APPENDIX B

RECORD OF NON-APPLICABILITY (RONA)

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RECORD OF NON-APPLICABILITY (RONA) FOR GENERAL CONFORMITY

NAME OF PROJECT:	Tumwater Readiness Center Construction and Operation, Thurston County, WA
PROJECT ID NUMBER:	MILCON #530129
POINT OF CONTACT:	LTC Adam Iwaszuk
PHONE/E-MAIL:	253-512-8702/Adam.M.Iwaszuk.mil@mail.mil
START DATE:	FY 2017

General Conformity under the *Clean Air Act, Section 1.76* has been evaluated for the project described above according to the requirements of *40 CFR 93 Subpart B*. The requirements of this rule are not applicable to this project/action because:

☐ The project/action qualifies as an exempt action. The applicable exemption citation is 40 CFR 93.153:

☒ Total direct and indirect emissions from this project/action have been estimated at (only include information for applicable pollutants):

1.7234 Tons per year of CO

0.11801 Tons per year of $PM_{2.5}$

These levels are below the conformity threshold values established at 40 CFR 93.153(b)--NO_x, $PM_{2.5}$, CO, SO_2: 100 tons per year, **AND** this project/action is not considered regionally significant under 40 CFR 93.153(i).

Supporting documentation and emission estimates are:

☒ Attached

☐ Appear in NEPA documentation -- ______________________(cite reference)

☐ Other ______________________ (cite reference)

THOMAS O. SKJERVOLD Thomas O. Skjervold Dec 11, 2015

ENVIRONMENTAL COORDINATOR (title and signature) DATE

Emissions Calculation for Tumwater Readiness Center Construction Project

Nonroad Equipment

Equipment	Equipment Population [a]	Operations (Days) [a]	CO Emission Factor (tons/day) [b]	$PM_{2.5}$ Emission Factor (tons/day)[b]	PM10 Emission Factor (tons/day)	CO Emissions (TPY)	$PM_{2.5}$ Emissions (TPY)	PM10 Emissions (TPY)
Paver	1	5	0.000346	0.000076	0.000078	0.001730	0.000380	0.000392
Rollers	1	40	0.000322	0.00007	0.000072	0.012880	0.002800	0.002887
Scrapers			0.000447	0.0001	0.000103	0.000000	0.000000	0.000000
Paving equipment			0.000286	0.00006	0.000062	0.000000	0.000000	0.000000
Surfacing equipment	1	25	0.000273	0.000054	0.000056	0.006825	0.001350	0.001392
Signal boards/ light plants	2	120	0.000052	0.00008	0.000082	0.012480	0.019200	0.019794
Trenchers			0.000288	0.000059	0.000061	0.000000	0.000000	0.000000
Bore/drill rigs			0.000179	0.000036	0.000037	0.000000	0.000000	0.000000
Cranes	1	40	0.000224	0.000055	0.000057	0.008960	0.002200	0.002268
Graders	1	25	0.000408	0.000092	0.000095	0.010200	0.002300	0.002371
Off-highway trucks			0.000787	0.000188	0.000194	0.000000	0.000000	0.000000
Tractors/ loaders/ backhoes	2	350	0.000413	0.000071	0.000073	0.289100	0.049700	0.051237
Crawler tractors/ dozers	1	40	0.000386	0.000087	0.000090	0.015440	0.003480	0.003588
Dumpers/ tenders			0.000179	0.000037	0.000038	0.000000	0.000000	0.000000
Other equipment	2	300	0.000297	0.000061	0.000063	0.178200	0.036600	0.037732
TOTAL						**0.535815**	**0.11801**	**0.12166**

[a]This number may change from project to project

[b]EPA Nonroad Emissions Model, Version 2005 1.0, June 2006

Onroad Vehicles							
Equipment	**Total Days[a]**	**Average trips/day[a]**	**Average miles/trip[a]**	**Total miles**	**CO Emission Factor (gram/mile)[b]**	**Conversion Factor (lb/gram)[b]**	**CO Emissions (TPY)**
Heavy duty diesel vehicle	300	11	30	99000	9.520000	0.0022	1.0367
Light duty diesel vehicle	500	6	30	90000	1.524000	0.0022	0.1509
Heavy duty gasoline vehicle				0	4.136000	0.0022	0.0000
						TOTAL	**1.1876**

[a]This number may change from project to project

[b]EPA Nonroad Emissions Model, Version 2005 1.0, June 2006

Total CO Emissions (TPY)	**1.7234**
Total PM2.5 Emissions (TPY)	**0.11801**
Total PM_{10} Emissions (TPY)	**0.121659794**

NOTE: These emissions estimates were based on a similar MILCON project (Pierce County Readiness Center) project. Calculations will be revised, if needed, when the TRC project design details are available and/or if there would be any changes in the regulatory requirement at the national and/or local level.

APPENDIX C

NOISE STUDY BY SSA ACOUSTICS, 2015

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Tumwater Readiness Center Project
Tumwater, WA

Noise Analysis

Prepared by:

SSA Acoustics, LLP
222 Etruria Street, Suite 100
Seattle, WA 98109
October 2015

Contact: William Stewart
(206) 839-0819

This document is provided to address necessary elements of the Environmental Assessment concerning noise for the construction of the new Tumwater Readiness Center. This report is completed to satisfy the requirements of 23 CFR 772 to be included with other relevant information associated to the project.

Sound is technically described in terms of amplitude (loudness) and frequency (pitch). The standard unit of sound amplitude measurement is the decibel (dB). The decibel scale is a logarithmic scale that describes the physical intensity of the pressure vibrations that make up any sound. The pitch of the sound is related to the frequency of the pressure vibration. Since the human ear is not equally sensitive to a given sound level at all frequencies, a special frequency-dependent rating scale has been devised to relate noise to human sensitivity. The A-weighted decibel scale (dBA) provides this compensation by discriminating against frequencies in a manner approximating the sensitivity of the human ear.

Noise, on the other hand, is typically defined as unwanted sound. A typical noise environment consists of a base of steady "background" noise that is the sum of many distant and indistinguishable noise sources. Superimposed on this background noise is the sound from individual local sources. These can vary from an occasional aircraft or train passing by to virtually continuous noise from, for example, traffic on a major highway.

Several rating scales have been developed to analyze the adverse effect of community noise on people. Since environmental noise fluctuates over time, these scales consider that the effect of noise upon people is largely dependent upon the total acoustical energy content of the noise, as well as the time of day when the noise occurs. Those that are applicable to this analysis are as follows:

- L_{eq} – An L_{eq}, or equivalent energy noise level, is the average acoustic energy content of noise for a stated period of time. Thus, the L_{eq} of a time-varying noise and that of a steady noise are the same if they deliver the same acoustic energy to the ear during exposure. For evaluating community impacts, this rating scale does not vary, regardless of whether the noise occurs during the day or the night.

- Equivalent Sound Level (L_{eq}) – The equivalent steady state sound level which in a stated period of time would contain the same acoustical energy. For the duration of time within this report 1-hour was the given period of time.

- L_{max} – The maximum instantaneous noise level experienced during a given period of time. For the duration of time within this report 1-hour was the given period of time.

- Day-Night Level (DNL) – The energy average of the A-weighted sound levels occurring during a 24-hour period, with 10 dB added to the A-weighted sound levels occurring during the period from 10:00 PM to 7:00 AM to account for the increased sensitivity of some individuals to noise levels during nighttime hours.

Noise environments and consequences of human activities are usually well represented by median noise levels during the day, night, or over a 24-hour period. For residential uses, environmental noise levels are generally considered low when the DNL is below 60 dBA, moderate in the 60–70 dBA range, and high above 70 dBA. Noise levels greater than 85 dBA can cause temporary or permanent hearing loss. Examples of low daytime levels are isolated, natural settings with noise levels as low as 20 dBA and quiet suburban residential streets with noise levels around 40 dBA. Noise levels above 45 dBA at night can disrupt sleep. Examples of moderate level noise environments are urban residential or semi-commercial areas (typically 55–60 dBA) and commercial locations (typically 60 dBA). People may consider louder environments adverse, but most will accept the higher levels associated with more noisy urban residential or residential-commercial areas (60–75 dBA) or dense urban or industrial areas (65–80 dBA).

Noise levels from a particular source generally decline as distance to the receptor increases. Other factors, such as the weather and reflecting or barriers, also help intensify or reduce the noise level at any given location. A commonly used rule of thumb for roadway noise is that for every doubling of distance from the source, the noise level is reduced by about 3 dBA at acoustically "hard" locations (i.e., the area between the noise source and the receptor is nearly complete asphalt, concrete, hard-packed soil, or other solid materials) and 4.5 dBA at acoustically "soft" locations (i.e., the area between the source and receptor is normal earth or has vegetation, including grass). Noise from stationary or point sources is reduced by about 6 to 7.5 dBA for every doubling of distance at acoustically hard and soft locations, respectively. Noise levels are also generally reduced by 1 dBA for each 1,000 feet of distance due to air absorption. Noise levels may also be reduced by intervening barriers designed to control noise. Based on their height, distance to the source, and distance to the receiver, as much as 18 dB(A) can be reduced from a particular source.

Impact Criteria

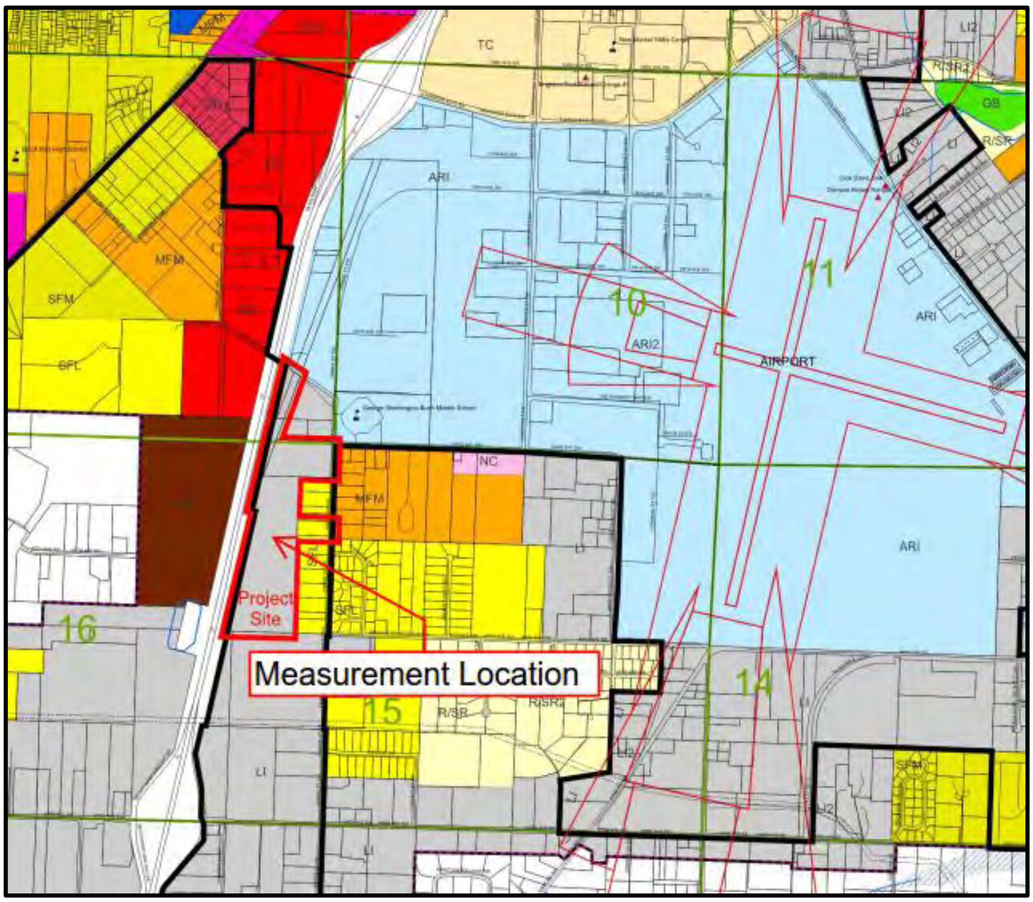
The Federal Transit Administration established impact criteria with "Transit Noise and Vibration Impact Assessment Guide", FTA-VA-90-1003-06. May 2006. This guide establishes activity categories for land usage with applicable maximum noise levels for development without mitigation. These levels are provided in Table 1.

Table 1. GHWA Noise Abatement Criteria

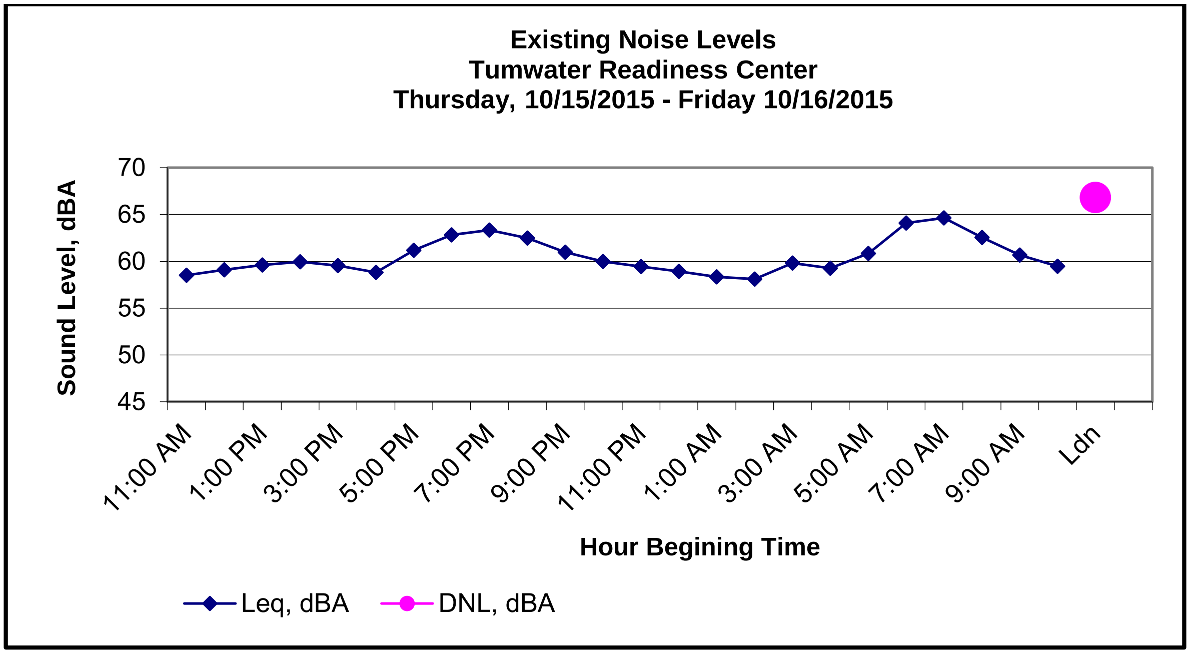
Activity Category	Hourly A-weighted Sound Level	Description of Activity Category
A	57 Exterior	Lands on which serenity and quiet are of extraordinary significance and serve an important public need and where the preservation of those qualities is essential if the area is to continue to serve its intended purposes.
B	67 Exterior	Picnic areas, recreation areas, playgrounds, active sports areas, parks, residences, motels, hotels, schools, churches, libraries, and hospitals.
C	72 Exterior	Developed lands, properties, or activities not included in Categories A or B above.
D	----------	Undeveloped lands.
E	52 Interior	Residences, motels, hotels, public meeting rooms, schools, churches, libraries, hospitals, and auditoriums.

Existing Noise Conditions:

In accordance with Washington Administrative Code 173-60, screening criteria, field measurements involving the use of noise meters at the site, was used to evaluate the ambient environmental noise. The site is located along Kimmie Street due west of the Olympia Regional Airport within Thurston County. It is bounded by Frontage Road to the north, Kimmie Street and a number of residential properties to the east, undeveloped land to the south, and Interstate 5 to the west. Noise to the site is dominated by traffic along Interstate 5 and to a lesser extent local traffic along Kimmie Street and aircraft from Olympia Reginal Airport. Sound level measurements were made to document the ambient noise levels at the site by monitoring conditions from a location setback to parcel 51850001200 from Kimmie Street shown in the map below. This location represented an average of the noise levels from both the east and west sources of the property. Aircraft exposure to the site was secondary to traffic noise generated by Interstate 5.

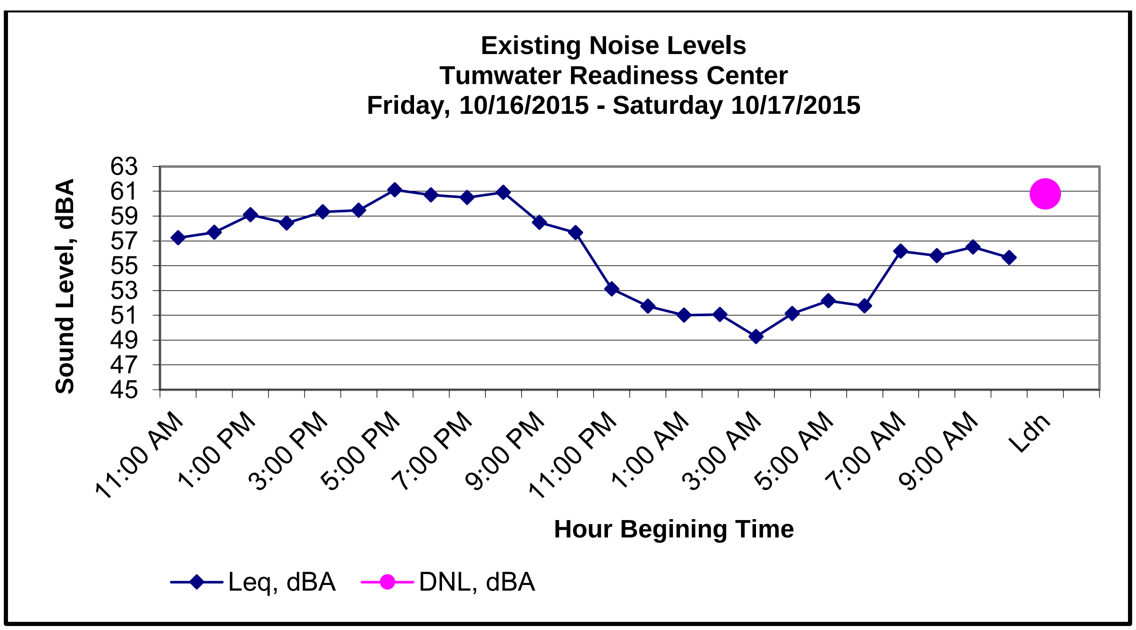


The results of these measurements are shown in Graphs 1, 2 and 3 below. The period selected was from Thursday, October 15th through Sunday, October 18. This period represents the most active portion of the week for a readiness center conducting training and drills around weekends.



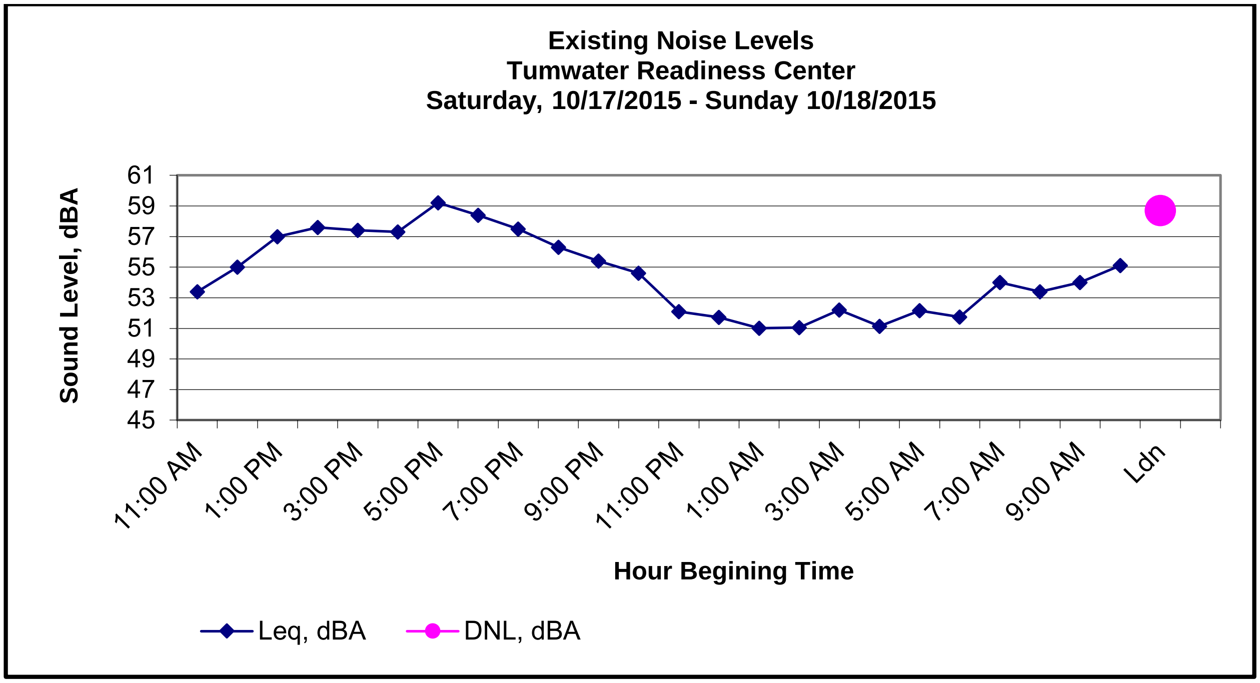
Graph 1: Hourly L_{eq} values for weekday site noise.

Graph 1 shows the hourly L_{eq} values that were recorded between noon October 15, 2015 and 10 am October 16, 2015. The hourly sound levels at the site are driven primarily by the traffic along Interstate 5. Noise levels on an hourly average do not exceed 65 dB(A) on a typical weekday. The Leq hourly averages were between 53.8 dB(A) and 64.7 dB(A). The DNL or L_{dn}, is a descriptor to evaluate noise to sensitive occupancies for auto, rail, and aircraft noise. Established by the Federal government, it provides a single value based on a 24-hour period and a correction for nighttime sleeping periods. The purple dot in the upper right of the graph provides this value and can be compared to the tables presented earlier in this report.



Graph 2: Hourly L_{eq} values for noise on Saturday.

Graph 2 shows the hourly L_{eq} values that were recorded between noon October 16, 2015 and 10 am October 17, 2015. The hourly sound levels at the site are driven primarily by the traffic along Interstate 5. Noise levels on an hourly average do not exceed 65 dB(A) on a typical Saturday. The Leq hourly averages were between 49.2 dB(A) and 61.1 dB(A).



Graph 3: Hourly Leq values for noise on Sunday.

Graph 3 shows the hourly L_{eq} values that were recorded between noon October 17, 2015 and 10 am October 18, 2015. The hourly sound levels at the site are driven primarily by the traffic along Interstate 5. Noise levels on an hourly average do not exceed 65 dB(A) on a typical Saturday. The Leq hourly averages were between 51.1 dB(A) and 59.0 dB(A).

Environmental Assessment of Noise Impact:

Based on field measurements taken at the site, the use of this property for a readiness center is well within the guidelines for land use category B which limits exterior noise to 67 dBA. Standard construction practices for building envelope can be utilized to achieve acceptable interior noise environments.

Construction Noise:

Construction noise is limited at the site by standards set forth in the Thurston County Municipal Code, Chapter 10.36.040 providing an exemption for sounds created by temporary construction sites as a result of construction activities between 7:00 A.M. and 10:00 P.M.

Operational Noise

Stationary Noise Sources

New stationary sources of noise, such as rooftop mechanical HVAC equipment would be installed at the proposed Readiness Center at the project site. The design of this equipment would be required to comply with Washington Administrative Code 173-60-040 Maximum Permissible Environmental Noise Levels, which prohibits noise from air conditioning, refrigeration, heating, pumping, and filtering equipment from exceeding the levels established by the level acceptable to the site usage EDNA A, Residential Zone, EDNA B Commercial Zone, and EDNA C Industrial Zone. These limits are set forth in the following table:

EDNA OF NOISE SOURCE	EDNA OF RECEIVING PROPERTY		
	Class A	Class B	Class C
CLASS A	55 dBA	57 dBA	60 dBA
CLASS B	57	60	65
CLASS C	60	65	70

In addition, between the hours of 10:00 p.m. and 7:00 a.m. the noise limitations of the foregoing table shall be reduced by 10 dBA for receiving property within Class A EDNAs. At any hour of the day or night the applicable noise limitations above may be exceeded for any receiving property by no more than:

(i) 5 dBA for a total of 15 minutes in any one-hour period; or

(ii) 10 dBA for a total of 5 minutes in any one-hour period; or

(iii) 15 dBA for a total of 1.5 minutes in any one-hour period.

Based on the planned setback of the structure from George Bush Middle School and the residential receivers to the east of the site along Kimmie St SW, no impact from HVAC equipment on the site is anticipated.

Further Study Required:

No further analysis is required regarding noise.

APPENDIX D-1

SOILS REPORT FOR STORM DRAINAGE PURPOSES, KIMMIE STREET INDUSTRIAL PARK, PARNELL ENGINEERING, 2008

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Kimmie Street Industrial Park

Soils Report For Storm Drainage Purposes

Site Address: Kimmie Street SW, Olympia WA 98512
TPN: 09520004000, 09520003000, 09230006000, 09230019000, 51850001200, 51850000400

Prepared For: Shea, Carr & Jewell, Inc
2102 Carriage Drive SW, Building H
Olympia, WA 98502
(360) 352-1465
Contact: Amy Head, P.E.

Prepared By: Parnell Engineering, LLC
10623 Hunters Lane S.E.
Olympia, WA 98513
(360) 491-3243
Contact: William Parnell, P.E.

PE
PARNELL ENGINEERING, LLC

SOIL EVALUATION REPORT
FORM 1: GENERAL SITE INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park — SHEET: 1 OF 2
PE PROJECT NO.:08117 — DATE: 6/13/08
PREPARED BY: William Parnell, P.E.

1. SITE ADDRESS: Kimmie Street SW, Olympia WA 98512

   TPN: 09520004000, 09520003000, 09230006000, 09230019000,51850001200,51850000400

2. PROJECT DESCRIPTION: Create an eleven lot commercial/ industrial development.

3. SITE DESCRIPTION: The project site is currently occupied by two metal buildings and one collapsed structure located on the northern portion of the site. Current access is located west of the Kimmie Street SW and 83rd Avenue SW intersection. Site topography is nearly level with elevations ranging from 190 ft. to 196 ft. Site vegetation consists of a conifer and deciduous forest of moderate density with indigenous brush ground cover on 60% of the project site. The center of the site has been previously logged and is now vegetated with indigenous brush and scotch broom. Site distinguishing features include a wetland located on the southern end of the site. The project site is bounded by Interstate 5 to the west, partially developed property to the north and east and undeveloped property to the south. The on-site soils are mapped by the NRCS as a combination of Cagey loamy sand, Everett very gravelly sandy loam and Norma silt loam. The Cagey soils are mapped on the central portion of the site. These soils are generally deep, moderately well drained and formed in sandy glacial drift. The Everett soils are mapped on the northern and southwestern portion of the site. These soils are generally a very deep, somewhat excessively drained and formed in glacial outwash. The Norma soils are mapped as a 300' wide band crossing diagonally across the southwestern portion of the site. These soils are generally very deep, poorly drained and formed in alluvium.

4. SUMMARY OF SOILS WORK PERFORMED: Six test pits were excavated by trackhoe to a maximum depth of 192" below the existing grade. Soils were inspected by entering and visually logging each test pit to a depth of four feet. Soils beyond four feet were inspected by examining backhoe tailings. Six double ring infiltration tests were completed. Test pit soil log data sheets and infiltration test results are included in this report.

5. ADDITIONAL SOILS WORK RECOMMENDED: Additional soils work may be necessary when the final site plan design is completed and the storm drainage infiltration facilities locations are determined.

6. FINDINGS: The Natural Resource Conservation Service soil survey for Thurston County mapped the majority of the on-site soils as a Cagey loamy sand (20) with some Norma (76) and Everett (32) series soils included.

Test pits #1 and #2 were mapped as an Everett series but test pit profiles revealed soils consistent with an Indianola soil overlying a deeper Everett series soil; loamy sand stratum overlying an extremely gravelly sand substratum. Test pit #1 presented a water table at 117" with possible winter water table indicators at 62"- 65" below the existing grade. Test pit #2 presented water table at 68" with possible winter water table indicators at 24". Double ring Infiltration tests completed at 12" below the existing grade in test pit #1 and at 6" in test pit #2 yielded infiltration rates of 23.6 in/hr and 8.18 in/hr respectively.

Test pits #3, #4 and #5 revealed profiles consistent with a Cagey series; loamy sand stratum overlying a sand to loamy sand substratum. The Cagey series is generally associated with an apparent winter water table that resides in the upper horizons of the soil profile. Test pit #3 presented a water table at 180" below the existing grade with possible winter water table indicators at 30"+. A double ring infiltration test completed at 9" below the existing grade yielded an infiltration rate of 16.36 in/hr. Test pit #4 presented a water table at 168" below the existing grade with possible winter water table indicators at 58"+. A double ring infiltration test completed at 9" below the existing grade yielded an infiltration rate of 9.73 in/hr. Test pit #5 did not present a water table but soils were wet at 101" and close to saturation at 180" below the existing grade with possible winter water table indicators at 48"+. A double ring infiltration test completed at 12" below the existing grade yielded an infiltration rate of 13.33 in/hr.

Test pit #6 revealed a profile consistent with a Norma series; sandy loam stratum overlying a sandy loam/loamy sand substratum. The Norma series is generally associated with an apparent winter water table that resides in the upper horizons of the soil profile. The test pit did not exhibit a water table but possible winter water table indicators were present at 28"+. A double ring infiltration test completed at 9" below the existing grade yielded an infiltration rate of 5.3 in/hr.

7. RECOMMENDATIONS: The Indianola/Everett soil series are somewhat excessively drained soils formed in sandy glacial drift/outwash. The Cagey soil series is a moderately well drained soil that formed in sandy glacial drift. Infiltration rates are generally rapid where unimpeded by a water table presence. All soil test pits revealed water table indicators indicating the project site may have a winter water table that resides at a fairly shallow depth. All infiltration testing was completed in the upper horizon soils no deeper than 12" below the existing grade. Infiltration rates varied from a low of 5.3 in/hr to a high of 23.6 in/hr. An average of all double ring infiltration tests completed resulted in a free drainage infiltration rate for the entire site of 12.75 in/hr. For design purposes, use an overall site infiltration rate of 5.0 in/hr or less as the free drainage rate for preconstruction, undisturbed and uncompacted conditions, non-influenced by winter water table mounding. It is recommended that infiltration tests be completed in all proposed stormwater drainage infiltration facilities once final design locations are determined so a more accurate site specific infiltration rate can be determined for each facility. Winter water table must be a consideration when evaluating the suitability of the on-site soils for drainage infiltration facilities.

During construction, care must be taken to prevent the erosion of exposed soils. Drainage facility infiltration surfaces must be properly protected from contamination by fine-grained soils and from compaction by construction site activities. Soils not properly protected will cause drainage infiltration facilities to prematurely fail.

I hereby certify that I prepared this report, and conducted or supervised the performance of related work. I certify that I am qualified to do this work. I represent my work to be complete an accurate within the bounds of uncertainty inherent to the practice of soils science, and to be suitable for its intended use.

SIGNED: William Purnell

DATE: 6/13/08

WILLIAM W. PURNELL
STATE OF WASHINGTON
26152
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 6-8-09

SOIL EVALUATION REPORT
FORM 2: SOIL LOG INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park PE PROJECT NO.:08117 PREPARED BY: William Parnell, P.E.		SHEET: 1 OF 6 DATE: 6/4/08		
SOIL LOG: #1 LOCATION: 80 ft. south and 230 ft. west of the northeast property corner at 83rd Ave SW.				
1. TYPES OF TEST DONE: Double Ring Infiltration Test	2. NRCS SOILS SERIES: Indianola/Everett	3. LAND FORM: Terrace		
4. DEPOSITION HISTORY: Glacial Drift/Outwash	5. HYDROLOGIC SOIL GROUP: A	6. DEPTH OF SEASONAL HW: Unknown		
7. CURRENT WATER DEPTH: 117"	8. DEPTH TO IMPERVIOUS LAYER: 62"-65"	9. MISCELLANEOUS: Level		
10. POTENTIAL FOR:		EROSION	RUNOFF	PONDING
		Minimal	Slow	Minimal
11. SOIL STRATA DESCRIPTION: See Following chart				
12. SITE PERCOLATION RATE: See FSP				
13. FINDINGS & RECOMMENDATIONS: The C4 horizon was moderately dense. The C5 horizon was very dense and compacted with faint streaky mottles. A double ring infiltration test completed at 12" below the existing grade yielded an infiltration rate of 23.6 in/hr.				

Soils Strata Description
Soil Log #1

Horz	Depth	Color	Texture	%CL	%ORG	CF	STR	MOT	IND	CEM	ROO	<X>	FSP
BC	0"- 9"	10YR3/3	LmFSa	<10	<1	<1	1SBK	-	-	-	ff	6-20	4
C1	9"- 17"	10YR4/3	LmMSa	<8	-	<1	1SBK	-	-	-	ff	6-20	23.6
C2	17"- 46"	10YR5/2	MSa	<2	-	<5	SG	-	-	-	-	>20	>20
C3	46"- 60"	10YR5/1	C-MSa	<1	-	<10	SG	-	-	-	-	>20	-
C4	60"- 62"	10YR5/2	LmC-MSa	<4	-	<10	SG	-	-	-	-	6-20	-
C5	62"- 65"	10YR5/3	ExGrSa	<2	-	<90	Mas	F1F	Mod	-	-	-	-
C6	65"-108"	10YR5/1	ExGrCSa	<1	-	<85	SG	-	-	-	-	>20	-
C7	108"-132"	10YR5/1	VGrCSa	<1	-	<75	SG	-	-	-	-	>20	-
C8	132"-156"	10YR5/1	ExGrSa	<1	-	<80	SG	-	-	-	-	>20	-

SOIL EVALUATION REPORT
FORM 2: SOIL LOG INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park
PE PROJECT NO.:08117
PREPARED BY: William Parnell, P.E.

SHEET: 2 OF 6
DATE: 6/4/08

SOIL LOG: #2
LOCATION: 230 ft. south and 690 ft. west of the northeast property corner at 83rd Ave SW.

1. TYPES OF TEST DONE: Double Ring Infiltration Test	2. NRCS SOILS SERIES: Indianola/Everett	3. LAND FORM: Terrace		
4. DEPOSITION HISTORY: Glacial Drift/Outwash	5. HYDROLOGIC SOIL GROUP: A	6. DEPTH OF SEASONAL HW: Unknown		
7. CURRENT WATER DEPTH: 68"	8. DEPTH TO IMPERVIOUS LAYER: Greater Than Bottom of Hole	9. MISCELLANEOUS: Level		
10. POTENTIAL FOR:		EROSION	RUNOFF	PONDING
		Minimal	Slow	Minimal

11. SOIL STRATA DESCRIPTION: See Following chart

12. SITE PERCOLATION RATE: See FSP

13. FINDINGS & RECOMMENDATIONS: The C3 horizon had pockets of moderate manganese staining and heavy mottles. The C4 horizon had pockets of heavy mottling. The C5 horizon was moderately dense and compacted. A double ring infiltration test completed at 6" below the existing grade yielded an infiltration rate of 8.18 in/hr.

Soils Strata Description
Soil Log #2

Horz	Depth	Color	Texture	%CL	%ORG	CF	STR	MOT	IND	CEM	ROO	<X>	FSP
BC	0"- 3"	10YR3/3	LmFSa	<10	<1	<1	1SBK	-	-	-	ff	6-20	4
C1	3"- 10"	10YR4/3	LmMSa	<8	-	<1	1SBK	-	-	-	ff	6-20	8.18
C2	10"- 24"	10YR5/2	MSa	<2	-	<5	SG	-	-	-	-	>20	-
C3	24"- 33"	10YR5/2	ExGrCSa	<1	-	<90	SG	F3P	-	-	-	>20	-
C4	33"- 60"	10YR5/1	ExGrCSa	<1	-	<90	SG	C3P	-	-	-	>20	-
C5	60"- 72"	10YR5/2	ExGrSa	<2	-	<70	SG	-	-	-	-	>20	-
C6	72"-120"	10YR5/1	ExGrSa	<1	-	<80	SG	-	-	-	-	>20	-

SOIL EVALUATION REPORT
FORM 2: SOIL LOG INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park PE PROJECT NO.:08117 PREPARED BY: William Parnell, P.E.		SHEET: 3 OF 6 DATE: 6/4/08		
SOIL LOG: #3 LOCATION: 1170 ft. south and 710 ft. west of the northeast property corner at 83rd Ave SW.				
1. TYPES OF TEST DONE: Double Ring Infiltration Test	2. NRCS SOILS SERIES: Cagey (20)	3. LAND FORM: Terrace		
4. DEPOSITION HISTORY: Sandy Glacial Drift	5. HYDROLOGIC SOIL GROUP: C	6. DEPTH OF SEASONAL HW: Unknown		
7. CURRENT WATER DEPTH: 180"	8. DEPTH TO IMPERVIOUS LAYER: Greater than bottom of hole	9. MISCELLANEOUS: Nearly Level		
10. POTENTIAL FOR:		EROSION	RUNOFF	PONDING
		Minimal	Slow	Minimal
11. SOIL STRATA DESCRIPTION: See Following chart				
12. SITE PERCOLATION RATE: See FSP				
13. FINDINGS & RECOMMENDATIONS: The C1 horizon had streaky mottles at 30" transitioning to moderate mottles at 40" below the existing grade. Roots stopped at 30". Soils were wet at 102" and saturated at 180" below the existing grade. A double ring infiltration test completed at 9" below the existing grade yielded an infiltration rate of 16.36 in/hr.				

Soils Strata Description
Soil Log #3

Horz	Depth	Color	Texture	%CL	%ORG	CF	STR	MOT	IND	CEM	ROO	<X>	FSP
A	0"- 3"	10YR3/2	LmVFSa	<12	<5	<1	1SBK	-	-	-	fm	6-20	2
Bw	3"- 26"	10YR4/3	LmVFSa	<10	<3	<1	1SBK	-	-	-	fm	6-20	16.36
C1	26"- 42"	10YR5/3	LmVFSa	<10	<1	<1	1SBK	-	-	-	fm	6-20	-
C2	42"-102"	2.5Y5/2	LmVFSa	<10	-	<1	1SBK	F2D	-	-	-	6-20	-
C3	102"-126"	2.5Y4/2	SiLm	<25	-	<1	Mas	-	-	-	-	.6-2.0	-
C4	126"-180"	2.5Y4/1	FSa	<8	-	<1	SG	-	-	-	-	6-20	-

SOIL EVALUATION REPORT
FORM 2: SOIL LOG INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park PE PROJECT NO.:08117 PREPARED BY: William Parnell, P.E.		SHEET: 4 OF 6 DATE: 6/4/08
SOIL LOG: #4 LOCATION: 1350 ft. south and 200 ft. west of the northeast property corner at 83rd Ave SW.		
1. TYPES OF TEST DONE: Double Ring Infiltration Test	2. NRCS SOILS SERIES: Cagey (20)	3. LAND FORM: Terrace
4. DEPOSITION HISTORY: Sandy Glacial Drift	5. HYDROLOGIC SOIL GROUP: C	6. DEPTH OF SEASONAL HW: Unknown
7. CURRENT WATER DEPTH: 168"	8. DEPTH TO IMPERVIOUS LAYER: Greater than bottom of hole	9. MISCELLANEOUS: Nearly Level
10. POTENTIAL FOR:		EROSION: Minimal / RUNOFF: Slow / PONDING: Minimal
11. SOIL STRATA DESCRIPTION: See Following chart		
12. SITE PERCOLATION RATE: See FSP		
13. FINDINGS & RECOMMENDATIONS: The C3 and C4 horizons were moderately mottled. Roots stopped at 60". A double ring infiltration test completed at 9" below the existing grade yielded an infiltration rate of 9.73 in/hr.		

Soils Strata Description
Soil Log #4

Horz	Depth	Color	Texture	%CL	%ORG	CF	STR	MOT	IND	CEM	ROO	<X>	FSP
A	0"- 8"	10YR3/2	LmVFSa	<12	<5	<1	1SBK	-	-	-	fm	6-20	2
Bw	8"- 20"	10YR4/3	LmFSa	<8	<3	<1	1SBK	-	-	-	fm	6-20	9.73
C1	20"- 36"	10YR5/2	LmFSa	<8	<1	<1	1SBK	-	-	-	fm	6-20	-
C2	36"- 58"	2.5Y5/2	FSa	<6	<1	<1	1SBK	-	-	-	fm	6-20	-
C3	58"- 64"	10YR5/2	SaLm	<20	-	<1	Mas	C2D	-	-	-	2-6	-
C4	64"- 96"	2.5Y5/3	LmVFSa	<12	-	<1	1SBK	F2D	-	-	-	6-20	-
C5	96"-144"	2.5Y5/1	FSa	<2	-	<1	SG	-	-	-	-	6-20	-
C6	144"-156"	2.5Y5/2	LmFSa	<8	-	<1	SG	-	-	-	-	6-20	-
C7	156"-180"	10YR5/1	ExGrSa	<3	-	<75	SG	-	-	-	-	>20	-

SOIL EVALUATION REPORT
FORM 2: SOIL LOG INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park PE PROJECT NO.:08117 PREPARED BY: William Parnell, P.E.		SHEET: 5 OF 6 DATE: 6/4/08
SOIL LOG: #5 LOCATION: 1730 ft. south and 800 ft. west of the northeast property corner at 83rd Ave SW.		
1. TYPES OF TEST DONE: Double Ring Infiltration Test	2. NRCS SOILS SERIES: Cagey (20)	3. LAND FORM: Terrace
4. DEPOSITION HISTORY: Sandy Glacial Drift	5. HYDROLOGIC SOIL GROUP: C	6. DEPTH OF SEASONAL HW: Unknown
7. CURRENT WATER DEPTH: 168"	8. DEPTH TO IMPERVIOUS LAYER: Greater than bottom of hole	9. MISCELLANEOUS: Nearly Level
10. POTENTIAL FOR:		EROSION: Minimal RUNOFF: Slow PONDING: Minimal
11. SOIL STRATA DESCRIPTION: See Following chart		
12. SITE PERCOLATION RATE: See FSP		
13. FINDINGS & RECOMMENDATIONS: Spotty mottles were present at 48" transitioning to heavy mottles at 57"-84" below the existing grade. Soils were wet at 101". A double ring infiltration test completed at 12" below the existing grade yielded an infiltration rate of 13.33 in/hr.		

Soils Strata Description
Soil Log #5

Horz	Depth	Color	Texture	%CL	%ORG	CF	STR	MOT	IND	CEM	ROO	<X>	FSP
A	0"- 2"	10YR3/2	LmVFSa	<12	<5	<1	1SBK	-	-	-	ff	6-20	2
Bw	2"- 10"	10YR4/3	LmVFSa	<10	<3	<1	1SBK	-	-	-	ff	6-20	6
C1	10"- 24"	10YR3/6	LmVFSa	<8	<1	<1	1SBK	-	-	-	ff	6-20	13.33
C2	24"- 57"	10YR5/3	LmVFSa	<8	-	<1	1SBK	F1F	-	-	-	6-20	-
C3	57"- 68"	10YR5/6	Lm	<20	-	<1	Mas	M3P	-	-	-	2-6	-
C4	68"- 84"	2.5Y4/3	LmVFSa	<10	-	<1	1SBK	M3P	-	-	-	6-20	-
C5	84"-101"	2.5Y4/4	Lm	<20	-	<1	Mas	F1F	-	-	-	2-6	-
C6	101"-192"	2.5Y4/1	F-VFSa	<6	-	<1	SG	-	-	-	-	6-20	-

SOIL EVALUATION REPORT
FORM 2: SOIL LOG INFORMATION

PROJECT TITLE: Kimmie Street Industrial Park PE PROJECT NO.:08117 PREPARED BY: William Parnell, P.E.		SHEET: 6 OF 6 DATE: 6/4/08
SOIL LOG: #6 LOCATION: 2170 ft. south and 850 ft. west of the northeast property corner at 83rd Ave SW.		
1. TYPES OF TEST DONE: Double Ring Infiltration Test	2. NRCS SOILS SERIES: Norma (76)	3. LAND FORM: Till Plain
4. DEPOSITION HISTORY: Alluvium	5. HYDROLOGIC SOIL GROUP: D	6. DEPTH OF SEASONAL HW: Unknown
7. CURRENT WATER DEPTH: Greater Than Bottom of Hole	8. DEPTH TO IMPERVIOUS LAYER: Greater than bottom of hole	9. MISCELLANEOUS: Nearly Level
10. POTENTIAL FOR:		EROSION: Minimal RUNOFF: Slow PONDING: Minimal
11. SOIL STRATA DESCRIPTION: See Following chart		
12. SITE PERCOLATION RATE: See FSP		
13. FINDINGS & RECOMMENDATIONS: Spotty mottles were present at 28" transitioning to heavy mottles at 36" below the existing grade. Medium roots were present to 24". A double ring infiltration test completed at 9" below the existing grade yielded an infiltration rate of 5.3 in/hr.		

Soils Strata Description
Soil Log #6

Horz	Depth	Color	Texture	%CL	%ORG	CF	STR	MOT	IND	CEM	ROO	<X>	FSP
A	0"- 12"	10YR3/2	SaLm	<20	<5	<1	2SBK	-	-	-	fm	6-20	5.3
Bw	12"- 22"	10YR4/3	LmVFSa	<10	<3	<1	1SBK	-	-	-	mm	6-20	-
C1	22"- 70"	10YR3/3	LmVFSa	<8	<1	<1	1SBK	M3P	-	-	fm	6-20	-
C2	70"- 88"	10YR5/1	VFSa	<4	-	<1	1SBK	F1F	-	-	-	6-20	-
C3	88"-168"	10YR5/2	LmVFSa	<8	-	<1	Mas	M3P	-	-	-	6-20	-

Abbreviations

Textural Class (Texture)	Structure (STR)	Grades of Structure
Cobbley -Cob	Granular - Gr	Strong - 3
Stoney - St	Blocky - Blky	Moderate - 2
Gravelly - Gr	Platy - Pl	Weak - 1
Sandy - Sa	Massive - Mas	
Loamy - Lm	Single Grained - SG	
Silty - Si	Sub-Angular Blocky - SBK	
Clayey - Cl		
Coarse - C		
Very - V		
Extremely - Ex		
Fine - F		
Medium - M		

Induration & Cementation (IND) (CEM)
Weak - Wk
Moderate - Mod
Strong - Str

Mottles (MOT)		
1 Letter Abundance	1st Number Size	2nd Letter Contrast
Few - F	Fine - 1	Faint - F
Common - C	Medium - 2	Distinct - D
Many - M	Coarse - 3	Prominent - P

Roots (ROO)	
1st Letter Abundance	2nd Letter Size
Few - f	Fine - f
Common - c	Medium - m
Many - m	Coarse - c

<X> - Generalized range of infiltration rates from SCS soil survey (<X>)
FSP - Estimated **F**ield **S**aturated **P**ercolation rate based on horizon specific factors.

DOUBLE RING INFILTRATION TESTS
Kimmie Road Industrial Park
Completed By : William Parnell, P.E.
PE Job : #08117

Test Pit # **1** (completed @ 12" below existing ground surface)
Location: 80' south & 230' west of the northeast property corner at 83rd Ave SW
Test Date: 6/4/08

Start (H: M' S")	Stop (H: M' S")	Elapsed Time (H: M' S")	Total Drop (Inches)	Infiltration Rate (In/Hr)
0: 00' 00"	0: 10' 00"	0: 10' 00"	6	
0: 11' 00"	0: 21' 30"	0: 10' 30"	6	
0: 24' 00"	0: 35' 30"	0: 11' 30"	6	
0: 37' 30"	0: 49' 30"	0: 12' 00"	6	
0: 52' 00"	1: 03' 00"	0: 11' 00"	6	
1: 08' 00"	1: 21' 00"	0: 13' 00"	6	
1: 23' 00"	1: 37' 00"	0: 14' 00"	6	
1: 38' 00"	1: 52' 00"	0: 14' 00"	6	
1: 54' 00"	2: 07' 30"	0: 13' 30"	6	
		Soaking Period		
2: 30' 00"	2: 43' 00"	0: 13' 00"	6	
2: 48' 00"	3: 02' 15"	0: 14' 15"	6	
3: 04' 00"	4: 19' 15"	0: 15' 15"	6	
3: 21' 00"	3: 36' 00"	0: 15' 00"	6	
3: 38' 00"	3: 53' 15"	0: 15' 15"	6	
3: 55' 00"	4: 10' 15"	0: 15' 15"	6	**23.6**

Test Pit # **2** (completed @ 6" below existing ground surface)
Location: 230' south & 690' west of the northeast property corner at 83rd Ave SW
Test Date: 6/4/08

Start (H: M' S")	Stop (H: M' S")	Elapsed Time (H: M' S")	Total Drop (Inches)	Infiltration Rate (In/Hr)
0: 00' 00"	0: 42' 00"	0: 42' 00"	6	
0: 44' 00"	1: 27' 00"	0: 43' 00"	6	
1: 29' 00"	2: 10' 00"	0: 41' 00"	6	
2: 12' 00"	2: 53' 00"	0: 41' 00"	6	
2: 55' 00"	3: 38' 00"	0: 43' 00"	6	
3: 40' 00"	4: 24' 00"	0: 44' 00"	6	
4: 26' 00"	5: 10' 00"	0: 44' 00"	6	**8.18**

DOUBLE RING INFILTRATION TESTS
Kimmie Road Industrial Park
Completed By : William Parnell, P.E.
PE Job : #08117

Test Pit # **3** (completed @ 9" below existing ground surface)				
Location: 1170' south & 710' west of the northeast property corner at 83rd Ave SW				
Test Date: 6/10/08				
Start	Stop	Elapsed Time	Total Drop	Infiltration Rate
(H: M' S")	(H: M' S")	(H: M' S")	(Inches)	(In/Hr)
0: 00' 00"	0: 18' 30"	0: 18' 30"	6	
0: 20' 00"	0: 38' 45"	0: 18' 45"	6	
0: 40' 00"	0: 59' 15"	0: 19' 15"	6	
1: 00' 30"	1: 20' 00"	0: 19' 30"	6	
1: 21' 00"	1: 41' 00"	0: 20' 00"	6	
1: 42' 00"	2: 02' 00"	0: 20' 00"	6	
2: 03' 00"	2: 23' 30"	0: 20' 30"	6	
2: 25' 00"	2: 45' 45"	0: 20' 45"	6	
2: 47' 00"	3: 07' 45"	0: 20' 45"	6	
3: 09' 00"	3: 31' 00"	0: 22' 00"	6	
3: 33' 00"	3: 55' 00"	0: 22' 00"	6	**16.36**
3: 56' 30"	4: 18' 15"	0: 21' 45"	6	

Test Pit # **4** (completed @ 9" below existing ground surface)				
Location: 1350' south & 200' west of the northeast property corner at 83rd Ave SW				
Test Date: 6/9/08				
Start	Stop	Elapsed Time	Total Drop	Infiltration Rate
(H: M' S")	(H: M' S")	(H: M' S")	(Inches)	(In/Hr)
0: 00' 00"	0: 32' 00"	0: 32' 00"	6	
Soaking Period				
1: 05' 00"	1: 40' 00"	0: 35' 00"	6	
1: 41' 00"	2: 14' 00"	0: 33' 00"	6	
2: 15' 00"	2: 52' 00"	0: 37' 00"	6	
2: 54' 00"	3: 31' 00"	0: 37' 00"	6	**9.73**
3: 34' 00"	4: 10' 30"	0: 36' 30"	6	

DOUBLE RING INFILTRATION TESTS
Kimmie Road Industrial Park
Completed By : William Parnell, P.E.
PE Job : #08117

Test Pit # **5** (completed @ 12" below existing ground surface)
Location: 1730' south & 800' west of the northeast property corner at 83rd Ave SW
Test Date: 6/10/08

Start (H: M' S")	Stop (H: M' S")	Elapsed Time (H: M' S")	Total Drop (Inches)	Infiltration Rate (In/Hr)
0: 00' 00"	0: 09' 30"	0: 09' 30"	6	
0: 11' 00"	0: 34' 15"	0: 23' 15"	6	
0: 36' 00"	1: 00' 45"	0: 24' 45"	6	
1: 02' 00"	1: 27' 30"	0: 25' 30"	6	
1: 29' 00"	1: 55' 00"	0: 26' 00"	6	
1: 56' 30"	2: 23' 00"	0: 26' 30"	6	
2: 24' 00"	2: 50' 30"	0: 26' 30"	6	
2: 52' 00"	3: 18' 30"	0: 26' 30"	6	
3: 21' 00"	3: 48' 00"	0: 27' 00"	6	
3: 50' 00"	4: 17' 00"	0: 27' 00"	6	**13.33**

Test Pit # **6** (completed @ 9" below existing ground surface)
Location: 2170' south & 850' west of the northeast property corner at 83rd Ave SW
Test Date: 6/9/08

Start (H: M' S")	Stop (H: M' S")	Elapsed Time (H: M' S")	Total Drop (Inches)	Infiltration Rate (In/Hr)
0: 00' 00"	1: 05' 00"	1: 05' 00"	6	
Soaking Period				
2: 06' 00"	3: 06' 00"	1: 00' 00"	6	
3: 09' 00"	4: 17' 00"	1: 08' 00"	6	**5.3**
4: 19' 00"	5: 26' 00"	1: 07' 00"	6	

SEC 09, T 17 N, R 2 W, WM

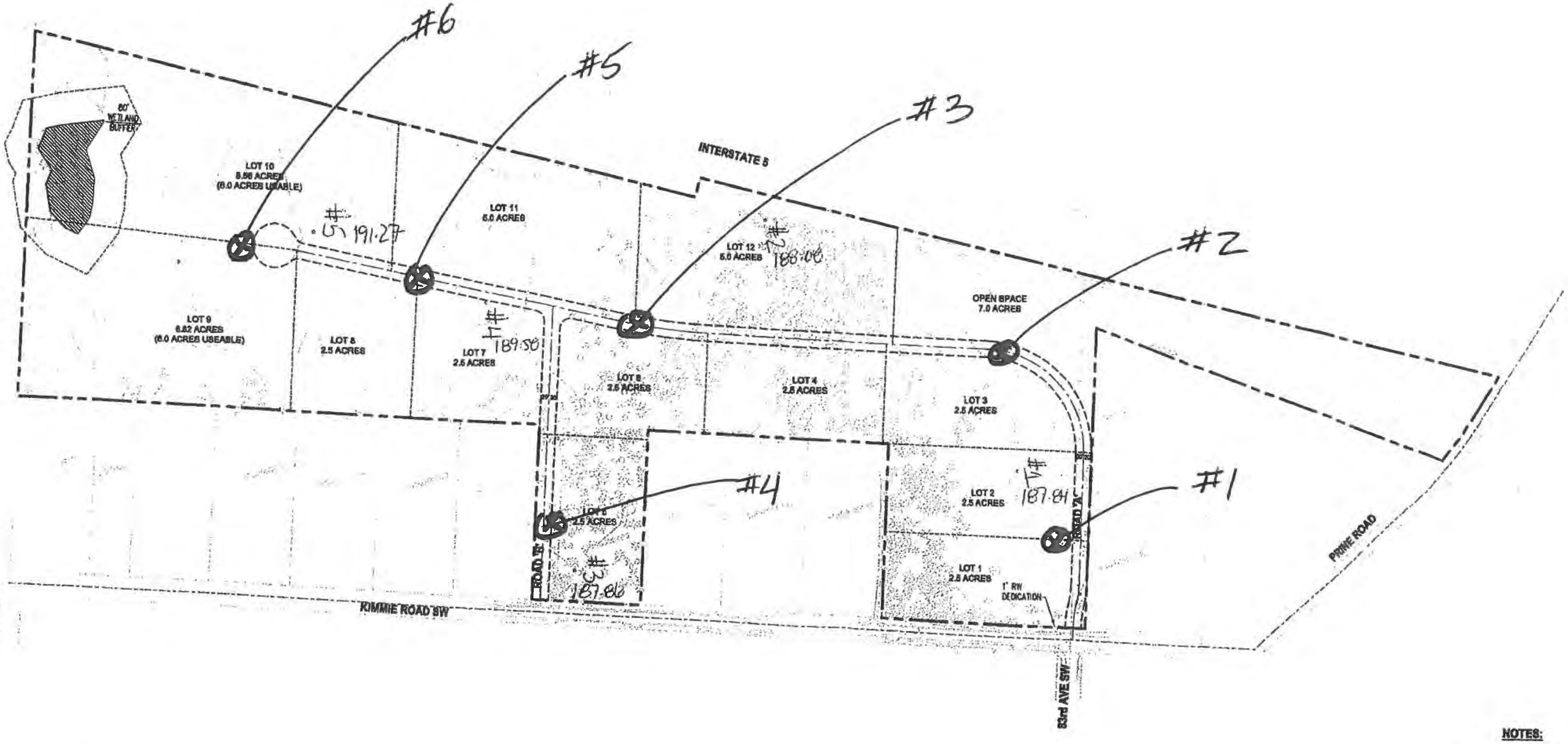
Test Pit Location Map

LEGEND:

- PROPERTY LINE
- ROADWAY EASEMENT
- ROADWAY CENTERLINE/LOT LINE
- LOT LINE

SITE DEVELOPMENT AREA:

TOTAL SITE AREA: 53 ACRES



CALL BEFORE YOU DIG

HE CONTRACTOR SHALL BE FULLY RESPONSIBLE FOR THE LOCATION AND ROTECTION OF ALL EXISTING UTILITIES. THE CONTRACTOR SHALL VERIFY ALL TILITY LOCATIONS PRIOR TO CONSTRUCTION BY CALLING THE UNDERGROUND OCATE LINE AT (800) 424-5555 A MINIMUM OF 48 HOURS PRIOR TO ANY XCAVATION.

