Geotechnical Design Memorandum
North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

September 27, 2019

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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GEOTECHNICAL DESIGN MEMORANDUM
NORTH MERCER INTERCEPTOR AND
ENATAI INTERCEPTOR UPGRADE PROJECT
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

This geotechnical design memorandum (GDM) presents the results of our geotechnical engineering studies completed for the North Mercer Interceptor and Enatai Interceptor Upgrade Project (Project) located on Mercer Island and in Bellevue, Washington (Figure 1). This GDM was prepared for the exclusive use of King County and the Tetra Tech design team and their representatives to assist with advancing the Project to final design. Included in this GDM are a site and Project description, an overview of the completed and existing geotechnical explorations and laboratory testing, the interpreted subsurface soil and groundwater conditions, engineering recommendations, construction considerations, and recommendations for geotechnical instrumentation. The geotechnical data, which provides the basis for the interpretations and recommendations presented in this report, are included in the Geotechnical Data Report (GDR) (Shannon & Wilson, 2019a).

Our studies were performed for final design purposes and should not be used for construction. This GDM should not be used without our approval if any of the following occurs:

- Conditions change due to natural forces or human activity under, at, or adjacent to the site.
- Assumptions stated in this GDM have changed.
- Project details change or new information becomes available such that our recommendations may be affected.
- If the site ownership or land use has changed.
- More than five years has passed since the date of this GDM.

If any of these occur, we should be retained to review the applicability of our recommendations.

Our services to prepare this GDM were conducted in accordance with Task 3200 as described in Amendment 8 to our agreement with Tetra Tech (Tetra Tech Job No. 200-12539-18001) and King County Contract No. E00306E13. Our scope of services did not include:

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- Evaluating the presence of cultural resources around the site.
- Performing wetland delineation at the site.
- Removing observation wells that we installed at the site. It is the Owner's responsibility to properly decommission subsurface installations in accordance with state regulations when use of the observation wells is no longer needed.
- Restoring pavement to pre-exploration conditions.

If a service is not specifically indicated in this report, do not assume that it was performed.

2.0 SITE AND PROJECT DESCRIPTION

The Project is located on Mercer Island and in the Enatai neighborhood of Bellevue (Figure 1). The primary objective of the Project is to increase the reliability and capacity of the existing North Mercer Island Interceptor and Enatai Interceptor components of the regional wastewater system by increasing capacity and replacing the conveyance from the North Mercer Pump Station (NMPS) to the Sweyolocken Pump Station (SPS). For contracting purposes, the Project has been separated into two design packages, designated as Contract No. C01339C20 and C01340C20. Contract No. C01339C20 includes modifications to the NMPS, modifications to the City of Mercer Island Lift Station No. 11 (LS 11), and sections of new and rehabilitated force main from LS 11 to the new conveyance in Contract No. C01340C20. Contract No. C01340C20 includes the new conveyance pipeline and existing pipeline rehabilitation from the NMPS to the SPS.

The modifications to the existing NMPS (Contract No. C01339C20) include a new generator building, BIOXIDE® storage tank, fuel storage tank, electric transformer, electric vault, odor control vessel, and about 165 feet of dual force mains. The Project will also include new concrete retaining walls along the west side of the Project site. In addition to these permanent facilities, the Project will require a temporary pump station maintenance hole, valve vault, meter vault, and temporary force main. The NMPS facilities locations are presented in the Site and Exploration Plan, North Mercer Pump Station, Contract No. C01339C20, Figure 3.

The new generator building at the NMPS will be a 30-foot-wide by 40-foot-long, single-story aboveground structure founded on shallow spread or continuous footings. Although the generator building is primarily above ground, the west side of the building will be constructed into the existing slope and will be founded about 6 to 8 feet below ground surface (bgs). The temporary pump station, which will be used during modifications to the pump station, will be a 12-foot-diameter structure founded about 35 feet bgs. The valve vault for the temporary pump station will be an 8-foot-wide by 10-foot-long buried structure founded approximately 8 feet bgs.

The new BIOXIDE® storage (relocated), fuel storage, and transformer will be aboveground facilities and founded on slabs on grade. The new odor control vessel will be a 9-foot-wide by 9-foot-long buried structure founded approximately 8 feet bgs. It is our understanding that the new retaining walls along the west side of the Project site will be conventional reinforced concrete cantilever walls that are 4 to 8 feet high. The depth of the dual force mains between the NMPS and the right-of-way of SE 22nd Street ranges from 9 to 15 feet bgs.

The modifications to the existing LS 11 (Contract No. C01339C20) include renovations to the existing dry and wet wells, a new valve vault, about 340 feet of existing 10-inch-diameter asbestos concrete force main that will be rehabilitated using cured-in-place pipe (CIPP) methods, about 275 feet of new 10-inch-diameter ductile iron force main, and a new maintenance hole. The new valve vault for LS 11 will be an approximate 8-foot-wide by 14-foot-long precast concrete vault founded approximately 11 feet bgs. The depth of the new 10-inch-diameter ductile iron force main from LS 11 will vary from 5 to 10 feet bgs. The locations of the proposed LS 11 facilities are presented in the Site and Exploration Plan, Conveyance, Contract No. C01340C20, Figure 2.

The new conveyance (Contract No. C01340C20) includes a dual force mains that extend from the NMPS to the cul-de-sac at the end of 90th Place SE. The force main alignment will run primarily along the right-of-way of SE 22nd Street, SE 22nd Place, 78th Avenue SE, SE 24th Street, 81st Avenue SE, N Mercer Way, the Mountains to Sound Greenway Trail (I-90 Trail), and 90th Place SE. At the end of 90th Place SE, the force mains transition to a gravity sewer that continues southeasterly along the I-90 Trail to just west of E Mercer Way, where the gravity sewer transitions to a triple-barrel siphon. The triple-barrel siphon extends to the east along SE 35th Place and through the Washington State Department of Transportation (WSDOT) right-of-way (City of Mercer Island boat launch) to the west shoreline of the East Channel of Lake Washington (East Channel). The triple-barrel siphon then crosses the East Channel to the north of the I-90 East Channel bridge as shallow buried in-water pipelines to the east shoreline of the East Channel. From the east shoreline in Enatai, the flows are directed either to the existing Enatai Interceptor through a new gravity pipeline in Enatai Beach Park or through a new deep siphon beneath Enatai and a new gravity pipeline between the siphon and the SPS. The depth of the new dual force mains, gravity sewer, and triple-barrel siphon on Mercer Island generally ranges from 5 to 25 feet bgs. For the East Channel crossing, the triple-barrel siphon is typically 2 to 11 feet below the existing mudline. For the deep siphon, the depth of the pipeline ranges from 15 feet to as much as 160 feet bgs. The proposed pipeline alignment and profile are presented in the Site and Exploration Plan, Conveyance, Contract No. C01340C20, Figure 2 and the Generalized Subsurface Profile, Conveyance, Contract No. C01340C20, Figure 5.

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It is our understanding that the force mains, gravity sewer, diversion sewer, and triple-barrel siphons on Mercer Island and Enatai will be installed using conventional trenching methods, and the triple-siphon crossing of the East Channel will be installed using barge-mounted excavators or dredging equipment. It is also our understanding that the deep siphon beneath Enatai will be installed using horizontal directional drilling (HDD) methods.

As part of Contract No. C01340C20, there are new maintenance or transition structures and additional sewer and siphon connections that will be required to collect and divert existing flows into or out of the new conveyance system. The new structures along the conveyance include (a) three force main inspection and air/vacuum release structures; (b) two force main drain vaults; (c) NMPS force main discharge structure; (d) East Channel siphon rock catcher, inlet, maintenance, and diversion structures; (e) Enatai siphon inlet and outlet structures; and (f) three odor control structures each located near the NMPS force main discharge, East Channel siphon inlet, and East Channel siphon diversion structures. The additional pipeline connections include the conversion of an existing pipeline beneath I-90 into a siphon that diverts flow into the new gravity sewer at 96th Avenue SE and a new connector sewer in Enatai Beach Park that diverts flows from the new conveyance pipeline to the existing Enatai Interceptor. In addition to these pipeline sections, the existing Enatai Interceptor will be replaced in Enatai Beach Park, repaired using grouting or CIPP point repair at angle structures along the south shoreline of Enatai, and rehabilitated using CIPP methods along the west side of the Mercer Slough. The general proposed locations for the structures and additional pipeline connections are shown in Figure 2.

Each of the three-force main inspection and air/vacuum release structures will be 14 feet in diameter and founded 9 to 13 feet bgs. Each of the two force main drain vaults will be 8 feet in diameter and founded about 11 and 15 feet bgs. The NMPS force main discharge structure will be located at the end of 90th Place SE on Mercer Island. The force main discharge structure will be 8 feet wide by 12 feet long and founded 12 feet bgs. The East Channel siphon rock catcher and inlet structures will be located in the SE 35th Place cul-de-sac, just west of E Mercer Way on Mercer Island. The East Channel siphon rock catcher will be 6 feet wide by 14 feet long and founded 15 feet bgs. The East Channel siphon inlet structure will be 14 feet wide by 17 feet long and will be founded about 15 feet bgs. The East Channel siphon maintenance structure will be located in the City of Mercer Island boat launch on WSDOT property. The maintenance structure will be about 9.5 feet wide by 16 feet long and founded about 10 feet bgs. The proposed force main discharge and East Channel siphon rock catcher, inlet, and maintenance structures are presented in plan and profile in Figures 2 and 5, respectively.

The East Channel siphon flow diversion structure will be located beneath the East Channel bridge in Enatai. The structure will be 20 feet wide by 21 feet long and will be founded about

14 feet bgs. The Enatai siphon inlet structure, located just north of the East Channel bridge in Enatai, will be 10 feet wide by 12 feet long and will be founded about 21 feet bgs. The Enatai siphon outlet structure will be located just south of the SPS in Enatai and will consist of an 10-foot-wide by 12-foot-long structure that is founded about 14 feet bgs. The proposed East Channel siphon flow diversion, Enatai siphon inlet, and Enatai siphon outlet structures are presented in plan and section in Figures 2 and 5, respectively.

The three odor control structures will be located near the end of the dual force mains on 90th Place SE and near the East Channel siphon inlet and flow diversion structures. The 90th Place SE odor control structure will be 17 feet wide by 24 feet long and founded about 11 feet bgs. The East Channel siphon inlet odor control structure will be 17 feet wide by 23 feet long and founded about 13.5 feet bgs. The East Channel siphon flow diversion odor control structure will be 19 feet wide by 25 feet long and will be founded about 14.5 feet bgs. The proposed odor control structures are shown in plan and section in Figures 2 and 5, respectively.

The depth of the new diversion sewer in Enatai Beach Park will range from 12 to 20 feet bgs. Similar to the conveyance pipelines discussed above, we understand that these pipelines will be installed using conventional trenching methods.

The vertical datum for this Project is the King County (KC) Metro datum. The KC Metro datum is 100 feet above the National Geodetic Vertical Datum of 1929 and 96.41 feet above the North American Vertical Datum of 1988.

3.0 SUBSURFACE EXPLORATIONS

Shannon & Wilson completed a geotechnical exploration program to characterize the soil and groundwater conditions present along current and previous alternative Project alignments. The geotechnical explorations included collecting and reviewing existing subsurface exploration data near the Project alignment and completing Project subsurface explorations.

3.1 Existing Subsurface Explorations

The collected existing subsurface explorations in the Project vicinity include 63 exploration logs from borings and test pits from previously completed projects. The samples from these explorations are not available for review and we cannot confirm that these explorations are representative of the site conditions. These exploration logs were collected from the Washington State Division of Geology and Earth Resources Subsurface Database, WSDOT, KC records, and from Shannon & Wilson's files. The approximate locations of the existing subsurface explorations are shown in the conveyance alignment (Contract No. C01340C20) and the NMPS

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site (Contract No. C01339C20) in Figures 2 and 3, respectively. The associated boring and test pit logs are presented in Appendix E of the GDR (Shannon & Wilson, 2019a).

3.2 Current Subsurface Explorations

A total of 45 geotechnical borings were drilled and sampled to characterize the subsurface conditions along the alignment. The field testing included standard penetration tests in borings, measuring the depth to groundwater in monitoring wells, and readings of vibrating wire piezometers installed in borings. Geotechnical laboratory tests included visual classification and index testing to determine the natural water content, grain size distribution, and Atterberg Limits. The results from the geotechnical explorations, field testing, and laboratory testing are included in the GDR (Shannon & Wilson, 2019a). The locations of the Project borings are shown in the Site and Exploration Plan, Conveyance, Figure 2, and Site and Exploration Plan, North Mercer Pump Station, Figure 3.

4.0 SUBSURFACE CONDITIONS

The geology and subsurface conditions along the Project alignment were inferred from soil samples and information obtained from borings and observation wells, data gathered from existing projects in the vicinity, geologic maps of the area, field reconnaissance, and our experience on other projects in the area. Our observations are specific to the locations and depths noted on the logs and profiles and may not be applicable to all areas of the site. No amount of explorations or testing can precisely predict the characteristics, quality, or distribution of subsurface and site conditions. Potential variation includes, but is not limited to the following:

- The conditions between and below explorations may be different.
- The passage of time or intervening causes (natural and manmade) may result in changes to site and subsurface conditions.
- Groundwater levels and flow directions may fluctuate due to seasonal variations.
- Groundwater flow between different aquifers can occur. No soil layer should be assumed to be continuous and/or watertight.
- Penetration test results in gravelly soils may be unrealistic. Actual soil density may be lower than estimated if the test was performed on gravel or cobble.
- Obstructions such as boulders, piles, foundations, rubble, etc., may be present in the subsurface.

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If conditions different from those described herein are encountered, we should be advised so we can review our description of the subsurface conditions and reconsider our conclusions and recommendations.

The following sections include a description of the general geology, soil types, tectonic setting, subsurface soil, and groundwater conditions encountered at the NMPS site (Contract No. C01339C20) and along the conveyance alignments (Contract No. C01340C20), geologic hazards, and soil properties.

4.1 General Geology

The topography in the Project area is the result of the last glaciation of the central Puget Sound Lowland between approximately 15,000 and 13,500 years ago and the geologic processes since that time. The subsurface geologic conditions may involve soils deposited during one or more of at least six glacial advances and intervening interglacial periods that have occurred within the Puget Sound area within the last 2 million years. During the last glaciation (Vashon Stade of the Fraser Glaciation) in the Puget Sound area, glaciolacustrine clay and silt, glacial outwash sand and gravel, and till and till-like soils were deposited by the glacier and were consolidated by the weight of about 3,000 feet of ice (Thorson, 1989). As the last ice to reach the Puget Lowland (Vashon Stade) retreated to the north, deposits of gravel, sand, silt, and clay were laid down by meltwater streams issuing from the glacial ice front. These deposits are termed glacial recessional soils and are not glacially consolidated. Since the last glacier retreated, lacustrine and landslide soils were deposited. Where development has occurred, these soils have been covered with fill, structures, and roadways, or potentially removed.

The non-glacially consolidated soil types observed in the existing geotechnical data and current borings along the alignment include the following:

- **Fill:** Fill deposits are placed by humans and can be both engineered and nonengineered. The deposits consist of various compositions of clay, silt, sand, and gravel and may contain other materials, including debris, cobbles, and boulders. Typically, engineered fill is dense or stiff and nonengineered fill is very loose to medium dense or very soft to stiff.
- **Landslide Deposits:** Deposits of landslides, normally at and adjacent to the toe of slopes but could also be landslide debris moved by human activity after deposition. The deposits consist of disturbed, heterogeneous material with several soil types, including organic debris. These soils are typically loose or soft with random dense or hard pockets. Wood, cobbles, and boulders may be present.

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- **Lacustrine Deposits:** Glaciolacustrine and lake sediments deposited as and after the glacier retreated. Lacustrine sediments consist of very soft to medium stiff, silty clay and clayey silt or very loose to medium dense silt and sandy silt. Cobbles and boulders may be present.
- **Recessional Deposits:** Recessional sediments were deposited as the glacier retreated and, therefore, were not overridden. Ice-Contact and Ablation Till consist of heterogeneous soils deposited during the wasting of glacial ice consisting of silty sand with gravel, sandy silt with gravel, and clayey sand. Recessional outwash consists of clean to silty sand, gravelly sand, and sandy gravel. Typically, these deposits are loose to dense and may contain cobbles or boulders.

The glacially overconsolidated soil types observed in the existing geotechnical data and current borings along the alignment include the following:

- **Till and Till-Like Deposits:** This unit was directly deposited as either lodgment till at the base of an advancing glacier or was ice margin deposits that were subglacially reworked (till-like) and subsequently overridden by the ice. Glacial till and till-like deposits generally consist of silty sand with gravel, sandy silt with gravel, or silty sand. Cobbles and boulders are present. Typically, glacial till and till-like deposits are dense to very dense.
- **Glacial Outwash and Nonglacial Fluvial Deposits:** Glaciofluvial sediments were deposited by glacial activity and were subsequently overridden. Nonglacial fluvial sediments were deposited in fluvial environments and subsequently overridden by advancing glaciers. Glacial outwash and nonglacial fluvial deposits are typically dense to very dense, clean to silty sand; gravelly sand; and sandy gravel with cobbles and boulders present.
- **Glaciolacustrine Deposits:** Glaciolacustrine deposits are fine-grained sediments deposited in a lake in front of the glacier. These deposits were subsequently overridden by advancing glaciers. Glaciolacustrine deposits typically are very stiff to hard silty clay and clayey silt or dense to very dense silt and sandy silt. Cobbles and boulders may be present.
- **Nonglacial Lacustrine Deposits:** Nonglacial lacustrine deposits are fine-grained sediments deposited in a lake. These deposits were then overridden as glaciers continued to advance. Nonglacial lacustrine deposits typically are very stiff to hard silty clay and clayey silt or dense to very dense silt and sandy silt. Cobbles and boulders may be present.

4.2 Tectonic Setting

The Puget Lowland is located near the Cascadia Subduction Zone. The tectonics and seismicity of the region are the result of the relative northeastward subduction of the Juan de Fuca Plate beneath the North American Plate. North-south compression is accommodated beneath the

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Puget Lowland by a series of west- and northwest-trending faults that extend to depths of about 12 miles. The nearest active faults to the Project are termed the Seattle Fault Zone, which consists of a series of four or more east-west-trending, south-dipping fault splays beneath Seattle. Based on the Department of Natural Resources (DNR) Washington Interactive Geologic Maps, the Project alignment is situated within the Seattle Fault Zone. It extends from Bremerton to south Bellevue along an alignment that is roughly coincident with I-90. Geologic evidence discovered in the 1990s indicates that ground surface rupture from movement on this fault zone occurred as recently as 1,100 years before present. One interpreted east-west-trending splay of the Seattle Fault Zone crosses the alignment near Station (Sta.) 80+50 and roughly parallels the alignment to the SPS. It should be noted that the location of this east-west-trending fault splay is interpreted and deep and not representative of the location near the surface.

4.3 Subsurface Conditions

Geologic profiles showing the anticipated soil and groundwater conditions along the Project alignment are presented in Figures 5 and 6. Variations between the interpretation shown and actual conditions will exist. The legend and notes for the geologic profile, including a listing and description of the interpreted soil types encountered in the borings, are presented in Figure 4.

4.3.1 NMPS (Sta. 0+00 to 1+64)

The existing NMPS was constructed in the 1960s and included the placement of fill to level the site. Based on existing information, up to 7 feet of medium dense fill, consisting of well-graded sand with silt, was placed across the site with the thickest fill along the east side of the existing driveway. In addition, the existing influent sewer was constructed using pipe-jacking methods and a construction shaft was located in the existing parking area just north of the pump station. The size and shape of the shaft is not known. We conducted a boring, designated as NME-7, near the existing influent manhole in the parking area and encountered about 36 feet of very loose to medium dense fill, which we presume is shaft backfill. The fill consisted of poorly graded sand with silt and scattered wood debris. Groundwater was encountered in the backfill at a depth of 8 feet bgs.

Based on the current borings conducted along the west side of the site near the new generator building, retaining walls, and temporary pump station, the subsurface soils consist of about 4 feet of fill over 5 to 8 feet of very soft to stiff clay. The fill generally consisted of medium stiff lean clay with sand and loose to medium dense silt with sand. Underlying the very soft to stiff clay is stiff to very stiff and hard clay to depths of greater than 50 feet bgs. Groundwater was encountered on the site at depths of about 7 to 17 feet bgs.

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Based on the borings along the dual force main alignment, the subsurface soils consist of about 7 feet of fill, consisting of soft clay with sand, over about 8 feet of medium stiff clay. Underlying the medium stiff clay is stiff to very stiff clay to a depth of about 22 feet bgs. Groundwater was encountered along the force main alignment at a depth of about 4 feet bgs.

4.3.2 NMPS to N Mercer Way (Sta. 1+64 to 29+25)

This portion of the alignment includes the dual force mains from the pump station along the right-of-way of SE 22nd Street, SE 22nd Place, 78th Avenue SE, SE 24th Street, and 81st Avenue SE. This portion of the alignment also includes two force main inspection and air/vacuum release structures (Sta. 18+50 and 27+63) and one force main drain vault (Sta. 22+30). The subsurface soils along this portion of the alignment consist of shallow fill over dense to very dense till and till-like deposits or very stiff to hard glaciolacustrine deposits. The fill is generally up to 11 feet thick and consists of medium dense, silty sand and sandy silt with gravel. Where till and till-like deposits are encountered (Sta. 7+00 to 21+00), they consist of sandy silt and silty sand and are up to 15 feet thick. Underlying these till and till-like deposits are very stiff to hard glaciolacustrine deposits consisting of silt, clayey silt, and silty clay. Groundwater was encountered along this section of the alignment at a depth of about 7 feet bgs.

4.3.3 N Mercer Way to 90th Place SE (Sta. 29+25 to 62+00)

This portion of the alignment includes the dual force mains along N Mercer Way and the I-90 Trail between 81st Avenue SE and 90th Place SE, one force main inspection and air/vacuum release structure (Sta. 48+70), and one force main drain vault (Sta. 42+70). The subsurface soils along this portion of the alignment consist of loose to very dense fill over medium stiff to hard glaciolacustrine deposits. The depth of fill varies from 0 to over 30 feet bgs and consists of silty sand with gravel and sandy gravel. The underlying glaciolacustrine deposits consist primarily of silty clay. Groundwater was encountered along this section of the alignment at depths of 5 to 22 feet bgs.

4.3.4 90th Place SE (Sta. 62+00 to 71+75)

This portion of the alignment includes the dual force mains along 90th Place SE and the NMPS force main discharge and odor control structures near the end of 90th Place SE. Along this portion of the alignment, the subsurface soils consist of medium dense to dense fill over dense to very dense till and till-like deposits. The depth of fill varies from 0 to over 15 feet bgs and consists of silty sand and silty sand with gravel. The underlying till and till-like deposits consist of sandy silt, silty sand, and silty sand with gravel. The till and till-like deposits contain

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seams and lenses of clean wet sand. Groundwater was encountered along this section of the alignment at depths of 6 to 8 feet bgs.

4.3.5 90th Place SE to SE 35th Street (Sta. 71+75 to 88+00)

This portion of the alignment includes the gravity sewer along the I-90 Trail from the end of 90th Place SE to SE 35th Street. Along this portion of the alignment, the subsurface soils consist of about 7 feet of loose to very dense fill over 15 to 20 feet of very soft to stiff and very loose to loose landslide debris. Underlying the landslide debris is about 5 feet of dense recessional outwash deposits over medium dense glaciolacustrine deposits consisting of silt and silt with sand. The fill consists of silty sand with gravel, the recessional outwash consists of sandy gravel, and the landslide debris is highly variable, ranging from silt and clay to silty sand. Based on existing information, it appears that the landslide deposits were reworked and placed as mass fill during I-90 construction. Groundwater was encountered along this section of the alignment at a depth of about 13 to 22 feet bgs.

4.3.6 SE 35th Street to E Mercer Way (Sta. 88+00 to 104+13)

This portion of the alignment includes the gravity sewer along the I-90 Trail from SE 35th Street to E Mercer Way and the East Channel siphon rock catcher, inlet, and odor control structures in the cul-de-sac at the end of SE 35th Place. From SE 35th Street to past 97th Avenue SE, the subsurface soils consist of about 7 to at least 25 feet of loose to very dense fill over medium dense to dense and stiff to very stiff glaciolacustrine deposits consisting of silt, sandy silt, and silty clay. The fill consists of silt, sandy silt, and silty sand with gravel. From near 97th Avenue SE to the East Channel siphon rock catcher, recessional deposits are present beneath the fill and above the glaciolacustrine deposits. The recessional deposits consist of soft to stiff, silty clay and sandy silt and loose to very dense sandy silt and silty gravel with sand. Groundwater was encountered along this section of the pipeline alignment at depths of 8 to 25 feet bgs.

At the East Channel siphon catcher, siphon inlet, and odor control structures, the fill is underlain by about 10 feet of soft to stiff and loose recessional deposits consisting of silty clay and silty sand. Very dense recessional deposits are present over the dense to very dense glaciolacustrine deposits. The glaciolacustrine deposits consist of silt and silt with sand. Groundwater was encountered at these structures at a depth of 15 feet bgs.

4.3.7 E Mercer Way to East Channel (Sta. 201+00 to 210+40)

This portion of the alignment includes the triple-barrel siphon along SE 35th Place from E Mercer Way to the west shore of the East Channel. Along this portion of the alignment, the

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subsurface soils consist of up to 10 feet of loose to medium dense fill over about 2 to 10 feet of recessional deposits. The recessional deposits consist of medium dense, sandy silt. Underlying the recessional deposits is dense to very dense glaciolacustrine deposits consisting of silt, sandy silt, and silt with sand. Groundwater was encountered along this section of the alignment at a depth of 17 to 21 feet bgs.

4.3.8 East Channel Crossing (Sta. 210+40 to 224+30)

This portion of the alignment includes the triple-barrel siphon beneath the East Channel. Along this portion of the alignment, the subsurface soils consist of loose to dense recessional deposits to depths of up to 15 feet below mudline. Underlying the recessional deposits is dense to very dense weathered and intact till and till-like deposits. The recessional deposits and till and till-like deposits consist of silty sand, silty sand with gravel, silty gravel with sand, and sandy gravel.

4.3.9 East Channel to Enatai Siphon Inlet (Sta. 224+30 to 226+92)

This portion of the alignment includes the triple-barrel siphon, East Channel siphon flow diversion structure, Enatai siphon inlet structure, odor control structure, and ancillary pipelines. Along this portion of the alignment, the subsurface soils consist of up to 10 feet of dense fill over about 10 feet of medium dense recessional deposits. The fill and recessional deposits consist of silty sand. Underlying the recessional deposits are very dense glacial outwash deposits consisting of silty sand with gravel. Groundwater was encountered along this section of the alignment at a depth of 10 feet bgs.

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4.3.10 Enatai Siphon Inlet to Siphon Outlet (Sta. 227+00 to 255+13)

This portion of the alignment includes the deep siphon beneath Enatai and the Enatai siphon outlet structure located near the SPS. From the Enatai inlet structure to about Sta. 232+00, the subsurface conditions consist of about 15 feet of medium dense to dense fill and recessional deposits. Underlying the recessional deposits are very dense glacial outwash deposits to a depth of over 100 feet bgs. The recessional deposits and glacial outwash deposits consist of silty sand and sandy gravel.

From Sta. 232+00 to 245+50, the Enatai hill is generally mantled by till and till-like deposits that are 10 to 25 feet thick. Underlying the till and till-like deposits are glacial outwash deposits and older sequences of till and till-like glacial outwash and glaciolacustrine deposits. The glacial outwash generally consists of very dense, silty sand and gravelly sand but may contain layers of predominately gravel in reaches. Boring NME-22, located near Sta. 234+25, indicates that multiple 3- to 7-foot-thick layers of poorly graded gravel exist within the glacial outwash deposits. From about Sta. 236+50 to 245+50, nonglacial lacustrine deposits with interbedded nonglacial fluvial deposits underlie the glacial outwash deposits. The nonglacial lacustrine deposits consist of very dense silt; sandy silt; and silty, fine sand, and the nonglacial fluvial deposits consist of very dense, silty sand and sandy silt. On the east side of the Enatai hill, Sta. 245+50 to the SPS, the till and till-like deposits dip to the east below the roadway fill for the I-90 on/off ramps (Bellevue Way SE). Underlying the till and till-like deposits are nonglacial lacustrine deposits with nonglacial fluvial interbeds. The roadway fill is up to 38 feet thick and consists of medium dense, silty sand with gravel to gravelly sand. Underlying the fill is very dense till and till-like deposits that are up to about 55 feet thick. Near the SPS, the subsurface conditions consist of 14 to 22 feet of loose to dense fill and ablation till over dense to very dense till and till-like deposits. Within the till and till-like deposits near the SPS, there are layers of very dense glacial outwash that range from 5 to 10 feet thick. One outwash layer located between about elevation 75 and 85 feet has confined or artesian groundwater levels to elevation 124.5 feet or about 0.5 foot above the ground surface at the SPS site. Groundwater elevations beneath the Enatai hill range from 120 to 125 feet.

4.3.11 Enatai Interceptor Connector Sewer

The Enatai Interceptor connector sewer runs through the Enatai Beach Park starting at the East Channel siphon flow diversion structure beneath the East Channel Bridge and ending at the existing Enatai Interceptor, located in the southeast corner of the park. The subsurface conditions consist of about 10 feet of dense to very dense fill over 7 feet of medium dense to dense recessional deposits. Underlying the recessional deposits is very dense till and till-like deposits and glacial outwash to a depth of over 25 feet bgs. The recessional, till and till-like, and glacial outwash deposits consist of silty sand, silty sand with gravel, and sandy gravel. The fill consists of silty sand with gravel, concrete, wood, and ballast debris. Groundwater is anticipated along the diversion sewer at about elevation 115 to 120 feet.

4.3.12 LS 11 Force Main and Valve Vault

The LS 11 force main runs along 97th Avenue SE between LS 11 and just north of the I-90 Trail. The force main consists of an existing section of the force main between LS 11 and SE 34th Street and a new section of force main between SE 34th Street and just north of the I-90 Trail. The existing section of force main will be rehabilitated using CIPP methods. The subsurface conditions along the new section of force main will vary with the south end near SE 34th Street consisting of about 5 feet of very soft recessional lacustrine silts over very stiff to hard glaciolacustrine clays and the north end near the I-90 Trail consisting of 7 feet of loose to medium dense fill over dense or very stiff glaciolacustrine silt and sandy silt. Groundwater is anticipated along the new section of force main at a depth of about 8 feet bgs.

The modifications to LS 11 include a new valve vault and a concrete slab for the electrical cabinets. The new valve vault for LS 11 will be an approximate 8-foot-wide by 14-foot-long precast concrete vault founded approximately 11 feet bgs. The electrical cabinet slab will be about 6 feet wide by 12 feet long. The subsurface conditions at the vault and slab locations consist of about 7 feet of fill over medium dense glaciolacustrine silts and sandy silts. Groundwater was measured at 2 feet bgs.

4.4 Geologic Hazards

Cobbles and boulders may be encountered in all soil units, particularly in the fill and glacial deposits along the Project alignment. Based on our experience, cobbles and boulders associated with glacial deposits are generally igneous or metamorphic rock with relatively high unconfined compressive strengths. Cobbles and boulders in the fill could be imported from other areas and likely have a wider range of unconfined compressive strengths.

Wood fragments were encountered in the fill and landslide deposits and should be expected to contain larger wood debris that could be difficult to penetrate and could cause other problems for excavation and shoring installation.

Published critical area maps from the City of Mercer Island indicate that there are landslide and seismic hazards along the Project alignment. Based on these maps, we have identified six sections of the alignment that are within published landslide and seismic hazard areas. For landslide hazards, these sections include Sta. 1+64 to 5+50, Sta. 71+75 to 73+11, Sta. 74+62 to 78+05, Sta. 81+23 to 82+35, Sta. 89+87 to 90+30, and Sta. 102+24 to 102+96. For seismic hazards, these sections include Sta. 1+64 to 5+50, Sta. 19+20 to 24+58, Sta. 25+60 to 29+40, Sta. 61+32 to 61+67, Sta. 88+80 to 90+30, and Sta. 97+08 to 99+37. In addition, we have identified one section of the LS 11 alignment that is within published landslide and seismic hazard areas. These sections include Sta. 3+40 to 4+05 for the landslide hazard and Sta. 3+40 to 6+14 for the seismic hazard.

The pipeline between Sta. 1+64 and 5+50 is located within both the landslide and seismic hazard areas. This section of pipeline will consist of dual force mains installed using conventional open-cut construction methods. The force mains will be installed within the right-of-way of SE 22nd Street to depths of 8 to 12 feet bgs. The existing grade of SE 22nd Street along this section ranges from relatively flat near Sta. 1+64 to about 11% near Sta. 5+50. The soils along this section of the alignment consist of about 7 feet of medium dense fill over stiff to very stiff clay.

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The pipeline will be founded primarily in stiff to very stiff clay, which are not susceptible to seismic hazards including liquefaction or lateral spreading. Based on the gradual grade of SE 22nd Street and that the pipeline will be installed perpendicular to the slope of the roadway, it is our opinion that the installation of the pipeline will not increase the risk of landslides within the roadway or the neighboring properties. In addition, there are no records of historic landslides along this section of roadway and we have not observed any signs of past landslides in this area.

The pipeline between Sta. 19+20 and 24+58 and between Sta. 25+60 and 29+40 is located within seismic hazard areas. These sections of pipeline will consist of dual force mains installed using conventional open-cut construction methods. The force mains will be installed within the right-of-way of SE 24th Street and 81st Avenue SE to depths of 5 to 12 feet bgs. The existing grades of SE 24th Street and 81st Avenue SE are relatively flat. The soils along SE 24th Street consist of about 5 feet of medium dense fill over about 7 feet of stiff silts and clays (recessional lacustrine deposits), and groundwater was not encountered. Along 81st Avenue SE, the soils consist of very dense or hard silts and clays (glaciolacustrine deposits), and groundwater was encountered at a depth of 35 feet bgs. In our opinion, the soils and groundwater along both these sections are not potentially liquefiable or susceptible to lateral spreading.

The pipeline between Sta. 61+32 to 61+67 is located within an identified seismic hazard area. This section of pipeline will consist of dual force mains installed using conventional open-cut construction methods. The force mains will be installed within the right-of-way of Shorewood Drive to a depth of about 10 feet bgs. The soils along this section consist of about 17 feet of medium dense to very dense fill over about 5 feet of loose recessional outwash with groundwater at a depth of about 17 feet bgs. In our opinion, these soils are potentially liquefiable but not susceptible to lateral spreading. Further discussion of liquefaction-induced settlement is presented in Section 5.5.2.

The pipeline between Sta. 71+75 to 73+11 is located within an identified landslide hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut construction methods. The gravity sewer will be installed to depths of 12 to 16 feet bgs within the right-of-way of 90th Place SE and a steep slope within the WSDOT right-of-way. The soils along this section consist of dense to very dense till and till-like deposits. In our opinion, these soils, due to their high shear strength, are not susceptible to landslide activity. However, due to the steepness of the grade in this area, trench dams should be considered to prevent groundwater migration and erosion of the backfill soils.

The pipeline between Sta. 74+62 to 78+05 is located within an identified landslide hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut

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construction methods. The gravity sewer will be installed to a depth of 8 feet bgs within the I-90 Trail. The soils along this section consist of about 8 feet of loose to very dense fill over loose landslide debris. Existing information from WSDOT indicates older landslides occurred to the south of I-90 and that the toe of these slides is located about 100 feet south of the Project alignment. In our opinion, it is likely that the landslide debris encountered in this section is old landslide debris used as fill to raise the roadway grade. The topography around the landslide has since been altered to a degree that would not suggest the same landslide susceptibility as was present at the time of the landslide. There is currently no evidence that landslides have occurred in this area post-I-90 construction. In addition, the gravity sewer will be constructed at the top of the existing slope through this section and the final installed pipeline will weigh the same or less than the current condition. Consequently, in our opinion, the construction of the pipeline through this section will not increase the risk of landslides.

The pipeline between Sta. 81+23 to 82+35 is located within an identified landslide hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut construction methods. The gravity sewer will be installed to a depth of 17 feet bgs within the I-90 Trail. The soils along this section consist of about 20 feet of loose to medium dense landslide debris over dense recessional outwash deposits and very stiff glaciolacustrine deposits. As discussed above, it is our opinion that the landslide debris encountered in this section is old landslide debris used as fill to raise the roadway grade and that the topography around the landslide has since been altered to a degree that would not suggest the same landslide susceptibility as was present at the time of the landslide.

The pipeline between Sta. 88+80 and 90+30 is located within a seismic hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut construction methods. The gravity sewer will be installed within the right-of-way of SE 35th Street to a depth of 25 feet bgs. The soils along SE 35th Street consist of dense to very dense fill, and groundwater is deeper than 25 feet. In our opinion, the soils and groundwater along this section are not potentially liquefiable or susceptible to lateral spreading.

The pipeline between Sta. 89+87 to 90+30 is located within an identified landslide hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut construction methods and will be installed to a depth of about 20 feet bgs within SE 35th Street. The soils along this section consist of dense to very dense fill with groundwater deeper than 25 feet bgs. The gravity sewer will be constructed at the top of the existing slope through this section, and the final installed pipeline will weigh the same or less than the current condition. Consequently, in our opinion, the construction of the pipeline through this section will not increase the risk of landsliding in this area.

The pipeline between Sta. 97+08 and 99+37 is located within a seismic hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut construction methods. The gravity sewer will be primarily installed on private property at 3425 97th Avenue SE to a depth of about 8 feet bgs. The soils along this section consist of 8 feet of loose to medium dense fill over very stiff to hard glaciolacustrine deposits with groundwater at about 8 feet bgs. In our opinion, the soils and groundwater along this section are not potentially liquefiable or susceptible to lateral spreading.

The pipeline between Sta. 102+24 to 102+96 is located within an identified landslide, steep slope hazard area. This section of pipeline will consist of a gravity sewer installed using conventional open-cut construction methods and will be installed to a depth of about 10 to 14 feet bgs within a steep slope on WSDOT right-of-way. The soils along this section consist of about 8 feet of loose to medium dense fill over about 14 feet of loose to medium dense recessional outwash with groundwater at a depth of 15 feet bgs. The gravity sewer will be constructed at the middle of the slope, and the final installed pipeline will weigh the same or less than the current condition. Consequently, in our opinion, the construction of the pipeline through this section will not increase the risk of landsliding in this area.

The LS 11 force main between Sta. 3+40 and 4+05 is located within the landslide hazard area. The force main will be installed within the right-of-way of 97th Avenue SE to depths of 7 to 10 feet bgs. The existing grade of 97th Avenue SE along this section is about 8%. The soils along this section of the alignment consist of about 5 feet of very soft recessional lacustrine silts over very stiff to hard glaciolacustrine clays. Based on the gradual grade of 97th Avenue SE and that the force main will be installed perpendicular to the slope of the roadway, it is our opinion that the installation of the pipeline will not increase the risk of landslides within the roadway or the neighboring properties. In addition, there are no records of historic landslides along this section of roadway and we have not observed any signs of past landslides in this area.

The LS 11 force main between Sta. 3+40 and 6+14 is located within the seismic hazard area. The force main will be installed within the right-of-way of 97th Avenue SE to depths of 5 to 10 feet bgs. As discussed above, the soils along this section of the alignment consist of about 5 feet of very soft recessional lacustrine silts over very stiff to hard glaciolacustrine clays with groundwater at a depth of 8 feet bgs. The pipeline will be founded primarily in very stiff to hard clay, which are not susceptible to seismic hazards including liquefaction or lateral spreading.

Note that for any site located on or near a slope, there are slope instability risks that present and future owners have to accept, including, but not limited to

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- Natural factors: soil and groundwater conditions, steep topography, heavy rainfall events, erosion, and vegetation conditions.
- Human-related factors: water leaks, pipe breaks, improper drainage, lack of maintenance of vegetation or drainage facilities, fill or debris placement, excavation, and/or removal of trees/vegetation.

Similar circumstances or other unknown conditions may also affect slope stability. Our evaluation and recommendations described herein are not a guarantee or warranty of future stability.

4.5 Soil Properties

For design purposes, soil engineering properties are presented in Table 1 for the geologic units encountered during our geotechnical investigations. The values in this table are based on relationships with laboratory test results and our experience with these soil units on similar projects.

TABLE 1
SOIL ENGINEERING PROPERTIES

Soil Unit	Total Unit Weight (pcf)	Drained Shear Strength	
		C' (tsf)	ϕ' (degrees)
Fill	120	0	28
Landslide	115	0	28
Lacustrine	110	0	20
Recessional Deposits	125	0	34
Till and Till-Like	130	0	40
Glacial Outwash/Nonglacial Fluvial	130	0	40
Glaciolacustrine	125	0.3	28
Nonglacial Lacustrine	125	0	40

Notes:
pcf = pounds per cubic foot
tsf = tons per square foot

5.0 ENGINEERING STUDIES AND CONSTRUCTION CONSIDERATIONS

The following sections present our engineering studies, engineering recommendations, and construction considerations for assisting the Tetra Tech design team during development of the 90% design of the Project. If the Tetra Tech design team develops additional information or revises the final alignment and facility configurations, the recommendations presented herein

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may need to be revised. Shannon & Wilson should be made aware of the revised or additional information so that we can evaluate our recommendations.

For purposes of our analyses and recommendations, it was necessary for us to assume that the results of the explorations and testing are representative of conditions along the Project alignment. However, as stated earlier in this GDM, subsurface conditions should be expected to vary. We may need to revise our recommendations if revisions are made to the alignment or facilities and if different conditions are encountered during construction.

5.1 Excavation

Based on the subsurface conditions and proposed depths of the land-based pipelines and appurtenant facilities and structures on Mercer Island and in Enatai, we anticipate that the excavations can be made using conventional excavating equipment such as rubber-tired backhoes or tracked hydraulic excavators. Most of the anticipated soils, including fill, landslide, lacustrine, recessional, glacial outwash, and glaciolacustrine deposits, are favorable soils for conventional excavation equipment. Where very dense till and till-like deposits are encountered, excavation may be difficult and impact breakers or hoe-rams may be necessary to break up these soils prior to excavation. Cobbles and boulders will be encountered in all soil types, particularly in the till and till-like deposits and near contacts between soil types. In addition, debris such as bricks, concrete, and wood are common in fill soils and wood debris may also be encountered in the landslide deposits.

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Temporary excavation slopes may be possible where there are sufficient working limits and the excavations are either above the groundwater table or the groundwater is adequately controlled. Consistent with conventional practice, temporary excavation slopes should be made the responsibility of the Contractor since the Contractor is able to observe full time the nature and conditions of the subsurface materials encountered, including groundwater, and has the responsibility for methods, sequence, and schedule of construction. All temporary excavation slopes should be accomplished in accordance with local, state, and federal safety regulations. For estimating purposes, temporary excavation slopes should be no steeper than 2 Horizontal to 1 Vertical (2H:1V) in the fill, landslide, lacustrine, and recessional deposits and 1.5H:1V for the till and till-like, glacial outwash, and glaciolacustrine deposits.

The excavations for the generator building, temporary pump station, retaining walls, and force mains at the NMPS will encounter medium stiff to hard glaciolacustrine deposits in the subgrade. In addition, portions of the pipeline on Mercer Island will also encounter lacustrine, glaciolacustrine, and landslide deposits in the pipeline subgrade. These soils are considered to be moisture-sensitive and easily disturbed and, therefore, the last 2 feet of the excavation should be made using an excavating bucket equipped with a smooth, flat steel plate over the digging teeth to reduce construction disturbance of the subgrade soil and, therefore, reduce post-construction settlements.

Based on the proposed depth of the siphons and the subsurface conditions along the East Channel crossing, we anticipate that the pipeline can be installed using barge-mounted excavators or dredging equipment. The anticipated soils, including lacustrine, recessional, and weathered till and till-like deposits, are favorable soils for an excavator or dredge. Cobbles and boulders will be common in these deposits and shallow logs and debris will likely be encountered along the crossing.

It is anticipated that the East Channel siphons will be installed in water using an open-cut trench with no shoring. The submerged temporary cut slopes will vary depending on the soils encountered. For planning purposes, we would assume that the loose to medium dense weathered till and till-like deposits and the medium dense to dense recessional deposits in the upper 5 feet of the open-cut trench will require 3H:1V temporary slopes to maintain stability.

Potentially contaminated soils were not encountered in the current borings conducted for this Project. However, based on a review of the Environmental Data Resources reports, there were two reported incidences of diesel spills on the NMPS site. Consequently, for planning and costing purposes, assume that diesel-contaminated soils will be encountered in all excavations along the existing driveway, parking area, and along the force main excavation beneath the creek. Also, assume that the soils encountered in the upper 10 feet of the temporary pump station excavation will be diesel-contaminated. We do not anticipate diesel-contaminated soils in excavations for the generator building and retaining walls to the west of the driveway and parking area. Excavated soils that are contaminated will require special handling and disposal in a Resource Conservation and Recovery Act Subtitle D facility.

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5.2 Groundwater Control

Based on the proposed pipeline and structure elevations and the groundwater conditions observed in the current and existing subsurface explorations, we anticipate that portions of the pipeline and some structures along the alignment will be constructed at or beneath the groundwater table or will be influenced by perched groundwater. Consequently, some form of groundwater control will be necessary to complete the work. The following discussion provides our assumptions regarding the likely groundwater control that could be selected by the Contractor. These assumptions are provided for the purpose of estimating the probable construction costs and are not intended for design. The responsibility for the selection, design, and performance of the groundwater control system should be the sole responsibility of the Contractor.

At the NMPS, the excavations for the generator building, odor control vessel, temporary pump station, retaining walls, and force mains will be conducted in loose to medium dense fill and medium stiff to hard glaciolacustrine silts and clays. Based on existing borings at the site, only perched groundwater is anticipated to be encountered in most of the fill and glaciolacustrine deposits, and in our opinion, groundwater control can likely be accomplished using sumps and pumps. The exception is near the backfilled pipe-jacking construction shaft used to install the existing influent sewer. As discussed previously, the shaft appears to have been backfilled with poorly graded sand with silt and scattered wood debris. Groundwater in the shaft backfill is about 8 feet bgs. Excavations near or within the backfilled shaft, particularly the excavations for the new force mains, may encounter "trapped" groundwater within the shaft. Consequently, well points or a deep well may be required to dewater and maintain the trapped groundwater below the base of the trench excavation.

For most of the pipeline alignment on Mercer Island, the groundwater levels vary from below to not more than 3 feet above the base of the trench excavation. Based on the pipeline depth and the geotechnical borings, up to 3 feet of water could be encountered in the trench excavations from Sta. 2+00 to 18+00 and from Sta. 65+00 to 72+00. In our opinion, the use of sumps and pumps is generally appropriate for controlling groundwater where it is not more than about 3 feet above the bottom of the excavation. Consequently, we anticipate that dewatering with properly constructed sumps and pumps will be sufficient to control groundwater in most of the pipeline excavations on Mercer Island. However, there are a few sections of pipeline, including the creek crossing near the NMPS (Sta. 0+75 to 2+00) and near the end of 97th Avenue SE (Sta. 96+50 to 101+50), where groundwater levels are more than 3 feet above the base of the trench excavation. At the creek crossing (Sta. 0+75 to 2+00), the groundwater is about 8 feet above the base of the excavation and sumps and pumps will likely not be sufficient. For this section, we recommend assuming closely spaced well points (10-foot centers) will be required to lower the groundwater level to below the bottom of the excavation. Near the end of 97th Avenue SE (Sta. 96+50 to 101+50), the groundwater is about 6 feet above the base of the excavation. For this section, we also recommend assuming closely spaced well points will be required to lower the groundwater to below the base of the excavation.

For the triple-siphon section in Enatai (Sta. 224+30 to 226+30), the groundwater level varies from 0 feet at the shoreline to 9.5 feet bgs. For the first 100 feet of the pipeline trench from the shoreline, the groundwater is 3 to 8 feet above the base of the excavation and sumps and pumps

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will not be sufficient to dewater the trench. For this section of pipeline trench, we recommend assuming that closely spaced well points will be required to dewater the trench. For the remainder of the section, the groundwater is anticipated to be less than 3 feet above the bottom of the trench and properly constructed sumps and pumps will likely be sufficient for groundwater control.

For the pipeline between the flow diversion structure and the Enatai siphon inlet structure, the groundwater level is 10 feet bgs or about 4 to 5 feet above the base of the excavation. Consequently, sumps and pumps will likely not be sufficient to dewater most of the trench and we recommend assuming that closely spaced well points will be required to dewater the trench.

For most of the Enatai Interceptor connector sewer in Enatai Beach Park, the groundwater level varies from below to not more than 3 feet above the base of the trench excavation and any perched or static groundwater encountered can be handled with properly constructed sumps and pumps. The exception is within about 60 feet of the connection to the Enatai Interceptor where groundwater levels are anticipated to be 6 feet or more above the base of the excavation and sumps and pumps will not be sufficient to control the groundwater. We recommend assuming closely spaced well points (10-foot centers) will be required in this area to lower the groundwater level to below the bottom of the excavation.

For the three-force main inspection and air/vacuum release structures and the two force main drain vaults, the measured groundwater levels are not more than 3 feet above the base of the structures. Consequently, in our opinion, any perched or static groundwater encountered in the excavations can be handled with properly constructed sumps and pumps.

