Exhibit 12.D.5

HWA GEOSCIENCES INC.
Geotechnical & Pavement Engineering • Hydrogeology • Geoenvironmental • Inspection & Testing

April 9, 2019
HWA Project No. 2017-039-21 T100-1400

Lochner H.W.
915 118th Avenue SE

Suite 130

Bellevue, Washington 98005

Attention: Mark Burrus, P.E.

Subject: **DRAFT GEOTECHNICAL REPORT**

24th Street SE and 91st Avenue SE Extensions

City of Lake Stevens, Washington

Dear Mr. Burrus:

This report presents the results of geotechnical engineering studies performed for the proposed 91st Avenue SE Extension Project in the City of Lake Stevens, Washington. The focus of this report is on the design and analysis of the embankment crossing the City of Everett transmission lines using lightweight cellular concrete as backfill. The report will subsequently be consolidated with the reports from the earlier tasks into one geotechnical report at a later date.

1. INTRODUCTION

1.1 GENERAL

The proposed 24th Street SE and 91st Avenue SE Extension Project in Lake Stevens, Washington, includes extending 91st Avenue SE southwards to its intersection with the proposed extension of 24th Street SE. Extending 91st Avenue SE requires placement of up to 17 feet of embankment fill over three City of Everett (COE) water transmission lines located about 200 feet south of 20th Street SE. The location of the crossing is shown on Figure 1, the Vicinity Map.

The transmission lines consist of two bar-wrapped concrete pipes and one steel pipe. The bar-wrapped pipes, designated TL-2 and TL-4, are 48 and 52 inches in internal diameter, respectively. TL-2 was installed in 1993 and TL-4 was installed 1958/1959. The steel pipe, designated TL-3, was installed in 1936 with an internal diameter of

21312 30th Drive SE
Suite 110
Bothell, WA 98021.7010

Tel: 425.774.0106
Fax: 425.774.2714
www.hwageo.com

approximately 48 inches. HWA previously evaluated the effect of the proposed additional roadway fill on the transmission lines. HWA's evaluation indicated there is a potential vulnerability that the pipes could experience excessive deflection due to the placement of the proposed embankment fill. This vulnerability was identified given that the proposed overburden heights are greater than the maximum fill depth recommended for this type of pipe (American Water Works Association (AWWA) M9, 2008).

As a condition for allowing construction of the proposed embankment over the COE water transmission lines, the City of Everett has requested that the City of Lake Stevens (COLS) propose a method of constructing the embankment such that the pipes will not be overloaded by the weight of the material and future access to the pipes is feasible. To satisfy this condition, COSL has proposed placing lightweight fill material to construct the 91st Avenue SE crossing to reduce the load and hence the embankment's impact on the pipes. The fill material currently proposed is lightweight cellular concrete. HWA reviewed the feasibility of using of cellular concrete and providing recommendations for design and installation for the roadway embankment. As part of this evaluation, HWA also addresses the concerns of the COE as they pertain to the operation and maintenance of the pipelines. This report summarized our findings and provided recommendations for the design and construction considerations to be addressed in the project design documents.

1.2 PROJECT BACKGROUND

The City of Lake Stevens is planning to extend 24th Street SE approximately 1,400 feet westward from SR 9 to its crossing with 91st Avenue SE, which will be extended 900 feet southward from 20th Street SE to connect with the 24th Street SE extension. The topography slopes downward steeply south of 20th Street SE and embankment fill will need to be placed to reduce the steepness of the roadway grades. The fill embankment for the 91st Avenue SE alignment will extend over 100 feet south of 20th Street SE and will cross three buried COE water transmission lines. The depth of the embankment fill will range from two (2) to 17 feet resulting in a total fill in the range of 7 to 22 feet above the top of the pipes. The pipes consist of two bar wrapped concrete pipes installed in 1994 and 1958 and with internal diameters of about 48 and 52 inches, respectively. The third pipe is a steel pipe installed in 1936 with an internal diameter of about 48 inches. To minimize the potential impact on the pipelines, COLS plans to construct the embankment at the crossing with lightweight material in place of conventional soil.

Due to space constraints, the fill near the intersection of 20th Street SE and the proposed 91st Avenue SE extension will be retained by a wall that will be constructed in front of, or replace, the existing wall along the southwest corner of the intersection of 20th Street SE with 91st Avenue SE. Construction of a structural earth wall would likely require shoring near the south edge of 20th Street SE to provide room for the reinforced soil zone. A permanent soldier pile wall is considered more cost effective. The extents of the wall along the west side of the road will be restricted by the need to maintain future accessibility to the water transmission mains for the COE. To maintain accessibility, the lightweight fill will be sloped so a wall does

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not need to be constructed over the water mains. Walls will extend from both the south and north to limit the extents of the sloped areas, as illustrated in Figure 2, the Site and Cellular Concrete Plan.

1.3 SCOPE AND AUTHORIZATION

HWA's scope for this phase included reviewing existing available data, design plans and cross sections; performing engineering analyses and providing design and construction recommendations for the lightweight (cellular concrete) embankment. Geotechnical information required for the tasks were obtained from HWA's previous work in the area and no additional geotechnical exploration was performed. Authorization to perform the work was through an email dated December 20, 2018 from Jorge Garcia from Lochner.

2. PREVIOUS EXPLORATIONS

Required geotechnical information was obtained from our previous geotechnical exploration program in the area and included three borings, designated BH-4 through BH-6. The borings were drilled on March 6 and 7, 2018 and were each advanced to approximately 31.5 feet below the ground surface. The locations of the explorations are shown on the Site and Cellular Concrete Plan, Figure 2. Additional geotechnical information was also obtained from two test pits designated TP- 27 and TP-28, that were part of the soil exploration for the final design phase and were excavated on September 14, 2017. The test pits were advanced to approximately 10 feet below ground surface. The locations of the test pits are also shown on Figure 2. The logs and test results from these explorations are provided in Appendix A.

3. SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site has gently rolling topography with vertical relief of about 10 to 20 feet in places. Beginning about 350 feet south of the water transmission lines, the ground slopes downward to the north at approximately 16 percent grade for about 100 feet before decreasing to approximately 5 percent grade for another 150 feet along the proposed 91st Avenue SE alignment. The ground then slopes upward (to the north) at approximately 9 percent grade to meet the intersection with 20th Street SE.

To the southwest of the intersection with 20th Street SE and 91st Avenue SE, surface runoff collects in the low area that forms a stormwater pond. Most of the site is currently wooded except for a few residential houses and a small pump station located southwest of the intersection between 20th Street SE and 91st Avenue SE. South of 20th Street SE, there is a Lock-Block wall about 10 feet tall located on the western edge of the proposed roadway prism for the 91st Avenue SE alignment.

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3.2 GENERAL GEOLOGIC CONDITIONS

The project alignment is located within the Puget Lowland. The Puget Lowland has repeatedly been occupied by a portion of the continental glaciers that developed during the ice ages of the Quaternary period. During at least four periods, portions of the ice sheet advanced south from British Columbia into the lowlands of Western Washington. The southern extent of these glacial advances was near Olympia, Washington. Each major advance included numerous local advances and retreats, and each advance and retreat resulted in its own sequence of erosion and deposition of glacial lacustrine, outwash, till, and drift deposits. Between and following these glacial advances, sediments from the Olympic and Cascade Mountains accumulated in the Puget Lowland.

General geologic information for the site was obtained from the Geologic Map of the Snohomish Quadrangle, Snohomish County, Washington (Minard, 1985). The map indicates that the geology along the project corridor includes glacial till near the ground surface from the most recent continental glaciation, the Fraser Glaciation. Minard indicates patches of recessional outwash and advance outwash, consisting of predominantly sands and gravels, were observed near the ground surface to the west of the site.

3.3 SUBSURFACE SOIL CONDITIONS

Our interpretations of subsurface conditions are based on our previous geotechnical explorations performed during the previous phases as described in Section 2, Previous Exploration. The exploration data indicate that the subsurface soils consist of very loose to loose fill above glacial deposits consisting of a thin layer (up to three feet thick) of recessional outwash, 7 to 10 feet of glacial till overlying advanced outwash to the depths of exploration. On the east side of 91st Avenue SE, the fill overlies buried topsoil as observed in two test pits excavated during the previous explorations. The presence of buried topsoil was not apparent in any of the three boreholes drilled. Each of these soil units is described below with the youngest in origin described first.

- **Fill:** Fill soils were encountered over glacial till in the three boreholes drilled. In BH-4, these soils extended from the ground surface to approximately 5 feet deep, and consisted of very loose to loose, gravelly sand and thin layer of gravel. The fill in boreholes BH-5 and BH-6 consists of two to three feet of topsoil overlying 2 to 3 feet of very loose, gravel in BH-5 and loose, gravelly, sand in BH-6.

- **Buried Topsoil:** Buried topsoil was observed below the fill in test pits TP-27 and TP-28 excavated during the previous phase. This layer was 1 to 2 feet thick and extended to a depth of 7 feet in TP-27 and a depth of 5 feet in TP-28.

- **Recessional Outwash:** Along 91st Avenue SE, the recessional outwash was typically a 2- to 3-foot thick layer of silty, gravelly, sand with abundant sub-rounded gravel particles, and was observed in test pits TP-27 and TP-28.

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- **Glacial Till:** Very dense, silty sand with varying amounts of gravel was encountered in each of the three boreholes and the test pits. The test pits were terminated in this layer while, the borings extended below the till. At these locations, the till ranges in thickness from approximately 7 to 10 feet. This material was deposited as a deforming bed of material along the sole of the glacial ice as lodgement till. It consists of an unsorted mixture of clay, silt, sand and gravel, and is very dense, having been consolidated by the weight of over 3,000 feet of glacial ice. It is also known to contain scattered cobbles and boulders.
- **Advanced Outwash:** Advanced outwash was encountered below the glacial till in each of the boreholes and they were all terminated within this layer. The deposit consisted of very dense alternating layers of gravelly, silty sand and slightly silty to clean sand.

3.4 GROUND WATER CONDITIONS

Ground water was encountered within the advance outwash in each of the three boreholes drilled varying from 13 feet below ground surface in BH-6 to 21 feet below ground surface in BH-5. The ground water readings were observed during drilling and are not necessarily representative of the static ground water table, which is typically higher than the depth at which the ground water was observed during drilling. Based on this, we have assumed that the advance outwash is saturated for our design and analysis.

4. CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Construction of the proposed 91st Avenue SE extension with a conventional soil supported roadway will result in significant additional load being placed on the City of Everett's water transmission lines that underly the alignment. This additional load will induce pipe deformation that may exceed the recommended maximum values, as provided by AWWA (2008), especially when the pipes are empty. To minimize these potentially damaging additional loads, we recommend that the road extension crossing the water transmission lines be constructed with a roadway section utilizing cellular concrete fill. Our analyses indicated that using cellular concrete resulted in overburden plus traffic load that were within the recommended values for pipes of the same materials and dimensions, as defined by AWWA (2008) and is adequate to provide protection of the existing pipelines. Therefore, the cellular concrete embankment as proposed is not expected to have an adverse impact on the existing pipelines.

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4.2 SEISMIC CONSIDERATIONS

4.2.1 Seismic Design Parameters

Earthquake loading for the proposed project was developed in accordance with Section 3.4 of the AASHTO *Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition*, 2011 (AASHTO, 2011 with 2012, 2014 and 2015 Interim Revisions) and the Bridge Design Manual (WSDOT, 2017). For seismic analysis, the Site Class is required to be established and is determined based on the average soil properties in the upper 100 feet below the ground surface. Based on available geotechnical information for the project corridor, the underlying soils represent AASHTO Site Class C.

Table 1 presents the seismic coefficients, associated with AASHTO Site Class C that were used for the geotechnical evaluations discussed in this report. These parameters are based upon a design event with a 7 percent probability of exceedance in 75 years (equal to a return period of 1,033 years). Ground motions for the site are based on probabilistic earthquake hazard mapping efforts including those conducted by the United States Geological Survey (Frankel, et al., 2002) and (Peterson, et al., 2014). The parameters provided include the 2014 update and were based on the design memorandum from WSDOT, dated January 8, 2017, and using the BridgeLink tool (BridgeLink, 2017) developed by WSDOT. Accordingly, a Seismic Design Category C, as given by AASHTO (AASHTO, 2011), should be used.

