December 12, 2019

Mr. Douglas Luetjen
BSRE Point Wells, LP
c/o Karr Tuttle Campbell
701 Fifth Avenue, Suite 3300
Seattle, Washington 98177

Re: Subsurface Conditions Report Addendum - Revised
Point Wells Redevelopment
Unincorporated Snohomish County, Washington
17203-57

Dear Mr. Luetjen:

In this letter, we provide additional geotechnical information to address items in Snohomish County's May 9, 2018 Supplemental Staff Recommendation document, the County's May 9, 2018 landslide hazard area memorandum (from Randolph Sleight), and the County's June 1, 2018 Findings of Fact/Conclusions of Law document for the hearing examiner. We clarify project geotechnical information provided in the Subsurface Conditions Report (Hart Crowser 2018) and provide supplemental geotechnical information for the Point Wells Redevelopment (Project) in unincorporated Snohomish County, Washington. This letter is an addendum to our April 20, 2018 Subsurface Conditions geotechnical report. Subsequent sections are organized using the general headings from the County's May 9, 2018 Staff Recommendation document.

1. Feasibility and Code Compliance of Second Access Road

Subitem (2) claims the 2018 geotechnical report lacks sufficient geotechnical analysis to demonstrate compliance with Snohomish County Code (SCC) 30.62B.140(1)(b) and refers to Item 8 for more details on substantial conflicts with code compliance.

8. Code Provisions Regarding Geologically Hazardous Areas

Geologically Hazardous Areas

Landslide Hazard Areas Deviation Request

Our revised landslide hazard area (LHA) deviation request letter (December 6, 2019) discusses specific County deviation requirements. The sections below discuss the geotechnical items related to this request. The intent of the LHA deviation request letter is to determine if the deviation requests are approvable by the County once the final design is completed following the general slope stabilization approach suggested. If these deviation requests are not approvable at this time, the letter requests the opportunity to discuss with the County what specific additional final design items would be needed to receive approval.

Secondary Access Road Location Alternatives

We understand the Secondary Access Road location is required to be different than the existing site southern access via Richmond Beach Drive, which leaves access routes to the northeast and southeast as possible options. Our August 2016 report, Appendix E (Hart Crowser 2016), shows access routes considered to the northeast (Abandoned Access Road) and southwest (current Secondary Access Road). Both locations are in landslide hazard areas. The northeastern route requires more grading in wet areas and the Abandoned Access Road is displaced in places, which suggests less stable conditions (Figure 5, Hart Crowser 2018). The current (revised December 2019) southeast Secondary Access Road location shown on Plan A-051 and in the geotechnical report (Figures 5 and 10, Hart Crowser 2018) encounters fewer geologic critical areas, especially landslide hazard areas, than the northeast location. The southeast location is also in an area that has shorter and flatter average slopes (Figure 4, Sections E, F, and G, Hart Crowser 2018). Thus, based on these factors, the southeastern access route option is more suitable than the northeast route. However, final design will need to follow final geotechnical design recommendations for subgrade preparation, drainage, and stabilization measures, as well as meeting County requirements.

Secondary Access Road Retaining Wall Improves Slope Stability

The proposed retaining wall for the Secondary Access Road would improve slope stability above current conditions to satisfy the required factors of safety (FS) in SCC 30.62B.340(3)(b), as discussed in Sections 5.1.6.1 and 7.1.1 of the geotechnical report (Hart Crowser 2018), with one exception discussed below. In summary, FS for current conditions are below values in SCC 30.62B.340(3)(b) (see Table 3a below) but would be increased to meet the SCC requirements by installing a permanent retaining wall, except for one seismic case (Table 3a, Figure 23a). The seismic case has one small area where the proposed seismic condition is just below (1.04) the SCC target seismic FS (1.10). Note that certain public agencies have target seismic FS values of 1.05, or do not require seismic FS values (Washington State Department of Transportation [WSDOT] Geotechnical Design Manual [GDM], Sections 7.4 and 6.1.2.1) for slope instability that would not cause collapse of adjacent structures or is not adjacent to critical structures (e.g., bridges, etc.). Mitigation options for this small area are discussed later in this section.

Table 3a – Summary of Slope Stability Factors of Safety

Section		Scenario (Piezometric Surface Estimated from Vibrating Wire Piezometers [VWP])	Factor of Safety (FS)	
			Static	Pseudostatic
G-G'	East Slope	Existing Conditions – Shallow/Critical Slip Surface	1.22	0.87
		East Retaining Wall Force Calculation	NA	1.10
		Proposed Wall + Backfill – Shallow/Critical Slip Surface (95.1 kips per foot shoring force, 10 feet minimum depth/target FS at 11_feet minimum depth)	1.51	1.04/1.10
		Same as above with Rain on Snow Surcharge	1.54	NA
		Proposed Wall + Backfill – Medium Slip Surface (95.1 kips per foot shoring force, >15 feet minimum depth)	2.00	1.21
		Proposed Wall + Backfill – Buttress Option (95.1 kips per foot shoring force)	–	1.11
	Near BNSF	Existing Conditions – Critical Slip Surface	3.59	1.44
		West Retaining Wall Force Calculation	NA	1.15
		Proposed Wall + Backfill – Critical Slip Surface (19 kips per foot shoring force)	3.20	1.62

Notes:

a. County minimum FS for development in landslide areas are 1.5 for static and 1.1 for seismic cases, per SCC 30.62B.340(3)(b).

b. FS values valid to one decimal; but, reported to two decimals for comparison purposes.

c. Figures are included for shaded cells. Blue font represents one case not meeting SCC target FS value east of the site in a small area (see Figure 23a and 23d).

d. Results use non-circular, auto-refine search methods. Minimum, or critical slip surface FS are shown in green font/green slip surface on attached figures.

The following items clarify how the stability analysis for the retaining wall demonstrates it is feasible to achieve the required FS in SCC 30.62B.340(3)(b).

- The permanent retained height of the retaining wall (Figures 22, 22a, 22b, 23, 23a through 23d; [a, b, etc.] designate new, updated figures attached) is about 40 feet above final grades. The lowermost 20 feet below grade would temporarily support building basement wall lateral earth pressures until building basement floor slabs and walls are complete, depending on sequencing. Once complete, building walls and slabs would transfer lateral earth loads on the east side of the basement to soil on the opposite, or west, side of the building. The number of rows of tiebacks can be adjusted to include the lower 20 feet of wall at different times to accommodate different building phasing.

- Geotechnical slope stability analysis/calculation results on Figures 22a, 22b, 23a through 23d show how a generic retaining wall providing a resisting force of 77 kips (kip = 1,000 pounds) per foot of wall length increases FS to the County code-required values. Several retaining wall options could be used. The critical seismic case on Figures 23b and 23c demonstrate with calculations how a permanent soldier pile and tieback retaining wall system is feasible to provide these resisting forces (including soldier pile and tieback geometry and loads). Note the following specifics about the stability analysis.

    - Section 5.1.6.1 of our geotechnical report (Hart Crowser 2018, page 23) discusses how a high strength (i.e., a cohesion of 10,000 pounds per square foot [psf]) was used in the stability analysis (results in Figures 20 – 25) to represent the retaining wall (typically steel and concrete). Later structural design would be done so the wall is structurally strong enough so slip surfaces do not go through, but rather under it.

    - A high cohesion (10,000 psf) was not used for subgrade or retained soil, as noted in the text and on the stability figures.

    - Our slope stability analyses/calculations were completed using commercially available limit-equilibrium software (SLIDE 2, by Rocscience) that is widely accepted and used by many geotechnical engineers, as noted in our geotechnical report (Hart Crowser 2018). Prior stability analysis results (2018) used a minimum slip surface depth of 30 feet to show more significant critical slip surfaces affecting more of the east slope. Current (2019) stability results used a minimum slip surface depth of 10 feet so show that critical/minimum slip surfaces are generally small, shallow surfaces that are associated with the steepest grades.

    - The permanent retaining wall resisting force of 95.1 kips per foot of wall is lower than loads used on other of our slope stabilization projects (170 to 190 kips per foot of wall). Thus, if during final design, some additional load resistance is required, additional capacity can be provided, which also supports the feasibility of the proposed slope stabilization method.

- The horizontal force required to retain the Secondary Access Road was calculated using an iterative analysis method for the critical pseudostatic slope stability load case. The same profile, soil properties, and conditions were used in the calculation. In this method, a horizontal force was applied at the mid-point of the retained section to represent the resultant force applied by multiple rows of tiebacks. This force was increased until the critical slip surface reached the SCC required seismic FS of 1.10 or greater. Figures 23b and 23c (attached) show the critical slip surfaces, which are the general shape of an active wedge in the lateral earth pressure analysis. By stabilizing this area of the slope (i.e. achieving a FS of 1.1 against failure within the roadway embankment) with a retaining force, the critical slip surfaces in the east slope now occur upslope in the location shown in Figures 22a, 22b, 23a, and 23d, rather than through the retained backfill for the Secondary Access

Road. The horizontal force is not directly providing a stabilizing force to the overall critical slip surface in the slope (Figure 23 and 23a, FS = 1.109 and 1.04 to 1.21). Instead, as discussed in Section 5.1.6.1 of our geotechnical report (Hart Crowser, 2018), the stabilized/retained section of the Secondary Access Road acts as a buttress for the east slope to improve the overall stability over existing conditions. This result is demonstrated in our geotechnical report by the FS increasing from the existing conditions (Figures 18a and 19a, below code minimum FS, Table 3a) to the retaining wall with backfill option (Figures 22a, 23a, and 23d, near or above code minimum FS). This is also demonstrated in Table 3a. Note the following specifics about the stability analysis.

- The stability results figures attached include a 250 psf traffic surcharge on the road.
- Stability results on Figures 22a through 23d include permanent basement wall drainage (see Drainage Plan in Geotechnical Report section).
- The stability results on Figure 22b include a rain on snow surcharge of 95 psf (equivalent to 1.5 feet of water). This is included for the static case only since it is an extreme event that is unreasonable to combine with the seismic extreme event.
- Figure 23d shows one feasible slope remedy by simply adding a small soil buttress/berm at the base of the East slope to increase the FS from 1.04 to greater than the code-required seismic FS of 1.10.

■ Figures 22a and 23a include excavation west of the railroad to an elevation of +6 feet, showing FS above the code-required values. This excavation would be temporary for either removal of contaminated soil or construction of building basements and would likely include sheetpile shoring. Final grades just west of the railroad will be raised to about an elevation of 50 feet, which will act as a resisting force to potential global instability extending from the east slopes under the railroad, which is unlikely. These figures only show the two minimum/critical slip surfaces above the proposed retaining wall and west of the railroad. However, the analysis included larger surfaces starting above the retaining wall and extending under both the retaining wall and the railroad; but, the safety factors were well above the code-required values.

