NELSON GEOTECHNICAL ASSOCIATES, INC.

**105 Palouse Street
Wenatchee, WA 98801
(509) 665-7696
www.nelsongeotech.com**

November 1, 2024

Joe Irwin
Helion Energy Inc.
Via E-Mail: Joe.Irwin@helionenergy.com

Geotechnical Engineering and Infiltration Evaluation
Helion Energy Power Plant Facility
Chelan County Parcel Number 212205000050
Chelan County, Washington
NGA File No. 1549124

Dear Mr. Irwin:

We are pleased to submit the attached report titled **"Geotechnical Engineering and Infiltration Evaluation – Helion Energy Power Plant Facility – Chelan County Parcel Number 212205000050 – Chelan County, Washington."** This report summarizes our explorations of the surface and subsurface conditions within the site and provides general recommendations for the proposed site development. Our services were completed in general accordance with the proposal dated September 13, 2024, and signed by you on September 24, 2024.

The property consists of a rural utility parcel directly south and west of Rock Island Dam. The proposed location of the power plant lies within the southwestern portion of the parcel, east of Colockum Road. The site is generally level with undulations and covered in bare earth and sagebrush.

Based on a preliminary undated site plan provided by Helion Energy, we understand that project plans include an approximately 100,000-square-foot main building, a 25,000-square-foot assembly building, and a support building. You have requested that we evaluate the site subsurface conditions and provide our opinions and recommendations regarding the proposed site development. Preliminary evaluation of on-site stormwater infiltration was also requested.

We explored the subsurface soil and groundwater conditions on October 10 and 11, 2024, with three geotechnical borings ranging from 25.5 to 51.5 feet below existing grade. In general, we encountered brown to gray, native gravel with some sand and trace gravel throughout the site. The native soil is generally in loose to medium-dense condition. We did not encounter groundwater or seepage in any of the explorations.

It is our opinion, from a geotechnical standpoint, that the planned development is feasible, provided that our recommendations are incorporated into project plans. The attached report includes recommendations for earthwork, foundation, and slab-on-grade support, temporary and permanent slopes, pavement subgrade, site drainage, and erosion control.

We appreciate the opportunity to provide service to you on this project. Please contact us if you have any questions regarding this report or require further information.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Khaled M. Shawish, PE
Principal

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Geotechnical Engineering Evaluation
Helion Energy Power Plant Facility
Chelan County Parcel Number 212205000050
Chelan County, Washington

INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Helion Energy Power Plant Facility project located north of Rock Island Dam Road (Nixon Rapids Road) near Rock Island Dam in Chelan County, Washington. The property consists of a 401.0-acre rural utility parcel directly south and west of Rock Island Dam. The proposed location of the power plant lies within the southwestern portion of the parcel east of Colockum Road within an area of approximately 81 acres. The location of the proposed development area is shown on the Vicinity Map in Figure 1.

Based on a preliminary undated site plan provided by Helion Energy, we understand that project plans include a 100,000-square-foot main building, a 25,000-square-foot assembly building, and a support office. There will also be paved access and parking areas as well as underground utilities.

SCOPE

The purpose of this study is to explore and characterize the site surface and subsurface conditions and provide geotechnical engineering and infiltration recommendations for the planned site development.

Specifically, our scope of services included the following:

1. Reviewing available soil and geologic maps of the area as well as other relevant geotechnical information and historical documents.
2. Exploring the subsurface soil and groundwater conditions within the proposed development areas with three, 25- to 50-foot-deep geotechnical boreholes using a sonic drill track rig. Drilling services were subcontracted by NGA.
3. Assessing the site for potential geologic hazards, including landslide, seismic, erosion, avalanche, and flood hazards per Chelan County Code.
4. Providing long-term design infiltration rates based on laboratory analysis on soil samples obtained at the site.
5. Performing laboratory analysis on selected soil samples obtained from the explorations, as necessary.
6. Providing recommendations for earthwork and foundation support.
7. Providing seismic design parameters, including site class, short- and long-period spectral accelerations, and amplification factors.

8. Providing an evaluation of the liquefaction potential of the site soils.

9. Providing recommendations for retaining walls.

10. Providing recommendations for temporary and permanent cut and fill slopes.

11. Providing recommendations for slab-on-grade subgrade preparation.

12. Providing recommendations for pavement, including subgrade preparation and pavement section thicknesses.

13. Providing recommendations for site drainage and erosion control.

14. Documenting our observations, explorations, conclusions, and recommendations in a written geotechnical engineering report.

SITE CONDITIONS

Surface Conditions

We visited the site and made our observations and explorations on October 10 and 11, 2024. The proposed power plant location is currently undeveloped range land owned by Public Utility District No. 1 of Chelan County. The surrounding parcels are also undeveloped range land owned by Alcoa Wenatchee. The site is generally level with generally east- to west-trending undulations and is covered with bare earth, small shrubs, and grass.

Subsurface Conditions

Geology The geologic units for this area are mapped on the Geologic Map of the Wenatchee 1:100,000 Quadrangle, Central Washington, by R. W. Tabor, et al., (USGS, 1982). The project site is generally mapped as being within deposits described as Columbia River Floods Deposits - Gravel of lower-level bars (Qcgl). The lower-level bar deposits are described as surfaces of cobble-to-boulder gravel generally 60 to 90 meters above the river and embellished with giant current dunes. Our explorations encountered cobbles, gravel, and sand at depth consistent with flood deposits.

Explorations: The subsurface conditions within the site were explored on September 3, 2024, with three geotechnical borings to depths ranging from 25.5 to 51.5 feet below existing grade. The approximate locations of our explorations are shown on the Site Plan in Figure 2. A Geologist from Nelson Geotechnical Associates, Inc. (NGA) was present during the boring explorations, examined the soils and geologic conditions encountered, obtained samples of the different soil types, and maintained logs of the explorations.

A Standard Penetration Test (SPT) was performed on each of the samples during drilling to document relative soil density at depth. The SPT consists of driving a 2-inch outer-diameter, split-spoon sampler 18 inches using a 140-pound hammer with a drop of 30 inches. The number of blows required to drive the sampler the final 12 inches is referred to as the “N” value and is presented on the boring logs. The N value is used to evaluate the strength and density of the deposit.

The soils were visually classified in general accordance with the Unified Soil Classification System, presented in Figure 3. The exploration logs are presented in Figures 4 through 6. Sieve analysis results for two samples from Boring 1 and Boring 3, respectively, are presented as Figures 7 and 8.

We present a brief summary of the subsurface conditions in the following paragraph. For a detailed description of the subsurface conditions, the exploration logs should be reviewed.

In general, our explorations encountered approximately 50.0 feet of Columbia River flood deposits consisting of brown to grey, boulder-to-cobble gravel with some sand and trace silt throughout the site. Based on the SPT blow counts, the density of the deposits appears to be very dense. It is our opinion that the SPT blow counts are overstated due to the cobbles within the deposits. Based on observed surface deposits and caving during the explorations, the soils encountered consist of unconsolidated fluvial deposits and are generally in a loose to medium-dense condition.

