FINAL GEOTECHNICAL REPORT
Pom Pom Road Improvements
Yakama Nation, Washington

HWA Project No. 2021-097-21

Prepared for
KPFF Consulting Engineers

June 26, 2025

HWA GEOSCIENCES INC.
DBE/MWBE

Geotechnical Engineering
Pavement Engineering
Geoenvironmental
Hydrogeology
Inspection & Testing

June 26, 2025
HWA Project No. 2021-097-21

KPFF Consulting Engineers
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Seattle, Washington 98101

Attention: Anne Fabrello-Streufert, P.E., S.E

Subject: **FINAL GEOTECHNICAL ENGINEERING REPORT**
Pom Pom Road Improvements
Yakama Nation, Washington

Dear Anne,

We are pleased to present this final geotechnical engineering report prepared in support of the Pom Pom Road Improvements project, at Toppenish Creek, within the Yakama Nations Reservation. The purpose of this study was to evaluate the soil and groundwater conditions along the project alignment and to provide geotechnical recommendations in support of the proposed improvements.

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.

Joe Westergreen

Donald J. Huling

Joe Westergreen, P.E.
Geotechnical Engineer

Donald Huling, P.E.
Geotechnical Engineer, Principal

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FINAL GEOTECHNICAL ENGINEERING REPORT
POM POM ROAD IMPROVEMENTS
YAKAMA NATION, WASHINGTON

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical engineering study performed by HWA GeoSciences Inc. (HWA) in support of the Pom Pom Road Improvements project, at Toppenish Creek, within Yakama Nations. The purpose of this study was to evaluate the soil and groundwater conditions along the alignment to aid in development of project improvements along the corridor. The approximate location of the project site is shown on the Vicinity Map, Figure 1, and on the Site and Exploration Plan, Figures 2A through 2C.

Our work for this project included performing a site reconnaissance, preparing and conducting a site investigation program, performing geotechnical engineering analyses, and providing recommendations for geotechnical aspects of design. Field work included drilling three machine-drilled borings in support of the three proposed culverts along the alignment, and two machine-drilled borings in support of the proposed bridge foundations.

Appropriate laboratory tests were conducted on selected soil samples from our explorations to determine relevant engineering properties of the subsurface soils. In this report, we present a summary of the subsurface and groundwater conditions observed, as well as geotechnical recommendations for the proposed improvements.

1.2 PROJECT UNDERSTANDING

It is our understanding that Yakama Nations would like to implement improvements to a portion of Pom Pom Road. Proposed improvements include installing three new culverts and constructing a new bridge to support a restoration of the Toppenish Creek watershed. The project area starts approximately 1,200 feet south of the intersection of Pom Pom Road and Marion Drain Road and extends about 2,000 feet to the south to an existing pre-cast concrete bridge that carries Pom Pom Road over the current main channel of Toppenish Creek. The new bridge will be within the northern portion of the project alignment. We understand that the proposed improvement will require raising road grades. We understand that road grades near the proposed culverts will increase by up to 4.5 feet, and that roadway grades for the bridge will require raising grades by up to about 9 feet.

There is an existing water line that extends down the west side of the road. As part of the project improvements, we understand that the existing water line will be relocated below the proposed culverts and will be attached to the new bridge structure.

2.0 FIELD AND LABORATORY TESTING

2.1 GEOTECHNICAL FIELD INVESTIGATION

Our geotechnical exploration program included surface reconnaissance of the alignment and drilling five machine-drilled borings, designated HWA-1 through HWA-5, to depths ranging from 31.5 to 51.5 feet below ground surface (bgs). Drilling was performed on January 4th through 7th, 2023 by Holt Services of Edgewater, Washington, under subcontract to HWA using a track-mounted Sonic drill rig. The approximate locations of these borings are shown on the Site and Exploration plans, Figures 2A through 2C. Logs for each boring are presented in Appendix A of this report.

Standard Penetration Testing (SPT) was performed in each boring using a 2-inch outside diameter, split-spoon sampler driven by a 140-pound automatic hammer. During the test, a sample was obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows required for each 6 inches of sampler penetration was recorded. The N-value (or resistance in terms of blows per foot) is defined as the number of blows recorded to drive the sampler the final 12 inches. If a total of 50 blows was recorded within a single 6-inch interval, the test was terminated, and the blow count was recorded as 50 blows for the number of inches of penetration achieved. This resistance, or N-value, provides an indication of the relative density of granular soils and the relative consistency of cohesive soils.

Additionally, two relatively undisturbed samples were obtained in Shelby tubes from HWA-1 and HWA-4. Sampling with a Shelby tube consists of pushing a 3-inch O.D., thin-walled steel tube (bolted to the bottom of the sampling rods) 30 inches into undisturbed soil below the bottom of the borehole using drill rig hydraulics. The sample tube is allowed to equilibrate for a few minutes before retrieval to promote adequate recovery.

The explorations were completed under the full-time observation of a geotechnical engineer from HWA, who collected pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence as the explorations were advanced. Soils were classified in general accordance with the classification system described on Figure A-1, which also provides a key to the exploration log symbols. The exploration logs are presented on Figures A-2 through A-6.

The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types. Actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

2.2 Laboratory Testing

Laboratory tests were conducted on selected samples retrieved from the explorations to characterize relevant engineering properties and index parameters of the soils encountered at the site. The tests included visual classification, natural moisture content determination, grain size distribution analysis, Atterberg Limits, and consolidation testing. The tests were conducted in the HWA laboratory in general accordance with appropriate American Society of Testing and Materials (ASTM) standards and are discussed in further detail in Appendix B. The test results are also presented in Appendix B, and/or displayed on the exploration logs in Appendix A, as appropriate.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project area extends about 2,000 feet along Pom Pom Road from the current Toppenish Creek bridge to about 1,200 feet south of the intersection of Pom Pom Road and Marion Drain Road. Within the project area Pom Pom Road consists of a two-lane gravel road and generally slopes gently down to the north with surface elevations ranging from about 1,018 to 1,032 feet along the alignment. Overhead power lines are present along the eastern side of the roadway.

3.2 General Geologic Conditions

The project is located within the Yakima Valley. The Yakima Valley has repeatedly been inundated by outburst floods associated with the most recent Quaternary glacial period. Flooding was the result of glacial ice sheets damming the Clark Fork River in northern Idaho, resulting in the formation of Glacial Lake Missoula. Periodically, the ice sheet dam would breach, releasing flood waters that would travel through eastern Washington to the Columbia River. Flood waters transported sediments ranging from clay to coarse gravel, with the capacity to transport boulders as well.

General geologic information for the site was obtained from the publication *Geologic Maps of Part of the Yakima Fold Belt, Northeastern Yakima County, Washington* (Bentley et al., 1993). The map indicates that the surficial geology in the project area generally consists of Catastrophic Flood Slackwater Sediments (Qfs), and Quaternary alluvium (Qas). In general, Qfs is described as bedded silt with varying percentages of sand and gravel, and was deposited by outburst flood waters in a low energy region of the flood zone. In general, Qas is described as silt, sand and gravel transported by Yakima River tributaries, such as Toppenish Creek, after the most recent glacial period. Qas is likely overlying Qfs in the project area.

3.3 Subsurface Conditions

3.3.1 Soils

In general soils encountered during our investigation are consistent with those identified on the geologic map. 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-6) provide more detail of subsurface conditions observed at specific locations and depths. The soils encountered in the explorations are described as follows:

- **Fill** – Fill was encountered in all borings extending to depths between about 4 and 6 feet bgs. The fill material generally consisted of medium dense, slightly silty to silty gravel with variable amounts of sand, and medium dense sandy gravel with variable amounts of silt. The gravel varied from subrounded to angular.
- **Alluvium** – Fine- and coarse-grained alluvium was encountered underlying the fill and extending to depths ranging from approximately 11 to 24 feet. The fine-grained alluvium was encountered in all borings and generally consisted of soft to medium stiff silty clay with variable amounts of sand and sandy silt. In HWA-1, and HWA-3 through HWA-5, there was about 1 to 6 feet of coarse-grained alluvium below the fill and above the fine-grained material that generally consisted of loose silty sand with variable amounts of gravel.
- **Outburst Flood Deposits**– Outburst flood deposits were encountered in all borings, underlying the alluvium and extending to the termination depths of the borings. The outburst flood deposits generally consisted of loose to very dense, clean to silty gravel with variable amounts of sand and cobbles. A medium dense silty sand bed was encountered in HWA-1 from 42 to 43.5 feet, and a stiff clay bed was encountered in HWA-5 from 40 to 41.3 feet. This unit represents Outburst Flood soils deposited during a period of very high transport energy. The abundant gravels and cobbles can affect blow counts, and densities could be falsely elevated.

3.3.2 Groundwater

During our explorations, groundwater was encountered at depths between about 10 to 13.5 feet below the existing roadway surface while drilling. Groundwater monitoring wells were installed in borings HWA-1 and HWA-5 to monitor fluctuations in groundwater levels. Pressure transducers were installed to monitor groundwater fluctuation in the wells. Since drilling, groundwater depths in the monitoring well in HWA-1 have ranged from about 13 to 16 feet bgs. A plot of groundwater depths from January 13, 2023, through November 13, 2023, obtained from the pressure transducer is included on Figure 3A.

During our last visit to collect the groundwater data in November 2023, we were unable to locate the monitoring well that we installed in boring HWA-5, during drilling in January 2023. The only sign of the well that we were able to locate was a piece of concrete in the ditch that looked like it was the concrete that the steel flush monument had been set with. It appears that the well monument likely got destroyed by snowplows or road maintenance equipment sometime after our site visit in February 2023. Therefore, we only have groundwater depths from HWA-5 from January 13, 2023, through February 21, 2023. Plots of groundwater depths from the pressure transducer for HWA-5 are included on Figure 3B. Based on the groundwater variations observed in HWA-1, we anticipate a seasonal high groundwater level of about 5 feet below the existing roadway grade at HWA-5.

Variations in groundwater conditions should be expected to occur seasonally and with changes in precipitation. In addition, perched water may be encountered at shallow depths during periods of wet weather. We anticipate that water could perch within the loose silty sand above the less permeable silt and clay.

Excavations below the groundwater table will require dewatering. If excavations extend into the underlying gravelly outburst flood deposits high flow rates should be expected due to the high permeability and large particle size of the gravel and cobbles. We recommend the contractor retain a dewatering expert to design required dewatering systems, as the contractor is responsible for construction dewatering.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

The subsurface soils along the alignment consist of existing roadway fill, over alluvium, over outburst flood deposits. The existing alluvium generally consists of loose silty sand and soft to medium stiff silty clay or sandy silt. The alluvium will not provide adequate foundation for the proposed bridge. We recommend that the bridge be supported on deep foundations that extend in the gravelly outburst flood deposits. Based on the large cobbles, we recommend that bridge foundations consist of drilled shafts constructed using the oscillator method. We recommend that drilled shafts extend into the dense to very dense gravel starting at about 40 feet bgs. Groundwater has been measured at depths between about 6.5 to 7.5 feet below the roadway surface at the time of drilling in the monitoring well installed in HWA-5 near the proposed bridge structure, as such, wet drilling conditions should be anticipated. The outburst flood deposits underlying the site contain significant amounts of cobbles and occasional boulders, difficult drilling conditions should be anticipated.

Based on the 80 percent design plans, we understand that the proposed 4-sided concrete box culverts will be constructed approximately 8 to 9.5 feet below existing grades. Based on our subsurface explorations we anticipate loose very silty sand will be present at the subgrade elevation for the southern culvert, and that soft silty clay will present at the subgrade elevation for the other two culverts. To support the culverts subgrade stabilization will be required.

We understand the bridge crossing will require raising grades up to about 9 feet, and that at the proposed culverts grades will be increased by up to 4.5 feet. The fine-grained alluvium underlying the project alignment is compressible and will be subject to consolidation settlements with the application of additional loads. These soft/loose soils will require subgrade improvement to support construction of the culverts.

We understand that bridge and culvert wing walls will consist of welded wire face Structural Earth Walls (SEWs). Based on the 80 percent design plans, the base of the walls are generally anticipated to be founded in the alluvium consisting of loose very silty sand and soft silty clay. These soft/loose soils will require subgrade improvement to support the SEWs.

4.2 SEISMIC DESIGN CONSIDERATIONS

4.2.1 Design Parameters

Earthquake loading for the project was developed in accordance with the General Procedure provided in Section 3.10 of the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, 9th Edition, 2020. For seismic analysis, the Site Class is required to be established and is determined based on the average soil properties in the upper 100 feet below the ground surface. Based on our subsurface explorations and understanding of site geology, it is our opinion that the site is underlain by soils that are consistent with Site Class D.

The mapped seismic design coefficients for the design level event, which has a probability of exceedance of 7 percent in 75 years (equal to an approximate return period of approximately 1033 years), were obtained using the USGS Unified Hazard Tool to incorporate the probabilistic seismic hazard parameters from the 2014 updates to the National Hazard Maps (Peterson, et al., 2014). The recommended seismic coefficients for the design event are provided in Table 1. Site Coefficients were obtained from Tables 3.10.3.2-1 through 3.10.3.2-3 (AASHTO, 2020). The spectral acceleration coefficient at 1-second period (SD_1) is between 0.3 and 0.5 at the project location; therefore, Seismic Performance Zone 3, as given by AASHTO Table 3.10.6-1 (AASHTO, 2020), should be used.

Table 1.
Seismic Coefficients Using AASHTO Guide Specifications
Calculated by USGS Seismic Uniform Hazard Tool
Location: Lat. 46.342612; Long. -120.711592

Site Class	Peak Horizontal Bedrock Acceleration PBA, (g)	Spectral Bedrock Acceleration at 0.2 sec S_S, (g)	Spectral Bedrock Acceleration at 1.0 sec S_1, (g)	Site Coefficients			Peak Horizontal Acceleration PGA (A_s), (g)
				F_{pga}	F_a	F_v	
D	0.1467	0.324	0.1039	1.507	1.548	2.392	0.221

4.2.2 Liquefaction

Liquefaction is a temporary loss of soil shear strength due to earthquake shaking. Loose, saturated cohesionless soils are highly susceptible to earthquake-induced liquefaction. However, research has shown that certain silts and low-plasticity clays are also susceptible. Primary factors controlling the development of liquefaction include the intensity and duration of strong ground motions, the characteristics of subsurface soils, in-situ stress conditions and the depth to groundwater.

Based on our explorations the soil below the groundwater table generally consists of silty clay or large gravels and cobbles in a clay matrix. Soil liquefaction is not anticipated during a seismic event.

4.3 STATIC SETTLEMENT CONSIDERATIONS

We understand that grades will be increased along the project alignment to incorporate the project improvements. Based on correspondence with KPFF, we understand that between about 1.5 to 4 feet of fill is anticipated along the roadway within the vicinity of the proposed culverts, and about 8.5 to 9 feet of fill is anticipated near the proposed bridge abutments.

The loads associated with these grade increases are expected to induce consolidation settlement of the underlying compressible soils. As installation of the culverts will result in removal of material, we anticipate that the culverts will settle slightly less than the adjacent roadway surface. Consolidation settlement will also induce down drag loads on the drilled shafts with the settlement of the abutments and embankment fill. We recommend that the bridge include approach slabs to account for the settlement. Further discussion of anticipated settlements is provided below.

4.3.1 Anticipated Consolidation Settlement Magnitude

Settlement analysis was performed to evaluate the magnitude of settlement due to proposed grade increases in the project area. The weight of the fill soils and the thickness of the soft compressible fine-grained alluvium will have the largest impacts on the estimated static consolidation settlement. In our explorations near the proposed improvements, we observed between approximately 5 to 16 feet of compressible fine-grained material.

The computer program SETTLE 3D was used to evaluate the expected settlement magnitudes along the roadway near the proposed structures. In our analysis we assumed conventional structural fill will be used with an approximate unit weight of 135 pounds per cubic foot (pcf). Table 2 presents our settlement estimates of the roadway surface near each proposed structure. At the proposed culverts we anticipate that the settlement will be about a ½ inch less than the adjacent roadway surface, due to the weight reduction from the culvert opening.

Table 2. Consolidation Settlement Estimates

Boring	Location	Structure Type	Proposed Fill Height (Feet)	Thickness Compressible Material (Feet)	Anticipated Settlement (Inches)
HWA-1	STA 4+25	Culvert	1.5	12	1-1.5
HWA-2	STA 6+00	Culvert	3	16	2-2.5
HWA-3	STA 8+12	Culvert	4	13	1.5-2
HWA-4	STA 15+25	South Abutment	8.5	9	3-3.5
HWA-5	STA 16+75	North Abutment	9	5	2.5-3

We expect that the underlying compressible soils would take between 3 to 6 months to complete primary consolidation settlement. After the primary consolidation settlement has occurred, the fine-grained soils will continue to experience settlement due to ongoing secondary consolidation, these settlements are not included in Table 2. Secondary consolidation can continue for many years after the application of load and can even occur in areas where no loads were applied. Based on our subsurface explorations we anticipate the magnitude of secondary settlement will be on the order of 10% of the primary consolidation values presented in Table 2.

4.4 Culvert Design

4.4.1 General

We understand the project includes installing three culverts as part of the flood plain improvements. Based on the 80 percent design plans, the culverts will consist of 4-sided concrete box culverts with associated concrete wing walls and head walls. We understand the culverts will be about 21 feet wide, about 8.5 to 12 feet tall, and about 40 feet in length.

The bottom of the culverts will be about 8 to 9.5 feet below existing roadway grades. Based on our explorations the base of the culvert excavations is anticipated to encounter loose very silty sand or soft silty clay. These soils will provide poor subgrade support and will require improvement. Subgrade improvement is anticipated to consist of over-excavation and replacement with stabilization material. In addition, the base of the culvert excavations after the necessary over-excavation is anticipated to be near the groundwater table. Wet excavation conditions should be anticipated, and temporary dewatering will be required for subgrade preparation and culvert installation. Temporary excavations should be completed in accordance with Section 5.3.

4.4.2 Culvert Structure Elevations and Settlement

Construction of the proposed culverts and associated grade increases is expected to result in consolidation settlement of the underlaying compressible soils. This consolidation settlement is expected to occur after the culvert structures are installed. Therefore, settlement of the Culvert structures is expected. Based on the proposed fill heights of 1.5 to 4 feet we anticipate total settlement of up to 2 inches with differential settlement of up to 1 inch for the proposed culvert structures.

If it is critical to maintain a specific head space, within the culvert structures, we recommend designing the proposed culverts to possess a minimum of 2-inches of increased head space. This will allow the structures to possess adequate head space upon completion of expected settlements.

4.4.3 Culvert Foundation Preparation

The loose and soft alluvial soils, at the anticipated base of the culverts, will not provide an adequate working surface to place the 4-sided culvert structures. To create a stable subgrade to support the 4-sided concrete box culverts we recommend a minimum three-foot over-excavation below the base of the culverts. The over-excavation should extend at least 3 feet laterally beyond the edge of the structure's foundation. To stabilize the over-excavated subgrade, we recommend tamping in about 2 feet of quarry spalls as defined in Section 9.13.1(5) of the WSDOT Standard Specifications (WSDOT, 2023). Stabilization material should be tamped into the subgrade with an excavator bucket until a firm and well keyed condition is established.

A geotextile fabric should be placed over the compacted stabilization material. Geotextiles for Separation should meet the requirements of Table 3 in Section 9-33.2(1) of the WSDOT *Standard Specifications* (WSDOT, 2023). Then the remainder of the remainder of the three-foot section of over-excavated area should be backfilled with crushed surfacing base course (CSBC) and compacted over the geotextile. The CSBC should be at least 12 inches thick and should be compacted to at least 95 percent of the maximum dry density determined using ASTM D 1557 (Modified Proctor).

4.4.4 Culvert Excavation Dewatering

Our explorations suggest that that the required culvert excavation will extend below the static groundwater table at each location. Dewatering will be required to lower the groundwater table to an elevation below the bottom of the proposed culvert foundations. Given the fine-grained nature of the alluvial soils at the culvert foundation elevations, we do not expect that the use of dewatering wells would be appropriate means of dewatering. We expect that dewatering will consist of the use of sumps and pumps to lower the groundwater level to an appropriate elevation.

To limit dewatering efforts at each culvert location, the foundation over excavation and replacement with quarry spalls could be completed partially within the wet.

4.4.5 Culvert Lateral Loading

Culvert structures will need to be designed to withstand lateral loads imposed on the sides of the culverts and walls by the surrounding soils. In addition, the culverts will need to be designed to withstand the soil overburden pressure and traffic loading. A unit weight of 135 pcf can be used to account for structural fill placed over the culverts.

Due to their proposed orientations and structural connections, the culvert structures are assumed to **not** be free to deflect by at least 0.001H, where H is the retained height of the wall, to allow active conditions to develop. Therefore, at-rest earth pressures should be assumed.

In consideration of these assumptions, an at-rest equivalent fluid pressure of 52 pounds per cubic foot can be assumed for static conditions. This assumes the culverts are backfilled with Gravel Barrow as defined by WSDOT Standard Specification Section 9-03.14(1) compacted to at least 95 percent of its maximum dry density in accordance with ASTM D 1557 (Modified Proctor). To account for live loading from traffic, it is conventional to assume a uniform vertical surcharge pressure of 250 pounds per square foot. This vertical surcharge will be experienced as a uniform rectangular horizontal pressure distribution of 95 pounds per square foot of wall. Therefore, under static loading conditions the culvert structures should be designed for a 95 psf uniform rectangular horizontal pressure, plus 52 pcf equivalent fluid pressure. These earth pressures assume the water level within the culvert structures is equal to the water level outside the culvert structures, and no unbalanced hydrostatic pressures develop. Under most conditions, we expect the water level outside the culvert to match the level inside the culvert. However, the culvert should be designed to resist unbalanced hydrostatic pressures associated with the design extreme event.

Under earthquake loading conditions, the culvert structures and their wings walls will experience an incremental additional horizontal earth pressure. Due to their configurations and structural connections, we do not expect that the culvert structures and their wing walls will be able to yield sufficiently, during the design seismic events, to develop active earth pressures. Therefore, we recommend the use of at-rest pressures for the design seismic events. The at-rest plus seismic equivalent earth pressures can be approximated using the Mononobe-Okabe method utilizing 1.0 times the PGA for the site. The recommended at-rest plus seismic equivalent fluid pressure of 70 pcf should be assumed.

Lateral resistance can also be resisted by friction as the base of the culverts. For culvert subgrade prepared as recommended above, an unfactored sliding coefficient of 0.5 times the effected stress can be used for precast concrete against the crushed rock base.

4.4.6 Culvert Headwall Lateral Loading

Culvert headwalls will need to be designed to withstand lateral loads imposed by the soil above the walls. We expect that the culvert headwalls will consist of cast in place concrete walls that are structurally connected to the culvert structures. We assume that the soil placed above the proposed culvert structures will be sloped at a maximum slope of 2H:1V. Due to their proposed orientations and structural connections to the culvert structures, the culvert headwalls are assumed to **not** be free to deflect by at least 0.001H, where H is the retained height of the wall, to allow active conditions to develop. Therefore, at-rest earth pressures should be assumed.

In consideration of these assumptions, an at-rest equivalent fluid pressure of 61 pounds per cubic foot can be assumed for static conditions. To account for live loading from traffic, it is conventional to assume a uniform vertical surcharge pressure of 250 pounds per square foot. This vertical surcharge will be experienced as a uniform rectangular horizontal pressure distribution of 112 pounds per square foot of wall. Therefore, under static loading conditions the culvert structures and their wing walls should be designed for a 112 psf uniform rectangular horizontal pressure, plus 61 pcf equivalent fluid pressure. If the anticipated traffic surface is to be located outside of a 1H:1V zone of influence of the proposed culvert headwalls, the traffic surcharge can be ignored in design of the headwalls. These earth pressures assume no accumulation of water behind the headwall and no development of hydrostatic pressures.

Under earthquake loading conditions, the culvert headwalls will experience an incremental additional horizontal earth pressure. Due to their configurations and structural connections, we do not expect that the culvert headwalls will be able to yield sufficiently, during the design seismic events, to develop active earth pressures. Therefore, we recommend the use of at-rest pressures for the design seismic events. The at-rest plus seismic equivalent earth pressures can be approximated using the Mononobe-Okabe method utilizing 1.0 times the PGA for the site. The recommended at-rest plus seismic equivalent fluid pressure of 90 pcf should be assumed.

4.4.7 Culvert Regrading

Raising grade along the roadway is expected to result in settlement of the proposed roadway and culvert structures. As the culverts will result in a void under the roadway, we expect settlement of the culvert structures to be less than the roadway on either side of the culvert. Therefore, we expect a sag within the roadway could result on either side of each culvert structure. Our calculation indicated that the majority of roadway settlement will occur within 3-6 months of placement. We recommend that the culvert structures be placed, and the roadway grade be raised early in the construction sequence. This will allow for settlement to occur during construction of the bridge structure. At the end of construction, we recommend that the roadway grade be leveled. This should reduce settlement impacts on the roadway grade.

4.5 BRIDGE FOUNDATIONS

4.5.1 Bridge Foundation Type

HWA evaluated the subsurface conditions at the proposed bridge abutments. Our explorations encountered a layer of soft compressible fine-grained soils near the ground surface that will not provide adequate bearing capacity to support the use of shallow spread footings for the bridge structure. Therefore, we recommend that the bridge abutments be supported on deep foundations bearing below the fine-grained compressible soils. Our explorations encountered gravelly soils that contain cobbles and potentially boulders. Cobbles and boulders, if encountered, could obstruct driven piles. Therefore, we do not recommend that driven piles be used to support the proposed bridge structure. We recommend that the bridge structure be supported on drilled shaft foundations bearing in the dense gravelly soils encountered at depth. We understand that 4-foot diameter drilled shafts are planned for the project. The density of the upper portion of the gravelly soils is variable, therefore we recommend a minimum tip elevation of 40 feet below existing grades (elevation of approximately 980 feet). Due to the potential of cobbles and boulders within the bearing soils, we do not recommend drilled shafts with a diameter less than 4-feet.

4.5.2 Drilled Shaft Axial Capacity

Axial shaft capacities were evaluated using Load and Resistance Factor Design (LRFD) methods in general conformance with the procedures referenced in the FHWA Drilled Shafts Manual, 8th Edition (Brown, et al., 2018). This method provides a revised method to the Reese and O'Neill method (1989). Axial capacities will be derived from both shaft friction and end bearing. Nominal axial shaft capacities versus embedment depths for the southern and northern abutments are presented in Figures 4 and 5 for 4-foot diameter shafts.

