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JN 17357

Seattle Land Use Co, LLC
1100 Dexter Avenue North, Suite 275
Seattle, Washington 98109

Attn: Michael Pollard

via email: michael@seattlelanduseco.com

Subject: **Transmittal Letter – Preliminary Geotechnical Engineering Study**
Proposed Mixed-Use Building
104, 110, and 124 – 12th Avenue
Seattle, Washington

Dear Mr. Pollard:

We are pleased to present this preliminary geotechnical engineering report for the mixed-use building to be constructed in Seattle, Washington. The scope of our services consisted of exploring site surface and subsurface conditions, and then developing this report to provide recommendations for general earthwork and design criteria for foundations, retaining walls, and temporary shoring. This work was authorized by your acceptance of our proposal dated June 21, 2017.

The attached report contains a discussion of the study and our recommendations. Please contact us if there are any questions regarding this report, or for further assistance during the design and construction phases of this project.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

James H. Strange, P.E.
Associate

TRC/JHS:mw

PRELIMINARY GEOTECHNICAL ENGINEERING STUDY
Proposed Mixed-Use Building
104, 110, and 124 – 12th Avenue
Seattle, Washington

This report presents the findings and recommendations of our preliminary geotechnical engineering study for the site of the proposed mixed-use building to be located in the International District neighborhood.

Development of the property is in the planning stage, and plans were not made available to us. Based on conversations with our client, we understand that the development will consist of a multi-story mixed-use building with one or two levels of underground parking. Cuts of up to about 24 feet are possible.

If the scope of the project changes from what we have described above, we should be provided with revised plans in order to determine if modifications to the recommendations and conclusions of this report are warranted.

SITE CONDITIONS

SURFACE

The Vicinity Map, Plate 1, illustrates the general location of the site in the International District. The site is bordered to the west by 12th Avenue, to the south by East Yesler Way, to the north by East Fir Street, and to the east by a property with three warehouses that are utilized by King County.

The site is composed of four parcels of land and occupies roughly the western third of a city block.

The north central part of the northern parcel contains a one-story auto shop. The building is masonry, and exhibits some diagonal cracking that are indicative of past differential settlement. The remainder of that parcel is surfaced with pavement or gravel and is utilized for vehicle parking. It is flat to gently sloping down toward the southeast.

The central parcel has a commercial building that covers its west half. The building has one story and a basement that daylights toward the east. The east half of the parcel slopes moderately to gently down toward the south. It is vegetated with grass and blackberry vines that were cut close to the ground.

The southwest parcel is covered with a commercial building that has one story and a basement that daylights toward the east. The walls of the basement level are concrete and the upper walls are brick. We observed some diagonal cracks extending from window corners, which indicate that the building has experienced differential settlement.

The southeastern parcel is a paved parking lot that slopes slightly down toward the south.

We obtained historic street profiles form the City of Seattle. Those show that up to 18 feet of fill were placed at the east side of 12th Avenue adjacent to the subject site. The street profiles also show that a few feet of fill were placed adjacent to the site along East Fir Street and East Yesler Way.

Two warehouses abut the northern two-thirds of the east edge of the property. The west side of the northern warehouse is masonry, and shows some vertical and diagonal cracks that are indicative of differential settlement. The southern warehouse also borders the north edge of the south central part of the site. It appears to be a concrete tilt up structure and does not exhibit cracks.

SUBSURFACE

The subsurface conditions were explored by drilling six test borings at the approximate locations shown on the Site Exploration Plan, Plate 2. Our exploration program was based on the proposed construction, anticipated subsurface conditions and those encountered during exploration, and the scope of work outlined in our proposal.

The borings were drilled on June 30 and July 12 and 13, 2017 using a track- and truck-mounted, hollow-stem auger drill. Samples were taken at approximate 5-foot intervals with a standard penetration sampler. This split-spoon sampler, which has a 2-inch outside diameter, is driven into the soil with a 140-pound hammer falling 30 inches. The number of blows required to advance the sampler a given distance is an indication of the soil density or consistency. A geotechnical engineer from our staff observed the drilling process, logged the test borings, and obtained representative samples of the soil encountered. The Test Boring Logs are attached as Plates 3 through 14.

Soil Conditions

The test borings revealed fill at the ground surface that extended to depths of approximately 2.5 to 16 feet. The fill consisted of silty sand with gravel that was generally loose. Most of the borings encountered up to a few feet of topsoil, peat, or organic silt below the fill. Those materials were followed by layers of silty sand, sand, silt, and clayey silt. Those soils were loose to medium-dense to depths of about 26 to 46 feet, and then were medium-dense to dense or very stiff to hard.

Although our explorations did not encounter cobbles or boulders, they are often found in soils that have been deposited by glaciers or fast-moving water.

Groundwater Conditions

Groundwater seepage was observed at depths of 15 to 50 feet in the test borings, which were left open for only a short time period. Therefore, the seepage levels on the logs represent the location of transient water seepage and may not indicate the static groundwater level. Groundwater levels encountered during drilling can be deceptive, because seepage into the boring can be blocked or slowed by the auger itself. It should be noted that groundwater levels vary seasonally with rainfall and other factors and are generally highest during the normally wet winter and spring months.

The stratification lines on the logs represent the approximate boundaries between soil types at the exploration locations. The actual transition between soil types may be gradual, and subsurface conditions can vary between exploration locations. The logs provide specific subsurface information only at the locations tested. If a transition in soil type occurred between samples in the borings, the depth of the transition was interpreted. The relative densities and moisture descriptions indicated on the test boring logs are interpretive descriptions based on the conditions observed during drilling.

CONCLUSIONS AND RECOMMENDATIONS

GENERAL

THIS SECTION CONTAINS A SUMMARY OF OUR STUDY AND FINDINGS FOR THE PURPOSES OF A GENERAL OVERVIEW ONLY. MORE SPECIFIC RECOMMENDATIONS AND CONCLUSIONS ARE CONTAINED IN THE REMAINDER OF THIS REPORT. ANY PARTY RELYING ON THIS REPORT SHOULD READ THE ENTIRE DOCUMENT.

The test borings conducted for this study encountered up to 16 feet of fill and then native soils that were loose to medium-dense to depths of about 26 to 46 feet. Those marginal soils were followed by more competent soils that were medium-dense to dense or very stiff to hard. It the proposed building is supported with footings bearing on the loose soils it would undergo settlement in excess of typical construction tolerances. Based on our analysis, there is also a potential for the soils that underlie the site to liquefy following a strong seismic event. For these reasons the structure should be supported with deep foundations such as auger-cast piles or drilled piers. Alternatively, ground improvements would reduce the potential for static and dynamic foundation settlement. If that option is selected, the ground should be improved below the entire structure foundation to achieve a reasonably consistent foundation subgrade. Potential vibrations would need to be considered for ground improvements undertaken near adjacent structures, so piling could be required for portions of the foundation even if the majority of the site utilized ground improvements.

We anticipate that shoring will be required around most or all of the site perimeter. The shoring should consist of cantilevered or tied-back soldier piles. The shoring will need to be designed to support surcharge loads of the adjacent warehouses, unless those buildings are underpinned.

Because a specific development scheme has not been determined, the recommendations in this report are only general and should be considered preliminary in nature. We can provide more detailed and specific recommendations after the project plans have been prepared.