In Boring 1, an approximately 5.0-foot-thick layer of silt was encountered at a depth of 13.0 feet below the current ground surface. We consider this to be a laterally discontinuous low-energy fluvial deposit, which may be encountered at depth within the site.

Hydrogeologic Conditions

We did not encounter groundwater seepage during our explorations. During wet weather, a perched water condition may develop on this site. Perched water occurs when surface water infiltrates through less dense, more permeable soils and accumulates on top of underlying, less permeable soils. Perched water does not represent a regional groundwater "table" within the upper soil horizons. Perched water tends to vary spatially and is dependent upon the amount of rainfall. We would expect the amount of perched water to decrease during drier times of the year and increase during wetter periods.

GEOLOGIC HAZARD EVALUATION

Erosion Hazard

The criteria used for determination of the erosion hazard for affected areas include soil type, slope gradient, vegetation cover, and groundwater conditions. The erosion sensitivity is related to vegetative cover and the specific surface soil types, which are related to the underlying geologic soil units. The Web Soil Survey, by the Natural Resources Conservation Service (NRCS), was reviewed, to determine the erosion hazard of the on-site soils. The site surface soils were classified using the NRCS classification system as Pogue fine sandy loam, 3 to 8 percent slopes (PoB), Pogue gravelly fine sandy loam, 3 to 8 percent slopes (PrB), Pogue gravelly fine sandy loam, 8 to 15 percent slopes (PrC), and Cashmont sandy loam, 3 to 8 percent slopes (CcB). These soils are listed as having a slight to moderate erosion hazard where soil is exposed or disturbed. The establishment of vegetation will significantly reduce the erosion on site.

Landslide Hazard

We did not observe indications of significant downslope movement within the site. Steep slopes are located several hundred feet to the west of this site, but we did not observe any indications of chronic landsliding or instability. The potential of catastrophic geologic landslides for native soil conditions is considered low for this site.

Flood Hazard

We did not observe evidence of seasonal stream channels or active alluvial fans within this site. The area does not indicate recent flooding or fan accumulations within the site. Accordingly, the potential for catastrophic flooding, inundation, or debris flows should be considered low.

Seismic Hazard

We reviewed the 2021 International Building Code (IBC) and the ASCE 7-16 for seismic site classification for this project. Since glacial sand and gravel deposits were encountered at depth within the subject site, the site conditions best fit the IBC description for Site Class D.

Table 1 below provides seismic design parameters for the site that are in conformance with the 2021 IBC, which specifies a design earthquake having a two percent probability of occurrence in 50 years (return interval of 2,475 years), and the 2014 USGS seismic hazard maps.

Table 1. 2021 IBC Seismic Design Parameters

Site Class	Spectral Acceleration at 0.2 sec. (g) S_S	Spectral Acceleration at 1.0 sec. (g) S_1	Site Coefficients		Design Spectral Response Parameters	
			F_a	F_v	S_{DS}	S_{D1}
D	0.435	0.179	1.452	2.242	0.421	0.267

The spectral response accelerations were obtained from the USGS Earthquake Hazards Program Interpolated Probabilistic Ground Motion website (2014 data) for the project latitude and longitude.

Hazards associated with seismic activity include liquefaction potential and amplification of ground motion. Liquefaction is caused by a rise in pore pressures in a loose, fine sand deposit beneath the groundwater table. It is our opinion that the loose to medium-dense flood sand and gravel deposits interpreted to underlie the site have a low to moderate potential for liquefaction or amplification of ground motion.

Snow Avalanche Hazard

No evidence of snow avalanches was observed within or nearby this area, accordingly the potential for destructive avalanches is very low.

INFILTRATION TESTING

The native subsurface soils encountered generally consisted of gravel and cobbles with some sand and trace gravel. In accordance with the 2019 Department of Ecology Stormwater Management Manual for Eastern Washington (SWMMEW), we utilized the grain size analysis method with one sample obtained from Boring 3. The infiltration results utilizing the grain size distribution method are summarized in Table 2 below.

Table 2. Grain Size Analysis Results

Exploration Identification	**USCS Soil Type**		**Depth of Sample (feet)**	**Grain Size Distribution Infiltration Rate (Inches/Hour)**	**Preliminary Infiltration Design Rate (Inches/Hour)** *Correction factor of 0.33 applied to calculated infiltration rate.
B-3	GP-GM		20.0	25.06	8.27

It is our opinion that the cobble-to-gravel flood deposits encountered at depth are suitable for onsite stormwater infiltration. The native gravel deposits were encountered across the site at depth in all borings.

Per Table 6.4 of section 6.3.3 of the 2019 SWMMEW, correction factors for site variability (CF_v), test method (CF_t), and siltation (CF_m) were applied to the grain size distribution infiltration rate of 25.06 inches per hour. A total correction factor of 0.33 was determined and applied to the field infiltration rates, resulting in a long-term design infiltration rate of 8.27 inches per hour. We have selected 8.27 inches per hour as a representative preliminary long-term design infiltration rate to be utilized for design of any infiltration galleries within this site. An overflow component should be incorporated into the design of onsite infiltration systems, if possible.

Supplemental field infiltration testing should be performed to verify design rates and system sizing prior to finalizing the design. The stormwater manual recommends a five-foot separation between the base of an infiltration system and any underlying bedrock, impermeable horizon, or groundwater. Due to the silt deposit encountered at depth within Boring 1, NGA should confirm soil conditions for any future stormwater infiltration system location.

We also recommend that any proposed infiltration systems be placed to not negatively impact any nearby structures and meet all required setbacks from existing property lines, structures, and sensitive areas as discussed in the drainage manual. Infiltration systems should not be located within proposed fill areas within the site associated with site grading as such conditions could lead to failures of the placed fills and/or retaining structures. We should be retained to evaluate the infiltration system design and installation during construction to confirm specific soil conditions exposed along the base of the systems.

CONCLUSIONS AND RECOMMENDATIONS

General

It is our opinion, from a geotechnical standpoint, that the proposed development is generally feasible as planned, provided that the geotechnical recommendations presented in this report are incorporated into project plans and followed during construction. Due to potential loose soil conditions, specific subgrade preparation should be performed to maintain site area stability and minimize post-construction distress to the planned structure. Organic rich topsoil should be stripped from the areas to be developed, including buildings and pavement areas, and the native alluvial subgrade should be compacted to a dense and unyielding condition. NGA should be retained to review final development plans prior to construction.

The on-site soils are considered to be moisture sensitive and will disturb when wet. We recommend that construction take place during extended periods of dry weather. If construction takes place during wet weather, additional expenses and delays should be expected due to the wet conditions. Additional expenses could include the need to additionally export on-site soil, the import of clean, granular soil for fill, and the need to place a blanket of rock spalls or crushed rock in the construction traffic areas and on exposed subgrades prior to placing structural fill or structural elements.

We recommend that we review geotechnical aspects of the final project plans prior to construction. We also recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications.

