Preliminary Seismic Hazard and Geotechnical Design Recommendations

Timber Trestle Asset Management Study
of 12 Terminals in the Puget Sound Area

for
Washington State Ferries

June 15, 2012

GeoEngineers
Earth Science + Technology

Preliminary Seismic Hazard and Geotechnical Design Recommendations

Timber Trestle Asset Management Study
of 12 Terminals in the Puget Sound Area

for
Washington State Ferries

June 15, 2012

GeoEngineers

8410 154th Avenue NE
Redmond, Washington 98052
425.861.6000

Preliminary Seismic Hazard and Geotechnical Design Recommendations

Timber Trestle Asset Management Study 12 Terminals in the Puget Sound Area

File No. 0180-284-00

June 15, 2012

Prepared for:

Washington State Ferries
2901 3rd Avenue, Suite 500
Seattle, Washington 98121

Attention: Nicole McIntosh

Prepared by:

GeoEngineers, Inc.
8410 154th Avenue NE
Redmond, Washington 98052
425.861.6000

Heidi Disla

Heidi P. Disla, EIT
Geotechnical Engineer

King H. Chin, PE
Associate

KING H. CHIN
STATE OF WASHINGTON
38761
REGISTERED
PROFESSIONAL ENGINEER

6/15/2012

David A. Cook, LG, CPG
Principal

HPD:KHC:nlu

GEOENGINEERS

Table of Contents

LIST OF TABLES

LIST OF FIGURES

Table of Contents (continued)

APPENDICES

INTRODUCTION

This report presents a summary of our preliminary geotechnical design recommendations and the results of our preliminary geotechnical engineering analyses completed to support the conceptual retrofit design evaluation of timber trestles as part of the Timber Trestle Asset Management Study at 12 Washington State Ferries (WSF) Terminals in the Puget Sound Area. Our services were completed in general accordance with the consultant agreement No. Y-10747, Task Order AF, executed August 5, 2011.

WSF owns and operates 20 ferry terminals in the Puget Sound area. Fourteen of the 20 terminals still utilize timber trestles to load/offload vehicles to/from the ferry boats and land. The timber trestles were constructed between 1952 and 1982 (portions of the Colman Dock trestle date to 1938) and were not designed to sustain earthquakes with predicted seismic loading per modern building codes. The timber trestles have been identified to pose the highest risk to life/safety and operation of the ferry terminals in the event of an earthquake.

The Terminal Engineering Group (Terminals) at WSF has developed and is considering implementing a large trestle replacement program to reduce the seismic risk of ferry terminal operation in the Puget Sound area. The timber trestles at five terminals have been replaced. The trestles at 14 terminals have been identified as needing replacement or upgrade. Two of these terminals, Seattle and Eagle Harbor, have been programmed to be upgraded and were not included as part of this study. The remaining 12 terminals that were part of this study, include (and are generally listed based on their geographic position from north Puget Sound to South Puget Sound): Friday Harbor on San Juan Island, Lopez Island, Shaw Island, Orcas Island, Anacortes, Mukilteo, Edmonds, Fauntleroy, Vashon Island, Southworth, Tahlequah and Point Defiance.

In keeping with their asset management program, WSF's goal was to evaluate the return on investment of the trestle replacement program and consider more cost effective alternatives to reduce the seismic risk, yet maintain life/safety and operational capacity of the 12 terminals. This study was unique in that it was developed by BIS Consulting LLC, GeoEngineers Inc., KPFF Consulting Engineers and WSF. The BIS, GeoEngineers, KPFF team tapped the collective expertise of lifecycle cost modeling (BIS), seismic analyses as it relates to soil and structural performance (GeoEngineers and KPFF, respectively).

The more cost effective alternatives include retrofitting the existing trestles by either soil stabilization and/or structural means are considered as the alternatives to the replacement option. This report presents the results of GeoEngineers' preliminary analyses related to seismic hazard and foundations, which were used as input to the structural analyses completed by KPFF and the life cycle cost modeling completed by BIS. We understand that more detailed engineering analyses may be completed for use in the final design of the seismic retrofit of the timber trestles evaluated in this study.

SUBSURFACE CONDITIONS

Subsurface Soils

The subsurface soil conditions at the sites were evaluated by reviewing the logs of exploratory borings completed near the existing timber trestles at each terminal, and by reviewing the USGS geologic map of the area. The geologic logs of the borings that we reviewed were provided by WSF. In general, the soils observed in the explorations for all 12 sites can be divided based on their geographic location within Puget Sound and by the site specific geologic units observed in the exploratory borings. In the North Puget Sound, around the San Juan Islands, soil consisted of loose unconsolidated sand and gravel with variable amounts of silt over bedrock while in the Central and South Puget Sound soil consisted of loose unconsolidated sand and gravel with variable amounts of silt over glacially consolidated soil. The following presents a general description of the geology starting with the most recently deposited unit. For specific subsurface soils information reviewed for each terminal, refer to Appendix A of this report.

- **Unconsolidated Deposits:** Unconsolidated deposits were encountered in the borings completed at most of the ferry terminals and generally consisted of loose sand and gravel with variable amounts of silt.
- **Glacially Consolidated Deposits:** Glacially consolidated deposits were encountered beneath the unconsolidated sand and gravel deposits in the borings completed at most of the ferry terminals. The glacially consolidated deposits generally consisted of dense to very dense sand with variable amounts of silt and gravel, and/or very stiff to hard clay.
- **Bedrock:** Bedrock was encountered at four ferry terminals (Friday Harbor, Lopez Island, Shaw Island, and Orcas Island) and was generally mapped as consisting of meta-sedimentary formations and conglomerate. The rock quality designation (RQD, developed by Deere, et al 1967 to estimate rock mass quality) number for the top 5 feet of the bedrock encountered generally ranges from 0 to 67 percent (0 to 50 being poor quality or highly fractured rock, 50 to 90 good quality or slightly to moderately fractured rock, and 90 to 100 excellent quality or intact rock).

Seismic Site Class Designation

Using the boring data, we established the Seismic Site Class and weighted average shear wave velocity within the top 30 meters of soil ($V_{s\text{-}30}$) for each ferry terminal, as presented in Table 1. The $V_{s\text{-}30}$ values were determined using published correlations with the standard penetration blow counts developed by Seed et al (1986) and Imai & Tonouchi (1982).

ENGINEERING ANALYSES

Preliminary Seismic Hazard

Response Spectra

The site specific ground surface response spectra for each of the 12 ferry terminals were determined using the 2008 USGS probabilistic seismic hazard model (https://geohazards.usgs.gov /deaggint/2008/). The response spectra curves were calculated using the $V_{s\text{-}30}$ values presented in Table 1, for design earthquakes with return periods of 72, 224, 475 and 975 years. Figures 1 through 12 present the response spectra curves and data points developed for use in the structural engineering analyses for the timber trestles at each of the twelve ferry terminals.

Liquefaction Potential

Soil liquefaction refers to the condition by which vibration or shaking of the ground, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength. In general, soils that are susceptible to liquefaction include sites with very loose to medium dense, clean to silty sands and non-plastic silts that are below the water table.

The evaluation of liquefaction potential is complex and dependent on numerous parameters, including soil type, grain-size distribution, soil density, depth to groundwater, in-situ static ground stresses, earthquake-induced ground stresses and excess pore water pressure generated during seismic shaking.

We completed soil liquefaction analyses using the boring data provided, and the peak ground acceleration values and mean magnitude determined using the 2008 USGS seismic hazard model. We evaluated liquefaction potential using the simplified method developed by Youd et al (2001). Based on our evaluation of the liquefaction potential of the soils at the 12 ferry terminals, we concluded that all the terminals have potential for liquefaction to occur after an earthquake event, with the exception of the Friday Harbor, Orcas Island, Tahlequah and Point Defiance terminals, where the liquefaction potential of the site soils is low. More detailed results are presented in Table 2.

Lateral Spreading Induced Load on Piles

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of 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. In the case of the ferry terminals, lateral spreading could occur during earthquakes resulting in the movement of soil or sediment onto below-water piles, or from movement of bulkhead soils onto downslope terminal facilities (including piles).

We completed lateral spreading analyses using the results of the soil liquefaction analysis and the MLR simplified method developed by Youd et al (1999). Based on our analysis, we concluded that the liquefiable soils at each terminal will spread laterally under the design earthquake levels.

The effect of the lateral spreading on pile foundations is represented by lateral soil pressure that should be included in the structural analysis. Based on back analysis of case histories, the average lateral spreading induced soil pressure on piles is estimated to be about 30 percent of the overburden pressure. For the conceptual design evaluation, we recommend that a rectangular soil pressure equal to 19H be used, where H is the thickness of critical slope failure surface and in this case equals to the thickness of liquefiable soils presented in Table 2. The additional lateral spreading load should be determined by applying the pressure over two pile diameters in the structural analysis.

PRELIMINARY FOUNDATION DESIGN RECOMMENDATIONS

Axial Pile Capacities

Existing Timber Piles

Based on our review of the as-built drawings and pile driving records provided for each ferry terminal, we understand that the timber piles were driven to practical refusal with a recommended axial downward capacity of 20 tons. We also recommended that the uplift capacity of the timber piles be neglected in the structural analysis because of the shallow pile embedment depth.

New Steel Pipe Piles

We understand that one of the retrofit options will require driving new piles along the perimeter of the existing timber trestle. The piles being considered are 16-, 24- and 36-inch-diameter steel pipe piles. We recommend that the piles be driven open-ended. For the 16- and 24-inch-diameter steel pipe piles, we estimated the axial pile capacities (both downward and uplift) assuming the piles will be plugged at the end of driving. For the 36-inch-diameter steel pipe piles, we estimated the axial pile capacities assuming unplugged conditions at the pile tip at the end of driving. We recommend that a factor of safety (FS) of 3.0 and 1.5 be used to determine the allowable downward and uplift capacities, respectively. Figures 18 through 55 present the ultimate vertical downward and uplift capacities of 16-, 24- and 36-inch-diameter steel pipe piles, for each of the ferry terminals with the exception of the Shaw Island and Mukilteo Terminals. We understand WSF will not pursue a retrofit of the Shaw Island terminal, and plans to construct a new terminal to replace the existing Mukilteo Terminal. Also not included in this section are the Friday Harbor and Lopez Island Terminals where the use of micropiles is anticipated. Refer to the “Micropile design recommendations” section of this report for information on these two terminals.

Soil Parameters for LPILE Analysis

Our recommendations for LPILE parameters to be used in seismic lateral pile analyses for each terminal are provided in Figures 56 through 65, with the exception of the Shaw Island and Mukilteo Terminals, we understand WSF will not pursue the retrofit of the Shaw Island terminal, and plans to construct a new terminal to replace the existing Mukilteo Terminal. Since the timber piles were spaced at least three pile diameters center-to-center, no reduction for pile group action needs to be made. For the potentially liquefiable soils, a load-reduction multiplier (p-multiplier) of 0.1 should be applied.

Micropile Design Recommendations

For the ferry terminals where shallow bedrock was encountered (e.g. Friday Harbor and Lopez Island), anchored micropiles are considered in the retrofit option. We understand that the micropiles considered generally consist of a 8⅝-inch steel casing with a 5-inch-diameter grouted anchor below the steel casing, per Washington State Department of Transportation (WSDOT) details developed for the Friday Harbor Preservation project completed in 2004. For design of the anchored micropiles, we recommend an allowable downward bearing capacity of 300 kips per square foot (ksf) and allowable side friction/uplift capacity of 15 kips per foot for the 8⅝-inch steel casing. For the uncased 5-inch-diameter grouted column, we recommend an allowable downward capacity of 300 ksf and allowable side friction/uplift capacity of 30 kips per foot to be used in the design.

BULKHEAD GLOBAL STABILITY ANALYSES

Slope stability analyses were completed to evaluate the global stability of the bulkhead at 7 ferry terminals (i.e., Anacortes, Mukilteo, Edmonds, Fauntleroy, Southworth, Tahlequah and Point Defiance). The objective of the analysis was to evaluate the potential for added soil pressure onto the trestle that results from instability of the bulkhead wall under the design earthquake events. We completed slope stability analyses in accordance with the analytical procedure presented in WSDOT's Geotechnical Design Manual (GDM) using the computer program SLOPE/W (GEO-SLOPE International, Ltd., 2005).

We evaluated five loading conditions:

1. Static condition (Existing soil condition);
2. Seismic conditions with acceleration coefficients of 0.1 g, 0.2g and 0.3g, representing a small, moderate and large sized earthquake, respectively; and
3. Post earthquake conditions with residual strength for the potential liquefiable soils as appropriate.

Bulkhead Stability Results

SLOPE/W evaluates the stability of the critical failure surfaces identified using vertical slice limit-equilibrium methods. This method compares the ratio of forces driving slope movement with forces resisting slope movement for each trial failure surface, and presents the result as the FS. Figures 66 through 72 present the critical failure surface, and FS for the different loading conditions evaluated, for the seven ferry terminals.

Based on the results of our global stability analyses, we concluded that the bulkhead at Anacortes, Mukilteo and Edmonds terminals would likely be unstable and exert additional load onto the trestles under the design earthquake events. The additional load exerted from the bulkhead is determined to be 35.6 and 7.4 kips per foot of bulkhead at Anacortes and Edmonds terminals, respectively. The additional bulkhead load for Mukilteo terminal was not provided because we understand that the bulkhead will be replaced.

PRELIMINARY GROUND IMPROVEMENT DESIGN FOR BULKHEAD STABILIZATION

In order to mitigate the bulkhead stability issues at both the Anacortes and Edmonds terminals, we recommend that ground improvement consisting of either stone columns or compaction grouting be installed behind the bulkhead. Figures 66 and 68 show the preliminary ground improvement zone determined for Anacortes and Edmonds Terminal, respectively. Based on our preliminary analysis, we determined that a 30-foot-wide compaction grouting zone or 50-foot-wide stone column zone be installed behind the bulkhead. The depth of the compaction grouting or stone columns is estimated to be about 30 feet. The minimum replacement ratio for the compaction grouting and stone columns is estimated to be 10 percent.

LIMITATIONS

We have prepared this report for WSF, their authorized agents and regulatory agencies for the WSF Timber Trestles project.

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices for geotechnical engineering in this area at the time this report was prepared.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments should be considered a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

Please refer to Appendix B titled "Report Limitations and Guidelines for Use" for additional information pertaining to use of this report.

REFERENCES

GEO-SLOPE International, Ltd. (2005). "Slope/W" Version 6.17.

Idriss, I.M., and Boulanger R.W. (2008). "Soil Liquefaction During Earthquakes," Earthquake Engineering Research Institute EERI Monograph MNO-12, Oakland, California.

Imai, T. and Tonouchi, K. (1982). "Correlation of N-value with S-wave velocity and shear modulus,"Proceedings, 2nd European Symposium on Penetration Testing, Amsterdam, pp. 57-72.

International Building Code (2009): International Code Council, Sec. 1613, 2006.

Seed, H. B., Wong, R.T., Idriss, I.M., and Tokimatsu, K. (1986). "Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils," Journal of Geotechnical Engineering, ASCE, Vol. 112, No. 11, November, pp. 1016-1032.

USGS, 2008 Interactive Deaggregations, U.S. Geological Survey Earthquake Hazards Program, http://eqint.cr.usgs.gov/deaggint/2008/index.php, 2008.

Youd, T.L., Hansen, C.M., and Bartlett, S.F., 1999, "Revised MLR Equations for Prediction Lateral Spread Displacement," Proceedings, 7th U.S.-Japan Workshop on Earthquake Resistant Design of Lifeline Facilities and Countermeasures Against Liquefaction, Seattle, Washington, Multidisciplinary Center for Earthquake Engineering Research Technical Report MCEER-99-0019, p99-114.

Youd, T.L., Idriss, I.M., Andrus R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F. Jr., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F. III., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H. II. 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." J. Geotech. Geoenviron. Eng. 127, 817.

TABLES

GeoEngineers

Table 1

Site Class and Vs-30

Washington State Ferries Timber Trestles Project
Puget Sound Area

Ferry Terminal	Site Class	Average weighted Shear wave velocity within the top 30-meter of soil (Vs-30 in m/s)
Friday Harbor	B/C	760
Lopez Island	B/C	760
Shaw Island	B/C	760
Orcas Island	C	626
Anacortes	D	345
Mukilteo[1]	D	280
	E	180
Edmonds	D	303
Fauntleroy	D	274
Vashon Island	D	335
Southworth	D	298
Tahlequah	D	299
Point Defiance	D	313

Notes:

[1]Site Class D for the design earthquakes that do not trigger liquefaction of soil deeper than 20 feet (e.g. the 72-year earthquake), and Site Class E for the design earthquakes that will trigger liquefaction at depth deeper than 20 feet (e.g. 224-, 475-, 975-year earthquakes).

Table 2

Average Thickness of Potentially Liquefiable Soils - All Design Earthquakes

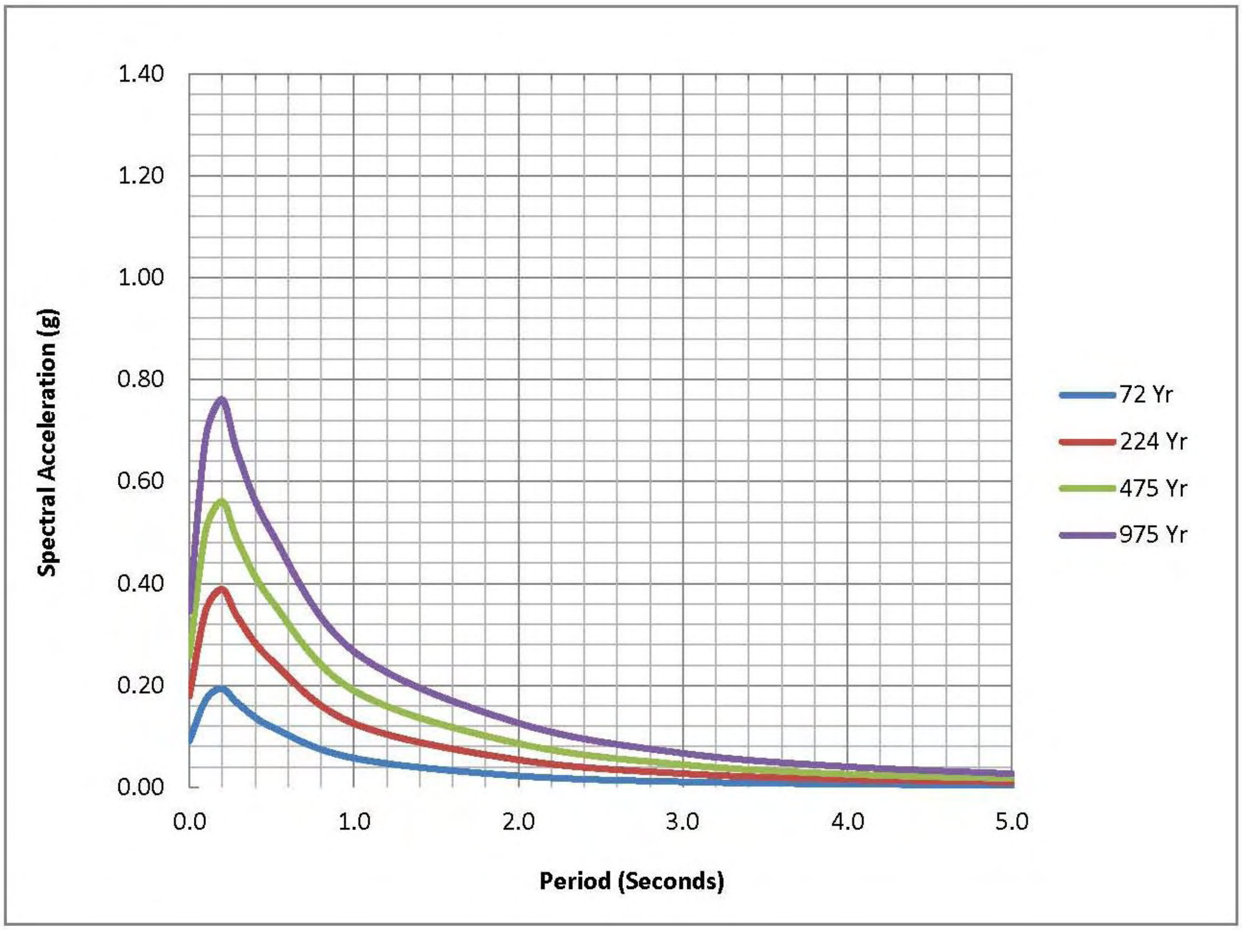
Washington State Ferries Timber Trestles Project

Puget Sound Area

Desgin Earthquake Return Period	Potentially Liquefiable Soils Average Thickness (feet)											
	Friday Harbor	Lopez Island	Shaw Island	Orcas Island	Anacortes	Mukilteo	Edmonds	Fauntleroy	Vashon Island	Southworth	Tahlequah	Point Defiance
72- year EQ	0	See Figure 13	See Figure 14	0	See Figure 15	23	4	See Figure 16	5	See Figure 17	0	0
224- year EQ	0	See Figure 13	See Figure 14	0	See Figure 15	64	8	See Figure 16	5	See Figure 17	0	0
475- year EQ	0	See Figure 13	See Figure 14	0	See Figure 15	67	11	See Figure 16	5	See Figure 17	0	0
975- year EQ	0	See Figure 13	See Figure 14	0	See Figure 15	80	11	See Figure 16	5	See Figure 17	0	0

FIGURES

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.0909	0.1791	0.2566	0.3448
0.1000	0.1717	0.3470	0.5038	0.6864
0.2000	0.1934	0.3887	0.5613	0.7611
0.3000	0.1637	0.3308	0.4800	0.6493
0.5000	0.1188	0.2492	0.3662	0.5000
1.0000	0.0575	0.1255	0.1900	0.2667
2.0000	0.0227	0.0542	0.0861	0.1260
3.0000	0.0111	0.0272	0.0445	0.0667
4.0000	0.0065	0.0157	0.0260	0.0403
5.0000	0.0044	0.0107	0.0175	0.0266



Ground Surface Response Spectra

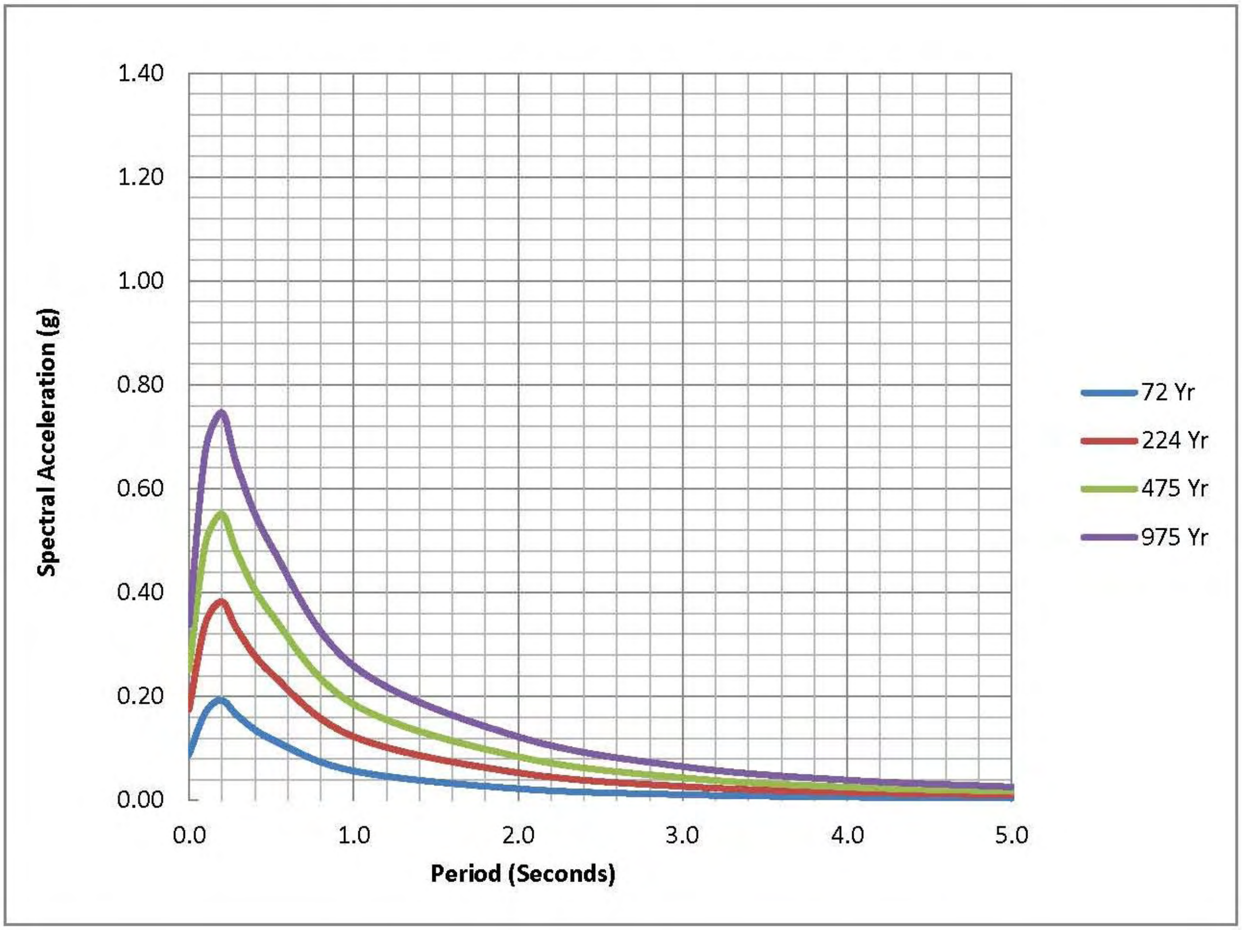
Washington State Ferries Timber Trestles Project
Friday Harbor Terminal
Puget Sound Area

GeoEngineers

Figure 1

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.0891	0.1757	0.2513	0.3379
0.1000	0.1696	0.3408	0.4943	0.6740
0.2000	0.1928	0.3822	0.5505	0.7467
0.3000	0.1619	0.3250	0.4705	0.6369
0.5000	0.1182	0.2446	0.3583	0.4899
1.0000	0.0569	0.1230	0.1851	0.2589
2.0000	0.0227	0.0534	0.0842	0.1226
3.0000	0.0111	0.0271	0.0436	0.0651
4.0000	0.0065	0.0156	0.0256	0.0391
5.0000	0.0044	0.0107	0.0174	0.0259



Ground Surface Response Spectra

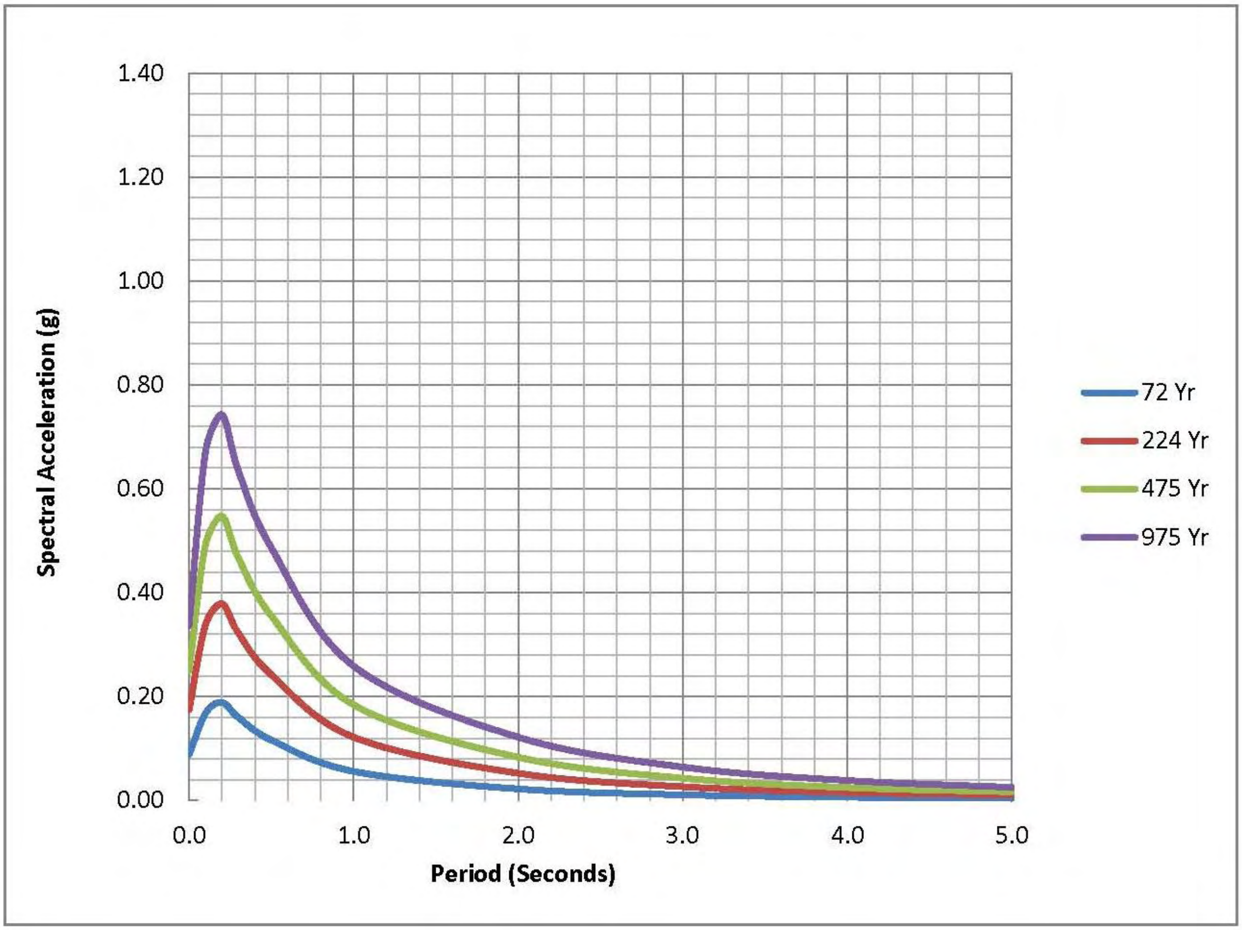
Washington State Ferries Timber Trestles Project
Lopez Island Terminal
Puget Sound Area

GeoEngineers

Figure 2

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.0879	0.1744	0.2500	0.3366
0.1000	0.1674	0.3380	0.4912	0.6703
0.2000	0.1888	0.3787	0.5472	0.7431
0.3000	0.1599	0.3223	0.4674	0.6349
0.5000	0.1164	0.2422	0.3558	0.4871
1.0000	0.0561	0.1220	0.1845	0.2590
2.0000	0.0223	0.0525	0.0836	0.1220
3.0000	0.0109	0.0265	0.0432	0.0646
4.0000	0.0064	0.0153	0.0253	0.0387
5.0000	0.0043	0.0105	0.0170	0.0255



Ground Surface Response Spectra

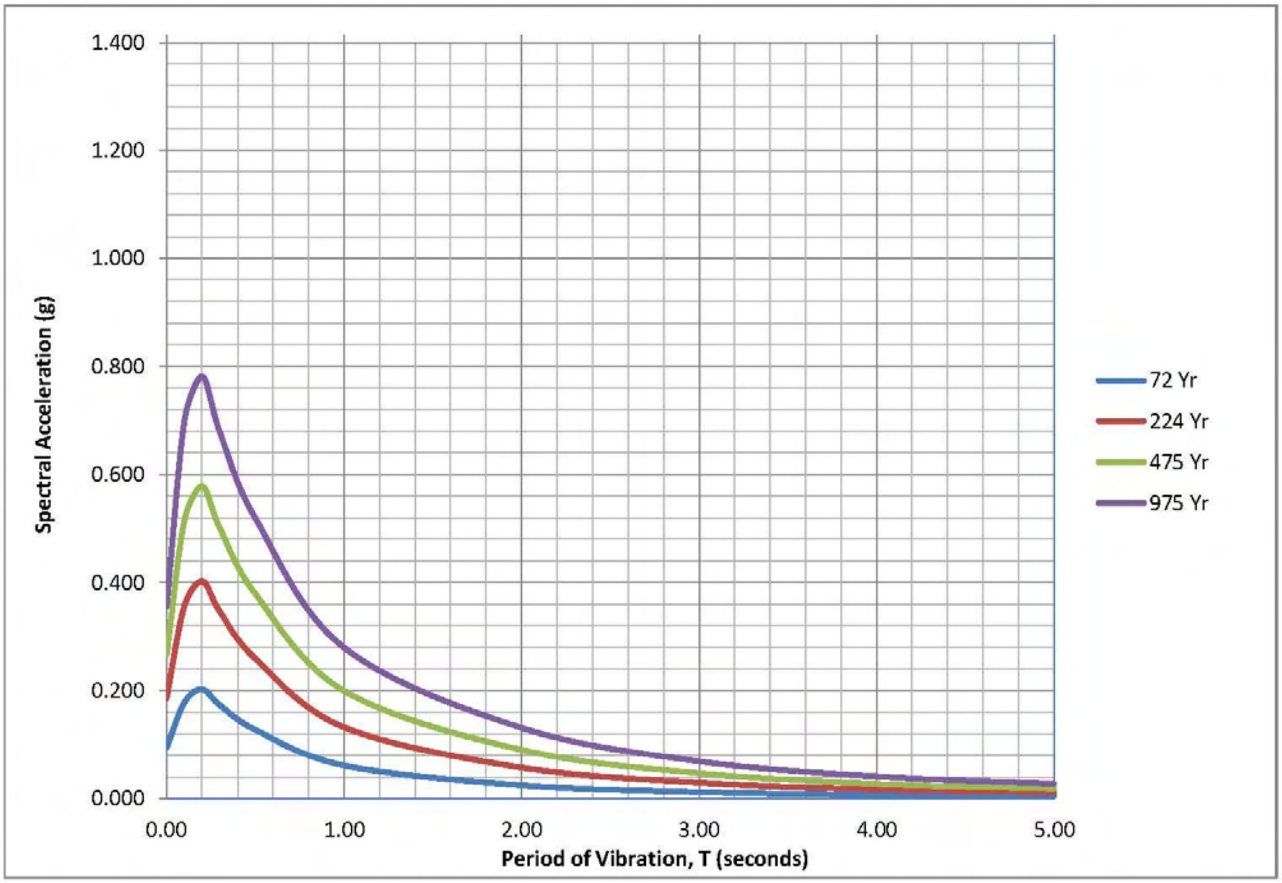
Washington State Ferries Timber Trestles Project
Shaw Island Terminal
Puget Sound Area

GeoEngineers

Figure 3

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period Spectral Acceleration (g)	224Yr Return Period Spectral Acceleration (g)	475Yr Return Period Spectral Acceleration (g)	975Yr Return Period Spectral Acceleration (g)
0.000	0.094	0.186	0.266	0.356
0.100	0.178	0.356	0.514	0.697
0.200	0.202	0.402	0.579	0.782
0.300	0.173	0.347	0.501	0.679
0.500	0.128	0.259	0.379	0.519
1.000	0.061	0.132	0.199	0.280
2.000	0.024	0.057	0.090	0.132
3.000	0.012	0.029	0.047	0.069
4.000	0.007	0.016	0.027	0.041
5.000	0.005	0.011	0.018	0.027



Ground Surface Response Spectra

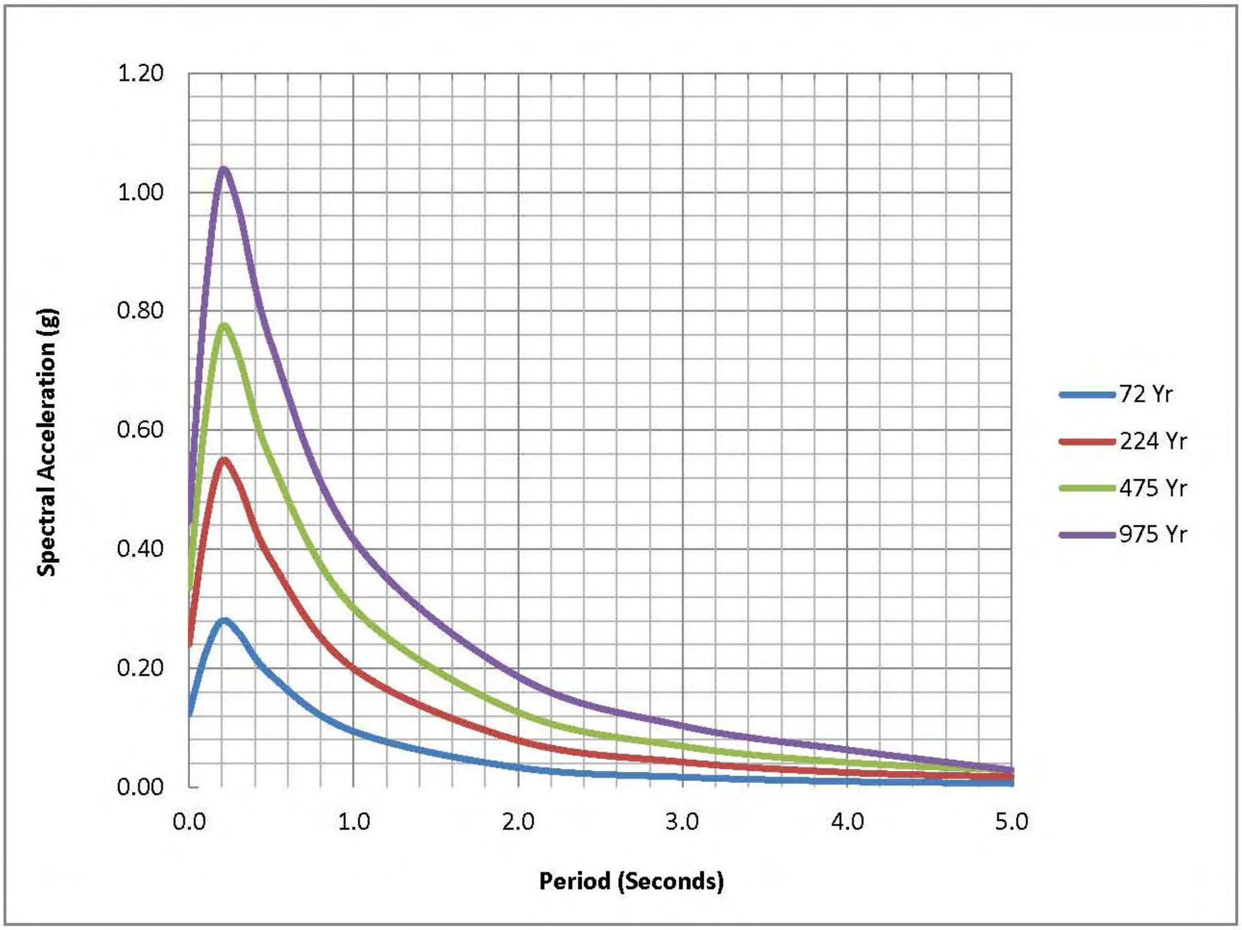
Washington State Ferries Timber Trestles Project
Orcas Island Terminal
Puget Sound Area