The erosion control measures needed during the site development will depend heavily on the weather conditions that are encountered. We anticipate that a silt fence will be needed around the downslope sides of any cleared areas. Existing pavements, ground cover, and landscaping should be left in place wherever possible to minimize the amount of exposed soil. Rocked staging areas and construction access roads should be provided to reduce the amount of soil or mud carried off the property by trucks and equipment. Wherever possible, the access roads should follow the alignment of planned pavements. Trucks should not be allowed to drive off of the rock-covered areas. Cut slopes and soil stockpiles should be covered with plastic during wet weather. Following clearing or rough grading, it may be necessary to mulch or hydroseed bare areas that will not be immediately covered with landscaping or an impervious surface. On most construction projects, it is necessary to periodically maintain or modify temporary erosion control measures to address specific site and weather conditions.

The drainage and/or waterproofing recommendations presented in this report are intended only to prevent active seepage from flowing through concrete walls or slabs. Even in the absence of active seepage into and beneath structures, water vapor can migrate through walls, slabs, and floors from the surrounding soil, and can even be transmitted from slabs and foundation walls due to the concrete curing process. Water vapor also results from occupant uses, such as cooking and bathing. Excessive water vapor trapped within structures can result in a variety of undesirable conditions, including, but not limited to, moisture problems with flooring systems, excessively moist air within occupied areas, and the growth of molds, fungi, and other biological organisms that may be harmful to the health of the occupants. The designer or architect must consider the potential vapor sources and likely occupant uses, and provide sufficient ventilation, either passive or mechanical, to prevent a buildup of excessive water vapor within the planned structure.

Geotech Consultants, Inc. should be allowed to review the final development plans to verify that the recommendations presented in this report are adequately addressed in the design. Such a plan review would be additional work beyond the current scope of work for this study, and it may include revisions to our recommendations to accommodate site, development, and geotechnical constraints that become more evident during the review process.

As with any project that involves demolition of existing site buildings and/or extensive excavation and shoring, there is a potential risk of movement on surrounding properties. This can potentially translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. However, the demolition, shoring, and/or excavation work could just translate into *perceived* damage on adjacent properties. Unfortunately, it is becoming more and more common for adjacent property owners to make unsubstantiated damage claims on new projects that occur close to their developed lots. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring, and/or commencing with the excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners. Additionally, any adjacent structures should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods.

We recommend including this report, in its entirety, in the project contract documents. This report should also be provided to any future property owners so they will be aware of our findings and recommendations.

SEISMIC CONSIDERATIONS

In accordance with the International Building Code (IBC), the site class within 100 feet of the ground surface is best represented by Site Class Type D (Stiff Soil). As noted in the USGS website, the mapped spectral acceleration value for a 0.2 second (S_s) and 1.0 second period (S_1) equals 1.37g and 0.53g, respectively.

The IBC and ASCE 7 require that the potential for liquefaction (soil strength loss) be evaluated for the peak ground acceleration of the Maximum Considered Earthquake (MCE), which has a probability of occurring once in 2,475 years (2 percent probability of occurring in a 50-year period). The MCE peak ground acceleration is adjusted for site class effects (F_{PGA}) and equals 0.56g. The soils beneath the site are susceptible to seismic liquefaction under the ground motions of the MCE.

Sections 1803.5 of the IBC and 11.8 of ASCE 7 require that other seismic-related geotechnical design parameters (seismic surcharge for retaining wall design and slope stability) include the potential effects of the Design Earthquake. The peak ground acceleration for the Design Earthquake is defined in Section 11.2 of ASCE 7 as two-thirds (2/3) of the MCE peak ground acceleration, or 0.38g.

The site is underlain by loose to medium-dense soils that are saturated. Some of these soils have been demonstrated to have a moderate potential for liquefaction during a large earthquake. The current SBC requires that the MCE be used to analyze the potential impacts of seismic liquefaction. Using procedures developed by Seed, Idriss, et al. we calculated the approximate total ground settlement that could result if liquefaction were to occur in the saturated soils as the result of the design earthquake. Based on this analysis, it is probable that soil liquefaction could occur following an MCE. Our calculations indicate that total ground settlement of approximately 6 inches could result during the MCE. The recommended deep foundations or ground improvements would maintain support of the proposed building during the MCE.

FOUNDATION AND RETAINING WALLS

Retaining walls backfilled on only one side should be designed to resist the lateral earth pressures imposed by the soil they retain. The following recommended parameters are for walls that restrain level backfill:

PARAMETER	VALUE
Active Earth Pressure *	40 pcf
Passive Earth Pressure	300 pcf
Coefficient of Friction (Conventional Foundations only)	0.50
Soil Unit Weight	130 pcf

Where: pcf is Pounds per Cubic Foot, and Active and Passive Earth Pressures are computed using the Equivalent Fluid Pressures.

*** For a restrained wall that cannot deflect at least 0.002 times its height, a uniform lateral pressure equal to 10 psf times the height of the wall should be added to the above active equivalent fluid pressure.**

The design values given above do not include the effects of any hydrostatic pressures behind the walls and assume that no surcharges, such as those caused by slopes, vehicles, or adjacent foundations will be exerted on the walls. If these conditions exist, those pressures should be added to the above lateral soil pressures. Where sloping backfill is desired behind the walls, we will need to be given the wall dimensions and the slope of the backfill in order to provide the appropriate design earth pressures. The surcharge due to traffic loads behind a wall can typically be accounted for by adding a uniform pressure equal to 2 feet multiplied by the above active fluid density. Heavy construction equipment should not be operated behind retaining and foundation walls within a distance equal to the height of a wall, unless the walls are designed for the additional lateral pressures resulting from the equipment.

The values given above are to be used to design only permanent foundation and retaining walls that are to be backfilled, such as conventional walls constructed of reinforced concrete or masonry. It is not appropriate to use the above earth pressures and soil unit weight to back-calculate soil strength parameters for design of other types of retaining walls, such as soldier pile, reinforced earth, modular or soil nail walls. We can assist with design of these types of walls, if desired. The passive pressure given is appropriate only for a shear key poured directly against undisturbed native soil, or for the depth of level, well-compacted fill placed in front of a retaining or foundation wall. The values for friction and passive resistance are ultimate values and do not include a safety factor. Restrained wall soil parameters should be utilized for a distance of 1.5 times the wall height from corners or bends in the walls. This is intended to reduce the amount of cracking that can occur where a wall is restrained by a corner.

Wall Pressures Due to Seismic Forces

The surcharge wall loads that could be imposed by the design earthquake can be modeled by adding a uniform lateral pressure to the above-recommended active pressure. The recommended surcharge pressure is 8**H** pounds per square foot (psf), where **H** is the design retention height of the wall. Using this increased pressure, the safety factor against sliding and overturning can be reduced to 1.2 for the seismic analysis.

Retaining Wall Backfill and Waterproofing

Backfill placed behind retaining or foundation walls should be coarse, free-draining structural fill containing no organics. This backfill should contain no more than 5 percent silt or clay particles and have no gravel greater than 4 inches in diameter. The percentage of particles passing the No. 4 sieve should be between 25 and 70 percent. If the native silty sand is used as backfill, a drainage composite similar to Miradrain 6000 should be placed against the backfilled retaining walls. The drainage composites should be hydraulically connected to the foundation drain system. Free-draining backfill or gravel should be used for the entire width of the backfill where seepage is encountered.

The purpose of these backfill requirements is to ensure that the design criteria for a retaining wall are not exceeded because of a build-up of hydrostatic pressure behind the wall. Also, subsurface drainage systems are not intended to handle large volumes of water from surface runoff. The top 12 to 18 inches of the backfill should consist of a compacted, relatively impermeable soil or topsoil, or the surface should be paved. The ground surface must also slope away from backfilled walls to reduce the potential for surface water to percolate into the backfill. Water percolating through pervious surfaces (pavers, gravel, permeable pavement, etc.) must also be prevented from flowing toward walls or into the backfill zone. The compacted subgrade below pervious surfaces and any associated drainage layer should therefore be sloped away. Alternatively, a membrane and subsurface collection system could be provided below a pervious surface.

