GEOTECHNICAL REPORT
PROPOSED DEVELOPMENT
710 – 7th Avenue South
Seattle, Washington

PROJECT NO. 23-253
September 2023

Prepared for:

LIHI

PanGEO INCORPORATED

Geotechnical & Earthquake Engineering Consultants

PanGEO
INCORPORATED
Geotechnical & Earthquake
Engineering Consultants

September 6, 2023
File No. 23-253

Mr. Johnny Wheeler
Low Income Housing Institute
1253 South Jackson Street, Suite A
Seattle, Washington 98144

Re: Geotechnical Report
Proposed Development
710 – 7th Avenue South, Seattle, Washington

Dear Mr. Wheeler:

As requested, PanGEO, Inc. is pleased to present the attached geotechnical report for the proposed development located at 710 – 7th Avenue South in Seattle, Washington. In preparing this report, we reviewed previous test borings near the site, completed three test borings at the site, and conducted our engineering analyses. Based on the results of our evaluations, the principal geotechnical findings are as follows:

- Assuming one level of basement below street grade along 7th Avenue South, competent bearing soils are anticipated to be present at the footing subgrade elevation. Hence, conventional footings are considered adequate support the proposed building;
- Assuming the lowest basement floor will be located about 10 to 12 feet below the street grade along 7th Avenue South to the west, the excavation should not encounter a significant amount of groundwater; however, minor amount of perch groundwater may be present and, if encountered, likely can be controlled with sumps and pumps.
- It is our opinion that the temporary shoring systems, where needed, should consist of soldier piles with timber lagging and, if needed, tiebacks.

We appreciate the opportunity to work on this project. Please call if there are any questions.

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Sincerely,

Siew L. Tan, P.E.
Principal Geotechnical Engineer
(stan@pangeoinc.com)

Encl: Geotechnical Report

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1.0 INTRODUCTION

This report presents the results of our geotechnical study that was undertaken to support the proposed development located at 710 – 7th Avenue South in Seattle, Washington. This study was conducted in general accordance with our proposal dated July 24, 2023, which was approved on August 7, 2023. Our service scope included reviewing readily available geologic and geotechnical data, conducting a site reconnaissance, completing three test borings at the site, and developing the conclusions and recommendations presented in this report.

2.0 PROJECT AND SITE DESCRIPTION

The subject site consists of a rectangular shaped parcel (King County Parcel #524780-2540) located at 710 – 7th Avenue South in Seattle, Washington. According to King County records, the parcel has an area of approximately 21,600 square feet. It borders South Lane Street to the north, an improved alley (Canton Alley South) to the east, 7th Avenue South to the west, and two, one- to two-story, at-grade buildings to the south. The approximate location of the site is shown on attached Figure 1 (Vicinity Map).

The site is occupied by a two-story warehouse/retail building at approximately the southern half of the site. The northern half of the site includes a gravel and asphalt parking area at the approximate grade of Canton Alley South. As the parking area continues to the west, an approximate 10-foot-tall retaining wall at the northwest corner of the site provides support for the asphalt parking area maintaining the alley grade. Parking is also provided beneath this asphalt parking structure along 7th Avenue South. The general layout of the site is shown on the attached Figure 2 (Site and Exploration Plan). The existing site conditions are shown in Plate 1 on the following page.

Based on review of topographic data available on the Seattle Department of Construction & Inspections (SDCI) GIS portal, the northern parking portion of the site is relatively flat (approximate Elevation 74 feet). As the site progresses to the south, the grade slopes down, following the approximate grades of Canton Alley South and 7th Avenue South, to approximate Elevation 63 feet. These elevations should be reevaluated following a topographic survey of the site.

Plate 1. Site conditions – viewed from the corner of 7th Avenue South and South lane Street

We understand that the project will include demolition of the existing structures and to construct a new eight-story apartment building with one level of basement below the street grade of 7th Avenue South, or two levels below the alley. No other design information is available at this time.

Review of the SDCI GIS map indicates the there are no Environmental Critical Areas (e.g., steep slope, potential landslide, or liquefaction-prone) mapped at the site.

The conclusions and recommendations in this report are based on our understanding of the proposed development, which is in turn based on the project information provided. If the above project description is incorrect, or the project information changes, we should be consulted to review the recommendations contained in this study and make modifications, if needed. In any case, PanGEO should be retained to provide a review of the final design to confirm that our geotechnical recommendations have been correctly interpreted and adequately implemented in the construction documents.

3.0 ADJACENT BUILDINGS

Two existing buildings are located immediately south of the project site (see Plate 2, below), including a masonry building that is likely sensitive to ground movements. Our test boring PG-1, which was drilled near the existing building, encountered several feet of fill, suggesting that a portion of the existing buildings may be found in marginal soils. Depending on the location of the proposed south basement wall, it may be necessary to underpin the adjacent buildings to the south.

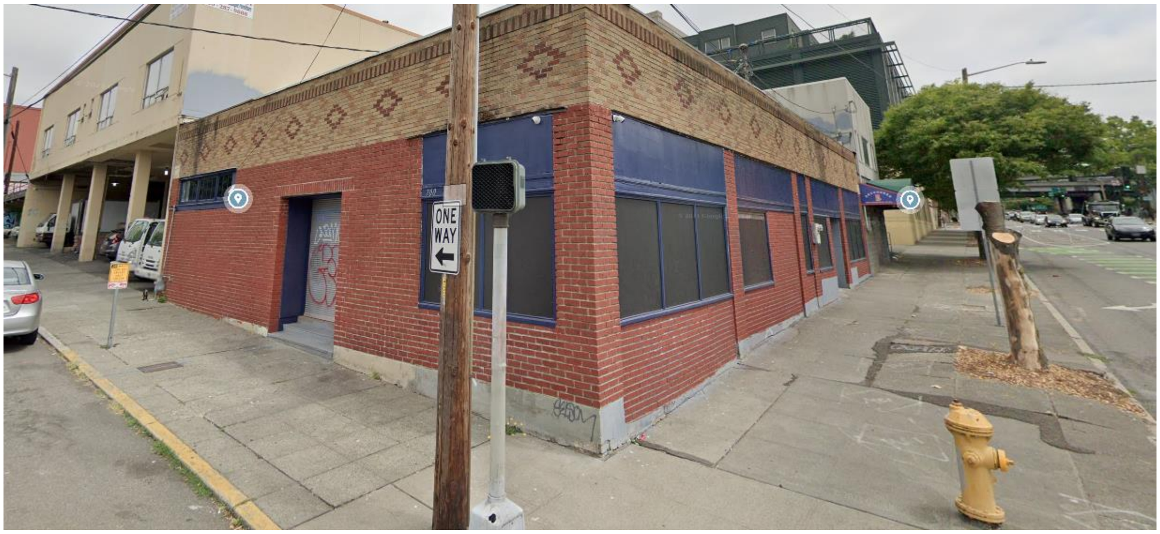

Plate 2. Existing neighboring buildings to the south of the site

Two existing multi-story mixed use buildings are located on the east side of the alley. Installation of tiebacks, if needed, may extend below the existing buildings. We recommend the designer verify the actual extent and depth of the adjacent foundations prior to the final design. Potential conflicts with existing basements should be taken into design considerations.

If tiebacks are needed, construction easements will be needed from the adjacent property owners to the south and to the east.

4.0 SUBSURFACE EXPLORATIONS

4.1 TEST BORINGS

On August 23, 2023, we completed three test borings (PG-1 through PG-3) at the approximate locations shown on Figure 2. The test borings were advanced to about 26½ to 41½ feet below the existing grade using hollow stem augers. Soil samples were obtained from the borings at 2½- and 5-foot depth intervals in conjunction with Standard Penetration Test (SPT) sampling methods in general accordance with ASTM test method D-1586, in which the samples are obtained using a 2-inch outside diameter split-spoon sampler. The sampler was driven into the soil a distance of 18 inches using a 140-pound weight falling a distance of 30 inches. The number of blows required for each 6-inch increment of sampler penetration was recorded. The number of blows required to achieve the last 12 inches of sample penetration is defined as the SPT N-value. The N-value provides an empirical measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils.

An engineer from PanGEO was present on a full-time basis to observe the drilling, assist in sampling, and to describe and document the soil samples obtained from the borings. The soil samples were described, and field classified in general accordance with the symbols and terms outlined in Figure A-1, and the summary boring logs are included as Figures A-2 through A-4.

4.2 LABORATORY TESTING

The following laboratory tests were performed on two representative soil samples collected from our borings:

- Moisture Content (ASTM D 2216)
- Atterberg Limits (ASTM D 4318)

The test results are noted on the boring logs in Appendix A, where appropriate. The Atterberg limit tests are also plotted and included as Figure B-1 in Appendix B.

4.3 PREVIOUS TEST BORINGS AND LABORATORY TESTING

We collected and reviewed readily available test boring logs from previous explorations near and at the site to supplement our test borings. Specifically, our research indicates that two test borings were previously completed for the design of the apartment building to the east, across the alley (Applied Geotechnology, Inc., 1993). The approximate locations of these two test borings (AG-1 and AG-2) are shown on the attached Figure 2. The summary test boring logs are included in Appendix C, and the associated laboratory tests are included as Appendix D.

5.0 SUBSURFACE CONDITIONS

5.1 GEOLOGY

According to the geologic map of the Seattle area compiled by Troost and others (2005), the project site is underlain by Pre-Olympia-aged glacial deposits (Map Unit Qpog). This soil unit has been further differentiated into Pre-Olympia Coarse-Grained Deposits (Qpoc) and Pre-Olympia Fine-Grained deposits (Qpof):

- Pre-Olympia Coarse-Grained Deposits are described by Troost et al. (2005) as clean to silty sand and gravel with some silt layers and is commonly iron oxide stained.

- Pre-Olympia Fine-Grained Deposits are described as a laminated to massive silt and clays with sandy interbeds.

Both soil units have been overridden by multiple advancing ice sheets and as such are typically very dense or very stiff to hard.

5.2 SOIL CONDITIONS

The following is a generalized description of the soils encountered in the borings. For a more detailed description of the subsurface conditions encountered at each exploration location for this study, please refer to the boring logs provided in Appendices A and C of this report.

> ***Unit 1: Fill (Hf)* -** At test boring PG-1, which was drilled in the loading dock near the southeast corner of the site, encountered approximately 2 inches of asphalt, underlain by about 4 feet of medium dense silty sand to sandy silt. At test boring PG-2 at the elevated parking lot area similarly, a 2-inch layer of asphalt was encountered above a loose to medium dense silty sand to sandy silt and extended to about 9 feet below grade (the approximate grade of 7th Avenue South). We interpret this surficial soil layer as fill.

At test boring PG-3 (at the approximate grade of Canton Alley South), an approximate 6-inch layer of crushed gravel was encountered bearing directly on the Qpoc unit described below. The thickness of existing fill at test boring locations generally correlates with historic street grading profile.

Previous test borings AG1 and AG2, located east of the alley (see Figure 2), encountered approximately 10 feet of medium dense to loose surficial fill soils in AG1 and minimal amounts of fill in AG2.

Unit 2: Pre-Olympia Coarse-Grained Deposits (Qpoc) - Underlying the existing fill in all test borings, a medium dense to dense interbedded silty sand and sandy silt with varying amounts of gravel was encountered. This unit extended to approximately 18 feet below grade in PG-1, 21 feet in PG-2, 8 feet in PG-3, and approximately 10 feet below-grade in previous boring AG2. Previous boring AG1 did not appear to encounter this unit. This soil unit appeared consistent with a coarse-grained unit of Pre-Olympia Glacial Deposits mapped in the vicinity.

Unit 3: Pre-Olympia Fine-Grained Deposits (Qpof) - Underlying the coarse-grained deposits of PG-1, PG-2, PG-3, and AG2, and beneath the fill in AG1, a very stiff to hard, gray clay was encountered to the bottom of each boring. As indicated by results of our laboratory tests (see Appendix B), this clay layer has high plasticity.

In borings PG-3 and AG-2, near the northeast portion of the site, this unit became coarser, and very dense with depth beginning at approximately 29 feet and 23 feet below grade, respectively.

We interpret this soil unit to be consistent with the fine-grained unit of the Pre-Olympia Glacial Deposits mapped in the vicinity.

It should be noted that the stratigraphic contacts indicated on the boring logs and subsurface profiles represent the approximate depth to boundaries between soil units. Actual transitions between soil units may be more gradual or occur at different elevations. The descriptions of groundwater conditions and depths are likewise approximate.

5.3 GROUNDWATER

During drilling, perched groundwater seepage was encountered at the contact of coarse- and fine-grained deposits in test boring PG-1 (southwest corner of the site, in the loading dock area), from about 16 to 18 feet below grade. Also encountered in PG-1 was a very moist to wet sand lens at approximately 25½ feet below grade. Soil samples in previous boring AG1 were also described as ‘wet’ at approximately 5 feet below grade within the fill unit described above at time of drilling in December 1992.

Evidence of groundwater was not observed in borings PG-2 or PG-3, nor noted on the log of AG2 at time of drilling.

Groundwater levels and seepage rates may vary depending on the season, local subsurface conditions, and other factors. Groundwater levels are normally highest during the winter and early spring.

6.0 GEOTECHNICAL RECOMMENDATIONS

6.1 SEISMIC DESIGN CONSIDERATIONS

6.1.1 Seismic Site Class

We assume that the seismic design of the building will be accomplished using the 2018 or 2021 editions of International Building Code (IBC), which specifies a design earthquake having a two percent probability of occurrence in 50 years (return interval of 2,475 years). The IBC seismic design parameters are in part based on the site soil conditions and site classifications defined in Chapter 20 of ASCE 7-16. Based on Chapter 20 of ASCE 7-16 and the results of current and test borings, the site soils should be classified as Site Class D (Stiff Soils).

6.1.2 Liquefaction Potential

Liquefaction is a process that can occur when soil loses shear strength for short periods of time during a seismic event. Ground shaking of sufficient strength and duration results in the loss of grain-to-grain contact and an increase in pore water pressure, causing the soil to behave as a fluid. Soils with a potential for liquefaction are typically cohesionless, predominately silt and sand, and are below the groundwater table. Based on the subsurface conditions at the site, it is our opinion that the potential for liquefaction is negligible and building design related to soil liquefaction is not necessary for this project.

6.2 FOUNDATIONS

Based on the conditions encountered in the test borings, glacially consolidates soils (very stiff to hard silts and clays, and dense silty sands to sandy silts) are anticipated to be present at the basement subgrade elevation. In our opinion, building support can be provided using conventional footings bearing on the undisturbed glacially consolidated soils, or adequately compacted structural fill.

