Geotechnical Report
Genesee and Henderson Combined Sewer Overflow Reduction Project
Henderson Basin 44 (Seward Park)
Preliminary Design
Seattle, Washington

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GEOTECHNICAL REPORT
GENESEE AND HENDERSON COMBINED SEWER OVERFLOW REDUCTION PROJECT
HENDERSON BASIN 44 (SEWARD PARK) PRELIMINARY DESIGN
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of subsurface explorations and provides preliminary geotechnical engineering conclusions and recommendations in support of the preliminary engineering phase for selected combined sewer overflow (CSO) reduction alternatives located in Henderson Basin 44. The project alternatives considered in this report are located on the west shore of Lake Washington, in the south portion of Seward Park, as shown in Vicinity Map, Figure 1.

We presented our preliminary geotechnical evaluation for the project in our report titled, "Preliminary Geotechnical Evaluation – Genesee and Henderson Combined Sewer Overflow Reduction Project, Seattle, Washington," dated August 10, 2009 (August 2009 Report) (Shannon & Wilson, 2009). Subsurface explorations and preliminary geotechnical recommendations associated with other considered alternatives in the Henderson Basin were presented in our letter report titled "Genesee and Henderson Combined Sewer Overflow Reduction Project, Henderson Basin Preliminary Design Subsurface Explorations, Seattle, Washington," dated January 19, 2011 (Shannon & Wilson, 2011).

2.0 SITE AND PROJECT DESCRIPTION

The CSO reduction alternatives are configured as shown in Figure 2, and each generally consists of a 2.4 million gallon storage tank and associated mechanical and electrical facility vaults, with overall dimensions of 428 feet long, 52 feet wide, and 20 feet deep. Two potential locations for the storage tank are being considered: Alternative 1, Storage at Seward Park Parking Lot; and Alternative 2, Storage at Seward Park Tennis Courts. Alternative 1 would be constructed at the site of an existing parking lot in Seward Park at the east end of Seward Park Road. The tank alignment would be approximately east-west, and would be bounded on the north by a multi-use Park path, and on the south by Lake Washington. Alternative 2 would be constructed at the site of an existing parking lot and tennis court in Seward Park south of Seward Park Road. The tank alignment would be approximately north-south, and would be bounded on the west by private property and on the east by Lake Washington. We understand that the tanks would have about 4 feet of cover, making the base of the tanks about 24 feet below the ground surface.

3.0 SUBSURFACE EXPLORATIONS

The preliminary evaluation of the subsurface conditions at the site was accomplished by reviewing available subsurface data summarized in our August 2009 Report, completing additional field explorations, and conducting laboratory testing on soil and rock samples collected from the additional field explorations. The following sections summarize these efforts.

3.1 Previous Subsurface Investigations

Before performing additional field explorations, we reviewed existing subsurface data included as part of our August 2009 Report. The logs of pertinent previous explorations are included in Appendix F. The approximate locations of the previous field explorations are shown in Figure 2.

3.2 Current Subsurface Explorations

The current subsurface exploration program for the preliminary engineering phase of the project consisted of drilling three borings, designated B-4 through B-6, to supplement existing subsurface data for evaluation of subsurface conditions at the locations of the considered CSO reduction alternatives. The locations of borings B-4 through B-6 are shown in the Site and Exploration Plan, Figure 2, and on the Geology Map of Seward Park Area, Figure 3. The locations of these borings and outcrops were estimated from existing features and, therefore, reflect approximate locations.

Borings B-4, B-5, and B-6 were drilled to depths of approximately 65, 60, and 65 feet below the existing ground surface, respectively. Mud-rotary drilling methods were used to advance the borings in soil, and rock coring methods, utilizing an HQ-size, triple, split-tube core barrel to retrieve relatively intact core samples of the sandstone and siltstone. After each boring was completed, a 2-inch polyvinyl chloride (PVC) monitoring well with a 5-foot-long screen was installed, and the holes were backfilled with clean sand in the screened interval and with bentonite chips elsewhere, according to state regulations. The logs of the borings are shown in Appendix A as Figures A-2 through A-4.

3.3 Geotechnical Laboratory Testing

Geotechnical laboratory tests were performed on soil and rock samples retrieved from borings B-4 through B-6. The laboratory tests were performed to provide data to classify the materials into similar geologic groups and to estimate likely engineering behavior. Visual classification tests, natural water content determinations, grain size analyses, and Atterberg Limit tests were conducted on selected soil samples; slake durability tests, point load tests, and unconfined

compression tests were conducted on selected rock samples. The visual classifications, water contents, grain size analyses, and Atterberg Limits are incorporated into the boring logs in Figures A-2 through A-4. Atterberg Limits and results of slake durability tests are included in Figure B-2. Summaries of point load test results are provided in Table B-1 and B-2 (Appendix B). Unconfined compressions tests are included in Appendix B.

3.4 Rock Outcrop Mapping

A geologist from Shannon & Wilson mapped four accessible bedrock exposures in the project vicinity to identify and characterize lithologic units and boundaries and rock mass structure, including discontinuity sets, fillings, and coatings. We also measured Schmidt Hammer rebound numbers, which may be correlated with unconfined compressive strengths of the rock. The four mapped outcrops are shown on the Geology Map of Seward Park Area, Figure 3. Field data from this mapping were used to provide a preliminary assessment of the stability of proposed CSO structure excavations and ground support requirements.

Appendix C summarizes the rock mass properties that we observed and characterized. The rock mass properties include orientation, persistence (length), terminations, spacing (distance between joints), aperture (width), joint filling, surface profile, and roughness.

All four outcrops consist of sandstone of the Blakely Formation. The sandstone is less susceptible to slaking and is of higher compressive strength, whereas the siltstone is highly slakable and is of lower compressive strength and, therefore, the sandstone may be over-represented in the outcrops. The orientation data indicate that there are three joint sets with a persistence ranging from 1 to 20 feet and spacing ranging from 0.01 to 1.0 foot. The outcrop data is included in Appendix C, Tables C-1 through C-4. A stereoplot of the discontinuity orientation data is included in Appendix C.

3.5 Hydraulic Conductivity Testing

Single-well field hydraulic conductivity tests (slug tests) were performed to provide an estimate of the horizontal hydraulic conductivity of the soils. Observation wells B-4, B-5, and B-6 were slug-tested for the project. The slug tests were applied to 5-foot-long screens backfilled with clean sand. A slug test provides an in situ means of estimating the horizontal hydraulic conductivity of the saturated sediments or bedrock surrounding the screened zone of a well. Slug tests do not provide data regarding large-scale aquifer properties, aquifer geometry, or boundary conditions affecting groundwater flow.

Slug testing consists of rapidly raising or lowering the water level within an observation well and measuring the recovery of the water level over time to near the static level. Raising the water level is achieved by lowering a slug (a sealed, sand-filled, PVC pipe) below the static water level to displace water within the well casing. This procedure is termed a “falling head test” because the water level falls with time back to the static level. Lowering the water level is achieved by quickly removing the slug from the well. This is termed a “rising head test” because the water level rises back to the static level after the slug is removed. A series of three rising and falling head tests were performed as part of the slug testing at each location.

Prior to slug testing, Shannon & Wilson developed each well to remove additional drilling fluid and sediment generated during the drilling process from the screen interval. The development work consisted of surging while removing 5 gallons of water from each 1-foot section of screen. The total water volume removed from each well during development was 25 gallons.

Field staff measured and recorded the variation in water level during the slug testing period at each well using a downhole combination pressure transducer/data logger, with additional water level measurements being made with an electronic water level indicator. The transducer was secured in the well below the depth to which the slug would be lowered, and water level measurements were made every second by the transducer and recorded by the data logger. The pressure transducer/data logger used for each test was an unvented model and required correction for barometric pressure changes. A barometric pressure transducer/data logger was installed within the well monument during each test to record barometric pressure changes every second during the slug tests. A total of six tests (three falling head and three rising head) were completed for each well.

The Bouwer and Rice equation is used to calculate the hydraulic conductivity of an aquifer based on drawdown versus time data from an observation well during a slug test. We used software package AQTESOLV (Duffield, 2007) and selected the Bouwer and Rice method (Bouwer and Rice, 1976 and 1989) to analyze the slug test data. The plots for the slug tests analyzed at each well are included as figures in Appendix D.

4.0 SITE SUBSURFACE CONDITIONS

4.1 General

The subsurface conditions at the selected CSO reduction alternatives were evaluated by reviewing previous subsurface data, the results of the new borings B-4 through B-6, the results of geophysical seismic surveys, hydraulic conductivity testing, and mapping of four surface rock

outcrops. Locations of current and previous subsurface soil explorations and proposed structures are shown in the Site and Exploration Plan, Figure 2. Major geologic units, the Seattle Fault location and boundaries, and mapped rock outcrop locations are shown in the Geologic Map of the Seward Park Area, Figure 3.

4.2 Geology

Seward Park is located near the middle of the approximately 6-mile-wide Seattle Fault Zone. The northern half of the Park is in glacial soils overlain by residual and alluvial soils. Roughly the southern half of Seward Park is mapped as Blakely Formation and is generally comprised of highly weathered to fresh sandstone, siltstone, and claystone (collectively called mudstone) containing marine fossils and shells (Troost and others, 2005). At other sites in the Seattle and Bellevue areas where the Blakely Formation has been encountered in the vicinity of the Seattle Fault, it has been significantly disturbed (sheared or fractured), likely related to seismic movement and stress relief associated with tectonic uplift and glacial retreat. While not encountered in the explorations, such disturbed conditions should be expected in the Seward Park area.

The proposed alternative tanks are located in the southern half of Seward Park and the subsurface conditions generally consists of completely weathered to fresh, very low to low strength mudstone and sandstone bedrock (Blakely Formation) overlain in some localized areas by relatively thin recent fills. A summary table of rock parameters is presented below and is based on laboratory results in Appendix B.

TABLE 1
PHYSICAL PROPERTIES OF THE SANDSTONE AND SILTSTONE OF THE BLAKELY FORMATION

Rock Type	Unit Weight (pcf)	Unconfined Compressive Strength (psi)	Youngs Modulus (psi)	Slake	RQD (%)	Hydraulic Conductivity (cm/sec)
Sandstone	129-133 (4)	710-5,310 (18)	407,000-1,710,000 (4)	Non-slakable	0-100	2.4×10^{-2} to 5.8×10^{-4} (2 mixed rock)
Siltstone	127 (1)	610-3,446 (8)	760,000 (1)	Rapid partial to completely slaked	0-100	2.7×10^{-4} cm/sec (1)

Notes:
cm/sec = centimeters per second
pcf = pounds per cubic foot
psi = pounds per square inch
RQD = rock quality designation
Value in ( ) is number of tests

The three borings indicated 0.5 to 10 feet of medium stiff clay fill overlying the siltstone and sandstone of the Blakely Formation. Below the fill, completely to highly weathered sandstone and siltstone were encountered in the upper few feet to over 15 feet, becoming slightly weathered to fresh with increasing depth.

The siltstone occurs as 1- to 17-foot-thick layers, interbedded with 3- to 48-foot-thick layers of sandstone. The siltstone is fine-grained, gray, very low to low strength, with an average strength of 1,644 pounds per square inch (psi), based on field and laboratory tests. It is notable that the siltstone is generally highly slakable, with chunks of the core falling apart and disaggregating to its soil constituents within a few minutes to a few hours of being submerged in water. It is likely that this slaking characteristic contributes to an absence of siltstone outcrops in Seward Park. The sandstone is gray, dense, fine- to medium-grained sandstone with shell fragments and up to 4-inch-diameter limy concretions and scattered bedding. The sandstone is apparently cemented and, therefore, does not slake, and has an average strength of 2,850 psi, based on laboratory unconfined strength tests. As a result of these properties, the sandstone is represented in Seward Park by several outcrops.

Joint data from the drilling logs show dip angles similar to those noted on the rock outcrop discontinuity data. These joints may be part of the same joint set. However, the dip direction orientation from the rock core is unknown. In addition, the drilling logs indicate relatively shallow angle (less than or equal to 20 degrees) jointing that was not observed at the rock outcrops.

Shannon & Wilson retained a local geophysical consultant to complete seismic refraction surveys at the two proposed storage tank locations. In general, the surveys indicate approximate depths to bedrock ranging from 2 to 10 feet below ground surface at Alternative 2 and from 3 to 12 feet below ground surface at Alternative 1. Seismic velocities within the overburden ranged from 1,000 to 1,800 feet per second (fps) and within the bedrock ranged from 5,500 to 6,900 fps. The seismic refraction survey report is included as Appendix E of this report.

Based on the considered alternatives at the time of the preparation of this report and the encountered and interpreted subsurface conditions, it is anticipated that the tanks will be constructed almost entirely within sedimentary rock of the Blakely Formation, with 0 to 15 feet of existing fill near the ground surface. The tank bottom (at approximately 24 feet below the existing ground surface) would likely be founded on low strength, moderately to highly fractured sandstone and/or siltstone.

4.3 Faulting and Seismicity

The east-west-trending Seattle Fault Zone, extends for about 7 to 8 miles from Atlantic City Park on the south to Leschi Park on the north along the west coast of Lake Washington, and is the nearest potentially active fault to the project site. The fault extends from the Olympic Mountains on the west to the foothills of the Cascades on the east. Recent geologic evidence indicates that ground surface rupture from movement on this fault zone occurred as recently as 1,100 years ago. Preliminary estimates of recurrence rates for the Seattle Fault are on the order of 3,000 to 5,000 years with a slip rate of 0.03 to 0.04 inch per year. Earthquake magnitudes of up to 7.7 have been postulated for movement on this fault.

One of the splays of the fault cuts east-west along the middle of Seward Park, as shown in Figure 3. As a result of the vertical offset, the south side of the Park has bedrock exposures at ground surface, while on the north half of the Park, only soil exposures are visible.

4.4 Hydrogeology

Groundwater levels near the edge of Lake Washington are generally coincident with the lake level and rise in subdued mimicry of the topography away from the lake. Therefore, it is likely that groundwater flows from relatively high groundwater areas along the 200- to 300-foot-high north-south-trending ridge that parallels the west shore of Lake Washington towards the relatively low groundwater areas near the west shore of Lake Washington. Relatively high groundwater is also likely to occur in the central, higher elevation area of the Seward Park peninsula, some of which would flow south toward Lake Washington. Perched water levels are also likely to occur in sand layers and lenses underlain by much less permeable silt, clay, and bedrock. Groundwater levels observed in the vibrating wire piezometers installed in boring B-1 indicate that the groundwater is generally within about 10 feet of the ground surface. Groundwater levels observed in B-4, B-5, and B-6, prior to slug testing, were 3.18, 4.53, and 5.43 feet below top of casing, respectively.

Hydraulic conductivity, based on the slug tests performed at observation wells B-4, B-5, and B-6, was estimated as 1.6 feet per day (ft/day) (5.6×10^{-4} centimeters per second [cm/sec]), 68 ft/day (2.4×10^{-2} cm/sec), and 0.8 ft/day (2.8×10^{-4} cm/sec) respectively. These estimates were obtained by taking the geometric mean of the results of the six tests (three falling head and three rising head) completed at each observation well. Detailed results of each slug test are included in Appendix D.

5.0 PRELIMINARY ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

5.1 General

Based on the subsurface conditions encountered in the current and existing explorations and our understanding of the project, engineering studies were performed to develop conclusions and recommendations regarding the following: (a) seismic design considerations, (b) groundwater control, (c) excavation and temporary shoring, (d) lateral pressures for shoring and permanent walls, (e) foundation support, and (f) uplift resistance. A discussion of our studies, analyses, conclusions, and recommendations is presented in the following sections.

5.2 Seismic Design Considerations

We assume that the seismic design of the facility will be in accordance with the International Building Code 2009. Computation of forces used for seismic design for this code is based on seismological input and site soil response factors.

The seismological inputs are short-period spectral acceleration, S_S, and spectral acceleration at the 1-second period, S_1, which were determined using the probabilistic ground motion studies conducted by the U.S. Geological Survey and Frankel and others (2002). S_S and S_1 are for a maximum considered earthquake, which correspond to ground motions with a 2 percent probability of exceedance in 50 years, or about a 2,475-year return period.

The site soil response factors are based on the determination of the Site Class. Based on the subsurface explorations at the site, it is our opinion that the site can be characterized as Site Class C. Parameters for seismic design of structures are presented in Table 2.

TABLE 2
PARAMETERS FOR SEISMIC DESIGN OF STRUCTURES

Spectral Response Acceleration (SRA) and Site Coefficients	Short Period	1-Second Period
Mapped SRA[(1)]	$S_s = 1.52$	$S_1 = 0.52$
Site Coefficients (Site Class C)	$F_a = 1.0$	$F_v = 1.3$
Design SRA[(1)]	$SD_s = 1.01$	$SD_1 = 0.45$

Notes:
[1] Mapped SRA and Design SRA values are in units of gravity.

5.3 Groundwater Control and Dewatering

Within each of the three borings, a 5- to 6-foot-long screened section, containing representative fractures was evaluated with in situ slug tests to be highly to moderately permeable. These fractured bedrock zone may be in direct connection with Lake Washington. Therefore, any excavation system will likely have to be sealed against groundwater inflow or an extensive dewatering program will likely be required. In addition, the local overall groundwater flow is toward the lake, as discussed in Section 4.4, and consequently any excavation adjacent to the lake would experience recharge from the uplands to the north and to the west, and directly from Lake Washington, via more permeable soil units and along fractures in the bedrock. Preliminary calculations of the steady state groundwater flow into the proposed excavations ranges from 50 gallons per minute (gpm) to 1,700 gpm. Based on the bedrock encountered in the borings, the groundwater flows are expected to be relatively high along fractures in the bedrock and low within the intact unfractured bedrock. The flow estimates assume that groundwater is entering the excavation though 50 percent of the excavation face. If flows are allowed to continue along fractures in the bedrock for prolonged periods of time during excavation, then some erosion of the fine-grained soils that typically partially fill a fracture and erosion and plucking of the bedrock will likely contribute to increased flow rates. These groundwater flow estimates are not intended for design purposes. Further evaluation of aquifer properties at the proposed storage tank location will be required to obtain information for design purposes and may include additional borings and well installations, pumping tests, and packer tests.

5.4 Excavation Methods

Appropriate excavation methods are a function of the rock material strengths, degree and characteristics of the natural fracturing or jointing, slakability of the rock, and likely behavior of the rock mass. The relatively consistent seismic velocity measurements across the two sites indicate that the top of rock is at a fairly uniform depth and is fairly uniform in strength and degree of fracturing. There are no indications of filled valleys or gullies in the top of rock that might reflect more erodible rock conditions in some areas, as might be represented by a very high degree and wide zone of fractures or shear zones.

There are several potential excavation methods for the relatively soft rock, including ripping with dozers, backhoe-mounted hydraulic breakers or hoe-rams, a track-mounted roadheader, or boom excavator and rock blasting. From the data on rock conditions available thus far, it appears that all of these excavation approaches would be capable of breaking up and excavating the rock, but some methods would result in less damage to the wall and floor of the resulting excavation.

5.4.1 Ripping

The planned surface area of the tank excavation is relatively large and will permit the effective use of ripping equipment if the rock is readily rippable.

Ripping production has been correlated with the following:

- The seismic velocity
- Bedding thickness, joints and fractures
- Rock strength
- Ripper tip configuration (short, intermediate and long)
- Ripper style (centerline and penetration)

Based on the seismic velocity measurements of around 6,000 fps, measured unconfined strength of the rock of generally less than 4,000 psi, and the fracture spacing observed in the borings and rock outcrops, it appears that the rock should be rippable with a D9 or D10 Caterpillar excavator equipped with a vibratory ripping tooth mounted on the back. The production curves for various excavator models, rock types and seismic velocities in the Caterpillar Performance Handbook (2011) predicts production rates in the range of 500 to 1,000 cubic yards per hour (cy/hr) for D9 and D10 dozers. The actual ripping rates and overall excavation productivity will, of course, depend on the quality of the equipment, and the experience level of the operator, spoil excavation, and removal rates, as well as the rock properties indicated above.

There are several disadvantages to excavating rock with a heavy dozer equipped with a ripper tooth. Ripping will create a fairly ragged surface on the perimeter walls, and it may be difficult to excavate the corners of the tank excavation. Ripping close to the final walls may also fracture a grout curtain or freeze zone around the excavation, resulting in increased groundwater inflow. However, ripping might be augmented with the hydraulic breaker or roadheader to excavate the corners and within 5 to 10 feet of the vertical walls of the tank excavation, thus resulting in a smoother, more stable vertical rock surface. The ripping operation, as well as the hoe-ram and roadheader are fairly dusty and noisy excavation methods.

5.4.2 Hydraulic Impact Hammer or Hoe-ram

Hydraulic impact hammers or hoe-rams have been used to effectively break up rock with unconfined strengths up to 25,000 psi. Hoe-rams can excavate relatively smooth surfaces in the soft rock and has the versatility to reach into the corners of the excavation. Excavation rates for a large hydraulic breaker have been reported to be on the order of 1,000 cy/day. The Caterpillar

Performance Handbook (2011) indicates that hydraulic impact hammer excavation rates are likely to range from 10 cy/hr to about 100 cy/hr for various sizes of hammers, and depending on the specific ground conditions, nature of the equipment, and experience of the operator. As with ripping, the efficiency of a hydraulic impact hammer for rock excavation will also depend on the ability of the spoil removal system to provide fresh rock working faces to the hydraulic hammer operator.

5.4.3 Roadheader

Roadheaders or boom-excavators consist of a rotating drum with replaceable abrasive cutting teeth or picks, mounted on hydraulically actuated boom that is typically mounted tracked transport system. Typically, large-size roadheaders of 30 to 50 tons are capable of excavating up to 12,000 to 15,000 psi rock. The presence of joints or fractures will enable the roadheader boom to pluck out the rock as well as grinding it. Excavation rates for large roadheaders have been documented at 30 to 50 cy/hr in low to medium strength sedimentary rock.

5.4.4 Blasting

Blasting is a feasible excavation approach for the rock mass conditions and volumes at Seward Park. Specialty smooth wall blasting techniques would be required in an attempt to create relatively smooth vertical walls and preserve the integrity of the rock mass. Drill holes for blasting are typically on a 12- to 24-inch spacing along the walls, and a 2- to 5-foot spacing elsewhere. Blasting lifts typically do not exceed about 20 feet, due to difficulties in maintaining hole alignment. The excavation floor will also be “dimpled” to depths of 3 to 5 feet corresponding the bottom of each blast hole. With an experienced, qualified blasting crew, the vertical walls should be within $\pm$ 2 feet or better. However, an inexperienced blaster can severely damage the excavation walls and floor, resulting in overbreaks of 5 to 10 feet and possibly damaging a grouted or frozen groundwater cutoff zone around the tank excavation.

5.5 Temporary Shoring

The shallow cover of soil over locally highly weathered bedrock will require support and/or replacement of the soil and weathered rock to support the ground loads and to develop a cutoff wall to prevent inflow of lake water directly through the soil or weathered rock. Below the upper variable weathered zone, the bedrock is generally only slightly weathered and ranges from massive to highly fractured. As noted above, slug tests indicated that the fractured bedrock has relatively high permeability along fractures. Consequently, a precursor to any excavation will be to construction of an impermeable boundary between the rock excavation and Lake Washington.

This impermeable boundary might also serve as temporary or even permanent support for the tank excavation. Some potential methods of shoring and reducing permeability are discussed below.

The two potential tank excavation sites are very similar, both having an approximately 20-foot minimum buffer between the edge of the tank excavation and the lake shoreline. The proposed site located in the tennis courts (Alternative 2) appears to have a 20- to 30-foot buffer between the excavation and Lake Washington; whereas, the proposed site located in the eastern parking lot (Alternative 1) appears to have a 20- to 50-foot-wide buffer between the edge of the tank excavation and the lake.

The upper 0 to 15 feet of the site excavation will likely be in soil, underlain by up to 15 feet of weathered to slightly weathered, fractured bedrock that transitions into relatively sound, widely fractured rock, with zones of closely spaced fractures. The bedrock consists of interlayered layers of sandstone and siltstone. Each of the three borings consisted of a mix of sandstone and siltstone, with the two rock types occurring in 2 to over 10-foot-thick layers. In the borings, the bedding dips at varying inclinations from 40 to 55 degrees,

Due to the fractured nature of the bedrock and the potentially high permeability of the fractured bedrock and the close proximity to Lake Washington, the tank excavation will need to be designed to accommodate full hydrostatic head that is equivalent to the high water elevation in Lake Washington.

Shoring systems we considered for this conceptual study included secant piles, cutter soil mixing, slurry walls, and rock bolts with shotcrete.

5.5.1 Secant Piles

Secant pile walls consist of a series of intersecting drilled piles that form a continuous, semi-rigid, watertight wall. Typical construction practice is to install alternating or initial piles along the line of the wall, leaving a clear space between the initial piles of less than one pile diameter. The intermediate or secondary pile is then drilled between and into the adjacent initial piles, forming a continuous wall. The initial and secondary piles typically overlap by at least 6 inches. The initial piles are typically backfilled with lean concrete, and the secondary piles are backfilled with steel reinforcement (section or cage) and structural concrete. The advantages of secant pile walls are that they are semi-rigid and can be used as part of the final structure. If the internal structure is structurally connected to the secant pile walls, the secant pile walls can provide both vertical foundation support and resistance to hydrostatic uplift forces. During pile

installation, the piles must be carefully aligned so that openings between piles do not occur. Misalignment of piles could result in leakage, ground losses, and settlement around the excavation. In the looser and softer soils encountered in the project borings, we recommend that casing or drill mud be used to prevent the adjacent soils from caving or loosening.

For conceptual design purposes, the secant pile walls should consist of 30- to 42-inch-diameter interlocking piles. To assure that the secant piles perform as a continuous wall, a reinforced cap beam is typically installed along the top of each secant pile wall. At a minimum, the secant pile shoring for the tank excavation should extend deep enough below the base of the excavation to provide groundwater flow cutoff or head reduction. Additional explorations and testing to refine potential rock fracture geometry would be required to determine this elevation, if this shoring option is selected, and once a final tank site has been determined.

Internal bracing or tiebacks will be necessary to anchor and stabilize the walls against lateral movement. Alternatively, the walls may be embedded sufficiently to provide cantilever support. However, for cantilever support against groundwater pressures and soil and rock loading, the required embedment will be on the order of one to two times the exposed wall height.

5.5.2 Cutter Soil Mixing (CSM)

CSM is a construction process similar to deep soil mixing (DSM) that involves mixing the in situ soil with a self-hardening slurry (usually cementitious) to form cutoff or retaining walls. The result of the process is a strengthened body of soil with its boundaries roughly defined by the excavation/mixing tool. CSM differs from more traditional DSM in that the CSM mixing tool rotates about a horizontal axis, while DSM tools rotate about a vertical axis. CSM techniques can be used to form continuous walls through the construction of individual overlapping panels, installed in an alternating sequence similar to secant piles. CSM slurry mix designs vary with application and subsurface conditions, but can be designed to accommodate requirements for low permeability (using bentonite), workability, and unconfined compressive strength. The main advantages of the CSM technique are high productivity rates, little spoils, and the use of in situ soil as a construction material. The main disadvantages associated with CSM are relatively high mobilization costs associated with specialty equipment and its ability to penetrate massive rock layers. While there does seem to be some worldwide experience with CSM techniques in weak or weathered rock, this expertise is not currently locally available, and there does not appear to be any local or regional projects where the technique has been used in

similar geologic conditions. We, therefore, do not believe that CSM is currently a viable alternative, based on our discussions with local specialty contractors experienced with CSM.

5.5.3 Slurry Walls

A slurry wall is a continuous wall excavated in panels using slurry to support the soil during excavation. Each panel is backfilled with reinforced concrete before excavating adjacent panels. A slurry wall could form the permanent storage tank structural wall. Excavation equipment is typically available that is suitable for excavating many soil types, including dense sand, gravel, and soil with cobbles and boulders. This method is not commonly performed in the Puget Sound region on small shoring projects, and equipment and expertise required to construct this system in rock also does not appear to be locally available. Therefore, the required construction equipment and expertise for the relatively small excavation may have relatively large mobilization costs.

5.5.4 Rock Improvement to Reduce Permeability

If one of the tight shoring systems discussed above is not used, then the rock might be dewatered or the rock might be sealed with grouting or freezing methods. Once the rock mass is sealed against large groundwater inflows, for approximately 20 to 30 feet below planned excavation bottom, then the excavated rock surface can be supported with rock bolts and shotcrete. Either freezing or pre-grouting the fractured rock around the perimeter of the excavation would likely require the drilling of three or more rows of holes, with holes ultimately spaced 3 to 5 feet apart, to a depth of 20 to 30 feet below the planned bottom of the excavation. The depth of the holes will ultimately be designed to develop a cutoff wall to extend the flow paths and disrupt direct flow of water from the lake into the excavation. The locations of the freeze and possibly the grout holes will have to be accurately surveyed and accurately drilled in order to locate and orient the bolt holes as discussed in the next section.

With freezing, an up to 10-foot-thick wall will be developed with closely spaced freeze pipes. The freezing process will take one to two months after the system is fully installed and turned on to develop a continuous freeze wall. The freezing process will have to be maintained until a permanent tank support system is installed. Any leakage through the freeze zone, of comparatively warm groundwater, will contribute to progressive melting of the freeze wall. Also. any breaks in the freeze pipe will release brine into the groundwater, making it much more difficult to freeze.

The grouting program would occur in stages, with primary, secondary, and tertiary grout holes being grouted in 5- to 10-foot-long packer-isolated intervals from bottom to top of each hole. The primary, secondary, and tertiary stage grout holes would be utilized to locate, fully grout and validate the effective sealing of high permeability fractures and fracture zones in the bedrock. Grout materials might range from standard cement or microfine cement, to more exotic low viscosity, penetrating, chemical grouts. Grouting would be closely monitored and controlled to limit the grout volumes and pressures utilized throughout the grouting process in order to minimize the potential for grout losses into Lake Washington.

5.5.5 Rock Bolts and Shotcrete

Grouted rock bolts and shotcrete are typically used to support fractured rock in near-vertical cuts where groundwater flows have been minimized with pre-grouting or freezing, as discussed above. The spacing, length, and diameter of rock bolts are determined based on the orientation and length of joints and shear zones mapped or logged in the rock mass. The outcrops in the area indicate the presence of several joint sets. Additional borings and outcrop mapping will be needed to develop a refined rock bolt design. However, the actual presence and location of fracture-bounded rock wedges will only be determined on the basis of mapping during excavation. In the meantime, the probable presence and locations of rock wedges will be extrapolated from available information from the borings and outcrops. With fractures dipping 45 to 60 degrees, it is likely that rock bolts would be spaced 4 to 6 feet apart, and would be longest, approximately 30 feet, at the top of the cut, and decreasing in steps to as little as 15 feet near the bottom of the cut. If bolts are only needed for short-term or temporary support, over a period of one to two years until the tank structure is installed, then temporary grouted No. 7 to No. 9 bolts would likely be adequate. However, if the rock bolts are to be relied upon for long-term support, then double–corrosion protected permanent grouted bolts would be required.

Wire mesh and shotcrete would likely be installed between the bolts, to act as lagging and rock fall protection, as well as sealing the rock against slaking and deterioration. The shotcrete thickness would likely range from 3 to 6 inches, depending on the amount of overbreak that occurs during excavation, the presence of the weaker more slakable siltstone, and the depth of the cut. Approximately 4 to 6 inches of shotcrete would likely be required over the slakable siltstone layers and near the top of the cut over the weathered rock. Over the sandstone and near the bottom of the cut, 3 to 4 inches of shotcrete should be adequate.

5.6 Earth Pressures for Temporary Shoring and Permanent Walls

Shoring and permanent walls should be designed for lateral earth, groundwater, and surcharge pressures. The total design pressure acting on the shoring is the sum of these pressures. Earth pressures will depend on the type of shoring system employed, and will be provided once the system is identified. Lateral pressures against buried walls depend on many factors, including surcharge loads, soil type and density or consistency, drainage provisions, and whether or not the wall can yield or deflect laterally or rotate at the top during and after excavation. If the wall is free to yield at the top an amount equal to approximately 0.001 times the height of the wall, the soil pressures will be less (active case) than if this amount of movement is not allowed due to stiffness or resistance of the wall (at-rest condition). For conceptual engineering purposes, we recommend using an equivalent fluid weight of 34H (pounds per square foot [psf]) for active conditions, and 54H (psf) for at-rest conditions, where H is the exposed height of the wall. Full hydrostatic pressure should be assumed to act on the wall, starting at 3 feet below the ground surface (bgs).

5.7 Foundation Support

We understand that the proposed tank structure will be founded about 24 feet bgs, in siltstone comprising the Blakely Formation. We recommend a net allowable bearing capacity of 10 tons per square foot (factor of safety greater than 3.0) and a coefficient of subgrade of reaction of 350 pounds per cubic inch for the proposed structure.

5.8 Uplift Resistance

Watertight, permanent buried structures will be subjected to hydrostatic uplift pressures. Based on the geotechnical data, groundwater levels range from 3 to 5 feet bgs at the locations of the proposed storage tanks. We recommend that uplift pressures equivalent to the full hydrostatic pressure at the base of the tank be used for conceptual design purposes. Resistance to uplift can be achieved through the weight of the tank structure itself, or through structural supports or anchorages (such as piles, micropiles, or tiedowns). If the structure is connected to the shoring walls, the weight of the walls and uplift resistance along the walls can be used to resist uplift forces. Alternatively, if the structure is not connected to the shoring walls and is only supported by piles or micropiles beneath the structure footprint, the weight of the piles and uplift resistance along the piles can be used to counteract uplift forces. Ultimate uplift resistance for the chosen shoring and/or support systems can be provided once the systems have been selected.