GeoEngineers

Figure 4

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1238	0.2402	0.3349	0.4482
0.1000	0.2243	0.4360	0.6202	0.8287
0.2000	0.2797	0.5481	0.7736	1.0353
0.3000	0.2603	0.5121	0.7268	0.9745
0.5000	0.1881	0.3810	0.5492	0.7451
1.0000	0.0942	0.1995	0.3012	0.4165
2.0000	0.0335	0.0787	0.1261	0.1860
3.0000	0.0175	0.0424	0.0688	0.1030
4.0000	0.0102	0.0249	0.0417	0.0629
5.0000	0.0071	0.0176	0.0288	0.0288



Ground Surface Response Spectra

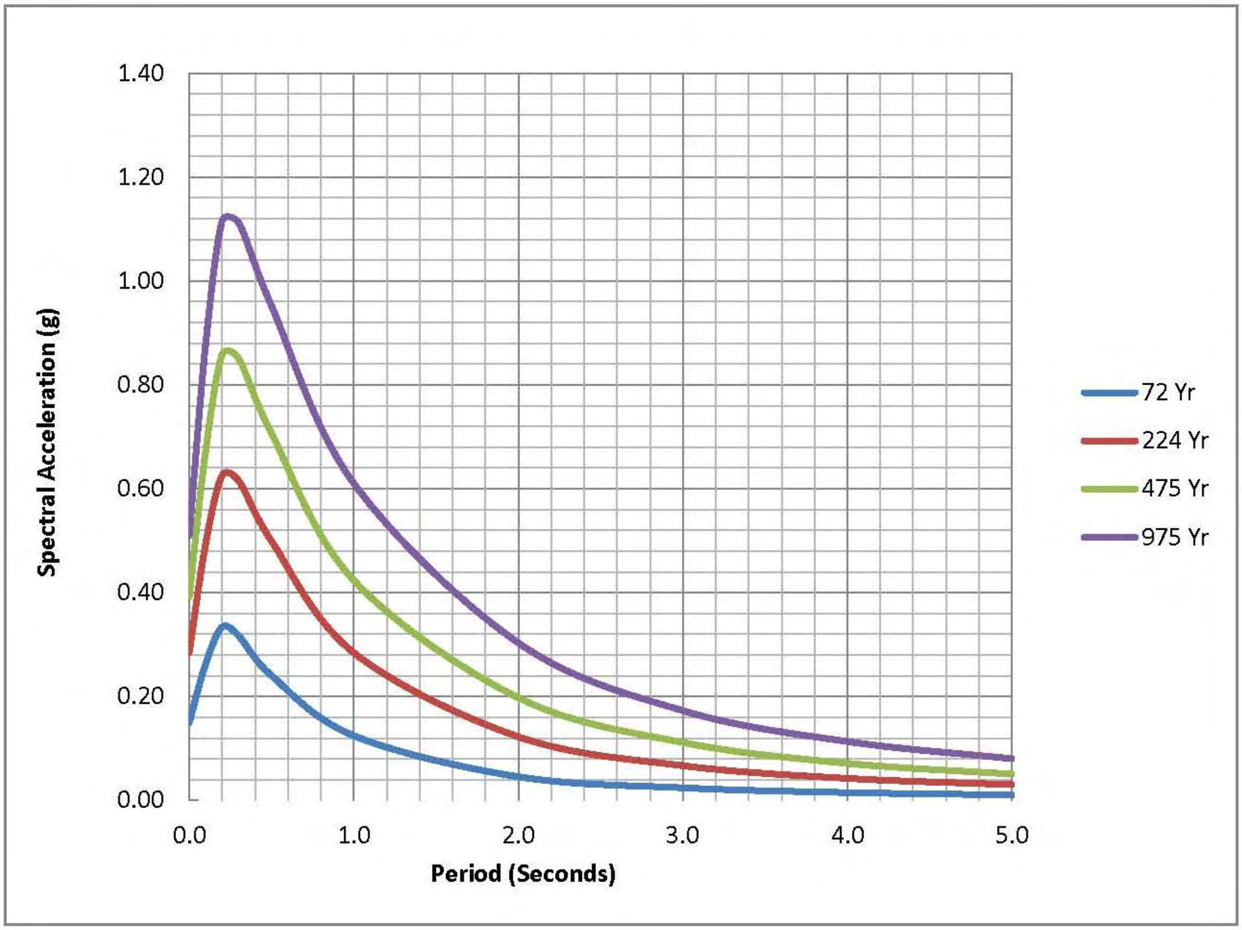
Washington State Ferries Timber Trestles Project
Anacortes Terminal
Puget Sound Area

GeoEngineers

Figure 5

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1498	0.2847	0.3922	0.5096
0.1000	0.2630	0.4936	0.6725	0.8779
0.2000	0.3340	0.6253	0.8578	1.1149
0.3000	0.3170	0.6153	0.8505	1.1121
0.5000	0.2405	0.4993	0.7066	0.9520
1.0000	0.1246	0.2844	0.4237	0.6103
2.0000	0.0455	0.1227	0.1978	0.3031
3.0000	0.0241	0.0670	0.1116	0.1726
4.0000	0.0150	0.0426	0.0714	0.1131
5.0000	0.0104	0.0304	0.0507	0.0800



Ground Surface Response Spectra

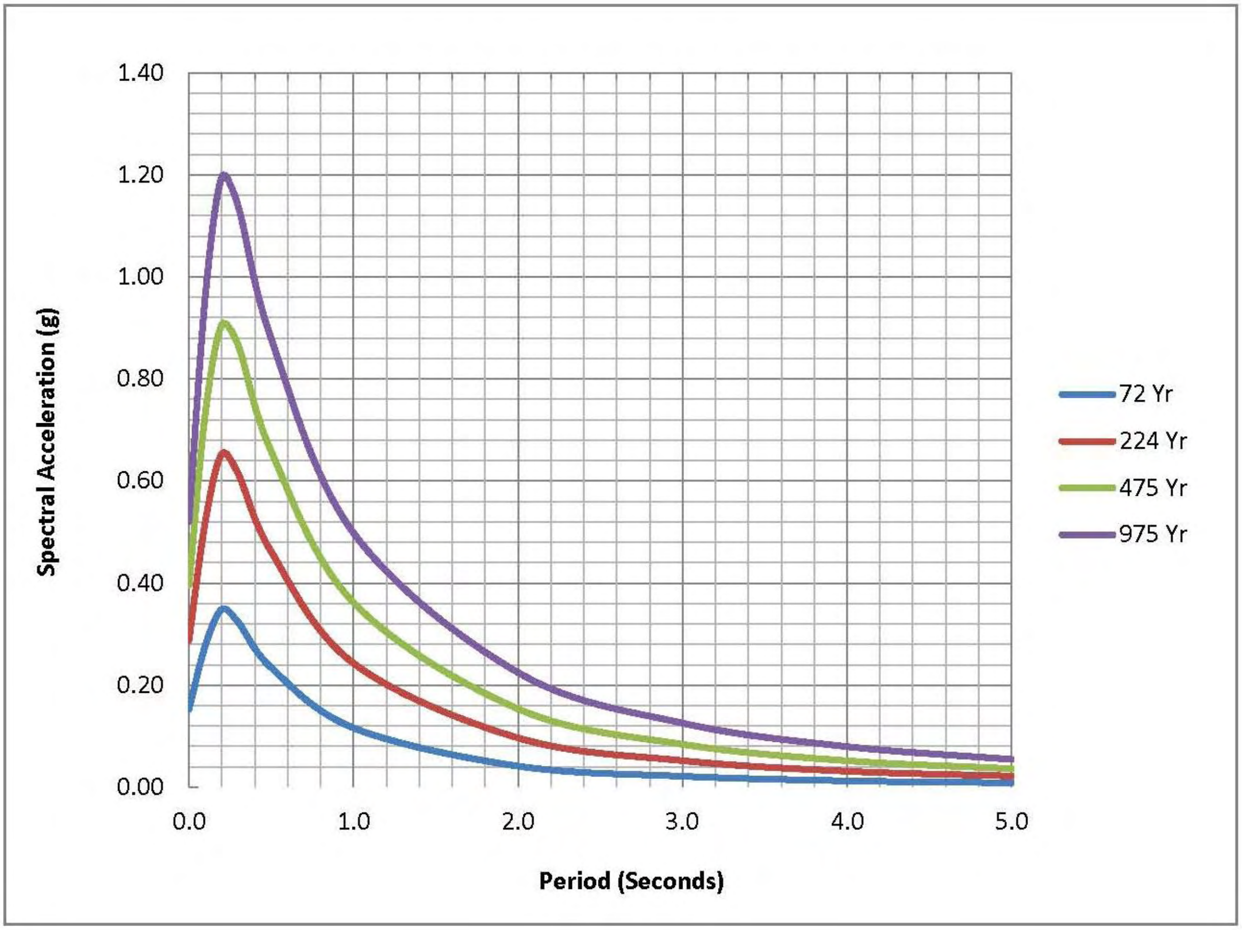
Washington State Ferries Timber Trestles Project
Mukilteo Terminal
Puget Sound Area

GEOENGINEERS

Figure 6

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1523	0.2870	0.3951	0.5209
0.1000	0.2774	0.5204	0.7315	0.9621
0.2000	0.3483	0.6532	0.9067	1.1961
0.3000	0.3231	0.6140	0.8661	1.1383
0.5000	0.2346	0.4617	0.6587	0.8796
1.0000	0.1165	0.2429	0.3621	0.4987
2.0000	0.0416	0.0964	0.1536	0.2251
3.0000	0.0217	0.0518	0.0840	0.1258
4.0000	0.0127	0.0311	0.0518	0.0798
5.0000	0.0089	0.0216	0.0363	0.0547



Ground Surface Response Spectra

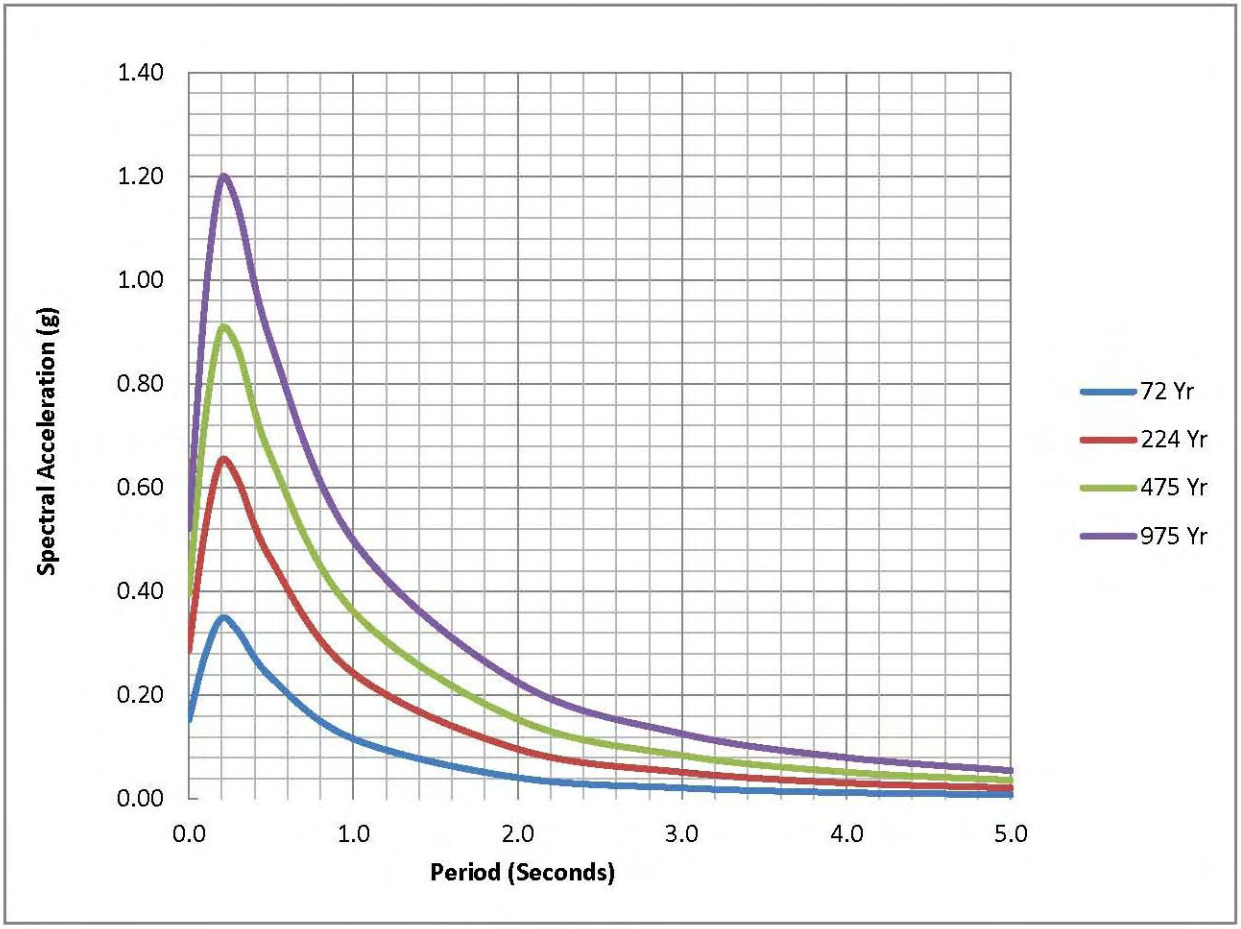
Washington State Ferries Timber Trestles Project
Edmonds Terminal
Puget Sound Area

GeoEngineers

Figure 7

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1523	0.2870	0.3951	0.5209
0.1000	0.2774	0.5204	0.7315	0.9621
0.2000	0.3483	0.6532	0.9067	1.1961
0.3000	0.3231	0.6140	0.8661	1.1383
0.5000	0.2346	0.4617	0.6587	0.8796
1.0000	0.1165	0.2429	0.3621	0.4987
2.0000	0.0416	0.0964	0.1536	0.2251
3.0000	0.0217	0.0518	0.0840	0.1258
4.0000	0.0127	0.0311	0.0518	0.0798
5.0000	0.0089	0.0216	0.0363	0.0547



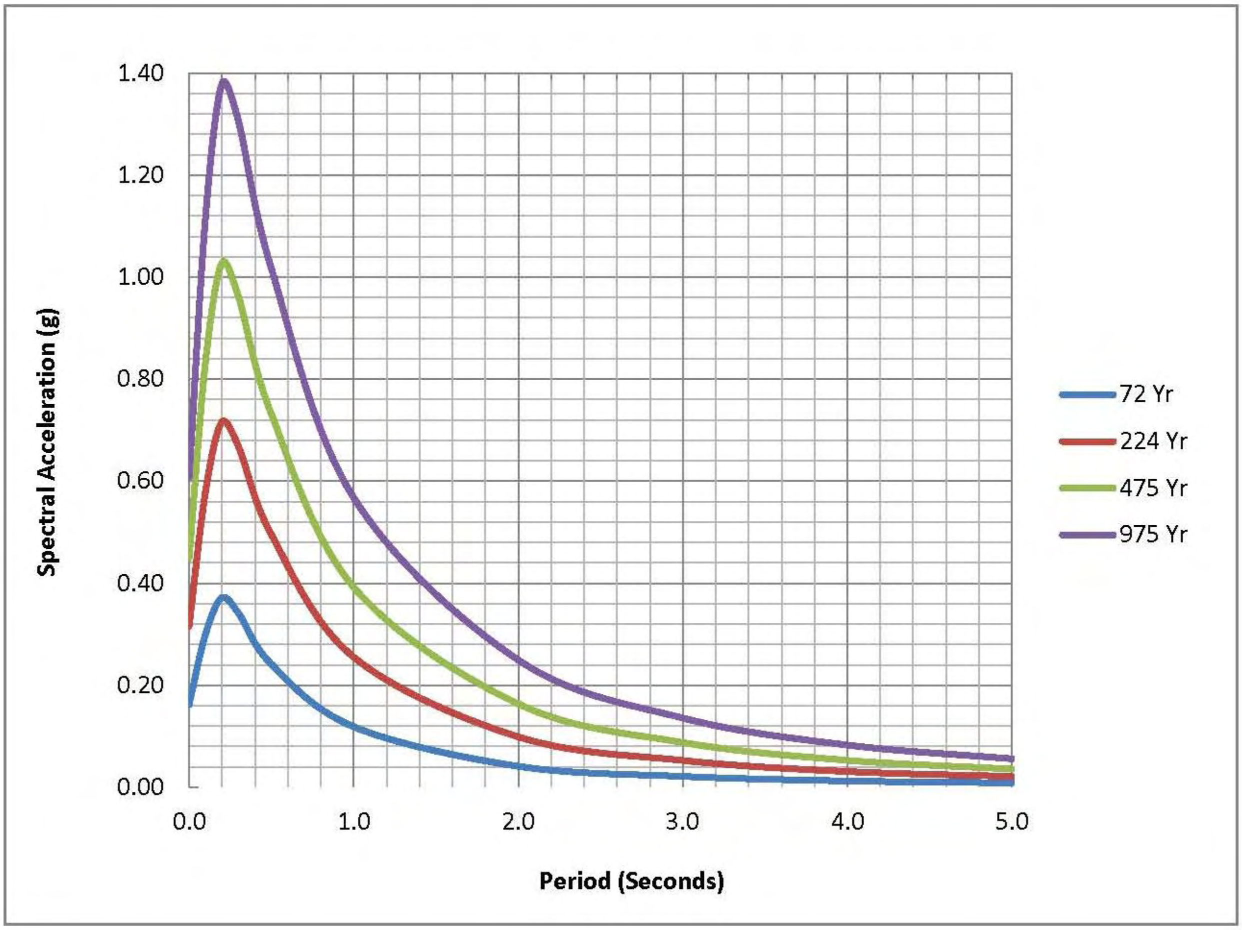
Ground Surface Response Spectra

Washington State Ferries Timber Trestles Project
Fauntleroy Terminal
Puget Sound Area

GeoEngineers

Figure 8

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1620	0.3152	0.4495	0.6083
0.1000	0.3002	0.5795	0.8336	1.1171
0.2000	0.3717	0.7164	1.0278	1.3791
0.3000	0.3417	0.6684	0.9625	1.3060
0.5000	0.2413	0.4956	0.7318	1.0170
1.0000	0.1187	0.2551	0.3925	0.5679
2.0000	0.0411	0.0988	0.1637	0.2498
3.0000	0.0213	0.0524	0.0879	0.1360
4.0000	0.0123	0.0307	0.0530	0.0830
5.0000	0.0086	0.0211	0.0358	0.0560



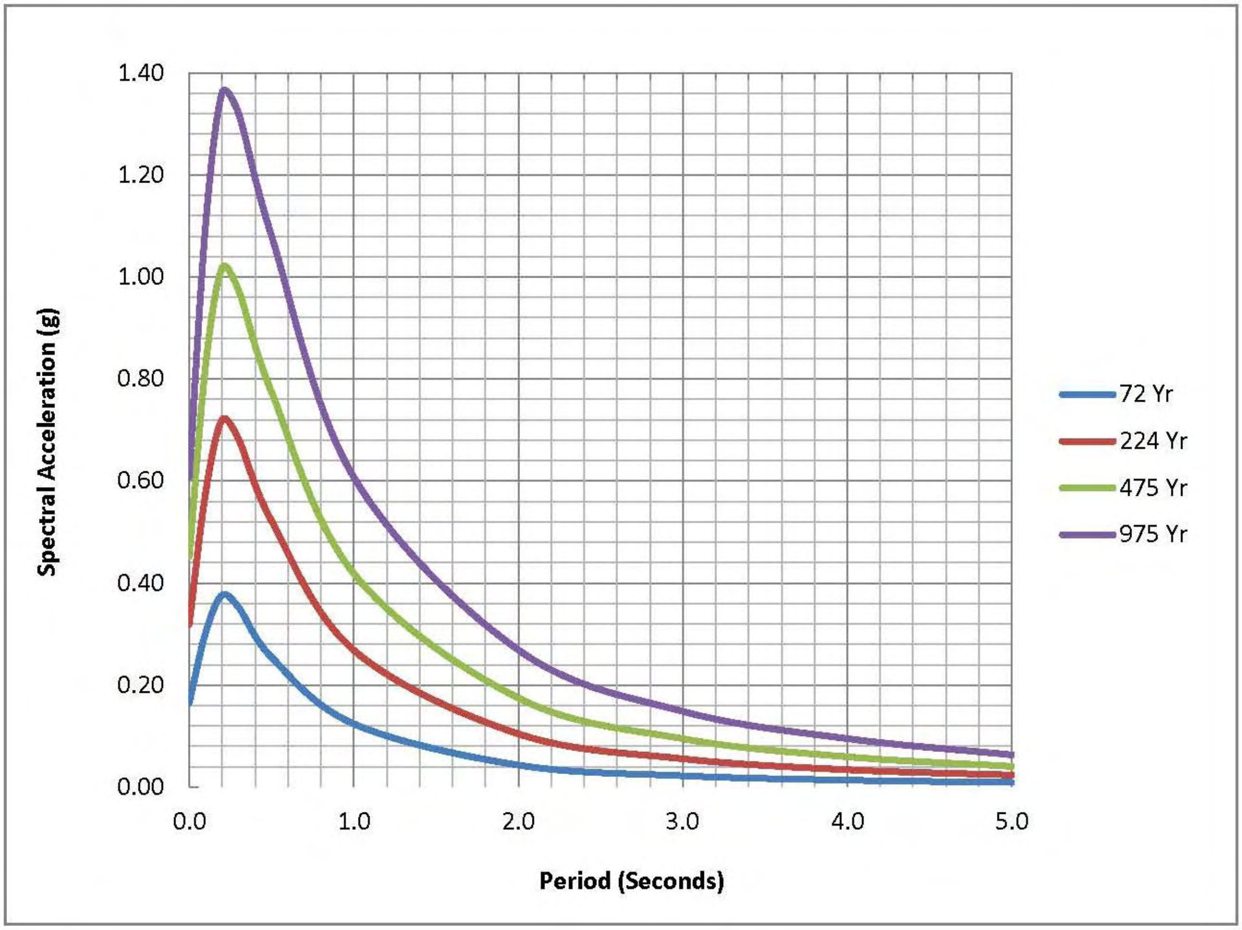
Ground Surface Response Spectra

Washington State Ferries Timber Trestles Project
Vashon Island Terminal
Puget Sound Area

GeoEngineers

Figure 9

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1651	0.3195	0.4518	0.6080
0.1000	0.3033	0.5778	0.8274	1.1040
0.2000	0.3767	0.7201	1.0178	1.3609
0.3000	0.3526	0.6819	0.9733	1.3200
0.5000	0.2551	0.5223	0.7746	1.0809
1.0000	0.1244	0.2686	0.4188	0.6077
2.0000	0.0435	0.1048	0.1749	0.2691
3.0000	0.0226	0.0559	0.0950	0.1486
4.0000	0.0140	0.0343	0.0596	0.0951
5.0000	0.0096	0.0240	0.0408	0.0633



Ground Surface Response Spectra

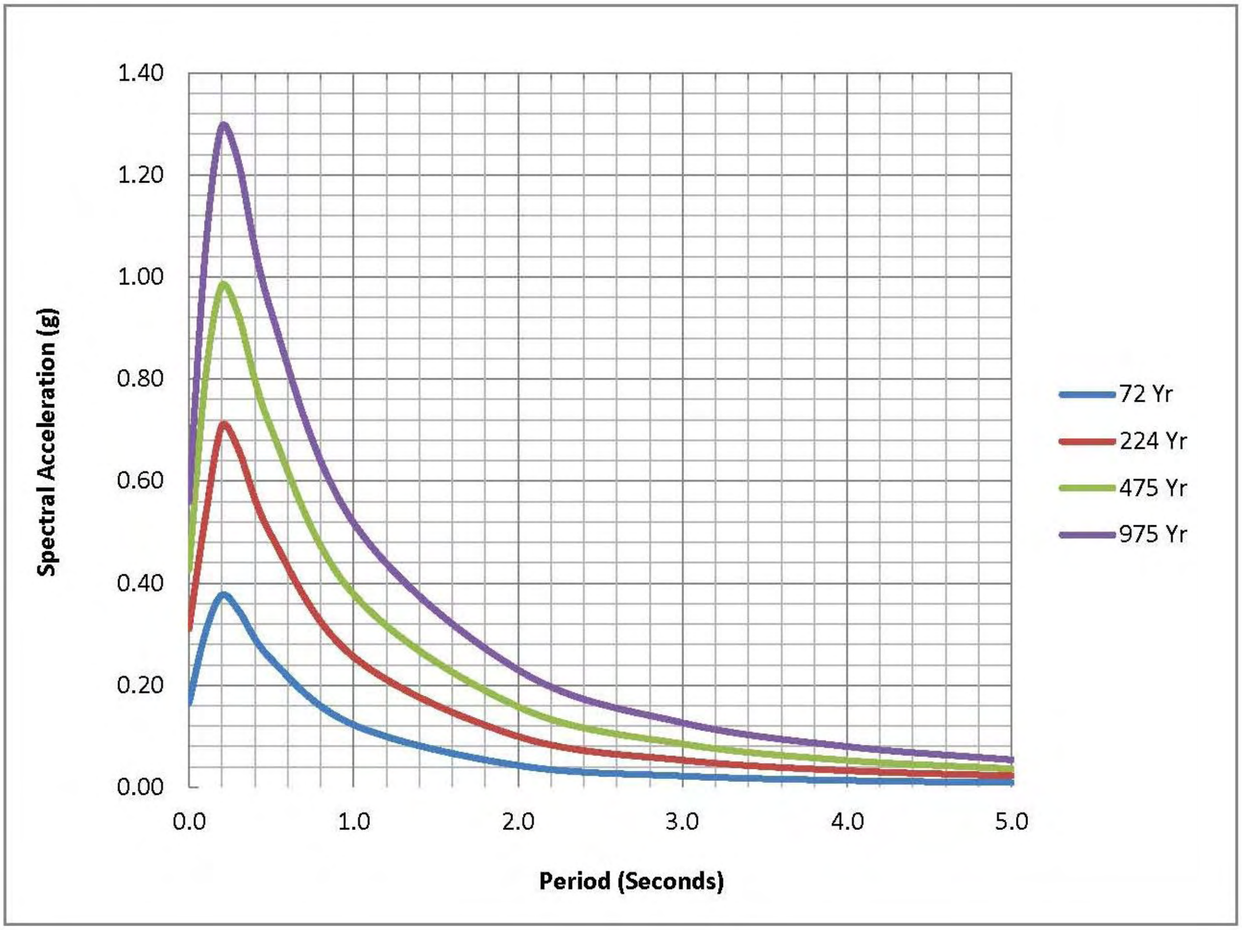
Washington State Ferries Timber Trestles Project
Southworth Terminal
Puget Sound Area

GeoEngineers

Figure 10

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1647	0.3102	0.4264	0.5606
0.1000	0.3029	0.5289	0.8000	1.0512
0.2000	0.3767	0.7079	0.9829	1.2952
0.3000	0.3473	0.6625	0.9258	1.2261
0.5000	0.2507	0.4934	0.7025	0.9319
1.0000	0.1226	0.2556	0.3786	0.5187
2.0000	0.0429	0.0996	0.1579	0.2301
3.0000	0.0222	0.0529	0.0851	0.1263
4.0000	0.0136	0.0325	0.0528	0.0797
5.0000	0.0094	0.0223	0.0364	0.0537



Ground Surface Response Spectra

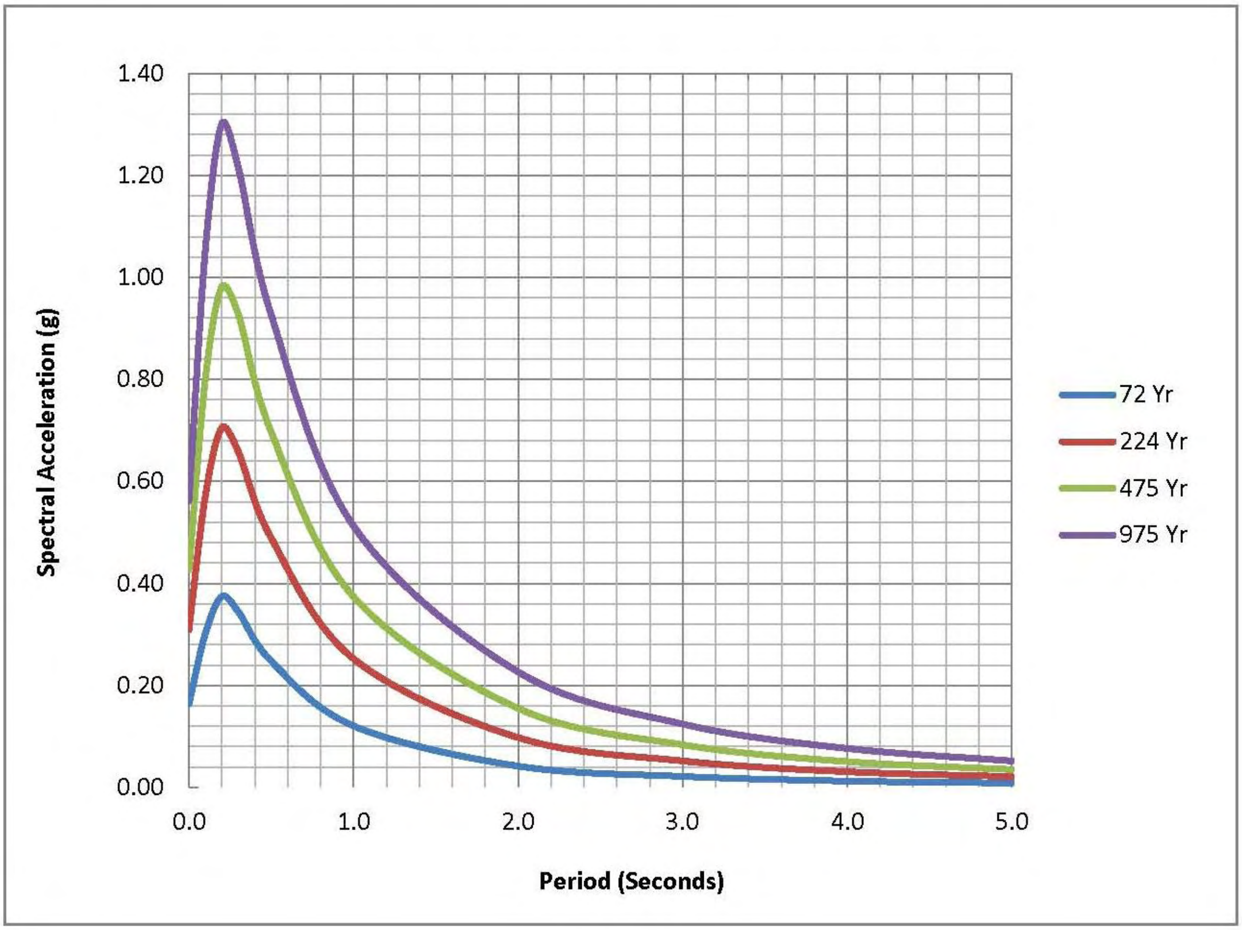
Washington State Ferries Timber Trestles Project
Tahlequah Terminal
Puget Sound Area

GEOENGINEERS

Figure 11

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12

Period of Vibration	72Yr Return Period SA	224Yr Return Period SA	475Yr Return Period SA	975Yr Return Period SA
0.0000	0.1637	0.3088	0.4260	0.5614
0.1000	0.3016	0.5714	0.8006	1.0538
0.2000	0.3746	0.7051	0.9816	1.3010
0.3000	0.3448	0.6585	0.9278	1.2158
0.5000	0.2480	0.4891	0.6968	0.9265
1.0000	0.1208	0.2523	0.3738	0.5127
2.0000	0.0420	0.0977	0.1551	0.2265
3.0000	0.0217	0.0518	0.0835	0.1242
4.0000	0.0126	0.0306	0.0505	0.0763
5.0000	0.0088	0.0211	0.0349	0.0519



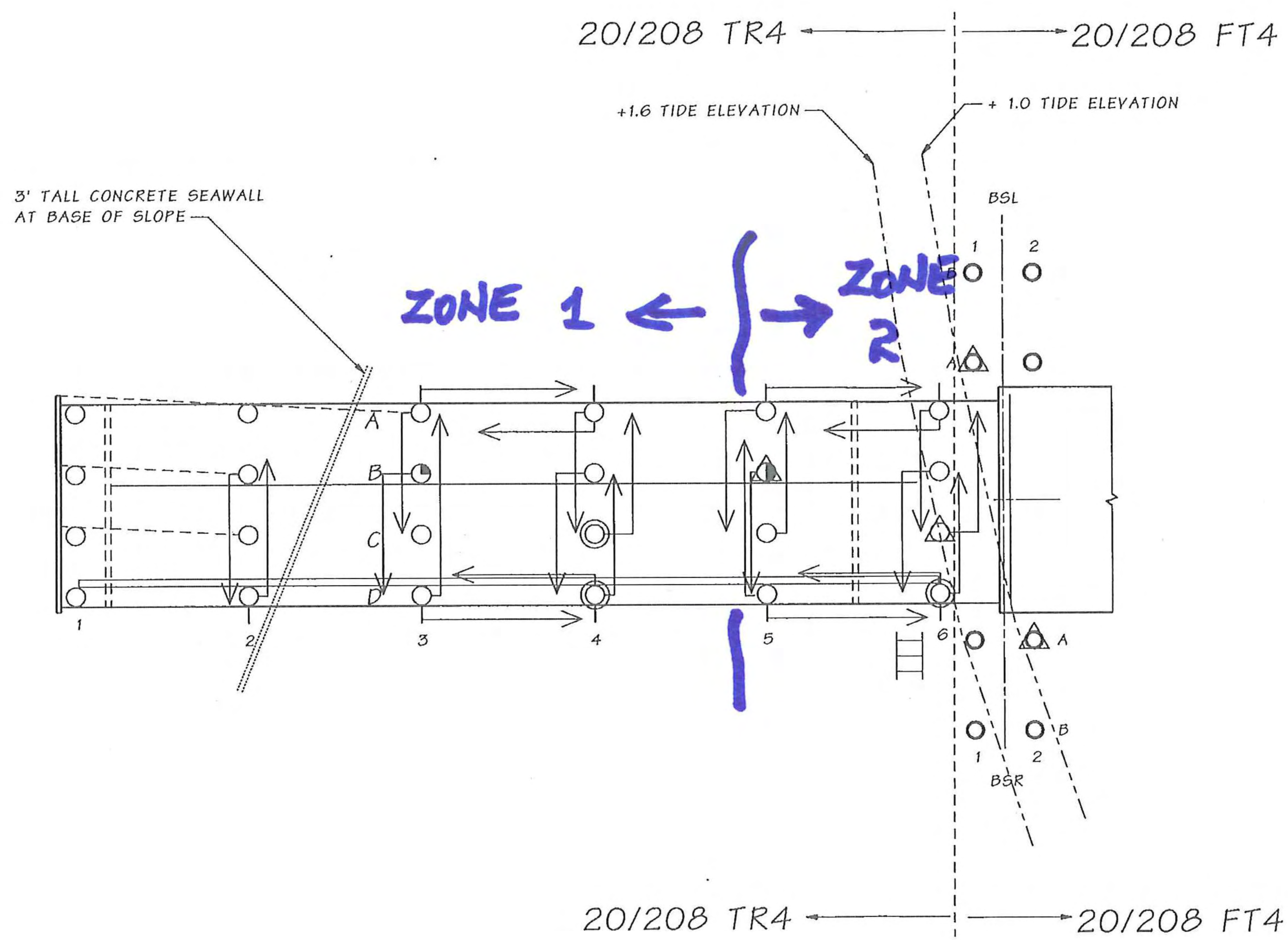
Ground Surface Response Spectra

Washington State Ferries Timber Trestles Project
Point Defiance Terminal
Puget Sound Area

GeoEngineers

Figure 12

SP/0180-284-00/Finals/Figures 1 through 12. Ground Surface Response Spectra.ppt NLU 3/23/12



AVERAGE THICKNESS OF POTENTIALLY LIQUEFIABLE SOILS UNDER ALL DESIGN EQ LEVELS

Trestle Zone	Average Thickness of Potentially Liquefiable Soils (feet)
1	0
2	5.0

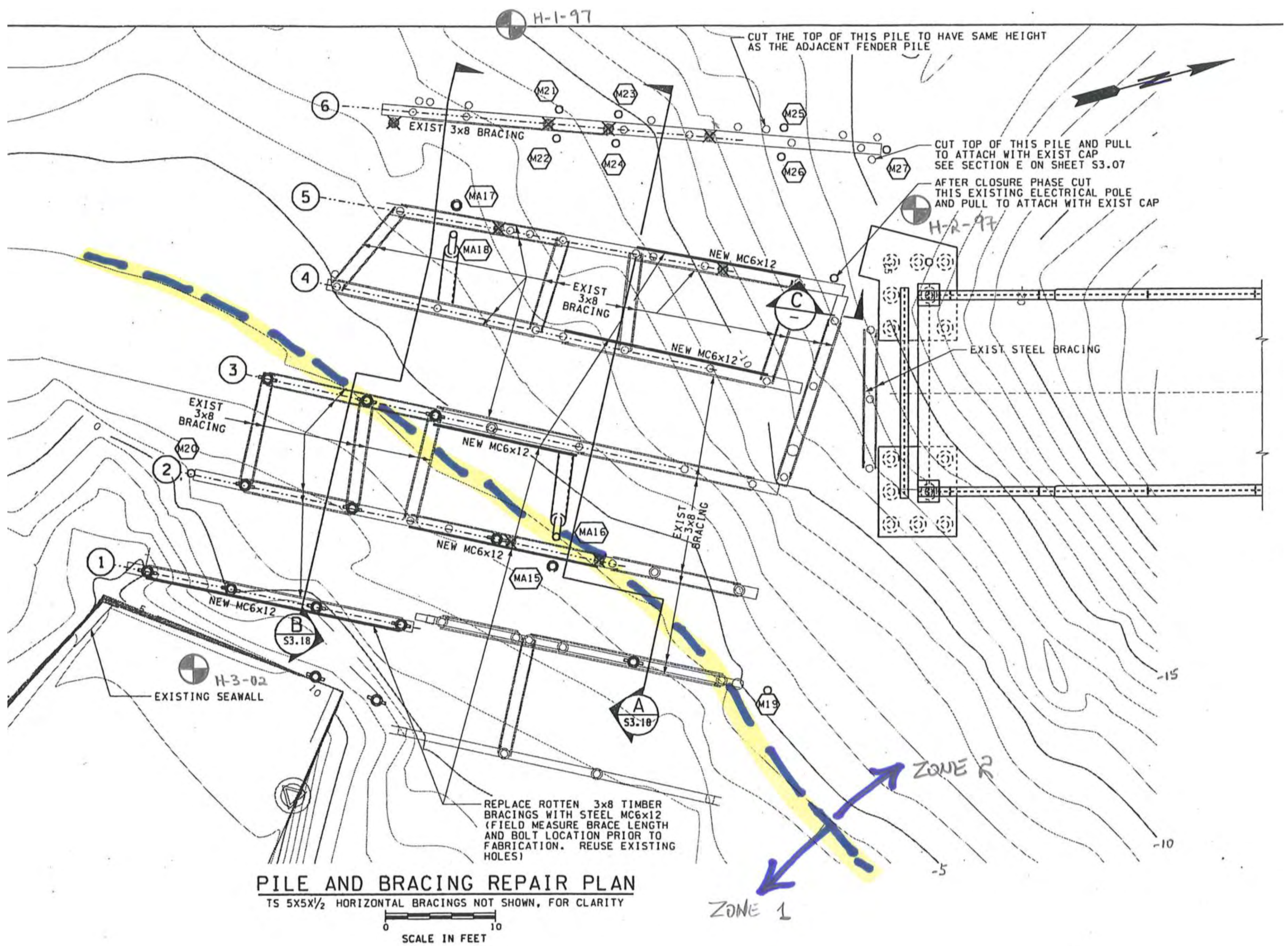
Average Thickness of Potentially Liquefiable Soils