6.2.1 Allowable Bearing Pressure

A maximum allowable soil bearing pressure of 8,000 pounds per square foot (psf) may be used to size the footings that bear directly on the undisturbed native soils or on CDF placed on undisturbed native soils. Footings bear on more than 2 feet of compacted structural fill should be sized using a reduced pressure of 4,000 psf.

For allowable stress design, the recommended allowable bearing pressure may be increased by one-third for transient loading conditions such as those due to wind and seismic forces.

Continuous and individual spread footings should have minimum widths of 18 and 24 inches, respectively. For frost protection considerations, footings should be placed at least 18 inches below adjacent finished grade and at least 12 inches below top of slab on grade floors.

6.2.3 Lateral Resistance

Lateral loads acting on footings may be resisted by passive earth pressure developed against the embedded portion of the footings and by frictional resistance developed at the base of the footings.

- For footings bearing directly on the native soils, CDF or compacted structural fill, a frictional coefficient of 0.35 may be used to evaluate sliding resistance.
- Passive soil resistance may be calculated using an equivalent fluid pressure of 400 pcf, assuming the footings are backfilled with structural fill and level ground surface. Unless

covered by pavements or slabs, the passive resistance in the upper 12 inches of soil should be neglected.

The above values include a factor of safety of 1.5.

6.2.4 Footing Subgrade Preparation

Footings should bear directly on the undisturbed native soils expected to be encountered at or near the footing subgrade elevation. It should be noted that fill soils were encountered to approximately 4-foot-deep at the southwest corner of the site within the existing loading dock area, and thicker fill may be present at other locations. Loose or soft soils should be removed from the footing excavations and replaced with Control Density Fill (CDF) or adequately compacted structural fill. Please note that a lower bearing pressure should be used to size footings that will be placed on structural fill.

The site soils are highly moisture sensitive and can become disturbed and softened when exposed to moisture and construction traffic. Protection of the foundation bearing soils should be the responsibility of the contractor.

The adequacy of the footing subgrade should be verified by a representative of PanGEO, prior to placing forms or rebar.

6.2.5 Foundation Performance

Total and differential settlements are anticipated to be within tolerable limits for footings designed and constructed as discussed above. Footing settlement under static loading conditions is estimated to be about 1 inch, and differential settlement between adjacent columns should be about ½ inch. Most settlement will be realized during construction as the deadloads are applied.

6.3 BASEMENT WALL DESIGN PARAMETERS

Basement walls should be designed to resist the lateral earth pressures exerted by the soils behind the wall. Adequate drainage provisions should also be provided behind the walls to intercept and remove groundwater that may collect behind the walls. Our geotechnical recommendations for the design and construction of the below-grade walls built with drainage provisions are presented below.

6.3.1 Lateral Earth Pressures

Cantilevered retaining walls should be designed for an active equivalent fluid earth pressure of 35 pcf with level backslope and 50 pcf with a maximum backslope of 2H:1V (Horizontal:Vertical). Basement walls (i.e., walls restrained at the top) should be designed for an at-rest equivalent fluid pressure of 50 pcf for a level backslope condition and 60 pcf for a maximum backslope condition of 2H:1V.

In addition, the walls should be designed for a uniform lateral pressure of 9H pounds per square foot (psf) for seismic loading, where H corresponds to the retained height of the wall. The recommended lateral pressures assume the backfill behind the wall consists of a free draining and properly compacted fill with adequate drainage provisions.

6.3.2 Surcharge

If the below-grade walls will be subjected to the influence of building or traffic surcharge loading within a horizontal distance equal to or less than the height of the walls, a uniform horizontal pressure of 80 psf should be used to represent the traffic surcharge. The traffic surcharge should be limited to the uppermost 15 feet of the wall.

Other surcharges may be evaluated using the recommendations provided in Figure 3.

6.3.3 Wall Drainage

We recommend that perimeter wall/footing drains be installed to provide permanent control of subsurface water adjacent to the new structure. For walls constructed against temporary shoring, a geocomposite drainage mat can be used between the shoring and permanent basement walls. The geocomposite drain should be connected to weep pipes cast into the base of the permanent wall. The weep pipes should be connected to a minimum 4-inch diameter footing drain around the interior perimeter of the building, below the lower slab on grade floor or below the top of the mat foundation.

For free-standing concrete retaining walls, a minimum, 4-inch diameter perforated drainpipe should be installed next to the base of the wall foundation and embedded in 12 to 18 inches of clean gravel. The gravel should be wrapped in a geotextile filter fabric to prevent the migration of fines into the drain system. The drainpipe should be graded to direct water to a suitable outlet. We recommend a minimum 18-inch-wide zone of free draining granular soils (i.e., washed rock or equivalent) be placed adjacent to the wall for the full height of the wall. Alternatively, a geocomposite drainage mat may be used in lieu of the clean washed rock.

Waterproofing considerations are beyond our expertise and scope of services. A building envelope specialist should be consulted to determine appropriate damp-proofing or water-proofing measures.

6.3.4 Wall Backfill

Wall backfill, if needed, should consist of imported, free draining granular material, such as a soil meeting the requirements of Gravel Borrow as defined in Section 9-03.14(1) of the WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT, 2023); or City of Seattle Type 17 aggregate. In areas where space is limited between the wall and the face of excavation, pea gravel may be used as backfill with minimal compaction.

Wall backfill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than about one foot in thickness, and systematically compacted to a dense and relatively unyielding condition, as verified by PanGEO personnel. If density tests will be performed, the test results should demonstrate at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557. Within 5 feet of retaining walls, the backfill should be compacted with hand-operated equipment to at least 90 percent of the maximum dry density.

6.4 CONCRETE SLAB ON GRADE FLOORS

The floor slabs for the proposed building may be constructed using conventional concrete slab-on-grade floor construction. The floor slabs should be supported on competent native soil or adequately compacted structural fill. Any over-excavation, if needed, should be backfilled with structural fill. The adequacy of the slab subgrade should be verified by PanGEO during construction.

Concrete slab-on-grade floors should be underlain by a minimum 4-inch-thick capillary break. The capillary break material should meet the gradational requirements provided in Table 1, below. A minimum 10-mil polyethylene vapor barrier should also be placed directly below the slab. We also recommend that control joints be incorporated into the floor slab to control cracking.

TABLE 1: Capillary Break Gradation

Sieve Size	Percent Passing
¾-inch	100
No. 4	0 – 10
No. 100	0 – 5
No. 200	0 – 3

6.5 PERMANENT SURFACE DRAINAGE CONSIDERATIONS

Permanent control of surface water and roof runoff should be incorporated into the final grading design. All collected runoff should be directed into conduits that carry the water away from the structure and into storm drain systems or other appropriate outlets. Adequate surface gradients should be incorporated into the grading design such that surface runoff is directed away from the structure. Collected water from surface runoff should not be allowed to discharge into retaining wall or footing drain systems.

6.6 PERMANENT CUT AND FILL SLOPES

Permanent cut and fill slopes should be constructed no steeper than 2H:1V (Horizontal:Vertical).

Cut slopes should be observed by PanGEO during excavation to verify that conditions are as anticipated. Supplementary recommendations can then be developed, if needed, to improve stability, including flattening of slopes or installation of surface or subsurface drains.

Permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve stability of the surficial layer of soil.

7.0 TEMPORARY EXCAVATIONS AND SHORING

7.1 TEMPORARY EXCAVATION SLOPES AND SHORING CONSIDERATIONS

Temporary excavations should be made in accordance with Part N of the WAC (Washington Administrative Code) 296-155. The contractor is responsible for maintaining safe excavation slopes and/or shoring. For planning purposes, if space is available and there are no conflicts with existing underground utilities or structures, temporary excavations should be sloped no steeper than 1H:1V (Horizontal:Vertical), assuming no critical structures or buildings located within 10 feet of the top of the excavation.

Temporary excavations should be evaluated in the field during construction based on actual observed soil conditions. If seepage is encountered, excavation slope inclinations may need to be reduced. During wet weather, the cut slopes may need to be flattened to reduce potential erosion or should be covered with plastic sheeting.

The cut slopes should be covered with plastic sheeting. We also recommend that heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance equal to one-third of the slope height from the top of the excavation.

For the proposed east and south shoring walls adjacent to existing buildings and right-of-way, design considerations will need to be made regarding temporary shoring adjacent to the existing neighboring footings which are envisaged to be of at-grade construction. We recommend the designer verify the actual extent and depth of the adjacent foundations and basement walls prior to the final design to avoid conflicts with the existing and proposed site configurations.

It is also understood that the basement level of the existing warehouse at the site daylights on the west side of the parcel (7th Avenue South) and follows a similar finished floor elevation across to the east side of the parcel (Canton Alley South) where the basement level is below the alley grade. As such, the current building basement walls are envisaged to shore up the alley soils in their current configuration.

The shoring system should be designed to provide adequate protection for the workers, adjacent structures, utilities, and other facilities. Excavations should be performed in accordance with the current requirements of WISHA. Construction should proceed as rapidly as feasible, to limit the time temporary excavations are open.

7.2 SOLDIER PILE SHORING WALL

Where needed, excavation shoring should consist of soldier piles with timber lagging. A soldier pile wall consists of vertical steel beams, typically spaced from 6 to 8 feet apart along the proposed excavation wall, spanned by timber lagging. Prior to the start of excavation, the steel beams are installed into holes drilled to a design depth and then backfilled with CDF or lean mix concrete. As the excavation proceeds downward and the steel piles are subsequently exposed, timber lagging is installed between the piles to further stabilize the walls of the excavation.

For excavations more than 12 to 15 feet deep, tiebacks are typically used to reduce the pile size and pile deflections.

7.2.1 Design Lateral Pressures

We recommend the earth pressures depicted in Figure 4 be used to design soldier pile shoring. Above the bottom of the excavation, the active and surcharge pressures should be applied over the full width of pile spacing. Below the bottom of excavation, the active and surcharge pressures should be applied over one pile diameter, and the passive resistance should be applied over two times the pile diameter.

The lateral earth pressures shown on Figure 4 should be increased for any surcharge loads resulting from traffic, construction equipment, building loads or excavated soil if they are located within the height dimension of the wall. The surcharges may be calculated based on Figure 3.

Heavy point loads such as outriggers for concrete pump trucks and cranes may apply additional loads to the lagging. These loads should be individually analyzed and where appropriate should be included in the shoring design calculations.

7.2.2 Temporary Lagging

The following minimum lagging sizes shown in Table 2 below should be used for this site, in accordance with the *FHWA Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored Systems*, based on presence of competent soils at the site.

TABLE 2: Recommended Lagging Thickness

Depth (feet)	Recommended thickness of lagging (roughcut) for clear spans of:		
	6 feet	7 feet	8 feet
0 – 26 feet	3 inches	3 inches	3 inches
26 – 50 feet	3 inches	3 inches	4 inches

Along the south wall, we recommend that any voids behind timber lagging be backfilled with CDF. Aggregate fill may be used behind the other three walls.

7.2.3 Vertical Soldier Pile Capacity

We recommend the vertical capacity of the soldier piles be determined using an allowable skin friction value of 1.0 ksf for the portion of the pile below the bottom of the excavation, and an allowable end bearing value of 30 ksf.

7.3 TIEBACKS

The manner in which the tieback anchors carry load depends on the type of anchor selected, the method of installation, and the soil conditions surrounding the anchor. Accordingly, we recommend the use of a performance specification requiring the shoring contractor to install anchors capable of satisfactorily achieving the design structural loads, with a pullout resistance factor of safety of 2.0. The performance specification should be field verified per our recommendations outlined below.

7.3.1 Assumed Tieback Design Parameters and Considerations

For planning purposes, however, the anchors may be sized using an assumed allowable skin friction value of 2.5 kips per linear foot of anchor bond length, assuming that small diameter (about 6 inches) tiebacks will be used. Pressure grouting and multiple post-grouting may be needed to achieve the assumed capacity.

If the contractor believes that, based on their proposed installation method in similar soil conditions, the assumed value should be revised, the tieback lengths should be revised accordingly.

The bond zone portion of the tiebacks must be located behind a no-load zone as defined in Figure 4. The tiebacks should have a minimum bond length of 15 feet beyond the no-load zone; longer tiebacks may be needed based on the design calculations.

Tiebacks will extend beyond the property boundaries and tieback easements will need to be obtained. The easements should be obtained as early in the design process as possible because the project costs could be significantly impacted without the construction easements. Although soldier piles may also be internally supported by braces or rakers, such construction methods will be significantly more costly than tiebacks and will impact the construction sequence.

Excessive pile top deflections could occur before the first row of tiebacks is installed. To improve the performance of the tieback wall, it may be necessary to limit the first row of tiebacks to no more than 10 feet below pile top unless steel beams of sufficient size will be used to limit the magnitude of the cantilever deflection.

In the tieback construction, a bond breaker shall be constructed in the no load zone if the installation procedure uses single stage grouting.

7.3.2 Tieback Testing – Verification Test

The actual capacity of the anchors should be confirmed with verification tests that test the tiebacks up to 200 percent of the design load. The anchor testing should be conducted in accordance with the latest edition of the Post-Tensioning Institute (PTI) *Recommendations for Prestressed Rock and Soil Anchors*. Verification testing procedure should adhere to the following recommendations:

- Prior to installing production anchors, perform a minimum of two tests each on each anchor type, installation method, and soil type with the tested anchors constructed to the same dimensions as production anchors. Contractor may choose to install the test anchors as part of the production anchors at its own risk.

- Test locations to be determined in conjunction and approved by the geotechnical engineer.

- Verification test anchors, which will be loaded to 200 percent of the design load, may require additional steel tendons so that the stress will not exceed 80 percent of the ultimate tensile strength.

- The verification test anchors should be loaded to a maximum 200 percent design load in 25 percent load increments, holding each incremental load until stable and recording deflection of the anchor head at various times within each hold to the nearest 0.01 inch.

- At the 150 percent design load, the holding period shall be at least 60 minutes.

- At the 200 percent design load, the holding period shall be for at least 10 minutes.

- An acceptable test shall provide a measured creep rate of 0.04 inches or less at the 150 percent load between 1 and 10 minutes, and 0.08 inches between 6 and 60 minutes, and both shall have a creep rate that is linear or decreasing with time. The applied load must remain constant during all holding periods (i.e., no more than 5 percent variation from the specified load).

Verification tested anchors or extended creep proof tested anchors not meeting the acceptance criteria will require a redesign by the contractor to achieve the acceptance criteria.