NOTES:

1. ROADWAY WILL BE PRIVATE AND WILL BE WITHIN AN EASEMENT. PARCEL LOT LINES TO EXTEND TO ROADWAY CENTERLINE.

SCALE: 1"=150'-0"

150' 0 150' 300'
Scale FEET

N

22

PRELIMINARY SITE PLAN

SHEA CARR JEWELL

PROJECT DESIGN: AMY HEAD
DESIGNER: AMY HEAD
DRAWN BY: L HURTADO
CADD CHECKED BY: S HOEY LEES
CHECKED BY: AMY HEAD
CHECKED/APPROVED: PERRY SHEA
STAMPED BY: AMY HEAD
HORIZONTAL SCALE: N/A
VERTICAL SCALE: N/A
DATE: MAY, 2008
JOB No: 578-03
SP-

APPENDIX D-2

INFILTRATION REPORT FOR THE NORTH SECTION OF THE TRC FACILITY SITE, HARTCROWSER, 2017

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MEMORANDUM

DATE: February 10, 2017

TO: Ross Whitehead, AIA
Schreiber Starling Whitehead Architects

FROM: Roy Jensen, LHG, Hart Crowser, Inc.

RE: **Pilot Infiltration Tests Analysis and Results**
Tumwater Readiness Center
Tumwater, Washington
19202-00

Hart Crowser performed two pilot infiltration tests to support construction at the proposed Tumwater Readiness Center in Tumwater, Washington. The purpose of the infiltration tests is to determine infiltration rates for design of stormwater infiltration facilities. The infiltration rate obtained from the proposed infiltration tests are considered to be a short-term infiltration rate. Short-term infiltration rates are adjusted through correction factors to account for site variability and number of tests conducted, degree of long-term maintenance and influent pre-treatment/control, and potential for long-term clogging due to siltation and bio-buildup. The infiltration test and analysis procedures are consistent with the test procedures provided in the 2010 City of Tumwater Drainage Design and Erosion Control Manual (December 2009).

Project Background

The soil layers observed during the preliminary field exploration program consisted of the following soil units, described in the order they were encountered from the ground surface down.

- **Loose to Medium Dense Silty Sand.** From the ground surface to a depth generally ranging from 1 to 4 feet below ground surface (bgs) the borings encountered loose to medium dense, slightly silty to silty sand.

- **Medium Dense to Dense Sand and Gravel.** A medium dense to dense sand and gravel unit was encountered directly under the Loose to Medium Dense Silty Sand and extended to depths ranging from 20 to 27 feet bgs. This unit was generally observed to vary between sandy to very sandy gravel and very gravelly sand with trace amounts of silt and layers of sand.

3131 Elliott Avenue, Suite 600
Seattle, Washington 98121
Tel 206.324.9530

- **Very Dense Sand and Gravel.** A very dense sand and gravel unit was encountered directly under the Medium Dense to Dense Sand and Gravel. This unit was observed to be layers of sand and sandy to very sandy gravel and extended to the bottom of all the borings drilled.

Groundwater was encountered during drilling the borings for preliminary field exploration program. Groundwater levels observed at time of drilling (ATD) ranged from about 7.5 to 11 feet bgs, or approximately elevation 181 to 184 feet.

GENERAL PROCEDURE – PILOT INFILTRATION TEST

The general procedures for the pilot infiltration tests are presented below.

- Excavate a test pit using an excavator to the depth of the bottom of the proposed infiltration test. The dimensions of the test pit for this project were generally 5 feet wide by 5 feet long corresponding to the area of the bottom of the test pit of approximately 25 square feet.

- Document soil conditions observed during excavation and along the side walls of the excavation. Record the size and geometry of the test pit, before beginning the field test.

- Install a vertical measuring rod marked in half inch increments in the pit bottom. The rod was used to record water levels in the test pit.

- Use a rigid 6-inch-diameter pipe with a splash plate on the bottom to convey water to the pit and reduce side wall erosion or excessive disturbance of the pond bottom.

- Conduct the constant head portion of the test by adding water to the test pit at a rate that will maintain a water level between 3 and 4 feet above the bottom of the pit. A flow meter verified with a bucket test was used to measure the flow rate into the pit.

- Record the cumulative volume and instantaneous flow rate in gallons per minute (gpm) necessary to maintain the water level at the same point (1 foot) on the vertical measuring rod. Water levels in the test pits were also monitored with a pressure transducer.

- Continue adding water to the pit while maintaining constant water level in the test pit for 6 to 8 hours.

- At the end of the constant head test, water flow into the test pit was turned off and the drop in water level was recorded for a period of at least 1 hour. This phase of the infiltration test is referred to as the falling head test.

TEST RESULTS

Two infiltration tests were completed at the site between September 12 and 16, 2016. The locations of the infiltration test pits are shown on Figure 1. The results of the individual infiltrations tests are summarized below.

Infiltration Test PIT-102

- The dimensions of Infiltration PIT-102 were about 5 by 5 feet and 2 feet deep. Soils observed in the test pit include soil cover with rootlets (0 to 2 feet) and sandy Gravel at the bottom of the test cell.

- Infiltration Test 1 was conducted on September 12, 2016. The constant head test was conducted for nearly 12 hours starting at 13:50. and ending at 19:12. The following falling head test was monitored from 19:12 until 20:08.

- During the constant head test, the water level in the test pit was maintained at approximately 1 foot at a flow rate of between 5 to 5.7 gpm. The average flow rate was 5.4 gpm. Water levels and flow rates during the constant head test are presented on Figure 2.

- During the falling head test, water levels dropped from 1 foot to 0.1 foot in 86 minutes. Water levels monitored during the falling head test are presented on Figure 3.

- The results of the constant head test indicate that at a constant head of 1 foot, the field infiltration rate is 0.2 gpm/ft^2 or 20 inches per hour (in./hr).

Infiltration Test PIT-101

- The dimensions of Infiltration Test PIT-101 were about 5 by 5 feet and 1.5 feet deep. Soils observed in the test pit were slightly silty fine to medium Sand.

- Infiltration Test PIT-101 was conducted on September 15, 2016. The constant head test was conducted for nearly 12 hours starting at 8:00 and ending at 16:00. The following falling head test was conducted from 16:00 until 16:46.

- Water levels were maintained at approximately 1 foot at flow rate of 2.32 to 3.36 gpm during the constant head test. At the end of the test the average flow rate was 2.6 gpm. Water levels and flow rates during the constant head test are presented on Figure 4.

- During the falling head test water levels dropped from 1 foot to 0.1 foot in about 90 minutes. Water levels during the falling head test are presented on Figure 5.

- The results of the constant head test indicate that at a constant head of 1 foot the field infiltration rate is 0.1 gpm/ft^2 or 10 in./hr.

SUMMARY AND CONCLUSIONS

- Two pilot infiltration tests were completed in the study area. Soil encountered in the test pits consist of an upper unit of sandy Gravel to gravelly Sand with organics and outwash gravel unit generally consisting of very sandy Gravel. The infiltration tests were conducted in the top of the outwash gravel unit.

- The infiltration tests consisted of a constant head test and a falling head test. The field infiltration rates based on the constant head tests ranged from 10 to 20 in./hr (Table 1).

- Infiltration rates are head dependent. The higher the head, the higher the infiltration rate. The infiltration rates developed in this study are based on a head of 1 foot.

- For design purposes, a correction factor of about 3 was used to adjust the infiltration rates to develop design infiltration rates in shallow soil units at the site. The design infiltration rate for the sandy gravel unit based on Infiltration Test PIT-102 is 7 in./hr and the design infiltration rate for the silty sand unit based on Infiltration Tests PIT-101 is 3 in./hr.

Attachments:

Table 1 - Summary of Infiltration Test Results
Figure 1 - Infiltration Test Location Map
Figure 2 - Infiltration Test PIT-102 - Constant Head Test
Figure 3 - Infiltration Test PIT-102 - Falling Head Test
Figure 4 - Infiltration Test PIT-101 - Constant Head Test
Figure 5 - Infiltration Test PIT-101 - Falling Head Test

Table 1 - Summary of Infiltration Test Results
Tumwater Readiness

Infiltration Test Number	Length in Feet	Width in Feet	Area in Square Feet	Steady-State Head in Feet	Steady-State Flow in gpm	Infiltration Rate in gpm/ft^2	Infiltration Rate in in./hr	Correction Factor	Scaled Infiltration Rate in in./hr	Recommended Design Infiltration Rate in in./hr
PIT-101	5.25	5.0	26.3	1	2.6	0.1	9.6	3	3	3
PIT-102	5.25	5.0	26.3	1	5.4	0.2	19.9	3	7	7

Legend

Proposed Explorations and Testing

- HC-101 Boring
- HC-MW-107 Monitoring Well
- TP-101 Test Pit
- PIT-101 Pilot Infiltration Test (PIT)

Existing Explorations and Wells

- HC-1 Hart Crowser Preliminary Investigation Boring
- GSW-1 Existing Well
- TP-9 Historical Test Pit

0 100 200
Scale in Feet

N

Tumwater Readiness Center
Tumwater, Washington

Proposed Explorations and Field Testing - North Site

19202-00 7/16

HARTCROWSER

Figure 1

EAL 07/22/16 1920200-006 (SPlan).dwg

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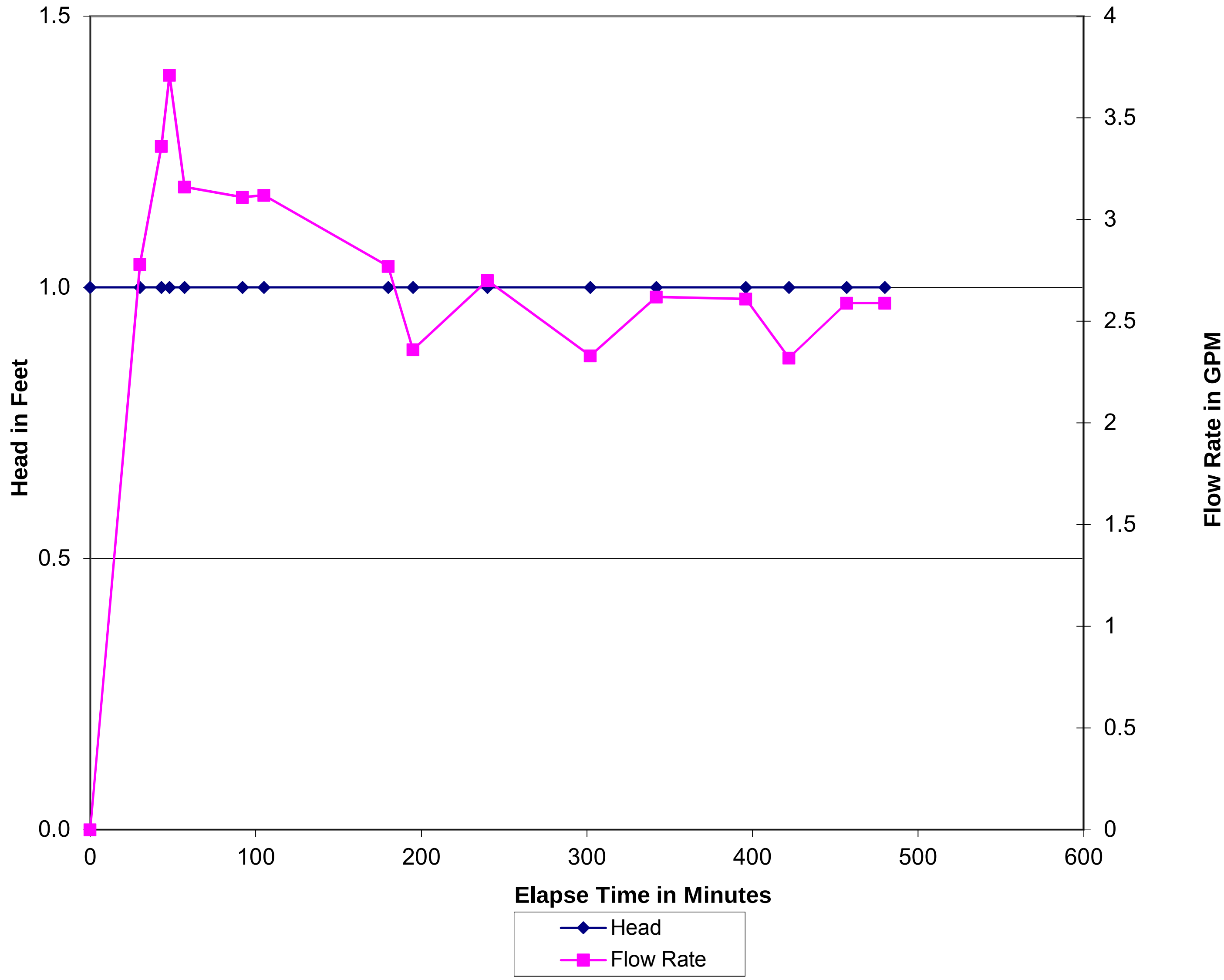

Tumwater Readiness Center

Infiltration Test PIT-102
Constant Head Test

19202-00

9/16

HARTCROWSER

Figure
2

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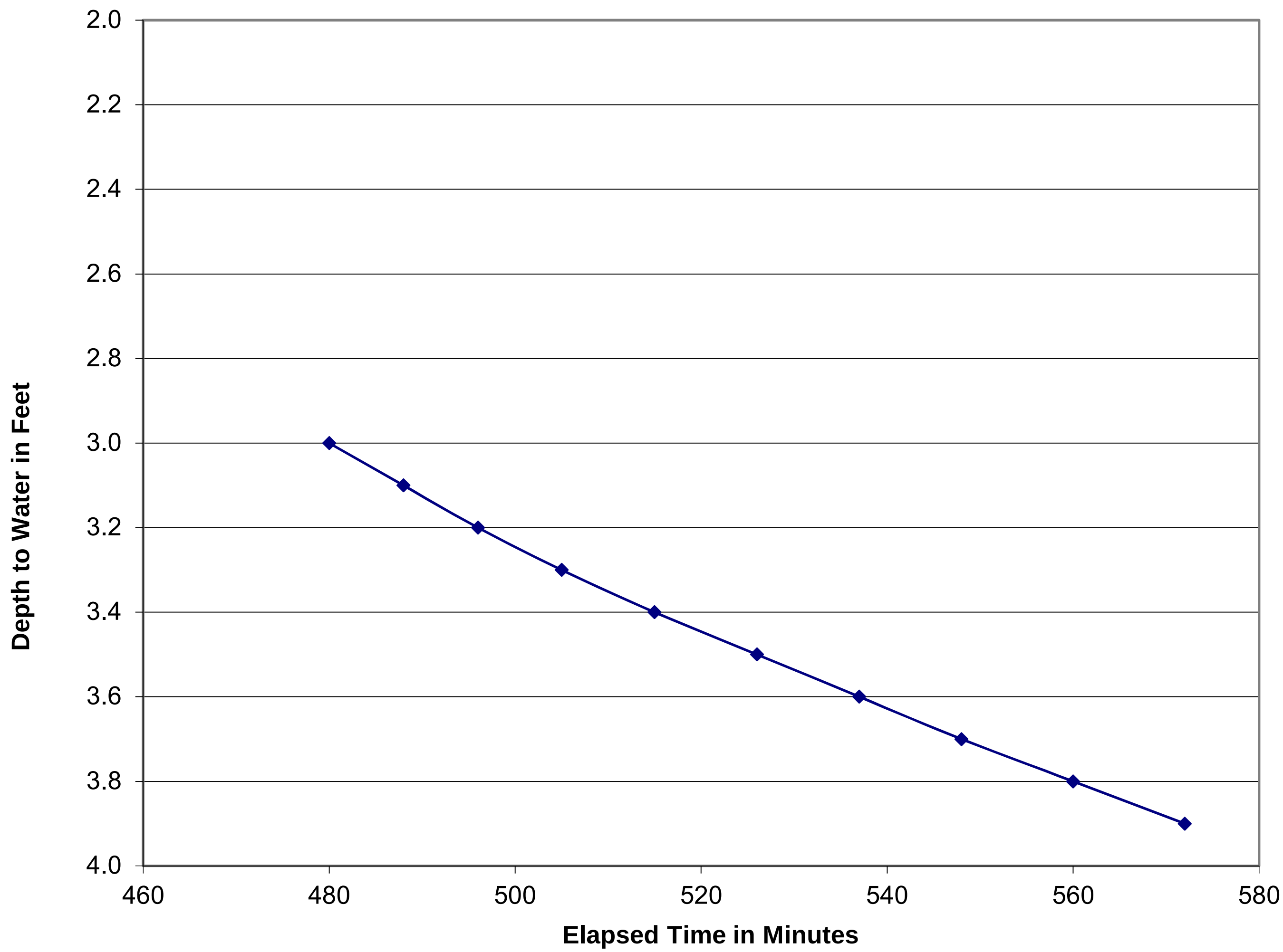

Tumwater Readiness Center

Infiltration Test PIT-102
Falling Head Test

19202-00

9/16

Figure

3

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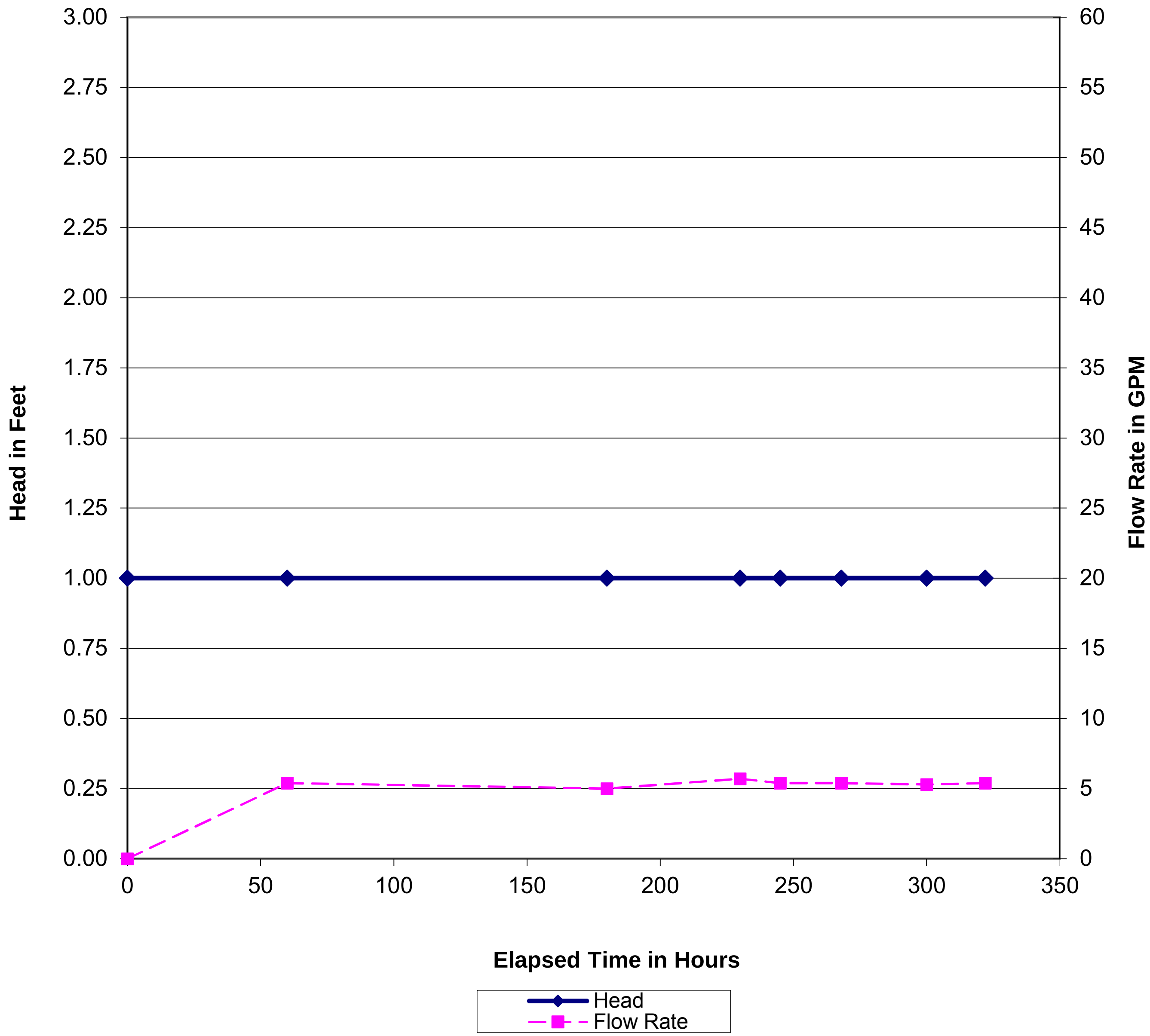

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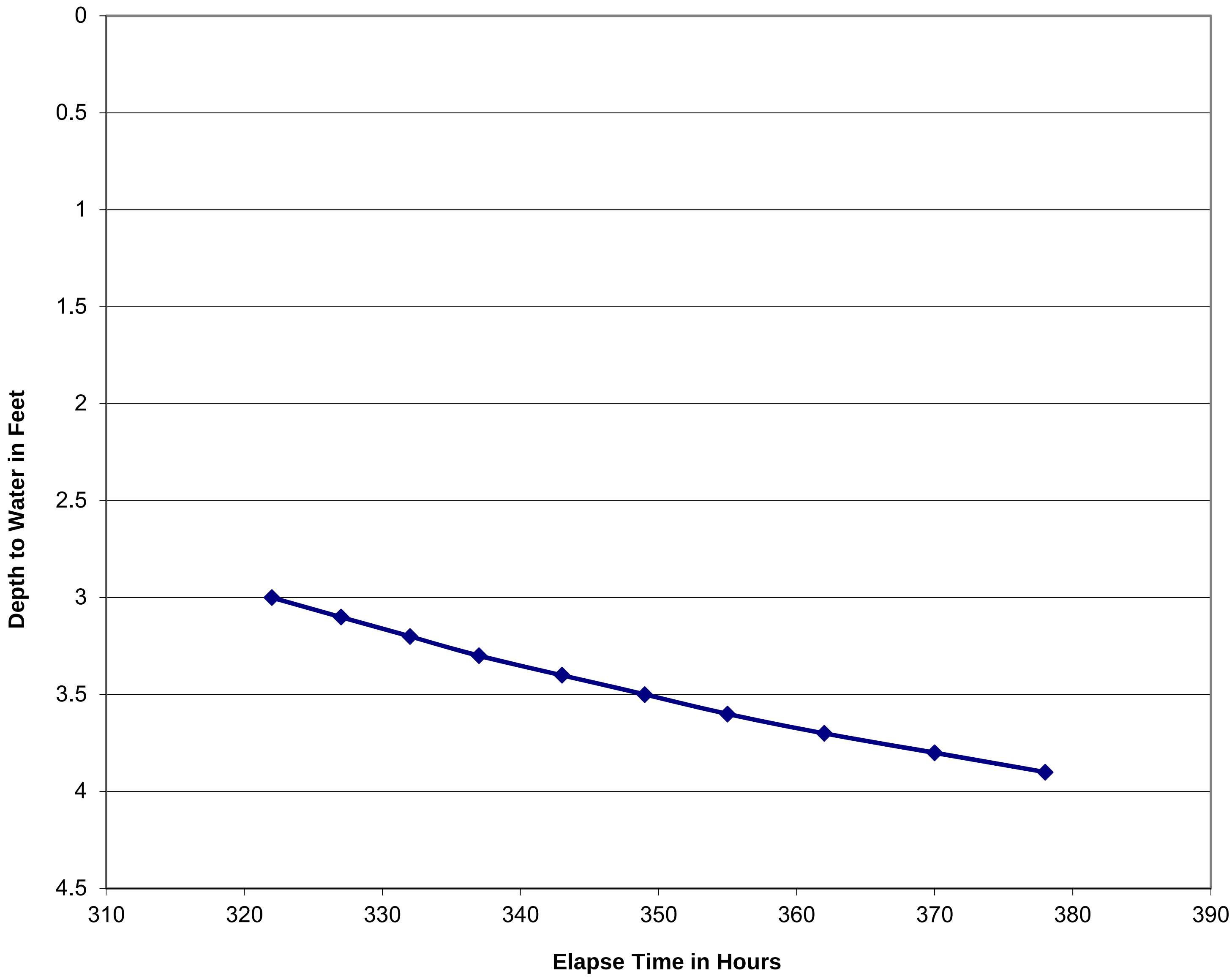
19202-00

9/16

Infiltration Test PIT-101
Constant Head Test

Tumwater Readiness Center

Figure
4



HARTCROWSER

19202-00 9/16

Infiltration Test PIT-101
Falling Head Test

Tumwater Readiness Center

Figure **5**

APPENDIX E

LINEAR REGRESSION FOR KIMMIE STREET INDUSTRIAL PARK, ROBINSON/NOBLE/SALTBUSH, 2008

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575-03
Regression Analysis

ROBINSON
NOBLE SALTBUSH INC. Established 1947
GROUNDWATER & ENVIRONMENTAL SCIENTISTS
1947 60 Years 2007

May 16, 2008

Mr. Todd Hansen
Todd Hansen Construction
9300 Kimmie St. SW
Olympia, WA 98512

Subject: Linear Regression Analysis for the Kimmie Street property

Mr. Hansen:

Per your request, Robinson, Noble & Saltbush has completed the water level monitoring and linear regression analysis for the proposed commercial development located at the Blomberg Industrial Park. These tasks were detailed as tasks 1 and 2 in our scope letter dated October 3, 2007. The Kimmie Street property requires this initial approach for stormwater planning due to the fact that the site is located in the Salmon Creek Basin, which has historically experienced significant flooding. Six piezometers were installed on the property in late 2007, five of these were equipped with data logging pressure transducers. Water level data was collected at all of these locations on 15-minute intervals from January 15, 2008 to May 10, 2008. Several visits were made during the data collection period to verify water levels and data logger operation, as well as retrieve the data logger records.

Once data collection for the period of interest was completed, the data sets were corrected for barometric influence. This was accomplished by subtracting the pressure recorded concurrently by an on-site barometric logger from each data point. The resulting feet of water was then referenced to the land surface elevation, as surveyed by Skillings-Connolly, Inc.

Available County water level monitoring data was collected and reviewed. At the time the analysis was completed, complete data sets were available for our period of record from eight of the wells in the 11-well network. These wells were: LRS-08, LRS-07A, LRS-01A, LRS-11A, LRS-09, LRS-12, TC MW-3A and TC MW-5. The remaining wells did not have complete data sets available for the period required. Of the wells that had complete data sets, several did not have historical data sets available, which makes them unusable in a regression analysis. Our data sets were culled to match the most complete County data sets by selecting the data point closest to the County measurement.

Analysis
The linear regression analysis was performed by comparing the site-specific data from each of the wells to the nearby County wells, as described in the guidance document. The water level for the on-site wells are plotted against the time-synchronous County water levels. A linear best-fit line is fitted to each of the plots. The equation for this line and the statistics of the fit are recorded for each fit. Plots for each of the wells on site with the respective county data sets are presented in the appendix. A summary of the fit properties and statistics is presented in the following table.

Table 1: Linear Regression Results and Statistics

Historical Data Available				
	LRS-08	LRS-07	LRS-01A	LRS-11A
MW-1	$y=0.8305x+28.46$ $r^2=0.9756$	$y=0.7705x+47.82$ $r^2=0.5212$	$y=1.218x-36.89$ $r^2=0.8830$	$y=0.5190x+83.61$ $r^2=0.5505$
MW-2	$y=0.8707x+21.00$ $r^2=0.9676$	$y=0.8924x+26.60$ $r^2=0.6310$	$y=1.221x-37.46$ $r^2=0.8006$	$y=0.6006x+68.14$ $r^2=0.6654$
MW-3	$y=0.8482x+25.09$ $r^2=0.9723$	No Correlation $r^2<0.5$	$y=1.252x-43.06$ $r^2=0.8912$	$y=0.5216x+83.01$ $r^2=0.5312$
MW-4	$y=0.8849x+18.88$ $r^2=0.9894$	$y=0.8709x+30.83$ $r^2=0.5949$	$y=1.261x-44.07$ $r^2=0.8449$	$y=0.5864x+71.32$ $r^2=0.6280$
MW-5	$y=0.9469x+7.668$ $r^2=0.9546$	$y=1.081x-5.368$ $r^2=0.7718$	$y=1.244x-40.81$ $r^2=0.6929$	$y=0.7264x+45.12$ $r^2=0.8117$

No Historical Data Available				
	LRS-12	LRS-09	TC MW-5	TC MW-3
MW-1	No Correlation $r^2<0.5$	$y=0.7471x+44.42$ $r^2=0.8198$	$y=0.4791x+93.51$ $r^2=0.5569$	No Correlation $r^2<0.5$
MW-2	$y=0.5561x+77.24$ $r^2=0.6128$	$y=0.8127x+32.32$ $r^2=0.8754$	$y=0.5343x+83.31$ $r^2=0.6251$	No Correlation $r^2<0.5$
MW-3	No Correlation $r^2<0.5$	$y=0.7588x+42.16$ $r^2=0.8081$	$y=0.4884x+91.69$ $r^2=0.5531$	No Correlation $r^2<0.5$
MW-4	$y=0.5418x+80.42$ $r^2=0.5758$	$y=0.8142x+32.55$ $r^2=0.8699$	$y=0.5283x+84.91$ $r^2=0.6050$	No Correlation $r^2<0.5$
MW-5	$y=0.6808x+54.57$ $r^2=0.7660$	$y=0.9235x+12.71$ $r^2=0.9428$	$y=0.6202x+68.24$ $r^2=0.7024$	No Correlation $r^2<0.5$

The closest County well with that shows a good correlation to the wells on site and has both a current and historical data set available is location LRS-08, approximately 3,500 feet to the east of the site. Water levels from the on-site wells show a relatively strong linear correlation with water levels from this well, with an average r^2 value of 0.9719.

As the correlation coefficient for the site wells/LRS-08 well pairs averages 0.9719, the relationship is adequate for the completion a linear regression analysis. Regression equations were applied to the County historical data set from LRS-08 to calculate a projected groundwater elevation at the site based on the measured historical data and the mathematically derived relationship. The highest water level from LRS-08 was recorded on February 25, 1999, when the elevation of the water in the well was 191.899 feet above sea level. The land surface elevation at this monitoring point is 191.1 feet, indicating flooding of the local area at that time.

According to Thurston County guidelines, the ground water beneath the proposed stormwater facility must maintain a six-foot separation from the base of that facility. For the purposes of these calculations, the linear regression was considered to be successful if water levels were projected to be greater than six feet below land surface during the historical high water period.

Applying the best-fit linear equation developed for each of the site wells and LRS-08 indicate that all of the wells on site have projected water levels within six feet of land surface during the historical high water period. The calculated values are presented in the following table.

Table 2: Projected Historical Water Levels based on Linear Regression to LRS-08

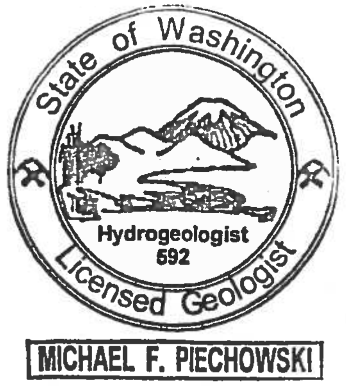
	LRS-08	Historical High Water at LRS-08	Projected Site High Water	Difference from Land Surface	Surface
MW-1	y=0.8305x+28.46 r^2=0.9756	191.8986	187.841	0.819	188.66
MW-2	y=0.8707x+21.00 r^2=0.9676	191.8986	188.083	0.117	188.20
MW-3	y=0.8482x+25.08 r^2=0.9723	191.8986	187.859	5.96	193.819
MW-4	y=0.8849x+18.88 r^2=0.9894	191.8986	189.577	3.42	192.99
MW-5	y=0.9469x+7.668 r^2=0.9546	191.8986	191.273	-1.07	190.20

Conclusions

As this linear regression analysis did not result in a positive result, further analysis of the site and proposed stormwater design will be required for the completion of an appropriate stormwater design for this property. The requirements are defined in the Salmon Creek Basin Interim Guidance Document, which stipulates that further analysis will require the preparation of a finite-difference numerical model of the site and surrounding area.

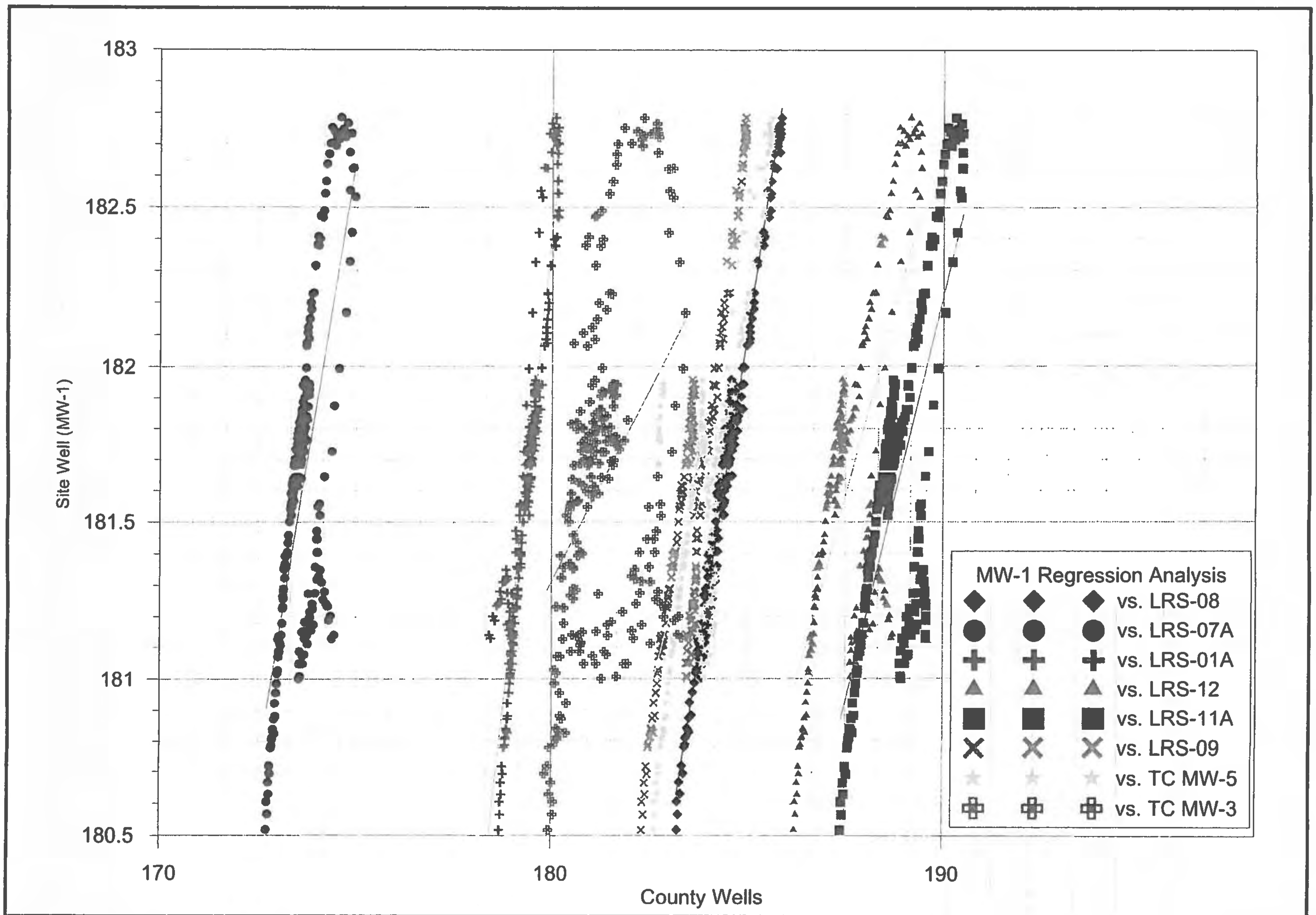
We appreciate this opportunity to provide our services to Todd Hansen Construction. Should you have any concerns regarding this project, or require further clarification of the information presented, please do not hesitate to contact us.

Sincerely,
Robinson, Noble & Saltbush, Inc.



Michael F. Piechowski, L.HG.
Senior Hydrogeologist

attachments



ROBINSON NOBLE SALTBUSH INC.
GROUNDWATER & ENVIRONMENTAL SCIENTISTS

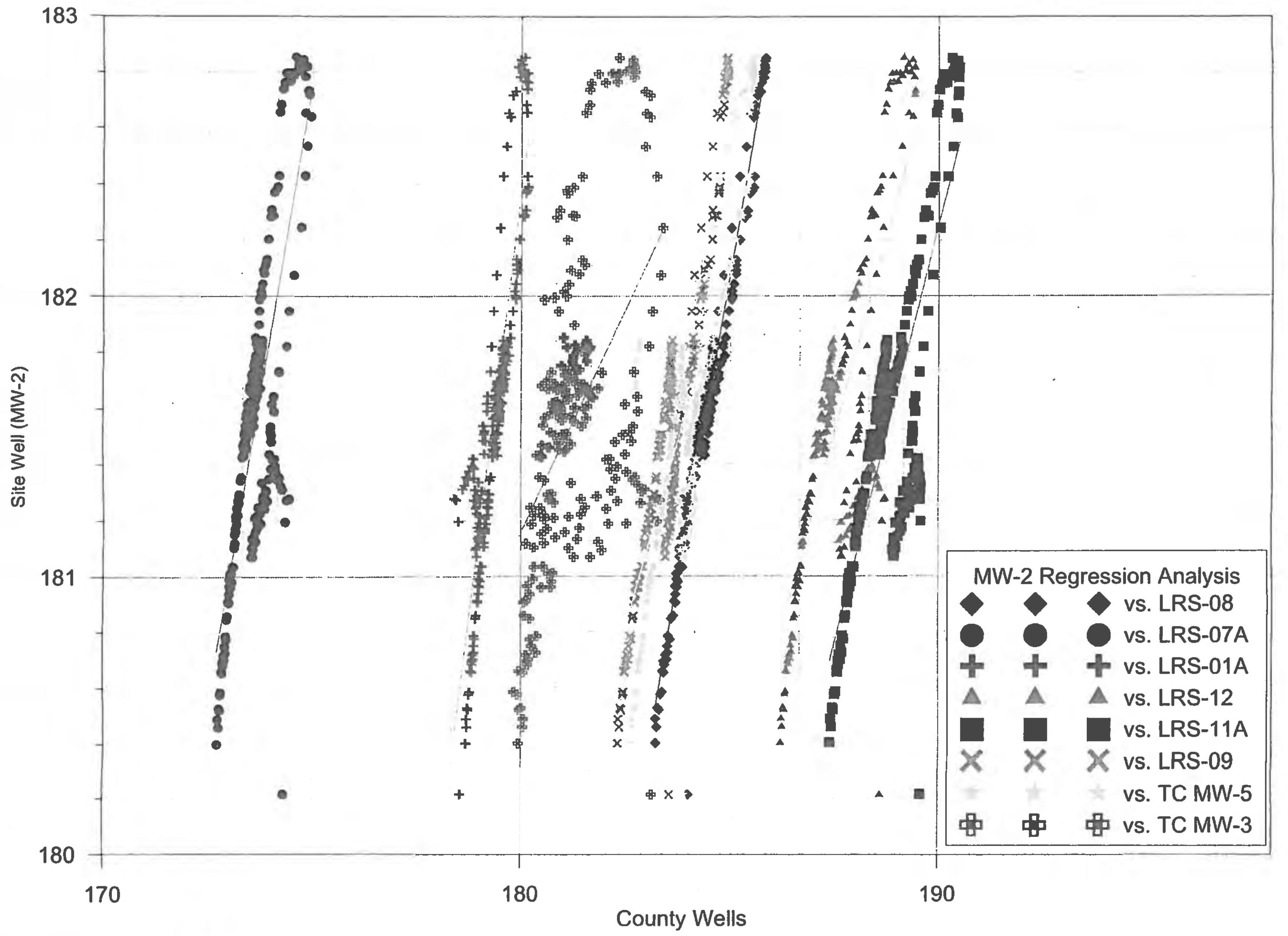
Date: 5/2008
Job#: 1357-005A
PM: BGC

Water Level Record from 1-08 to 5-08
Presented as feet above MSL

Todd Hansen Construction
Kimmie St. Linear Regression Analysis
MW-1 vs County Wells (all data)

Figure 1

26



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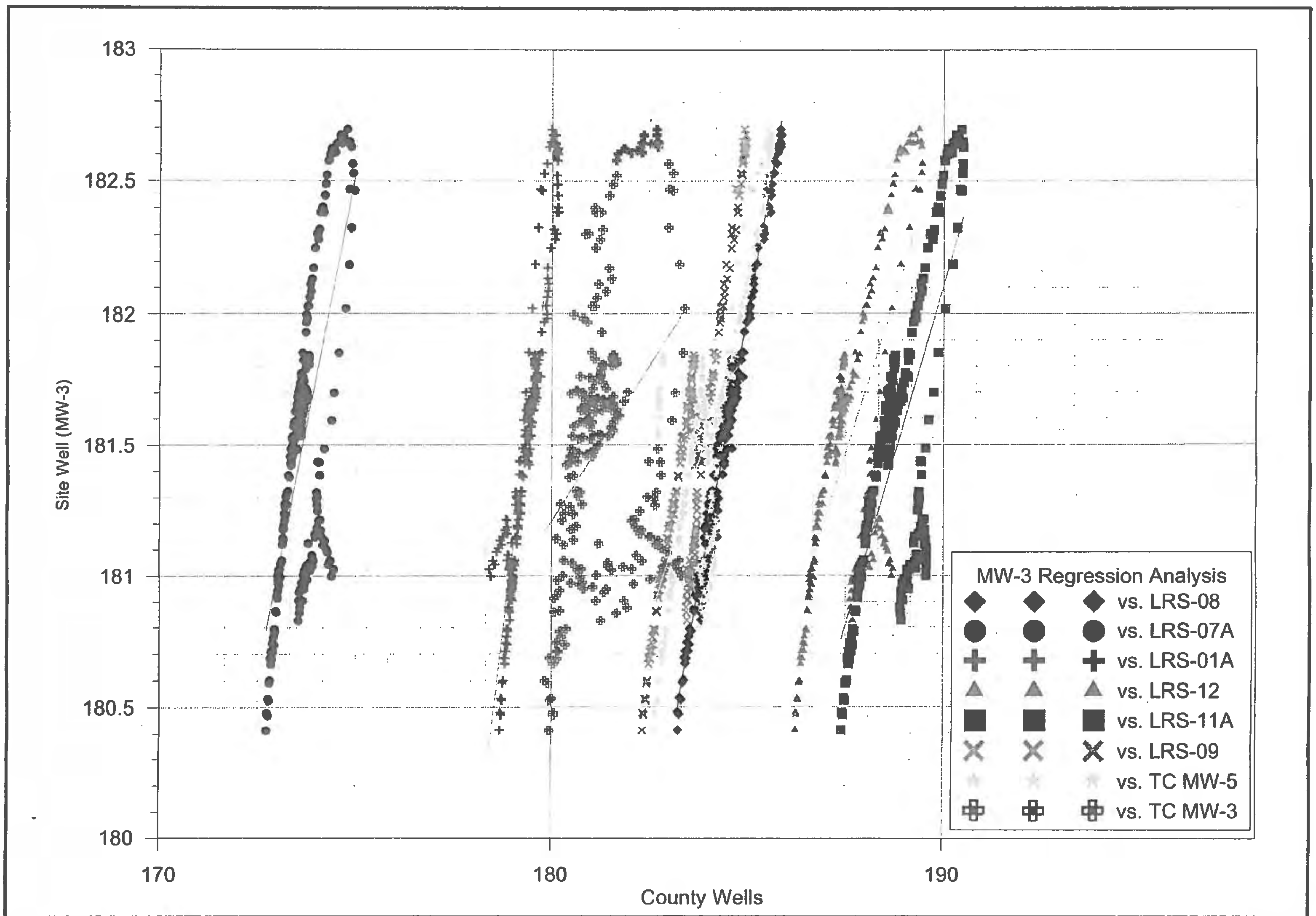
ROBINSON NOBLE SALTBUSH INC. GROUNDWATER & ENVIRONMENTAL SCIENTISTS	Date: 5/2008 Job#: 1357-005A PM: BGC	Water Level Record from 1-08 to 5-08 Presented as feet above MSL	Todd Hansen Construction Kimmie St. Linear Regression Analysis MW-2 vs County Wells (all data)

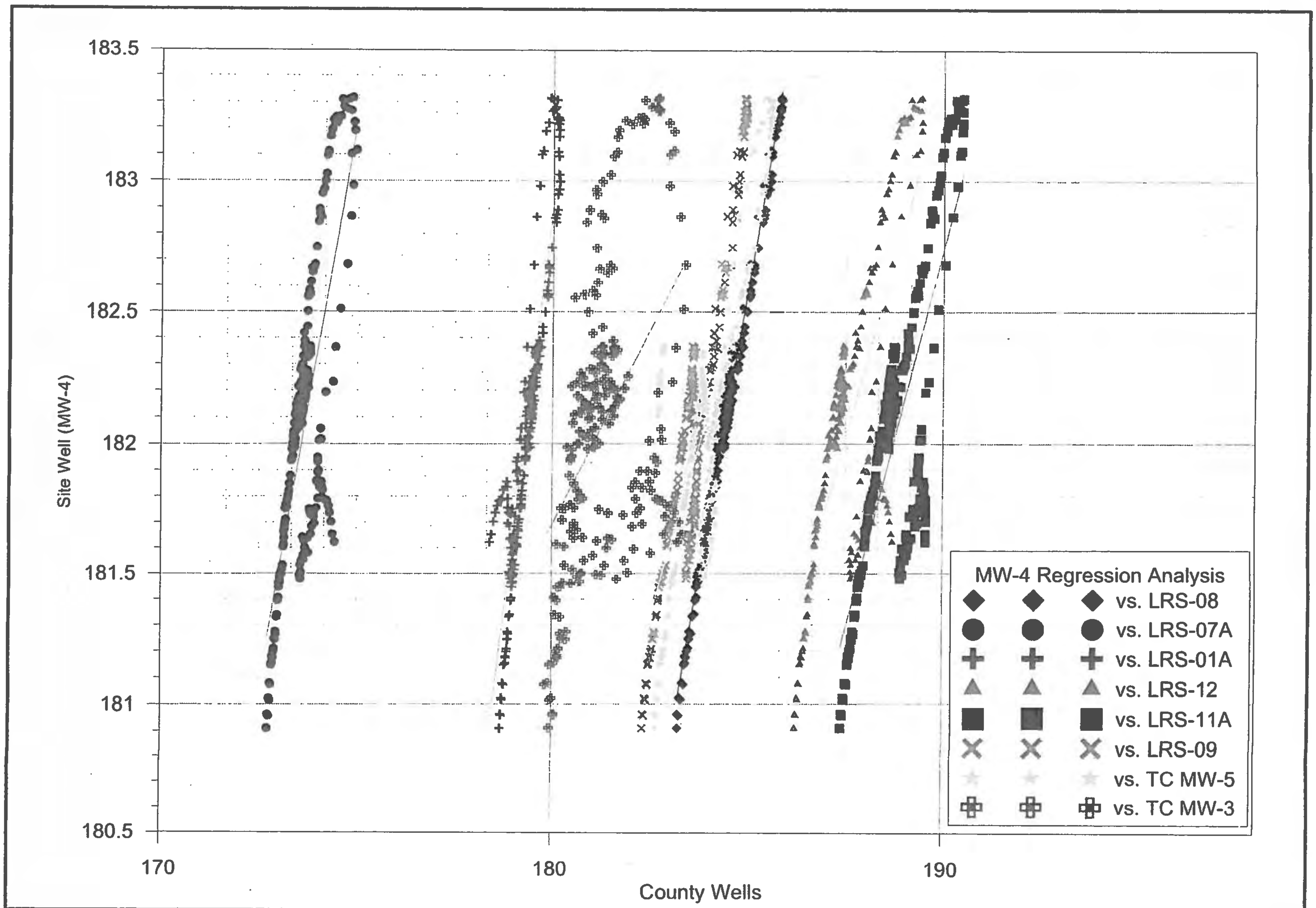
Figure 2



ROBINSON NOBLE SALTBUSH INC. GROUNDWATER & ENVIRONMENTAL SCIENTISTS	Date: 5/2008 Job#: 1357-005A PM: BGC	Water Level Record from 1-08 to 5-08 Presented as feet above MSL	Todd Hansen Construction Kimmie St. Linear Regression Analysis MW-3 vs County Wells (all data)

Figure 3

28



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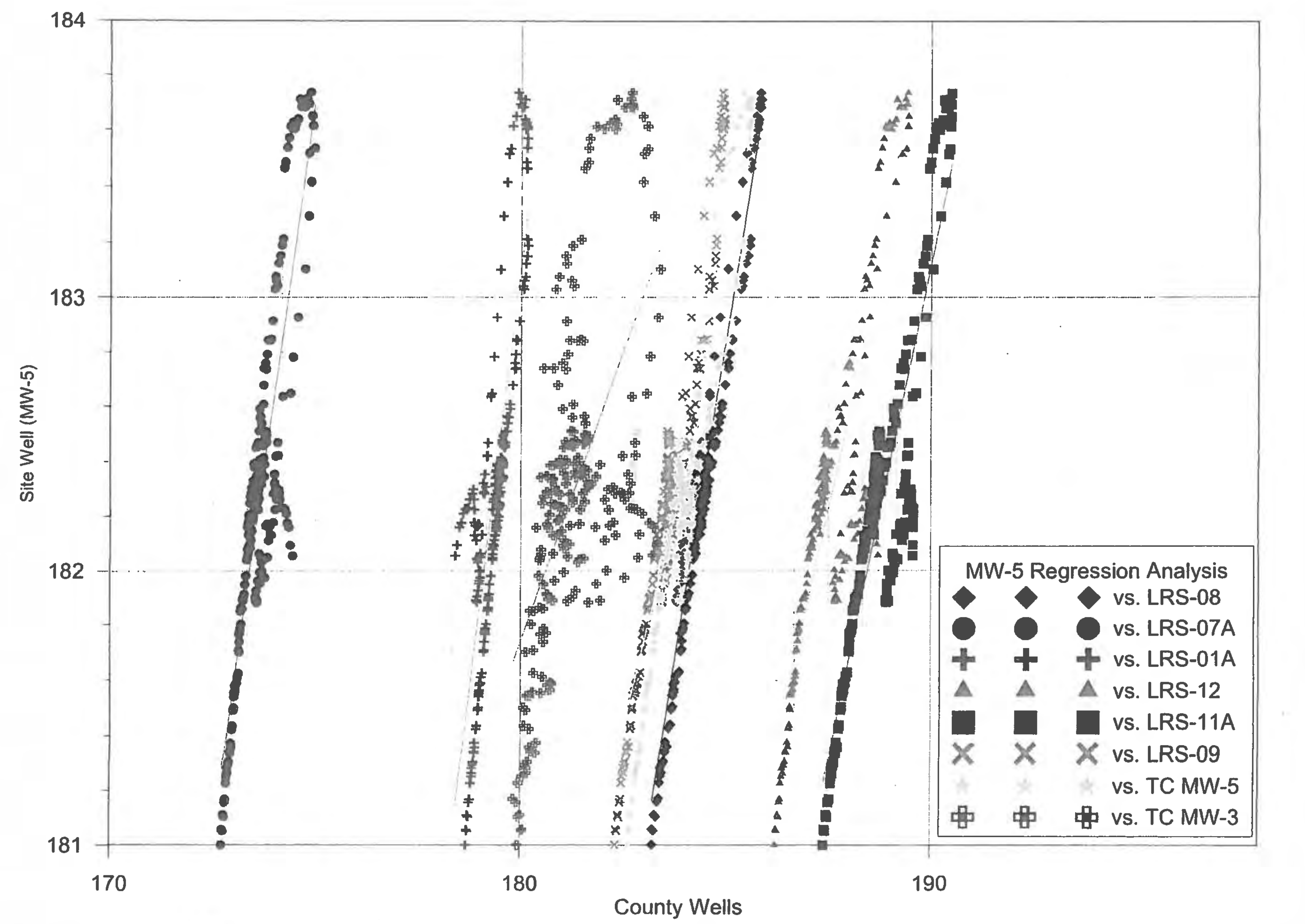
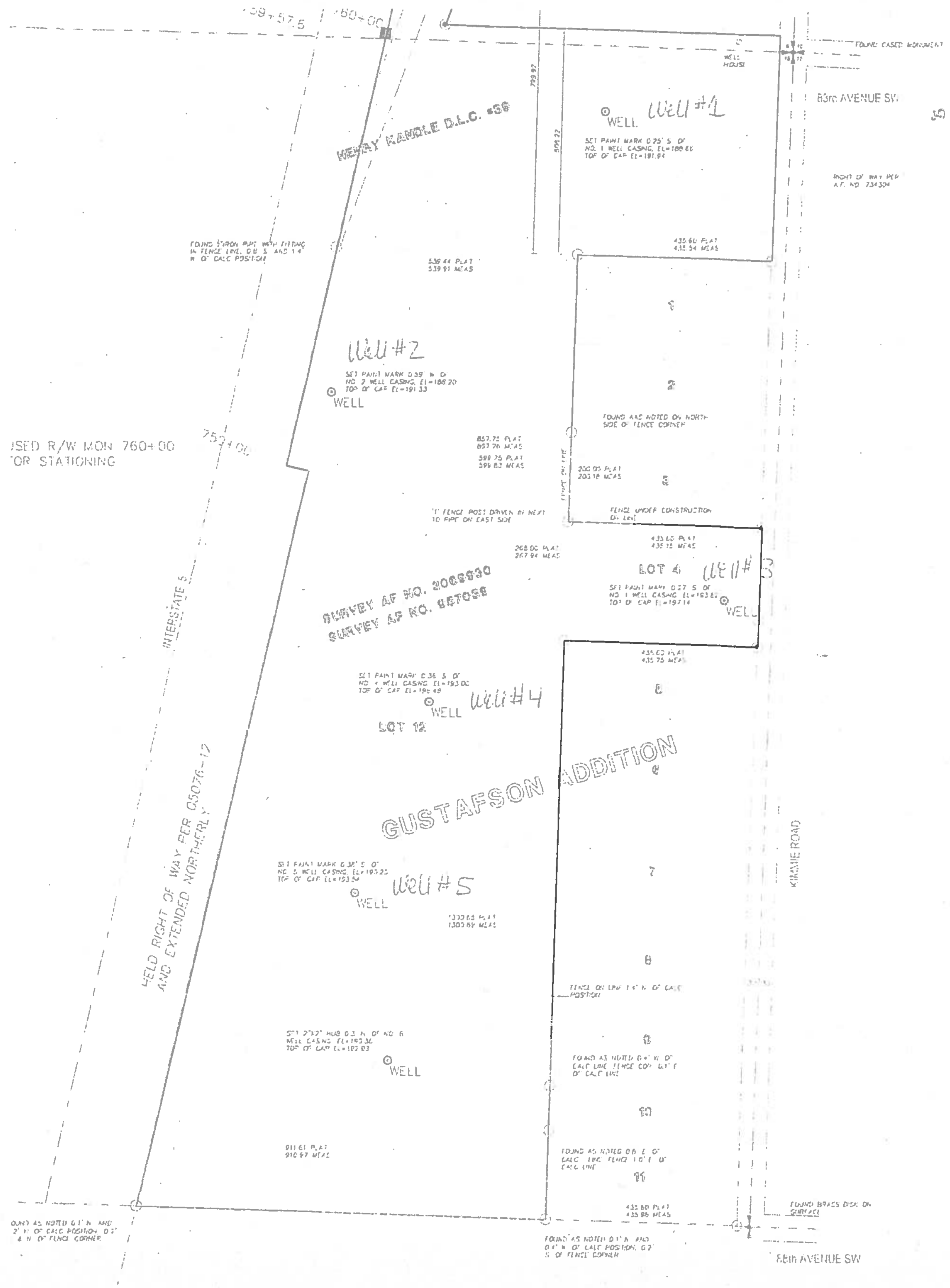
ROBINSON NOBLE SALTBUSH INC. GROUNDWATER & ENVIRONMENTAL SCIENTISTS	Date: 5/2008 Job#: 1357-005A PM: BGC	Water Level Record from 1-08 to 5-08 Presented as feet above MSL	Todd Hansen Construction Kimmie St. Linear Regression Analysis MW-4 vs County Wells (all data)