Washington State Ferries Timber Trestles Project
Lopez Island Terminal
Puget Sound Area

GeoEngineers

Figure 13

SP/0180-284-00/Finals/Figures13 – 17..ppt NLU 3/23/12



AVERAGE THICKNESS OF POTENTIALLY LIQUEFIABLE SOILS UNDER ALL DESIGN EQ LEVELS

Trestle Zone	Average Thickness of Potentially Liquefiable Soils (feet)
1	0
2	7.0

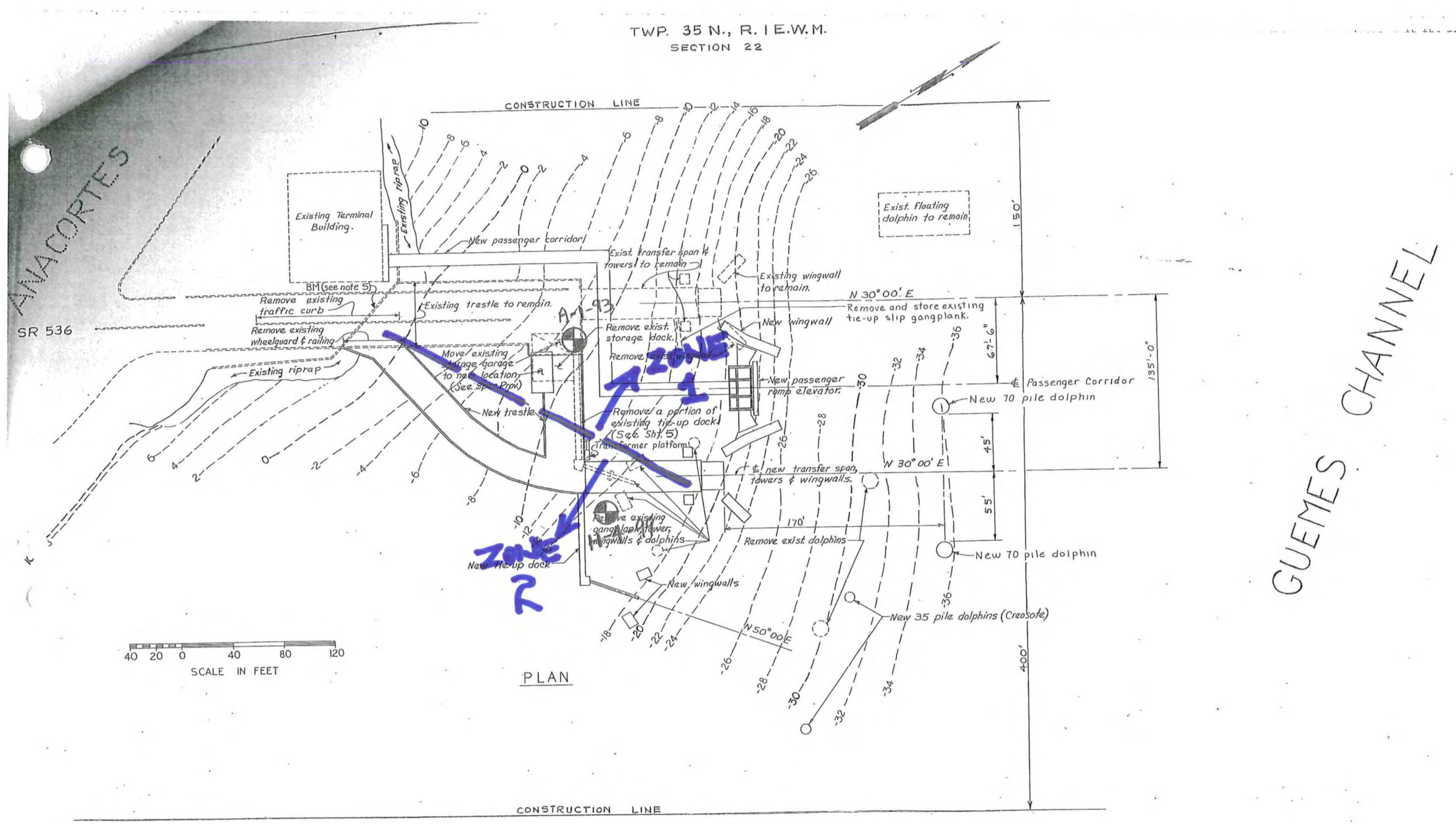
Average Thickness of Potentially Liquefiable Soils

Washington State Ferries Timber Trestles Project
Shaw Island Terminal
Puget Sound Area

GeoEngineers

Figure 14

SP/0180-284-00/Finals/Figures13 – 17.ppt NLU 3/23/12



AVERAGE THICKNESS OF POTENTIALLY LIQUEFIABLE SOILS UNDER ALL DESIGN EQ LEVELS

Trestle Zone	Average Thickness of Potentially Liquefiable Soils (feet)
1	5.0
2	11.0

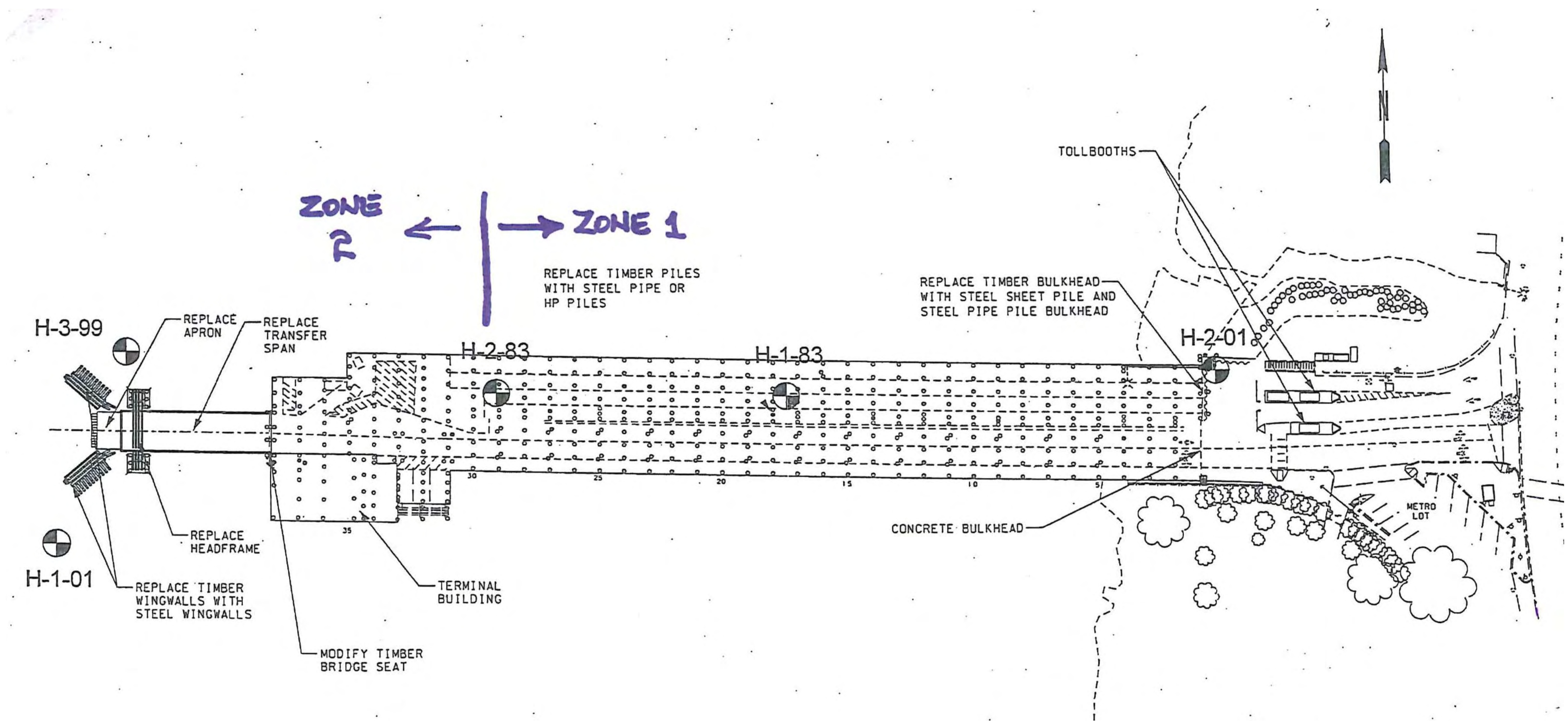
Average Thickness of Potentially Liquefiable Soils

Washington State Ferries Timber Trestles Project
Anacortes Terminal
Puget Sound Area

GeoEngineers

Figure 15

SP/0180-284-00/Finals/Figures13 – 17.ppt NLU 3/23/12



Trestle Zone	Design Earthquake Return Periods (years)	Average Thickness of Potentially Liquefiable Soils (feet)
1	72, 224, 475 & 975	4.0
2	72	5.0
	224	12.0
	475	22.0
	975	22.0

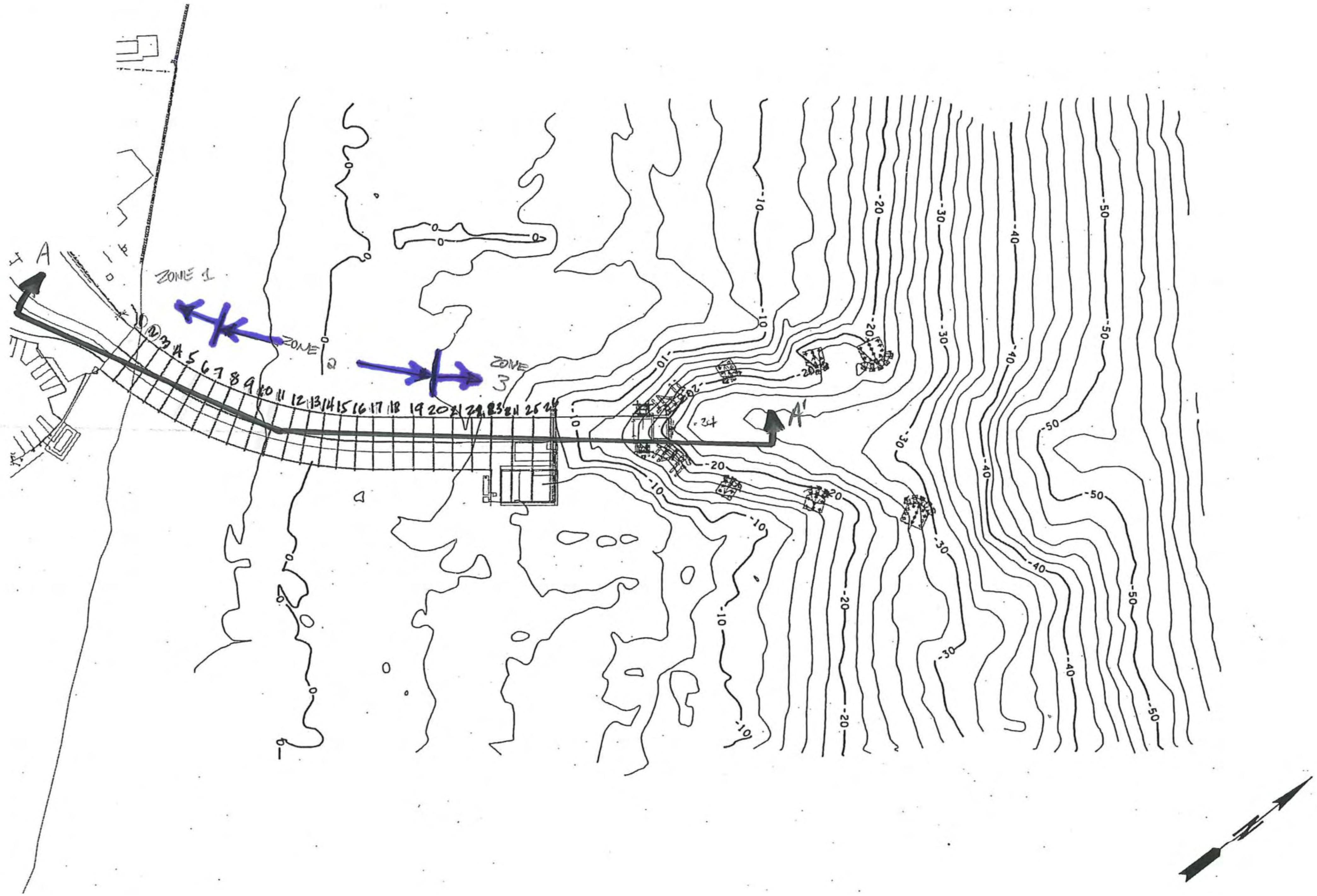
Average Thickness of Potentially Liquefiable Soils

Washington State Ferries Timber Trestles Project
Fauntleroy Terminal
Puget Sound Area

GeoEngineers

Figure 16

SP/0180-284-00/Finals/Figures13 – 17.ppt NLU 3/23/12



AVERAGE THICKNESS OF POTENTIALLY LIQUEFIABLE SOILS UNDER ALL DESIGN EQ LEVELS

Trestle Zone	Average Thickness of Potentially Liquefiable Soils (feet)
1	2.5
2	7.5
3	10.0

Average Thickness of Potentially Liquefiable Soils

Washington State Ferries Timber Trestles Project
Southworth Terminal
Puget Sound Area

GEOENGINEERS

Figure 17

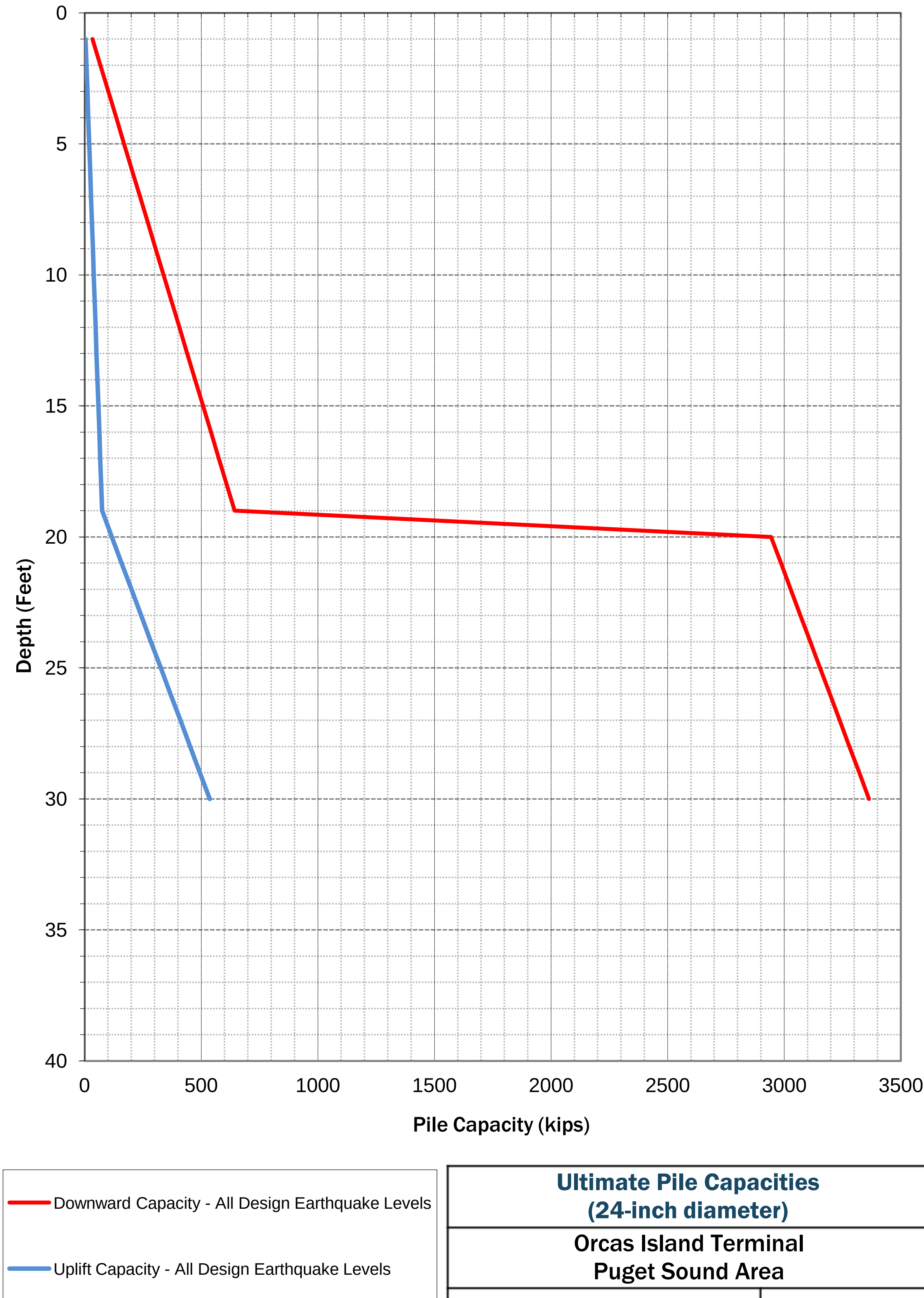
SP/0180-284-00/Finals/Figures13 – 17.ppt NLU 3/23/12

Ultimate Pile Capacities (16-inch diameter)

Orcas Island Terminal
Puget Sound Area

GeoEngineers

Figure 18



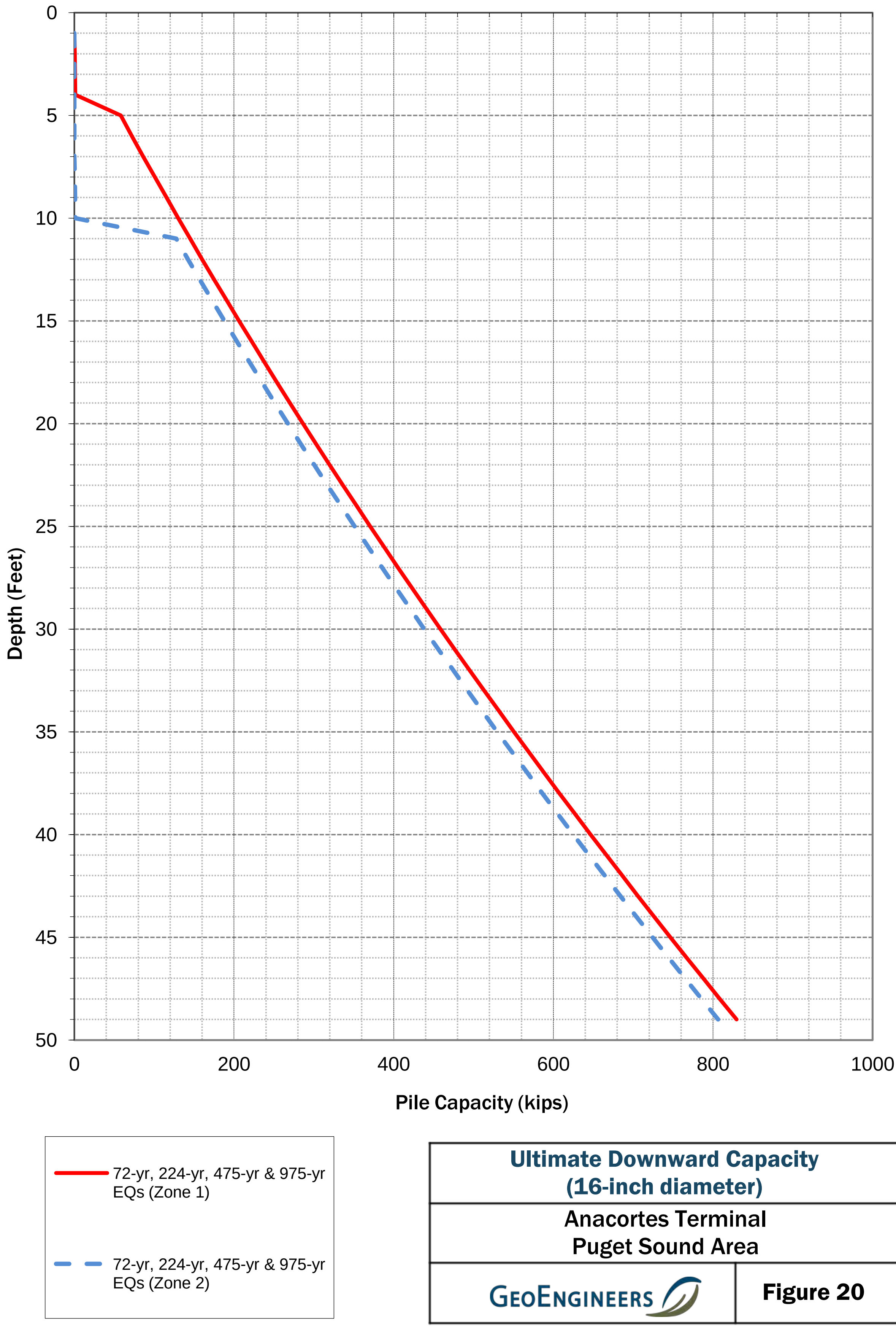
SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 1/30/12

Ultimate Pile Capacities (24-inch diameter)

Orcas Island Terminal Puget Sound Area

GeoEngineers

Figure 19



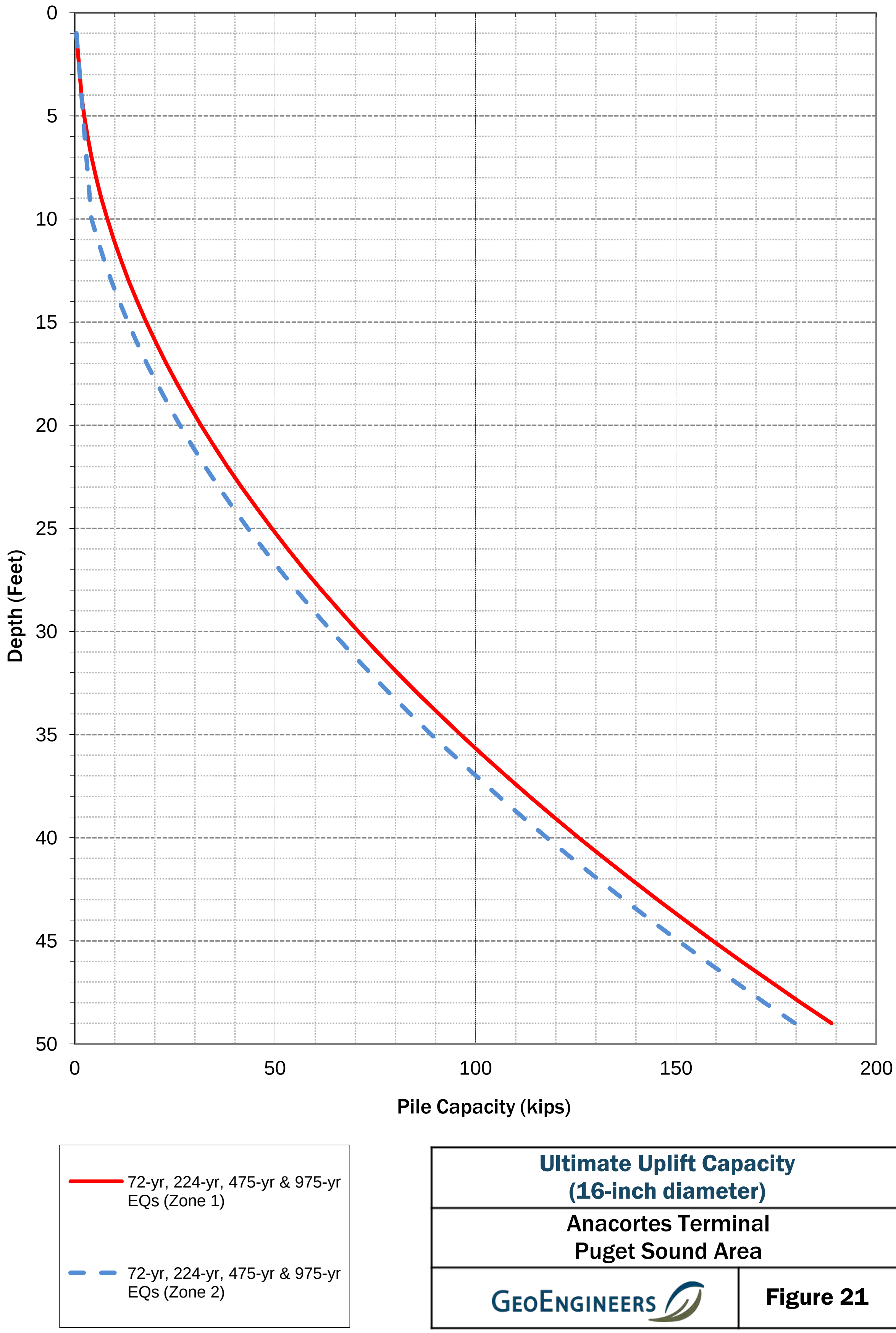
Ultimate Downward Capacity (16-inch diameter)

Anacortes Terminal
Puget Sound Area

GEOENGINEERS

Figure 20

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



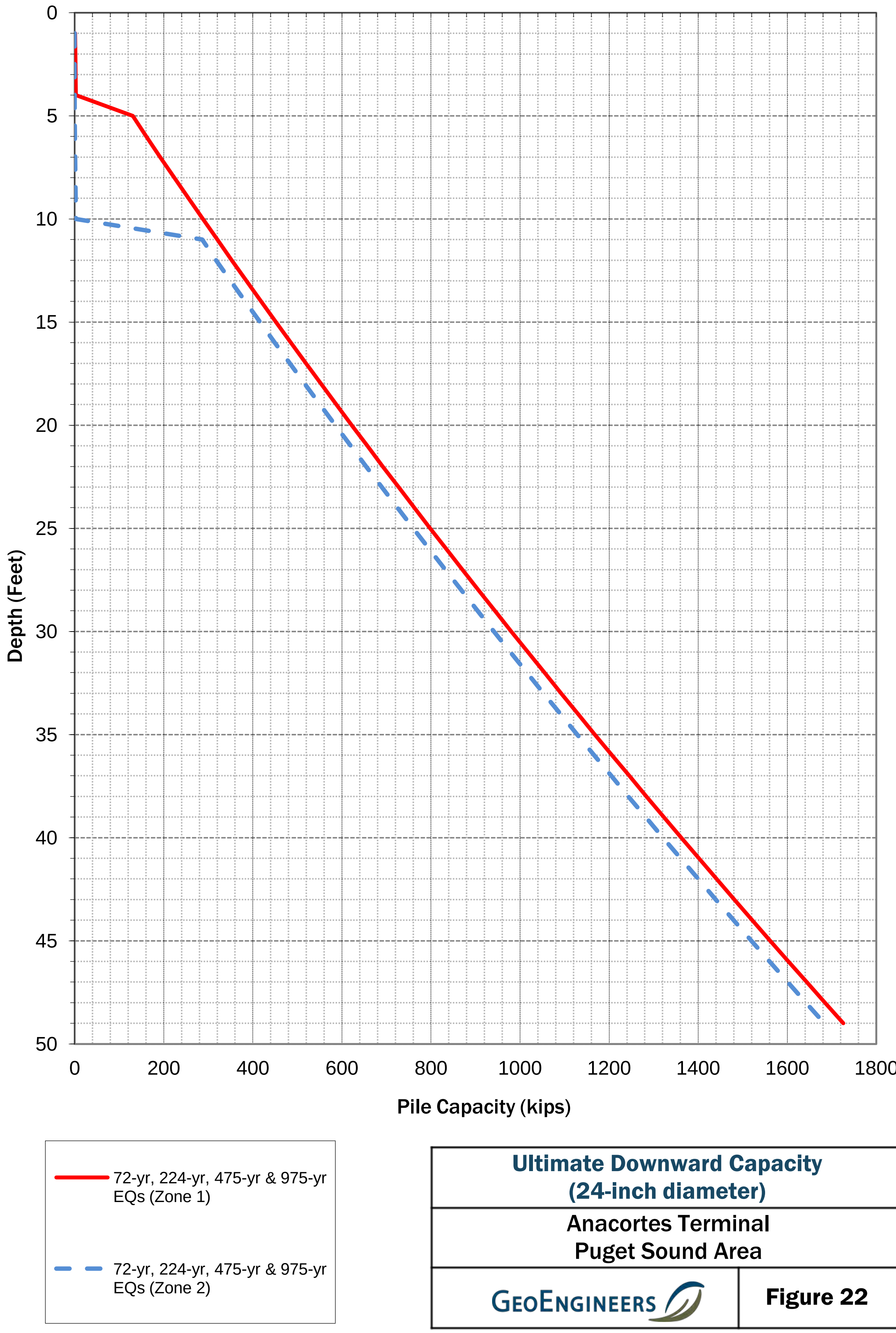
Ultimate Uplift Capacity (16-inch diameter)

Anacortes Terminal
Puget Sound Area

GEOENGINEERS

Figure 21

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



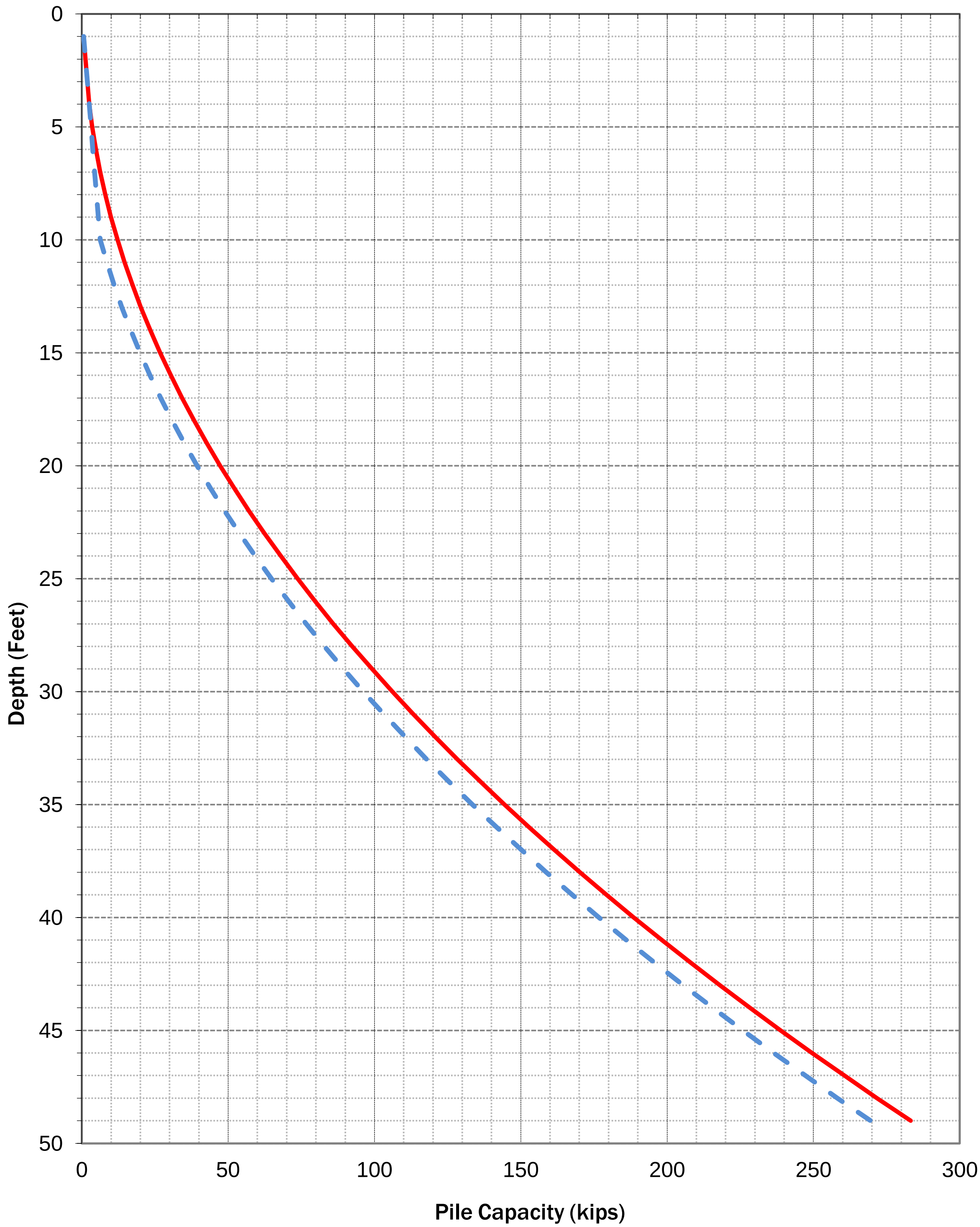
Ultimate Downward Capacity (24-inch diameter)

Anacortes Terminal
Puget Sound Area

GEOENGINEERS

Figure 22

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 1)

72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 2)

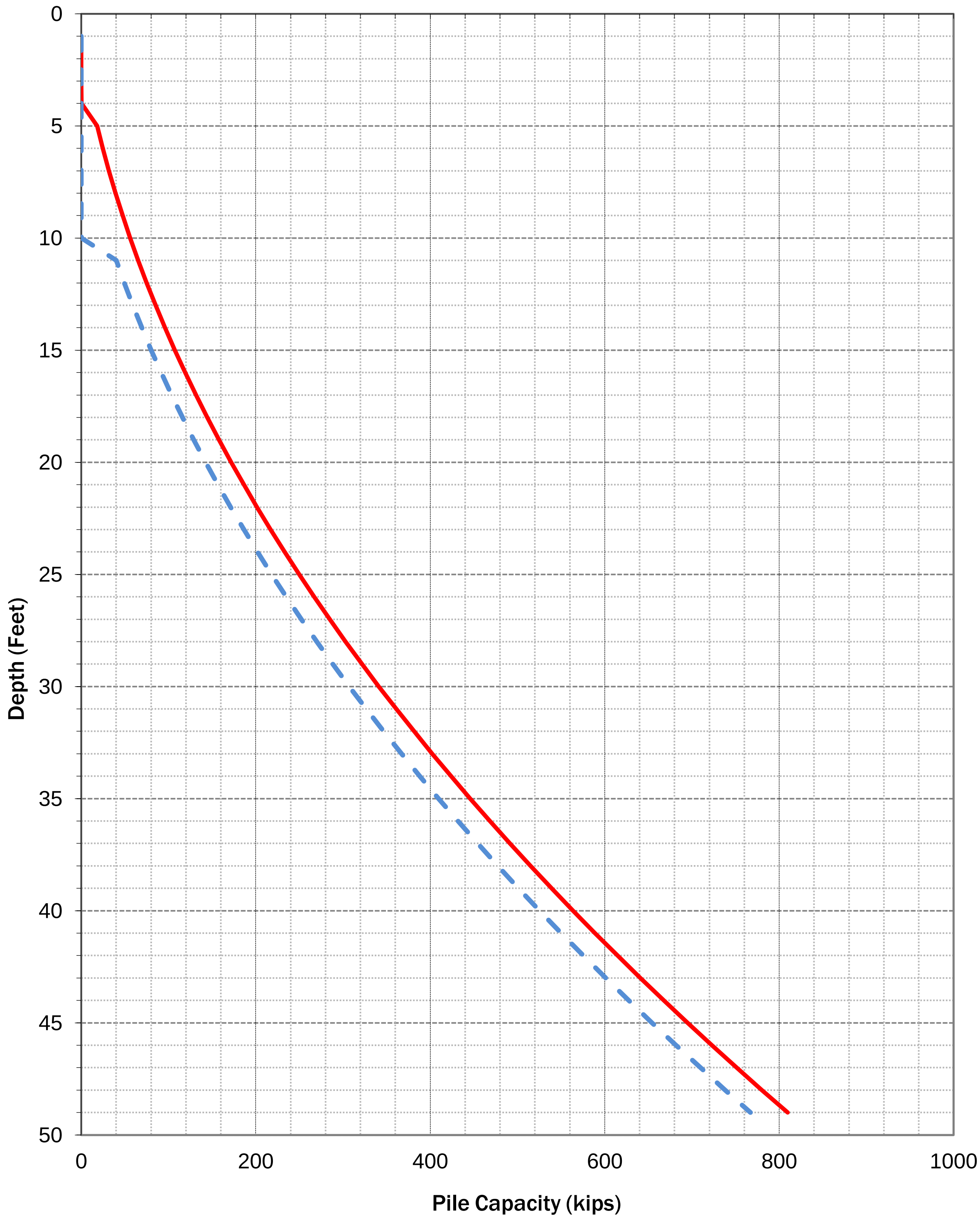
Ultimate Uplift Capacity (24-inch diameter)

Anacortes Terminal Puget Sound Area

GEOENGINEERS

Figure 23

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 1)

