NOTE FROM SMR:
This report contains site soil and groundwater conditions. CHE continues to monitor the site conditions and will be updating this report with the new information. Certain site and building elements will be updated when that information is available.

HOUSING AUTHORITY OF SNOHOMISH COUNTY TIMBERGLEN & PINEWOOD REDEVELOPMENT

GEOTECHNICAL ENGINEERING REPORT

CIANI & HATCH ENGINEERING

Lynnwood, WA

for

Housing Authority of Snohomish County
April 5, 2024

Geotechnical Engineering Report

Housing Authority of Snohomish County
Timberglen & Pinewood Redevelopment
Lynnwood, Washington

CHE File No. 2002-001-00
April 5, 2024

Prepared for:

Housing Authority of Snohomish County
12711 4th Ave W
Everett, WA 98204

Attention: Chris Bogdan

Prepared by:

CIANI & HATCH ENGINEERING, PLLC

Mikayla Hatch, PE
Geotechnical Engineer

MIKAYLA SON HATCH
STATE OF WASHINGTON
21018811
REGISTERED
PROFESSIONAL ENGINEER
4/5/2024

W
Associate

CEM:MSH:WLC

CIANI & HATCH
ENGINEERING

Housing Authority of Snohomish County
Timberglen & Pinewood Redevelopment

CONTENTS

FIGURES

TABLES

APPENDICES

ABBREVIATIONS AND ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
AL	Atterberg Limit
ASCE	American Society of Civil Engineers
ATD	at time of drilling
bgs	below ground surface
CA	combined analysis
CHE	Ciani & Hatch Engineering, PLLC
CSBC	crushed surfacing base course
DOE	Washington State Department of Ecology
HASCO	Housing Authority of Snohomish County
IBC	International Building Code
ksf	kips per square foot
pcf	pounds per cubic foot
pci	pounds per cubic inch
USGS	U.S. Geological Survey
WAC	Washington Administrative Code
WSDOT	Washington State Department of Transportation

This page intentionally left blank.

1.0 INTRODUCTION

This report presents geotechnical engineering conclusions and recommendations based on the results of Ciani & Hatch Engineering, PLLC's (CHE) field investigation and laboratory testing program for the proposed Timberglen & Pinewood Redevelopment project (project) in Lynnwood, Washington. The general project location is shown on the Project Vicinity Map (Figure 1); the site is shown relative to existing features and infrastructure on the Geotechnical Exploration Site Plan (Figure 2).

1.1 PROJECT UNDERSTANDING

This report was prepared based on our discussions with, and information provided by, the project owner, Housing Authority of Snohomish County (HASCO). CHE understands HASCO proposes to demolish the existing multi-family housing structures and construct new multi-family housing units to increase capacity. The Site comprises three Snohomish County Parcels (00565300001501, 00565300001502, and 00565300001505) with a total site area of 2.57 acres.

Preliminary design concept plans were provided to CHE by HASCO on January 31, 2024. Based on these plans, the new housing will consist of up to five new buildings with up to two partially below-grade parking structures that will daylight downslope to the east. Concept plans indicate limited to no excavation will be required to install these below-grade structures due to the natural existing slope of the site.

1.2 SCOPE OF SERVICES

The purpose of our design services for the Timberglen & Pinewood redevelopment was to explore the subsurface soil and groundwater conditions at the site as a basis for providing geotechnical design recommendations for the project. Our scope of services have been provided in accordance with our proposal for geotechnical engineering services dated February 2, 2024. The services in that proposal were incorporated into the subcontract between HASCO and CHE executed February 5, 2024.

2.0 EXISTING CONDITIONS

The following sections provide a description of the existing surface conditions, the geologic setting, and a summary of the subsurface soil and groundwater conditions observed in CHE's field investigation.

2.1 SURFACE CONDITIONS

The site is currently occupied by six two-story apartment buildings and associated asphalt pavement parking lots along 200th St SW between 60th Ave W and 56th Ave W. The buildings and parking lots are bordered by 200th St SW to the north and residential properties to the east, south, and west. The existing ground surface slopes from west to east from approximate elevation 385 feet to elevation 360 feet.[1] The property is landscaped with grass and mature coniferous trees.

Buried utilities are located within and near the project site and within the public right of way along the adjacent streets. These utilities include, but are not limited to, electrical, telecommunication, water, sanitary sewer, and storm drains.

[1] All project elevations reference the North American Vertical Datum of 1988 (NAVD88).

2.2 GEOLOGIC SETTING

Geologic information for the project area was obtained from the Geologic Map of the Edmonds East and part of the Edmonds West quadrangles, Washington (Minard, 1983). Surficial deposits mapped within the project area consist of Vashon till.

Vashon till generally consists of an unsorted mixture of boulders, cobbles, gravel, and sand within a matrix of silt and clay. Vashon till is comprised of Pleistocene deposits that have been mixed and transported before being deposited, overridden, and compacted by the advancing glacier during the Vashon Glaciation. This produced a largely unsorted deposit of clay, silt, sand, and gravel that has low permeability and high strength. Weathered till was encountered in some exploratory borings and has significantly lower strength. Water tends to percolate through the weathered and remolded till more readily but typically pools and moves laterally along the undisturbed glacial till.

The soils encountered during CHE's field investigation program were generally consistent with the mapped geology.

2.3 SUBSURFACE EXPLORATION

Subsurface conditions were explored between February 22 and 23, 2024, by advancing nine hollow-stem auger borings (CHE-B1-24 through CHE-B9-24) to depths of 21.5 to 26.5 ft below ground surface (bgs). Two of these borings (CHE-P3-24 and CHE-P8-24) were completed as standpipe piezometer groundwater monitoring wells. The approximate locations of the explorations are shown on Figure 2.

CHE personnel monitored the field explorations, collected representative soil samples, and maintained detailed logs of the subsurface soil and groundwater conditions. Summary boring logs and a description of the geotechnical field investigation are provided in Appendix A.

Soil samples were transported to HWA GeoScience's AASHTO-accredited geotechnical laboratory in Bothell, Washington, for further examination and testing. Test results and a description of the geotechnical laboratory testing program are provided in Appendix B.

2.3.1 Soil Conditions

The soils observed underlying existing surface conditions (i.e., asphalt pavement section, gravel surfacing) were characterized into four general units:

Fill: Encountered in borings CHE-B1-24, CHE-P3-24, CHE-B7-24, and CHE-P8-24 to depths up to 13 feet below grade. The fill generally consisted of sand with gravel and variable silt content, portions likely deriving from remolded till deposits used as fill. The fill encountered was typically dark brown in color, very loose to medium dense, and included scattered organic material.

Peat: Peat was observed underlying the fill in exploration CHE-B7-24. This unit was approximately 1.5 feet in thickness and typically consisted of medium stiff organic silt.

Weathered glacial till: Weathered glacial till was observed underlying the peat in exploration CHE-B7-24 and the fill in CHE-P3-24. This unit ranged in thickness of 4.5 to 7.5 feet and typically consisted of loose to dense, silty, fine to coarse sand with varying gravel and cobble content. This unit was typically brown to gray in color with occasional mottling and generally in a moist condition.

Glacial till: Glacial till was observed underlying the weathered glacial till in explorations CHE-P3-24 and CHE-B7-24, the fill in explorations CHE-B1-24 and CHE-P8-24, and the pavement in explorations CHE-B2-24, CHE-B4-24, CHE-B5-24, CHE-B6-24, and CHE-B9-24; all explorations terminated in glacial till deposits. The glacial till typically consisted of gray, fine to coarse sand with varying silt and gravel content and sandy silt with oxidation staining. The glacial till was generally dense to very dense or hard and in a moist to wet condition.

Cobbles and boulders are often encountered in glacial deposits and may be present at the project site. The presence of cobbles, as noted on the boring logs, is inferred from observations of drilling action and broken pieces of cobbles in samplers. Contractors should be prepared to deal with these conditions.

2.4 EXISTING PAVEMENT CONDITIONS

All borings were advanced in a paved parking lot. Characteristics of the existing pavement sections are summarized below in Table 1.

Table 1. Summary of Existing Pavement Conditions

Boring Designation	Asphalt Concrete Pavement Thickness (inches)	Base Course Thickness (inches)
CHE-B1-24	2	2
CHE-B2-24	2	2
CHE-P3-24	2	3
CHE-B4-24	2	2
CHE-B5-24	2	0
CHE-B6-24	3	0
CHE-B7-24	2	3
CHE-P8-24	1.5	2
CHE-B9-24	1	2

2.4.1 Groundwater Conditions

Groundwater conditions were observed during the geotechnical drilling program. Relative moisture content was noted for each sample and, when first indicated by an increase in soil moisture or from wet sampling equipment, drilling was paused for 20 to 30 minutes to allow groundwater to collect and equilibrate within the borehole. After this pause, a depth-to-groundwater measurement was made through the hollow-stem auger with an electronic water-level indicator. At time of drilling (ATD) groundwater measurements are noted in the summary boring logs provided in Appendix A.

Groundwater levels observed at the time of drilling require time to equilibrate due to fines present in the soil and drilling action and may not be representative of actual groundwater conditions at the site. Drilling action can result in smearing of fine-grained soils along the shaft of the borehole, decreasing the hydraulic conductivity of the soil. Groundwater can take up to hours or days to fully equilibrate.

Groundwater measurements were taken in the standpipe piezometer wells (borings CHE-P3-24 and CHE-P8-24) immediately following well installation on February 22 and 23, 2024, and approximately 1 week later on March 5, 2024. Recorded groundwater levels are provided in Table 2.

Table 2. Summary of Groundwater Measurements

Monitoring Well ID (Boring)	Ground Surface Elevation (ft)	Depth to Groundwater, ft [Groundwater Elevation, ft]		
		2/22/24	2/23/24	3/5/2024
BPG 809 (CHE-P3-24)	366.12	7.4 [358.72]	4.67 [361.45]	3.6 [362.52]
BPG 810 (CHE-P8-24)	380.29	-	11 [369.29]	6.1 [374.19]

a. Vertical datum is NAVD88
b. ft = feet
c. ID = identification

The groundwater information reported herein and on the summary logs contained in Appendix A is for the specific location and date indicated. We anticipate that groundwater conditions will vary depending on local subsurface conditions, weather conditions, season, and other factors. Groundwater levels in the project area are expected to fluctuate seasonally, with maximum groundwater levels generally occurring during the late winter and early spring.

2.5 ENVIRONMENTALLY CRITICAL AREAS

Environmentally Critical Areas Inventory maps provided by the City of Lynnwood were reviewed for the project site. A Type F Stream (Fish Bearing) is mapped adjacent to the east side of the site (City of Lynnwood 2024a). There are no mapped native growth protection areas, wetlands, steep slopes, erosion hazard areas, potential landslide, frequently flooded areas, or critical aquifer recharge areas mapped within the project limits (City of Lynnwood 2024b-f).

3.0 CONCLUSIONS AND RECOMENDATIONS

A summary of the primary geotechnical considerations is provided below. This summary is provided for introductory purposes only and should be used in conjunction with the complete recommendations presented in this report.

- The site is designated as Site Class C per the 2021 IBC.
- The glacial till deposits encountered in our site explorations represent good bearing soils for shallow foundations. Building foundations bearing on glacial till can be supported on spread or mat foundations with a design bearing pressure of 9 kips per square foot (ksf).
- Recent deposits and weathered glacial till were encountered overlying competent glacial till to depths of up to 20 feet along the eastern extents of the site (see borings CHE-B7-24 and CHE-P3-24). Based on discussions with the project team and the anticipated structure locations, one or more of the proposed buildings may be partially founded within the recent and/or weathered glacial deposits. For buildings founded or partially founded along the eastern extent of the site, we recommend that pin piles or ground improvement be utilized to transfer foundation loads to the glacial-till bearing layer. Alternatively, recent deposits and weathered glacial till soils may be over-excavated and replaced with compacted structural fill. Building foundations bearing on improved ground or structural fill extending to competent glacial till may be designed with a bearing pressure of 6 ksf.

- For the current development plan, site excavations for the majority of the Site will not extend more than 6 feet deep, which is the recommended design groundwater table depth for the western extents of the Site. Accordingly, no temporary dewatering is anticipated to complete the planned improvements in this area. If over-excavation and replacement is selected for support of foundations along the eastern portion of the site, excavations are anticipated to extend below the static groundwater table and active dewatering may be required.
- Slabs-on-grade are considered appropriate for the site and should be underlain by a 6-inch-thick layer of clean crushed rock. The underslab drainage system for each proposed building should consist of a perimeter foundation drain and one longitudinal drain.
- The potential for on-site infiltration is low, based on the presence of shallow glacial till and the results of our laboratory testing program .

Specific geotechnical engineering recommendations are presented in the following sections of this report.

3.1 EARTHQUAKE ENGINEERING

3.1.1 Seismicity

The Puget Sound area is a seismically active region that has experienced thousands of earthquakes in historical time. Seismicity in this region is attributed primarily to the interaction between the Pacific, Juan de Fuca, and North American plates. Potential seismic hazards from earthquakes include ground shaking, liquefaction, ground rupture from lateral spreading, and surface fault rupture. Our opinions regarding the likelihood of these seismic hazards occurring at the site are presented below. These opinions are based on the seismicity criteria recommended by the 2021 edition of the IBC.

3.1.2 Ground Shaking

There is a risk of earthquake-induced ground shaking at the site, as with all sites in the Puget Sound region, and the intensity of the ground shaking could be severe. The severity of ground shaking will mostly be a function of the earthquake’s magnitude and proximity to the site. In our opinion, strong ground shaking should be considered in the design of the planned structures at the site. We recommend that the seismic ground shaking at the site be evaluated in accordance with the 2021 IBC.

3.1.3 2021 IBC Seismic Design Information

We recommend the 2021 IBC parameters for Site Class, short period spectral response acceleration (S_S), 1-second period spectral response acceleration (S_1), and Seismic Coefficients F_A and F_V as presented in Table 3.

Table 3. 2021 IBC Seismic Design Parameters

2018 IBC Parameter	Recommended Value
Risk Category	II
Site Class	C
Short Period Spectral Response Acceleration, S_s (percent g)	1.298
1-Second Period Spectral Response Acceleration, S_1 (percent g)	0.458
Site amplification factor at 0.2s, F_a	1.2
Site amplification factor at 1.0s, F_v	1.5

3.1.4 Liquefaction Potential

Liquefaction refers to a condition where vibration or shaking of the ground, usually from earthquake forces, results in development of excess pore pressures in saturated soils and subsequent loss of strength in the deposits of soil so affected. Ground settlement, lateral spreading, and/or sand boils may result from soil liquefaction. Structures supported on liquefied soils could suffer foundation settlement or lateral movement that could be severely damaging to structures. Conditions favorable to liquefaction occur in loose to medium dense, clean to moderately silty sand that is below the groundwater level.

We evaluated the liquefaction-triggering potential (Youd et al. 2001; Idriss and Boulanger 2014) and liquefaction-induced settlement (Tokimatsu and Seed 1987; Ishihara and Yoshimine 1992; Idriss and Boulanger 2014) for the soils encountered in the borings at the site. Based on our evaluation and analysis, the site poses a low risk of liquefaction hazard. However, the loose fill soils within CHE-P3-24 contain a localized liquefaction hazard due to the presence of perched/shallow groundwater. We estimate up to 0.7 inches of liquefaction-induced settlement could occur if the loose fill soils are not removed and replaced with structural fill or recompacted to a firm and unyielding condition.

3.1.5 Surface Fault Rupture

The site is located within the southern Whidbey Island fault zone and is approximately 0.32 miles from a mapped fault (USGS 2023). Based on the fault distance from the site, the risk of fault displacement resulting in ground rupture at the site surface is considered low.

3.2 FOUNDATION SUPPORT

We anticipate a conventional shallow foundation system (e.g., spread or continuous footings) will be feasible for the majority of the site due to the presence of shallow, glacially consolidated soil. Geotechnical explorations completed for this study indicate approximately 15 to 20 feet of loose and compressible soil along the eastern extent of the site that are unsuitable for shallow foundation support. This soil must be removed and replaced with adequately compacted structural fill extending to competent glacial till. Depending on the design bearing elevation in this area, removal and replacement of the unsuitable soil may not be feasible. If this is the case, we anticipate targeted use of pin piles or a conventional shallow foundation system underlain by soils improved with engineered aggregate piers (EAPs]) will be required.

The following sections discuss each of these foundation options.

3.2.1 Shallow Spread Footings

Conventional shallow foundations transfer bearing load near the ground surface. These foundations are cost effective and ideal when the upper layer of soil possesses adequate bearing capacity to support the structure without the need for extensive excavation or deep penetration into the ground. Shallow spread footings bearing on undisturbed glacial till, or on structural fill extending down to these soils, will be feasible for most of the proposed structures across the site.

The design frost depth for the Puget Sound area is 12 inches; therefore, we recommend that all exterior footings be founded at least 18 inches below the lowest adjacent finished grade. Interior footings should be founded at least 12 inches below bottom of slab or adjacent finished grade. For shallow foundation support of the new structures, we recommend widths of at least 24 and 48 inches for continuous wall and isolated column footings, respectively.