Figure 4



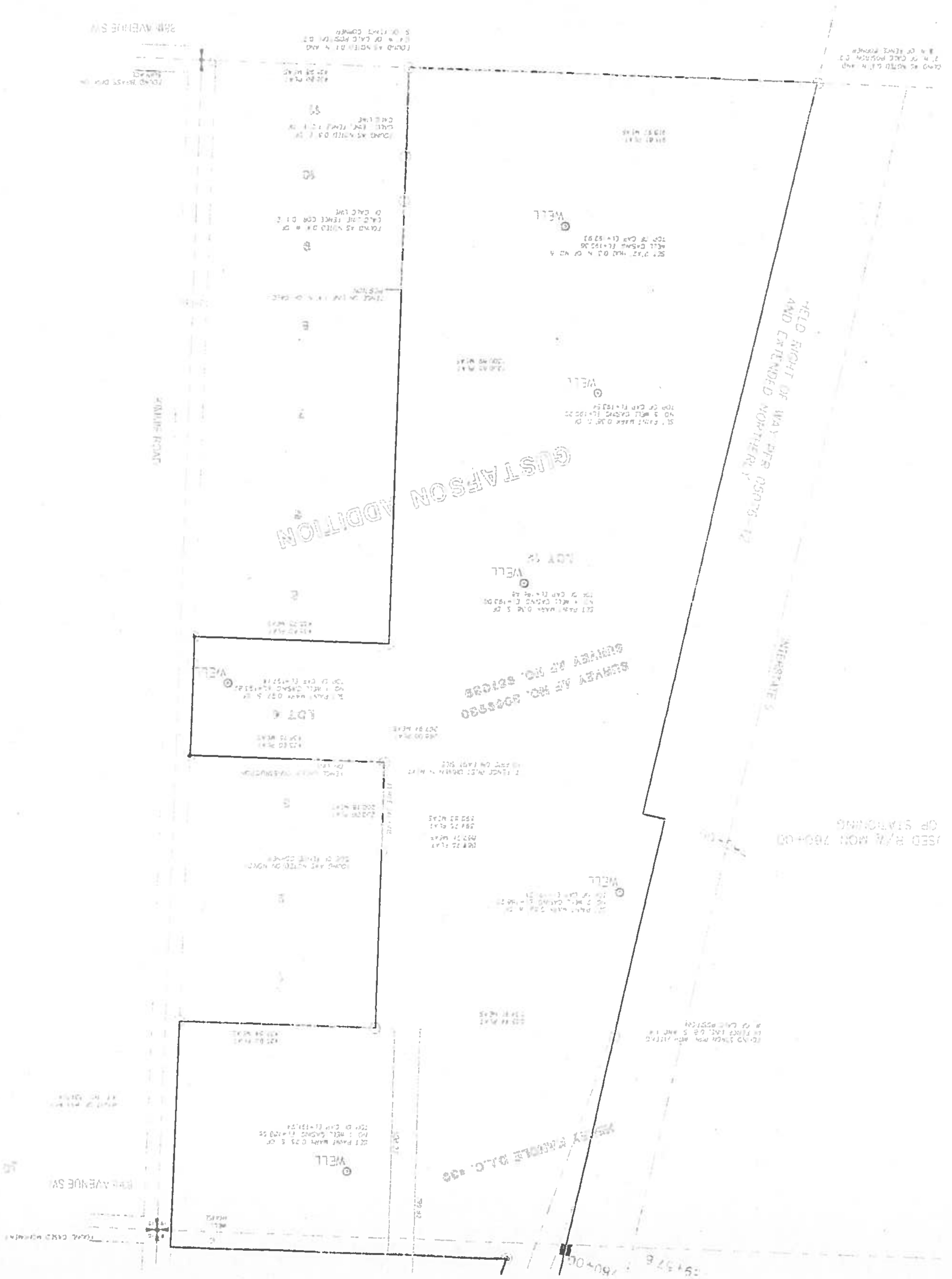
ROBINSON NOBLE SALTBUSH INC. GROUNDWATER & ENVIRONMENTAL SCIENTISTS	Date: 5/2008 Job#: 1357-005A PM: BGC	Water Level Record from 1-08 to 5-08 Presented as feet above MSL	Todd Hansen Construction Kimmie St. Linear Regression Analysis MW-5 vs County Wells (all data)

Figure 5



HELD RIGHT OF WAY PER 05076-12
AND EXTENDED NORTHERLY
INTERSTATE 5
JSED R/W MON 760+00
'OR STATIONING
HENRY KANDLE D.L.C. #38
GUSTAFSON ADDITION
LOT 4
LOT 12
Well #1
Well #2
Well #3
Well #4
Well #5
WELL
83rd AVENUE SW
KIMMIE ROAD
88th AVENUE SW

31



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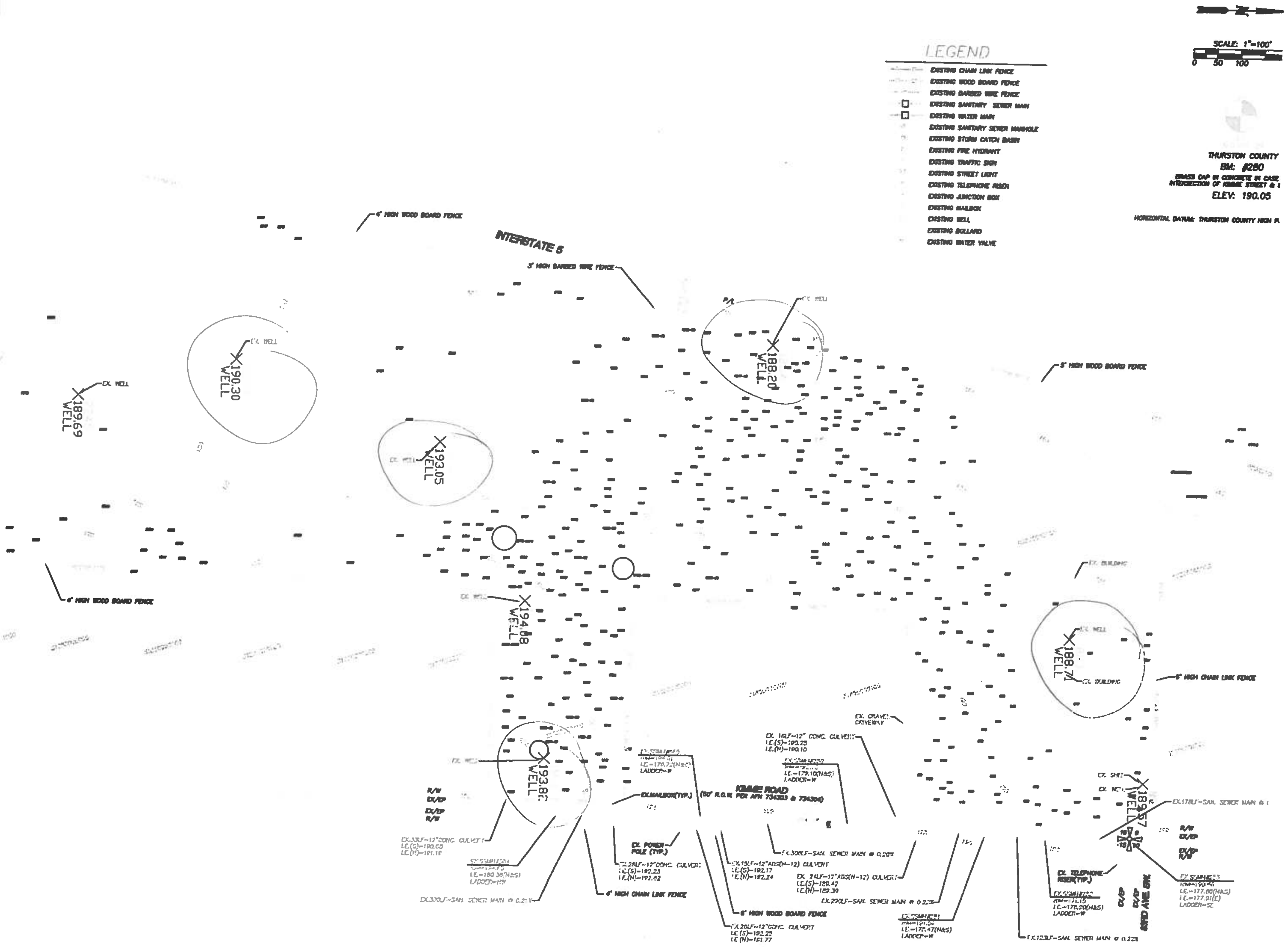
LEGEND
EXISTING CHAIN LINK FENCE
EXISTING WOOD BOARD FENCE
EXISTING BARBED WIRE FENCE
EXISTING SANITARY SEWER MAIN
EXISTING WATER MAIN
EXISTING SANITARY SEWER MANHOLE
EXISTING STORM CATCH BASIN
EXISTING FIRE HYDRANT
EXISTING TRAFFIC SIGN
EXISTING STREET LIGHT
EXISTING TELEPHONE RISER
EXISTING JUNCTION BOX
EXISTING MAILBOX
EXISTING WELL
EXISTING BOLLARD
EXISTING WATER VALVE
N
SCALE: 1"=100'
0 50 100
THURSTON COUNTY
BM: #280
ELEV: 190.05
INTERSTATE 5
3' HIGH BARBED WIRE FENCE
KIMMIE ROAD
189.69 WELL
190.30 WELL
193.05 WELL
188.20 WELL
194.88 WELL
193.80 WELL
188.71 WELL
6' HIGH CHAIN LINK FENCE
6' HIGH WOOD BOARD FENCE

33

APPENDIX F-1

SCHEMATIC STORMWATER DESIGN AND SITE GRADING, KIMMIE ST. PROPERTY, AHBL INC., 2014; UPDATED SCHEMATIC STORMWATER PLAN 2016

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December 11, 2014

Mr. Thomas Skjervold
Environmental Programs Manager
Washington Military Department
Building 36, Quartermaster Road
Camp Murray, WA 98430-5050

Project: Thurston County Readiness Center Site Feasibility Study, AHBL No. 2140515.10
Subject: Schematic Stormwater Design and Site Grading

Dear Tom:

We are pleased to provide you with this schematic grading and drainage plan and summary letter for the Thurston County Readiness Center Site located at Kimmie St SW and 83rd Avenue SW. Our schematic design and analysis presents data and findings relative to the physical and regulatory opportunities and constraints affecting development.

The focus of this analysis is on the high groundwater conditions and their impact on stormwater management facilities and site grading. The methodology used to complete the study included review of projects in the vicinity, correspondence with the City of Tumwater review engineer, review of stormwater drainage requirements for areas with seasonally high groundwater, GIS topography, and other research as necessary.

The factors influencing final development potential are provided below.

Site Description

The subject property is located in the City of Tumwater in a portion of Section 16, Township 17 North, and Range 2 West. The project site is identified as Thurston County Parcel Nos. 5185001200, 09520003000, and 09520004000. The parcels are situated between Interstate 5 and Kimmie Street SW.

The 3 parcel areas are 35.97 acres, 9.71 acres, and 1.57 acres for a total site area of 47.25 acres. The residential parcel was neglected in this analysis. The subject parcels of land are predominantly undeveloped and forested.

The northern parcel 09520003000 has access to the intersection of Kimmie St and 83rd Avenue SW. The Large parcel 5185001200 has access to Kimmie St south of the intersection with Burns Dr SW.

Civil Engineers

Structural Engineers

Landscape Architects

Community Planners

Land Surveyors

Neighbors

TACOMA

2215 North 30th Street
Suite 300
Tacoma, WA 98403-3350
253.383.2422 TEL
253.383.2572 FAX

www.ahbl.com

Proposed Project

The proposed project is to develop the site with two buildings consisting of a Readiness Center and a vehicle storage shed. A conceptual plan of the site has been prepared depicting the location of the two buildings, driveways, privately owned vehicle (POV) parking lot, and Military Vehicle Parking. The conceptual property site plan and enlarged site plan are included as Figures C-1 and C-2. The proposed development includes a two-story Readiness Center with a footprint of 56,650 SF, a 55,094 SF vehicle storage shed, and an 18,834 SY Military Vehicle Parking area. There are a total of 242 parking spaces proposed for POV parking in a 9,100 SY lot. A single-access driveway to the site is provided at the east side of the parcel at to Kimmie Street SW. This analysis does not consider impervious areas from future developments.

Topography

The site is undulating and gently slopes from south to north; existing grades range between ±195.00 and ±185.00.

Soils

Well logs were dug by Arcadia Drilling Inc. in 2008 for ground water monitoring at the project site. The well logs encounter brown silty sands in the top 24 inches overlaying sandy gravels.

Storm Drainage

The City of Tumwater has adopted the *2010 Drainage Design and Erosion Control Manual (DDECM)*. Development of the site will require that the stormwater be controlled and treated to meet water quality requirements. Flow control options include detention and release to a downstream conveyance system, flow dispersion, and infiltration to the site's subsoils.

A conveyance system is not available near the project site; therefore a combination of flow dispersion and infiltration is the preferred method of disposal of stormwater. The design of these facilities will meet minimum requirements in Tumwater's *2010 DDECM.* The requirements which apply to this site include:

Regulatory Requirements for Full Dispersion: Tumwater has adopted LID stormwater management requirements, prescribed in section 2.2.8, Volume V, of the 2010 DDECM.

- Retain 65% of site as native vegetation (approximately 31 acres of 47 acre site).
- Only 10% of site impervious can be dispersed (maximum 4.5 acres of approximately 8.5 acres of proposed impervious surfaces)
- Dispersion shall follow design guidelines for roof downspouts (LID.04) and driveway dispersion (LID.06 and LID .07).

Regulatory Requirements for Infiltration: The site lies to the north of the Salmon Creek Basin. Historical flooding problems within the Salmon Creek Basin have occurred due to high groundwater. Because of high groundwater, Tumwater has adopted stricter drainage requirements, categorized in Section 2.3.2, Volume III, of the *2010 DDECM.*

- The base of all infiltration basins or trench systems shall be a minimum of 6 feet above known or estimated high groundwater levels. This elevation may be determined using groundwater monitoring data gathered through a minimum of one wet period (December through April). Per conversations with the city 3.0' separation is allowed with a mounding analysis.

- A mounding analysis is required to determine the impact of groundwater mounding on the estimated design infiltration rate, and the known or estimated high groundwater elevation at the property boundary and at any onsite or offsite features that might be impacted by groundwater mounding.

- The mounding analysis must demonstrate there will be no breakout of groundwater to the surface in the vicinity of the project.

- A minimum separation to groundwater from the building foundation will be at least 3 feet.

- The increase in groundwater level at the property boundary due to mounding is less than 1 foot.

High Groundwater: A groundwater regression analysis was prepared by Robinson, Noble & Saltbush, Inc in 2008 at the project site. A color Map, "Depth to High Groundwater Conditions" was prepared indicating groundwater levels sloping from south to north from elevation 191 to elevation 188. The preliminary study concluded that additional analysis should be performed to achieve a "finite-difference numerical model of the site and surrounding area".

Analysis of High Groundwater Impacts: The development scenario has established minimum design grades for finish floor elevations, paving elevations, and storm pond bottom elevations to maintain separation from high groundwater. Refer to Figure C-3 for a schematic stormwater basin map.

The maximum separation from groundwater occurs near the central portion of the site. This area is also the least wooded area of the site. This is the recommended location for this development. The existing grades range between elevation 191 and 194 and groundwater is assumed to be at elevation 189. This location also minimizes the length of access road required to connect to Kimmie Avenue.

Full Dispersion and Infiltration Systems: Based on preliminary review of the dispersion BMP's we have assumed that 150LF of paved surfaces on the north and south side of the site can sheet flow to native vegetation and will not require additional stormwater management. Stormwater retention basins have been located as close to the source of runoff as practical to minimize facility depth for the remaining development area. We have assumed a minimum depth to finish floor of 4.25' will be required to drain to the stormwater retention facilities. Factors during design may require additional system depth which would require the site to be raised. We anticipate that finish floors of 6' above high groundwater should accommodate the unknowns.

Building Finish Floors: Two structures are proposed with this development. A vehicle storage shed and the readiness center building. The minimum finish floor elevation for these buildings per Tumwater requirements is 3 feet above the assumed or known high groundwater elevation. Based on our schematic stormwater concept the finish floors must be at least 4.25′ above the assumed high groundwater elevation and may by up to 6.0′ above groundwater.

BUILDING	FF = 4.25′ ABOVE G.W.	FF = 6.0′ ABOVE G.W.	TOTAL FILL	IMPORTED FILL ($20)	BORROW PIT FILL ($6)
READINESS CENTER	BALANCE	3,000 CY FILL	3,000 CY	$60,000	$18,000
VEHICLE SHED	3,000 CY FILL	7,000 CY FILL	10,000 CY	$200,000	$60,000

Fill costs may be mitigated by creating an onsite borrow pit for use as structural fill. A geotechnical engineer will need to review the site soils and provide recommendations for suitability of onsite soils for structural fill. Assuming onsite soils are not suitable the import costs for buildings could be approximately $200,000.00.

Please note that sewer system may control the building finish floor. We have not evaluated the location or depth of sewer serving the site. If sufficient depth is not available to gravity sewer from the site then the buildings either need to be raised or install a force main sewer and pump to discharge sewer flows. We have evaluated a range of fill scenarios using the minimum 4.25′ separation and a preferred 6′ separation.

Conclusion

Due to the size of the development full flow dispersion techniques cannot mitigate all of the stormwater runoff generated from the site. Therefore building finish floors will need to be set sufficiently high to accommodate drainage conveyance to ponds or trenches. Stormwater facilities need to maintain 2.5′ to 3.0′ of separation from groundwater. We anticipate that imported fill costs for the buildings could range between $60,000 to $200,000.00.

If you have any questions regarding this report, please do not hesitate to contact me at (253) 383-2422.

Sincerely,

J. Matthew Weber, PE
Principal

STK/lsk

Enclosures:

Figure C-1 – Property Site Plan

Figure C-2 – Enlarged Site Plan
Figure C-3 – Schematic Stormwater Basin Map
Figure C-4 - Calculations

This study is limited in scope. The statements and observations were derived from secondary information provided by local service providers. There may be additional information, records, or legal documents pertaining to the subject property that were not available to us during this feasibility assessment.

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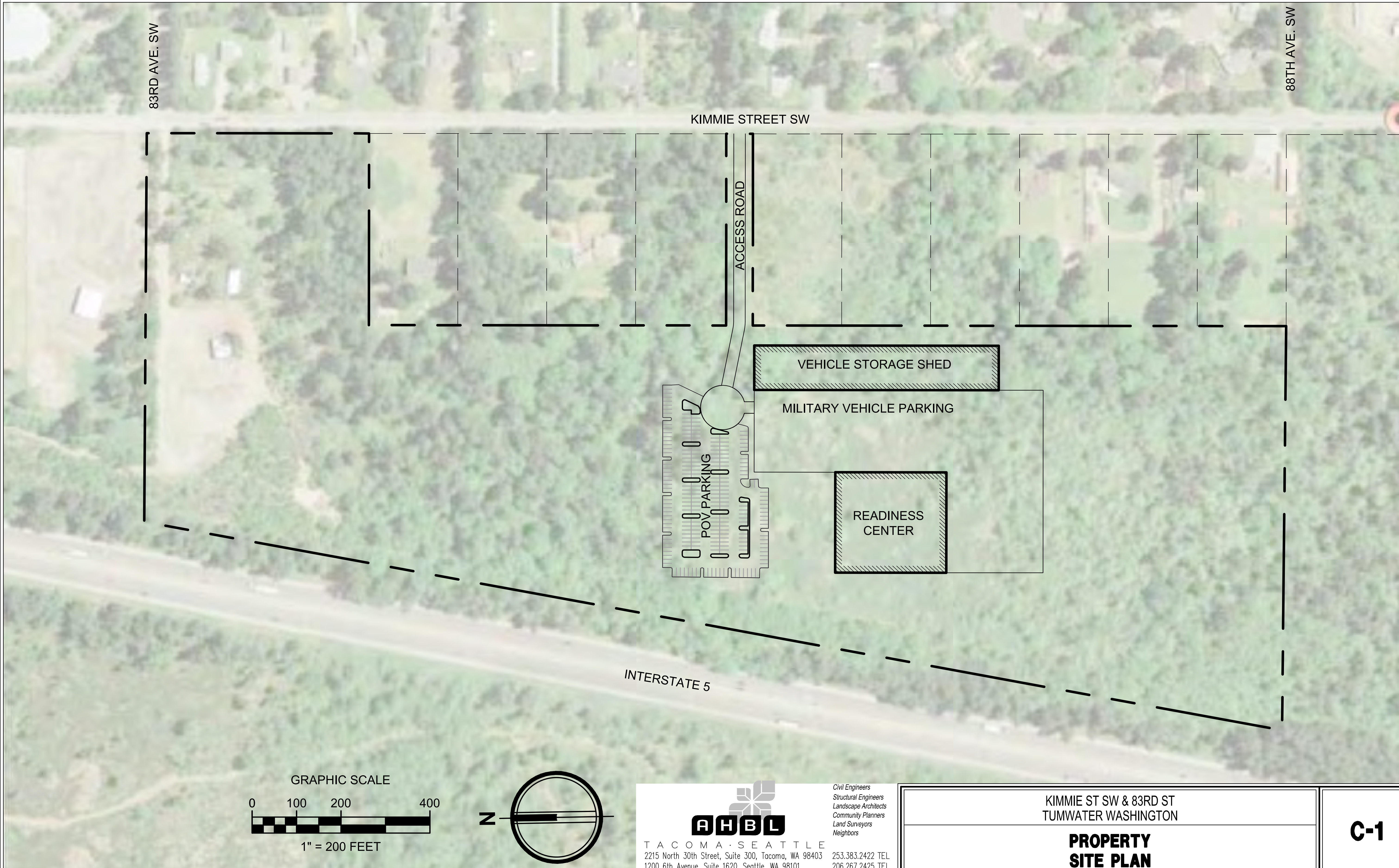


AHBL
TACOMA · SEATTLE
2215 North 30th Street, Suite 300, Tacoma, WA 98403
1200 6th Avenue, Suite 1620, Seattle, WA 98101

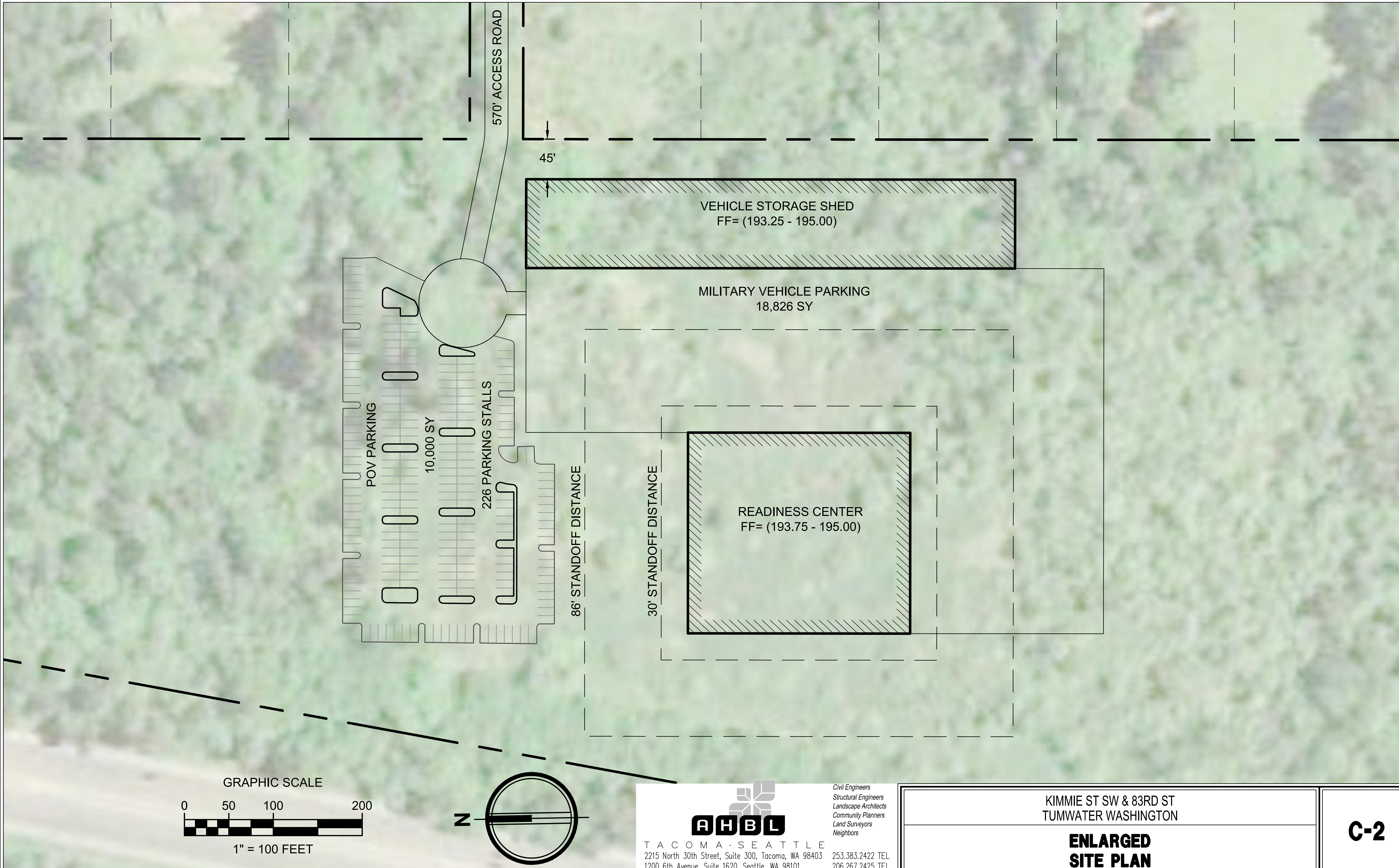
Civil Engineers
Structural Engineers
Landscape Architects
Community Planners
Land Surveyors
Neighbors

253.383.2422 TEL
206.267.2425 TEL

KIMMIE ST SW & 83RD ST
TUMWATER WASHINGTON

PROPERTY SITE PLAN

C-1



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1200 6th Avenue, Suite 1620, Seattle, WA 98101 206.267.2425 TEL

Civil Engineers
Structural Engineers
Landscape Architects
Community Planners
Land Surveyors
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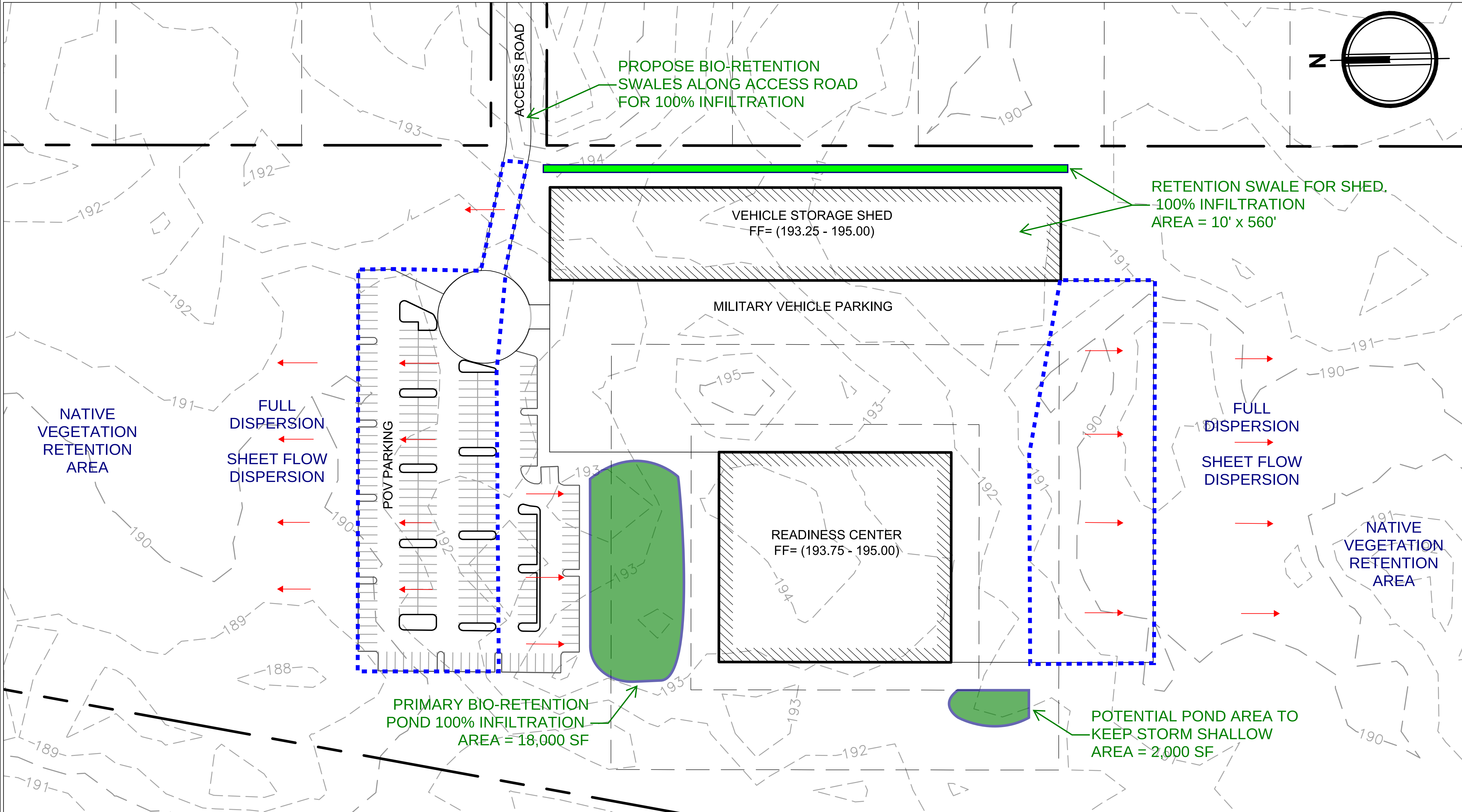
KIMMIE ST SW & 83RD ST
TUMWATER WASHINGTON

ENLARGED SITE PLAN

C-2



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1200 6th Avenue, Suite 1620, Seattle, WA 98101 206.267.2425 TEL

KIMMIE ST SW & 83RD ST
TUMWATER WASHINGTON

SCHEMATIC STORMWATER BASIN MAP

C-3

Project KIMMIE ST
Subject
With/To
Address
Date 12/10/2014

Project No. 2140515.10
Phone
Fax #
Faxed Pages
By KAUL

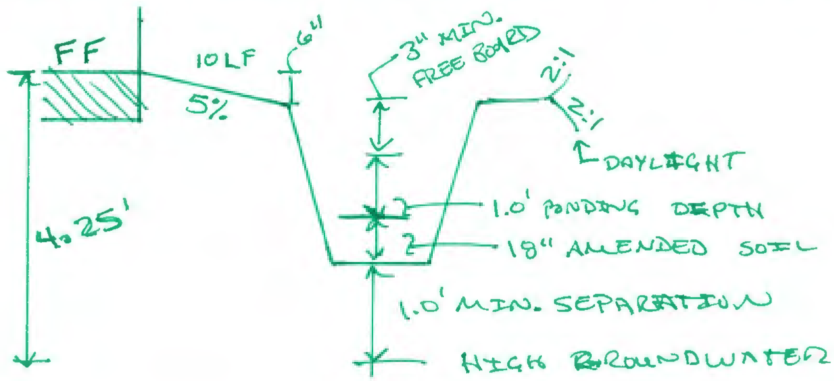
☐ Page ____ of ____
☐ Calculations
☐ Fax
☐ Memorandum
☐ Meeting Minutes
☐ Telephone Memo

AHBL

Civil Engineers
Structural Engineers
Landscape Architects
Community Planners
Land Surveyors
Neighbors

EVALUATE BLDG FILL :

ROOF DRAINS INTO BIO-RETENTION SWALE OR PONDS



BASED ON MINIMUM FF (4.25' ABOVE GROUNDWATER)

☐ READINESS CENTER COULD BE SET APPROXIMATELY AT GRADE (NO IMPORT)
MORE IDEAL SEPARATION WOULD BE 6.0' ABOVE G.W.
WOULD REQUIRE APPROXIMATELY 3,000 CY FILL

☐ VEHICLE STORAGE SHED ~~COULD BE~~ REQUIRES FILL

MINIMUM FILL ~ 3,000 CY (4.25' ABOVE G.W.)
PREFERRED FILL ELEV ~ 7,000 CY (6.00' ABOVE G.W.)

PROJECT ESTIMATED FILL FOR BLDGS 3,000 CY TO 10,000 CY

MINE SITE FOR FILL (BORROW PIT) ⇒ $6 /CY
IMPORT FILL TO SITE ⇒ $ 20 / CY

COST RANGE ⇒ $18,000 TO $ 200,000
3,000 @ $6 10,000 @ $20

APPENDIX F-2

UPDATED SCHEMATIC STORMWATER DESIGN AND SITE GRADING, KIMMIE ST. PROPERTY, SCHREIBER STARLING WHITEHEAD 2017

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SSW ARCHITECTS
SCHREIBER
STARLING
WHITEHEAD
901 FIFTH AVE NO 3100
SEATTLE, WA 98164
206-682-8300
SSWARCHITECTS.COM

February 23, 2017

Ms. Rowena Valencia-Gica Ph.D.
Environmental Specialist
Washington Military Department
36 Quartermaster Road
Camp Murray, WA 98430

Subject: **Tumwater Readiness Center: Civil Design Narrative**
Agreement No. 2016-008 A (1)

Dear Rowena:

To assist your Environmental Assessment re-submittal effort, our civil engineer AHBL provided the following narrative in our Schematic Design package:

Site Development – The project site is located in the City of Tumwater in Thurston County. The site is located along Kimmie Road due west of the Olympia Regional Airport. The site is bounded by Frontage Road to the north, Kimmie Road and a number of residential parcels to the east, undeveloped land to the south, and Interstate 5 to the west. The site is situated on six parcels totaling 53 acres in size. The site is largely undeveloped with the exception of gravel surfacing near the north side of the site. The site has two access points to Kimmie Road via parcel 09520003000 and parcel 51850000400.

Proposed project improvements include constructing a readiness center building and vehicle storage shed as well as POV parking lots and concrete plaza and pedestrian walkways. Frontage improvements are not anticipated at this time.

Grading – The existing site elevations range from approximately 183 feet to 197 feet, although the site is relatively flat with average elevations in the low 190's. An east – west ridge divides the site into two basins. The south basin drains to a wetland near the south property line. The north basin drains to a depression near the northwest corner of the site. The proposed development will be located in the northern portion of the site. The finished floor elevation of the readiness center building and vehicle storage shed are proposed at 196.

Existing site grades will be modified as a result of project improvements, but slope patterns will generally remain as is. Removal of unsuitable soils and import of structural fill is anticipated. It is unlikely that the site grading will result in a balanced condition and a net import is anticipated.

The proposed project will exceed one acre of disturbed area and will require an NPDES General Construction Stormwater permit. The contractor will be required to follow the project's stormwater pollution prevention plan and install temporary erosion and sediment control best management practices (BMPs).

Stormwater – All stormwater improvements will be designed to meet the requirements of the latest edition of the City of Tumwater Drainage and Erosion Control Manual. Stormwater from the project site currently discharges at two depressions on site. One is located at the south property line and the other near the northwest corner of the site. The project proposes to infiltrate all storm water by means of pervious pavement and rain gardens for roof downspouts. This strategy is intended to mimic the predeveloped condition as closely as possible and comply with the storm water drainage manual.

Treatment is required for runoff from pollutant generating surfaces such as drive aisles and parking spaces. This will be provided by a sand filtration layer in the subgrade of the pervious pavements.

Flow control is required for stormwater runoff and primarily will be provided by the infiltration methods described above. High groundwater is present on the site and as such, facilities will be placed at elevations required to achieve separation from groundwater. In addition, a groundwater mounding analysis will be required that shows compliance with the Tumwater Drainage Manual and that we are not excessively impacting groundwater elevations outside the site boundaries.

The stormwater conveyance system will consist of catch basins, enclosed pipes, and open conveyance in a series of bioretention cells and swales. It will also consist of a roof drainage network for runoff from the new buildings.

Onsite stormwater management BMPs will be required where feasible and appropriate. This is being addressed by directing stormwater runoff to a series of bioretention facilities (rain gardens).

Roads and Surfacing –The project site is bordered to the east by Kimmie Road. In general access will be from Kimmie Road and extend onto the site to the POV parking areas as well as the paved area between the readiness center facility and the vehicle storage shed. All vehicle pavements are proposed as pervious concrete. Site walkways and plazas are proposed as standard concrete.

Water and Fire Suppression Services – The project site is located within the City of Tumwater water service area. An existing 16-inch diameter PVC water main is located under Kimmie Street.

Water improvements will include a new 8 inch water main loop around the proposed readiness center building that connects to the water main in Kimmie Road. The water main loop will be approximately 1,600 linear feet. Improvements also include a new domestic service for the new readiness center building. Fire suppression improvements will include installing a new 6 inch service line, remote fire department connection, and four fire hydrants.

Sanitary Sewer Service – Sewer service is provided by the City of Tumwater. A 12" PVC sanitary main is located in Kimmie Street. The proposed connection point is at a manhole located at the intersection of Kimmie Street and 83rd Avenue SW and is roughly 14 feet deep. The project proposes roughly 750 feet of 8" pipe to be extended onsite to serve the proposed buildings. Two oil water separators, a grease interceptor, and trench drains are anticipated to be connected to the sewer system.

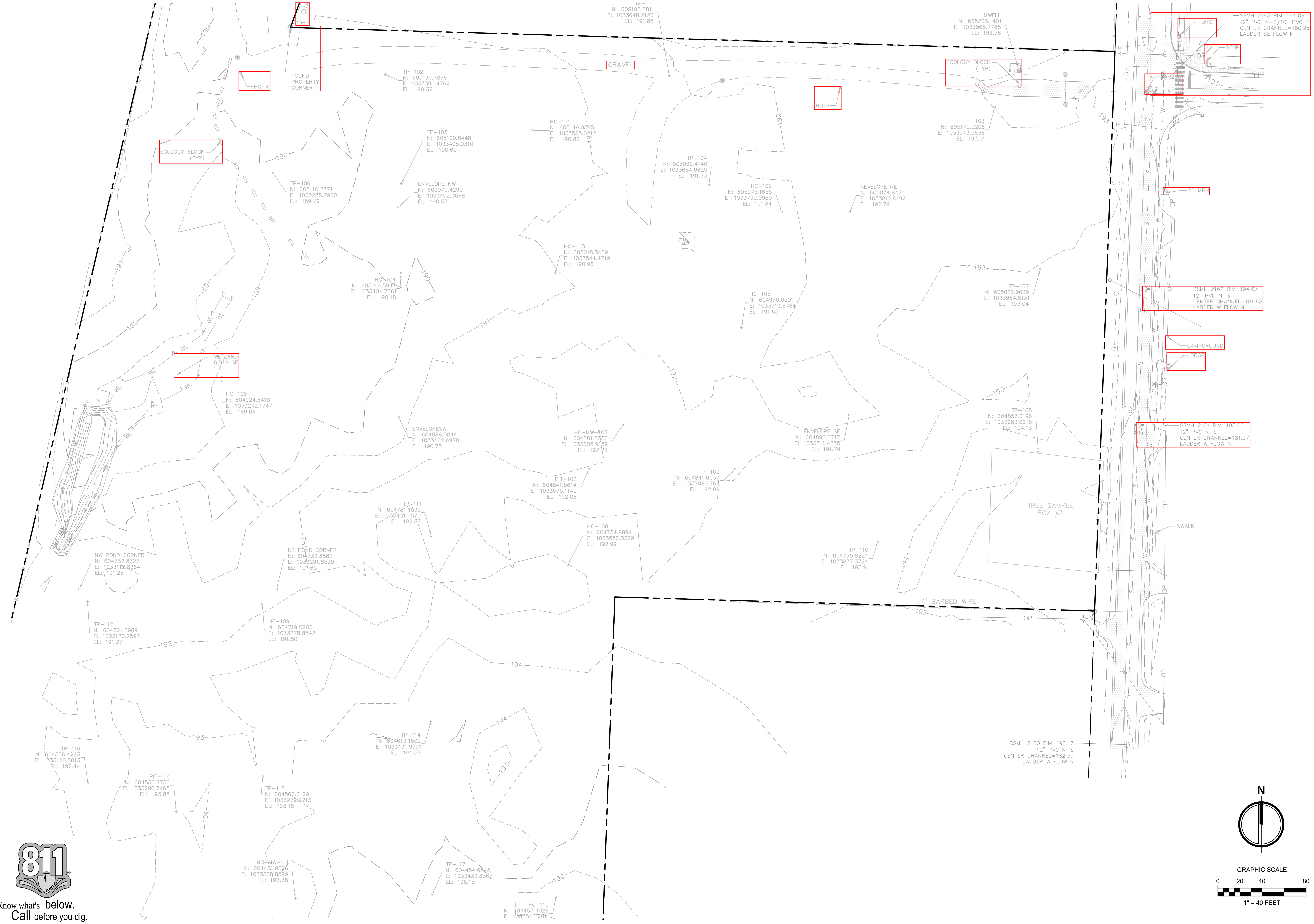
Natural Gas Service – Natural gas service is provided to the site by Puget Sound Energy through a 4 inch main located in Kimmie Street SW. Approximately 375 feet of gas service and a meter will be required to be extended to the readiness center.

We trust this is a sufficient description of our civil design approach, and in particular our approach to stormwater, now that the site development is focused on the north end of the Tumwater property. Should you need additional information, please do not hesitate to ask.

Respectfully,

Ross Whitehead AIA PRINCIPAL
Schreiber Starling Whitehead Architects

encl:	Schematic Design Drawings C1.0 (Existing Conditions & Demolition Plan), C1.1 (Civil Grading & Storm Drainage Plan), and C1.2 (Civil Utility Plan)
cc:	Ron Cross; Thomas Skjervold; Dino Othieno
File:	15018/COR



Know what's below. Call before you dig.

N

GRAPHIC SCALE

0 20 40 80

1" = 40 FEET

SSW ARCHITECTS

SCHREIBER STARLING WHITEHEAD

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PRELIMINARY

AHBL

TACOMA · SEATTLE · SPOKANE · TRI-CITIES

2215 North 30th Street, Suite 300 Tacoma, WA 98403
253.383.2422 TEL 253.383.2572 FAX www.ahbl.com WEB

Tumwater Readiness Center

Schematic Design

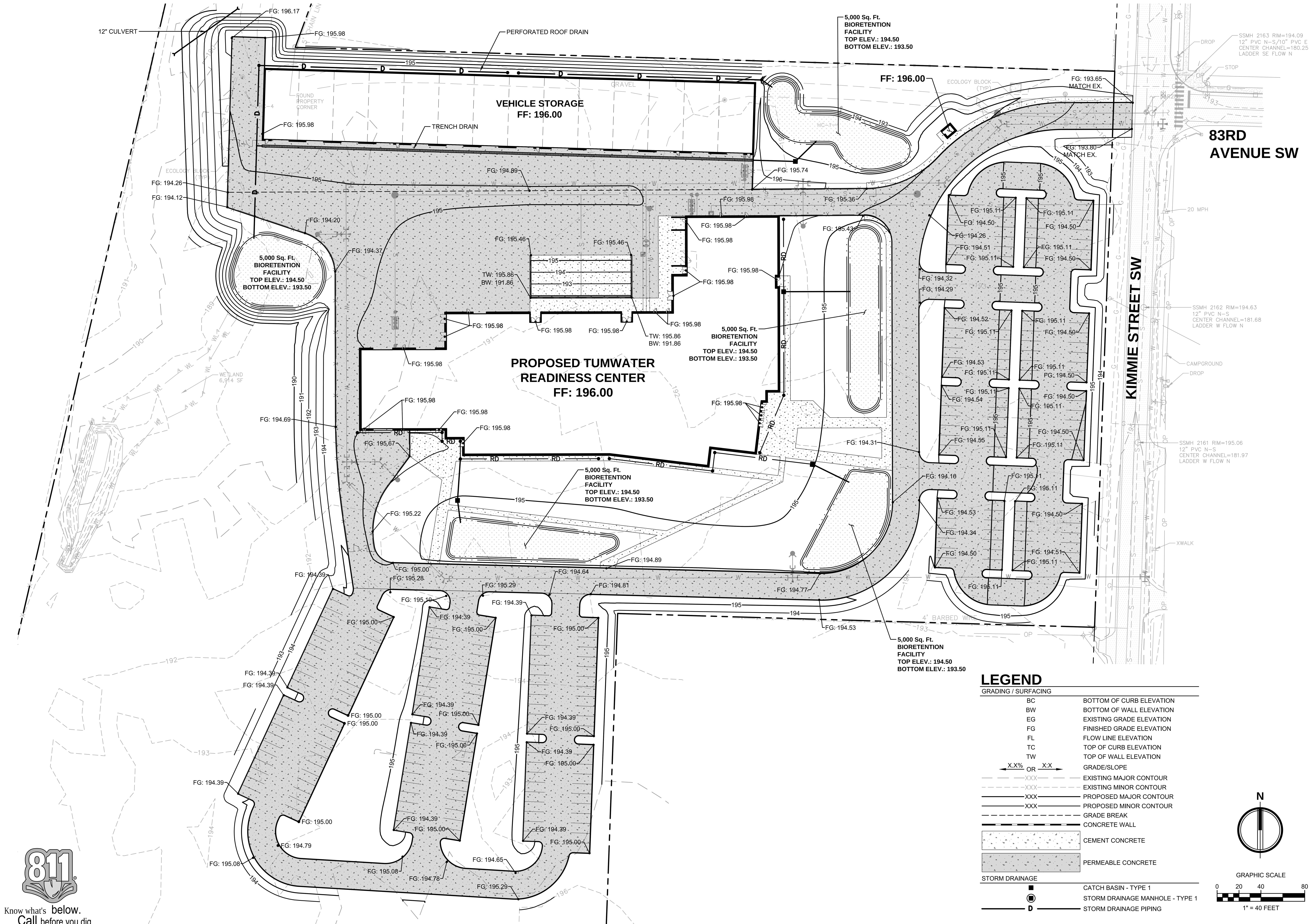
SCHEMATIC DESIGN EXISTING CONDITIONS AND DEMOLITION PLAN

Client Project No.: 2150623.10

SSW Architects Project No.: 15018

Date: 09.2016

C1.0



LEGEND

GRADING / SURFACING

BC	BOTTOM OF CURB ELEVATION
BW	BOTTOM OF WALL ELEVATION
EG	EXISTING GRADE ELEVATION
FG	FINISHED GRADE ELEVATION
FL	FLOW LINE ELEVATION
TC	TOP OF CURB ELEVATION
TW	TOP OF WALL ELEVATION
X.X% OR X:X	GRADE/SLOPE
XXX	EXISTING MAJOR CONTOUR
XXX	EXISTING MINOR CONTOUR
XXX	PROPOSED MAJOR CONTOUR
XXX	PROPOSED MINOR CONTOUR
	GRADE BREAK
	CONCRETE WALL
	CEMENT CONCRETE
	PERMEABLE CONCRETE

STORM DRAINAGE

■	CATCH BASIN - TYPE 1
◙	STORM DRAINAGE MANHOLE - TYPE 1
D	STORM DRAINAGE PIPING

N

GRAPHIC SCALE

0 20 40 80

1" = 40 FEET

Know what's below. Call before you dig.

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PRELIMINARY

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2215 North 30th Street, Suite 300 Tacoma, WA 98403

Tumwater Readiness Center

Schematic Design

SCHEMATIC DESIGN CIVIL GRADING AND STORM DRAIAGE PLAN

Client Project No.: 2150623.10

SSW Architects Project No.: 15018

Date: 09.2016

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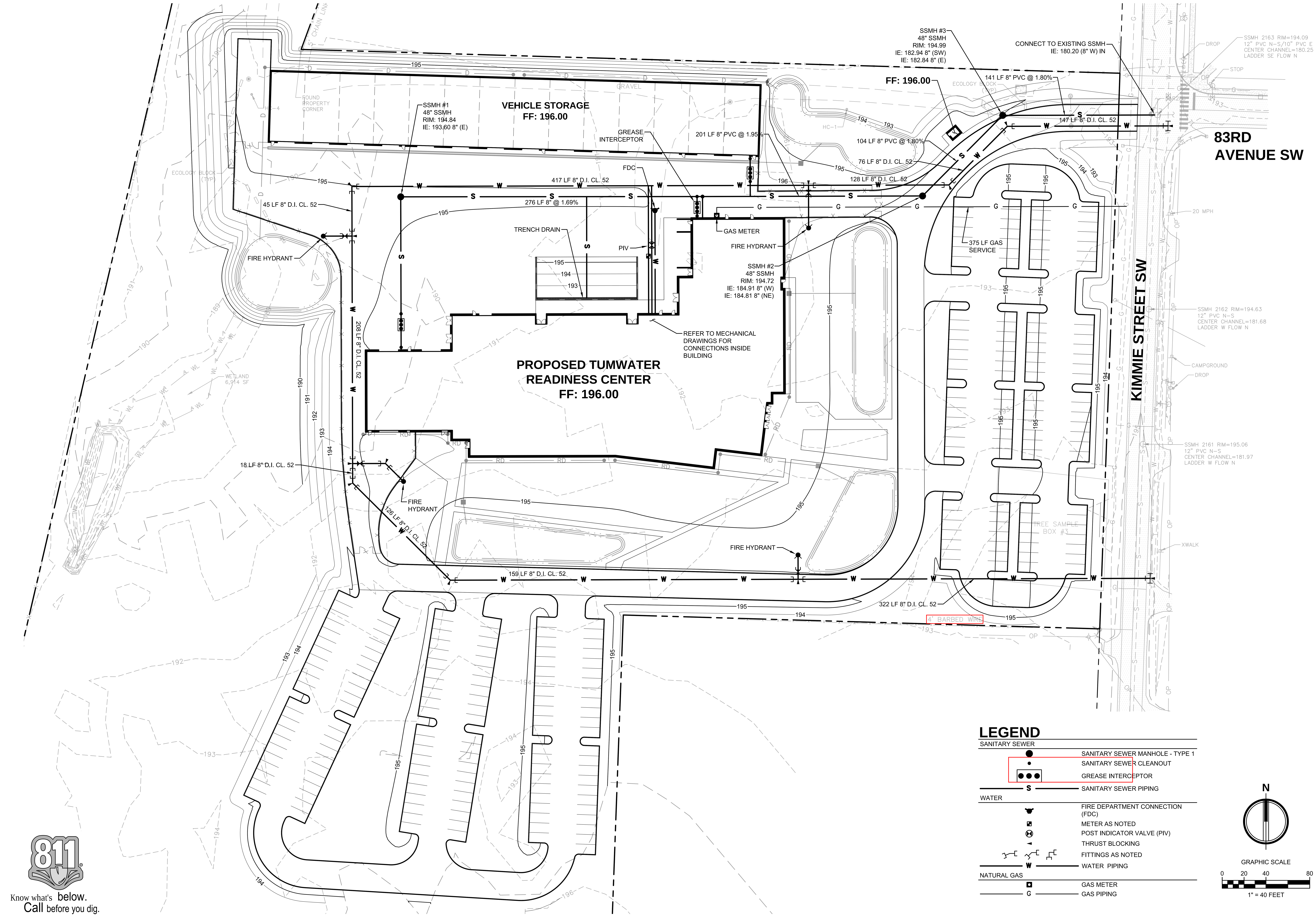

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2215 North 30th Street, Suite 300 Tacoma, WA 98403
253.383.2422 TEL 253.383.2572 FAX www.ahbl.com WEB

Tumwater Readiness Center

Schematic Design

SCHEMATIC DESIGN
CIVIL UTILITY PLAN

Client Project No.: 2150623.10

SSW Architects Project No.: 15018

Date: 09.2016

C1.2

APPENDIX G-1

PRELIMINARY GEOTECHNICAL ENGINEERING REPORT, SOUTH SOUND GEOTECHNICAL CONSULTING, 2015

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Preliminary Geotechnical Engineering Report
Army Readiness Center
Tumwater, Washington

Prepared for: AHBL, Inc.
2215 North 30th Street, Suite 300
Tacoma, WA 98403-3350

Attention: Ms. Lisa Klein

Prepared by: *South Sound* Geotechnical Consulting
P.O. Box 39500
Lakewood, WA 98496

January 16, 2015

January 15, 2015

AHBL, Inc.
2215 North 30th Street, Suite 300
Tacoma, WA 98403-3350

Attention: Ms. Lisa Klein

Subject: Preliminary Geotechnical Engineering Report
Army Readiness Center
Kimmie Street SW
Tumwater Washington
SSGC Project Number: 15001

Dear Ms. Klein:

South Sound Geotechnical Consulting (SSGC) has prepared this geotechnical engineering report regarding the above referenced project. Our services have been completed in general conformance with our proposal (P14080) dated December 21, 2014 and authorized per AHBL's Subconsultant Agreement dated January 7, 2015. The purpose of our services was to characterize subsurface conditions on the property to assess general geotechnical issues in support of siting the proposed development. Our scope of services included completing ten (10) test pits on the site, engineering analyses, and preparation of this report.