7.3.3 Tieback Testing – Proof Test

All production anchors should be proof tested as outlined below:

- Load test all production anchors to 130 percent of the design load in 25 percent load increments, holding each incremental load until a stable deflection is achieved (record deflection of the anchor head at various times within each hold to the nearest 0.01 inch).

- At the 130 percent design load, the holding period shall be at least 10 minutes.

- An acceptable test shall provide a measured creep rate of 0.04 inches or less at the 130 percent design load between 1 and 10 minutes. The creep rate must be linear or decreasing with time. The applied load must remain constant during the holding period (i.e., no more than 5 percent variation from the specified load). Anchors failing this proof testing creep acceptance criteria may be held an additional 50 minutes for creep measurement. Acceptable performance would equate to a creep of 0.08 inches or less between 6 and 60 minutes with a linear or decreasing creep rate.

7.4 GROUNDWATER, CAVING, AND OBSTRUCTION CONSIDERATIONS

Perched groundwater conditions were encountered in test boring PG-1 between approximately 16 to 18 feet below grade at time of drilling, which is deeper than the planned excavation. However, the drilling for soldier piles may extend into perched groundwater or groundwater lenses should be expected near the contact of coarse- and fine-grained glacially consolidated units. The contractor should be prepared to stabilize the holes, including the use of drilling mud and temporary casings.

Where more than a foot of groundwater is present in the bottom of drilled soldier pile holes, the concrete should be placed using a tremie pipe. When placing timber lagging, the height of each lift may need to be limited if wet soils are encountered. The actual allowable vertical cut for timber lagging placement should be determined in the field, based on the actual conditions observed.

It should also be noted that gravel was encountered throughout each unit of soil encountered in our borings and cobbles and boulders may also be present. As such, obstructions due to large cobbles and boulders may be encountered during drilling for soldier piles. If obstructions cannot be cleared with typical drilling methods, alternative locations and sizes for shoring elements should be considered.

7.5 BASELINE SURVEY AND PERFORMANCE MONITORING

Ground movements will occur because of excavation activities. As such, the condition of adjacent properties and city right-of-way should be documented prior to commencing earthwork to provide accurate baseline data.

As a minimum, shoring monitoring shall conform to the guidelines summarized below:

- Prior to installation of soldier piles, establish baseline survey markers on adjacent buildings to the south and to the east. The monitoring points shall be monitored at least twice weekly for vertical and horizontal displacement during shoring installation and excavation.

- Establish monitoring points for baseline readings along the concrete curbs, along the centerline of the roadways, and at distances up to the wall height, H, behind the wall in private properties. These monitoring points should be spaced no more than 20 feet apart. These monitoring points need not be monitored unless the shoring wall exhibits lateral movements approaching or more than 0.5 inches.

- After installation of soldier piles but prior to mass excavation, establish monitoring points for baseline reading at the top of every other soldier pile. The monitoring points shall be monitored at least twice weekly for vertical and horizontal displacement during shoring installation and excavation.

- Submit survey data to the project team each week. Notify the project team immediately if unusual or significantly increased movements occur, or if 0.5 inches of movement occurs between two consecutive readings.

The optical survey frequency may be decreased after completion of the initial foundation footings if the data indicates no or little additional movement. Surveying must continue until the permanent structure is completed up to the adjacent alley grade.

We also recommend that the existing conditions along the public right of way and the adjacent private properties be photo-documented prior to commencing on any earthworks at the site.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 STRUCTURAL FILL

Aggregate structural fill, where needed, should consist of imported, well-graded, granular material such as WSDOT Gravel Borrow (*WSDOT Standards and Specifications,* 2023, Section 9-03.14(1)), City of Seattle Type 17, or an approved equivalent. Based on the lack of groundwater anticipated in the excavation, recycled crushed concrete may also be considered as structural fill, except in the City's right-of-way.

Structural fill should be moisture conditioned to near its optimum moisture content, placed in loose, horizontal lifts less than about 12 inches in thickness, and systematically compacted to a dense and relatively unyielding condition. The adequacy of the compaction should be verified by PanGEO. If density tests will be performed, the test results should indicate at least 95 percent of the maximum dry density, as determined using test method ASTM D-1557. Within 5 feet of basement and shoring walls, the backfill should be compacted to 90 percent of the maximum dry density.

Depending on the type of compaction equipment used and depending on the type of fill material, it may be necessary to decrease the thickness of each lift to achieve adequate compaction. PanGEO can provide additional recommendations regarding structural fill and compaction during construction.

8.2 MATERIAL REUSE

Within the anticipated excavation depth, the soils underlying the site are anticipated to consist of highly moisture sensitive glacially consolidated soils and will become disturbed and soft when exposed to inclement weather conditions. In our opinion, the site soils should not be used as structural fill.

8.3 GENERAL TEMPORARY EROSION CONTROL CONSIDERATIONS

In our opinion, the potential for erosion at the site can be adequately mitigated by employing best management practices (BMPs). During construction, erosion control should include measures for reducing concentrated surface runoff and for reducing the potential of off-site sediment transport by protecting disturbed or exposed surfaces. The construction stormwater control plan (CSC) plan should include the following:

- Where practical, maintain vegetation buffers around cleared areas.
- The ground surface within the construction area should be graded to prevent ponding of water and to prevent runoff from reaching site slopes
- Adequately cover soil stockpiles with plastic sheeting.
- Divert water away from the top of slopes.
- Use silt fences, straw bales, and/or straw waddles around the site perimeter.
- If possible, stage construction such that the amount of exposed soil and exposure time is minimized.

The erosion control measures should be inspected on a regular basis to verify they are functioning as intended.

8.4 SURFACE DRAINAGE CONSIDERATIONS

Adequate drainage provisions are imperative to improve the performance of the structures and other site improvements. We recommend both short term and long-term drainage measures be incorporated into the project design and construction. Surface runoff can be controlled during construction by careful grading practices, as described above in Section 8.3. All collected water should be directed under control to a positive and permanent discharge system.

For post-construction, erosion can be controlled by revegetating disturbed areas and controlling surface water runoff. Permanent control of surface water should be incorporated in the final grading design. Adequate surface gradients and drainage systems should be incorporated into the design such that surface runoff is directed away from structures and away from any site slopes. All collected water from surface runoff and from downspouts should be routed into an appropriate discharge point and should *not* drain into retaining wall or footing drain systems.

8.5 WET EARTHWORK RECOMMENDATIONS

It is our opinion that construction of the project can be accomplished during the wet season. However, performing earthwork activities during the wet season is anticipated to be more costly than during dry weather conditions. General recommendations relative to earthwork performed in wet weather or in wet conditions are presented below:

- All footing surfaces should be protected against inclement weather unless the footings can be poured immediately after the subgrade is exposed. It is the contractor's responsibility to protect the footing subgrade from disturbance. If needed, one option is to place 2 to 3 inches of lean-mix concrete or 4 to 6 inches of crushed rock or recycled concrete on the exposed foundation subgrade as soon as the subgrade is exposed. Alternatively, the footing pour may be made immediately after the footing excavation is completed. This will require the reinforcing steel to be prefabricated and lowered into the footing excavation once the excavation is completed.

- Earthwork should be performed in small areas to minimize subgrade exposure to wet weather. Excavation or the removal of unsuitable soil should be followed promptly by the placement and compaction of clean structural fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance.

- During wet weather, the allowable fines content of the structural fill should be reduced to no more than 5 percent by weight based on the portion passing the ¾-inch sieve. The fines should be non-plastic.

- Geotextile silt fences should be strategically located to control erosion and the movement of soil. Erosion control measures should be installed along all the property boundaries.

9.0 LIMITATIONS

We have prepared this report for the Low Income Housing Institute and project design team. Recommendations contained in this report are based on a site reconnaissance, a subsurface exploration program, review of pertinent subsurface information, and our understanding of the project. The study was performed using a mutually agreed-upon scope of work.

Variations in soil conditions may exist between the explorations and the actual conditions underlying the site. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at the site that are different from those described in this report, we should be notified immediately to review the applicability of our recommendations. Additionally, we should also be notified to review the applicability of our recommendations if there are any changes in the project scope.

The scope of our work does not include services related to construction safety precautions. Our recommendations are not intended to direct the contractors' methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Additionally, the scope of our work specifically excludes the assessment of environmental characteristics, particularly those involving hazardous substances. We are not mold consultants nor are our recommendations to be interpreted as preventative of mold development. A mold specialist should be consulted for all mold-related issues.

This report may be used only by the client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both off and on-site), or other factors including advances in our understanding of applied science, may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 24 months from its issuance. PanGEO should be notified if the project is delayed by more than 24 months from the date of this report so that we may review the applicability of our conclusions considering the time lapse.

It is the client's responsibility to see that all parties to this project, including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk. Any party other than the client who wishes to use this report shall notify PanGEO of such intended use and for permission to copy this report. Based on the intended use of the report, PanGEO may require that additional work be performed and that an updated report be reissued. Noncompliance with any of these requirements will release PanGEO from any liability resulting from the use of this report.

Within the limitation of scope, schedule and budget, PanGEO engages in the practice of geotechnical engineering and endeavors to perform its services in accordance with generally accepted professional principles and practices at the time the report or its contents were prepared. No warranty, express or implied, is made.

We appreciate the opportunity to be of service to you on this project. Please feel free to contact our office with any questions you have regarding our study, this report, or any geotechnical engineering related project issues.

Sincerely,

PanGEO Inc.

SIEW LAN TAN
30342
September 6, 2023

Siew L. Tan, P.E.
Principal Geotechnical Engineer

10.0 REFERENCES

Applied Geotechnology, Inc, 1993. *Seattle Chinatown International District PDA.* Test Borings AG1 and AG2, dated March 8, 1993.

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Troost, K.G., Booth, D. B., Wisher, A. P., Shimmel, S. A., 2005, *The Geologic Map of Seattle-A Progress Report, Seattle, Washington – U. S. Geological Survey Open File Report 2005-1252, scale 1:24,000.*

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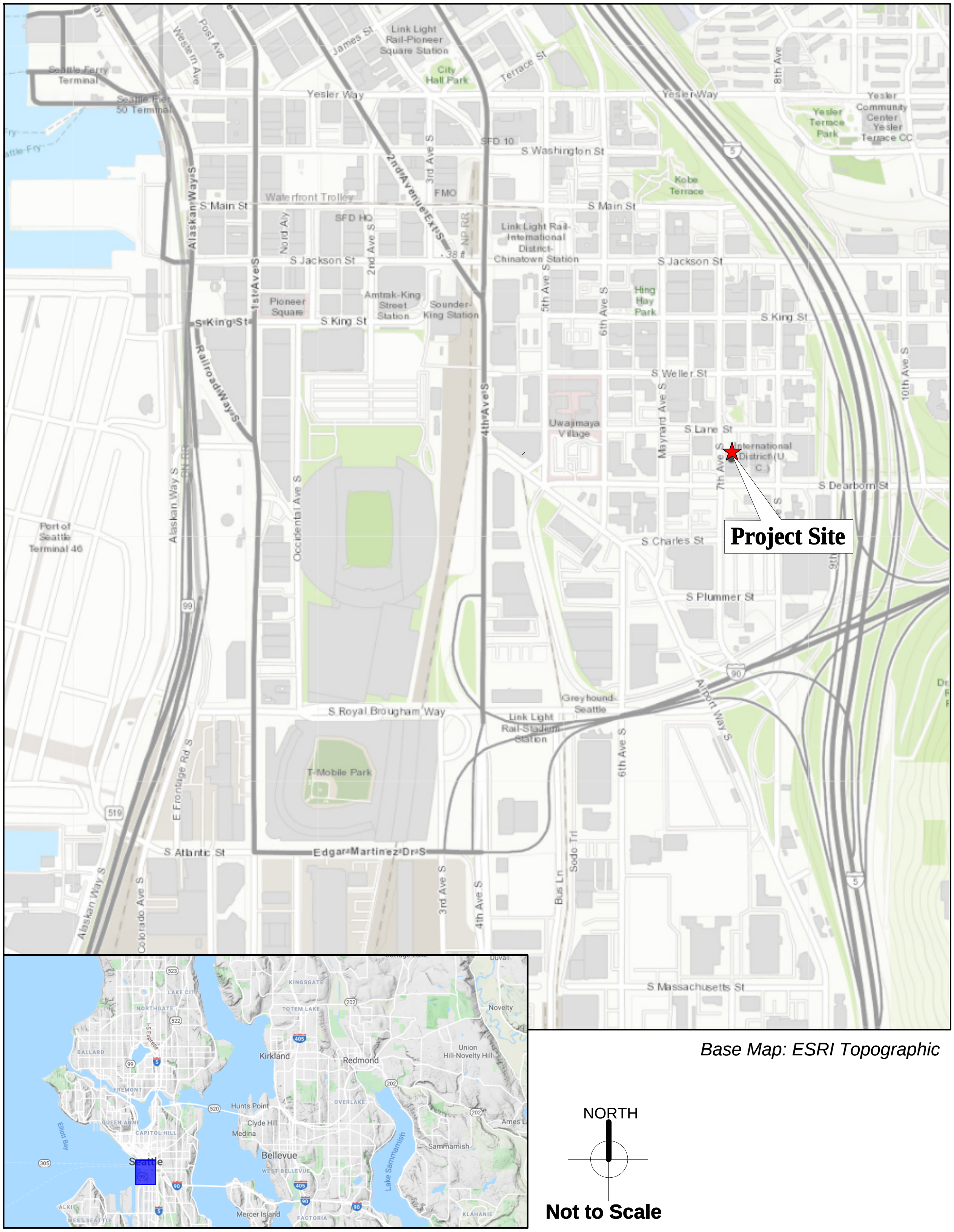

Base Map: ESRI Topographic

Not to Scale

23-253 Fig 1 - Vicinity Map.grf 8/28/23 (4:50:08) SPS

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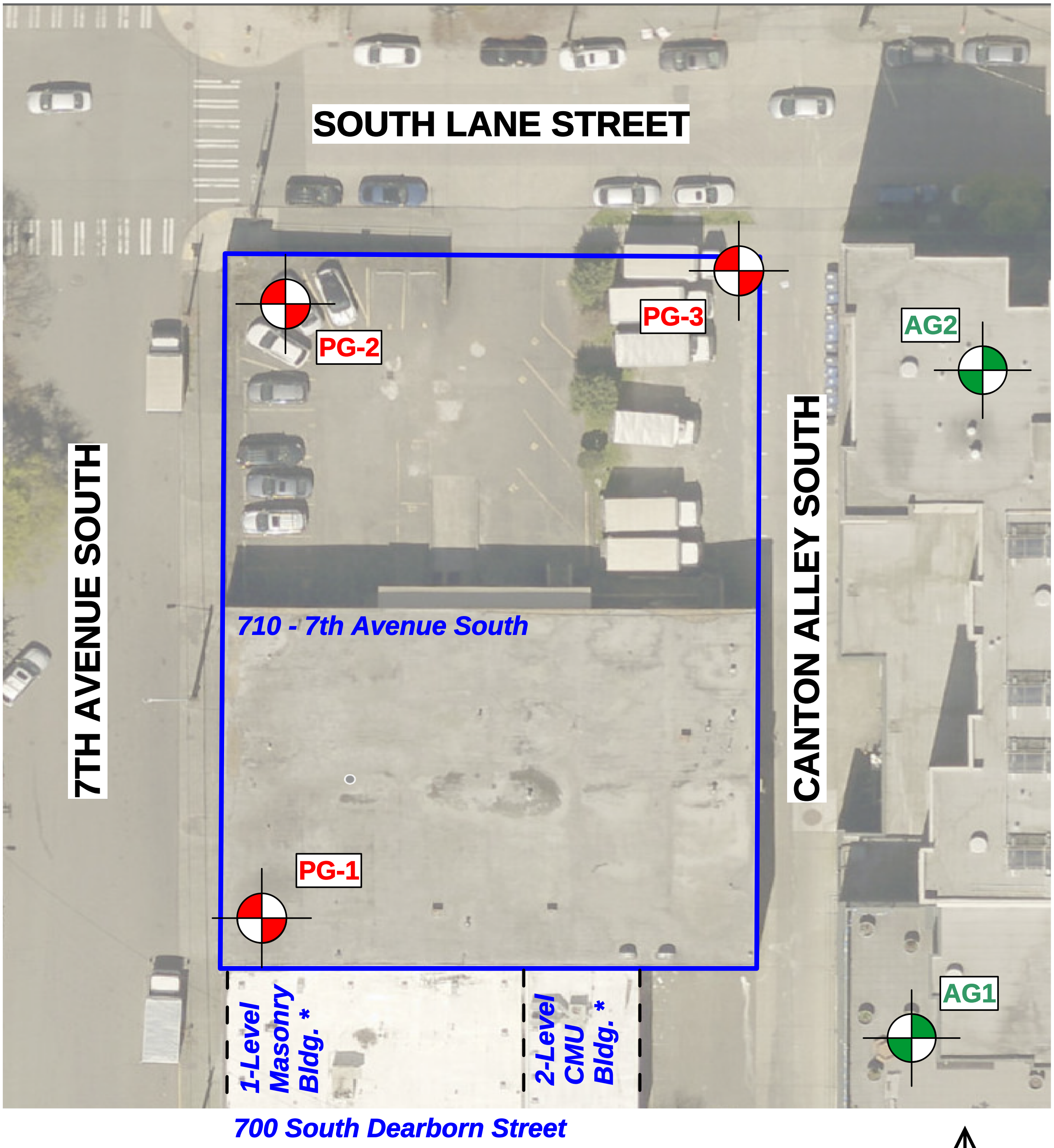
710 - 7th Avenue South
Seattle, Washington

VICINITY MAP

Project No. **23-253**

Figure No. **1**



Legend:

Approx. Test Boring Location
PanGEO, Inc., August 2023

Previous Test Boring Location
AGI, December 1992

Approximate Parcel Boundary

N

Approx. Scale
1" = 40'

**Adjacent footing elevations to south to be confirmed by contractor prior to final design*
Base map modified from 2021 Aerial, King County iMap, Accessed August 28, 2023

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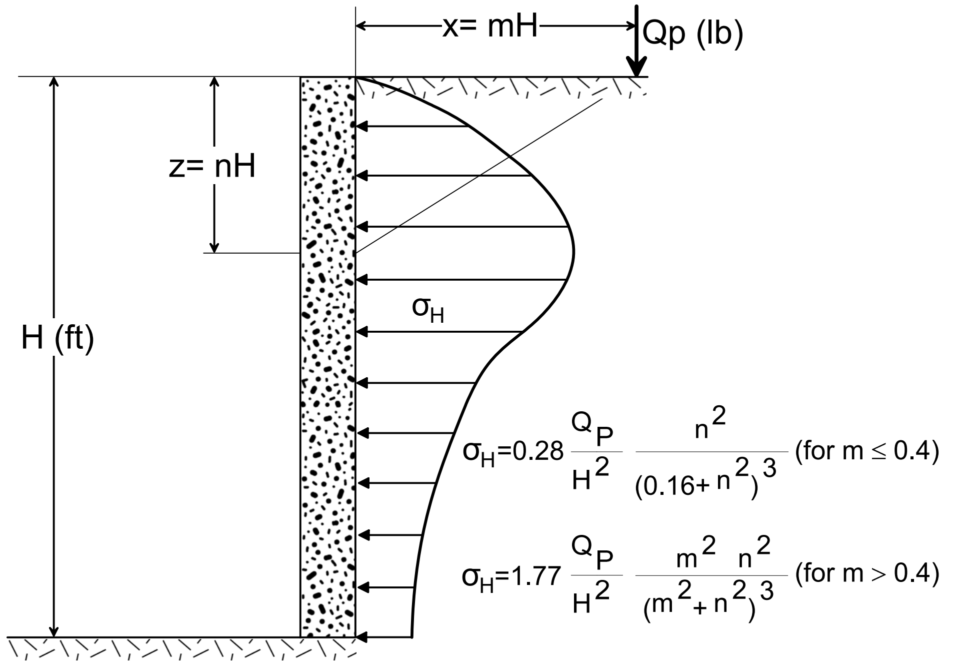
Proposed Development
710 - 7th Avenue South
Seattle, Washington

SITE AND EXPLORATION PLAN

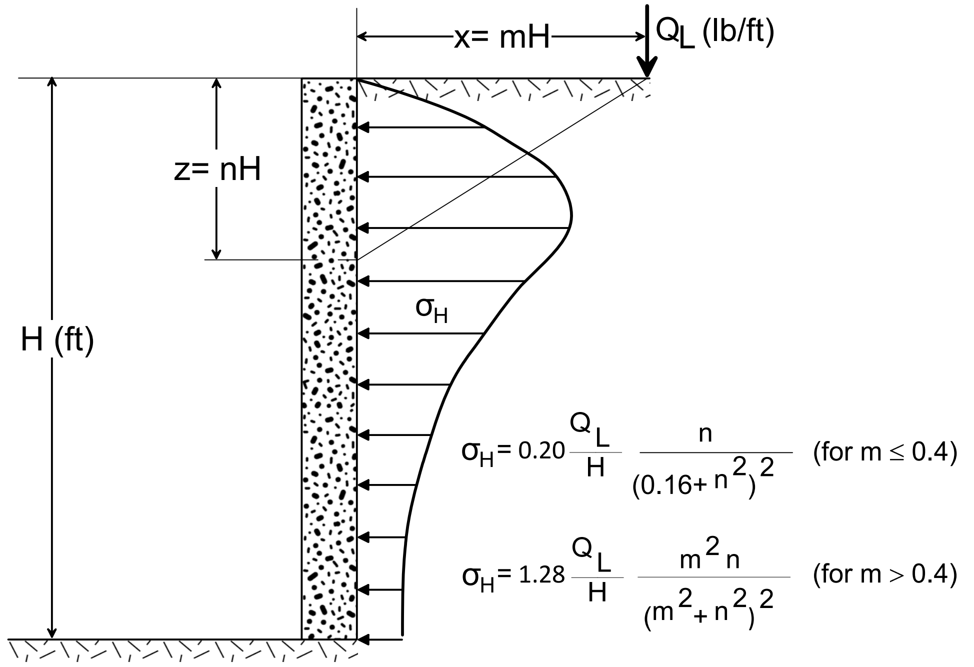
Project No. **23-253**

Figure No. **2**

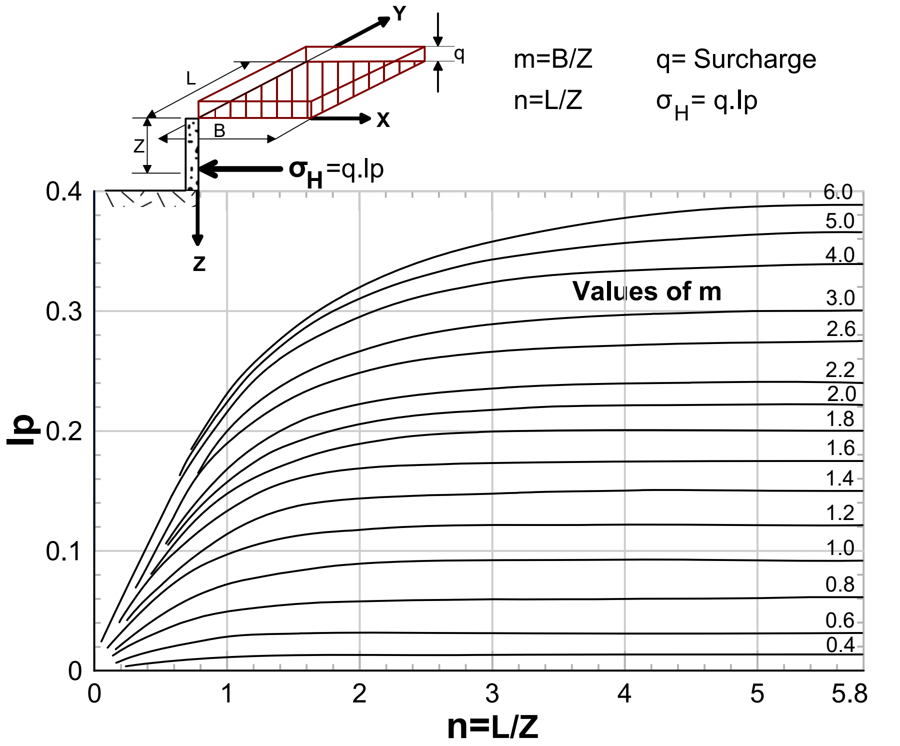
23-253 - Site and Exploration Plan.gpj 8/28/23 (5:50:07) SPS



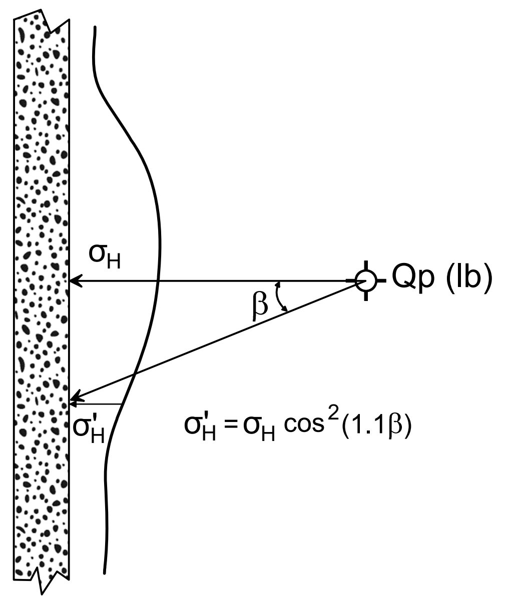
A-1) Lateral Pressure Due to Point Load- Elevation View



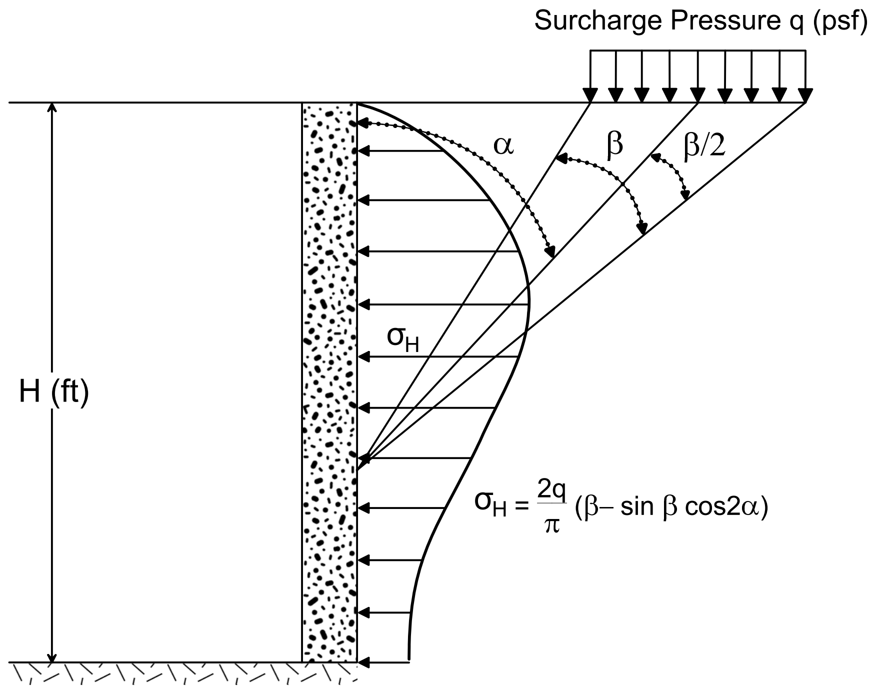
B) Lateral Pressure Due to Line Load-Parallel to the Wall



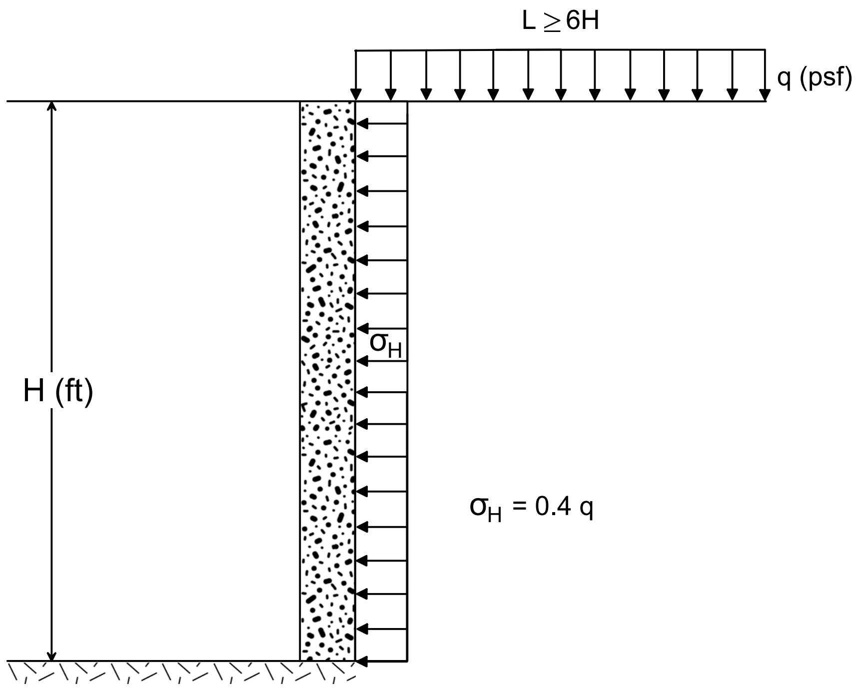
D) Lateral Pressure Due to Adjacent Footing



A-2) Lateral Pressure Due to Point Load- Plan View



C) Lateral Pressure Due to Strip Load-Perpendicular to the Wall



E) Lateral Pressure Due to Uniform Surcharge.
(For L ≤ 6H Use Chart D Above)

* σ_H in psf.

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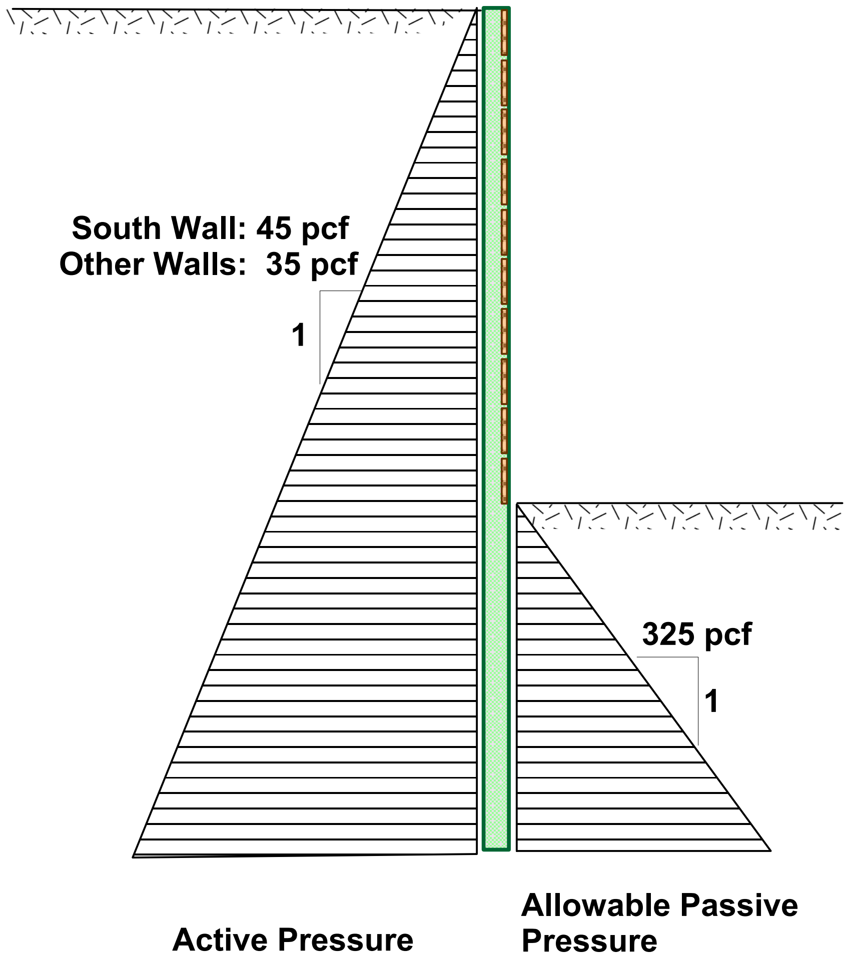
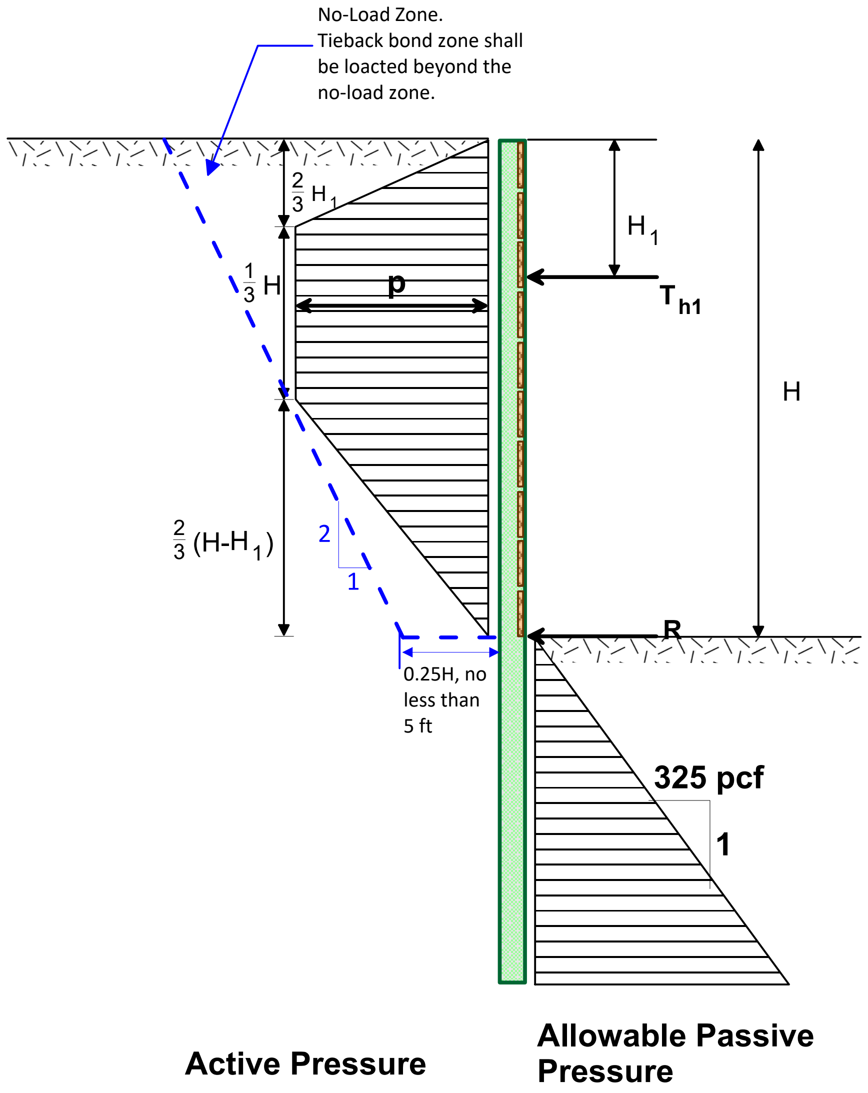
Seattle, Washington

SURCHARGE LOADING ON WALLS

Project No.	Figure No.
23-241	4



a) Cantilever Wall

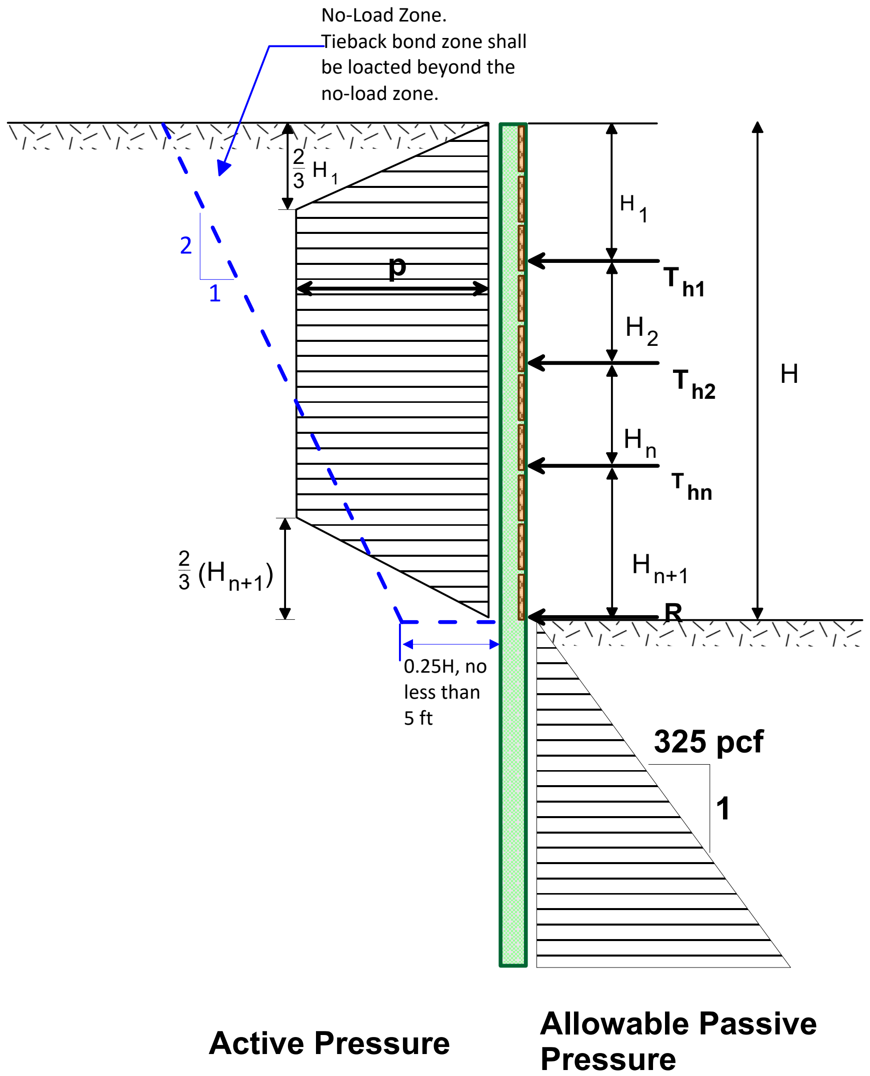


Total Load= $0.65\ K_A\ \gamma\ H^2$

= $30H^2$ for South Wall, $23H^2$ for other walls

$$p= \frac{\text{Total Load}}{\frac{2}{3}H}$$

b) Walls with One Level of Ground Anchors



Total Load= $0.65\ K_A\ \gamma\ H^2$

= $30H^2$ for South Wall, $23H^2$ for other walls

$$p= \frac{\text{Total Load}}{H - \frac{1}{3}H_1 - \frac{1}{3}H_{n+1}}$$

c) Walls with Multiple Levels of Ground Anchors

Legend:

H_1 = Distance from the ground surface to uppermost ground anchor.

H_{n+1}= Distance from the base of excavation to lowermost ground anchor.

T_{hi}= Horizontal load in ground anchor i.

R= Reaction force to be resisted by subgrade
(i.e., below the base of excavation).

p= maximum ordinate of diagram.

K_A= Active earth pressure coefficient.

γ = Soil unit weight (pcf).

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Apparent Earth Pressures Temporary Soldier Pile Walls

Project No.	Figure No.
23-253	4

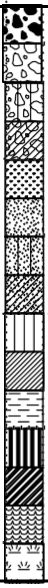
APPENDIX A

SUMMARY TEST BORING LOGS

RELATIVE DENSITY / CONSISTENCY

SAND / GRAVEL			SILT / CLAY		
Density	SPT N-values	Approx. Relative Density (%)	Consistency	SPT N-values	Approx. Undrained Shear Strength (psf)
Very Loose	<4	<15	Very Soft	<2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Med. Dense	10 to 30	35 - 65	Med. Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	>50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	>30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM



MAJOR DIVISIONS		GROUP DESCRIPTIONS	
Gravel 50% or more of the coarse fraction retained on the #4 sieve. Use dual symbols (eg. GP-GM) for 5% to 12% fines.	GRAVEL (<5% fines)	GW	Well-graded GRAVEL
		GP	Poorly-graded GRAVEL
	GRAVEL (>12% fines)	GM	Silty GRAVEL
		GC	Clayey GRAVEL
Sand 50% or more of the coarse fraction passing the #4 sieve. Use dual symbols (eg. SP-SM) for 5% to 12% fines.	SAND (<5% fines)	SW	Well-graded SAND
		SP	Poorly-graded SAND
	SAND (>12% fines)	SM	Silty SAND
		SC	Clayey SAND
Silt and Clay 50%or more passing #200 sieve	Liquid Limit < 50	ML	SILT
		CL	Lean CLAY
		OL	Organic SILT or CLAY
	Liquid Limit > 50	MH	Elastic SILT
		CH	Fat CLAY
		OH	Organic SILT or CLAY
Highly Organic Soils		PT	PEAT

Notes:

1. Soil exploration logs contain material descriptions based on visual observation and field tests using a system modified from the Uniform Soil Classification System (USCS). Where necessary laboratory tests have been conducted (as noted in the "Other Tests" column), unit descriptions may include a classification. Please refer to the discussions in the report text for a more complete description of the subsurface conditions.
2. The graphic symbols given above are not inclusive of all symbols that may appear on the borehole logs. Other symbols may be used where field observations indicated mixed soil constituents or dual constituent materials.

DESCRIPTIONS OF SOIL STRUCTURES

Layered:	Units of material distinguished by color and/or composition from material units above and below
Laminated:	Layers of soil typically 0.05 to 1mm thick, max. 1 cm
Lens:	Layer of soil that pinches out laterally
Interlayered:	Alternating layers of differing soil material
Pocket:	Erratic, discontinuous deposit of limited extent
Homogeneous:	Soil with uniform color and composition throughout
Fissured:	Breaks along defined planes
Slickensided:	Fracture planes that are polished or glossy
Blocky:	Angular soil lumps that resist breakdown
Disrupted:	Soil that is broken and mixed
Scattered:	Less than one per foot
Numerous:	More than one per foot
BCN:	Angle between bedding plane and a plane normal to core axis

COMPONENT DEFINITIONS

COMPONENT	SIZE / SIEVE RANGE	COMPONENT	SIZE / SIEVE RANGE
Boulder:	> 12 inches	Sand	
Cobbles:	3 to 12 inches	Coarse Sand:	#4 to #10 sieve (4.5 to 2.0mm)
Gravel		Medium Sand:	#10 to #40 sieve (2.0 to 0.42 mm)
Coarse Gravel:	3 to 3/4 inches	Fine Sand:	#40 to #200 sieve (0.42 to 0.074 mm)
Fine Gravel:	3/4 inches to #4 sieve	Silt	0.074 to 0.002 mm
		Clay	<0.002 mm

TEST SYMBOLS

for In Situ and Laboratory Tests listed in "Other Tests" column.

ATT	Atterberg Limit Test
Comp	Compaction Tests
Con	Consolidation
DD	Dry Density
DS	Direct Shear
%F	Fines Content
GS	Grain Size
Perm	Permeability
PP	Pocket Penetrometer
R	R-value
SG	Specific Gravity
TV	Torvane
TXC	Triaxial Compression
UCC	Unconfined Compression

SYMBOLS

Sample/In Situ test types and intervals

2-inch OD Split Spoon, SPT (140-lb. hammer, 30" drop)

3.25-inch OD Spilt Spoon (300-lb hammer, 30" drop)

Non-standard penetration test (see boring log for details)

Thin wall (Shelby) tube

Grab

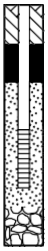
Rock core

Vane Shear

MONITORING WELL

▽ Groundwater Level at time of drilling (ATD)

▼ Static Groundwater Level



Cement / Concrete Seal

Bentonite grout / seal

Silica sand backfill

Slotted tip

Slough

Bottom of Boring

MOISTURE CONTENT

Dry	Dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water

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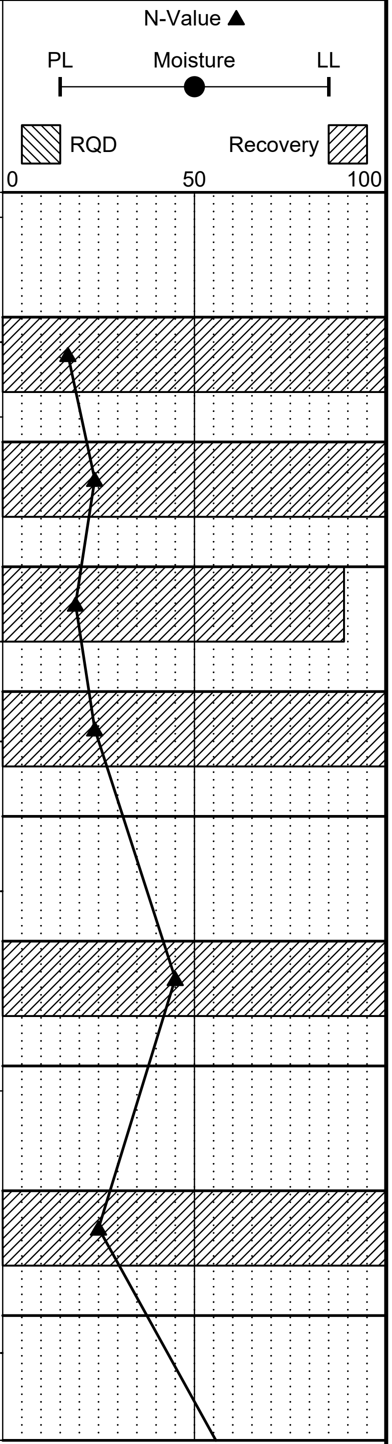
Terms and Symbols for Boring and Test Pit Logs

Figure A-1

Project:	LIHI - 7th Ave LLC	Surface Elevation:	~63 ft
Job Number:	23-253	Top of Casing Elev.:	N/A
Location:	710 - 7th Avenue South	Drilling Method:	CAT track drill rig, hollow stem auger
Coordinates:	Northing: 47.596148, Easting: -122.32362	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						2 inches ASPHALT above 4 inches crushed surface base coarse.
						FILL - [Hf] Medium dense, gray, silty SAND with gravel, disturbed texture, non-plastic, moist.
2.5	S-1	☒	5 7 10			Medium dense, brown with orange, SILT with sand and gravel, disturbed texture, non-plastic, moist; scattered iron oxide staining.
5.0	S-2	☒	8 10 14			**PRE-OLYMPIA COARSE GRAINED DEPOSITS - [Qpoc]** Medium dense, orangish-brown to gray-brown, silty fine to medium SAND with gravel, non-plastic, moist; minor iron oxide staining.
7.5	S-3	☒	11 10 9			*--silt and gravel content decreases.*
						Medium dense, gray-brown, sandy SILT, trace gravel, non-plastic, moist; minor iron oxide staining observed at 11-foot contact.
10.0	S-4	☒	5 9 15			Medium dense, gray-brown, silty SAND with gravel, non-plastic, moist; minor iron oxide staining.
12.5						
						Dense, orangish-brown, poorly-graded SAND with silt, non-plastic, moist to wet.
15.0	S-5	☒	12 21 24		▽	*--perched groundwater observed between approximately 16 to 18 feet.*
17.5						
						PRE-OLYMPIA FINE GRAINED DEPOSITS - [Qpof] Very stiff, gray, silty CLAY, low plasticity, moist.
20.0	S-6	☒	5 10 15			*--occasional slickenslide features and coarse sand lenses encountered.*
22.5						
						Hard, gray, fat CLAY, high plasticity, moist to wet.
25.0						*(continued on next page).*



Completion Depth:	26.5ft
Date Borehole Started:	8/23/23
Date Borehole Completed:	8/23/23
Logged By:	L. Dunham
Drilling Company:	Geologic Drill

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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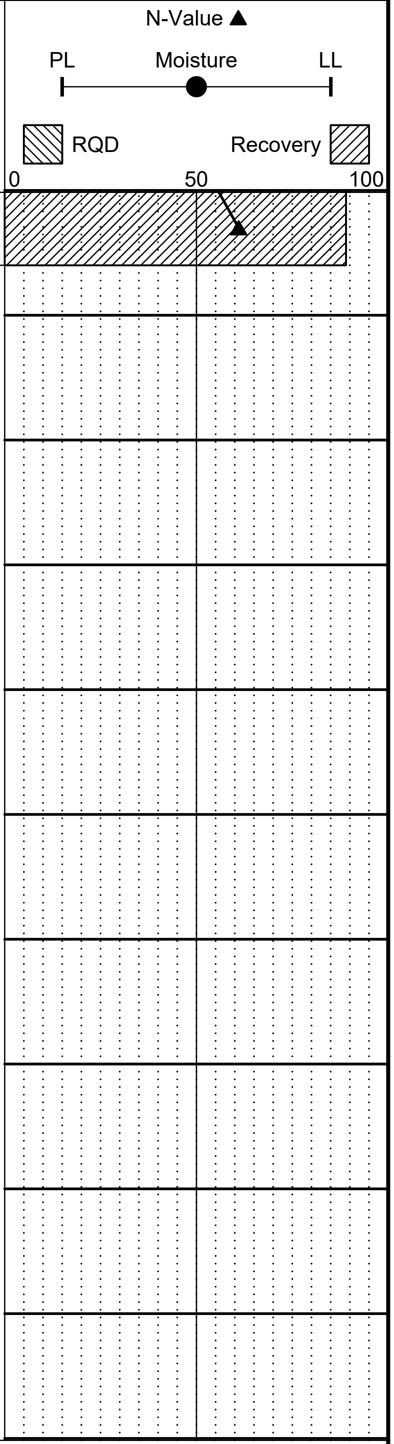
LOG OF TEST BORING PG-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	LIHI - 7th Ave LLC	Surface Elevation:	~63 ft
Job Number:	23-253	Top of Casing Elev.:	N/A
Location:	710 - 7th Avenue South	Drilling Method:	CAT track drill rig, hollow stem auger
Coordinates:	Northing: 47.596148, Easting: -122.32362	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
25.0	S-7	☒	12 16 45		▨	*--three-inch perched layer of very moist to wet, gray clayey SAND.*
27.5						Boring terminated at about 26.5 feet below grade. Groundwater was observed minor perched ground water at approximately 16 to 18 feet and 25.5 to 26 below ground surface during drilling.
30.0						
32.5						
35.0						
37.5						
40.0						
42.5						
45.0						
47.5						
50.0						



Completion Depth:	26.5ft
Date Borehole Started:	8/23/23
Date Borehole Completed:	8/23/23
Logged By:	L. Dunham
Drilling Company:	Geologic Drill

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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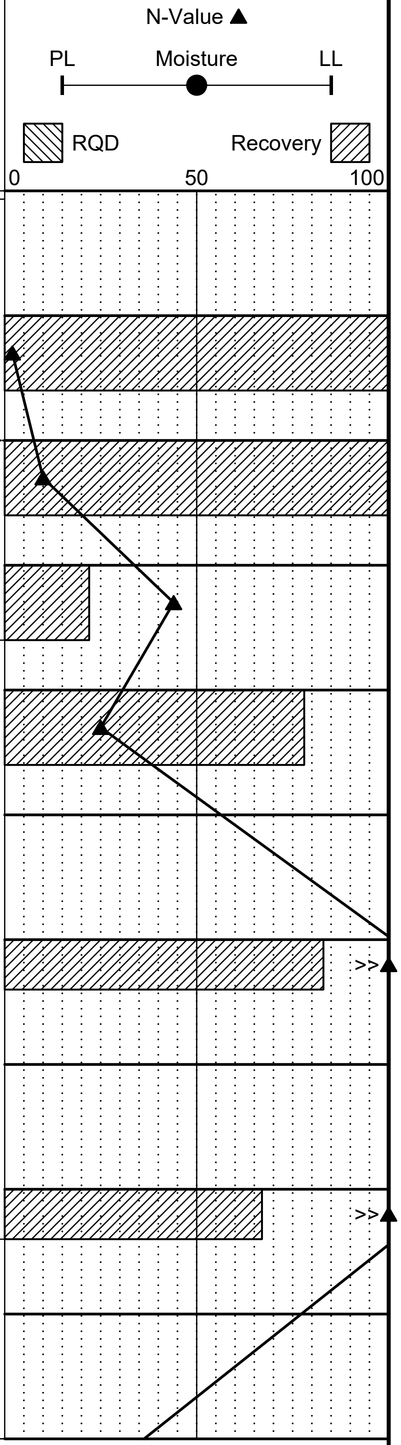
LOG OF TEST BORING PG-1

Figure A-2

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	LIHI - 7th Ave LLC	Surface Elevation:	~74 ft
Job Number:	23-253	Top of Casing Elev.:	N/A
Location:	710 - 7th Avenue South	Drilling Method:	CAT track drill rig, hollow stem auger
Coordinates:	Northing: 47.596574, Easting: -122.323626	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Asphalt - 2 inches.
						FILL - [Hf] Very loose, light brown to gray, sandy SILT with gravel, disturbed texture, non-plastic, moist.
2.5	S-1	☒	1 1 1			*--becomes mottled gray with blue.*
5.0	S-2	☒	4 4 6			Loose to medium dense, mottled with orange, SILT with sand and gravel, disturbed texture, non-plastic, moist; iron oxide staining.
7.5	S-3	☒	9 16 28			*--becomes dense at approximately 9 feet; minimal returns; likely inflated blow counts.*
						PRE-OLYMPIA COARSE GRAINED DEPOSITS - [Qpoc] Medium dense, brown to gray-brown, silty medium to coarse SAND, trace gravel, non-plastic, moist.
10.0	S-4	☒	7 11 14			
12.5						
15.0	S-5	☒	33 50/6			*--becomes very dense at 15 feet.*
17.5						
						--becomes gray, with gravel, diamict (till-like) texture.
20.0	S-6	☒	18 50/6			
						PRE-OLYMPIA FINE GRAINED DEPOSITS - [Qpof] Hard to very stiff, gray, fat CLAY, high plasticity, moist.
22.5						
25.0						*(continued on next page).*



Completion Depth:	41.5ft
Date Borehole Started:	8/23/23
Date Borehole Completed:	8/23/23
Logged By:	L. Dunham
Drilling Company:	Geologic Drill

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

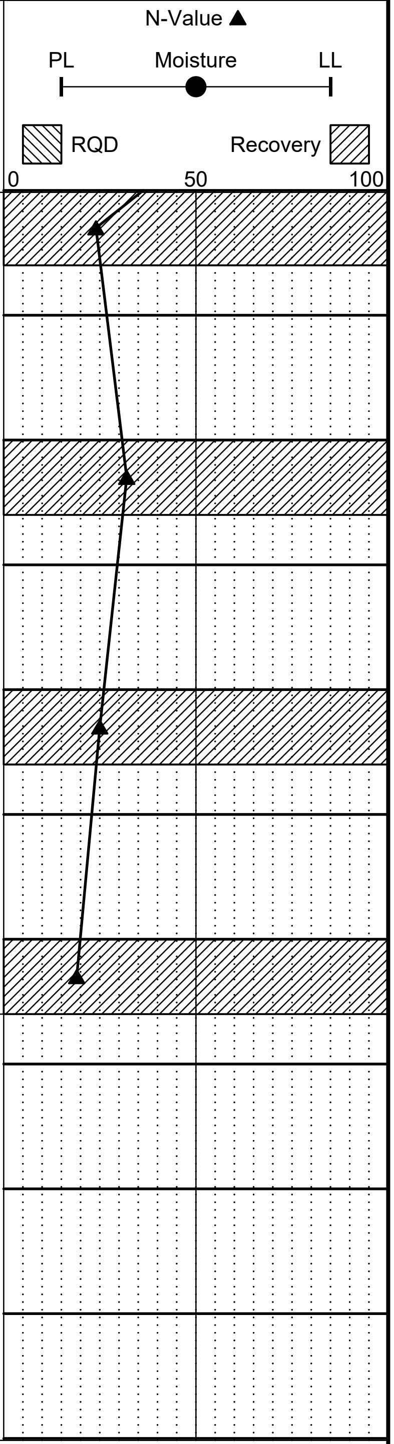
LOG OF TEST BORING PG-2

Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project:	LIHI - 7th Ave LLC	Surface Elevation:	~74 ft
Job Number:	23-253	Top of Casing Elev.:	N/A
Location:	710 - 7th Avenue South	Drilling Method:	CAT track drill rig, hollow stem auger
Coordinates:	Northing: 47.596574, Easting: -122.323626	Sampling Method:	SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
25.0	S-7	☒	8 10 14		▨	Very stiff, gray, silty CLAY, massive texture, high plasticity, moist.
27.5					▨	
30.0	S-8	☒	25 17 15		▨	*--becomes hard, occasional gravels encountered.*
32.5					▨	
35.0	S-9	☒	9 11 14		▨	*--becomes very stiff.*
37.5					▨	
40.0	S-10	☒	6 8 11		▨	
42.5						Boring terminated at about 41.5 feet below grade. Groundwater or groundwater seepage was not observed during drilling.
45.0						
47.5						
50.0						



Completion Depth:	41.5ft
Date Borehole Started:	8/23/23
Date Borehole Completed:	8/23/23
Logged By:	L. Dunham
Drilling Company:	Geologic Drill

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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LOG OF TEST BORING PG-2

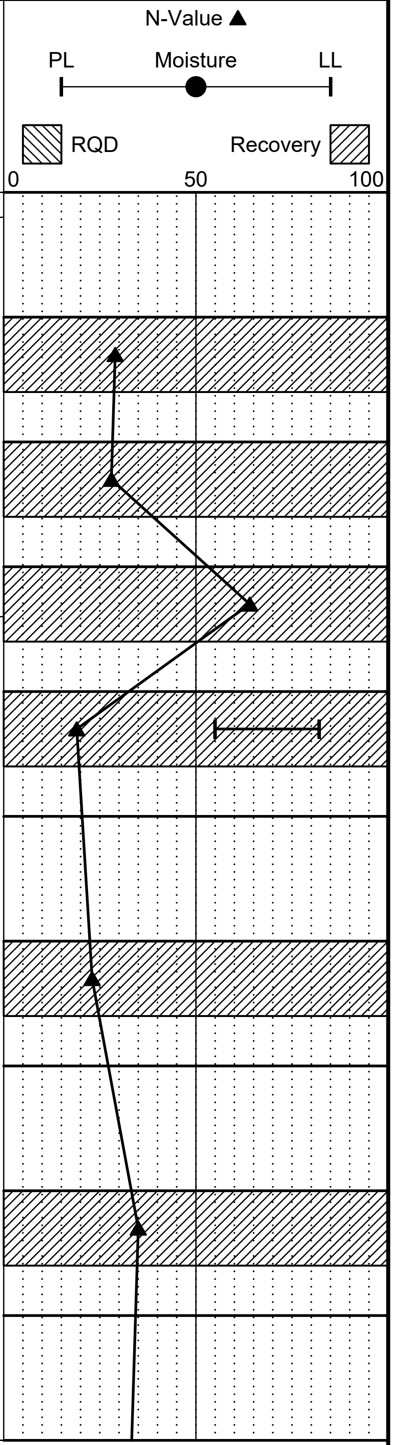
Figure A-3

The stratification lines represent approximate boundaries. The transition may be gradual.

Project: LIHI - 7th Ave LLC
Job Number: 23-253
Location: 710 - 7th Avenue South
Coordinates: Northing: 47.596574, Easting: -122.32321

Surface Elevation: ~74 ft
Top of Casing Elev.: N/A
Drilling Method: CAT track drill rig, hollow stem auger
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
0.0						Approximately 6 inches of crushed gravel parking surface. **PRE-OLYMPIA COARSE GRAINED DEPOSITS - [Qpoc]** Medium dense, silty fine to medium SAND with gravel, diamict (till-like) texture, non-plastic, moist.
2.5	S-1	☒	6 11 18			
5.0	S-2	☒	12 14 14			*--hard gray silt lens encountered at about 7 feet; increase in gravel content, likely inflated blow counts.*
7.5	S-3	☒	18 45 19			**PRE-OLYMPIA FINE GRAINED DEPOSITS - [Qpof]** Very stiff, gray, fat CLAY, trace gravel, high plasticity, moist.
10.0	S-4	☒	5 7 12	ATT		Sample S-4 : MC = 37%, LL = 82, PL = 55, PI = 27.
12.5						
15.0	S-5	☒	5 10 13			*--occasional slickenslide features and coarse sand lenses encountered.*
17.5						
20.0	S-6	☒	8 14 21			
22.5						*(continued on next page).*
25.0						



Completion Depth: 31.5ft
Date Borehole Started: 8/23/23
Date Borehole Completed: 8/23/23
Logged By: L. Dunham
Drilling Company: Geologic Drill

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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LOG OF TEST BORING PG-3

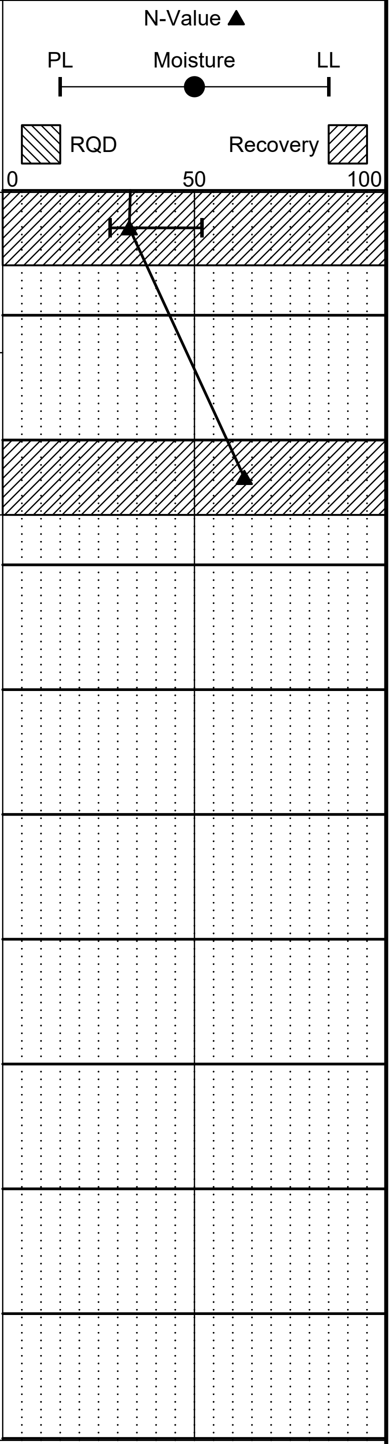
Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

Project: LIHI - 7th Ave LLC
Job Number: 23-253
Location: 710 - 7th Avenue South
Coordinates: Northing: 47.596574, Easting: -122.32321

Surface Elevation: ~74 ft
Top of Casing Elev.: N/A
Drilling Method: CAT track drill rig, hollow stem auger
Sampling Method: SPT

Depth, (ft)	Sample No.	Sample Type	Blows / 6 in.	Other Tests	Symbol	MATERIAL DESCRIPTION
25.0	S-7	☒	9 14 19	ATT	▨	Hard, gray, silty CLAY, trace gravel, low plasticity, moist. Sample S-7 : MC = 27%, LL = 52, PL = 28, PI = 24.
27.5						
						Very dense, brown, interbedded SILT and silty SAND, non-plastic, moist.
30.0	S-8	☒	8 18 45			
32.5						Boring terminated at about 31.5 feet below grade. Groundwater or groundwater seepage was not observed during drilling.
35.0						
37.5						
40.0						
42.5						
45.0						
47.5						
50.0						



Completion Depth: 31.5ft
Date Borehole Started: 8/23/23
Date Borehole Completed: 8/23/23
Logged By: L. Dunham
Drilling Company: Geologic Drill

Remarks: Standard penetration test (SPT) sampler driven with a 140 lb. safety hammer. Hammer operated with a rope and cathead mechanism. Coordinates and elevation are approximate and based on their relative location to known site features. This information is provided for relative information only and is not a substitution for field survey. **Datum: WGS84/NAVD88**

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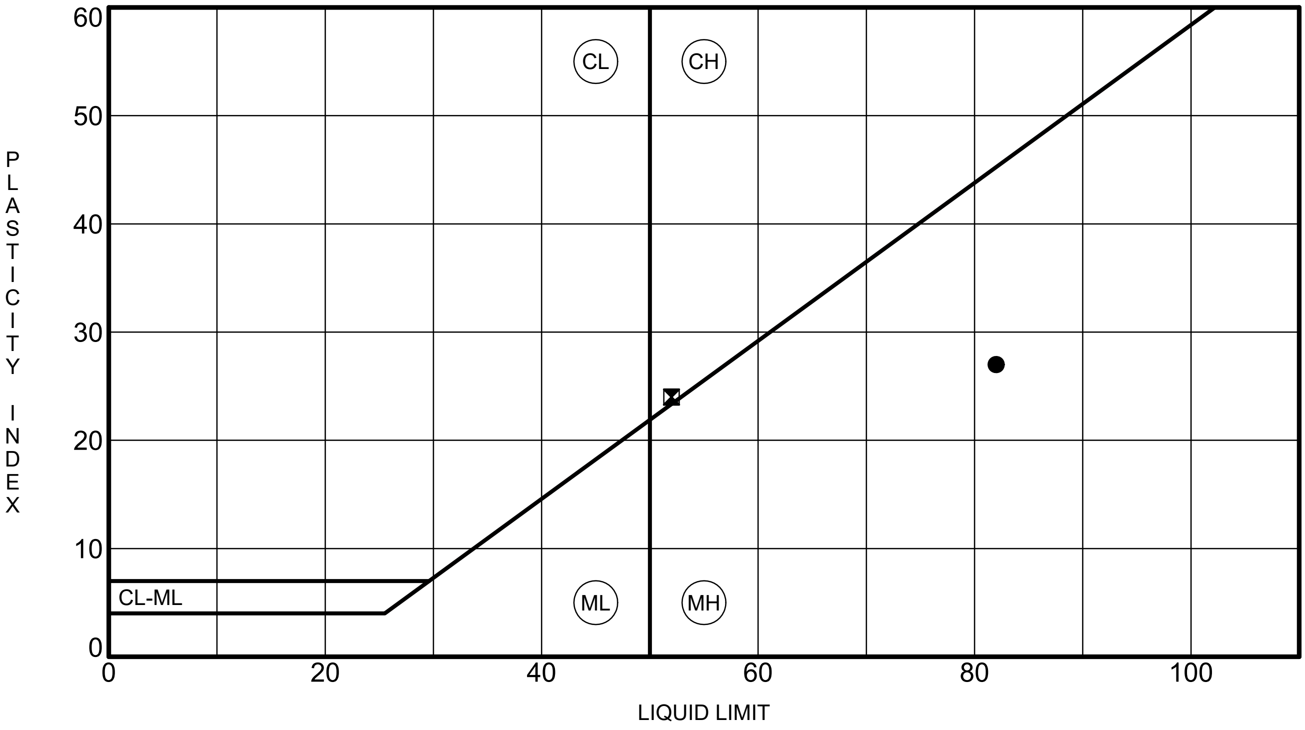
LOG OF TEST BORING PG-3

Figure A-4

The stratification lines represent approximate boundaries. The transition may be gradual.

APPENDIX B

LABORATORY TEST RESULTS



	Specimen Identification		M	LL	PL	PI	Fines	Classification
●	**PG-3**	**10.0**		**82**	**55**	**27**		
⧗	**PG-3**	**25.0**		**52**	**28**	**24**		

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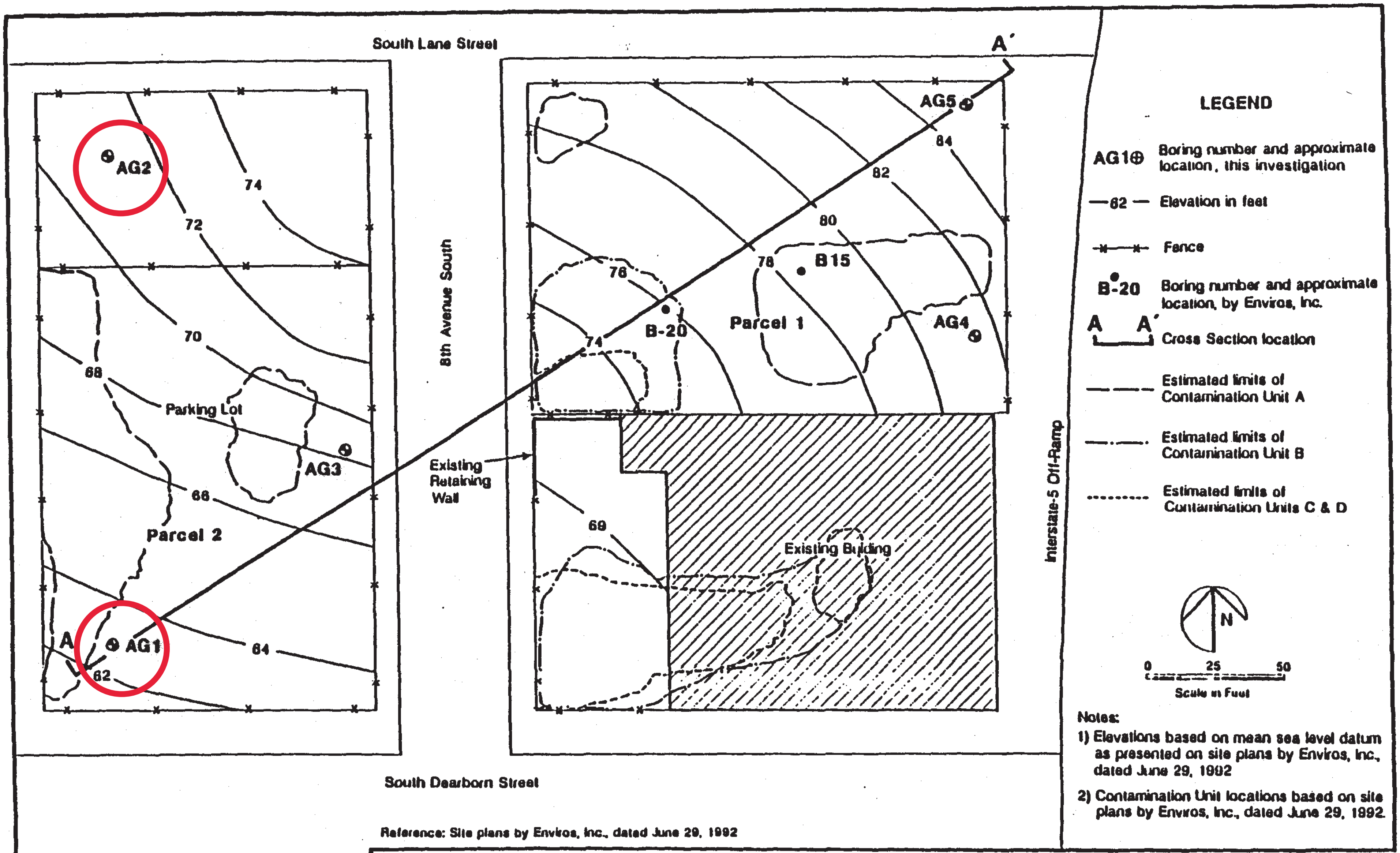
ATTERBERG LIMITS

Project: LIHI - 7th Ave LLC
Job Number: 23-253
Location: 710 - 7th Avenue South

Figure B-1

APPENDIX C

PREVIOUS TEST BORING LOGS



Notes:

1) Elevations based on mean sea level datum as presented on site plans by Enviros, Inc., dated June 29, 1992

2) Contamination Unit locations based on site plans by Enviros, Inc., dated June 29, 1992

Reference: Site plans by Enviros, Inc., dated June 29, 1992

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Site Plan

Seattle Chinatown International District PDA
Seattle, Washington

Figure C-1

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,715 001	SLB	TSM	3 March 1993		

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN NO. 200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN NO. 200 SIEVE	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 CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

LEGEND

SAMPLE

- ■ "Undisturbed"
- ▨ Bulk/Grab
- ☐ Not Recovered
- ▥ Recovered, Not Retained

CONTACT BETWEEN UNITS

- ——— Well Defined Change
- ——— Gradational Change
- — — — Obscure Change
- ▬▬▬ End of Exploration

BLOWS/FOOT

Hammer is 300 pounds with 30-inch drop, unless otherwise noted

S - SPT Sampler (2.0-inch O.D.)
T - Thin Wall Sampler (2.8-inch Sample)
H - Split Barrel Sampler (2.4-inch Sample)

MOISTURE DESCRIPTION

Dry - Considerably less than optimum for compaction
Moist - Near optimum moisture content
Wet - Over optimum moisture content
Saturated - Below water table, in capillary zone, or in perched groundwater

LABORATORY TESTS

Consol - Consolidation
LL - Liquid Limit
PL - Plastic Limit
Gs - Specific Gravity
SA - Size Analysis
TxS - Triaxial Shear
TxP - Triaxial Permeability
Perm - Permeability
Po - Porosity
MD - Moisture/Density
DS - Direct Shear
VS - Vane Shear
Comp - Compaction

UU - Unconsolidated, Undrained
CU - Consolidated, Undrained
CD - Consolidated, Drained

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Soil Classification/Legend

Seattle Chinatown International District PDA
Seattle, Washington

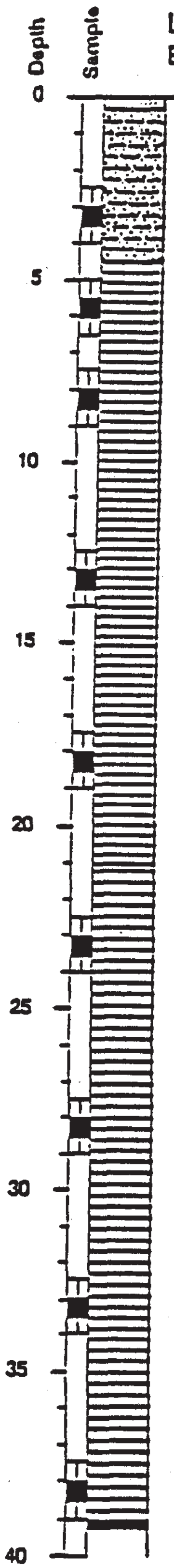
Figure C-2

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15.715.001	SES	TSM	16 Feb. 93		

Equipment Mobile Drill B-61

Land Surface Elevation 63 feet* Date 12/29/92

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows per Foot
MD	13.2	121	18
MD	16.8	107	8
			7
MD	19.6	113	29
MD	10.8	125	46
PI,MD	24.6	101	44
			46
			36
			55



Asphaltic Concrete Pavement (3").
DARK BROWN SILTY SAND (SM) medium dense, moist; with gravel.

GRAY SANDY CLAY (CL) medium stiff, wet; with a trace of gravel.

Becomes stiff, with a trace of organics.

Becomes very stiff, with some gravel.

GRAY CLAY (CH) hard, moist; fissured and slickensided.

Fissured and slickensided.

Fissured and slickensided.

Fissured and slickensided.
Groundwater not encountered during drilling.
Boring backfilled with bentonite chips on 12-29-92.

*Mean Sea Level Datum based on site plan by Enviros, Inc., dated June 29, 1992

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Log of Boring AG1

Seattle Chinatown International District PDA
Seattle, Washington

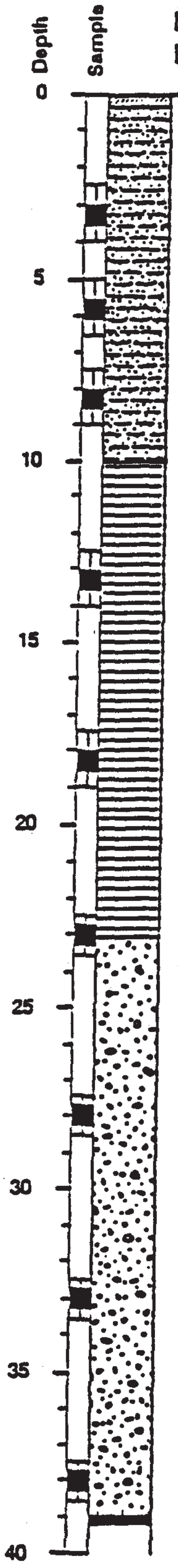
Figure C-3

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,715.001	SES	TSM	16 Feb. 93		

Equipment Mobile Drill B-61

Land Surface Elevation 71 feet Date 12/29/92

Laboratory Tests	Moisture Content (%)	Dry Density (pcf)	Blows per Foot
			33
MD	15.5	118	49
			28
MD	40.3	85	27
CONSOL	30.2	94	
MD	27.5	97	57
MD	28.2	98	50/5"
MD	13.9	119	
MD	6.2	112	50/5"
			50/6"
			50/5"



Asphaltic Concrete Pavement (3").

GRAY BROWN SILTY SAND (SM) medium dense, mosit; fine to coarse grained, with some fine gravel.

GRAY CLAY (CL) very stiff, moist; fissured and slickensided.

Fissured and slickensided.

Fissured and slickensided.

GRAY BROWN SAND (SW) very dense, moist; fine to coarse grained, with some gravel.

Groundwater not encountered during drilling. Boring backfilled with bentonite chips on 12-29-92.

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Log of Boring AG2

Seattle Chinatown International District PDA
Seattle, Washington

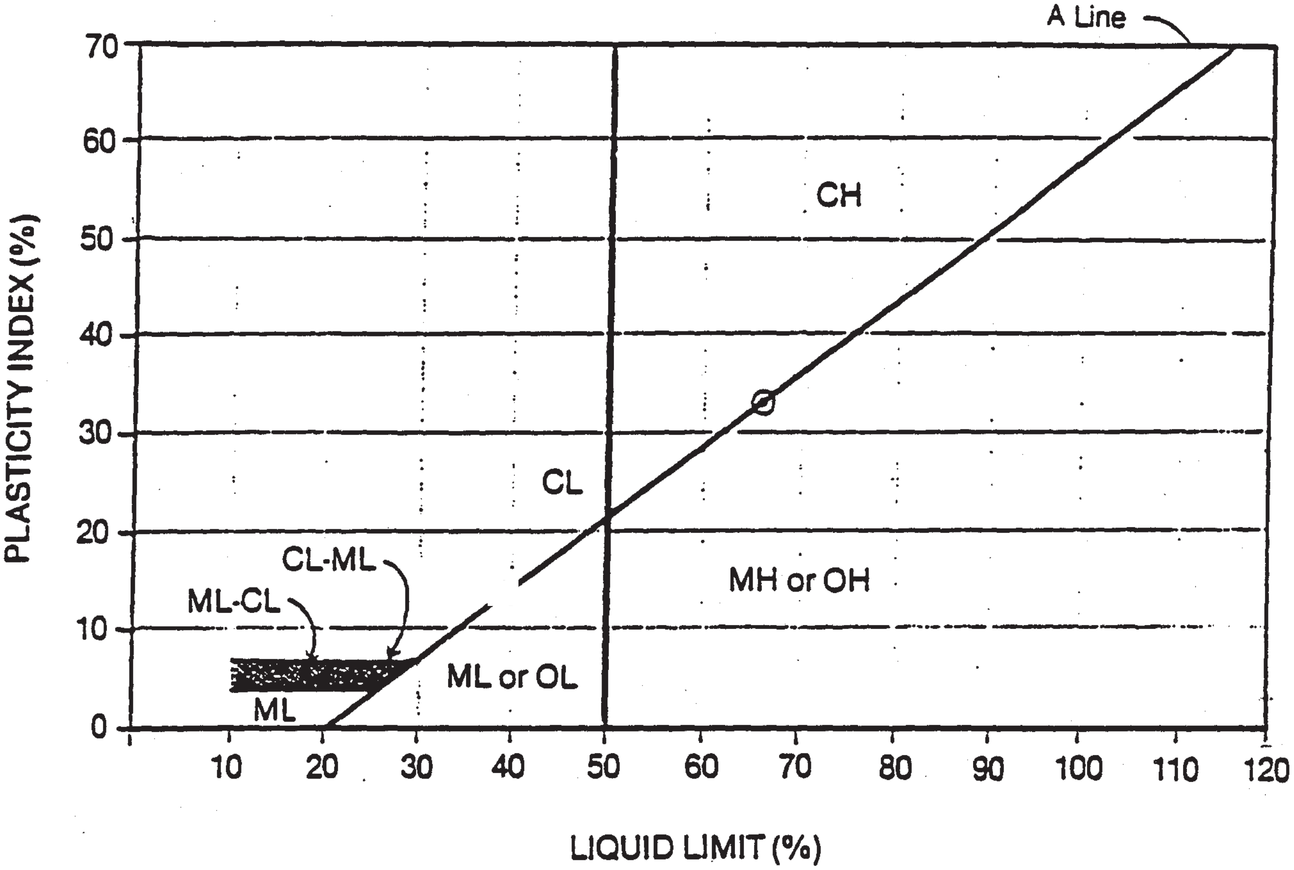
Figure C-4

JOB NUMBER	DRAWN	APPROVED	DATE	REVISED	DATE
15,715.001	SES	TSM	16 Feb. 93		

APPENDIX D

PREVIOUS LABORATORY TEST RESULTS

Reference: ASTM D 423, 424



Symbol	Source	Classification	Natural M.C. (%)	Liquid Limit (%)	Plasticity Index (%)	% Passing #200 Sieve
⊙	AG1 @ 22.5'	Gray Sandy Clay (CH)	24.6	66	33	

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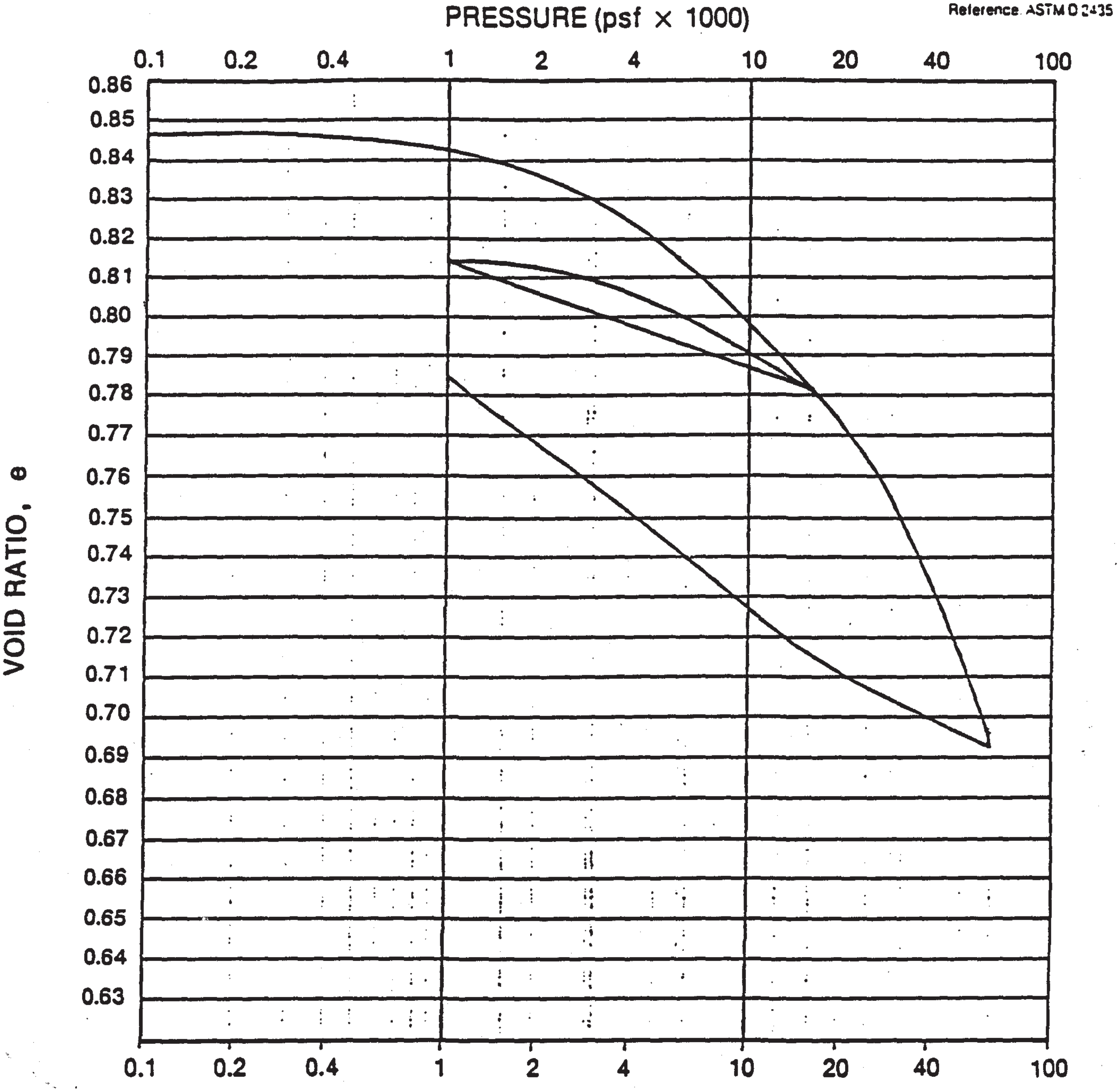
Plasticity Chart

Seattle Chinatown International District PDA
Seattle, Washington

Figure D-1

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
KM	15.715.001	TSM	3 March 1993		



VOID RATIO — PRESSURE CURVE

Type of Specimen	D & M	Condition		Before Test		After Test
Diameter (in.) 2.410	Height (in.) 0.739	Water Content	w_o	30.2 %	w_f	29.9 %
Overburden Press., P_o	1,450 psf	Void Ratio	e_o	0.878	e_f	0.783
Preconsol. Press., P_c	12,000 psf	Saturation	S_o	99.6 %	S_f	100.0 %
Recompression Index, C_{cr}	0.027	Dry Density	γ_d	94 pcf	γ_d	98 pcf
LL ——	PL ——	PI ——		G_s 2.8		
Classification	Gray Clay (CL)	Source		AG2 @ 12.5 feet		

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Consolidation Test Report

Seattle Chinatown International District PDA
Seattle, Washington

Figure D-2

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
KM	15,715.001	TSM	3 March 1993		