3.2.1.1 ALLOWABLE BEARING PRESSURE

For footings supported on undisturbed and unweathered glacial till or glacially consolidated soils, an allowable soil bearing pressure of 9 ksf may be used. For footings supported on structural fill extending down to undisturbed glacial soils described above or on weathered glacial till, an allowable soil bearing pressure of 6 ksf may be used. The zone of structural fill below the foundation should extend beyond the faces of the footing a distance at least equal to the thickness of the structural fill. The zone of structural fill should be compacted to at least 95 percent of the maximum dry density (MDD) in general accordance with ASTM International (ASTM) D 1557. The allowable soil bearing value applies to the total of dead and long-term live loads and may be increased by up to one-third for wind or seismic loads.

3.2.1.2 FOUNDATION SETTLEMENT

We estimate that the post-construction settlement of footings founded as recommended above will be less than 1 inch. Differential settlement between comparably loaded column footings or along a 25-foot section of continuous wall footing should be less than ½ inch. We expect most of the footing settlements will occur during construction as loads are applied. Loose or disturbed soils not removed from footing excavations prior to placing concrete will result in additional settlement.

3.2.1.3 LATERAL RESISTANCE

Lateral foundation loads may be resisted by passive resistance on the sides of footings and by friction on the base of the shallow foundations. For shallow foundations supported on native soils, the allowable frictional resistance may be computed using a coefficient of friction of 0.4 applied to vertical dead-load forces.

Passive resistance may be estimated using an equivalent fluid density of 450 pounds per cubic foot (pcf) above the groundwater table and 335 pcf below the groundwater table. These values assume that the footings and below-grade elements are poured directly against undisturbed glacially consolidated soils or are backfilled with structural fill placed and compacted as recommended.

The above coefficient of friction and passive equivalent fluid density values incorporate a factor of safety of about 1.5.

3.2.1.4 FOOTING DRAINAGE

We recommend that perimeter footing drains be installed around each building addition and the parking garage and at the base of the exterior footings and below-grade walls. The perimeter footing drains should be provided with cleanouts and should consist of at least 4-inch-diameter perforated pipe placed on a 2-

inch bed of, and surrounded by, 6 inches of drainage material enclosed in a non-woven geotextile such as TenCate Mirafi 140N (or approved equivalent) to prevent fine soil from migrating into the drainage material.

We recommend that the drain pipe consist of either heavy-wall perforated pipe (SDR-35 PVC, or equal) or rigid corrugated smooth interior polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for footing drain pipes. The drainage material should consist of Gravel Backfill for Drains per Section 9-03.12(4) of the 2024 Washington State Department of Transportation (WSDOT) Standard Specifications (WSDOT 2024).

The perimeter drains should be sloped to drain by gravity, if practicable, to a suitable discharge point (preferably a storm drain). We recommend that the cleanouts be covered and placed in flush-mounted utility boxes. Water collected in roof downspout lines must not be routed to the footing drain lines.

3.2.1.5 CONSTRUCTION CONSIDERATIONS

Immediately prior to placing concrete, all debris and loose soils that have accumulated in the footing excavations during forming and steel placement must be removed. Debris or loose soils not removed from the footing excavations will result in increased settlement.

If wet-weather construction is planned, we recommend that all footing subgrades be protected using a lean concrete mud mat. The mud mat should be placed the same day that the footing subgrade is excavated and approved for foundation support.

We recommend that all completed footing excavations be observed by a representative of our firm prior to placing mud mat, reinforcing steel, and structural concrete. Our representative will confirm that the bearing surface has been prepared in a manner consistent with our recommendations and that the subsurface conditions are as expected.

3.2.2 Eastern Portion of Site Foundation Support

Unsuitable bearing soils were observed in borings CHE-P3-24 and CHE-B7-24 to depths of 15 to 20 feet bgs, respectively. Where removal and replacement of these soils is not feasible, we recommend structural loads be transferred to the glacial-till bearing layer using Engineered Aggregate Piers or Pin Piles, as described in the following sections.

3.2.2.1 ENGINEERED AGGREGATE PIERS

EAP systems are installed using either a displacement or a replacement (drilled) method. Drilled EAPs are ideal for non-caving soils and are constructed by augering 14- to 36-inch-diameter holes below the bottom of footing and backfilling the holes with thin lifts of compacted aggregate. Displacement EAPs are ideal for caving soils and/or environmentally impacted soils and are constructed by driving a 12- to 20-inch hollow mandrel into the ground and then feeding aggregate through the bottom of the mandrel to construct the pier. The displacement mandrel provides temporary casing during installation that prevents the caving soils from contaminating the pier.

In both methods, the first lift of aggregate prestresses/prestrains the soils at the bottom of the EAP, creating a solid base from which to construct the rest of the pier. The pier is then built in subsequent compacted lifts to the ground surface. Pier construction is performed with a high-frequency impact hammer that delivers vertical ramming energy to the aggregate backfill. In addition to creating a dense column of aggregate, the pier installation process also forces the aggregate laterally into the sidewalls of the hole, increasing the lateral stress in the matrix soils surrounding the EAP elements. The end result is a composite mass of dense pier elements surrounded by stiffened matrix soils that allow for higher design bearing pressures and reduced settlement.

For this project, we anticipate the EAPs will extend to depths in the range of 15 to 20 feet bgs to improve the loose and compressible fill soils, recent deposits (silt and peat), and weathered glacial till observed in borings advanced in this area of the project site. After pier installation is complete, the spread footings can be constructed directly on or over the EAP elements and no structural connection is required between the footing and the EAP elements. If EAPs are the preferred foundation solution, CHE should be retained to provide guidance and review of the contractordesigned system.

3.2.2.2 PIN PILES

Pin piles serve as a practical and efficient solution for building foundations, particularly in areas with challenging soil conditions or limited space for traditional foundation methods. Their versatility and adaptability make them well suited for supporting various types of structures, including residential, commercial, and industrial buildings.

When deep deposits of loose, unsuitable bearing soils are present, pin piles offer a reliable means of transferring structural loads to deeper, more stable layers of soil. They provide a cost-effective alternative to deep foundation systems such as drilled shafts or driven piles, as they require less installation effort, mobilization, and material while still offering substantial load-bearing capacity. Furthermore, pin piles can be installed quickly and with minimal disruption to surrounding areas, making them suitable for projects in urban or densely populated areas where noise and vibration must be minimized. Overall, the adaptability, efficiency, and reliability of pin pile foundations make them a preferred choice for building construction projects seeking robust support systems in challenging soil conditions.

Pin piles typically consist of 4- to 10-inch-diameter steel pipe piles. Pin piles are installed using pneumatic hammers and are driven to refusal. Pin piles at this site would be anticipated to extend 15 to 25 feet bgs. Pin piles would require structural detailing to tie the piles into the foundation system. If pin piles are the preferred foundation solution, CHE should be retained to provide final design recommendations.

3.3 FLOOR SLAB

Conventional slabs on grade are appropriate when the structures are supported on undisturbed native glacial soils or on at least 2 feet of structural fill, provided the subgrade soils are prepared as recommended in Section 3.7.2. Based on the borings completed at the site, we anticipate that competent dense to very dense glacial till will be present 0 to 20 feet below existing site grades. We recommend that where undisturbed native glacial soils are not present, the upper 2 feet of disturbed or loose fill soils be removed and replaced with properly compacted structural fill.

3.3.1 Subgrade Preparation

We recommend that concrete slabs-on-grade be constructed on a 6-inch-thick layer of clean crushed rock to provide uniform support and to act as a capillary break. The 6-inch crushed rock layer should be placed on structural fill or approved native glacial soils. Prior to placing the capillary break layer or structural fill, the exposed subgrade should be proof-rolled or evaluated as described in Section 3.7.2. Where existing fill is exposed, the subgrade soils should be recompacted to a firm condition.

3.3.2 Design Parameters

The capillary break layer should consist of at least 6 inches of clean crushed rock with a maximum particle size of 1 inch and negligible sand and silt, in accordance with Section 9-03.1(4)C, grading No. 67 of the 2024 WSDOT Standard Specifications. The clean crushed rock will provide uniform support and form a capillary break beneath the slab. For slabs designed as a beam on an elastic foundation, a modulus of subgrade reaction of 75 pounds per cubic inch (pci) may be used for subgrade soils prepared as recommended above.

If water vapor migration through the slabs is objectionable and/or moisture cannot be tolerated (such as areas that will have vinyl, tile, or carpet), the capillary break material should be covered with a heavy plastic sheet to act as a vapor retarder. The vapor barrier should have sufficient thickness to withstand foot traffic during placement of reinforcement and slab concrete and be constructed in accordance with the American Concrete Institute (ACI 302.1R). The contractor should be made responsible for maintaining the integrity of the vapor barrier during construction.

3.3.3 Below-Slab Drainage

Groundwater could accumulate below the structure slabs-on-grade because of perched groundwater flow and relatively shallow groundwater. To mitigate this condition, we recommend that the slabs-on-grades be constructed with a below-slab drainage system to collect and discharge groundwater. The below-slab drainage system should include an interior perimeter drain and one longitudinal drain consisting of a 4-inch-diameter, heavy-wall perforated collector pipe installed in a shallow trench placed directly below the capillary break layer. The trench should measure about 18 inches wide by at least 18 inches deep and should be backfilled with the drainage material as described in Sections 3.7.4 and 3.7.5. The drainage material should be wrapped in a needle-punched nonwoven geotextile, such as Mirafi 140N.

The collector pipes should be sloped to drain and discharge into the stormwater collection system to convey water off site. If connected to the footing drain pipe, the invert of the below-slab drain pipe must be at a higher elevation than the footing drain pipe to prevent water from flowing under the structures from the perimeter systems. The pipe should also incorporate cleanouts, if possible. The cleanouts could be extended through the foundation walls to be accessible from the outside or could be placed in flush-mounted access boxes cast into the floor slabs.

The project civil engineer should develop a conceptual below-slab drainage plan for CHE to review.

3.4 BELOW-GRADE WALLS

Permanent below-grade walls may be designed using the earth pressure diagrams presented in Figure 3. Earth pressures are provided for two conditions: Not Restrained Against Rotation and Restrained Against Rotation. Walls that are free to rotate/move laterally at the top a distance of at least one thousandth the height of the wall (H/1000) should be designed using active earth pressures provided for the Not Restrained Against Rotation case. Walls that will be restrained from rotation/lateral movement should be designed using at-rest earth pressures provided for the Restrained Against Rotation case.

Foundation surcharge loads from adjacent buildings should be incorporated into the design of the below-grade walls using the surcharge pressures presented in Figure 4. If vehicles can approach the tops of exterior walls to within half the height of the wall, a traffic surcharge should be added to the wall pressure. Other surcharge loads, such as from construction equipment or construction staging areas, should be considered on a case-by-case basis and can be provided, if necessary, as design progresses.

The lateral earth pressures provided on Figure 3 assume that the ground surface behind the wall is horizontal/flat. If the ground surface behind the wall is sloped steeper than 4H:1V (horizontal to vertical), CHE should be contacted to provide revised earth pressures.

These recommendations assume that all retaining walls will be provided with adequate drainage. The values for soil bearing, frictional resistance, and passive resistance presented above for foundation design are applicable to retaining wall design. Walls located in level ground areas should be founded at a depth of 18 inches below the adjacent grade.

3.4.1 Wall Drainage

In areas where temporary cut slopes and conventional cast-in-place techniques are used to build the below-grade walls, a conventional footing drain should be located on the outside of the building. Positive drainage should be provided behind cast-in-place retaining walls by placing a 2-foot-wide zone of free-draining material against the wall. Free-draining material should consist of Gravel Backfill for Walls per Section 9-03.12(2) of the 2024 WSDOT Standard Specifications. The zone of wall drainage material should extend from the base of the wall to within 2 feet of the ground surface. The wall drainage material should be covered with 2 feet of less-permeable material, such as the on-site silty sand that is properly moisture conditioned and compacted. A geotextile separator should be placed between the wall drainage material and overlying cover soil.

A 4-inch-diameter perforated drain pipe should be installed within the free-draining material at the base of each wall. We recommend using either perforated heavy-wall pipe (SDR-35 PVC) or rigid corrugated polyethylene pipe (ADS N-12, or equal). We recommend against using flexible tubing for the wall drain pipe. The footing drain recommended in Section 3.2.1.4 can be incorporated into the bottom of the drainage zone and used for this purpose.

The pipes should be laid with minimum slopes of one-quarter percent and discharged into the stormwater collection system to convey the water off site. The pipe installations should include a cleanout riser with cover located at the upper end of each pipe run. The cleanouts could be placed in flush-mounted access boxes. Collected downspout water should be routed to appropriate discharge points in separate pipe systems.

3.4.2 Waterproofing

The recommendations in this section are provided to reduce the potential for buildup of hydrostatic pressures behind below-grade walls and hydrostatic uplift forces below building slabs. If no special waterproofing measures are taken, leaks or seepage may occur in localized areas of the below-grade portion of the buildings, even if the recommended wall drainage and below-slab drainage provisions are constructed. If leaks or seepage is undesirable, below-grade waterproofing should be specified. A waterproofing consultant should be contracted to provide recommendations for below-grade waterproofing for this project.

3.5 PAVEMENT RECOMMENDATIONS

3.5.1 Subgrade Preparation

We recommend the subgrade soils in new pavement areas be prepared and evaluated as described in Section 3.7.2. If the subgrade soils are excessively loose or soft, it may be necessary to excavate localized areas and replace them with additional gravel borrow or gravel base material. Pavement subgrade conditions should be observed and proof-rolled during construction and prior to placing the gravel base material in order to evaluate the presence of unsuitable subgrade soils and the need for over excavation and placement of a geotextile separator.

3.5.2 New Hot-Mix Asphalt Pavement

Hot-mix asphalt (HMA) pavement recommendations are provided below for typical automobiles, delivery trucks, and buses based on our engineering judgment. In light-duty pavement areas (e.g., automobile parking), we recommend a pavement section consisting of at least a 2-inch thickness of HMA (Class ½-inch, PG 58) over a 4-inch thickness of densely compacted crushed rock base course. In heavy-duty pavement areas (e.g., materials delivery and vehicle drive aisles) around the building, we recommend a pavement section consisting of at least a 3-inch thickness of HMA (Class ½-inch, PG 58) over a 6-inch thickness of densely compacted crushed rock base course. The subbase and base course should be compacted to at least 95 percent of the MDD (ASTM D 1557). We recommend that a proof-roll of the compacted base course be observed by a representative from our firm prior to paving. Soft or yielding areas observed during proof-rolling may require over-excavation and replacement with compacted structural fill.

The pavement sections recommended above are based on our experience. Thicker asphalt sections may be needed based on the actual subgrade conditions, traffic data, and intended use, especially for pavements supporting heaving trucking or bus traffic.

3.6 INFILTRATION FACILITIES

For preliminary planning purposes, the infiltration characteristics of the fill soil samples (upper 5 feet) from the project site were evaluated using correlations between saturated hydraulic conductivity and ASTM gradation (sieve) testing presented in the 2019 Department of Ecology Stormwater Management Manual for Western Washington (SWMMWW; DOE 2019). We estimate that long-term design infiltration rate of site soils will be less than 1 inch per hour, as summarized in Table 4.

Table 4. Saturated Hydraulic Conductivity Based on Grain Size Analysis

Exploration ID	Sample Depth (ft)	Sample Elevation (ft)	Sample Geologic Unit	Fines Content (%)	D_{10} (mm)	D_{60} (mm)	D_{90} (mm)	K_{sat} initial (in/hr)
CHE-B1-24	5.0	363.8	Glacial till	38	0.013	0.270	7.0	0.7
CHE-P3-24	5.0	361.1	Fill	40	0.015	0.730	21.5	0.4
CHE-B5-24	5.0	369.7	Glacial till	30	0.008	0.198	6.5	0.9

The saturated hydraulic conductivity correlation provided in the 2019 SWMMWW is appropriate for normally consolidated soil deposits and may be less accurate for glacially consolidated soils. Based on the relatively shallow groundwater table and presence of glacially consolidated soils with moderate to high fines content, it is our opinion that infiltration feasibility at the site is low. If infiltration facilities are planned for the project development, CHE recommends that site-specific testing (i.e., pilot infiltration testing) be completed to determine design infiltration rates.

3.7 EARTHWORK

CHE understands that earthwork will include excavating and backfilling for foundations, slabs-on-grade, below-grade walls, at-grade parking, and utility trenches.

3.7.1 Clearing and Site Preparation

Construction of the proposed buildings will require demolition of existing buildings, hardscape, utilities, and other pertinent structures. Existing debris should be excavated and removed from the site. Demolished concrete sidewalks, asphalt pavement, and other concrete structures may be ground up and reused on-site as structural fill; otherwise, it should be hauled off site. Existing utilities and associated trench backfill should be removed from below the proposed building footprints. All excavations that extend below slabs-on-grade or foundation subgrades should be backfilled with structural fill.