■ Perched groundwater was encountered in the five VWPs installed in three borings for the Secondary Access Road, as noted in Table 2 of our report. As noted in Section 5.1.6.1 (Section G-G' subsection, pages 22 to 23), perched groundwater was encountered at different elevations in the VWPs in the sand layers within the Lawton silt/clay layer. However, the stability analysis uses a conservative groundwater assumption that all soil below the highest perched groundwater elevation is saturated. Based on this conservative groundwater assumption, stability analysis shows that groundwater drainage control is not required, up the slope where the slip surfaces exist (Figures 22a and 23a), to achieve the required FS for the Secondary Access Road. We are currently recording water levels in these VWPs for use in future stability analysis, but the groundwater elevations have not changed significantly, as shown in Table 2a below. See Drainage Plan in Geotechnical Report section later in this letter for discussion about building basement permanent drainage.

- Landslide runout does not have a broadly accepted standard of practice calculation method, nor methods for how it is applied in conjunction with slope stability analysis. In our opinion, the existing landslide runout records are suitable to be used for reference, but should be used with caution for design purposes. Site slopes range from about 40 percent near Section B to 20 percent near Section G, which are much less than the estimated pre-slide slopes of the Woodway landslide (70 percent). Thus, in our opinion, a Woodway-type slide runout is highly unlikely east of this project. From the runout studies we found, estimated runout distances for the 50th to 90th percentile slides were between about 200 to 300 feet, respectively, from the head scarp of landslides. If these rough estimated runout distances start from the head scarp of slip surfaces estimated in our slope stability analysis, the runout may not reach the base of the slope near the Secondary Access Road and Upper Plaza buildings. However, the shallow 20 percent slopes at Section G are likely closer to the lower end of the runout distances in the studies we reviewed (see small, shallow critical [lowest FS] slides on Figure 23a). Additional measures that can be considered during final design to address the potential for runout from shallow slides above the wall that may reach the base of the slope include: a) one wall on either side of the Secondary Access Road (i.e., Figures 24 and 25 of the 2018 geotechnical report), b) increasing the height of retaining walls to extend above grade and designing them to contain slide runout from shallow slides starting higher upslope, c) adding a retaining wall on the upslope side of the secondary access road (Screen Wall added to Plan Sheet C-300) to contain slide runout, and/or d) designing the east side of buildings to have walls to withstand/retain slide runout for some height above final grades (e.g., reinforced concrete without windows or doors).

Table 2a – Vibrating Wire Piezometer Water Level Measurements

Boring ID	Approx. Ground Surface Elevation in Feet	VWP Elevation in Feet[a]	Date	Measured Head in Feet	Groundwater Depth in Feet	Groundwater Elevation in Feet
HC-1[a]	243	229	May 6, 2015	7.6	6.4	236.6
			May 21, 2015	6.9	7.1	235.9
			May 26, 2015	6.9	7.1	235.9
		184	May 6, 2015	39.0	19.8	223.2
			May 21, 2015	40.0	18.7	224.3
			May 26, 2015	40.5	18.3	224.7
		129	May 6, 2015	55.3	58.7	184.3
			May 21, 2015	57.2	56.8	186.2
			May 26, 2015	58.0	56.0	187.0
		89	May 6, 2015	38.4	115.6	127.4
			May 21, 2015	38.2	115.8	127.2
			May 26, 2015	38.4	115.6	127.4
HC-10[b]	180	151	March 23, 2018	16.8	12.6	167.4
			to April 20, 2018	16.4 – 17.9	-	167 – 168.5
		121	March 23, 2018	50.5	9.0	171.0
			to April 20, 2018	50 – 51.3	-	170.5 – 171.8
		91	March 23, 2018	65.2	24.2	155.8
			to April 20, 2018	65.1 – 66.4	-	155.7 – 157
HC-11[b]	142	112	March 23, 2018	22.1	7.5	134.5
			to April 20, 2018	21.6 – 23.1	-	134 – 135.5
HC-12[b]	47	31	March 23, 2018	18.7	-2.2[c]	48.8
			to April 20, 2018	18.5	-2.1[c]	48.6

Notes:

a. HC-1 VWPs installed on April 22, 2015.

b. HC-10, -11, and -12 VWPs installed on February 22, 2018, February 26, 2018, and February 19, 2018 respectively.

c. Groundwater appears to be slightly above the ground surface due to either slight artesian conditions or VWP locations that shifted slightly during installation from their original elevations.

- A Sounder Station is planned to be located adjacent to the railroad under the northern overpass where the Secondary Access Road crosses over the railroad. This structure will have a retaining wall on its east side and provide access from the Urban Plaza down to the railroad for train transit. The retaining wall for this structure should have similar loads as the retaining wall for the Secondary

Access Road since the slopes above it are similar height and slope angle as Section G-G'. As a result, items in this section are also applicable to the Sounder Station.

The stability analysis/calculations and information in this section show that the proposed retaining wall just east of the Urban Plaza is feasible to achieve the code-required FS (with minor grading for the seismic case) and provide greater protection than standard landslide setbacks and existing slope stability with FS below code required values. Note that setbacks simply locate structures farther from potential landslides, but do not improve slope stability. The proposed retaining wall/slope stabilization would be (during final design), and has preliminarily been designed to increase slope stability to code-required FS (improved from existing conditions) for the proposed Secondary Access Road and other structures. Future final design work can be done to adjust slope stabilization measures to achieve the county code FS, which will be more stable than standard landslide setbacks and existing slope stability (see Table 3a).

Urban Plaza Building Location Alternatives and Retaining Wall

Location Alternatives. We understand from the project architect (Perkins+Will) that buildings in the Urban Plaza (including the Sounder Station) need to be located in the front part of the site because the multi-modal transportation center has to be located here by the railroad, existing entry road, and proposed Secondary Access Road; and for other reasons noted in Attachment 1 of our December 12, 2019 landslide hazard deviation request letter.

Secondary Access Road Retaining Wall Protects Urban Plaza Buildings. The large retaining wall downslope of the Secondary Access Road, the Screen Wall (upslope edge of secondary access road on Sheet C-300) and Service Access road below the secondary access (Sheet A-050) would protect the Urban Plaza Buildings in a similar manner. The slope stability information in the Secondary Access Road Retaining Wall section above is applicable for the Urban Plaza Buildings.

Geotechnical Report

Purpose and Scope. Our 2018 geotechnical report (Hart Crowser 2018), as well as 2015 and 2016 geotechnical reports referenced in our 2018 report, were developed to support preparation of an Environmental Impact Study (EIS) and address specific geotechnical engineering questions from EA Engineering and County Planning and Development Services (PDS), per Section 2.1 and 2.2 of our 2018 report. Additional geotechnical engineering was completed based on PDS comments in their October 6, 2017 review letter. Our geotechnical reports indicate that analyses and calculations are preliminary to support planning-level decisions and demonstrate feasibility of site development concepts, but that final design analyses are required. As noted in the County's May 9, 2018 Supplemental Staff Recommendation (page 22), "It is appropriate for an applicant to provide specific details regarding the design of structures at a later stage, such as the time of building permit review. However, at this stage in the permitting process, the applicant must demonstrate the *feasibility* of the structures."

Geotechnical Feasibility. In our opinion, as professional geotechnical engineers, our analyses and preliminary recommendations are adequate to demonstrate that the geotechnical engineering aspects of the proposed development (slope stability, foundation support in liquefiable soil, etc.) are feasible to design and construct as discussed in this letter and in our reports. We have indicated items that would require additional geotechnical investigation, analysis, and design recommendations during later final design stages of the project. Such items that we indicate can be completed later are less critical items that, in our professional opinion, are not needed to demonstrate the geotechnical feasibility of the project. The following list discusses and/or clarifies items PDS staff indicate are critical to determine the feasibility of geotechnical aspects of the project at this time.

- Our 2018 geotechnical report and this letter include slope stability analysis at locations, that, in our opinion, represent critical conditions for a location (i.e., combination of steep slope, high slope height, high groundwater, etc). We did this to determine existing slope stability and demonstrate slope stabilization is feasible where needed above the Secondary Access Road. Section G-G’ has steeper and higher slopes than other locations above the Secondary Access Road.

  Section B-B’ at the north part of the east slopes has steeper and higher slopes than other areas along the east slopes. Buildings west of these slopes, and west of the railroad, are beyond the landslide hazard area setback and west of the proposed grade separation wall on the west side of the railroad. The grade separation wall would essentially block landslide runout from reaching these buildings.

- Building basement excavations west of the railroad may encounter groundwater that may require temporary construction dewatering. Section A (Figure 7, Hart Crowser 2018) shows some borings with time of drilling groundwater levels a few feet above the proposed basement bottom elevation of 6 feet, but other groundwater levels have groundwater below an elevation of 6 feet. Dewatering only a few feet is not a critical item that would determine if excavation is feasible or not. Several methods may be used to lower the water level, if needed, including ditching and sump pumps, wells, or well points. These methods are routinely used and would be determined during a later design stage. At that time, potential impacts of dewatering on the railroad would be determined. If this is a concern, a sheet pile cutoff wall (or continuous secant piles or soil freezing) could be installed near the railroad such that dewatering would not detrimentally affect the railroad.

- **Drainage Plan.** Permanent drainage of the slope above the Urban Plaza is not necessary to stabilize the slope above the Secondary Access Road, based on slope stability results (attached Figures). However, surface water along the upslope edge of the Secondary Access Road would be collected (with typical subgrade drains consisting of drainage geotextile, drainage rock, and perforated pipes) and conveyed to the creek diversion structure, or other suitable drainage piping (Sheet C-300). Subsurface drainage adjacent to the upslope edge of the road and building basement retaining walls is needed to avoid the buildup of hydrostatic pressure on the retaining wall(s), as shown on Figure

2A (attached). The civil drainage plans (Sheet C-300) for the Upper Bench and access road show relocating the existing diversion structure further up Chevron Creek above the retaining wall. Both the existing and new diversion structures collect the creek into a pipe with an outfall into Puget Sound. Figure 2A shows one feasible drainage configuration for surface and subsurface drainage for the road, retaining wall, and building basement wall. The permanent basement wall drainage would likely include a typical wall drainage layer and perimeter perforated collection pipe with either gravity drainage in a solid wall pipe or a pump to convey water to the existing creek diversion pipe or other drainage piping. Final drainage configurations would be determined during final design.