Table 1. Seismic Coefficients for Geotechnical Evaluations Using AASHTO

Site Class	Peak Horizontal Bedrock Acceleration PBA, (g)	Spectral Bedrock Acceleration at 0.2 sec S_s, (g)	Spectral Bedrock Acceleration at 1.0 sec S_1, (g)	Site Coefficients			Peak Horizontal Acceleration PGA, (g)
				F_{pga}	F_a	F_v	
C	0.347	0.785	0.232	1.200	1.200	1.500	0.416

4.2.2 Soil Liquefaction

The potential for soil liquefaction must be considered during the design of any soil-supported structure. Soil liquefaction is a phenomenon where loose, saturated, granular deposits temporarily lose strength and behave as a liquid in response to moderate to strong earthquake shaking. Primary factors controlling the development of liquefaction include the intensity and duration of strong ground motions, the characteristics of subsurface soils, in-situ stress conditions and the depth to ground water. Geotechnical data from boreholes and test pits shows the granular layers encountered below the water table are of a dense nature, indicating that liquefaction will likely not be a design consideration for the crossing.

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4.3 EVALUATION OF ROAD EMBANKMENT IMPACTS ON WATER TRANSMISSION LINES

HWA previously performed a preliminary evaluation of the impacts the construction of the proposed road embankment using conventional soil will have on the existing COE water transmission lines. The purpose of the evaluation was to assess the need of implementing measures to protect the existing pipelines. The evaluation was detailed in our report "*Revised Draft Supplementary Geotechnical Report, 24th Street Final Design, Lake Stevens, Washington*".

Results of the analyses are provided below:

- The bar-wrapped concrete cylinder pipe (CCP) TL2, the center transmission line, has an estimated pipe deflection (i.e. deflection due to the total external load from the existing soil plus the proposed road embankment) that is within the maximum recommended by *M9 Concrete Pressure Pipe* 3rd Edition (AWWA, 2008) for bar CCP. HWA also compared the proposed depth of cover of 14 feet with the allowable depth of cover for a 48-inch CCP (based on the assumed soil stiffness provided in the as-built Plans) as provided in Table 7-3 of AWWA (2008), which is 15 feet. The result indicates the use of conventional soil for construction of the proposed embankment is within the limits for depth of cover and deflection and is not likely to overstress the pipe, provided it is in good condition and has not been damaged by corrosion or other defects.
- For the steel pipe TL3, which runs along the north side of the COE easement, HWA estimated a total deflection of greater than greater than 2% times the pipe diameter. Given its age, it was installed 82 years ago, the steel pipe is anticipated to be near the end of its useful life. Due to these factors, the steel pipe is considered vulnerable and backfilling the proposed embankment with conventional soil could overstress pipe.
- The bar-wrapped CCP TL4, that runs along the south side of the COE easement, has an estimated pipe deflection that is within the maximum recommended by AWWA for this type of pipe. However, based the approximate existing depth of cover and the soil stiffness provided on the as-built plans, the total depth of cover with the new embankment fill would be approximately 16 feet, which is greater than the maximum allowable depth of 15 feet recommended for CCP in Table 7-3 of *M9* (AWWA, 2008). Although the estimated deformation of the pipe does not indicate too much deflection, the depth of cover indicates the pipe could be near the point of being overstressed if the proposed embankment is constructed with conventional fill.

The evaluation indicated that construction of the embankment with conventional fill will exceed the recommended maximums for depth of cover for one pipe and could be detrimental to the steel pipe. Some types of alternate construction method are needed to limit the impacts the embankment will have on the existing water transmission mains.

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4.4 USE OF ALTERNATIVE BACKFILLS FOR THE ROADWAY EMBANKMENT

Lightweight material could be used to significantly reduce the load on the underlying transmission lines and minimize impact to the pipelines. It can also provide a high strength material to increase the stability of the embankment slopes and allow steeper embankment slopes limiting the footprint of the proposed improvements.

Several lightweight fill materials are available and have been used on past projects with success. These materials include Geofoam, bottom ash, volcanic rock, glass cutlet and lightweight cellular concrete. Geofoam consists of proprietary lightweight Styrofoam blocks that are readily available to contractors and have been used successfully on numerous road projects. One drawback in transportation related applications is that Geofoam blocks require encapsulation with a gasoline resistant geomembrane. Bottom Ash is a byproduct of coal fired power plants and weighs between 45 and 75 pounds per cubic foot. Bottom ash has been used on several road projects but is becoming hard to obtain. Volcanic rock has been used as lightweight fill in the past. Volcanic rock generally weights approximately 45 to 60 pounds per cubic foot. However, there are no readily available sources of volcanic rock in Washington State. Therefore, the cost associated with importing the material would be prohibitive for this project. Another option is to use lightweight cellular concrete which is both cost effective and technically viable. The following paragraphs and sections described cellular concrete in greater details.

4.5 CELLULAR CONCRETE EMBANKMENT

Lightweight cellular concrete is a proprietary product that can be manufactured onsite to a wide range of unit weights (27 to 120 pcf) and compressive strengths to match project requirements. It is relatively strong, low-density, and can have a bearing capacity higher than compacted fill. Cellular concrete is produced by adding a pre-formed foam to a slurry of Portland cement, fly ash, water, and occasionally aggregates. The resulting mixture is highly flowable and pumpable as well as self-leveling and will harden between 2 to 6 hours after production depending on the mix design and admixture. Cellular concrete is also known by other names including foam cement, foamed concrete, or lightweight flowable fill. There are two methods for producing cellular concrete: the batch production method in which externally generated foam is injected into the drum of a mixer for a calculated amount of time and the continuous production method in which foam is injected in-line, on the discharge side of a pump.

Cellular concrete is widely available in Washington State and has been used successfully on road projects. Cellular concrete can be placed around existing utilities and can be excavated for future utility crossings if needed. A major advantage of using cellular concrete for this project is placement of cellular is done using a pump outside the embankment area thereby minimizing the use of heavy equipment over the pipes.

Cellular concrete is like standard concrete in that it should be protected from direct contact with the adjacent wetlands to avoid environmental issues. The crossing is not located adjacent to a wetland in this project and is not expected to be in direct contact with a wetland. Cellular

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concrete does have the same pH effect on adjacent waterbodies as would be expected from the placement of standard concrete.

The cost of cellular concrete is dependent on-site access conditions, volume of placement and desired unit weight. For high volume lightweight applications, cellular concrete can range from $50 to $80 per cubic yard. For low volume and high unit weight applications cellular concrete can cost as much as $250 per cubic yard. For preliminary estimating purposes a unit cost of $80 per cubic yard should be assumed. Cellular Concrete can be placed a rate as high as 250 yards per day and for this project, completion within a week can be expected.

Cellular is specified in terms of unit weight and maximum compressive strength by classes, including Class I to VI, or more. The higher the class, the higher the density and compressive strength. Properties corresponding to commonly used classes of cellular concrete by ACI are shown in Table 2.

Table 2: Cellular Concrete Types and Properties (ACI 213R-03) *

	Class II	Class III	Class IV
Maximum Cast Density (pcf)	30	36	42
Minimum Compressive Strength at 28 days (psi)	41	80	120

*Guide for Structural Lightweight-Aggregate Concrete, Reported by American Concrete Institute (ACI) Committee 213

4.5.1 Proposed Embankment Dimensions

The construction of the cellular concrete embankment will involve the removal of approximately two feet of the existing soil and placement of cellular concrete. We propose using Class II (maximum cast density 30 pcf) to within a foot of the final roadway grade, followed by placement of approximately 6 inches of crushed surfacing base course (CSBC) and 6 inches of hot mix asphalt (HMA). Table 3 below summarizes the approximate depths of each material at the maximum depth of the proposed embankment (west edge) along each pipeline. These values were used to analysis the approximate stresses imposed on the pipes by the proposed embankment.

Table 3: Embankment Material Thicknesses over the Pipelines at the Maximum Embankment Height

Embankment Material	Pipeline		
	TL2	TL3	TL4
Thickness of soil removed	2 ft	2 ft	2 ft
Unit weight of soil removed	100 pcf	100 pcf	100 pcf
Thickness of remaining soil cover	2.92 ft	3.67 ft	5 ft
Unit weight of remaining soil cover	125 pcf	125 pcf	125 pcf
Cellular concrete depth	15 ft	17 ft	13 ft
Unit weight of cellular concrete	30 pcf	30 pcf	30 pcf
CSBC thickness	0.5 ft	0.5 ft	0.5 ft
Unit weight of CSBC	140 pcf	140 pcf	140 pcf
HMA thickness	0.5 ft	0.5 ft	0.5 ft
Unit weight of HMA	145 pcf	145 pcf	145 pcf

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4.5.2 Cellular Concrete Embankment Analyses

Based on the proposed depths of cellular concrete provided in the previous section, evaluation of the impact of constructing the embankment with cellular concrete over the pipelines was performed based on methods provided by the American Water Works Association for concrete and steel pipes. For concrete pipes, recommendations from *M9 Concrete Pressure Pipe* Manual of Water Supply Practices, 3rd Edition (AWWA, 2008) were used. For steel pipe, recommendations from *M11 Steel Pipe: A Guide for Design and Installation*, 4th Edition (AWWA, 2004) were used. Evaluations were performed to determine the anticipated deflections of the pipes. For concrete pipes, maximum deflections are taken to be equal to $D^2/4{,}000$ in inches, where D is the diameter in inches. For steel pipes, the maximum deflection is taken to be 5% of the pipe internal diameter. The results of our analyses, with the maximum deflections for each pipeline, are summarized in Table 4. The impact of the live load was considered negligible given that the heights of the embankment are greater than 8 feet.

Evaluation of construction loading was considered for this analysis; however, we do not recommend that heavy equipment be allowed to track across the pipelines, particularly once two feet of soil has been removed prior to placement of the cellular concrete. Lightly loaded, tracked equipment is expected to have equal or lesser maximum vertical stresses compared with those estimated for the proposed final embankment heights.

Table 4: Summary of Pipeline Deflection with Cellular Concrete Embankment

Pipeline	Type	Pipe diam. (in.)	Approx. Depth of max. fill(1) (ft)	Max. estimated vertical stress(2) (psf)	Deflection for Final Embankment Loading(3)	Recommended limit
TL2	Bar-wrapped CCP	48	19	758	0.05 in. (0.10%)	0.58 in. (AWWA M9)
TL3	Steel	48	22	911	0.66 in (1.4%)	5% (AWWA M11)
TL4	Bar-wrapped CCP	52	19	958	0.09 in (0.16 %)	0.68 in. (AWWA-M9)

(1) At maximum embankment height
(2) At the top of pipe (include remaining soil cover and road pavement)
(3) Deflection is provided in terms of inches with percent of the pipe diameter in parentheses

The results indicate that constructing the embankment with Class II lightweight cellular concrete (average density = 30 pcf) will keep the total weight of the embankment well within the maximum limits recommended by AWWA. The analyses indicate that impact to the pipes would be greater during construction, if operation of heavy equipment is permitted over the pipelines. We recommend that restrictions of operating heavy equipment be placed such that only light wide-tracked equipment be allowed to operate over the pipes during construction.

4.5.3 Settlement

The cellular concrete has a bearing capacity higher than compacted fill but is much lighter. Placing cellular concrete in a uniform layer for the embankment reduces the overall weight of the embankment load acting on top of the pipes. The increase in load at the maximum overburden depth is equivalent to approximately 4 feet of conventional soil with an average unit weigh of 125 pcf. The pipeline was installed within very dense glacial till and settlement due to four feet of soil on this material is negligible. In the long term, settlement of the pipe due to the cellular embankment is negligible.

4.6 Cellular Concrete Design Recommendations

The objective of the cellular concrete embankment design is to minimize the load on the water pipes. Our analyses indicate that using cellular concrete with a density of 30 pcf results in maximum overburden stresses and deflections that are within the limits that the pipelines are typically designed for (results provided in Table 4). Therefore, we recommend construction the proposed embankment with Class II lightweight cellular concrete.