Areas to be developed or graded should be cleared of surface and subsurface deleterious matter including any debris, brush, trees, and associated stumps and roots. Graded areas should be stripped of organic soils. Based on our site observations, we estimate that stripping depths will typically be on the order of 2 to 6 inches to remove the sod layer. Deeper stripping and grubbing depths will be required to remove localized deeper deposits of topsoil.

The organic soils can be stockpiled and processed for landscaping purposes or may be spread over disturbed areas following completion of grading. If spread out, the organic strippings should be placed in a layer less than 1-foot thick, should not be placed on slopes greater than 3H:1V, and should be track-walked to a uniformly compacted condition. Materials that cannot be used for landscaping or protection of disturbed areas should be removed from the project site.

3.7.2 Subgrade Preparation

Preparation of footing subgrades and slab subgrade areas should follow the recommendations provided previously in this report. All topsoil, existing fill, and organic soils should be removed from below the building footprint. Prior to placing concrete, new fills, pavement base course materials, or structural fill, all subgrade areas should be evaluated by means of a ½-inch-diameter steel probe rod or by proof-rolling to locate any soft or pumping soils. Proof-rolling can be completed using a piece of heavy tire-mounted equipment such as a loaded dump truck. During wet weather, the exposed subgrade areas should be probed to evaluate the presence and determine the extent of soft soils. If soft or pumping soils are observed, they should be removed and replaced with structural fill.

3.7.3 Subgrade Protection

Site soils contain significant fines content (silt/clay) and will be highly sensitive to increases in moisture content. The exposed subgrade soils can deteriorate rapidly in wet weather and under equipment loads.

The contractor should take necessary measures to prevent site subgrade soils from becoming disturbed or unstable. During periods of wet weather, CHE recommends subgrade soils be protected with a lean concrete mud mat. The mud mat should be placed immediately following subgrade approval. Construction traffic during the wet season should be restricted to specific areas of the site, preferably areas that are surfaced with crushed rock not susceptible to wet weather disturbance.

3.7.4 Structural Fill

All fill that will support foundations, floor slabs, or pavements and hardscape areas, or to be placed against retaining walls or in utility trenches, should meet the criteria for structural fill presented below. The suitability of soil for use as structural fill depends on its gradation and moisture content.

3.7.4.1 MATERIALS

Materials used to construct the building pad, placed under foundations and hardscape, and used to backfill utility trenches are classified as structural fill for the purpose of this report. Quality of structural fill material varies depending upon its use as described below:

- Structural fill placed below new buildings should consist of imported Gravel Borrow, as described in Section 9-03.14(1) of the 2024 WSDOT Standard Specifications, with the additional requirement that the fines content be limited to no more than 5 percent.

- Structural fill placed below landscape areas to backfill utility trenches in the dry summer months may consist of imported Common Borrow per WSDOT Section 9-03.14 (3) provided no particles are larger than 3 inches and the soil is moisture conditioned within 2 percent of the optimum moisture content for the required compaction. If structural fill is placed during wet weather, the structural fill should consist of imported gravel borrow.
- Structural fill placed within utility trenches and below pavement and sidewalk areas should meet the requirements of Gravel Borrow, as described in Section 9-03.14(1) of the 2024 WSDOT Standard Specifications.
- Structural fill placed around perimeter footing drains, under slab drains, and cast-in-place wall drains should consist of Gravel Backfill for Drains per Section 9-03.12(4) of the 2024 WSDOT Standard Specifications.
- Structural fill placed as crushed surfacing base course (CSBC) below pavements should conform to Section 9-03.9(3) of the 2024 WSDOT Standard Specifications.
- Structural fill placed behind below-grade walls within the wall drainage zone should consist of Gravel Backfill for Walls, conforming to Section 9-03.12(2) of the 2024 WSDOT Standard Specifications.
- Structural fill placed as capillary break below slabs should consist of 1-inch-minus clean crushed rock with negligible sand or silt in conformance with Section 9-03.1(4)C, grading No. 67 of the 2024 WSDOT Standard Specifications.

3.7.4.2 REUSE OF ON-SITE NATIVE SOILS

The existing fill, weathered glacial till, and glacial till all contain a high percentage of fines and will be sensitive to changes in moisture content and difficult to handle and compact during wet weather. On-site soils should not be planned for reuse during the wet season (October through May) or during wet weather conditions. Site soils that cannot be reused as structural fill should be disposed of at a suitable off-site location or in landscaped areas where several inches of post-construction settlement is acceptable.

The contractor should plan to cover and maintain all stockpiles with plastic sheeting if planned to be used as structural fill. The reuse of on-site soil is highly dependent on the skill of the contractor and schedule, and we will work with the design team and contractor to maximize the reuse of on-site soils during the wet and dry seasons.

Cobbles and boulders may be encountered during construction excavation. The contractor should be prepared to manage oversized materials (e.g., by screening or manually picking/removing) to ensure reused soils meet gradation requirements.

3.7.4.3 REUSE OF EXISTING ASPHALT AND CONCRETE RUBBLE

Existing asphalt pavement, base course, and Portland cement concrete (PCC) rubble may be reused as structural fill if properly crushed during demolition. Recycled asphalt pavement should not be used as structural fill under building foundations, floor slabs, or in landscape areas. PCC rubble and base course materials may be reused as structural fill throughout the project, except in landscape areas. For use as structural fill, the asphalt and concrete rubble should be crushed or otherwise ground up and should meet the gradation requirements for gravel borrow as described in Section 9-03.14(1) of the 2024 WSDOT Standard Specifications. Recycled concrete may also be used as base course under asphalt parking areas and under hardscape. If used as base course, recycled concrete should be crushed or otherwise ground up and meet the gradation requirements for CSBC as described in Section 9-03.9(3) of the 2024 WSDOT Standard Specifications.

3.7.4.4 FILL PLACEMENT AND COMPACTION CRITERIA

Structural fill should be mechanically compacted to a firm, non-yielding condition. Structural fill should be placed in loose lifts not exceeding 12 inches in thickness if using heavy compactors, or 4 inches if using hand-operated equipment. The actual thickness will be dependent on the structural fill material used and the type and size of compaction equipment. Each lift should be conditioned to the proper moisture content and compacted to the specified density before placing subsequent lifts. Compaction of all structural fill should be in accordance with the ASTM D 1557 (modified proctor) test method. Structural fill should be compacted to the following criteria:

1. Structural fill placed below building foundations and concrete slabs-on-grade should be compacted to at least 95 percent of the MDD.
2. Structural fill placed behind below-grade walls should be compacted to between 90 to 92 percent of the MDD. Care should be taken when compacting fill near the face of below-grade walls to avoid over-compaction (and, hence, overstressing the walls). Hand operated compactors should be used within 5 feet behind the wall. The contractor should keep all heavy construction equipment away from the top of retaining walls a distance equal to half the height of the wall, or at least 5 feet, whichever is greater.
3. Structural fill in new pavement and hardscape areas, including utility trench backfill, should be compacted to at least 90 percent of the MDD, except that the upper 2 feet of fill below final subgrade should be compacted to at least 95 percent of the MDD.
4. CSBC below pavements and hardscape should be compacted to at least 95 percent of the MDD.
5. Non-structural fill, such as fill placed in landscape areas, should be compacted to at least 90 percent of the MDD, unless otherwise specified by the landscape architect.

3.7.4.5 WEATHER CONSIDERATIONS

Surficial site soils are considered moisture sensitive. Imported fill could also be moisture sensitive. When the moisture content of soil is more than a few percent above or below optimum, these soils become muddy and unstable, and operation of equipment on these soils is difficult. Disturbance of near-surface soils should be expected if earthwork is completed during periods of wet weather or in wet conditions.

In western Washington, the wet weather season generally lasts from October through June; however, periods of wet weather can occur throughout the year. During wet weather, CHE recommends:

- The ground surface be sloped so surface water is directed away from the work area to an approved collection/dispersion point.
- Earthwork activities do not take place during periods of heavy precipitation or in wet conditions.
- Measures are taken to prevent site soils and soil stockpiles from becoming wet or unstable.
- The amount of fines in imported fill materials is limited to 5 percent or less, by dry weight, based on the fraction passing the ¾-inch sieve.
- A smooth-drum roller is used to seal the ground surface prior to periods of precipitation to reduce the extent to which the soil becomes wet or unstable.
- Foundation subgrade soils be protected with a lean concrete mud mat.
- Construction traffic is restricted to specific areas of the site, preferably areas surfaced with materials not susceptible to wet weather disturbance.

- Contingencies are included in the project schedule and budget to allow for the above measures.

3.7.5 Utility Trenches

Trench excavation, pipe bedding, and trench backfilling should be completed using the general procedures described in the 2024 WSDOT Standard Specifications or other suitable procedures specified by the project civil engineer. The native glacial deposits and fill soils encountered at the site are generally of low corrosivity based on our experience in the Puget Sound area. If corrosivity of site soils is a concern, then additional testing should be completed.

Utility trench backfill should consist of structural fill and should be placed in lifts of 8 inches or less (loose thickness) such that adequate compaction can be achieved throughout the lift. Sand backfill, containing less than 5 percent fines, may be compacted in loose lifts not exceeding 12 inches. Each lift must be compacted prior to placing the subsequent lift. Prior to compaction, the backfill should be moisture conditioned to within 3 percent of the optimum moisture content, if necessary.

3.7.6 Piezometer Decommissioning

The piezometers installed in borings CHE-P3-24 and CHE-P8-24 can be decommissioned at any time; however, the contractor may choose to leave the piezometer in place until construction has been completed. The piezometer should be decommissioned by a licensed well driller, in accordance with Section 173-160-460 (2)(a) of the WAC. The piezometer should be filled from bottom to top with bentonite, bentonite slurry, neat cement grout, or neat cement.

3.8 EXCAVATION SUPPORT

At this time, we expect that temporary open cut slopes will be suitable for excavations required for the project and understand temporary shoring is not anticipated for the site. The following sections summarize the general excavation recommendations for temporary cut slopes and permanent cut-and-fill slopes.

3.8.1 Temporary Cut Slopes

The stability of open cut slopes is a function of soil type, groundwater seepage, slope inclination, slope height, and nearby surface loads. The use of inadequately designed open cuts could impact the stability of adjacent work areas, existing utilities, and endanger personnel. In our opinion, the contractor will be in the best position to observe subsurface conditions continuously throughout the construction process and to respond to variable soil and groundwater conditions. Therefore, the contractor should have the primary responsibility for deciding whether or not to use open cut slopes rather than some form of temporary excavation support, and for establishing the safe inclination of the cut slope.

Acceptable slope inclinations should be determined during construction. All open cut slopes and temporary excavation support should be constructed or installed and maintained in accordance with the requirements of the appropriate governmental agency.

For planning purposes, temporary unsupported cut slopes more than 4 feet high may be inclined no steeper than 1.5H:1V in the fill and weathered deposits. Cut slopes in the glacially overridden soils may be inclined no steeper than 1H:1V. It may be necessary to flatten the cut slopes if seepage is present on the cut face or if localized sloughing occurs. We recommend that a representative from our firm observe excavation of temporary cuts over 6 feet in height to assess stability and confirm recommended cut slope inclination.

The above guidelines assume that surface loads such as equipment loads and storage loads will be kept a sufficient distance away from the top of the cut so that the stability of the excavation is not affected. We recommend that this distance be not less than half the height of the cut or 5 feet, whichever is greater.

Water entering excavations must be collected and routed away from prepared subgrade areas. We expect that this may be accomplished by installing a system of drainage ditches and sumps along the toe of the cut slopes. Some sloughing and raveling of the cut slopes should be expected. Temporary covering, such as heavy plastic sheeting with appropriate ballast, should be used to protect these slopes during periods of wet weather. Surface water runoff from above cut slopes should be prevented from flowing over the slope face by using berms, drainage ditches, swales, or other appropriate methods.

If temporary cut slopes experience excessive sloughing or raveling during construction, it may become necessary to modify the cut slopes to maintain safe working conditions. Slopes experiencing problems can be flattened, regraded to add intermediate slope benches, or additional dewatering can be provided if the poor slope performance is related to groundwater seepage.

3.8.2 Permanent Cut and Fill Slopes

We recommend that permanent cut or fill slopes be constructed at inclinations of 2H:1V or flatter, and be blended into existing slopes with smooth transitions. Steeper slopes can be constructed if the fill is reinforced with geogrid or other types of reinforcement designed for fill slopes. To achieve uniform compaction, we recommend that fill slopes be overbuilt slightly and subsequently cut back to expose well-compacted fill. Structural fill placed on slopes inclined steeper than 5H:1V should be properly benched or keyed into the slope in accordance with Section 2-03.3(14) of the WSDOT Standard Specifications.

To reduce erosion, newly constructed slopes should be planted or hydroseeded shortly after completion of grading. Until the vegetation is established, some sloughing and raveling of the slopes should be expected. This may necessitate localized repairs and reseeding. Temporary covering, such as heavy plastic sheeting, jute fabric, or erosion control blankets (such as American Excelsior Curlex 1 or North American Green SC150) could be used to protect the slopes during periods of rainfall.

3.9 SEDIMENTATION AND EROSION CONTROL

In our opinion, the erosion potential of the on-site soils is low to moderate, if properly managed. Construction activities including stripping and grading will expose soils to the erosional effects of wind and water. The amount and potential impacts of erosion are partly related to the time of year that construction occurs. Wet-weather construction will increase the amount and extent of erosion and potential sedimentation.

Erosion and sedimentation control measures may be implemented by using a combination of interceptor swales, straw bale barriers, silt fences, and straw mulch for temporary erosion protection of exposed soils. All disturbed areas should be finish graded and seeded as soon as practical to reduce the risk of erosion. Erosion- and sedimentation-control measures should be installed and maintained in accordance with the requirements of the county or other applicable procedures specified by the project civil engineer.

4.0 FINAL DESIGN AND CONSTRUCTION SUPPORT

A geotechnical engineer familiar with the project should conduct a design review of the plans and specifications for compliance with the recommendations presented in this report. CHE also recommends that monitoring, testing, and consultation be provided during construction to confirm that the conditions encountered are consistent with those observed in the field explorations, to provide expedient recommendations should conditions be revealed during construction that differ from those anticipated, and to evaluate whether geotechnical-related construction activities comply with projects plans, specifications, and the recommendations contained in this report.

Geotechnical-related activities include evaluation of foundation and pavement subgrade, placement and compaction of backfill material, observation of temporary slopes, and other earthwork activities. CHE would be pleased to provide these services for you.

5.0 LIMITATIONS

Ciani & Hatch Engineering, PLLC (CHE) has prepared this report for the exclusive use of the Housing Authority of Snohomish County and their authorized agents for explicit application to the Timberglen & Pinewood Redevelopment project located in Lynnwood, Washington. No other party is entitled to rely on the information, conclusions, and recommendations included in this document without the express written consent of CHE. Reuse of the information, conclusions, and recommendations provided herein for extensions of the project or for any other project, without review and authorization by CHE, shall be at the user's sole risk.

This report, along with its conclusions and interpretations, should not be considered a guarantee of the subsurface conditions. The analysis and recommendations provided in this report rely on the data gathered from each specific boring location and other information detailed herein. This report does not account for potential variances that might exist between different boring locations, across the site, or due to the influence of weather conditions or events. The true subsurface conditions can only be confirmed during the actual earthwork construction phase. In the event of any observed variations, we recommend you promptly notify CHE so that we can confirm that the soil conditions encountered are consistent with those observed in our explorations and provide supplementary recommendations as needed.

An evaluation of environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials, or conditions was not completed and is not part of CHE's scope. If the potential for such contamination or pollution is a concern, an environmental consultant should be retained to complete such studies.

CHE recommends the project design and specifications are reviewed by CHE to verify recommendations have been interpreted and implemented as intended. Should site or project conditions change, CHE should be retained to review the recommendations and conclusions in this report to determine if the recommendations are still applicable or to provide supplemental information, if needed.

Within the limitations of scope, schedule, and budget, CHE's services have been provided in a manner in accordance with the level of skill and care ordinarily exercised in the field of geotechnical engineering currently practicing in the same area and under similar conditions. No other warranty, either expressed or implied, should be understood.

CHE cannot guarantee the accuracy and content of electronic files (email, text, table, figures, etc.). The master file of this document is stored by CHE and will serve as the official document of record.

6.0 REFERENCES

American Society of Civil Engineers (ASCE). 2017. Minimum Design Loads and Associated Criteria for Buildings and Other Structures: ASCE/SEI 7-16, Supplement 1

City of Lynnwood. 2024a. Streams Map.

———. 2024b. Native Growth Protection Areas Map.

———. 2024c. Wetlands Map.

———. 2024d. Geologically Hazardous Areas Map.

———. 2024e. Frequently Flooded Areas Map.

———. 2024f. Critical Aquifer Recharge Areas Map.

HWA Geosciences Inc. 2024. Materials Laboratory Report, HASCO – Timberglen & Pinewood Development.

International Code Council, 2018. International Building Code.

