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Geotechnical Report
West Bay Drive Northwest Sidewalks Project
Olympia, Washington

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Prepared for

The City of Olympia
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EXECUTIVE SUMMARY

The City of Olympia (City) proposes to construct a new sidewalk along the west side of West Bay Drive NW to improve pedestrian movement and safety along West Bay Drive NW in Olympia, Washington. The addition of the sidewalks to the west of the road will require cutting into the adjacent slopes and building four cast-in-place (CIP) concrete retaining walls (designated as Walls 1 through 4). In the vicinity of the proposed retaining walls, the hillside rises at an approximate 30 to 35 degree angle about 15 to 35 ft above the roadway.

Subsurface soil and groundwater conditions within the limits of the project area were explored by advancing and sampling six borings (B-101 through B-104, and B-106 and B-107) and five hand-auger explorations (HA-1 to HA-5) along West Bay Drive NW and in the slopes located above the proposed retaining walls. Subsurface soil conditions observed in the borings and hand-augers generally consist of soft to medium stiff, non-plastic to low-plasticity silt (recessional lacustrine deposits). Groundwater was observed in our explorations near the elevation of West Bay Drive NW. An approximate 2-ft deep scarp and ground cracks were observed at the top of the slope located above Wall 2 between about Station 24+50 and 25+50. The scarp and ground cracks are indicative of past soil movement.

The steep slopes located above the proposed retaining wall classify as a Landslide Hazard area per Section 18.32.605 of the Olympia Municipal Code (OMC). The OMC requires that proposed improvements within a landslide hazard area be as safe as if they were developed outside of a landslide hazard area. The soils exposed along the slopes of the West Bay Drive NW corridor (from Harrison Avenue to the Tugboat Annies Restaurant) are relatively weak and are over-steepened. As such, the slopes west of West Bay Drive NW are susceptible to downslope movement under static or seismic loading. Therefore, it will not be feasible to show that the development is as safe as if it were not located within a landslide hazard area. Rather, the purpose of our analysis is to show that the proposed retaining walls will not negatively impact the stability of the slopes above the wall.

An assessment of the impacts of constructing the proposed retaining walls on the stability of the slopes was completed. The analysis indicates the City’s proposal for Walls 1, 3, and 4 will not decrease the stability of the adjacent slopes provided the retaining wall is constructed and backfilled in a short period of time. At Wall 2, between Station 24+50 and 25+50, Spiralnails are recommended to be installed through the landslide mass to stabilize the slope prior to constructing the CIP wall.

In order to construct the retaining walls, temporary cuts into the toe of the slope will be required. If the cut is left unsupported for an extended period of time, the recessional lacustrine deposits may creep (i.e., move) downslope. Creep movement could occur over an extended period of time or it could be quite sudden. In order to reduce the risk, the project specifications should limit the duration of time that the cut

is left exposed and require that the contractor continuously monitor the slopes for movement. The contractor should be prepared to buttress the slope or install temporary shoring should creep be detected.

The near-surface soil along the alignment contains an appreciable amount of fine sand and silt and is moisture sensitive. It will likely be impractical to moisture condition and compact the on-site soil during periods of wet weather. In order to provide a suitable bearing surface for the sidewalk and retaining walls, it is anticipated that the subgrade soil will need to be overexcavated and replaced with import structural fill. Over excavation and replacement will be necessary to provide adequate foundation support. Large lateral earth pressures will be imposed on the retaining walls due to the steep backslopes. Recommended lateral earth pressure coefficients for both static and seismic loading conditions are provided in the report.

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1.0 INTRODUCTION

This report presents the results of our field investigations and provides geotechnical engineering conclusions and recommendations for design and construction of the retaining walls associated with the West Bay Drive Northwest (NW) Sidewalks Project in Olympia, Washington. The recommendations contained in this report supersede the recommendations contained in our earlier July 10, 2012 report for this study (Landau Associates 2012). The purpose of this study was to complete additional subsurface explorations at the locations of the proposed retaining walls to further characterize subsurface soil and groundwater conditions, to complete a geologic reconnaissance of the slopes at the location of the walls, and to develop revised geotechnical conclusions and recommendations for design and construction of the proposed retaining walls.

The general project location is shown on the Vicinity Map (Figure 1). The Site and Exploration Plans (Figures 2A through 2E) show the locations of the proposed retaining walls, some of the surrounding features, and the approximate locations of the explorations completed for this study. The figures also show the location of areas meeting the criteria for Landslide Hazard Areas per Title 18, Section 32.605 (18.32.605) of the Olympia Municipal Code (OMC). Appendix A presents a description of the field exploration program and summary logs of conditions observed in the explorations completed for this study. Appendix A also includes a description of the laboratory testing program and a summary of laboratory test results.

This report has been prepared based on our discussions with representatives of Skillings Connolly and the City of Olympia (City); base maps of the project area provided by Skillings Connolly; our review of readily available subsurface information in the project area; the results of the explorations completed for this project; our familiarity with geologic conditions within the vicinity of the project area; and our experience on similar projects.

1.1 PROJECT DESCRIPTION

The purpose of the West Bay Drive NW Sidewalks Project is to improve pedestrian movement and safety along West Bay Drive NW in Olympia, Washington. Based on information provided by Skillings Connolly, this project will add sidewalks to the west side of West Bay Drive NW. The addition of sidewalks to the west side of the road will require cutting into the adjacent slopes and building walls. As currently envisioned, the retaining walls will consist of cast-in-place (CIP) concrete retaining walls. In some areas, Hilfiker Spiralnails are proposed to stabilize the slope located above the Wall 2.

1.2 SCOPE OF SERVICES

Landau Associates was subcontracted by the City to provide continued geotechnical services to support the project. Our geotechnical services were provided in accordance with terms and conditions of our existing on-call contract with the City and our revised proposal for geotechnical engineering services dated January 3, 2014.

To support the proposed project, we provided the following specific services:

- Completed a geologic reconnaissance of the slopes located above the proposed retaining walls to identify the exposed soil type on the slope, the presence of any springs or seeps, and any indications of global instability such as scarps, hummocky terrain, bowed trees, etc.
- Explored the subsurface soil and groundwater conditions on the slopes above the proposed retaining walls by advancing five hand-auger explorations (HA-1 through HA-5) and two additional soil borings (B-106 and B-107). The hand-auger explorations were advanced to depths of between 3½ and 5 feet (ft) below existing ground surface (BGS) while the soil borings were advanced to depths of 21 ½ and 24 ft BGS
- Logged each soil unit observed in the exploratory borings and recorded pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence
- Completed additional geotechnical engineering analyses and developed revised geotechnical engineering conclusions and recommendations in accordance with the Standard *Specifications for Road, Bridge, and Municipal Construction* published by the Washington State Department of Transportation (WSDOT Standard Specifications; WSDOT 2014) and the *LRFD Bridge Design Specifications* published by the American Association of State Highway and Transportation Officials (AASHTO 2012)
- Prepared and submitted this revised report summarizing our findings, conclusions, and recommendations for the project. This report includes:
    - A site plan showing the locations of the explorations completed for this investigation
    - Descriptive summary logs of the conditions encountered in the explorations completed for this study
    - A summary of surface and subsurface conditions observed in the vicinity of the proposed retaining walls
    - Results of geotechnical laboratory testing
    - An assessment of the geologic hazards along West Bay Drive NW including suggested mitigation measures
    - Recommendations for earthwork including: clearing, grubbing and stripping; wet weather construction considerations; temporary and permanent slopes; subgrade preparation; structural fill; and backfill and compaction criteria
    - Recommendations for CIP concrete wall design.

2.0 EXISTING SITE CONDITIONS

This section provides a discussion of the general surface and subsurface conditions observed along the project corridor at the time of our investigations. Interpretations of the site conditions are based on the results of our review of available information, site reconnaissance, subsurface explorations, and laboratory testing.

2.1 SURFACE CONDITIONS

At the time of our field explorations, the portion of West Bay Drive NW, in the vicinity of retaining Walls 1 through 4, was paved with asphalt pavement. The west shoulder of the road is generally paved for 2 to 3 ft beyond the fog line, with a drainage ditch approximately 6 to 8 ft west of the fog line. The approximate beginning (bottom) of the slope to be constrained by the retaining walls is typically a few feet west of the existing drainage ditch. West Bay Drive NW consists of a single travel lane in each direction, with occasional pullouts and driveways.

Utility poles and overhead utility lines parallel the east (non-project) side of West Bay Drive NW in the vicinity of retaining walls 1, 2, and 3, and the west (project) side of the road in the vicinity of retaining wall 4. Several underground utilities exist throughout the project area which daylight to the surface in the form of fire hydrants, stormwater inlets, sanitary sewer manholes, etc.

The overall topography along West Bay Drive NW is relatively flat (elevation of about 20 ft) as it parallels the shore of West Bay to the east. The east side of the road, toward the bay, is generally flat and is supported primarily by fill. On the west side of the road, a drainage ditch separates the roadway from a steep east-facing hillside. In the vicinity of the proposed retaining walls, the hillside rises at an approximate 30 to 35 degree angle about 15 to 35 ft above the roadway. An approximate 2-ft deep scarp and ground cracks were observed at the top of the slope located above Wall 2 between about Station 24+50 and 25+50.

Properties adjacent to West Bay Drive NW are generally a mix of commercial and residential. Vegetation throughout the project site is a mixture of large evergreen and deciduous trees with undergrowth typical of western Washington. Tree limbs and leaning trees overhang the roadway in several places.

2.2 GEOLOGIC SETTING

Geologic information for the project area was obtained from the *Geologic Map of the Tumwater 7.5-Minute Quadrangle, Thurston County, Washington* (Walsh et al. 2003) published by the Washington State Department of Natural Resources. According to Walsh et al., the project alignment parallels a contact between advance outwash exposed on the hillside to the west and fill from the base of the slope to the shoreline.

The conditions observed in our explorations and during our slope reconnaissance are more consistent with recessional outwash, which is mapped to the south in downtown Olympia. Recessional outwash typically consists of stratified deposits of sand and silt (lacustrine). Recessional outwash is transported by meltwater streams and deposited in streams and pools emanating from the face of an ablating glacier and has not been glacially overridden. Recessional outwash consisting of sand are generally loose to medium dense in density while lacustrine deposits are generally soft to medium stiff in consistency. Recessional outwash deposits consisting of sand are generally permeable while recessional lacustrine deposits generally have very low permeability. The recessional outwash deposits at the project site are generally fine-grained, and are referred to as recessional lacustrine deposits throughout this report. In some of the explorations completed for this study, the recessional lacustrine deposits are overlain by fill associated with previous site development.

2.3 FIELD EXPLORATIONS AND LABORATORY TESTING

Subsurface conditions along the project alignment were explored by advancing and sampling six hollow-stem auger borings (B-101 through B-104, B-106, and B-107) and five hand auger borings (HA-1 through HA-5). Boring B-101 was advanced in the vicinity of proposed retaining wall 1, B-102 in the vicinity of proposed Wall 2, B-103 in the vicinity of proposed retaining Wall 3, and B-104 in the vicinity of proposed retaining Wall 4. Borings B-106 and B-107 were advanced on the slope above Wall 2. Hand augers HA-1 through HA-5 were advanced on the slopes above the proposed walls. A detailed discussion of the field exploration program, together with edited logs of the exploratory borings, is presented in Appendix A.

Prior to drilling, potholing/air vacuum holes were excavated to locate marked utilities adjacent to borings B-101 and B-102 on April 26, 2012. Potholing operations were completed by Applied Professional Services, Inc. of North Bend, Washington under subcontract to Landau Associates.

Hollow-stem auger borings B-101 through B-104 were completed with a track-mounted, hollow-stem, auger drill rig on April 30 and May 1, 2012. These borings were advanced to depths ranging from about 25 to 26 ft BGS. These exploratory auger borings were completed by Holocene Drilling, Inc. of Puyallup, Washington under subcontract to Landau Associates. Exploration locations are shown on Figures 2A through 2E of this report.

Hollow-stem auger borings B-106 and B-107 were completed with an Acker drill rig advancing hollow-stem auger on February 11, 2014. These borings were advanced to depths of 21½ and 24 ft BGS, respectively. These exploratory auger borings were completed by Boretec, Inc. of Spangle, Washington under subcontract to Landau Associates. Exploration locations are shown on Figure 2C of this report.

Hand auger borings HA-1 through HA-5 were completed with a hand auger on January 8, 2014. These hand-auger borings were advanced to depths ranging from 3 to 5 ft BGS and were completed by a Landau Associates' geotechnical engineer. Exploration locations are shown on Figures 2A through 2E of this report.

Geotechnical laboratory testing consisted of natural moisture content determinations, sieve analyses, and plasticity tests on selected samples collected during the exploration program. A discussion of the geotechnical laboratory test procedures and test results are presented in Appendix A.

2.4 SUBSURFACE SOIL CONDITIONS

Based on the results of the field exploration program and our review of available geologic information, the project alignment is interpreted to be underlain throughout the depths explored by recessional lacustrine over advance outwash. The following subsections describe the subsurface conditions along West Bay Drive NW in more detail.

2.4.1 RETAINING WALL 1

Boring B-101 was advanced about 5 ft east of the center of proposed retaining Wall 1. Soft to medium stiff silt, interpreted as recessional lacustrine deposits, was encountered to a depth of about 7½ ft BGS. Soil interpreted as advance outwash was encountered below the recessional lacustrine deposit to the bottom of the boring at about 26 ft BGS. Advance outwash consists of medium dense to dense, sandy, silty gravel and very dense, sandy gravel and gravelly to very gravelly sand with variable silt content.

Hand auger HA-3 was advanced near the center of the slope above the proposed wall. Soil interpreted as recessional lacustrine deposits was encountered throughout the depth explored (to about 4½ ft BGS). Recessional lacustrine deposits consist of medium stiff silt and loose to medium dense very silty sand.

