FINAL GEOTECHNICAL ENGINEERING REPORT
Lake to Sound Trail – Westside Section
King County, Washington

HWA Project No. 2010-100-21

Prepared for
Parametrix, Inc.

December 27, 2012

HWA GEOSCIENCES INC.

- Geotechnical Engineering
- Hydrogeology
- Geoenvironmental Services
- Inspection & Testing

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December 27, 2012
HWA Project No. 2010-100

Parametrix, Inc.
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Attention: Ms. Yammie Ho, P.E.

Subject: **FINAL GEOTECHNICAL ENGINEERING REPORT**
Lake to Sound Trail - Westside Section
King County, Washington

Dear Ms. Ho:

We are pleased to present our final geotechnical engineering report prepared in support of the proposed Westside Section of the Lake to Sound Trail, along Des Moines Memorial Drive South in Burien and SeaTac, Washington. The purpose of this study was to evaluate the soil and ground water conditions along the proposed trail alignment and provide geotechnical information for the design of infiltration systems and geotechnical recommendations for design and construction of retaining walls, rockeries, pavement sections, an elevated boardwalk, and general earthworks, as represented in the 60% Project Plans.

We appreciate the opportunity to have provided geotechnical engineering services on this project. If you have any questions regarding this report or require additional information or services, please contact the undersigned at your convenience.

Sincerely,

HWA GEOSCIENCES INC.

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FINAL GEOTECHNICAL ENGINEERING REPORT
LAKE TO SOUND TRAIL – WESTSIDE SECTION
KING COUNTY, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical study performed by HWA GeoSciences Inc. (HWA) for the proposed Westside Section of the Lake to Sound Trail in King County, Washington. The purpose of this study was to evaluate the soil and ground water conditions along the proposed trail alignment and provide geotechnical information for infiltration and recommendations for design and construction of retaining walls, rockeries, pavement sections, an elevated boardwalk, and general earthwork.

1.2 PROJECT DESCRIPTION

The project location and general trail alignment are shown on the Vicinity Map, Figure 1. We understand that the paved trail will be constructed adjacent to the eastern side of Des Moines Memorial Drive South. The trail will extend approximately 1.4 miles between Ambaum Boulevard at the south end and South 156th Street at the north. The existing road embankment will generally be widened to accommodate a 10 to 12 foot wide paved trail with 2-foot gravel shoulders. There will also be a 3-foot wide planting strip separating the trail from the roadway. Near the southern end of the alignment, a section approximately 1,050 feet long will deviate onto South 176th Street and along an existing sewer line easement, before rejoining Des Moines Memorial Parkway, near the north end of Airport Park. A 400-foot section of the trail will be supported on an elevated boardwalk. Curb and gutter will be constructed along the eastern edge of the roadway, as well as catch basins and storm sewers to collect road runoff.

Widening of the existing road embankment will require low sliver fills and relatively short retaining walls to support fill sections along extensive portions of the alignment. Nine Concrete Masonry Unit (CMU) walls will be constructed; some with layers of geogrid creating Mechanically Stabilized Earth (MSE) walls. The fill wall faces will generally range from 2 to 5 feet tall above existing road embankment slopes. Three cut areas will be faced with rockeries, with one of these walls having a maximum wall height of about 8 feet.

1.3 SCOPE OF SERVICES AND AUTHORIZATION

Our scope of work was developed in consultation with Parametrix, and our work to date was performed in general accordance with our proposed scope and cost estimate dated June 25, 2010

and the Subconsultant Agreement executed November 1, 2010 by Parametrix. Eight explorations were advanced along the existing road embankment. Per the original scope of work all embankment explorations were to consist of test pits; however, field conditions dictated that two locations be explored with limited-acceess drilling equipment in order to have room to work and minimize disturbance to existing site features. In addition, two borings were drilled, per our scope of work, at the toe of the road embankment adjacent to wetlands for foundation design of a boardwalk.

Laboratory testing was conducted on selected soil samples obtained from the explorations to determine relevant engineering properties.

The objective of our investigation was to provide recommendations for the design of fill MSE retaining walls and a cut slope rockery, provide long-term infiltration rates for stormwater management and geotechnical recommendations for pile and spread footing design. In addition, recommendations are provided for pavement design, site preparation, fill placement, subgrade preparation, and provisions for wet weather earthwork.

2.0 FIELD AND LABORATORY TESTING

2.1 FIELD INVESTIGATION

The field investigation included review of available geologic and geotechnical data for the project corridor, surface reconnaissance of the alignment, excavating six test pits, designated TP-1 through TP-6, and drilling four boreholes, designated BH-1 through BH-4. Borings BH-1 and BH-2 were drilled at the toe of the road embankment for the proposed pile-supported section of the trail. Borings BH-3 and BH-4 were drilled within the existing road embankment. The locations of our borings are presented on Figures 2A through 2Q (Site and Exploration Plans). Additional information pertaining to the subsurface investigations and summary exploration logs are presented in Appendix A.

2.2 LABORATORY TESTING

Laboratory tests were conducted at HWA's laboratory in Bothell, Washington, on selected samples retrieved from the test pits and borings to determine relevant index and engineering properties of the soils encountered. The tests included natural moisture content, Atterberg limits, and grain size distribution. The tests were conducted in general accordance with appropriate American Society of Testing and Materials (ASTM) standards. The test results and a discussion of the laboratory test methodologies are presented in Appendix B, and/or are displayed on the exploration logs in Appendix A, as appropriate.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

Des Moines Memorial Drive South runs generally south-southwest to north-northeast and consists of one lane in each direction, with gently rolling grade changes. In many areas the roadway is adjacent to wetlands, and long sections of the road are supported by embankment fill along the east side. The embankment fill height varies from 0 to 10 feet high above the adjacent terrain, with the highest portion adjacent to wetlands in the vicinity of boreholes BH-1 and BH-2, where the elevated boardwalk section is proposed.

3.2 GENERAL GEOLOGIC CONDITIONS

General geologic information for the site was obtained from the publication *Geologic Map of the Des Moines Quadrangle, King County, Washington* (Waldron, 1962). The map indicates that the surficial geology of the site consists of Vashon recessional outwash over glacial till, which were deposited by the Puget Lobe of the Cordilleran Ice Sheet during the Vashon Stade of the Fraser Glaciation. Recessional outwash in the vicinity consists of glacial meltwater deposits of stratified sand with variable gravel and silt content. This material was deposited by the retreating glaciers and has not been glacially overconsolidated, hence its loose to medium dense condition. This material is relatively permeable and will provide suitable foundation support when thoroughly compacted.

Glacial till is an unsorted, non-stratified deposit of silt, sand, and gravel with scattered cobbles and boulders, commonly referred to as “hardpan”. This material is relatively impermeable and is typically dense to very dense, as a result of being overconsolidated by the advancing glaciers. Areas of glacial till were observed at the surface in locations along the project alignment.

3.3 SUBSURFACE CONDITIONS

3.3.1 Soils

Brief descriptions of the soil units observed in our explorations are presented below in order of deposition, beginning with the most recently deposited. The geotechnical logs in Appendix A (Figures A-2 through A-11) provide more detail of subsurface conditions observed at specific locations and depths. The soils encountered in the explorations are described as follows:

- **Topsoil** – A 12-inch thick (or less) layer of organic-rich topsoil was encountered at the surface of the existing road embankment, typically below grass sod.
- **Fill** – Up to 2 feet of embankment fill was encountered in test pits TP-3 and TP-5. Borings BH-1 and BH-2 also encountered fill with thicknesses of 2 and 3 feet, respectively. The fill material generally consisted of loose, silty sand with variable gravel content.

- **Lacustrine** – Boreholes BH-1 and BH-2 encountered medium stiff to stiff silt and clay, with varying sand content, to depths of at least 23 feet at the wetland area where the boardwalk is proposed. This material appeared to have been deposited in a former lake, and may be transitional from older recessional outwash deposits.
- **Alluvium** – Alluvium consisting of loose to medium dense, silty sand, with varying gravel content, was encountered in borehole BH-2 below the lacustrine, at a depth of about 23 feet. Alluvium was also encountered in borehole BH-4 at a depth of 1 to 7.5 feet and test pit in TP-6. The alluvium encountered in test pit TP-6 contained an approximately 2.5 feet thick layer of organic silt, with an organic content of about 15%.
- **Weathered Drift** – Many of the test pits encountered a one- to five-foot thick B-horizon beneath the topsoil, which appeared to be weathered recessional outwash or weathered till and was labeled as such.
- **Recessional Outwash –** Stratified deposits of medium dense, clean to silty sand with little to no gravel were encountered in most of the test pits.
- **Glacial Till –** Six of the eight explorations adjacent to the road encountered glacial till at varying depths, and generally extended to the full depths explored. The till consisted of an unsorted and unstratified mixture of dense to very dense, gravelly, silty sand.
- **Advance Outwash** – Boreholes BH-3 and BH-4 encountered this unit beneath the glacial till to the full depths explored. The advance outwash consisted of very dense, clean to slightly silty sand with little or no gravel. This material was deposited by meltwater flowing from the advancing Puget Lobe, and was subsequently over-ridden by the glacier, thus over-consolidating it.
- **Glaciolacustrine** – Hard, bluish-gray clay was encountered in test pits TP-4 and TP-5 beneath the recessional outwash. Glaciolacustrine soils were deposited in ice-marginal standing water and were subsequently glacially overridden, hence the hard consistency.

3.3.2 Ground Water

At the times of exploration, ground water was observed at depths ranging from 4 to 15 feet in the eight explorations adjacent to the road, and at the ground surface in the two borings adjacent to the wetland. Variation in ground water conditions should be expected to occur seasonally and with changes in precipitation.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

The investigation showed that most of the existing embankment along Des Moines Memorial Drive consists of native glacial soils with minor amounts of fill in some areas. During

construction of the roadway, a ten-foot thick embankment fill was placed between Project STA 115+00 and 119+00. The toe of the embankment is adjacent to wetlands and an elevated boardwalk is proposed in this section.

Retention of new fill placed along the road embankment could be attained with MSE walls everywhere except where the elevated boardwalk section is planned. In our opinion, the boardwalk could be supported by pipe piles; however, end bearing depths are unknown due to the limits of the hand-portable drilling equipment used to access this area.

We understand that rockeries are planned in the areas of three proposed cuts.

Stormwater infiltration into the existing soils appears feasible, via porous pavement, for most of the alignment. Infiltration trenches could be used in the vicinity of test pits TP-1 and TP-3, because of the higher infiltration rates in those areas. We do not recommend infiltration in the area of the boardwalk, or any retaining walls, as this could influence the global stability of the slope and reduce the bearing capacity of spread footings.

The following sections present geotechnical recommendations for design of boardwalk pile and spread footing foundations, MSE walls, rockeries, stormwater infiltration, conventional and porous pavement structures, and general earthwork.

4.2 SEISMIC DESIGN CONSIDERATIONS

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2009 edition of the International Building Code (IBC, 2009). Based on the conditions encountered in the explorations, we recommend that Site Class D be used. The corresponding normalized design response spectra are considered adequate for this site. The design peak ground acceleration (PGA) is 0.38 g. This value should be provided for design of the CMU/MSE walls. For global stability analyses, we utilized a horizontal coefficient (k_h) equal to PGA/2 or 0.19 g.

Where the trail is constructed over native glacial soils, the road embankment, and thus the trail, is unlikely to experience significant permanent displacements resulting from ground surface rupture, landsliding, or liquefaction. However, we expect significant depths of liquefaction along the section with a 10-foot fill embankment adjacent to the wetlands, at the location of the proposed boardwalk. In this area, the majority of the lacustrine deposits are potentially liquefiable under the design seismic event.

4.3 BOARDWALK FOUNDATIONS

Between STA 115+00 and 119+00 the existing embankment has a steep narrow shoulder which drops approximately 10 feet in elevation into the adjacent wetland. Rather than place fill in this area, the intent of the project is to construct an elevated boardwalk parallel to the existing

roadway embankment, thus minimizing impacts to the wetland. The 60% Plans indicate the boardwalk will be supported on 8-inch diameter piles along the wetland side and shallow spread footings on the roadway side.

Our recommendations for the boardwalk foundations are based on information from the following sheets from the 60% design drawings (Parametrix, December 2011): Sheet C5, Plan and Profile; Sheet CS1, Typical Cross Sections; Sheet S1, Structural Notes; and Sheet S2, Boardwalk Details. The loads were obtained in an e-mail from Joe Merth (Parametrix) to Tom Kinney (HWA) on December 6, 2012.

Based on the information on Sheets S2 and S1, the concept is to support the road side of the trail on spread footings in the roadway fill and support the wetland side of the boardwalk on steel piles driven through the road fill and soft soils and into the underlying bearing layer.

For the purposes of design, we were provided with the following loads, which we understand are the loads applied at each bent. The bent spacing is approximately 20 feet center-to center.

Vertical Loads:

- Total DL (superstructure, overlay, and pier cap) = 29.3 k/pier
- Total LL (pedestrian load) = 21.6 k/pier
- Factored Service Load (1.0DL + 1.0 LL) = 50.9 k/pier
- Factored Strength Load (1.25DL + 1.75LL) = 74.4 k/pier
- Vertical earthquake loading is considered zero

Note: We have assumed the vertical loads are to be spread equally on each side of the trail.

Horizontal Loads:

- Factored Extreme Load (0.20 {DL+LL}) = 10.2 k/pier

Note: In our opinion, the horizontal load will not necessarily be spread equally between the spread footings and the piles as the lateral stiffness of these elements are different.

Figure 3 shows the critical section based on the contours provided on Sheet C5, which is located at approximate STA 116+00. We recommend using this section for the entire boardwalk except near the culvert. Between STA 115+00 and 119+00 the western edge of the wetland appears to be at roughly Elevation 222 to 224 feet. Elevation 222 feet is the lowest elevation shown on the site survey (see Fig 2D-2E, Site and Exploration Plans). Note that we have no information about the surface contours in the wetlands. For purposes of design we have assumed a continuation of the embankment fill for several feet below the lowest elevation shown, after which the slope flattens out somewhat, as shown in Figure 3.

4.3.1 Subsurface Information

Two borings were drilled along the boardwalk alignment including boring BH-1 at STA 115+75 and boring BH-2 at STA 118+75. Both borings were located near the toe of the embankment

slope. The borings encountered 2 to 3 feet roadway fill consisting of loose, silty sand over lacustrine deposits of slightly sandy to sandy silt and clay with Standard Penetration Test blow counts (N-values) generally between 5 and 12. Note that both borings were terminated due to limitations of the drilling equipment before encountering glacially consolidated materials. Boring BH-1 was terminated at a depth of about 22 feet, likely at the contact with alluvial soils, due to refusal on a cobble. Boring BH-2 was terminated at a depth of 31.5 feet in alluvial soils. However, based on the geologic mapping and our experience in this area, it is our opinion that glacially overridden soils underlie the soft sediments at some depth below the boardwalk location.

No explorations were performed within the embankment materials at the crest of the slope in this area. However surface observations and probing of the embankment slope and test pits at other locations along the road shoulders indicate that the embankment fill generally consists of loose to medium dense, silty sand to sandy silt.

4.3.2 Liquefaction

The sandy silt and silty sand with SPT N-values less than about 12 are loose enough to liquefy during the design seismic event. Where these underlie the roadway embankment, the embankment slopes are likely to slide or rotate such that they no longer provide any resistance to loading exerted on the soil by the boardwalk structure. However, we understand that it considered too costly to mitigate against the effects of soil liquefaction.

4.3.3 Vertical Pile Design

As stated above, it is our opinion that glacially consolidated soils suitable for support of the proposed piles is present at some depth below the lacustrine and alluvial deposits. To define the depth to the bearing layer additional explorations would be required. We understand that the decision has been made that the design team will not perform any additional explorations prior to construction. As part of the construction phase, the Contractor will be tasked with installing test piles that will be used to determine the pile lengths.

Acceptance of the piles should be determined using a driving criterion developed for the specific hammer system provided by the Contractor prior to performing the test pile installation. If a wave equation analysis is used to develop the driving criterion, the ultimate load for acceptance of the piles should be taken as the factored maximum design load (37.2 kips) times a resistance factor (ϕ) of 0.4. This resistance factor is based on the 2005 NHI ("Development of Geotechnical Resistance Factors and Downdrag Load Factors for LRFD for Strength Limit Design", Table 8). In our opinion it is possible to develop the ultimate load of 93 kips (37.2 kips/0.4) with steel piles driven into glacially overridden soils at the site.

The contractor should be required to drive two test piles at least 200 feet apart, with the proposed pile section and the pile-driving equipment to be used for production piles, to at least the penetration resistance calculated with wave equation analyses. The test piles could be used as production piles as long as they meet the requirements. The final pile tip elevation should be accepted on-site by a geotechnical engineer who is monitoring the installation of the piles based on the developed driving criteria.

We recommend using closed-ended pipe piles for several reasons. One is that these piles typically require less penetration than open-ended sections. It also allows visual inspection of the inside of the piles after driving to check for damage to the pile that may have occurred during driving. A steel H section with a plate welded on the bottom should not be used as it will reduce the lateral capacity of the piles in the upper materials.

4.3.4 Culvert

The section near the culvert has a higher embankment and steeper side slopes than the other sections. The bents for the boardwalk in this area should be supported solely on piles rather than with a pile on one side and a spread footing on the other. Based on the information we have, it appears that about 40 feet of the trail, between STA 116+05 and 116+45, is not appropriate for a spread footing. The pile design information provided is applicable for both piles in this section.

4.3.5 Horizontal Pile Design

We understand horizontal pile design is required for the Extreme I Limit State. For this report, we performed analyses assuming the entire lateral load from the abutment is applied to the head of the pile, which is 10.2 kips per pile. In our opinion, this horizontal load can be resisted by the design pile section within the lacustrine deposits. The piles will be installed near the toe of the slope. We recommend neglecting the soils above Elevation 222 feet. Below Elevation 222 feet we recommend using the following properties for the LPILE analyses based on rapid (earthquake) loading:

LPILE Parameters:

- Soil Type: Soft Clay (sandy silt under rapid loading)
- Effective density: 60 pcf
- Undrained cohesion: 1000 psf
- Strain factor (ε_{50}): 0.01

Note: The possible effects of liquefaction have been ignored as discussed earlier.

Based on a fixed head condition and a standard 8-inch pipe pile, the pile deformation is about 2.4 inches under the design load of 10.2 kips. For this analysis the lateral load was applied at Elevation 230 feet resulting in an unsupported pile length of about 8 feet.

4.3.6 Spread Footing Design

The spread footing will be constructed within the roadbed materials. The bridge structure will be structurally supported so the footing will have 18 inches of soil weight cover and the outside edge of the footing will be at least 4 feet back from the crest of the slope, as shown on Figure 3.

Bearing Capacity

For the Strength I Limit State, an ultimate bearing capacity of 6,600 psf may be used with a resistance factor of 0.45. For the Service I Limit State and the Extreme I Limit State, an ultimate bearing capacity of 2,830 psf may be used, with a resistance factor of 1.0. Note that global stability of the embankment controls the bearing capacity values for the Service I and Extreme I Limit States.

Settlement

For use with the Service I Limit State, the expected settlement is expected to be linear up to the ultimate bearing capacity or 0.1 inches for every 1000 psf applied to the foundation.

Sliding Resistance

We recommend using an ultimate friction value equal to 0.55 times the normal load applied. This should be used with a resistance factor of 0.80 for the Strength I Limit State and 1.0 for the Extreme I Limit State.

Slope Stability

The factor of safety of the slope is 1.5 for the Static Case, as shown on Figure 4. The factor of safety drops to about 1.1 under a horizontal earthquake loading of 0.19 g, as shown on Figure 5. Therefore, slope stability is adequate provided liquefaction is not considered.

4.3.7 Combined Foundations Under Horizontal Loading

Sliding resistance to horizontal loading of the spread footings will be developed under deflections significantly less than the horizontal deflection of the pipes. We would therefore expect the spread footing to carry almost all of the horizontal seismic loading before the piles would deflect enough to begin carrying a significant portion of the load. However, if the frictional resistance of the spread footing is exceeded then we would expect almost all of the horizontal force to be applied to the piles.

4.4 RETAINING WALLS

4.4.1 MSE Retaining Walls

There are nine sections where relatively short fill walls are required and the preferred retention system is a CMU (Concrete Masonry Unit) MSE (Mechanically Stabilized Earth) wall. For this

project we recommend geogrid reinforcement and commercially available blocks on the order of 6 to 8 inches high with positive connection between the CMU's and the geogrid. We understand that the MSE walls are required to be designed by the wall system manufacturer. Design parameters for use in MSE wall design are provided in Table 1.

Table 1. Recommended Design Parameters for MSE Walls

Soil Properties	Wall Backfill	Retained Soil	Foundation Soil
Unit Weight (pcf)	135	135	130
Friction Angle (deg)	36	36	35
Cohesion (psf)	0	0	0
		AASHTO Load Group I (EP+LL)	**AASHTO Load Group VII (EP+EQ)**
Allowable Bearing Resistance (ksf)		2.8	3.7
Acceleration Coefficient (g)		N/A	0.38

Existing embankment soils will need to be benched horizontally down to firm and unyielding soils. Geogrid will need to be placed horizontally, and structural fill placed and compacted in horizontal lifts.

We recommend drainage measures be included behind all walls constructed for this project. Wall drains typically consist of 4-inch to 6-inch diameter, perforated drain pipes embedded in clean, drainage aggregate at the base of, and behind, the wall. Drainage pipes should be directed to appropriate outlets.

4.4.2 Rockeries

We understand that there are three proposed rockery walls, one of which is up to 8 feet high, and will support cuts into a glacial till embankment. We recommend that the guidelines in the ARC Design Guide (Attachment 1 to this report) be used for rockery design and construction. The foundation materials are expected to be glacial till and adequate with a minimum of preparation. We recommend the soil be cut back to a slope of 3/4 horizontal to 1 vertical and the space between the slope and the rockery rocks be backfilled with crushed drain rock.

4.5 STORMWATER INFILTRATION CAPACITY

Table 2 provides estimates of long-term infiltration capacity of selected subgrade soils, based on the grain size analyses results presented in Appendix B, using the most recent Department of Ecology Guidelines (DOE, 2005). The results are in order from south to north along the alignment. Stormwater infiltration is feasible using porous pavement, assuming that the existing

road runoff is captured by proposed curbs and a new storm sewer drainage system. Infiltration trenches could be used in selected areas where infiltration rates appear to be higher, in the vicinity of test pits TP-1 and TP-3. At the other explorations, either the perched ground water level was too close to the ground surface, and/or the grain size distribution of the near-surface soils were too fine to infiltrate stormwater in an infiltration trench.

Table 2: Estimated Long –Term Infiltration Capacity

Exploration (Sample Depth)	Stationing	Soil Classification (ASTM)	Fines Content (%)	D_{10} size (mm)	Long-Term Infiltration Capacity (inches/hour)
TP-1 (3 to 3.5 feet)	104+75	Poorly graded SAND with silt	7	0.1	2
TP-1 (8 to 8.5 feet)	104+75	Silty SAND	33	<0.003	0.5
TP-2 (2 to 2.5 feet)	110+45	Silty GRAVEL with Sand	20	0.018	0.6
BH-3 (1.5 to 1.8 feet)	126+90	Silty SAND	29	0.006	0.5
BH-3 (2.5 to 4 feet)	126+90	Silty SAND with Gravel	25	0.006	0.5
TP-3 (0.6 to 1.3 feet)	135+10	Well graded SAND with silt and gravel	10	0.08	1
TP-3 (2 to 2.5 feet)	135+10	Poorly graded SAND with silt	5	0.15	2
TP-4 (1.5 to 2 feet)	145+65	Silty SAND	24	0.018	0.6
TP-4 (2.8 to 3.3 feet)	145+65	Poorly graded SAND with silt	10	0.077	1

(cont) Exploration (Sample Depth)	Stationing	Soil Classification (ASTM)	Fines Content (%)	D10 size (mm)	Long-Term Infiltration Capacity (inches/hour)
TP-5 (2.5 to 3 feet)	152+45	Silty SAND	34	<0.006	0.5
TP-5 (4 to 4.5 feet)	152+45	Silty SAND	49	<0.006	0.5
TP-6 (2.5 to 3 feet)	157+40	Silty SAND with Gravel	18	<0.004	0.5
BH-4 (2.5 to 4.0 feet)	165+30	Silty SAND with Gravel	17	0.02	0.6
BH-4 (7.5 to 9.0 feet)	165+30	Silty SAND with Gravel	27	0.005	0.5

4.6 HOT-MIX ASPHALT (HMA) PAVEMENT

4.6.1 Conventional HMA Pavement

We recommend the following minimum pavement structure for the trail:

Table 3. Minimum Pavement Structure Requirements

Material Description	Minimum Layer Thickness (inches)	WSDOT Standard Specification
HMA	2.5	5-04 & 9-02.1
CSTC	4	9-03.9(3)
Prepared Subgrade	Strip and proof roll native subgrade	

HMA: Hot Mix Asphalt – Class ½-inch
CSTC: Crushed Surfacing Top Course compacted as specified.

The structural layer thickness given in Table 3 is for a non-motorized trail used by pedestrians, cyclists, etc. The pavement structure could accommodate occasional passes of a lightly loaded maintenance vehicle, such as a pickup truck.

Subgrade preparation for pavement construction should consist of stripping the surficial organics and removing any other deleterious materials. In areas of competent subgrade consisting of native glacial soils, the exposed subgrade should be thoroughly compacted and then proof-rolled with a fully loaded dump truck prior to placing the base course. Any soft and yielding materials identified during the proof-rolling process should be removed and replaced with structural fill.

Crushed surfacing top course (CSTC) and structural fill should be compacted to at least 95% of the maximum dry density, as determined using test method ASTM D 1557 (Modified Proctor).

