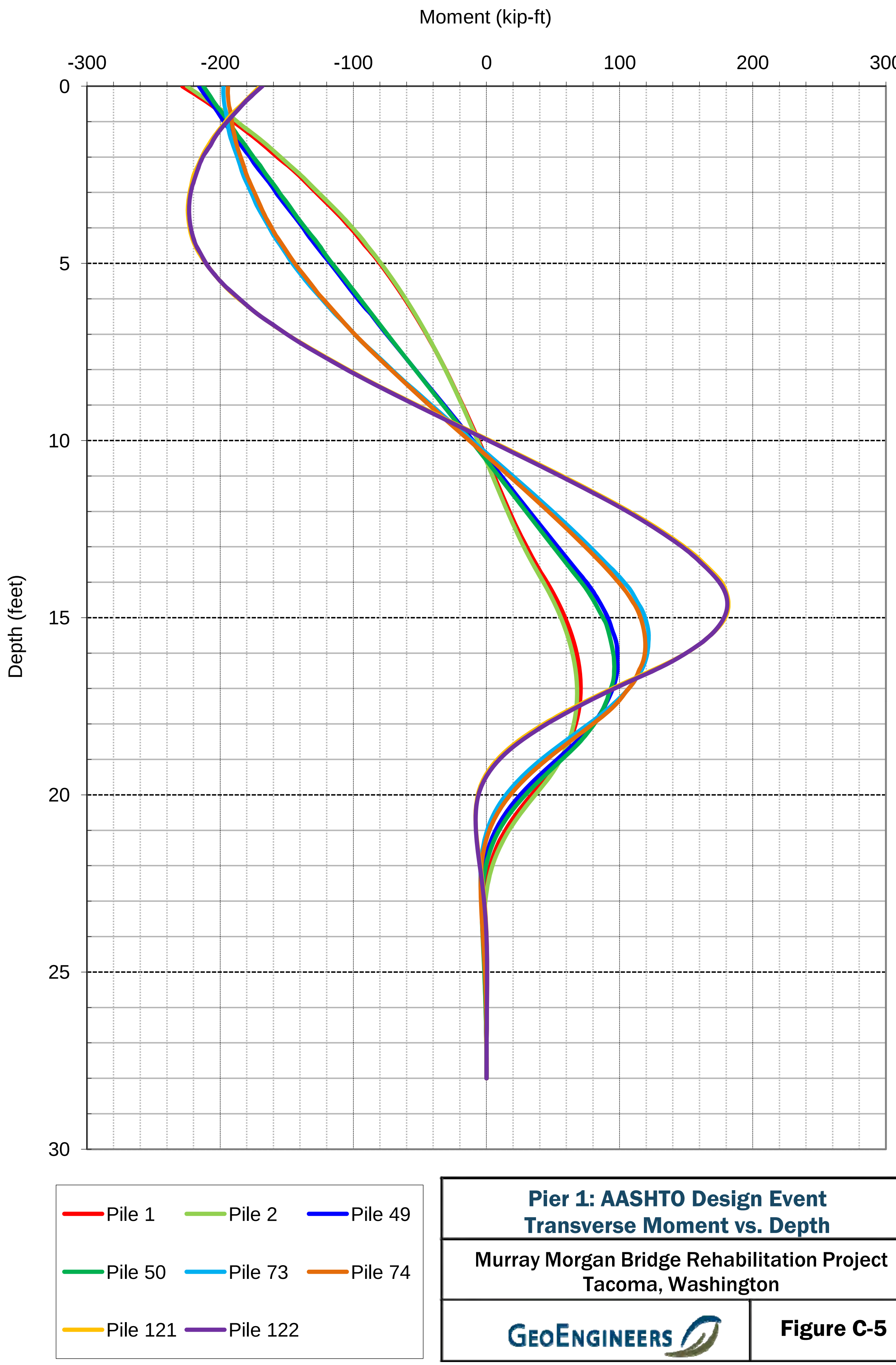
Pier 1: AASHTO Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-4

0570100002_FiguresC1_C16.xlsx WLT:khc 10/21/11



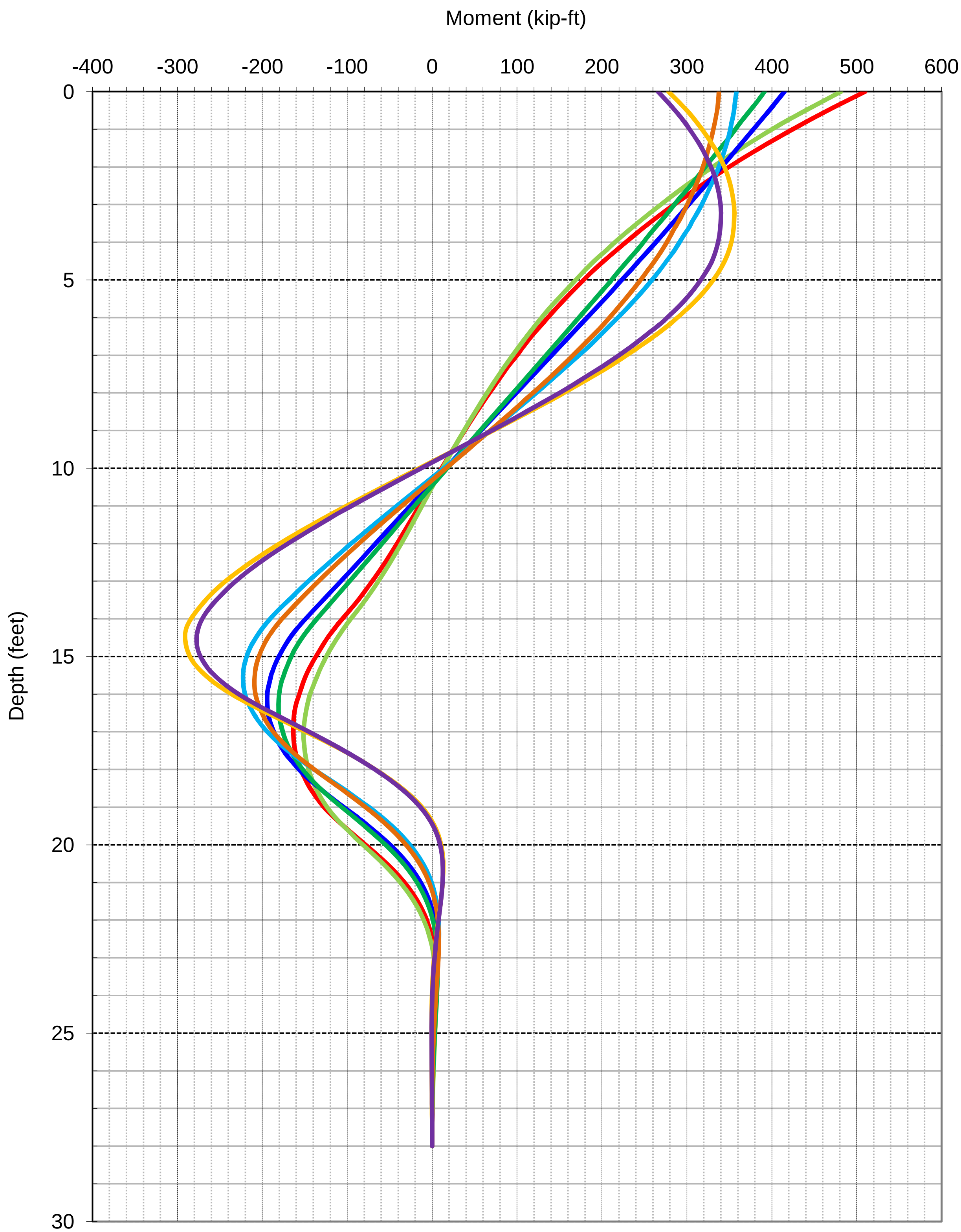
057010002_Figures C1_C16.xlsx WLT:khc 10/21/11

Pier 1: AASHTO Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-5



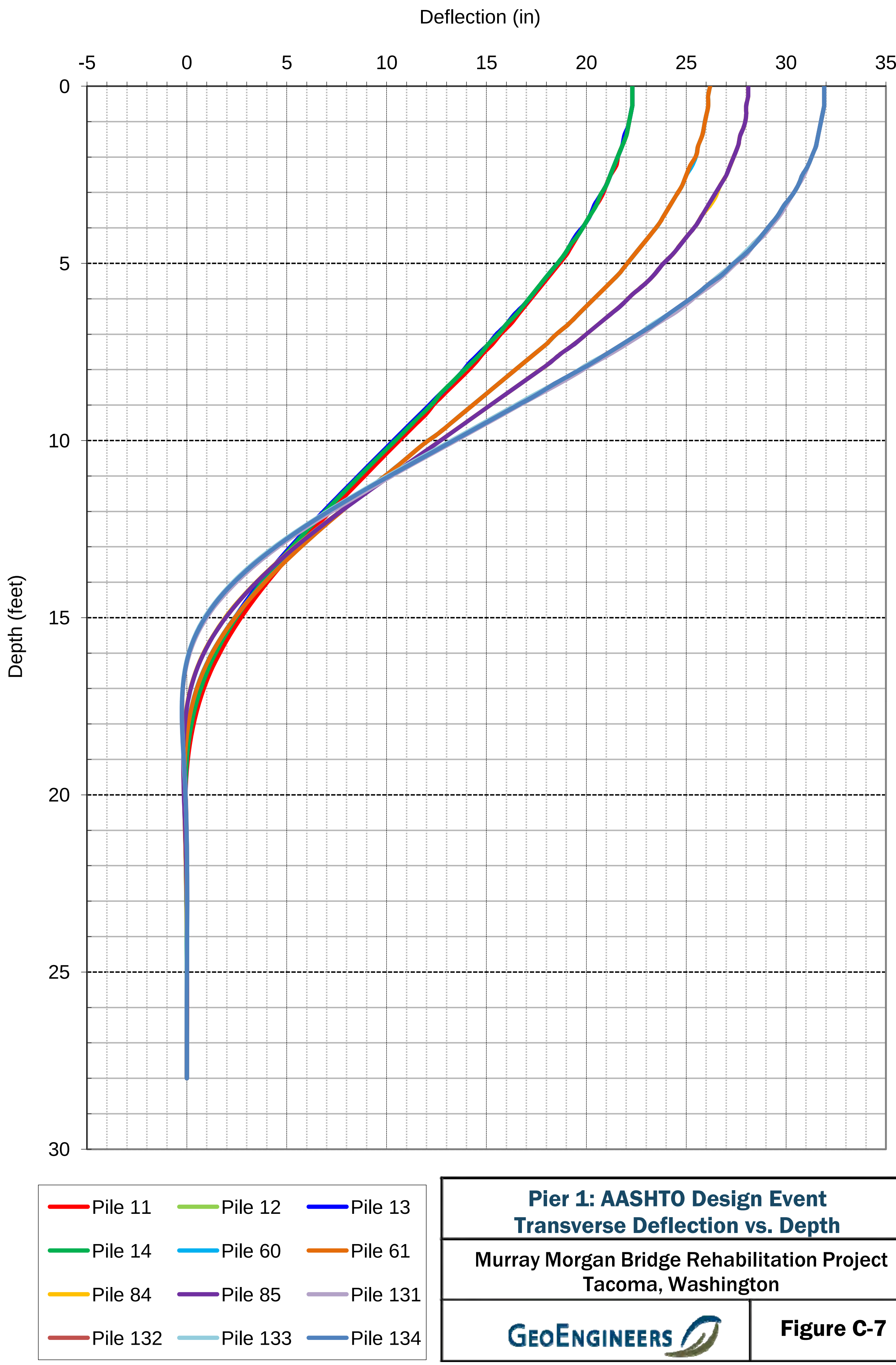
Pier 1: AASHTO Design Event Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-6

057010002_FiguresC1_C16.xlsx WLT:khc 10/21/11



057010002_Figures C1_C16.xlsx WLT:khc 10/21/11

Pier 1: AASHTO Design Event Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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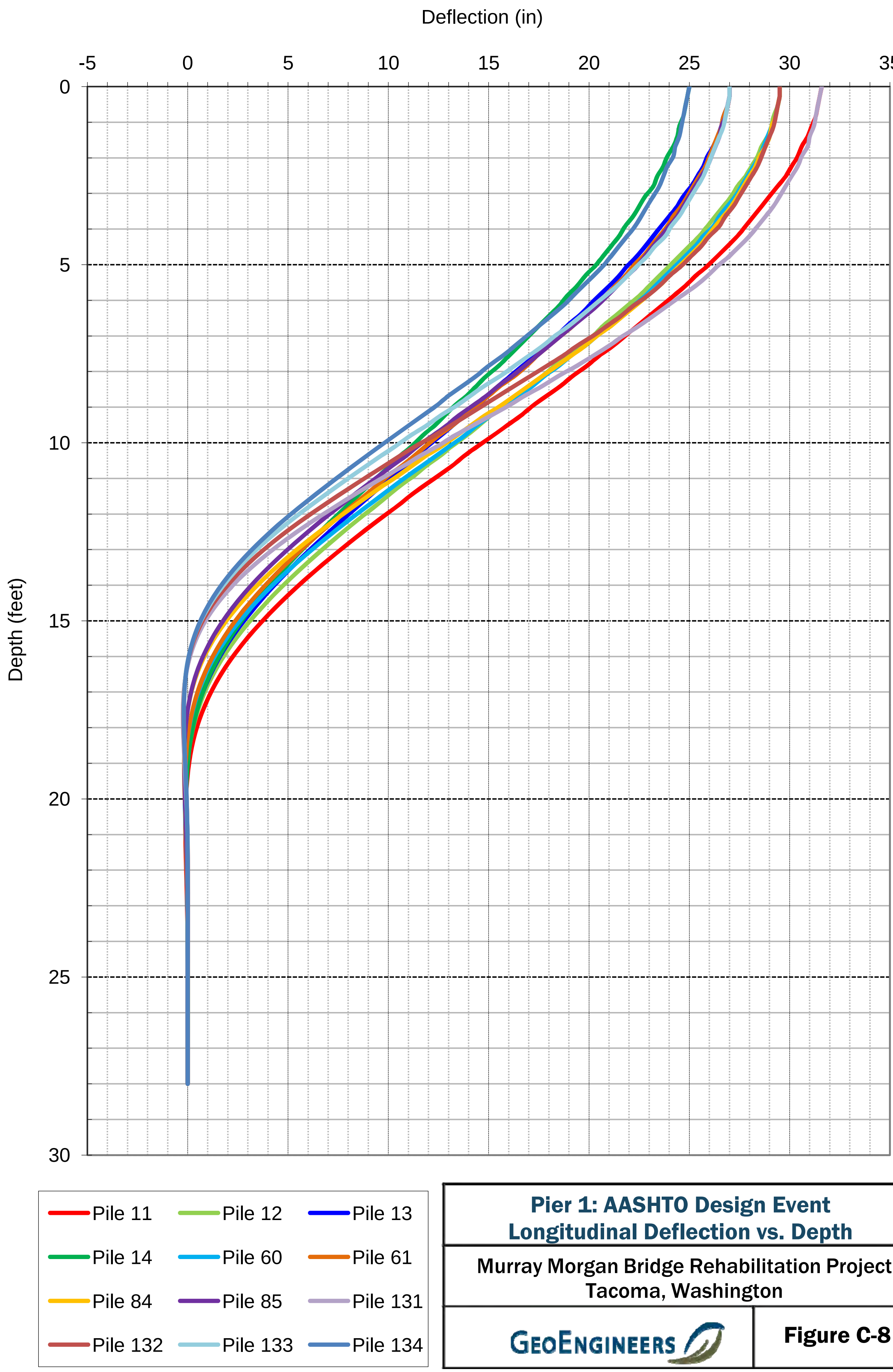
Figure C-7



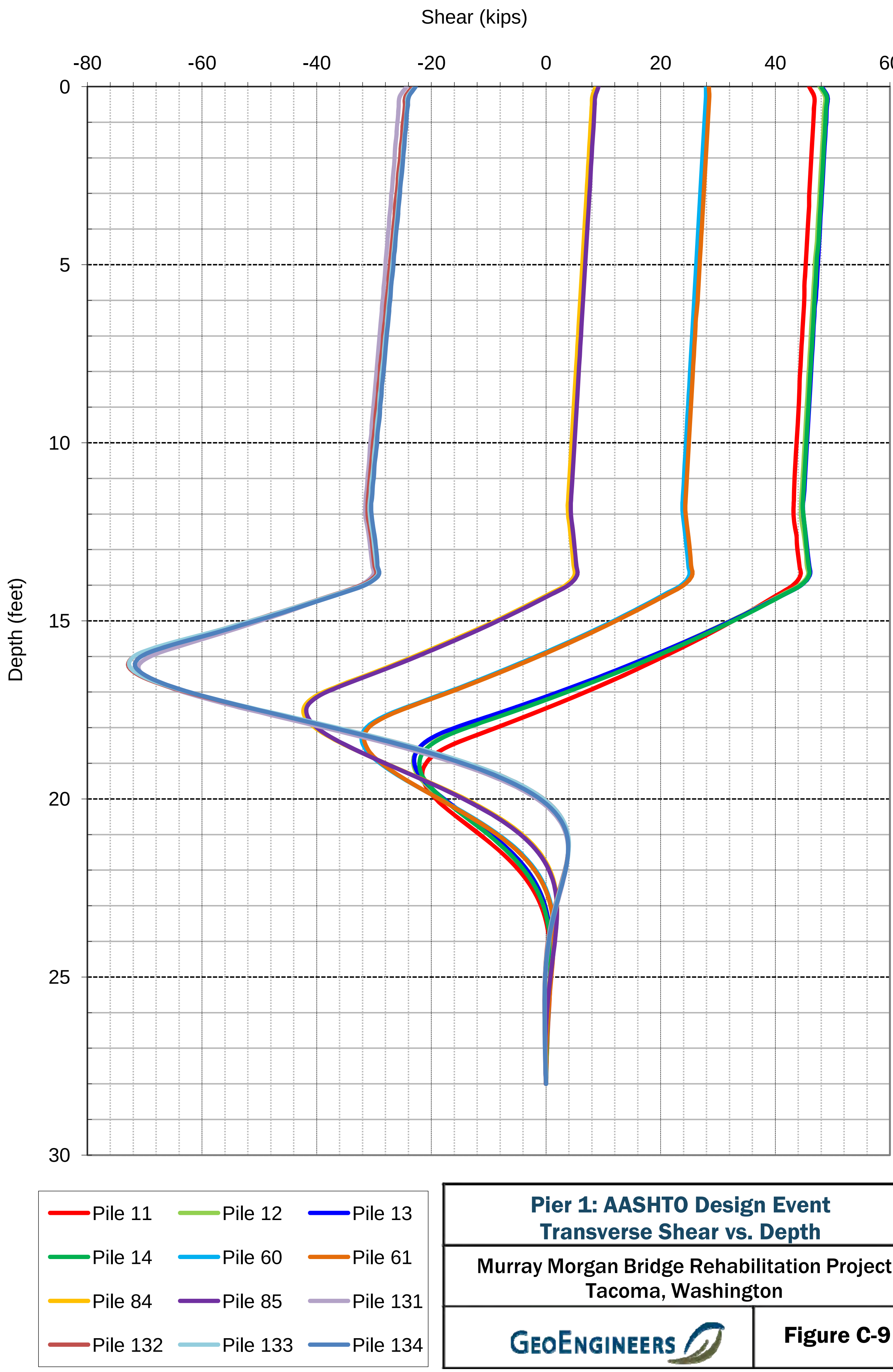
Pier 1: AASHTO Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-8

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



Pier 1: AASHTO Design Event Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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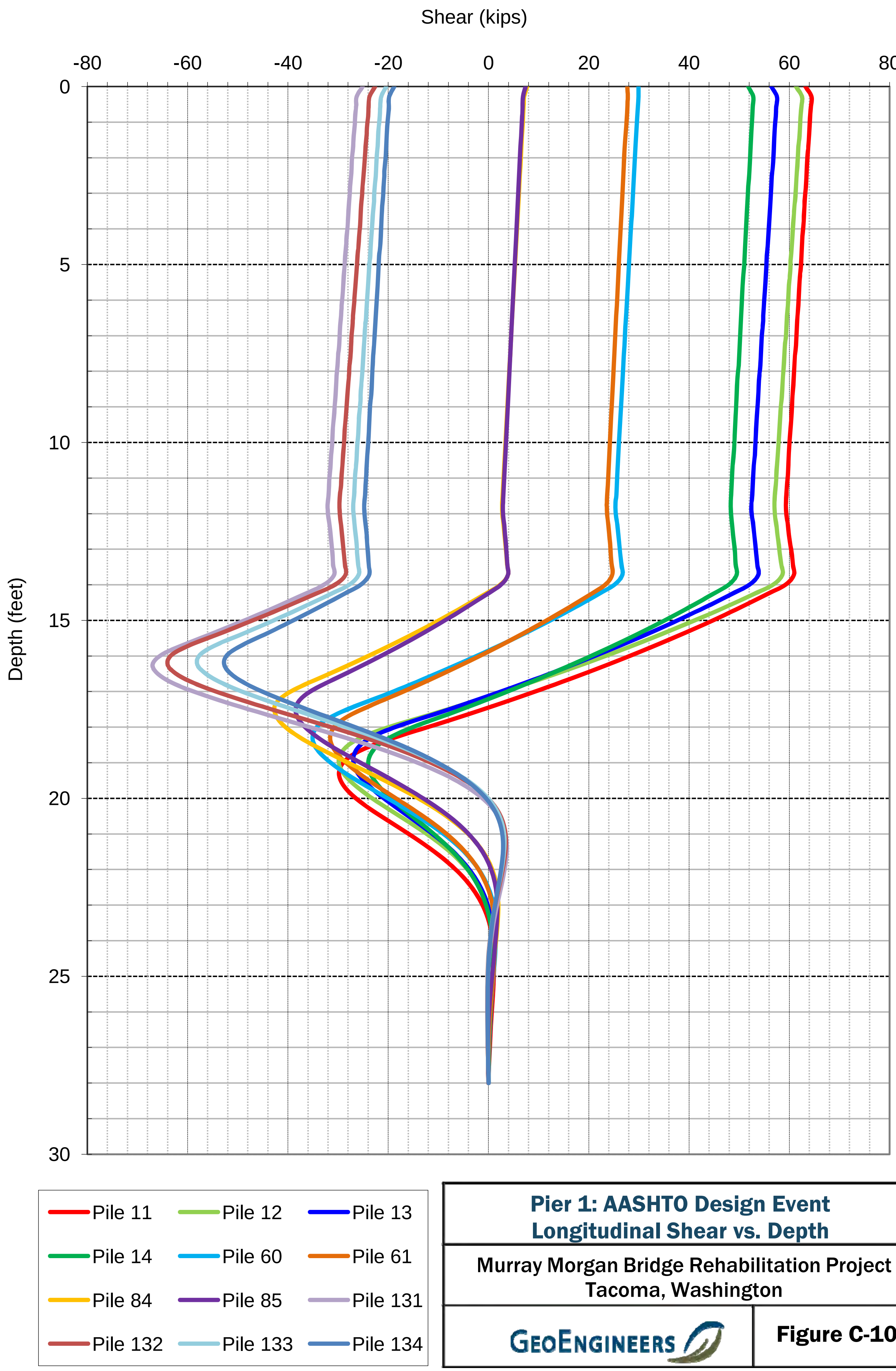
Figure C-9

057010002_FiguresC1_C16.xlsx WLT:khc 10/21/11



Pier 1: AASHTO Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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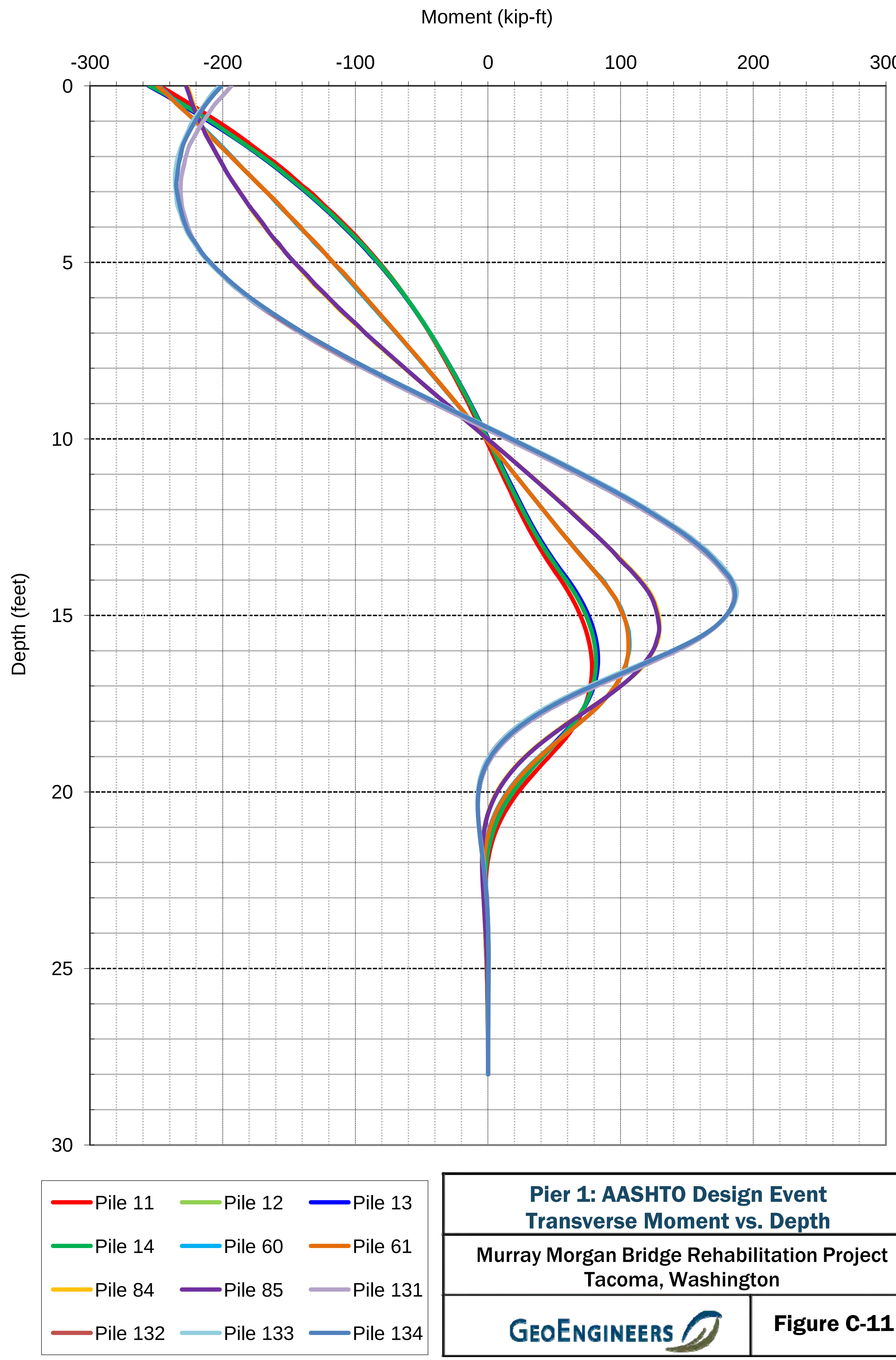
Figure C-10

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



Pier 1: AASHTO Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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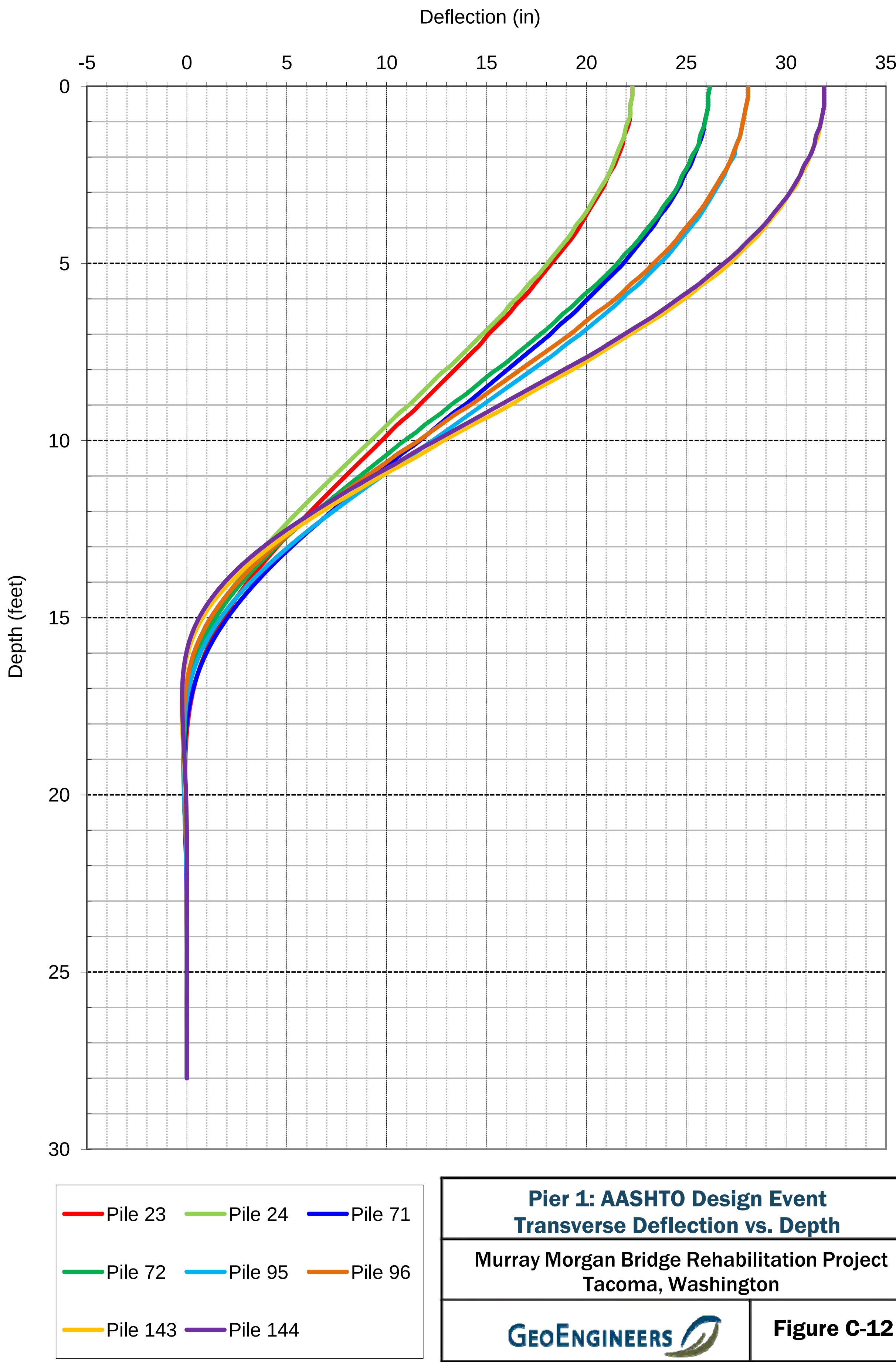
Figure C-11

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



Pier 1: AASHTO Design Event Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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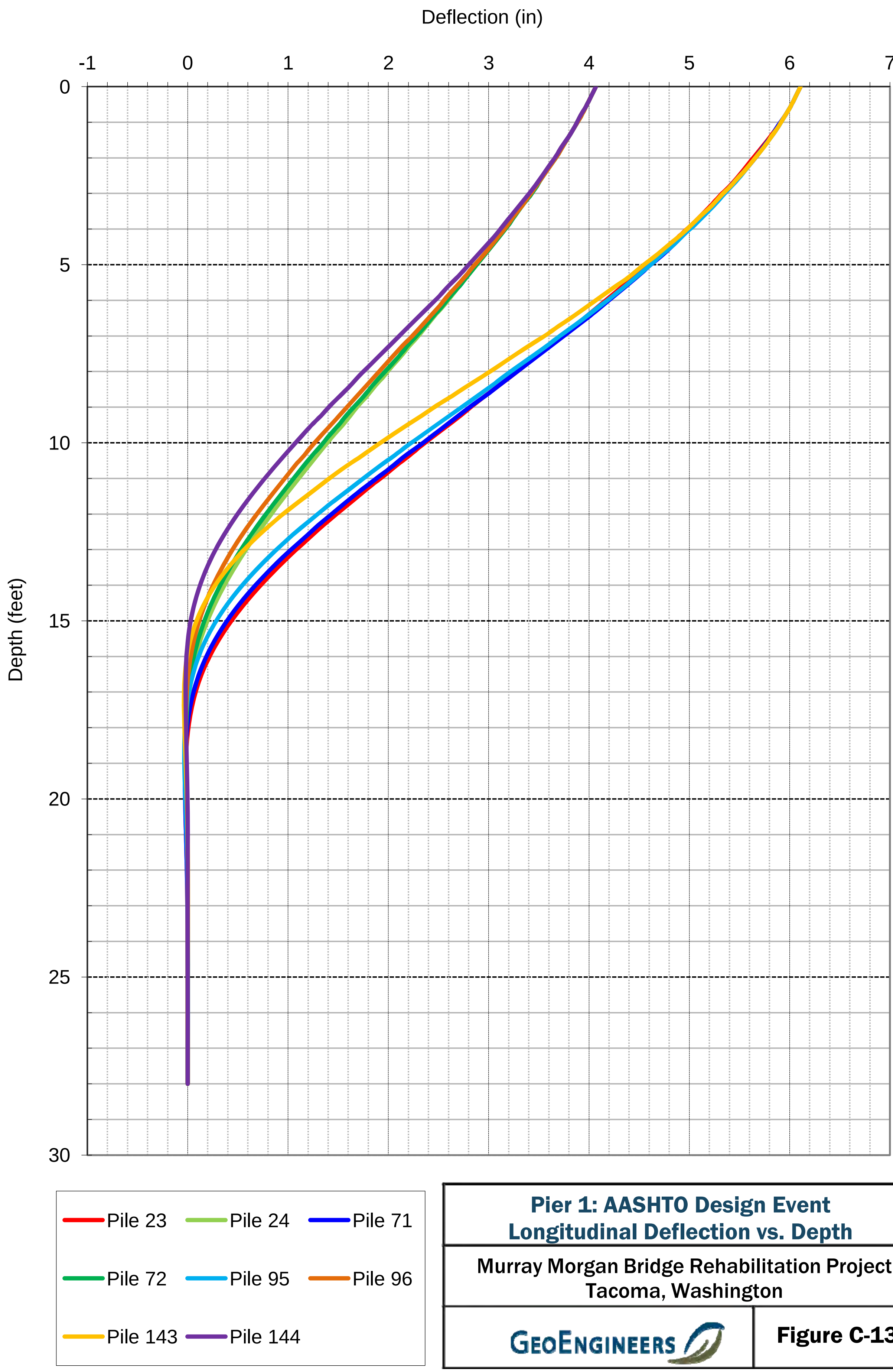
Figure C-12

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



057010002_Figures C1_C16.xlsx WLT:khc 10/21/11

Pier 1: AASHTO Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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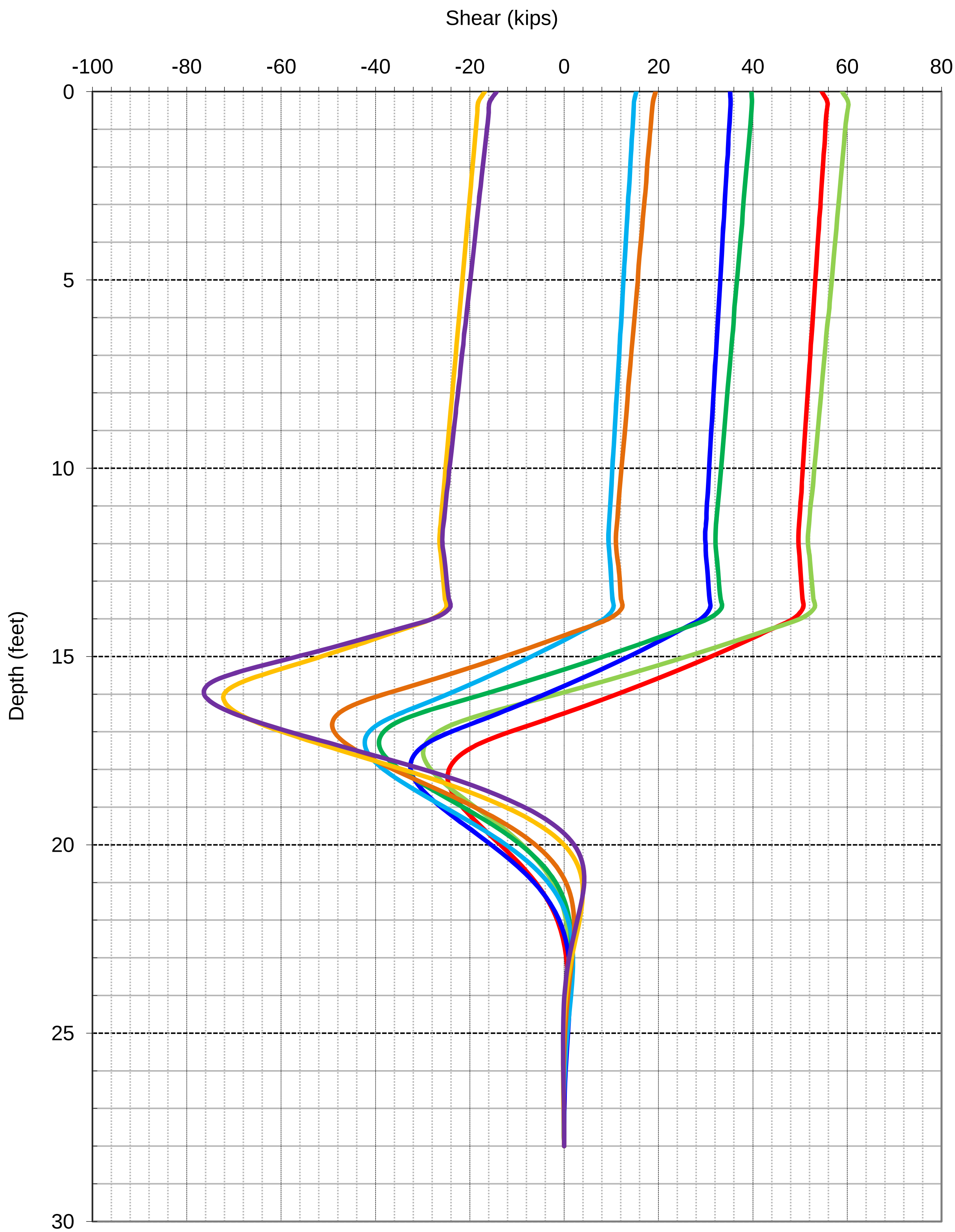
Figure C-13