- Section 6.2.2.1 of our 2018 geotechnical report (and our prior reports Hart Crowser 2005, 2016a, 2016b) indicates additional work needs to be done to provide geotechnical seismic design information for International Building Code (IBC)-based building structural design. However, sufficient information is available to determine the feasibility of building support in liquefiable soil. In our opinion, the site is suitable for development, provided that appropriate foundation support and/or ground improvement method(s) are used. Section 7.1.2 of our 2018 geotechnical report discusses several different methods that are feasible and likely to be used to support residential towers, including, but not limited to, ground improvement (e.g., stone columns, rammed aggregate piers, grouting, soil mixing, etc.), deep foundations (drilled shafts, augercast piles, driven piles, etc.), overexcavation and replacement with structural fill, and groundwater drainage. All these methods are commonly used in local practice for development in liquefaction-susceptible soil. Selection of a specific building support method would be done at a later design stage when structural load information is determined, and be based on geotechnical final design recommendations. One likely foundation support method would be to support buildings on deep foundations that transfer loads down to dense bearing soil below the liquefiable soil with ground improvement along the west part of the site to limit liquefaction and lateral spreading. We have used these methods on numerous recent local projects (see Local Projects section below).

- Liquefaction susceptibility on site was evaluated using the Idriss & Boulanger (2008) method for standard penetration test (SPT) blow counts from both historical and recent borings on site (Figure 26). For fine-grained soils, liquefaction susceptibility was further evaluated using the Bray & Sancio (2006) method, based on Atterberg limits and natural water contents of exploration samples (Figure 27). Fine-grained samples of higher plasticity and lower water contents are generally not expected to liquefy, and these characteristics were determined for the majority of samples that have Atterberg limits laboratory tests. Atterberg limits and water content information was used to classify the general geologic units (i.e. Lawton Clay, Glacial Outwash, Transitional Beds, etc.) as "non-susceptible". Figures 26 and 27 (new figures, attached) illustrate our findings on the liquefaction potential across the site: including the Lower Bench, the Upper Bench, and the East Slope.

  - **Slope:** Liquefaction potential in the slope east of the site is low (~5 percent of mid slope and ~4 percent of upper slope on site samples [5 or 112]). Some isolated, small pockets exist that may liquefy in the event of a design-level earthquake, although these are thin (generally 1- to 4-feet thick) and discontinuous within the slope (see Figure 9A). Some shallow areas of soil that exhibit liquefaction susceptibility do exist, but these are above the measured groundwater table, and thus will not liquefy. The groundwater table in the area of the Secondary Access Road has shown very minor fluctuation (less than a few feet in Table 2) for the duration of our measurements, suggesting saturation of these areas is unlikely.

- **Upper Bench:** The Upper Bench area, east of the railroad, shows potentially liquefiable soils within the top 20 feet in boring MW-122 at the western-most edge of the area and pockets of deeper susceptibility in E-101 (about 150 feet south of the site). Borings towards the east (HC-12) and north (MW-95), closer to the bottom of the slope, show high factors of safety against liquefaction. In the event of a design-level earthquake, the Upper Bench may experience liquefaction in western and southern portions of the area. The disparity between the two sides of the area is likely due to the transition between glacially deposited soils in the bluff to loose, granular shoreline deposits toward the Puget Sound, or loose fill to the west if a sidehill cut was used to create the bench. About 14 percent of samples in this site area are susceptible to liquefaction.

- **Lower Bench:** We performed liquefaction analysis on selected representative deeper widely distributed borings in the Lower Bench. This analysis shows a widespread potential for liquefaction (about 27 percent of samples) in the upper 50 feet. The shoreline deposits in this area are loose, granular, and saturated materials. Figure 26a shows no fine-grained samples in the borings analyzed from the Lower Bench.

- As noted in the Secondary Access Road section above and Section 5.1.6.1 of our 2018 geotechnical report, groundwater elevations were measured with several VW piezometers and included in slope stability analysis using conservative groundwater level assumptions. Thus, groundwater conditions in this area are reasonably well defined for this stage in a project.

- Site access includes bridges/overpasses over the railroad. These bridges could be supported by a variety of methods, including shallow foundations and retaining walls designed for static and seismic loads. This would likely require shallow foundations and retaining walls on ground improvement, as mentioned in Section 7.5.1 of our 2018 geotechnical report. Several deep foundation support options could also be used, such as drilled shafts, augercast piles, and/or driven piles, as discussed in Section 7.5.2 or our 2018 geotechnical report. The existing bridge over the railroad appears to be pile supported, indicating deep foundations are feasible. These methods are routinely used and would be determined during a later design stage, once structural loads are determined.

Local Projects with the Same Geotechnical Considerations

Several local projects have the same geologic hazards and used similar geotechnical analysis and design methods to address the geologic hazards. The following local projects have used the general methods discussed above and in our 2018 geotechnical report, which demonstrates the feasibility of the proposed geotechnical methods.

Amgen/Expedia Campus, Pier 88 and 89, Seattle, Washington

- Stone column ground improvement to prevent liquefaction induced lateral spreading.
- Deep foundation support in liquefiable soil.

Federal Center South, Duwamish Valley, Seattle, Washington

- Stone column ground improvement to prevent liquefaction-induced lateral spreading.
- Deep foundation support in liquefiable soil.

West Point Wastewater Treatment Plant, Discovery Park, Seattle, Washington

- Permanent soldier pile and tieback retaining wall in landslide area.
- Deep foundation support in liquefiable soil.

Puget Sound Bluff Estate, Shoreline, Washington

- Permanent soldier pile and tieback retaining wall (deep seated stabilization) and shallow ground anchor (shallow stabilization) in landslide area.

Issaquah Residential Campus, Issaquah, Washington

- Permanent drilled shaft, tieback, and anchor block retaining walls in landslide area.

Sound Transit Maintenance Building and Access Ramps, Duwamish Valley, Seattle, Washington

- Stone column ground improvement to prevent liquefaction near deep foundations.
- Stone column ground improvement to prevent liquefaction and support mechanically stabilized earth walls.
- Deep foundation support of structures in liquefiable soil.

West Seattle Bridge, Duwamish Valley, Seattle, Washington

- Stone column ground improvement to prevent liquefaction near deep foundations.
- Deep foundation support of structures in liquefiable soil.

We trust this letter provides the required information. Please let us know if you or others have any questions about the content of this letter.

Sincerely,

HART CROWSER, INC.

N. JOHN BINGHAM, PE
Senior Associate, Geotechnical Engineer

Attachments:

Figure 2a	Site and Conceptual Drainage Plan for Secondary Access Road
Figure 9a	Liquefaction and Slickenside Potential in Generalized Subsurface Cross Section G-G’
Figure 18a	Section G-G’ – East Slope Existing Conditions, Static
Figure 19a	Section G-G’ – East Slope Existing Conditions, Pseudostatic
Figure 22a	Section G-G' – Wall with Backfill, Static
Figure 22b	Section G-G' – Wall with Backfill, Static, Rain on Snow Surcharge
Figure 23a	Section G-G' – Wall with Backfill, Pseudostatic
Figure 23b	Section G-G’ – East Wall Retaining Force Calculation, Pseudostatic
Figure 23c	Section G-G’ – West Wall Retaining Force Calculation, Pseudostatic
Figure 23d	Section G-G' – Wall with Backfill, Buttress Option, Pseudostatic
Figure 26	Liquefaction Analysis of SPT Samples
Figure 27	Fine Grained Soils Liquefaction Susceptibility

References

Bray, J. D., & Sancio, R. B. (2006). Assessment of the liquefaction susceptibility of fine-grained soils. Journal of Geotechnical and Geoenvironmental Engineering, 132(9), 1165-1177.

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Idriss, I. M., & Boulanger, R. W. (2008). Soil Liquefaction During Earthquakes. Earthquake Engineering Research Institute.

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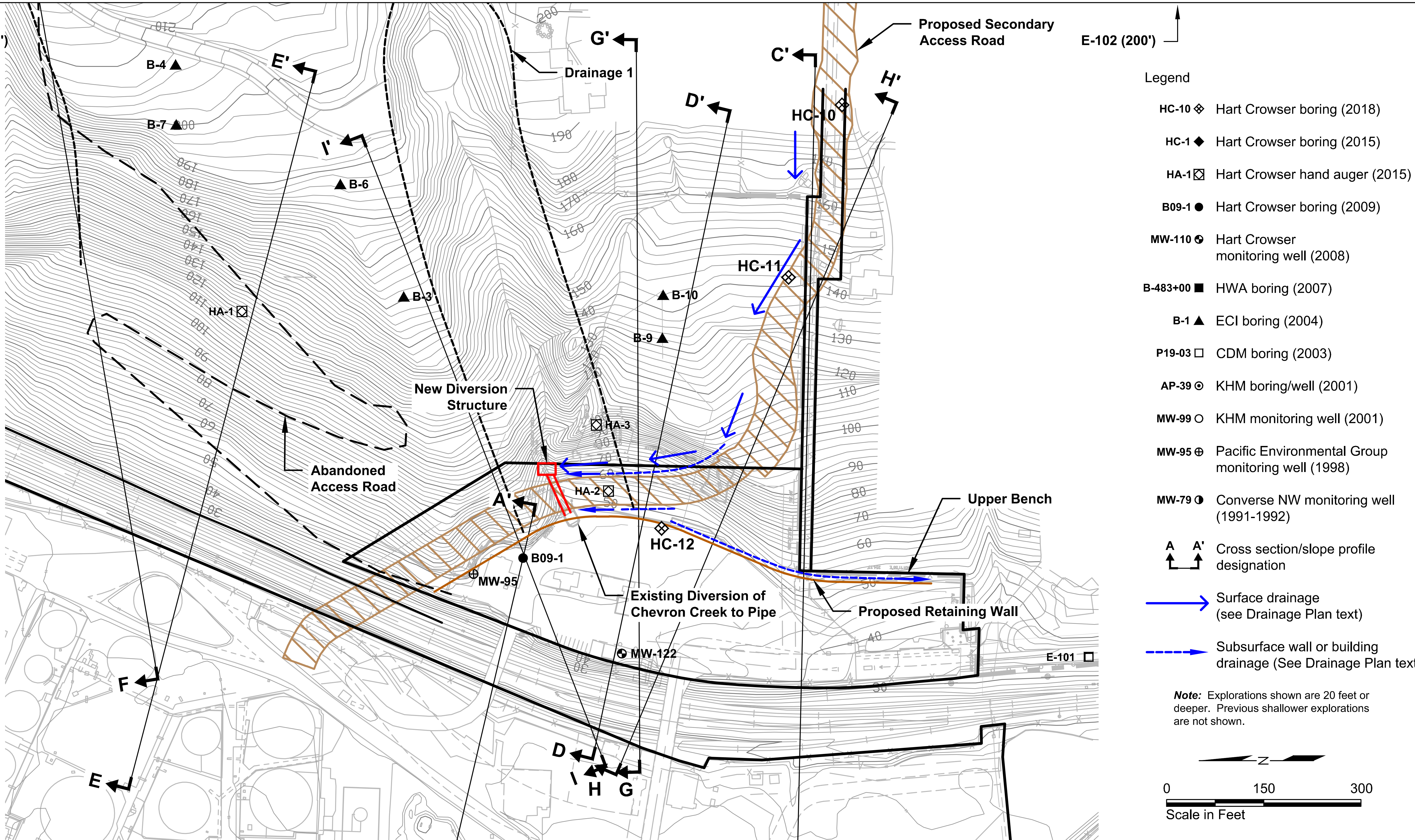