For the NMPS force main discharge and odor control structures at the end of 90th Place SE, the measured groundwater level is about 8 feet bgs. Since the planned depth of each structure is about 12 feet bgs, the groundwater will be about 4 feet above the base of the excavation. However, based on boring NME-11, the excavations for the structures will be primarily in dense to very dense glacial till deposits. In general, these deposits have a very low permeability and groundwater is likely perched in sand lenses and layers within the deposits. The quantity of perched groundwater in these sand lenses and layers is typically small and they tend to drain off quickly. Consequently, in our opinion, sumps and pumps will be sufficient to control the groundwater during construction.

For the East Channel siphon rock catcher and associated maintenance holes, inlet, and odor control structures located in the SE 35th Place cul-de-sac, just west of E Mercer Way, the base of the structures are 12 to 15 feet deep and the groundwater level is at or below the bottom of the

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structures at a depth of 15 feet bgs. Although the groundwater is at or below the structures, boring NME-40 indicates that the structures are underlain by 4 to 5 feet of loose recessional outwash deposits. As discussed below in Sections 5.5.2 and 5.6.3, these loose soils are potentially liquefiable and are not considered to be suitable foundation soils for the structures. Consequently, we anticipate that approximately 4 to 5 feet of overexcavation and replacement with structural fill will be required beneath these structures, which will require closely spaced well points or deep sumps to lower the groundwater to a depth of 19 to 20 feet bgs.

For the East Channel siphon maintenance structure, located in the City of Mercer Island boat launch property, the base of the structure is about 10 feet deep and the groundwater level is about 21 feet bgs. Consequently, the groundwater is well below the structure, but perched water may be encountered, which can be handled using sumps and pumps.

For the East Channel siphon flow diversion and odor control structures in Enatai, the base of the structures is about 13 to 15 feet deep and the groundwater level is about 10 feet bgs. Consequently, the groundwater is about 3 to 5 feet above the base of the structures and well points or deep sumps will likely be required to lower and maintain the groundwater during construction.

For the Enatai siphon inlet structure, located just north of the East Channel bridge in Enatai, the base of the structure will be about 21 feet deep and the groundwater level is about 10 feet bgs. Consequently, the groundwater is about 11 feet above the base of the structure and well points or deep wells will be required to lower and maintain the groundwater during construction.

The Enatai siphon outlet structure, located just south of the SPS in Enatai, will be founded about 13 feet deep, and the groundwater level is about 6 feet bgs. Consequently, the groundwater is about 7 feet above the base of the structure and well points or deep wells will be required to lower and maintain the groundwater during construction.

For the LS 11 valve vault, located next to LS 11, the base of the structure will be about 11 feet deep, and the groundwater level is about 2 feet bgs. Consequently, the groundwater is about 9 feet above the base of the structure and well points will be required to lower and maintain the groundwater during construction. In addition, the existing force main near LS 11 and SE 34th Street will require a retrieval and/or launch pit for the CIPP rehabilitation. Based on force main depths and groundwater levels, the existing force main is about 1 to 2 feet below the groundwater level and sumps and pumps should be sufficient to lower and maintain groundwater during the CIPP work.

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5.2.1 Construction Dewatering Discharge Estimates

Our evaluation of dewatering discharge rates is based on estimated excavation dimensions and depths provided by the Tetra Tech design team, subsurface boring information, observation well readings, grain size analyses, and correlations between soil type and hydraulic conductivity. For our analysis, we used an empirical equation for groundwater inflow. These rates are provided for permitting purposes. The actual rates will depend on the Contractor's groundwater control and shoring designs as well as the sequencing of the work and time of year.

Portions of the force main (Sta. 2+00 to 18+00 and Sta. 65+00 to 72+00) will require groundwater to be lowered up to 3 feet within the trench. For our analysis, we assumed permeable shoring, sumps and pumps, and trench widths of about 8 and 5 feet for the force mains. We also assumed that not more than 50 feet of trench is open at one time. Based on these assumptions, we estimated discharge rates could range from 20 to 30 gallons per minute (gpm).

For the trench excavation across the creek at the NMPS (Sta. 0+75 to 2+00), the groundwater will have to be lowered about 8 feet. For our analysis, we assumed permeable shoring, closely spaced well points, and a trench width of 8 feet. We also assumed that not more than 50 feet of trench is open at one time. Based on these assumptions, we estimated a discharge rate of 60 gpm during the initial stages of dewatering, decreasing to about 40 gpm after about seven days of dewatering.

For the trench excavation near the end of 97th Avenue SE (Sta. 96+50 to 101+50), the groundwater will have to be lowered about 6 feet. For our analysis, we assumed permeable shoring, closely spaced well points, and a trench width of about 5 feet. We also assumed that not more than 50 feet of trench is open at one time. Based on these assumptions, we estimated discharge rates of 60 gpm during the initial stages of dewatering, decreasing to about 40 gpm after about seven days of dewatering.

For the triple-siphon section in Enatai (Sta. 224+30 to 226+30), the groundwater will have to be lowered by up to 8 feet. For our analysis, we assumed permeable shoring, closely spaced well points, and a trench width of 10 feet. We also assumed that not more than 50 feet of trench is open at one time. Based on these assumptions, we estimated discharge rates of 40 to 70 gpm during the initial stages of dewatering, decreasing to about 25 to 50 gpm after about seven days of dewatering.

For the pipeline between the flow diversion structure and the Enatai siphon inlet structure, the groundwater will have to be lowered by up to 5 feet. For our analysis, we assumed

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permeable shoring, closely spaced well points, and a trench width of 5 feet. We also assumed that not more than 50 feet of trench is open at one time. Based on these assumptions, we estimated discharge rates of 30 to 50 gpm during the initial stages of dewatering, decreasing to about 20 to 40 gpm after about seven days of dewatering.

When the diversion sewer in Enatai Beach Park is within about 60 feet of the connection to the Enatai Interceptor, the groundwater will have to be lowered by about 6 feet. For our analysis, we assumed permeable shoring, closely spaced well points, and a trench width of 5 feet. We also assumed that not more than 50 feet of trench is open at one time. Based on these assumptions, we estimated discharge rates of about 65 gpm during the initial stages of dewatering, decreasing to about 50 gpm after about seven days of dewatering.

For the force main inspection and air/vacuum release structures and the two force main drain vaults, perched groundwater will have to be controlled in the excavations. For our analysis, we assumed permeable shoring, sumps and pumps, and excavation dimensions of 16 feet by 16 feet for the force main inspection and air/vacuum release structures and 10 feet by 10 feet for the force main drain vaults. Based on these assumptions, we estimated discharge rates of 20 to 30 gpm.

For the NMPS force main discharge and odor control structures at the end of 90th Place SE, perched groundwater will have to be controlled in the excavations. For our analysis, we assumed permeable shoring, sumps and pumps, and excavation dimensions of 10 feet by 14 feet for the force main discharge and 19 feet by 26 feet for the odor control structure. Based on these assumptions, we estimated discharge rates of 20 to 30 gpm.

For the East Channel siphon rock catcher, inlet, and odor control structures, groundwater will have to be controlled in the excavations. For our analysis, we assumed permeable shoring, closely spaced well points or deep sumps, and excavation dimensions of 8 feet by 16 feet for the rock catcher structure, 16 feet by 19 feet for the siphon inlet structure, and 19 feet by 25 feet for the odor control structure. Based on these assumptions, we estimated discharge rates of 30 to 50 gpm.

For the East Channel maintenance structure, groundwater is well below the base of the excavation, but perched water may be encountered during construction. For our analysis, we assumed permeable shoring, sumps and pumps, and excavation dimensions of 11.5 feet by 18 feet. Based on these assumptions, we estimated discharge rates of 10 to 15 gpm.

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For the East Channel siphon flow diversion and odor control structures in Enatai, the groundwater will have to be lowered about 3 to 5 feet. For our analysis, we assumed permeable shoring, closely spaced well points or deep sumps, and excavation dimensions of 22 feet by 23 feet for the siphon outlet and flow diversion structure and 21 feet by 27 feet for the odor control structure. Based on these assumptions, we estimated discharge rates of 35 to 45 gpm during the initial stages of dewatering, decreasing to about 20 to 30 gpm after about 14 days of dewatering.

For the Enatai siphon inlet structure, the groundwater will have to be lowered about 11 feet. For our analysis, we assumed permeable shoring, closely spaced well points or deep wells, and excavation dimensions of 12 feet by 14 feet. Based on these assumptions, we estimated discharge rates of 100 gpm during the initial stages of dewatering, decreasing to about 75 gpm after about 14 days of dewatering.

For the Enatai siphon outlet structure, the groundwater will have to be lowered about 7 feet. For our analysis, we assumed permeable shoring, closely spaced well points or deep wells, and excavation dimensions of 12 feet by 14 feet. Based on these assumptions, we estimated discharge rates of 60 gpm during the initial stages of dewatering, decreasing to about 40 gpm after about 14 days of dewatering.

For the LS 11 valve vault, the groundwater will have to be lowered about 9 feet. For our analysis, we assumed permeable shoring, closely spaced well points, and excavation dimensions of 10 feet by 16 feet. Based on these assumptions, we estimated discharge rates of 60 gpm during the initial stages of dewatering, decreasing to about 40 gpm after about 14 days of dewatering. For the CIPP access pits along the existing LS 11 force main, our analysis assumed permeable shoring, sumps and pumps, and excavation dimensions of 4 feet by 8 feet. Based on these assumptions, we estimated discharge rates of 20 to 30 gpm.

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5.2.2 Dewatering-Induced Settlements

The magnitude of dewatering-induced settlements is dependent on the amount of groundwater drawdown and the elastic and consolidation parameters of the underlying soils. The settlements below assume that conditions adjacent to the Project are the same as those interpreted from the explorations completed for the Project and the existing exploration logs.

For the NMPS, the excavations for the generator building, temporary pump station, retaining walls, and force mains are anticipated to encounter primarily perched groundwater in the fill and glaciolacustrine deposits. Consequently, in our opinion, settlements due to dewatering are expected to be ⅛ inch or less.

For most of the pipeline alignment where dewatering will be required, groundwater drawdown during construction will be less than 3 feet and the underlying soils are medium dense fill and dense to very dense or very stiff to hard glacial soils. For these areas, dewatering-induced settlements are expected to be ⅛ inch or less. For the sections of pipeline where more than 3 feet of dewatering will be required, including the creek crossing near the NMPS (Sta. 0+75 to 2+00), near the end of 97th Avenue SE (Sta. 96+50 to 101+50), the first 100 feet from the shoreline in Enatai (Sta. 224+30 to 225+30), and the pipeline between the flow diversion structure and the Enatai siphon inlet structure (Sta. 226+52 to 226+88), the underlying soils are medium dense to very dense recessional outwash, very dense glacial outwash, or stiff to very stiff glaciolacustrine deposits and dewatering-induced settlements are anticipated to be ⅛ inch or less.

Dewatering for the force main discharge, East Channel siphon flow diversion, odor control, and Enatai siphon inlet structures will require 3 to 11 feet of groundwater drawdown. Since these structures are primarily in dense to very dense glacial till and outwash deposits, dewatering-induced settlements are anticipated to be nominal. As discussed above, the East Channel siphon rock catcher and associated maintenance holes, inlet, and odor control structures are underlain by 4 to 5 feet of loose recessional outwash deposits that will likely require overexcavation and replacement with structural fill. To overexcavate and replace these loose soils, groundwater will have to be lowered 4 to 5 feet, which will result in about 0.25 inch of dewatering-induced settlement near the excavations.

The excavation for the Enatai siphon outlet structure will be about 13 feet deep and will require about 7 feet of groundwater drawdown. The excavation will be primarily in loose to medium dense fill, alluvium, and recessional outwash deposits. For this excavation and based on our understanding of the subsurface conditions at the site, dewatering-induced settlements are expected to be about 1 inch at the excavation perimeter and about 0.5 inch at a distance of 100 feet from the excavation. The excavation is located near the SPS and an existing WSDOT retaining wall along the east side of the I-90 ramps (Bellevue Way). It is our understanding that the SPS facilities are on deep foundations and the WSDOT retaining wall is pile supported. Although the SPS and WSDOT retaining wall are on deep foundations, settlements of these magnitudes could adversely affect surface facilities and pavements at the SPS and could cause downdrag forces on the WSDOT retaining wall piles. In addition, peat is locally present in the area near the Enatai siphon outlet structure and if the Contractor's dewatering of the excavation results in lowering the groundwater level in the peat, excessive settlement could occur. Therefore, the Contractor should be responsible for designing a system of shoring and

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groundwater control that prevents the lowering of groundwater at adjacent property lines to less than 6 inches.

5.3 Temporary Shoring

The proposed pipeline and related structures will require excavation depths ranging from 5 to 35 feet bgs. Due to site constraints, excavation depths, soil and groundwater conditions, and the presence of existing structures and utilities near the pipeline alignment, temporary shoring will be required to support the soils and provide protection for the workers. It is our understanding that the design and the method of construction of the shoring will be the responsibility of the Contractor.

The type of temporary shoring used will depend on many factors, including depth of excavation, soil types, groundwater, adjacent structures and utilities, and site constraints. For temporary shored excavations, construction practice in the Puget Sound region generally includes trench boxes, slide-rail shoring systems, interlocking steel sheet piles, soldier piles and horizontal lagging, secant pile walls, and cutter soil-mix walls. Most of the soils along the Project alignment and at the proposed structure locations are dense to very dense or stiff to hard glacial soils and, therefore, are not considered favorable, unless predrilled, for the installation of sheet piles. Secant pile and cutter soil-mix walls are commonly used for deep excavations where watertight shoring is required. These walls are relatively stiff and expensive to install. Consequently, we anticipate the Contractor will likely select trench boxes, slide-rail shoring systems, or drilled soldier piles and horizontal lagging walls. Regardless of the method selected, the shoring system should provide adequate protection for workers and should prevent damage to adjacent structures, utilities, streets, and other facilities.

5.3.1 North Mercer Pump Station (NMPS)

Modifications to the NMPS will include several belowground facilities that will require shoring. These facilities include the temporary pump station, valve and meter vaults for the temporary pump station, odor control vessel, dual force mains, and a temporary force main.

The temporary pump station will be a 12-foot-diameter structure founded about 35 feet bgs and the appurtenant valve vault will be 8 feet wide by 10 feet long by 8 feet deep. The odor control vessel will be a 9-foot-wide by 9-foot-long buried structure founded approximately 8 feet bgs. The subsurface soils for the all three structures consist of about 5 feet of stiff or loose to medium dense fill over stiff to hard glaciolacustrine silts and clays. For the temporary pump station, we anticipate the excavation can be shored using drilled soldier piles and horizontal lagging walls and internal bracing. For the valve and meter vaults and odor control vessel, we

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anticipate that the excavations could be shored using a slide-rail or similar shoring system. The shoring for the temporary pump station, valve and meter vaults, and odor control vessel should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures are shown in Figure 7. There are three lateral earth pressures provided for the shoring including cantilevered walls, single-braced walls, and multiple braced walls. This figure also includes the lateral resistance from passive pressures for the embedded portion of the shoring. The shoring should have adequate toe penetrations to withstand the wall loading without causing undesirable lateral movement and disturbance to adjacent soils. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

For the dual force mains and temporary force main, trench excavations up to 15 feet deep will be required to install the pipe. The subsurface soils along the pipeline alignments generally consist of up to 7 feet of loose to medium dense fill over stiff to very stiff glaciolacustrine silts and clays. We anticipate that trench shoring could consist primarily of single or stacked trench boxes with sumps and pumps for groundwater control.

The generator building and retaining walls will require excavations to install the building footings and the cantilevered retaining walls. We anticipate excavation depths of up to 8 feet to install these elements. The subsurface soils for these excavations consist of medium stiff to very stiff glaciolacustrine silts and clays and we anticipate that the excavations can be made using temporary excavation slopes or cantilevered shoring walls. The temporary excavation slopes should be made the responsibility of the Contractor since the Contractor is able to observe full time the nature and conditions of the subsurface materials encountered, including groundwater, and has the responsibility for methods, sequence, and schedule of construction. The temporary excavation slopes should be accomplished in accordance with local, state, and federal safety regulations. For estimating purposes, temporary excavation slopes should be no steeper than 2H:1V. If cantilevered shoring walls are used, the walls should be designed for the lateral earth and surcharge pressures shown in Figures 7 and 10, respectively.

5.3.2 Pipeline Shoring

For most of the land-based pipeline alignment, excavation depths will range from 5 to 15 feet and we anticipate that trench shoring could consist primarily of single or stacked trench boxes with sumps and pumps or well points for groundwater control. For those sections of the pipeline alignment with excavation depths that exceed 15 feet, we anticipate that a combination of sloped open cut and trench boxes or slide-rail shoring systems with sumps and pumps or well points for groundwater control will be used.

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5.3.3 Force Main Inspection Structures and Drain Vaults

Three force main inspection and air/vacuum release structures and the two force main drain vaults will be installed along the force main alignment. The force main inspection and air/vacuum release structures will be about 12 feet in diameter and founded 9 to 13 feet bgs. The force main drain vaults will be about 6 feet in diameter and founded about 11 and 15 feet bgs. The subsurface soil conditions at the three force main inspection structures consist of either dense to very dense glacial till and till-like deposits or very stiff to hard glaciolacustrine deposits with groundwater levels up to 3 feet above the base of the excavations. The subsurface conditions at the two force main drain vaults consist of either stiff recessional lacustrine deposits or dense fill with groundwater levels below the base of the excavation. We anticipate that the excavations for these structures could be shored using slide-rail shoring systems in conjunction with sumps and pumps to control the groundwater.

The shoring for the force main inspection structures and drain vaults should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive pressures for the embedded portion of the shoring, are shown in Figure 7. There are two lateral earth pressures provided for the shoring, the first for single-braced walls and the second for multiple braced walls. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.4 90th Place SE Force Main Discharge and Odor Control Structures

The 90th Place SE force main discharge and odor control structures will be located in the cul-de-sac at the end of 90th Place SE. The force main discharge structure will be about 6 feet wide by 10 feet long and the odor control structure will be 17 feet wide by 24 feet long. Both structures will be founded 11 to 13.5 feet bgs. The subsurface soil conditions consist of dense to very dense glacial till deposits and the groundwater level is about 8 feet bgs or about 4 feet above the base of the excavations. As discussed above, these deposits have a very low permeability and groundwater is likely perched in sand lenses and layers within the deposits and can be controlled using sumps and pumps. Consequently, we anticipate that the excavations for these structures could be shored using slide-rail shoring systems or soldier pile and lagging walls with internal bracing in conjunction with sumps and pumps to control the groundwater.

The shoring for the force main discharge and odor control structures should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive

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pressures for the embedded portion of the shoring, are shown in Figure 7. There are two lateral earth pressures provided for the shoring, the first for single-braced walls and the second for multiple braced walls. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.5 East Channel Siphon Rock Catcher, Inlet, and Odor Control Structures

The East Channel siphon rock catcher, inlet, and odor control structures will be located in the SE 35th Place cul-de-sac, just west of E Mercer Way, on Mercer Island. The rock catcher structure will be 6 feet wide by 14 feet long and the siphon inlet structure will be 14 feet wide by 17 feet long. Both these structures will be founded 15 feet bgs. The odor control structure will be 17 feet wide by 23 feet long and will be founded about 13.5 feet bgs. The subsurface soils consist of about 10 feet of medium dense to dense and medium stiff fill over about 10 feet of soft to stiff and loose recessional deposits. Underlying the recessional deposits are very dense glacial outwash deposits and dense to very dense glaciolacustrine deposits. Groundwater was encountered at a depth of 15 feet bgs. As discussed above, the loose and soft recessional deposits between the depths of 15 and 20 feet are potentially liquefiable and are not considered to be suitable foundation soils for the structures. Consequently, we anticipate that approximately 4 to 5 feet of overexcavation and replacement with structural fill will be required beneath these structures. We anticipate the excavations for the three structures could be shored using drilled soldier piles and horizontal lagging walls and internal bracing or slide-rail shoring systems in conjunction with well points or deep sumps to control perched groundwater.

The shoring for the East Channel siphon rock catcher, inlet, and odor control structures should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive pressures for the embedded portion of the shoring, are shown in Figure 7. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.6 East Channel Siphon Maintenance Structure

The East Channel siphon maintenance structure will be located in the City of Mercer Island boat launch. The maintenance structure will be 9.5 feet wide by 16 feet long and will be founded 10 feet bgs. The subsurface soils consist of about 10 feet of medium dense fill over about 2 feet of medium dense recessional deposits. Underlying the recessional deposits are very dense glaciolacustrine deposits. Groundwater was encountered at a depth of 21 feet bgs. We anticipate the excavation for the maintenance structure could be shored using slide-rail shoring systems in conjunction with sumps and pumps to control perched groundwater.

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The shoring for the East Channel siphon maintenance structure should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for this structure, including the lateral resistance from passive pressures for the embedded portion of the shoring, are shown in Figure 7. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.7 East Channel Siphon Flow Diversion and Odor Control Structures

The East Channel siphon flow diversion and odor control structures will be located beneath the East Channel bridge in Enatai. The East Channel siphon flow diversion structure will be 20 feet wide by 21 feet long and the associated odor control structure will be 17 feet wide by 23 feet long. The two structures will be founded about 14 to 14.5 feet bgs. The subsurface soil conditions consist of up to 10 feet of dense fill over about 10 feet of medium dense recessional outwash deposits. Underlying the recessional outwash deposits is very dense glacial outwash deposits. Groundwater was encountered at a depth of 10 feet bgs or about 3 to 5 feet above the base of the excavations. We anticipate the excavations for these structures could be shored using drilled soldier piles and horizontal lagging walls and internal bracing or slide-rail shoring systems in conjunction with well points or deep sumps to control perched groundwater.

The shoring for the East Channel siphon flow diversion and odor control structures should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive pressures for the embedded portion of the shoring, are shown in Figure 7. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.8 Enatai Siphon Inlet Structure

The Enatai siphon inlet structure, located just north of the East Channel bridge in Enatai, will be 10 feet wide by 12 feet long and will be founded about 21 feet bgs. The subsurface soil conditions consist of up to 10 feet of dense fill over about 10 feet of medium dense recessional outwash deposits. Underlying the recessional outwash deposits is very dense glacial outwash deposits. Groundwater was encountered at a depth of 10 feet bgs or about 11 feet above the base of the excavation. We anticipate the excavation for this structure could be shored using drilled soldier piles and horizontal lagging walls and internal bracing or a slide-rail shoring system in conjunction with well points or deep sumps to control groundwater.

The shoring for the Enatai siphon inlet structure should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive pressures for the

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embedded portion of the shoring, are shown in Figure 7. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.9 Enatai Siphon Outlet Structure

The Enatai siphon outlet structure will be located just south of the SPS in Enatai and will consist of an 10-foot-wide by 12-foot-long structure that is founded about 14 feet bgs. The subsurface soil conditions consist of 12 feet of loose to medium dense fill over 2 feet of dense ablation till. Underlying the ablation till is very dense glacial till deposits. Groundwater was encountered at a depth of 6 feet bgs or about 7 feet above the base of the excavation. We anticipate the excavation for this structure could be shored using drilled soldier piles and horizontal lagging walls and internal bracing or a slide-rail shoring system in conjunction with well points to control groundwater. As discussed earlier, the glacial till deposits near the SPS contain layers of very dense glacial outwash that range from 5 to 10 feet thick. One outwash layer located between about elevation 75 and 85 feet has confined or artesian groundwater levels to elevation 124.5 feet or about 0.5 foot above the ground surface. To limit the potential for leakage from the confined aquifer, the penetration depths of soldier piles, if used, should be restricted so they do not extend below elevation 95 feet. In addition, all soldier piles should be backfilled with concrete or controlled density fill and left in place. Also, because of the potential presence of peat near the site, consideration could be given for requiring the use of watertight shoring or recharge wells to limit groundwater drawdown adjacent to the site. If sheet piles are used, pile penetration depths should also be limited to elevation 95 feet.

The shoring for the Enatai siphon outlet structure should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive pressures for the embedded portion of the shoring, are shown in Figure 7. If watertight shoring is used, additional earth pressure factors will need to be developed by the Contractor. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.3.10 LS 11 Valve Vault

The LS 11 valve vault will be located just south of the existing LS 11 on Mercer Island and will consist of a 6-foot-wide by 12-foot-long structure that is founded about 11 feet bgs. The subsurface soil conditions consist of medium dense glaciolacustrine deposits. Groundwater was encountered at a depth of 2 feet bgs or about 9 feet above the base of the excavation. We anticipate the excavation for this structure could be shored using a slide-rail shoring system in conjunction with well points to control groundwater.

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The shoring for the LS 11 valve vault should be designed for lateral earth and surcharge pressures. The recommended lateral earth pressures for the design of the temporary shoring for these structures, including the lateral resistance from passive pressures for the embedded portion of the shoring, are shown in Figure 7. The recommended lateral surcharge pressures for shoring design are provided in Figure 10.

5.4 Horizontal Directional Drilling (HDD)

The section of pipeline beneath the Enatai hill (Sta. 227+00 to 255+13) will be installed as a 2,813-foot-long deep siphon using HDD methods. We understand that the pipe will consist of a 32-inch-diameter, high-density polyethylene pipe. The proposed pipeline profile includes an entry angle of 26.91% and an exit angle of 8.75%. We understand that conductor casings will not be used for this project.

Based on the subsurface conditions shown in Figure 5, the HDD bore will encounter loose to medium dense fill and recessional deposits from the entry location (Sta. 255+53) to Sta. 255+00 and then primarily very dense glacial till deposits to about Sta. 254+00. From Sta. 254+00 to approximately Sta. 253+80, the bore will encounter very dense glacial outwash deposits with confined or artesian groundwater levels to elevation 124.5 feet or 0.5 foot above the existing ground surface. From Sta. 253+80 to 252+30, the bore will encounter very dense glacial till deposits and then very dense nonglacial lacustrine and nonglacial fluvial deposits to about Sta. 239+60. The bore will then encounter primarily very dense glacial outwash to Sta. 226+80 and then medium dense to dense recessional deposits and fill to the exit location at Sta. 225+92. Groundwater ranges from elevation 120 feet near the east and west flanks of the Enatai hill to elevation 125 feet beneath the hill or up to about 70 feet above the proposed pipe at its deepest point.

5.4.1 Ground Behavior

The HDD bore will encounter loose to dense fill in the first 50 feet of the exit/entry locations near the East Channel and SPS. Since the fill was placed by humans, it can be highly variable and may consist of any materials, including cobbles, boulders, wood, and debris. These obstructions may impact the performance of the HDD boring equipment and could require open excavations to remove obstructions from the alignment. The fill is not anticipated to squeeze during HDD operations, but the lower density and shallow nature of these deposits are expected to increase the susceptibility to hydraulic fracturing and release of slurry to the surrounding ground (inadvertent drilling fluid release).

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The majority of the HDD alignment will encounter very dense glacial outwash, nonglacial lacustrine, nonglacial fluvial, and till and till-like deposits. These soils provide generally favorable conditions for HDD construction because they are very dense and have relatively high shear strengths and low compressibility. However, these soil units may contain cobbles and boulders of varying diameters and concentrations. In addition, all of these deposits are saturated with hydrostatic heads of up to 70 feet above the proposed alignment and confined or artesian groundwater will be encountered in an outwash layer near the HDD entry location. These conditions will have to be controlled to prevent borehole washout. For the confined or artesian layer near the HDD entry location, drilling from the existing ground surface with drilling slurry should be sufficient to prevent borehole washout and leakage from the confined or artesian layer. If washout or leakage does occur, mitigation measures such as depressurization wells may be required in the layer during HDD drilling and pipe installation. Layers of gravel and cobbles exist along the bore path, particularly within the glacial outwash deposits, that can be difficult to stabilize with slurry and may require modifying the drilling fluid properties or pumping lost circulation materials to stabilize the bore. Boring NME-22, located near Sta. 234+20, encountered multiple 3- to 7-foot-thick layers of poorly graded gravel with sand near the design elevation of the pipe. Adjacent borings NME-4, NME-21, and NME-45 did not encounter these gravel layers at the design elevation of the pipe.

5.4.2 Hydraulic Fracturing and Fluid Release

Hydraulic fracturing, or "frac outs," occur when slurry pressure that is necessary to create the bore is in excess of the confining stresses in the ground, allowing slurry escape, either by creating soil fractures or by release of slurry into permeable ground (drilling fluid loss to the formation). For HDD construction, drilling fluid loss to the formation may be expected to occur through highly permeable soils, disturbed or loose soils such as fill, along contacts where there is significant variation in density, such as between fill and glacial soils, and in areas where depth of cover is less than 15 feet. Contractors can use a variety of methods, including vactor trucks, excavating pits, and/or constructing straw bale and sandbag dams, to control the released slurry. Proactively, they may install pressure relief wells to control frac outs.

There are two primary areas along the alignment where frac outs and inadvertent drilling fluid release may occur. These areas include both entry/exit locations where shallow cover or fill and recessional outwash deposits will be encountered.

It should be noted that inadvertent drilling fluid release may occur for reasons other than hydrofracture, such as fluid migration along desiccation cracks, roots, or seams. We recommend that the Contractor provide a contingency action plan for remediation should an event occur.

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The plan should include methods for identifying when an event has occurred, when and who needs to be notified, and what will be the immediate action by the Contractor to control the event. Long-term cleanup, if necessary, will depend on the nature of the event and can be decided at a later time based on site-specific situations.

5.4.3 Ground Settlement

In our opinion, overexcavation within the HDD bore will be difficult to measure but could potentially lead to settlement above the area of overexcavation. Local experience and tunneling case histories indicate that the amount of settlement and the width of the affected area along the alignment are dependent on the volume of overexcavation, the alignment depth, and the properties of the soil.

For the HDD alignment, most of the alignment is deep and within glacial deposits. Because of the depth and the existence of a relatively thick till-like deposit above most of the alignment, a large portion of ground losses, if they occur, will not reach the ground surface, and settlements are anticipated to be 1/8 inch or less. Near the HDD entry and exit locations, the HDD will pass beneath an existing WSDOT retaining wall (Wall 6) and adjacent to the northernmost footing of Pier 9 of the East Channel bridge. Additional settlement analyses were conducted for these structures and the results are discussed further in Section 5.9 of this report.

5.5 Seismic Design Considerations

The potential seismic hazards within the Project area include seismic-induced liquefaction and surcharge loading of buried structures. We anticipate that seismic design considerations will apply to the structures including the force main inspection structures; force main drain vaults; NMPS force main discharge and odor control structure; East Channel siphon rock catcher, inlet, odor control, and maintenance structures; East Channel siphon flow diversion and odor control structures; and Enatai siphon inlet and outlet structures. In addition, seismic design considerations will apply to the generator building, odor control vault, and retaining walls at the NMPS and the valve vault at LS 11. Recommendations for seismic surcharge loading for these structures is presented in Section 5.7 and a discussion of seismic-induced liquefaction for these structures is presented later in this section. We anticipate that all other Project elements, including the force mains, gravity sewer, siphons, and maintenance holes, will not incorporate seismic design considerations. We evaluated liquefaction potential along the pipeline alignment for the purposes of estimating settlement.

As discussed above in Section 4.2, the Project alignment is situated within the Seattle Fault Zone and is roughly parallel to an interpreted east-west-trending fault splay. Consequently, in addition

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to seismic-induced liquefaction and surcharge loading, there is also a risk of ground surface rupture from movement along fault splays. Mitigation of potential ground surface rupture and movement along the pipeline could include the use of flexible earthquake joints across the fault splays. However, this assumes that the locations of the fault splays are well defined along the alignment, which they are not. Consequently, since the locations of the fault splays are not known, it is our opinion that mitigation measures, such as the use of earthquake joints on pipes, is not warranted.

5.5.1 Seismic Design Criteria

We assume that the seismic design of the buried structures discussed above will be in accordance with the 2015 International Building Code (IBC). Computation of forces used for seismic design for this code is based on seismological input and site soil response factors. The site soil response factors are based on the determination of the site class. In our opinion, Site Class D can be used for design. Table 2 summarizes the spectral response values. Table 2 also presents site amplification factors for Site Class D. These factors should be applied to the Site Class B spectral response values.

TABLE 2
PARAMETERS FOR SEISMIC DESIGN OF STRUCTURES

Spectral Values[1]			Site Class D		
PGA (g)	**S_S (g)**	**S_1 (g)**	**F_a**	**F_v**	**PGA(g)[2]**
0.63	1.572	0.610	1.0	1.5	0.42

Notes:
[1] Spectral values for soft rock (Site Class B) site condition
[2] Design PGA value adjusted for soil factor for geotechnical design
Fv = soil factor at 1.0 second
PGA = peak ground acceleration
S_S = horizontal spectral acceleration at 0.2 second
S_1 = horizontal spectral acceleration at 1.0 second

5.5.2 Liquefaction Potential Analysis

Since there is no specified seismic design code for the sewer pipeline and buried structures, we elected to use seismic parameters that are representative of a 2,475-year return period ground motion. This is consistent with how seismic parameters are selected in the 2015 IBC. We performed our liquefaction analysis for an earthquake of magnitude 7.0 and peak ground acceleration of 0.42.

In general, most of the structures are not underlain by potentially liquefiable soils. The exceptions are the East Channel siphon rock catcher, inlet, and odor control structures located in

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the SE 35th Place cul-de-sac, just west of E Mercer Way on Mercer Island. These structures are underlain by 4 to 5 feet of loose recessional outwash deposits that are potentially liquefiable. Based on our analysis, liquefaction-induced settlements of 1 to 2 inches may occur beneath these structures. To mitigate the potential liquefaction and settlement of these structures, we recommend that the loose recessional deposits be overexcavated and replaced with structural fill. This will result in excavation depths of 19 to 20 feet bgs.

We understand that KC does not require seismic design of pipelines. However, we performed liquefaction potential analyses to estimate the likely range of liquefaction-induced settlement along the Project alignment. We performed this analysis because, as discussed in Section 4.4 of this report, a portion of the pipeline maybe located within potentially liquefiable soils. Our analyses indicated that liquefaction may occur in four areas along the pipeline alignment. The first area is near boring NME-10 or between Sta. 55+00 and 58+00 where the dual force mains are underlain by about 4 feet of loose fill, which is potentially liquefiable. Our analysis indicates that liquefaction-induced settlements of 0.5 to 1 inch may occur in this area. The second area is near boring NME-35 or between Sta. 60+00 and 62+00 where the dual force mains are underlain by about 8 feet of loose recessional outwash, which is potentially liquefiable. Our analysis indicates that liquefaction-induced settlements of 2 to 2.5 inches may occur in this area. The third area is near boring NME-13 or between Sta. 81+00 and 84+00, where the gravity sewer is underlain by very loose to loose landslide deposits, which are potentially liquefiable. Our analysis indicates that liquefaction-induced settlements of 1.5 to 3 inches may occur in this area. The fourth area is near boring NME-40 or between Sta. 101+50 and 104+13 and between Sta. 201+00 and 203+00, where the gravity sewer and triple siphon are underlain by loose recessional outwash, which is potentially liquefiable. Based on our analysis, liquefaction-induced settlements of 1 to 2 inches may occur in this area.

5.6 Foundation Support

The pipelines, maintenance holes, and related structures will be founded on a variety of soils. Except for the very loose to loose landslide deposits from about Sta. 72+50 to 84+00, most of the soils along the alignment are considered to be suitable foundation soils for the pipelines, maintenance holes, and structures.

The allowable bearing capacities provided below for all structures are for static loading conditions. For seismic loading, these allowable bearing capacities can be increased by 50%. For all structures, we recommend that the foundation soils be protected using a minimum 12 inches of crushed rock.

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5.6.1 Pipelines

Based on the depth of the pipelines and maintenance holes and the interpreted subsurface conditions, most of the pipelines and maintenance holes are anticipated to be founded on medium dense to dense fill, dense to very dense recessional deposits, very stiff to hard glaciolacustrine deposits, and medium dense to very dense weathered and intact till and till-like deposits. These soils are considered to be good to very good foundation soils for pipelines and maintenance holes.

The exception is between Sta. 72+50 and 84+00 where very loose to loose landslide deposits were encountered. These soils are not considered to be good foundation soils. However, due to the shallow depth of the gravity sewer through this area and the limited increase in loading, these soils are acceptable provided that the subgrade soils are improved by overexcavating the subgrade approximately 2 feet and replacing them with geosynthetic-wrapped backfill. After overexcavating the unsuitable soils, the geosynthetic filter fabric (Mirafi 500X or equivalent) should be placed across the bottom of the overexcavated trench and up the sidewalls of the shoring. The filter fabric should then be backfilled up to the design trench base elevation using clean quarry spalls meeting the gradational requirements specified in the KC Standard Specifications for fill material Type R. After the filter fabric is wrapped and overlapped over the backfill, the trench would be ready for bedding and pipe or maintenance hole placement.

5.6.2 North Mercer Pump Station (NMPS)

The foundation soils for the generator building, temporary pump station, odor control vessel, and the new retaining walls are anticipated to consist of medium stiff to very stiff glaciolacustrine deposits. These are considered to be suitable foundation soils.

The aboveground generator building will be constructed on the west side of the NMPS site, just west and north of the existing NMPS. We anticipate that the generator building will be constructed on shallow footings with a finished top of floor slab at about elevation 135 feet. Consequently, the east side of the generator building will be at the existing grade and the west side of the building will be partially buried into the existing 5H:1V slope on the west side of the Project site. This will result in the base of the footings being founded at an elevation of about 132 feet or about 3 feet bgs on the east side of the building and 8 to 10 feet bgs on the west side of the building. Based on borings NME-26 through NME-28, the building footings will be founded on very soft to stiff glaciolacustrine silts and clay. The very soft glaciolacustrine clays were encountered in boring NME-27, which is located near the southeast corner of the proposed building. The very soft glaciolacustrine clays were encountered between the depths of 4 and

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7 feet bgs or elevation 128 to 131 feet. These very soft glaciolacustrine clays are not suitable foundation soils and we recommend that they be overexcavated and replaced with compacted structural fill. We also recommend that during foundation excavation, a geotechnical engineer be on site to evaluate the foundation soils and to observe the overexcavation and replacement, particularly along the east side of the building. The other two borings, NME-26 and NME-28, indicated that the footings will be founded on medium stiff to stiff glaciolacustrine silts and clays. For shallow footings founded on medium stiff to stiff glaciolacustrine silts and clays, we recommend an allowable bearing capacity of 2,000 pounds per square foot (psf) with an estimated settlement of ½ inch. For footings founded on compacted structural fill, an allowable bearing capacity of 3,000 psf could be used, provided the structural fill extends at least 2B, where B is the footing width, below the base of the footing. If native clay is present within the zone of 2B, we would recommend using an allowable bearing capacity of 2,000 psf.

The retaining wall footings will be founded at about elevation 132 feet or 7 to 10 feet bgs. Based on borings NME-26 and NME-29, the wall footings will be founded on medium stiff to stiff glaciolacustrine silts and clays. For footings founded on medium stiff to stiff glaciolacustrine silts and clays, we recommend an allowable bearing capacity of 2,000 psf. The settlement is expected to be about ½ inch or less.

The odor control vessel will be founded about 8 feet bgs or about elevation 127 feet. Based on boring NME-28, the odor control vessel will be founded on stiff glaciolacustrine clays. For this structure, we recommend a net allowable bearing capacity of 5,000 psf.

The temporary pump station will be founded on hard glaciolacustrine clays. For deep structures founded on hard glaciolacustrine clays, we recommend a net allowable bearing capacity of 5,000 psf.

5.6.3 Force Main Inspection Structures and Drain Vaults

The foundation soils for the proposed force main inspection structures (Sta. 18+46, 27+63, and 48+60) consist of either dense to very dense glacial till and till-like deposits or very stiff to hard glaciolacustrine deposits. These deposits are considered to be good foundation soils. For all force main inspection structures, we recommend a net allowable bearing capacity of 4,000 psf. The settlement for all structures is expected to be about ½ inch or less.

The foundation soils for the proposed force main drain vaults (Sta. 22+30 and 42+71) consist of either stiff recessional lacustrine deposits or dense fill. These deposits are considered to be good foundation soils. For both force main drain vaults, we recommend a net allowable

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bearing capacity of 4,000 psf. The settlement for all structures is expected to be about ½ inch or less.

5.6.4 90th Place SE Force Main Discharge and Odor Control Structures

The foundation soils for the proposed force main discharge and odor control structure at the end of 90th Place SE are anticipated to consist of very dense weathered till and till-like deposits. These deposits are considered to be very good foundation soils. We recommend a net allowable bearing capacity of 4,500 psf for the structures. The settlement is expected to be about ½ inch or less.

5.6.5 East Channel Siphon Rock Catcher, Inlet, and Odor Control Structures

The foundation soils for the proposed East Channel siphon rock catcher structure and associated maintenance holes and the inlet structure are anticipated to consist of about 5 feet of loose recessional deposits over very dense glacial outwash and glaciolacustrine deposits. The loose recessional deposits are not considered to be good foundation soils. Consequently, as discussed above, we recommend that the loose recessional deposits between the depths of 15 and 20 feet be overexcavated and replaced with compacted structural fill. Provided that the loose recessional deposits underlying these structures are replaced with structural fill, we recommend a net allowable bearing capacity of 6,000 psf for these structures. The settlement is expected to be about ½ inch or less.

For the proposed odor control structure, the foundation soils are anticipated to consist of about 3 feet of soft lacustrine deposits over about 5 feet loose recessional outwash deposits. These deposits are not considered to be good foundation soils. Underlying these soft and loose deposits are very dense glacial outwash and glaciolacustrine deposits. As per the East Channel siphon rock catcher and inlet structures, we recommend that the soft lacustrine and loose recessional deposits between the depths of 12 and 20 feet be overexcavated and replaced with compacted structural fill. Provided that the soft and loose deposits are overexcavated and replaced with structural fill, we recommend a net allowable bearing capacity of 4,500 psf for this structure. The settlement is expected to be about ½ inch or less.

5.6.6 East Channel Siphon Maintenance Structure

The foundation soils for the proposed maintenance structure located in the City of Mercer Island boat launch property are anticipated to consist of medium dense recessional to very dense glaciolacustrine deposits. These deposits are considered to be good to very good foundation

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soils. We recommend a net allowable bearing capacity of 4,000 psf for the structure. The settlement is expected to be about ½ inch or less.

5.6.7 East Channel Siphon Flow Diversion and Odor Control Structures

The foundation soils for the proposed East Channel siphon flow diversion and odor control structures are anticipated to consist of medium dense to dense recessional outwash deposits over very dense glacial outwash deposits. These soils are considered to be good foundation soils. We recommend a net allowable bearing capacity of 3,500 psf for the East Channel siphon flow diversion structure. For the odor control structure, we recommend a net allowable bearing capacity of 2,500 psf. The settlement for all three structures is expected to be about ½ inch or less.

5.6.8 Enatai Siphon Inlet Structure

The foundation soils for the proposed Enatai siphon inlet structure are anticipated to consist of very dense glacial outwash deposits. These soils are considered to be very good foundation soils. We recommend a net allowable bearing capacity of 6,000 psf for this structure. The settlement is expected to be about ½ inch or less.

5.6.9 Enatai Siphon Outlet Structure

The foundation soils for the proposed Enatai siphon outlet structure are anticipated to consist of dense recessional deposits over very dense glacial till deposits. These soils are considered to be very good foundation soils. We recommend a net allowable bearing capacity of 5,500 psf for this structure. The settlement is expected to be about ½ inch or less.

5.6.10 LS 11 Valve Vault and Electrical Cabinet Slab

The modifications at LS 11 include a new valve vault and a slab on-grade for the electrical cabinets. The foundation soils for the proposed LS 11 valve vault are anticipated to consist of medium dense glaciolacustrine silt deposits. These soils are considered to be good foundation soils. We recommend a net allowable bearing capacity of 4,500 psf for this structure. The settlement is expected to be about ½ inch or less.

The foundation soils for the proposed slab for the electrical cabinets are anticipated to consist of granular fill over medium dense glaciolacustrine silt deposits. The density of the fill is not known, but based on observations during potholing, we recommend an allowable bearing capacity of 2,000 psf for the slab.

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5.7 Loads on Permanent Structures, Maintenance Holes, and Pipelines

5.7.1 Permanent Structures

All permanent buried cast-in-place structures for the Project should be designed for lateral earth, groundwater, seismic, and surcharge pressures. The total design pressure acting on the structures is the sum of these pressures. The permanent buried cast-in-place structures at the NMPS include the retaining walls, portions of the generator building, and the odor control vessel. Along the pipeline alignment, the permanent buried cast-in-place structures include the NMPS force main discharge and odor control structure on 90th Place SE; East Channel siphon rock catcher, inlet, maintenance, and odor control structures; East Channel siphon flow diversion and odor control structures; and the Enatai siphon inlet and outlet structures.

For the design of the retaining walls and the buried portion of the generator building at the NMPS, the recommended lateral earth and seismic pressures are presented in Figure 8. Along the base of the retaining walls and the building foundations, we recommend a friction coefficient of 0.25 for walls on the native glaciolacustrine silts and clays and 0.35 for walls on a minimum of 12 inches of crushed rock or structural fill. For the odor control vessel at the NMPS and all the permanent buried cast-in-place structures along the pipeline alignment, the recommended lateral earth, groundwater, and seismic pressures are presented in Figure 9. The recommended surcharge pressures for the retaining walls, the buried portions of the generator building, and the odor control vessel at the NMPS and all permanent buried cast-in-place structures along the pipeline alignment are presented in Figure 10.

5.7.2 Buried Precast Concrete Maintenance Holes, Vaults, and Structures

We understand that precast buried structures including, but not limited to, maintenance holes, force main inspection structures, air/vacuum release vaults, drain vaults, and the LS 11 valve vault will be installed along the pipe alignment. Unyielding precast concrete structures above the groundwater level should be designed to resist an at-rest lateral earth pressure using an equivalent fluid weight of 55 pounds per cubic foot (pcf). Unyielding precast concrete structures below the groundwater level should be designed to resist an at-rest lateral earth pressure using an equivalent fluid weight of 90 pcf. In our experience, unyielding, precast maintenance holes that extend both above and below the groundwater level are typically designed using an equivalent fluid weight of 90 pcf. The recommended at-rest lateral earth pressures assume that a well-compacted structural fill, meeting the gradational requirements specified in the KC Standard Specifications for fill material Type C, will be placed around the structures. Maintenance holes and structures should also be designed to withstand lateral loads imposed by equipment and other surcharges (e.g., materials) on the adjacent ground surface. Buried, rigid, precast concrete structures designed for at-rest lateral earth pressures and water pressure do not need to be designed for a seismic lateral soil pressure. The recommended surcharge pressures for maintenance holes are presented in Figure 10.

5.7.3 Pipelines

General recommendations regarding backfill and surcharge loading on buried pipes are presented in Figure 11. We anticipate that trenching would be used to install the proposed pipe; therefore, Case (b) for a conduit in a trench would likely apply. We recommend that the effect of backfill loads, as shown in Figure 11 from Case (b) and the H-20 live load shown in Case (c), be added (where appropriate) to obtain the total load on the pipe under vehicular traffic. We recommend using a unit weight for the structural backfill of 130 pcf.

5.8 Uplift Resistance

Watertight, permanent buried pipes, maintenance holes, and structures may be subjected to hydrostatic uplift pressures. Based on the available geotechnical data, the depth to groundwater varies from about 2 to more than 20 feet bgs along the land-based pipeline alignment. We recommend that all land-based pipes and structures be checked for uplift assuming a groundwater level at 5 feet bgs, except for the LS 11 valve vault, which should be checked for a groundwater level at the surface. For in-water pipelines, we recommend that uplift be checked assuming groundwater at the surface.

The recommended values for use in calculating uplift resistance for the land-based pipes, maintenance holes, and structures are presented in Figures 12 through 14. Figure 12 is included for buried maintenance holes and structures. It is presented in a general form so that it can be used for maintenance holes and structures with and without an extended base. Figures 13 and 14 are included for buried pipes with and without extended bases or pipe sleds, respectively.

5.9 Ground Movement and Settlement

Ground movements and settlement could result from dewatering (discussed earlier), lateral deformation of temporary shoring systems, and trenchless construction. The ground settlement estimates presented below should be reviewed relative to the proximity and condition of adjacent structures, improvements, utilities, pavements, and facilities. If the settlements appear to be excessive and could pose a risk of unacceptable damage, the Contractor would generally be required to alter their construction means and methods to limit ground movements. In all cases, a monitoring program should be established to evaluate performance during construction.

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Lateral deformations of the temporary shoring system during excavation will likely result in settlement behind the support systems. The magnitude of lateral deformation and the resulting settlement is a function of the soil and groundwater conditions, the stiffness of the temporary shoring system, and the means and methods selected by the Contractor. Based on work performed by Clough and O'Rourke (1990), the maximum anticipated settlement resulting from ground movements could range between about 0.15 and 0.5% of the height of the excavation, depending on the type of support. The typical model for the settlement trough behind temporary shoring is linear from the point of maximum settlement located immediately behind the shoring to less than ⅛ inch of settlement at a horizontal distance equal to one to 1.5 times the depth of the excavation. For pipeline excavations and shoring, the depth of excavation ranges from 5 to 25 feet. Based on an average of 0.3% of the excavation depth, settlements caused by shoring deformations are estimated to range from 0.2 to 1 inch immediately behind the shoring and decreasing linearly to ⅛ inch at 5 to 40 feet away from the temporary shoring walls.