6.0 ADDITIONAL SUBSURFACE EXPLORATIONS

Once a site has been selected and conceptual engineering studies have determined the probable structure type, dimensions, support methods, and shoring alternatives, we recommend completing an additional subsurface exploration program to provide parameters and recommendations relevant to final design. The additional subsurface explorations would focus on determining rock fracture spacing and characteristics, hydrogeologic characteristics of the rock mass, and other issues. Specifically, the additional subsurface explorations might consist of:

- Four (4) to 6 additional borings with packer-isolated groundwater permeability tests to determine groundwater inflow characteristics of specific fracture zones.
- Downhole geophysics to assess fracture spacing, orientation, thickness, and infill.
- Well pumping test(s) if dewatering is pursued as part of an excavation regime.
- Additional field mapping to locate additional rock outcrops and map and measure the orientations, surface conditions, and continuity of additional fractures and shear zones.
- Test pits to assess the condition of top of rock, the standup time of the soils, and groundwater inflow rates.

7.0 LIMITATIONS

This report was prepared for the exclusive use of HDR Engineering for specific application to the conceptual level design of facilities discussed in this report. The report is provided for information of factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the results of the subsurface explorations made for this project represent the subsurface conditions throughout the sites; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. The current preliminary explorations are not sufficient for the development of final design. We anticipate that additional explorations should and will be performed to final design of the selected project components.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or completing test pit excavations. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

The scope of our services for this report did not include any evaluation regarding the presence or absence of wetlands. Nor were assessments or evaluations regarding the presence or absence of hazardous or toxic substances in the soil, groundwater, or air on or below this site in our scope of work.

Shannon & Wilson, Inc. has prepared a document (Appendix G), "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.

Michael D. Harney, P.E.
Principal Engineer

Robert A. (Red) Robinson, L.E.G., L.G.
Senior Vice President

CWA:MDH:RAR/mdh

Items related to engineering interpretation of data were prepared by or prepared under the direct supervision of Michael D. Harney, P.E.

Items related to geological interpretation of explorations were prepared by or prepared under the direct supervision of Robert A. (Red) Robinson, L.E.G., L.G.

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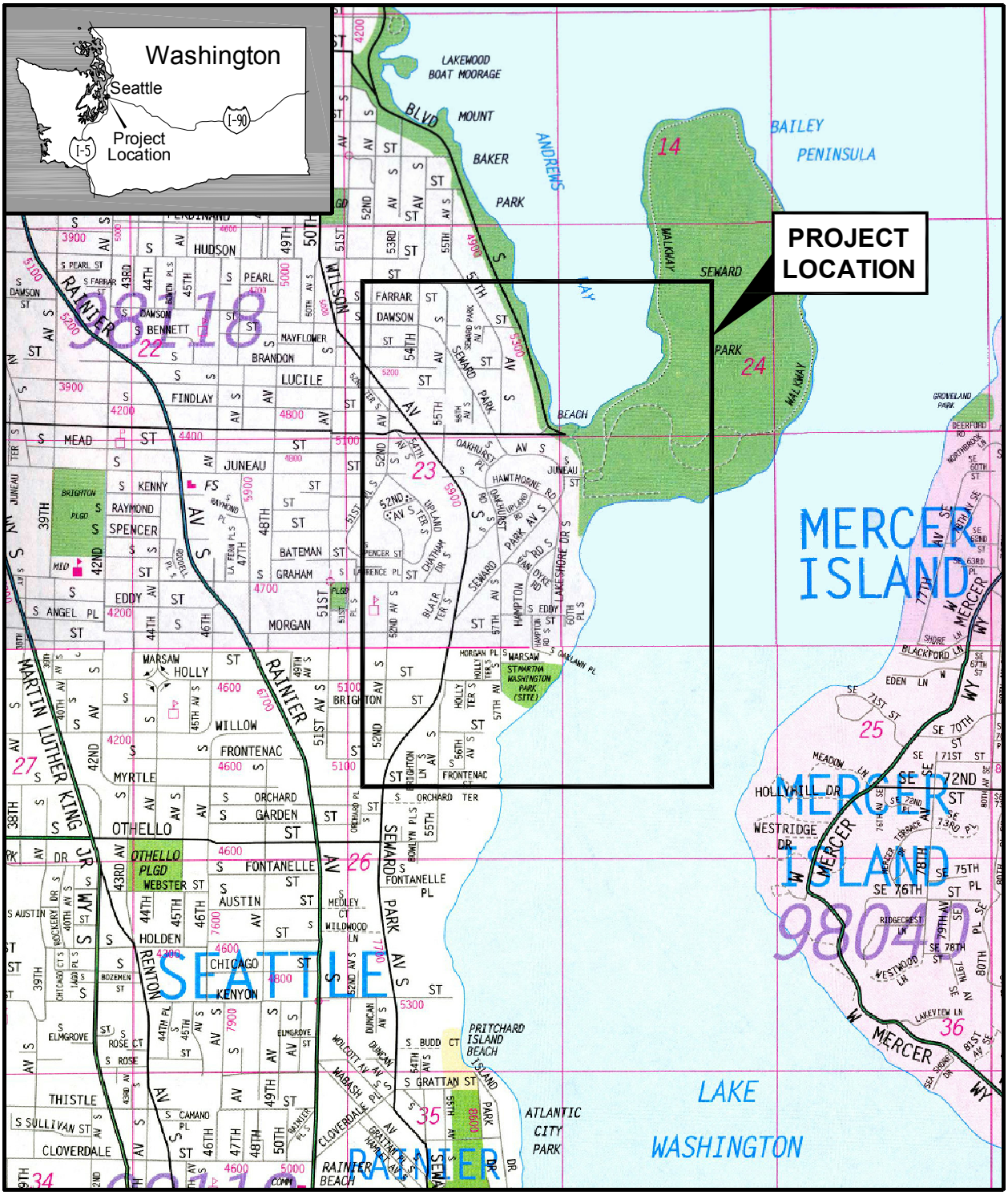

NOTE

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Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

VICINITY MAP

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FIG. 1

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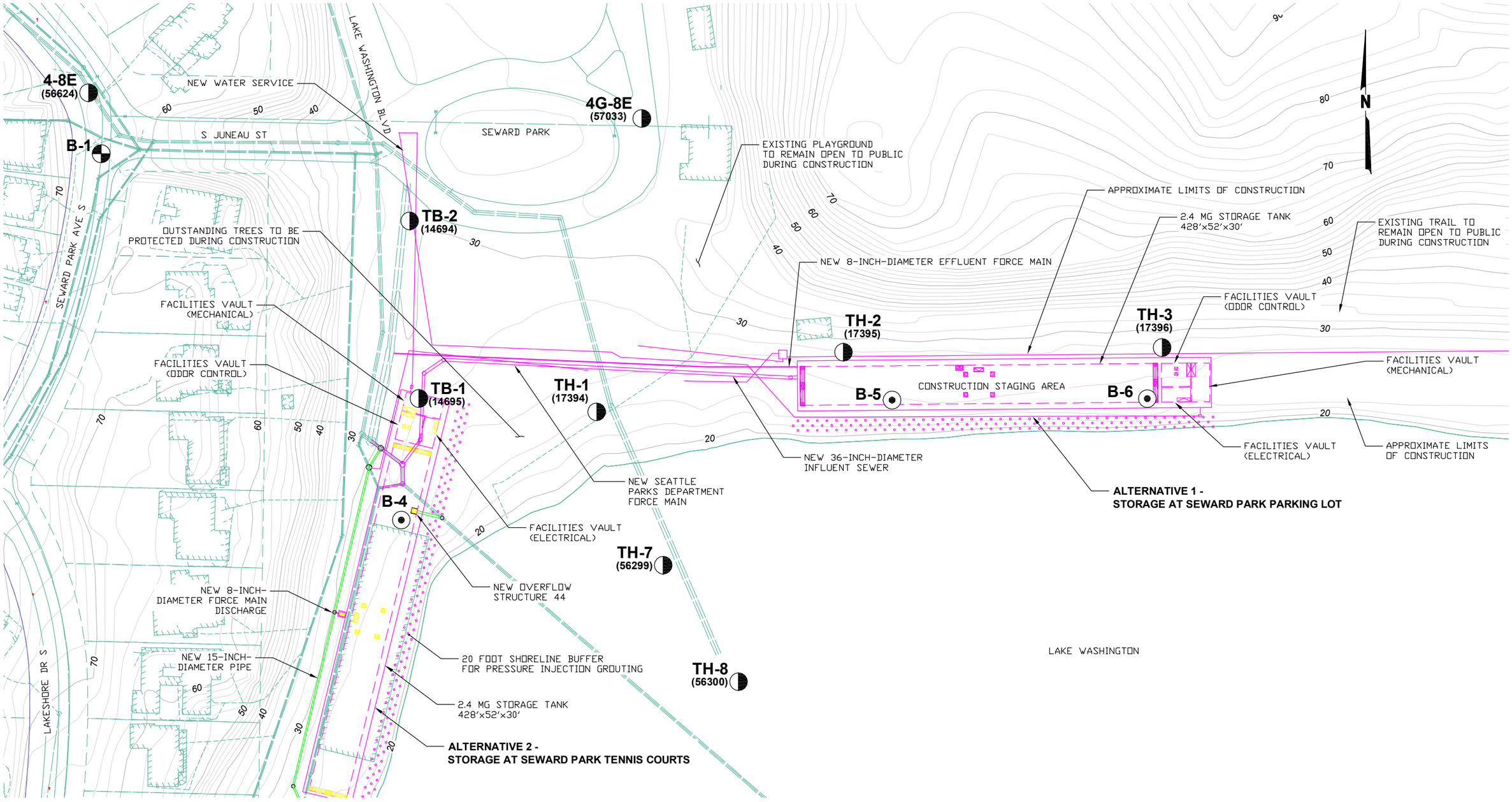

0 100 200

Scale in Feet
Contour Interval = 2 Feet

LEGEND

TH-1 (17394) — Boring Designation and Approximate Location (GeoMap Arch)

B-1 — Phase I Boring Designation and Approximate Location (Shannon & Wilson, January 2011)

B-4 — Phase II Boring Designation and Approximate Location (Shannon & Wilson, August 2011)

NOTE

Figure adapted from client files, *"C304102_X_Hen_PERC.dwg"*, *"X_REF_STORAGE_TANK_PARKING_LOT.dwg"*, and *"C304102_X_STORAGE_TANK_TENNIS_COURT.dwg"*, received 8-5-11.

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SITE AND EXPLORATION PLAN

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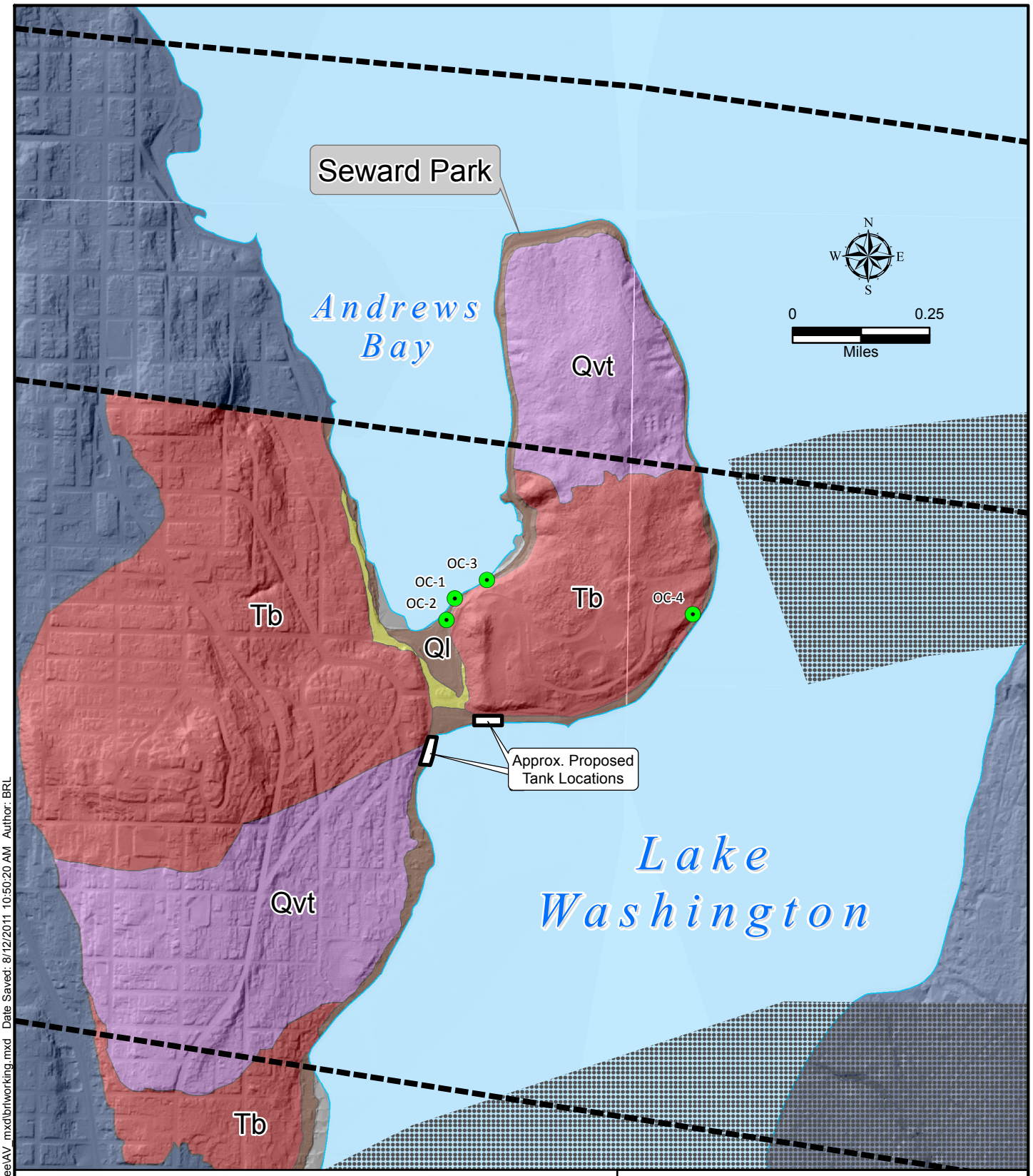
FIG. 2

Filename: J:\21\21144-007\21-1-21144-007 Site Plan.dwg Date: 09-15-2011 Login: sac



LEGEND

GEOLOGIC UNITS

- Qb — Beach Deposits
- Ql — Lake Deposits
- Qvt — Vashon Till
- Tb — Blakeley Formation
- Not In Project Area – Intentionally Blank

FAULT SPLAY AREAS

- Johnson, 1999
- Blakely, 2002

FEATURES

- Observed Outcrop Location
- Approx. Proposed Tank Locations

NOTE: Simplified 'Geologic Units' based on USGS Open File Report 2005-1252.

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GEOLOGIC MAP

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FIG. 3

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APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

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A.4 REFERENCE A-3

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A-1 Soil Classification and Log Key (2 sheets)
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APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

Three soil/rock borings, completed June 28 to July 1, 2011, were drilled for the preliminary design alternatives phase of the Henderson Basin 44 portion of the Henderson-Genesee Combined Sewer Overflow Reduction project, in Seward Park near the west shore of Lake Washington in Seattle, Washington. The locations of the borings are shown in the Site and Exploration Plan (Figure 2) presented in this report. The approximate locations shown were established by measuring distances from known physical objects in the field.

A.2 BORINGS

Three borings, designated B-4 through B-6, were drilled to evaluate subsurface conditions and develop parameters for preliminary engineering studies. The logs for the borings are presented as Figures A-2 through A-4. The Unified Soil Classification System, as described in Figure A-1, was used to classify the soils encountered in the borings.

The borings were drilled by Boart Longyear, Inc., under subcontract to Shannon & Wilson, Inc. The borings were drilled with a truck-mounted rig generally using mud-rotary drilling techniques in the upper soil deposits and using HQ coring equipment attached to the drill rods to core the underlying bedrock. Depths of drilling ranged from about 60 to about to 65 feet below the ground surface. In general, the mud-rotary drilling procedure consisted of drilling the geologic formation materials and removing the cuttings by circulation of drilling mud. The cuttings were deposited in a settling tank at the ground surface. The drilling mud used was a mixture of water and baroid-zeogel (bentonite).

Drilling through the rock was accomplished using a swivel-type, double-tube HQ core barrel. This type of sampler consists of an outer, rotating barrel and an inner, stationary barrel. The inner barrel protected the core sample from drilling fluid and torsional forces transmitted to the core sample. Drill mud consisting of de-ionizing polymer and water slurry was circulated from a mud tank, down the drill rods, through the bit, up the annular space between the drill rods and the borehole, and back into the mud tank. The circulation of drilling mud removes the cuttings from the hole and carries them to the surface where they settle out in the mud tank. The drill mud also aids in preventing caving of the drill hole during sampling.

A.3 SAMPLING METHODS

A.3.1 Soil Samples

Soil samples were generally obtained by removing the tri-cone bit and lowering a soil sampler down the casing. Split-spoon sampling was generally performed in conjunction with a Standard Penetration Test (SPT). SPTs were generally performed in the borings at 2.5-foot intervals. The tests were performed in general accordance with ASTM International (ASTM) Designation: D-1586, Standard Method for Penetration Test and Split-Barrel Sampling of Soils. The SPT consists of driving a 2-inch outside-diameter (O.D.) split-spoon sampler a total distance of 18 inches into the bottom of the boring with a 140-pound hammer falling 30 inches. The number of blows required to achieve each of three 6-inch increments of sampler penetration is recorded. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). If the penetration resistance exceeded 50 blows for 6 inches or less of penetration, the test was terminated and the number of blows was recorded along with the penetration distance.

Penetration resistance values were recorded by our field geologist and are presented graphically in the boring logs. These values provide a means for evaluating the consistency of cohesive soils and the relative density of cohesionless (granular) soils. The terminology used to describe the relative density and consistency based on the N-value is shown in Figure A-1. Soil samples obtained from the split-spoon sampler were placed in plastic jars, labeled, and transported to the S&W laboratory in Seattle, Washington, for further evaluation.

A.3.2 Core Samples

When SPT N-values, samples or drill cuttings indicated that the material in the borehole was bedrock, Boart Longyear began core drilling. Continuous core runs were obtained typically in 5-foot intervals. Rock samples were obtained using 3-inch O.D. HQ core samplers.

During coring operations, the rate of penetration, the estimated quantity of return drill mud, and the action of the drill rig (whether rough, jerky, or smooth) were recorded on the field log. Attention was paid to the drilling characteristics so that probable reasons for core loss could be recorded.

Core samples recovered from the borings were logged and placed into core boxes furnished by Boart Longyear. The core boxes were labeled and the core was arranged in descending sequence beginning at the upper left end of the core box partition and continuing in the other partitions from left to right. Each core run was separated from the preceding run by

foam or wood blocks labeled with the depth of the core interval. Zones of core loss were indicated with blocks and labeled with the interval where the loss occurred. If the zone of core loss was uncertain, the core loss was assigned to the bottom of the run. Empty spaces within the core boxes were then filled with blocks to reduce core movement.

A.4 REFERENCE

ASTM International (ASTM), 2006, Annual book of standards, construction, v. 4.08, Soil and rock, (I): D 420 – D 5611: West Conshohocken, Pa.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

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

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WOH	Weight of hammer
WOR	Weight of drill rods
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

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SOIL AND ROCK CLASSIFICATION AND LOG KEY

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FIG. A-1
Sheet 1 of 3

BORING_CLASS1 21-21144.GPJ SWNEW.GDT 8/5/11

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From USACE Tech Memo 3-357)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW		Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM		Silty gravels, gravel-sand-silt mixtures
			GC		Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW		Well-graded sands, gravelly sands, little or no fines
			SP		Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL		Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH		Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH		Inorganic clays of medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH		Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* 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.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

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SOIL AND ROCK CLASSIFICATION AND LOG KEY

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FIG. A-1
Sheet 2 of 3

BORING_CLASS2 21-21144.GPJ SWNEW.GDT 8/5/11

WEATHERING OR ALTERATION

TERM	DESCRIPTION
Fresh	No evidence of alteration
Slightly	Slight discoloration on surface
Moderately	Discoloring evident; Alteration penetrating well below rock surface
Highly	Entire rock mass discolored
Completely	Rock reduced to a soil with relict rock texture

STRENGTH

TERM	APPROX. UCS (psi x 1000)
Very Low	<0.7
Low	0.7 to 4
Moderate	4 to 7
Medium High	7 to 15
High	15 to 36
Very High	>36

JOINT ROUGHNESS COEFFICIENT (JRC)

COEFFICIENT	DESCRIPTION
14 to 20	VERY ROUGH: Near vertical edges evident
10 to 14	ROUGH: Smooth ridges, surface abrasion
6 to 10	SLIGHTLY ROUGH: Asperities on surface can be felt
2 to 6	SMOOTH: Appears and feels smooth
0 to 2	SLICKENSIDED: Visible polishing, striated surface

DISCONTINUITY DATA

SPACING	
TERM	SPACING
Very Wide	>10 ft.
Wide	3 to 10 ft.
Moderately Close	1 to 3 ft.
Close	2 in. to 1 ft.
Very Close	<2 ft.

APERTURE WIDTH	
TERM	SPACING
Very Tight	<0.1mm
Tight	0.1 to 0.25mm
Partly Open	0.25 to 0.5mm
Open	0.5 to 2.5mm
Moderately Wide	2.5 to 10mm
Wide	10mm to 1cm
Very Wide	1 to 10cm
Extremely Wide	10 to 100cm
Cavernous	>1m

DISCONTINUITY TERMS

FRACTURE - Collective term for any natural break excluding shears, shear zones, and faults

JOINT (JT) - Planar break with little or no displacement

FOLIATION JOINT (FJ) or BEDDING JOINT (BJ) - Joint along foliation or bedding

INCIPIENT JOINT (IJ) or INCIPIENT FRACTURE (IF) - Joint or fracture not evident until wetted and dried; breaks along existing surface

RANDOM FRACTURE (RF) - Natural, very irregular fracture that does not belong to a set

BEDDING PLANE SEPARATION or PARTING - A separation along bedding after extraction from stress relief or slaking

FRACTURE ZONE (FZ) - Planar zone of broken rock without gouge

MECHANICAL BREAK (MB) - Breaks due to drilling or handling; drilling break (DB), hammer break (HB)

SHEAR (SH) - Surface of differential movement evident by presence of slickensides, striations, or polishing

SHEAR ZONE (SZ) - Zone of gouge and rock fragments bounded by planar shear surfaces

FAULT (FT) - Shear zone of significant extent; differentiation from shear zone may be site-specific

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

SOIL AND ROCK CLASSIFICATION AND LOG KEY

September 2011 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 3 of 3

File: J:\211\21144-007\21-1-21144-007 Rock Log Key.dwg Date: 09-15-2011 Author: sac

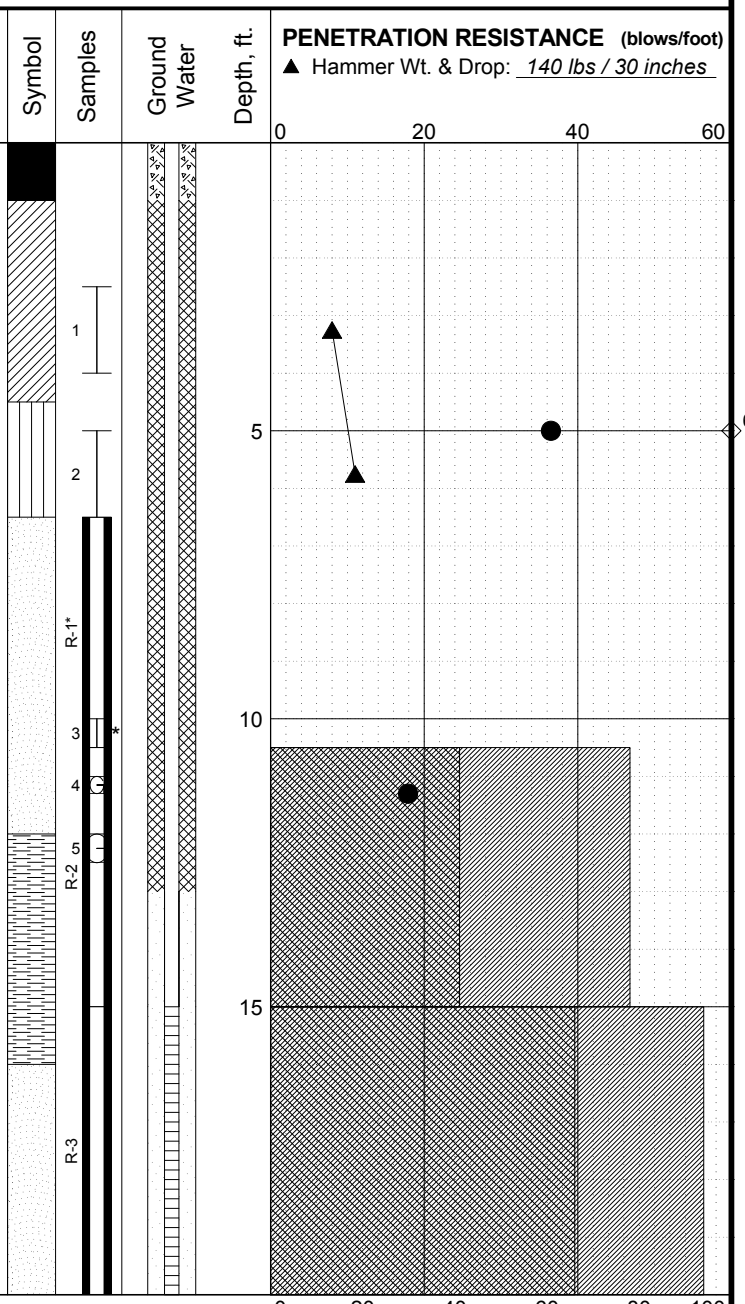
Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59	Hammer Type:	
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

Description	Depth, ft.
ASPHALT	1.0
Medium stiff, mottled brown, gray, and orange-brown, sandy, silty CLAY, trace of gravel; moist to wet; scattered iron-oxide staining; (Hf) CL.	4.5
Stiff, mottled, brown, gray-brown, and blue-gray, slightly clayey, fine sandy SILT; moist; iron-oxide staining, trace of roots and burnt organics; (Hf) ML.	6.5
SANDSTONE: Very low to low strength, gray, fine-grained, medium-spaced, low- to high-angle joints, slightly weathered to fresh; (Blakeley Formation).	12.0
SILTSTONE: Very low strength, gray, very closely spaced, low- to high-angle joints, slightly weathered to fresh; (Blakeley Formation).	16.0
SANDSTONE: Very low to moderate strength, gray, fine-grained, low- to high-angle, very closely to medium-spaced joints, slightly weathered to fresh, trace of shell fragments; (Blakeley Formation).	

CONTINUED NEXT SHEET



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

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.

Genesee - Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-4

September 2011 | 21-1-21144-007

SHANNON & WILSON, INC.
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FIG. A-2
Sheet 1 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

SOIL DESCRIPTION

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

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

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

0 20 40 60

- Bedded at 40 degrees with some joint sets along bedding between 25 and 30 feet.

30.0

SANDSTONE: Very low strength, gray, fine-grained, low- to high-angle, medium- to widely spaced joints, fresh, trace of shell fragments, black, linear features, (bedding?) at about 40 to 50 degrees;
(Blakeley Formation).

- Scattered subrounded to rounded concretions (react with HCl) between 35 to 36 feet.

- Scattered concretions up to 1.5-inch diameter observed between 35 and 40 feet. Majority of concretions were plucked during drilling, leaving pits.

CONTINUED NEXT SHEET

Samples: 6, R-4, R-5, 7, R-6, R-7, 8, 9

Depth, ft.: 25, 30, 35

0 20 40 60 80 100

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content ← (use scale at top)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

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2. Groundwater level, if indicated above, is for the date specified and may vary.
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Genesee - Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-4

September 2011 21-1-21144-007

SHANNON & WILSON, INC.
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FIG. A-2
Sheet 2 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59	Hammer Type:	
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.	PENETRATION RESISTANCE (blows/foot) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches
SANDSTONE: Very low strength, gray, no bedding observed, low angle, very closely to closely spaced joints, trace of shell fragments; (Blakeley Formation).	40.0		10 R-8			0 20 40 60
SANDSTONE: Very Low strength, gray, no bedding observed, low- to high-angle, very cllsely to closely spaced joints, trace of concretions up to 3 inches in diameter (plucked during drilling), trace of shell fragments; (Blakeley Formation).	45.0		R-9		45	
- No recovery between 52.5 and 55 feet.			11 R-10 R-11*		50	
SANDSTONE: Very low strength, dark gray, fine-grained, no bedding observed, low- to high-angle, very closely to medium-spaced joints, fresh, trace of shell fragments; (Blakeley Formation). - Highly weathered, friable zone between 57.2 and 57.5 feet. Possible shear zone.	55.0		12 R-12		55	

CONTINUED NEXT SHEET

0 20 40 60 80 100

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content (use scale at top)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

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.
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Genesee - Henderson CSO Reduction Project
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Seattle, Washington

LOG OF BORING B-4

September 2011 21-1-21144-007

SHANNON & WILSON, INC.
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FIG. A-2
Sheet 3 of 4

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

SOIL DESCRIPTION

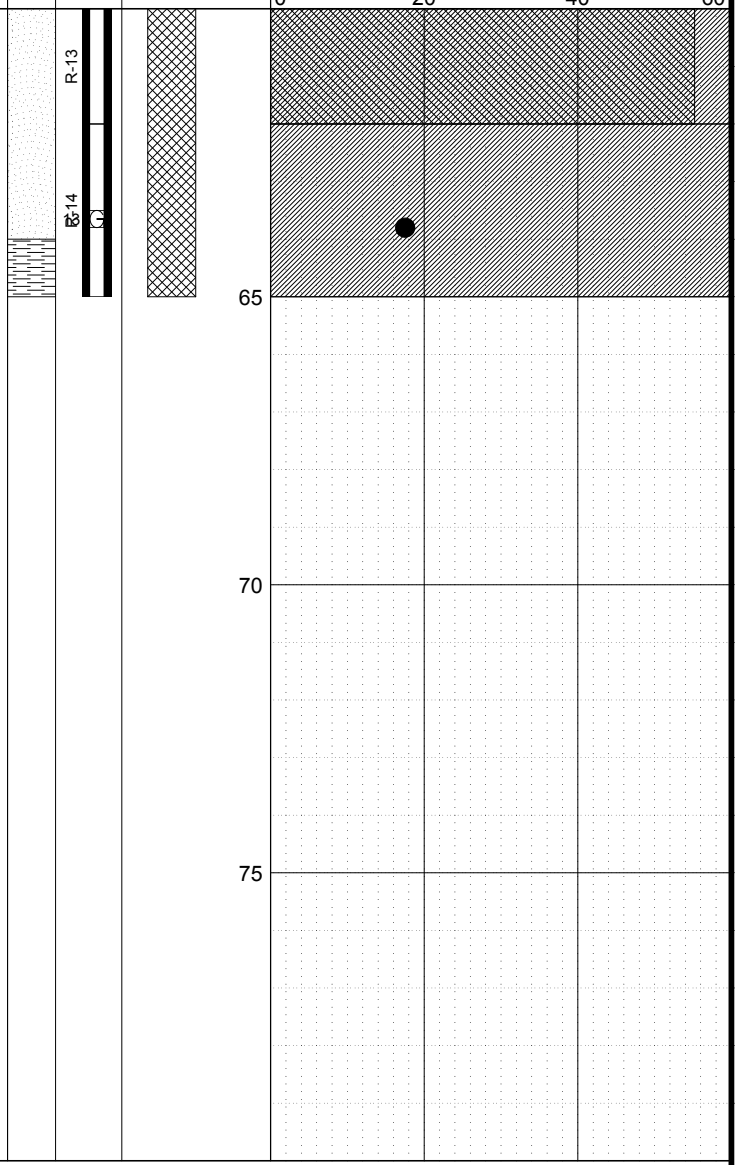
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

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

0 20 40 60



SILTSTONE: Low strength, gray, low- to high-angle, very closely to closely spaced joints, fresh; (Blakeley Formation).

BOTTOM OF BORING
COMPLETED 6/30/2011

0 20 40 60 80 100

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content (use scale at top)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

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2. Groundwater level, if indicated above, is for the date specified and may vary.
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Genesee - Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-4

September 2011 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-2
Sheet 4 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	60 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

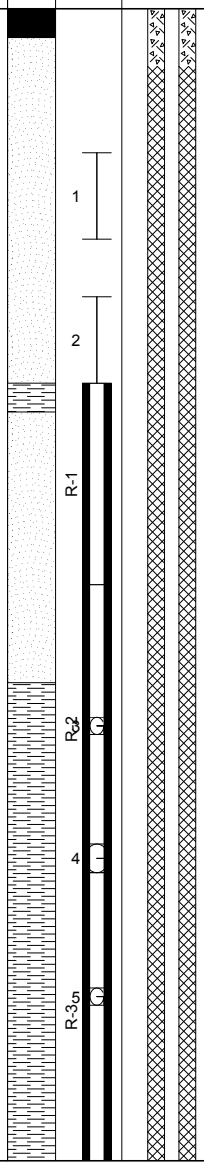
SOIL DESCRIPTION

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

Description	Depth, ft.
ASPHALT	0.5
WEATHERED SANDSTONE: Very low strength, brown to orangish brown, iron-oxide-stained, fine-grained, completely weathered, trace of roots; remolds to slightly gravelly, silty sand, trace of clay; (Blakeley Formation).	6.5
SILTSTONE: Low strength, gray, low- to high-angle, very closely spaced joints, lightly weathered to fresh; (Blakeley Formation).	7.0
SANDSTONE: Low strength, gray, fine-grained, low- to high-angle, very closely to closely spaced joints, slightly weathered to fresh, trace of shells, scattered siltstone clasts, localized zones react with HCL; (Blakeley Formation).	11.7
SILTSTONE: Very low strength, gray to light gray, low- to high-angle, very closely to medium-spaced joints, slightly weathered to fresh sandstone clasts up to 3 inches in diameter, bedding (?) at 50 degrees; (Blakeley Formation).	