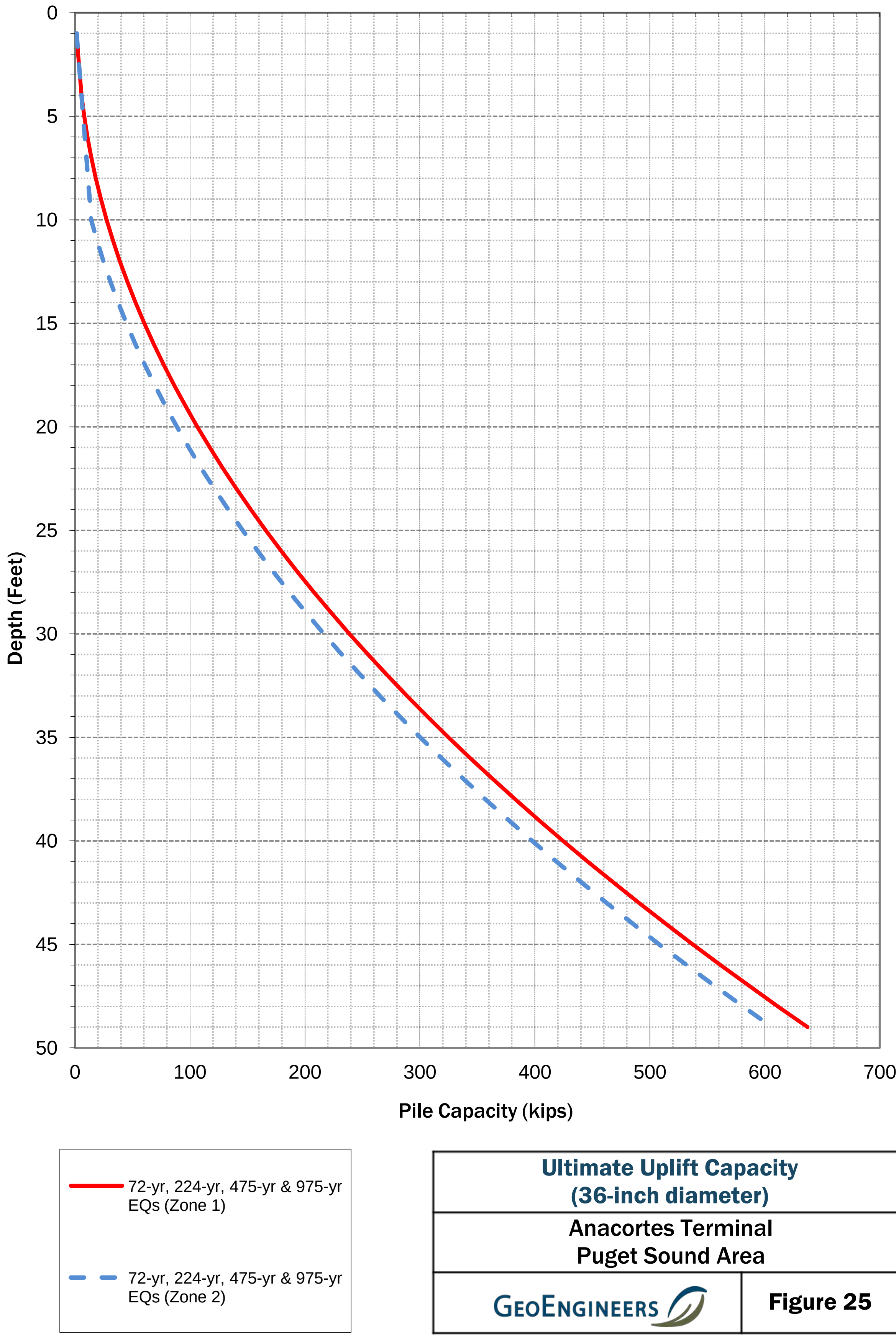
72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 2)

Ultimate Downward Capacity (36-inch diameter)

Anacortes Terminal Puget Sound Area

GeoEngineers

Figure 24



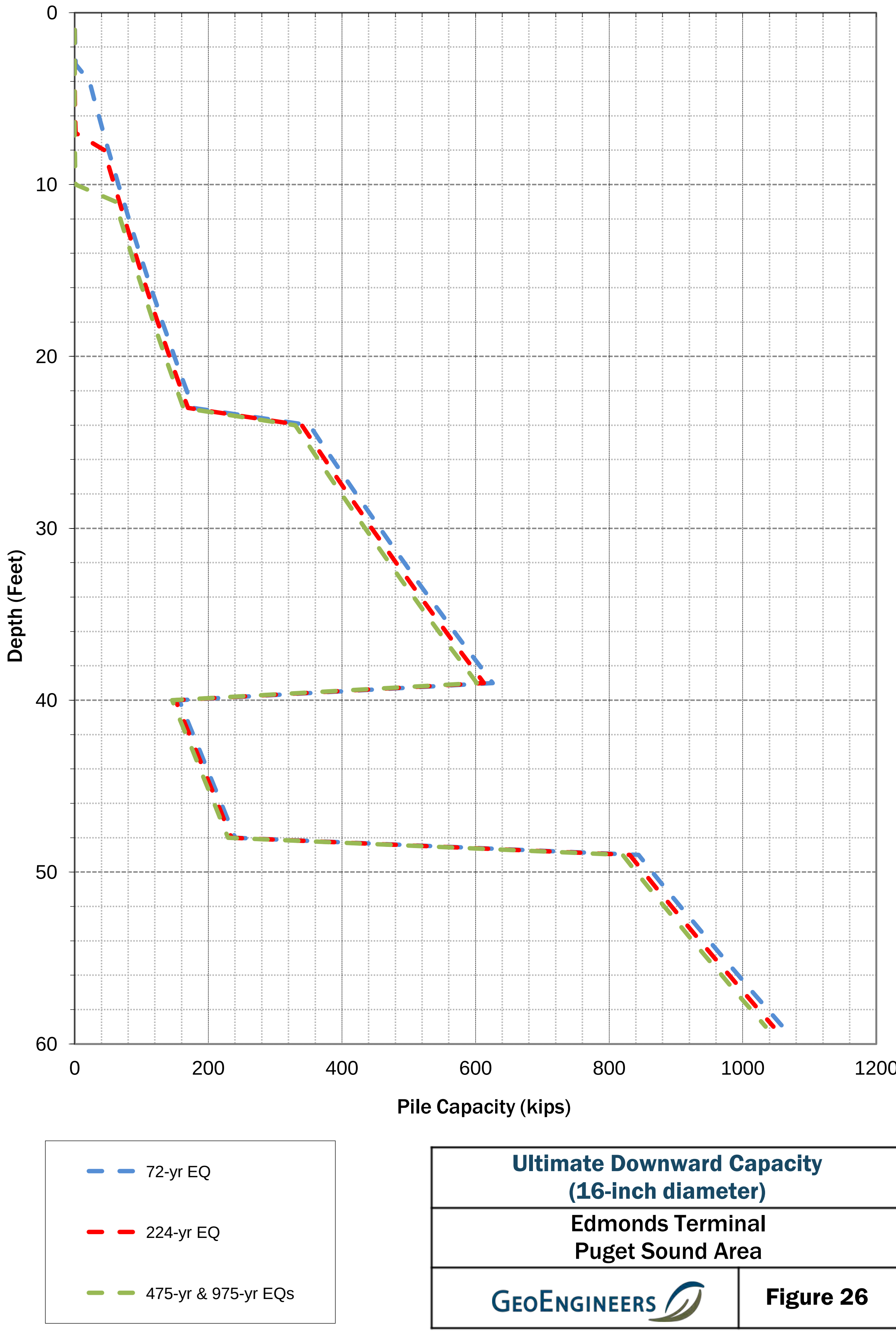
Ultimate Uplift Capacity (36-inch diameter)

Anacortes Terminal
Puget Sound Area

GEOENGINEERS

Figure 25

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



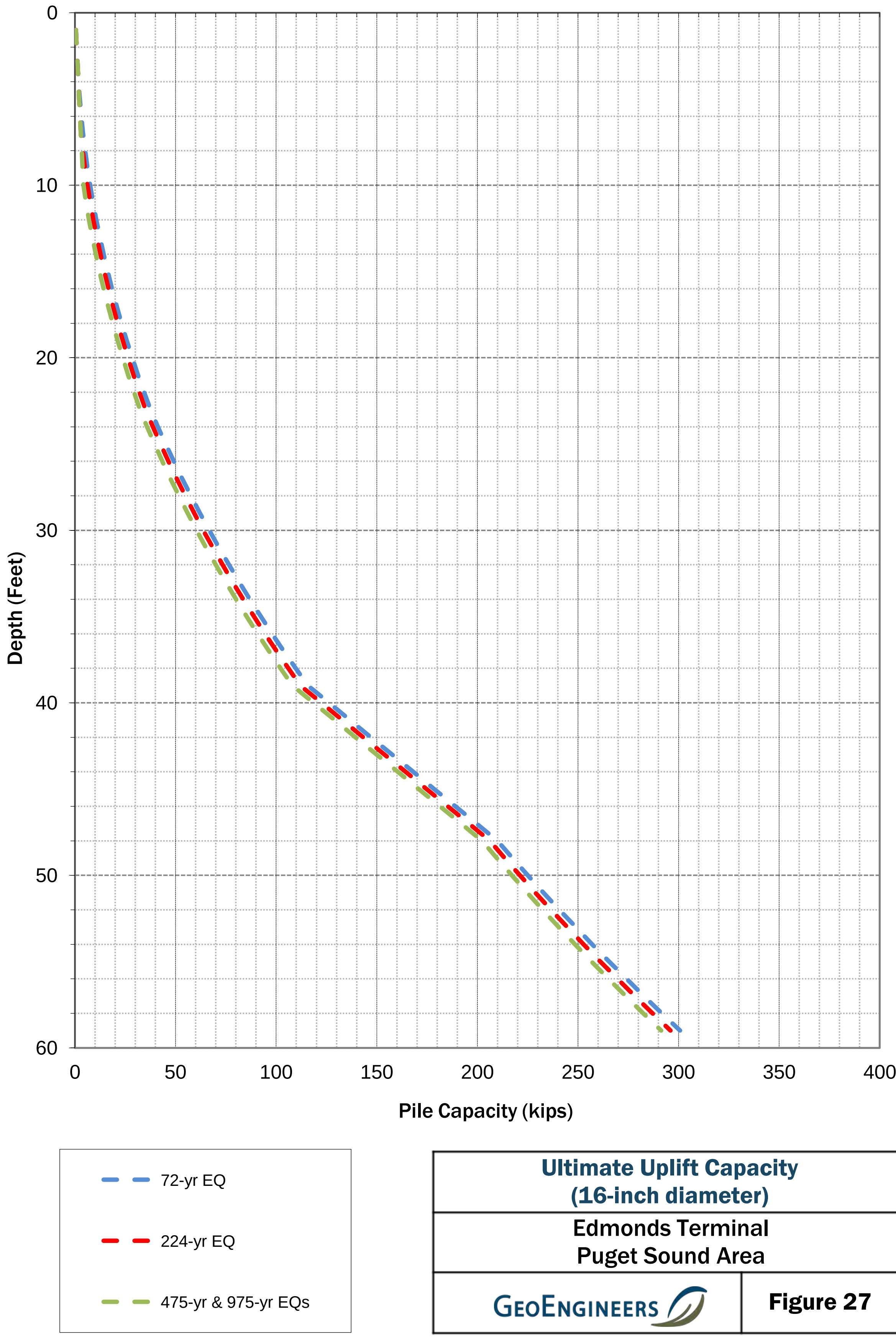
Ultimate Downward Capacity (16-inch diameter)

Edmonds Terminal
Puget Sound Area

GEOENGINEERS

Figure 26

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



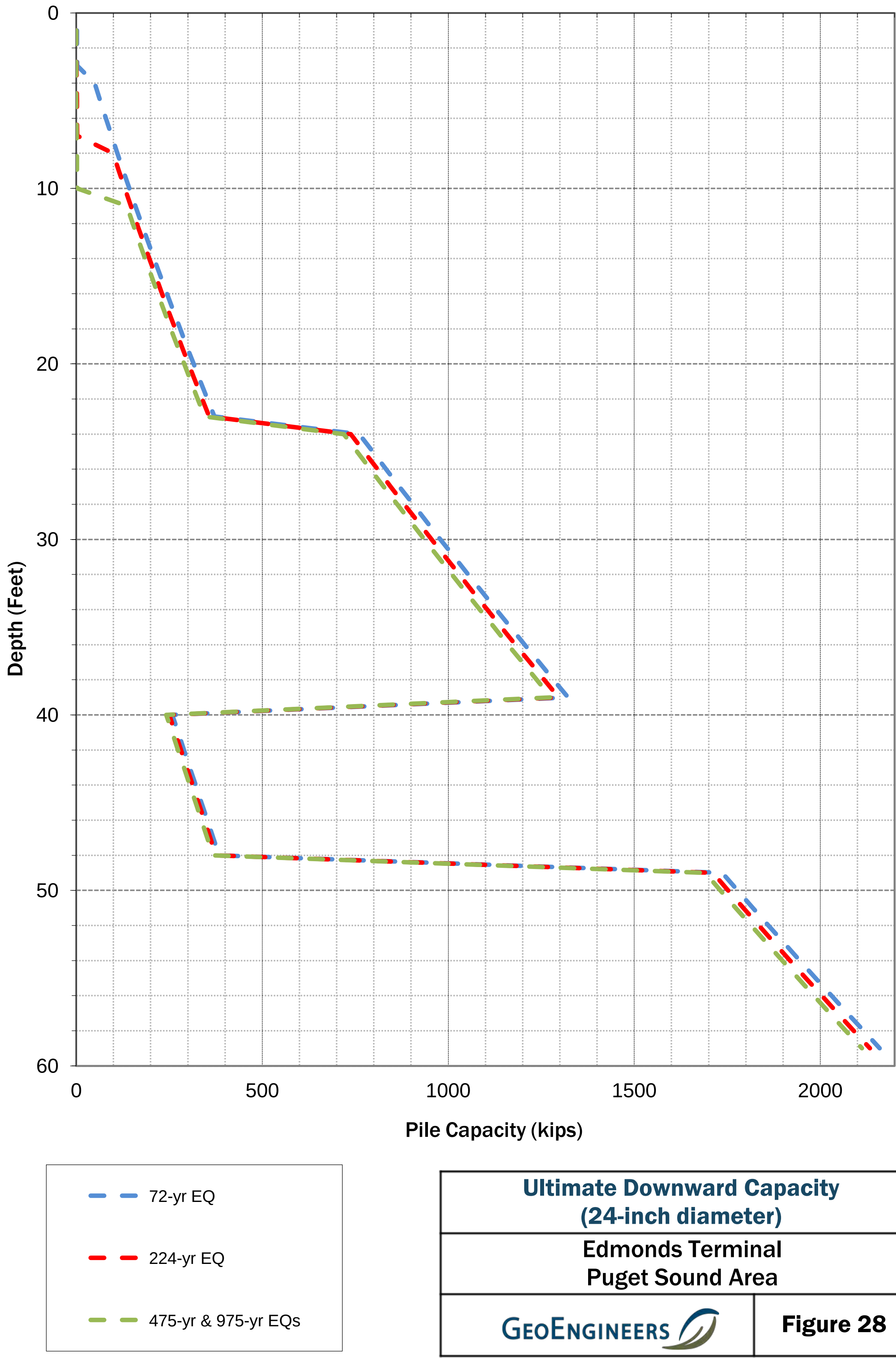
Ultimate Uplift Capacity (16-inch diameter)

Edmonds Terminal Puget Sound Area

GEOENGINEERS

Figure 27

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



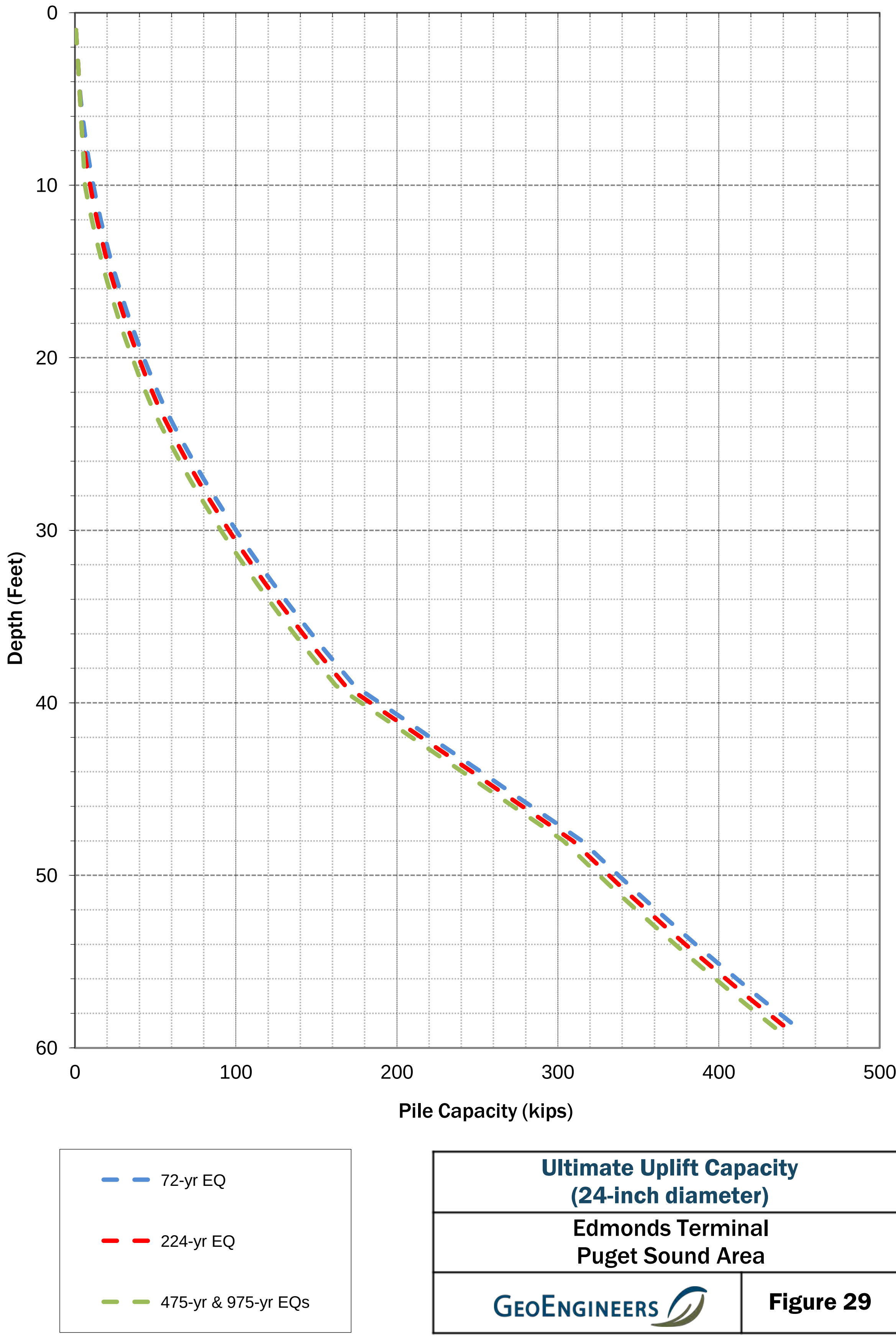
Ultimate Downward Capacity (24-inch diameter)

Edmonds Terminal
Puget Sound Area

GeoEngineers

Figure 28

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



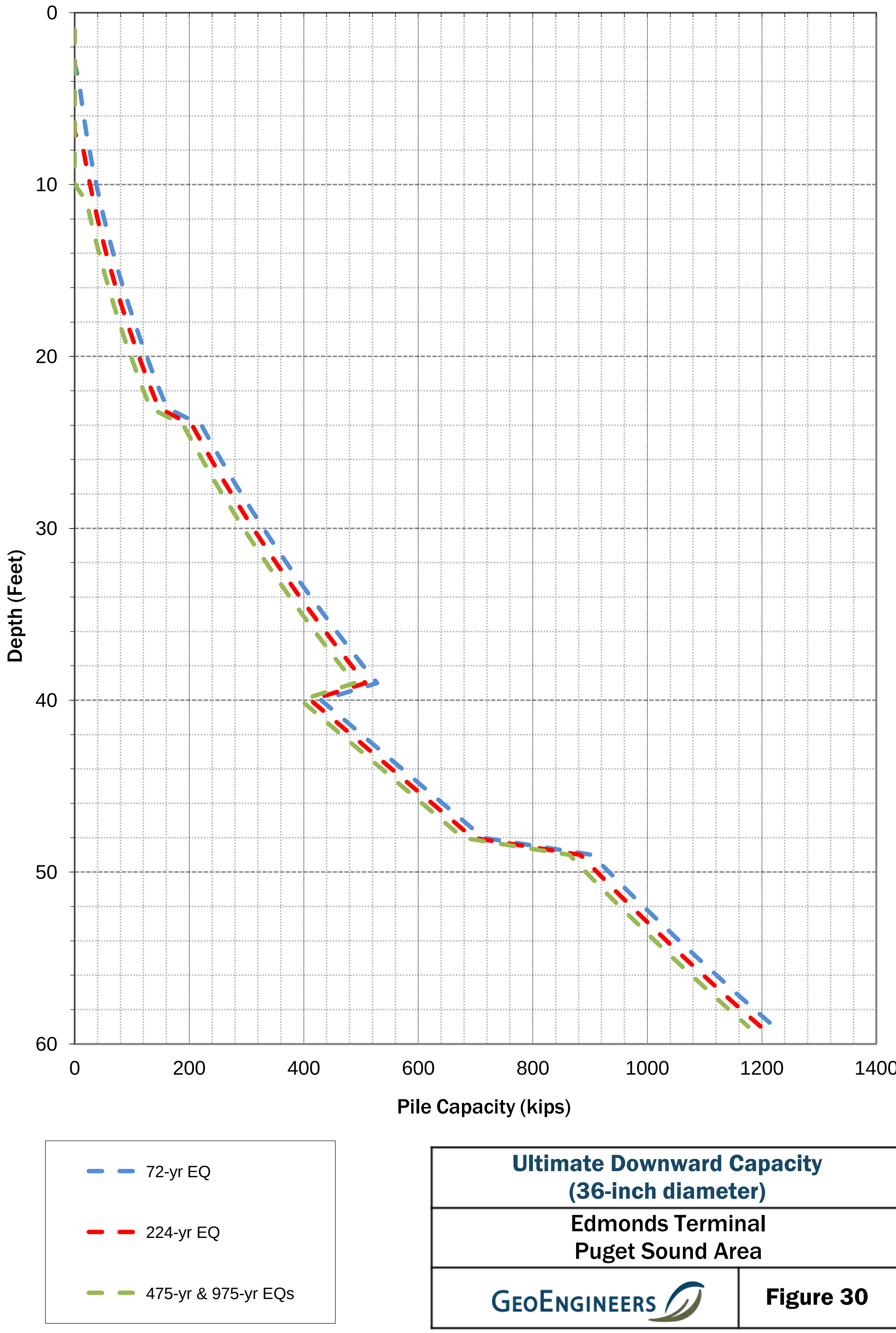
Ultimate Uplift Capacity (24-inch diameter)

Edmonds Terminal
Puget Sound Area

GEOENGINEERS

Figure 29

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



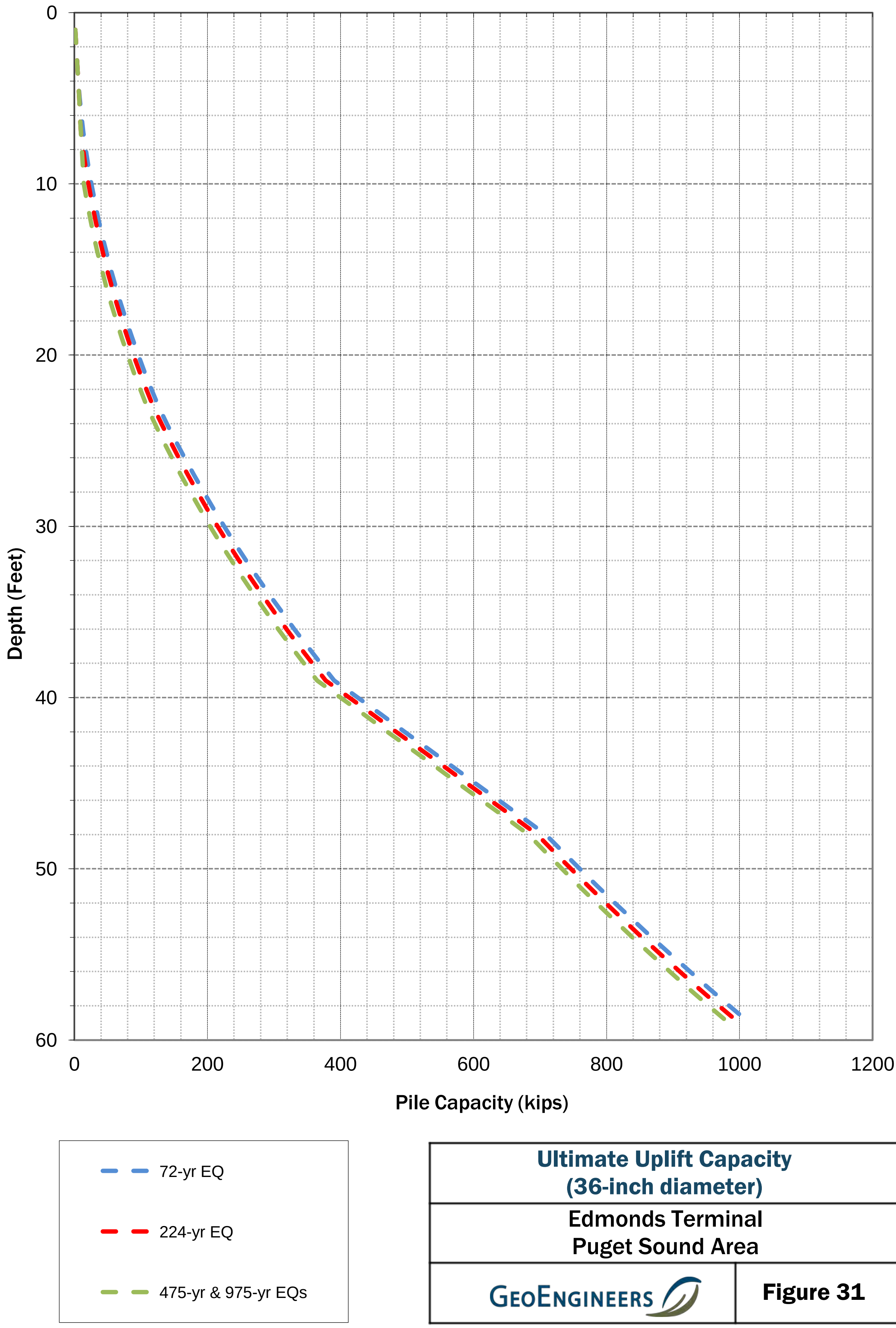
Ultimate Downward Capacity (36-inch diameter)

Edmonds Terminal
Puget Sound Area

GeoEngineers

Figure 30

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

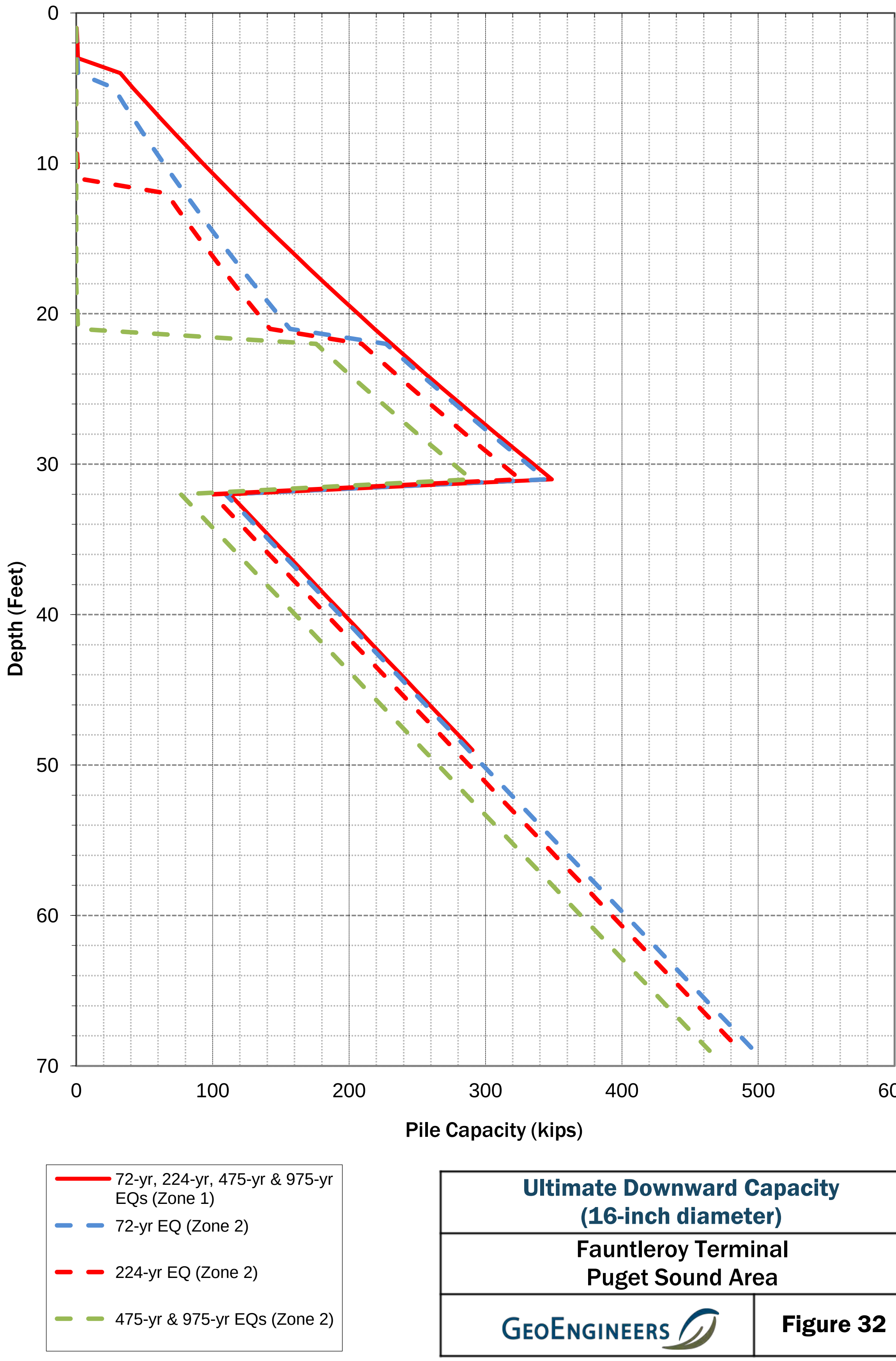


Ultimate Uplift Capacity (36-inch diameter)

Edmonds Terminal
Puget Sound Area

GeoEngineers

Figure 31

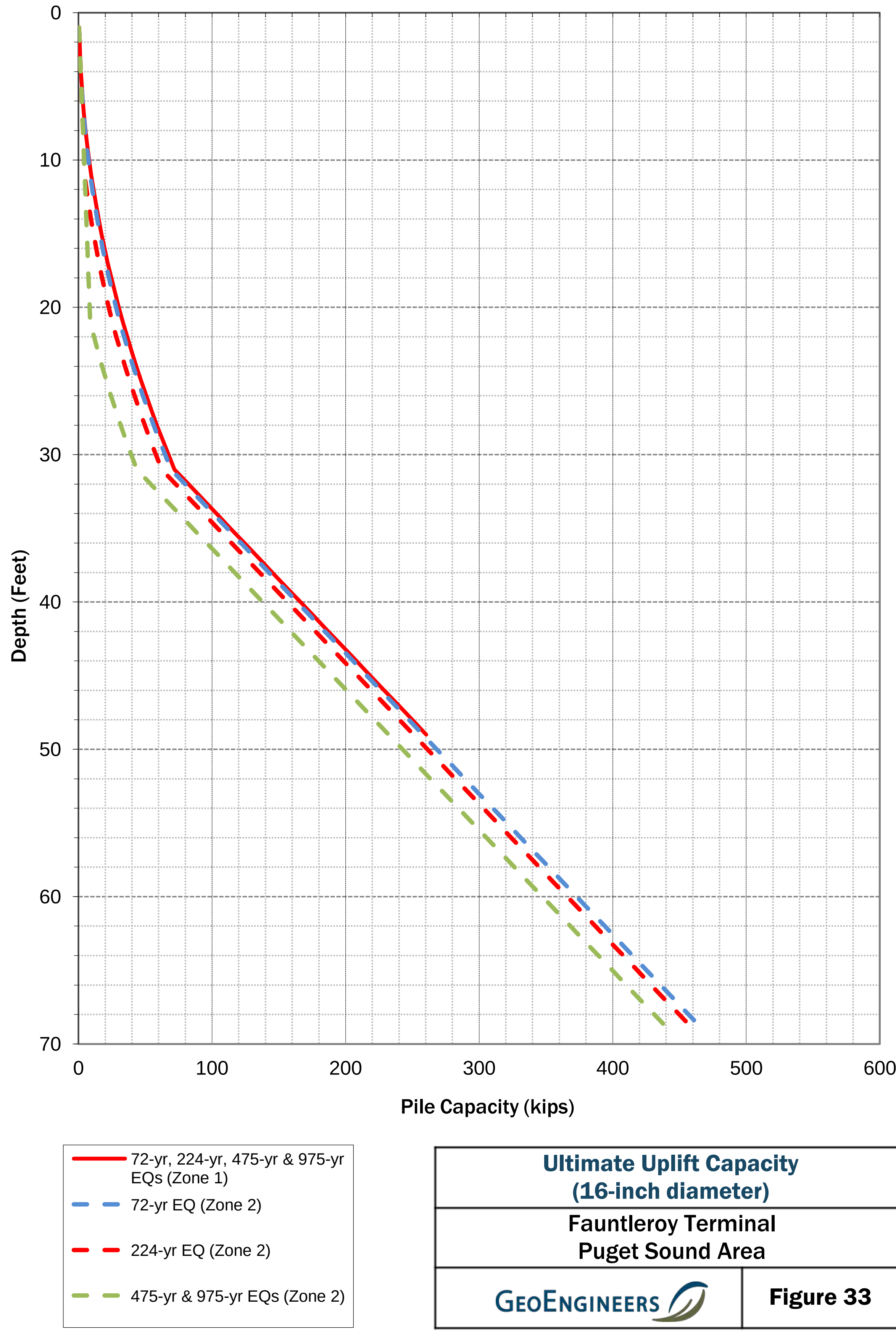


Ultimate Downward Capacity (16-inch diameter)

Fauntleroy Terminal
Puget Sound Area

GEOENGINEERS

Figure 32



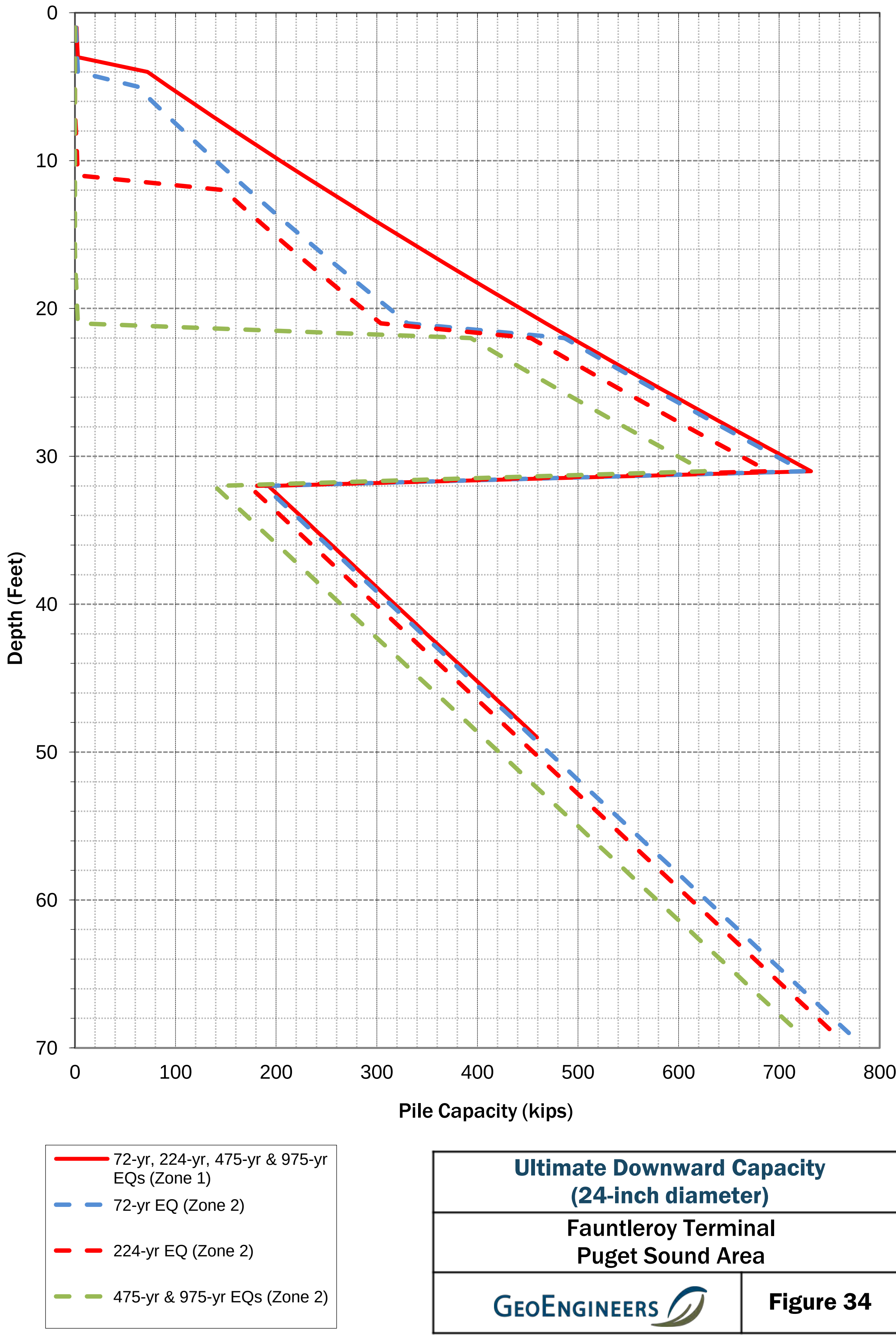
Ultimate Uplift Capacity (16-inch diameter)