For most of the Project alignment, there are no structures located within a horizontal distance equal to 1.5 times the depth of excavation. The alignment will parallel I-90 from about Sta. 29+00 to 61+00, Sta. 72+50 to 96+50, Sta. 100+00 to 104+13, and Sta. 201+00 to 210+50. Along these sections, there are conventional cantilevered retaining walls of various heights. It is our understanding that the trench locations are not within the WSDOT influence zone for the wall footings and, therefore, the trench excavations should not adversely affect the walls. The Project alignment also parallels the East Channel bridge footings in Piers 1 through 6 from about Sta. 208+00 to 220+00. The trench excavations along this section of the alignment are 2 to 11 feet deep and located a distance of 20 to 60 feet from the footings. Consequently, along this section of the alignment (208+00 to 220+00), the trench excavations should not adversely affect the bridge foundations. From about Sta. 220+00 to 225+70, the pipeline alignments trend to the south and the trench excavation gets within about 7 feet of the northernmost footing in Pier 7 and within 16 feet of the two northernmost footings in Pier 8. The trench excavations near the Pier 7 and 8 footings is about 6 to 8 feet deep and the footings are founded about 18 feet bgs or about 10 feet below the bottom of the pipeline trench. Consequently, the trench excavations near Piers 7 and 8 should not adversely affect the bridge foundations.

There are two residences, one between Sta. 97+50 and 98+25 (3425 97th Avenue SE) and the other between 98+10 and 98+70 (3421 97th Avenue SE), that are located within 6 to 8 feet of the pipeline trench excavation. At these residences, the excavation depth is about 7 feet deep. If settlement extends out to 1.5 times the depth of the excavation, the residences could settle about ⅛ inch or more, which could cause some cracking, but not structural damage.

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Although there are few structures located within a horizontal distance equal to 1.5 times the depth of excavation, there are many potentially sensitive utilities within that distance. The potentially sensitive utilities include storm drains, sewers, and water mains that will likely require settlement monitoring during construction. Instrumentation recommendations are included below in Section 6.0.

As discussed earlier, most of the HDD alignment is deep and within glacial deposits. Because of the pipeline depth and the existence of a relatively thick till-like deposit above the alignment, a large portion of ground losses, if they occur, will not reach the ground surface, and settlements are anticipated to be negligible. The HDD alignment crosses beneath the existing WSDOT retaining wall (Wall W-6) near the SPS (Sta. 253+66). The existing wall is supported by three rows of 14-inch-diameter augercast piles with two rows of piles battered at 1H:3V (4 inches in 12 inches) and one row of vertical piles. We understand that the piles were typically installed approximately 2 feet into the glacial till deposits (pile tip elevation of 99 feet) and were designed as 40-ton piles. The crown of the HDD bore will be drilled approximately 19 feet below the tips of the piles primarily though glacial till and outwash deposits. As discussed earlier, the outwash deposits within the glacial till has confined or artesian ground levels at elevation 124.5 feet or 0.5 foot above the ground surface. To estimate settlement of the Wall W-6 piles due to the drilling of the HDD bore beneath the wall, a soil-structure interaction analysis was performed. The analysis consisted of finite difference modeling to predict the performance of the piles under static loading conditions. The results of the analysis indicated that most of the ground settlement of 0.5 to 2.5 inches occurs within about 1 foot or less of the bore crown and that ground settlement at the pile tips was only 0.02 inch, which should not adversely affect the performance of the wall. Details of the analysis and the results are presented in a technical memorandum dated August 8, 2019 (Shannon & Wilson, 2019b). For reference, this technical memorandum is included in Appendix A.

In addition to the undercrossing of Wall W-6, the HDD alignment parallels the East Channel bridge between Sta. 227+00 and 231+00. Near the northernmost footing of Pier 9, the HDD is about 30 feet north and 25 feet below the bottom of the footing, which puts the HDD bore just outside the influence zone of the footing. However, due to steering and guidance system tolerances for the HDD, it is possible that the HDD bore could be near or slightly within the influence zone of the footing. During the drilling of the HDD bore, ground losses could occur resulting in a zone of loosened ground above the bore. This loosened zone could result in additional settlement and a reduction in bearing capacity of the Pier 9 footing. A settlement influence zone above the HDD bore was developed by Staheli Trenchless, Inc. and was used in conjunction with finite-element and limited equilibrium analyses to estimate potential settlement

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and decrease in bearing capacity for the Pier 9 footing. The analyses indicated that the estimated settlements beneath the footing are very small and not uniform due to the proximity of the north side of the footing to the bore. Consequently, the model predicted larger settlement on the north side of the footing than the south side. For the proposed HDD alignment, including bore path tolerances, the analyses indicated very small settlements of 0.02 to 0.05 inch as a result of the influence zone above the HDD bore, with only 0.01 to 0.04 inch of differential settlement occurring between the north and south sides of the footing. Settlements of these magnitudes are very small, not measurable, and should not negatively impact the Pier 9 footing. The analyses also indicated that the existing Pier 9 footing has a factor of safety (FS) of 3.61 in bearing capacity and by introducing the influence zone over the HDD bore, the FS dropped to minimum of 3.29. Since the FS remained well above 3 in bearing capacity, the influence zone above the HDD bore did not appear to have any significant effect on the bearing capacity of the footing. Details of the analyses and the results are presented in a memorandum dated September 28, 2018 (Tetra Tech, 2018). For reference, this technical memorandum is included in Appendix B.

5.10 Backfill Placement and Compaction

Although portions of the excavated material along the alignment may be suitable for reuse as backfill, for planning purposes, we recommend that imported fill be used to backfill the excavations. This is primarily due to the presence of relatively high fines content of some of the fill and native soils and the difficulty in segregating, transporting, and storing the excavated soils.

5.10.1 Pipe Bedding

We recommend that the land-based pipe bedding consist of imported granular bedding material meeting the following gradational requirements as specified in the WSDOT and American Public Works Association (APWA) Standard Specifications (2020):

- WSDOT right-of-way: 9-03.12(3), Gravel Backfill for Pipe Zone Bedding.
- City of Mercer Island right-of-way: 9-03.9(3), Crushed Surfacing Top Course.
- City of Bellevue right-of-way: 9-03.9(3), Crushed Surfacing Top Course.

The bedding should extend a minimum of 6 inches below the bottom of the pipe and up to 12 inches above the top of the pipe.

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5.10.2 Subsequent Backfill

We recommend that the land-based trench backfill, above the pipe bedding materials, meet the following gradational requirements as specified in the WSDOT and APWA Standard Specifications (2020):

- WSDOT right-of-way: 9-03.19, Bank Run Gravel for Trench Backfill.
- City of Mercer Island right-of-way: 9-03.9(3), Crushed Surfacing Top Course.
- City of Bellevue right-of-way: 9-03.14(1), Gravel Borrow.

The U.S. Army Corps of Engineers and DNR may require different material for use within the East Channel. In addition, for the East Channel crossing, surface backfill should consist of habitat-friendly fill, as required by the permitting agencies.

5.10.3 Trench Foundation Backfill

We recommend that the trench foundation backfill, to replace unsuitable overexcavated materials, meet the gradational requirements specified in 9-03.17, Foundation Material Class A of the WSDOT and APWA Standard Specifications (2020).

5.10.4 Structural Fill

We recommend that backfill materials for permanent structures should meet the gradational requirements specified in 9-03.10, Aggregate for Gravel Base of the WSDOT and APWA Standard Specifications (2020), except that the percent passing the No. 200 sieve should not exceed 5%.

5.10.5 Retaining Wall Backfill

We recommend that the retaining walls at the NMPS be backfilled using an imported, free-draining granular material meeting the gradational requirements specified in 9-03.12(2), Gravel Backfill for Walls of the WSDOT and APWA Standard Specifications (2020).

5.10.6 Compaction

The on-land pipe bedding and subsequent backfill should be placed in a maximum loose backfill lift thickness of 6 inches. The pipe bedding backfill should be carefully worked under the pipe by means of slicing with a shovel, vibration, tamping, or other approved method. Heavy mechanical compaction equipment should not be allowed within 2 feet of the pipes. The pipe bedding and subsequent backfill should be placed in uniform lifts and compacted to a dense and

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unyielding condition and to 95% of its Modified Proctor maximum dry density (ASTM Designation D1557, Method C or D).

Imported structural fill should be at a moisture content near optimum (±2%) to allow proper compaction. We recommend that the material be compacted to a dense, unyielding condition. To avoid overstressing, heavy compaction equipment should not be used in the immediate vicinity of structural walls. For compaction within 3 feet of walls, smaller, vibrating-plate compactors should be used. We recommend a maximum loose backfill lift thickness of 9 inches for heavy compaction equipment or 6 inches for hand-operated equipment. If a backhoe-mounted plate compactor is used, the maximum loose lift thickness could be increased to 18 inches. Whatever equipment and lift thicknesses are used, all soil within the lift should be compacted to the applicable KC Standard Specifications. We recommend that the above limitations on compaction equipment use be incorporated into the Project specifications. All compacted surfaces should be sloped to drain to prevent ponding.

5.11 Wet Weather Considerations

In the Project area, wet weather generally begins about mid-October and continues through May. While the Contractor should be responsible for selecting the equipment and methods necessary to complete the work in accordance with the specifications, in our experience, the following procedures are required if wet weather earthwork is unavoidable:

- The ground surface in the construction area should be sloped to promote the rapid runoff of precipitation away from work areas and to prevent ponding of water.
- Covering work areas or slopes with plastic, sloping, ditching, using sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. Excavation, or the removal of unsuitable soil, should be followed immediately by the placement of concrete or compaction of a suitable thickness (generally 12 inches or more) of clean structural fill. The size and type of construction equipment and its mode of mobility (wheels or track) should be selected to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, Gradall, or equivalent, located so that the equipment does not traffic over the excavated area; thus, subgrade disturbance caused by equipment traffic will be reduced.
- Uncompacted soil should not be left exposed to moisture. Where vibration settlement-sensitive facilities are not located within 10 feet, a smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.

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- In-place soils or fill soils that are, or become, wet and unstable and/or are too wet to suitably compact should be removed and replaced with clean granular soil.
- Excavation and placement of structural fill material should be observed on a full-time basis by an engineer or engineer's representative experienced in earthwork, to determine that all work is being accomplished in accordance with the intent of the specifications.

6.0 GEOTECHNICAL INSTRUMENTATION

6.1 Pre-Construction Survey

For planning purposes, pre-construction surveys to document the existing condition of residences above the HDD pipeline beneath the Enatai hill should be considered prior to the start of construction. Although damage to the homes is not expected, these pre-construction surveys could assist with settling disputes regarding construction impacts. In general, pre-construction surveys should be conducted on each house in which a subterranean easement is required.

The pre-construction survey should include diagrams, sketches, and photographs. These records should include, but not be limited to, the number of cracks, locations of cracks, length and width of existing cracks, etc. The surveys should be conducted by a Professional Engineer registered in the State of Washington and should be completed with the Owner, Contractor, and KC present. A formal report should then be developed and signed by each member of the group.

6.2 Geotechnical Instrumentation

Geotechnical instrumentation should be installed to monitor the response of the ground and adjacent structures, utilities, and pavement to the construction of the pipeline, maintenance holes, and appurtenant structures. Data collected from the monitoring program would be used to assess

- The validity of any claims.
- Effectiveness of remedial measures.
- Performance of the shoring.
- Performance of the dewatering system.

The construction of the Project will require relatively deep shored trenches and excavations for maintenance holes and structures, dewatering, and trenchless construction. Each of these construction activities could result in deformations or ground losses that may lead to vertical settlements adjacent to excavations, which may affect adjacent structures, utilities, and pavements. Each of these and other related elements should be monitored prior to construction and during construction, as required. For 90% design, we recommend assuming the following geotechnical instrumentation systems:

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- Surface settlement points for monitoring curbs, sidewalks, and roadways that will remain after construction that are within a distance equal to 1.5 times the excavation depth or within an area that could be influenced by construction vibrations. At the NMPS, surface settlement points should be established along the driveway on the west side and the street on the north side of the Project site.
- Utility settlement points should be established on settlement-sensitive utilities such as sewers, storm drains, and water mains that cross above and/or parallel the pipe excavations and are within a distance equal to 1.5 times the excavation depth or within an area that could be influenced by construction vibrations. As an alternative to utility settlement points, video surveys could be conducted on sewers and storm drains prior to and after construction to evaluate settlement or damage.
- Structure settlement points should be established on all residences where pre-construction surveys are conducted and structures within a distance equal to 1.5 times the excavation depth. We anticipate that portions of the WSDOT retaining walls along the alignment will require structure settlement points. For these walls, we recommend at least two structure points be installed on each wall panel, one near each end of the panel. In addition, we recommend that structure settlement points be installed on the WSDOT wall near the SPS where the HDD will be installed beneath the wall. On this wall we would recommend at least two structure points be installed on two wall panels, one on each side of the undercrossing. We also recommend structural settlement points on the SPS and that two structure settlement points be established on all East Channel bridge columns that are within a distance equal to 1.5 times the excavation depth.
- Slope monitoring points should be established on slopes above the generator building and retaining walls at the NMPS and along portions of the pipeline alignment where existing slopes are above the excavations. This will include the south-facing slope beneath 90th Place SE (Sta. 72+00 to 74+00) and the north-facing slope between 97th Avenue SE and E Mercer Way (Sta. 101+00 to 103+00).

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The proposed instrument locations and details will be included on the 90% design drawings and the installation and monitoring requirements will be included in the specifications.

6.3 Groundwater Monitoring

Groundwater drawdown and pressure reduction associated with the construction and construction dewatering can influence areas beyond the Project boundaries. Groundwater lowering and pressure reduction can result in ground settlement, which could impact nearby property, structures, utilities, and other improvements.

The Contractor should use dewatering, groundwater recharge, shoring, and excavation methods that limit groundwater drawdown and ground movement outside the excavations and protect nearby properties, structures, utilities, and existing improvements from settlement and movement that could result from the work.

Prior to commencing construction activities for the Project, we recommend the construction Contractor and/or Project Owner perform and document a detailed visual survey of the interior and exterior of all nearby structures within a 100-foot radius of the planned dewatering activities. This visual pre-condition survey should include high-resolution digital photographs of all exposed surfaces, areas, and observed preexisting damage.

We recommend the construction Contractor and/or Project Owner install and monitor groundwater monitoring wells around the perimeter of the Project site boundaries and stop dewatering if drawdown of the water elevations is detected outside the Project site boundaries, which should include, at a minimum, daily monitoring from immediately prior to the start of any dewatering activities until after all dewatering activities have stopped.

The Contractor should engage a Professional Engineer to evaluate potential impacts associated with groundwater drawdown and pressure reduction and to design mitigation measures to prevent damage to nearby properties, structures, utilities, and other facilities. The Contractor should implement those mitigation measures prior to operating dewatering systems.

The Contractor should have sole responsibility for proper design, furnishing, installation, operation, maintenance, and failure of the dewatering system or any component of the dewatering systems.

The Contractor should install and operate the dewatering system so that the groundwater elevation and piezometric pressure outside the excavation is not drawn down to an extent that would damage or endanger adjacent property, structures, utilities, or other improvements. The Contractor should be responsible for abandoning the wells and restoration after construction in conformance with Chapter 173-160 of the Washington Administrative Code, Minimum Standards for Construction and Maintenance of Wells.

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7.0 CLOSURE

The recommendations and conclusions in this GDM are based on:

- The limitations of our approved scope, schedule, and budget as described in Amendment 8 to our agreement with Tetra Tech (Tetra Tech Job No. 200-12539-18001) and King County Contract No. E00306E13.
- Our understanding of the Project and information provided by the Tetra Tech design team.
- Subsurface conditions we observed in the borings as they existed during drilling.
- The results of testing performed in the explorations and on samples we collected from the explorations.
- A subsurface exploration plan developed with the Tetra Tech design team to consider Project-specific factors, risk tolerance, schedule, and budget.
- Assumed construction methods for the pipeline.

We have prepared an Appendix C, "Important Information About Your Geotechnical/ Environmental Report," to assist you and others in understanding the use and limitations of this GDM. Please read this document to learn how you can lower your risks for this Project.

SHANNON & WILSON, INC.

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Michael S. Kucker, PE
Vice President

JAA:MSK/jaa

8.0 REFERENCES

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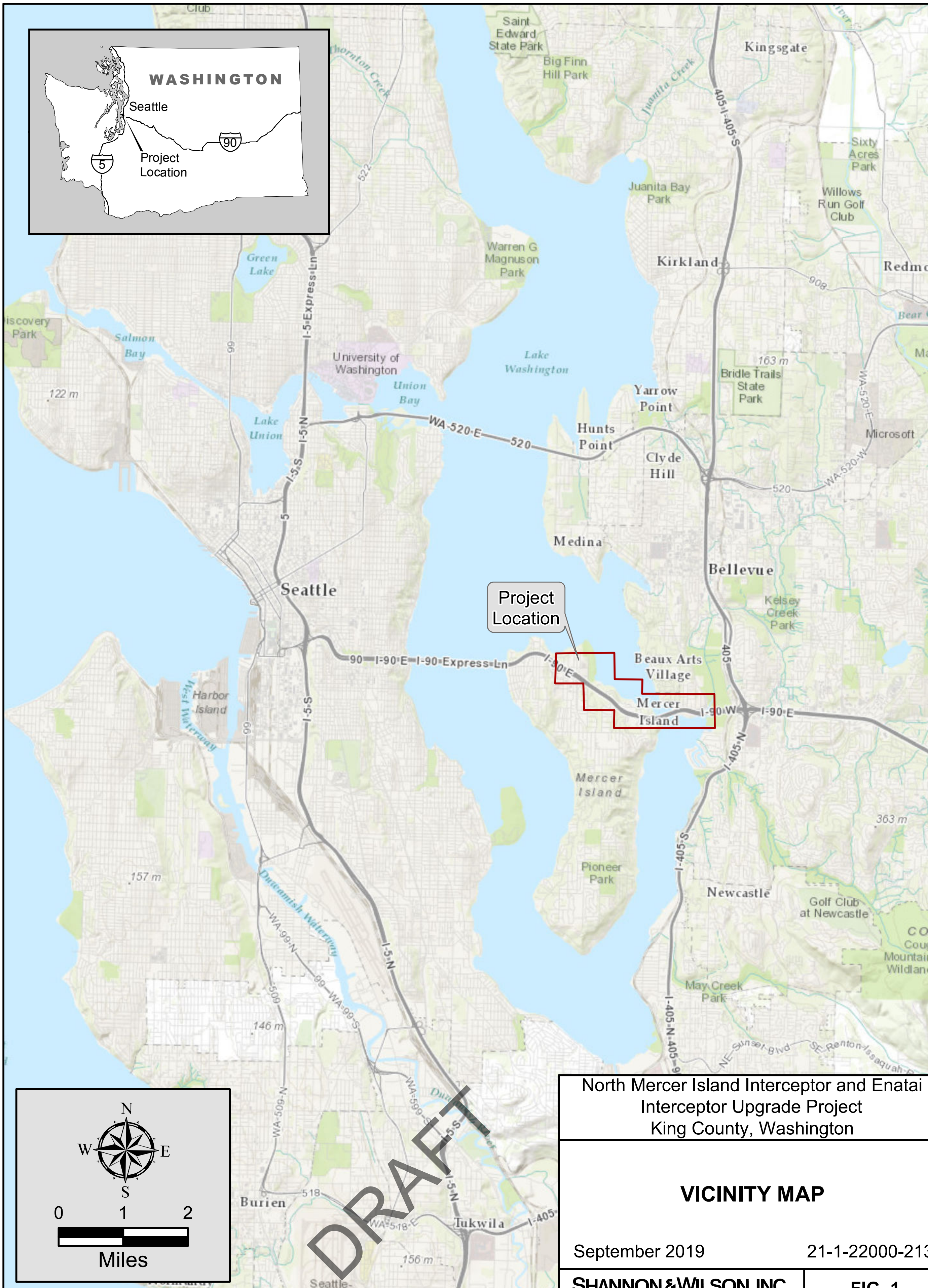

North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

VICINITY MAP

September 2019 | 21-1-22000-213

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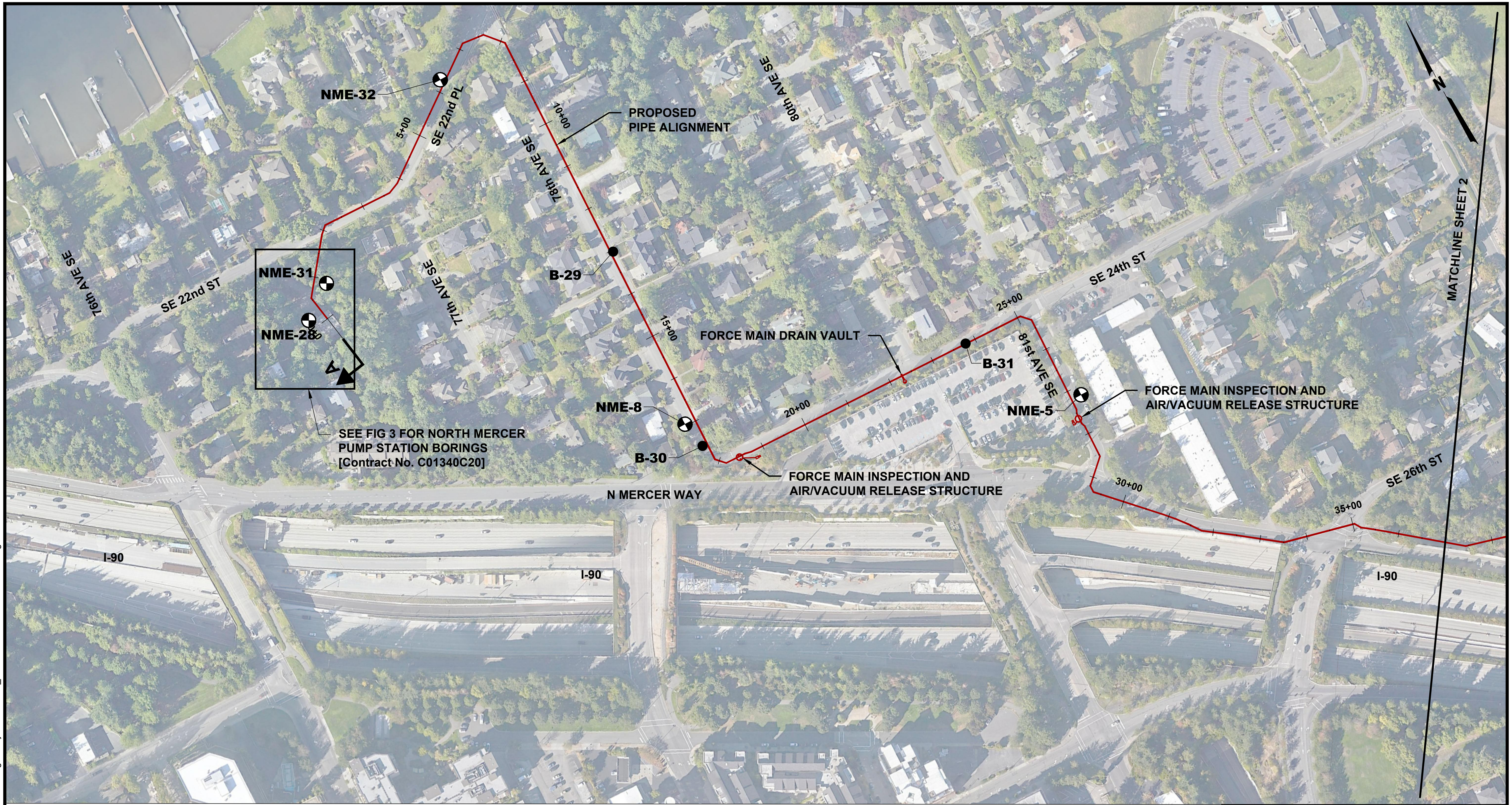
FIG. 1



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LEGEND

NME-1 Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1, TP-1 Existing Exploration Designation and Approximate Location (Completed by Others)

0 200 400

Scale in Feet

NOTE

Figure adapted from JACOBS' file *2019-06-27_Overall.dwg and Tetra Tech's files 2019-08-15_LS11_96th Ave Siphon.dwg* and *E00306E13-PIPE-ALGN-EIR_EAST.dwg dated 2019-08-15.*

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN
CONVEYANCE
CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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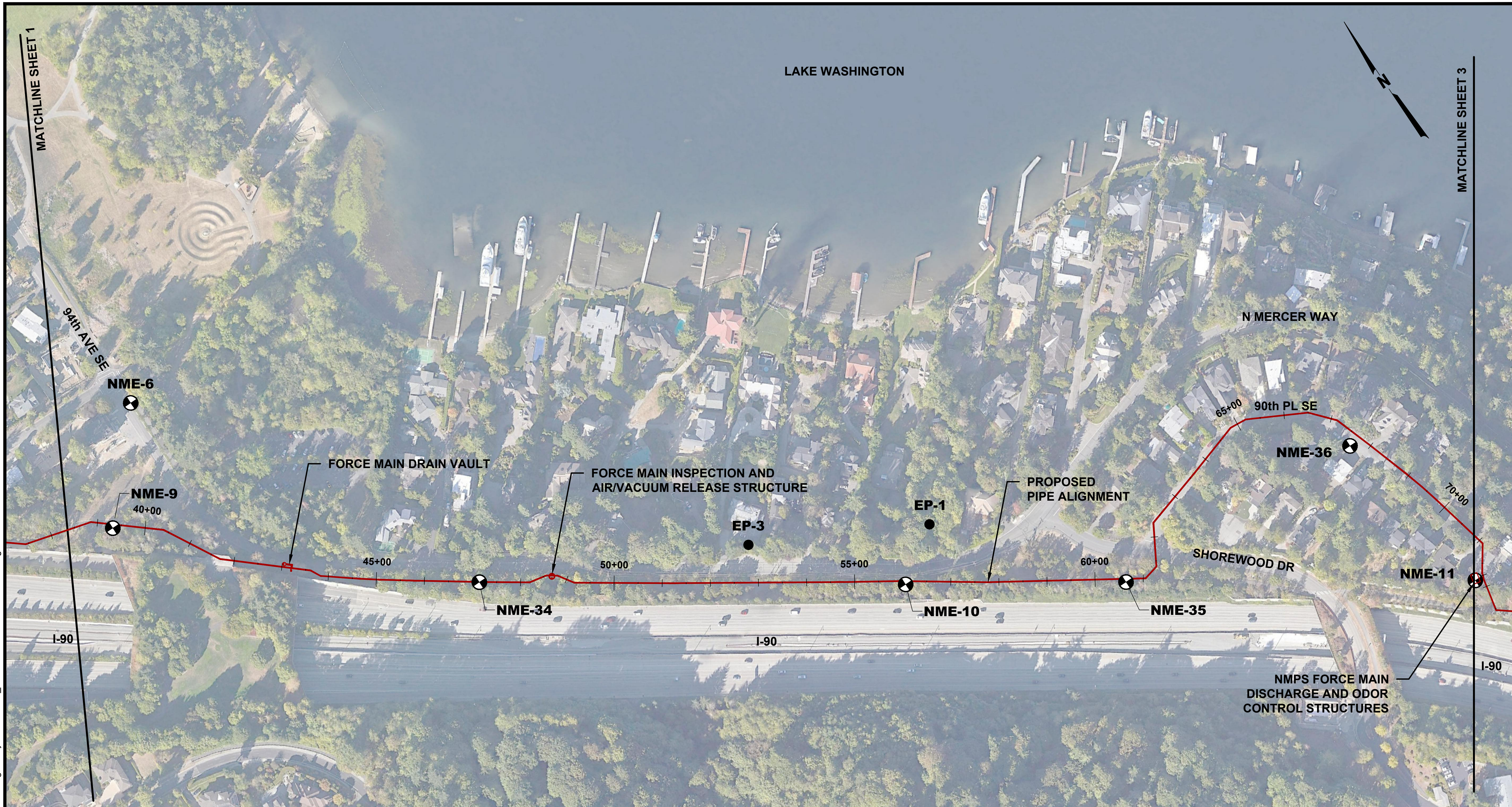
FIG. 2
Sheet 1 of 6



LEGEND

NME-1 — Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1, TP-1 — Existing Exploration Designation and Approximate Location (Completed by Others)

0 200 400

Scale in Feet

NOTE

Figure adapted from JACOBS' file *2019-06-27_Overall.dwg and Tetra Tech's files 2019-08-15_LS11_96th Ave Siphon.dwg* and *E00306E13-PIPE-ALGN-EIR_EAST.dwg dated 2019-08-15.*

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN
CONVEYANCE
CONTRACT NO. C01340C20

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FIG. 2
Sheet 2 of 6

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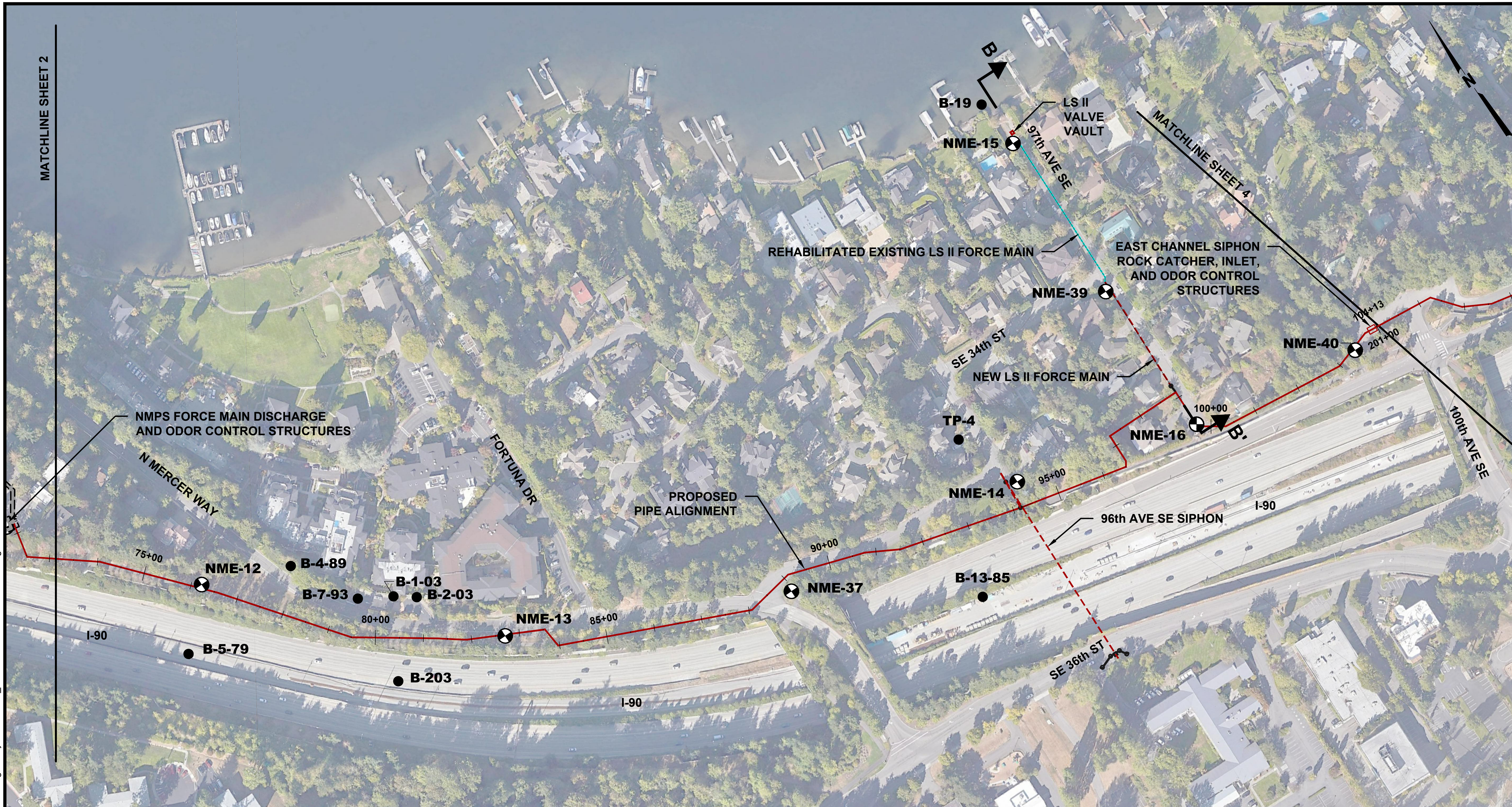

LEGEND

NME-1 — Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1, TP-1 — Existing Exploration Designation and Approximate Location (Completed by Others)

0 200 400

Scale in Feet

NOTE

Figure adapted from JACOBS' file *2019-06-27_Overall.dwg and Tetra Tech's files 2019-08-15_LS11_96th Ave Siphon.dwg* and *E00306E13-PIPE-ALGN-EIR_EAST.dwg dated 2019-08-15.*

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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FIG. 2
Sheet 3 of 6

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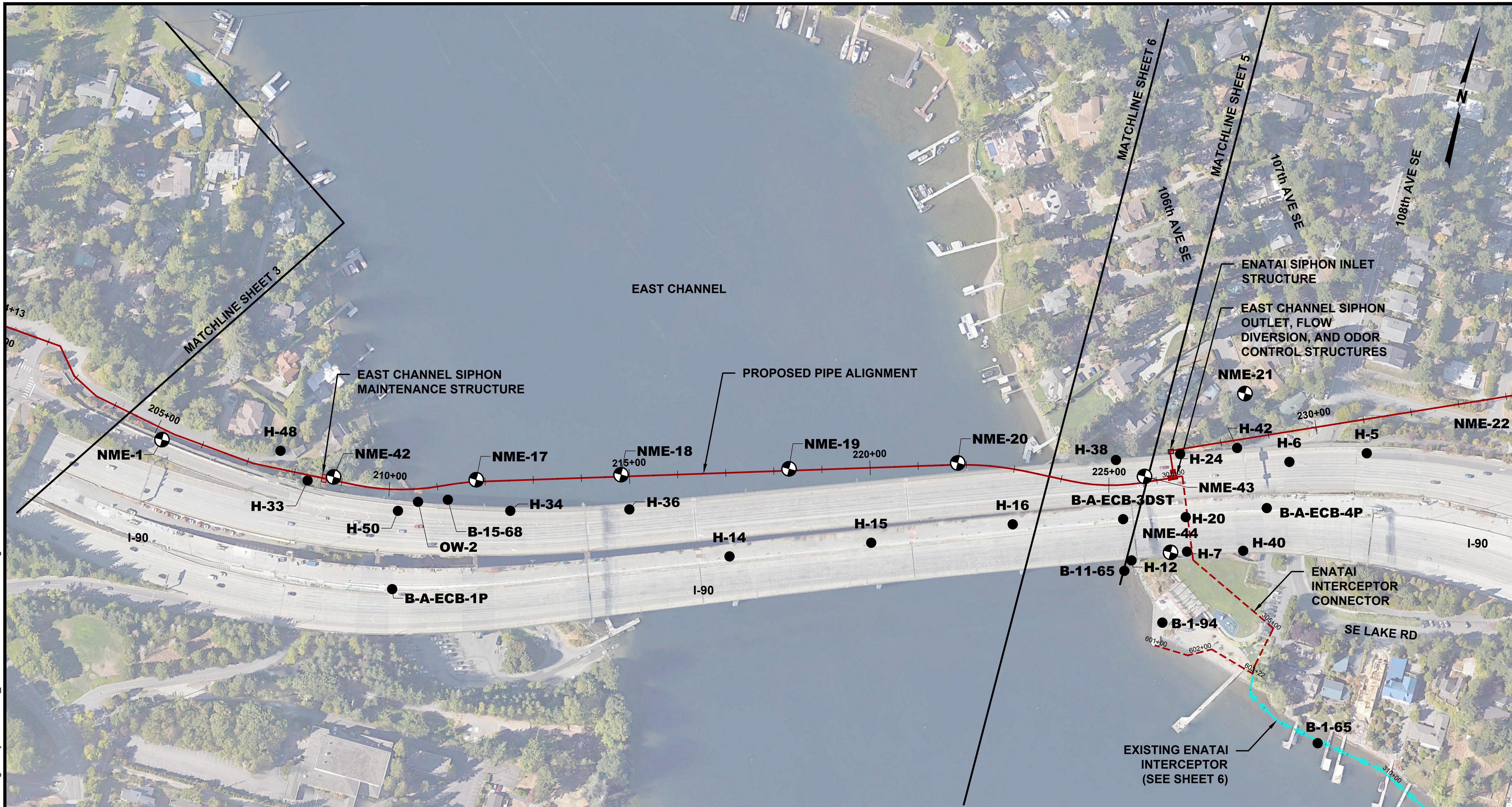

LEGEND

NME-1 — Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1, TP-1 — Existing Exploration Designation and Approximate Location (Completed by Others)

0 200 400

Scale in Feet

NOTE

Figure adapted from JACOBS' file *2019-06-27_Overall.dwg and* *Tetra Tech's files 2019-08-15_LS11_96th Ave Siphon.dwg* and *E00306E13-PIPE-ALGN-EIR_EAST.dwg dated 2019-08-15.*

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 2
Sheet 4 of 6

Filename: J:\211\22000\21\21-1-22000 Plans.dwg Layout: Sheet 4_GDM Date: 09-03-2019 Login: sac

LEGEND

NME-1 Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1, TP-1 Existing Exploration Designation and Approximate Location (Completed by Others)

0 200 400

Scale in Feet

NOTE

Figure adapted from JACOBS' file *2019-06-27_Overall.dwg and* Tetra Tech's files *2019-08-15_LS11_96th Ave Siphon.dwg* and *E00306E13-PIPE-ALGN-EIR_EAST.dwg dated 2019-08-15.*

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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FIG. 2
Sheet 5 of 6

Filename: J:\211\22000\21-1-22000 Plans.dwg Layout: Sheet 5_GDM Date: 09-03-2019 Login: sac

LEGEND

NME-1 — Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1, TP-1 — Existing Exploration Designation and Approximate Location (Completed by Others)

0 200 400

Scale in Feet

NOTE

Figure adapted from JACOBS' file *2019-06-27_Overall.dwg and Tetra Tech's files 2019-08-15_LS11_96th Ave Siphon.dwg* and *E00306E13-PIPE-ALGN-EIR_EAST.dwg dated 2019-08-15.*

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN CONVEYANCE CONTRACT NO. C01340C20

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FIG. 2
Sheet 6 of 6

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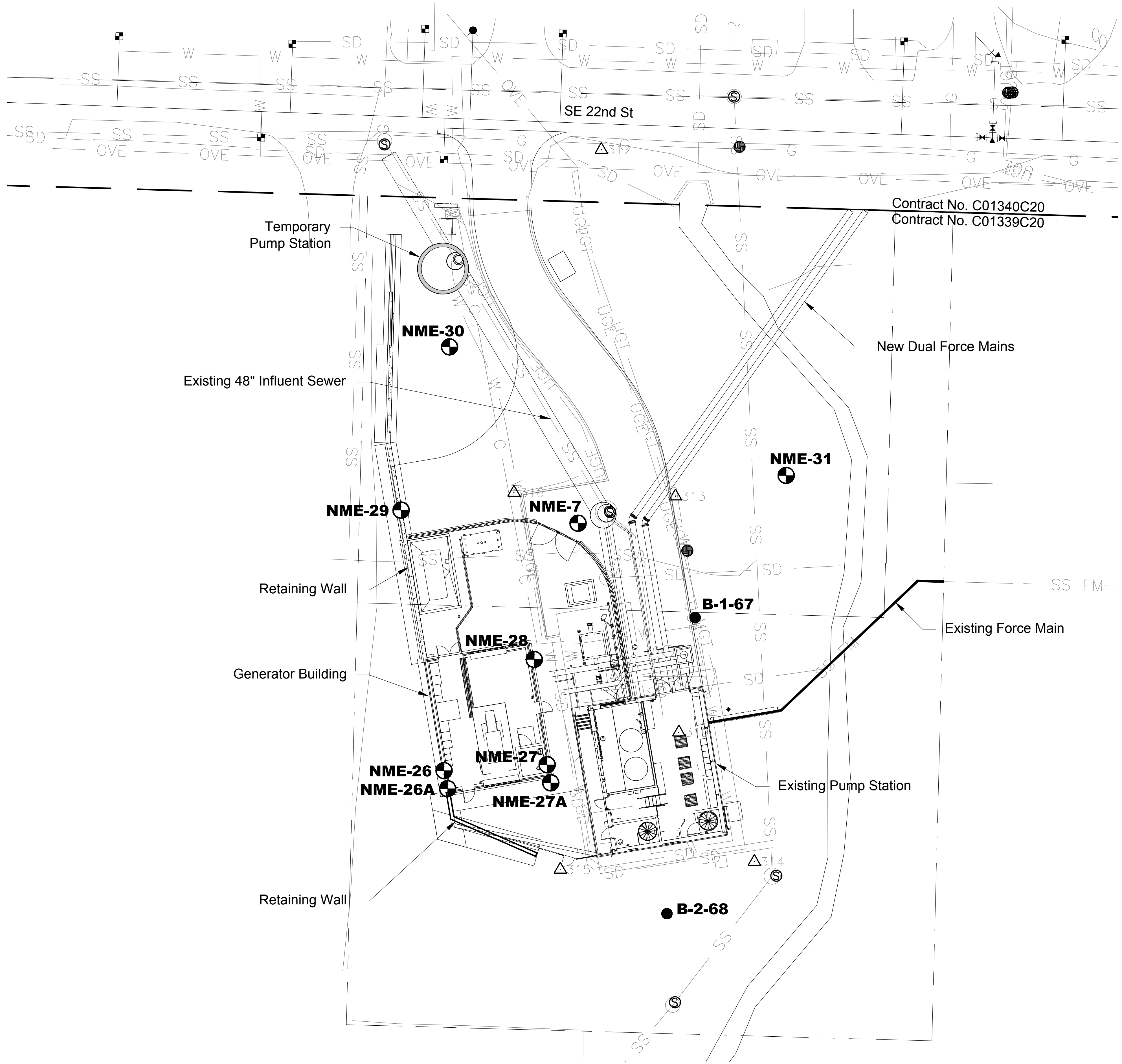

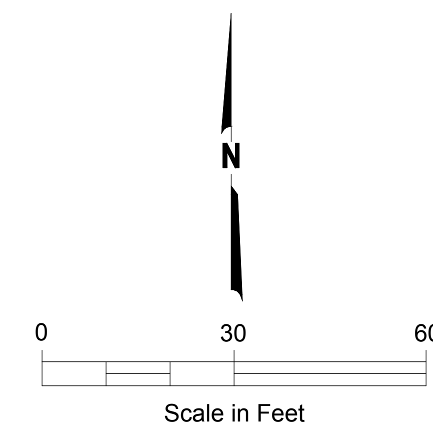

LEGEND

NME-7 — Current Exploration Designation and Approximate Location (Completed by Shannon & Wilson)

B-1-67 — Existing Exploration Designation and Approximate Location (Completed by Others)

NOTE

Figure adapted from TetraTech file, *333-E00306E13G011.dwg,* received 8-14-19.

North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

SITE AND EXPLORATION PLAN
NORTH MERCER PUMP STATION
CONTRACT NO. C01339C20

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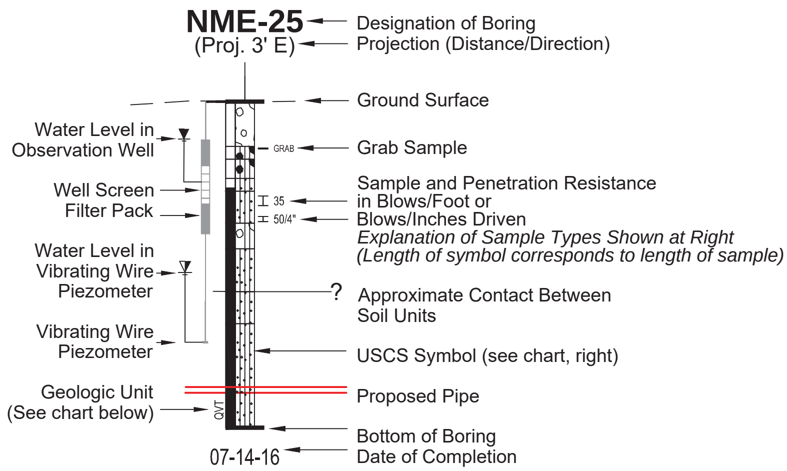
FIG. 3

Filename: J:\211\22000\21\21-1-22000-212 Fig 3 - Pump Station Plan.dwg Layout: Layout Date: 08-30-2019 Login: sac

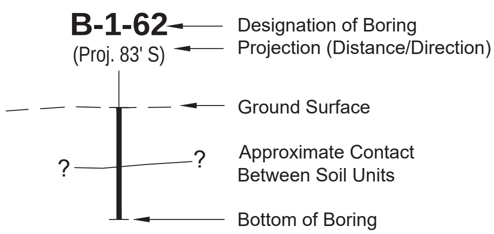
BORING LOG LEGEND

(Current Borings)



BORING LOG LEGEND

(Existing Borings)



UNIFIED SOIL CLASSIFICATION SYSTEM

(From USACE Tech Memo 3-357)

GW	SM
GP	SC
GW-GM	CL
GP-GM	ML
GM	OL
GC	CH
SW	MH
SP	OH
SW-SM	PT or wood
SP-SM	

SAMPLE TYPES

- * Sample Not Recovered
- 2" O.D. Split Spoon Sample or with 140 lb. Hammer (standard penetration test - SPT)
- Grab Sample
- Sonic Core

1. Dual Symbols (symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND) are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.
2. Borderline symbols (symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND) indicate that the soil may fall into one of two possible basic groups. The graphic symbol of only the first group symbol is shown on the profile.

NOMENCLATURE

GEOLOGIC AGE DESIGNATION		DEPOSITIONAL ENVIRONMENT, GEOLOGIC PROCESS, OR LITHOLOGY		
H = Holocene		f = fill	l = lucustrine	ls = landslide
Q = Quaternary	v = Vashon	r = recessional	o = outwash l = lacustrine	i = ice-contact
		gl = glacial lacustrine		
		at = ablation till		
		t = till (lodgment) d = till-like (diamict)	gl = glaciolacustrine a = advance outwash	
	p = Pre-Vashon 6 or more glacial and interglacial episodes	n = nonglacial (interglacial)	f = fluvial	l = lacustrine
		g = glacial	o = outwash l = lacustrine	d = till-like m = marine

GEOLOGIC NOMENCLATURE

Each geologic unit has a two- to four-letter abbreviation composed of a leading capital letter signifying geologic age, followed by one or more lowercase letters indicating further breakdown of geologic age, depositional environment or geologic process.

LEGEND

Glacially Overridden Soil Units Below Line

NOTES

1. The profiles are constructed from surface elevations based on the King County Metro Datum. The subsurface conditions shown are derived from borings conducted by Shannon & Wilson, Inc. for this study and from borings conducted by others for previous studies. Elevations and contacts should be considered approximate. Variations between the profile and actual conditions are likely to exist.
2. Water levels may fluctuate seasonally and may have changed since the last reading. Groundwater fluctuations should be expected.

SOIL UNITS

- Fill (Hf)
- Lacustrine (Hl, Qvrl)
- Recessional Deposits (Qvat, Qvri, Qvro)
- Landslide Deposits (Hls)
- Till and Till-Like Deposits (Qvt, Qvd, Qpgd)
- Glacial Outwash/Nonglacial Fluvial (Qva, Qpgo, Qpnf)
- Glaciolacustrine (Qvgl, Qpgl, Qpgm)
- Nonglacial Lacustrine (Qpnl)

NOTE: See Section 4.1 of this report for descriptions of the soil units.