It is critical that the wall backfill be placed in lifts and be properly compacted, in order for the above-recommended design earth pressures to be appropriate. The wall design criteria assume that the backfill will be well-compacted in lifts no thicker than 12 inches. The compaction of backfill near the walls should be accomplished with hand-operated equipment to prevent the walls from being overloaded by the higher soil forces that occur during compaction. The section entitled ***General Earthwork and Structural Fill*** contains additional recommendations regarding the placement and compaction of structural fill behind retaining and foundation walls.

The above recommendations are not intended to waterproof below-grade walls, or to prevent the formation of mold, mildew or fungi in interior spaces. Over time, the performance of subsurface drainage systems can degrade, subsurface groundwater flow patterns can change, and utilities can break or develop leaks. Therefore, waterproofing should be provided where future seepage through the walls is not acceptable. This typically includes limiting cold-joints and wall penetrations, and using bentonite panels or membranes on the outside of the walls. There are a variety of different waterproofing materials and systems, which should be installed by an experienced contractor familiar with the anticipated construction and subsurface conditions. Applying a thin coat of asphalt emulsion to the outside face of a wall is not considered waterproofing, and will only help to reduce moisture generated from water vapor or capillary action from seeping through the concrete. As with any project, adequate ventilation of basement and crawl space areas is important to prevent a buildup of water vapor that is commonly transmitted through concrete walls from the surrounding soil, even when seepage is not present. This is appropriate even when waterproofing is applied to the outside of foundation and retaining walls. We recommend that you contact an experienced envelope consultant if detailed recommendations or specifications related to waterproofing design, or minimizing the potential for infestations of mold and mildew are desired.

The ***General***, ***Slabs-On-Grade***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

SLABS-ON-GRADE

The building floors can be constructed as slabs-on-grade atop competent native soil, or on structural fill. The subgrade soil must be in a firm, non-yielding condition at the time of slab construction or underslab fill placement. Any soft areas encountered should be excavated and replaced with select, imported structural fill.

Even where the exposed soils appear dry, water vapor will tend to naturally migrate upward through the soil to the new constructed space above it. This can affect moisture-sensitive flooring, cause imperfections or damage to the slab, or simply allow excessive water vapor into the space above the slab. All interior slabs-on-grade should be underlain by a capillary break drainage layer consisting of a minimum 4-inch thickness of clean gravel or crushed rock that has a fines content (percent passing the No. 200 sieve) of less than 3 percent and a sand content (percent passing the No. 4 sieve) of no more than 10 percent. Pea gravel or crushed rock are typically used for this layer. This capillary break/drainage layer is not necessary if an underslab drainage system is installed.

As noted by the American Concrete Institute (ACI) in the *Guides for Concrete Floor and Slab Structures*, proper moisture protection is desirable immediately below any on-grade slab that will be covered by tile, wood, carpet, impermeable floor coverings, or any moisture-sensitive equipment or products. ACI also notes that vapor *retarders* such as 6-mil plastic sheeting have been used in the past, but are now recommending a minimum 10-mil thickness for better durability and long term performance. A vapor retarder is defined as a material with a permeance of less than 0.3 perms, as determined by ASTM E 96. It is possible that concrete admixtures may meet this specification, although the manufacturers of the admixtures should be consulted. Where vapor retarders are used under slabs, their edges should overlap by at least 6 inches and be sealed with adhesive tape. The sheeting should extend to the foundation walls for maximum vapor protection. If no potential for vapor passage through the slab is desired, a vapor *barrier* should be used. A vapor barrier, as defined by ACI, is a product with a water transmission rate of 0.01 perms when tested in accordance with ASTM E 96. Reinforced membranes having sealed overlaps can meet this requirement.

We recommend that the contractor, the project materials engineer, and the owner discuss these issues and review recent ACI literature and ASTM E-1643 for installation guidelines and guidance on the use of the protection/blotter material.

The ***General***, ***Permanent Foundation and Retaining Walls***, and ***Drainage Considerations*** sections should be reviewed for additional recommendations related to the control of groundwater and excess water vapor for the anticipated construction.

EXCAVATIONS AND SLOPES

Excavation slopes should not exceed the limits specified in local, state, and national government safety regulations. Temporary cuts to a depth of about 4 feet may be attempted vertically in unsaturated soil, if there are no indications of slope instability. However, vertical cuts should not be made near property boundaries, or existing utilities and structures. Based upon Washington Administrative Code (WAC) 296, Part N, the soil at the subject site would generally be classified as Type B. Therefore, temporary cut slopes greater than 4 feet in height should not be excavated at an inclination steeper than 1:1 (Horizontal:Vertical), extending continuously between the top and the bottom of a cut. Unless approved by the geotechnical engineer of record, it is important that vertical cuts not be made where the overall depth of the temporary cut slopes is taller than 4 feet.

The above-recommended temporary slope inclination is based on the conditions exposed in our explorations, and on what has been successful at other sites with similar soil conditions. It is possible that variations in soil and groundwater conditions will require modifications to the inclination at which temporary slopes can stand. Temporary cuts are those that will remain unsupported for a relatively short duration to allow for the construction of foundations, retaining walls, or utilities. Temporary cut slopes should be protected with plastic sheeting during wet weather. It is also important that surface runoff be directed away from the top of temporary slope cuts. Cut slopes should also be backfilled or retained as soon as possible to reduce the potential for instability. Please note that sand or loose soil can cave suddenly and without warning. Excavation, foundation, and utility contractors should be made especially aware of this potential danger. These recommendations may need to be modified if the area near the potential cuts has been disturbed in the past by utility installation, or if settlement-sensitive utilities are located nearby.

All permanent cuts into native soil should be inclined no steeper than 2:1 (H:V). Water should not be allowed to flow uncontrolled over the top of any temporary or permanent slope. All permanently exposed slopes should be seeded with an appropriate species of vegetation to reduce erosion and improve the stability of the surficial layer of soil.

TEMPORARY SHORING

This section presents design considerations for cantilevered or tied-back soldier-pile walls. Since the most suitable choice is primarily dependent on a number of factors under the contractor's control, we suggest that the contractor work closely with the structural engineer during the shoring design.

As discussed above, the sensitivity of adjacent buildings and utilities must be considered in the design to reduce the risk of causing settlement of these adjacent elements. Regardless of the system used, all shoring systems will deflect in toward the excavation. Therefore, there is always a risk of noticeable settlement occurring on the ground behind the shoring wall. These risks are reduced, but not entirely eliminated, by using more rigid shoring systems, such as soldier piles.

The shoring design should be submitted to Geotech Consultants, Inc. for review prior to beginning site excavation. We are available and would be pleased to assist in this design effort.

Soldier Pile Installation

Soldier pile walls would be constructed after making planned cut slopes, and prior to commencing the mass excavation, by setting steel H-beams in a drilled hole and grouting the space between the beam and the soil with concrete for the entire height of the drilled hole. The contractor should be prepared to case the holes or use the slurry method if caving soil is encountered. Excessive ground loss in the drilled holes must be avoided to reduce the potential for settlement on adjacent properties. If water is present in a hole at the time the soldier pile is poured, concrete must be tremied to the bottom of the hole.