4.6.2 General Requirements for HMA Placement

Placement of HMA should be in accordance with Section 5-04 of the WSDOT *Standard Specifications* (WSDOT, 2012). Particular attention should be paid to the following:

- HMA should not be placed until the engineer has accepted the previously constructed pavement layers.
- HMA should not be placed on any frozen or wet surface.
- HMA should not be placed when precipitation is anticipated before the pavement can be compacted, or before any other weather conditions which could prevent proper handling and compaction of HMA.
- HMA should not be placed when the average surface temperatures are less than 45° F.
- HMA temperature behind the paver should be in excess of 240° F. Compaction should be completed before the mix temperature reduces below 180° F. Comprehensive temperature records should be kept during the HMA placement.
- For cold joints, tack coat should be applied to the edge to be joined and the paver screed should be set to overlap the first mat by 1 to 2 inches.

4.6.3 Porous HMA Pavement

The long-term infiltration results given in Table 2 indicate that porous pavement is feasible for the entire alignment. We recommend that porous pavement be designed for long-term infiltration rates between 0.5 and 2 inches per hour as given in Table 2. We further recommend that HWA inspect the subgrade after stripping to confirm that suitable permeable material is present. It should be noted that actual infiltration rates will vary significantly along the

alignment, depending on the soil type (particularly the percentage of fines), presence of buried utilities, etc.

Table 4 provides minimum pavement structure requirements for a porous pavement:

Table 4: Minimum Pavement Structure Requirements

Material Description	Minimum Layer Thickness (inches)	Specification Requirement
Porous HMA	2.5	-
Bedding Course	1.5	3/8-inch to #4 crushed rock with 90% to 100% fractured faces (ASTM C33 gradation)
Permeable Crushed Base Course	8	1-inch minus permeable crushed rock with 90% to 100% crushed faces (WSDOT 9-03.1(24) gradation)
Non-woven geotextile	--	Construction Geotextile for Separation (WSDOT 9-33.2(1) Table 3)
Gravel Borrow, or Native Subgrade	Inspect to confirm suitability of native subgrade	Minimal compaction to preserve infiltration rates of the subgrade soils

The following notes apply to the porous HMA pavement:

- The structural layer thickness given in Table 4 is for a non-motorized trail used by pedestrians, cyclists, etc. Because porous HMA is typically compacted to a relatively low density to facilitate infiltration, the pavement is only able to accommodate occasional passes of lightly loaded maintenance vehicles.
- The thickness of open-graded crushed rock base should be at least 8 inches for structural design purposes, but will likely need to be greater to provide sufficient storage capacity for hydrologic design considerations.
- Permeable crushed base should be spread in maximum 6-inch loose lifts. For each lift, make at least two passes of the compactor in the vibratory mode, then at least two in the static mode until there is no visible movement of the crushed rock.
- The pavement should be provided with an outlet to discharge excess flow that may occur during very wet periods. Build-up or accumulation of water within the base course supporting the HMA should be prevented to prevent its deterioration.

- Infiltration capacity will reduce with time as the asphalt surfacing becomes dirty, and clogging of the native soils from fines carried down into the pavement. In particular, runoff from the adjacent slopes should be prevented from flowing across the porous HMA surfacing. An allowance must be made for future pavement maintenance. Pressure washing and vacuuming could be performed once or twice a year to remove fines from the pavement surface.
- Regular maintenance is also required because porous HMA is susceptible to frost damage if water becomes trapped in the base directly below the surfacing.
- Porous HMA is expected to have the same design life as regular HMA, of around 15 years.
- Consideration should be given to providing a ditch between the edge of the porous HMA trail paving and the toe of the slope to prevent runoff from the slope being infiltrated through the pavement.
- Porous pavement is most susceptible during construction, and therefore it is important that the construction be undertaken in such a way as to prevent:
    - Over-compaction of the underlying soil;
    - Contamination of the Permeable Crushed Base Course with fines;
    - Tracking of soil onto pavement surface;
    - Drainage of sediment laden waters onto porous surface.
- Staging, construction practices, and erosion and sediment control must all be taken into consideration when using porous pavements.

4.7 GENERAL EARTHWORK

4.7.1 Temporary Excavations

Any temporary excavations deeper than 4 feet should be sloped or shored in accordance with current State of Washington Labor and Industries Safety and Health guidelines. Per these guidelines, any fill, alluvial soils and loose glacial soils are classified as Type C soils. Unsupported excavations within Type C soils should be sloped no steeper than 1.5H:1V. Dense to very dense glacial till is classified as Type A soil. Unsupported excavations within Type A soils should be sloped no steeper than 0.75H:1V. The recommended maximum slope is applicable to temporary excavations above the water table only; flatter side slopes would be required for excavations below the water table and/or where aggressive seepage is occurring.

The Contractor should monitor the stability of all temporary excavations and adjust the construction schedule and slope inclination accordingly. The Contractor should be responsible

for control of ground and surface water and should employ sloping, slope protection, ditching, sumps, dewatering, and other measures, as necessary, to prevent sloughing/erosion of soils.

4.7.2 Structural Fill and Compaction

For the purposes of this report, material used to raise grades is classified as structural fill. In general, structural fill should consist of clean, relatively free-draining, granular soils that are free from organic matter or other deleterious materials. Such materials should comprise particles of less than 4 inches, or 1/2 the lift height in maximum dimension, whichever is less, with less than 7% fines (portion passing the U. S. Standard No. 200 sieve), as specified for "Gravel Borrow" in Section 9-03.14(1) of the WSDOT *Standard Specifications* (WSDOT, 2012). The fine-grained portion of structural fill soils should be non-plastic. Advance outwash excavated from the site is likely to be suitable for structural fill, but should be evaluated by the geotechnical engineer at the time of construction.

Structural fill soils should be moisture conditioned, placed in loose horizontal lifts less than 8-inches thick, and compacted to at least 95% of the maximum dry density as determined using test method ASTM D1557 (Modified Proctor). Achievement of proper density of a compacted fill depends on the size and type of compaction equipment, the number of passes, thickness of the layer being compacted, and soil moisture-density properties. In areas where limited space restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough layers to achieve the required relative compaction. Generally, loosely compacted soils result from poor construction technique and/or improper moisture content. Soils with high fines contents are particularly susceptible to becoming too wet and coarse-grained materials easily become too dry, for proper compaction.

Structural fill used in geogrid-reinforced zones of MSE walls should not be angular and meet the requirements of Gravel Borrow for Geosynthetic Retaining Wall, Section 9-03.14(4) of the 2012 WSDOT *Standard Specifications*. In addition compaction of all fill within 2 feet of any retaining structure should be limited to hand operated equipment, requiring thinner lifts to achieve adequate compaction.

4.7.3 Construction of Sliver Fills

Site preparation for construction of sliver (thin) fills should consist of removing the surface vegetation from the road shoulder and excavating benches so the new fill can be placed in horizontal layers and compacted in accordance with the requirements for structural fill.

Sliver fills should be constructed in a manner so that they do not obstruct existing drainage conditions within the slopes.

4.7.4 Pipe Bedding and Utility Trench Backfill

General recommendations relative to pipe bedding and utility trench backfill are presented below:

- Pipe bedding material, placement, compaction, and shaping should be in accordance with the project specifications and the pipe manufacturer's recommendations. As a minimum, the pipe bedding should meet the gradation requirements for Gravel Backfill for Pipe Zone Bedding, Section 9-03.12(3) of the 2012 WSDOT *Standard Specifications*.

- Pipe bedding materials should be placed on relatively undisturbed native soils, or compacted fill soils. If the native subgrade soils are disturbed, the disturbed material should be removed and replaced with compacted bedding material.

- If areas of deep alluvium are encountered, it will be necessary to over-excavate the unsuitable material and backfill with pipe bedding material. In wet conditions, 1¼-inch-minus granular fill meeting the gradation requirements for Crushed Surfacing, as described in Section 9-03.9 of the 2012 *Standard Specifications*, may be used to backfill the over-excavated portion of the trench.

- Pipe bedding should provide a firm, uniform, cradle for the pipe. We recommend that a minimum 4-inch thickness of bedding material beneath the pipe be provided.

- Pipe bedding material and/or backfill around the pipe should be placed in layers and tamped to obtain complete contact with the pipe.

- We recommend that trench backfill meet the specifications for structural fill, as described in this report. During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Trench backfill should be placed in 8-inch (maximum) lifts and compacted using mechanical equipment to at least 95% of its maximum dry density, as determined by testing in general accordance with ASTM D 1557.

4.7.5 Wet Weather Earthwork

Existing site soils are moisture sensitive to varying degrees, and may be difficult to handle or traverse with construction equipment during periods of wet weather. Therefore, general recommendations relative to earthwork performed in wet weather or in wet conditions are presented below. These recommendations should be incorporated into the contract specifications and should be required when earthwork is performed in wet conditions:

1) Site stripping and fill placement should be accomplished in small sections to minimize exposure to wet weather. Excavation or removal of unsuitable soil should

be followed promptly by placement and compaction of a suitable thickness of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

2) Material used as structural fill should consist of clean granular soil, of which not more than 5% passes the U.S. Standard No. 200 sieve, based on wet sieving the fraction passing the ¾-inch sieve. The fine-grained portion of structural fill soils should be non-plastic.

3) No soil should be left uncompacted so it can absorb water. Stockpiles should be protected from moisture using anchored plastic sheeting, or other suitable methods.

4) Excavation and placement of fill should be monitored by someone experienced in wet weather earthwork to determine that the work is being accomplished in accordance with the project specifications and the recommendations contained herein.

5.0 CONDITIONS AND LIMITATIONS

We have prepared this report for use by Parametrix and King County in design of a portion of this project. This report should be provided in its entirety to prospective contractors for bidding or estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that subsurface 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 that HWA be retained to review the plans and specifications and to monitor the geotechnical aspects of construction, particularly temporary excavations, subgrade preparation, pipe bedding and backfill, pile installation, wall construction, and rockery construction.

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.

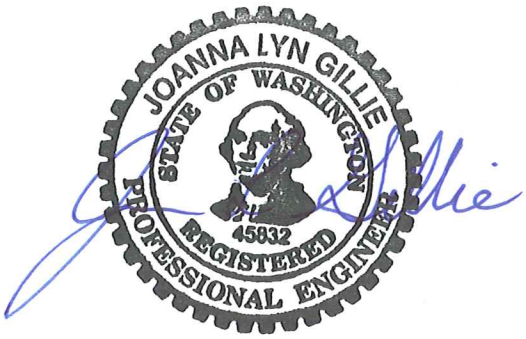
HWA does not practice or consult in the field of safety engineering. We will not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any of the recommended actions presented herein unsafe.

December 27, 2012
HWA Project No. 2010-100

We appreciate this opportunity to be of service.

Sincerely,

HWA GEOSCIENCES, INC.



Bryan K. Hawkins, P.E.
Senior Geotechnical Engineer

JoLyn Gillie, P.E.
Geotechnical Engineer

6.0 REFERENCES

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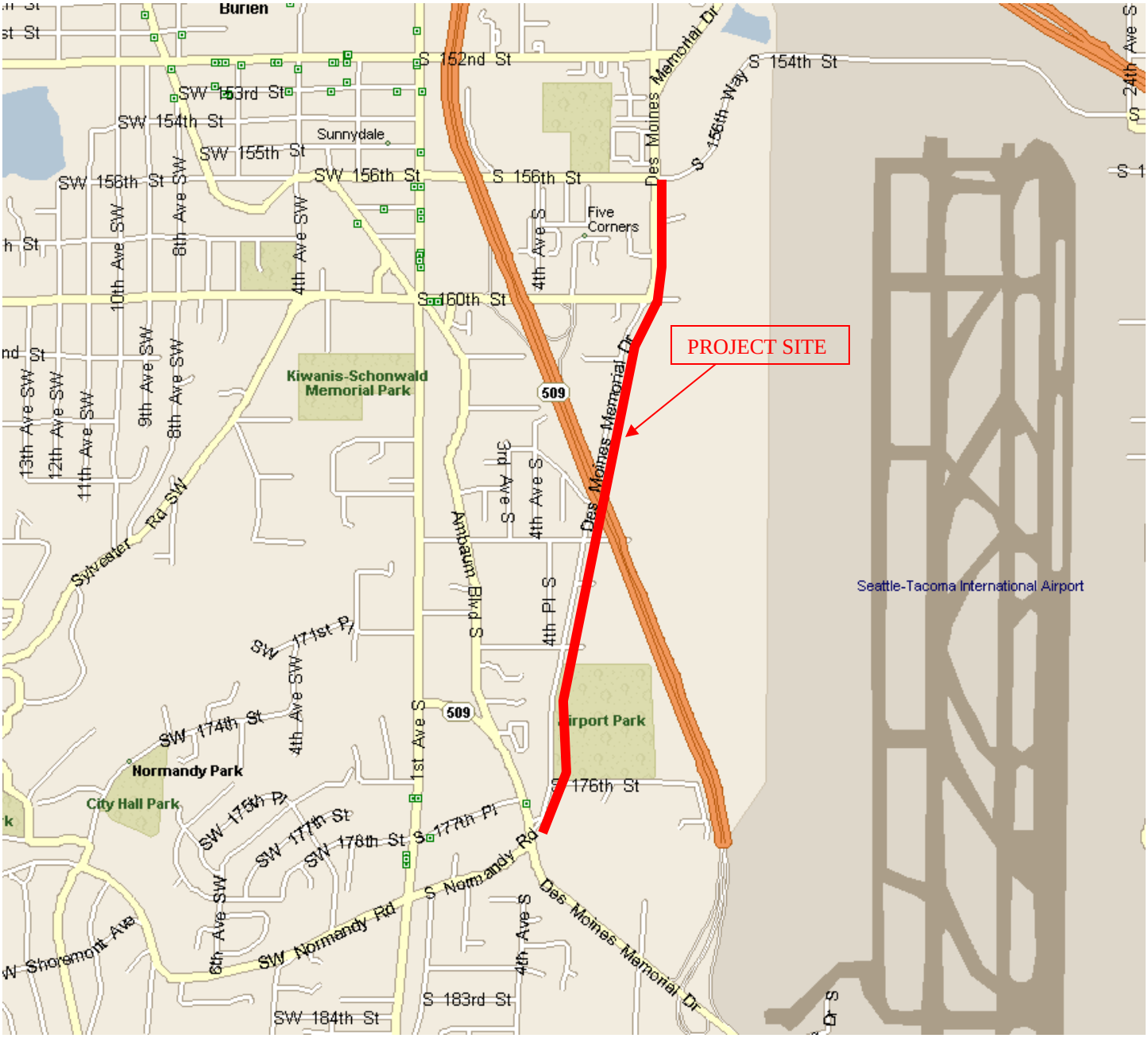


NORTH

NOT TO SCALE

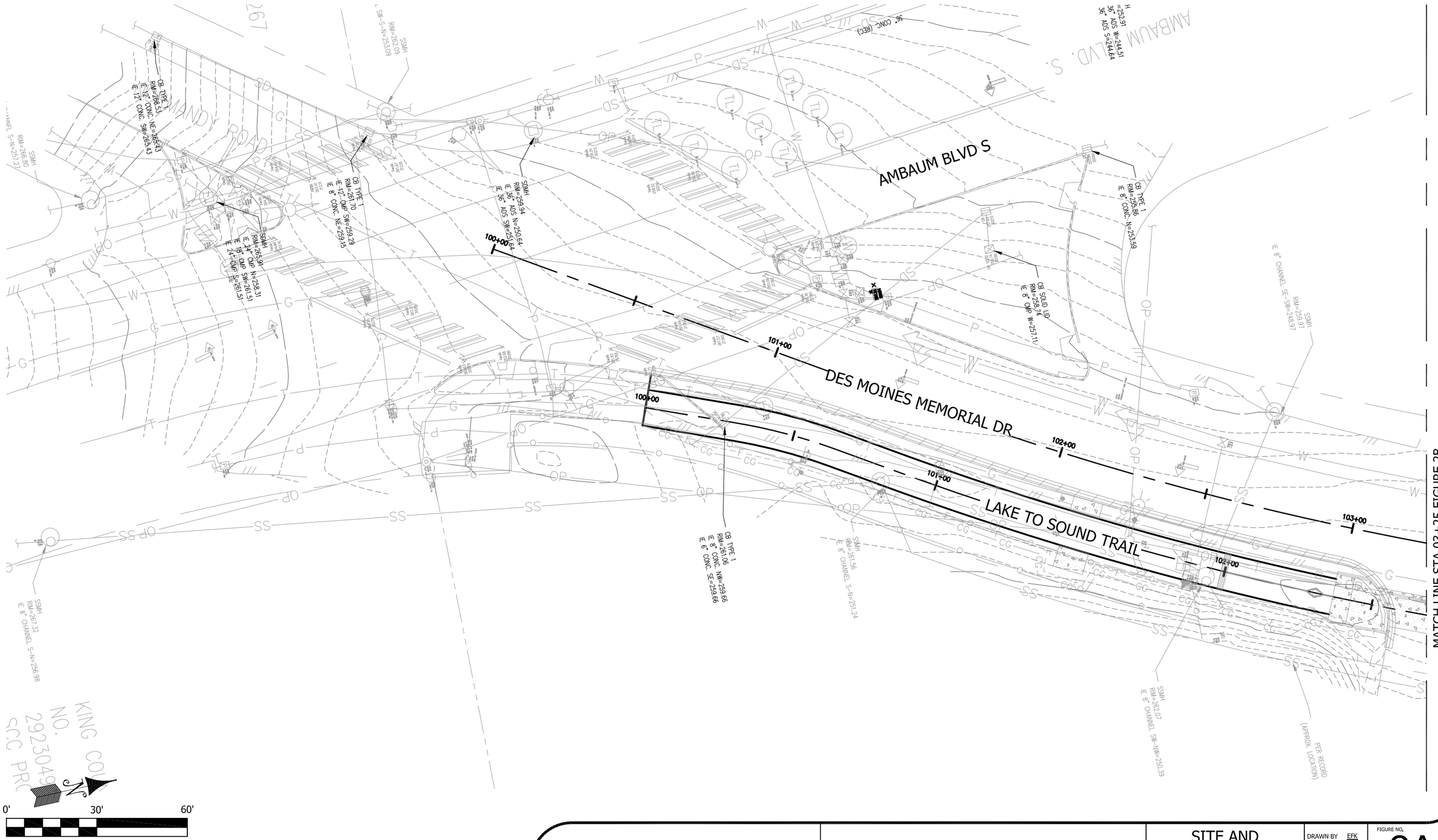
HWA GEOSCIENCES INC.

VICINITY MAP

LAKE TO SOUND TRAIL – WESTSIDE SECTION
KING COUNTY, WASHINGTON

FIGURE NO. 1

PROJECT NO. 2010-100



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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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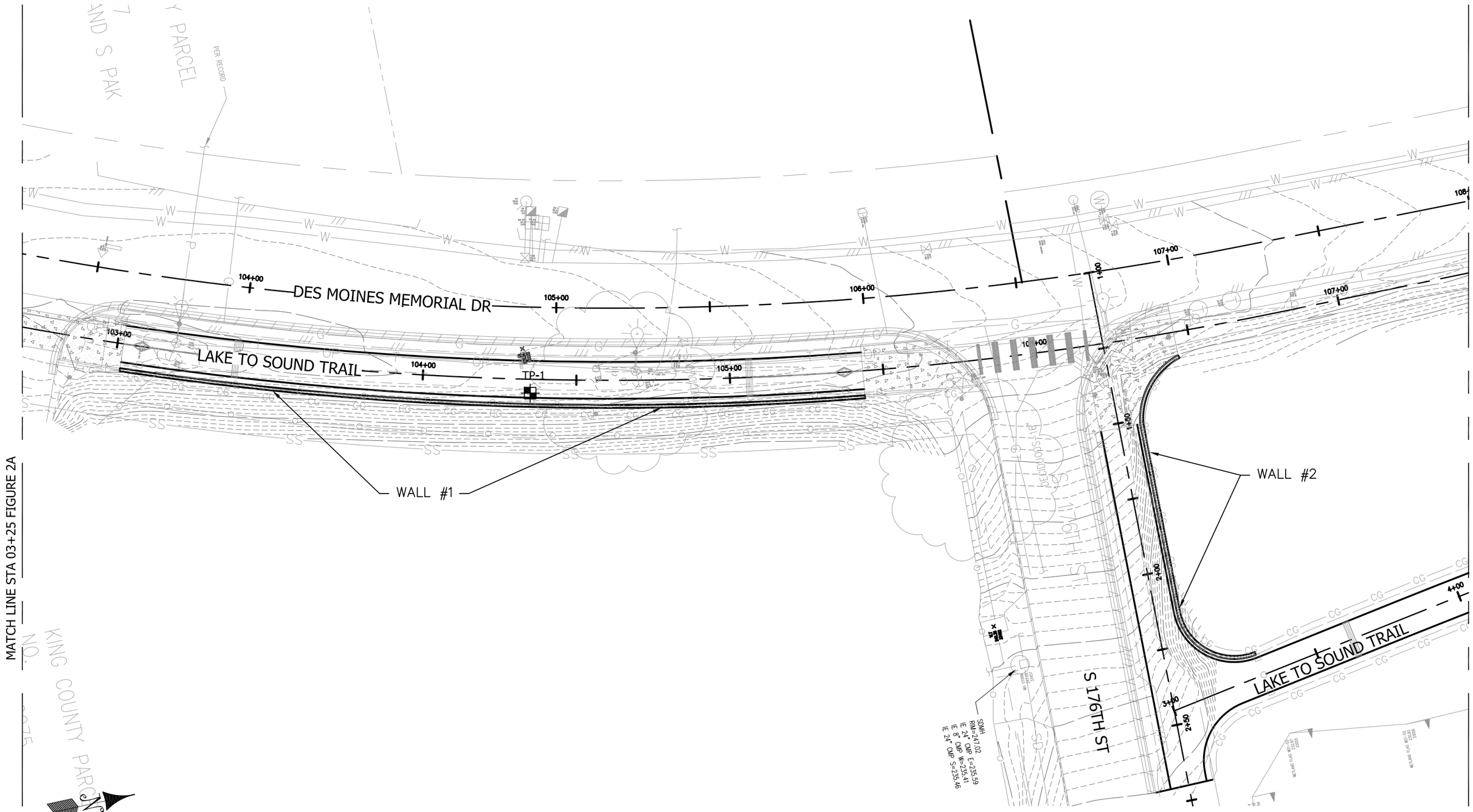
DATE	12.06.12

FIGURE NO. 2A

PROJECT NO. 2010-100-21 Task 1100



TP-1

TEST PIT DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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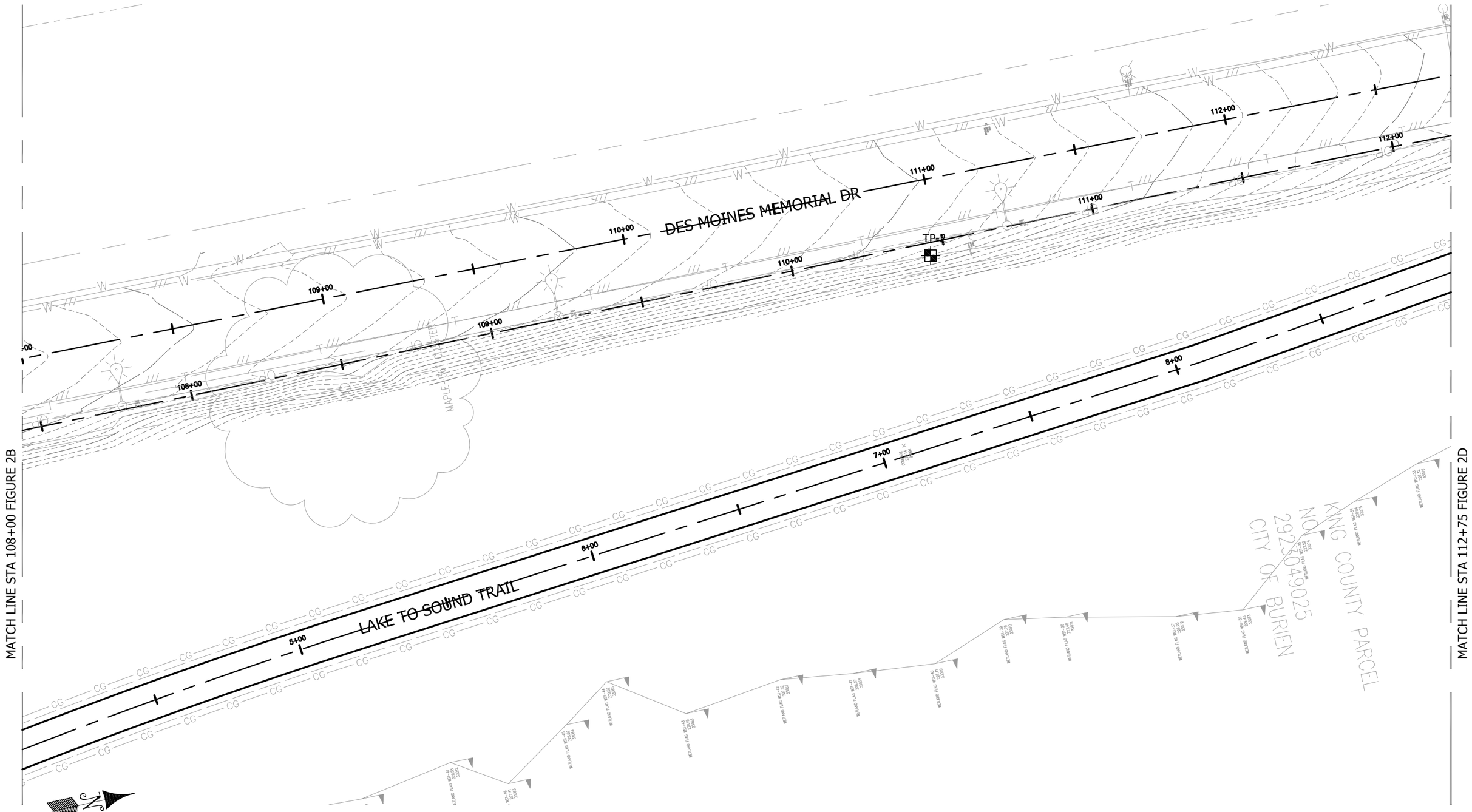
DATE 12.06.12	



