Willow Creek Daylight Project
Conceptual Level Geotechnical Assessment
Edmonds, Washington

November 24, 2014

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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21-1-12393-406

November 24, 2014

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RE: WILLOW CREEK DAYLIGHT PROJECT, CONCEPTUAL LEVEL GEOTECHNICAL ASSESSMENT, EDMONDS, WASHINGTON

Dear Mr. Shuster:

This letter report presents a summary of our geotechnical review of proposed channel excavation activities for the Willow Creek Daylight Project in Edmonds, Washington. The location of the project site is shown on the Vicinity Map, Figure 1. The purpose of this geotechnical assessment is to evaluate the potential effects of proposed channel excavations on adjacent property and structures and to develop conceptual level design recommendations to mitigate hazards if necessary. Shannon & Wilson, Inc. reviewed existing data and performed subsurface explorations to evaluate the stability of the proposed excavations and other geotechnical considerations for conceptual design for this Final Feasibility Phase. Results are presented herein.

BACKGROUND

The project site is located at the western edge of Edmonds (Figure 1, Vicinity Map). The City of Edmonds proposes daylighting the downstream section of Willow Creek to improve fish passage to the Edmonds Marsh, as part of a larger restoration project. Willow Creek flows from uplands through Edmonds Marsh into a stormwater pipe and into Puget Sound, as shown on the Willow Creek Restoration Area drawing, Figure 2. The downstream section of Willow Creek currently flows through culverts underneath the BNSF Railway Company (BSNF) Railroad, into a stormwater pipe along Admiralty Way, and under Marina Beach Park (the Park) to an outfall in Puget Sound. The proposed daylight channel will connect to the existing channel along BNSF and Chevron/Unocal property. It will then extend underneath the existing BNSF bridge, underneath a proposed new pedestrian and maintenance vehicle bridge at the Park, and then westward into Puget Sound, as shown in the Site and Exploration Plan, Figure 3. This general alignment selected as the preferred alternative alignment during the Early Feasibility Phase of this project. The preferred alignment through Marina Beach Park is yet to be determined, but is proposed as either Option A that extends through the off-leash dog park area or Option B that extends through the north end of the Park through the lawn to the beach (Figure 3).

Conceptual designs for this alignment include making a channel excavation from the existing open channel along the BNSF Railroad for a distance of about 750 feet to the Park (Figures 2 and 3). The preliminary dimensions of the excavations are expected to be 5 to 10 feet deep with a bottom width of 14 feet and a top width of 40 to 50 feet. Side slopes along the BNSF and Unocal property are 2 Horizontal to 1 Vertical (2H:1V). Immediately upstream from the BNSF bridge the east bank side slope is shown as 2H:1V with the possibility of a soldier pile wall installed where the channel meets the toe of the steep slope or a reduction in channel width at this location. Downstream from the bridge the side slopes are 3H:1V.

Subsurface explorations were conducted along both Park channel alignment options to characterize materials and evaluate geologic conditions present at the Park. Access limitations at this time prevented exploration in the daylight channel section along the Chevron/Unocal property and BNSF Railroad and adjacent to a steep slope just east of the BNSF bridge. We reviewed available background data and subsurface information from Arcadis reports and BNSF bridge designs to evaluate conditions for these areas where we did not have access.

SUBSURFACE EXPLORATIONS

The locations of the boring and test pits completed for this project are shown in the Site and Explorations Plan, Figure 3. Descriptions of the drilling programs, test pit programs, and the boring and test pit logs are presented in Appendix A.

Shannon & Wilson, Inc. explored subsurface conditions at seven locations in the Park (Figure 3). Subsurface explorations were performed for soil characterization, geotechnical analyses, and contamination testing on August 28 and 29 and September 5, 2014. A representative from Shannon & Wilson, Inc. was present during the field exploration periods to observe the drilling and sampling operations, retrieve representative soil samples for subsequent laboratory testing, and to prepare descriptive field logs. Additionally, an archeologist was on-site during field explorations to document the presence of pre-historic and historical items (Cultural Resource Consultants, Inc. [CRC], 2014).

Borings B-1 and B-2 were drilled by Holt Services, Inc. in two locations in the off-leash dog park. These borings extended to 45 feet below ground surface (bgs) and 20 feet bgs, respectively. The borings were drilled using mud rotary drilling techniques to advance below the ground level. Standard Penetration Tests were performed at select depth intervals and samples were collected for visual classification, water content determinations, and grain size analysis.

Test pits were excavated by Clear Creek Contractors on September 5, 2014. Test pits TP-1, TP-2, and TP-3 were excavated in the off-leash dog park along the Option A alignment to depths ranging between 9.5 and 11 feet (bgs). Test pits TP-4 and TP-5 were excavated in the park along the Option B alignment to depths of 14 and 8.3 feet, respectively. Samples were collected at select depth intervals for visual classification, water content determinations, and grain size analysis.

We screened samples on site for contamination based on visual, olfactory, or other indication of contamination. We screened samples collected near the water table, where encountered, for volatile organic compounds using a photoionization detector. No indications of hydrocarbon contamination were observed in the test pit or boring samples.

LABORATORY ANALYSES

Geotechnical laboratory tests were performed on select samples retrieved from the explorations to characterize the index and engineering properties of the subsurface soils at the project site. Laboratory testing included visual soil classification, moisture content determinations, and grain size analyses. The geotechnical laboratory testing was performed in the Shannon & Wilson, Inc. laboratory in Seattle, Washington, and in general accordance with the American Society of Testing and Materials/ASTM International (ASTM) standard procedures (ASTM, 2000 – 2011). A brief description of the laboratory test procedures and the laboratory test results are presented in Appendix B.

GEOLOGIC INTERPRETATION

We interpreted the geology and subsurface conditions along the project alignment from samples collected from geotechnical borings and test pits performed from this phase of the project, from data gathered from existing projects in the vicinity, and from geologic maps of the area. The following includes a description of geologic setting, of interpreted geologic units, and the subsurface conditions encountered in the project area from our explorations and explorations by others.

Geologic Setting

Geologists generally agree that the Puget Sound area was subjected to six or more major glacial events. Each glaciation deposited new sediment and partially eroded previous sediments. During the intervening periods when glacial ice was not present, normal stream processes, wave action, weathering, and landsliding eroded and reworked some of the glacially derived sediment, further complicating the geologic setting.

During the most recent Fraser Glaciation of the Vashon Stade that covered the central Puget Lowland, approximately 18,000 to 16,000 years before present (Porter and Swanson, 1998), the glacial ice is estimated to have been about 3,000 feet thick in the project area (Thorson, 1989). The weight of the glacial ice resulted in compaction of the glacial and nonglacial soils beneath the ice. The glacial and nonglacial deposits are overlain by younger (Holocene Epoch), relatively loose and soft, post-glacial soils that include peat, beach, and fill deposits.

Existing Information

According to geologic maps (Washington State Department of Natural Resources [DNR], 2011 and Minard, 1983), the soils along the daylight channel alignment consist of fill. The adjacent steep slope to the east consists of nonglacial soils of the Whidbey Formation, which are glacially over-ridden and typically consist of locally cross-bedded sand with silt and clay layers.

Additionally, we reviewed geologic and subsurface explorations and interpretations in the following documents include:

- Final Conceptual Site Model (Arcadis, 2013),
- Final 2011 Site Investigation Completion Report (Arcadis, 2012), and
- BNSF Final Design Services (BNSF, 2010), including borings by HWA Geosciences Inc. (HWA, 2008)

Arcadis conducted remedial site investigations for the former Unocal Edmonds Bulk Fuel Terminal property on behalf of Chevron Environmental Management Company, with reports dating back to 2001. These studies have included remediation stages involving site history, subsurface exploration, groundwater monitoring, and soil and groundwater testing in the vicinity of the daylight channel alignment east of the BNSF Railroad. Arcadis (2012) identified five geologic units along the daylight channel alignment, including:

- 2008 Fill is remediation backfill materials that consist of poorly graded, coarse gravel generally 6 to 12 inches above observed groundwater, overlain by fine to medium sand, trace silt, and fine to medium gravel to the ground surface.
- 1929 Fill consists of silty sands with gravel and sandy silts with gravel from 8 to 15 feet bgs interpreted as fill material placed circa 1929 or later.
- Marsh Deposits consists of a 6- to 12-inch-thick layer of silty and sandy silt with organic matter such as peat, wood debris, and decomposing vegetation beneath the 1929 Fill. It was generally encountered from about 8 to 14 feet bgs. The unit is directly below the 1929 Fill material and interpreted to be representative of the former marsh.
- Beach Deposits consists of poorly graded, fine to medium sand with fine gravel that contains organic material such as driftwood and seashells. This layer is interpreted to represent of the former beach environment in the area prior to development.
- Whidbey Formation. This material is a poorly graded sand layer consisting of fine to medium sand with fine gravel that contains interbedded sand with silt, and interbedded silt and sandy silt ranging in thickness from 1 inch to several feet.

Figure A-9 in Appendix A shows depths of the remediation gravel backfill of the 2008 Fill (Arcadis, 2012) and monitoring well MW-149R (Figure A-10) (Arcadis, 2013) shows the stratigraphy of remediation gravel in the north end of the daylight channel alignment east of the BNSF Railroad.

Boring logs BH-1 and BH-2 from the geotechnical report that accompanied the design plans for the BNSF Railroad bridge foundations were used in subsurface interpretations and are presented in Appendix A-11 and A-12.

Geologic Units

We identified geologic units to group the complex sediment and soil types encountered in the project explorations. The geologic unit descriptions are described herein and are shown on the boring logs presented in Figures A-2 through A-12 in Appendix A and Figure 4.

The subsurface conditions we encountered in explorations in the project area generally consist of a fill (Hf) layer overlying beach deposits (Hb) locally interlayered with a 0.5- to 1-foot-thick marsh deposit (Hm). These units are further described as:

- **Fill (Hf) –** Explorations encountered 6 to 8 feet of fill soil with variable properties. Hf generally consists of silty sand with gravel and cobbles to clayey sand with gravel and cobbles to 6 feet bgs at TP-4 at Marina Beach Park lawn area. This fill may be associated with a glacial till source. Hf encountered in Marina Beach Park outside of the lawn area consists of poorly graded sand with gravel to 8 feet bgs, and may be derived from a nearby excavation in a similar beach environment. Based on the historic land uses in this area, some deposits resembling beach deposits have been interpreted as fill.
- **Beach Deposits (Hb) –** Explorations encountered more than 20 feet to 46.5 feet of Hb below the fill unit. Hb generally consists of medium dense, poorly graded sand with silt to poorly graded sand and gravel with variable amounts of silt and wood fragments. Below about 35 feet, Hb becomes dense.
- **Marsh Deposits (Hm) –** Test pit explorations locally encountered a thin ½- to 1-foot-thick layer of silty sand laminated with sandy silt and peat between 6 to 8 feet bgs. Metal debris was found on top of, and in, the marsh deposits in TP-2 and TP-3. We encountered trace iron-oxide staining was found in marsh deposits in TP-5.

Subsurface Conditions

Interpreted subsurface conditions along the daylight channel alignment based on existing information and explorations performed for this project are presented in Cross Sections A-A' through D-D' of the Typical Stream Channel Cross Section, Figure 4.

Option A of the daylight channel alignment consists of Hb with possible fill (Hf) from a beach source in the upper 6 to 8 feet bgs as presented in Cross Section A-A' (Figure 4). Option B of the daylight channel alignment consists of fill (Hf) to 6 feet bgs, possibly from a glacial till source, overlying Hb as presented in Cross Section B-B'.

Subsurface conditions at the location of the proposed pedestrian bridge are underlain by Hb and Hf deposits of a beach origin as presented in Cross Section D, Sheet 2 of Figure 4.

Subsurface conditions at the adjacent steep slope and the base of the steep slope, where the daylight channel alignment meets the toe of the slope is shown in Cross Section C-C'. Cross Section C-C' indicates Hb and Hf are present at the base of the slope and mapped Whidbey Formation underlies the slope. There is likely a layer of colluvium mantling the slope with variable thicknesses but the exact configuration of these layers is unknown at this time. Fill in the form of remediation gravels backfilled to between 4 to 6 feet bgs will likely be encountered north of Cross Section C-C'.

Groundwater was encountered at about 9.5 feet bgs (elevation 6 feet NAVD88) at B-1, B-2, and TP-1 at Cross Section A-A'. At TP-5, on the beach, groundwater was encountered at 8 feet bgs (elevation 3.5 feet NAVD88), possibly due to close proximity to tide levels.

GEOTECHNICAL CONCLUSIONS AND RECOMMENDATIONS

Daylighting of Willow Creek will require excavation of the daylight channel at the following locations:

- Along BNSF and Chevron/Unocal property near the Washington State Department of Transportation stormwater pipe and manhole,
- Underneath the existing BNSF Railroad bridge,
- Underneath a proposed new pedestrian and maintenance vehicle bridge at the Park, and
- Into the Park preferred alternative alignment of the beach outlet.

We have performed a geotechnical assessment to evaluate the potential effects on adjacent property and structures, and to develop recommendations for preliminary design of mitigation measures. We note that a site topographic survey and a geotechnical reconnaissance of the Unocal property was not performed due to access limitations. Therefore, our assessment of the surface features, exposed geology and stability of the Unocal property and the steep slope on the east boundary of the Unocal property was not performed as part of this study and remains to be performed during the design phase.

GEOLOGIC HAZARDS

Potential geologic hazards that may affect the site include slope failure of the steep slope; liquefaction and associated effects (lateral spreading, differential settlement, and reduced bearing capacity foundations); and fault rupture. Our review of these hazards is based on historical mapping and results of subsurface explorations.

Landslides are movement of a rock and/or soil mass on a slope caused by shear failure within the rock and/or soil. Based on the Washington State Coastal Atlas (Washington State Department of Ecology [Ecology], 1979), the project site is mapped as unstable due to the steep slope east of the railroad tracks. The closest mapped landslide occurred about ½-mile south of the site, along the shoreline. Landslides can occur quickly or progressively over time, and can be either deep-seated or shallow. Potential causes that can increase the risk of landsliding include: seismically induced ground movement, increasing the water and porewater pressures in the rock and/or soil, increasing the loading on or above the slope, removing material at the toe of the slope, and strain-softening of overconsolidated clay. At the project site, it is unlikely that seismic shaking would cause a deep-seated landslide because of the dense nature of the Whidbey Formation soils that underlie the slope. Surficial sloughing of loose colluvium on the surface of the slope is possible. We estimate that the potential for this type of movement is low to moderate over most of the hillside and high in some areas where local topography is steeper.

The proposed excavation of soils for channel construction at the toe of the steep slope just east of the BNSF bridge is potentially destabilizing. In our opinion, this proposed excavation over a distance of about 50 to 100 feet will likely require either construction of a retaining wall at the toe of the slope to accommodate the 2H:1V sloped bank on the east side of the creek or a reduction in channel width. If a retaining wall option is selected, it would likely consist of a soldier pile and lagging wall, as shown on Figure 4. Vertical members (soldier piles) consist of steel sections placed in predrilled holes spaced 6 to 8 feet apart and typically backfilled with lean mix concrete. Penetration depths below the final excavation level should be designed for kick-out resistance. We anticipate that the soldier pile embedment bgs may need to be up to two times the cantilevered height of the wall. We recommend that permanent lagging be installed between soldier piles. Permanent lagging may consist of precast concrete panels and should be installed as the excavation proceeds. In general, not more than 4 feet (measured vertically) of unsupported excavation should be exposed at any one time; however, that should be evaluated after the actual soil conditions at the wall location are determined by making subsurface explorations. The actual height of vertical, unsupported excavation may vary depending on the soils encountered. The final design embedment depths should be determined by the structural designer with input from the geotechnical engineer.

To protect the base of the wall from scour it may be necessary to construct a reinforced soil slope in front of the wall. Use of a geogrid-reinforced slope is one way to accomplish this. We have prepared a sketch illustrating this concept in Figure 5, Schematic Soldier Pile Wall. A vegetated surface (green screen or green wall) can be installed in this area to provide the benefits of overhanging vegetation to this section of the channel while visually hiding the constructed wall, as shown in Figure 5.

Soil liquefaction is a phenomenon in which excess pore pressure in loose, saturated, granular soils increases during ground shaking to a level near the initial effective stress, thus resulting in a reduction of shear strength of the soil (a quicksand-like condition). Because of this reduction in shear strength during liquefaction, ground settlement and lateral spreading (ground movement on very gentle slopes) may occur. Vertical and lateral foundation restraint may also be significantly reduced. In general, the soils below about 14 feet at the site are sufficiently dense to preclude liquefaction. There is a thin layer of medium dense sand between about 10 and 14 feet that could liquefy; however, in our opinion, this would result in minimal ground settlement and no lateral spreading.

The fault nearest to the project site is the South Whidbey Island Fault, which is 7.2 miles away. Based on the distance to the nearest fault and the apparent lack of recent movement on this fault, it is our opinion that the potential for fault rupture at the site is relatively low and not a design issue.

Based on the mapped information and geotechnical analyses in the vicinity, of the potential for geologic hazards at the site is considered low provided the slope instability mitigation measures discussed above are included in the design.

Channel Side Slope Stability

In general, the proposed Willow Creek channel alignment alternatives are underlain by loose to dense, granular fill materials and beach deposits that will provide relatively stable side slopes ranging from 2H:1V to 3H:1V. During our subsurface explorations, we observed groundwater at elevation 6 feet in TP-1, 3.5 feet in TP-5, 6 feet in B-1, and 7 feet in B-2. It is likely that the groundwater elevation will fluctuate with the tides and in response to rainfall. The proposed bottom of channel is elevation 4 feet. Therefore, the proposed channel excavation will extend below the groundwater level in some areas. Groundwater control and temporary dewatering will be required in order to maintain stable slopes and allow excavation to be performed under “dry” conditions.

