SAGE
GEOTECHNICAL

DRAFT TECHNICAL MEMORANDUM

TO: Trevor Louviere, PE, Project Manager, Dalton, Olmsted & Fuglevand, Inc.

FROM: Calvin McCaughan, PE, and Daniel Simpson, PE

DATE: November 8, 2024

RE: Summary of Geotechnical Engineering Services
Former Arkema Manufacturing Site
Tacoma, Washington
Sage Project No. 046001

INTRODUCTION

At the request of Dalton, Olmsted & Fuglevand, Inc. (DOF, prime engineer), Sage Geotechnical, LLC (Sage) provided geotechnical engineering services to support the design of a vibrating beam containment wall (VBCW) and soil cap for the Former Arkema Manufacturing Site project in Tacoma, Washington (site; Figure 1).

The following services were provided in accordance with the scope outlined in Sage's February 21, 2024, proposal:

- A review of existing subsurface and geotechnical laboratory data, including boring logs prepared by others (see boring locations on Figure 2).
- An analysis of the amount of settlement that may occur following placement of the soil cap.
- An analysis of the VBCW performance during a seismic event.
- An assessment of the impact the VBCW may have on shoreline slope stability.

The conclusions presented in Sage's August 19, 2024, memorandum have been supplemented with the geotechnical recommendations noted in the following section.

NOVEMBER 2024 UPDATE

On October 17, 2024, DOF authorized Sage to develop the following supplemental recommendations (collectively, "site development recommendations"):

- Geotechnical recommendations for earthwork and utilities.
- Pile design recommendations for outfall structures.
- Lateral bearing capacity recommendations for light pole foundations.

The site development recommendations are provided in Attachment 1.

SETTLEMENT ANALYSIS

The site will be filled to an elevation of approximately 22 feet (ft), or to 2 to 8 ft above the existing grade at the proposed VBCW location. Compressible soils at this location may experience consolidation settlement when fill is placed. Sage estimated the magnitude of ground surface settlement and drag loads that could be imposed on the VBCW.

Settlement Calculations

Sage used data generated by DOF[1] and Budinger & Associates, Inc. (geotechnical testing laboratory)[2] to evaluate the potential for soil compression. Based on its evaluation, Sage concluded that the units identified as the Upper Aquitard and Lower Aquitard contain fine-grained, settlement-sensitive soils.

Soils in the Lower Aquitard will likely experience a small to moderate change in vertical effective stress relative to existing stress. Detailed calculations were not performed for each boring location. Instead, a representative thickness of 7 ft, coupled with an average normalized virgin compression index of 0.1, was used to estimate settlement in the Lower Aquitard. Sage estimates that approximately 1 inch of consolidation settlement will occur in the Lower Aquitard.

Sage used the soil conditions reported at each boring location to calculate consolidation settlement in the Upper Aquitard. Budinger & Associates performed three consolidation tests in support of the project; however, two of the tests appear to have been completed on coarse-grained or non-plastic, fine-grained soil.[3] The results of these two tests were disregarded, and the normalized virgin compression index from the third test (0.17) was used to complete the calculations. Settlement calculations for the Upper Aquitard are summarized in Table 1.

The test data indicate that the pre-consolidation pressure is near, or just above, the *in-situ* vertical effective stress. There is no known site history to suggest that alluvial sediments are pre-consolidated. Therefore, the settlement calculations are based on the assumption that soils will behave in accordance with the virgin compression trend.

Settlement Conclusions

Sage recommends assuming an average, sitewide settlement of 6 inches. This quantity should be used to estimate the volume of imported fill. Sage estimates that settlement will be complete 1 to 2 months after the final lift of fill has been placed. The fill elevation should be monitored for several weeks to confirm that settlement has sufficiently decayed.

---

[1] Boring logs and barrier wall design cross sections.
[2] Soil laboratory data.
[3] Based on the initial void ratio and the change in the void ratio over the stress range of testing.

The settlement calculations in Table 1 are conservative, as the Upper Aquitard appears to contain zones of low-plasticity, fine-grained soils with a compression index that is likely less than that assumed. This variability was not accounted for in the calculation.

Long-term secondary compression may occur. The majority of the site will likely experience less than 1 inch of secondary compression over the next 50 years. If present, isolated pockets of organic-rich waste, soil, or estuarine deposits would increase the potential for secondary compression beyond 1 inch.

Drag Loads

Drag loads, also referred to as negative skin friction, occur on piles or other buried structures when the surrounding soil moves downward. The proposed VBCW could be subject to drag loads caused by consolidation settlement.

The drag load is a function of friction between the *in-situ* soil and the VBCW. Sage understands that the slurry will consist of 5 percent clay, 18 percent granular ground blast furnace slag, and 77 percent water. Sage assumes that the slurry will cure into a material with a consistency similar to soft clay with sand. When estimating the drag load, Sage assumed that the clay would have an internal angle of friction of 22 degrees, and the slurry shear strength would control the drag load.

The drag load is also a function of horizontal stress in the soil. Sage has assumed a horizontal-to-vertical stress ratio of 0.5.

Based on the above parameters, Sage estimates a total drag load of $10H^2$ pounds per foot of wall, where H is the height of the wall. This calculation accounts for both sides of the wall. For example, a 30-ft-deep VBCW would accumulate a drag load of approximately 9,000 pounds per foot of wall. The VBCW cured slurry and native site soils exhibit a similar stiffness; as such, the drag load will likely cause the VBCW to compress with the surrounding soil. The toe of the wall may punch lower in elevation, depending on the toe bearing capacity and wall material shear strength. The total vertical movement of the wall is unlikely to exceed sitewide settlement (6 inches).

SEISMIC ANALYSIS

Sage calculated design-response spectra for three earthquake events: an operational level earthquake (OLE), a contingency level earthquake (CLE), and a design earthquake (DE). Sage's assessment was completed in general accordance with the American Society of Civil Engineers' *Seismic Design of Piers and Wharves* (hereafter, *ASCE 61-14*). Over 50 years, the seismic hazard exceedance probability is 50 percent for the OLE, 10 percent for the CLE, and 2 percent for the DE. These exceedance probabilities correspond to return periods of approximately 75 years for the OL; 500 years for the CLE; and 2,500 years for the DE.

Liquefaction and lateral spreading analyses were also performed for each hazard level. Because seismic analyses were based on guidance in *ASCE 61-14,* the results may not be applicable to structures that will be designed in accordance with *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE 2017) and/or the *2021 International Building Code* (ICC 2021). As such, the selected foundation type [for future site redevelopment] should not be based on the ground deformation estimates in this memorandum.

Overall seismic performance was evaluated using subsurface data collected from 15 shallow soil borings advanced by DOF (2023) and five deep soil borings advanced by Landau Associates, Inc. (2009). The approximate locations of the borings are shown on Figure 2.

Seismic Response Spectra

Prior to calculating the design-response spectra, Sage evaluated the seismic site class in accordance with *ASCE 61-14.* The evaluation was completed using a downhole shear wave velocity (Vs) profile measured by cone penetration test (RRI-CPT-168B) and a suspension-logged Vs profile measured in a borehole (RRI-B-167B) near the site (Landau 2009). Both Vs profiles resulted in seismic Site Class E.

Design-response spectra were calculated in general accordance with *ASCE 61-14;* however, the U.S. Geological Survey's 2008 National Seismic Hazard Map (NSHM) was used to determine the seismic parameters for the OLE, CLE, and DE, as *ASCE 61-14* uses an older NSHM that is no longer readily available. As a result, the design-response spectra in Table 2 are generally more intense than those required by *ASCE 61-14.*

Liquefaction Analysis Approach

Grain size and plasticity data typically are used to determine soil liquefaction susceptibility, with low-plasticity soils deemed susceptible to liquefaction and high-plasticity soils not susceptible. Unified Soil Classification System soils with group symbols ending in "H" (i.e., high-plasticity soils, like MH, CH, and OH) and soils classified as "CL" were deemed not susceptible, and all other soils were deemed susceptible. Where Atterberg limits data were available, the susceptibility criteria described above were compared to recommendations by Bray and Sancio (2006) and generally found to be consistent.