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

LEGEND AND NOTES FOR SUBSURFACE PROFILES AND SECTIONS

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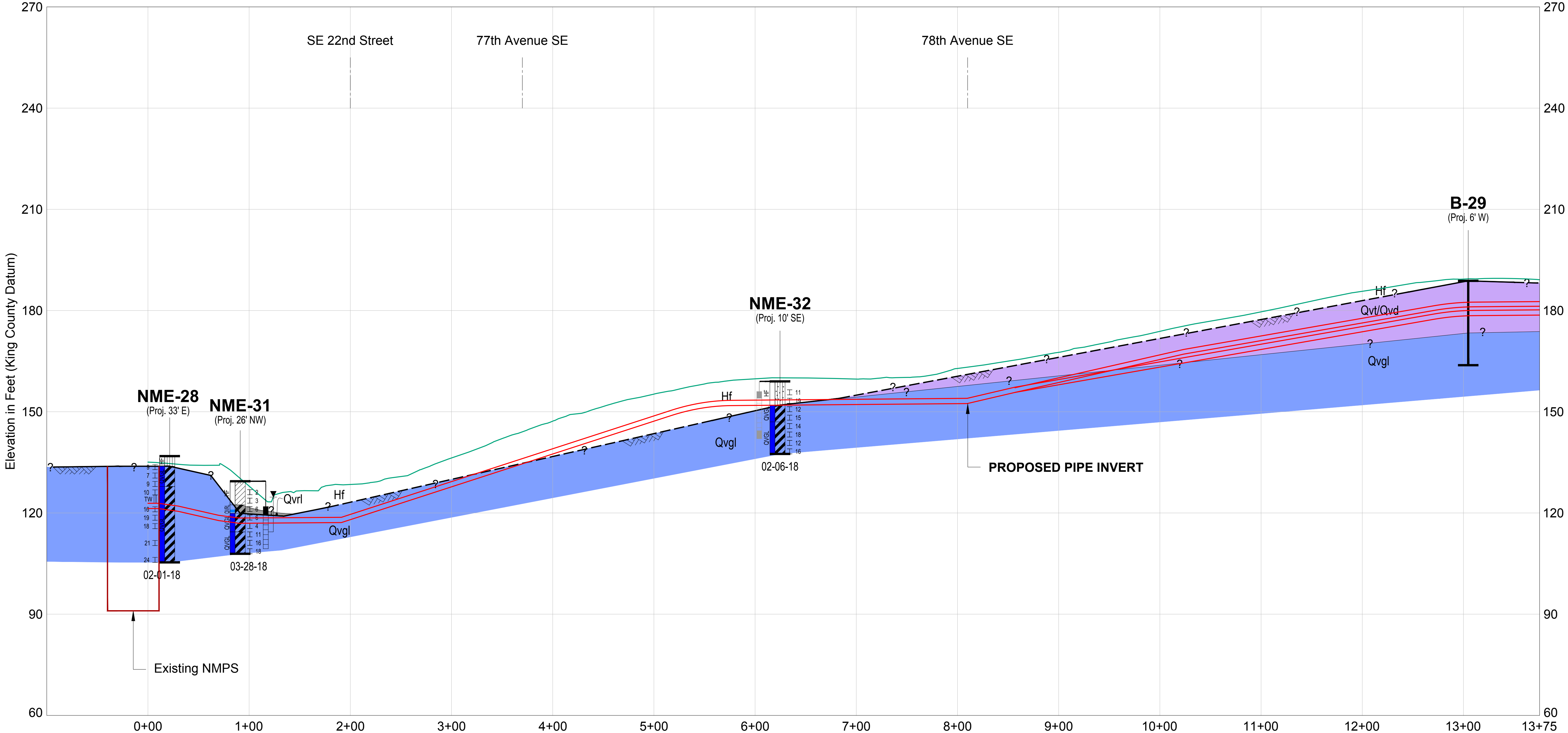
FIG. 4

Filename: J:\211\22000\213\21-1-22000-213 Fig 4 - Legend.dwg Date: 06-07-2018 Login: SAC



0 30 60

Vertical Scale in Feet

0 100 200

Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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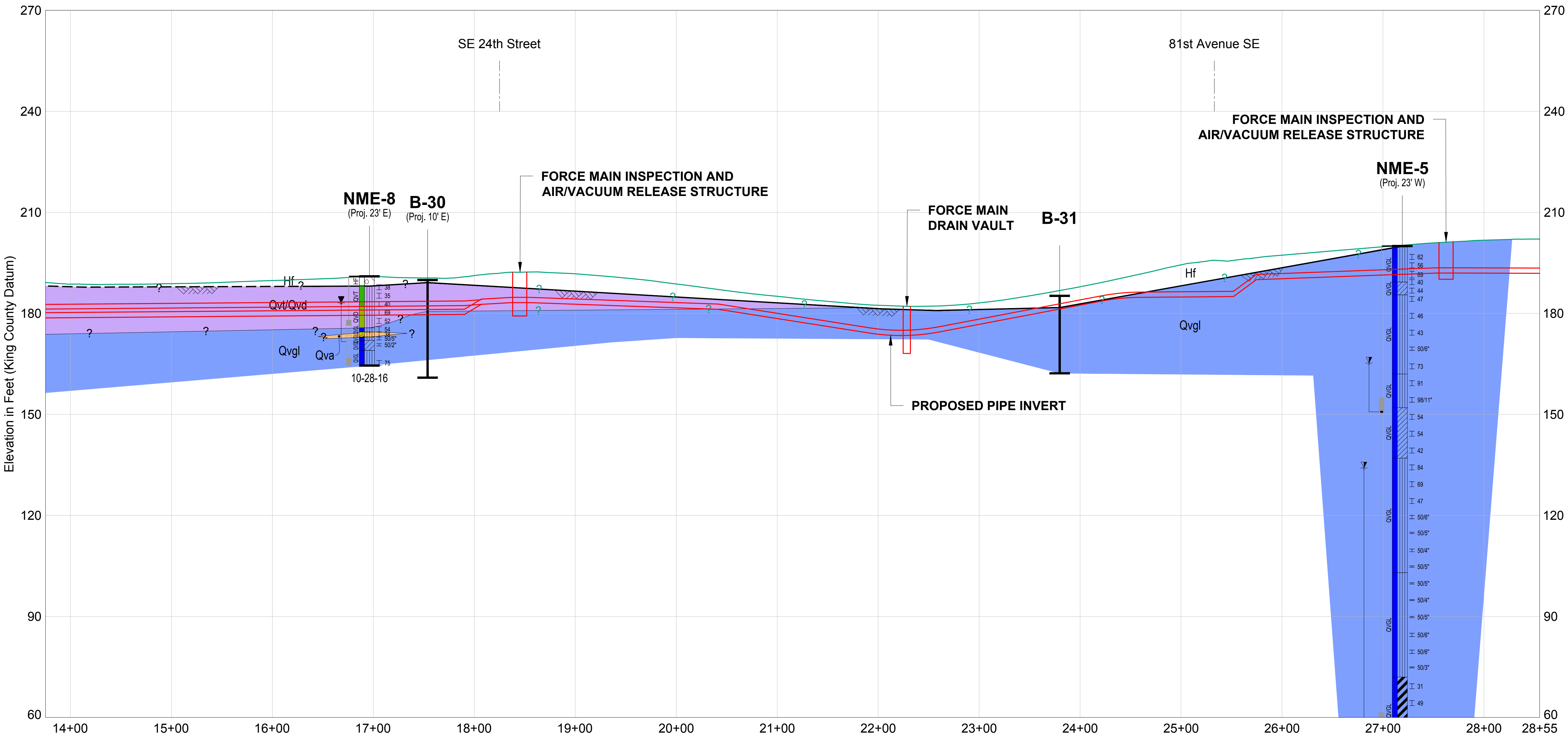
FIG. 5
Sheet 1 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 1 of 11 Date: 09-03-2019 Login: sac



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Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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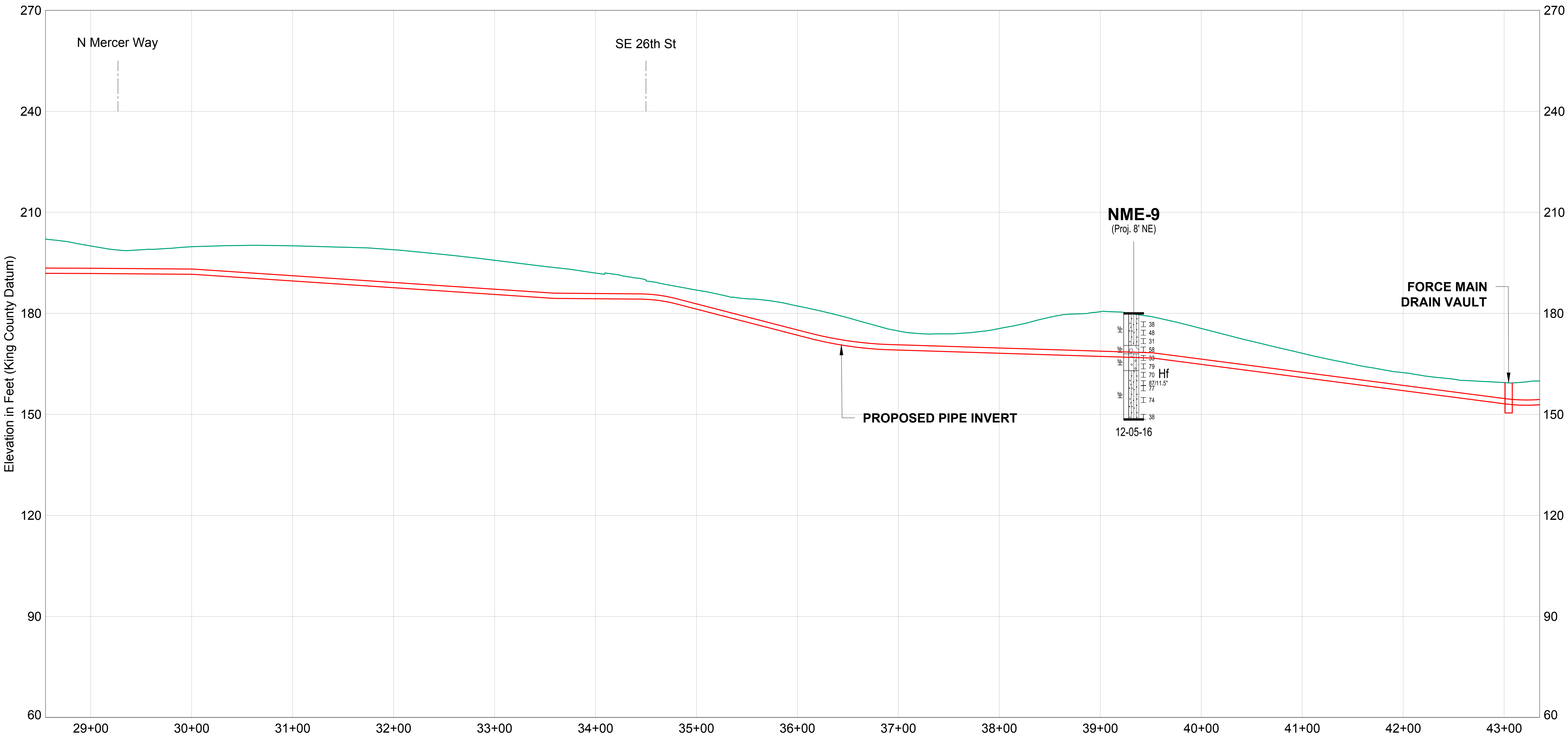
FIG. 5
Sheet 2 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 2 of 11 Date: 09-03-2019 Login: sac



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NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

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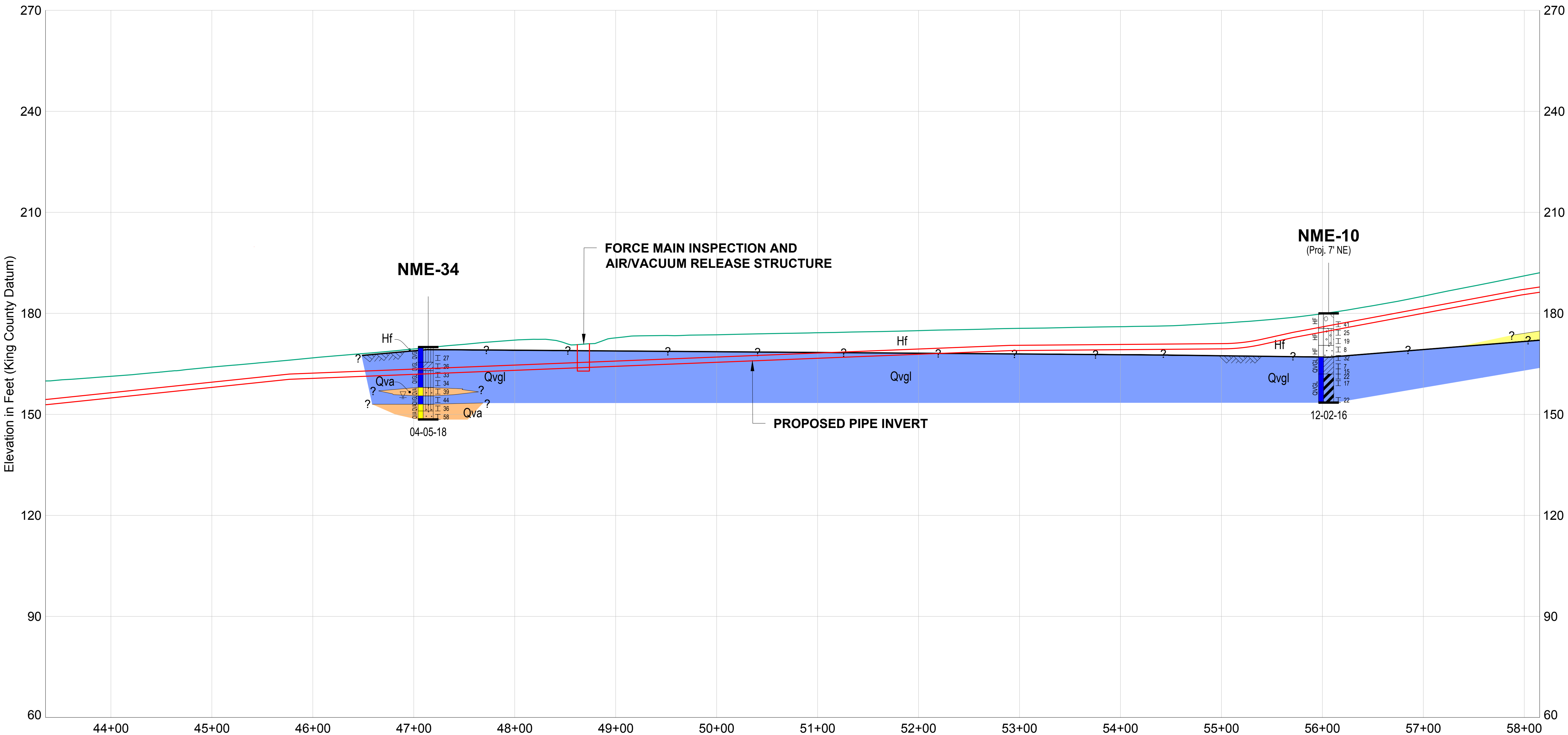
FIG. 5
Sheet 3 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 3 of 11 Date: 09-03-2019 Login: sac



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Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

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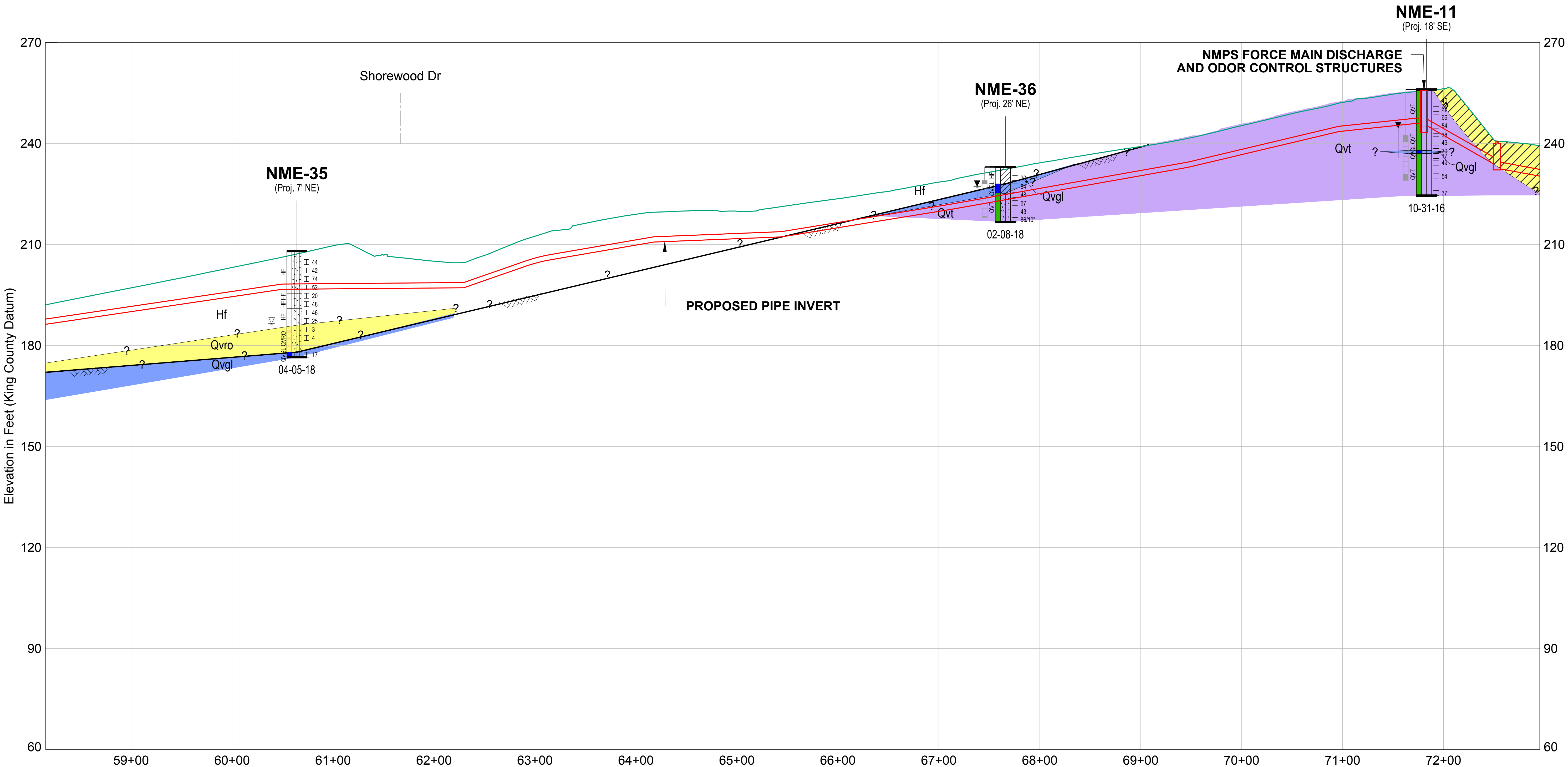
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5
Sheet 4 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 4 of 11 Date: 09-03-2019 Login: sac



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Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

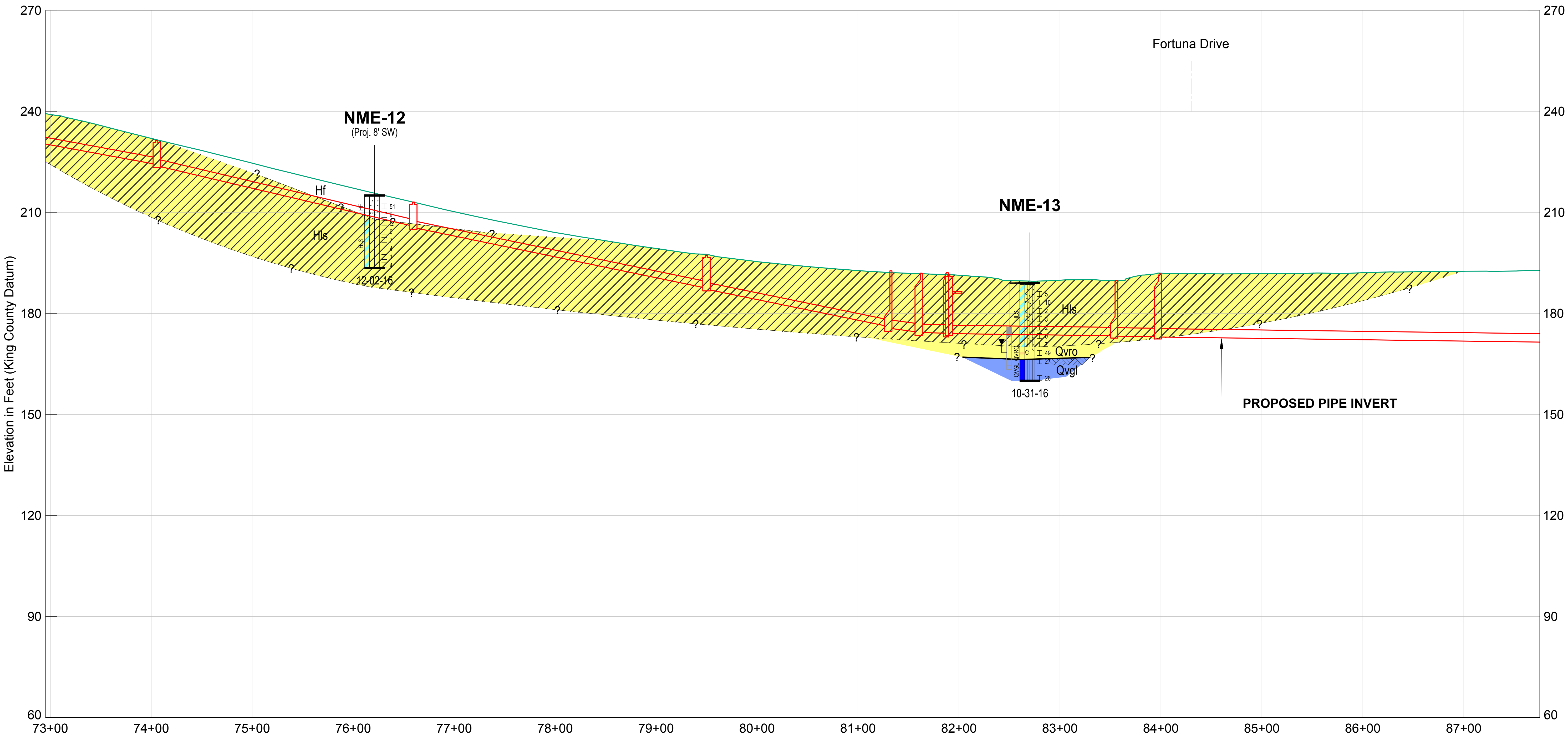
GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5
Sheet 5 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 5 of 11 Date: 09-03-2019 Login: sac



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Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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FIG. 5
Sheet 6 of 11

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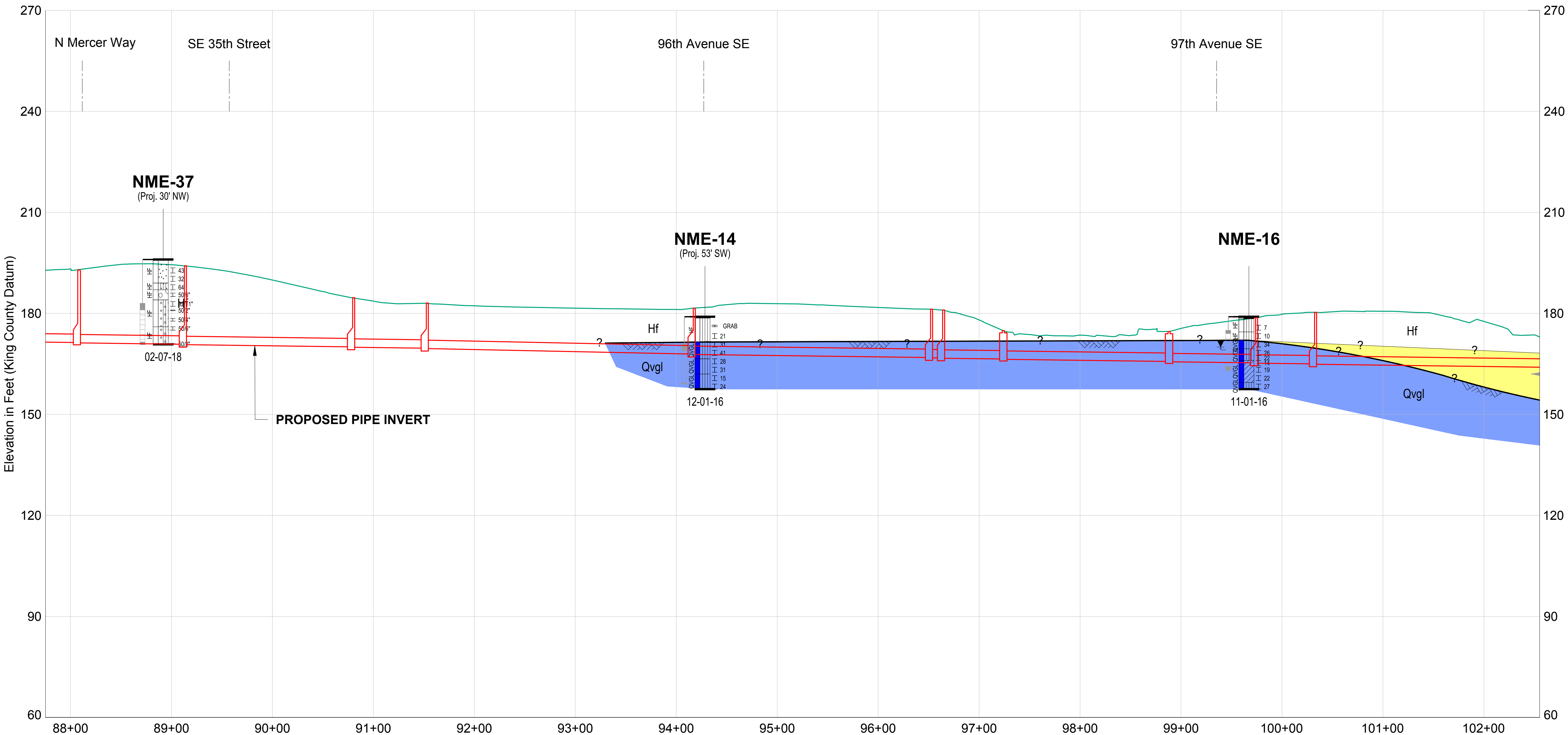

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Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

SHANNON & WILSON, INC.
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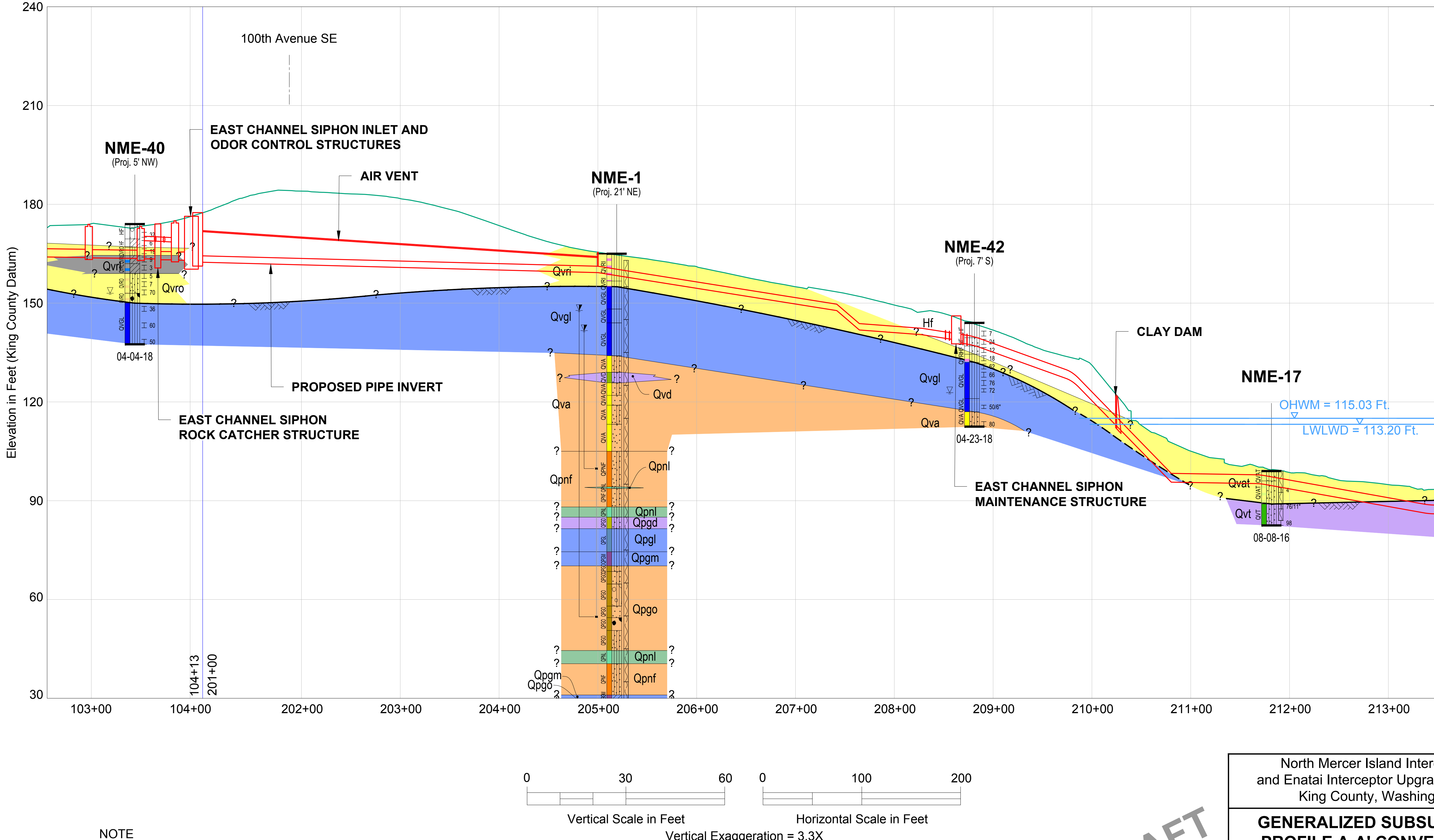
FIG. 5
Sheet 7 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 7 of 11 Date: 09-03-2019 Login: sac



NOTE

See Figure 4 for profile notes and legend.

North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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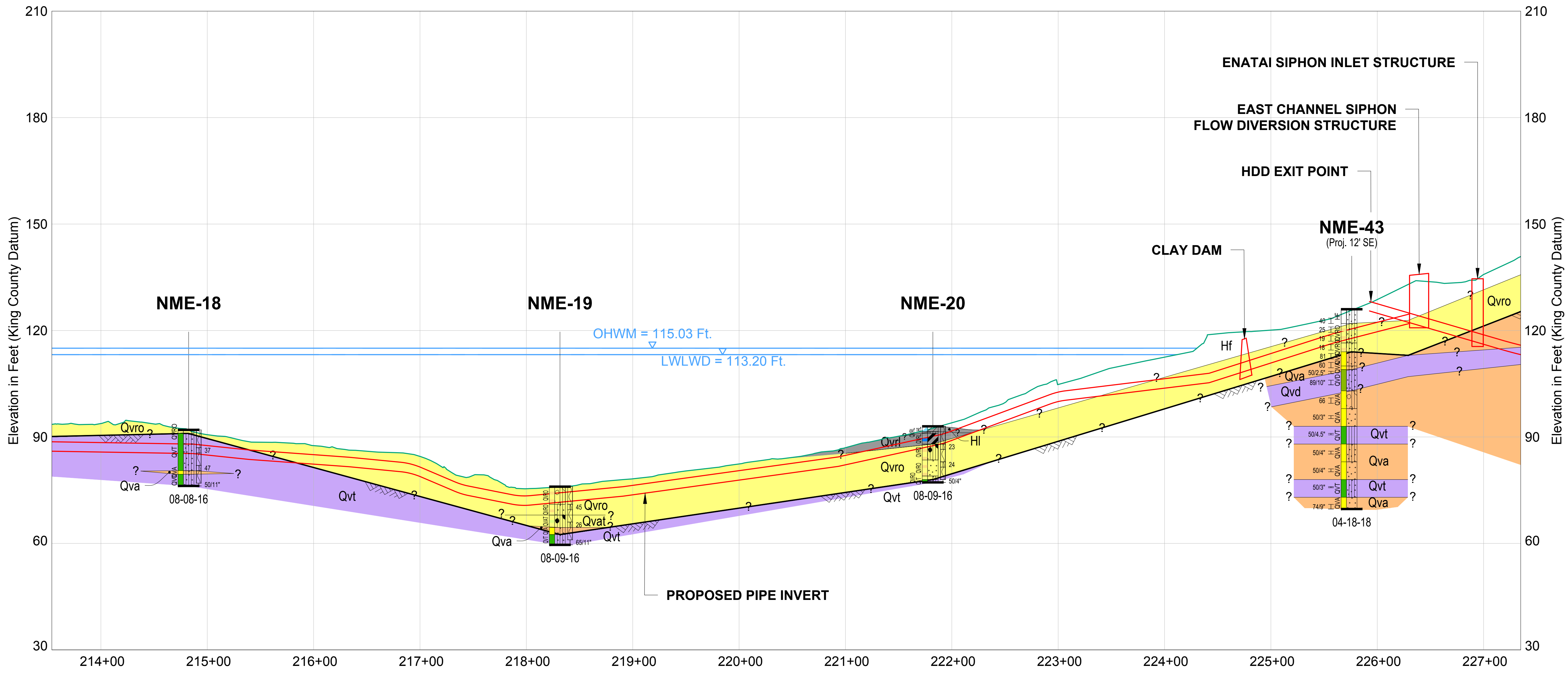
FIG. 5
Sheet 8 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 8 of 11 Date: 09-03-2019 Login: sac



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Vertical Scale in Feet

0 100 200
Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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King County, Washington

GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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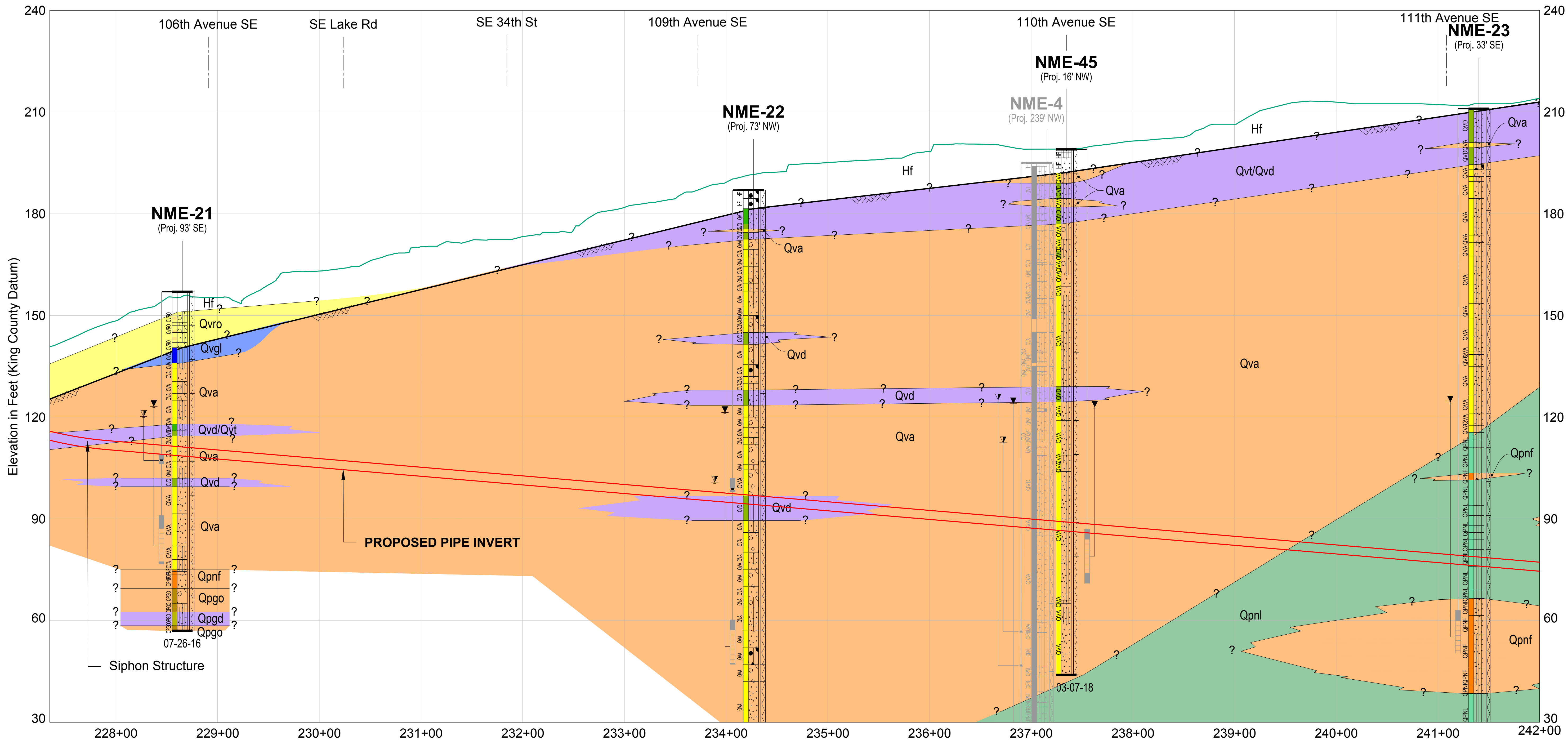
FIG. 5
Sheet 9 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 9 of 11 Date: 09-24-2019 Login: sac



0 30 60
Vertical Scale in Feet

0 100 200
Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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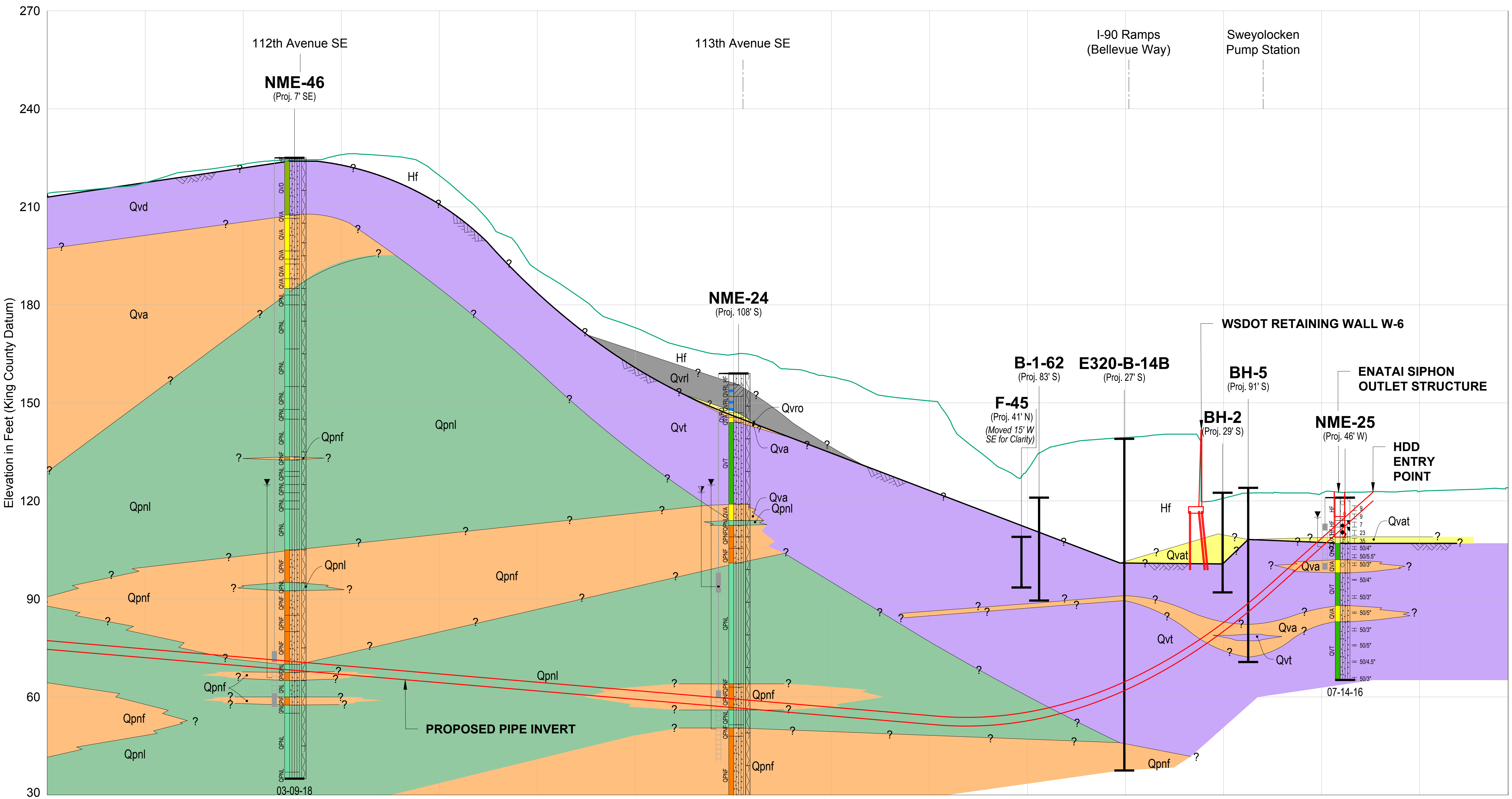
FIG. 5
Sheet 10 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 10 of 11 Date: 09-03-2019 Login: sac



0 30 60

Vertical Scale in Feet

0 100 200

Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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GENERALIZED SUBSURFACE PROFILE A-A' CONVEYANCE CONTRACT NO. C01340C20

September 2019 21-1-22000-213

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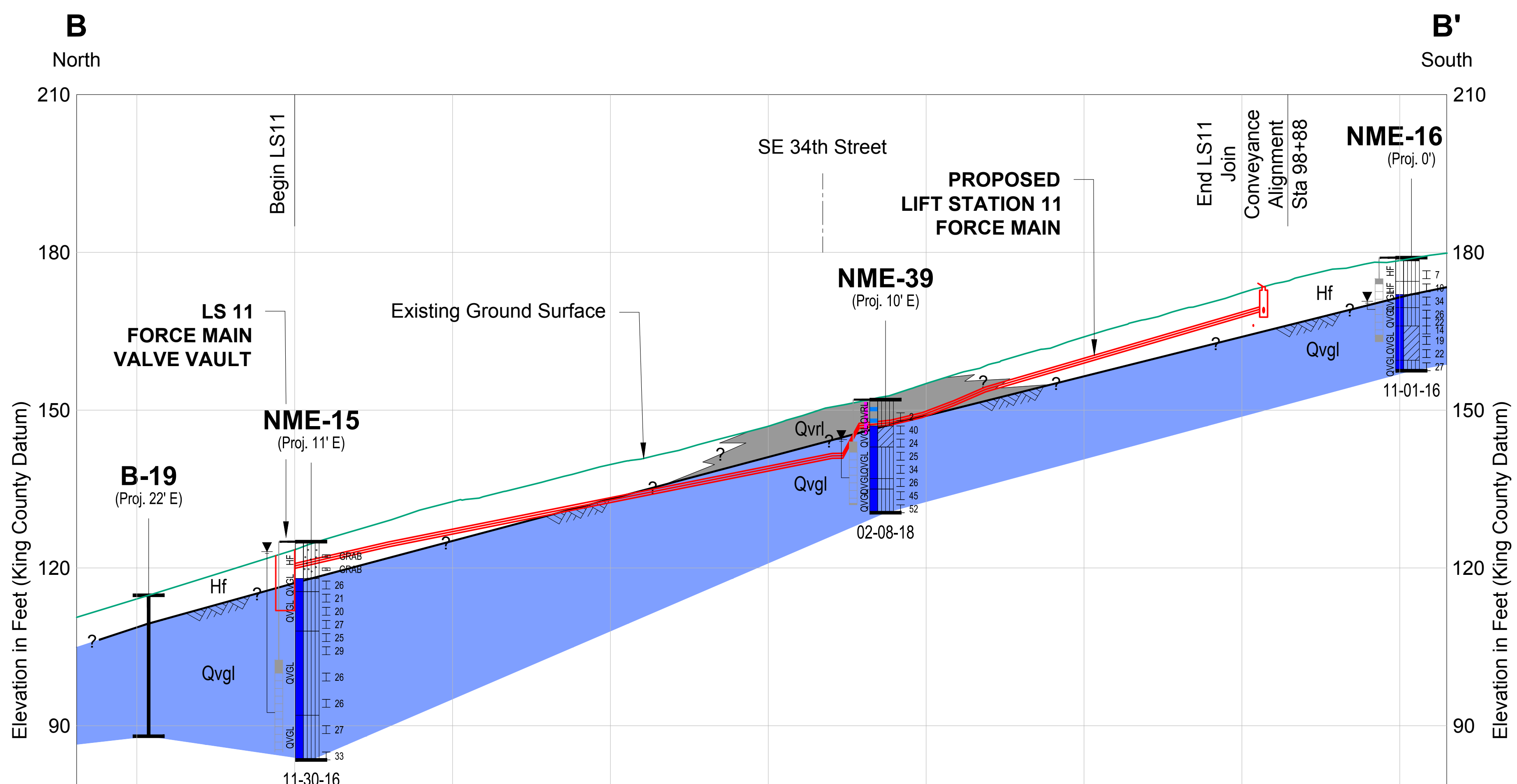
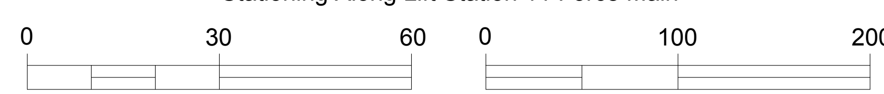
FIG. 5
Sheet 11 of 11

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: 11 of 11 Date: 09-03-2019 Login: sac



Stationing Along Lift Station 11 Force Main



Vertical Scale in Feet

Horizontal Scale in Feet

Vertical Exaggeration = 3.3X

NOTE

See Figure 4 for profile notes and legend.

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GENERALIZED SUBSURFACE PROFILE B-B'
LIFT STATION 11 FORCE MAIN
CONTRACT NO. C01739C02

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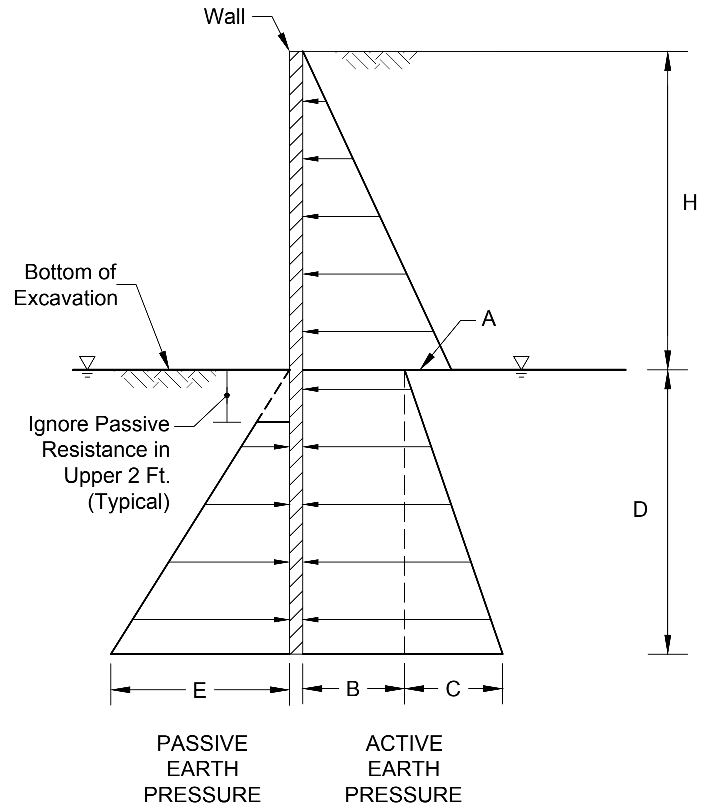
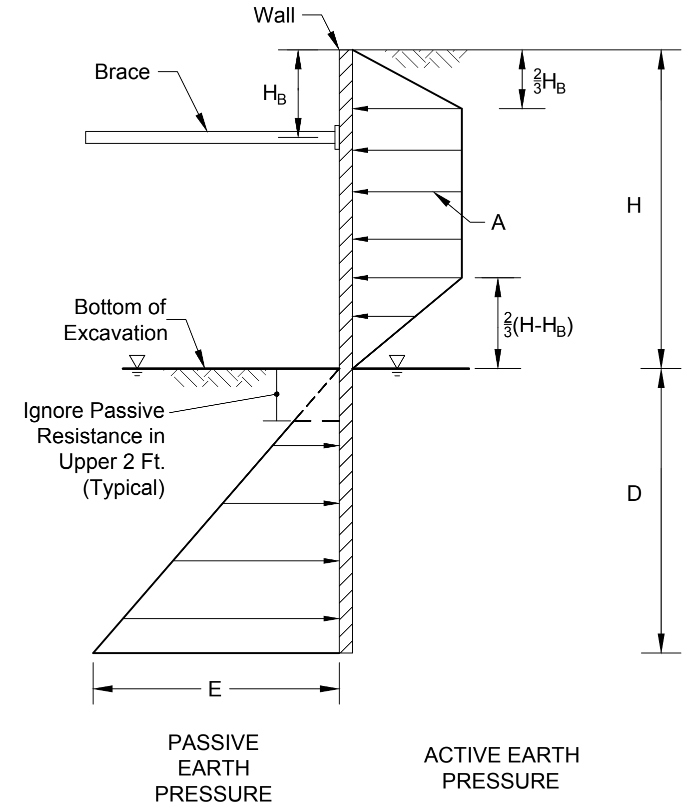
FIG. 6

Filename: J:\211\22000\213\21-1-22000-213 Profiles.dwg Layout: LSFM Date: 09-03-2019 Login: sac

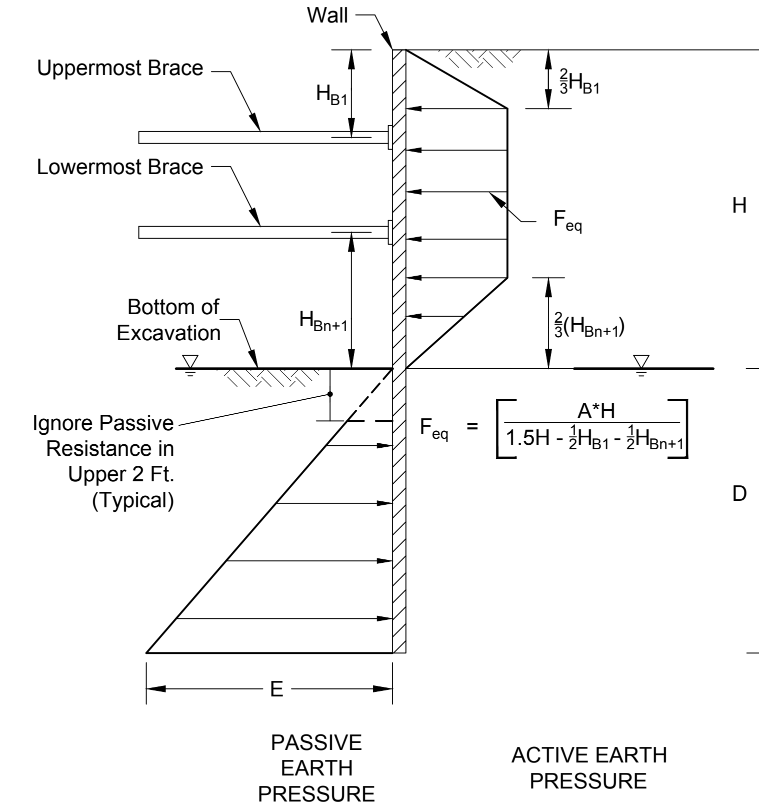
Recommended Temporary Earth Pressures for Cantilevered Wall



Recommended Temporary Earth Pressures for Single Braced Wall



Recommended Temporary Earth Pressures for Multiple Braced Wall



RECOMMENDED EARTH PRESSURE FACTORS

STRUCTURE	A	B	C	E
NMPS Temporary Pump Station, Vaults, Odor Control, and Retaining Wall	40H	40H	20D	210D
Force Main Inspection Structures	32H	32H	16D	355D
Force Main Drain Vault (Sta ~ 22+30)	36H	32H	16D	355D
Force Main Drain Vault (Sta ~42+71)	32H	32H	16D	355D
NMPS FM Discharge and 90th Place SE Odor Control	32H	32H	16D	355D
East Channel Siphon Rock Catcher, Inlet, and Odor Control	37H	33H	17D	355D
East Channel Siphon Maintenance	36H	32H	17D	400D
East Channel Siphon Flow Diversion and Odor Control	34H	34H	17D	290D
Enatai Siphon Inlet	34H	28H	15D	520D
Enatai Siphon Outlet	34H	28H	15D	520D
Lift Station 11 Valve Vault	39H	39H	19D	220D

NOTES

1. The recommended pressure diagrams are based on a continuous wall system. If soldier piles with lagging are used, apply active or at-rest pressure over the width of the soldier piles below the bottom of the excavation and apply passive resistance over three times the diameter of the piles or the spacing of the piles, whichever is smaller.
2. The total temporary lateral shoring pressure is the sum of the active earth pressure and surcharge pressure (Figure 10).
3. All earth pressures are in units of pounds per square foot.
4. Groundwater is assumed to be lowered and maintained at the base of the excavation during construction.
5. Passive pressures include F.S. =1.5.
6. Wall embedment (D) should consider kickout resistance. Embedment should be determined by satisfying horizontal static equilibrium about the bottom of the pile. Minimum recommended embedment is 10 feet.
7. Design lagging for 50% of lateral earth and surcharge pressure.
8. Diagrams are not to scale.
9. This figure shall not be used for construction or included in the construction contract.