0570100002_Figures C1_C16.xlsx WLT:khc 10/21/11

Pier 1: AASHTO Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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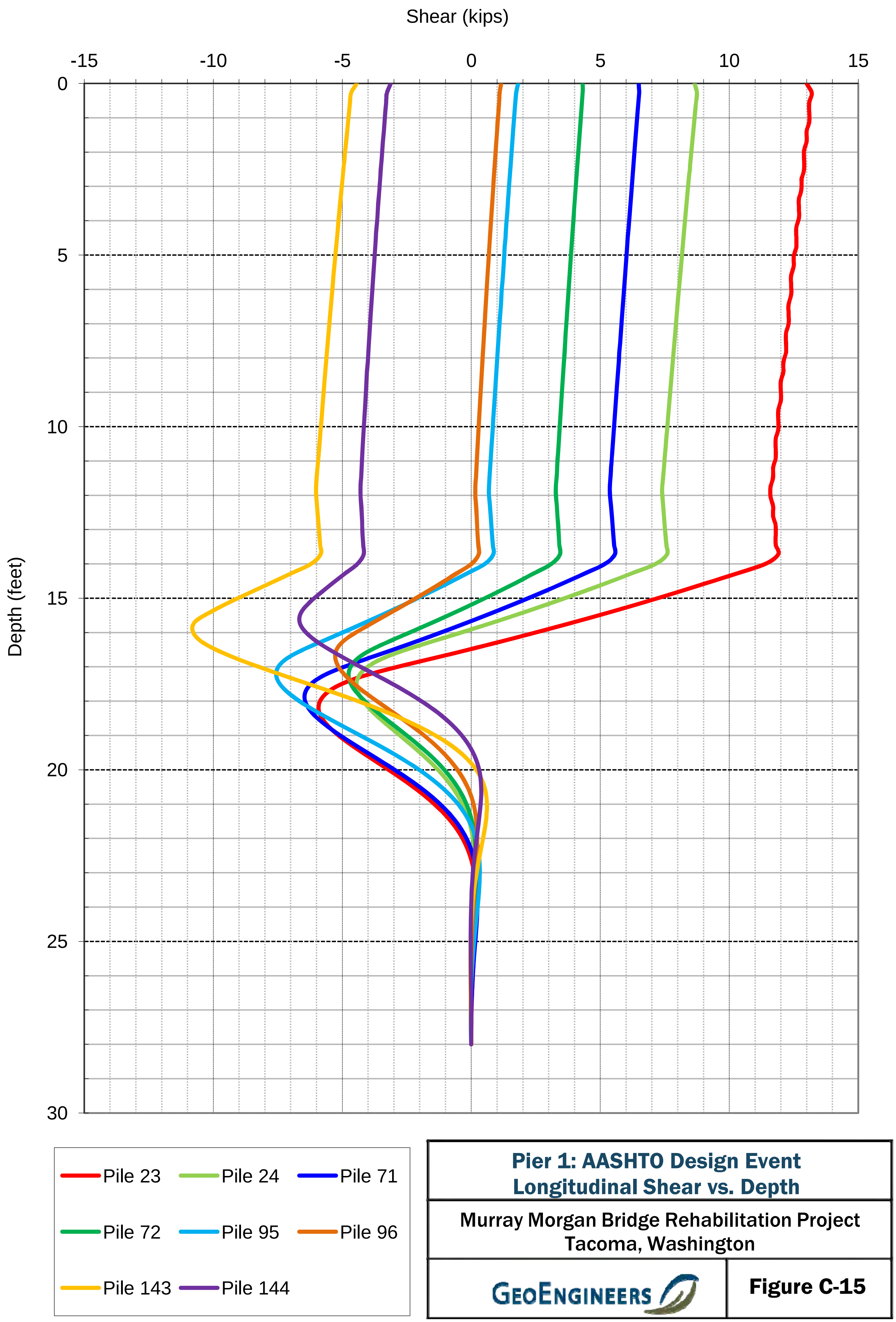
Figure C-14



Pier 1: AASHTO Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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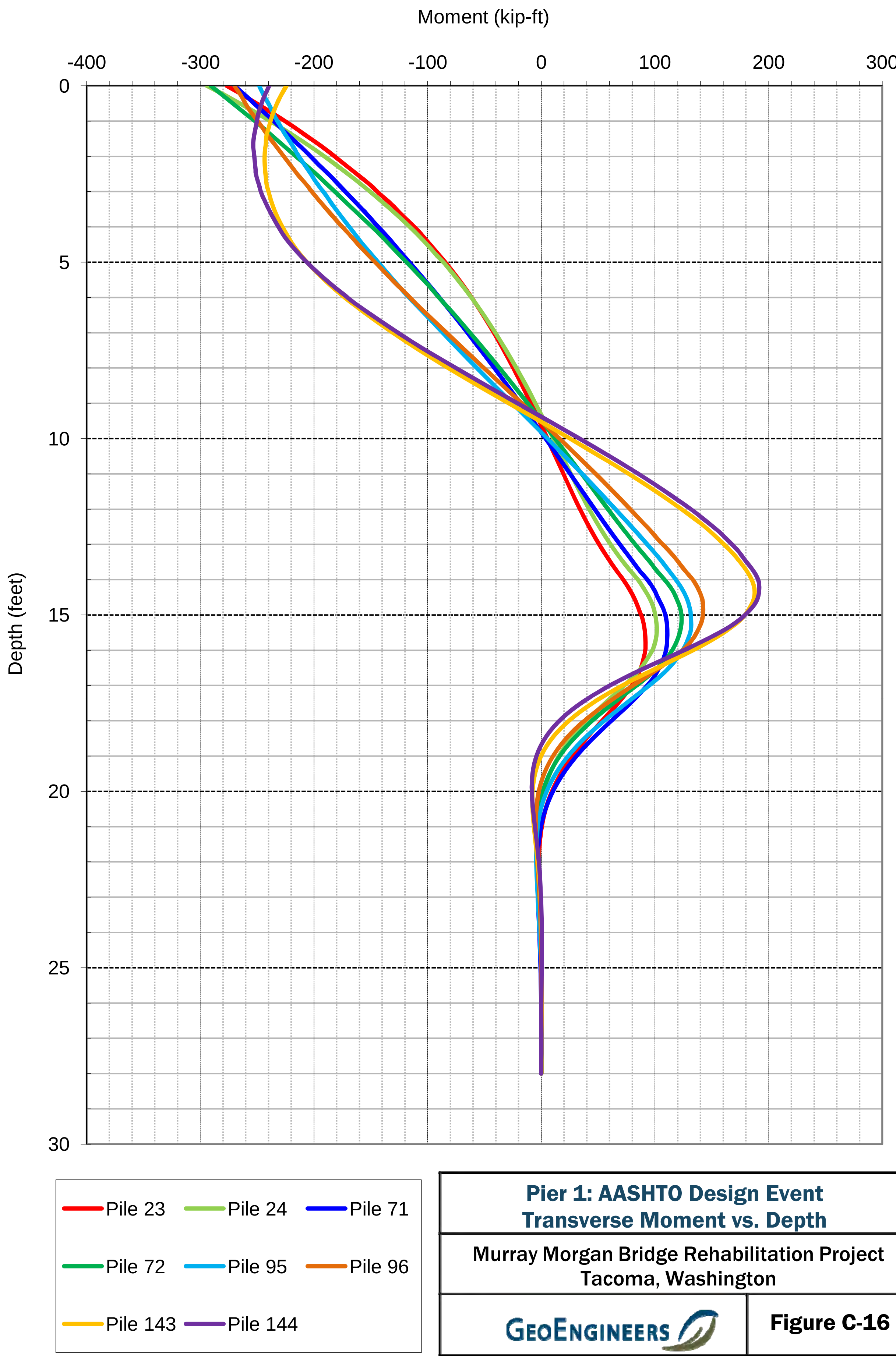
Figure C-15

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



Pier 1: AASHTO Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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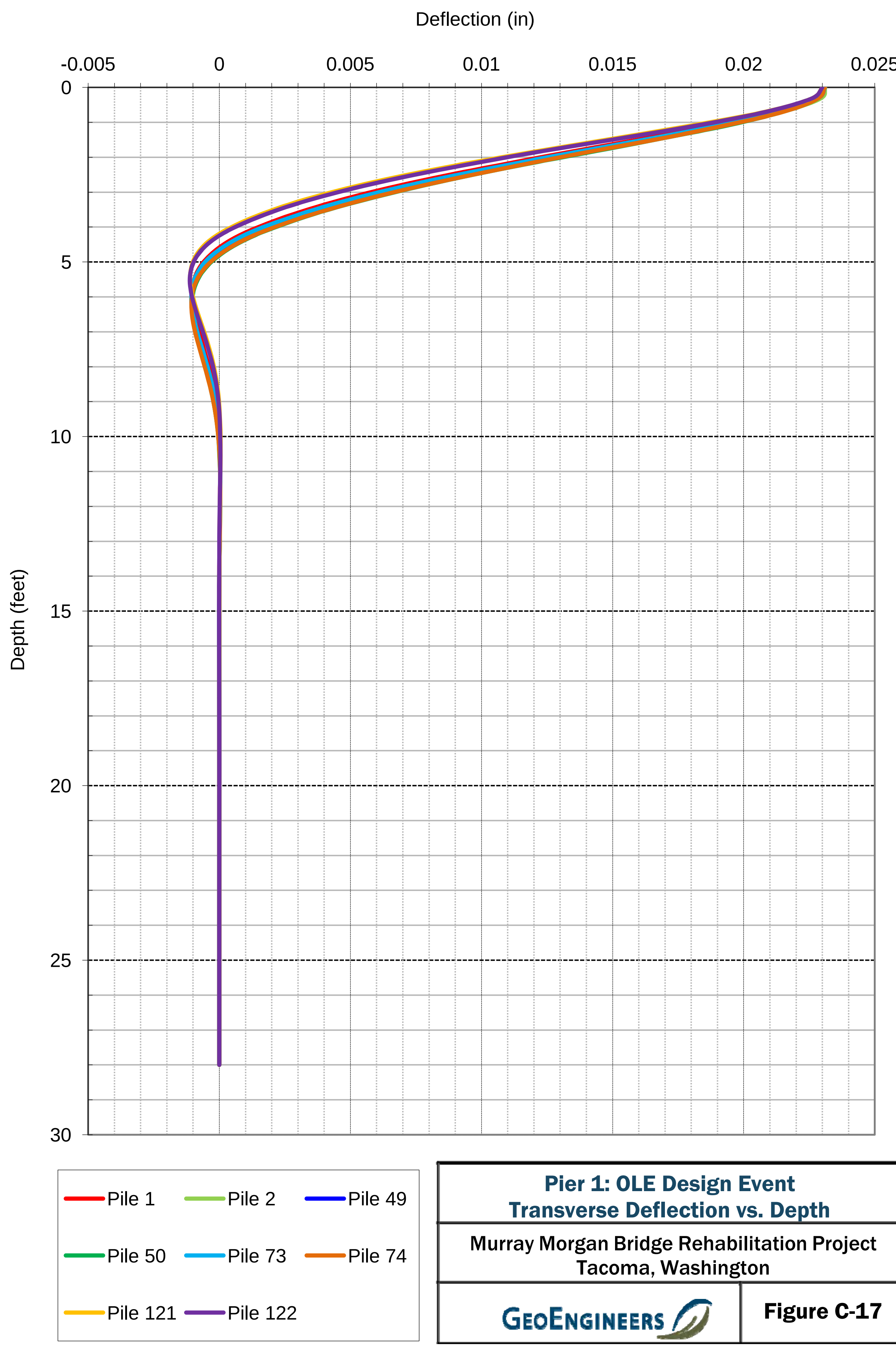
Figure C-16

057010002_Figures C1_C16.xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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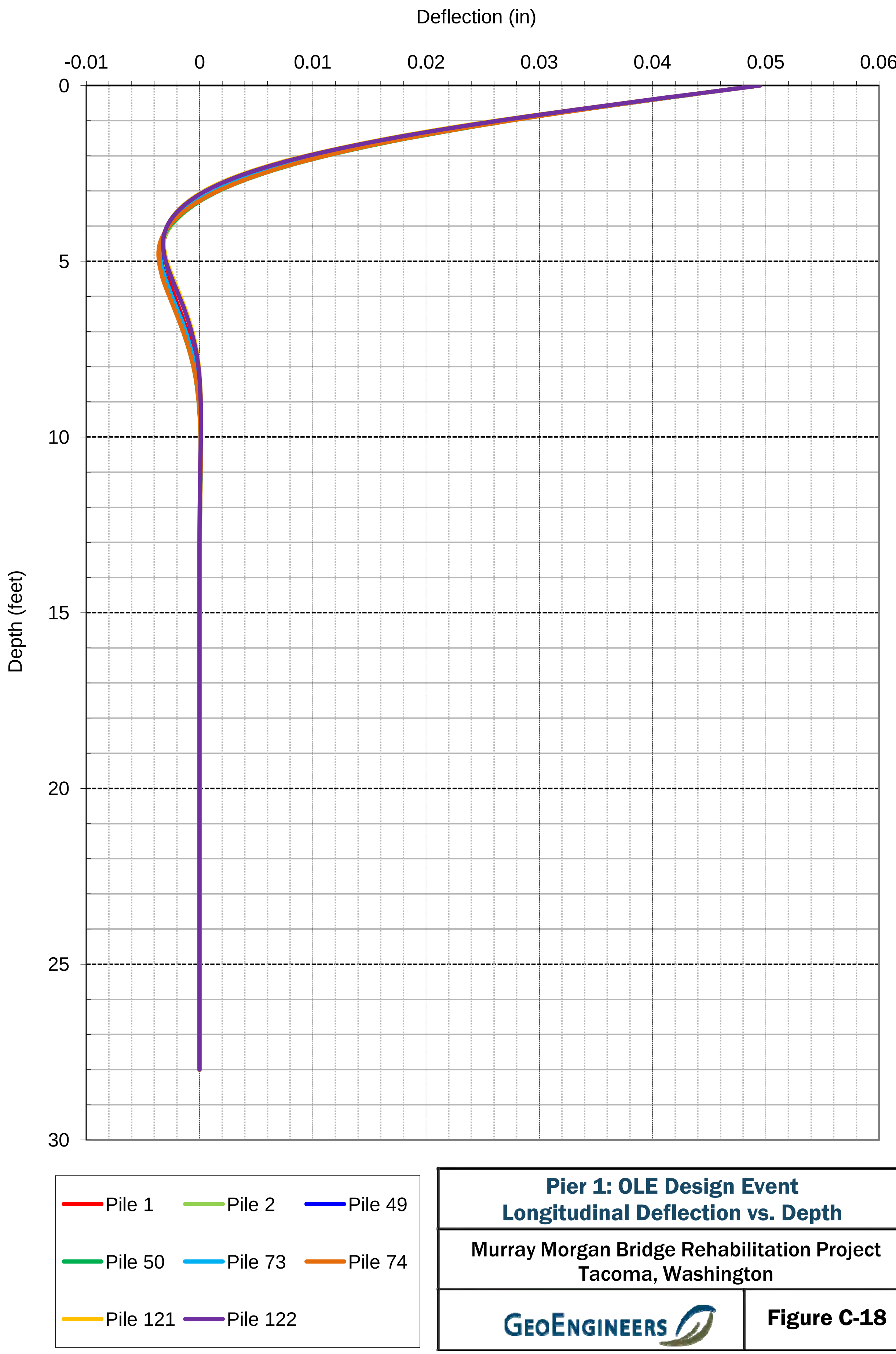
Figure C-17

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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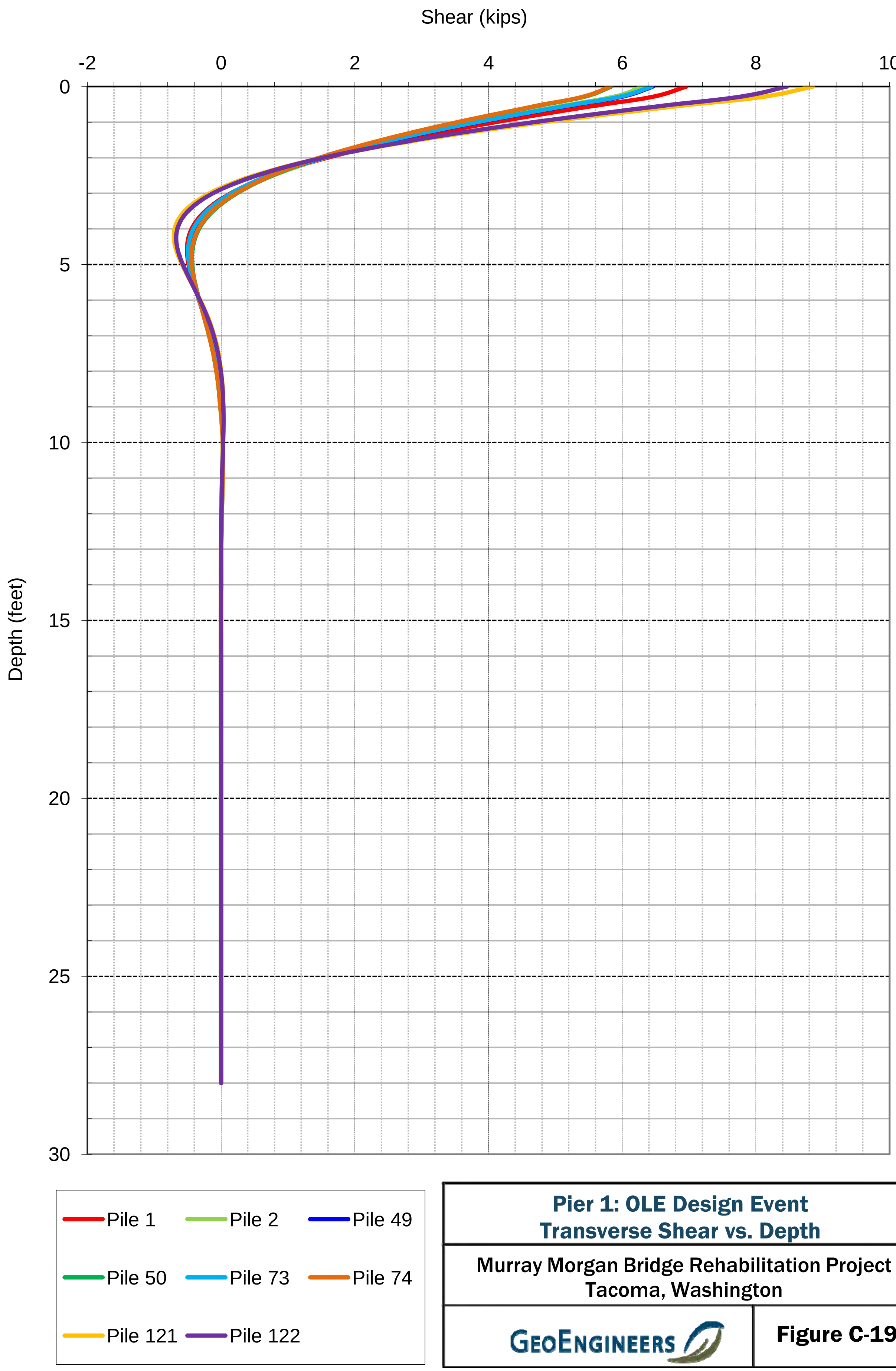
Figure C-18

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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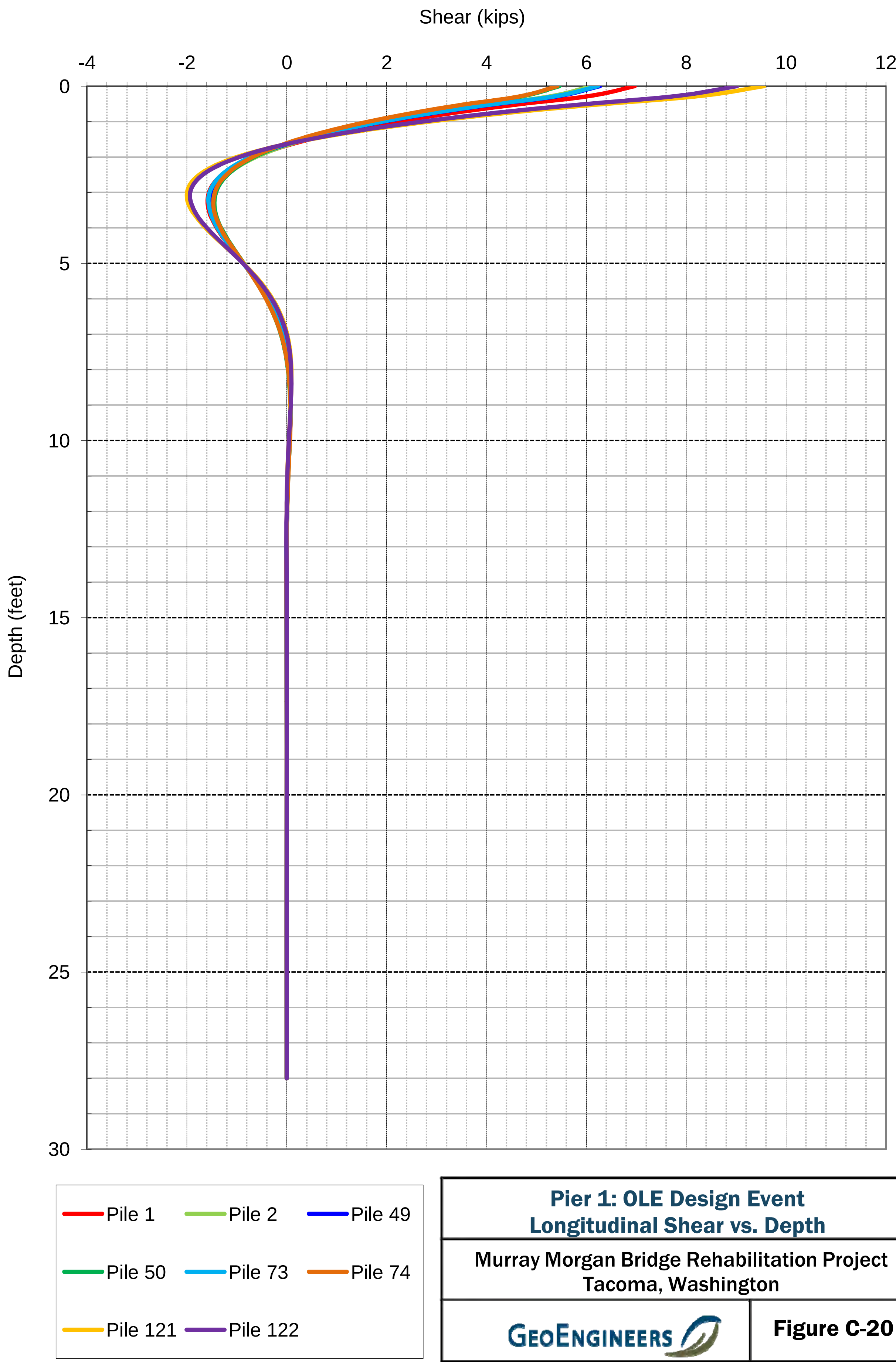
Figure C-19

0570100002_Figures C17_C34..xlsx WLT:khc 10/21/11



057010002_Figures C17_C34..xlsx WLT:khc 10/21/11

Pier 1: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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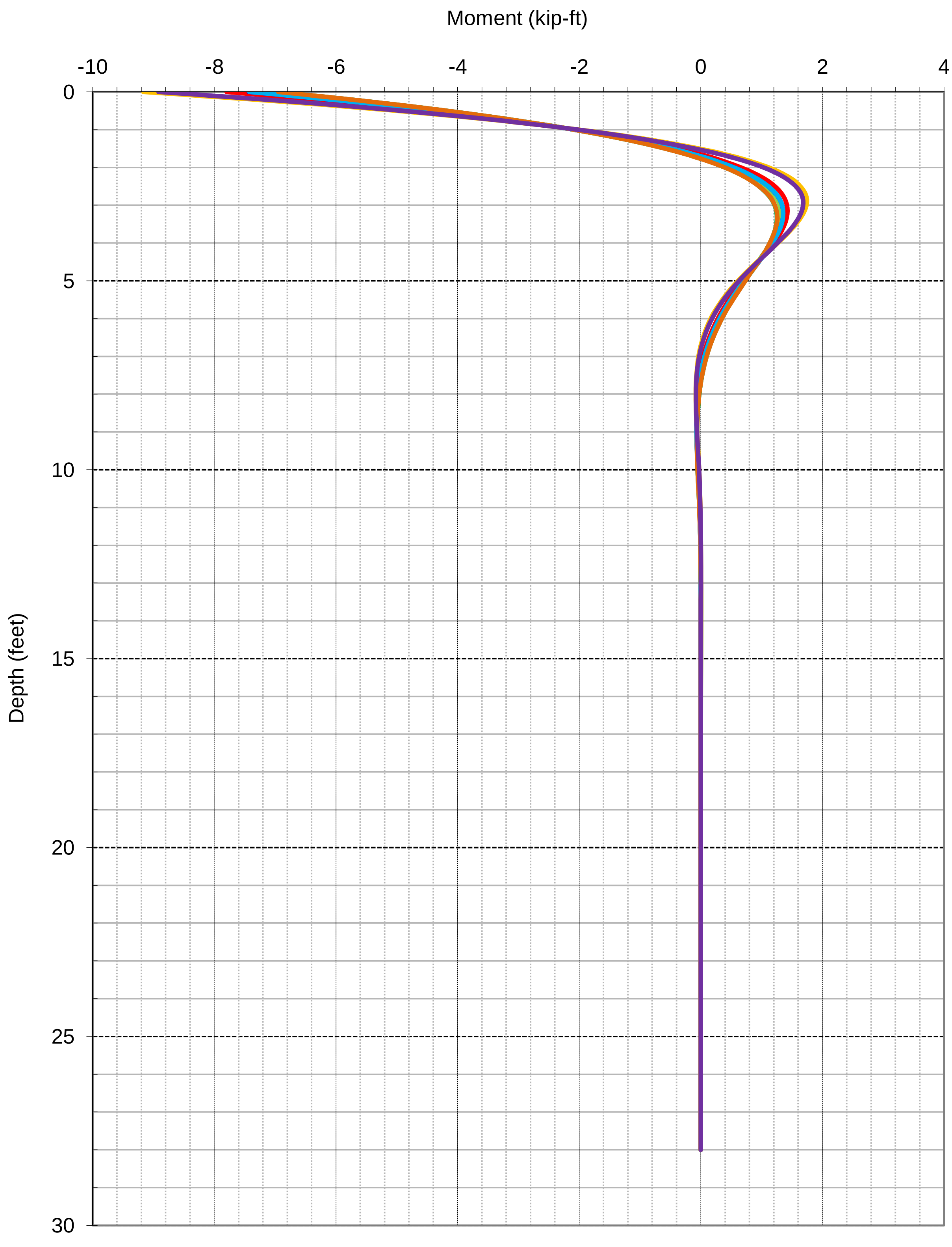
Figure C-20



057010002_Figures C17_C34..xlsx WLT:khc 10/21/11

Pier 1: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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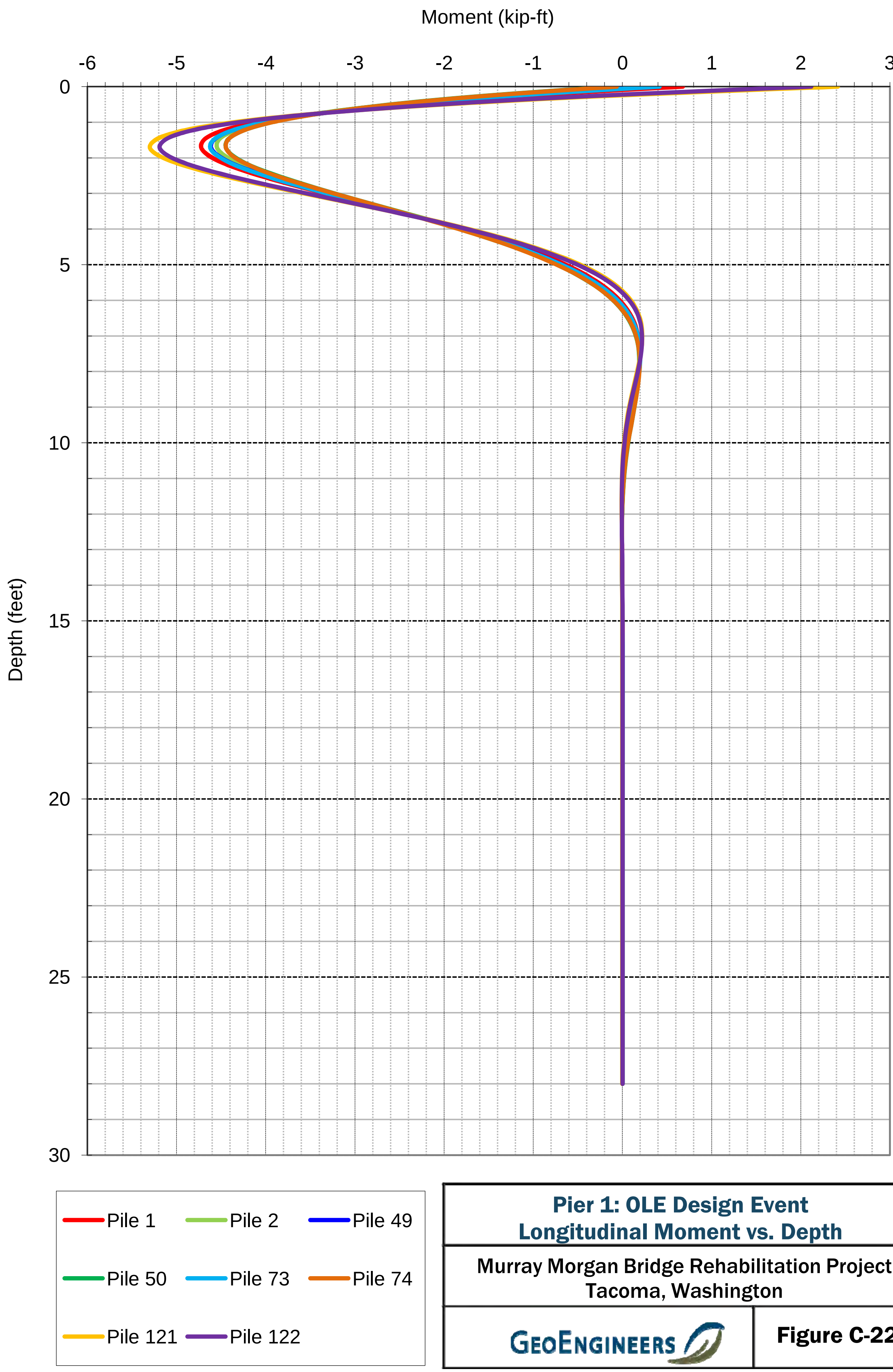
Figure C-21



Pier 1: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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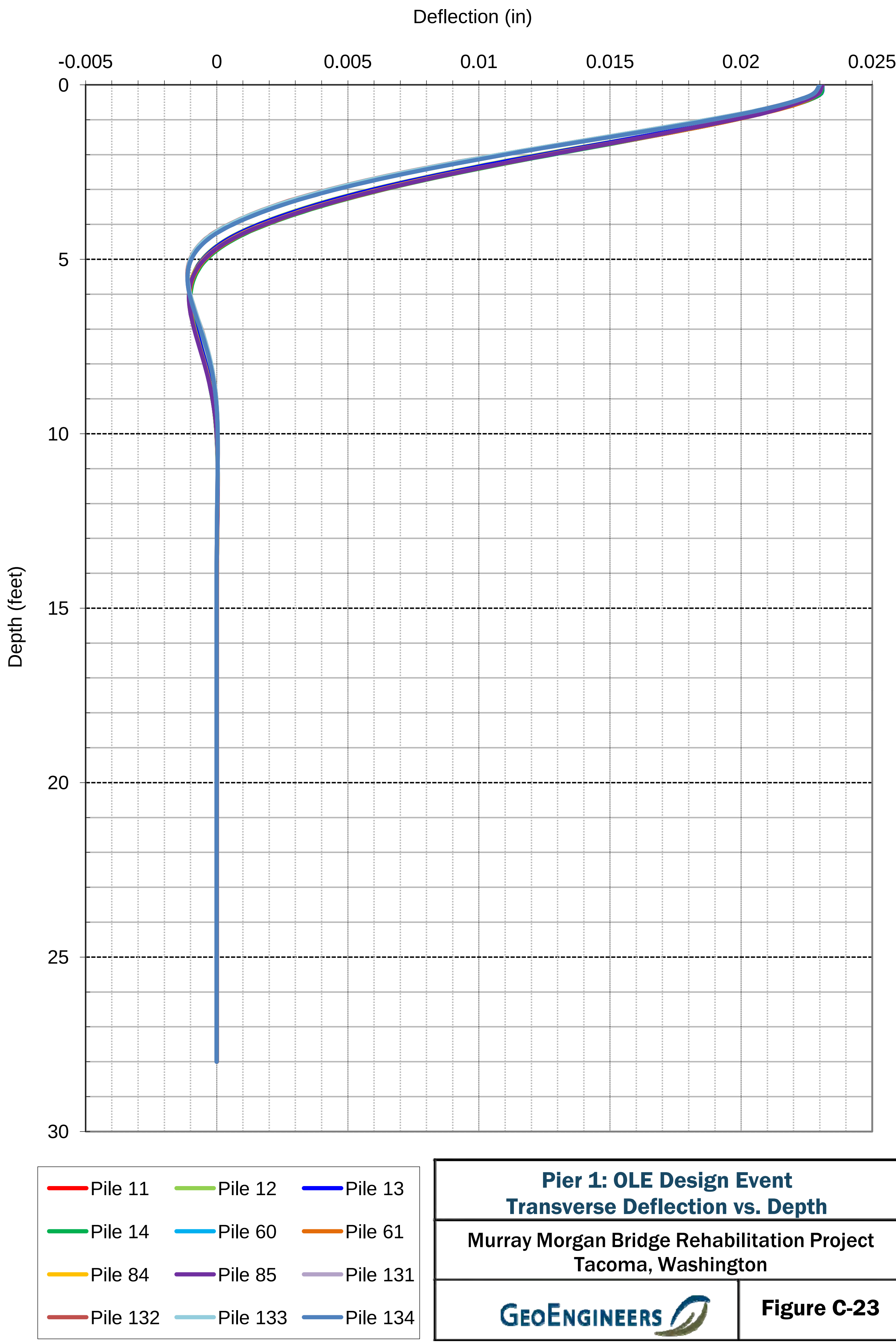
Figure C-22

057010002_FiguresC17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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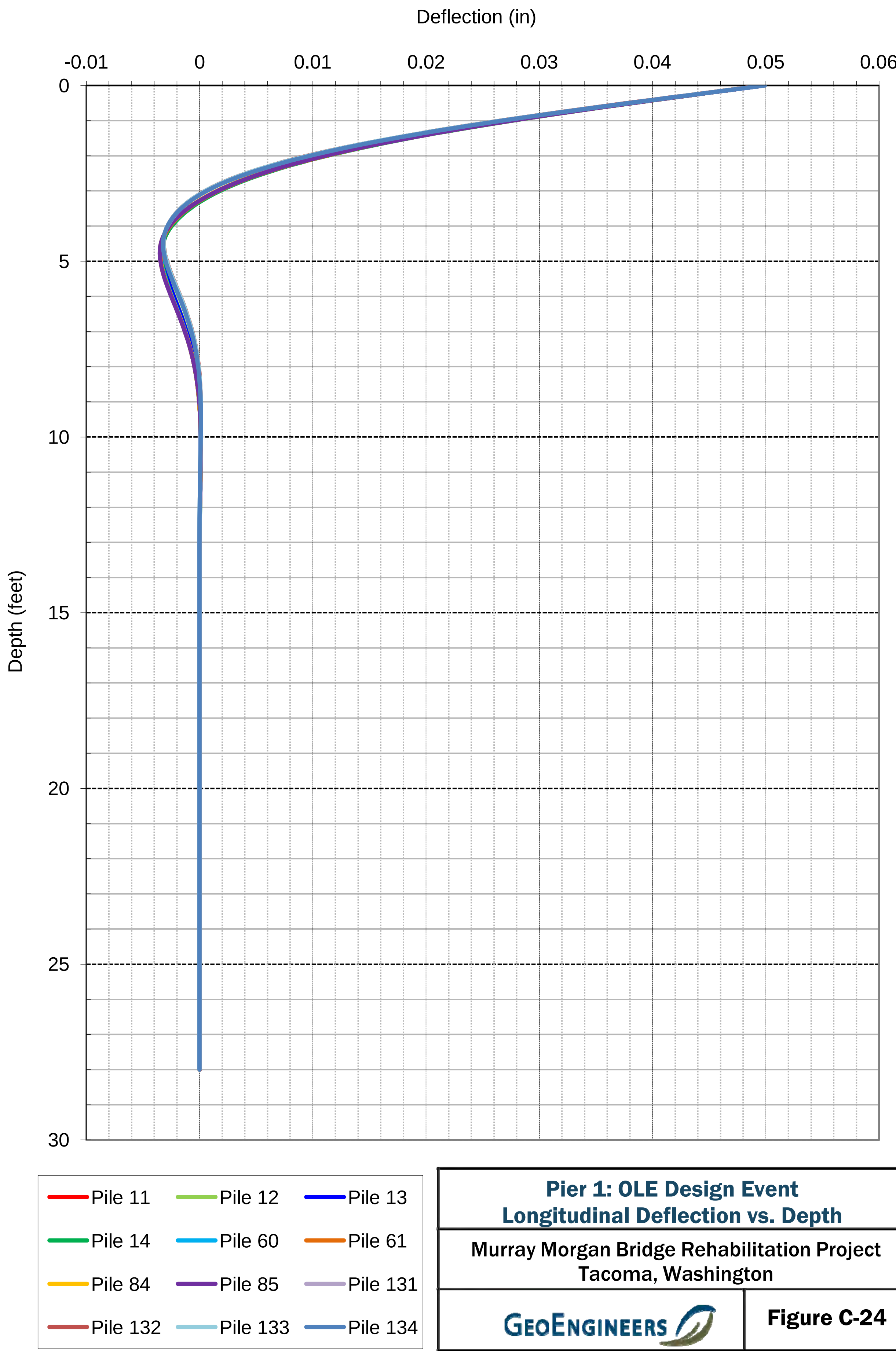
Figure C-23