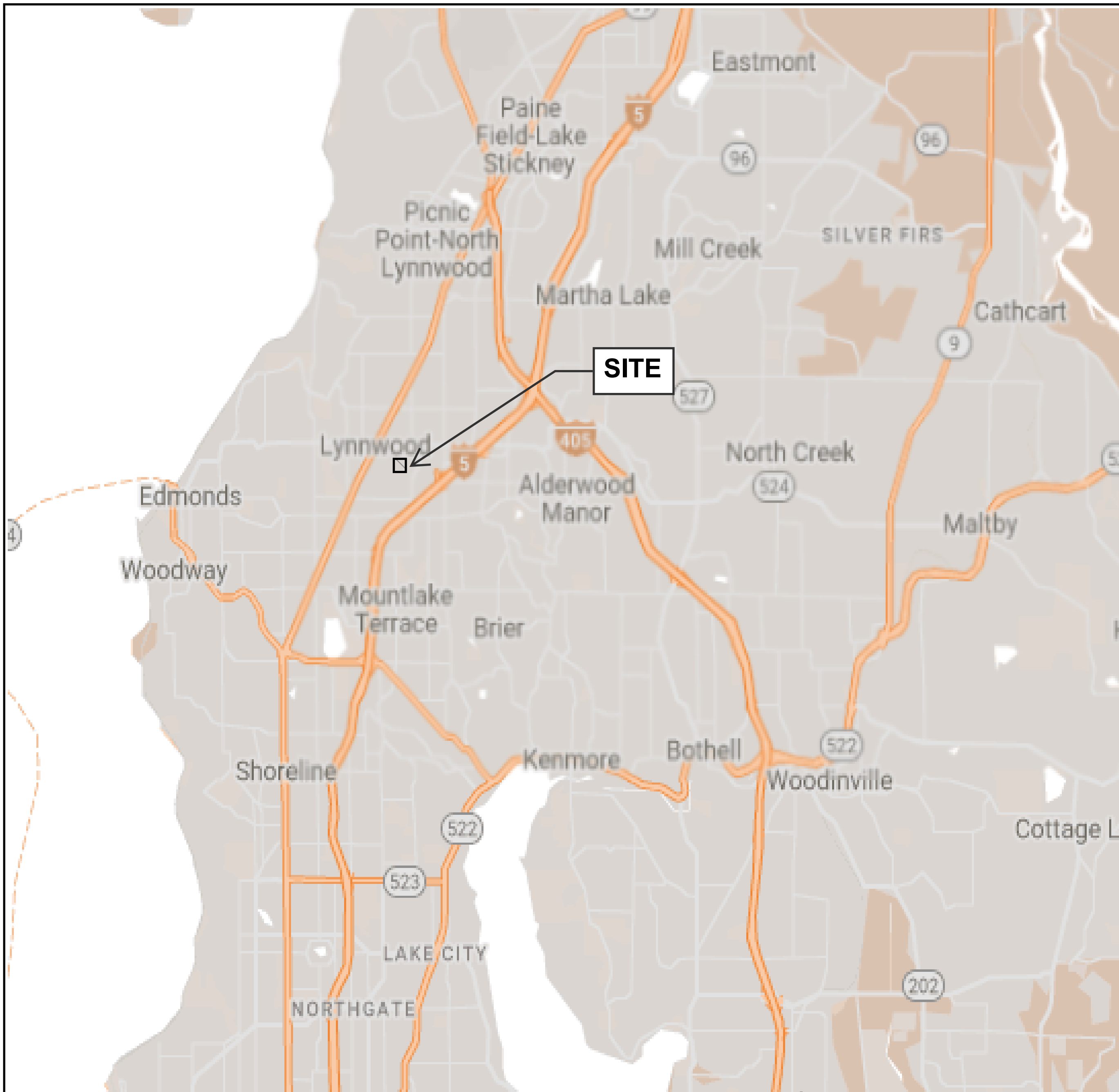
Minard, J.P. 1983. Geologic Map of the Edmonds East and part of the Edmonds West quadrangles, Washington.

U.S. Geological Survey (USGS). 2022. Interactive U.S. Fault Map. Interactive U.S. Fault Map | U.S. Geological Survey (usgs.gov).

Washington State Department of Ecology (DOE). 2019. Stormwater Management Manual for Western Washington.

Washington State Department of Transportation (WSDOT). 2024. Standard Specifications for Road, Bridge, and Municipal Construction. M 41-10.

Figures



NOTES:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes only. It is intended to assist in showing features discussed in an attached document. Ciani & Hatch Egineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

Map Source: Map data 2023 Google

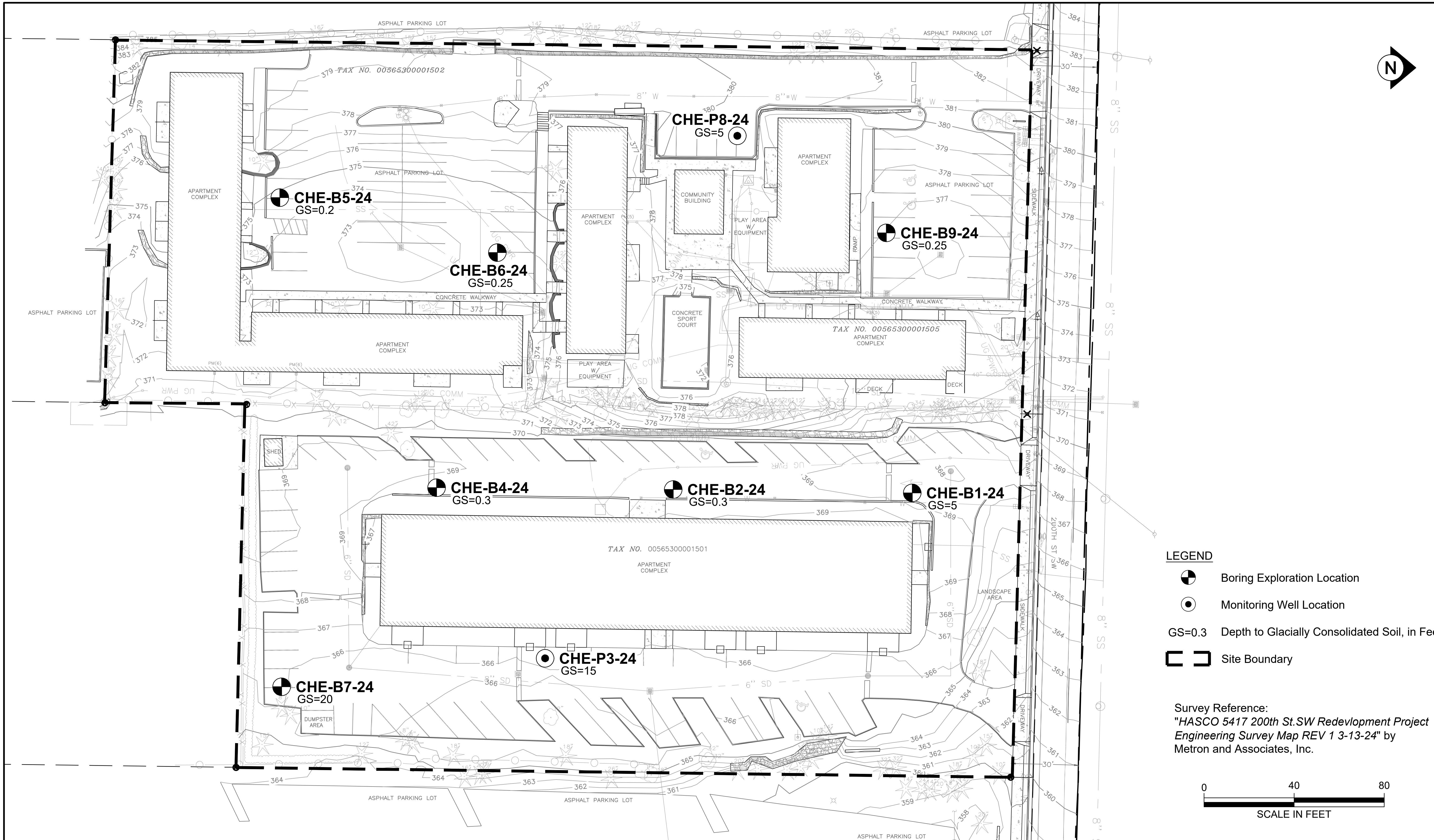
N

CIANI & HATCH
ENGINEERING

Project Vicinity Map

Timberglen & Pinewood Redevelopment Project
Lynnwood, Washington

FIGURE 1

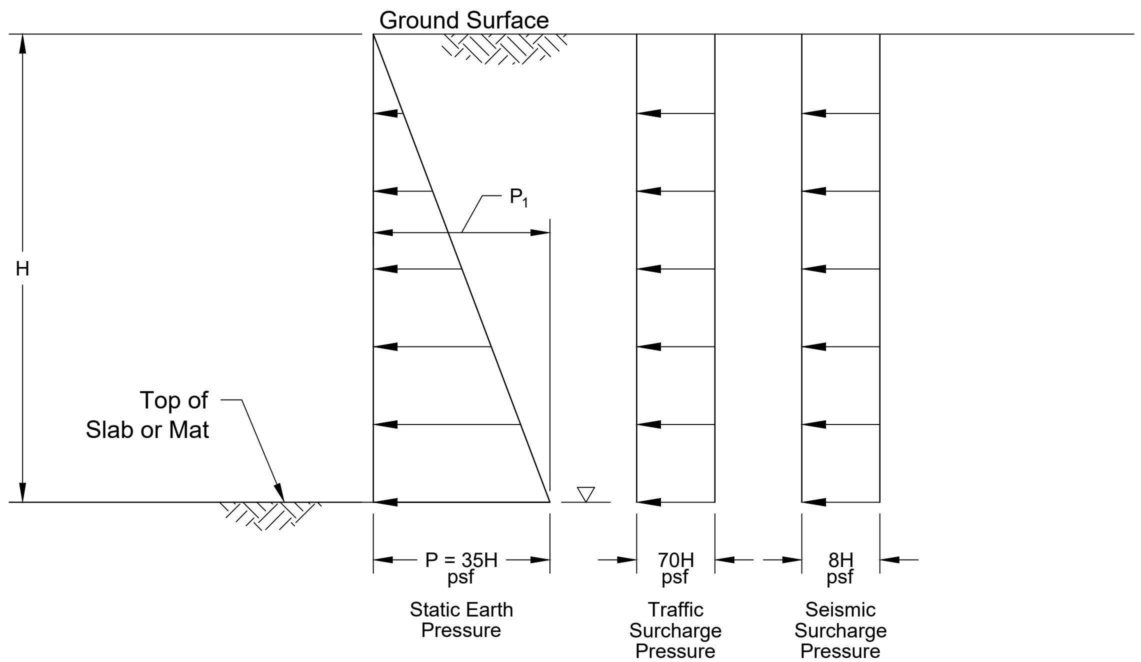


CIANI&HATCH ENGINEERING	PROJECT MANAGER: M. Hatch	PROJECT NUMBER: 2002-001-00	**Geotechnical Exploration Site Plan**
	DRAWN BY: C. Taylor	SCALE: As Shown	Housing Authority of Snohomish County
	CHECKED BY: M. Hatch	FILE NAME: 2002-001-00 Hasco Redev.dwg	Timberglen & Pinewood Redevelopment
	APPROVED BY: W. Ciani	DATE: 3/14/2024	Lynnwood, Washington

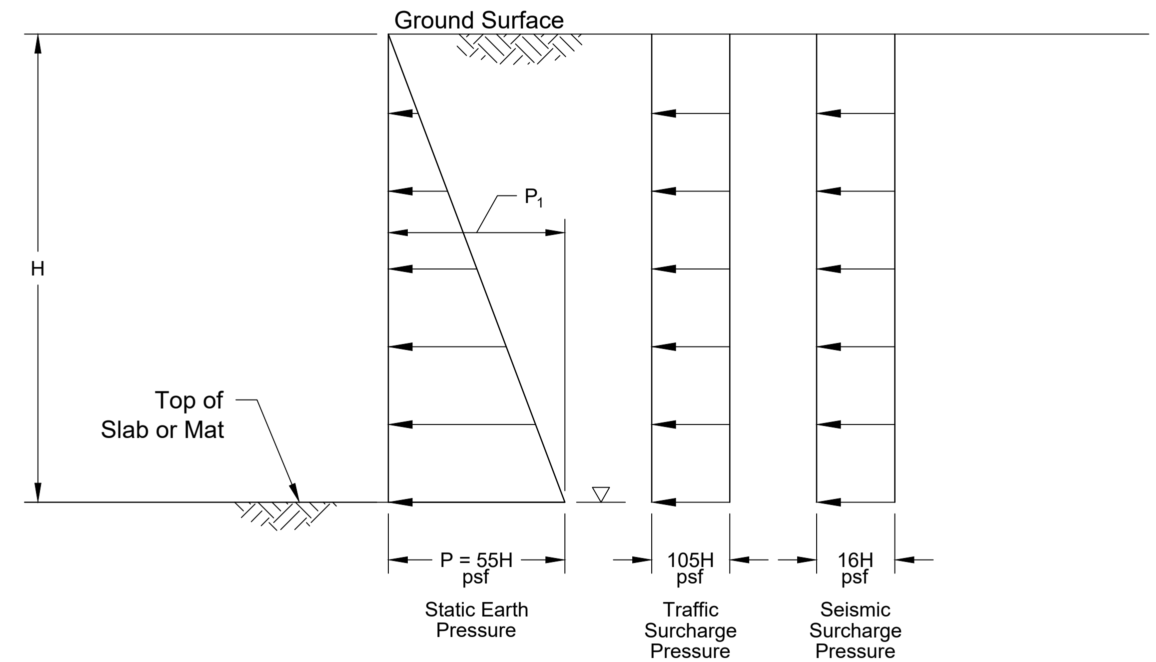
FIGURE

2

Below Grade Walls
Not Restrained Against Rotation



Below Grade Walls
Restrained Against Rotation



Not to Scale

NOTES

1. Additional surcharge from footings of adjacent buildings should be included in accordance with recommendations provided on Figure 4.
2. This pressure diagram is appropriate for permanent below grade walls and retaining structures which are not adjacent to temporary shoring walls and are not restrained against movement.
3. Walls are assumed to be restrained if top movement during backfilling is less than H/1000, where H is the wall height.
4. These lateral soil pressures assume that the ground surface behind the wall is horizontal.
5. The static earth pressure does not include a factor of safety and represents the actual anticipated static earth pressure.
6. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

LEGEND

H = Height of Basement Wall, Feet

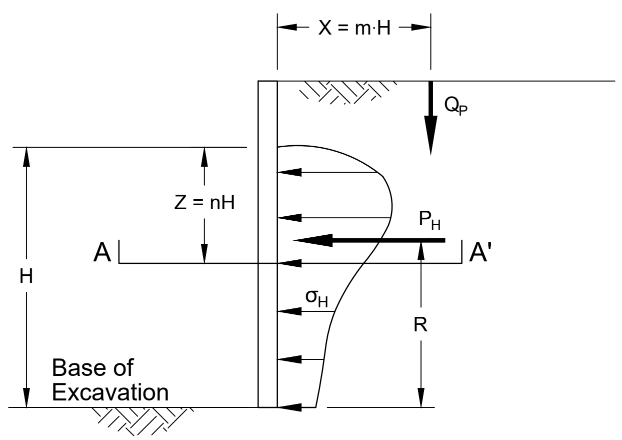
P_1 = Maximum Static Earth Pressure Pounds per Square Foot

▽ Design Groundwater Elevation for Drained Walls

CIANI & HATCH ENGINEERING

PROJECT MANAGER	M. Hatch	PROJECT NUMBER	2002-001-00	**Permanent Below Grade Walls**	FIGURE
DRAWN BY	C. Taylor	SCALE	As Shown	Housing Authority of Snohomish County Timberglen & Pinewood Redevelopment Lynnwood, Washington	3
CHECKED BY	M. Hatch	FILE NAME	2002-001-00 Hasco Details.dwg		
APPROVED BY	W. Ciani	DATE	3/22/2024		

Lateral Earth Pressure From Point Load, Q_P
(Spread Footing)



FOR m ≤ 0.4

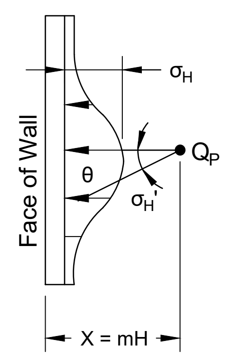
$$\sigma_H = \frac{0.28 Q_P n^2}{H^2(0.16+n^2)^3}$$