As indicated on these figures, resistance factors (ϕ) of 0.55 and 0.45 should be applied to the nominal side resistance, for the Strength I Limit State for cohesionless and cohesive soils, respectively. Resistance factors of 0.5 and 0.4 should be applied to the nominal base resistance for Strength I Limit State design for cohesionless and cohesive soils, respectively. For the Extreme I and the Service I Limit States, the resistance factor (ϕ) should be 1.0 for both shaft friction and end bearing.

It should be noted that no positive vertical capacity is assumed to develop above the base of the compressible fine-grained soils. This is due to the fact that down drag loading is expected to develop within the compressible soils due to consolidation settlement associated with proposed grade increases along the roadway. Estimated down drag loads are discussed in Section 4.5.3 below.

For the Service I Limit state, total shaft resistance (i.e., friction plus end bearing) is provided for an allowable settlement of 1 inch. If a Service I Limit State capacity for a different settlement value (e.g. 2 inches or ½ inch) is needed, we should be contacted to revise our calculations.

If the drilled shaft cap beam is to be constructed in firm contact with the ground, then no vertical capacity reduction is required for shaft spacing no closer than 3 shaft diameters. If the shaft cap is not to be designed in firm contact with the ground, then vertical capacity efficiency factors will need to be applied to account for shaft center-to-center spacing of less than 6 shaft diameters.

4.5.3 Down Drag Loading Parameters

Down drag loading on shafts occurs when the surrounding soil settles or otherwise moves downward relative to the shaft. Downward movements on the order of ¼ inch are sufficient to fully mobilize negative shaft resistance or down drag. Due to the presence of compressible soils along the bridge alignment and proposed fill placements, each drilled shaft will experience down drag loading as a result of the consolidation of the compressible soils. Down drag loads associated with consolidation of the fine-grained soils should be added to other foundation loads for the Strength Limit State. The unfactored down drag force for the proposed 4-foot diameter shafts at each abutment location are presented in Table 3.

Table 3.
Down Drag Loading Due to Consolidation of Fine-grained Soils (Per Shaft)

Shaft Diameter (ft)	South Abutment (Kips)	North Abutment (Kips)
4	210	158

A load factor of 1.05 should be applied to the values in Table 3.

These downdrag loads only account for negative skin friction along the perimeter of the shafts. To avoid the development of downdrag loads on the abutment wall and pile cap, we recommend that a bond breaker be placed between the abutment wall and the associated embankment fill.

This bond breaker could consist of multiple layers of overlapped plastic sheeting draped over the abutment wall prior to backfilling the wall with structural fill.

4.5.4 Drilled Shaft Lateral Design Parameters

Lateral loads may be resisted by the passive earth pressure against deep foundations and foundation caps. The magnitude of lateral resistance developed by drilled shafts depends on the subsurface conditions encountered and the moment capacity at the foundation cap connection. We recommend ignoring the friction sliding resistance at the base of the foundation cap, because a deep foundation-supported cap may not transmit load directly to the soil beneath it.

We understand that the design team desires to use a conventional p-y method of lateral analysis (i.e., LPILE) to estimate shears, moments, and deflections of the shafts. Soil parameters for use in LPILE analyses are provided in Appendix C. Liquefaction potential is considered low; therefore, the values provided are for both static and seismic conditions.

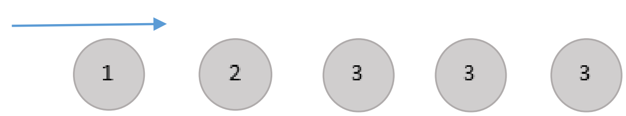
The p-y curves generated by the lateral parameters provided in Appendix C must be modified by the applicable p-multipliers to account for group reduction effects. The 80 percent design plans indicate approximately 16 feet from center to center of the proposed shafts. The p-multipliers for a shaft spacing of 4 shaft diameters (4-foot diameter shafts on 16-foot spacing) are provided in Table 4. If center-to-center spacings for the proposed foundations vary from these assumptions, HWA should be contacted to provide updated p-multipliers.

Table 4.
P-Multipliers for Center-to-Center Spacing of 4 Shaft Diameters

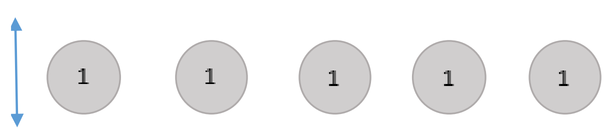
Row	P-Multiplier
1	0.90
2	0.63
3 or more	0.50

The same p-multiplier factor should be applied parallel and perpendicular to the group shaft alignment. The following diagram shows how the p-multipliers should be assigned with respect to the load direction and shaft orientation.

Parallel Direction



Perpendicular Direction



4.5.5 Drilled Shaft Construction Considerations

The drilled shafts will be drilled through existing gravelly fill, alluvium consisting of loose silty sand and soft silty clay, and into variable gravelly outburst flood deposits. The outburst flood deposits are generally loose to medium dense and become dense to very dense with depth. The drilled shaft contractor should be prepared to encounter and handle cobbles and boulders, this may require rock coring using a core barrel. The contractor should be prepared to drill for extended periods of time to advance through particularly dense layers or obstructions. Based on the large cobbles, we recommend that drilled shafts be constructed using the oscillator method.

The contractor should be prepared to construct the shafts below the groundwater level and provide appropriate methods for stabilizing the sides and bottom of the shaft excavations. Soils excavated from the shafts will likely be saturated and could require decanting prior to being transported off-site. The contractor should be prepared to undertake decanting of the soil excavated from the drilled shafts.

Drilled shaft bottoms should be cleaned to the extent practical using appropriate excavation methods to provide for a relatively undisturbed shaft base. After the shaft bottoms are cleaned, concrete should be placed by the tremie method into the shafts. Temporary casing should be withdrawn such that the level of concrete is maintained above the bottom of the casing at all times and at such elevations to counteract any potential hydrostatic effects associated with groundwater conditions that may be present at the location of the work.

4.5.6 Bridge Abutment Lateral Loading

We expect that the abutment walls will be free to be able to defect by at least 0.001H, where H is the retained height of the wall, to allow active conditions to develop. Therefore, active earth pressures should be assumed for both abutment walls. An active equivalent fluid pressure of 35 pounds per cubic foot can be assumed for static conditions. To account for live loading from traffic, it is conventional to assume a uniform vertical surcharge pressure of 250 pounds per square foot. This vertical surcharge will be experienced as a uniform rectangular horizontal pressure distribution of 60 pounds per square foot of wall. Therefore, under static loading conditions the abutment walls should be designed for a 60 psf uniform rectangular horizontal pressure, plus 35 pcf equivalent fluid pressure. These earth pressures assume no accumulation of water behind the walls. Proper wall drainage should be constructed to ensure that hydrostatic pressures do not develop behind the wall structures.

Under earthquake loading conditions, the abutment walls will experience an incremental additional horizontal earth pressure. We expect that the abutment walls will be able to yield sufficiently, during the design seismic events, to develop active earth pressures. Therefore, we recommend the use of active pressures for the design seismic events. The active plus seismic equivalent earth pressures can be approximated using the Mononobe-Okabe method utilizing 0.5

times the PGA for the site. A recommended active plus seismic equivalent fluid pressure of 41 pcf should be assumed.

4.6 BRIDGE SCOUR PROTECTION

We understand that scour protection will be required in the new channel to protect the abutments and line the channel. Based on correspondence with the design team we understand that installation of rip rap for scour protection will require excavations down to elevations as deep as about elevation 1005 feet. Based on our explorations, these excavations will extend below the groundwater table, through the fine-grained alluvium, and into the outburst flood deposits. The outburst flood deposits underlying the alluvium consist of large open graded gravels and will require extensive de-watering to achieve a dry excavation.

In our opinion, the large rip rap material can be placed in the wet. However, it should be noted that if excavations extend below the water table significant sloughing and caving should be anticipated, resulting in larger temporary excavations. In our opinion, placement of rip rap below the water table is feasible if small segmental excavations are done to the required scour depths and then rip rap is immediately placed. If the rip rap is placed before the drilled shafts oversized casing should be placed at each shaft location, prior to placement of rip rap, to ensure that rip rap does not migrate to the shaft locations. Alternatively, drilled shafts can be installed prior to stream channel excavation and placement of scour protection.

4.7 RETAINING WALLS

We understand that gravity block or Structural Earth Walls (SEWs) will be used for bridge and culvert wing walls. Based on the 80 percent plans and updated wall profiles for the bridge to account for the post fire scenario, we understand that the base of the wing walls will generally be at elevations of between about 1011 to 1022 feet, with total wall heights (including required embedment) between about 4 to 13 feet in height. Based on our geotechnical borings, we anticipate that the base of the walls will generally be founded within the loose very silty sand and soft silty clay that we encountered near the base of the proposed culverts. Subgrade stabilization will be required to provide adequate wall support. The walls are generally anticipated to retain the existing roadway embankment and new structural fill that will be required to achieve the desired road grades.

4.7.1 Wall Design Parameters

We assume that SEW walls will consist of a proprietary wall system that the wall supplier will design for internal stability. The walls should be designed in accordance with the most current version of the AASHTO *LRFD Bridge Design Manual* and Section 6.13 of the WSDOT *Standard Specifications* (WSDOT, 2023). We recommend that the walls be designed using the parameters presented in Table 5. We understand that the design for these walls will be performed using LRFD. Appropriate AASHTO resistance factors should be used for design of all retaining walls.

For the Extreme Event I Limit State, the walls shall be designed for a horizontal seismic acceleration coefficient K_h of one-half the peak ground acceleration or 0.111g and vertical seismic coefficient Kv of 0.0g (assuming the wall is free to move during a seismic event). Extreme Event I Limit State is defined in the AASHTO Standard Specifications as a safety check involving an extreme load even resulting from an earthquake in combination with the dead load and a fraction of the live loads.

Table 5. Recommended Design Parameters for SEW Walls

Soil Properties	**Reinforced soil**	**Retained Soil**	**Foundation Soil**
Unit Weight (pcf)	135	135	120
Friction Angle (deg)	36	34	28
Cohesion (psf)	0	0	0
		Strength Limit State (EP+LL)	**Extreme Limit State (EP+EQ)**
Ultimate Bearing Resistance (ksf)		4.0	4.0
Horizontal Seismic Acceleration Coefficient (k_h) (g)		N/A	0.111

To satisfy global wall stability requirements we recommend that walls over 5 feet in height be embedded at least 2 feet below existing grades; walls 5 feet or less in height should be embedded at least 1 foot below existing grades. These minimum embedment depths assume grades in front of the walls of up to 3H:1V. For walls with grades in front of the wall of up to 2H:1V we recommend a minimum embedment of at least 2 feet below existing grades.

It is important that the walls be designed per specific toe- and back-slope geometry at each wall location. Additionally, vertical, and lateral dead loads such as pavement, guard rails, and chain-link fences, and live loads such as vehicular, pedestrian, construction equipment loading should be considered in design of each retaining wall. An unfactored coefficient of friction of 0.5 times the effective stress at the base of the wall can be used for sliding resistance, provided the wall subgrade is prepared as recommended below.

4.7.2 Wall Settlement

We anticipate that the SEWs will experience magnitudes of settlement similar to those calculated in Table 2 of the Geotechnical Report based on the various proposed fill heights and associated wing wall heights. The amount of settlement will be dependent on wall height and thickness of compressible material. We anticipate total wall settlement could range from 1 and 3 inches.

For the Service Limit State, the wall shall be designed to accommodate a differential settlement of up to 2 inches per 100 feet of wall length.

4.7.3 Subgrade Preparation

The loose and soft alluvial soils, at the anticipated base of most of the SEWs, will not provide an adequate working surface for wall construction. To create stable subgrade for the SEWs we recommend assuming that up to 2 feet of over excavation and replacement with stabilization material and structural fill will be required to provide adequate wall subgrade support. We recommend an HWA geotechnical engineer, or their representative, be present during construction to verify the assumptions made for the foundations of the walls are met.

To stabilize the over-excavated subgrade, we recommend tamping in about 1.5 foot of quarry spalls as defined in Section 9.13.1(5) of the WSDOT Standard Specifications (WSDOT, 2023). Stabilization material should be tamped into the subgrade with an excavator bucket until a firm and well keyed condition is established.

A geotextile fabric should be placed over the compacted stabilization material. Geotextiles for Separation should meet the requirements of Table 3 in Section 9-33.2(1) of the WSDOT *Standard Specifications* (WSDOT, 2023). Then the remainder of the two-foot section of over-excavated area should be backfilled with crushed surfacing base course (CSBC) and compacted over the geotextile. The CSBC should be at least inches thick and should be compacted to at least 95 percent of the maximum dry density determined using ASTM D 1557 (Modified Proctor). This leveling pad should be graded to establish the proper wall batter.

4.7.4 Wall Backfill

Wall backfill materials should consist of Gravel Backfill for Walls, as described in Section 9-03.12(2) of the WSDOT *Standard Specifications* (WSDOT, 2023) and should be compacted to at least 95% of the maximum dry density as determined by ASTM D 1557 (Modified Proctor). The wall backfill should be placed and compacted in layers as each row of blocks is placed.

The Contractor should consider the weight of construction equipment operating within the fill zone behind the wall. For compaction, materials within about 3 feet of the wall face should be compacted with lighter equipment to limit the loading on the back of the wall.

4.7.5 Wall Drainage

Drainage should be provided behind all walls to prevent buildup of hydrostatic pressures and should consist of a 4- to 6-inch diameter, perforated, rigid plastic pipe, bedded and backfilled with Gravel Backfill for Drains, as specified in Section 9-03.12(4) of the WSDOT *Standard*

Specifications (WSDOT, 2023). The drain rock should surround the drainpipe by at least 6 inches. The pipes should slope to drain to a suitable outlet.

4.8 WATERLINE

It is our understanding that a pressurized waterline currently runs along the project alignment, and the waterline is being reconfigured as part of the project. Based on the 80 percent plans the waterline will be installed below the proposed culvert structures. The waterline will need to tolerate anticipated differential settlements at the culvert locations and at the bridge abutments. We understand that this will be accomplished with flexible piping, flexible connections, and over-sized steel casings. Dewatering and temporary shoring may also be required at the culvert locations.

4.8.1 Waterline Temporary Shoring

Waterline trench shoring should be the responsibility of the contractor. However, based on typical construction practices, we expect that temporary shoring for the waterline trench will likely consist of conventional trench shields (trench boxes). It should be noted that trench boxes are designed to protect the life and safety of the workers within the excavation but may not effectively apply sufficient active pressure against the excavation walls necessary to mitigate cave-ins.

4.8.2 Waterline Dewatering

HWA anticipates that the local groundwater table varies seasonally across the site. Observed groundwater levels vary across the site from about 5 to 16 feet below existing ground surface. It is our understanding that the waterline will extend under the proposed culvert structures and will have a maximum invert depth of about 13 feet below the existing roadway grade. In order to account for soft soil conditions and pipe bedding requirements, maximum trench depths should be increased by 1 foot below pipe invert, as described in Section 4.8.3. Therefore, we anticipate waterline excavation depths will vary from as shallow as 7 feet at the bridge locations to as deep as 14 feet at the culvert under crossings. Based on these depths, we expect that dewatering may be required for installation of portions of the waterline. We expect that dewatering for the waterline will consist of sumps and pumps, with the largest dewatering effort at the culvert under crossings. If the contractor wishes to install the waterline under the culverts in the wet, HWA should review the proposed installation method prior to installation.

4.8.3 Utility Trench Subgrade Preparation

The subgrade soils along the proposed waterline alignment are expected to consist of soft alluvial soils. We expect that these soils will provide highly variable subgrade support along the proposed waterline. To minimize required over-excavation and replacement, we recommend that all trench excavations be over-excavated 1-foot below the base of proposed pipe bedding.

Once this over-excavation is made, the base of the trench should be inspected for the presence of unsuitable materials.

Unsuitable materials may be present in the alluvial soils, such as peat, organics, or deleterious material (e.g., logs, stumps etc.), if encountered at the base of the trench, these unsuitable materials should be removed. Such materials should be over-excavated, and the exposed subgrade compacted to a firm state, as determined by the geotechnical engineer. Over-excavated areas should be backfilled, with ledge rock or quarry spalls that are tamped into an unyielding condition. Over-excavations should extend on either side of the pipe a distance equal to the depth of the over-excavation beneath the invert elevation or full width of the trench.

Once over-excavated areas have been backfilled, the subgrades should be compacted to a firm state as determined by the geotechnical engineer. Trench bottoms should be free of debris and standing water. If subgrade soils are disturbed, the disturbed material should be removed down to undisturbed soil and replaced with properly placed and compacted structural fill bedding material. To minimize trench subgrade disturbance during excavation, the excavator should use a smooth-edged bucket rather than a toothed bucket.

Once the trench subgrade is properly prepared, a geotextile separator fabric meeting the requirements of Table 3 of Section 9-37.2 of the WSDOT standard specifications (WSDOT, 2023) should be placed across the base of the excavation. The placement of the geotextile separator fabric will allow for placement aggregate with limited loss to the subgrade soils.

4.8.4 Waterline Pipe Bedding

Once the subgrade soils have been stabilized, we recommend the waterline be founded on suitable pipe bedding material, such as Gravel Backfill for Pipe Zone Bedding, meeting the requirements of Section 9-03.12(3) of the WSDOT, 2023). Based on the results of the laboratory testing program, the onsite soils encountered at the site generally do not meet the requirements for pipe bedding. Therefore, pipe bedding material will need to be imported.

Pipe bedding should provide a firm uniform cradle for support of the pipes. A minimum 12-inch thickness of bedding material beneath the pipe should be provided. Prior to installation of the pipe, the pipe bedding should be shaped to fit the lower part of the pipe exterior, with reasonable closeness, to provide uniform support along the pipe. Pipe bedding material should be placed in layers and tamped around the pipes to obtain complete contact. To protect the pipe, bedding material should extend at least 12 inches above the top of the pipe.

4.8.5 Waterline Material

Due the anticipated consolidation settlement from fill placement and resulting settlement of the culvert structures we recommend that the waterline be constructed with fused HDPE pipe or other pipe capable for tolerating anticipated differential settlements.

4.8.6 Waterline Trench Backfill

Native and fill soils encountered in the explorations generally contain high fines contents and will not be suitable for reuse as trench backfill, we do not recommend that the native soil be reused as trench backfill for this project.

Imported trench backfill should consist of Gravel Borrow, meeting the requirements of Section 9-03.14(1) of the WSDOT *Standard Specifications* (WSDOT, 2023). Trench backfill should be free of organics and other debris. The project specifications and cost estimate should account for imported trench backfill.

4.8.7 Waterline Trench Backfill Placement and Compaction

Proper preparation, placement and compaction of the trench backfill is extremely important to limit future settlement of the ground surface around structures and along trenches. Given the depth of the proposed trench, failure to achieve proper compaction could result in significant settlement on the order of several inches, resulting in observed surface settlement.

Trench backfill should be uniformly moisture conditioned to within about 3 percent of optimum moisture content prior to placement in the trench. Properly prepared backfill should be placed in successive layers with the minimum cover to be determined based on the pipe material utilized, and the following layers not exceeding 12 inches in loose thickness with each layer being compacted in a systematic manner using appropriately sized compaction equipment to achieve at least 95 percent of the maximum dry density as determined using ASTM D-1557. Smaller loose lifts may be necessary to achieve compaction where handheld compaction equipment such as jumping jacks, hoe-packs, or plate compactors are used. The contractor should develop compaction methods that consistently produce adequate compaction levels.

During placement of the initial lifts, the trench backfill material should not be bulldozed into the trench or dropped directly on the pipe. Furthermore, heavy vibratory equipment should not be permitted to operate directly over the pipe until a minimum of 2 feet of backfill has been placed over the pipe bedding to an in-place density of at least 90 percent of the maximum dry density as determined using ASTM D 1557.

4.8.8 Waterline Flexible Connections

We understand that the waterline will be attached to the bridge structure. The embankment fill is anticipated to cause consolidation settlement of the underlying compressible soils. The bridge structure will be supported on drilled shafts and not experience settlement, creating differential settlement at the bridge connection. To withstand this differential settlement between the grade supported waterline section and bridge supported waterline section we recommend that flexible connections capable of tolerating up to 6-inches of vertical displacement be used at the connection points.

4.9 EMBANKMENT SLOPES

We recommend that compacted roadway embankment slopes for the project be constructed no steeper than 2H:1V (Horizontal:Vertical). For fill slopes constructed at 2H:1V or flatter, and comprised of fill soils placed and compacted as structural fill as described in Sections 5.1 and 5.2 of this report, we anticipate that adequate factors of safety against global failure will be maintained. Measures should be taken to prevent surficial instability and/or erosion of embankment material. This can be accomplished by conscientious compaction of the embankment fills in level lifts, benched cuts into the slope face, maintaining adequate drainage, and planting the disturbed slope face with vegetation as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to finish grade.

4.10 CULVERT REMOVAL

We understand that an existing culvert will be removed at approximately Station 11+60. After removal of the culvert about 17 feet of fill will be required to achieve the proposed roadway grades from the bottom of the removed culvert. Based on the elevation and location of the existing culvert, we anticipate that soft and wet subgrade conditions will be present after removal. Subgrade stabilization will be necessary at this location prior to embankment construction.

To stabilize the over-excavated subgrade, we recommend tamping in about 2 feet of quarry spalls as defined in Section 9.13.1(5) of the WSDOT Standard Specifications (WSDOT, 2023). Stabilization material should be tamped into the subgrade with an excavator bucket until a firm and well keyed condition is established.

A geotextile fabric should be placed over the compacted stabilization material. Geotextiles for Separation should meet the requirements of Table 3 in Section 9-33.2(1) of the WSDOT *Standard Specifications* (WSDOT, 2023). Then the embankment can be constructed as recommended in Section 4.9 of this report.

To limit the impacts to the wetland, a retaining wall could be installed within the embankment fill. We anticipate that the wall would be a contractor designed SEW wall, similar to the proposed bridge and wing walls. The wall should be designed in accordance with the recommendations in Section 4.7 of this report. As the wall is anticipated to be founded within a new embankment of up to 2H:1V the wall should be embedded a minimum of at least 2 feet below existing grades.

5.0 EARTHWORK

5.1 STRUCTURAL FILL

Structural fill should consist of imported clean, free-draining, granular soils free from organic matter or other deleterious materials. Such materials should be less than 4 inches in maximum particle dimension, with less than 7 percent fines (portion passing the U.S. Standard No. 200 sieve), as specified for Select Borrow in Section 9-03.14(2) of the 2023 WSDOT *Standard Specifications*. The fine-grained portion of structural fill soils should be non-plastic.

The onsite near surface existing gravel fill can be re-used as structural fill provided that it can be properly moisture conditioned and compacted. The underlying native alluvium soils are highly variable and moisture sensitive. We do not recommend reusing the onsite native soils as structural fill for this project.

It is our understanding that Yakama Nation owns and operates borrow pits in close proximity to the project site. If importation of structural fill from these pits is desired, the aggregate generated from the pits should be evaluated and additional recommendations provided.

5.2 COMPACTION

Structural fill soils should be moisture conditioned and compacted to the requirements specified in Section 2-03.3(14), Method C, of the 2023 WSDOT *Standard Specifications*, except that maximum dry densities should be obtained using ASTM D 1557 (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.

5.3 TEMPORARY EXCAVATION

We expect that excavations completed onsite can be accomplished with conventional excavating equipment such as track hoes. Temporary excavations are anticipated to install the culverts, bridge abutments, and wing walls. The onsite soils will be prone to sloughing and raveling.

Maintenance of safe working conditions, including temporary excavation stability is the responsibility of the contractor. In accordance with Part N of Washington Administrative Code (WAC) 296-155, all temporary cuts more than 4 feet in height must be either sloped or shored prior to entry by personnel. The existing fill, alluvial soils, and upper outburst flood deposits are generally classified as Type C soils per WAC 296-155. Where shoring is not used, temporary cuts in Type C soils should be sloped no steeper than 1.5H:1V (horizontal: vertical); however, if groundwater seepage is observed on cut slopes, shallower inclinations may be needed to maintain safe working conditions.

The design, installation, maintenance and removal of temporary shoring should be the responsibility of the contractor. The shoring system should be designed by a qualified and licensed engineer experienced with shoring design for deep excavations within similar soil conditions. We recommend that the design of the temporary shoring system be submitted by the contractor, for approval, prior to starting excavation. HWA should be allowed to review shop drawings and calculations for proposed shoring systems to check for consistency with the recommendations included in this report.

5.4 WET WEATHER EARTHWORK

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.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation of unsuitable and/or softened soil should be followed promptly by placement and compaction of clean structural fill. The size and type of construction equipment used may need to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic.

- Material used as excavation backfill in wet weather should consist of clean granular soil with less than 5 percent passing the U.S. No. 200 sieve, based on wet sieving the fraction passing the ¾-inch sieve. The fines should be non-plastic. It should be noted this is an additional restriction on the structural fill materials specified.

- The ground surface within the construction area should be graded to promote surface water run-off and to prevent ponding.

- Within the construction area, the ground surface should be sealed on completion of each shift by a smooth drum vibratory roller, or equivalent, and under no circumstances should soil be left uncompacted and exposed to moisture infiltration.

- Excavation and placement of backfill materials should be monitored by a geotechnical engineer experienced in wet weather earthwork to determine that the work is being accomplished in accordance with the project specifications and the recommendations contained herein.

6.0 CONDITIONS AND LIMITATIONS

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

We recommend HWA be retained to review the plans and specifications to verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

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

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

We appreciate this opportunity to be of service.

Sincerely,

HWA GEOSCIENCES, INC.