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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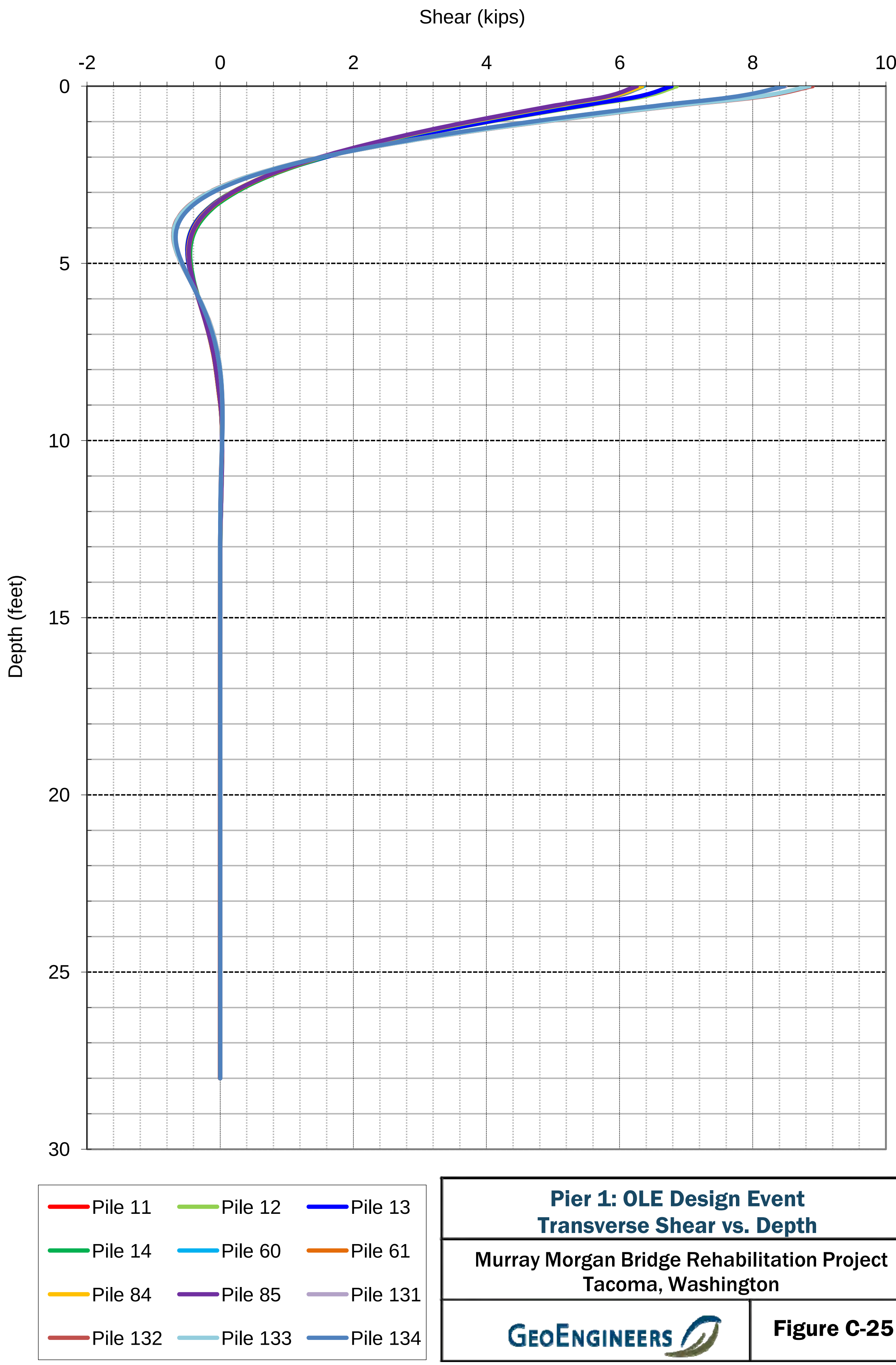
Figure C-24

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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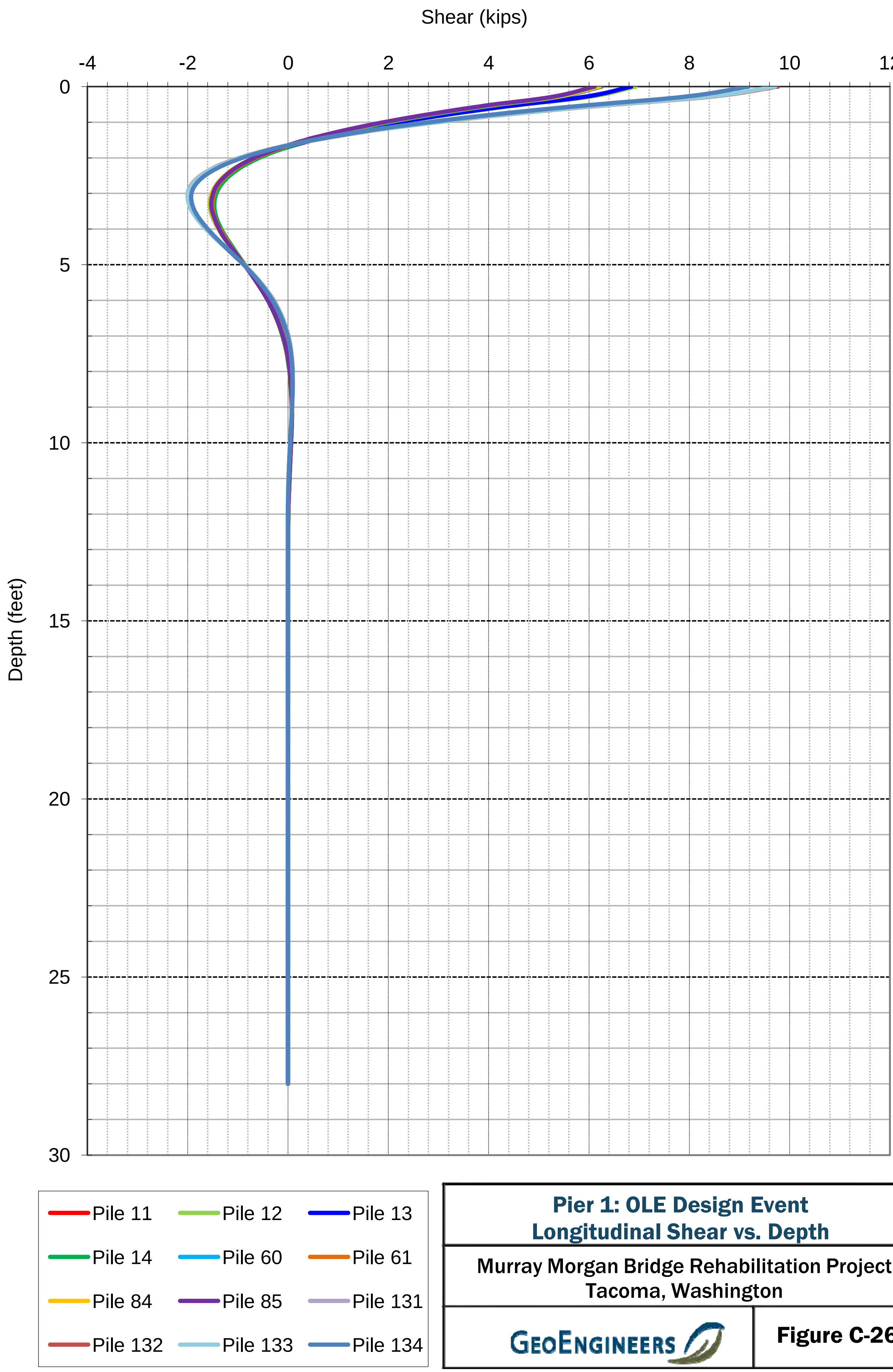
Figure C-25

057010002_Figures C17_C34.xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
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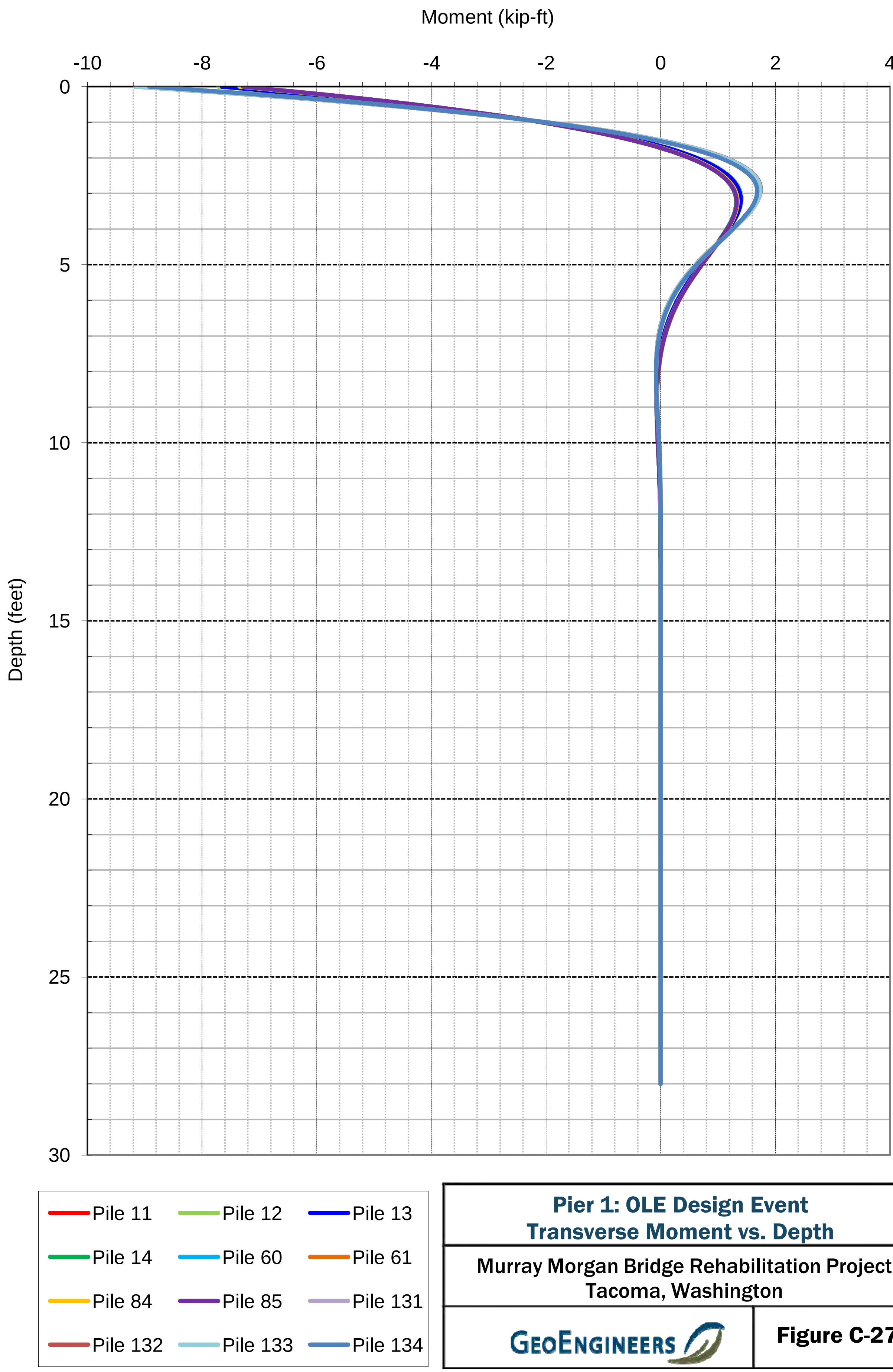
Figure C-26

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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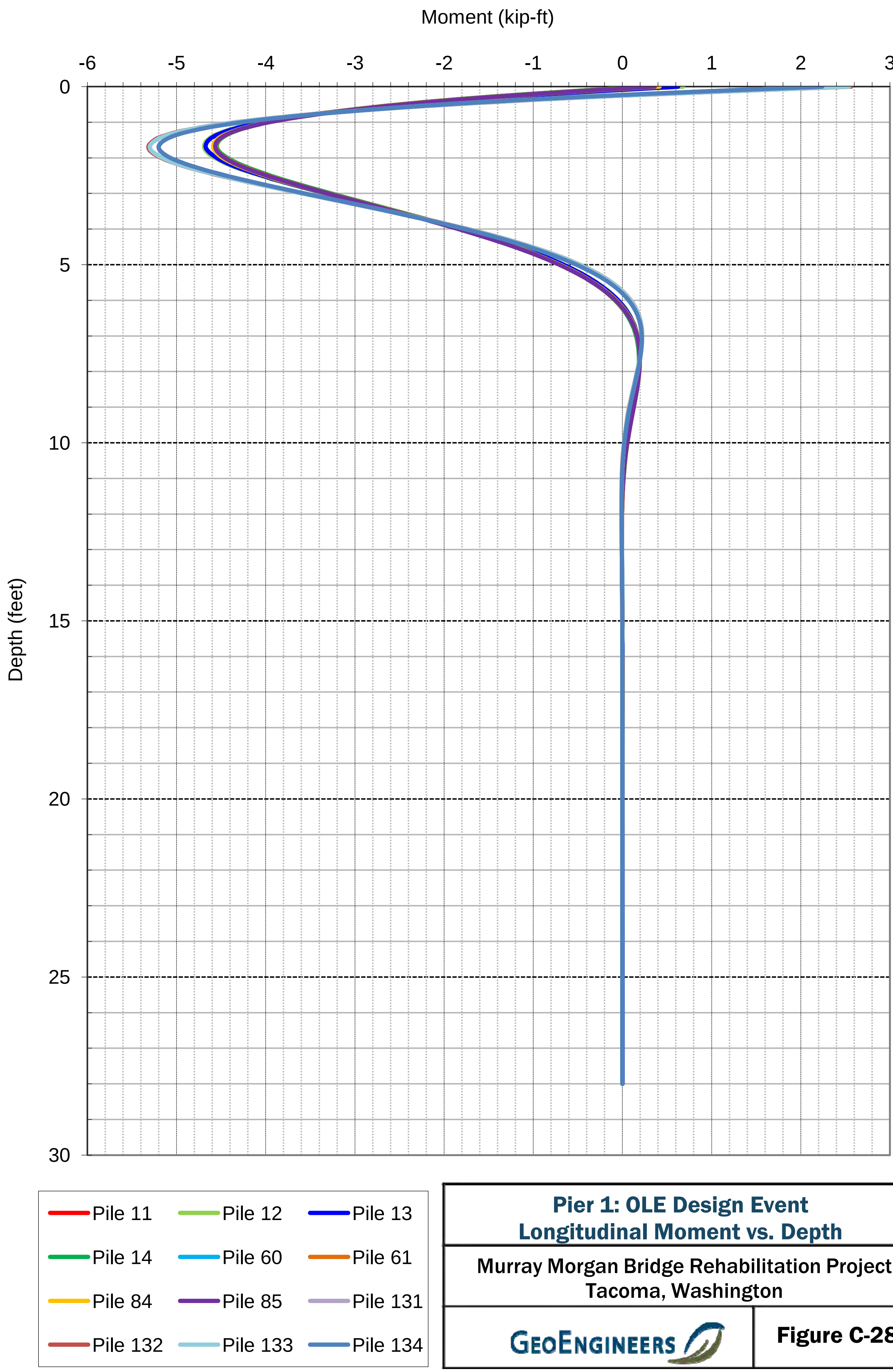
Figure C-27

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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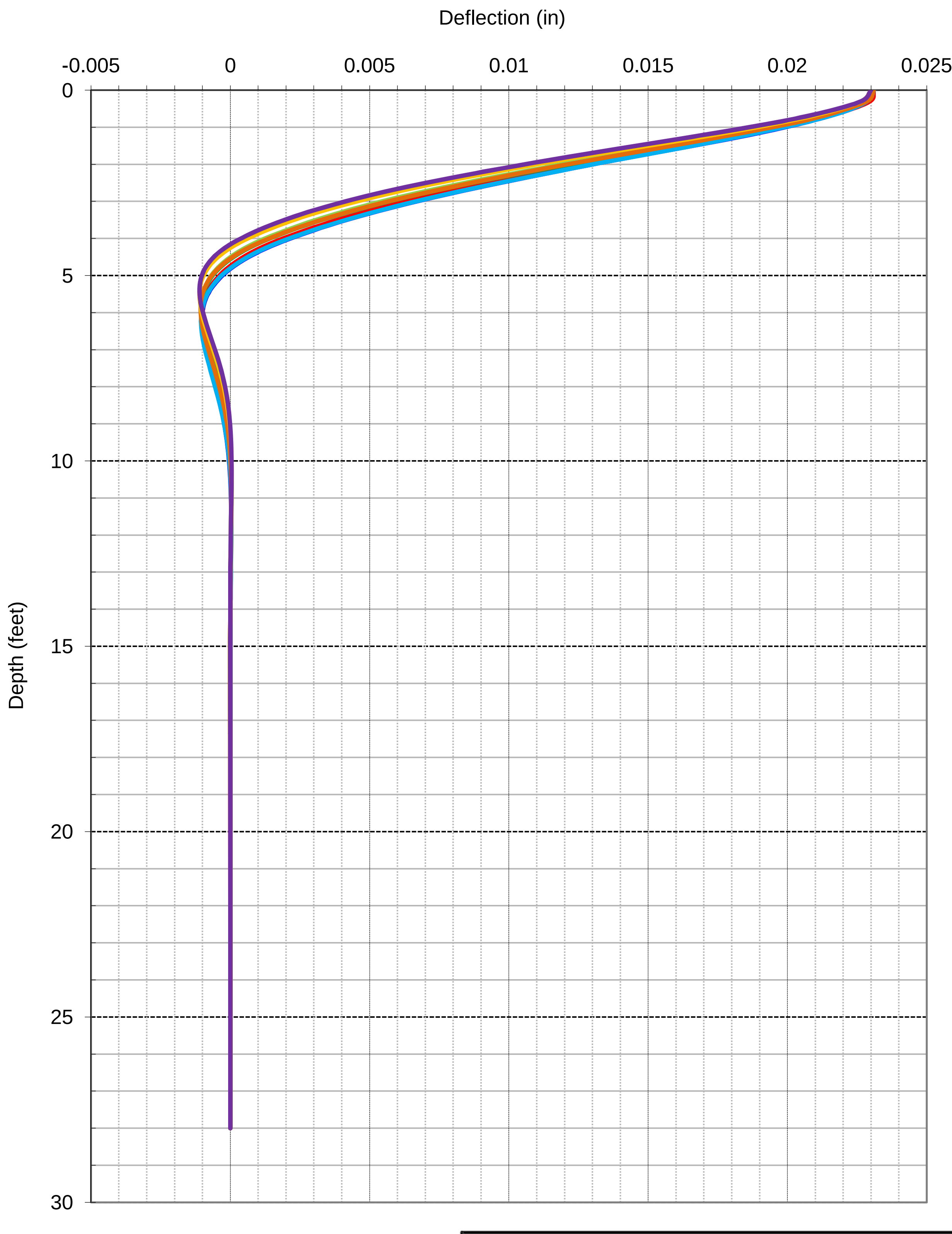
Figure C-28

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
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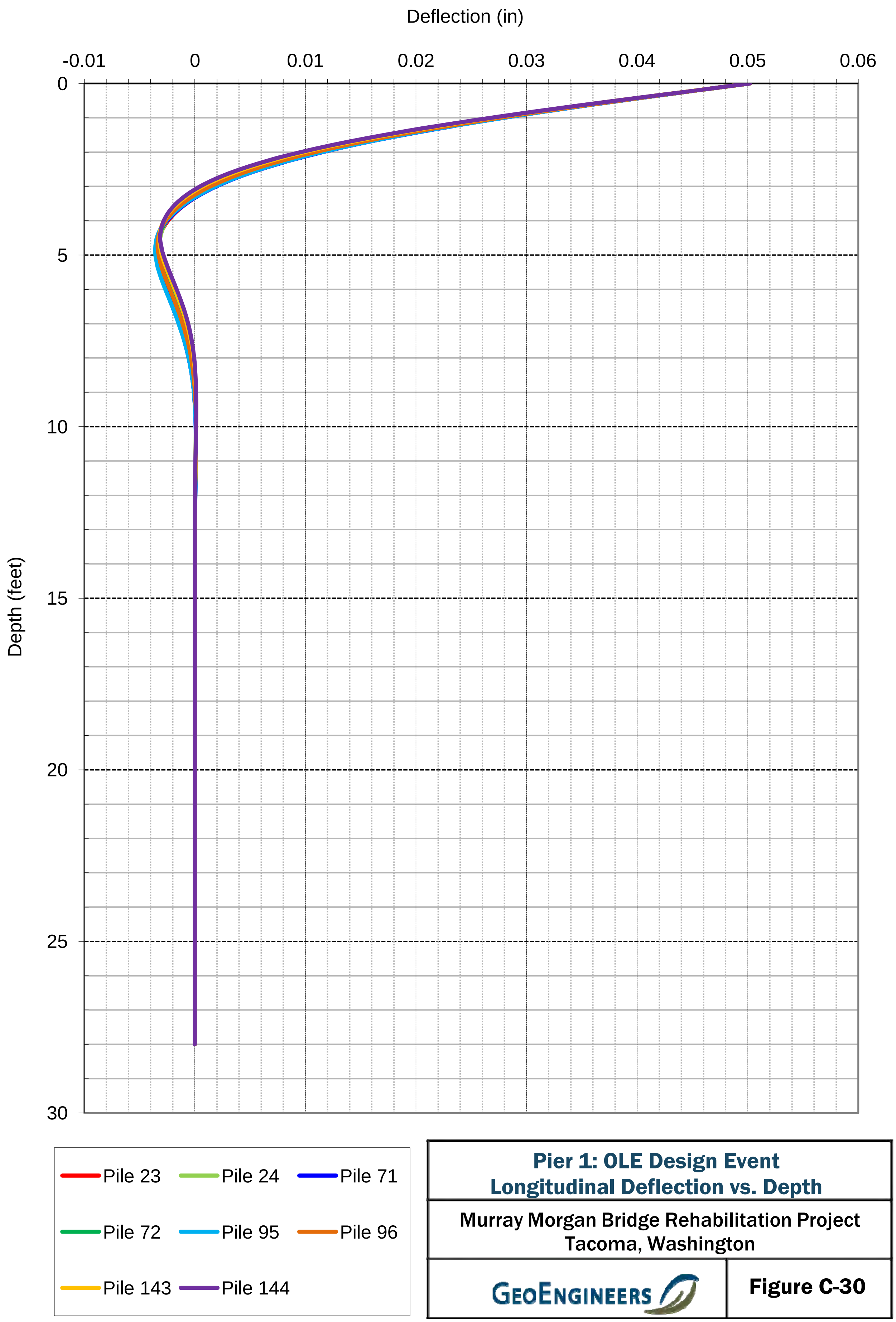
Figure C-29

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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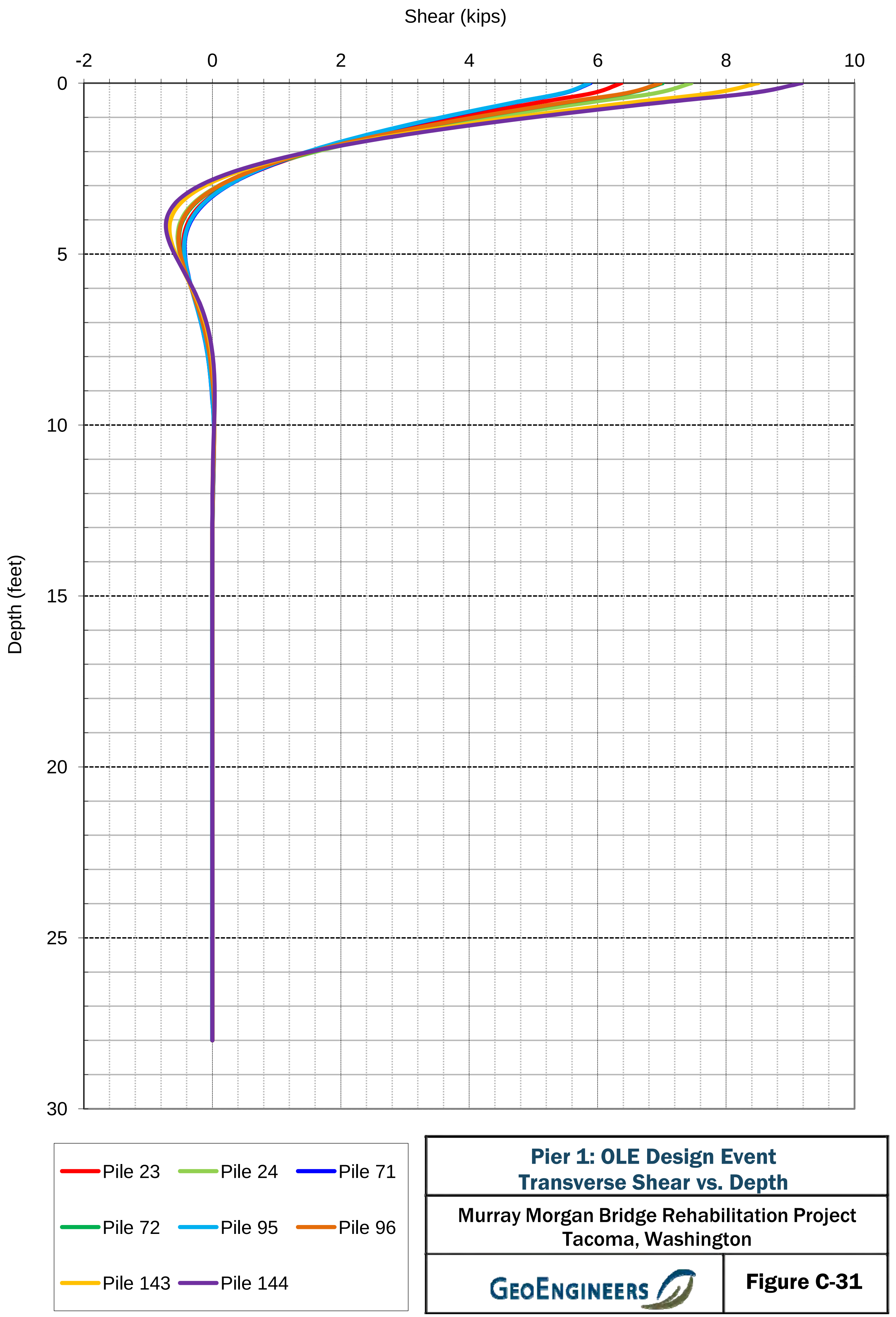
Figure C-30

057010002_FiguresC17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
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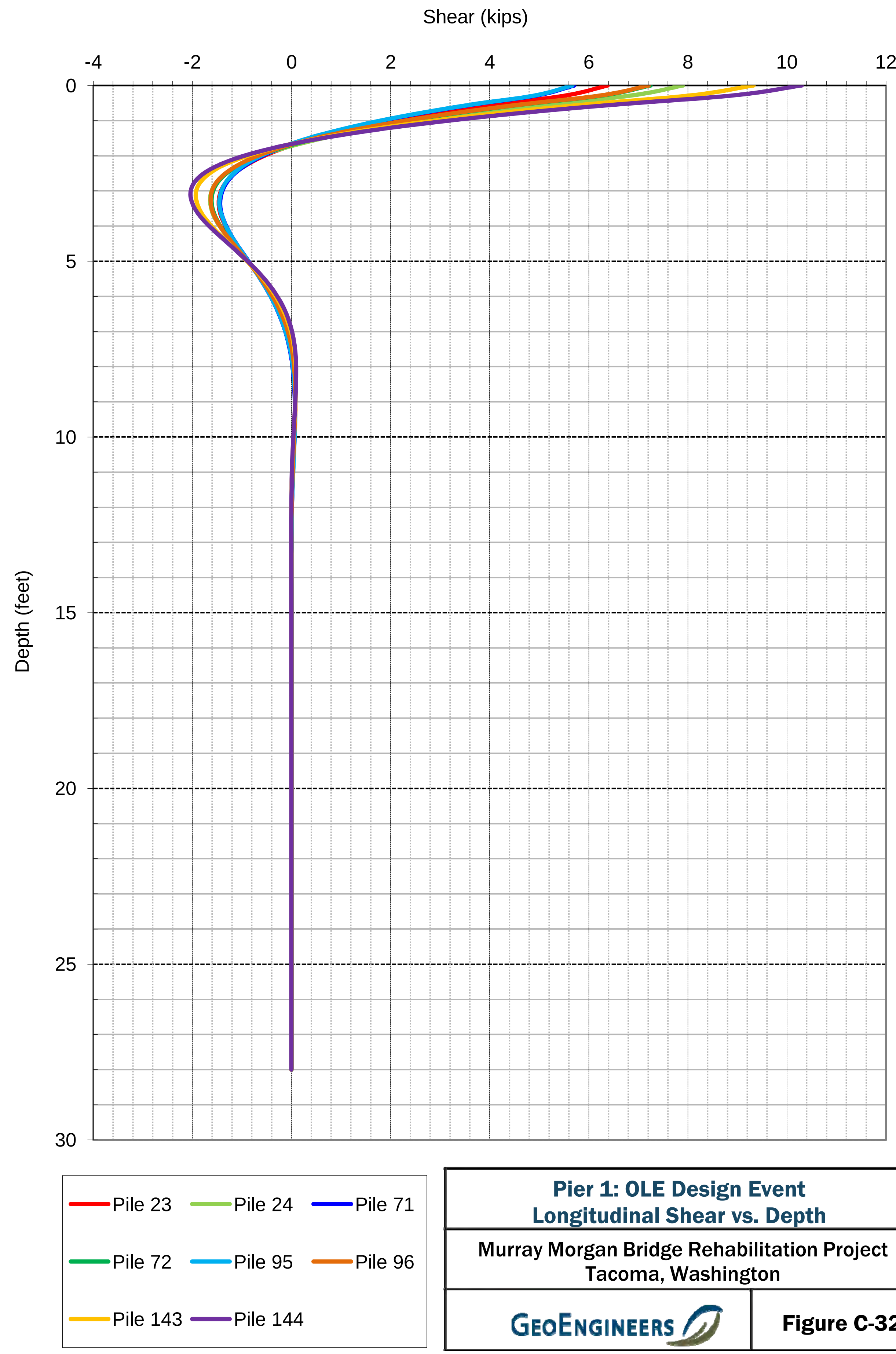
Figure C-31

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
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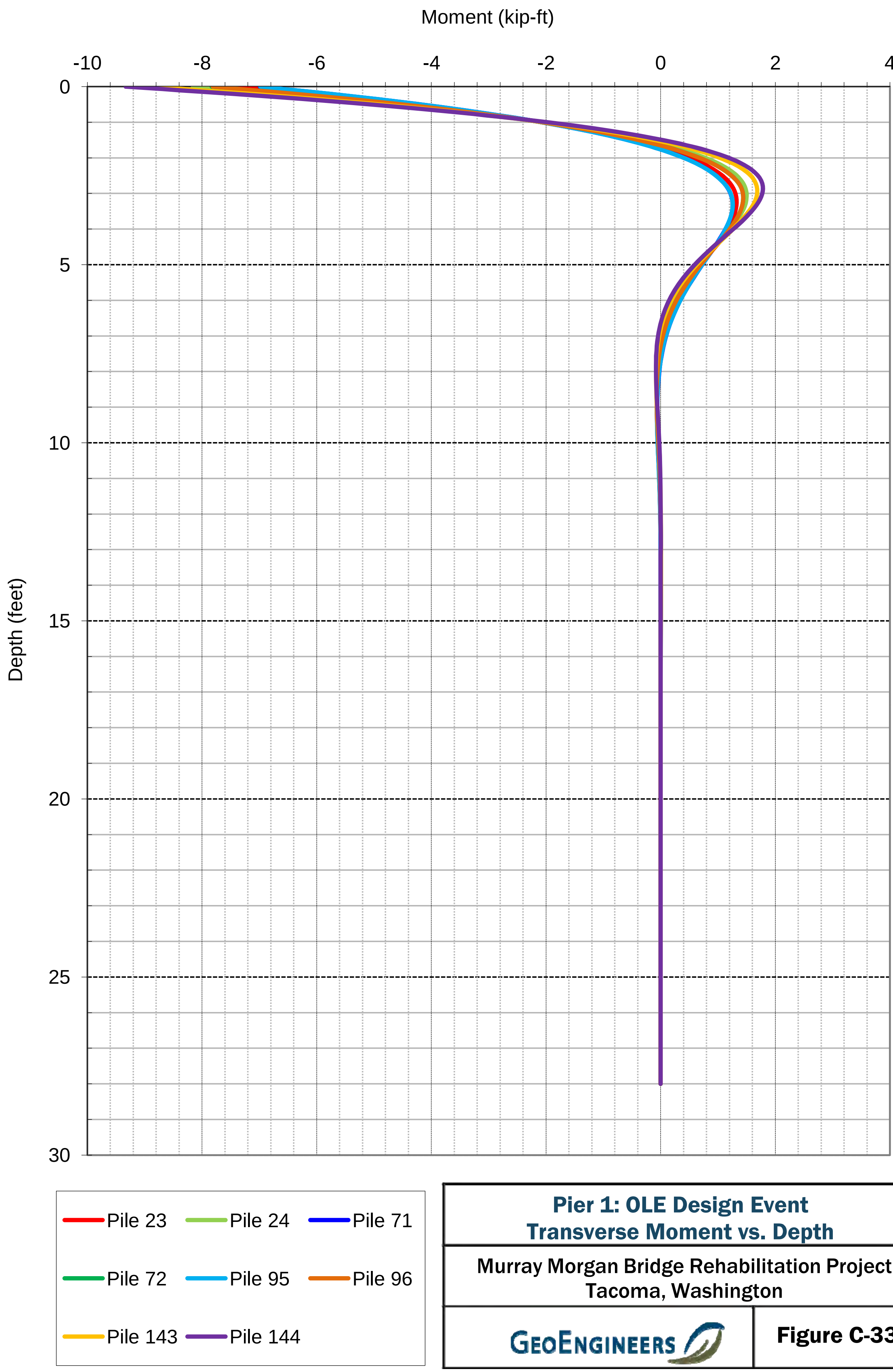
Figure C-32

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



Pier 1: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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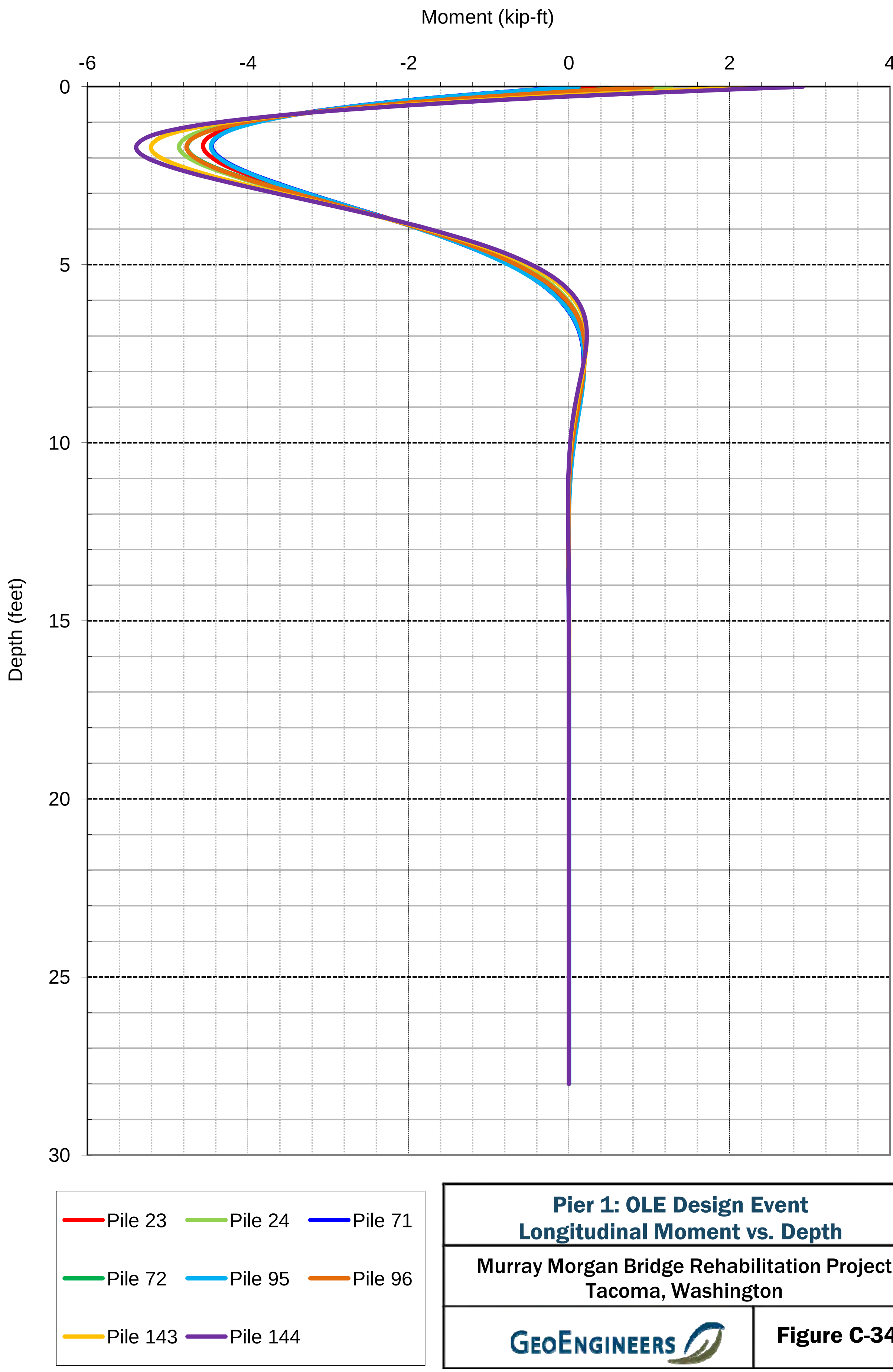
Figure C-33

057010002_Figures C17_C34..xlsx WLT:khc 10/21/11



057010002_Figures C17_C34..xlsx WLT:khc 10/21/11

Pier 1: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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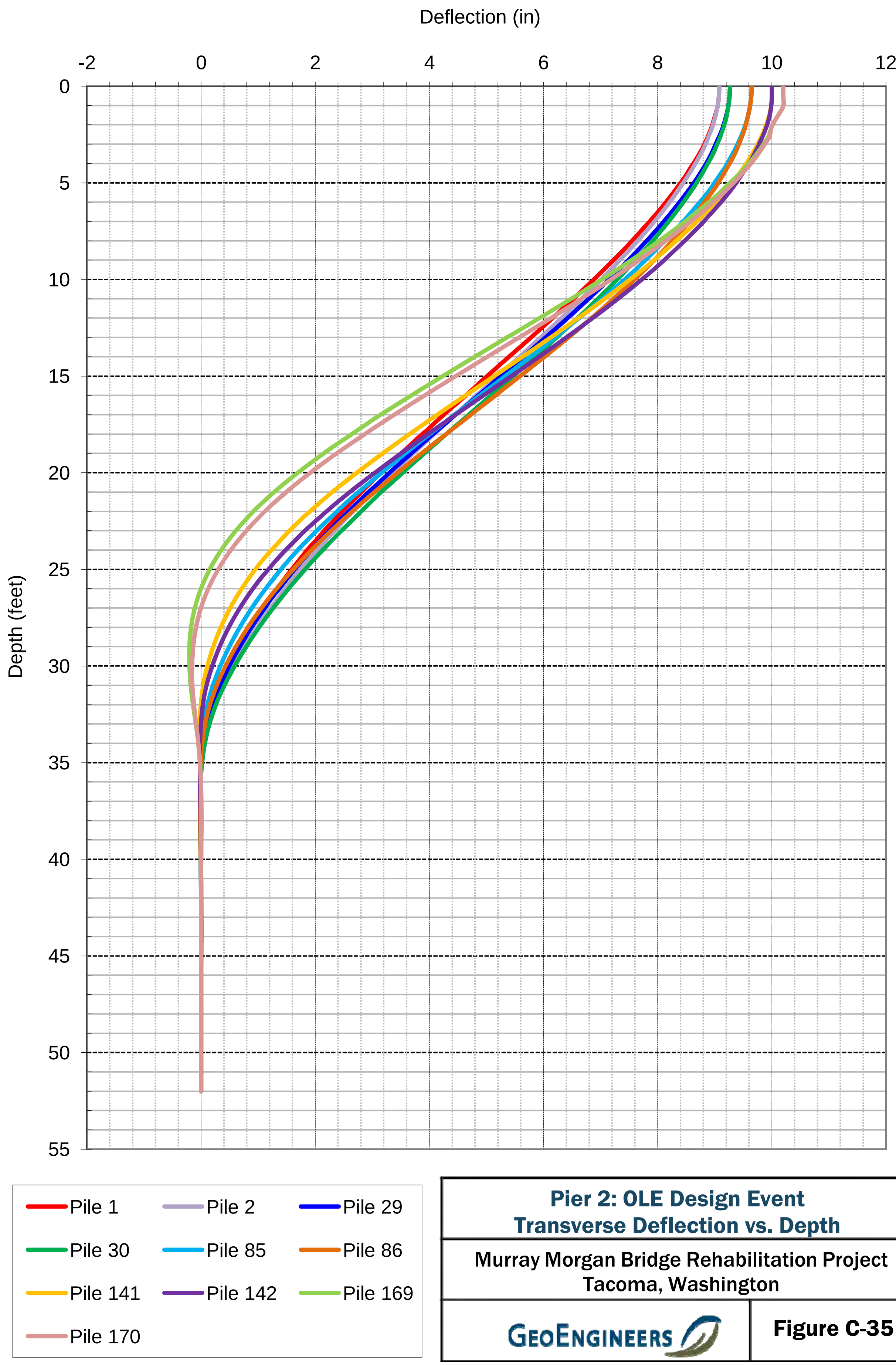
Figure C-34