Fauntleroy Terminal Puget Sound Area

GEOENGINEERS

Figure 33

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



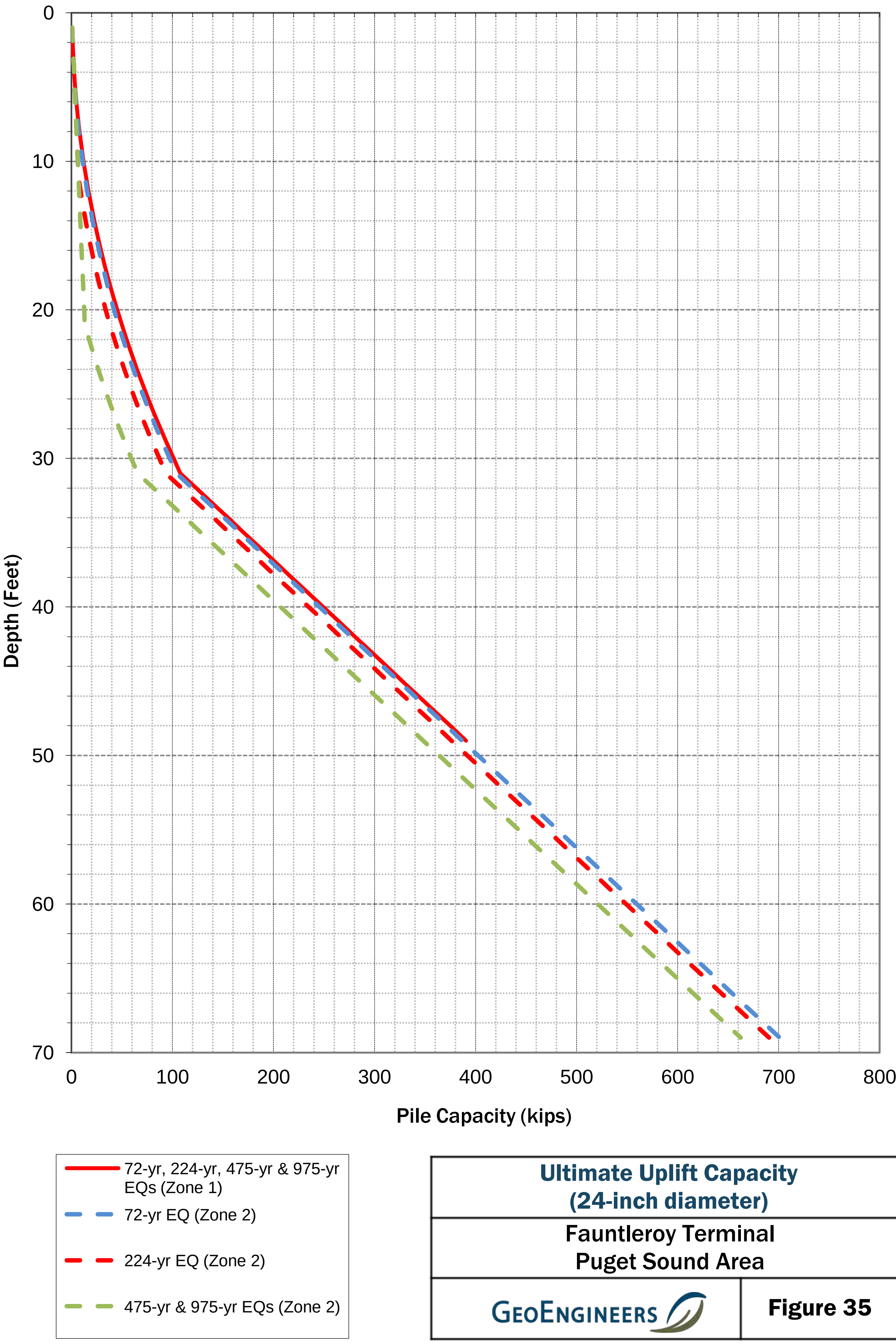
Ultimate Downward Capacity (24-inch diameter)

Fauntleroy Terminal
Puget Sound Area

GEOENGINEERS

Figure 34

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



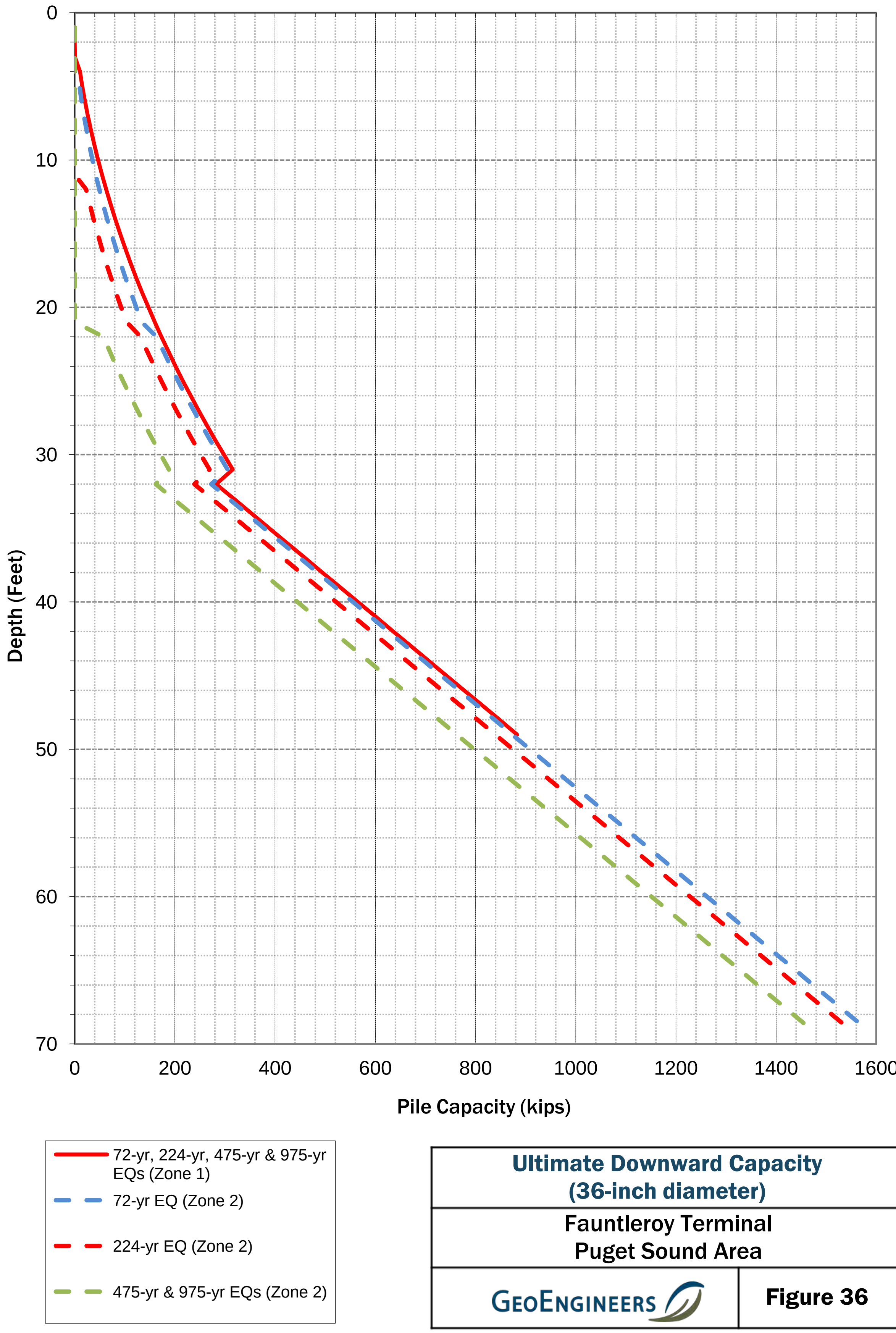
Ultimate Uplift Capacity (24-inch diameter)

Fauntleroy Terminal
Puget Sound Area

GEOENGINEERS

Figure 35

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



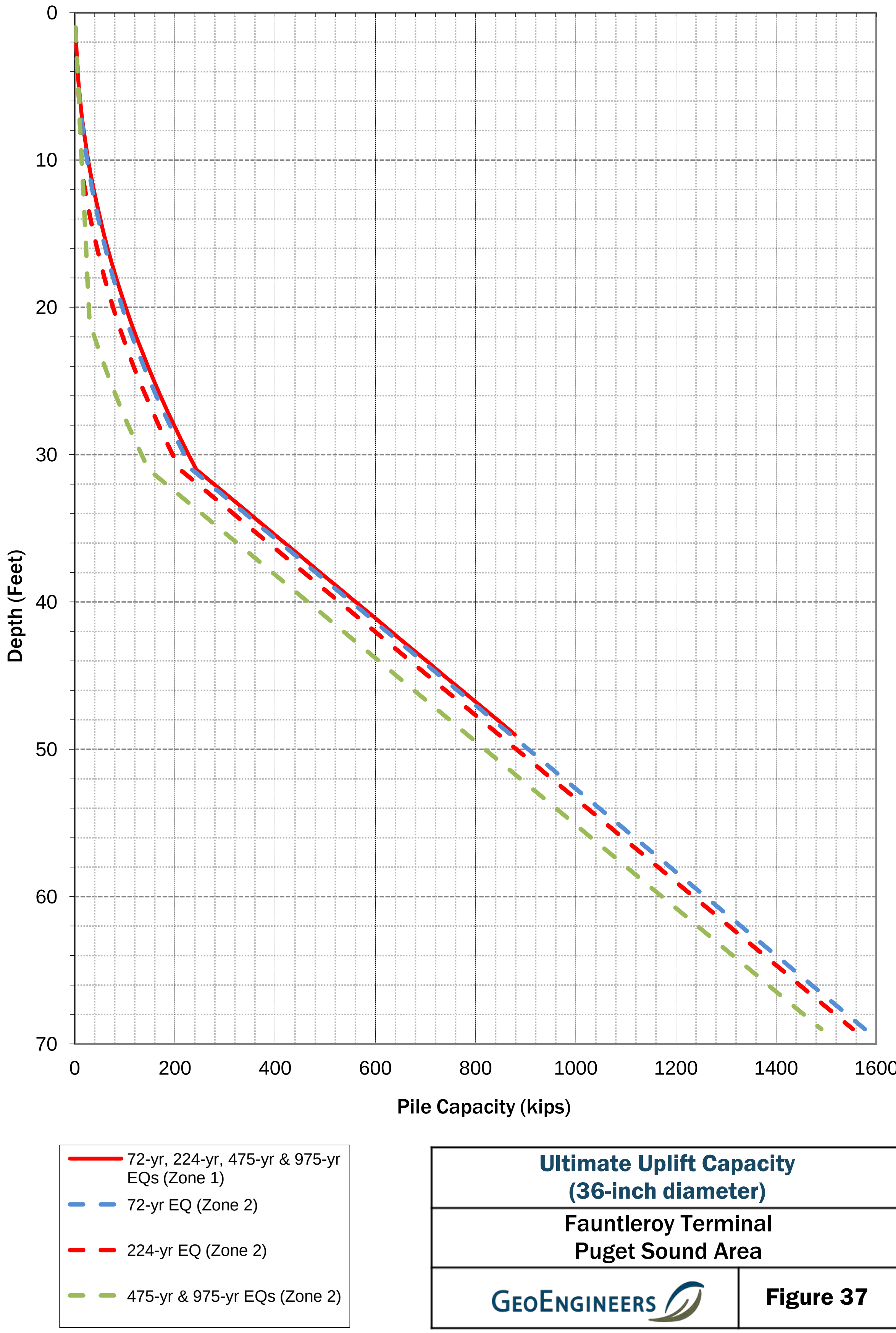
Ultimate Downward Capacity (36-inch diameter)

Fauntleroy Terminal
Puget Sound Area

GEOENGINEERS

Figure 36

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



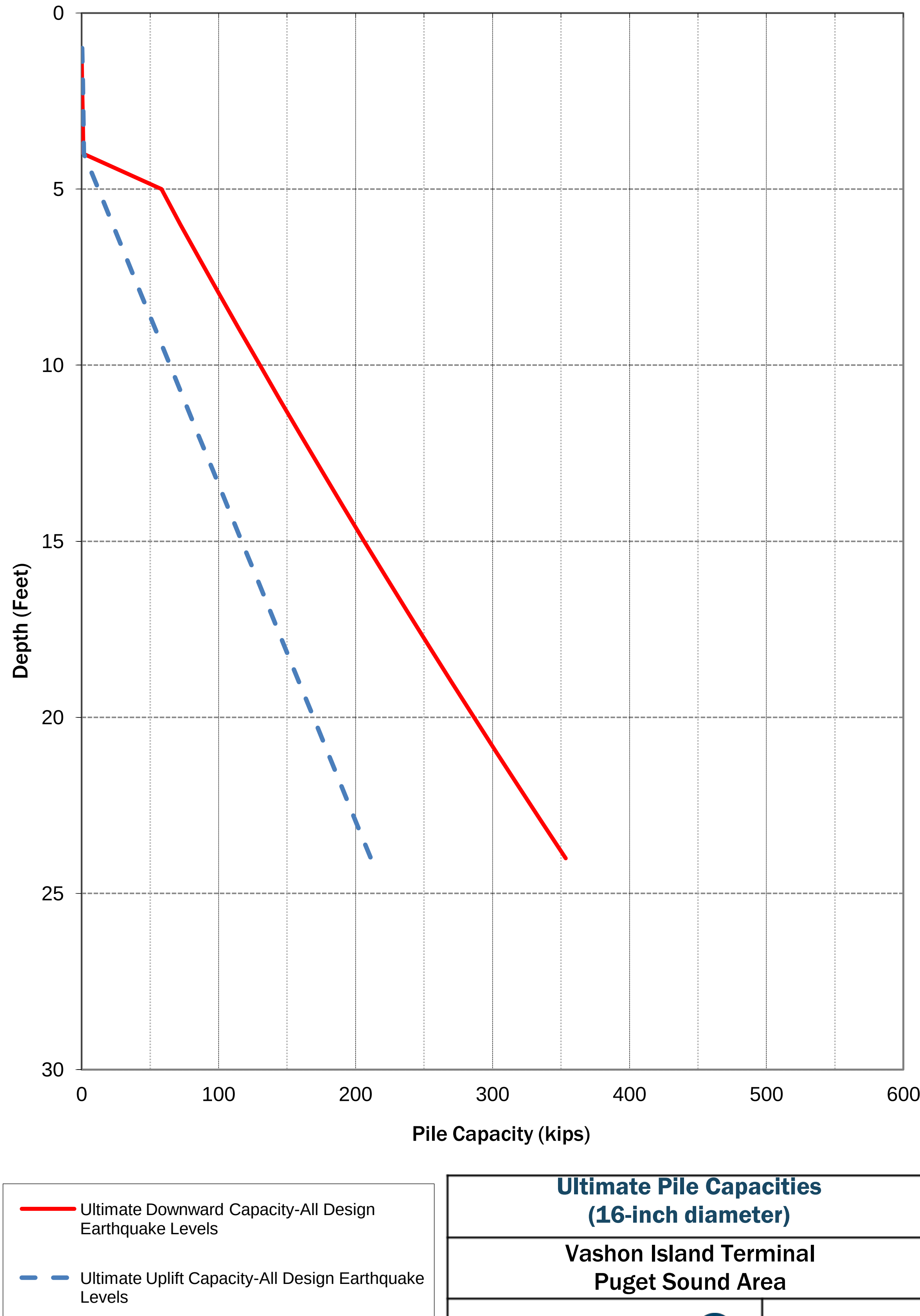
Ultimate Uplift Capacity (36-inch diameter)

Fauntleroy Terminal
Puget Sound Area

GEOENGINEERS

Figure 37

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



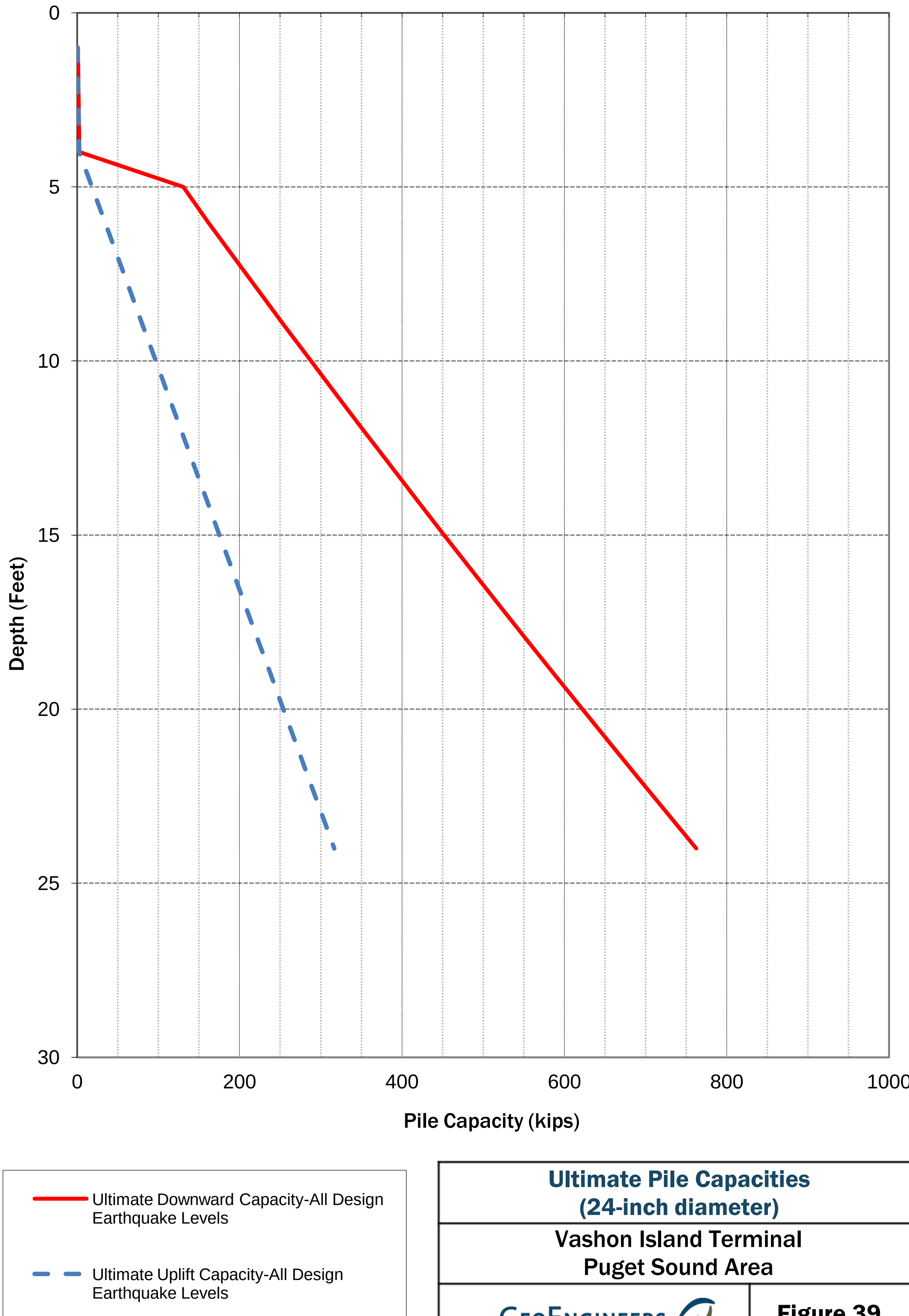
Ultimate Pile Capacities (16-inch diameter)

Vashon Island Terminal
Puget Sound Area

GEOENGINEERS

Figure 38

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity_16inch.xls khc:hpd 11/12/11

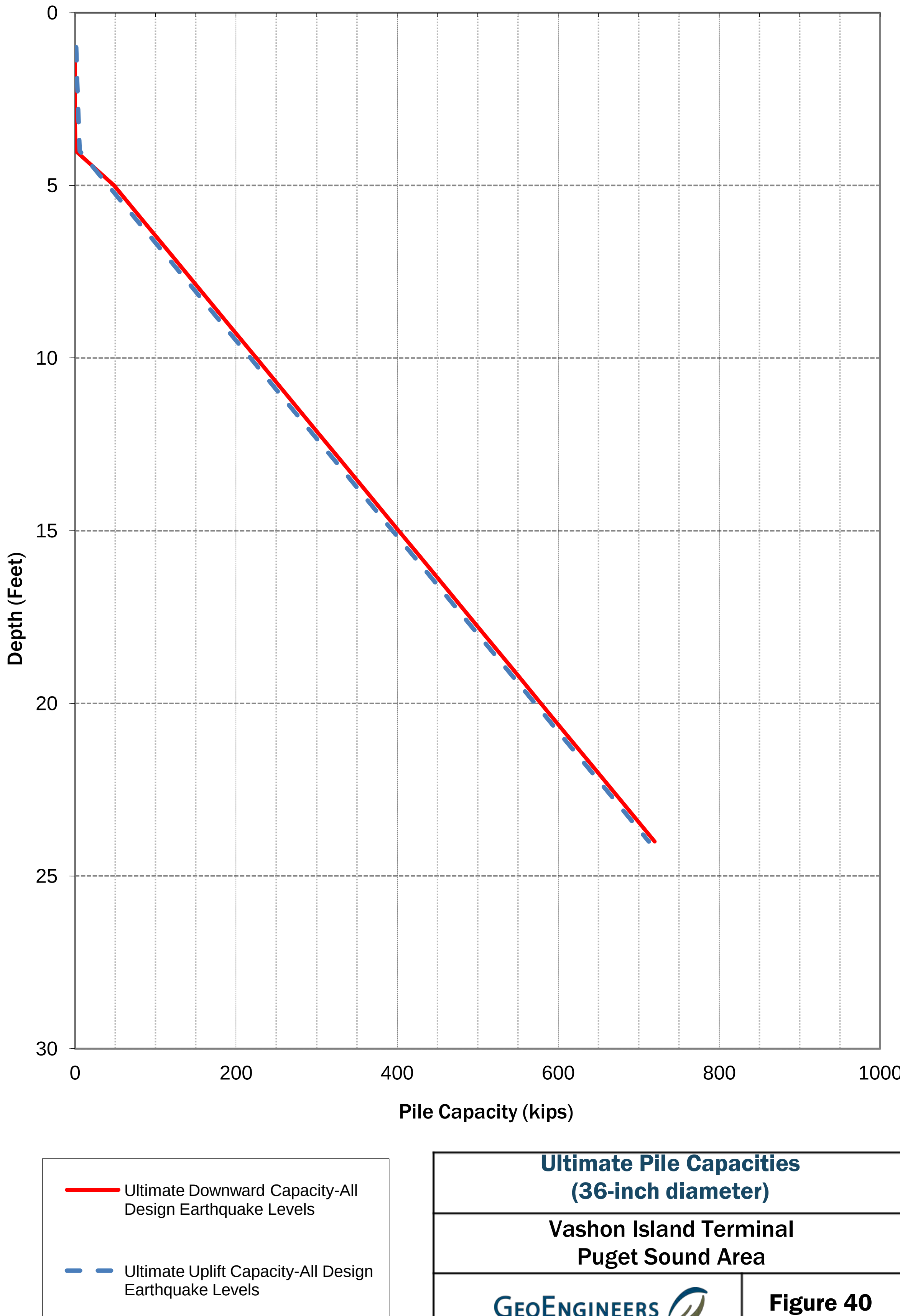


Ultimate Pile Capacities (24-inch diameter)

Vashon Island Terminal Puget Sound Area

GEOENGINEERS

Figure 39

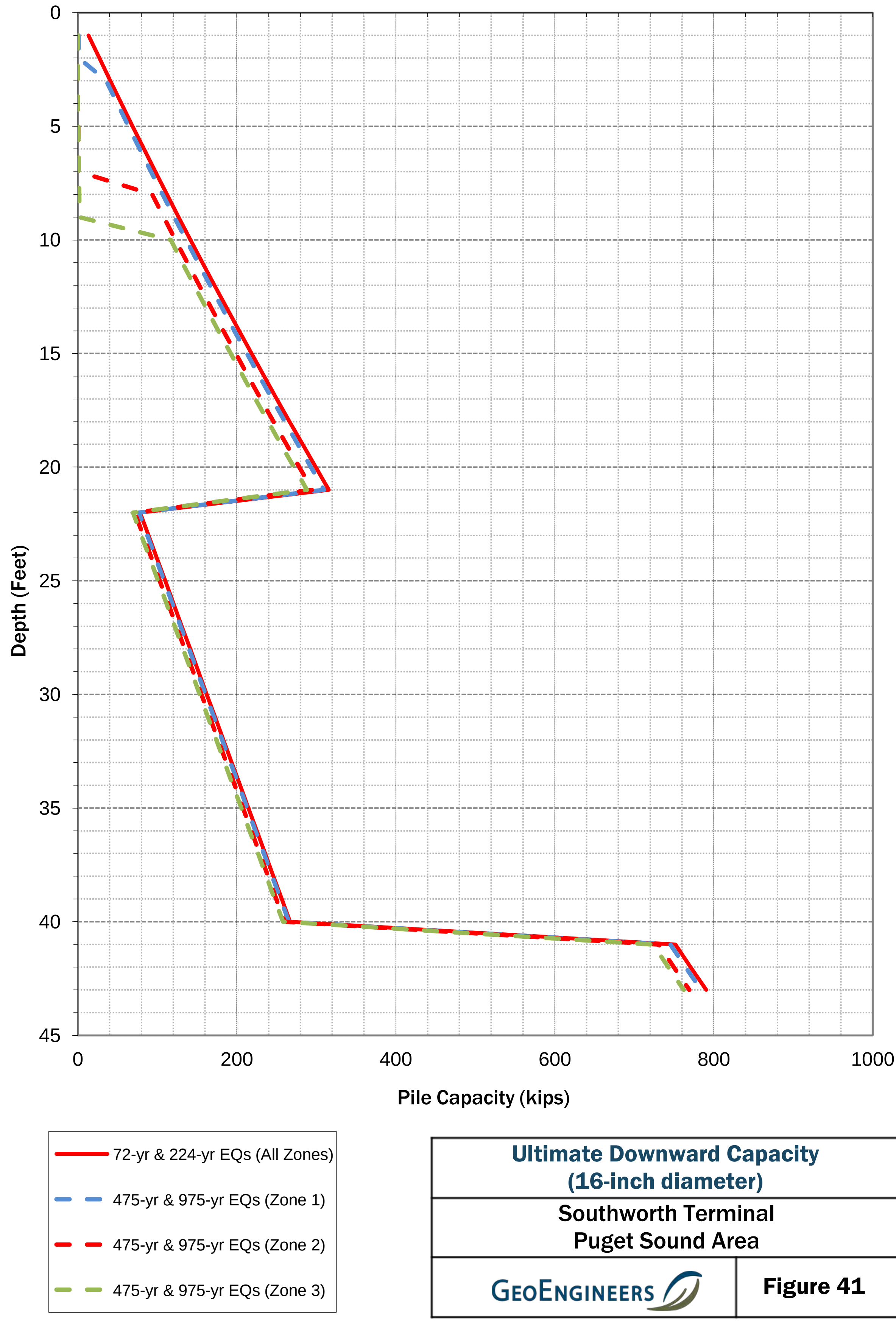


Ultimate Pile Capacities (36-inch diameter)

Vashon Island Terminal
Puget Sound Area

GeoEngineers

Figure 40

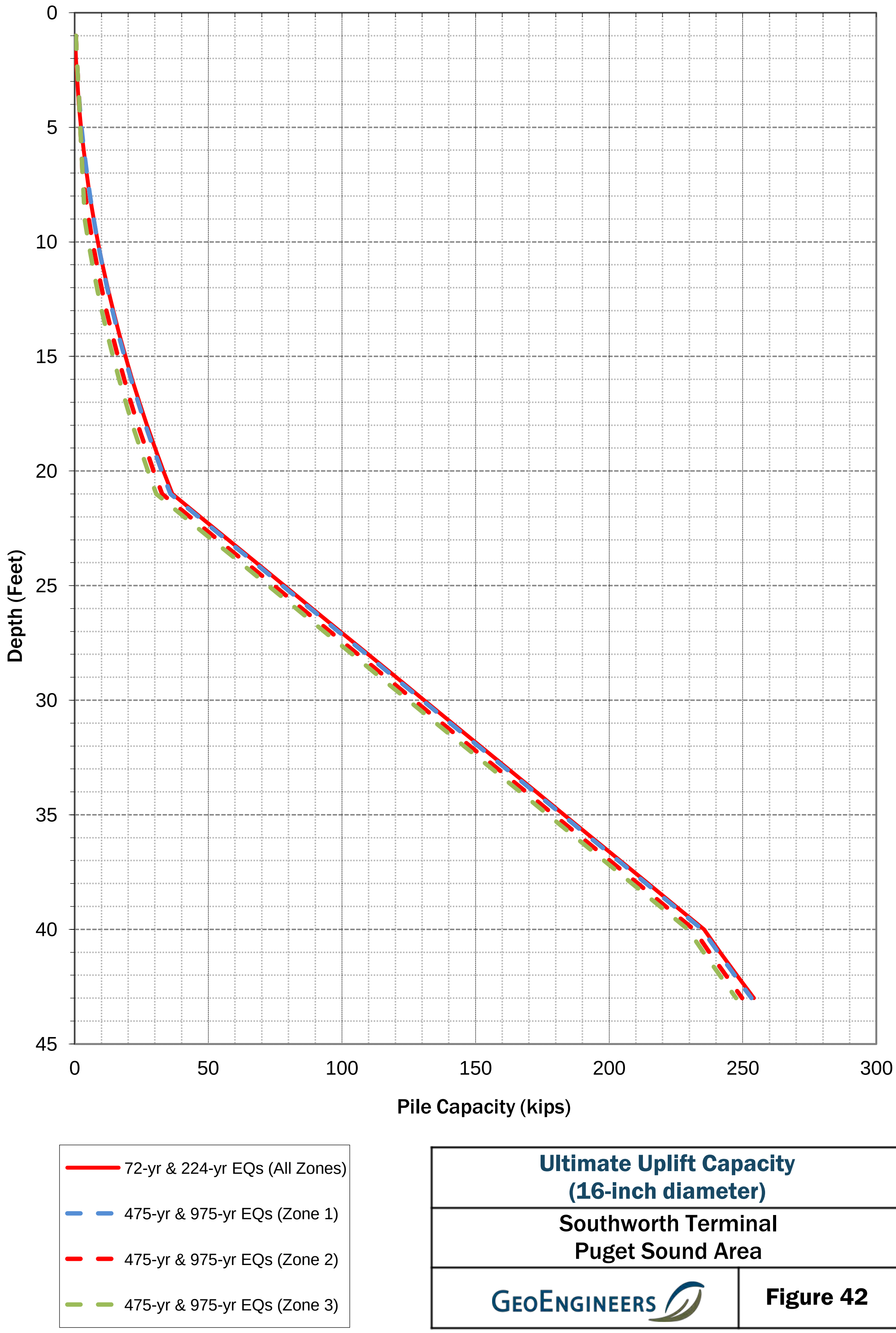


Ultimate Downward Capacity (16-inch diameter)

Southworth Terminal
Puget Sound Area

GeoEngineers

Figure 41

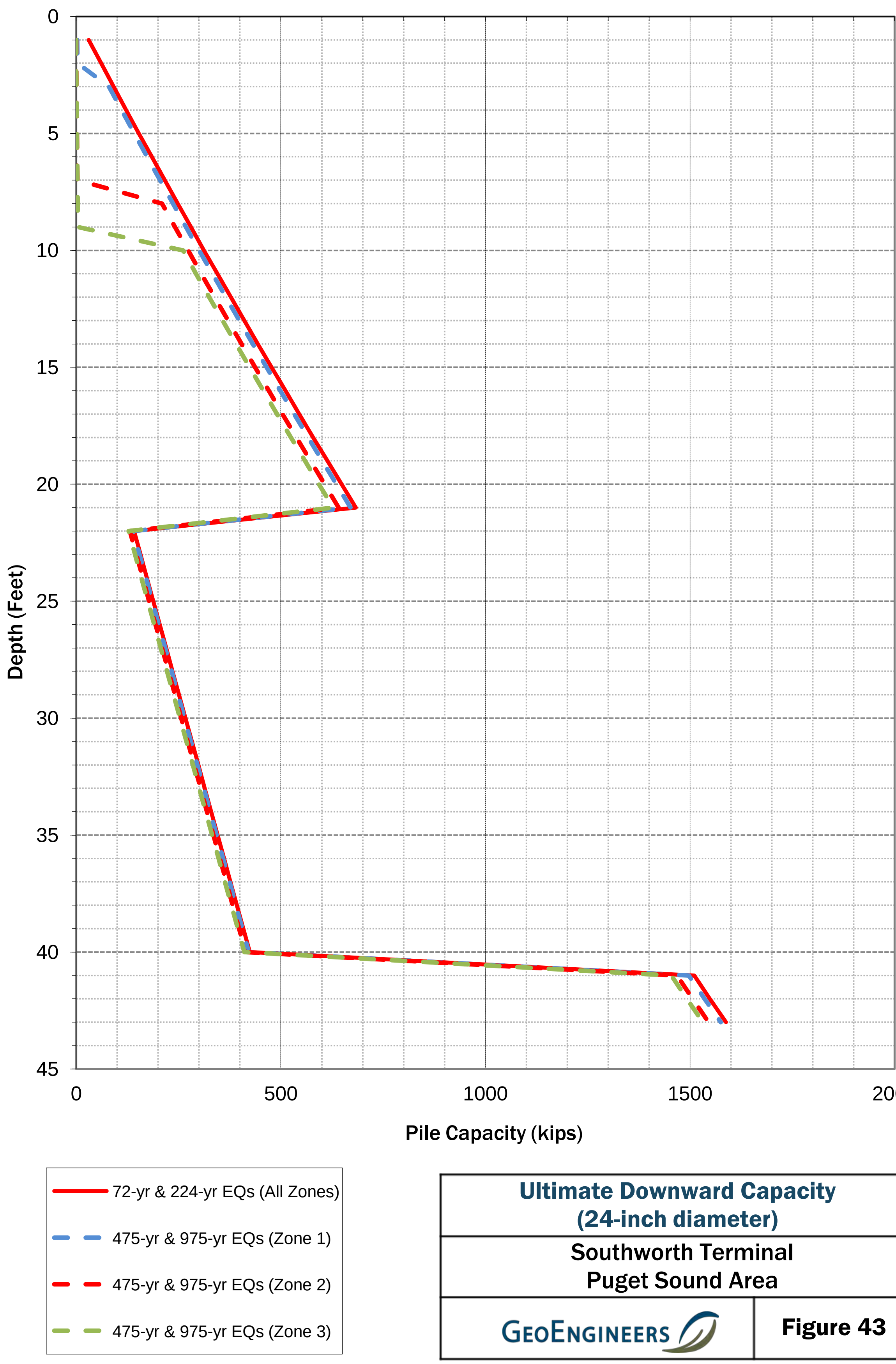


Ultimate Uplift Capacity (16-inch diameter)

Southworth Terminal
Puget Sound Area

GeoEngineers

Figure 42



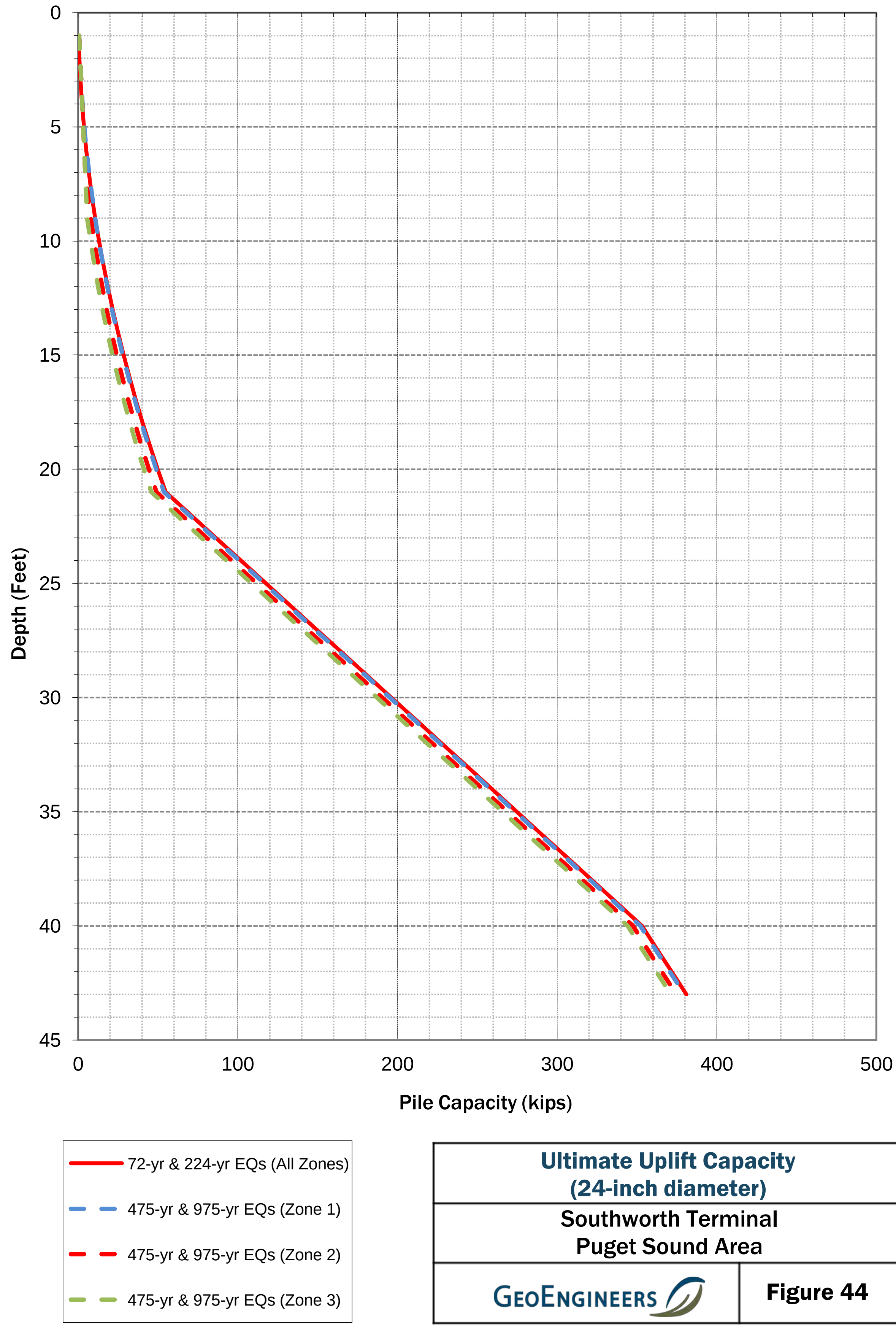
SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

Ultimate Downward Capacity (24-inch diameter)