Erosion Control Measures

The erosion hazard for the on-site soils is listed as slight to moderate, but the actual hazard will be dependent on how the site is graded and how water is allowed to concentrate. Best Management Practices (BMPs) should be used to control erosion. Areas disturbed during construction should be protected from erosion. Erosion control measures may include diverting surface water away from the stripped or disturbed areas. Silt fences should be erected to prevent muddy water from leaving the site or flowing over the site slopes. Stockpiles should be covered with plastic sheeting during wet weather. Disturbed areas should be planted as soon as practical, and the vegetation should be maintained until it is established. The erosion potential for areas that have been adequately revegetated should be low.

Site Preparation and Grading

After erosion control measures are implemented, site preparation should consist of stripping any organics, undocumented fill, or loose/soft soils to expose medium-dense or better bearing soils in foundation, slab, and pavement subgrade areas. Based on our explorations, we anticipate a stripping depth of up to 1.0 to 3.0 feet throughout the site. The stripped soil should be removed from the site or stockpiled for later use as a landscaping fill.

If the exposed subgrade after site stripping should appear to be loose, it should be compacted to a non-yielding condition. Areas observed to pump or weave during compaction should be reworked to structural fill specifications or over-excavated and replaced with properly compacted crushed rock or rock spalls. If significant surface water flow is encountered during construction, this flow should be diverted around areas to be developed and the exposed subgrades should be maintained in a semi-dry condition. We should be retained to observe and evaluate all subgrades prior to placing foundation concrete or hard surfaces. The subgrade preparation recommendations provided in the **Foundation Support** and **Slabs-on-Grade** subsections of this report should be followed for footings and slabs.

If wet conditions are encountered, alternative site grading techniques might be necessary. These could include using large excavators equipped with wide tracks and a smooth bucket to complete site grading and covering exposed subgrade with a layer of crushed rock for protection. If wet conditions are encountered or construction is attempted in wet weather, the subgrade should not be compacted, as this could cause further subgrade disturbance. In wet conditions, it may be necessary to cover the exposed subgrade with a layer of crushed rock as soon as it is exposed to protect the extremely moisture-sensitive soils from disturbance by machine or foot traffic during construction. The prepared subgrade should be protected from construction traffic and surface water should be diverted around areas of prepared subgrade.

Temporary and Permanent Slopes

Temporary cut slope stability is a function of many factors, including the type and consistency of soils, depth of the cut, surcharge loads adjacent to the excavation, length of time a cut remains open, and the presence of surface or groundwater. It is exceedingly difficult under these variable conditions to estimate a stable, temporary, cut slope angle. Therefore, it should be the responsibility of the contractor to maintain safe slope configurations at all times, as indicated in OSHA guidelines for cut slopes. The following information is provided solely for the benefit of the owner and other design consultants and should not be construed to imply that Nelson Geotechnical Associates, Inc. assumes responsibility for job site safety. Job site safety is the sole responsibility of the project contractor. For planning purposes, we recommend that temporary cuts in the on-site soils be no steeper than 1.5 horizontal to 1.0 vertical (1.5H:1.0V).

If temporary cut excavations are not able to achieve safe inclinations, we recommend temporary shoring be considered for the planned cuts. We are available to provide additional options and recommendations for temporary shoring, if needed, as the project plans are developed. If significant groundwater seepage or surface water flow were encountered, we would expect that flatter inclinations would be necessary. We recommend that cut slopes be protected from erosion. The slope protection measures may include covering cut slopes with plastic sheeting and diverting surface runoff away from the top of cut slopes. We do not recommend vertical slopes for cuts deeper than four feet if worker access is necessary. We recommend that cut slope heights and inclinations conform to appropriate OSHA/WISHA regulations. Permanent cut and fill slopes should be no steeper than 2.0H:1.0V. However, flatter inclinations may be required in areas where loose soils are encountered. Permanent slopes should be vegetated, and the vegetative cover maintained until established.

Foundation Support

The foundation support recommendations and ground improvements provided in the **Site Preparation and Grading** subsection above are intended to improve foundation performance and reduce the potential for post-construction total and differential settlements. We should review final foundation plans and be on site during earthwork construction to evaluate foundation area over-excavation and structural fill and rock spalls placement and compaction. We recommend that all foundations bear on native soil compacted to medium dense or better condition, or structural fill extending down to component native soils.

We should be on site during earthwork construction to evaluate foundation area excavation and any over-excavation, structural fill placement, and compaction. Building foundations should extend at least 24 inches below the lowest adjacent finished ground surface for frost protection and bearing capacity considerations. Footings should be sized based on the anticipated loads and allowable soil bearing pressure and should conform to current IBC guidelines. Water should not be allowed to accumulate in footing excavations. All loose or disturbed soil should be removed from the foundation excavation prior to placing concrete.

For foundations constructed as outlined above we recommend an allowable design bearing pressure of not more than 3,000 pounds per square foot (psf) be used for sizing the footings. We estimate that footings constructed in accordance with these recommendations will experience total settlements less than 1 inch and differential settlements less than ½ inch across a distance of about 20 feet. A representative of NGA should evaluate the foundation excavations. We should be consulted if higher bearing pressures are needed. Current IBC guidelines should be used when considering increased allowable bearing pressure for short-term transitory wind or seismic loads, and for snow loading.

Lateral loads may be resisted by friction on the base of the footing and passive resistance against the subsurface portions of the foundation. A coefficient of friction of 0.35 may be used to calculate the base friction and should be applied to the vertical dead load only. Passive resistance may be calculated as a triangular equivalent fluid pressure distribution. An equivalent fluid density of 200 pounds per cubic foot (pcf) should be used for passive resistance design for a level ground surface adjacent to the footing. This level surface should extend a distance equal to at least three times the footing depth. To achieve this value of passive resistance, the foundations should be poured “neat” against medium-dense soils or compacted fill should be placed against the footing. These recommended values incorporate safety factors of 1.5 and 2.0 applied to the estimated ultimate values for frictional and passive resistance, respectively.

Structural Fill

General: Fill placed beneath foundations, slabs-on-grade, pavement, or other settlement-sensitive structures should be placed as structural fill. Structural fill, by definition, is placed in accordance with prescribed methods and standards, and is monitored by an experienced geotechnical professional or soils technician. Field monitoring procedures would include the performance of a representative number of in-place density tests to document the attainment of the desired degree of relative compaction. The area to receive the fill should be suitably prepared as described in the **Site Preparation and Grading** subsection prior to beginning fill placement.

Materials: Structural fill should consist of good quality, granular soil, free of organics and other deleterious material, and be well graded to a maximum size of about three inches. If fill will be placed during wet weather or if wet conditions are present, the fill materials should contain no more than five percent fines (soil finer than U.S. No. 200 sieve, based on that fraction passing the U.S. 3/4-inch sieve). The use of some of the on-site soils as structural fill may be feasible, but this will be dependent on the moisture content of the material at the time construction takes place. Particles over 3.0 inches in diameter should be removed from material intended for use as structural fill. We should be retained to evaluate proposed structural fill material prior to placement.