LEGEND

H = Total Excavation Height, feet

D = Total Embedment Depth, feet

A, B, C, E = Earth Pressure Factors; See Table

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

TEMPORARY LATERAL SHORING PRESSURES

September 2019 21-1-22000-213

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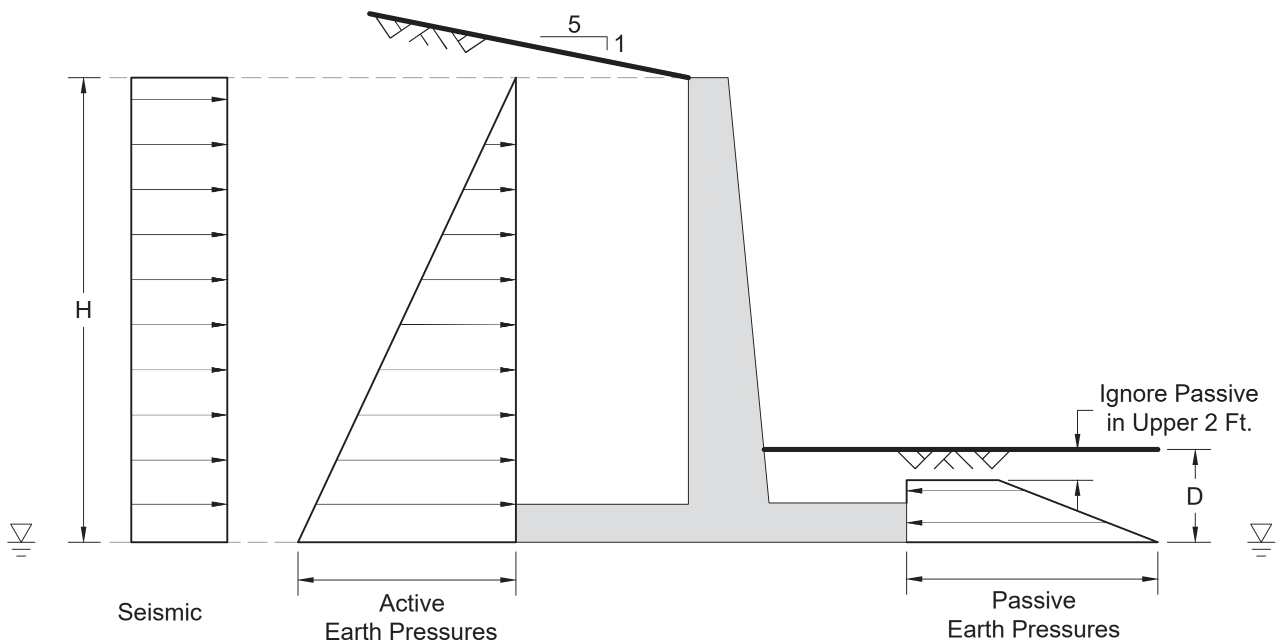
FIG. 7

Filename: J:\211\22000\213\21-1-22000-213 Fig 7 - Temporary Lateral Shoring Pressures.dwg Date: 09-25-2019 Login: sac



Not to Scale

Active	Passive	Seismic
50 H	365 D	21 H

NOTES

1. All pressures are in units of pounds per square foot. Total design pressure is the sum of the above pressures and applicable surcharge pressures (Figure 10).

2. Along the base of the retaining wall, use a Base Friction Coefficient equal to 0.25 and 0.35 for walls on native soil and structural fill, respectievely.

3. Passive pressures include F.S. = 1.5. Ignore passive resistance in upper 2 feet.

4. It is assumed that drainage is provided so that water pressures do not act on the wall.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
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LATERAL EARTH PRESSURES
CANTILEVER RETAINING WALL

September 2019 21-1-22000-213

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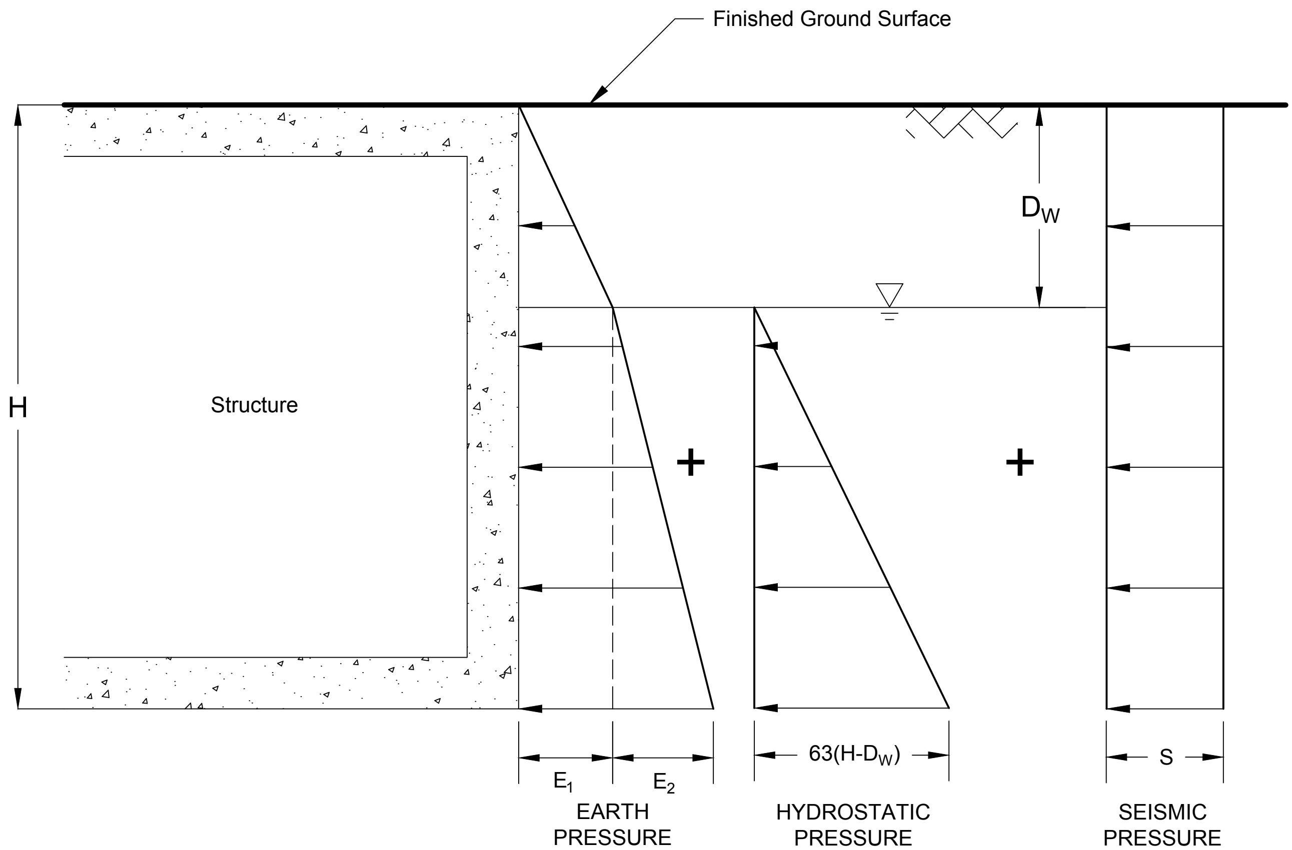
FIG. 8

File: J:\211\22000\213\21-1-22000-213 Fig 8 - Cantilever Wall LEP.dwg Date: 06-07-2018 Author: SAC



RECOMMENDED EARTH PRESSURE FACTORS

STRUCTURE	E_1	E_2	S	D_w (ft)
NMPS Temporary Pump Station, Vaults, and Odor Control	$67D_w$	$32(H-D_w)$	9H	9
NMPS Force Main Discharge and 90th Place SE Odor Control	$55D_w$	$29(H-D_w)$	9H	7
East Channel Siphon Rock Catcher, Inlet, and Odor Control	$62D_w$	$30(H-D_w)$	9H	15
East Channel Siphon Maintenance	$65D_w$	$32(H-D_w)$	9H	10
East Channel Siphon Flow Diversion, and Odor Control	$57D_w$	$29(H-D_w)$	9H	10
Enatai Siphon Inlet	$57D_w$	$29(H-D_w)$	9H	10
Enatai Siphon Outlet	$59D_w$	$29(H-D_w)$	9H	6

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NOTES

1. All pressures are in units of pounds per square foot (psf). Total design pressure is the sum of the above pressures and appropriate surcharge pressures (Figure 10).
2. This figure shall not be used for construction or included in the construction contract.
3. Diagrams are not to scale.

North Mercer Island Interceptor
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PERMANENT LATERAL PRESSURES FOR BURIED STRUCTURES

September 2019 | 21-1-22000-213

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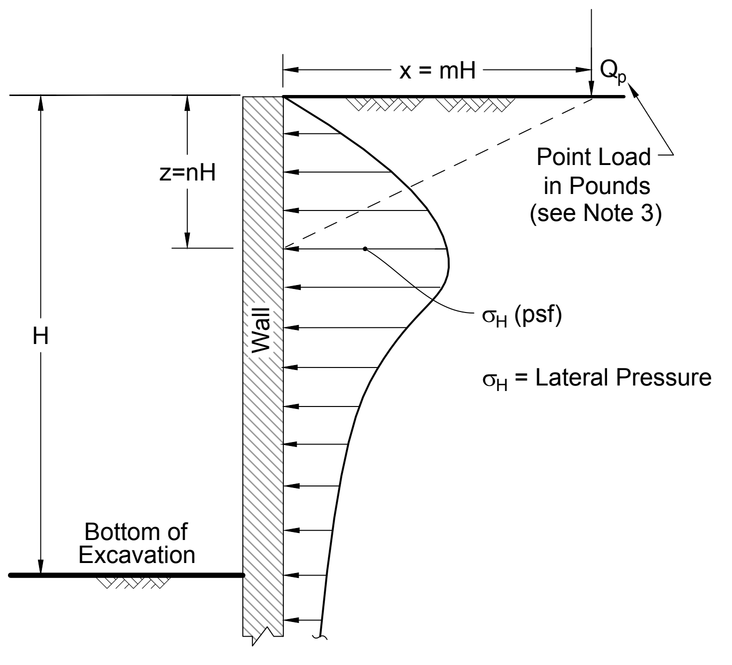
FIG. 9

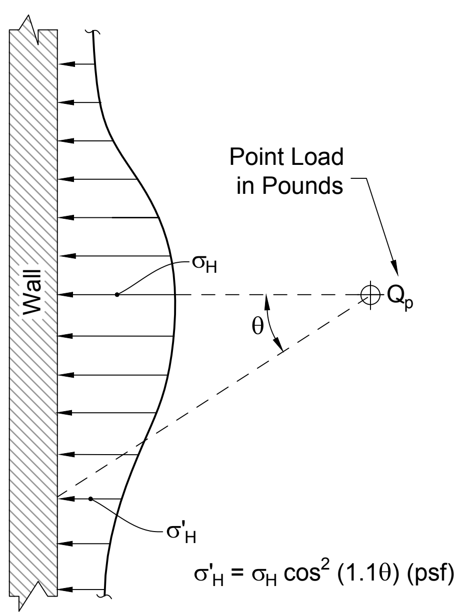
Filename: J:\211\22000\213\21-1-22000-213 Fig 9 - Permanent Lateral Pressures.dwg Layout: Layout Date: 09-24-2019 Login: sac



ELEVATION VIEW

For m ≤ 0.4: $\sigma_H = 0.28 \dfrac{Q_p}{H^2} \dfrac{n^2}{(0.16 + n^2)^3}$ (psf) (see Note 3)

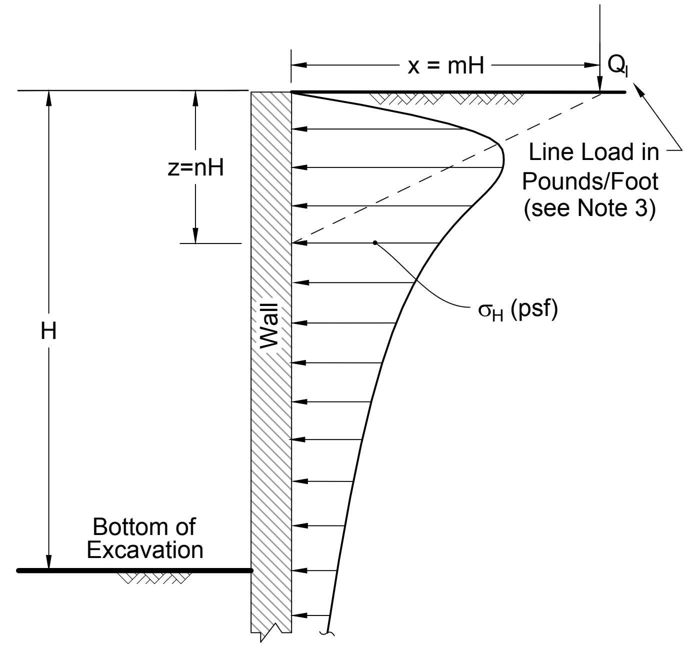
For m > 0.4: $\sigma_H = 1.77 \dfrac{Q_p}{H^2} \dfrac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)



PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD
i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



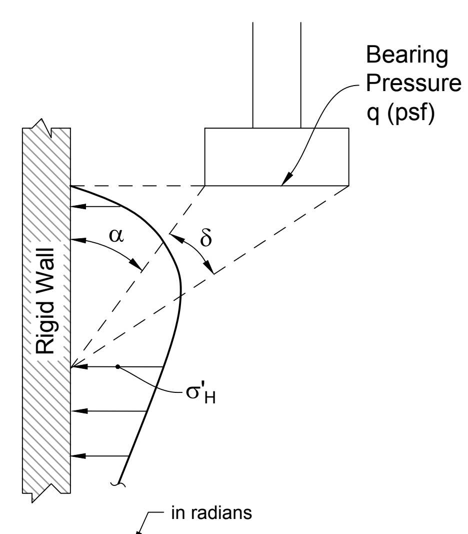
ELEVATION VIEW

For m ≤ 0.4: $\sigma_H = 0.20 \dfrac{Q_l}{H} \dfrac{n}{(0.16 + n^2)^2}$ (psf) (see Note 3)

For m > 0.4: $\sigma_H = 1.28 \dfrac{Q_l}{H} \dfrac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD
i.e. NARROW CONTINUOUS FOOTING
PARALLEL TO WALL

(NAVFAC DM 7.02, 1986)

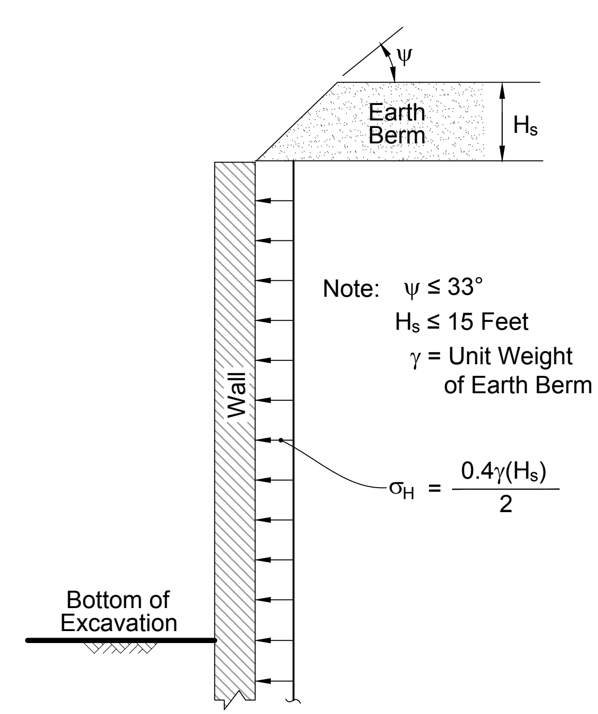


in radians

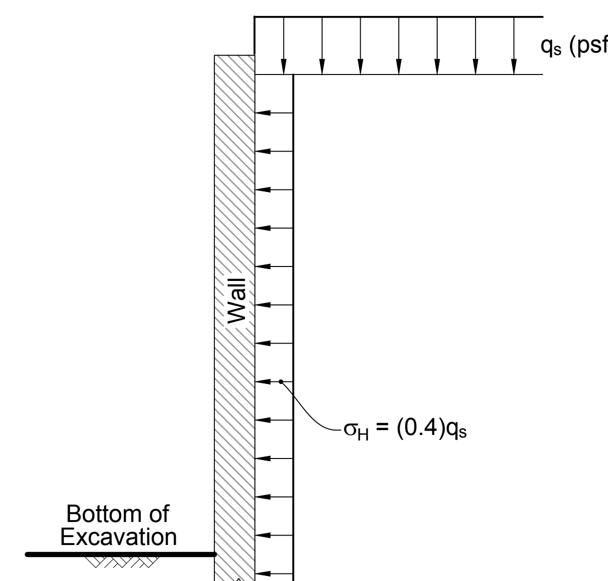
$\sigma_H = \dfrac{2q}{\pi} [\delta - \sin \delta \cos (\delta + 2\alpha)]$ (psf)

C) LATERAL PRESSURE DUE TO STRIP LOAD

(AASHTO LRFD Bridge Design Specifications, 2017)



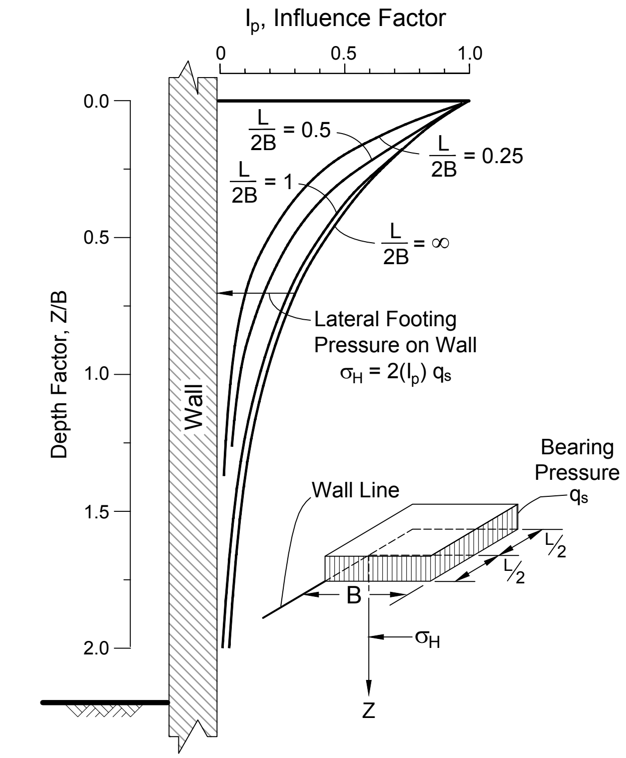
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM
OR UNIFORM SURCHARGE

(derived from Poulos and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE
TO ADJACENT FOOTING
(see Notes 4 and 5)

(derived from NAVFAC DM 7.02, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that m ≤ 0.4, it may be more appropriate to model the actual load distribution (i.e., Detail E) or use more rigorous analysis methods.
4. The stress is estimated on the back of the wall at the center of the length, L, of loading.
5. The estimated stress is based on a Poisson's ratio of 0.5.

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North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project
King County, Washington

RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

September 2019 | 21-1-22000-213

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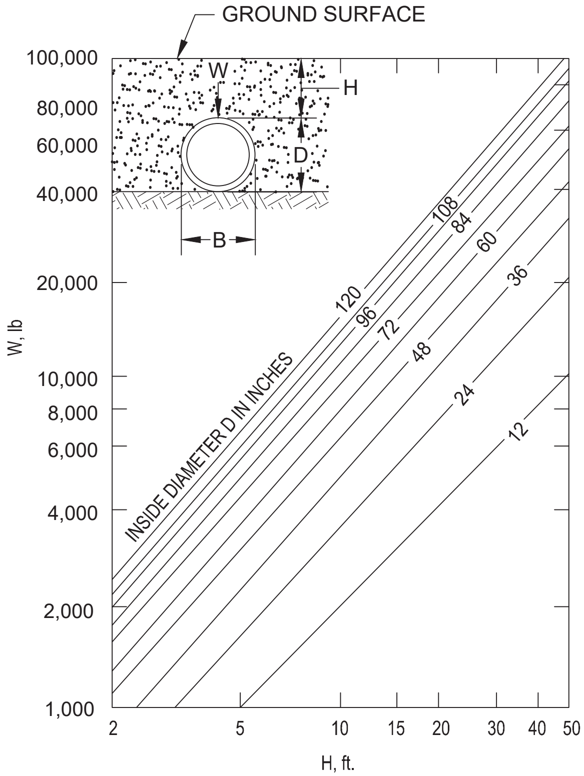
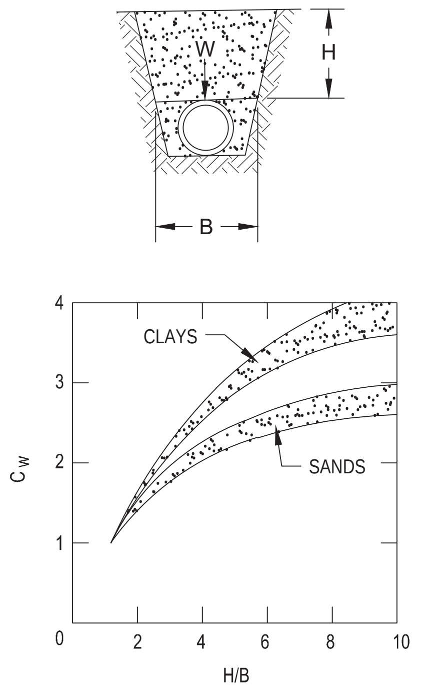
FIG. 10

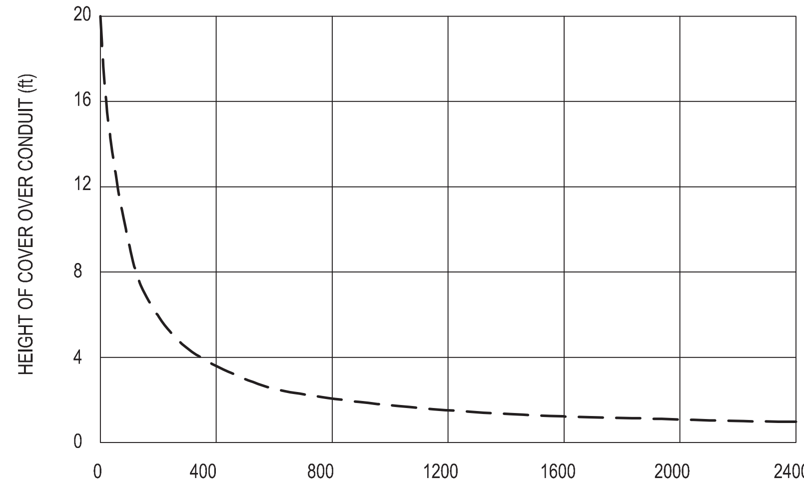
File: J:\211\22000\213\21-1-22000-213 Fig 10 - Surcharge Loading.dwg Date: 09-03-2019 Author: sac



(a) EMBANKMENT DEAD LOAD W ON A CONDUIT BURIED IN A SOIL EMBANKMENT.



(b) C_W FOR CONDUIT IN A TRENCH



(c) VERTICAL PRESSURE DUE TO H-20 LIVE LOAD ON CONDUIT (PSF)

NOTES

1. W = total dead load per unit length.

2. Embankment dead loads shown in (a) are based on soil unit weight of 100 pcf. For different soil unit weights, adjust the loads proportionately.

3. For trench backfill shown in (b): $W = C_W (\gamma)(B)^2$

   where: γ = soil unit weight.
   B = trench width at top of pipe level.

   If backfill compacted adequately, a unit weight of 130 pcf is recommended for evaluation.

4. Live loads shown in (c) include effect of impact.

5. This figure was adapted from NAVFAC DM7.

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LOADS ON BURIED UTILITIES

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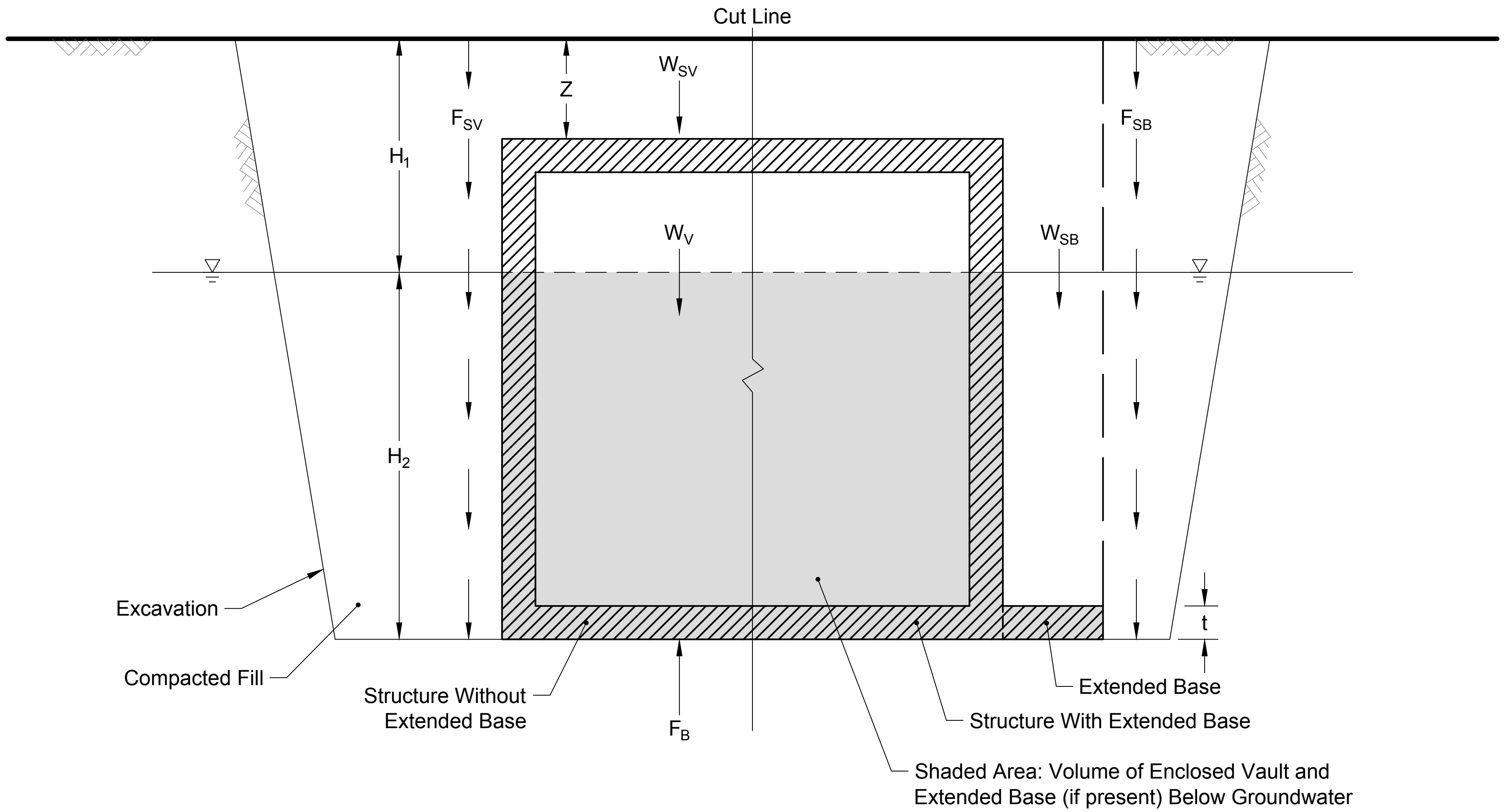
FIG. 11

FIG. 11



Not to Scale

NOTATION

γ_T = Total Unit Weight of Soil Above Groundwater Level, γ_T = 125 pcf

γ_B = Buoyant Unit Weight of Soil Below Groundwater Level, γ_B = 62.6 pcf

γ_W = Unit Weight of Water, γ_W = 62.4 pcf (Fresh Water)

ϕ = Soil Effective Stress Friction Angle, ϕ = 30°

δ = Soil - Structure Interface Friction Angle, $\delta = \frac{2}{3}\phi$ (Precast Concrete)

K = K_o for Unshored Excavations, K = K_a otherwise

K_o = At-Rest Earth Pressure Coefficient, $K_o = 1 - \sin\phi$

K_a = Active Earth Pressure Coefficient, $K_a = \frac{1-\sin\phi}{1+\sin\phi}$

A_{Top} = Area of Top of Vault, Ft^2

A_{EB} = Area of Extended Base, Ft^2

P = Perimeter Distance Around Base of Vault and Extended Base (if present), Ft.
May Not Be Same Shape as Shown

t = Thickness of Extended Base, Ft.

V = Volume of Enclosed Vault and Extended Base (if present) Below Groundwater, Ft^3.
May Not Be Same Shape as Shown

Z = Depth to Top of Vault

H_1 = Depth to Groundwater = 5 feet for All Structures Except for LS11 Valve Vault;
= Depth to Groundwater = 0 feet for LS11 Valve Vault

H_2 = Depth from Groundwater to Base of Vault

∇ = Groundwater Level

EQUATIONS

$$\text{Factor of Safety without Extended Base} = \frac{W_V + W_{SV} + F_{SV}\,(P)}{F_B}$$

$$\text{Factor of Safety with Extended Base} = \frac{W_V + W_{SV} + W_{SB} + F_{SB}\,(P)}{F_B}$$

W_V = Weight of Vault and Extended Base (if present), lb

W_{SV} = Weight of Soil Above Vault (if present), lb

Buried Vault with Groundwater Below Top of Vault: $W_{SV} = \gamma_T\, Z\, A_{Top}$ (lb)

Buried Vault with Groundwater Above Top of Vault: $W_{SV} = [\gamma_T H_1 + \gamma_B (Z - H_1)]\, A_{Top}$ (lb)

W_{SB} = Weight of Soil Above Extended Base (if present), lb

$= [\gamma_T H_1 + \gamma_B (H_2 - t)]\, A_{EB}$

F_{SV} = Shearing Resistance of Soil to Vault Wall, lb/Ft

$= (\frac{1}{2})\,(\tan\delta)\,(K)\,(2\,\gamma_T H_1 H_2 + \gamma_T H_1^2 + \gamma_B H_2^2)$

F_{SB} = Shearing Resistance of Soil, lb/Ft

$= (\frac{1}{2})\,(\tan\phi)\,(K)\,(2\,\gamma_T H_1 H_2 + \gamma_T H_1^2 + \gamma_B H_2^2)$

F_B = Buoyant Force, lb

$= V\,\gamma_W$

NOTES

1. Uplift could result in high moments in bottom slab.
2. If temporary or permanent shoring is left in place adjacent to vault, F_{SV} and F_{SB} should be ignored.

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UPLIFT RESISTANCE FOR BURIED MAINTENANCE HOLES AND STRUCTURES

September 2019 | 21-1-22000-213

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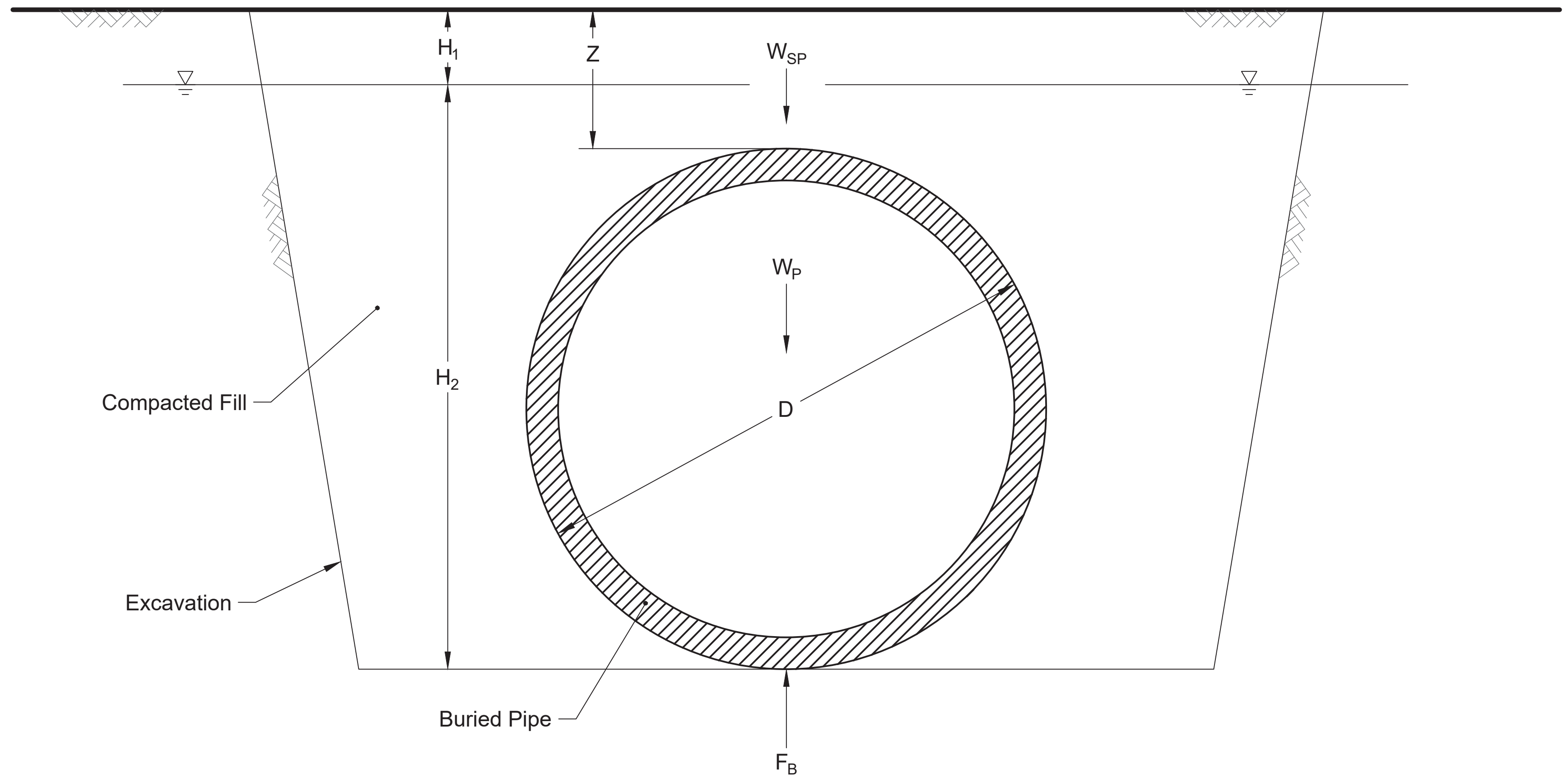
FIG. 12

Filename: J:\211\22000\213\21-1-22000-213 Fig 12- Uplift Resistance Buried Maint Holes.dwg Layout: Layout Date: 09-03-2019 Login: sac



Not to Scale

NOTATION

γ_T = Total Unit Weight of Soil Above Groundwater Level, γ_T = 125 pcf

γ_B = Buoyant Unit Weight of Soil Below Groundwater Level, γ_B = 62.6 pcf

γ_W = Unit Weight of Water, γ_W = 62.4 pcf (Fresh Water)

D = Outside Diameter of Pipe, Ft.

A = Area of Enclosed Pipe Below Groundwater, Ft^2. May Not Be Same Shape as Shown

A_{TOP} = Plan View Area of Pipe, DUnit Length Ft^2

Z = Depth to Top of Pipe

H_1 = Depth to Groundwater = 5 Feet for Land-based Pipes and 0 Feet for In-Water Pipes

H_2 = Depth from Groundwater to Base of Pipe

$\overline{\nabla}$ = Groundwater Level

EQUATIONS

$$\text{Factor of Safety} = \frac{W_P + W_{SP}}{F_B}$$

W_P = Weight of Pipe, lb/lft

W_{SP} = Weight of Soil Above Pipe, lb/lft

$= [\gamma_T H_1 + \gamma_B (Z - H_1)] A_{Top}$

F_B = Buoyant Force, lb/lft

$= A \gamma_W$

NOTE

Permanent tiedowns could also be used to resist uplift.

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King County, Washington

BURIED PIPE
UPLIFT RESISTANCE

September 2019 21-1-22000-213

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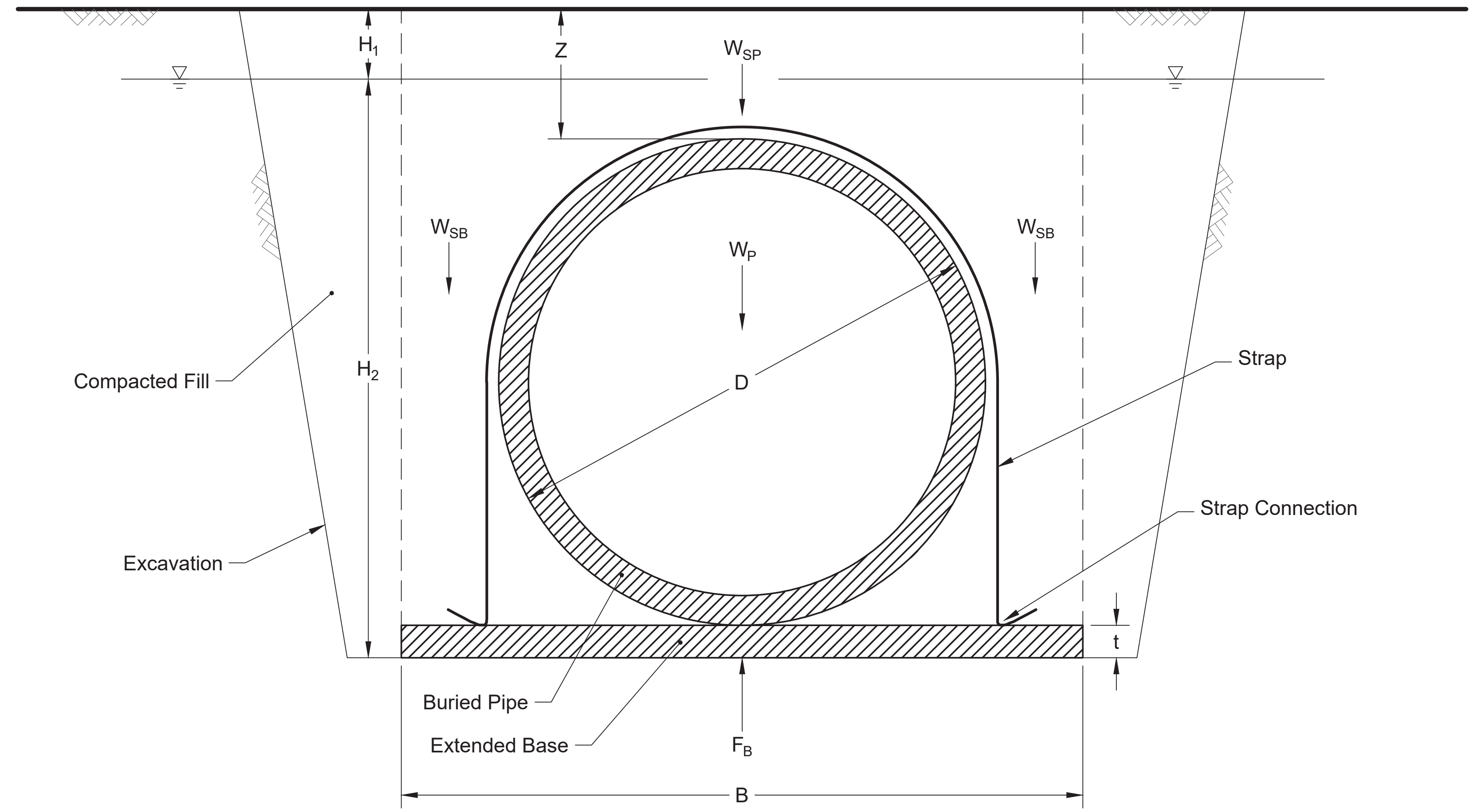
FIG. 13

Filename: J:\211\22000\213\21-1-22000-213 Uplift Resistance Buried Pipe.dwg Layout: Layout Date: 06-07-2018 Login: SAC



Not to Scale

EQUATIONS

$$\text{Factor of Safety with Extended Base} = \frac{W_P + W_{SP} + W_{SB}}{F_B}$$

W_P = Weight of Pipe and Extended Base, lb/lft

W_{SP} = Weight of Soil Above Pipe, lb/lft

= $[\gamma_T H_1 + \gamma_B (Z - H_1)] A_{Top}$

W_{SB} = Weight of Soil Above Extended Base, lb/lft

= $[\gamma_T H_1 + \gamma_B (Z - t)] (B-D)$

F_B = Buoyant Force, lb/lft

= $A \gamma_W$

NOTATION

γ_T = Total Unit Weight of Soil Above Groundwater Level, γ_T = 125 pcf

γ_B = Buoyant Unit Weight of Soil Below Groundwater Level, γ_B = 62.6 pcf

γ_W = Unit Weight of Water, γ_W = 62.4 pcf (Fresh Water)

t = Thickness of Extended Base, Ft.

A = Area of Enclosed Pipe and Extended Base Below Groundwater, Ft^3. May Not Be Same Shape as Shown

Z = Depth to Top of Pipe

H_1 = Depth to Groundwater = 5 Feet for Land-based Pipes and 0 Feet for In-Water Pipes

H_2 = Depth from Groundwater to Base of Pipe

D = Outside Diameter of Pipe, Ft.

B = Length of Extended Base

NOTES

1. Uplift could result in high moments in bottom slab.
2. Strap, strap connection, and strap spacing should be structurally designed.

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North Mercer Island Interceptor and Enatai Interceptor Upgrade Project
King County, Washington

BURIED PIPE UPLIFT RESISTANCE WITH EXTENDED BASE

September2019 21-1-22000-213

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FIG. 14

Filename: J:\211\22000\213\21-1-22000-213 Uplift Resistance Buried Pipe.dwg Layout: with Extended Base Date: 06-07-2018 Login: SAC

APPENDIX A

NUMERICAL ANALYSIS FOR WSDOT WALL W-6

Draft

APPENDIX A

NUMERICAL ANALYSIS FOR WSDOT WALL W-6

TABLE OF CONTENTS

TECHNICAL MEMORANDUM

Technical Memorandum from Shannon & Wilson, Inc., to Tetra Tech, Inc., 8/09/2019 (21 pages) Revised Numerical Analysis for the WSDOT Wall W-6 Undercrossing, North Mercer Island and Enatai Interceptor Upgrade Project, Bellevue, Washington

Draft

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

ALASKA
CALIFORNIA
COLORADO
FLORIDA
MISSOURI
OREGON
WASHINGTON

TECHNICAL MEMORANDUM

TO: Grizelda Sarria, Tetra Tech, Inc.

c: David Scott, Tetra Tech, Inc.
James Chae, Jacobs Civil

FROM: Michael Kucker, Shannon & Wilson, Inc.
Hollie Ellis, Shannon & Wilson, Inc.

DATE: August 8, 2019

RE: REVISED NUMERICAL ANALYSIS FOR THE WSDOT WALL W-6 UNDERCROSSING, NORTH MERCER ISLAND AND ENATAI INTERCEPTOR UPGRADE PROJECT, BELLEVUE, WASHINGTON

This technical memorandum (TM) describes the results of our revised numerical analysis for the pipeline undercrossing of the Washington State Department of Transportation (WSDOT) Wall W-6, located along the east side of the Interstate 90 (I-90) on/off ramps (Bellevue Way SE) in Bellevue, Washington (Figure 1). A previous numerical analysis had been conducted for the wall and was documented in our September 26, 2018 TM. Recently, additional geotechnical information was made available which changed the interpreted geology beneath the wall. An outwash layer, which had been encountered to the east of the wall, was found to extend further west and beneath the wall. The revised numerical analysis and this TM were conducted and prepared in general accordance with our scope of work under Task 3210, Unplanned Geotechnical Explorations and Testing, dated August 23, 2018.

In summary, the revised numerical analysis presented in this TM is similar to our previous September 26, 2018 analysis, except the revised model includes a 10-foot thick layer of very dense glacial outwash between elevation 75 and 85 feet or about 14 feet below the tips of the piles. Groundwater in the outwash layer is confined with a hydrostatic head of about 0.5 feet above the ground surface (elevation 124.5 feet). The revised numerical analysis assumed the same pipeline alignment and the same pipe diameter (36-inch) and borehole size (48 inches) as used in the previous analysis. The results of the revised numerical analysis are very similar to the previous analysis with ground settlement at the pile tips of about 0.02-inch or less. We also note, however, that the final design will include a smaller pipe (32-inch diameter) and borehole. We anticpate that the results for the smaller pipe and borehole will be similar or less than the results for the 36-inch pipe and 48-inch borehole.

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PROJECT DESCRIPTION

The North Mercer Island and Enatai Interceptor Upgrade project includes the installation of about 3,000 feet of pipeline beneath the Enatai hill using horizontal directional drilling (HDD) methods. On the east end of the proposed alignment, the pipeline will be constructed beneath the I-90 on/off ramps and an existing WSDOT retaining wall, designated as Wall W-6. A plan and profile along the proposed pipeline alignment is shown in Figure 2. The vertical datum for the project is the King County (KC) Metro datum. The KC Metro datum is 100 feet above the National Geodetic Vertical Datum of 1929 and 96.41 feet above the National Geodetic Vertical Datum of 1988.

Based on the as-built drawings, Wall W-6 is a conventional reinforced concrete retaining wall. At the proposed undercrossing (Wall W-6 Sta. 3+55), the wall is approximately 23 feet high with a footing thickness of 2 feet and width of 14 to 15 feet. The wall is supported on 14-inch diameter augercast piles at a spacing of 4.75 feet along the wall. In section, there are three augercast piles, two battered at 4 inches in 12 inches near the toe of the wall footing and one vertical pile near the heel of the wall footing. Based on the pile installation records, the augercast piles were all installed to a final tip elevation of about 99 feet. A plan and section showing the pile layout and depth is shown in Figure 3.

The subsurface soil conditions at the site are based on four existing borings, designated as BH-2, BH-5, NME-25, and E320-B-14B, which are located in the vicinity of the wall. Based on these borings, the subsurface conditions below the base of the wall footing consist of 6 feet of loose to medium dense fill over 8 feet of loose to medium dense alluvium. Underlying the alluvium is 16 feet of very dense glacial till and 10 feet of very dense glacial outwash. Underlying the glacial outwash is very dense glacial till to at least elevation 65 feet. Groundwater was encountered at a depth of about 6 feet (elevation 116 feet) in the upper fill and alluvium and confined groundwater, with a hydrostatic head about 0.5 feet above the ground surface (elevation 124.5 feet), was encountered in the glacial outwash. The approximate locations of the existing borings are shown in Figure 2 and the interpreted subsurface soil conditions are shown in Figures 2 and 3.

The pipeline will be installed using HDD methods. This will include the drilling of a pilot hole, assumed to be 10 inches in diameter, using drilling mud to balance the confined groundwater in the outwash layer, to transport the cuttings back to the entry pit, and to help maintain an open bore. Subsequently, the pilot hole is enlarged by back reaming the bore in gradual steps with drilling mud to a bore hole size sufficient to pull the pipe into place. We anticipate that a final bore diameter of 48 inches will be required to pull the 36-inch diameter pipe into place. Based on recommendations from the design team, the numerical analysis described below assumes a post-installation drilling fluid loss of about five percent of the bore volume. Based on the proposed bore path, the crown of the 48-inch diameter bore will be at an elevation of about 81 feet or a minimum of 18 feet below the Wall W-6 pile tips. The proposed bore location in relation to the Wall W-6 piles is shown in section in Figure 3.