Southworth Terminal
Puget Sound Area

GEOENGINEERS

Figure 43



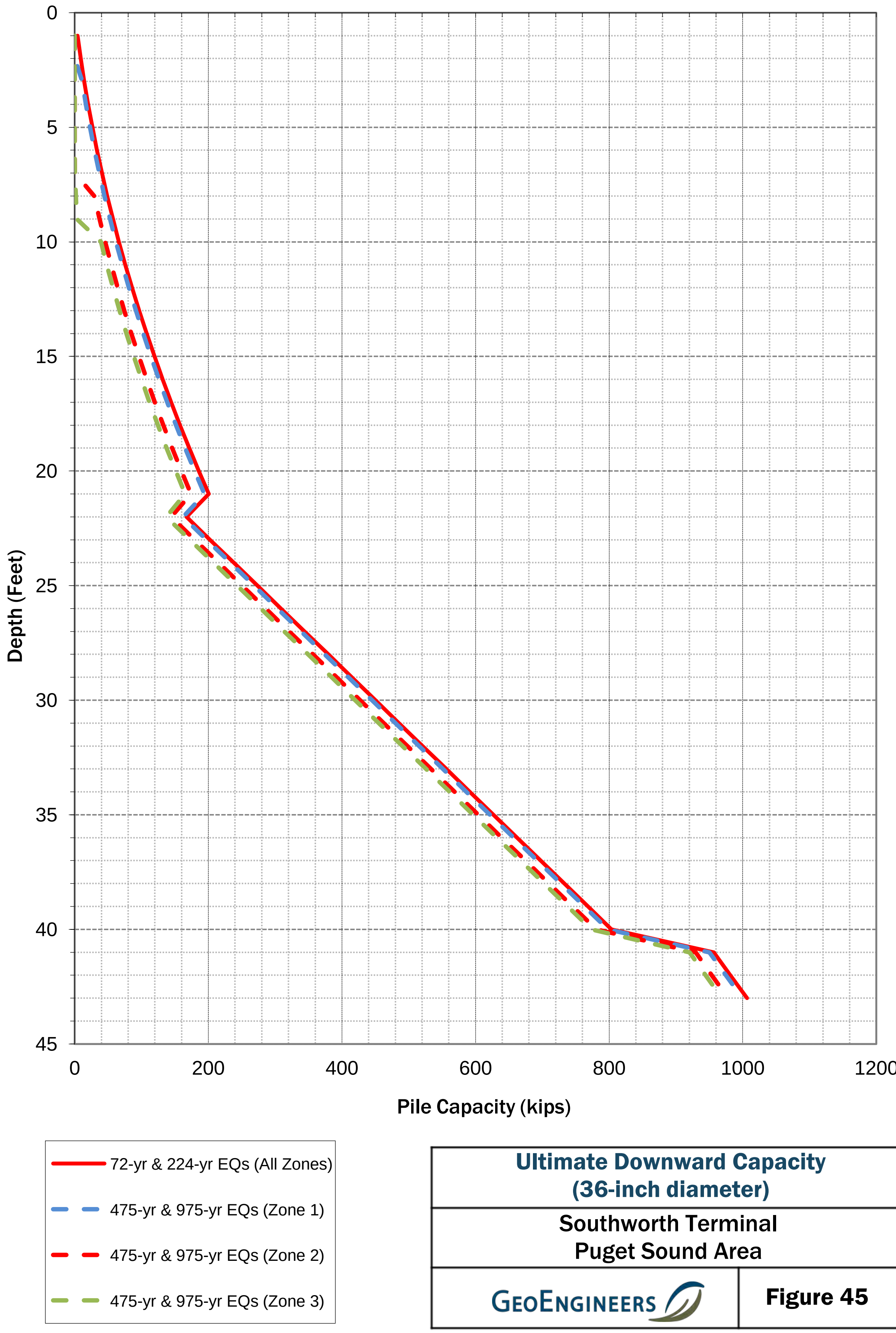
Ultimate Uplift Capacity (24-inch diameter)

Southworth Terminal
Puget Sound Area

GEOENGINEERS

Figure 44

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

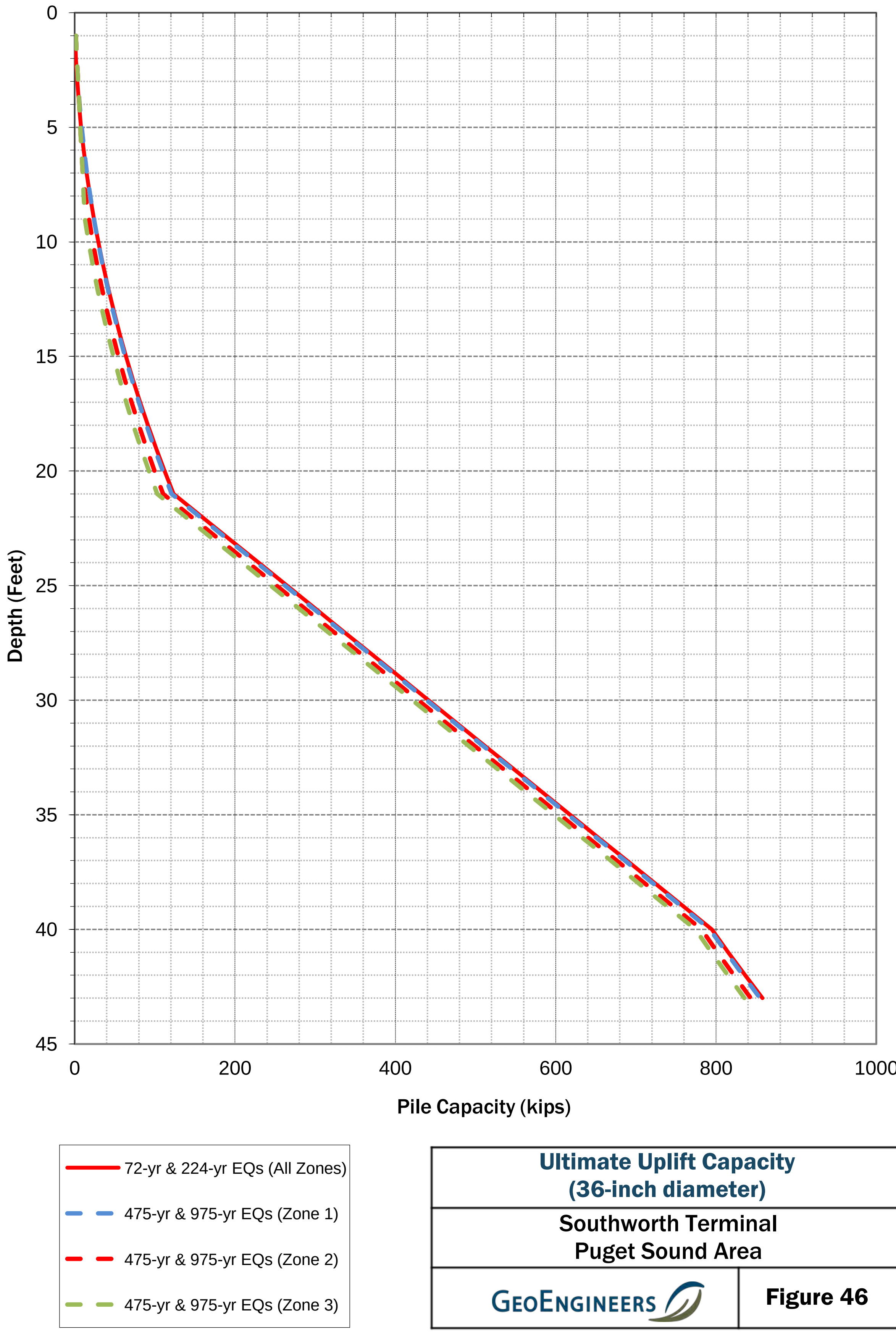


Ultimate Downward Capacity (36-inch diameter)

Southworth Terminal
Puget Sound Area

GeoEngineers

Figure 45

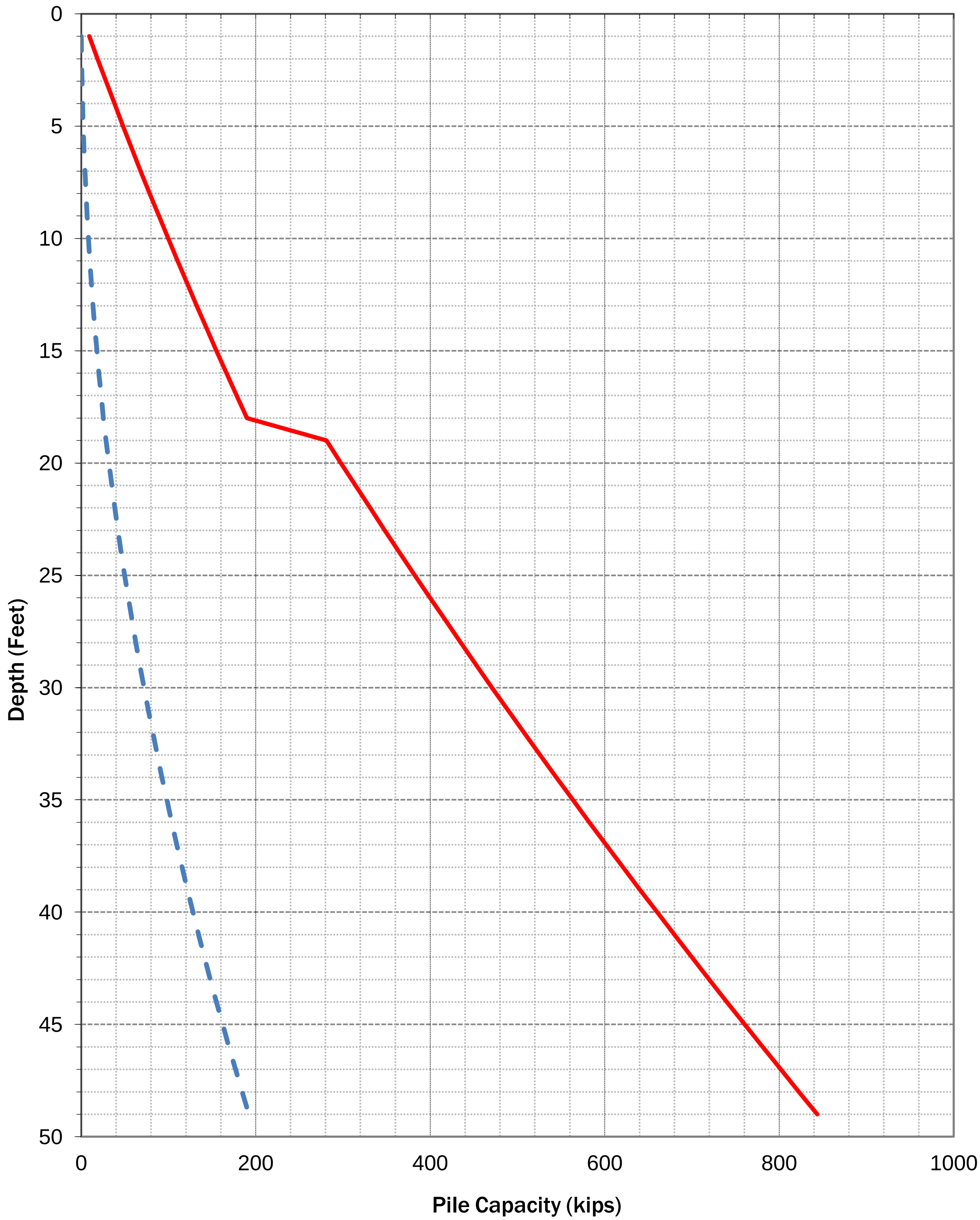


Ultimate Uplift Capacity (36-inch diameter)

Southworth Terminal Puget Sound Area

GEOENGINEERS

Figure 46



Ultimate Downward Capacity-All Design Earthquake Levels

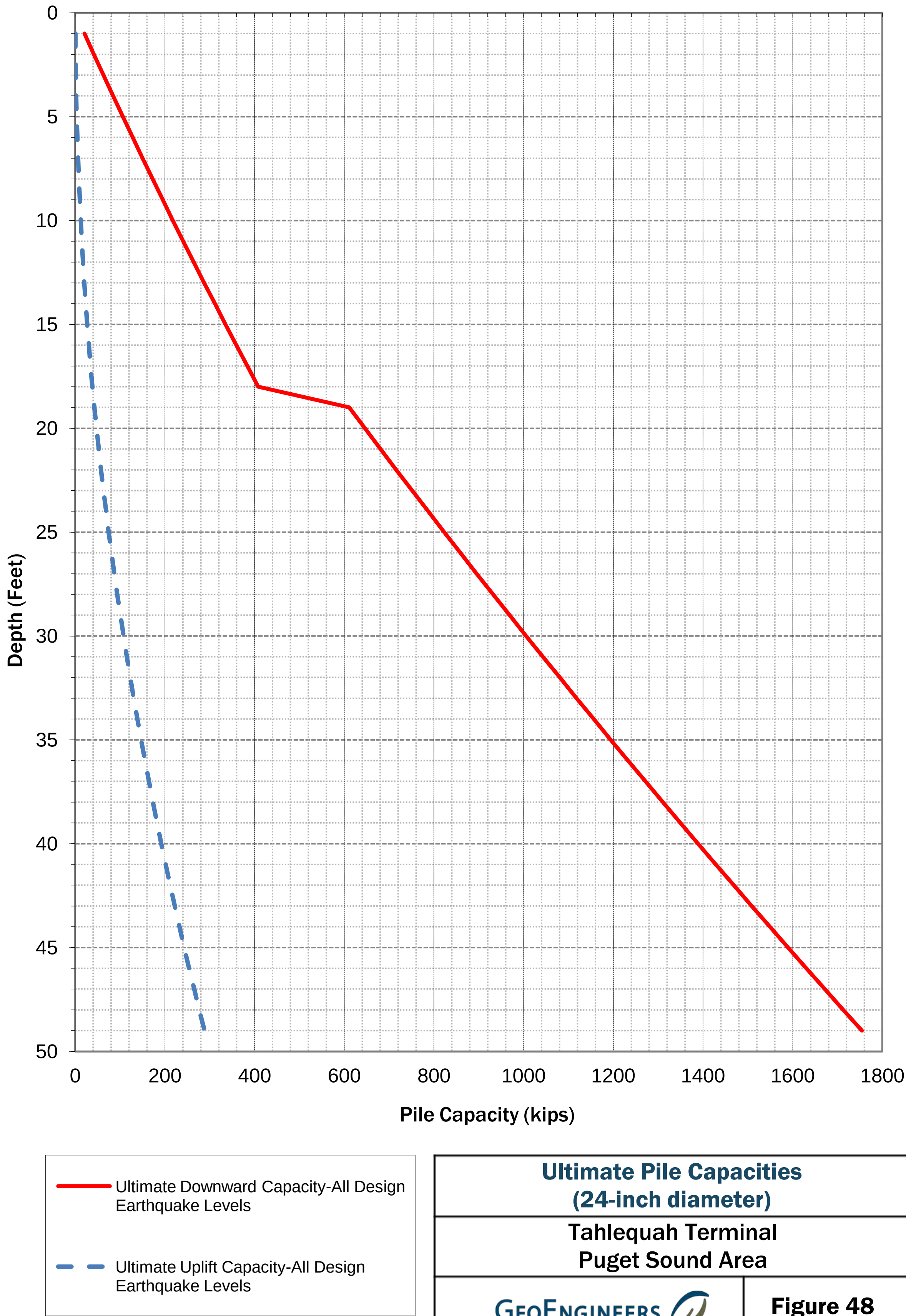
Ultimate Uplift Capacity-All Design Earthquake Levels

Ultimate Pile Capacities (16-inch diameter)

Tahlequah Terminal
Puget Sound Area

GEOENGINEERS

Figure 47



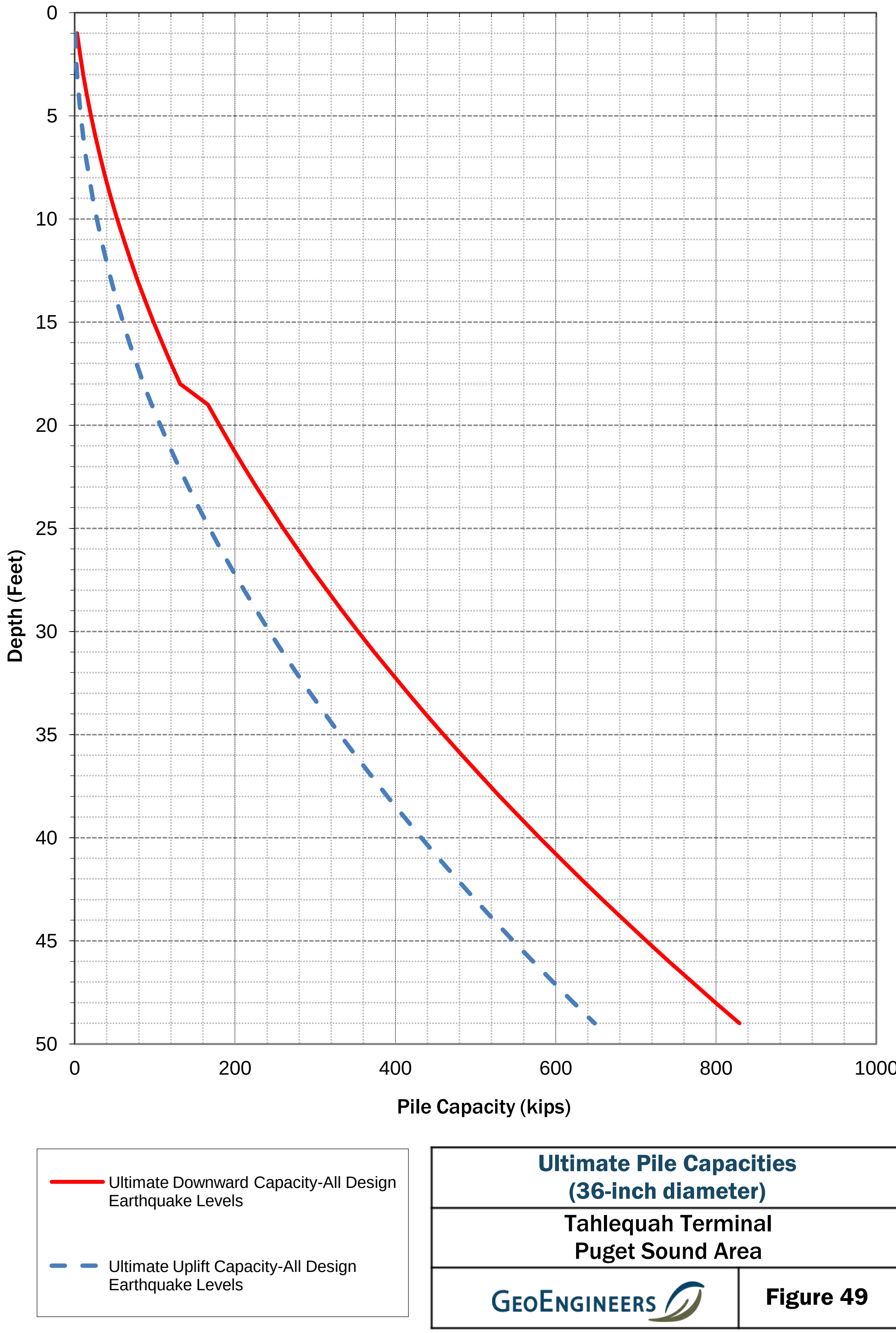
Ultimate Pile Capacities (24-inch diameter)

Tahlequah Terminal
Puget Sound Area

GeoEngineers

Figure 48

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

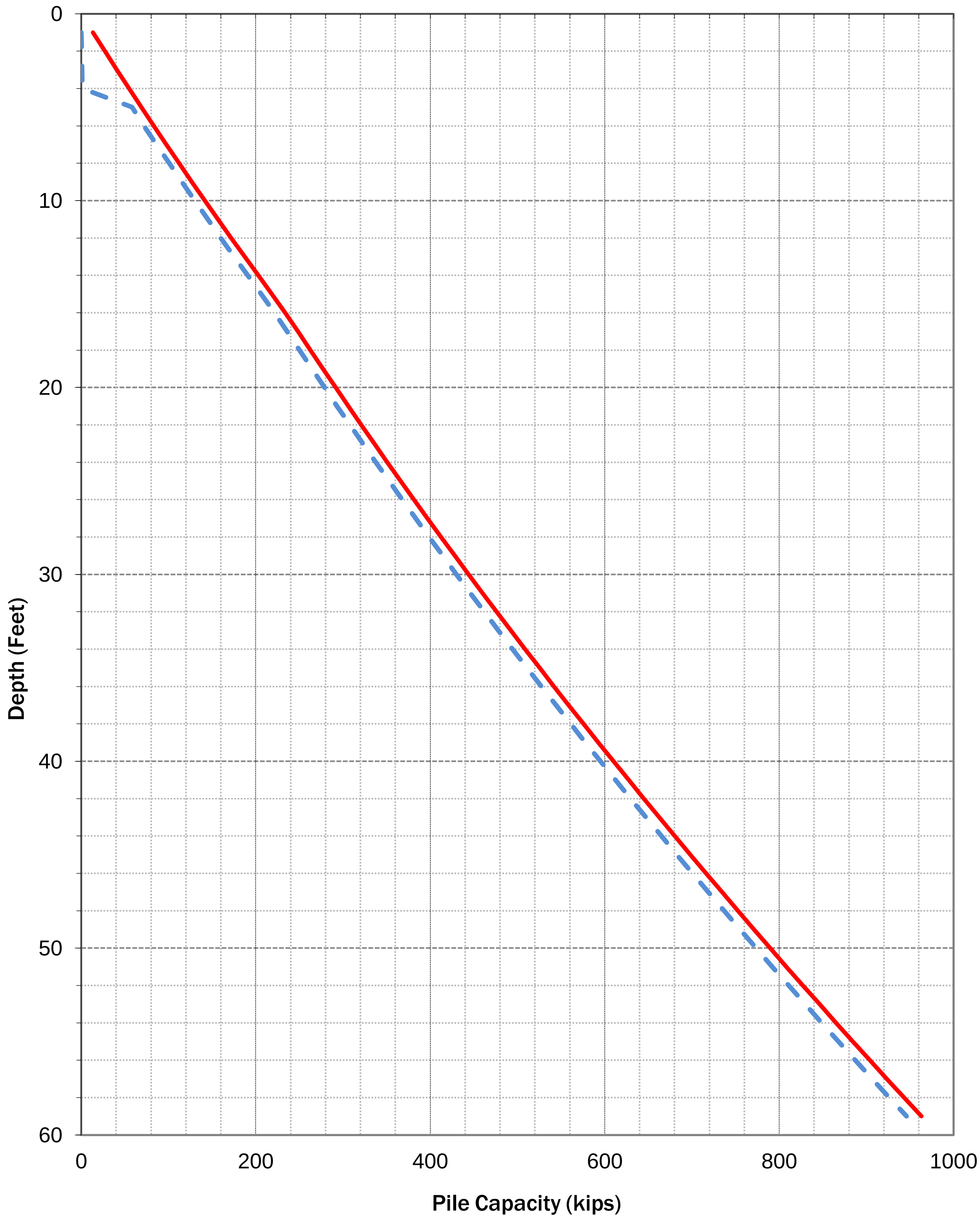


Ultimate Pile Capacities (36-inch diameter)

Tahlequah Terminal
Puget Sound Area

GeoEngineers

Figure 49



72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 1)

72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 2)

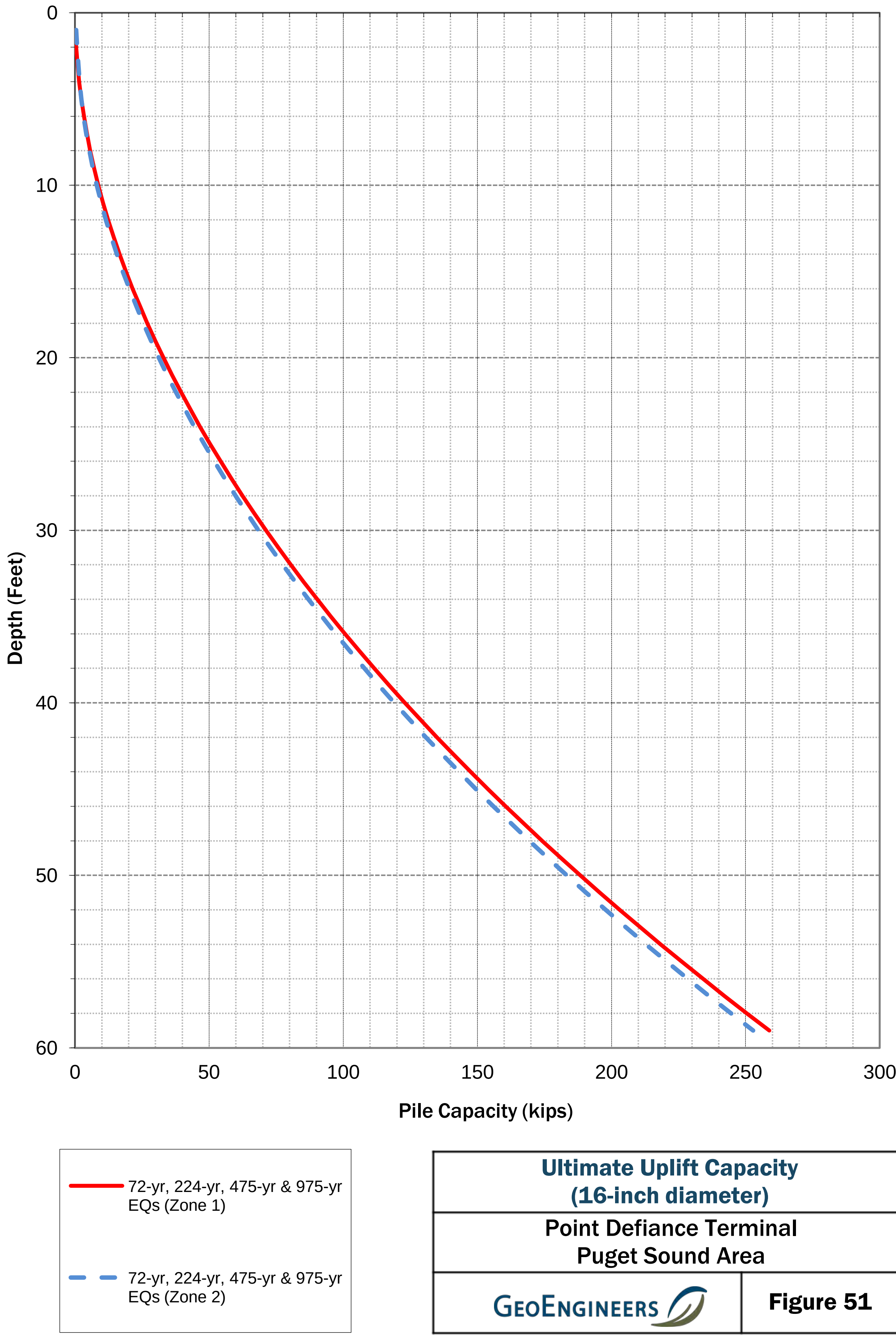
SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

Ultimate Downward Capacity (16-inch diameter)

Point Defiance Terminal
Puget Sound Area

GEOENGINEERS

Figure 50



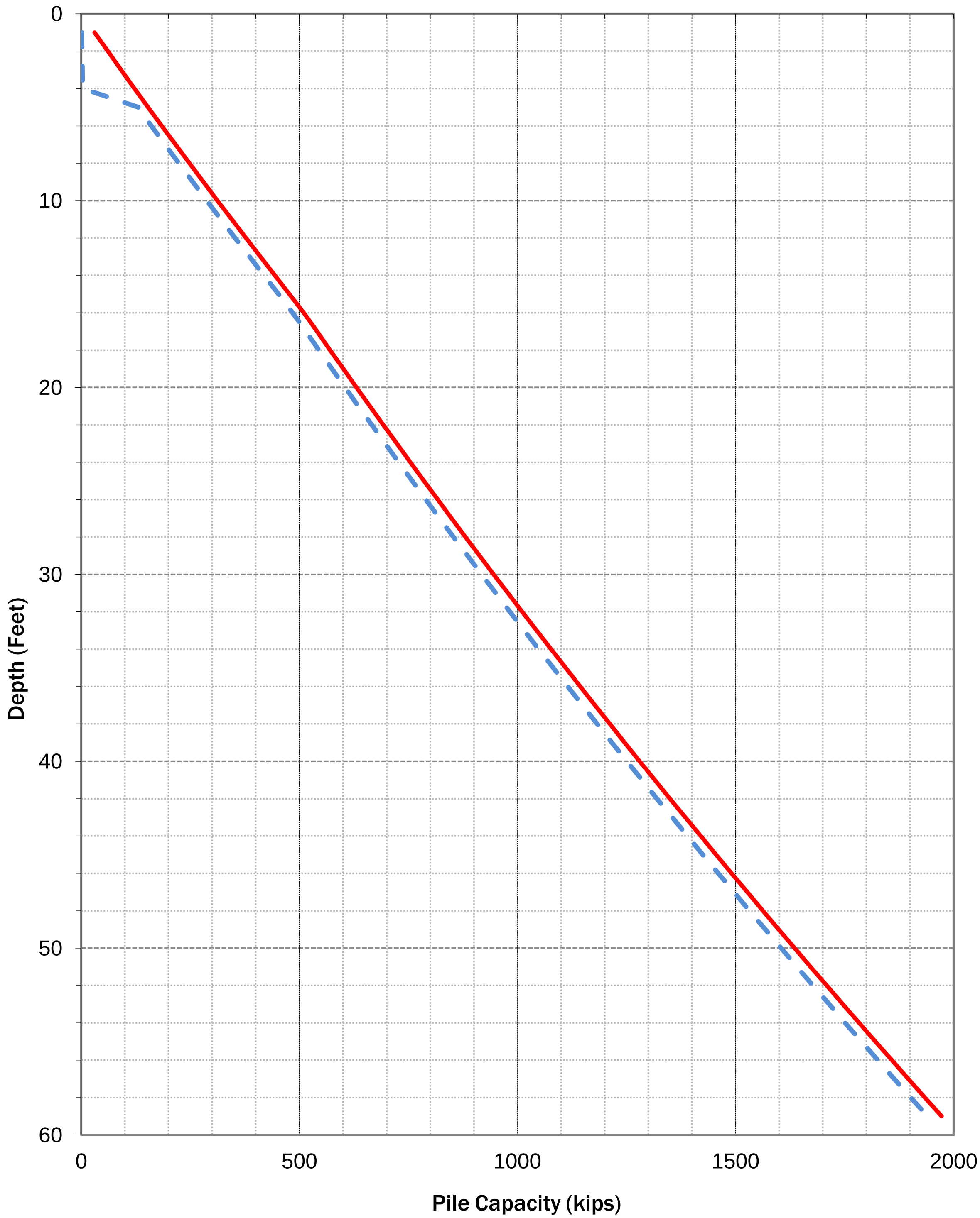
Ultimate Uplift Capacity (16-inch diameter)

Point Defiance Terminal Puget Sound Area

GEOENGINEERS

Figure 51

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 1)

72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 2)

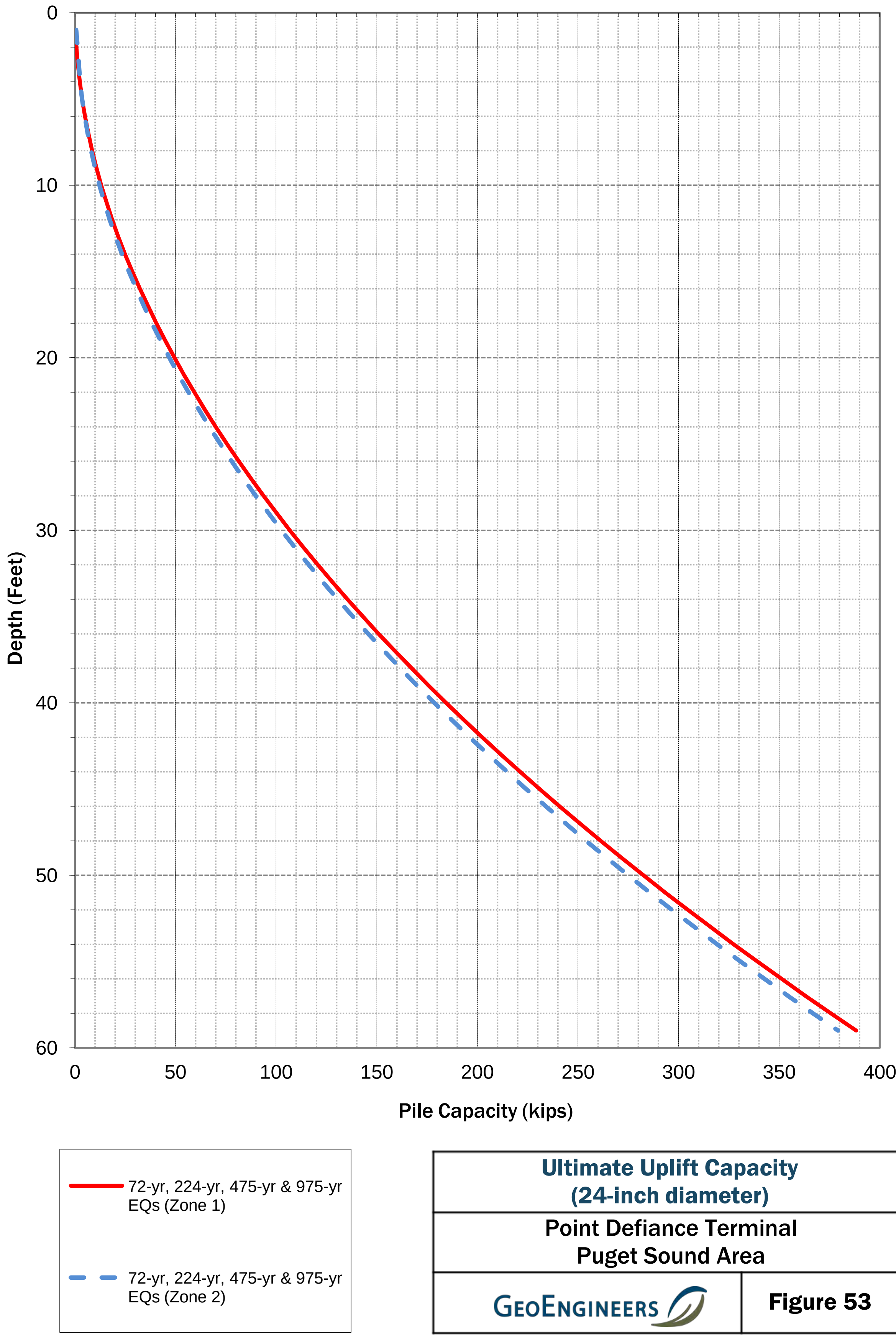
Ultimate Downward Capacity (24-inch diameter)

Point Defiance Terminal Puget Sound Area

GEOENGINEERS

Figure 52

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



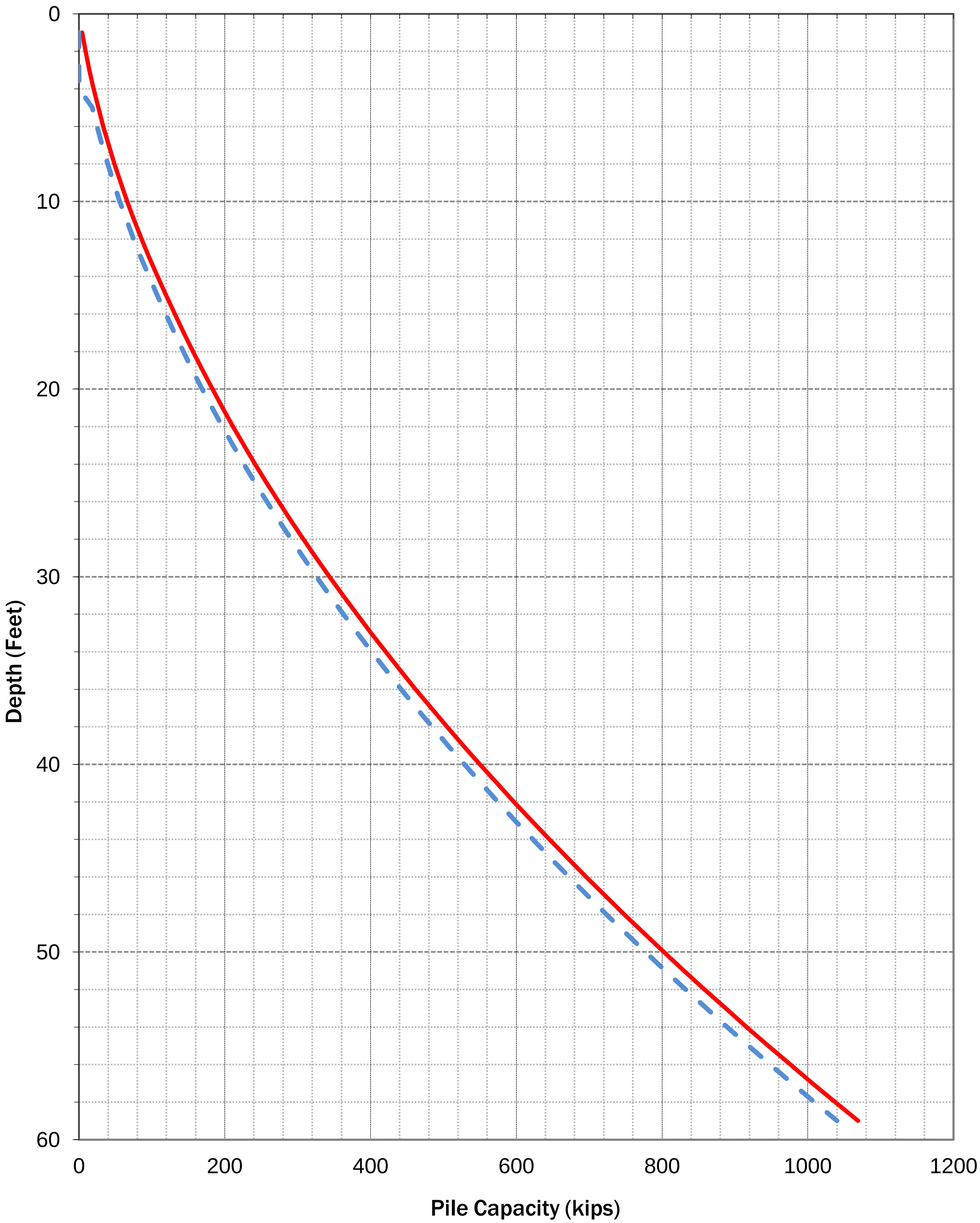
SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

Ultimate Uplift Capacity (24-inch diameter)

Point Defiance Terminal
Puget Sound Area

GEOENGINEERS

Figure 53



72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 1)

72-yr, 224-yr, 475-yr & 975-yr EQs (Zone 2)