Fill Placement: Following subgrade preparation, placement of structural fill may proceed. All fill placements should be accomplished in uniform lifts up to eight inches thick. Each lift should be spread evenly and be thoroughly compacted prior to placement of subsequent lifts. All structural fill underlying building areas and pavement subgrade should be compacted to a minimum of 95 percent of its maximum dry density. Maximum dry density, in this report, refers to that density as determined by the ASTM D-1557 Compaction Test procedure. The moisture content of the fill soils to be compacted should be within about two percent of optimum so that a readily compactable condition exists. It may be necessary to over-excavate and remove wet soils in cases where drying to a compactable condition is not feasible. It may be necessary to add moisture to dry soil so that a readily compactable condition is achieved. All compaction should be accomplished by equipment of a type and size sufficient to attain the desired degree of compaction and should be tested.

Slab-on-Grade

Slabs-on-grade both exterior and interior should be supported on subgrade soils prepared as described in the **Site Preparation and Grading** subsection of this report. Due to the presence of potentially loose soil encountered in our explorations, we recommend the subgrade under all slab areas be compacted to a dense and unyielding state or over-excavated a minimum of 12 inches and backfilled with 2- to 4-inch rock spalls or railroad ballast. The crushed rock should be placed and compacted as structural fill. The exposed subgrade should be thoroughly compacted prior to placing the crushed rock. We recommend that all interior floor slabs be underlain by at least six inches of free-draining sand or gravel for use as a capillary break. A suitable vapor barrier, such as heavy plastic sheeting (6-mil minimum), should be placed over the capillary break material. An additional 2-inch-thick layer of sand may be used to cover the vapor barrier. This sand layer is optional and mainly intended to protect the vapor barrier membrane during construction. The subgrade of slab areas should be compacted to a firm and unyielding state prior to placing the capillary break layer.

Pavements

Pavement subgrade should be prepared as described in the **Site Preparation and Grading** subsection of this report. The pavement section is determined by expected loading conditions, traffic volume, and desired longevity of the parking lot, among other factors. For the expected traffic type and volume, the recommended pavement section should consist of 8 inches of 1 ¼-inch to 2-inch clean crushed aggregate overlain by 4 inches of hot mix asphalt (HMA). The pavement subgrade should be compacted to a firm condition, then proof rolled using a loaded dump truck prior to placing the crushed rock. Areas observed to pump or weave during the proof roll test should be over-excavated and replaced with crushed rock.

Site Drainage

Surface Drainage: Water should not be allowed to collect in any area where footings are to be constructed. Final site grades should allow for drainage away from the structures. We suggest that the finished ground be sloped at a gradient of three percent minimum for a distance of at least 10 feet away from the structures and site slopes. Surface water should be collected by permanent catch basins and drain lines and be discharged to a suitable outlet. Surface drains should be maintained separately and not be interconnected with foundation or wall drains. The drains should consist of a minimum 4-inch diameter rigid, slotted, or perforated PVC pipe surrounded by free-draining material wrapped in a filter fabric. The drain should discharge into a tightline leading to an appropriate collection and discharge point with convenient cleanouts.

Subsurface Drainage: If groundwater seepage is encountered or if excessive rainfall or snowmelt occurs during construction, we recommend that the contractor slope the bottom of the excavations and direct the water to ditches and small sump pits. The collected water can then be pumped to a suitable discharge point. We recommend the use of footing drains around structures. Footing drains should be installed at least one foot below planned finished floor elevation. The drains should consist of a minimum 4-inch-diameter, rigid, slotted or perforated, PVC pipe surrounded by free-draining material wrapped in a filter fabric. We recommend that the free-draining material consist of an 18-inch-wide zone of clean (less than three-percent fines), granular material placed along the back of walls, extending up the wall to one foot below finished grade. Washed rock is an acceptable drain material or drainage composite may be used instead. The top foot of soil should consist of low permeability soil placed over plastic sheeting or building paper to minimize the migration of surface water or silt into the footing drain. Footing drains should discharge into tightlines leading to an appropriate collection and discharge point with convenient cleanouts to prolong the useful life of the drains. Roof drains should not be connected to wall or footing drains.

USE OF THIS REPORT

NGA has prepared this report for **Helion Energy**, and their agents, for use in the planning and design of the project planned on this site only. The scope of our work does not include services related to construction safety precautions and our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. There are possible variations in subsurface conditions between the explorations and also with time. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions. A contingency for unanticipated conditions should be included in the budget and schedule.

We recommend that NGA be retained to review project plans and consult with the design team during final design. We also recommend that NGA be retained to provide monitoring and consultation services during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork and foundation installation activities comply with contract plans and specifications. We should be contacted a minimum of one week prior to construction activities and could attend pre-construction meetings if requested.

Within the limitations of scope, schedule, and budget, our services have been performed in accordance with generally accepted geotechnical engineering practices in effect in this area at the time this report was prepared. No other warranty, expressed or implied, is made. Our observations, findings, and opinions are a means to identify and reduce the inherent risks to the owner.

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We appreciate the opportunity to provide service to you on this project. If you have any questions or require further information, please call.

Sincerely,

NELSON GEOTECHNICAL ASSOCIATES, INC.