Pier 2: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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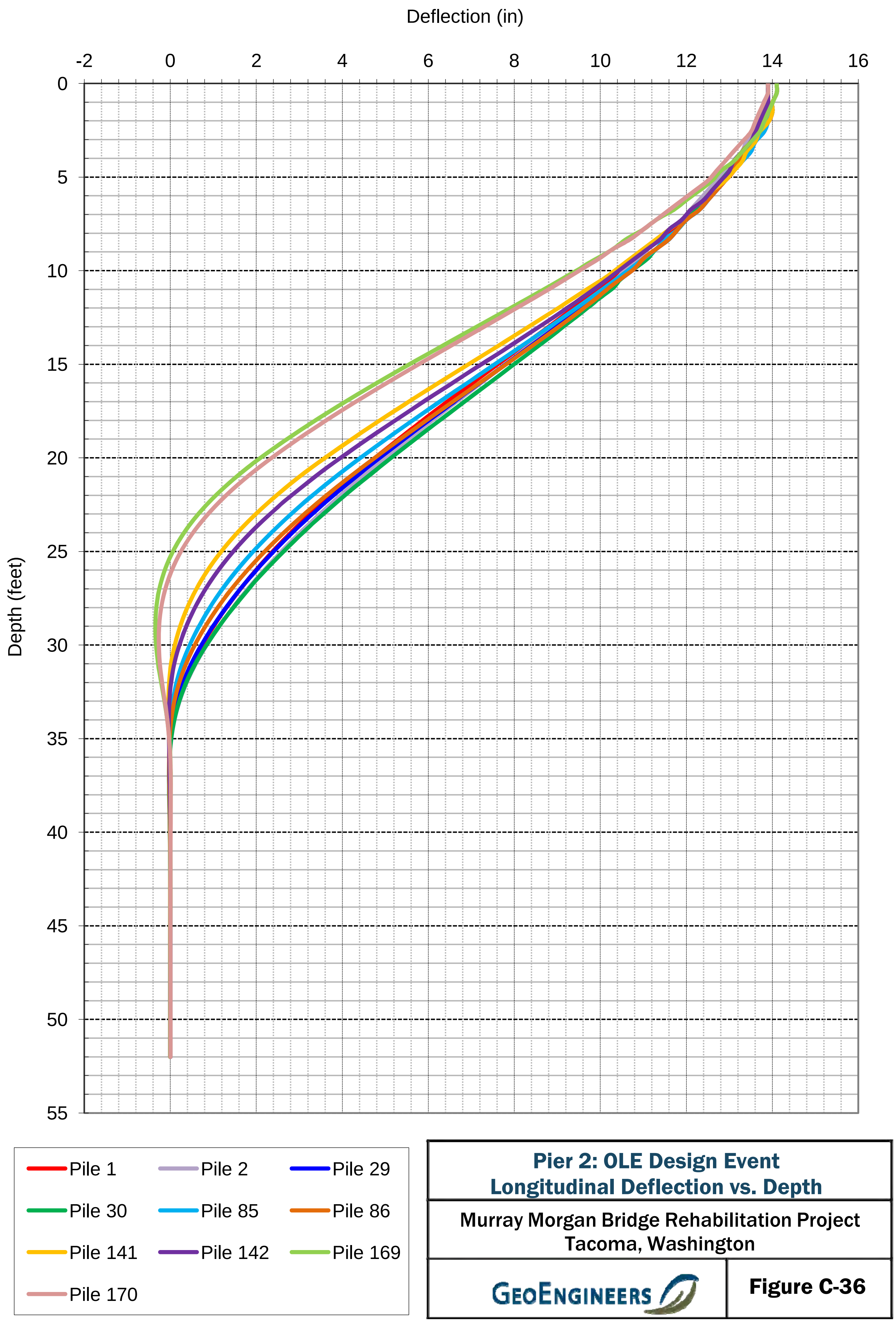
Figure C-35

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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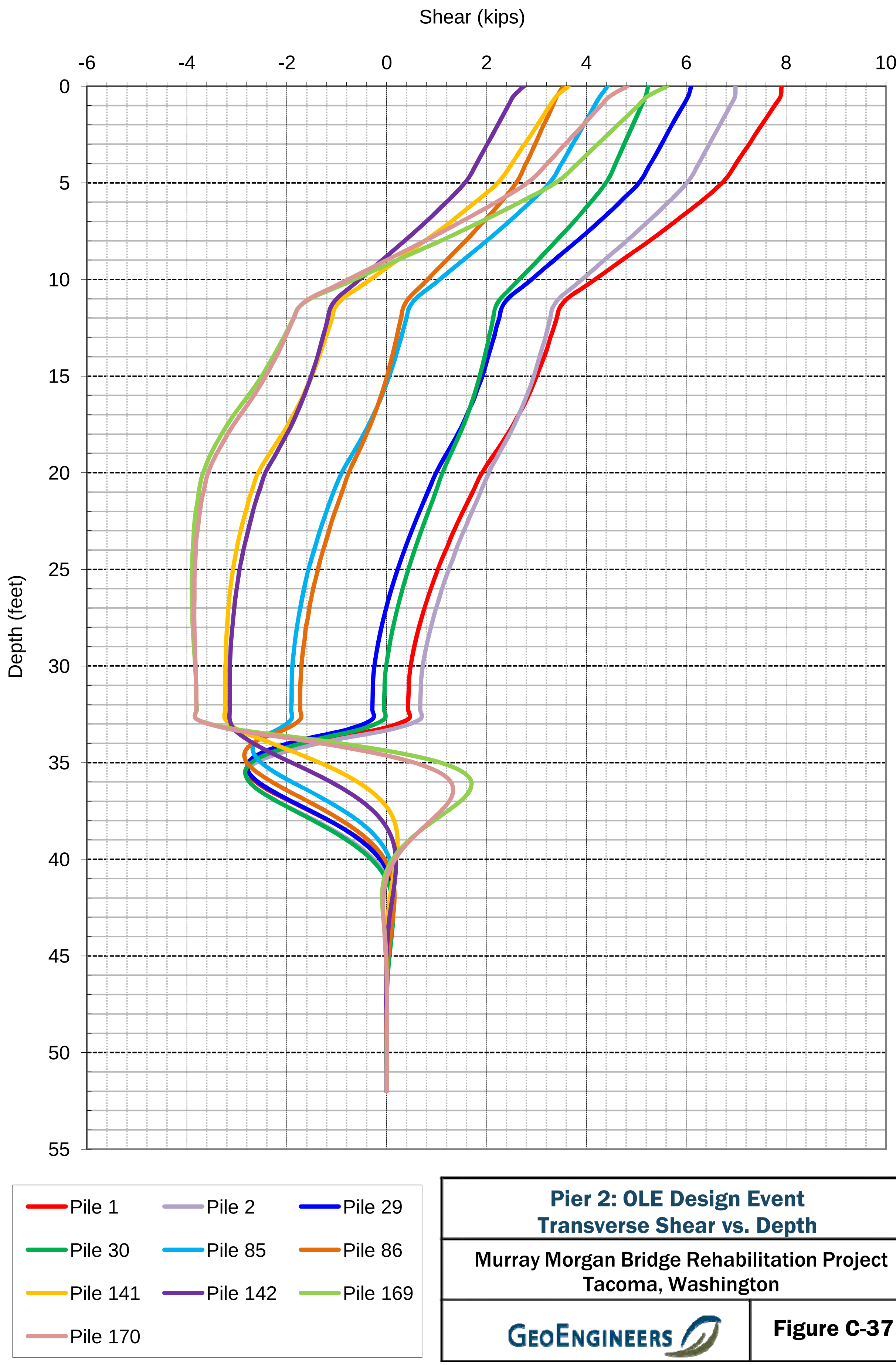
Figure C-36

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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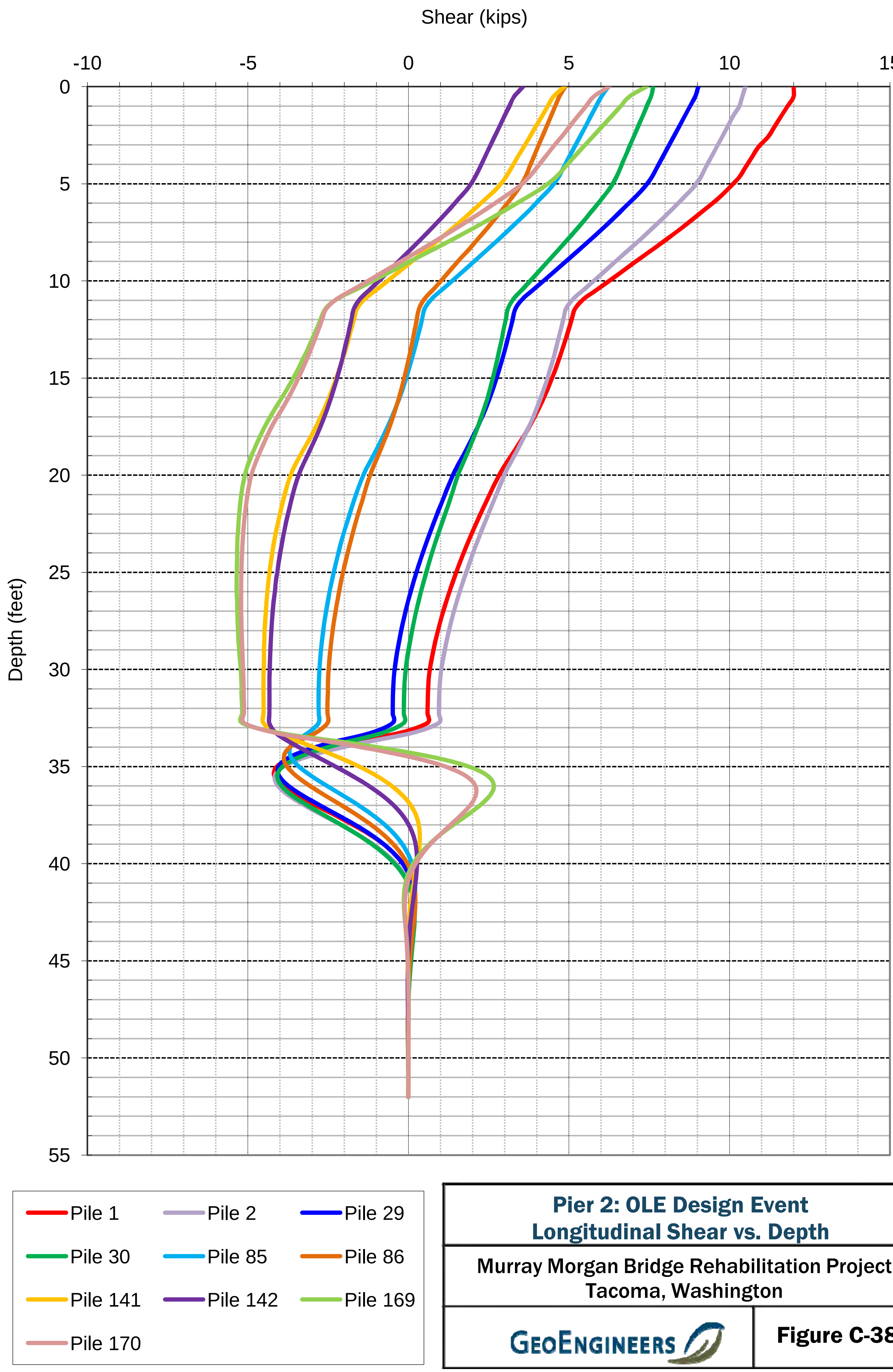
Figure C-37



Pier 2: OLE Design Event Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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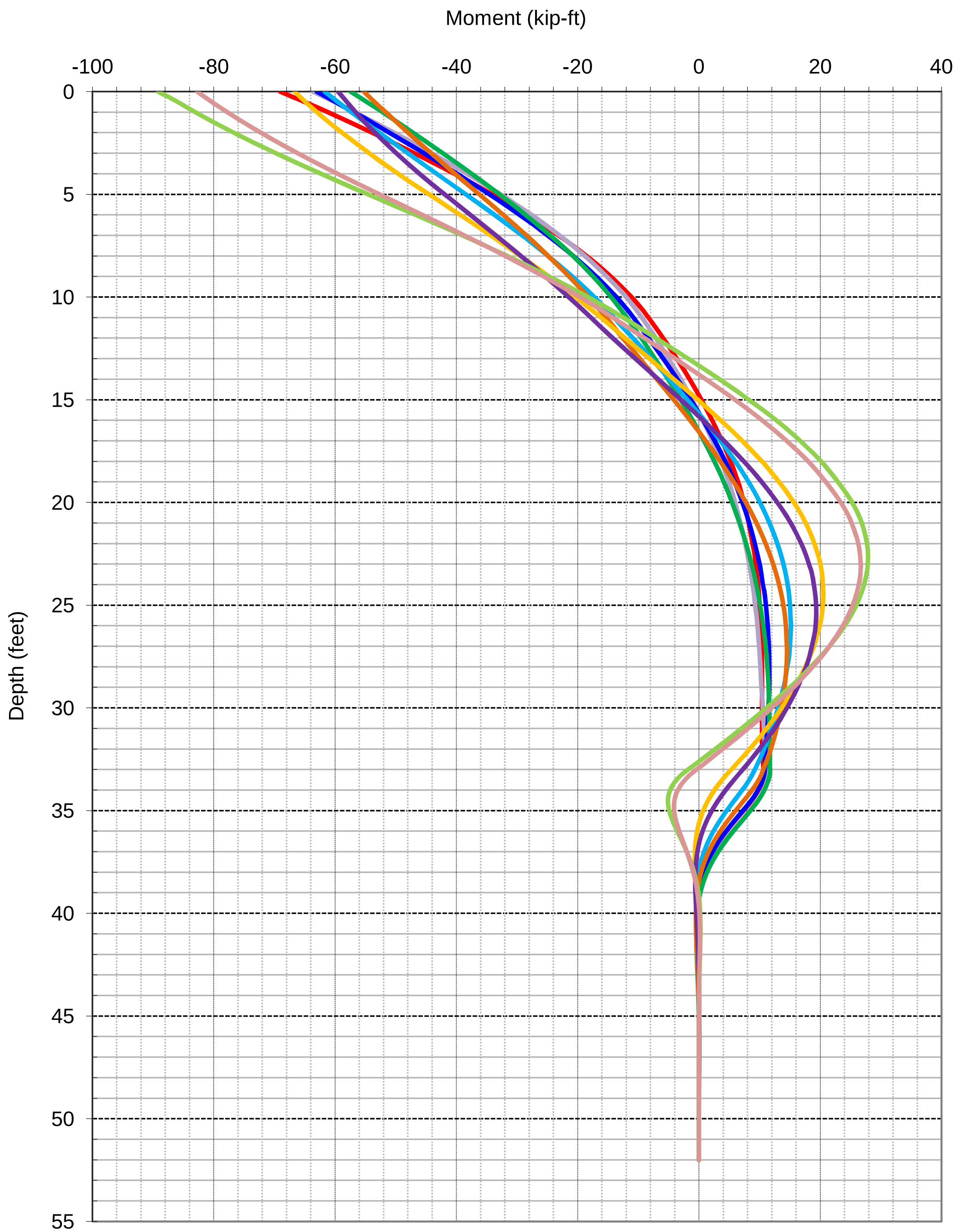
Figure C-38

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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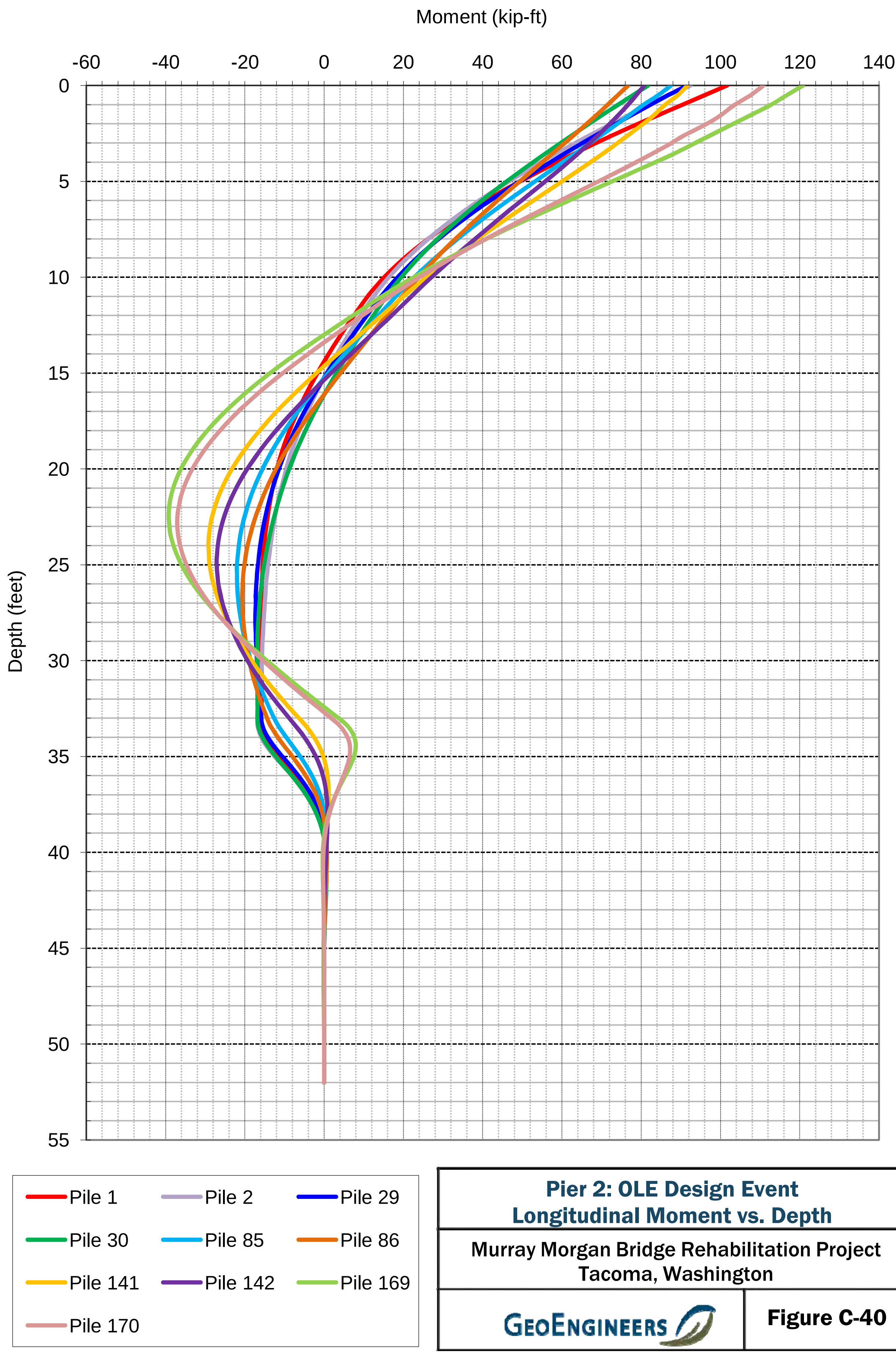
Figure C-39

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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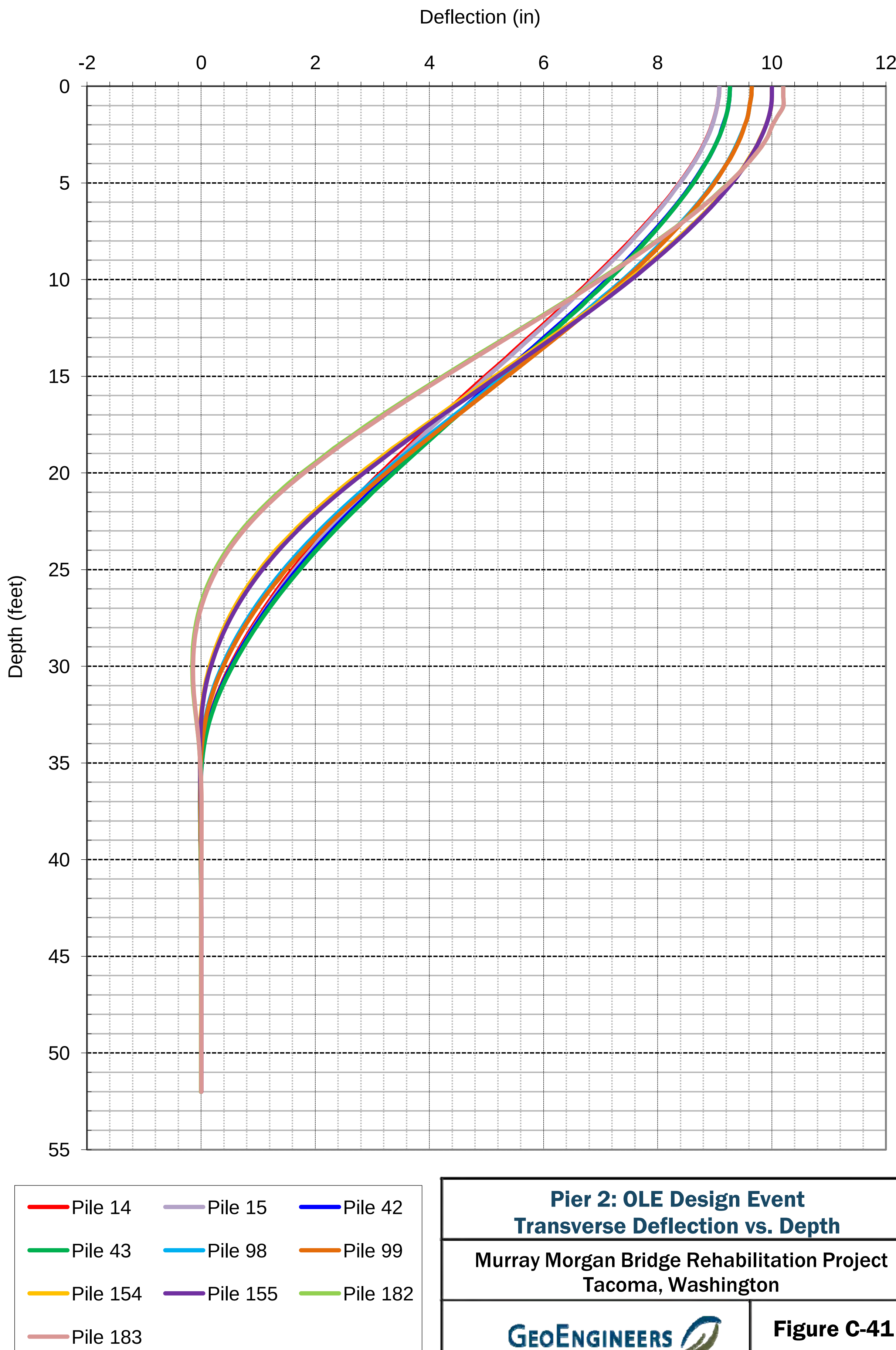
Figure C-40

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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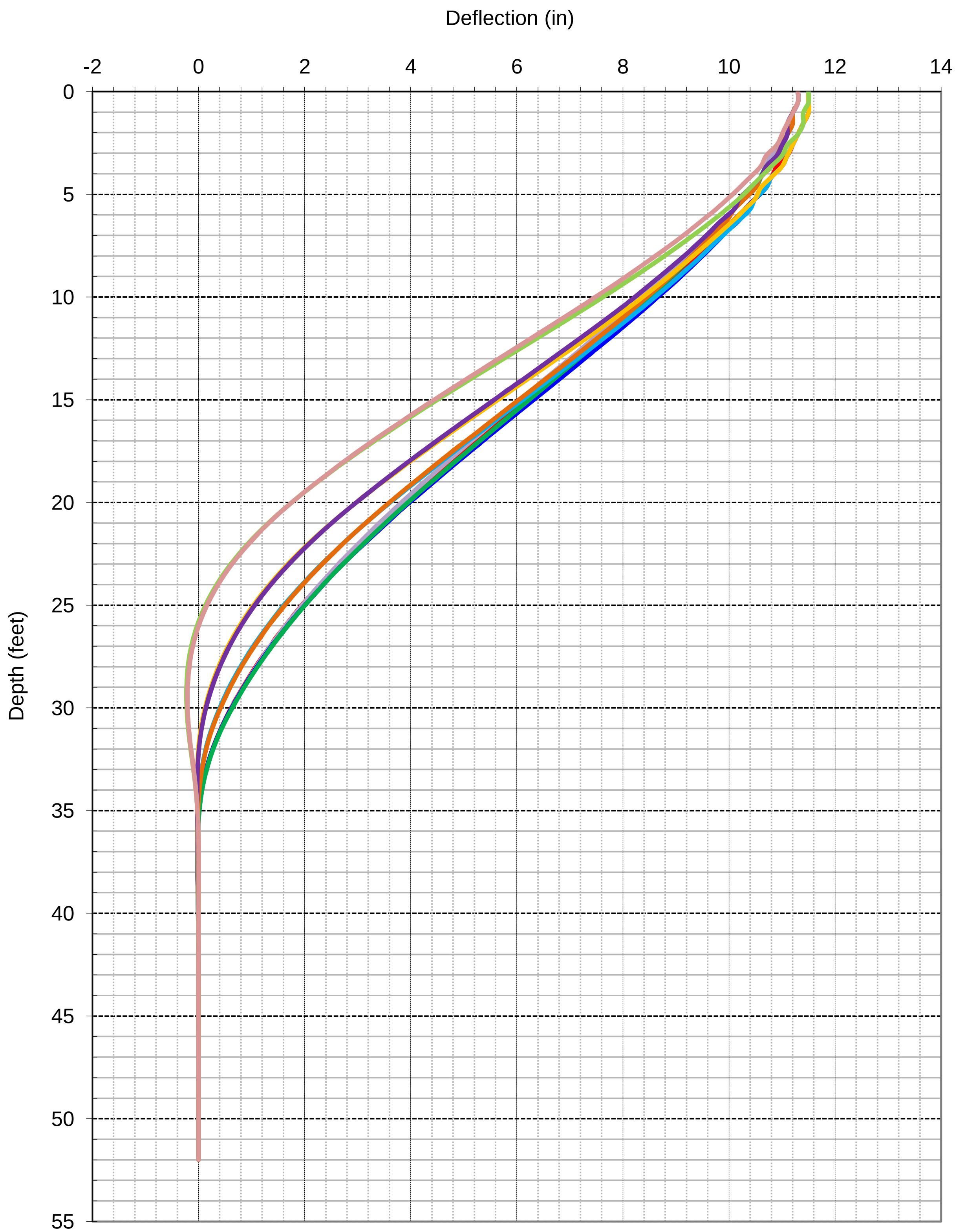
Figure C-41

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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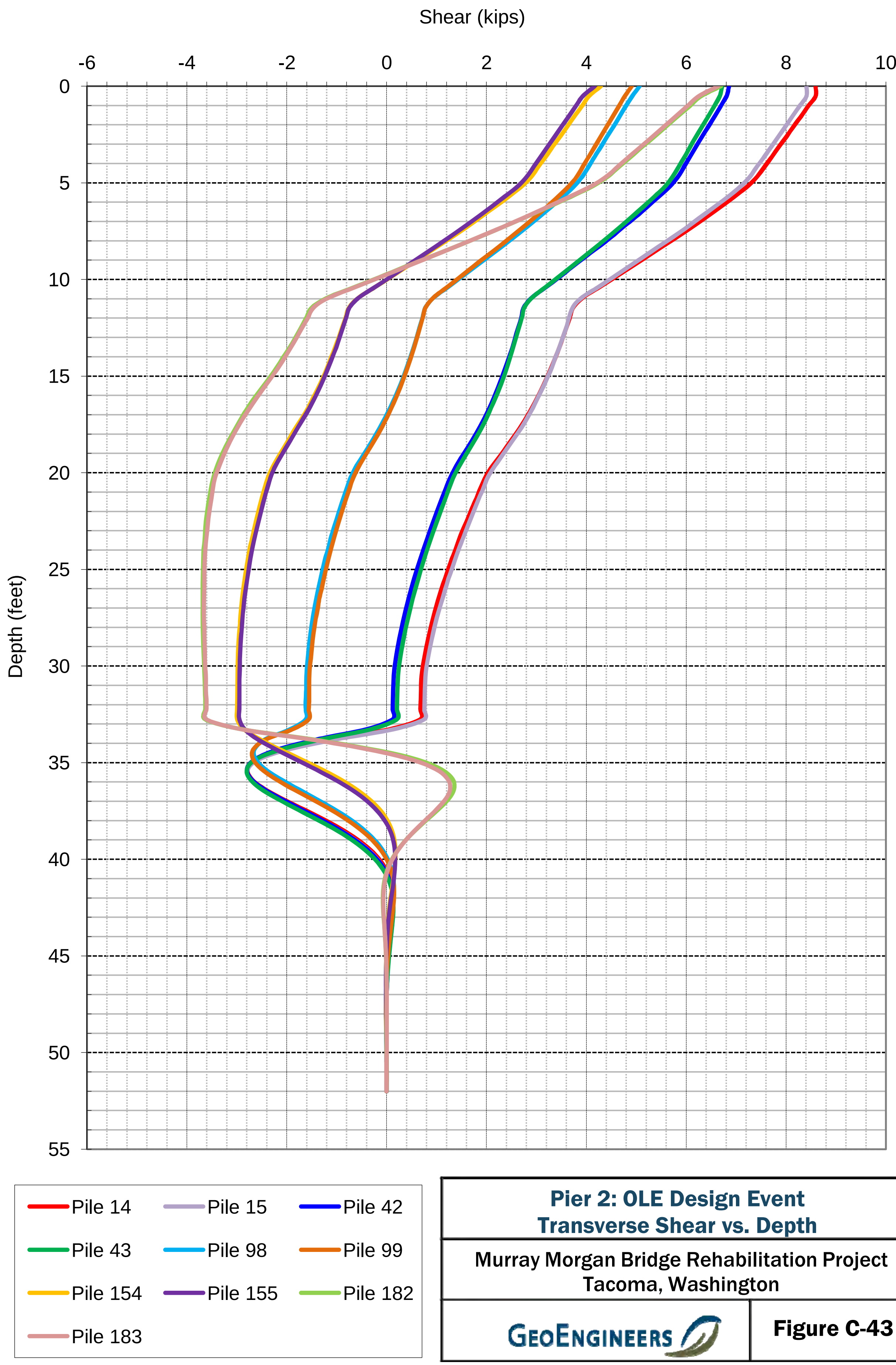
Figure C-42

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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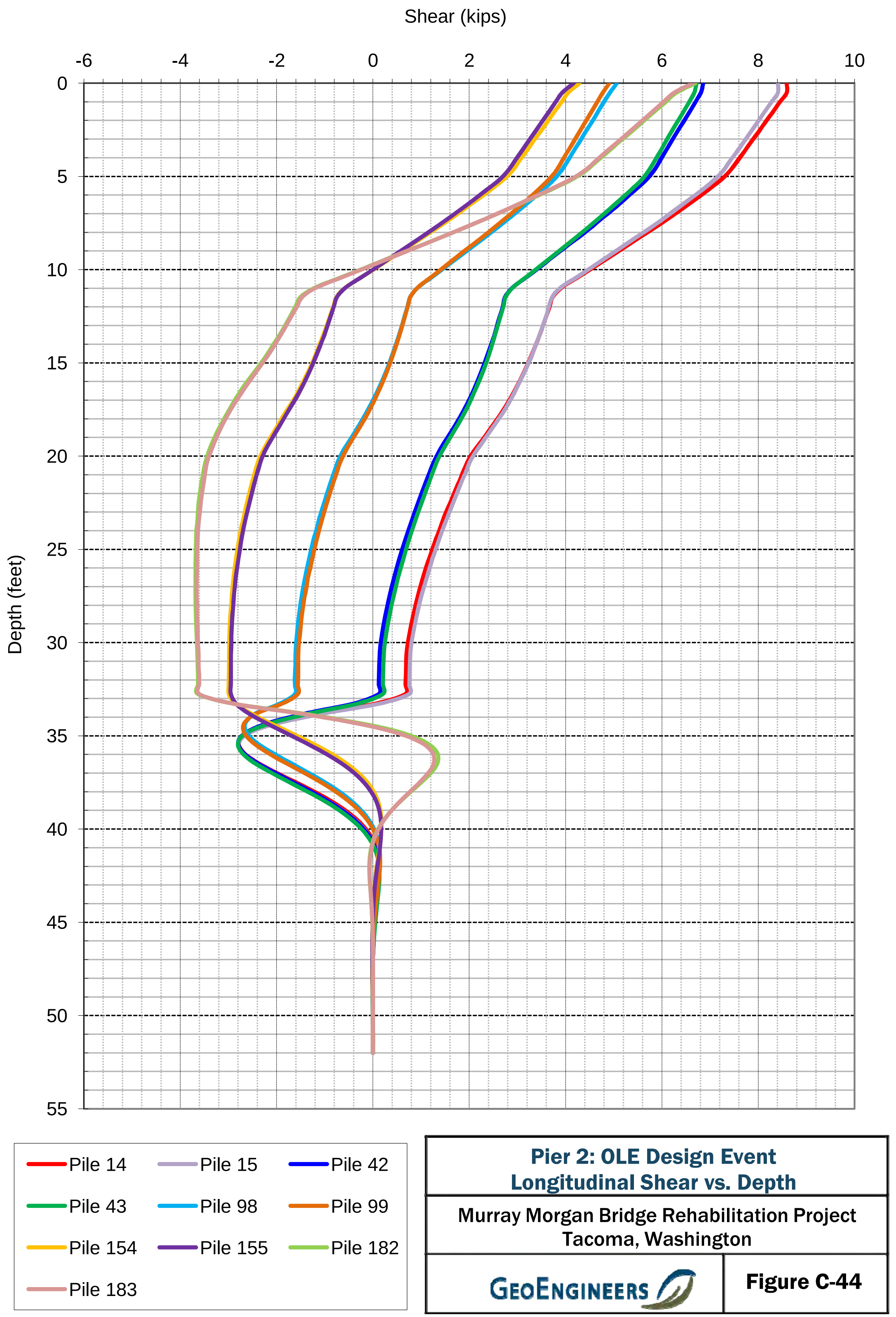
Figure C-43

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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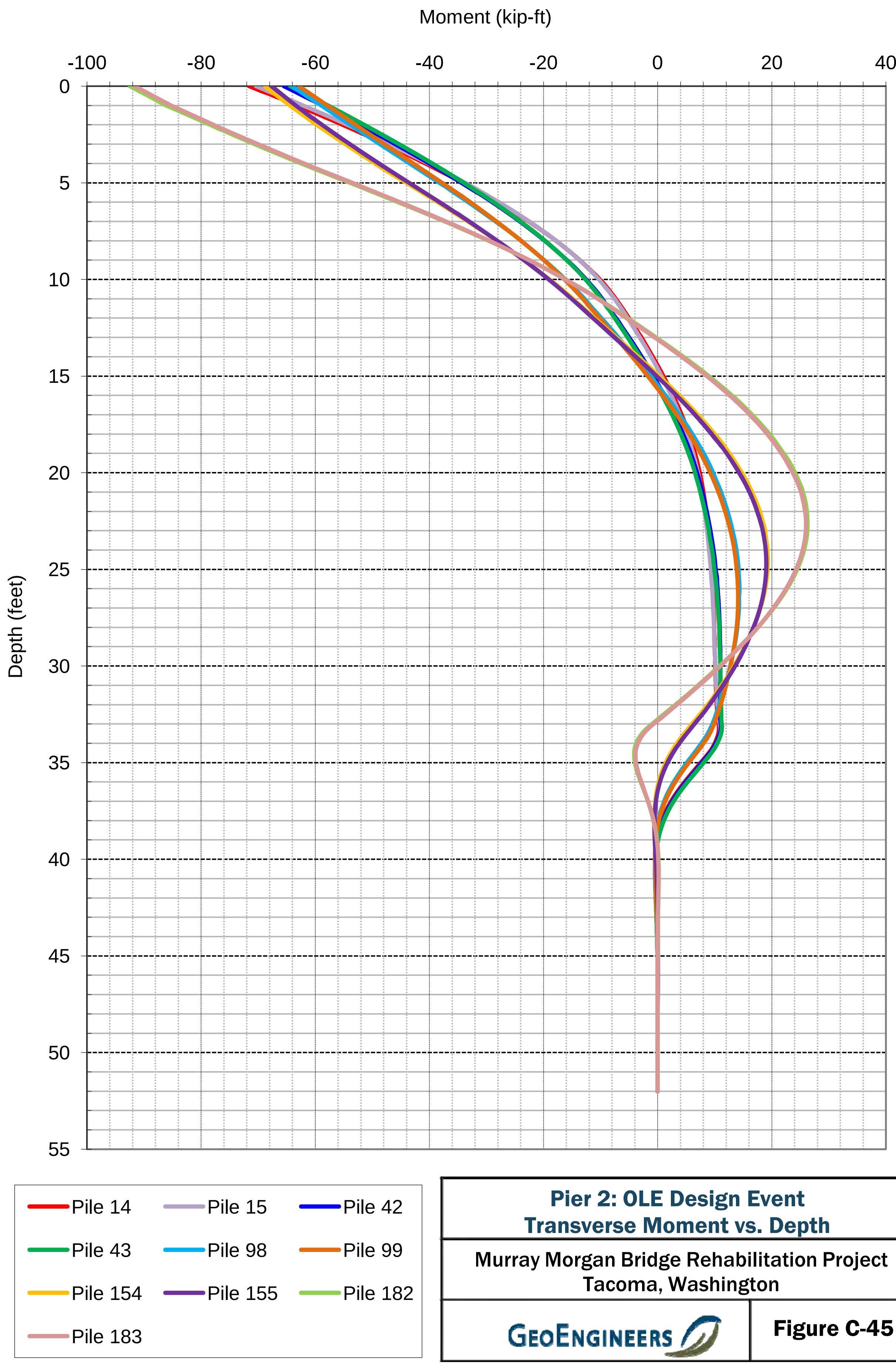
Figure C-44

