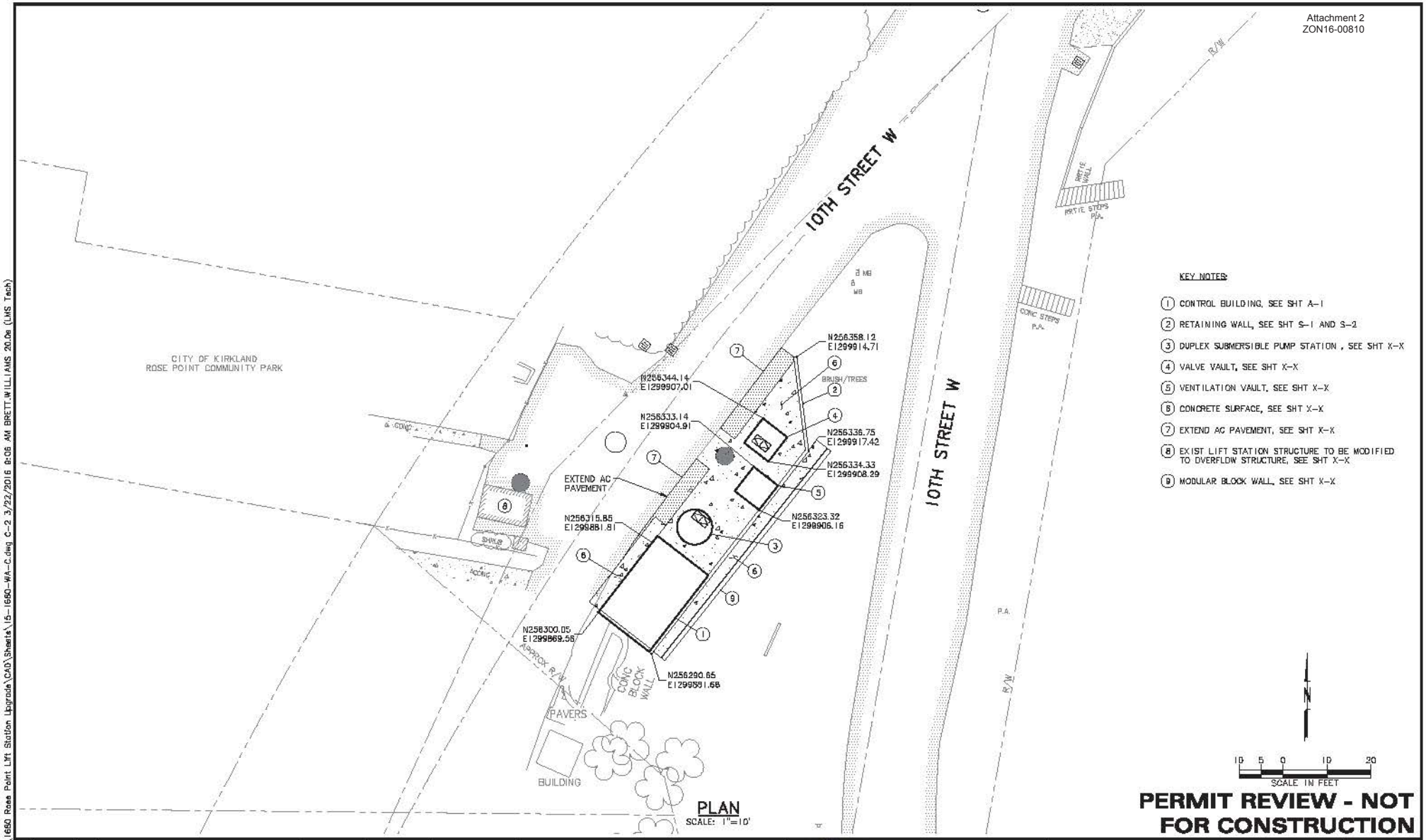
Attachment 2
ZON16-00810



PLAN
SCALE: 1"=10'

PERMIT REVIEW - NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE
0 ½ 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

BMC DESIGNED
HCM/BAW DRAWN
BMC CHECKED

MSA Murray, Smith & Associates, Inc. Engineers/Planners
2707 Colby Avenue, Suite 1110
Everett, Washington 98201-3520
PHONE 425.252.9003
FAX 425.252.9983

CITY OF KIRKLAND, WASHINGTON
ROSE POINT LIFT STATION IMPROVEMENTS

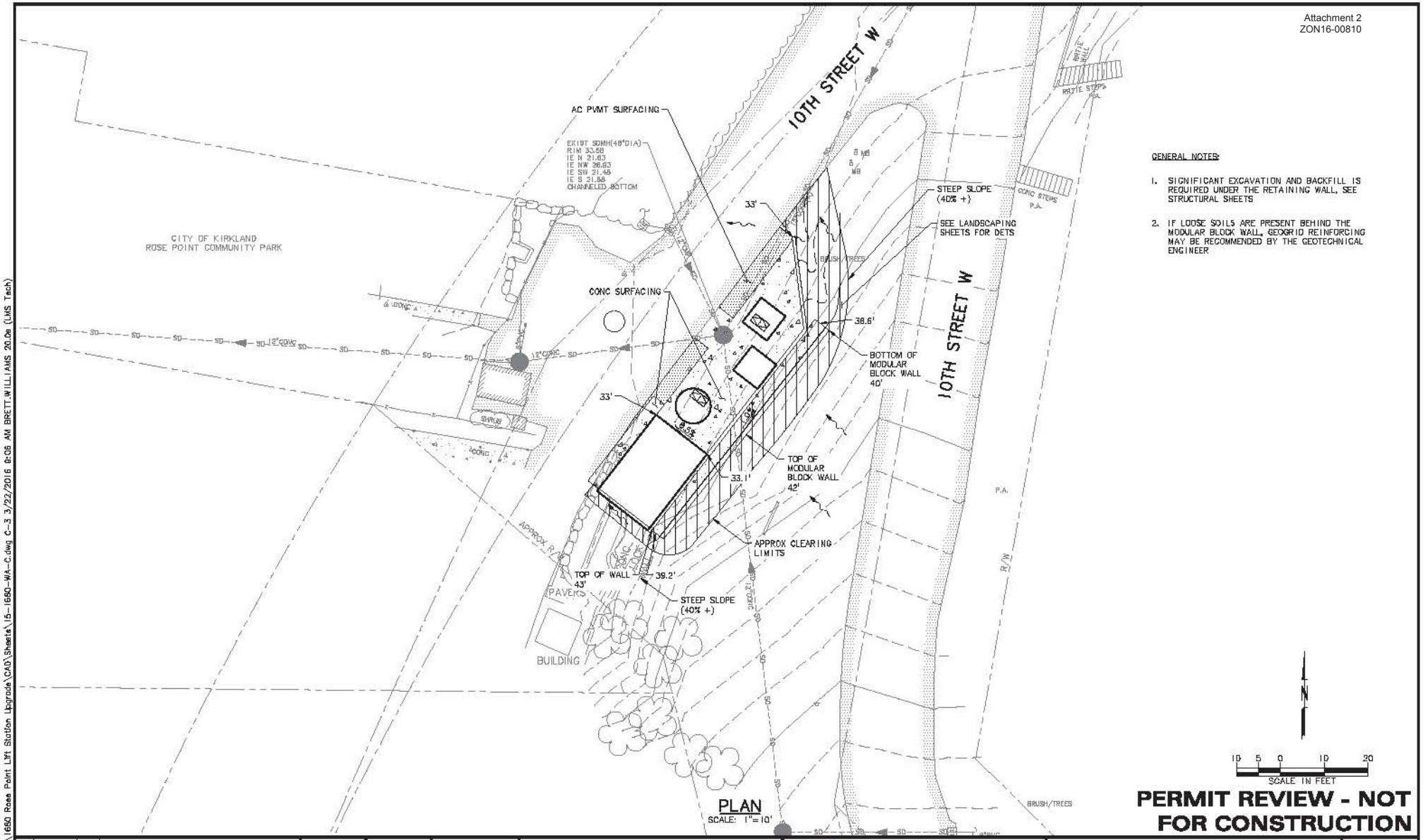
SITE PLAN

PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: MARCH 2016

SHEET C-2
X of X

K:\TAC_Projects\15\1680 Rose Point Lift Station Upgrade\CAD\Sheets\15-1680-WA-C.dwg C-2 3/22/2016 9:06 AM BRETT.WILLIAMS 20.0s (LMS Tech)

Attachment 2
ZON16-00810



PLAN
SCALE: 1"=10'

GENERAL NOTES:

1. SIGNIFICANT EXCAVATION AND BACKFILL IS REQUIRED UNDER THE RETAINING WALL, SEE STRUCTURAL SHEETS
2. IF LOOSE SOILS ARE PRESENT BEHIND THE MODULAR BLOCK WALL, GEOGRID REINFORCING MAY BE RECOMMENDED BY THE GEOTECHNICAL ENGINEER

PERMIT REVIEW - NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE
0 ½ 1
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BMC DESIGNED
HCM/BAW DRAWN
BMC CHECKED

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FAX 425.252.9983

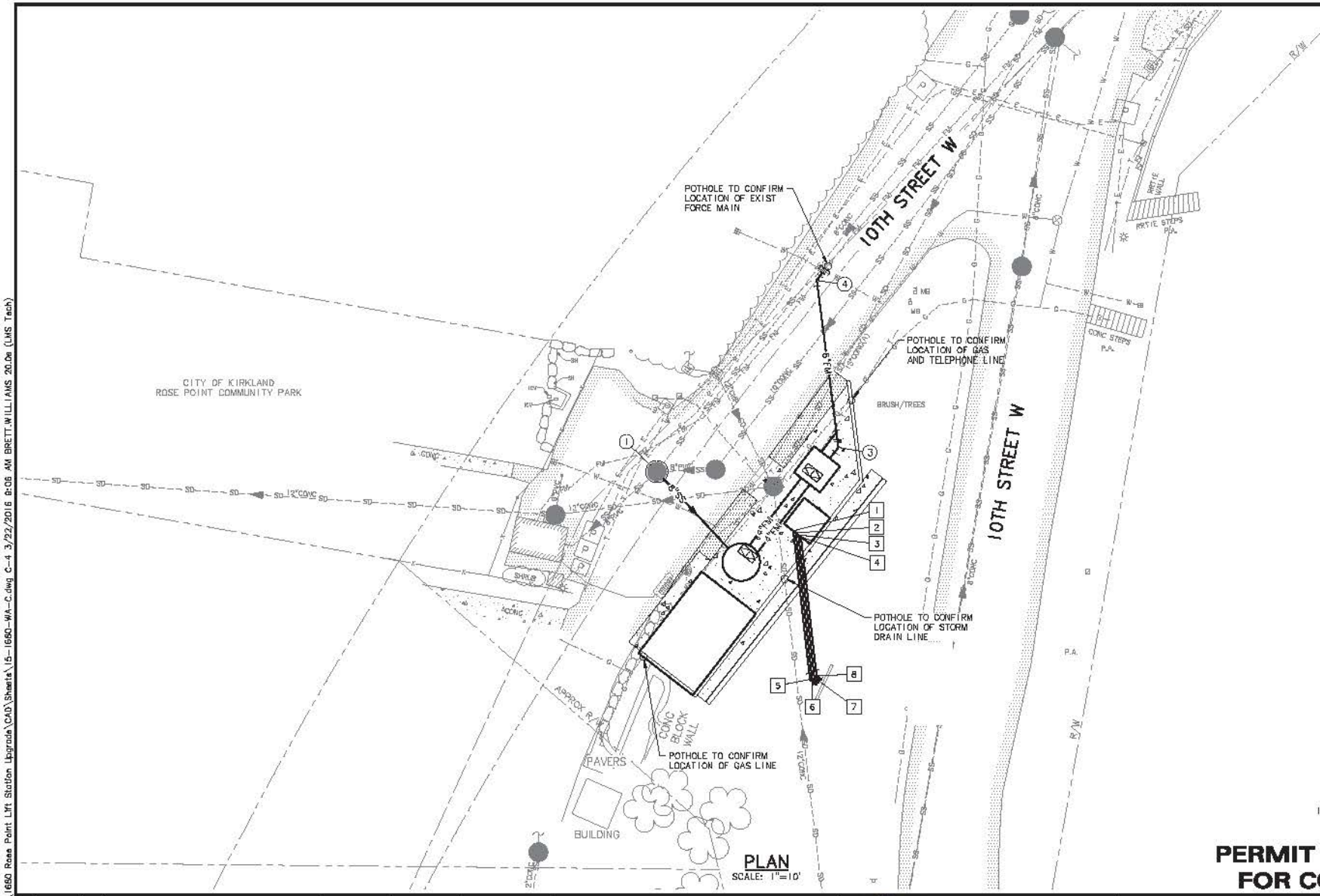
CITY OF KIRKLAND, WASHINGTON
ROSE POINT LIFT STATION IMPROVEMENTS

GRADING AND DRAINAGE PLAN

PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: MARCH 2016

SHEET C-3
X of X

K:\TAC_Projects\15\1680 Rose Point Lift Station Upgrade\CAD\Sheets\15-1680-WA-C.dwg C-3 3/22/2016 8:06 AM BRETT.WILLIAMS 20.0s (LMS Tech)



PLAN
SCALE: 1"=10'

SEWER PIPING SCHEDULE:

1. N___E___
EXIST 48" MH
RIM=31.70' (EXIST)
IE (E) 8" (IN)=19.30' (EXIST)
IE (NE) 8" (IN)=19.20' (EXIST)
IE (SW) 8" (IN)=19.20' (EXIST)
IE (SE) 8" (OUT)=19.10'+/-

2. N___E___
FURNISH & INSTALL:
8' DIAMETER WET WELL
RIM=33.00'
IE (NW) 8" (IN)=18.30'+/-
IE (NE) 4" (OUT)=

3. N___E___
FURNISH & INSTALL:
1 - 4" 45° BEND
1 - 4" WYE
1 - 4"X6" RDCR

4. N___E___
FURNISH & INSTALL:
1 - 6" 45° BEND
1 - 6" LONG BODY SLEEVE

SEWER PIPING SCHEDULE:

1. N___E___
FURNISH & INSTALL:
1 - 4" SEWER VENT PIPE

2. N___E___
FURNISH & INSTALL:
1 - 4" SEWER VENT PIPE

3. N___E___
FURNISH & INSTALL:
1 - 4" SEWER VENT PIPE

4. N___E___
FURNISH & INSTALL:
1 - 4" SEWER VENT PIPE

5. N___E___
FURNISH & INSTALL:
1 - 4" 45° BEND

6. N___E___
FURNISH & INSTALL:
1 - 4" 45° BEND

7. N___E___
FURNISH & INSTALL:
1 - 4" 45° BEND

8. N___E___
FURNISH & INSTALL:
1 - 4" 45° BEND

N

10 5 0 10 20
SCALE IN FEET

PERMIT REVIEW - NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE
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BMC DESIGNED
HCM/BAW DRAWN
BMC CHECKED

MSA Murray, Smith & Associates, Inc. Engineers/Planners
2707 Colby Avenue, Suite 1110
Everett, Washington 98201-3520
PHONE 425.252.9003
FAX 425.252.9983