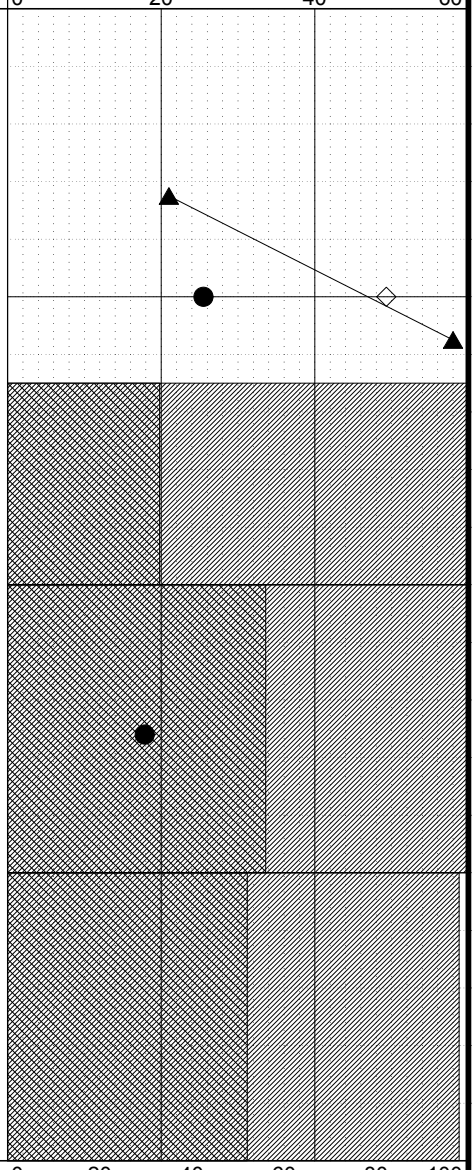
CONTINUED NEXT SHEET

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.



PENETRATION RESISTANCE (blows/foot)

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



RQD (%) Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content ← (use scale at top)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

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2. Groundwater level, if indicated above, is for the date specified and may vary.
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Genesee - Henderson CSO Reduction Project
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Seattle, Washington

LOG OF BORING B-5

September 2011 | 21-1-21144-007

SHANNON & WILSON, INC.
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FIG. A-3
Sheet 1 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	60 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59	Hammer Type:	
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 indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

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

0 20 40 60

21.5

SANDSTONE: Low strength, gray, fine- to medium-grained, no bedding observed, low- to high-angle, closely spaced jointing, fresh; (Blakeley Formation).

23.3

SILTSTONE: Low strength with very low strength from 26.3 to 26.7 feet, gray, low- to high-angle, very closely to medium-spaced joints with some quartz infilling, fresh, trace of shells;
(Blakeley Formation).
- Highly fractured between 25.2 and 27.3 feet.

27.3

SANDSTONE: Low strength, gray, no bedding observed, fine-grained, fines downward, low- to high-angle, very closely to medium-spaced joints, fresh, trace of shell fragments;
(Blakeley Formation).

- High-angle, highly fractured coal seam at 27.3 feet.
- Highly fractured between 30.6 and 31.2 feet.
- Highly fractured between 33.7 and 35.2 feet.
- Light gray 0.5-inch seam, very reactive to HCL.

CONTINUED NEXT SHEET

R-4 R-5 6 R-6 R-7 8

25 30 35

0 20 40 60 80 100

RQD (%) Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content ← (use scale at top)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

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.
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Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-5

September 2011 21-1-21144-007

SHANNON & WILSON, INC.
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FIG. A-3
Sheet 2 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

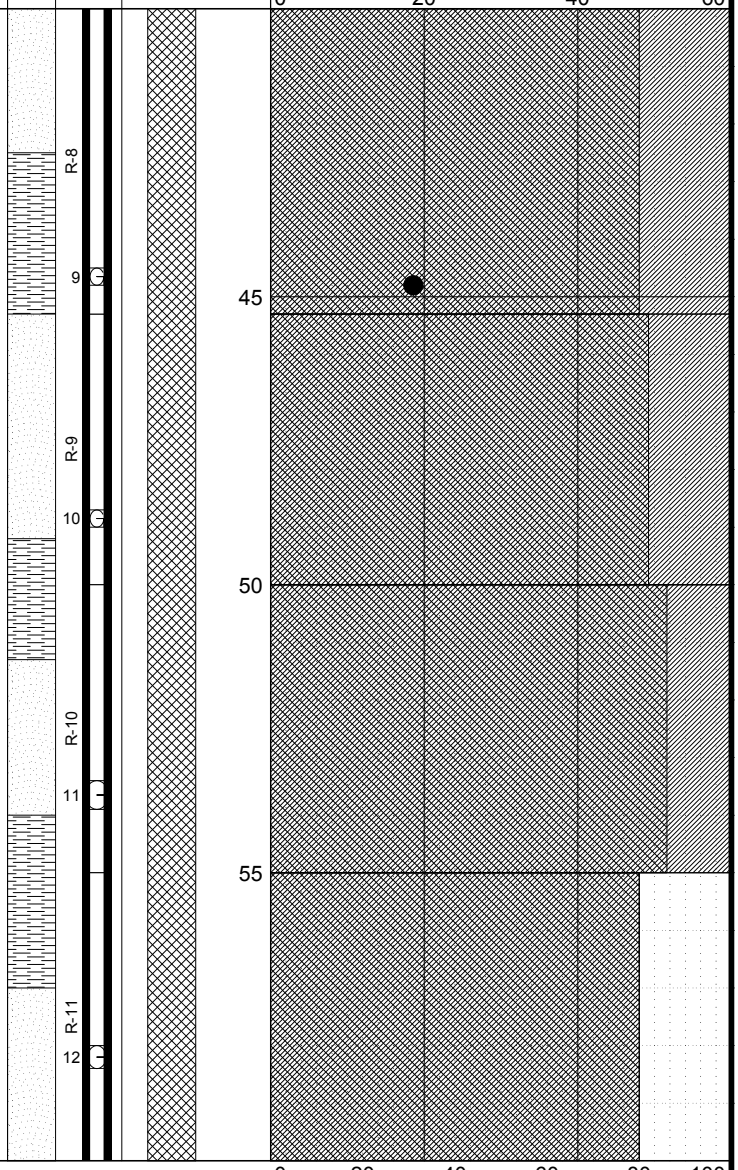
Total Depth:	60 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59	Hammer Type:	
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

Description	Depth, ft.
SILTSTONE: Very low strength, gray to light gray, low- to high-angle, very closely to widely spaced joints, fresh, trace of shells, bedded at approximately 55 degrees, chaotic in zones, localized zones react with HCL; (Blakeley Formation).	42.5
- Light gray, 3- to 4-inch seam reacts with HCL.	45.3
SANDSTONE: Low strength, gray, fine-grained, low- to high-angle, medium- to closely spaced joints, fresh, faintly bedded at approximately 50 degrees, trace of shell fragments, trace of siltstone clasts, matrix reacts with HCL in localized zones; (Blakeley Formation).	49.2
SILTSTONE: Low strength, gray to light gray, low- to high-angle, closely spaced joints, fresh, chaotic bedding, gradational contact with overlying sandstone, matrix reacts with HCL below 49.6 to 50 feet; (Blakeley Formation).	51.3
SANDSTONE: Low strength, gray to light gray, fine-grained, low-angle, closely spaced joints, fresh; bedded at approximately 50 degrees, (Blakeley Formation).	54.0
SILTSTONE: Very low strength, gray to light gray, low- to high-angle, medium-spaced joints, fresh, chaotic bedding direction trending at approximately 45 degrees, no HCL reaction; (Blakeley Formation).	57.0

CONTINUED NEXT SHEET



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

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LOG OF BORING B-5

September 2011 | 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3
Sheet 3 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	60 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.	PENETRATION RESISTANCE (blows/foot) ▲ Hammer Wt. & Drop: 140 lbs / 30 inches 0 20 40 60
SANDSTONE: Low strength, gray to light gray, fine-grained, high-angle, medium-spaced joints, fresh, trace of shells, no HCL reaction; (Blakeley Formation).	60.0					
BOTTOM OF BORING COMPLETED 7/1/2011					65 70 75	

0 20 40 60 80 100
RQD (%) Recovery (%)
◇ % Fines (<0.075mm)
● % Water Content (use scale at top)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

NOTES

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Seattle, Washington

LOG OF BORING B-5

September 2011 21-1-21144-007

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FIG. A-3
Sheet 4 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

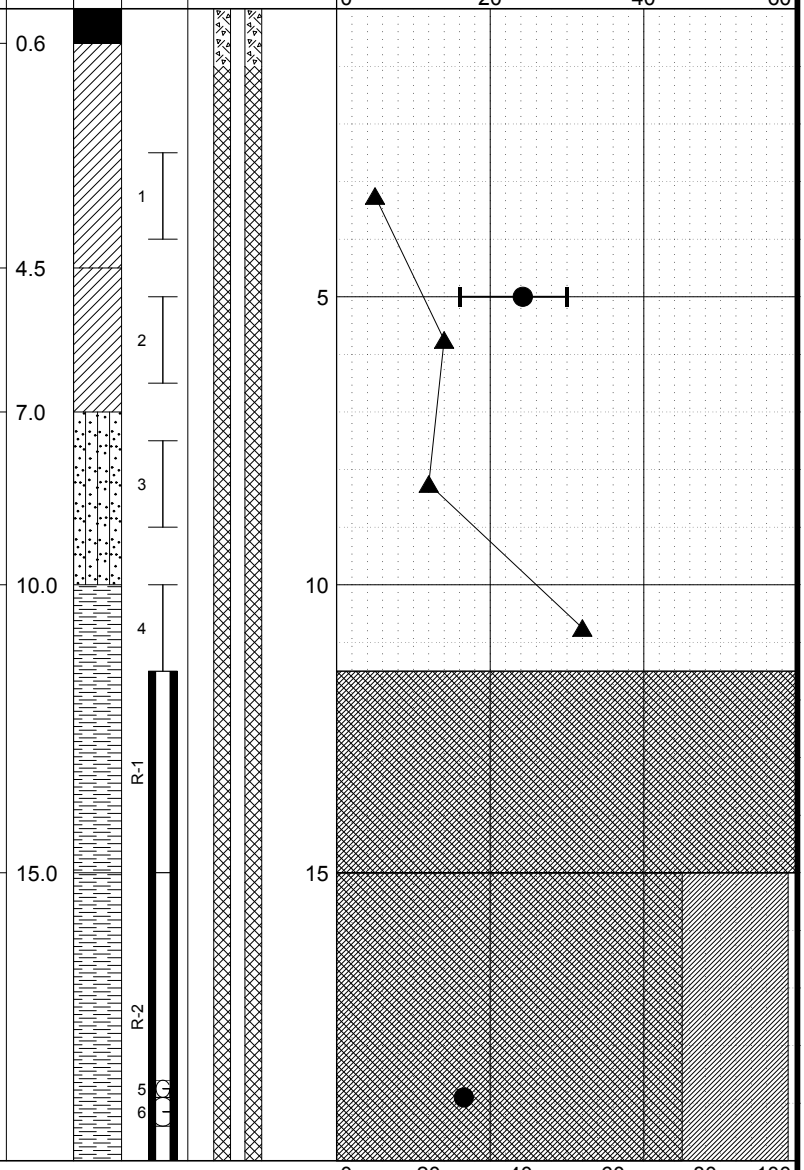
Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

SOIL DESCRIPTION

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

Depth, ft.	Soil Description
	ASPHALT
0.6	Medium stiff, mottled, olive-brown, gray, and orange, silty CLAY; moist; trace of organics, iron-oxide mottling, faintly stratified; (Hf) CL.
4.5	Stiff, mottled, olive-gray, yellowish-brown, and gray, slightly gravelly to gravelly, sandy, silty CLAY; moist; gravel is angular siltstone; (Hf) CL.
7.0	Medium dense, mottled, gray, brown, and orange-brown, silty, fine to medium SAND, trace of gravel; wet; iron-oxide-stained, angular, siltstone gravel; (Hf) SM.
10.0	WEATHERED SILTSTONE: Very low strength, gray, highly weathered angular siltstone clasts within completely weathered matrix, low- to high-angle joints; (Blakeley Formation).
15.0	SILTSTONE: Very low strength, gray, low- to high-angle, very closely to closely spaced joints, moderately to highly weathered; (Blakeley Formation).

CONTINUED NEXT SHEET



● % Water Content ←(use scale at top)
Plastic Limit ⊢●⊣ Liquid Limit

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

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.
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Genesee - Henderson CSO Reduction Project
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Seattle, Washington

LOG OF BORING B-6

September 2011 | 21-1-21144-007

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FIG. A-4
Sheet 1 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

SOIL DESCRIPTION

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

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

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

0 20 40 60

21.7

SILTSTONE: Very low to low strength, gray, low- to high-angle, closely to moderately closely spaced joints, moderately to slightly weathered;
(Blakeley Formation).

- Small shell fragments at 25 feet.
- Very closely spaced joints between 27 and 29.5 feet.
- Highly fractured jointed zone from 29.5 to 30 feet.
- No core recovery between 30 and 35 feet.

36.9

SANDSTONE: Very low strength, gray, fine-grained, low- to high-angle, medium- to closely spaced joints, fresh, trace of shell fragments;
(Blakeley Formation).

39.0

CONTINUED NEXT SHEET

R-3 | 7 | R-4 | R-5* | 8 | R-6

25 30 35

0 20 40 60 80 100

RQD (%) Recovery (%)

● % Water Content ←(use scale at top)

Plastic Limit ├─●─┤ Liquid Limit

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

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.
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Genesee - Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-6

September 2011 21-1-21144-007

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FIG. A-4
Sheet 2 of 4

MASTER LOG E 21-21144.GPJ SHAN WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59	Hammer Type:	
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 indicated below represent the approximate boundaries between material types, and the transition may be gradual.

- HCL bubbles from 37.2 to 37.4 feet.

SILTSTONE: Very low to low strength, gray, low- to high-angle, medium- to very closely spaced joints, fresh, trace of shell fragments, trace of angular to subangular coarse sand in siltstone matrix;
(Blakeley Formation).
- Fault gouge (?) between 39.4 and 49 feet.
- HCL reaction at 39.4 feet.
- HCL reaction from 40 to 49 feet on white flecks (shell fragments?). No HCL reaction on matrix.

48.2

SANDSTONE: Very low strength, gray, fine-grained, low- to high-angle, closely to very closely spaced joints, fresh, trace of shell fragments inclined at about 40 degrees;
(Blakeley Formation).

51.0

SILTSTONE: Very low to low strength, gray, high-angle, closely to medium-spaced joints, fresh, trace of shell fragments and angular to subangular coarse sand to fine gravel in siltstone matrix;
(Blakeley Formation).

52.7

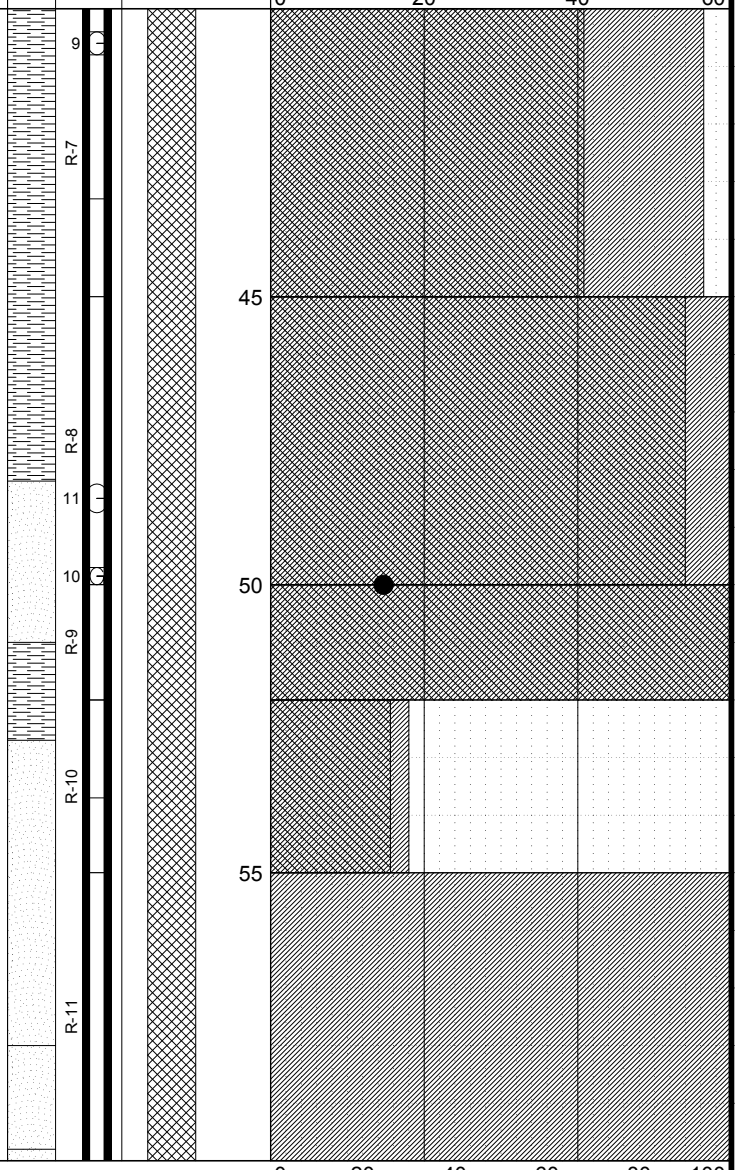
SANDSTONE: Low strength, gray, fine-grained, low- to high-angle, close to very closely spaced joints, fresh, trace of shell fragments 40 degrees;
(Blakeley Formation).
- Infilling with silty soil and angular bedrock clasts (shear zone?) from 56.7 to 56.9 feet.

58.0

SANDSTONE: Low strength, gray, fine-grained, low-angle, closely to medium-spaced joints, fresh, trace of shell

CONTINUED NEXT SHEET

59.8



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

● % Water Content ←(use scale at top)

Plastic Limit ├──●──┤ Liquid Limit

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.

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Seattle, Washington

LOG OF BORING B-6

September 2011 | 21-1-21144-007

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FIG. A-4
Sheet 3 of 4

Log: CWA Rev: AJC Typ: CLP

MASTER LOG E 21-21144.GPJ SHAN WIL.GDT 9/15/11

Total Depth:	65 ft.	Northing:		Drilling Method:	Mud Rotary & Rock Core
Top Elevation:	~	Easting:		Drilling Company:	Boart Longyear
Vert. Datum:		Station:		Drill Rig Equipment:	Foremost Mobile B-59
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.:
Hammer Type:

SOIL DESCRIPTION

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

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

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

0 20 40 60

fragments; (Blakeley Formation).
- Highly fractured/jointed between 58.5 to 59.8 feet.

SANDSTONE: Low strength, gray, fine-grained, low-angle, closely- to medium-spaced joints, fresh, trace of shell fragments; (Blakeley Formation).

R-12

65.0

BOTTOM OF BORING
COMPLETED 6/28/2011

65

70

75

0 20 40 60 80 100

RQD (%) Recovery (%)

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Rock Core
- Grab Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

● % Water Content ←(use scale at top)

Plastic Limit ├─●─┤ Liquid Limit

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.

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Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-6

September 2011 21-1-21144-007

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FIG. A-4
Sheet 4 of 4

MASTER_LOG_E_21-21144.GPJ SHAN_WIL.GDT 9/15/11 Log: CWA Rev: AJC Typ: CLP

APPENDIX B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

APPENDIX B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

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APPENDIX B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

B.1 INTRODUCTION

This appendix contains descriptions of the procedures and the results of the geotechnical laboratory tests performed on soil samples obtained from the borings performed for the preliminary design alternatives phase of the Henderson Basin 44 portion of the Henderson-Genesee Combined Sewer Overflow Reduction project near the west shore of Lake Washington in Seward Park in Seattle, Washington. The samples were tested to determine the basic index and physical properties of the site soils and rock. The laboratory testing was performed by an engineer or an experienced technician at the Shannon & Wilson, Inc. laboratory in Seattle in July and August 2011.

B.2 VISUAL CLASSIFICATION

All of the soil samples recovered from the borings were visually reclassified in our laboratory using a system based on ASTM International (ASTM) Designation: D-2487, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D-2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). This visual classification method allows for convenient and consistent comparison of soils from widespread geographic areas. Using this method, the soils can be classified by using the Unified Soil Classification System. The individual sample classifications have been incorporated into the boring logs presented in Appendix A.

B.3 WATER CONTENT DETERMINATION

The natural water content of selected soil samples recovered from the borings were determined in general accordance with ASTM Designation: D-2216, Standard Method of Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Comparison of natural water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. The water contents are plotted on the boring logs presented in Appendix A.

B.4 GRAIN SIZE ANALYSIS

Grain size analyses were performed on selected samples of granular soil in general accordance with ASTM Designation: D-422, Standard Method for Particle-Size Analysis of Soils. Three

general procedures to determine the grain size distribution of a soil include sieve analysis, hydrometer analysis, and combined analysis.

Grain size distribution is used to assist in classifying soils and evaluating their liquefaction potential, and to provide correlation with soil properties, including permeability and capillarity. The results of the grain size analyses are plotted on the grain size distribution curves presented in Figure B-1.

B.5 ATTERBERG LIMITS DETERMINATION

The Atterberg limits were determined on selected samples of fine-grained soil obtained in the field explorations in general accordance with ASTM Designation: D-4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). They are generally used to assist in classification of soils, indicate soil consistency (when compared with natural water content), and provide correlation to soil properties including compressibility and strength. The results of the Atterberg limits determination are shown graphically on the plasticity chart presented in Figure B-2.

B.6 POINT LOAD TESTING

Point load tests were performed on rock core samples obtained from the borings. The point load tests were performed in general accordance with ASTM D 5731, Standard Test Method for Determination of the Point Load Strength Index of Rock and Application to Rock Strength Classifications. The point load test was used as an index test for strength classification of bedrock. The bedrock samples were tested by application of concentrated load through a pair of conical platens. Point load data is presented in Table B-1.

B.7 UNCONFINED COMPRESSION TEST

GeoTesting Express performed unconfined compressive strength tests on five samples in general accordance with ASTM D 7012 Method D, Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens. We chose samples based on rock type (siltstone or sandstone), location (boring location and sample depth), and degree of weathering. Unconfined compressive strength test results and associated laboratory data are presented in Figures B-3 through B-17.

B.8 JAR SLAKE TEST

Jar Slake testing of core specimens were performed in accordance with Colorado Procedure 26-90, which uses the following descriptive behavior to classify soil durability using the Jar Slake test (Method A).

Behavior Index, I_J	Behavior Description	Behavior Index, I_J	Behavior Description
1	Degrades into a pile of flakes or mud	4	Breaks slowly and/or forms several fractures
2	Breaks rapidly and/or forms many chips	5	Breaks slowly and/or forms few fractures
3	Breaks rapidly and/or forms few chips	6	No change

A small, oven-dry specimen is selected and placed in a beaker of water. Observations are made of the degradation of the sample and a descriptive value assigned based on breakdown of the specimen. The final value is assigned after a period of 24 hours. Results of Jar Slake testing are included on boring logs in Appendix A.

B.8 REFERENCE

ASTM International (ASTM), 2006, Annual book of standards, construction, v. 4.08, Soil and rock, (I): D 420 – D 5611: West Conshohocken, Pa.

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Table B-1: Point Load Test

ASTM D5731-08

Project Name: Henderson/Genesee CSO
Job Number: 21-1-21144-007

Tested By: JJM 7/21/11
Checked By: JFL 7/25/11

Boring No.	Depth Interval (feet) From	Depth Interval (feet) To	Type of Test (D/A/B/L)	Diameter D (inches)	Point Load P (lbs)	Type of Break (I/S/C)	Break Remarks	Is (psi)	Is(50) (psi)	UCS (psi)
B-4	11	11.3	D	2.37	241	S	Broke lengthwise	43	47	1160
B-4	20	20.3	D	2.38	1800	S	Broke lengthwise	318	346	8540
B-4	30	30.4	D	2.37	463	I		82	89	2190
B-4	39	39.5	D	2.35	600	I		109	118	2900
B-4	42	42.3	D	2.34	263	C	Multiple break points	48	52	1270
B-4	52.1	52.5	D	2.36	587	I		105	114	2800
B-4	63.5	63.8	D	2.35	148	I		27	29	710
							Arithmetic Mean of all B-4 Point Load Tests=			2800
							Arithmetic Mean of all Intact and Combined Failure B-4 Point Load Tests=			1970
							Median of all Intact and Combined Failure B-4 Point Load Tests=			2190
B-5	12.3	12.6	D	2.34	234	S	Possible clay with sand lenses	43	46	1130
B-5	17	17.3	D	2.36	434	S	Did not go through both contact points	78	85	2090
B-5	28.7	29	D	2.36	1110	C		199	216	5310
B-5	39.7	40	D	2.36	976	I		175	190	4670
B-5	44.5	44.8	D	2.37	701	C	Multiple break points	125	136	3350
B-5	48.7	49	D	2.36	888	I		159	172	4230
B-5	58	58.4	D	2.36	357	S	Broke immediately	64	69	1700
							Arithmetic Mean of all B-5 Point Load Tests=			3210
							Arithmetic Mean of all Intact and Combined Failure B-5 Point Load Tests=			4390
							Median of all Intact and Combined Failure B-5 Point Load Tests=			4450
B-6	18.6	18.9	D	2.32	205	S	Very soft	38	41	1000
B-6	25.7	26	D	2.32	225	C	Break began intact, finished structural	42	45	1100
B-6	36.6	36.9	D	2.3	124	S	Pre-existing crack/joint	23	25	610
B-6	40.1	40.4	D	2.3	171	S	Pre-existing crack/joint	32	34	830
B-6	49.7	50	D	2.31	209	S	Broke into multiple pieces	39	42	1020
B-6	53.3	53.7	D	2.32	355	I	Soft	66	71	1730
							Arithmetic Mean of all B-6 Point Load Tests=			1050
							Arithmetic Mean of all Intact and Combined FailureB-6 Point Load Tests=			1420
							Median of all Intact and Combined Failure B-6 Point Load Tests=			1420
							Arithmetic Mean of all Point Load Tests=			2420

Type of Test:
D = Diametral
A = Axial
B = Block

Type of Break:
I = Intact
S = Structure
C = Combined

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Table B-1 (continued): Point Load Test

ASTM D5731-08

Project Name: Henderson/Genesee CSO
Job Number: 21-1-21144-007

Tested By: JJM 7/21/11
Checked By: JFL 7/25/11

Boring No.	Depth Interval (feet) From	Depth Interval (feet) To	Type of Test (D/A/B/L)	Diameter D (inches)	Point Load P (lbs)	Type of Break (I/S/C)	Break Remarks	Is (psi)	Is(50) (psi)	UCS (psi)
B-4	30	30.4	D	2.37	463	I		82	89	2190
B-4	39	39.5	D	2.35	600	I		109	118	2900
B-4	42	42.3	D	2.34	263	C	Multiple break points	48	52	1270
B-4	52.1	52.5	D	2.36	587	I		105	114	2800
B-4	63.5	63.8	D	2.35	148	I		27	29	710
B-5	28.7	29	D	2.36	1111	C		199	216	5310
B-5	39.7	40	D	2.36	976	I		175	190	4670
B-5	44.5	44.8	D	2.37	701	C	Multiple break points	125	136	3350
B-5	48.7	49	D	2.36	888	I		159	172	4230
B-6	25.7	26	D	2.32	225	C	Break began intact, finished structural	42	45	1100
B-6	53.3	53.7	D	2.32	355	I	Soft	66	71	1730

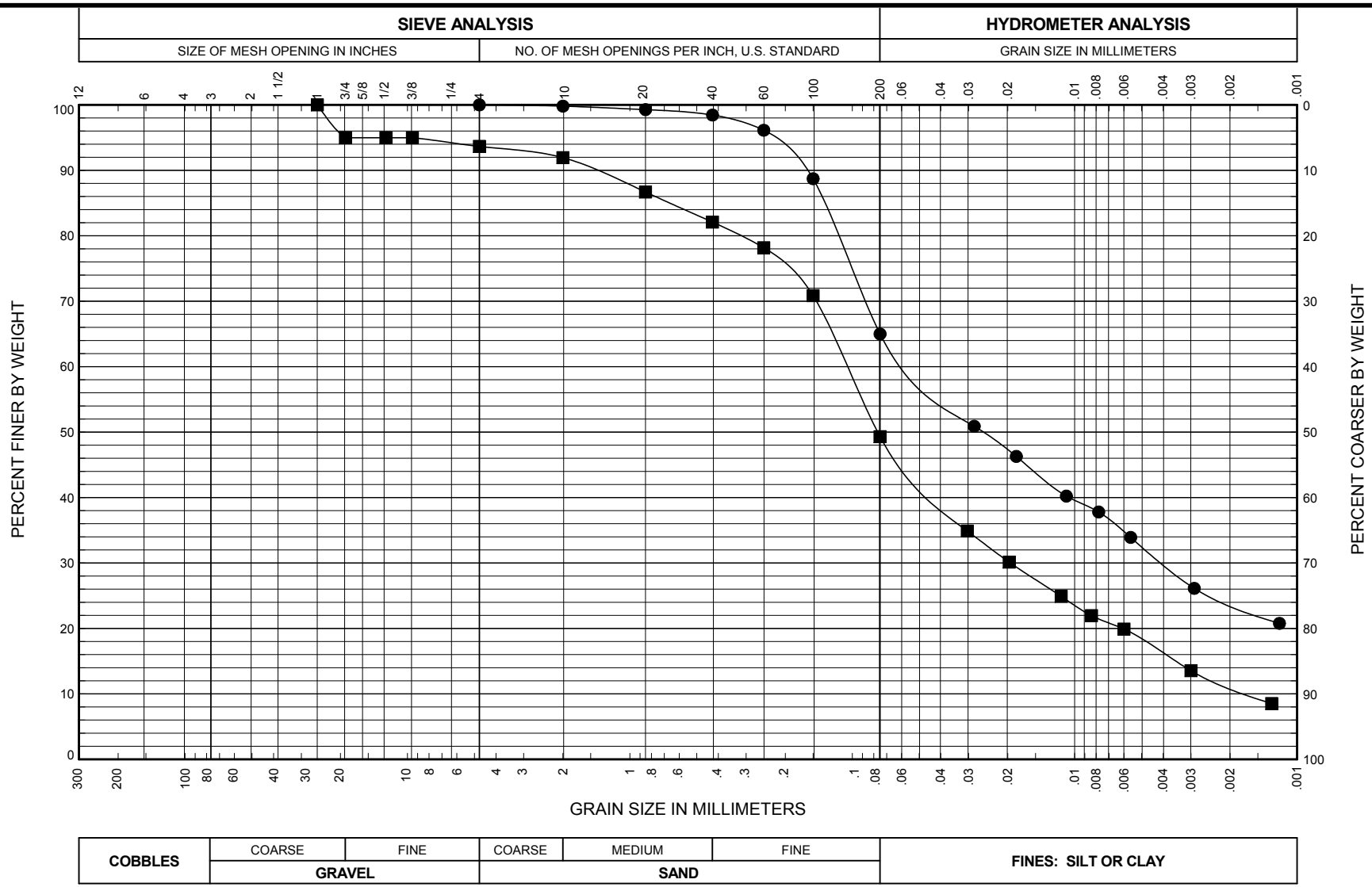
Arithmetic Mean of all Intact and Combined Failure Point Load Tests= 2750
Median of all Intact and Combined Failure Point Load Tests= 2800

B-4	11	11.3	D	2.37	241	S	Broke lengthwise	43	47	1160
B-4	20	20.3	D	2.38	1798	S	Broke lengthwise	318	346	8540
B-5	12.3	12.6	D	2.34	234	S	Possible clay with sand lenses	43	46	1130
B-5	17	17.3	D	2.36	434	S	Did not go through both contact points	78	85	2090
B-5	58	58.4	D	2.36	357	S	Broke immediately	64	69	1700
B-6	18.6	18.9	D	2.32	205	S	Very soft	38	41	1000
B-6	36.6	36.9	D	2.3	124	S	Pre-existing crack/joint	23	25	610
B-6	40.1	40.4	D	2.3	171	S	Pre-existing crack/joint	32	34	830
B-6	49.7	50	D	2.31	209	S	Broke into multiple pieces	39	42	1020

Arithmetic Mean of all Structural Failure Point Load Tests= 2010
Median of all Structural Failure Point Load Tests= 1130

Type of Test:
D = Diametral
A = Axial
B = Block

Type of Break:
I = Intact
S = Structure
C = Combined



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-4, S-2	5.0	ML	Gray and brown, slightly clayey, fine sandy SILT; oxidation staining	65.0	36.5			
■ B-5, S-2	5.0	SM	Brown, slightly gravelly, silty SAND, trace of clay; oxidation staining, scattered organics	49.3	25.5			