We appreciate the opportunity to work with you on this project. Please contact us if we can be of further assistance.

Respectfully,

South Sound Geotechnical Consulting

Timothy H. Roberts, P.E., R.G.
Member/Geotechnical Engineer

Table of Contents

EXECUTIVE SUMMARY

South Sound Geotechnical Consulting (SSGC) has completed a preliminary geotechnical evaluation for the planned Army Readiness Center on Kimmie Street SW in Tumwater, Washington. Based on subsurface conditions observed in the explorations completed, most of the site is considered feasible for development from a geotechnical perspective. The following information is intended only as a summary of geotechnical considerations for development of the site:

- Site Conditions: The site is generally level with elevation changes on the order of 5 (+/-) feet. Previous development including buildings is in the northern portion, with mixed forest growth and open areas covering the remainder of the property.

- Soil Conditions: Native soils generally consisted of sand with variable silt. Coarser gravelly sand was observed below the finer sand in test pits in the northern portion of the site.

- Groundwater: Groundwater was observed in two test pits at the time of excavation. Depth to groundwater was on the order of 11 feet in the south-central portion of the site, and at about 7 feet in the most northern test pit.

- Foundations: Conventional spread footings founded on properly prepared native subgrades and/or structural fill are considered suitable for support of planned buildings. Footings on properly prepared subgrades can be designed for an allowable bearing pressure of 1,500 pounds per square foot (psf).

- Floor Slabs: Floor slabs may be supported on properly prepared native subgrades or structural fill. Floor slab design should include a 6-inch minimum thickness capillary break.

- Seismic Considerations: Site Class D is recommended to classify the site per the 2012 International Building Code (IBC) with the subsurface information available. Site soils are considered to have a low risk to liquefaction.

- Pavements: Properly prepared native soils and/or structural fill should provide suitable support for pavement sections. Conventional asphalt concrete pavement sections should include a minimum 4 inches of HMA over 6 inches of crushed top or base course in main access lanes. A minimum pavement section of 2 inches HMA over 4 inches of crushed course can be used in car parking areas.

This executive summary should not be used for design and/or construction purposes. The entire report must be read for a comprehensive understanding of the information and recommendations presented as specific details are not included or fully developed in this executive summary.

PROJECT INFORMATION

A Readiness Center is being proposed on the approximate 50 (+) acre property to the west of Kimmie Street SW and east of Interstate I-5, between 83rd Avenue SW and 88th Avenue SW in Tumwater, Washington. Planned improvements currently include a Readiness Center building and paved parking. A vehicle storage/maintenance building may be constructed in the future. We understand that about 16 acres of the property will be required for the planned development.

The property is reported to have a generally high groundwater table with isolated wetlands in the southern portion. We have been provided with maps of observed water levels across the property which suggest that the general center portion would be the most feasible to develop. Standing surface water had been mapped in the northern and southern portions of the site.

We anticipate that the Readiness Center will be a one to two-story building. Conventional spread footing foundations and slab-on-grade floors are planned. Asphalt paved access ways and parking is anticipated. Access would be from Kimmie Street SW via a designated easement near the center of the property.

SITE CONDITIONS

Most of the site is undeveloped and covered with mixed forest growth and open areas vegetated with thick brush and weeds. The very northern portion of the property has several abandoned buildings and asphalt paved areas. The site is generally considered level with elevation changes estimated on the order of 5 (+/-) feet. A ditch along the western portion of the property adjacent I-5 was estimated at about 8 to 10 feet deep. Standing water was in the ditch and estimated at about 7 feet below adjacent grades.

SUBSURFACE CONDITIONS

Subsurface conditions on the lot were completed by excavating ten (10) test pits on January 14, 2015. Approximate test pit locations are shown on Figure 1, Site Plan. Test pits were advanced to depths ranging from about 5.5 to 11.5 feet below surface grades. A summary description of observed subgrade conditions is provided below, with complete logs of the explorations provided in Appendix A. Please note that subsurface conditions can vary across the site from those observed at the exploration locations.

Soil Conditions

Topsoil/Forest duff was observed in most of the test pits and ranged from about 6 to 12 inches. In several of the test pits, thick roots were observed to about 18 inches below the surface. Native soils below the topsoil consisted predominantly of silty sand to sand with trace to some silt. These soils were generally in a loose to medium dense condition. Silt content was observed to be variable, and some soil layers were interpreted to be sandy silt. Coarser sand and gravel soils were observed in the northern portion of the site (test pits TP-9 and TP-10) below an upper silty sand layer. These granular soils were typically in a medium dense to dense condition.

Fill consisting of mixed silt, sand, and gravel was observed below the surface in test pit TP-10, in the previous developed portion of the site. Fill was generally in a medium dense condition and was about 1.5 feet thick. We anticipate that fill will be present in other areas of the developed portion of the site and will vary in thickness. .

Groundwater Conditions

Groundwater was observed in two of the test pits at the time of excavation. Groundwater level was at about 7 feet in test pit TP-10 in the northwest portion of the site, and at about 11 feet in test pit TP-7 in the southwest portion of the planned development area. Iron oxide staining in test pit TP-10 suggests groundwater levels may rise to a depth of about 6 feet. Groundwater levels should be expected to fluctuate on the order of 1 to 2 feet due to seasonal precipitation patterns, and off- and on-site drainage sources.

Geologic Setting

Soils on the site have been classified by the Washington State Department of Natural Resources (DNR). The mapping of the area is presented in the "Geologic Map of the Maytown 7.5-minute Quadrangle, Thurston County, Washington" issued in 2009. Surface geology is mapped as Vashon recessional outwash. This unit is described as "Sand and silt with minor gravel interbeds". Soils observed in the test pits are interpreted to be glacial outwash.

GEOTECHNICAL DESIGN CONSIDERATIONS

It is our opinion, from a geotechnical perspective, that the majority the property (encompassed within our field explorations) is considered suitable for development. Observed soil conditions are considered feasible for support of buildings and roads. Groundwater was observed in only two test pits at depths between 7 and 11 feet below the surface and should not adversely impact development of the central and northern portions of the property. Additional explorations are recommended to assess the southernmost portion of the site, if future development is planned in that area.

Native silty sand soils were generally in a loose condition near the surface. As such, subgrade preparation methods in building and pavement areas will require adequate compaction to achieve suitable subgrades for support of planned improvements. Alternatively, localized removal and replacement with structural fill may be needed.

Recommendations presented in the following sections are based upon the subsurface conditions observed in the explorations completed and our current understanding of project plans. Recommendations presented in this report assume that finish grades will be near existing grades. It should be noted that subsurface conditions across the site may vary from those described on the test pit logs, and can change with time. Therefore, proper site preparation will depend upon the weather and soil conditions encountered at the time of construction. We recommend that SSGC review final plans and assess subgrade conditions for foundations, floor slabs, and pavements at the time of construction.

Site Preparation

Preparation for site grading and earthwork should include procedures intended to drain ponded water and control surface water runoff. Grading the site without adequate drainage control measures may negatively impact site soils, resulting in increased export of impacted soil and import of fill materials. This can potentially increase the cost of the earthwork and subgrade preparation phases of the project.

Site grading should include removal (stripping) of topsoil, thick roots, and any fill in building and pavement areas. Stripping depths of topsoil will range from about 6 inches to perhaps 1.5 to 2 feet where thicker roots are encountered. Foundation and pavement subgrades should consist of undisturbed native soils following stripping.

Subgrade Preparation

After stripping the site to suitable native subgrades, we recommend that exposed building and pavement subgrades are proofrolled using a large roller or other mechanical compaction equipment to assess subgrade conditions. Proofrolling efforts should result in the upper 2 feet of subgrade soils achieving a firm, unyielding condition, and at least 95 percent of the maximum dry density (MDD) per the ASTM D1557 test method. Wet, loose, or soft subgrades should be compacted or removed and replaced with structural fill. A representative of SSGC should be present to assess subgrade conditions during proofrolling.

Grading and Drainage

Positive drainage should be provided during construction and maintained throughout the life of the development. Allowing surface water into foundation excavations or utility trenches should be prevented during construction.

Roof downspouts should discharge into an approved stormwater receptor or onto splash blocks or extensions when the ground surface is not protected by exterior slabs or paving. Sprinkler systems should not be installed within five feet of foundation elements. Landscaped irrigation adjacent to foundations should be minimized or eliminated.

Structural Fill Materials

The suitability of soil for use as structural fill depends primarily on the gradation and moisture content of the soil when it is placed. As the amount of fines (soil fraction passing the U.S. No. 200 sieve) increases, soils can become increasingly sensitive to small changes in moisture content. It is often difficult to achieve adequate compaction if soil moisture is outside of optimum condition for soils that contain more than 5 percent fines. In general, optimum moisture is within about +/- 2 percent of the moisture content required to achieve the maximum density per the ASTM D-1557 test method.

Site Soils: Observed finer grained soils (upper silty sand) will be difficult to use as their higher fines (silt and/or clay) content make them moisture sensitive. Under optimum moisture conditions, they could be used as structural fill. Native gravelly sand to sandy gravel soils are considered suitable for structural fill.

Structural Fill Materials: We recommend that import structural fill placed during dry weather periods consist of material which meets the specifications for *Gravel Borrow* as described in Section 9-03.14(1) of the 2014 Washington State Department of Transportation (WSDOT) Specifications for Road, Bridge, and Municipal Construction (Publication M 41-10). Gravel Borrow should be placed in horizontal lifts not exceeding 10 inches in loose thickness. Each lift must be conditioned to the proper moisture content and uniformly compacted to a firm, unyielding condition using mechanical equipment. Gravel Borrow fill must be protected from disturbance if exposed to wet conditions after placement.

During wet weather, or for backfill on wet subgrades, import soil suitable for compaction in wet conditions should be provided. Imported fill for use in wet conditions should generally conform to specifications for *Select Borrow* as described in Section 9-03.14(2), or *Crushed Surfacing* per Section 9-03.9(3) of the 2014 WSDOT M-41 manual, with the modification that a maximum of 5 percent by weight shall pass the U.S. No. 200 sieve for these soil types.

Placement of structural fill is often weather-dependent. Delays due to inclement weather are common, even when using select granular fill. We recommend that site grading and earthwork be scheduled for the drier months of the year. Structural fill should not consist of frozen material.

Structural Fill Placement

We recommend that structural fill is placed in lifts not exceeding 10 inches in loose measure. It may be necessary to adjust lift thickness based on site and fill conditions during placement and compaction. Structural fill should be compacted to attain the recommended levels presented in Table 1, Compaction Criteria.

Table 1. Compaction Criteria

Fill Application	Compaction Criteria*
Footing areas (below structures and retaining walls)	95 %
Upper 2 feet in pavement areas, slabs and sidewalks, and utility trenches	95 %
Below 2 feet in pavement areas, slabs and sidewalks, and utility trenches	92 %
Utility trenches or general fill in non-paved or -building areas	90 %

*Per the ASTM D 1557 test method.

Trench backfill within about 2 feet of utility lines should not be over-compacted to reduce the risk of damage to the line. In some instances the top of the utility line may be within 2 feet of the surface. Backfill in these circumstances should be compacted to a firm and unyielding condition.

We recommend that all fill procedures include maintaining grades that promote drainage and do not allow for ponding of water within the fill area. The contractor should protect compacted fill subgrades from disturbance during wet weather. In the event of rain during structural fill placement, the exposed fill surface should be allowed to dry prior to placement of additional fill. Alternatively, the wet soil can be removed. We recommend that consideration be given to protecting haul routes and other high traffic areas with free-draining granular fill material (i.e. sand and gravel containing less than 5 percent fines) or quarry spalls to reduce the potential for disturbance to the subgrade during inclement weather.

Earthwork Procedures

Conventional earthmoving equipment should be suitable for earthwork at this site. Subgrade soils that become disturbed due to elevated moisture conditions should be overexcavated to expose firm, non-yielding, non-organic soils and backfilled with compacted structural fill. We recommend that the earthwork portion of this project be completed during extended periods of dry weather. If earthwork is completed during the wet season (typically November through May) it may be necessary to take extra precautionary measures to protect subgrade soils. Wet season earthwork may require additional mitigative measures beyond that which would be expected during the drier months of the year.

If earthwork takes place during freezing conditions, we recommend that the exposed subgrade be allowed to thaw and be recompacted prior to placing subsequent lifts of structural fill. Alternatively, the frozen soil can be removed by excavation to unfrozen soil and replaced with non-frozen structural fill.

The contractor is responsible for designing and constructing stable, temporary excavations (including utility trenches) as required to maintain stability of both the excavation sides and bottom. Excavations should be sloped or shored in the interest of safety following local, state, and federal regulations, including current OSHA excavation and trench safety standards.

A qualified geotechnical engineer and materials testing firm should be retained during the construction phase of the project to observe earthwork operations and to perform necessary tests and observations during subgrade preparation, placement and compaction of structural fill, backfilling of excavations, and prior to construction of foundations.

Permanent Cut and Fill Slopes

We recommend that permanent cut and fill slopes have a maximum inclination of 2H:1V (Horizontal:Vertical).

Foundations

Foundations can be placed on firm native soils or on structural fill that have been prepared as described in this report. The following recommendations have been prepared for conventional spread footing foundations.

Bearing Capacity (net allowable):	1,500 pounds per square foot (psf) for footings supported on native soils or structural fill prepared as described in this report.
Footing Width (Minimum):	18 inches (Strip) 24 inches (Column)
Embedment Depth (Minimum):	18 inches (Exterior) 12 inches (Interior)
Settlement:	Total: < 1 inch Differential: < 1/2 inch over 30 foot span
Allowable Lateral Passive Resistance:	275 psf/ft* (below 12 inches)
Allowable Coefficient of Friction:	0.35* (Native soils and structural fill)

*These values include a factor of safety of approximately 1.5

The net allowable bearing pressures presented above may be increased by one-third to resist transient, dynamic loads such as wind or seismic forces. Lateral resistance to footings should be ignored in the upper 12-inches from exterior finish grade.

Foundation Construction Considerations

All foundation subgrades should be free of water and loose soil prior to placing concrete, and should be prepared as recommended in this report. Concrete should be placed soon after excavating and compaction to reduce disturbance to bearing soils. Should soils at foundation level become excessively dry, disturbed, saturated, or frozen, the affected soil should be removed prior to placing concrete. We recommend that SSGC observe all foundation subgrades prior to placement of concrete.

We recommend that a working surface of sand and gravel (e.g. base course) is placed on the finer grained subgrade soil in footing areas. This layer should be at least 4 inches thick and compacted to a firm and unyielding condition. The purpose of this layer is to protect the more sensitive fine grained native soil from disturbance during form and rebar placement.

Foundation Drainage

We recommend that footing drains are installed around new perimeter footings of buildings. Footing drains should include a minimum 4-inch diameter perforated rigid plastic or metal drain line installed at the base of the footing. The perforated drain lines should be connected to a tight line pipe that discharges to an approved storm drain receptor. The drain line should be surrounded by a zone of clean, free-draining granular material having less than 5 percent passing the No. 200 sieve or meeting the requirements of section 9-03.12(2) "Gravel Backfill for Walls" in the 2014 WSDOT Standard Specifications for Road, Bridge, and Municipal Construction manual (M41-10). The free-draining aggregate zone should be at least 12 inches wide and wrapped in filter fabric. The granular fill should extend to within 6 inches of final grade where it should be capped with compacted fill containing sufficient fines to reduce infiltration of surface water into the footing drains. Alternately, the ground surface can be paved with asphalt or concrete. Cleanouts are recommended for maintenance of the drain system.

Seismic Considerations

The following seismic parameters and values presented in Table 2 are recommended based on the 2012 International Building Code (IBC).

Table 2. Seismic Parameters

PARAMETER	VALUE
2012 International Building Code (IBC) Site Classification[1]	D
Site Latitude	N 46.96281°
Site Longitude	W 122.93209°
S_s Spectral Acceleration for a Short Period	1.298g
S_1 Spectral Acceleration for a 1-Second Period	0.54g
F_a Site Coefficient for a Short Period	1.0
F_v Site Coefficient for a 1-Second Period	1.5

[1] Note: In general accordance with *2012 International Building Code,* Section 1613.3.2 for risk categories I,II,III. IBC Site Class is based on the specified characteristics of the upper 100 feet of the subsurface profile. S_s, S_1, F_a, and F_v values based on the USGS US Seismic Design Maps website using referenced site latitude and longitude. The 2012 IBC requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. Test pits completed on the site do not satisfy the required 100 foot soil profile determination. The recommended seismic site class considers that a stiff soil profile continues below the maximum depth of the test pits and is based on the referenced maps in this report and other geologic information in the area.

Liquefaction

Soil liquefaction is a condition where loose, typically granular soils located below the groundwater surface lose strength during ground shaking, and is often associated with earthquakes. The risk of liquefaction at this site is low for the design level earthquake based on the Washington DNR's Olympia-Lacey-Tumwater Urban Area, Washington: Liquefaction Susceptibility Map (GM-47), dated 1999.

On-Grade Floor Slabs

On-grade floor slabs should be placed on native soils or structural fill prepared as described in this report. We recommend a modulus subgrade reaction of 175 pounds per square inch per inch (psi/in) for firm native soils and structural fill.

We recommend that a capillary break is provided between the prepared subgrade and bottom of slab. Capillary break material should be a minimum of 6 inches thick and consist of compacted clean, free-draining, well graded course sand and gravel. The capillary break material should contain less than 5 percent fines, based on that soil fraction passing the U.S. No. 4 sieve. Alternatively, a clean angular gravel such as No. 7 aggregate per Section 9-03.1(4) C of the 2014 WSDOT (M41-10) manual could be used for this purpose.

We recommend that positive separations and/or isolation joints are provided between slabs and foundations, and columns or utility lines to allow independent movement, where needed. Backfill in interior trenches beneath slabs should be compacted in accordance with recommendations presented in this report.

A vapor retarder should be considered beneath concrete slabs that will be covered with moisture sensitive or impervious coverings (such as tile, wood, etc.), or when the slab will support equipment or stored materials sensitive to moisture. We recommend that the slab designer refer to ACI 302 and/or ACI 360 for procedures and limitations regarding the use and placement of vapor retarders.

Pavements

We understand that concrete asphalt pavements will be used for access ways and parking areas. Subgrades for pavement areas should be prepared as described in the site and subgrade preparation and structural fill sections of this report. Subgrade soils below pavements should be compacted to at least 95 percent of the maximum dry density (ASTM D 1557) within at least one foot of the base of the section. Subgrades below pavement sections should also be graded or crowned to promote drainage and not allow for ponding of water beneath the section. If drainage is not provided and ponding occurs, the subgrade soils could become saturated, lose strength, and result in premature distress to the pavement. In addition, the pavement surfacing should also be graded to promote drainage and reduce the potential for ponding of water on the pavement surface.

Pavement section design has been prepared and is based on AASHTO design guidelines and the following assumed design parameters:

- 20-year life span;
- Estimated design life Equivalent Single Axle Loads (18 kips) of 250,000;
- Estimated subgrade CBR of 4;
- Terminal serviceability of 2.0; and,
- Level of reliability 85 percent.

Minimum recommended pavement sections for conventional pavement areas include:

Table 3. Preliminary Pavement Sections

Traffic Area	Preliminary Recommended Minimum Pavement Section Thickness (inches)	
	Asphalt Concrete Surface[1]	Top/Base Course[2]
Light Duty (Car Parking)	2	4
Access Ways	4	6

[1] 1/2 –inch nominal aggregate hot-mix asphalt per WSDOT 9-03.8(1)
[2] Top or base course per WSDOT 9-03.9(3)

The above recommended pavement sections should only be considered for preliminary design purposes. Final pavement sections should be based on actual traffic design loads. The estimated CBR value may not be suitable depending on final road subgrades which could affect the preliminary pavement sections. When traffic loads and final pavement subgrade elevations are known, SSGC should review and verify or modify the preliminary pavement sections.

Pavement Maintenance

The performance and lifespan of pavements can be significantly impacted by future maintenance. The above pavement sections represent minimum recommended thicknesses and, as such, periodic maintenance should be completed. Proper maintenance will slow the rate of pavement deterioration, and will improve pavement performance and life. Preventive maintenance consists of both localized maintenance (crack and joint sealing and patching) and global maintenance (surface sealing). Added maintenance measures should be anticipated over the lifetime of the pavement section if any existing fill or topsoil is left in-place beneath pavement sections.

REPORT CONDITIONS

This report has been prepared for the exclusive use of AHBL, Inc as discussed and has been prepared in accordance with generally accepted geotechnical engineering practices in the area. No warranties, either express or implied, are intended or made. Site safety and earthwork construction procedures are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless SSGC reviews the changes and either verifies or modifies the conclusions of this report in writing.

The analysis and recommendations presented in this report are based upon the data obtained from the test pits completed at the indicated locations and from other information as discussed. This report does not reflect variations of subsurface conditions that may occur between explorations, across the site, or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided, as warranted.

The scope of services for this project does not include any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for contamination or pollution, other studies should be completed.

Legend

TP - 1

Approximate Test Pit Location

Scale: NTS

Base map from "Property Site Plan" by AHBL.

South Sound Geotechnical Consulting
P.O. Box 39500
Lakewood, WA 98496
(253) 973-0515

Figure 1 – Site Plan

Army Readiness Center
Tumwater, WA

SSGC Project #15001

Appendix A

Field Exploration Procedures and Logs

Field Exploration Procedures

Our field exploration for this project included ten (10) test pits completed on January 14, 2015. The approximate exploration locations are shown on the Site Plan (Figure 1). The exploration locations were determined by pacing from site features. Ground surface elevations referenced on the logs were inferred from USGS 7.5 minute quadrangle maps. Exploration locations and elevations should be considered accurate only to the degree implied by the means and methods used.

A private excavating contractor subcontracted to SSGC excavated the test pits. Soil samples were collected and stored in moisture tight containers for further identification and laboratory testing. Test pits were backfilled with excavated soils and tamped when completed. Please note that backfill in the test pits will likely settle with time. Should test pits be discovered in building or pavement areas, the backfilled material should be re-excavated and recompacted, or replaced with structural fill.

The following logs indicate the observed lithology of soils and other materials observed in the explorations at the time of excavation. Where a soil contact was observed to be gradational, our log indicates the average contact depth. Our logs also indicate the approximate depth to groundwater (where observed at the time of excavation), along with sample numbers and approximate sample depths. Soil descriptions on the logs are based on the Unified Soil Classification System.

Test Pit TP-1

Depth (feet)	Material Description
0 – 0.75	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
0.75 – 4	Silty SAND: Loose, moist, brown. (Sample S-1 @ 2 feet)
4 – 6	SAND with silt: Loose, moist, light brown. (Sample S-2 @ 5 feet)
6 – 10.5	Sand with trace to some silt: Medium dense, moist, light gray/brown. (Sample S-3 @ 6.5 feet)

Test pit completed at approximately 10.5 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 200 feet

Test Pit TP-2

Depth (feet)	Material Description
0 – 1	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown. Moderate roots to 1.5 feet.
1 – 2	SAND with silt: Loose, moist, light brown.
2 – 8	Silty SAND: Loose to medium dense, moist, grayish brown.
8 – 10	Silty SAND: Medium dense, moist, gray/brown mottled.
10 – 10.5	SAND with silt: Medium dense, moist, gray. (Sample S-1 @ 10.5 feet)

Test pit completed at approximately 10.5 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 201 feet

Test Pit TP-3

Depth (feet)	Material Description
0 – 1	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
1 – 5	SAND with silt: Medium dense, moist, brown.
5 – 7	Silty SAND: Medium dense, moist, grayish brown.
7 – 10	Sand with trace to some silt: Medium dense, moist, brown/gray.

Test pit completed at approximately 10 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 203 feet

Test Pit TP-4

Depth (feet)	Material Description
0 – 1	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
1 – 3	Silty SAND: Loose, moist, brown.
3 – 5.5	SAND with silt: Loose, moist, brown/gray.

Test pit completed at approximately 5.5 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 203 feet

Test Pit TP-5

Depth (feet)	Material Description
0 – 0.75	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
0.75 – 6	Silty SAND: Loose, moist, brown.
6 – 7.5	Silty SAND/Sandy SILT: Medium dense, moist, gray/brown.
7.5 - 10	SAND with silt: Medium dense, moist, brownish gray. Grades gray at about 9 feet.

Test pit completed at approximately 10 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 203 feet

Test Pit TP-6

Depth (feet)	Material Description
0 – 0.75	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown. Heavy roots to 1.5 feet.
0.75 – 4	Silty SAND: Loose, moist, brown.
4 – 7	Silty SAND/Sandy SILT: Loose to medium dense, moist, grayish brown with some mottling.
7 – 9.5	SAND with silt: Medium dense, moist, brownish gray.

Test pit completed at approximately 9.5 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 203 feet

Test Pit TP-7

Depth (feet)	Material Description
0 – 0.5	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
0.5 – 6	Silty SAND: Loose to medium dense, moist, brown.
6 – 7	SAND with silt: Medium dense, moist, grayish brown.
7 – 11.5	Silty SAND: Medium dense, moist, gray.

Test pit completed at approximately 11.5 feet on 1/14/15.
Groundwater observed at about 11.5 feet at time of excavation.
No caving observed.
Approximate surface elevation: 201 feet

Test Pit TP-8

Depth (feet)	Material Description
0 – 0.5	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown. Moderate roots to 1 foot.
0.5 – 5	Silty SAND: Loose to medium dense, moist, brown.
5 – 6.5	SAND with silt: Medium dense, moist, brownish gray.
6.5 – 9.5	Silty SAND/Sandy SILT: Medium dense, moist, gray with slight mottling.
9.5 - 11	SAND with silt: Medium dense, moist, gray.

Test pit completed at approximately 11 feet on 1/14/15.
No groundwater observed at time of excavation.
No caving observed.
Approximate surface elevation: 201 feet

Test Pit TP-9

Depth (feet)	Material Description
0 – 0.75	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
0.75 – 3	Silty SAND: Loose, moist, light brown.
3 – 9.5	Gravelly SAND with trace to some silt: Medium dense to dense, moist, brown/gray. (Sample S-1 @ 4 feet)

Test pit completed at approximately 9.5 feet on 1/14/15.
No groundwater observed at time of excavation.
Moderate caving below 4 feet.
Approximate surface elevation: 200 feet

Test Pit TP-10

Depth (feet)	Material Description
0 – 1.5	**Fill:** Gravelly SAND with some silt: Medium dense, moist, brown.
1.5 – 2.5	Silty SAND: Loose, moist, light brown.
2.5 – 6	Gravelly SAND with trace to some silt: Medium dense to dense, moist, brown grading to gray.
6 – 6.5	Gravelly SAND with trace to some silt: Medium dense, moist, red oxidized.
6.5 – 7.5	Sandy GRAVEL with trace silt: Loose to medium dense, reddish brown.

Test pit completed at approximately 7.5 feet on 1/14/15.
Groundwater observed at about 7 feet at time of excavation.
Slight caving below 3.5 feet.
Approximate surface elevation: 200 feet

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
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 [C]	Cu ≥ 4 and 1 ≤ Cc ≤ 3 [E]	GW	Well-graded gravel [F]
			Cu < 4 and/or 1 > Cc > 3 [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel [F,G,H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu ≥ 6 and 1 ≤ Cc ≤ 3 [E]	SW	Well-graded sand [I]
			Cu < 6 and/or 1 > Cc > 3 [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G,H,I]
			Fines classify as CL or CH	SC	Clayey sand [G,H,I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots on or above "A" line [J]	CL	Lean clay [K,L,M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OL	Organic clay [K,L,M,N]
					Organic silt [K,L,M,O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K,L,M]
			PI plots below "A" line	MH	Elastic Silt [K,L,M]
		Organic:	Liquid limit - oven dried / Liquid limit - not dried < 0.75	OH	Organic clay [K,L,M,P]
					Organic silt [K,L,M,Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains ≥ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.

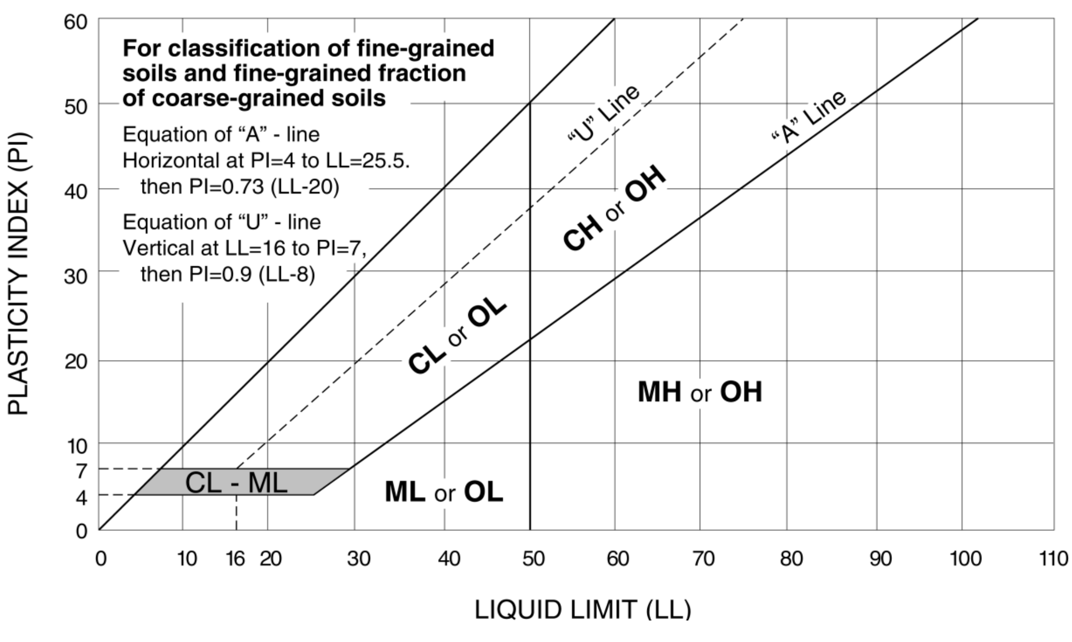
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI ≥ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



APPENDIX G-2

FINAL GEOTECHNICAL ENGINEERING REPORT, HARTCROWSWER, 2017

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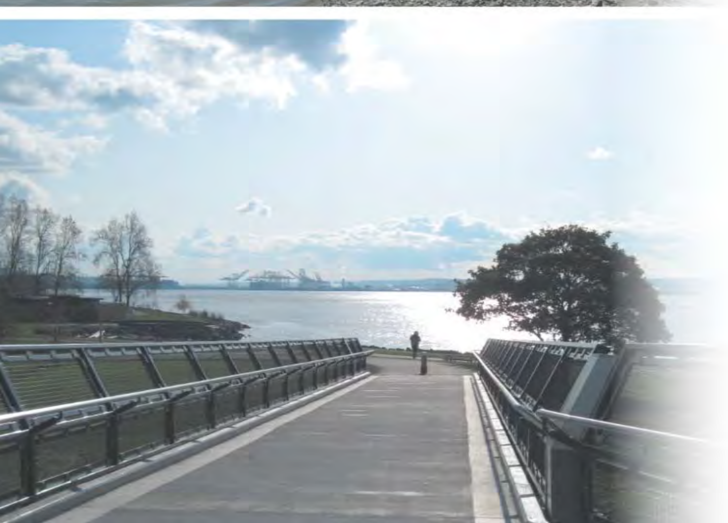

Geotechnical Engineering Design Report

Tumwater Readiness Center

Tumwater, WA

Prepared for

Schreiber Starling Whitehead Architects

February 10, 2017
19202-00

HARTCROWSER

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www.hartcrowser.com

Geotechnical Engineering Design Report

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February 10, 2017
19202-00

Prepared by

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Geotechnical Engineering Design Report

Tumwater Readiness Center

Tumwater, WA

This report presents the results of our geotechnical engineering design study for the Tumwater Readiness Center (TRC) north site option in Tumwater, WA. We previously completed a geotechnical engineering design study for the central site location and prepared a draft report dated April 5, 2016. Due to the presence of potentially liquefiable soils to a depth of about 25 feet at the central site location, a new design study was requested at the north site. The proposed readiness center includes a two-story building, military vehicle storage, military owned vehicle parking, and privately owned vehicle (POV) parking. A Vicinity Map showing the site location is on Figure 1.

This report is divided into several sections. Following the introduction, which describes the organization and purpose of this report, our principal geotechnical engineering design recommendations are organized as follows:

- Project Background;
- Geotechnical Engineering Conclusions and Recommendations; and
- Geotechnical Recommendations for Construction

Our scope of work included:

- Reviewing existing data and reports for the site;
- Advancing 15 borings (4 as part of the north site option preliminary investigation) and completing two borings as monitoring well installations;
- Excavating 18 test pits;
- Performing two pilot infiltration tests (PIT);
- Monitoring long-term groundwater levels through the balance of the 2016 rainy season in four on-site monitoring wells;
- Testing soil samples in our laboratory;
- Completing geotechnical engineering analyses; and
- Producing this geotechnical engineering design report.

We completed this work in general accordance with amendment 1 (dated September 12, 2016) to our original agreement (dated January 20, 2016). This report is for the exclusive use of Schreiber Starling Whitehead Architects[1]; Washington Military Department; and their consultants for specific application to the subject project and site. We completed this work in accordance with generally accepted geotechnical engineering practices for the nature and conditions of the work completed in the same or

[1] Previously Schreiber, Starling, and Lane Architects, PS

similar localities, at the time the work was performed. We make no other warranty, express or implied.

PROJECT BACKGROUND

Site and Project Description

Our understanding of the project is based on information provided in the State of Washington Military Department's Predesign Study dated April 20, 2015 and the north site option preliminary schematic design provided by Schreiber Starling Whitehead Architects dated September 29, 2016. The proposed TRC site is located in Tumwater, Washington immediately east of Interstate 5 between Exits 101 and 99 and west of the Olympia Regional Airport. Figure 1 shows the project location on a Vicinity Map. It is bounded by Frontage Road to the north, Kimmie Road and a number of residential properties to the east, undeveloped land to the south, and Interstate 5 to the west. The TRC site is 53 acres in size with proposed development covering approximately 10 acres at the north end of the site (Figure 2).

The overall site consists largely of undeveloped land which was selectively logged in the past and is now partially reforested. A trucking company occupied the north end of the property approximately 30 years ago, and several minor structures remain from that time.

The site topography over the proposed north site development area generally slopes gently down from east to west with elevations generally ranging between 189 and 194 feet, based on survey information provided by AHBL dated March 28, 2016. Note all elevations presented in this report are relative to NAVD 88.

We understand that the proposed TRC will consist of a two-story, approximately 82,000 gross-square-foot (gsf) readiness center building, military vehicle (MILV) parking, 29,700 gsf unheated vehicle storage building, and private vehicle (POV) parking. The proposed site plan is shown on Figure 2 (note this figure only depicts the northern portion of the site).

We performed our analysis and prepared this report in general accordance with the 2012 International Building Code (IBC).

Subsurface Conditions

Our understanding of the subsurface conditions is based on borings, test pits, and laboratory analyses performed by Hart Crowser for this project, as well as historical explorations in the project vicinity. The borings performed for this project include 4 borings conducted as part of the north site option preliminary investigation. The boring logs by Hart Crowser are provided in Appendix A and the laboratory analysis by Hart Crowser is in Appendix B. Historical exploration logs reviewed are included in Appendix C of this report. Results for infiltration testing performed on site are presented in Appendix D.

The subsurface information used for this study represents conditions at discrete locations across the project site and actual conditions in other areas could vary. Furthermore, the nature and extent of any variations may not become evident until additional explorations are performed or until construction begins. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

Soil Conditions

From June 29 to 30, 2016, we drilled four borings (HC-1 through HC-4) at the previously proposed approximate corners of the north site option structures. After this preliminary investigation, we conducted a full subsurface investigation of the north site option, advancing eleven additional borings (HC-101 through HC-MW-111) and excavating eighteen test pits (TP-101 through TP-118) across the project site from August 30 to September 9, 2016 . Locations of the soil explorations are illustrated on our site and exploration plan (Figure 2) and two subsurface cross section through the proposed development area are shown on Figure 3 and 4.

The soil layers observed during the field explorations program were broadly categorized based on their engineering properties into Engineering Soil Units (ESU). In general, the soils observed in the explorations consist of the following soil units, described in the order they were encountered from the ground surface down.

- **ESU 1 – Loose to Medium Dense Silty Sand.** From the ground surface to a depth generally ranging from 1 to 5 feet below ground surface (bgs) the borings encountered loose to medium dense, slightly silty to silty sand. In general, this layer is not suitable for foundation bearing and is susceptible to liquefaction under the IBC design earthquake conditions, during periods of high groundwater levels.

- **ESU 2 – Medium Dense to Dense Sandy Gravel.** A medium dense to dense sandy gravel unit was encountered directly under ESU 1 and extended to depths ranging from 20 to 27 feet bgs. This unit was generally observed to vary between sandy to very sandy gravel and very gravelly sand with trace amounts of silt and layers of sand. Isolated zones of this soil unit are potentially susceptible to liquefaction under the IBC design earthquake. As long as structures are designed to tolerate the estimated liquefaction-induced settlement, this layer is suitable for shallow foundation support.

- **ESU 3 – Dense to Very Dense Gravel.** A very dense sand and gravel unit was encountered directly under ESU 2. This unit was observed to be sandy to very sandy gravel with occasional sand seams that extended to the bottom of all the borings drilled. This layer does not appear to be susceptible to liquefaction under the IBC design earthquake and is suitable for foundation support.

Groundwater

Groundwater was encountered during drilling the borings for the north site option investigation. Groundwater levels observed at time of drilling (ATD) in HC-1 through HC-4 (preliminary investigation) ranged from about 7.5 to 11 feet bgs, or approximately elevation 181 to 184 feet. Groundwater levels observed ATD in the August and September investigation ranged from about 11.5 to 16.5 feet bgs, or approximately elevation 177 to 180 feet. Groundwater levels from measurements in monitoring wells are more reliable than ATD groundwater levels, as the groundwater takes time to reach equilibrium after the disturbance from drilling. Two existing wells are located at the north site location (Figure 2): a monitoring well (BAH-815) and a groundwater supply well (GSW-1). Measurements in these wells on June 30, 2016 during the preliminary investigation indicated groundwater was about 9 to 10 feet bgs, or approximately elevation 183 feet.

Note that the groundwater levels were obtained on the dates and at the times indicated on the logs. Groundwater elevations vary depending on location, season, and precipitation.

Long-term groundwater level monitoring was performed from the beginning of March through mid to late June 2016 at the central site location (summary memorandum provided August 4, 2016). This long-term groundwater data generally showed the central portion of the site had peak groundwater elevations of about 193 feet in mid-March 2016. Based on comparison of groundwater level measurements in BAH-821 at the central site location and in BAH-815 and GSW-1 at the north site location on June 30, 2016, it appears the groundwater at the north site is approximately 2 feet lower than at the central site location. Our preliminary estimate of the design groundwater level at the north end is about elevation 191 feet.

We did not assess conditions at the north end during our original fieldwork for the central site location. However, the preliminary design groundwater elevation of 191 feet appears approximately consistent with the surveyed ponded water elevation of 190 feet at the north end by AHBL on March 15, 2016. To better estimate what the design groundwater level should be at the north end, we have installed four pressure transducers, three in the north site and one in the central site, to provide long-term groundwater monitoring. The 2015-2016 rainy season appeared to be the wettest on record and produced higher groundwater levels than estimated from the previous regression analysis for the site. Because of this we will perform new regression analyses using the four newly installed transducers. We will provide a preliminary regression analysis after about two months of groundwater data collection and a final regression analysis utilizing data through the 2016-17 rainy season. This will allow us to better correlate north end groundwater elevations with those measured at the central site location from March 2016 through June 2016.

Upon completion of long-term groundwater level monitoring, we will provide a memo documenting our findings and any revised recommendations as necessary. In general, geotechnical analysis and recommendations are not likely to be significantly affected by minor changes in design groundwater following completion of long-term monitoring; however, infiltration and building siting may be affected.

Infiltration Testing

Our scope of work included two pilot infiltration tests (PITs). PIT-101 was conducted at a depth of 1.5 feet with the bottom of the test cell located within ESU 1 – slightly silty fine to medium sand. PIT-102 was conducted at a depth of 2 feet with the bottom of the test cell located within ESU 2 – very sandy gravel. The infiltration tests provided design infiltration rates in Table 1 below. Because ESU 1 was stripped from the test cell in PIT-102, design infiltration rates are representative more of the engineering soil unit in which they were conducted rather than their geographic location. If on-site soils are compacted due to construction related activities, design infiltration rates will be lower than provided below.

Table 1 – Design Infiltration Rates

PIT/ESU	Design Rate (inches/hour)
PIT-101 (ESU-1)	3
PIT-102 (ESU-2)	7

PITs were conducted from September 12 to 16, 2016 with results shown in Appendix D. Grain size distribution results from the PITs can be found on figure B-7 of Appendix B.

GEOTECHNICAL ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Our recommendations are based on our current understanding of the project and the subsurface conditions interpreted from explorations at and near the site by Hart Crowser and others. If the nature or location of the facilities is different than we have assumed, we should be notified so we can review, change, and/or confirm our recommendations.

Earthquake Engineering

The site is located in a seismically active area. In this section, we describe the seismic setting at the project site, provide seismic design parameters, and discuss earthquake-induced geotechnical hazards.

Seismic Setting

The seismicity of Western Washington is dominated by the Cascadia Subduction Zone, in which the offshore Juan de Fuca Plate subducts beneath the continental North American Plate. Three types of earthquakes are associated with subduction zones: intraslab subduction, interface subduction, and crustal earthquakes.

Subduction Zone Sources. The offshore Juan de Fuca Plate is subducting below the North American Plate. This causes two distinct types of events. Large-magnitude interface subduction earthquakes occur at shallow depths near the Washington coast at the interface between the two plates (e.g., the 1700 earthquake with magnitude of approximately 9.0). A deeper zone of seismicity is associated with bending of the Juan de Fuca Plate below the Puget Sound Region that produces intraslab subduction earthquakes at depths of 40 to 70 kilometers (e.g., the 1949, 1965, and 2001 earthquakes).

Crustal Sources. Recent fault trenching and seismic records in the Puget Sound area indicate a distinct shallow zone of crustal seismicity (e.g., Seattle and Tacoma Faults) which may have surficial expressions and can extend 25 to 30 kilometers deep.

Seismic Basis of Design

We understand that the TRC must meet the seismic design requirements of the 2012 IBC. The basis of structural design for the IBC is two-thirds of the hazard associated with the Risk-Targeted Maximum Considered Earthquake (MCE_R). The basis of soil liquefaction evaluation for the IBC is the Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration (PGA), which is not adjusted for targeted risk. The Maximum Considered Earthquake for IBC is a seismic event with 2 percent probability of exceedance in 50 years, which corresponds to an average return period of 2,475 years (often referred to as the 2,500-year event). The probability for such an event to occur during the design life of the structure is considered low. A design objective for the IBC earthquake is that if this event occurs, the structure may experience a major failure but still maintain life safety. Therefore, the structure should be designed to have adequate strength and ductility to prevent collapse. Note, however, that stricter performance criteria apply for essential facilities.

Seismic Design Parameters

We obtained the seismic hazard parameters for the MCE_R and MCE_G from the United States Geologic Survey U.S. Seismic Design Maps (USGS 2014) for the site location at Latitude 46.966 and Longitude –122.932. We provide the seismic design parameters in accordance with 2012 IBC in Table 2. Note that these parameters correspond to Soil Site Class B and should be adjusted for the actual site soil using the site coefficients provided.

Table 2 – 2012 IBC Seismic Design Parameters

Parameter	Value
Latitude	46.966
Longitude	–122.932
Site Class (based on liquefaction susceptibility / based on SPT) – see text	F / D
Assumed Design Site Class for Site Coefficients Provided – see text	D
Structural Design	
Risk Category	IV
Mapped MCE_R spectral response acceleration at short periods, S_S	1.299 g
Mapped MCE_R spectral response acceleration at 1-second periods, S_1	0.540 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.5
Liquefaction Evaluation	
Mapped MCE_G peak ground acceleration, PGA	0.5 g
Magnitude (mean) from USGS (2008) hazard deaggregation	7.42
Site Coefficient, F_{PGA}	1.0

As indicated in Table 1, the Site Class is F based on liquefaction potential (see the following section). According to IBC, structures built on Site Class F soil profiles require site-specific site response analysis. However, the IBC provides an exception for structures having fundamental periods of vibration equal to or less than 0.5 second on Site Class F soils; for these structures, a site class is permitted to be determined based on *in situ* testing and soil properties. Based on communication with the structural engineer (PCS) it is anticipated the building period will be 0.5 seconds or less. Assuming the IBC exception will apply, the soil site class was determined based on a weighted average of the blow counts observed to a depth of 100 feet bgs (extrapolating to 100 feet, as necessary). Based on this method for determining site class the site soils are Site Class D. We recommend using Site Class D for determining site coefficients. A site-specific site response analysis will be required if the building fundamental period of vibration is greater than 0.5 second.

Seismically Induced Geotechnical Hazards

Potential seismically induced geotechnical hazards may include surface rupture, liquefaction, lateral spreading, and landslides. Our review of these hazards is based on the soils encountered in our explorations, regional experience, and our knowledge of local seismicity.

Surface Rupture

There are no mapped crustal faults in the direct vicinity of Tumwater. However, there is a contribution to the seismic hazard from gridded crustal seismicity. Gridded sources (or seismicity-based background sources) estimate seismicity of unidentified or uncharacterized faults by spatially smoothing historical seismicity. In our opinion, the risk of surface rupture at this site is low.

Liquefaction and Settlement

Liquefaction is caused by a rapid increase in excess porewater pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils that rely on inter-particle friction for strength are susceptible to liquefaction until the excess porewater pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess porewater pressures dissipating upward, carrying soil particles with the draining water. In general, loose, saturated sandy soils with low silt and clay content are susceptible to liquefaction. Silty soils with low plasticity are also susceptible to liquefaction. For any soil type, the soil must be saturated for liquefaction to occur. Liquefaction can cause ground surface settlement and lateral spreading. We used empirical methods to estimate liquefaction potential based on the SPT data obtained at the site. We used SPT-based liquefaction triggering procedures after Idriss and Boulanger (2008).

Due to some uncertainty in the liquefaction triggering analysis methods, we assume soil layers with a factor of safety against liquefaction (FSliq) of less than or equal to 1.1 are assumed to be susceptible liquefy, while soil layers with an FSliq greater than 1.1 are generally considered not susceptible to liquefaction.

Subsurface conditions and our analyses indicate that the susceptibility of ESU 1 to liquefaction is high. Figures 3 and 4 present the results of the liquefaction triggering analysis along the subsurface profiles for the IBC earthquake.

Post-liquefaction settlement results from densification and redistribution of liquefiable soils after an earthquake. The ground surface may not settle uniformly across the area of liquefaction, so differential settlement may be greater than the difference in settlement predicted at each boring location. We estimated liquefaction-induced ground surface settlement using SPT corrections by Idriss and Boulanger (2008) and volumetric strain formulations by Ishihara and Yoshimine (1992). Estimates of liquefaction-induced settlement at the ground surface of the borings that fall within or near the building footprint are generally less than 2 inches. HC-105 showed slightly higher liquefaction induced settlement than the other 15 borings we reviewed and was considered to be an anomaly. This may be the influence of previous man-made activities such as a previous excavation and backfilling. Estimated liquefaction-induced settlements outside (primarily to the south) of the building area could be up to 3 to 4 inches.

If mitigation options to address liquefaction susceptibility are deemed necessary, ground improvement or deep foundations are possibilities.

Lateral Spreading

Lateral spreading is typically associated with lateral movement on sloping ground or ground near a steep slope, and is caused by liquefaction or a shear strength reduction of soils within or under a slope. Lateral spreading can affect a structure by increasing the lateral force exerted on the subsurface walls or within pile foundations. Given the generally flat site gradient, the risk of lateral spreading is considered low and does not warrant special design considerations.

Landslides

Based on the site location, slope inclination, and lack of reported landslides in the area, the landslide hazard at the site is considered low.

General Considerations

We understand from discussions with the civil engineer (AHBL) and review of the predesign documents that Thurston County requires the finish floor elevation of structures be a minimum of three feet above the high groundwater. Based on available groundwater measurements, we assume the high groundwater level at the site is at about elevation 191 feet (NAVD 88). Based on Thurston County requirements the minimum finish floor elevation is assumed to be 194 feet, which is close to the existing grades. However, for consideration of separation for infiltration facilities the minimum finish floor elevation is estimated to be 4.25 to 6 feet above the high groundwater level, or elevation 195.25 to 197 feet, which would require raising grades over significant portions of the site. Additionally, long-term monitoring is on-going, which could increase the high groundwater elevation and the proposed site grade elevations.

Building Foundations

Based on our field observations, subsurface explorations, and liquefaction susceptibility analysis and the essential facility classification, shallow foundations appear feasible for support of the TRC structures at the north site, if the structures are designed to tolerate the potential liquefaction-induced differential settlement following the design earthquake (approximately 2 inches). Discussions with the design team regarding the preliminary post-liquefaction settlement of shallow foundations indicates the building would meet both life safety and essential facilities functionality after the design earthquake. However, there may be the need for minor re-leveling of slabs, cosmetic repair of cracking, etc. If this potential, minor damage is not acceptable, we can assess options for mitigating liquefaction (e.g., ground improvement) or structurally supporting structures on non-liquefiable soils (e.g., deep foundations).

We recommend a maximum net allowable bearing capacity of 4,000 psf for foundation bearing on ESU 2 (Medium Dense to Dense Sandy Gravel), or structural fill if localized overexcavation is necessary. Note that a minimum net allowable bearing pressure of 2,000 psf is required by the National Guard Bureau. For typical buildings constructed at grade, foundations would typically bear approximately 3 feet below grade, which is near the top of the Medium Dense to Dense Sand and Gravel. Because the site grades will be raised to accommodate the high groundwater elevation, some local overexcavation and replacement of ESU 1 (Loose to Medium Dense Silty Sand) may be required prior to raising grade. Alternatively, shallow foundations could be constructed on the Medium Dense to Dense Sand and Gravel prior to raising the site grade with thicker footings or taller embedded columns.

Other recommendations for design of shallow footings include:

- Isolated and strip footings should have a minimum width of 24 and 18 inches, respectively. Place the base of all footings at least 18 inches below the lowest adjacent grade for consideration of frost protection.

- Depth of footings should also ensure that they are founded outside of an imaginary 1H:1V plane projected upward from the nearest bottom edge of adjacent footings or utility trenches.

Foundation Resistance to Lateral Loads

For shallow foundations, resistance to lateral loads is from passive soil resistance against the side(s) of the footing and frictional resistance along the base of the footing. For passive resistance to lateral loads, we recommend applying passive equivalent fluid pressure and sliding resistance using the values in Table 3. A factor of safety of at least 1.5 has been applied to these values. The allowable coefficient of friction provided assumes that the footing concrete is poured neat against the native soil. The allowable passive resistance assumes structural backfill will be placed in accordance with the recommendations in our report in the passive zone on the sides of the foundations.

Table 3 – Passive Resistance to Lateral Loads for Spread Foundations

Soil Type	Allowable Passive Equivalent Fluid Density	Allowable Coefficient of Friction
ESU 1	200 pcf above water 100 pcf below water	N/A[a]
ESU 2 or Structural Fill	300 pcf above water 150 pcf below water	0.3

Notes:
a. Bottom of foundations should not be located in ESU 1

Floor Slab

We anticipate that the floor slab for the TRC building will be constructed as a slab-on-grade. We recommend the floor slab be underlain by a capillary break/drainage layer.

In general the upper 0.5 to 2 feet of ESU 1 contains forest duff, topsoil, and organics throughout the site. Where encountered, we recommend overexcavating the portions of the near surface ESU 1 unit that contain debris or significant organic material (roots and organic silt) up to 2 feet deep below the bottom of the floor slab and replacing with structural fill (includes the thickness of the capillary break/drainage layer). Where structural fill will be used to raise grade, at a minimum near surface soils (approximately upper 0.5 feet) with the most significant organics should be stripped off, and the final depth from the top of the raised subgrade to the top of native soils with limited organics that may still be present should be at least 2 feet.

Subgrade conditions should be verified in the field by proof-rolling the subgrade soils. If soft spots are encountered during preparation of the subgrade, they should be overexcavated and replaced with structural fill. We make the following recommendations:

- Compact the capillary break/drainage layer beneath the slab-on-grade as described for compaction of structural fill in this report.

- We assume site grades will be raised and floor slabs will be constructed on structural fill. For preliminary design, we recommend using a modulus of subgrade reaction in the range of 100 to 250 pci for design of floor slabs on structural fill.

- Sliding friction between the slab in direct contact with the capillary break/drainage layer may be determined using an allowable coefficient of 0.3. If a vapor barrier is placed between the slab and the capillary break/drainage layer the interface allowable sliding coefficient should be neglected.

The above recommendations are based on anticipated conditions and should be confirmed in the field. It should be noted that many variables, including weather conditions and construction techniques could affect the suitability of *in situ* soil as slab support.

Permanent Drainage

Groundwater is anticipated to be below the elevation of the proposed development, based on raising site grade to maintain a minimum separation of 3 feet between the highest anticipated groundwater elevation. Unanticipated groundwater level fluctuations may occur, posing potential issues that can be readily handled by following the recommendations below.

Foundation and Under Slab Drainage

We recommend the following for permanent drainage of the floor slab around the structure.

- Install perimeter drains near the base of perimeter wall footings. The perimeter drains should be a minimum 4-inch-diameter perforated pipe and should be surrounded by at least 6 inches of drainage material. All drainage pipes should be sloped to drain.

- All slabs should be underlain directly by a capillary break/drainage layer at least 6 inches thick hydraulically connected to the perimeter drains.

- The capillary break/drainage layer should consist of well-graded, free-draining sand and gravel with less than 3 percent fines based on the minus 3/4-inch fraction. This layer is intended to reduce the potential buildup of hydrostatic pressures beneath the slab and to provide a hydraulic connection to the perimeter drains.

- Compact the capillary break/drainage layer to the criteria of structural fill in in this report.

Site Drainage

The site should be graded in such a way that surface water will not pond near the structures. Roof drains should not be connected to the subgrade drainage system and should be sloped and tightlined to a suitable outlet away from the proposed building.

Pavement Design

We recommend that all pavement sections be constructed over a subgrade surface consisting of either non-yielding native bearing soil or compacted structural fill. In general, the upper 0.5 to 2 feet of ESU 1 contains forest duff, topsoil, and organics throughout the site. Where encountered, we recommend overexcavating the portions of ESU 1 that contain debris or significant organic material up to 2 feet deep below the bottom of the pavement and replacing with structural fill. Where structural fill will be used to raise grade, at a minimum near surface soils (approximately upper 0.5 feet) with the most significant organics should be stripped off, and the final depth from the top of the raised subgrade to the top of native soils with organics should be at least 2 feet.

Ground surface settlement due to potential liquefaction was estimated for the IBC design earthquake. This provides an estimate of the magnitude of settlement the pavement would need to accommodate (approximately 2 inches). It is likely damage to pavement areas would occur that would need to be repaired after the earthquake, if liquefaction is not mitigated in these areas. The disruption of uneven/damaged pavement needs to be assessed relative to the vehicles that will need to access or depart from these areas following a design level earthquake. If mitigation is determined to be necessary, ground improvement may be necessary in critical pavement areas.

Pervious Pavement

We understand pervious pavement is being considered for the paved areas of the TRC: concrete sections in areas with heavy military vehicle loading and asphalt sections in areas of personal vehicle use. The WSDOT Pavement Policy (WSDOT 2015) recommends pervious pavements be considered for application to sidewalks, bicycle trails, light vehicle access areas, public and municipal parking lots, and driveways. In general, pervious pavement is typically applied to very low volume, slow speed locations with infrequent truck traffic.