6-26-2025

6-26-2025

Joe Westergreen, P.E.
Geotechnical Engineer

Donald Huling, P.E.
Geotechnical Engineer, Principal

7.0 REFERENCES

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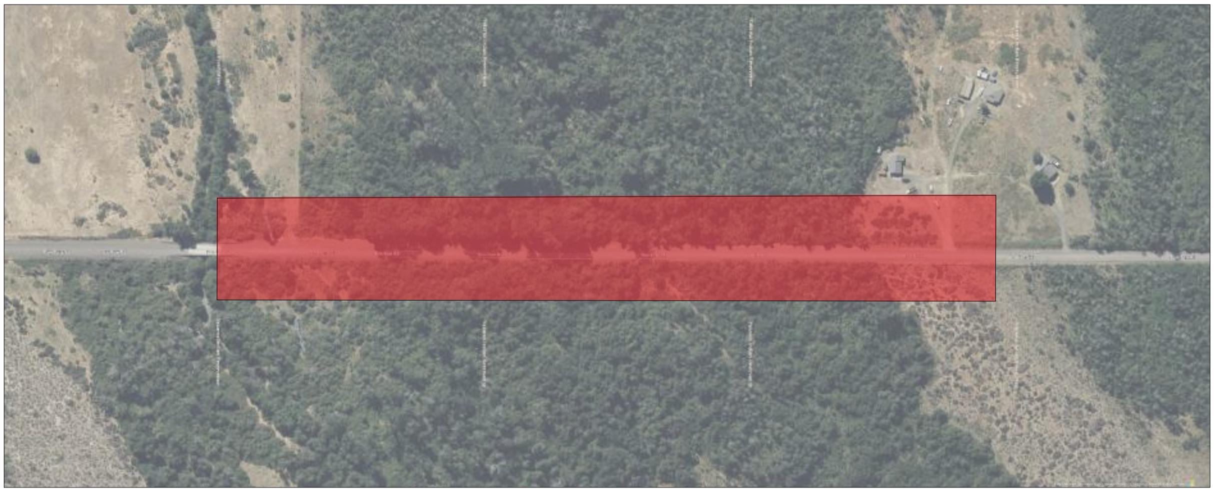
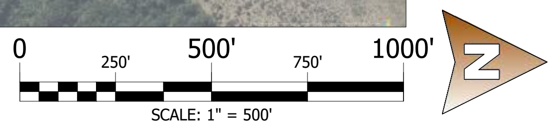
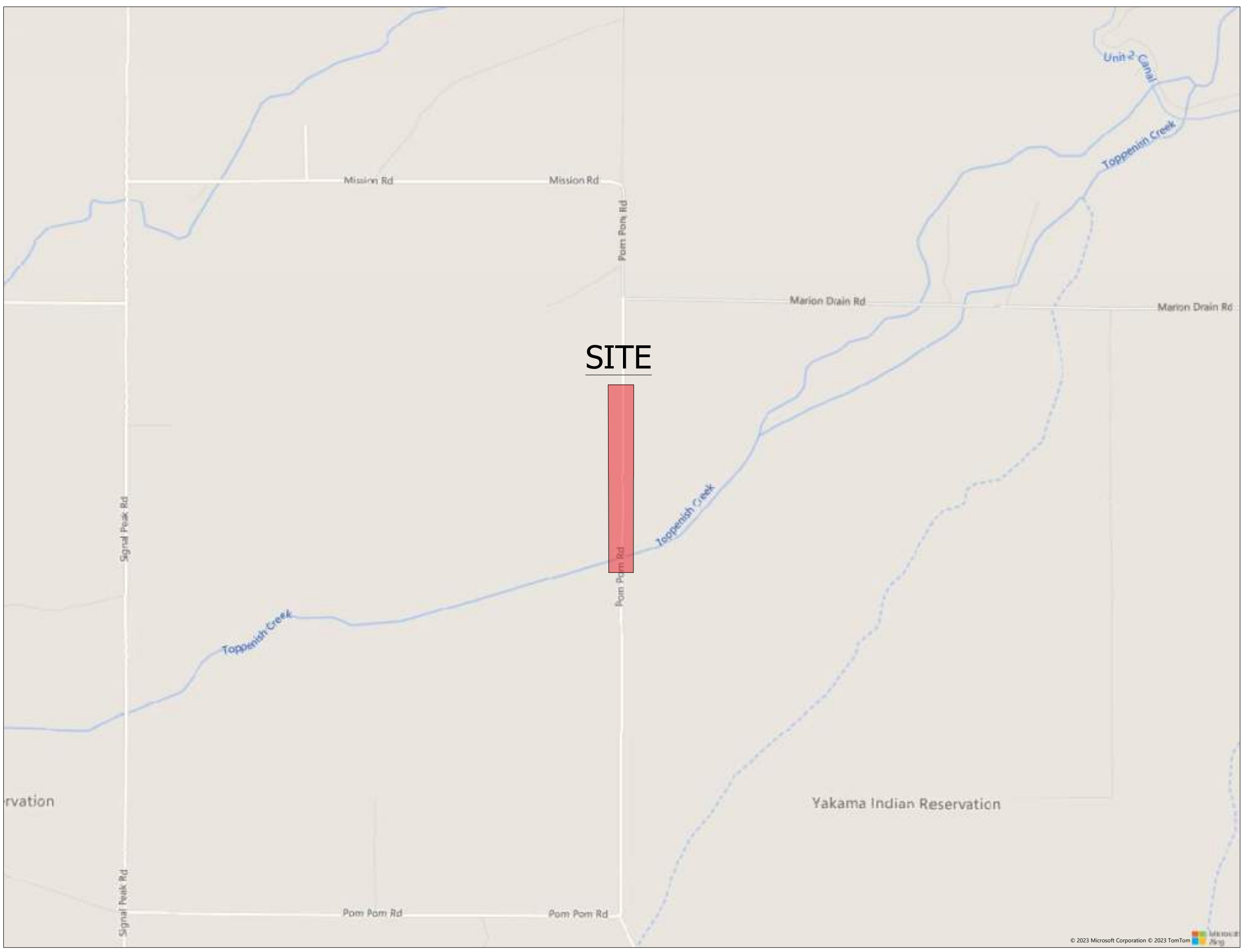
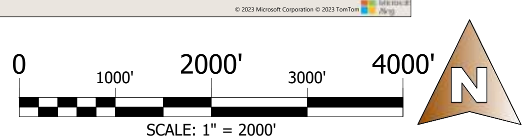


SITE MAP





VICINITY MAP



HWA GEOSCIENCES INC.
DBE/MWBE

SITE AND VICINITY MAP		FIGURE NO.: 1
POM POM ROAD IMPROVEMENTS YAKAMA NATION, WASHINGTON	DRAWN BY: CF	CHECK BY: DJH
	PROJECT # 2021-097-21	

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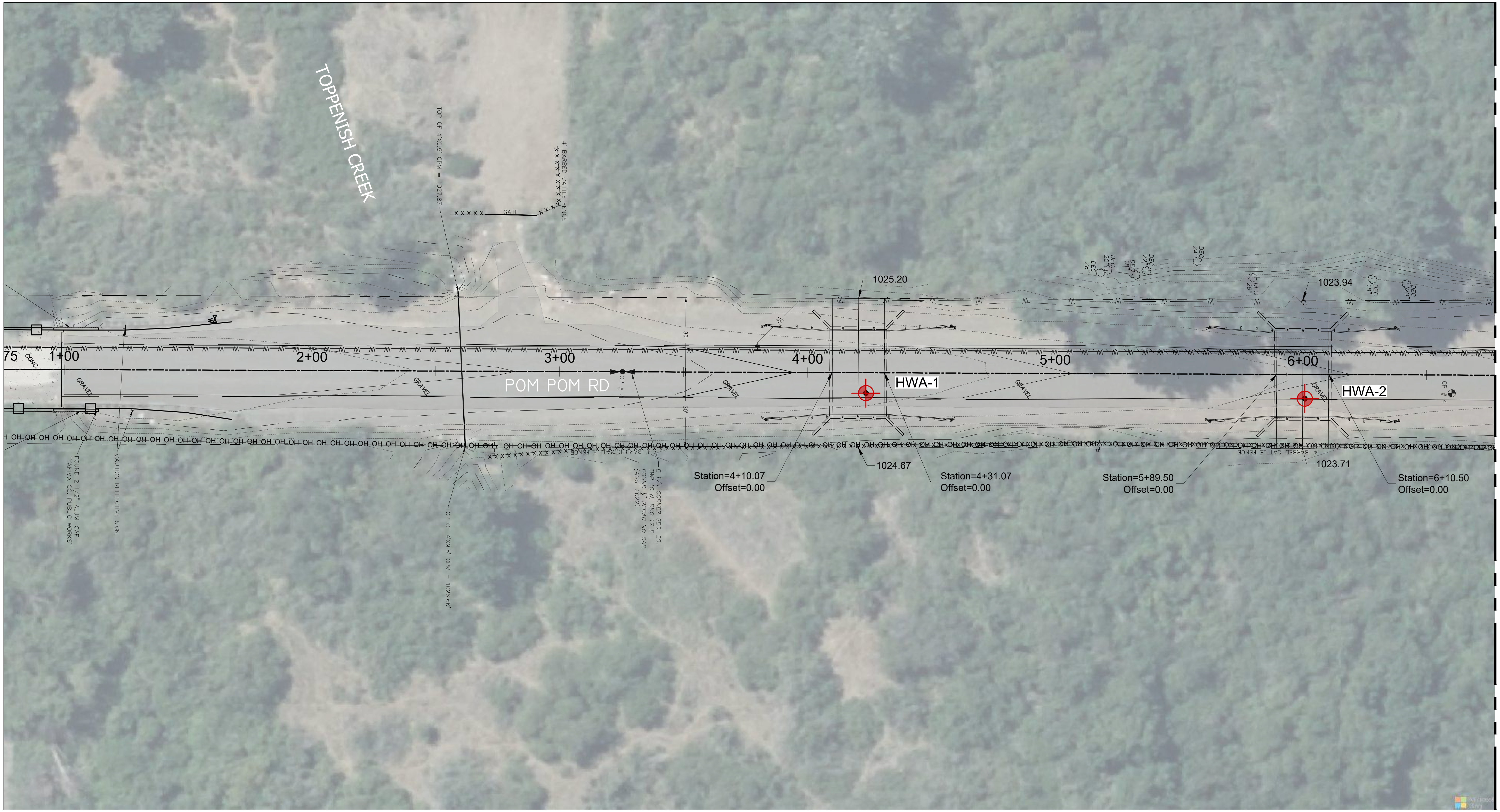

POM POM ROAD

Scale: 1" = 40'-0"

EXPLORATION LEGEND

HWA-1 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

0 20 40 60 80

SCALE: 1" = 40'

HWA GEOSCIENCES INC. DBE/MWBE

POM POM ROAD IMPROVEMENTS
YAKAMA NATION, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2A
CHECK BY:	PROJECT NO.:
DJH/JTW	2021-097-21

BASE MAP PROVIDED BY: BING AND KPFF 3.3.2023

S:\2021 PROJECTS\2021-097-21 POM POM BRIDGE DESIGN\CAD\2021-097-21 POM POM BRIDGE DESIGN.DWG <2A> Plotted: 3/28/2023 1:01 PM

MATCHLINE SEE 2A

MATCHLINE SEE 2C

POM POM ROAD

Scale: 1" = 40'-0"

EXPLORATION LEGEND

HWA-3 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

N

0 20 40 60 80

SCALE: 1" = 40'

GEOSCIENCES INC. DBE/MWBE

POM POM ROAD IMPROVEMENTS
YAKAMA NATION, WASHINGTON

SITE & EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2B
CHECK BY:	PROJECT NO.:
DJH/JTW	2021-097-21

BASE MAP PROVIDED BY: BING AND KPFF 3.3.2023

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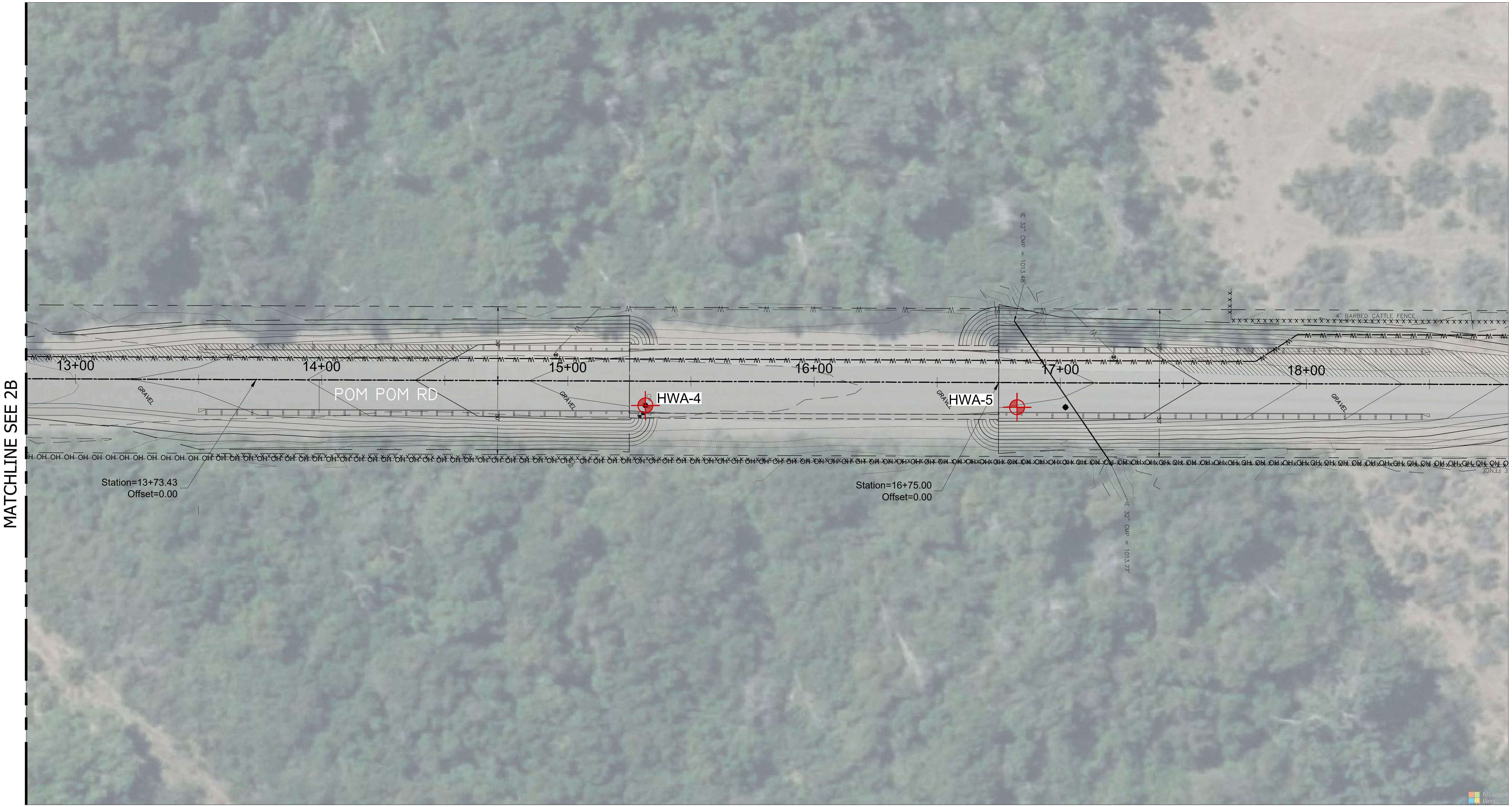

POM POM ROAD
Scale: 1" = 40'-0"

EXPLORATION LEGEND

HWA-5 BOREHOLE DESIGNATION AND APPROXIMATE LOCATION

0 20 40 60 80
SCALE: 1" = 40'

HWA GEOSCIENCES INC. DBE/MWBE

POM POM ROAD IMPROVEMENTS
YAKAMA NATION, WASHINGTON

SITE &
EXPLORATION PLAN

DRAWN BY:	FIGURE NO.:
CF	2C
CHECK BY:	PROJECT NO.:
DJH/JTW	2021-097-21

BASE MAP PROVIDED BY: BING AND KPFF 3.3.2023

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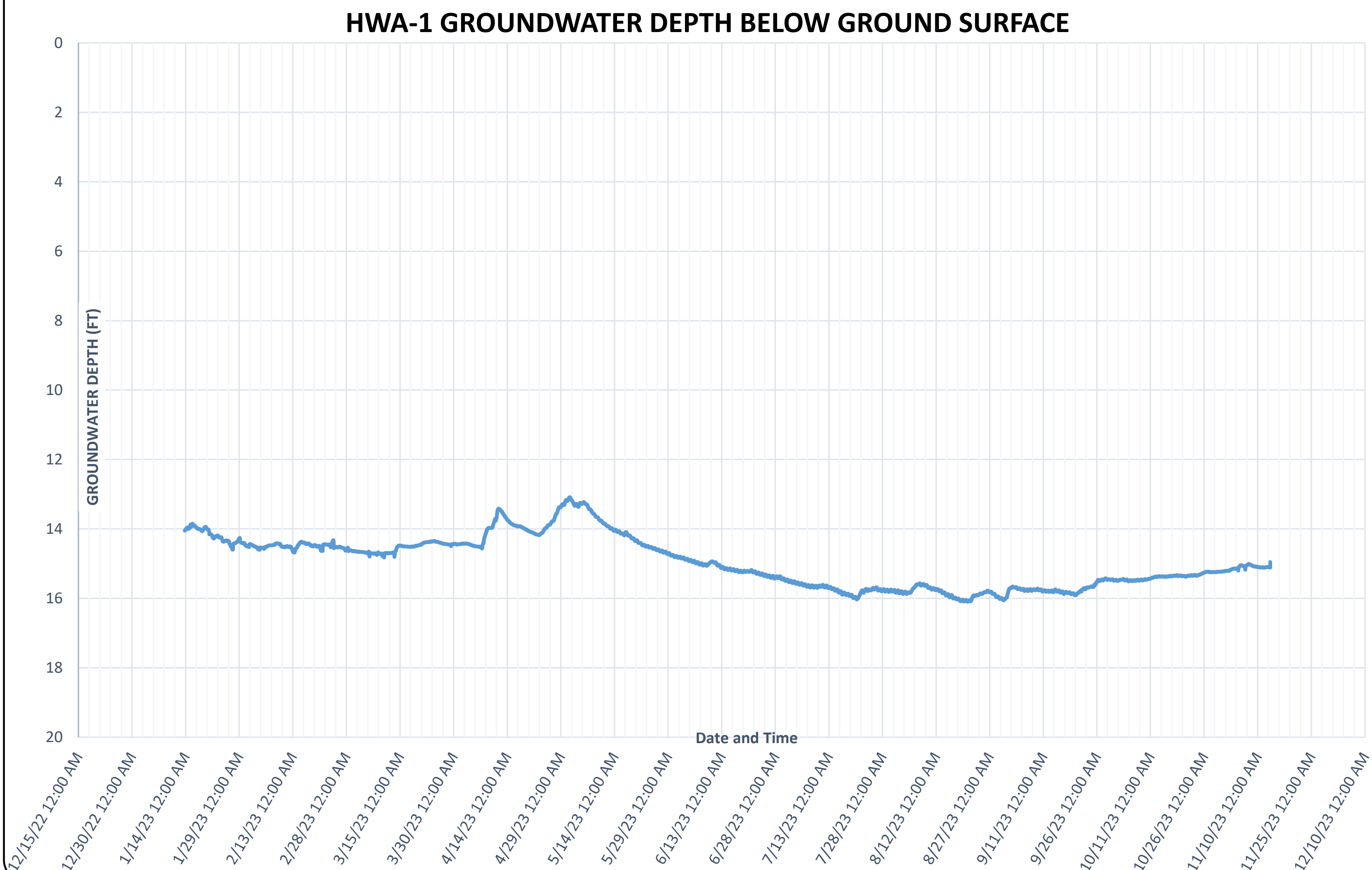


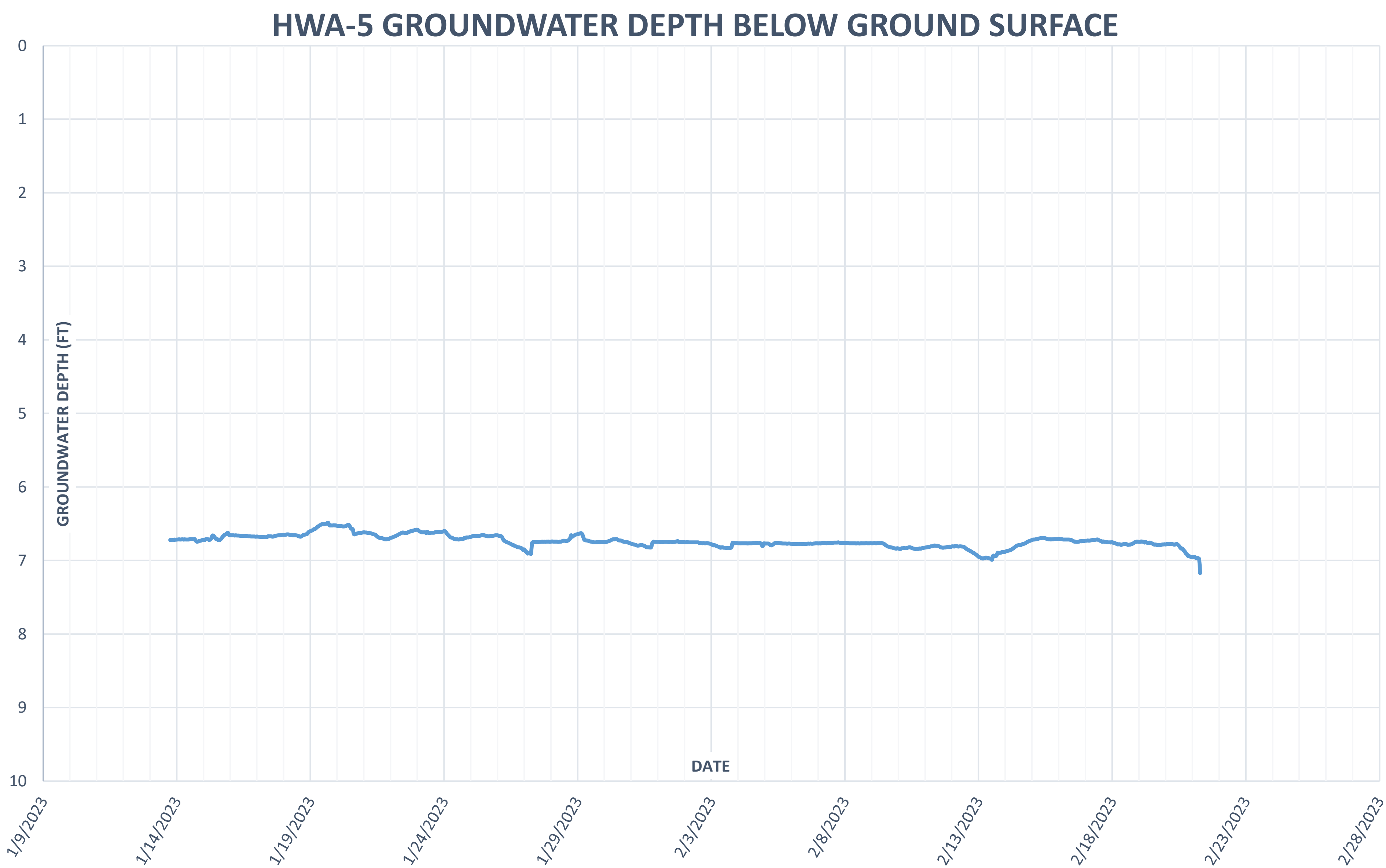
HWA GEOSCIENCES INC. DBE/MWBE

HWA-1 GROUNDWATER LEVEL READINGS

POM POM ROAD IMPROVEMENTS
YAKAMA NATION, WASHINGTON

FIGURE NO. 3A

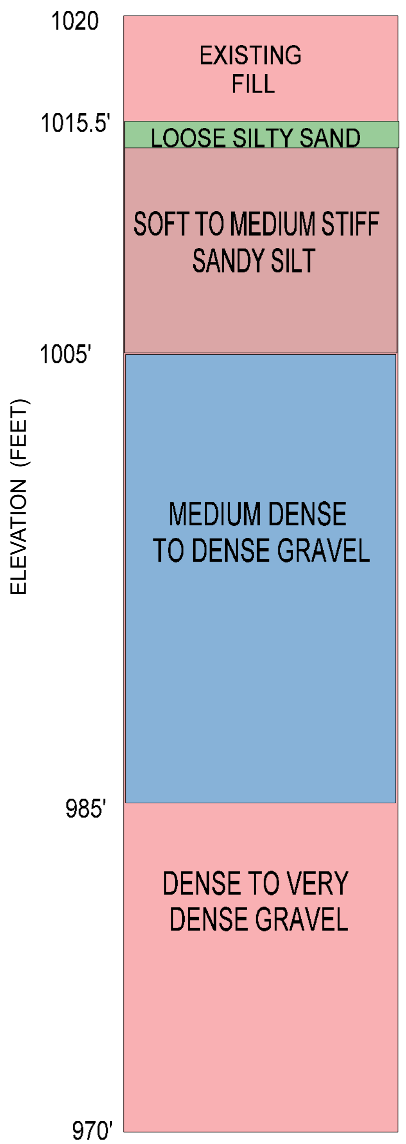
PROJECT NO. 2021-097-21



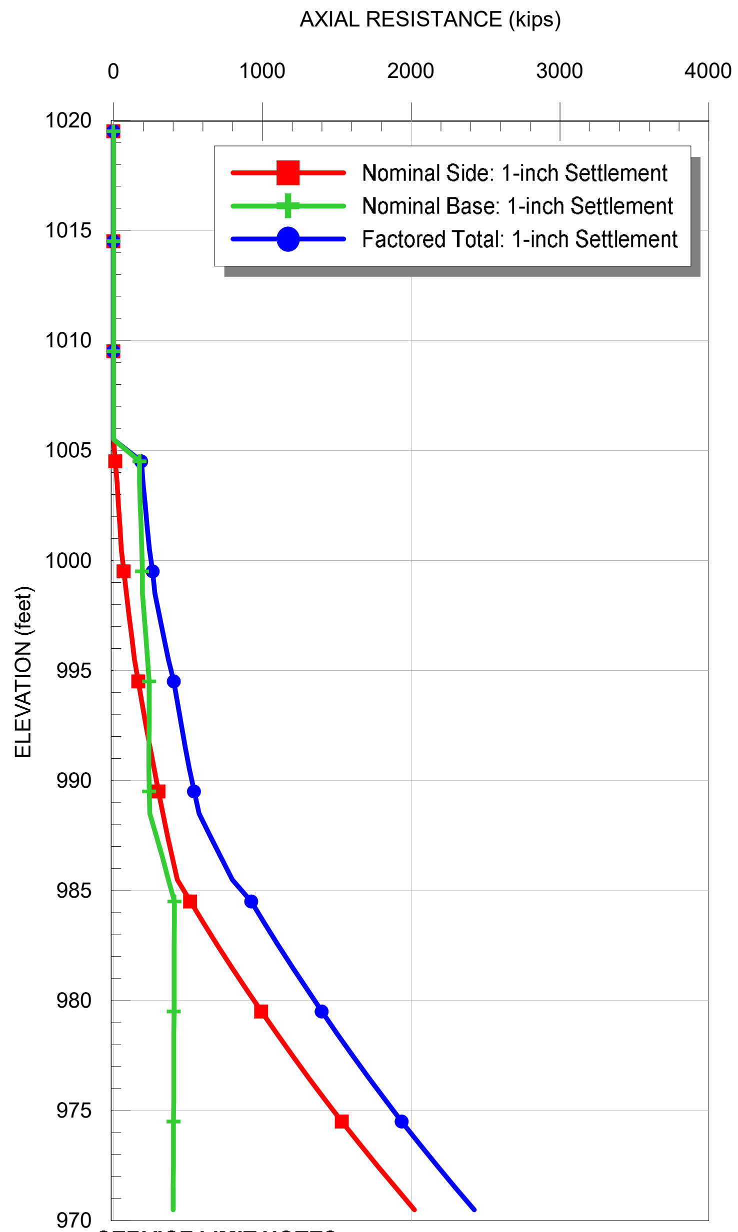
HWA GEOSCIENCES INC.
DBE/MWBE

HWA-5 GROUNDWATER LEVEL READINGS	FIGURE NO.	3B
POM POM ROAD IMPROVEMENTS YAKAMA NATION, WASHINGTON	PROJECT NO.	2021-097-21