Point Wells
Richmond Beach, Washington

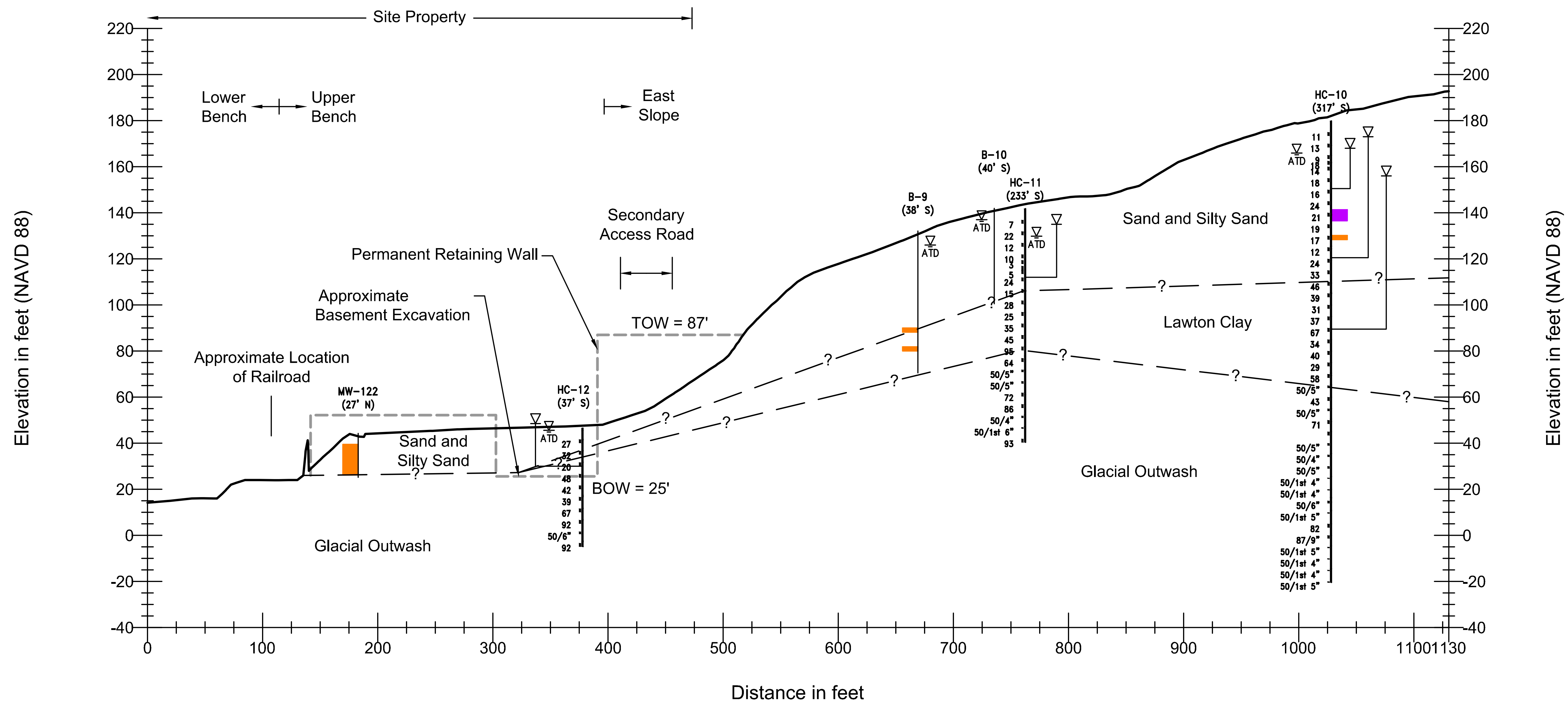
Site and Conceptual Drainage Plan for Secondary Access Road

17203-57 12/19

HARTCROWSER

Figure **2A**

File: L:\Notebooks\1720357_Point_Wells_County_Hearing_Responses_On-Call\CAD\1720357-001 (SPlan).dwg Layout:Apr2018SiteMap Date: 12-09-2019 Author: evinfairchild



Horizontal Scale in Feet
0 100 200
0 50 100
Vertical Scale in Feet
Vertical Exaggeration x 2

Notes:

1. Contacts between soil units are based upon interpolation between borings and represent our interpretation of subsurface conditions based on currently available data.
2. Slopes appear steeper than they actually are because of vertical exaggeration used for figure clarity. See Figure 4 for slope profile without vertical exaggeration.
3. See text and Figures 26 and 27 for further discussion of the potential liquefaction and slickensides observed in the borings shown here and in other borings across the site.
4. Liquefaction is not typically expected within the Lawton Clay. Possible zones of liquefaction in the Lawton Clay in boring B-9 represent thin layers of interbedded sands within the geologic unit.

Legend

HC-102 (34.5' E) — Exploration Number (Offset Distance and Direction)

Exploration Location

ATD — Water Level

Vibrating Wire Piezometer and Measured Groundwater Head

9 — Standard Penetration Resistance in Blows per Foot

Possible Slickensides

Possible Liquefaction

Point Wells
Richmond Beach, Washington

Liquefaction and Slickenside Potential in Generalized Subsurface Cross Section G-G'

17203-57 11/19

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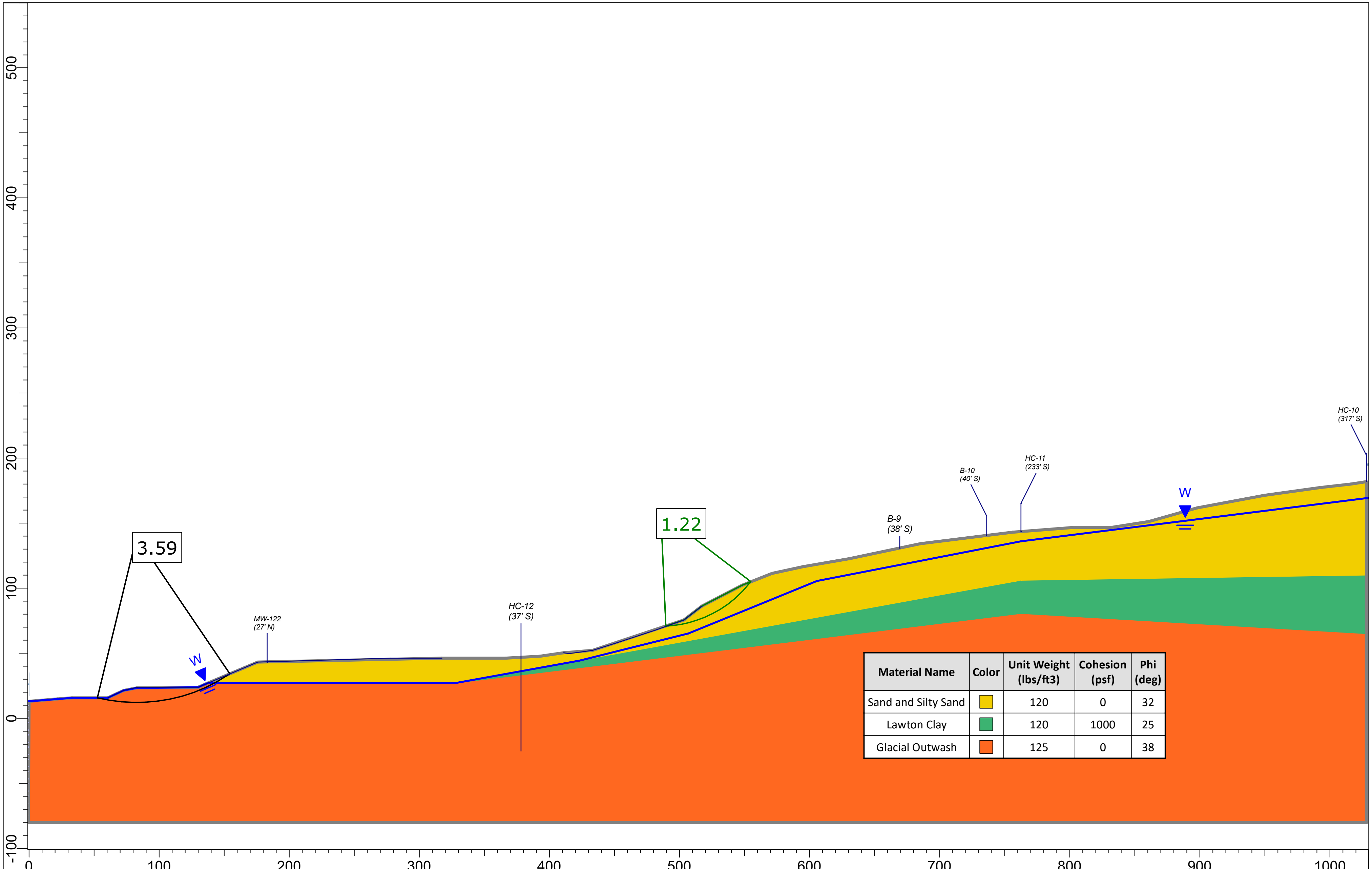
Figure
9A

File: \\seafs\Projects\Notebooks\1720357_Point_Wells_County_Hearing_Responses_On-Call\CAD\1720357-002 (XSec).dwg Layout:11x17 - H (3) Date: 11-26-2019 Author: melissaschweitzer



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	0	32
Lawton Clay		120	1000	25
Glacial Outwash		125	0	38

Notes:
1) All Figures: Green factor of safety (FS) value and slip surface are the minimum or critical slip surface based on search results.
2) All Figures: Results use non-circular, auto-refine search methods.