TP-2

TEST PIT DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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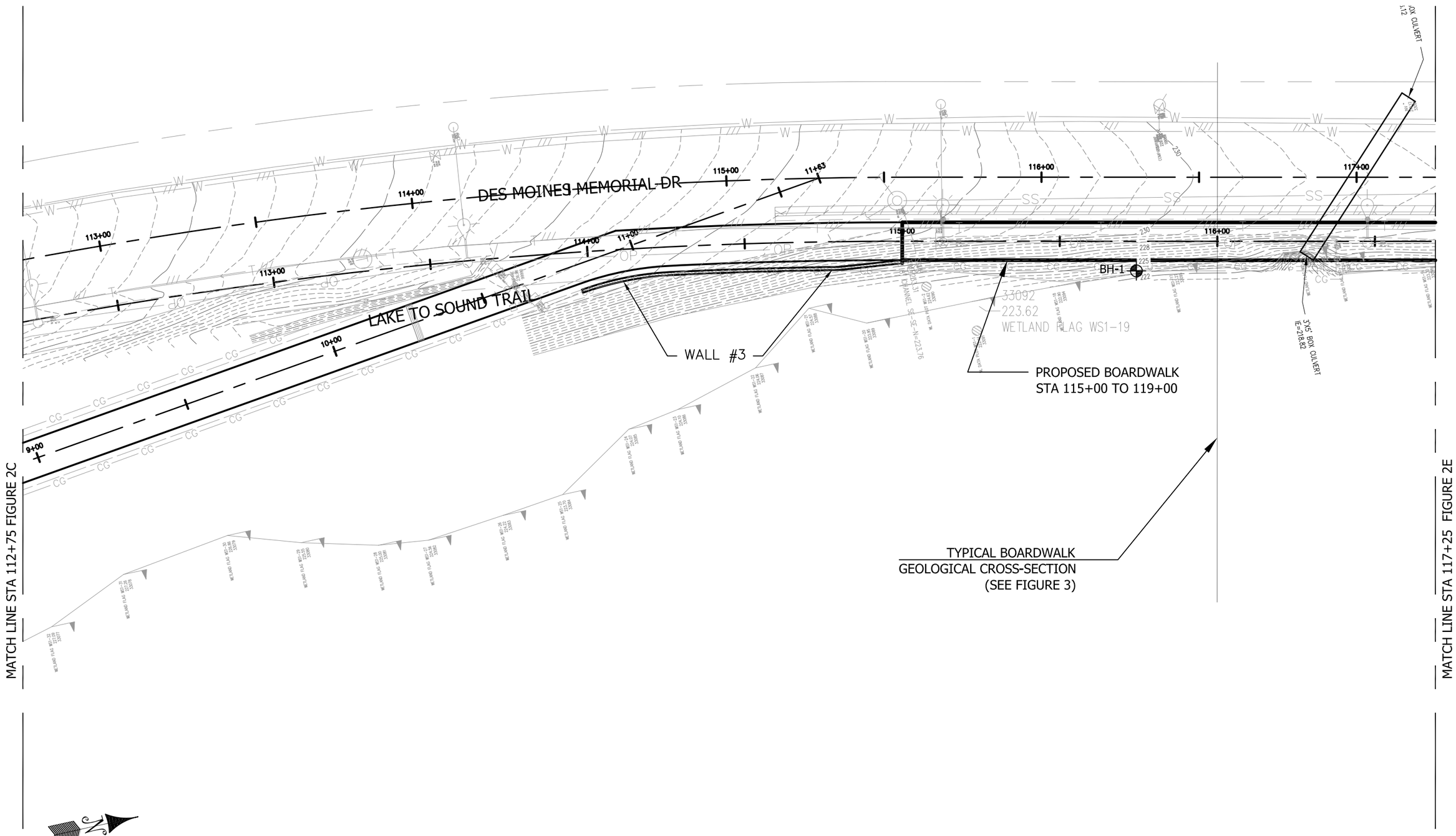
FIGURE NO.	2C
PROJECT NO.	2010-100-21 Task 1100



0' 30' 60'

BH-1

BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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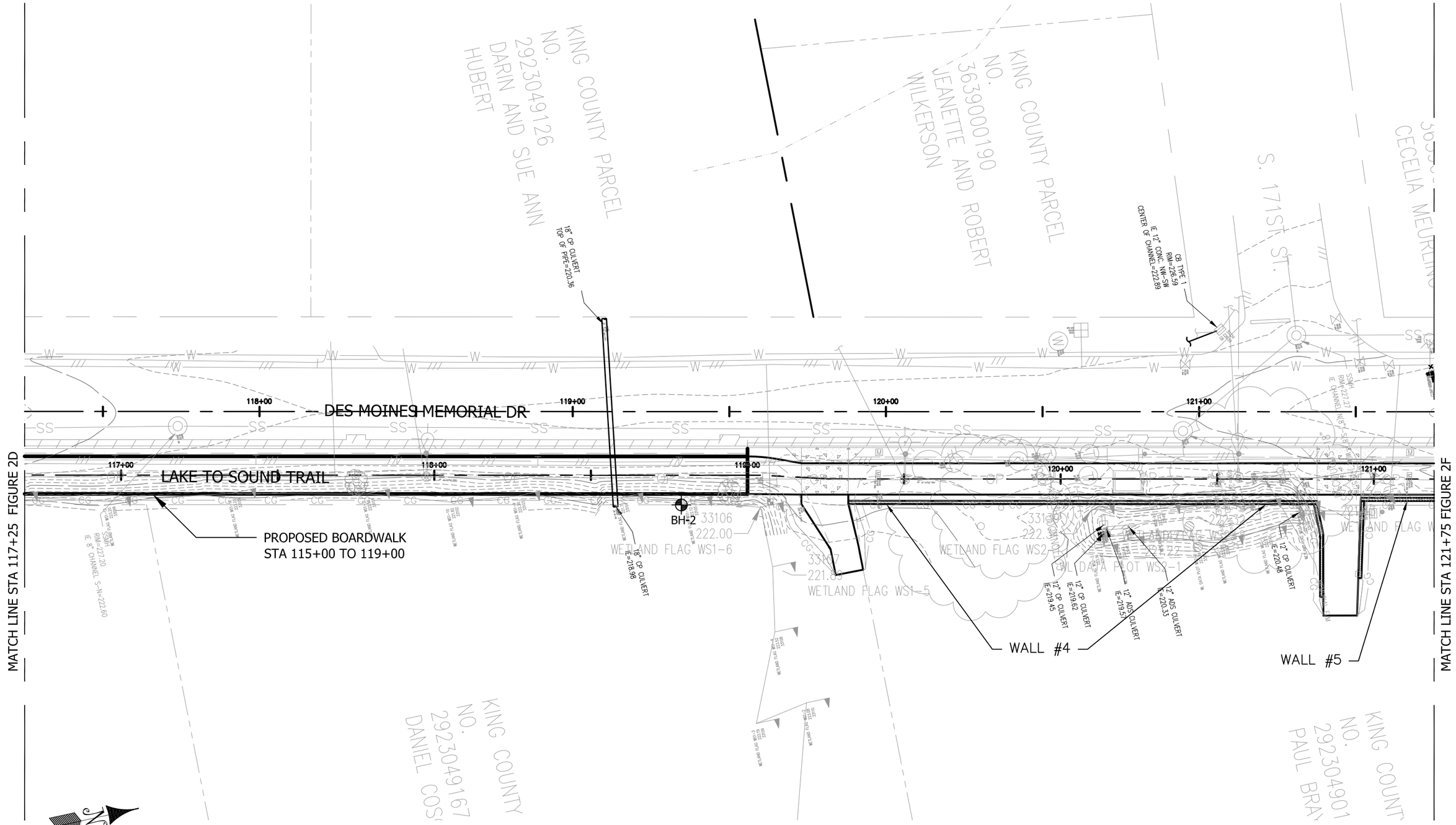
FIGURE NO. 2D

PROJECT NO. 2010-100-21 Task 1100



BH-2

BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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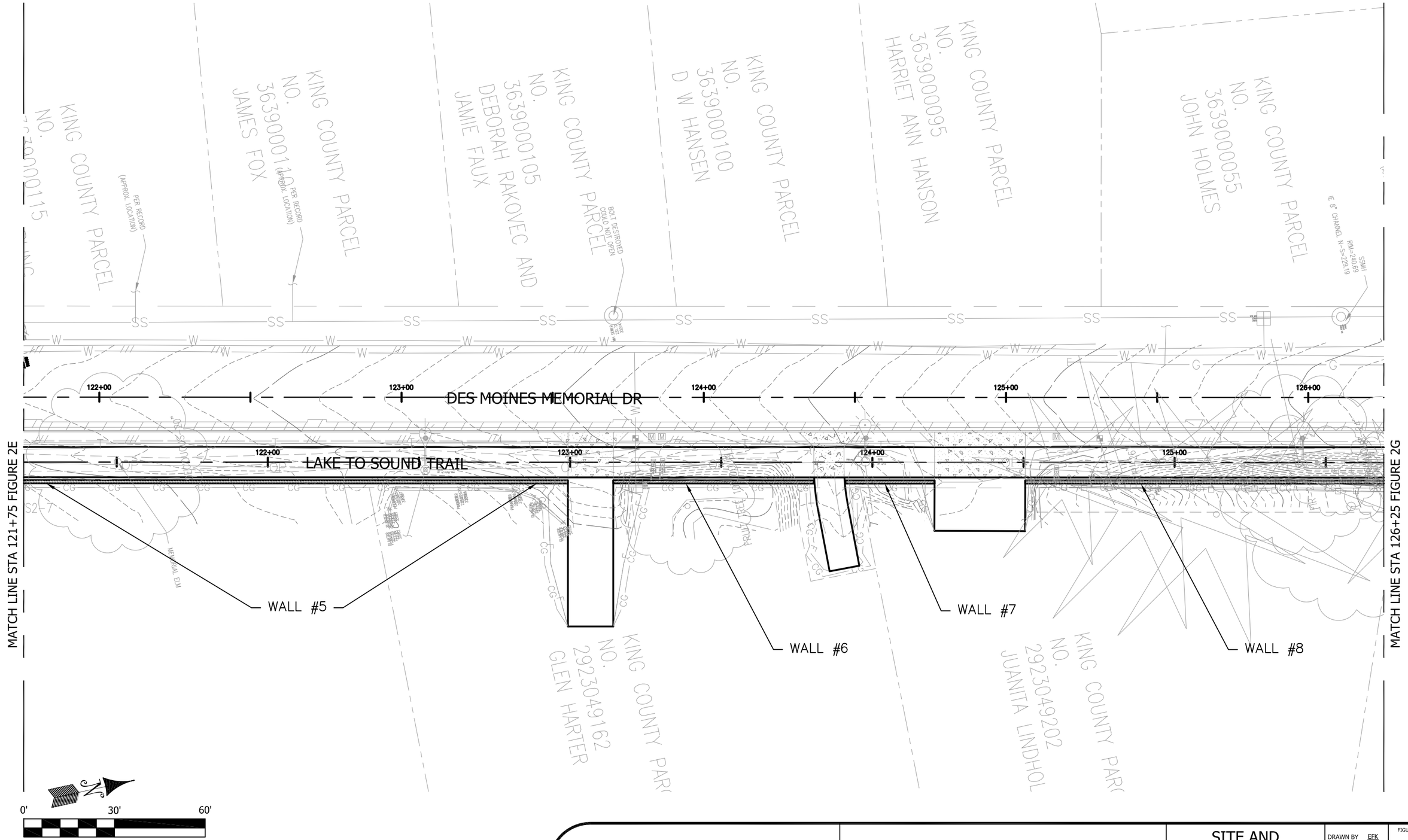
DATE	12.06.12

FIGURE NO. 2E

PROJECT NO. 2010-100-21 Task 1100



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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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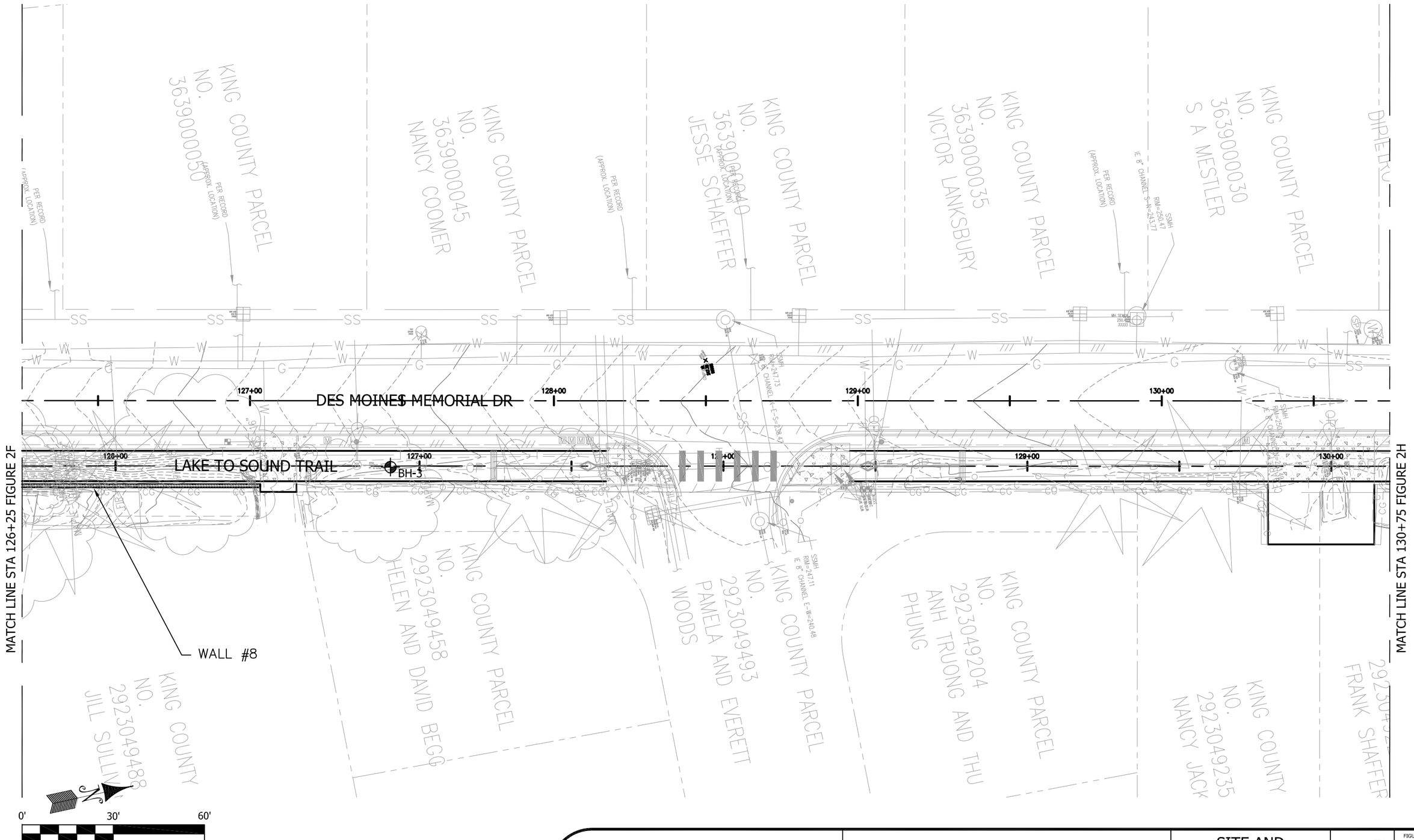
FIGURE NO. 2F

PROJECT NO. 2010-100-21 Task 1100



BH-3

BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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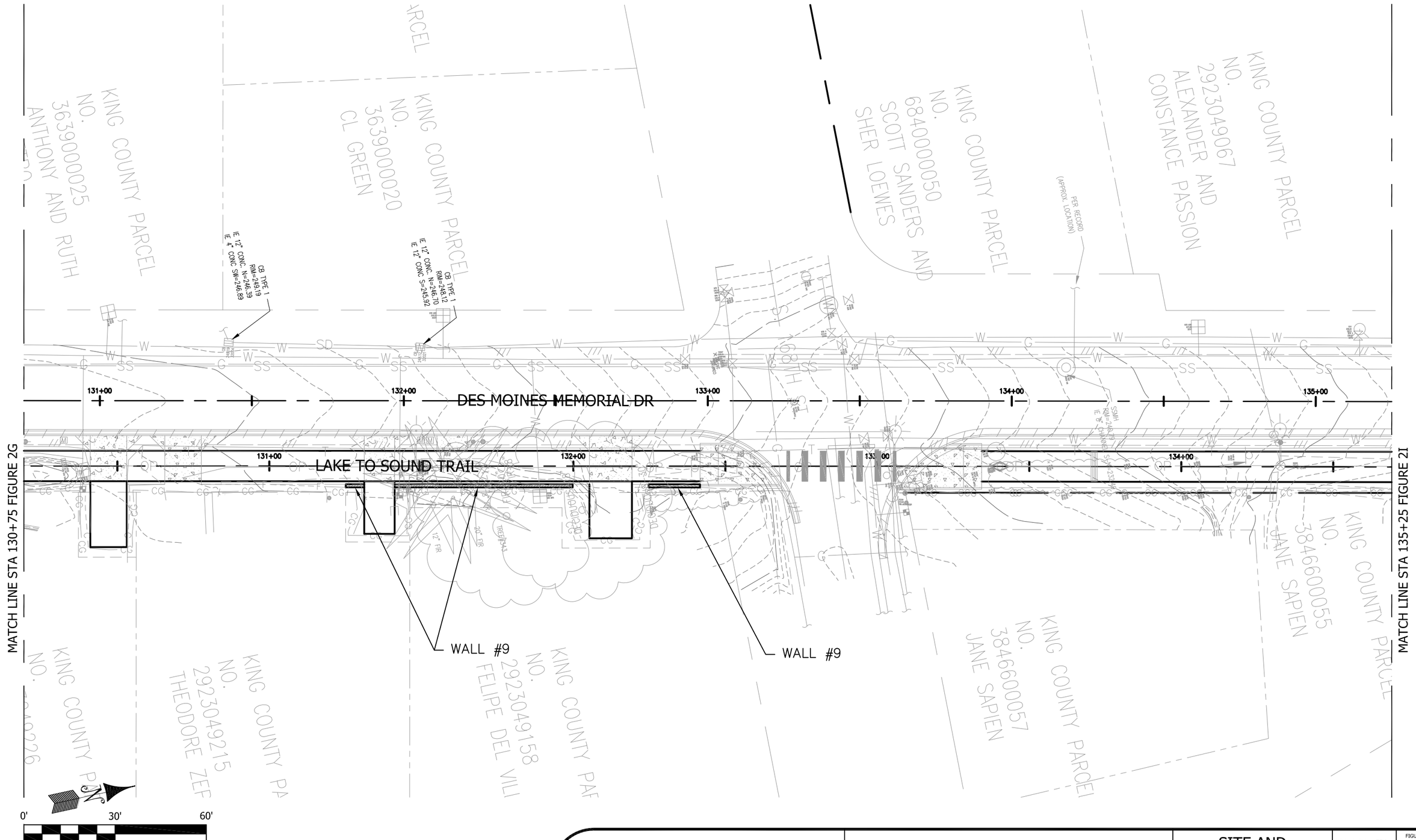
DATE	12.06.12

FIGURE NO. 2G

PROJECT NO. 2010-100-21 Task 1100



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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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DATE	12.06.12

FIGURE NO. 2H

PROJECT NO. 2010-100-21 Task 1100

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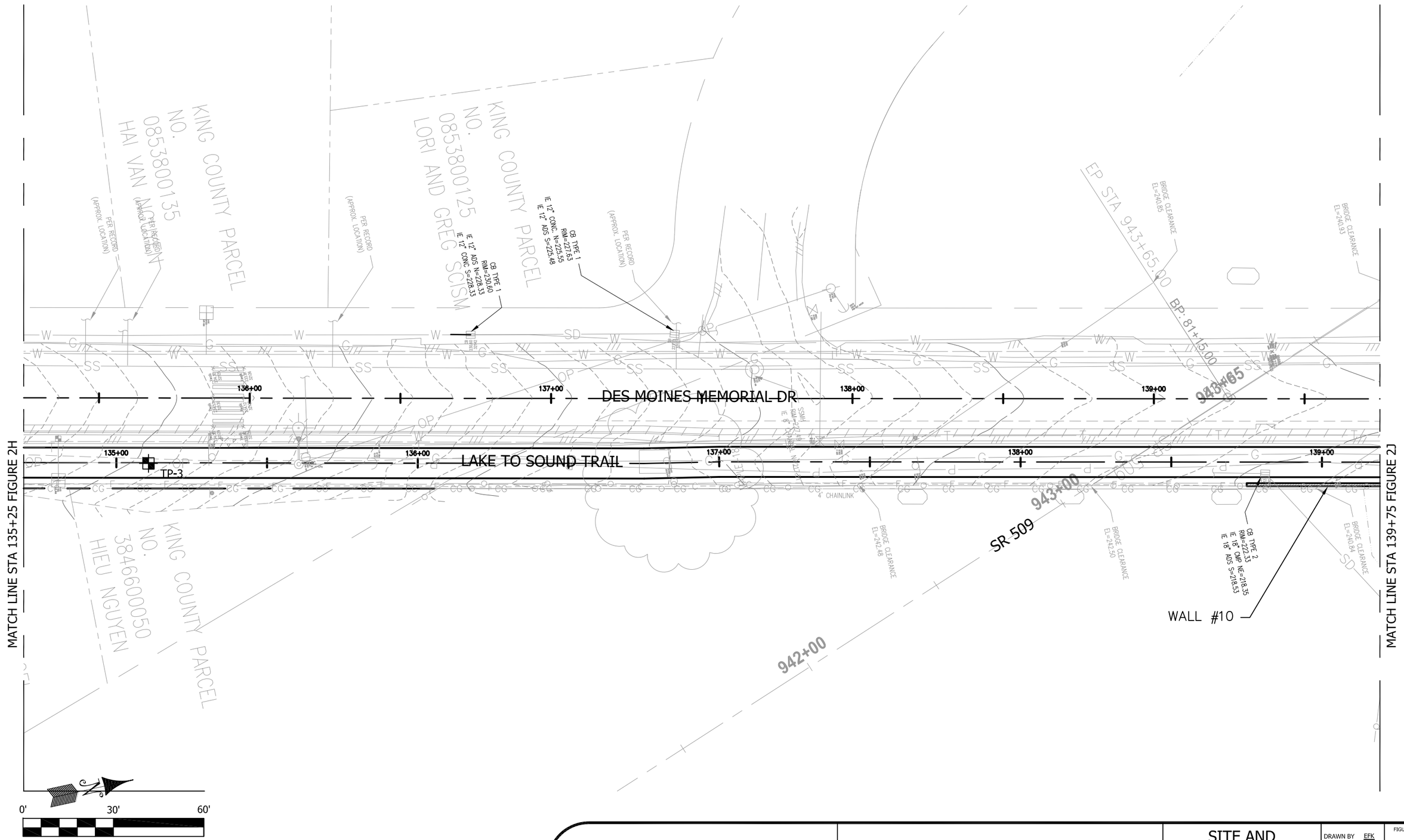

TP-3

TEST PIT DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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FIGURE NO. 2I

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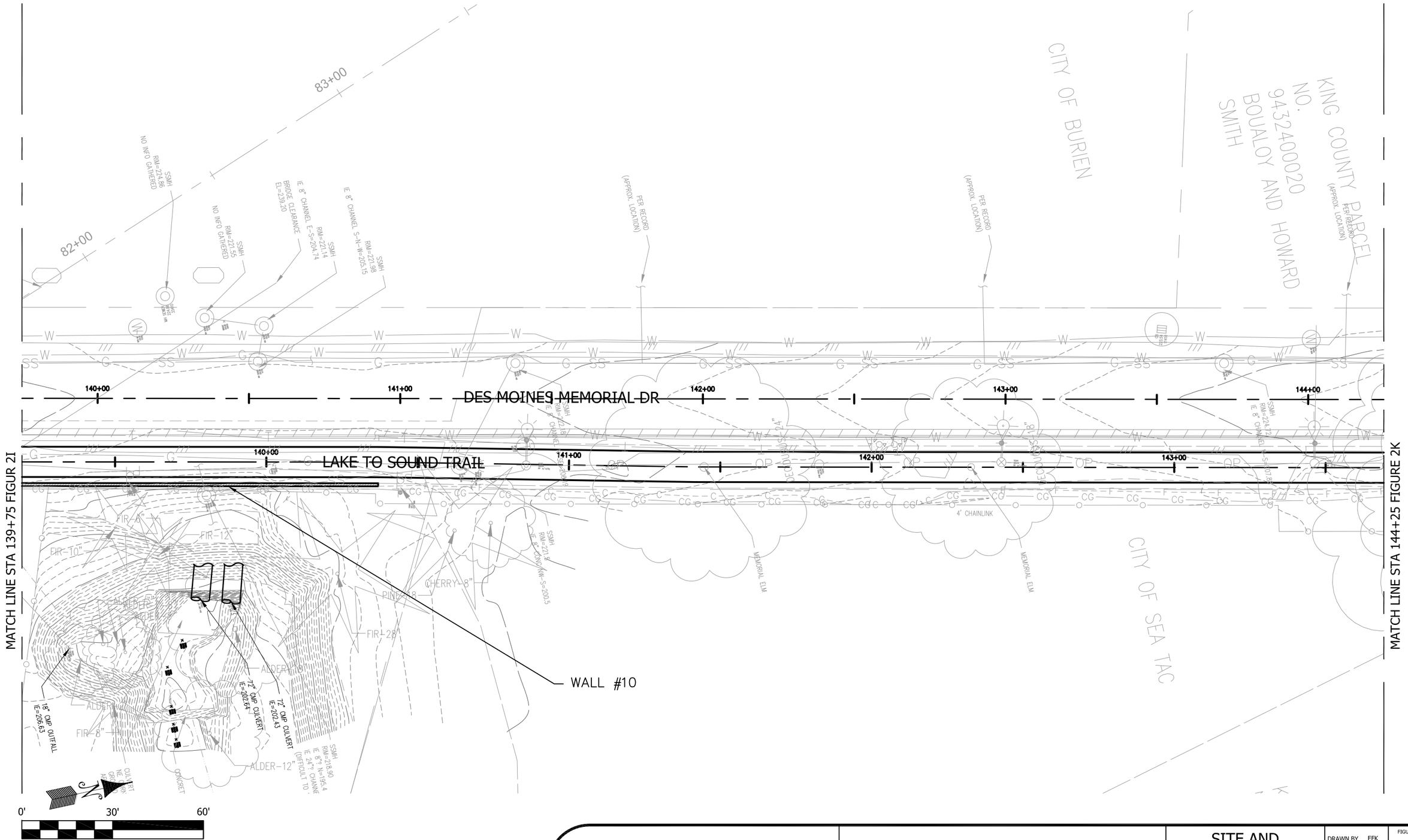