NUMERICAL ANALYSES

Soil-Structure Interaction Analyses

A soil-structure interaction (SSI) analysis was completed to estimate settlement of the Wall W-6 piles due to the drilling of the HDD bore beneath the wall and potential ground loss or voids associated with the loss of drilling fluid around the installed pipe. The analysis consisted of finite difference modeling to predict the performance of the piles under static loading conditions. The software used for the finite difference analysis was FLAC2D 8.00 (Itasca, 2019).

An SSI analysis uses a numerical model to evaluate the effects of internal and external loads on the soils and structures at the site. An SSI analysis consists of modeling the site soils and structures in an existing static condition and then modeling the construction events to observe the performance of the existing structure and foundation soils during and after the construction events.

The static soil properties used for the analysis were selected based on the subsurface soil conditions inferred from existing subsurface explorations and test results. The static properties of the Wall W-6 structure and piles were selected based on as-built drawings, pile installation records, and historical photographs.

Model Design and Properties

Geologic Cross Section and Soil Properties

The geologic layering used in the SSI analysis was based on existing subsurface explorations BH-2 (HWA GeoSciences Inc., 2000), BH-5 (HWA GeoSciences Inc., 2003), NME-25 (Shannon & Wilson, 2016), and E320-B-14B (HJH Final Design Partners, 2013). Based on these existing explorations, the soils beneath the wall footing consist of about 14 feet of loose to medium dense fill and alluvium over about 16 feet of very dense glacial till deposits. Beneath the glacial till is 10 feet of very dense outwash deposits over very dense glacial till. The contact elevation between the alluvium and glacial till was interpreted to be 101 feet, which is consistent with the existing explorations and observations described in the pile installation records. The upper and lower contact elevations of 85 and 75 feet for the glacial outwash deposits is consistent with the existing subsurface explorations.

The engineering properties of the soils were developed based on the existing explorations and our experience with similar soils in the general vicinity of the site. The inferred soil properties are presented in Table 1.

Model Meshes

Because two-dimensional models were used for these analyses, two separate model meshes were created. A two-model approach is necessary because the HDD bore is perpendicular to the wall.

The first mesh, identified as Model 1 (Figure 4), is a section perpendicular to the wall. Model 1 was used to estimate the current state of stress in the piles and soils beneath the wall. The second mesh, identified as Model 2 (Figure 5), is a section parallel to the wall at a distance from the wall at approximately the location of the battered pile tips. The results of Model 1 were used to determine the appropriate pile and soil stresses to be used in Model 2. Model 2 was used to estimate settlement at the battered pile tip elevations due to construction of the HDD bore and subsequent loss of drilling fluid (about five percent of the bore volume).

Because it is unknown if the wall footing is in good contact with the underlying soils, a soft soil zone immediately beneath the footing was included in the two models. This approach conservatively causes the wall and backfill load to be primarily supported by the piles.

The vertical faces of the meshes are truncation boundaries (roller boundaries) and are set at a distance from the area of interest to limit interference with the calculation of displacements and stress changes due to static loading. Nodes at the vertical faces are fixed against displacement in the direction normal to the face but are free to move vertically and horizontally in the plane of the face. The bottom boundary of the mesh is also defined as a roller boundary and is fixed against vertical movement.

Constitutive Equations

A constitutive equation describes the stress-strain behavior of a material. FLAC2D provides several constitutive equations for soil and rock materials. FLAC2D also provides beam, cable, shell, pile, geogrid, and liner elements for the structural components of a model. The choice of the soil or rock constitutive equation depends on the objectives and complexity of the model and on the information available to define soil or rock stress-strain behavior.

Existing soils and backfill soils in the models were assigned an M-C constitutive equation that defines a material stress-strain relationship as purely elastic-purely plastic. The M-C equation allows the material to deform elastically under compressive load until its shear limit is reached, at which point the material deforms plastically without further change in stress. Under tensile load, the M-C equation allows the material to deform elastically until its tensile limit is reached, at which point the tensile stress and tension limit are set to zero, preventing further development of tensile stress. The elastic properties specified for the M-C equation are applied in compression and tension.

The material properties that can be specified for the FLAC2D implementation of the M-C equation are mass density, cohesion, angle of internal friction, shear modulus, bulk modulus, tension limit, and dilation angle. Properties default to a value of zero if no value is specified. As discussed previously, the soil properties used in the SSI analyses are presented in Table 1.

The retaining wall and footing were represented with continuum elements and a purely elastic constitutive equation. The FLAC2D purely elastic constitutive equation uses the same elastic properties in compression and tension and allows for unlimited compressive and tensile stresses. The property values required to define the purely elastic constitutive equation are mass density, shear modulus, and bulk modulus. The concrete properties used for the retaining wall and footing are presented in Table 1. It should be noted that due to the age and service of the structure, we assumed cracked concrete properties for the analyses.

The piles were modeled using FLAC's pile element. A pile element is a one-dimensional beam element with the added feature of a user-defined interface between the pile and surrounding soils. The pile beam is defined by its unit weight per foot, cross-section area, perimeter, elasticity, moment of inertia, and pile spacing. The pile-soil interface is defined by stiffness, friction angle, and cohesion in the shear and normal directions. The properties used for the battered and vertical augercast piles in Model 1 and Model 2 are presented in Tables 2 and 3, respectively. Because the piles in this case are augercast concrete piles with steel reinforcement, composite properties based on the relative cross-section area of concrete and steel were used. For the vertical piles, the steel reinforcement consisted of a single 1-inch diameter steel bar installed down the middle of the pile. For the battered piles, steel reinforcement cages were installed in the piles. Details of the steel reinforcement cages were not documented in the pile installation records and, consequently, we had to review historical photographs of the installations. Based on the review of these photographs, we assumed that the cages consisted of six, evenly spaced, number 4 bars with a spiral.

Groundwater

The FLAC software uses an effective stress analysis method; therefore, groundwater was included in the two models. The groundwater surface in the fill and alluvium above the glacial till is at approximately elevation 116 feet and the confined groundwater head in the glacial outwash layer is at elevation 124.5 feet.

Analytical Steps

Existing Condition

The first step of an SSI analysis is to develop the model with a stress state that approximates existing conditions in the soil and structures, if any. For Model 1, this consisted of establishing a stress state that approximates the stress state in the soil, retaining wall, backfill, and piles. The existing stress conditions were estimated by the following steps:

1. Estimate soil stress before construction of the wall. This was accomplished by defining the existing soil layers and groundwater conditions in Model 1 and running the model to static equilibrium.

2. Install the piles. Pile elements were entered in the model and the model was run to static equilibrium.

3. Construct footing and wall. Material zones representing the footing and wall were entered in the model and the model was run to static equilibrium.

4. Backfill behind the wall. Material zones representing the backfill were entered in the model in twelve increments from the top of the footing to the final grade elevation. The model was run to static equilibrium for each increment.

5. The stresses in the pile and soil at the end of Step 4 are assumed to be the current state of stress.

Model 2 was developed with steps like those of Model 1, except the wall was not included and the fourth step consisted of applying a normal stress to the top of the wall footing. The applied normal stress was selected to yield approximately the same pile and soil stresses in Model 2 that were calculated after Step 4 of Model 1.

Analysis of the HDD Bore

The final step of the numerical analysis was to simulate the effects of the HDD bore and subsequent loss of drilling fluid (about five percent of the bore volume) in the bore using Model 2.

The bore was simulated by removing the soil zones in the 4-foot diameter bore location and applying a normal internal stress to the inside perimeter of the bore. The applied normal internal stress was equal to the unit weight of the drilling mud times the elevation difference between the bore location and the ground surface (about 2,900 psf). The model was then run to static equilibrium to estimate the stress change and deformation that would occur in the surrounding soil and to estimate the settlement that would occur at the elevation of the pile tips.

Subsequent loss of drilling fluid (about five percent of the bore volume) in the bore was simulated by deforming the bore hole above the springline until an approximately five percent reduction of cross section area of the bore hole had been achieved. The deformation was accomplished by applying a small, fixed velocity to the nodes at the perimeter of the bore hole and stepping the model until the target area change had been achieved. The Model 2 mesh used for the loss of drilling fluid is shown in Figure 6.

Numerical Analyses Results

The numerical analyses were conducted to evaluate the effects of the HDD bore and potential loss of drilling fluid around the installed casing. The primary effects evaluated included ground and pile settlement and pile axial and shear loading.

The estimate of ground settlements above the HDD bore after drilling and loss of drilling fluid are shown in Figures 7 and 8. As shown in Figure 7, the model indicates that most of the ground settlement of 0.5 to 2.5 inches occurs within about one foot or less of the bore crown. As shown in Figure 8, the model indicates that ground settlement at the pile tips (elevation of 99 feet) is about 0.02-inch. The estimate of pile settlements after the HDD bore was completed, after subsequent loss of drilling fluid occurred, and total are shown in Figure 9. The maximum total settlement occurs directly above the HDD bore and is about 0.02 inches with approximately 65 percent of the settlement due to drilling fluid loss after construction of the HDD bore.

Figure 10 illustrates the redistribution of pile axial forces due to the drilling of the HDD bore and loss of drilling fluid. As shown in Figure 10, the piles immediately above the HDD bore show a loss in axial force of about 3 percent; whereas, the piles at some distance from the bore show a gain in axial force of about one percent. In general, the piles immediately above the HDD bore exhibit a greater change in forces than those at some distance from the bore because only a few piles immediately above the bore loss axial force; whereas, more piles at distance from the bore are taking up the axial force changes. The model also shows that the axial force on the existing piles, prior to any construction, ranges from about 17 to 19 tons. This loading is about half of the allowable pile capacity of 40 tons and, therefore, the existing factor of safety for the piles is about four. Consequently, the redistribution of axial loading on the adjacent piles, due to the HDD bore and loss of drilling fluid, should not adversely affect the performance of the wall.

If you have any questions or comments regarding this technical memorandum, please contact me at 206-695-6856.

SHANNON & WILSON, INC.

Michael S. Kucker, PE
Vice President

MSK:HLE/msk

Enc: Table 1 – Soil and Concrete Properties
Table 2 – Battered Augercast Pile Properties
Table 3 – Vertical Augercast Pile Properties
Figure 1 – Vicinity Map
Figure 2 – Enatai Siphon HDD Plan and Profile, Sta. 26+00 to Sta. 32+00
Figure 3 – Wall W-6 Pile Layout and Section
Figure 4 – Model 1 Mesh and Layers
Figure 5 – Model 2 Mesh and Layers
Figure 6 – Model 2 Bore Shrinkage
Figure 7 – Model 2 Settlement Above Bore, After HDD Drilling and Drilling Fluid Loss
Figure 8 – Model 2 Settlement Above Bore, After HDD Drilling and Drilling Fluid Loss
Figure 9 – Settlement at Pile Tip Elevation
Figure 10 – Pile Axial Force

Table 1
Soil and Concrete Properties

	Fill	Alluvium	Till	Outwash	Concrete	Concrete (cracked)
Unit Weight, pcf	125	125	130	130	155	155
Unit Weight, slugs	3.88	3.88	4.04	4.04	4.81	4.81
Friction, degrees	28	32	40	36	NA	NA
Cohesion, psf	0	0	0	0	NA	NA
Tension, psf	0	0	0	0	NA	NA
Dilation, degrees	0	0	0	0	NA	NA
Poisson's Ratio	0.35	0.35	0.35	0.35	0.15	0.15
Young's Modulus ($E_{s)}$, psi	18,000	25,000	40,000	30,000	4,000,000	2,000,000
Young's Modulus (E_s), psf	2.59E+06	3.60E+06	5.76E+06	4.32E+06	5.76E+08	2.88E+08
Shear Modulus ($G_{s)}$, psf	9.60E+05	1.33E+06	2.13E+06	1.60E+06	2.50E+08	1.25E+08
Bulk Modulus ($K_{s)}$, psf	2.88E+06	4.00E+06	6.40E+06	4.80E+06	2.74E+08	1.37E+08

Table 2
Battered Augercast Pile Properties

		Steel	Concrete	Composite
	Spacing, feet	NA	NA	4.75
	Diameter, feet (1 #4 bar)	0.04	1.17	1.17
	Perimeter, feet	NA	3.67	3.67
	Area, ft^2 (6 #4 bars)	0.008	1.06	1.07
	Unit Weight, pcf	490	155	157
	Unit Weight, slugs	15.22	4.81	4.89
	Young's Modulus, psi	39,000,000	2,000,000	2,281,060
	Young's Modulus, psf	5.62E+09	2.88E+08	3.28E+08
	Moment of Inertia, ft^4	1.48E-07	9.09E-02	9.09E-02
Footing	Shear Cohesion, psf	NA	NA	1.00E+08
	Shear Friction, degrees	NA	NA	0
	Shear Stiffness, lb/ft	NA	NA	1.00E+07
	Normal Stiffness, lb/ft	NA	NA	1.00E+07
Fill	Shear Cohesion, psf	NA	NA	0
	Shear Friction, degrees	NA	NA	20
	Shear Stiffness, lb/ft	NA	NA	1.00E+05
	Normal Stiffness, lb/ft	NA	NA	1.00E+07
Alluvium	Shear Cohesion, psf	NA	NA	0
	Shear Friction, degrees	NA	NA	20
	Shear Stiffness, lb/ft	NA	NA	1.00E+05
	Normal Stiffness, lb/ft	NA	NA	1.00E+07
Outwash	Shear Cohesion, psf	NA	NA	NA
	Shear Friction, degrees	NA	NA	NA
	Shear Stiffness, lb/ft	NA	NA	NA
	Normal Stiffness, lb/ft	NA	NA	NA
Till	Shear Cohesion, psf	NA	NA	1.00E+05
	Shear Friction, degrees	NA	NA	0
	Shear Stiffness, lb/ft	NA	NA	1.00E+07
	Normal Stiffness, lb/ft	NA	NA	1.00E+07

Table3
Vertical Augercast Pile Properties

		Steel	Concrete	Composite
	Spacing, feet	NA	NA	4.75
	Diameter, feet	0.08	1.17	1.17
	Perimeter, feet	NA	3.67	3.67
	Area, ft^2	0.005	1.06	1.07
	Unit Weight, pcf	490	155	157
	Unit Weight, slugs	15.22	4.81	4.86
	Young's Modulus, psi	39,000,000	2,000,000	2,186,759
	Young's Modulus, psf	5.62E+09	2.88E+08	3.15E+08
	Moment of Inertia, ft^4	2.37E-06	9.09E-02	9.09E-02
Footing	Shear Cohesion, psf	NA	NA	1.00E+08
	Shear Friction, degrees	NA	NA	0
	Shear Stiffness, lb/ft	NA	NA	1.00E+07
	Normal Stiffness, lb/ft	NA	NA	1.00E+07
Fill	Shear Cohesion, psf	NA	NA	0
	Shear Friction, degrees	NA	NA	20
	Shear Stiffness, lb/ft	NA	NA	1.00E+05
	Normal Stiffness, lb/ft	NA	NA	1.00E+07
Alluvium	Shear Cohesion, psf	NA	NA	0
	Shear Friction, degrees	NA	NA	20
	Shear Stiffness, lb/ft	NA	NA	1.00E+05
	Normal Stiffness, lb/ft	NA	NA	1.00E+07
Outwash	Shear Cohesion, psf	NA	NA	NA
	Shear Friction, degrees	NA	NA	NA
	Shear Stiffness, lb/ft	NA	NA	NA
	Normal Stiffness, lb/ft	NA	NA	NA
Till	Shear Cohesion, psf	NA	NA	1.00E+05
	Shear Friction, degrees	NA	NA	0
	Shear Stiffness, lb/ft	NA	NA	1.00E+07
	Normal Stiffness, lb/ft	NA	NA	1.00E+07

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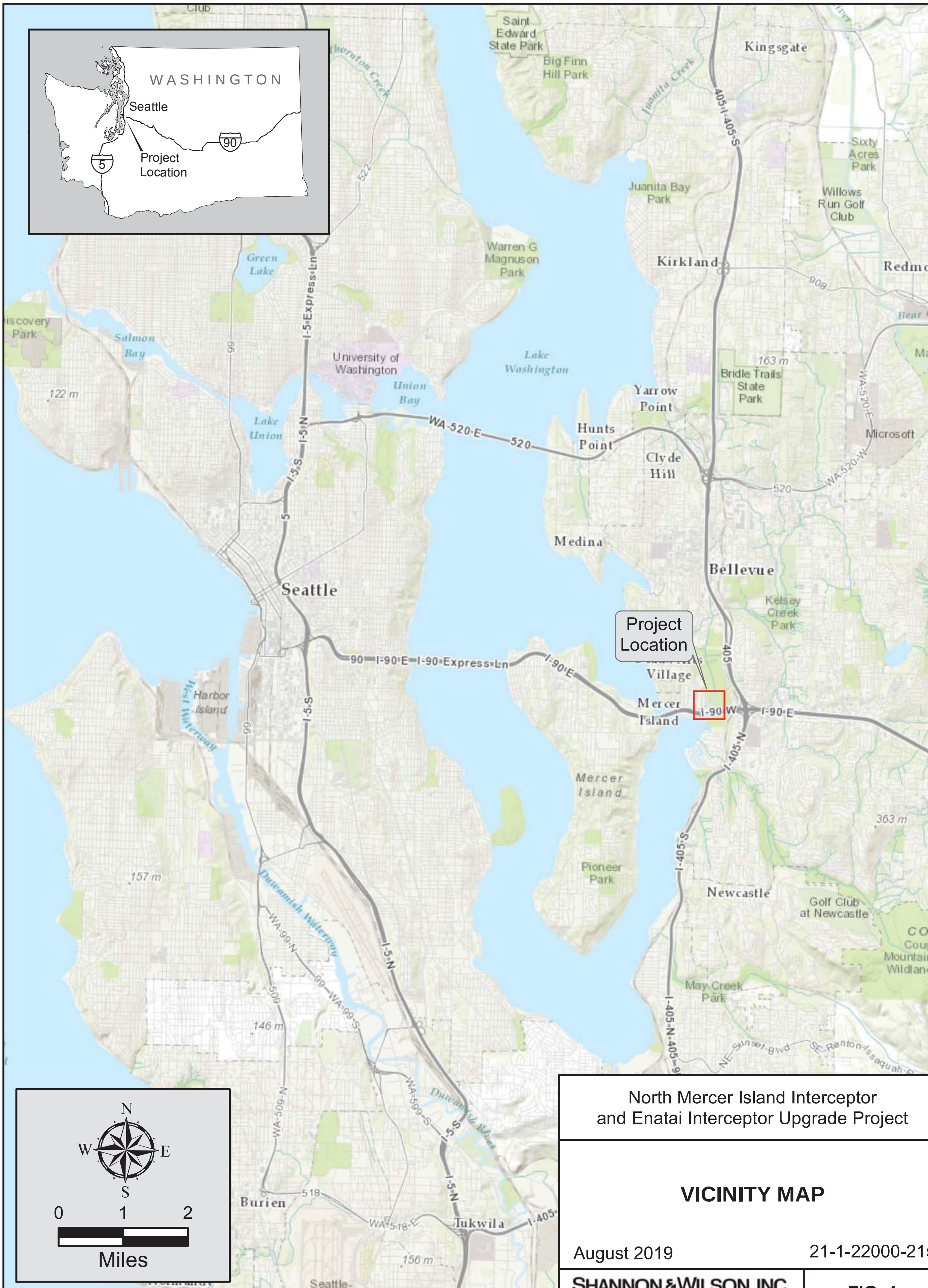

North Mercer Island Interceptor
and Enatai Interceptor Upgrade Project

VICINITY MAP

August 2019

21-1-22000-215

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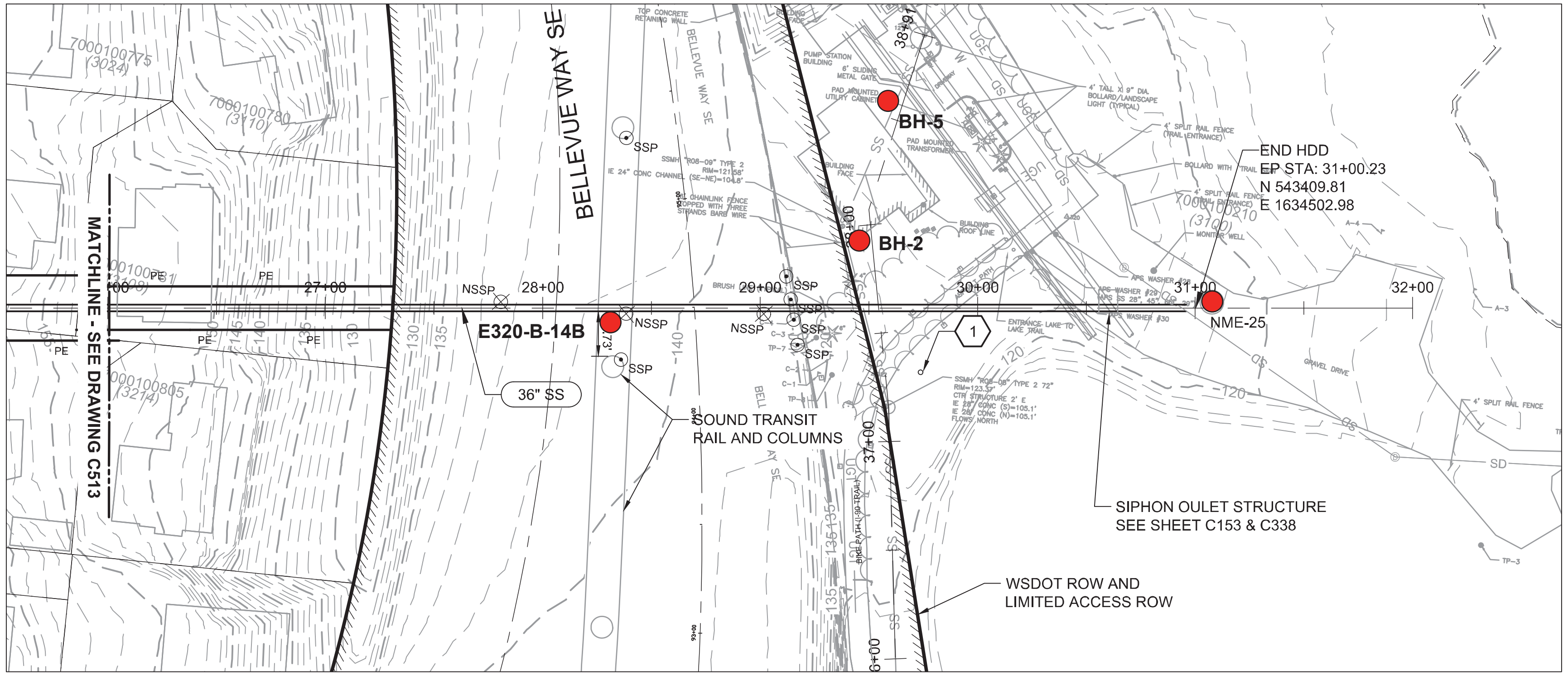
FIG. 1



PLAN

SCALE: 1" = 40'

NOTES:

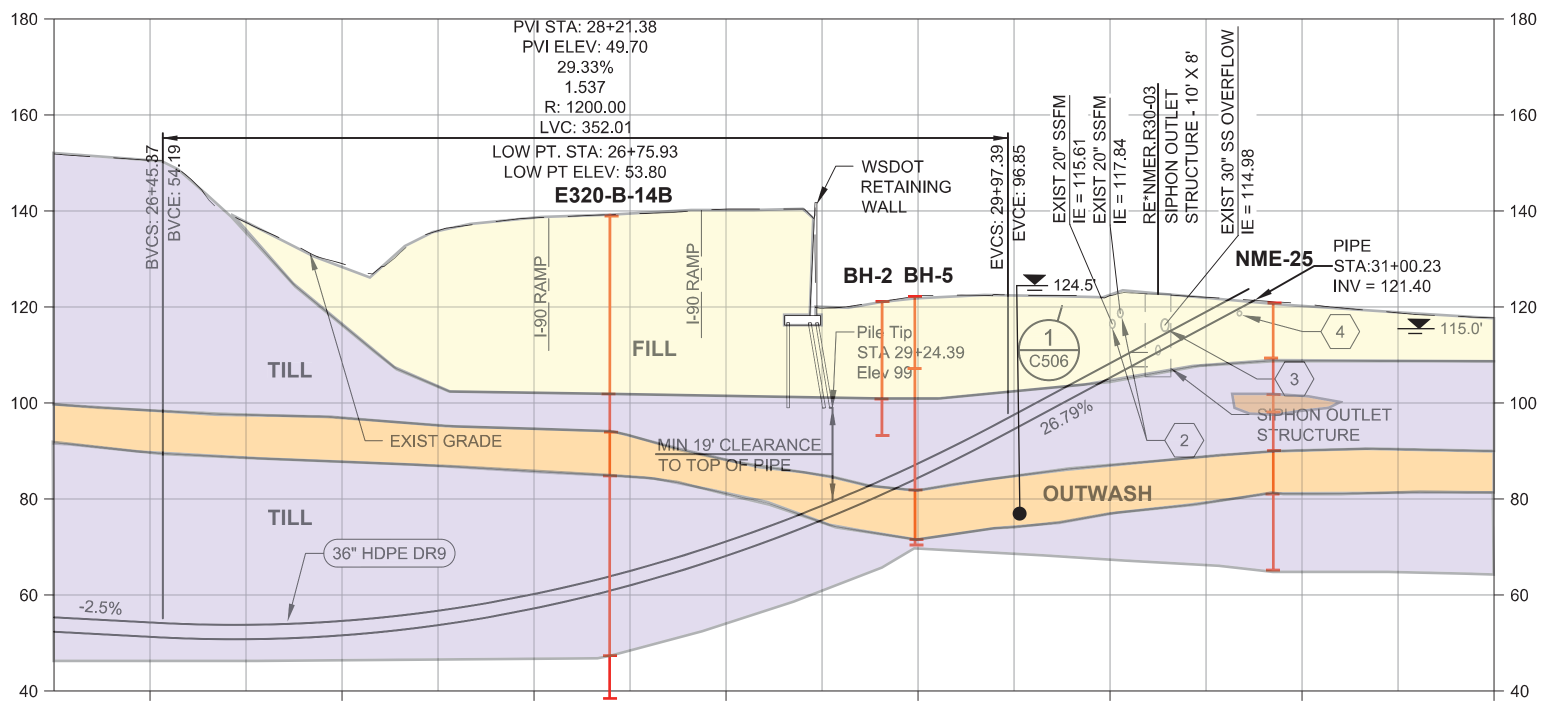
1. PLACE STRUCTURE SETTLEMENT POINTS ON BUILDING CORNERS OF PROPERTIES CONTAINING SUBTERRANEAN EASEMENTS.
2. INSTALL NSSP, SSP, OR VMS SETTLEMENT MONITORING WHERE SYMBOL IS SHOWN ON DRAWING. SEE GEOTECHNICAL INSTRUMENTATION DETAILS DRAWING FOR TYPE OF MONITORING EQUIPMENT REQUIRED.
3. SIPHON OUTLET STRUCTURE IS SHOWN FOR LOCATION COORDINATION AND WILL BE INSTALLED AS A SEPARATE WORK STAGE. SEE SHEETS C238 AND C338.
4. SOUND TRANSIT INFRASTRUCTURE SHOWN ON THE PLAN IS PROPOSED TO BE COMPLETED IN 2019.

KEY NOTES:

1. RELIEF WELL FINAL LOCATION TBD BETWEEN STA 29+60 AND 30+ 40. OFFSET MAXIMUM 20 FEET FROM CENTERLINE FROM HDD ALIGNMENT. RELIEF WELL TO TERMINATE 10 FEET ABOVE CROWN OF PILOT BORE. SEE LOCATION ON SHEET C503 AND DETAIL ON SHEET C506.
2. 20 INCH SS FORCE MAINS TO BE TEMPORARILY SUPPORTED. COORDINATE WITH WORK ON C238.
3. 30 INCH SS OVERFLOW TO BE TEMPORARILY RE-ROUTED DURING HDD WORK AND PERMANENTLY RE-ROUTED AFTER DEMO OF HDD EQUIPMENT. SEE PLAN ON SHEET C338
4. 12 INCH STORM DRAIN TO BE RE-ROUTED DURING HDD WORK AND REPLACED AFTER DEMO OF HDD EQUIPMENT. COORDINATE WITH WORK ON C238.

NOTE TO REVIEWER:

1. STATIONING WILL BE UPDATED AFTER 60% TO BE CONTINUOUS THROUGH ENATAI SIPHON.
2. SETTLEMENT MONITORING POINT NO TO BE ADDED POST 60%.
3. STABILIZING BEAM AND MICROPILES WILL BE ADDED TO EXISTING WSDOT RETAINING WALL DUE TO PROXIMITY OF HDD BORE. CURRENTLY UNDER DISCUSSION WITH WSDOT PERMIT SUBMITTAL. TO BE FINALIZED SUMMER 2018.
4. GRADE BEAM AND MICROPILES FOR MITIGATION OF EXISTING WSDOT RETAINING WALL PILES DUE TO PROXIMITY WITH HDD BOREHOLE IS BEING DISCUSSED WITH WSDOT. DRAWINGS WILL BE UPDATED FOR WSDOT PERMIT SUBMITTAL AFTER DISCUSSIONS IN SUMMER 2018.
5. ADDITIONAL DETAIL WILL BE DEVELOPED FOR TEMPORARY PROTECTION OF EXISTING SEWER AND STORM PRIOR TO 90% DESIGN.



PROFILE

SCALE HORIZ 1" = 40'
VERT 1" = 20'

FIG. 2

0 20 40 60
SCALE IN FEET

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NO	REVISION DESCRIPTION	BY	APVD	DATE

TETRA TECH
JACOBS

PRELIMINARY ISSUE DRAWING
INFORMATION ONLY
60% REVIEW
JUNE 2018

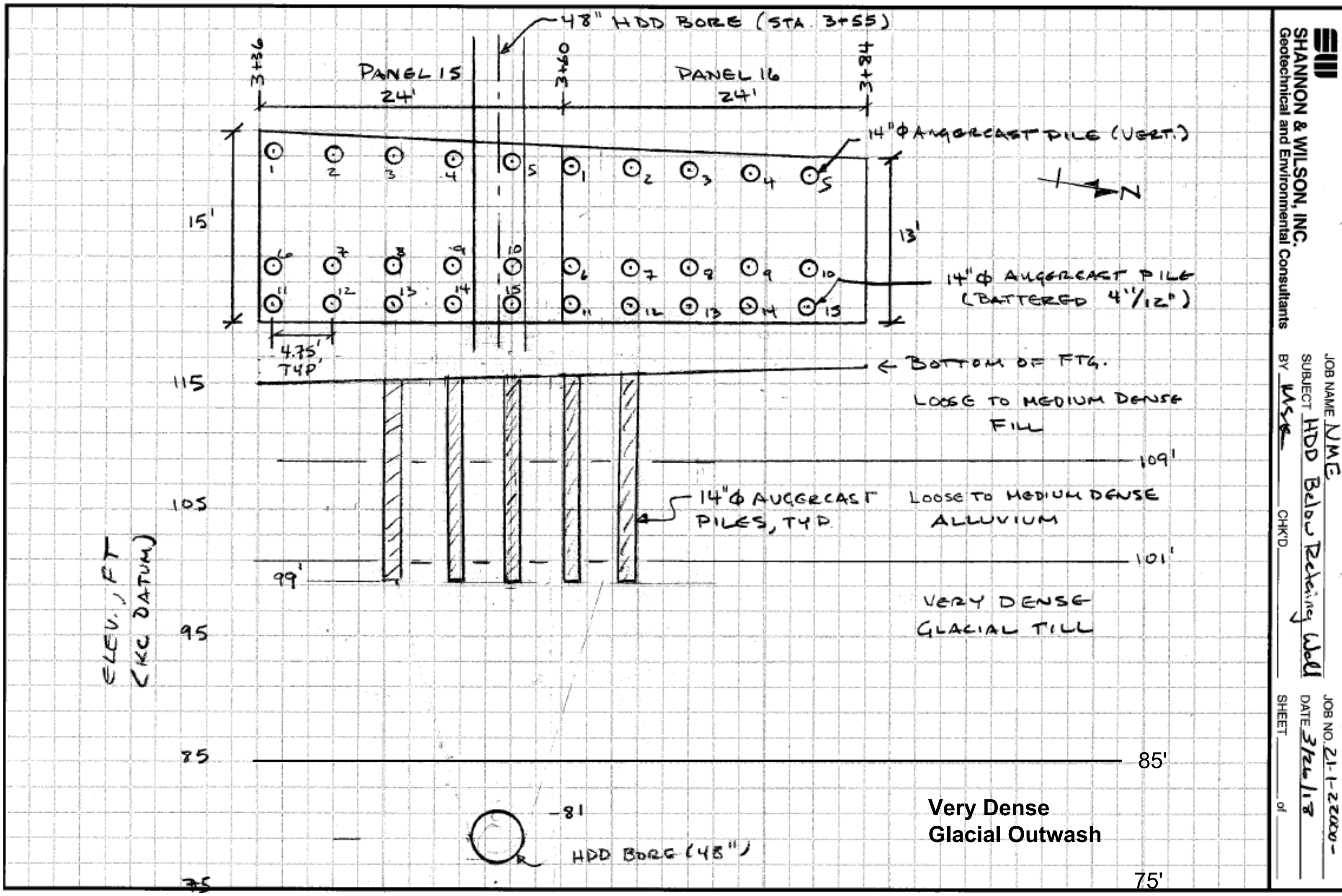
DESIGNED/DRAWN: K.STAHELI	SCALE: AS NOTED		
PROJECT ENGINEER: J. CHAE	REFERENCE 0 1"		
DESIGN APPROVAL: R. BROWNE	FACILITY NUMBER: TBD		
PROJECT ACCEPTANCE: S. YILDIZ	CONTRACT NO: TBD		

King County

DEPARTMENT OF NATURAL RESOURCES & PARKS
WASTEWATER TREATMENT DIVISION
NORTH MERCER ISLAND INTERCEPTOR AND ENATAI INTERCEPTOR UPGRADE

ENATAI SIPHON HDD PLAN AND PROFILE STA 26+00 TO STA 32+00

DCN: TBD
DATE:
PROJECT FILE NO: TBD
DRAWING NO: C514
SHT NO / TOTAL: /
REV NO:



Very Dense
Glacial Till

65'

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

WALL W-6 PILE LAYOUT AND SECTION

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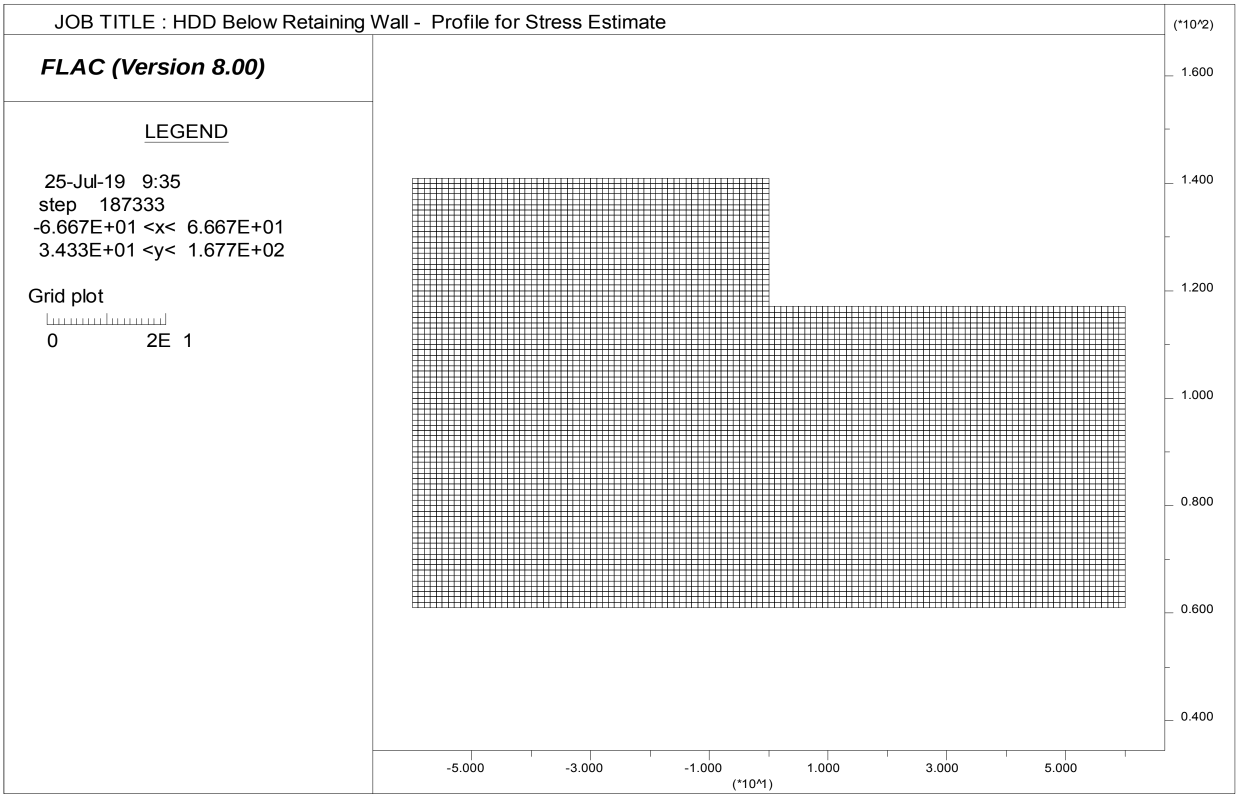
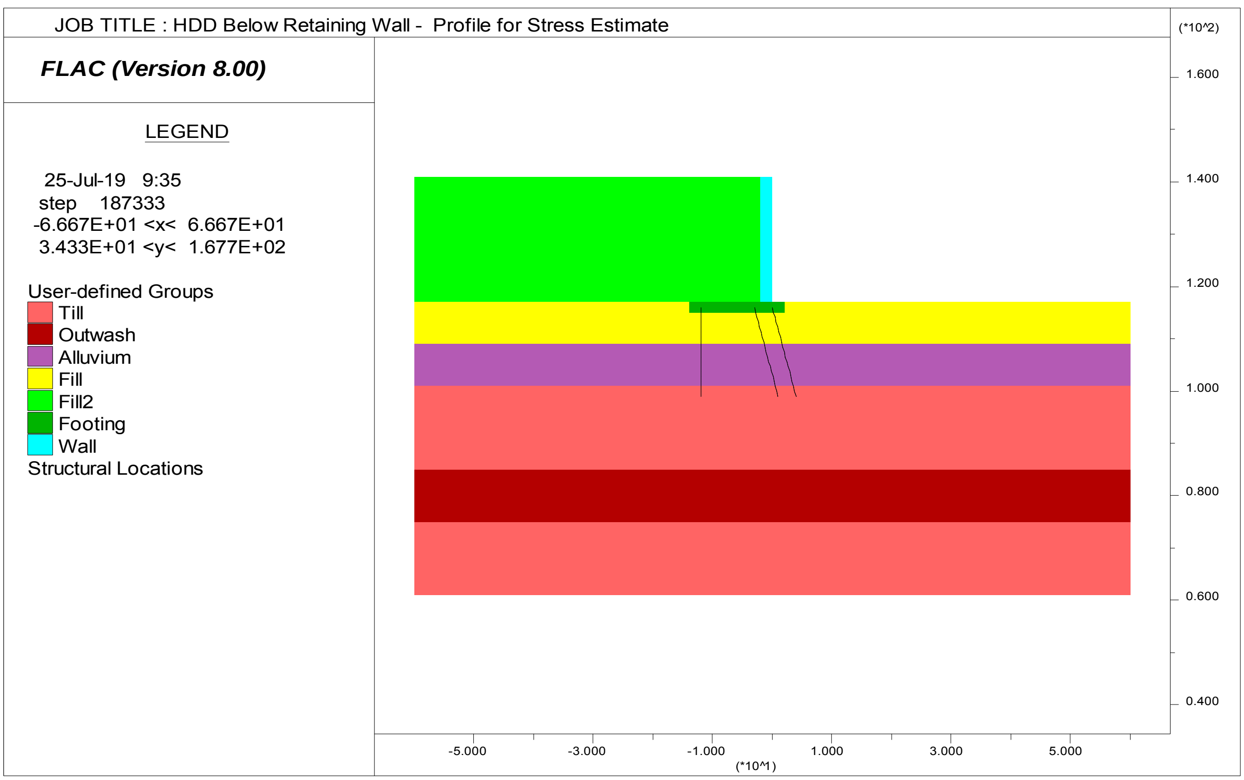
FIG. 3

FIG. 3





North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

MODEL 1 MESH AND LAYERS

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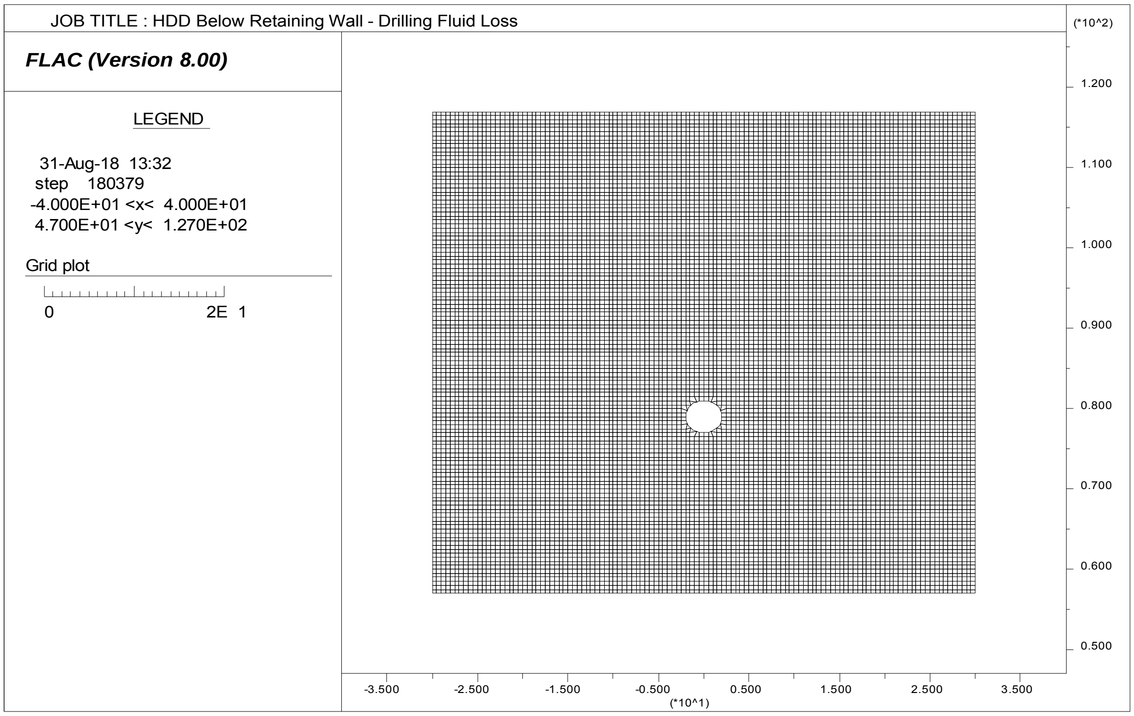
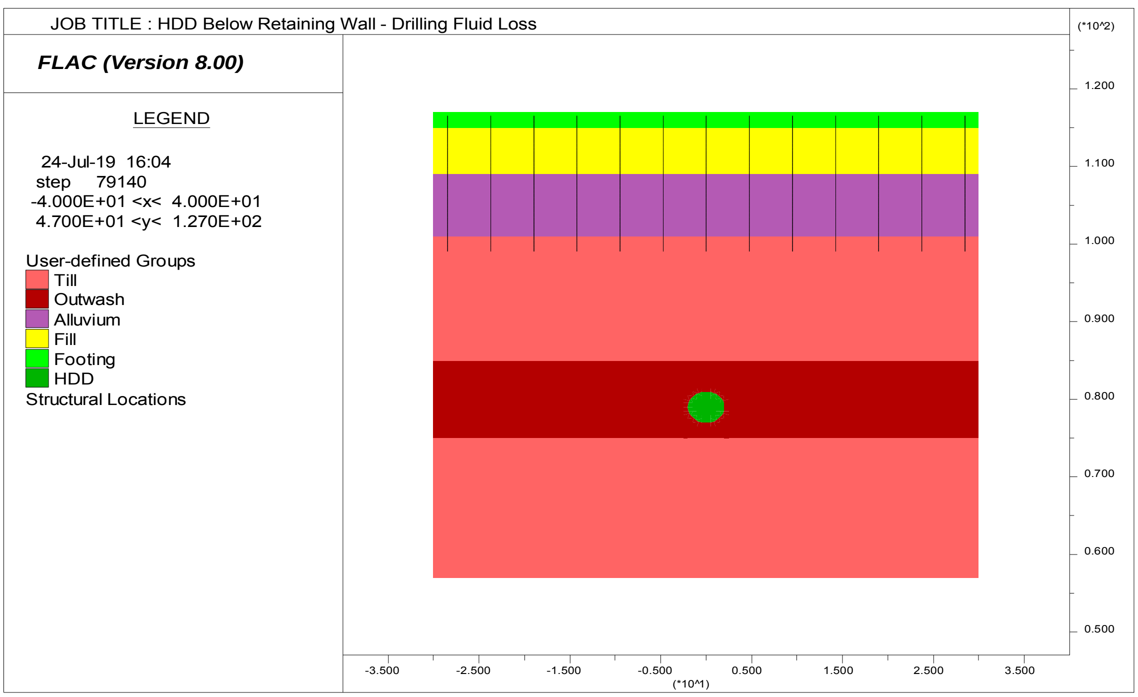
FIG. 4





North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

MODEL 2 MESH AND LAYERS

August 2019 | 21-1-22000-215

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FIG. 5

JOB TITLE : HDD Below Retaining Wall - Drilling Fluid Loss

FLAC (Version 8.00)

LEGEND

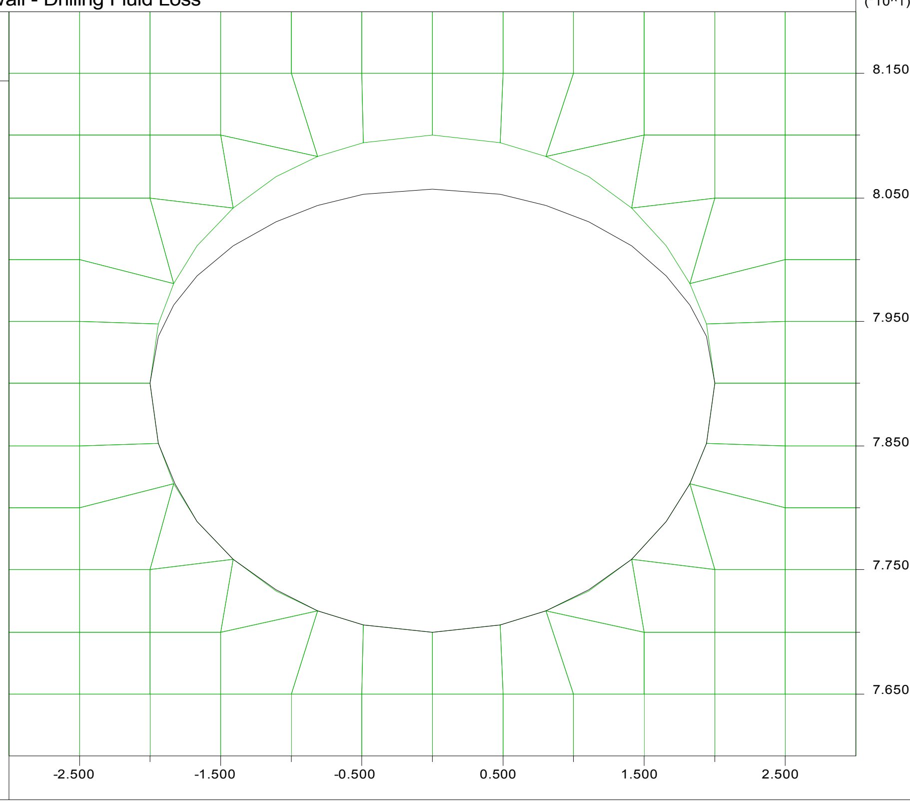
31-Aug-18 13:59
step 180379
-3.000E+00 <x< 3.000E+00
7.600E+01 <y< 8.200E+01

Grid plot

0 1E 0

Exaggerated Boundary Disp.