Point Wells
Richmond Beach, Washington

Section G-G'
East Slope Existing Conditions, Static

17203-57 | Scale 1:1200 | 12/4/2019

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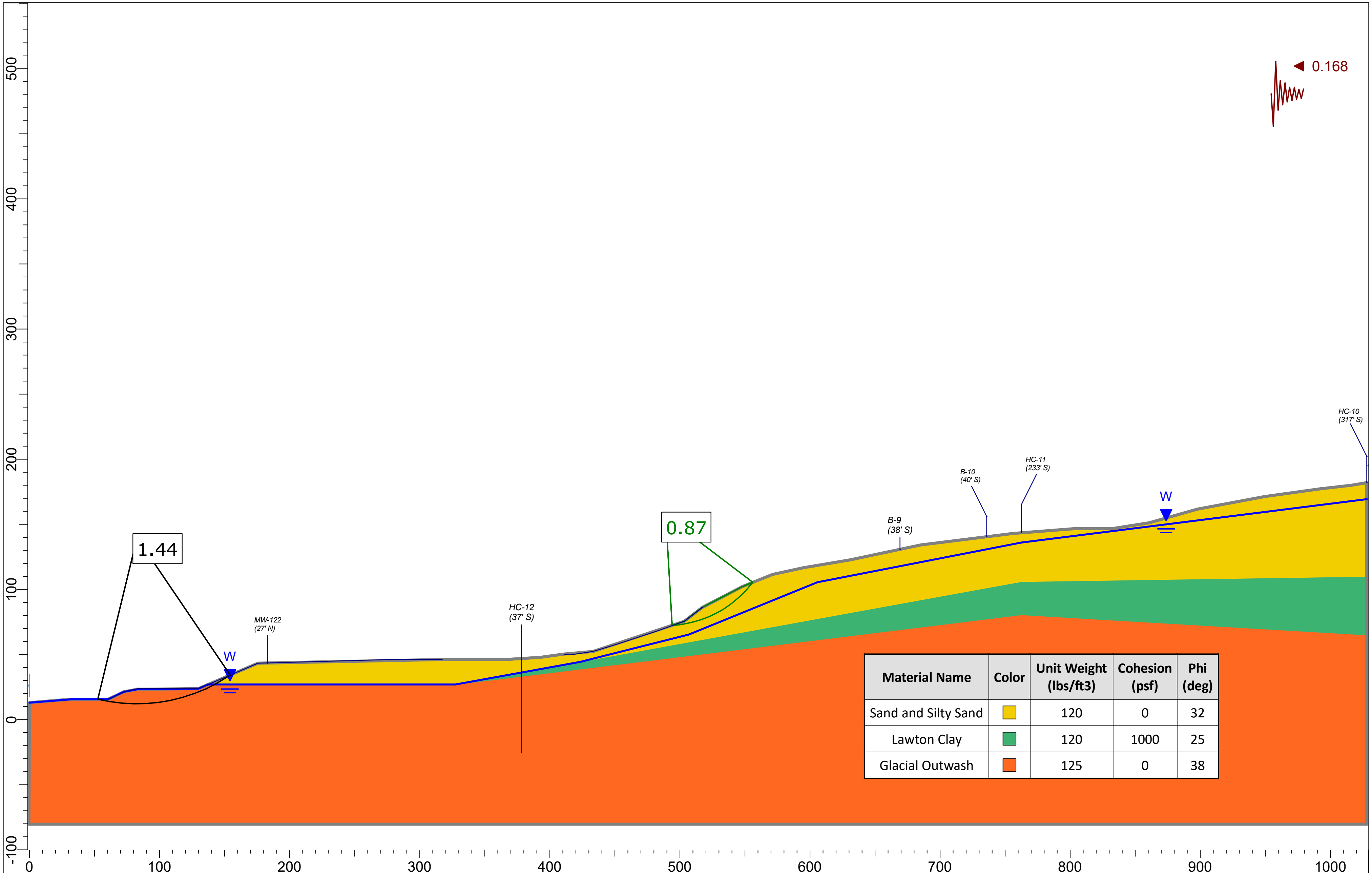
Figure
18a



Material Name	Color	Unit Weight (lbs/ft3)	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	0	32
Lawton Clay		120	1000	25
Glacial Outwash		125	0	38

Notes:
1) All Figures: Green factor of safety (FS) value and slip surface are the minimum or critical slip surface based on search results.
2) All Figures: Results use non-circular, auto-refine search methods.

Point Wells
Richmond Beach, Washington

Section G-G'
East Slope Existing Conditions, Psuedostatic

17203-57 Scale 1:1200 12/4/2019

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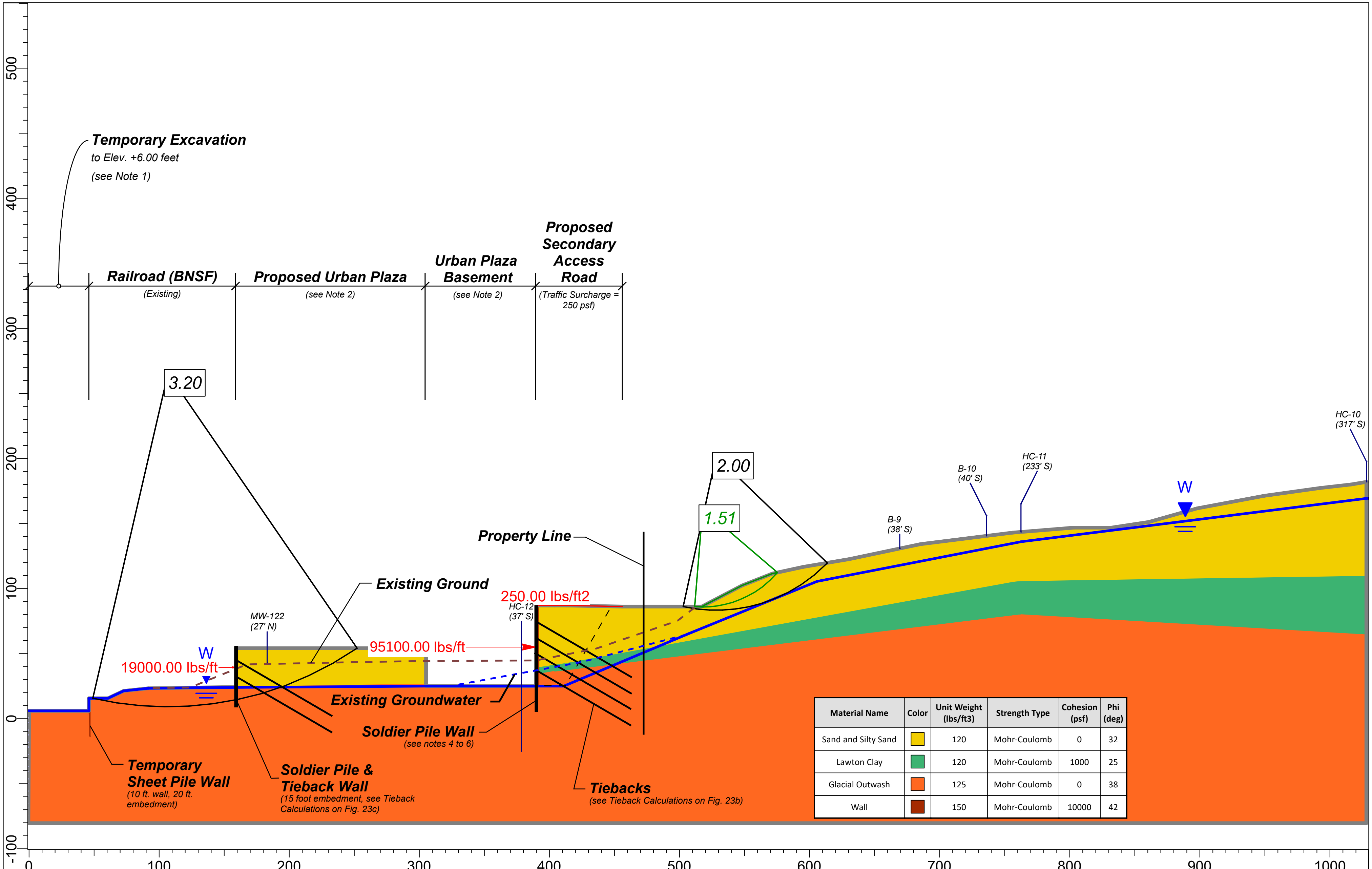
Figure
19a



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	Mohr-Coulomb	0	32
Lawton Clay		120	Mohr-Coulomb	1000	25
Glacial Outwash		125	Mohr-Coulomb	0	38
Wall		150	Mohr-Coulomb	10000	42

Notes:
1) Temporary excavation for remedial excavation or building basements. Basement structure will take permanent earth loads and grade around buildings will be raised to about elevation 50 feet west of railroad.
2) Building basement floors will support lateral earth pressures below existing grade.
3) Permanent wall drainage required since existing groundwater level above base of excavation.
4) Embed soldier piles 20-feet below base of excavation.
5) Permanent retaining wall height only about 40-feet above existing grade (see note 2).
6) Permanent wall drainage required (see note 3).