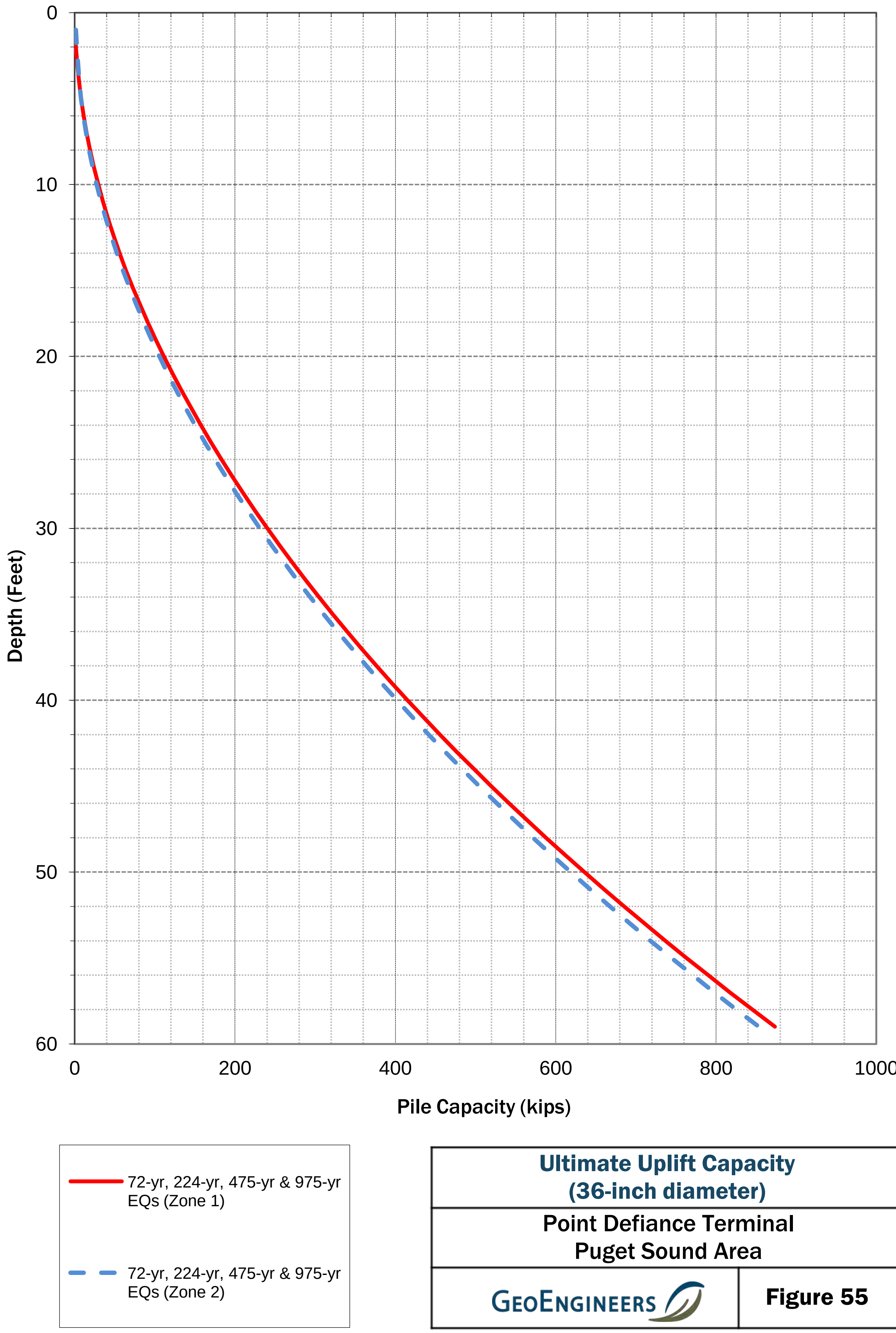
Ultimate Downward Capacity (36-inch diameter)

Point Defiance Terminal
Puget Sound Area

GEOENGINEERS

Figure 54

SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11



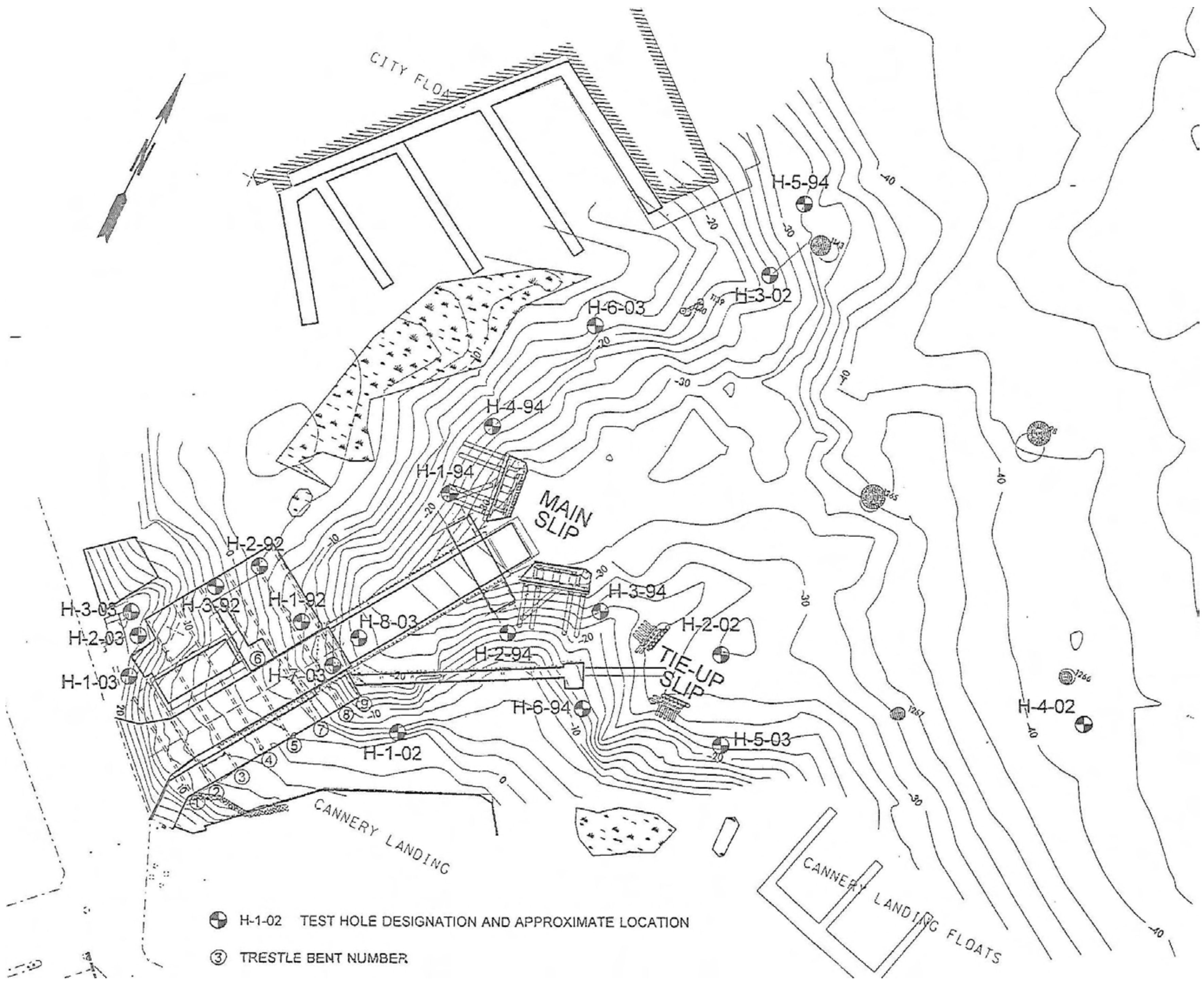
SharePoint: 0180-284-00\Technical Analysis\Summary of Pile Capacity Evaluation.xls hpd 11/1/11

Ultimate Uplift Capacity (36-inch diameter)

Point Defiance Terminal Puget Sound Area

GEOENGINEERS

Figure 55



Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	Uniaxial Compressive Strength (psi)	LPILE Soil Model
Overburden soil (gravelly sand or marine deposits, non-liquefiable)	Varies	0.035	60	36	N/A	N/A	Reese et al (1974)
Marine Meta Sedimentary Rock	Below overburden soils	0.050	N/A	N/A	N/A	40,000	Strong Rock (Vuggy Limestone)

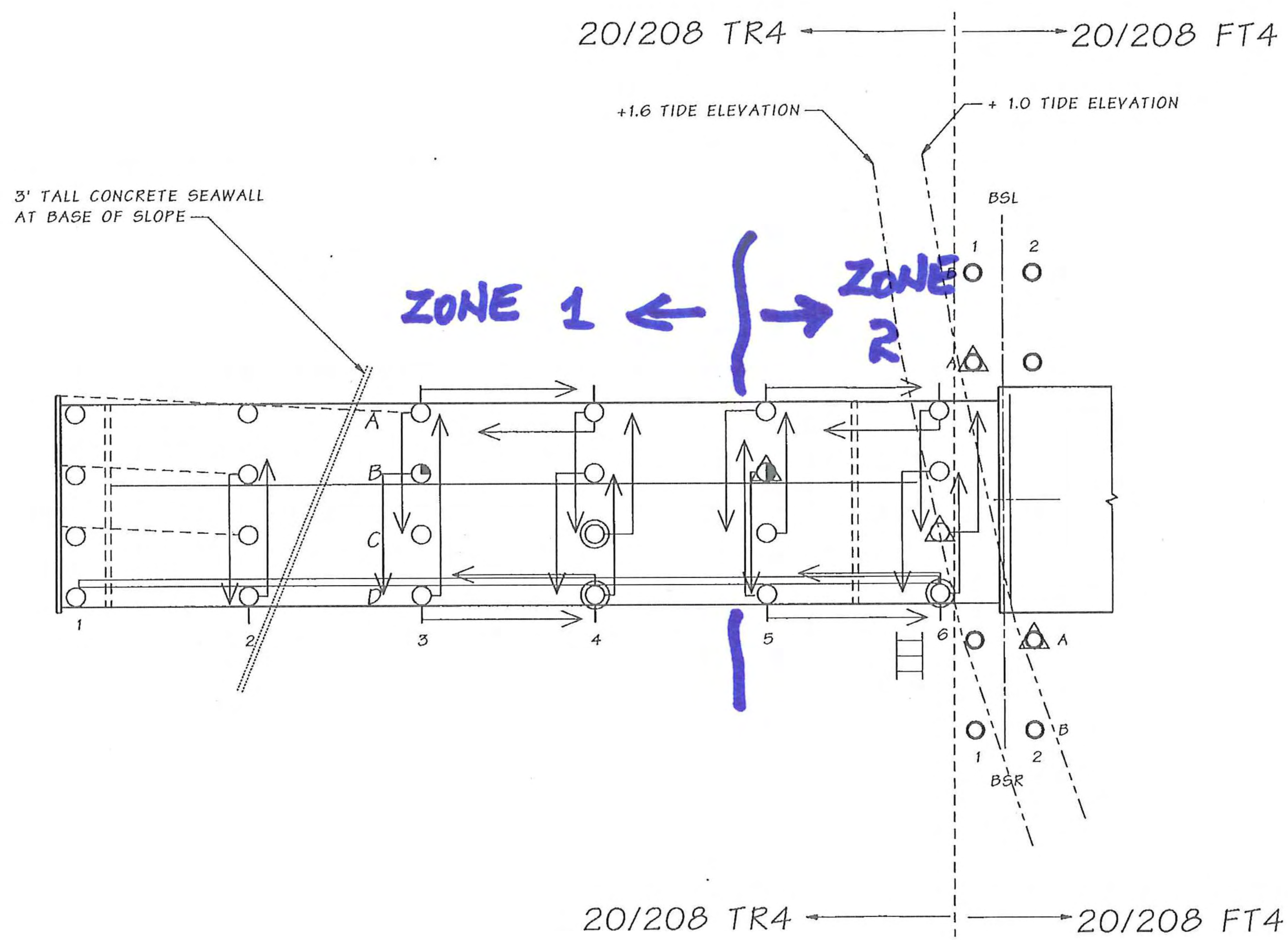
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Friday Harbor Terminal
Puget Sound Area

GeoEngineers

Figure 56

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	Uniaxial Compressive Strength (psi)	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)	0 – 5	0.035	30	36	0.1	N/A	Reese et al (1974)
Moderately Strong Conglomerate	5 – 7	0.050	N/A	N/A	N/A	20,000	Strong Rock (Vuggy Limestone)
Strong Conglomerate	7–36	0.050	N/A	N/A	N/A	40,000	Strong Rock (Vuggy Limestone)

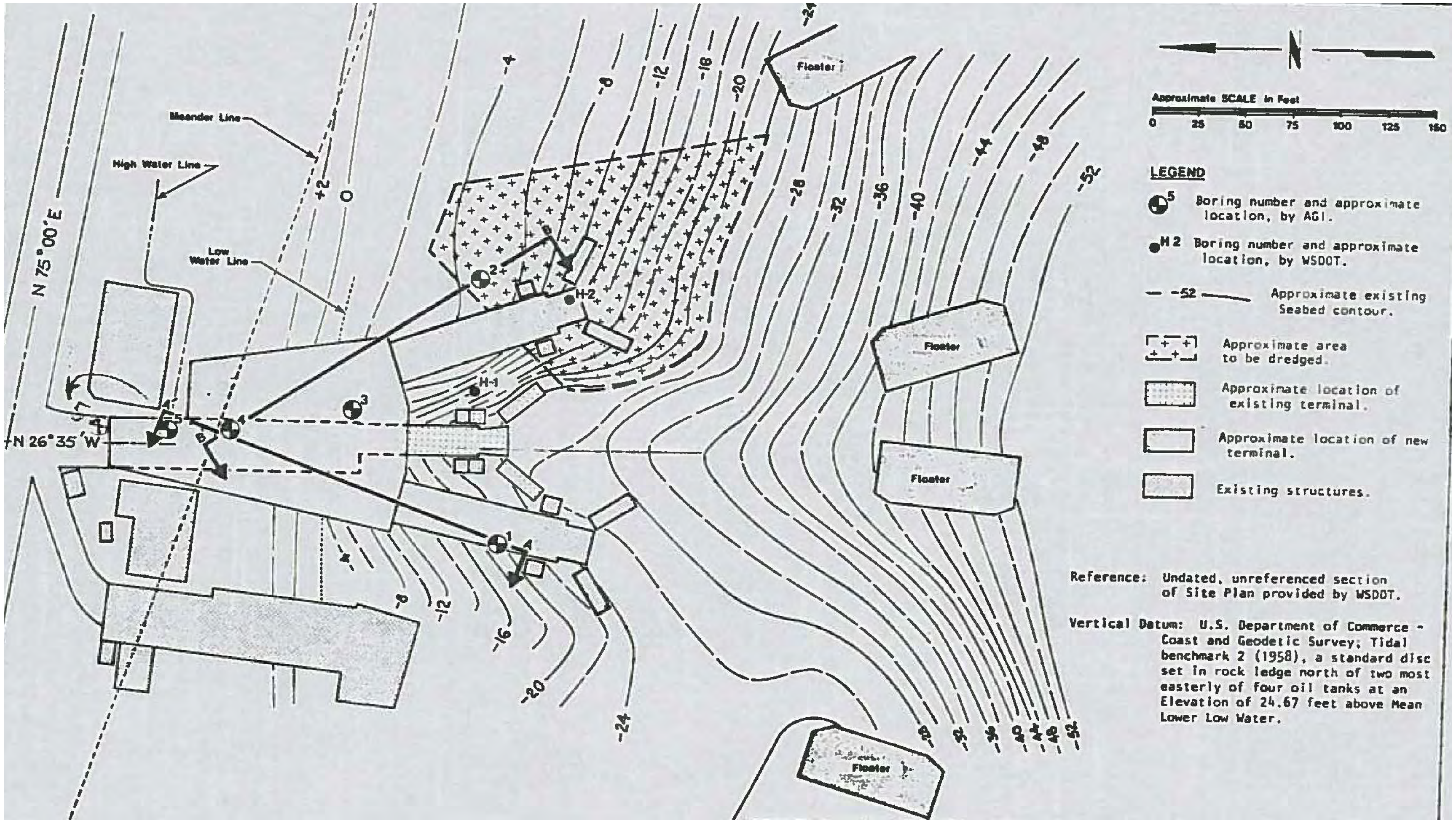
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Lopez Island Terminal
Puget Sound Area

GeoEngineers

Figure 57

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Very Dense Sand (Glacially consolidated)	0–20	0.041	100	40	N/A	N/A	Reese et al (1974)

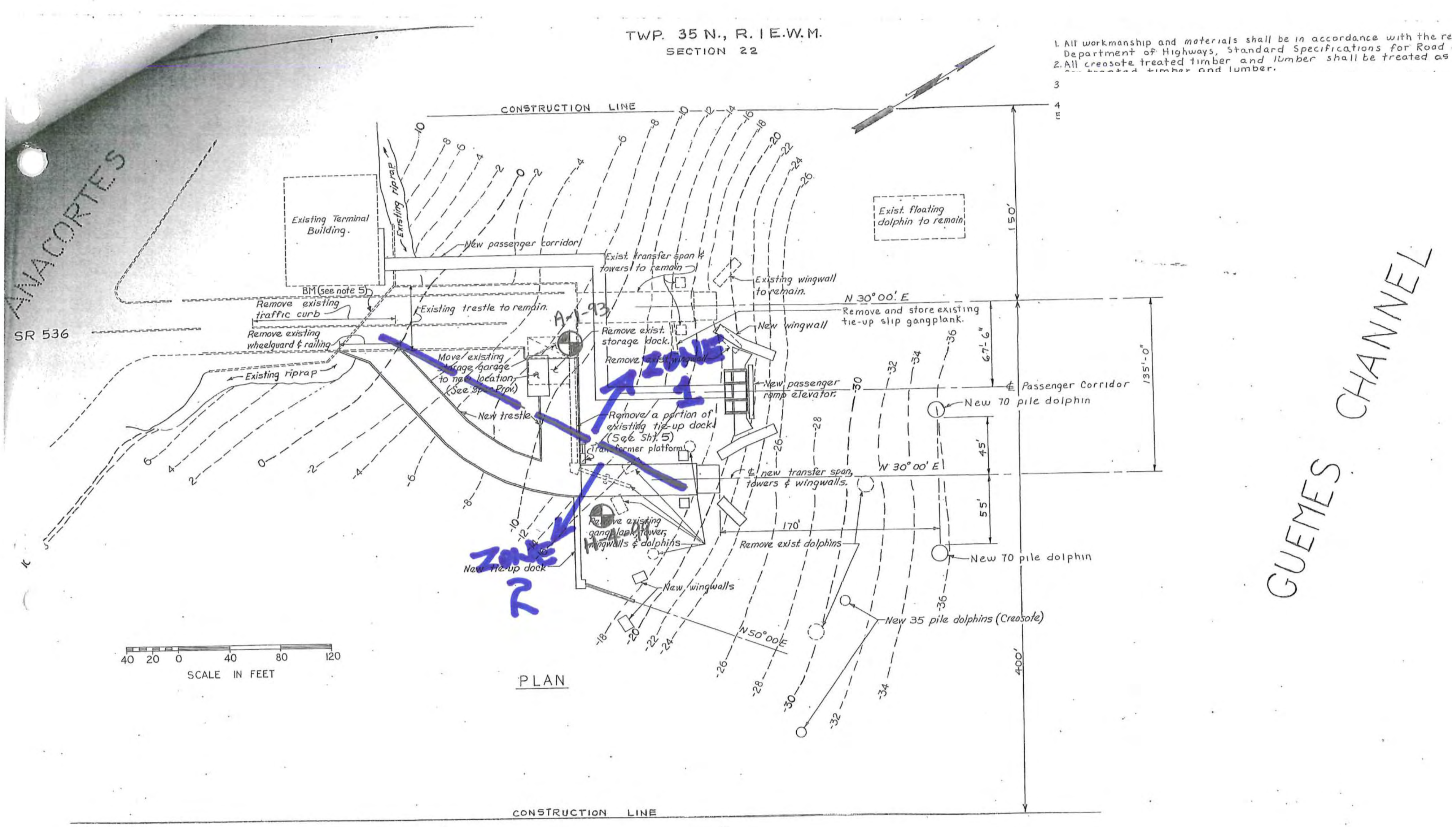
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Orcas Island Terminal
Puget Sound Area

GeoEngineers

Figure 58

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



	Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Zone 1	Unconsolidated Soils (Potentially liquefiable Soils)	0–5	0.0353	30	36	0.1	N/A	Reese et al (1974)
	Very Dense Sand (Glacially consolidated)	5 – 100	0.041	100	40	N/A	N/A	Reese et al (1974)

	Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Zone 2	Unconsolidated Soils (Potentially liquefiable Soils)	0–11	0.0353	30	36	0.1	N/A	Reese et al (1974)
	Very Dense Sand Glacially consolidated)	11–100	0.041	100	40	N/A	N/A	Reese et al (1974)

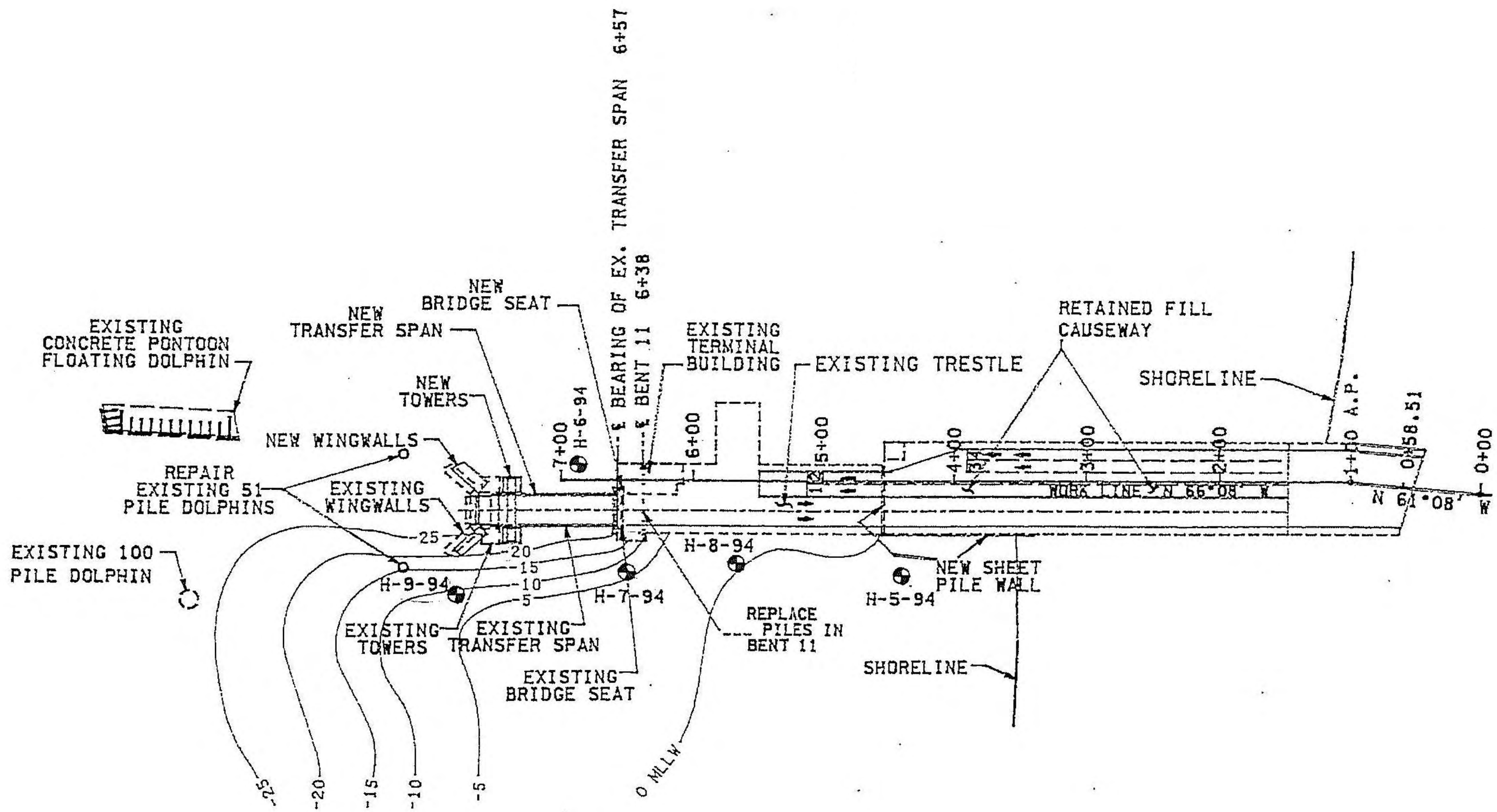
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Anacortes Terminal
Puget Sound Area

GeoEngineers

Figure 59

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)[1]	0–11	0.0353	30	36	0.1	N/A	Reese et al (1974)
Loose to Dense Sand	11–24	0.041	100	38	N/A	N/A	Reese et al (1974)
Very Dense Sand (Glacially consolidated)	24–40	0.041	100	40	N/A	N/A	Reese et al (1974)
Hard Peat	40–49	0.0353	100	N/A	N/A	0.01	Stiff Clay w/free water
Very Dense Sand	49–100	0.041	100	40	N/A	N/A	Reese et al (1974)

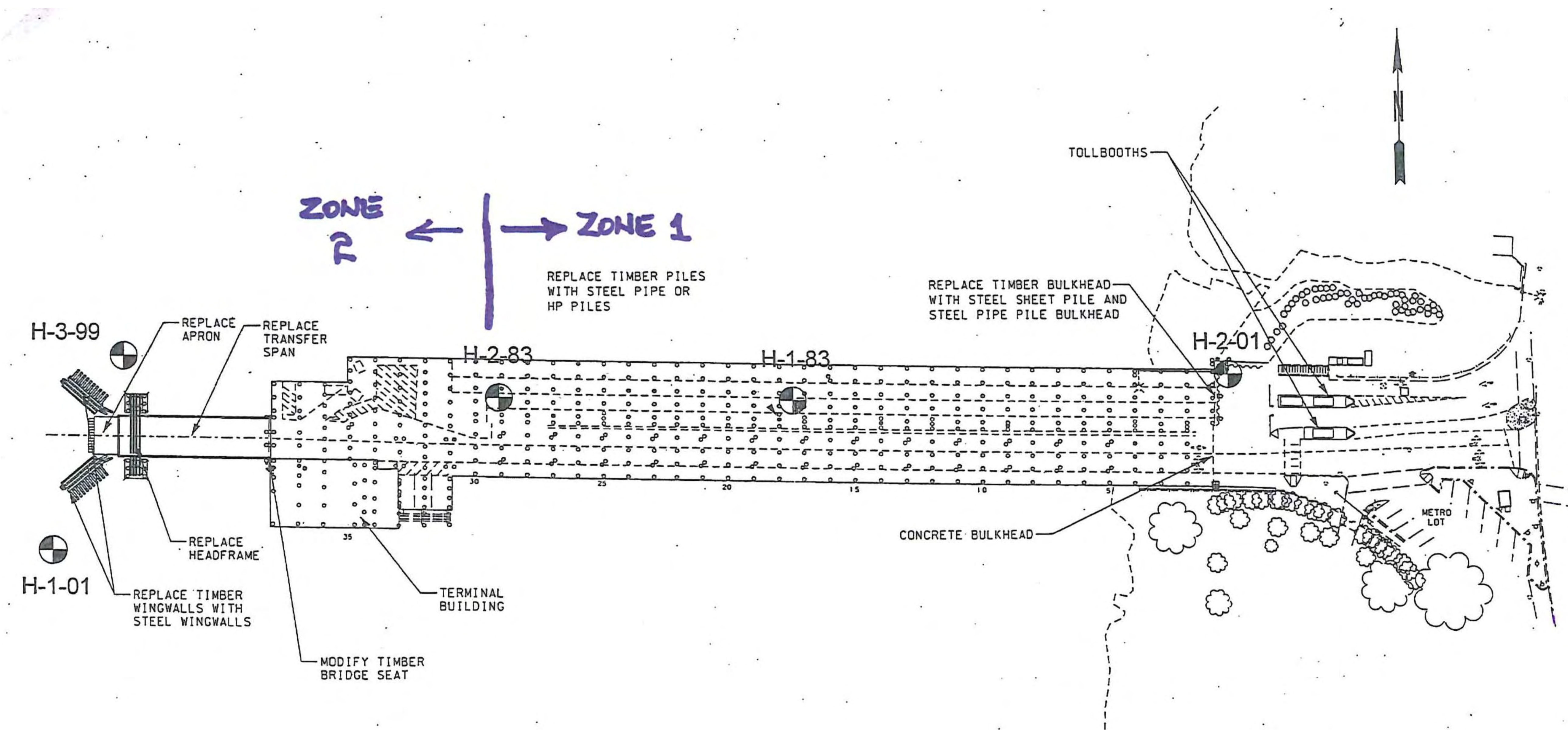
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Edmonds Terminal
Puget Sound Area

GeoEngineers

Figure 60

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Zone 1

Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)	0–4	0.0353	30	36	0.1	N/A	Reese et al (1974)
Dense Sand	4 - 42	0.041	100	38	N/A	N/A	Reese et al (1974)
Glacial Consolidated Clay	42–100	0.0353	100	38	N/A	0.005	Stiff Clay without free water using k

Zone 2

Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)[1]	0–22	0.0353	30	36	0.1	N/A	Reese et al (1974)
Dense Sand	22 - 42	0.041	100	38	N/A	N/A	Reese et al (1974)
Glacial Consolidated Clay	42–100	0.0353	100	38	N/A	0.005	Stiff Clay without free water using k

Note: For non-liquefiable soils use a p-multiplier of 1.0

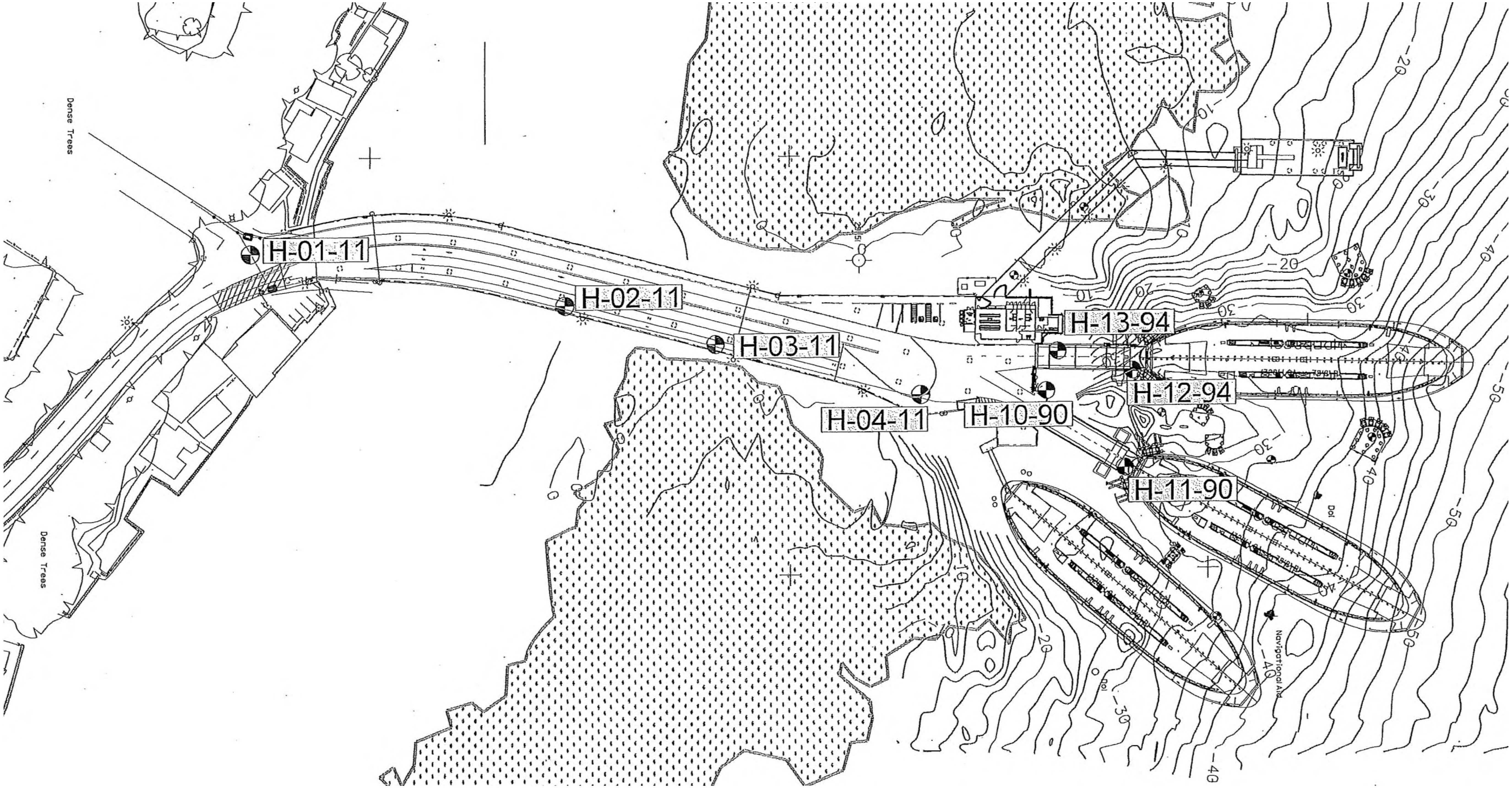
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Fauntleroy Terminal
Puget Sound Area

GEOENGINEERS

Figure 61

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	LPILE Soil Model
Unconsolidated Soils	0–5	0.0353	30	32	0.1	Reese et al (1974)
Glacial Consolidated Soils	5–100	0.0353	100	38	N/A	Reese et al (1974)

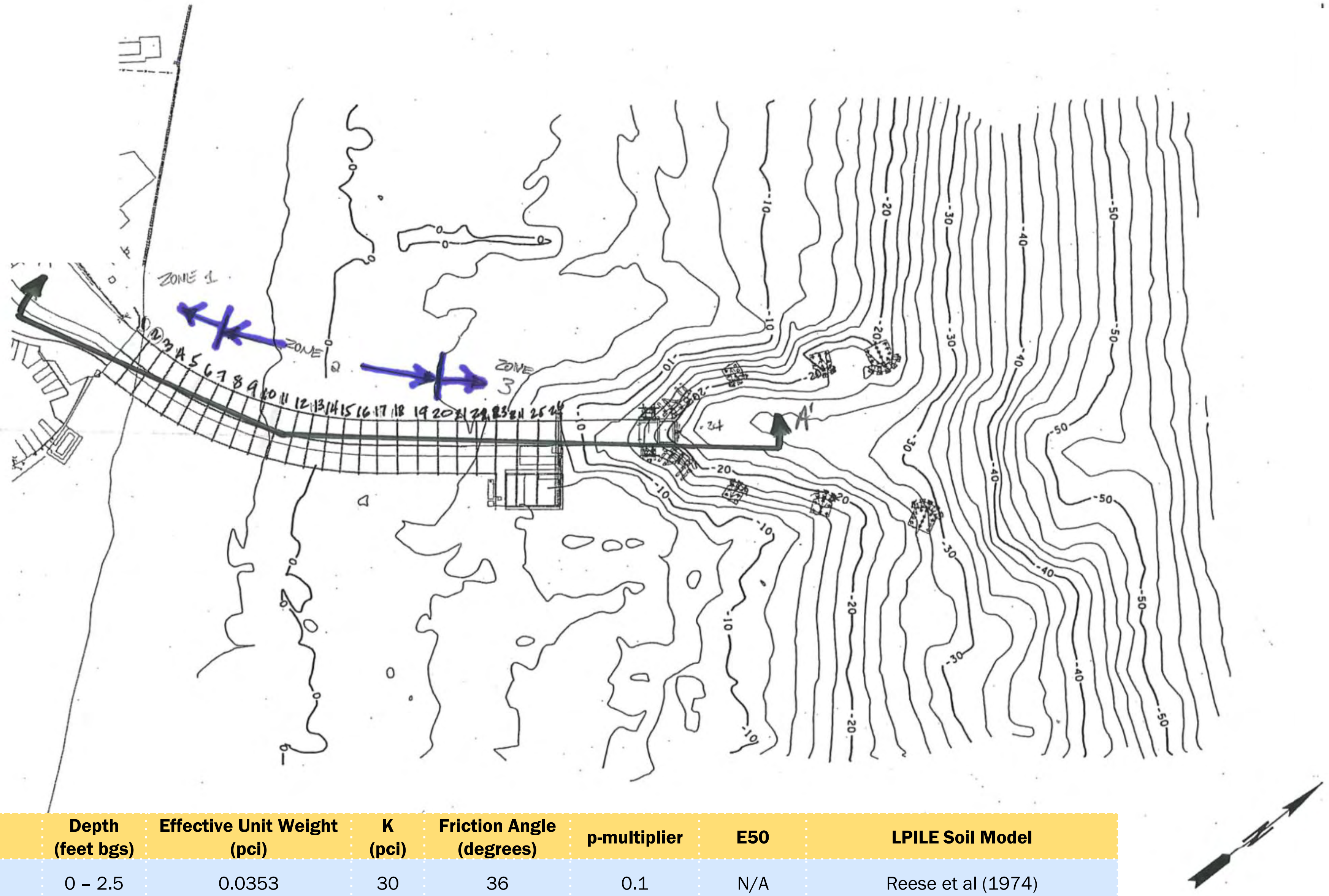
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Vashon Island Terminal
Puget Sound Area

GeoEngineers

Figure 62

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Zone 1

Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)	0 – 2.5	0.0353	30	36	0.1	N/A	Reese et al (1974)
Medium Dense to Very Dense Sand (Glacially consolidated)	2.5 – 22	0.041	100	40	N/A	N/A	Reese et al (1974)
Glacial Consolidated Clay	22 – 41	0.0353	100	38	N/A	0.005	Stiff Clay without free water using k
Glacial Consolidated Silt	41–100	0.041	100	40	N/A	0.005	Silt (Cemented c-phi Soil)

Zone 2

Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)	0 – 5	0.0353	30	36	0.1	N/A	Reese et al (1974)
Medium Dense to Very Dense Sand (Glacially consolidated)	5 – 22	0.041	100	40	N/A	N/A	Reese et al (1974)
Glacial Consolidated Clay	22 – 41	0.0353	100	38	N/A	0.005	Stiff Clay without free water using k
Glacial Consolidated Silt	41–100	0.041	100	40	N/A	0.005	Silt (Cemented c-phi Soil)