0570100002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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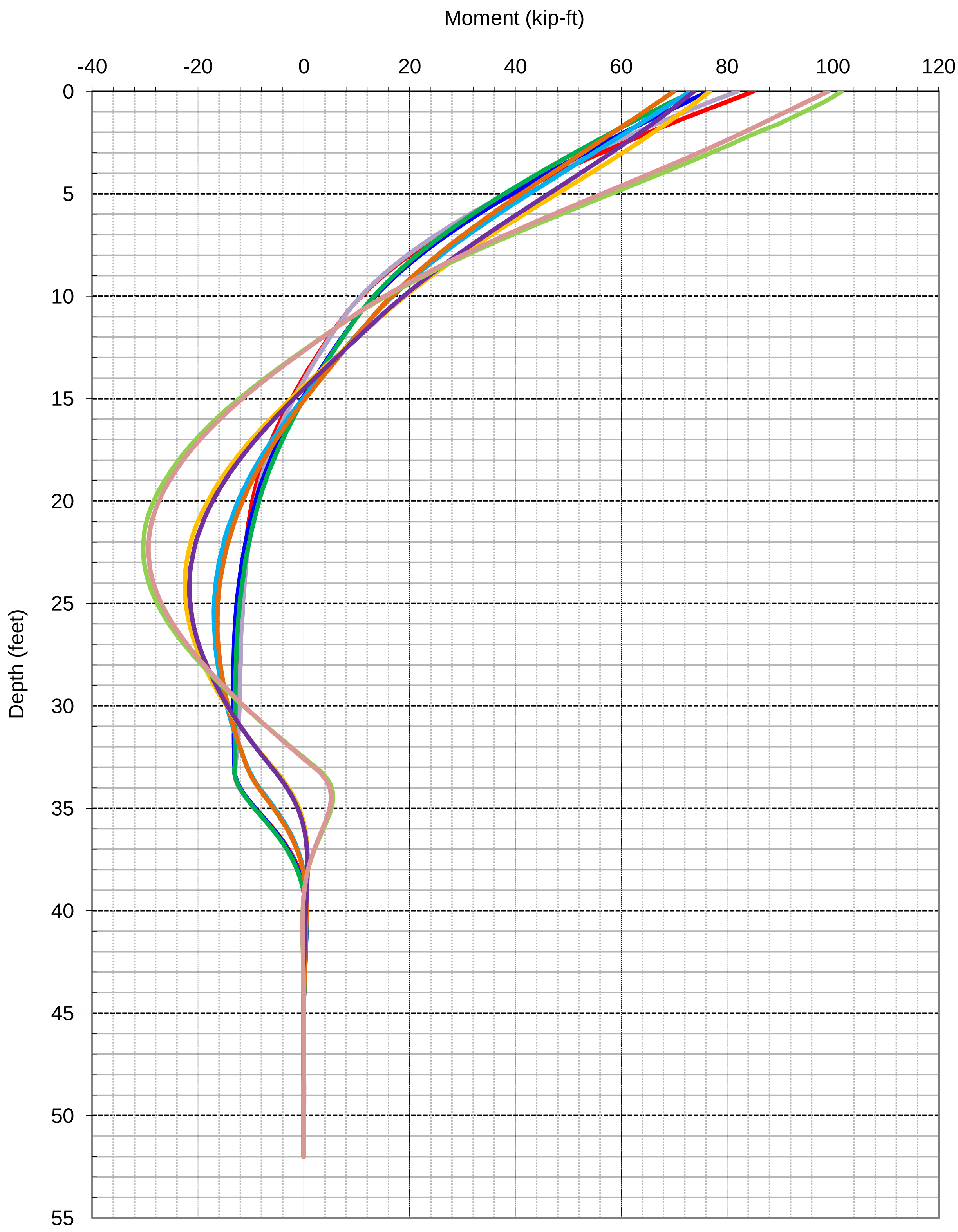
Figure C-45

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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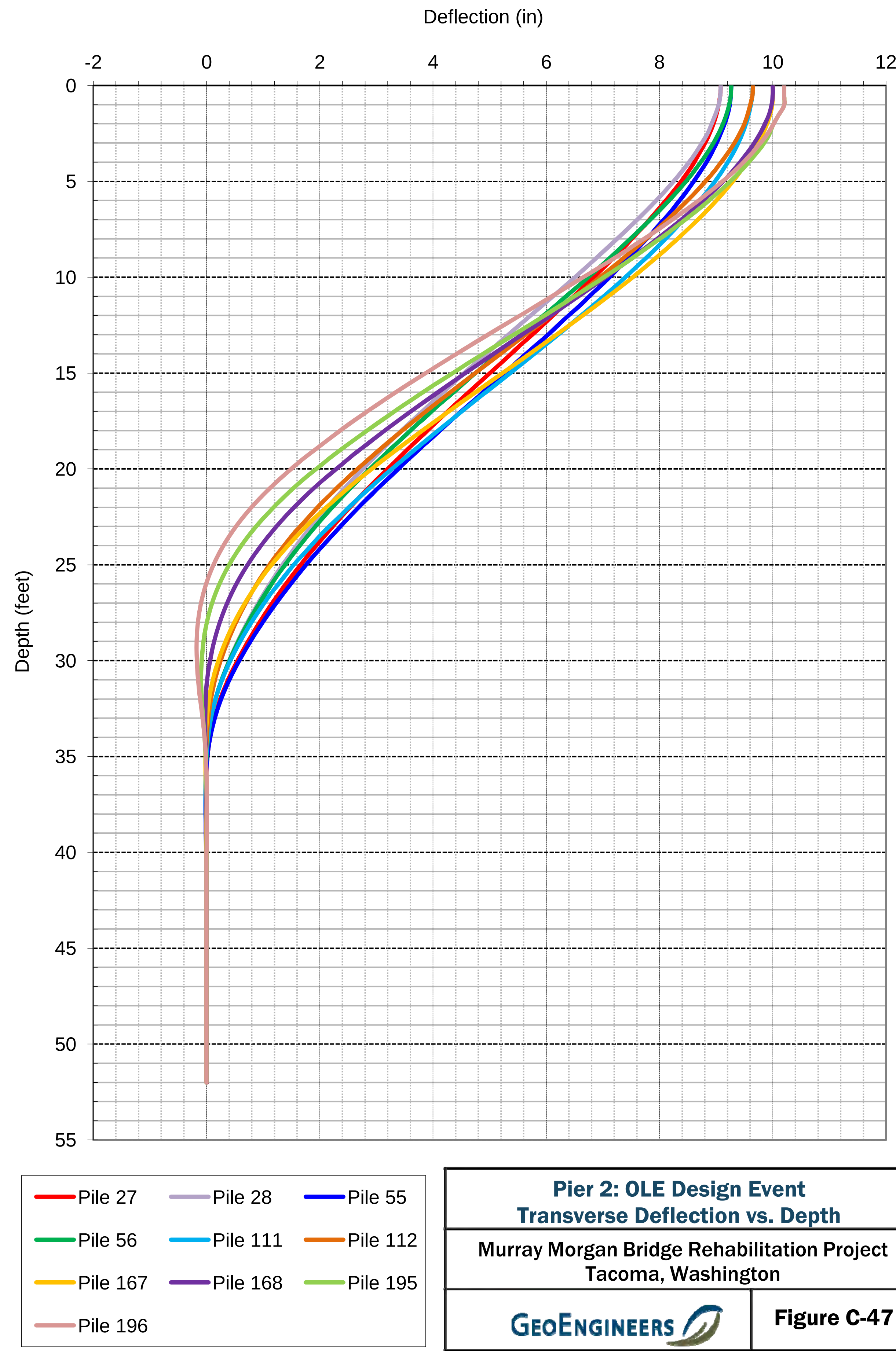
Figure C-46

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



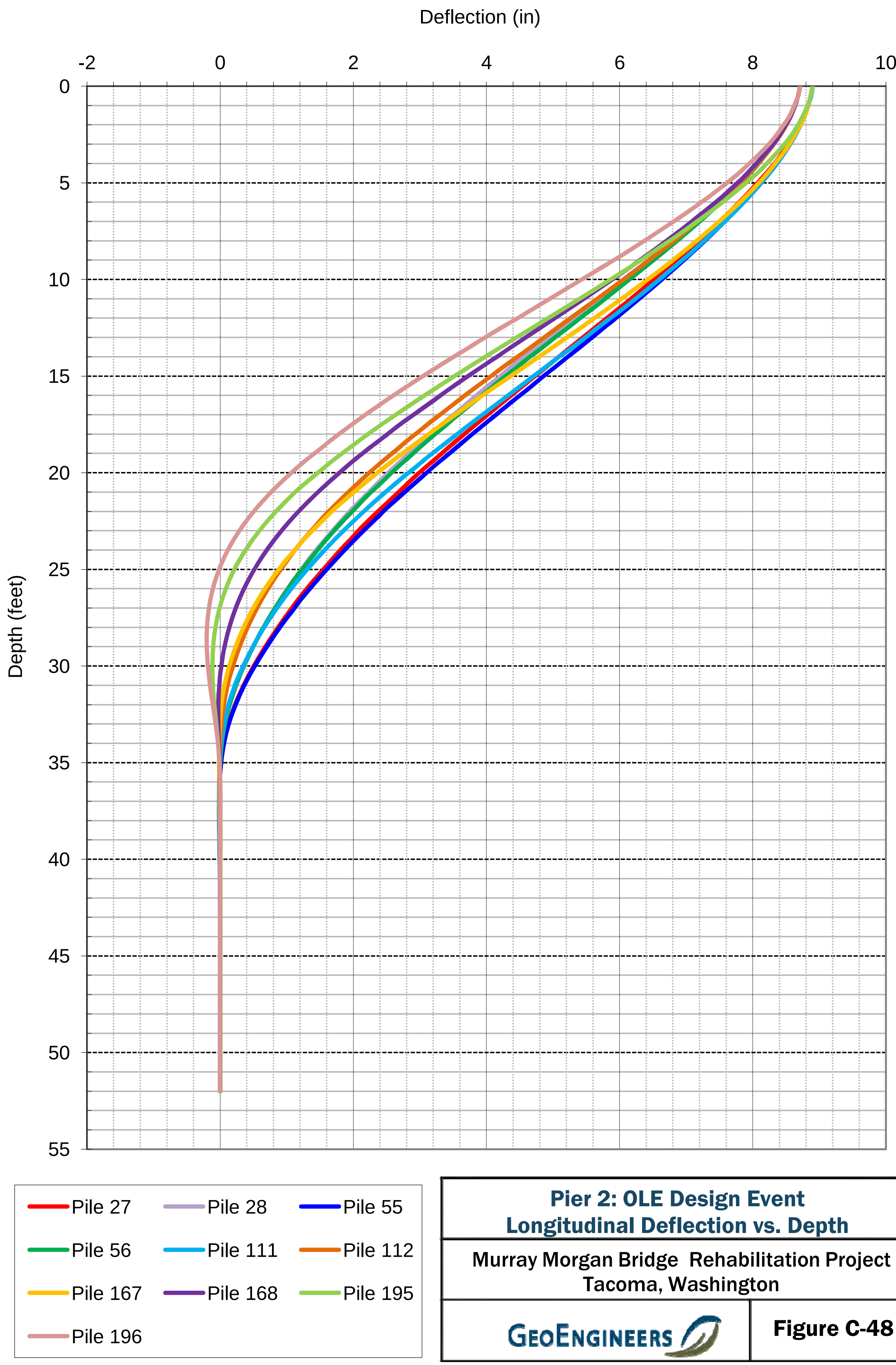
Pier 2: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-47

0570100002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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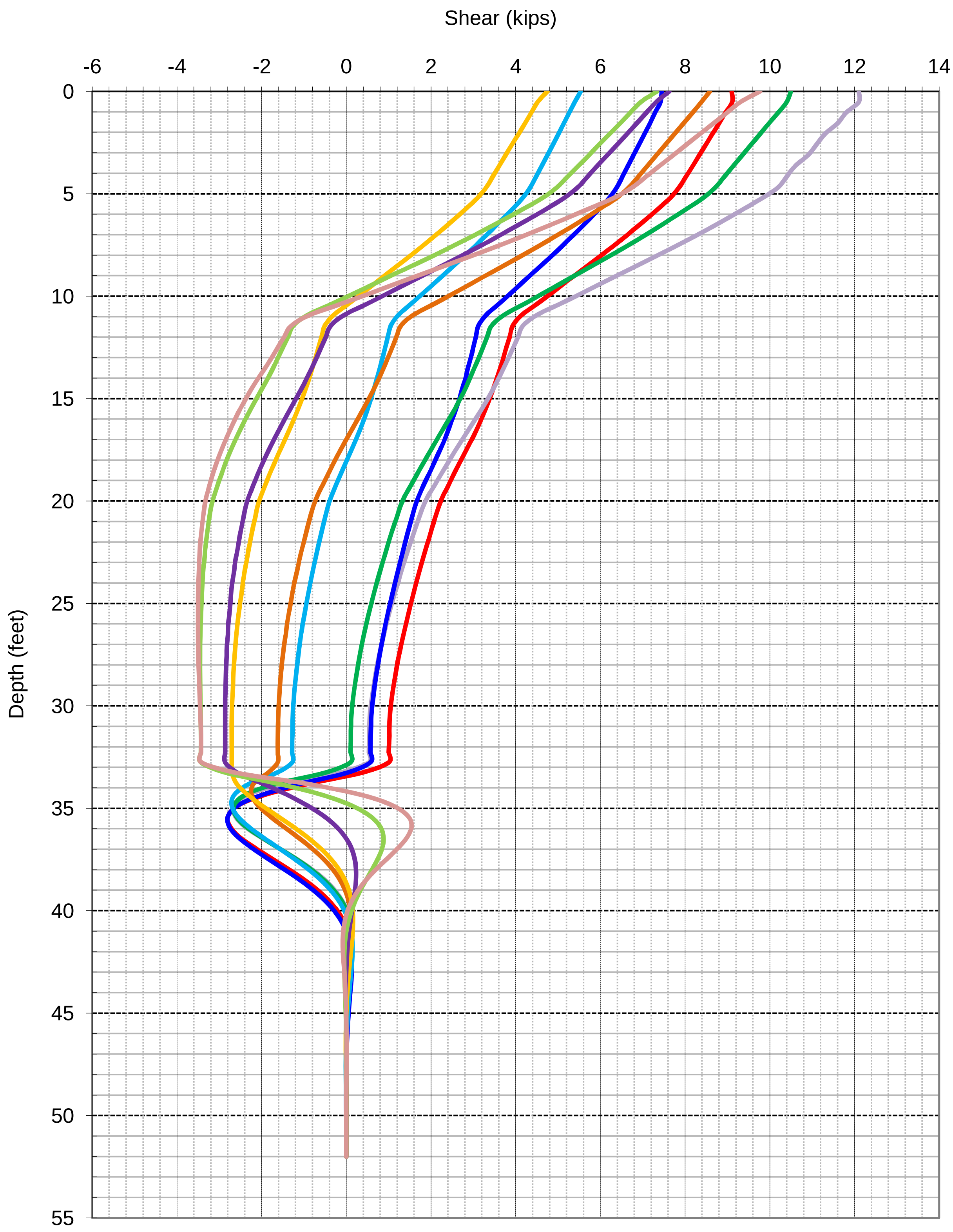
Figure C-48

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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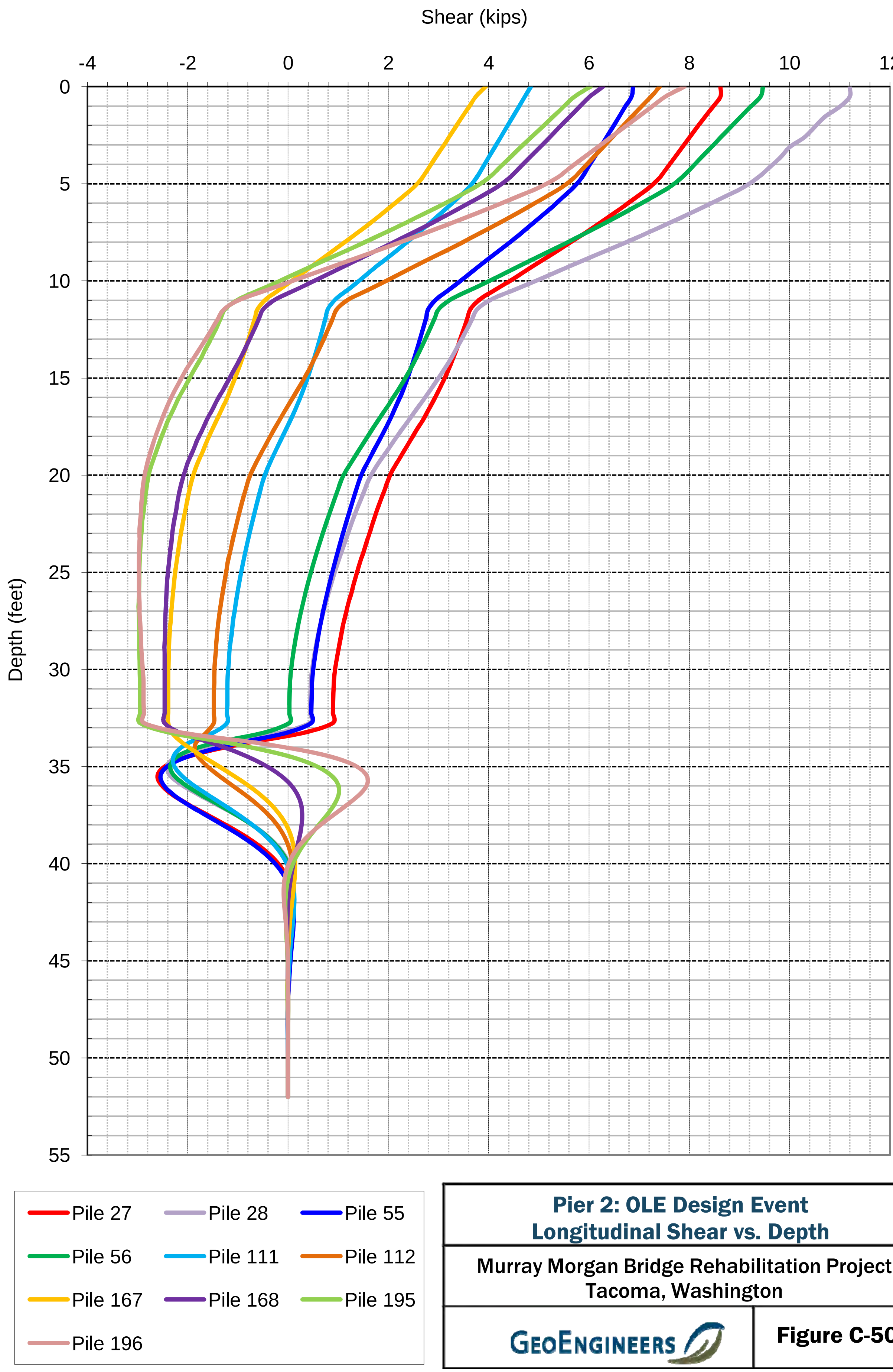
Figure C-49



Pier 2: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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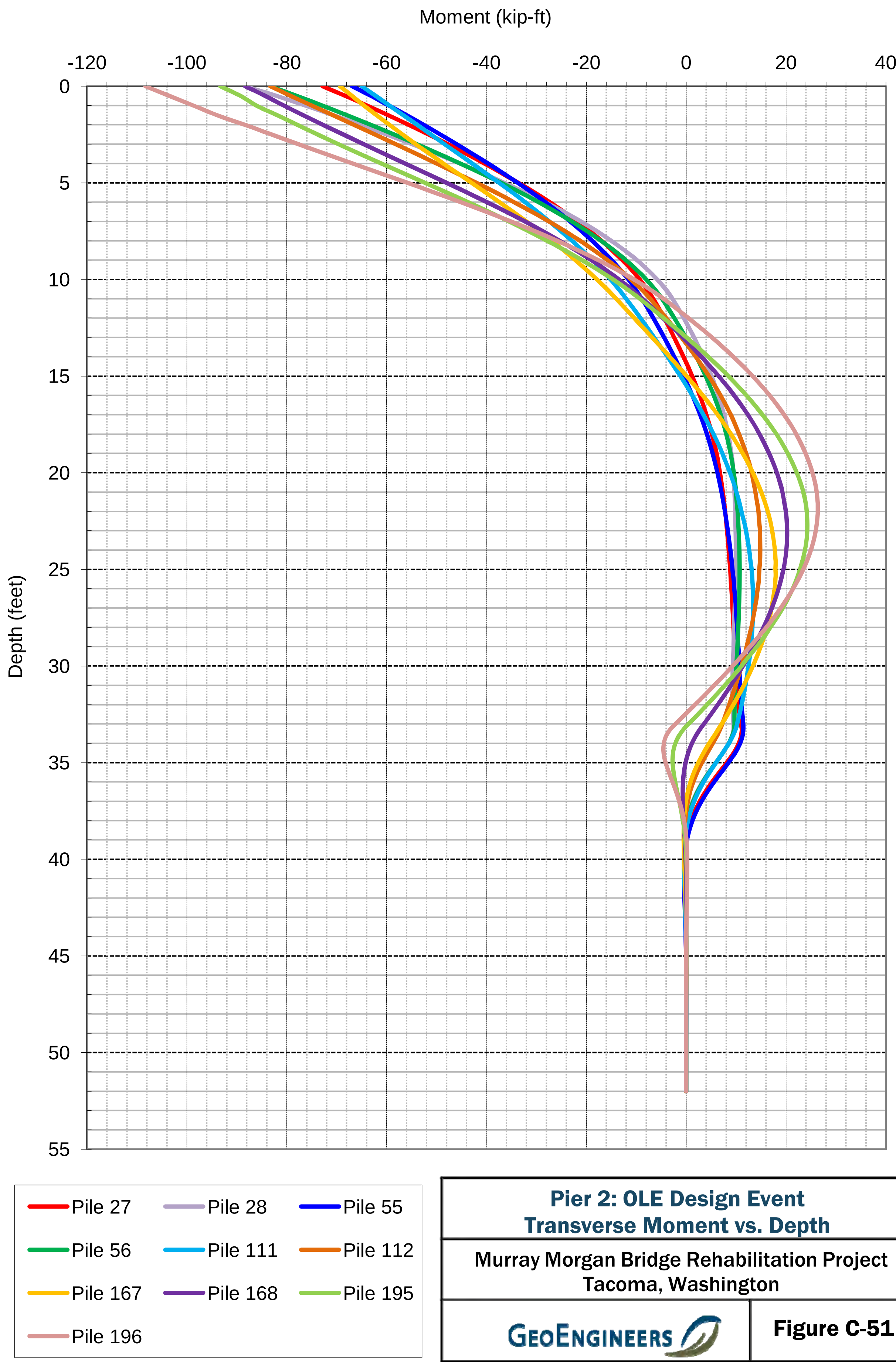
Figure C-50

0570100002 Figures C-35_C52.xlsx WLT:khc 10/21/11



Pier 2: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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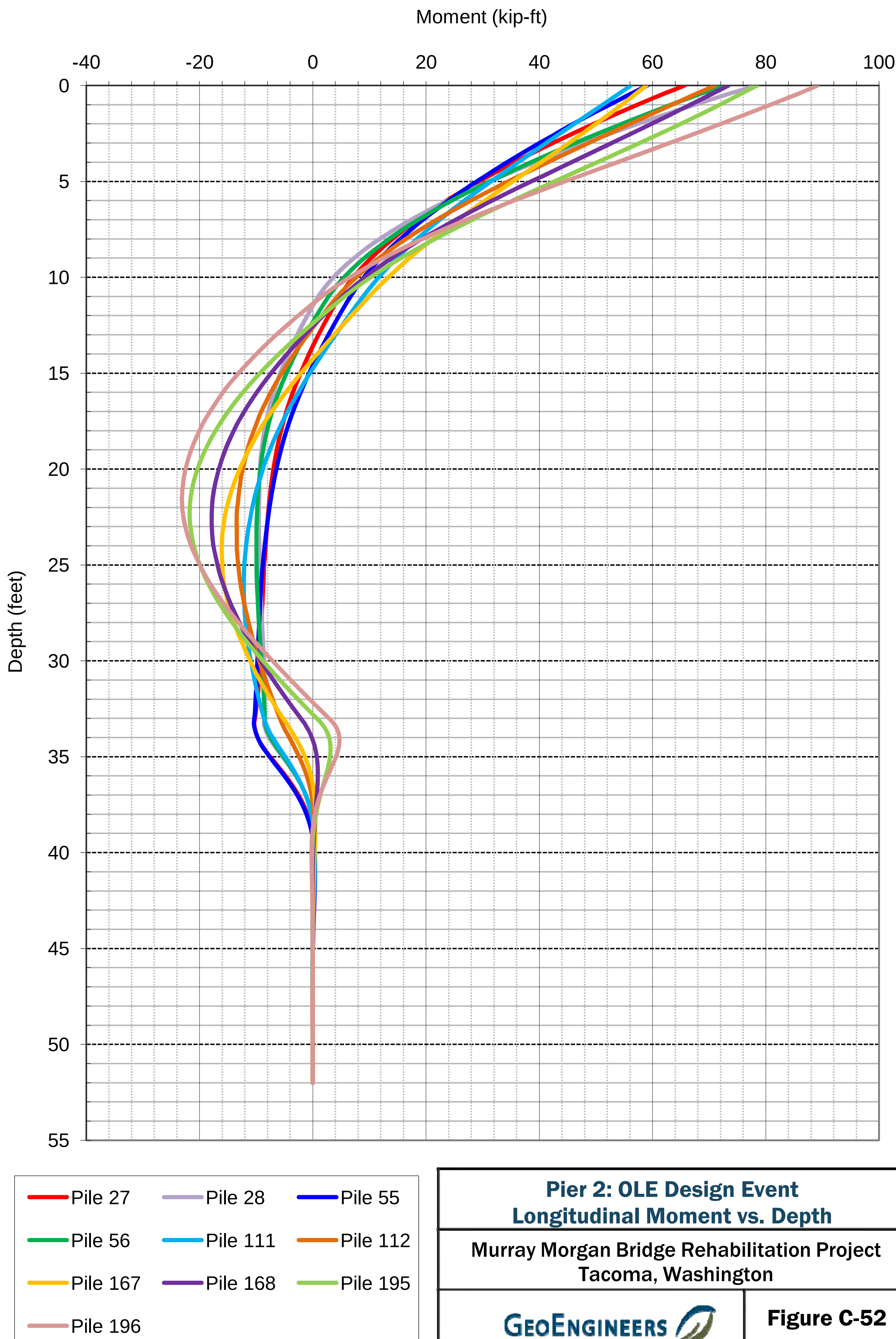
Figure C-51

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



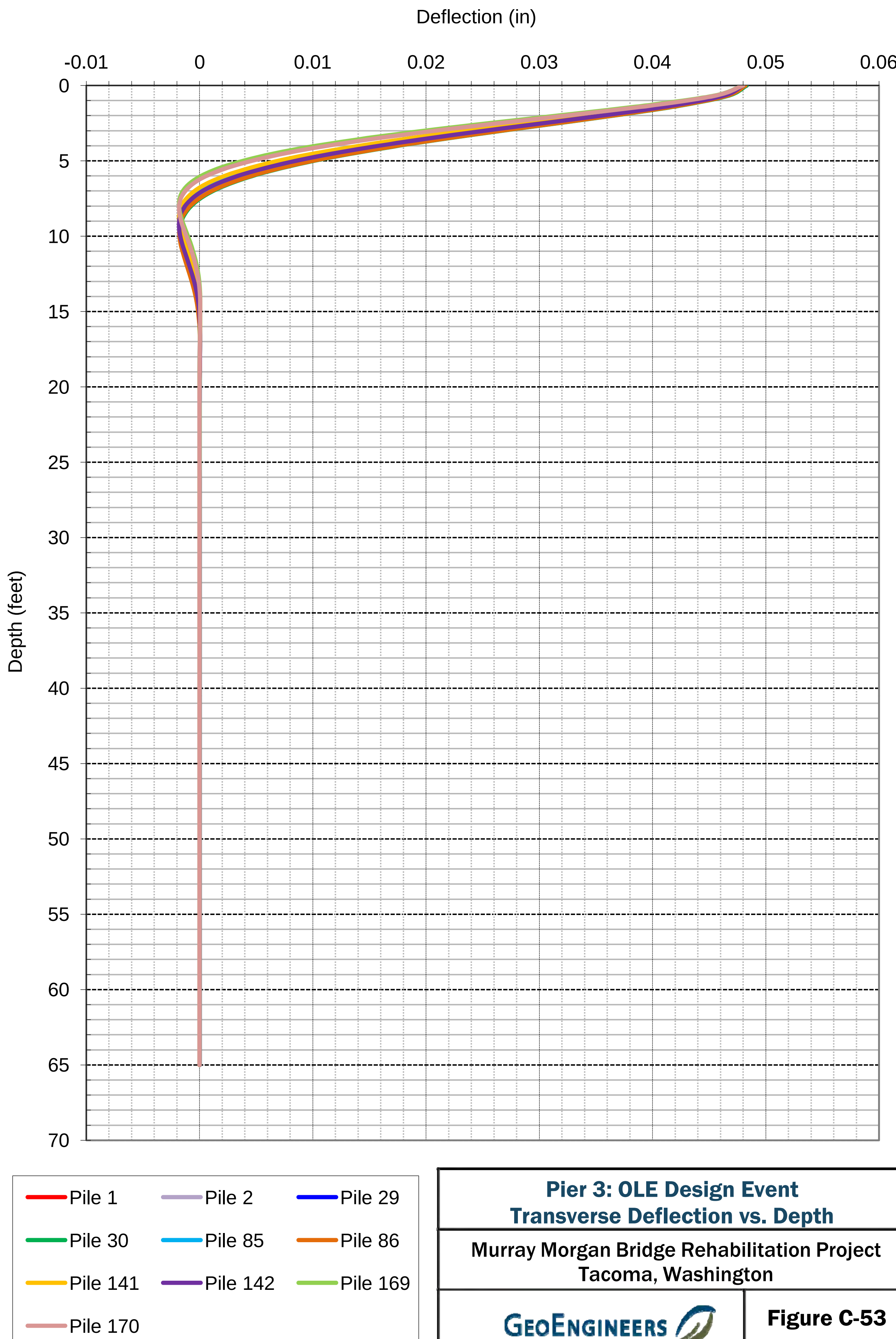
Pier 2: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-52

057010002 Figures C-35_C52.xlsx WLT:khc 10/21/11



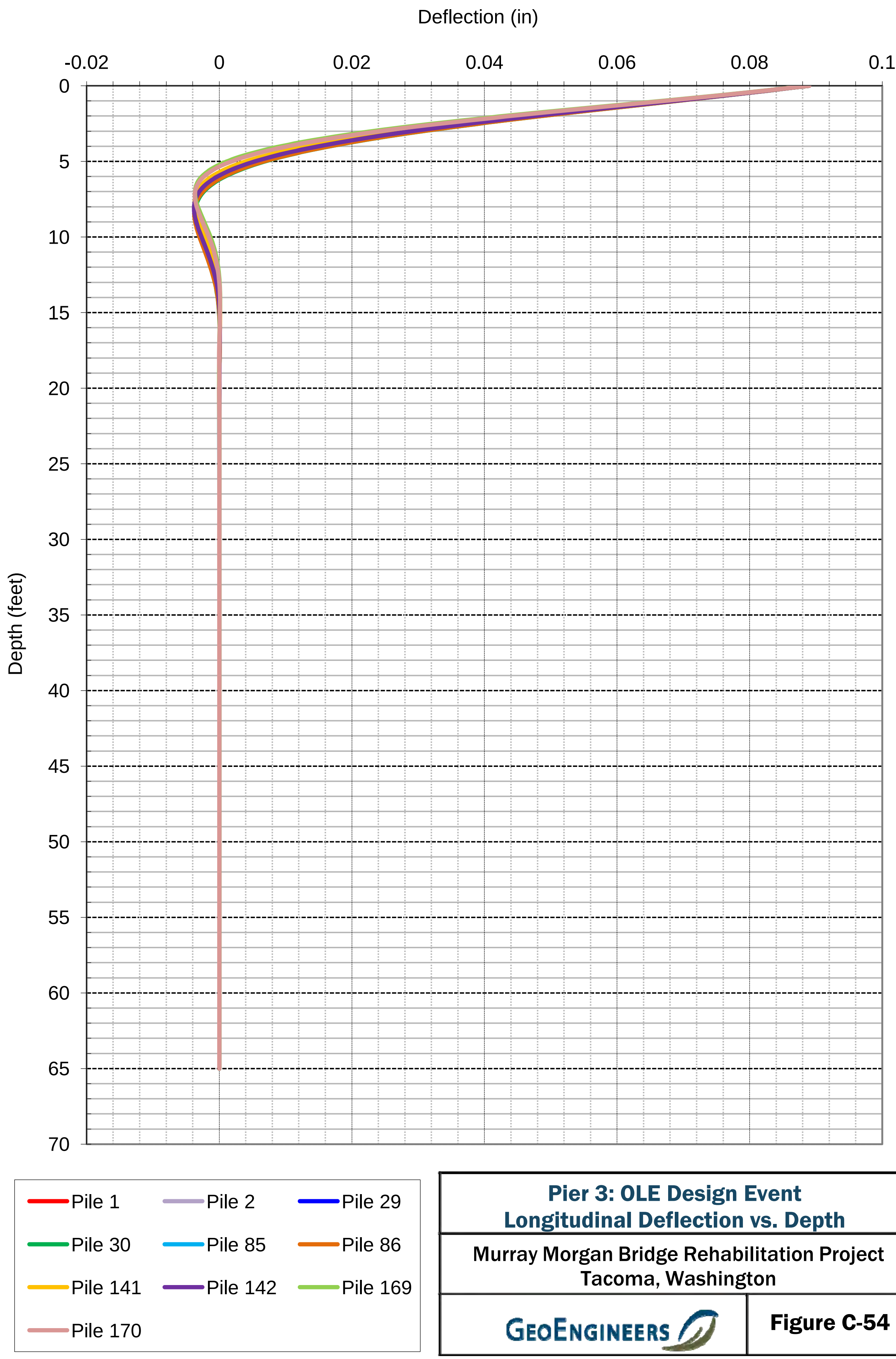
Pier 3: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-53

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11

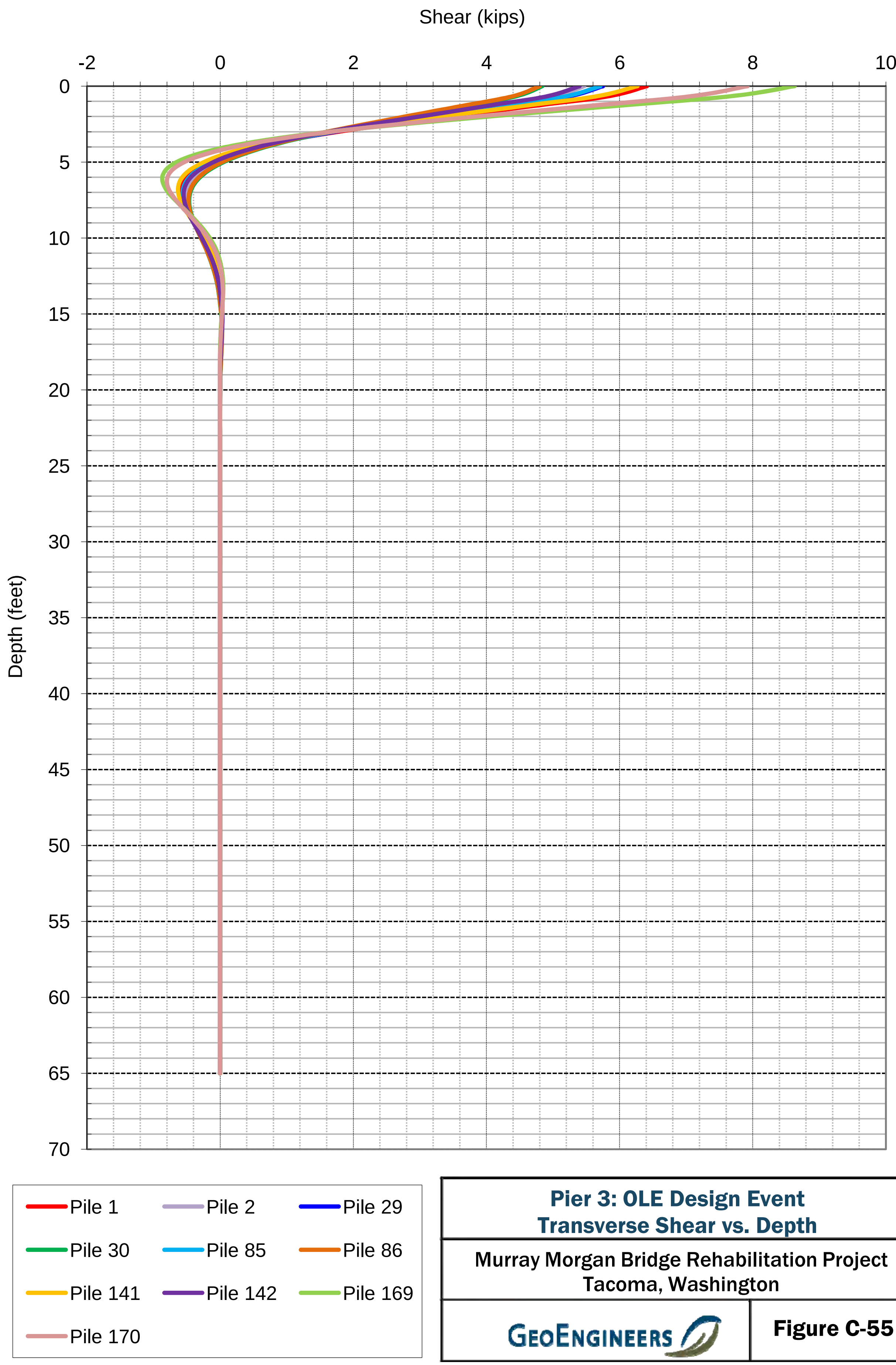


Pier 3: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-54



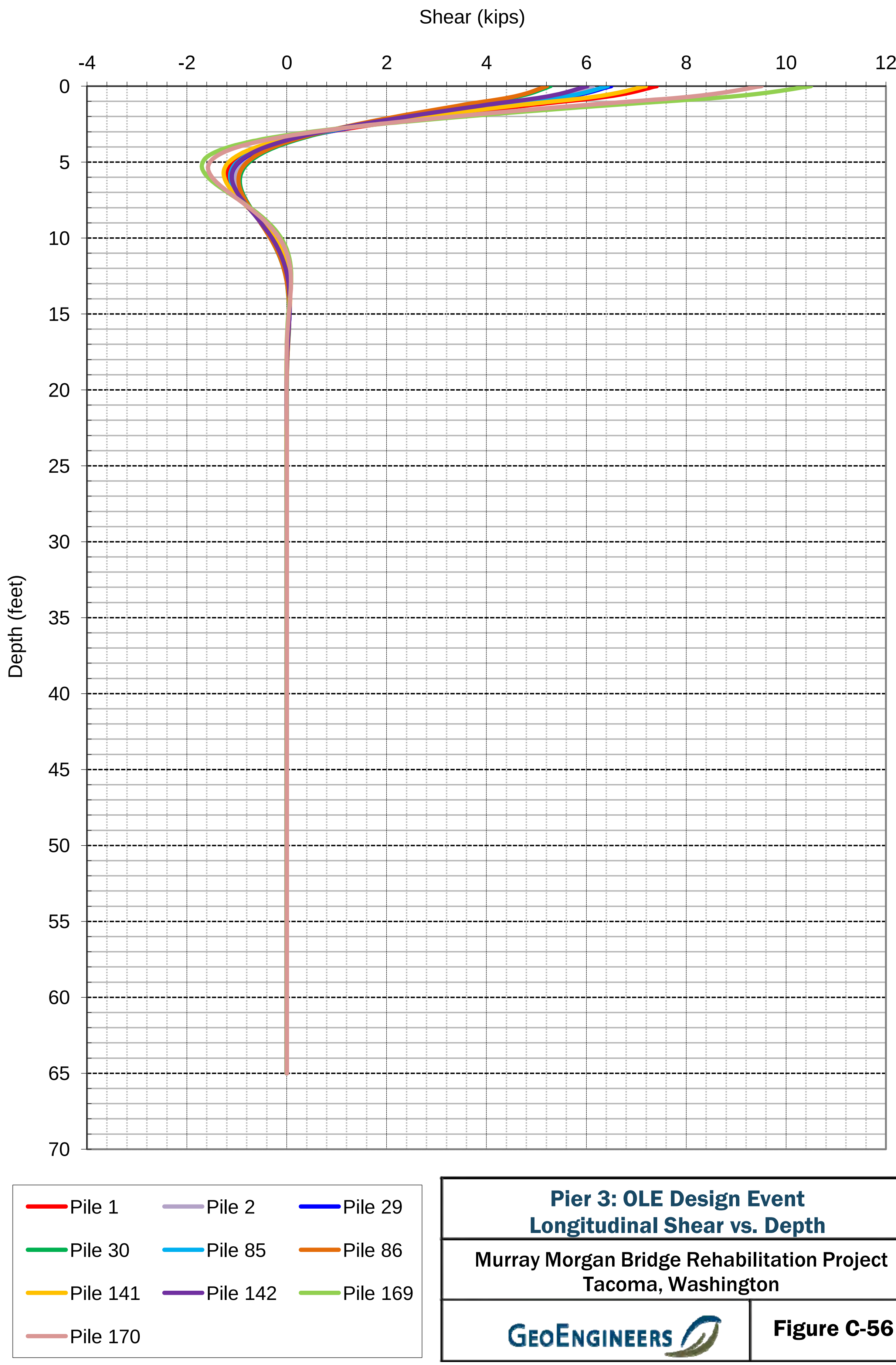
Pier 3: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-55