Christopher Ward-Guthrie

Chris Ward-Guthrie, GIT
Project Geologist

KHALED M. SHAWISH
35215
11/1/24

Khaled M. Shawish, PE
Principal

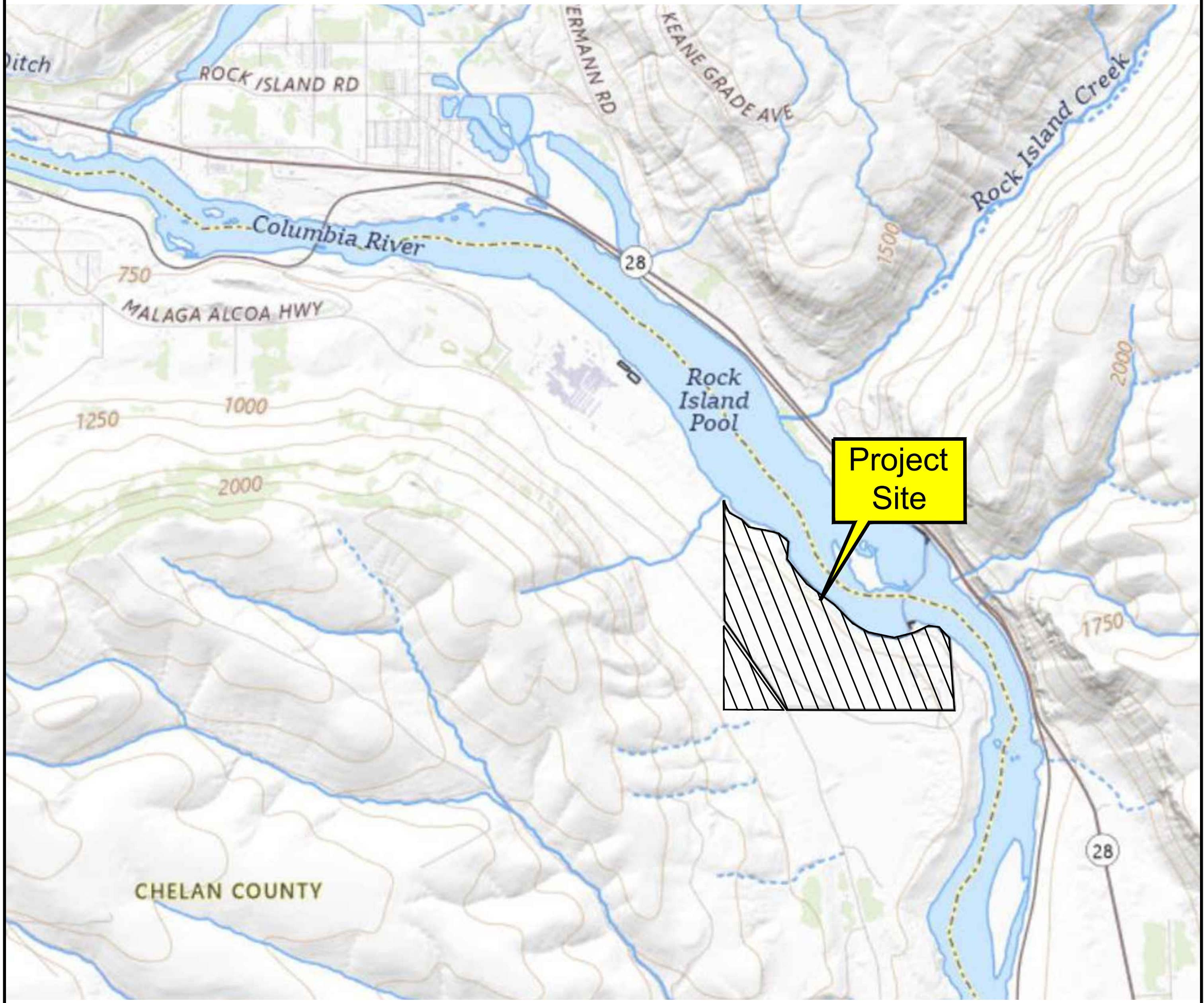
CWG:ABR:KMS:as

Eight Figures Attached

VICINITY MAP

Not to Scale

N



Chelan County, WA

Project Number
1549124

Figure 1

Helion Energy
Power Plant Facility
Vicinity Map

NELSON GEOTECHNICAL ASSOCIATES, INC

www.nelsongeotech.com

Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	10/17/24	Original	CWG	ABR

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Site Plan

NOT TO SCALE

N

LEGEND

— · —	Property line
B-1 ⌖	Number and approximate location of borings

0 600 1200

Approximate Scale: 1 inch = 600 feet

Reference: Site Plan based on Google Earth imagery, dated June 27, 2022.

Project Number
1549124

Figure 2

Helion Energy
Power Plant
Facility Site Plan

NELSON GEOTECHNICAL
ASSOCIATES, INC

www.nelsongeotech.com

Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	10/17/24	Original	CWG	ABR

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UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE - GRAINED SOILS MORE THAN 50 % RETAINED ON NO. 200 SIEVE	GRAVEL MORE THAN 50 % OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVEL	GW	WELL-GRADED, FINE TO COARSE GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVEL WITH FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SAND MORE THAN 50 % OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SAND	SW	WELL-GRADED SAND, FINE TO COARSE SAND
			SP	POORLY GRADED SAND
		SAND WITH FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE - GRAINED SOILS MORE THAN 50 % PASSES NO. 200 SIEVE	SILT AND CLAY LIQUID LIMIT LESS THAN 50 %	INORGANIC	ML	SILT
			CL	CLAY
		ORGANIC	OL	ORGANIC SILT, ORGANIC CLAY
	SILT AND CLAY LIQUID LIMIT 50 % OR MORE	INORGANIC	MH	SILT OF HIGH PLASTICITY, ELASTIC SILT
			CH	CLAY OF HIGH PLASTICITY, FAT CLAY
		ORGANIC	OH	ORGANIC CLAY, ORGANIC SILT
HIGHLY ORGANIC SOILS			PT	PEAT

NOTES:

1) Field classification is based on visual examination of soil in general accordance with ASTM D 2488-93.

2) Soil classification using laboratory tests is based on ASTM D 2488-93.

3) Descriptions of soil density or consistency are based on interpretation of blowcount data, visual appearance of soils, and/or test data.

SOIL MOISTURE MODIFIERS:

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water.

Wet - Visible free water or saturated, usually soil is obtained from below water table

Project Number
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Figure 3

Helion Energy
Power Plant Facility
Soil Classification Chart

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www.nelsongeotech.com

Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

No.	Date	Revision	By	CK
1	10/17/24	Original	CWG	ABR

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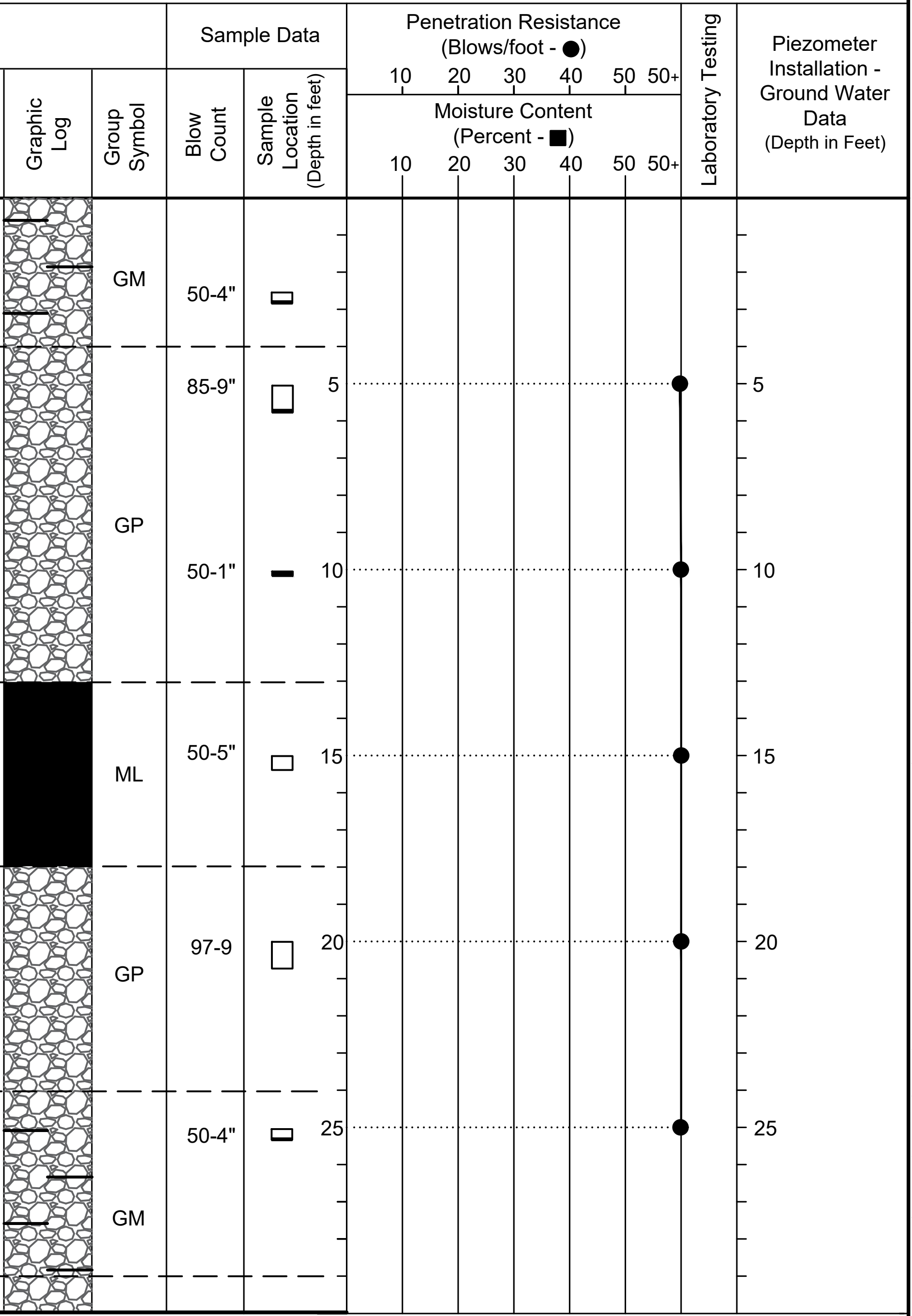
BORING LOG