4.6.1 Lateral Extents of Cellular Concrete

In the lateral direction (along the roadway centerline), the cellular concrete should extend 10 feet north of the north pipeline (TL3) at the base of the embankment and slope up at 1H:1V towards the north to its intersection with the proposed base of the pavement section. At the south end, the cellular concrete should extend 10 feet south of the south pipeline (TL4), at the base of the embankment and slope up at 1H:1V towards the south (Figure 3). These extents are provided so that conventional soil backfill is placed in areas that are outside zone of influence of the pipes

4.6.2 Cellular Concrete Embankment Slope

Along the western side of the embankment, a wall was considered to reduce the extents of the filled placed; however, we do not recommend that a wall constructed within the zone of influence of the proposed pipelines as it could transmit additional loading to the pipes as well as restrict future access to the pipes. To reduce the extent of the proposed embankment slope, we recommend constructing the embankment sides at a slope of 1.5 to 1 (horizontal: vertical). Sloping down at 1.5 to 1 (horizontal: vertical) also provides sufficient space for operation and maintenance of the Snohomish County PUD Pump Station to the north of the COE easement.

4.6.3 Future Access with Temporary Excavations for the Pipelines

Future access to the pipelines would be the same as if conventional soil had been used for the embankment. The cellular concrete is fully excavatable and if the excavation is less than 4 feet, shoring is not required. In accordance with Part N of Washington Administrative Code (WAC) 296-155, latest revisions, all temporary cuts in excess of 4 feet in height must be either sloped or shored prior to entry by personnel. Lightweight cellular concrete would classify as Type A soils, per WAC 296-155. Where shoring is not used, temporary cuts in Type A soils should be sloped no steeper than 3/4H:1V (horizontal: vertical).

4.6.4 Backfill of Excavations with Cellular Concrete

If excavation through the cellular concrete is required for maintenance or replacement of the existing transmission mains, the excavated material will need to be replaced with newly placed cellular concrete. Backfilling will need to be performed in lifts that will be restricted by the maximum depth that cellular concrete can be placed. For Class II lightweight cellular concrete this is 3 to 4 feet. The cure time will be defined per the manufacturer; however, the set time is typically of the order of 12 hours.

Lightweight cellular concrete is supplied by two to three local contractors, including one in Monroe. For typical maintenance work, suppliers typically request two weeks' notice in advance of lightweight cellular concrete being placed. For special/emergency repair work, 48 hours turnaround time is possible

4.6.5 Pavement Consideration

In determining the expected load on the pipes, we assumed pavement section consisting of 6 inches thick HMA pavement is adequate for the anticipated traffic. Pavement design is not within HWA's scope of work

4.6.6 Lateral Earth Pressures on Retaining Walls

Cellular Concrete

When the Class II cellular concrete is initially placed up against these walls, it will exert a fluid pressure equal to the wet unit weight of 30 pcf. As the cellular concrete cures overnight, the lateral pressure on the adjacent wall will be "locked in" and will remain acting on the structure. However, the shear strength of the cured cellular concrete will result in no additional lateral stress being exerted in the previous, cured lift, when the next lift of cellular concrete is poured, reducing the total load applied to the wall as backfill is placed. For Class II cellular concrete with a lift thickness of 3 feet, the maximum lateral pressure acting on the back of the wall can be represented as a uniform (rectangular) load equal to 45 psf over height of the cellular concrete section.

Wall Drainage

These earth pressures assume no accumulation of water behind the wall. We recommend placing a drain along the bottom of the base course on the west side of the proposed embankment. This should consist of a 4-inch diameter, Schedule 40, perforated, plastic drain pipe embedded within the CSBC above the top of the cellular concrete. This should be in addition to the wall drainage provided at the base of the wall, as addressed for wall design in our *Revised Draft Report*, dated April 24, 2018.

4.6.7 Buoyancy

Existing data suggests that the static groundwater table is currently located approximately 13 feet below the ground surface along the alignment within the advance outwash and below the till which is anticipated to have very low permeability. The groundwater elevation is below the proposed cellular concrete fill. We do not expect that the groundwater level would be able to rise to the ground surface under the current condition or the proposed roadway configurations therefore, is not expected to result in a buoyancy issue under static groundwater conditions.

Perpendicular to the roadway, the area is sloping east to west. In an extreme event the roadway can act like a dam causing surface water to rise on the east side. This can be mitigated with a drainage system on the east side of the embankment that are properly discharged and maintained so water cannot pond to a significant depth (see recommendations in the following section). Buoyancy, therefore, is not anticipated to be a design concern for this project.

4.6.8 Trench Drain on East Side of Embankment

We recommend installation of a trench drain along the east side of the proposed embankment for the length of the proposed cellular concrete section to intercept drainage that could collect on top of the cellular concrete. The trench drain should consist of a 4-inch diameter, Schedule 40, perforated, plastic drain pipe, as shown on Figure 4A to Figure 4C. The perforated pipe should be embedded within a zone of clean 5/8-inch minus crushed rock that is wrapped in a non-woven geotextile meeting the requirements for Geotextile for Separation as provided in Table 3 of Section 9-33 of the *Standard Specifications* (WSDOT, 2018). Discharge should be tight lined to a suitable outlet.

4.7 CELLULAR CONCRETE CONSTRUCTION

Key items that should be included in the contract documents include the required cast density. The Contractor should provide the required cast density at the point of placement. The cellular concrete material should consist of Class II cellular concrete, possessing a cast density ranging from 27 to 30 pcf and a minimum compressive strength of 40 to 50 psi. The Contractor should also provide a Work Plan, including the equipment and procedures proposed, with working drawings for form work for the cellular concrete showing work areas which shall be formed in segments prior to placement. Items in the submittal should include:

1) Material list of items; manufacturer's specifications for those items;
2) Mix design(s), including laboratory data using the mix design verifying mass and strength requirements; and
3) Working drawings of form work for the installation of the cellular concrete, including permanent structure and temporary form systems conforming to the manufacturer's specifications. Form work shall be water tight and must be capable of supporting lateral loads of 3 times the calculated load of the cellular concrete.

We recommend that the lightweight cellular concrete be placed in maximum 3-foot thick lifts, as conceptually depicted on Figure 4A to 4C of this report. The 3-foot thick layer should be placed per the manufacture’s recommendation and be allowed to flow neatly up against the sides of the formworks/excavation. The cellular concrete should extend from the west edge of the embankment and build up to the east side of the embankment as shown on Figure 4A to 4C.

Construction will need to consider potential impacts to the adjacent stormwater facility. It is important that the formwork and fill placement are carefully monitored to prevent any spills of cellular concrete outside of the forms, especially on the west side where there is a stormwater pond nearby.

After completing placement of cellular concrete, a 6-inch to a maximum of 1-foot layer of crushed rock should be placed over the cellular concrete to allow for leveling prior to placement of pavement.

4.7.1 Equipment Restrictions

Excavations for the lightweight cellular concrete installation can be accomplished with conventional excavating equipment such as track hoes. To reduce loading on the pipe during construction, we recommend excavators and other machinery needed for the work be limited to lightweight, tracked equipment.

4.7.2 Subgrade Preparation

Prior to installation, the upper two feet of the ground should be removed, along with all organic top soils, debris, sharp objects, and trees. Tree stumps shall be either removed or cut to the level of the ground surface. All wheel tracks or ruts more than 3 inches in depth shall be graded smooth or otherwise filled with soil to provide a reasonable smooth surface. The bottom of the excavation should be free of water prior to placement of lightweight cellular concrete.

4.7.3 Structural Fill and Compaction

All fill placed below pavements or sidewalks should meet the requirements of Crushed Surfacing Base Course (CSBC) materials meeting the requirements of Section 9-03.9(3) of the WSDOT *Standard Specifications* (WSDOT, 2018). The native on-site soils will not be suitable for re-use as structural fill, given their high fines content.

Structural fill should be placed in loose, horizontal, lifts of not more than 6 inches in thickness and compacted to at least 95% of the maximum dry density, as determined using test method ASTM D 1557 (Modified Proctor). At the time of placement, the moisture content of structural fill should be at or near optimum. The procedures required to achieve the specified minimum relative compaction depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and the soil moisture-density properties

5. CONDITIONS AND LIMITATIONS

We have prepared this report for the City of Lake Stevens and Lochner H.W. for use in design of this project. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as our warranty of the subsurface conditions. Experience has shown that soil and ground water conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations and may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HWA should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend HWA be retained to review the plans and specifications to verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm the conditions encountered are consistent with those indicated by the explorations, to provide

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recommendations for design changes should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, HWA attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in the area at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or ground water at this site.

HWA does not practice or consult in the field of safety engineering. We do not direct the contractor's operations and cannot be responsible for the safety of personnel other than our own on the site. As such, the safety of others is the responsibility of the contractor(s). The contractor(s) should notify the owner if it is considered that any of the recommended actions presented herein are unsafe.

We appreciate the opportunity to provide geotechnical services on this project. Should you have any questions or comments, or if we may be of further service, please do not hesitate to call.

Sincerely,

HWA GEOSCIENCES INC.

Fadzilah (Dila) Saidin, Ph.D. P.E.
Senior Geotechnical Engineer

JoLyn Gillie, P.E.
Geotechnical Engineer, Principal

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FIGURES (Following Text)

Figure 1	Vicinity Map
Figures 2	Site and Cellular Concrete Embankment Plan
Figure 3	Section A-A', Profile Through the Centerline
Figure 4A	Section B-B' Conceptual Cellular Concrete Fill Detail
Figure 4B	Section C-C' Conceptual Cellular Concrete Fill Detail
Figure 4C	Section D-D' Conceptual Cellular Concrete Fill Detail

Appendix A: Logs and Test Results from Previous Exploration

6. REFERENCES

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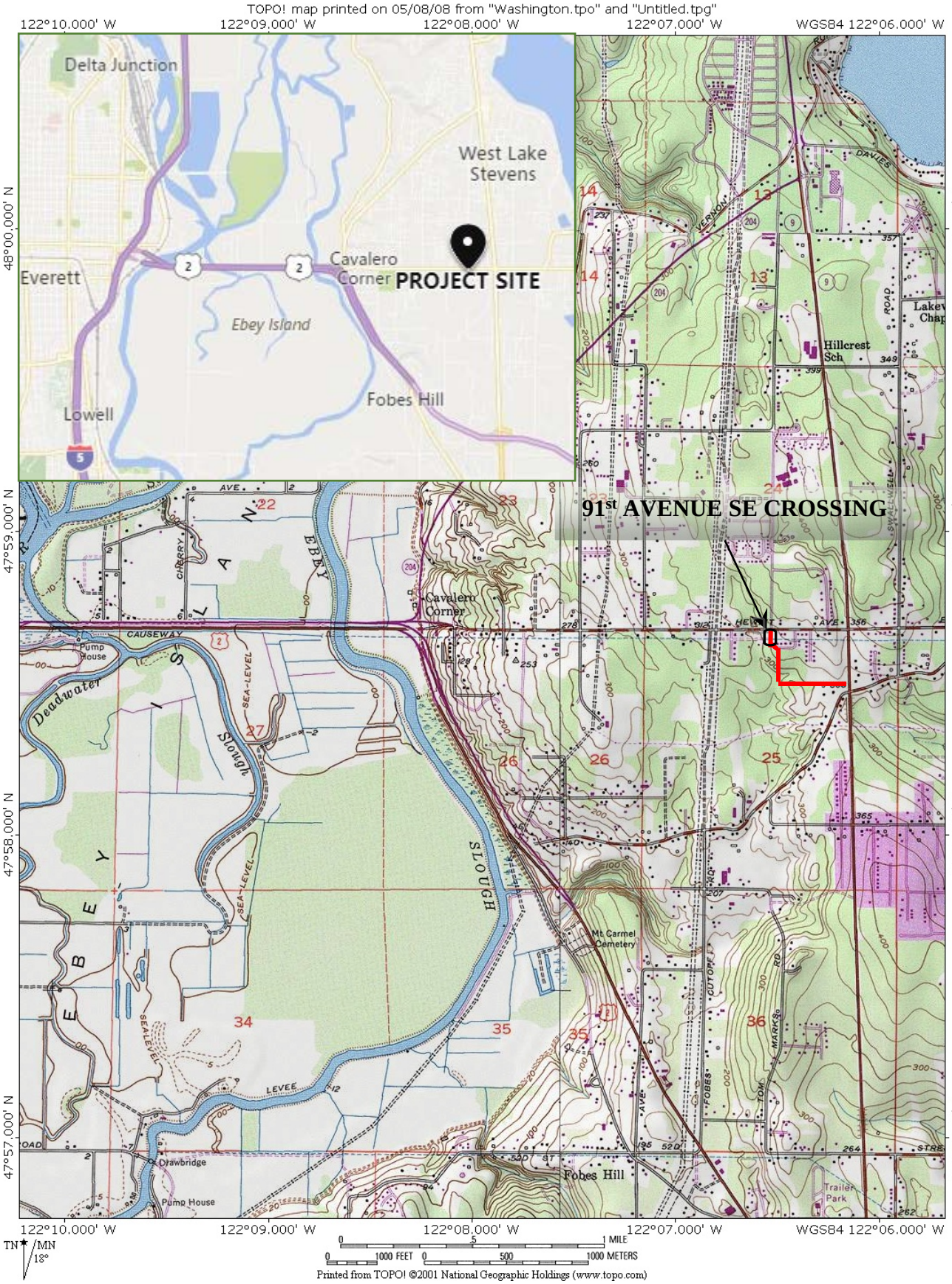