FOR m > 0.4

$$\sigma_H = \frac{1.77 Q_P m^2 n^2}{H^2(m^2+n^2)^3}$$

$$\sigma_H' = \sigma_H \cos^2(1.1\theta)$$

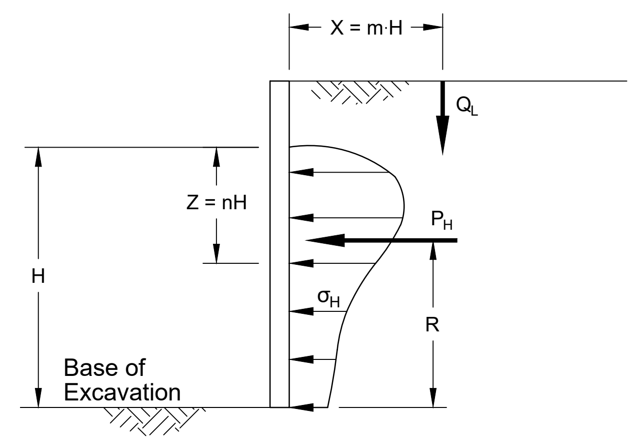
M	$P_H\left(\frac{H}{Q_P}\right)$	R
0.2	0.78	0.59H
0.4	0.78	0.59H
0.6	0.45	0.48H



SECTION A-A'

Pressures from Point Load Q_P

Lateral Earth Pressure From Line Load, Q_L
(Continuous Wall Footing)



FOR m ≤ 0.4

$$\sigma_H = \frac{0.2 n Q_L}{H(0.16+n^2)^2}$$

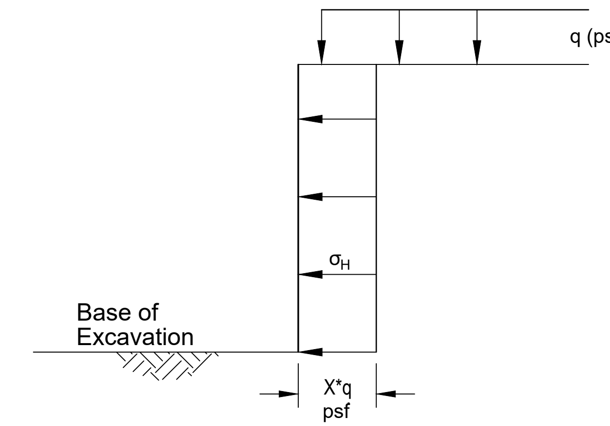
FOR m > 0.4

$$\sigma_H = \frac{1.28 m^2 n Q_L}{H(m^2+n^2)^2}$$

$$\text{Resultant } P_H = \frac{0.64 Q_L}{(m^2+l)}$$

M	R
0.1	0.60H
0.3	0.60H
0.5	0.56H
0.7	0.48H

Uniform Surcharges, q
(Floor Loads, Large Foundation Elements, or Traffic Loads)



σ_H = Lateral Surcharge Pressure From Uniform Surcharge

X = Lateral Pressure Coefficient as Shown in Table Below

	X
Wall Not Restrained Against Movement	0.24
Wall Restrained Against Movement	0.41

Not to Scale

NOTES

1. Procedures for estimating surcharge pressures shown above are based on Manual 7.02 Naval Facilities Engineering Command, September 1986 (NAVFAC DM 7.02).
2. Lateral earth pressures from surcharge should be added to earth pressures presented on Figures 3.
3. See report text for where surcharge pressures are appropriate.
4. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.

LEGEND

Q_P = Point load in pounds
Q_L = Line load in pounds/foot
H = Excavation height below footing, feet
σ_H = Lateral earth pressure from surcharge, psf
q = Surcharge pressure in psf
θ = Radians
σ_H' = Distribution of σ_H in plan view
P_H = Resultant lateral force acting on wall, pounds
R = Distance from base of excavation to resultant lateral force, feet

CIANI & HATCH ENGINEERING

PROJECT MANAGER	M. Hatch	PROJECT NUMBER	2002-001-00	
DRAWN BY	C. Taylor	SCALE	As Shown	
CHECKED BY	M. Hatch	FILE NAME	2002-001-00 Hasco Details.dwg	
APPROVED BY	W. Ciani	DATE	3/22/2024	

Recommended Surcharge Pressure

Housing Authority of Snohomish County
Timberglen & Pinewood Redevelopment
Lynnwood, Washington

FIGURE 4

Appendix A.

CHE FIELD EXPLORATIONS

APPENDIX A

FIELD EXPLORATIONS

Subsurface conditions at the site were explored by drilling nine borings (CHE-B1-24 through CHE-B9-24), two of which were completed as groundwater monitoring wells (CHE-P3-24 and CHE-P8-24). The borings were completed to depths between 21.5 and 26.5 feet below existing site grades. The drilling was performed by Bortec 1, Inc. subcontracted to Ciani & Hatch Engineering, PLLC (CHE) on February 22 and 23, 2024.

CHE personnel coordinated and monitored the explorations, obtained representative soil samples, maintained detailed logs of the subsurface soil and groundwater conditions observed, and described the soil encountered by visual and textural examination. Each representative soil type was described using the Unified Soil Classification System (USCS), in general accordance with ASTM International Standard D 2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)*. Summary logs of the explorations are presented on Figures A-2 through A-18. The stratigraphic contacts indicated on the logs represent the approximate boundaries between soil types; actual transitions may vary and be more gradual. The densities noted on the boring logs are based on the blow count data obtained in the borings. The soil and groundwater conditions illustrated are specific to the dates and locations mentioned in the report and may not necessarily reflect conditions in other areas or at different times. A key to the exploration logs is presented on Figure A-1.

Soil samples were obtained from the borings using a 1.5-inch-inside-diameter split-spoon sampler. A 140-pound motorized rope and cathead hammer, falling approximately 30 inches, was used to drive the sampler 18 inches (or a portion thereof) into the undisturbed soil. The number of blows required for each 6 inches of penetration was recorded. The blow count ("N-value") of the soil was calculated as the number of blows required for the final 12 inches of penetration. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. Where very dense soil conditions precluded driving the full 18 inches, the penetration resistance for the partial penetration was entered on the logs. The blow counts are shown on the boring logs at the respective sample depths.

Where first indicated by an increase in soil moisture or from wet sampling equipment, drilling was paused for 20 to 30 minutes to allow groundwater to collect and equilibrate within the borehole. After this pause, a depth-to-groundwater measurement was made through the hollow-stem auger with an electronic water-level indicator.

Upon completion of fieldwork, all borings (except CHE-P3-24 and CHE-P8-24) were decommissioned. In borings CHE-P3-24 and CHE-P8-24, a 2-inch-diameter polyvinyl chloride (PVC) monitoring well was installed. The installed well depth was determined based on our observations of subsurface soil and groundwater conditions encountered during drilling. The lower portion of the casing was slotted to allow entry of water into the well. Medium (12/20 Industrial) sand was placed in the borehole annulus surrounding the slotted portion of the casing. A bentonite seal was placed above the slotted portion of the casing. The monitoring well was protected by installing a flush-mount steel monument set in concrete. Completion details of monitoring well construction are included on the logs for those explorations. Borehole decommissioning and well construction were completed in accordance with the requirements in Chapter 173-160 of the Washington Administrative Code.

Soil samples were transported to the HWA GeoSciences, Inc. geotechnical laboratory for further examination and testing. Test results are presented in Appendix B.

Soil Classification Chart

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	LETTER DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	Well-graded gravels, gravel-sand mixtures, little or no fines
				GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
	MORE THAN 60% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	Silty gravels, gravel-sand-silt mixtures
				GC	Clayey gravels, gravel-sand-clay mixtures
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	Well-graded sands, gravelly sands, little or no fines
				SP	Poorly graded sands, gravelly sands, little or no fines
	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	Silty sands, sand-silt mixtures
				SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
				CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts and organic silty clays of low plasticity
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils
				CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat, humus, swamp soils with high organic contents

OTHER MATERIALS	LETTER SYMBOL	GRAPH SYMBOL
Asphalt Concrete	AC	
Cement Concrete	CC	
Base Course	BC	
Wood	W	

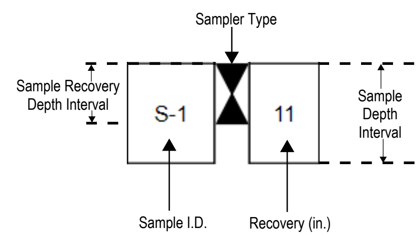
Field and Lab Testing

- %F Percent passing the No. 200 Sieve
- SA Grain size analysis
- CA Combined grain size analysis and hydrometer
- AL Atterberg Limits
- LL Liquid Limit
- PL Plastic Limit
- PI Plasticity Index
- MC Moisture Content
- WOH Weight of Hammer
- WOR Weight of Rod

Sampler Types

- SPT Standard Penetration Test
- CALI California Sampler
- GRAB Grab sample

Sampler Information



Geology Boundaries

Soil strata contact between soil geologic units

Soil strata contact within the same geologic unit

Groundwater

Groundwater level observed at time of drilling

Groundwater level observed post drilling

Notes:

1. Each representative soil type was described using the Unified Soil Classification System (USCS), in general accordance with ASTM International standard Standard D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedures). Dual letter symbols (e.g., SP-SM for sand or gravel) indicate soil with an estimated 5-15% fines. Multiple letter symbols (e.g., ML/CL) indicate borderline or multiple soil classifications.
2. Soil density or consistency descriptions are based on judgment using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.
3. These logs are for information purposes only. They are intended to assist in showing features discussed in an attached document. Ciani & Hatch Engineering PLLC (CHE) cannot guarantee the accuracy and content of electronic files. The master file is stored by CHE and will serve as the official record of this communication.
4. Descriptions on the logs are constrained to the specific limited exploration locations at the time the explorations were made. These logs are not warranted to be representative of subsurface conditions at other locations or times. These logs are to be interpreted with the discussion in the report text for a proper understanding of subsurface conditions.

CIANI & HATCH ENGINEERING

Key to Exploration Logs

Timberglen & Pinewood Redevelopment
Lynnwood, Washington

FIGURE A-1

CIANI & HATCH ENGINEERING

Borehole No: CHE-B1-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.817291	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.310595	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 368.79(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 20.3 ft	Date	: 02/22/2024	Loc Comment	: 5710 200th St SW

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery (in.)	Blows/6" (bpf)	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
									ASP	2 inches of Asphalt			
368	1								BC	2 inches of Base Course			
367	2								SM	Brown, silty SAND with gravel, asphalt fragments (medium dense, moist) (Fill)			
366 / 365	3 / 4		S-1		11	3 9 17 (N=26)	%F				10	19	
364 / 363	5 / 6		S-2		7	33 27 32 (N=59)	CA		SM	Brown, silty fine to coarse SAND with gravel and broken cobbles, oxidation staining (moist to wet, very dense) (Glacial till)	12	38	
362	7												Drill Chatter
361 / 360	8 / 9		S-3		18	12 16 18 (N=34)	%F		SM	Brown, silty SAND with gravel (moist to wet, dense)	12	31	
359 / 358 / 357	10 / 11 / 12		S-4		18	7 17 25 (N=42)	MC				15		
356	13												
355	14	▽											Ground water at 14.1' at time of drilling
354 / 353	15 / 16		S-5a S-5b		11	31 50/5in (N=50/5")			ML	Brown, SILT with sand, (moist, hard)			
352 / 351 / 350	17 / 18 / 19												
349	20												Drill Chatter
348	21		S-6		4	50/4in (N=50/4")			ML	Gray, blocky SILT with sand, broken cobbles (moist, hard)			
347 / 346	22									**CHE-B1-24 Terminated at 20.3 ft (Groundwater encountered at 14.1 ft bgs)**			

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B2-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816949	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.310591	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 369.41(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 20.4 ft	Date	: 02/22/2024	Loc Comment	: 5710 200th St SW

Depth (ft)	Sample Number	Recovery (in.)	Blows/6" (bpf)	Lab Testing	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
0					ASP	2 inches of Asphalt			
0.2					BC	2 inches of Base Course			
0.3					SM	Brown, silty SAND with gravel, broken cobbles (moist, very dense) (Glacial till)			
2.5	S-1	5	27 39 44 (N=83)	MC			9		
5	S-2	11	24 50/5in (N=50/5")	%F			8	35	
6									Drill Chatter
7.5	S-3	9	29 50/4in (N=50/4")	%F			7	31	
10	S-4	16	46 43 50/5in (N=93/11")						
15	S-5	11	29 50/5in (N=50/5")		ML	Brown, sandy SILT with gravel (moist to wet, hard)			
17.5									Drill Chatter
20	S-6	5	50/5in (N=50/5")		ML	Gray, sandy SILT with gravel (moist to wet, hard)			

CHE-B2-24 Terminated at 20.4 ft (No groundwater encountered at time of drilling)

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Borehole No: CHE-P3-24

CIANI & HATCH ENGINEERING

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816848	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.310284	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 366.09(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 21.5 ft	Date	: 02/22/2024	Loc Comment	: 5710 200th St SW

General comments: Department of Ecology ID Well Tag Number: BPG 810

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery	Blows/6"	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)
366	0								ASP	2" Asphalt		
									BC	3" Base Course		
365–361	1–5	▼ (4.7)	S-1 (3–4.5)		8	2, 1, 1 (N=2)	%F		SM	Red brown, silty SAND with gravel and organic matter, oxidation staining (moist, very loose) (Fill)	22.1	20.5
361–358	5–7.5	▽ (7.4)	S-2 (5.5–7)		13	3, 5, 5 (N=10)	CA		SM	Brown, silty SAND with gravel (wet, loose)	16.0	39.9
358	7.5		S-3 (8–9.5)		11	9, 9, 13 (N=22)			SM	Brown, silty SAND with gravel (wet, dense) (Weathered glacial till)		
			S-4 (10.5–12)		6	11, 19, 12 (N=31)						
351	15		S-5		9	37, 50/5in (N=50/5")			ML	Brown, sandy SILT with gravel, broken cobbles (wet, hard) (Glacial till)		
346	20		S-6a		18	16, 26, 36 (N=62)			SM	Brown, silty SAND (wet, very dense)		
345	21		S-6b						SP-SM	Gray, SAND with gravel and silt (wet, very dense)		

Well Diagram:
- Portland Concrete Cement
- 2 in. PVC Solid
- Bentonite
- 12/20 Industrial Sand
- 2 in. Schedule 40 PVC screen, 0.020-inch slot

CHE-P3-24 Terminated at 21.5ft (Groundwater encountered at 7.4 ft bgs ATD)

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B4-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816710	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.310592	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 369.27(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 21.5 ft	Date	: 02/22/2024	Loc Comment	: 5710 200th St SW

Depth (ft)	Water	Sample Number	Recovery (in.)	Blows/6" (bpf)	Lab Testing	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
0						ASP	2 inches of Asphalt			
0.2						BC	2 inches of Base Course			
0.3						SM	Brown, silty SAND with gravel, broken cobbles (moist, very dense) (Glacial till)			
2.5		S-1	9	3 28 28 (N=66)	%F			13	23	Drill Chatter
5		S-2	17	21 24 28 (N=52)	MC			11		
7						SM	Brown, silty SAND with gravel (wet, dense)			Groundwater observed at 7.5 ft bgs ATD
7.5	▽	S-3	14	22 23 17 (N=40)	%F			12	30	
10		S-4	14	12 16 18 (N=34)		ML	Brown, sandy SILT (wet, hard)			
15		S-5	4	50/5in (N=50/5")						
20		S-6	14	27 24 33 (N=57)		ML	Gray, sandy SILT with gravel (wet, hard)			

Elevation scale: 369 to 347 ft; Depth scale: 1 to 22 ft.

CHE-B4-24 Terminated at 21.5ft (Groundwater observed at 7.5 ft bgs.)