Zone 3

Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)	0 – 10	0.0353	30	36	0.1	N/A	Reese et al (1974)
Medium Dense to Very Dense Sand (Glacially consolidated)	10 – 22	0.041	100	40	N/A	N/A	Reese et al (1974)
Glacial Consolidated Clay	22 – 41	0.0353	100	38	N/A	0.005	Stiff Clay without free water using k
Glacial Consolidated Silt	41–100	0.041	100	40	N/A	0.005	Silt (Cemented c-phi Soil)

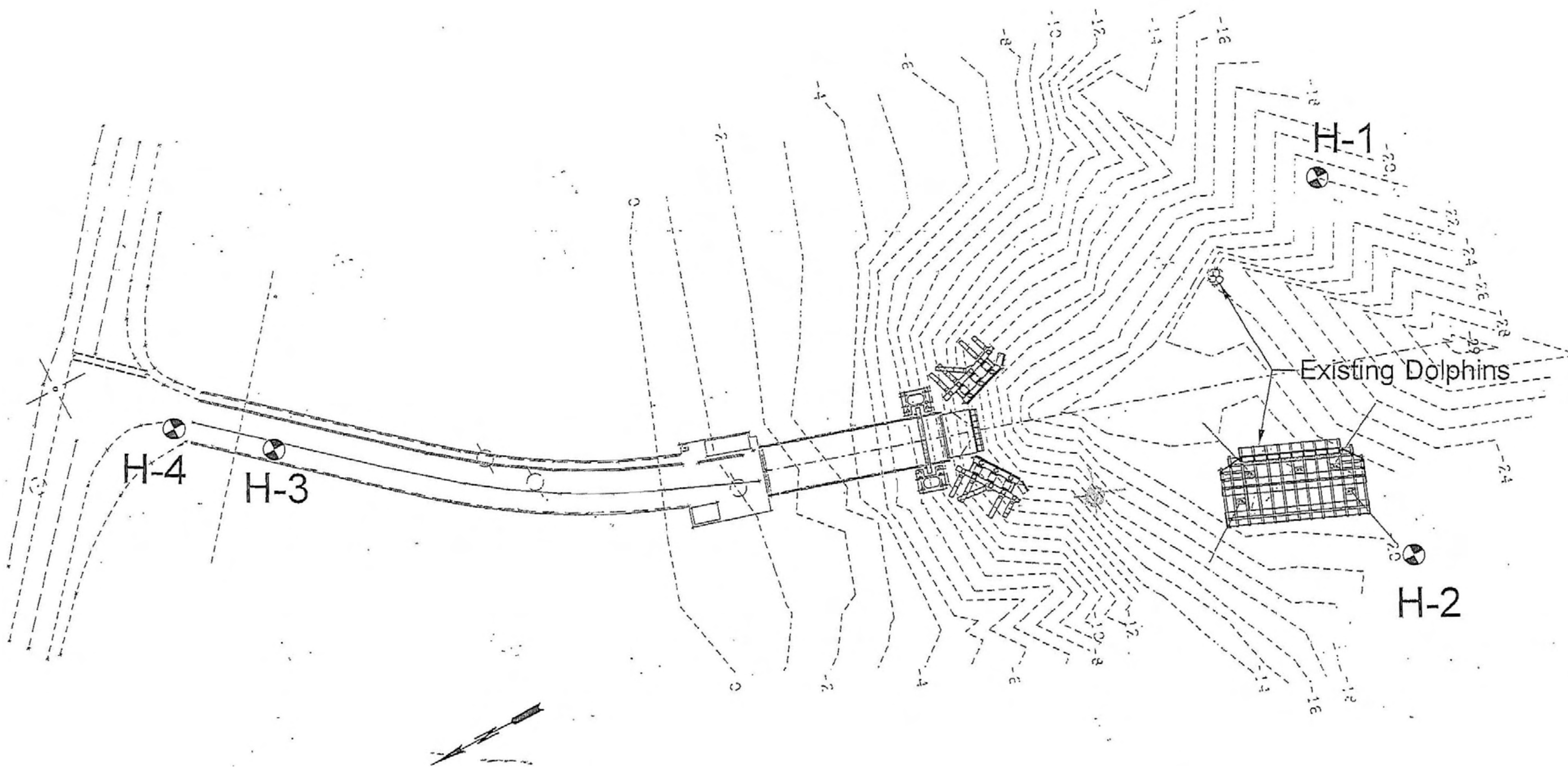
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Southworth Terminal
Puget Sound Area

GeoEngineers

Figure 63

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Medium Dense to Dense Silty Sand	0 – 19	0.041	100	38	N/A	N/A	Reese et al (1974)
Very Dense Gravel with Silt and Sand	19 – 45	0.041	100	40	N/A	N/A	Reese et al (1974)
Very Dense Sand (Glacially consolidated)	45–100	0.041	100	40	N/A	N/A	Reese et al (1974)

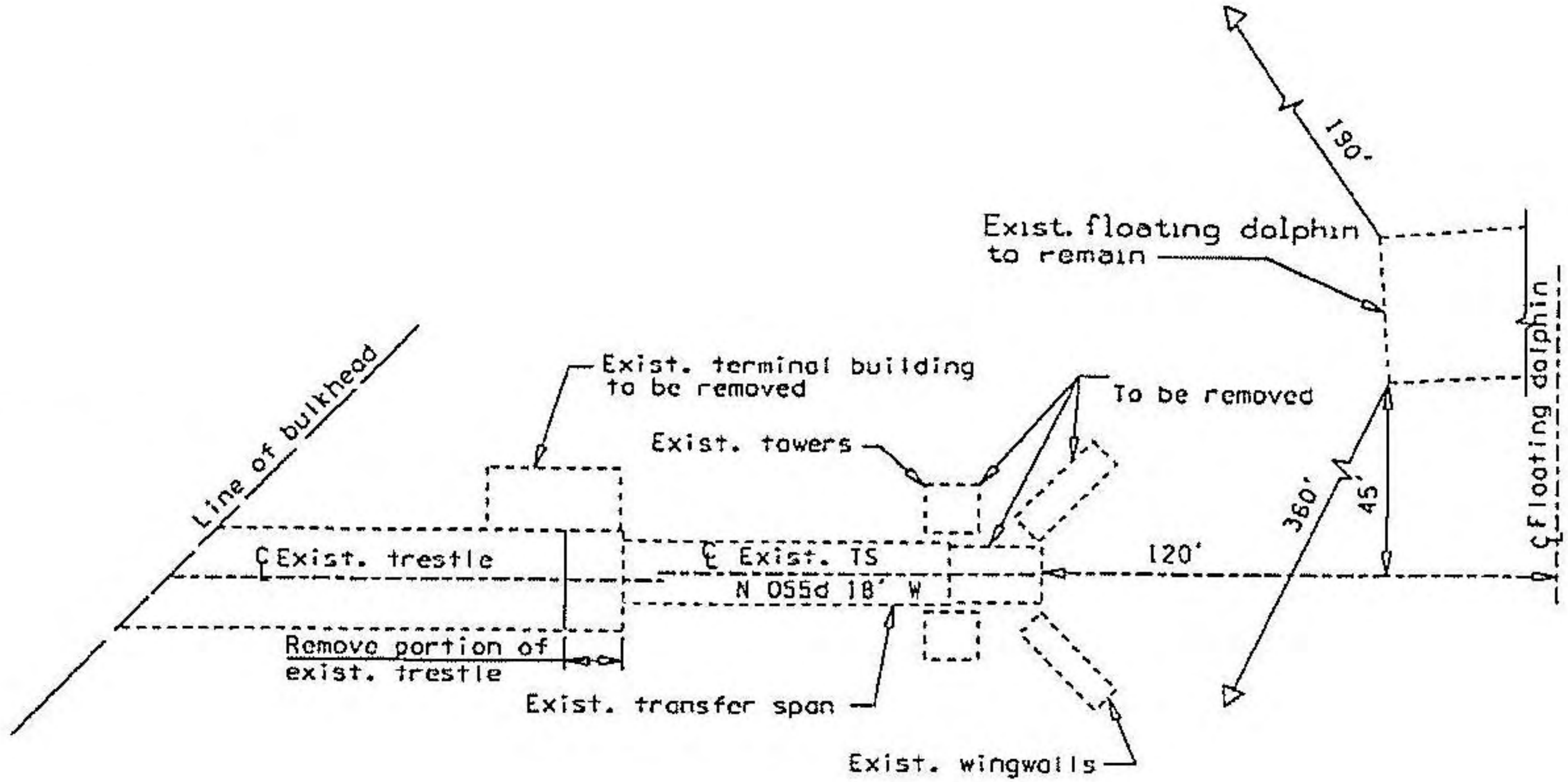
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Tahlequah Terminal
Puget Sound Area

GeoEngineers

Figure 64

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



PLAN-EXISTING LAYOUT

Soil Layer	Depth (feet bgs)	Effective Unit Weight (pci)	K (pci)	Friction Angle (degrees)	p-multiplier	E50	LPILE Soil Model
Unconsolidated Soils (Potentially liquefiable Soils)	0–5	0.0353	30	36	0.1	N/A	Reese et al (1974)
Very Dense Sandy Gravel	5 – 6	0.041	100	40	N/A	N/A	Reese et al (1974)
Very Dense Sand (Glacially consolidated)	6 – 16	0.041	100	40	N/A	N/A	Reese et al (1974)
Glacial Consolidated Silt	16–100	0.0353	100	40	N/A	0.005	Silt (Cemented c-phi Soil)

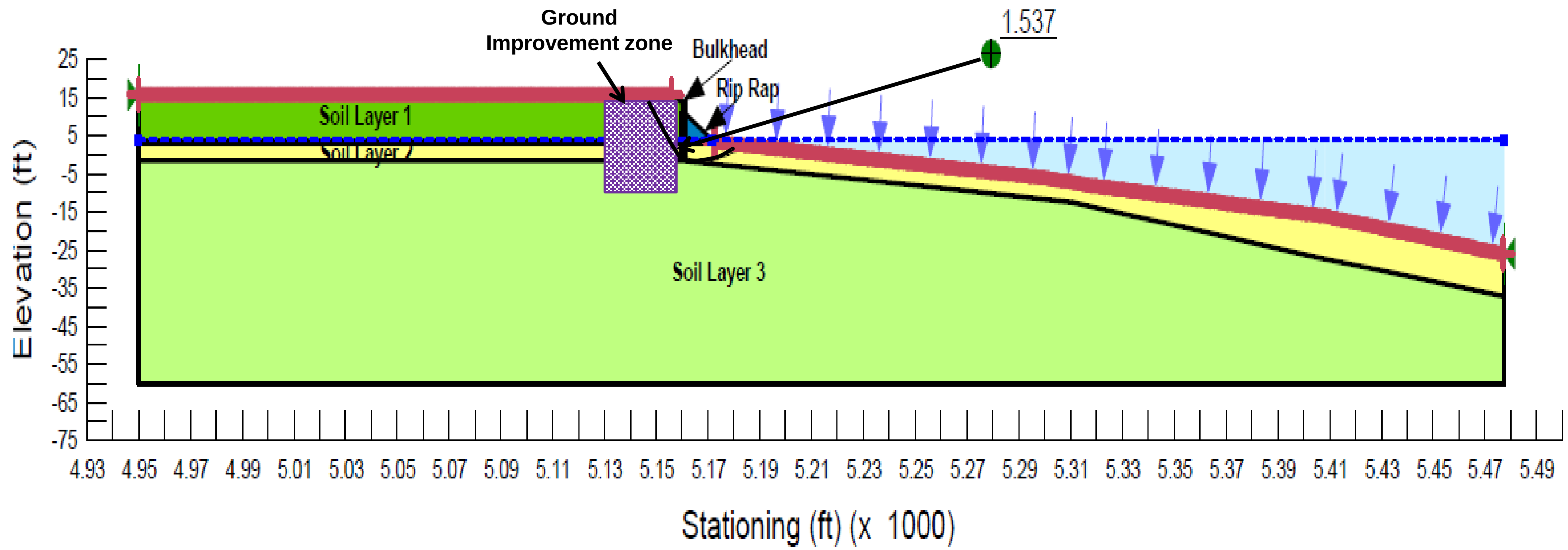
Soil Parameters for L-Pile Analysis

Washington State Ferries Timber Trestles Project
Point Defiance Terminal
Puget Sound Area

GeoEngineers

Figure 65

SP/0180-284-00/Finals/Figures 62 - 71.LPILE Parameters.ppt NLU 3/23/12



Loading Condition	FS
Static	1.537
*Seismic (0.1g - Small earthquake)	1.246
*Seismic (0.2g – Moderate earthquake)	1.021
*Seismic (0.3g – Large earthquake)	0.859
**Post-Earthquake (No-mitigation)	0.541
**Post-Earthquake (Mitigation)	1.045

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

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 the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot 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.

Reference: From SLOPE/W.

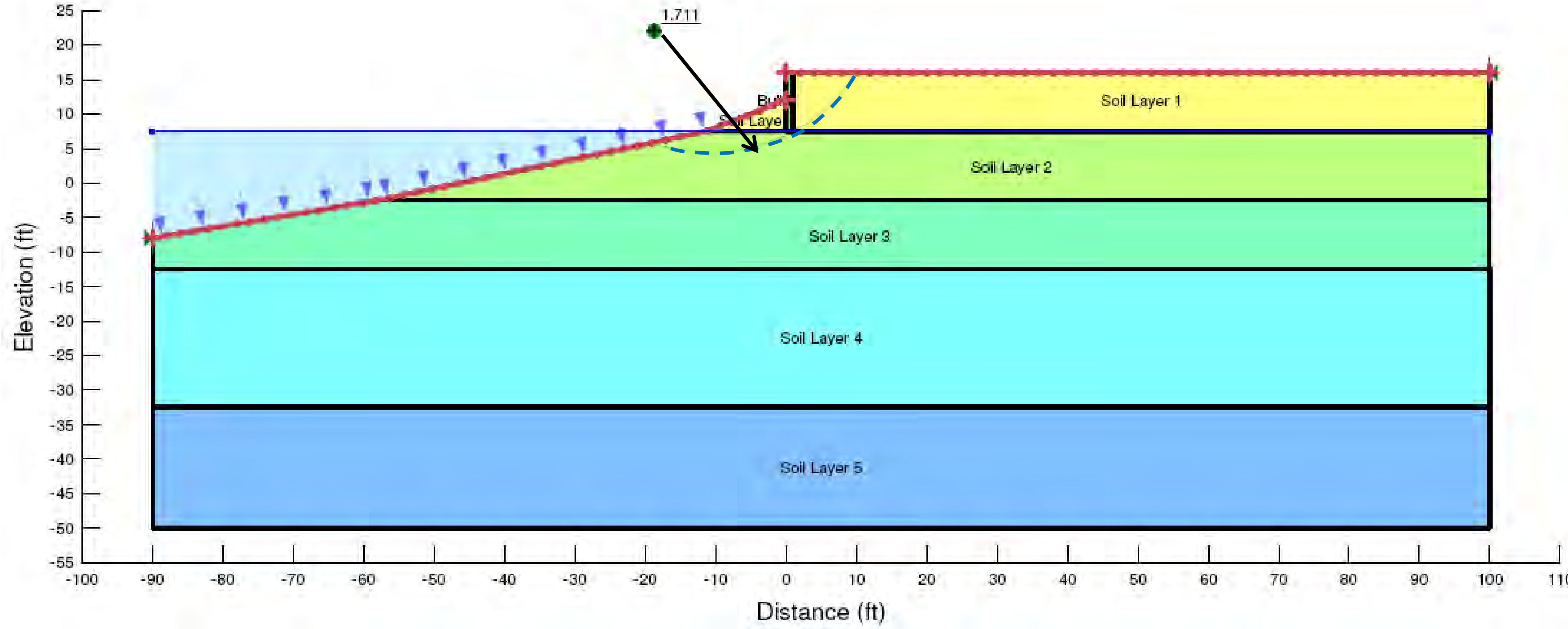
Anacortes Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GeoEngineers

Figure 66

: SP:\0\0180284\00\finals\Figures 72 – 78 Bulkhead Stability.ppt HPD 03/22/12



Loading Condition	FS
Static	1.711
*Seismic (0.1g - Small earthquake)	1.341
*Seismic (0.2g – Moderate earthquake)	1.083
*Seismic (0.3g – Large earthquake)	0.846
**Post-Earthquake (No-mitigation)	1.006

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

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 the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot 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.

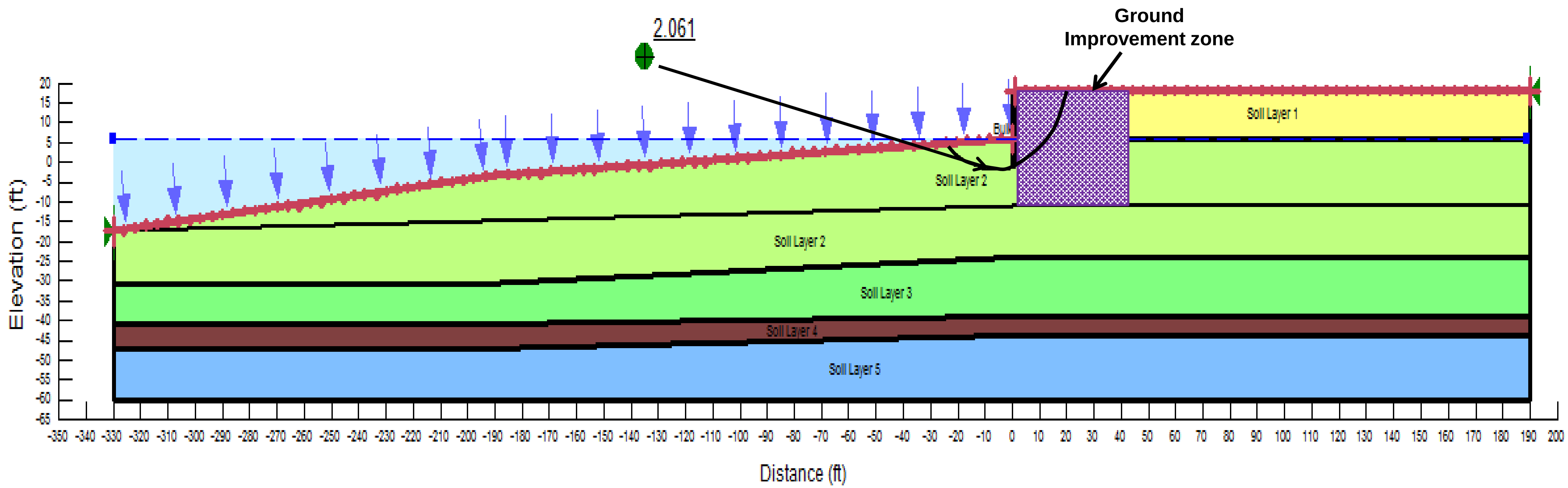
Reference: From SLOPE/W.

Mukilteo Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GEOENGINEERS

Figure 67



Loading Condition	FS
Static	2.061
*Seismic (0.1g - Small earthquake)	1.646
*Seismic (0.2g – Moderate earthquake)	1.362
*Seismic (0.3g – Large earthquake)	1.128
**Post-Earthquake (No-mitigation)	0.346
**Post-Earthquake (Mitigation)	1.031

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

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 the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot 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.

Reference: From SLOPE/W.

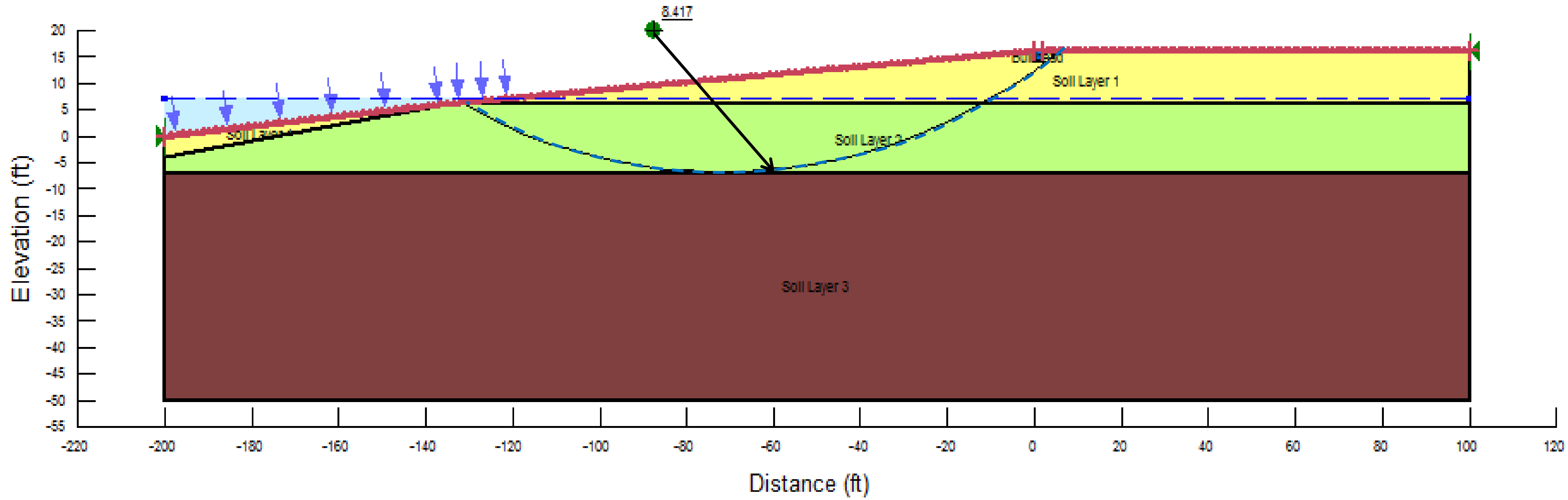
Edmonds Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GeoEngineers

Figure 68

: SP:\0\0180284\00\finals\Figures 72 – 78 Bulkhead Stability.ppt HPD 03/22/12



Loading Condition	FS
Static	8.417
*Seismic (0.1g - Small earthquake)	3.380
*Seismic (0.2g – Moderate earthquake)	2.071
*Seismic (0.3g – Large earthquake)	1.483
**Post-Earthquake (No-mitigation)	8.417

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

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 the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot 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.

Reference: From SLOPE/W.

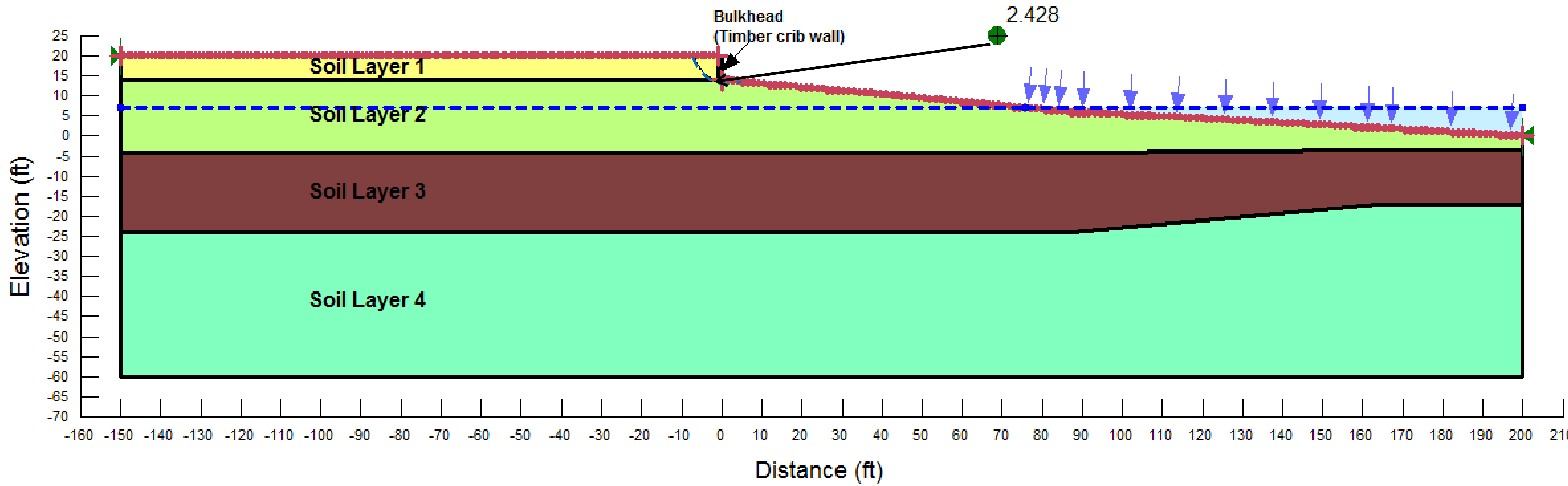
Fauntleroy Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GEOENGINEERS

Figure 69

: SP:\0\0180284\00\finals\Figures 72 – 78 Bulkhead Stability.ppt HPD 03/22/12



Loading Condition	FS
Static	2.428
*Seismic (0.1g - Small earthquake)	2.094
*Seismic (0.2g – Moderate earthquake)	1.810
*Seismic (0.3g – Large earthquake)	1.557

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

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 the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot 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.

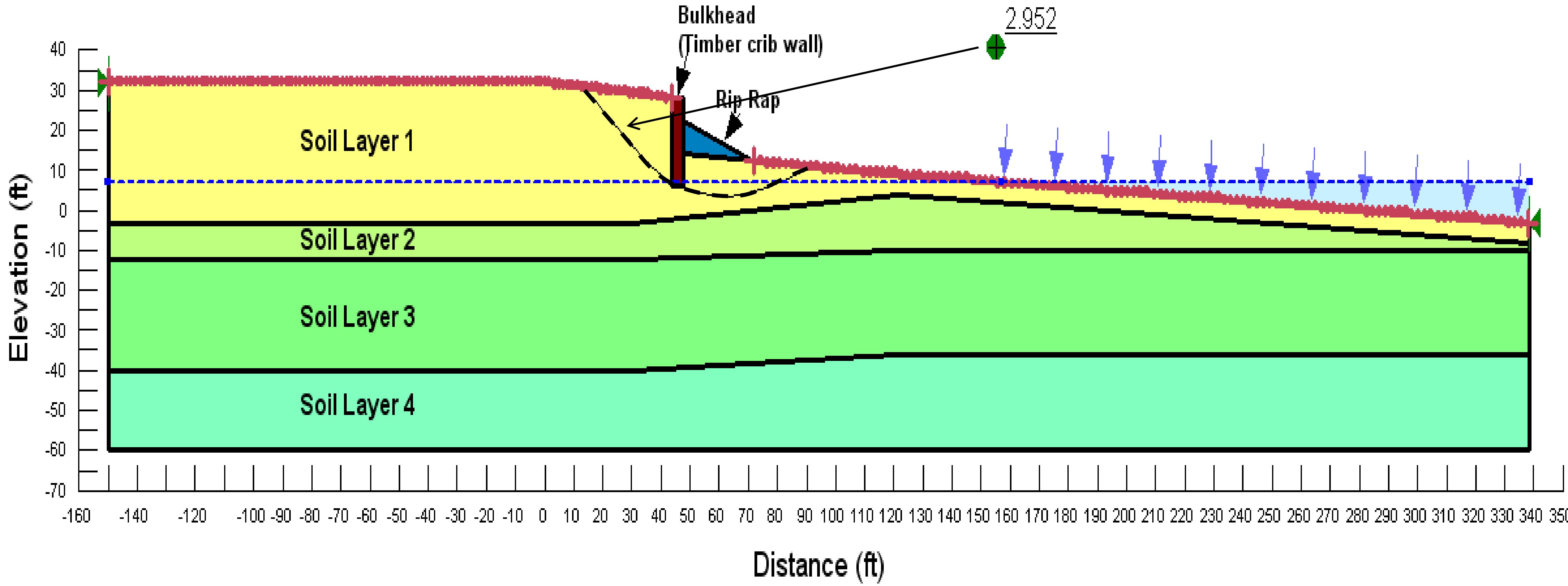
Reference: From SLOPE/W.

Southworth Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GeoEngineers

Figure 70



Loading Condition	FS
Static	2.952
*Seismic (0.1g - Small earthquake)	2.288
*Seismic (0.2g – Moderate earthquake)	1.777
*Seismic (0.3g – Large earthquake)	1.339

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

Notes:

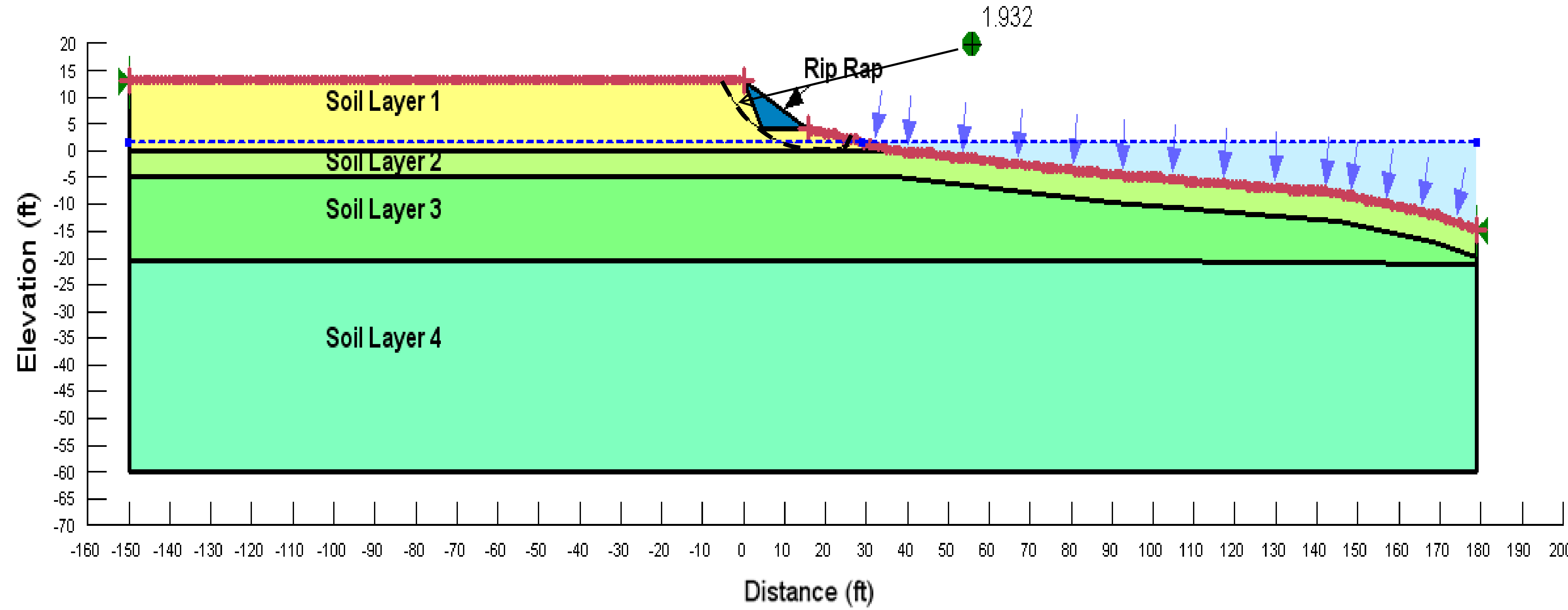
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Reference: From SLOPE/W.

Tahlequah Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GEOENGINEERS | **Figure 71**



Loading Condition	FS
Static	1.932
*Seismic (0.1g - Small earthquake)	1.526
*Seismic (0.2g – Moderate earthquake)	1.255
*Seismic (0.3g – Large earthquake)	1.037

Table Notes:
* Seismic condition – During earthquake; includes seismic load.
**Post-Earthquake condition – After earthquake, includes residual strength of Liquefied soils.

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 the Bulkhead Stability Results section of this report. GeoEngineers, Inc. cannot 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.

Reference: From SLOPE/W.

Point Defiance Ferry Terminal – Critical Failure Surface

Washington State Ferries Timber Trestles Project
Puget Sound Area

GeoEngineers

Figure 72

: SP:\0\0180284\00\finals\Figures 72 – 78 Bulkhead Stability.ppt HPD 03/22/12

APPENDICES

APPENDIX A
Review of Subsurface Soil Conditions

APPENDIX A
SUBSURFACE SOIL CONDITIONS

The subsurface soil conditions at the sites were evaluated by reviewing the logs of borings completed near the existing timber trestles at each terminal, and by reviewing the U.S. Geological Survey (USGS) geologic map of the area. The logs of the borings reviewed were provided by the Washington State Ferries (WSF). The following sections of this appendix describe the units encountered in each terminal in the order of deposition, starting with the most recent.

Friday Harbor Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-1-03 and H-7-03) completed near the existing timber trestles provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Unconsolidated deposits and Bedrock. The two soil units consisted of the following:

- **Unconsolidated Deposits** consist of about 6.5 feet of loose gravel with sand and silt.
- **Bedrock** was encountered at the mudline at the north end of the existing trestle and at depths of about 6.5 feet at the south end of the trestle, and generally consisted of meta-sedimentary formation, with very tightly spaced discontinuities. The rock quality designation (RQD) number for the top 5 feet of the bedrock encountered generally ranges from 0 percent to 22 percent.

Lopez Island Terminal

The subsurface soil conditions at the site were evaluated by reviewing the log of one boring (H-1-97) completed at the terminal provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Unconsolidated deposits and Bedrock. The two soil units consisted of the following:

- **Unconsolidated Deposits** consist of about 6 feet of loose sand with gravel.
- **Bedrock** was encountered at depths of about 6 feet, and consisted of a fresh conglomerate with an average of seven fractures per 0.3 m in the upper 6 m (20 feet). The RQD number for the top 5 feet of the bedrock encountered is about 67 percent.

Shaw Island Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-1-97, H-2-97 and H-3-02) completed near the existing timber trestles provided by the WSF and by reviewing the USGS geologic map of the area. In general three soil types were encountered in the explorations reviewed: Unconsolidated deposits, glacially consolidated soils and Bedrock. The three soil units consisted of the following:

- **Unconsolidated Deposits** consist of 5 to 10 feet of loose to medium dense unconsolidated sand and gravel.
- **Glacially Consolidated Soils** were encountered beneath the unconsolidated sand and gravel deposits. The glacially consolidated deposits ranged from 5 to 10 feet thick.

- **Bedrock** was encountered at the mudline at the south end of the existing trestle and at depths of about 15 to 16 feet at the north end of the trestle, and generally consisted of marine meta-sedimentary rock. The RQD number for the top 5 feet of the bedrock encountered generally ranges from 0 percent to 40 percent.

Orcas Island Terminal

The subsurface soil conditions within the footprint of the trestle were evaluated by reviewing the logs of the borings (3-85 through 5-85) provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Glacially consolidated soils and Bedrock. The two soil units consisted of the following:

- **Glacially Consolidated Soils** encountered were very dense and ranged from 14 to 26 feet thick.
- **Bedrock** was encountered at about 20 feet below the mudline at the north end of the existing trestle and at depths of about 26 feet at the south end of the trestle. The rock encountered in the boring was meta-sedimentary rock generally consisting of fine grained, strong rock with closely spaced discontinuities.

Anacortes Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (A-1-93 and H-4-99) completed near the existing trestle provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Unconsolidated deposits and Glacially consolidated soils. The two soil units consisted of the following:

- **Unconsolidated Deposits** consist of loose to medium dense sand with silt, encountered in the upper 3 to 16 feet.
- **Glacially Consolidated Soils** were encountered beneath the unconsolidated deposits, and consist of dense to very dense sand with silt and gravel.

Mukilteo Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings completed at the site for previous projects provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Tide flat deposits and Glacial drift. The two soil units consisted of the following:

- **Tide Flat Deposits** consist of loose sand with silt, encountered in the upper 10 to 17 feet.
- **Glacial Drift** was encountered beneath the tide flat deposits, and consists of medium dense sand with silt and gravel.

Edmonds Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-5-94 through H-9-94) completed near the existing timber trestles provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Unconsolidated deposits and Glacially consolidated soils. The two soil units consisted of the following:

- **Unconsolidated Deposits** consist of loose to medium dense unconsolidated sand and gravel, encountered in the upper 4 to 11 feet.
- **Glacially Consolidated Soils** were encountered beneath the unconsolidated deposits, and consist of dense to very dense sand with silt and gravel.

Fauntleroy Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-1-83 and H-2-83) completed at the site for previous projects provided by the WSF and by reviewing the USGS geologic map of the area. In general three soil types were encountered in the explorations reviewed: Artificial Fill, Beach deposits and Recessional glacial drift. The three soil units consisted of the following:

- **Artificial Fill** encountered was loose to medium dense.
- **Beach Deposits** encountered were medium dense to dense.
- **Recessional Glacial Drift** was encountered below the beach deposits in all of the borings reviewed.

Vashon Island Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-01-11 through H-03-11) completed at the site for previous projects provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Unconsolidated deposits and Glacially consolidated soils. The two soil units consisted of the following:

- **Unconsolidated Deposits** were encountered in the upper 3 to 10 feet of the borings, and consist of loose to medium dense sand with silt.
- **Glacially Consolidated Soils** were encountered beneath the unconsolidated deposits, and consist of dense to very dense sand with silt and gravel.

Southworth Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-1-99, H-2-99 and H-4-99) completed near the existing trestle provided by the WSF and by reviewing the USGS geologic map of the area. In general two soil types were encountered in the explorations reviewed: Unconsolidated deposits and Glacially consolidated soils. The two soil units consisted of the following:

- **Unconsolidated Deposits** were encountered in the upper 5 to 13 feet and generally consists of loose to medium dense sand with silt and gravel.
- **Glacially Consolidated Soils** were encountered beneath the unconsolidated deposits, and consist of very stiff to hard clay and dense to very dense silty sand and sandy silt soils.

Tahlequah Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (H-3-02 and H-4-02) completed near the existing timber trestles provided by the WSF and by reviewing the USGS geologic map of the area. Subsurface soils near the timber trestle generally consist of glacially consolidated soils. The soil unit consisted of the following:

- **Glacially Consolidated Soils** were encountered in all the explorations reviewed, and consist of medium dense to very dense sand with silt.

Point Defiance Terminal

The subsurface soil conditions at the site were evaluated by reviewing the logs of the borings (HQ-2, HQ-6 and HQ-7) completed near the existing timber trestles provided by the WSF and by reviewing the USGS geologic map of the area. Subsurface soils near the timber trestle generally consist of glacially consolidated soils. The soil unit consisted of the following:

- **Glacially Consolidated Soils** were encountered in all the explorations reviewed, and consist of dense to very dense sand with silt.

Appendix B
Report Limitations and Guidelines for Use

APPENDIX B
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 final report has been prepared for the exclusive use of the Washington State Ferries, and their authorized agents. 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 with the Client 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 draft report has been prepared for the Washintong State Ferries timber trestles project. 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.

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

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;
- 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.

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.

Have we delivered World Class Client Service?

Please let us know by visiting **www. geoengineers.com/feedback**.

GeoEngineers