Liquefaction triggering is evaluated by comparing the cyclic resistance ratio (strength) to the earthquake cyclic stress ratio (demand). Sage used the empirical procedure proposed by Idriss and Boulanger (2008, updated 2014) to complete a liquefaction triggering analysis.

Liquefaction-induced ground surface settlement and residual soil strength were also computed using the procedure proposed by Idriss and Boulanger (2008, updated 2014). Ground surface settlement at the shallow boring locations was computed by adding the average cumulative settlement from the deep borings at 40 ft (i.e., at the approximate base of the shallow borings).

Lateral Spreading Analysis Approach

Lateral spreading potential was estimated using an empirical Newmark sliding block analysis. First, a limit equilibrium model was developed using Rocscience Slide2 and liquefied (residual) soil strength properties where applicable. Residual strength profiles were prepared for the CLE and DE hazard levels. Sage developed the profiles using subsurface data from the 15 borings that extended approximately 35 to 40 ft below ground surface and the five borings that extended 70 to 80 ft below ground surface. The subsurface data indicate that pockets of liquefiable soil are present in all of the borings; the elevation, composition, and severity of the liquefiable soils vary. Because the lateral spreading failure surfaces are very large (i.e., engage vast areas of soil), the computed residual strengths for all borings were averaged and enveloped by a design profile, as shown on Figure 3. Because the CLE and DE hazard levels have nearly the same residual strengths, only the DE level was modeled to determine the yield acceleration.

The residual strength profile included the lateral extent of the limit-equilibrium model (Figure 4). The limit equilibrium model was developed to compute the magnitude, location, and distribution of seismic yield accelerations and corresponding slide masses (Figure 5).

Once yield accelerations had been calculated, the corresponding lateral spreading horizontal displacements were computed using empirical methods recommended by Bray and Macedo (2019) and Bray et al. (2018). These computations were used to create the lateral spreading contours shown on Figure 2; the contours depict estimated boundaries of equal horizontal displacement.

Seismic Analysis Conclusions

Soil layers in each of the 20 borings are susceptible to liquefaction following a DE or CLE event. Few soil layers are susceptible to liquefaction following an OLE event. The soil residual strength and degree of liquefaction-induced settlement are similar for DE and CLE events. Table 3 includes the estimated liquefaction-induced settlement for each boring location following DE, CLE, and OLE events. Based on these estimates, Sage recommends assuming an average free-field ground surface settlement of 13 inches for the CLE event and 16 inches for the DE event. Sage estimates that, following a DE or CLE event, approximately 13 inches of differential settlement will occur over a horizontal span of 50 ft. The OLE event is not anticipated to cause significant ground surface movement.

Although they have similar yield accelerations, the CLE event will result in less lateral spreading than the DE event, as the CLE event has a smaller seismic demand (i.e., strong ground motion magnitude). The lateral spreading contours shown on Figure 2 can be used to estimate lateral ground movement following a DE or CLE event. The OLE event is not anticipated to cause significant lateral ground movement in the upland portion of the site.

Sage reviewed information regarding an existing sheet pile wall and the proposed VBCW. Both structures are anticipated to have a negligible impact on liquefaction-induced lateral spreading and settlement. During lateral spreading, the sheet pile wall and VBCW will likely deform with the surrounding ground.

SHORELINE SLOPE STABILITY

Approach

Sage evaluated the stability of the segment of shoreline in the vicinity of the proposed VBCW. A typical cross section used for stability analyses is provided on Figure 6. The results of the analyses were used to assess whether ground vibrations caused by VBCW installation could contribute to slope instability.

A vibratory pile hammer, likely an APE Model 150 or comparable, will be used to install the VBCW. A hammer of this size typically vibrates at a frequency of approximately 30 hertz. Peak particle velocities (PPVs) for vibratory pile hammers typically range from 0.1 inches per second to 0.8 inches per second at 25 ft from the pile (Wilson, Ihrig & Associates, Inc. et al. 2012). The slurry pocket around the vibrating beam will likely attenuate vibrations produced by the pile hammer. Therefore, a PPV of 0.45 inches per second at 25 ft is assumed to be representative of VBCW installation.

A pseudo-static slope stability analysis was completed to evaluate the distribution of yield acceleration within the slope (Figure 6). Based on the results of the analysis, the minimum yield acceleration is approximately 0.11g, where “g” stands for units of gravity. The centroid of the slide mass is approximately 100 ft from the proposed VBCW location, and ground vibrations (PPVs) are likely to attenuate by the time they reach the most unstable portion of the slope.

Sage used the Federal Transit Administration’s (2018) proposed vibration-attenuation relationship to estimate a PPV of 0.16 inches per second at the centroid of the critical slip surface.

Given the wide frequency range and unknown amplitude of the ground vibrations, peak ground accelerations could not be calculated directly from the PPV. Instead, Sage reviewed literature on soil slope stability in the presence of controlled blasting (Wong and Pang 2000). Findings show that, for the slope materials, inclination, and PPV indicated, vibration-induced slope instability is unlikely to occur.

Vibrations from VBCW installation could cause soil liquefaction and ground settlement. Soil liquefaction, if any, is likely to be localized to the pile, as PPVs of more than 2 inches per second typically are required to liquefy loose sand when conducting blast-induced liquefaction studies (Ishimwe et al. 2020). Several inches of soil settlement are likely to occur within a horizontal distance of the VBCW that is equal to the beam depth.

Conclusions

Based on the results of its stability analysis, Sage concludes that VBCW installation is unlikely to cause slope instability. Several inches of soil settlement should be anticipated adjacent to the VBCW. During the final design phase, the project design team should discuss settlement monitoring of the sheet pile wall, as VBCW installation could cause the sheet piles to settle. The sheets are unlikely to settle more than the surrounding ground.

USE OF THIS TECHNICAL MEMORANDUM

Sage Geotechnical, LLC has prepared this technical memorandum for the exclusive use of Dalton, Olmsted & Fuglevand, Inc; the Port of Tacoma; and their designated representatives for specific application to the Former Arkema Manufacturing Site project in Tacoma, Washington. No other party is entitled to rely on the information, conclusions, and recommendations included in this document without the express written consent of Sage Geotechnical. Reuse of the information, conclusions, and recommendations provided herein for extensions of the project or for any other project, without review and authorization by Sage Geotechnical, shall be at the user’s sole risk. Sage Geotechnical warrants that, within the limitations of scope, schedule, and budget, its services have been provided in a manner consistent with that level of skill and care ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. Sage Geotechnical makes no other warranty, either express or implied.

CLOSING

We trust that this memorandum provides you with the information needed to proceed with the project. If you have questions or comments, or if we can be of further service, please contact Calvin McCaughan at calvinm@sagegeotechnical.com.

SAGE GEOTECHNICAL, LLC

Calvin McCaughan, PE
Principal Engineer

Daniel Simpson, PE
Principal Engineer

DCS/MCS/CAM
[HTTPS://SAGEGEOTECHNICAL.SHAREPOINT.COM/SITES/SAGEGEOTECHNICAL/SHARED DOCUMENTS/PROJECTS/046 DALTON OLMSTED FUGLEVAND/R/REVISED REPORT/FORMER ARKEMA MANUFACTURING SITE DRAFT TECHNICAL MEMORANDUM 11.8.2024.DOCX]

Attachments: Figure 1. Vicinity Map
Figure 2. Site Plan
Figure 3. Residual Strength Profiles
Figure 4. Lateral Spreading Profile A
Figure 5. Yield Acceleration Calculation
Figure 6. Shoreline Slope Stability
Table 1. Upper Aquitard Consolidation Settlement Summary
Table 2. Seismic Response Spectra
Table 3. Estimated Liquefaction Settlement
Attachment 1. Site Development Recommendations

REFERENCES

ASCE. 2017. *Minimum Design Loads and Associated Criteria for Buildings and Other Structures (7-16).* American Society of Civil Engineers.

ASCE. 2014. *Seismic Design of Piers and Wharves (61-14).* American Society of Civil Engineers.

Boulanger, R.W., and I.M. Idriss. 2014. "CPT and SPT-based Liquefaction Triggering Procedures." Report No. UCD/CGM-14.