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Seattle, Washington

GRAIN SIZE DISTRIBUTION

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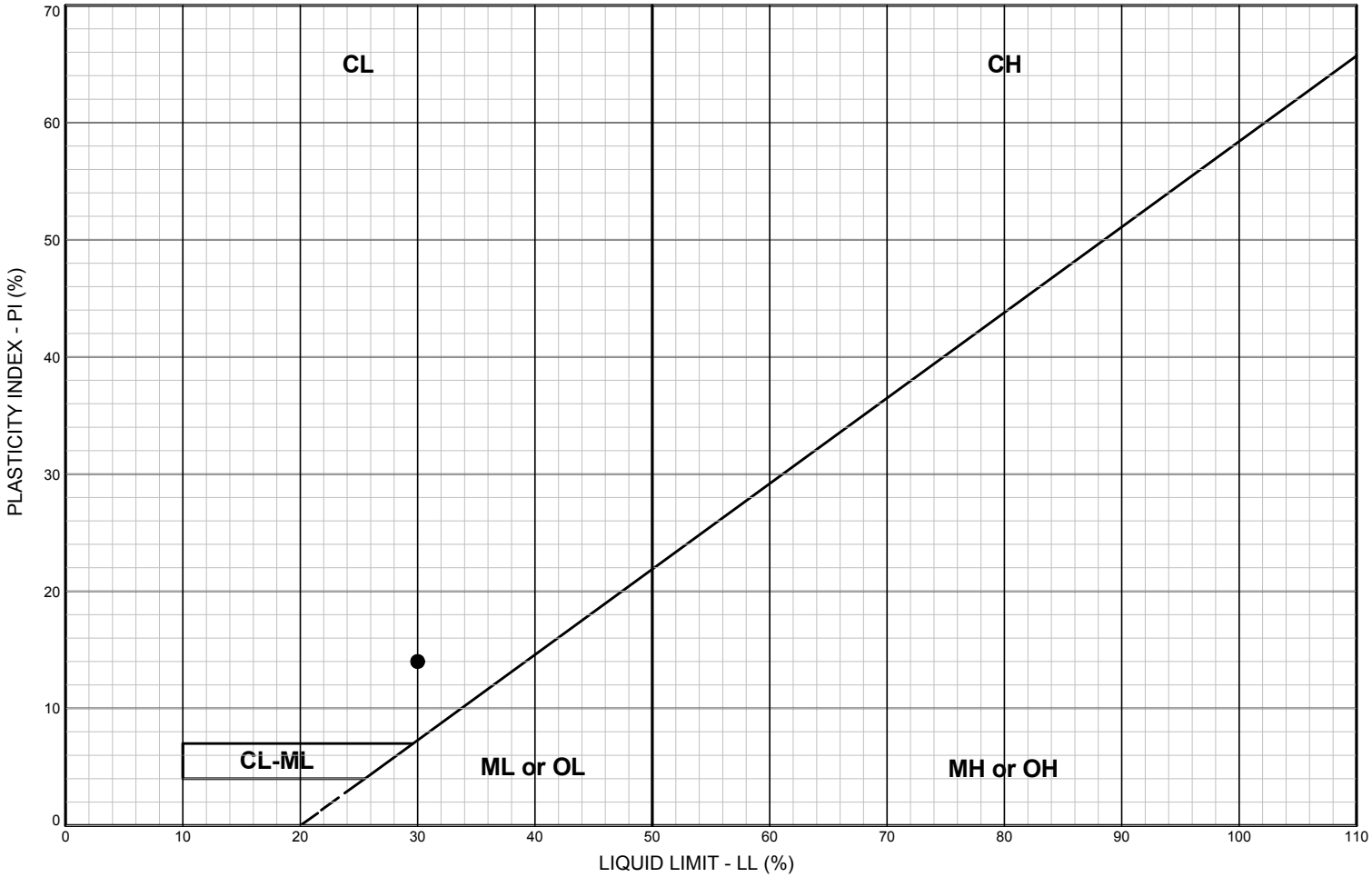
FIG. B-1

FIG. B-1

GSA_MAIN 21-21144.GPJ SHAN_WIL.GDT 9/15/11



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● B-6, S-2	5.0	CL	Brown, gravelly, sandy, silty CLAY	30	16	14	24.2	

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PLASTICITY CHART

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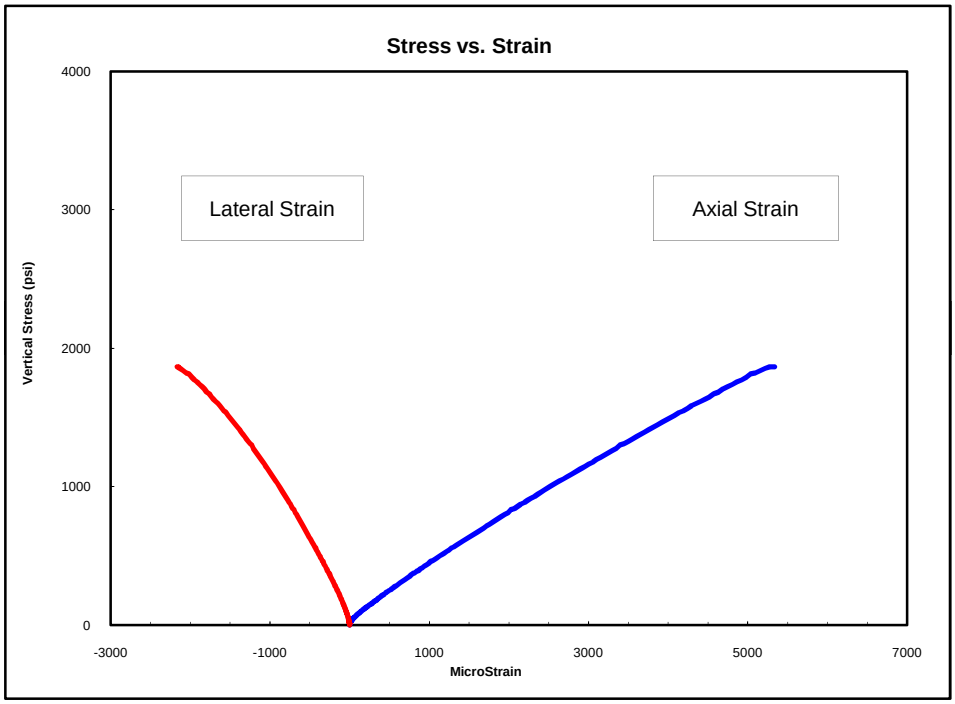
FIG. B-2

FIG. B-2

ATT_MAIN 21-21144.GPJ SHAN WIL.GDT 9/15/11

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-4
Sample ID:	S-8
Depth, ft:	38.60-39.00
Sample Type:	rock core
Sample Description:	See photographs Intact material failure L/D < 2

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 1,866 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-500	407,000	0.34
500-1000	363,000	0.37
1000-15000	330,000	0.41

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Ausenco Vector	Test Date:	08/04/11
Project Name:	Henderson-Genesee	Tested By:	daa
Project Location:	---	Checked By:	mpd
GTX #:	11031		
Boring ID:	B-4		
Sample ID:	S-8		
Depth:	38.60-39.00 ft		
Visual Description:	See Photographs	BEST EFFORT END PREPARATION	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.29	4.32	4.31
Specimen Diameter, in:	2.35	2.36	2.36
Specimen Mass, g:	638.49		
Bulk Density, lb/ft^3	129		
Length to Diameter Ratio:	1.83		

DEVIATION FROM STRAIGHTNESS

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Straightness Tolerance Met? YES

END FLATNESS

END 1

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

END 2

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

End Flatness Tolerance Met? YES

PARALLELISM Spherically Seated

END 1

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.008	Angle of Departure,°:	0.11	*Angle of departure < 0.25°?*	**YES**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.024	Angle of Departure,°:	0.32	*Angle of departure < 0.25°?*	**NO**

END 2

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.019	Angle of Departure,°:	0.25	*Angle of departure < 0.25°?*	**NO**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.01	Angle of Departure,°:	0.13	*Angle of departure < 0.25°?*	**YES**

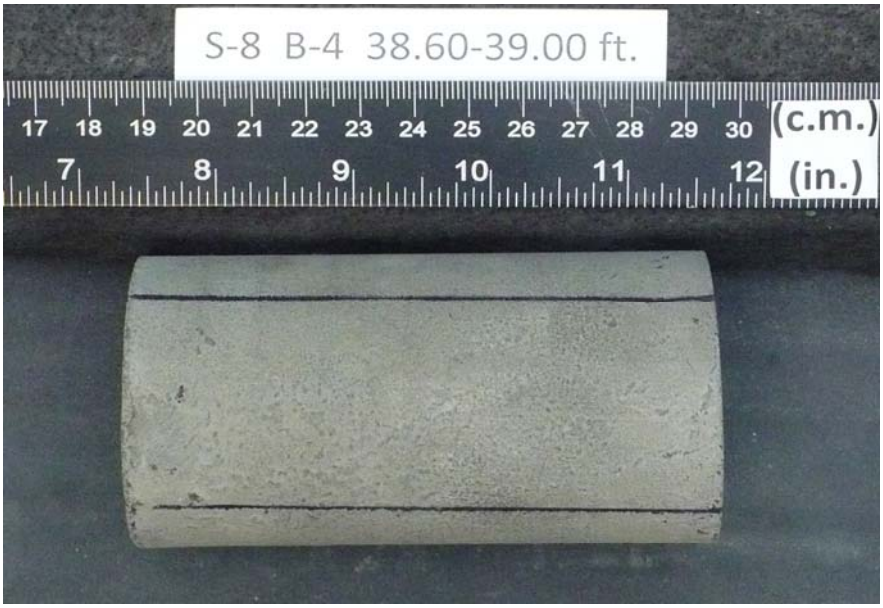
Parallelism Tolerance Met? NO

PERPENDICULARITY Spherically Seated

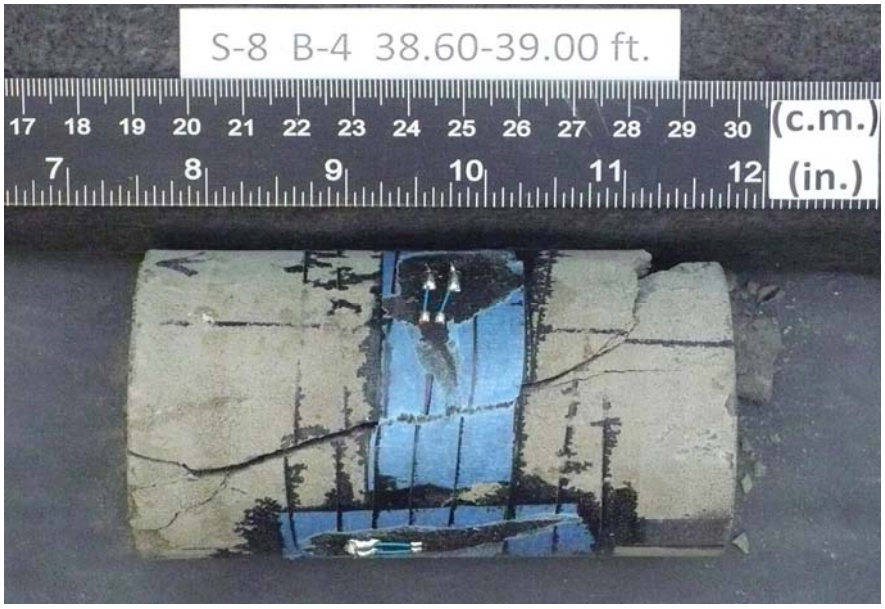
Maximum Gap Between Square and Top of Specimen, in = 0.094 Angle of Departure,°: 1.25 *Maximum angle of departure must be < 0.25°*

Perpendicularity Tolerance Met? NO

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-4
Sample ID:	S-8
Depth, ft:	38.60-39.00



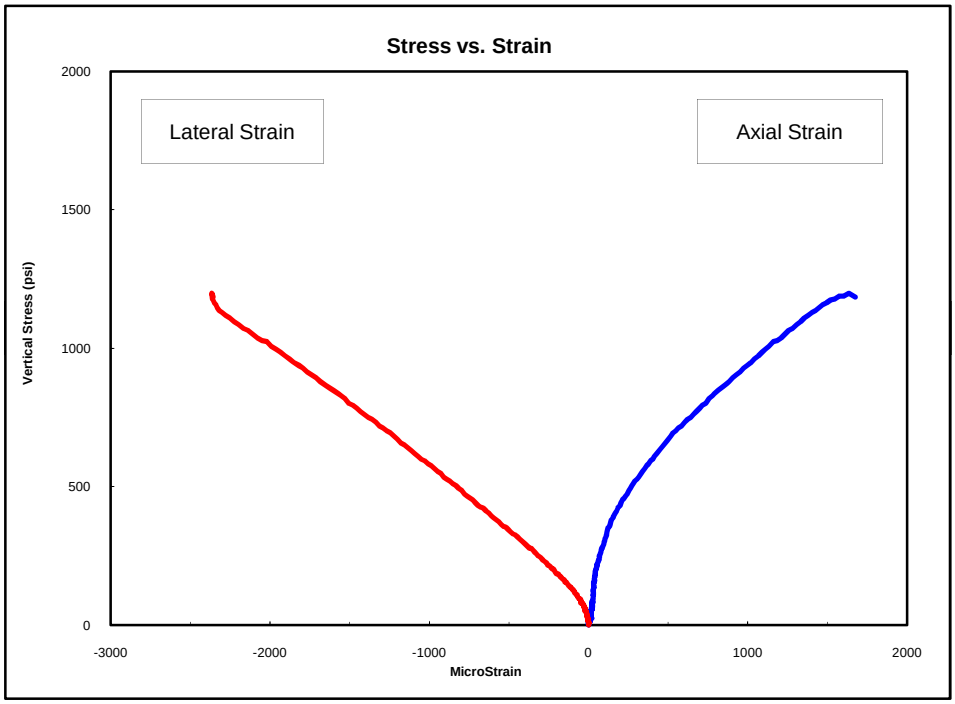
After cutting and grinding



After break

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-4
Sample ID:	S-12
Depth, ft:	57.82-58.26
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 1,198 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-500	1,710,000	---
500-1000	581,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Ausenco Vector	Test Date:	08/03/11
Project Name:	Henderson-Genesee	Tested By:	daa
Project Location:	---	Checked By:	mpd
GTX #:	11031		
Boring ID:	B-4		
Sample ID:	S-12		
Depth:	57.82-58.26 ft		
Visual Description:	See Photographs	BEST EFFORT END PREPARATION	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.47	4.49	4.48
Specimen Diameter, in:	2.29	2.27	2.28
Specimen Mass, g:	624.64		
Bulk Density, lb/ft^3	130		
Length to Diameter Ratio:	2.0		

DEVIATION FROM STRAIGHTNESS

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? NO

Straightness Tolerance Met? NO

END FLATNESS

END 1

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

END 2

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

End Flatness Tolerance Met? YES

PARALLELISM Spherically Seated

END 1

Diameter 1	Gap Between Square and Top of Specimen, in	<	0.001	Angle of Departure,°:	0.01	*Angle of departure < 0.25°?*	**YES**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.02	Angle of Departure,°:	0.26	*Angle of departure < 0.25°?*	**NO**

END 2

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.02	Angle of Departure,°:	0.26	*Angle of departure < 0.25°?*	**NO**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.028	Angle of Departure,°:	0.36	*Angle of departure < 0.25°?*	**NO**

Parallelism Tolerance Met? NO

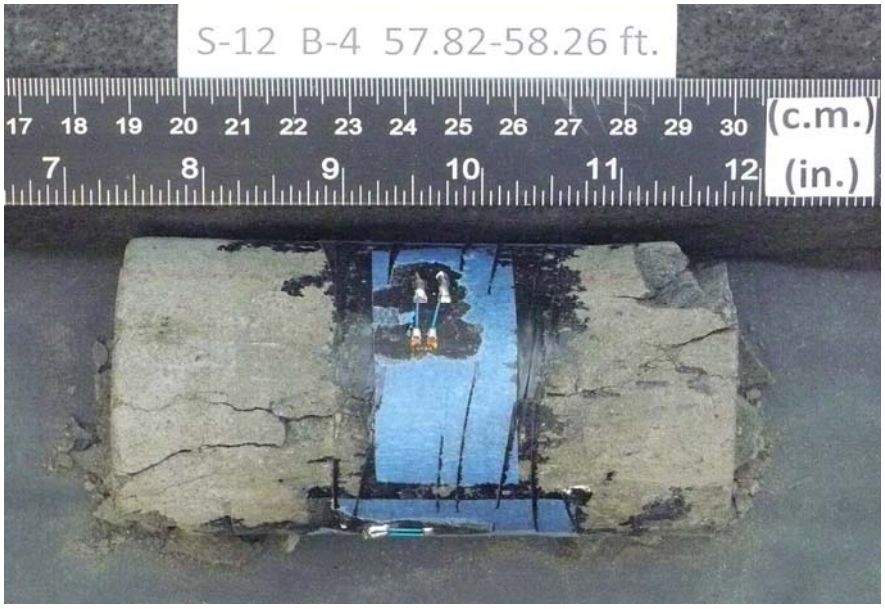
PERPENDICULARITY Spherically Seated

Maximum Gap Between Square and Top of Specimen, in = 0.11 Angle of Departure,°: 1.41 *Maximum angle of departure must be < 0.25°*

Perpendicularity Tolerance Met? NO

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-4
Sample ID:	S-12
Depth, ft:	57.82-58.26

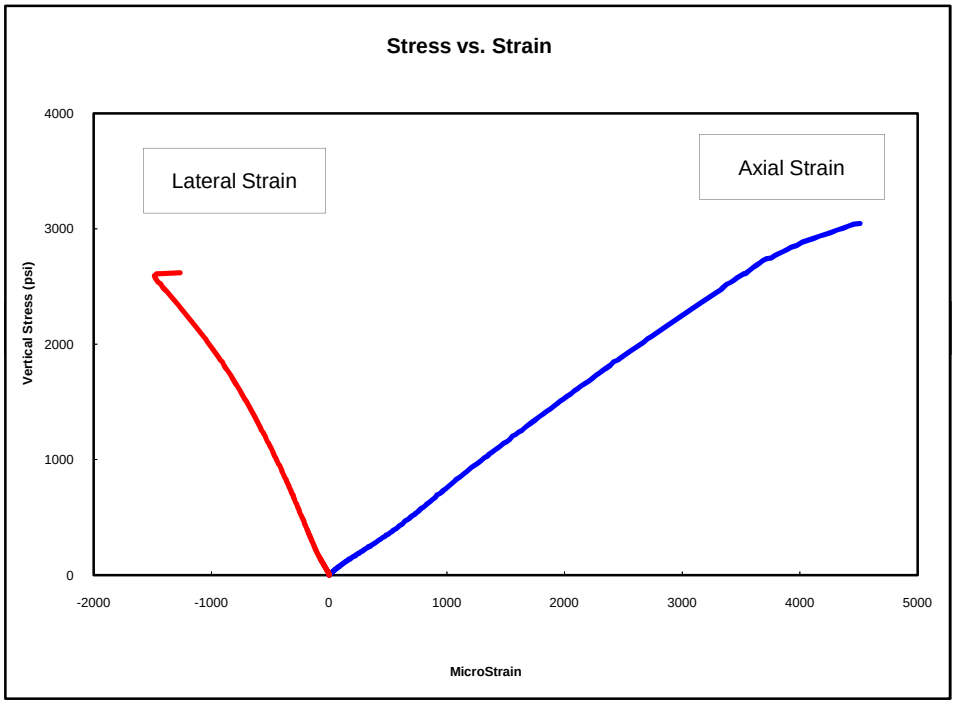
After cutting and grinding



After break

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-5
Sample ID:	S-4
Depth, ft:	14.58-14.92
Sample Type:	rock core
Sample Description:	See photographs Intact material failure L/D < 2

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 3,048 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-1000	760,000	0.32
1000-2000	746,000	0.42
2000-2500	693,000	---

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed. Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Ausenco Vector	Test Date:	08/04/11
Project Name:	Henderson-Genesee	Tested By:	daa
Project Location:	---	Checked By:	mpd
GTX #:	11031		
Boring ID:	B-5		
Sample ID:	S-4		
Depth:	14.58-14.92 ft		
Visual Description:	See Photographs	BEST EFFORT END PREPARATION	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	3.75	3.77	3.76
Specimen Diameter, in:	2.35	2.36	2.36
Specimen Mass, g:	545.28		
Bulk Density, lb/ft^3	127		
Length to Diameter Ratio:	1.60		

DEVIATION FROM STRAIGHTNESS

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Straightness Tolerance Met? YES

END FLATNESS

END 1

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

END 2

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

End Flatness Tolerance Met? YES

PARALLELISM Spherically Seated

END 1

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.01	Angle of Departure,°:	0.15	*Angle of departure < 0.25°?*	**YES**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.045	Angle of Departure,°:	0.69	*Angle of departure < 0.25°?*	**NO**

END 2

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.035	Angle of Departure,°:	0.53	*Angle of departure < 0.25°?*	**NO**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	<	0.001	Angle of Departure,°:	0.02	*Angle of departure < 0.25°?*	**YES**

Parallelism Tolerance Met? NO

PERPENDICULARITY Spherically Seated

Maximum Gap Between Square and Top of Specimen, in = 0.126 Angle of Departure,°: 1.92 *Maximum angle of departure must be < 0.25°*

Perpendicularity Tolerance Met? NO

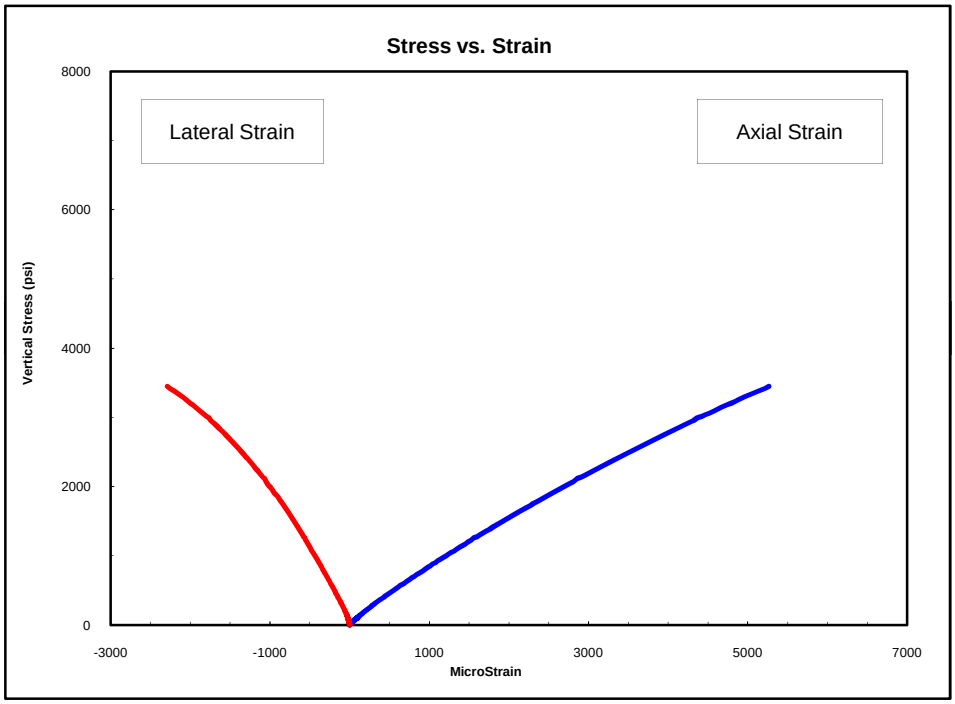
Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-5
Sample ID:	S-4
Depth, ft:	14.58-14.92

After cutting and grinding

After break

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-5
Sample ID:	S-6
Depth, ft:	37.33-37.68
Sample Type:	rock core
Sample Description:	See photographs Intact material failure L/D < 2

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 3,446 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-1000	811,000	0.37
1000-2000	674,000	0.38
2000-3000	588,000	0.46

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Ausenco Vector	Test Date:	08/04/11
Project Name:	Henderson-Genesee	Tested By:	daa
Project Location:	---	Checked By:	mpd
GTX #:	11031		
Boring ID:	B-5		
Sample ID:	S-6		
Depth:	37.33-37.68 ft		
Visual Description:	See Photographs	BEST EFFORT END PREPARATION	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	3.83	3.85	3.84
Specimen Diameter, in:	2.38	2.37	2.38
Specimen Mass, g:	596.22		
Bulk Density, lb/ft^3	133		
Length to Diameter Ratio:	1.62		

DEVIATION FROM STRAIGHTNESS

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Straightness Tolerance Met? YES

END FLATNESS

END 1

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

END 2

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

End Flatness Tolerance Met? YES

PARALLELISM Spherically Seated

END 1

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.018	Angle of Departure,°:	0.27	*Angle of departure < 0.25°?*	**NO**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.034	Angle of Departure,°:	0.51	*Angle of departure < 0.25°?*	**NO**

END 2

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.003	Angle of Departure,°:	0.04	*Angle of departure < 0.25°?*	**YES**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.042	Angle of Departure,°:	0.63	*Angle of departure < 0.25°?*	**NO**

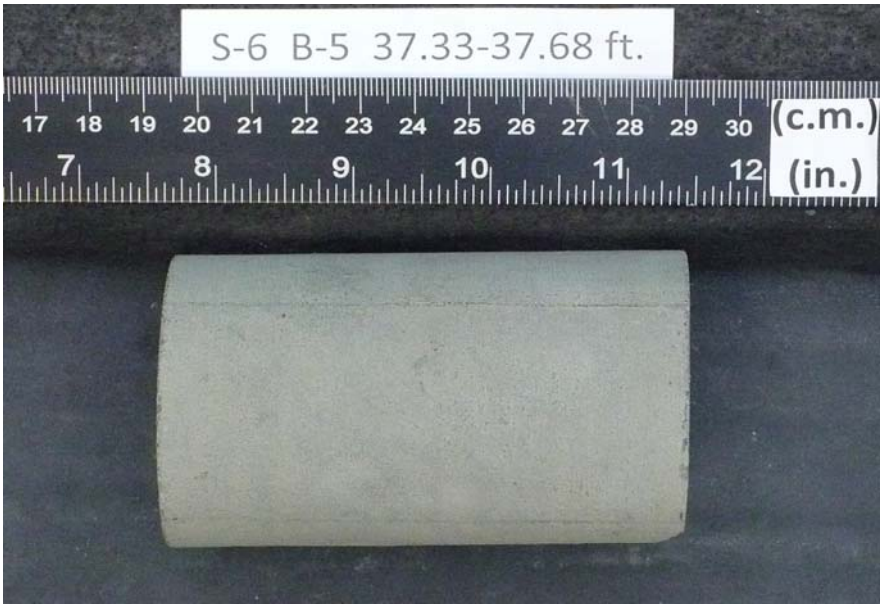
Parallelism Tolerance Met? NO

PERPENDICULARITY Spherically Seated

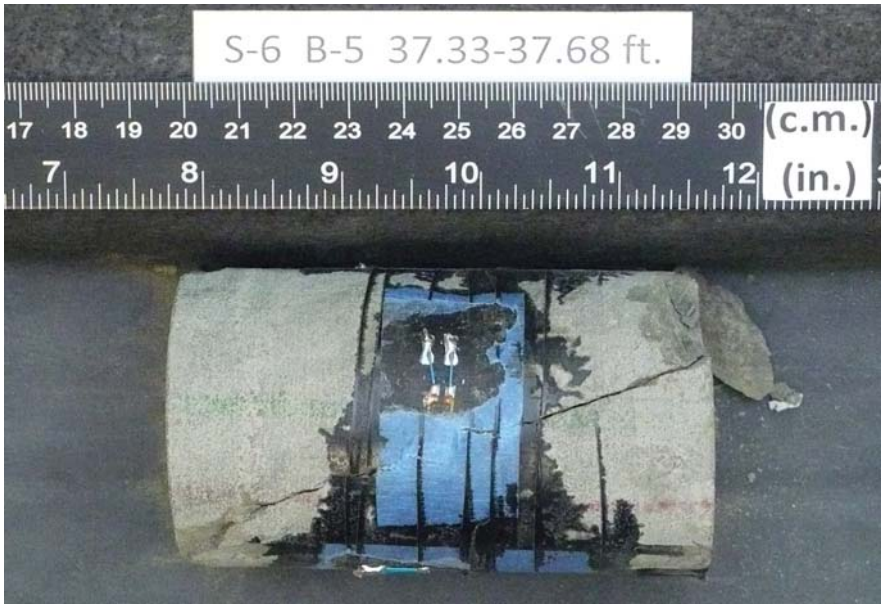
Maximum Gap Between Square and Top of Specimen, in = 0.131 Angle of Departure,°: 1.95 *Maximum angle of departure must be < 0.25°*

Perpendicularity Tolerance Met? NO

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-5
Sample ID:	S-6
Depth, ft:	37.33-37.68



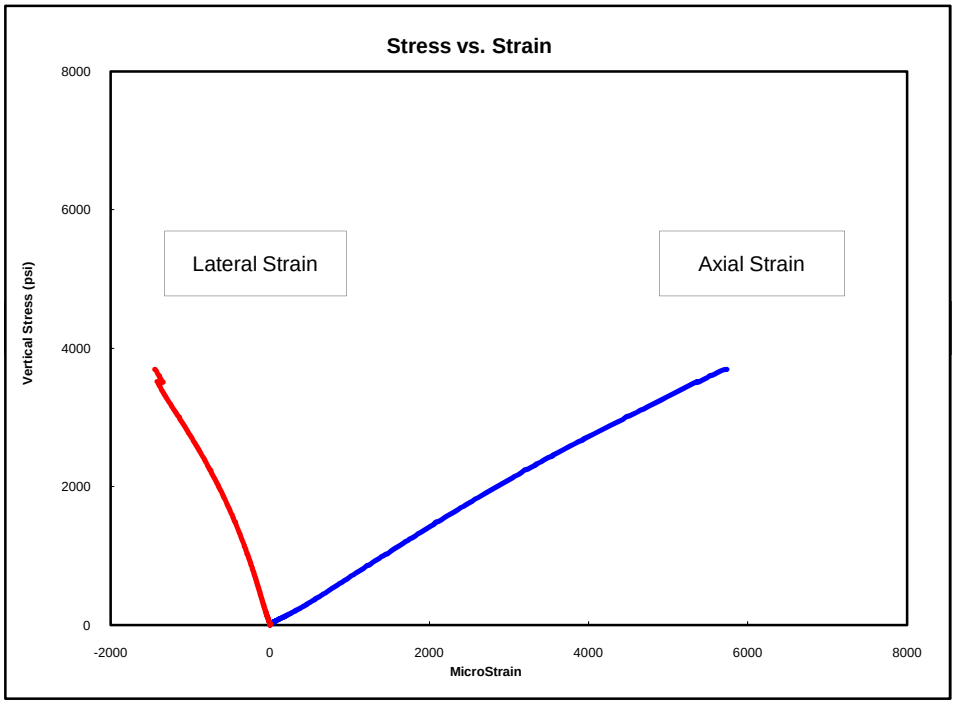
After cutting and grinding



After break

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-5
Sample ID:	S-10
Depth, ft:	53.47-53.90
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

Compressive Strength and Elastic Moduli of Rock by ASTM D 7012 - Method D



Peak Compressive Stress: 3,700 psi

The graph above may not include all data up to the peak shear stress value. Therefore, the highest value on the graph may not represent the peak shear stress value listed above.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
0-1000	707,000	0.19
1000-2000	704,000	0.26
2000-3000	613,000	0.31
3000-3700	567,000	0.25

Notes: Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

Client:	Ausenco Vector	Test Date:	08/04/11
Project Name:	Henderson-Genesee	Tested By:	daa
Project Location:	---	Checked By:	mpd
GTX #:	11031		
Boring ID:	B-5		
Sample ID:	S-10		
Depth:	53.47-53.90 ft		
Visual Description:	See Photographs	BEST EFFORT END PREPARATION	

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D 4543-04

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.66	4.67	4.67
Specimen Diameter, in:	2.36	2.37	2.37
Specimen Mass, g:	714.6		
Bulk Density, lb/ft^3	133		
Length to Diameter Ratio:	2.0		

DEVIATION FROM STRAIGHTNESS

Maximum gap between side of core and reference surface plate:

Is the maximum gap ≤ 0.02 in.? YES

Straightness Tolerance Met? YES

END FLATNESS

END 1

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

END 2

Diameter 1	Is the maximum gap < 0.001 in.?	YES
Diameter 2 (rotated 90°)	Is the maximum gap < 0.001 in.?	YES

End Flatness Tolerance Met? YES

PARALLELISM Spherically Seated

END 1

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.016	Angle of Departure,°:	0.20	*Angle of departure < 0.25°?*	**YES**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.012	Angle of Departure,°:	0.15	*Angle of departure < 0.25°?*	**YES**

END 2

Diameter 1	Gap Between Square and Top of Specimen, in	=	0.02	Angle of Departure,°:	0.25	*Angle of departure < 0.25°?*	**YES**
Diameter 2 (rotated 90°)	Gap Between Square and Top of Specimen, in	=	0.006	Angle of Departure,°:	0.07	*Angle of departure < 0.25°?*	**YES**

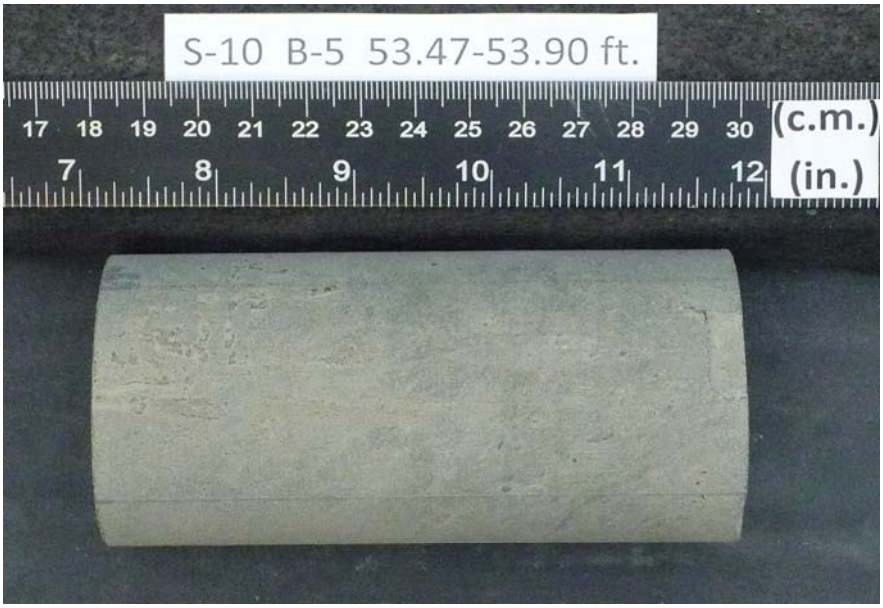
Parallelism Tolerance Met? YES

PERPENDICULARITY Spherically Seated

Maximum Gap Between Square and Top of Specimen, in = 0.065 Angle of Departure,°: 0.80 *Maximum angle of departure must be < 0.25°*

Perpendicularity Tolerance Met? NO

Client:	Ausenco Vector
Project Name:	Henderson-Genesee
Project Location:	---
GTX #:	11031
Test Date:	08/05/11
Tested By:	daa
Checked By:	mpd
Boring ID:	B-5
Sample ID:	S-10
Depth, ft:	53.47-53.90



After cutting and grinding

After break

APPENDIX C

ROCK OUTCROP MAPPING

APPENDIX C

ROCK OUTCROP MAPPING

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APPENDIX C

ROCK OUTCROP MAPPING

C.1 OUTCROP MAPPING

A geologist from Shannon &Wilson completed a reconnaissance on July 22, 2011, to study any rock outcrops exposed within Seward Park. The reconnaissance was completed by traveling the pedestrian paths and roadways throughout the park. A total of four outcrops (Figures C-1 through C-4) were observed and rock mass properties were noted at each outcrop.