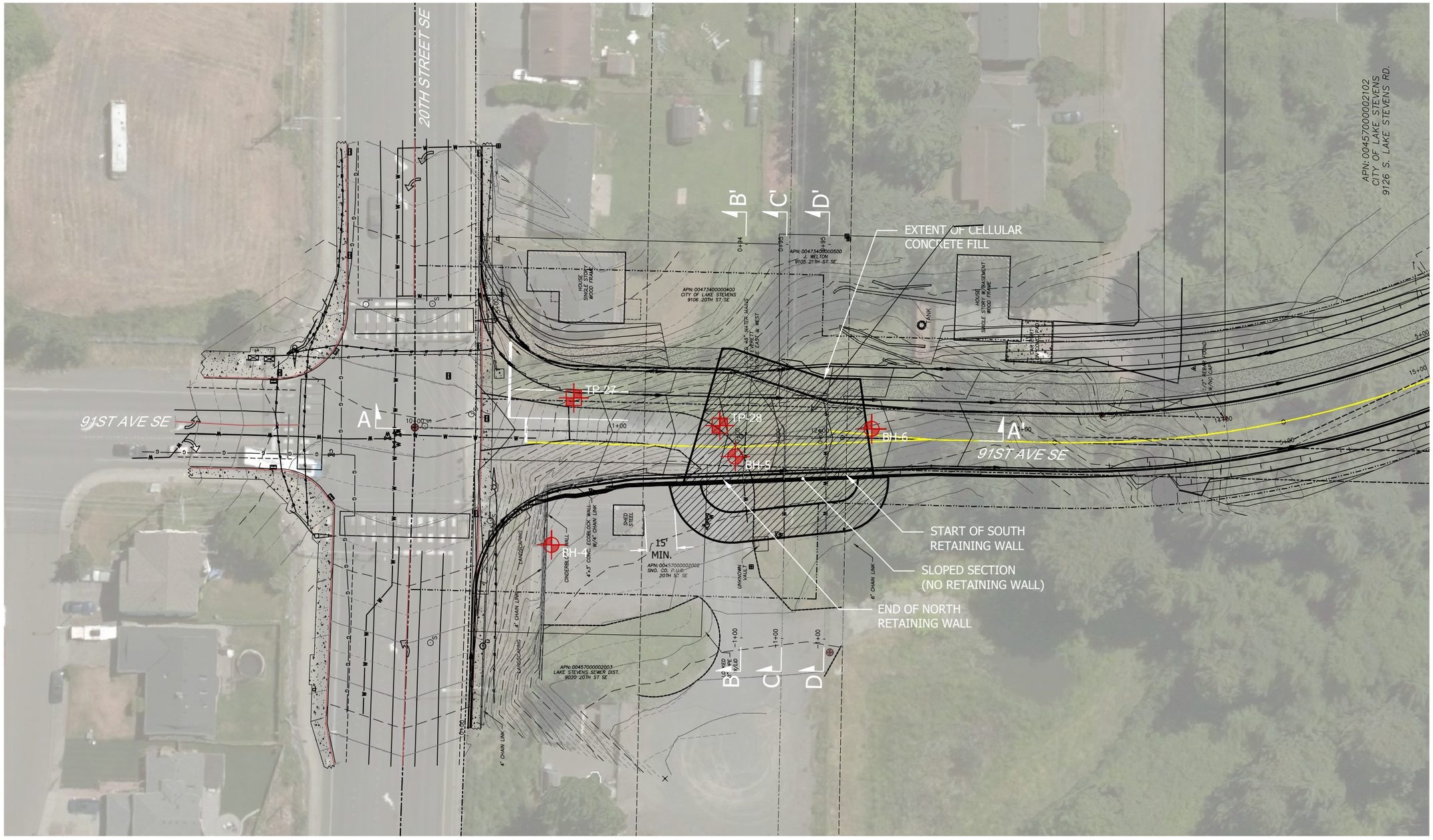
VICINITY MAP

24TH STREET SE AND 91ST AVENUE SE EXTENSION PROJECT
91ST AVENUE CROSSING
LAKE STEVENS, WASHINGTON

FIGURE NO. 1

PROJECT NO. 2017-039

HWA GEOSCIENCES INC.



0' 25' 50' 100'
SCALE: 1"=50'

LEGEND

BH-3 — BORE HOLE DESIGNATION AND APPROXIMATE LOCATION FOR CURRENT STUDY

TP-27 — TEST PIT DESIGNATION AND APPROXIMATE LOCATION FOR CURRENT STUDY

BASE MAP PROVIDED BY: LOCHNER

HWA GEOSCIENCES INC.

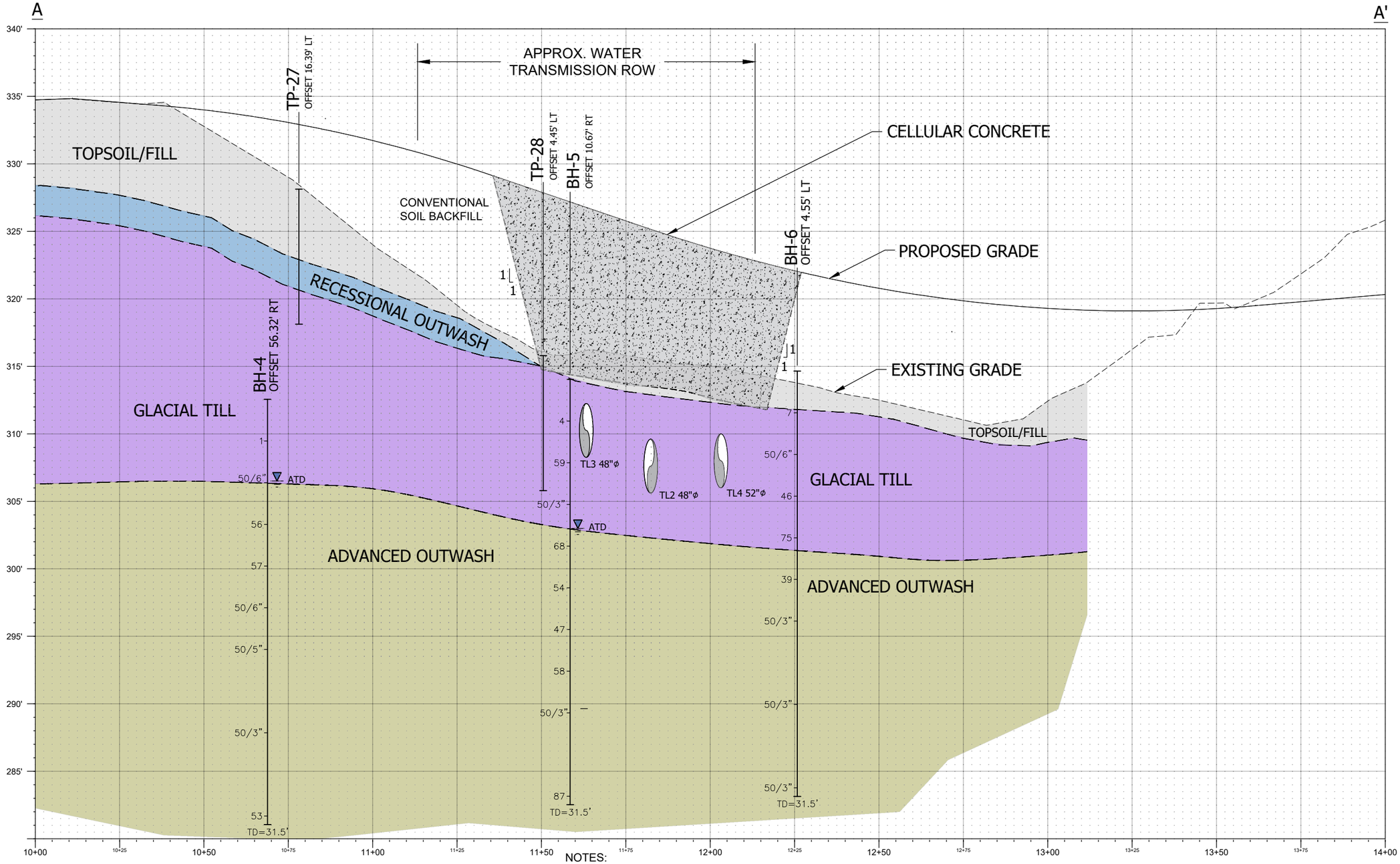
24th STREET SE AND 91st AVENUE SE
LAKE STEVENS, WASHINGTON

CELLULAR CONCRETE EMBANKMENT PLAN

DRAWN BY: BFM
CHECK BY: DS
DATE 04.02.2018

FIGURE NO. 2
PROJECT NO. 2017-039-21
TASK 100-1400

S:\2017 PROJECTS\2017-039-21 24TH STREET FINAL DESIGN\CAD\HWA 2017-039-21 TASK 100-1400.DWG <Fig 2 (Cellular Concrete)> Plotted: 4/9/2019 12:02 PM



LEGEND

BH-3 — EXPLORATION DESIGNATION

SM — USCS SOIL CLASS

30 — BLOW COUNT "N-VALUE"

WATER LEVEL AT TIME OF EXPLORATION

INFERRED GEOLOGIC CONTACT

46.5 — BOTTOM OF EXPLORATION

SOIL LEGEND

- TOPSOIL / FILL: LOOSE DARK BROWN SAND TO DARK GRAY SILTY SAND
- RECESSIONAL OUTWASH: MEDIUM DENSE, SILTY, GRAVELLY SAND WITH ABUNDANT SUB-ROUNDED GRAVEL
- GLACIAL TILL: VERY DENSE, OLIVE BROWN, SILTY SAND WITH GRAVEL
- ADVANCED OUTWASH: VERY DENSE, GRAY, SAND TO SILTY SAND WITH GRAVEL

NOTES:

1. LOCATIONS AND DEPTHS OF PIPELINES ARE INFERRED THROUGH POTHOLING.
2. THE SUBSURFACE CONDITIONS SHOWN ARE BASED ON WIDELY SPACED BORINGS AND SHOULD BE CONSIDERED APPROXIMATE. FURTHER, THE CONTACT LINES SHOWN BETWEEN UNITS ARE INTERPRETIVE IN NATURE AND MAY VARY LATERALLY OR VERTICALLY OVER RELATIVELY SHORT DISTANCES ON SITE.

SCALE 5V:25H

BASE MAP PROVIDED BY: LOCHNER

HWA GEOSCIENCES INC.