CITY OF KIRKLAND, WASHINGTON
ROSE POINT LIFT STATION IMPROVEMENTS

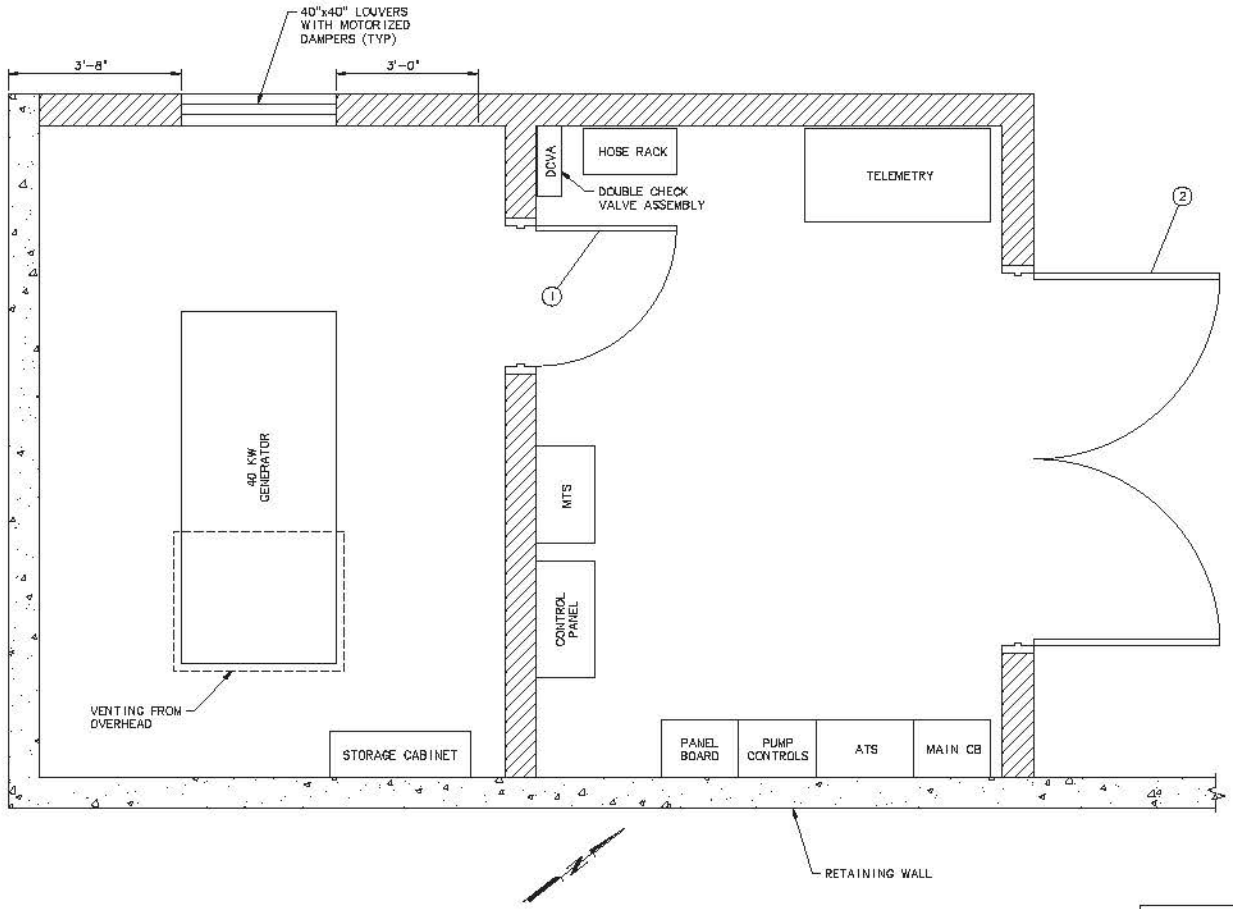
SITE PIPING PLAN

PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: MARCH 2016

SHEET C-4
X of X

K:\TAC_Projects\15\1680 Rose Point Lift Station Upgrade\CAD\Sheets\15-1680-WA-C.dwg C-4 3/22/2016 8:06 AM BRETT.WILLIAMS 20.0s (LMS Tech)

Attachment 2
ZON16-00810



PLAN
SCALE: ¾"=1'-0"

FINISH SCHEDULE

ITEM/SURFACE	MATERIAL	FINISH	COLOR	SPECIFICATION
EXTERIOR WALLS	SPLIT FACE CMU			
GENERATOR ROOM WALLS	SMOOTH FACE CMU			
CONTROL ROOM WALLS	SMOOTH FACE CMU			
CEILING/INTERIOR GABLE	EXPOSED RAFTERS/SHEATHING			
GENERATOR ROOM CEILING	ACOUSTICAL PANELS			
CONTROL ROOM CEILING	ACOUSTICAL PANELS			
INTERIOR FLOOR	SMOOTH SLAB ON GRADE			
ROOF	STANDING SEAM METAL			
LOUVERS	ALUMINUM			
DOORS	STEEL			
FASCIA/SOFFITS	FIBER CEMENT			

DOOR SCHEDULE

NO.	DESCRIPTION	ROUGH OPENING	SIZE	OPEN	HARDWARE	SPECIFICATION
1	STEEL DOOR	3'-4"x7'-4"	3'x7'	RHR		
2	STEEL DOUBLE DOOR	6'-8"x7'-4"	2@ 3'-7"	RH ACT LEAF		

PERMIT REVIEW - NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE
0 ½ 1
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JTS DESIGNED
HCM DRAWN
BMC CHECKED

MSA Murray, Smith & Associates, Inc. Engineers/Planners
2707 Colby Avenue, Suite 1110 Everett, Washington 98201-3590
PHONE 425.252.9003 FAX 425.252.9983

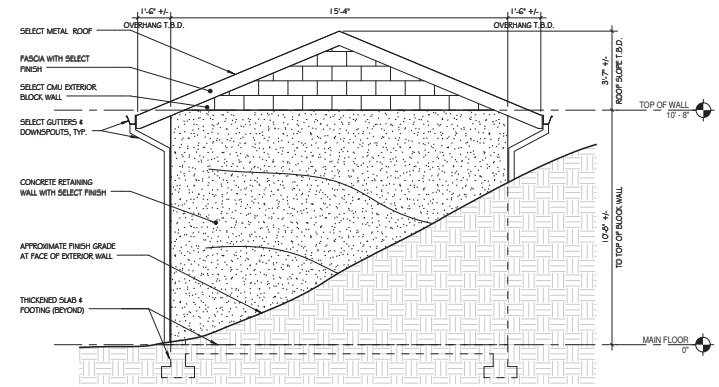
CITY OF KIRKLAND, WASHINGTON
ROSE POINT LIFT STATION IMPROVEMENTS

ARCHITECTURAL GENERAL NOTES, FLOOR PLAN, DOOR AND FINISH SCHEDULES

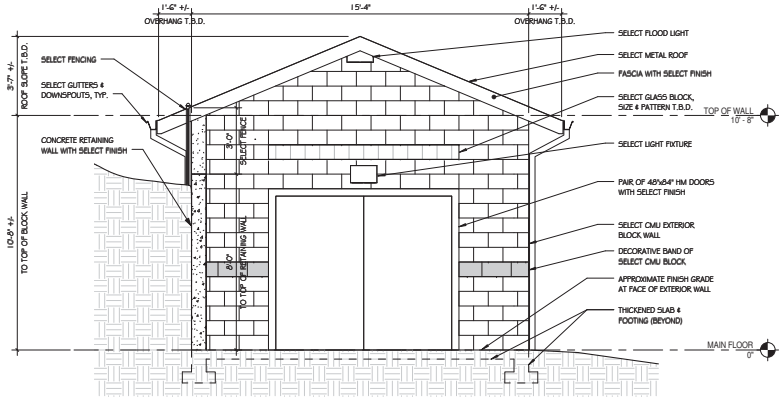
PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: MARCH 2016

SHEET A-1
X of X

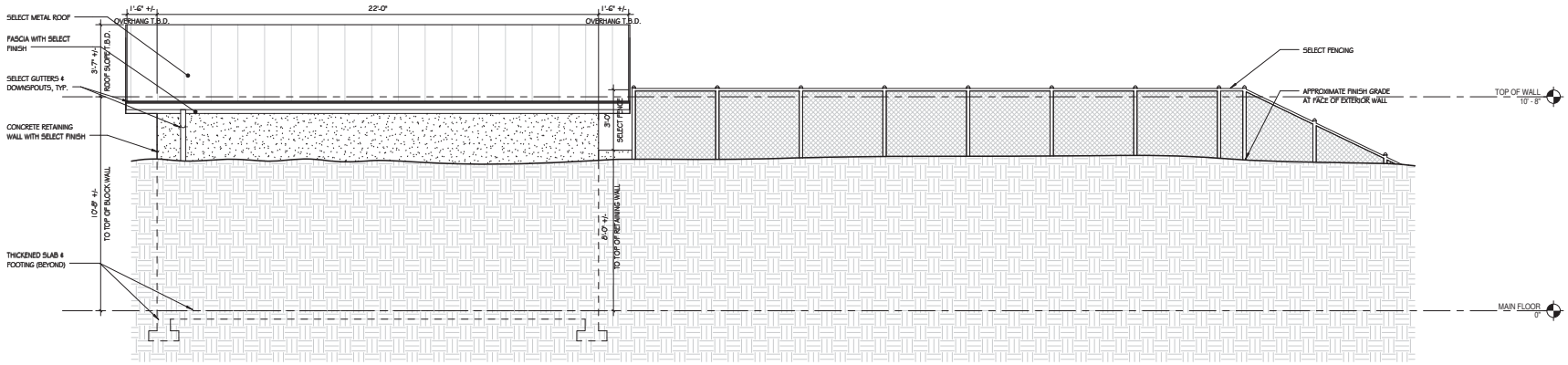
K:\TAC_Projects\15\1680 Rose Point Lift Station Upgrade\CAD\Sheets\15-1680-WA-A.dwg A-1 3/18/2016 10:25 AM RLF 20.0s (LMS Tech)



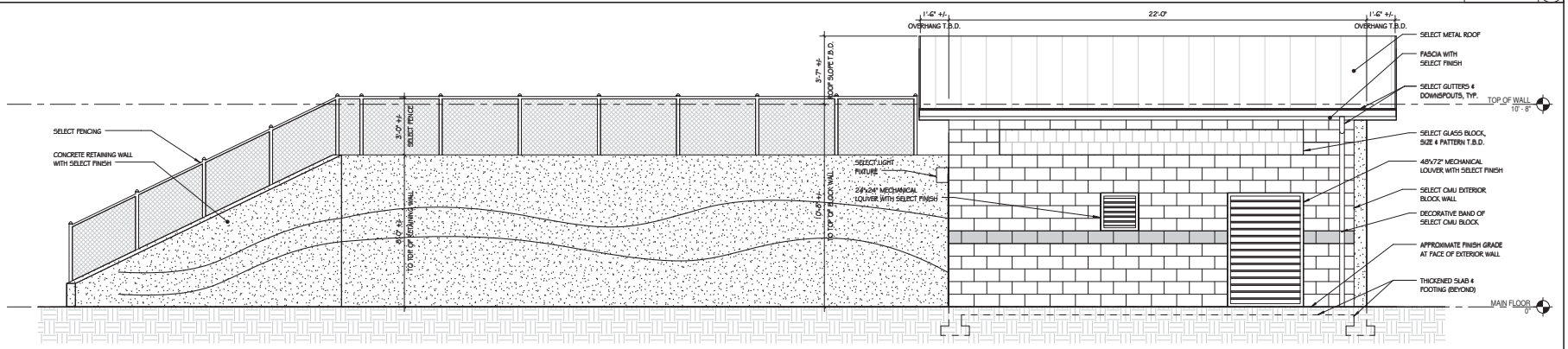
SOUTH ELEVATION | SCALE 3/8" = 1'-0" | 9



NORTH ELEVATION | SCALE 3/8" = 1'-0" | 1



EAST ELEVATION | SCALE 3/8" = 1'-0" | 2



WEST ELEVATION | SCALE 3/8" = 1'-0" | 4

The Driftmier Architects, P.S.
7983 Leary Way NE
Redmond, WA 98052
(425) 881-7506
mail@driftmier.com

CITY OF KIRKLAND ROSE POINT LIFT STATION

KIRKLAND, WASHINGTON

BUILDING ELEVATIONS

3540 REGISTERED ARCHITECT
Richard Prentice Driftmier
STATE OF WASHINGTON

THIS DOCUMENT REPRESENTS A PROPRIETARY DESIGN OWNED BY THE ARCHITECT AND SHALL NOT BE USED ON OTHER PROJECTS FOR ADDITIONS TO THIS PROJECT OR FOR COMPLETION OF THIS PROJECT BY OTHERS EXCEPT BY PRIOR AGREEMENT IN WRITING.

Revision Schedule

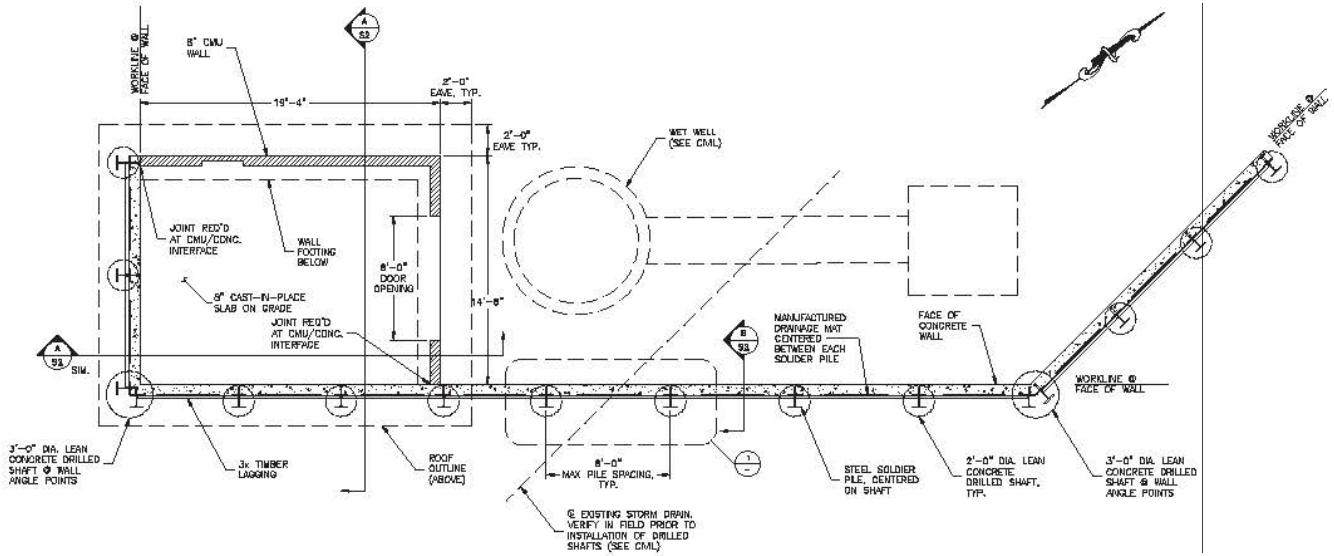
NO.	TYPE REVISION	DATE

ISSUE DATE:	3/25/16
DRAWN BY:	AMM
CHECKED BY:	RPD

PROJECT NUMBER
21601

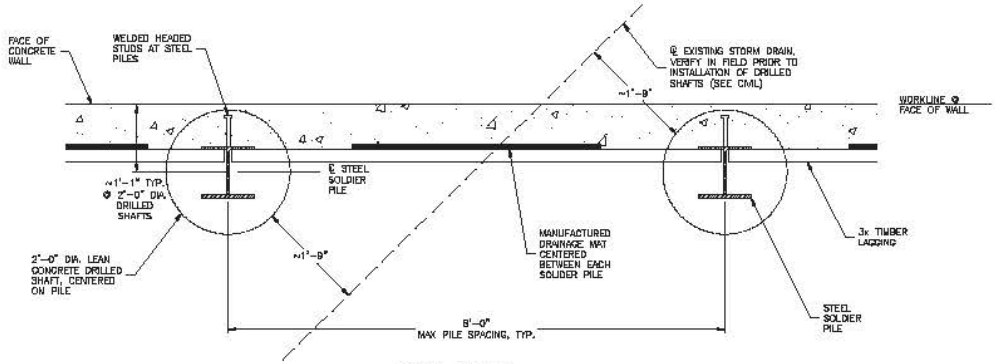
SHEET NUMBER
A3.0

3/29/2016 5:07:51 PM



PROJECT PLAN VIEW

1/4" = 1'-0"