C.2 ROCK MASS PROPERTIES

The rock mass properties noted during Shannon & Wilson's outcrop mapping include orientation, persistence (length), terminations, spacing (distance between joints), aperture (width), joint filling, surface profile, and roughness.

- **Orientation:** The orientation of the discontinuity (joint, fault, parting) expressed in dip and dip direction. The dip is the angle that the discontinuity is inclined from horizontal, and the dip direction is the angle in degrees from true north measured in the downdip direction.
- **Persistence:** The distance a particular discontinuity extends, measured in surface discontinuity surveys.
- **Terminations:** The number of ends or terminations of a discontinuity visible in an outcrop.
- **Spacing:** The distance between individual discontinuities, measured normal to the discontinuities.
- **Aperture:** The width of openings of the discontinuities.
- **Joint Filling:** The type of material occurring within joints.
- **Surface Profile and Roughness:** The profile of joints observed at the surface.

C.3 ROCK MASS PROPERTY RESULTS

The rock mass property results are provided in Tables C-1 through C-4. Dips, by Rocscience, provides graphical and statistical analysis of the orientation of rock structure such as joints, faults and bedding planes. Shannon & Wilson used Dips to plot rock structure data from Tables C-1 through C-4 onto a stereonet to show the relationship of the rock structures at the four outcrops. The stereonet of the data is provided as Figure C-5. The stereonet plot of the structural data indicates three joint sets are present.

Table C-1. Surface Geological Survey

Waypoint: OC-1 | Date: 7/22/2011 | Weather: Sunny | Temp.: 65 | Logged by: CWA | Page 1 of 1

Strike: 337 | Plunge: 82 | GPS ID: 47.55216N, 122.25676W | Location: Seward Park Pedestrian Path, west side of park, north of parking lots

Joint No.	Type	Dip	Dip Direction	Persistence	Termination	Aperture	Nature of Filling	Surface Shape	Surface Roughness	Spacing (Ft)	Water Flow	Waviness Wavelength	Waviness Amplitude	Comments/No. of Similar
1	2	60	81	15	1	3	2 , 8	2	1	0.3 to 1.0	O1			4
2	2	67	75	15	1	3	2 , 8	2	1	0.3 to 1.0	O1			
3	2	69	85	15	1	3	2 , 8	2	1	0.3 to 1.0	O1			
4	2	65	92	15	1	3	2 , 8	2	1	0.3 to 1.0	O1			
5	2, 5	63	348	3	2	5	2	3	1	0.04	O1			14
6	2, 5	60	351	3	2	5	2	3	1	0.04	O1			
7	2, 5	68	359	3	2	5	2	3	1	0.04	O1			
8	2	44	150	1	2	4	3 , 8	2	1	0.3	O1			8
9	2	39	174	1	2	4	3 , 8	2	1	0.3	O1			
10	2	49	132	1	2	4	3 , 8	2	1	0.3	O1			

Type:

1. Fracture
2. Joint
3. Foliation
4. Random Fracture
5. Bedding/Parting
6. Fracture Zone
7. Mechanical Break
8. Shear
9. Shear Zone
10. Fault
11. Other _____

Aperture (width):

	inch	mm
1. Very Tight	< .004	< .1
2. Tight	.004 - .01	.1 - .25
3. Partly Open	.01 - .02	.25 - .5
4. Open	.02 - .1	.5 - 2.5
5. Moderately Wide	.1 - .4	2.5 - 10
6. Wide	.4 - 4	10 - 100
7. Very Wide	4 - 40	100 -1000
8. Extremely Wide	> 40	> 1000

Persistence:

	English	Metric
1. Very Low	< 3 ft	< 1 m
2. Low	3 - 10 ft	1 - 3 m
3. Medium	10 - 30 ft	3 - 10 m
4. High	30 - 60 ft	10 - 20 m
5. Very High	> 60 ft	> 20 m

Surface Shape:

1. Stepped
2. Undulating
3. Planar

Surface Roughness:

1. Rough
2. Smooth
3. Polished
4. Slickensided

Termination:

0. Neither end visible
1. One end visible
2. Both end visible

Nature of Filling:

1. Clean
2. Surface Staining
3. Non-cohesive
4. Inactive Clay or Clay Mix
5. Swelling Clay or Clay Mix
6. Cemented
7. Chlorite, talc, or gypsum
8. Other _____

Compressive Strength of Infilling:

	MPa	10^{-3} psi
S1	< .025	< .004
S2	.025 - .05	.004 - .007
S3	.05 - .1	.007 - .01
S4	.1 - .25	.01 - .04
S5	.25 - .5	.04 - .07
S6	> .5	> .07
R1	< 1	< .1
R2	1 - 5	.1 - .7
R3	5 - 25	.7 - 4
R4	25 - 50	4 - 7
R5	50 - 100	7 - 15
R6	100 - 250	15 - 36
R7	> 250	> 36

Water Flow (Open):

O1. Dry
O2. Water Staining
O3. Damp
O4. Dripping
O5. Continuous Flow

Water Flow (Filled):

F1. Dry
F2. Damp
F3. Free Water
F4. Occasional Drops
F5. Outwash Channels

Comments & special features:

SANDSTONE: Low strength, light brown, fine grained; rough, very close to medium spaced , high angle joints; very thin to thinly bedded, highly weathered.

Schmidt Hammer: 31, 32, 32, 36, 34, 36, 36, 36, 42, 37

Right hand rule used.

Note: Corrected for 17deg East declination

Table C-2. Surface Geological Survey

Waypoint: OC-2 | Date: 7/22/2011 | Weather: Sunny | Temp.: 65 | Logged by: CWA | Page 1 of 1

Strike: 166 | Plunge: 79 | GPS ID: 47.55158N, 122.25708W | Location: Seward Park, west side of park, near north end of NW parking lot

Joint No.	Type	Dip	Dip Direction	Persistence	Termination	Aperture	Nature of Filling	Surface Shape	Surface Roughness	Spacing (Ft)	Water Flow	Waviness Wavelength	Waviness Amplitude	Comments/No. of Similar
1	2	38	162	3	0	5	3	3	1		O1			0
2	2	53	122	5	0	6	3	3	1		O1			0
3	5	70	335											0

Type:

1. Fracture
2. Joint
3. Foliation
4. Random Fracture
5. Bedding/Parting
6. Fracture Zone
7. Mechanical Break
8. Shear
9. Shear Zone
10. Fault
11. Other _____

Aperture (width):

	inch	mm
1. Very Tight	< .004	< .1
2. Tight	.004 - .01	.1 - .25
3. Partly Open	.01 - .02	.25 - .5
4. Open	.02 - .1	.5 - 2.5
5. Moderately Wide	.1 - .4	2.5 - 10
6. Wide	.4 - 4	10 - 100
7. Very Wide	4 - 40	100 -1000
8. Extremely Wide	> 40	> 1000

Persistence:

	English	Metric
1. Very Low	< 3 ft	< 1 m
2. Low	3 - 10 ft	1 - 3 m
3. Medium	10 - 30 ft	3 - 10 m
4. High	30 - 60 ft	10 - 20 m
5. Very High	> 60 ft	> 20 m

Surface Shape:

1. Stepped
2. Undulating
3. Planar

Surface Roughness:

1. Rough
2. Smooth
3. Polished
4. Slickensided

Termination:

0. Neither end visible
1. One end visible
2. Both end visible

Nature of Filling:

1. Clean
2. Surface Staining
3. Non-cohesive
4. Inactive Clay or Clay Mix
5. Swelling Clay or Clay Mix
6. Cemented
7. Chlorite, talc, or gypsum
8. Other _____

Compressive Strength of Infilling:

	MPa	10^3 psi
S1	< .025	< .004
S2	.025 - .05	.004 - .007
S3	.05 - .1	.007 - .01
S4	.1 - .25	.01 - .04
S5	.25 - .5	.04 - .07
S6	> .5	> .07
R1	< 1	< .1
R2	1 - 5	.1 - .7
R3	5 - 25	.7 - 4
R4	25 - 50	4 - 7
R5	50 - 100	7 - 15
R6	100 - 250	15 - 36
R7	> 250	> 36

Water Flow (Open):

O1. Dry
O2. Water Staining
O3. Damp
O4. Dripping
O5. Continuous Flow

Water Flow (Filled):

F1. Dry
F2. Damp
F3. Free Water
F4. Occasional Drops
F5. Outwash Channels

Comments & special features:

SANDSTONE: Very low strength, gray to light brown, fine grained; rough, closely spaced joints; very thinly bedded.

Schmidt- Hammer: 11,12,14,15,14,14,14,16,16,14

Right hand rule used for orientations.

Note: Corrected for 17deg East declination

Table C-3. Surface Geological Survey

Waypoint: OC-3 | Date: 7/22/2011 | Weather: Sunny | Temp.: 68 | Logged by: CWA | Page 1 of 1

Strike: 223 | Plunge: 78 | GPS ID: 47.55266N, 122.25552W | Location: Seward Park Pedestrian Path, west side of park, north of OC-2

Joint No.	Type	Dip	Dip Direction	Persistence	Termination	Aperture	Nature of Filling	Surface Shape	Surface Roughness	Spacing (Ft)	Water Flow	Waviness Wavelength	Waviness Amplitude	Comments/No. of Similar
1	2	37	368	6'	1	4 to 6	3	2	1		O2			0
2	5	82	326	Face				2	1	0.01	O1/O2	3'	0.2'	Dip Slope
3	5	88	337	Face				2	1	0.01	O1/O2	3'	0.2'	
4	5	63	332	Face				2	1	0.01	O1/O2	3'	0.2'	

Type:
1. Fracture
2. Joint
3. Foliation
4. Random Fracture
5. Bedding/Parting
6. Fracture Zone
7. Mechanical Break
8. Shear
9. Shear Zone
10. Fault
11. Other _____

Aperture (width):

	inch	mm
1. Very Tight	< .004	< .1
2. Tight	.004 - .01	.1 - .25
3. Partly Open	.01 - .02	.25 - .5
4. Open	.02 - .1	.5 - 2.5
5. Moderately Wide	.1 - .4	2.5 - 10
6. Wide	.4 - 4	10 - 100
7. Very Wide	4 - 40	100 -1000
8. Extremely Wide	> 40	> 1000

Persistence:

	English	Metric
1. Very Low	< 3 ft	< 1 m
2. Low	3 - 10 ft	1 - 3 m
3. Medium	10 - 30 ft	3 - 10 m
4. High	30 - 60 ft	10 - 20 m
5. Very High	> 60 ft	> 20 m

Surface Shape:
1. Stepped
2. Undulating
3. Planar

Surface Roughness:
1. Rough
2. Smooth
3. Polished
4. Slickensided

Termination:
0. Neither end visible
1. One end visible
2. Both end visible

Nature of Filling:
1. Clean
2. Surface Staining
3. Non-cohesive
4. Inactive Clay or Clay Mix
5. Swelling Clay or Clay Mix
6. Cemented
7. Chlorite, talc, or gypsum
8. Other _____

Compressive Strength of Infilling:

	MPa	10^{-3} psi
S1	< .025	< .004
S2	.025 - .05	.004 - .007
S3	.05 - .1	.007 - .01
S4	.1 - .25	.01 - .04
S5	.25 - .5	.04 - .07
S6	> .5	> .07
R1	< 1	< .1
R2	1 - 5	.1 - .7
R3	5 - 25	.7 - 4
R4	25 - 50	4 - 7
R5	50 - 100	7 - 15
R6	100 - 250	15 - 36
R7	> 250	> 36

Water Flow (Open):
O1. Dry
O2. Water Staining
O3. Damp
O4. Dripping
O5. Continuous Flow

Water Flow (Filled):
F1. Dry
F2. Damp
F3. Free Water
F4. Occasional Drops
F5. Outwash Channels

Comments & special features:

SANDSTONE: Very low strength, light brown, fine grained with areas of fine to coarse grained; one joint observed; face of outcrop is dip slope with very thin bedding with partings; highly weathered.

Schmidt Hammer: 12, 14, 12, 12, 12, 12, 12, 12, 11, 12

Right hand rule used.

Note: Corrected for 17deg East declination

Table C-4. Surface Geological Survey

Waypoint: OC-4 | Date: 7/22/2011 | Weather: Sunny | Temp.: 70 | Logged by: CWA | Page 1 of 1

Strike: 34 | Plunge: 84 | GPS ID: 47.55185N, 122.24746W | Location: Seward Park Pedestrian Path, east side of park, west of pedestrian path

Joint No.	Type	Dip	Dip Direction	Persistence	Termination	Aperture	Nature of Filling	Surface Shape	Surface Roughness	Spacing (Ft)	Water Flow	Waviness Wavelength	Waviness Amplitude	Comments/No. of Similar
1	2	50	342	20	0	2, 3	2, 3	2	1	0.3 to 0.9	O1, O2			3
2	2	51	328					2	1	0.3 to 0.9	O1, O2			
3	2	38	341					2	1	0.3 to 0.9	O1, O2			
4	2	35	134	2	2	1, 4	3	2	1	0.3 to 0.7	O1, O2			5
5	2	38	151					2	1	0.3 to 0.7	O1, O2			
6	2	63	167	2	2	1,4	3	2	1	0.3 to 0.7	O1, O2			
7	2	65	161					2	1	0.3 to 0.7	O1, O2			
8	2	75	357	2.5	2	1, 4	3	2	1	0.4	O1, O2			1

Type:

1. Fracture
2. Joint
3. Foliation
4. Random Fracture
5. Bedding/Parting
6. Fracture Zone
7. Mechanical Break
8. Shear
9. Shear Zone
10. Fault
11. Other _____

Aperture (width):

	inch	mm
1. Very Tight	< .004	< .1
2. Tight	.004 - .01	.1 - .25
3. Partly Open	.01 - .02	.25 - .5
4. Open	.02 - .1	.5 - 2.5
5. Moderately Wide	.1 - .4	2.5 - 10
6. Wide	.4 - 4	10 - 100
7. Very Wide	4 - 40	100 -1000
8. Extremely Wide	> 40	> 1000

Persistence:

	English	Metric
1. Very Low	< 3 ft	< 1 m
2. Low	3 - 10 ft	1 - 3 m
3. Medium	10 - 30 ft	3 - 10 m
4. High	30 - 60 ft	10 - 20 m
5. Very High	> 60 ft	> 20 m

Surface Shape:

1. Stepped
2. Undulating
3. Planar

Surface Roughness:

1. Rough
2. Smooth
3. Polished
4. Slickensided

Termination:

0. Neither end visible
1. One end visible
2. Both end visible

Nature of Filling:

1. Clean
2. Surface Staining
3. Non-cohesive
4. Inactive Clay or Clay Mix
5. Swelling Clay or Clay Mix
6. Cemented
7. Chlorite, talc, or gypsum
8. Other _____

Compressive Strength of Infilling:

	MPa	10^{-3} psi
S1	< .025	< .004
S2	.025 - .05	.004 - .007
S3	.05 - .1	.007 - .01
S4	.1 - .25	.01 - .04
S5	.25 - .5	.04 - .07
S6	> .5	> .07
R1	< 1	< .1
R2	1 - 5	.1 - .7
R3	5 - 25	.7 - 4
R4	25 - 50	4 - 7
R5	50 - 100	7 - 15
R6	100 - 250	15 - 36
R7	> 250	> 36

Water Flow (Open):

O1. Dry
O2. Water Staining
O3. Damp
O4. Dripping
O5. Continuous Flow

Water Flow (Filled):

F1. Dry
F2. Damp
F3. Free Water
F4. Occasional Drops
F5. Outwash Channels

Comments & special features:

SANDSTONE: Low to moderate strength, light brown to light gray, fine grained rough, closely spaced, high angle joints; iron oxide stained, moderately to highly weathered.

Schmidt Hammer: 30, 32, 31, 32, 36, 32, 30, 30, 32, 36

Right hand rule used.

Note: Corrected for 17deg East declination

Photo 1. Outcrop #1.

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21-1-21144-007

SITE PHOTOGRAPHS

FIG. C-1

FIG. C-1

Photo 2. Outcrop #2.

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21-1-21144-007

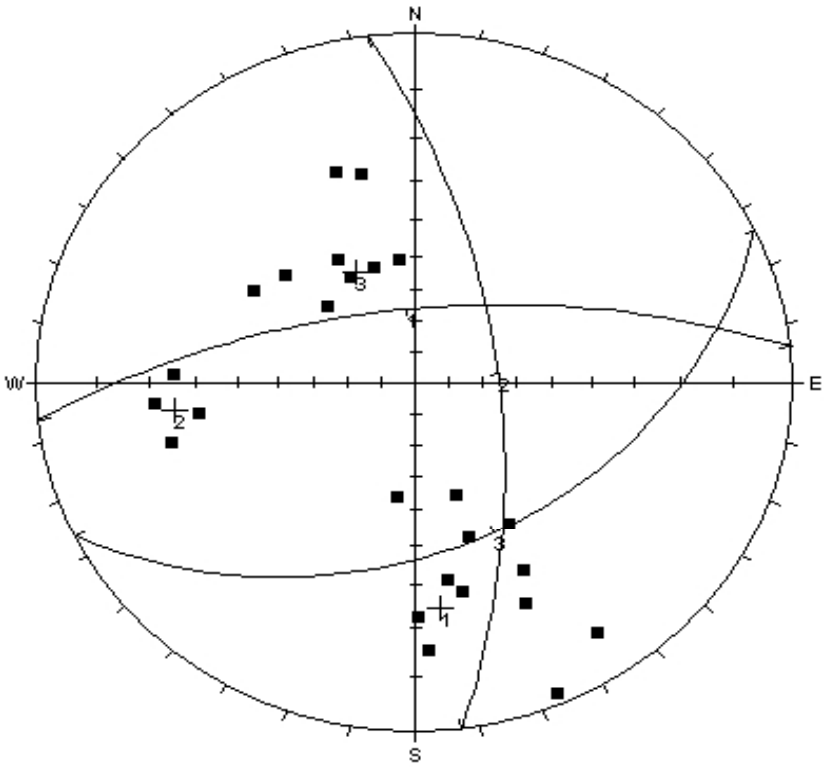
SITE PHOTOGRAPHS

FIG. C-2

Photo 3. Outcrop #3.

FIG. C-3

Photo 4. Outcrop #4.



Poles

Equal Angle
Lower Hemisphere
25 Poles
25 Entries

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Stereonet Rock Outcrop Data

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FIG. C-5

APPENDIX D

FIELD SLUG TESTS

APPENDIX D

FIELD SLUG TESTS

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APPENDIX D

FIELD SLUG TESTS

D.1 SLUG TEST

Slug tests were performed to evaluate the hydrogeologic conditions for the two proposed storage tank sites. Six slug tests per well (three falling head and three rising head) were performed using observation wells B-4, B-5, and B-6. Pressure transducers ("Levelogger" by Solinst Company) were installed in the observation wells to take continued measurement of the water level every second during the slug tests and a "Barologger" collected measurements of barometric pressure during the tests.

D.2 SLUG TEST ANALYSIS METHODS

The Bouwer and Rice equation is used to calculate the hydraulic conductivity of an aquifer based on drawdown versus time data from an observation well during a pumping test or a slug test. We used software package AQTESOLV (HydroSOLVE, Inc., 2010)[1] to analyze the slug test data.

D.3 SLUG TEST RESULTS

The slug test results are listed in Table D-1. AQTESOLV analysis for rising head and falling tests were presented in Figures D-1 to D-18. Averaged hydraulic conductivities obtained from the B-4 slug tests were about 1.6 feet per day (ft/day). Averaged hydraulic conductivity for B-5 was about 68 ft/day; the relatively high value likely represents a more highly fractured zone of the bedrock. Averaged hydraulic conductivity for B-6 was about 0.8 ft/day.

[1] HydroSOLVE, Inc., 2010, copyright© 1996-2007, AQTESOLV version 4.50 – Professional.

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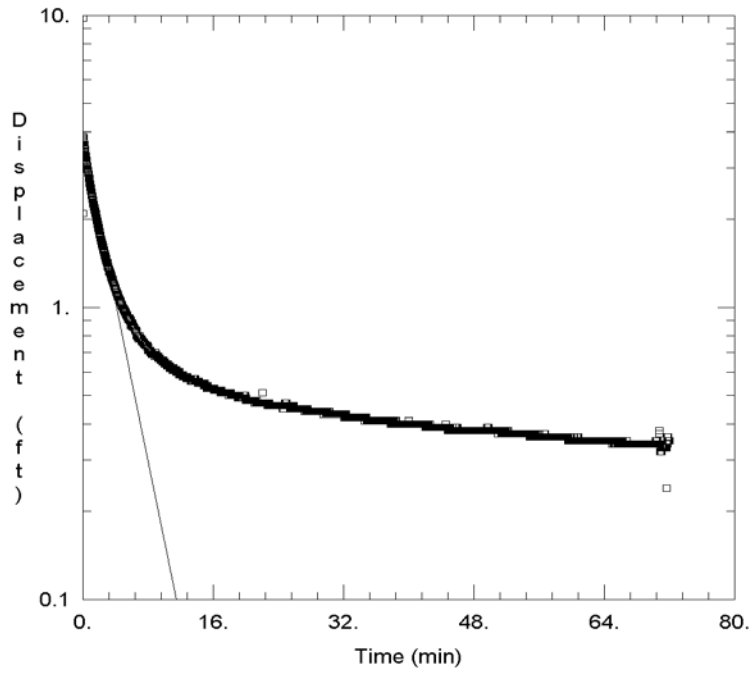
Table D-1. Slug Test Results Summary

Observation Well	Date Tested	Test No.	Estimated Hydraulic Conductivity (ft/day)	Estimated Hydraulic Conductivity (cm/sec)
B-4	7/15/11	Falling Head Test 1	1.37	4.86E-04
B-4	7/15/11	Rising Head Test 1	1.59	5.60E-04
B-4	7/15/11	Falling Head Test 2	2.24	7.91E-04
B-4	7/15/11	Rising Head Test 2	1.55	5.48E-04
B-4	7/18/11	Falling Head Test 3	1.86	6.56E-04
B-4	7/18/11	Rising Head Test 3	1.21	4.29E-04
		Geomean	**1.6**	**5.8E-04**
B-5	7/14/11	Falling Head Test 1	52.13	1.84E-02
B-5	7/14/11	Rising Head Test 1	75.57	2.67E-02
B-5	7/14/11	Falling Head Test 2	79.29	2.80E-02
B-5	7/14/11	Rising Head Test 2	60.43	2.14E-02
B-5	7/14/11	Falling Head Test 3	63.74	2.25E-02
B-5	7/14/11	Rising Head Test 3	77.06	2.72E-02
		Geomean	**68.0**	**2.4E-02**
B-6	7/14/11	Falling Head Test 1	0.66	2.33E-04
B-6	7/14/11	Rising Head Test 1	0.56	1.99E-04
B-6	7/15/11	Falling Head Test 2	0.96	3.38E-04
B-6	7/15/11	Rising Head Test 2	0.94	3.34E-04
B-6	7/14/11	Falling Head Test 3	0.84	2.96E-04
B-6	7/14/11	Rising Head Test 3	0.66	2.32E-04
		Geomean	**0.8**	**2.7E-04**

Notes:
1. Slug tests were performed by Shannon & Wilson personnel.
cm/sec = centimeters per second
ft/day = feet per day



WELL TEST ANALYSIS

Data Set: I:\...\B-4 SlugTest-F1.aqt
Date: 08/12/11 Time: 07:57:17

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-4
Test Date: 7/15/11

AQUIFER DATA

Saturated Thickness: 18.7 ft
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-4-Falling Head Test #1)

Initial Displacement: 3.852 ft
Total Well Penetration Depth: 16.9 ft
Casing Radius: 0.083 ft
Static Water Column Height: 16.9 ft
Screen Length: 5. ft
Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 1.374 ft/day
y0 = 3.647 ft

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Seattle, Washington

B-4 Falling Head Test #1

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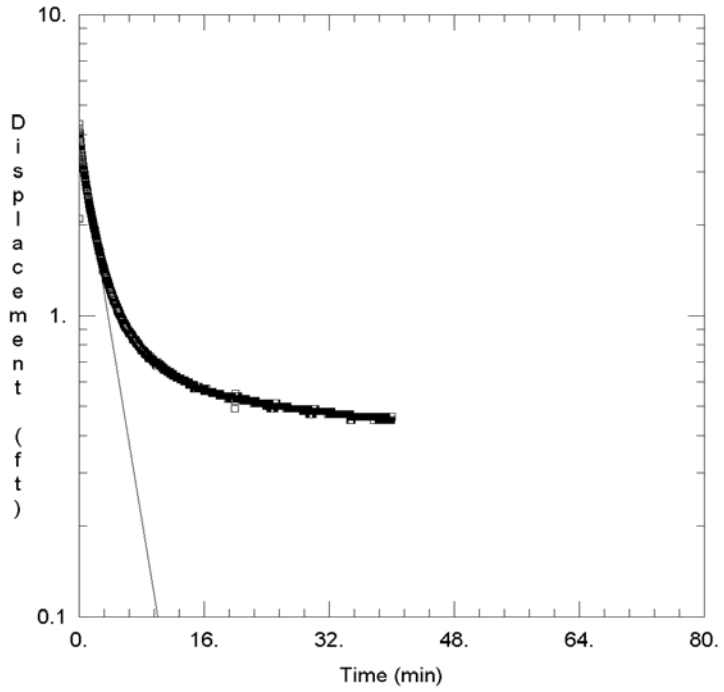
FIG. D-1



WELL TEST ANALYSIS

Data Set: I:\...\B-4 SlugTest-R1.aqt
Date: 08/12/11 Time: 08:28:54

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-4
Test Date: 7/15/11

AQUIFER DATA

Saturated Thickness: 18.7 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-4-Rising Head Test #1)

Initial Displacement: 4.33 ft Static Water Column Height: 18.82 ft
Total Well Penetration Depth: 16.9 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 1.585 ft/day y0 = 3.795 ft

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B-4 Rising Head Test #1

August 2011 21-1-21144-007

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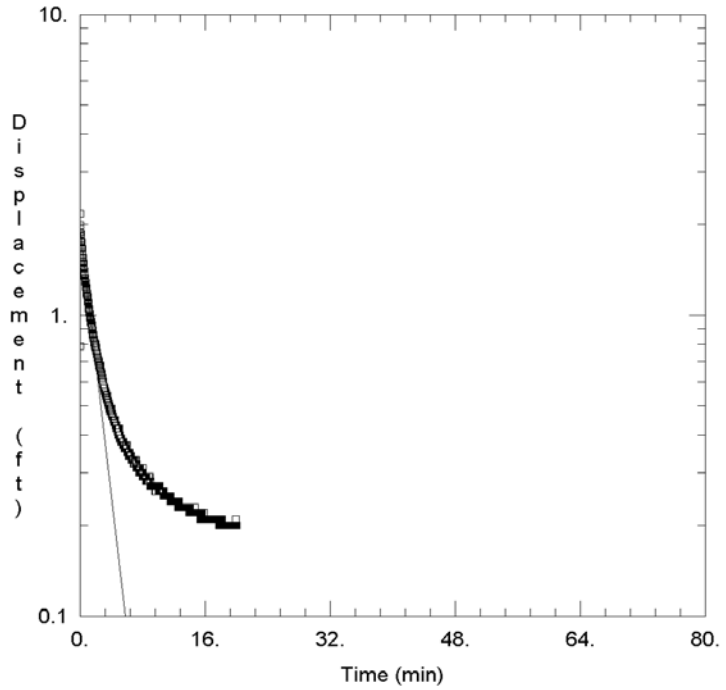
FIG. D-2



WELL TEST ANALYSIS

Data Set: I:\...\B-4 SlugTest-F2.aqt
Date: 08/12/11 Time: 08:30:28

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-4
Test Date: 7/15/11

AQUIFER DATA

Saturated Thickness: 18.82 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-4-Falling Head Test #2)

Initial Displacement: 2.17 ft Static Water Column Height: 18.82 ft
Total Well Penetration Depth: 16.9 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 2.238 ft/day y0 = 1.928 ft

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B-4 Falling Head Test #2

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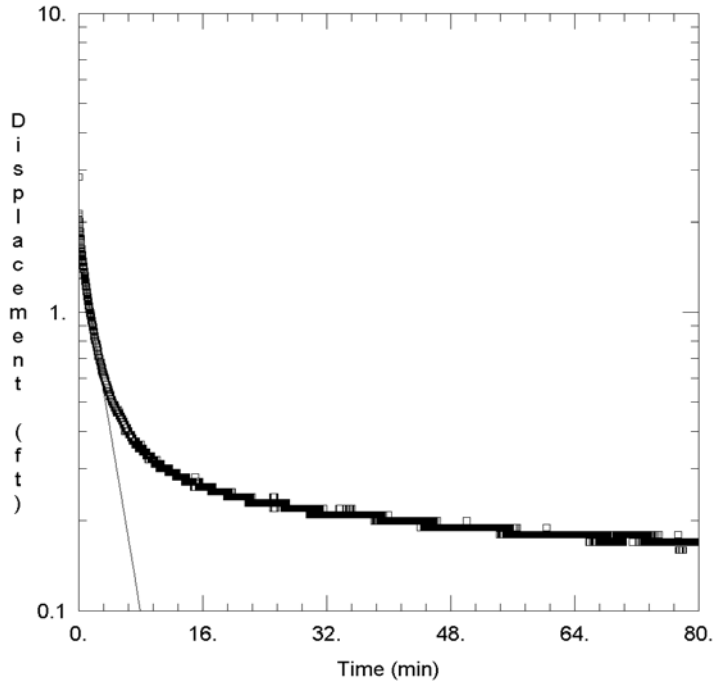
FIG. D-3



WELL TEST ANALYSIS

Data Set: I:\...\B-4 SlugTest-R2.aqt
Date: 08/12/11 Time: 08:31:41

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-4
Test Date: 7/15/11

AQUIFER DATA

Saturated Thickness: 18.7 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-4-Rising Head Test #2)

Initial Displacement: 2.83 ft Static Water Column Height: 18.82 ft
Total Well Penetration Depth: 16.9 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 1.552 ft/day y0 = 1.684 ft

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B-4 Rising Head Test #2

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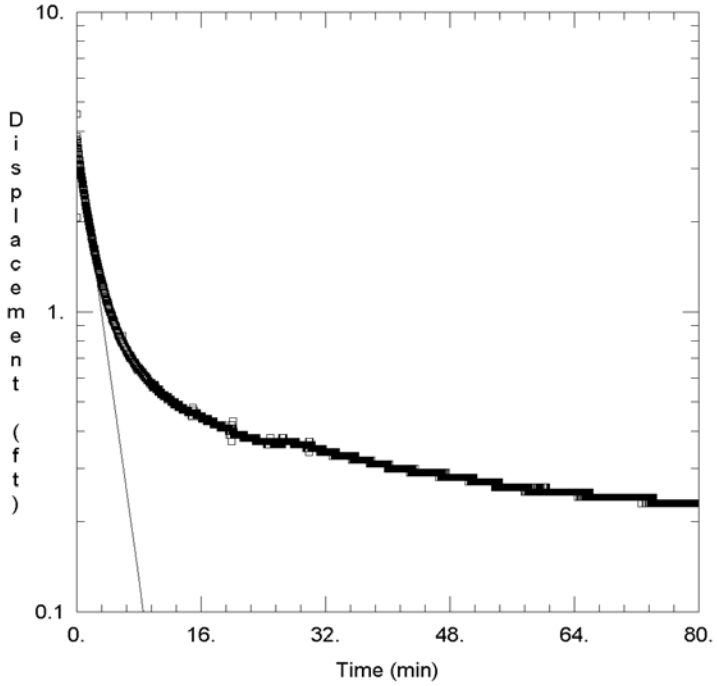
FIG. D-4



WELL TEST ANALYSIS

Data Set: I:\...\B-4 SlugTest-F3.aqt
Date: 08/12/11 Time: 08:31:03

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-4
Test Date: 7/15/11

AQUIFER DATA

Saturated Thickness: 18.7 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-4-Falling Head Test #3)

Initial Displacement: 3.83 ft Static Water Column Height: 18.82 ft
Total Well Penetration Depth: 16.9 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 1.857 ft/day y0 = 3.819 ft

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B-4 Falling Head Test #3

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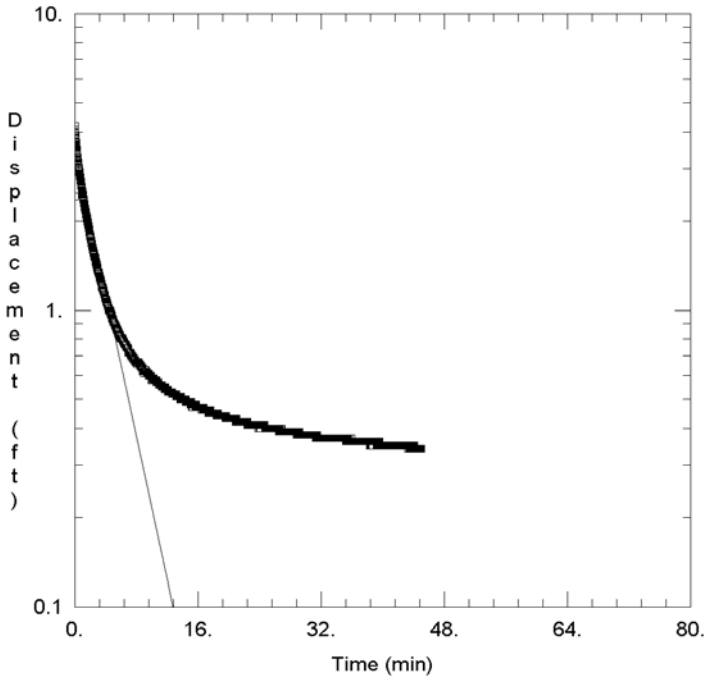
FIG. D-5



WELL TEST ANALYSIS

Data Set: I:\...\B-4 SlugTest-R3.aqt
Date: 08/12/11 Time: 08:32:16

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-4
Test Date: 7/15/11

AQUIFER DATA

Saturated Thickness: 18.7 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-4-Rising Head Test #3)

Initial Displacement: 4.17 ft Static Water Column Height: 18.82 ft
Total Well Penetration Depth: 16.9 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 1.214 ft/day y0 = 3.429 ft