2.4.2 RETAINING WALL 2

Boring B-102 was advanced about 5 ft east of the proposed retaining Wall 2. Soft to very stiff, sandy to very sandy silt, interpreted as recessional lacustrine deposits, was encountered to a depth of about 5 ft BGS. Advance outwash, consisting of very dense, very sandy gravel with silt and very gravelly sand was encountered beneath the recessional lacustrine deposits to the bottom of the boring at about 26 ft BGS.

Hand augers HA-4 and HA-5 were advanced on the slope above the proposed wall. Soil interpreted as recessional lacustrine deposits was encountered throughout the depth explored (to about 5 ft BGS in HA-4 and 3½ ft BGS in HA-5). Recessional lacustrine deposit consists of medium stiff silt and sandy silt and loose, silty sand.

Borings B-106 and B-107 were also advanced on the slope above the proposed wall. Both borings encountered about 7 to 9 inches of forest duff and topsoil over soil interpreted as recessional lacustrine deposits. In boring B-106, recessional lacustrine deposits consists of medium stiff to very stiff silt to about 17 ft BGS. Soil interpreted as advance outwash was encountered below the recessional lacustrine to the bottom of the boring at about 21½ ft BGS. Advance outwash consists of very dense, silty sand. In boring B-107, recessional lacustrine deposits consists of medium stiff to very stiff silt to the bottom of the boring at about 24 ft BGS.

2.4.3 RETAINING WALL 3

Boring B-103 was advanced about 5 ft east of the center of proposed retaining Wall 3. Soft to medium stiff silt with variable sand and gravel content, interpreted as recessional lacustrine, was encountered to a depth of about 4½ ft BGS. Soil interpreted as recessional outwash was encountered below the fill to a depth of about 9 ft BGS. Recessional outwash consists of medium dense sand and very stiff, sandy silt. Advance outwash was encountered below the recessional outwash to the bottom of the boring at about 26 ft BGS. Advance outwash consists of very dense, very silty sand and sand with gravel.

Hand auger HA-2 was advanced near the center of the slope above the proposed wall. Soil interpreted as recessional lacustrine deposits was encountered throughout the depth explored (to about 3 ft BGS). Recessional lacustrine deposits consist of medium stiff silt.

2.4.4 RETAINING WALL 4

Boring B-104 was advanced about 10 ft east of proposed retaining Wall 4. Loose to medium dense gravel base course was encountered to a depth of about 2 ft BGS. Soil interpreted as recessional outwash was encountered below the base course to a depth of about 13 ft BGS. Recessional outwash consists of medium dense sand and very silty sand, with a hard sandy silt layer from about 12 to 13 ft BGS. Advance outwash was encountered below the recessional outwash to the bottom of the boring at about 25½ ft BGS. Advance outwash consists of very dense, gravelly sand with silt.

Hand auger HA-1 was advanced near the center of the slope in the northern reaches of Wall 4. Soil interpreted as recessional lacustrine deposit was encountered throughout the depth explored (to about 4 ft BGS). Recessional lacustrine deposits consist of medium stiff silt.

2.5 GROUNDWATER

At the time of exploration (late April, 2012), groundwater was encountered in borings B-101 through B-104 at depths ranging from 0 to 12 ft BGS. These boreholes were left open for only a short period of time, so actual groundwater levels may have been higher than those observed at the time of drilling. Borings B-101 through B-103 were located within 1 to 2 ft of a drainage ditch containing standing water. Some of these explorations showed this to be a perched groundwater table, whereas in others, the soil remained wet throughout the depth of exploration. A reasonably conservative estimate is to assume the water table to be at the ground surface, which is likely the case during the wetter months of the year.

Subsequent explorations completed on the slope above the proposed Wall 2 (February 2014) encountered groundwater at about 15 ft BGS (B-106) and 23 ft BGS (B-107). In boring B-106, this corresponds to a groundwater depth near the West Bay Drive NW road surface. In boring B-107, this corresponds to a groundwater depth about 5 ft above the West Bay Drive NW road surface. The relatively shallow hand auger borings advanced on the slopes did not encounter groundwater.

It should be noted that the groundwater conditions reported on the summary logs are for the specific locations and dates indicated, and therefore may not necessarily be indicative of other locations and/or times. Furthermore, it is anticipated that groundwater conditions will vary depending on local subsurface conditions, the weather, and other factors. Groundwater levels in the project area are expected to fluctuate seasonally with maximum groundwater levels generally occurring during the winter and early spring months.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on conditions observed in the explorations and the results of our geotechnical engineering evaluation, construction of the proposed retaining walls associated with the West Bay Drive NW Sidewalks project is considered feasible using conventional means and methods. A discussion of landslide hazard areas and potential mitigation measures are presented in Section 3.1 of this report. Geotechnical conclusions and recommendations are presented in the following sections for earthwork and retaining wall design for CIP walls in Sections 3.2 and 3.3 of this report, respectively.

3.1 CRITICAL AREAS SUMMARY

This section of the report is intended to address the OMC, Title 18, Section 32.640 (18.32.600), which concerns landslide hazard areas. The OMC requires that proposed improvements within a landslide hazard area be as safe as if they were developed outside of a landslide hazard area. The soils exposed along the slopes of the West Bay Drive NW corridor (from Harrison Avenue to the Tugboat Annies Restaurant) are relatively weak and are over-steepened. As such, the slopes west of West Bay Drive NW are susceptible to downslope movement during static or seismic loading. Therefore, it will not be feasible to show that the development is as safe as if it were not located within a landslide hazard area. Rather, the purpose of our analysis is to show that the proposed retaining walls will not negatively impact the stability of the slopes above the wall, provided the suggested mitigation measures are implemented.

3.1.1 LANDSLIDE HAZARD AREAS

Landslide hazard areas are defined in Section 18.32.605 as areas "which are potentially subject to the risk of mass movement due to a combination of geologic, topographic and hydrologic factors; and where the vertical height is ten (10) feet or more." Such areas include:

- Slopes with grades greater than or equal to 40 percent
- Slopes of 15 percent or greater with impermeable soils and springs or groundwater seepage
- Any areas located on a landslide feature that has shown movement during the Holocene epoch or is underlain by mass wastage debris from that period.

Section 18.32.630 defines a buffer zone of undisturbed vegetation that shall be maintained around all landslide hazard areas. This buffer extends a distance back from the top of the slope equal to ⅓ of the height of the slope; away from the toe of the slope a distance equal to ½ the height of the slope; and 50 ft in all directions from a seep located in a landslide hazard areas. Buffer zones and slopes steeper than 40 percent are indicated on Figures 2A through 2E. Also shown on these figures is the location of a large landslide scarp located above the potential cut wall. By definition, this area would also classify as a landslide hazard area by Title 18, Section 32.605 of the OMC. Based on the results of our slope stability analysis, the presumptive buffer zones provided in the OMC are appropriate.

3.1.2 PROPOSED ALTERATIONS TO LANDSLIDE HAZARD AREAS

As currently envisioned, CIP retaining walls are planned at the base of four areas identified as landslide hazard areas. These areas, identified as Wall 1, Wall 2, Wall 3, and Wall 4, are shown in detail on Figures 2B through 2E, respectively. At all wall locations, plans include cutting into the toe of the slope to accommodate roadway widening and sidewalk construction. Cut faces will be about 4 ft tall and be supported by L-shaped CIP retaining walls. The affected slopes range from about 15 to 35 ft in total height and are as steep as about 80 percent (38 degrees).

The existing roadway extends well into the landslide hazard area buffer zone in many areas. The proposed improvements will extend beyond the buffer zone and into the landslide hazard area at the toe of the slopes.

3.1.3 LANDSLIDE HAZARD EVALUATION

The slopes above proposed retaining walls are all greater than 10 ft tall and steeper than 40 percent, thereby classifying as landslide hazard areas per Section 18.32.605 of the OMC. The OMC (18.32.625) allows for the expansion of existing roadway corridors and new facilities into buffer areas and landslide hazard areas, provided authorization is obtained by the hearing examiner. This section further states that crossings of landslide hazard areas shall be avoided to the extent possible and shall serve multiple properties/purposes whenever possible. Information contained in this section is derived from available mapping and a detailed slope reconnaissance of the slopes above the retaining walls on January 8, 2014. This reconnaissance was performed by a Landau Associates geologist and geotechnical engineer.

The slope above proposed Wall 1 is about 24 ft tall with an average grade of about 65 to 75 percent. The slope is heavily vegetated with fir trees, deciduous trees, and an understory typical of western Washington. An asphalt paved parking lot is situated at the top of the slope. During our site reconnaissance, we observed a crack in the asphalt setback about 15 ft from the top of the slope, paralleling the top of the slope. Additionally, pavement on the slope side of the crack appears to grade slightly downward toward the steep slope. The location of the storm drain system in the parking lot suggests this area was originally graded to slope away from the crest of the steep slope. These signs are indications of potential slope creep at this location.

The slope above proposed Wall 2 reaches a maximum of about 36 ft tall with an average grade of about 60 to 70 percent. The upper portion of the slope has a grade of about 100 percent for 6 to 10 vertical feet before transitioning to much flatter grades. The slope is heavily vegetated with fir trees, deciduous trees, and an understory typical of western Washington. Many of the trees along this slope are leaning out over West Bay Drive NW. During our site reconnaissance, we observed a scarp with a vertical offset of up to about 2 ft near the top of the slope. The location of the observed scarp is shown on Figure 2C and extends between about Station 24+50 and 25+50. In boring B-107, advanced on this slope, we observed a sheared surface in the soil sample collected at about elevation 39 ft (7½ ft BGS). These are indications that the slope is unstable and has exhibited movement in the recent past. We estimate that the unstable slope mass extends from the scarp at the top of the slope to near the existing roadway surface and is between 7 and 10 ft thick.

Additionally, a water seep was observed near the far south end of the proposed Wall 2 (shown on Figure 2C). The OMC recommends a 50-ft setback from the location of all seeps. The 50-ft setback is presented on Figure 2C. We also observed a very slow seep at the base of the slope in front of the proposed Cut Wall.

The slope above proposed Wall 3 reaches a maximum of about 36 ft tall with an average grade of about 70 to 80 percent. The slope is heavily vegetated with fir trees, deciduous trees, and an understory typical of western Washington. No signs of past or active slope instability were observed during our reconnaissance at the location of proposed Wall 3.

The slope above the northernmost reach of proposed Wall 4 is about 16 ft tall with an average grade of about 70 to 80 percent. The slope is heavily vegetated with fir trees, deciduous trees, and an understory typical of western Washington. No signs of past or active slope instability were observed during our reconnaissance at this location.

3.1.4 SLOPE STABILITY ASSESSMENT

In order to assess the impacts of the proposed wall construction on the slope located above the wall, a slope stability analysis was completed. The approach used in our slope stability analyses is as follows:

- Developed the slope geometry and subsurface profile based on existing site surveys, observations made during our geologic reconnaissance, borings advanced in the vicinity of the steep slopes, and groundwater data. In order to evaluate the impact of the walls on the slopes, slope profiles were developed for both the existing and the proposed slope geometry. Specifically, we looked at the following critical locations:
- Wall 1 at Station 20+50 – Approximately 21 ft tall, 35 degree slope with traffic surcharge located above the slope
- Wall 2 Station 25+00 – Approximately 35 ft tall, 30 degree slope located below scarp
- Wall 3 at Station 27+50 – Approximately 16 ft tall, 33 degree slope

- Wall 4 at Station 33+25 – Approximately 17 ft tall, 38 degree slope
- Assigned representative strength parameters for each subsurface unit
- Performed a series of stability analyses of the existing slope under both static and seismic conditions to estimate the factor of safety of the existing slope
- Included the retaining wall in the slope stability model and rerun the stability analysis to estimate the factors of safety of the slope with the retaining wall in place.

A computer slope stability program, SLIDE version 5.0 (Rocscience Inc. 2003), was used to determine factors of safety for the slope under both static and seismic conditions. SLIDE evaluates the stability of circular and non-circular failure surfaces in soil or rock using vertical slice limit equilibrium methods. For this project, the Spencer's method of slices was utilized. This method estimates slope stability by assuming numerous failure surfaces and calculating the forces that would cause slope movement (driving forces) and the forces resisting slope movement (resisting forces) for each selected failure surface. The ratio of resisting force to driving force for a given failure surface is referred to as the factor of safety. SLIDE uses a searching routine to determine the critical failure surfaces (i.e., those surfaces with the lowest factors of safety) for a given slope.

For Wall 1, a surcharge load of 250 pounds per square foot (psf) was applied to the top of the slope to account for vehicular parking under static loading conditions. For seismic loading conditions, a reduced traffic surcharge load of 125 psf was used in our assessment which is consistent with the policy adopted by WSDOT.

3.1.4.1 Seismic Loading

Retaining walls supporting slopes located adjacent to roads are typically designed in accordance with the AASHTO *LRFD Bridge Design Manual*. The AASHTO *LRFD Bridge Design Manual* suggests that roadways be designed to accommodate the potential impacts of an earthquake with a 7 percent probability of exceedance (PE) in 75 years (approximate 1,000/year recurrence interval). Based on our experience and our analysis, there is a widespread potential for significant damage (i.e., liquefaction, lateral spreading, slope movements) of the entire West Bay Drive NW corridor (i.e., from Harrison Avenue to Tugboat Annies Restaurant) as a result of the 1,000 year earthquake.

If the 1,000 year earthquake were to occur, it is our opinion that there could be some surficial movement of ground above the top of the proposed walls (on the order of several feet) and potentially some lateral deformation of the retaining walls (several inches) In our opinion, deformations of these kinds are not likely to impact life safety, provided that large leaning trees are felled and that the slope mitigations measures discussed in this report are implemented. Given the low risk to life safety and the significant level of damage anticipated elsewhere along West Bay Drive NW, it is our opinion that designing the retaining walls for the higher level of shaking may not be cost-effective.