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



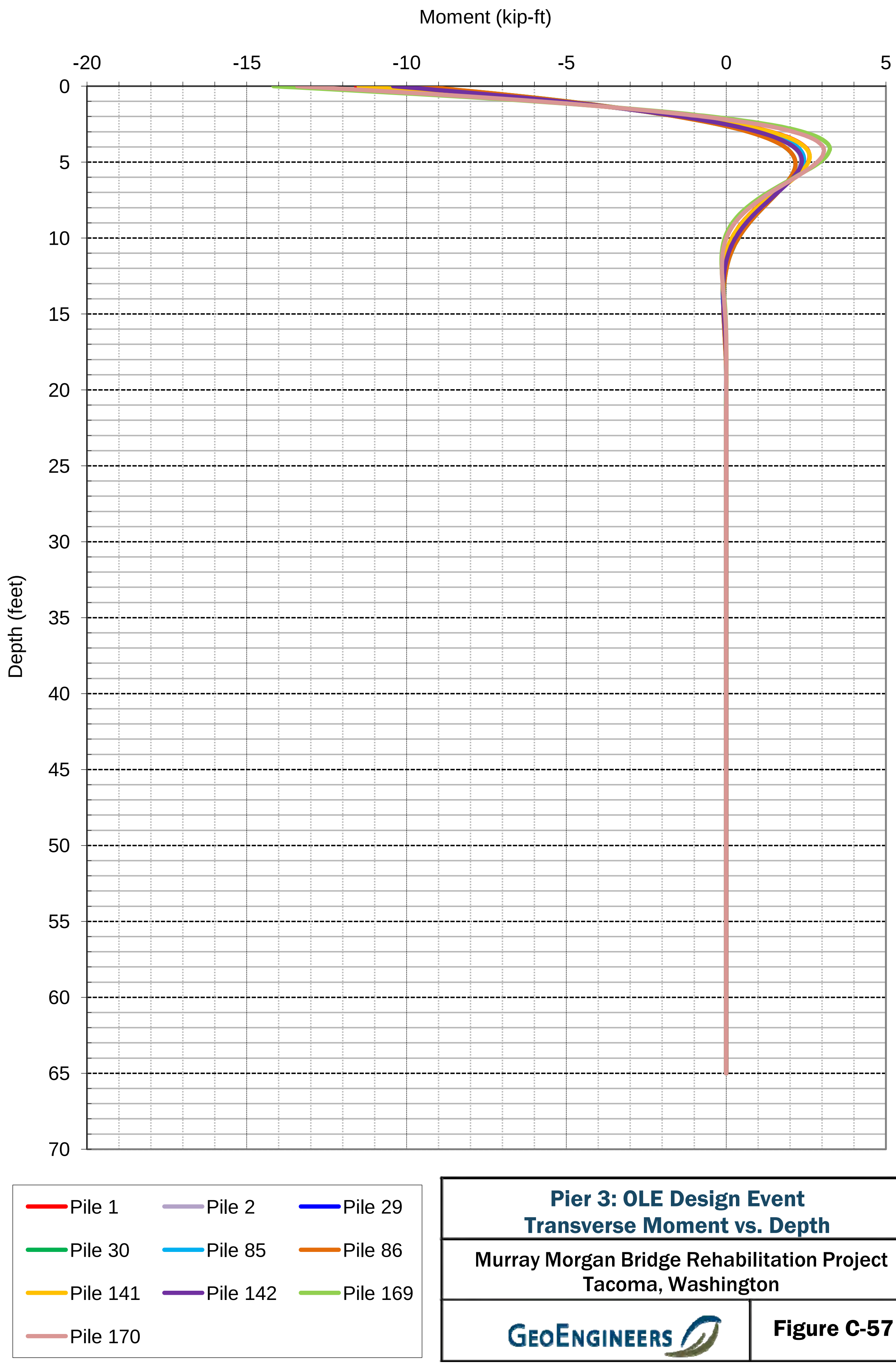
Pier 3: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-56

0570100002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



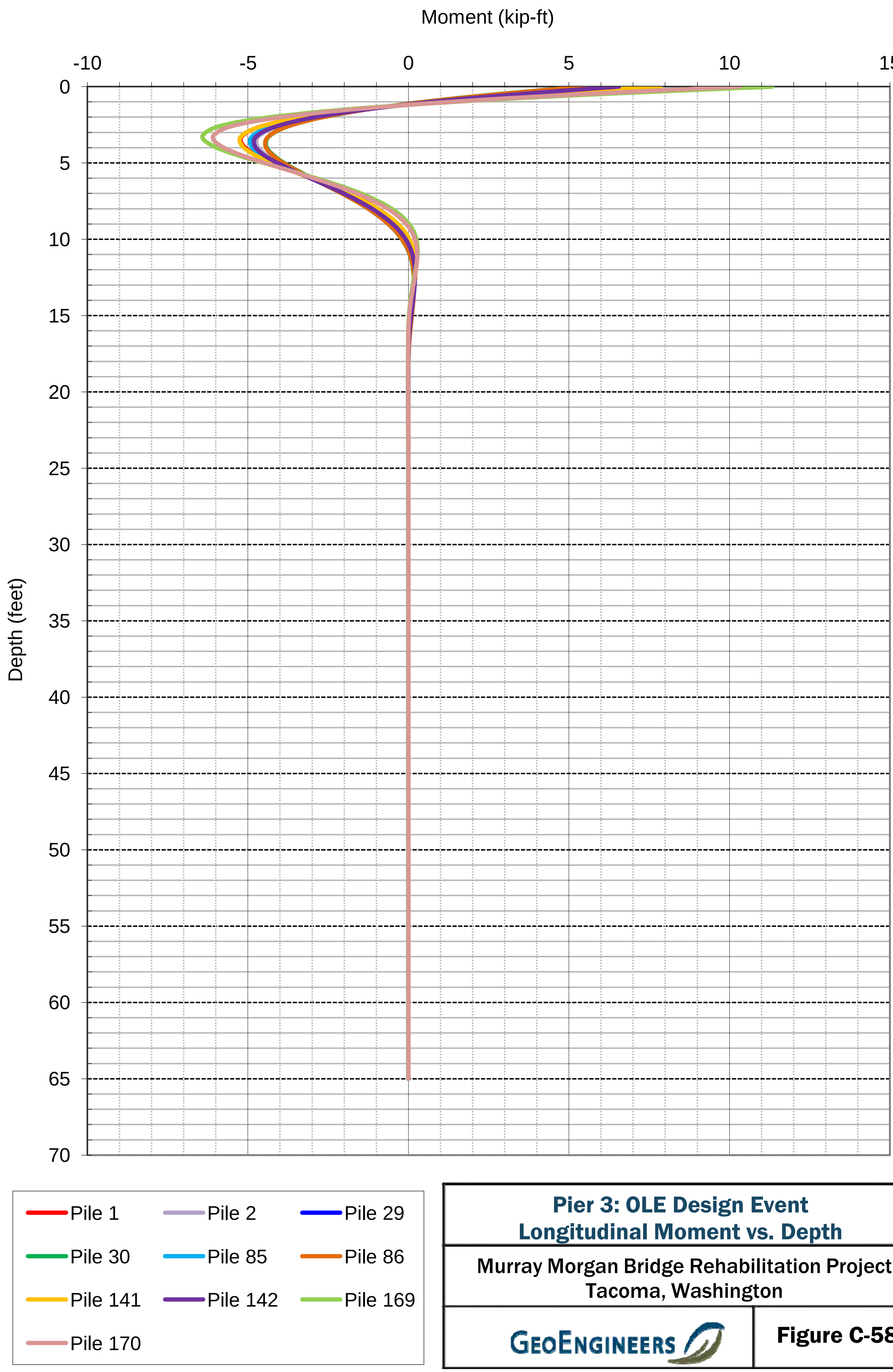
Pier 3: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-57

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



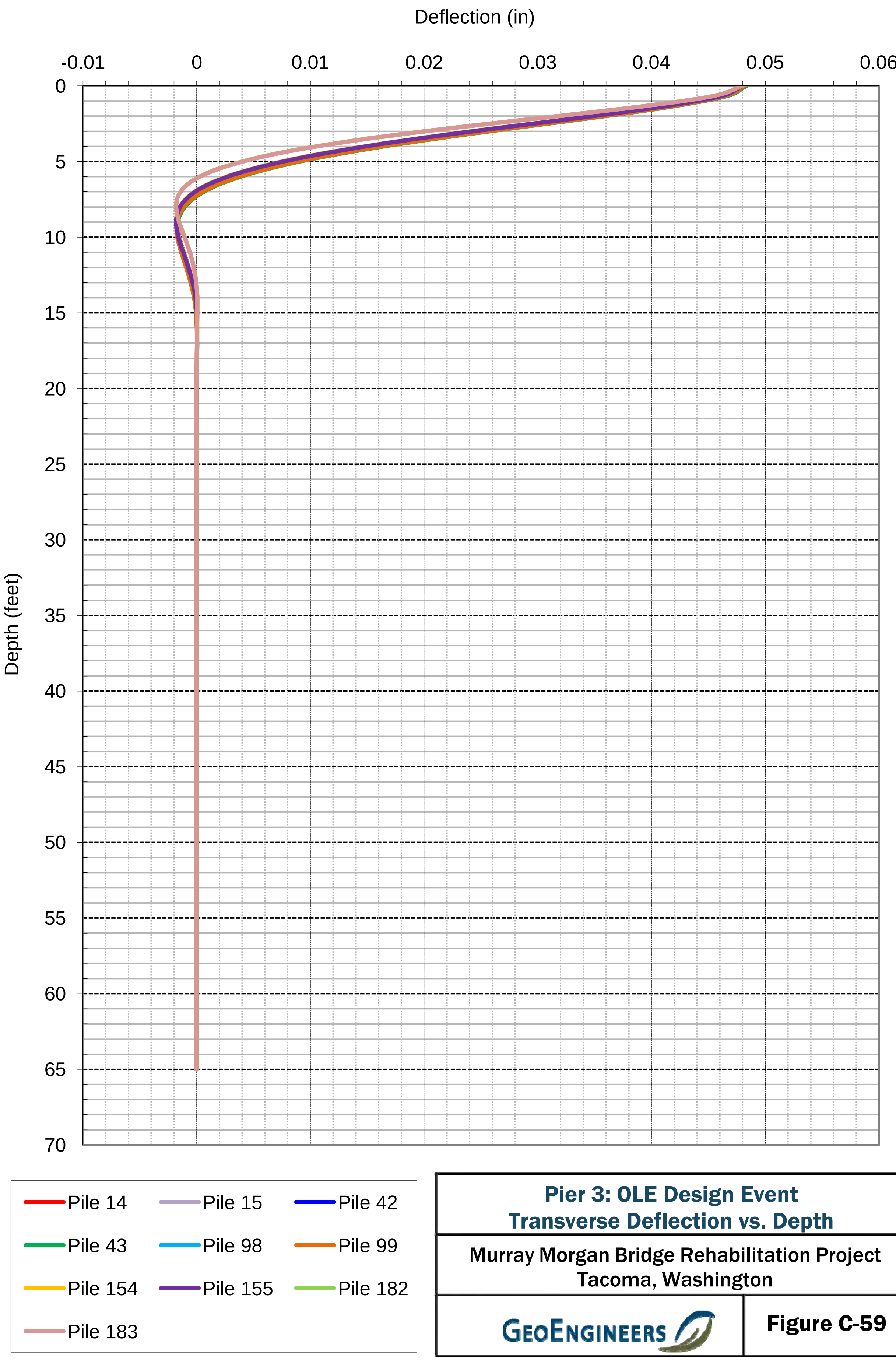
Pier 3: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-58

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11

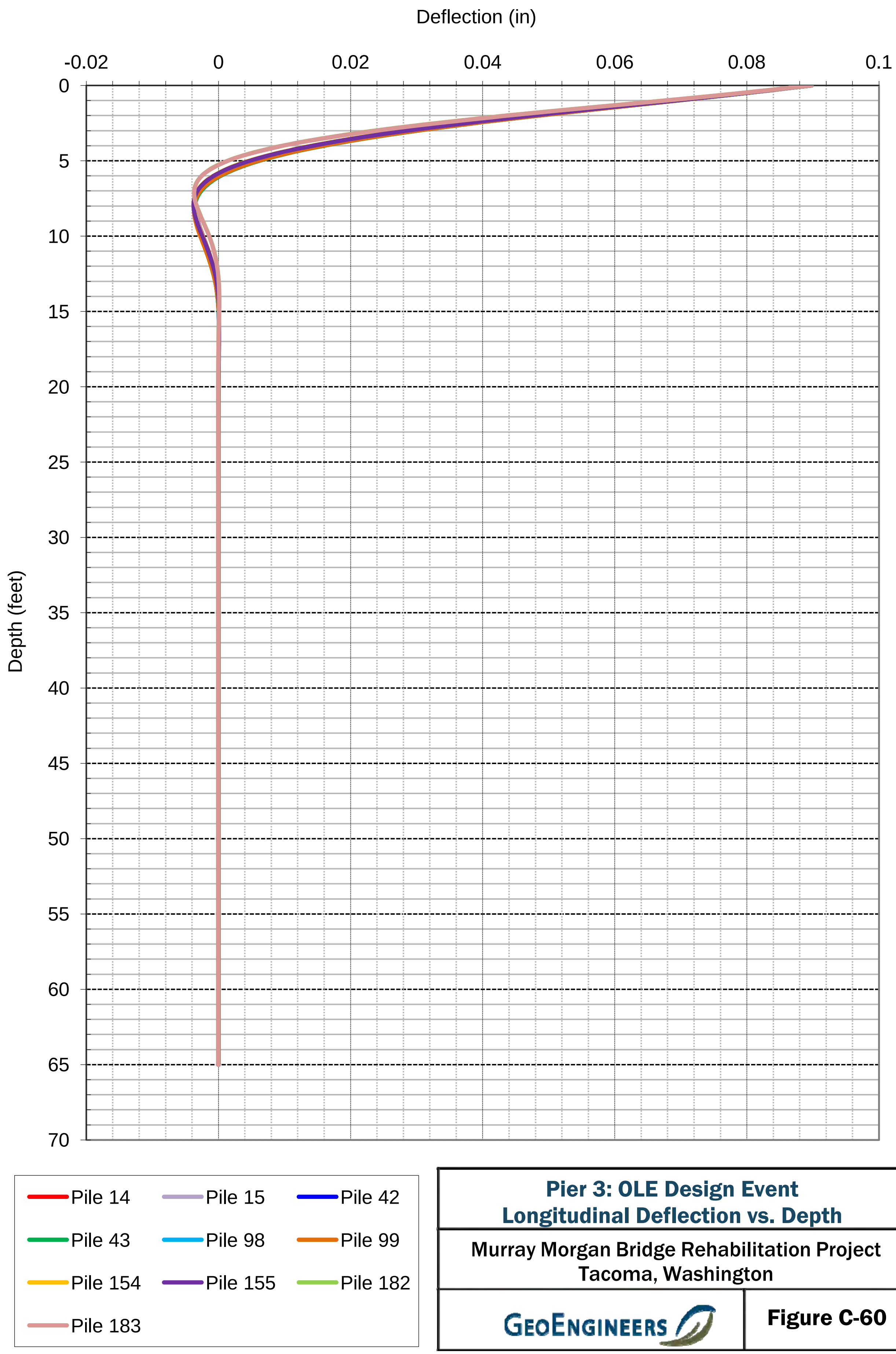


Pier 3: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-59



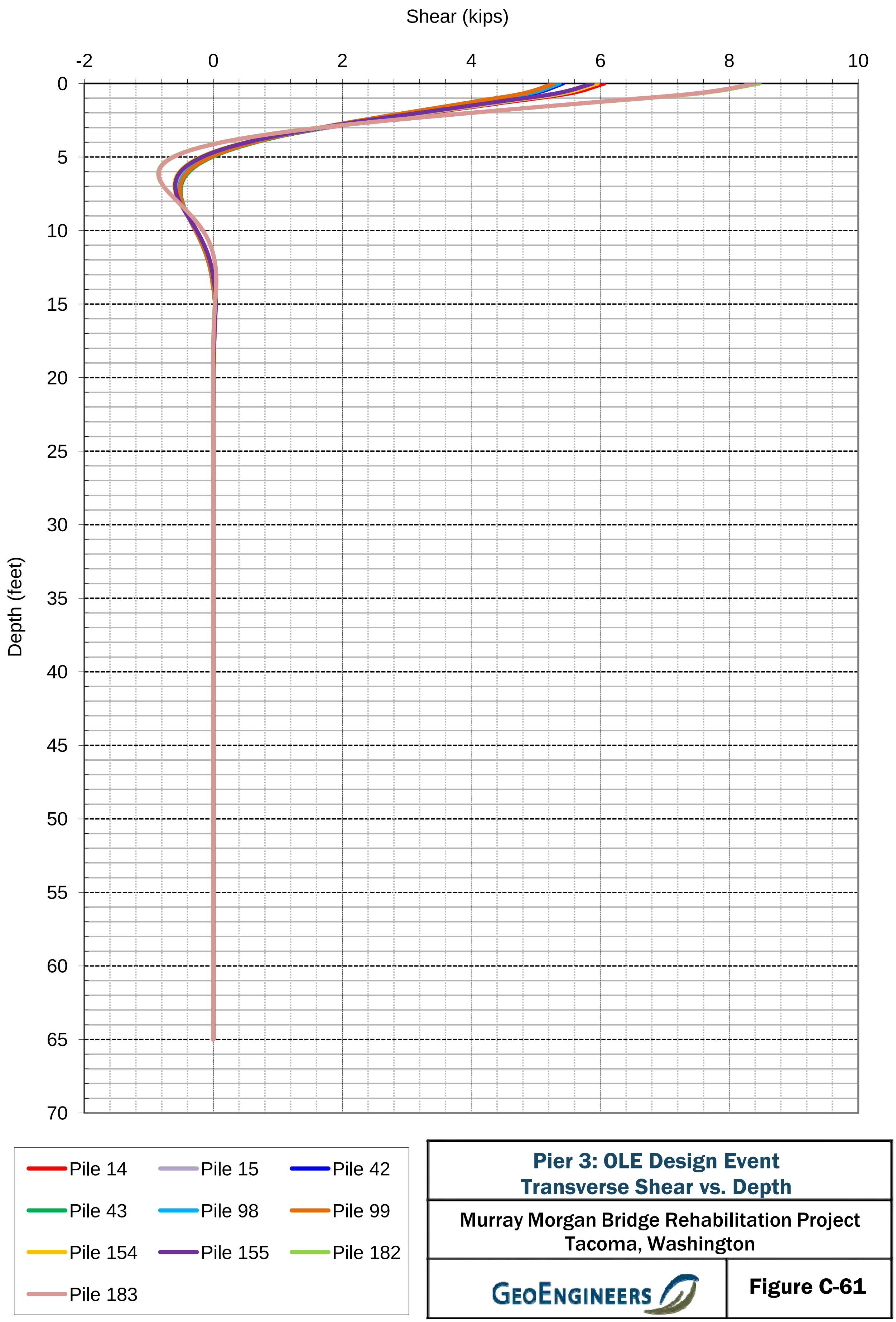
057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11

Pier 3: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-60



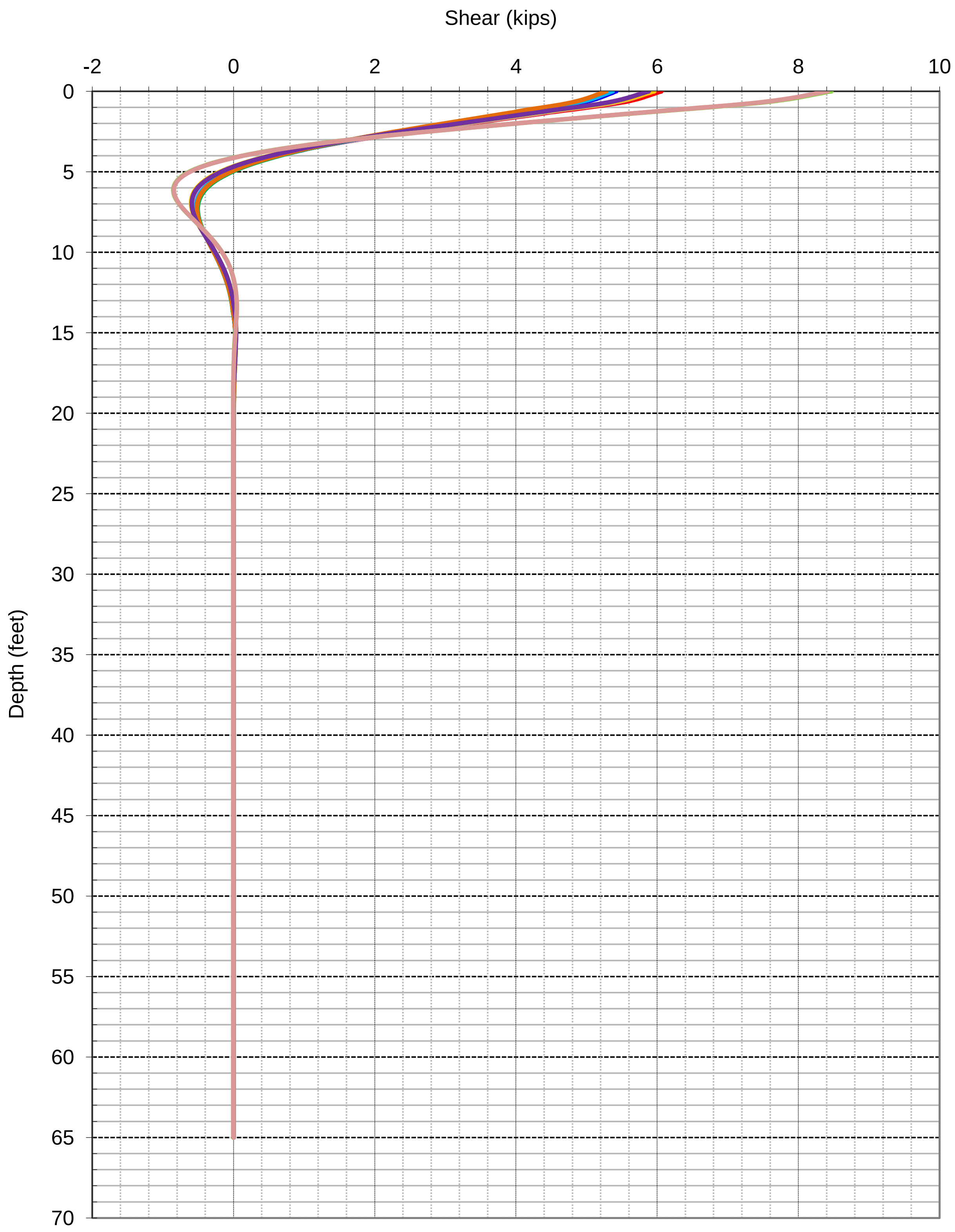
Pier 3: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-61

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11

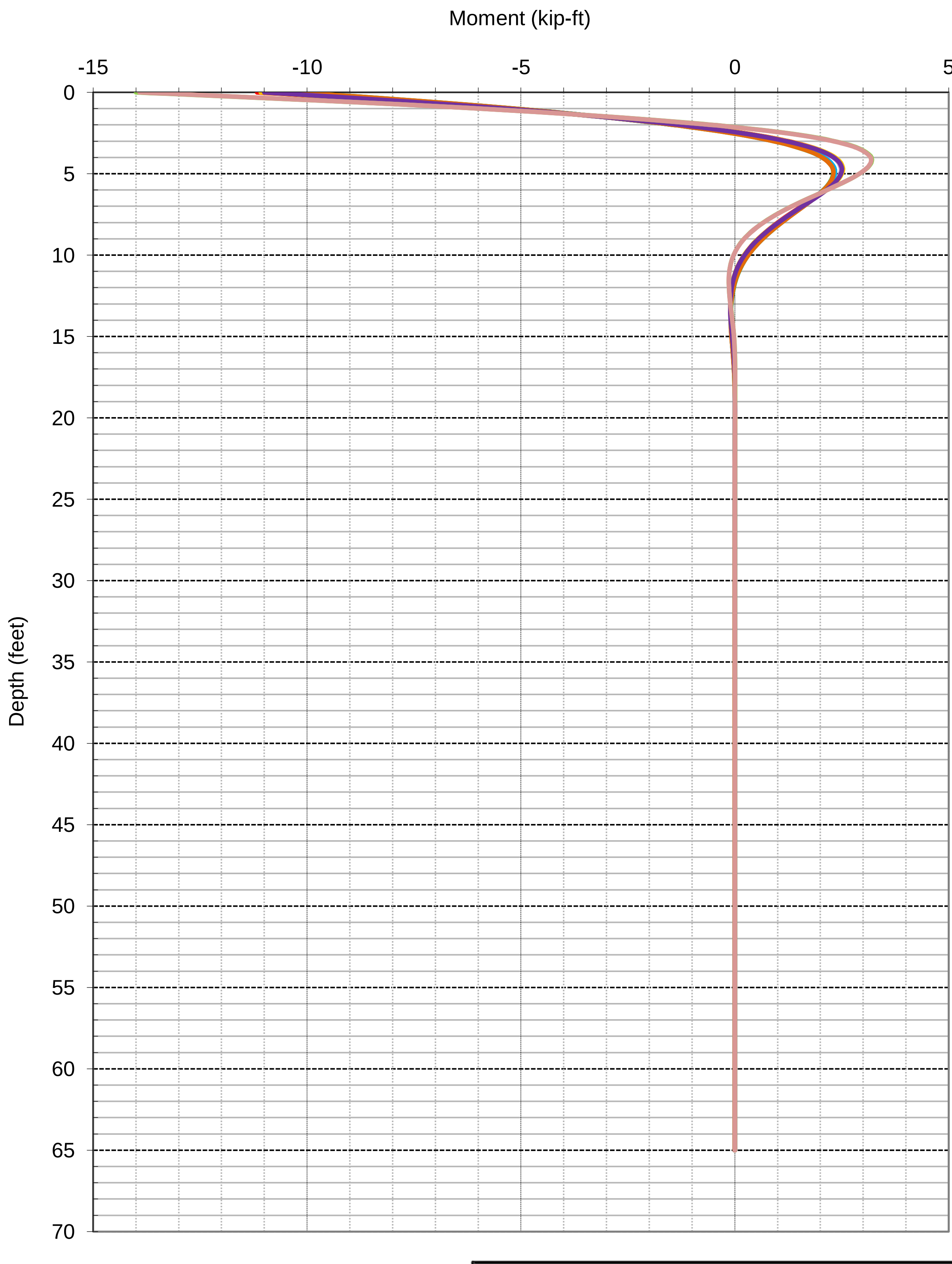


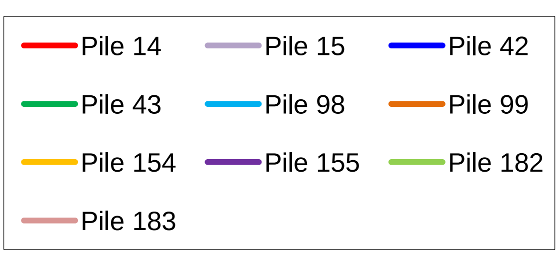
Pier 3: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-62





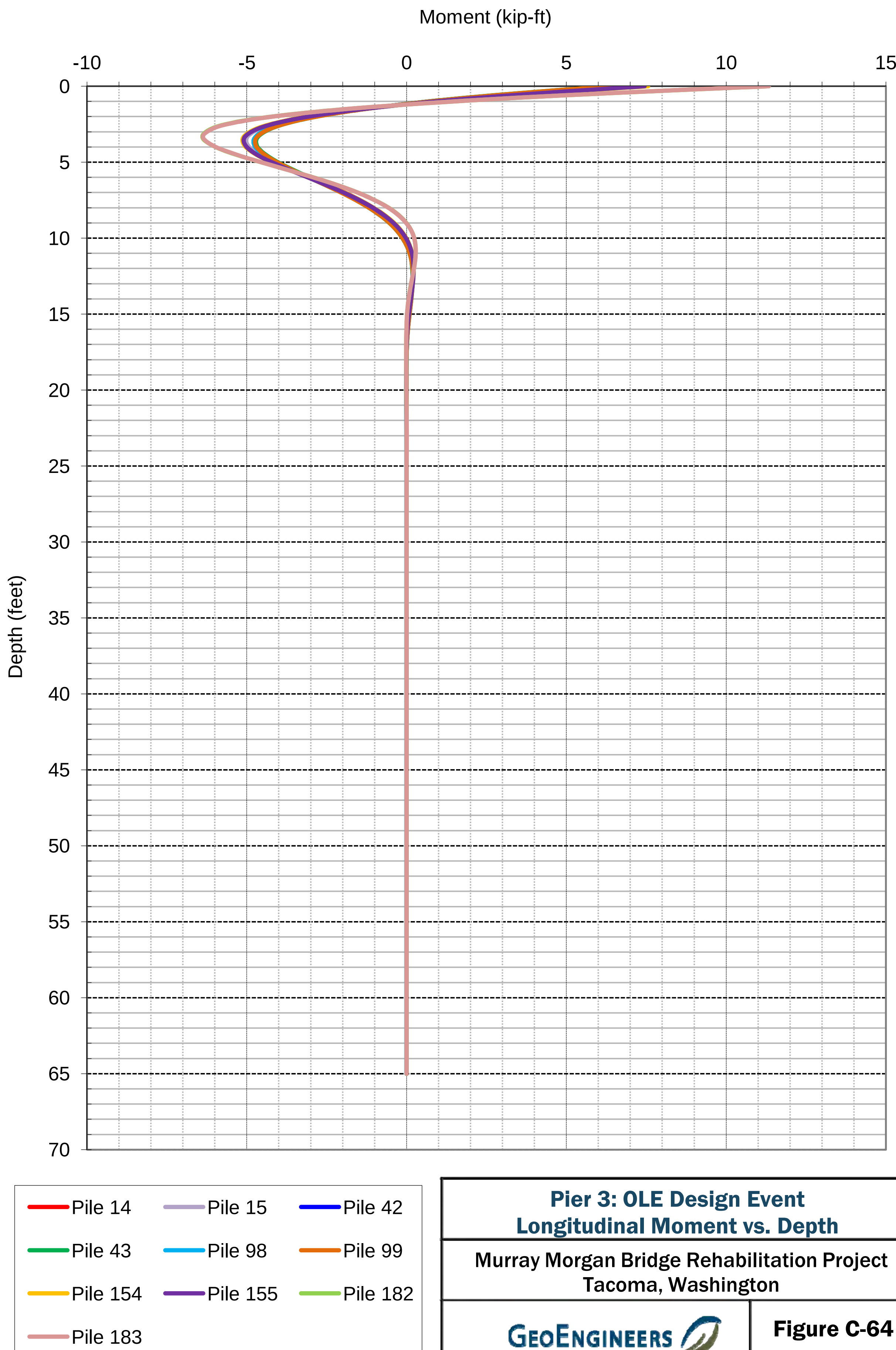
Pier 3: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-63

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



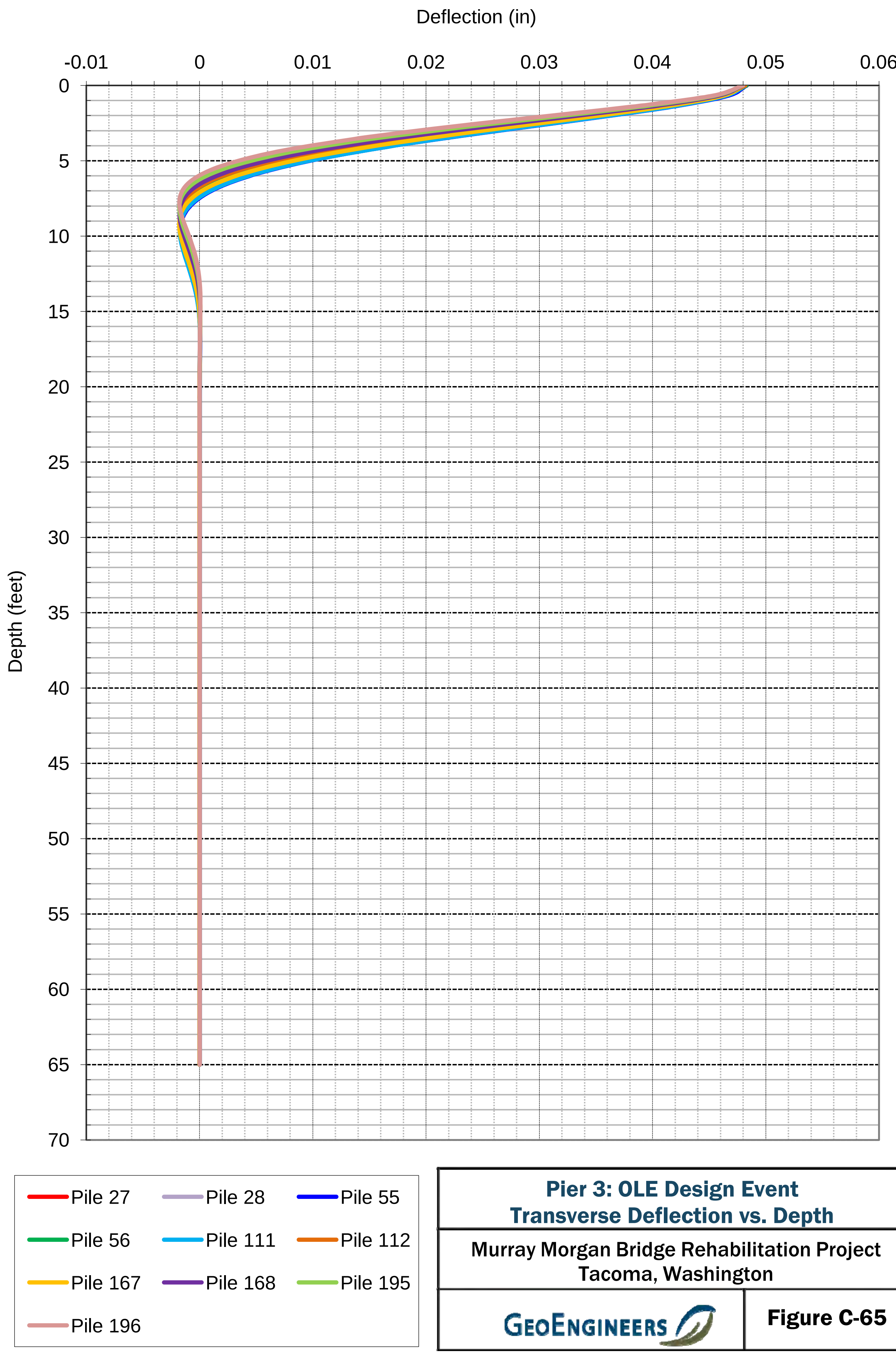
Pier 3: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-64

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



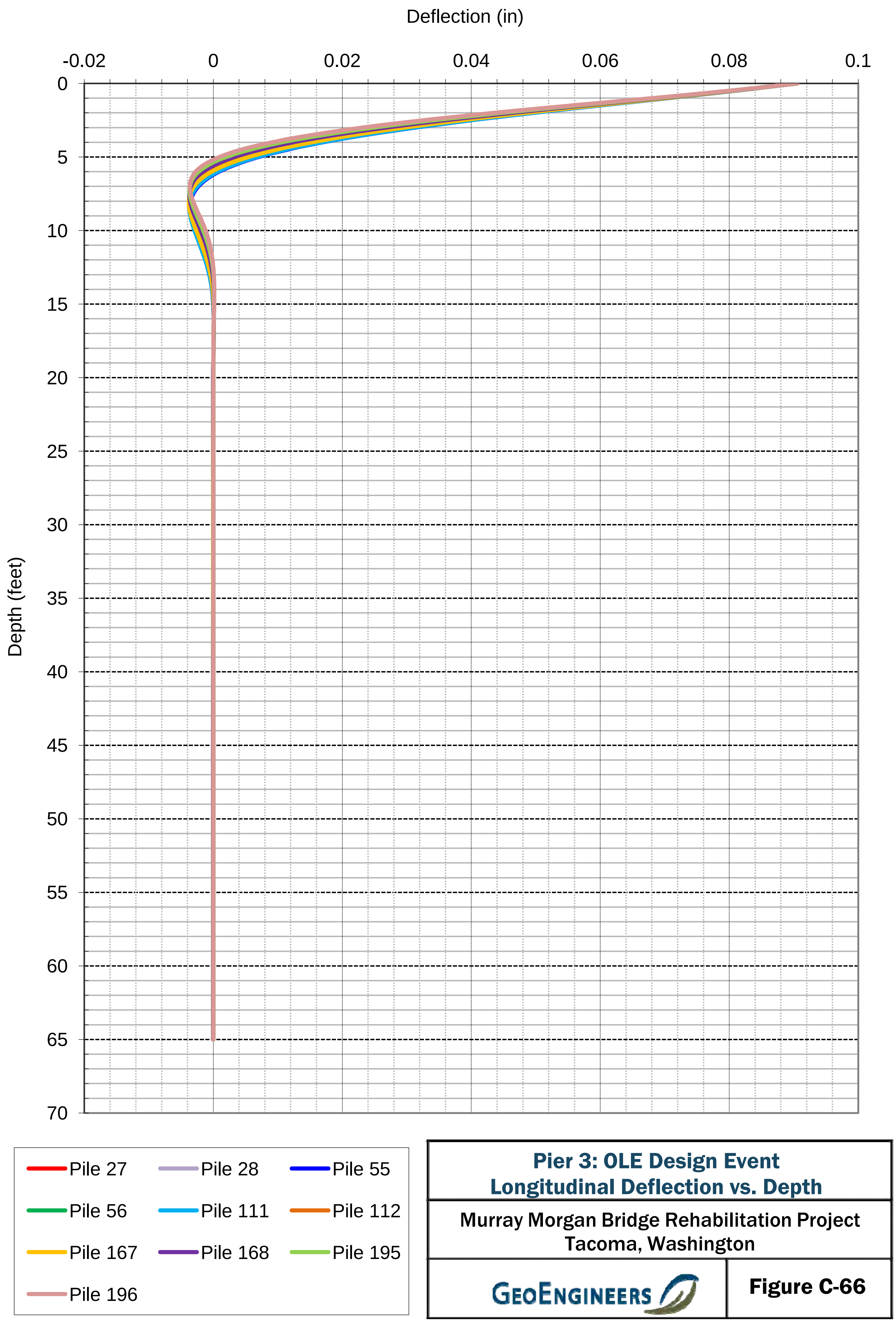
Pier 3: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-65