As excavation proceeds downward, the space between the piles should be lagged with timber, and any voids behind the timbers should be filled with pea gravel, or a slurry comprised of sand and fly ash. Treated lagging is usually required for permanent walls, while untreated lagging can often be utilized for temporary shoring walls. Temporary vertical cuts will be necessary between the soldier piles for the lagging placement. The prompt and careful installation of lagging is important, particularly in loose or caving soil, to maintain the integrity of the excavation and provide safer working conditions. Additionally, care must be taken by the excavator to remove no more soil between the soldier piles than is necessary to install the lagging. Caving or overexcavation during lagging placement could result in loss of ground on neighboring properties. Timber lagging should be designed for an applied lateral pressure of 30 percent of the design wall pressure, if the pile spacing is less than three pile diameters. For larger pile spacings, the lagging should be designed for 50 percent of the design load.

If permanent building walls are to be constructed against the shoring walls, drainage should be provided by attaching a geotextile drainage composite with a solid plastic backing, similar to Miradrain 6000, to the entire face of the lagging, prior to placing waterproofing and pouring the foundation wall. These drainage composites should be hydraulically connected to the foundation drainage system through weep holes placed in the foundation walls.

Soldier Pile Wall Design

Temporary soldier pile shoring that is cantilevered or restrained by one row of tiebacks, and that has a level backslope, should be designed for an active soil pressure equal to that pressure exerted by an equivalent fluid with a unit weight of 35 pounds per cubic foot (pcf). To design temporary tied-back shoring with more than one row of tiebacks and a level backslope, we recommend assuming that the lateral active soil pressure on the wall, expressed in pounds per square foot (psf), is equal to 22**H**, where **H** is the total height of the excavation in feet.

Traffic surcharges can typically be accounted for by increasing the effective height of the shoring wall by 2 feet. Existing adjacent buildings will exert surcharges on the proposed shoring wall, unless the buildings are underpinned. Slopes above the shoring walls will exert additional surcharge pressures. These surcharge pressures will vary, depending on the configuration of the cut slope and shoring wall. We can provide recommendations regarding slope and building surcharge pressures when the preliminary shoring design is completed.

If tieback easements cannot be obtained, it may be necessary to utilize internal braces (rakers) to restrain the soldier piles. Soldier piles restrained by rakers can undergo more deflection than do tied-back or cantilever piles, due to the temporary sloped excavation that is necessary in front of the soldier piles to install the rakers and thrust blocks. This requires that the shoring designer closely evaluate the temporary conditions that exist before raker installation. We should be contacted early in the design process if rakers are necessary, in order to provide the appropriate design considerations.

It is important that the shoring design provides sufficient working room to drill and install the soldier piles, without needing to make unsafe, excessively steep temporary cuts. Cut slopes should be planned to intersect the backside of the drilled holes, not the back of the lagging.

Lateral movement of the soldier piles below the excavation level will be resisted by an ultimate passive soil pressure equal to that pressure exerted by a fluid with a density of 300 pcf. No safety factor is included in the given value. This soil pressure is valid only for a level excavation in front of the soldier pile; it acts on two times the grouted pile diameter. Cut slopes made in front of shoring walls significantly decrease the passive resistance. This includes temporary cuts necessary to install internal braces or rakers. The minimum embedment below the floor of the excavation for cantilever soldier piles should be equal to the height of the "stick-up." Tied-back soldier piles should be embedded no less than 10 feet below the lowest point of the excavation, including footing and utility excavations.

The vertical capacity of soldier piles to carry the downward component of the tieback forces will be developed by a combination of frictional shaft resistance along the embedded length and pile end-bearing.

PARAMETER	*DESIGN VALUE*
Pile Shaft Friction	750 psf
Pile End-Bearing	5,000 psf

Where: psf is Pounds per Square Foot.

The above values assume that the excavation is level in front of the soldier pile and that the bottom of the pile is embedded a minimum of 10 feet below the floor of the excavation. For the pile end-bearing to be appropriate, the bottom of the drilled holes must be cleaned of loosened soil. The shoring contractor should be made aware of this, as it may affect their installation procedures. The concrete surrounding the embedded portion of the pile must have sufficient bond and strength to transfer the vertical load from the steel section through the concrete into the soil.

TIEBACK ANCHORS

We recommend installing tieback anchors at inclinations between 20 and 30 degrees below horizontal. The tieback will derive its capacity from the soil-grout strength developed in the soil behind the no-load zone. The minimum grouted anchor length should be 10 feet. The no-load zone is the area behind which the entire length of each tieback anchor should be located. To prevent excessive loss-of-ground in a drilled hole, the no-load section of the drilled tieback hole should be backfilled with a sand and fly ash slurry, after protecting the anchor with a bond breaker, such as plastic casing, to prevent loads from being transferred to the soil in the no-load zone. The no-load section could be filled with grout after anchor testing is completed.

During the design process, the possible presence of foundations or utilities close to the shoring wall must be evaluated to determine if they will affect the configuration and length of the tiebacks.

Based on the results of our analyses and our experience at other construction sites, we suggest using an adhesion value of 500 psf in the loose to medium-dense soil to design temporary anchors, if the mid-point of the grouted portion of the anchor is more than 10 feet below the overlying ground surface. This value applies to non-pressure-grouted anchors. Pressure-grouted or post-grouted anchors can often develop adhesion values that are two to three times higher than that for non-pressure-grouted anchors. These higher adhesion values must be verified by load testing.

Soil conditions, soil-grout adhesion strengths, and installation techniques typically vary over any site. This sometimes results in adhesion values that are lower than anticipated. Therefore, we recommend substantiating the anchor design values by load-testing all tieback anchors. At least two anchors in each soil type encountered should be performance-tested to 200 percent of the design anchor load to evaluate possible anchor creep. Wherever possible, the no-load section of these tiebacks should not be grouted until the performance tests are completed. Unfavorable results from these performance tests could require increasing the lengths of the tiebacks. The remaining anchors should be proof-tested to at least 135 percent of their design value before being "locked off." After testing, each anchor should be locked off at a pre-stress load of 80 to 100 percent of its design load.

If caving or water-bearing soil is encountered, the installation of tieback anchors will be hampered by caving and soil flowing into the holes. It will be necessary to case the holes, if such conditions are encountered. Alternatively, the use of a hollow-stem auger with grout pumped through the stem as the auger is withdrawn would be satisfactory, provided that the injection pressure and grout volumes pumped are carefully monitored.

All drilled installations should be grouted and backfilled immediately after drilling. No drilled holes should be left open overnight.

EXCAVATION AND SHORING MONITORING

As with any shoring system, there is a potential risk of greater-than-anticipated movement of the shoring and the ground outside of the excavation. This can translate into noticeable damage of surrounding on-grade elements, such as foundations and slabs. Therefore, we recommend making an extensive photographic and visual survey of the project vicinity, prior to demolition activities, installing shoring or commencing excavation. This documents the condition of buildings, pavements, and utilities in the immediate vicinity of the site in order to avoid, and protect the owner from, unsubstantiated damage claims by surrounding property owners.

Additionally, the shoring walls, and any adjacent foundations should be monitored during construction to detect soil movements. To monitor their performance, we recommend establishing a series of survey reference points to measure any horizontal deflections of the shoring system. Control points should be established at a distance well away from the walls and slopes, and deflections from the reference points should be measured throughout construction by survey methods. At least every other soldier pile should be monitored by taking readings at the top of the pile. Additionally, benchmarks installed on the surrounding buildings should be monitored for at least vertical movement. We suggest taking the readings at least once a week, until it is established that no deflections are occurring. The initial readings for this monitoring should be taken before starting any demolition or excavation on the site.