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B5-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816513	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311103	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 374.74(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 25.9 ft	Date	: 02/22/2024	Loc Comment	: 5720 200th St SW

Depth (ft)	Elevation (ft)	Water	Sample Number	Recovery (in.)	Blows/6" (bpf)	Lab Testing	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
0	374.74						ASP	2 inches of Asphalt			
0.2							SM	Gray, silty SAND with gravel, oxidation staining (moist, dense) (Glacial till)			
2.5–4			S-1	16	23 23 25 (N=48)	MC			9		
5–6.5			S-2	18	21 23 24 (N=47)	CA			9	30	Cobble fragments observed in sampler
7.2		▽									
7.5–9			S-3	18	15 15 28 (N=43)	%F	SM	Becomes wet	12	32	
10–11.5			S-4	11	21 50/5in (N=50/5")	MC	SM	Brown, silty SAND with gravel and inclusions of gray silt, (wet, very dense)	11		
15–16.5			S-5	6	27 50/2in (N=50/2")		SM	Brown, silty SAND with gravel (wet, very dense)			

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B5-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816513	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311103	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 374.74(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 25.9 ft	Date	: 02/22/2024	Loc Comment	: 5720 200th St SW

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery (in.)	Blows/6" (bpf)	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
354	21		S-6		1	50/1in (N=50/1")			SM	Brown, silty SAND with gravel (wet, very dense)			
353	22												
352	23												
351	24												
350	25												
349	26		S-7		9	29 50/5in (N=50/5")							
348	27									**CHE-B5-24 Terminated at 25.9 ft (Groundwater encountered at 7.2 ft bgs)**			
347	28												
346	29												
345	30												
344	31												
343	32												
342	33												
341	34												
340	35												
339	36												
338	37												
337	38												
336	39												
335													

CIANI&HATCH ENGINEERING

Borehole No: CHE-B6-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816779	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311013	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 373.46(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 25.2 ft	Date	: 02/23/2024	Loc Comment	: 5720 200th St SW

Depth (ft)	Water	Sample Number	Recovery (in.)	Blows/6" (bpf)	Lab Testing	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
0						ASP	3 inches of Asphalt			
0.25						SM	Brown-gray, silty SAND with gravel, broken cobbles (moist, dense to very dense) (Glacial till)			
2.5		S-1	18	24 26 24 (N=50)	%F			8	21	
5		S-2	7	16 21 36 (N=57)	MC	SM	Becomes very dense	10		
6.9	ATD GWT									ATD GWT at 6.9 ft bgs
7.5		S-3	11	18 23 13 (N=36)	MC	GM	Brown, silty GRAVEL with sand (wet, dense)	9		Drill Chatter
10		S-4	18	12 16 34 (N=50)	%F	SP-SM	Brown, poorly graded SAND with gravel and silt (wet, dense)	10	11	
15		S-5a	18	17 25 48 (N=73)		SM	Brown, silty SAND (moist, very dense)			
16		S-5b				ML	Brown-gray, sandy SILT with gravel (wet, hard)			

Elevation scale (ft): 373, 372, 371, 370, 369, 368, 367, 366, 365, 364, 363, 362, 361, 360, 359, 358, 357, 356, 355, 354

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B6-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816779	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311013	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 373.46(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 25.2 ft	Date	: 02/23/2024	Loc Comment	: 5720 200th St SW

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery (in.)	Blows/6" (bpf)	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
353–348	20.5–25		S-6		14	4 37 50/2in (N=87/8")			SM	Brown, silty SAND with gravel (moist to wet, very dense)			
348–347	25–25.2		S-7		2	50/2in (N=50/2")			SM	Brown, silty SAND (wet, very dense)			

Elevation scale: 353, 352, 351, 350, 349, 348, 347, 346, 345, 344, 343, 342, 341, 340, 339, 338, 337, 336, 335, 334

Depth scale: 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35, 36, 37, 38, 39

CHE-B6-24 Terminated at 25.2 ft (Groundwater encountered at 6.9 ft bgs at time of drilling)

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B7-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.816529	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.310223	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 366.03(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 21.5 ft	Date	: 02/22/2024	Loc Comment	: 5710 200th St SW

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery (in.)	Blows/6" (bpf)	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
366	0								ASP	2 inches of Asphalt			
365	1								BC	3 inches of Base Course			
364	2								SM	Brown, silty SAND with organics, inclusions of silt (moist, loose) (Fill)			
363	3		S-1		18	2 2 3 (N=5)	%F				14	22	
361	5		S-2		11	1 0 1 (N=1)	AL		SM	Brown, silty SAND with gravel (wet, very loose)	18		AL; (Non-Plastic)
358	8		S-3a		18	1 1 6 (N=7)	MC		ML	Gray, SILT with gravel (wet, medium stiff)	16		
357	9	▽	S-3b				OC, MC		OL	Dark brown, organic SILT with sand (wet, medium stiff)	146		25.6% Organic Matter
356	10		S-4		4	10 13 14 (N=27)	%F		SM	Brown, silty SAND with gravel (wet, very stiff), (Weathered glacial till)	12	28	
351	15		S-5		18	6 8 9 (N=17)							
346	20		S-6		18	35 36 48 (N=84)			ML	Brown, sandy SILT with gravel (wet, hard) (Glacial till)			
344	22									**CHE-B7-24 Terminated at 21.5ft (Groundwater encountered at 9.2 ft bgs ATD)**			

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Borehole No: CHE-P8-24

CIANI & HATCH ENGINEERING

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.817071	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311225	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 380.29(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 25.3 ft	Date	: 02/23/2024	Loc Comment	: 5720 200th St SW

General comments: Department of Ecology ID Well Tag Number: BPG 810

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery	Blows/6"	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Well Diagram
380	0								ASP	1.5 inches of Asphalt			Portland Concrete Cement
									BC	2 inches of Base Course			
379	1								SM	Brown, silty SAND with organics (moist, loose) (Fill)			
377	2.5–4		S-1		4	4, 4, 4 (N=8)							2 in. PVC Solid; Bentonite
375	5–6.5	▼ (approx. 6 ft)	S-2		13	8, 13, 31 (N=44)	%F		SM	Brown, silty SAND with gravel, broken cobbles (moist, dense) (Glacial till) Drill Chatter	13	20	
372	7.5–9		S-3		6	11, 19, 30 (N=49)	MC				14		12/20 Industrial Sand
370	10–11.5	∇ (approx. 11 ft)	S-4		14	9, 17, 17 (N=34)	MC				10		2 in. Schedule 40 PVC screen
365	15–16.5		S-5a / S-5b		18	11, 20, 29 (N=49)							
364	16								ML	Gray, sandy SILT (moist, hard)			

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Borehole No: CHE-P8-24

CIANI & HATCH ENGINEERING

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.817071	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311225	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 380.29(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 25.3 ft	Date	: 02/23/2024	Loc Comment	: 5720 200th St SW

General comments: Department of Ecology ID Well Tag Number: BPG 810

Elevation (ft)	Depth (ft)	Water	Sample Number	Samples	Recovery	Blows/6"	Lab Testing	Graphic Log	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Well Diagram
360–355	21–25		S-6		5	50/5in (N=50/5")			SM	Brown, silty SAND with gravel (moist, very dense) Drill Chatter			12/20 Industrial Sand
355–354	25–26		S-7			50/3in (N=50/3")				No Recovery; Refusal			

CHE-P8-24 Terminated at 25.3 ft (Groundwater encountered at 11 ft bgs ATD.)

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

CIANI & HATCH ENGINEERING

Borehole No: CHE-B9-24

UTM	: 10 n	Driller Rig	: EC95 (HE = 75%)	Job Number	: 2002-001-00
Latitude	: 47.817253	Driller Supplier	: Boretec1, Inc.	Client	: Housing Authority of Snohomish County
Longitude	: -122.311062	Logged By	: CEM	Project	: Timberglen & Pinewood Redevelopment
Elevation	: 376.92(ft)	Reviewed By	: MSH	Location	: Lynnwood, WA, USA
Total Depth	: 20.2 ft	Date	: 02/23/2024	Loc Comment	: 5720 200th St SW

Depth (ft)	Water	Sample Number	Recovery (in.)	Blows/6" (bpf)	Lab Testing	USCS	Soil Description	Moisture Content (%)	Fines Content (%)	Remarks
0						ASP	1 inch of Asphalt			
0.1						BC	2 inches of Base Course			
0.25						SM	Brown, silty SAND with gravel, broken cobbles, oxidation staining (moist, dense) (Glacial till)			
2.5		S-1	16	20 21 23 (N=44)	%F			10	25	
5		S-2	18	19 22 24 (N=46)	%F			10	28	
7.5		S-3	6	26 31 31 (N=62)	MC	SM	Becomes very dense	10		
10	▽	S-4	14	24 33 31 (N=64)	%F	SM	Brown, silty SAND with gravel (moist, very dense)	9	20	
15		S-5	5	50/5in (N=50/5")		SM	Brown, silty SAND (wet, very dense)			
20		S-6		50/2in (N=50/2")			No Recovery; Refusal			

Elevation scale (ft): 376 to 354; Depth scale (ft): 1 to 22.

CHE-B9-24 Terminated at 20.2 ft (Groundwater encountered at 10.1 ft bgs ATD)

This report must be read in conjunction with accompanying notes and abbreviations. It has been prepared for geotechnical purposes only, without attempt to assess possible contamination. Any references to potential contamination are for information only and do not necessarily indicate the presence or absence of soil or groundwater contamination.

Appendix B.

LABORATORY TESTING

APPENDIX B

LABORATORY TESTING

Soil samples obtained from the explorations were transported to the HWA GeoSciences, Inc. (HWA) geotechnical laboratory. HWA examined samples for natural moisture content, grain size distributions, Atterberg Limits (plasticity indices), and percent passing U.S. No. 200 sieve. The results of the laboratory testing program are provided in the appended Materials Laboratory Report dated March 5, 2024.

A summary of the grain size analyses and percent passing the No. 200 sieve analyses for use in evaluating the hydraulic conductivity of site soils is provided in Table B-1.

Table B-1. Grain Size and Percent Passing Analysis Data

Boring	Sample Depth (ft)	D_{90}	D_{60}	D_{10}	%Fines
CHE-B1-24	**2.5**	-	-	-	18.6
CHE-B1-24	**5.0**	7	0.27	0.013	37.7
CHE-B1-24	**7.5**	-	-	-	30.8
CHE-B2-24	**5.0**	-	-	-	34.6
CHE-B2-24	**7.5**	-	-	-	31.0
CHE-P3-24	**2.5**	-	-	-	20.5
CHE-P3-24	**5.0**	21.5	0.73	0.015	39.9
CHE-B4-24	**7.5**	-	-	-	30.0
CHE-B5-24	**5.0**	6.5	0.198	0.008	30.4
CHE-B5-24	**7.5**	-	-	-	31.6
CHE-B6-24	**2.5**	-	-	-	20.7
CHE-B6-24	**10.0**	-	-	-	10.8
CHE-B7-24	**2.5**	-	-	-	21.5
CHE-B7-24	**10.0**	-	-	-	28.0
CHE-P8-24	**5.0**	-	-	-	19.7
CHE-B9-24	**5.0**	-	-	-	28.4
CHE-B9-24	**10.0**	-	-	-	19.8

March 5, 2024
HWA Project No. 2023-177-23 Task 200

Ciani & Hatch Engineering
18875 67th Drive NE, Unit 1
Kenmore, WA 98028

Attention: Ms. Mikayla Hatch, P.E.

Subject: **Materials Laboratory Report**
HASCO – Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

Dear Ms. Hatch,

In accordance with your request, HWA GeoSciences Inc. (HWA) performed laboratory testing for the above referenced project. Herein we present the results of our laboratory analyses, which are summarized on the attached Figures. The laboratory testing program was performed in general accordance with your instructions and appropriate ASTM Standards as outlined below.

SAMPLE DESCRIPTION: The subject samples were delivered to our laboratory on February 27, 2024 by Ciani and Hatch personnel. The samples were delivered in re-sealable plastic bags and designated with exploration ID, sample number, and depth of sampling. The soil samples were classified using visual-manual methods. The descriptions may be found on the attached Summary of Material Properties, Figures 1 through 2.

MOISTURE CONTENT OF SOIL: The moisture content of the selected samples (percent by dry mass) was determined in general accordance with ASTM D 2216. The results are shown on the attached Summary of Material Properties, Figures 1 through 2.

PERCENTAGE FINER THAN #200 SIEVE: The percentage of material finer than the #200 sieve was determined for the selected samples in general accordance with ASTM D1140. The soil was oven dried and washed over a #200 sieve to determine the percentage of fines. The results are summarized on the attached Particle Size Analysis of Soils reports, Figures 3 through 9, which also provide information regarding the classification of the sample and the moisture content at the time of testing.

PARTICLE SIZE ANALYSIS OF SOILS (SIEVE AND HYDROMETER): Selected samples were tested to determine the particle size distribution in general accordance with ASTM D6913 and D7928. The results are summarized on the attached Particle Size Analysis of Soils reports, Figures 3 through 9, which also provide information regarding the classification of the sample and the moisture content at the time of testing.

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS (ATTERBERG LIMITS): The plasticity index of the selected sample was tested using method ASTM D4318, multi-point method. The results are reported on the attached Liquid Limit, Plastic Limit, and Plasticity Index report, Figure 10.

MOISTURE CONTENT, ASH, AND ORGANIC MATTER: The selected sample was tested in general accordance with method ASTM D 2974, using moisture content method 'A' (oven dried at 105^0 C) and ash content method 'C' (burned at 440^0 C). The results are percent by weight of dry soil and are summarized on Figure 2 of the attached Summary of Material Properties.

CLOSURE: Experience has shown that test values on soil and other natural materials vary with each representative sample. As such, HWA has no knowledge as to the extent and quantity of material the tested samples may represent. HWA also makes no warranty as to how representative either the samples tested or the test results obtained are to actual field conditions. It is a well-established fact that sampling methods present varying degrees of disturbance that affect sample representativeness.

No copy should be made of this report except in its entirety.

We appreciate the opportunity to provide laboratory testing services on this project. Should you have any questions or comments, or if we may be of further service, please call.

Sincerely,

HWA GEOSCIENCES INC.

Nick Johnson	Chad McMullen, P.E.
Materials Laboratory Manager	Quality Assurance Manager

Attachments:

Figures 1-2	Summary of Material Properties
Figures 3-9	Particle-Size Analysis of Soils
Figure 10	Liquid Limit, Plastic Limit and Plasticity Index of Soils

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
CHE-B1-24,1	2.5	4.0	10.3								18.6	SM	Olive-brown, silty SAND with gravel
CHE-B1-24,2	5.0	6.5	11.5						13.3	49.1	37.7	SM	Olive-brown, silty SAND
CHE-B1-24,3	7.5	9.0	11.9								30.8	SM	Olive, silty SAND with gravel
CHE-B1-24,4	10.0	11.5	14.9									SM	Dark grayish-brown, silty SAND
CHE-B2-24,1	2.5	4.0	8.9									SM	Dark grayish-brown, silty SAND with gravel
CHE-B2-24,2	5.0	6.5	8.0								34.6	SM	Dark gray, silty SAND with gravel
CHE-B2-24,3	7.5	9.0	7.4								31.0	SM	Olive-gray, silty SAND with gravel
CHE-B4-24,1	2.5	4.0	13.4								22.8	SM	Olive-brown, silty SAND with gravel
CHE-B4-24,2	5.0	6.5	11.2									SM	Olive-gray, silty SAND with gravel
CHE-B4-24,3	7.5	9.0	12.2								30.0	SM	Dark gray, silty SAND with gravel
CHE-B5-24,1	2.5	4.0	9.3									SM	Olive-gray, silty SAND with gravel
CHE-B5-24,2	5.0	6.5	9.2						24.5	45.1	30.4	SM	Olive-gray, silty SAND with gravel
CHE-B5-24,3	7.5	9.0	11.6								31.6	SM	Olive-gray, silty SAND with gravel
CHE-B5-24,4	10.0	11.5	10.5									SM	Olive-gray, silty SAND with gravel
CHE-B6-24,1	2.5	4.0	7.7								20.7	SM	Olive-gray, silty SAND with gravel
CHE-B6-24,2	5.0	6.5	9.8									SM	Olive-gray, silty SAND with gravel
CHE-B6-24,3	7.5	9.0	8.6									GM	Olive-gray, silty GRAVEL with silt and sand
CHE-B6-24,4	10.0	11.5	10.2								10.8	SP-SM	Olive-gray, poorly graded SAND with silt and gravel
CHE-B7-24,1	2.5	4.0	13.8								21.5	SM	Dark olive-gray, silty SAND with gravel and organics
CHE-B7-24,2	5.0	6.5	17.8			NP	NP	NP				SM	Dark olive-gray, silty SAND with gravel

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

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

SUMMARY OF MATERIAL PROPERTIES

PROJECT NO.: 2023-177 T200 FIGURE: 1

INDEX MATSUM 2 2023-177 T200.GPJ 3/4/24

EXPLORATION DESIGNATION	TOP DEPTH (feet)	BOTTOM DEPTH (feet)	MOISTURE CONTENT (%)	ORGANIC CONTENT (%)	SPECIFIC GRAVITY	ATTERBERG LIMITS (%)			% GRAVEL	% SAND	% FINES	ASTM SOIL CLASSIFICATION	SAMPLE DESCRIPTION
						LL	PL	PI					
CHE-B7-24,3a	7.5	8.0	15.8									ML	Olive-gray, SILT with gravel
CHE-B7-24,3b	8.0	9.0	146.0	25.6								OL	Very dark brown, organic SILT with sand
CHE-B7-24,4	10.0	11.5	11.7								28.0	SM	Olive, silty SAND with gravel
CHE-B9-24,1	2.5	4.0	9.5								24.7	SM	Olive-gray, silty SAND with gravel
CHE-B9-24,2	5.0	6.5	10.2								28.4	SM	Olive-gray, silty SAND with gravel
CHE-B9-24,3	7.5	9.0	9.6									SM	Olive-gray, silty SAND with gravel
CHE-B9-24,4	10.0	11.5	9.1								19.8	SM	Olive-gray, silty SAND with gravel
CHE-P3-24,1	2.5	4.0	22.1								20.5	SM	Dark brown, silty SAND with gravel and organics
CHE-P3-24,2	5.0	6.5	16.0						11.9	48.1	39.9	SM	Light olive-brown, silty SAND
CHE-P8-24,2	5.0	6.5	13.4								19.7	SM	Olive-brown, silty SAND with gravel and organics
CHE-P8-24,3	7.5	9.0	13.5									SM	Olive-brown, silty SAND with gravel and organics
CHE-P8-24,4	10.0	11.5	10.3									SM	Grayish-brown, silty SAND with gravel