At the proposed Marina Beach channel, shown in Figure 4, Sections A-A’ and B-B’, the soils that will form the channel side slopes consists of loose to dense sand and gravel fill over beach sands. The proposed channel cross sections indicate that the creek will consist of a 6-foot-wide low-flow channel and a 20-foot-wide bankfull channel. These soils will generally form stable 2H:1V side slopes, steeper than the proposed 3H:1V side slope. The soils encountered in boring B-2, located adjacent to the south side of the existing parking lot, consisted of medium dense sand and gravel (fill and beach deposits). In our opinion, the proposed channel excavation for channel alignment Option A, adjacent to the parking lot, will not create a slope stability issue for the parking lot.

At the proposed pedestrian bridge channel (Section D-D’), the soils that will form the channel side slopes consists of 7 feet of medium dense sand and gravel fill materials overlying medium dense beach sand and gravel. Groundwater was observed during drilling at 9.5 feet deep (elevation 6 feet). These soils will generally form stable 2H:1V side slopes. Scour protection will be required.

Based on our review of the BNSF bridge design drawings (Sheet 1 of 3, 90% Submittal by AECOM, dated December 8, 2008), the bridge was designed for a future 6-foot bottom width, with a channel invert elevation of 4.26 feet, with 1.5H:1V slopes extending down from the top of the bridge piers to the channel bottom. The geometry of the bridge (span is 37 feet long) is such that 2H:1V sloping side channels will not allow for a 6-foot-wide bottom channel. Thus, a steeper slope (1.5H:1V) will be required underneath the bridge. In our opinion, the steeper slope is acceptable; however, these slopes will need to be armored at the surface in order to limit erosion and scour which could cause undermining and sloughing of the slopes. Special precautions should be exercised during the excavation of soils from beneath the railroad bridge. We recommend that the exposed soils be systematically compacted with a backhoe-mounted hoepack as the excavation proceeds. This will densify the existing fill materials and beach deposits and reduce the potential for sloughing. We recommend armoring the side slopes with a 1-foot layer of 6- to 8-inch quarry spalls overlain by 1- to 2-foot riprap. Future excavations beneath the bridge will need to be coordinated with BNSF Railway operations and safety requirements.

Construction of the Willow Creek Channel improvements will require close coordination with BNSF. BNSF's primary concern will be the uninterrupted passage of trains, and work windows to perform construction may be as short as a couple of hours each day. It is important that this be considered in the in the design and constructability of the structure. We recommend that the design team meet with BNSF early on to discuss the project and better understand what their concerns are and how they will accommodate construction.

Geotechnical boring logs for the BNSF bridge project (borings BH-1 and BH-2 by HWA) indicated the presence of loose to medium dense sand and silt sand to 18.5 feet, followed by dense, slightly gravelly, silty sand and sand with gravel to the bottom of the boring at 41.5 feet deep. Based on our review of the soils data, it is likely, in our opinion, that the driven steel piles that support the BNSF bridge derive their bearing from soils below a depth of 18 feet. Thus, the proposed excavation that will remove soils from beneath the bridge will not have an adverse effect on foundation bearing capacity of the existing bridge.

At the proposed channel near the bluff, just east of the BNSF bridge (Section C-C'), the soils that will form the channel side slopes consists of granular fill materials to silt, sandy silt, and sands, as noted in boring logs MW-149R and BH-1, respectively. These soils will generally form stable 3H:1V side slopes; however, the current design shows a 2H:1V bank at the east side of the channel; however, the geometry of this section of creek channel will have to be modified to accommodate the property boundary and the steep slope that rises to the east. During an earlier data acquisition site visit, we noted the presence of a large old concrete structure extending along the toe of this steep slope. The structure may have been constructed to serve as a retaining wall at the toe of the slope. Given the close proximity of the proposed channel to the toe of the slope, it is possible that the proposed channel excavation could undermine the structure at the toe of the slope and thereby cause slope instability. We recommend that additional site investigations be performed to collect data on the slope, concrete structure, and condition of soils at this location. Site-specific slope stability analysis should then be performed to determine if mitigation measures are required. For feasibility level planning purposes, we recommend the preliminary design include a retaining wall structure along the toe of the steep slope (Figure 5).

Pedestrian and Maintenance Vehicle Access Bridge Design Considerations

Foundation Design

Structural design concepts for the proposed pedestrian bridge are not available at this time. However, we assume the bridge will span 30 to 35 feet over the proposed creek channel and be designed for HS-20 loading. Our analyses based on the results of boring B-1 indicate that the medium dense soils between 9 and 14 feet deep (below the groundwater level) at the proposed bridge location are susceptible to liquefaction during a design level seismic event. Thus, the upper 14 feet of soils at the proposed bridge site would be susceptible to settlements during a seismic event and shallow spread footing foundations will not be suitable. For this reason, we recommend that the proposed bridge be supported on deep foundations that derive their capacity from medium dense to dense granular soils below 14 feet. At this site, deep foundations may consist of either drilled piles, such as auger cast-in-place piles (augercast), or driven piles such as driven steel pipe. The following sections discuss design issues for each type of pile.

Pipe Pile Foundations

Piles develop resistance through friction between the side of the pile and the soil, and from end bearing at the tip of the pile. Piles are driven until a specified depth at which the amount of developed resistance is enough to withstand the proposed loading conditions. Pipe piles are typically installed by means of an impact hammer. Vibratory hammers can also be used during installation; however, vibratory hammer installation methods do not provide a means to evaluate that the pile has reached the correct driving criteria (driving resistance). Selection of the proper hammer for the driving conditions is important to the success of the installation. The hammer selection process requires an understanding of the pile diameter and required vertical compressive loads and uplift loads.

A drivability analysis should be performed in order to select the appropriate hammer. The drivability analysis should consist of dynamic load testing coupled with a Case Pile Wave Analysis Program and wave equation analysis. This will help determine the optimal driving equipment and confirm that the pile has sufficient capacity with the desired factor of safety. We recommend that a representative of the geotechnical engineer observe the installation of driven piles on a full-time basis to evaluate the adequacy of the construction procedures.

Augercast Pile Foundations

Augercast piles are installed by rotating a continuous-flight, hollow-stem auger to a predetermined depth. After the auger is rotated to the predetermined depth, a high-strength, sand-cement grout is pumped under controlled pressure through the center of the shaft as the auger is slowly withdrawn. By maintaining pressure in the grout line and extracting the auger no faster than an equivalent volume of grout is pumped, a continuous column of concrete is formed. A single reinforcing rod can be placed through the hollow stem of the auger and/or a reinforcing cage with centering guides can be placed in the column of wet grout. Where piles are expected to experience tensile/uplift forces, the central reinforcing rod should be extended for the full length of the pile.

The quality of the augercast concrete piles depends on the procedure and workmanship of the contractor who installs them. We recommend that a representative of the geotechnical engineer observe the installation of augercast piles on a full-time basis to evaluate the adequacy of the construction procedures.

Our conceptual evaluation of bridge foundations included a preliminary analysis of pile capacity. Assuming 12-inch steel pipe piles are selected, we estimate that a capacity of 50 tons can be achieved by driving the piles approximately 40 to 50 feet deep. We also considered 12-inch-diameter augercast piles. Augercast piles installed to a depth of 40 to 45 feet can develop up to 50 tons capacity. Greater capacities could be achieved by increasing the diameter of the piles or by increasing the depth of penetration.

Estimated Settlements of Pile Foundations

Based on the subsurface conditions encountered in the borings, estimated pile design loads, and installation techniques, relatively minor settlements will occur upon loading. We estimate total settlement of the piles would be on the order of ½ inch, with differential settlements of about ¼ inch. No long-term settlements are anticipated.

Lateral Resistance

Lateral loads acting on the structure may be resisted by the passive earth pressure against the pile caps and grade beams, the frictional resistance developed between the sides of the pile cap, and the lateral resistance provided by the vertical piles.

We recommend that passive earth pressure developed from compacted granular fill against the pile caps be estimated using an equivalent fluid weight of 300 pounds per cubic foot. This value applies to soils above the groundwater table and assumes that the pile caps are founded at least 2 feet below the adjacent grade. Lateral resistance analyses should be performed after the bridge pier design details are known.

LIMITATIONS

Within the limitations of scope, schedule, and budget, the conclusions and recommendations presented in this letter report were prepared in accordance with generally accepted professional geotechnical and environmental engineering principles and practices in this area at the time this letter report was prepared.

The data presented in this letter report are based on limited survey and phase of design development. It is also based on a limited number of samples. Shannon & Wilson, Inc. is not responsible for conditions or consequences arising from relevant facts that were concealed, withheld, or not fully disclosed at the time the letter report was prepared. We also note that the facts and conditions referenced in this letter report may change over time, and that the facts and conditions set forth here are applicable to the facts and conditions as described only at the time of this letter report. We believe that the conclusions stated here are factual, but no guarantee is made or implied.

This letter report was prepared for the exclusive use of City of Edmonds, and their respective representatives, and in no way guarantees that any agency or its staff will reach the same conclusions as Shannon & Wilson, Inc. This report did not include any evaluation regarding the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air on or below or around the site beyond those discussed in the report. We have prepared the enclosed Appendix C, "Important Information About Your Geotechnical/ Environmental Report," to help you and others in understanding our reports.

Sincerely,

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Subsurface characterization and geologic interpretation was performed by Stephanie A. Williams, L.G.
Geotechnical engineering findings and recommendations were prepared by Martin W. Page, P.E., L.E.G.

SAW:DRC:MWP/mwp

Enc: References
Figure 1 – Vicinity Map
Figure 2 – Willow Creek Restoration Area
Figure 3 – Site and Explorations Plan
Figure 4 – Typical Stream Channel Cross Section, Section A-A', Section B-B', Section C-C', Section D-D' (2 sheets)
Figure 5 – Schematic Soldier Pile Wall
Appendix A – Subsurface Explorations
Appendix B – Geotechnical Laboratory Test Procedures and Results
Appendix C – Important Information About Your Geotechnical/Environmental Report

REFERENCES

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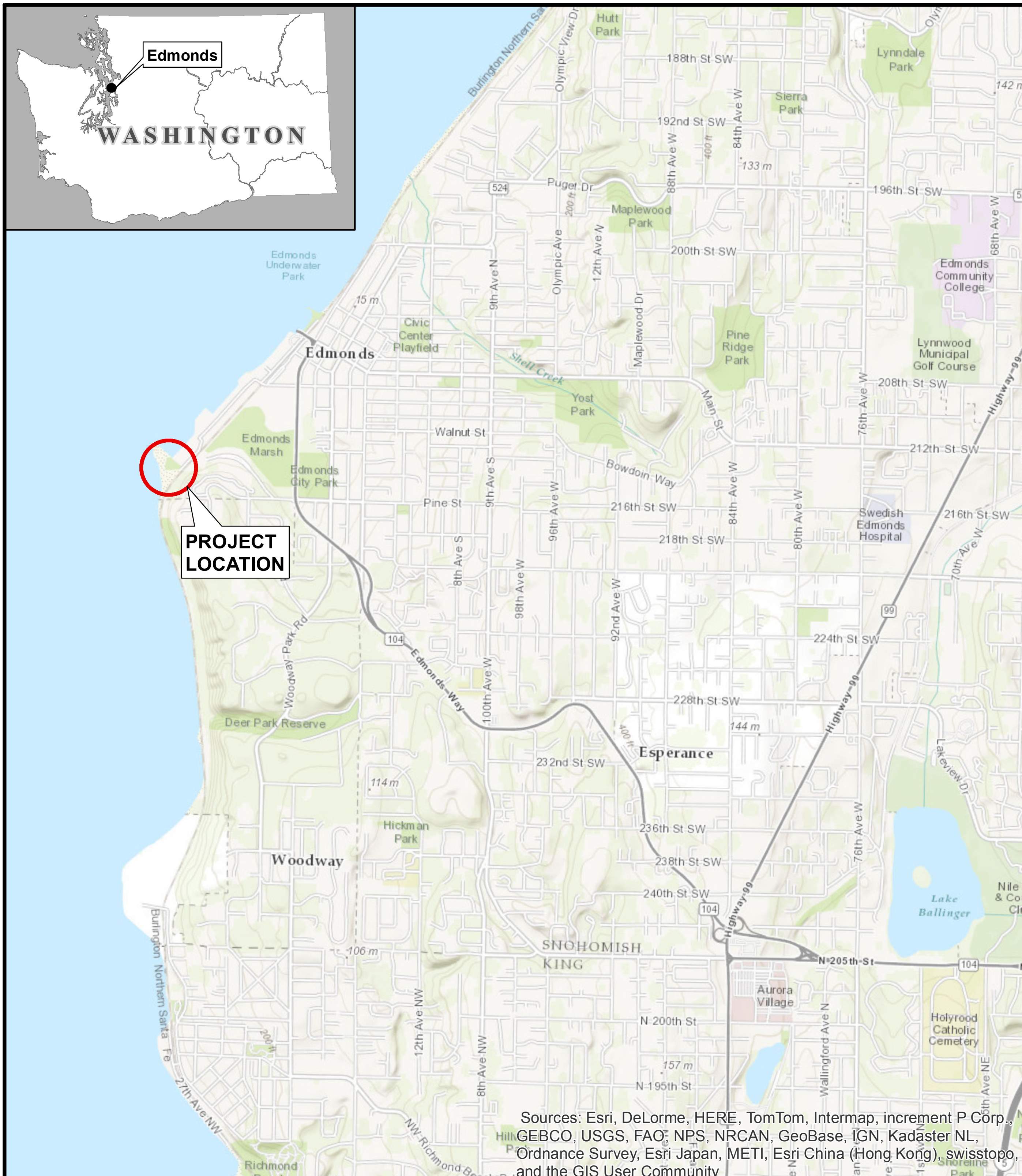

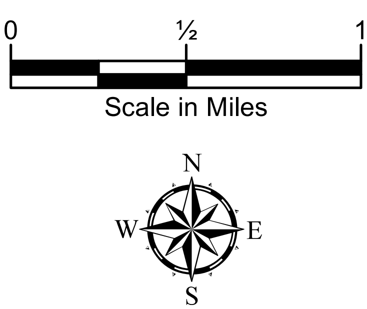

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Edmonds, Washington

VICINITY MAP

November 2014 | 21-1-12393-406

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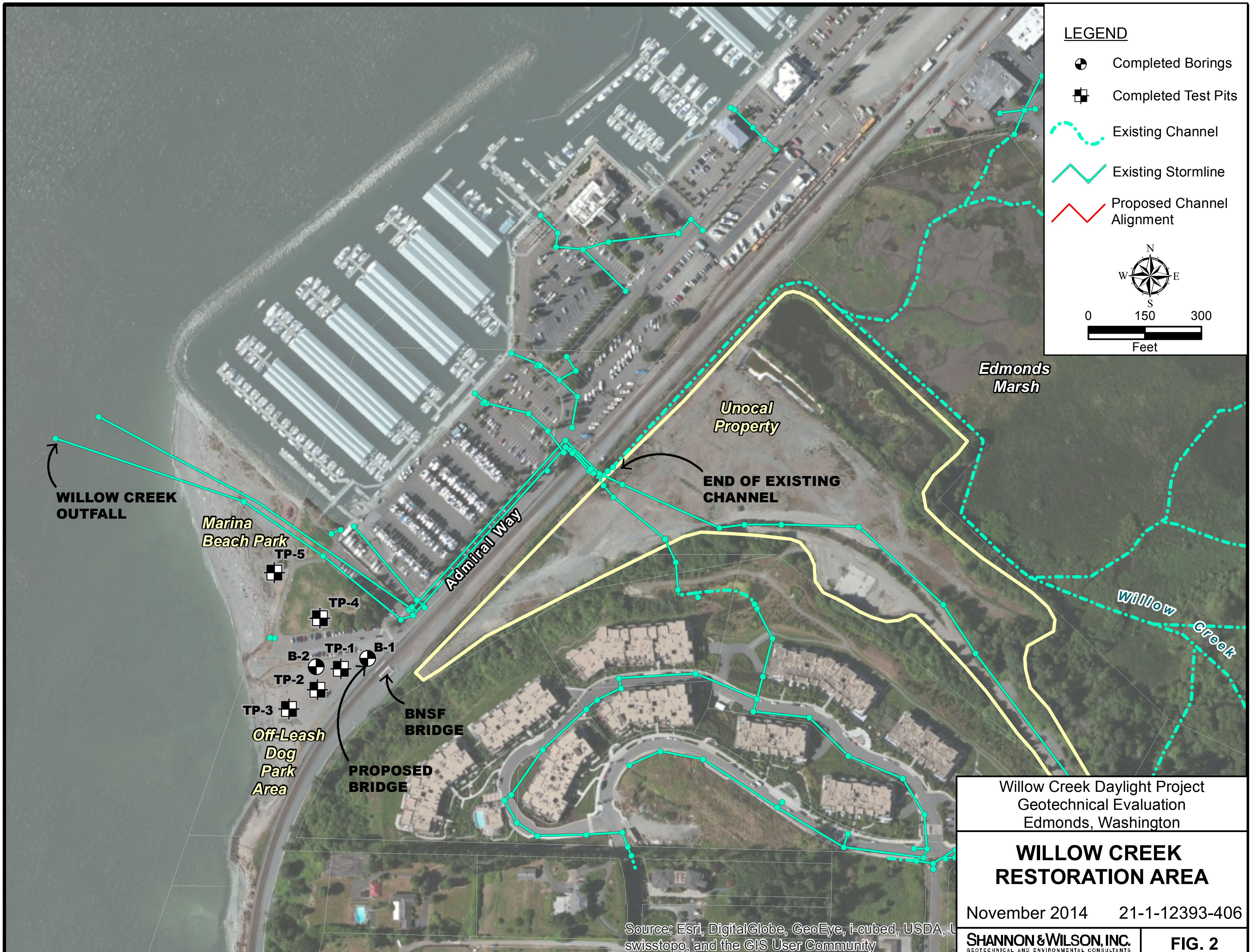
FIG. 1

Filename: I:\WIP\21-1\12393 Willow Creek Daylight\406. GEOTECH\Report\Figures\VicinityMap.mxd Date: 10/1/2014 SAW



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WILLOW CREEK RESTORATION AREA

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FIG. 2

Filename: I:\WIP\21-1\12393 Willow Creek Daylight\406. GEOTECH\Report\Figures\old\ExplorAndSurveyArea.mxd Date: 10/17/2014 SAW