DRAINAGE CONSIDERATIONS

We anticipate that permanent foundation walls will be constructed against the shoring walls. Where this occurs, a plastic-backed drainage composite, such as Miradrain, Battledrain, or similar, should be placed against the entire surface of the shoring prior to pouring the foundation wall. Weep pipes located no more than 6 feet on-center should be connected to the drainage composite and poured into the foundation walls or the perimeter footing. A footing drain installed along the inside of the perimeter footing will be used to collect and carry the water discharged by the weep pipes to the storm system. Isolated zones of moisture or seepage can still reach the permanent wall where groundwater finds leaks or joints in the drainage composite. This is often an acceptable risk in unoccupied below-grade spaces, such as parking garages. However, formal waterproofing is typically necessary in areas where wet conditions at the face of the permanent wall will not be tolerable. If this is a concern, the permanent drainage and waterproofing system should be designed by a specialty consultant familiar with the expected subsurface conditions and proposed construction.

Footing drains placed inside the building or behind backfilled walls should consist of 4-inch, perforated PVC pipe surrounded by at least 6 inches of 1-inch-minus, washed rock wrapped in a non-woven, geotextile filter fabric (Mirafi 140N, Supac 4NP, or similar material). At its highest point, a perforated pipe invert should be at least 6 inches below the level of a crawl space or the bottom of a floor slab, and it should be sloped slightly for drainage. Plate 15 presents typical considerations for footing drains and a typical shoring drain detail is on Plate 16. All roof and surface water drains must be kept separate from the foundation drain system.

If the structure includes an elevator, it may be necessary to provide special drainage or waterproofing measures for the elevator pit. If no seepage into the elevator pit is acceptable, it will be necessary to provide a footing drain and free-draining wall backfill, and the walls should be waterproofed. If the footing drain will be too low to connect to the storm drainage system, then it will likely be necessary to install a pumped sump to discharge the collected water. Alternatively, the elevator pit could be designed to be entirely waterproof; this would include designing the pit structure to resist hydrostatic uplift pressures.

Underslab drainage should also be provided where (1) a crawl space or slab will slope or be lower than the surrounding ground surface, (2) an excavation encounters significant seepage, or (3) an excavation for a building will be close to the expected high groundwater elevations. We can provide recommendations for interior drains, should they become necessary, during excavation and foundation construction.

As a minimum, a vapor retarder, as defined in the ***Slabs-On-Grade*** section, should be provided in any crawl space area to limit the transmission of water vapor from the underlying soils. Crawl space grades are sometimes left near the elevation of the bottom of the footings. As a result, an outlet drain is recommended for all crawl spaces to prevent an accumulation of any water that may bypass the footing drains. Providing even a few inches of free draining gravel underneath the vapor retarder limits the potential for seepage to build up on top of the vapor retarder.

Groundwater was observed during our field work. If seepage is encountered in an excavation, it should be drained from the site by directing it through drainage ditches, perforated pipe, or French drains, or by pumping it from sumps interconnected by shallow connector trenches at the bottom of the excavation.

The excavation and site should be graded so that surface water is directed off the site and away from the tops of slopes. Water should not be allowed to stand in any area where foundations, slabs, or pavements are to be constructed. Final site grading in areas adjacent to a building should slope away at least 2 percent, except where the area is paved. Surface drains should be provided where necessary to prevent ponding of water behind foundation or retaining walls. A discussion of grading and drainage related to pervious surfaces near walls and structures is contained in the ***Foundation and Retaining Walls*** section. Water from roof, storm water, and foundation drains should not be discharged onto slopes; it should be tightlined to a suitable outfall located away from any slopes.

GENERAL EARTHWORK AND STRUCTURAL FILL

All building and pavement areas should be stripped of surface vegetation, topsoil, organic soil, and other deleterious material. It is important that existing foundations be removed before site development. The stripped or removed materials should not be mixed with any materials to be used as structural fill, but they could be used in non-structural areas, such as landscape beds.

Structural fill is defined as any fill, including utility backfill, placed under, or close to, a building, behind permanent retaining or foundation walls, or in other areas where the underlying soil needs to support loads. All structural fill should be placed in horizontal lifts with a moisture content at, or near, the optimum moisture content. The optimum moisture content is that moisture content that results in the greatest compacted dry density. The moisture content of fill is very important and must be closely controlled during the filling and compaction process.

The allowable thickness of the fill lift will depend on the material type selected, the compaction equipment used, and the number of passes made to compact the lift. The loose lift thickness should not exceed 12 inches. We recommend testing the fill as it is placed. If the fill is not sufficiently compacted, it can be re-compacted before another lift is placed. This eliminates the need to remove the fill to achieve the required compaction.

The following table presents recommended relative compactions for structural fill:

LOCATION OF FILL PLACEMENT	MINIMUM RELATIVE COMPACTION
Beneath slabs or walkways	95%
Filled slopes and behind retaining walls	90%
Beneath pavements	95% for upper 12 inches of subgrade; 90% below that level

Where: Minimum Relative Compaction is the ratio, expressed in percentages, of the compacted dry density to the maximum dry density, as determined in accordance with ASTM Test Designation D 1557-91 (Modified Proctor).

Structural fill that will be placed in wet weather should consist of a coarse, granular soil with a silt or clay content of no more than 5 percent. The percentage of particles passing the No. 200 sieve should be measured from that portion of soil passing the three-quarter-inch sieve.

LIMITATIONS

The conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our exploration and assume that the soil and groundwater conditions encountered in the test borings are representative of subsurface conditions on the site. If the subsurface conditions encountered during construction are significantly different from those observed in our explorations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. Unanticipated conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking samples in test borings. Subsurface conditions can also vary between exploration locations. Such unexpected conditions frequently require making additional expenditures to attain a properly constructed project. It is recommended that the owner consider providing a contingency fund to accommodate such potential extra costs and risks. This is a standard recommendation for all projects.

This report has been prepared for the exclusive use of Seattle Land Use Co, LLC and its representatives for specific application to this project and site. Our conclusions and recommendations are professional opinions derived in accordance with our understanding of current local standards of practice, and within the scope of our services. No warranty is expressed or implied. The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design. Our services also do not include assessing or minimizing the potential for biological hazards, such as mold, bacteria, mildew and fungi in either the existing or proposed site development.

ADDITIONAL SERVICES

In addition to reviewing the final plans, Geotech Consultants, Inc. should be retained to provide geotechnical consultation, testing, and observation services during construction. This is to confirm that subsurface conditions are consistent with those indicated by our exploration, to evaluate whether earthwork and foundation construction activities comply with the general intent of the recommendations presented in this report, and to provide suggestions for design changes in the event subsurface conditions differ from those anticipated prior to the start of construction. However, our work would not include the supervision or direction of the actual work of the contractor and its employees or agents. Also, job and site safety, and dimensional measurements, will be the responsibility of the contractor.

During the construction phase, we will provide geotechnical observation and testing services when requested by you or your representatives. Please be aware that we can only document site work we actually observe. It is still the responsibility of your contractor or on-site construction team to verify that our recommendations are being followed, whether we are present at the site or not.

The following plates are attached to complete this report:

Plate 1	Vicinity Map
Plate 2	Site Exploration Plan
Plates 3 - 14	Test Boring Logs
Plate 15	Typical Footing Drain Detail
Plate 16	Typical Footing Drain Detail

We appreciate the opportunity to be of service on this project. Please contact us if you have any questions, or if we can be of further assistance.

Respectfully submitted,

GEOTECH CONSULTANTS, INC.

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REGISTERED
PROFESSIONAL ENGINEER
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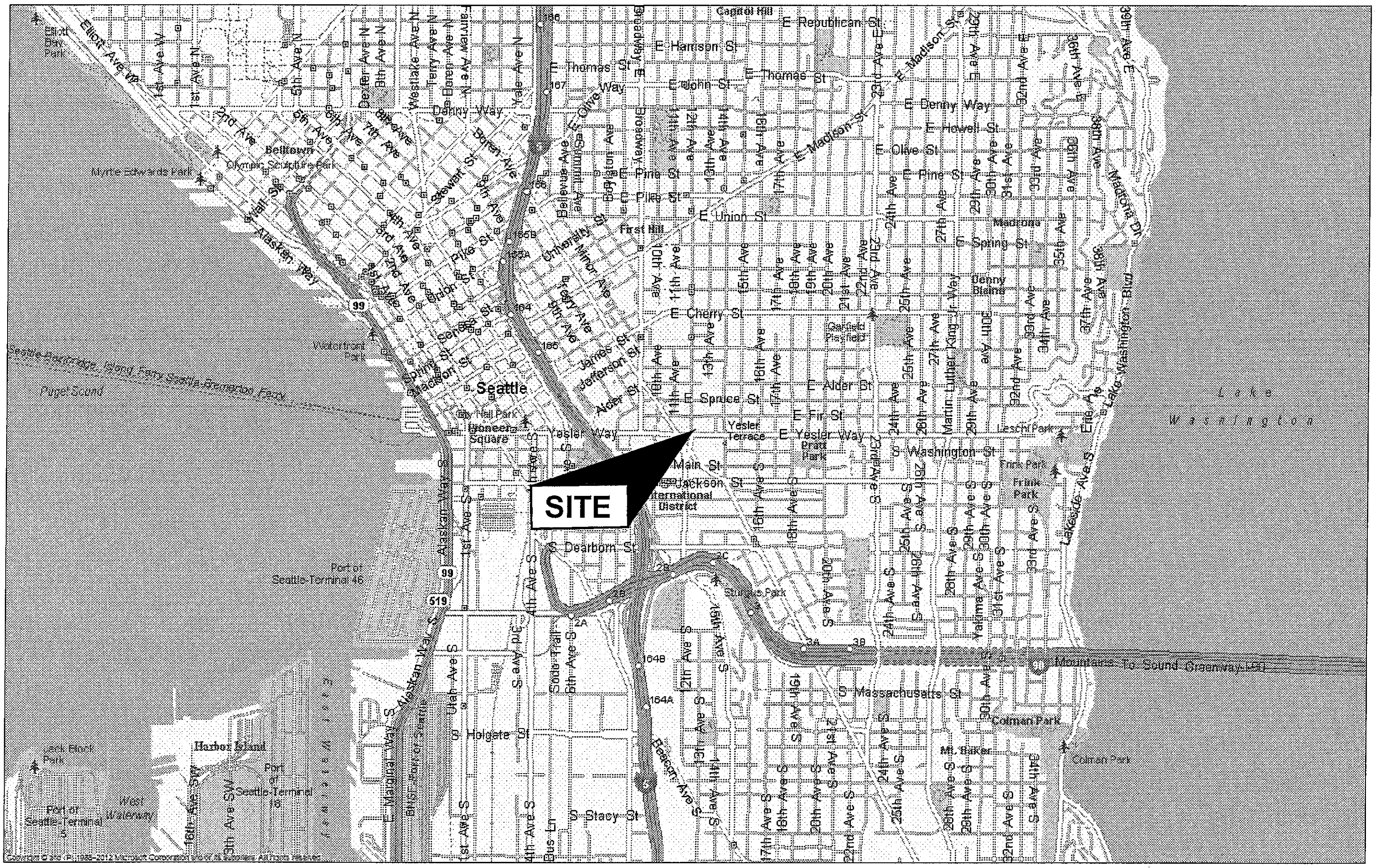
NORTH



(Source: Microsoft MapPoint, 2013)

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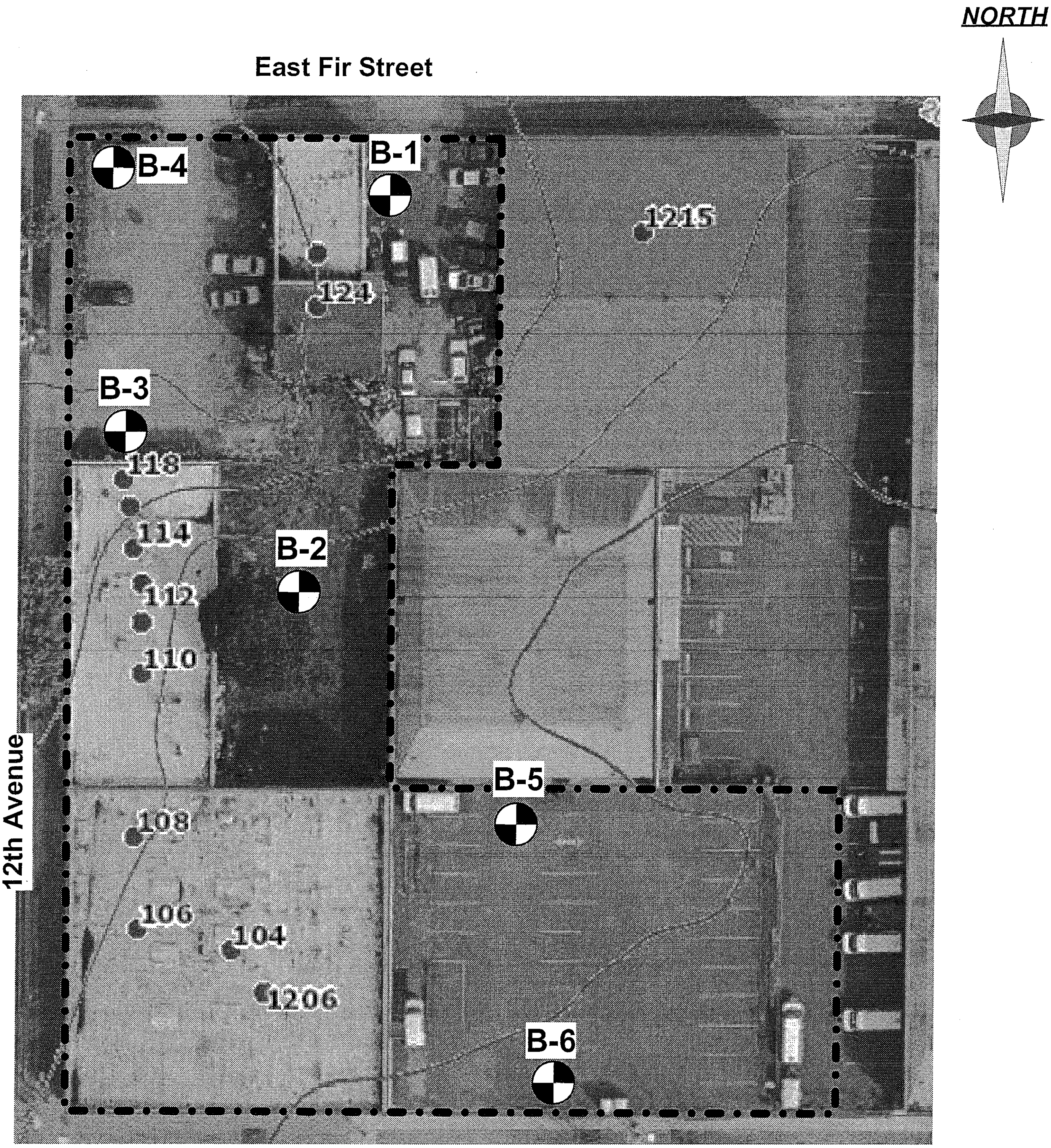
VICINITY MAP

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job No:	Date:		Plate:
17357	July 2017		1



Legend:

Test boring location

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SITE EXPLORATION PLAN

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job No:	Date:		Plate:
17357	July 2017	No Scale	2

BORING 1

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Asphalt over; Gray-brown silty SAND with gravel, fine to coarse-grained, moist, loose **(Fill)**
5			9	1		
10			9	2		-with organics and a piece of brick
15			18	3	SM	Rust-brown mottled gray silty SAND with gravel, fine to coarse-grained, moist to very moist, medium-dense
20			25	4		-becomes gray, with a 3 inch seam of wet sand
25			9	5		-becomes loose and very moist
30						

CONTINUED ON PLATE 4

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	*Date:*	*Logged by:*	*Plate:*
17357	July 2017	TRC	3

BORING 1 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
30		▼	26	6	SW	Gray SAND, fine to coarse-grained, wet, medium-dense
35			31	7	ML/SP	Gray SILT to SAND, non-plastic, very fine-grained, wet, dense
40						

*** Test boring was terminated at 36.5 feet on June 30, 2017.**
*** Groundwater was encountered at 32 feet during drilling.**

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	4

BORING 2

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
0					FILL	Topsoil over; Brown silty SAND with gravel, fine to coarse-grained, moist, loose **(Fill)**
5			4	1		
~6						Topsoil
10			15	2	SM	Brown-mottled gray SAND with silt, fine to coarse-grained, very moist, medium-dense
15		▼	14	3		-becomes silty and gray-brown
20			8	4	SM ML	Gray-brown silty SAND to sandy SILT, non-plastic, fine to medium-grained, very moist, loose
25			9	5		-becomes wet and gray
30						

CONTINUED ON PLATE 6

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	*Date:*	*Logged by:*	*Plate:*
17357	July 2017	TRC	5

BORING 2 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
30			7	6	ML	Gray sandy clayey SILT, plastic, fine to medium-grained, very moist, medium stiff
35			13	7	SW	Gray SAND, fine to medium-grained, moist, medium-dense
40			50/ 3"	8		
			46	9	ML	Gray clayey SILT, plastic, very moist, hard -refusal on a rock
45						
50						

*** Test boring was terminated at 43.5 feet on June 30, 2017.**
*** Groundwater was encountered at 15 feet during drilling.**

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	6

BORING 3

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Gravel over; Brown silty SAND with gravel, fine to coarse-grained, moist, loose **(Fill)**
			4	1		
5						
			13	2		-becomes medium-dense
10						
			7	3		-becomes loose and gray
					PT	Dark brown PEAT, wet, soft
15		▼				
			11	4	SM	Gray silty SAND with gravel, fine to coarse-grained, very moist, loose to medium-dense
20						
			11	5		
25						
			11	6	SW	Gray SAND, fine to coarse-grained, wet, loose to medium-dense
30						

CONTINUED ON PLATE 8

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	*Date:*	*Logged by:*	*Plate:*
17357	July 2017	TRC	7

BORING 3 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
30						
			22	7	ML	Gray clayey SILT, plastic, very moist, very stiff
35						
			19	8		
40						
			13	9	SP	Gray SAND, fine-grained, very moist, medium-dense
45						
			41	10	ML	Gray clayey SILT, plastic, very moist, hard
50						
55						

*** Test boring was terminated at 49 feet on July 12, 2017.**
*** Groundwater was encountered at 16 feet during drilling.**

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	8

BORING 4

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Gravel over; Concrete rubble Brown silty SAND with gravel, fine to coarse-grained, moist, loose **(Fill)**
			6	1		
5						
			3	2		-becomes very loose
10						
					PT	Dark brown PEAT, wet, very soft
			6	3	SM	Gray silty SAND with gravel, fine to coarse-grained, very moist, loose
15						
		▼				
			20	4		-becomes medium-dense and very moist to wet, reduced silt in some zones
20						
			23	5	SP	Gray SAND, fine-grained, very wet, medium-dense
25						
			19	6		
					ML	Gray clayey SILT, plastic, very moist, very stiff
30						

CONTINUED ON PLATE 10

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	*Date:*	*Logged by:*	*Plate:*
17357	July 2017	TRC	9

BORING 4 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
30						
35			20	7	ML	Gray clayey SILT, plastic, very moist, very stiff
40			18	8		
45			7	9	SP	Gray SAND, fine-grained, very wet, loose
50			46	10	SM/ML	Gray silty SAND to sandy SILT, very fine-grained, non-plastic, wet, dense
55						
60						

*** Test boring was terminated at 49 feet on July 12, 2017.**
*** Groundwater was encountered at 17 feet during drilling.**

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	10

BORING 5

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Asphalt over; Brown to gray-brown silty SAND with gravel, fine to coarse-grained, moist to very moist, very loose **(Fill)**
			3	1		
5						
			21	2		-becomes medium-dense and moist to wet, reduced silt in some zones
10						
			5	3		-becomes loose
15						
			31	4	SP	Gray SAND, fine-grained, moist, dense
20						
			12	5	SM	Gray silty SAND, fine to medium-grained, moist, medium-dense
25						
			12	6	ML	Gray SILT to sandy SILT, non-plastic, very moist, medium-dense
30						

CONTINUED ON PLATE 12

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	11

BORING 5 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
30–35			9	7	ML	Gray SILT to sandy SILT, non-plastic, fine-grained, very moist, loose
35–40			41	8	SW	Gray SAND, fine to coarse-grained, wet, dense
40–54			10	9	ML	Gray clayey SILT, plastic, very moist, stiff
			18	10		-becomes very stiff
50		▼				
			24	11		

* **Test boring was terminated at 54 feet on July 13, 2017.**
* **Groundwater was encountered at 50 feet during drilling.**

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	12

BORING 6

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
					FILL	Asphalt over; Gray-brown silty SAND with gravel, fine to coarse-grained, very moist, medium-dense **(Fill)**
			5	1		Dark brown organic SILT, very moist, loose
5					SM	Gray-brown silty SAND, fine to coarse-grained, moist, loose
			15	2		-becomes medium-dense, reduced silt in sone zones
10						
			6	3		-becomes loose
15						
			9	4		
20						
			11	5		-becomes medium-dense and very moist
25						
			16	6	ML	Gray clayey SILT, plastic, very moist, stiff
30						

CONTINUED ON PLATE 14

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TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	13

BORING 6 (CONTINUED)

Depth (ft.)	Moisture	Water Table	Blows per Foot	Sample	USCS	Description
30–36	37		7	▮	ML	Gray clayey SILT, plastic, very moist, very stiff
36–39	37	▼ (37 ft)	8	▮	SP	Gray SAND, fine-grained, very wet, loose

*** Test boring was terminated at 39 feet on July 13, 2017.**
*** Groundwater was encountered at 37 feet during drilling.**

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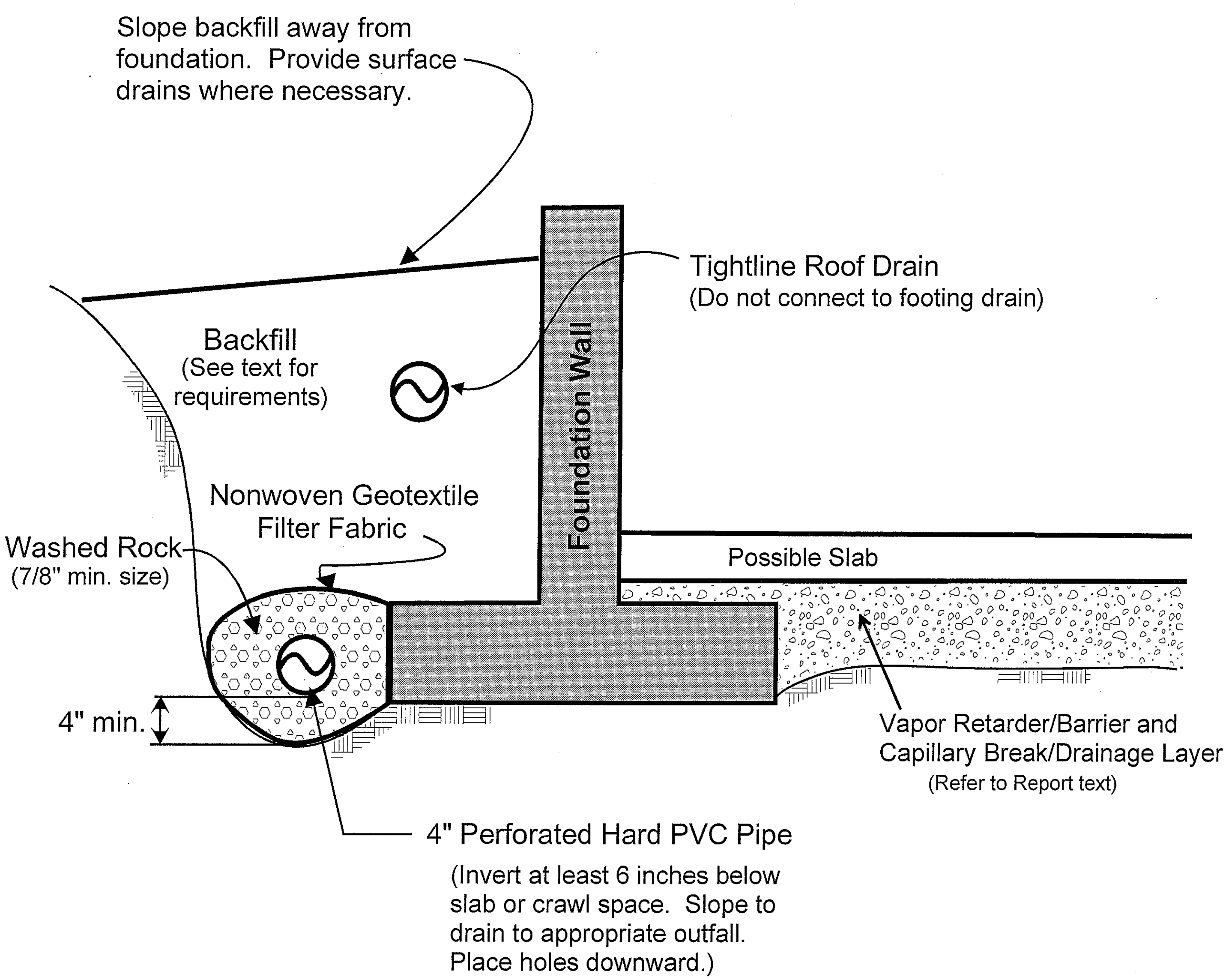
TEST BORING LOG

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job	Date:	Logged by:	Plate:
17357	July 2017	TRC	14



NOTES:

(1) In crawl spaces, provide an outlet drain to prevent buildup of water that bypasses the perimeter footing drains.

(2) Refer to report text for additional drainage, waterproofing, and slab considerations.

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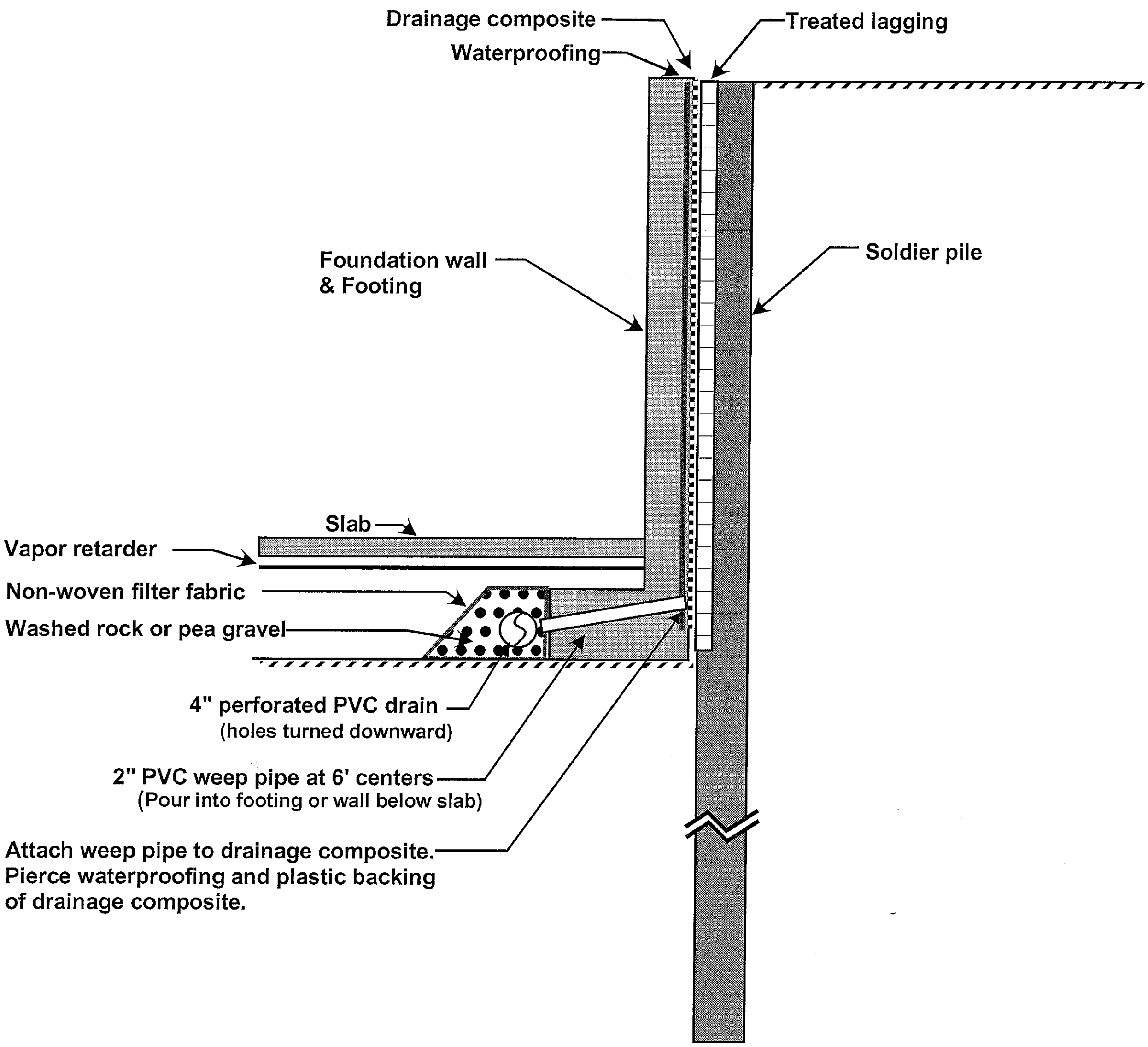
FOOTING DRAIN DETAIL

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job No:	Date:		Plate:
17357	July 2017		15



<u>Note</u> - Refer to the report for additional considerations related to drainage and waterproofing.

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SHORING DRAIN DETAIL

104, 110, and 124 - 12th Avenue
Seattle, Washington

Job No:	Date:		Plate:
17357	July 2017		16