ASSUMED SUBSURFACE PROFILE



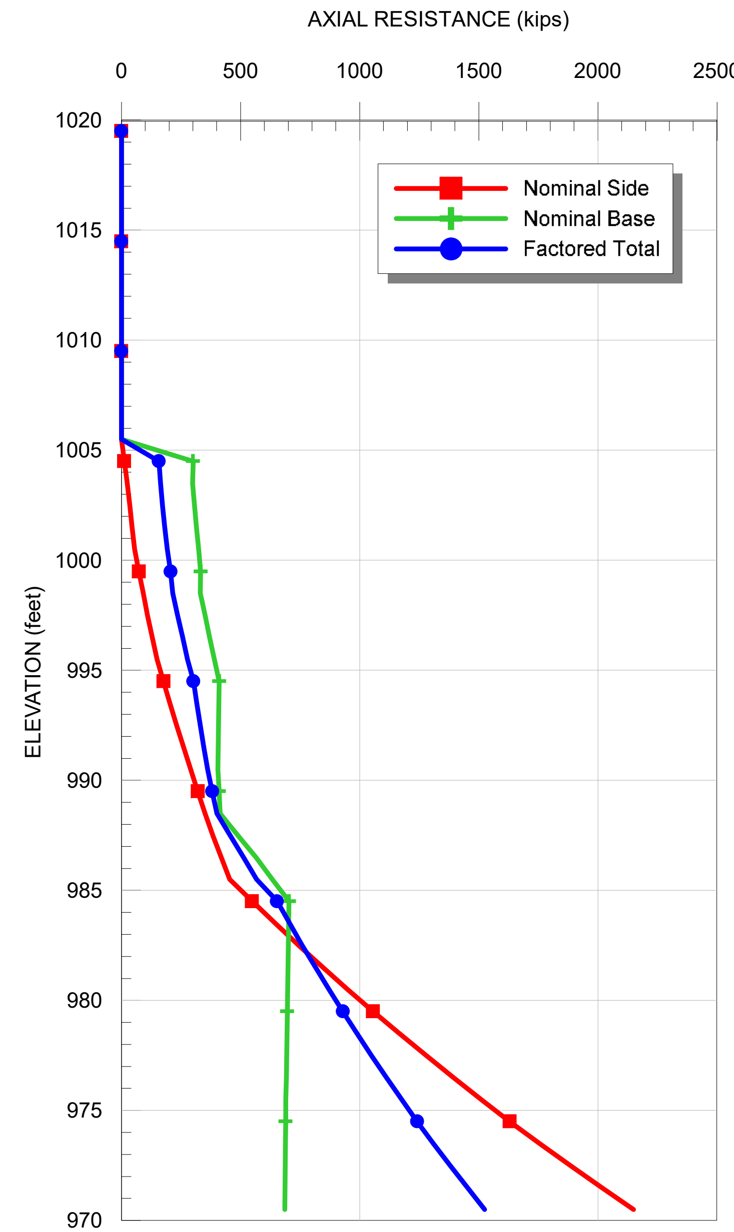
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

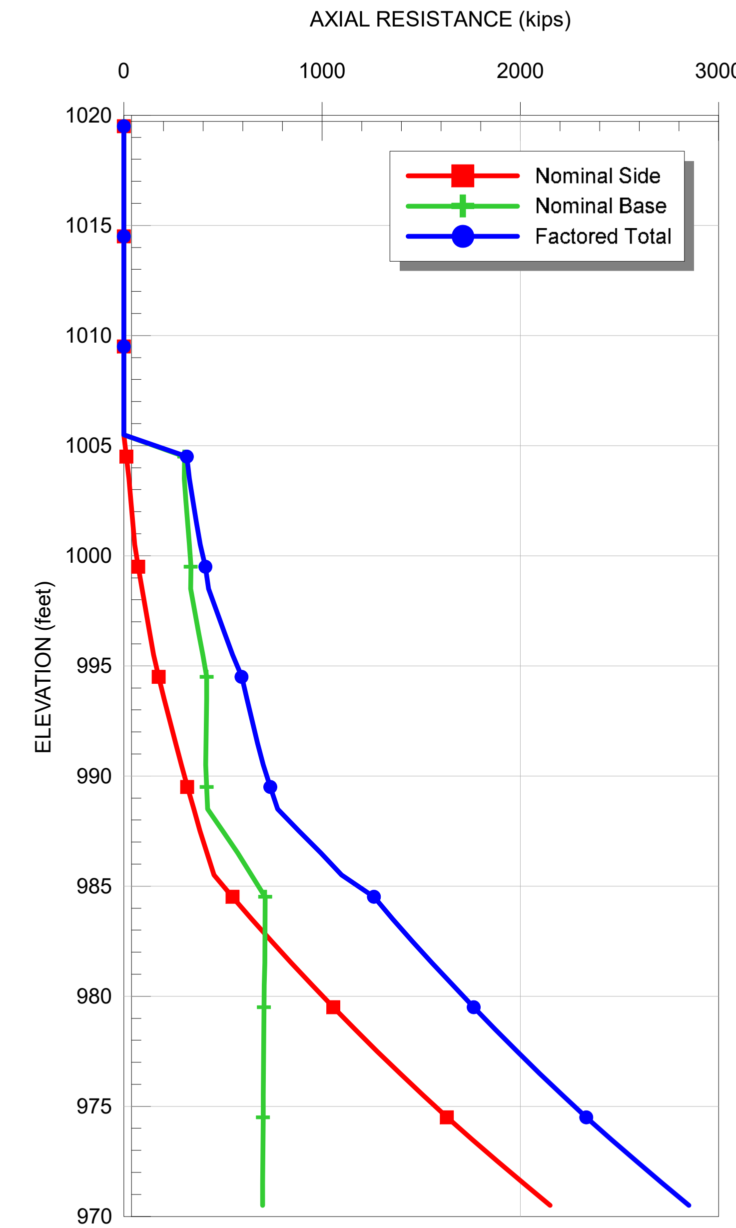
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.

2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.

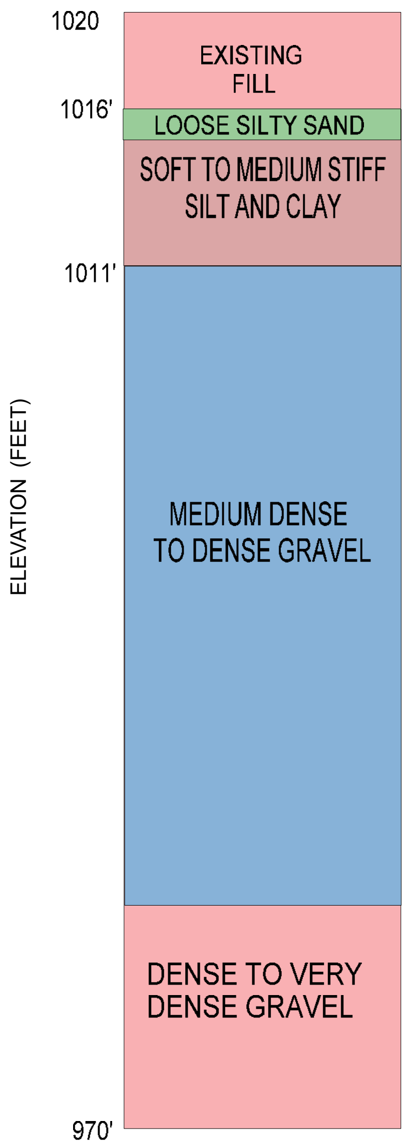
3. The nominal side and base resistance values presented do not include the resistance factors.

4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.

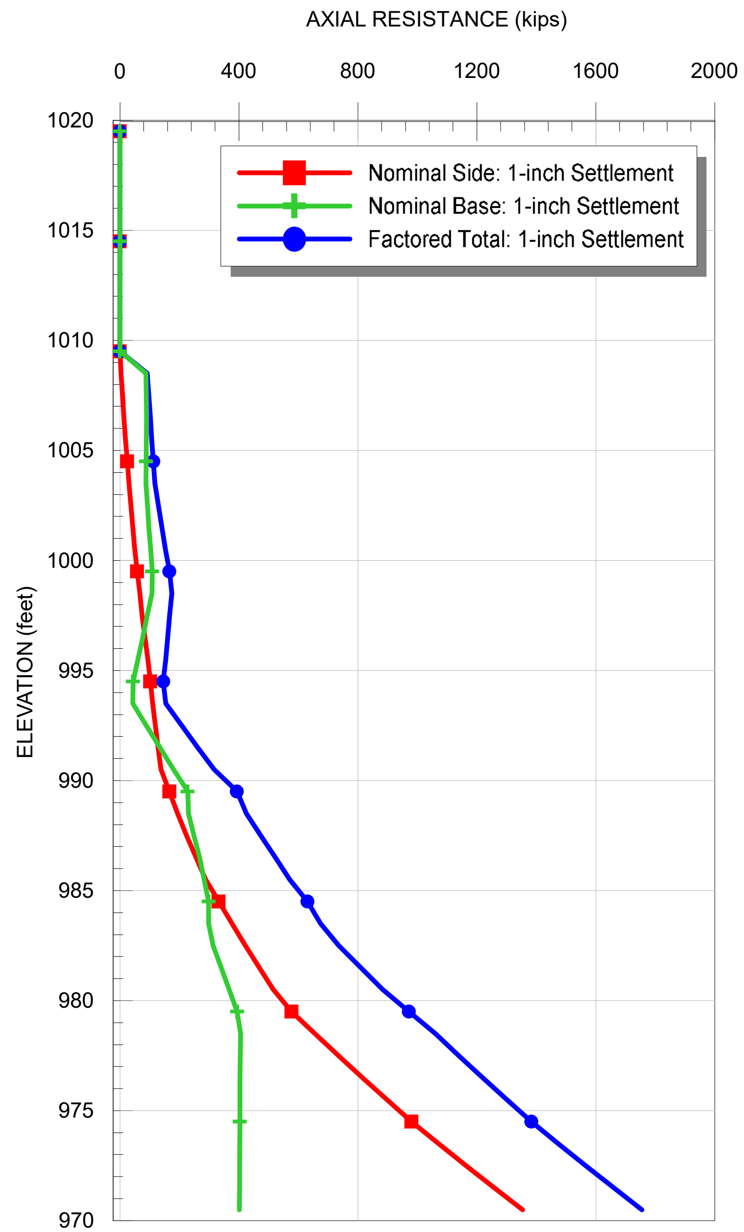
5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GEOSCIENCES INC.	POM POM ROAD IMPROVEMENTS YAKAMA NATION, WASHINGTON	SOUTH BRIDGE ABUTMENT (HWA-4) AXIAL SHAFT CAPACITIES 4-FOOT DIAMETER SHAFTS	DRAWN BY JTW CHECKED BY DJH DATE 2.23.2203	FIGURE NO. 4 PROJECT NO. 2021-097-21

ASSUMED SUBSURFACE PROFILE



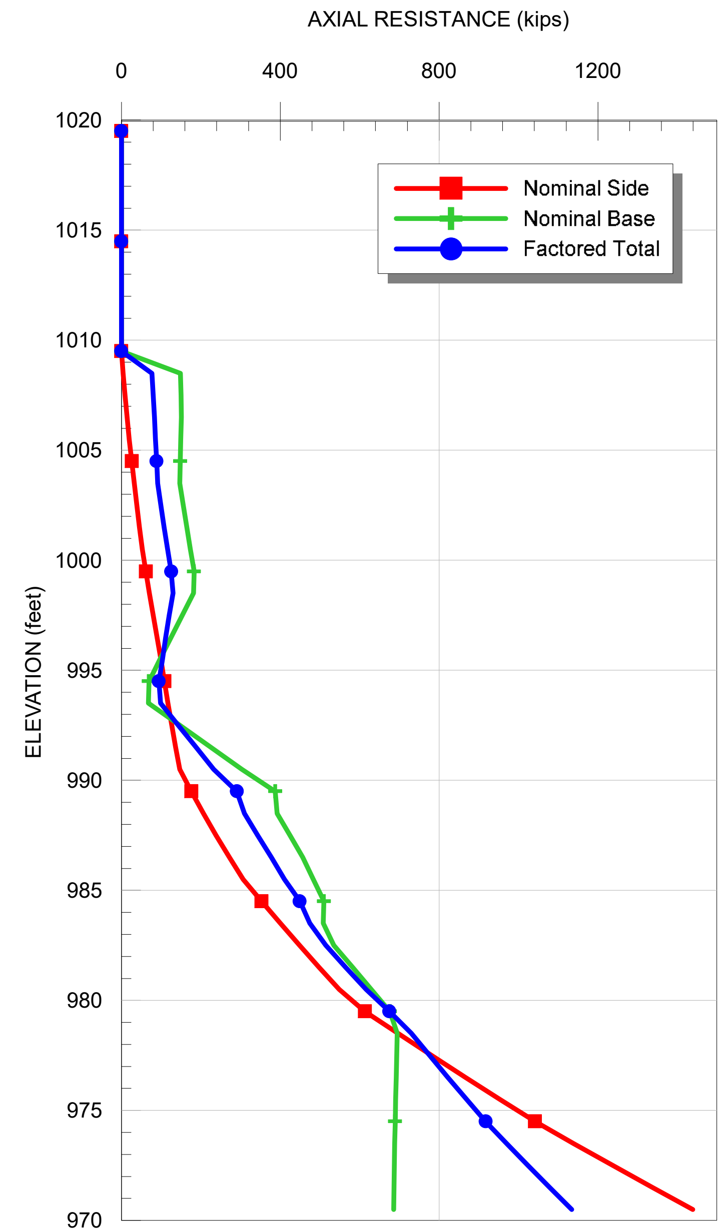
SERVICE LIMIT



SERVICE LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification is 1.0 for side resistance.
2. Settlements is based on a single shaft. No group action is considered.
3. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

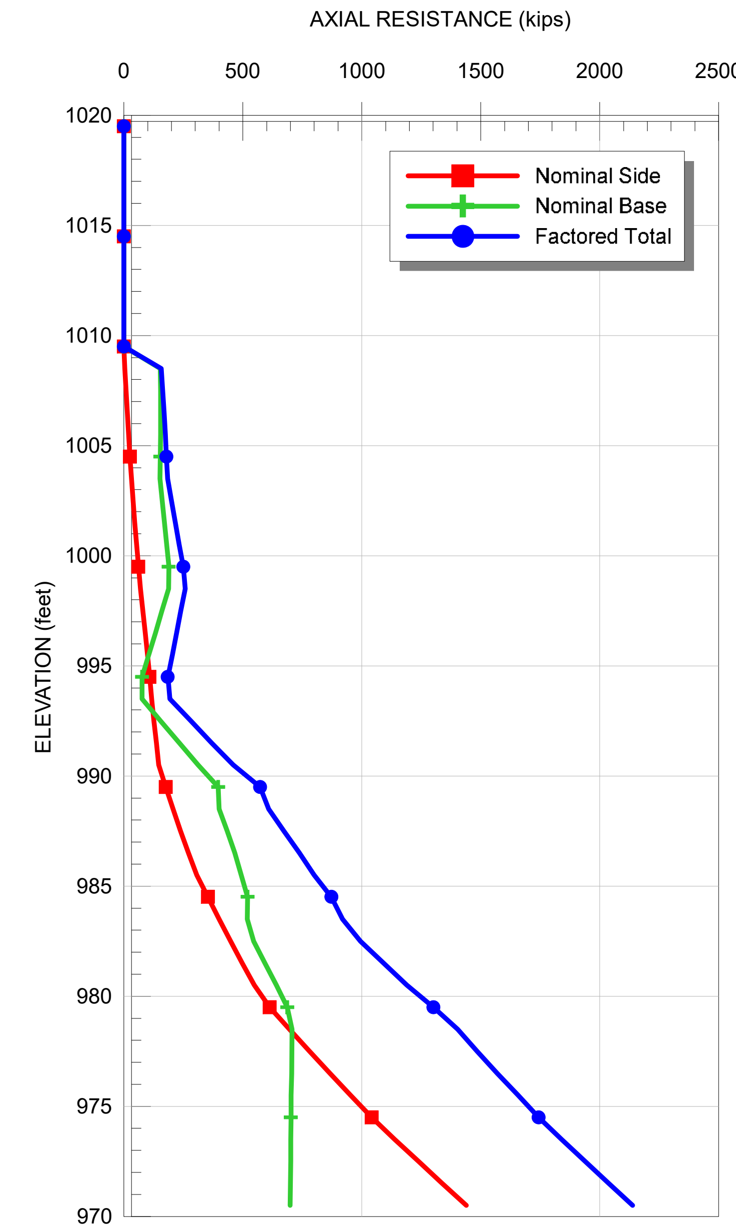
STRENGTH LIMIT



STRENGTH LIMIT NOTES:

1. Recommended resistance factors included in Factored Loads are 0.55 for cohesionless and 0.45 for cohesive for side resistance and 0.5 for cohesionless and 0.4 for cohesive for base resistance, as provided in AASHTO LRFD Bridge Design Specification.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.35 (per AASHTO LRFD Bridge Design Specification).
3. Recommended load factor of 1.25 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

EXTREME EVENT LIMIT



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per AASHTO LRFD Bridge Design Specification for both side and base resistance are 1.0 for compression and 0.8 for uplift.
2. Recommended load factor of 1.0 applied to weight of shaft per AASHTO LRFD Bridge Design Specifications.

GENERAL NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specification and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts.
2. Factored total shaft resistance shown on plots include the summation of the shaft's nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. The nominal side and base resistance values presented do not include the resistance factors.
4. The nominal base and total factored axial capacities provided have been are reduced to account for the weight of drilled shafts with appropriate load factors applied for each limit state.
5. The weight of the drilled shafts is calculated from the proposed shaft top elevation presented on the assumed subsurface profile.

HWA GEOSCIENCES INC.	POM POM ROAD IMPROVEMENTS YAKAMA NATION, WASHINGTON	NORTH BRIDGE ABUTMENT (HWA-5) AXIAL SHAFT CAPACITIES 4-FOOT DIAMETER SHAFTS	DRAWN BY JTW CHECKED BY DJH DATE 2.23.2203	FIGURE NO. 5 PROJECT NO. 2021-097-21

APPENDIX A

FIELD EXPLORATIONS

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 More than 50% Retained on No. 200 Sieve Size	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	Silt and Clay	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILT/Organic CLAY
	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
OC	Organic Content
pH	pH of Soils
PID	Photoionization Device Reading
PP	Pocket Penetrometer (Approx. Comp. Strength, tsf)
Res.	Resistivity
SG	Specific Gravity
CD	Consolidated Drained Triaxial
CU	Consolidated Undrained Triaxial
UU	Unconsolidated Undrained Triaxial
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
- Non-standard Penetration Test (3.0" OD Split Spoon with Brass Rings)
- Small Bag Sample
- Large Bag (Bulk) Sample
- Core Run
- 3-1/4" 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.

Pom Pom Road Improvements
Yakama Nation, Washington

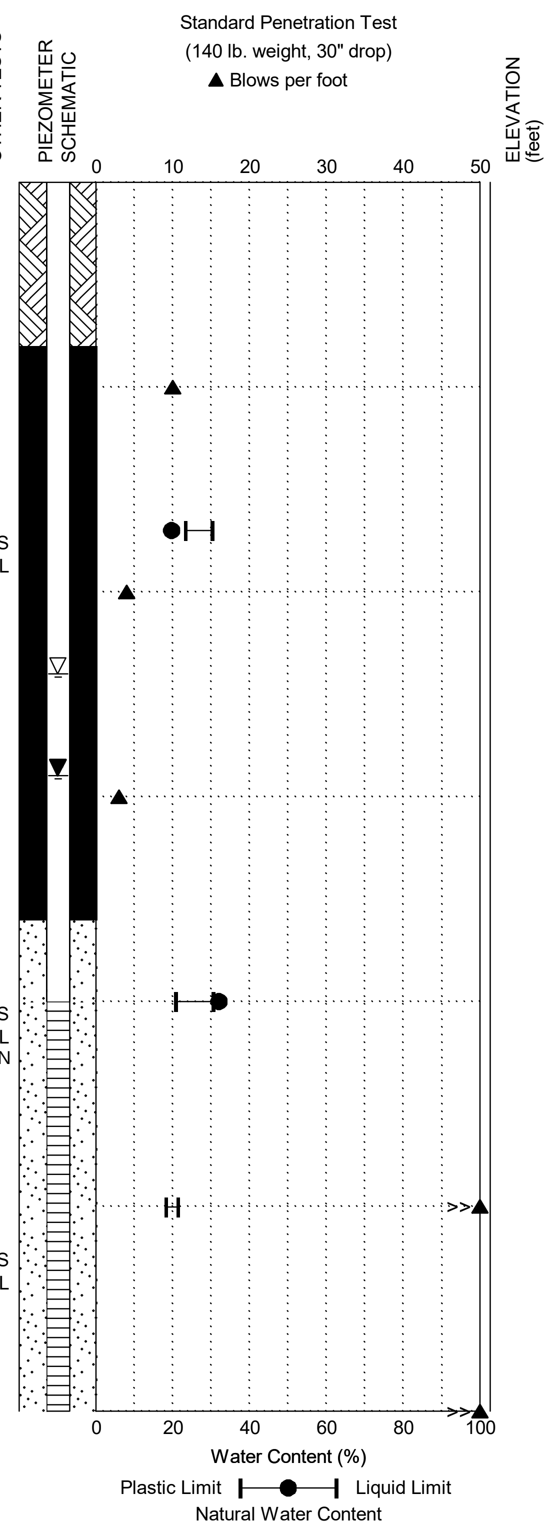
LEGEND OF TERMS AND SYMBOLS USED ON EXPLORATION LOGS

PROJECT NO.: 2021-097-21 FIGURE: A-1

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2A

DATE STARTED: 1/5/2022
DATE COMPLETED: 1/7/2022
LOGGED BY: W. Rosso

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
0	GM	Medium dense, brown, sandy, silty GRAVEL, moist. Fine to coarse angular to subrounded gravel, medium to coarse sand. **(FILL)**	S-1		
			S-2		
5	GP	Medium dense, brownish gray, sandy GRAVEL, moist. Fine to coarse angular to subangular gravel, medium to coarse sand, trace silt.	S-3		
	GM	Medium dense, brown, slightly sandy, silty GRAVEL, moist. Fine to coarse subrounded gravel, fine to medium sand, low plasticity.	S-4	5-6-4	
	SM	Loose, dark brown, gravelly, very silty SAND, moist. Fine to medium sand, low plasticity. **(ALLUVIUM)**	S-5		
			S-6		GS AL
10		Gravel particle size and content increases. Gravel content increases.	S-7	1-1-3	
		Becomes wet.	S-8		
	CL ML	Soft to medium stiff, olive yellow, sandy, silty CLAY, moist. Low to medium plasticity, fine sand, trace fine gravel.	S-9		
			S-10		
15			S-11	1-1-2	
			S-12		
			S-13		
20	CL	Soft to medium stiff, olive brown, sandy CLAY, moist.	S-14		GS AL CN
			S-15		
		Contained coarse rounded gravels.			
	GM	Very dense, greyish brown, sandy, silty GRAVEL, wet. Fine to coarse subrounded gravel, fine to medium sand, low plasticity.			
25		**(OUTBURST FLOOD DEPOSITS)**	S-16	11-27-50/5	
		Hammer bounce.	S-17		GS AL
			S-18		
		Becomes yellowish brown with orange mottling. Becomes reddish brown.			
30					



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.

Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-1

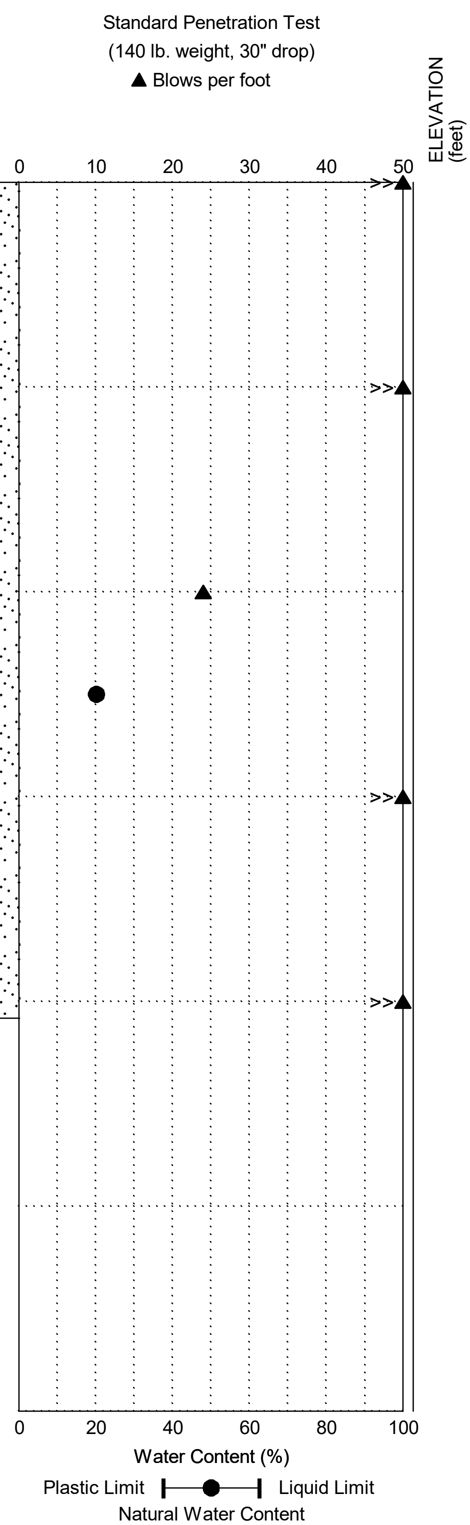
PROJECT NO.: 2021-097-21 FIGURE: A-2

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2A

DATE STARTED: 1/5/2022
DATE COMPLETED: 1/7/2022
LOGGED BY: W. Rosso

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
30		Becomes brown, fines content decreases.	S-19	50/5	
		Becomes yellowish brown, fines content increases.	S-20		
35			S-21	27-40-50/4	
40		Becomes greyish brown, medium dense, plasticity increases.	S-22	7-10-14	
			S-23		
	SM	Medium dense, olive yellow, silty, gravelly SAND, moist. Medium to coarse sand, fine to coarse subrounded to rounded gravel, weakly cemented.	S-24		GS
	GM	Very dense, grayish brown, silty, sandy GRAVEL, moist to wet. Fine to coarse subrounded to rounded gravel, fine to medium sand, low plasticity. Hammer bounce.			
45			S-25	33-50/3	
			S-26		
50		Hammer bounce.	S-27	50/4	
55					
60					

Borehole terminated at approximately 51.5 feet below ground surface (bgs).
Groundwater encountered at approximately 12 feet bgs during drilling.
2-inch diameter PVC Well installed: 20 feet solid PVC riser, 30 feet of PVC screen.
DOE Unique Well #BPL 229



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.