LEGEND

- Completed Boring
- Completed Test Pit
- Borings by Others
- Cross Section
- Proposed Channel Alignment

N W E S

0 100 200

Feet

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SITE AND EXPLORATIONS PLAN

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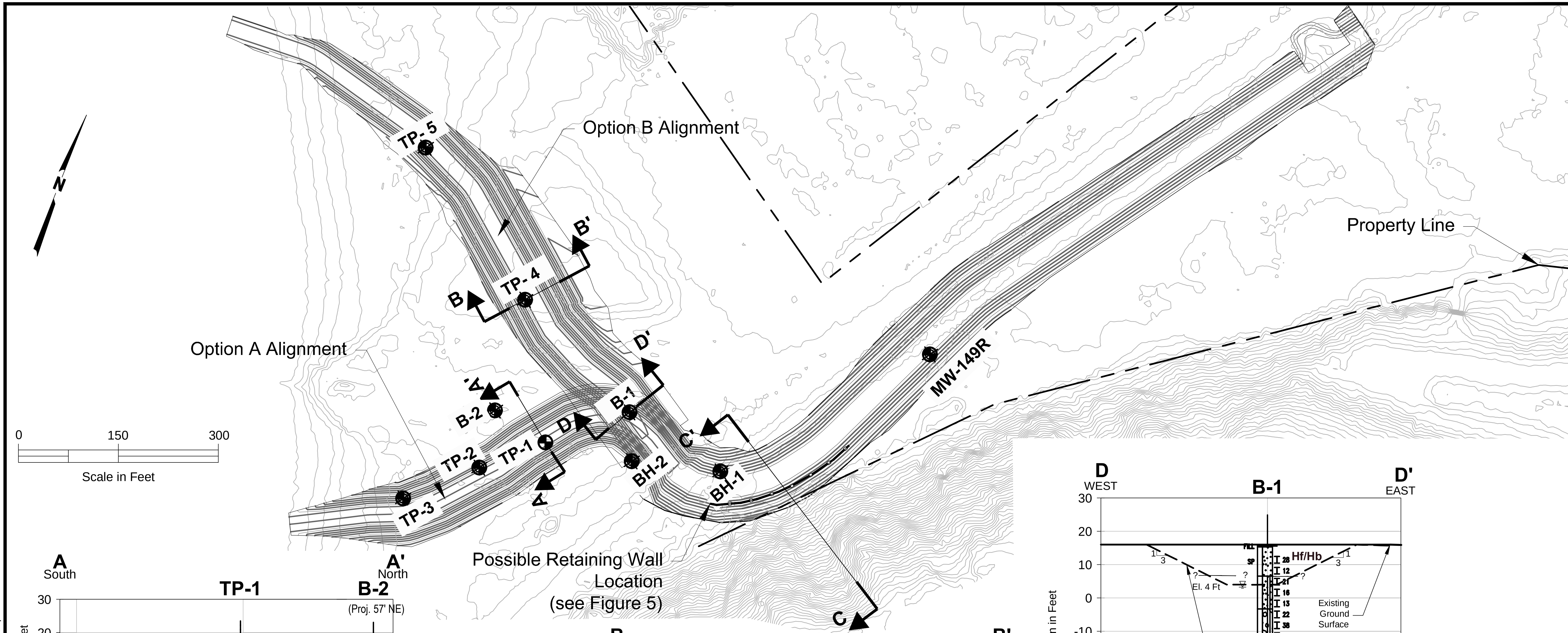
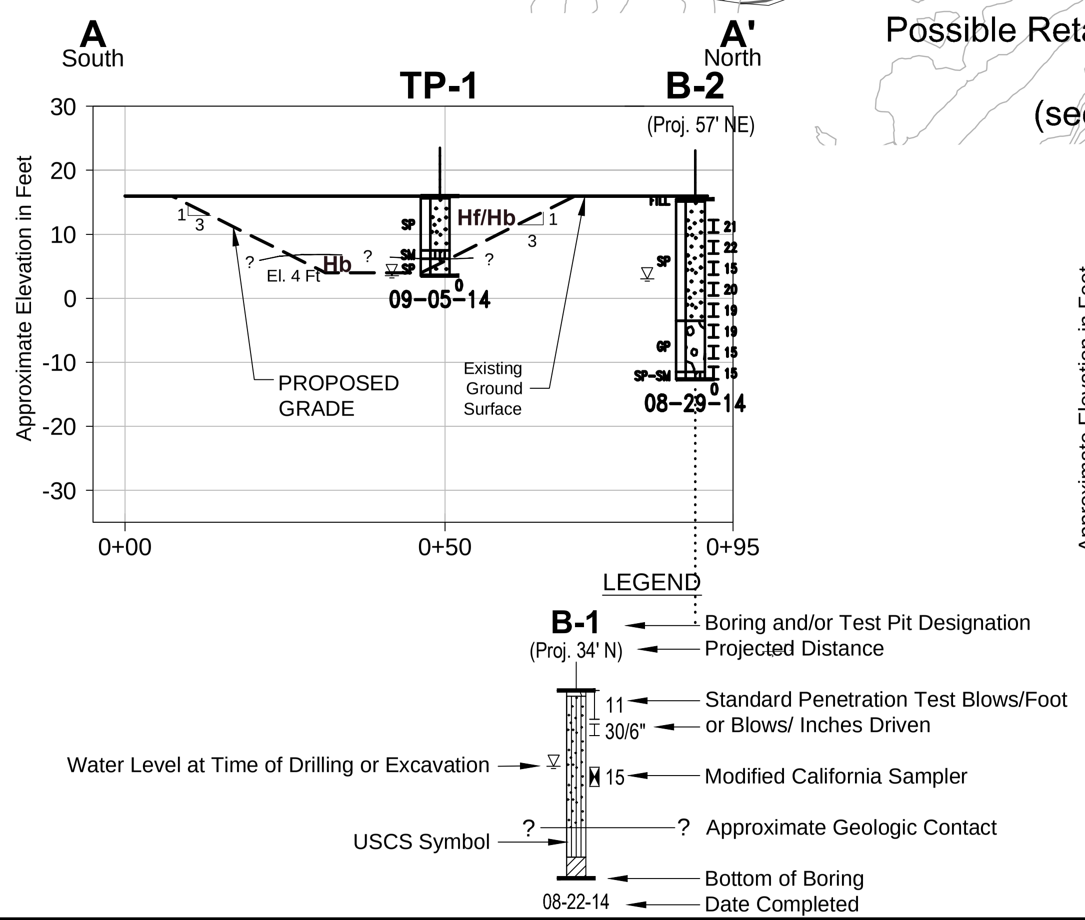
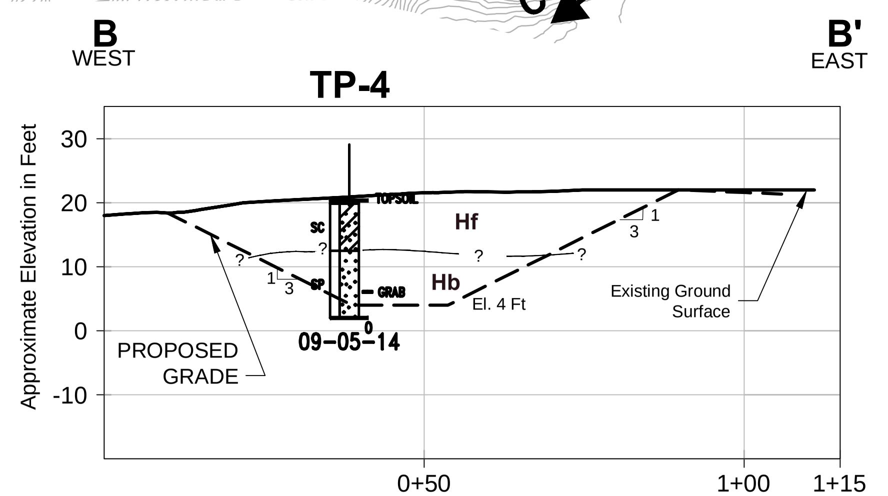
FIG. 3

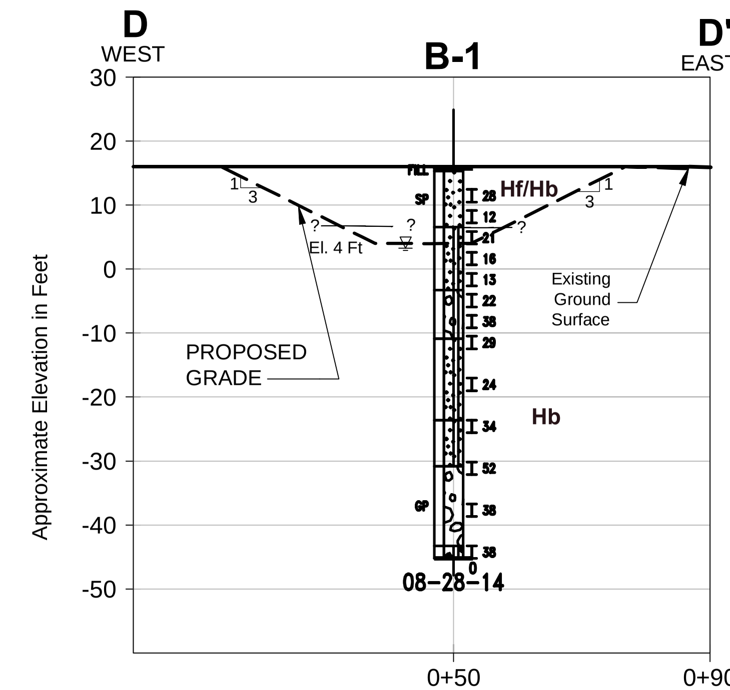
Filename: I:\WIP\21-1\12393 Willow Creek Daylight\406. GEOTECH\Report\Figures\old\Fig3_ExplorationPlan.mxd Date: 10/30/2014 SAW

Filename: J:\211\12393-406\21-1-12393 - 406 Fig 4.dwg Date: 10-28-2014 Login: drtemp









NOTES

1. Ground surface is adapted from Site Survey and LiDAR.
2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
3. See Sheet 2 of Fig 4 for Section C-C'.

0 30 60

Scale in Feet

Vertical = Horizontal

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TYPICAL STREAM CHANNEL CROSS SECTION
SECTION A-A'
SECTION B-B'
SECTION D-D'

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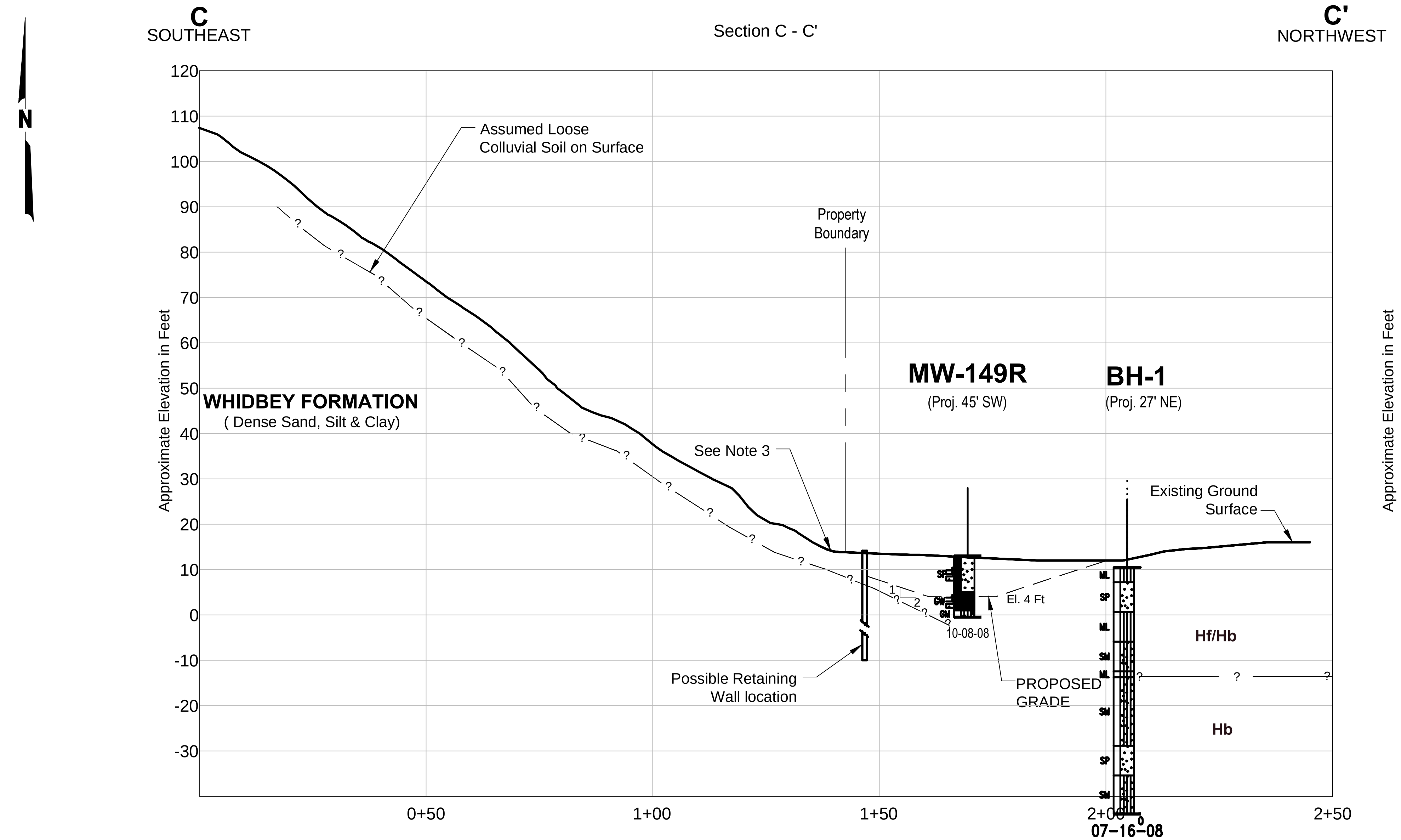
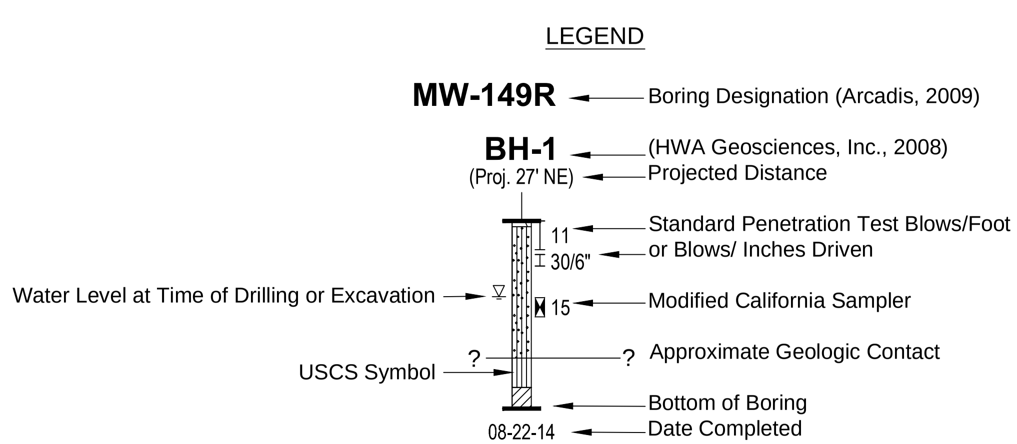
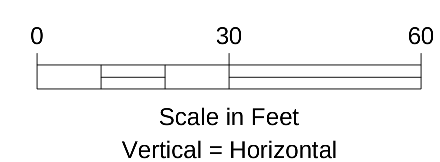
FIG. 4
Sheet 1 of 2







1. Ground surface is adapted from LiDAR.
2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
3. On-site Topographic survey data is not available for this area. Channel location and geometry may be revised. Subsurface conditions are not known in this area.

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TYPICAL STREAM CHANNEL CROSS SECTION

SECTION C-C'

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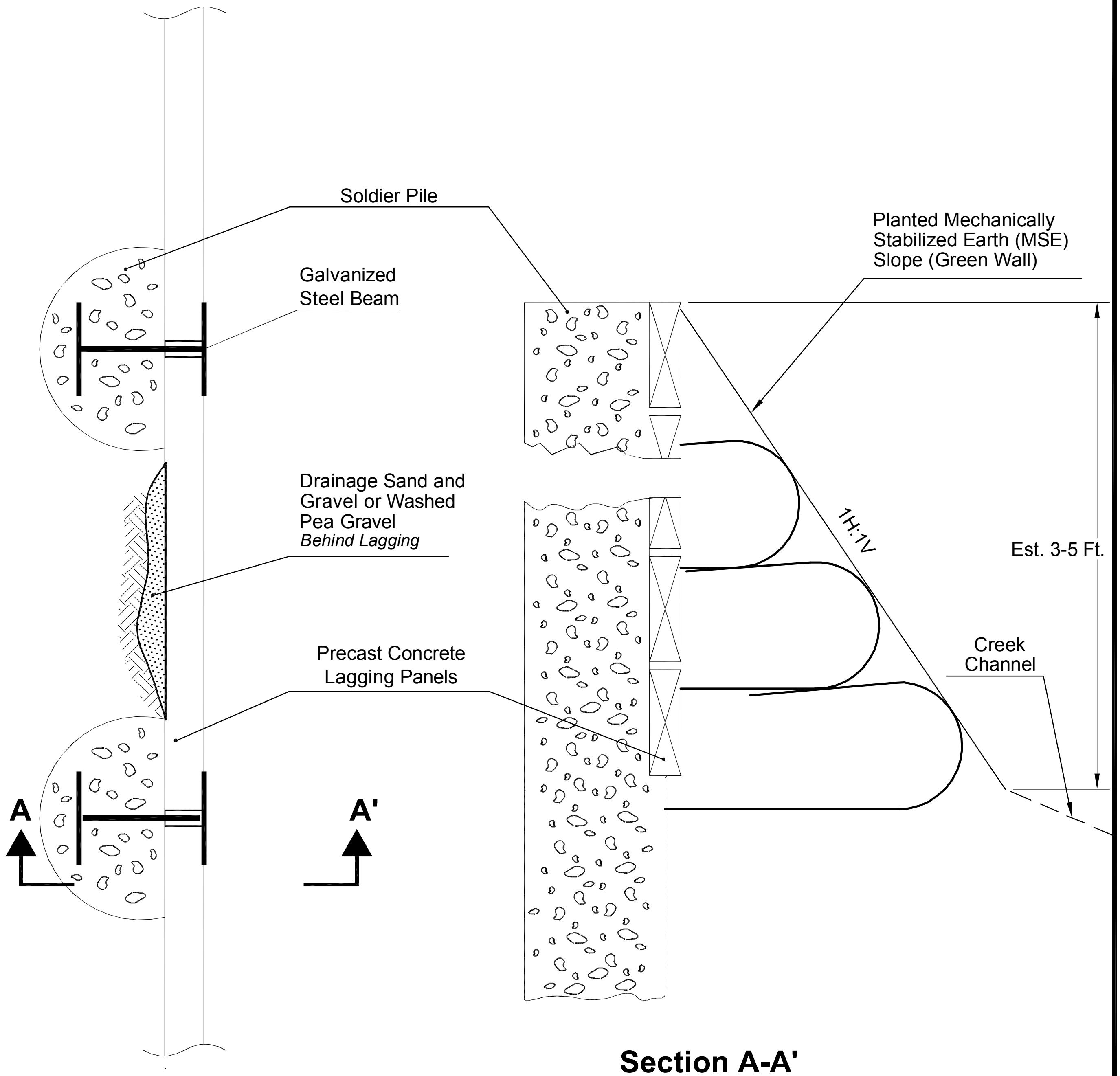
FIG. 4
Sheet 2 of 2

Filename: J:\211\12393-406\21-1-12393 - 406 Fig 4.dwg Date: 10-27-2014 Login: drtemp



Section A-A'

PLAN VIEW

Not to Scale

NOTE

This drawing depicts a retaining wall concept that may be considered during design of the creek channel alignment and geometry between stations 7+00 and 8+00 to avoid excavation into the toe of the steep slope along the property boundary.