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B-4 Rising Head Test #3

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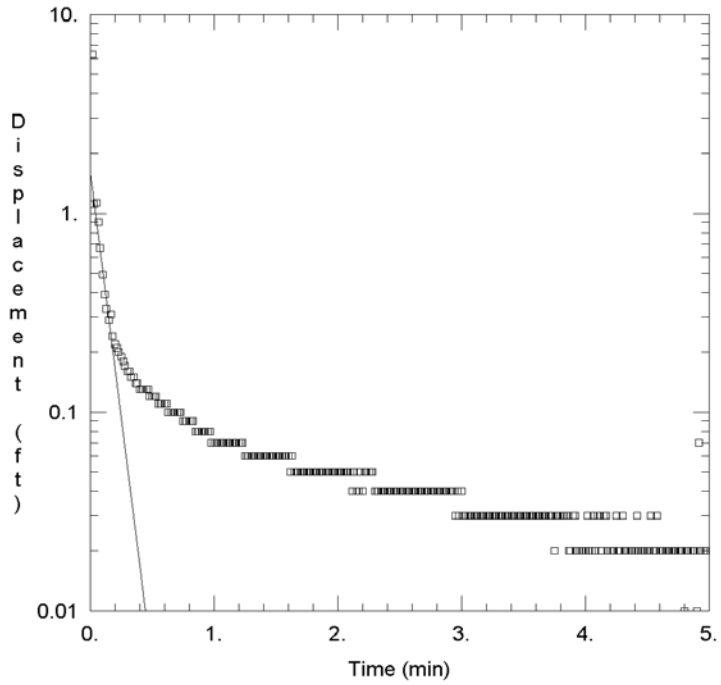
FIG. D-6



WELL TEST ANALYSIS

Data Set: I:\...\B-5 SlugTest-F1.aqt
Date: 08/12/11 Time: 08:33:41

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-5
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 27.47 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-5-Falling Head Test #1)

Initial Displacement: 1.11 ft Static Water Column Height: 25.52 ft
Total Well Penetration Depth: 25.52 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 52.13 ft/day y0 = 1.605 ft

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B-5 Falling Head Test #1

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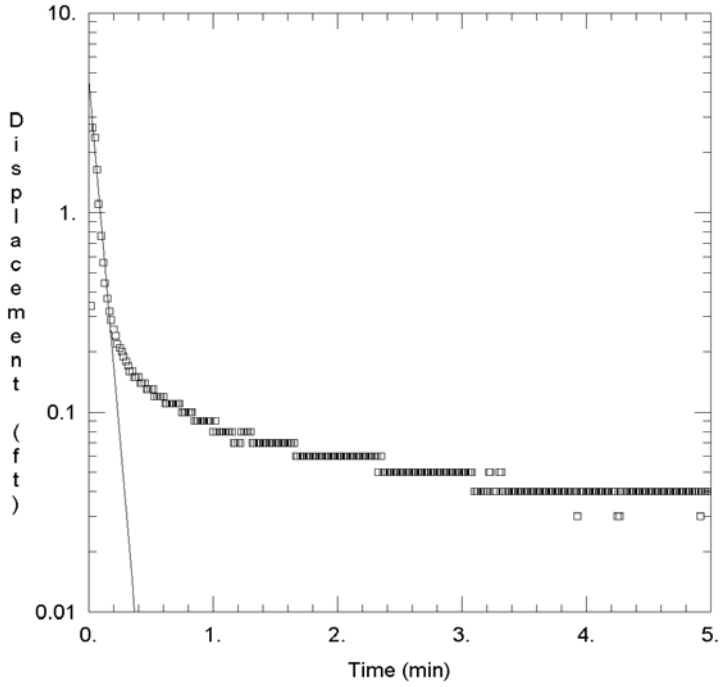
FIG. D-7



WELL TEST ANALYSIS

Data Set: I:\...\B-5 SlugTest-R1.aqt
Date: 08/12/11 Time: 08:35:12

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-5
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 27.47 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-5-Rising Head Test #1)

Initial Displacement: 2.66 ft Static Water Column Height: 25.52 ft
Total Well Penetration Depth: 25.52 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 75.57 ft/day y0 = 4.63 ft

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B-5 Rising Head Test #1

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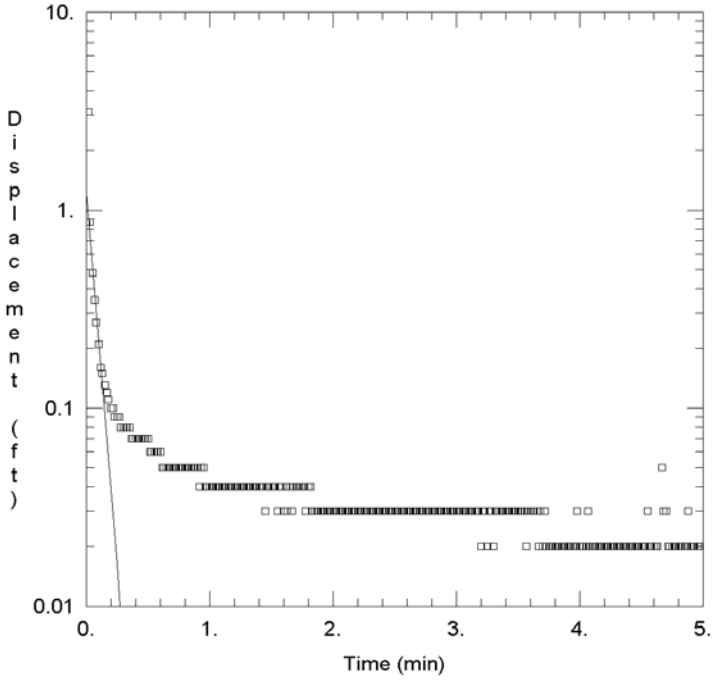
FIG. D-8



WELL TEST ANALYSIS

Data Set: I:\...\B-5 SlugTest-F2.aqt
Date: 08/12/11 Time: 08:34:12

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-5
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 27.47 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-5-Falling Head Test #2)

Initial Displacement: 0.87 ft Static Water Column Height: 25.52 ft
Total Well Penetration Depth: 25.52 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 79.29 ft/day y0 = 1.224 ft

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B-5 Falling Head Test #2

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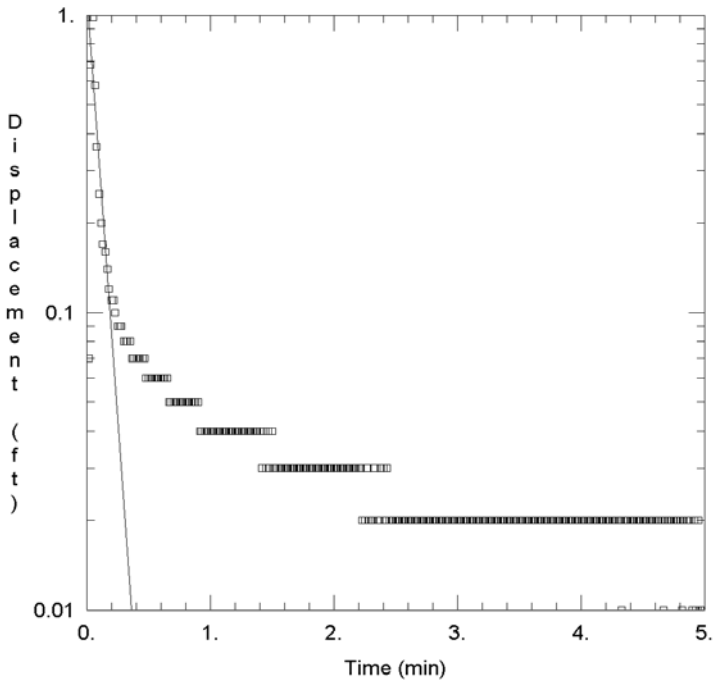
FIG. D-9



WELL TEST ANALYSIS

Data Set: I:\...\B-5 SlugTest-R2.aqt
Date: 08/12/11 Time: 08:35:37

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-5
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 27.47 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-5-Rising Head Test #2)

Initial Displacement: 0.98 ft
Total Well Penetration Depth: 25.52 ft
Casing Radius: 0.083 ft
Static Water Column Height: 25.52 ft
Screen Length: 5. ft
Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 60.43 ft/day
y0 = 1.183 ft

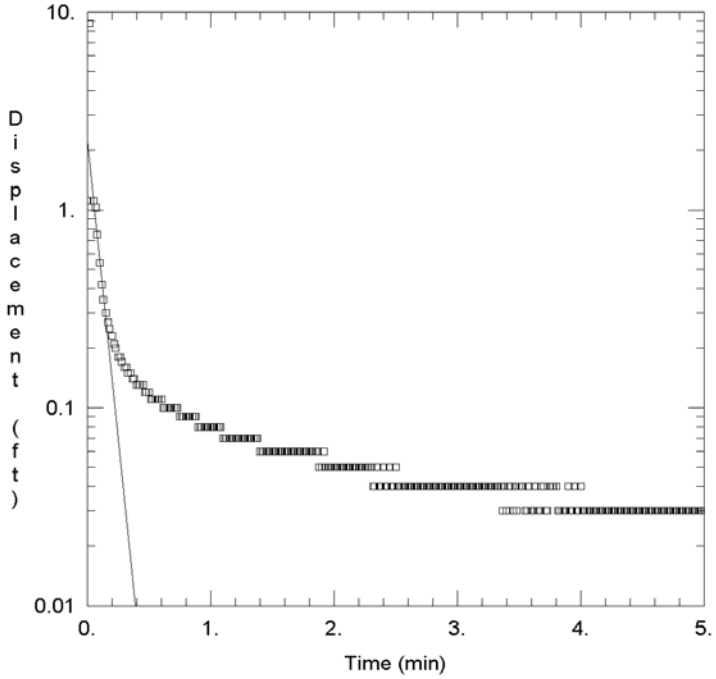
Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-5 Rising Head Test #2

August 2011 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D-10



WELL TEST ANALYSIS

Data Set: I:\...\B-5 SlugTest-F3.aqt
Date: 08/12/11 Time: 08:34:42

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-5
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 27.47 ft
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-5-Falling Head Test #3)

Initial Displacement: 1.11 ft
Total Well Penetration Depth: 25.52 ft
Casing Radius: 0.083 ft
Static Water Column Height: 25.52 ft
Screen Length: 5. ft
Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 63.74 ft/day
y0 = 2.249 ft

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-5 Falling Head Test #3

August 2011 21-1-21144-007

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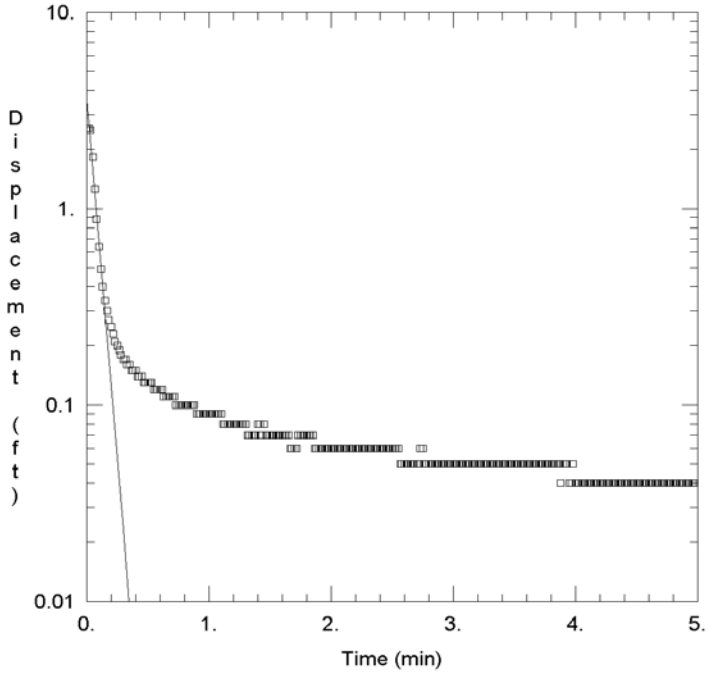
FIG. D-11



WELL TEST ANALYSIS

Data Set: I:\...\B-5 SlugTest-R3.aqt
Date: 08/12/11 Time: 08:36:02

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-5
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 27.47 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-5-Rising Head Test #3)

Initial Displacement: 2.56 ft Static Water Column Height: 25.52 ft
Total Well Penetration Depth: 25.52 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 77.06 ft/day y0 = 3.595 ft

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-5 Rising Head Test #3

August 2011 21-1-21144-007

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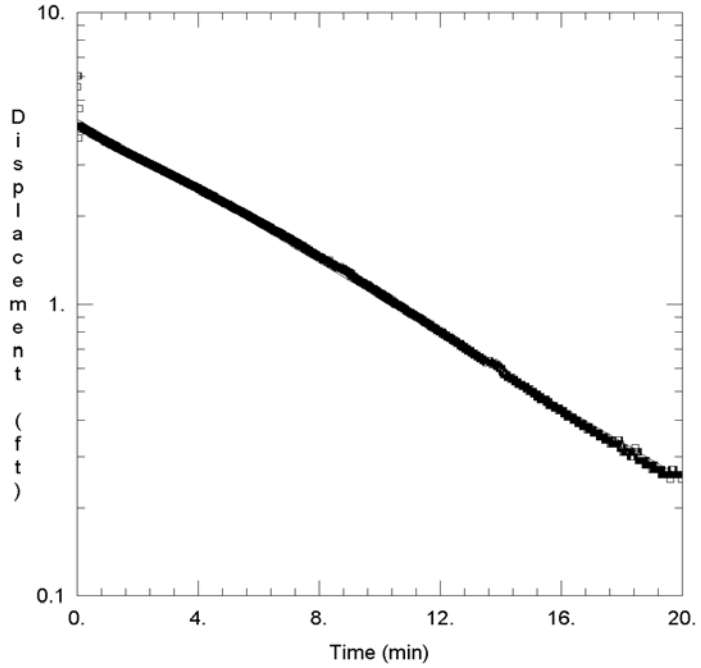
FIG. D-12



WELL TEST ANALYSIS

Data Set: I:\...\B-6 SlugTest-F1.aqt
Date: 08/12/11 Time: 08:36:36

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-6
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 31.57 ft
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-6-Falling Head Test #1)

Initial Displacement: 6.03 ft
Total Well Penetration Depth: 29.83 ft
Casing Radius: 0.083 ft
Static Water Column Height: 29.83 ft
Screen Length: 5. ft
Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 0.6606 ft/day
y0 = 4.292 ft

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-6 Falling Head Test #1

August 2011 21-1-21144-007

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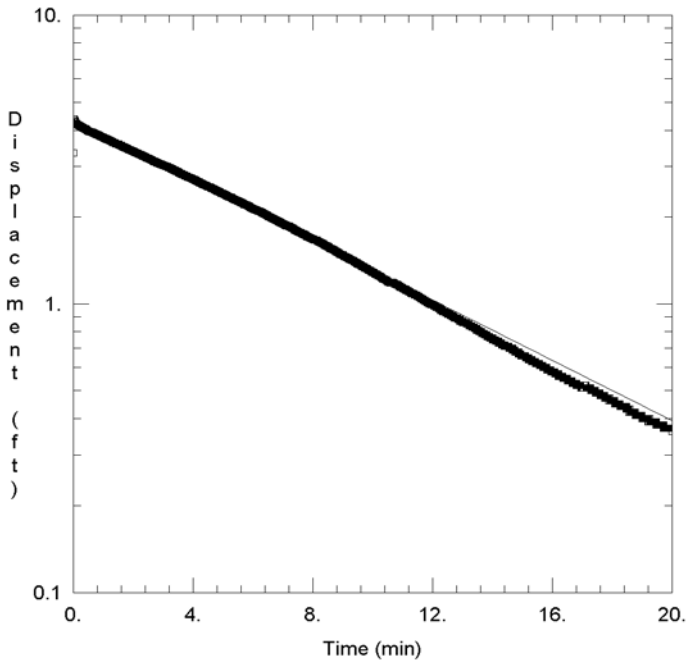
Geotechnical and Environmental Consultants

FIG. D-13



WELL TEST ANALYSIS

Data Set: I:\...\B-6 SlugTest-R1.aqt
Date: 08/12/11 Time: 08:37:54

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-6
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 31.57 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-6-Rising Head Test #1)

Initial Displacement: 4.37 ft Static Water Column Height: 29.83 ft
Total Well Penetration Depth: 29.83 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 0.5628 ft/day y0 = 4.379 ft

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-6 Rising Head Test #1

August 2011 21-1-21144-007

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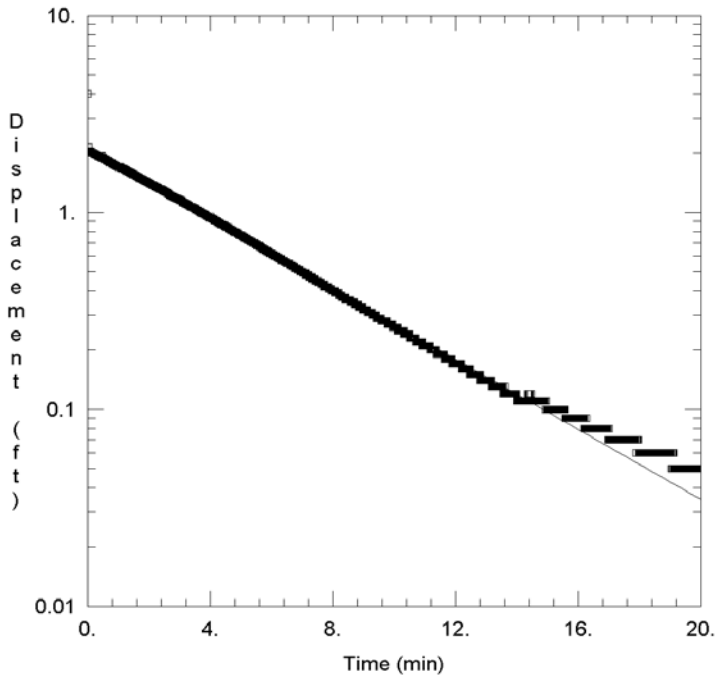
FIG. D-14



WELL TEST ANALYSIS

Data Set: I:\...\B-6 SlugTest-F2.aqt
Date: 08/12/11 Time: 08:37:02

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-6
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 31.57 ft
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-6-Falling Head Test #2)

Initial Displacement: 3.99 ft
Static Water Column Height: 29.83 ft
Total Well Penetration Depth: 29.83 ft
Screen Length: 5. ft
Casing Radius: 0.083 ft
Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 0.9574 ft/day
y0 = 2.107 ft

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-6 Falling Head Test #2

August 2011 21-1-21144-007

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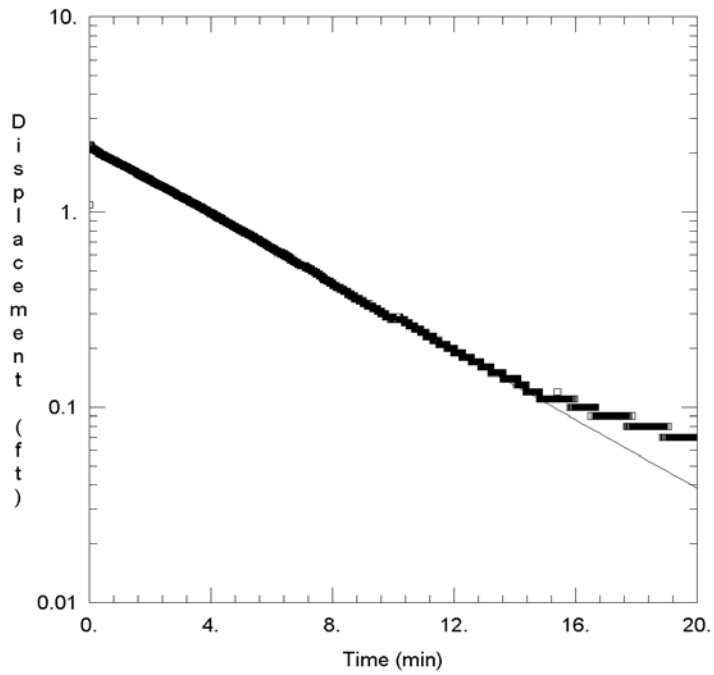
FIG. D-15



WELL TEST ANALYSIS

Data Set: I:\...\B-6 SlugTest-R2.aqt
Date: 08/12/11 Time: 08:38:18

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-6
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 31.57 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-6-Rising Head Test #2)

Initial Displacement: 2.21 ft Static Water Column Height: 29.83 ft
Total Well Penetration Depth: 29.83 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 0.9447 ft/day y0 = 2.199 ft

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Seattle, Washington

B-6 Rising Head Test #2

August 2011 21-1-21144-007

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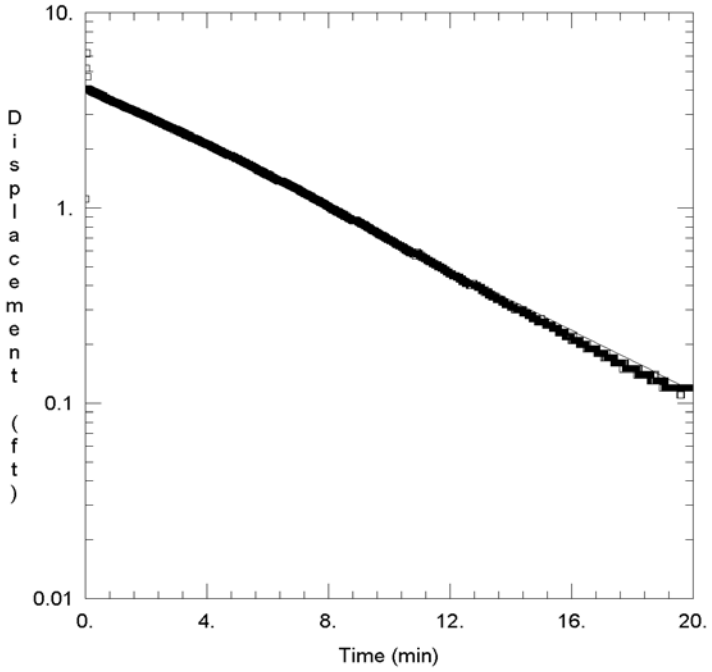
FIG. D-16



WELL TEST ANALYSIS

Data Set: I:\...\B-6 SlugTest-F3.aqt
Date: 08/12/11 Time: 08:37:24

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-6
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 31.57 ft Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-6-Falling Head Test #3)

Initial Displacement: 4.05 ft Static Water Column Height: 29.83 ft
Total Well Penetration Depth: 29.83 ft Screen Length: 5. ft
Casing Radius: 0.083 ft Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined Solution Method: Bouwer-Rice
K = 0.8368 ft/day y0 = 4.15 ft

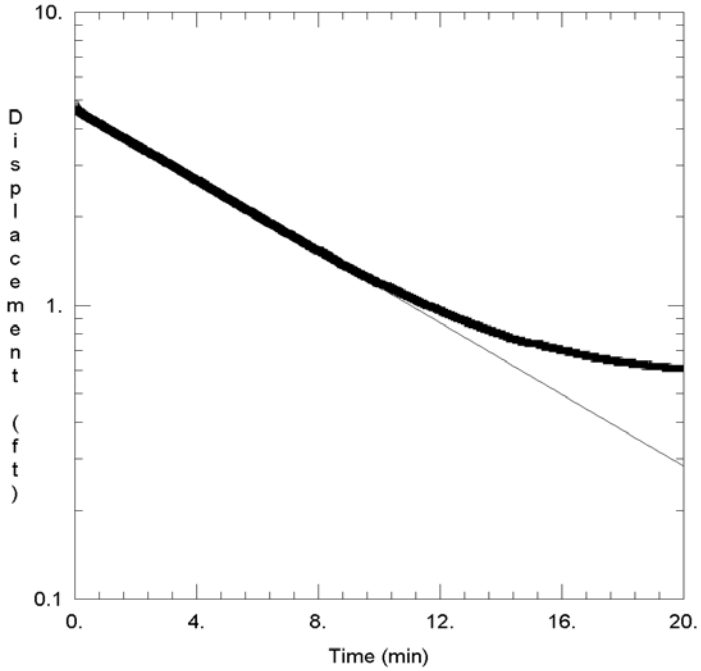
Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-6 Falling Head Test #3

August 2011 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D-17



WELL TEST ANALYSIS

Data Set: I:\...\B-6 SlugTest-R3.aqt
Date: 08/12/11 Time: 08:38:40

PROJECT INFORMATION

Company: Shannon & Wilson
Client: HDR
Project: 21-1-21144-007
Location: Seward Park
Test Well: B-6
Test Date: 7/14/11

AQUIFER DATA

Saturated Thickness: 31.57 ft
Anisotropy Ratio (Kz/Kr): 0.1

WELL DATA (B-6-Rising Head Test #3)

Initial Displacement: 4.83 ft
Static Water Column Height: 29.83 ft
Total Well Penetration Depth: 29.83 ft
Screen Length: 5. ft
Casing Radius: 0.083 ft
Well Radius: 0.083 ft

SOLUTION

Aquifer Model: Confined
Solution Method: Bouwer-Rice
K = 0.6567 ft/day
y0 = 4.719 ft

Genesee-Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

B-6 Rising Head Test #3

August 2011 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D-18

APPENDIX E

SEISMIC REFRACTION SURVEY

APPENDIX E

SEISMIC REFRACTION SURVEY

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REPORT

Duoos, Phil, Report, Seismic Refraction Survey, Seward Park Site, Seattle, Washington, September 15, 2011,

APPENDIX E

SEISMIC REFRACTION SURVEY

Phil Duoos performed seismic refraction surveys under subcontract with Shannon & Wilson from July 5 through 8, 2011. A total of 11 seismic lines were attempted during the work. Of the 11 lines, 5 lines at the proposed Alternative 1 site and 4 lines at the proposed Alternative 2 site provided data that could be analyzed. The two remaining lines were noted to have interference due to wind and wave noise. The results from the surveys are presented in this appendix titled, "Report, Seismic Refraction Survey, Seward Park Site, Seattle, Washington," dated September 15, 2011, by Phil Duoos. We used the seismic refraction method to evaluate general subsurface stratigraphy and estimate variations in rock strength at the two storage vault locations.

FINAL REPORT

SEISMIC REFRACTION SURVEY

SEWARD PARK SITE
SEATTLE, WASHINGTON

FOR

SHANNON & WILSON, INC.
SEATTLE, WASHINGTON

SEPTEMBER 15, 2011

PHILIP H. DUOOS
GEOPHYSICAL CONSULTANT

September 15, 2011 Our Ref: 985-11

Mr. Michael Harney, P.E.
Shannon & Wilson
400 North 34th Street, Suite 100
P.O. Box 98103
Seattle, WA 98103

FINAL REPORT: Seismic Refraction Survey
Seward Park Site
Seattle, Washington

Dear Mr. Harney:

This letter report contains the results of the seismic refraction survey that I performed at the site during the period of July 5 - 8, 2011. The purpose of the investigation was to determine the depth to rock and the seismic velocity of the rock to help determine rock rippability. A brief description of the seismic refraction method is attached (Attachment A).

Field Methodology

The locations of the seismic lines are shown on the Site Maps. Map 1 shows the locations of the lines in the East Parking Lot, and Map 2 shows the Tennis Court Area. Two seismic lines were oriented along the length of each site in grassy areas along the edges of the parking lot or tennis courts. Three crosslines were run in each area, and extended from Lake Washington and up onto the hillside as terrain and brush allowed. Not all of the data were interpreted at the Tennis Court Area. Lines 9a and 9b were attempted and the wind noise from the lake limited the quality of the data for portions of the lines. These long lines were shifted to the west (SL10 and SL11). Seismic Line SL3 was not interpreted as the tennis court pavement limited the quality of the data for portions of the line.

We used a tape measure, paint, chalk, and/or PVC pin flags to mark the locations of the geophones along each line. The ends of some of the lines in the brushy hillsides were marked with wood stakes where they could be safely placed away from the park activity. The seismic lines used geophone spacings ranging from 5 to 10 feet, and ranged in length from about 120 feet to 230 feet.

Relative elevation changes to each geophone location were obtained using a hand-level, and are estimated to be within +/- 1 foot of the actual elevations. The hand-level data were referenced to the water level, which was about 22.15 feet elevation at the time of the survey based on a gauge in Kenmore, Washington (obtained by an internet search).

Philip H. Duoos **13503 NE 78th Place, Redmond, Washington, 98052**
PH/FAX: (425) 882-2634 **Email: geopyg@aol.com**

The field investigation was performed using a 24-channel digital seismograph to record the data. A slide-hammer source was used to generate a seismic wave at 50 to 80 foot intervals along each seismic spread Aside from the wind noise on the lines along water near the tennis courts, and the line across the tennis court, the data quality was good. The parking lots were closed to vehicles during the survey, and we waited for pedestrians to be well away from the lines when acquiring data.

Interpretation Results

The results of the seismic survey are shown on the interpretation profiles for each seismic line. The profiles are grouped together for the East Parking Lot Area and the Tennis Court Area. The profiles show the geophone locations along the ground surface, the calculated depth points below each geophone, and the interpreted interfaces (dashed lines). Results from intersecting seismic lines (blue stars) and nearby borings (red lines) are also noted on each profile. Please note that the horizontal scale is at 1 inch = 30 feet, and the vertical scale is at 1 inch = 15 feet. The elevation ranges are the same for all of the profiles to make comparison of the lines easier.

The basic geologic units were identified based on the interpreted compressional wave velocities (in feet/second), the site-specific information (borings), and results from other seismic surveys I have performed in the region. Their probable classification is indicated on the following table.

TABLE 1

Seismic Velocity Classification

SYMBOL	SEISMIC VELOCITY (feet per second)	PROBABLE CLASSIFICATION
O1	1,000 - 1,800	Unconsolidated soil; dry, loose overburden, fill and alluvium
BX	5,500 – 6,900	Moderate velocity bedrock, competent with some weathering and/or fractured.

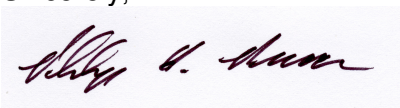
The layer of highly weathered rock overlying the more competent rock observed in the borings was not evident in the seismic data. It would be a relatively thin layer in most cases, and not observed in the data due to the physics of seismic refraction method (see Attachment A). However, the correlation between the seismic lines was very good; within a foot or two in most cases. The seismic results also compare well with the boring data in most cases. One exception is on Line SL2 in the Tennis Court Area. The middle portion of the line shows rock becoming shallower. This however puts rock too shallow at Boring B4, and also at the intersection of SL10. This shallower rock may be due to a “velocity pull up” which can occur if a thin, higher velocity zone is not observed and accounted for. Extrapolating the rock interface from the ends of Line SL2 in a smooth curving fashion to remove the “pull up” also ties in fairly well with the boring data and the SL10 results.

Summary

The depth correlations at the intersections of the seismic lines are fairly good, and the layer velocities calculated for all of the lines are very similar as well, indicating that the interpretation results are reasonable. The seismic results also agree in general to the boring results. As with any geophysical technique, these results are interpretive in nature and represent the best estimate of subsurface conditions considering the limitations of the geophysical method employed. Only direct observations using borings or other means can ultimately characterize subsurface conditions, using the geophysical results as a guide. Review of this information by someone familiar with the geology of the area may also provide additional insight into the seismic results.

Please feel free to contact me if you have any questions or comments regarding this information, or if you require further assistance. I appreciated the opportunity to work with you on this project and look forward to providing you with geophysical services in the future.