WALL DETAIL 1

1" = 1'-0"

Peterson Structural Engineers, Inc.
708 Broadway, Suite 100A
Tacoma, WA 98402
(253) 830-2140 8/26/15

PERMIT REVIEW - NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE: IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

WJS DESIGNED
WJS DRAWN
TGM CHECKED

PRELIMINARY ONLY
NOT FOR CONSTRUCTION
AUGUST 2015
MURRAY, SMITH & ASSOCIATES, INC. Engineers/Planners

MSA Murray, Smith & Associates, Inc. Engineers/Planners
1145 Broadway Plaza, Suite 1010 Tacoma, Washington 98402-4323
PHONE 253.627.1520 FAX 253.627.1923

CITY OF KIRKLAND WASHINGTON

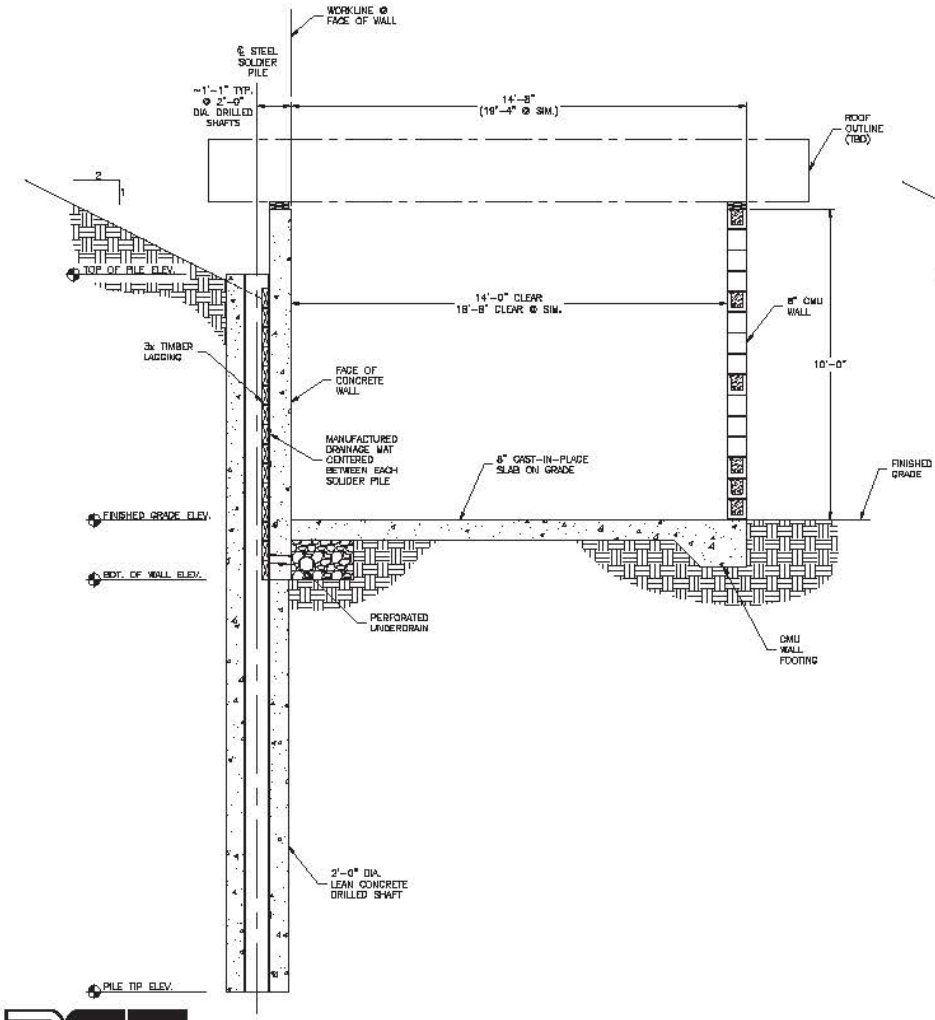
CITY OF KIRKLAND, WASHINGTON ROSE POINT LIFT STATION IMPROVEMENTS

BUILDING AND WALL PLAN

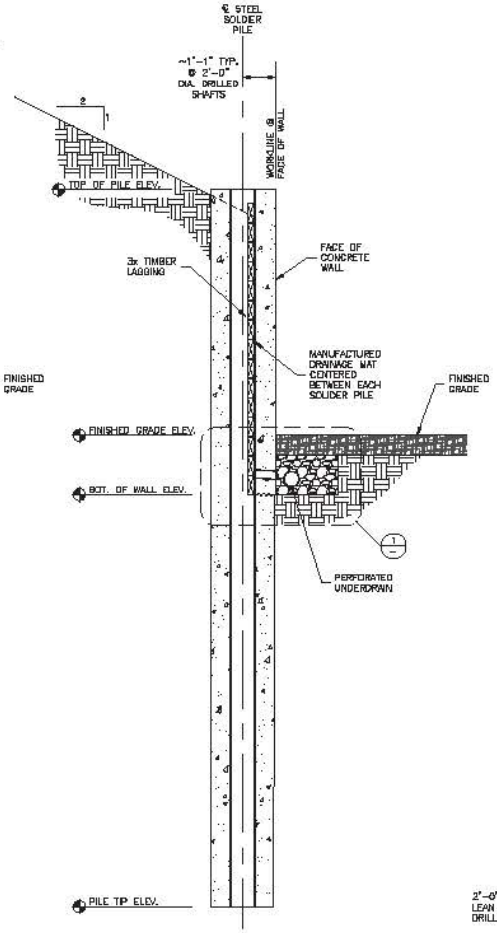
PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: AUGUST 2015

SHEET S-1 of X

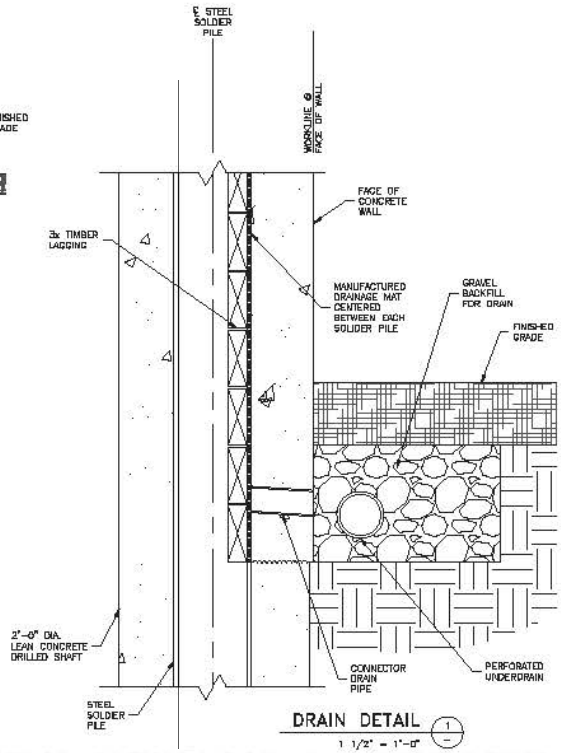
X:\2015\15-126 to 15-150\15-147\Acad\15-147-01 Kirkland Rose Point.dwg S1 8/26/2015 3:00 PM PSE-D09-PC 20.1a (LMS Tech)



BUILDING SECTION (A / S1)
1/2" = 1'-0"



WALL SECTION (B / S1)
1/2" = 1'-0"



DRAIN DETAIL (1 / –)
1 1/2" = 1'-0"

Peterson Structural Engineers, Inc.
708 Broadway, Suite 100A
Tacoma, WA 98402
(253) 830-2140 8/28/15

PERMIT REVIEW - NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE: 0 — 1 — IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED / DRAWN / CHECKED

PRELIMINARY ONLY — NOT FOR CONSTRUCTION — AUGUST 2015 — MURRAY, SMITH & ASSOCIATES, INC. Engineers/Planners

Murray, Smith & Associates, Inc.
Engineers/Planners
1145 Broadway Plaza, Suite 1010
Tacoma, Washington 98402-4331
PHONE 253.627.1520
FAX 253.627.1935

CITY OF KIRKLAND, WASHINGTON
ROSE POINT LIFT STATION IMPROVEMENTS

STRUCTURAL SECTIONS AND DETAILS

PROJECT NO.: 15-1580.203 | SCALE: AS SHOWN | DATE: AUGUST 2015

SHEET S-2 of X

X:\2015\15-128 to 15-150\15-147\Acad\15-147-01 Kirkland Rose Point.dwg S2 8/28/2015 3:00 PM PSE-D09-PC 20.1a (LMS Tech)

PLAN
SCALE: 1"=10'

PLANTING SCHEDULE

SCIENTIFIC NAME	COMMON NAME	SIZE	SPACING
SHRUBS			
MYRICA CALIFORNIA	SWEET GALE	1 GAL.	5'
OLEMARIA CERASIFORMIS	OSOBERRY	1 GAL.	5'
ROSA WOODII	WOOD'S ROSE	1 GAL.	4'
GROUNDCOVERS			
FESTUCA GLAUCA	BLUE FESCUE	1 GAL.	AS SHOWN
FRAGARIA CHILOENSIS	SAND STRAWBERRY	4" POT	18" ON-CENTER
RUBUS PENTALOBUS	CREEPING RASPBERRY	4" POT	18" ON-CENTER
GRASS			
SOD OR HYDROSEED TO MATCH EXISTING GRASS STRIP			

NOTES:

1. REFER TO CIVIL DRAWINGS FOR SITE PREPARATION AND TESC MEASURES.
2. REFER TO SHEET L-2 FOR PLANTING DETAILS.

N

10 5 0 10 20
SCALE IN FEET

PERMIT REVIEW – NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE
0 1
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GB DESIGNED
GB DRAWN
LA CHECKED

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MARCH 2016
MURRAY, SMITH & ASSOCIATES, INC.
Engineers/Planners

ESA
5309 Shilshole Avenue NW
Suite 200
Seattle, WA 98107
P: (206) 789-9658
F: (206) 789-9684

CITY OF KIRKLAND WASHINGTON

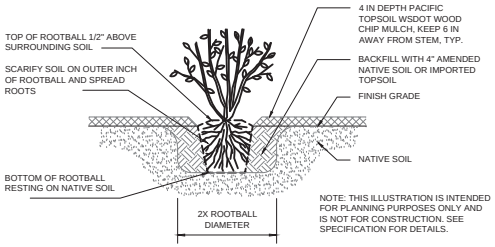
CITY OF KIRKLAND,
WASHINGTON
ROSE POINT
LIFT STATION
IMPROVEMENTS

LANDSCAPE PLAN

PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: MARCH 2016

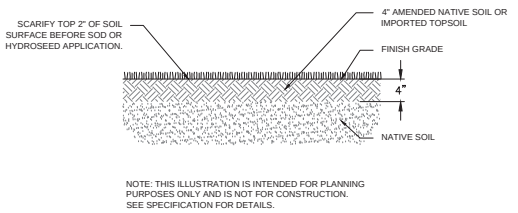
SHEET
L-1
X of X

G:\15cox\D150125.01 Rose Point Lift Station Upgrade\08 CAD\Dwgs\15-1680-WA-L-2.dwg L-1 3/25/2016 4:50 PM GBERGMAN 19.0e (LMS Tech)



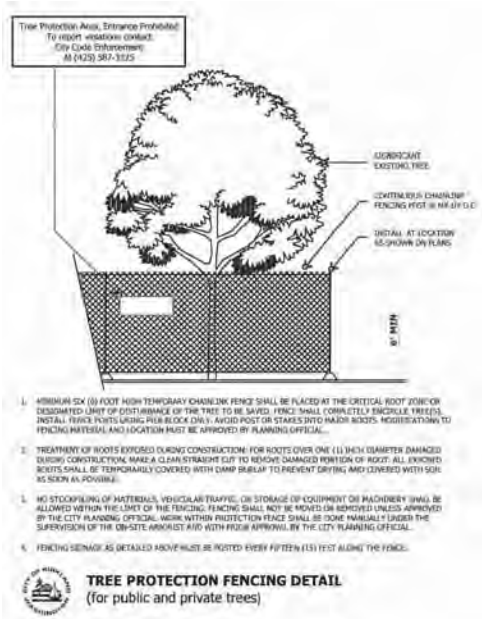
1 TYPICAL SHRUB (CONTAINER) PLANTING

NOT TO SCALE



2 TYPICAL LAWN AND SOIL PREPARATION DETAIL

NOT TO SCALE



3 CITY OF KIRKLAND TREE PROTECTION FENCING DETAIL

NOT TO SCALE

PERMIT REVIEW – NOT FOR CONSTRUCTION

NO.	DATE	BY	REVISION

NOTICE
0 1
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GB DRAWN
LA CHECKED

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MARCH 2016
MURRAY, SMITH & ASSOCIATES, INC.
Engineers/Planners

ESA
5309 Shilshole Avenue NW
Suite 200
Seattle, WA 98107
P: (206) 789-9658
F: (206) 789-9684

CITY OF KIRKLAND WASHINGTON

CITY OF KIRKLAND, WASHINGTON
ROSE POINT LIFT STATION IMPROVEMENTS

LANDSCAPE DETAILS

PROJECT NO.: 15-1680.203 | SCALE: AS SHOWN | DATE: MARCH 2016

SHEET
L-2
X of X

G:\15xxxx\D150125.01 Rose Point Lift Station Upgrade\08 CAD\Dwgs\15-1680-WA-L-2.dwg L-2 3/28/2016 9:22 AM GBERGMAN 19.0s (LMS Tech)

CITY OF KIRKLAND
Planning and Building Department
123 5th Avenue, Kirkland, WA 98033
425.587.3600 ~ www.kirklandwa.gov

DEVELOPMENT STANDARDS LIST
File: Rose Point Lift Station Replacement, ZON16-00810

PLANNING DEPARTMENT

SHORELINE MASTER PROGRAM STANDARDS

WAC173-27-190 Substantial Development Permit. Construction pursuant to a substantial development permit shall not begin and is not authorized until 21 days from the date of filing, or until all review proceedings initiated within 20 days from the date of filing have been terminated, except as provided in RCW90.58.140(5)(a) & (b).

ZONING CODE STANDARDS

85.25.1 Geotechnical Report Recommendations. The geotechnical recommendations contained in the report by GeoEngineers dated August 28, 2015 shall be implemented.