Bray, J.D., and J. Macedo. 2019. "Procedure for Estimating Shear-Induced Seismic Slope Displacement for Shallow Crustal Earthquakes." *Journal of Geotechnical and Geoenvironmental Engineering.* 145.12.

Bray, J.D., J. Macedo, and T. Travasarou. 2018. "Simplified Procedure for Estimating Seismic Slope Displacements for Subduction Zone Earthquakes." *Journal of Geotechnical and Geoenvironmental Engineering.* 144.3.

Bray, J.D., and R.B. Sancio. 2006. "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils." *Journal of Geotechnical and Geoenvironmental Engineering.* 132.9:1165–1177.

Budinger & Associates, Inc. 2023. Arkema Manufacturing Site 2023 Materials, Results of Laboratory Testing, Report #1. November 21.

DOF. 2024. Barrier Wall Design Cross Sections added to Preliminary 30% Design: Arkema Manufacturing Area Interim Action Containment Wall. Dalton, Olmsted & Fuglevand, Inc.

DOF. 2023. Boring Logs: Arkema Pre-Design Investigation. Dalton, Olmsted & Fuglevand, Inc. October.

FTA. 2018. *Transit Noise and Vibration Impact Assessment Manual.* FTA Report No. 0123. Federal Transit Administration. September.

ICC. 2021. *International Building Code.* International Code Council. September.

Idriss, I.M., and R.W. Boulanger. 2008. *Soil Liquefaction during Earthquakes.* Monograph MNO-12. Earthquake Engineering Research Institute.

Ishimwe, E., R.A. Coffman, and K.M. Rollins. 2020. "Predicting Blast-Induced Liquefaction Within the New Madrid Seismic Zone." *AIMS Geosciences.* 6(1):71–91.

Landau. 2009. Draft Preliminary Geotechnical Report: Road/Rail/Infrastructure Project, 60 Percent Design Phase, Port of Tacoma, Tacoma, Washington. Landau Associates, Inc. February 17.

USGS. 2008. Documentation for the 2008 Update of the United States National Seismic Hazard Maps. Open File Report 2008-1128. U.S. Department of the Interior. U.S. Geological Survey.

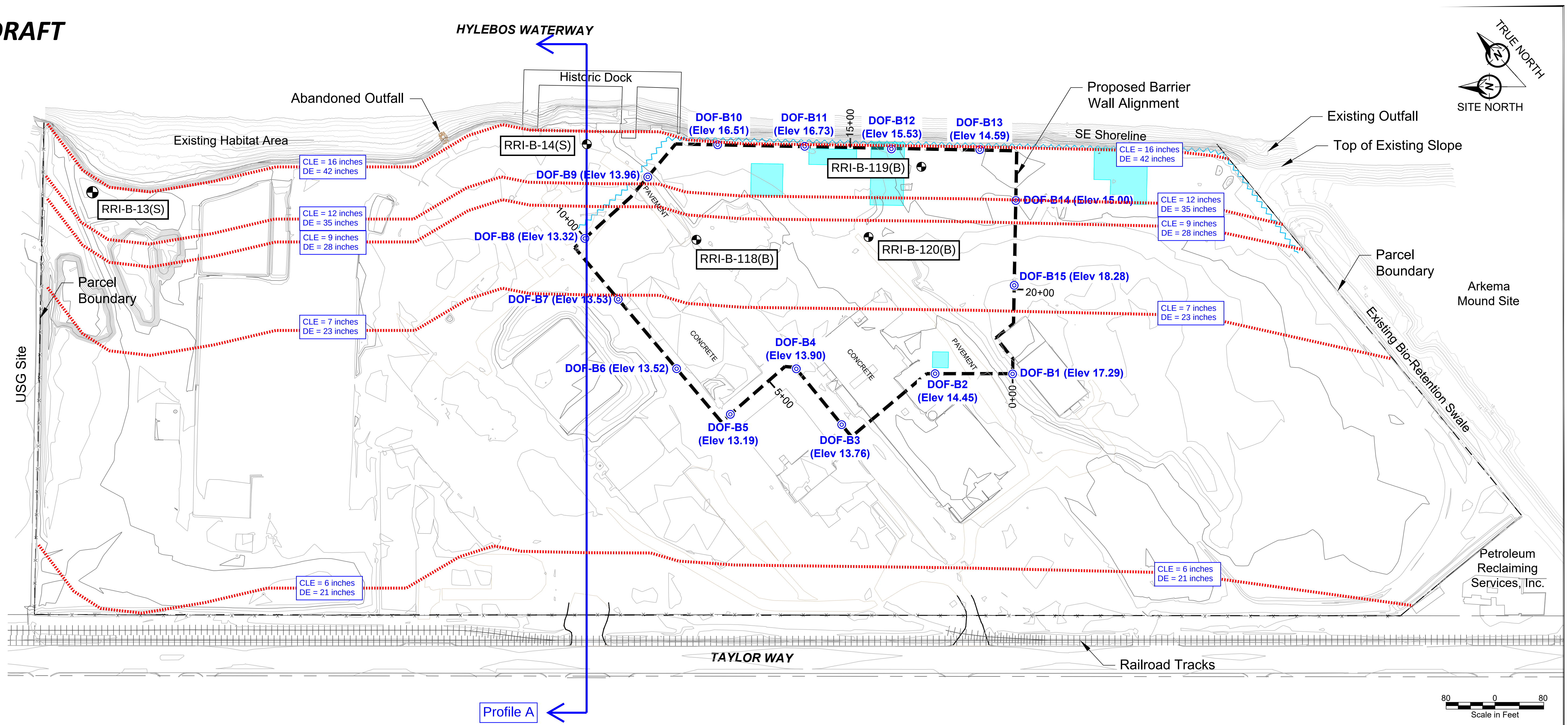
Wilson, Ihrig & Associates, Inc.; ICF International; and Simpson, Gumpertz & Heger, Inc. 2012. Report: Current Practices to Address Construction Vibration and Potential Effects to Historic Buildings Adjacent to Transportation Projects. NCHRP 25-25/Task 72. September.

Wong, H.N., and P.L.R. Pang. 2000. Assessment of Stability of Slopes Subjected to Blasting Vibration. Geo Report No. 15. Geotechnical Engineering Office, Civil Engineering Department, The Government of the Hong Kong Special Administrative Region. August.

SAGE GEOTECHNICAL

Vicinity Map	
Former Arkema Manufacturing Site	
Tacoma, Washington	Figure 1

DRAFT



Legend

◎	DOF (2023) exploration location and designaton
▬ ▬ ▬	Barrier Wall Alignment
0+00 —	Barrier Wall Alignment Stationing
—— · ——	Parcel Boundary
∿∿∿∿	Existing Sheet Pile Wall
—x—x—	Existing Fence
▭	Thin or Leaky First Aquitard Location
RRI-B-14(S)	Landau (2009) exploration location and designation
CLE = 6 inches / DE = 21 inches	Lateral spreading equal-displacement contour

Contingency level earthquake (CLE) and design earthquake (DE) lateral spreading horizontal displacement