B-1

Approximate Ground Surface Elevation:

Description	Group Symbol	Blow Count	Sample Location (Depth in feet)
Light brown, GRAVEL with sand and trace silt (very dense, dry)	GM	50-4"	
Gray, GRAVEL with medium to coarse sand (very dense, dry)	GP	85-9"	5
- Becomes moist		50-1"	10
Dark brown, SILT with some sand (very hard, moist) - No sample recovery	ML	50-5"	15
Gray, GRAVEL with coarse sand (very dense, moist) - No sample recovery	GP	97-9	20
Gray, GRAVEL with sand and silt (very dense, moist)	GM	50-4"	25
Gray, GRAVEL with coarse sand (very dense, moist)			



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler

- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit

- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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Helion Energy Power Plant
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Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
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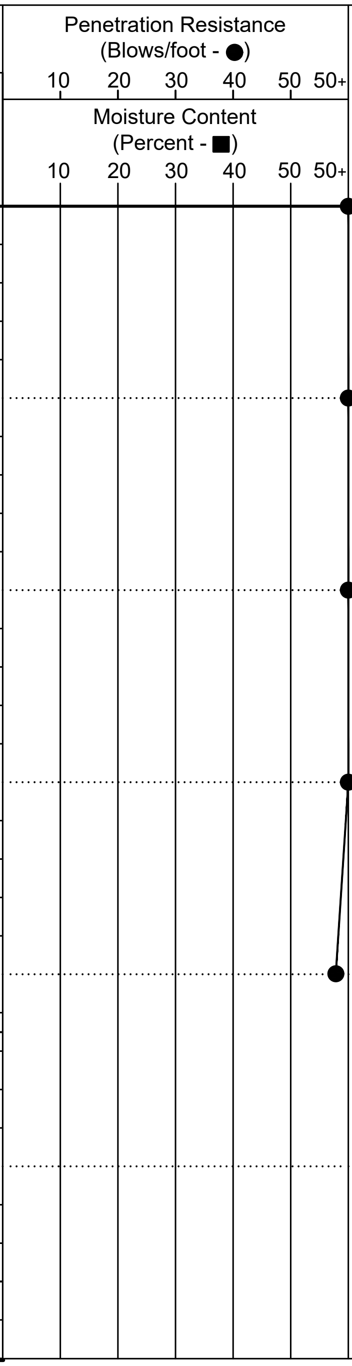
BORING LOG

B-1 (cont.)

Soil Profile			Sample Data			Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Description	Graphic Log	Group Symbol	Blow Count	Sample Location (depth in feet)			
			50-5"	▭			
- No sample recovery			50-3"	▭	35		35
- Increasing gravel size - No sample recovery		GP	50-3"	▭	40		40
					45		45
- No sample recovery			58-12"	▯	50		50
Boring completed at 51.5 feet below existing grade on 10/11/2024. No groundwater seepage was encountered during drilling.							
					55		55



LEGEND

- ▯ Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- ■ Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- 🟩 Solid PVC Pipe
- 🟩 Slotted PVC Pipe
- 🟥 Monument/ Cap to Piezometer
- * Liquid Limit
- + Plastic Limit
- ▨ Concrete
- ▨ Bentonite
- ▨ Native Soil
- ▨ Silica Sand
- ▼ Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

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Figure 4							
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Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
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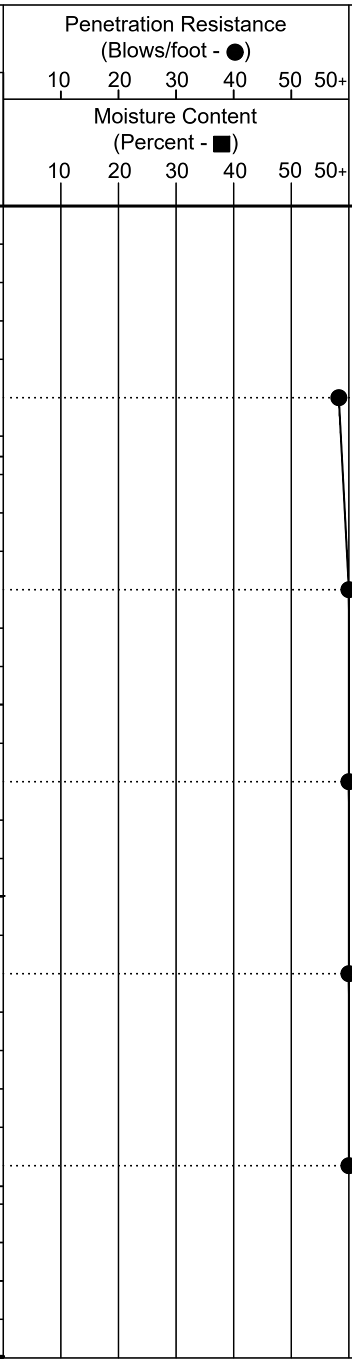
BORING LOG

B-2

Approximate Ground Surface Elevation:

Soil Profile		Sample Data		Laboratory Testing	Piezometer Installation - Ground Water Data (Depth in Feet)
Description	Group Symbol	Blow Count	Sample Location (Depth in feet)		
Brown, GRAVEL with silty fine sand (very dense, dry)	GM				
- No sample recovery		58	5		5
Brown, GRAVEL with fine sand and silt (very dense, dry)	GM				
- No sample recovery		50-3"	10		10
Brown, GRAVEL with fine sand (very dense, dry)	GP	50-6"	15		15
Brown, GRAVEL with fine silty sand (very dense, dry)	GM	50-4"	20		20
		50-2"	25		25
Boring completed at 25.5 feet below existing grade on 10/11/2024. No groundwater seepage was encountered during drilling.					