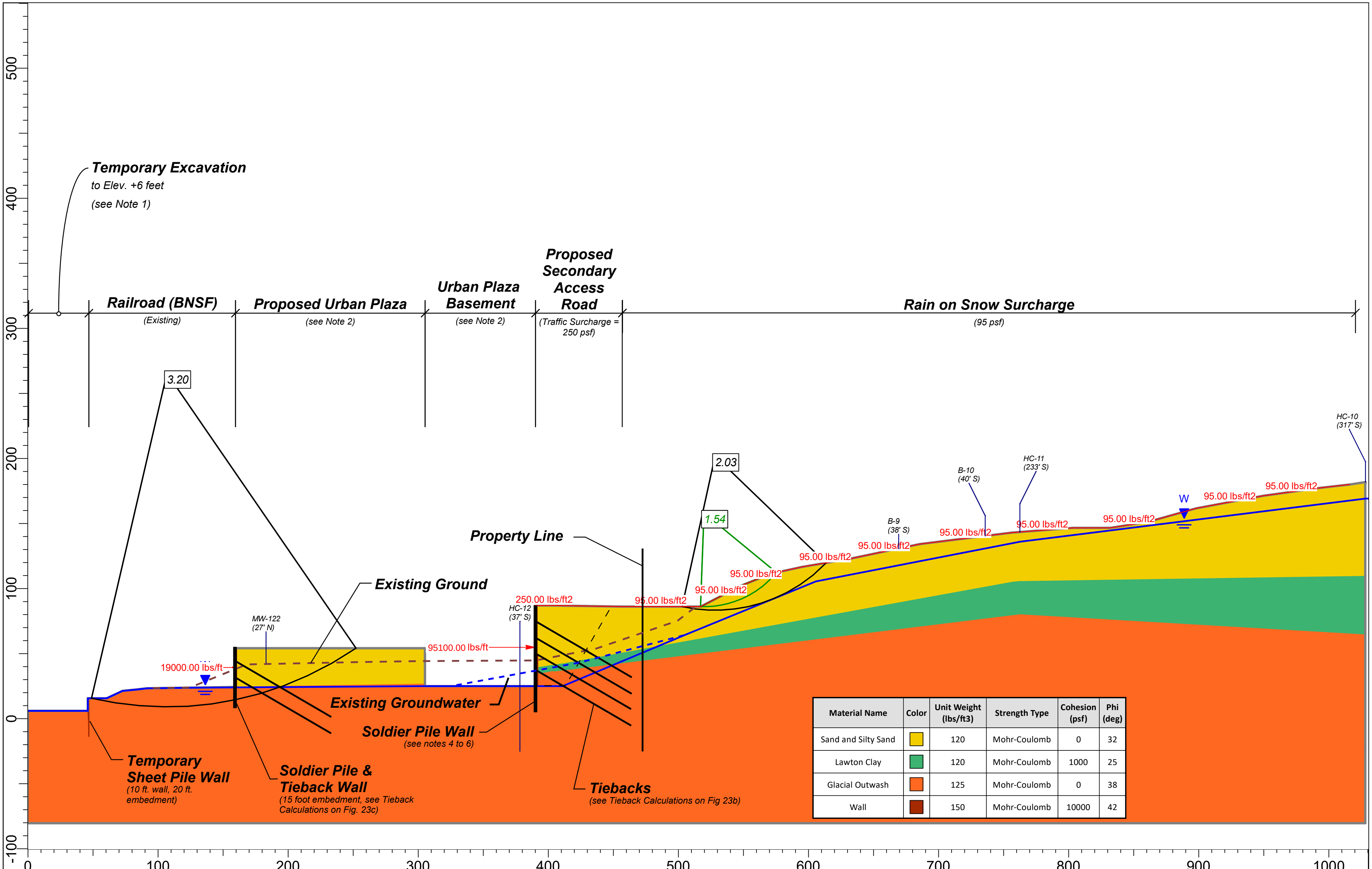
Point Wells
Richmond Beach, Washington

Section G-G'
Wall with Backfill, Static

17203-57 Scale 1:1200 12/3/2019

HARTCROWSER

Figure
22a



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	Mohr-Coulomb	0	32
Lawton Clay		120	Mohr-Coulomb	1000	25
Glacial Outwash		125	Mohr-Coulomb	0	38
Wall		150	Mohr-Coulomb	10000	42

Notes:
1) Temporary excavation for remedial excavation or building basements. Basement structure will take permanent earth loads and grade around buildings will be raised to about elevation 50 feet west of railroad.
2) Building basement floors will support lateral earth pressures below existing grade.
3) Permanent wall drainage required since existing groundwater level above base of excavation.
4) Embed soldier piles 20-feet below base of excavation.
5) Permanent retaining wall height only about 40-feet above existing grade (see note 2).
6) Permanent wall drainage required (see note 3).

Point Wells
Richmond Beach, Washington

Section G-G'
Wall with Backfill, Static with Snow Load

17203-57 Scale 1:1200 12/3/2019

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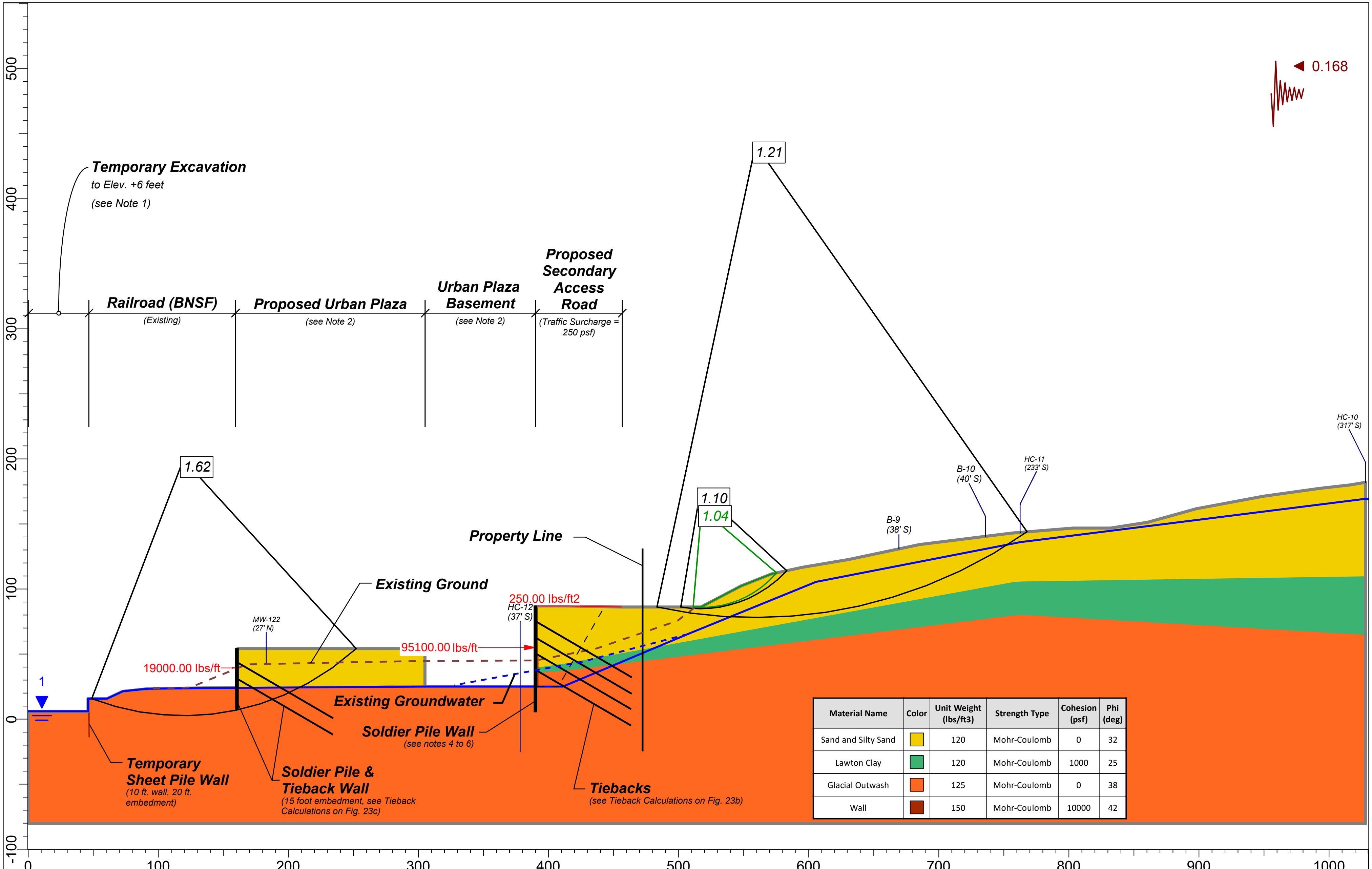
Figure
22b



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	Mohr-Coulomb	0	32
Lawton Clay		120	Mohr-Coulomb	1000	25
Glacial Outwash		125	Mohr-Coulomb	0	38
Wall		150	Mohr-Coulomb	10000	42

Notes:
1) Temporary excavation for remedial excavation or building basements. Basement structure will take permanent earth loads and grade around buildings will be raised to about elevation 50 feet west of railroad.
2) Building basement floors will support lateral earth pressures below existing grade.
3) Permanent wall drainage required since existing groundwater level above base of excavation.
4) Embed soldier piles 20-feet below base of excavation.
5) Permanent retaining wall height only about 40-feet above existing grade (see note 2).
6) Permanent wall drainage required (see note 3).

Point Wells
Richmond Beach, Washington

Section G-G'
Wall and Backfill, Pseudostatic

17203-57 | Scale 1:1200 | 12/4/2019

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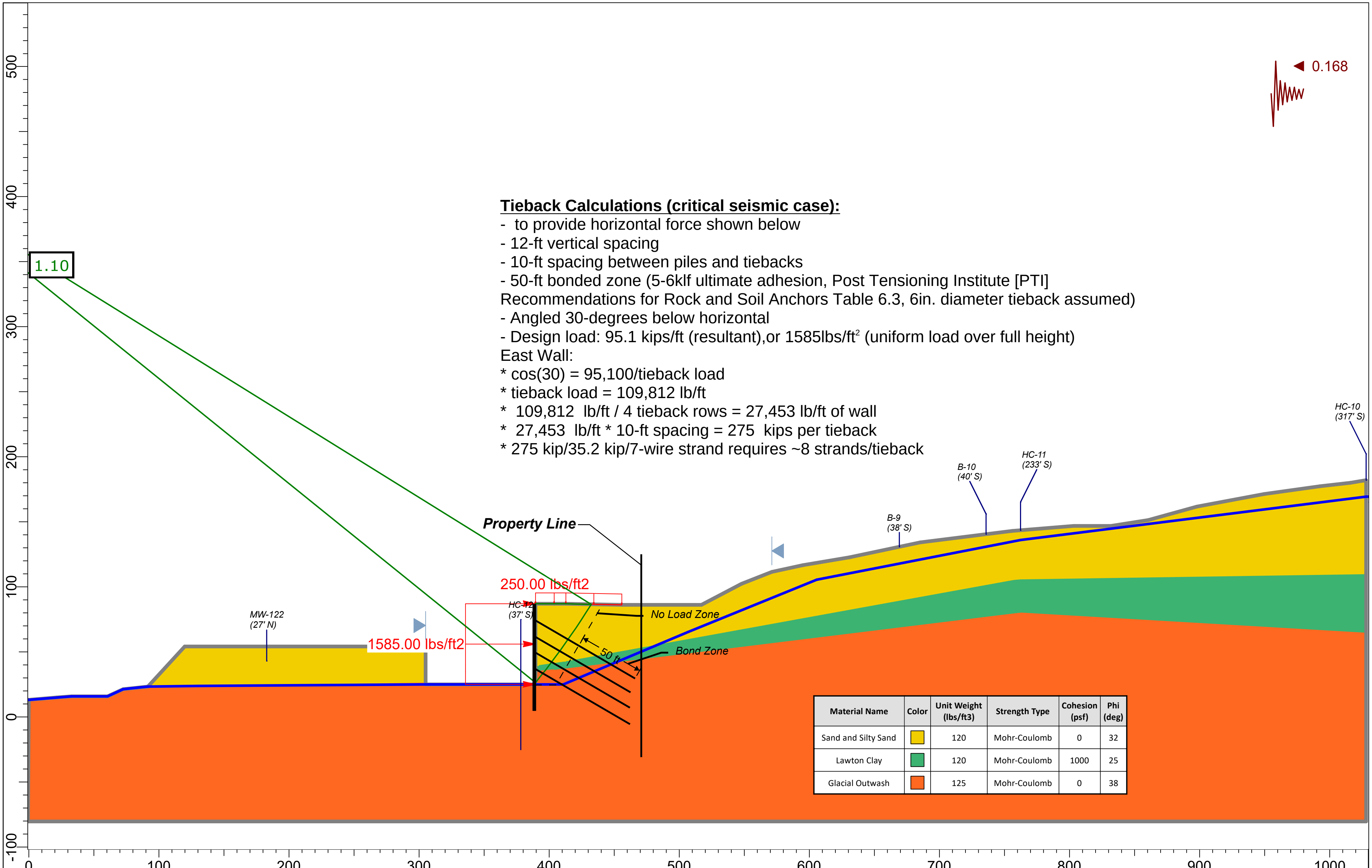
Figure
23a