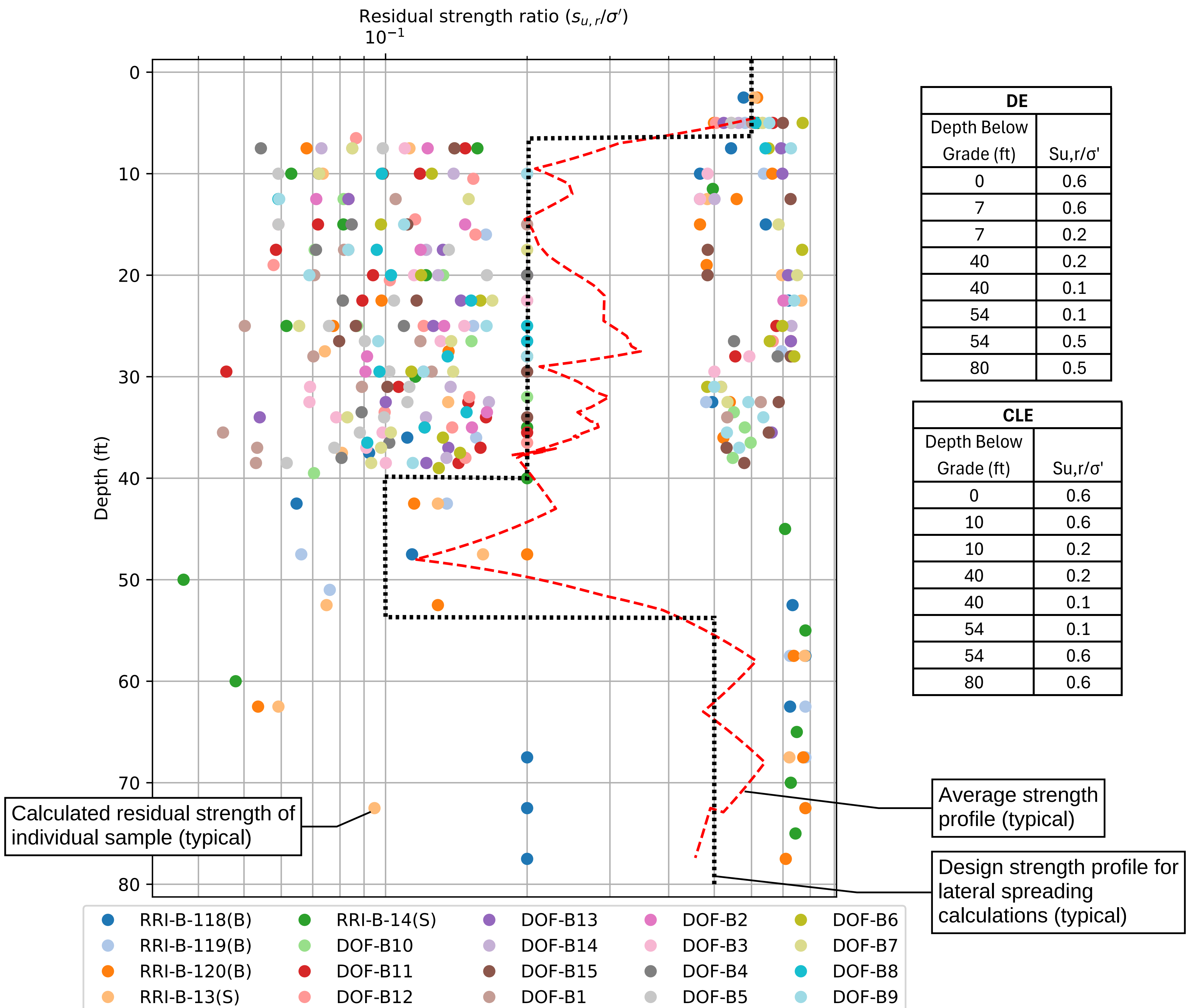
Source: DOF (2023)

SAGE
GEOTECHNICAL

Site Plan	
Former Arkema Manufacturing Site	
Tacoma, Washington	Figure 2

DRAFT

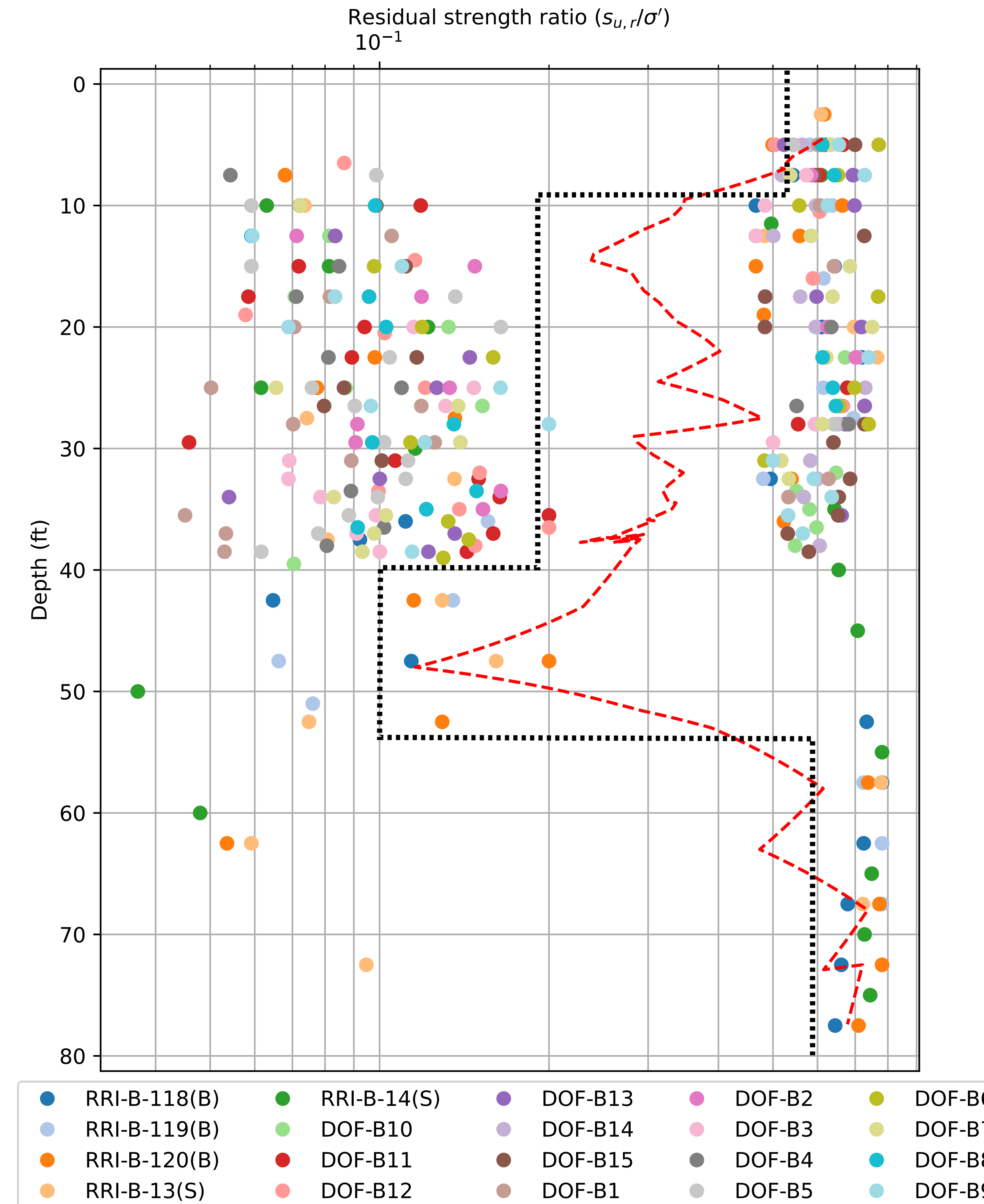
DE



DE	
Depth Below Grade (ft)	Su,r/σ'
0	0.6
7	0.6
7	0.2
40	0.2
40	0.1
54	0.1
54	0.5
80	0.5

CLE	
Depth Below Grade (ft)	Su,r/σ'
0	0.6
10	0.6
10	0.2
40	0.2
40	0.1
54	0.1
54	0.6
80	0.6

CLE

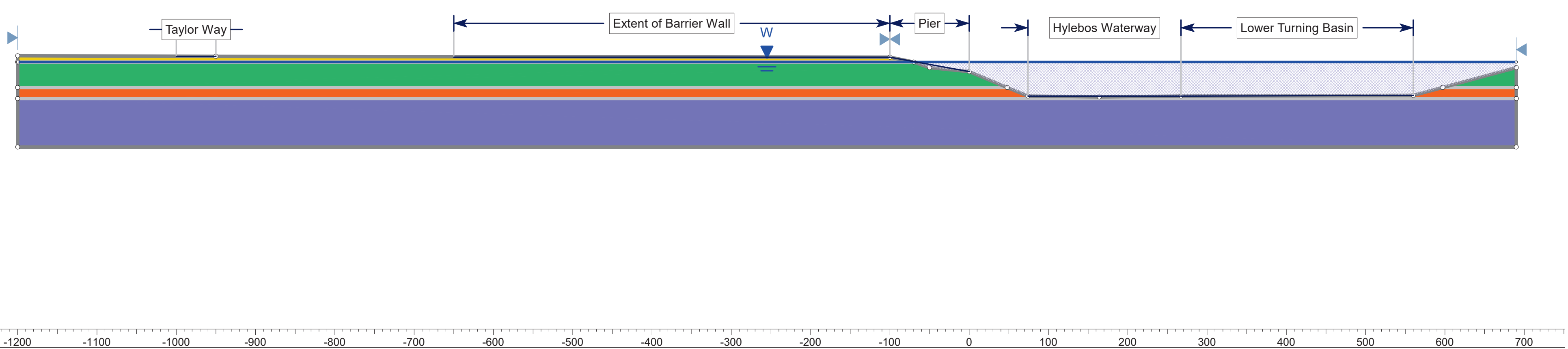


SAGE GEOTECHNICAL

Residual Strength Profiles	
Former Arkema Manufacturing Site	
Tacoma, Washington	Figure 3