Sincerely,



Philip H. Duoos
Geophysical Consultant

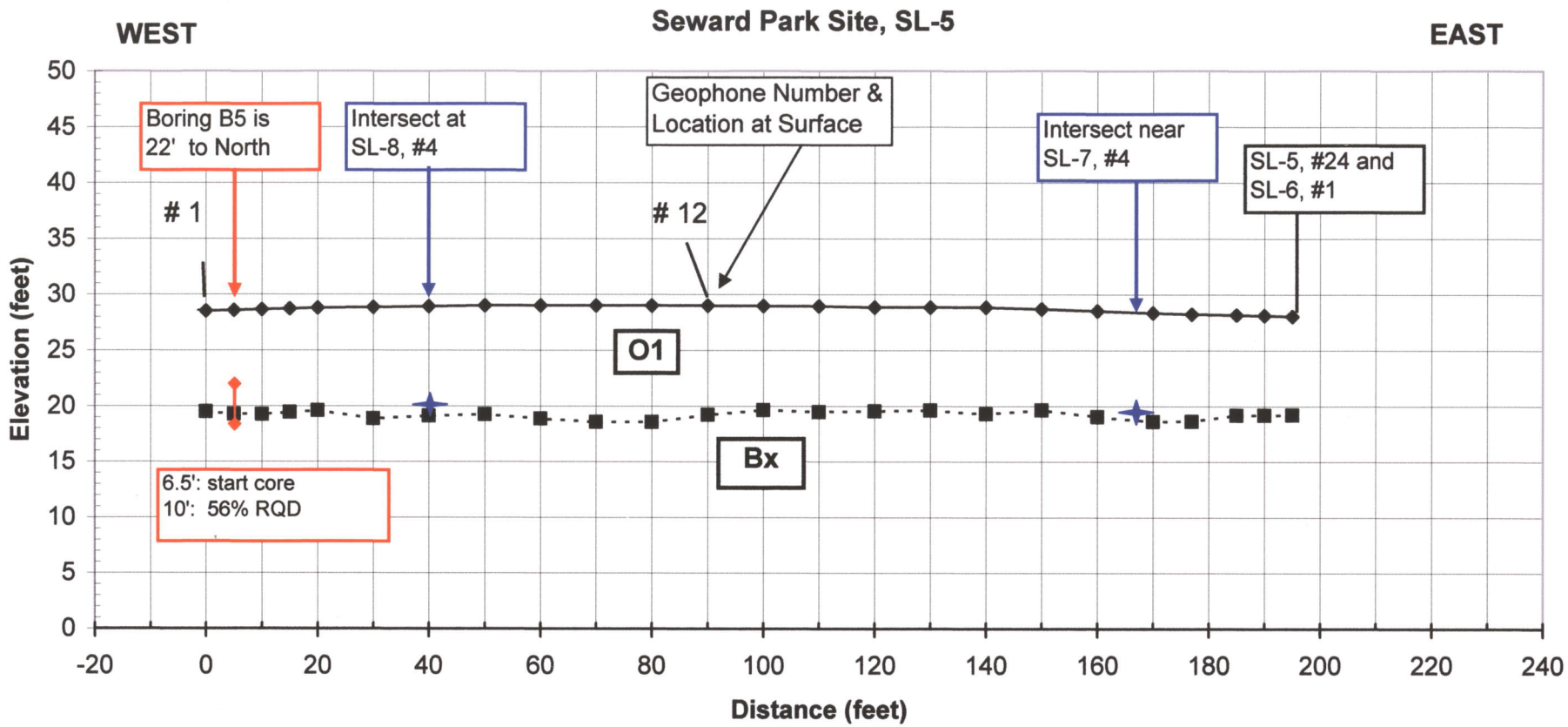
Attachments

Map 1: East Parking Lot Area
Map 2: Tennis Court Area
Profiles: East Parking Lot Area (Lines SL-1, 5, 6, 7, & 8)
Profiles: Tennis Court Area (Lines SL-2, 4, 10 and 11)
Attachment A: Seismic Refraction Methodology

SEISMIC REFRACTION PROFILES

EAST PARKING LOT AREA
SEWARD PARK, SEATTLE, WA

Lines SL-5 and SL-6, running West to East
Lines SL-1, SL-7 and SL-8 running South to North



Seismic Layer Velocities			
O1:	1,000	-	1,600 fps
Bx:	6,100	-	6,900 fps

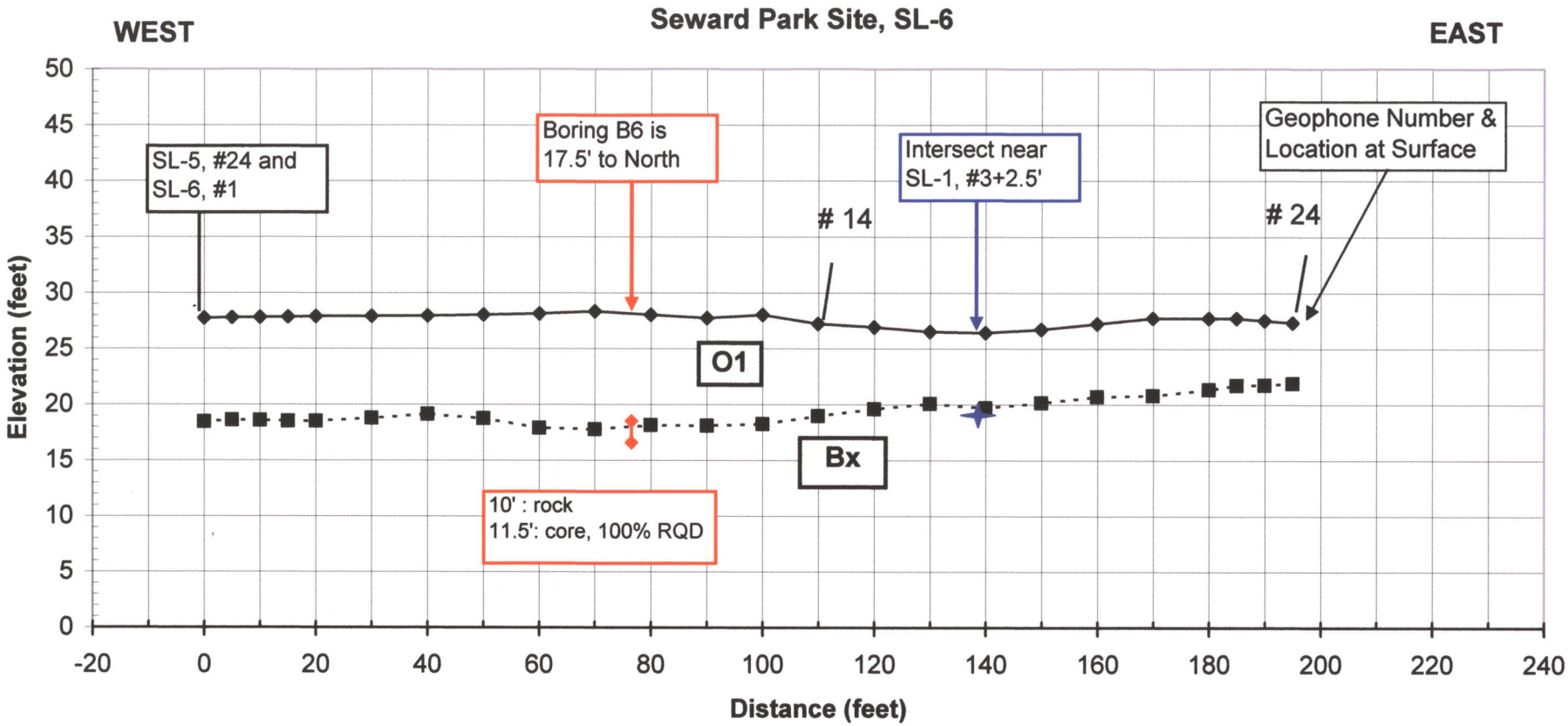
Horizontal Scale:	1" =	30'
Vertical Scale:	1" =	15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-5

EAST LOT, SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,000	-	1,300 fps
Bx:	5,800	-	6,400 fps

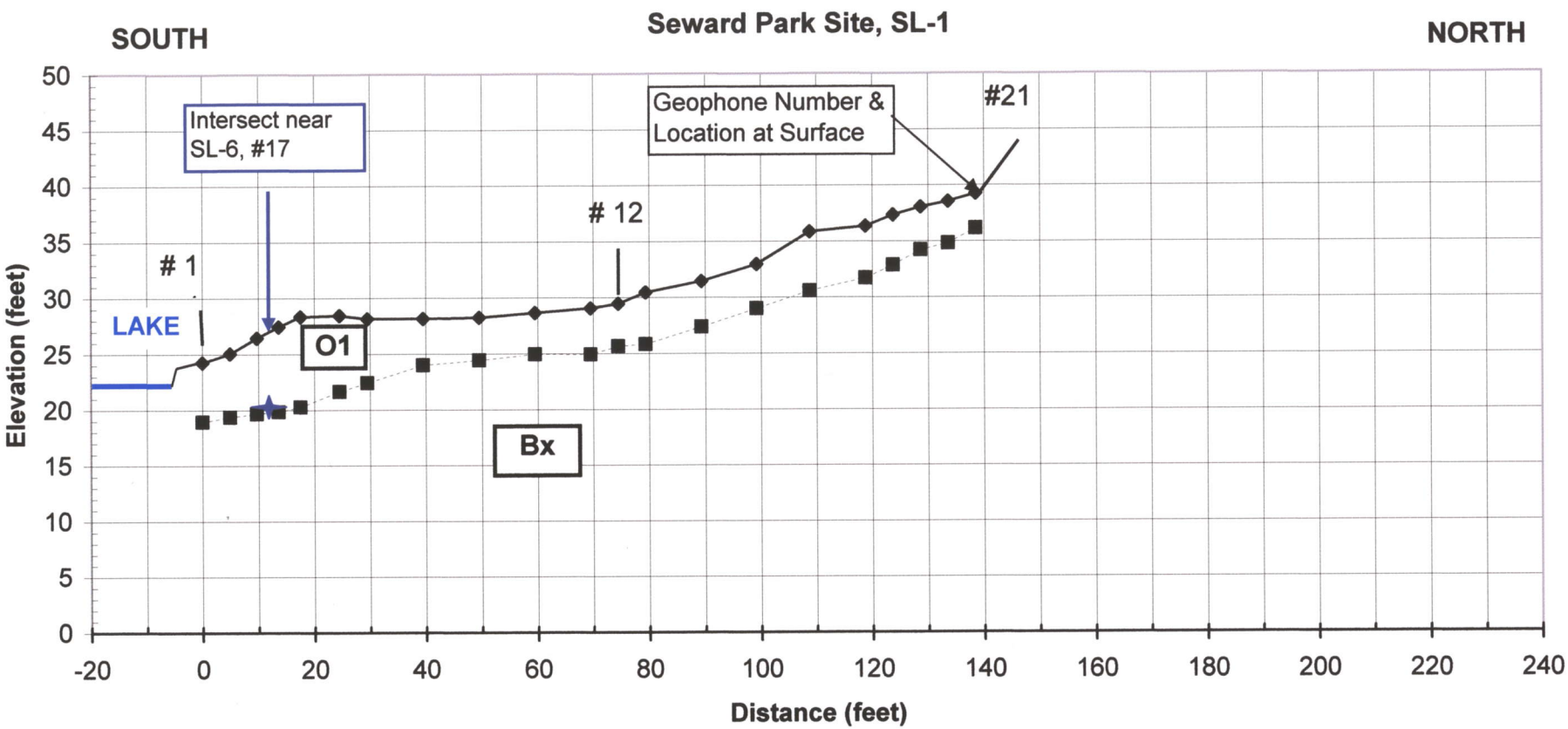
Horizontal Scale:	1" = 30'
Vertical Scale:	1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-6

EAST LOT, SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,000	-	1,500 fps
Bx:	5,700	-	6,200 fps

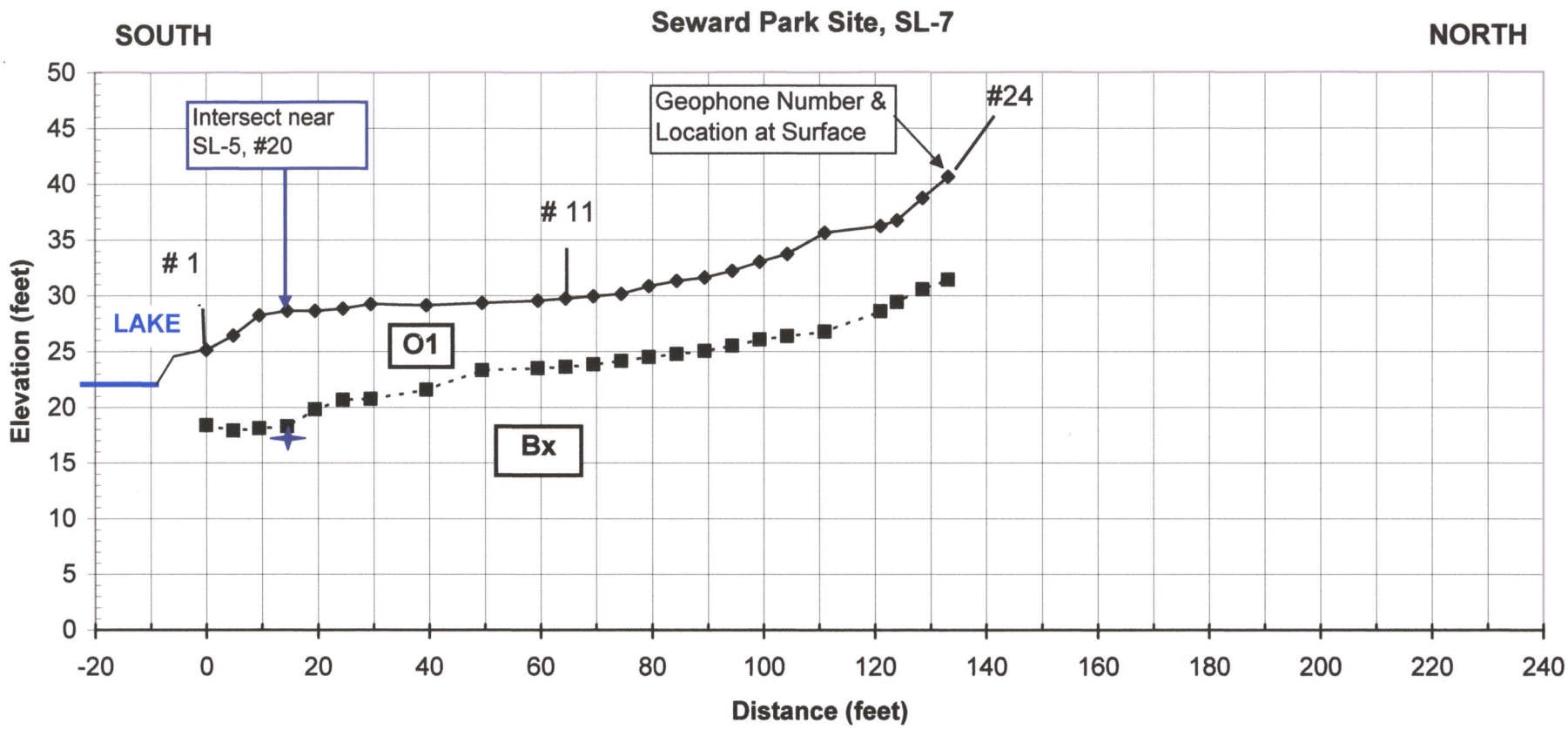
Horizontal Scale: 1" = 30'
Vertical Scale: 1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-1

EAST LOT, SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,100	-	1,800 fps
Bx:	5,900	-	6,800 fps

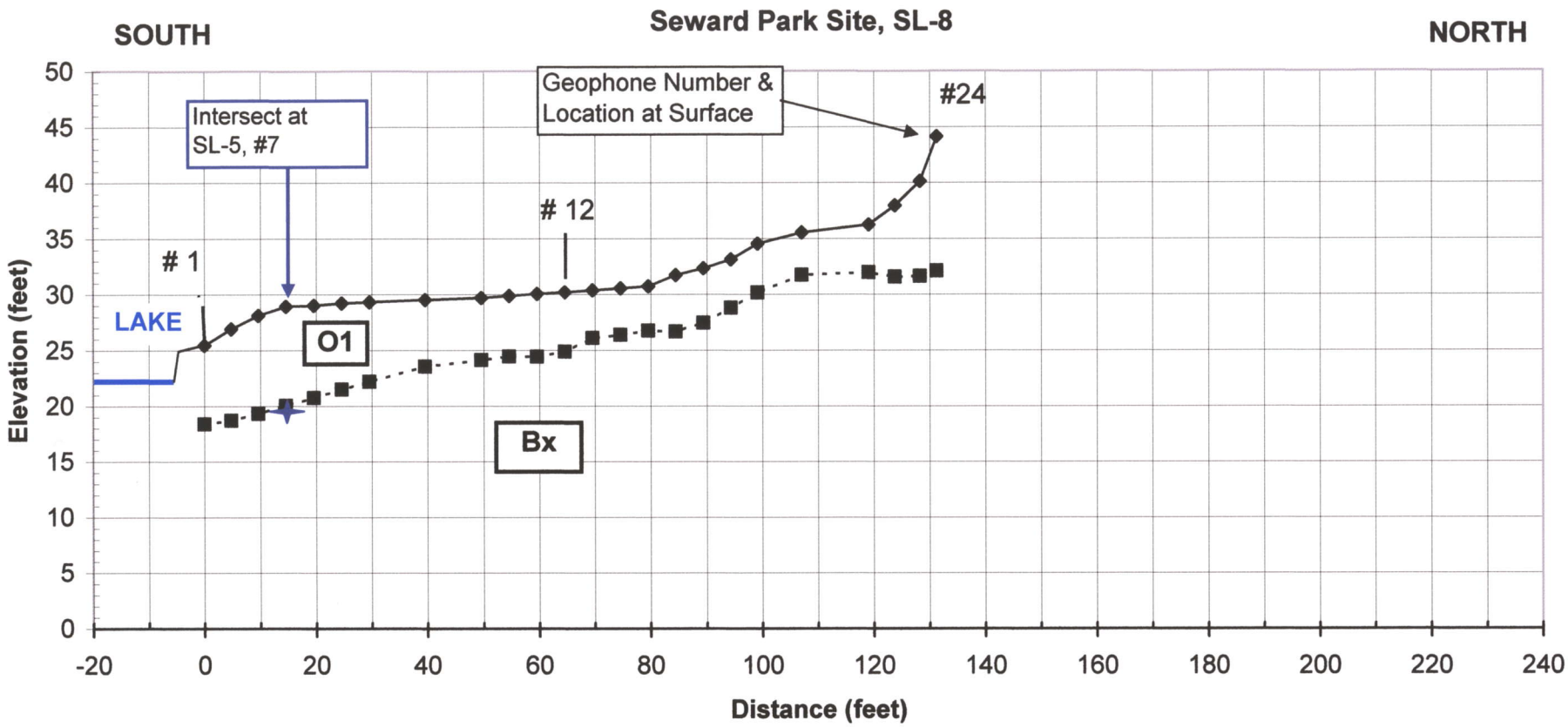
Horizontal Scale:	1" =	30'
Vertical Scale:	1" =	15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-7

EAST LOT, SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,000	-	1,300 fps
Bx:	5,800	-	6,400 fps

Horizontal Scale: 1" = 30'
Vertical Scale: 1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-8

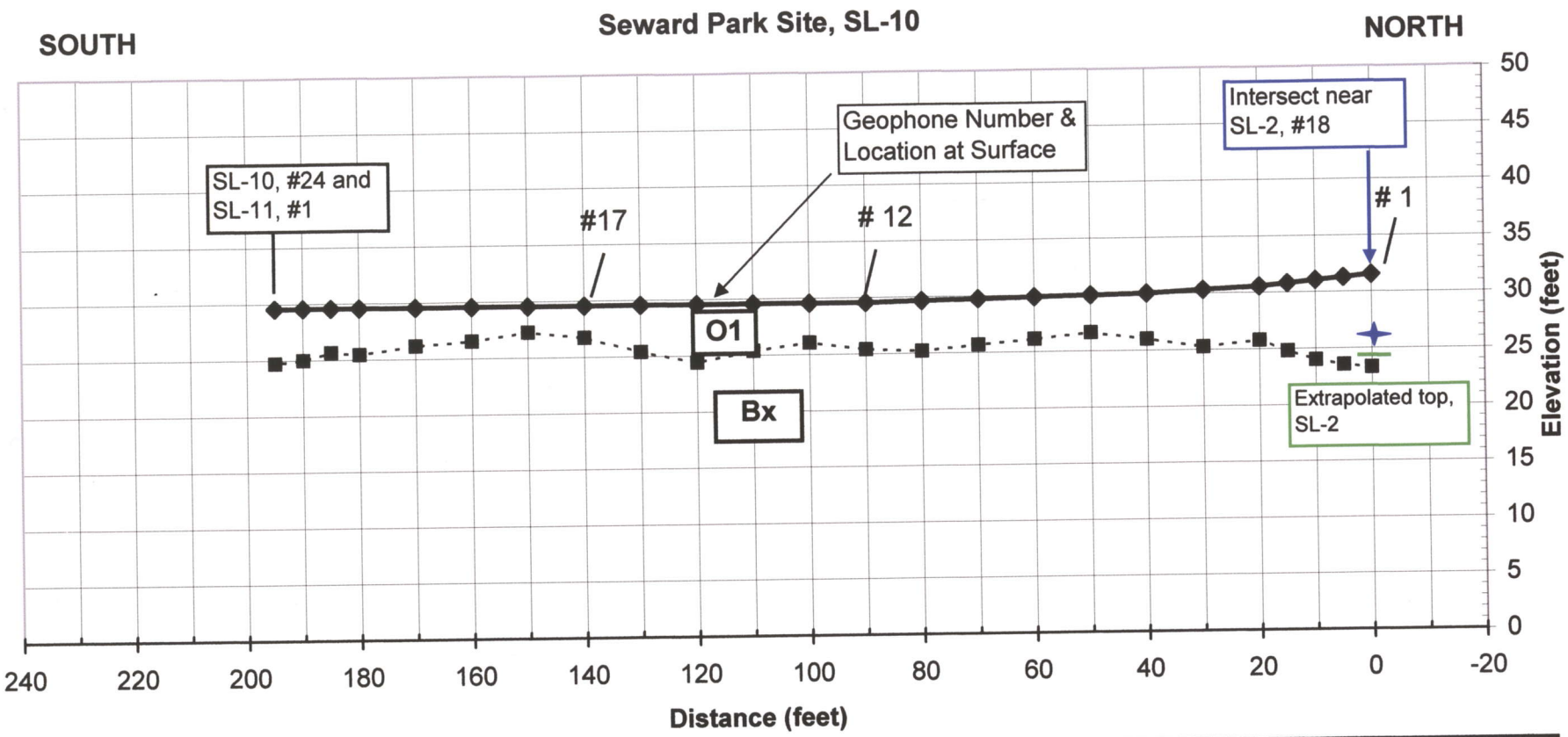
EAST LOT, SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011

SEISMIC REFRACTION PROFILES

TENNIS COURT AREA
SEWARD PARK, SEATTLE, WA

Lines SL-10 and SL-11, running South to North
Lines SL-2 and SL-4 running West to East



Seismic Layer Velocities			
O1:	1,050	-	1,450 fps
Bx:	5,800	-	6,700 fps

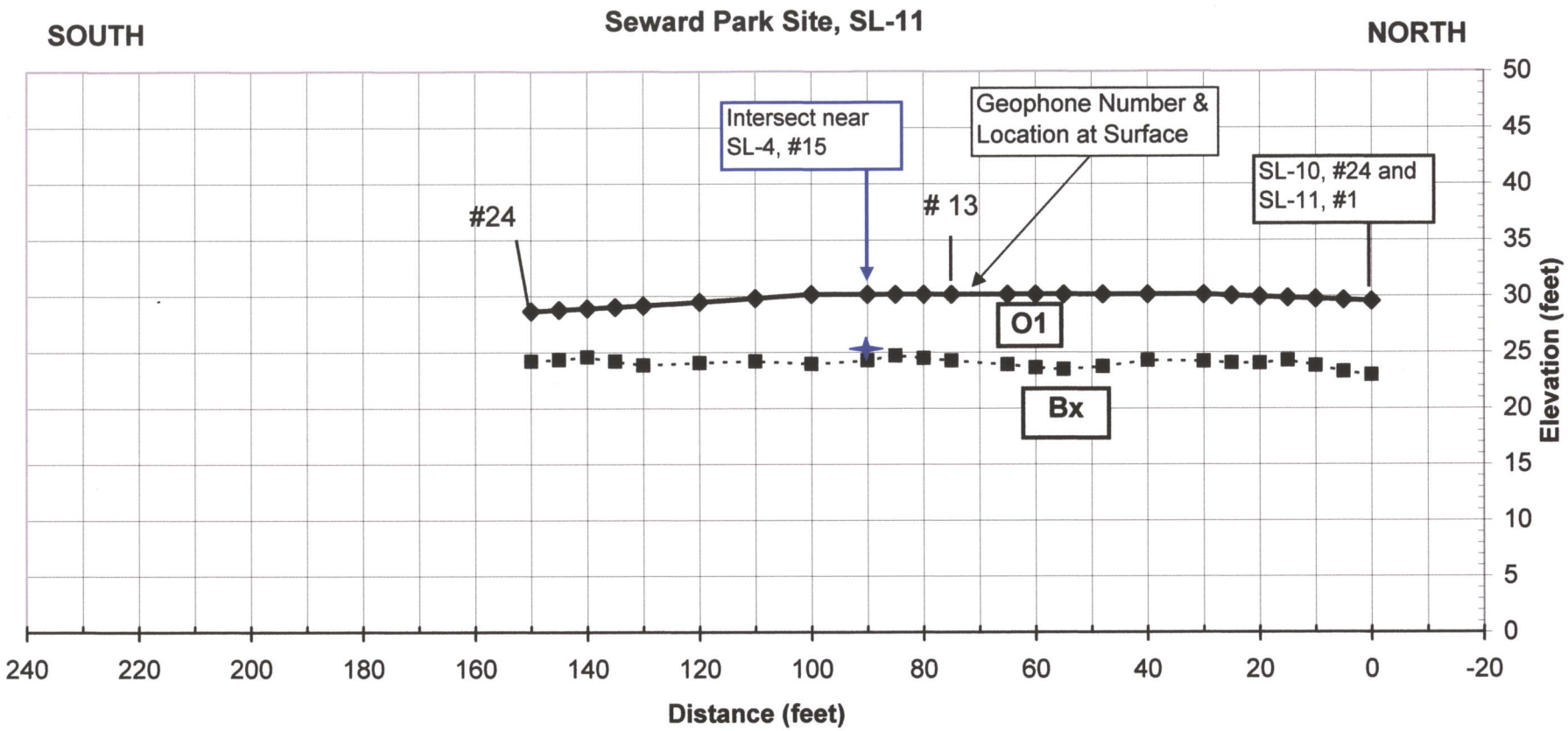
Horizontal Scale:	1" = 30'
Vertical Scale:	1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-10

TENNIS COURT AREA
SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,080	-	1,400 fps
Bx:	5,500	-	6,100 fps

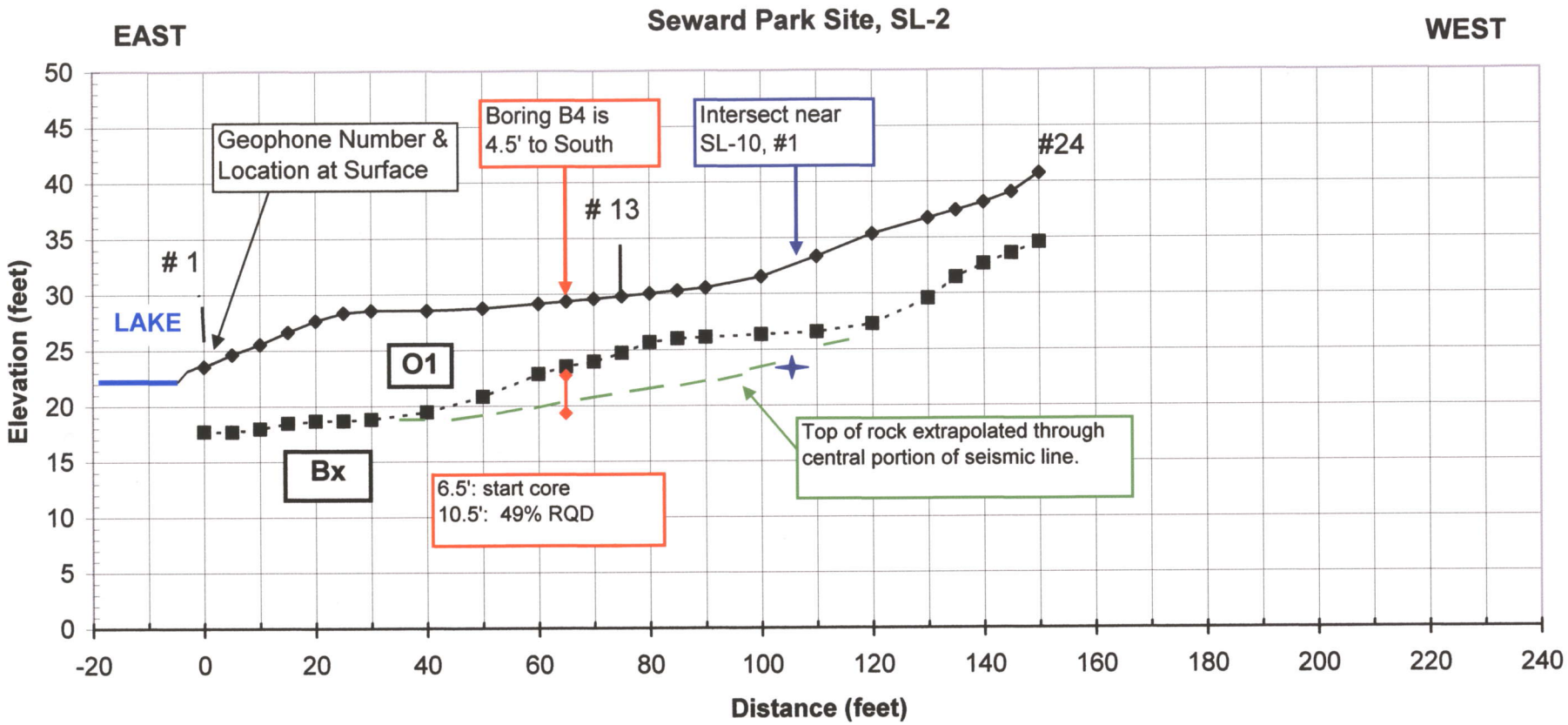
Horizontal Scale:	1" = 30'
Vertical Scale:	1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-11

TENNIS COURT AREA
SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,000	-	1,400 fps
Bx:	6,100	-	6,600 fps

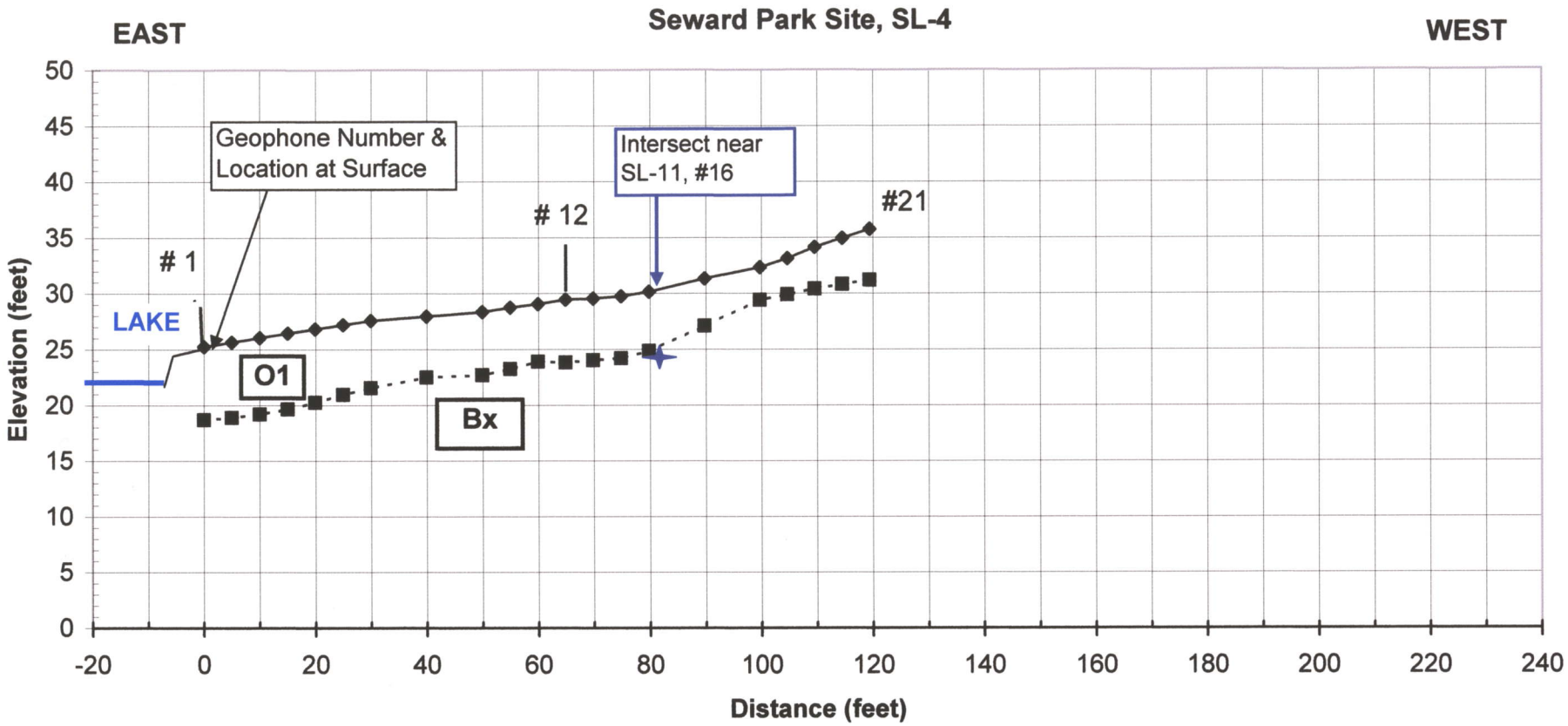
Horizontal Scale: 1" = 30'
Vertical Scale: 1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-2

TENNIS COURT AREA
SEWARD PARK
SEATTLE, WASHINGTON

For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



Seismic Layer Velocities			
O1:	1,000	-	1,200 fps
Bx:	5,600	-	6,300 fps

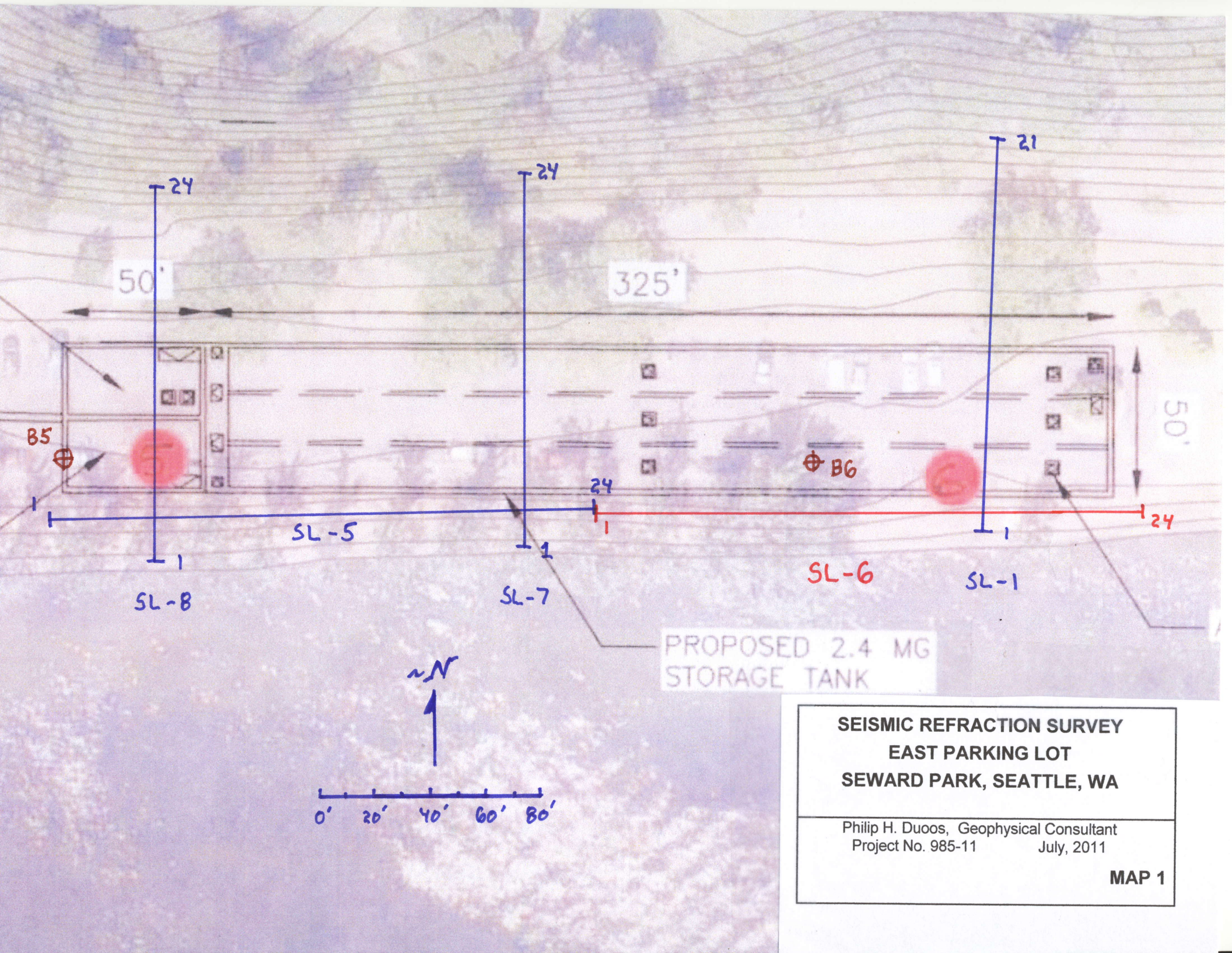
Horizontal Scale:	1" = 30'
Vertical Scale:	1" = 15'

Ground surface elevations are approximate using hand level measurements referenced to water surface elevation (~ 22.15 feet).