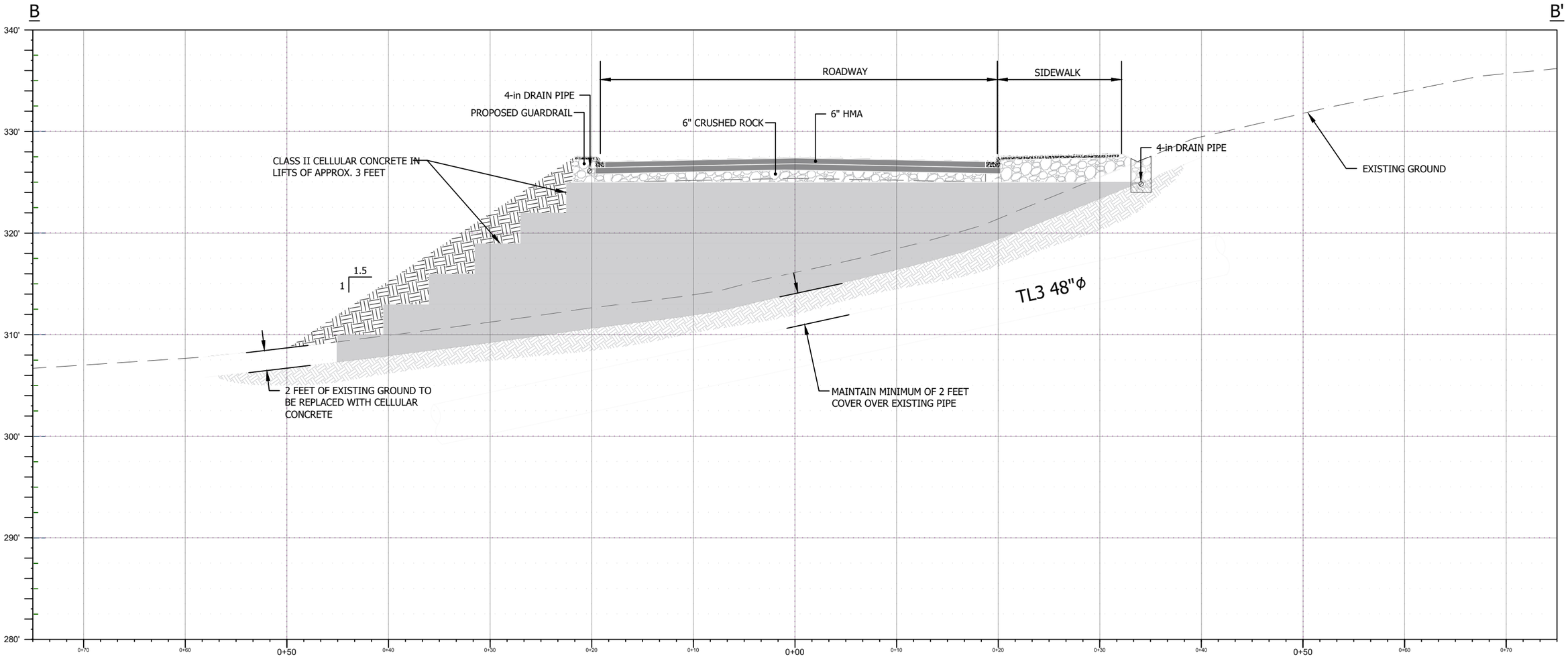
24TH STREET SE AND 91ST AVENUE SE EXTENSION PROJECT
91ST AVENUE CROSSING
LAKE STEVENS, WASHINGTON

SECTION A-A'
PROFILE THROUGH THE CENTERLINE

DRAWN BY: BFM
CHECK BY: DS
DATE 04.02.2018

FIGURE NO. 3
PROJECT NO. 2017-039-21
TASK 100-1400

S:\2017 PROJECTS\2017-039-21 24TH STREET FINAL DESIGN\CAD\HWA 2017-039-21 TASK 100-1300.DWG <Fig 3> Plotted: 4/9/2019 11:52 AM



GENERAL NOTES:

1. PROPOSED CONCEPTUAL CROSS-SECTION (ACTUAL GEOMETRY MAY VARY BASED ON CONTRACTOR MEANS AND METHODS).
2. LOCATIONS AND DEPTHS OF PIPELINES ARE INFERRED FROM POTHOLING INFORMATION.
3. 4-INCH DIAMETER, SCHEDULE 40, PERFORATED, PLASTIC DRAIN. THE SUB-DRAIN PIPE DISCHARGE SHOULD BE TIGHT LINED TO A SUITABLE OUTLET TO THE SOUTH.

SCALE 10V:10H

BASE MAP PROVIDED BY: LOCHNER

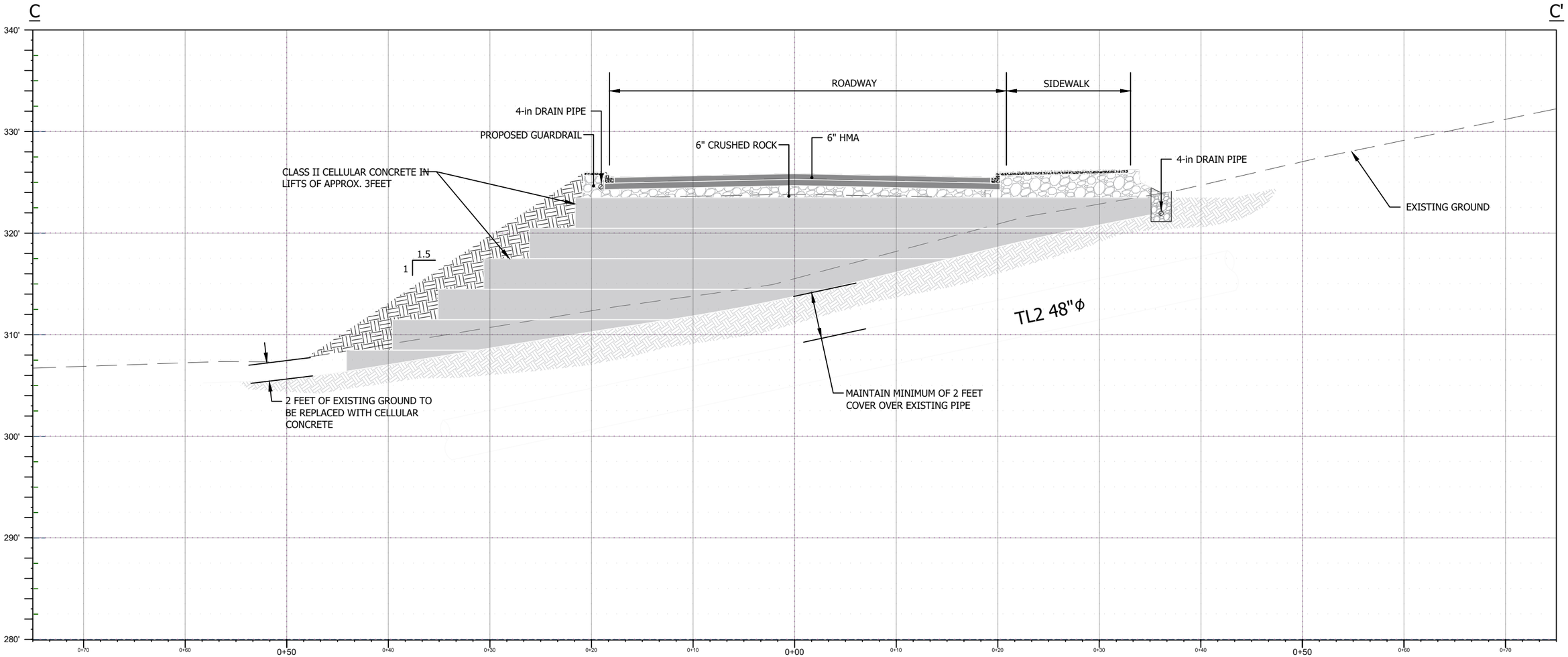
HWA GEOSCIENCES INC.

24TH STREET SE AND 91ST AVENUE SE EXTENSION PROJECT
91ST AVENUE CROSSING
LAKE STEVENS, WASHINGTON

SECTION B-B'
CONCEPTUAL CELLULAR CONCRETE FILL DETAIL

DRAWN BY: BFM
CHECK BY: DS
DATE 04.02.2018

FIGURE NO. 4A
PROJECT NO. 2017-039-21
TASK 100-1400



GENERAL NOTES:

1. PROPOSED CONCEPTUAL CROSS-SECTION (ACTUAL GEOMETRY MAY VARY BASED ON CONTRACTOR MEANS AND METHODS).
2. LOCATIONS AND DEPTHS OF PIPELINES ARE INFERRED FROM POTHOLING INFORMATION.
3. 4-INCH DIAMETER, SCHEDULE 40, PERFORATED, PLASTIC DRAIN. THE SUB-DRAIN PIPE DISCHARGE SHOULD BE TIGHT LINED TO A SUITABLE OUTLET TO THE SOUTH.

SCALE 10V:10H

BASE MAP PROVIDED BY: LOCHNER

HWA GEOSCIENCES INC.

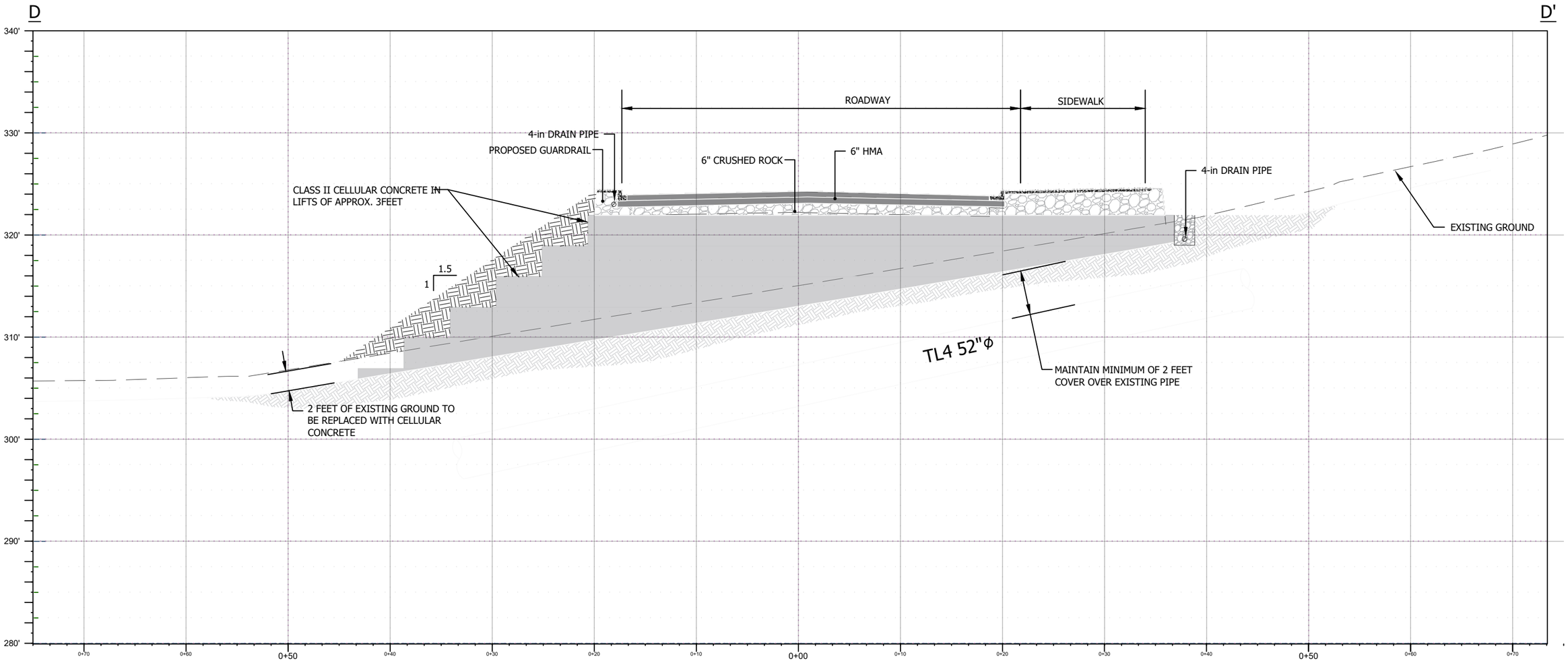
24TH STREET SE AND 91ST AVENUE SE EXTENSION PROJECT
91ST AVENUE CROSSING
LAKE STEVENS, WASHINGTON

SECTION C-C'
CONCEPTUAL CELLULAR CONCRETE FILL DETAIL

DRAWN BY: BFM
CHECK BY: DS
DATE 04.02.2018

FIGURE NO. 4B
PROJECT NO. 2017-039-21 TASK 100-1400

S:\2017 PROJECTS\2017-039-21 24TH STREET FINAL DESIGN\CAD\HWA 2017-039-21 TASK 100-1400.DWG <Fig (Section C)> Plotted: 4/9/2019 7:07 PM



GENERAL NOTES:

1. PROPOSED CONCEPTUAL CROSS-SECTION (ACTUAL GEOMETRY MAY VARY BASED ON CONTRACTOR MEANS AND METHODS).
2. LOCATIONS AND DEPTHS OF PIPELINES ARE INFERRED FROM POTHOLING INFORMATION.
3. 4-INCH DIAMETER, SCHEDULE 40, PERFORATED, PLASTIC DRAIN. THE SUB-DRAIN PIPE DISCHARGE SHOULD BE TIGHT LINED TO A SUITABLE OUTLET TO THE SOUTH.