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- \+ Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- ▼ Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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Helion Energy Power Plant
Boring Log

NELSON GEOTECHNICAL
ASSOCIATES, INC

Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
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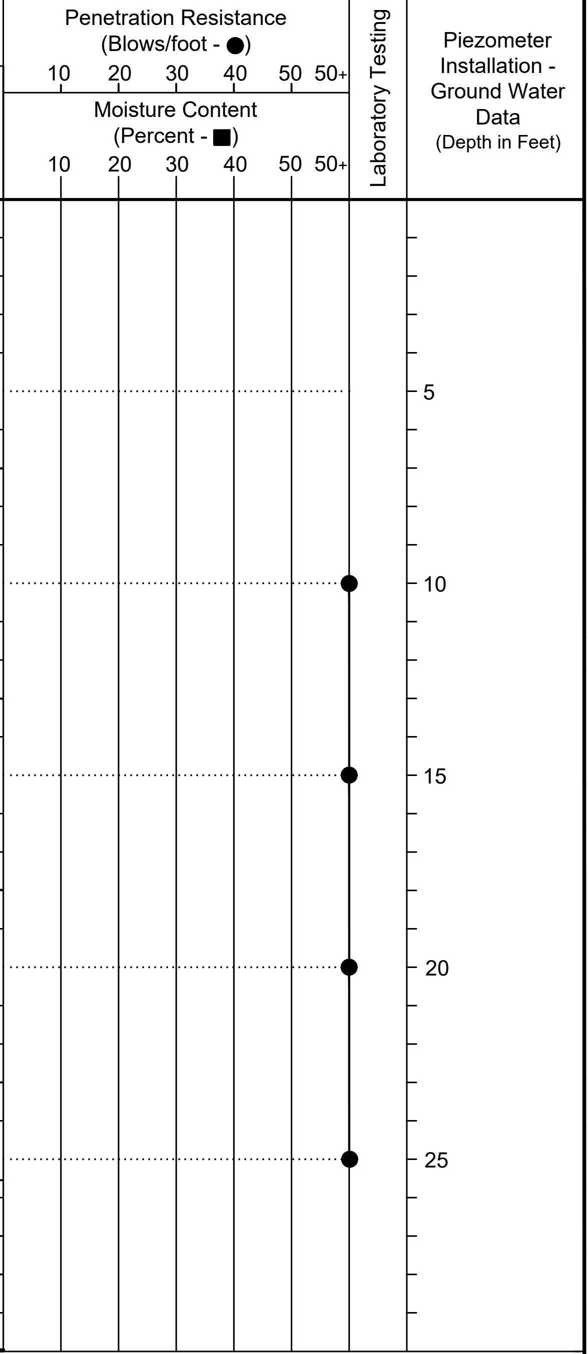
BORING LOG

B-3

Approximate Ground Surface Elevation:

Description	Group Symbol	Blow Count	Sample Location (Depth in feet)
Brown, GRAVEL with silt and sand (very dense, dry) - No sample due to severe caving	GM		5
Gray, GRAVEL with medium to coarse sand (very dense, dry) - Little sample recovery	GP	50-3"	10
Gray, silty GRAVEL with sand (very dense, dry) - Little sample recovery	GM	50-5"	15
Gray, silty GRAVEL (very dense, dry)	GP	50-4"	20
Gray, GRAVEL (very dense, moist) - No sample recovery	GM	50-2"	25
Boring completed at 25.5 feet below existing grade on 10/11/2024. No groundwater seepage was encountered during drilling.			



LEGEND

- Depth Driven and Amount Recovered with 2-inch O.D. Split-Spoon Sampler
- Depth Driven and Amount Recovered with 3-inch Shelby Tube Sampler
- Solid PVC Pipe
- Slotted PVC Pipe
- Monument/ Cap to Piezometer
- * Liquid Limit
- \+ Plastic Limit
- Concrete
- Bentonite
- Native Soil
- Silica Sand
- Water Level

M	Moisture Content
A	Atterberg Limits
G	Grain-size Analysis
DS	Direct Shear
PP	Pocket Penetrometer Readings, tons/ft
P	Sample Pushed
T	Triaxial

NOTE: Subsurface conditions depicted represent our observations at the time and location of this exploratory hole, modified by engineering tests, analysis and judgement. They are not necessarily representative of other times and locations. We cannot accept responsibility for the use or interpretation by others of information presented on this log.

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Boring Log

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Woodinville Office
17311-135th Ave. NE, A-500
Woodinville, WA 98072
(425) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
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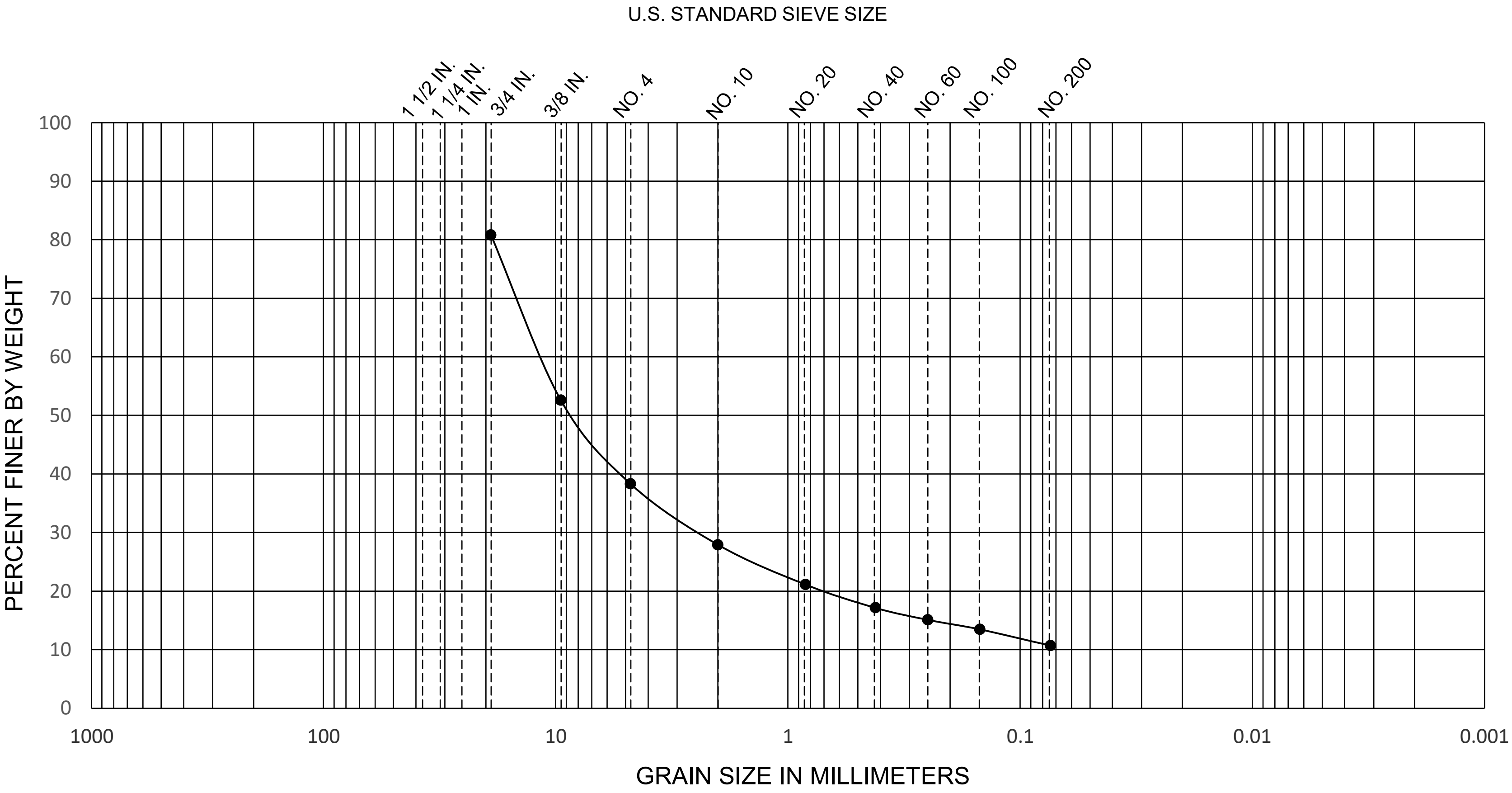