Willow Creek Daylight Project Geotechnical Evaluation Edmonds, Washington	
SCHEMATIC SOLDIER PILE WALL	
November 2014	21-1-12393-406
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 5**

File: J:\211\12393-406\21-1-12393-406 Soldier Pile Wall Schematic.dwg Date: 10-24-2014 Author: drtemp

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

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FIGURES

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 INTRODUCTION

To date, the field explorations performed by Shannon & Wilson, Inc. for the proposed Willow Creek Daylight Project have consisted of drilling and sampling two borings and excavating five test pits between August 28 and September 5, 2014. The borings were drilled using mud rotary drilling techniques and sampled using a 2-inch-diameter split-spoon and Standard Penetration Test (SPT). Boring B-1 was drilled to a depth of 45 feet and sampled to 46.5 feet below ground surface (bgs). Boring B-2 was drilled to a depth of 20 feet and sampled to 21.5feet. Driven soil samples were obtained generally at 2.5-foot intervals to 20 feet, then in 5-foot intervals. Five test pits were excavated to depths of between 8 and 14 feet bgs.

Approximate locations of the explorations performed at the project site are shown in Figure 2, Site and Exploration Plan. The exploration locations were recorded with a Trimble Global Positioning System device. A Soil Description and Log Key is presented in Figure A-1 as a reference for symbols and information presented on the boring logs. The logs of the explorations are presented as Figures A-2 through A-8.

A.2 EXPLORATIONS

A.2.1 Mud Rotary Drilling

Mud rotary borings are advanced by spinning a tri-cone bit attached to a string of drilling rods. Drilling mud consisting of water and bentonite or a biodegradable synthetic thickening agent is pumped out of a tank at the ground surface, down the drill rods and the tri-cone bit, up the annulus, and back into the mud tank. The circulation of drilling mud removes the cuttings generated during the drilling process from the hole and carries them to the surface, where they are screened and removed from the recirculating fluid. The drilling fluid also maintains the integrity of the borehole, thereby reducing caving or collapsing during drilling and sampling.

A.2.2 Test Pit Exavations

Test pits were excavated by Clear Creek Contractors, Inc. using a Hitachi ZAxis 75 Excavator. Contractors backfilled the test pits using the excavated material in approximately the same order it was removed from the hole.

A.3 SAMPLING

Disturbed soil samples were retrieved from the borehole and test pits locations. Disturbed soil samples from the boring were obtained by a split-spoon sampler in conjunction with an SPT and using the sonic core barrel. Grab samples were obtained from the test pits locations. The intervals where these samples were collected are shown on the boring log and test pit logs included in the Appendix A figures. Specific sampling procedures are described below.

A.3.1 Split-spoon Soil Samples

To obtain disturbed soil samples from the borings, SPTs were performed in general accordance with the ASTM International (ASTM) Designation: D1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM, 2009). The SPTs were generally performed at 5-foot intervals in between sonic core runs. After performing the SPT, the sampler was brought to the ground surface and soil collected inside the barrel was examined and logged by a Shannon & Wilson, Inc. geologist. The split-spoon samples collected from the borings were placed in plastic jars with screw lids for further review and testing.

A.3.2 Grab Samples

Grab samples were collected during test pit excavation from each location. Grab samples from soil layers within the test pits were collected from the backhoe bucket or spoil pile by a Shannon & Wilson, Inc. representative. Soil samples were collected in labeled plastic jars and 5-gallon plastic bags, sealed, and transported to our laboratory for further analyses and testing. Grab samples were also collected from specific depths within the sonic core during the review process. The grab samples collected during the sonic core review process are collected in the sample manner as grab samples collected on-site.

A Shannon & Wilson, Inc. representative was present throughout the drilling and test pit procedures to collect soil samples, visually classify the samples, and to prepare an exploration log for the boring and each test pit. After classification, representative soil samples were sealed to help preserve the natural moisture content of the soil and returned to our laboratory in Seattle, Washington, for analyses.

A.4 PENETRATION TEST

To obtain disturbed soil samples, SPTs are performed in general accordance with ASTM Designation: D1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM, 2009). The SPT consists of a 2-inch outside-diameter, 1.375-inch inside-diameter, split-spoon sampler driven 18 inches into the bottom of the borehole with a 140-pound hammer free falling 30 inches. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). Generally, when penetration resistances exceed 50 or more blows for 6 inches or less of penetration, the test is terminated, and the number of blows and corresponding penetration distance recorded. The SPT N-value is a useful parameter for estimating the relative density or consistency of the soil. This value is commonly used in engineering analyses to estimate soil strength and other characteristics.

The penetration resistances were recorded by our field representative and are plotted on the boring logs. These values are empirical parameters that provide a means of evaluating the relative density or compactness of cohesionless (granular) soils and the relative consistency (stiffness) of cohesive soils. The terminology used to describe the relative density or consistency of the soils is presented in Figure A-1.

The split-spoon sampler used during the penetration testing recovers a disturbed sample of the soil, which is useful for identification and classification purposes. The samples were classified and recorded on field logs by our geologist. The samples were sealed in jars and returned to our laboratory for testing.

A.5 EXPLORATION LOGS

Field exploration logs were prepared by our field representative for each exploration to record the encountered subsurface conditions at that time. Pertinent information, including depths, stratigraphy, engineering characteristics, and groundwater occurrence, were recorded. The summary boring logs and test pit logs presented in this report represent our interpretation of the field exploration log or test pit, and are a written record of the subsurface conditions encountered in the boring at the time of exploration, where applicable. It graphically shows the geologic units (layers) encountered in the boring and the Unified Soil Classification System symbol of each geologic layer. The stratigraphic contacts indicated on the summary logs represent the approximate boundaries between soil or rock types at those locations. The subsurface conditions were those recorded at the time of drilling, and may not necessarily represent those at other times and locations.

A.6 REFERENCE

ASTM International (ASTM), 2009, Annual book of ASTM standards, West Conshohocken, Pa.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Perforated or Screened Casing	Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

SOIL DESCRIPTION AND LOG KEY

November 2014 21-1-12393-406

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 1 of 3

2013_BORING_CLASS1 21-20699.GPJ SHAN_WIL.GDT 4/22/14

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW		Well-Graded Gravel; Well-Graded Gravel with Sand
			GP		Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM		Silty Gravel; Silty Gravel with Sand
			GC		Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW		Well-Graded Sand; Well-Graded Sand with Gravel
			SP		Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM		Silty Sand; Silty Sand with Gravel
			SC		Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL		Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH		Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH		Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH		Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat or other highly organic soils (see ASTM D4427)

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

SOIL DESCRIPTION AND LOG KEY

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FIG. A-1
Sheet 2 of 3

2013_BORING_CLASS2 21-20699.GPJ SHAN_WIL.GDT 4/22/14

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASTICITY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20
High	It takes considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

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2013_BORING_CLASS3 21-20699.GPJ SHAN_WIL.GDT 4/22/14

Willow Creek Daylight Project
Geotechnical Evaluation
Everett, Washington

SOIL DESCRIPTION AND LOG KEY

November 2014 21-1-12393-406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 3 of 3

Total Depth:	46.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	12 in.
Top Elevation:	~ 15.5 ft.	Easting:		Drilling Company:	Holt	Rod Diam.:	2-5/8" O.D.
Vert. Datum:		Station:		Drill Rig Equipment:	LA Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

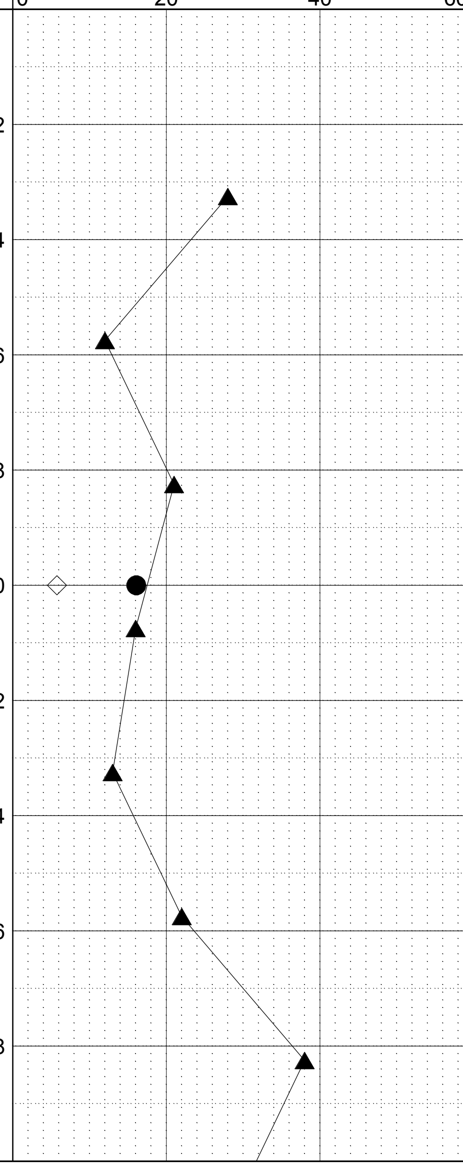
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	PID, ppm	Samples	Ground Water
Gray, chipped gravel over compacted sand and gravel.	0.3			
Medium dense, gray, *Poorly Graded Sand with Gravel (SP)*; moist; some fine to coarse, subangular to rounded gravel; fine to coarse sand; trace fines. - Sand becoming finer below 5 feet.			1 2	
Medium dense, gray to gray-brown, *Poorly Graded Sand with Silt (SP-SM)*; moist to wet, becoming wet below 9.5 feet; few fine, subrounded gravel; mostly fine to medium sand. - Groundwater assumed to be about 9.5 feet because the 10-foot sample was saturated. - Becoming more gravelly below 12.5 feet.	7.0	0 0	3 4 5	During Drilling
Medium dense to dense, gray to brown, *Poorly Graded Gravel with Silt and Sand (GP-GM)*; wet; fine to coarse, subangular to rounded gravel, mostly coarse gravel; fine to coarse sand. Fines content may be over estimated because of drilling fluid in samples S-6 and S-7. - Trace wood fragments noted by driller at 19 feet.	14.5		6 7	

CONTINUED NEXT SHEET

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



◇ % Fines (<0.075mm)

● % Water Content

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

LOG OF BORING B-1

November 2014 21-1-12393-406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2
Sheet 1 of 3

REV 3 - Approved for Submittal

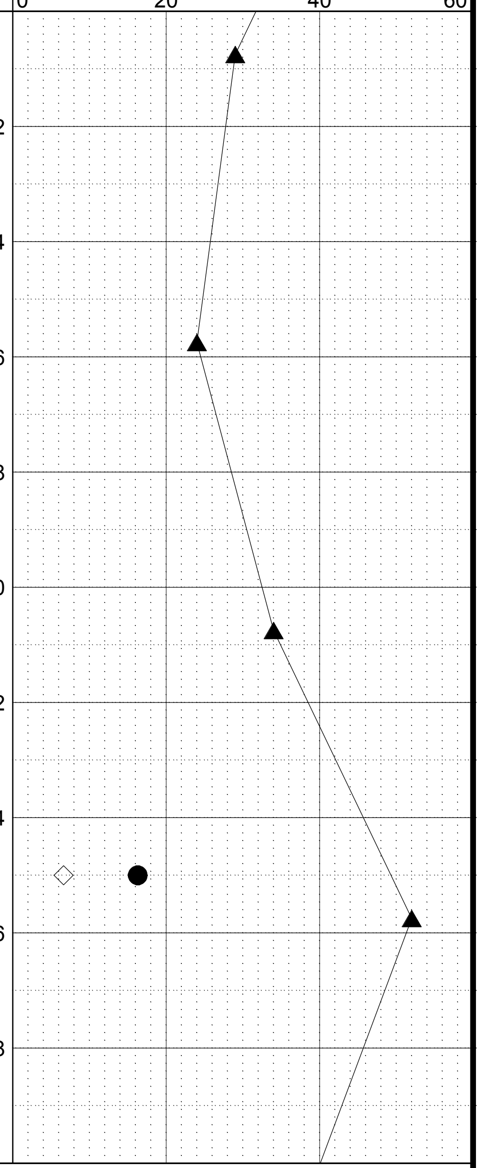
Total Depth:	46.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	12 in.
Top Elevation:	~ 15.5 ft.	Easting:		Drilling Company:	Holt	Rod Diam.:	2-5/8" O.D.
Vert. Datum:		Station:		Drill Rig Equipment:	LA Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	PID, ppm	Samples	Ground Water	Depth, ft.
Medium dense, dark gray, *Poorly Graded Sand with Silt (SP-SM)*; wet; few fine to coarse, subrounded to rounded gravel; fine to medium sand.	20.3			8		
						22
						24
				9		26
						28
Dense, dark gray, *Poorly Graded Sand with Silt (SP-SM)*, little fine, subrounded to rounded gravel; some fine to coarse sand, trace wood fragments.	30.0			10 *		30
						32
						34
Dense to very dense, dark gray, *Poorly Graded Gravel with Sand (GP)*; wet; fine to coarse, subrounded to rounded gravel; some fine to coarse sand; trace fines.	35.5			11		36
						38

CONTINUED NEXT SHEET

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

⌶ 2.0" O.D. Split Spoon Sample

⊻ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

LOG OF BORING B-1

November 2014 21-1-12393-406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

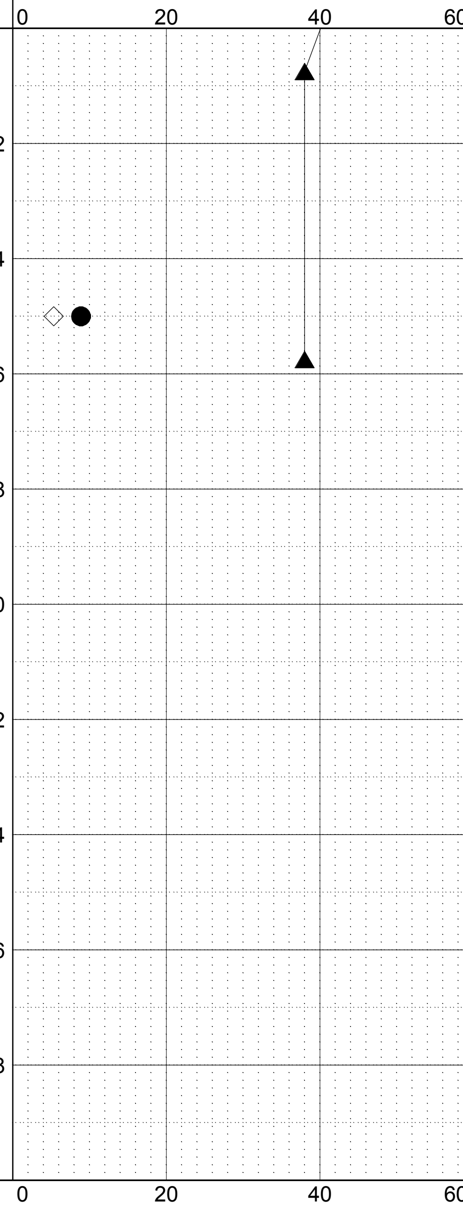
FIG. A-2
Sheet 2 of 3

REV 3 - Approved for Submittal

MASTER_LOG_E 21-12393.GPJ SHAN_WIL.GDT 10/20/14 Log: SAW Rev: JKP Typ: CLP

Total Depth:	46.5 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	12 in.
Top Elevation:	~ 15.5 ft.	Easting:		Drilling Company:	Holt	Rod Diam.:	2-5/8" O.D.
Vert. Datum:		Station:		Drill Rig Equipment:	LA Rig	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	PID, ppm	Samples	Ground Water	Depth, ft.
- Fine to medium sand interbed with little fines from 40 to 40.8 feet. - Wood fragments around 40 feet.				12		42, 44
Dense, gray to gray-brown, *Well-Graded Gravel with Silt and Sand (GW-GM)*; wet; fine to coarse, mostly fine, subangular to subrounded gravel; some fine to coarse sand.	45.0 46.5			13		46
BOTTOM OF BORING COMPLETED 8/28/2014						48, 50, 52, 54, 56, 58



LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

LOG OF BORING B-1

November 2014 | 21-1-12393-406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2
Sheet 3 of 3