SCALE 10V:10H

BASE MAP PROVIDED BY: LOCHNER

HWA GEOSCIENCES INC.

24TH STREET SE AND 91ST AVENUE SE EXTENSION PROJECT
91ST AVENUE CROSSING
LAKE STEVENS, WASHINGTON

SECTION D-D'
CONCEPTUAL CELLULAR CONCRETE FILL DETAIL

DRAWN BY: BFM
CHECK BY: DS
DATE 04.02.2018

FIGURE NO. 4C
PROJECT NO. 2017-039-21 TASK 100-1400

APPENDIX A

LOG AND TEST RESULTS FROM PREVIOUS EXPLORATION

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density(%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

USCS SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
	More than 50% of Coarse Fraction Retained on No. 4 Sieve	Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils	Clean Sand (little or no fines)	SW	Well-graded SAND
More than 50% Retained on No. 200 Sieve Size			SP	Poorly-graded SAND
	50% or More of Coarse Fraction Passing No. 4 Sieve	Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILT/Organic CLAY
Highly Organic Soils			PT	PEAT

TEST SYMBOLS

%F	Percent Fines
AL	Atterberg Limits: PL = Plastic Limit, LL = Liquid Limit
CBR	California Bearing Ratio
CN	Consolidation
DD	Dry Density (pcf)
DS	Direct Shear
GS	Grain Size Distribution
K	Permeability
MD	Moisture/Density Relationship (Proctor)
MR	Resilient Modulus
PID	Photoionization Device Reading
PP	Pocket Penetrometer Approx. Compressive Strength (tsf)
SG	Specific Gravity
TC	Triaxial Compression
TV	Torvane Approx. Shear Strength (tsf)
UC	Unconfined Compression

SAMPLE TYPE SYMBOLS

- 2.0" OD Split Spoon (SPT) (140 lb. hammer with 30 in. drop)
- Shelby Tube
- 3-1/4" OD Split Spoon with Brass Rings
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- Non-standard Penetration Test (3.0" OD split spoon)

GROUNDWATER SYMBOLS

- Groundwater Level (measured at time of drilling)
- Groundwater Level (measured in well or open hole after water level stabilized)

COMPONENT DEFINITIONS

COMPONENT	SIZE RANGE
Boulders	Larger than 12 in
Cobbles	3 in to 12 in
Gravel	3 in to No 4 (4.5mm)
Coarse gravel	3 in to 3/4 in
Fine gravel	3/4 in to No 4 (4.5mm)
Sand	No. 4 (4.5 mm) to No. 200 (0.074 mm)
Coarse sand	No. 4 (4.5 mm) to No. 10 (2.0 mm)
Medium sand	No. 10 (2.0 mm) to No. 40 (0.42 mm)
Fine sand	No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt and Clay	Smaller than No. 200 (0.074mm)

COMPONENT PROPORTIONS

PROPORTION RANGE	DESCRIPTIVE TERMS
< 5%	Clean
5 - 12%	Slightly (Clayey, Silty, Sandy)
12 - 30%	Clayey, Silty, Sandy, Gravelly
30 - 50%	Very (Clayey, Silty, Sandy, Gravelly)
Components are arranged in order of increasing quantities.	

NOTES: Soil classifications presented on exploration logs are based on visual and laboratory observation. Soil descriptions are presented in the following general order:

Density/consistency, color, modifier (if any) GROUP NAME, additions to group name (if any), moisture content. Proportion, gradation, and angularity of constituents, additional comments. (GEOLOGIC INTERPRETATION)

Please refer to the discussion in the report text as well as the exploration logs for a more complete description of subsurface conditions.

MOISTURE CONTENT

DRY	Absence of moisture, dusty, dry to the touch.
MOIST	Damp but no visible water.
WET	Visible free water, usually soil is below water table.

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

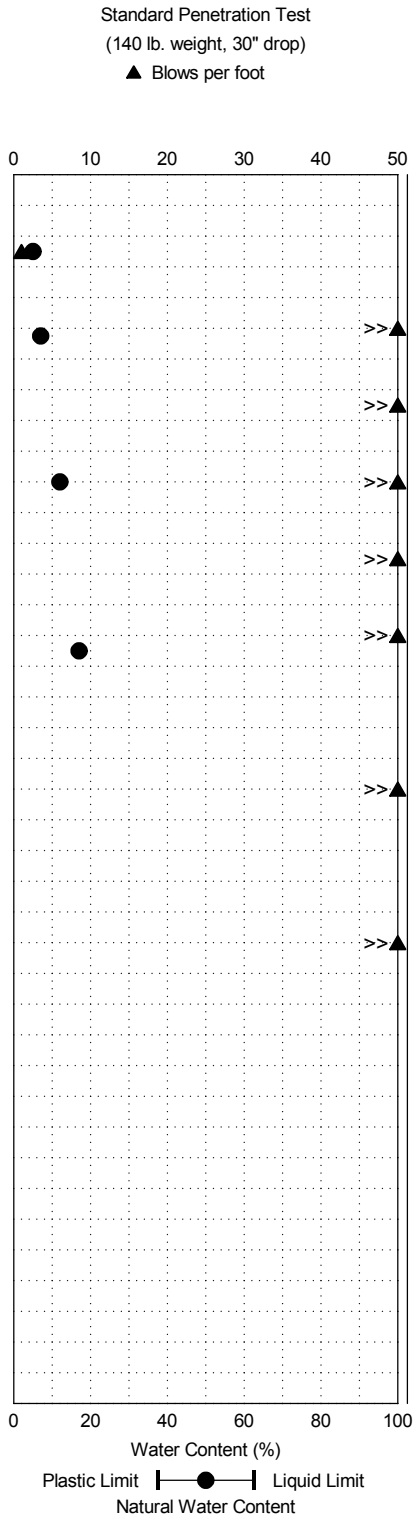
PROJECT NO.: 2017-039-21 FIGURE: A-1

DRILLING COMPANY: Environmental Drilling Inc.
DRILLING METHOD: Track-mounted B-53, HSA
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figures 2A-2B

DATE STARTED: 3/6/2018
DATE COMPLETED: 3/7/2018
LOGGED BY: S. Khandaker

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	SM	Loose, dark gray, gravelly, silty SAND, moist. **(FILL)**			
	SM	Very loose, dark gray, gravelly, SAND, moist. Subrounded gravel.	S-1	3-1-0	
5	GP SM	Very dense, gray, angular GRAVEL, moist. Pounding on a rock.	S-2a S-2b	50/6"	%F
		Very dense, olive-brown, silty SAND, with rounded gravel, moist. **(GLACIAL TILL)**	S-3	15-25-31	
10		Same, but lighter gray in color and gravel pieces more angular. Rust mottling observed. Very dense, yellowish brown, silty SAND with gravel, moist. Rust mottling observed.	S-4	17-22-35	GS AL
	GP	Very dense, dark gray, medium to coarse sandy, sub-angular GRAVEL, moist. Extensive rust mottling at the toe of sampler.	S-5	50/6"	
15	SM SP SM	**(ADVANCED OUTWASH)** Very dense, dark gray, very gravelly, coarse to medium SAND, wet.	S-6a S-6b	32-48-50/5"	
		Very dense, gray-brown, fine to medium SAND, moist.			
20		Very dense, gray, silty, gravelly SAND, wet. Sample transitions from fine to medium sand to fine gravel at the toe. 3 feet of heave. Mud added to auger.	S-7	11-27-50/3"	
25	SM	Very dense, brown-gray, gravelly, silty SAND, moist. Some rust mottling observed.	S-8	4-11-42	
30		Same with finer particles of sand.			
35		Borehole terminated at 31.5 feet below ground surface (bgs). Ground water seepage observed at 18 feet bgs. Borehole abandoned with bentonite chips.			
40					

GROUNDWATER



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

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

BORING:
BH-4

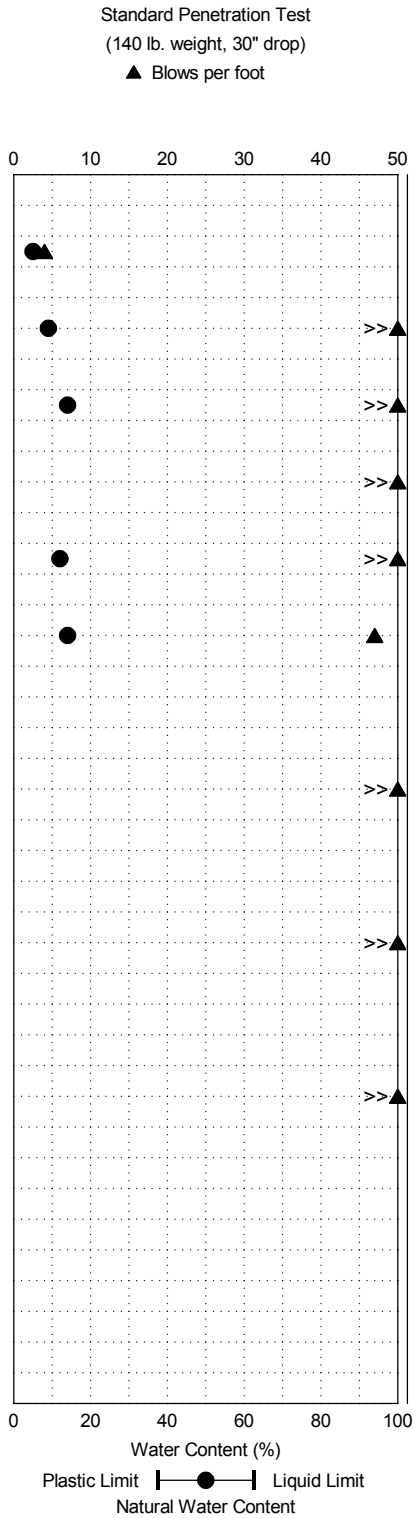
PROJECT NO.: 2017-039-21 FIGURE: A-2

DRILLING COMPANY: Environmental Drilling Inc.
DRILLING METHOD: Track-mounted B-53, HSA
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figures 2A-2B

DATE STARTED: 3/6/2018
DATE COMPLETED: 3/6/2018
LOGGED BY: S. Khandaker

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		Loose, brown, silty, medium to fine SAND, moist. **(TOPSOIL)**				
	GP	Very loose, gray, subangular GRAVEL, moist. Some topsoil present. **(FILL)**		S-1	11-3-1	
5	SM	Very dense, olive brown, silty, gravelly, fine to medium SAND, moist. **(GLACIAL TILL)**		S-2	4-27-32	
		Very dense, yellowish-brown, silty SAND, moist.		S-3	50/3	GS AL
10		Same, with bigger rounded medium gravel.		S-4	27-32-36	
		Very dense, olive brown, silty SAND with gravel, moist.		S-5a	11-26-28	GS AL
	SM	Very dense, red, silty, fine SAND, moist.		S-5b		
15	SM	**(ADVANCED OUTWASH)** Dense, gray, gravelly, silty, fine SAND, moist. 1" layer of red and black, fine sand with white minerals in the middle of sample. (Sample 6-b)		S-6	16-21-26	
20		Very dense, gray, gravelly, silty, SAND, moist. Some rust mottling observed.		S-7	15-16-42	
25	SP SM	Very dense, gray, silty, medium SAND, moist. Some rounded gravel pieces.		S-8	36-50/3	
30	SP SM	Very dense, gray, silty, gravelly, medium SAND, moist.		S-9	11-37-50	
35		Borehole terminated at 31.5 feet below ground surface (bgs). Ground water seepage observed at 21 feet bgs. Borehole abandoned with bentonite chips.				
40						