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



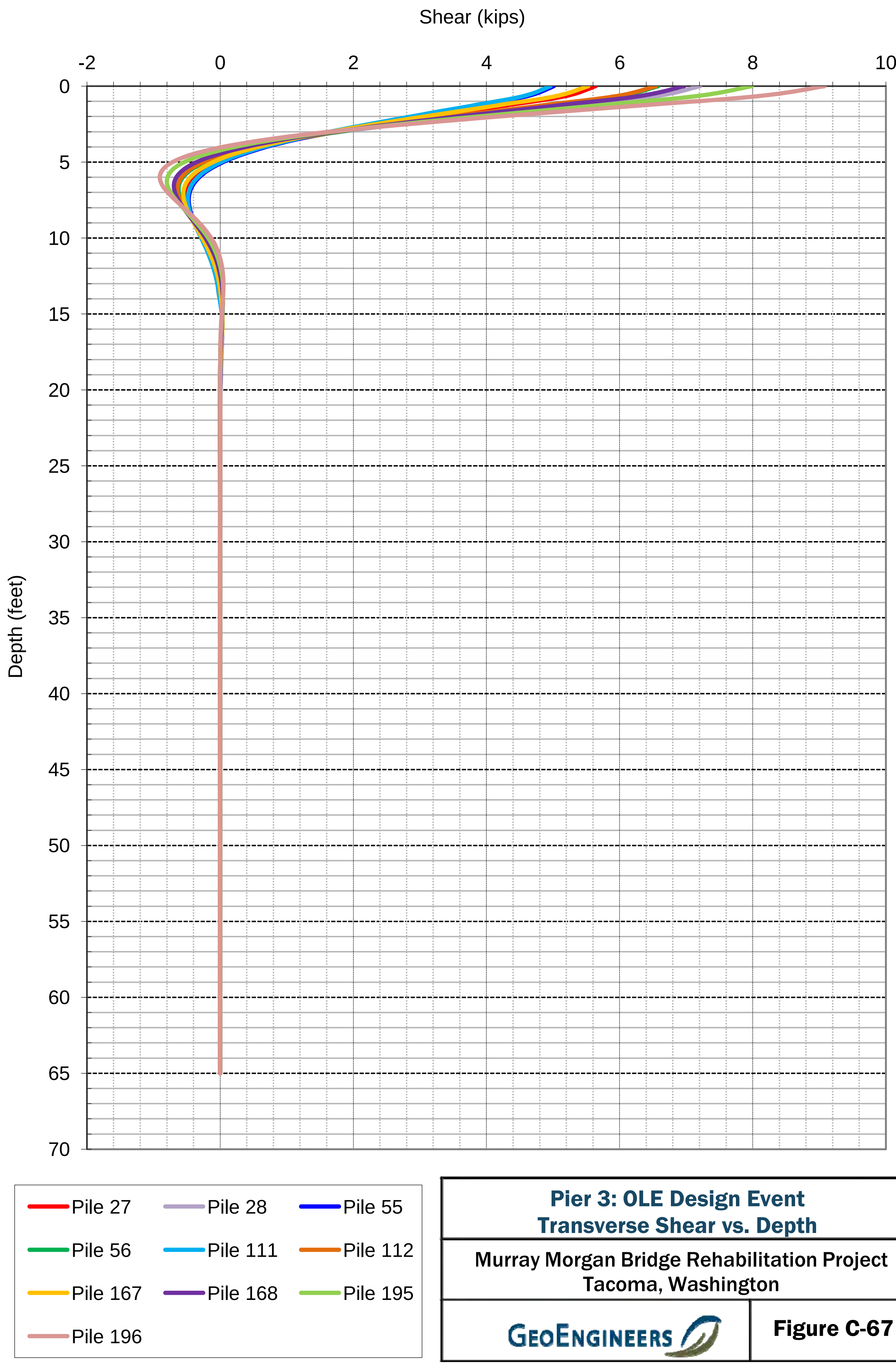
Pier 3: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-66

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



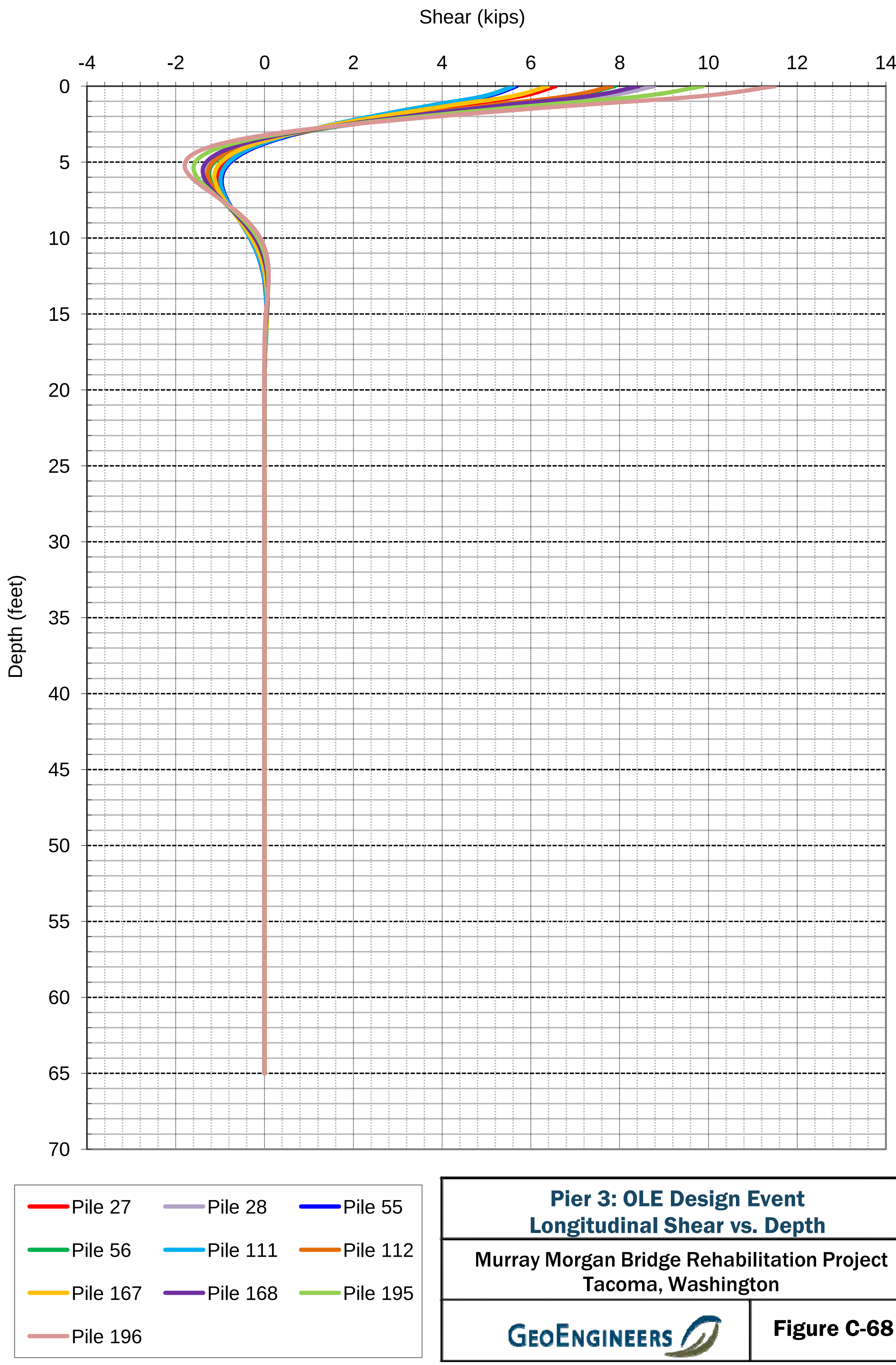
Pier 3: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-67

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11

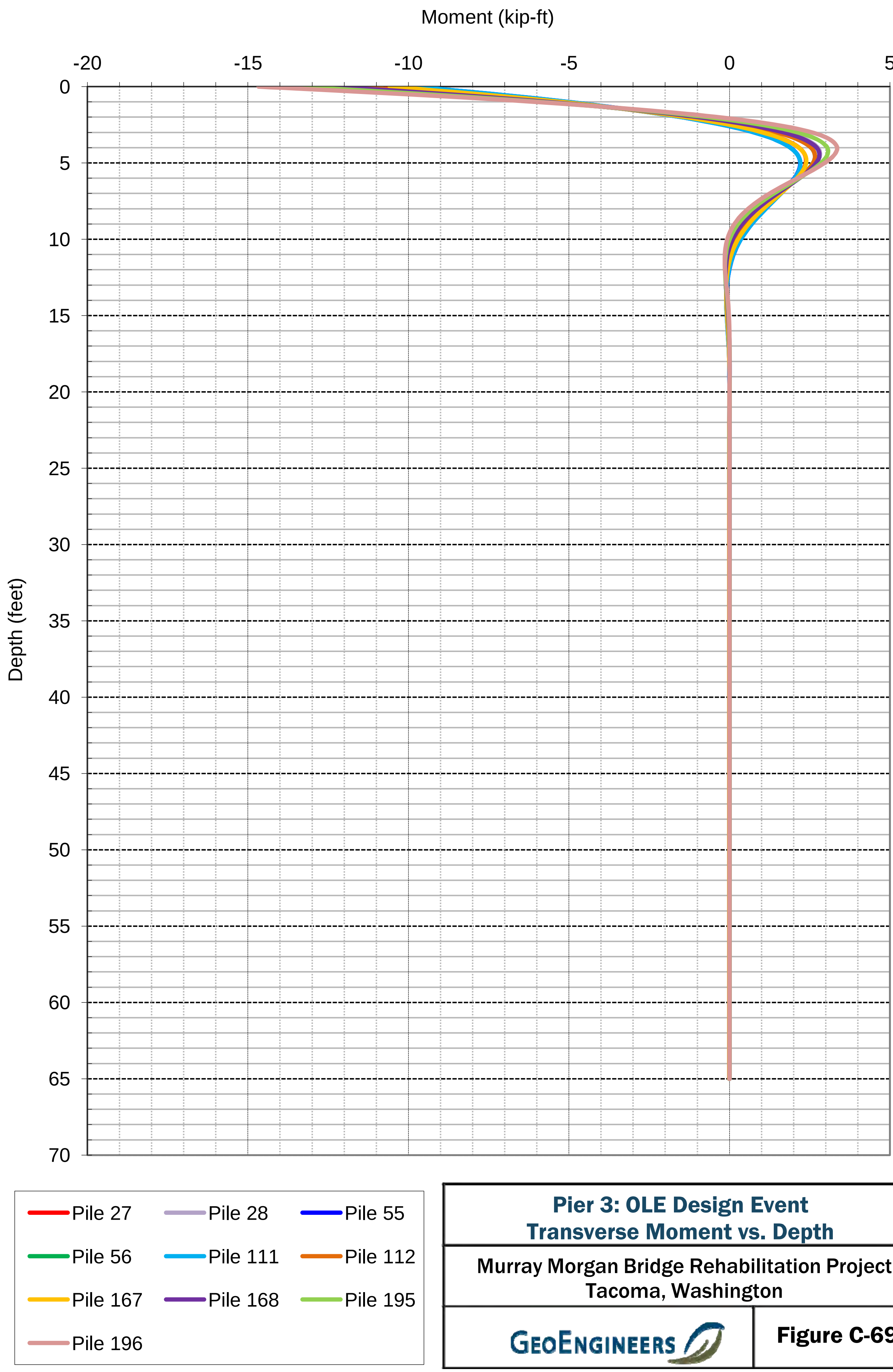


Pier 3: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-68



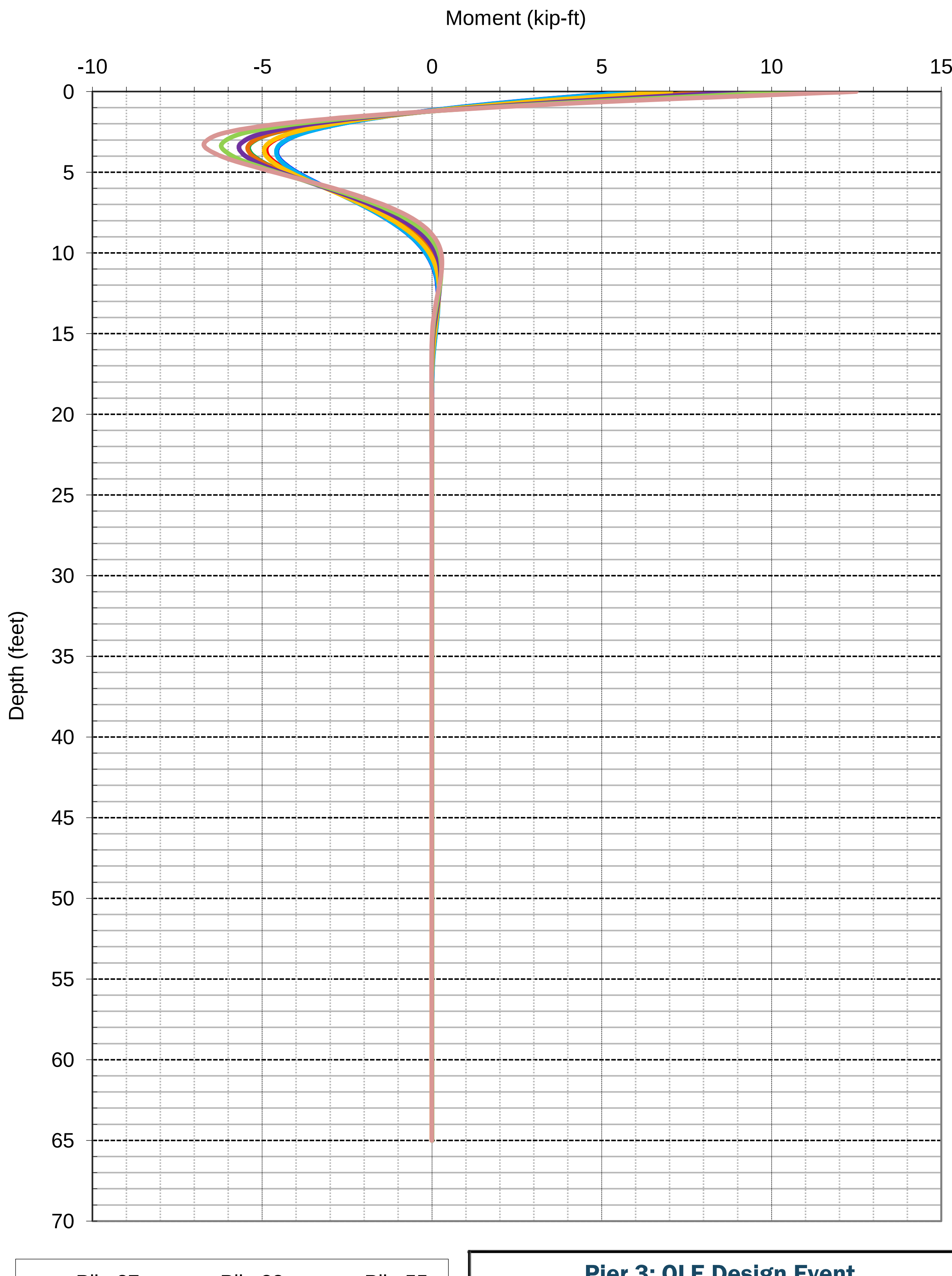
Pier 3: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-69

057010002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



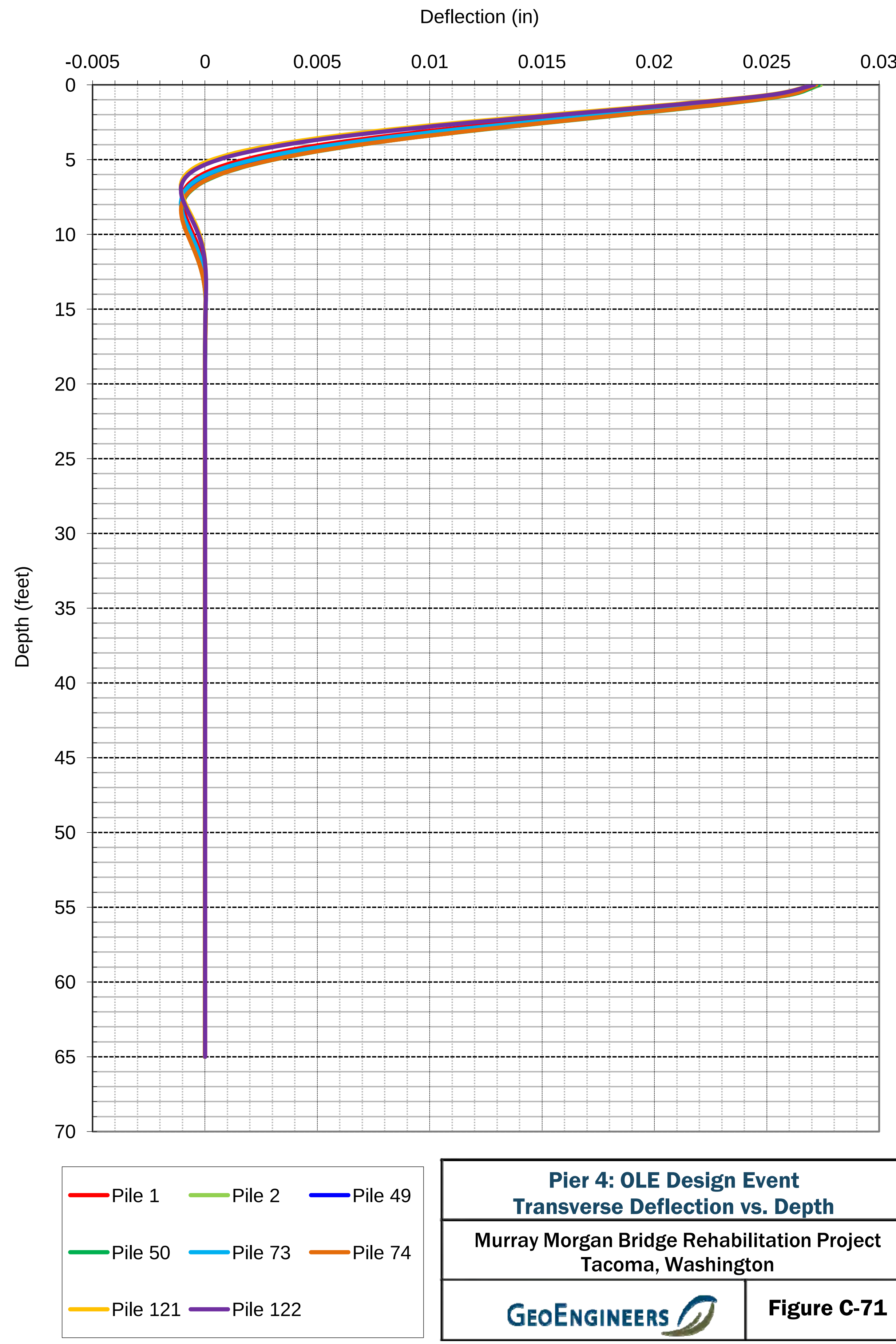
Pier 3: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-70

0570100002_FiguresC-53-C70.xlsx WLT:khc 10/21/11



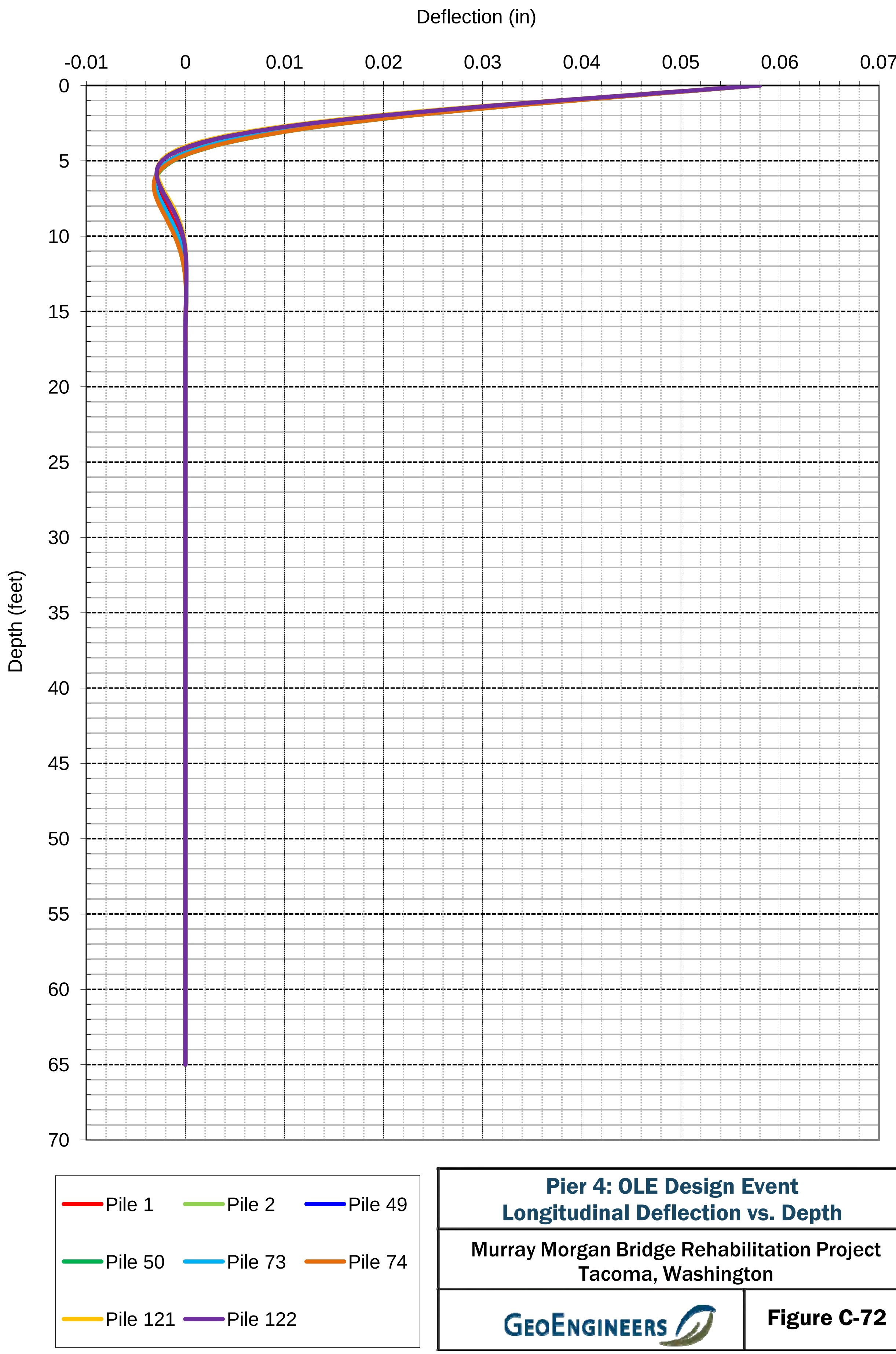
Pier 4: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-71

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



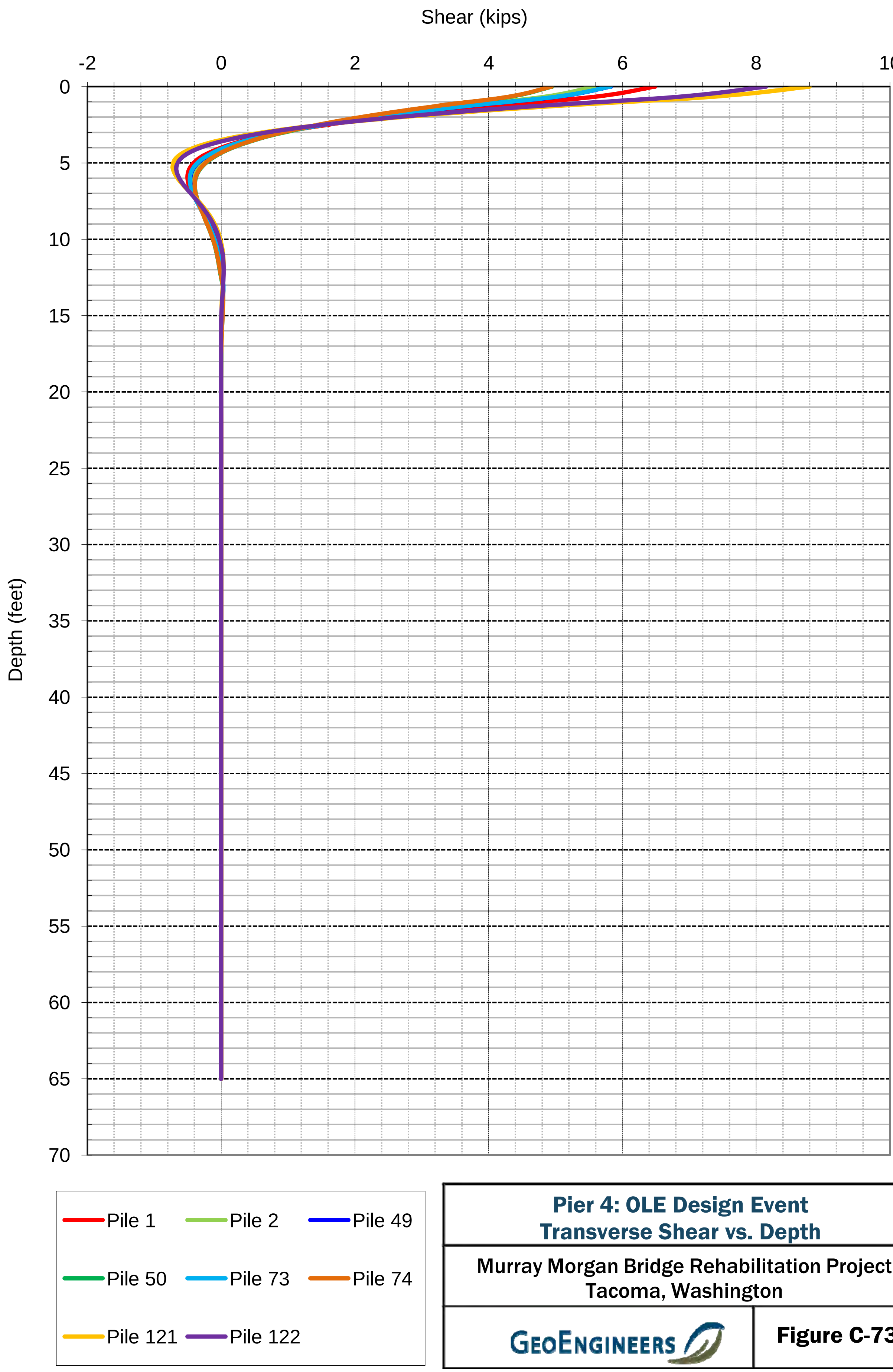
Pier 4: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-72

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



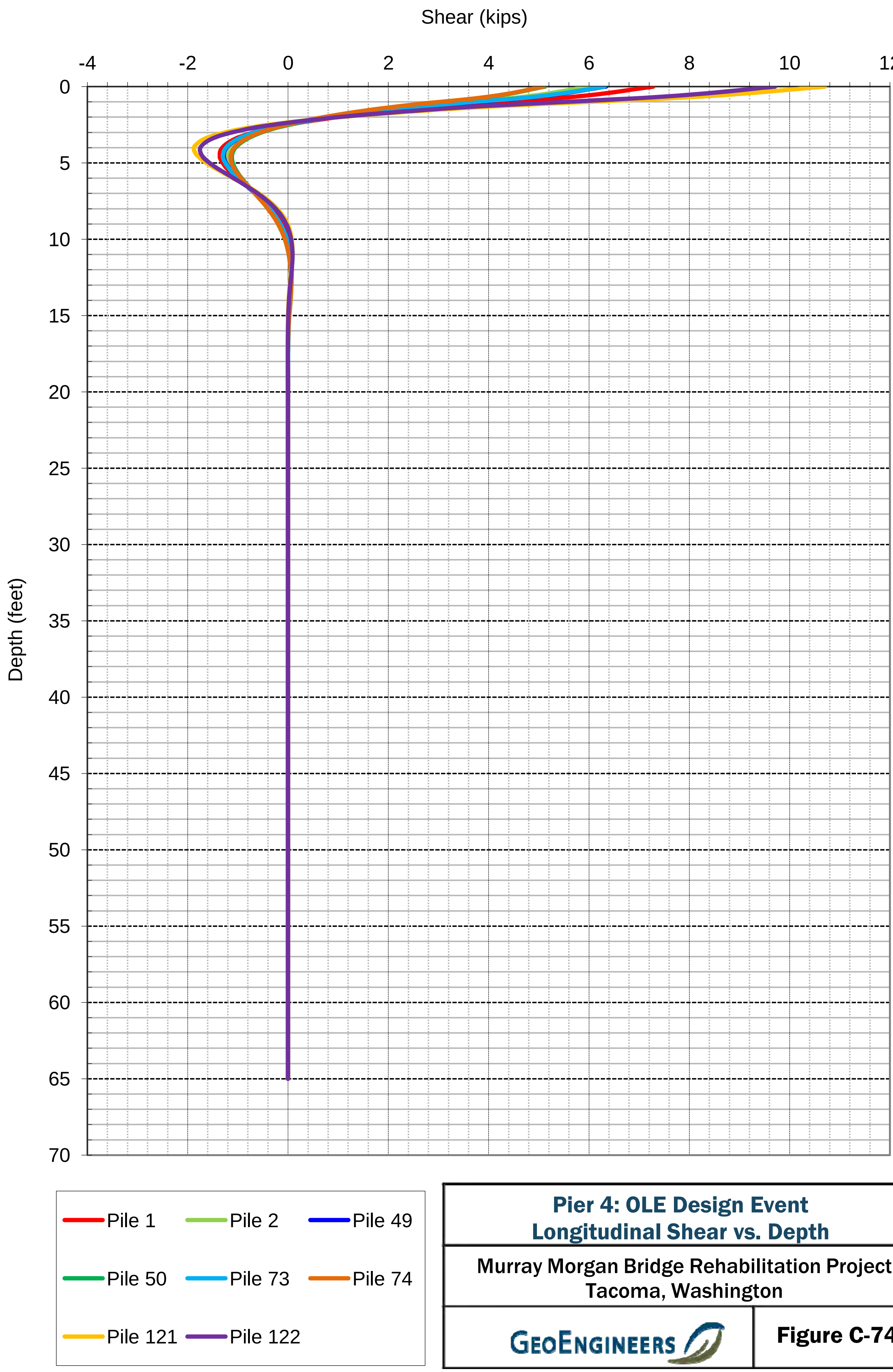
Pier 4: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-73

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



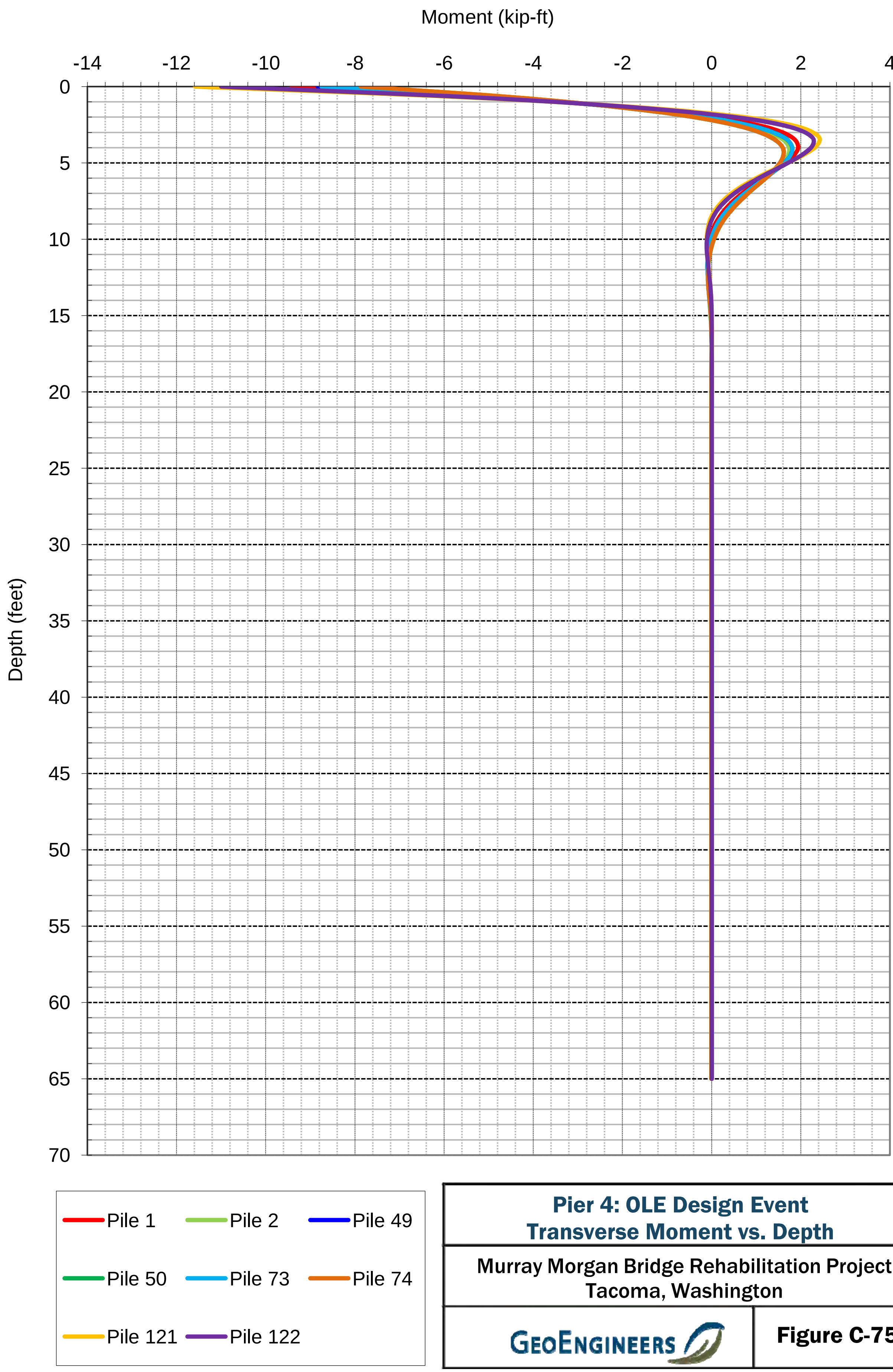
Pier 4: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-74

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



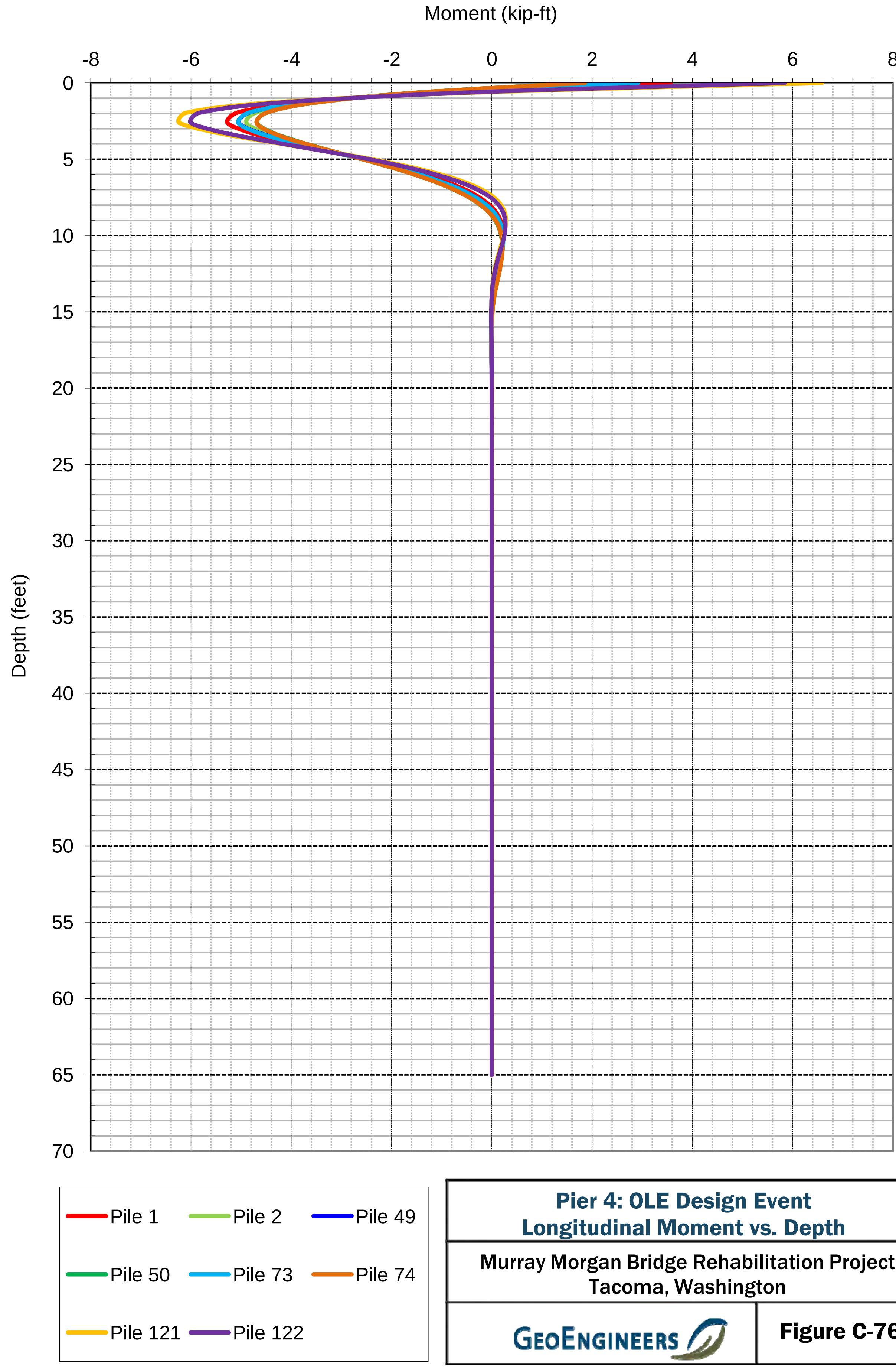
Pier 4: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-75