We recommend that the retaining walls and the slopes be evaluated for earthquake loading similar to what was experienced in the Olympia area as a result of the 2001 Nisqually earthquake. This earthquake has an approximate recurrence interval of 108-years. We are not aware of any significant movement of the slopes in the vicinity of the proposed retaining walls as a result of the 2001 Nisqually earthquake. This lower level of earthquake shaking was discussed at a meeting held at the Olympia City Hall Building on February 21, 2014. Based on our analysis, we anticipate that movement above the retaining walls will be negligible (provided mitigation measures are implemented) for a Nisqually level earthquake event.

The potential effect of seismic loading on the steep slopes was analyzed assuming a peak horizontal ground acceleration of 0.22 percent of gravity (g) for a seismic event with a PE of 50 percent in a 75-year period (108-year earthquake). The horizontal forces developed during earthquake shaking were represented in the stability analyses by a “pseudo-static” seismic coefficient, k_h. For natural slopes, it is typical to assume that k_h is equal to up to one-half of the horizontal peak ground acceleration from the design level earthquake. Therefore, for the purpose of our seismic stability analysis, k_h was assumed to be 0.11g. The pseudo-static slope stability analysis is typically completed on the critical failure surface determined during static loading.

3.1.4.2 Modeled Subsurface Conditions

The shear strength of the identified geologic units present in the slopes above the wall were modeled by assigning an angle of internal friction (ϕ') and cohesion (c’) to each geologic unit. Our scope of services did not include determining the shear strength parameters of the identified geologic units by conducting laboratory strength tests. Shear strength parameters were selected based on the results of our field investigations, available geologic literature, and our experience with similar geologic conditions.

The individual material properties used in the stability analyses of the slope stability assessment are summarized in the table below.

SUMMARY OF MATERIAL PROPERTIES UTILIZED IN ANALYSIS

Geologic Unit	Unit Weight [pounds per cubic ft (pcf)]	Cohesion, c’ (psf)	Internal Friction Angle, ϕ’ (degrees)
Slide Debris	120	0	27.5
Recessional Lacustrine	120	100	31

Based on the conditions observed in our explorations and during our geologic reconnaissance, groundwater is situated within several feet of the elevation of West Bay Drive NW. For the purpose of our analysis, groundwater was modeled as being at an elevation within 1 ft of the base of the proposed retaining wall.

3.1.4.3 Acceptable Safety Factors

Imminent slope failure is represented by a factor of safety equal to 1.0. Natural slopes are typically considered to be stable under permanent or sustained loading conditions (i.e., static loading conditions) if the calculated minimum factor of safety is equal to or greater than about 1.3. The minimum static safety factor mentioned above is consistent with the minimum static factors of safety established by WSDOT and other government agencies. Natural slopes are typically considered to be stable under seismic loading conditions if the calculated minimum factor of safety under a pseudo-static analysis is equal to or greater than 1.1.

3.1.4.4 Stability Analyses Results and Conclusions

The following table presents the results of our slope stability analysis for each of the analyzed sections.

SLOPE STABILITY ANALYSIS RESULTS

	Existing		Proposed	
Location	Static	Seismic	Static	Seismic
Wall 1 – Station 20+50	1.38	1.18	1.43	1.22
Wall 2 – Station 25+00 (in landslide area)	0.87	0.64	0.85	0.70
Wall 3 – Station 27+50	1.66	1.36	1.76	1.45
Wall 4 – Station 33+25	1.42	1.18	1.49	1.25

Our analysis indicates that existing slopes located at the proposed location of Walls 1, 3, and 4 have a factor of safety of at least 1.3 under static loading conditions and 1.1 under seismic loading conditions. For these wall locations, the critical failure surface generally extends from the crest to the toe of the slope. Our analysis indicates that construction of the wall does not cause a reduction in the calculated factor of safety, provided that the retaining walls are designed to accommodate the lateral earth pressures described in Section 3.3.3 of this report.

At the location of the Wall 2 and within the existing landslide area, our analysis indicates that the existing slope has a factor of safety of about 0.87 under static loading conditions and 0.64 under seismic loading conditions (i.e., the slope is unstable). Our analysis indicates that construction of the CIP wall does not increase the factor of safety to acceptable levels. In other words, the portion of the slope situated above the wall has a low factor of safety under static and seismic loading conditions. As discussed in Section 3.1.5.2 of this report, Hilfiker Spiralnails should be installed in the slope above the wall to mitigate the low factor of safety.

Based on our analysis for the other walls and our experience, the portion of the Wall 2 located outside of the landslide area (i.e., not between about Station 24+50 and 25+50), will have an acceptable factor of safety under static and seismic loading for the proposed condition.

3.1.5 MITIGATION

The City's proposal for Walls 1, 3, and 4 will not decrease the stability of the adjacent slopes provided the retaining wall is constructed and backfilled in a short period of time. In these areas, as the soil in the toe of the slope is removed, there is some risk for soil creep especially if the slope is left open for extended periods of time. Completing a test section and monitoring the slope above the test section slope for several weeks may be beneficial in assessing the stand up time and creep potential of the slopes. Potential locations for test sections should be away from public right-of-way in areas where the risk to the public or their property are minimal and have similar geologic conditions with similar backslopes. The slopes above the walls should be monitored, as described in Section 3.1.5.1 of this report.

At Wall 2, between Station 24+50 and 25+50, we identified a landslide scarp during our slope reconnaissance. This area will be stabilized prior to construction of the wall with Spiralnail. Recommendations for Spiralnails are provided in Section 3.1.5.2 of this report. In addition to these mitigation measures, the slopes should be revegetated in accordance with the project Tree Plan (Landau Associates 2014).

3.1.5.1 Slope Monitoring

In order to construct the retaining walls, temporary cuts into the toe of the slope will be required. If the cut is left unsupported for an extended period of time, the recessional lacustrine deposits may creep (i.e., move) downslope. The stand-up time of the slope will be dependent on a number of factors, including the means and methods employed by the contractor, the specific subsurface soil and groundwater conditions, the weather, and other factors and will not be known until construction. Creep movement could occur over an extended period of time or it could be quite sudden. In order to reduce the risk, the project specifications should limit the duration of time that the cut is left exposed. For worker safety, the specifications should also require that all formwork be constructed from in front of the wall and in no situation should workers be allowed to enter the area between the wall and the slope.

The contractor should continuously monitor the slopes located above the proposed retaining wall for signs of instability. In addition, we recommend that the contractor establish monitoring points on the slope above the walls. The monitoring points should consist of a steel rod driven 3 ft into the slope. Monitoring points should be established at 15 to 20 ft intervals at both the midpoint and the top of slope. The monitoring points should be optically monitored in both the vertical and horizontal direction immediately after installation and twice daily when the toe is open within 25 ft of the monitoring point. The monitoring data shall be submitted to the owner's geotechnical consultant for review as soon as it becomes available. If the contractor observes signs of instability or if the settlement monitoring points indicate downslope movement, the contractor should immediately buttress the toe of the slope with quarry spalls or other angular fill material and/or install temporary sheeting. If downslope movement is observed, constructing the wall in very short lengths in slot cut into the slope, temporary shoring, slope stabilization techniques such as Spiralnails (described below), or alternative wall types should be considered.

3.1.5.2 Hilfiker Spiralnails

Mitigation will be required to stabilize the portion of the slope located between about Station 24+50 and Station 25+50. Based on conversations with the design team, we understand that Hilfiker Spiralnails are the preferred mitigation technique for this area. Spiralnails consist of small-diameter, steel soil nails that will be driven into the underlying undisturbed recessional lacustrine deposits to stabilize the landslide mass. Specially designed "Spiders" or wire mesh placed on the slope work together with the Spiralnails to create a reinforced wire web over the slope face making it possible to stabilize the slope while leaving most of the vegetation intact. The Spiralnails are usually installed in a grid pattern, adjusted to avoid trees. The spacing of the elements depends on the slope inclination and soil strength as well as the required post-construction factor of safety. The system is proprietary and designed by Hilfiker of Humboldt, California (wire mesh slope facing) or by Aziz Engineering Company of Redmond, Washington ("Spider" slope facing).

We recommend that the Spiralnail system be designed to increase the factor of safety of the slope to at least 1.3 under static loading conditions and to at least 1.1 under seismic loading conditions. For seismic loading conditions, a peak horizontal ground acceleration of 0.22g, corresponding to the 108-year earthquake should be assumed. The soil parameters provided in Section 3.1.4.2 should be utilized in the assessment. Groundwater should be assumed to be situated at elevation 22 ft. The Spiralnail system should be submitted to the owner for review prior to construction. In addition, construction of the retaining wall at the base of the slope should not commence until after the Spiralnail system is constructed. All leaning trees should be removed from the slope – the root balls should remain in place.

Given the underbrush present on the slopes, the zone of slope instability may be greater than what we observed during our slope reconnaissance. Therefore, we recommend that the project include a contingency should the amount of slope area requiring stabilization be greater than that estimated during our slope reconnaissance.

3.1.5.3 Wall 1 Ground Crack

As discussed above, indications of soil creep were observed in the parking lot and on private property during our geologic reconnaissance of the area above Wall 1. The intrusion of groundwater into ground cracks or onto the slope face can lead to an increased risk of global instability. We recommend that the property owner seal the crack in the pavement be sealed to minimize the potential for water intrusion into the slope. Furthermore, we recommend that the property owner regrade the pavement to the east of the crack to promote drainage away from the slope and into the parking lot's existing storm drain system.

3.2 EARTHWORK

Earthwork to accommodate the proposed improvements is expected to consist of clearing, grubbing and stripping of areas where the roadway will be widened and where cuts and fills are required, slope construction and compaction, preparation of subgrade below wall foundations, and general fill placement and compaction.

3.2.1 WET WEATHER CONSIDERATIONS

Earthwork-related construction will be influenced by weather conditions. Most of the existing near-surface soil along the alignment consists of recessional lacustrine deposits. This type of soil, containing a significant amount of fine sand and silt, is moisture sensitive. Site grading activities using moisture-sensitive soil should normally occur during the relatively warmer and drier period between about mid-summer to early fall. Completing these activities outside of this normal construction window could lead to a significant increase in construction costs due to weather-related delays, repair of disturbed areas, and the increased use of "all-weather" import fill materials.

Because of the moisture sensitivity, unprotected site soil, in either a compacted or uncompacted state, will degrade quickly to a slurry-like consistency in the presence of water and construction traffic. If subgrade or fill soil becomes loosened or disturbed, affected soil should be overexcavated and replaced with properly compacted structural fill. For wet weather construction, the contractor may reduce the potential for disturbance of subgrades by the following:

- Protecting exposed subgrades from disturbance by construction activities by constructing gravel working mats
- Working off of the existing asphalt pavement

- Using a trackhoe with a smooth-bladed bucket to limit disturbance of the subgrade during excavation
- Suspending earthwork and other construction activities that may damage subgrades during rainy days
- Limiting and/or prohibiting construction traffic over unprotected soil
- Providing designated haul roads for construction equipment
- Sloping excavated surfaces to promote runoff
- Sealing the exposed surface by rolling with a smooth-drum compactor or rubber-tire roller at the end of each working day and removing wet surface soil prior to commencing filling each day.

3.2.2 DEMOLITION AND CLEARING, GRUBBING AND STRIPPING

Clearing and grubbing should be in accordance with the requirements in Section 2-01 of the 2014 WSDOT Standard Specifications. Material generated during clearing and grubbing should be properly disposed of at an approved offsite location. Topsoil, and/or other organic-rich soil from existing landscape areas along the roadway corridor, should be stripped to expose the underlying inorganic soil. Based on conditions observed in our borings, stripping depths are anticipated to be about ½ ft. Stripped material is not considered suitable for use as fill. Stripped material should either be wasted off site at an approved location, or stockpiled for later use as topsoil.

If required, the removal of existing improvements (e.g., existing pavement sections) should be in accordance with the requirements of Section 2-02 of the 2014 WSDOT Standard Specifications. Existing asphalt pavement that is removed to accommodate the proposed improvements may be pulverized, stockpiled, and recycled for use as structural fill, provided the asphalt is processed to meet the requirements in Section 9-03.21 of the 2014 WSDOT Standard Specifications. Disposal of the asphalt pavement at an approved offsite location is also a viable alternative.

Utilities that will be abandoned that are less than 3 ft deep below final grades should be removed and disposed of off site. Deeper lines left in place should be grouted full with controlled density fill (CDF) to reduce the potential for differential settlement resulting from collapsed pipes or erosion. CDF should meet the requirements in Section 2-09.3(1)E of the 2014 WSDOT Standard Specifications.

All incidental excavations associated with clearing and grubbing should be backfilled in accordance with the recommendations in Section 3.2.6 of this report.

3.2.3 TEMPORARY SLOPES

In order to accommodate the construction of new retaining walls, temporary excavations into the existing slopes along the west side of West Bay Drive NW will be required. Based on the soil conditions observed in our explorations, we anticipate that temporary excavations for retaining walls will generally encounter recessional lacustrine deposits consisting of medium stiff silt and sandy silt that would be expected to creep if left unsupported for extended periods of time.

Temporary excavation slopes should be the sole responsibility of the contractor. All local, state, and federal safety codes should be followed. The contractor should implement measures to prevent surface water runoff from entering excavations. All temporary excavation slopes should be monitored by the contractor during construction for any evidence of instability. If instability is detected, the contractor should be prepared to buttress the toe of the slope.

3.2.4 SUBGRADE PREPARATION

We recommend that the CIP concrete retaining (including sidewalk) wall be underlain by a minimum of 12 inches of import structural fill. After site preparation activities and any cuts needed to establish the planned subgrade elevation, we recommend that the subgrade be thoroughly proof-rolled with a heavy, rubber-tired equipment in the presence of a qualified geotechnical or civil engineer to check for the presence of soft, loose, and/or disturbed areas. If the qualified geotechnical or civil engineer determines that the subgrade soil is too soft or wet to be proof-rolled, alternative methods (i.e., probing with a ½-inch-diameter steel t-probe) identified by the qualified geotechnical or civil engineer could be used to identify soft, loose, and/or disturbed areas.

If any soft or disturbed areas are revealed during proof-rolling that cannot be compacted to a stable and uniformly firm condition, we recommend that the unsuitable soils be overexcavated and replaced with structural fill. The extent of overexcavation and replacement with structural fill will depend on a number of things, including; 1) final planned site grades relative to the area, 2) depth to groundwater and 3) planned use of the area (i.e., structural slab, driveway, retaining wall, etc.). The actual overexcavation depths should be determined in the field during construction based on the recommendations of the qualified geotechnical or civil engineer. It is anticipated that the maximum depth of overexcavation (including 12 inches of import structural fill described above) will be 2 ft.

If needed, to stabilize the soft/wet base of the overexcavation areas, a 6- to 12-inch layer of quarry spalls could be placed to establish a base on which to compact the structural fill. Quarry Spalls should meet the requirements of Section 9-13.6 of the 2014 WSDOT Standard Specifications. The quarry spalls should be pushed into the native subgrade by wheel rolling with a vibratory roller without the use of vibration. Alternatively, they could be pushed into the native subgrade soils with the back of an excavator or backhoe bucket. Import structural fill meeting the requirements of Section 3.2.5 should be utilized to backfill the remaining overexcavation. Import structural fill should be compacted to a firm and unyielding surface by track-walking with a dozer or non-vibratory compaction equipment.

A non-woven geotextile could be placed at the base of the excavation in order to limit the amount of overexcavation. The non-woven geotextile should meet the requirements for soil stabilization in Table 3 in Section 9-33.2 of the 2014 WSDOT Standard Specifications. The geotextile should be placed in accordance with the requirements of Section 2-12 of the 2014 WSDOT Standard Specifications. The geotextile should either be overlapped a minimum of 2 ft along all of the transverse and longitudinal joints or the seams shall be sewn together. The initial lift of structural fill over the geotextile should be a minimum of 12 inches thick. Under no circumstances should construction equipment be allowed on the geotextile fabric before placement of the initial lift of fill.

3.2.5 STRUCTURAL FILL

Structural fill is defined as material needed to establish planned subgrade elevations within the roadway corridor. The suitability of excavated soil or imported soil for use as structural fill will depend on the gradation and moisture content of the soil when it is placed. As the amount of fines increases, the soil becomes increasingly sensitive to small changes in moisture content and adequate compaction becomes more difficult to achieve. Soil containing more than about 5 percent fines cannot consistently be compacted to a dense, non-yielding condition when the water content is greater than about 2 to 3 percent above optimum moisture content. Optimum moisture content is the moisture content at which the greatest compacted dry density can be achieved.

The near-surface soil contains a significant quantity of silt and is well above the optimum moisture content for compaction and is not suitable for use as structural fill. Import structural fill will be required. Import structural fill should meet the requirements for Select Borrow in Section 9-03.14(2) of the 2014 WSDOT Standard Specifications with the exception that the maximum particle size should not exceed 3 inches. If wet weather construction is anticipated, the amount of fines (material passing a U.S. No. 200 sieve) should not exceed 5 percent, by dry weight, based on a wet sieve analysis of that portion passing the ¾-inch sieve.

3.2.6 BACKFILL AND COMPACTION REQUIREMENTS

Structural fill should be placed and compacted in accordance with Section 2-03.3(14)C, Method C of the 2014 WSDOT Standard Specifications. Compaction and moisture control tests should be done in accordance with Section 2-03.3(14)D of the 2014 WSDOT Standard Specifications. The maximum dry density and optimum moisture content may also be determined by the ASTM International D 1557 (modified Proctor) test procedure.

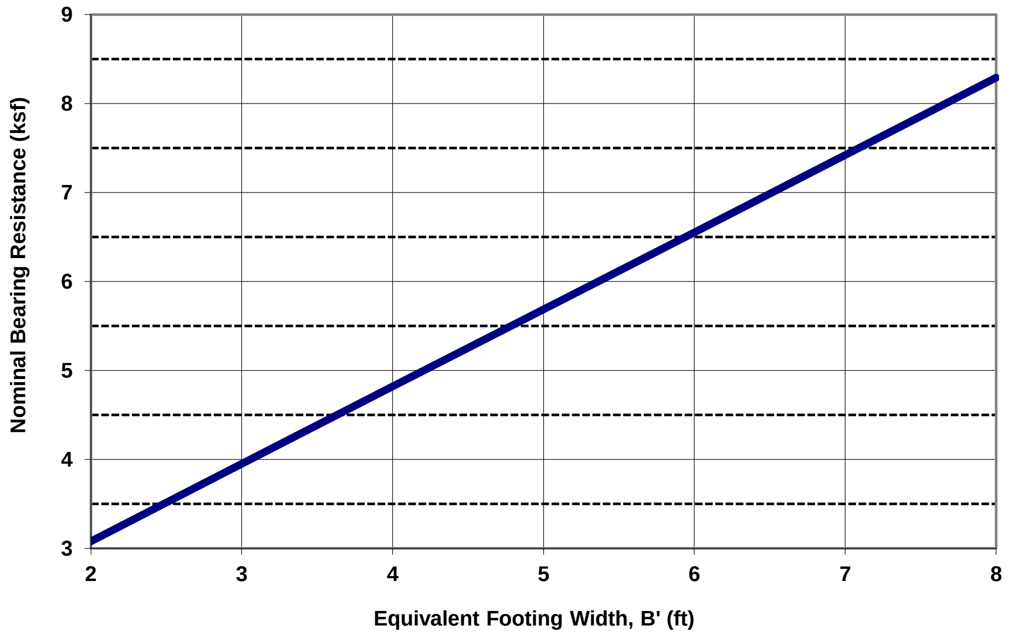
3.3 CAST-IN-PLACE CONCRETE RETAINING WALL DESIGN

The following section of this report provides geotechnical design recommendations for the proposed CIP concrete retaining walls. The foundation subgrade should be prepared as described in Section 3.2.4 of this report. Geotechnical recommendations are provided in the following section for nominal bearing resistance, foundation settlement, lateral pressures, resistance to lateral loads, external stability, and wall backfill and drainage considerations.

3.3.1 NOMINAL BEARING RESISTANCE

The nominal bearing resistance of shallow foundations is dependent on the depth of embedment, the equivalent footing width (B’), groundwater, and other factors. Groundwater levels are anticipated to be near the planned foundation subgrade elevations. The nominal bearing resistances [kips per square foot (ksf)] summarized on the following figure may be utilized for design of the wall foundations. The nominal bearing resistances assume that the subgrade has been prepared as described above and that the foundation is embedded at least 1 ft below the lowest adjacent site grade.

NOMINAL BEARING RESISTANCE FOR STRENGTH I LIMIT STATE AND EXTREME EVENT I LIMIT STATE FOR WALL DESIGN



The following table summarizes the recommended resistance factors for use in bearing capacity design:

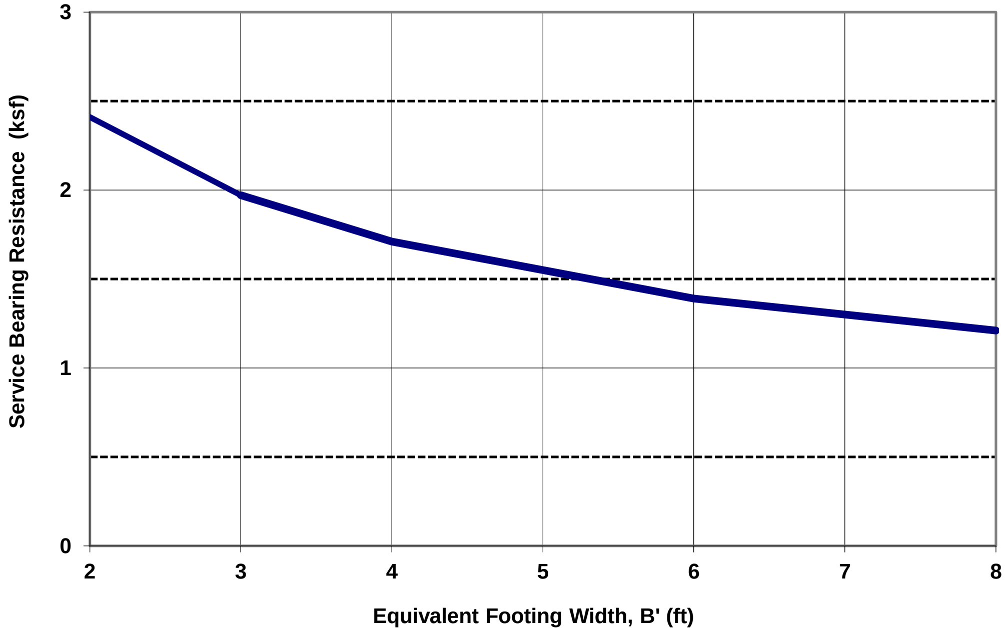
RESISTANCE FACTORS FOR BEARING CAPACITY AND SETTLEMENT DESIGN

Limit State	Resistance Factor
Service Limit State	1.0
Strength I Limit State	0.55
Extreme Event I Limit State	0.80

3.3.2 FOUNDATION SETTLEMENT

Settlement of shallow foundations depends on the foundation size and bearing pressure, as well as the strength and compressibility characteristics of the underlying bearing soil. The following figure can be utilized to estimate the maximum service pressure resulting in total foundation settlements of about 1 inch:

SERVICE BEARING PRESSURE FOR 1 INCH OF SETTLEMENT – SERVICE LIMIT STATE



Assuming similarly loaded foundation elements, differential settlement between two points spaced 50 ft away along the length of the wall will be ½ inch or less. Distortion due to differential settlement along the length of the wall should be less than $^{1}/_{300}$. If the foundations are dissimilarly loaded, differential settlement and distortion may be greater than what is estimated above. Most of the settlement described above will occur when the static loads are applied. Post-construction settlements should be negligible.

3.3.3 LATERAL EARTH PRESSURES

The lateral pressure that develops along the face of the wall is dependent on the amount the wall is allowed to rotate or yield. If the wall is unrestrained against rotating and yielding at least 0.001H, where H is the height of the wall (including embedment); the wall may be designed assuming active conditions. If the walls are restrained against rotating and yielding; the walls should be designed assuming at-rest conditions. It is assumed that the retaining walls are free to translate and should be designed utilizing active lateral earth pressure parameters. The following table provides soil properties for retaining wall design:

DESIGN PARAMETERS FOR LATERAL EARTH PRESSURE CALCULATIONS

Parameter	Backslope Conditions	
	Flat (Southern Portion of Wall 4)	Inclined (All other wall locations) (1)(2)
Moist Unit Weight (γ_m)	120 pcf	120 pcf
Buoyant Unit Weight ($\gamma_{;}$)	58 pcf	58 pcf
Active Earth Pressure Coefficient (k_a) (3)	0.32	0.44
Seismic Earth Pressure Coefficient (K_{ae}) (3)(4)	0.39	0.75
Passive Earth Pressure Coefficient (k_p) (5)	3.12	3.12

Notes:

(1) Active and seismic earth pressure coefficients for inclined slope above wall was estimated using general limit equilibrium approach as described by Anderson et al. (2008).
(2) Active and seismic earth pressure coefficients assume that the slopes are stable or have been mitigated using Spiralnails.
(3) Active lateral earth pressure coefficients assume drained conditions.
(4) Seismic active lateral earth pressure coefficient was developed for an earthquake with a 50 percent probability of exceedance in 75-years.
(5) Passive earth pressure coefficient is an ultimate value and has not been reduced to limit deflections. Passive earth pressure coefficient assumes ground slope is level in front of wall a distance equal to two times the embedment depth. For estimating passive resistance, we recommend assuming the soil providing passive resistance is saturated and utilizing buoyant unit weights.

Wall Backfill should consist of Gravel Backfill for Walls meeting the requirements in Section 9-03.12(2) of the 2014 WSDOT Standard Specifications. Wall backfill should be compacted in accordance with Section 2-09 of the 2014 WSDOT Standard Specifications. Wall drainage shall be in accordance with Section 6-02.3(22) of the 2014 WSDOT Standard Specifications. We recommend installing underdrains in general accordance with WSDOT *Standard Plan* D-4 (WSDOT 2013). The perforated under drain pipe should be supplied with clean-outs.

Design of any subsurface walls with a flat backslope, such as the southern portion of Wall 4, should include appropriate lateral earth pressures caused by any adjacent surcharge loads. For uniform surcharge pressures due to vehicular loading, a uniformly distributed horizontal load of 0.32 times the surcharge pressure should be included in the design. At a minimum, we recommend establishing a vertical surcharge load of 250 psf to account for vehicular traffic for the Service and Strength I limit states. For the Extreme Event I limit state, a vertical surcharge load of 125 psf should be utilized. Where large surcharge loads, such as heavy trucks, a crane, or other construction equipment will be located within a distance equal to one-half of the wall height, the retaining wall should be designed to accommodate the additional lateral pressures resulting from these surcharge loads.

For the Extreme Event I Limit State, the horizontal earth pressure should be calculated and distributed as a single triangular pressure as specified in Section 11.6.5 of the 2012 AASHTO *LRFD Bridge Design Specifications.* The resultant of the horizontal earth pressure can be assumed to act a point of ⅓H above the base of the wall. The horizontal earth pressure for the Extreme Event I Limit State includes both static and dynamic lateral pressures and must not be added to the static lateral earth pressure.

3.3.4 RESISTANCE TO LATERAL LOADS

Passive earth pressures acting against the sides of the foundations, in conjunction with friction developed between the base of the foundation and the supporting subgrade will resist lateral loads transmitted from the wall to its foundation. A nominal sliding coefficient of 0.55 may be used to compute the nominal sliding resistance between soil and foundation for cast-in-place concrete footings placed on import structural fill.

The passive pressure provided by soil in front of the foundation is a function of the soil in front of the foundation (both composition and density), the slope in front of the foundation, the depth of embedment, and the presence of groundwater. The groundwater table is in close proximity to the base of the wall. Therefore, we recommend utilizing an equivalent fluid weight (EFW) of 181 pcf for determining resistance to lateral loads. The EFW is a nominal value, and has not been reduced by a factor of 1.5 to limit deflections to 1 to 2 percent of the embedded depth. Passive resistance within 12 inches below the adjacent ground surface should be neglected in the design. The following table summarizes the recommended resistance factors for use in resistance to lateral load calculations:

RESISTANCE FACTORS FOR LATERAL LOAD DESIGN

	Resistance Factor	
Limit State	Shear Resistance, ϕ_τ	Passive Resistance, ϕ_{ep}
Service Limit State	1.0	1.0
Strength I Limit State	0.8	0.5
Extreme Event I Limit State	1.0	1.0

4.0 REVIEW OF DOCUMENTS AND CONSTRUCTION OBSERVATIONS

Landau Associates recommends that we review the geotechnical-related portions of the plans and specifications for the proposed project in advance of project bidding. The purpose of the review is to verify that the recommendations presented in this geotechnical report have been properly interpreted and implemented in the design and specifications.

We recommend that monitoring, testing, and consultation be provided during construction to confirm that the conditions encountered are consistent with those indicated by our explorations, to provide expedient recommendations should conditions be revealed during construction that differ from those anticipated, and to evaluate whether geotechnical-related activities comply with project plans and specifications and the recommendations contained in this report. Such geotechnical-related activities include observation of the prepared retaining wall foundation subgrade, retaining wall construction observation, and other geotechnical-related earthwork activities. The purpose of these services would be to observe compliance with the design concepts, specifications and recommendations of this report, and in the event subsurface conditions differ from those anticipated before the start of construction, provide revised recommendations appropriate to the conditions revealed during construction. Landau Associates would be pleased to provide these services for you.

5.0 USE OF THIS REPORT

This report was prepared for the exclusive use of the City of Olympia for specific application to the West Bay Drive NW Sidewalk Improvements project. The use by others, or for purposes other than intended, is at the user's sole risk. The findings, conclusions, and recommendations presented herein are based on our understanding of the project, review of available geotechnical and geologic information in the project vicinity, and on subsurface and pavement conditions observed during explorations completed on April 30, 2012; May 1, 2012; January 8, 2014; and February 11, 2014. Within the limitations of scope, schedule, and budget, the conclusions and recommendations presented in this report were prepared in accordance with generally accepted geotechnical engineering principles and practices in the area at the time the report was prepared. We make no other warranty, either express or implied.

There may be some variation in subsurface soil and groundwater conditions at the site, and the nature and extent of the variations may not become evident until construction. Accordingly, a contingency for unanticipated conditions should be included in the construction budget and schedule. We should be contacted if variations in subsurface conditions are encountered during construction.

We appreciate the opportunity to provide geotechnical services on this project and look forward to assisting you during the bidding and construction phases. If you have any questions or comments regarding the information contained in this report, or if we may be of further service, please call.

LANDAU ASSOCIATES, INC.

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JDE/BAB/EJH/jrc

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Data Source: Esri 2012

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Vicinity Map

Figure
1

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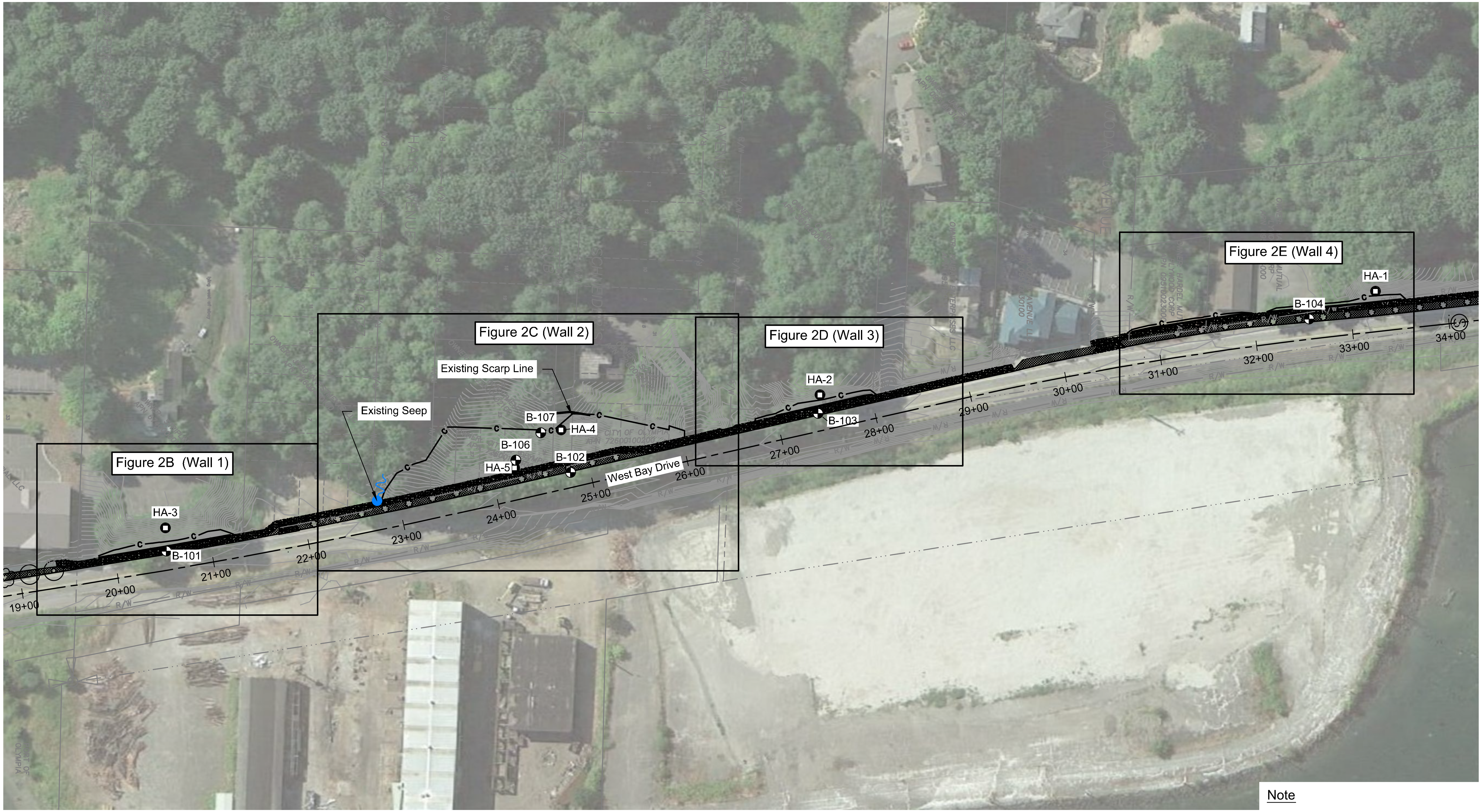

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

N

0 100 200

Scale in Feet

Source: City of Olympia 2013

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LANDAU ASSOCIATES

West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Site and Exploration Plan
Overview Map

Figure
2A

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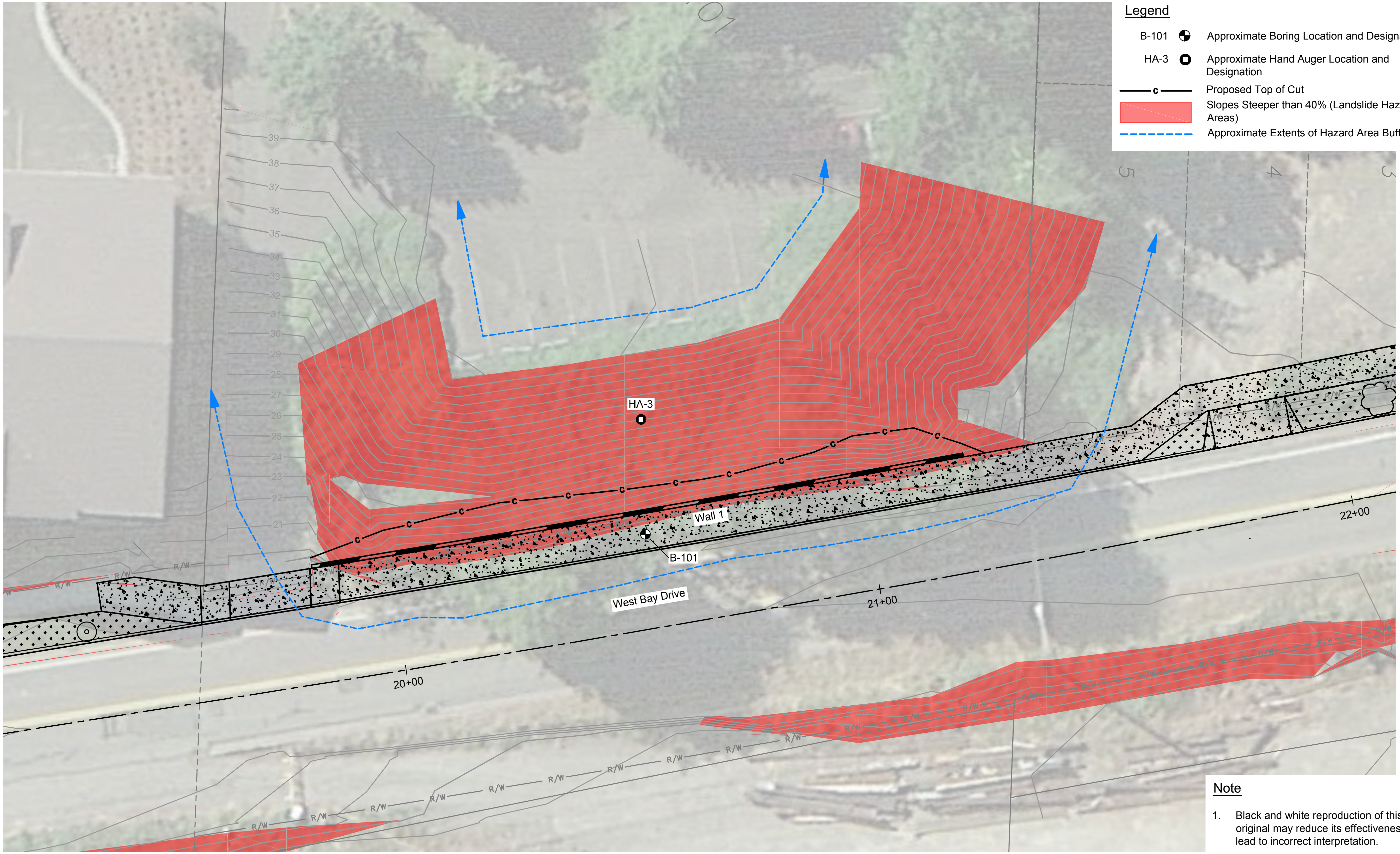

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0 20 40

Scale in Feet

Source: City of Olympia 2013

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Site and Exploration Plan
Wall 1

Figure
2B

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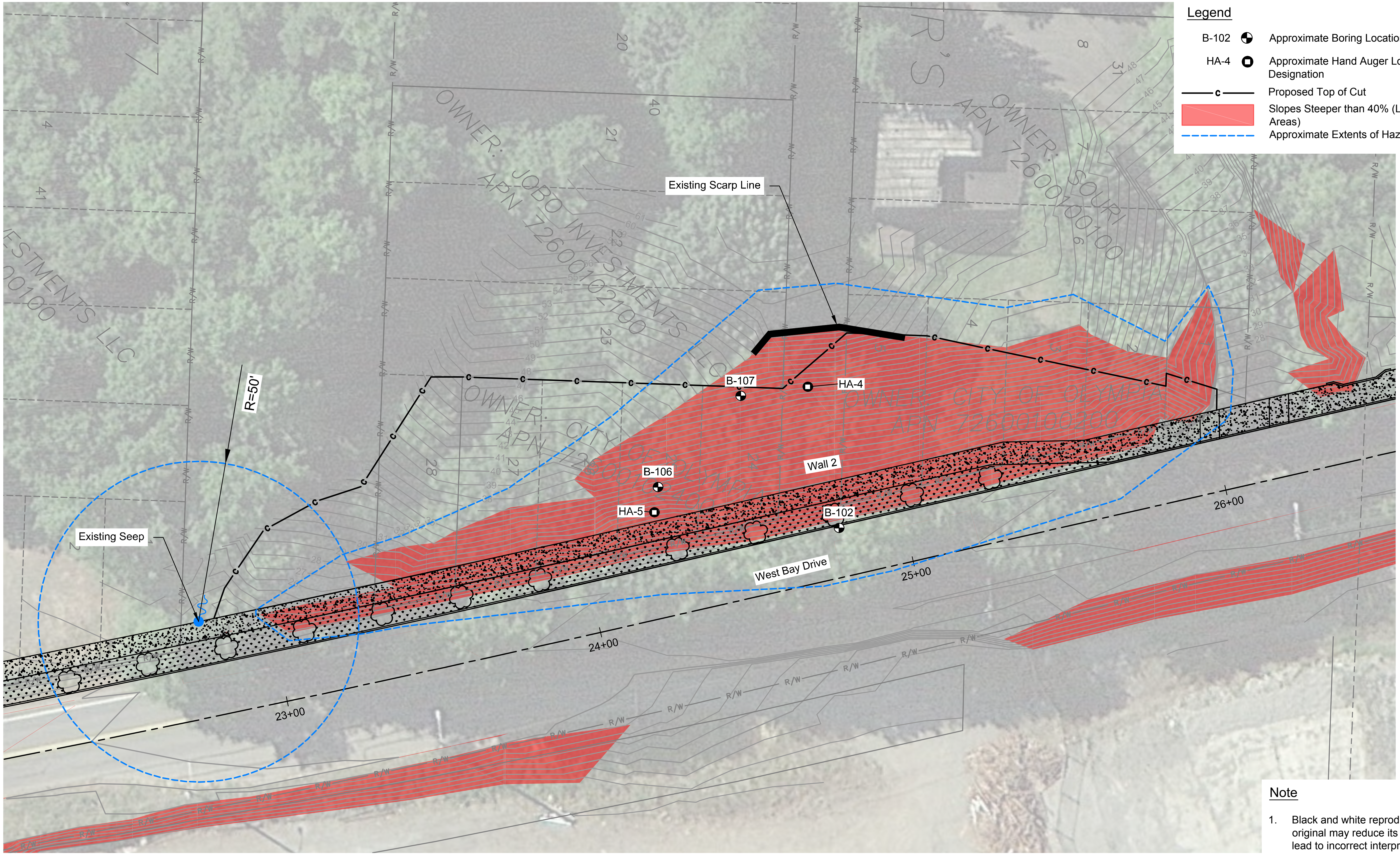

Legend

B-102 ◒	Approximate Boring Location and Designation
HA-4 ▣	Approximate Hand Auger Location and Designation
——c——	Proposed Top of Cut
🟥	Slopes Steeper than 40% (Landslide Hazard Areas)
- - - -	Approximate Extents of Hazard Area Buffer

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

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0 30 60
Scale in Feet

Source: City of Olympia 2013

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LANDAU ASSOCIATES

West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Site and Exploration Plan
Wall 2

Figure
2C

CP&D RECEIVED March 31, 2014

Legend

B-103 ◐	Approximate Boring Location and Designation
HA-2 ○	Approximate Hand Auger Location and Designation
—c—	Proposed Top of Cut
🟥	Slopes Steeper than 40% (Landslide Hazard Areas)
- - - -	Approximate Extents of Hazard Area Buffer

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

N

0 20 40
Scale in Feet

Source: City of Olympia 2013

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LANDAU ASSOCIATES

West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Site and Exploration Plan
Wall 3

Figure
2D

Legend

B-104 ⊕	Approximate Boring Location and Designation
HA-1 ◘	Approximate Hand Auger Location and Designation
—c—	Proposed Top of Cut
🟥	Slopes Steeper than 40% (Landslide Hazard Areas)
- - -	Approximate Extents of Hazard Area Buffer

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

N

0 20 40

Scale in Feet

Source: City of Olympia 2013

LANDAU ASSOCIATES, INC. | G:\Projects\258\031\050\051\F Wall Plans.dwg (A) "4" 3/19/2014

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Site and Exploration Plan
Wall 4

Figure
2E

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Subsurface conditions within the limits of the project area were explored on April 30, 2012; May 1, 2012; January 8, 2014; and February 11, 2014. The exploration program consisted of advancing and sampling six exploratory borings (B-101 through B-104, B-106, and B-107) and five hand-auger borings at the approximate locations shown on the Site and Exploration Plans (Figures 2A through 2E). Borings B-101 though B-104 were advanced to depths ranging from about 25 to 26 feet (ft) below ground surface (BGS) using a tracked drill rig advancing hollow-stem augers. The hollow-stem auger borings were advanced by Holocene Drilling, Inc. of Puyallup, Washington under subcontract to Landau Associates. Borings B-106 and B-107 were advanced to depths ranging from about 21½ to 24 ft BGS using a limited access Acker drill rig. The Acker borings were advanced by Boretec, Inc. of Spangle, Washington under subcontract to Landau Associates. The hand augers were advanced by Landau Associates personnel to depths ranging from about 3 to 5 ft BGS. The explorations were located approximately in the field by referencing existing physical features referenced on a site plan provided by Skilling Connolly. Ground surface elevations at the exploratory borings were approximated using a topographic map provided by Skillings Connolly.

The field exploration program was coordinated and monitored by a Landau Associates geotechnical engineer or geologist, who also obtained representative soil samples, maintained a detailed record of the observed subsurface soil and groundwater conditions, and described the soil encountered by visual and textural examination. Each representative soil type observed in our exploratory borings was described using the soil classification system shown on Figure A-1, in general accordance with ASTM International (ASTM) D 2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)*. Logs of the exploratory borings are presented on Figures A-2 through A-12. These logs represent our interpretation of subsurface conditions identified during the field exploration program. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific date and locations reported and, therefore, are not necessarily representative of other locations and times. A further discussion of the soil and groundwater conditions observed is contained in the text portion of this report.

Disturbed soil samples encountered from the hollow-stem auger borings were obtained at frequent intervals using a 1.5-inch inside diameter Standard Penetration Test split-spoon sampler. The sampler was driven up to 18 inches into the undisturbed soil ahead of the auger bit with a 140-pound hammer falling a distance of approximately 30 inches. An automatic trip hammer was utilized in borings B-101 through B-104 while a rope-and-cathead hammer was used for borings B-106 and B-107. The number of blows required to drive the sampler for the final 12 inches (or portion thereof) of soil penetration is noted on the boring logs adjacent to the appropriate sample notation. Soil samples collected in the borings were taken to our laboratory for further examination and testing. Upon completion of drilling and sampling, the boreholes were decommissioned in general accordance with the requirements of Washington Administrative Code 173-160.

The laboratory testing program, which was performed in general accordance with the ASTM standard test procedures described below, included visual inspection to confirm our field soil descriptions, determination of the natural moisture content and grain size distribution on selected samples, and Atterberg limits tests to assess the plasticity characteristics of fine grained soil. The natural moisture contents of selected soil samples obtained from our exploratory borings was determined in general accordance with ASTM D 2216 test procedures. The results from the natural moisture content determinations are indicated adjacent to the corresponding samples on the summary logs. The grain size distributions of selected soil samples obtained from our exploratory borings was determined in general accordance with ASTM D 422 test procedures. The results are presented in the form of grain size distribution curves on Figure A-13 and denoted with a “GS” in the Test Data column on the summary logs. Atterberg limits testing was performed in general accordance with ASTM D 4318 test procedures. The results are presented in the form of a summary graph on Figure A-14 and denoted with an “AL” in the Test Data column on the summary logs.

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL(1)	TYPICAL DESCRIPTIONS (2)(3)
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)	○	GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
			○	GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	▥	GM	Silty gravel; gravel/sand/silt mixture(s)
			▨	GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)	░	SW	Well-graded sand; gravelly sand; little or no fines
			░	SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)	▥	SM	Silty sand; sand/silt mixture(s)
			▨	SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)		▥	ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
			▨	CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
			▒	OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)		▥	MH	Inorganic silt; micaceous or diatomaceous fine sand
			▨	CH	Inorganic clay of high plasticity; fat clay
			▦	OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL		〰	PT	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT	■ ░	AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK	▧	RK	Rock (See Rock Classification)
WOOD	▒	WD	Wood, lumber, wood chips
DEBRIS	◇	DB	Construction debris, garbage

Notes:
1. USCS letter symbols correspond to symbols used by the Unified Soil Classification System and ASTM classification methods. 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 descriptions are based on the general approach presented in the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the Standard Test Method for Classification of Soils for Engineering Purposes, as outlined in ASTM D 2487.
3. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent: > 50% - "GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents: > 30% and ≤ 50% - "very gravelly," "very sandy," "very silty," etc.
> 15% and ≤ 30% - "gravelly," "sandy," "silty," etc.
Additional Constituents: > 5% and ≤ 15% - "with gravel," "with sand," "with silt," etc.
≤ 5% - "with trace gravel," "with trace sand," "with trace silt," etc., or not noted.

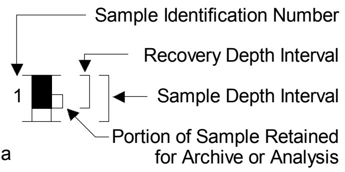
4. Soil density or consistency descriptions are based on judgement using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.

Drilling and Sampling Key

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Single-Tube Core Barrel
f	Double-Tube Core Barrel
g	2.50-inch O.D., 2.00-inch I.D. WSDOT
h	3.00-inch O.D., 2.375-inch I.D. Mod. California
i	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Vibrocore (Rotosonic/Geoprobe)
5	Other - See text if applicable

SAMPLE NUMBER & INTERVAL



Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

Groundwater

▽ Approximate water level at time of drilling (ATD)
▼ Approximate water level at time other than ATD

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Soil Classification System and Key

Figure
A-1

B-101

LAI Project No: 258031.050

SAMPLE DATA

Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
S-1	b2	4	W = 39 AL
S-2	b2	7	W = 30
S-3	b2	24	
S-4	b2	32	GS W = 14
S-5	b2	55	
S-6	b2	50/ 4"	
S-7	b2	50/ 4"	
S-8	b2	50/ 4"	

Depth (ft): 0, 5, 10, 15, 20, 25

Elevation (ft): 15, 10, 5, 0, -5

SOIL PROFILE

Drilling Method: Hollow-Stem Auger

Ground Elevation (ft): 19

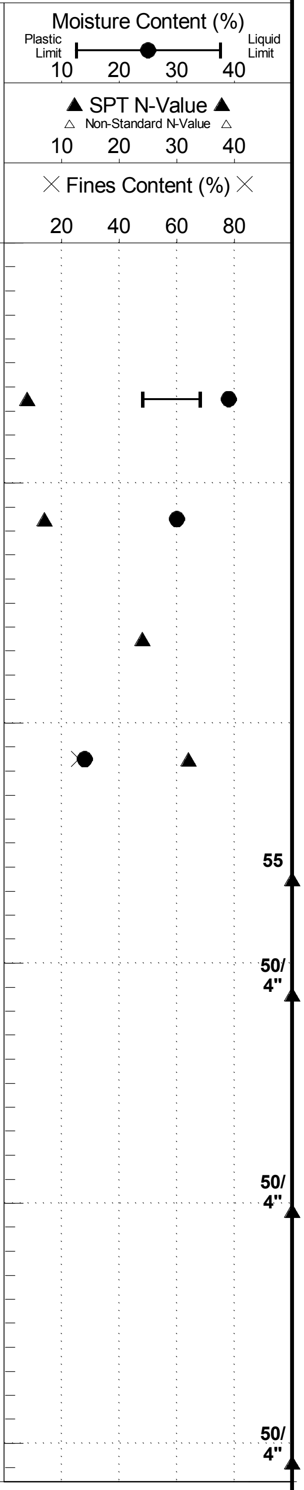
Drilled By: Holocene Drilling Inc.

Logged By: JDE Date: 04/30/12

USCS Symbol	
ML	Light brown SILT, frequent oxidation staining / rust coloring, low plasticity (soft to medium stiff, wet) **(RECESSIONAL LACUSTRINE)**
GM	Brown, sandy, silty GRAVEL (medium dense to dense, wet) **(ADVANCE OUTWASH)**
SP-SM	Brown, gravelly, fine to coarse SAND with silt (very dense, wet)
GP	Brown, sandy GRAVEL (very dense, wet)
SP	Brown, very gravelly, fine to coarse SAND (very dense, wet)
SP	Brown, gravelly, fine to coarse SAND (very dense, wet)

Groundwater

2.0 ft ATD



Boring Completed 04/30/12
Total Depth of Boring = 25.8 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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West Bay Drive NW
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Log of Boring B-101

Figure
A-2

B-102

LAI Project No: 258031.050

SAMPLE DATA

Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
S-1	b2	17	GS W = 25
S-2	b2	50/ 3"	GS W = 5
S-3	b2	50/ 6"	
S-4	b2	100/ 4"	
S-5	b2	50/ 4"	
S-6	b2	50/ 6"	
S-7	b2	50/ 6"	
S-8	b2	50/ 6"	

Depth (ft): 0, 5, 10, 15, 20, 25

Elevation (ft): 20, 15, 10, 5, 0

SOIL PROFILE

Drilling Method: Hollow-Stem Auger

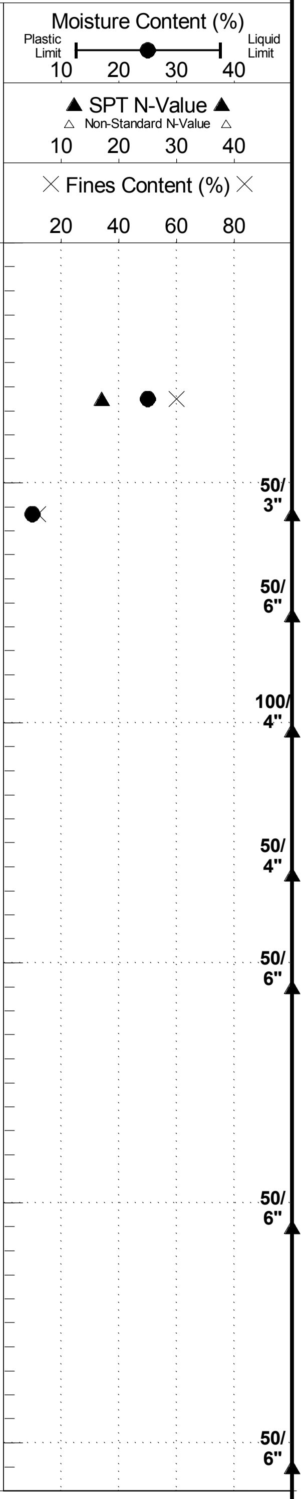
Ground Elevation (ft): 22

Drilled By: Holocene Drilling Inc.

Logged By: JDE Date: 04/30/12

USCS Symbol	Description
ML	Dark brown, sandy, SILT with trace organics (soft, wet) **(RECESSIONAL LACUSTRINE)**
ML	Grayish-brown, very sandy SILT with gravel (very stiff, wet)
GP-GM	Brown to gray, very sandy, GRAVEL with silt (very dense, dry to moist) **(ADVANCE OUTWASH)** Grades to gray
SW	Gray, very gravelly, fine to coarse SAND (very dense, wet)

Groundwater: 12.0 ft ATD



Boring Completed 04/30/12
Total Depth of Boring = 26.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Log of Boring B-102

Figure
A-3

B-103

LAI Project No: 258031.050

SAMPLE DATA

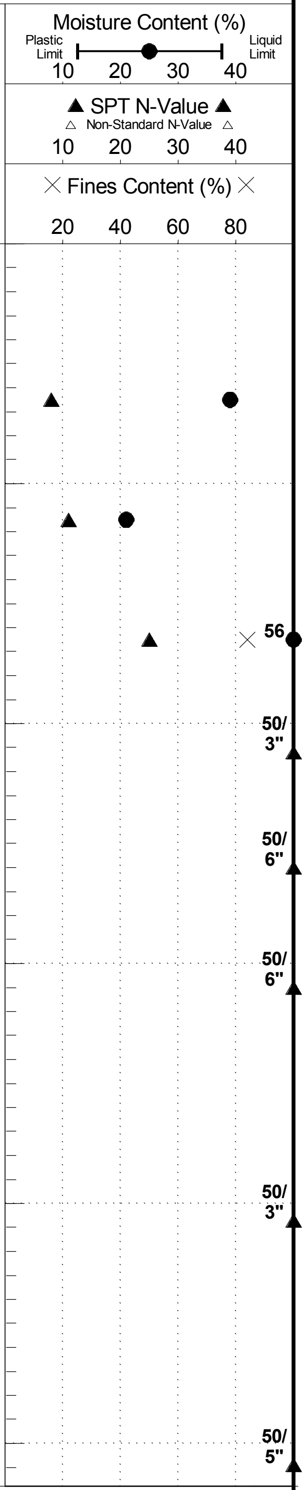
SOIL PROFILE

Drilling Method: Hollow-Stem Auger
Ground Elevation (ft): 23
Drilled By: Holocene Drilling Inc.
Logged By: JDE Date: 05/01/12

Groundwater: 0.0 ft ATD ◁

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
3	20	S-1	b2	8	W = 39
5.5		S-2	b2	11	W = 21
8	15	S-3	b2	25	GS W = 56
10.5		S-4	b2	50/ 3"	
13	10	S-5	b2	50/ 6"	
15.5		S-6	b2	50/ 6"	
20.5		S-7	b2	50/ 3"	
25.5		S-8	b2	50/ 5"	

USCS Symbol	Description
ML	Gray SILT (soft to medium stiff, wet) **(RECESSIONAL LACUSTRINE)**
ML	Gray SILT with sand and gravel (medium stiff, wet)
SP	Gray, fine to medium SAND (medium dense, wet) **(RECESSIONAL OUTWASH)**
ML	Brown, sandy SILT (very stiff, moist)
SM	Gray, very silty, fine SAND (very dense, moist to wet) **(ADVANCE OUTWASH)**
SW	Gray, fine to coarse SAND with gravel (very dense, wet)



Boring Completed 05/01/12
Total Depth of Boring = 25.9 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Sidewalk Improvements
Olympia, Washington

Log of Boring B-103

Figure
A-4

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B-104

LAI Project No: 258031.050

SAMPLE DATA

SOIL PROFILE

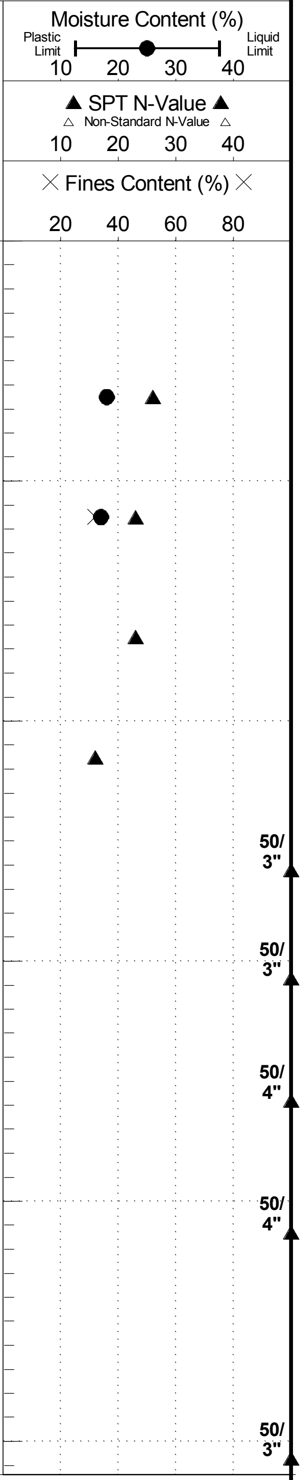
Drilling Method: Hollow-Stem Auger
Ground Elevation (ft): 25
Drilled By: Holocene Drilling Inc.
Logged By: JDE Date: 05/01/12

Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
S-1	b2	26	W = 18
S-2	b2	23	GS W = 17
S-3	b2	23	
S-4	b2	16	
S-5 S-5b	b2	50/ 3"	
S-6	b2	50/ 3"	
S-7	b2	50/ 4"	
S-8	b2	50/ 4"	
S-9	b2	50/ 3"	

Depth (ft): 0, 5, 10, 15, 20, 25
Elevation (ft): 25, 20, 15, 10, 5, 0

USCS Symbol	Description
GW	Gray, fine to coarse GRAVEL (loose to medium dense, moist) **(GRAVEL BASE COURSE)**
SM	Gray, very silty, fine SAND (medium dense, moist) **(RECESSIONAL OUTWASH)**
SP	Gray, fine to medium SAND (medium dense, wet)
ML	Brown, sandy SILT (hard, moist to wet)
SP-SM	Brownish-gray, gravelly, fine to coarse SAND with silt (very dense, wet) **(ADVANCE OUTWASH)**

Groundwater: ▽ 7.0 ft ATD



Boring Completed 05/01/12
Total Depth of Boring = 25.7 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

258031.05 2/25/14 Y:\258031.050\T\258031.050.GPJ SOIL BORING LOG WITH GRAPH

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Log of Boring B-104

Figure
A-5

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B-106

LAI Project No: 258031.050

SAMPLE DATA

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
0	35				
		S-1	b2	7	
5	30				
		S-2	b2	7	
		S-3	b2	9	
10	25				
		S-4	b2	10	
15	20				
		S-5	b2	28	
20	15				
		S-6	b2	52	
25					

SOIL PROFILE

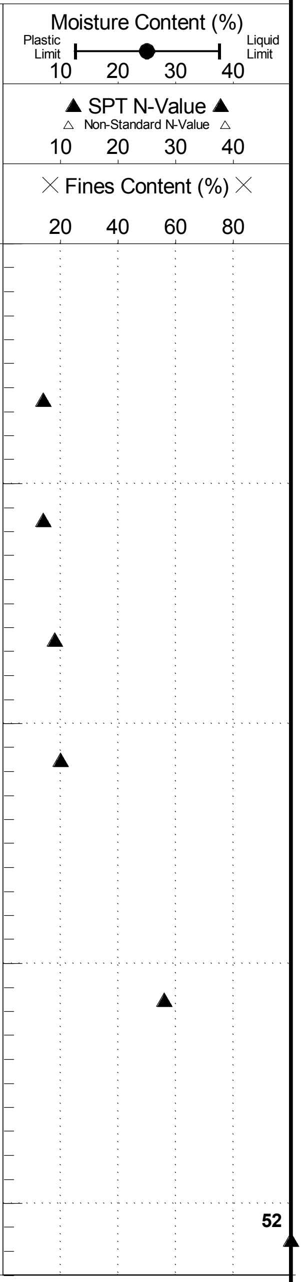
Drilling Method: Hollow-Stem Auger

Ground Elevation (ft): 35

Drilled By: Boretec, Inc.

Logged By: DAR Date: 02/11/14

Graphic Symbol	USCS Symbol	Description	Groundwater
	ML	2 inches of Forest Duff over 5 inches of topsoil (loose, moist) **(TOPSOIL)**	
	ML	Mottled orange/brown SILT (medium stiff, wet) **(RECESSIONAL LACUSTRINE)**	
	ML	Gray SILT (stiff, wet)	▽ ATD
	ML	Gray SILT with sand and gravel (very stiff, wet)	
	SM	Gray, silty, fine to medium SAND (very dense, wet) **(ADVANCE OUTWASH)**	



Boring Completed 02/11/14
Total Depth of Boring = 21.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Sidewalk Improvements
Olympia, Washington

Log of Boring B-106

Figure
A-6

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B-107

LAI Project No: 258031.050

SAMPLE DATA / SOIL PROFILE

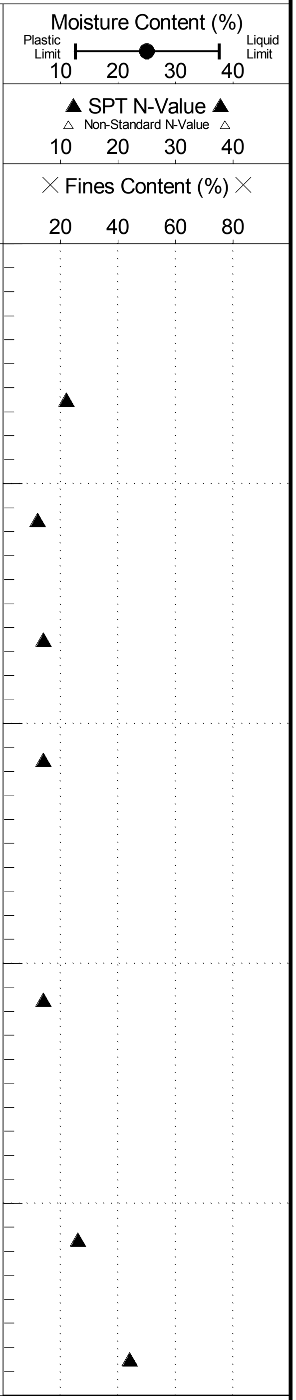
Drilling Method: Hollow-Stem Auger

Ground Elevation (ft): 47

Drilled By: Boretec, Inc.

Logged By: DAR Date: 02/11/14

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Soil Profile	Groundwater
0							ML	4 inches of forest duff over 5 inches of topsoil (soft, moist) **(TOPSOIL)**	
							ML	Mottled orange/brown, SILT (stiff, wet) **(RECESSIONAL LACUSTRINE)**	
	45								
		S-1	b2	11					
5								Grades to medium stiff	
		S-2	b2	6					
	40								
								Sheared surface in sample	
		S-3	b2	7					
10									
		S-4	b2	7					
	35								
15									
		S-5	b2	7					
	30								
20								Grades to stiff	
		S-6	b2	13					
							ML	Gray SILT (very stiff, wet)	
	25								ATD ▽
		S-7	b2	22					
25									



Boring Completed 02/11/14
Total Depth of Boring = 24.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

258031.05 2/25/14 Y:\258031.050\T\258031.050.GPJ SOIL BORING LOG WITH GRAPH

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Log of Boring B-107

Figure
A-7

HA-1

LAI Project No: 258031.050

Drilling Method: Hand Auger

Ground Elevation (ft): Not Measured

Drilled By: Landau Associates

Logged By: JDE   Date: 01/08/14

SAMPLE DATA						SOIL PROFILE				Moisture Content (%) Plastic Limit — Liquid Limit 10 20 30 40 ▲ SPT N-Value ▲ △ Non-Standard N-Value △ 10 20 30 40 × Fines Content (%) × 20 40 60 80
Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol		Groundwater	
0							ML	Light brown SILT with trace roots; some mottling (medium stiff, wet) **(RECESSIONAL LACUSTRINE)**	Groundwater Not Encountered	
2										
4										
6										

Boring Completed 01/08/14
Total Depth of Boring = 4.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Log of Boring HA-1

Figure
A-8

HA-2

LAI Project No: 258031.050

SAMPLE DATA

SOIL PROFILE

Drilling Method: Hand Auger
Ground Elevation (ft): Not Measured
Drilled By: Landau Associates
Logged By: JDE Date: 01/08/14

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
0							ML	Light brown SILT with trace roots; some mottling (medium stiff, wet) **(RECESSIONAL LACUSTRINE)**	Groundwater Not Encountered
2		S-1	d						

Moisture Content (%): Plastic Limit ├─●─┤ Liquid Limit; 10 20 30 40

▲ SPT N-Value ▲; △ Non-Standard N-Value △; 10 20 30 40

× Fines Content (%) ×; 20 40 60 80

Boring Completed 01/08/14
Total Depth of Boring = 3.0 ft.

4

6

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Log of Boring HA-2

Figure
A-9

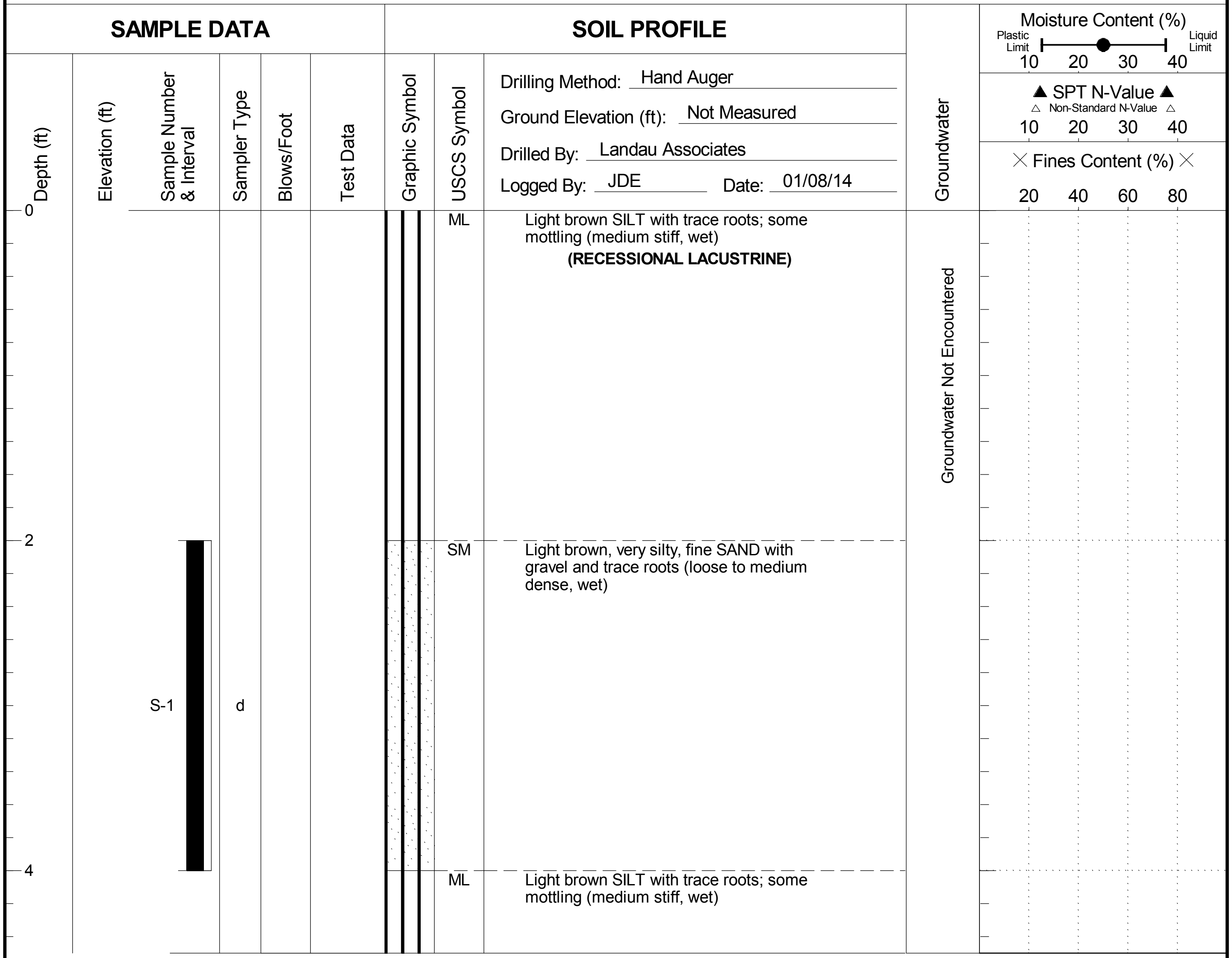
HA-3

LAI Project No: 258031.050

SAMPLE DATA

SOIL PROFILE

Drilling Method: Hand Auger
Ground Elevation (ft): Not Measured
Drilled By: Landau Associates
Logged By: JDE Date: 01/08/14



Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description	Groundwater
0						ML	Light brown SILT with trace roots; some mottling (medium stiff, wet) **(RECESSIONAL LACUSTRINE)**	Groundwater Not Encountered
2		S-1	d			SM	Light brown, very silty, fine SAND with gravel and trace roots (loose to medium dense, wet)	
4						ML	Light brown SILT with trace roots; some mottling (medium stiff, wet)	
6								

Boring Completed 01/08/14
Total Depth of Boring = 4.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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West Bay Drive NW
Sidewalk Improvements
Olympia, Washington

Log of Boring HA-3

Figure
A-10

CP&D RECEIVED March 31, 2014

HA-4

LAI Project No: 258031.050

SAMPLE DATA

SOIL PROFILE

Drilling Method: Hand Auger
Ground Elevation (ft): Not Measured
Drilled By: Landau Associates
Logged By: JDE Date: 01/08/14

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
0							ML	Light brown SILT with trace roots; some mottling (medium stiff, wet) **(RECESSIONAL LACUSTRINE)**	Groundwater Not Encountered
							SM	Light brown, silty, fine SAND (loose, wet)	
2									
		S-1	d				ML	Light brown SILT with trace roots; some mottling (medium stiff, wet)	
4									
							ML	Gray, sandy SILT (medium stiff, wet)	

Moisture Content (%)
Plastic Limit — Liquid Limit
10 20 30 40

▲ SPT N-Value ▲
△ Non-Standard N-Value △
10 20 30 40

× Fines Content (%) ×
20 40 60 80

Boring Completed 01/08/14
Total Depth of Boring = 5.0 ft.

6

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Log of Boring HA-4

Figure
A-11

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HA-5

LAI Project No: 258031.050

SAMPLE DATA

SOIL PROFILE

Drilling Method: Hand Auger

Ground Elevation (ft): Not Measured

Drilled By: Landau Associates

Logged By: JDE
Date: 01/08/14

Moisture Content (%)
Plastic Limit ├──●──┤ Liquid Limit
10 20 30 40

▲ SPT N-Value ▲
△ Non-Standard N-Value △
10 20 30 40

× Fines Content (%) ×
20 40 60 80

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol		Groundwater
0							ML	Light brown SILT with trace roots; some mottling (medium stiff, wet) **(RECESSIONAL LACUSTRINE)**	Groundwater Not Encountered
2									

Boring Completed 01/08/14
Total Depth of Boring = 3.5 ft.

4

6

Notes:
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3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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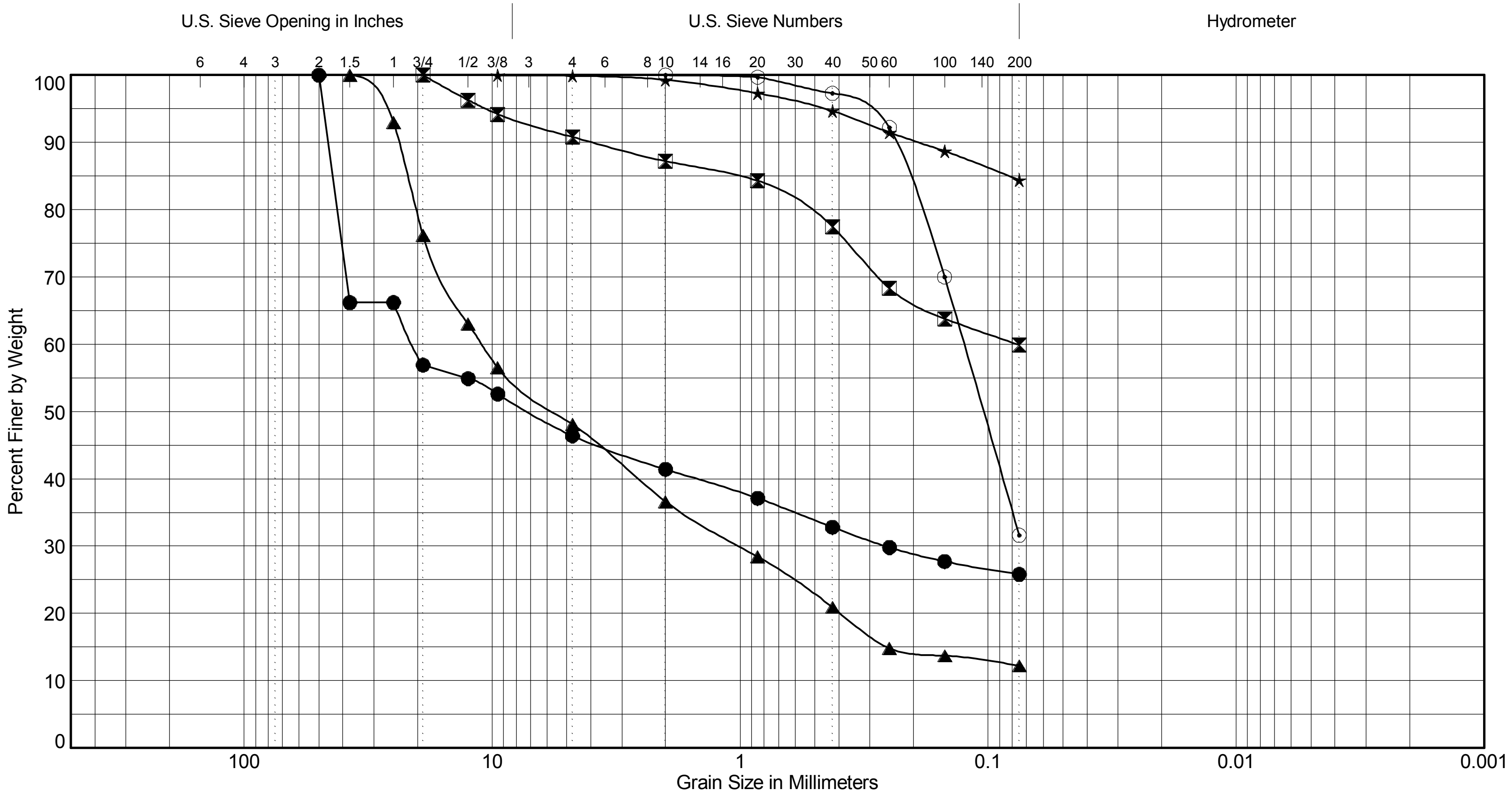
Log of Boring HA-5

Figure
A-12



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-101	S-4	10.0	14	Sandy, silty GRAVEL	GM
⧗	B-102	S-1	2.5	25	Very sandy SILT with gravel	ML
▲	B-102	S-2	5.0	5	Very sandy GRAVEL with silt	GP-GM
★	B-103	S-3	7.5	56	Sandy SILT	ML
⊙	B-104	S-2	5.0	17	Very silty SAND	SM

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Grain Size Distribution

Figure
A-13

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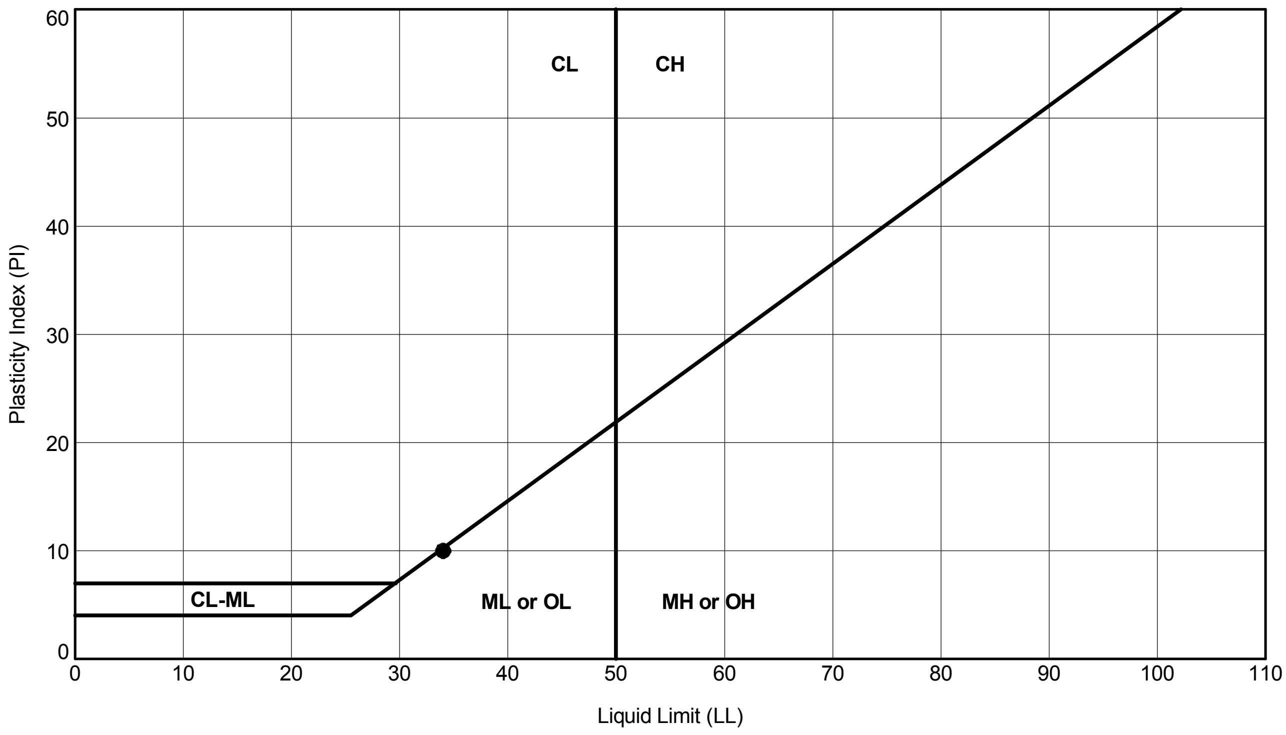

ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-101	S-1	2.5	34	24	10	39	SILT	ML

ASTM D 4318 Test Method

258031.05 2/25/14 Y:\258\031.050\T\258031.050.GPJ ATTERBERG LIMITS FIGURE

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Plasticity Chart

Figure
A-14