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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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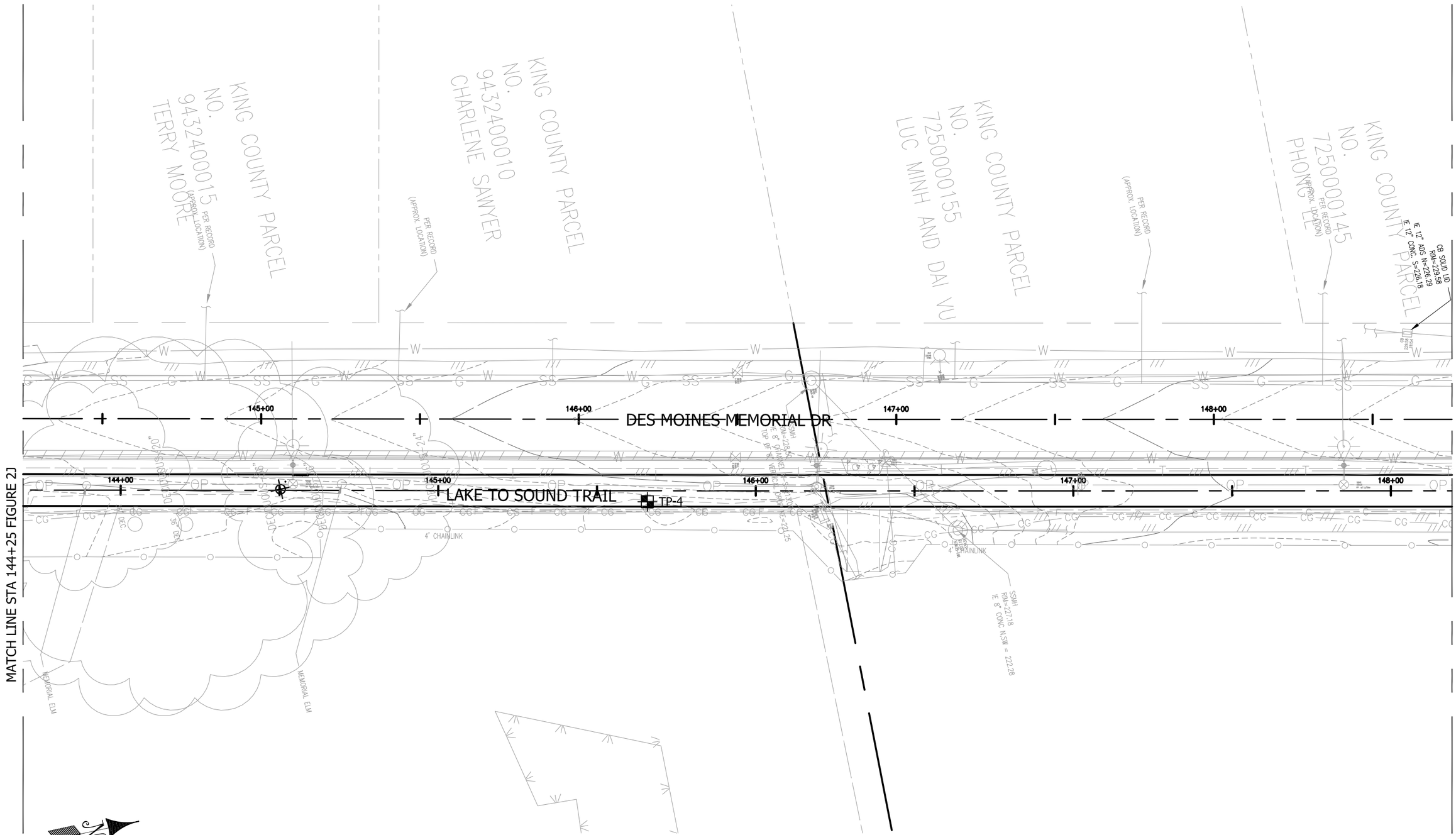
FIGURE NO. 2J

PROJECT NO. 2010-100-21 Task 1100



TP-4

TEST PIT DESIGNATION AND APPROXIMATE LOCATION

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LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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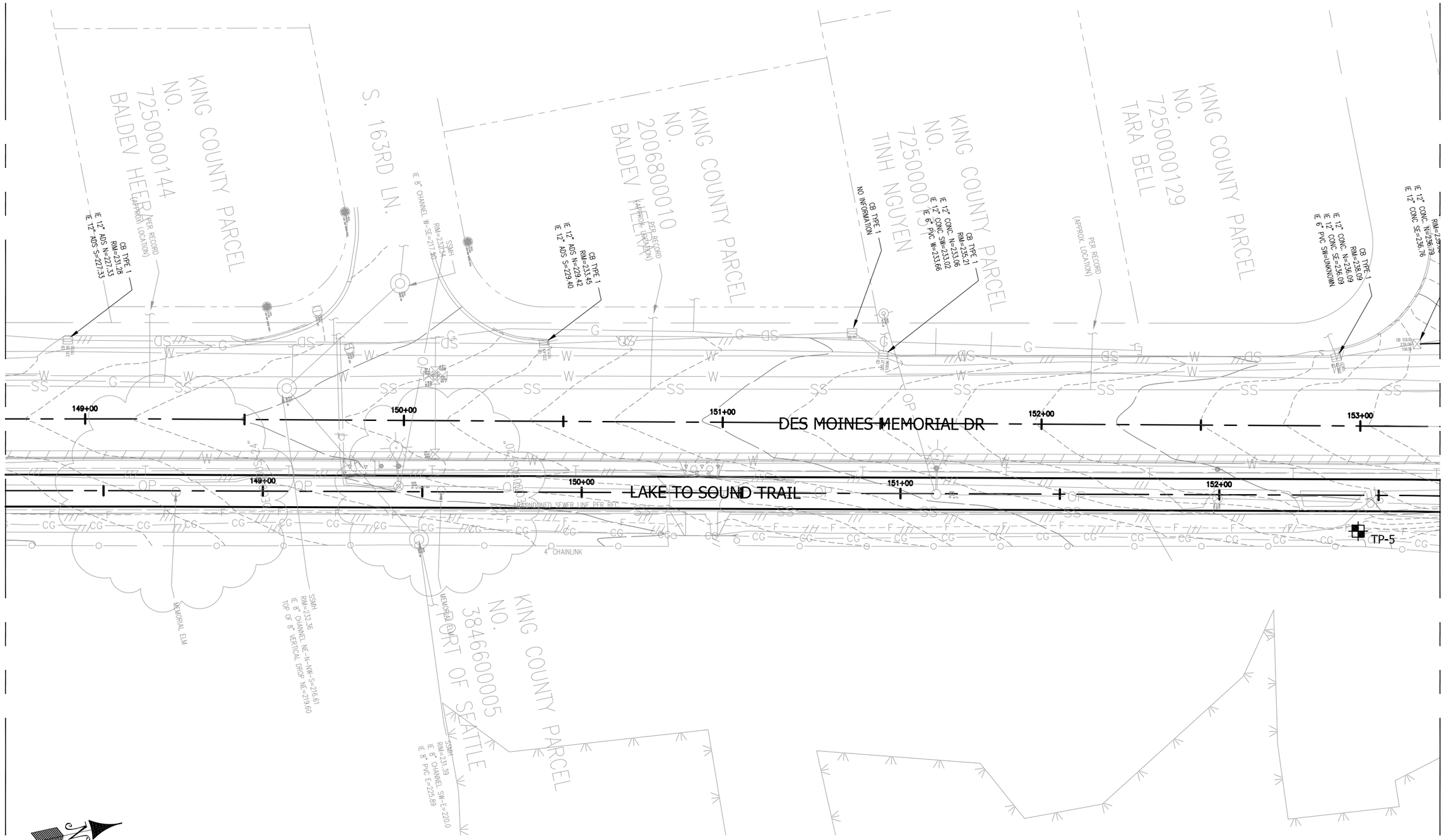
FIGURE NO. 2K

PROJECT NO. 2010-100-21 Task 1100



TP-5

TEST PIT DESIGNATION AND APPROXIMATE LOCATION

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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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DATE	12.06.12

FIGURE NO. 2L

PROJECT NO. 2010-100-21 Task 1100

TP-6

TEST PIT DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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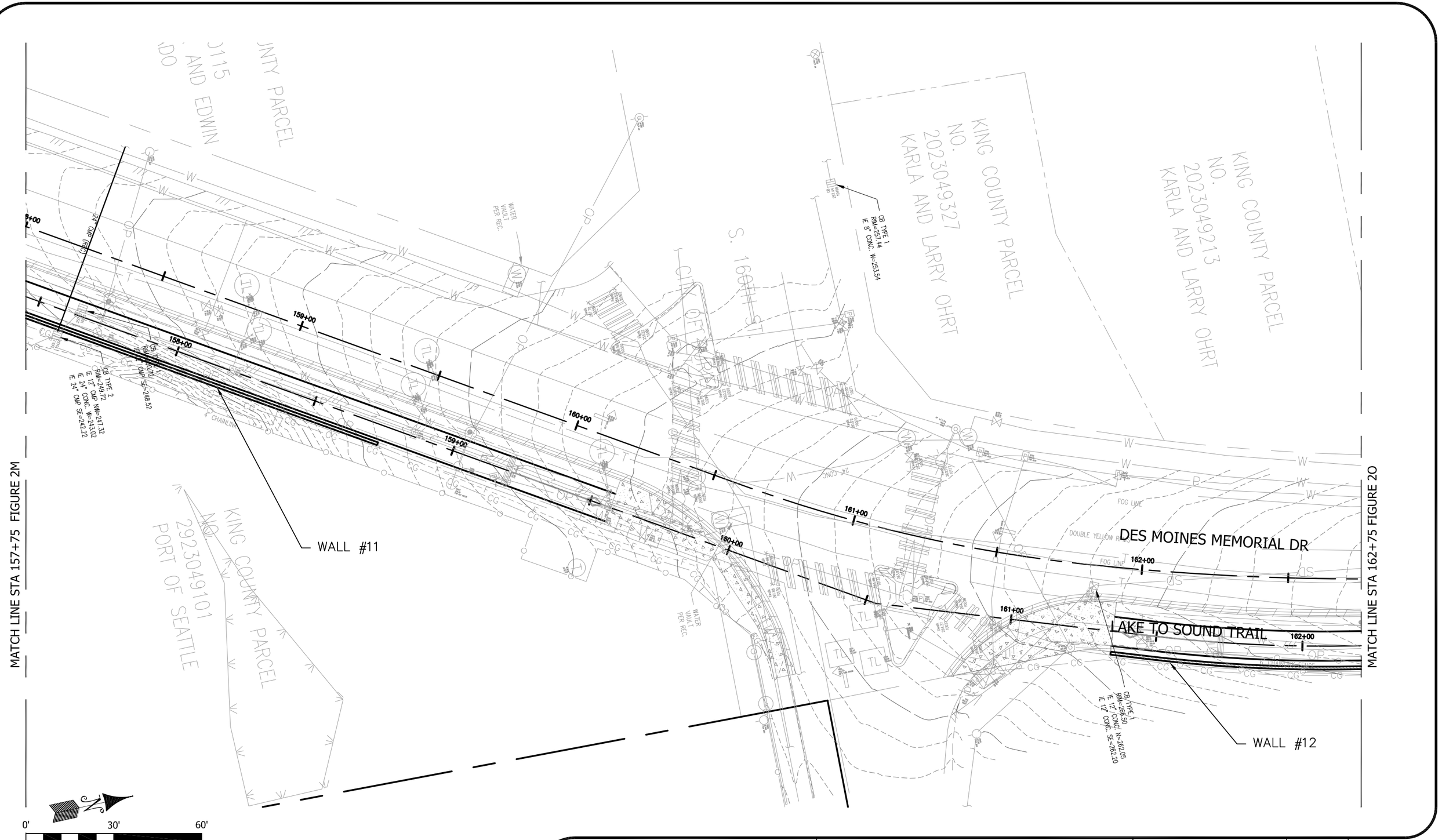
FIGURE NO. 2M

PROJECT NO. 2010-100-21 Task 1100



HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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DATE	12.06.12

FIGURE NO. 2N

PROJECT NO. 2010-100-21 Task 1100

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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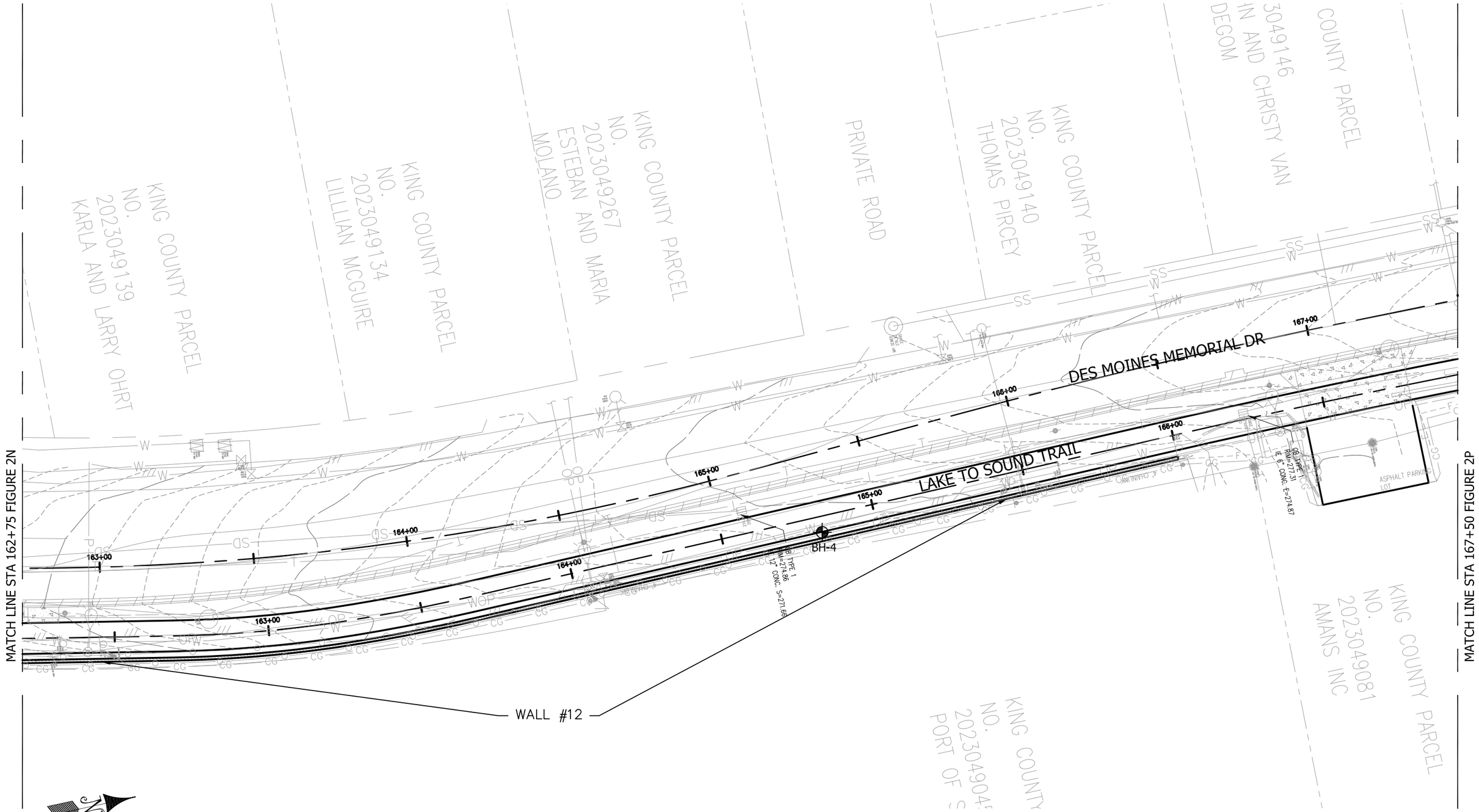

BH-4

BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

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HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

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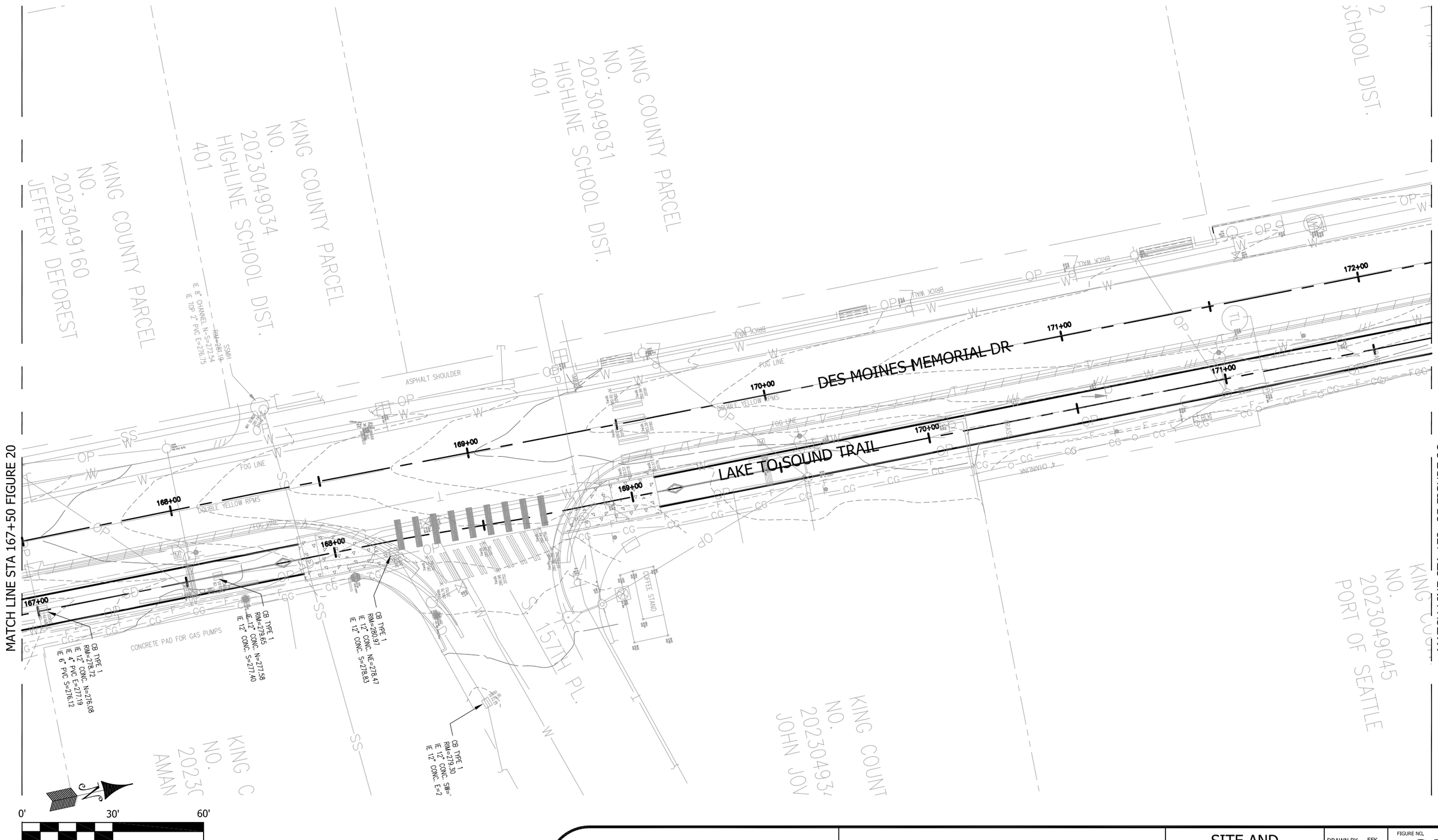
FIGURE NO.	20
PROJECT NO.	2010-100-21 Task 1100



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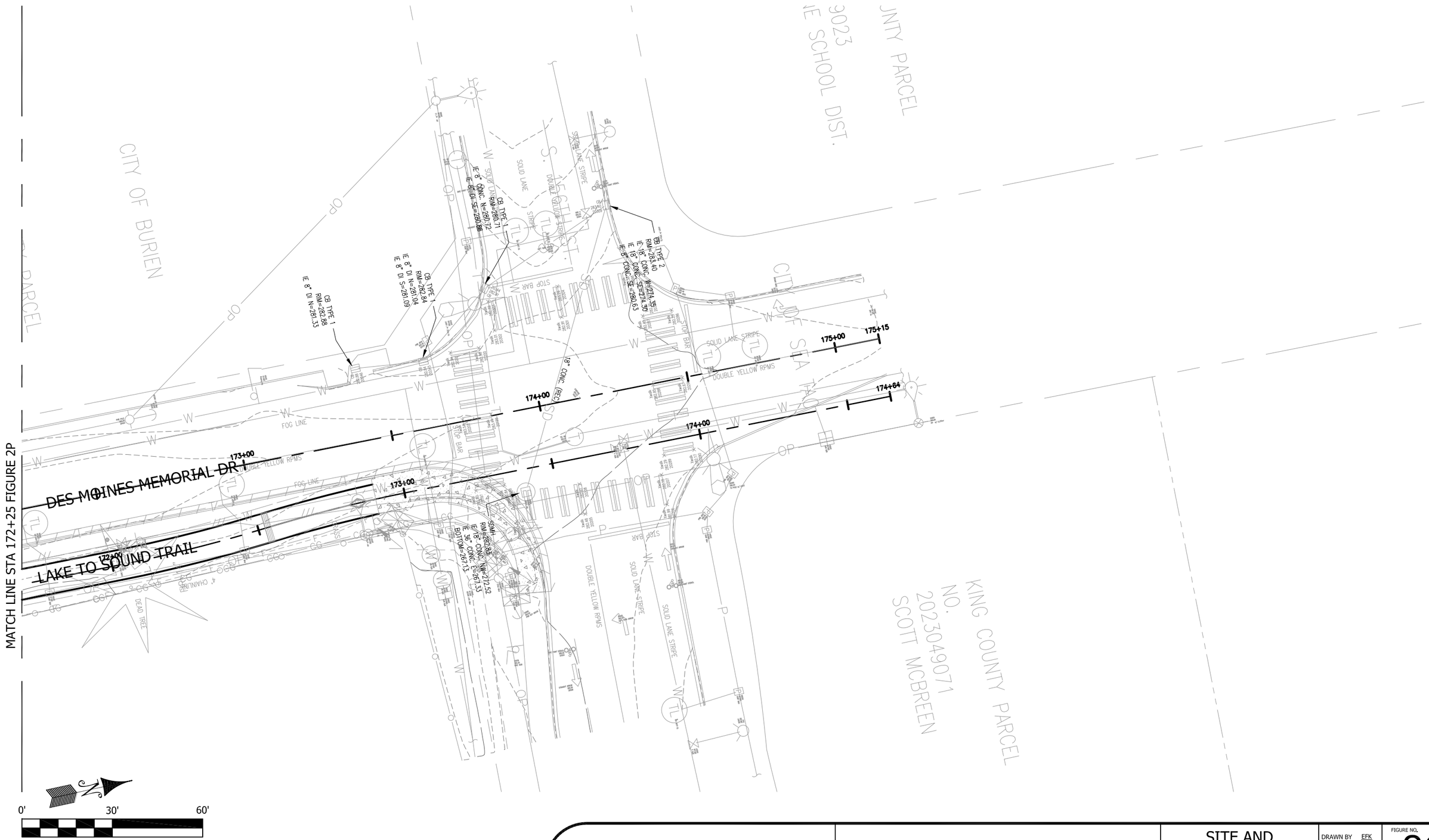
HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

DRAWN BY	EFK
CHECK BY	BWT
DATE	12.06.12
FIGURE NO.	2P
PROJECT NO.	2010-100-21 Task 1100



BASE MAP PROVIDED BY PARAMETRIX DATED: 11.10.10

S:\2010 PROJECTS\2010-100-21 LAKE TO SOUND TRAIL\CAD\HWA 2010-100.DWG <11x17 flg> Plotted: 12/27/2012 12:16 PM

HWA GEOSCIENCES INC.