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



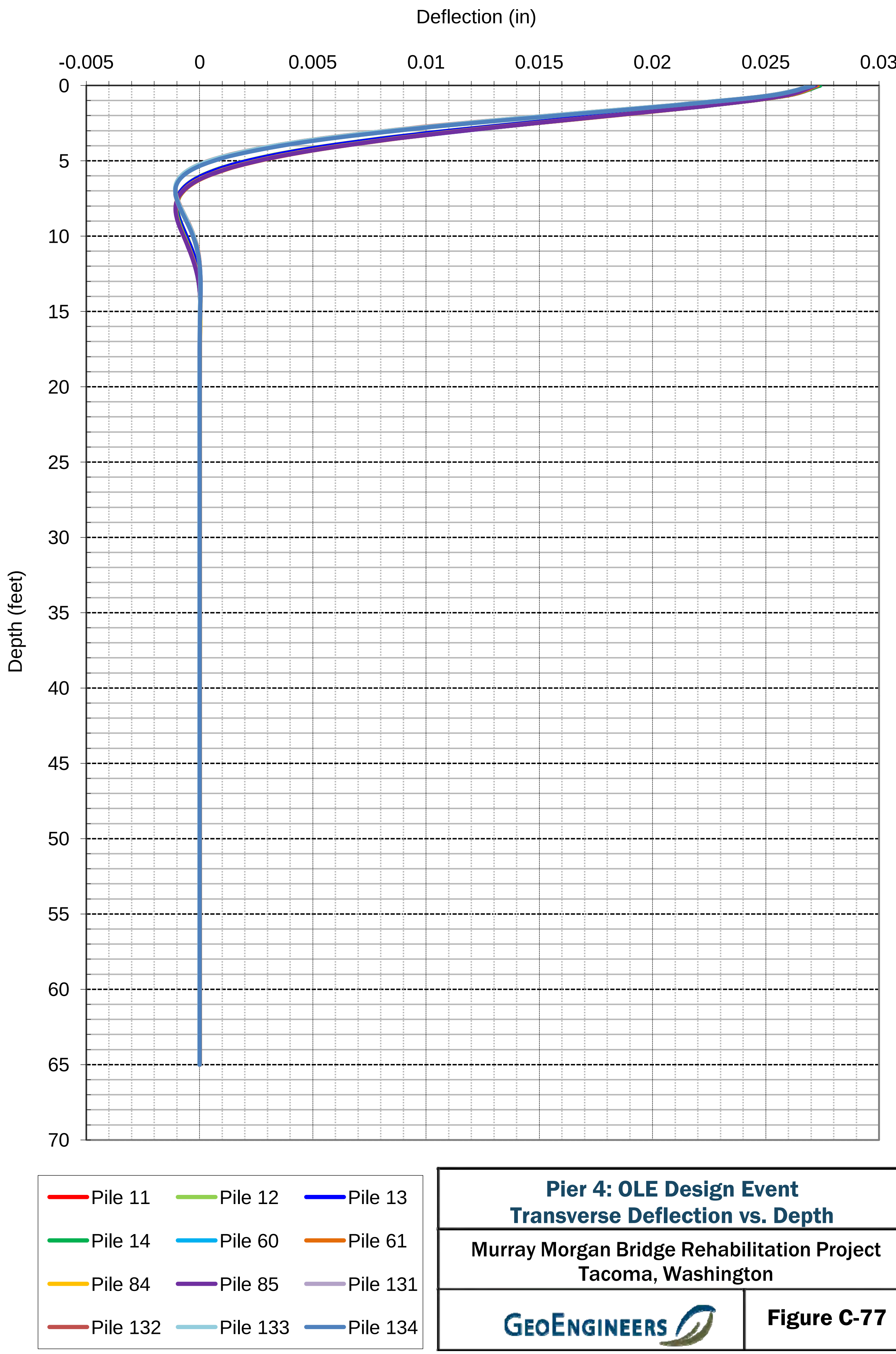
057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11

Pier 4: OLE Design Event
Longitudinal Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-76



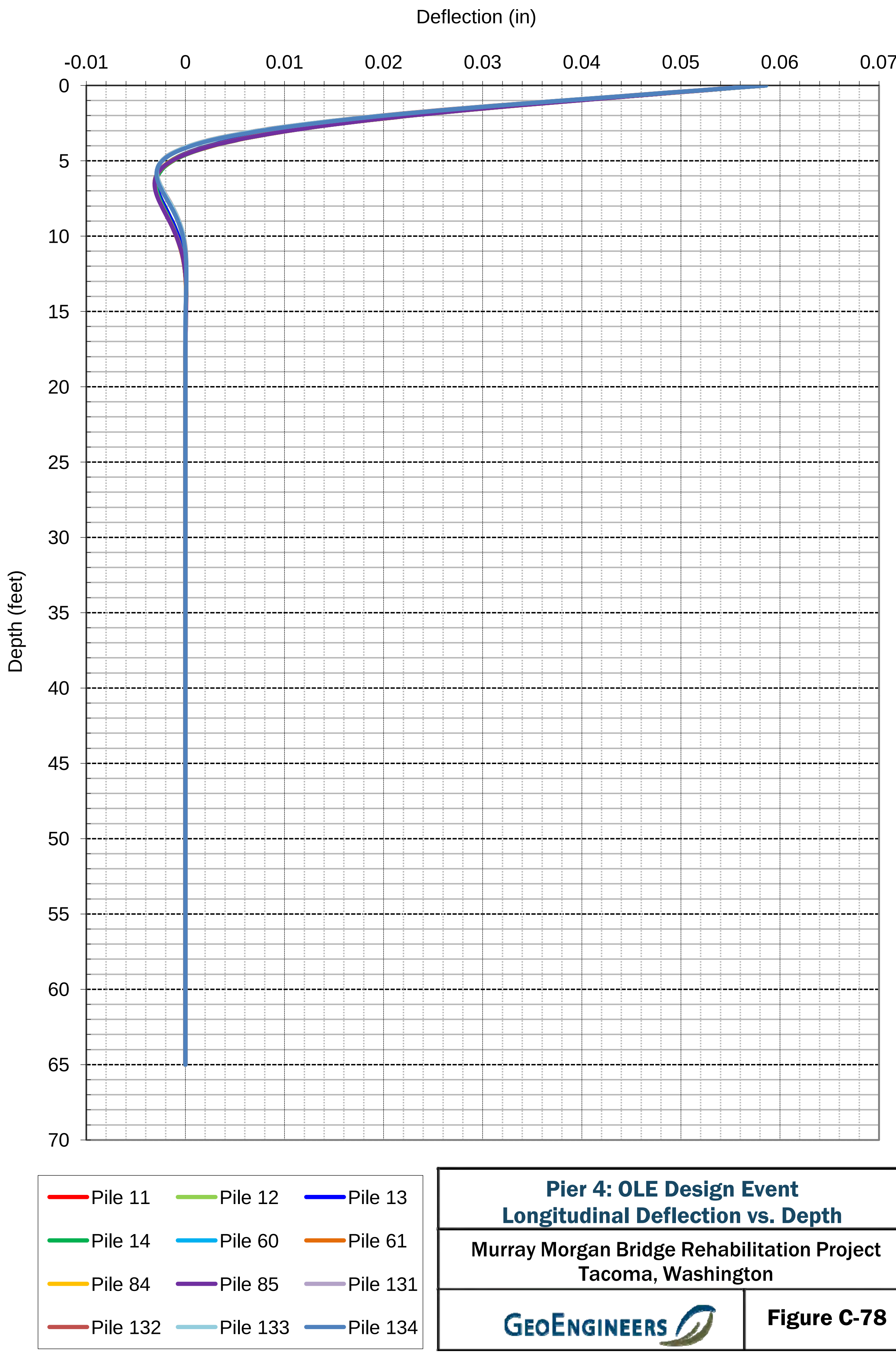
057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11

Pier 4: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-77



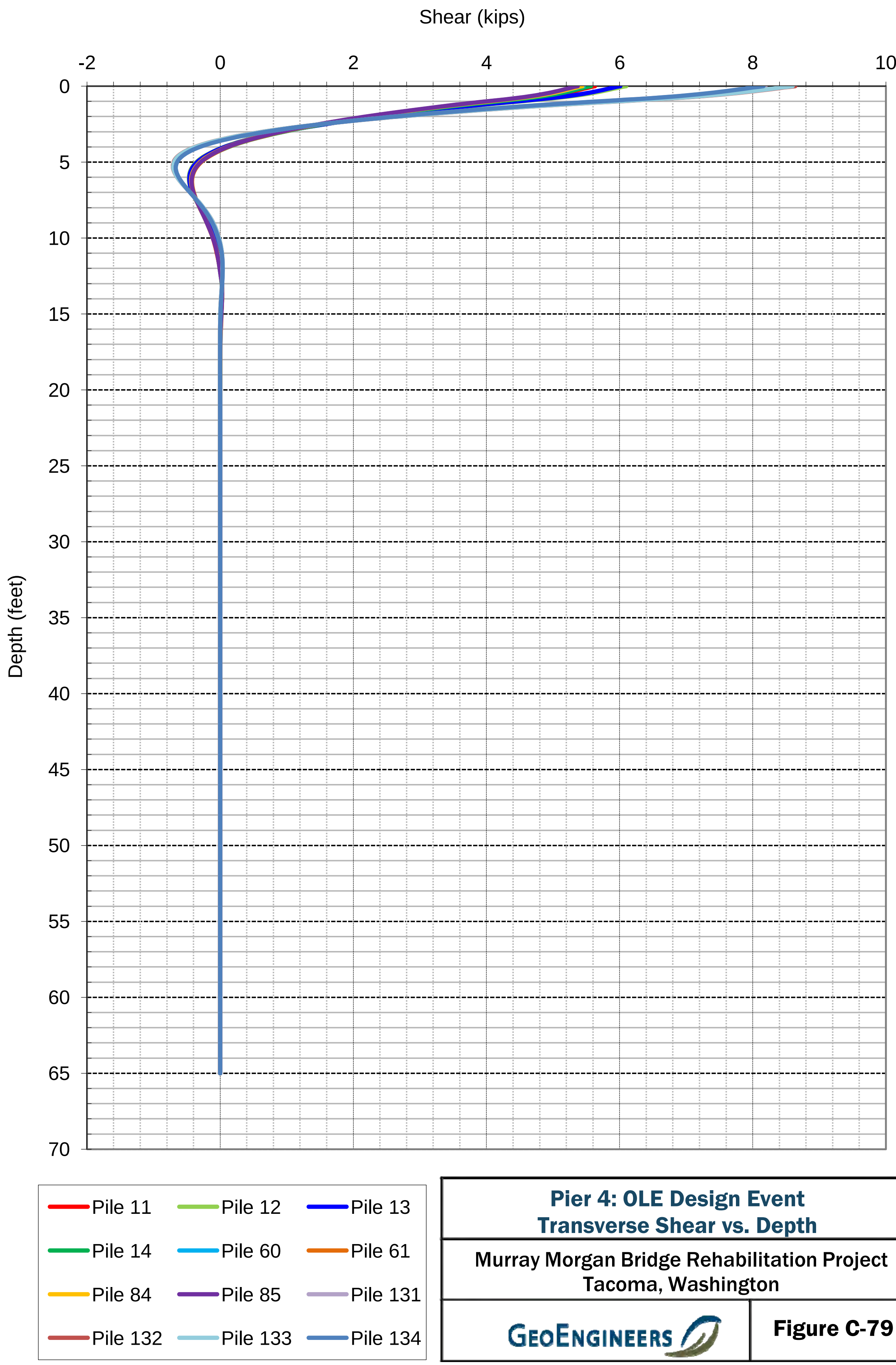
Pier 4: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-78

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



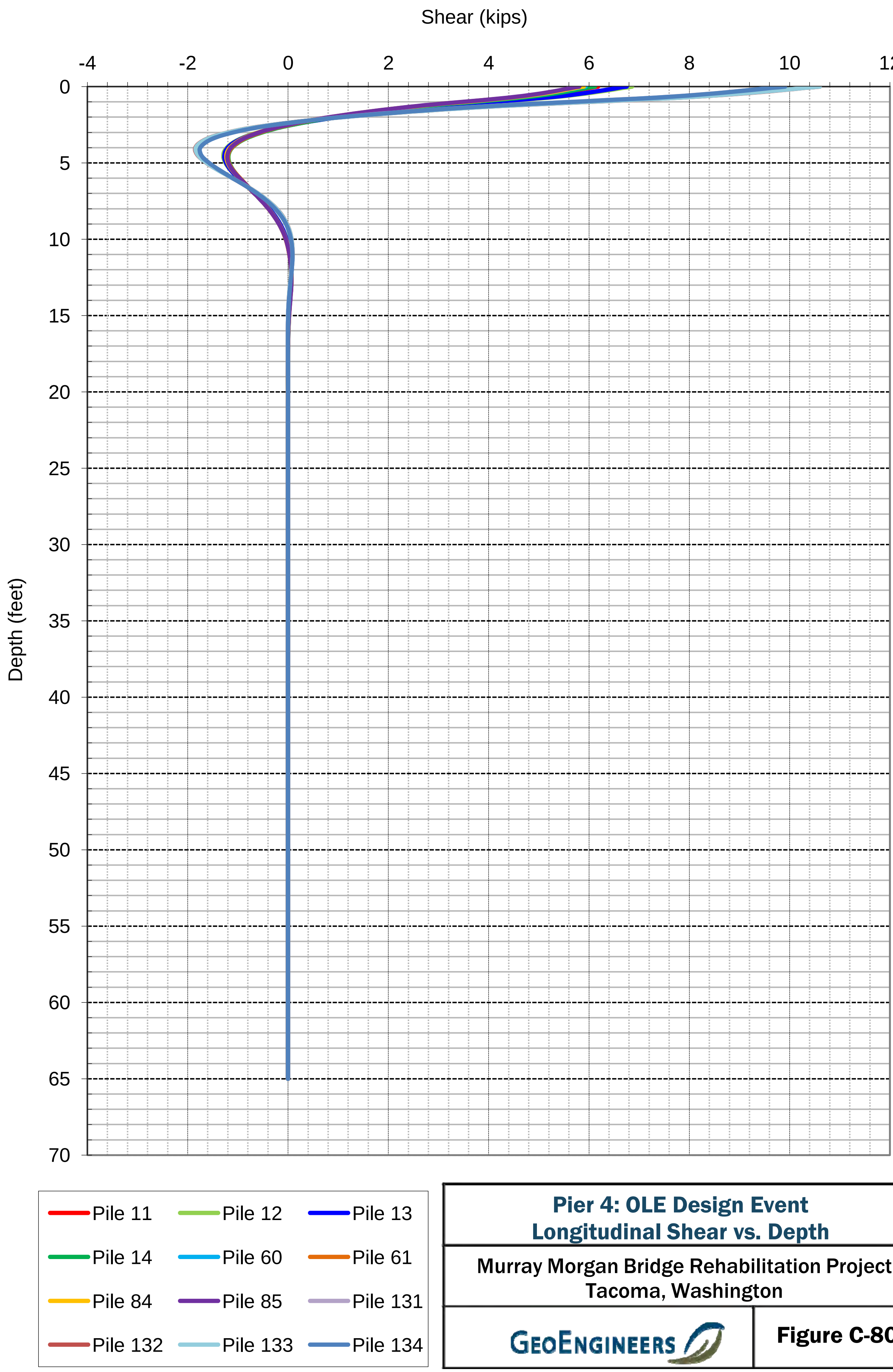
Pier 4: OLE Design Event Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GeoEngineers

Figure C-79

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



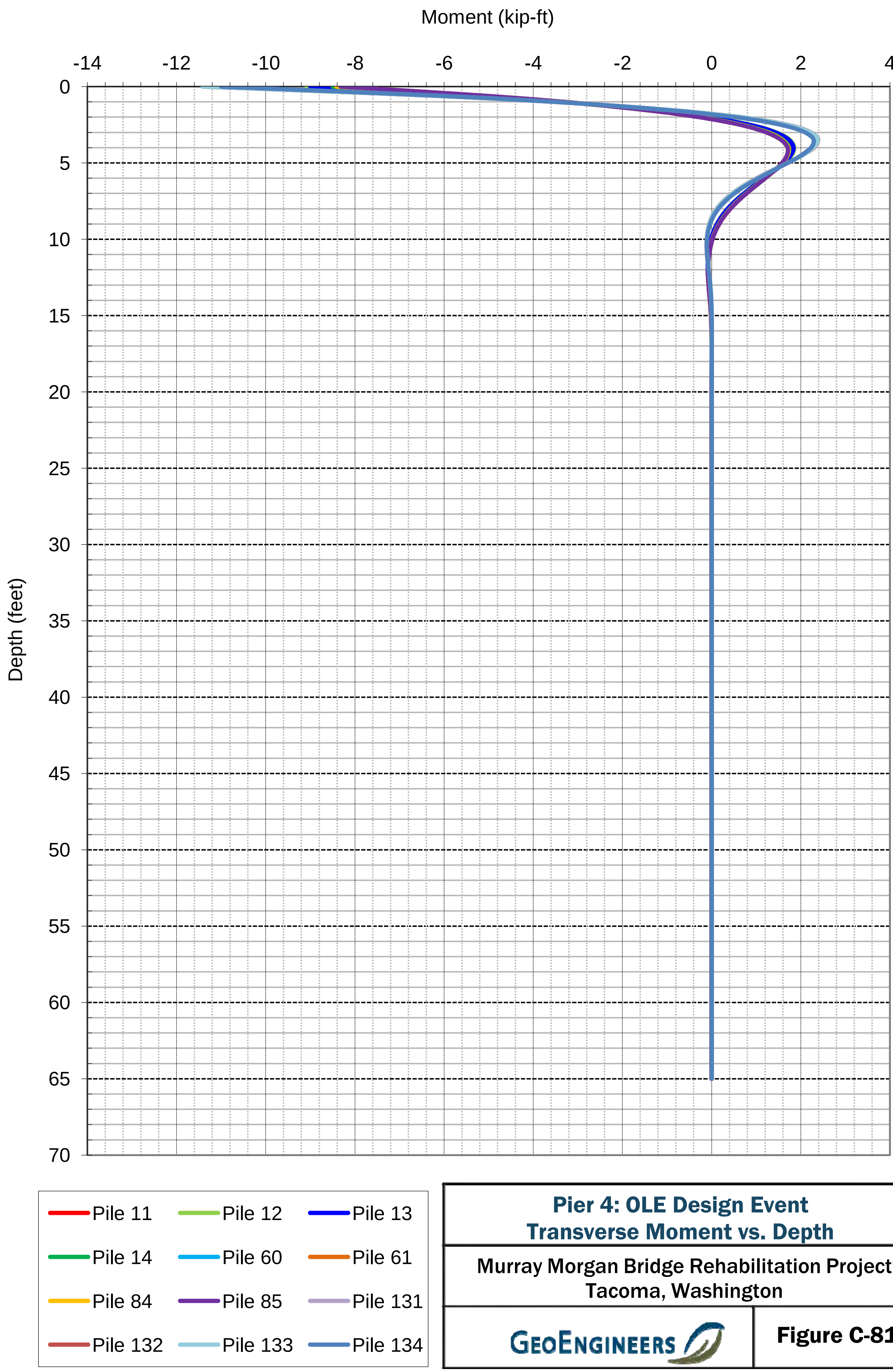
Pier 4: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-80

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



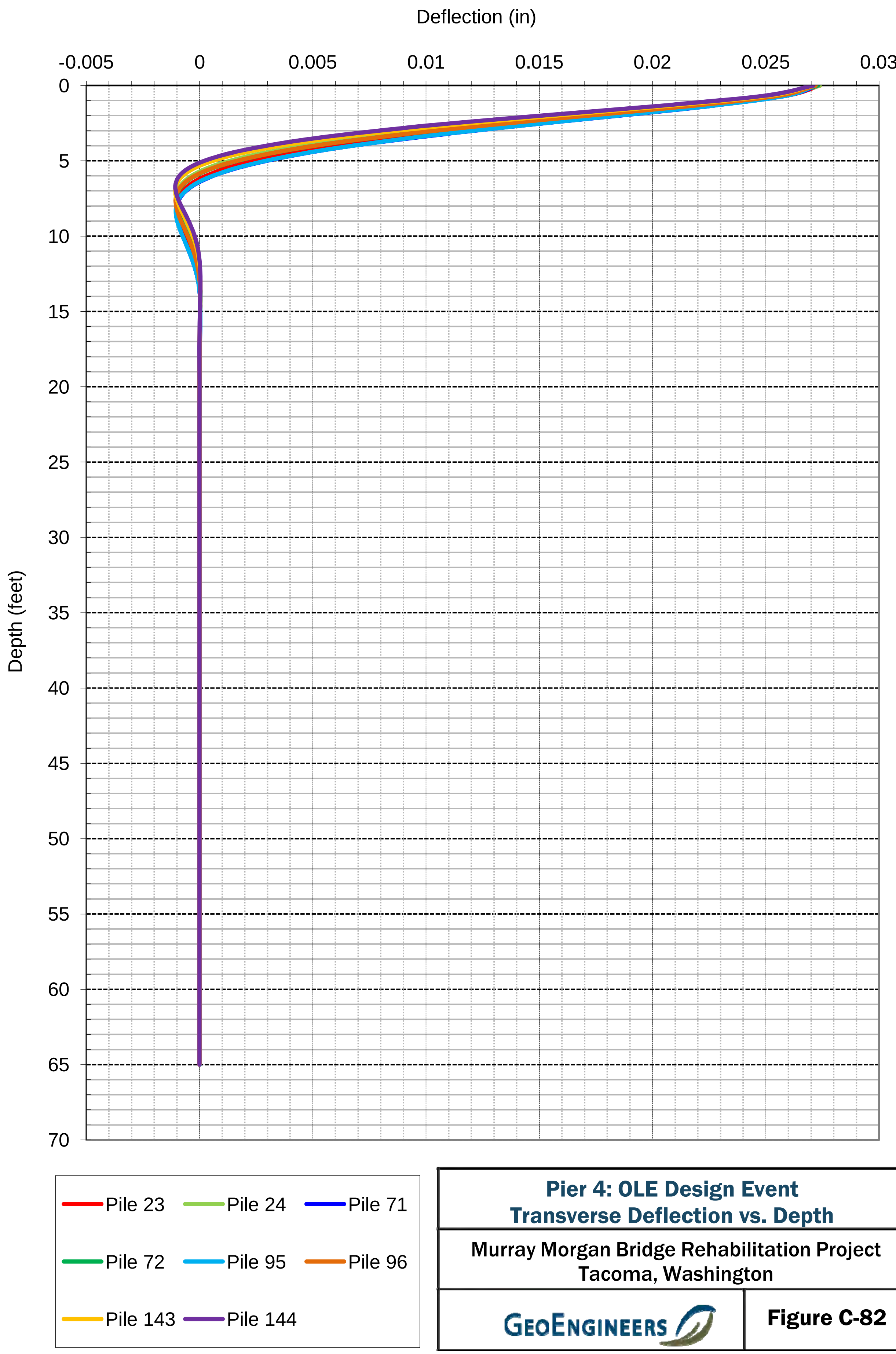
Pier 4: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-81

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11

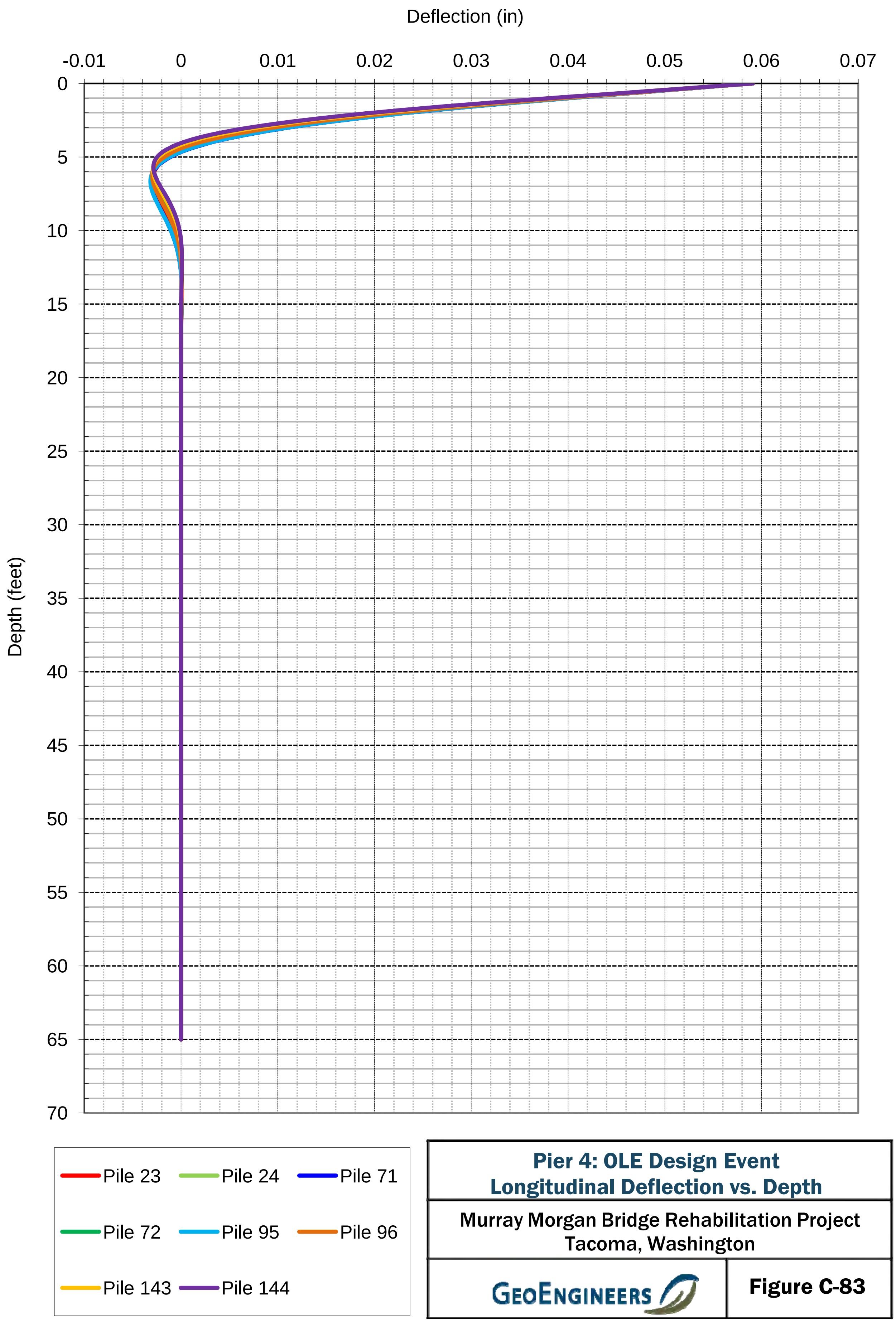


Pier 4: OLE Design Event
Transverse Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-82



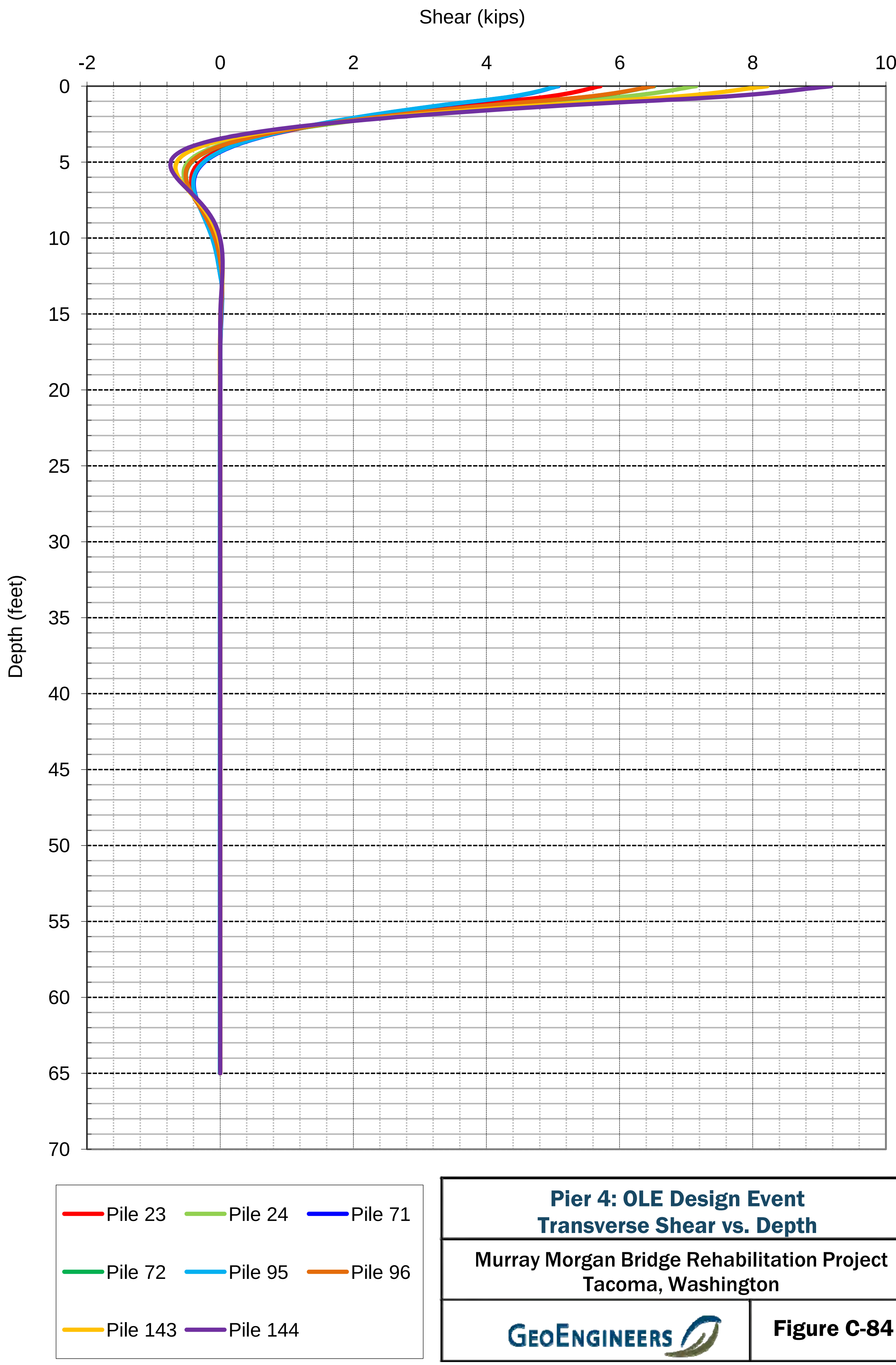
Pier 4: OLE Design Event
Longitudinal Deflection vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-83

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11



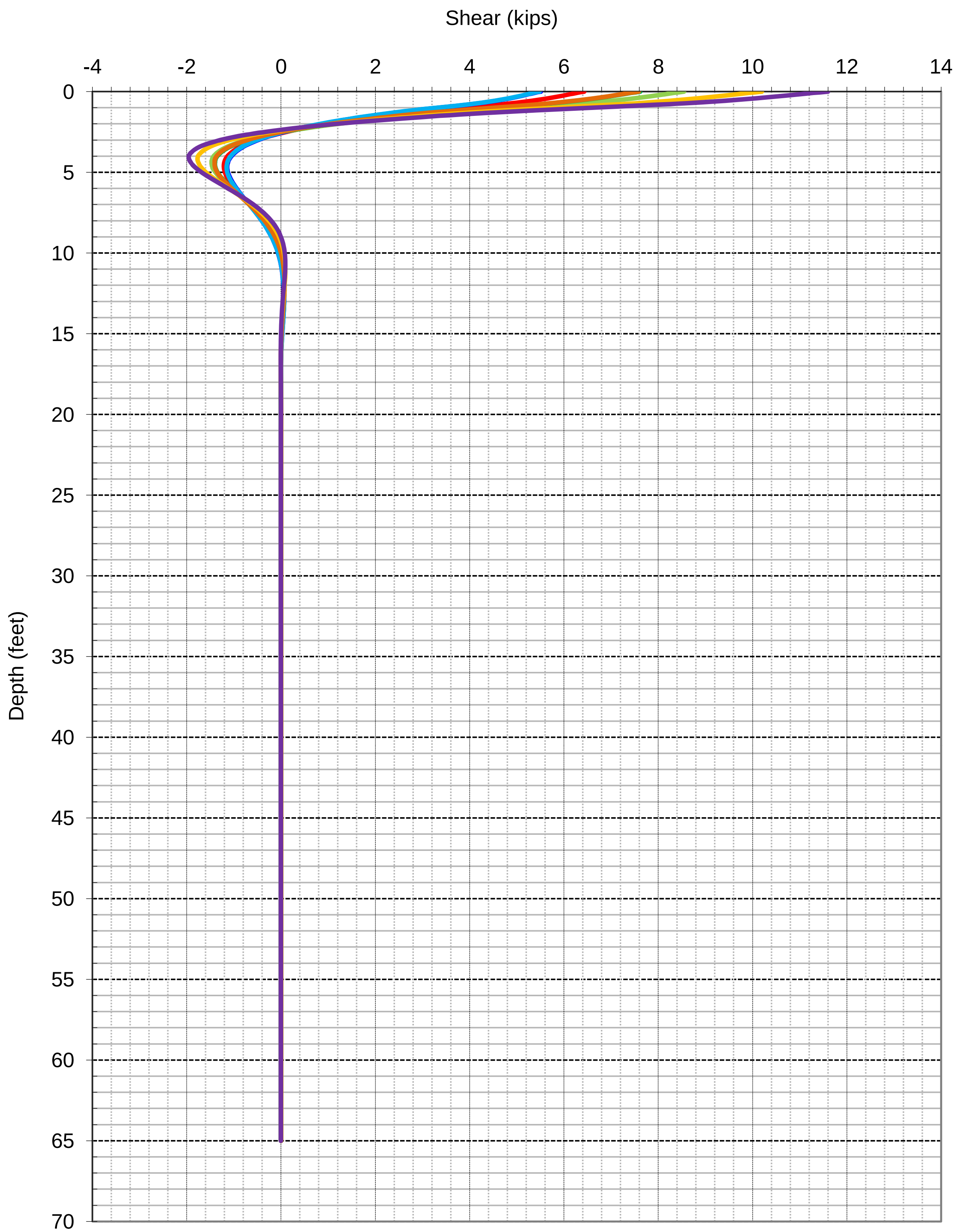
Pier 4: OLE Design Event
Transverse Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-84

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11

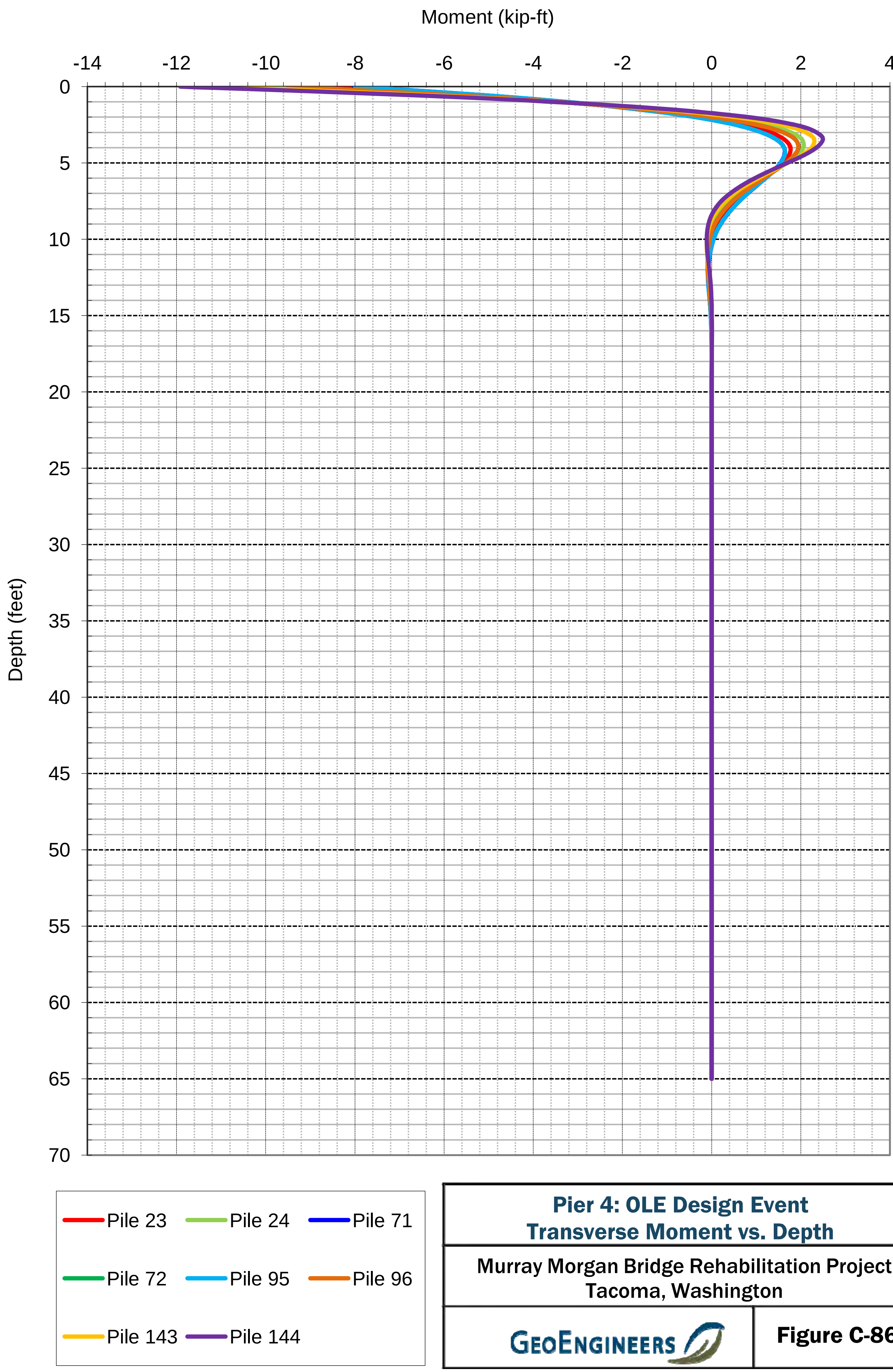


Pier 4: OLE Design Event
Longitudinal Shear vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

GEOENGINEERS

Figure C-85



Pier 4: OLE Design Event
Transverse Moment vs. Depth

Murray Morgan Bridge Rehabilitation Project
Tacoma, Washington

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Figure C-86

057010002_Figures C-71-C86.xlsx WLT:khc 10/21/11

APPENDIX D
Report Limitations and Guidelines for Use

APPENDIX D
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of the City of Tacoma and their authorized agents. This report may be made available to other members of the design team. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This report has been prepared for the Murray Morgan Bridge Rehabilitation project in Tacoma, Washington. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- the function of the proposed structure;
- elevation, configuration, location, orientation or weight of the proposed structure;

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- composition of the design team; or
- project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Topsoil

For the purposes of this report, we consider topsoil to consist of generally fine-grained soil with an appreciable amount of organic matter based on visual examination, and to be unsuitable for direct support of the proposed improvements. However, the organic content and other mineralogical and gradational characteristics used to evaluate the suitability of soil for use in landscaping and agricultural purposes was not determined, nor considered in our analyses. Therefore, the information and recommendations in this report, and our logs and descriptions should not be used as a basis for estimating the volume of topsoil available for such purposes.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor’s procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory “limitations” provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these “Report Limitations and Guidelines for Use” apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers’ Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of biological pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of biological pollutants and no conclusions or inferences should be drawn regarding biological pollutants, as they may relate to this project. The term “biological pollutants” includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.