DRAFT

Material Name	Color	Unit Weight (lbs/ ft3)	Strength Type	Vertical Stress Ratio	Minimum Shear Strength (psf)	Water Surface	Ru Value
Layer 1		120	Vertical Stress Ratio	0.6	100	None	0
Layer 2		120	Vertical Stress Ratio	0.2	0	None	0
Layer 3		120	Vertical Stress Ratio	0.1	0	None	0
Layer 4		120	Vertical Stress Ratio	0.5	0	None	0



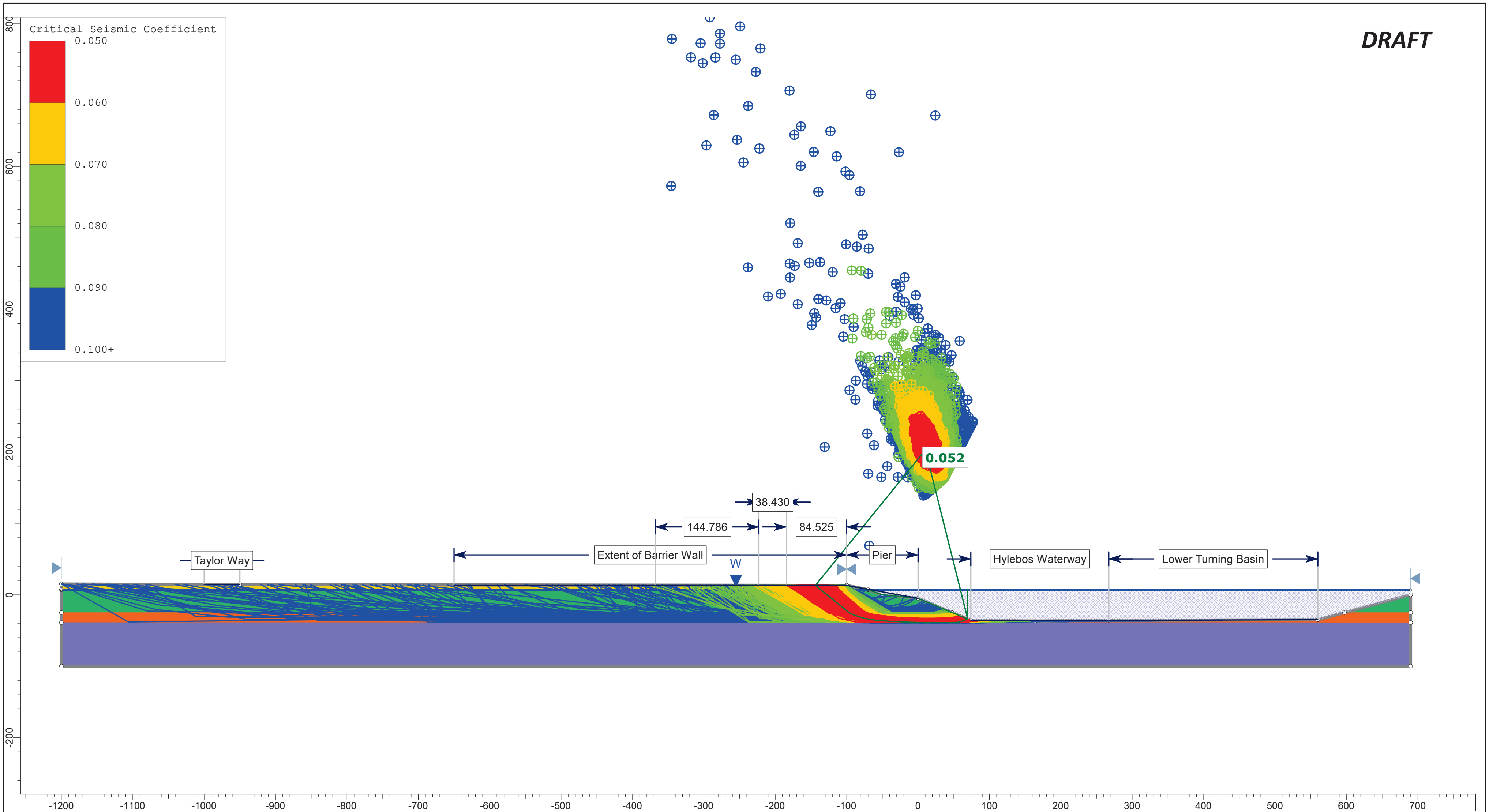
SAGE GEOTECHNICAL

Lateral Spreading Profile A

Former Arkema Manufacturing Site

Tacoma, Washington

Figure 4

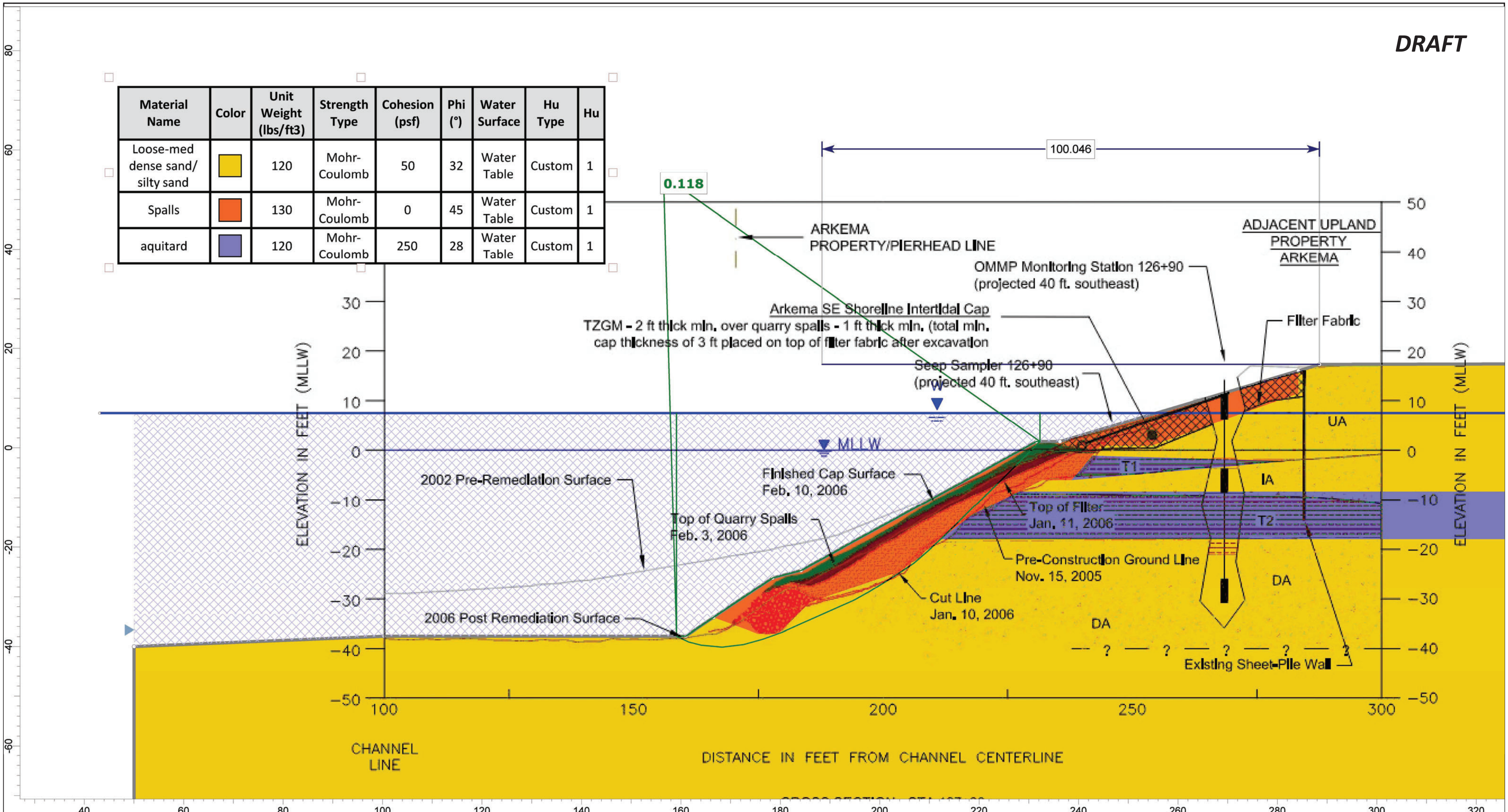


SAGE GEOTECHNICAL

Yield Acceleration Calculation	
Former Arkema Manufacturing Site	
Tacoma, Washington	Figure 5

DRAFT

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Water Surface	Hu Type	Hu
Loose-med dense sand/ silty sand		120	Mohr-Coulomb	50	32	Water Table	Custom	1
Spalls		130	Mohr-Coulomb	0	45	Water Table	Custom	1
aquitard		120	Mohr-Coulomb	250	28	Water Table	Custom	1



Background image adapted from PGG 2006.

SAGE GEOTECHNICAL

Shoreline Slope Stability	
Former Arkema Manufacturing Site	
Tacoma, Washington	Figure 6

DRAFT

Table 1.
Upper Aquitard Consolidation Settlement Summary
Former Arkema Manufacturing Site
Tacoma, Washington

Assumed Groundwater Elevation:	10	ft
Assumed ESU2 Normalized Virgin Compression Index, Ccε:	0.17	
Assumed Unit Weight of Fill:	130	pcf
Assumed Average Unit Weight of *In-Situ* Soil:	110	pcf

Boring ID	Existing grade (ft)	Finished grade (ft)	Elev. Top of ESU 2 (ft)	Elev. Bottom of ESU2 (ft)	σ'_{vo} @ midpoint of ESU 2 (psf)	$\Delta\sigma$ (psf)	Estimated Primary Consol. Settlement (in.)
DOF-B1	17.5	22	-2.5	-5.5	1491.4	585	0.9
DOF-B2	14.5	22	1.5	-1	959.1	975	1.6
DOF-B3	14	22	3.5	-5	951.7	1040	5.6
DOF-B4	14	22	3.5	-5	951.7	1040	5.6
DOF-B5	14	22	-2	-11.5	1237.3	1040	5.1
DOF-B6	14	22	2	-6	1011.2	1040	5.0
DOF-B7	14	22	2	-6.5	1023.1	1040	5.3
DOF-B8	14.5	22	2.5	-5.5	1042.4	975	4.7
DOF-B9	14.5	22	-4.5	-5.5	1209	975	0.5
DOF-B10	16.5	22	-3	-5	1381.4	715	0.7
DOF-B11	16.5	22	4	-1	1119.6	715	2.2
DOF-B12	15.5	22	0	-4	1176.2	845	1.9
DOF-B13	15	22	3.5	-3.5	1026	910	3.9
DOF-B14	15	22	3.5	-4	1037.9	910	4.2
DOF-B15	18.5	22	3.5	-6	1470.5	455	2.3
Averages			1.16666667	-5			3.3

Notes:

ESU 2 = Upper Aquitard
σ'vo = initial vertical effective stress