SEISMIC PROFILE, SL-4

TENNIS COURT AREA
SEWARD PARK
SEATTLE, WASHINGTON

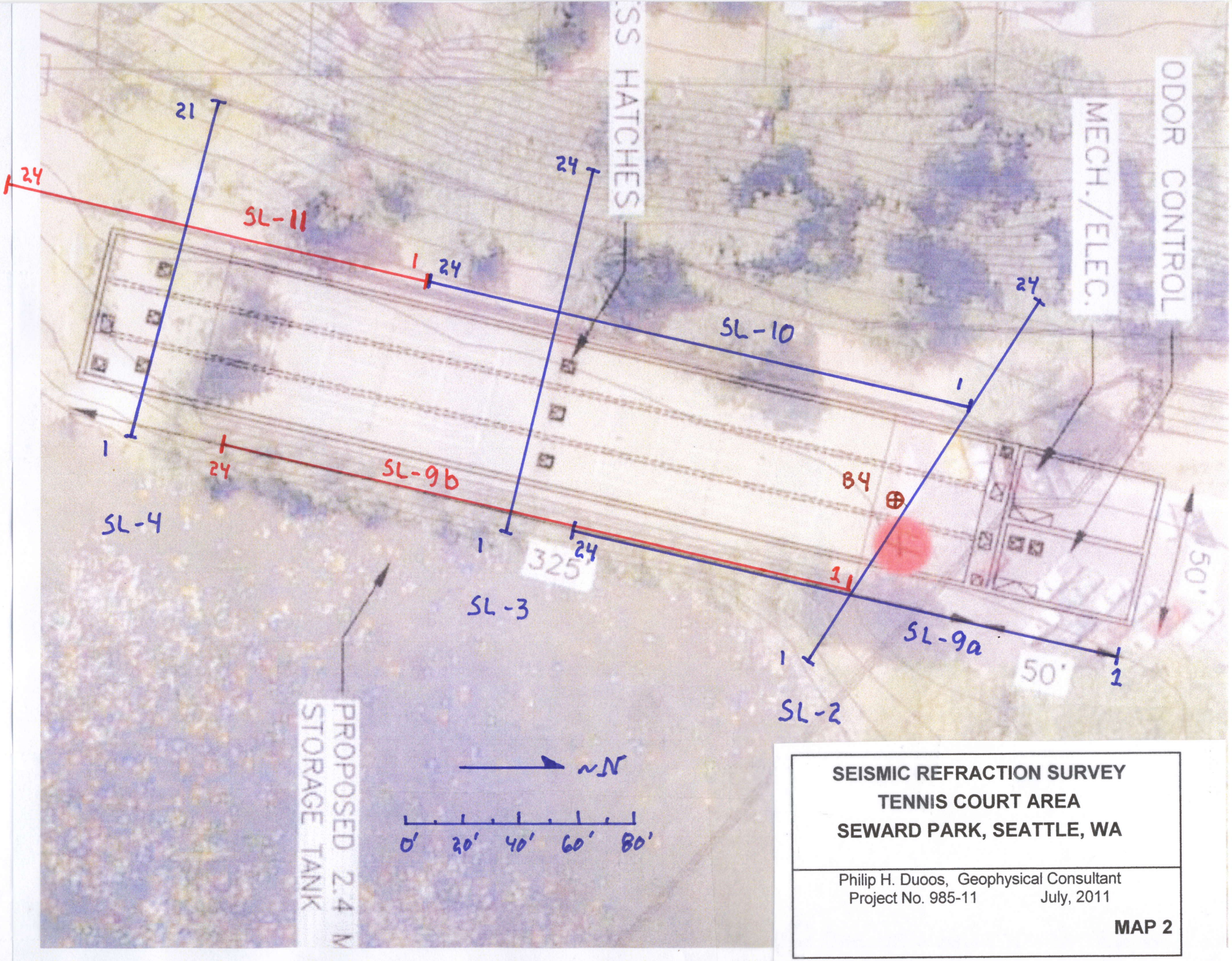
For Shannon & Wilson, Inc.
Philip H. Duoos, Geophysical Consultant
PN 985-11, July 31, 2011



SEISMIC REFRACTION SURVEY
EAST PARKING LOT
SEWARD PARK, SEATTLE, WA

Philip H. Duoos, Geophysical Consultant
Project No. 985-11 July, 2011

MAP 1



SEISMIC REFRACTION SURVEY
TENNIS COURT AREA
SEWARD PARK, SEATTLE, WA

Philip H. Duoos, Geophysical Consultant
Project No. 985-11 July, 2011

MAP 2

SEISMIC REFRACTION METHODOLOGY

Overview

The seismic refraction method is used to evaluate numerous subsurface conditions; including depth to and strength (rippability) of rock, depth to water, and general subsurface stratigraphy.

The seismic refraction method uses an induced shock wave. As the shock wave propagates through the earth, it is affected by the materials through which it passes. Geophones placed on the ground surface record the ground motion caused by the resultant wave. A seismograph measures the time required for the resultant wave to arrive at each geophone. These geophones are located at selected distances from the wave source. Analysis of the data (travel times and distances) provides seismic velocities of subsurface material and depths to significant velocity interfaces.

Geologic conditions yielding higher seismic velocities include increased amounts of water, clay, cobbles, and rock fragments, greater compaction of overburden materials, and greater competency of rock. Several factors can affect the effectiveness of the seismic method including the proximity of cultural interferences (such as powerlines and traffic noise), surface conditions (such as loose soil), the size and depth of the target, and the seismic wave velocity contrast between stratigraphic units. Seismic velocities must increase with depth for a reliable interpretation of the data.

Calculations

The description of the travel of seismic refraction waves through the earth uses the same equation that describes the refraction of light: Snell's Law. The following is a brief summary of the basic theory for a simple two-layer geologic model as discussed by Redpath (Redpath, 1973).

Snell's Law is stated as:

$$\frac{SIN\alpha}{SIN\beta} = \frac{V_1}{V_2}$$

and at the critical angle of incidence for a refracted seismic wave (β=90°), it becomes:

$$SIN\alpha = \frac{V_1}{V_2}$$

where V_1 and V_2 are the seismic wave velocities for the upper and lower layers, respectively.

The seismic refraction method measures the amount of time it takes the seismic energy to travel from the energy source to the geophones placed along the ground surface. The arrival time for the seismic wave at each geophone is plotted corresponding to the distance of the geophone from the energy source, creating a time-distance graph (Figure 1).

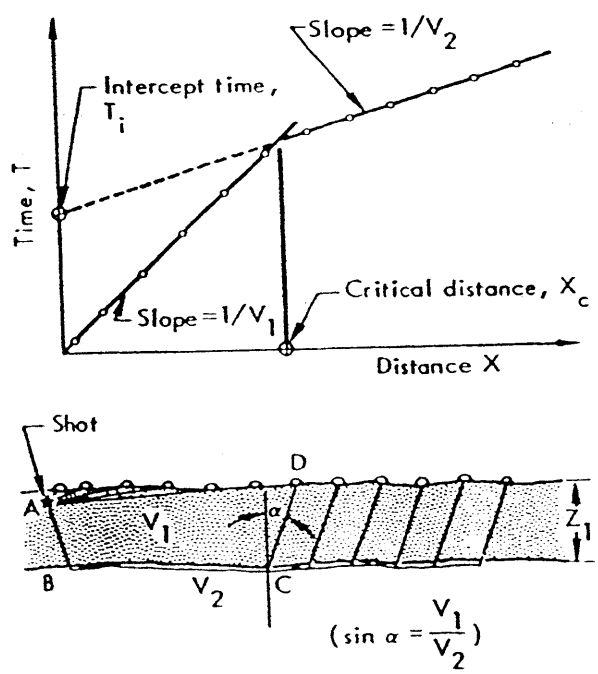


Figure 1: Two-layer geologic model and associated time-distance graph (Redpath, 1973).

The time required for the energy to reach the geophones near the source (direct wave arrivals) is based only on the seismic velocity of the energy traveling though the upper (low velocity) layer. At a certain distance from the source, called the critical distance, the first seismic waves to reach the

geophones will be those that have refracted from a deeper, higher velocity layer. Although these waves have traveled a greater distance than the direct waves, they have traveled at a greater velocity over most of their path, and thus arrive before the slower direct arrivals to the geophones farther from the source. Successively deeper layers with higher velocities affect the time-distance graph in a similar manner.

Using the time-distance graph, the velocities of the layers can be calculated (based on the slope of the arrival times), and the layer thicknesses can be calculated using the intercept times. The equation used in the time-intercept method to determine thicknesses is:

$$Z_1 = \frac{T_1 V_1}{2COS(SIN^{-1} V_1 / V_2)} + \frac{\text{SHOT DEPTH}}{2};$$

Figure 2 is a sketch of a multiple layer case and the corresponding time distance curve showing the intercept times.

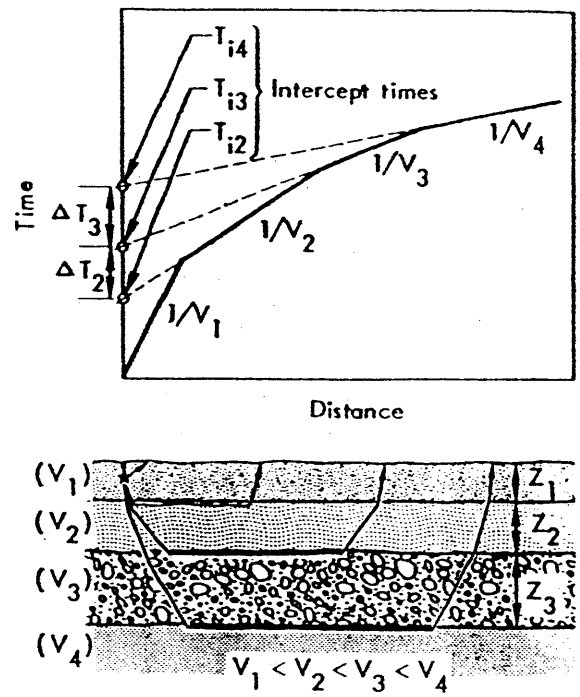


Figure 2: Multiple layer geologic model and associated time-distance graph (Redpath, 1973).

For more complex geologic models, as is usually observed, additional energy source locations are required at both ends of a seismic line as was done for this survey. The layer velocities are calculated using the data from all of the time-distance curves (delay-time method).

Limitations

Two types of geologic conditions can cause a *hidden zone* problem. One type of hidden zone is a layer with a lower velocity than the layer above it. Energy approaching the layer at the critical angle will pass through the layer, and will not be refracted back to the surface until it encounters a deeper layer with a higher velocity, so no first arrivals are observed from the low-velocity layer. The presence of an unknown low-velocity layer will cause the calculated depths to be greater than the actual depths.

The other type of hidden zone is a layer with a greater velocity than the layer above it, but one that is too thin and/or does not have a large enough velocity contrast. The effect of a thin layer will cause the calculated depths to be shallower than the actual depths.

In areas with hidden zones, the amount of error can be determined based on direct observations (such as test pits or boreholes), and can be compensated for over the rest of the seismic lines.

References

Redpath, Bruce B. (1973). "Seismic Refraction Exploration for Engineering Site Investigations." *Tech. Report E-73-4*, U.S. Army Engineer Waterways Experiment Station Explosive Excavation Research Laboratory, Livermore, CA

APPENDIX F

PREVIOUS EXPLORATIONS

APPENDIX F

PREVIOUS EXPLORATIONS

TABLE OF CONTENTS

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F.1 EXISTING INFORMATION F-1

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FIGURE

F-1 Shannon & Wilson Boring B-1, 8/2010

Boring Logs by Others

Boring 4-8E
Boring 4G-8E
Boring TB-1
Boring TB-2
Boring TH-1
Boring TH-2
Boring TH-3
Boring TH-7
Boring TH-8

APPENDIX F

PREVIOUS EXPLORATIONS

F.1 EXISTING INFORMATION

Soil borings were collected from reports in or near the potential alternative sites. All reports presented were found from the Shannon & Wilson archives or from GeoMapNW at the University of Washington. The information included in this appendix represents the borings that were most applicable in determining engineering and construction related issues at each location. The selected reports are shown below in reference to their Shannon & Wilson project number or GeoMapNW document ID number.

GeoMapNW ID No.	Shannon & Wilson Job No.	Document Name	Document Date	Document Author
	21-1-21144-001	Genesee and Henderson Combined Sewer Outfall (CSO) Reduction	8-10-09	Shannon & Wilson
3659		Pritchard/Seward Park CSO	8-1-84	Seattle
4004		Seward Park	7-31-69	Seattle
10047		Lake Washington Drainage	12-31-70	Seattle
10051		Unknown (Seattle Pacific University [SPU] Records Vault)	Unknown	CH2M Hill

F.2 REFERENCES

GeoMapNW, The Pacific Northwest Center for Geologic Mapping Studies, 2009, Online geodatabase: Seattle, Wash., GeoMapNW, available: http://geomapnw.ess.washington.edu/index.php?toc=maintoc&body=services/geodata/geodata.htm.

Total Depth:	40 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	~	Easting:		Drilling Company:	Holocene Drilling	Rod Diam.:	NWJ 2-5/8-inch
Vert. Datum:		Station:		Drill Rig Equipment:	Diedrich D-120	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 indicated below represent the approximate boundaries between material types, and the transition may be gradual.

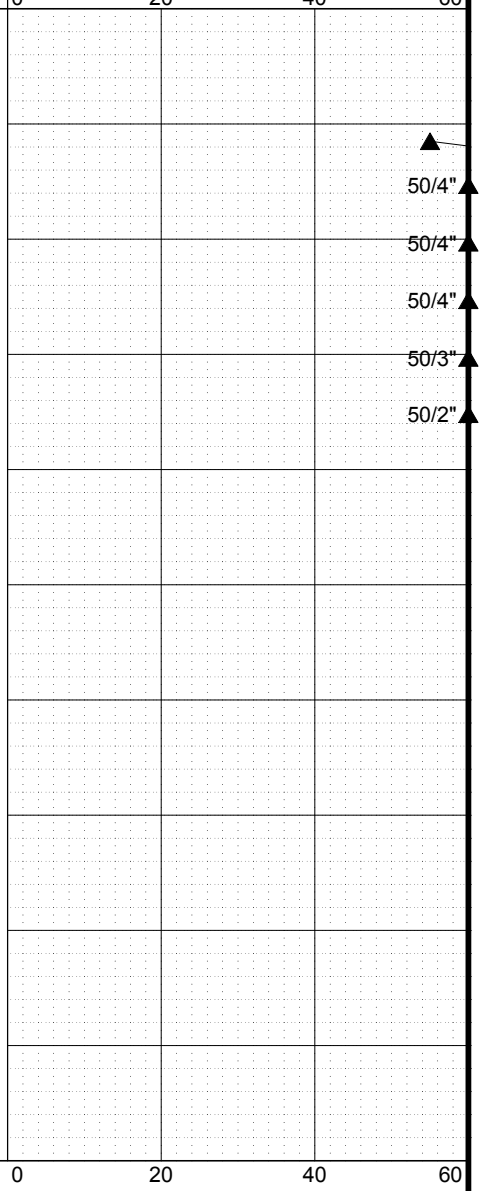
Depth, ft.	Description
1.0	ASPHALT PAVING.
4.5	Mottled, brown, orange-brown, brown, and gray, trace to slightly fine gravelly, silty, clayey SAND to sandy CLAY; moist; scattered to numerous iron-oxide-stained, highly weathered sandstone clasts, scattered gravelly, sandy clay clasts; (Fill) SC/CL.
7.0	Very dense, orange-brown and yellow-brown, silty, fine SAND, trace of clay; moist; fractured, scattered clay pockets, iron-oxide-stained; (Weathered Bedrock) SM.
	SANDSTONE: Very low strength, orange-brown and yellow-brown, fine-grained; very closely spaced, low to high angle joints with iron-oxide coatings; highly to completely weathered, iron-oxide-stained.
	SANDSTONE: Very low to low strength, gray and brown-black, fine-grained; steeply dipping bedding, scattered coarse sand to fine gravel-sized coal and rock clasts, scattered thin layers of gravelly sand, scattered black laminated zones; widely spaced low to high angle joints; slightly weathered to fresh.
40.9	BOTTOM OF BORING COMPLETED 8/27/2010

Samples: 1, 2, 3, 4, 5, 6, 7, 8, 9

Ground Water: During Drilling; Dry (12/14/2010)

PENETRATION RESISTANCE (blows/foot)

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



LEGEND

- * Sample Not Recovered
- Grab Sample
- Standard Penetration Test
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level ATD
- Ground Water Level in VWP

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.

Genesee - Henderson CSO Reduction Project
Henderson Basin Preliminary Engineering
Seattle, Washington

LOG OF BORING B-1

September 2011 | 21-1-21144-007

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F-1

REV 0

MASTER LOG E 21-21144.GPJ SHAN WIL.GDT 9/15/11 Log: CWA Rev: DPO Typ: LKN

The Pacific Northwest Center for Geologic Mapping Studies

Point Data for Point 56624:

Point ID	56624
Document ID	10047
Point Name	4-8E
Point Type	Boring
Northing	203824
Easting	1288383
Location Confidence	Less than 20 feet
Point Depth	14.5
Point Elevation	0.0
Elevation from DEM	63.0
Elevation Source	Not applicable
Datum name	Not applicable
Author name	Seattle Engineering Department
Point Date	1969-04-30 00:00:00
Boring Method	Unknown
Contractor	Unknown
Number of Wells	0

Document Information:

Document ID	10047 (PDF)
Document Type	Report
Source Name	Seattle Public Utilities - Records Vault
Author Name	Seattle Engineering Department
Document Name	Lake Washington Drainage
Document Date	1970-12-31 00:00:00
Project Type	Utility
Project Address	S McClellan - Ranier; Beacon Ave - Lake Wash Blvd
Local ID 1	05977-06069
Local ID 2	0

Layer Data for Point 56624:

Layer No.	Top Depth	Bottom Depth	Layer Description	USCS on Log
1	0.0	6.0	SILT, ML-CL, clay, trace gravel, med, moist, brn	CL-ML
2	6.0	11.0	SAND, SM, silt, gravel, compact, wet, brn	SM
3	11.0	14.5	SAND, SP-Sandstone, silt, very compact, gray	SP

Comment Data:

Comment No.	Depth	Comment Description
1	3.0	Water level 5-6-69
2	12.0	Harder drilling
3	17.0	Pulled out

The Pacific Northwest Center for Geologic Mapping Studies

Point Data for Point 57033:

Point ID	57033
Document ID	10047
Point Name	4G-8E
Point Type	Boring
Northing	203767
Easting	1288966
Location Confidence	Greater than 100 feet
Point Depth	30.0
Point Elevation	0.0
Elevation from DEM	29.0
Elevation Source	Not applicable
Datum name	Not applicable
Author name	Seattle Engineering Department
Point Date	1969-06-03 00:00:00
Boring Method	Unknown
Contractor	Unknown
Number of Wells	0

Document Information:

Document ID	10047 (PDF)
Document Type	Report
Source Name	Seattle Public Utilities - Records Vault
Author Name	Seattle Engineering Department
Document Name	Lake Washington Drainage
Document Date	1970-12-31 00:00:00
Project Type	Utility
Project Address	S McClellan - Ranier; Beacon Ave - Lake Wash Blvd
Local ID 1	05977-06069
Local ID 2	0

Layer Data for Point 57033:

Layer No.	Top Depth	Bottom Depth	Layer Description	USCS on Log
1	0.0	6.0	Fine SAND, SP, well graded, soft, wet, gray, brn	SP
2	6.0	13.0	Fine SAND, SM & PT, trace organ, very soft, wet, gray, brn; SAND, SM & PT, organic, very soft, wet, gray, brn	Non-standard
3	13.0	18.0	SAND, SILT, SM & OL, organic, soft, wet, gray, brn; CLAY, ML-CL, sand, silt, soft, wet, gray, brn, 44% passed #200	SM
4	18.0	21.0	SAND, SILT, SM, clay & trace peat, 35.1% passed #200, med, wet,	SM

			gray	
5	21.0	26.0	CLAY, SM, sand & silt, 42.7% passed #200, very compact, moist, gray	SM
6	26.0	30.0	SAND, SILT, SM-sandstone, clay, very compact, moist, gray	SM

Comment Data:

Comment No.	Depth	Comment Description
1	3.0	Water level 6-5-69

CS 7.241

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 7·2·84 HOLE NO. 1

PROJECT Pritchard/Seward CSO Contract 3 GRD. ELEV. +20

LOCATION S of Juneaus on Lake Wash. Blvd

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT	STD. PEN.	DESCRIPTION OF MATERIAL: COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
Loose to medium SILT & fine sandy SILT		1A	1 1	1	SANDY SILT w/trace clay	LOOSE	MOIST	GRAY	
	5								
		B	3 6 8	14	SILT disturbed	Medium	Moist to Dry	Gray	▽ 7/3/84
	10				Hard Drilling				
Very Compact weathered SILTSTONE w/sand layers		C	50/5"	50+	SILT w/siltstone	very comp	WET	GRAY	
	15								
		D	19 40 52/2"	50+	weathered SILTSTONE w/ fine sand layer	VERY COMP	MOIST w/ WET LAYERS	GRAY	PIEZO INSTALLED SAND IN TIP
BOTTOM 7.2.84	20								
	25								

10594

INSPECTOR J Ramm

CS 7.241

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 7·2·84 HOLE NO. 2

PROJECT PRITCHARD / SEWER PARK CSO CR3 GRD. ELEV. +20

LOCATION 85'S ℄ S. Juneau St on LAKE WA BLVD

STRATA	DEPTH	SAMPLE NO.	BLOW COUNT	STD. PEN.	DESCRIPTION OF MATERIAL: COMPOSITION	CONSISTENCY	MOISTURE	COLOR	WATER LEVEL
Loose to Firm fine sandy SILT		A	1 2	2	fine SAND-SILT w/trace clay in layer	LOOSE	WET	MOTTLED RUST-GRAY	
	5								▽ 7/3/84
		B	2 5 6	11	SAME	FIRM	MOIST	GRAY	
	10								
Hard Drilling		C	9 25 35	60	NO RECOVERY ON A ROCK				
	15				Hard Drilling				Open Hole
		D	5 5/6"	50+	NO RECOVERY ON A ROCK 2½" Gravel				
BOTTOM 7·2·84	20								

10594

INSPECTOR J Kamm

No Site Plan Provided WO B907 06150

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DAH (4004)

DATE 6-30-68 HOLE NO.

PROJECT SEWARD PARK WEATHER

LOCATION △#1 100' AHD on ℄

WATER TABLE

GRD. EL.	SAMPLE NO.	BLOW COUNT			STD. PEN.	DESCRIPTION OF MATERIAL: BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
3 JULY 68 ▽										
	A	1	1	1	2	SAND	TRACE SILT & ORGANIC	LOOSE	SATURATED	BRN.
5										
	B	1	2	1	3	12" SLI		SOFT	WET	BRN
	B.					60" SA			WET	GRAY
10										
			(12")							
	C		1	X	1+	SILT		VERY SOFT	WET	DARK BRN.
15										
	D	2	2	3	5	CLAY	SILT & SAND LAYERS	SOFT	WET	GRAY
20										
	E	2	4	8	12	SAND	SILT, TRACE GRAVEL	FIRM	SATURATED TO WET	GRAY
25										
	F	16	27	32		SILT	SAND & GRAVEL LAYERS	VERY STIFF	MOIST	GRAY
30										

Missing log for TH #10 location of TH #20?

INSPECTOR:

00150

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 6-30-69 HOLE NO. [redacted]

PROJECT SEWARD PARK WEATHER

LOCATION A #2 95' AHD. ON ℄

WATER TABLE

GRD. EL.	SAMPLE NO.	BLOW COUNT	STD. PEN.	DESCRIPTION OF MATERIAL: BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
		6"						
30JUN69 5	A	50 X X	50+	SILT	SILTSTONE SAND	VERY STIFF	DRY	GRAY
ADDED WATER								
		1½"						
10	B	50 X X	50+	SILT	SILTSTONE SAND	VERY STIFF	DRY	GRAY
PULLED OUT								
	C							
15								
	D							
20								
	E							
25								
	F							
30								

INSPECTOR:

06151

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 6-30-69 HOLE NO. [redacted]

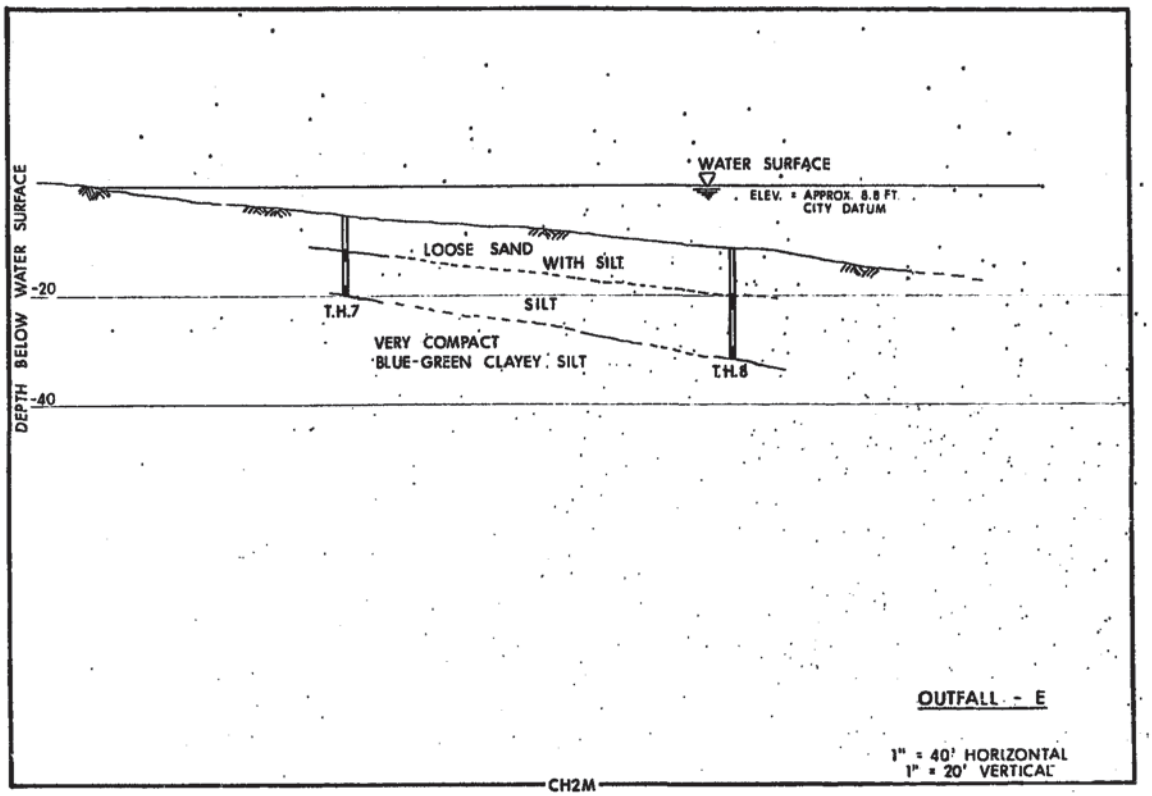
PROJECT SEWARD PARK WEATHER

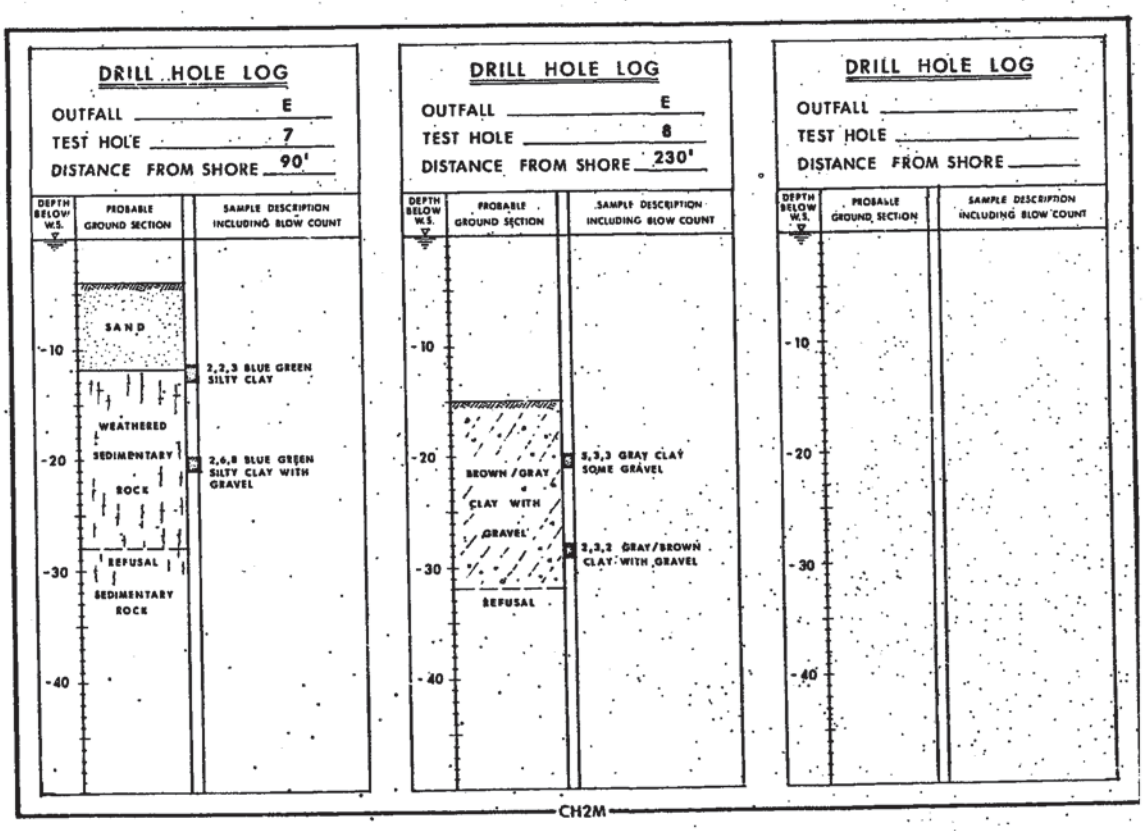
LOCATION 75' BK OF A #3

WATER TABLE

GRD. EL.	SAMPLE NO.	BLOW COUNT			STD. PEN.	DESCRIPTION OF MATERIAL: BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
3 July 69			4½"							
	A	41	50	X	50+	SILT	SILTSTONE TRACE SAND	VERY STIFF	DRY	GRAY
5										
ADDED WATER										
	B	13	21	22	43	SILT	SILTSTONE	STIFF	MOIST	GRAY
10										
		5"								
	C	50	X	X	50+	SILT	SILTSTONE	VERY STIFF	—	GRAY
DRILLING TIME 1 HOUR FOR 5' 15										
		3"								
	D	50	X	X	50+	NO	SAMPLE			
PULLED OUT 20										
	E									
25										

INSPECTOR:


WATER SURFACE
ELEV. = APPROX. 8.8 FT. CITY DATUM
LOOSE SAND WITH SILT
SILT
VERY COMPACT BLUE-GREEN CLAYEY SILT
T.H.7
T.H.8
DEPTH BELOW WATER SURFACE
-20
-40
OUTFALL - E
1" = 40' HORIZONTAL
1" = 20' VERTICAL
CH2M


DRILL HOLE LOG
OUTFALL E
TEST HOLE 7
DISTANCE FROM SHORE 90'
DEPTH BELOW W.S.
PROBABLE GROUND SECTION
SAMPLE DESCRIPTION INCLUDING BLOW COUNT
SAND
WEATHERED SEDIMENTARY ROCK
REFUSAL
SEDIMENTARY ROCK
2,2,3 BLUE GREEN SILTY CLAY
2,6,8 BLUE GREEN SILTY CLAY WITH GRAVEL
-10
-20
-30
-40
DRILL HOLE LOG
OUTFALL E
TEST HOLE 8
DISTANCE FROM SHORE 230'
BROWN/GRAY CLAY WITH GRAVEL
REFUSAL
3,3,3 GRAY CLAY SOME GRAVEL
3,3,2 GRAY/BROWN CLAY WITH GRAVEL
DRILL HOLE LOG
OUTFALL
TEST HOLE
DISTANCE FROM SHORE
CH2M

APPENDIX G

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-21144-007

Date: September 16, 2011
To: HDR Engineering, Inc.
Attn: Ms. Edith M. Hadler

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