GROUNDWATER



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

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

BORING:
BH-5

PROJECT NO.: 2017-039-21 FIGURE: A-3

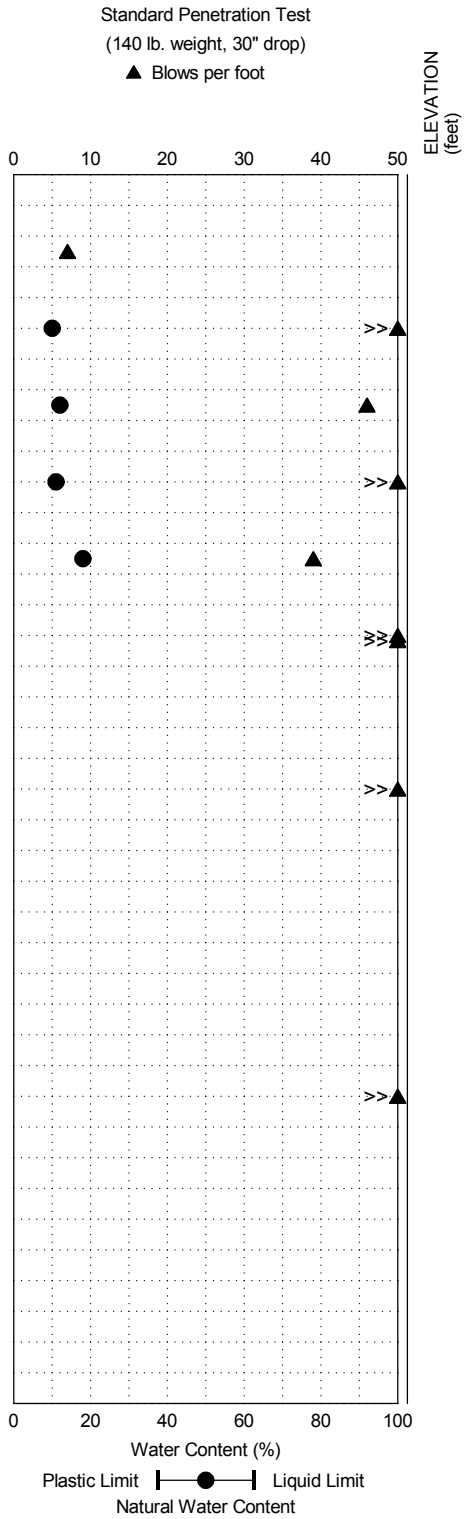
DRILLING COMPANY: Environmental Drilling Inc.
DRILLING METHOD: Track-mounted B-53, HSA
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figures 2A-2B

DATE STARTED: 3/7/2018
DATE COMPLETED: 3/7/2018
LOGGED BY: S. Khandaker

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)
0		Loose, dark brown, silty, fine to medium SAND, moist. **(TOPSOIL)**		
	SM	Loose, olive brown, silty, slightly gravelly, fine SAND, moist. **(FILL)**	S-1a, S-1b	3-4-3
5	SM	Very dense, olive brown, silty SAND with gravel, moist. Pounding on a rock. **(GLACIAL TILL)**	S-2	29-50/6"
		Dense, dark grayish brown, silty, SAND with gravel, moist. Gravel subrounded. Some minor rust mottling.	S-3	18-21-25
10		More gravel peices and some white minerals observed.	S-4	11-33-42
	SP	Very dense, gray, medium SAND, with some gravel pieces, moist. **(ADVANCED OUTWASH)**	S-5	11-13-26
15	SM	Mud added to auger after water observed on rods, to prevent heave.	S-6, S-7	42-46-50/3", 6-50/3"
		Very dense, gray, slightly gravelly, silty, fine to medium SAND, moist. White minerals and rust mottling observed.		
20	SP SM	Very dense, gray, silty, medium to coarse SAND, wet. Pounding on a rock.	S-8a	32-50/3"
25		Very dense, gray, silty, fine to coarse SAND, moist. Sample gets coarser with depth.	S-8b	
	SP SM	Very dense, gray, silty, fine to medium SAND, moist.		
30		No recovery.	S-9	24/50/3"
35				
40				

Borehole terminated at 31.5 feet below ground surface (bgs).
Groundwater seepage observed at 13 feet bgs.
Borehole abandoned with bentonite chips.

GROUNDWATER



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

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

BORING:
BH-6

PROJECT NO.: 2017-039-21 FIGURE: A-4

EXCAVATION COMPANY: City of Lake Stevens
EXCAVATING EQUIPMENT: Yanmar Rubber Tracked Excavator

LOCATION: See Figures 2A-2B
DATE COMPLETED: 9/14/17
LOGGED BY: Z. Ngoma

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose, dark brown, silty, gravelly SAND, dry. With abundant roots and rootlets. **(TOPSOIL)**		S-1		
		SM	Medium dense, olive-gray to light olive-brown, silty, gravelly SAND, moist. **(FILL)**		S-2	5	
5			Medium dense, dark brown, silty, gravelly SAND, moist. Occasional cobbles and minor organics. **(BURIED TOPSOIL)**		S-3		
		GM	Medium dense, yellow-brown, silty, sandy, GRAVEL, moist. **(RECESSIONAL OUTWASH)**		S-4	7	
10		SM	Very dense, olive-gray, silty, gravelly SAND, moist. **(GLACIAL TILL)**		S-5		
15			Test pit terminated at 10 feet below ground surface. No caving. No groundwater seepage observed at the time of exploration.				

TEST PIT PHOTO

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

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

LOG OF TEST PIT
TP-27

PROJECT NO.: 2017-039-21 FIGURE: C-2

EXCAVATION COMPANY: City of Lake Stevens
EXCAVATING EQUIPMENT: Yanmar Rubber Tracked Excavator

LOCATION: See Figures 2A-2B
DATE COMPLETED: 9/14/17
LOGGED BY: Z. Ngoma

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Loose, dark brown, silty, gravelly SAND, dry. With abundant roots and rootlets. **(TOPSOIL)**				
		SM	Medium dense, olive-gray to light olive-brown, silty, gravelly SAND, moist. **(FILL)**		S-1		
			Medium dense, dark brown, silty, gravelly SAND, moist. Occasional cobbles and minor organics. **(BURIED TOPSOIL)**		S-2		
5		SM	Medium dense, yellow-brown, silty, gravelly SAND, moist. **(RECESSIONAL OUTWASH)**				
		SM	Very dense, olive-gray, silty, gravelly SAND, moist. **(GLACIAL TILL)**		S-3	8	
10			Test pit terminated at 10 feet below ground surface. No caving. No groundwater seepage observed at the time of exploration.				
15							

TEST PIT PHOTO

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

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

LOG OF TEST PIT
TP-28

PROJECT NO.: 2017-039-21 FIGURE: C-3

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
BH-1,S-8	20.0	21.5	20.5			31	24	7				ML	Gray, SILT
BH-2,S-2	5.0	6.0	8.5						24.5	45.5	29.9	SM	Light olive-brown, silty SAND with gravel
BH-2,S-4	10.0	10.5	6.2						19.1	54.8	26.2	SM	Gray, silty SAND with gravel
BH-3,S-2	5.0	6.5	8.9						24.5	45.9	29.6	SM	Grayish-brown, silty SAND with gravel
BH-3,S-9	25.0	26.5	25.0			34	27	7				ML	Gray, SILT
TP-16,S-1	2.5	3.0	5.9						72.9	24.6	2.5	GP	Yellowish-brown, poorly graded GRAVEL with sand
TP-16,S-3	4.5	5.0	6.0						36.3	40.3	23.4	SM	Gray, silty SAND with gravel
TP-17,S-2	2.0	2.5	5.5						36.2	43.1	20.7	SM	Light olive-brown, silty SAND with gravel
TP-17,S-4	4.0	4.5	5.6						34.0	40.6	25.3	SM	Light olive-brown, silty SAND with gravel
TP-18,S-1	2.5	3.0	4.8						63.9	28.8	7.3	GW-GM	Yellowish-brown, well-graded GRAVEL with silt and sand
TP-19,S-1	2.5	3.0	27.8									SM	Brownish-yellow, silty SAND with gravel
TP-19,S-2	5.5	6.0	7.5						27.6	42.3	30.1	SM	Dark gray, silty SAND with gravel
TP-20,S-1	2.5	3.0	12.0									SM	Brownish-yellow, silty SAND with gravel
TP-21,S-1	1.5	2.0	11.3									SM	Brownish-yellow, silty SAND with gravel
TP-21,S-2	2.5	3.0	10.9						24.0	47.0	29.0	SM	Gray, silty SAND with gravel
TP-21,S-3	6.0	6.5	13.8						25.4	44.1	30.5	SM	Gray, silty SAND with gravel
TP-22,S-1	1.5	2.0	7.3						42.4	44.6	13.0	SM	Brownish-yellow, silty SAND with gravel
TP-22,S-2	4.0	4.5	9.0						21.4	49.2	29.4	SM	Light olive-brown, silty SAND with gravel
TP-23,S-1	3.5	4.0	8.4						30.0	46.0	24.0	SM	Brownish-yellow, silty SAND with gravel
TP-24,S-1	1.5	2.0	19.6									SM	Brownish-yellow, silty SAND with gravel

Notes: 1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

SUMMARY OF
MATERIAL PROPERTIES

PROJECT NO.: 2017-039-21 FIGURE: B-1

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
TP-25,S-1	2.5	3.0	7.7									SM	Olive-brown, silty SAND with gravel
TP-26,S-1	3.0	3.5	6.7						52.9	28.6	18.5	GM	Brownish-yellow, silty GRAVEL with sand
TP-27,S-2	2.5	3.0	4.9						28.1	35.6	36.3	SM	Light yellowish-brown, silty SAND with gravel
TP-27,S-4	7.0	7.5	7.4						40.4	37.6	21.9	GM	Brownish-yellow, silty GRAVEL with sand
TP-28,S-3	7.5	8.0	7.7						32.5	44.9	22.6	SM	Grayish-brown, silty SAND with gravel