REV 3 - Approved for Submittal

Total Depth:	*21.5 ft.*	Northing:		Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*~ 15.5 ft.*	Easting:		Drilling Company:	*Holt*	Rod Diam.:	*2-5/8" O.D.*
Vert. Datum:		Station:		Drill Rig Equipment:	*LA Rig*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

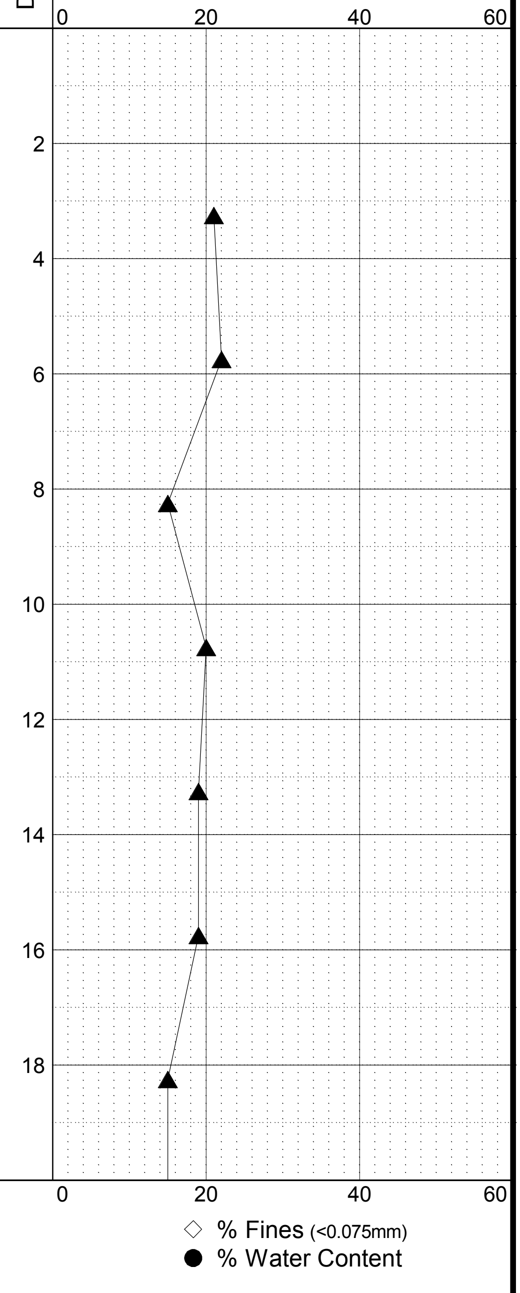
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	PID, ppm	Samples	Ground Water
Gravel chip over compacted sand and gravel.	0.3			
Medium dense, gray, *Poorly Graded Sand with Gravel (SP)*; moist to wet; some fine to coarse, subrounded and broken to rounded gravel; fine to coarse sand; trace fines. Beach Sand or Fill.			1	
- More coarse gravel from 5 to 6.5 feet.			2	
		0	3	
			4	During Drilling ∇
- Finer gravel from 12.5 feet.			5	
Medium dense, gray, *Poorly Graded Gravel with Sand (GP)*; wet; fine to coarse, subrounded to rounded gravel, mostly fine gravel; little fine to coarse sand; trace fines.	14.5		6	
			7	

CONTINUED NEXT SHEET

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

⌶ 2.0" O.D. Split Spoon Sample

∇ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

LOG OF BORING B-2

November 2014 | 21-1-12393-406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3
Sheet 1 of 2

REV 3 - Approved for Submittal

MASTER_LOG_E 21-12393.GPJ SHAN_WIL.GDT 10/20/14 Log: SAW Rev: JKP Typ: CLP

Total Depth: 21.5 ft.	Northing:	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation: ~ 15.5 ft.	Easting:	Drilling Company:	Holt	Rod Diam.:	2-5/8" O.D.
Vert. Datum:	Station:	Drill Rig Equipment:	LA Rig	Hammer Type:	Automatic
Horiz. Datum:	Offset:	Other Comments:			

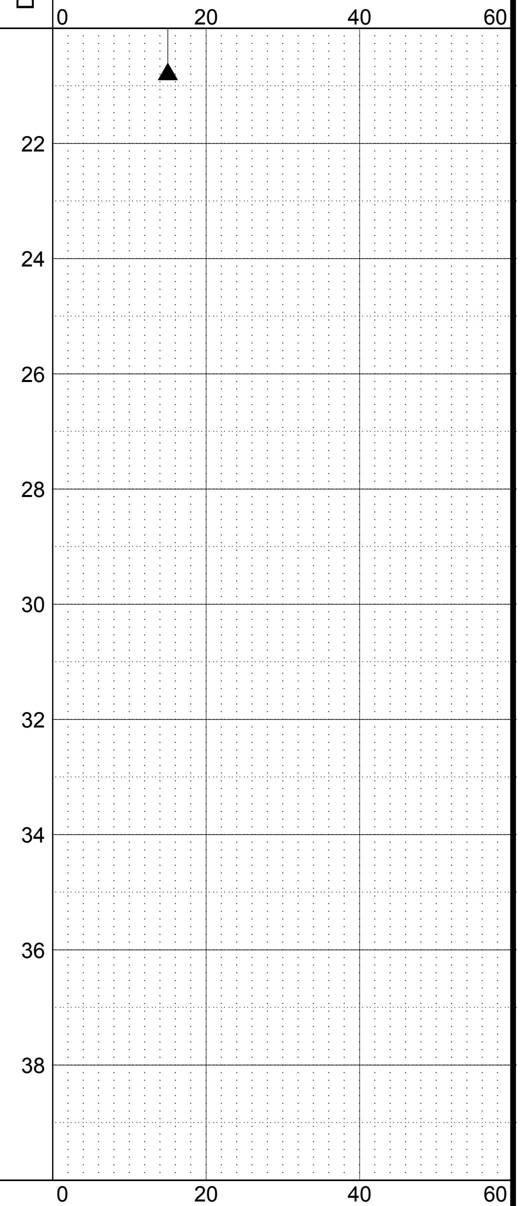
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.	Symbol	PID, ppm	Samples	Ground Water
- Becoming fine gravel and coarse sand below 20 feet.	20.6			8	
Medium dense, gray, *Poorly Graded Sand with Silt and Gravel (SP-SM)*; wet; fine to coarse, subrounded gravel; mostly fine to coarse sand.	21.5				
BOTTOM OF BORING COMPLETED 8/29/2014					

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



◇ % Fines (<0.075mm)

● % Water Content

LEGEND

* Sample Not Recovered

⌶ 2.0" O.D. Split Spoon Sample

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

LOG OF BORING B-2

November 2014 21-1-12393-406

SHANNON & WILSON, INC.
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FIG. A-3
Sheet 2 of 2

REV 3 - Approved for Submittal

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-1

JOB NO: 21-1-12393-406 DATE: 9-5-2014 LOCATION: See Site and Exploration Plan

PROJECT: Willow Creek Daylight Project, Geotechnical Evaluation

Sketch of Southeast Pit Side

Surface Elevation: Approx. 15.5 Ft.

SOIL DESCRIPTION

1. Gray, chipped gravel over Compacted Sand and Gravel.

2. Brown to gray-brown, *Poorly Graded Sand with Gravel (SP)*; moist; fine to coarse subangular to rounded gravel; fine to coarse sand; trace fines; interbedded with 1- to 2-inch thick fine to coarse gravel layers from 3 to 5 feet. Unit fining downward to fine to medium sand and trace shell below 5 ft.

3. Dark gray to brown with red-brown to yellow-brown locally, *Silty Sand (SM)*; moist; laminated with 1/8- to 1/4-inch thick *Sandy Silt (ML)*; moist; 1/4-inch thick sand layers consist of mostly fine sand; fibrous organics and mica minerals in silt layers; little wood debris throughout unit.

4. Gray, Poorly Graded Sand with Gravel (SP), moist; some fine to coarse, subangular to rounded gravel; fine to medium sand; trace fines.

Ground Water: 9.5 ft. ▽

% Water Content

Samples: G-1, G-2, G-3

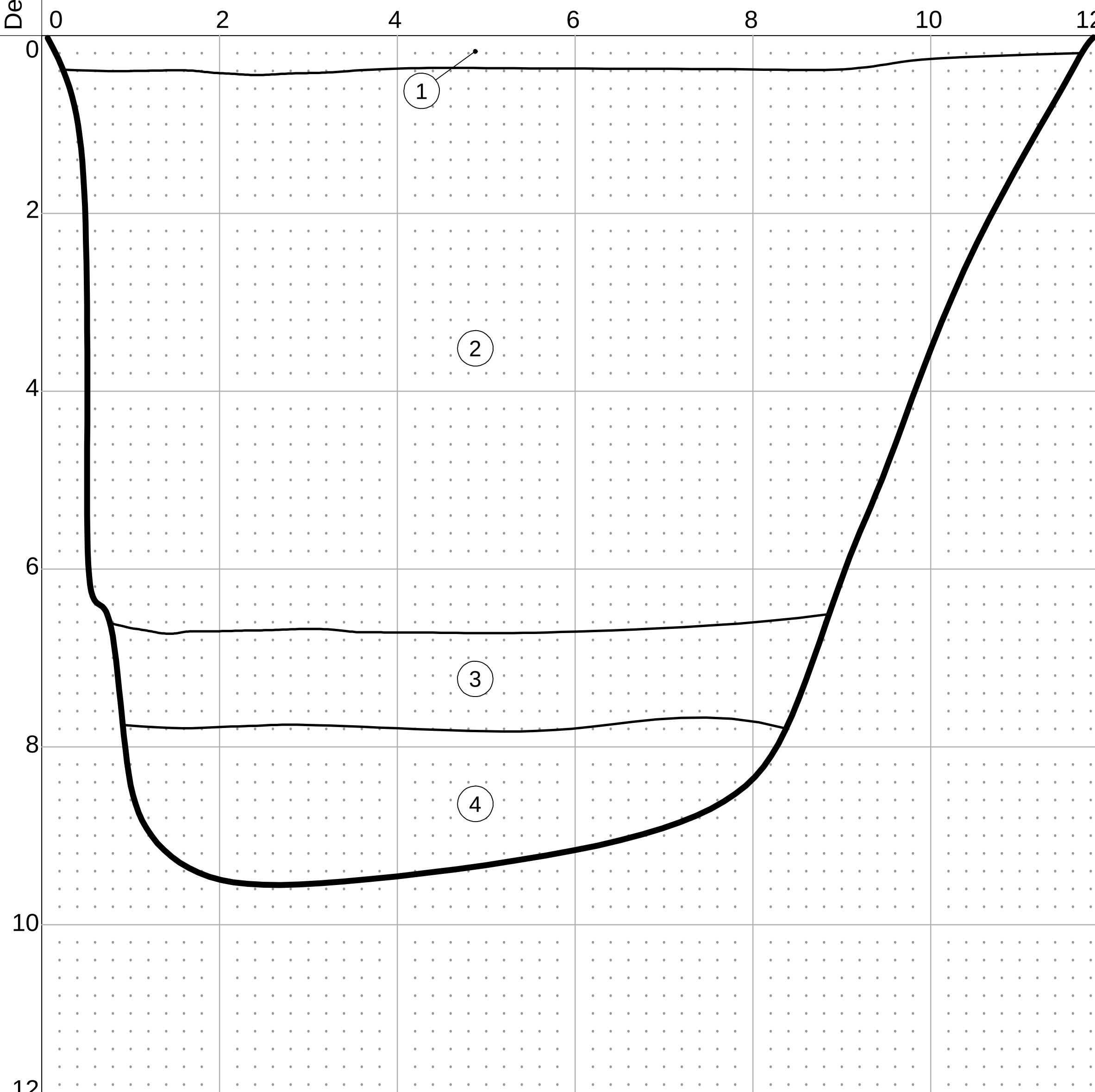


FIG. A-4

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-2

JOB NO: 21-1-12393-406 DATE: 9-5-2014 LOCATION: See Site and Exploration Plan

PROJECT: Willow Creek Daylight Project, Geotechnical Evaluation

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Gray, chipped gravel over compacted sand and gravel.	None Observed			0
(2) Brown to gray-brown, *Poorly Graded Sand with Gravel (SP);* moist; little fine to coarse, subangular to rounded gravel; fine to coarse sand; trace fines. • Sand cross bedded locally from 0.5 to 2.5 feet.				2, 4
(3) Dark gray to brown with red-brown to yellow-brown locally, *Silty Sand (SM)*; moist; laminated with 1/8- to 1/4-inch thick *Sandy Silt (ML)*; moist; 1/4-inch thick sand layers consist of mostly fine sand; fibrous organics and mica minerals in silt layers; little wood debris throughout unit; trace metal debris and iron oxide staining found at top of unit.		3.1	G-1	6
(4) Gray, *Poorly Graded Sand with Gravel (SP)*; moist; some fine to coarse, subangular to rounded gravel; fine to medium sand; trace fines.			G-2	8, 10, 12

Sketch of Southeast Pit Side Surface Elevation: Approx. 16 Ft.

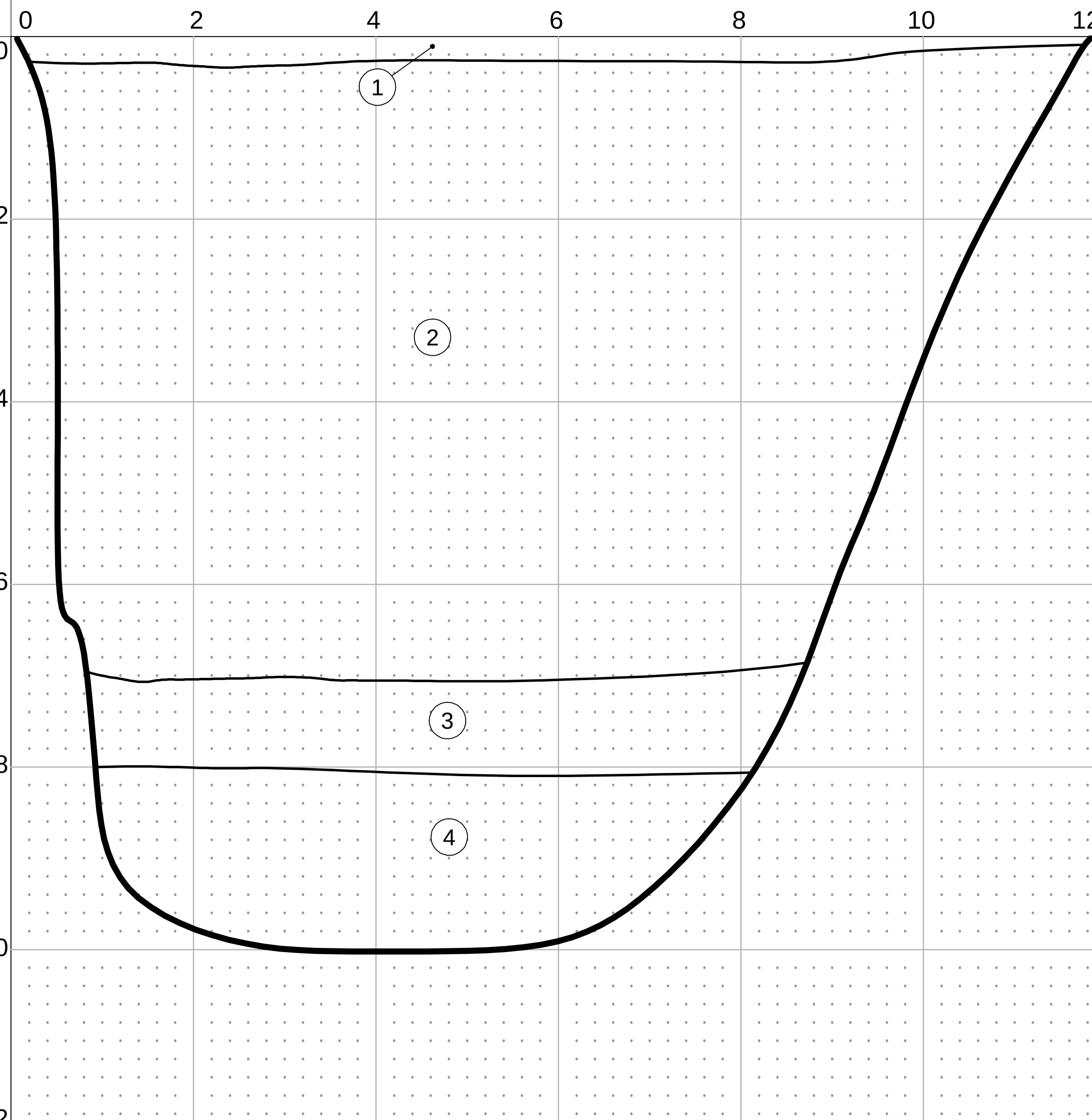


FIG. A-5

Filename: J:\211\12393-406\21-1-12393-406 TPs.dwg Date: 10-21-2014 Login: drtemp

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-3

JOB NO: 21-1-12393-406 DATE: 9-5-2014 LOCATION: See Site and Exploration Plan

PROJECT: Willow Creek Daylight Project, Geotechnical Evaluation

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Gray, chipped gravel.	None Observed			0
(2) Brown, *Poorly Graded Sand with Gravel (SP)*; moist; little fine to coarse, subangular to rounded gravel; fine to coarse sand; trace fines; interbedded with 1- to 2-inch thick fine to coarse gravel about every 1/2 foot from 2.5 to 5 feet.			G-1	2, 4
(3) Dark gray to brown with red-brown to yellow-brown locally, *Silty Sand (SM)*; moist; laminated with 1/8- to 1/4-inch thick *Sandy Silt (ML)*; moist; 1/4-inch thick sand layers consist of mostly fine sand; fibrous organics and mica minerals in silt layers; little wood debris throughout unit; trace metal debris and iron oxide staining found at top of unit.			G-2	6, 8
(4) Gray, *Poorly Graded Sand with Gravel (SP);* moist; little fine to coarse, subangular to rounded gravel; fine to medium sand; trace fines.				10, 12

Sketch of West Pit Side

Surface Elevation: Approx. 15.5 Ft.