Δσ = change in vertical effective stress

Abbreviations and Acronyms:

consol = consolidation
elev. = elevation
ESU = engineering stratigraphic unit
ft = feet
ID = identification
in = inches
pcf = pounds per cubic foot
psf = pounds per square foot

DRAFT

Table 2.
Seismic Response Spectra
Former Arkema Manufacturing Site
Tacoma, Washington

DE (2% in 50 year)	
T (s)	Sa,d (g)
0	0.30912
0.1	0.533976867
0.2	0.758833735
0.20621118	0.7728
0.3	0.7728
0.4	0.7728
0.5	0.7728
0.6	0.7728
0.7	0.7728
0.8	0.7728
0.9	0.7728
1	0.7728
1.031055901	0.7728
1.1	0.724363636
1.2	0.664
1.3	0.612923077
1.4	0.569142857
1.5	0.5312
1.6	0.498
1.7	0.468705882
1.8	0.442666667
1.9	0.419368421
2	0.3984
2.1	0.379428571
2.2	0.362181818
2.3	0.346434783
2.4	0.332
2.5	0.31872
2.6	0.306461538
2.7	0.295111111
2.8	0.284571429
2.9	0.274758621
3	0.2656
3.1	0.257032258
3.2	0.249
3.3	0.241454545
3.4	0.234352941
3.5	0.227657143
3.6	0.221333333
3.7	0.215351351
3.8	0.209684211
3.9	0.204307692
4	0.1992

CLE (10% in 50 year)	
T (s)	Sa,d (g)
0	0.1668
0.1	0.290770295
0.2	0.414740589
0.201822542	0.417
0.3	0.417
0.4	0.417
0.5	0.417
0.6	0.417
0.7	0.417
0.8	0.417
0.9	0.417
1	0.417
1.00911271	0.417
1.1	0.382545455
1.2	0.350666667
1.3	0.323692308
1.4	0.300571429
1.5	0.280533333
1.6	0.263
1.7	0.247529412
1.8	0.233777778
1.9	0.221473684
2	0.2104
2.1	0.200380952
2.2	0.191272727
2.3	0.182956522
2.4	0.175333333
2.5	0.16832
2.6	0.161846154
2.7	0.155851852
2.8	0.150285714
2.9	0.145103448
3	0.140266667
3.1	0.135741935
3.2	0.1315
3.3	0.127515152
3.4	0.123764706
3.5	0.120228571
3.6	0.116888889
3.7	0.11372973
3.8	0.110736842
3.9	0.107897436
4	0.1052

OLE (50% in 50 year)	
T (s)	Sa,d (g)
0	0.06072
0.1	0.110959622
0.181291173	0.1518
0.2	0.1518
0.3	0.1518
0.4	0.1518
0.5	0.1518
0.6	0.1518
0.7	0.1518
0.8	0.1518
0.9	0.1518
0.906455863	0.1518
1	0.1376
1.1	0.125090909
1.2	0.114666667
1.3	0.105846154
1.4	0.098285714
1.5	0.091733333
1.6	0.086
1.7	0.080941176
1.8	0.076444444
1.9	0.072421053
2	0.0688
2.1	0.06552381
2.2	0.062545455
2.3	0.059826087
2.4	0.057333333
2.5	0.05504
2.6	0.052923077
2.7	0.050962963
2.8	0.049142857
2.9	0.047448276
3	0.045866667
3.1	0.044387097
3.2	0.043
3.3	0.04169697
3.4	0.040470588
3.5	0.039314286
3.6	0.038222222
3.7	0.037189189
3.8	0.036210526
3.9	0.035282051
4	0.0344