85.25.3 Geotechnical Professional On-Site. A qualified geotechnical professional shall be present on site during land surface modification and foundation installation activities.

95.51.2.a Required Landscaping. All required landscaping shall be maintained throughout the life of the development. The applicant shall submit an agreement to the city to be recorded with King County which will perpetually maintain required landscaping. Prior to issuance of a certificate of occupancy, the proponent shall provide a final as-built landscape plan and an agreement to maintain and replace all landscaping that is required by the City.

95.45 Parking Area Landscape Buffers. Applicant shall buffer all parking areas and driveways from the right-of-way and from adjacent property with a 5-foot wide strip as provided in this section.

95.50 Tree Installation Standards. All supplemental trees to be planted shall conform to the Kirkland Plant List. All installation standards shall conform to Kirkland Zoning Code Section 95.45.

95.52 Prohibited Vegetation. Plants listed as prohibited in the Kirkland Plant List shall not be planted in the City.

115.25 Work Hours. It is a violation of this Code to engage in any development activity or to operate any heavy equipment before 7:00 am. or after 8:00 pm Monday through Friday, or before 9:00 am or after 6:00 pm Saturday. No development activity or use of heavy equipment may occur on Sundays or on the following holidays: New Year's Day, Memorial Day, Independence Day, Labor Day, Thanksgiving, and Christmas Day. The applicant will be required to comply with these regulations and any violation of this section will result in enforcement action, unless written permission is obtained from the Planning official.

115.47 Service Bay Locations. All uses must locate service bays away from pedestrian areas. If not feasible must screen from view.

115.95 Noise Standards. The City of Kirkland adopts by reference the Maximum Environmental Noise Levels established pursuant to the Noise Control Act of 1974, RCW 70.107. See Chapter 173-60 WAC. Any noise, which injures, endangers the comfort, repose, health or safety of persons, or in any way renders persons insecure in life, or in the use of property is a

violation of this Code.

115.115.3.g **Rockeries and Retaining Walls.** Rockeries and retaining walls are limited to a maximum height of four feet in a required yard unless certain modification criteria in this section are met. The combined height of fences and retaining walls within five feet of each other in a required yard is limited to a maximum height of 6 feet, unless certain modification criteria in this section are met.

115.135 **Sight Distance at Intersection.** Areas around all intersections, including the entrance of driveways onto streets, must be kept clear of sight obstruction as described in this section.

150.22.2 **Public Notice Signs.** Within seven (7) calendar days after the end of the 21-day period following the City's final decision on the permit, the applicant shall remove all public notice signs.

Prior to issuance of a grading or building permit:

85.25.1 **Geotechnical Report Recommendations.** A written acknowledgment must be added to the face of the plans signed by the architect, engineer, and/or designer that he/she has reviewed the geotechnical recommendations and incorporated these recommendations into the plans.

95.30(4) **Tree Protection Techniques.** A description and location of tree protection measures during construction for trees to be retained must be shown on demolition and grading plans.

95.34 **Tree Protection.** Prior to development activity or initiating tree removal on the site, vegetated areas and individual trees to be preserved shall be protected from potentially damaging activities. Protection measures for trees to be retained shall include (1) placing no construction material or equipment within the protected area of any tree to be retained; (2) providing a visible temporary protective chain link fence at least 6 feet in height around the protected area of retained trees or groups of trees until the Planning Official authorizes their removal; (3) installing visible signs spaced no further apart than 15 feet along the protective fence stating "Tree Protection Area, Entrance Prohibited" with the City code enforcement phone number; (4) prohibiting excavation or compaction of earth or other damaging activities within the barriers unless approved by the Planning Official and supervised by a qualified professional; and (5) ensuring that approved landscaping in a protected zone shall be done with light machinery or by hand.

Prior to final inspection (occupancy):

85.25.3 **Geotechnical Professional On-Site.** The geotechnical engineer shall submit a final report certifying substantial compliance with the geotechnical recommendations and geotechnical related permit requirements.

95.51.2.a **Required Landscaping.** All required landscaping shall be maintained throughout the life of the development. Prior to issuance of a certificate of occupancy, the proponent shall provide a final as-built landscape plan and an agreement to maintain and replace all landscaping that is required by the City

BUILDING DEPARTMENT

You may contact Tanya Elder at 425-587-3614 for Building Department questions related to this permit.

1. The approved plans shall not be changed, modified, or altered without authorization from the building official. The approved plans are required to be on the job site.

2. This ZON Permit does not authorize any cutting or digging for new footings or foundations for retaining walls, shoring, block walls or buildings. A SEPARATE BUILDING PERMIT MUST BE ISSUED PRIOR TO ANY FOOTING OR FOUNDATION WORK.
3. No excavation or fill is authorized to encroach upon a neighboring property without explicit agreement by the adjoining property owner.
4. A Separate Building Permit must be obtained for the retaining wall(s), block walls, building(s) and shoring.
5. The geotechnical report providing the final (rather than preliminary) geotechnical design and construction considerations shall be provided at the time of building permit submittal.
6. Structural calculations, design criteria, and material data relevant to shoring, retaining walls, blocks walls, roof structure, etc., shall be provided at the time of building permit submittal. Compliance with structural provisions in accordance with the 2015 IBC is required and geotechnical engineering design recommendations.
7. No structural or fire and life safety review has occurred for the proposed construction, as that will occur under the separate building permit application for review and approval.

PUBLIC WORKS DEPARTMENT

This project is managed by the Public Works Capital Improvement Projects Team. The project will be subject to all 2016 Public Works Pre-approved Plans and Policies as well all applicable state and federal regulations.

Geotechnical Engineering Services

Rose Point Lift Station Upgrade
10th Street West and 18th Avenue West
Kirkland, Washington

for
City of Kirkland and
Murray, Smith & Associates, Inc.

August 28, 2015

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Services

Rose Point Lift Station Upgrade
10th Street West and 18th Avenue West
Kirkland, Washington

for
City of Kirkland and
Murray, Smith & Associates, Inc.

August 28, 2015

GeoEngineers

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Rose Point Lift Station Upgrade
10th Street West and 18th Avenue West
Kirkland, Washington

File No. 0231-087-00

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INTRODUCTION AND PROJECT UNDERSTANDING

This report presents the results of our geotechnical engineering services for the proposed upgrades to the Rose Point Lift Station located near the intersection of 10th Street West and 18th Avenue West in the Rose Point neighborhood of Kirkland, Washington. The location of the project site is shown on the Vicinity Map, Figure 1. Figure 2, Site Plan, shows the lift station site in relation to surrounding physical features. Figure 3, Site Plan, shows a portion of the route of a new force main that will replace an existing force main extending northeast of the lift station.

Written authorization to proceed with our services was provided as a Task Order under Agreement for Services on Continuing Basis for Murray, Smith & Associates, Inc. (MSA) Project No. 15-1680 dated June 9, 2015. The Continuing Services Agreement is dated January 24, 2015.

A draft version of this report dated July 29, 2015 was submitted to MSA and other members of the project team for review and comment. This final version incorporates the input from the project team.

We understand that relocation of the existing lift station is being considered due to the age and insufficient capacity at peak flows during major storm events. Station improvements will include a new 8-foot-diameter wet well, a control building with new system controls and an adjacent ventilation vault, a valve vault, a new 6-inch force main from the valve vault that will connect with the existing force main, and 1,800 linear feet of 8-inch ductile iron or C900 PVC force main that will extend northeast of the lift station and through Juanita Bay Park.

We understand that the control building will be located at the toe of the steep slope east of the existing lift station. A retaining wall integral with the uphill (east) side of the control building will be constructed and will extend about 60 feet north of the building.

PURPOSE AND SCOPE

The purpose of our services is to complete subsurface explorations and review site conditions as a basis for evaluating preliminary geotechnical design and construction considerations. Our specific scope of services includes the following tasks:

1. Review available geologic maps, topographic maps and previous geotechnical exploration data in the vicinity, as available.
2. Complete a site visit to conduct a geologic reconnaissance of the slope above the existing lift station and to mark boring locations. Contact the One Call Utility Locate Service, then complete a second site visit to confirm utilities are marked, prior to completing the explorations. Retain a private locating company to check boring locations for utilities.
3. Explore subsurface soil and groundwater conditions at the lift station site by drilling three borings (B-1 through B-3) with track-mounted, hollow stem auger equipment and standard penetration test (SPT) sampling. We drilled one boring adjacent to the existing lift station to a depth of 31½ feet, one boring near the toe of the steep slope east of the lift station to a depth of 31½ feet, and a third boring at the top of the slope to a depth of 51½ feet. A groundwater monitoring well was installed in boring B-2 at

the toe of the slope. Traffic control was required for partial closure of 10th Avenue West during drilling of boring B-3.

4. Complete one hand auger hole (HA-1) on the slope above the lift station and four hand auger holes (HA-2 through HA-5) along the route of the force main segment within Juanita Bay Park.
5. Evaluate pertinent physical and engineering characteristics of the soils encountered based on geotechnical laboratory tests conducted on samples obtained from the explorations. The laboratory tests included moisture content, percent fines (particles passing the No. 200 sieve), and gradation (sieve analysis) tests.
6. Complete preliminary geotechnical evaluations and engineering analyses, and provide conclusions and recommendations, as follows:
    a. Describe subsurface conditions including soil and groundwater conditions encountered in the explorations and evaluated on the basis of laboratory tests.
    b. Make a preliminary assessment of the stability of the slope above the existing lift station, including consideration of temporary cut slopes, temporary shoring and permanent walls.
    c. Provide preliminary geotechnical input for design and construction of the lift station improvements including excavation and structural backfill, use of on-site soils, below grade walls, and foundation support.
    d. Comment on construction dewatering considerations including depth to groundwater encountered during drilling, two subsequent groundwater level measurements following drilling, seasonal variations in the groundwater level, and estimated permeability coefficients based on laboratory test results.
    e. Evaluate pipe support considerations along the force main alignment based on the results of the hand auger explorations.
    f. Comment on anticipated construction difficulties identified from the results of our explorations and from our experience with similar projects in the vicinity.
7. Prepare draft and final versions of a report presenting our preliminary conclusions and design recommendations together with detailed exploration logs, site plans, laboratory test results, and other supporting information. This final report incorporates comments from the project team.
8. Provide follow up consultation during preliminary design and participate in one project team conference call.

SITE CONDITIONS

Geology

Geologic information for the project vicinity was obtained from the map entitled "Geologic Map of the Kirkland Quadrangle, Washington" (Minard, 1983), published by the United States Geological Survey (USGS). The native geologic units mapped in the site vicinity consist of recessional outwash, glacial till, advance outwash, and transitional beds (Fraser glaciation). Each of these units is described separately below. Fill overlies the native soils in places.

Recessional Outwash

Recessional outwash, derived from meltwater flowing off the receding Vashon glacier, occurs as stratified sand and gravel with variable amounts of silt and clay. The outwash can include cobbles and boulders. Recessional outwash in the map area occurs as channel fill deposits. One such channel is mapped in the Juanita Bay and Juanita Bay Park area north of the lift station.

Glacial Till

Glacial till typically consists of a dense to very dense heterogeneous mixture of sand, gravel, cobbles and occasional boulders in a silt and clay matrix that was deposited beneath a glacier. Lenses of sorted, stratified sand and gravel occur within the till in places.

A zone of weathered till typically overlies the dense glacial till to depths of several feet below the ground surface. This weathered zone is somewhat drained, whereas the unweathered till is a barrier to vertical flow. Water percolating into the weathered till will usually pond and migrate laterally near the contact between the weathered and unweathered till layers.

Advance Outwash

Advance outwash generally underlies the glacial till cap and consists of sand and gravel with some silt and clay deposited by meltwater flowing from an advancing glacier. Fine-grained silty sand is common in the lower part of this unit in the site vicinity. This unit tends to be more permeable for groundwater flow and more prone to erosion and mass wasting processes than other glacially deposited soils.

Transitional Deposits

Transitional deposits are mapped beneath the advance outwash, beneath the glacial till where the advance outwash is absent, or at the ground surface in some areas. These deposits are both glacial and non-glacial and consist mostly of laminated clay and silty clay in the lower part, grading upward into silt and fine-grained sand in the upper part.

Surface Conditions

Lift Station Area

The project site is located near the east shoreline of Lake Washington at 1805 10th Street West. Figure 2 shows existing site features and the lift station location at the south end of a short extension of 10th Street West. This street serves several residences that are accessed by a private driveway extending to the south. A private community beach serving the Rose Point neighborhood is located northwest of the lift station.

The lift station is located in close proximity to the toe of an existing slope that slopes up at an inclination of about 2½H:1V (horizontal to vertical) to the main road segment of 10th Street West (Figure 2). The vertical height of the slope varies from about 17 to 26 feet at this location and is vegetated with tall grass, trees and brush. The lower portion of the slope is mapped as a Landslide High Hazard Area by the City of Kirkland. We did not observe indications of deep seated or shallow slope instability, ground cracking or seepage zones during our site reconnaissance. In addition, we observed localized near vertical cuts made near the toe of the slope along the private driveway that extends south of the lift station. The vertical cuts were made to accommodate placement of a shed and appear relatively stable with minor surficial raveling. The existing

ground surface on the pavement surrounding the lift station is at about Elevation 31 to 32 feet. The elevation of the main segment of 10th Street West at the top of the slope varies from about Elevation 48 to 58 feet in the vicinity of the lift station.

Force Main Extension

The force main alignment extends northeast of the lift station along 10th Street West and enters the southern portion of Juanita Bay Park near the intersection of 10th Street West and 20th Avenue West. The route then extends to the east and crosses a gentle, northwest facing slope within the park (Figure 3). Ground surface elevations along this segment of the route range from about Elevation 32 feet to 56 feet. This segment of the route is vegetated with lawn and scattered trees. The route continues eastward and then southeastward to 98th Avenue NE (Market Street).

Subsurface Conditions

Field Explorations and Laboratory Testing

We evaluated subsurface soil and groundwater conditions by drilling three borings (B-1 through B-3) with a track-mounted, hollow stem auger drill rig. The borings were drilled to depths of 31½ to 51½ feet below the existing ground surface. Boring B-2 was completed as a monitoring well to allow for groundwater level measurements following drilling. We also completed a series of hand auger holes (HA-1 through HA-5) near the existing lift station and along a segment of the new force main route using hand tools. The approximate locations of the explorations are shown on the Site Plans, Figures 2 and 3. Appendix A includes logs of the borings (Figures A-2 through A-4), hand auger holes (Figures A-5 through A-9) and details of the subsurface exploration program.

Soil samples obtained from the explorations were transported to our Redmond geotechnical laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils. Representative samples were selected for laboratory testing consisting of moisture content tests, percent fines tests and sieve analyses. Appendix B includes a brief discussion of the laboratory tests and the test results.

Soil Conditions

Subsurface soils encountered in our explorations are generally consistent with the mapped geology. We interpret the soils encountered in the borings to consist primarily of recessional outwash and transitional deposits. Surficial fill is also present at the site and along the force main alignment. The fill is associated with previous construction of the lift station, roadway and the existing force main.