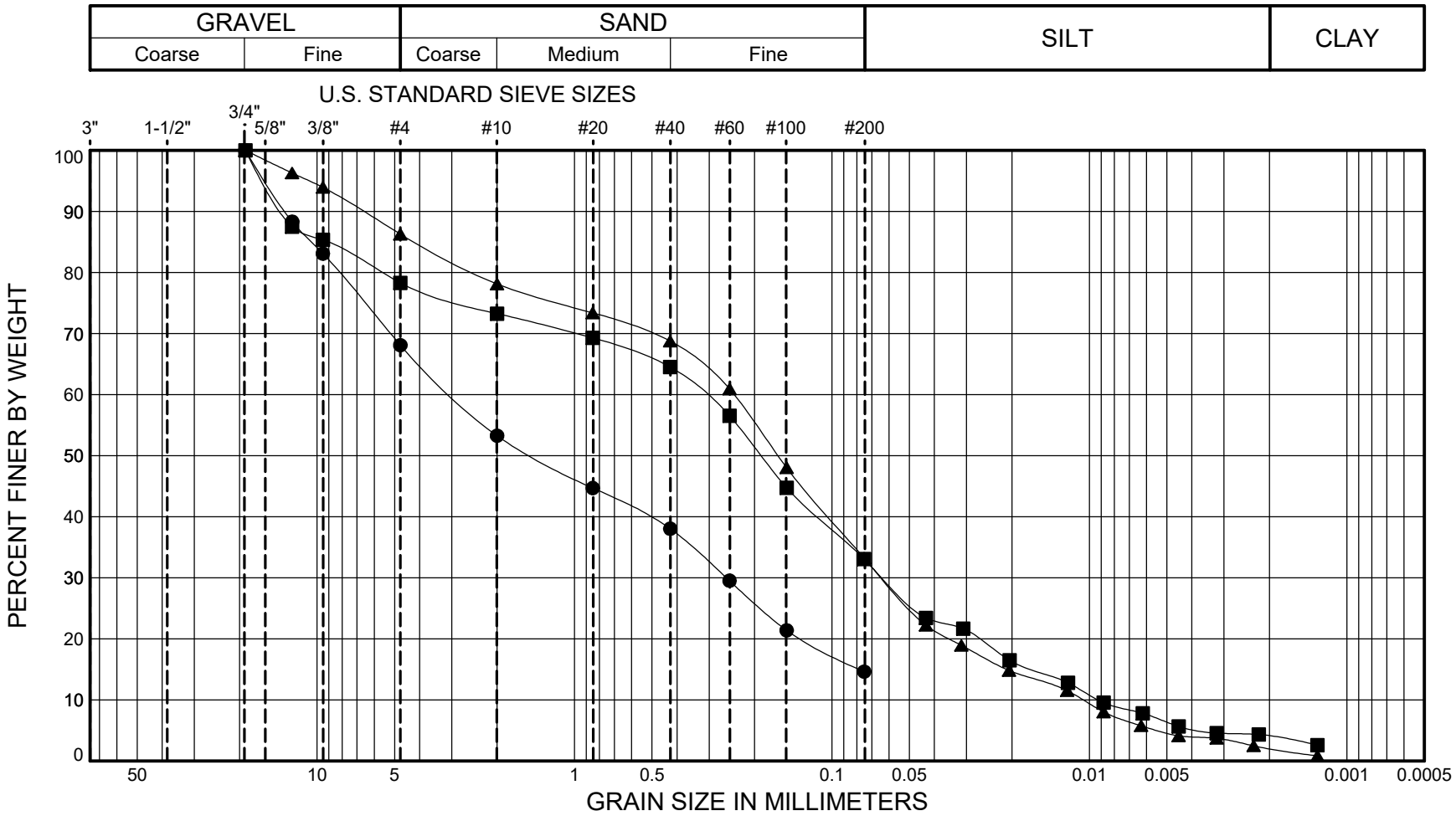
Notes:
1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

SUMMARY OF
MATERIAL PROPERTIES

PROJECT NO.: 2017-039-21 FIGURE: B-1



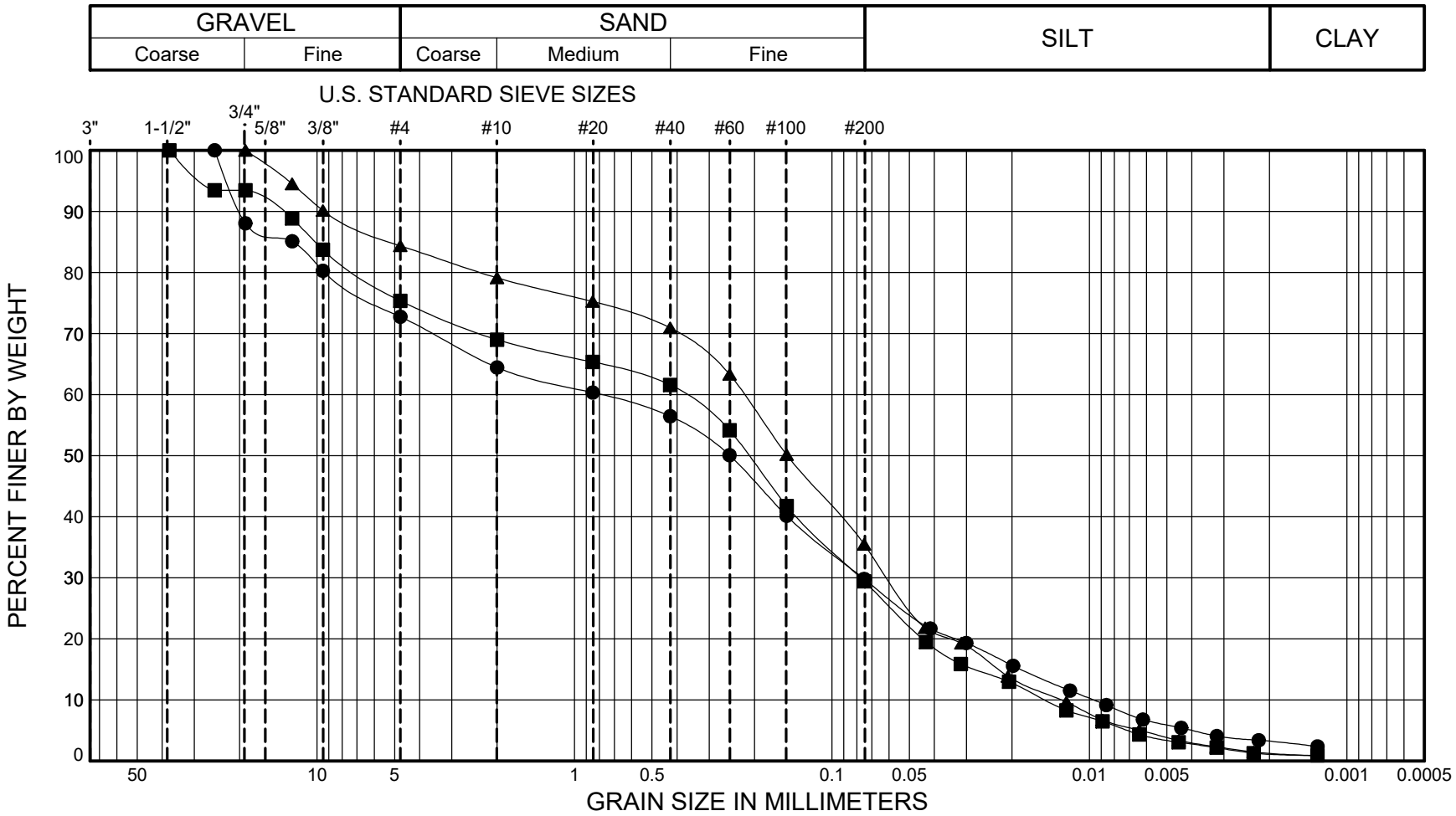
SYMBOL	SAMPLE		DEPTH (ft.)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-4	S-2b	5.3 - 5.5	(SM) Olive-brown, silty SAND with gravel	7				31.9	53.5	14.6
■	BH-4	S-4	10.0 - 11.5	(SM) Yellowish-brown, silty SAND with gravel	12	NP	NP	NP	21.7	45.2	33.1
▲	BH-5	S-3	7.5 - 8.2	(SM) Yellowish-brown, silty SAND	14	NP	NP	NP	13.7	53.1	33.2

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D6913

PROJECT NO.: 2017-039-21 FIGURE: B-3



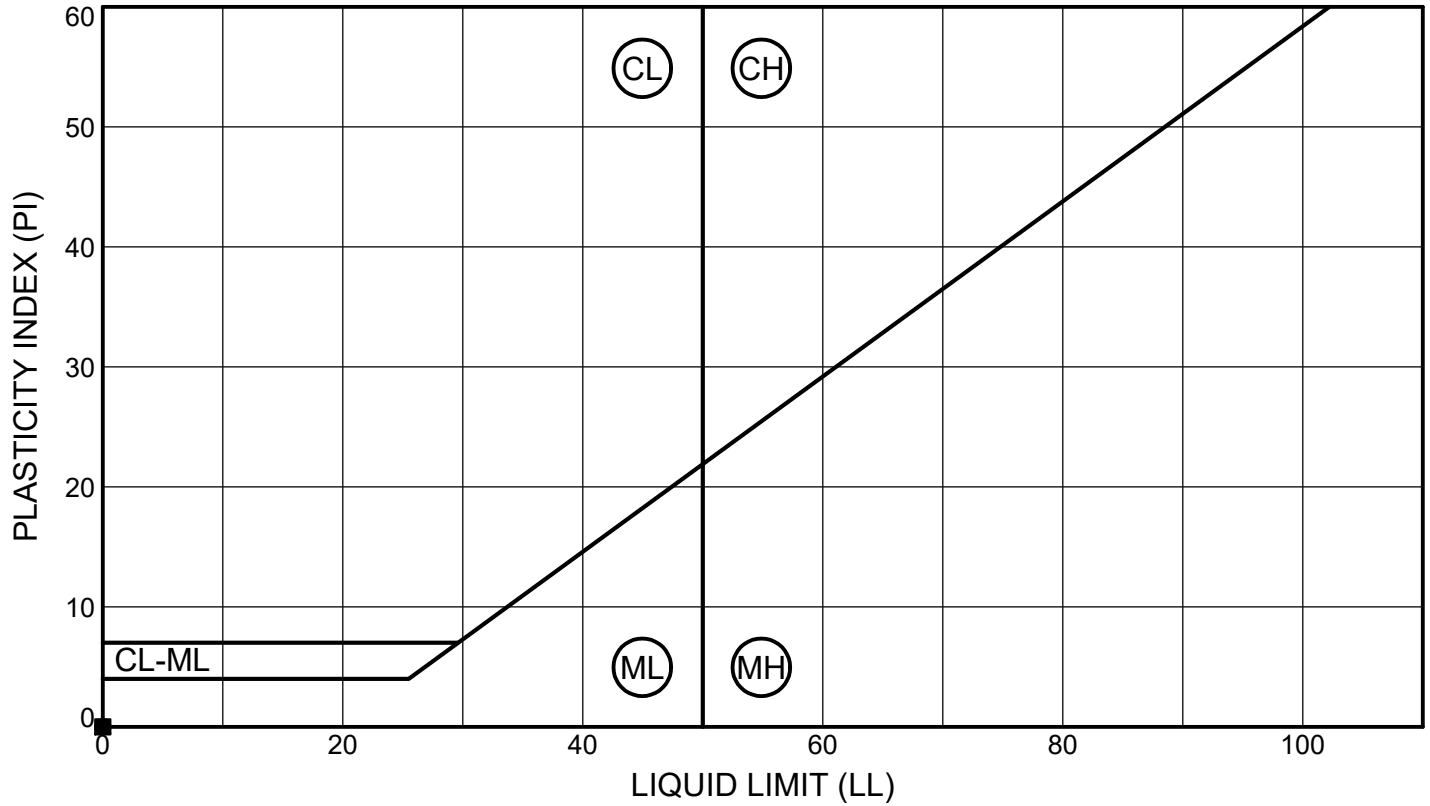
SYMBOL	SAMPLE		DEPTH (ft.)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-5	S-5a	12.5 - 13.8	(SM) Olive-brown, silty SAND with gravel	12	NP	NP	NP	27.3	42.9	29.8
■	BH-6	S-2	5.0 - 5.8	(SM) Olive-brown, silty SAND with gravel	10				24.6	45.9	29.4
▲	BH-6	S-3	7.5 - 9.0	(SM) Dark grayish-brown, silty SAND with gravel	12	NP	NP	NP	15.6	48.9	35.5

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
FINAL DESIGN
LAKE STEVENS, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D6913

PROJECT NO.: 2017-039-21 FIGURE: B-4



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-4	S-4	10.0 - 11.5	(SM) Yellowish-brown, silty SAND with gravel	12	NP	NP	NP	33.1
■	BH-5	S-3	7.5 - 8.2	(SM) Yellowish-brown, silty SAND	14	NP	NP	NP	33.2
▲	BH-5	S-5a	12.5 - 13.8	(SM) Olive-brown, silty SAND with gravel	12	NP	NP	NP	29.8
○	BH-6	S-3	7.5 - 9.0	(SM) Dark grayish-brown, silty SAND with gravel	12	NP	NP	NP	35.5

HWA GEOSCIENCES INC.

24TH STREET SE PROJECT
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LAKE STEVENS, WASHINGTON

LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX OF SOILS
METHOD ASTM D4318

PROJECT NO.: 2017-039-21 FIGURE: B-2