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	Mohr-Coulomb	0	32
Lawton Clay		120	Mohr-Coulomb	1000	25
Glacial Outwash		125	Mohr-Coulomb	0	38

Notes:

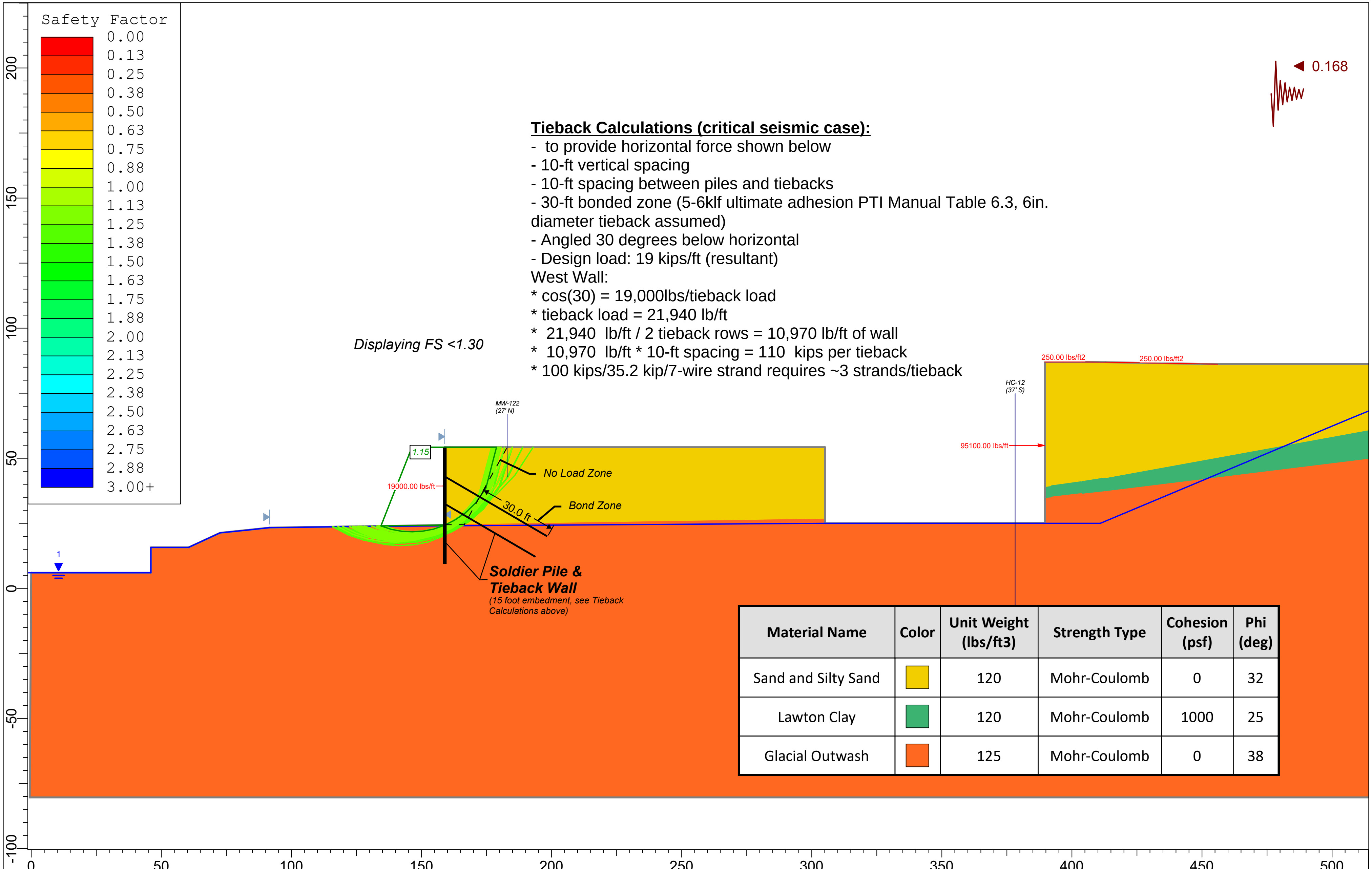
Point Wells
Richmond Beach, Washington

Section G-G' East Retaining Wall
Force Calculation, Pseudostatic

17203-57 | Scale 1:1200 | 12/5/2019

HARTCROWSER

Figure
23b



Tieback Calculations (critical seismic case):

- to provide horizontal force shown below
- 10-ft vertical spacing
- 10-ft spacing between piles and tiebacks
- 30-ft bonded zone (5-6klf ultimate adhesion PTI Manual Table 6.3, 6in. diameter tieback assumed)
- Angled 30 degrees below horizontal
- Design load: 19 kips/ft (resultant)

West Wall:

* cos(30) = 19,000lbs/tieback load
* tieback load = 21,940 lb/ft
* 21,940 lb/ft / 2 tieback rows = 10,970 lb/ft of wall
* 10,970 lb/ft * 10-ft spacing = 110 kips per tieback
* 100 kips/35.2 kip/7-wire strand requires ~3 strands/tieback

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	Mohr-Coulomb	0	32
Lawton Clay		120	Mohr-Coulomb	1000	25
Glacial Outwash		125	Mohr-Coulomb	0	38

Notes:

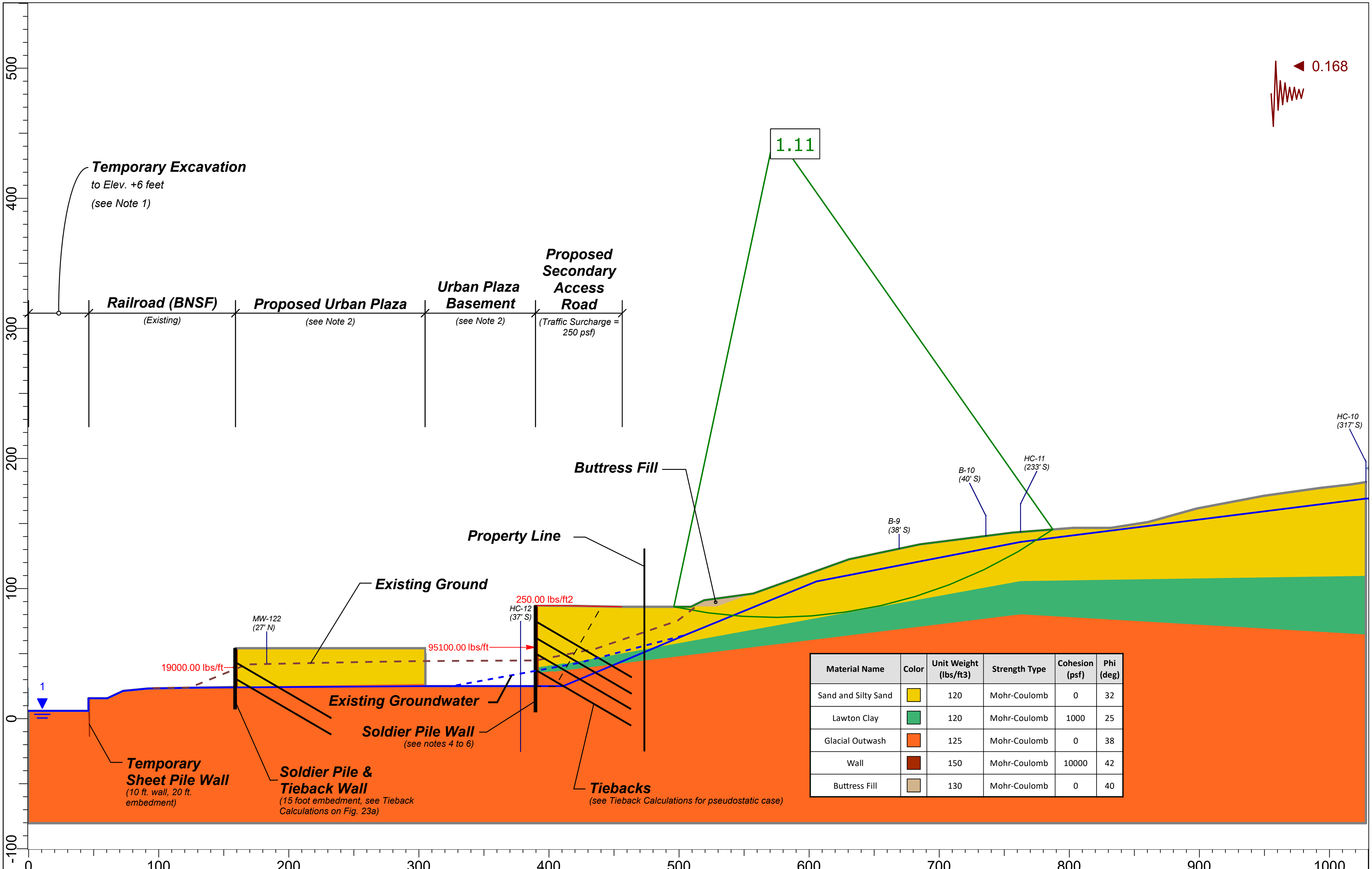
Point Wells
Richmond Beach, Washington

Section G-G' West Retaining Wall
Force Calculation, Pseudostatic

17203-57 Scale 1:600 12/4/2019

HARTCROWSER

Figure
23c



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Sand and Silty Sand		120	Mohr-Coulomb	0	32
Lawton Clay		120	Mohr-Coulomb	1000	25
Glacial Outwash		125	Mohr-Coulomb	0	38
Wall		150	Mohr-Coulomb	10000	42
Buttress Fill		130	Mohr-Coulomb	0	40

Notes:
1) Temporary excavation for remedial excavation or building basements. Basement structure will take permanent earth loads and grade around buildings will be raised to about elevation 50 feet west of railroad.
2) Building basement floors will support lateral earth pressures below existing grade.
3) Permanent wall drainage required since existing groundwater level above base of excavation.
4) Embed soldier piles 20-feet below base of excavation.
5) Permanent retaining wall height only about 40-feet above existing grade (see note 2).
6) Permanent wall drainage required (see note 3).