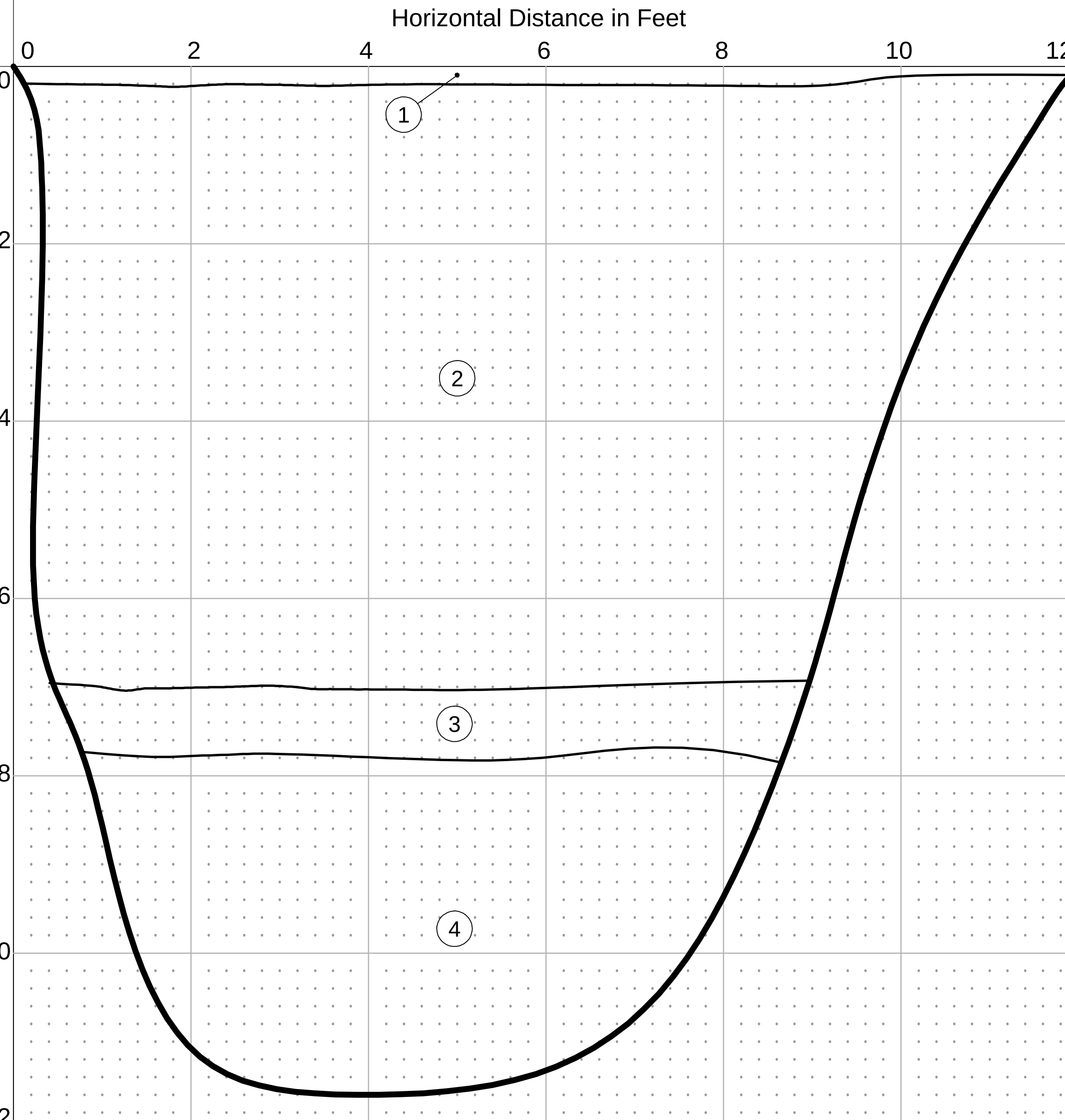


FIG. A-6

Filename: J:\211\12393-406\21-1-12393-406 TPs.dwg Date: 10-21-2014 Login: drtemp

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-4

JOB NO: 21-1-12393-406 DATE: 9-5-2014 LOCATION: See Site and Exploration Plan

PROJECT: Willow Creek Daylight Project, Geotechnical Evaluation

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Grass sod and brown top soil.	None Observed			0
(2) Brown to gray-brown, *Silty Sand with Gravel and Cobbles (SM) to Clayey Sand with Gravel and Cobbles (SC);* moist; few 3- to 4-inch cobbles; little fine to coarse, subangular to rounded gravel; fine to coarse sand. Fill, possibly from a glacial till source.				3
(3) Gray to gray-brown, *Poorly Graded Sand with Gravel (SP);* moist; some fine to coarse, subrounded to rounded gravel; fine to coarse sand; trace fines; 1- to 3-inch thick fine to coarse gravel interbeds from 7 to 10 feet. • Trace shell fragments below 10 feet. • 1 broken brick found at about 10 feet.			G-1	6
			G-2	9
		2.8	G-3	
				12
			G-4	
				15
				18

Sketch of West Pit Side Surface Elevation: Approx. 20 Ft.

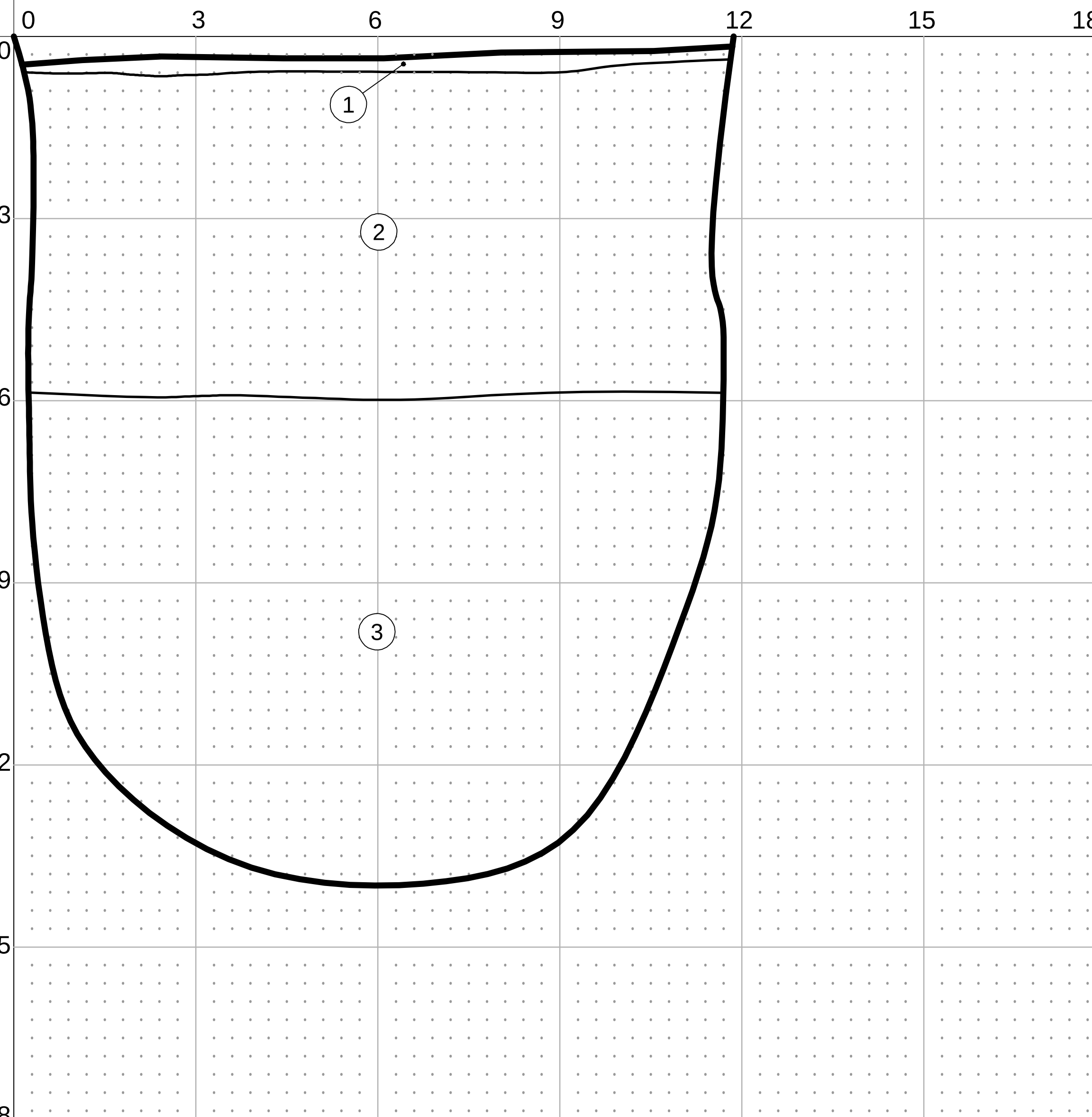


FIG. A-7

Filename: J:\211\12393-406\21-1-12393-406 TPs.dwg   Date: 10-21-2014   Login: drtemp

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-5

JOB NO: 21-1-12393-406   DATE: 9-5-2014   LOCATION: See Site and Exploration Plan

PROJECT: Willow Creek Daylight Project, Geotechnical Evaluation

Sketch of South Pit Side   Surface Elevation: Approx. 11.5 Ft.

SOIL DESCRIPTION

1. Gray to gray-brown, *Poorly Graded Sand with Gravel (SP);* moist, little fine to coarse, subangular to rounded gravel; fine to coarse sand; trace fines; shell fragments; interbedded with 2- to 3-inch thick fine gravel and coarse sand layers about every 1/2 foot from 2.5 to 5.5 feet.

2. Dark gray to brown with red-brown to yellow-brown locally, *Silty Sand (SM)*; moist; laminated with 1/8- to 1/4-inch thick *Sandy Silt (ML)*; moist; 1/4-inch thick sand layers consist of mostly fine sand; fibrous organics and mica minerals in silt layers; little wood debris throughout unit; trace iron oxide staining in sand.

3. Gray, *Poorly Graded Sand with Gravel (SP);* moist, wet below 8 feet; little fine to coarse, subangular to rounded gravel; fine to coarse sand; trace fines.

Ground Water	% Water Content	Samples	Depth, Ft.
		G-1	~5.1
		G-2	~6.1
8.0 ft. ▽		G-3	~8.1

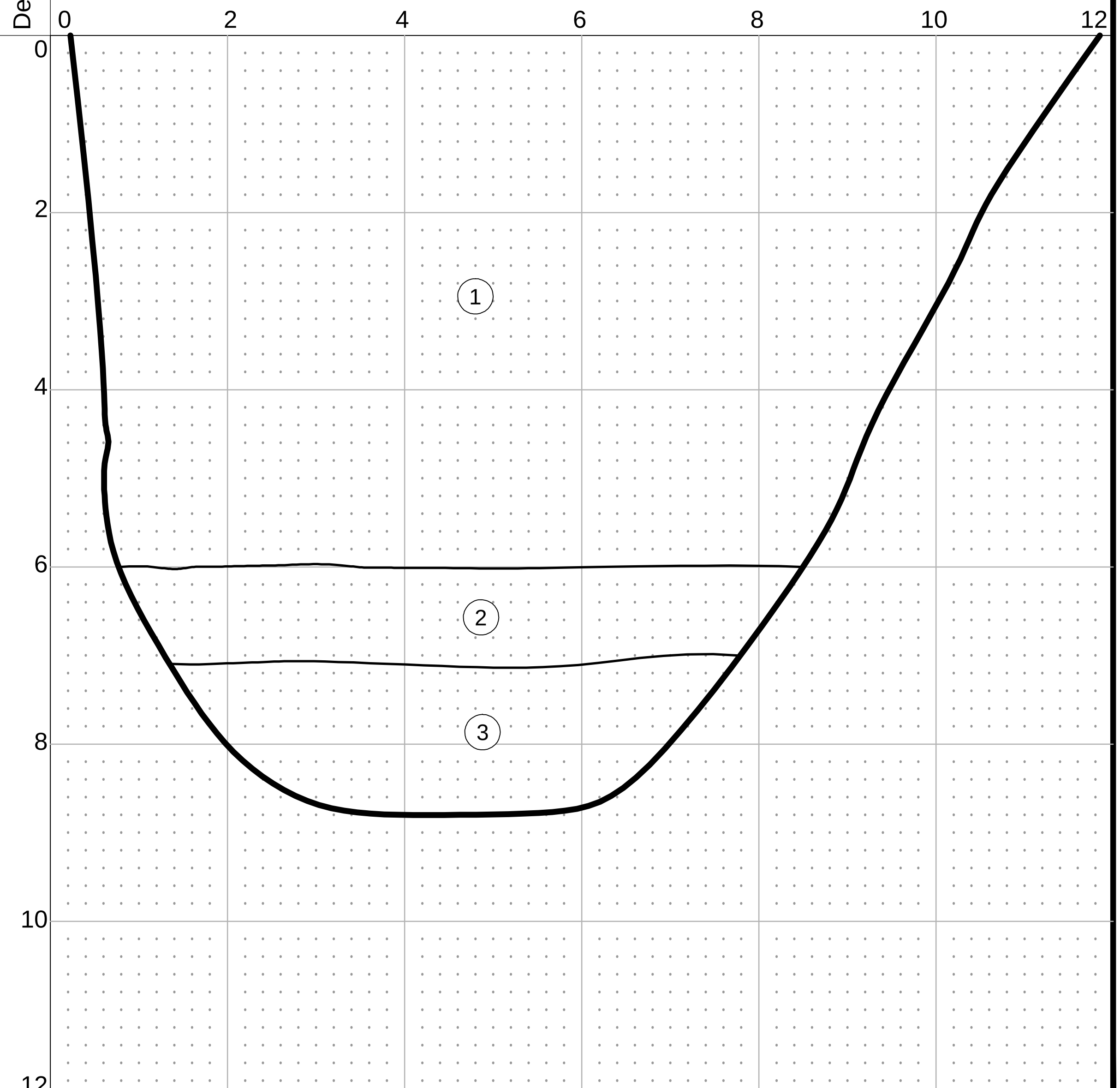
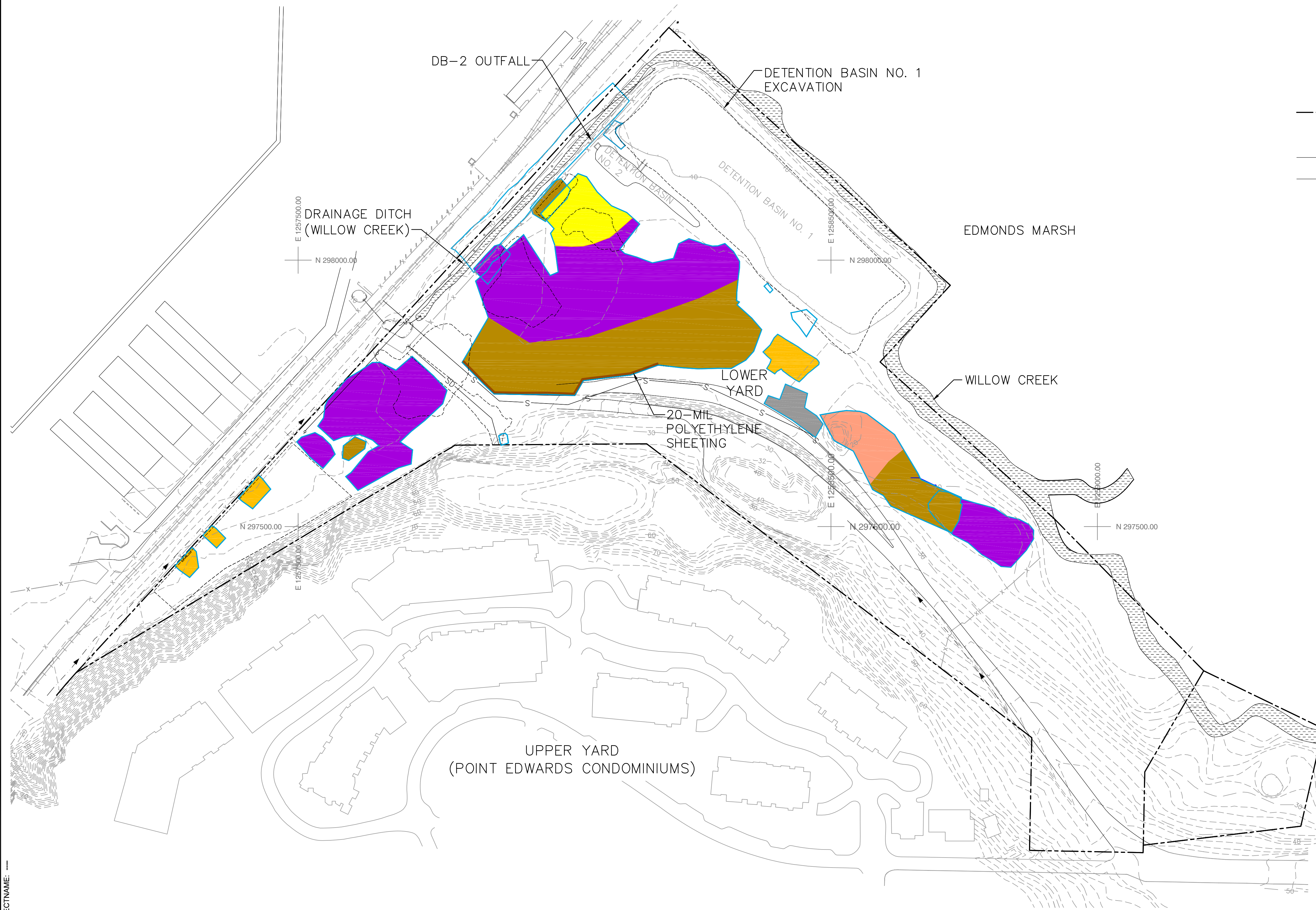


FIG. A-8



LEGEND:

- 2001 AND 2003 SOIL EXCAVATIONS BELOW GROUNDWATER TABLE
- LOWER YARD PROPERTY BOUNDARY
- 2007/2008 EXCAVATION BOUNDARIES
- S — WSDOT STORM DRAIN LINE
- SD — POINT EDWARDS STORM DRAIN LINE

TOP OF GRAVEL BACKFILL ELEVATION:

- 2 – 3 FT AMSL
- 4 – 5 FT AMSL
- 5 – 6 FT AMSL
- 6 – 7 FT AMSL
- 7 – 8 FT AMSL
- 9 – 11 FT AMSL

NOTES:

1. MONITORING WELLS MW-129R, MW-139R, MW-8R, AND MW-149R WERE ABANDONED DURING INTERIM ACTION AND REPLACED IN OCTOBER 2008.
2. STAFF GAUGE D-1 RE-ESTABLISHED PRIOR TO JUNE 2009 SAMPLING EVENT.
3. STAFF GAUGE D-4 WAS ESTABLISHED PRIOR TO JUNE 2009 SAMPLING EVENT TO REPLACE STAFF GAUGE D-7 WHICH IS NOT WITHIN THE WILLOW CREEK CHANNEL.
4. STAFF GAUGES WERE RESURVEYED BY OTAK INCORPORATED JUNE 1, 2009. STAFF GAUGES WERE SURVEYED FROM TOP OF GAUGE AND WATER LEVELS ARE NOW MEASURED FROM TOP DOWN TO WATER.
5. 20-MIL POLYETHYLENE SHEETING INSTALLED UPON COMPLETION OF PHASE I EXCAVATION. SHEETING REACHES TO APPROXIMATELY 7.5 FEET ABOVE MEAN SEA LEVEL.
6. HORIZONTAL DATUM: WASHINGTON STATE COORDINATE SYSTEM NORTH ZONE (NAD 83/98).
VERTICAL DATUM: N.A.V.D. 88
UNITS: U.S. SURVEY FEET
HORIZONTAL AND VERTICAL CONTROL ESTABLISHED BY GPS VIA VERTICAL REFERENCE STATION NETWORK (VRSN).
7. FT AMSL = FEET ABOVE MEAN SEA LEVEL.