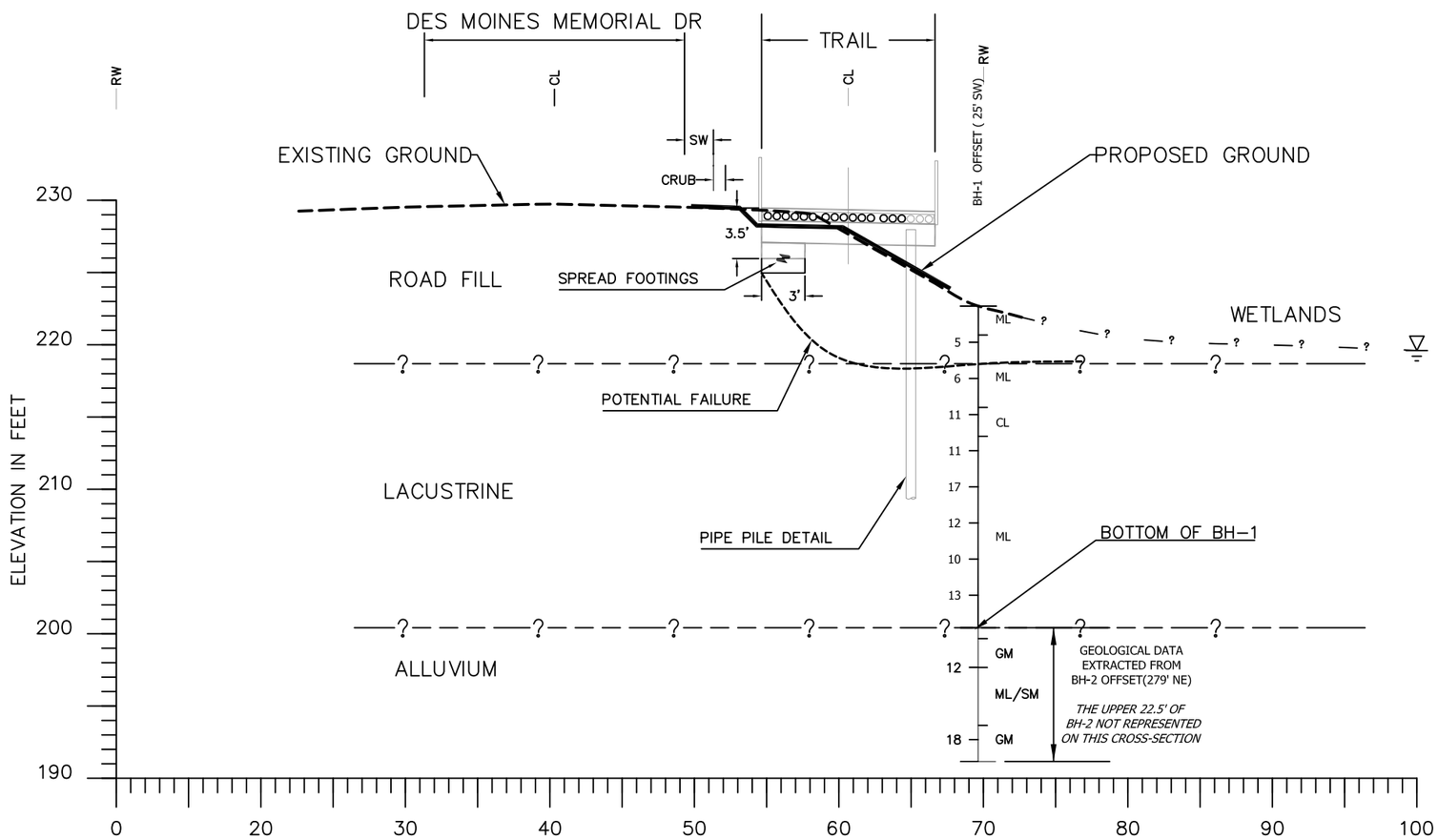
LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

SITE AND EXPLORATION PLAN

DRAWN BY	EFK
CHECK BY	BWT
DATE	12.06.12

FIGURE NO. 2Q

PROJECT NO. 2010-100-21 Task 1100



NOTE:

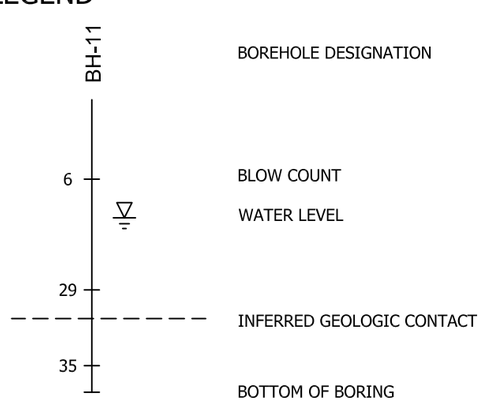
Soils
- The borings were terminated in lacustrine and alluvial materials due to drilling equipment limitation.
- The soils encountered above a depth of 28 ft generally consisted of medium stiff to stiff SILT and loose to medium dense SAND.
- We anticipate that a bearing layer of glacially consolidated soils exists at some depth below the site.

Vertical support
- We recommend driving piles to the bearing layer.
- The piles should be accepted based on driving criteria developed for the Contractor's specific hammer system.

Seismic
- The alluvial and lacustrine soils are potentially liquefiable, which may cause the slope to slide into the adjacent wetlands.
- Liquefaction would likely result in horizontal displacement of the trail, up to several feet.
- The design team has determined that it is impractical to provide mitigation measures or design the structure to withstand the forces imposed due to liquefaction.

The subsurface conditions shown are based on widely spaced borings and/or test pits 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.

LEGEND



HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL - WESTSIDE SECTION
KING COUNTY, WASHINGTON

TYPICAL BOARDWALK GEOLOGICAL CROSS-SECTION STA 116+00

DRAWN BY EFK
CHECK BY JG
DATE 12.20.12

FIGURE NO. 3

PROJECT NO. 2010-100-21 Task 1100

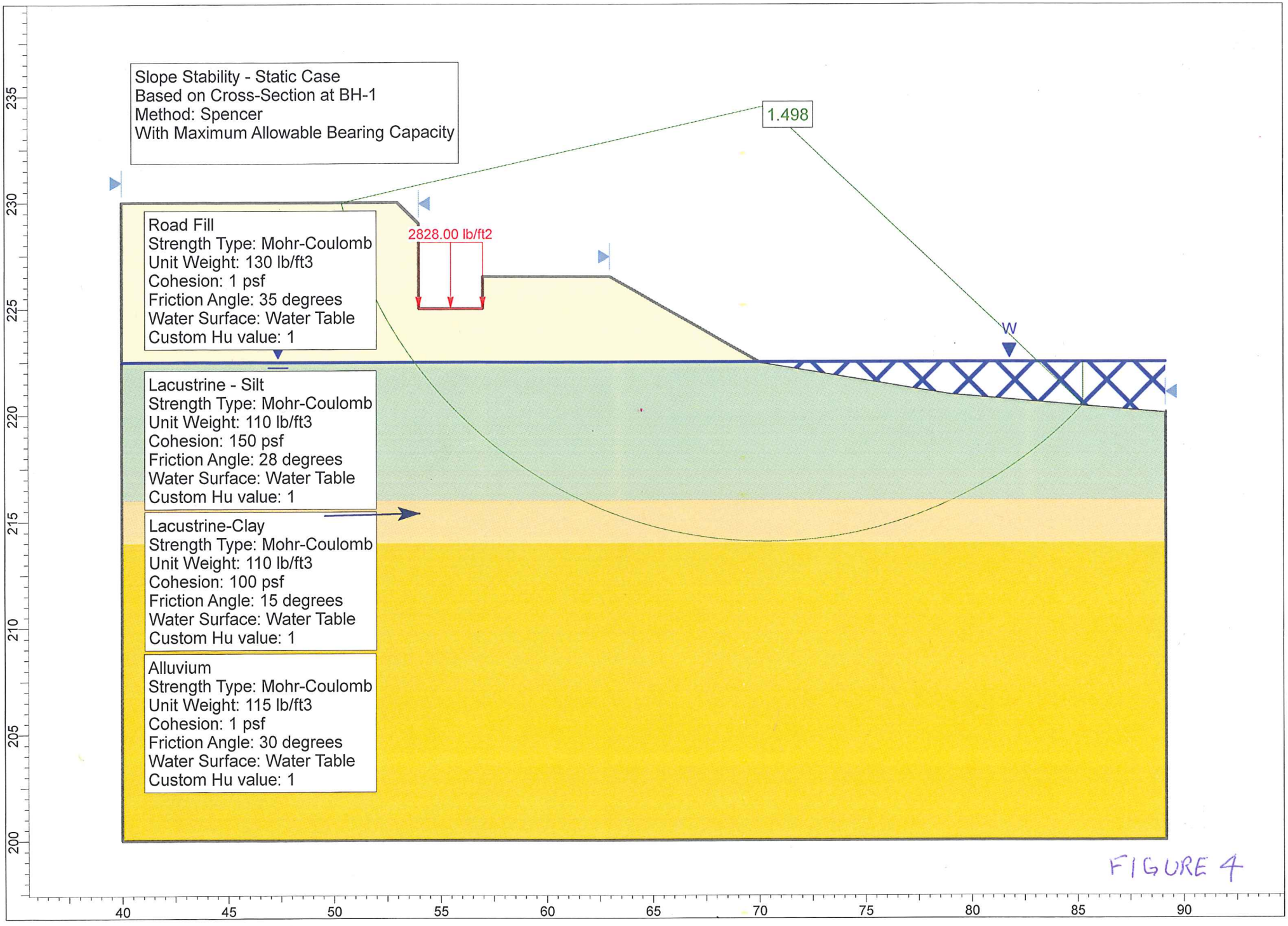
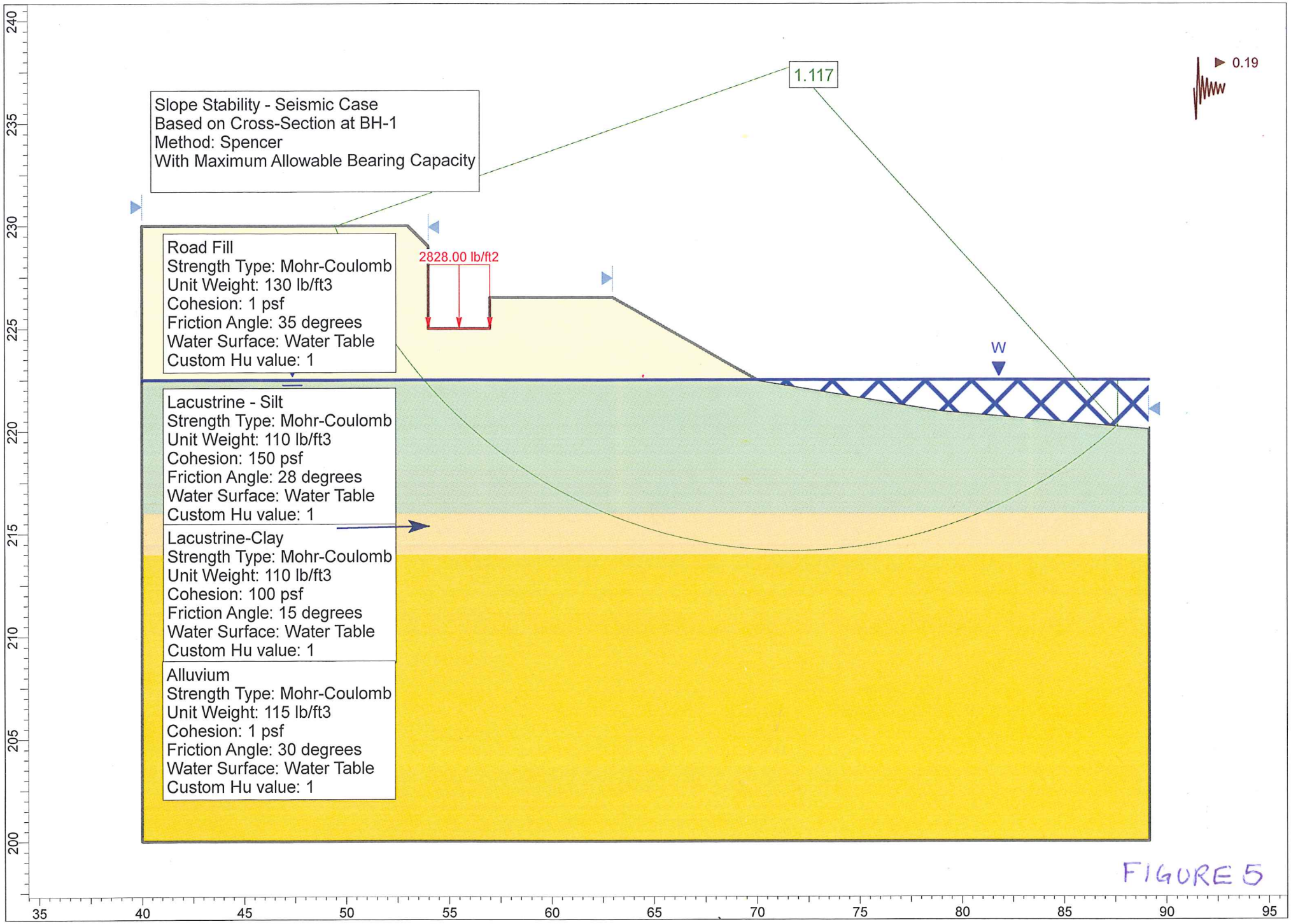

Slope Stability - Static Case
Based on Cross-Section at BH-1
Method: Spencer
With Maximum Allowable Bearing Capacity
1.498
2828.00 lb/ft2
W
Road Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 1 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1
Lacustrine - Silt
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 150 psf
Friction Angle: 28 degrees
Water Surface: Water Table
Custom Hu value: 1
Lacustrine-Clay
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 100 psf
Friction Angle: 15 degrees
Water Surface: Water Table
Custom Hu value: 1
Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 1 psf
Friction Angle: 30 degrees
Water Surface: Water Table
Custom Hu value: 1
235
230
225
220
215
210
205
200
40
45
50
55
60
65
70
75
80
85
90

FIGURE 4


Slope Stability - Seismic Case
Based on Cross-Section at BH-1
Method: Spencer
With Maximum Allowable Bearing Capacity
1.117
0.19
2828.00 lb/ft2
W
Road Fill
Strength Type: Mohr-Coulomb
Unit Weight: 130 lb/ft3
Cohesion: 1 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1
Lacustrine - Silt
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 150 psf
Friction Angle: 28 degrees
Water Surface: Water Table
Custom Hu value: 1
Lacustrine-Clay
Strength Type: Mohr-Coulomb
Unit Weight: 110 lb/ft3
Cohesion: 100 psf
Friction Angle: 15 degrees
Water Surface: Water Table
Custom Hu value: 1
Alluvium
Strength Type: Mohr-Coulomb
Unit Weight: 115 lb/ft3
Cohesion: 1 psf
Friction Angle: 30 degrees
Water Surface: Water Table
Custom Hu value: 1
240
235
230
225
220
215
210
205
200
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90
95

FIGURE 5

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS

A geotechnical subsurface exploration program and site reconnaissance was conducted by HWA in November and December, 2011. The field investigation consisted of excavating six test pits with a backhoe, drilling two borings along the existing road embankment, and drilling two borings at the foot of the embankment for a proposed pin-pile supported boardwalk. The exploration locations were chosen in consultation with Parametrix based on proposed fills, cuts, and the boardwalk. Locations were determined in the field by taping distances from surveyed site features and plotted on the Site and Exploration Plans (Figures 2A through 2Q).

The test pits were excavated by Kelly's Excavating, Inc. of Auburn, Washington, under subcontract to HWA. The test pits were excavated on November 29 and 30, 2011 with a Komatsu rubber-tired backhoe to depths of 8 to 12.5 feet below ground surface (BGS). Each of the pits was excavated from the road shoulder. Soil samples were obtained at selected intervals.

The boreholes were drilled on November 30 and December 1, 2011 by Geologic Drill Explorations, Inc. of Renton, Washington, under subcontract to HWA. Boreholes BH-1 and BH-2 were drilled at the toe of the road embankment adjacent to wetlands with a hand-portable Acker Soil Mechanic drill rig. Boreholes BH-3 and BH-4 were drilled in the road shoulder with a compact Bobcat MT-52 tracked drill rig. Both rigs advanced 2.25-inch inside diameter (ID), continuous-flight, hollow-stem augers. The boreholes were advanced to depths from 20.5 to 31.5 feet BGS.

Soil samples from the boreholes were collected at 2.5- to 5-foot intervals using Standard Penetration Test (SPT) sampling in general accordance with ASTM D-1586. SPT sampling consisted of using a 2-inch outside diameter, split-spoon sampler driven with a 140-pound drop hammer using a rope and cathead. During the test, a sample is obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches per stroke. The number of blows required for each 6 inches of penetration is recorded. The Standard Penetration Resistance ("N-value") of the soil is calculated as the number of blows required for the final 12 inches of penetration. If a total of 50 blows is recorded within a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the number of inches of actual penetration. This resistance, or N-value, provides an indication of the relative density of granular soils and the relative consistency of cohesive soils. Upon completion of drilling, the boreholes were backfilled using bentonite chips.

The borings and test pits were advanced under the full-time observation of an HWA engineering geologist. Soil samples obtained from the explorations were classified in the field and

representative portions were placed in plastic bags. These soil samples were then returned to our Bothell, Washington, laboratory for further examination and testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and ground water occurrence was recorded and used to develop logs of the explorations. A legend to the terms and symbols used on the exploration logs is presented on Figure A-1; summary logs of the explorations are presented on Figures A-2 through A-11. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and ground water conditions depicted are only for the specific dates and locations reported and, therefore, are not necessarily representative of other locations and times.

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
More than 50% Retained on No. 200 Sieve Size	Sand and Sandy Soils	Clean Sand (little or no fines)	SW	Well-graded SAND
			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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

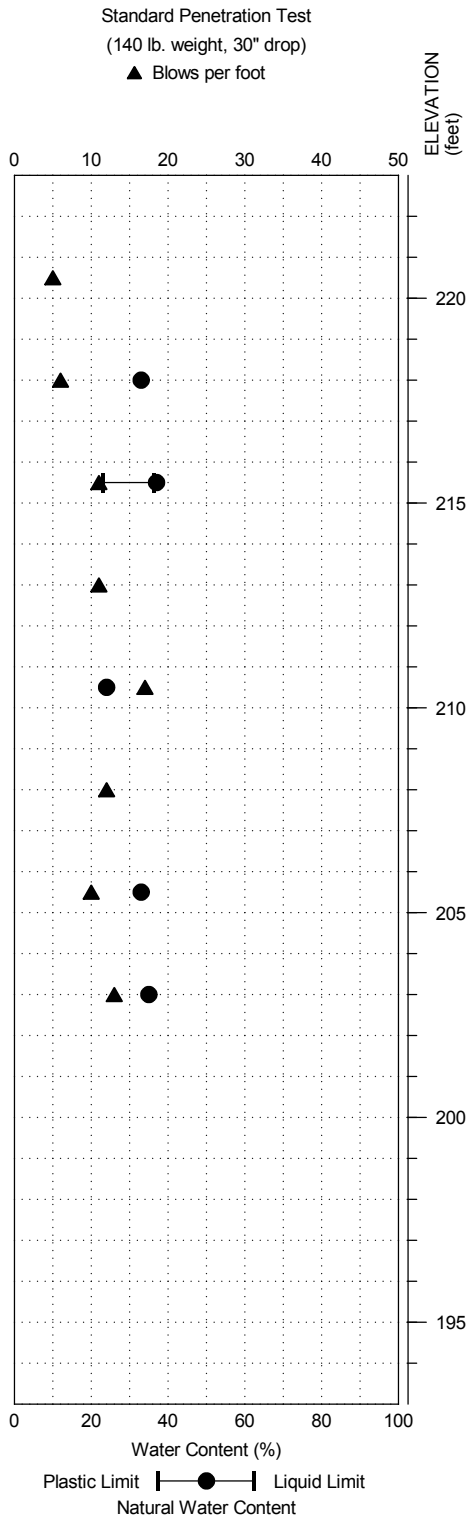
PROJECT NO.: 2010-100 FIGURE: A-1

DRILLING COMPANY: Geologic Drill, Inc.
DRILLING METHOD: Acker 4-inch OD Hollow Stem Auger
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figure 2D, STA 115+75

DATE STARTED: 11/30/2010
DATE COMPLETED: 11/30/2010
LOGGED BY: P. Pearson
SURFACE ELEVATION: 223.0 ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0		ML	Soft, brown, sandy SILT, wet. Abundant organics. Trace gravel. **(FILL)**		S-1	NA	
		ML	Medium stiff, brown to gray, slightly sandy SILT, wet. Organics. Gravel in sample tip. **(LACUSTRINE)**		S-2	1-1-4	
5			Medium stiff, olive gray, slightly sandy SILT, moist. Rust mottled. Some clay.		S-3	2-2-4	
		CL	Stiff, gray, silty CLAY, moist. Rust mottled. Trace sand.		S-4	3-6-5	AL
		ML					
10			Stiff, gray, fine sandy SILT, moist. Some rust mottling.		S-5	4-5-6	
			Very stiff, gray, silty fine SAND to fine sandy SILT, wet.		S-6	4-8-9	
15			Stiff, gray, slightly sandy SILT, moist to wet.		S-7	4-6-6	
			Stiff, gray, clayey SILT, moist. Trace sand.		S-8	3-4-6	
20			Stiff, gray, fine sandy SILT to silty fine SAND, wet. Trace gravel in sample tip.		S-9	3-6-7	
			Driller notes gravel.				
25			Boring terminated at 22.25 feet below ground surface due to refusal on cobble. Ground water observed at ground surface.				
30							

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

BOREHOLE
BH-1

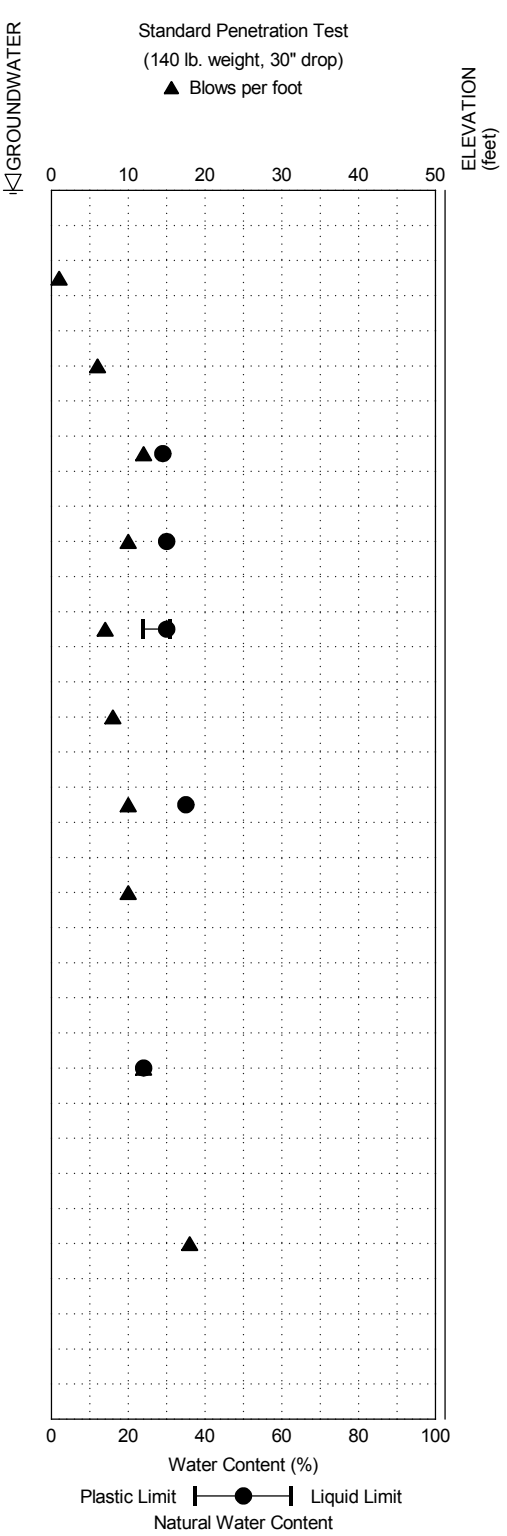
PROJECT NO.: 2010-100 FIGURE: A-2

DRILLING COMPANY: Geologic Drill, Inc.
DRILLING METHOD: Acker 4-inch OD Hollow Stem Auger
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figure 2E, STA 118+80

DATE STARTED: 11/30/2010
DATE COMPLETED: 11/30/2010
LOGGED BY: P. Pearson
SURFACE ELEVATION: 10' below road grade ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	SM	Loose, brown, silty SAND, wet. Some organics. **(FILL)**	S-1	NA	
	ML	Very loose, gray to dark gray, silty, fine to medium SAND, wet. Trace organics.	S-2	1-1-0	
5		Medium stiff, olive gray, slightly sandy SILT, moist to wet. Some clay. Laminated. **(LACUSTRINE)**	S-3	2-3-3	
	ML SM	Stiff, gray, fine sandy SILT, wet. Finely laminated.	S-4	6-7-5	GS
10			S-5	2-4-6	
	ML	Medium stiff, gray, slightly sandy SILT, wet.	S-6	3-3-4	AL
15		Medium stiff, gray, sandy SILT, moist to wet.	S-7	4-3-5	
		Stiff, gray, slightly sandy SILT, moist to wet. Fine laminations.	S-8	3-5-5	
20			S-9	4-4-6	
	GM	Driller notes gravel from 23 to 25 feet. **(ALLUVIUM)**			
25	ML SM	Stiff, gray, sandy SILT to silty SAND, moist to wet. Trace gravel.	S-10	4-4-8	
	GM	Driller notes gravel from 29 to 30 feet.			
30	ML	No recovery.	S-11	5-8-10	
35					

Boring terminated at 31.5 feet below ground surface.
Ground water observed at ground surface.