Point Wells
Richmond Beach, Washington

Section G-G' Wall and Backfill
Buttress Option, Pseudostatic

17203-57 | Scale 1:1200 | 12/4/2019

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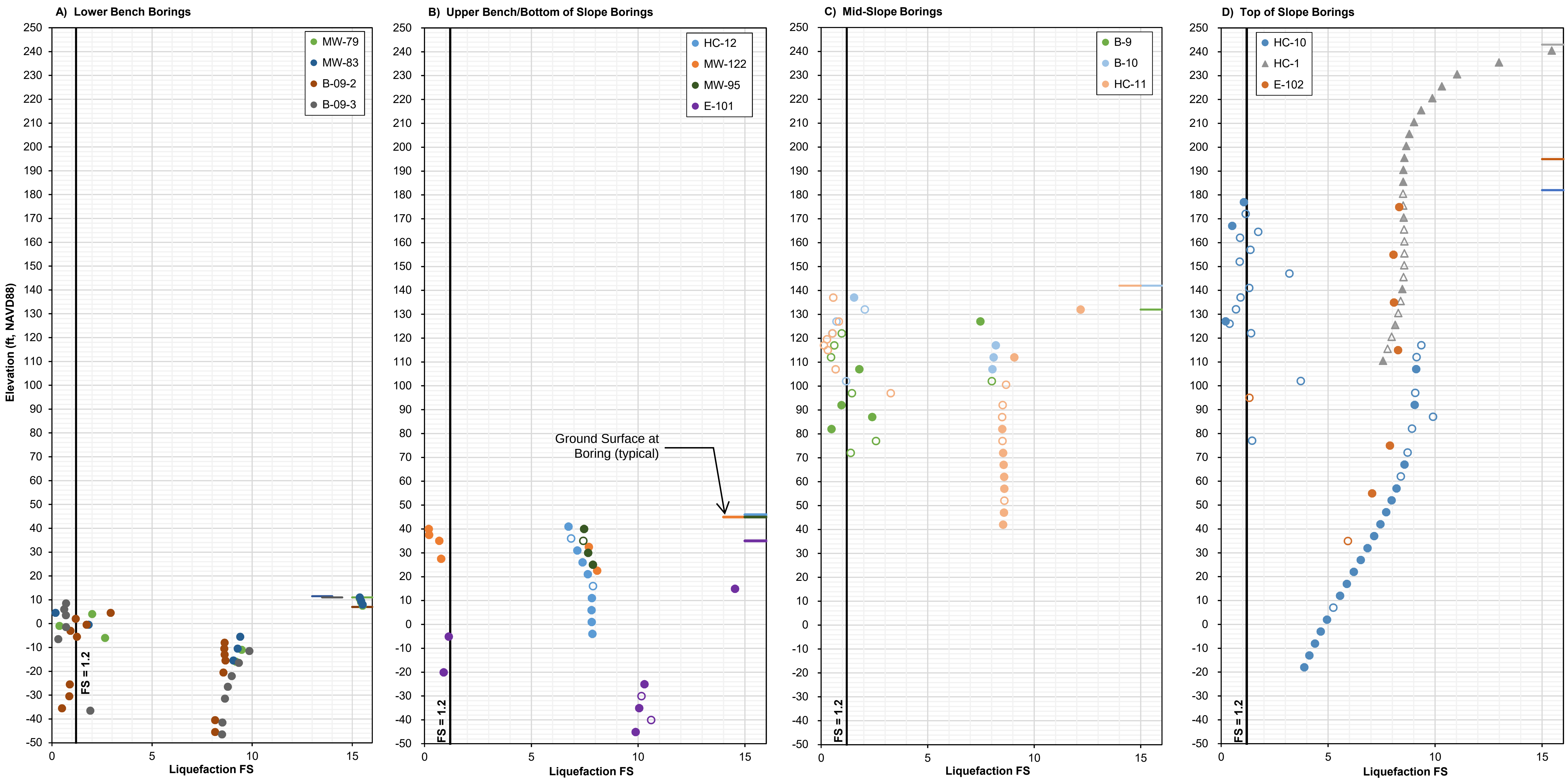
Figure
23d



Notes:
1. Liquefaction potential evaluated based on SPT blow counts using the Idriss & Boulanger (2008) method.
2. Hollow data points represent fine-grained samples that are not expected to liquefy, regardless of FS shown above. Fine grain liquefaction susceptibility was evaluated for select samples using Atterberg lab data and the criteria of Wc/LL<0.8 and PI>18 (Bray & Sancio, 2006). See text for further explanation.
3. A factor of safety of 1.2 is used per the Washington Dept of Transportation Geotechnical Design Manual (GDM) section 6.4.2.3.

Point Wells
Richmond Beach, Washington

Liquefaction Analysis of SPT Samples

17203-54 06/18

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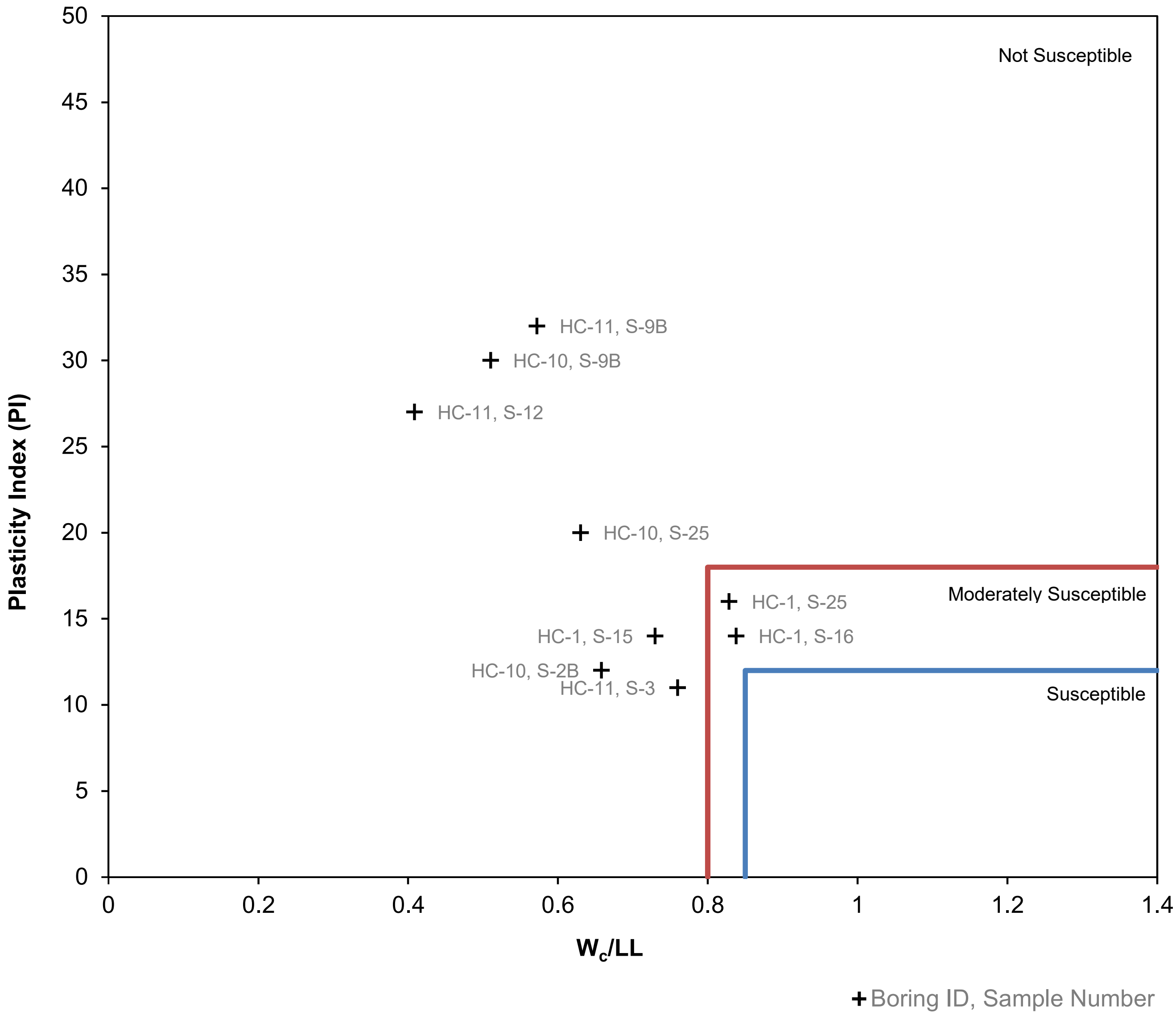
Figure
26

BFT 06/12/2018 L:\Notebooks\1720354_Point Wells EIS Geotech Analyses\Analysis and Calcs\Liquefaction - 2018\1720354_Liquefaction and Sr from SPT.XLSM



Notes:
1. Liquefaction susceptibility of fine grained soils was evaluated using the method presented in Bray & Sancio (2006).
2. The data plotted above represents samples from both historical and recent borings on site. See Figure 2 of the Subsurface Conditions Report (Hart Crowser, 2018) for boring locations.

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Fine Grained Soils Liquefaction Susceptibility

17203-54 06/18

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Figure

27