Lift Station Area

Borings B-1 and B-2 were completed in the vicinity of the existing lift station. A limited pavement section was encountered in the borings consisting of 1 to 2½ inches of asphalt concrete with no observed underlying base course. Loose to medium dense silty sand and sand with silt fill underlies the asphalt and extends to depths of about 7 to 9½ feet. Stiff sandy silt interpreted as lake bed (lacustrine) deposits was encountered below the fill in boring B-1, extending to a depth of about 13 feet.

Recessional outwash consisting of medium dense silty sand and sand with silt were encountered below the fill and lacustrine deposits. These soils extend to depths of about 28 and 16 feet in borings B-1 and B-2,

respectively. Transitional deposits consisting of very stiff to hard silt and medium dense sand were encountered below the recessional outwash, and extend to the bottom of both borings.

Slope Above Lift Station

Hand auger hole HA-1 and boring B-3 were completed on the slope above the lift station. An approximate 12-inch thickness of sod and topsoil was encountered in both explorations. Medium dense silty sand fill was encountered beneath the topsoil to a depth of about 4½ feet in boring B-3. Loose to medium dense silty sand and sand with silt (interpreted as recessional outwash) was encountered below the fill in boring B-3, and below the topsoil in HA-1. The recessional outwash extends to a depth of 3 feet below the slope surface in HA-1, and 10½ feet below the pavement surface in B-3.

Transitional deposits underlie the recessional outwash and include stiff to hard silt, sandy silt and dense silty sand and sand. These deposits extend to the bottom of the explorations.

Our interpretation of soil and groundwater conditions within and northwest of the slope is shown in Cross Section A-A' presented in Figure 4. The location of this cross section is indicated in Figure 2.

Force Main Segment in Juanita Bay Park

Hand auger holes HA-2 through HA-5 were completed along the segment of the new force main route within Juanita Bay Park (Figure 3).

These hand auger holes encountered 2 to 4 inches of sod overlying topsoil that typically extends to a depth of 1 foot. Fill consisting of loose to medium dense silty sand and sand with silt was encountered below the topsoil. This fill may have been placed during construction of the force main or during development of the golf course. Hand auger hole HA-2 met practical refusal in the fill at a depth of 3½ feet on gravel or a cobble.

Recessional outwash consisting of medium dense sand and sand with silt was encountered below the fill and extends to the bottom of hand auger holes HA-3 through HA-5.

Groundwater Conditions

Groundwater was encountered during drilling at variable depths ranging from 9 to 35 feet below the existing ground surface. Groundwater seepage was not observed in the shallow hand auger holes. Groundwater indications during drilling are often inaccurate because of the influence of the drilling techniques and the limited amount of time the drilled hole is open. Groundwater was measured in the monitoring well installed in boring B-2 at a depth of about 10 feet below the ground surface on June 23, 2015. We anticipate that the groundwater level will fluctuate as a function of season, precipitation, the level of Lake Washington, and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Summary of Geotechnical Considerations

From a geotechnical perspective, it is our opinion that the Rose Point Lift Station upgrade project can be satisfactorily completed with appropriate temporary shoring walls. Construction dewatering will be required to lower the groundwater level below the bottom of the excavation.

The geotechnical considerations presented in this report should be incorporated into the project planning and design. A summary of primary geotechnical considerations is provided below. The summary is presented for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The site is designated as seismic Site Class D per the 2012 International Building Code (IBC).
- The slope extending up from the extension of 10th Street West to the main segment of the street appears to be stable with respect to deep-seated sliding. Some localized shallow sliding could occur in the slope due to weathering or prolonged exposure during wet weather. We anticipate that the stability of the slope will not be significantly impacted by construction of the new facility, provided that an appropriate shoring system is used and that disturbance of the slope within work areas is kept to a minimum.
- If practical, we recommend that site preparation, earthwork, lift station construction and pipe installation activities be completed in the generally drier summer to early fall months in order to reduce earthwork and construction dewatering costs associated with these activities.
- The existing fill and recessional outwash deposits contain a high percentage of fines (particles passing the No. 200 sieve), are highly moisture sensitive, and will be difficult to compact when wet. These soils may be used as structural fill during extended dry weather conditions (typically occurring in June through September) provided the soils are properly moisture conditioned. The fine-grained transitional deposits are generally not suitable for structural fill, but may be used in landscaping areas. Imported gravel borrow should be used as structural fill during wet weather conditions and during the wet seasons (typically October through May).
- Effective erosion and sedimentation control must be implemented during construction so that potential impacts to adjacent areas are reduced. The erosion and sediment control measures used for this project should be in accordance with the requirements of the City of Kirkland.
- Based on the conditions encountered in our borings, we anticipate excavations will be relatively easy and can be completed with conventional equipment. If excavations extend into the dense and hard glacially consolidated soils, more effort will be required. Although not encountered during drilling of the small diameter borings, cobbles and boulders are frequently present in glacial deposits. The contractor should anticipate cobbles and boulders and be prepared to remove these materials.
- Temporary shoring and construction dewatering will be necessary to construct the new lift station and associated piping. We provide recommendations for conventional shoring and for cantilever soldier pile walls in a subsequent section. Lateral soil pressures and construction considerations are discussed in detail in the following sections.
- Groundwater will be encountered in the lower portion of the wet well excavation and possibly in deeper trenches for the associated piping and vaults. Construction dewatering will be required to adequately lower the groundwater below the bottom of the excavation. For shallow excavations, it may be possible to handle groundwater seepage by excavating collector trenches and pumping from sumps. Dewatering wells or well points should be considered for removing water from deeper excavations as discussed in the “Construction Dewatering” section of this report.
- The wet well will extend below the groundwater level and should be evaluated for buoyancy and uplift resistance.

- The base slab of the wet well is expected to bear on medium dense recessional outwash or very stiff transitional deposits. These soils contain a high percentage of fines and can become easily disturbed when wet. We recommend placing a minimum 6-inch-thick layer of crushed rock beneath new base slabs to provide a level foundation pad and protect the subgrade from disturbance. The subgrade should be clean and free of loose soil prior to placing the base rock.
- Foundations for the control building should bear on at least 2 feet of compacted structural fill that replaces the existing loose to medium dense fill.
- Based on the conditions encountered in the borings, we estimate less than 1 inch of ground settlement may occur at the site as the result of lowering the water table approximately 5 feet. This is a preliminary estimate based on the conditions encountered and our understanding of the project. We should review seasonal groundwater conditions and the final design configuration, when available to evaluate settlement estimates.
- We recommend temporary cut slopes in the upper fill and native soils be inclined at 1½H:1V or flatter. These slopes may need to be modified depending on the total excavation depth, seepage conditions, localized sloughing, and the dewatering methods utilized during construction.
- New hot mix asphalt (HMA) pavement sections should consist of at least 2 inches of HMA over 4 inches of base course in light-duty pavement areas and at least 3 inches of HMA over 6 inches of base course in heavy-duty pavement areas.

Earthquake Engineering

2012 IBC Seismic Design Information

We recommend the 2012 IBC parameters for Site Class, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1), and Seismic Coefficients F_A and F_V presented in the table below.

2012 IBC Parameter	Recommended Value
Site Class	D
Short Period Spectral Response Acceleration, S_S (percent g)	125
1-Second Period Spectral Response Acceleration, S_1 (percent g)	48
Seismic Coefficient, F_A	1.00
Seismic Coefficient, F_V	1.52

Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in saturated soils and subsequent loss of strength. Ground settlement, lateral spreading and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils can suffer foundation settlement or lateral movement that can be severely damaging.

Based on the subsurface soil and groundwater conditions encountered in the explorations completed at the site, and the preliminary planned excavation, there is a low to moderate risk of the site experiencing liquefaction below the lift station during a moderate to large earthquake. The medium dense outwash

deposits below the water table have a moderate risk of liquefaction and the very dense/hard deposits encountered below a depth of about 30 feet have a low risk. Based on the soil conditions and groundwater measured in the boring, we estimate less than 2 inches of ground settlement may occur due to liquefaction during a moderate to large earthquake.

Surface Fault Rupture

Based on USGS maps of active faults in the Puget Sound region, the site is located more than 5 miles from any known active fault (roughly midway between the South Whidbey Island Fault and the Seattle Fault Zone). Because of the thickness of Quaternary sediments below the site, which are commonly more than 1,000 feet thick, and lack of fault displacement evidence in the area, the potential for surface fault rupture is considered low.

Slope Stability Considerations

The lower portion of the slope is mapped as a Landslide High Hazard Area and the upper portion is mapped as a Landslide Medium Hazard Area by the City of Kirkland. We did not complete detailed quantitative slope stability analyses as part of our services, however, we observed no evidence of slope instability or shallow failures during our site reconnaissance.

In our opinion, the lift station and related piping can be satisfactorily constructed within the lower portion of the slope, provided that the recommendations presented in this report are implemented during design and construction of the project. This includes design of suitable temporary shoring and permanent retaining walls, earthwork and excavation considerations, and control of surface and subsurface water. The design was in the preliminary stages at the time of this report. The plans and specifications should be reviewed during final design to confirm geotechnical design recommendations and considerations are incorporated as intended.

Excavation

Excavations at the site will include trench excavations to construct new piping and a deep excavation during construction of the wet well. Shallow trench excavations may be feasible using open cuts, while a trench box combined with a partially sloping cut may be considered where the pipe is deeper. All excavations should be fully dewatered. Lateral soil pressures for trench shoring design and for the deeper wet well excavation are presented in the following section, “Temporary Shoring.”

Construction of the proposed improvements will require demolition of existing facilities, such as the existing lift station and appurtenant structures, and possibly also existing sewer pipes and other utilities. Concrete, asphalt and other materials generated from demolition should be removed from the site.

Excavation of the upper fill and loose to medium dense native soils will be relatively easy. The very dense/hard transitional deposits will be more difficult to excavate and may require removal of cobbles and boulders. We expect that conventional heavy excavators will be adequate to excavate site soils up to about 20 feet in depth

Since the contractor has control of the construction operations, the contractor should be made responsible for the dewatering methods used, shoring, stability of cut slopes, and the safety of the excavations. A specialty dewatering subcontractor with demonstrated experience and suitably qualified dewatering design staff should be retained. The contractor is present at the site continuously and is best able to

observe site conditions and monitor the performance of excavations. Slope inclinations will need to be modified by the contractor if localized sloughing occurs or if significant seepage occurs. All dewatering, shoring and temporary slopes should conform to applicable local, state and federal safety regulations. All temporary cut slopes and shoring must comply with the provisions of Part N of the Washington Administrative Code (WAC) 296-155.

Temporary Cut Slopes

We recommend that temporary slopes in the upper fill and native soils be inclined at 1½H:1V or flatter. Flatter cut slopes may be necessary depending on control of groundwater seepage and dewatering methods. In addition, we recommend the following procedures for open cuts at the site:

- No traffic, construction equipment, or supplies should be allowed at the top of cut slopes for a distance of at least 5 feet from the top of the cut.
- Exposed soil along temporary cut slopes should be protected from surface erosion using waterproof tarps or visqueen, when appropriate.
- Construction should be scheduled so that the length of time the temporary cut is left open is minimized.
- Erosion control measures should be constructed as appropriate to reduce runoff from the site.
- Surface water flow should be diverted away from all excavations.
- The general condition of the temporary cut slopes should be observed periodically by a geotechnical engineer to identify potential problems.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions and protect adjacent facilities or structures. Slopes experiencing excessive sloughing or raveling can be flattened, supported with shoring, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

Temporary Shoring

Excavations deeper than 4 feet should be shored or laid back at a stable slope if workers are required to enter. Because of the diversity of available shoring systems and construction techniques, the design of temporary shoring is most appropriately left up to the contractor proposing to complete the installation. However, we recommend that the shoring be designed by a Professional Engineer (PE) licensed in the State of Washington, and that the PE-stamped shoring plans and calculations be submitted to the City of Kirkland and the Engineer for review prior to construction. The following paragraphs present general recommendations for the type of shoring system and design parameters that we conclude are appropriate for the subsurface conditions at the project site.

We anticipate that the majority of excavations will be shored using conventional shoring methods (trench boxes, sheet piles, a braced system, or a slide rail system.) A soldier pile wall may also be considered for retaining walls or shoring located within the existing slope. . The upper portion of the soldier pile wall can be designed as a permanent retaining structure to retain the existing slope above the top of the new below-grade structure.

Lateral Pressures for Conventional Shoring

The lateral soil pressures acting on temporary shoring will depend on the nature and density of the soil behind the wall, the inclination of the ground surface behind the wall, and the groundwater level. For walls that are free to yield at the top at least one thousandth of the height of the wall (i.e., wall height times 0.001), soil pressures will be less than if movement is restrained. The design of temporary shoring should allow for lateral pressures exerted by the adjacent soil, and for surcharge loads resulting from structures, traffic, construction equipment, temporary stockpiles adjacent to the excavation, etc. Lateral load resistance can be mobilized through the use of braces, tiebacks, anchor blocks and passive pressures on members that extend below the bottom of the excavation. Temporary shoring used to support trench excavations typically uses internal bracing such as hydraulic shoring or trench boxes.

We recommend that yielding walls retaining loose fill and native soils be designed using an equivalent fluid density of 40 pounds per cubic foot (pcf), for horizontal ground surfaces. For non-yielding (i.e., braced) systems, we recommend that the shoring be designed for a uniform lateral pressure of 26H in pounds per square foot (psf), where H is the depth of the planned excavation in feet below a level ground surface. These values assume that the ground behind the shoring has been dewatered such that the ground water table is at least 2 feet below the base of the excavation. Temporary dewatering recommendations are discussed in a subsequent section of this report.

If the dewatering system is not designed to lower the groundwater level behind the shoring walls (e.g. sheet pile walls with dewatering system inside the shored excavation), hydrostatic pressures must be included in the shoring design. For this condition, temporary shoring should be designed using a lateral pressure equal to an equivalent fluid density of 85 pcf, for horizontal ground conditions adjacent to the excavation.

The above lateral soil pressures do not include traffic, structure or construction surcharges that should be added separately, if appropriate. Shoring should be designed for a traffic influence equal to a uniform lateral pressure of 100 psf acting over the depth of the trench. More conservative pressure values should be used if the designer deems them appropriate.

The soil pressure available to resist lateral loads against shoring is a function of the passive resistance that can develop on the face of below-grade elements of the shoring as those elements move horizontally into the soil. The allowable passive resistance on the face of embedded shoring elements may be computed using an equivalent fluid density of 160 pcf for the sand fill soils and 300 pcf for the very dense/hard native soils below the water table. This passive equivalent fluid density value includes a factor of safety of about 1.5.

Soldier Pile Walls

Soldier pile walls consist of steel beams that are concreted into drilled vertical holes located along the wall alignment, typically about 8 feet on center. After excavation to specified elevations, tiebacks are installed, if necessary. Once the tiebacks are installed, the pullout capacity of each tieback is tested, and the tieback is locked off to the soldier pile at or near the design tieback load. Tiebacks typically consist of steel strands that are installed into pre-drilled holes and then either tremie or pressure grouted. Timber lagging is typically installed behind the flanges of the steel beams to retain the soil located between the soldier piles. Geotechnical design recommendations for a cantilever soldier pile wall are presented in the following section.

Soldier Piles

Cantilever soldier pile walls with a horizontal back slope can be designed using a triangular soil pressure distribution of 35H, where H is the height of the wall. The lateral soil pressure should be increased to 60H for a back slope of 2H:1V above the wall. Depending on the depth and location of the wet well, tiebacks may be necessary for additional lateral restraint. We can provide earth pressure diagrams for soldier pile walls with tiebacks and recommendations for tieback ground anchors during final design if needed.