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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

BOREHOLE
BH-2

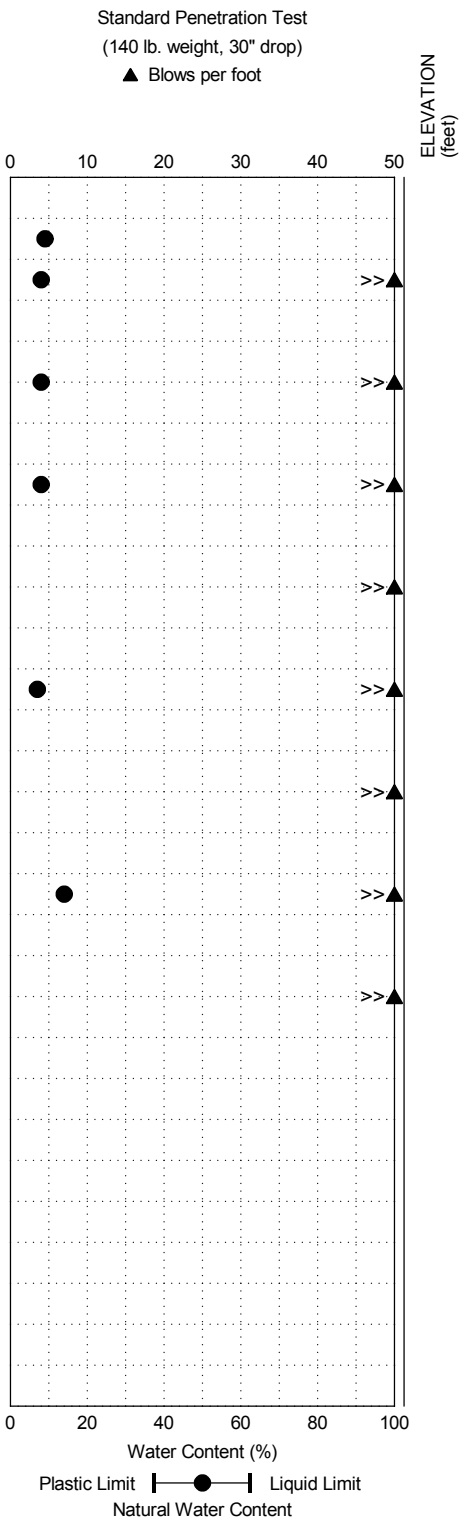
PROJECT NO.: 2010-100 FIGURE: A-3

DRILLING COMPANY: Geologic Drill, Inc.
DRILLING METHOD: Track Mounted 4-inch OD Hollow Stem Auger
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figure 2G, STA 126+90

DATE STARTED: 12/1/2010
DATE COMPLETED: 12/1/2010
LOGGED BY: D. Coltrane
SURFACE ELEVATION: Road grade ± feet

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	SM	Loose, brown, silty SAND with gravel, roots and rootlets, moist. **(TOPSOIL)**		S-1		GS
		Very dense, olive brown, gravelly, silty, fine to medium SAND, moist. **(GLACIAL TILL)**		S-2	19-25-44	GS
5				S-3	50/5.5"	
	SP SM	Very dense, gray, clean to slightly silty, fine to medium SAND, moist.		S-4	50/6"	
	SM	Very dense, olive brown, silty fine SAND, moist. Trace fine gravel to coarse sand. 1-2 inch layer of coarse sand slough on top of sample most likely from water bearing sand seam.		S-5	50/5.5"	
10		Very dense, olive gray, gravelly, silty fine SAND, moist.		S-6	50/6"	
15		Grades to gravelly, sandy SILT, moist.		S-7	50/5"	
		Very dense, olive gray, gravelly, silty, fine to medium SAND, wet. Ground water observed between 15 and 17.5 feet below ground surface.		S-8	23-40-50/5"	
20	SP	Very dense, gray, fine to medium SAND with trace silt, wet. **(ADVANCE OUTWASH)**		S-9	43-50/4"	
		Boring was terminated at 21 feet below ground surface (BGS). Ground water seepage was observed between 15 and 17.5 feet BGS.				
25						
30						

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

BOREHOLE
BH-3

PROJECT NO.: 2010-100 FIGURE: A-4

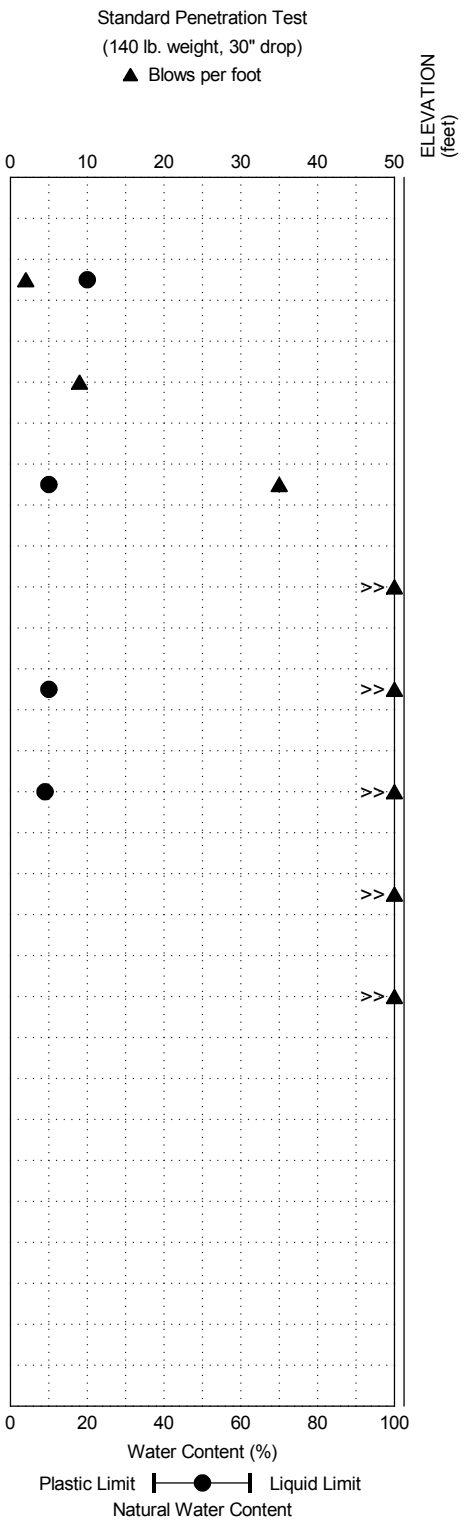
DRILLING COMPANY: Geologic Drill, Inc.
DRILLING METHOD: Track Mounted 4-inch OD Hollow Stem Auger
SAMPLING METHOD: SPT w/ Cathead
LOCATION: See Figure 2O, STA 165+30

DATE STARTED: 12/1/2010
DATE COMPLETED: 12/1/2010
LOGGED BY: D. Coltrane
SURFACE ELEVATION: 1.5' below road grade ± feet

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0			Loose, brown, gravelly, silty SAND with roots and rootlets, moist.				
		SM	**(TOPSOIL)**				
			Very loose, brown, slightly gravelly, silty SAND, moist. Roots and rootlets present. **(ALLUVIUM)**		S-1	1-1-1	GS
5			Rock stuck in sampler tip, no recovery besides rock. Ground water observed perching on till between 5 and 7.5 feet below ground surface.		S-2	3-5-4	
		SM	Very dense, olive brown, gravelly, silty SAND, moist. **(GLACIAL TILL)**		S-3	8-13-22	GS
10					S-4	15-20-33	
			Grades to gravelly.		S-5	15-28-32	
15					S-6	11-34-50/4"	
		SP	Very dense, gray, slightly silty, fine to medium SAND, wet. **(ADVANCE OUTWASH)**		S-7	19-29-50/4"	
20		ML	Hard, olive brown, sandy SILT with trace gravel, moist.		S-8	50/6"	
25							
30							

Boring terminated at 20.5 feet below ground surface (BGS). Ground water seepage was observed between 5 and 7.5 feet, and 12.5 and 15 feet BGS.

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

BOREHOLE
BH-4

PROJECT NO.: 2010-100 FIGURE: A-5

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu Rubber tired extenda-hoe
SURFACE ELEVATION: Road grade±feet

LOCATION: See Figure 2B, STA 104+35
DATE COMPLETED: 11/29/10
LOGGED BY: D. Coltrane

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0			Grass and gravel at surface. Topsoil is light brown, slightly gravelly, silty SAND, moist. **(TOPSOIL)**	S-1	9	
			Medium dense, olive brown, slightly silty, fine SAND, moist. **(RECESSIONAL OUTWASH)**	S-2	5	GS
		SP SM	Grades to interbedded, clean, slightly fine gravelly, fine to medium SAND, moist.	S-3		
5			Becoming gravelly from 6.0-7.0 feet BGS.	S-4	6	
		SM	Medium dense, brownish gray, slightly gravelly, silty, fine to medium SAND, moist.	S-5	15	GS
		GM	Very dense, light gray, silty, sandy, fine to coarse GRAVEL, moist. **(GLACIAL TILL)**	S-6	8	
10			Test pit was terminated at 10.0 feet below ground surface. No groundwater seepage was observed during the excavation.			
15						

TEST PIT PHOTO

EAST

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LOG OF TEST PIT
TP-1

PROJECT NO.: 2010-100 FIGURE: A-6

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu Rubber tired extenda-hoe
SURFACE ELEVATION: Road grade±feet

LOCATION: See Figure 2C, STA 110+45
DATE COMPLETED: 11/29/10
LOGGED BY: D. Coltrane

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0			Grass and plants at surface. Topsoil is brown, gravelly, silty SAND, moist. Abundant roots and rootlets. **(TOPSOIL)**				
		GM	Medium dense, light reddish brown, silty, very sandy GRAVEL, moist. **(WEATHERED RECESSIONAL OUTWASH)**		S-1	13	GS
					S-2	16	
		SM	Dense, light orangish brown to gray, slightly gravelly, silty SAND, moist. Trace rootlets. **(WEATHERED GLACIAL TILL)**				
5			Very dense, light gray with rust mottling, gravelly, silty SAND, moist. **(GLACIAL TILL)** Increasing moisture content with depth and consistent rust mottling.		S-3		
		SM					
					S-4		
10							
15			Test pit terminated at 12 feet below ground surface (BGS) due to heavy ground water seepage. Ground water seepage was observed at 12 feet BGS. After the excavation ground water seepage was observed from sidewalls at 10.0 feet and 7.0 feet BGS.				

TEST PIT PHOTO

WEST

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LOG OF TEST PIT
TP-2

PROJECT NO.: 2010-100 FIGURE: A-7

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu Rubber tired extenda-hoe
SURFACE ELEVATION: 0.25' below road grade±feet

LOCATION: See Figure 2I, STA 135+10
DATE COMPLETED: 11/29/10
LOGGED BY: D. Coltrane

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE / SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SW SM	Grass and trash at surface. Topsoil is brown, well-graded SAND, moist. Abundant roots and rootlets.	S-1	5	GS
		SM	**(TOPSOIL)**			
		SP SM	Loose, brownish gray, slightly silty, fine gravelly, fine to coarse SAND, moist. **(FILL)**	S-2	8	GS
			Loose, brown, slightly gravelly, silty SAND, moist.			
5		ML SM	Medium dense, dark yellowish brown, clean to slightly silty, fine SAND, moist. **(RECESSIONAL OUTWASH)**	S-3	16	
		SM	Dense, light orangish brown, to gray, fine sandy SILT , moist. Trace rootlets **(WEATHERED GLACIAL TILL)**	S-4		
			Very dense, light gray, gravelly, silty SAND, moist. **(GLACIAL TILL)**			
		SM	Becomes dense, light brown to light gray, silty fine SAND, moist.	S-5	19	
10		SM	Very dense, light gray, slightly gravelly, silty SAND, moist to wet.	S-6		
15			Test pit terminated at 12.5 feet below ground surface (BGS) due to ground water seepage. Ground water seepage observed at 12.5 feet BGS.			

TEST PIT PHOTO

WEST

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LOG OF TEST PIT
TP-3

PROJECT NO.: 2010-100 FIGURE: A-8

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu Rubber tired extenda-hoe
SURFACE ELEVATION: 1.0' below road grade±feet

LOCATION: See Figure 2K, STA 145+65
DATE COMPLETED: 11/30/10
LOGGED BY: D. Coltrane

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0			Grass and gravel at surface. Topsoil is light brown, slightly gravelly, silty SAND, moist. **(TOPSOIL)**				
		SM	Loose, reddish brown, silty fine SAND, moist. Abundant roots and rootlets. **(WEATHERED OUTWASH)**		S-1	21	GS
			Medium dense, light olive brown with oxide mottling, slightly silty, fine SAND, moist. **(RECESSIONAL OUTWASH)**		S-2	20	GS
					S-3	21	
5		SP SM	Sand becomes coarser, increasing moisture content.		S-4		
			Ground water perching on clay.				
10		CL	Hard, bluish gray, CLAY with scattered gravel and sand, moist. **(GLACIOLACUSTRINE)**		S-5		
15			Test pit terminated at 11.0 feet below ground surface (BGS) due to caving in the sand layers caused by perched water on the clay layer. Ground water seepage was observed at 4.75 BGS feet BGS from SE corner.				

TEST PIT PHOTO

WEST

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LOG OF TEST PIT
TP-4

PROJECT NO.: 2010-100 FIGURE: A-9

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu Rubber tired extenda-hoe
SURFACE ELEVATION: 0.5' below road grade±feet

LOCATION: See Figure 2L, STA 152+45
DATE COMPLETED: 11/30/10
LOGGED BY: D. Coltrane

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0		SM	Grass and gravel at surface. Topsoil is light brown, slightly gravelly, silty SAND, moist to wet. **(TOPSOIL)**				
		SP	Loose, brown, slightly gravelly, silty SAND, moist. Roots and rootlets present. **(FILL)**				
		SM	Loose, reddish brown, slightly gravelly, silty SAND, moist. Scattered roots and rootlets. **(WEATHERED OUTWASH)**		S-1	19	GS
			Medium dense, light olive brown with oxide mottling, very silty fine SAND, moist. **(RECESSIONAL OUTWASH)**		S-2	22	GS
5			Layers of oxidized silt interbedded with fine sand. Boulder encountered.		S-3	25	
		CL	Hard, bluish gray, CLAY with trace gravel, moist. **(GLACIOLACUSTRINE)**		S-4		
10		SM	Very dense, gray, gravelly, silty SAND, moist. **(GLACIAL TILL)**		S-5		
15			Test pit terminated at 11.5 feet below ground surface (BGS). Ground water seepage was observed at bottom of sod and at 7.5 feet BGS perching on the glaciolacustrine layer.				

TEST PIT PHOTO

WEST

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LOG OF TEST PIT
TP-5

PROJECT NO.: 2010-100 FIGURE: A-10

EXCAVATION COMPANY: Kelly's Excavating
EXCAVATING EQUIPMENT: Komatsu Rubber tired extenda-hoe
SURFACE ELEVATION: 1.5' below road grade±feet

LOCATION: See Figure 2M, STA 157+40
DATE COMPLETED: 11/30/10
LOGGED BY: D. Coltrane

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	MOISTUE CONTENT (%)	OTHER TESTS
0			Grass and gravel at surface. Topsoil is light brown, gravelly, silty SAND, moist. **(TOPSOIL)**				
		SP	Medium dense, light olive brown with oxide mottling, gravelly silty fine to medium SAND with, moist to wet. **(ALLUVIUM)**		S-1	12	GS
		SM	Medium dense, bluish gray, slightly gravelly, silty, fine to medium SAND, wet. Trace organics.		S-2		
5					S-3	16	
					S-4	72	
		OH	Soft, brown, ORGANIC SILT, with wood debris and gravel, moist. Organic Content = 14.65%				
		GP	Dense, gray, sandy GRAVEL to gravelly SAND, wet. **(RECESSIONAL OUTWASH)**				
10			Test pit terminated at 8.0 feet below ground surface (BGS) due to caving caused by ground water seepage at 4 feet BGS. Ground water seepage observed at bottom of sod layer and 4 feet BGS.				
15							

TEST PIT PHOTO

WEST

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.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

LOG OF TEST PIT
TP-6

PROJECT NO.: 2010-100 FIGURE: A-11

APPENDIX B

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

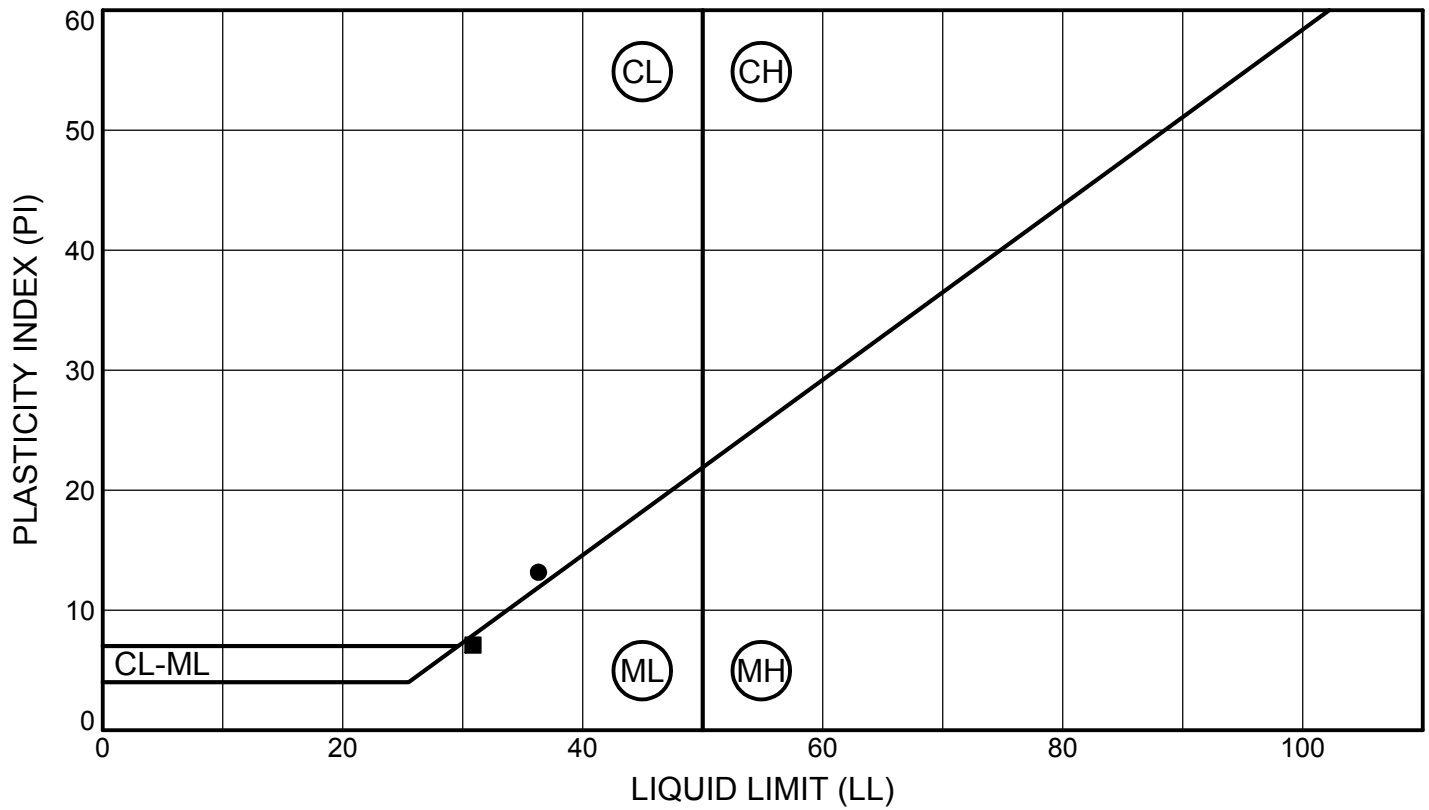
HWA personnel performed laboratory tests in general accordance with appropriate ASTM test methods. We tested selected soil samples to determine moisture content, grain-size distribution, and Atterberg Limits. The test procedures and results are briefly discussed below.

MOISTURE CONTENT OF SOIL: Laboratory tests were conducted to determine the natural moisture content of selected soil samples, in general accordance with ASTM D-2216. Test results are indicated at the sampled intervals on the appropriate exploration logs in Appendix A.

ORGANIC CONTENT OF SOIL: Laboratory tests were conducted to determine the organic content of selected soil samples, in general accordance with ASTM D-2216. Test results are indicated at the sampled intervals on the appropriate exploration logs in Appendix A.

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ATTERBERG LIMITS): Selected samples were tested using method ASTM D 4318, multi-point method. The results are reported on the attached Liquid Limit, Plastic Limit, and Plasticity Index report, Figure B-1.

PARTICLE SIZE ANALYSIS OF SOILS: Selected samples were tested to determine the particle size distribution of material in general accordance with ASTM D422. The results are summarized on the attached Grain Size Distribution reports, Figures B-2 through B-6, and provide information regarding the classification of the sample.



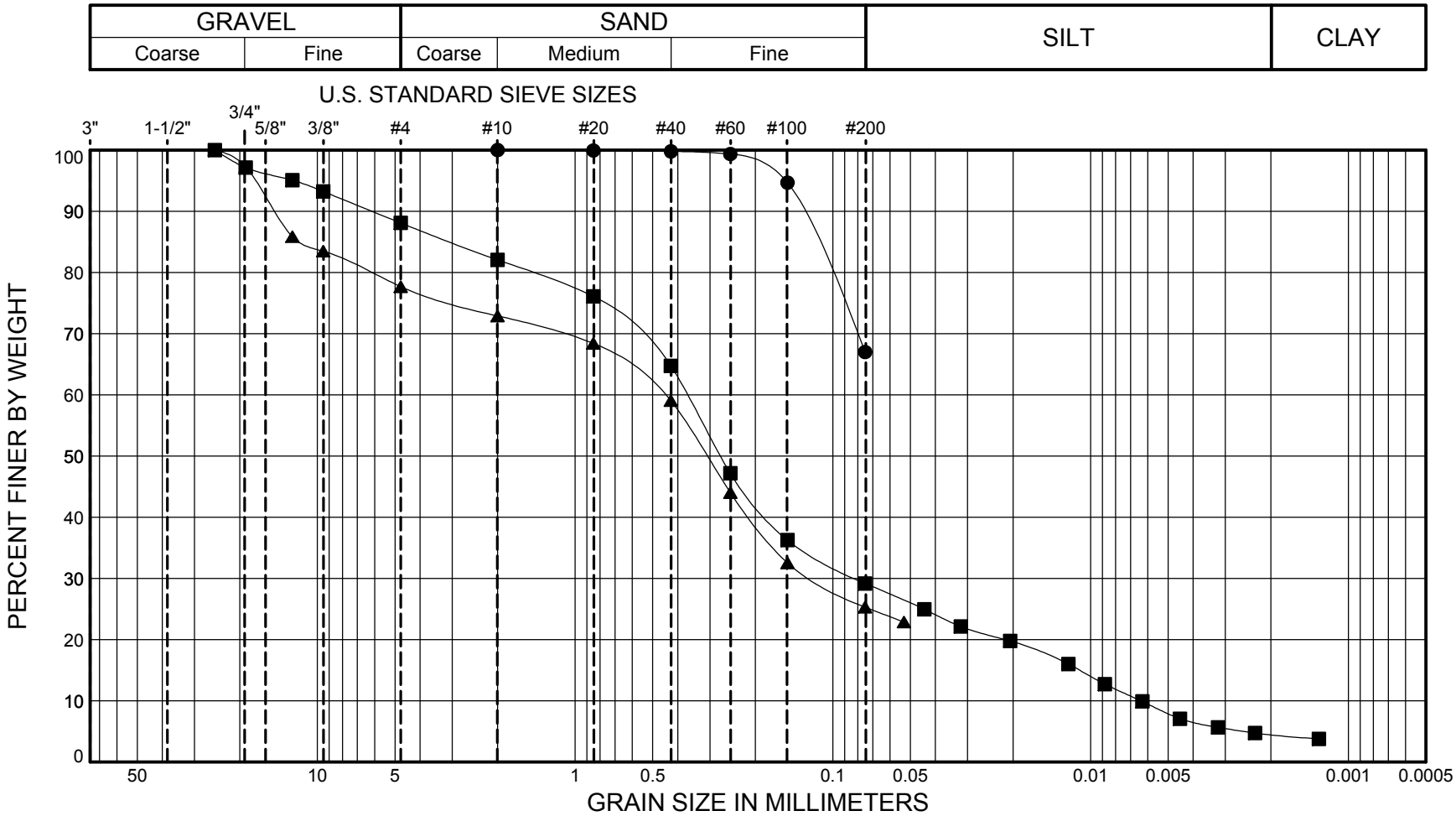
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	BH-1	S-4	7.5 - 9.0	(CL) Olive gray, lean CLAY	37	36	23	13	
■	BH-2	S-6	12.5 - 14.0	(ML) Dark gray, SILT	30	31	24	7	