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

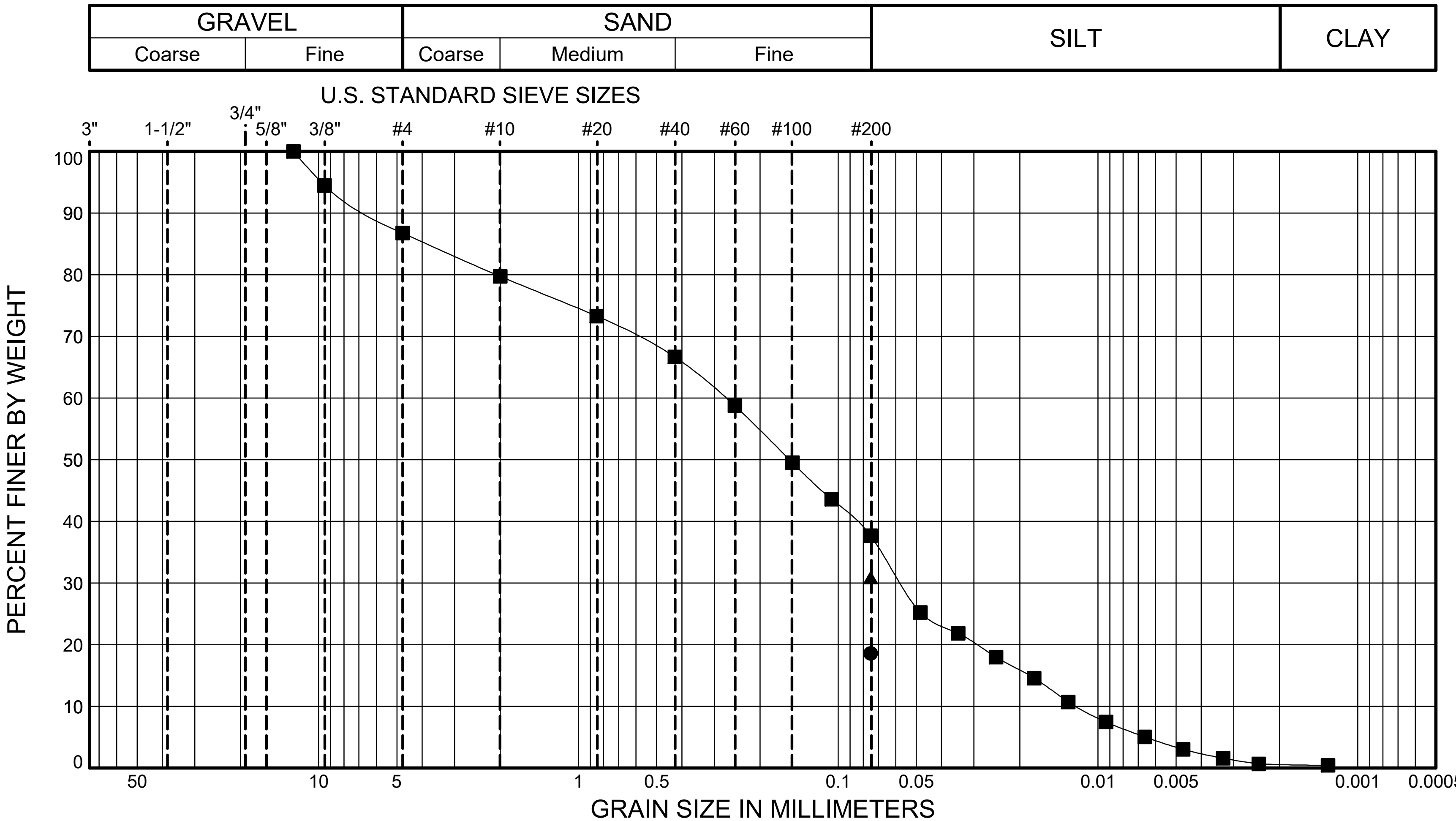
HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

SUMMARY OF MATERIAL PROPERTIES

PROJECT NO.: 2023-177 T200 FIGURE: 2

INDEX MATSUM 2 2023-177 T200.GPJ 3/4/24



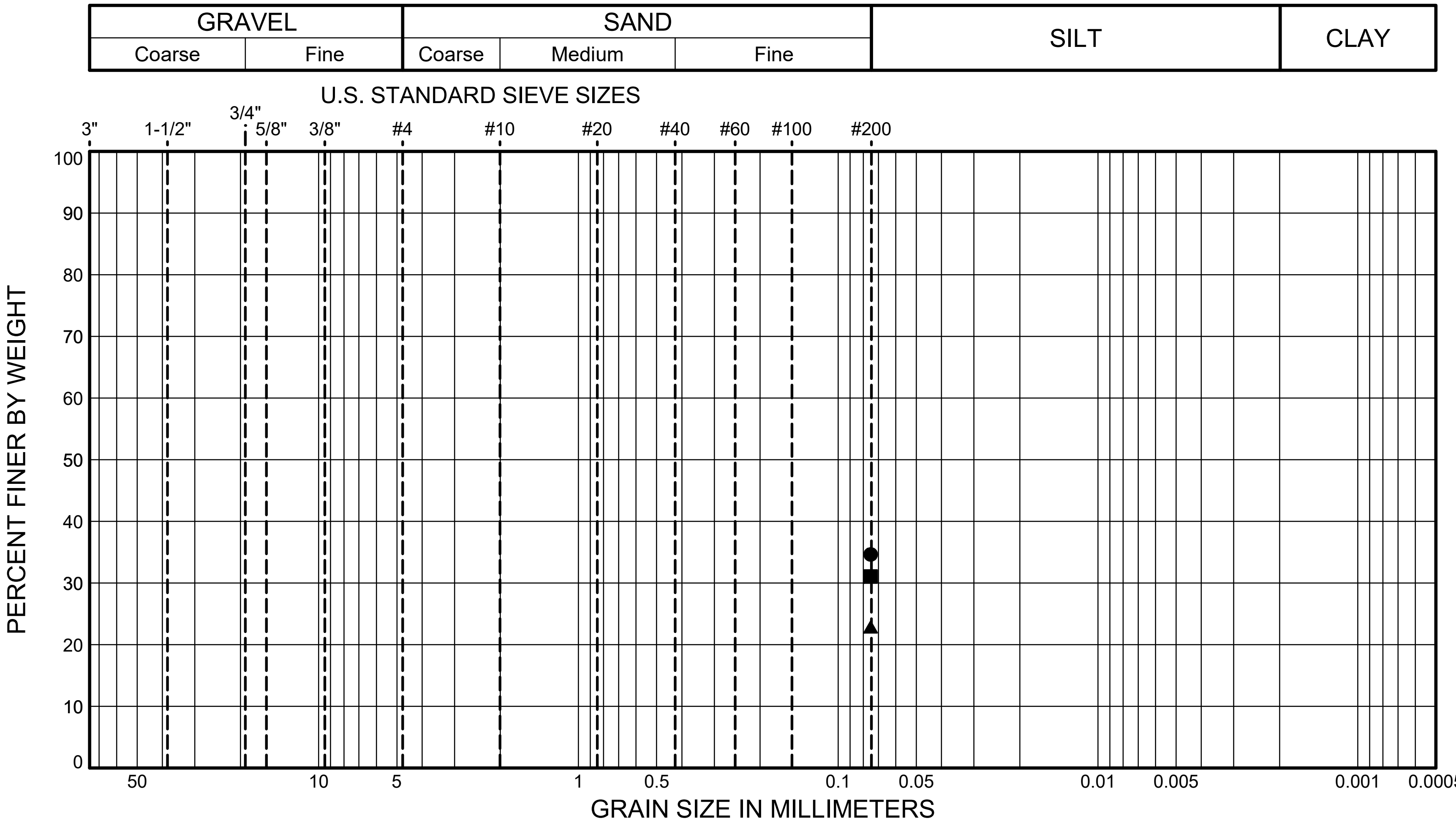
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-B1-24	1	2.5 - 4.0	(SM) Olive-brown, silty SAND with gravel	10								18.6
■	CHE-B1-24	2	5.0 - 6.5	(SM) Olive-brown, silty SAND	12				13.3	49.1	37.1	0.6	
▲	CHE-B1-24	3	7.5 - 9.0	(SM) Olive, silty SAND with gravel	12								30.8

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS
OF SOILS
METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 3



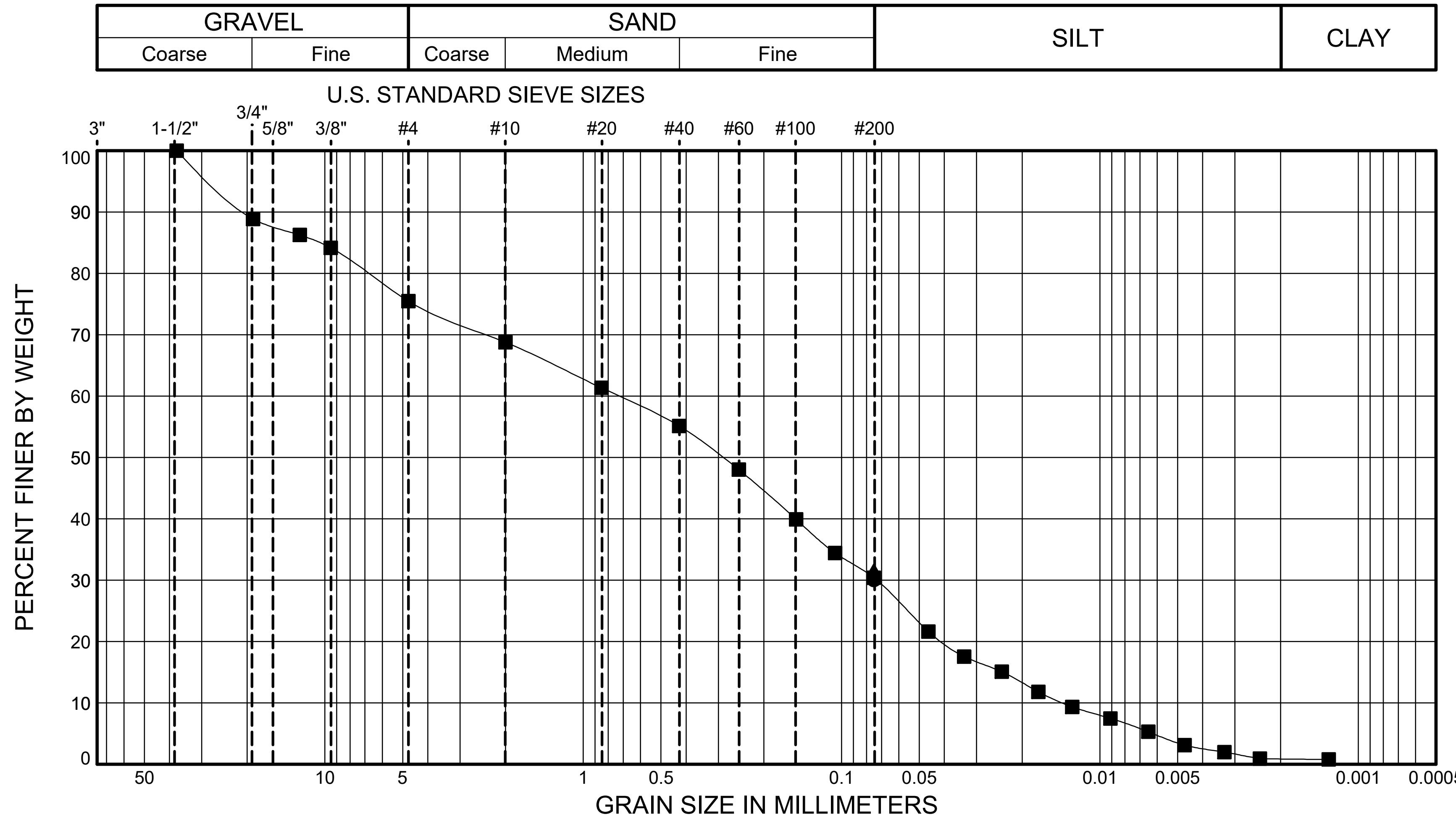
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-B2-24	2	5.0 - 6.5	(SM) Dark gray, silty SAND with gravel	8								34.6
■	CHE-B2-24	3	7.5 - 9.0	(SM) Olive-gray, silty SAND with gravel	7								31.0
▲	CHE-B4-24	1	2.5 - 4.0	(SM) Olive-brown, silty SAND with gravel	13								22.8

GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS
OF SOILS
METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 4



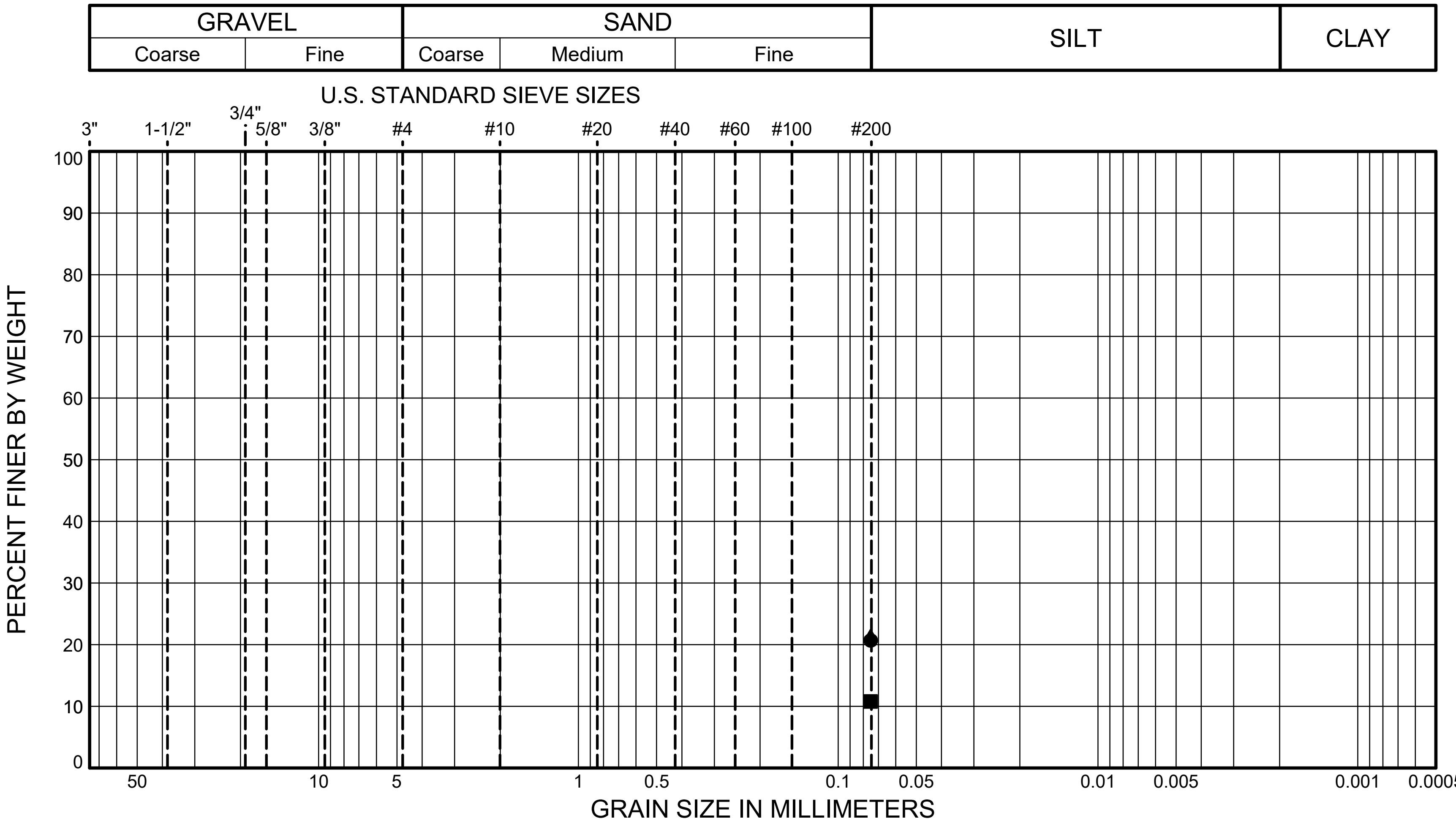
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-B4-24	3	7.5 - 9.0	(SM) Dark gray, silty SAND with gravel	12								30.0
■	CHE-B5-24	2	5.0 - 6.5	(SM) Olive-gray, silty SAND with gravel	9				24.5	45.1	29.5	0.9	
▲	CHE-B5-24	3	7.5 - 9.0	(SM) Olive-gray, silty SAND with gravel	12								31.6

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS
OF SOILS
METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 5