Pom Pom Road Improvements
Yakama Nation, Washington

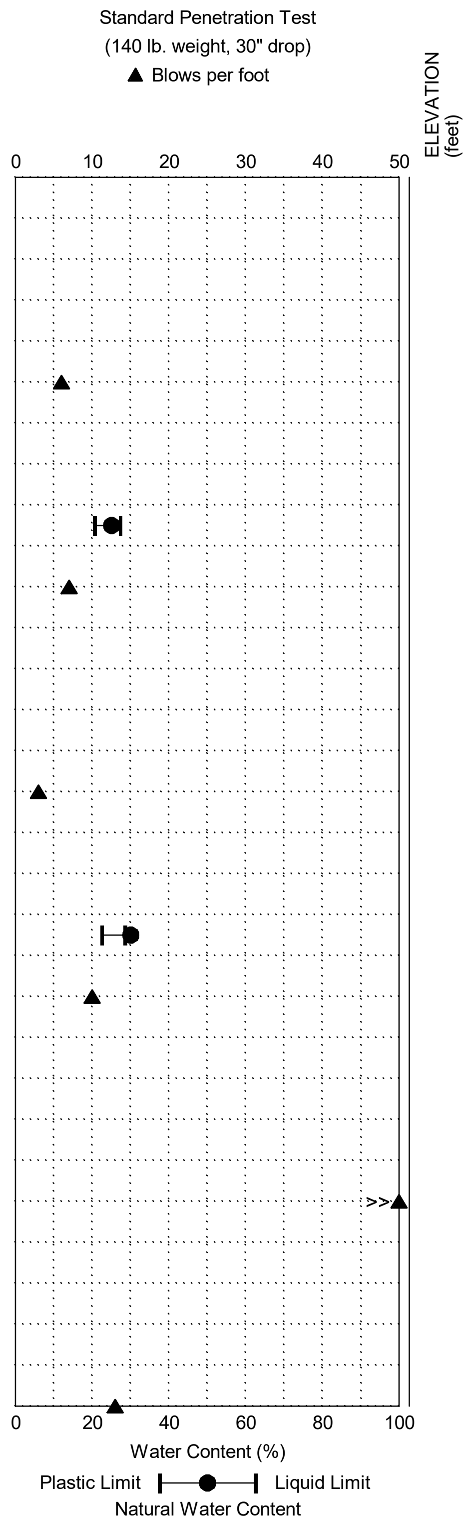
BORING:
HWA-1

PROJECT NO.: 2021-097-21 FIGURE: A-2

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2A

DATE STARTED: 1/5/2022
DATE COMPLETED: 1/5/2022
LOGGED BY: W. Rosso

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GM	Medium dense, brown, silty, sandy GRAVEL, moist. Fine to coarse angular to subrounded gravel, medium to coarse sand. **(FILL)**	S-1			
5	GP	Medium dense, brownish grey, sandy GRAVEL, moist. Fine to coarse angular to subangular gravel, medium to coarse sand, trace silt.	S-2	2-3-3		
	CL ML	Soft, dark brown, sandy, silty CLAY, moist. Fine to medium sand, low plasticity, trace gravel. **(ALLUVIUM)**	S-3			
			S-4		GS AL	
10		Gravel particle size and content increases, becomes wet.	S-5	3-3-4		▽
	CL ML	Soft to medium stiff, olive yellow, slightly sandy, silty CLAY. Low to medium plasticity, fine sand, trace fine gravel.	S-6			
15			S-7	1-1-2		
			S-8		GS AL	
20			S-9	2-2-8		
	GM	Loose to medium dense, dark brown to yellowish brown, silty, sandy GRAVEL, wet. Fine to coarse subrounded gravel, fine to medium sand, low plasticity. **(OUTBURST FLOOD DEPOSITS)**	S-10			
25			S-11	15-21-34		
			S-12			
30		Becomes reddish brown.				



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.

Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-2

PAGE: 1 of 2

PROJECT NO.: 2021-097-21 FIGURE: A-3

BORING-DSM 2021-097.GPJ 6/26/25
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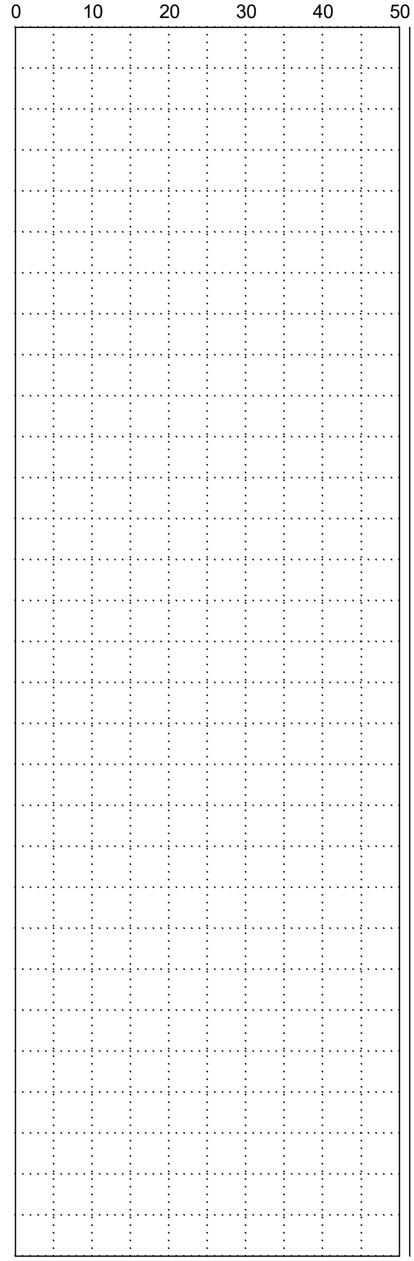
DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2A

DATE STARTED: 1/5/2022
DATE COMPLETED: 1/5/2022
LOGGED BY: W. Rosso

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30					S-13	5-6-7		
			Borehole terminated at approximately 31.5 feet below ground surface (bgs). Groundwater encountered at approximately 11 feet bgs during drilling. Boring abandoned with 3/8" bentonite chips.					
35								
40								
45								
50								
55								
60								



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.

Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-2

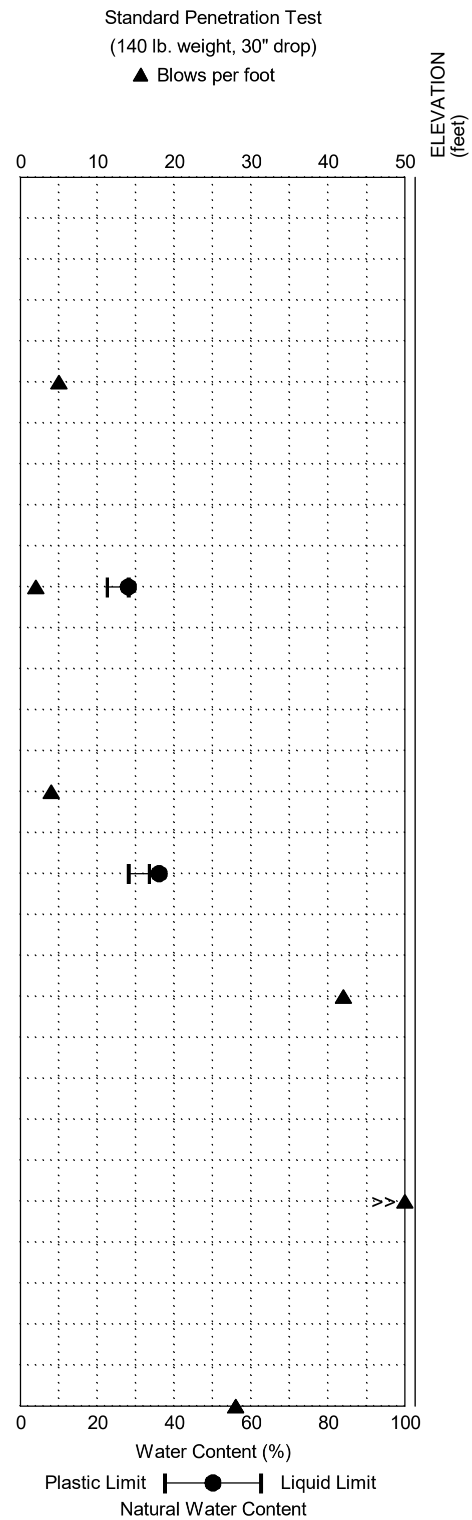
PROJECT NO.: 2021-097-21 FIGURE: A-3

BORING-DSM 2021-097.GPJ 6/26/25
Library: Q:\LIBRARY\LIBRARY - BOTHELL BACKUP BACKUP.GLB

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2B

DATE STARTED: 1/6/2022
DATE COMPLETED: 1/6/2022
LOGGED BY: W. Rosso

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GM	Medium dense, brown, silty, sandy GRAVEL, moist. Fine to coarse angular to subrounded gravel, medium to coarse sand. **(FILL)**				
			S-1			
5	GP GM	Medium dense, brownish grey, slightly silty, sandy GRAVEL, moist. Fine to coarse angular to subangular gravel, medium to coarse sand.		3-2-3		
	SM	Loose, dark brown, slightly gravelly, very silty SAND, moist. Fine to medium sand, low plasticity, fine to coarse subangular gravel. **(ALLUVIUM)**	S-2			
	CL ML	Soft to medium stiff, olive yellow, sandy, silty CLAY, moist. Low to medium plasticity, fine sand, trace fine gravel.				
10		trace dark brown sand and gravel from 10 to 12 feet bgs	S-3	1-1-1	GS AL	
						▽
		Becomes wet.	S-4			
15		Sand content increases, moisture content decreases, orange mottling.	S-5	1-1-3		
	ML	Soft to medium stiff, olive yellow, sandy SILT, moist.	S-6		GS AL	
20			S-7	7-7-35		
	GM	Medium dense to very dense, dark brown to yellowish brown, slightly sandy, silty GRAVEL, wet. Fine to coarse subrounded gravel, fine to medium sand, low plasticity. **(OUTBURST FLOOD DEPOSITS)**				
			S-8			
25		Hammer bounce.	S-9	27-50/5		
		Becomes reddish brown.	S-10			
30						



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.

Pom Pom Road Improvements
Yakama Nation, Washington

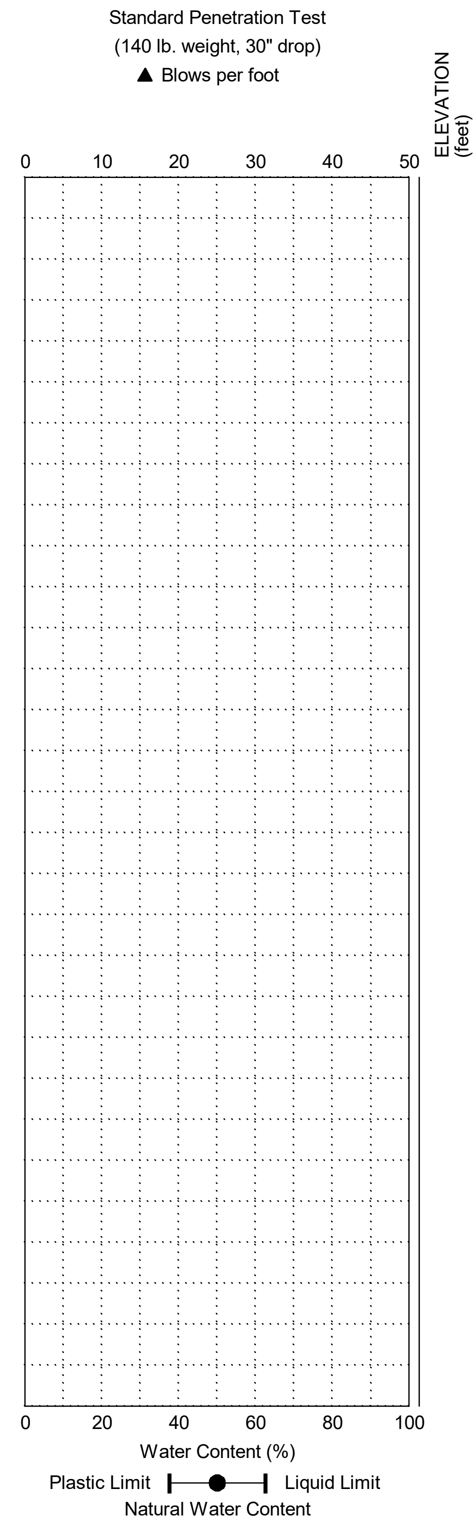
BORING:
HWA-3

PROJECT NO.: 2021-097-21 FIGURE: A-4

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2B

DATE STARTED: 1/6/2022
DATE COMPLETED: 1/6/2022
LOGGED BY: W. Rosso

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30					S-11	6-13-15		
			Borehole terminated at approximately 31.5 feet below ground surface (bgs). Groundwater encountered at approximately 13.5 feet bgs during drilling. Boring abandoned with 3/8" bentonite chips.					
35								
40								
45								
50								
55								
60								



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.

Pom Pom Road Improvements
Yakama Nation, Washington

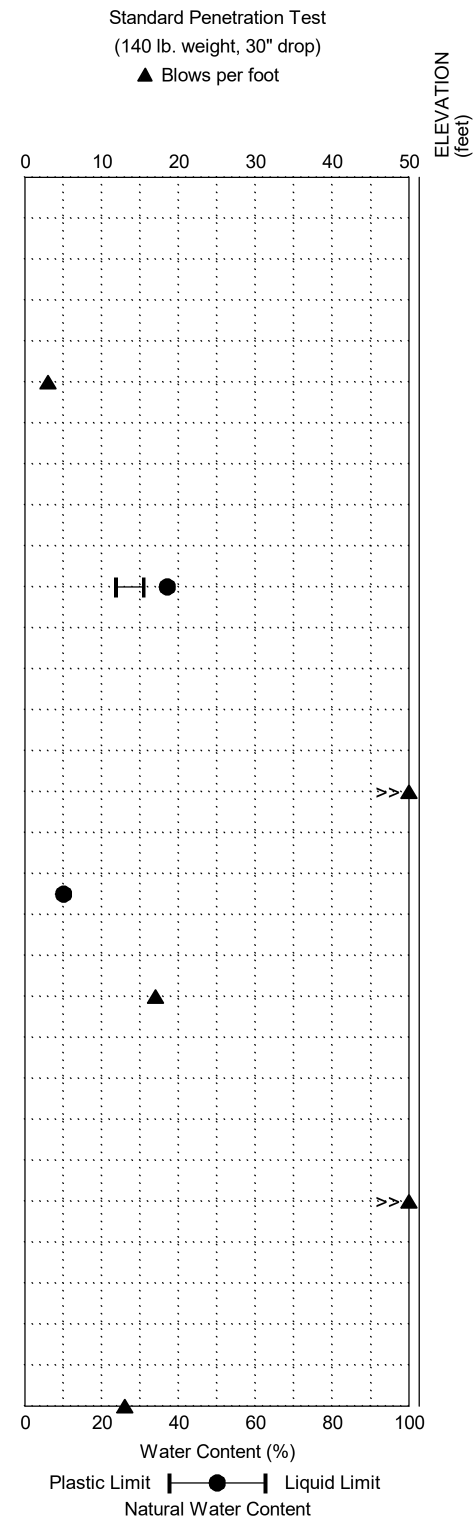
BORING:
HWA-3

PROJECT NO.: 2021-097-21 FIGURE: A-4

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2C

DATE STARTED: 1/6/2022
DATE COMPLETED: 1/7/2022
LOGGED BY: W. Rosso

DEPTH (feet)	USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
0	GP GM	Medium dense, brown, slightly silty, slightly sandy GRAVEL, moist. Fine to coarse angular to subrounded gravel, medium to coarse sand. **(FILL)**				
		Becomes brownish grey	S-1			
5	SM	Loose, dark brown, slightly gravelly, very silty sand, moist. Fine to medium sand, low plasticity, fine to coarse subangular gravel. **(ALLUVIUM)**		2-1-2		
	ML	Soft to medium stiff, olive yellow, sandy SILT, moist, low plasticity, fine sand, trace fine gravel.	S-2			
			S-3			
10		Becomes very dark brown. Coarse angular gravel in sample.	S-4		GS AL CN	▽
15	GP GM	Medium dense to very dense, dark brown to yellowish brown, slightly silty, slightly sandy GRAVEL, wet. Fine to coarse subrounded gravel, fine to medium sand, low plasticity. **(OUTBURST FLOOD DEPOSITS)**	S-5	4-30-34		
		Medium dense to very dense, dark brown to yellowish brown, slightly silty, sandy GRAVEL, wet.	S-6		GS	
20		Moisture content decreases, fines content increases, becomes grey. Fines content decreases, moisture content increases.	S-7	4-6-11		
		Becomes reddish brown.				
25		Hammer bounce.	S-8	21-30-33		
30		Becomes grey.				



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.

Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-4

PAGE: 1 of 2

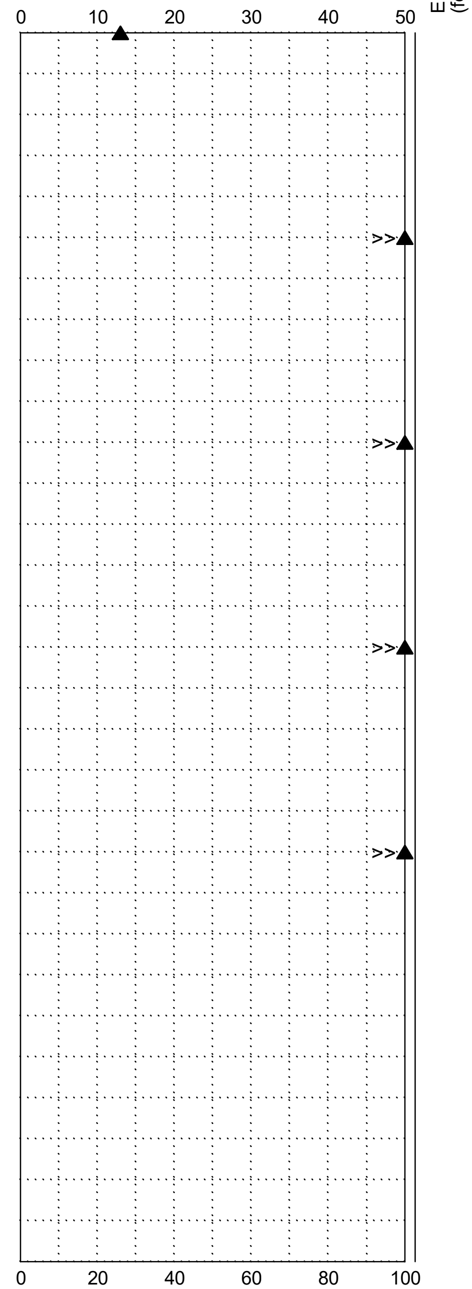
PROJECT NO.: 2021-097-21 FIGURE: A-5

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2C

DATE STARTED: 1/6/2022
DATE COMPLETED: 1/7/2022
LOGGED BY: W. Rosso

DEPTH (feet)	SYMBOL	USCS SOIL CLASS	DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS	GROUNDWATER
30			Moisture content increases.		S-9	8-11-10		
			Moisture content decreases>					
35			Hammer bounce.		S-10	2-15-50/1		
			Drilling effort increaes, coarse angular gravel fragments in sample.					
40			Becomes grey, moisture content decreases.		S-11	4-15-50/4		
			4 inch rock core through larger cobble or boulder.					
45			Hammer bounce.		S-12	27-50/1		
			Dry yellow gravel and sand lense 48.7-49. Becomes red brown, moist.					
50			Hammer bounce.		S-13	50/4		
			Borehole terminated at approximately 51.5 feet below ground surface (bgs). Groundwater encountered at approximately 10 feet bgs during drilling. Boring abandoned with 3/8" bentonite chips.					
55								
60								

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot



Water Content (%)

Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

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.

Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-4

PROJECT NO.: 2021-097-21 FIGURE: A-5

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2C

DATE STARTED: 1/4/2022
DATE COMPLETED: 1/6/2022
LOGGED BY: W. Rosso

USCS SOIL CLASS	DESCRIPTION	SAMPLE NUMBER	PEN. RESISTANCE (blows/6 inches)	OTHER TESTS
GP	Medium dense, brown, slightly sandy GRAVEL, moist. Fine to coarse angular to subangular gravel, medium to coarse sand, trace silt. **(FILL)** Coarse gravel fraction increases, gravel becomes subangular to subrounded.	S-1, S-2		
SM	Loose, dark brown, very silty SAND, moist, fine to medium sand, low plasticity **(ALLUVIUM)**	S-3, S-4	3-2-1	
CL ML	Soft to medium stiff, olive yellow, slightly sandy, silty CLAY, moist. Low to medium plasticity, fine sand, trace fine gravel.	S-5, S-6		
	Gravel particle size and content increases, becomes wet.	S-7	1-1-2	
GM	Loose, dark brown, silty, sandy GRAVEL, wet. Fine to coarse subrounded gravel, fine to medium sand, nonplastic to low plasticity, cobbles up to 4 inches. **(OUTBURST FLOOD DEPOSITS)**	S-8		GS AL
GP	Loose to medium dense, slightly sandy GRAVEL, wet. Fine to coarse subangular to subrounded gravels, fine to medium sand.	S-9	2-2-7	
	Becomes brownish grey, fine gravel content increases.	S-10	3-10-19	
	Medium dense, brownish grey, GRAVEL, moist.	S-11, S-12		GS
GC	Loose to dense, dark brown to yellowish brown, sandy, clayey GRAVEL, wet. Fine to coarse subrounded gravel, fine to medium sand, low plasticity.	S-13	3-3-2	
		S-14		

Standard Penetration Test
(140 lb. weight, 30" drop)
▲ Blows per foot

Water Content (%)
Plastic Limit ⊢●⊣ Liquid Limit
Natural Water Content

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.

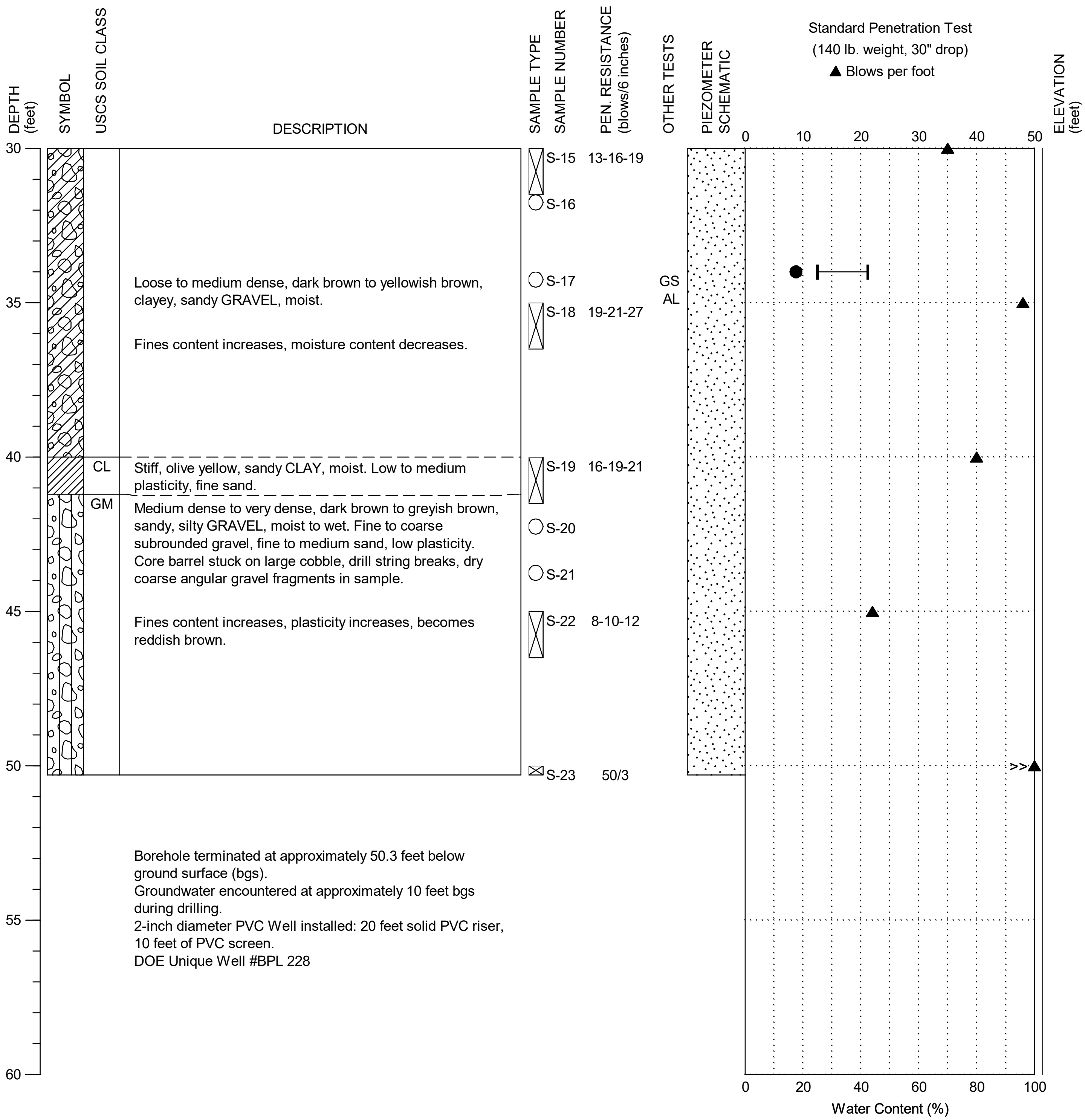
Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-5

PROJECT NO.: 2021-097-21 FIGURE: A-6

DRILLING COMPANY: Holt Services, Inc.
DRILLING METHOD: Sonic - TSi 150CC
SAMPLING METHOD: SPT W/Autohammer
LOCATION: See Figure 2C

DATE STARTED: 1/4/2022
DATE COMPLETED: 1/6/2022
LOGGED BY: W. Rosso



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.