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

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

PROJECT NO.: 2010-100 FIGURE: B-1



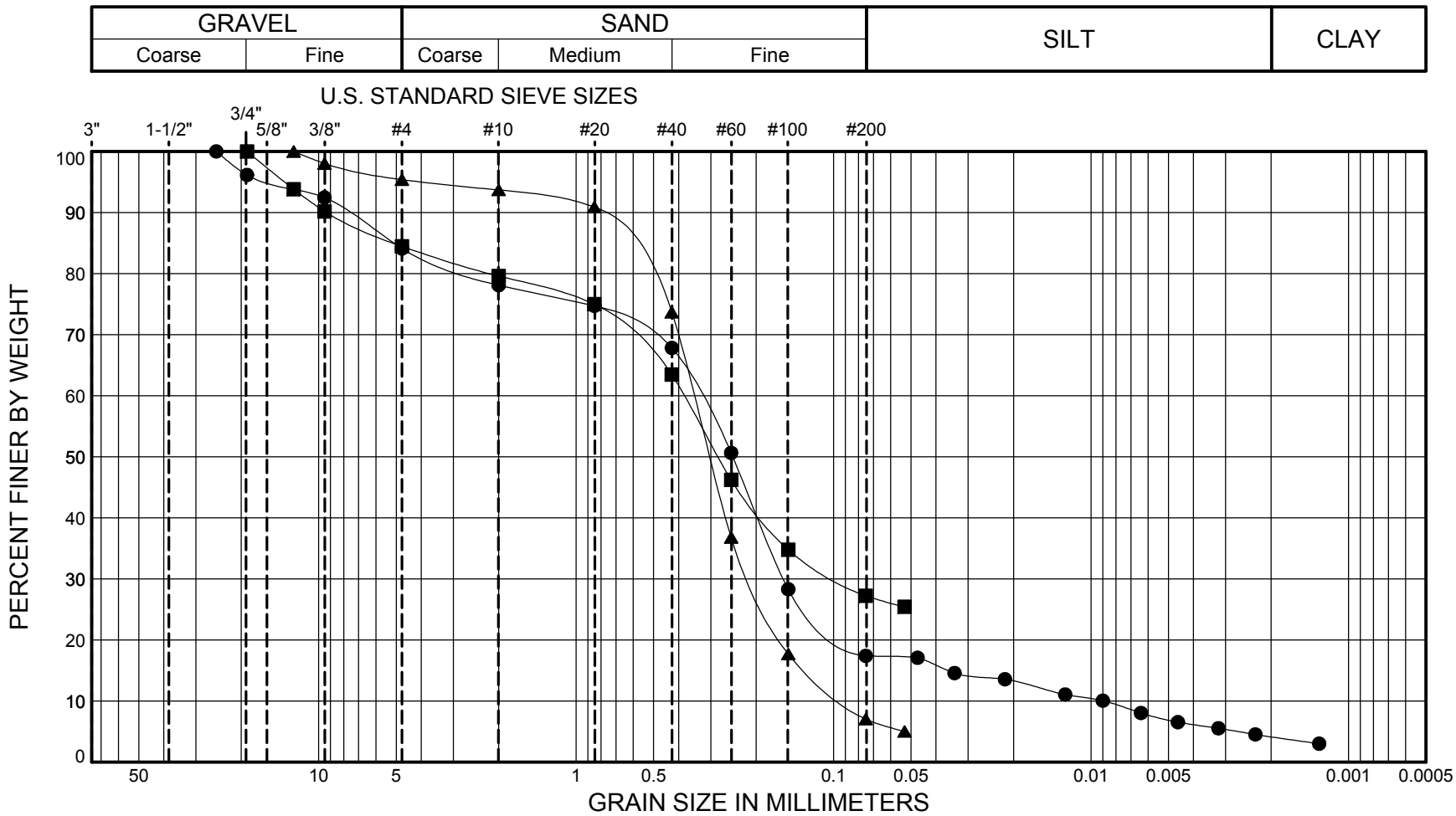
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-2	S-4	7.5 - 9.0	(ML) Gray, sandy SILT	29				0.0	33.0	67.0
■	BH-3	S-1	1.5 - 1.8	(SM) Grayish brown, silty SAND	9				11.9	58.9	29.2
▲	BH-3	S-2	2.5 - 4.0	(SM) Olive brown, silty SAND with gravel	8				22.3	52.4	25.3

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2010-100 FIGURE: B-2



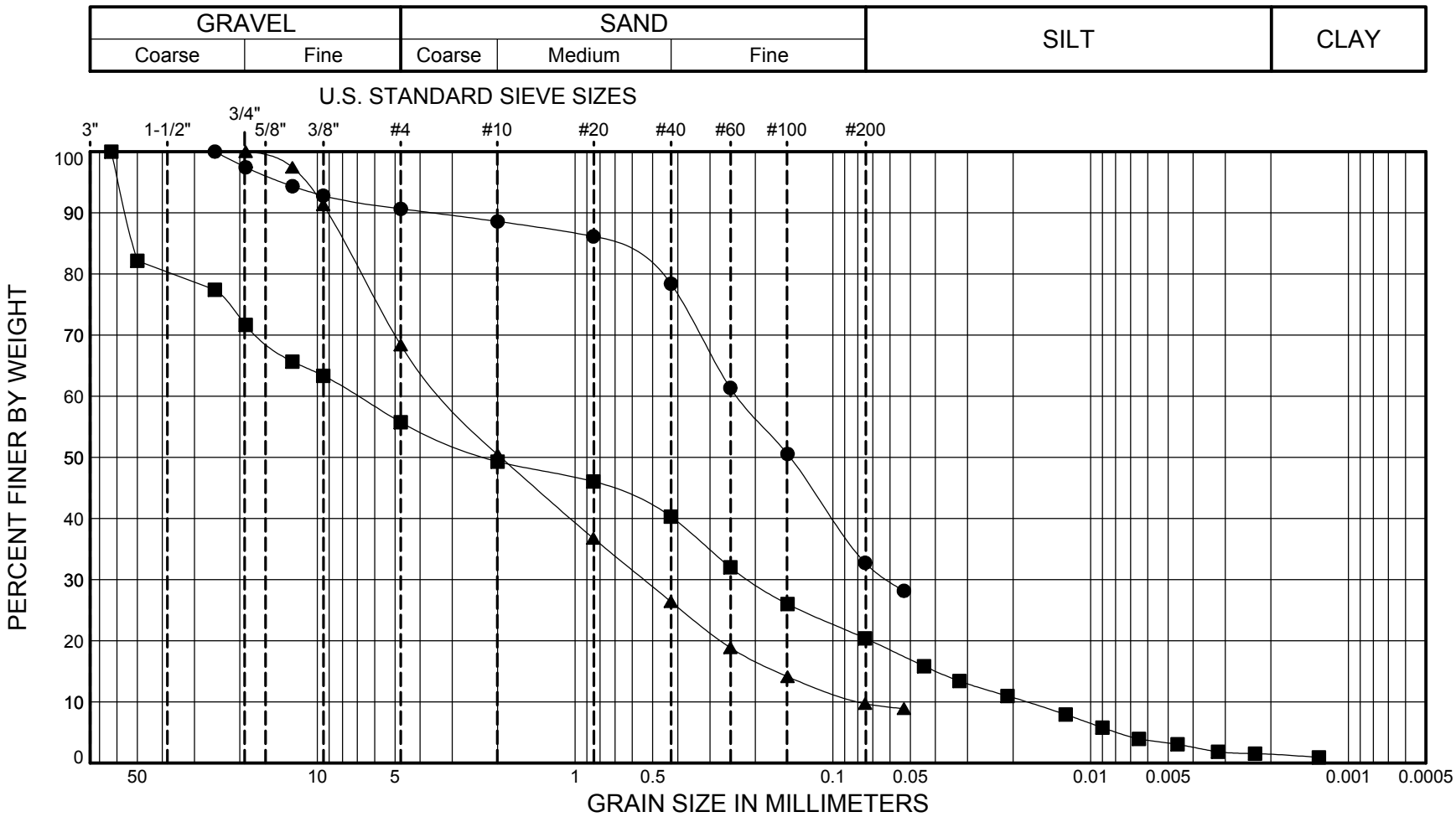
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	BH-4	S-1	2.5 - 4.0	(SM) Brown, silty SAND with gravel	20				15.9	66.7	17.4
■	BH-4	S-3	7.5 - 9.0	(SM) Olive brown, silty SAND with gravel	10				15.5	57.2	27.2
▲	TP-1	S-2	3.0 - 3.5	(SP-SM) Olive brown, poorly graded SAND with silt	5				4.6	88.3	7.0

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2010-100 FIGURE: B-3



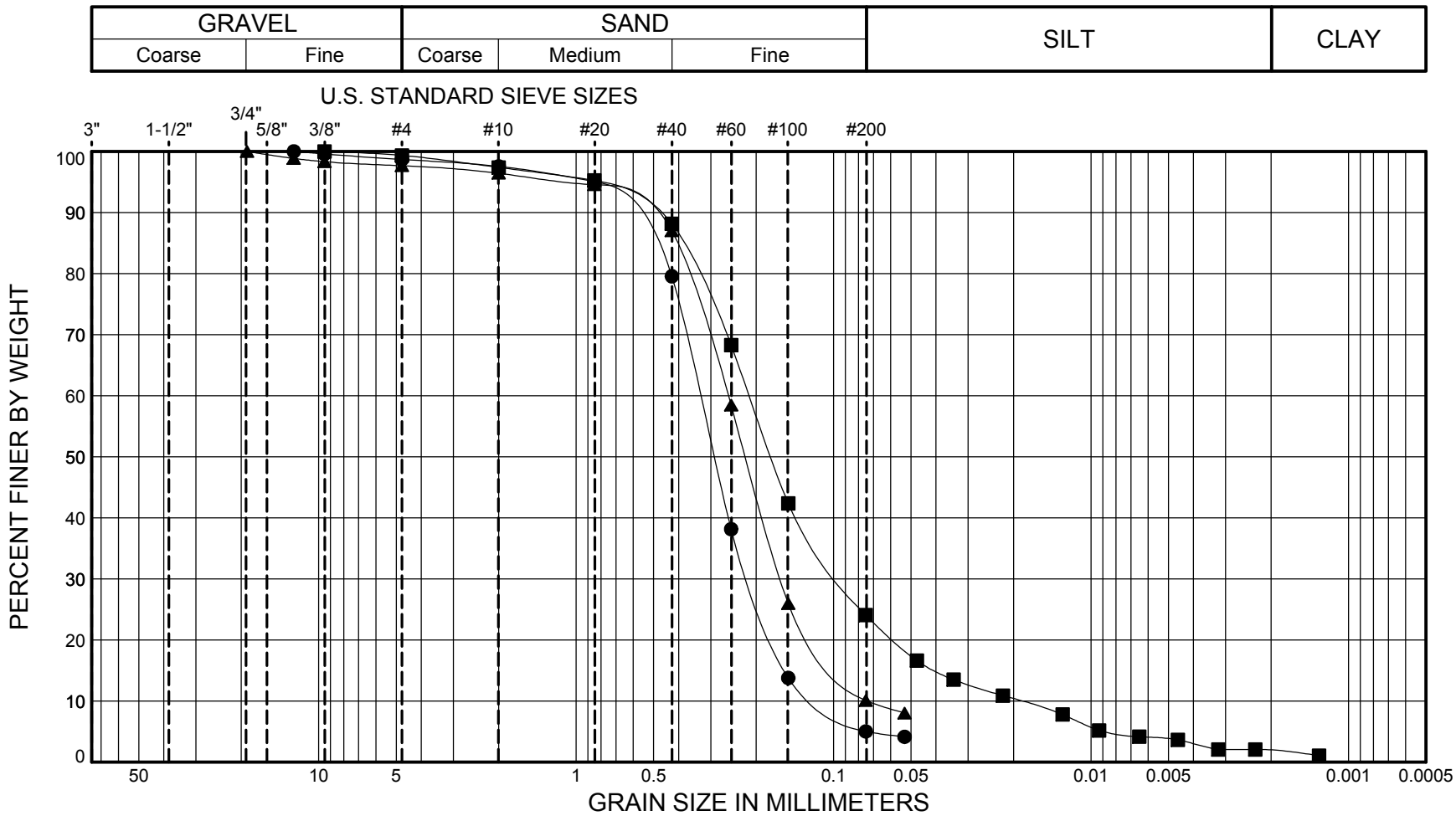
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	TP-1	S-5	8.0 - 8.5	(SM) Olive brown, silty SAND	15				9.4	57.9	32.7
■	TP-2	S-1	2.0 - 2.5	(GM) Brown, silty GRAVEL with sand and organic material	13				44.3	35.3	20.4
▲	TP-3	S-1	0.6 - 1.3	(SW-SM) Brown, well graded SAND with silt and gravel	5				31.6	58.7	9.7

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2010-100 FIGURE: B-4



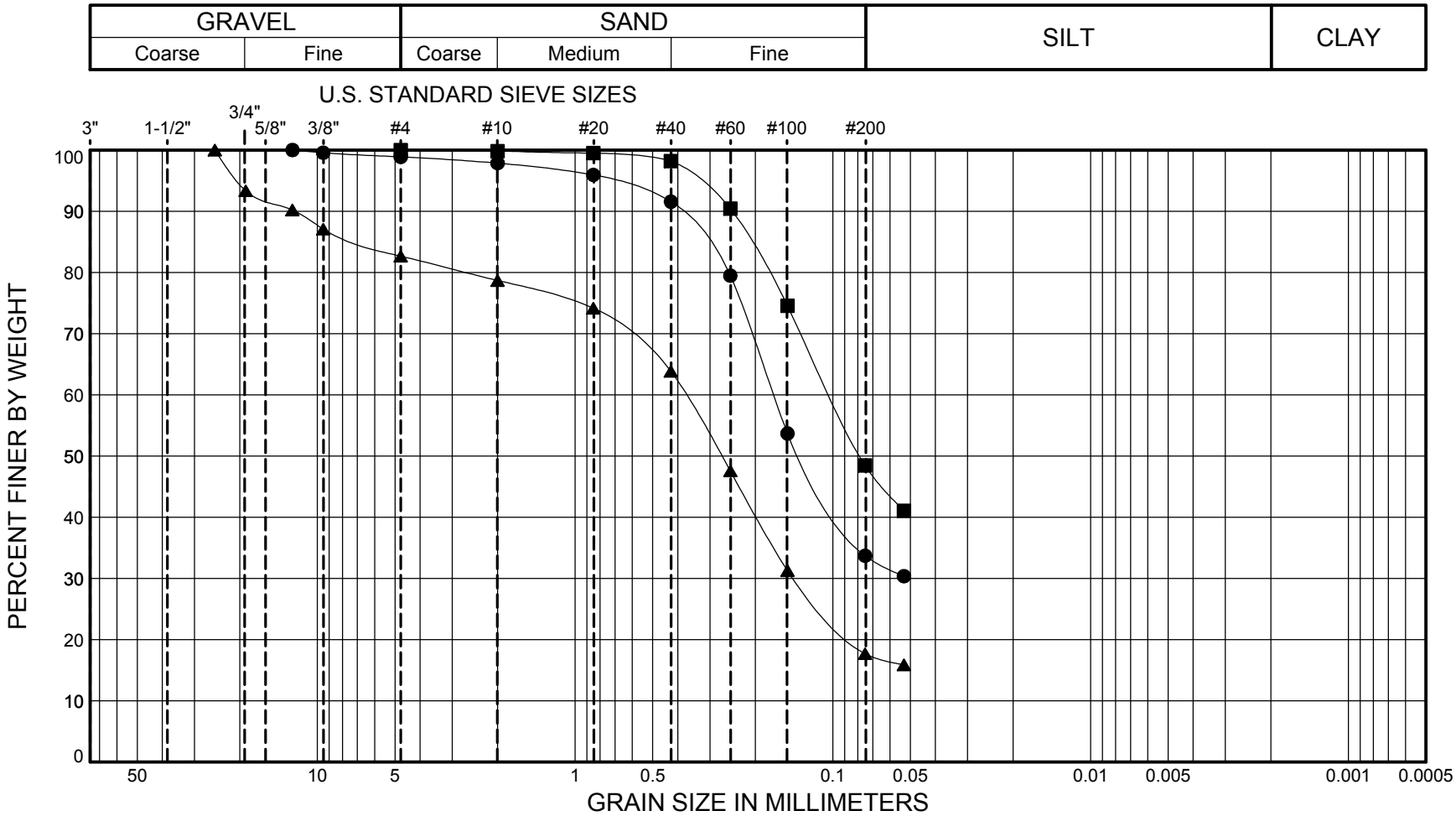
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	TP-3	S-2	2.0 - 2.5	(SP-SM) Dark yellowish brown, poorly graded SAND with silt	8				1.3	93.7	5.0
■	TP-4	S-1	1.5 - 2.0	(SM) Reddish brown, silty SAND	21				0.7	75.3	24.1
▲	TP-4	S-2	2.8 - 3.3	(SP-SM) Dark yellowish brown, poorly graded SAND with silt	20				2.3	87.6	10.1

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2010-100 FIGURE: B-5



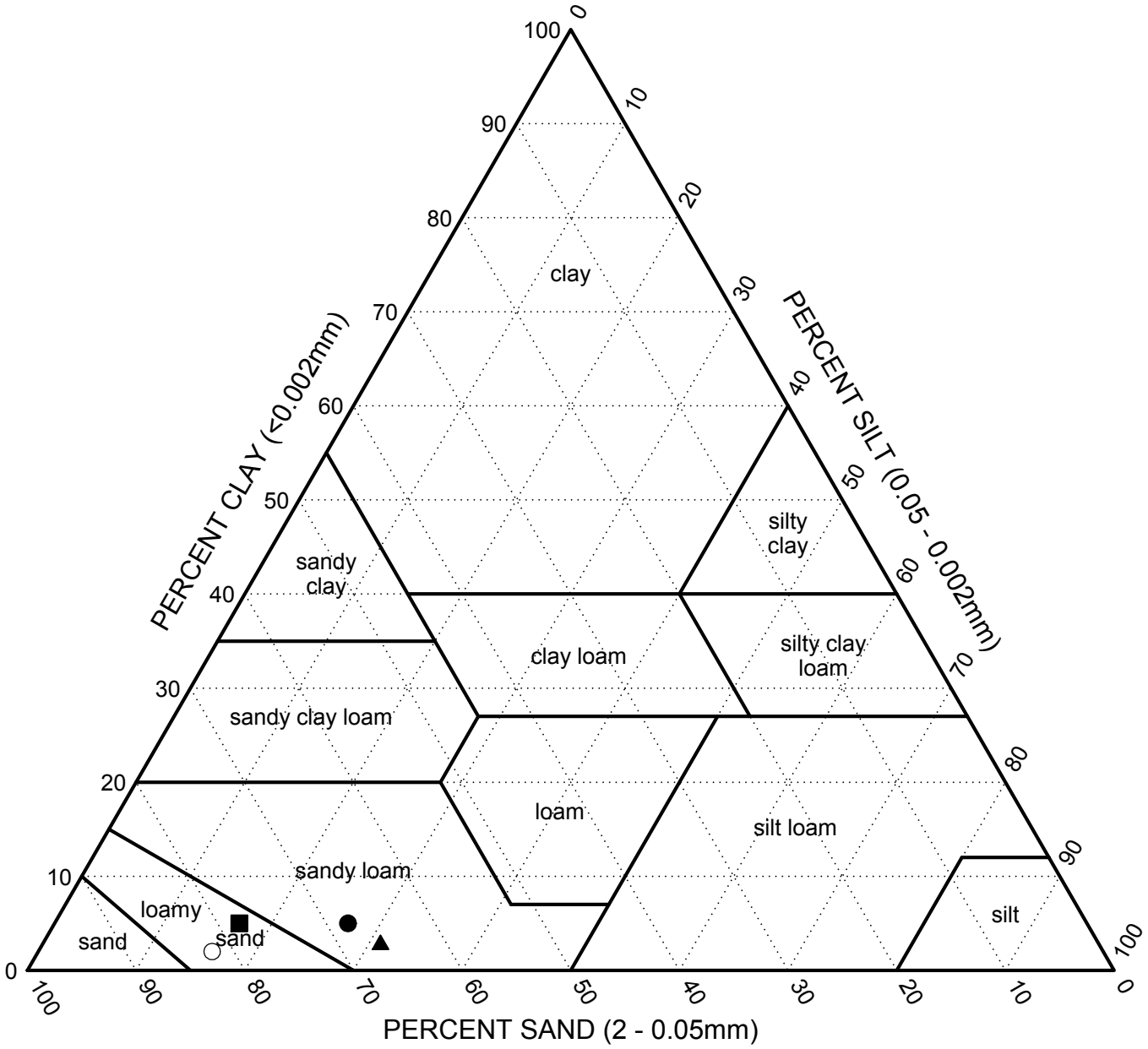
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION OF SOIL- ASTM D2487 Group Symbol and Name	% MC	LL	PL	PI	Gravel %	Sand %	Fines %
●	TP-5	S-1	2.5 - 3.0	(SM) Brown, silty SAND	19				1.1	65.2	33.7
■	TP-5	S-2	4.0 - 4.5	(SM) Olive brown, silty SAND	22				0.0	51.5	48.5
▲	TP-6	S-1	2.5 - 3.0	(SM) Olive brown, silty SAND with gravel	12				17.3	65.0	17.7

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D422

PROJECT NO.: 2010-100 FIGURE: B-6



	Borehole	Depth (ft)	USDA Classification	Sand (%)	Silt (%)	Clay (%)
●	**BH-3**	**1.5**	**SANDY LOAM**	**68.4**	**26.1**	**5.5**
■	**BH-4**	**2.5**	**LOAMY SAND**	**78.1**	**16.6**	**5.3**
▲	**TP-2**	**2.0**	**SANDY LOAM**	**65.8**	**31.5**	**2.8**
○	**TP-4**	**1.5**	**LOAMY SAND**	**82.1**	**16.0**	**1.9**

Fractions normalized to 100% passing the 2mm (#10) sieve

HWA GEOSCIENCES INC.

LAKE TO SOUND TRAIL
WESTSIDE SECTION
KING COUNTY, WASHINGTON

USDA Textural Classification Chart

PROJECT NO.: 2010-100 FIGURE: B-7

ATTACHMENT 1

ARC
STANDARD
ROCK WALL CONSTRUCTION
GUIDELINES

ARC

Associated Rockery Contractors

Standard Rock Wall Construction Guidelines

P.O. Box 1794 • Woodinville, Washington 98072

Association Representatives
(~~206~~ 425) 481-3456 or (206) 481-7222

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- 1.01.2 Goal 1

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Post Construction Guideline Figure A 8

TYPICAL DETAIL Native Cut, Any Height Over 4 Feet 9

TYPICAL DETAIL Overbuild Fill Construction, Rock Wall less than 8 Feet in Height 10

TYPICAL DETAIL Geogrid Reinforced Fill Construction, Rock Wall 8 Feet or More in Height 11

ARC STANDARD ROCKERY CONSTRUCTION GUIDELINES

1.01 Introduction:

1.01.1 Historical Background These standard rock wall construction guidelines have been developed in an effort to provide a more stringent degree of control on materials and construction methodology in the Pacific Northwest. They have been assembled from numerous other standards presently in use in the area, from expertise provided by local geotechnical engineers, and from the wide experience of the members of the Associated Rockery Contractors (ARC).

1.01.2 Goal The primary goals of this document are to standardize the methods of construction for rock walls over four feet in height, and to provide a means of verifying the quality of materials used in construction and the workmanship employed in construction. This standard has also been developed in a manner that makes it, to the best of ARC's knowledge, more stringent than the other standards presently in use by local municipalities.

2.01 Materials:

2.01.1 Rock Quality All rock shall be sound, angular ledge rock that is resistant to weathering. The longest dimension of any individual rock should not exceed three times its shortest dimension. Acceptability of rock will be determined by laboratory tests as hereinafter specified, geologic examination and historical usage records.

All rock delivered to and incorporated in the project shall meet the following minimum specifications:

a. Absorption ASTM C127 AASHTO T-85	*Not more than 2.0% for igneous and metamorphic rock types and 3.0% for sedimentary rock types.*
b. Accelerated Expansion *(15 days)* CRD-C-148 *1, *2	*Not more than 15% breakdown.*
c. Soundness *(MsS04 at 5 cycles)* ASTM C88 CRD-C-137	*Not greater than 5% loss.*
d. Unconfined Compressive Strength ASTM D 2938	*Intact strength of 6,000 psi, or greater.*
e. Bulk Specific Gravity (155pcf) ASTM C127 AASHTO T-85	*Greater than 2.48*

*1. The test sample will be prepared and tested in accordance with Corps of Engineers Testing procedure CRD-C-148, "Method of Testing Stone for Expansive Breakdown on Soaking in Ethylene Glycol.".

*2. Accelerated expansion tests should also include analyses of the fractures and veins found in the rock.

2.01.2 Frequency of Testing Quarry sources shall begin a testing program when either becoming a supplier or when a new area of the source pit is opened. The tests described in Section 2.01.1 shall be performed for every four thousand (4000) tons for the first twelve thousand (12,000) tons of wall rock supplied to establish that specific rock source. The tests shall then be performed once a year, every 40,000 tons, or at an apparent change in material. If problems with a specific area in a pit or with a particular material are encountered, the initial testing cycle shall be restarted.

2.01.3 Rock Density Recognizing that numerous sources of rock exist, and that the nature of rock will vary not only between sources but also within each source, the density of the rock shall be equal to, or greater than, one hundred fifty-five (155) pcf. Typically, rocks used for rock wall construction shall be sized approximately as follows:

Rock Size	Rock Weight	Average Dimension
One man	50-200 pounds	12 to 18 inches
Two man	200-700 pounds	18 to 28 inches
Three man	700-2000 pounds	28 to 36 inches
Four man	2000-4000 pounds	36 to 48 inches
Five Man	4000-6000 pounds	48 to 54 inches
Six Man	6000-8000 pounds	54 to 60 inches

In rock walls eight feet and over in height, it should not be possible to move the large sized rocks (four to six-man size) with a pry bar. If these rocks can be moved, the rock wall should not be considered capable of restraining any significant lateral load. However, it is both practical and even desirable that smaller rocks, particularly those used for "chinking" purposes, can be moved with a pry bar to achieve the "best fit".

2.01.4 Submittals The rock source shall present current geologic and test data for the minimum guidelines described in Section 2.01.1 on request by either the rock wall contractor, the owner, or the applicable agency.

3.01 Rock Wall Construction:

3.01.1 General Rock wall construction is a craft and depends largely on the skill and experience of the builder. A rock wall is a protective system which helps to retard the weathering and erosion process acting on an exposed cut or fill soil face. While by its nature (the mass, size and shape of the rocks) it will provide some undetermined degree of retention, it is not a designed or engineered system in the sense a reinforced concrete retaining wall would be considered designed or engineered. The degree of retention achieved is dependant on the size of rock used; that is, the "mass" or weight, and the height of the rock wall being constructed. The larger the rock, the more competent the rock wall. To accomplish an appropriate

degree of competency, all rock walls in excess of four feet in height should be built on a "mass" basis, i.e. by the ton.

To provide a competent and adequate rock wall structure, all rock walls constructed in front of either cuts or fills eight feet and over in height should be bid and constructed in accordance with these standard guidelines and the geotechnical engineer's supplemental recommendations. Both the standard guidelines and the supplemental geotechnical recommendations should be provided to prospective bidders before bidding and the start of construction.

3.01.2 Geotechnical Engineer The geotechnical engineer retained to provide necessary supplemental rock wall construction guidelines shall be a practicing geotechnical/civil engineer licensed as a professional civil engineer in the State of Washington who has had at least four years of professional employment as a geotechnical engineer in responsible charge, including experience with fill construction and stability and rock wall construction. The geotechnical engineer should be hired either by the rock wall contractor or the owner.