Magnification = 2.143E+00
Max Disp = 2.000E-01 **feet**



North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

MODEL 2 BORE SHRINKAGE

August 2019 21-1-22000-215

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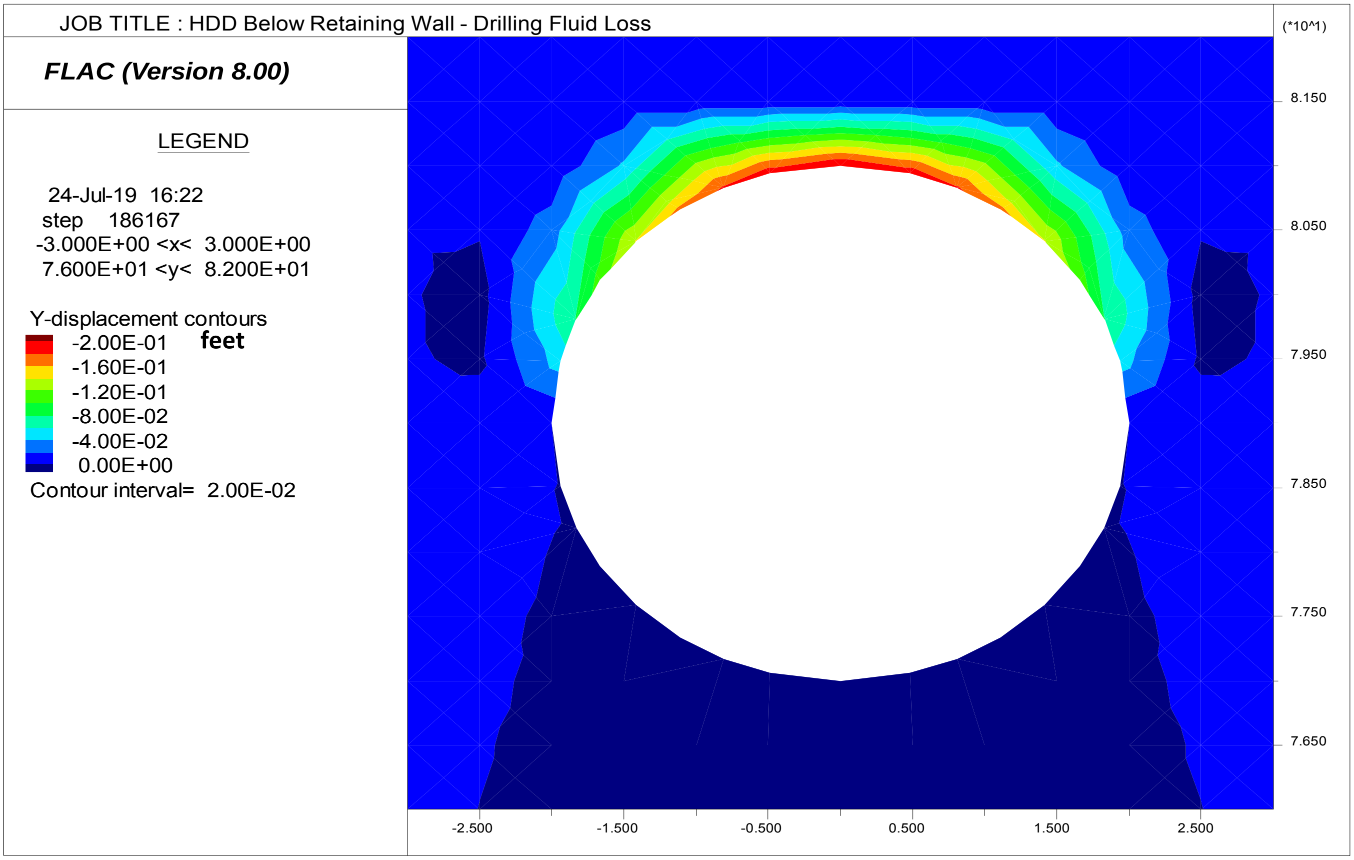
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 6



North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

MODEL 2 SETTLEMENT ABOVE BORE AFTER HDD DRILLING AND DRILLING FLUID LOSS

August 2019 21-1-22000-215

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Geotechnical and Environmental Consultants

FIG. 7

JOB TITLE : HDD Below Retaining Wall - Drilling Fluid Loss

FLAC (Version 8.00)

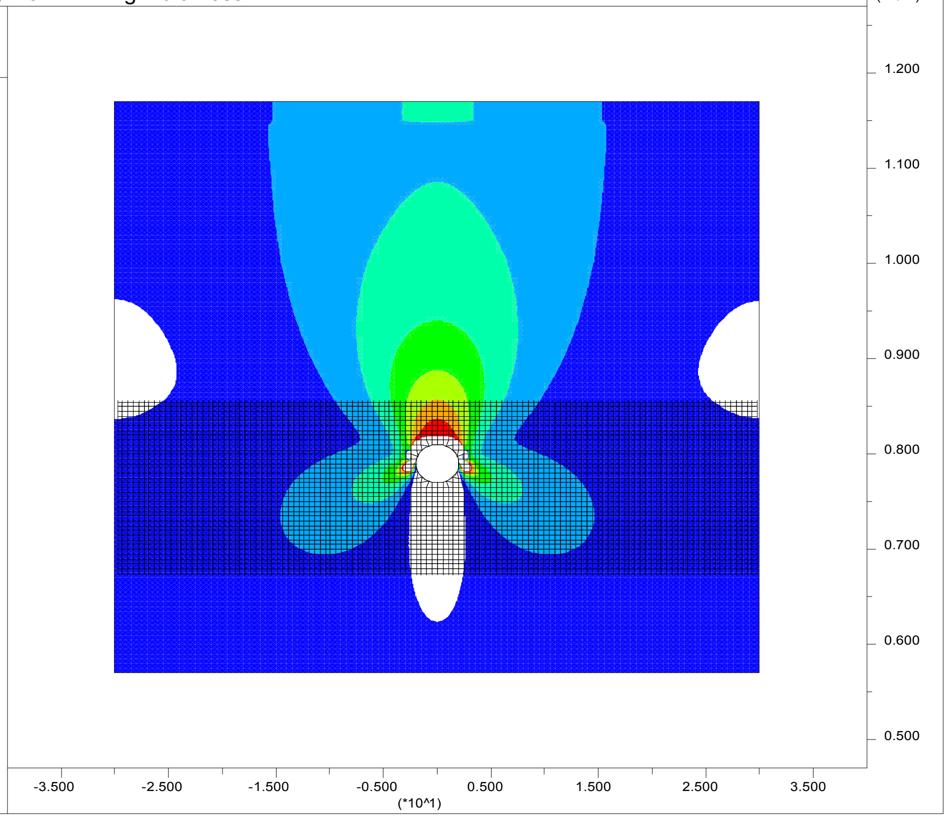
LEGEND

25-Jul-19 9:48
step 186167
-4.000E+01 <x< 4.000E+01
4.700E+01 <y< 1.270E+02

Y-displacement contours
-3.50E-03 **feet**
-3.00E-03
-2.50E-03
-2.00E-03
-1.50E-03
-1.00E-03
-5.00E-04
0.00E+00

Contour interval= 5.00E-04
Boundary plot

0 2E 1



North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

MODEL 2 SETTLEMENT ABOVE BORE AFTER HDD DRILLING AND DRILLING FLUID LOSS

August 2019 | 21-1-22000-215

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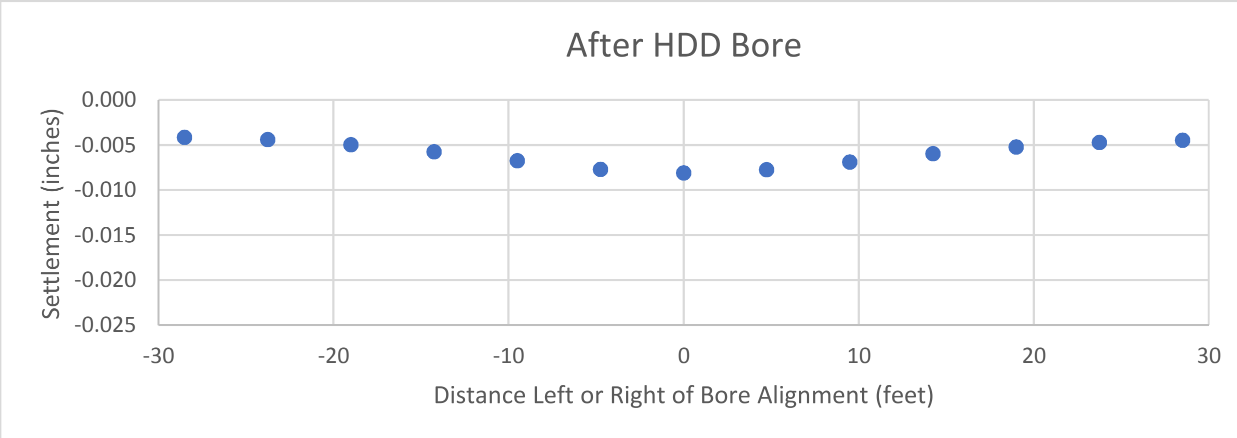
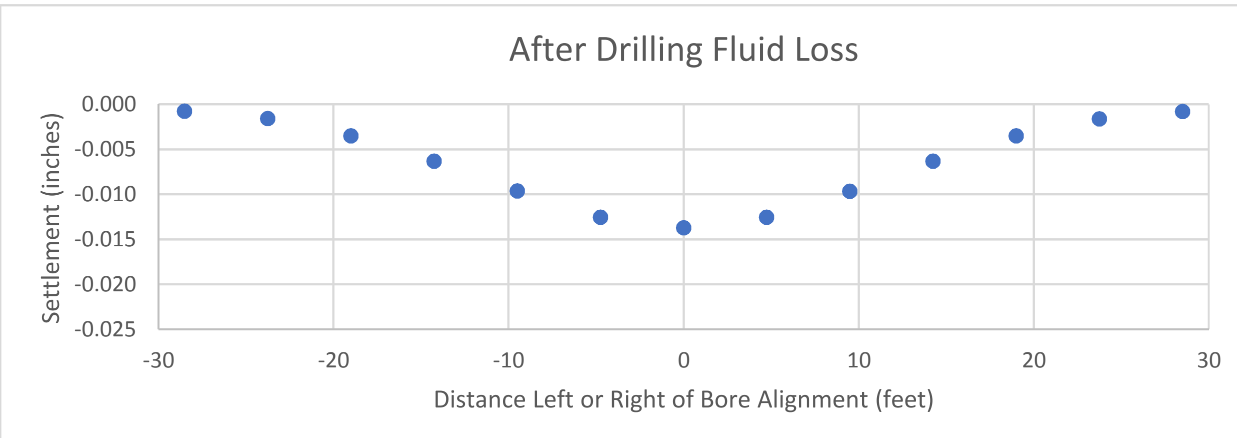
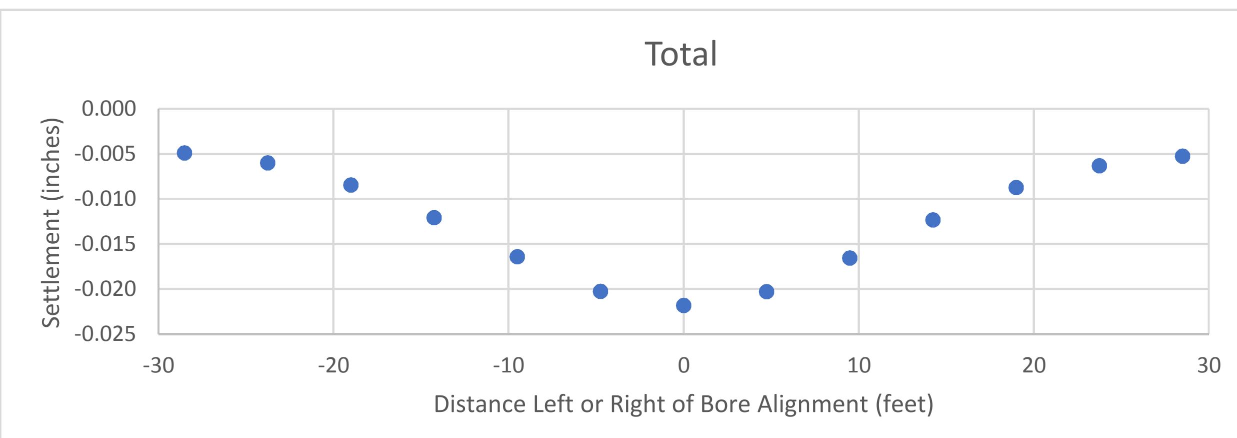
FIG. 8







North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

SETTLEMENT AT PILE TIP ELEVATION

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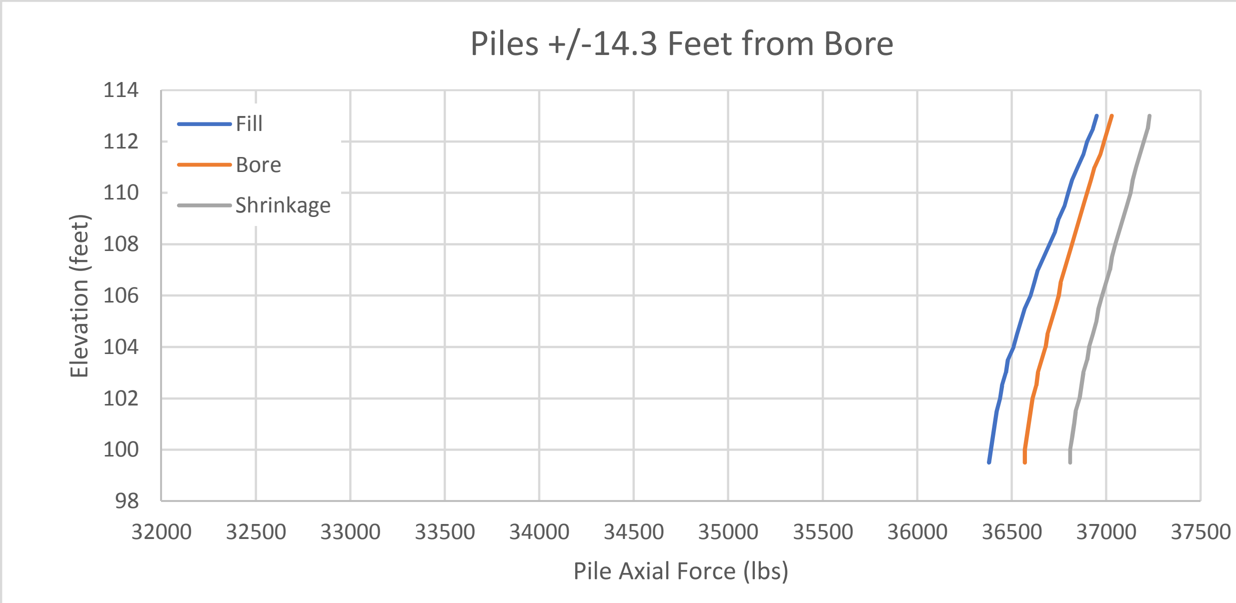
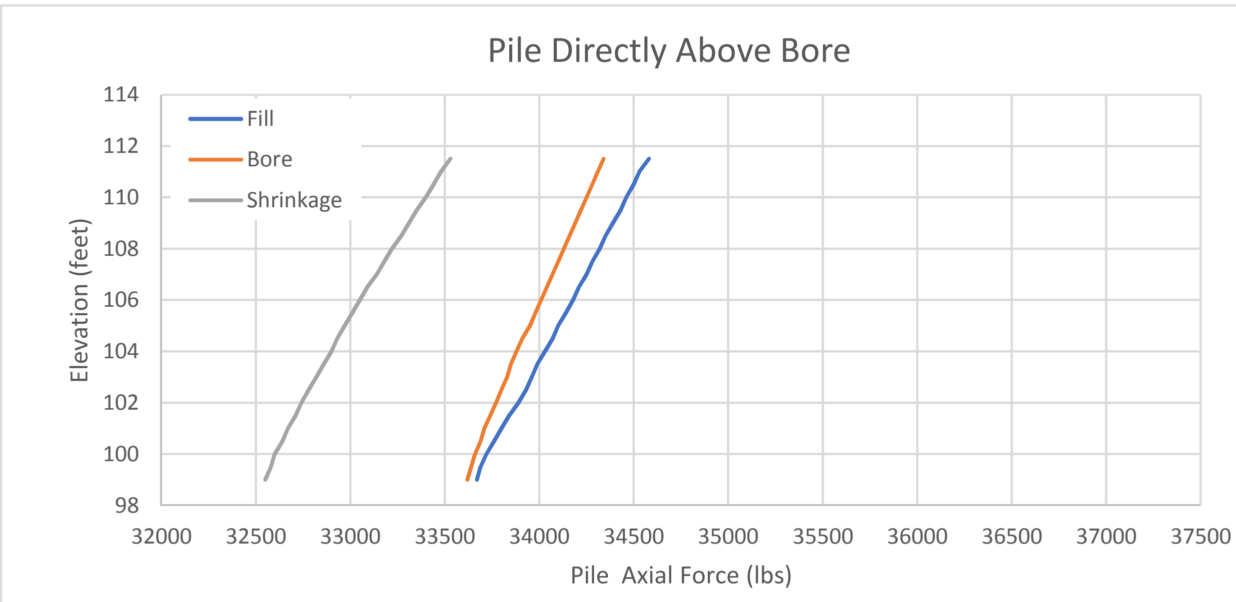
FIG. 9





North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

PILE AXIAL FORCE

August 2019 | 21-1-22000-215

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FIG. 10

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APPENDIX B

ANALYSIS OF HDD ALIGNMENT NEAR I-90 BRIDGE PIERS

APPENDIX B

ANALYSIS OF HDD ALIGNMENT NEAR I-90 BRIDGE PIERS

TABLE OF CONTENTS

MEMORANDUM

Memorandum from Shannon & Wilson, Inc. and Staheli Trenchless Consultants, to King County, 9/28/2018 (18 pages) Geotechnical Analysis of HDD Alignment near I-90 Bridge Piers at Enatai Beach Park

Draft

Subject	**Geotechnical Analysis of HDD Alignment near I-90 Bridge Piers at Enatai Beach Park**	**Project Name**	North Mercer Island Interceptor & Enatai Interceptor Upgrade Project
Attention	Sibel Yildiz, King County	**Project No.**	E00306E11
From	Mike Kucker, PE, Shannon & Wilson, Inc. Kim Staheli, PE, Staheli Trenchless Consultants		
Date	9/28/18		
Copies to	Courtney Schaumberg (King County), Mann-Ling Thibert (King County), Kevin Dour (Tetra Tech), James Chae (Jacobs), David Scott (Tetra Tech)		

1. Purpose of Memorandum

King County is proposing the construction of a 30-inch HDPE pipeline as part of the North Mercer Interceptor and Enatai Interceptor Upgrade Project. The 3,000-foot siphon pipeline will be constructed with horizontal directional drilling (HDD) that will connect the entry point at the King County-owned Sweyolocken Pump Station, to the exit location at the Enatai Beach Park within the Washington State Department of Transport (WSDOT) ROW, north of the Interstate 90 (I-90) bridge. The purpose of this memo is to document the results of the analysis performed to determine if installation of the siphon pipeline with HDD could negatively impact the I-90 Bridge Piers.

2. Pipeline Alignment

The proposed alignment, shown in Attachment 1, was designed to minimize any negative impacts of HDD construction to WSDOT structures, including the I-90 on/off ramps, as well as the I-90 Bridge Piers. As such, the HDD entry will be located at the Sweyolocken Pump Station with the vast majority of the HDD construction operations taking place near the entry. The bore will be constructed from East to West, beneath the Enatai neighborhood hill, with a maximum depth approaching 120 feet. As the bore approaches the exit location, the last 300 feet (approx.) will be within the I-90 ROW, along the north side of the I-90 bridge in the Enatai Beach Park, north of WSDOT Piers 8 and 9. The proposed HDD bore location is approximately 31 feet north and 25.6 feet below the bottom of the Pier 9 footing. The HDD bore will exit the ground prior to entering the influence zone of Pier 8 (Shown in the 8 section). The proposed alignment in relationship to the Pier 9 footing is shown in Attachment 2.

3. GEOTECHNICAL ANALYSIS

A systematic settlement analysis in the soil mass above the HDD was conducted by Staheli Trenchless Consultants, Inc. (STC) to determine the magnitude of ground deformations or settlement above the bore due to the HDD process (see Attachment 9). The conceptual model for the analysis is based upon work performed by Cording and Hansmire (1975) which assumes that the soil surrounding the bore collapses into the annulus between the pipe and the excavation. The maximum volume of collapse is calculated as a percentage of the total annular volume based upon the amount of drilling fluid which remains within the annular space or escapes into the formation. If 100% of the fully hydrated drilling fluid remains in the annulus (the ideal condition), no collapse or settlement will occur, resulting in no ground deformation. The model further assumes that if settlement were to occur, due to the phenomena of soil arching (Terzaghi, 1948) the load above the bore will be distributed in the soil mass and the resulting settlement will form a settlement trough centered above the centerline of the bore. The settlement trough is based on an inverted Gaussian distribution curve, wherein the maximum settlement is assumed to occur directly above the pipe and dissipate based upon the soil stiffness. The maximum settlement is a function of depth, soil strength, annular volume, soil bulking factor, and estimated drilling fluid loss.

Based on the settlement analysis, STC developed a settlement influence zone above the HDD bore that was bounded by inclined planes extending from the spring-line of the bore to the ground surface at an angle of 64 degrees from the horizontal. If settlement occurs above the HDD bore, the soils within the settlement influence zone would expand to fill the volume of ground loss, resulting in a loss of density and a decrease in shear strength and modulus. Although the influence zone does not occur directly beneath the Pier 9 footing, ground deformations and settlement would result in a loosened zone adjacent to the footing that could result in additional settlement and reduction in bearing capacity of the footing. The settlement analysis and settlement influence zone developed by STC was provided to Shannon & Wilson, Inc. to determine how the zone of influence could impact the settlement and bearing capacity of the Pier 9 footing.

The settlement and bearing capacity analyses for the Pier 9 footing included some variations on pipe elevation and lateral location to account for the location tolerance of the horizontal directional drilling installation method, with regard to both the ability to steer the drilling tools in non-homogeneous soils and the accuracy of the HDD locating equipment that varies with the depth of the bore. The steering tolerance for the HDD bore will be specified as +/- 5 feet from the design centerline and will be established during drilling of the pilot bore (the first phase of drilling when all steering operations take place). Once the pilot bore is established, all subsequent reaming phases follow the general alignment established by the pilot bore. The accuracy of most HDD steering tools that are used for installation depths beyond 40 feet vary with depth and are accurate to approximately 2% of the depth (i.e. the guidance system is accurate by +/- 2 feet at a depth of 100 feet).

To evaluate the effects of the HDD and possible imposed settlement of the Pier 9 footing, a commercially available finite-element program, SIGMA/W from GEO-SLOPE International Ltd, was used. The subsurface conditions were modeled based on an existing WSDOT boring, designated as H-42, which was conducted near the footing location. The log for boring H-42 is provided in Attachment 3. Based on boring H-42, the subsurface conditions consist of 5 feet of medium dense fill over 5 feet of very dense recessional outwash. Underlying the recessional outwash is very dense advanced outwash to a depth of more than 40 feet. The soil properties used in the model for each soil unit, including the soils in the settlement influence zone, are presented in Attachment 4. Groundwater was modeled at a depth of about 30 feet, which is consistent with nearby borings. Based on as-built information from WSDOT, the footing is modeled to be 25 feet wide by 39 feet long and founded in the very dense advanced outwash at a depth of about 14 feet (elevation 136.12 feet). The allowable design footing pressure, also based on WSDOT as-builts, is 6 tons per square foot or 12,000 pounds per square foot.

Models were initially developed for the existing condition and the proposed siphon alignment, with the influence zone above the HDD bore, as described above. To account for alignment deviations within the specified bore path tolerance, three additional models were developed for the proposed siphon alignment, one that is shifted 5 feet south and 5 feet shallower (T1), one that is shifted 5 feet south (T2), and one that is shifted 5 feet south and 5 feet deeper (T3). The location of the pipeline for each model case is shown in section in Attachment 2. The results of the finite-element analyses are presented in Attachment 4. As shown, the analyses indicate that the estimated settlements beneath the footing are very small, and not uniform due to the proximity of the north side of the 25-ft wide footing that is closer to the bore than the south side. As a result the model predicts settlement for the north side of the footing that is larger than predicted settlement on the south side of the footing. For the proposed siphon alignment including bore path tolerances, the analyses indicate very small settlements of 0.02 to 0.05-inch will occur as a result of the influence zone above the HDD bore, with only 0.01 to 0.04-inch of differential settlement occurring between the north and south sides of the footing. Settlements of these magnitudes are very small, not measurable, and will not negatively impact the Pier 9 footing.

To evaluate a potential reduction in bearing capacity of the Pier 9 footing, a limited equilibrium analysis (Morgenstern-Price) using a commercially available program, SLOPE/W from GEO-SLOPE International Ltd, was used. Similar to the settlement analysis, five models including one for the existing condition, one for the proposed siphon alignment, and three for the bore path tolerance were developed. The same subsurface conditions, soil properties, footing size, footing depth, and allowable design footing pressure as described for the settlement analysis were used. The results of the bearing capacity analyses are presented in Attachments 5 through 9. As shown, the existing Pier 9 footing has a factor of safety (FS) of 3.61 in bearing capacity. By introducing the influence zone over the HDD bore, the FS drops to minimum of 3.29. Since the FS remains well above 3 in bearing capacity, the influence zone above the HDD bore does not appear to have any significant effect on the bearing capacity of the footing.

Based on the results of the bearing capacity analysis, the influence zone above the HDD bore does not appear to have any significant effect on the footing capacity. The estimated settlements of 0.02 to 0.05-inch for the proposed alignment and bore path tolerances are very small and should not adversely affect the footing.

4. CONCLUSIONS

Based on an evaluation of the proposed siphon, the following conclusions related to the I-90 Bridge piers were determined:

- The proposed HDD bore exits the ground prior to entering the influence zone of Pier 8.
- The finite element analysis clearly shows that the estimated settlement of the Pier 9 footing is very small and of a magnitude that could not be practically detected (1/48-inch to 1/24-inch over the width of the footing), and will not negatively impact the footing.
- The limited equilibrium analysis clearly shows that the factor of safety for bearing capacity of the Pier 9 footing remains above 3 and, therefore, the influence zone above the HDD bore does not appear to have any significant effect on the capacity of the footing.
- Our analysis assumed that the Sound Transit light rail loading does not exceed the WSDOT maximum allowable pier footing pressure of 6 tons per square foot. We anticipate that this is the case as the existing footings have not been modified by Sound Transit; however, we will confirm our geotechnical analyses after we receive Sound Transit's light rail loadings.

Attachments

1. HDD Plan and Profiles

2-9 Geotechnical Analysis Figures

2-10 Settlement Analysis

North Mercer Island Interceptor and Enatai Interceptor Upgrade Project

Geotechnical Analysis of HDD Alignment Near I-90 Bridge Piers at Enatai Beach Park

CERTIFICATION

The technical material and data contained in this document were prepared by or under the supervision and direction of the undersigned, whose seal, as a professional engineer licensed to practice as such, is affixed below.

The interpretation of the subsurface conditions and soil properties and the settlement and bearing capacity analyses for the Pier 9 footing were prepared by or prepared under the direct supervision of Michael S. Kucker, PE.

The determination of the zone of influence above the HDD was prepared by or under the direct supervision of Kimberlie Staheli, P.E.

Prepared By

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REGISTERED
PROFESSIONAL ENGINEER
9/26/18

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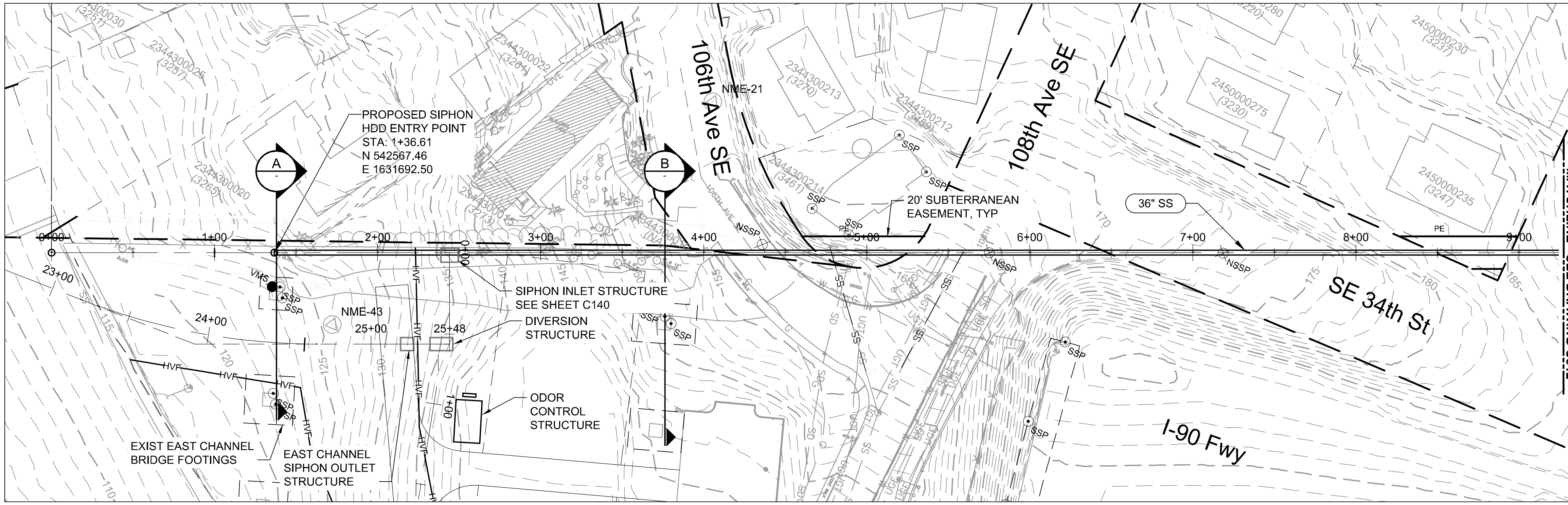
9/26/18

Michael S. Kucker, P.E., Shannon & Wilson, Inc.

Kimberlie Staheli, Ph.D., P.E., Staheli Trenchless Consultants



PLAN

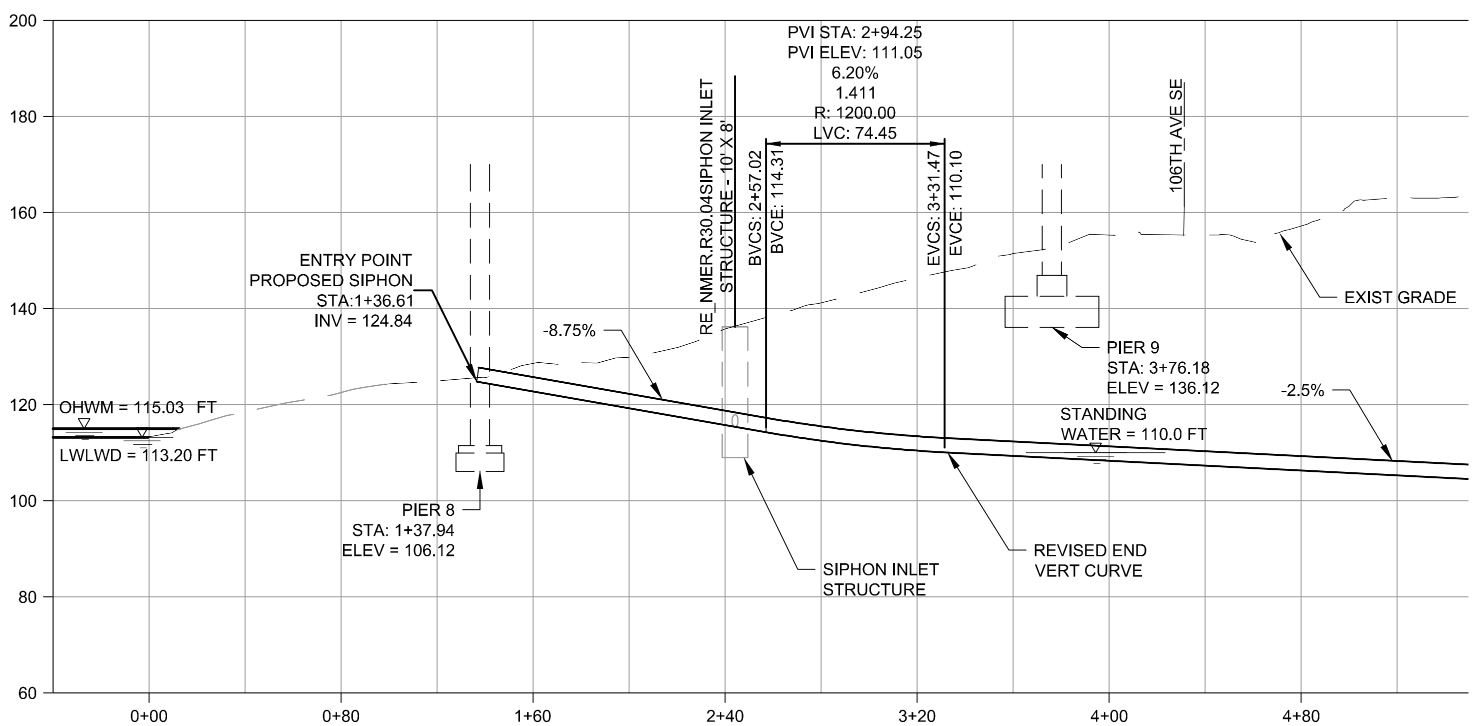
SCALE: 1" = 40'

NOTES:

1. PLACE STRUCTURE SETTLEMENT POINTS ON BUILDING CORNERS OF PROPERTIES CONTAINING SUBTERRANEAN EASEMENTS.
2. INSTALL NSSP, SSP, OR VMS SETTLEMENT MONITORING WHERE SYMBOL IS SHOWN ON DRAWING. SEE GEOTECHNICAL INSTRUMENTATION DETAILS DRAWING FOR TYPE OF MONITORING EQUIPMENT REQUIRED.
3. SIPHON INLET STRUCTURE IS SHOWN FOR LOCATION COORDINATION AND WILL BE INSTALLED DURING A SEPARATE WORK STAGE. SEE SHEETS C236 AND C336.

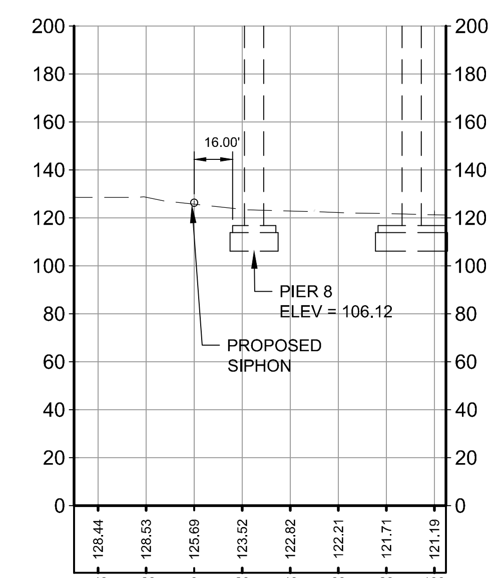
NOTE TO REVIEWER:

1. STATIONING WILL BE UPDATED AFTER 60% TO BE CONTINUOUS THROUGH ENATAI SIPHON.
2. ADDITIONAL SETTLEMENT MONITORING POINTS AT HOUSES WILL BE REVIEWED AND ADDED TO THE PLANS WHERE NECESSARY AFTER 60%.
3. SETTLEMENT MONITORING POINT NUMBERS TO BE ADDED POST 60%.



PROFILE

SCALE HORIZ 1" = 40'
VERT 1" = 20'



SECTION A

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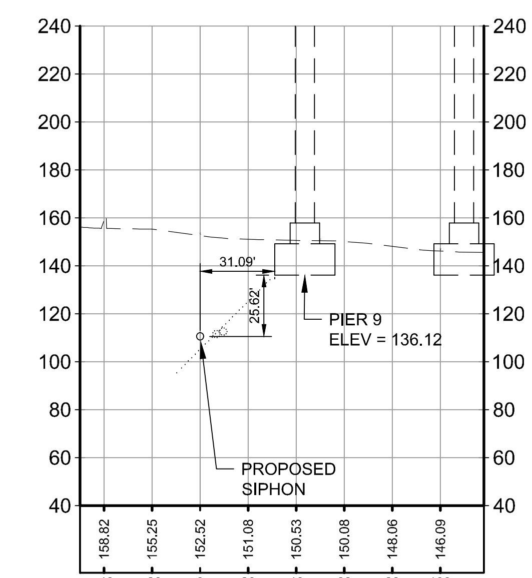

SECTION B

SCALE HORIZ 1" = 40'
VERT 1" = 1'

SCALE IN FEET: 0, 20, 40, 60

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King County

DEPARTMENT OF NATURAL RESOURCES & PARKS
WASTEWATER TREATMENT DIVISION
NORTH MERCER ISLAND INTERCEPTOR AND ENATAI INTERCEPTOR UPGRADE

ENATAI SIPHON HDD - POST 60% PLAN AND PROFILE STA 0+80 TO STA 9+20

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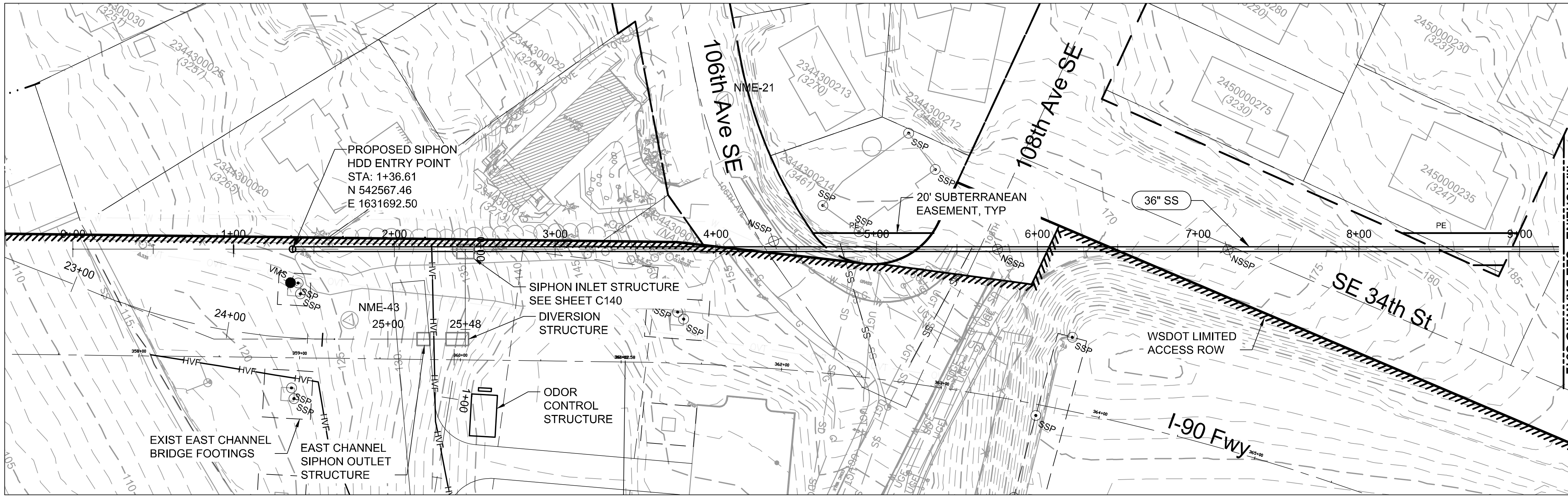

PLAN

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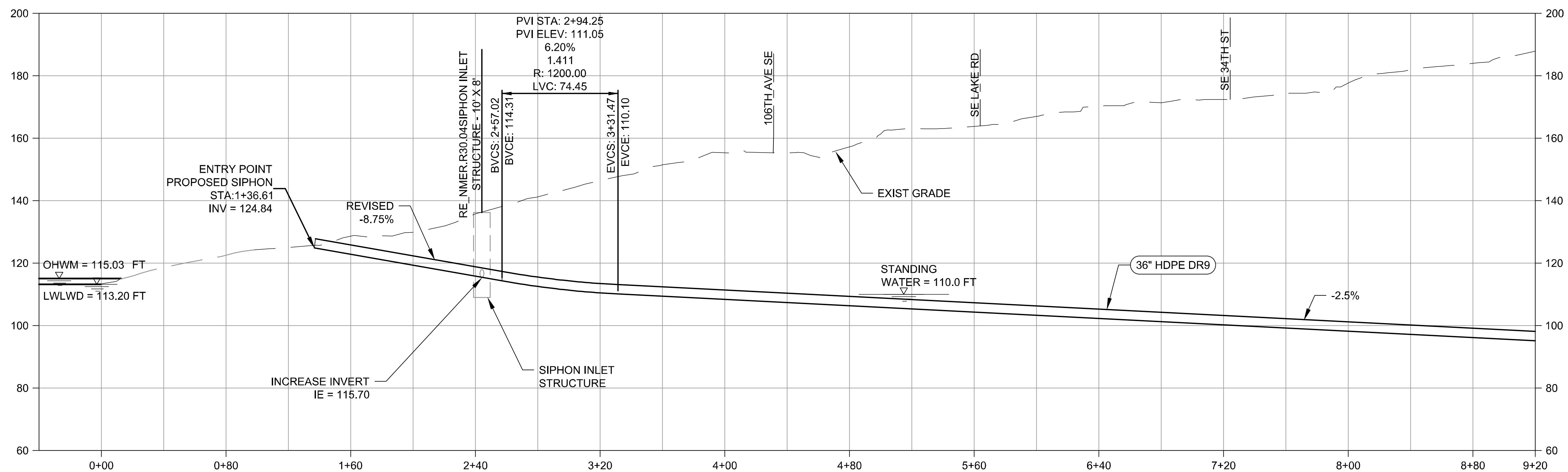

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SCALE HORIZ 1" = 40'
VERT 1" = 20'

SCALE IN FEET 0 20 40 60

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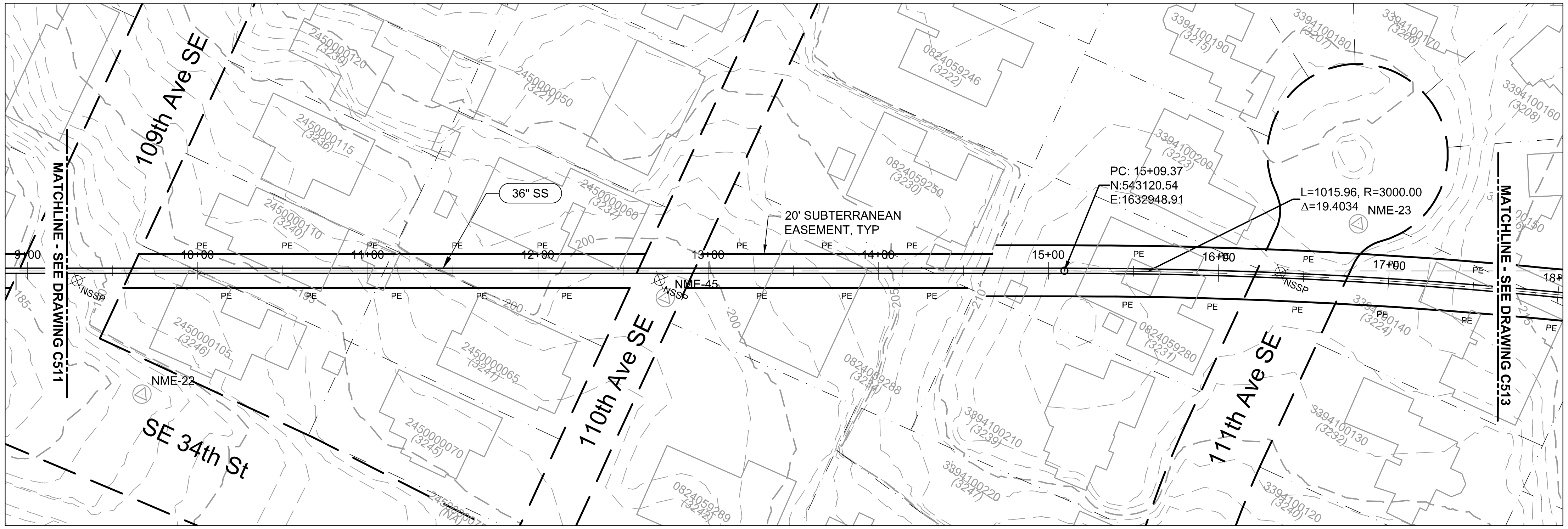

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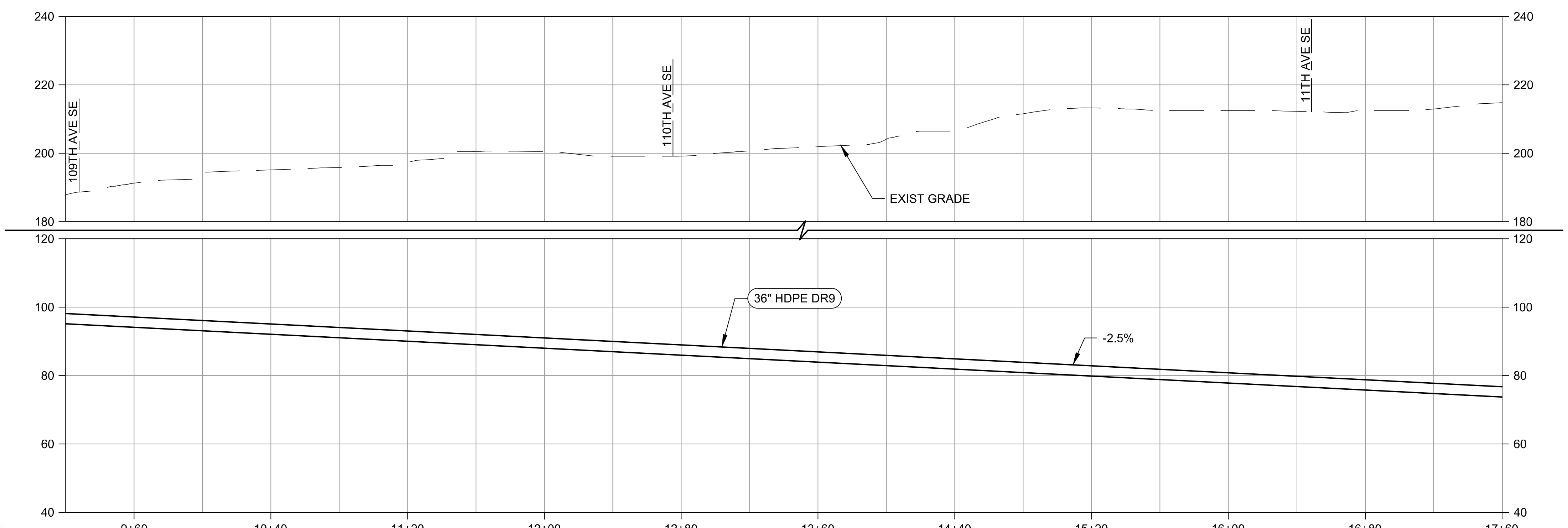

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VERT 1" = 20'

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0 20 40 60
SCALE IN FEET

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ENATAI SIPHON HDD PLAN AND PROFILE STA 9+20 TO STA 17+60

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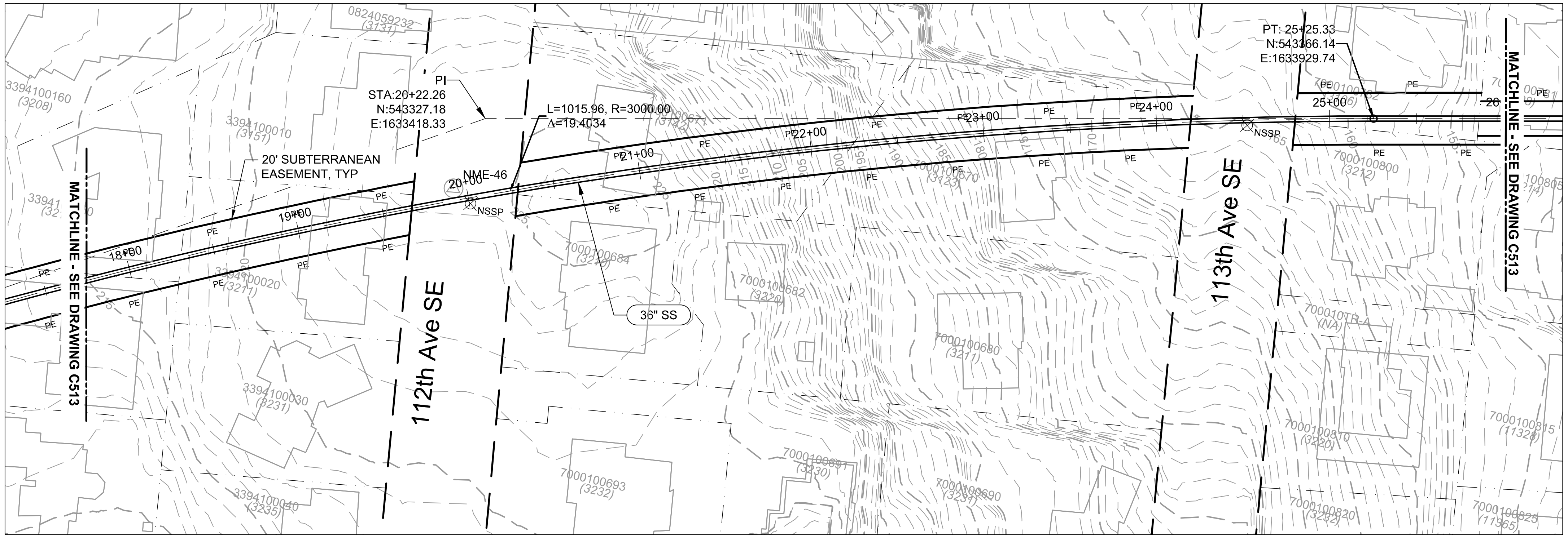