Pom Pom Road Improvements
Yakama Nation, Washington

BORING:
HWA-5

PAGE: 2 of 2

PROJECT NO.: 2021-097-21 FIGURE: A-6

APPENDIX B

LABORATORY TESTING

APPENDIX B

Laboratory Testing

Representative soil samples obtained from our explorations were placed in plastic bags to prevent loss of moisture and transported to our Bothell, Washington, laboratory for further examination and testing. Laboratory tests were conducted on selected soil samples to characterize relevant engineering and index properties of the site soils. Laboratory testing was conducted as described below: A Summary of Material Properties is provided on Figure B-1.

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 and on the Summary of Materials Properties 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 6913. 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.

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, Figures B-7 and B-8.

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOIL: The consolidation properties of selected compressible soil samples were measured in general accordance with ASTM D 2435. Saturation was maintained by inundation of the sample throughout the test. The samples were subjected to increasing increments of total stress, the duration of which was selected to exceed the time required for completion of primary consolidation as defined in the Standard, Method B. The test results are presented on the attached One-Dimensional Consolidation reports, Figures B-9 through B-17.

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	ATTERBERG LIMITS (%)			% COBBLES	% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
					LL	PL	PI						
HWA-1,S-6	8.5	9.0	19.6		30	23	7		17.8	36.9	45.3	SM	Dark brown, silty SAND with gravel
HWA-1,S-14	20.0	21.0	32.0		30	20	10			12.6	87.4	CL	Olive-brown, lean CLAY
HWA-1,S-17	25.0	27.5	11.3		21	18	3		57.7	29.7	12.6	GM	Grayish-brown, silty GRAVEL with sand
HWA-1,S-24	42.5	45.0	20.1						63.6	20.2	16.2	GM	Yellowish-brown, silty GRAVEL with sand
HWA-2,S-4	8.5	9.0	24.8		27	20	7		0.4	37.2	62.4	CL-ML	Light olive-brown, sandy silty CLAY
HWA-2,S-8	18.5	19.0	30.4		29	22	7			22.1	77.9	CL-ML	Light olive-brown, silty CLAY with sand
HWA-3,S-3	10.0	11.5	27.7		28	22	6			15.1	84.9	CL-ML	Light olive-brown, silty CLAY with sand
HWA-3,S-6	17.0	17.5	36.1		33	28	5			15.7	84.3	ML	Light olive-brown, SILT with sand
HWA-4,S-4	10.0	12.0	37.3		31	24	7		1.0	30.6	68.4	ML	Very dark brown, sandy SILT
HWA-4,S-6	17.5	20.0	10.0					5.3	75.1	12.7	6.9	GP-GM	Olive-brown, poorly graded GRAVEL with silt and cobbles
HWA-5,S-8	12.5	15.0	10.7		30	23	7	13.3	68.5	11.1	7.1	GP-GM	Olive-brown, poorly graded GRAVEL with silt and cobbles
HWA-5,S-12	22.0	24.0	10.1						82.8	13.3	3.9	GP	Dark olive-brown, poorly graded GRAVEL
HWA-5,S-17	34.0	34.5	17.5		42	24	18		68.8	17.1	14.2	GC	Dark yellowish-brown, clayey GRAVEL with sand

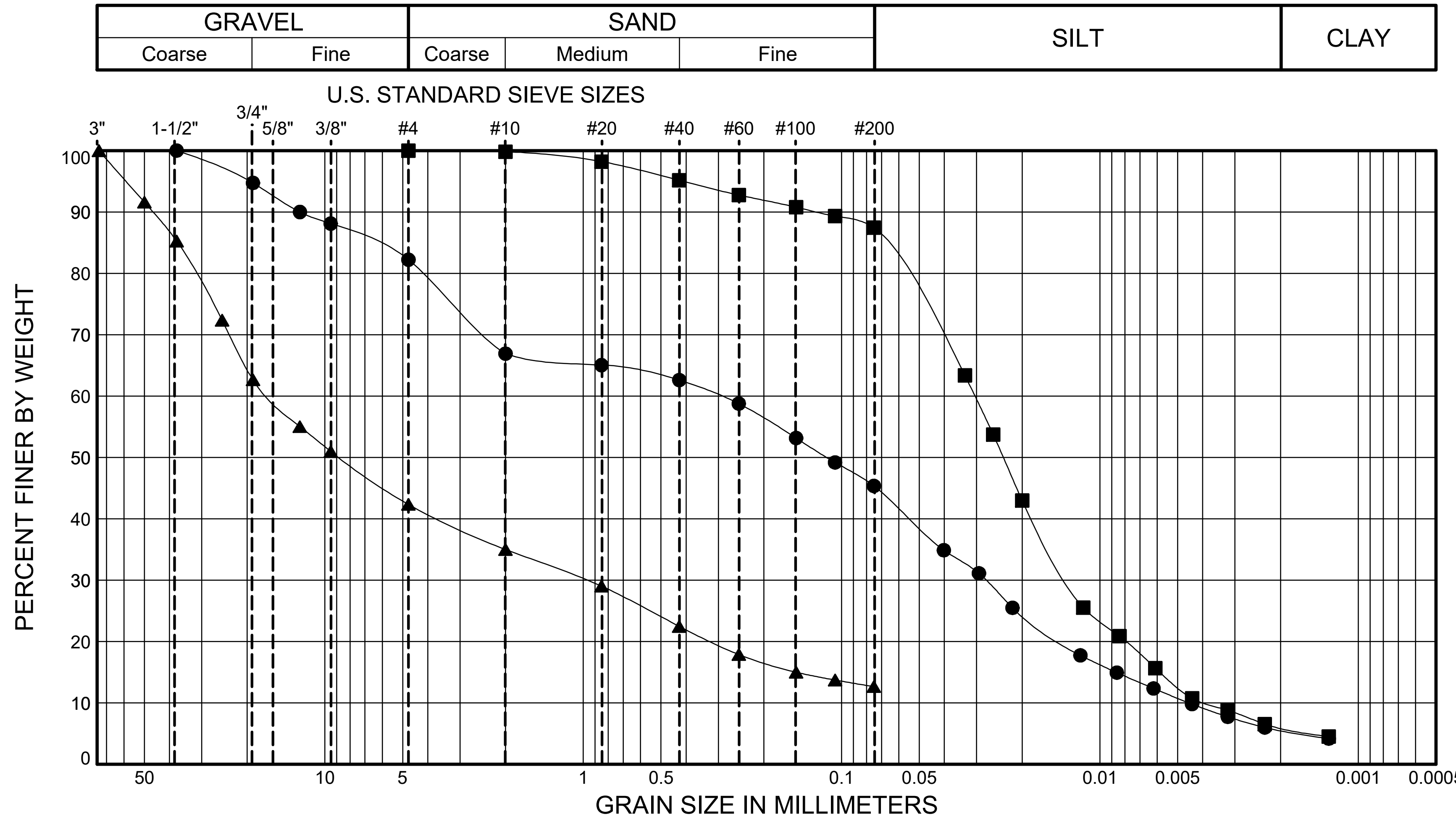
Notes:
1. This table summarizes information presented elsewhere in the report and should be used in conjunction with the report test, other graphs and tables, and the exploration logs.
2. The soil classifications in this table are based on ASTM D2487 and D2488 as applicable.

HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

SUMMARY OF
MATERIAL PROPERTIES

PROJECT NO.: 2021-097-21 FIGURE: B-1



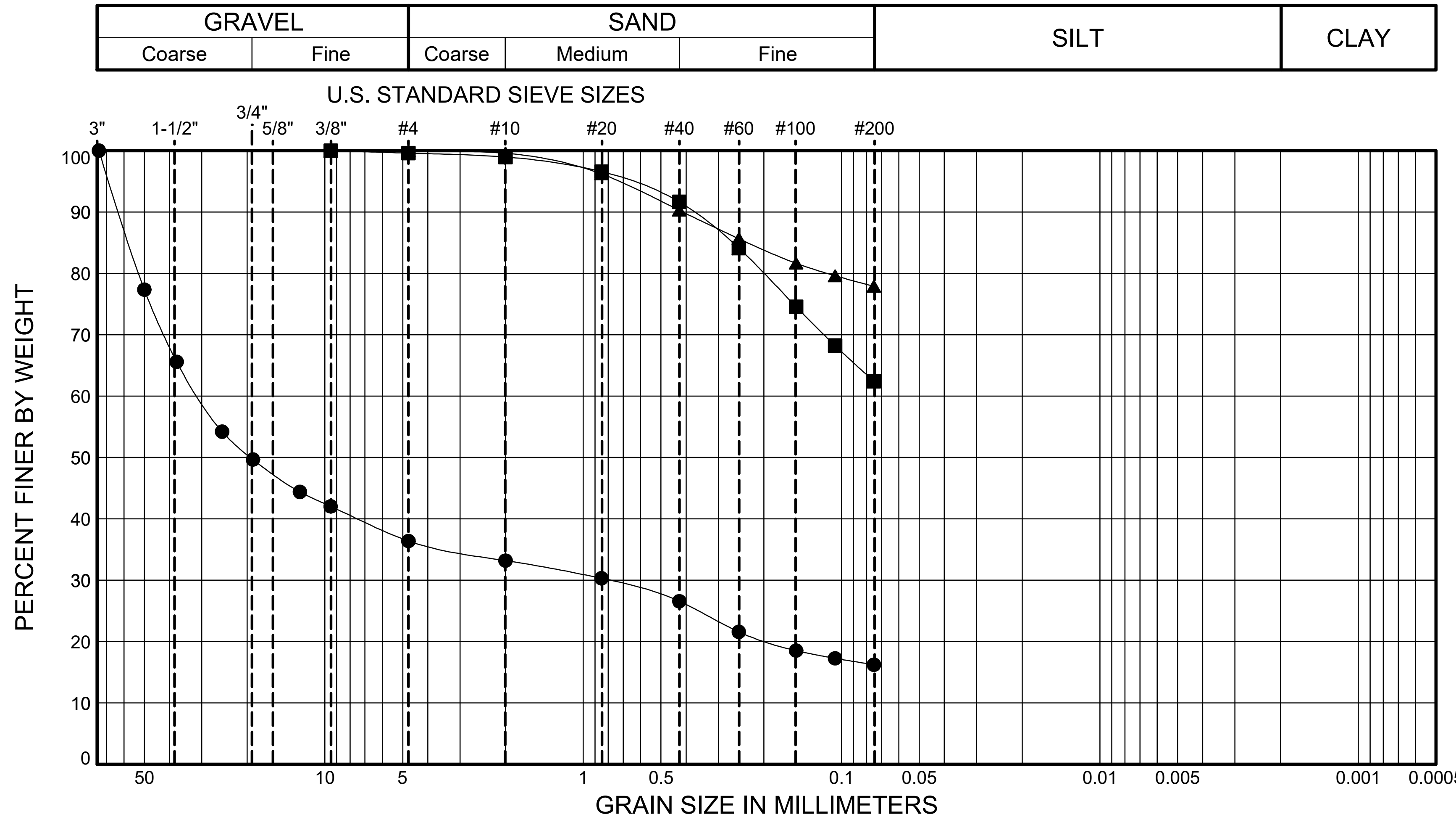
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Cobble %	Gravel %	Sand %	Silt %	Clay %	Fines %
●	HWA-1	S-6	8.5 - 9.0	(SM) Dark brown, silty SAND with gravel	20	30	23	7		17.8	36.9	39.8	5.5	
■	HWA-1	S-14	20.0 - 21.0	(CL) Olive-brown, lean CLAY	32	30	20	10			12.6	81.4	6.0	
▲	HWA-1	S-17	25.0 - 27.5	(GM) Grayish-brown, silty GRAVEL with sand	11	21	18	3	0.0	57.7	29.7			12.6

HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHODS ASTM D6913/D7928

PROJECT NO.: 2021-097-21 FIGURE: B-2



SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Cobble %	Gravel %	Sand %	Silt %	Clay %	Fines %
●	HWA-1	S-24	42.5 - 45.0	(GM) Yellowish-brown, silty GRAVEL with sand	20				0.0	63.6	20.2			16.2
■	HWA-2	S-4	8.5 - 9.0	(CL-ML) Light olive-brown, sandy silty CLAY	25	27	20	7		0.4	37.2			62.4
▲	HWA-2	S-8	18.5 - 19.0	(CL-ML) Light olive-brown, silty CLAY with sand	30	29	22	7			22.1			77.9

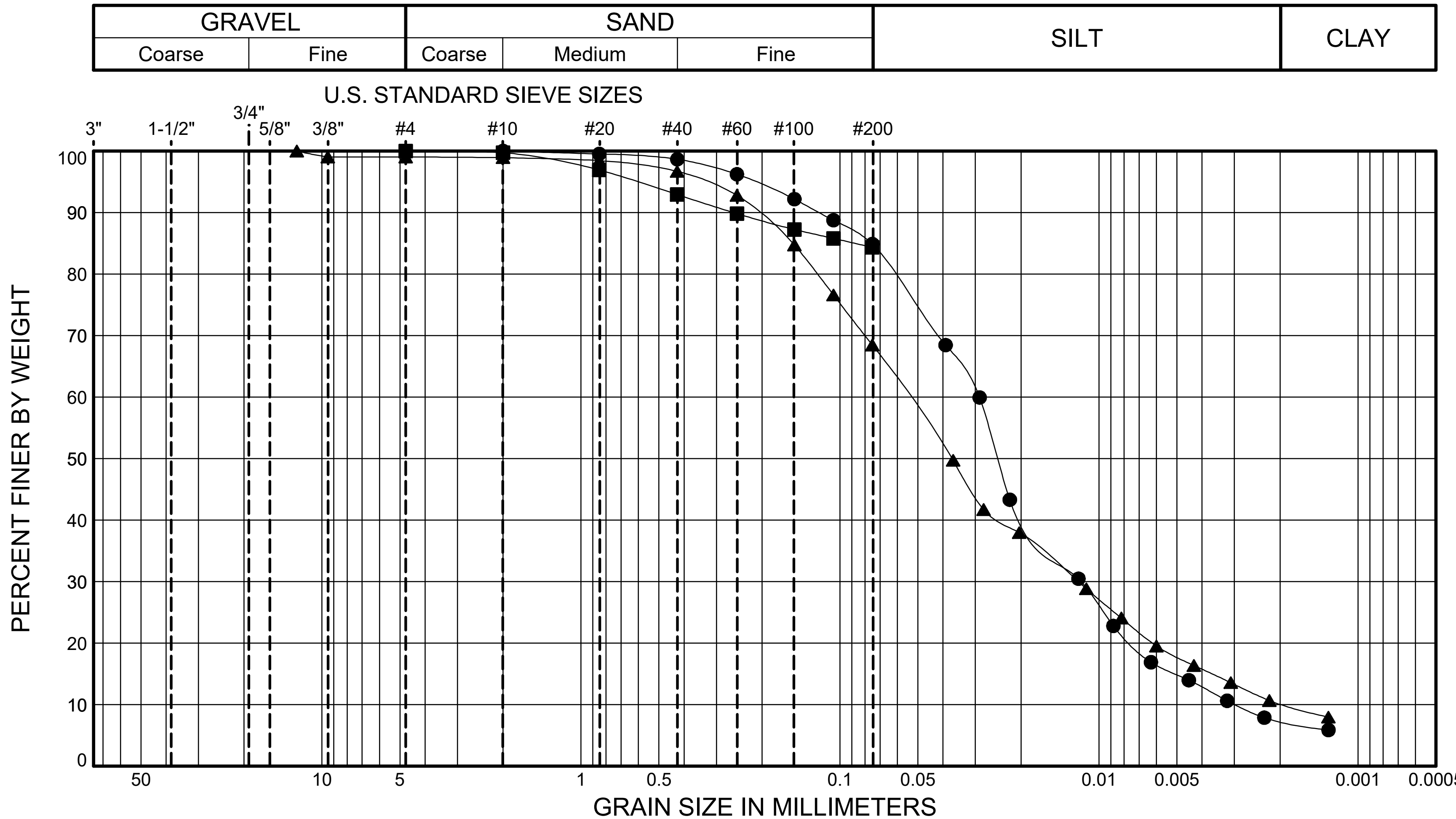
HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

PARTICLE-SIZE ANALYSIS
OF SOILS
METHOD ASTM D6913

PROJECT NO.: 2021-097-21 FIGURE: B-3

HWAGRSZ SILT-CLAY PERCENTAGE W/COBBLES 2021-097.GPJ 2/20/23



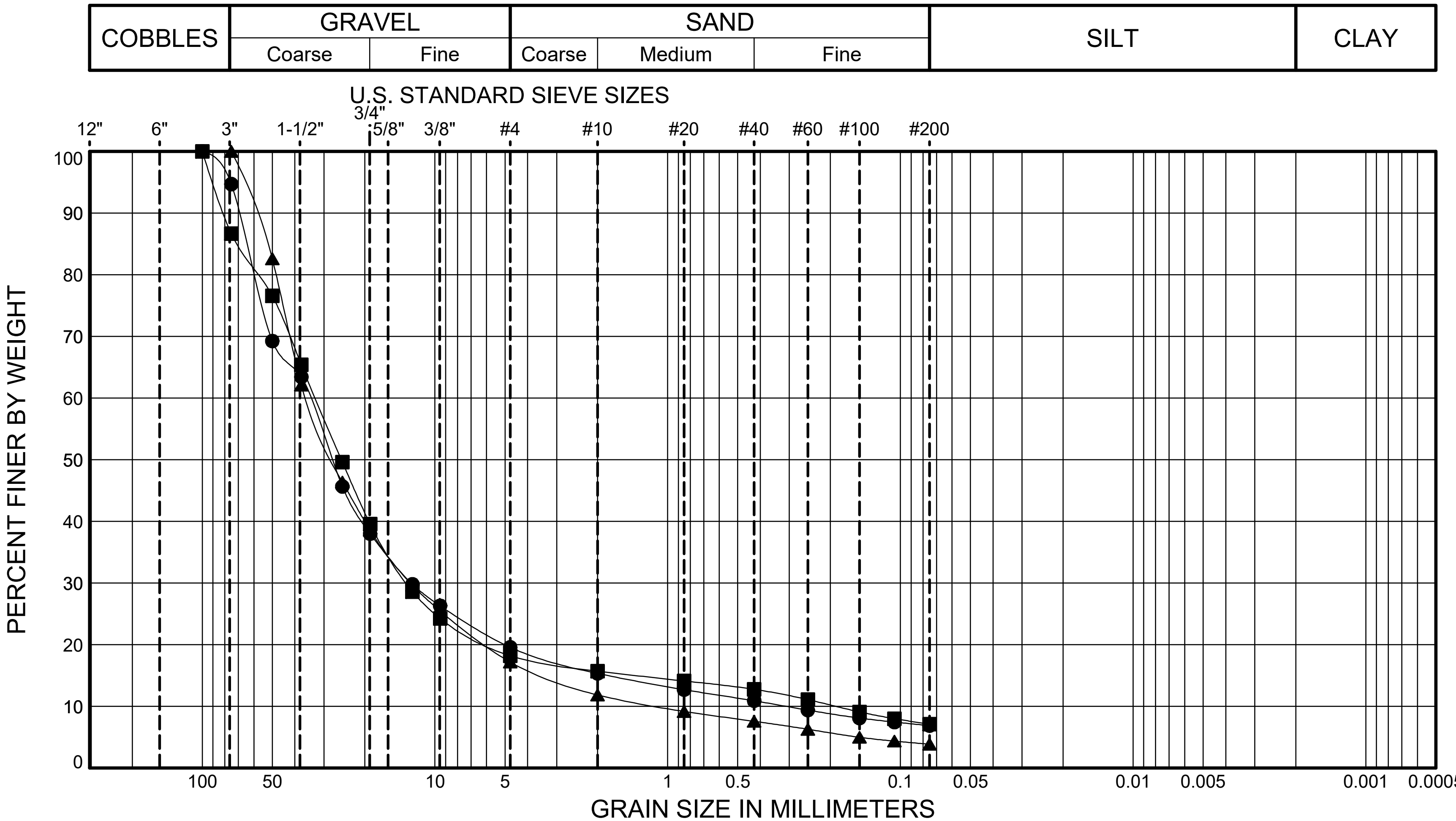
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Cobble %	Gravel %	Sand %	Silt %	Clay %	Fines %
●	HWA-3	S-3	10.0 - 11.5	(CL-ML) Light olive-brown, silty CLAY with sand	28	28	22	6			15.1	77.5	7.4	
■	HWA-3	S-6	17.0 - 17.5	(ML) Light olive-brown, SILT with sand	36	33	28	5			15.7			84.3
▲	HWA-4	S-4	10.0 - 12.0	(ML) Very dark brown, sandy SILT	37	31	24	7		1.0	30.6	58.3	10.2	

HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

PARTICLE-SIZE ANALYSIS OF SOILS METHODS ASTM D6913/D7928

PROJECT NO.: 2021-097-21 FIGURE: B-4



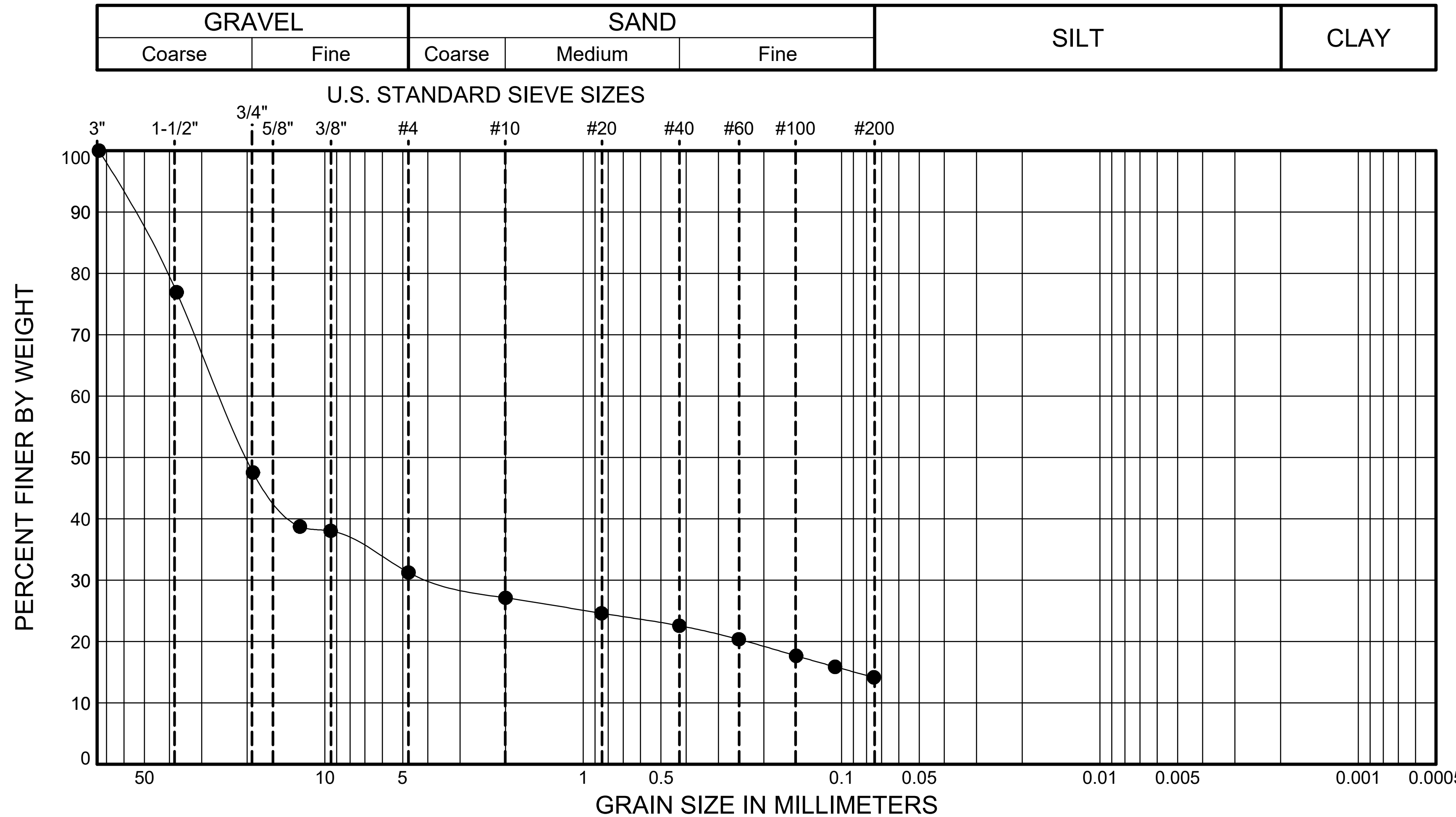
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Cobble	% Gravel	% Sand	% Fines
●	HWA-4	S-5	17.5 - 20.0	(GP-GM) Olive-brown, poorly graded GRAVEL with silt and cobbles	10				5.3	75.1	12.7	6.9
■	HWA-5	S-8	12.5 - 15.0	(GP-GM) Olive-brown, poorly graded GRAVEL with silt and cobbles	11	30	23	7	13.3	68.5	11.1	7.1
▲	HWA-5	S-12	22.0 - 24.0	(GP) Dark olive-brown, poorly graded GRAVEL	10				0.0	82.8	13.3	3.9

HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

PARTICLE-SIZE ANALYSIS OF SOILS METHOD ASTM D6913

PROJECT NO.: 2021-097-21 FIGURE: B-5



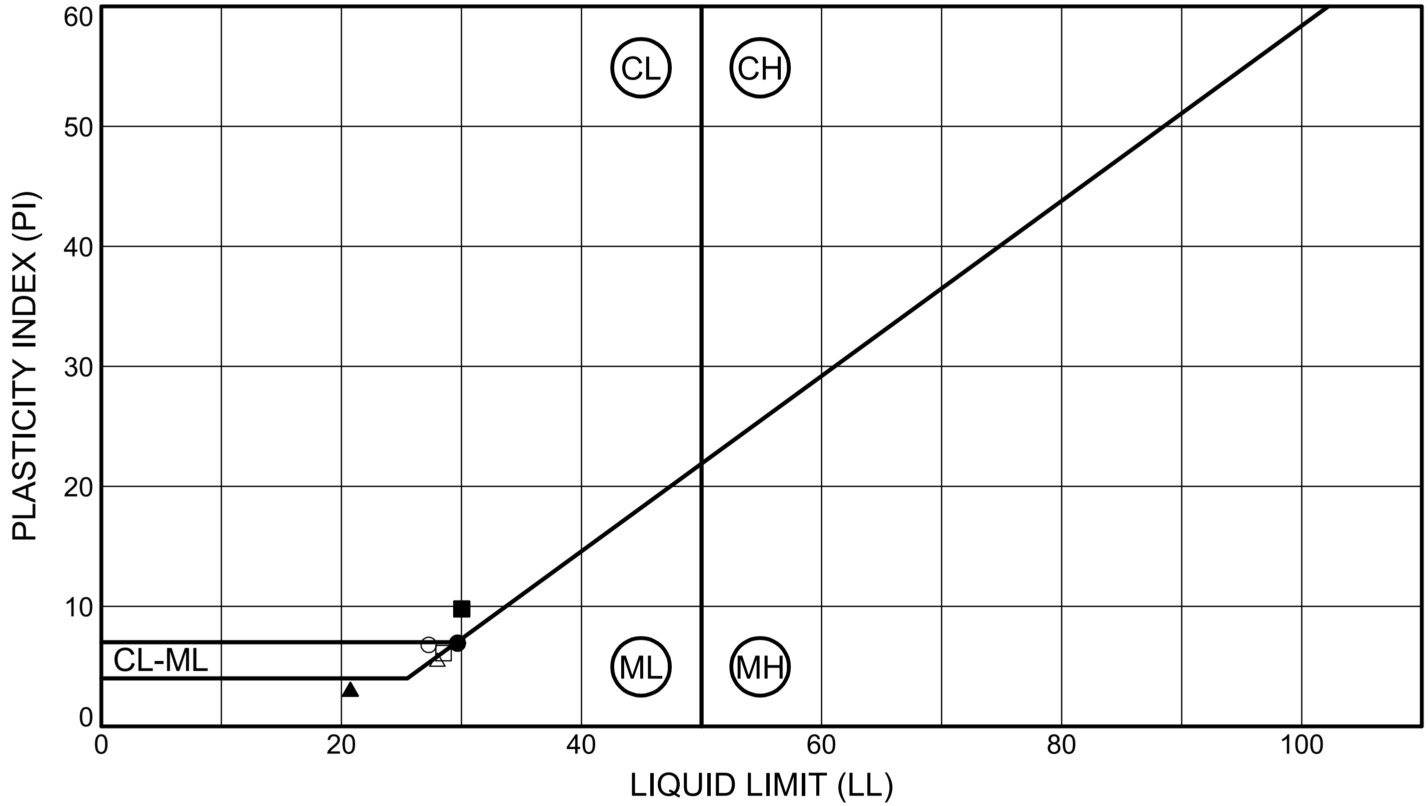
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Cobble %	Gravel %	Sand %	Silt %	Clay %	Fines %
●	HWA-5	S-17	34.0 - 34.5	(GC) Dark yellowish-brown, clayey GRAVEL with sand	17	42	24	18	0.0	68.8	17.1			14.2

HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

PARTICLE-SIZE ANALYSIS OF SOILS METHOD ASTM D6913

PROJECT NO.: 2021-097-21 FIGURE: B-6



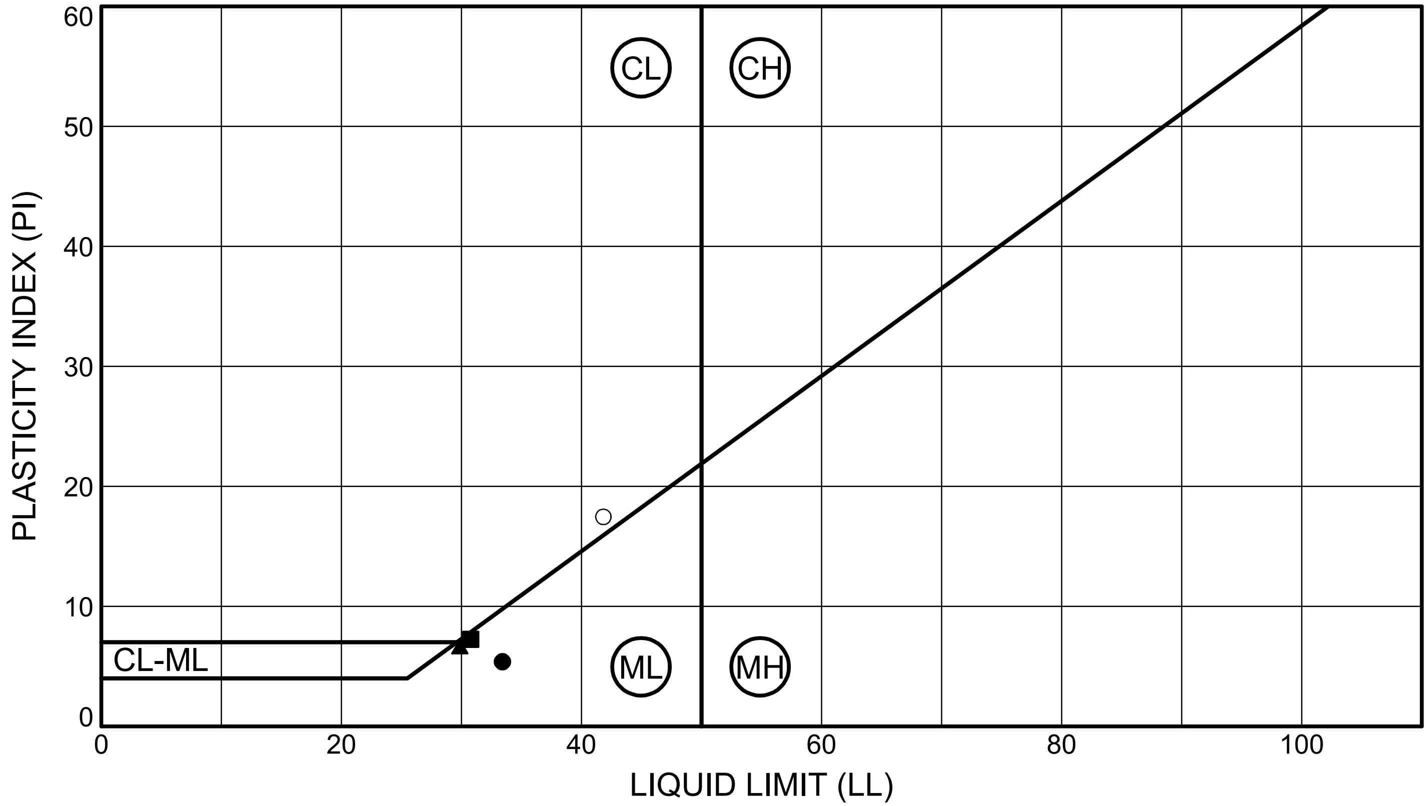
SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	HWA-1	S-6	8.5 - 9.0	(SM) Dark brown, silty SAND with gravel	20	30	23	7	45.3
■	HWA-1	S-14	20.0 - 21.0	(CL) Olive-brown, lean CLAY	32	30	20	10	87.4
▲	HWA-1	S-17	25.0 - 27.5	(GM) Grayish-brown, silty GRAVEL with sand	11	21	18	3	12.6
○	HWA-2	S-4	8.5 - 9.0	(CL-ML) Light olive-brown, sandy silty CLAY	25	27	20	7	62.4
□	HWA-2	S-8	18.5 - 19.0	(CL-ML) Light olive-brown, silty CLAY with sand	30	29	22	7	77.9
△	HWA-3	S-3	10.0 - 11.5	(CL-ML) Light olive-brown, silty CLAY with sand	28	28	22	6	84.9

HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

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

PROJECT NO.: 2021-097-21 FIGURE: B-7



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	HWA-3	S-6	17.0 - 17.5	(ML) Light olive-brown, SILT with sand	36	33	28	5	84.3
■	HWA-4	S-4	10.0 - 12.0	(ML) Very dark brown, sandy SILT	37	31	24	7	68.4
▲	HWA-5	S-8	12.5 - 15.0	(GP-GM) Olive-brown, poorly graded GRAVEL with silt and cobbles	11	30	23	7	7.1
○	HWA-5	S-17	34.0 - 34.5	(GC) Dark yellowish-brown, clayey GRAVEL with sand	17	42	24	18	14.2

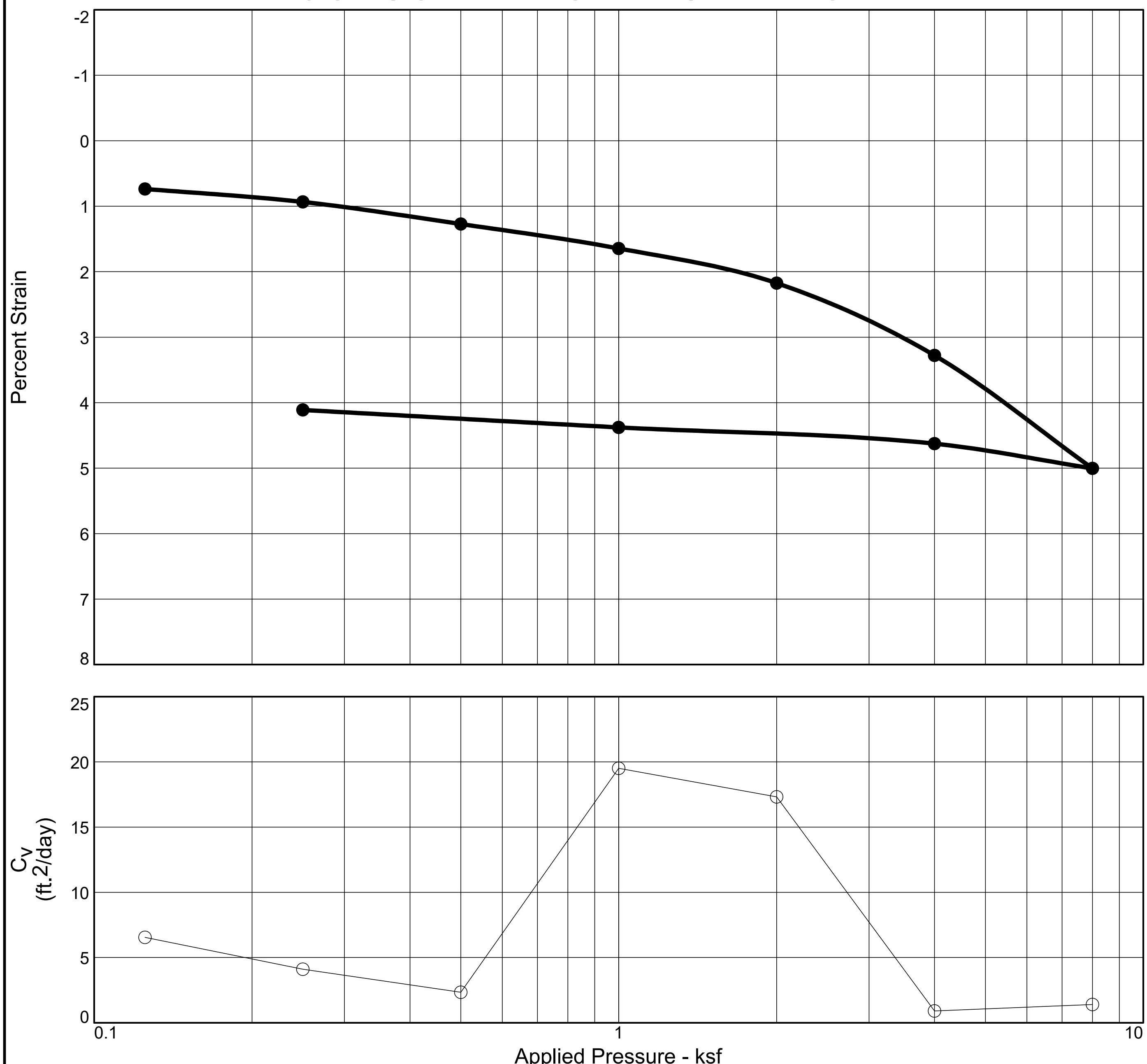
HWA GEOSCIENCES INC.

Pom Pom Road Improvements
Yakama Nation, Washington

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

PROJECT NO.: 2021-097-21 FIGURE: B-8

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
106.1 %	32.0 %	89.8	30	10	2.65	CL		0.798

MATERIAL DESCRIPTION

Olive-brown, lean CLAY

Project No. 2021-097 **Client:** KPFF

Project: Pom Pom Road Improvements

Source of Sample: HWA-1 **Sample Number:** S-14

HWA GEOSCIENCES INC. DBE/MWBE

Remarks:

Specific Gravity Assumed

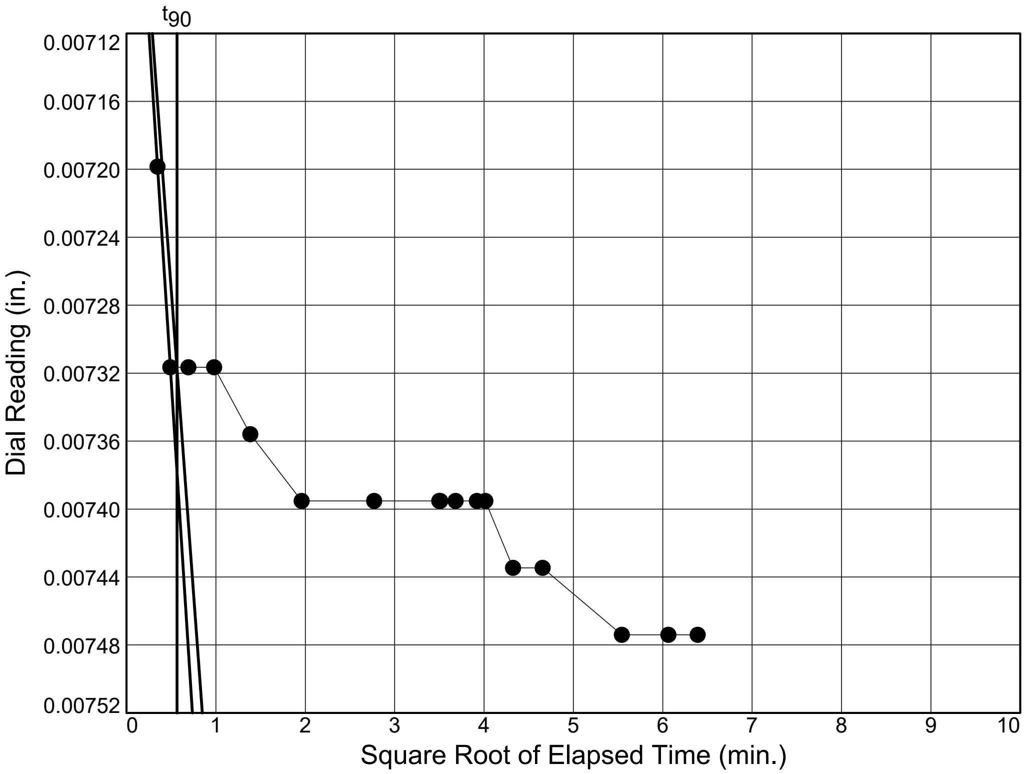
Figure B-9

Tested By: AH **Checked By:** SEG

Dial Reading vs. Time

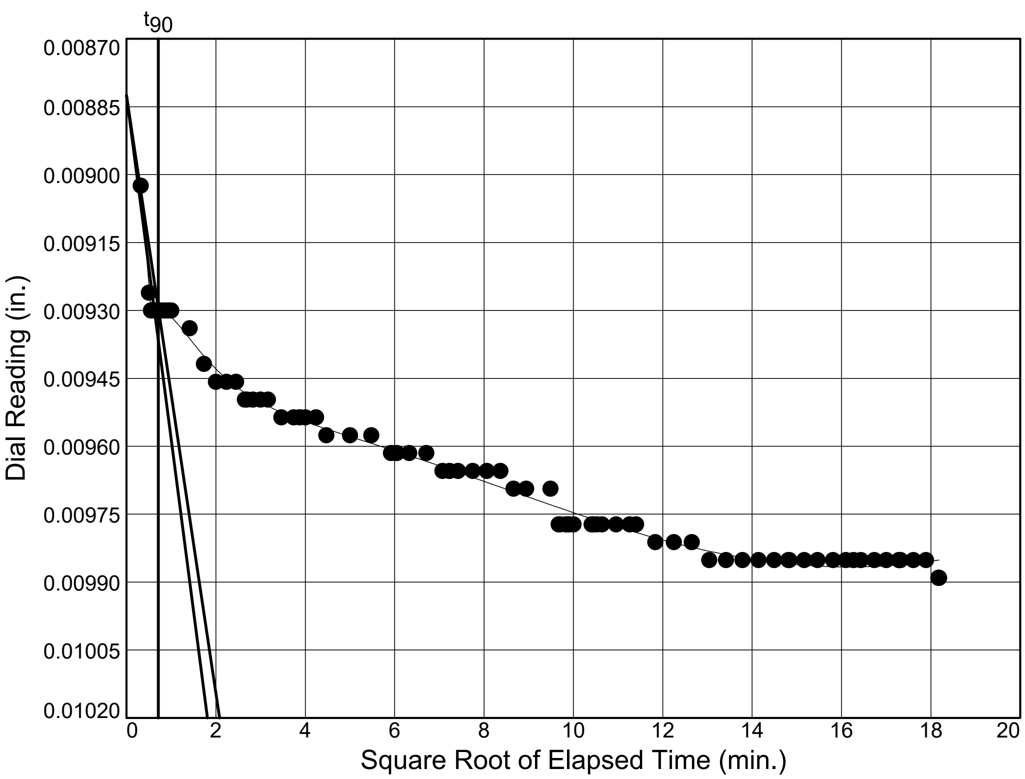
Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-1 Sample Number: S-14



Load No.= 1
Load= 0.13 ksf
D_0 = 0.0069
D_{90} = 0.0073
D_{100} = 0.0074
T_{90} = 0.32 min.

C_v @ T_{90}
6.552 ft.^{2}/day



Load No.= 2
Load= 0.25 ksf
D_0 = 0.0088
D_{90} = 0.0093
D_{100} = 0.0093
T_{90} = 0.51 min.

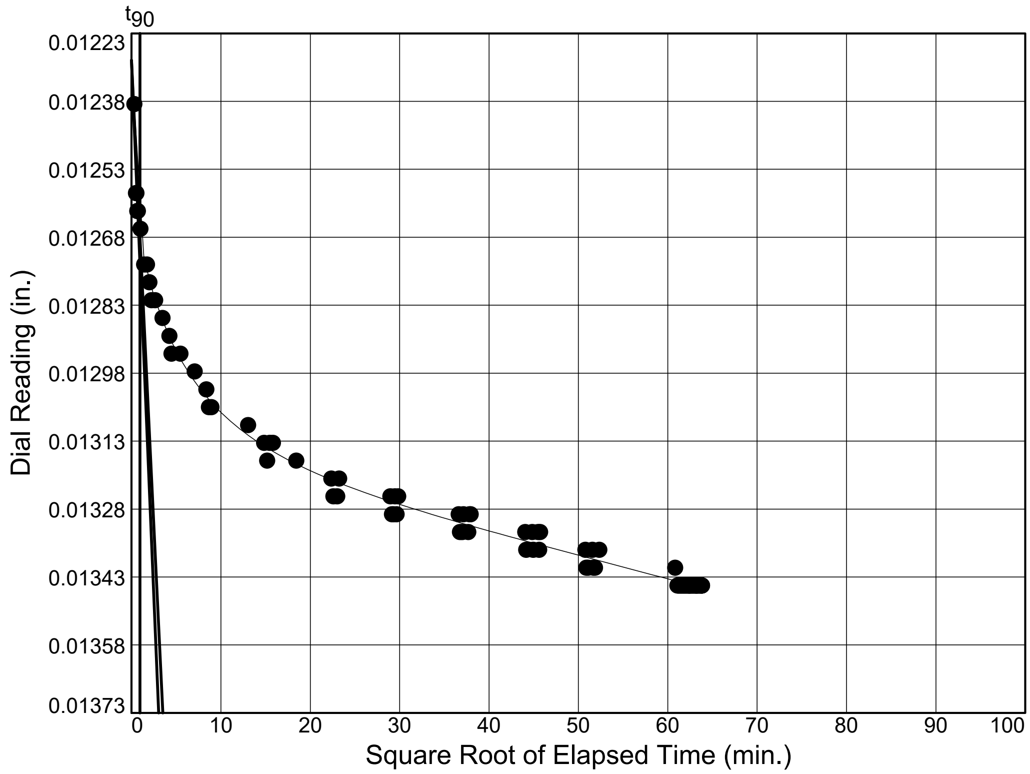
C_v @ T_{90}
4.112 ft.^{2}/day

Figure B-10

Dial Reading vs. Time

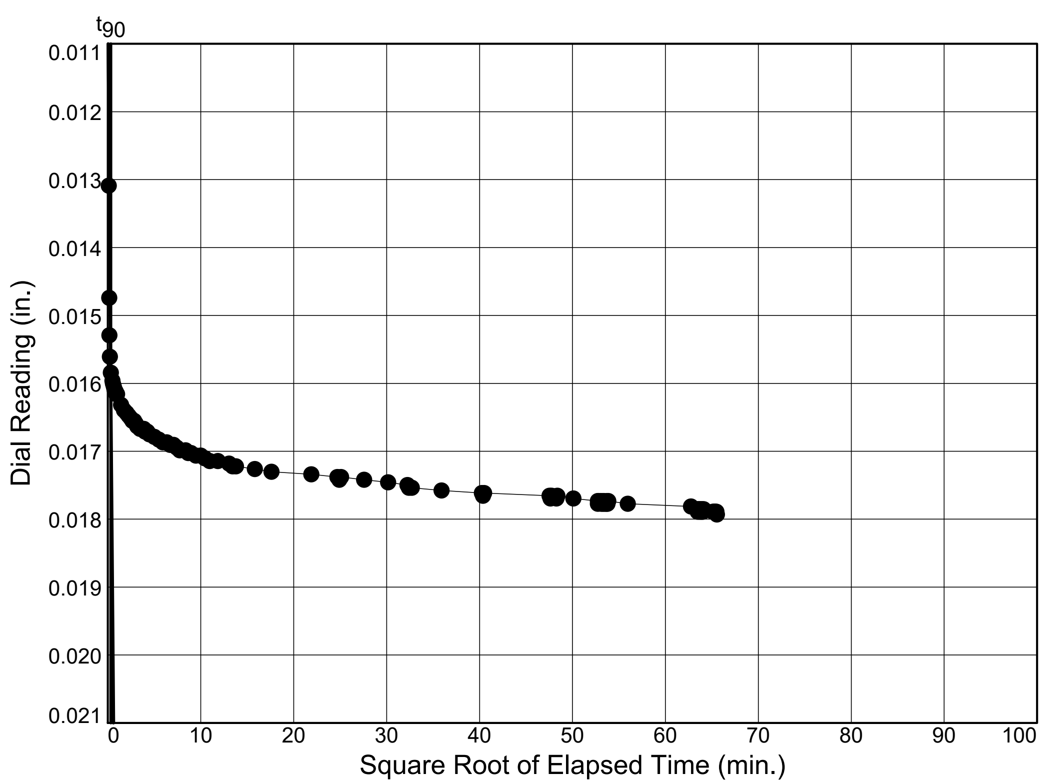
Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-1 Sample Number: S-14



Load No.= 3
Load= 0.50 ksf
D_0 = 0.0123
D_{90} = 0.0127
D_{100} = 0.0127
T_{90} = 0.88 min.

C_v @ T_{90}
2.336 ft.^{2}/day



Load No.= 4
Load= 1.00 ksf
D_0 = 0.0105
D_{90} = 0.0158
D_{100} = 0.0164
T_{90} = 0.11 min.

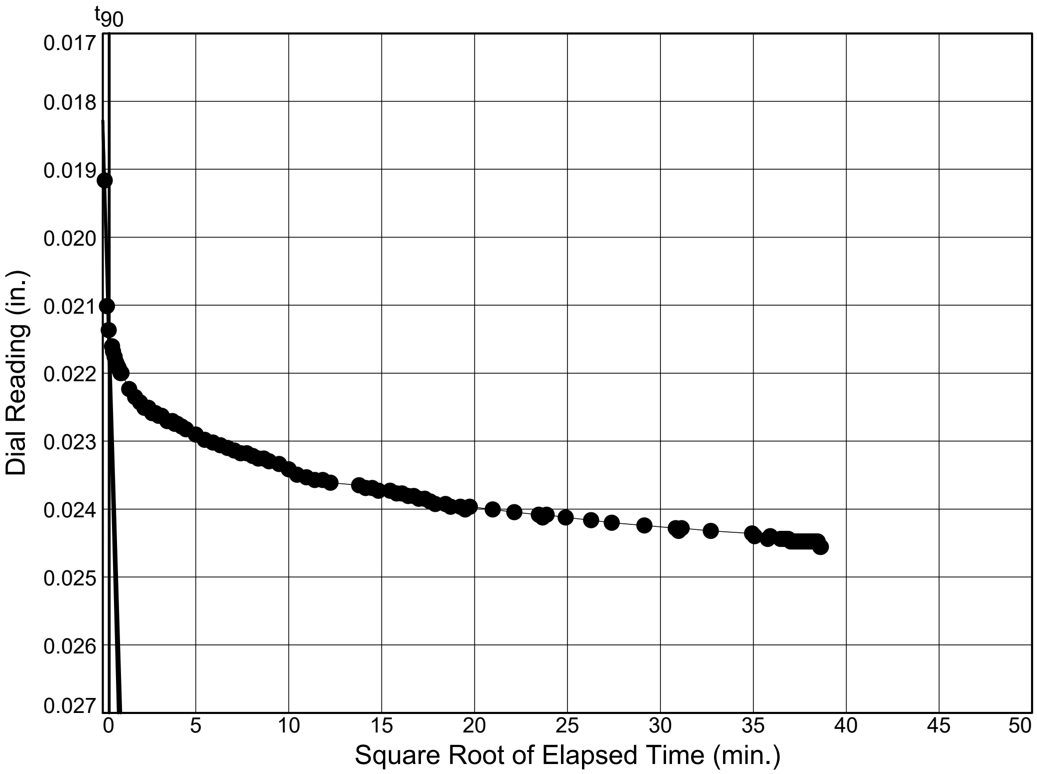
C_v @ T_{90}
19.512 ft.^{2}/day

Figure B-11

Dial Reading vs. Time

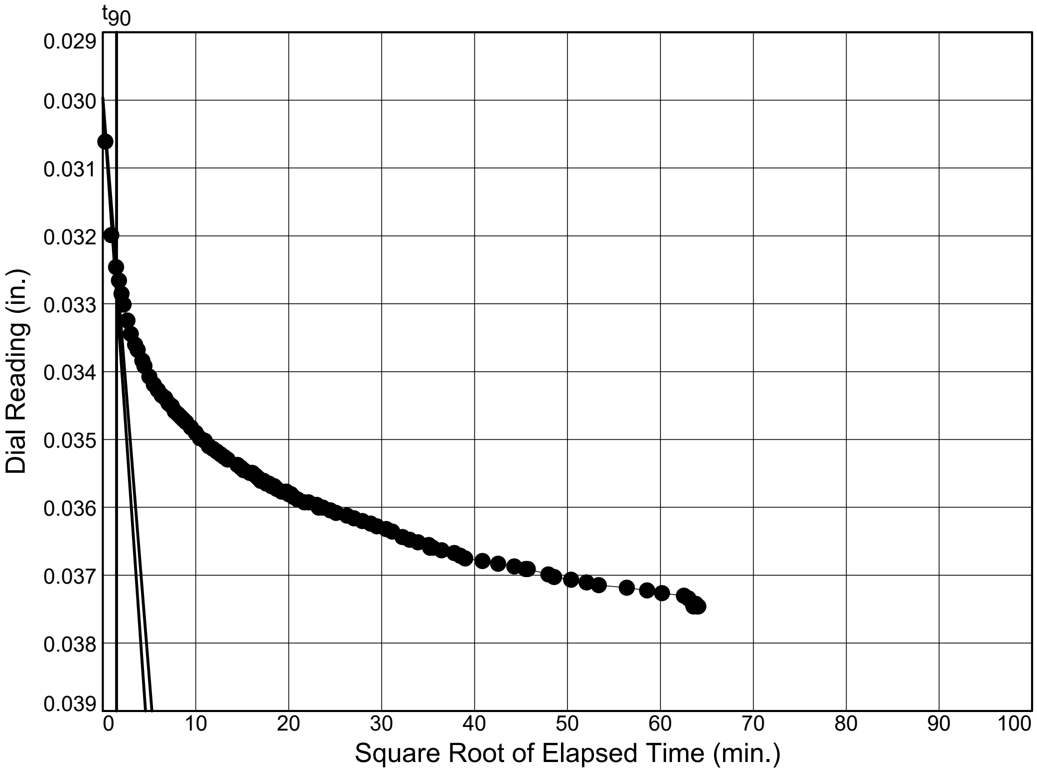
Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-1 Sample Number: S-14



Load No.= 5
Load= 2.00 ksf
D_0 = 0.0183
D_{90} = 0.0214
D_{100} = 0.0217
T_{90} = 0.12 min.