Traffic Loading

Traffic loading assumptions are based on information provided by Schreiber Starling Whitehead and Washington State Military Department. Table 4 provides vehicle types and respective axle loads for vehicles that will access the TRC. We understand the military vehicles will be stored onsite the majority of the time and only used once a month with trips to and from the nearby Joint Base Lewis McChord (JBLM) or the Yakima Training Center. Personal vehicles of TRC employees will access the site daily, with the possibility of personal vehicle use in the evening for recreational activities. The resulting traffic loading yields relatively high vehicle and axle weights with a low number of passes, requiring the typical Equivalent Single Axle Load (ESAL) application for pavement design to be used in conjunction with engineering judgment.

Table 4 – Vehicle Loading for Pavement Section Design

Vehicle	Total Weight (lbs)	Number of Axles[a]	Individual Axle Load (lbs)[b]
Passenger Car	6,000	2	3,000
Humvee	16,000	2	8,000
L-ATV	30,000	2	15,000
M35	18,000	2	9,000
M777 Howitzer	9,300	1	9,300
Stryker	50,000	4	12,500
HEMTT	76,000	4	19,000
LMTV	35,000	2	17,500

Notes:
a. M35 vehicle consists of one front steering axle and one rear dual tandem axle. All other axles are single axles.
b. Even distribution of vehicle weight across axles is assumed

Design Parameters

Infiltration rates and necessary stormwater storage capacity of the pavement section have been assessed by the Civil Engineer (AHBL). We used the following design parameters to obtain minimum layer thicknesses of the pavement sections to achieve structural capacity under the assumed traffic loading. Material specifications are typically provided by specific vendors since the type of material has a direct effect on hydraulic storage capacity. Once a final pavement section is determined, we should be notified to verify the section complies with assumed design strength parameters.

Soil Subgrade

Although site grading may result in pavement sections being founded on structural fill in some locations, pavement sections were designed to bear in ESU 1. A resilient modulus of 3,000 pounds per square inch (psi) was used for ESU 1, which is consistent with recommended values from the PerviousPave Software by the American Concrete Pavement Association (ACPA 2010).

Sand Treatment Layer

We understand a minimum 18-inch sand treatment layer is required for filtration of potential contaminants in the surface water. We assumed this layer will consist of clean fine to medium sand with a resilient modulus of 3,000 psi.

Aggregate Storage Layer

We understand approximately 6 inches of permeable ballast is required as a reservoir layer for stormwater storage under the design storm event conditions. A resilient modulus of 20,000 psi, a structural layer coefficient of 0.10, and a drainage coefficient of 0.8 were assumed for structural design of pavement sections. A minimum 2-inch (and maximum 4-inch) layer of smaller choker/leveling aggregate should be placed between the permeable ballast and the pavement to provide a smooth surface for application of the pavement surfacing.

Permeable Asphalt Concrete (AC)

A structural layer coefficient of 0.35 was used for design of asphalt concrete sections. This is a reduced value from the WSDOT recommended 0.5 for conventional hot mix AC.

Permeable Portland Cement Concrete (PCC)

For permeable PCC section design we used an elastic modulus of 2,500,000 psi, typical of permeable concrete with a minimum flexural strength of 350 psi and a slump between 0 and 1 inch.

Pavement Sections

The following are preliminary pavement sections. When specific permeability rates and necessary storage capacity of the pavement section materials is determined, we will confirm the structural capacity.

Personally Owned Vehicle (POV) Areas

For POV areas, we recommend using 6 inches of permeable AC, over 6 inches of permeable ballast storage rock, placed on a geotextile drainage filter fabric, over 18 inches of a sand treatment layer, and founded on native soil subgrade or structural fill. If PCC sections are required in POV areas, we recommend replacing the 6-inch AC section with 8 inches of undoweled permeable PCC per WSDOT Pavement Policy minimum thickness requirements. Asphaltic permeable pavement should not be applied to areas that will receive anything but POVs.

Military Vehicle Parking, Storage, and Access Areas

For areas that will experience military vehicle loading, we recommend using 10 inches of undoweled permeable PCC, over 6 inches of permeable ballast storage rock, placed on a geotextile drainage filter fabric, over 18 inches of a sand treatment layer, and founded on native soil subgrade or structural fill.

Utility Considerations

Ground surface settlement due to potential liquefaction was estimated for the IBC design earthquake at the locations of the 15 borings across the project site. While the ground surface under the building footprint has the potential for liquefaction induced settlement up to 2 inches, utilities around the building perimeter and in the paved areas may be subject to up to about 3 to 4 inches of settlement. If no interruption in utility service is desired, utilities will need to be designed to accommodate the differential settlement, or emergency backup accommodations would be required.

GEOTECHNICAL RECOMMENDATIONS FOR CONSTRUCTION

General Considerations

A qualified geotechnical representative should be on site to note compliance with the design concepts, specifications, or recommendations, and to allow design changes or evaluation of appropriate construction measures in the event that subsurface conditions differ from those anticipated prior to the start of construction. At a minimum, the representative should observe:

- Excavation and preparation of subgrade for fill placement, pavement, and floor slabs.
- Placement and testing of compacted material.
- Installation of the permanent drainage system.
- Preparation of shallow foundation subgrades.

The purpose of these observations and services is to note compliance with the design concepts, specifications, or recommendations, and to allow design changes or evaluation of appropriate construction measures in the event that subsurface conditions differ from those anticipated before construction starts.

Site Preparation and Grading

Site preparation may involve demolishing existing buildings and foundations, removing pavement, surface vegetation and landscaping, and removing other obstructions that may interfere with new construction. All visible organic material (sod, humus, roots, and/or other plant material), debris, and other unsuitable material should be removed from subgrade areas. We recommend conducting all site grading and paving, as well as any utility trenching, during relatively dry weather.

It may be necessary to relocate or abandon some utilities. Excavation of these utility lines will probably occur through fill. Abandoned underground utilities should be removed or completely grouted. The ends of remaining abandoned utility lines should be sealed to prevent soil or water from entering the pipe. Soft or loose backfill should be removed, and excavations should be backfilled with structural fill. Coordination with the utility owners is generally required to address existing utilities.

Permeable Pavement

We recommend that the permeable pavement be constructed by a contractor with at least 3 years of successful experience with permeable pavements. The soil subgrade beneath permeable pavements should be relatively flat (less than 3 percent slope) to prevent uneven ponding of water. The subgrade should be cut from the edges and not trafficked by heavy machinery. Native subgrade soils should be compacted to a depth of 6 inches using a lightweight static steel drum roller to retain the subgrade's natural infiltration rate or no more than 90 percent of the maximum dry density as determined by the modified Proctor (ASTM D 1557) test method. If heavy compaction of the subgrade occurs, tilling may be necessary to a depth of 2 feet or more below material placement.

A layer of geotextile drainage filter fabric should be placed above the sand treatment layer to prevent migration fines between material types. The geotextile should meet WSDOT Standard Specifications (WSS 2016) 9-33.2 Table 1 for moderate survivability and WSS 9-33.2 Table 2 for Class A permittivity. The geotextile should be installed in conformance with WSS 2-12 – Construction Geotextile.

The sand treatment layer should be relatively flat (less than 3 percent slope) to prevent uneven ponding of water within the storage aggregate. The treatment layer should be placed in maximum 12-inch-thick lifts and compacted with a lightweight static steel drum roller or small vibratory sled plate compactor.

Permeable ballast and choker/leveling layer should be placed in maximum 12-inch lifts and compacted until proofrolling indicates that a firm, unyielding surface is present. A qualified geotechnical representative should be on site to observe proofrolling.

Landscaping areas that are adjacent to pervious pavements should be designed to prevent runoff from washing over the pavements, otherwise sediment can clog the pervious materials.

During and after construction, stockpiles of landscaping materials (i.e., topsoil, bark dust, etc.) and construction materials (i.e., sand, gravel, etc.) should not be placed on the pervious pavements. Extreme care should be taken to prevent trafficking of muddy construction equipment over pervious pavements.

Maintenance should consist of periodic cleaning by vacuuming and flushing with high volume water at low pressures. Based on available information, vacuuming and flushing should occur at least one time per year. We note that sweeping is not an effective method for cleaning of pervious pavements, in fact available information indicates that sweeping may decrease the permeability of pervious pavements by clogging pores.

Foundations and Floor Slabs

A qualified geotechnical representative should be on site to assess and document the suitability of the subgrade during construction, prior to placement of footings or concrete. Compact all exposed subgrades to a firm, non-yielding condition. Subgrades will need to be protected from groundwater, surface water, and precipitation. Softened subgrades will need to be overexcavated and replaced with suitable structural fill, or lean mix depending on the location.

Given the soil conditions encountered in the borings and test pits, we expect that conditions will not be favorable for working on the native soils during the wet season. We recommend that the contractor consider grading during drier months and placing a 4-inch lean concrete mud slab to protect exposed subgrades below slabs on grade and footing excavations. The mud slab should be poured before placing the drainage system.

Note that if the bottom of the floor slab excavation is soft, wet, or disturbed, the contractor should be prepared to place a temporary working surface. This surface cannot count as part of the capillary break.

It may be necessary to build a working surface for equipment to prevent excessive site disturbance that could result in additional overexcavation and structural fill placement.

Structural Fill Selection, Placement, and Compaction

Backfill placed within the building area, or below paved areas, should be considered structural fill. The following sections include our recommendations for structural fill selection, placement, and compaction.

Reuse of Site Soil as Structural Fill

The suitability of excavated site soils for compacted structural fill will depend upon the gradation and moisture content of the soil when it is placed. As the amount of fines (that portion passing the No. 200 sieve) increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soil containing more than about 5 percent fines cannot be consistently compacted to a dense non yielding condition when the water content is greater than about 2 percent above or below optimum. Reusable soil must also be free of organic and other unsuitable material.

Our explorations and laboratory tests indicate that ESU 1, which is the near surface unit in which grading will occur appears to vary from zones with about 0 percent fines to zones with about 25 percent fines. Thus, in general it appears these soils may be difficult to compact to structural fill requirements, depending on weather and soil moisture conditions. It may be possible to use some of these materials for structural fill, if conditions are favorable or moisture conditioning can be accomplished. We recommend Hart Crowser assess the potential suitability of these materials based on observations and laboratory testing during construction. Onsite soils that cannot be compacted to the structural fill requirements may be considered for use for non-structural purposes such as landscaping.

Selection of Import Fill

For import soil to be used as structural fill, we recommend using a non-silty, well graded sand or sand and gravel with less than 5 percent passing the U.S. No. 200 sieve by dry weight (based on the minus 3/4-inch fraction) for imported structural fill placed during wet weather. Compaction of material containing more than about 5 percent fine material may be difficult if the material is wet or becomes wet during rainy weather. During dry weather, imported soil can contain 20 to 30 percent by weight passing the No. 200 mesh sieve (based on the minus 3/4-inch fraction) provided it is compacted at a moisture content within 2 percent of the optimum moisture content.

Placement and Compaction of Structural Fill

We make the following recommendations for the placement and compaction of structural fill:

- Before fill control can begin, the compaction characteristics of proposed fill material must be determined from representative samples of the structural and drainage fill. Samples should be obtained as soon as possible, but at least 5 days before use on site. A study of compaction characteristics should include determination of grain size distribution and optimum and natural moisture content of the soil at the time of placement.

- Structural fill can consist of either imported soil or recompacted on-site soil, if its moisture content is suitable and weather conditions allow.

- Compact structural fill to a minimum of 95 percent of the maximum dry density (ASTM D 1557) test method.

- Maintain moisture content within 2 percent of the optimum moisture content (ASTM D 1557).

- Place structural fill only on dense, non-yielding subgrade soils.

- Place and compact all structural fill in even lifts with a loose thickness no greater than 10 inches. If small, hand-operated compaction equipment is used to compact structural fill, fill lifts should not exceed 6 to 8 inches in loose thickness.

- In wet subgrade areas, clean material with a gravel content (material coarser than a U.S. No. 4 sieve) of at least 30 to 35 percent may be necessary.

- The compacted densities of all lifts should be verified by testing. Any material to be used as structural fill should be sampled and tested prior to use on site, to determine its maximum dry density and gradation.

Temporary Open Cuts

The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil;
- The presence and amount of any seepage;
- The depth of cut;
- The proximity of the cut to any surcharge loads near the top of the cut, such as stockpiled material, traffic, structures, etc. and the magnitude of these surcharges;
- The duration of the open excavation; and
- The care and methods used by the contractor.

Temporary soil cuts for site excavations that are more than 4 feet deep should be adequately sloped back to prevent sloughing and collapse in accordance with Washington Department of Occupational Safety and Health (DOSH) guidelines (WAC Chapter 296-155 Part N). Based on these guidelines, the near surface soils at the site in which grading would be performed would be classified as Type C. We make the following recommendations for open cuts:

- Use a maximum allowable slope for excavation less than 20 feet deep of:
  - 1.5H:1V for cuts in Soil Type C.
- Use a maximum allowable slope of 1.5H:1V or flatter if groundwater seepage is encountered within the excavation slopes.
- Do not excavate below the bearing elevation of existing footings or structural elements. Consult with the geotechnical engineer during construction to limit the size of these excavations and the amount of time that they remain open.
- Protect the slope from erosion by using plastic sheeting, especially during wet weather excavation.
- Limit the maximum duration of the open excavation to the shortest time period possible.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of the slope, in general. However, more or less stringent requirements may apply depending on field conditions and actual surcharge loads.

Because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated prior to construction. We recommend that stability of the temporary slopes used for construction be the sole responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface. All excavations should be made in accordance with all local, state, and federal safety requirements.

Site Drainage

Due to the high groundwater table and observed standing water in portions of the site, groundwater/surface water may be encountered during construction even for shallow excavations and grading. The contractor will need to maintain positive site drainage to avoid allowing foundation, slab, or pavement subgrades to soften during construction. Generally, we expect that any groundwater seepage at the site can likely be temporarily controlled during construction using sumps and pumps. However, potential deeper utility installations may encounter more significant groundwater.

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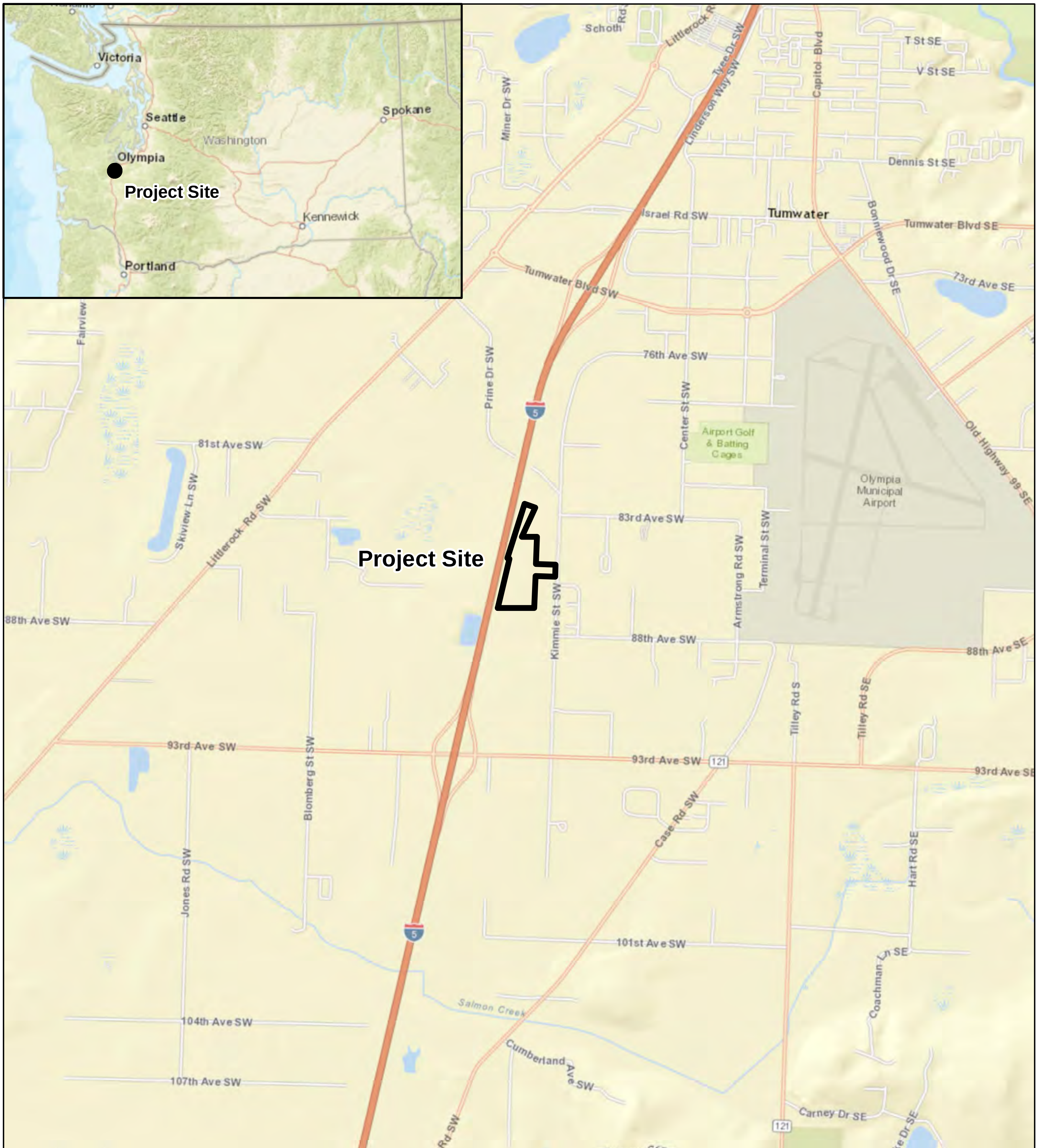

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3,100 1,550 0 3,100

Scale in Feet

Tumwater Readiness Center
Tumwater, Washington

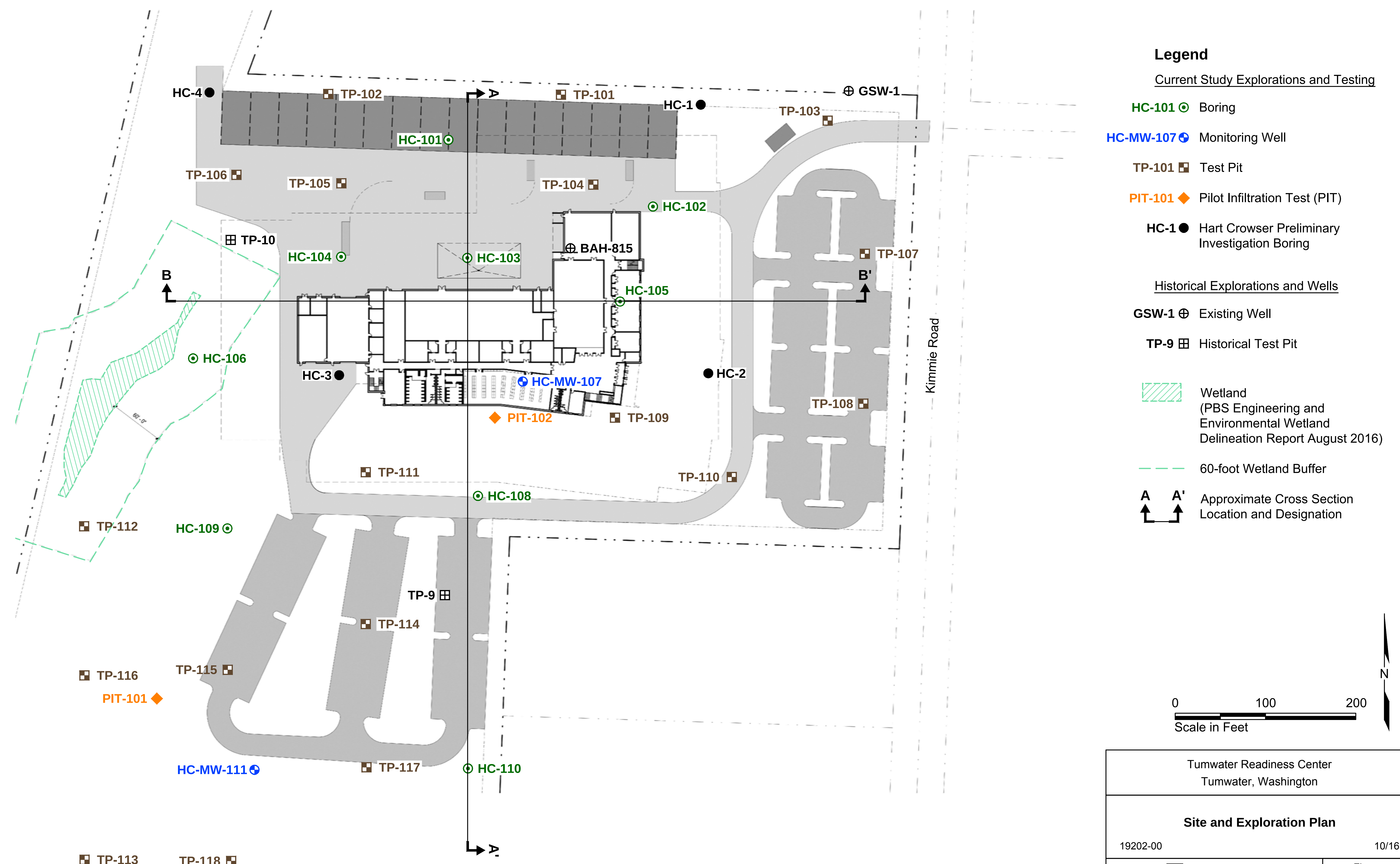
Vicinity Map

19202-00 10/16

HARTCROWSER

Figure **1**

EAL 10/05/16 1920200-AA (VMap).mxd



Tumwater Readiness Center
Tumwater, Washington

Site and Exploration Plan

19202-00 10/16

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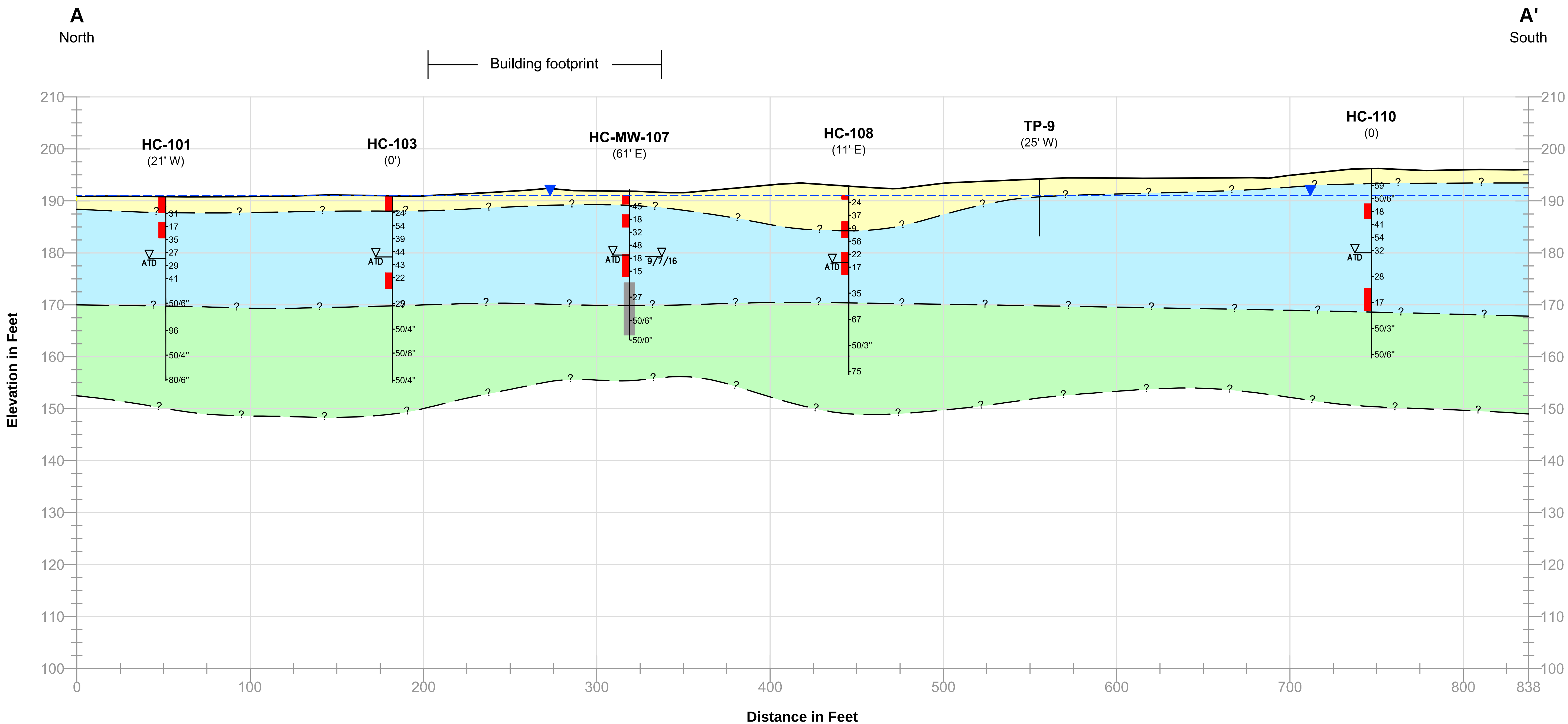
Figure 2

EAL 10/4/16 1920200-006 (SPlan).dwg



Legend

HC-101 (21' W)	Exploration number (Offset and distance)
	Exploration location
Current design groundwater elevation (long-term monitoring ongoing)	
ATD	Water level
9	Standard penetration resistance in blows per foot
Approximate range of soils susceptible to liquefaction	
	Screened interval

- ESU 1 - Loose to medium dense, light brown to brown, slightly silty to silty, fine to medium SAND with scattered gravel and cobbles
- ESU 2 - Medium dense to dense, gray to gray-brown, sandy to very sandy GRAVEL
- ESU 3 - Dense to very dense, gray to gray-brown, sandy GRAVEL with occasional sand seams

Horizontal Scale in Feet: 0, 60, 120

Vertical Scale in Feet: 0, 20, 40

Vertical Exaggeration x 3

Note:
Contact between soil units is interpolated between borings and represents our interpretation of subsurface conditions based on currently available data.

EAL 10/4/16 1920200-007 (XSec).dwg

Tumwater Readiness Center
Tumwater, Washington

Generalized Soil Profile
Cross Section A-A

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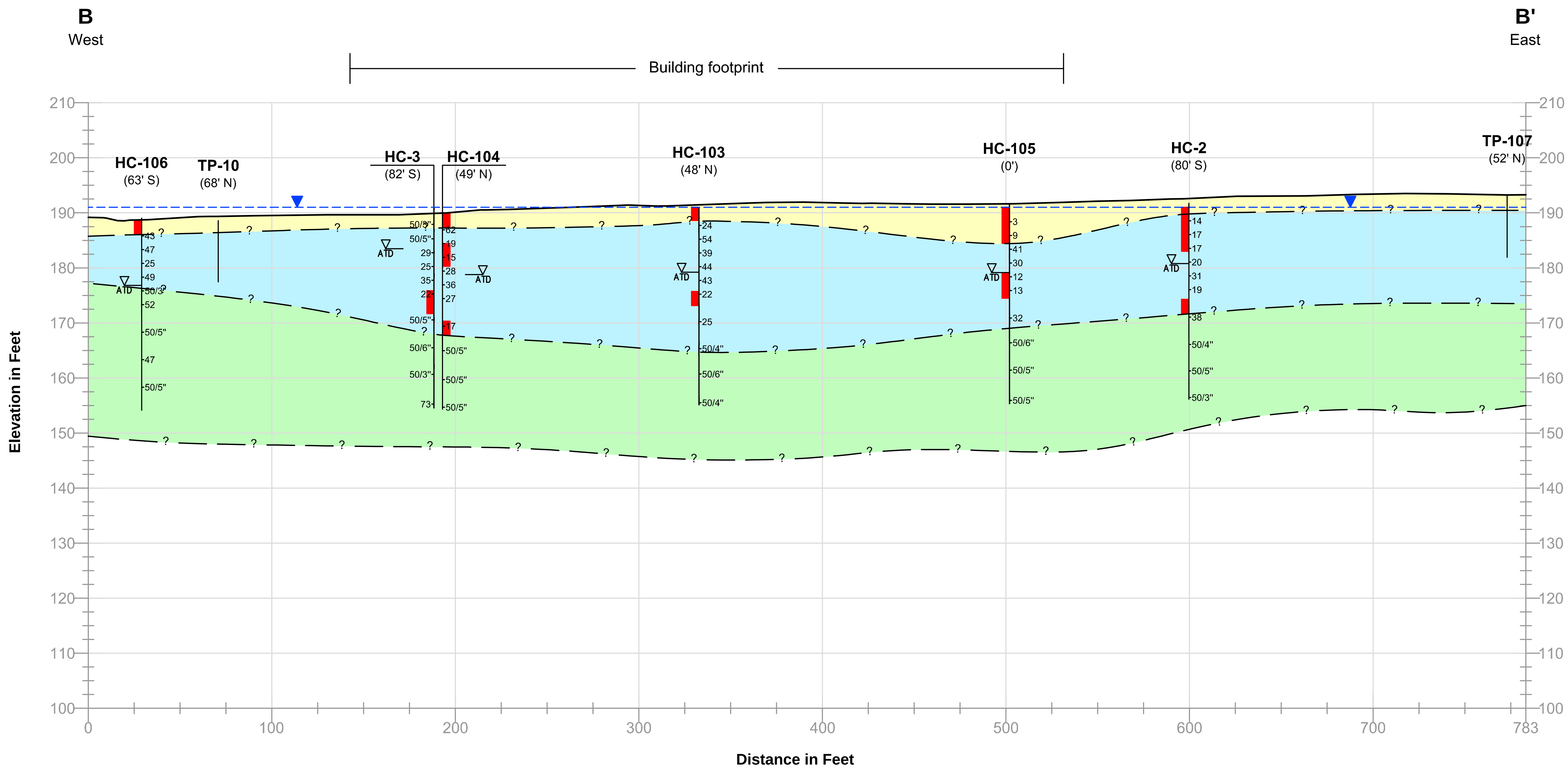
Figure **3**



Legend

HC-101 (21' W) Exploration number (Offset and distance)

Exploration location

Current design groundwater elevation (long-term monitoring ongoing)

ATD Water level

9 Standard penetration resistance in blows per foot

Approximate range of soils susceptible to liquefaction

Screened interval

ESU 1 - Loose to medium dense, light brown to brown, slightly silty to silty, fine to medium SAND with scattered gravel and cobbles

ESU 2 - Medium dense to dense, gray to gray-brown, sandy to very sandy GRAVEL

ESU 3 - Dense to very dense, gray to gray-brown, sandy GRAVEL with occasional sand seams

Note:
Contact between soil units is interpolated between borings and represents our interpretation of subsurface conditions based on currently available data.

Horizontal Scale in Feet
0 60 120

0 20 40
Vertical Scale in Feet
Vertical Exaggeration x 3

Tumwater Readiness Center
Tumwater, Washington

Generalized Soil Profile
Cross Section B-B

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Figure
4

EAL 10/4/16 1920200-007 (XSec).dwg

APPENDIX A
Field Exploration Methods and Analysis

APPENDIX A

Field Exploration Methods and Analysis

This appendix documents the processes Hart Crowser used in determining the nature (and quality) of the soil and groundwater underlying the project site addressed by this report. The discussion includes information on the following subjects:

- Explorations and Their Location;
- Hollow-Stem Auger Borings;
- Standard Penetration Test (SPT) Procedures;
- Excavation of Test Pits; and
- Monitoring Well Installation

Explorations and Their Location

Subsurface explorations for this project include HC-1 through HC-4, HC-101 through HC-111, and TP-101 through TP-118. The exploration logs within this appendix show our interpretation of the drilling, excavating, sampling, and testing data. The logs indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on Figure A-1 - Key to Exploration Logs. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

Location of Explorations. Figure 2 shows the location of the explorations. This report shows the actual locations and ground surface elevations, presented on the exploration logs, as they were established during a site survey by AHBL, dated March 28, 2016.

Hollow-Stem Auger Borings

With depths ranging from 29 to 36.5 feet below the ground surface, 15 hollow-stem auger borings were drilled from June 29 to September 7, 2016. The borings used a 3 ¼ -inch inside diameter hollow-stem auger. The borings were all advanced with a Diedrich 50 track-mounted drill rig subcontracted by Hart Crowser. The drilling was continuously observed by an engineering geologist from Hart Crowser. Detailed field logs were prepared of each boring. Using the Standard Penetration Test (SPT), we obtained samples at 2-1/2- to 5-foot-depth intervals.

The borings logs are presented on Figures A-2 through A-16 at the end of this appendix.

Standard Penetration Test (SPT) Procedures

This test is an approximate measure of soil density and consistency. To be useful, the results must be used with engineering judgment in conjunction with other tests. The SPT (as described in ASTM D 1586) was used to obtain disturbed samples. This test employs a standard 2-inch outside diameter split-spoon sampler. Using a 140-pound autohammer, free-falling 30 inches, the sampler is driven into the soil for 18 inches. The number of blows required to drive the sampler the last 12 inches only is the

Standard Penetration Resistance. This resistance, or blow count, measures the relative density of granular soils and the consistency of cohesive soils. The blow counts are plotted on the boring logs at their respective sample depths.

Soil samples are recovered from the split-barrel sampler, field classified, and placed into water-tight jars. They are then taken to Hart Crowser's laboratory for further testing.

In the Event of Hard Driving

Occasionally very dense materials preclude driving the total 18-inch sample. When this happens, the penetration resistance is entered on logs as follows:

Penetration less than 6 inches. The log indicates the total number of blows over the number of inches of penetration.

Penetration greater than 6 inches. The blow count noted on the log is the sum of the total number of blows completed after the first 6 inches of penetration. This sum is expressed over the number of inches driven that exceed the first 6 inches. The number of blows needed to drive the first 6 inches are not reported. For example, a blow count series of 12 blows for 6 inches, 30 blows for 6 inches, and 50 (the maximum number of blows counted within a 6-inch increment for SPT) for 3 inches would be recorded as 80/9.

Excavation of Test Pits

18 test pits, designated TP-101 through TP-118, were excavated across the site from August 30 to September 9, 2016, with a backhoe subcontracted by Hart Crowser. The sides of these excavated pits offer direct observation of the subgrade soils. The test pits were located by AHBL the same as the borings, and excavated under the direction of a geologist from Hart Crowser. The geologist observed the soil exposed in the test pits and reported the findings on a field log. Our geologist took representative samples of soil types for testing at Hart Crowser's laboratory. Groundwater levels or seepage were noted during excavation. The density/consistency of the soils (as presented parenthetically on the test pit logs to indicate their having been estimated) is based on visual observation only as disturbed soils cannot be measured for in-place density in the laboratory.

The test pit logs are presented on Figures A-17 through A-25.

Monitoring Well Installation

Two monitoring wells, HC-MW-107 and HC-MW-111, were installed to allow long-term groundwater level monitoring, assess groundwater quality, and to provide water quality data for the site. The monitoring wells were installed on September 6 (HC-MW-107) and September 7 (HC-MW-111), 2016.

The boreholes for the wells were drilled using a Diedrich 50 track-mounted drill rig. Two-inch-diameter Schedule 80 PVC riser pipe and 1-1/2-inch-diameter 0.020-inch machine-slotted screen were used for the well casings and screens. The well screen and casing riser were lowered down through the hollow-stem auger/casing/open hole. As the auger/casing was withdrawn, No. 10/20 silica sand was placed in the annular space from the base of the boring to approximately 13 feet above the top of the well screen in HC-MW-107 and 17.5 feet above in HC-MW-111.

Well seals were constructed by placing bentonite chips in the annular space on top of the filter sand to 1 foot below the ground surface. The remaining annular space was backfilled with concrete to complete the surface seal. The monitoring wells were each completed with an 8-inch Morris flushmount set in concrete. The monitoring well construction details are illustrated on the boring logs on Figures A-12 and A-16.

The monitoring wells were installed in accordance with Washington State Department of Ecology regulations.

Key to Exploration Logs

Sample Description

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT, additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Density	Standard Penetration Resistance (N) in Blows/Foot	SILT or CLAY Consistency	Standard Penetration Resistance (N) in Blows/Foot	Approximate Shear Strength in TSF
Very loose	0 to 4	Very soft	0 to 2	<0.125
Loose	4 to 10	Soft	2 to 4	0.125 to 0.25
Medium dense	10 to 30	Medium stiff	4 to 8	0.25 to 0.5
Dense	30 to 50	Stiff	8 to 15	0.5 to 1.0
Very dense	>50	Very stiff	15 to 30	1.0 to 2.0
		Hard	>30	>2.0

Sampling Test Symbols

- 1.5" I.D. Split Spoon
- Grab (Jar)
- 3.0" I.D. Split Spoon
- Shelby Tube (Pushed)
- Bag
- Cuttings
- Core Run

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS — MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS — MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS — MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SILTY SANDS, SAND - SILT MIXTURES
			SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS — MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Moisture

Dry	Little perceptible moisture
Damp	Some perceptible moisture, likely below optimum
Moist	Likely near optimum moisture content
Wet	Much perceptible moisture, likely above optimum

Minor Constituents

Minor Constituents	Estimated Percentage
Trace	<5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Laboratory Test Symbols

GS	Grain Size Classification
CN	Consolidation
UU	Unconsolidated Undrained Triaxial
CU	Consolidated Undrained Triaxial
CD	Consolidated Drained Triaxial
QU	Unconfined Compression
DS	Direct Shear
K	Permeability
PP	Pocket Penetrometer Approximate Compressive Strength in TSF
TV	Torvane Approximate Shear Strength in TSF
CBR	California Bearing Ratio
MD	Moisture Density Relationship
AL	Atterberg Limits
PID	Photoionization Detector Reading
CA	Chemical Analysis
DT	In Situ Density in PCF
OT	Tests by Others

Water Content in Percent
Liquid Limit
Natural
Plastic Limit

Groundwater Indicators

- Groundwater Level on Date or (ATD) At Time of Drilling
- Groundwater Seepage (Test Pits)

Sample Key

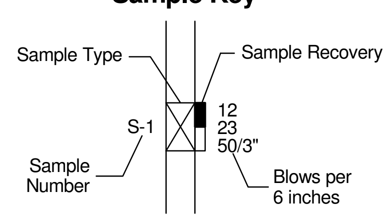

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Figure A-1

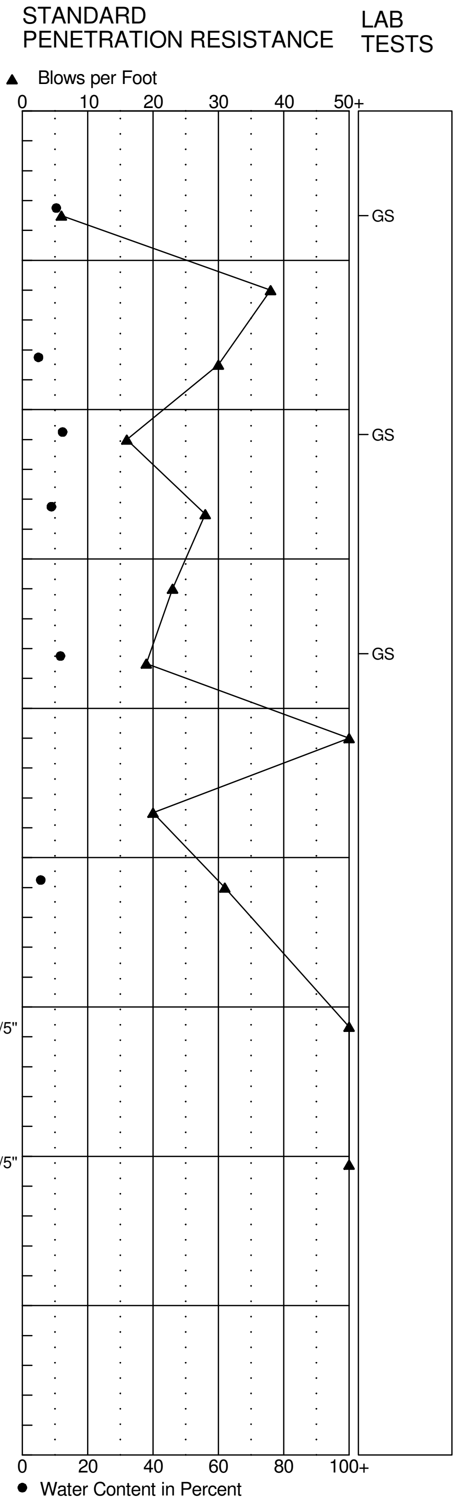
Boring Log HC-1

Location: N 605187.9 E 1033803
Approximate Ground Surface Elevation: 192.55 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich 50 Track/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 4 inches
Logged By: B. McDonald   Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SP-SM	2 inches of Sod over loose, moist, red-brown, slightly silty, fine SAND with scattered small roots.	0
GP	Medium dense to dense, moist, gray-brown, sandy GRAVEL.	5
SP	Medium dense, wet, gray-brown, very gravelly SAND with trace silt.	10 (ATD)
GP	Medium dense to dense, wet, gray-brown, sandy GRAVEL with occasional gravelly sand zones.	20
GP	Very dense, wet, gray-brown, sandy GRAVEL with possible scattered cobbles.	27
	Bottom of Boring at 35.4 Feet. Started 06/30/16. Completed 06/30/16.	35.4

Sample	Blows	Lab Tests
S-1	3 3 3	GS
S-2	12 19 19	
S-3	12 15 15	
S-4	9 9 7	GS
S-5	11 13 15	
S-6	6 11 12	
S-7	6 8 11	GS
S-8	30 45 36	
S-9	5 6 14	
S-10 A/B	8 13 18	
S-11	22 50/5"	
S-12	50/5"	



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

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Figure A-2

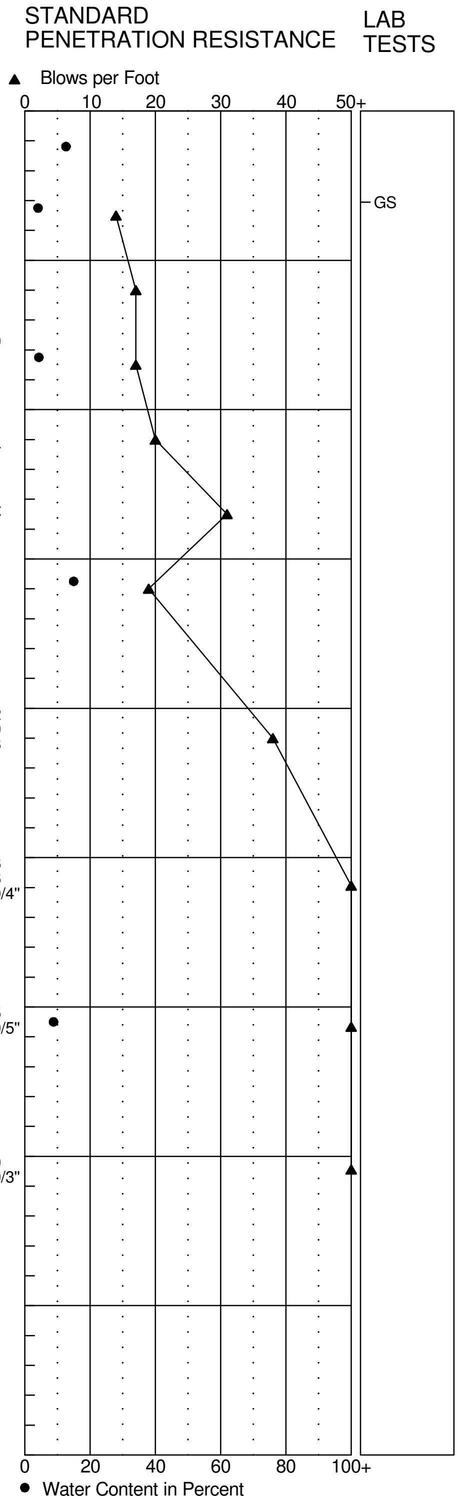
Boring Log HC-2

Location: N 604890.7 E 1033811
Approximate Ground Surface Elevation: 191.79 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich 50 Track/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 4 inches
Logged By: B. McDonald Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SM	2 inches of Forest Duff over (loose), moist, red-brown, silty, fine SAND, with small roots and scattered gravel.	0
GP	Medium dense, moist, brown to gray-brown, very sandy GRAVEL with trace silt.	
SP	Medium dense, moist, gray-brown, fine to medium SAND with occasional gravel.	5
GP	Medium dense to dense, moist to wet, gray-brown, sandy GRAVEL.	10 (ATD)
SP	Medium dense, wet, gray-brown SAND with scattered gravel.	15
GP	Dense to very dense, wet, gray-brown to light gray, sandy GRAVEL.	20
	Bottom of Boring at 35.7 Feet. Started 06/29/16. Completed 06/29/16.	

Sample	Blows
S-1	
S-2	5 7 7
S-3	5 8 9
S-4	10 9 8
S-5	5 6 14
S-6	8 14 17
S-7	5 8 11
S-8	12 20 18
S-9	16 26 50/4"
S-10	35 50/5"
S-11	30 50/3"



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

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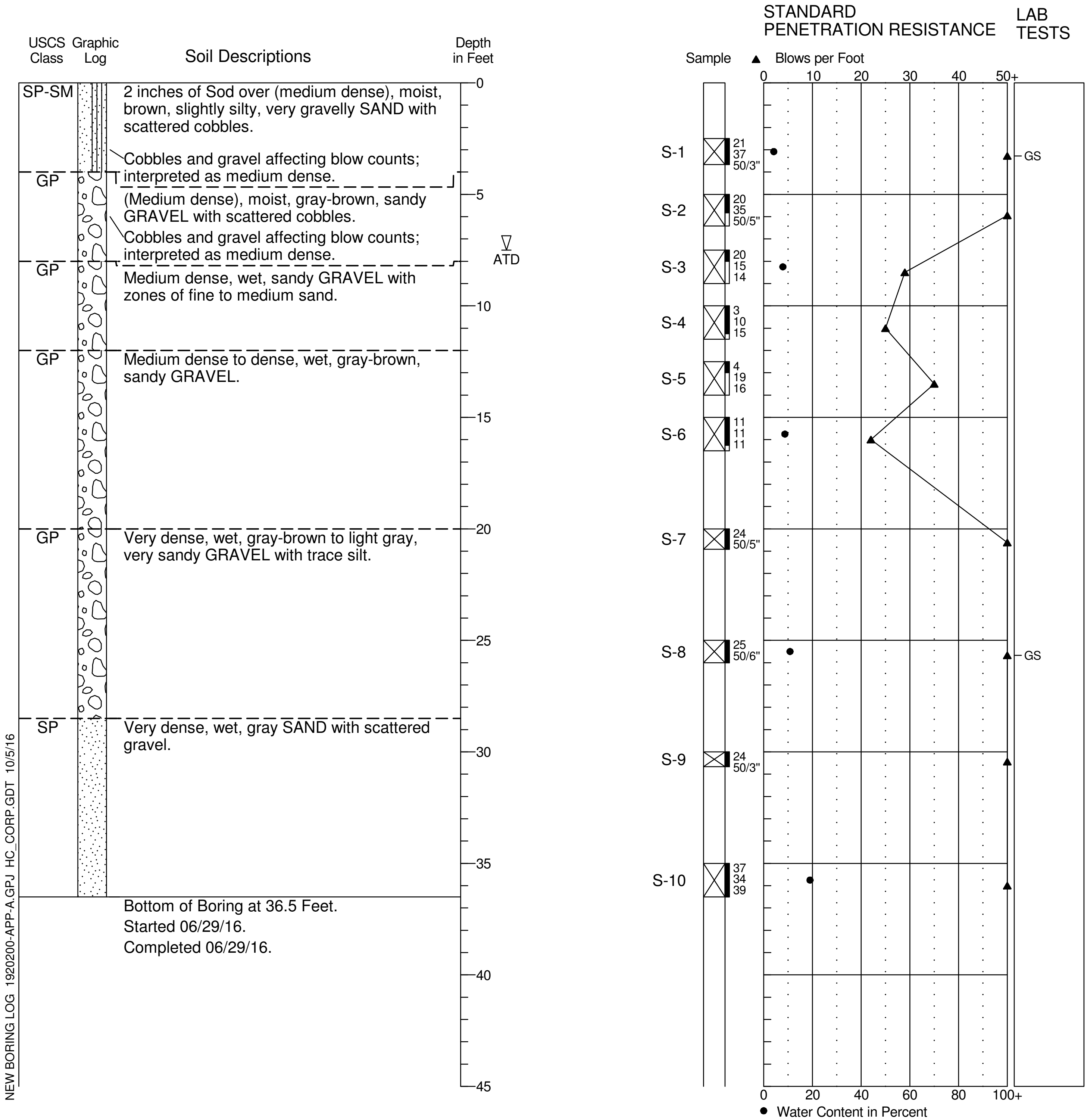
Figure A-3

Boring Log HC-3

Location: N 604888.6 E 1033403
Approximate Ground Surface Elevation: 190.75 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich 50 Track/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 4 inches
Logged By: B. McDonald   Reviewed By: J. Thomas



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

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Figure A-4

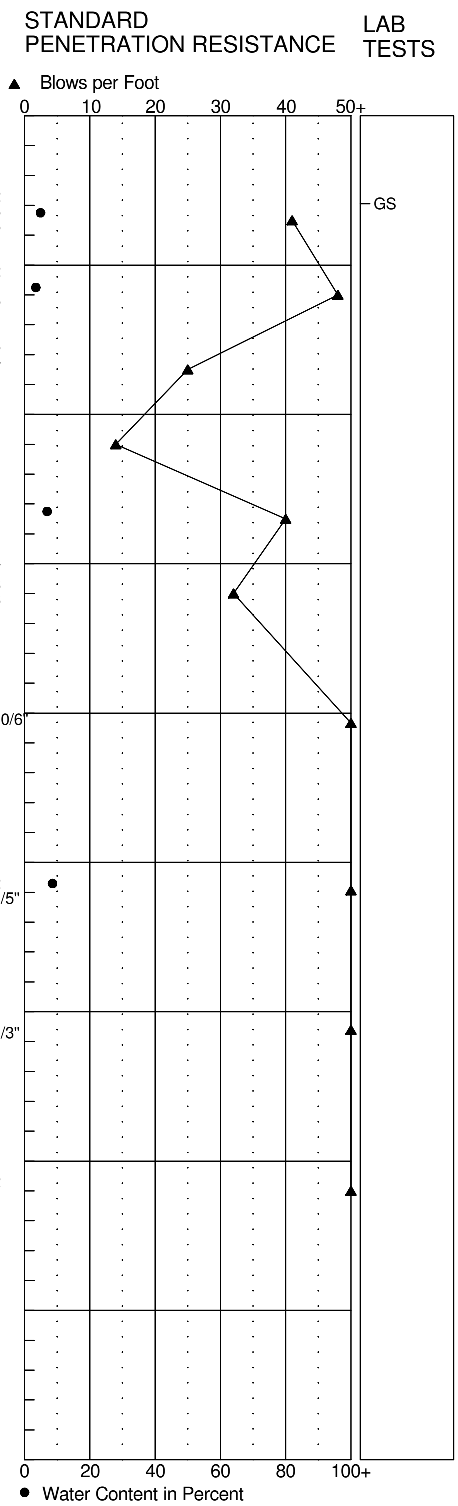
Boring Log HC-4

Location: N 605201.4 E 1033259
Approximate Ground Surface Elevation: 190.66 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich 50 Track/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 4 inches
Logged By: B. McDonald   Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SM	2 inches of Sod over (loose), moist, brown, silty, fine SAND with occasional scattered gravel and small roots.	0
GW-GM	(Medium dense), moist to wet, gray-brown, slightly silty, very sandy GRAVEL.	
	∇ ATD	
SP	Medium dense, wet, gray, fine to medium SAND.	
GP	Medium dense to dense, wet, gray-brown, sandy GRAVEL.	10
GP	Very dense, wet, gray, sandy GRAVEL with occasional scattered cobbles.	20
SP	Very dense, wet, gray SAND with scattered gravel.	
	Blows in first six inches affected by heave and clean-out.	35
	Bottom of Boring at 36.5 Feet. Started 06/29/16. Completed 06/29/16.	

Sample	Blows	Lab Tests
S-1		
S-2	12 16 25	GS
S-3	12 23 25	
S-4	38 14 11	
S-5 A/B	7 6 8	
S-6	9 19 21	
S-7	17 16 16	
S-8	100/6"	
S-9	29 42 50/5"	
S-10	30 50/3"	
S-11	5 22 29	



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

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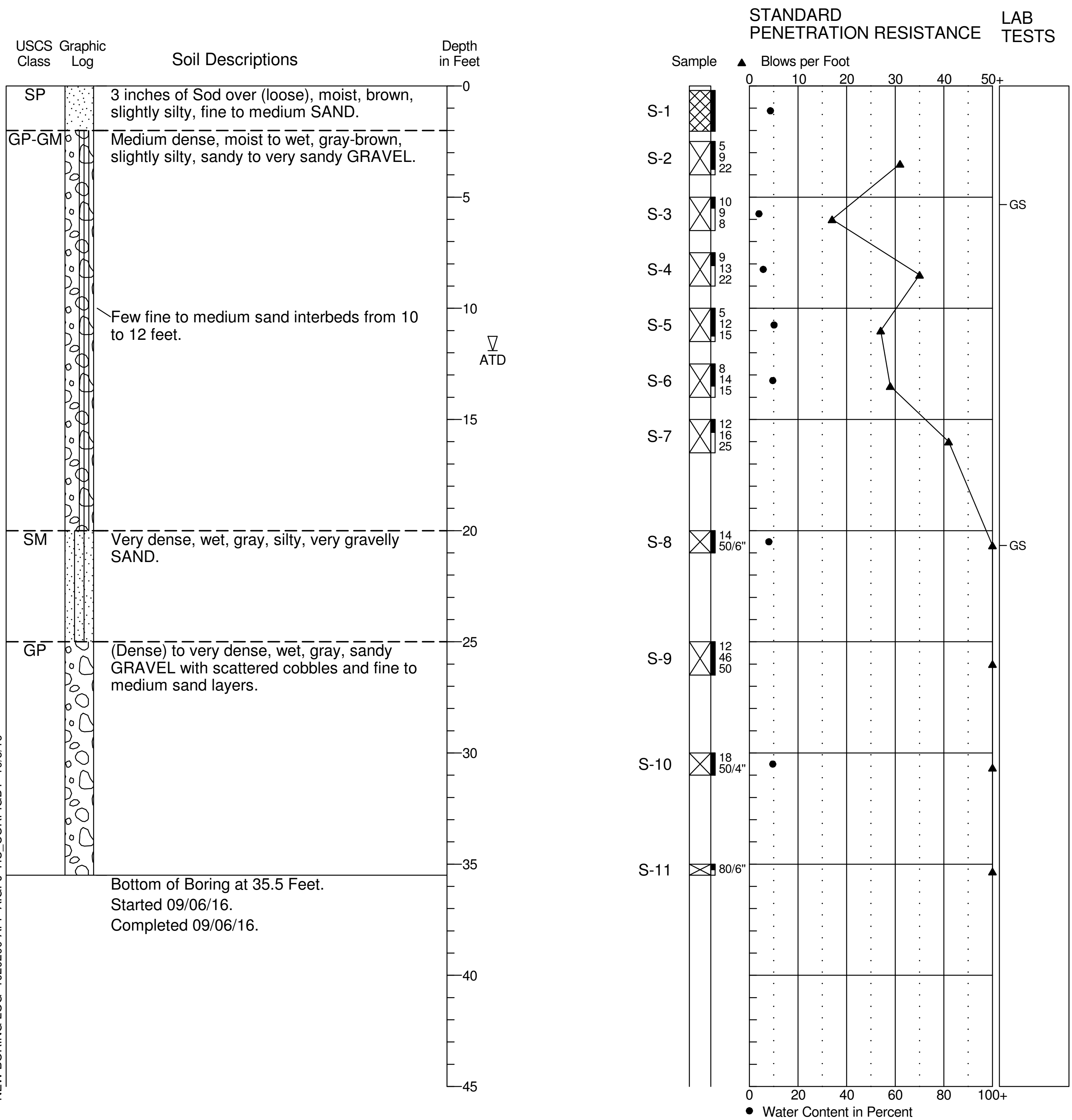
Figure A-5

Boring Log HC-101

Location: N 605149.06 E 1033523.62
Approximate Ground Surface Elevation: 190.82 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich D-50/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: B. McDonald   Reviewed By: J. Thomas



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

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Figure A-6

Boring Log HC-102

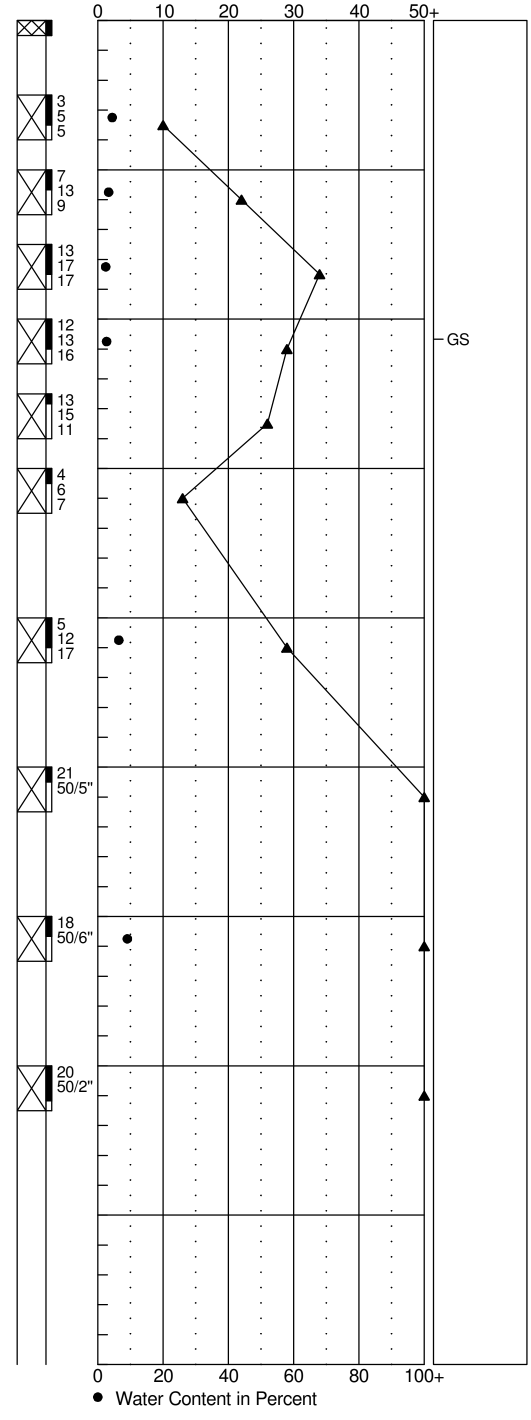
Location: N 605075.17 E 1033750.1
Approximate Ground Surface Elevation: 191.84 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: D50 Track Rig/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: M. Miller   Reviewed By: J. Thomas

USCS Class	Depth in Feet	Soil Descriptions
SW	0–10	Loose to medium dense, moist, light brown, slightly gravelly to gravelly SAND.
GP	10–25	Medium dense, wet, gray, sandy GRAVEL.
GP	25–33.5	Very dense, wet, gray, sandy GRAVEL.
SW	33.5–36.5	Very dense, wet, gray SAND with trace gravel.
		Bottom of Boring at 36.5 Feet. Started 09/07/16. Completed 09/07/16.

Groundwater: ATD (approx. 12.5 ft)

Sample	Blows	Lab Tests
S-1		
S-2	3, 5, 5	
S-3	7, 13, 9	
S-4	13, 17, 17	
S-5	12, 13, 16	GS
S-6	13, 15, 11	
S-7	4, 6, 7	
S-8	5, 12, 17	
S-9	21, 50/5"	
S-10	18, 50/6"	
S-11	20, 50/2"	



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

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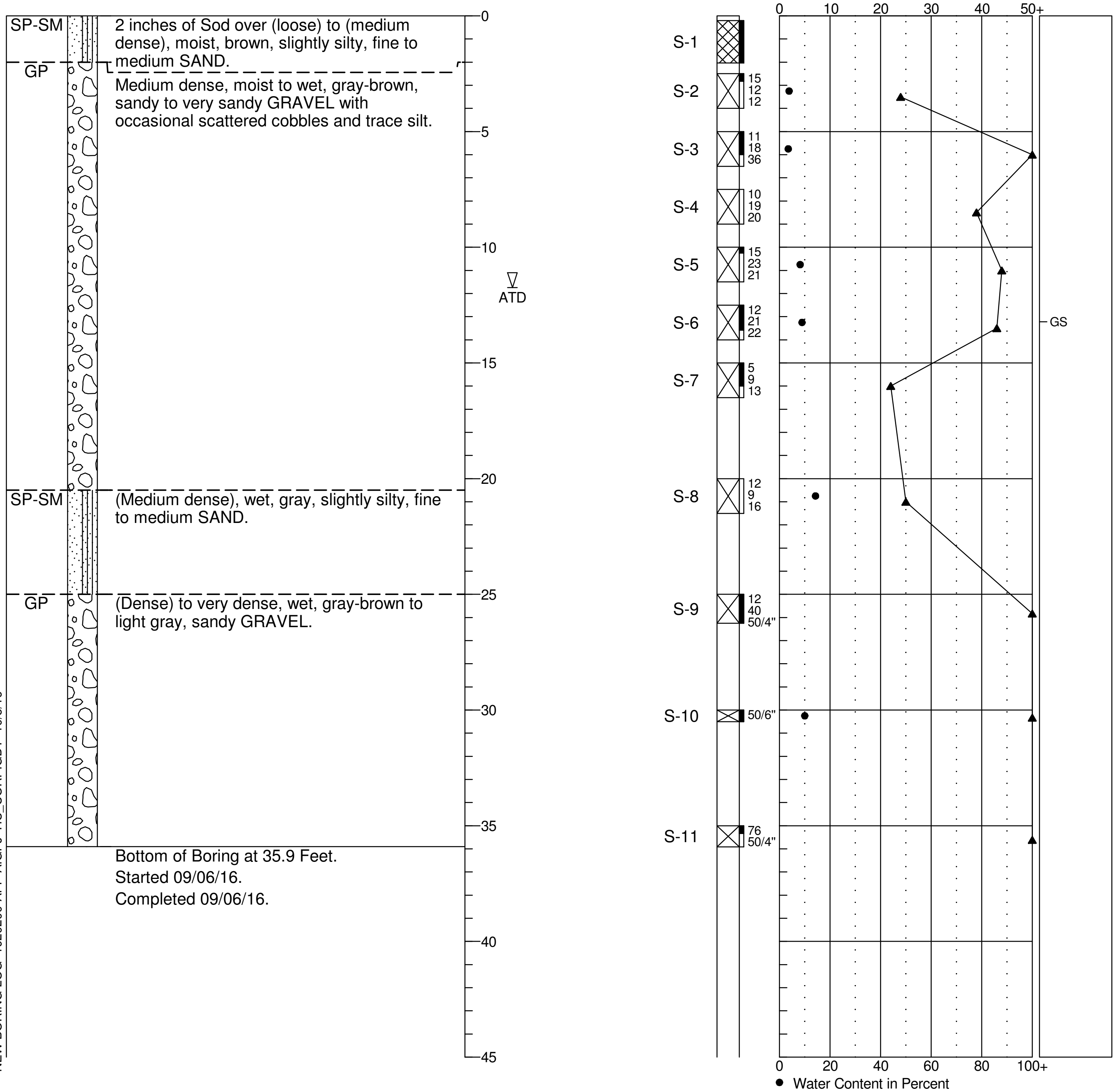
Figure A-7

Boring Log HC-103

Location: N 605018.35 E 1033544.47
Approximate Ground Surface Elevation: 190.96 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich D-50/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: B. McDonald Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SP-SM	2 inches of Sod over (loose) to (medium dense), moist, brown, slightly silty, fine to medium SAND.	0
GP	Medium dense, moist to wet, gray-brown, sandy to very sandy GRAVEL with occasional scattered cobbles and trace silt.	2
SP-SM	(Medium dense), wet, gray, slightly silty, fine to medium SAND.	20
GP	(Dense) to very dense, wet, gray-brown to light gray, sandy GRAVEL.	25
	Bottom of Boring at 35.9 Feet. Started 09/06/16. Completed 09/06/16.	35.9



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

19202-00 *9/16*

Figure A-8

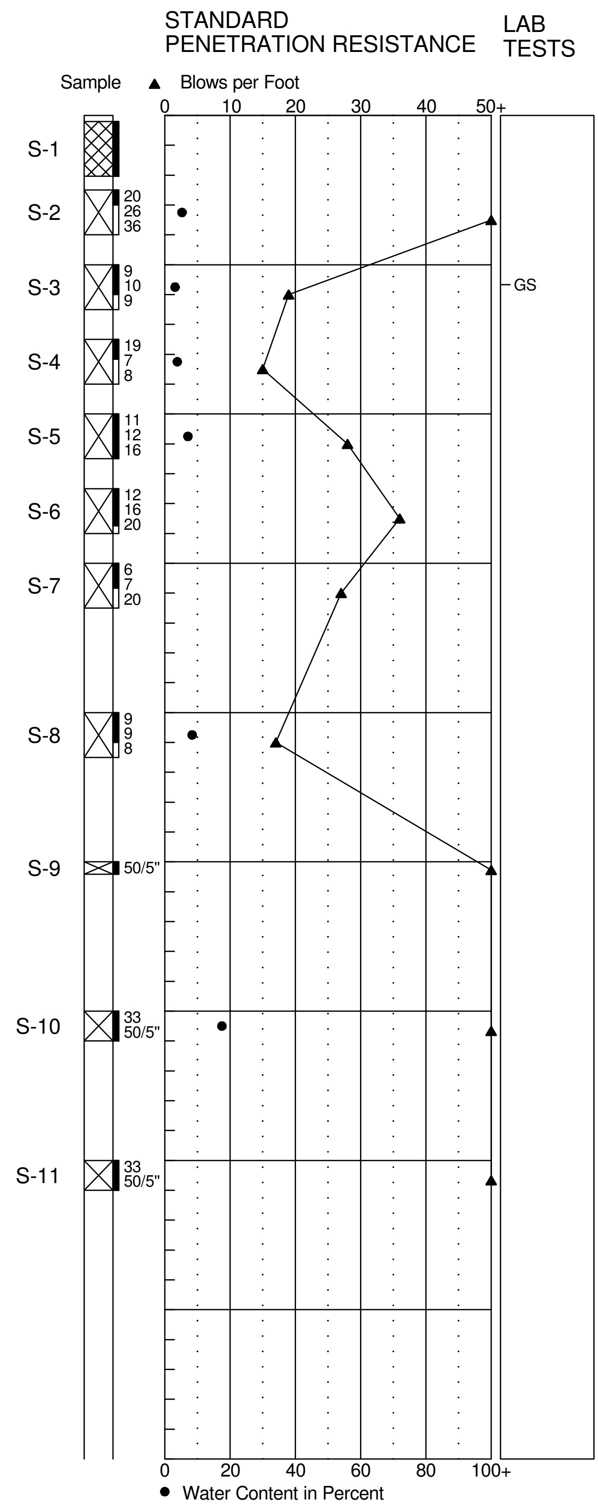
Boring Log HC-104

Location: N 605019.59 E 1033404.76
Approximate Ground Surface Elevation: 190.18 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich D-50/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: B. McDonald Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SP-SM	2 inches of Sod over (loose), moist, brown, slightly silty to silty, fine to medium SAND with scattered gravels.	0
	Becomes gravelly,	6
GP	Medium dense, moist to wet, gray-brown, sandy GRAVEL.	6.5
	Blow counts may not be representative; affected by gravel.	12
	Gravelly drill action.	18
GP	Dense to very dense, wet, gray-brown to gray, sandy GRAVEL.	22
	Scattered fine to medium sand layers.	30
	Gravelly drill action.	33
	Bottom of Boring at 35.9 Feet. Started 09/06/16. Completed 09/06/16.	35.9

▽ ATD (at approximately 11 feet)



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

19202-00 *9/16*

Figure A-9

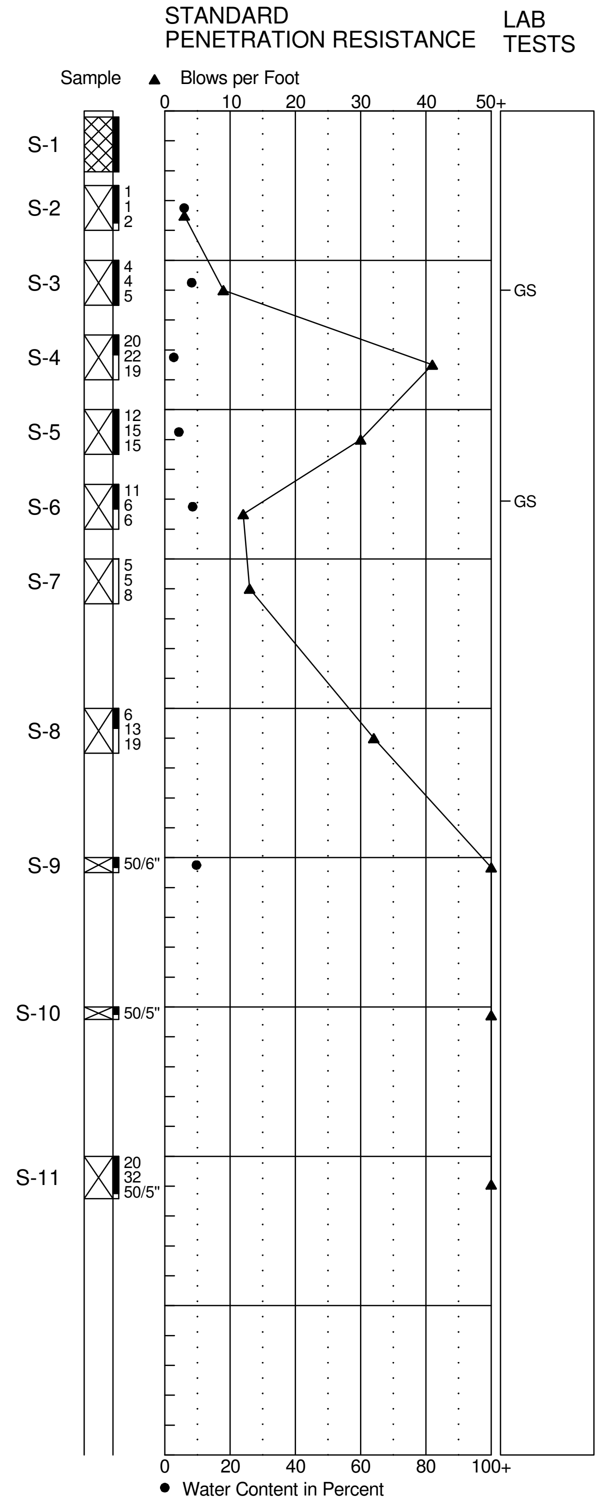
Boring Log HC-105

Location: N 604970.05 E 1033713.68
Approximate Ground Surface Elevation: 191.65 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich D-50/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: B. McDonald   Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SP-SM	2 inches of Sod over loose, moist, brown, slightly silty to silty, fine to medium SAND with scattered gravel.	0
GP	Medium dense, moist to wet, gray-brown, sandy GRAVEL with trace silt. Blow counts may not be representative; affected by gravel.	7
	Faster bit penetration.	15
GP	(Dense) to very dense, wet, gray-brown to gray, sandy GRAVEL with scattered cobbles.	22
	Very gravelly drill action.	28
	Bottom of Boring at 36.4 Feet. Started 09/06/16. Completed 09/06/16.	36.4

Groundwater: ATD at approx. 12 feet.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

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Figure A-10

Boring Log HC-106

Location: N 604907.16 E 1033240.86
Approximate Ground Surface Elevation: 189.08 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: D50 Track Rig/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: M. Miller   Reviewed By: J. Thomas

USCS Class	Depth in Feet	Soil Descriptions
SM	0	Loose, dry, light brown, silty, very gravelly SAND.
SW	2.5	Dense, moist, light brown, gravelly SAND.
		Becomes gray.
		Becomes wet.
GW-GM	12.5	Very dense, wet, gray, slightly silty, sandy to very sandy GRAVEL.
		Occasional sand seams.
	35	Bottom of Boring at 35.0 Feet. Started 09/07/16. Completed 09/07/16.

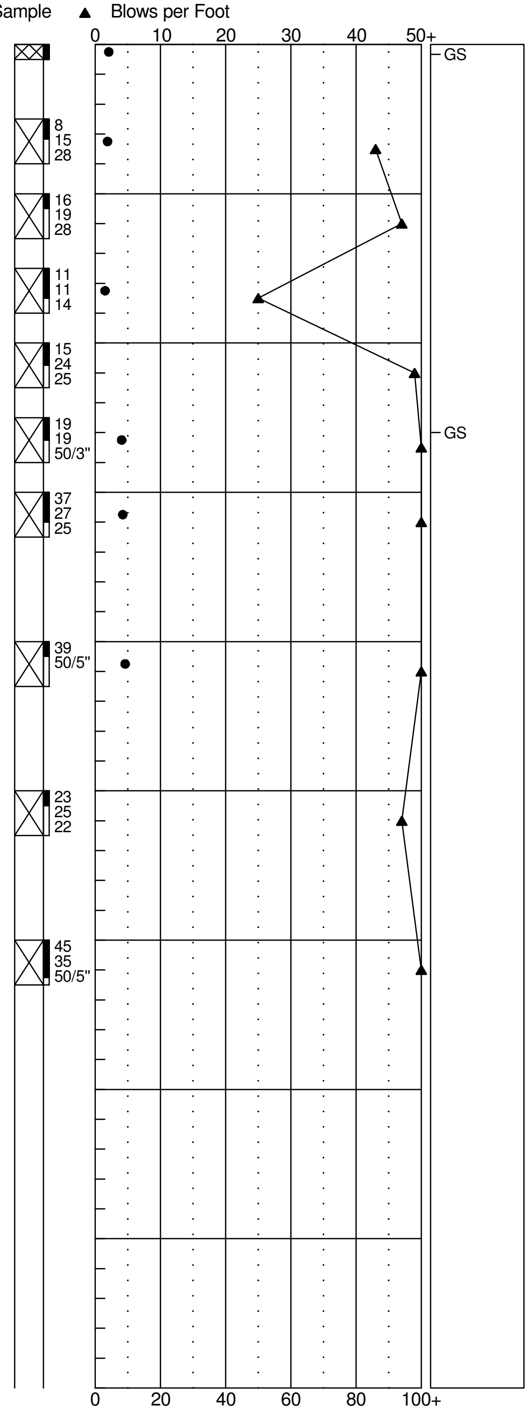
Graphic Log

ATD

STANDARD PENETRATION RESISTANCE

LAB TESTS

Sample	Blows	Lab Tests
S-1		GS
S-2	8 15 28	
S-3	16 19 28	
S-4	11 11 14	
S-5	15 24 25	
S-6	19 19 50/3"	GS
S-7	37 27 25	
S-8	39 50/5"	
S-9	23 25 22	
S-10	45 35 50/5"	



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

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Figure A-11

Boring Log HC-MW-107

Location: N 604881.54 E 1033605.92
Approximate Ground Surface Elevation: 192.23 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: Diedrich D-50/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: B. McDonald   Reviewed By: J. Thomas

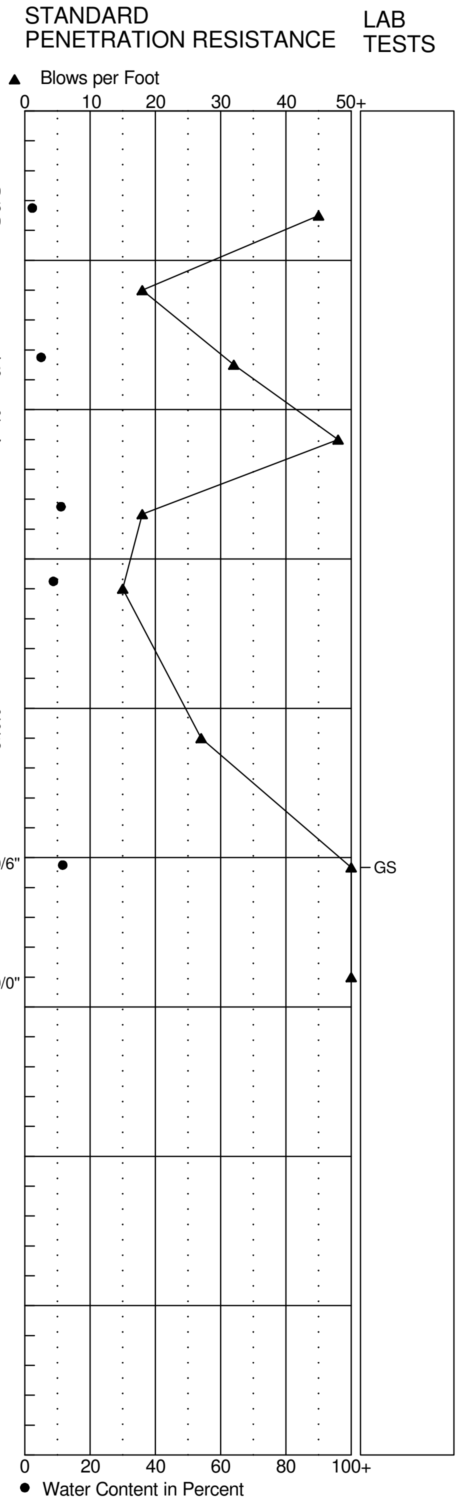
USCS Class	Soil Descriptions	Depth in Feet
SM	3 inches of Forest Duff over (medium dense), moist, brown, silty, fine to medium SAND with scattered small roots.	0
GP	Medium dense, moist to wet, gray-brown, sandy GRAVEL.	2
	Fine to medium sand interbeds from approximately 6 to 8 feet.	
	Scattered cobbles.	
SP-SM	(Dense) to very dense, wet, gray, slightly silty, gravelly SAND with scattered cobbles.	22
	Auger refusal at 29 feet.	
	Bottom of Boring at 29.0 Feet.	29

Started 09/06/16.
Completed 09/06/16.

Ecology Well Tag #BIK-381

Well Construction: Flush mount monument; Concrete; Bentonite chips; 10-20 Silica sand; Screened 1-1/2" PVC. Groundwater: ATD 9/7/16

Sample	Blows
S-1	
S-2	19, 26, 19
S-3	5, 9, 9
S-4	6, 14, 18
S-5	12, 21, 27
S-6	7, 9, 9
S-7	4, 7, 8
S-8	12, 12, 15
S-9	50/6"
S-10	50/0"



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

19202-00 *9/16*

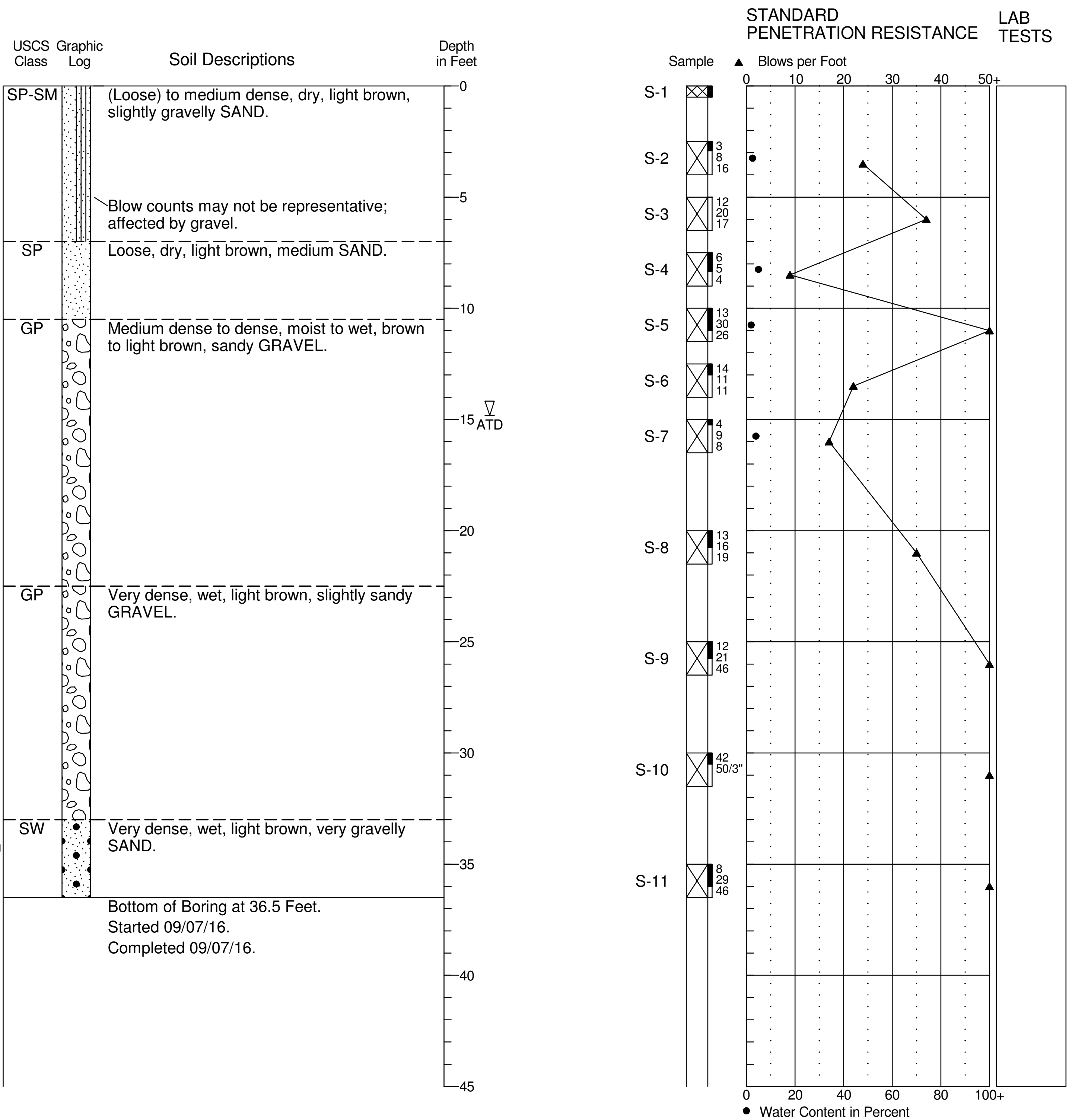
Figure A-12

Boring Log HC-108

Location: N 604754.99 E 1033556.33
Approximate Ground Surface Elevation: 192.99 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: D50 Track Rig/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: M. Miller   Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SP-SM	(Loose) to medium dense, dry, light brown, slightly gravelly SAND. Blow counts may not be representative; affected by gravel.	0
SP	Loose, dry, light brown, medium SAND.	
GP	Medium dense to dense, moist to wet, brown to light brown, sandy GRAVEL.	10
GP	Very dense, wet, light brown, slightly sandy GRAVEL.	
SW	Very dense, wet, light brown, very gravelly SAND.	
	Bottom of Boring at 36.5 Feet. Started 09/07/16. Completed 09/07/16.	36.5



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

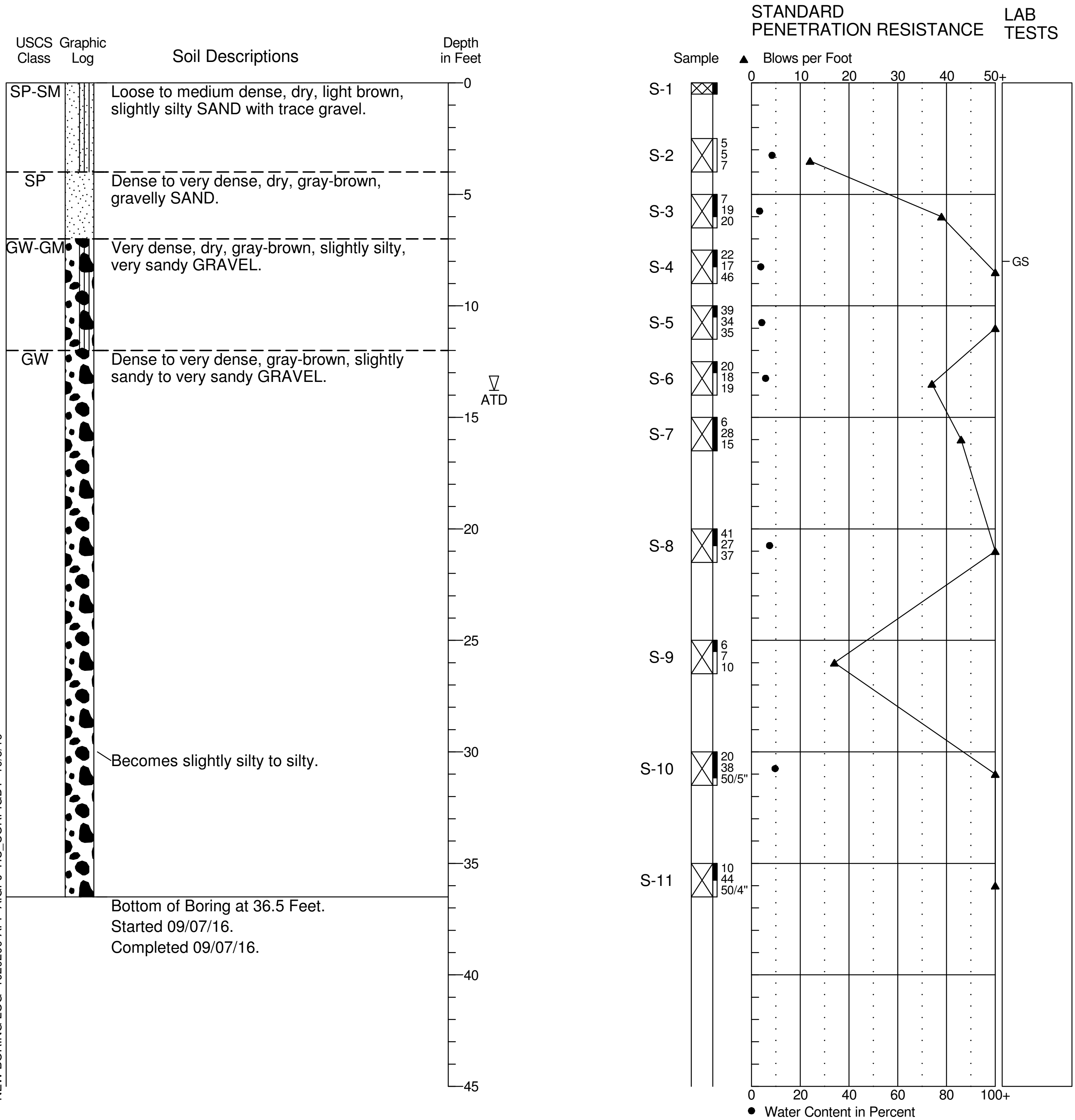
19202-00 *9/16*

Figure A-13

Boring Log HC-109

Location: N 604719.02 E 1033278.85
Approximate Ground Surface Elevation: 191.8 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: D50 Track Rig/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: M. Miller   Reviewed By: J. Thomas



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

19202-00 9/16

Figure A-14

Boring Log HC-110

Location: N 604453.4 E 1033545.28
Approximate Ground Surface Elevation: 196.22 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

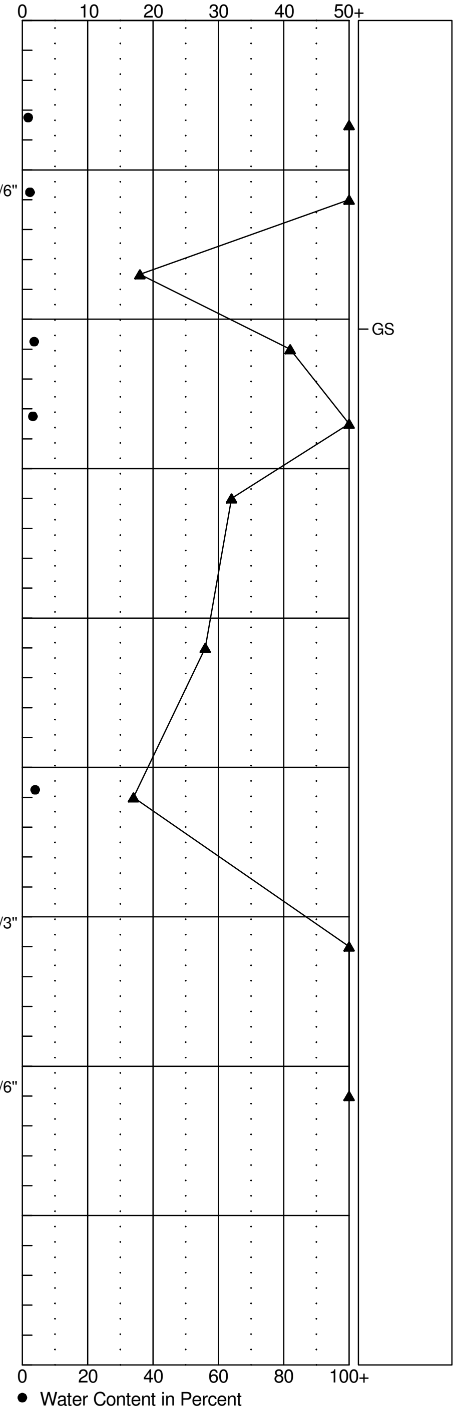
Drill Equipment: D50 Track Rig/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: M. Miller   Reviewed By: J. Thomas

USCS Class	Soil Descriptions	Depth in Feet
SW-SM	Loose, dry, brown, slightly silty, very gravelly SAND.	0
GP	Very dense, dry, gray, sandy GRAVEL.	
SP-SM	Medium dense to dense, moist, gray-brown, slightly silty, gravelly SAND.	
GP	Medium dense to dense, dry, gray-brown, sandy GRAVEL.	
	∇ ATD	15
	Blow counts may not be representative; bouncing off gravels.	30
	Bottom of Boring at 36.5 Feet. Started 09/07/16. Completed 09/07/16.	36.5

Sample	Blows
S-1	
S-2	9, 22, 37
S-3	16, 50/6"
S-4	9, 7, 11
S-5	9, 14, 27
S-6	18, 24, 30
S-7	10, 18, 14
S-8	5, 9, 19
S-9	7, 7, 10
S-10	50/3"
S-11	27, 50/6"

STANDARD PENETRATION RESISTANCE

LAB TESTS



NEW BORING LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
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4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

19202-00 *9/16*

Figure A-15

Boring Log HC-MW-111

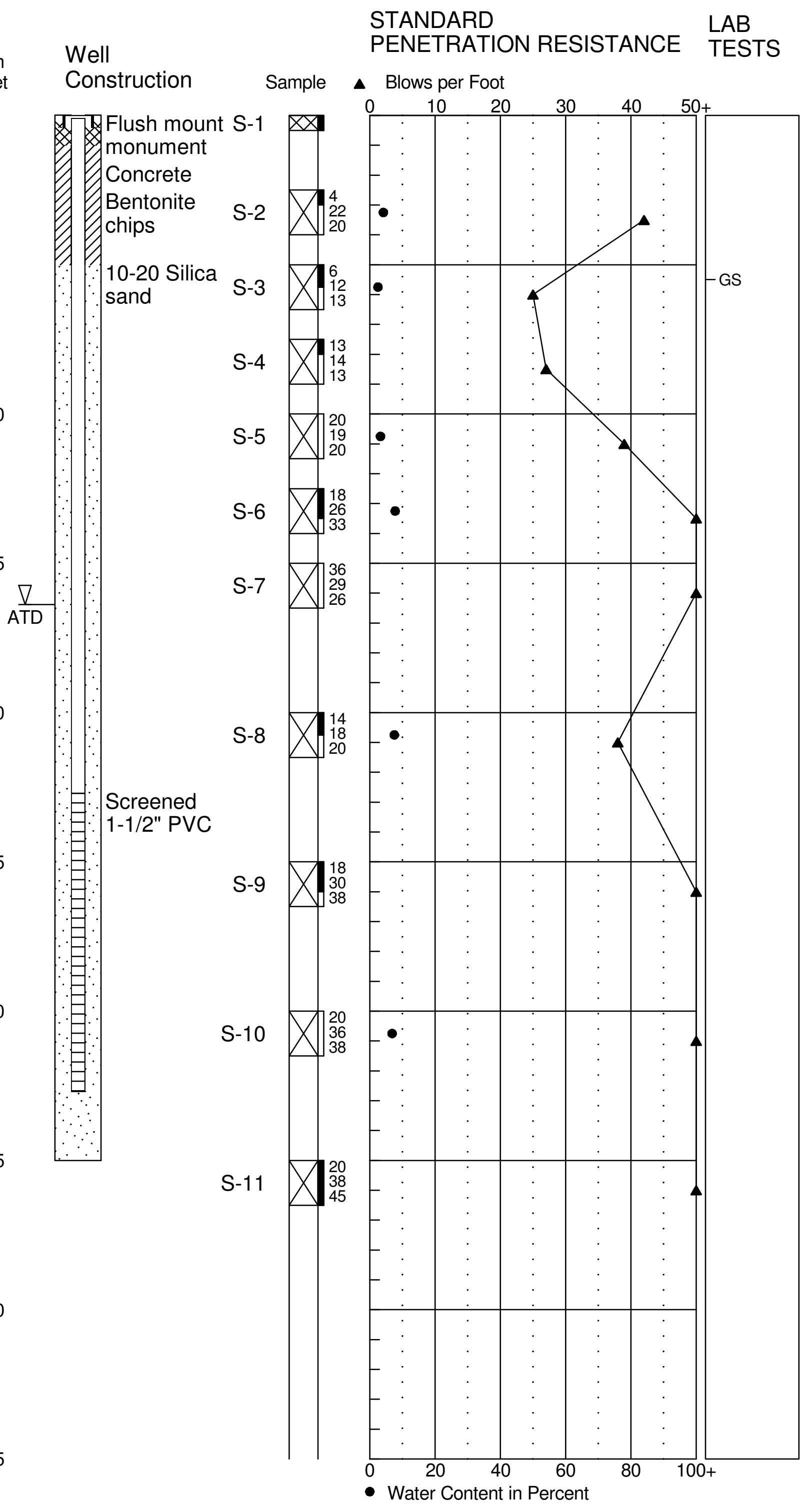
Location: N 604451.93 E 1033308.86
Approximate Ground Surface Elevation: 193.38 Feet
Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

Drill Equipment: D50 Track Rig/HSA
Hammer Type: SPT w/140 lb. Automatic hammer
Hole Diameter: 3-1/4 I.D. inches
Logged By: M. Miller   Reviewed By: J. Thomas

USCS Class	Depth in Feet	Soil Descriptions	Sample	Blows (per 6 in.)	Lab Tests
SP	0	Loose, dry, brown, slightly silty SAND.	S-1		
SW		Medium dense to dense, dry, slightly gravelly to gravelly SAND.	S-2	4, 22, 20	
GP-GM	5	Medium dense to very dense, wet, gray-brown, slightly silty, very sandy GRAVEL.	S-3	6, 12, 13	GS
			S-4	13, 14, 13	
	10		S-5	20, 19, 20	
			S-6	18, 26, 33	
	15		S-7	36, 29, 26	
	20		S-8	14, 18, 20	
	25	Silty sample at 25 feet.	S-9	18, 30, 38	
	30	Gravelly drill action.	S-10	20, 36, 38	
	35	Occasional sand seams.	S-11	20, 38, 45	

Bottom of Boring at 36.5 Feet.
Started 09/07/16.
Completed 09/07/16.

Ecology Well Tag #BIK-382



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.
5. Densities in parentheses are estimated based on field observations. Blow counts may be affected by cobbles and gravels.

HARTCROWSER

19202-00 *9/16*

Figure A-16

Test Pit Log TP-101

Location: N 605198.88 E 1033648.21
Approximate Ground Surface Elevation: 191.88 Feet
Logged By: B. McDonald   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SW-SM	(Dense), moist, gray, slightly silty, very gravelly SAND.	0	S-1	2		GS
SP	(Medium dense), moist, brown SAND with trace gravel and organics.		S-2			
GP	(Medium dense), moist, brownish gray, sandy to very sandy GRAVEL with scattered cobbles and occasional interbedded sand layers.		S-3	1		
		5 ∇	S-4			
			S-5	2		
	Bottom of Test Pit at 10.0 Feet. Started 09/01/16. Completed 09/01/16.	10				
		15				

Test Pit Log TP-102

Location: N 605199.79 E 1033390.48
Approximate Ground Surface Elevation: 190.32 Feet
Logged By: B. McDonald   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GP	(Medium dense), moist, brownish gray, slightly silty, sandy GRAVEL. (FILL)	0	S-1	3		
SM	(Medium dense), moist, grayish brown, silty, gravelly SAND with scattered organics.		S-2			
GP	(Medium dense), moist, brown to gray, sandy to very sandy GRAVEL with cobbles and trace silt.		S-3			
			S-4			
		5				
			S-5	3		GS
			S-6			
	Bottom of Test Pit at 10.0 Feet. Started 09/01/16. Completed 09/01/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

HARTCROWSER

19202-00 *9/16*

Figure A-17

Test Pit Log TP-103

Location: N 605170.22 E 1033943.56
Approximate Ground Surface Elevation: 193.01 Feet
Logged By: B. McDonald Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GP		(Loose), moist, brownish gray, sandy GRAVEL with roots. (FILL)	0	S-1			
SP-SM		(Medium dense), moist, brown, slightly silty SAND with roots.		S-2	7		
			5				
SP		(Medium dense), moist, brownish gray SAND.		S-3			
ML		(Medium stiff), moist, light brownish gray, slightly sandy SILT.		S-4			
GP		(Medium dense), moist, brownish gray, sandy GRAVEL with scattered cobbles.		S-5	3		
		Bottom of Test Pit at 10.0 Feet. Started 09/01/16. Completed 09/01/16.	10				
			15				

Test Pit Log TP-104

Location: N 605099.41 E 1033684.06
Approximate Ground Surface Elevation: 191.73 Feet
Logged By: B. McDonald Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM		Sod over (medium dense), moist, brown, silty SAND with organics and occasional gravel.	0	S-1	7		
				S-2			
GP		(Medium dense), moist, brownish gray, sandy GRAVEL with scattered cobbles.	5	S-3			
				S-4			
		Bottom of Test Pit at 10.0 Feet. Started 09/01/16. Completed 09/01/16.	10				
			15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

HARTCROWSER

19202-00 *9/16*

Figure A-18

Test Pit Log TP-105

Location: N 605100.84 E 1033405.03
Approximate Ground Surface Elevation: 190.6 Feet
Logged By: B. McDonald   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GP	(Medium dense), moist, brownish gray, sandy GRAVEL. (FILL)	0	S-1			
			S-2			
SP-SM	(Medium dense), moist, brown, silty to clean SAND with trace organics.		S-3	8		GS
			S-4			
GP	(Medium dense), moist, brownish gray, sandy GRAVEL with scattered cobbles.	5	S-5			
			S-6	4		
	Bottom of Test Pit at 10.0 Feet. Started 08/30/16. Completed 08/30/16.	10				
		15				

Test Pit Log TP-106

Location: N 605110.24 E 1033288.79
Approximate Ground Surface Elevation: 189.79 Feet
Logged By: B. McDonald   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
GM	(Medium dense), moist, brownish gray, silty, sandy GRAVEL with scattered organics.	0	S-1			
SM	(Loose to medium dense), moist, dark brown, very silty SAND with organics and structural debris. (FILL)		S-2	25		
GP	(Medium dense), moist, brownish gray, very sandy GRAVEL with cobbles.	5	S-3			
			S-4			
SP-SM	(Medium dense), moist, brown, slightly silty SAND.		S-5			
	Bottom of Test Pit at 10.0 Feet. Started 09/01/16. Completed 09/01/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

Test Pit Log TP-107

Location: N 605022.96 E 1033984.81
Approximate Ground Surface Elevation: 193.04 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM	5 inches of Duff over (loose to medium dense), dry, light brown, slightly gravelly, silty SAND.	0	S-1			
GP	(Medium dense), dry, light brown to gray, sandy GRAVEL.		S-2	3		
		5	S-3			
	Bottom of Test Pit at 10.0 Feet. Started 09/09/16. Completed 09/09/16.	10				
		15				

Test Pit Log TP-108

Location: N 604857.02 E 1033983.09
Approximate Ground Surface Elevation: 194.13 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP	6 to 8 inches of Duff over (loose to medium dense), dry, light brown, slightly gravelly SAND with trace organics.	0	S-1			
GP	(Medium dense), dry, light brown, slightly sandy to very sandy GRAVEL with cobbles and trace silt.		S-2			
		5	S-3	2		GS
	Bottom of Test Pit at 10.0 Feet. Started 09/09/16. Completed 09/09/16.	10				
		15				

NEW TEST PIT LOG 1920200-APP-A.GPJ HC_CORP.GDT 10/5/16

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

HARTCROWSER

19202-00   *9/16*

Figure A-20

Test Pit Log TP-109

Location: N 604841.6037 E 1033708.077
Approximate Ground Surface Elevation: 192.89 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP-SM	4 inches of Duff over (loose), dry, light brown, slightly silty SAND with trace gravel and organics.	0	S-1	8		
GP	(Medium dense), dry, light brown, slightly sandy GRAVEL.		S-2			
SP	(Medium dense), damp, gray, slightly gravelly SAND with trace organics.	5	S-3			
SP	(Medium dense), damp, gray SAND with trace gravel.		S-4	4		
	Bottom of Test Pit at 10.0 Feet. Started 09/09/16. Completed 09/09/16.	10				
		15				

Test Pit Log TP-110

Location: N 604775.82 E 1033837.37
Approximate Ground Surface Elevation: 193.91 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP-SM	5 inches of Duff over (loose to medium dense), dry, light brown, slightly silty SAND with gravel.	0	S-1	6		GS
GP	(Medium dense), dry, light brown, sandy GRAVEL.	5	S-2			
SP	(Medium dense), damp, gray SAND with trace gravel.		S-3			
	Bottom of Test Pit at 10.0 Feet. Started 09/09/16. Completed 09/09/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

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Figure A-21

Test Pit Log TP-111

Location: N 604781.15 E 1033431.96
Approximate Ground Surface Elevation: 192.87 Feet
Logged By: B. McDonald   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP	3 inches of Duff over (loose to medium dense), moist, brown, silty SAND with scattered gravel and organics.	0	S-1	8		
SP-SM	(Medium dense), moist, brown, slightly silty, gravelly SAND with scattered organics.		S-2			
GW	(Medium dense), moist, brownish gray, very sandy GRAVEL with cobbles and trace silt.		S-3			
		5				
	Fine to medium sand lens observed.		S-4	4		
			S-5			
			S-6	4		GS
	Bottom of Test Pit at 9.5 Feet. Started 09/01/16. Completed 09/01/16.	10				
		15				

Test Pit Log TP-112

Location: N 604721.4 E 1033120.26
Approximate Ground Surface Elevation: 191.27 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM	8 inches of Duff over (loose to medium dense), dry, light brown, silty SAND with trace gravel.	0				
			S-1	7		
		5				
SP GP	(Medium dense to dense), dry, gray, gravelly SAND to sandy GRAVEL.		S-2			
	Bottom of Test Pit at 10.0 Feet. Started 09/08/16. Completed 09/08/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

Test Pit Log TP-113

Location: N 604352.28 E 1033120.66
Approximate Ground Surface Elevation: 193.04 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP	6 inches of Duff over (medium dense), dry, light brown SAND with trace gravel and silt.	0				
			S-1	7		
SP-SM	(Medium dense to dense), moist, light brown to gray, slightly silty SAND.	5				
			S-2	10		
	Grades to moist to wet.	10				
	Bottom of Test Pit at 11.0 Feet. Started 09/08/16. Completed 09/08/16.	15				

Test Pit Log TP-114

Location: N 604613.16 E 1033432
Approximate Ground Surface Elevation: 194.57 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP	4 to 6 inches of Duff over (loose), dry, brown SAND with organics.	0	S-1			
SM	(Medium dense), dry, brown, silty SAND with trace gravel.		S-2	6		GS
SP	(Medium dense), dry, light brown, slightly gravelly SAND with trace organics.		S-3			
GW	(Loose to medium dense), damp, light brown GRAVEL with trace cobbles and boulders.	5				
			S-4	1		
	Bottom of Test Pit at 10.0 Feet. Started 09/08/16. Completed 09/08/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

Test Pit Log TP-115

Location: N 604562.61 E 1033279.22
Approximate Ground Surface Elevation: 193.18 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM	4 inches of Duff over (loose to dense), dry, brown, silty SAND with organics.	0	S-1			
			S-2	7		
SP	(Dense), moist, gray to brown SAND.	5	S-3			
ML	(Medium stiff to stiff), moist, brown and gray, slightly sandy SILT.		S-4	34		GS
GP	(Dense), moist, dark gray, sandy GRAVEL with trace cobbles.	10	S-5			
	Bottom of Test Pit at 11.0 Feet. Started 09/08/16. Completed 09/08/16.	15				

Test Pit Log TP-116

Location: N 604556.42 E 1033120.5
Approximate Ground Surface Elevation: 192.44 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP-SM	3 to 4 inches of Duff over (loose to dense), dry to damp, brown to gray, slightly silty to silty SAND.	0				
			S-1	7		GS
		5				
GM	(Dense), damp, dark gray, silty, sandy GRAVEL with trace cobbles and boulders.		S-2	4		
	Bottom of Test Pit at 10.0 Feet. Started 09/08/16. Completed 09/08/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

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Figure A-24

Test Pit Log TP-117

Location: N 604454.68 E 1033432.84
Approximate Ground Surface Elevation: 195.15 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SM	6 to 8 inches of Duff over (loose), dry, brown, silty SAND with trace gravel.	0	S-1	8		
GW	(Loose to medium dense), dry, gray, sandy GRAVEL with cobbles.		S-2			
	6- to 12-inch lens of coarse SAND observed.	5				
	6- to 12-inch lens of coarse SAND observed.					
	Bottom of Test Pit at 10.0 Feet. Started 09/08/16. Completed 09/08/16.	10				
		15				

Test Pit Log TP-118

Location: N 604351 E 1033282.96
Approximate Ground Surface Elevation: 194.27 Feet
Logged By: M. Miller   Reviewed By: J. Bruce

Horizontal Datum: WA State Plane S, NAD 83
Vertical Datum: NAVD 88

USCS Class	Soil Descriptions	Depth in Feet	Sample	Water Content in Percent	PID	LAB TESTS
SP	3 inches of Duff over (loose), dry, tan SAND with organics.	0				
SP	(Medium dense), tan to brown SAND with trace silt.		S-1			
		5				
SM	(Medium dense to dense), moist, brown to gray, silty SAND.		S-2	18		GS
	Bottom of Test Pit at 11.0 Feet. Started 09/08/16. Completed 09/08/16.	10				
		15				

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater conditions, if indicated, are at time of excavation. Conditions may vary with time.
5. Densities in parentheses are estimated based on field observations.

APPENDIX B
Laboratory Testing Program

APPENDIX B
LABORATORY TESTING PROGRAM

A laboratory testing program was performed for this study to evaluate the basic index and geotechnical engineering properties of the site soils. Both disturbed and relatively undisturbed samples were tested. The tests performed and the procedures followed are outlined below.

Soil Classification

Field Observation and Laboratory Analysis. Soil samples from the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled laboratory environment. Field and laboratory observations include density/consistency, moisture condition, and grain size and plasticity estimates.

The classifications of selected samples were checked by laboratory tests such as Atterberg limits determinations and grain size analyses. Classifications were made in general accordance with the Unified Soil Classification (USC) System, ASTM D 2487, as presented on Figure B-1.

Water Content Determinations

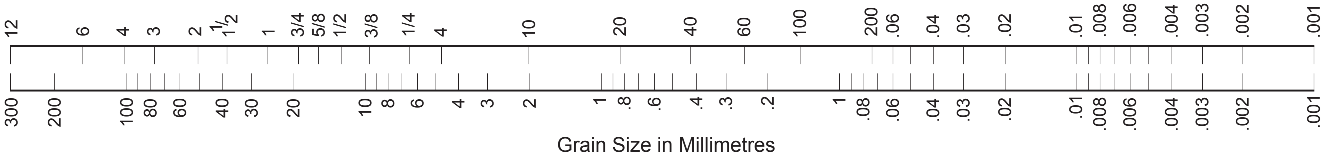
Water contents were determined for most samples recovered in the explorations in general accordance with ASTM D 2216, as soon as possible following their arrival in our laboratory. Water contents were not determined for very small samples nor samples where large gravel contents would result in values considered unrepresentative. The results of these tests are plotted at the respective sample depth on the exploration logs. In addition, water contents are routinely determined for samples subjected to other testing. These are also presented on the exploration logs.

Grain Size Analysis (GS)

Grain size distribution was analyzed on representative samples in general accordance with ASTM D 422. Wet sieve analysis was used to determine the size distribution greater than the U.S. No. 200 mesh sieve. The results of the tests are presented as curves on Figures B-2 through B-10 plotting percent finer by weight versus grain size.

Unified Soil Classification (USC) System
Soil Grain Size

Size of Opening In Inches	Number of Mesh per Inch (US Standard)	Grain Size in Millimetres



COBBLES	GRAVEL	SAND	SILT and CLAY
Coarse-Grained Soils			Fine-Grained Soils

Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
Clean GRAVEL <5% fines		* GRAVEL with >12% fines		Clean SAND <5% fines		* SAND with >12% fines	
GRAVEL >50% coarse fraction larger than No. 4				SAND >50% coarse fraction smaller than No. 4			
Coarse-Grained Soils >50% larger than No. 200 sieve							

G W and S W $\left(\frac{D_{60}}{D_{10}}\right)$ >4 for G W, >6 for S W & $1 \leq \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \leq 3$

G P and S P Clean GRAVEL or SAND not meeting requirements for G W and S W

G M and S M Atterberg limits below A line with PI <4

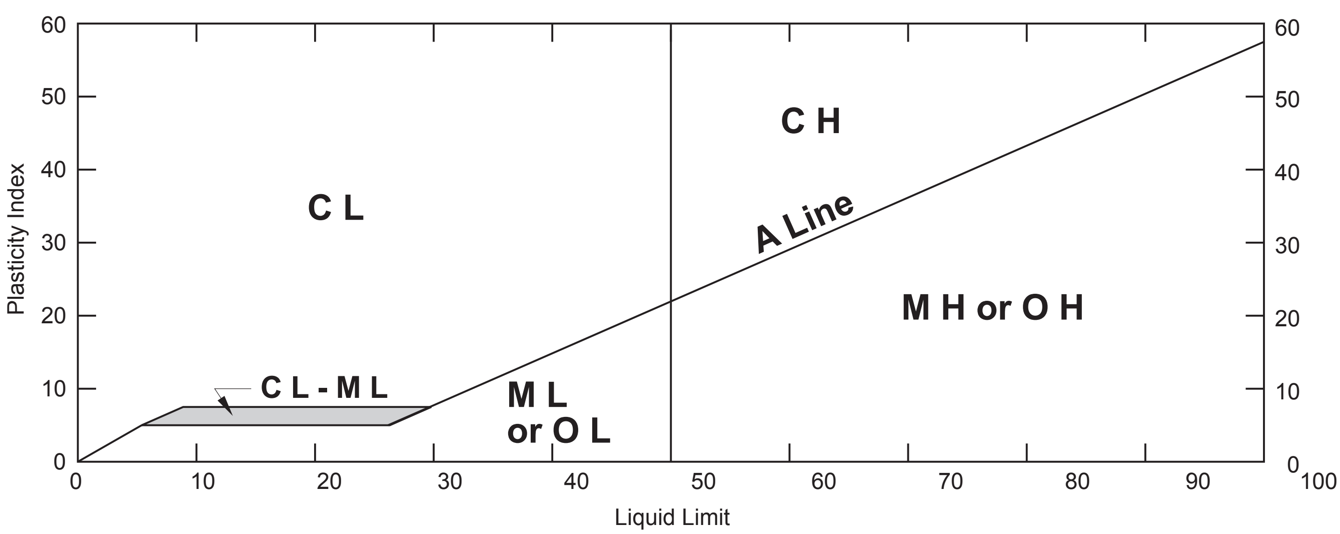
G C and S C Atterberg limits above A Line with PI >7

* Coarse-grained soils with percentage of fines between 5 and 12 are considered borderline cases requiring use of dual symbols.