We recommend that the lateral soil pressures be taken to act over the pile center to center spacing above the base of the excavation, and over one pile diameter below this level. The earth pressures do not include loading from traffic or other surcharge loads such as cranes, construction equipment or construction staging areas. These surcharge pressures should be evaluated on a case-by-case basis. No seismic earth pressures are required if the shoring will be temporary. For permanent walls with a 2H:1V back slope, we recommend seismic loading be approximated using a uniform lateral pressure equal to 12H psf, where H is the height of the wall.

Passive resistance to resist "kick-out" of the soldier piles may be computed on the basis of an equivalent fluid density of 160 pcf below the groundwater level and acting over the embedded depth of the soldier pile below the excavation. Passive soil pressures should be assumed to act over 2½ times the concrete diameter of the soldier pile or the pile spacing, whichever is less. The upper 2 feet of soil should be ignored when considering the passive resistance. The passive equivalent fluid density value includes a factor of safety of about 1.5.

We recommend that the embedded portion of the soldier piles be at least 2 feet in diameter and extend a minimum distance of 10 feet below the base of the excavation to resist "kick-out." The axial capacity of the soldier piles must resist the downward component of the tieback anchor loads and other vertical loads, as appropriate. We recommend using an allowable end bearing value of 25 kips per square foot (ksf) for piles supported on the recessional outwash or transitional deposits.

The allowable end bearing value should be applied to the base area of the drilled hole into which the soldier pile is concreted. This value includes a factor of safety of about 2.5. The allowable end bearing value assumes that the shaft bottom is cleaned out immediately prior to concrete placement. If necessary, an allowable pile skin friction of 1 ksf may be used on the embedded portion of soldier piles within the recessional outwash or transitional deposits to resist vertical loads.

Lagging

We recommend that temporary timber lagging for the soldier pile walls be sized using the procedures outlined in the Federal Highway Administration's Geotechnical Engineering Circular No. 4. The site soils are best described as competent soils. The following table presents recommend lagging thicknesses (rough cut) as a function of soldier pile clear span and depth.

Depth (feet)	Recommended Lagging Thickness (rough cut) for clear spans of:					
	5 feet	6 feet	7 feet	8 feet	9 feet	10 feet
0 to 25	2 inches	3 inches	3 inches	3 inches	4 inches	4 inches
25 to 50	3 inches	3 inches	3 inches	4 inches	4 inches	5 inches

Lagging should be installed promptly after excavation, especially in areas where seepage is encountered or where cleaner sand lenses of soils are present and caving soils conditions are likely. The workmanship associated with lagging installation is important for maintaining the integrity of the excavation.

The space behind the lagging should be filled with soil as soon as practicable, or within the same day as excavation. Placement of this material will help reduce the risk of voids developing behind the wall and movement or settlement behind the wall. If voids develop behind the lagging, they should be backfilled immediately with controlled-density fill or lean concrete, or as recommended by the Engineer. Material used as backfill in voids located behind the lagging should not cause buildup of hydrostatic pressure behind the wall.

Drainage

A suitable drainage system should be installed to prevent the buildup of hydrostatic groundwater pressures behind the soldier pile and lagging wall. It may be necessary to cut weep holes through the lagging in wet areas. Seepage flows at the bottom of the excavation should be contained and controlled.

Construction Considerations

Temporary casing or drilling fluid will be required to install the soldier piles and possibly the tiebacks where:

- Loose fill or native granular deposits are present;
- The native soils do not have adequate cementation or cohesion to prevent caving or raveling; and/or
- Groundwater is present.

GeoEngineers should be allowed to observe and document the installation and testing of the soldier pile wall shoring to verify conformance with the design assumptions and recommendations.

Shoring Monitoring

We recommend a shoring monitoring and instrumentation program be established prior to construction. This should include a series of survey points to monitor horizontal and vertical movements of the shoring, surrounding streets, buildings, and any other adjacent facilities. Preexisting cracks should be noted and photographed where observed in exposed foundations and stem walls, concrete slabs and asphalt pavements. Locations of the monitoring points should be established when the final shoring design is complete. Additional recommendations can be provided for the shoring monitoring program and for the ground anchor load tests when the final location of the structure and shoring is selected.

Construction Dewatering Considerations

We recommend that the groundwater level be maintained at least 2 feet below the bottom of excavations during construction. The dewatering effort will depend upon the dewatering method, the size of the excavation and the time of year during which construction is accomplished. We anticipate the groundwater level varies by several feet throughout the seasons of the year. We recommend that construction be completed in the late summer or early fall months when the groundwater level is typically at its lowest elevation. A review of dewatering issues that should be considered for construction are provided in the following paragraphs. The contractor should have the responsibility to determine the dewatering measures needed for the project.

Dewatering Methods

The purpose of this report section is to present geotechnical and hydrogeological data that will influence temporary construction dewatering and to describe in general terms various types of dewatering techniques that may be feasible at the site.

We recommend that the contractor retain a specialty dewatering subcontractor to design and install an appropriate dewatering system that will adequately lower the groundwater level without adversely affecting offsite structures. The dewatering subcontractor should be experienced in dewatering in the subsurface conditions anticipated at the site. The contractor's dewatering plan should be reviewed by GeoEngineers to assess whether the proposed method is feasible and that the design is consistent with our recommendations. The contractor should also be required to install piezometers at several locations around the excavation(s) to verify that groundwater levels are lowered adequately by the dewatering system.

Based on the soil consistency and fines content, and depth to groundwater measured in boring B-2, we anticipate that groundwater inflow to excavations may occur at a moderate rate following an initial higher influx of groundwater. Hydraulic conductivity for medium dense silty sand fill and native soils are typically in the range of 10^{-3} to 10^{-4} centimeters per second (cm/sec), and the values for native transitional deposits are typically in the range of 10^{-4} to 10^{-6} cm/sec. In areas where the groundwater levels need to be lowered by less than about 3 feet, it may be possible to dewater by pumping from sumps augmented with gravel-lined trenches. The excavation for the sump and the drainage trenches should be backfilled with clean gravel or crushed rock to reduce the amount of sediment in the water pumped from the sump (i.e., to serve as a filter).

Dewatering wells or vacuum wellpoints may be required to adequately lower the groundwater around the temporary shoring for the deeper excavation of the wet well construction. Because of the limitations of suction lift, vacuum wellpoint systems typically are capable of lowering the water table to a maximum depth of about 20 feet below the header pipe that connects the wellpoints. A wellpoint system offers the advantage over deep wells in that additional wellpoints may be installed economically as the excavation proceeds, should zones of higher groundwater inflow be encountered. We anticipate that wellpoints may typically be spaced in the range of 4 to 8 feet on-centers around excavations to effectively dewater the moderate to relatively higher permeability soils above the hard/very dense transitional deposits.

Wellpoint tips (screened portion) should extend to depths at least 10 feet below the base of the wet well excavation to accomplish depressurization of the deeper transitional deposits and reduce uplift pressures. The wellpoint borings should be packed with clean, uniform sand to form a continuous filter pack that will facilitate vertical drainage of shallow water down to the wellpoint screens. Surface seals consisting of at least 3 lineal feet of hydrated bentonite chips or pellets should be placed in the annulus at the top of each wellpoint boring to allow the applied vacuum to extend into the filter pack and surrounding soils.

Alternatively, dewatering wells equipped with individual submersible pumps may be considered, although these will likely be more costly to construct and operate, considering the relatively low individual well yields likely to be achieved.

Water Disposal

Groundwater at the site is assumed to be free of contaminants but has not been sampled or tested. If environmental review suggests potential sources of contamination exist in the vicinity, water produced during the initial testing of the dewatering systems should be sampled and submitted to a certified laboratory for appropriate testing before the dewatering system is put into operation.

Groundwater pumped from the wellpoints, after they have been fully developed and put into operation, is expected to be clear, clean, low in turbidity, and free of suspended sediment. As such, groundwater can be discharged to a suitable storm drain system.

The development of wellpoints will produce turbid water laden with suspended sediment. This water must not be discharged to directly to the storm drain system. Development water must be pumped from each wellpoint and contained in a settlement tank before discharge to the storm drain system.

Other Considerations

An important issue for any significant dewatering project is the potential impact of lowering the groundwater table beneath adjacent facilities. When the groundwater table is lowered in loose sands or soft silt, the increase in effective weight or reduction in buoyancy tends to cause these materials to settle. This settlement, if excessive, can cause damage to buried utilities or to shallow foundations. The potential off-site impacts from dewatering could be serious, with numerous potential sources for claims (e.g. broken utilities, damage to roads and utilities). Therefore, it is critical that the dewatering program be designed to minimize off-site impacts. It is also critical for the owner and contractor's protection to initiate a monitoring program where groundwater impacts could occur.

Based on the depth of dewatering and subsurface soil conditions, we estimate that up to 1 inch of settlement can occur at the site as the result of lowering the groundwater by about 5 feet. We expect the settlement to be observed as a gentle downwarping of the adjacent area and may require some maintenance following construction. Additional recommendations for detailed settlement monitoring should be provided during final design of the lift station upgrade.

Wet Well Structure

Lateral Earth Pressures

For preliminary design, we recommend the permanent walls of the structure be designed using a uniform lateral soil pressure of 15H in psf, where H is the depth of the structure below a level ground surface. This design value assumes a full hydrostatic head condition (a long-term groundwater level as high as the existing ground surface). The hydrostatic pressure should be added to the 15H psf lateral soil pressure. In addition, we recommend seismic loading be approximated using a uniform lateral pressure equal to 8H psf, where H is the depth in feet below grade of the structure. This seismic lateral pressure is in addition to and should be superimposed upon the static soil and hydrostatic pressures. Depending on the location of the structure and permanent retaining wall design along the existing slope, additional lateral pressures may need to be considered. We recommend the design lateral pressures be reviewed when the final location and design configuration is established.

These lateral soil pressures do not include traffic or other surcharges that should be added separately, if appropriate. For traffic loading, if applicable, we recommend that below grade walls be designed for a

uniform surcharge pressure determined by increasing the apparent height of the backfill around the wall by 2 feet (250 psf). Other surcharge loads should be included as appropriate.

Buoyancy and Uplift

The wet well structure will extend below the groundwater level and should be evaluated for buoyancy and uplift resistance. Resistance to uplift can be developed by the dead weight of the structure, friction along the sides of the structure, and the weight of zones of soil which are located above the slab floor which protrude beyond the permanent walls. For preliminary design purposes, we recommend that hydrostatic uplift pressures be considered to the ground surface.

Frictional resistance can be computed using a coefficient of friction of 0.35 applied to the lateral soil pressures. This coefficient of friction value includes a factor of safety of about 1.5. We recommend that lateral soil pressures for uplift resistance be computed using an equivalent fluid density of 20 pcf considering groundwater is present. Backfill above the slab floor may be assumed to have a moist unit weight of 125 pcf and a submerged unit weight of 60 pcf.

Foundation Support

Subsurface soil conditions encountered at depth in our borings B-1 and B-2 consist of medium dense recessional outwash and transitional deposits consisting of hard sandy silt and very dense silty fine sand. These soils are competent load bearing soils, but become easily disturbed during excavation and when wet. To provide a level foundation pad and prevent disturbance, we recommend placing a minimum 6-inch-thick layer of crushed rock beneath the slab. We recommend the crushed rock consist of Permeable Ballast conforming to the 2014 Washington State Department of Transportation(WSDOT) Standard Specifications, Section 9-03.9(2).

All loose soil should be removed from the subgrade prior to placing the crushed rock. We recommend that slab subgrade areas be evaluated by a representative of our firm immediately prior to placing the crushed rock to confirm that subsurface conditions are as expected and that the bearing surface has been prepared adequately.

Permanent Dewatering and Waterproofing

Our preliminary design recommendations assume an undrained condition and a design groundwater level at the ground surface. It may be necessary to construct a sump within the base slab to collect any groundwater that seeps through construction joints and add the seepage to the waste stream.

Nominal waterproofing could also be considered in the final design, but will not be sufficient to provide a fully watertight structure. Without a complete and continuous multi-layer waterproofing system, moisture ingress and limited seepage through the walls and base slab will likely occur throughout the life of the facility.

Control Building Foundation Support

Footing Excavation

We recommend that all footings for the control building be supported on a pad of structural fill having a thickness of at least 2 feet that replaces the existing loose fill. We recommend that the compacted structural fill extend horizontally out from each edge of the footing a distance equal to the depth of the

excavation below footing subgrade level. Subgrades for footings supported on the fill should be evaluated and recompacted as necessary just prior to placing footing concrete.

We recommend that a representative of our firm observe the footing excavations before placement of structural fill.

Design Criteria

We recommend that all perimeter and interior footings be founded at least 18 and 12 inches, respectively, below the lowest adjacent finished grade. Isolate column footings should be at least 24 inches wide. Continuous strip footings should be at least 18 inches wide.

Individual column footings and continuous footings constructed as recommended may be designed using an allowable soil bearing pressure of 2,500 psf. This value applies to the total of all dead plus long-term live loads and may be increased by one-third when considering earthquake or wind loads.

We estimate that post construction settlements for wall and column footings will be less than ¾ inch. Differential settlements of up to ½ inch may be experienced along the width of the control building.

Lateral loads on the building can be resisted by passive resistance on the sides of footings and other below-grade structural elements and by friction on the base of footings. Base friction on floor slabs can also be used if the interface between the foundations and slabs can adequately transfer the loads.

Passive resistance may be estimated using an equivalent fluid density of 160 pcf, assuming that the footings and below-grade elements are backfilled with structural fill compacted to at least 95 percent of the maximum dry density (MDD). Frictional resistance may be estimated using 0.4 for the coefficient of base friction. The above values incorporate a factor of safety of about 1.5.

Floor Slab Support

We recommend that the floor slab for the control building be supported on a 4-inch thick gravel base layer to provide uniform support and to act as a capillary break. The exposed subgrade of floor slab area should be compacted to a firm, non-yielding condition prior to placement of the gravel base. The gravel base should consist of clean crushed gravel with a maximum particle size of 1½ inches and negligible sand or silt, such as WSDOT 9-03.1(4)C Grading No. 57.

For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) may be used.

Conventional Cast-in-Place Retaining Walls

Conventional cast-in-place retaining walls should be designed for lateral soil pressures based on an equivalent fluid density of 60 pcf, assuming a permanent backslope of 2H:1V above the wall and the ability of the top of the wall to move laterally a distance at least one thousandth the height of the wall. Walls that are restrained from movement during backfilling should be designed for a lateral earth pressure corresponding to an equivalent fluid density of 85 pcf. In addition, we recommend seismic loading be approximated using a uniform lateral pressure equal to 12H psf, where H is the height of the wall.

These values apply to wall backfill placed and compacted as recommended below. Care should be taken not to over-compact the backfill against the wall.

The recommended fluid density also assumes a free-draining condition behind the wall. This may be achieved by placing a 12- to 18-inch-wide zone of sand and gravel containing less than 3 percent fines behind the wall. A perforated drainpipe sloped to drain to a suitable discharge point should be installed at the bottom of the sand and gravel zone along the base of the wall.

Values for allowable soil bearing pressure, frictional resistance and passive pressure presented above for control building foundation support are also applicable to design of cast-in-place retaining walls.