C_v @ T_{90}
17.318 ft.^{2}/day



Load No.= 6
Load= 4.00 ksf
D_0 = 0.0300
D_{90} = 0.0325
D_{100} = 0.0328
T_{90} = 2.19 min.

C_v @ T_{90}
0.909 ft.^{2}/day

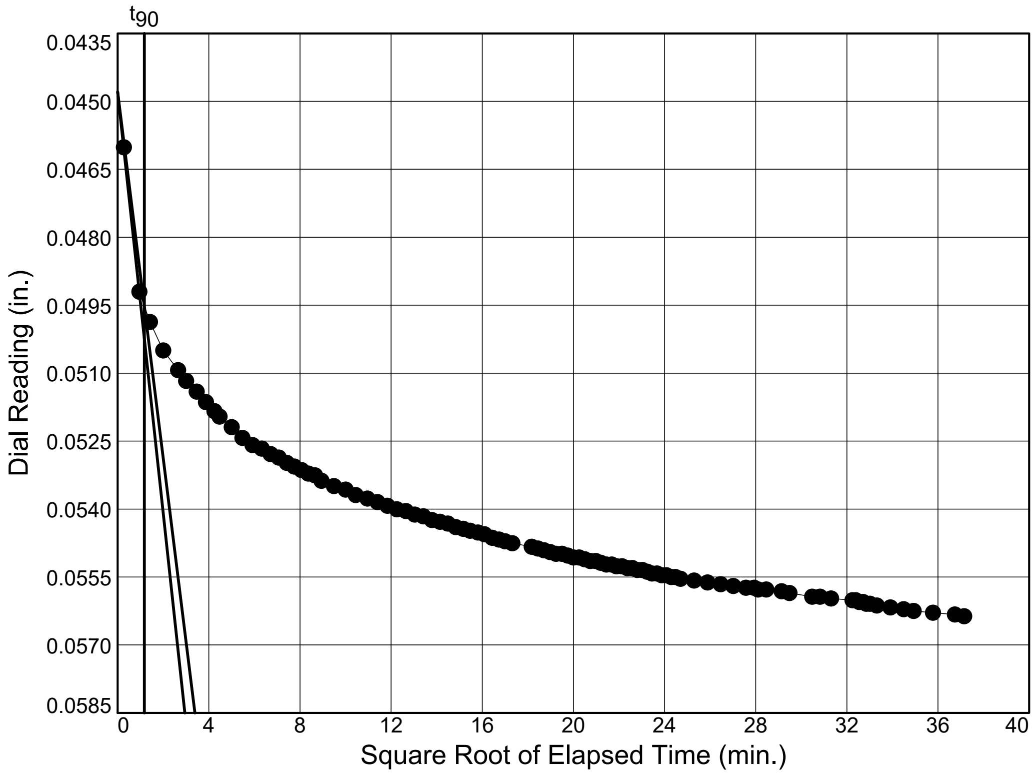
Figure B-12

HWA GeoSciences Inc.

Dial Reading vs. Time

Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-1 Sample Number: S-14



Load No.= 7

Load= 8.00 ksf

D_0 = 0.0448

D_{90} = 0.0495

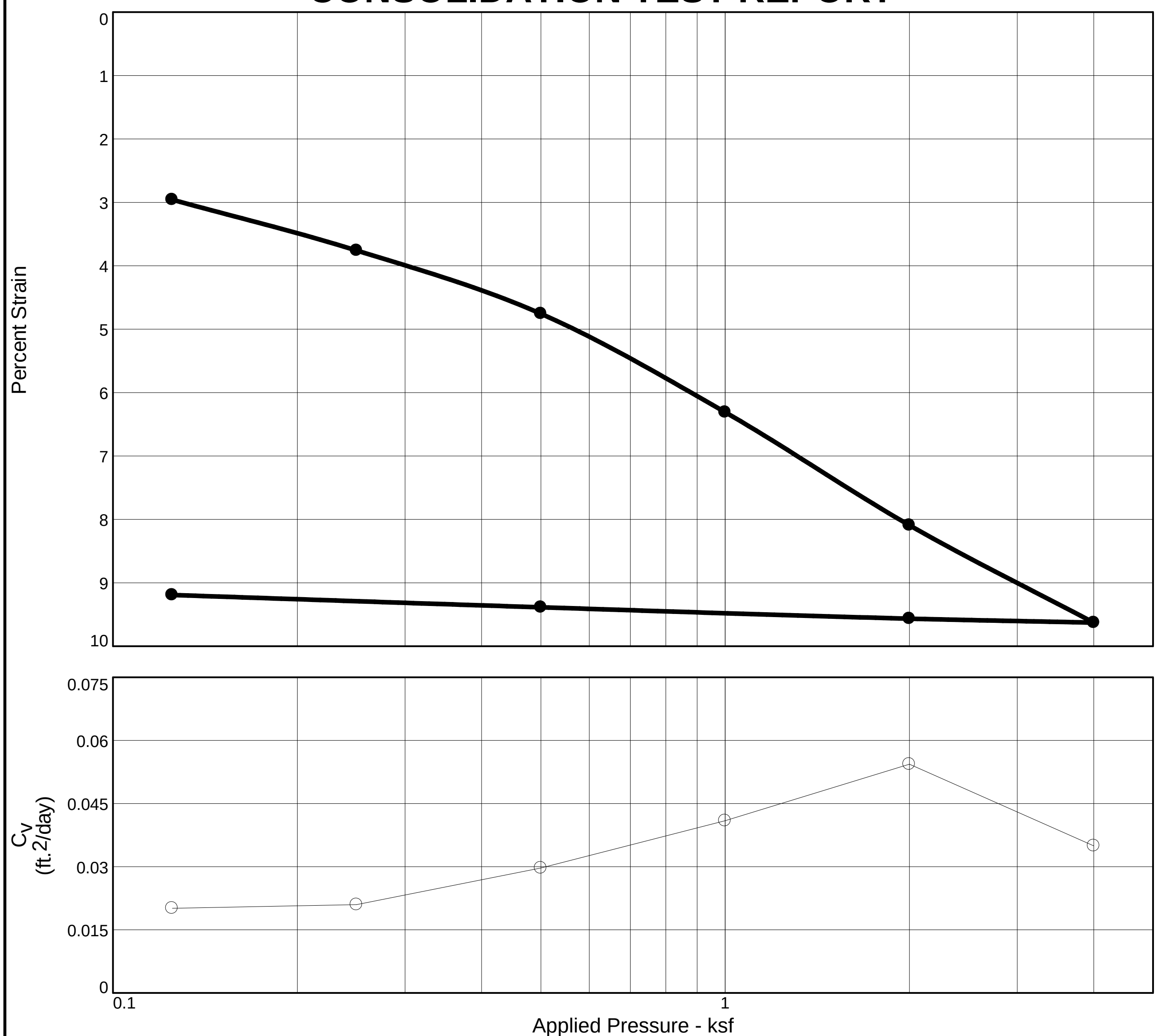
D_{100} = 0.0500

T_{90} = 1.37 min.

C_v @ T_{90}
1.406 ft.^{2}/day

Figure B-13

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
104.2 %	37.3 %	80.7	31	7	2.65	ML		0.950

MATERIAL DESCRIPTION

Very dark brown, sandy SILT

Project No. 2021-097 **Client:** KPFF

Project: Pom Pom Road Improvements

Source of Sample: HWA-4 **Sample Number:** S-4

Remarks:
Specific Gravity Assumed

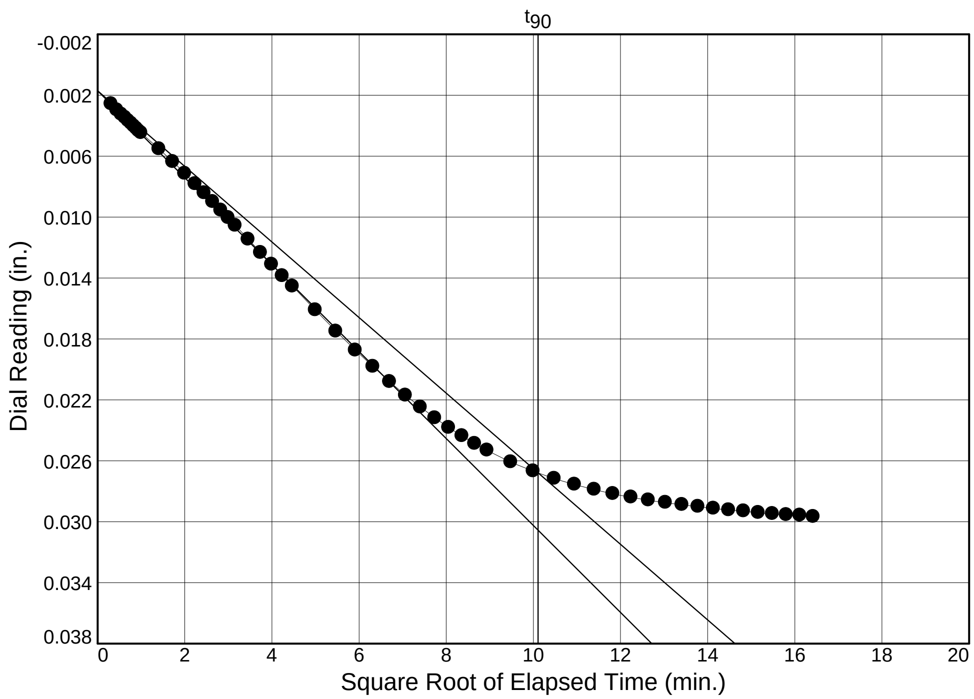
Figure B-14

Tested By: AH **Checked By:** SEG

Dial Reading vs. Time

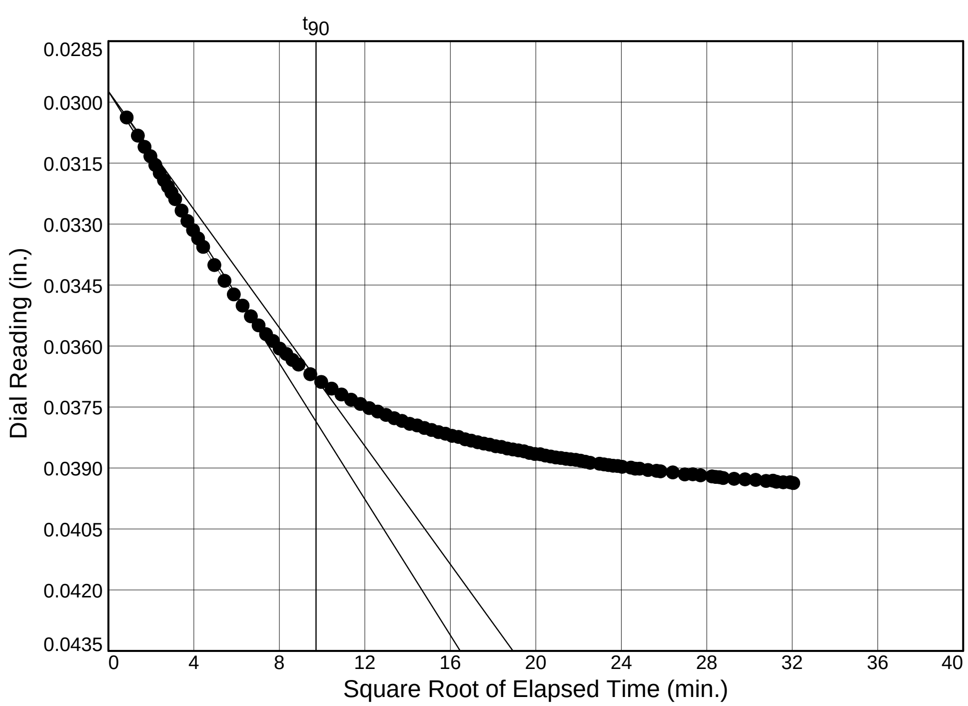
Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-4 Sample Number: S-4



Load No.= 1
Load= 0.13 ksf
D_0 = 0.0017
D_{90} = 0.0268
D_{100} = 0.0296
T_{90} = 102.17 min.

C_v @ T_{90}
0.020 ft.^{2}/day



Load No.= 2
Load= 0.25 ksf
D_0 = 0.0297
D_{90} = 0.0368
D_{100} = 0.0376
T_{90} = 94.36 min.

C_v @ T_{90}
0.021 ft.^{2}/day

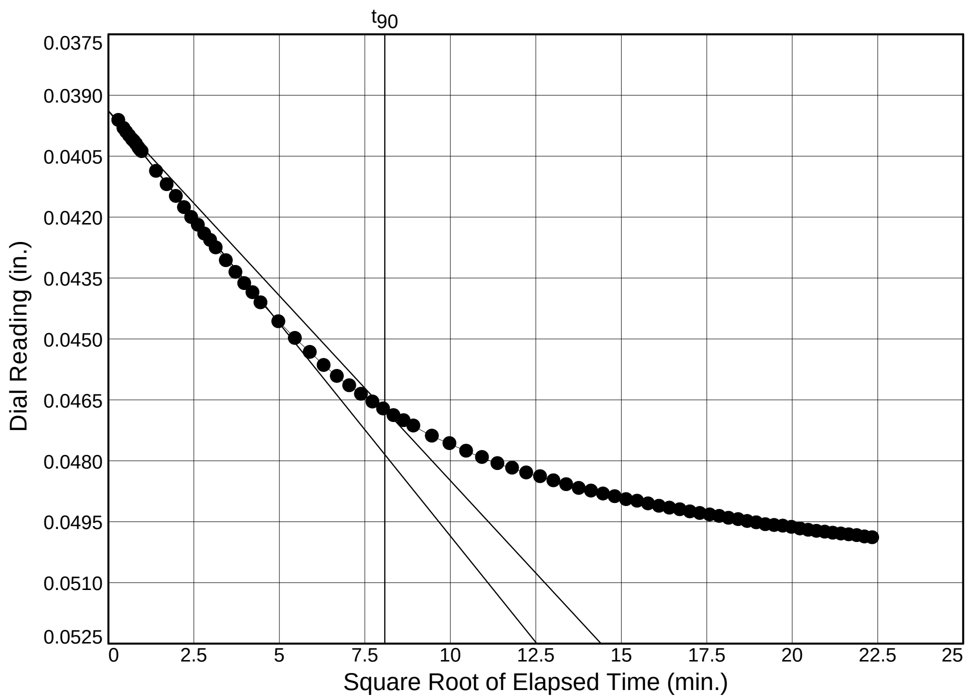
Figure B-15

HWA GeoSciences Inc.

Dial Reading vs. Time

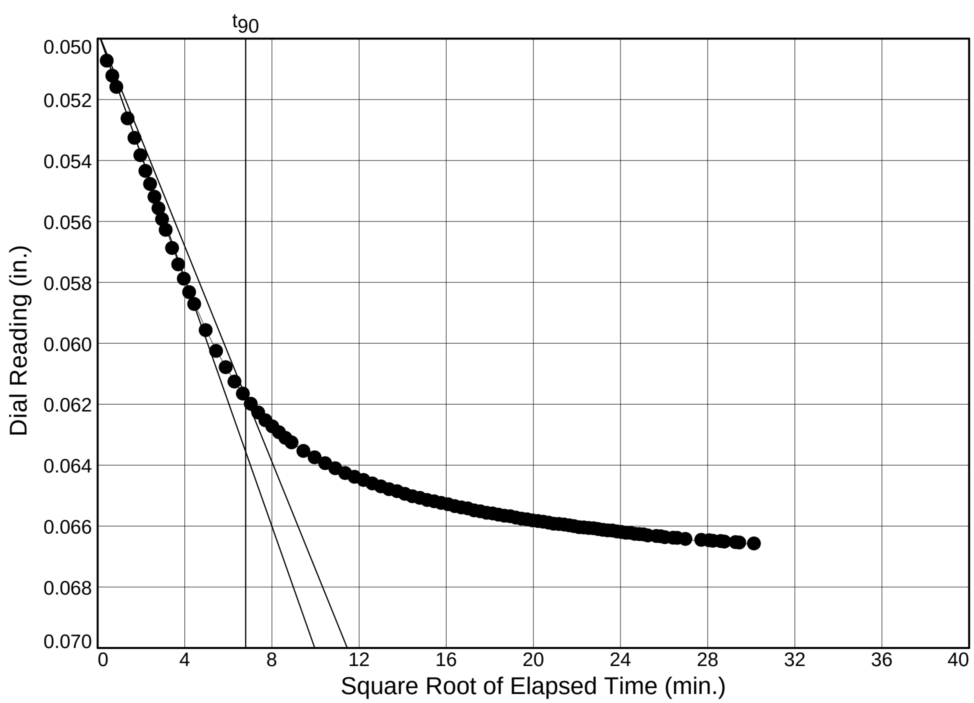
Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-4 Sample Number: S-4



Load No.= 3
Load= 0.50 ksf
D_0 = 0.0394
D_{90} = 0.0467
D_{100} = 0.0476
T_{90} = 65.33 min.

C_v @ T_{90}
0.030 ft.^{2}/day



Load No.= 4
Load= 1.00 ksf
D_0 = 0.0498
D_{90} = 0.0618
D_{100} = 0.0631
T_{90} = 46.13 min.

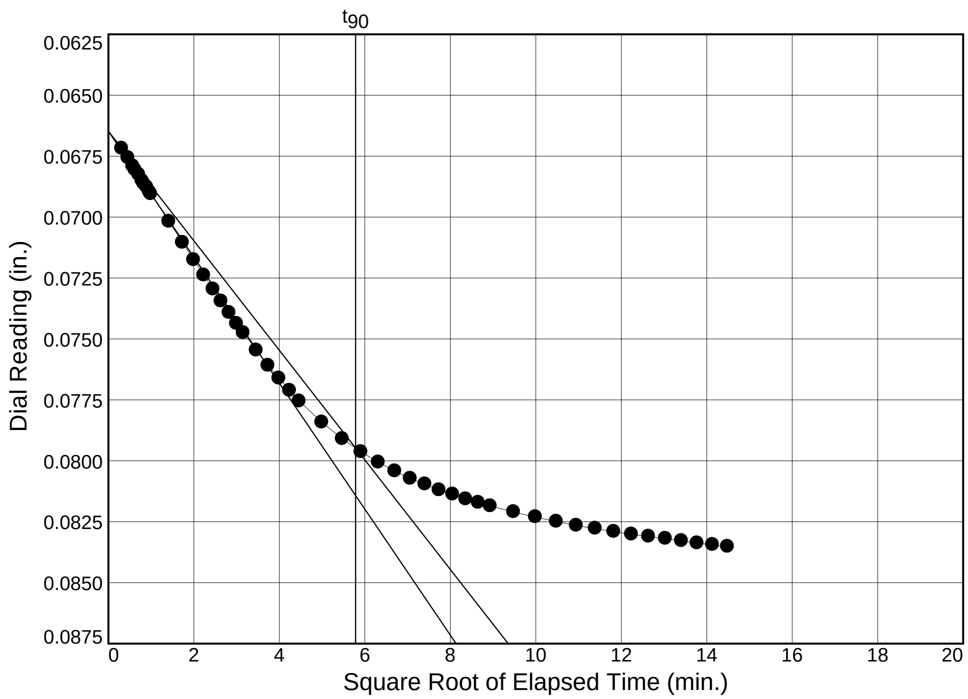
C_v @ T_{90}
0.041 ft.^{2}/day

Figure B-16

Dial Reading vs. Time

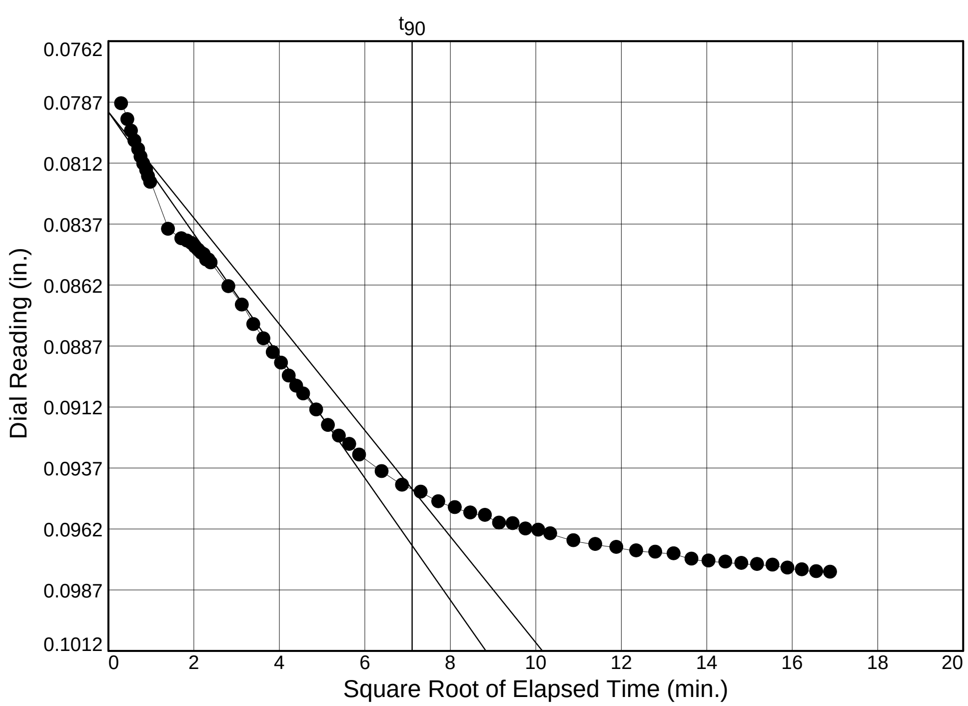
Project No.: 2021-097
Project: Pom Pom Road Improvements

Source of Sample: HWA-4 Sample Number: S-4



Load No.= 5
Load= 2.00 ksf
D_0 = 0.0665
D_{90} = 0.0795
D_{100} = 0.0809
T_{90} = 33.47 min.

C_v @ T_{90}
0.054 ft.^{2}/day



Load No.= 6
Load= 4.00 ksf
D_0 = 0.0791
D_{90} = 0.0946
D_{100} = 0.0963
T_{90} = 50.51 min.

C_v @ T_{90}
0.035 ft.^{2}/day

Figure B-17

APPENDIX C

LPILE PARAMETERS

South Abutment - LPILE Parameters

Existing Ground Surface Elevation at South Abutment (Boring HWA-4) = 1020 Feet

Soil Layer	Soil Type (p-y model)	Top of Layer (ft)	Bottom of Layer (ft)	Effective Unit Wt, y' (pcf)[1]	Friction Angle (deg)	Undrained Shear Strength C_u (psf)[2]	p-y Modulus Static, k (pci)	p-y Modulus Seismic, k (pci)	Strain Factor, ε_{50} (dim)
Existing Fill	Sand (Reese)	0		125	32	--	90	90	--
			4.5	125	32	--	90	90	--
Loose Silty Sand	Sand (Reese)	4.5		125	32				
			6	125	32				
Soft to Medium Stiff Sandy Silt	Soft Clay (Matlock)	6		100	--	400.0	30	--	0.02
			10	100	--	400.0	30	--	0.02
Soft to Medium Stiff Sandy Silt (Below Groundwater)	Soft Clay (Matlock)	10		38	--	400.0	30	--	0.02
			15	38	--	400.0	30	--	0.02
Medium Dense to Dense Gravel	Sand (Reese)	15		73	34	--	90	90	--
			35	73	34	--	90	90	--
Dense to Very Dense Gravel	Sand (Reese)	35		73	36	--	125	125	--
			50	73	36	--	125	125	--

North Abutment - LPILE Parameters

Existing Ground Surface Elevation at North Abutment (Boring HWA-5) = 1020 Feet

Soil Layer	Soil Type (p-y model)	Top of Layer (ft)	Bottom of Layer (ft)	Effective Unit Wt, y' (pci)[1]	Friction Angle (deg)	Undrained Shear Strength C_u (psf)[2]	p-y Modulus Static, k (pci)	p-y Modulus Seismic, k (pci)	Strain Factor, ε_{50} (dim)
Existing Fill	Sand (Reese)	0		125	32	--	90	90	--
			4	125	32	--	90	90	--
Loose Silty Sand	Sand (Reese)	4		125	32				
			6	125	32				
Soft to Medium Stiff Silty Clay (Below Groundwater)	Soft Clay (Matlock)	6		38	--	400.0	30	--	0.02
			11	38	--	400.0	30	--	0.02
Medium Dense to Dense Gravel	Sand (Reese)	11		73	34	--	90	90	--
			40	73	34	--	90	90	--
Dense to Very Dense Gravel	Sand (Reese)	40		73	36	--	125	125	--
			50	73	36	--	125	125	--

1: Total Unit Weight (pcf) = Effective Unit Weight + 62.4 (for layers below water table)
2: Undrained Shear Strength, C=Cu=Su

[1]: Total Unit Weight (pcf) = Effective Unit Weight + 62.4 (for layers below water table)
[2]: Undrained Shear Strength, C = Cu = Su