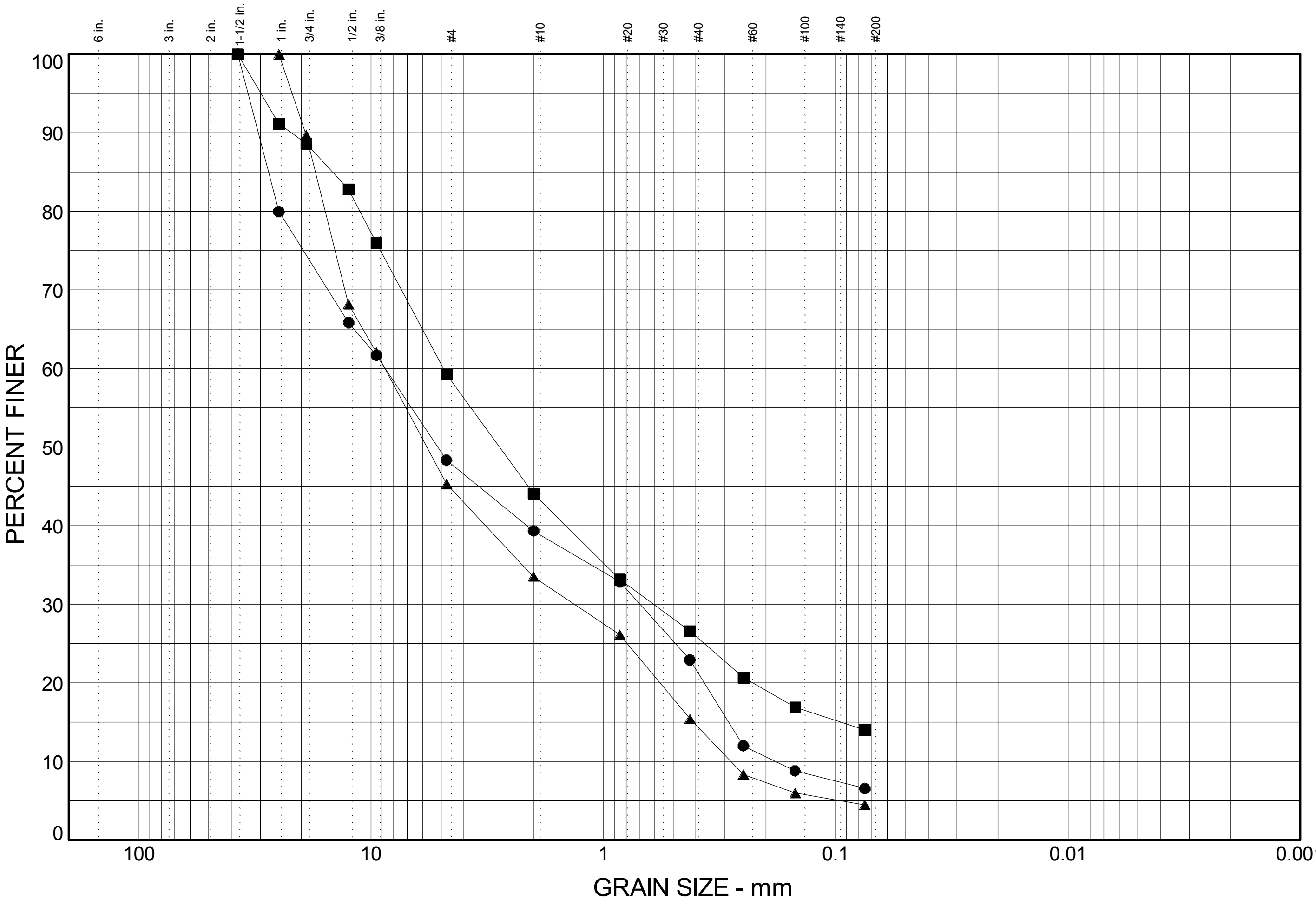
D_{10}, D_{30}, and D_{60} are the particles diameter of which 10, 30, and 60 percent, respectively, of the soil weight are finer.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Highly Organic Soils
Soils with Liquid Limit <50%			Soils with Liquid Limit >50%			
Fine-Grained Soils >50% smaller than No. 200 sieve						



Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	51.6	41.8	6.6	
■	0.0	40.8	45.2	14.0	
▲	0.0	54.7	40.9	4.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			27.679	8.7	5.174	0.696	0.289	0.181	0.31	48.04
■			14.605	4.907	2.805	0.608	0.095			
▲			17.336	8.727	5.765	1.329	0.411	0.283	0.72	30.86

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, very sandy GRAVEL	GP-GM	3.9%
■ silty, very gravelly SAND	SM	8.0%
▲ very sandy GRAVEL, trace silt	GP	2.7%

Remarks:
●

■

▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

	Source	Sample No.	Depth
●	Source: HC-101	Sample No.: S-3	Depth: 5.0 to 6.5
■	Source: HC-101	Sample No.: S-8	Depth: 20.0 to 21.0
▲	Source: HC-102	Sample No.: S-5	Depth: 10.0 to 11.5

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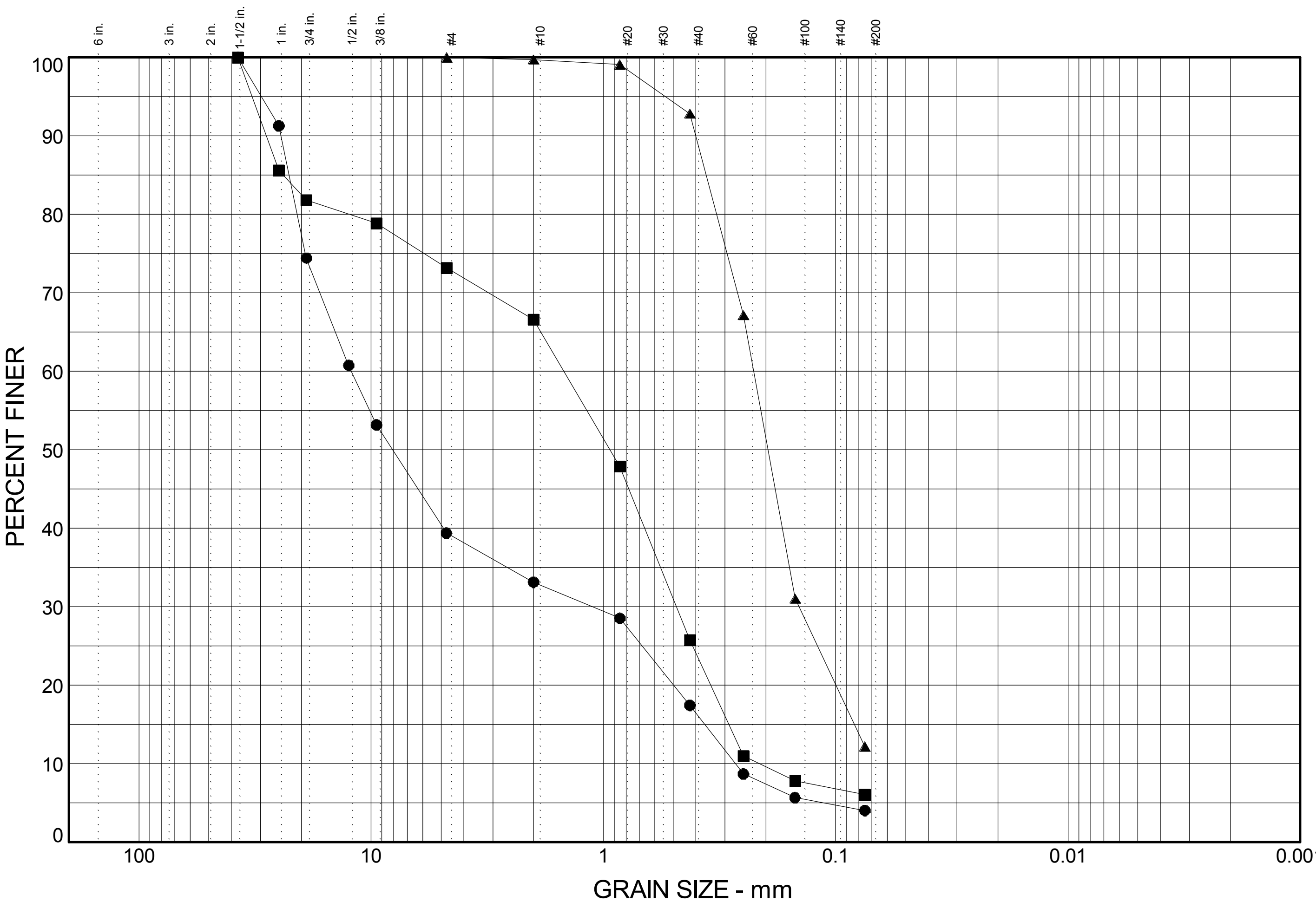
Figure B-2

GRAIN SIZE 1920200-BL-9-16.GPJ HC_CORP.GDT 10/5/16

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	60.6	35.3	4.0	
■	0.0	26.8	67.2	6.0	
▲	0.0	0.0	87.8	12.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			22.566	12.159	8.097	1.116	0.366	0.271	0.38	44.93
■			23.909	1.479	0.936	0.485	0.289	0.213	0.75	6.94
▲			0.361	0.226	0.196	0.144	0.083		1.33	3.27

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● very sandy GRAVEL, trace silt	GP	8.9%
■ slightly silty, gravelly SAND	SP-SM	3.1%
▲ silty SAND	SM	8.2%

Remarks:

●

■

▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

	Source	Sample No.	Depth
●	Source: HC-103	Sample No.: S-6	Depth: 12.5 to 14.0
■	Source: HC-104	Sample No.: S-3	Depth: 5.0 to 6.5
▲	Source: HC-105	Sample No.: S-3	Depth: 5.0 to 6.5

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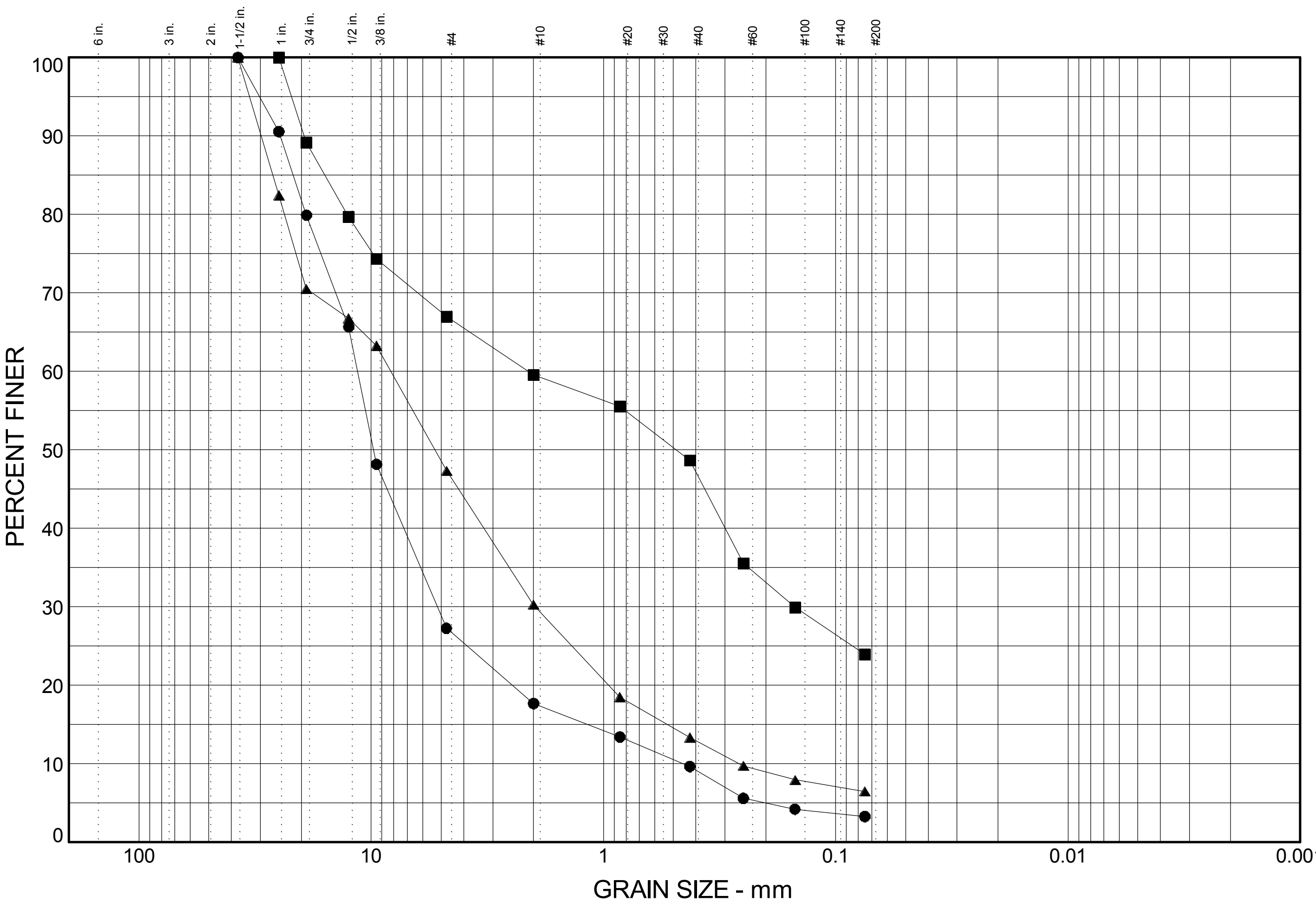
Figure B-3

GRAIN SIZE 1920200-BL-9-16.GPJ HC_CORP.GDT 10/5/16

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	72.7	24.0	3.3	
■	0.0	33.0	43.1	23.9	
▲	0.0	52.7	40.8	6.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			21.669	11.439	9.779	5.203	1.169	0.453	5.22	25.25
■			15.811	2.11	0.485	0.151				
▲			26.528	8.247	5.338	1.962	0.531	0.26	1.80	31.73

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● sandy GRAVEL, trace silt	GP	8.5%
■ silty, very gravelly SAND	SM	4.2%
▲ slightly silty, very sandy GRAVEL	GW-GM	8.1%

Remarks:

●

■

▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: HC-105	Sample No.: S-6	Depth: 12.5 to 14.0
■ Source: HC-106	Sample No.: S-1	Depth: 0.0 to 0.5
▲ Source: HC-106	Sample No.: S-6	Depth: 12.5 to 14.0

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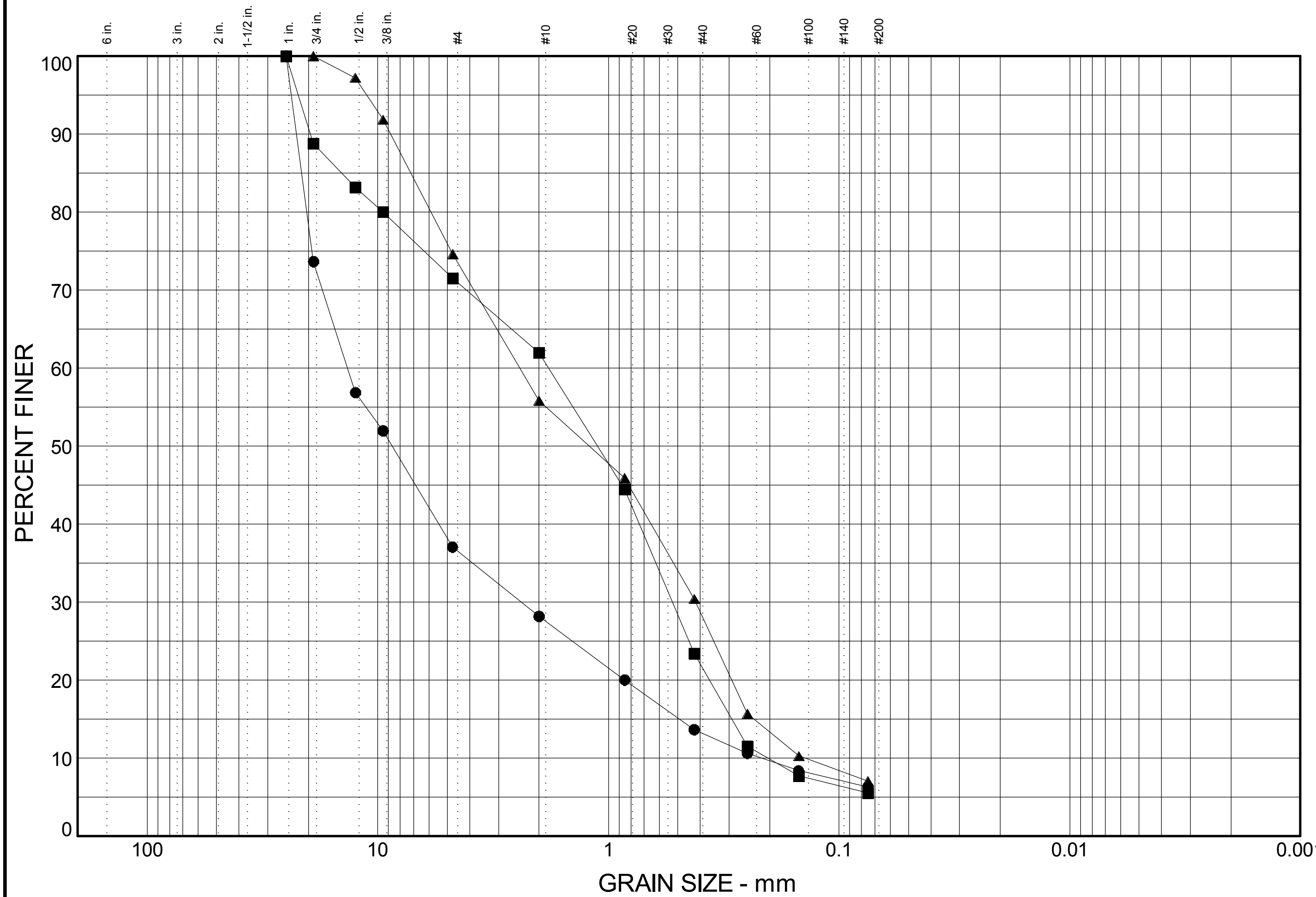
Figure B-4

GRAIN SIZE 1920200-BL-9-16.GPJ HC_CORP.GDT 10/5/16

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	62.9	30.8	6.3	
■	0.0	28.4	66.0	5.5	
▲	0.0	25.4	67.6	7.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			21.383	13.515	8.671	2.387	0.492	0.217	1.95	62.35
■			14.315	1.815	1.113	0.528	0.292	0.204	0.75	8.90
▲			7.206	2.42	1.208	0.419	0.234	0.141	0.51	17.16

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, very sandy GRAVEL	GW-GM	3.8%
■ slightly silty, gravelly SAND	SP-SM	3.6%
▲ slightly silty, gravelly SAND	SP-SM	11.6%

Remarks:
●
■
▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: HC-109	Sample No.: S-4	Depth: 7.5 to 9.0
■ Source: HC-110	Sample No.: S-5	Depth: 10.0 to 11.5
▲ Source: HC-MW-107	Sample No.: S-9	Depth: 25.0 to 25.5

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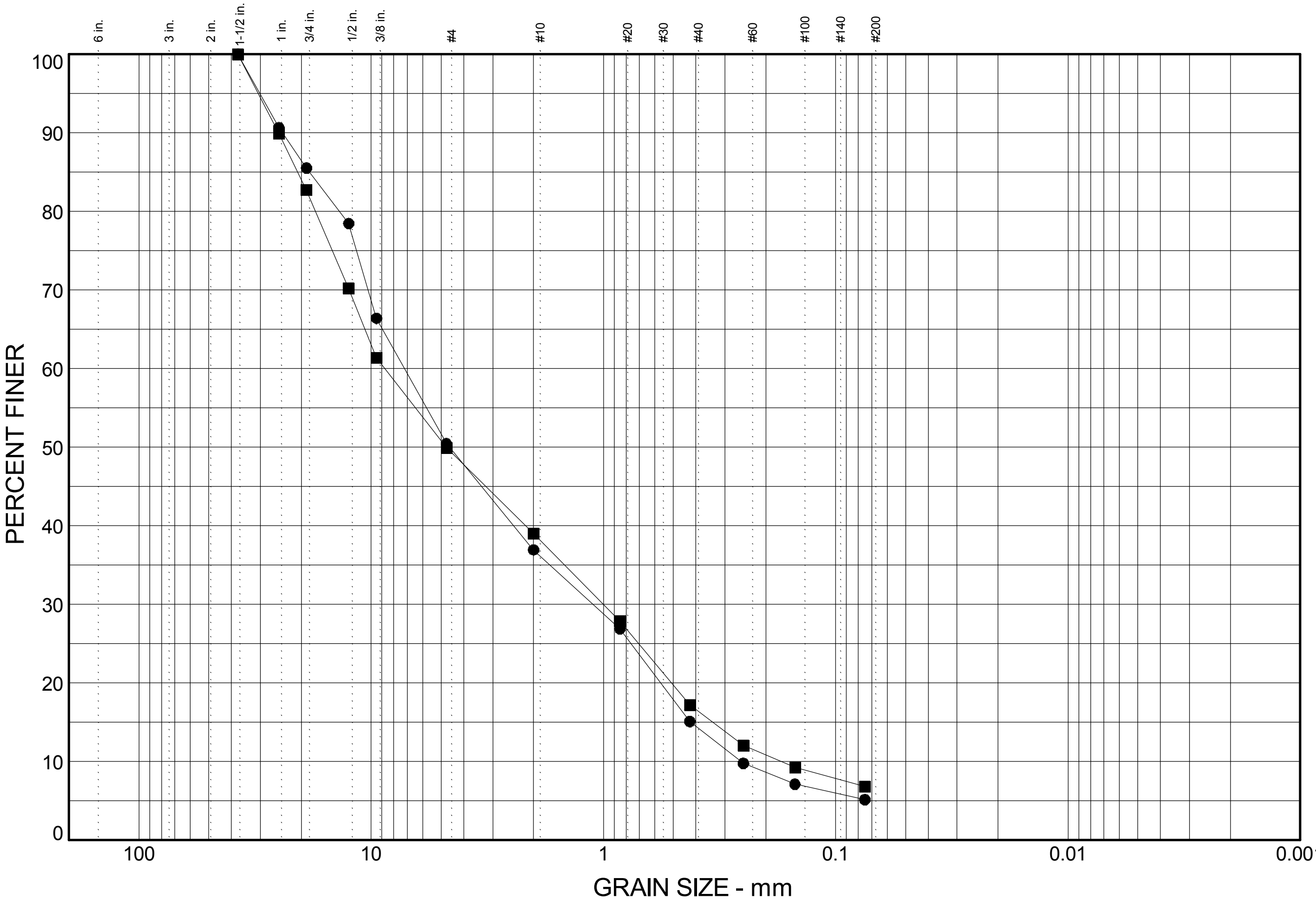
Figure B-5

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	49.5	45.3	5.1	
■	0.0	50.1	43.1	6.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			18.418	7.19	4.612	1.108	0.421	0.256	0.67	28.11
■			20.693	8.748	4.77	1	0.338	0.172	0.66	50.72

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty, very sandy GRAVEL	GP-GM	2.5%
■ slightly silty, very sandy GRAVEL	GP-GM	6.8%

Remarks:
●

■

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: HC-MW-111 Sample No.: S-3 Depth: 5.0 to 6.5
■ Source: HC-MW-111 Sample No.: S-10 Depth: 30.0 to 31.5

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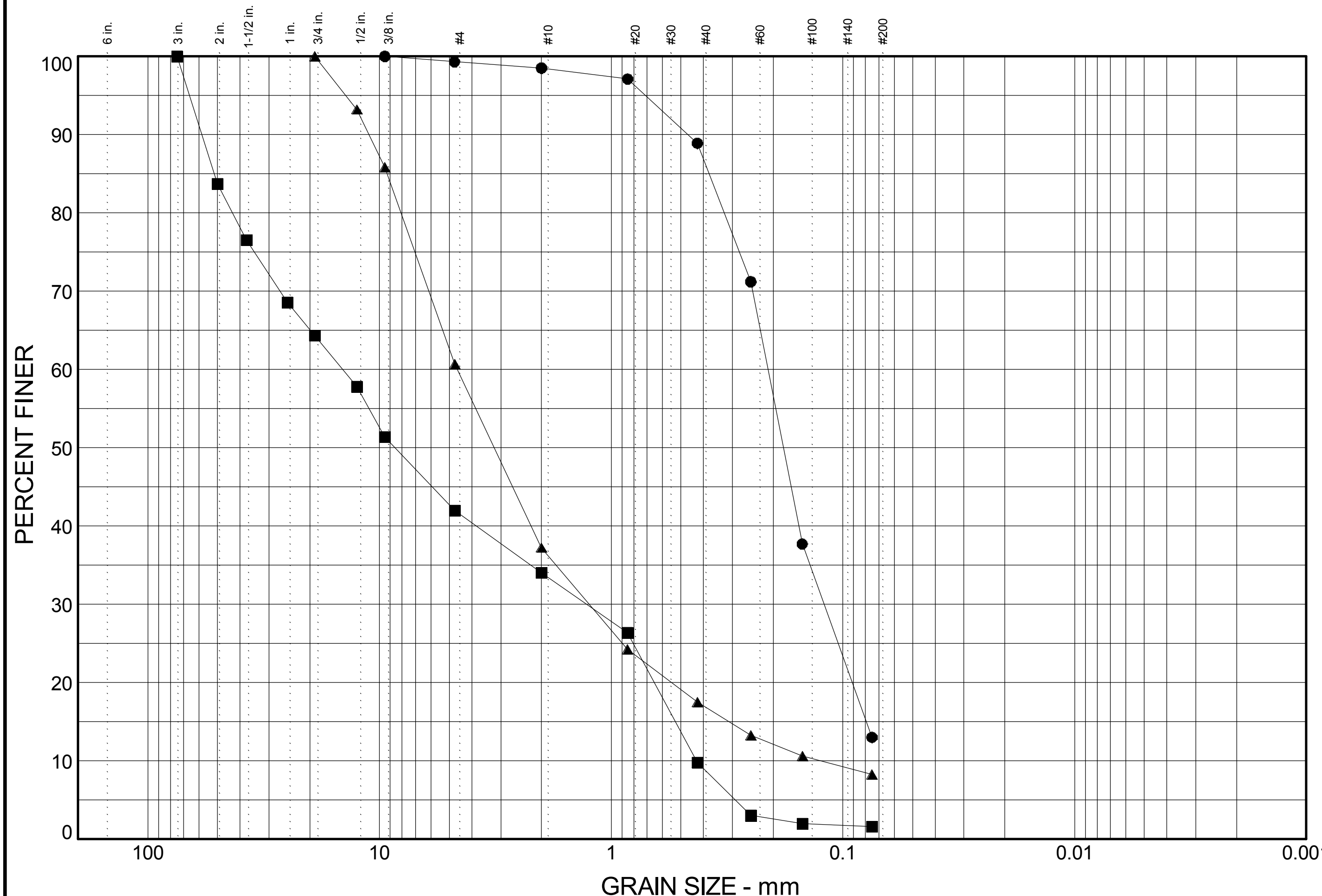
Figure B-6

GRAIN SIZE 1920200-BL-9-16.GPJ HC_CORP.GDT 10/5/16

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.7	86.3	13.0	
■	0.0	58.0	40.4	1.6	
▲	0.0	39.3	52.4	8.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.378	0.211	0.181	0.121	0.079			
■			51.643	14.403	8.568	1.275	0.528	0.429	0.26	33.61
▲			9.281	4.63	3.203	1.243	0.311	0.125	2.66	36.91

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● silty SAND	SM	25.3%
■ very sandy GRAVEL, trace silt	GP	7.1%
▲ slightly silty, very gravelly SAND	SW-SM	1.7%

Remarks:
●

■

▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: PIT-101	Sample No.: S-1	Depth: 2.0 to 3.5
■ Source: PIT-102	Sample No.: S-1	Depth: 4.0 to 5.0
▲ Source: TP-101	Sample No.: S-1	Depth: 0.3 to 1.3

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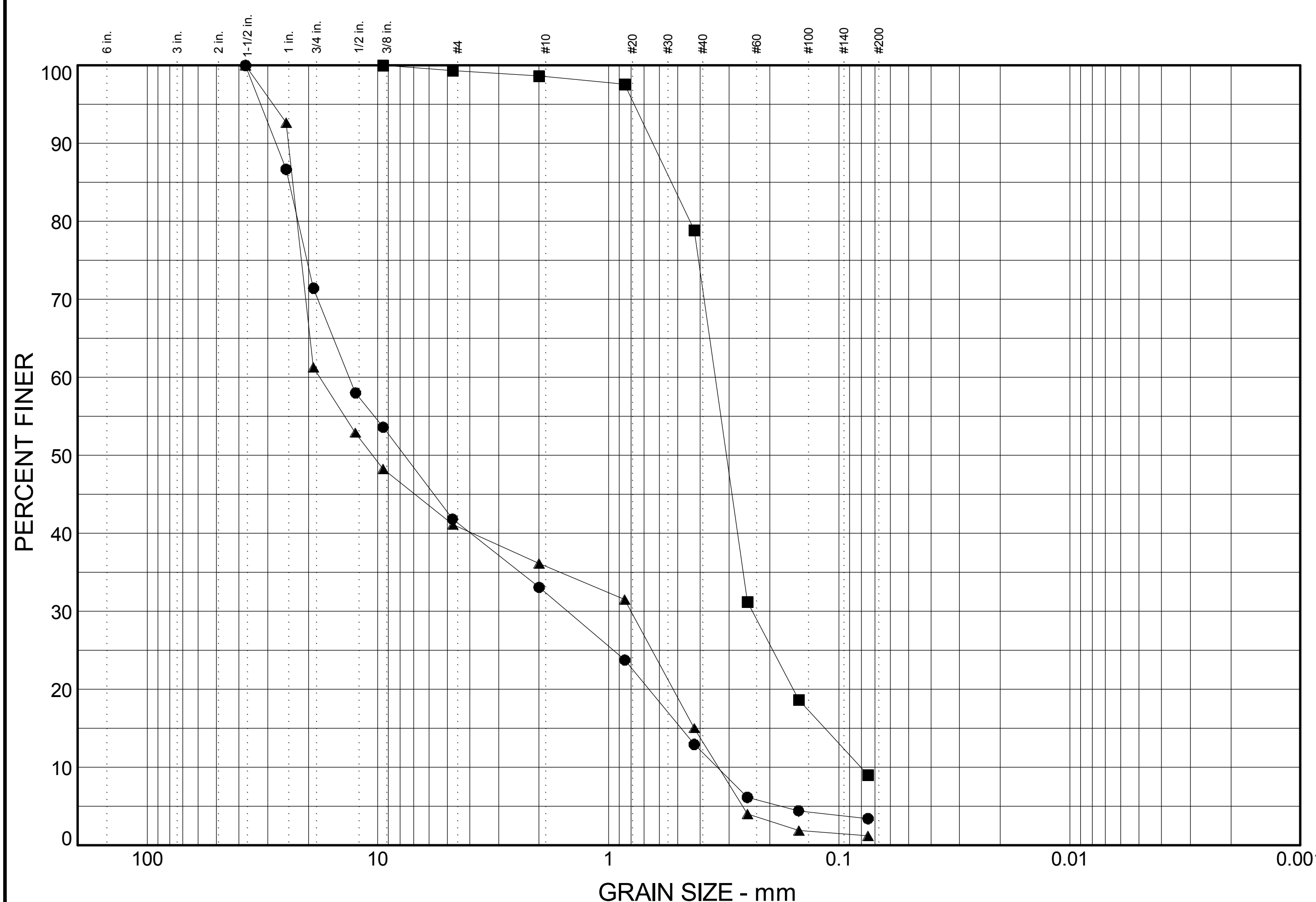
Figure B-7

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	58.2	38.4	3.4	
■	0.0	0.7	90.3	9.0	
▲	0.0	58.9	39.9	1.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			24.258	13.301	7.682	1.507	0.485	0.338	0.51	39.39
■			0.533	0.344	0.308	0.238	0.115	0.08	2.04	4.28
▲			23.379	17.803	10.524	0.797	0.424	0.334	0.11	53.37

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● very sandy GRAVEL, trace silt	GP	3.4%
■ slightly silty SAND	SP-SM	8.4%
▲ very sandy GRAVEL, trace silt	GP	2.2%

Remarks:

●

■

▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: TP-102 Sample No.: S-5 Depth: 7.0 to 8.0
■ Source: TP-105 Sample No.: S-3 Depth: 1.5 to 3.0
▲ Source: TP-108 Sample No.: S-3 Depth: 5.0 to 6.0

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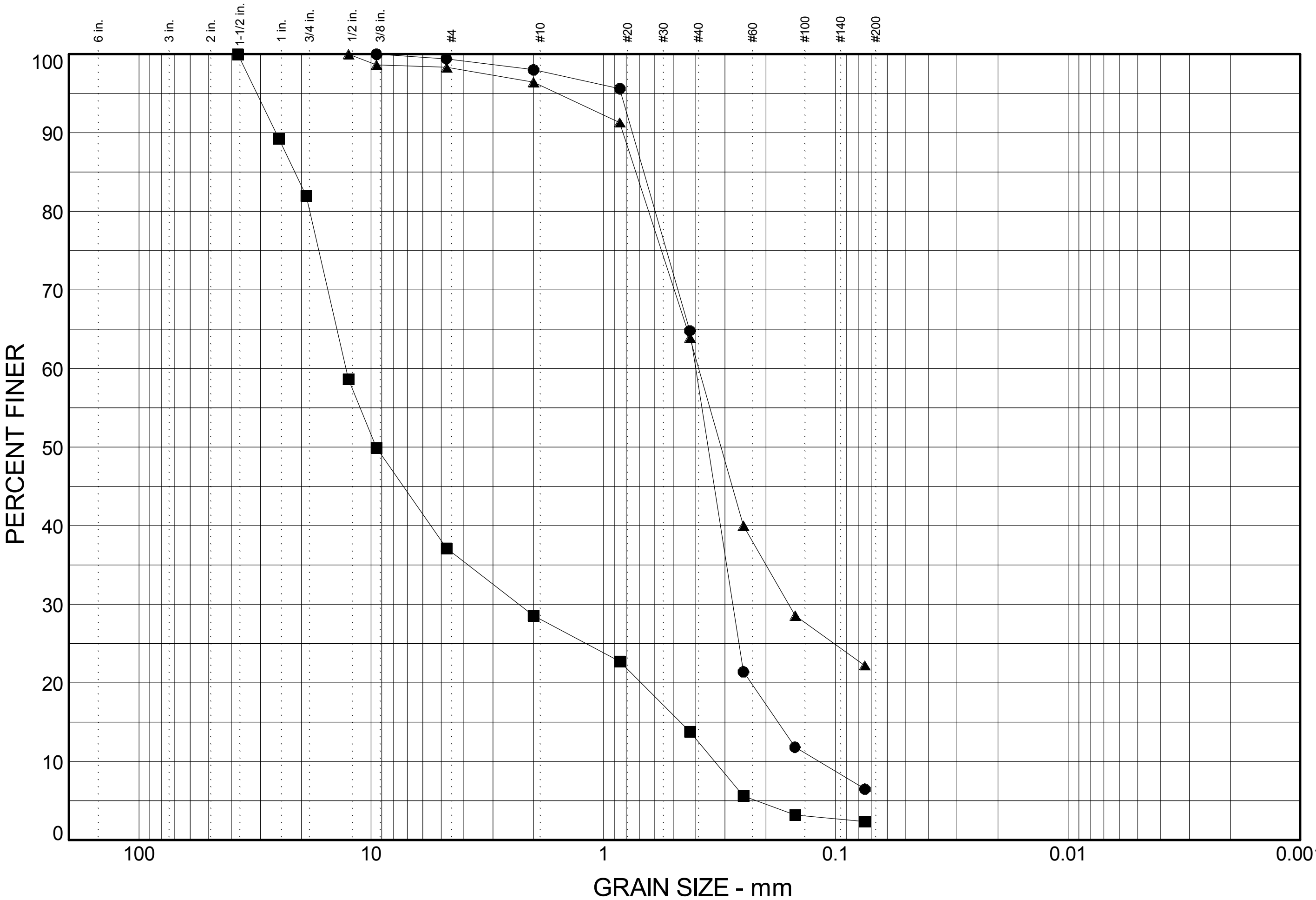
Figure B-8

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.6	92.9	6.5	
■	0.0	62.9	34.7	2.4	
▲	0.0	1.7	76.1	22.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.669	0.401	0.355	0.278	0.178	0.118	1.63	3.39
■			21.302	12.815	9.526	2.314	0.466	0.332	1.26	38.63
▲			0.724	0.389	0.312	0.16				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly silty SAND	SP-SM	6.4%
■ very sandy GRAVEL, trace silt	GW	3.9%
▲ silty SAND, trace gravel	SM	5.8%

Remarks:

●

■

▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: TP-110	Sample No.: S-1	Depth: 2.0 to 3.0
■ Source: TP-111	Sample No.: S-6	Depth: 8.5 to 9.5
▲ Source: TP-114	Sample No.: S-2	Depth: 2.0 to 3.0

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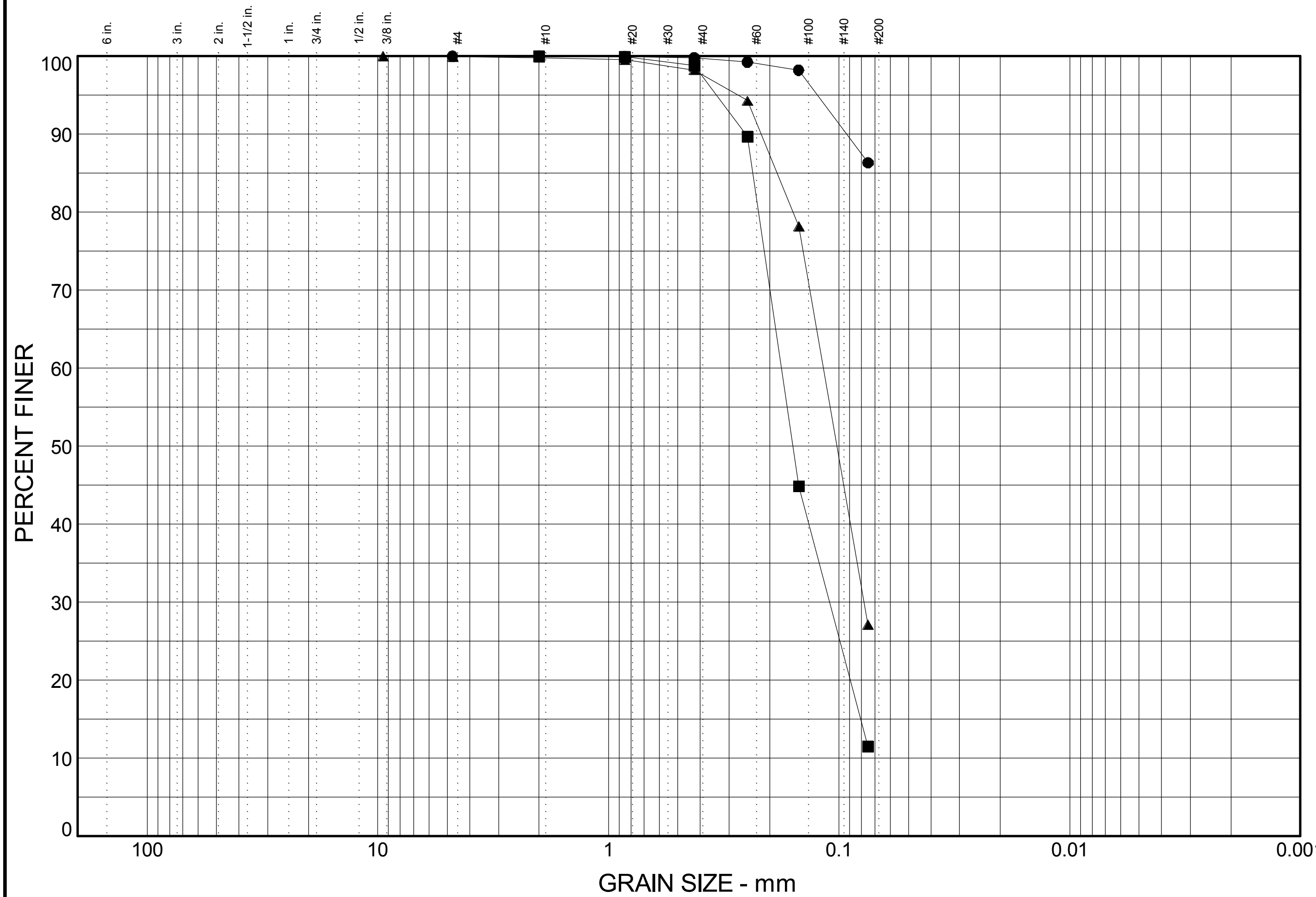
19202-00 *10/16*

Figure B-9

GRAIN SIZE 1920200-TP-9-16.GPJ HC_CORP.GDT 10/5/16

Particle Size Distribution Test Report



	% COBBLES	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	13.7	86.3	
■	0.0	0.0	88.5	11.5	
▲	0.0	0.1	72.8	27.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●										
■			0.237	0.178	0.159	0.11	0.081		0.94	2.45
▲			0.186	0.117	0.102	0.078				

MATERIAL DESCRIPTION	USCS	NAT. MOIST.
● slightly sandy SILT	ML	34.1%
■ slightly silty SAND	SP-SM	7.0%
▲ silty SAND	SM	18.5%

Remarks:
●
■
▲

Project: Tumwater Readiness Center

Client: Schreiber, Starling, and Whitehead Architects, PS

● Source: TP-115	Sample No.: S-4	Depth: 9.0 to 10.0
■ Source: TP-116	Sample No.: S-1	Depth: 2.5 to 3.5
▲ Source: TP-118	Sample No.: S-2	Depth: 8.0 to 9.0

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19202-00 *10/16*

Figure B-10

GRAIN SIZE 1920200-TP-9-16.GPJ HC_CORP.GDT 10/5/16

APPENDIX C
Historical Explorations

APPENDIX C
HISTORICAL EXPLORATIONS

This appendix provides available historical well logs and test pit logs for the site. Logs and test reports by others are included as they were produced by others for reference only and Hart Crowser is not responsible for the accuracy or completeness of the information presented in the logs. Approximate locations of the explorations by others are shown on Figure 2 in the main text; actual locations may differ from those shown.

Test Pit TP-9

Depth (feet)	Material Description
0 – 0.75	**Topsoil/Duff:** Silty SAND with organics: Very loose, moist, dark brown.
0.75 – 3	Silty SAND: Loose, moist, light brown.
3 – 9.5	Gravelly SAND with trace to some silt: Medium dense to dense, moist, brown/gray. (Sample S-1 @ 4 feet)

Test pit completed at approximately 9.5 feet on 1/14/15.
No groundwater observed at time of excavation.
Moderate caving below 4 feet.
Approximate surface elevation: 200 feet

Test Pit TP-10

Depth (feet)	Material Description
0 – 1.5	**Fill:** Gravelly SAND with some silt: Medium dense, moist, brown.
1.5 – 2.5	Silty SAND: Loose, moist, light brown.
2.5 – 6	Gravelly SAND with trace to some silt: Medium dense to dense, moist, brown grading to gray.
6 – 6.5	Gravelly SAND with trace to some silt: Medium dense, moist, red oxidized.
6.5 – 7.5	Sandy GRAVEL with trace silt: Loose to medium dense, reddish brown.

Test pit completed at approximately 7.5 feet on 1/14/15.
Groundwater observed at about 7 feet at time of excavation.
Slight caving below 3.5 feet.
Approximate surface elevation: 200 feet

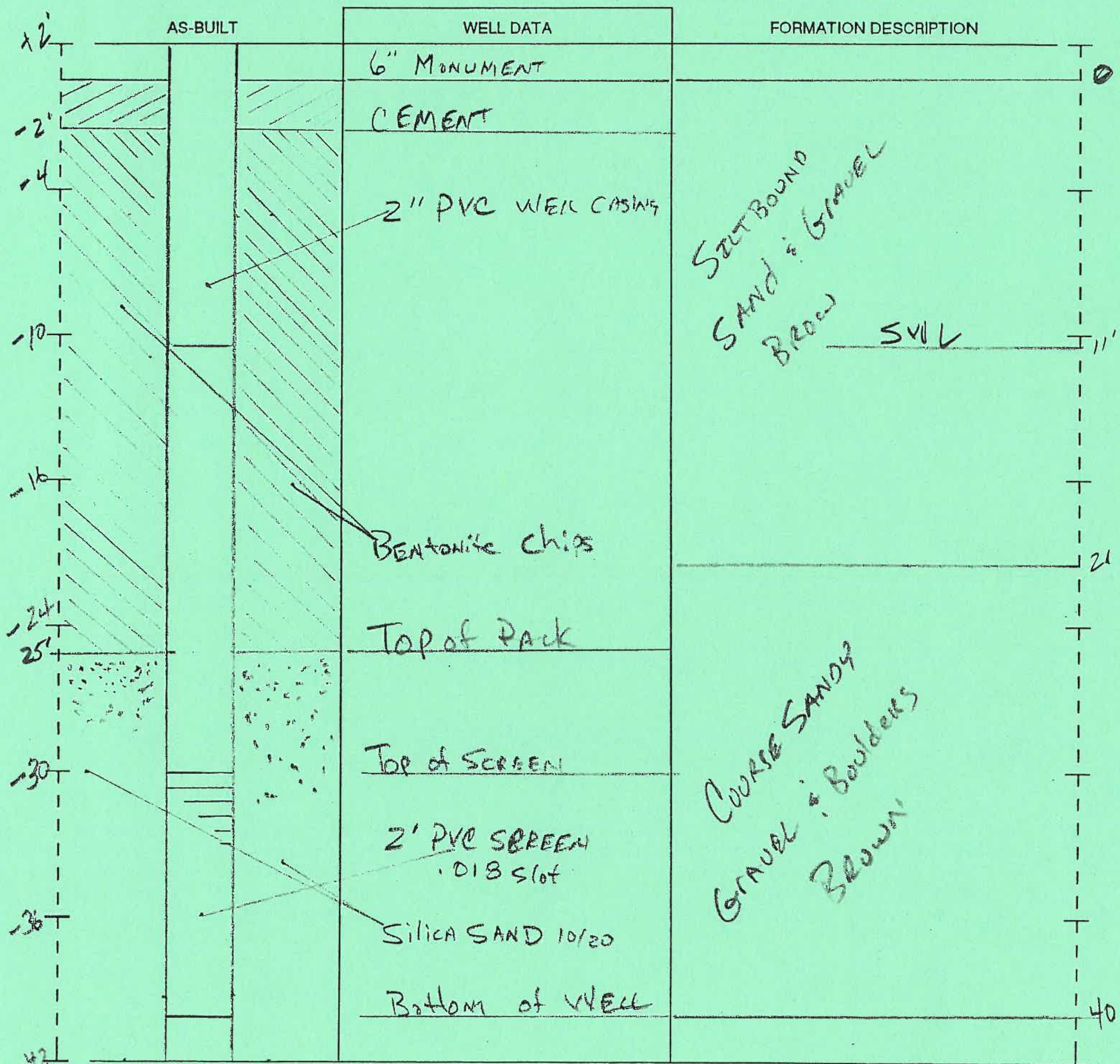
REC'D JAN 21 2008

RESOURCE PROTECTION WELL REPORT

START CARD NO. RE02109

PROJECT NAME: TODD A. HANSEN INC.
WELL IDENTIFICATION NO. BAH815
DRILLING METHOD: Rotary
DRILLER: William M. Neal III #1445
FIRM: Arcadia Drilling Inc.
SIGNATURE: [signature]
CONSULTING FIRM:
REPRESENTATIVE:

COUNTY: THURSTON
LOCATION: NE 1/4 NE 1/4 Sec 16 Twn 17N R 2W
STREET ADDRESS OF WELL: Kimmie Street SW
Tumwater
WATER LEVEL ELEVATION:
GROUND SURFACE ELEVATION:
INSTALLED:
DEVELOPED:



SCALE: 1" = N.T.S.   PAGE 1 OF 1

ECY 050-12 (Rev. 11/89)

APPENDIX D
Infiltration Testing

MEMORANDUM

DATE: February 10, 2017

TO: Ross Whitehead, AIA
Schreiber Starling Whitehead Architects

FROM: Roy Jensen, LHG, Hart Crowser, Inc.

RE: **Pilot Infiltration Tests Analysis and Results**
Tumwater Readiness Center
Tumwater, Washington
19202-00

Hart Crowser performed two pilot infiltration tests to support construction at the proposed Tumwater Readiness Center in Tumwater, Washington. The purpose of the infiltration tests is to determine infiltration rates for design of stormwater infiltration facilities. The infiltration rate obtained from the proposed infiltration tests are considered to be a short-term infiltration rate. Short-term infiltration rates are adjusted through correction factors to account for site variability and number of tests conducted, degree of long-term maintenance and influent pre-treatment/control, and potential for long-term clogging due to siltation and bio-buildup. The infiltration test and analysis procedures are consistent with the test procedures provided in the 2010 City of Tumwater Drainage Design and Erosion Control Manual (December 2009).

Project Background

The soil layers observed during the preliminary field exploration program consisted of the following soil units, described in the order they were encountered from the ground surface down.

- **Loose to Medium Dense Silty Sand.** From the ground surface to a depth generally ranging from 1 to 4 feet below ground surface (bgs) the borings encountered loose to medium dense, slightly silty to silty sand.

- **Medium Dense to Dense Sand and Gravel.** A medium dense to dense sand and gravel unit was encountered directly under the Loose to Medium Dense Silty Sand and extended to depths ranging from 20 to 27 feet bgs. This unit was generally observed to vary between sandy to very sandy gravel and very gravelly sand with trace amounts of silt and layers of sand.

- **Very Dense Sand and Gravel.** A very dense sand and gravel unit was encountered directly under the Medium Dense to Dense Sand and Gravel. This unit was observed to be layers of sand and sandy to very sandy gravel and extended to the bottom of all the borings drilled.

Groundwater was encountered during drilling the borings for preliminary field exploration program. Groundwater levels observed at time of drilling (ATD) ranged from about 7.5 to 11 feet bgs, or approximately elevation 181 to 184 feet.

GENERAL PROCEDURE – PILOT INFILTRATION TEST

The general procedures for the pilot infiltration tests are presented below.

- Excavate a test pit using an excavator to the depth of the bottom of the proposed infiltration test. The dimensions of the test pit for this project were generally 5 feet wide by 5 feet long corresponding to the area of the bottom of the test pit of approximately 25 square feet.

- Document soil conditions observed during excavation and along the side walls of the excavation. Record the size and geometry of the test pit, before beginning the field test.

- Install a vertical measuring rod marked in half inch increments in the pit bottom. The rod was used to record water levels in the test pit.

- Use a rigid 6-inch-diameter pipe with a splash plate on the bottom to convey water to the pit and reduce side wall erosion or excessive disturbance of the pond bottom.

- Conduct the constant head portion of the test by adding water to the test pit at a rate that will maintain a water level between 3 and 4 feet above the bottom of the pit. A flow meter verified with a bucket test was used to measure the flow rate into the pit.

- Record the cumulative volume and instantaneous flow rate in gallons per minute (gpm) necessary to maintain the water level at the same point (1 foot) on the vertical measuring rod. Water levels in the test pits were also monitored with a pressure transducer.

- Continue adding water to the pit while maintaining constant water level in the test pit for 6 to 8 hours.

- At the end of the constant head test, water flow into the test pit was turned off and the drop in water level was recorded for a period of at least 1 hour. This phase of the infiltration test is referred to as the falling head test.

TEST RESULTS

Two infiltration tests were completed at the site between September 12 and 16, 2016. The locations of the infiltration test pits are shown on Figure 1. The results of the individual infiltrations tests are summarized below.

Infiltration Test PIT-102

- The dimensions of Infiltration PIT-102 were about 5 by 5 feet and 2 feet deep. Soils observed in the test pit include soil cover with rootlets (0 to 2 feet) and sandy Gravel at the bottom of the test cell.

- Infiltration Test 1 was conducted on September 12, 2016. The constant head test was conducted for nearly 12 hours starting at 13:50. and ending at 19:12. The following falling head test was monitored from 19:12 until 20:08.

- During the constant head test, the water level in the test pit was maintained at approximately 1 foot at a flow rate of between 5 to 5.7 gpm. The average flow rate was 5.4 gpm. Water levels and flow rates during the constant head test are presented on Figure 2.

- During the falling head test, water levels dropped from 1 foot to 0.1 foot in 86 minutes. Water levels monitored during the falling head test are presented on Figure 3.

- The results of the constant head test indicate that at a constant head of 1 foot, the field infiltration rate is 0.2 gpm/ft^2 or 20 inches per hour (in./hr).

Infiltration Test PIT-101

- The dimensions of Infiltration Test PIT-101 were about 5 by 5 feet and 1.5 feet deep. Soils observed in the test pit were slightly silty fine to medium Sand.

- Infiltration Test PIT-101 was conducted on September 15, 2016. The constant head test was conducted for nearly 12 hours starting at 8:00 and ending at 16:00. The following falling head test was conducted from 16:00 until 16:46.

- Water levels were maintained at approximately 1 foot at flow rate of 2.32 to 3.36 gpm during the constant head test. At the end of the test the average flow rate was 2.6 gpm. Water levels and flow rates during the constant head test are presented on Figure 4.

- During the falling head test water levels dropped from 1 foot to 0.1 foot in about 90 minutes. Water levels during the falling head test are presented on Figure 5.

- The results of the constant head test indicate that at a constant head of 1 foot the field infiltration rate is 0.1 gpm/ft^2 or 10 in./hr.

SUMMARY AND CONCLUSIONS

- Two pilot infiltration tests were completed in the study area. Soil encountered in the test pits consist of an upper unit of sandy Gravel to gravelly Sand with organics and outwash gravel unit generally consisting of very sandy Gravel. The infiltration tests were conducted in the top of the outwash gravel unit.

- The infiltration tests consisted of a constant head test and a falling head test. The field infiltration rates based on the constant head tests ranged from 10 to 20 in./hr (Table 1).

- Infiltration rates are head dependent. The higher the head, the higher the infiltration rate. The infiltration rates developed in this study are based on a head of 1 foot.

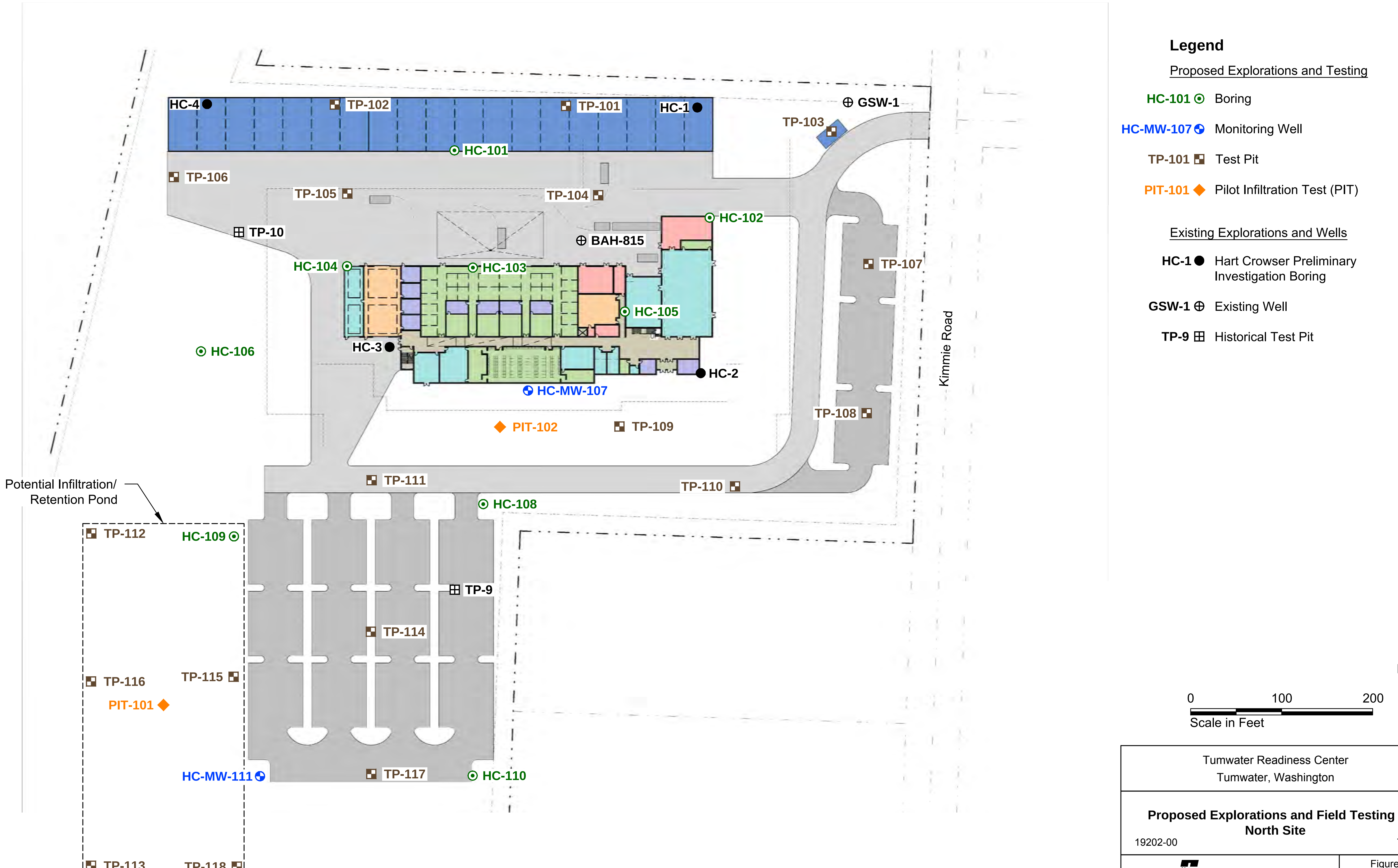
- For design purposes, a correction factor of about 3 was used to adjust the infiltration rates to develop design infiltration rates in shallow soil units at the site. The design infiltration rate for the sandy gravel unit based on Infiltration Test PIT-102 is 7 in./hr and the design infiltration rate for the silty sand unit based on Infiltration Tests PIT-101 is 3 in./hr.

Attachments:

Table 1 - Summary of Infiltration Test Results
Figure 1 - Infiltration Test Location Map
Figure 2 - Infiltration Test PIT-102 - Constant Head Test
Figure 3 - Infiltration Test PIT-102 - Falling Head Test
Figure 4 - Infiltration Test PIT-101 - Constant Head Test
Figure 5 - Infiltration Test PIT-101 - Falling Head Test

Table 1 - Summary of Infiltration Test Results
Tumwater Readiness

Infiltration Test Number	Length in Feet	Width in Feet	Area in Square Feet	Steady-State Head in Feet	Steady-State Flow in gpm	Infiltration Rate in gpm/ft^2	Infiltration Rate in in./hr	Correction Factor	Scaled Infiltration Rate in in./hr	Recommended Design Infiltration Rate in in./hr
PIT-101	5.25	5.0	26.3	1	2.6	0.1	9.6	3	3	3
PIT-102	5.25	5.0	26.3	1	5.4	0.2	19.9	3	7	7



Legend

Proposed Explorations and Testing

HC-101 Boring

HC-MW-107 Monitoring Well

TP-101 Test Pit

PIT-101 Pilot Infiltration Test (PIT)

Existing Explorations and Wells

HC-1 Hart Crowser Preliminary Investigation Boring

GSW-1 Existing Well

TP-9 Historical Test Pit

Tumwater Readiness Center
Tumwater, Washington

Proposed Explorations and Field Testing - North Site

19202-00

7/16

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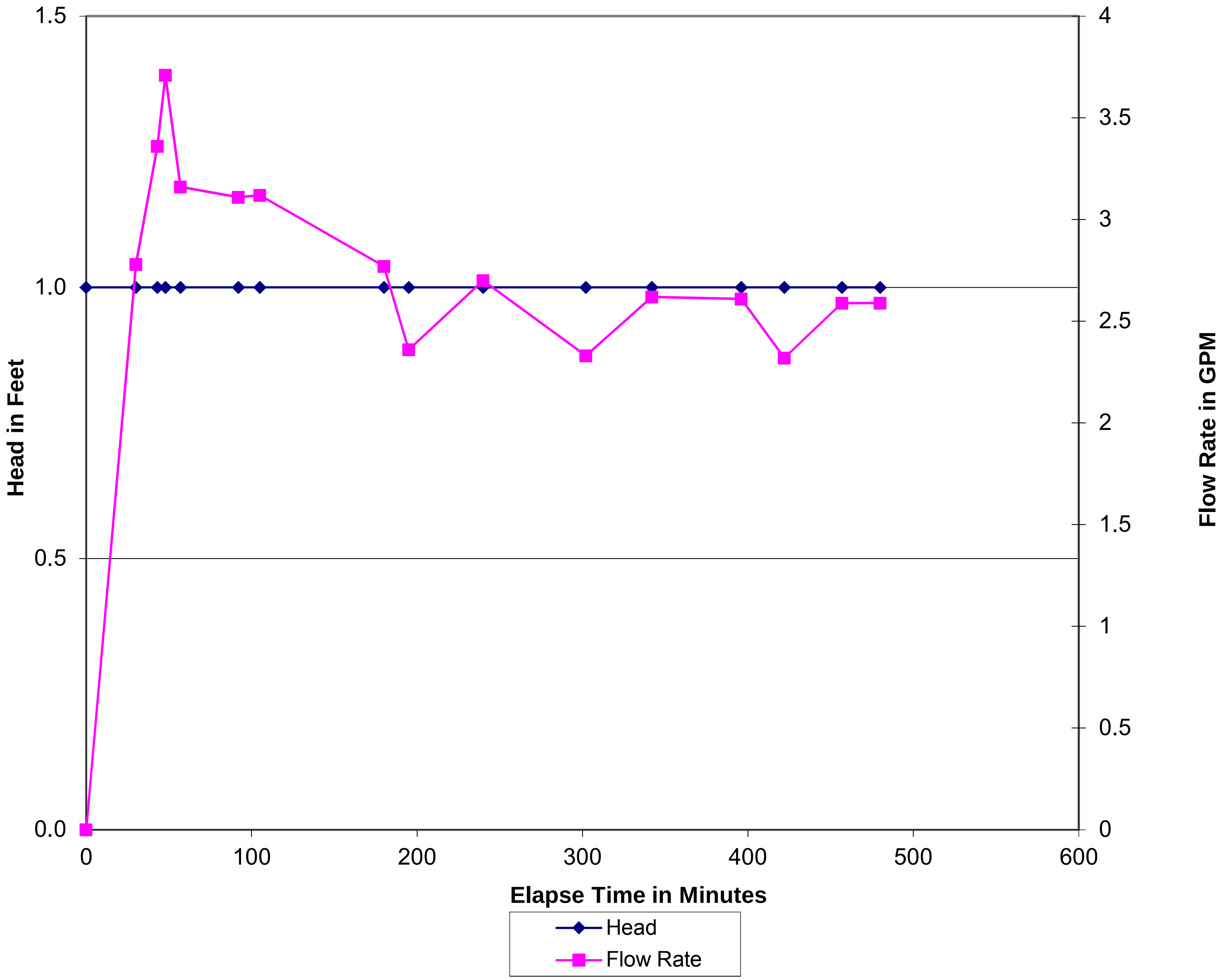
Figure

1

EAL 07/22/16 1920200-006 (SPlan).dwg



Tumwater Readiness Center

Infiltration Test PIT-102
Constant Head Test

19202-00

9/16

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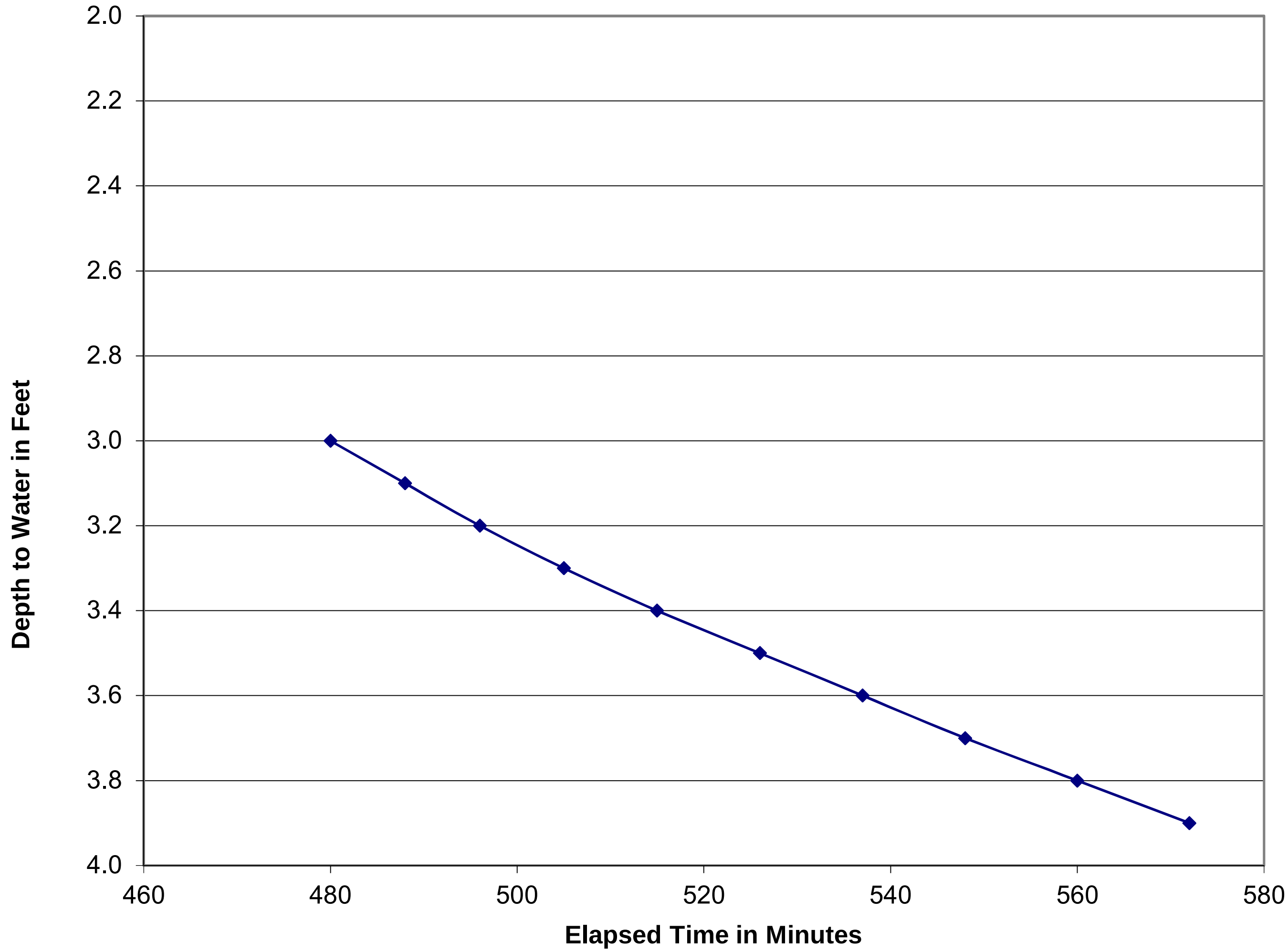
Figure
2



Tumwater Readiness Center

Infiltration Test PIT-102
Falling Head Test

19202-00

9/16

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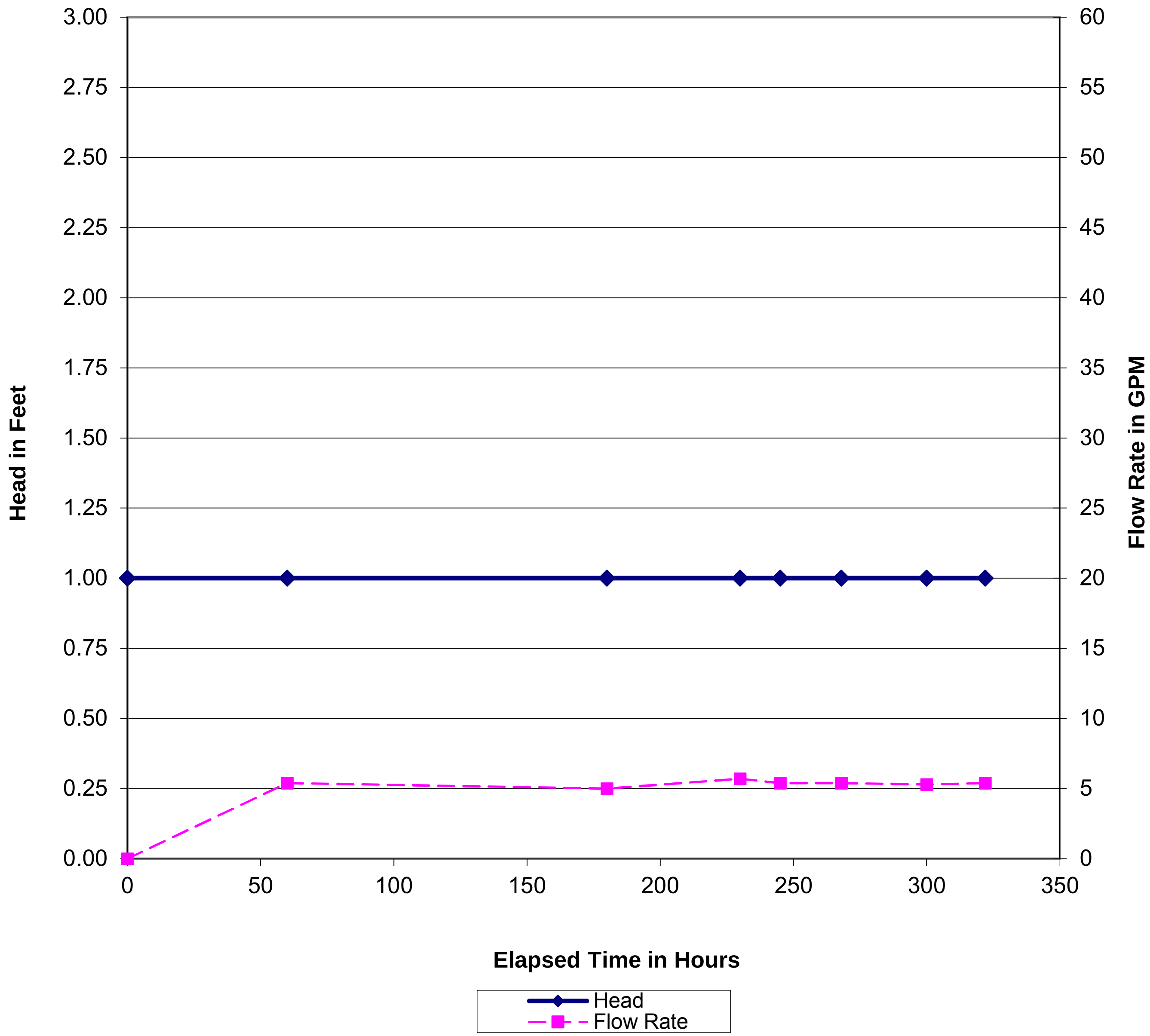
Figure
3

sde 09/15/08 j:\jobs\1749300\Infiltration Tests Figures.xls



Tumwater Readiness Center

Infiltration Test PIT-101
Constant Head Test

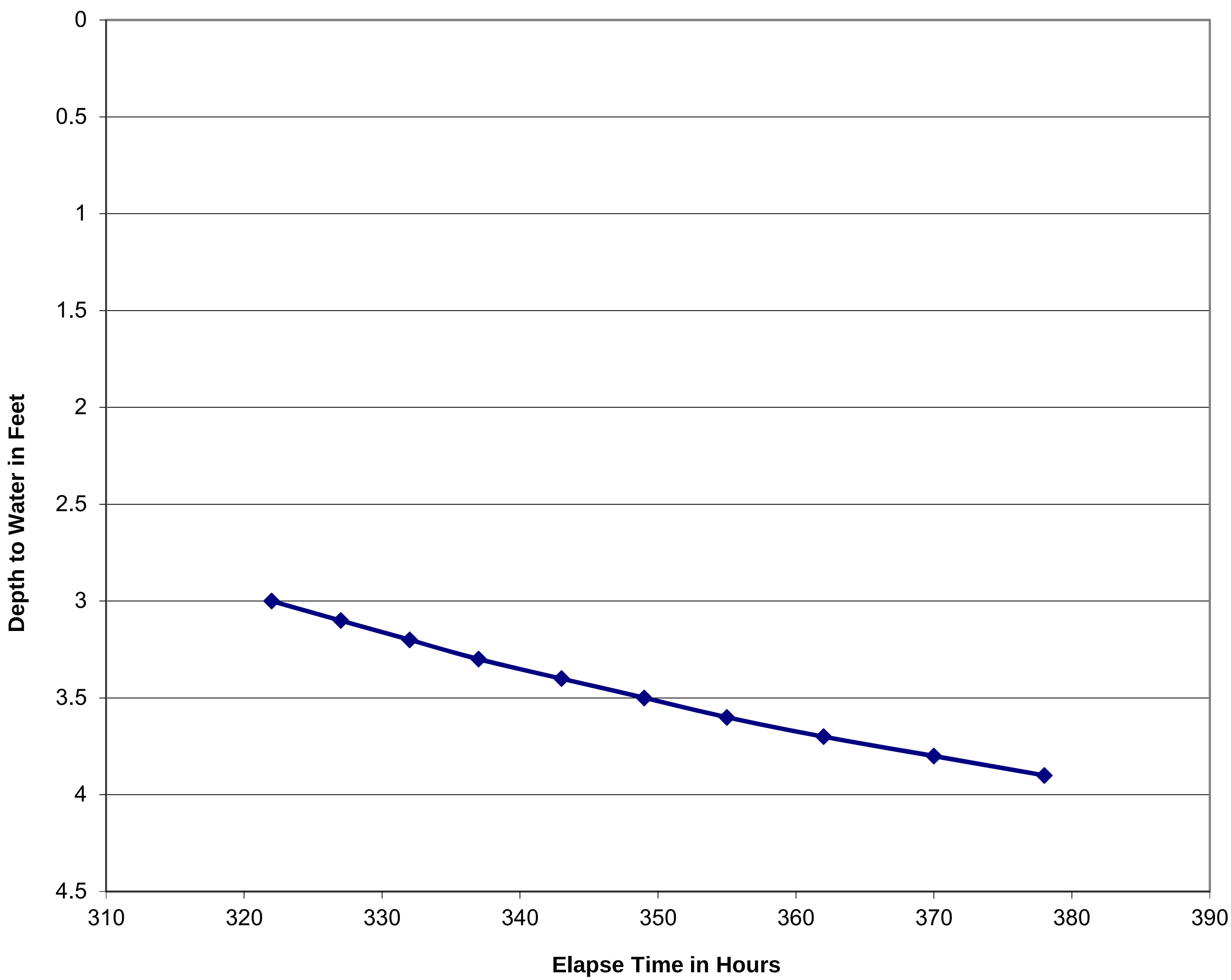
19202-00

9/16

HARTCROWSER

Figure
4

sde 09/15/08 j:\jobs\1749300\Infiltration Tests Figures.xls



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19202-00

9/16

Infiltration Test PIT-101
Falling Head Test

Tumwater Readiness Center

Figure
5

APPENDIX H

STORMWATER SITE PLAN REVIEW, AHBL INC., 2015

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May 26, 2015
Revised June 3, 2015

Mr. Thomas Skjervold
Environmental Programs Manager
Washington Military Department
Building 36, Quartermaster Road
Camp Murray, WA 98430-5050

Project: Tumwater Readiness Center Site Feasibility Study, AHBL No. 2140515.10
Subject: Stormwater Site Plan Review

Dear Tom:

We are pleased to provide you with our revised stormwater analysis of the pre-design site plan for the Kimmie Street Property located in Tumwater, Washington. This letter summarizes our review of the site plan in relation to our Schematic Stormwater Design and Site Grading letter that was revised in March of this year.

The major components of this review include the following:

- Can we retain 65 percent of the site as native vegetation?
- Is stormwater dispersion being used to the maximum extent practicable? If not, how can we modify the site plan to utilize dispersion?
- What are the anticipated earthwork quantities associated with the pre-design site plan?
- Preliminary review of a Field Maintenance Shop (FMS) site at the north end of the property.

Pre-Design Site Plan

The Pre-Design Site Plan includes a Readiness Center and Vehicle Storage Shed with supporting Military Vehicle Parking and POV parking areas, and associated internal access roads and standoff areas. The site statistics are summarized in the table below:

	Readiness Center	**Vehicle Shed**	**Military Vehicle Parking**	**POV Parking**	**FMS[1]**	**Total Development Footprint[2]**
Proposed (sf)	55,000	39,000	90,000	96,000	N/A	640,000
Future Expansion (sf)	16,000	N/A	N/A	N/A	67,000	805,000

[1] The FMS area is based on the Seattle FMS, including a planned 24-foot wide expansion.
[2] The Total Development Footprint includes impervious surfaces, as well as existing and anticipated disturbed areas and stormwater facilities.

Based on the current site plans, we anticipate that the fully developed site will create 9.5 acres of impervious surfaces within a development footprint of approximately 16 acres. The 16 acres include the future Readiness Center expansion and FMS and associated access roads. An additional 2.5 acres of the 52.9-acre property is existing gravel surface, for a total footprint of 18.5 acres.

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Tacoma, WA 98403-3350
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www.ahbl.com

Vegetation Retention

The entire property is approximately 52.9 acres. To utilize full dispersion, the property must retain and protect 65 percent of the property as native vegetation. With full dispersion, future development will not be allowed within the vegetation retention areas. The maximum development area allowed is 35 percent of the site or approximately 18.5 acres. Considering that the site already has approximately 2.5 acres of existing disturbed gravel areas, the property can develop up to 16 acres of the site.

The pre-design site plan (Exhibit 1) develops approximately 16 acres of the site, and the suggested pre-design site plan (Exhibit 2) develops approximately 15 acres of the site. Exhibit 2 allows for up to 1 acre of additional developed area not shown. The development concepts meet the requirements for use of full dispersion for stormwater management.

The native vegetation areas can be used as passive recreation with pedestrian trails, as long as the cleared areas associated with this use do not exceed 8 percent of the preserved native vegetation area.

Full Dispersion

The primary benefit of sheet flow dispersion for stormwater management is related to earthwork. Since there is no required separation from groundwater, an area can be placed at or near existing grade and disperse runoff through native vegetation. “Full dispersion” does not eliminate the need for stormwater control. The full dispersion concept allows up to 10 percent of the impervious area (approximately 5 acres) to be dispersed without stormwater control. The remaining impervious area will still require stormwater control. For an infiltration facility, 2.5 feet of separation from the base of the facility to high groundwater is required.

The Readiness Center, Vehicle Storage Shed, and future FMS building require a minimum of 3 feet of separation between finish floor and groundwater. The buildings and paving adjacent to the buildings will have some site filling associated with them. Therefore, the greatest opportunity to realize cost savings associated with sheet flow dispersion is at the POV area, since grading of the area is not directly tied to the elevation of the buildings.

In the pre-design concept, the POV area is planned for a bioswale. AHBL has reviewed the pre-design concept against a dispersion alternative. See the table below for our summary:

	Fill Quantity	Bio-Retention Facility	Fill Cost ($20 - $24)	Bio-Retention Cost (Soil & Plantings)	Anticipated Cost Impact
Bio-Retention Swale	11,300 CY	12,000 SF	$226,000 $271,000	$35,000	$306,000
Dispersion into Native	900 CY	7,000 CY	$18,000 $22,000	N/A	$22,000

The two options, including our modeling assumptions, are covered in detail within Exhibits 3 and 4. We recommend modifying the POV parking lot similar to that shown in Exhibit 4 in order to disperse 100 percent of the parking lot into adjacent native vegetation. Not only will dispersion provide cost savings during construction, this option will also have a life cycle benefit by eliminating maintenance associated with a bioretention swale.

The POV parking lot will create approximately 95,000 square feet of impervious surface. City of Tumwater code allows for 10 percent of the site impervious to be fully dispersed; therefore, up to 135,000 square feet of additional site impervious could be dispersed. We anticipate that some portions of the Military Vehicle Parking area and access road can be graded to achieve full flow dispersion and minimize the size of required stormwater facilities.

Building Finish Floors

The minimum finish floor elevation for these buildings, per City of Tumwater requirements, is 3 feet above the assumed or known high groundwater elevation. Refer to Exhibits 5 and 6 for the Readiness Center and future FMS grading concepts.

A minimum finish floor elevation of 194 feet at the Storage Shed appears to be feasible. This would set the finish floor a minimum of 4.5 feet above groundwater. It appears that roof drains could splash block into an at-grade swale along the rear of the building for stormwater retention.

A minimum finish floor elevation of 194.5 feet at the Readiness Center appears to be feasible. This would set the finish floor a minimum 5 feet above groundwater. This assumes roof drain collection lines consist of shallow infiltration trenches and overflow to an adjacent bio-retention area to minimize depth.

The future FMS site will require a minimum finish floor of 192 feet, based on groundwater separation requirements. A summary of the earthwork associated with each building is listed in the table below:

	Building Fill	Associated Site Fill	Total Fill	Imported Fill ($20)	Imported Fill ($24)
Readiness Center	2,500 CY	9,500 CY	12,000 CY	$240,000	$290,000
Vehicle Storage Shed	1,000 CY	Included with R.C.	1,000 CY	$20,000	$24,000
FMS & Supporting Facilities	500 CY	2,000 CY	2,500 CY	$50,000	$60,000

The total fill quantities above do not account for stripping, nor do they consider pavement and foundation sections. A geotechnical engineer will need to review the site soils and provide recommendations for structural sections. Additionally they should review the suitability of native soils for reuse as structural fill. The table above assumes native soils are not suitable; therefore, import costs will range between $260,000 to $315,000 for the proposed Readiness Center, and another $60,000 for the future FMS site.

Our letter dated March 2015 estimated earthwork costs around $260,000 ($20/CF) for the Readiness Center development. The table above considers the entire development, including associated pavement areas, and is consistent with our original findings.

Conclusion

Maximizing the use of full stormwater dispersion on this property will provide a significant cost savings over conventional stormwater facilities. The greatest benefit is at the POV parking lot. Refer to Exhibit 2 for our recommended revision to the pre-design site plan. The parking configuration could be changed, but should take into consideration the benefit of sheet flowing runoff into native vegetation.

If you have any questions regarding this report, please do not hesitate to contact me at (253) 383-2422.

Sincerely,

J. Matthew Weber, PE
Principal

STK/lsk

Enclosures:

- Ex-1 – Pre-Design Site Development Map
- Ex-2 – Pre-Design Suggested Site Development Map
- Ex-3 – POV Parking – Bioretention Swale Concept
- Ex-4 – POV Parking – Dispersion Concept
- Ex-5 – Readiness Center and Vehicle Storage Concept
- Ex-6 – FMS Concept

This study is limited in scope. The statements and observations were derived from secondary information provided by local service providers. There may be additional information, records, or legal documents pertaining to the subject property that were not available to us during this feasibility assessment.

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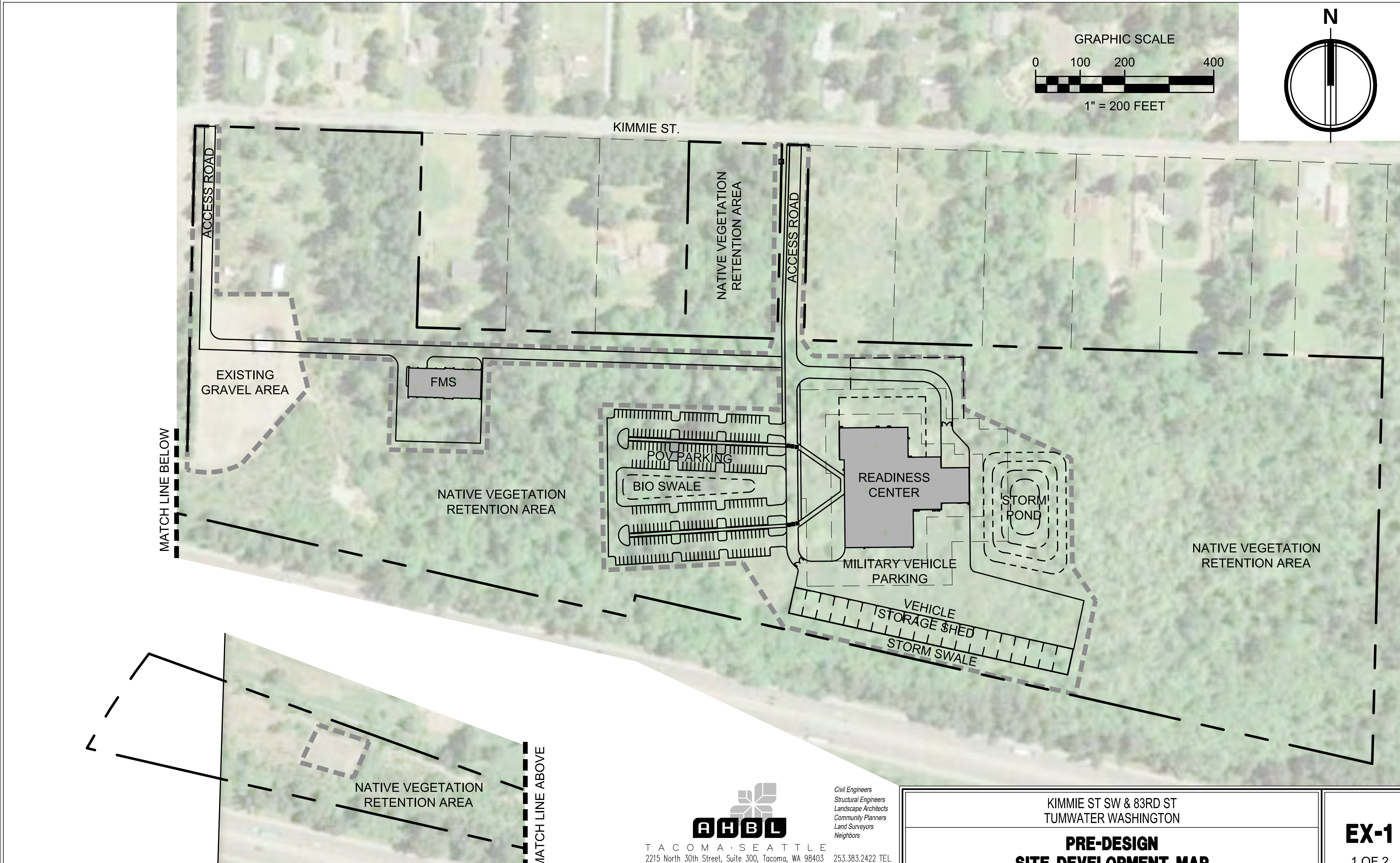

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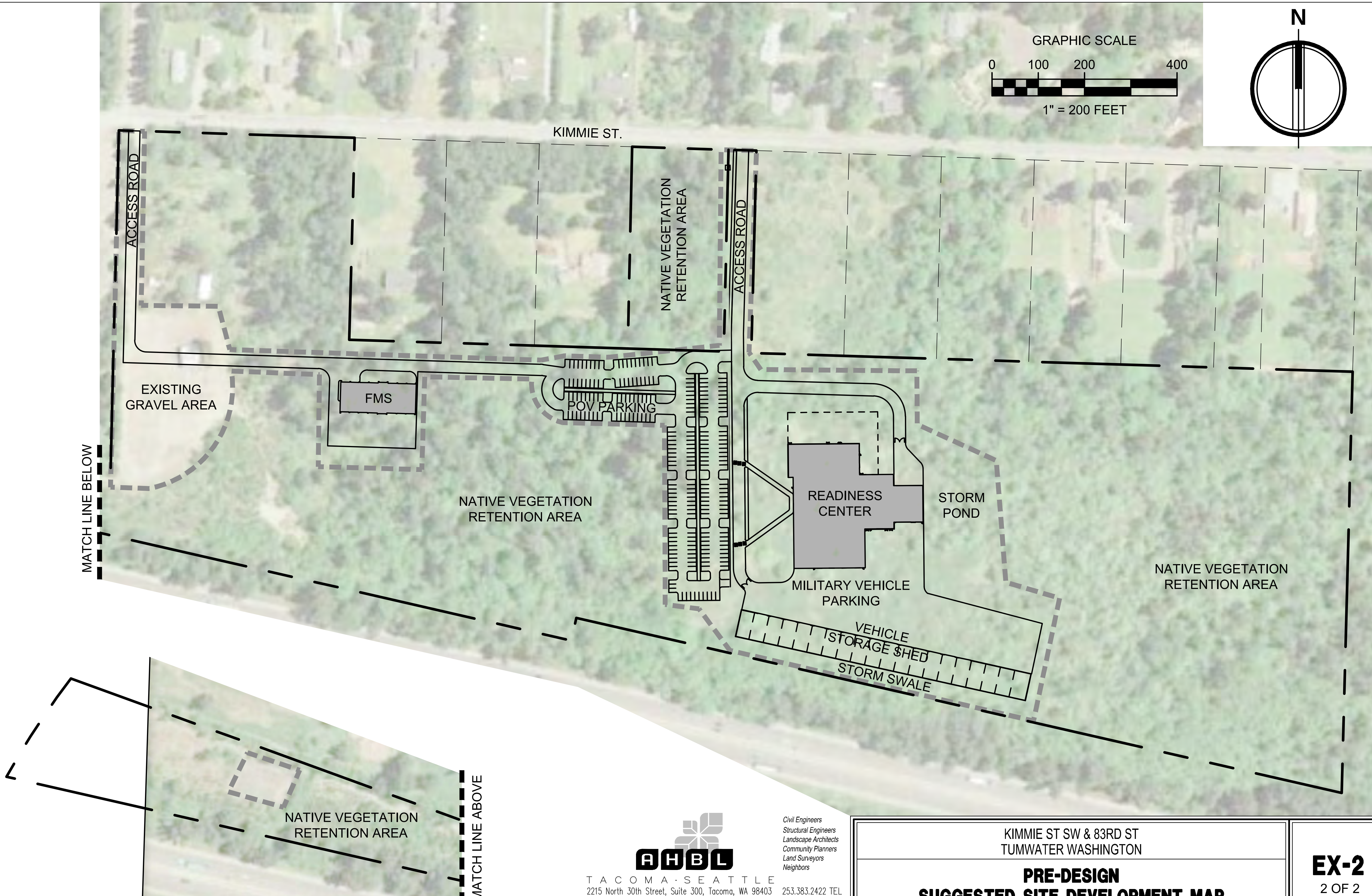
TUMWATER WASHINGTON

PRE-DESIGN
SITE DEVELOPMENT MAP

EX-1

1 OF 2



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TUMWATER WASHINGTON

PRE-DESIGN
SUGGESTED SITE DEVELOPMENT MAP

EX-2

Project	READINESS CENTER	Project No.	2140515.10
Subject	STORM IMPACT	Phone	
With/To		Fax #	
Address		# Faxed Pages	
Date	5/15/2015	By	KAUL

☑ Calculations
☐ Fax
☐ Memorandum
☐ Meeting Minutes
☐ Telephone Memo

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PARKING LOT WITH BIOSWALE

USING BIORETENTION SWALE REQUIRES 1.0' MINIMUM SEPARATION FROM GROUNDWATER. ADD 1.5' AMENDED SOIL

2.5' TO BOTTOM ⟹ GROUNDWATER ANTICIPATED @ 188.5' ±

THEREFORE SWALE BOTTOM 191.00' = (188.5' + 2.5')

USING 2.4"/HR INFILTRATION RATE
3.08 AC PARKING LOT BASIN
2.25 AC IMPERVIOUS AREA

REQUIRES APPROX. 12,000 SF SWALE @ 6" STORAGE DEPTH PLUS 6" FREEBOARD

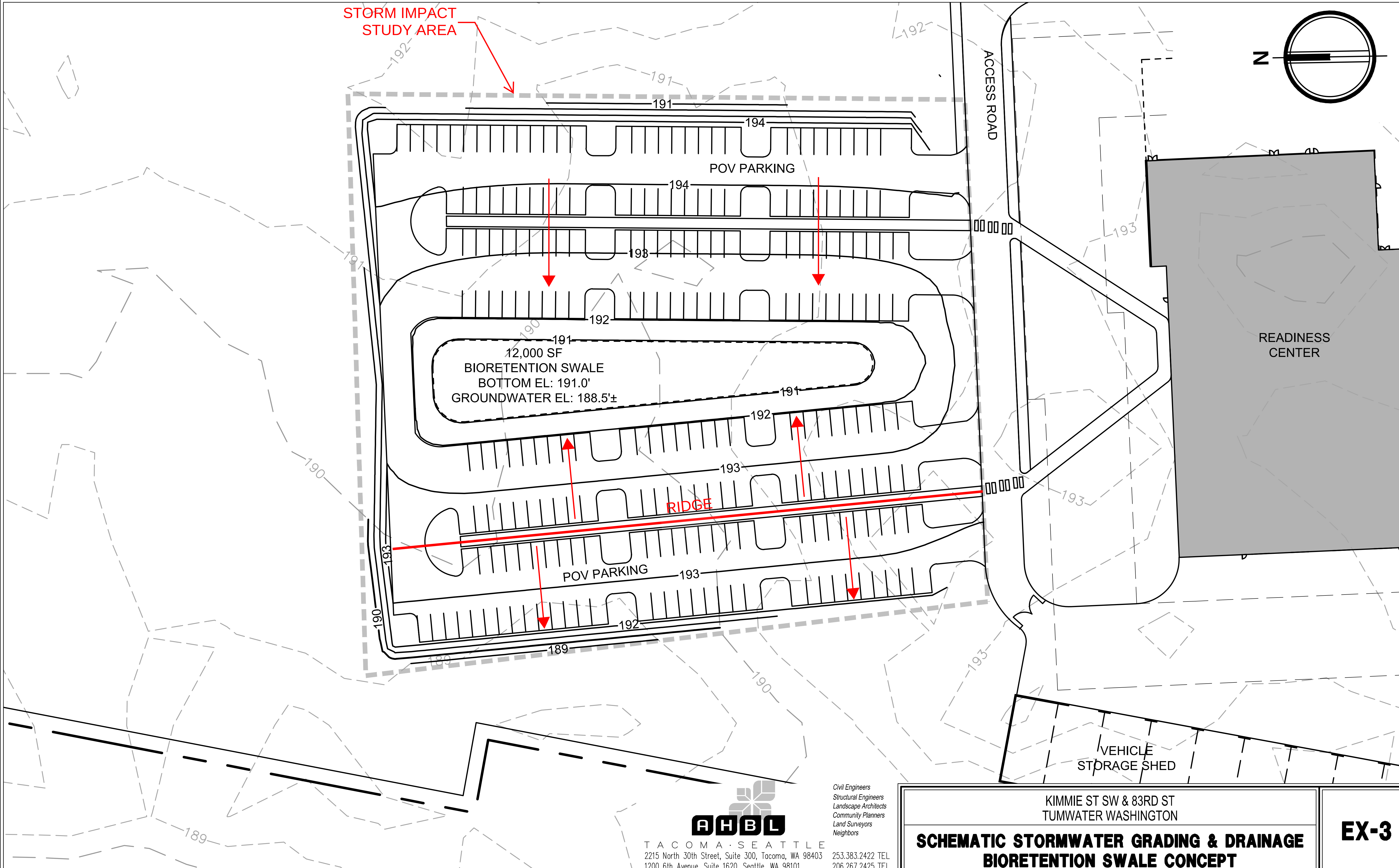
AHBL USED GOOGLE SRTM DATA TO APPROXIMATE EARTHWORK FOR THE SWALE PARKING LOT CONCEPT. REFER TO ATTACHED PRELIM GRADE EXHIBIT #1

NEGLECTING STRIPPING AND PAVEMENT SECTION THIS CONCEPT WILL GENERATE APPROXIMATELY 12,000 CY OF FILL
APPROXIMATELY 700 CY WILL BE THE BIOSWALE TOPSOIL

700 CY SWALE @ \$35/CY ≅ \$25,000
11,300 CY IMPORT @ \$20-\$24 ≅ \$226,000 - \$271,000

COST FOR THIS OPTION **\$250,000 - \$300,000**

COMPARE OPTION ABOVE TO FLOW DISPERSION CONCEPT
USE NATIVE VEGETATION TO TREAT AND DISPERSE PARKING LOT



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TUMWATER WASHINGTON

SCHEMATIC STORMWATER GRADING & DRAINAGE
BIORETENTION SWALE CONCEPT

EX-3

Project		Project No.	214 0515
Subject	STORM IMPACT	Phone	
With/To		Fax #	
Address		# Faxed Pages	
Date	5/15/2015	By	KAUL

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PARKING LOT WITH FULL DISPERSION
LOW SIDE OF PARKING LOT SET AT EXISTING GRADE
GRADE AWAY AT 1.5%
NO MANMADE STORM SYSTEM

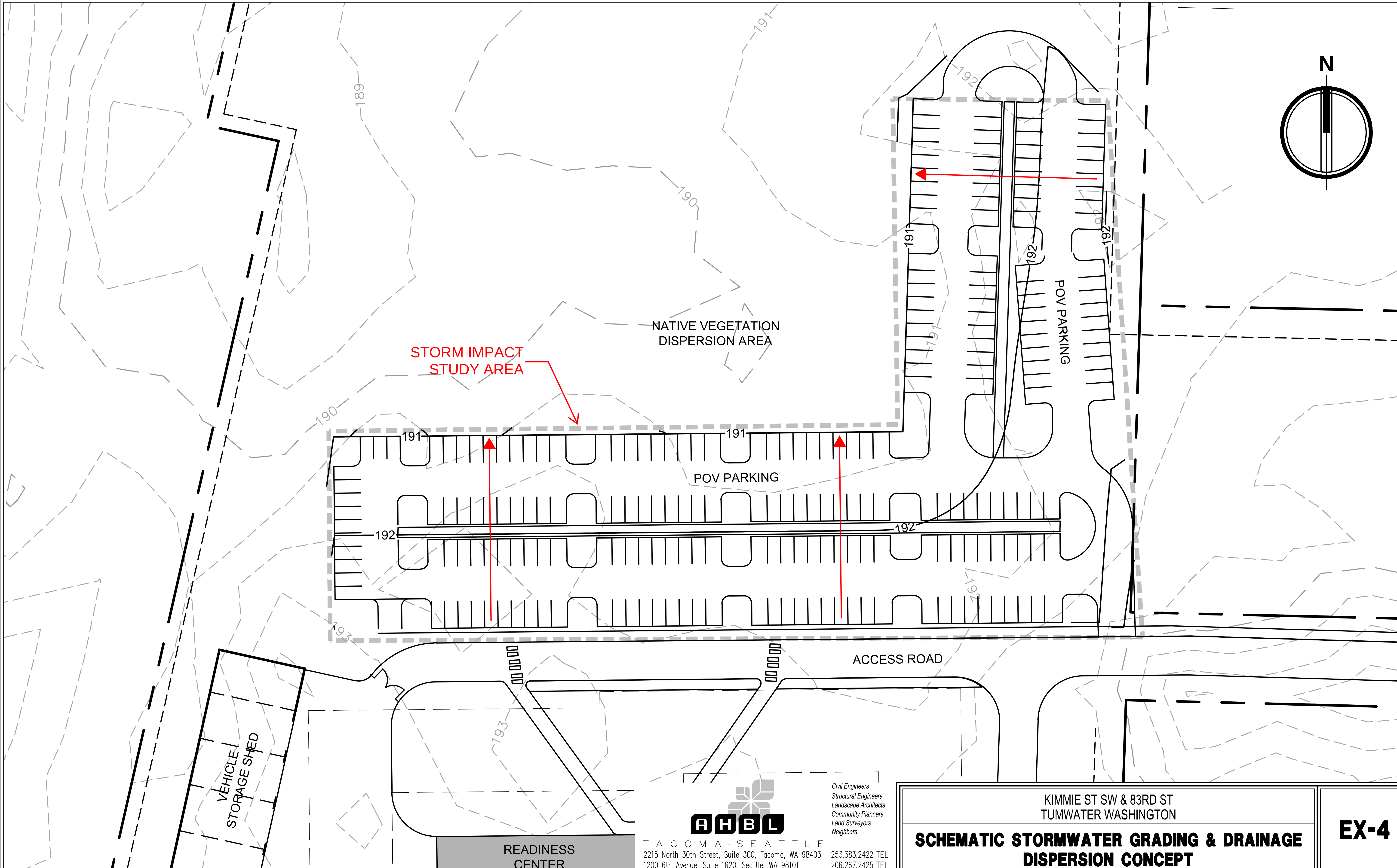
AHBL USED GOOGLE SRTM DATA TO APPROXIMATE
EARTHWORK FOR THE SWALE PARKING LOT CONCEPT.

NEGLECTING STRIPPING AND PAVEMENT SECTION
THIS CONCEPT WILL GENERATE APPROXIMATELY 900 CY OF FILL

900 CY IMPORT @ \$20 - \$24 ≅ \$18,000 - \$22,000

FULL DISPERSION WILL PROVIDE SIGNIFICANT
COST SAVINGS COMPARED TO BIOSWALE ALTERNATIVE

SEE EXHIBIT 2 FOR A DISPERSION CONCEPT



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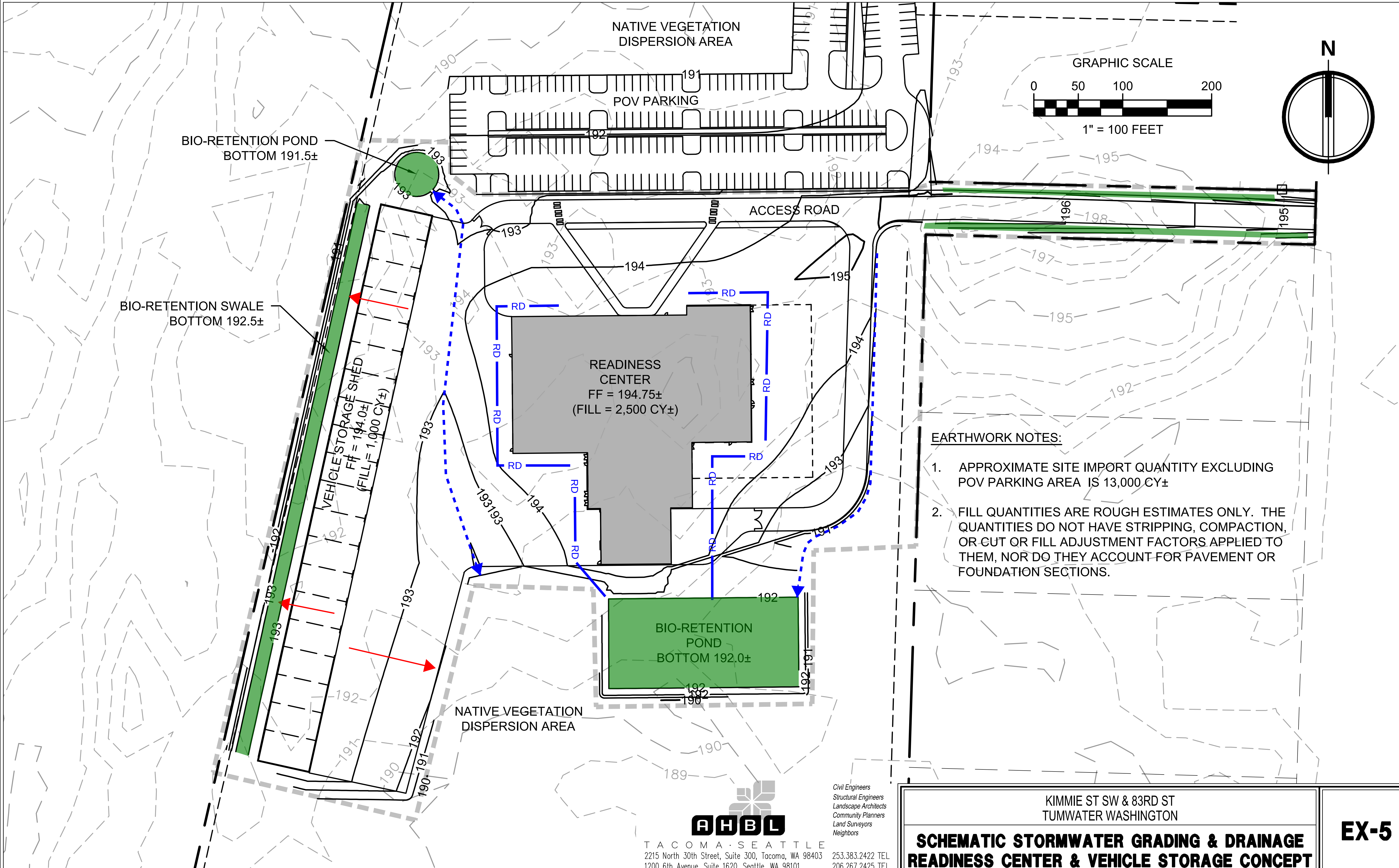
KIMMIE ST SW & 83RD ST
TUMWATER WASHINGTON

SCHEMATIC STORMWATER GRADING & DRAINAGE DISPERSION CONCEPT

EX-4



EARTHWORK NOTES:

1. APPROXIMATE SITE IMPORT QUANTITY EXCLUDING POV PARKING AREA IS 13,000 CY±
2. FILL QUANTITIES ARE ROUGH ESTIMATES ONLY. THE QUANTITIES DO NOT HAVE STRIPPING, COMPACTION, OR CUT OR FILL ADJUSTMENT FACTORS APPLIED TO THEM, NOR DO THEY ACCOUNT FOR PAVEMENT OR FOUNDATION SECTIONS.

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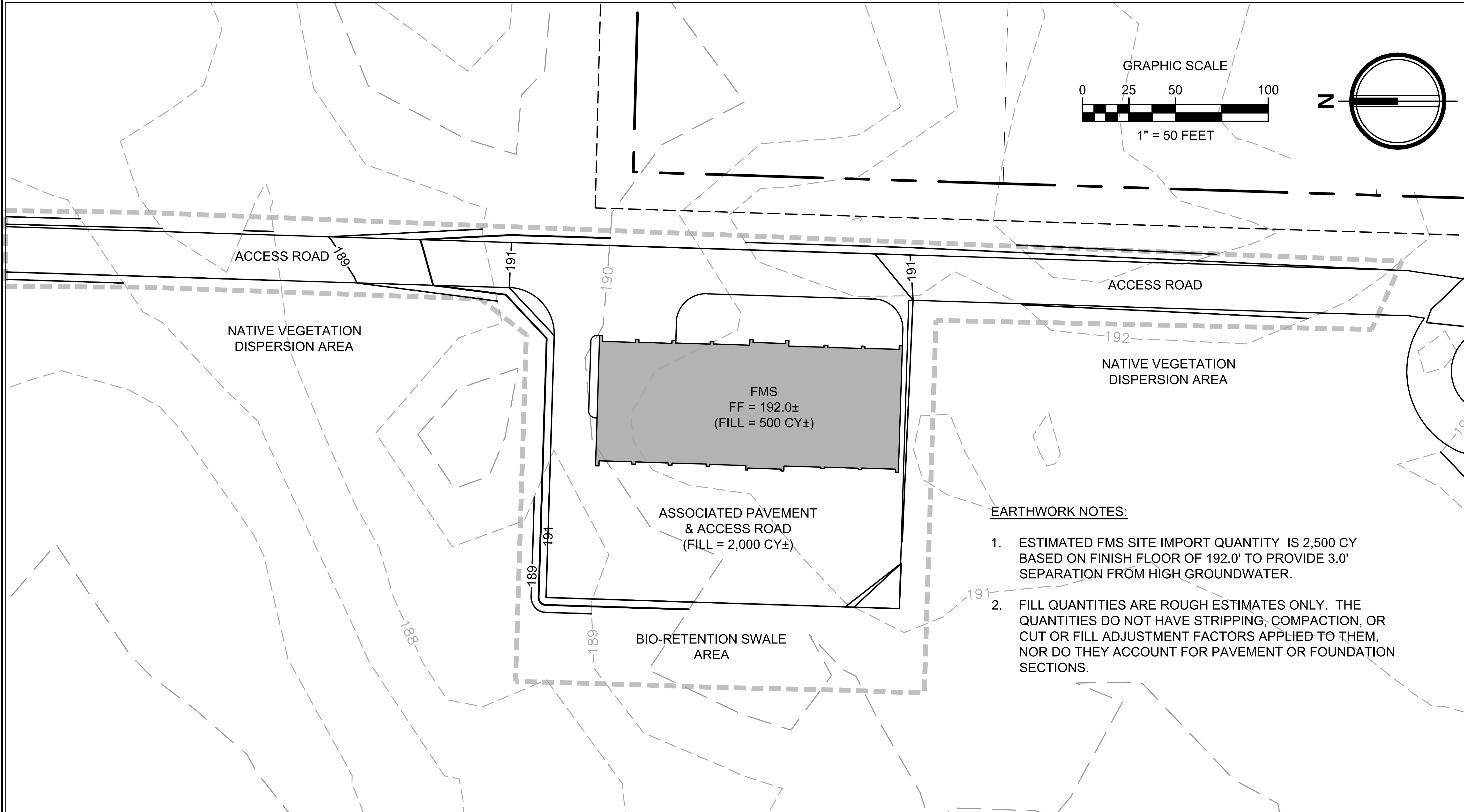
SCHEMATIC STORMWATER GRADING & DRAINAGE
READINESS CENTER & VEHICLE STORAGE CONCEPT

EX-5



EARTHWORK NOTES:

1. ESTIMATED FMS SITE IMPORT QUANTITY IS 2,500 CY BASED ON FINISH FLOOR OF 192.0' TO PROVIDE 3.0' SEPARATION FROM HIGH GROUNDWATER.
2. FILL QUANTITIES ARE ROUGH ESTIMATES ONLY. THE QUANTITIES DO NOT HAVE STRIPPING, COMPACTION, OR CUT OR FILL ADJUSTMENT FACTORS APPLIED TO THEM, NOR DO THEY ACCOUNT FOR PAVEMENT OR FOUNDATION SECTIONS.

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SCHEMATIC STORMWATER GRADING & DRAINAGE FMS CONCEPT

EX-6