0 200' 400'
GRAPHIC SCALE

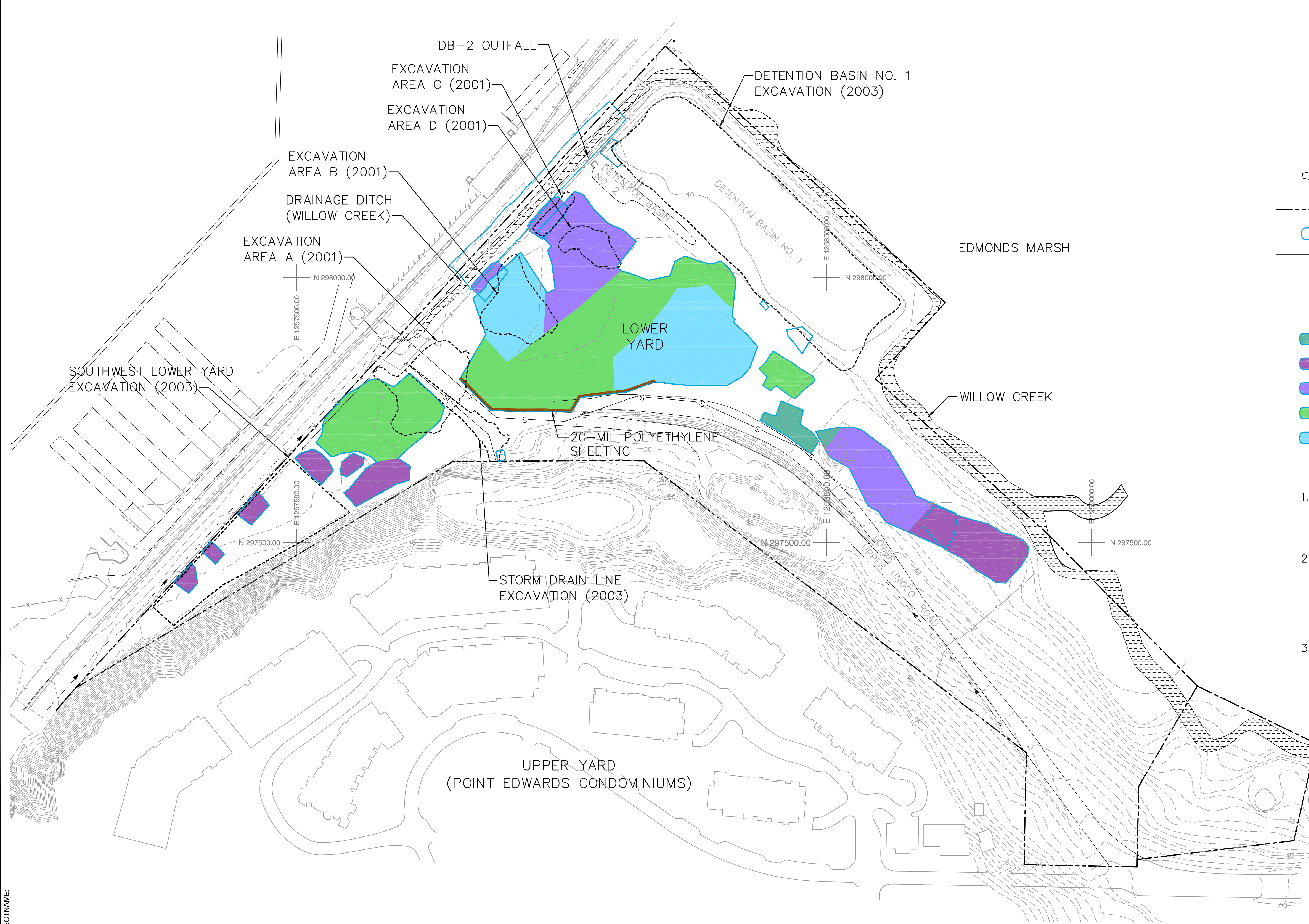
CHEVRON ENVIRONMENTAL MANAGEMENT COMPANY
FORMER UNOCAL EDMONDS TERMINAL, LOWER YARD
EDMONDS, WASHINGTON
CONCEPTUAL SITE MODEL

TOP OF 2008 GRAVEL BACKFILL ELEVATIONS

ARCADIS

FIGURE
A-9.1

CITY: SYRACUSE, NY DIV/GROUP: ENV/IM-DV DB: K. DAVIS, R. ALLEN, P. LISTER PM/TM: R. ANDRESEN TR: A. PATEL LYR: ON=*;OFF=*REF* G:\ENVCAD\SYRACUSE\ACT\B0045362\0003\00016\DWG\45362B02.dwg LAYOUT: 10 SAVED: 6/3/2013 10:39 AM ACADVER: 18.1S (LMS TECH) PAGESETUP: --- PLOTSTYLETABLE: PLTFULL.CTB PLOTTED: 6/3/2013 10:40 AM BY: LISTER, PAUL
XREFS: 45362X02 45362XDL 45362X03 IMAGES: PROJECTNAME: ---



LEGEND:

- 2001 AND 2003 SOIL EXCAVATIONS BELOW GROUNDWATER TABLE
- LOWER YARD PROPERTY BOUNDARY
- 2007/2008 EXCAVATION BOUNDARIES
- S — WSDOT STORM DRAIN LINE
- SD — POINT EDWARDS STORM DRAIN LINE

BOTTOM OF GRAVEL BACKFILL ELEVATION:

- 8 – 9 FT AMSL
- 6 – 4 FT AMSL
- 4 – 2 FT AMSL
- 2 – 0 FT AMSL
- 0 TO –2 FT AMSL

NOTES:

1. 20-MIL POLYETHYLENE SHEETING INSTALLED UPON COMPLETION OF PHASE I EXCAVATION. SHEETING REACHES TO APPROXIMATELY 7.5 FEET ABOVE MEAN SEA LEVEL.
2. HORIZONTAL DATUM: WASHINGTON STATE COORDINATE SYSTEM NORTH ZONE (NAD 83/98).
   VERTICAL DATUM: N.A.V.D. 88
   UNITS: U.S. SURVEY FEET
   HORIZONTAL AND VERTICAL CONTROL ESTABLISHED BY GPS VIA VERTICAL REFERENCE STATION NETWORK (VRSN).
3. FT AMSL = FEET ABOVE MEAN SEA LEVEL.

0 200' 400'
GRAPHIC SCALE

CHEVRON ENVIRONMENTAL MANAGEMENT COMPANY
FORMER UNOCAL EDMONDS TERMINAL, LOWER YARD
EDMONDS, WASHINGTON
CONCEPTUAL SITE MODEL

BOTTOM OF 2008 GRAVEL BACKFILL ELEVATIONS

ARCADIS

FIGURE
A-9.2

CITY: SYRACUSE, NY DIV/GROUP: ENV/IM-DV DB: K. DAVIS, R. ALLEN, P. LISTER PM/TM: R. ANDRESEN TR: D. RASAR LYR: ON=*;OFF=*REF* (FRZ)
G:\ENVCAD\SYRACUSE\ACT\B0045362\0003\00016\DWG\45362B04.dwg LAYOUT: 11 SAVED: 6/3/2013 10:49 AM ACADVER: 18.1S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: PLTFULL.CTB PLOTTED: 6/3/2013 10:49 AM BY: LISTER, PAUL
XREFS: 45362X02 45362XDL 45362X03 IMAGES: PROJECTNAME: ----

Date Start/Finish: 10/08/08
Drilling Company: Cascade Drilling Inc.
Driller's Name: Andy Flanagen
Drilling Method: Hollow Stem Auger
Auger Size: 8"
Rig Type: CME-75
Sampling Method: 2" Split Spoon

Northing: 297500.80
Easting: 1257354.15
Casing Elevation: 12.18

Borehole Depth: 13.5
Surface Elevation:

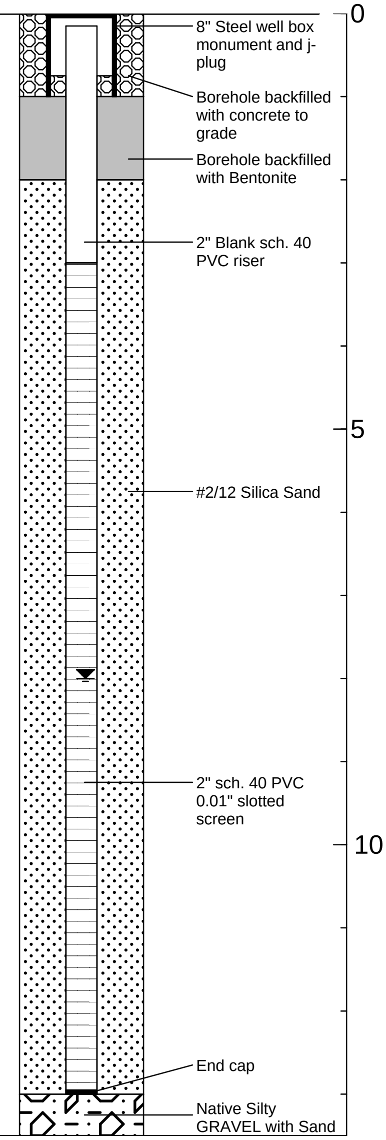
Descriptions By: Russ Greisler

Well/Boring ID: **MW-149R**

Client: Chevron

Location: 11720 Unoco Rd., Edmonds, WA

DEPTH	ELEVATION	Sample Run Number		Recovery (feet)	Blow Counts	N - Value	PID Headspace (ppm)	Analytical Sample	USCS Code	Geologic Column	Stratigraphic Description	Well/Boring Construction
0									SP		SAND (SP); brown, fine to medium, few fine subrounded to subangular gravels, trace fines. (FILL)	
	10	1		1.5			0.0					
5	5	2		1.5	5 6 4	10	0.0					
									GW		GRAVEL (GW); brown, subrounded to subangular, trace sand and non plastic fines. (FILL)	
10												
	0	3		1.5			0.3		GM		Silty GRAVEL with Sand (GM); dark brown, fine, subrounded to subangular, some non plastic fines, trace sand.	



Remarks: bgs = below ground surface

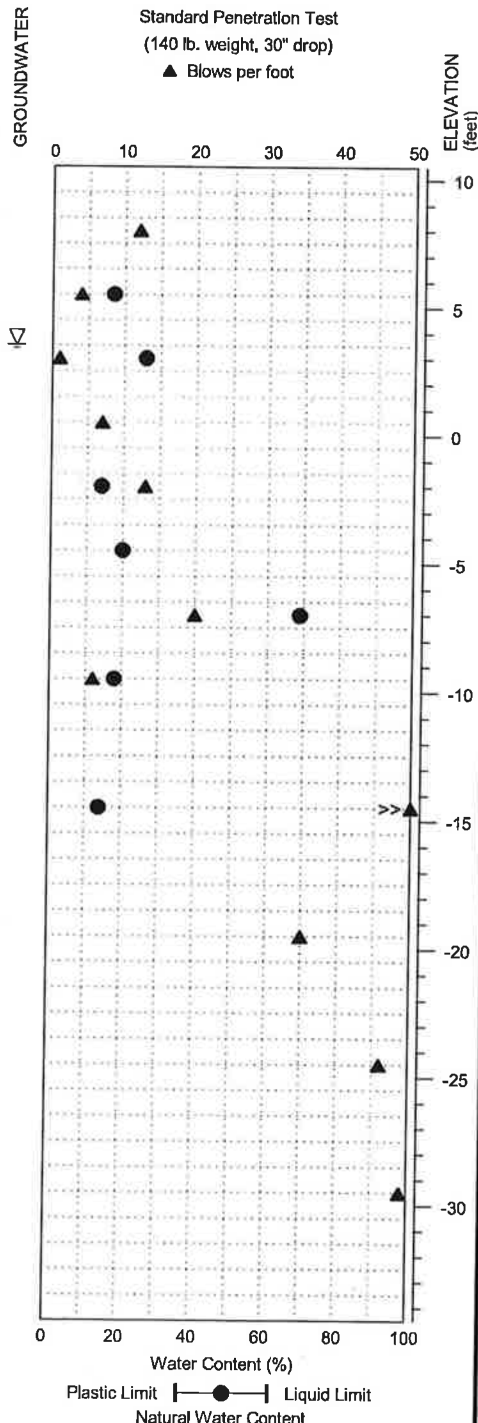
ARCADIS
Infrastructure, environment, buildings

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger, CME-850 track mounted rig
SAMPLING METHOD: SPT with autohammer
LOCATION: See Figure 2: Site and Exploration Plan

DATE STARTED: 07/16/2008
DATE COMPLETED: 07/16/2008
LOGGED BY: J. Speck
SURFACE ELEVATION: 10.5 ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	ML	Light brown, sandy SILT.			
	SP	Loose to medium dense, dark olive brown, gravelly, fine to medium SAND, moist.	S-1	3-5-7	
5			S-2	4-3-1	MC
	ML SM	Loose, dark olive brown, gravelly, fine to medium silty SAND to sandy SILT, wet.	S-3	0-0-1	MC
10			S-4	1-2-5	
	SM	Loose to medium dense, dark gray, fine to medium silty SAND, wet.	S-5	5-7-6	MC
15		8 inch heave noted, water added.	S-6	5-4-2	GS MC
	ML	Medium dense, dark olive brown, slightly gravelly, organic SILT, wet.	S-7	14-10-10	MC
20	SM	Dense, dark olive brown, slightly gravelly, fine to medium silty SAND, wet. Trace decomposed wood (black). 2 inch lense of sandy SILT at 20.25 feet.	S-8	6-2-4	GS MC
25		Some siltier lenses 4 inch thick at 25.25 feet, 2 inch thick at 26 feet.	S-9	15-29-36	GS MC
30	SP	Dense, dark gray, fine to medium SAND with trace fines and gravel, wet.	S-10	1-8-27	
35	SM	Dense, dark olive brown, silty fine to medium SAND with gravel, wet.	S-11	7-17 -29	
40			S-12	8-16 -33	
45					

Borehole completed at TD = 41.5 feet.
Groundwater noted at 7 feet during exploration.



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

WILLOW CREEK CULVERT REPLACEMENT
UNDER BNSF MAINLINE

BORING:
BH-1

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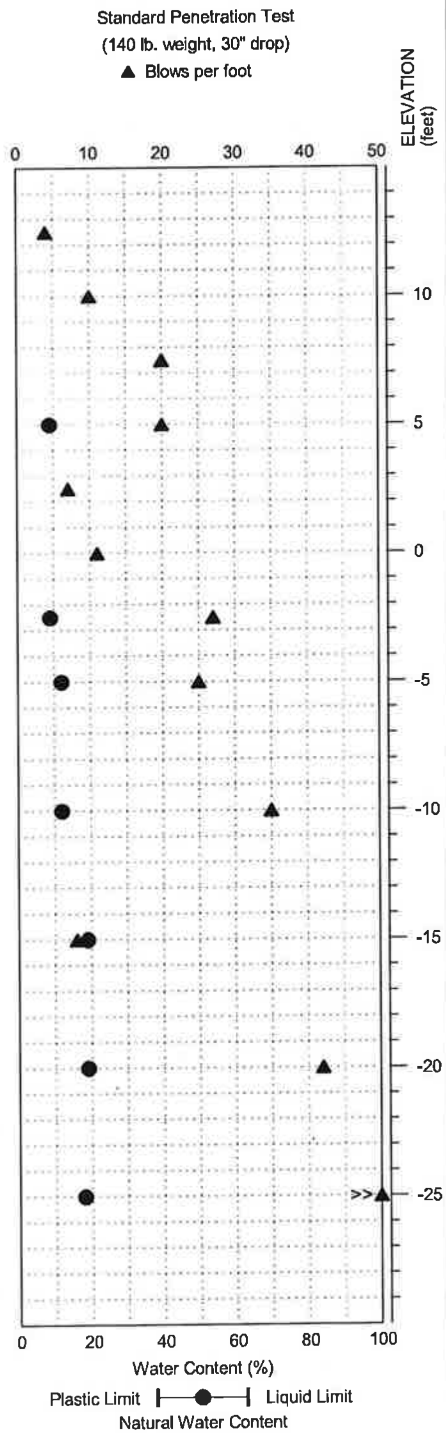
PROJECT NO.: 2007-147-21 FIGURE: 4

BORING-DSM 2007142.GPJ 2/26/10

DRILLING COMPANY: Holocene Drilling
DRILLING METHOD: Hollow Stem Auger, CME-850 track mounted rig
SAMPLING METHOD: SPT with autohammer
LOCATION: See Figure 2: Site and Exploration Plan

DATE STARTED: 07/17/2008
DATE COMPLETED: 07/17/2008
LOGGED BY: J. Speck/B. Blanchette
SURFACE ELEVATION: 15.0 ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	SM	Cuttings, brown, sandy SILT to fine to medium silty SAND with gravel, moist.			
		No recovery	S-1	4-2-2	
5	SP	Medium dense, olive brown, gravelly, fine to coarse SAND, moist.	S-2	3-4-6	
			S-3	4-8-12	
10		Very gravelly, gravel is fine to coarse. Wet.	S-4	6-9-11	GS MC
			S-5	4-3-4	
15			S-6	2-6-5	
			S-7	8-13-14	MC
20	SP SM	Medium dense, olive brown, gravelly, fine to coarse slightly silty SAND, wet. Gravel is fine. Noticeably more fine sand than above sample.	S-8	11-10-15	GS MC
25	SM	Dense, dark gray, gravelly, fine to medium silty SAND, wet.	S-9	9-15-20	MC
30		Material becomes loose with a 3 inch thick silty sand lense at 31 feet. Color changes to dark gray at 31 feet. Wood fragments noted at 30.5 feet.	S-10	8-3-5	GS MC
35			S-11	13-21 -21	MC
40			S-12	31 -50/3	MC
45		Borehole completed at TD = 41.5 feet. Groundwater was first noted at 9.5 feet during exploration.			