DRAFT

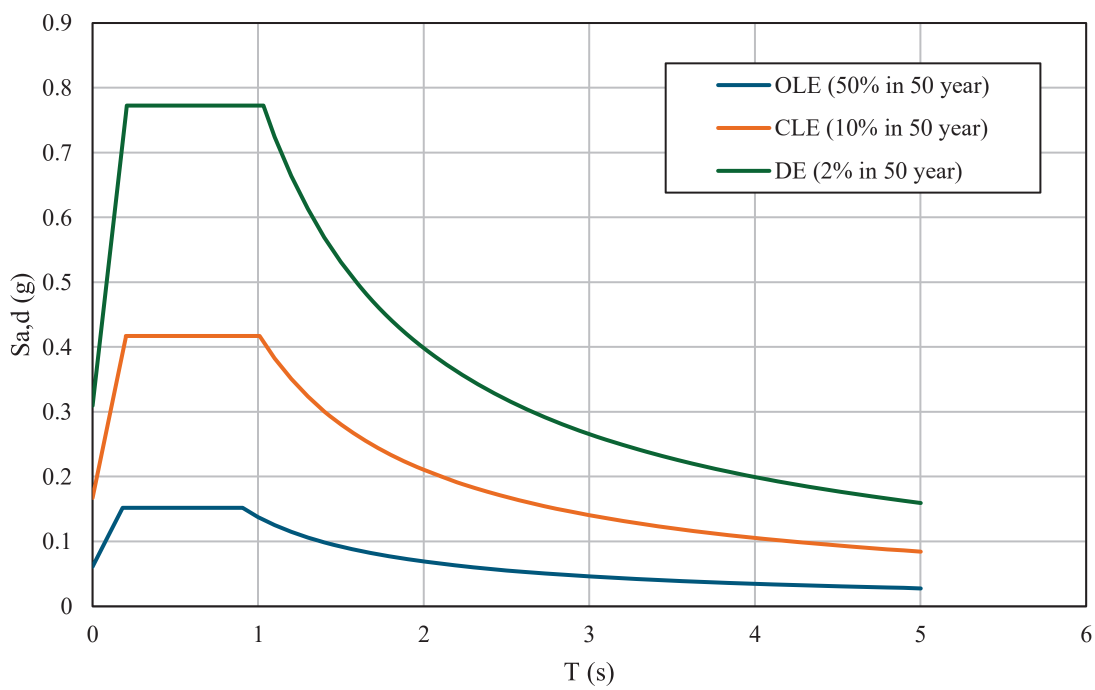
Table 2.
Seismic Response Spectra
Former Arkema Manufacturing Site
Tacoma, Washington

DE (2% in 50 year)	
T (s)	**Sa,d (g)**
4.1	0.194341463
4.2	0.189714286
4.3	0.185302326
4.4	0.181090909
4.5	0.177066667
4.6	0.173217391
4.7	0.169531915
4.8	0.166
4.9	0.162612245
5	0.15936

CLE (10% in 50 year)	
T (s)	**Sa,d (g)**
4.1	0.102634146
4.2	0.100190476
4.3	0.097860465
4.4	0.095636364
4.5	0.093511111
4.6	0.091478261
4.7	0.089531915
4.8	0.087666667
4.9	0.085877551
5	0.08416

OLE (50% in 50 year)	
T (s)	**Sa,d (g)**
4.1	0.033560976
4.2	0.032761905
4.3	0.032
4.4	0.031272727
4.5	0.030577778
4.6	0.029913043
4.7	0.029276596
4.8	0.028666667
4.9	0.028081633
5	0.02752

Abbreviations and Acronyms:
CLE = contingency level earthquake
DE = design earthquake
OLE = operational level earthquake
Sa,d (g) = design spectral acceleration at T
T (s) = period in seconds



DRAFT

Table 3.
Estimated Liquefaction Settlement
Former Arkema Manufacturing Site
Tacoma, Washington

Boring Name	Estimated Settlement (in.)		
	DE	CLE	OLE
RRI-B-118(B)	8	6	< 0.5
RRI-B-119(B)	11.5	8	< 0.5
RRI-B-120(B)	12	8.5	< 0.5
RRI-B-13(S)	20	18	< 0.5
RRI-B-14(S)	18	16	< 0.5
DOF-B10	16	12.5	< 0.5
DOF-B11	18	16	< 0.5
DOF-B12	17	12	< 0.5
DOF-B13	15	13	< 0.5
DOF-B14	14	8	< 0.5
DOF-B15	14	11.5	< 0.5
DOF-B1	19	17	< 0.5
DOF-B2	15.5	12.5	< 0.5
DOF-B3	17	15	< 0.5
DOF-B4	19.5	19.5	< 0.5
DOF-B5	21	19	< 0.5
DOF-B6	14.5	12	< 0.5
DOF-B7	15	12	< 0.5
DOF-B8	18	15	< 0.5
DOF-B9	15.5	13.5	< 0.5
Average	**16**	**13**	**N/A**
Min	**8**	**6**	**N/A**
Max	**21**	**19.5**	**N/A**

Abbreviations and Acronyms:

DE = design earthquake
CLE = contingency level earthquake
in = inches
max = maximum
min = minimum
N/A = not applicable
OLE = operational level earthquake

ATTACHMENT 1

Site Development Recommendations

ATTACHMENT 1.
SITE DEVELOPMENT RECOMMENDATIONS

Conditions Review Summary

The site will be filled to an elevation of approximately 22 feet (ft), or to 2 to 8 ft above the existing grade at the proposed vibrating beam containment wall location. Gravel Borrow will be used to raise site grades. The pavement sections shown on the current design drawings (i.e., 8 inches of hot-mix asphalt over 16 inches of Crushed Surfacing Base Course) will be supported by the Gravel Borrow.

Filling will be completed in phases over 10 years or fewer.

The site may experience up to 6 inches of consolidation settlement; settlement is likely to occur in the 2 months following filling. Please note that Sage's settlement estimates are likely conservative.

Granular fill soils (sand, silty sand, and Gravel Borrow) will be encountered in most of the proposed earthwork and utility excavations. Fine-grained soils are likely to be encountered in excavations that extend below elevation 8 ft; this likelihood increases with depth.

The site has a high water table that can fluctuate with season and tide.

Conclusions Summary

Neither soil preloading nor ground improvement is necessary to mitigate consolidation settlement associated with the proposed regrading activities/soil cap placement. These measures are not warranted because the estimated settlement period is short; the compressible soil layers are thin (and will be overlain by 10 to 15 ft of granular soil); and the site has been developed (i.e., pre-loaded) previously.

Sage should be contacted if development plans change to include buildings or defined container-stacking areas, as soil preloading could be advantageous to both.

Before pavement sections are constructed, mass grading (with Gravel Borrow) should be placed 3 inches high, then cut or topped off after a 1- to 2-month settlement period.

Sage recommends that settlement-sensitive wet gravity utilities are installed after filling. If this is not feasible, the design team may consider updating the project plans so the first phase of filling includes fill placement above wet utility corridors. Additionally, flexible pipe connections should be considered for settlement-sensitive utilities.

Given the groundwater conditions previously documented at the site, Sage recommends that underground utilities and buried structures (e.g., vaults, manholes, etc.) are designed to withstand buoyancy. The water table should be assumed to be coincident with the finished ground surface.

Recommendations

Earthwork

Subgrade Preparation

After it has been cleared and stripped, the exposed subgrade should be compacted with a large, vibratory roller. Before structural fill is placed, the prepared subgrade should be proof rolled in the presence of a qualified civil or geotechnical engineer, who is familiar with the site and can check for soft/disturbed areas. Areas of limited access can be evaluated with a steel T-probe. If probing or proof-rolling reveals loose and/or disturbed soils, the upper 1 ft of subgrade should be scarified; moisture-conditioned; and compacted to a firm, unyielding condition. Alternatively, unsuitable subgrade soils may be overexcavated and replaced with compacted structural fill, as directed by the engineer.

Wet Weather Earthwork

Moisture-sensitive, near-surface soils may contain up to 20 percent fines. These soils can become unstable if earthwork construction is completed during the wet season (i.e., October through April). Bids for site-grading operations should be based on the time of year construction will occur.

Wet weather earthwork will necessitate soil overexcavation or stabilization. Stabilization may be achieved with quarry spall working surfaces, covered soil stockpiles, and runoff collection and controls. The severity of construction disturbance will depend, in part, on the contractor’s efforts to protect moisture-sensitive site soils.

If wet weather construction is anticipated, the fines content of the Gravel Borrow should be less than 5 percent by weight of the fraction passing the ¾-inch sieve. This requirement should be incorporated into the project plans and specifications and clearly stated on the bid documents.

Structural Fill

Gravel Borrow, as described in Section 9-03.14(1) of the Washington State Department of Transportation’s 2024 *Standard Specifications for Road, Bridge, and Municipal Construction* (hereinafter, *2024 WSDOT Standard Specifications*), is a suitable source of import structural fill. During periods of wet weather, the fines content should not exceed 5 percent, based on the minus ¾-inch fraction.

Structural fill should be placed and compacted in accordance with the requirements in Section 2-03.3(14)C, Method C of the *2024 WSDOT Standard Specifications.*

Each layer of structural fill should be compacted to at least 95 percent of the maximum dry density, determined in accordance with the compaction control tests in Section 2-03.3(14)D of the *2024 WSDOT Standard Specifications.* Alternatively, the maximum dry density can be determined using ASTM International standard test method D1557, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3)).*

Temporary Excavations and Shoring

Utility excavations may extend below the water table. The excavations should be shored or laid back and dewatered before backfill or piping is placed. Trench boxes or sheet systems are feasible shoring methods.

The soil parameters in Table 1 can be used as a guide for preliminary design of temporary shoring systems. The contractor should be responsible for selecting the final shoring design parameters. Shoring design shall be prepared by a civil or structural engineer licensed in the State of Washington and reviewed by the owner's engineer prior to acceptance.

Table 1. Idealized Soil Parameters for Design of Temporary Shoring

Soil Unit	Typical Elevation Range (ft)	Moist Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (degrees)
New Fill	17–22	135	0	36
Existing Fill	8–17	120	0	32
Silt	<8	105	0	28

Note: Hydrostatic pressure should be added where appropriate.
ft = feet
pcf = pounds per cubic foot
psf = pounds per square foot

The contractor should be responsible for shoring and excavation stability; shoring and excavation should comply with the safety standards in Chapter 296-155 of the Washington Administrative Code (WAC). The site is underlain by Type C soils (WAC 296-155-664), and temporary slopes should be no steeper than 1.5 horizontal to 1 vertical. Flatter slopes may be required below groundwater. The contractor should be responsible for maintaining the stability of temporary slopes.

Dewatering

Dewatering should be used to lower water levels to 2 ft below the base of construction excavations. The contractor should select, design, and implement the dewatering system.

Underground Utilities and Utility Structures

Pipe Foundation Support, Pipe Zone Bedding, and Trench Backfill

The utility details shown on the current project drawings are consistent with Sage's geotechnical recommendations. Utility and utility structure excavations that extend below elevation 8 ft are more likely than shallower excavations to require subgrade improvements to amend soft soil conditions. To account for potential soft soil conditions, Sage recommends that a new detail for deep trenches is created or that Detail C2.1-8 is amended to include the following:

Where directed by the engineer, subgrade improvements (along utility trench bottoms) should consist of Quarry Spalls that conform to the requirements in Section 9-13.1(5) of the 2024 WSDOT Standard Specifications. *An excavator bucket should be used to push the Quarry Spalls into the soft subgrade. Vibratory compaction should be avoided. Class A Foundation Material should be placed between the Quarry Spalls and the overlying pipe bedding. The Class A Foundation material should conform to the requirements in Section 9-03.17 of the* 2024 WSDOT Standard Specifications.

Lateral Earth Pressures

Sage recommends assuming a design groundwater elevation equal to the finished ground surface. Lateral earth pressures acting on the sides of below-grade structures can be computed using an equivalent fluid density of 82 pounds per cubic foot (pcf) for active conditions and 95 pcf for at-rest conditions. The active and at-rest pressures include hydrostatic pressure.

The design of subsurface walls should account for lateral pressures exerted by adjacent surcharge loads. For uniform surcharge pressures, uniformly distributed lateral pressures of 0.44 times the surcharge pressure should be added for non-yielding walls.

Buoyancy Considerations

Buried structures should be evaluated for global uplift and structural capacity. When developing buoyancy calculations, Sage recommends assuming that the water table could rise to ground surface.

Buoyant forces can be resisted by skin friction along the edges of the structure (Rs), by the weight of soil over footings (Rw), and by the weight of the structure itself. Sage should be contacted if additional resistance is required. Buoyancy can be computed in accordance with Diagram 1 and the equations that follow.

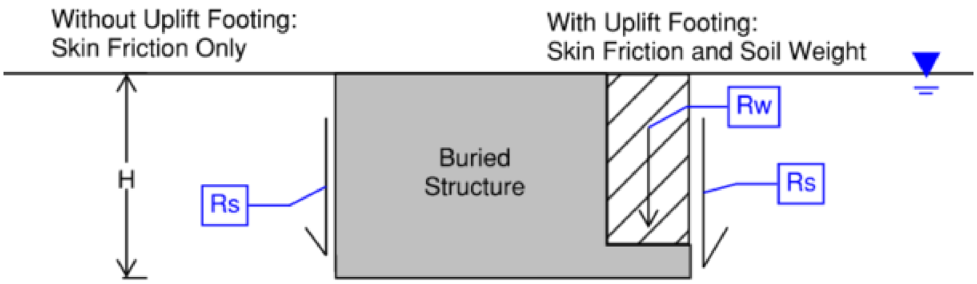


Diagram 1. Buried structure uplift resistance.

With:

$$Rs = 2.2H^2\ lbs\ per\ foot\ perimeter.$$

$$Rw = 73\ pcf * H * (uplift\ footing\ area).$$

Buoyancy may be a concern for buried utility pipes if the water table rises above the crown of the pipe when the pipe is not full. The weight of the pipe and the overlying soil, W_s (pounds), will provide resistance to buoyant forces. If the water table rises to ground surface, the weight of the soil along a 1-ft length of pipe can be interpreted as:

$W_s = \gamma * H * D$ (McGrath 1998),

where:

γ = buoyant unit weight of trench backfill (73 pcf).

H = distance from the ground surface to the crown of the pipe in feet.

D = diameter of the pipe in feet.

Light Pole Foundations

Light pole foundations should be designed in accordance with the manufacturer's recommendations. Sage recommends placing at least 6 inches of compacted crushed gravel or controlled density fill beneath the foundations.

Sage recommends an allowable soil bearing pressure (vertical) of 1,000 pounds per square foot (psf) for foundations established below elevation 10 ft, and 2,000 psf for foundations established above elevation 10 ft. This allowable soil bearing pressure applies to long-term dead and live loads, exclusive of the weight of the footing and any overlying backfill.

Sage recommends an allowable lateral bearing resistance of 1,000 psf below elevation 8 ft, and 2,500 psf above elevation 8 ft. Lateral bearing resistances were developed in accordance with Section 17.2.1 of the 2023 *Geotechnical Design Manual* (WSDOT 2022).

Pile Foundations (Stormwater Outfall)

Driven pile foundations for stormwater outfall structures are shown on Sheet C2.12. Based on conversations with Sitts & Hill Engineers, Inc. (project structural engineer), Sage understands that current project plans include 12-inch-diameter pipe piles with a required (allowable) capacity of 100 kips.

Sage analyzed axial pile capacities and reviewed subsurface conditions pertinent to pile design and installation. Based on its findings, Sage recommends that the following information is incorporated into the project plans and specifications:

- Pipe piles shall be open-ended and 12 inches in diameter with a nominal (ultimate) capacity of 180 kips.
- Pile driving shall be completed in accordance with Section 6-05 of the *2024 WSDOT Standard Specifications.*
- North outfall – Minimum tip elevation: -35 ft, estimated tip elevation: -70 ft, overdriving allowance: 0 kips.
- South outfall: Minimum tip elevation: -35 ft, estimated tip elevation: -60 ft, overdriving allowance: 0 kips.

References

ASTM. 2024. *Volume 04.08: Soil and Rock (I): D420–D5876/D5876m.* ASTM International.

LNI. 2020. Construction Work. Chapter 296-155 WAC; Part N. Excavation, Trenching, and Shoring. Washington State Department of Labor and Industries. Effective October.

McGrath, T.J. 1998. "Calculating Loads on Buried Culverts Based on Pipe Hoop Stiffness." Presented at the 1999 TRB Annual Meeting in Washington, D.C.

WSDOT. 2023. *M41-10: Standard Specifications for Road, Bridge, and Municipal Construction.* 2024 Edition. Washington State Department of Transportation.

WSDOT. 2022. *M46-03: Geotechnical Design Manual.* Washington State Department of Transportation. February 10.