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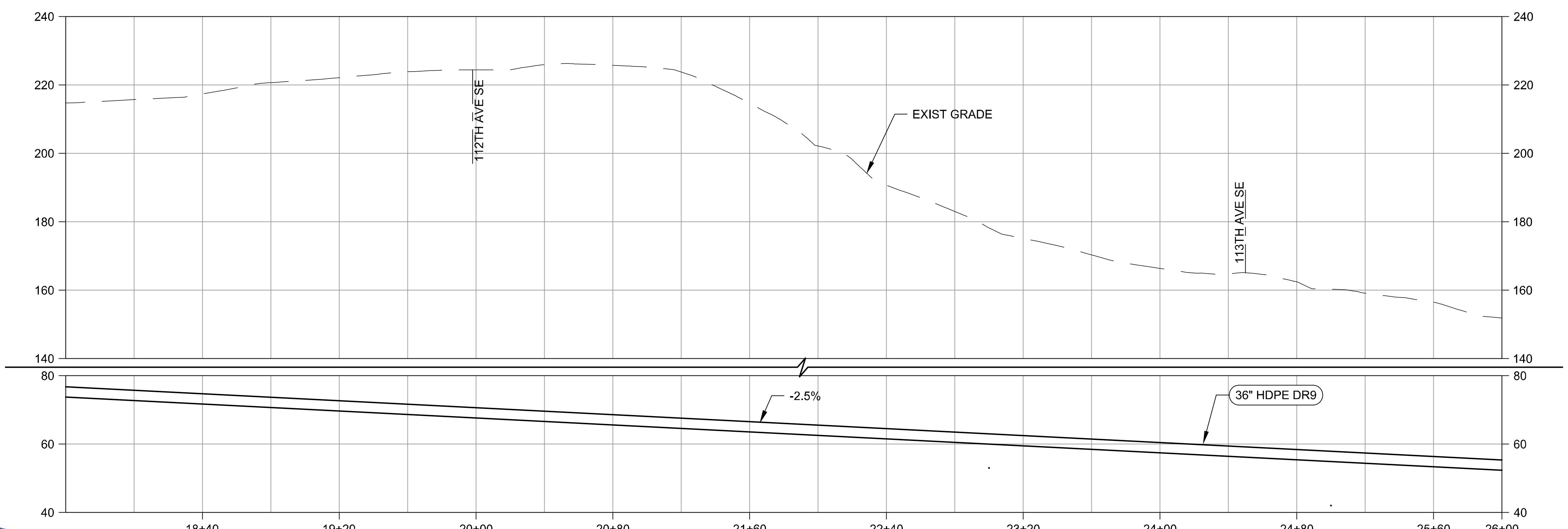

PROFILE

SCALE HORIZ 1" = 40'
VERT 1" = 20'

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0 20 40 60
SCALE IN FEET

STAHELI TRENCHLESS CONSULTANTS

NO	REVISION DESCRIPTION	BY	APVD	DATE

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JUNE 2018

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King County

DEPARTMENT OF NATURAL RESOURCES & PARKS
WASTEWATER TREATMENT DIVISION
NORTH MERCER ISLAND INTERCEPTOR AND ENATAI INTERCEPTOR UPGRADE

ENATAI SIPHON HDD PLAN AND PROFILE STA 17+60 TO STA 26+00

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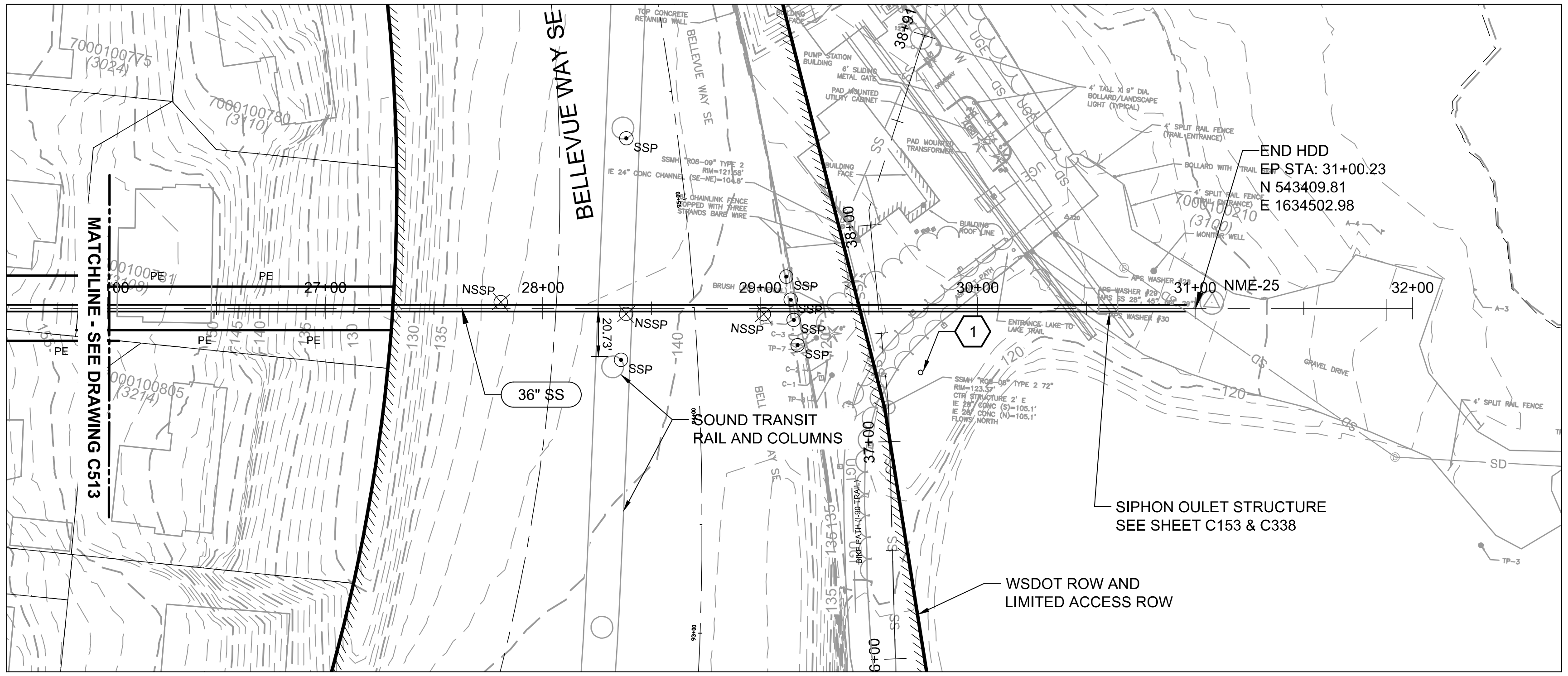

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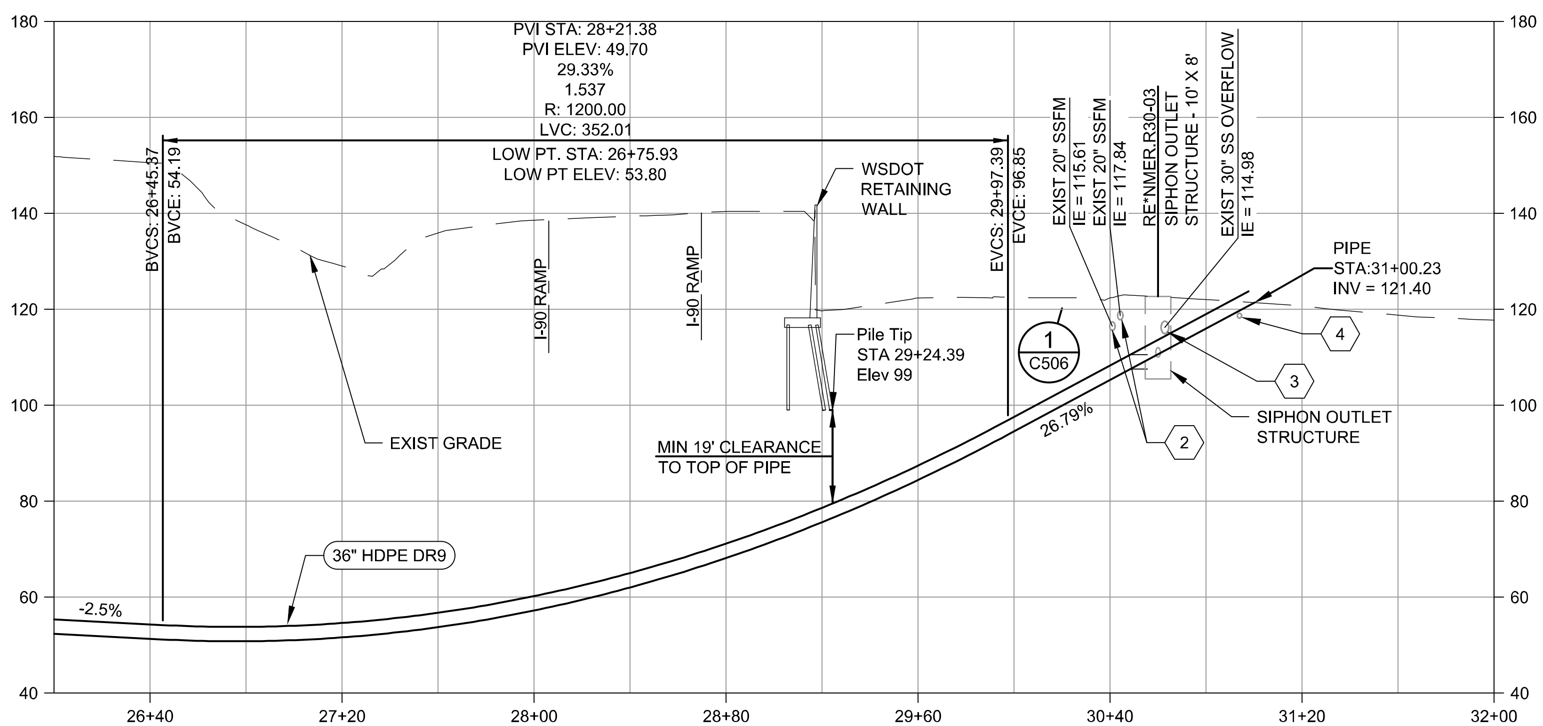


PROFILE

SCALE HORIZ 1" = 40'
VERT 1" = 20'

NOTES:

1. PLACE STRUCTURE SETTLEMENT POINTS ON BUILDING CORNERS OF PROPERTIES CONTAINING SUBTERRANEAN EASEMENTS.
2. INSTALL NSSP, SSP, OR VMS SETTLEMENT MONITORING WHERE SYMBOL IS SHOWN ON DRAWING. SEE GEOTECHNICAL INSTRUMENTATION DETAILS DRAWING FOR TYPE OF MONITORING EQUIPMENT REQUIRED.
3. SIPHON OUTLET STRUCTURE IS SHOWN FOR LOCATION COORDINATION AND WILL BE INSTALLED AS A SEPARATE WORK STAGE. SEE SHEETS C238 AND C338.
4. SOUND TRANSIT INFRASTRUCTURE SHOWN ON THE PLAN IS PROPOSED TO BE COMPLETED IN 2019.

KEY NOTES:

1. RELIEF WELL FINAL LOCATION TBD BETWEEN STA 29+60 AND 30+ 40. OFFSET MAXIMUM 20 FEET FROM CENTERLINE FROM HDD ALIGNMENT. RELIEF WELL TO TERMINATE 10 FEET ABOVE CROWN OF PILOT BORE. SEE LOCATION ON SHEET C503 AND DETAIL ON SHEET C506.
2. 20 INCH SS FORCE MAINS TO BE TEMPORARILY SUPPORTED. COORDINATE WITH WORK ON C238.
3. 30 INCH SS OVERFLOW TO BE TEMPORARILY RE-ROUTED DURING HDD WORK AND PERMANENTLY RE-ROUTED AFTER DEMO OF HDD EQUIPMENT. SEE PLAN ON SHEET C338
4. 12 INCH STORM DRAIN TO BE RE-ROUTED DURING HDD WORK AND REPLACED AFTER DEMO OF HDD EQUIPMENT. COORDINATE WITH WORK ON C238.

NOTE TO REVIEWER:

1. STATIONING WILL BE UPDATED AFTER 60% TO BE CONTINUOUS THROUGH ENATAI SIPHON.
2. SETTLEMENT MONITORING POINT NO TO BE ADDED POST 60%.
3. STABILIZING BEAM AND MICROPILES WILL BE ADDED TO EXISTING WSDOT RETAINING WALL DUE TO PROXIMITY OF HDD BORE. CURRENTLY UNDER DISCUSSION WITH WSDOT PERMIT SUBMITTAL. TO BE FINALIZED SUMMER 2018.
4. GRADE BEAM AND MICROPILES FOR MITIGATION OF EXISTING WSDOT RETAINING WALL PILES DUE TO PROXIMITY WITH HDD BOREHOLE IS BEING DISCUSSED WITH WSDOT. DRAWINGS WILL BE UPDATED FOR WSDOT PERMIT SUBMITTAL AFTER DISCUSSIONS IN SUMMER 2018.
5. ADDITIONAL DETAIL WILL BE DEVELOPED FOR TEMPORARY PROTECTION OF EXISTING SEWER AND STORM PRIOR TO 90% DESIGN.

0 20 40 60
SCALE IN FEET

STAHELI TRENCHLESS CONSULTANTS

NO	REVISION DESCRIPTION	BY	APVD	DATE

TETRA TECH
JACOBS

PRELIMINARY ISSUE DRAWING
INFORMATION ONLY
60% REVIEW
JUNE 2018

DESIGNED/DRAWN: K.STAHELI	SCALE: AS NOTED
PROJECT ENGINEER: J. CHAE	REFERENCE 0 1"
DESIGN APPROVAL: R. BROWNE	FACILITY NUMBER: TBD
PROJECT ACCEPTANCE: S. YILDIZ	CONTRACT NO: TBD

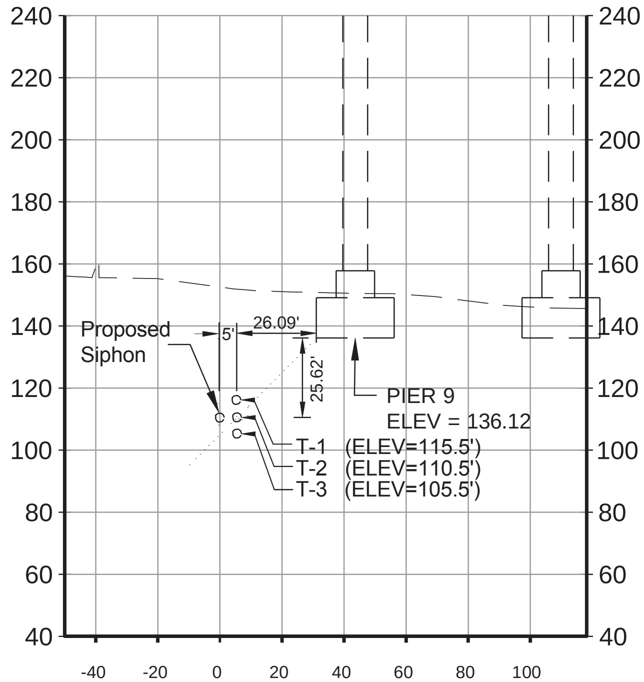
King County

DEPARTMENT OF NATURAL RESOURCES & PARKS
WASTEWATER TREATMENT DIVISION
NORTH MERCER ISLAND INTERCEPTOR AND
ENATAI INTERCEPTOR UPGRADE

ENATAI SIPHON HDD PLAN AND PROFILE STA 26+00 TO STA 32+00

DCN: TBD
DATE:
PROJECT FILE NO: TBD
DRAWING NO: C514
SHT NO / TOTAL /
REV NO:

BORDER FILE EDITION: KCWTD-2012-Dsize-TB-Border
FILE LOCATION:o:\Projects\Seattle\12539\200-12539-18001\CAD\sheetfiles\E00306E13-PIPE-C514.dwg
PLOT DATE: 9/13/2018 10:10 AM
PLOTTED BY: Thomasy, Bryan



SECTION B

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

SECTION AT PIER 9 FOOTING

September 2018 21-1-22000-211

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

ATTACHMENT 2

ATTACHMENT 2

H. F. 26.66 (Rev. 5-67)

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS

Original to Materials Engineer
Copy to Bridge Engineer
Copy to District Engineer
Copy to

LOG OF TEST BORING

S.H. ______ S.R. 90 Section Lake Washington Job No. L-2058

Hole No. H-42 Sub Section East Channel Bridge Cont. Sec. 1705

Station 361+59 Pier No. 9 Offset 110' Lt. ℄ Rev. Ground El. 43.9

Type of Boring Wash Bore & Rotary Casing 3" X 22'6" W.T. El. 13.9

Inspector Scovill, Nebgen, Duvall Date August 14-19, 1969 Sheet 1 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
	23		8 / 11 / 12 / 9 Std. Pen 1	SILT, SAND, SILTY CLAY, MIXED & LAYERED thinly scattered gravels, brown, damp to wet
	22		8 / 10 / 12 / 17 Std. Pen 2	
5				
	63		29 / 30 / 33 / 37 Std. Pen 3	
10				SAND - gravelly, silty, brown, wet occasional scattered cobble & boulder
	135		45 / 90 Std. Pen 4	
15				
				20'6" blast 2 sticks 50% 8" X 1 1/8" cobble or boulder
	100/4"		100 Std. Pen 5	
20				

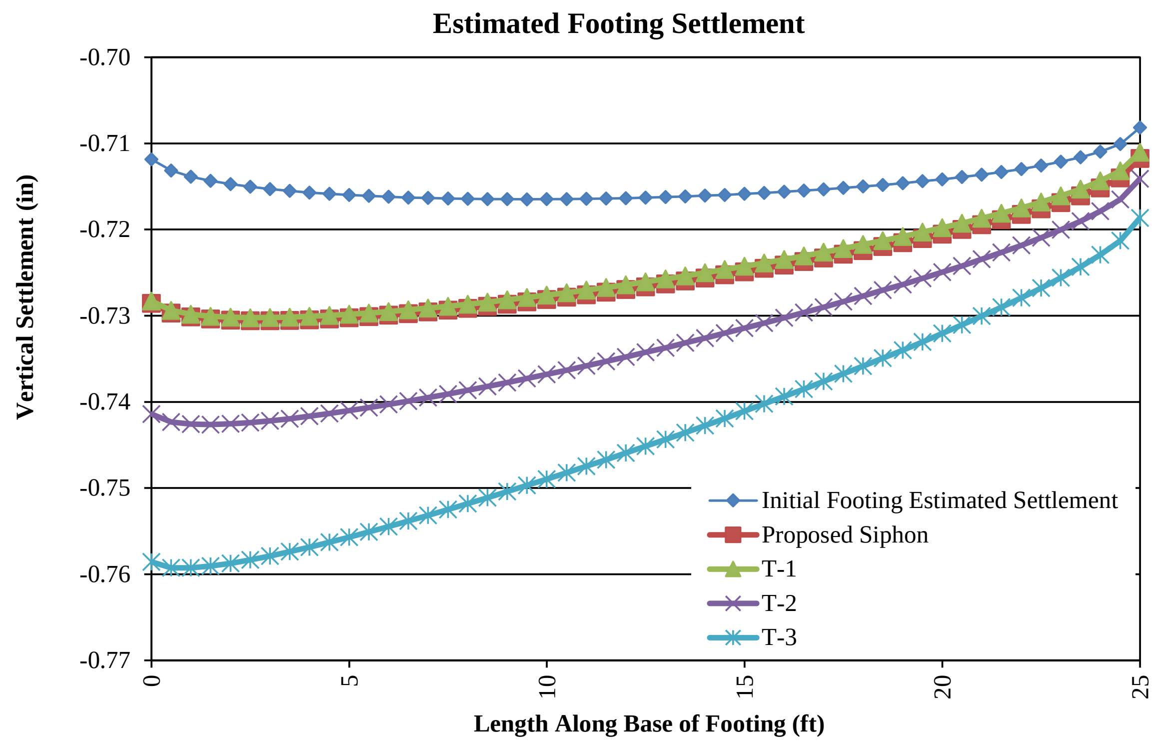
ATTACHMENT 3
(Sheet 1 of 2)

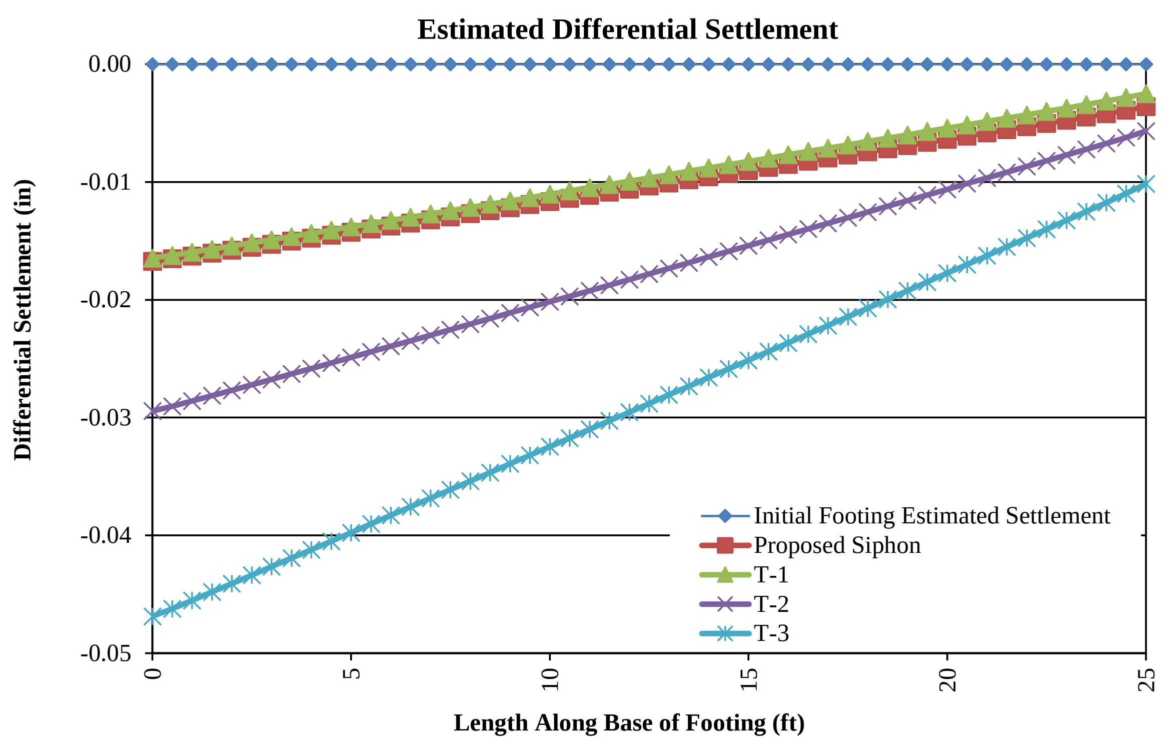
F. 26.66-A (Rev. 5-67)

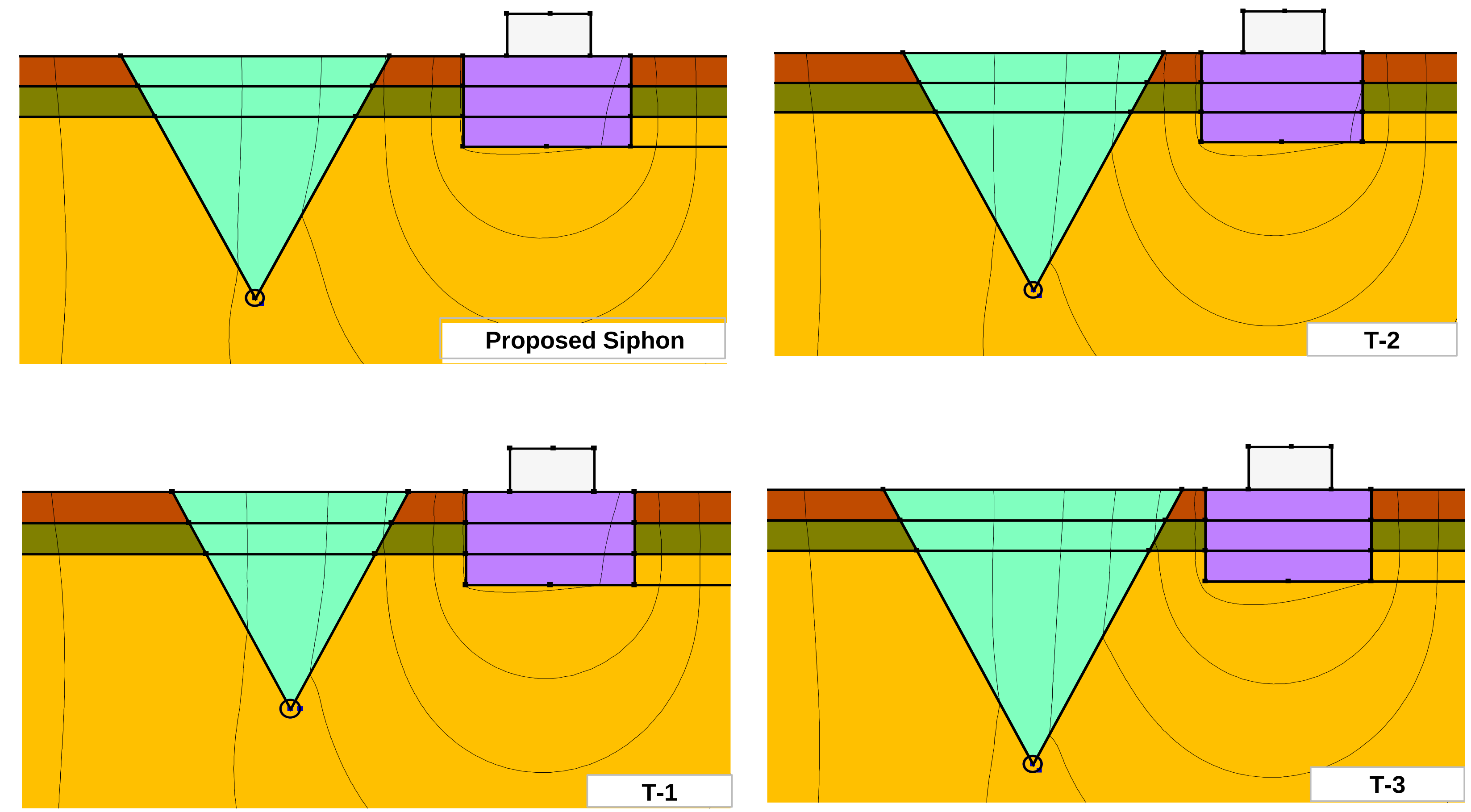
Hole No. H-42 Sub Section East Channel Bridge Sheet 2 of 2

DEPTH	BLOWS PER FT.	PROFILE	SAMPLE TUBE NOS.	DESCRIPTION OF MATERIAL
				SAND - gravelly, silty, brown wet occasional
				scattered cobble & boulder
	175/5"		175 Std. Pen 6	
				SAND with layers silty sand, thinly
25				scattered gravel, brown & grayish brown
				damp
	118/8"		40 68 50/2" Std. Pen 7	
30				
				Gravel, sandy, brown
	194		30 88 106 Std. Pen 8	Sand with layers silty sand thinly
				scattered gravel, brown & grayish brown
35				damp
				Sand - gravelly, silty, brown, damp
40				
	120/6"		120 Std. Pen 9	STOPPED TEST BORING AT 40'-6"
45				

ATTACHMENT 3
(Sheet 2 of 2)







Color	Geologic Unit	Friction Angle	Unit Weight pcf	Elastic Modulus ksf
	Hf	32	125	245
	Qvro	38	130	1,050
	Qva	40	135	5,500
	Influence Zone	30	120	245

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

FOOTING SETTLEMENT
FOOTING LOADING = 12,000 PSF

September 2018 21-1-22000-211

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

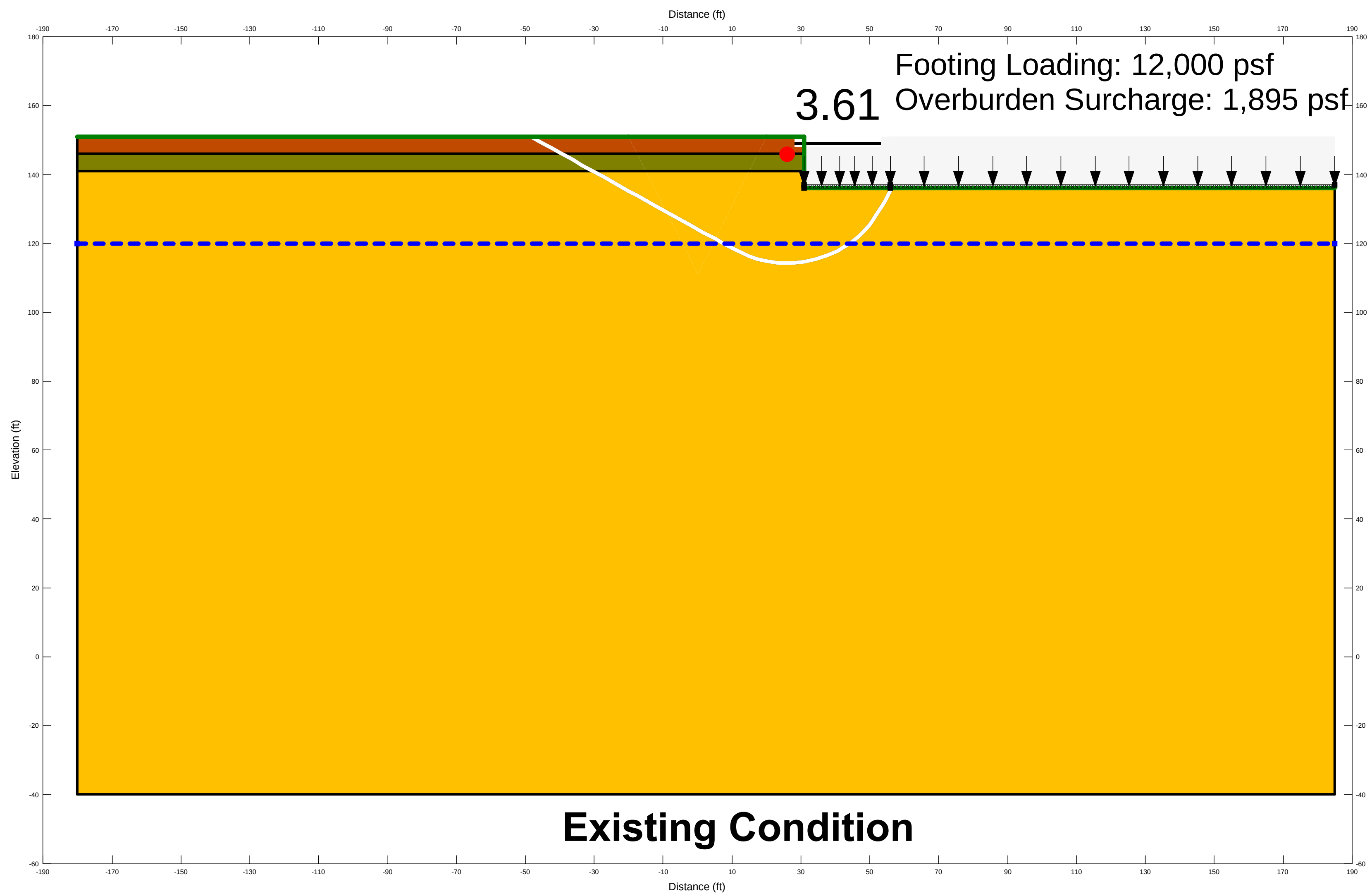
ATTACHMENT 4

ATTACHMENT 4

I:\EF\21-1 SEA\22000s\22000 North Mercer Enatai\Analysis\Enatai Bridge Pier\Revised Upper Alignment_12k\Proposed Siphon\

NORTH

SOUTH



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Hf	Mohr-Coulomb	125	0	32
	Qvro	Mohr-Coulomb	130	0	38
	Qva	Mohr-Coulomb	135	0	40

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

BEARING CAPACITY F.S.
Footing Loading = 12,000 psf

September 2018

21-1-22000-211

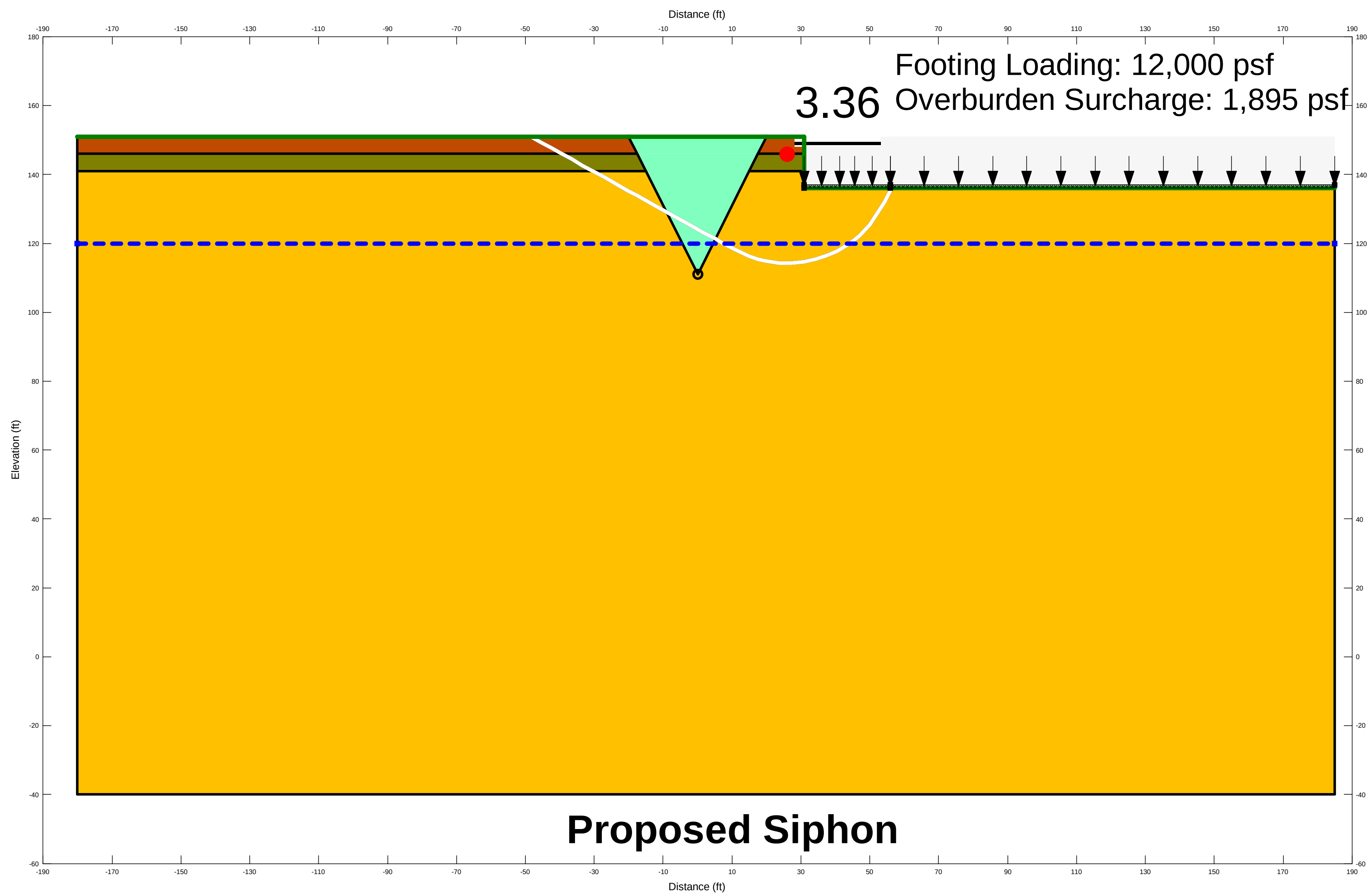
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

ATTACHMENT 5

I:\EF\21-1 SEA\22000s\22000 North Mercer Enatai\Analysis\Enatai Bridge Pier\Revised Upper Alignment_12k\Proposed Siphon\

NORTH

SOUTH



Proposed Siphon

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Hf	Mohr-Coulomb	125	0	32
	Qvro	Mohr-Coulomb	130	0	38
	Qva	Mohr-Coulomb	135	0	40
	Influence Zone	Mohr-Coulomb	120	0	30

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

BEARING CAPACITY F.S.
Footing Loading = 12,000 psf

September 2018

21-1-22000-211

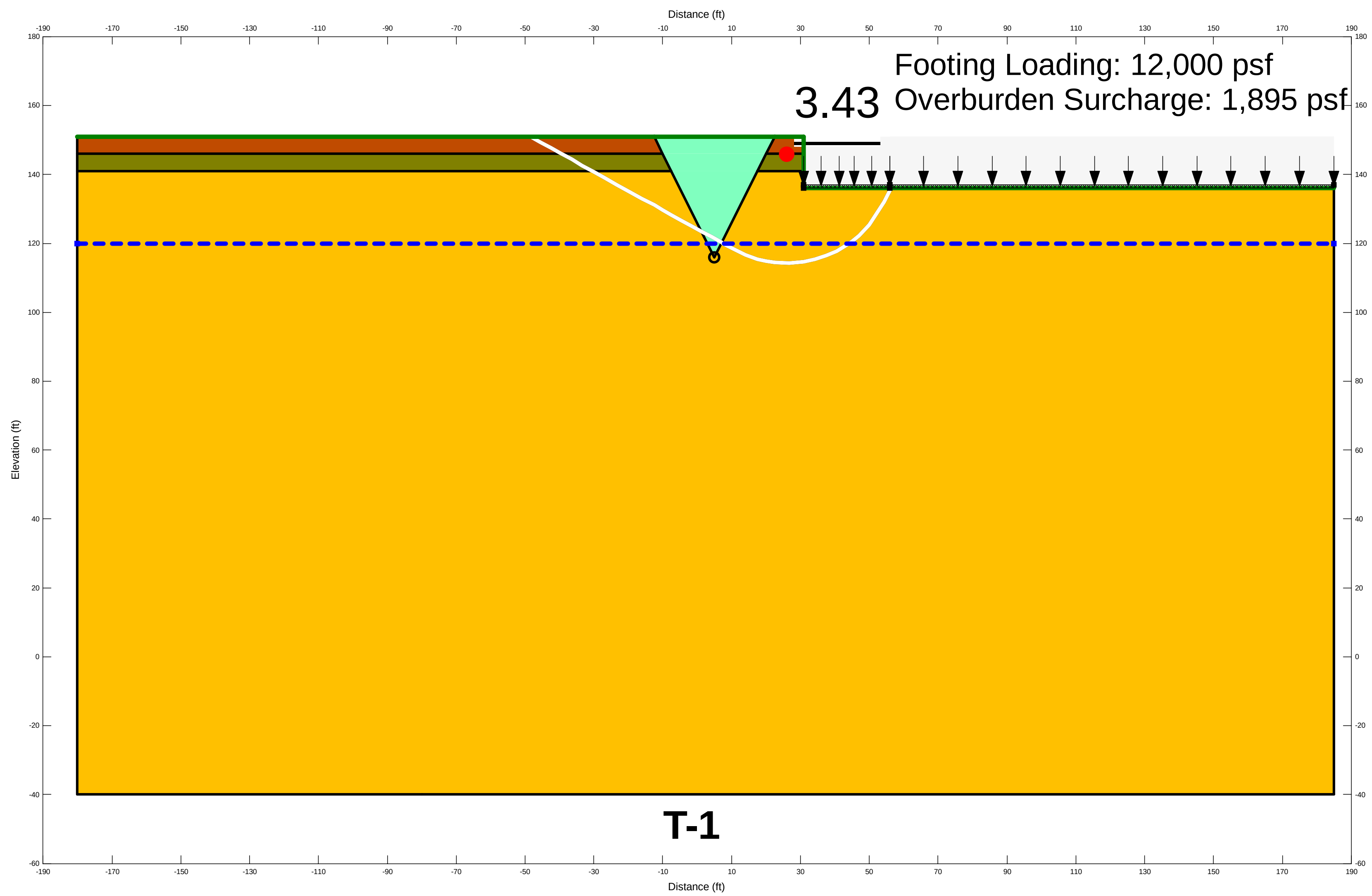
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

ATTACHMENT 6

I:\EF\21-1 SEA\22000s\22000 North Mercer Enatai\Analysis\Enatai Bridge Pier\Revised Upper Alignment_12k\Proposed Siphon - T-1\

NORTH

SOUTH



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Hf	Mohr-Coulomb	125	0	32
	Qvro	Mohr-Coulomb	130	0	38
	Qva	Mohr-Coulomb	135	0	40
	Influence Zone	Mohr-Coulomb	120	0	30

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

BEARING CAPACITY F.S.
Footing Loading = 12,000 psf

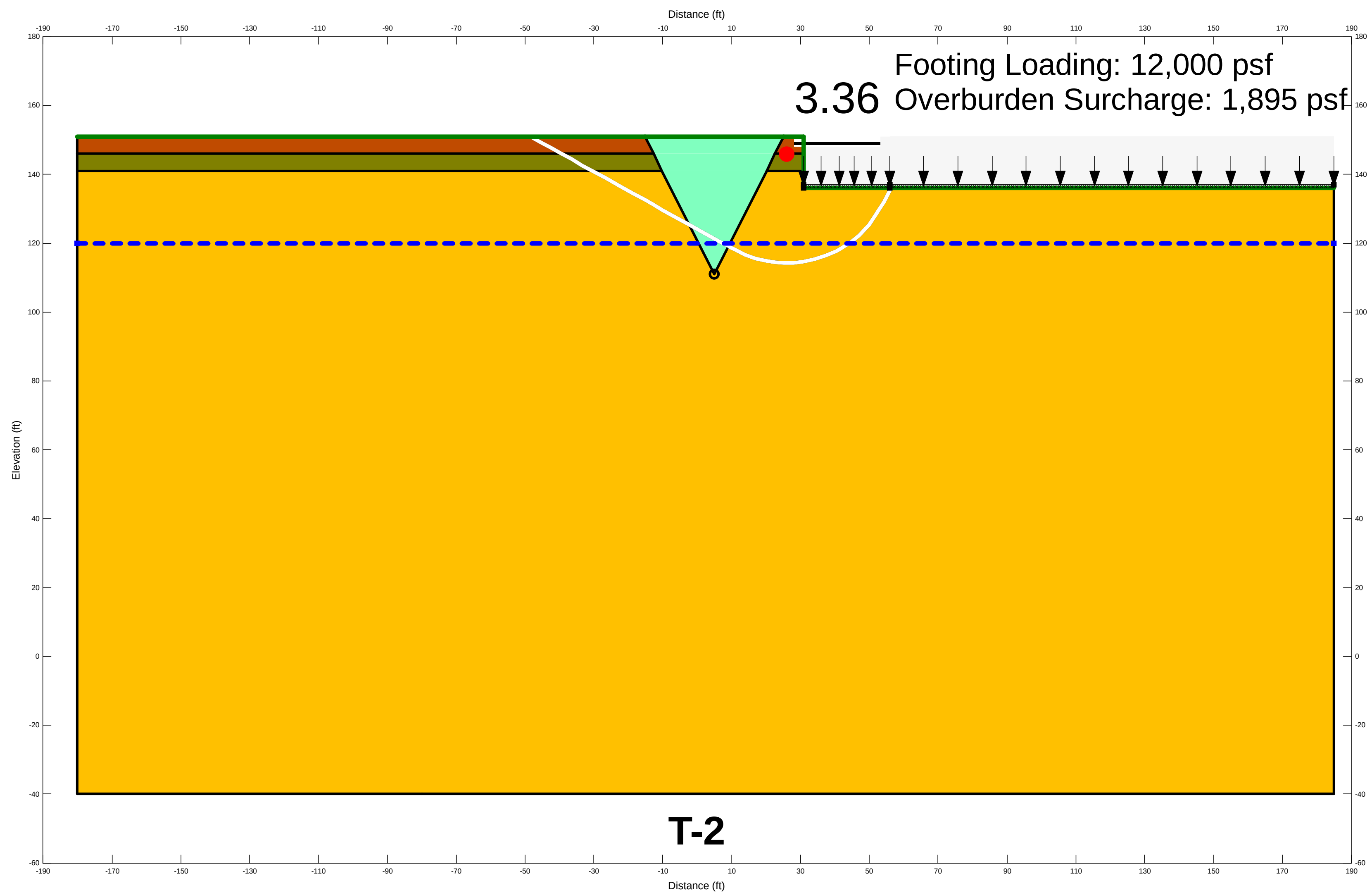
September 2018 21-1-22000-211

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

ATTACHMENT 7

NORTH

SOUTH



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Hf	Mohr-Coulomb	125	0	32
	Qvro	Mohr-Coulomb	130	0	38
	Qva	Mohr-Coulomb	135	0	40
	Influence Zone	Mohr-Coulomb	120	0	30

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

BEARING CAPACITY F.S.
Footing Loading = 12,000 psf

September 2018

21-1-22000-211

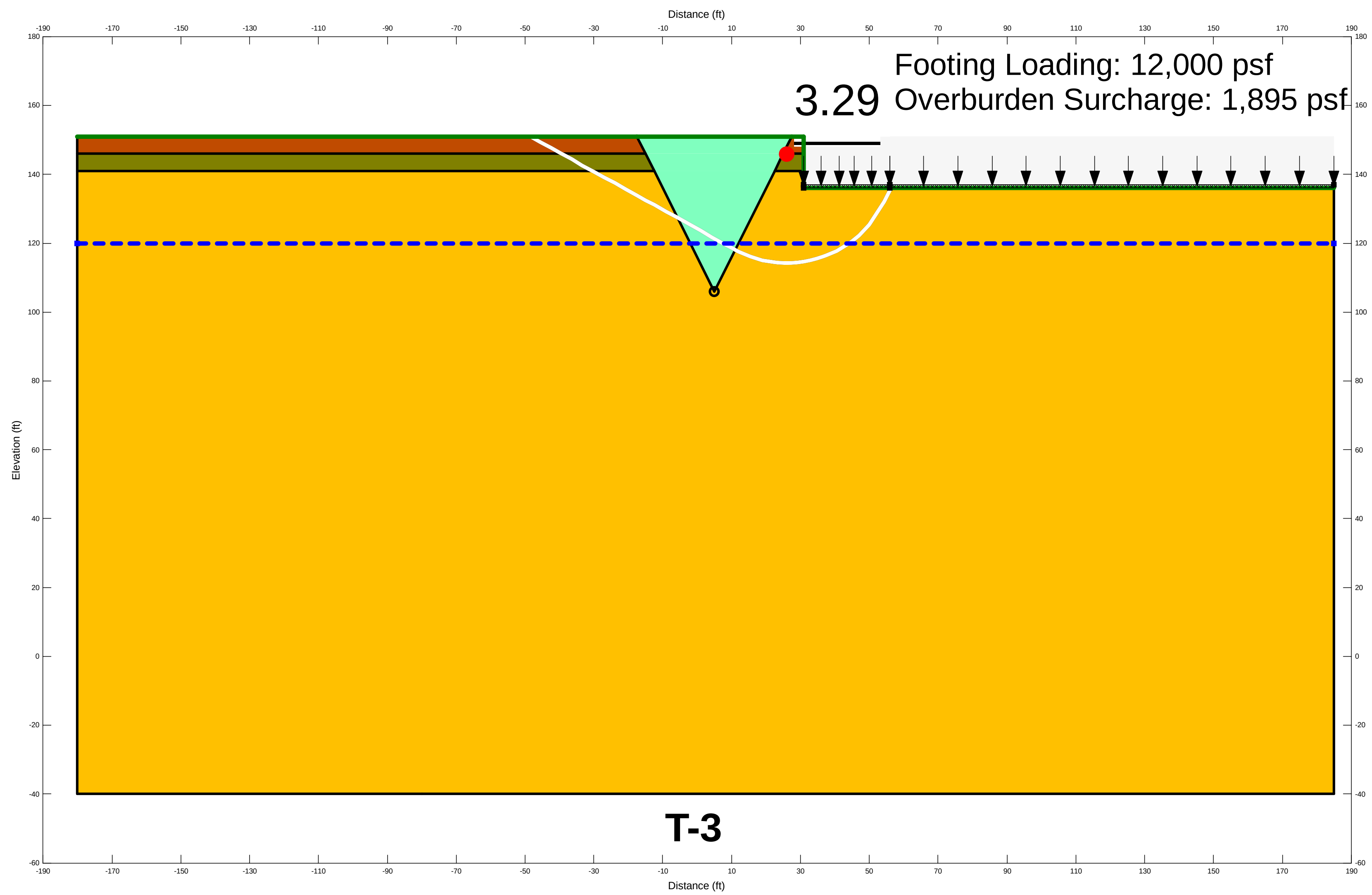
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

ATTACHMENT 8

I:\EF\21-1 SEA\22000s\22000 North Mercer Enatai\Analysis\Enatai Bridge Pier\Revised Upper Alignment_12k\Proposed Siphon - T-2\

NORTH

SOUTH



Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Hf	Mohr-Coulomb	125	0	32
	Qvro	Mohr-Coulomb	130	0	38
	Qva	Mohr-Coulomb	135	0	40
	Influence Zone	Mohr-Coulomb	120	0	30

North Mercer Island Interceptor
and Enatai Interceptor
Upgrade Project

BEARING CAPACITY F.S.
Footing Loading = 12,000 psf

September 2018 21-1-22000-211

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

ATTACHMENT 9

I:\EF\21-1 SEA\22000s\22000 North Mercer Enatai\Analysis\Enatai Bridge Pier\Revised Upper Alignment_12k\Proposed Siphon - T-3\

Draft

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

Attachment to and part of Report 21-1-22200-2103

Date: September 27, 2019
To: Ms. Grizelda Sarria
Tetra Tech, Inc.

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ ENVIRONMENTAL REPORT

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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