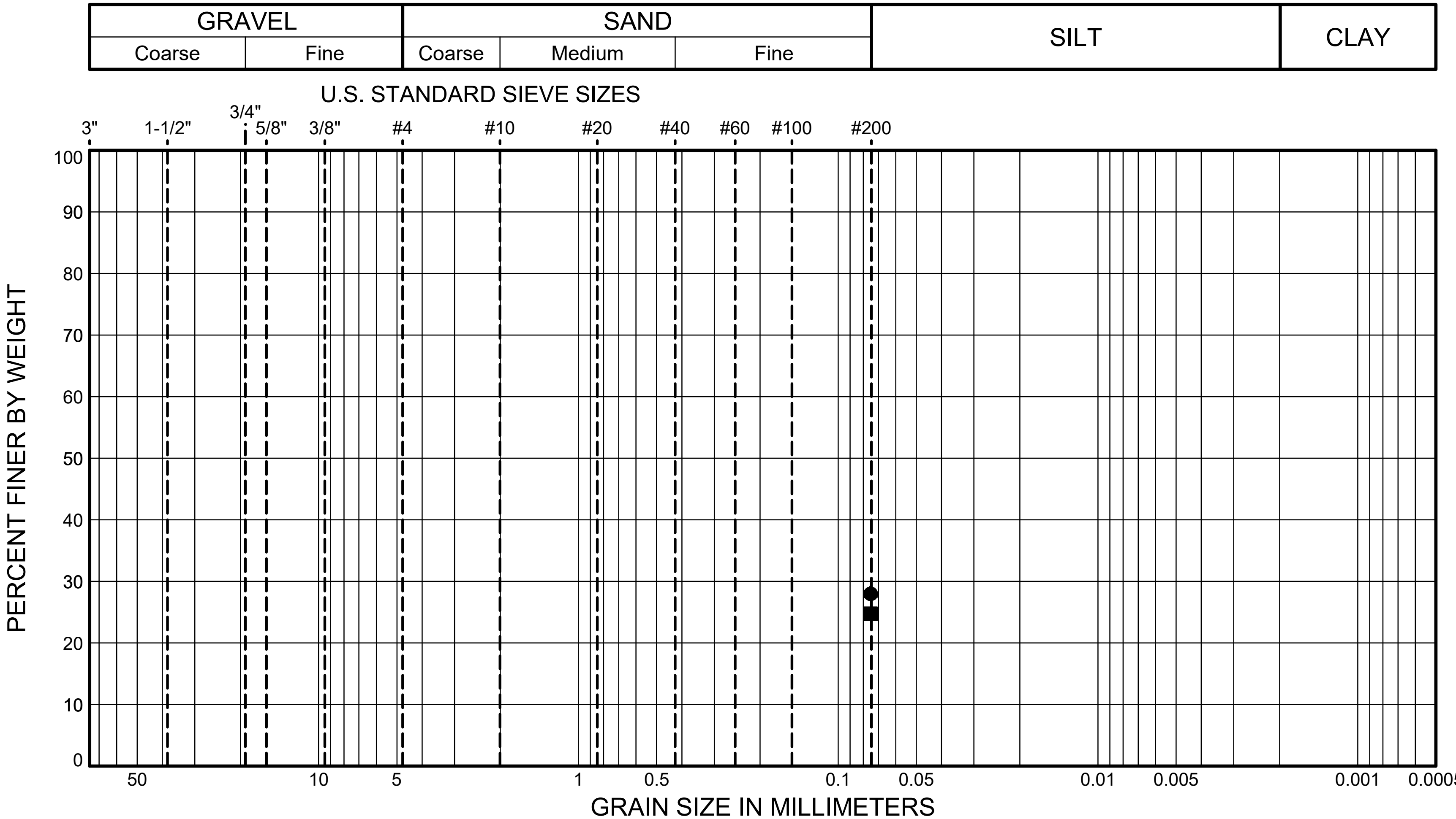
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-B6-24	1	2.5 - 4.0	(SM) Olive-gray, silty SAND with gravel	8								20.7
■	CHE-B6-24	4	10.0 - 11.5	(SP-SM) Olive-gray, poorly graded SAND with silt and gravel	10								10.8
▲	CHE-B7-24	1	2.5 - 4.0	(SM) Dark olive-gray, silty SAND with gravel and organics	14								21.5

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS OF SOILS METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 6



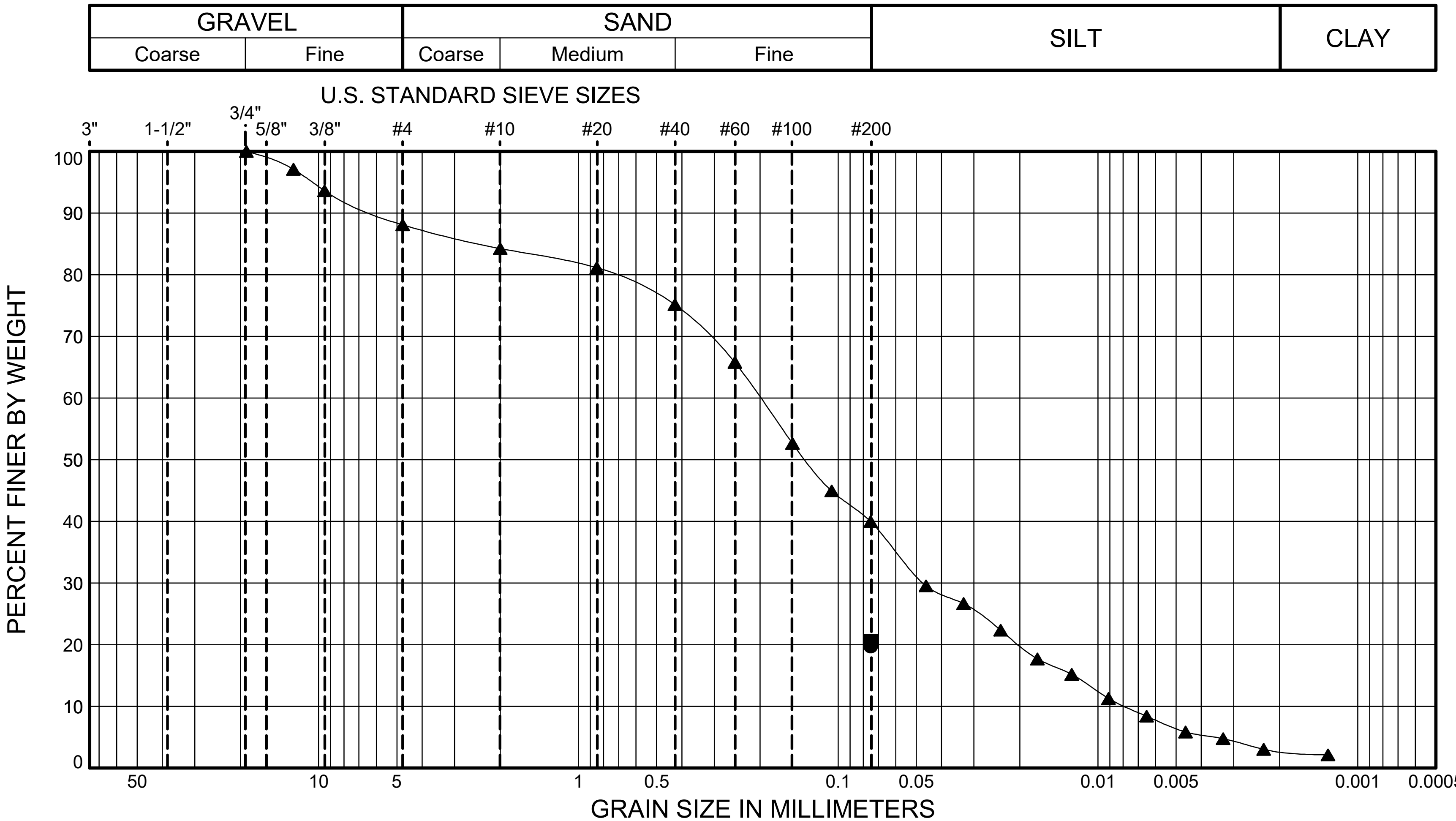
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-B7-24	4	10.0 - 11.5	(SM) Olive, silty SAND with gravel	12								28.0
■	CHE-B9-24	1	2.5 - 4.0	(SM) Olive-gray, silty SAND with gravel	10								24.7
▲	CHE-B9-24	2	5.0 - 6.5	(SM) Olive-gray, silty SAND with gravel	10								28.4

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS
OF SOILS
METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 7



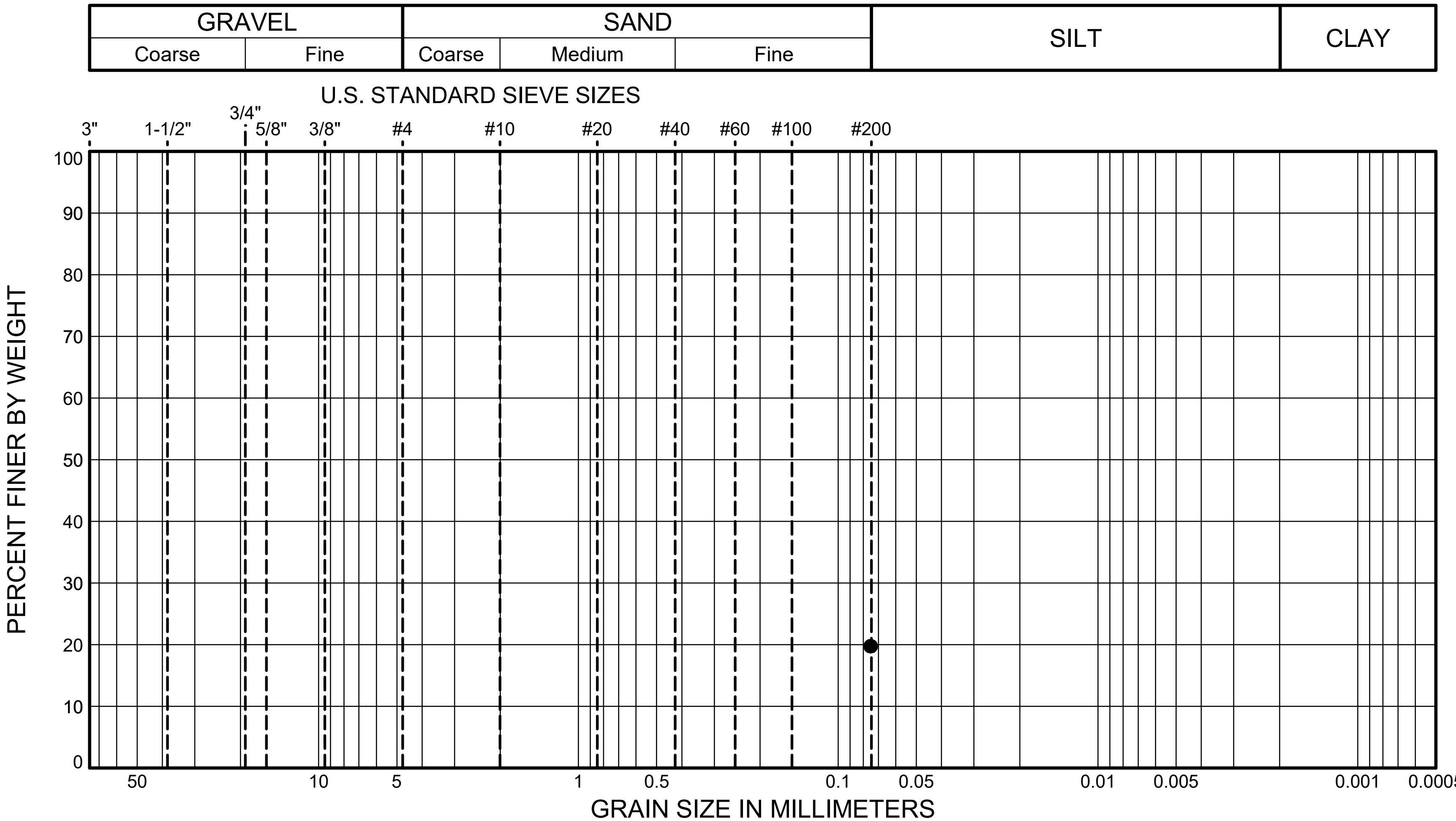
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-B9-24	4	10.0 - 11.5	(SM) Olive-gray, silty SAND with gravel	9								19.8
■	CHE-P3-24	1	2.5 - 4.0	(SM) Dark brown, silty SAND with gravel and organics	22								20.5
▲	CHE-P3-24	2	5.0 - 6.5	(SM) Light olive-brown, silty SAND	16				11.9	48.1	37.1	2.8	

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS
OF SOILS
METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 8



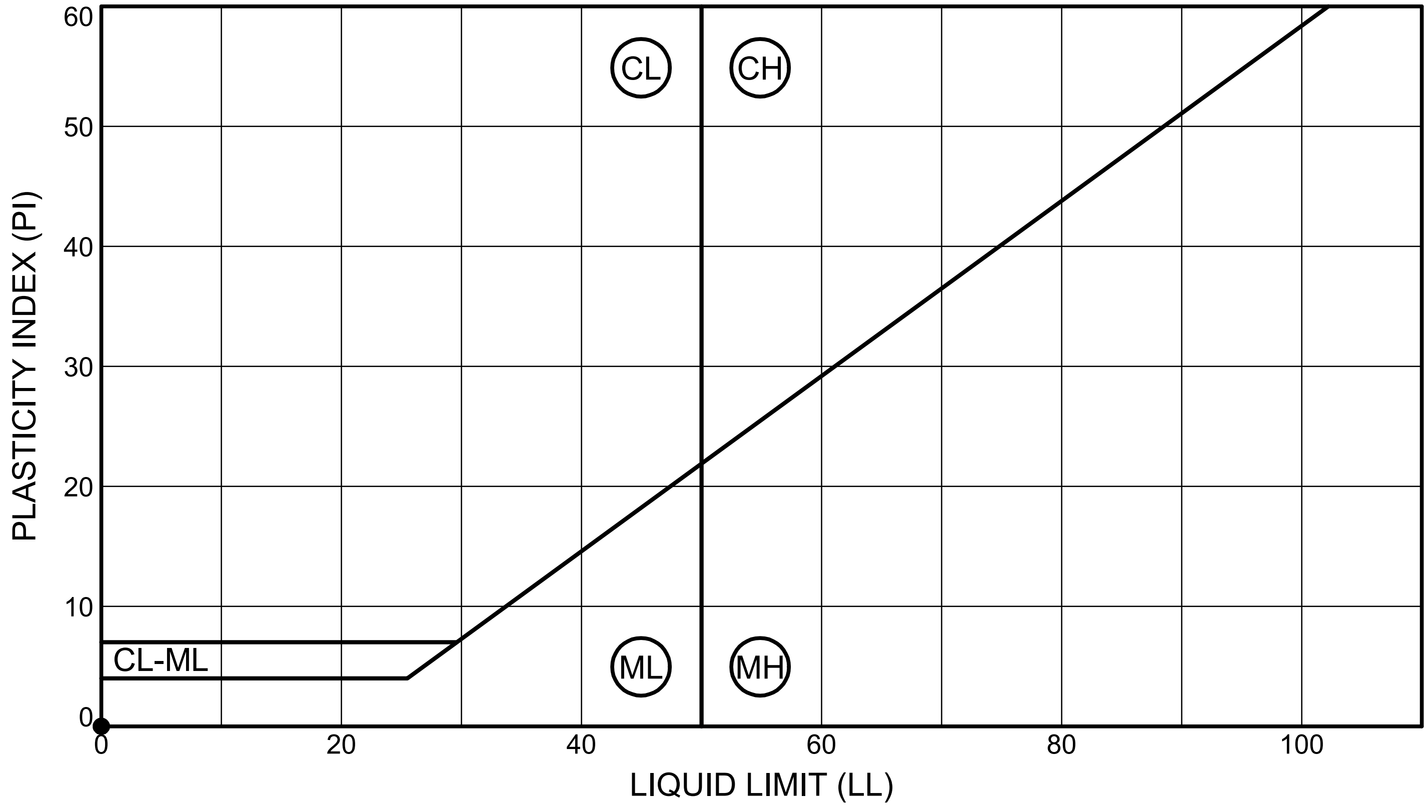
SYMBOL	SAMPLE		DEPTH (ft.)	ASTM SOIL CLASSIFICATION	% MC	LL	PL	PI	Gravel %	Sand %	Silt %	Clay %	Fines %
●	CHE-P8-24	2	5.0 - 6.5	(SM) Olive-brown, silty SAND with gravel and organics	13								19.7

GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

PARTICLE-SIZE ANALYSIS OF SOILS
METHODS ASTM D6913/D7928/D1140

PROJECT NO.: 2023-177 T200 FIGURE: 9



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	CHE-B7-24	2	5.0 - 6.5	(SM) Dark olive-gray, silty SAND with gravel	18	NP	NP	NP	

HWA GEOSCIENCES INC.

Laboratory Testing for Ciani & Hatch Engineering
HASCO - Timberglen & Pinewood Development
Client Project No.: 2002-001-00 Task 1

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

PROJECT NO.: 2023-177 T200 FIGURE: 10