Excavation Backfill

Structural Fill Content

Re-use of most of the on-site soils will likely not be practical due to the high fines content and groundwater conditions at the site. If control building foundation excavation and force main trench excavation takes place during dry weather conditions, it may be possible to re-use the existing fill and recessional outwash soils as trench backfill.

We recommend structural backfill meet the criteria for Common Borrow as described in the WSDOT Standard Specifications Section 9-03.14(3). Common Borrow will be suitable for use as structural fill during dry weather conditions only. If structural fill is placed during wet weather, the structural fill should consist of Gravel Borrow, WSDOT Standard Specifications Section 9-03.14(1), with the additional restriction that the fines content be limited to 5 percent.

Crushed rock placed below pavements should consist of Crushed Surfacing Base Course, WSDOT Standard Specifications Section 9-03.9(3).

Structural Fill Placement

Structural fill should be placed in horizontal lifts which are 10 inches or less in loose thickness. The moisture content of the fill soil must be adjusted as necessary to achieve the required degree of compaction. Each lift must be compacted to the appropriate specification before placing subsequent layers.

We recommend that structural fill below the control building foundations and floor slab be compacted to at least 95 percent of the MDD obtained using ASTM D 1557 test procedures. Elsewhere, the structural fill should be compacted to at least 90 percent of the MDD, except that the upper 2 feet of fill in pavement areas should be compacted to at least 95 percent of the MDD. Crushed rock placed below the wet well structure should be compacted to at least 95 percent of the MDD.

All backfill placed on existing slopes should be ‘keyed’ into the slope by cutting a series of horizontal benches into the slope. We recommend the horizontal benches be a minimum of 3 feet wide. A representative of our firm should observe the preparation for, placement, and compaction of structural fill. An adequate number of in-place density tests should be performed in the fill to evaluate if the specified degree of compaction is being achieved.

Pipe Bedding

We recommend that all structural fill placed as pipe bedding meet the criteria for Gravel Backfill for Pipe Zone Bedding as described in Section 9-03.12(3) of the WSDOT Standard Specifications. Pipe bedding should be placed in accordance with WSDOT Standard Specification 7-08.3(1)C. Where soft or loose soils are encountered below the pipe alignment, we recommend they be removed to a depth of 12 inches, or to firm material as directed by the engineer.

Wet Weather Construction

As previously discussed, the existing soils throughout the project area contain significant fines (material passing the U.S. No. 200 sieve) and are therefore moisture sensitive. If site preparation is completed during wet weather, earthwork and construction operations should be planned to reduce the exposure of subgrade areas to wet weather and construction traffic. In addition, we recommend the following considerations during wet weather:

- The ground surface in and around the work area should be sloped so that surface water is directed away from the work area. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work area.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- The site soils should not be left un-compacted and exposed to moisture. Sealing the surficial soils by rolling with a smooth-drum roller prior to periods of precipitation will reduce the extent to which these soils become wet or unstable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are surfaced with materials not susceptible to wet weather disturbance.
- Construction activities should be scheduled so that the length of time that soils are left exposed to moisture is reduced to the extent practical.

Pavement Recommendations

Subgrade Preparation

Prior to placing new fills or pavement base course materials, new pavement subgrade areas should be evaluated by proof rolling or probing to identify zones of soft or pumping soils. Proof rolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather, the exposed subgrade should be probed to identify the extent of soft soils. If zones of soft or pumping soil are identified, they should be removed and replaced with structural fill.

It is critical that all construction traffic be kept off the silty subgrade soils during wet weather to prevent disturbance (rutting and weaving) from occurring. We recommend placing a minimum 8-inch-thick gravel borrow base to perform as a drainage layer between the silty soils and the pavement base course. The minimum thickness is not intended to serve as a working surface for construction traffic during wet weather.

In light duty pavement areas, we recommend a pavement section consisting of at least a 2-inch thickness of ½-inch HMA (PG 58-22) over a 6-inch thickness of densely compacted crushed rock base course. In heavy-duty pavement areas, we recommend a pavement section consisting of at least a 3-inch thickness

of ½-inch HMA (PG 58-22) over a 6-inch thickness of densely compacted crushed rock base course. The base course should conform to WSDOT 9-03.9(2).

The base course should be compacted to at least 95 percent of the MDD (ASTM D 1557). The HMA should be compacted to a minimum of 90 percent of the reference maximum density in accordance with WSDOT 5-04.3(10).

The pavement sections recommended above are based on our experience. Thicker asphalt sections may be needed based on the actual traffic data and intended use.

Erosion and Sedimentation Control

Potential sources or causes of erosion and sedimentation depend upon construction methods, slope length and gradient, amount of soil exposed and/or disturbed, soil type, construction sequencing and weather. The project's impact on erosion-prone areas can be reduced by implementing an erosion and sedimentation control plan. The plan should be designed in accordance with applicable City of Kirkland standards. The plan should incorporate basic planning principles including:

- Scheduling grading and construction to reduce soil exposure;
- Retaining existing vegetation whenever feasible;
- Revegetating or mulching denuded areas;
- Directing runoff away from denuded areas;
- Minimizing the length and steepness of slopes with exposed soils;
- Decreasing runoff velocities;
- Confining sediment to the project site;
- Inspecting and maintaining control measures frequently;
- Covering soil stockpiles; and
- Implementing proper erosion control best management practices (BMPs).

Temporary erosion protection should be used and maintained in areas with exposed or disturbed soils to help reduce the potential for erosion and reduce transport of sediment to adjacent areas. Temporary erosion protection should include the construction of a silt fence around the perimeter of the work area prior to the commencement of grading activities. Permanent erosion protection should be provided by reestablishing vegetation using hydroseeding and/or landscape planting.

Until the permanent erosion protection is established and the site is stabilized, site monitoring should be performed by qualified personnel to evaluate the effectiveness of the erosion control measures and repair and/or modify them as appropriate. Provisions for modifications to the erosion control system based on monitoring observations should be included in the erosion and sedimentation control plan.

Recommended Additional Geotechnical Services

Throughout this report, recommendations are provided where we consider additional geotechnical services to be appropriate. These additional services are summarized below:

- GeoEngineers should be retained to provide addition input for final design of the geotechnical elements of the project, and to review the project plans and specifications when complete to confirm that our design recommendations have been implemented as intended.
- During construction, GeoEngineers should observe excavation and shoring installation, evaluate temporary and permanent slope conditions, observe installation of dewatering and subsurface drainage measures, observe and evaluate the suitability of foundation and slab subgrades, observe and test structural backfill, evaluate the suitability of pavement subgrades and other appurtenant structures, and provide a summary letter of our construction observation services. The purposes of GeoEngineers' construction phase services are to confirm that the subsurface conditions are consistent with those observed in the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, to evaluate whether or not earthwork and foundation installation activities are completed in accordance with our recommendations, and other reasons described in Appendix C, Report Limitations and Guidelines for Use.

LIMITATIONS

We have prepared this report for use by the City of Kirkland and Murray, Smith & Associates, Inc. in design and construction of the Rose Point Lift Station Upgrade project located in Kirkland, Washington. Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted geotechnical practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood. Please refer to Appendix C titled Report Limitations and Guidelines for Use for additional information pertaining to use of this report.

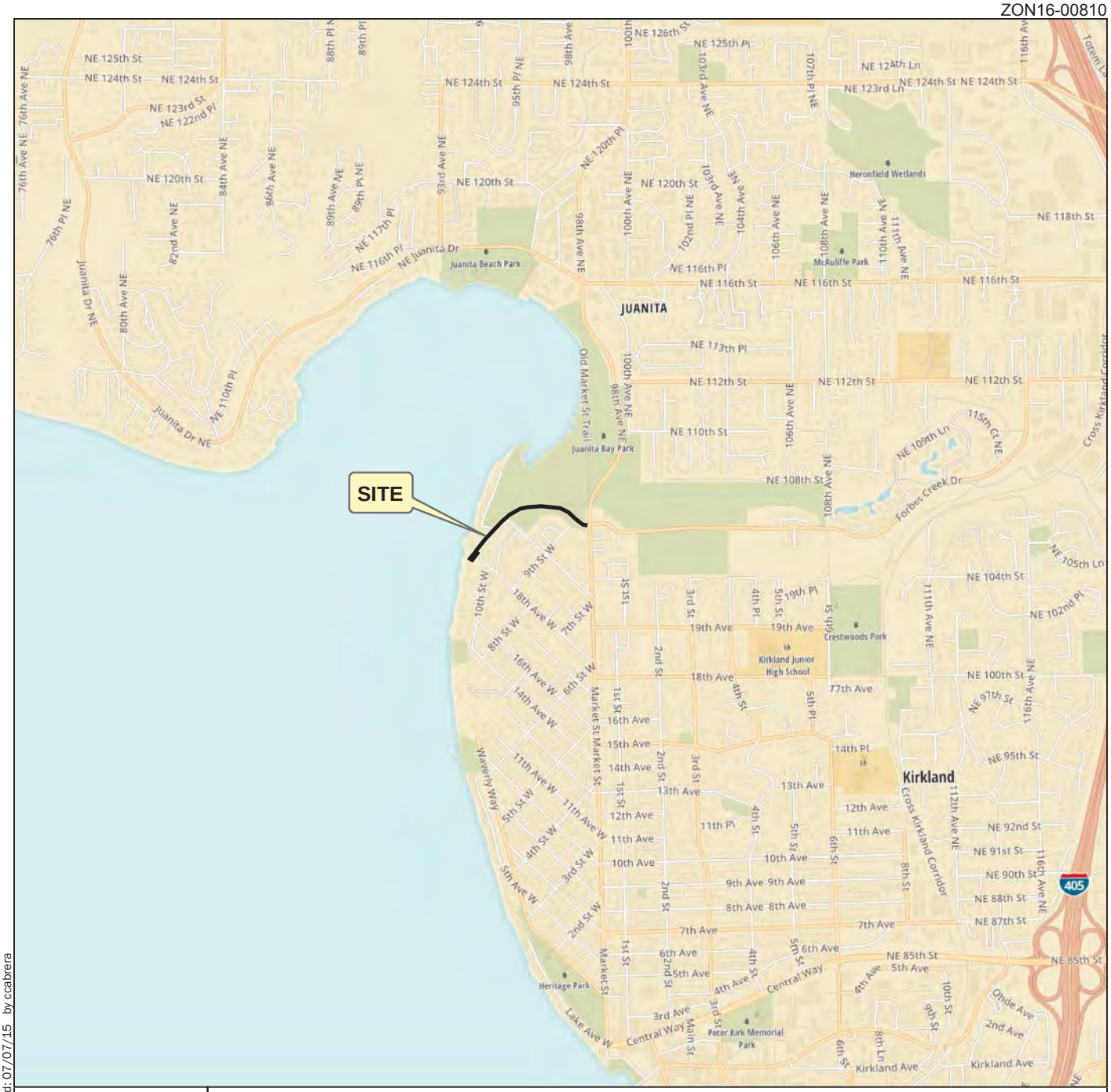
Our services were provided to assist in the design of structures on sloping ground. Our recommendations are intended to improve the overall stability of the site and to reduce the potential for future property damage related to earth movements, drainage or erosion. Qualified engineering and construction practices can help mitigate the risks inherent in construction on slopes, although those risks cannot be eliminated completely. Favorable performance of structures in the near term is useful information for anticipating future performance, but it cannot predict or imply a certainty of long-term performance, especially under conditions of adverse weather or seismic activity.

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by GeoEngineers, Inc. and will serve as the official document of record.

We appreciate this opportunity to be of service to the design team on this project. Please call if you have any questions regarding this report or we can provide additional assistance.

FIGURES

GeoEngineers



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Feet

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Mapbox Open Street Map, 2015

Projection: NAD 1983 UTM Zone 10N

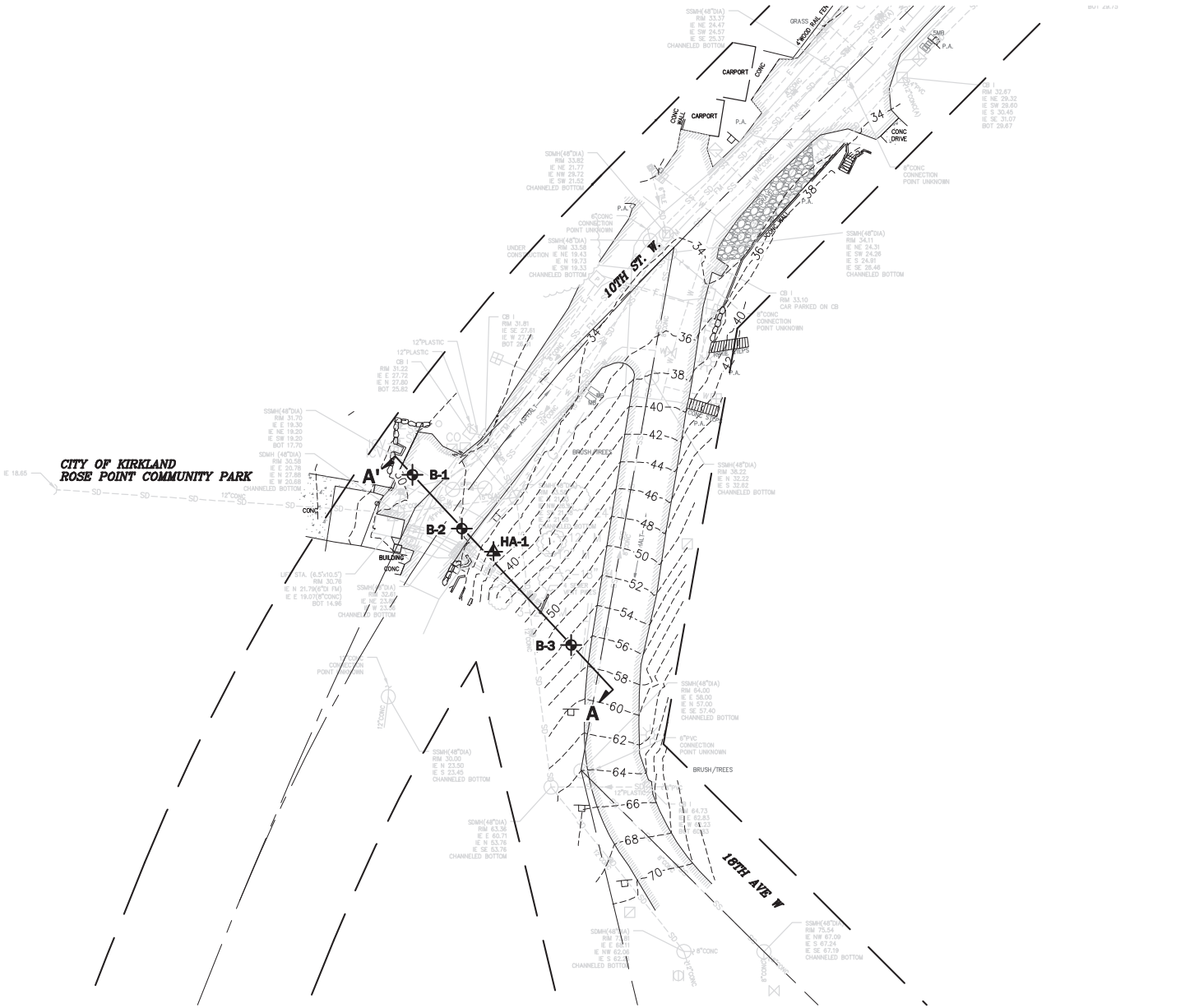
Vicinity Map

Rose Point Lift Station Upgrade
Kirkland, Washington

GEOENGINEERS

Figure 1

\\red\projects\0\0231087\GIS\023108700_Tsk100_F1_VM.mxd Date Exported: 07/07/15 by ccabrera



Legend

B-1 Boring by GeoEngineers, 2015

HA-1 Hand Auger by GeoEngineers, 2015

A A' Cross-Section Location

N
W E
S

40 0 40
FEET

Notes:

1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Base survey by Duane Hartman & Associates, Inc. dated 7-02-2015.