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
GP-GM	B-3	20.0 feet	Grey, poorly graded GRAVEL with some sand and trace silt	Gravel = 61.7% Sand = 27.6% Fines = 10.7%

Helion Energy Power Plant Facility
Infiltration Sieve Analysis

NELSON GEOTECHNICAL ASSOCIATES, INC

Woodinville Office
17311 135th Ave. NE, A-500
Woodinville WA 98072
(25) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

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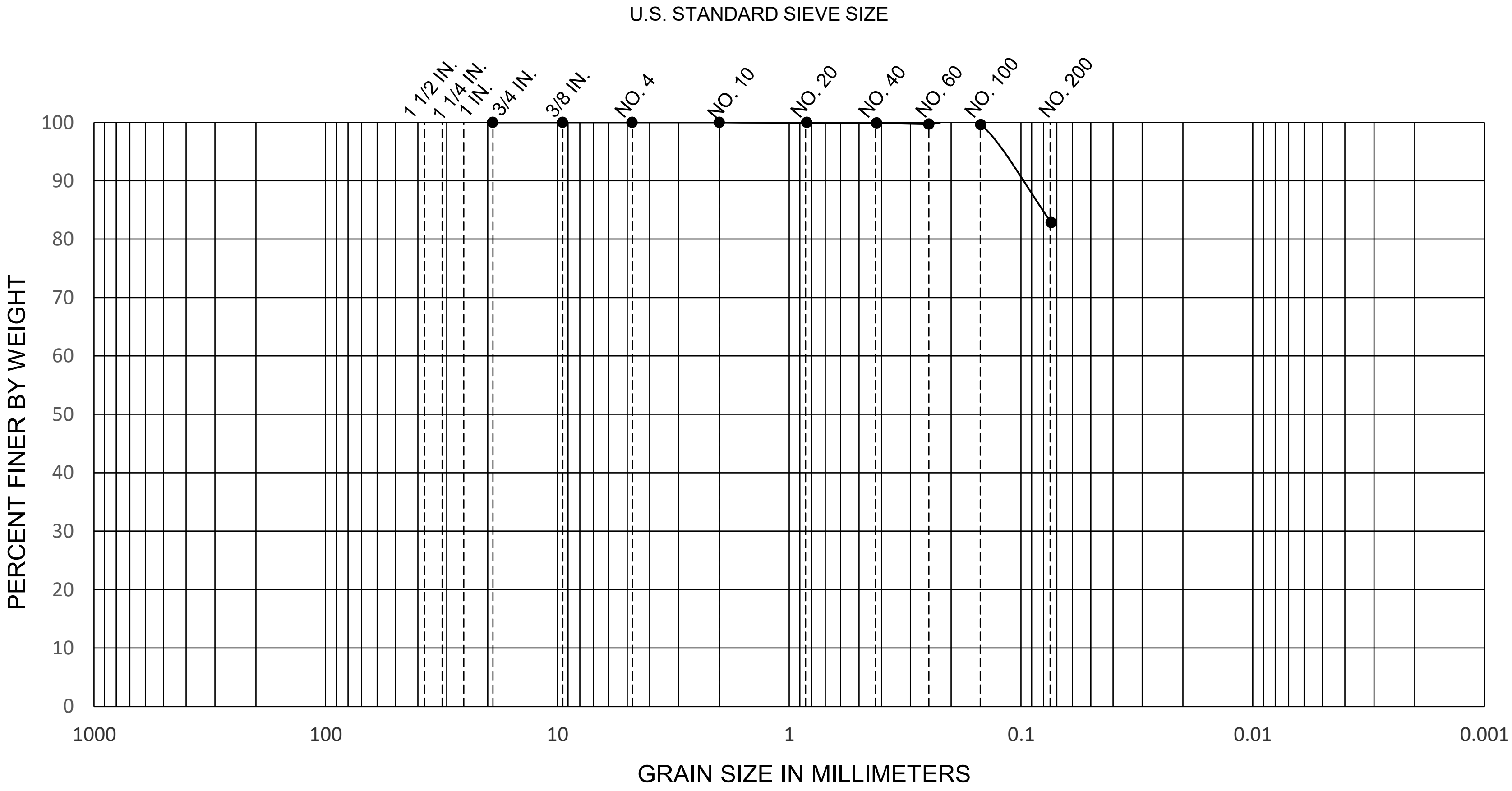
Figure 7



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

USCS SYMBOL	EXPLORATION NUMBER	SAMPLE DEPTH	SOIL DESCRIPTION	SOIL DISTRIBUTION
ML	B-1	13.0 feet	Dark brown, SILT with some sand	Gravel = 0.0% Sand = 17.2% Fines = 82.8%

Helion Energy Power Plant Facility
Sieve Analysis

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ASSOCIATES, INC

Woodinville Office
17311 135th Ave. NE, A-500
Woodinville WA 98072
(25) 486-1669 / Fax: 481-2510

Wenatchee Office
105 Palouse St.
Wenatchee, WA 98801
(509) 665-7696 / Fax: 665-7692

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Figure 8