3.01.3 Responsibility The ultimate responsibility for standard rock wall construction should remain with the rock wall builder. However, rock walls protecting moderate to thick fills, with steep sloping surfaces above or below them, with multiple steps, with foundation or other loads affecting them, protecting sandy or gravelly soils subject to ravelling, with seepage or wet conditions, or that are eight feet or more in height, all represent special design conditions and require consultation and/or advice from qualified experts.

3.01.4 Workmanship All workmanship is guaranteed by the rock wall contractor and all materials are guaranteed by the supplying quarry for a period of six years from the date of completion of erection, providing no modification or changes to the conditions existing at the time of completion are made.

3.01.5 Changes to Finished Product Such changes include, but are not necessarily limited to, temporary excavation of ditches or trenches for any utility within a distance of less than five feet from the back of the top of the rock wall; excavation made either within a distance equal to at least two thirds of the free-standing wall height in front of the toe of a rock wall, or that will penetrate an imaginary line extended at a 1H:1V (Horizontal: Vertical) slope from the front edge of the rock wall toe (see Figure A); removal of any material from the subgrade in front of the wall, excavation of material from any location behind the rock wall within a distance at least equal to the rock wall's height, the addition of any surcharge or other loads within a similar distance of the top of the rock wall, or surface or subsurface water forced, directed, or otherwise caused to flow behind the rock wall in any quantity.

3.01.6 Slopes Slopes above rock walls should be kept as flat as possible, but should not exceed 2H:1V unless the rock wall is designed specifically to provide some restraint to the load imposed by the slope. Any slope existing above a completed rock wall should be covered with vegetation by the owner to help reduce the potential for surface water flow induced erosion. It should consist of a deep rooted, rapid growth vegetative mat, will typically be placed by hydroseeding and covered with a mulch. It is often useful to overlay the seed and mulch with either pegged

in-place jute matting, or some other form of approved geotextile, to help maintain the seed in-place until the root mat has an opportunity to germinate and take hold.

3.01.7 Monitoring

All rock walls constructed against cuts or fills eight feet and over in height shall be periodically monitored during construction by the geotechnical engineer to verify that the nature and quality of the materials being used are appropriate, that the construction procedures are appropriate, and that the rock wall is being constructed in a generally professional manner and in accordance with this ARC guideline and any supplemental recommendations.

On completion of the rock wall, the geotechnical engineer should submit to the client, the rock wall contractor, and to the appropriate municipality, copies of his rock wall examination reports along with a final report summarizing rock wall construction.

3.01.8 Fill Compaction

Where rock walls are constructed in front of a fill, it is imperative that the owner ensure the fill be placed and compacted in a manner that will provide a competent fill mass. To achieve this goal, all fills should consist of relatively clean, organic and debris free granular materials with a maximum size of four inches. Ideally, but particularly if placement and compaction is to take place during the wet season, they should contain no more than seven percent fines (silt and clay sized particles) passing the number 200 mesh sieve.

All fills should be placed in thin lifts not exceeding ten (10) inches in loose thickness. Each lift should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM Test Method D-1557-78 (Modified Proctor), before any additional fill is placed and compacted. In-place density tests should be performed at random locations within each lift of the fill to verify that this degree of compaction is being achieved.

3.01.9 Fill Construction Reinforcement

There are two methods of constructing a fill. The first, which typically applies to rock walls of less than eight feet in height, is to overbuild and then cut back the fill. The second, which applies to all rock walls eight feet and over in height, is to construct the fill using a geogrid or geotextile reinforcement.

Overbuilding the fill allows for satisfactory compaction of the fill mass out beyond the location of the fill face to be protected. Overbuilding also allows the earthwork contractor to use larger and more effective compaction equipment in his compactive efforts, thereby typically achieving a more competent fill mass. Cutting back into the well compacted fill also typically results in construction of a competent near vertical fill face against which to build the rock wall.

For the higher rock walls the use of a geogrid or geotextile fabric to help reinforce the fill results in construction of a more stable fill face against which to construct the rock wall. This form of construction leads to a longer lasting and more stable rock wall and helps reduce the risk of significant long term maintenance.

This latter form of construction requires a design by the geotechnical engineer for each specific case. The vertical spacing of the reinforcement, the specific type of reinforcement and the distance to which it must extend back into the fill, the amount of lapping and the construction sequence must be determined on a case by case basis.

3.01.10 Rock Wall Keyway

The first step in rock wall construction, after general excavation, is to construct a keyway in which to build the rock wall. The keyway shall comprise a shallow trench of at least twelve (12) inches in depth, extending for the full length of the rock wall. The keyway subgrade should be slightly inclined back towards the face being protected. It is typically dug as wide as the rock wall (including the width of the rock filter layer). If the condition of the cut face is of concern, the keyway should be constructed in sections of manageable length, that is, of a length that can be constructed in one shift or one day's work.

The competency of the keyway subgrade to support the rock wall shall be verified by probing with a small diameter steel rod. The rod shall have a diameter of between three-eighths and one-half inch, and shall be pushed into the subgrade in a smooth unaided manner under the body weight of the prober only. Penetration of up to six inches, with some difficulty, shall indicate a "competent" keyway subgrade unless other factors in the geotechnical engineer's opinion shall indicate otherwise.

Penetration in excess of six inches, with ease, shall indicate a "soft" subgrade and one that could require treatment. Shallow soft areas of the subgrade can be "firmed up" by tamping a layer of coarse quarry spalls into the subgrade.

3.01.11 Keyway Drainage

Upon completion of keyway excavation, a shallow ditch or trench, approximately twelve (12) inches wide and deep, should be dug along the rear edge of the key way. A minimum four-inch diameter perforated or slotted rigid ADS drain pipe, or equivalent, approved by an engineer, should be placed in this shallow trench and should be bedded on and surrounded by a free-draining crushed rock. Burial of the drain pipe in this shallow trench provides protection to the pipe and helps prevent it from being inadvertently crushed by pieces of the rock wall rock. This drain pipe should be installed with sufficient gradient to initiate flow, and the outfall should be connected to a positive and permanent discharge.

Positive and permanent drainage should be considered to mean an existing or to be installed storm drain system, a swale, ditch or other form of surface water flow collection system, a detention or retention pond, or other stable native site feature or previously installed collection system.

3.01.12 Rock Wall Thickness

The individual rock wall thickness should be equal to the thickness of the recommended size of rock plus the thickness of the drain rock layer. This thickness, which will be determined on a case by case basis, will be dependant on the specific rock sizes recommended for each individual rock wall. For example, if four-man rock is used the rock wall thickness will be approximately five feet.

3.01.13 Rock Selection The contractor should have sufficient space available so that he can select from among a number of stockpiled rocks for each space in the rock wall to be filled. Rocks which have shapes which do not match the spaces offered by the previous course of rock should be placed elsewhere to obtain a better fit. Rock should be of a generally cubical, tabular or rectangular shape and selected in accordance with Section 2.01.3. Any rocks of basically rounded or tetrahedral form should be rejected or used for filling large void spaces.

3.01.14 Rock Placement The first course of rock should be placed on firm unyielding soil. There should be full contact between the rock and soil, which may require shaping of the ground surface or slamming or dropping the rocks into place so that the soil foundation conforms to the rock face bearing on it. The bottom of the first course of rock should be a minimum of twelve (12) inches below the lowest adjacent site grade.

As the rock wall is constructed, the rocks should be placed so that there are no continuous joint planes in either the vertical or lateral direction. Wherever possible, each rock should bear on at least two rocks below it. Rocks should be placed so that there is some bearing between flat rock faces rather than on joints. Joints between courses (the top surface of rock), should slope back towards the cut face and away from the face of the rock wall.

Smaller rocks (one to two-man size) are often used to create an aesthetically pleasing "top edge" to a rock wall. This is an acceptable practice provided none of the events described in Section 3.01.5 occur, and that people are prevented from climbing or walking on the finished wall. ***This is the owner's responsibility.***

3.01.15 Face Inclination The face of the rock wall should be inclined at a gradient of about 1H:6V back towards the face being protected. The inclination should not be constructed flatter than 1H:4V.

3.01.16 Voids Because of the nature of the product used to construct a rock wall, it is virtually impossible to avoid creating void spaces between individual rocks. However, it should be recognized that voids do not necessarily constitute a problem in rock wall construction. As the size of rock used to build a rock wall increases, i.e. to six-man size, the void spaces between individual rocks should be expected to be larger.

Where voids of greater than six inches in dimension exist in the face of a rock wall they should be visually examined to determine if contact between the rocks exists within the thickness of the rock wall. If contact does exist, no further action is required. However, if there is no rock contact within the rock wall thickness the void should be "chinked" with a smaller piece of rock.

3.01.17 Drain Rock Layer In order to provide some degree of drainage control behind the rock wall, and as a means of helping to prevent loss of soil through the face of the rock wall, a rock drainage filter shall be installed between the rear face of the rock wall and the soil face being protected. This drain rock layer should be at least twelve (12) inches thick; and for rock walls eight feet in height or higher, it should be at

least eighteen (18) inches thick. It should be composed of 4 to 2-inch sized crushed rock quarry spalls, crushed concrete, or other material approved by the geotechnical engineer. If a random wall rock extends back to the exposed soil face, it is not necessary that the filter rock layer extend between it and the soil face.

Depending on soil type and potential water seepage, a geotextile fabric may or may not be required. This can be determined on a case by case basis by the geotechnical engineer during design and prior to bidding.

3.01.18 Surface Drainage It is the owner's responsibility to intercept surface drainage from above the rock wall and direct it away from the rock wall to a positive and permanent discharge well below and beyond the toe of the rock wall. Use of other drainage control measures should be determined on a case-by-case basis by the geotechnical engineer prior to bidding on the project.

Post Construction Guidline

Schematic Only - Not to Scale

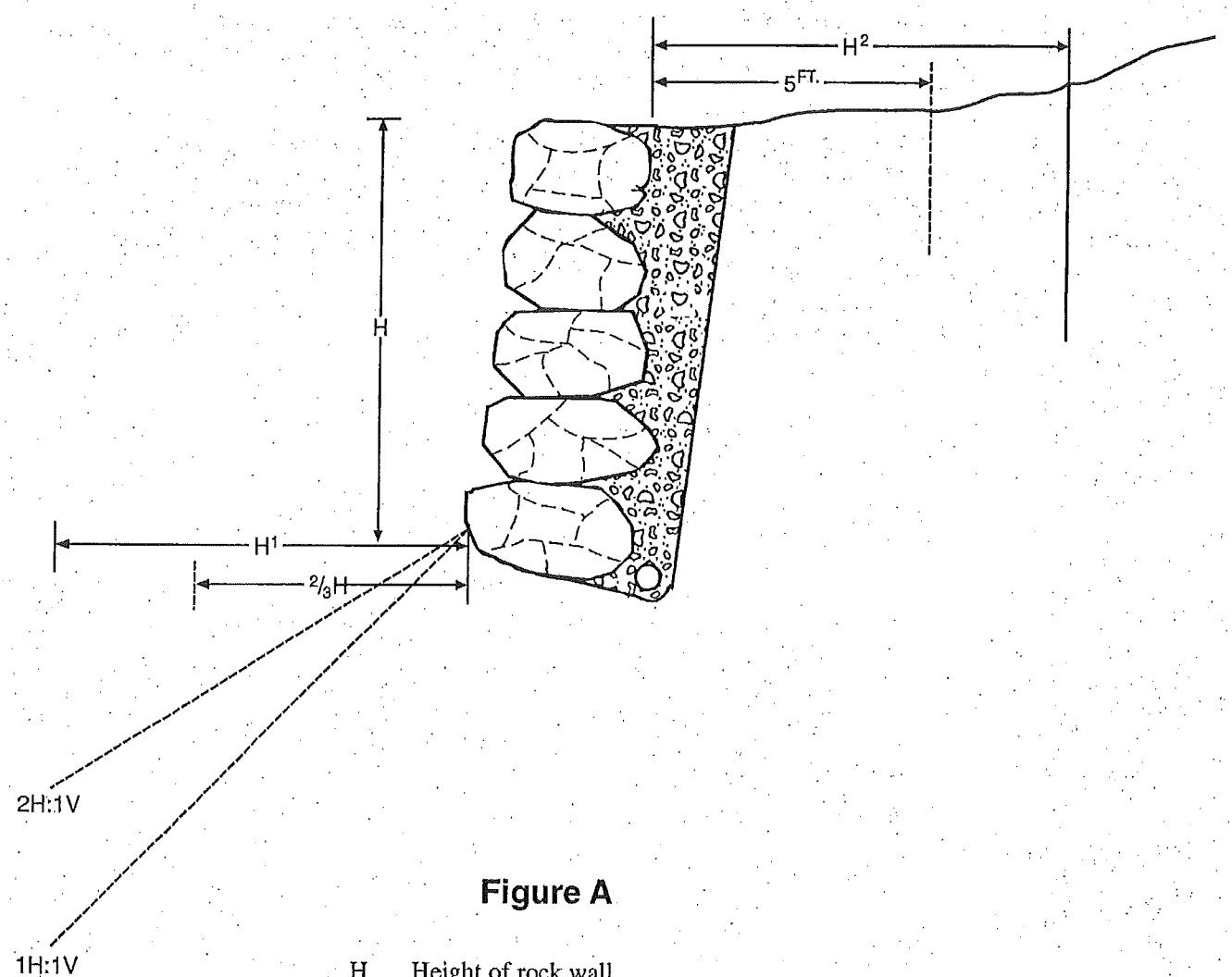
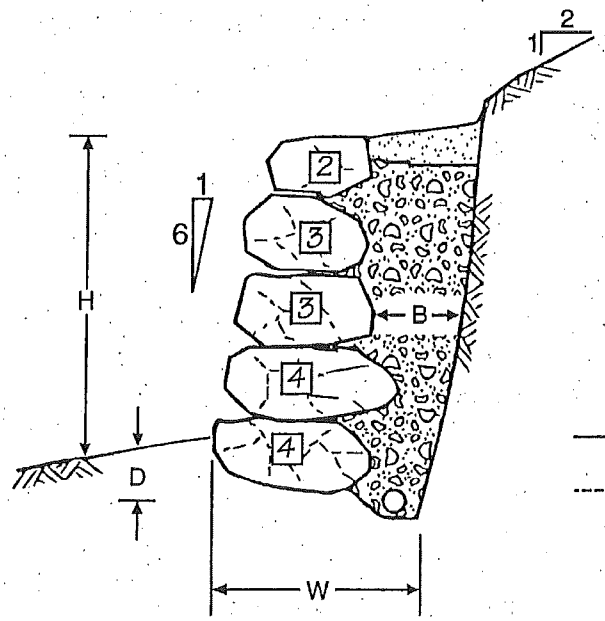


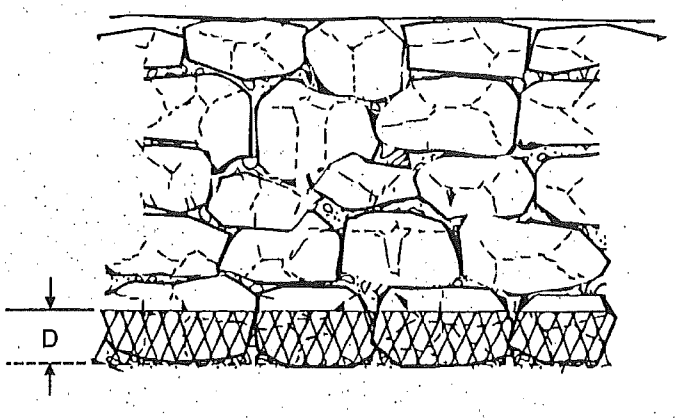
Figure A

H	Height of rock wall.
5FT	No temporary excavation or trenching.
2/3 H	No temporary excavation or trenching.
H^1	No permanent removal of subgrade allowed.
H^2	No excavation of material or any surcharge allowed.
1H:1V	No excavation below allowed.
2H:1V	Maximum finished grade or permanent excavation .

Schematic Only - Not to Scale



Rock Wall Section



Rock Wall Elevation

NOTES:

- Rock wall construction is a craft and depends largely on the skill and experience of the builder.
- A rock wall is a protective system which helps retard the weathering and erosion process on an exposed soil face.
- While by its nature (mass, size and shape of the rocks) it will provide some degree of retention, it is not a designed or engineered system in the sense a reinforced concrete retaining wall would be considered designed or engineered.
- The degree of retention achieved is dependent on the size of the rock used; that is, the mass or weight, and the height of the wall being constructed. The larger the rock, the more competent the rock wall should be.
- Rock walls should be considered maintenance items that will require periodic inspection and repair. They should be located so that they can be reached by a contractor if repairs become necessary.
- Maximum inclination of the slopes above and behind rock walls should be 2:1 (Horizontal:Vertical).
- Minimum thickness of rock filter layer B = 12 inches, Minimum embedment D = 12 inches undisturbed native soil or compacted fill placed in accordance with report recommendations.
- Maximum rock wall height H = _____ feet.
- Rock walls greater than 8 feet in height to be installed under periodic or full time observation of the geotechnical engineer.
- Rock should be placed to gradually decrease in size with increasing wall height in accordance with geotechnical engineers recommendations.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

- The long dimension of the rocks should extend back towards the cut or fill face to provide maximum stability. Rocks should not be stacked like shoe boxes. They should be placed to avoid continuous joint planes in vertical or lateral directions. Whenever possible each rock should bear on two or more rocks below it, with good flat-to-flat contact.
- All rock walls over 4 feet in height should be constructed on basis of wall mass, not square footage of face.

Size	Approximate Weight - lbs.	Approximate Diameter
1 Man	50 - 200	12 - 18"
2 Man	200 - 700	18 - 28"
3 Man	700 - 2000	28 - 36"
4 Man	2000 - 4000	36 - 48"
5 Man	4000 - 6000	48 - 54"
6 Man	6000 - 8000	54 - 60"

Reference: Local quarry weight study using average weights of no less than six rocks of each man size conducted in January 1, 1988.

LEGEND:

Drainage materials to consist of clean angular 4 to 2 inch spalls, or other material , approved by the geotechnical engineer

Surface seal: may consist of impervious soil or a fine free draining granular material.

Undisturbed firm Native soil

Drainpipe: 4-inch minimum diameter, perforated or slotted rigid plastic ADS pipe laid with a positive gradient to discharge under control well away from the wall.

4 Designates size of rock required, i.e. 4 man.

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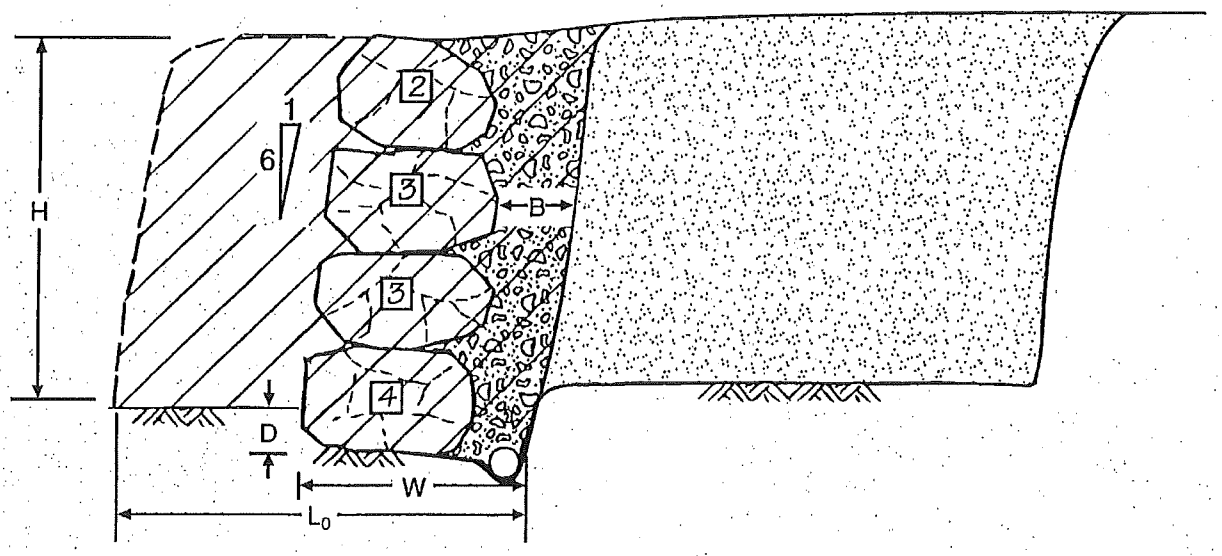
Geotechnical Engineers, Geologists & Environmental Scientists

TYPICAL DETAIL
NATIVE CUT, ANY HEIGHT OVER 4 FEET

Plate
A

Schematic Only - Not to Scale



LEGEND

Crushed rock or other apporved material ranging between 4 to 2 inches in size and free of organics, with less than 5 percent fines (silt and clay size particles passing the No. 200 mesh sieve).

Structural fill overbuild, compacted to at least 95% of maximum dry density as determined by ASTM Test Method D-1558-78 (Modified Proctor).

Compacted structural fill consisting of free-draining, organic-free material with a maximum size of 4 inches. Should contain no more than 7 percent fines (described above), compacted to at least 95 percent of ASTM D-1557-78 maximum dry density.

Undisturbed firm Native soil

Perforated or slotted drain pipe with 4 inch minimum diameter bedded on and surrounded by crushed rock filter material, described above.

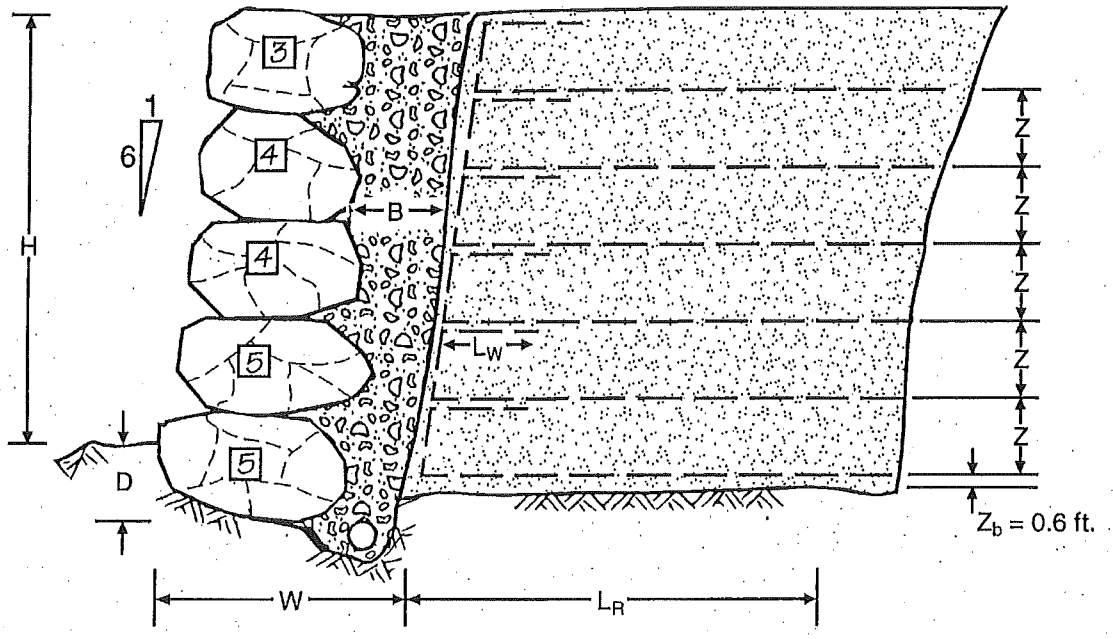
4 Designates size of rock required, i.e. 4 man.

NOTES

- All fill should be placed in thin lifts not exceeding 10 inches in loose thickness. Each layer should be compacted to no less than 95 percent of maximum dry density, as determined by ASTM D-1557-78 (Modified Proctor).
- Thickness of crushed filter rock layer, B, should be no less than 12 inches.
- Depth of burial of basal layer of rock, D, should be no less than 12 inches.
- Height of rock wall, H, should not exceed _____ feet.
- Lateral extent of fill overbuild, L_0 should be no less than H feet.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

Creative Engineering Options INC. CEO Geotechnical Engineers, Geologists & Environmental Scientists	TYPICAL DETAIL OVERBUILD FILL CONSTRUCTION ROCK WALL LESS THAN 8 FEET IN HEIGHT	Plate B

Schematic Only - Not to Scale



LEGEND

Crushed rock or crushed concrete drain rock material ranging between 4 and 2 inches in size and free of organics, with less than 5 percent fines (silt and clay size particles passing the No. 200 mesh sieve).

Compacted structural fill consisting of free-draining, organic-free material with a maximum size of 4 inches. Should contain no more than 7 percent fines (described above), compacted to a least 95 percent of ASTM D-1557-78 maximum dry density.

Undisturbed firm Native soil

Geogrid reinforcement approved by geotechnical engineer.

Perforated or slotted drain pipe with 4 inch minimum diameter bedded on and surrounded by crushed rock filter material, described above.

5 Designates size of rock required, i.e. 5 man.

NOTES

- All fill should be placed in thin lifts not exceeding 10 inches in loose thickness. Each layer should be compacted to no less than 95 percent of maximum dry density, as determined by ASTM D-1557-78 (Modified Proctor).
- Minimum length of geogrid wrap over top of fill, L_W, should be no less than 3 feet.
- Length of reinforcing geogrid, L_R shall be _____ feet.
- Geogrid reinforcement layer spacing Z shall be _____ feet as determined by the geotechnical engineer's design.
- Height of rock wall, H, should not exceed _____ feet.
- Thickness of crushed drain rock layer, B should be no less than 18 inches.
- Depth of burial of basal layer of rock, D, should be no less than 12 inches.
- Minimum width of keyway excavation, W, should be equal to the thickness of the basal rock (as determined by geotechnical engineer's design) plus B.

Creative Engineering Options INC. CEO Geotechnical Engineers, Geologists & Environmental Scientists	TYPICAL DETAIL GEOGRID REINFORCED FILL CONSTRUCTION ROCK WALL 8 FEET OR MORE IN HEIGHT	Plate C