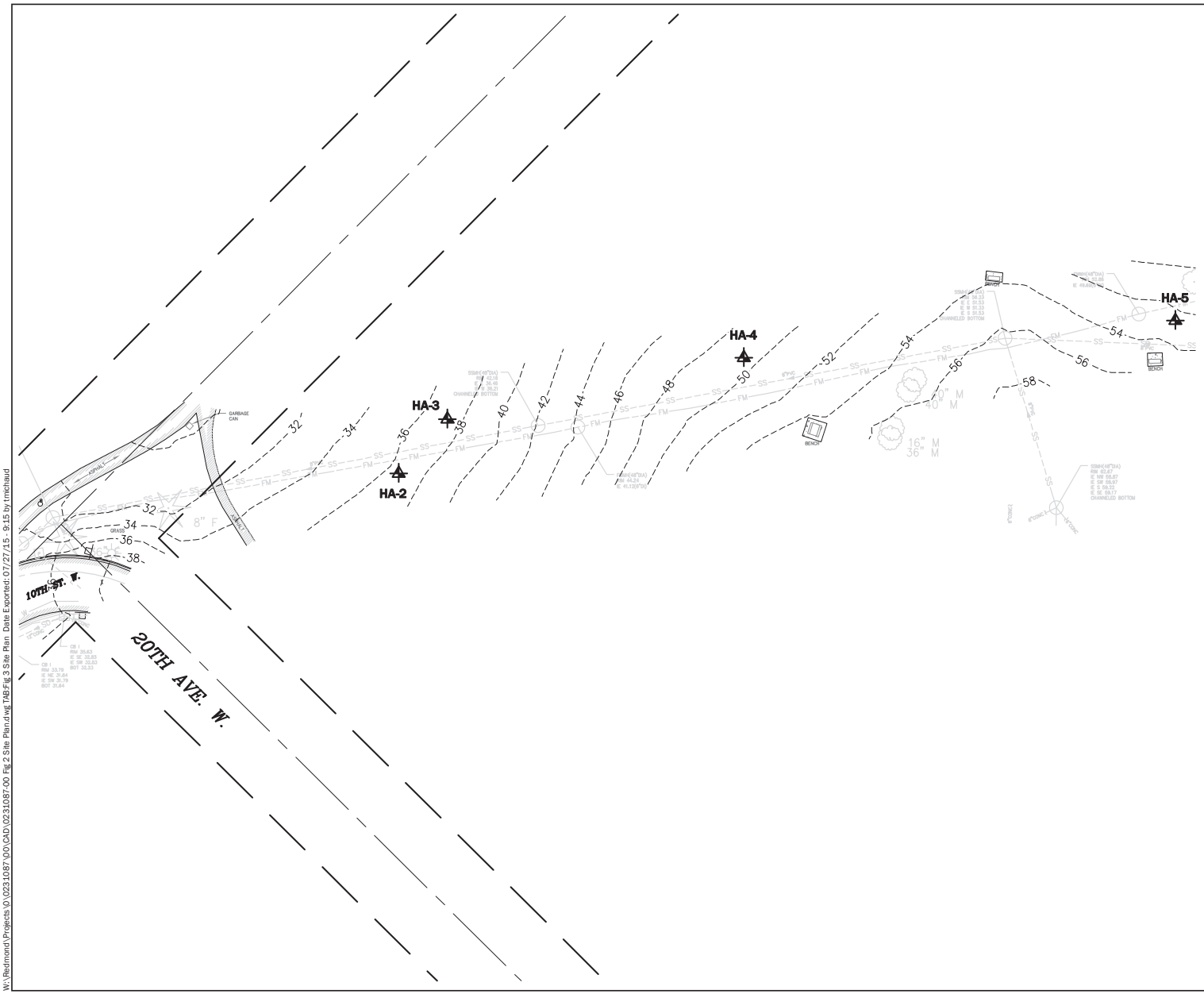
Site Plan

Rose Point Lift Station Upgrade
Kirkland, Washington

GEOENGINEERS

Figure 2

W:\Redmond\Projects\0\0231087\00\CAD\0231087-00 Fig 2 Site Plan.dwg TAB:Fig 2 Site Plan Date Exported: 07/27/15 - 9:15 by tmichaud



Legend

HA-1 Hand Auger by GeoEngineers, 2015

N
W E
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50 0 50
FEET

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source:
Base survey by Duane Hartman & Associates, Inc. dated 7-02-2015.

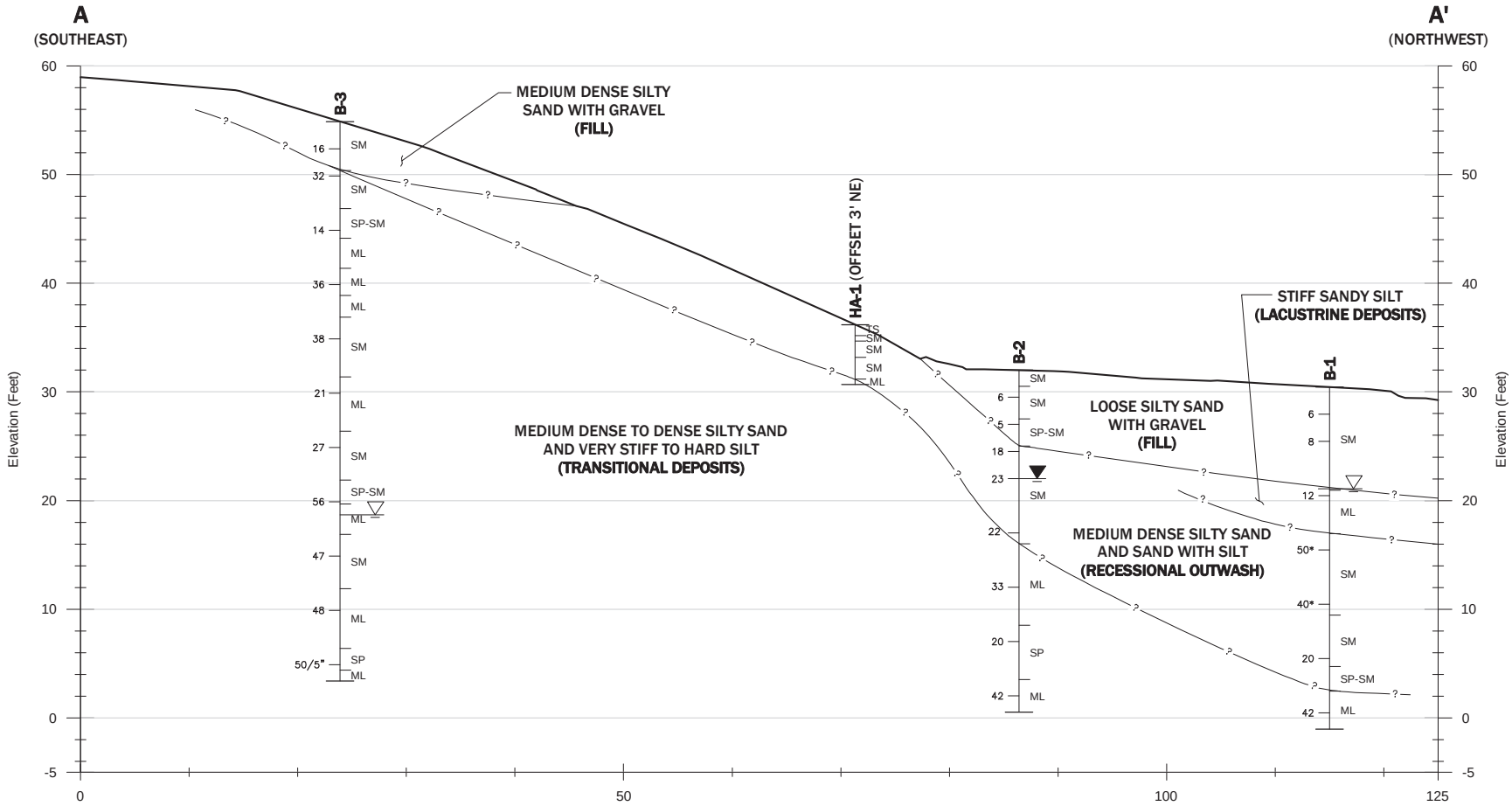
Site Plan

Rose Point Lift Station Upgrade
Kirkland, Washington

GEOENGINEERS

Figure 3

W:\Redmond\Projects\0\0231087\00\CAD\0231087-00 Fig 2 Site Plan.dwg TAB:Fig 3 Site Plan Date Exported: 07/27/15 - 9:15 by tmichaud



Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.
2. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The hard copy is stored by GeoEngineers, Inc. and will serve as the official document of record.
3. Datum: NAVD 88, unless otherwise noted.

Legend

B-1 Boring

? Inferred Soil Contact

Groundwater Level Observed During Drilling

Groundwater Level Observed in Piezometer

SM Soil Classification

25 Blow Count
* Blow count exaggerated due to heave.

10 0 10
Horizontal Scale in Feet

10 0 10
Vertical Scale in Feet
Vertical Exaggeration: 1X

Cross-Section A-A'

Rose Point Lift Station Upgrade
Kirkland, Washington

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Figure 4

W:\Redmond\Projects\0\0231087\00\CAD\0231087-00 Fig2 Site Plan.dwg TAB:Fig 4 CrossSection Date Exported: 07/27/15 - 9:33 by tmichaud

APPENDICES

GeoEngineers

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Subsurface soil and groundwater conditions were evaluated by drilling three borings (B-1 through B-3) and completing five hand auger holes (HA-1 through HA-5) at the approximate locations shown on the Site Plans, Figures 2 and 3. The borings were drilled to depths ranging from about 31½ to 51½ feet below the existing ground surface using a track-mounted Diedrich D-50 track-mounted drill rig owned and operated by Geologic Drill Exploration, Inc. The hand auger holes were completed to depths of 3½ to 8½ feet using hand tools.

Exploration locations were estimated in the field by measuring distances from existing site features. Ground surface elevations at the exploration locations were estimated by interpolation from contours on preliminary topographic survey plans dated July 2, 2015 and prepared by Duane Hartman & Associates, Inc. Exploration locations and elevations should be considered accurate to the degree implied by the methods used.

The explorations were continuously observed by a member of our geotechnical engineering staff who evaluated and classified the soils encountered, obtained representative soil samples, and observed groundwater conditions. Our representative maintained a detailed log of each exploration. Disturbed samples of the representative soil types were obtained from the borings using standard penetration test (SPT) sampling procedures. SPT sampling was performed using a 2-inch outside diameter split-spoon sampler driven with a standard 140-pound hammer in accordance with ASTM D 1586. Disturbed bag samples were obtained from the hand auger holes.

The soils encountered in the borings were typically sampled at 2½- to 5-foot vertical intervals with the SPT split spoon sampler. Samples were obtained by driving the sampler into the soil with a hammer free-falling 30 inches. The number of blows required to drive the sampler the final 12 inches or other specified distance is noted on the boring logs.

Soils encountered in the borings and hand auger holes were classified in the field using ASTM D 2488, which is summarized in Figure A-1. Logs of the borings are provided in Figures A-2 through A-4, and logs of the hand auger holes are provided in Figures A-5 through A-9. The logs reflect our interpretation of the field conditions and the results of geotechnical laboratory evaluation and testing of samples. They also indicate the depths at which the soil types or their characteristics change, although the change may be gradual. If the change occurred between samples, it was interpreted.

The soil samples were logged, sealed in plastic bags and transported to our Redmond geotechnical laboratory. The field classifications were further evaluated in our laboratory.

A monitoring well (1-½-inch diameter) was installed in boring B-2 to allow measurement of groundwater levels following drilling. We measured the groundwater level in the well on June 23, 2015, 1 day after drilling. The groundwater level measurement is indicated on the boring log.

The monitoring well is the property of the City of Kirkland. The well should be decommissioned by a licensed well driller in accordance with Chapter 173-160 of the Washington Administrative Code (WAC) when it is no longer needed for data collection. Alternatively, the well could be kept intact for use during project bidding and then be decommissioned under the construction contract.

Soil cuttings generated from drilling were transported for eventual disposal at an off-site facility.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	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
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		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 NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		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% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, 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 SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	TS	Topsoil/ Forest Duff/Sod

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Material Description Contact

Distinct contact between soil strata or geologic units

Approximate location of soil strata change within a geologic soil unit

Sampler Symbol Descriptions

2.4-inch I.D. split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

A "P" indicates sampler pushed using the weight of the drill rig.

Laboratory / Field Tests

%F	Percent fines
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PP	Pocket penetrometer
PPM	Parts per million
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen
NT	Not Tested

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS

GEOENGINEERS

FIGURE A-1

Drilled Start 6/22/2015 End 6/22/2015	Total Depth (ft) 31.5	Logged By MWR Checked By HRP	Driller Geologic Drill Exploration, Inc.	Drilling Method Hollow-stem Auger
Surface Elevation (ft) Vertical Datum 31		Hammer Data Auto 140 (lbs) / 30 (in) Drop		Drilling Equipment Deidrich D 50 Track Rig
Latitude Longitude		System Datum Geographic		Groundwater Date Measured / Depth to Water (ft) / Elevation (ft)
Notes: Auger Data: 3¼-inch I.D. , 6¼-inch O.D.				See Remarks

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Water Level	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0								AC	1" asphalt concrete pavement			
30									SM	Brown silty fine to coarse sand with gravel, trace organic matter and asphalt fragments (loose, moist) (fill)			
			9	6		1 / MC					5		
	5		2	8		2				Grades to wet			
25													Groundwater seepage observed at 9 feet during drilling
	10		12	12		3 / SA			ML	Gray and orange brown sandy silt with lenses of silty sand (stiff, wet) (lacustrine deposits)	28	59	
20									SM	Brown silty fine to medium sand with lenses of sandy silt and silt and occasional gravel (medium dense, wet) (recessional outwash)			
	15		18	50*		4 / SA					25	14	3 feet of heave encountered before sampling *Full sampler - blowcount overstated
15													
	20		18	40*		5							Water added to borehole to control heave *Full sampler - blowcount overstated
10									SM	Orange brown silty fine to medium sand with lenses of silt (medium dense, wet)			
	25		18	20		6				Grades to brown			
5									SP-SM	Gray fine to medium sand with silt and silt lenses (medium dense, wet)			
									ML	Gray brown silt with sand, thin sand lenses, and occasional organic matter (hard, wet) (transitional deposits)			
	30		16	42		7							
0													

Notes: See Figure A-1 for explanation of symbols.

Log of Boring B-1

GEOENGINEERS

Project: Rose Point Lift Station Upgrade
Project Location: Kirkland, Washington
Project Number: 0231-087-00

Figure A-2
Sheet 1 of 1

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