NOTE: This log of subsurface conditions applies only at the specified location and on the date indicated and therefore may not necessarily be indicative of other times and/or locations.

HWA GEOSCIENCES INC.

WILLOW CREEK CULVERT REPLACEMENT
UNDER BNSF MAINLINE

BORING:
BH-2

PAGE: 1 of 1

PROJECT NO.: 2007-147-21 FIGURE: 5

BORING-DSM 2007142.GPJ 2/26/10

Fig. A-12

APPENDIX B

GEOTECHNICAL LABORATORY TEST PROCEDURES AND RESULTS

APPENDIX B

GEOTECHNICAL LABORATORY TEST PROCEDURES AND RESULTS

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FIGURES

APPENDIX B

GEOTECHNICAL LABORATORY TEST PROCEDURES AND RESULTS

B.1 INTRODUCTION

This appendix contains descriptions of the procedures and the results of the geotechnical laboratory tests performed on select soil samples obtained from the subsurface explorations completed for the Willow Creek Daylight Project. The samples were tested to evaluate the basic index and physical properties of the native soil. The laboratory test program included visual classifications, water content determinations, and grain size analyses. The laboratory testing was performed by an experienced technician at the Shannon & Wilson, Inc. laboratory in Seattle, Washington.

B.2 VISUAL CLASSIFICATION

The soil samples recovered from the exploratory borings and test pits were visually reclassified in our laboratory using a system based on American Society for Testing and Materials/ASTM International (ASTM, 2000 – 2011) Designation: D2487, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), and ASTM Designation: D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). This visual classification method allows for convenient and consistent comparison of soils from widespread geographic areas. The terminology used and the definition of modifying terms are presented on Figure A-1 in Appendix A. The sample classifications are presented on the individual boring and test pit logs in Appendix A.

B.3 WATER CONTENT DETERMINATION

The natural water content of select samples recovered was determined in general accordance with ASTM Designation: D2216, Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil by Mass. Comparison of the natural water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. The organic contents are shown graphically on the boring logs in Appendix A.

B.4 GRAIN SIZE ANALYSES

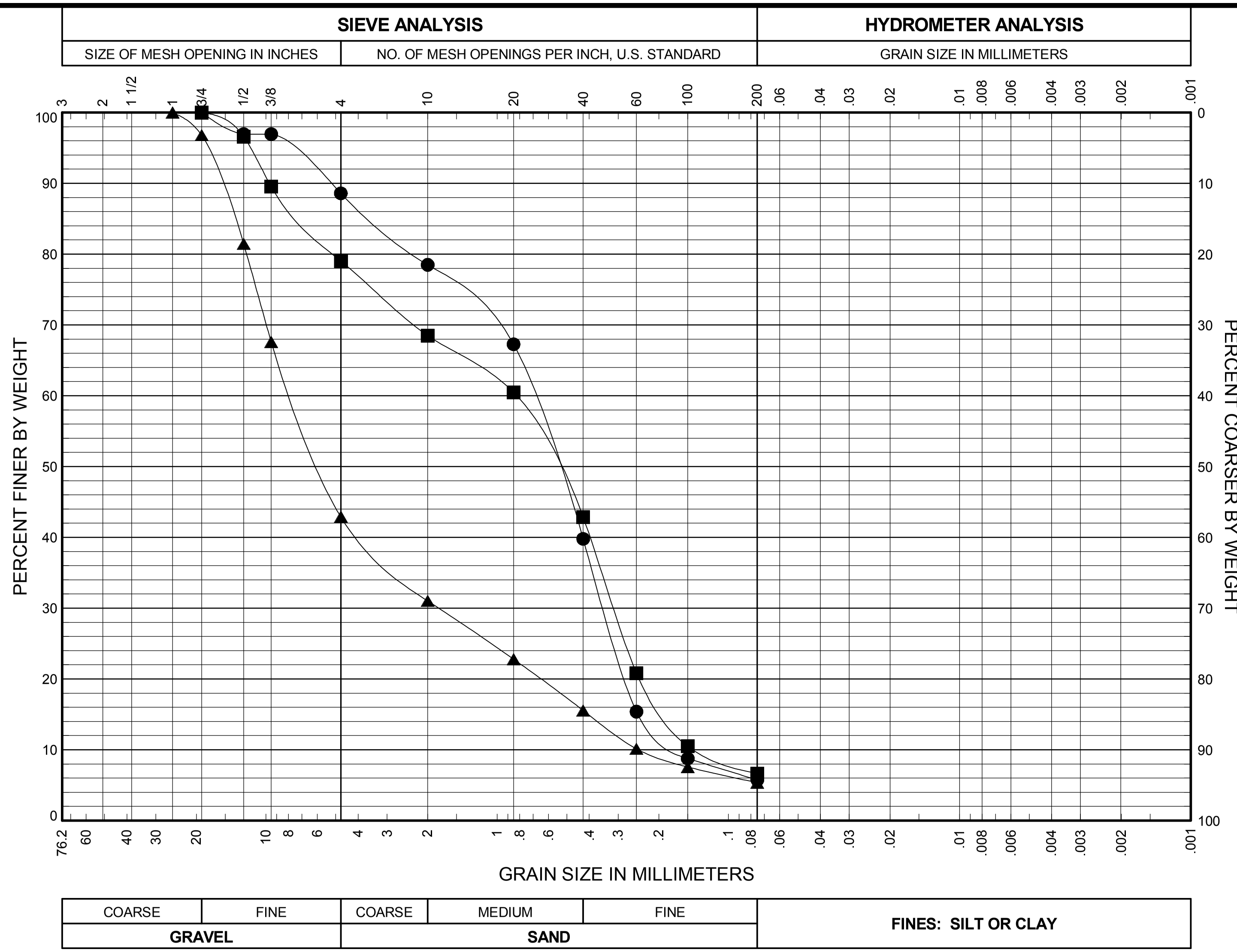
Grain size analyses were performed on selected samples of granular soils in general accordance with ASTM Designation: D6913, Standard Test Method for Particle-Size Analysis of Soils. Results of these analyses are presented as grain size distribution curves in Figures B-1 through

B-3 in this appendix. Along with each grain size distribution is a tabulated summary containing the sample description, Unified Soil Classification System symbol for the soil group, percentage of fines passing the No. 200 sieve, and the natural water content.

Grain size distribution is used to assist in classifying soils and to provide correlation with soil properties, including hydraulic conductivity, capillary action, liquefaction potential, and sensitivity to moisture.

B.5 REFERENCE

American Society for Testing and Materials/ASTM International (ASTM), 2000 - 2011, 2000 – 2011 annual book of standards, construction, volume 04.08, soil and rock (I): West Conshohocken, Penn.



LEGEND

USCS: Unified Soil Classification System

COBBLE REM %: Percentage of cobbles removed from specimen; based on pre-removal total dry mass

< 2 μm %: Percentage of soil particles finer than 2 micrometers (0.002 mm); clay-size fraction

NAT WC %: Natural water content

Cu: Coefficient of uniformity

Cc: Coefficient of curvature

ASTM DES: ASTM International test standard designation

*: ***Sample specimen weight did not meet required minimum mass for ASTM test method.***

BORING AND SAMPLE NO.	DEPTH (feet)	USCS SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	< 2 μm %	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-1, S-4*	10.0	SP-SM	Poorly Graded Sand with Silt	11	83	5.7			16.1	4.3	1.0	AKV	JFL	D422
■ B-1, S-11*	35.0	SP-SM	Poorly Graded Sand with Silt and Gravel	21	72	6.6			16.3	6.1	0.8	AKV	JFL	D422
▲ B-1, S-13*	45.0	GW-GM	Well-Graded Gravel with Silt and Sand	57	38	5.3			8.9	31.5	1.7	AKV	JFL	D422

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

GRAIN SIZE DISTRIBUTION
BORING B-1

November 2014 21-1-12393-406

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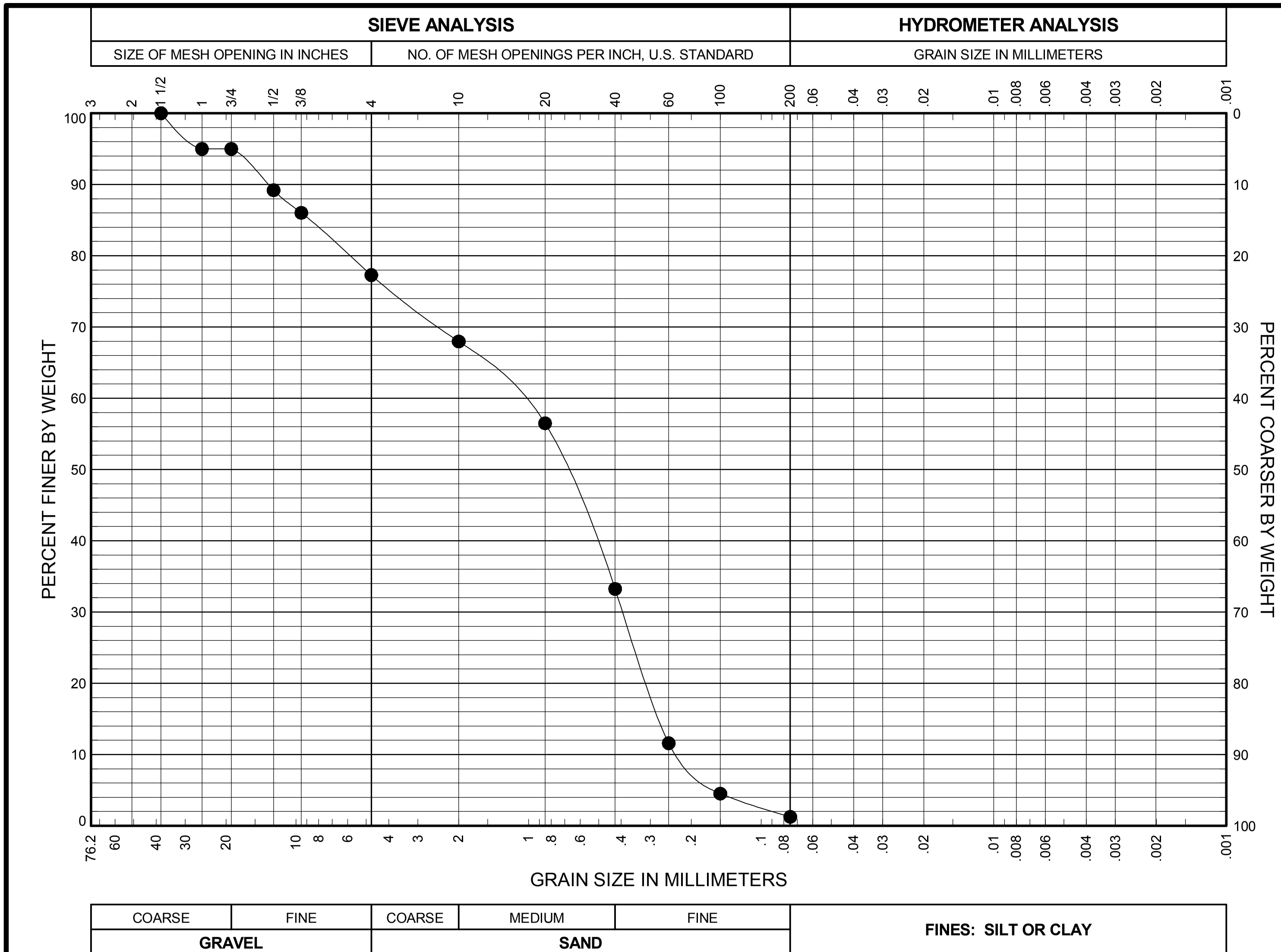
FIG. B-1
Sheet 1 of 1



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BORING AND SAMPLE NO.	DEPTH (feet)	USCS SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	< 2 µm %	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● TP-2, G-1*	5.0	SP	Poorly Graded Sand with Gravel	23	76	1.3			3.1	5.0	0.6	AKV	JFL	D422

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

GRAIN SIZE DISTRIBUTION
TEST PIT TP-2

November 2014 21-1-12393-406

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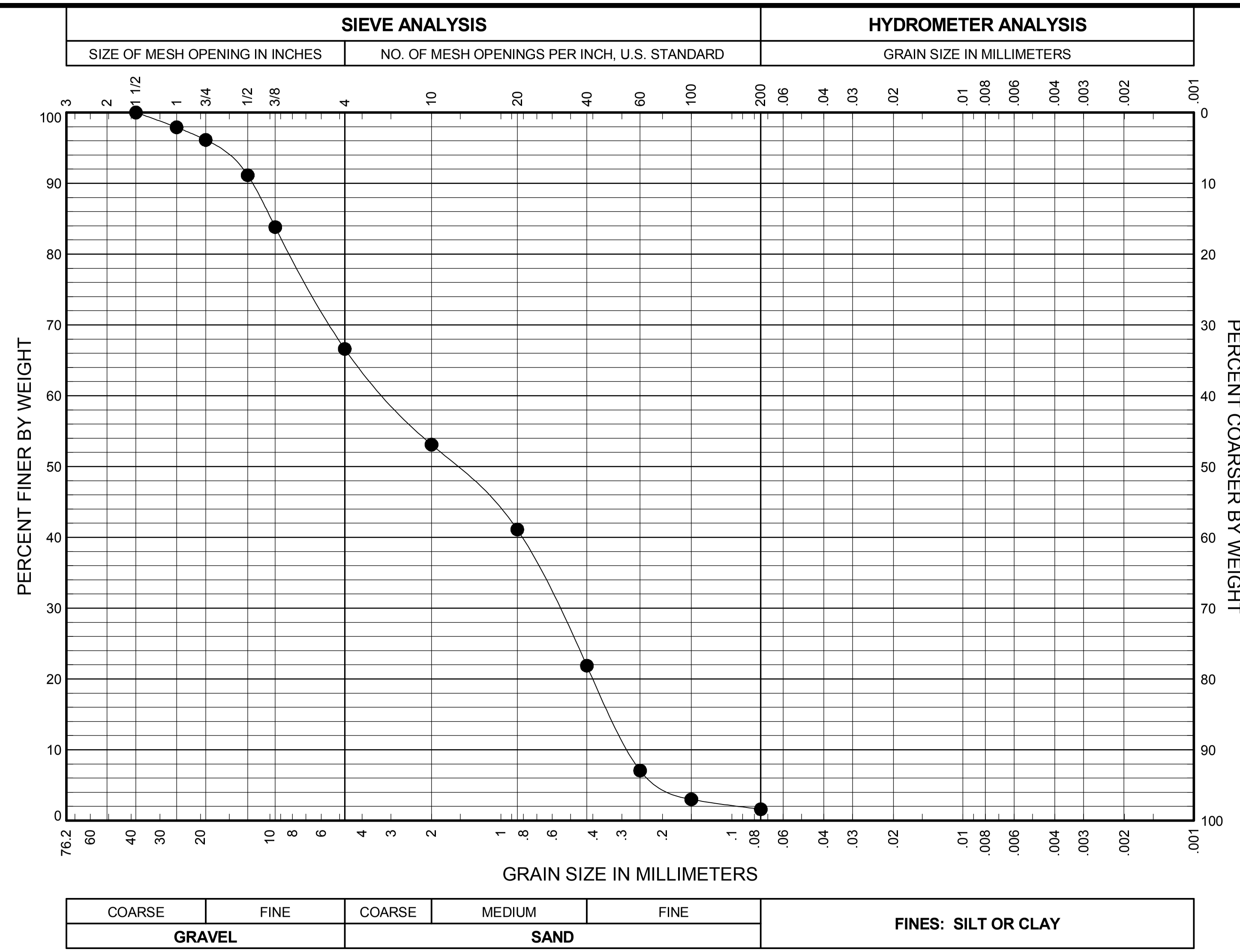
FIG. B-2
Sheet 1 of 1



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Cu: Coefficient of uniformity

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ASTM DES: ASTM International test standard designation

*: ***Sample specimen weight did not meet required minimum mass for ASTM test method.***

BORING AND SAMPLE NO.	DEPTH (feet)	USCS SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	< 2 µm %	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● TP-4, G-3*	10.8	SP	Poorly Graded Sand with Gravel	33	65	1.6			2.8	11.2	0.4	AKV	JFL	D422

Willow Creek Daylight Project
Geotechnical Evaluation
Edmonds, Washington

GRAIN SIZE DISTRIBUTION
TEST PIT TP-4

November 2014 21-1-12393-406

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FIG. B-3
Sheet 1 of 1

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland