Geotechnical Report
Ballard Mixed Use Facility
5214 Ballard Avenue NW
Seattle, Washington

Project 1466-01
July 17, 2007

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1.0 INTRODUCTION

The Galli Group performed a geotechnical investigation on the property located at 5214 and 5216 Ballard Avenue NW, in Seattle, Washington. The purpose of our investigation was to identify the subsurface soil conditions on the site and to provide recommendations for earthwork and foundation support on the site.

This geotechnical report summarizes observations from our research and subsurface exploration performed for the above referenced property. It also presents our recommendations for the geotechnical design elements of the project.

2.0 PROJECT DESCRIPTION

The project site is located between Ballard Avenue NW and 20th Avenue NW in the Ballard area of Seattle (see Vicinity Map, Figure 1). The lot accesses both 20th Avenue and Ballard Avenue, and is bounded on the South, North and East by existing buildings. The site currently contains a single story structure and an older two-story garage that will be demolished as part of the proposed improvements. The planned development includes a mixed-use facility for retail, commercial, and residential housing. Underground parking is planned with access from 20th Avenue NW. Utilities are provided from Ballard and 20th Avenue NW.

We understand that the proposed improvements call for construction of a four-story structure with three levels of below-grade parking. The structure will be supported on conventional spread footings founded in the dense soils at the base of the excavation. The adjacent buildings will be underpinned during the shoring operation with soldier piles. The shoring will consist of soldier pile shoring with two levels of tiebacks.

3.0 SITE FEATURES

3.1 GEOLOGY AND EXISTING CONDITIONS

3.1.1 Existing Conditions

The existing site is relatively flat grading gradually toward the southwest from 20th Avenue NW to Ballard Avenue NW a total elevation drop of about 2 feet. A two-story brick building and single-story wood framed building are located on the site. The remainder of the property serves as temporary storage and parking.

The building to the north and east (Olympic Athletic Club) is supported on spread footings founded approximately 8 feet below existing grade. The adjacent building includes a basement with concrete slab. The building to the south appears supported on a spread footing founded on the dense soil approximately 4 feet below existing grade. The southeasterly existing brick building ground floor is wood-framed above a crawl space.

3.1.2 Site Geologic Conditions

Geologic maps of the area indicate that the vicinity is underlain by glacial till deposits (*Preliminary Geology of Seattle and Vicinity*, Waldron, 1962). Glacial till typically consists of an unsorted mixture of silt, sand, clay and gravel that was pushed over the existing landscape in front of the advancing glacier thousands of years ago. A unique aspect of glacial till is its ability to stand unsupported in near vertical relief.

Bedded or braided deposits of advance outwash often underlie the glacial till unit. Glacial outwash was deposited in meltwater environments in front of the advancing glacier thousands of years ago. Subsequently the glacial outwash was overridden by tons of ice and thus tends to appear very dense. Glacial outwash tends to consist of sand and pebbly gravel that appear in layers or beds with varying permeability. These layers can transmit seasonal perched or permanent groundwater toward lower topographic elevations or open excavations. We anticipate that the proposed foundation support will be within the dense glacial outwash. The existing adjacent buildings are likely supported on the glacial till unit.

3.1.3 Seismic Design Parameters

The nearest mapped fault is approximately 5 miles south of the project site. We do not anticipate surficial faults or ground rupture to be a factor on this site.

The site appears underlain by glacially consolidated silty SAND and very dense SAND with silt. Permanent water table will likely be at or near the proposed base of excavations however the soils appear very dense at that depth. Based upon these site factors seismic liquefaction does not appear to be a significant concern. The risk of seismically induced slope movement is not significantly increased with the proposed construction and does not represent a significant threat to the project site.

Based upon the latitude and longitude of the site we consulted the USGS Seismic Hazards Maps and estimated the site coefficients for an event with 2 percent probability of exceedance in 50 years. In conformance with the 2003 International Building Code (Section 1615) the following design parameters should be used for the project site:

TABLE 1
Seismic Site Coefficients

Site Class	S_l Spectral Acceleration (1 second period)	F_v Site Coefficient	S_s Spectral Acceleration (0.2 second period)	F_a Site Coefficient
C	0.45	1.35	1.31	1.0

F_v from Table 1615.1.2(2) 2003 Seattle Building Code
F_a from Table 1615.1.2(1)

3.2 SITE SOIL AND GROUNDWATER CONDITIONS

The Galli Group conducted a subsurface investigation of the project site. We also researched borings accomplished for site development on a nearby site[1] to develop a more comprehensive assessment of the site soil and groundwater conditions. We observed the exposed soil conditions in the open excavation as the above project was being constructed and discussed the soil conditions with the excavating contractor of the Leary Avenue project. The results of our investigation and research are provided in the sections below.

3.2.1 Subsurface Exploration

On May 14, 2007, we conducted a subsurface investigation of the site. Using a truck-mounted hollow stemmed auger we advanced 3 borings to depths varying from 46.5 to 56.5 feet sampling every 5 feet. Using an automatic trip hammer we performed Standard Penetration Tests for each sample, dropping a 140-lb hammer 30 inches and recording the number of blows for each 6 inches of penetration. Samples were retrieved and logged in the field by a geotechnical engineer. The results of our drilling and sampling program are provided in the Appendix, Figures A-1 to A-8.

On May 21, 2007, we returned to drill an additional boring in the vicinity of the existing garage. Using a portable Acker Soil Mechanic we advanced one boring, B-4, to 24 feet depth and retrieved samples at periodic intervals. Samples were retrieved and logged in the field by an engineer from our office. The results of that boring are provided on Figure A-8 in the appendix.

The location of the borings is shown on Figure 2, Site Sketch. Representative samples from our subsurface exploration were taken to a laboratory for natural moisture content testing and grain size analyses. The results of the laboratory testing are provided in the Appendix.

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[1] *Draft Geotechnical Report Jefferson at Ballard Project, Seattle, Washington*, April 6, 1999, by GeoEngineers

A monitoring well was installed in Boring B-2 in order to allow for ongoing monitoring of the water level at the site. The well was constructed using 2-inch diameter slotted PVC pipe from 20 to 30 feet below grade. The pipe was capped at the bottom and the hole was sealed below the pipe and above the slotted pipe from 18 feet to grade. A monument cap was placed at grade. A diagram of the well construction is provided on Figure A-4.

3.2.2 Subsurface Soil Conditions
Based upon our subsurface investigation the site appears underlain by three general soil units. The uppermost unit from grade to about 6 or 9 feet below grade appeared comprised of loose to medium dense silty SAND with bits of organics or bricks. We described this unit as undocumented fill. The second unit from depths of about 4 feet to about 21 feet below grade appeared as very dense, silty SAND with trace amounts of fine gravel. We interpreted this unit as glacial till. Underlying the glacial till at about 21 feet below grade in all of the borings we encountered a unit of medium SAND with silt, interbedded with layers of silty SAND and occasional gravel lenses. We interpreted this unit, the deepest unit encountered in our explorations as glacial outwash.

The generalized soil stratigraphy of the site shown across two cross sections is depicted on Figures 3 and 4. Generally the site appears by very dense glacial till to a depth of about 21 feet from existing grade. The till appears underlain by glacial outwash. The entire site might be blanketed by fill to depths on the order of about 4 to 6 feet.

3.2.3 Groundwater Conditions
We encountered groundwater in all of the borings on the site. The driller used water or drilling mud to avoid heave of the sample rod and auger making precise determination of the depth to water table difficult to assess. The depth of the water at time of drilling is provided in Figure 2 below.

TABLE 2
Depth of Water at time of Drilling

Boring Number	Depth at Time of Drilling	Depth in Monitoring Well	Comments
B-1	15 feet	NA	perched?
B-1	31 feet	NA	outwash layer
B-2	26.5 feet	10.5 feet (piezo) 21 feet	Could bail well to depth of 21 feet (see text)
B-3	23 feet	NA	outwash layer
B-4	20 feet	NA	estimated

On June 14, 2007, we returned to the site to bail the well in Boring B-2. The piezometric water level was at 10.5 feet below grade. We were able to bail the well to a depth of 20.8 feet before we could not bail the well fast enough to keep up with the recharge of the well. Based upon these results, we concluded that the permanent water table is likely at about 21 feet within the glacial outwash unit. The water on the site is likely fed from higher elevations and therefore appears with piezometric head evidenced in the static position of the water table in the well. However, we could bail the well to near the contact depth of the advance outwash unit indicating that the infiltration rate is not excessive.

We anticipate that permanent groundwater will be found at depths on the order of about 21 feet below grade with bands of seepage possibly appearing in the glacial till unit. The primary seepage is likely to be within the glacial outwash unit. Construction dewatering will be required on the site. Settlement of adjacent structures does not appear to be a significant concern because of the dense consistency of the outwash unit. Because of the interbedded nature of the glacial outwash unit, the rates of seepage might vary across the site. The water layers might also be perched within the outwash unit.

We discussed the excavating operations with the excavating contractor for the construction just north between Leary and Ballard Avenue. That site appeared to have similar soil conditions to this project site. The contractor indicated that they were able to dewater the site using numerous sump pumps. The infiltration rate was not sufficient to cause collapse of the soils on that site, and the seepage rate varied across the site. They utilized a series of small sump pumps that discharged to a collection sump in the base of the 23-foot excavation. The collection sump discharged to a sedimentation tank prior to discharging to the storm drain.[2]

The contractor might choose to use well points to dewater the excavation. Wellpoint systems are most useful for dewatering excavations where the water table does not need to be lowered more than about 18 feet below the installation level of the wellpoint system.

The contractor should be responsible for construction dewatering and selecting an appropriate dewatering system that will enable the work to be completed. Seepage rates and means of dewatering can vary significantly from site to site in the same region as well as across the same small site. We have provided the above comments and recommendations so that the contractor can anticipate what to expect on this job site. However, the selection of means and methods should be up to the contractor based upon field conditions.

4.0 CONCLUSIONS

Based upon our subsurface investigation and review of available information from nearby projects, the project site appears underlain by very dense, silty SAND (glacial till) that is underlain by fine to medium SAND with silt (glacial outwash.) The upper few feet of the site appears blanketed with medium dense silty SAND and undocumented fill. Adjacent buildings appear founded on the dense glacial till unit. We anticipate that new footings will likely be founded in the dense glacial outwash unit. We anticipate that groundwater will be encountered at about 21 feet depth and that construction dewatering will be required. Permanent dewatering of the parking garage will also be necessary.

[2] Phone conversation with Bill Spooner, Spooner Excavating, LLC, June 18, 2007, regarding 5440 Leary Ave NW.

The following elements should be addressed during design and construction of the proposed project:

0. *Temporary Shoring with Tiebacks.* The depth of the excavation will be approximately 26 feet from grade. New walls will be immediately adjacent to the property line requiring temporary shoring. We anticipate that a soldier pile shoring system with at least two rows of tiebacks will be required for the project.
0. *Underpinning Adjacent Structures.* Adjacent structures are supported on spread footings. These structures will need to be underpinned by the soldier pile shoring system.
0. *Construction Dewatering.* We anticipate that the significant seepage will be enoucntered during site excavation at depths on the order of 21 feet. The contractor should be prepared to select a method and conduct construction dewatering of the lower reaches of the excavation and the footings.
0. *Permanent Dewatering.* Due to the presence of year-round seepage within the outwash unit, the below-grade garage and elevator pit will need to be permanently dewatered.
0. *Construction of Spread Footings.* The proposed structure will be supported on spread footings at the base of the excavation. These might be constructed in wet conditions requiring dewatering and careful control of the footing subgrade.
0. *Construction of Permanent Retaining Walls.* Permanent braced retaining walls supported on spread footings will be constructed against the shoring system. Lateral earth pressures are provided for design of the walls in our report.
0. *Temporary Erosion Control.* Site development will require considerable mass excavation and transport of spoils. Erosion control measures are recommended in the report.
0. *Stormwater quality control.* The contractor will also be required to collect and discharge seepage and stormwater runoff from the site. The contractor will need to monitor water quality of the discharge and arrange for appropriate permits.

5.0 RECOMMENDATIONS

The site appears underlain by glacial outwash at depth and blanketed by a unit of very dense glacial till. We anticipate that the spread footings will be supported in the outwash unit and that groundwater seepage will be encountered at the lower depths of the excavation. Temporary shoring with tiebacks will be required to support the excavation and to underpin adjacent structures.

The sections below address these geotechnical issues and other aspects of site development for the proposed project. Provided the recommendations provided in this report are followed during design and construction of the residence, the proposed construction may proceed safely under appropriate geotechnical supervision.

5.1 SITE GRADING AND EARTHWORK

5.1.1 Temporary Erosion Control Measures

Site development will result in a large excavation footprint within glacial till and outwash soils. Much of the excavated soil will consist of silty SAND that can become unworkable when wet or disturbed by equipment traffic. Best Management Practices commonly observed should be employed during construction. We anticipate these will include the following:

0. Mass excavation will require construction of a ramp into the excavation and loading of trucks within the right-of-way. Arrangements should be made with SDOT for necessary permits for use of the right-of-way.

0. It is important to avoid tracking sediment onto the roadway. The contractor should monitor the tracking of sediment from the site and clean up as necessary. Sand and silt tracked from the site should be removed or cleaned by the contractor.

0. Collected stormwater runoff or seepage should be discharged into a tank or basin to remove sediment and suspended material. The contractor will likely need to collect the water and pump it into a portable poly tank such as provided by Baker Tanks to provide for sediment removal and to decrease turbidity. This is discussed in more detail in Section 5.1.4.

0. Spoils should be removed immediately from the site or protected during wet weather by use of plastic sheeting (BMP E1.20). Generally stockpiles should not remain uncovered for more than 2 days during the wet season or 5 days during the drier summer months.

0. The contractor should monitor the performance of the erosion control measures and contact the geotechnical engineer if the TESC measures do not provide the intended function.

5.1.2 Construction of Working Pad

The site soils consist of silty SAND and SAND with silt. When undisturbed, the material is expected to perform fairly well for temporary excavations and with respect to sediment transport. However, once the soil is disturbed it could become unworkable and retard construction operations. In order to decrease the transport of material off site, to maintain water quality control of water leaving the site, and to expedite construction work, the contractor might consider constructing a working pad.

The working pad should consist of imported gravel placed over the native undisturbed soil. The subgrade should first be graded so that it sheds water toward one end of the footprint where a collection sump can collect and discharge water. The soil bearing should be verified by the geotechnical engineer prior to placing the imported fill. The imported sand and gravel should meet City of Seattle standard specification 9-03.11 for clean crushed gravel (Type 21 or Type 22). It should be placed and compacted according to the compaction criteria described in Section 5.6 of this report. Areas prepared for footings should be excavated through the working pad immediately prior to forming footings and soil bearing should be verified by the geotechnical engineer. The working pad should be at least 4 inches thick and should be at least 12 inches thick in areas anticipated to receive heavy equipment traffic.

5.1.3 Construction Staging

The contractor should submit a construction staging schedule that demonstrates how the excavation can progress while maintaining erosion and water quality control. Shoring will have to be installed from within the building footprint, and then access provided for equipment during installation of the lagging and forming the footings. The approach is left up to the contractor, but the contractor must demonstrate to the geotechnical engineer on site that the concerns regarding erosion control, sediment transport, and stormwater quality are addressed throughout the project.

5.1.4 Construction Stormwater Water Quality Control

Because the site is located near receiving water bodies it is imperative that the contractor be vigilant about erosion control and monitor the water quality of runoff leaving the site. We anticipate that the water from seepage or stormwater runoff may be handled by means of shallow trenches and sumps. Standard BMP measures such as silt fencing and straw barriers will help reduce sediment transport, but turbidity remains a concern.

We recommend that collected runoff be temporarily stored in a sump, settling basin, or tank so that the suspended silt and sediment may have adequate time to settle out. Because of the limited space we recommend using a poly tank to collect the runoff. The collection and discharge of the collected water should be timed so that turbidity may be sufficiently reduced. The water should then be discharged from the tank to a suitable outlet point. The contractor must arrange with SPU (Seattle Public Utilities) for appropriate permits for discharge of construction dewatering. Storage of the poly tank should be arranged with SDOT for appropriate permits and use of the right-of-way.

If this treatment does not appear to adequately remove turbidity and suspended soils arrangements for additional measures such as sand filters or the introduction of flocculants and discharge into either the storm or sanitary sewer might be required. The potential for this increases if excavation is conducted during wet weather or after October 31. The contractor should be aware of this possibility. The turbidity levels of water leaving the site must satisfy the requirements dictated by the City of Seattle or SPU during the review process.

5.2 TEMPORARY EXCAVATIONS AND SHORING

5.2.1 Unsupported Excavations

Temporary excavations should be shaped or benched to protect workers below and adjacent buildings. The entire excavation will be shored on this site. Temporary excavations will be limited to those performed prior to installing the piles and those conducted during installation of the lagging. No excavation below the base of the adjacent buildings should be permitted without installation of the shoring system. Maintaining safe excavations shall be the ongoing responsibility of the contractor.

5.2.2 Soldier Pile Shoring System

We recommend the following for the design of the shoring system:

1) The cantilevered soldier pile shoring system should consist of drilled shafts, grouted and reinforced with steel beams. The shafts shall consist of structural concrete placed up to at least the bottom of the proposed excavation and lean mix concrete above that grade. As the site is excavated, the lean mix is chipped away and lagging is placed between the flanges of the H-beams.

2) The drilled piers should be advanced to a depth sufficient to provide support for the anticipated excavation depth. The structural engineer should provide detailed calculations for piles after selecting precise locations, height, spacing, diameter of the hole, reinforcing elements, and material strength.

3) The contractor shall select the means and method to install the drilled piers. We anticipate that either drilling mud or casing will be needed to excavate the piers below water table. The holes will also likely need to be installed at a slight batter or oversized in some locations in order to place the piles at the desired location.

4) Based upon the SPT values obtained during drilling, and the consistency of the underlying soil encountered, we have recommended earth pressures as shown in Figure 5 for design of the shoring system. Forces above the base of the excavation should be considered to act on the spacing of the piles. Below the excavation, passive forces should be considered to act on 2 pile diameters. The bottom of the perimeter footings shall be considered to be the base of the excavation.

5) Surcharge loads within 12 feet of the wall such as traffic loads, material stockpiles, equipment or structure loads should be included in the design of the wall. It does not appear that adjacent structures will impact the loading on the wall since they will be underpinned and supported by the piles.

6) The steel shall be adequately sized to resist the lateral forces during each stage of the excavation. Critical stages tend to be when the excavation is just below the installation of the first row of anchors, and immediately prior to the installation of the second row of anchors. A schematic showing the lateral earth pressures at each stage is provided in Figure 6 and Figure 7, Construction Stage Earth Pressure Diagrams.

7) A pressure equivalent to 70 percent of the design active pressure may be used to size the timber lagging, provided the pile spacing does not exceed 8 feet. Pressure treated timber lagging should be placed as excavation proceeds. Voids behind the lagging should be filled with free draining material such as pea gravel or a clean sand slurry. Maximum height of the exposed cut should not exceed 4 feet before placing lagging. Lagging should be completed to the base of the excavation at then end of every working day.

8) The ultimate axial capacity of a 24-inch diameter shaft embedded 10 feet below the base of the excavation should be on the order of 142 tons. Applying a factor of safety of three, the working load or design capacity will be 95 kips per pile.

The Galli Group should review the design and should monitor the installation of the soldier pile shoring system. Please contact us if you need additional clarification regarding the intent of this section. We understand that the shoring system will not be used to support the permanent walls and that the permanent walls will be designed independent of the soldier pile shoring system.

The design of the shoring wall and the piles shall be the responsibility of the structural engineer, utilizing the design parameters provided in this report. Additional requirements related to concrete strength, grout, reinforcing elements, construction monitoring, and material specifications should be provided by the structural engineer.

5.2.3 Shoring Wall Tiebacks

Construction of the shoring system will require installation of temporary tiebacks. The tiebacks can be spaced at each soldier pile or spaced as needed using a waler across the face of the soldier piles. Because of space constraints we anticipate that the tiebacks will be at the location of the soldier piles. We recommend the following for design of the tiebacks:

0) Anchors must extend beyond the "no load" zone shown on Figure 5. The no load zone consists of the area immediately behind the shoring wall described as beginning at a point H/4 (where H is the height of the wall) beyond the base of the excavation and extending upward at 60 degrees away from the wall toward the existing grade. Typically a minimum embedment of 15 feet beyond the no load zone is required.

0) The anchors should be inclined downward at 15 to 30 degrees from horizontal. The inclination can be determined by the structural engineer and architect as needed to avoid utility conflicts and other site-specific criteria.

0) Anchors can be either strand anchors or bars. Selection is up to the contractor. The design load shall not exceed 60 percent of the specified minimum tensile strength (SMTS) of the prestressing. Lock-off load shall not exceed 70 percent of the SMTS and the maximum test load shall not exceed 80 percent of the SMTS. The steel in the anchors shall be at least 150 ksi steel. If the selection of the anchor type is different from the plan documents or requires a different anchor assembly than shown on the plans, the contractor must submit shop drawings to the structural engineer prior to installing the anchorage system.

0) The structural engineer shall provide the anchor head assembly detail including trumpet and connection to the pile. Web stiffeners may be required at the connection between the pile and the anchor head assembly.

0) All anchors should be designed to withstand at least 150 percent of the design load. A performance test shall be conducted on the first two production anchors and then on at least 3 percent of the remaining anchors. Details regarding the performance test are provided in the section below. The remaining anchors should be proof tested.

5.2.4 Tieback Testing

A minimum of two of the first production anchors shall be performance tested and thereafter at least 3 percent of the anchors shall be performance tested. The contractor shall be responsible for supplying the testing equipment. The geotechnical engineer shall monitor the testing. A performance test shall be conducted as follows:

Performance Test

0) An alignment load (AL) no more than 5 percent of the design load shall be applied to the anchor and the displacement equipment zeroed thereafter.
0) The anchors shall be loaded in 25 percent increments of the design load with the incremental movement of the anchor recorded at each loading cycle. Following each incremental load, the anchor load is reduced to the alignment load.
0) The anchor shall be reloaded in increasing increments until the test load is reached.
0) The test load shall be 133 percent of the design load. The load must be held for ten minutes with movements recorded at 1, 2, 3, 4, 5, 6, and 10 minutes. The geotechnical engineer must monitor the performance testing.
0) Reduce the load to the design load and lock off the anchor.

Each additional anchor not subjected to a performance test shall be proof tested. The proof test shall be conducted as follows:

Proof Test

0) An initial alignment load shall be applied to the anchor and gauges adjusted to zero. The alignment load shall not exceed 5 percent of the design load (DL.)
0) Successively apply and record total movements for the following load increments: 0.25DL, 0.50DL, 0.75DL, 1.00DL, 1.20DL, and 1.33DL. The test load shall be 133 percent of the design load.
0) Hold the test load for 10 minutes and record total movement.
0) At the discretion of the geotechnical engineer, he or she might require some of the anchors to be unloaded to the alignment load to record residual movement.
0) Reduce the load on anchors that pass the acceptance criteria to the lock-off load.

Acceptance Criteria

Each performance tested anchor shall be considered acceptable if it passes the following:

- Creep of the anchor shall not exceed 1mm (0.045 inches) between 1 and 10 minutes. If it does not pass this test then the creep test shall be extended to 60 minutes. The anchor shall be considered acceptable if the total movement over the interval from 6 to 60 minutes does not exceed 2 mm, or 2mm per log cycle of time.

Lock-off Load and Lift-off Testing
The anchors shall be locked off at 80 percent of the design load. After the load has been transferred from the jack to the anchorage, the contractor should perform a lift-off test to verify the magnitude of the loaded anchor. The anchor shall be gradually stressed until the wedge plate lifts off the bearing plate. If the load measured during lift-off should be within five percent of the lock-off load. If this criterion is not met, the anchor load should be adjusted and the lift-off test repeated.

5.2.5 Monitoring of Shoring System Performance
The contractor shall provide a monitoring program to evaluate the performance of the shoring system and the impact of the excavation and dewatering on adjacent structures and property. We recommend that horizontal and vertical survey points be established on the shoring piles and adjacent structures. A licensed surveyor should establish the coordinates of the points and read the points at the following times: 1) prior to commencing excavation, 2) weekly during dewatering and excavation activity, and 3) prior to commencing backfilling or construction of the retaining walls. If deflection of the piles exceeds ½ inch then more frequent readings might be required. The results of the performance monitoring should be supplied to the structural engineer and geotechnical site inspector.

In addition we recommend documenting existing conditions of the adjacent building walls and footings by digital camera prior to commencing excavation and again prior to backfilling or construction of the walls. This helps protect all parties involved in the process.

5.2.6 Additional Contractor Responsibility
Numerous shoring methods and approaches are available to the contractor even within the parameters of an owner-designed system. Means and methods for installation of the system are up to the contractor. Our recommendations are presented to provide the project owners and contractor with information regarding soils, groundwater, and type of shoring needed to protect adjacent properties, as well as level of effort needed to construct the shoring system. The contractor shall be responsible to review the factual data presented in this report, perform any additional investigations and tests deemed necessary to characterize the soil and groundwater conditions on the site, and to develop independent conclusions regarding the design, construction, and execution of the shoring, excavation, and dewatering systems.

5.3 LATERAL EARTH PRESSURES AND RETAINING WALLS

The proposed site development plan includes a three-story below grade parking garage. The garage includes retaining walls that are 6 inches away from the property lines and formed immediately adjacent to the temporary shoring system. The shoring system is designed as a temporary system. Therefore the retaining walls must resist the permanent lateral earth pressures. We anticipate that the proposed wall will be a braced wall with intermediate slabs bracing across the excavation.

The table below provides soil parameters used in the analyses for this project.

TABLE 3

Soil design parameters used in determination of lateral earth pressures

Soil Type	Unit Weight pcf	Passive Resistance (EFW)	Active Earth Pressure (EFW)	At-Rest Earth Pressure (EFW)
Loose Fill	115	200 pcf	35 pcf	55 pcf
Dense silty SAND	130	250 pcf	35 pcf	55 pcf
Dense outwash	130	250 pcf	35 pcf	55 pcf
Compacted Fill	125	250 pcf	35 pcf	

(EFW) = Equivalent Fluid unit Weight in pounds per cubic foot

We anticipate that the walls will be supported on spread footings within the glacial outwash unit. Care must be taken that the proposed footings are formed on undisturbed soil or properly prepared footing areas since groundwater seepage might result in sedimentation of the excavated footing areas.

5.3.1 Lateral Earth Pressures for Permanent Retaining Walls

The earth pressure diagram for design of the permanent walls is provided on Figure 8, Earth Pressure Diagrams for Permanent Retaining Walls. We recommend the following for design of the retaining walls.

0. All walls must be supported on native undisturbed soil. We recommend using an allowable bearing capacity of 3,500 psf for design.
0. For braced walls or restrained walls, a lateral at-rest earth pressure of 55 pcf should be used for design of the walls.
0. Lateral resistance for basement retaining walls may be calculated at 250 pcf per foot of overburden.

5.3.2 Wall Drainage

A backwall drainage system must be supplied for the walls. The drainage system shall include sheet drains that are fastened to the lagging of the shoring wall. The sheet drains must connect to a collection drain at the base of the wall or be allowed to discharge through the wall at each bay of the shoring system. The discharge should be through a PVC pipe above the footing and below the proposed slab. A collection system beneath the slab should connect the wall drains and route them to the permanent drainage system.

5.4 FOUNDATIONS

The proposed structure will be supported on shallow spread footings within the excavation. We anticipate that these footings will be excavated from within the excavation once shoring is complete. The sections below address specific recommendations for the shallow foundations.

5.4.1 Spread Footings and Wall Footings
Column or wall loads within the excavation may be supported on spread footings. For spread footings within the excavation we recommend the following:

0. An allowable bearing pressure of 3,500 psf may be used for footings bearing on undisturbed glacial soil. Allowable bearing of 2,000 psf should be used for footings on structural fill. This may be increased by 1/3 for temporary loads such as wind loads or seismic loads.

0. The footing area must be free from loose or wet soil prior to placing reinforcing or pouring concrete. The geotechnical engineer should verify the bearing.

5.4.2 Footings Constructed in Wet Conditions
Wet conditions within the excavation might require modifications to typical construction practices for the footings. Sump pumps might need to be supplied immediately following excavation. In order to preserve the base of the footings, clean crushed rock or 2"-4" quarry spalls or lean mix concrete might need to be placed in the base of the footing to preserve the footing soils and prevent sedimentation while forming and steel reinforcement is completed. The means and method of preserving the bearing capacity of the soil surface is up to the contractor, but all footings must be poured against an unyielding, dense subgrade.

5.4.3 Subsurface Drainage Recommendations
All backwall drainage systems and slab drains should drain via tightline pipe to a suitable collection point or discharge system. Because of seepage conditions anticipated near the base of the excavation, we anticipate that many of the footings will be poured in wet conditions. Construction dewatering will be important to maintain the performance of the subgrade soils. Permanent subslab drains are also recommended in order to prevent wet conditions in the elevator pits and lower level of the parking garage.

We recommend installing subslab drains and/or a drainage blanket beneath the proposed garage slab in order to prevent wet conditions and hydrostatic loading of the basement slab. The drainage blanket consists of a geotextile filter fabric placed against the subgrade or working pad at or near the base of the footings. The subgrade should be shaped to direct water toward a collection point or PVC perforated pipe should be placed to collect water and direct it to the collection point. Above the filter fabric a layer of clean crushed rock is placed to form the drainage blanket. The collected water is pumped from a sump to the construction discharge point and then later to the storm drain.

An alternative to the subslab drainage blanket a system of trenches with perforated PVC pipe and backfilled with clean crushed rock could be constructed that collects the seepage from the excavation and directs it to the collection sump.

5.5 SLAB-ON-GRADE FLOORS

Reinforced concrete floors that are beneath structures ringed with perimeter footings or walls can be supported on a 6-inch drain rock layer placed over properly prepared subgrade soils. For slabs on grade, we recommend that granular import be placed as soon as the subgrade is prepared to protect the subgrade soil. If the subslab drainage blanket is installed, the drain rock layer is not required. The vapor barrier and reinforcing of the slab should be according to the plans.

5.6 BACKFILL AND COMPACTION REQUIREMENTS

5.6.1 Drain Rock

Material intended for drainage material beneath slabs or for footing drains shall be clean ¾"-1¼" crushed rock with less than 2 percent passing the No. 200 sieve. Ballast intended for placement as a drainage mat or for protection of footing areas must be smaller than 2½ inches in maximum size and contain less than 2 percent passing the No. 100 sieve (WSDOT 9-03.9(2).

5.6.2 Structural Fill Material

Structural fill material intended for placement beneath structures, footings, or pavement shall consist of granular material less than 4 inches in maximum size and free of debris and organic content. Structural fill should contain less than 7 percent passing the No. 200 sieve. All fill material should be placed in thin enough lifts to achieve uniform compaction throughout the entire lift. The compaction criteria are described below.

5.6.3 Compaction Recommendations

Imported fill soils used as backfill behind walls and under slabs should be moisture conditioned to within 3 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and compacted to at least 92 percent of the maximum dry density, as determined using ASTM D1557 (Modified Proctor). The 92 percent compaction criteria should apply to any material intended to support pavement or intended as backfill behind walls. If structures are planned to be supported on the structural fill the compaction criteria should be 95 percent of the Modified Proctor. In areas not constructed as fill slopes or not intended to support pavement or structures, fill material should be placed in loose lifts less than 12 inches in thickness and compacted to at least 90 percent of the maximum dry density.

5.7 PERMANENT DEWATERING RECOMMENDATIONS

Groundwater was encountered at depths varying from 15 to 31 feet during drilling. We anticipate that groundwater will consistently seep into the excavation through the sand seams within the outwash unit below depths on the order of 21 feet. Construction dewatering performed by the contractor is intended to dewater the site during construction. Permanent dewatering of the excavation will be important to prevent wet conditions in the parking garage, and to avoid hydrostatic uplift against buried structures, footings, and openings.

We recommend constructing a permanent collection sump within the garage to accommodate groundwater dewatering. A system of subslab and backwall drains should direct groundwater toward the sump where it can be pumped to an acceptable discharge point. If gravity drains cannot keep up with the seepage, then more aggressive means such as pumping might be required. The rate of groundwater seepage should be evaluated during construction dewatering and a permanent dewatering system should be designed based upon the discharge rates of construction dewatering pumps.

6.0 ADDITIONAL SERVICES AND LIMITATIONS

6.1 ADDITIONAL SERVICES

Additional services by the geotechnical engineer are important to help insure that report recommendations are correctly interpreted in final project design and to help verify compliance with project specifications during the construction process. For this project we anticipate additional services may include the following:

0. Review final design and construction drawings for conformance with geotechnical recommendations.
0. Monitor temporary excavations and verify soil bearing.
0. Monitor installation of the shoring system and testing of tiebacks.
0. Monitor drainage installation behind the retaining walls.
0. Monitor subsurface drainage installation
0. Monitor temporary erosion control measures.
0. Provide periodic construction field reports, as requested by the client and required by the building department.
0. Provide final report in accordance with City of Seattle requirements.

We would provide these additional services on a time-and-expense basis in accordance with our Standard Fee Schedule and General Conditions already in place for this project.

6.2 LIMITATIONS

This geotechnical investigation was planned and conducted in accordance with generally accepted engineering standards practiced presently within this geographic area. Geotechnical investigations performed by these standards reveal with reasonable regularity soils that are representative of subsurface conditions throughout the site under consideration. Recommendations contained in this report are based upon the assumption that soil conditions encountered in explorations are representative of actual conditions throughout the building site. However, inconsistent conditions can occur between exploratory borings or test pits and not be detected by a geotechnical study. If, during construction or subsequent exploration, subsurface or slope conditions are encountered which differ from those anticipated based upon results of this investigation, The Galli Group should be notified so that we can review and revise our recommendations where necessary. If conditions change prior to the proposed construction, we should be consulted so that we may alter our recommendations if necessary.

This report is prepared for the exclusive use of the owner or the owner's consultants for specific application on this project at this particular site. Copies of this report should be made available to the design team, and should be included with the contract drawings issued to the contractor. Our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions on the site and should not be applied to neighboring sites. No warranty, expressed or implied is made. We recommend that geotechnical observation and testing be provided during the construction phases to verify that the recommendations provided in this report are incorporated into the actual construction. If our firm is not utilized to provide these services or if the contractor fails to notify us or request construction monitoring we cannot be held responsible for performance of the geotechnical design elements.

Respectfully submitted,

THE GALLI GROUP

Paul L. Stoltenberg, P.E.
Project Geotechnical Engineer

Appendix

Logs of Exploratory Borings
Laboratory Testing

Site Sketch

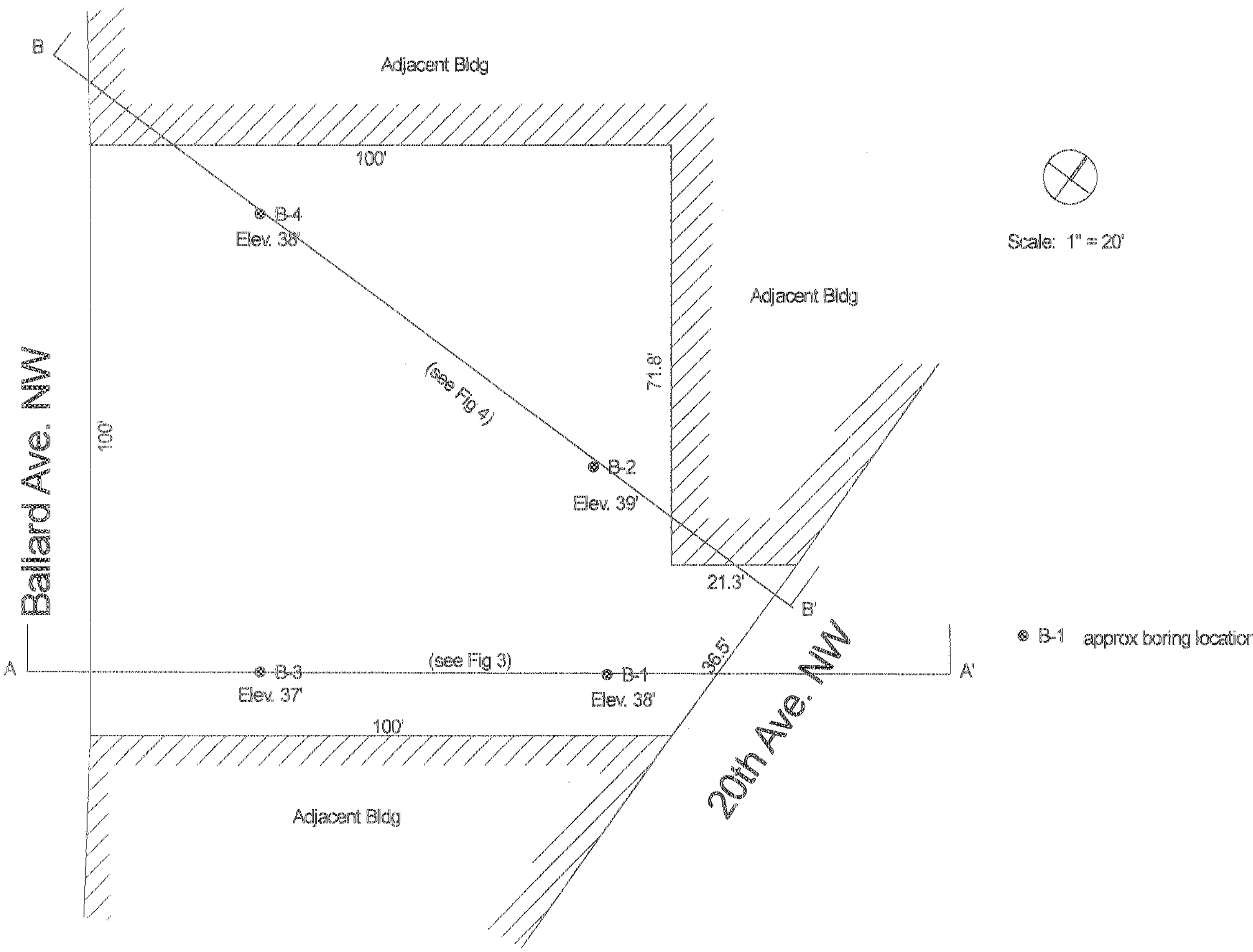

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Figure 2

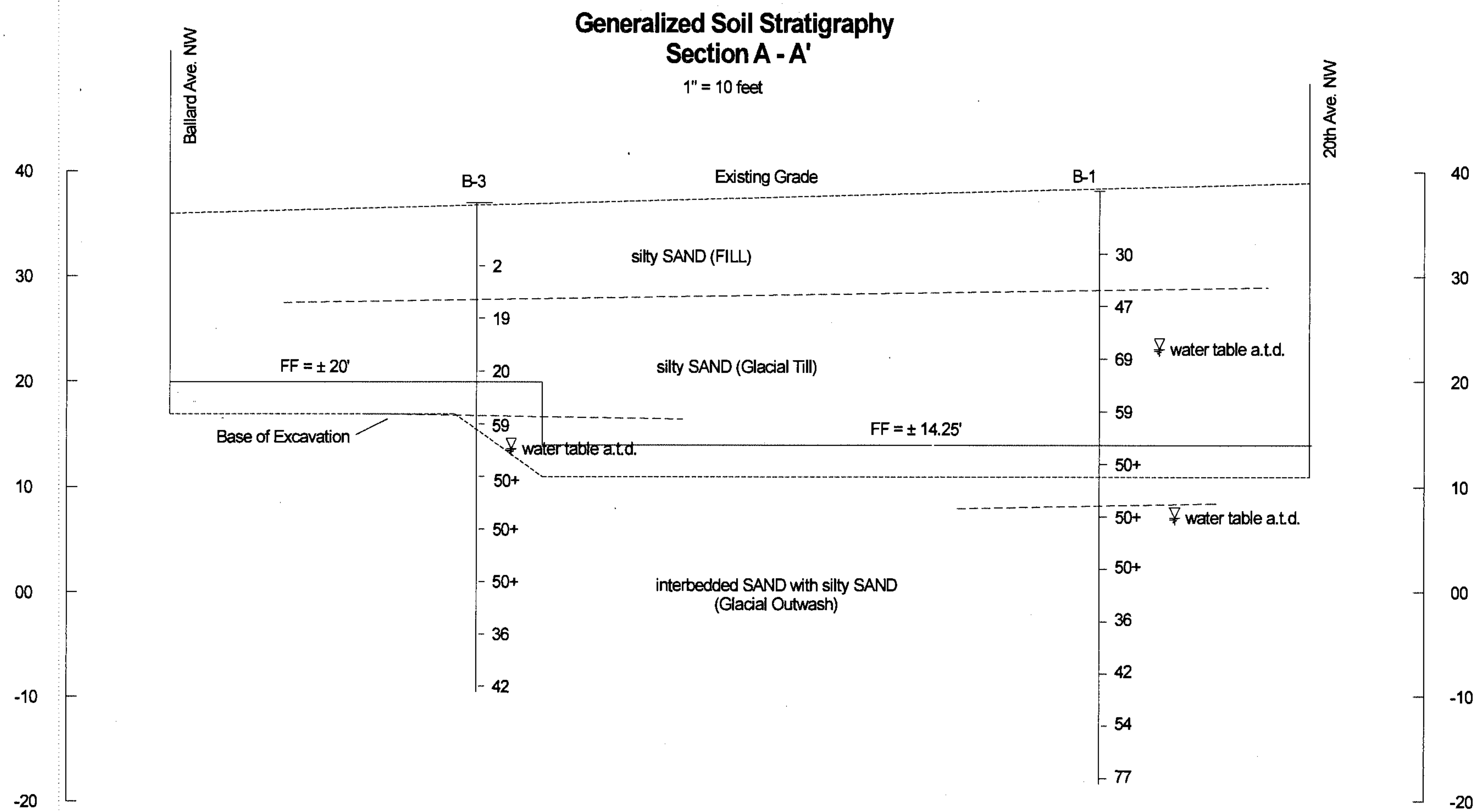

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Figure 3

Generalized Soil Stratigraphy
Section B-B'

1" = 10 feet

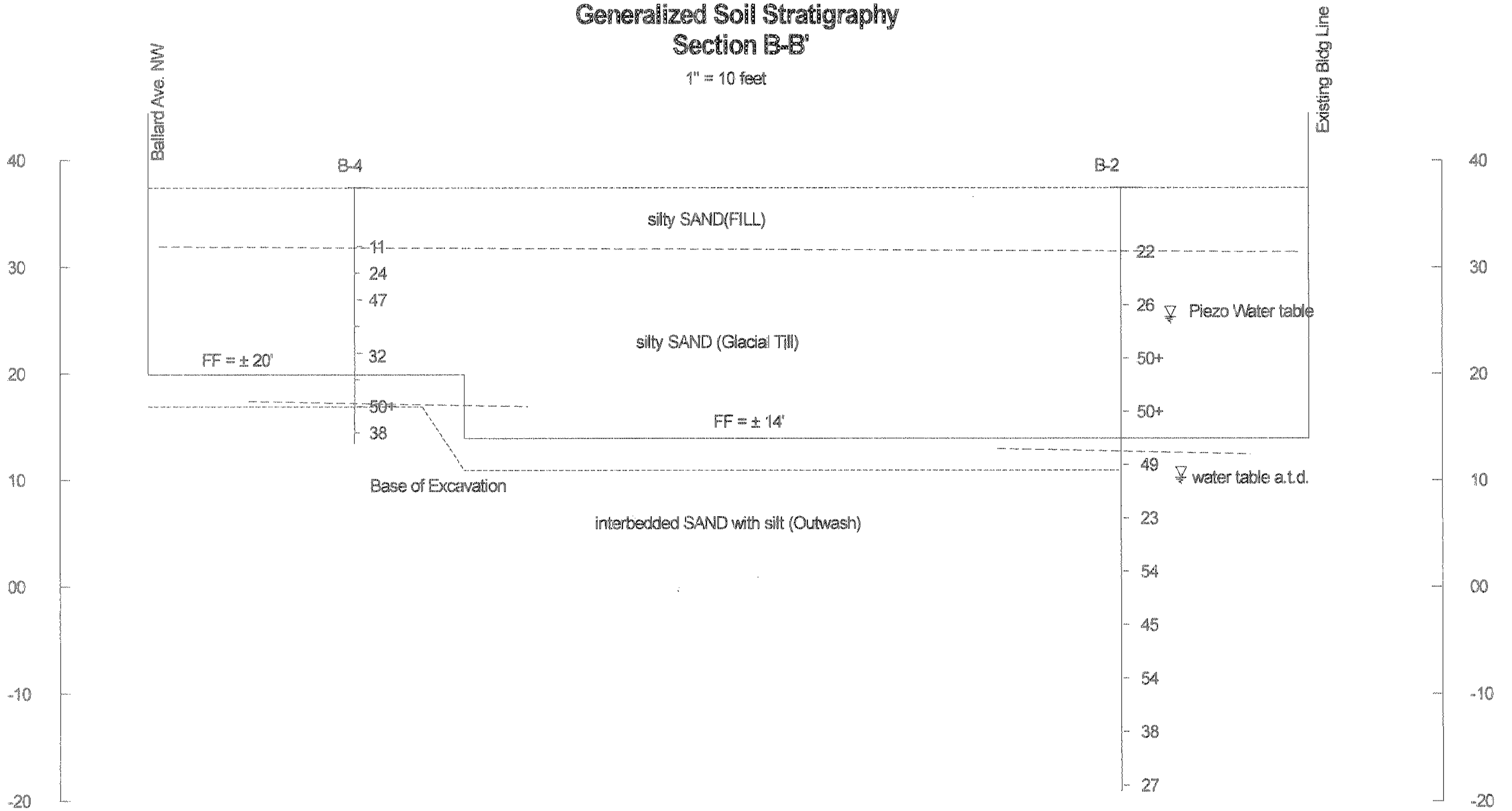

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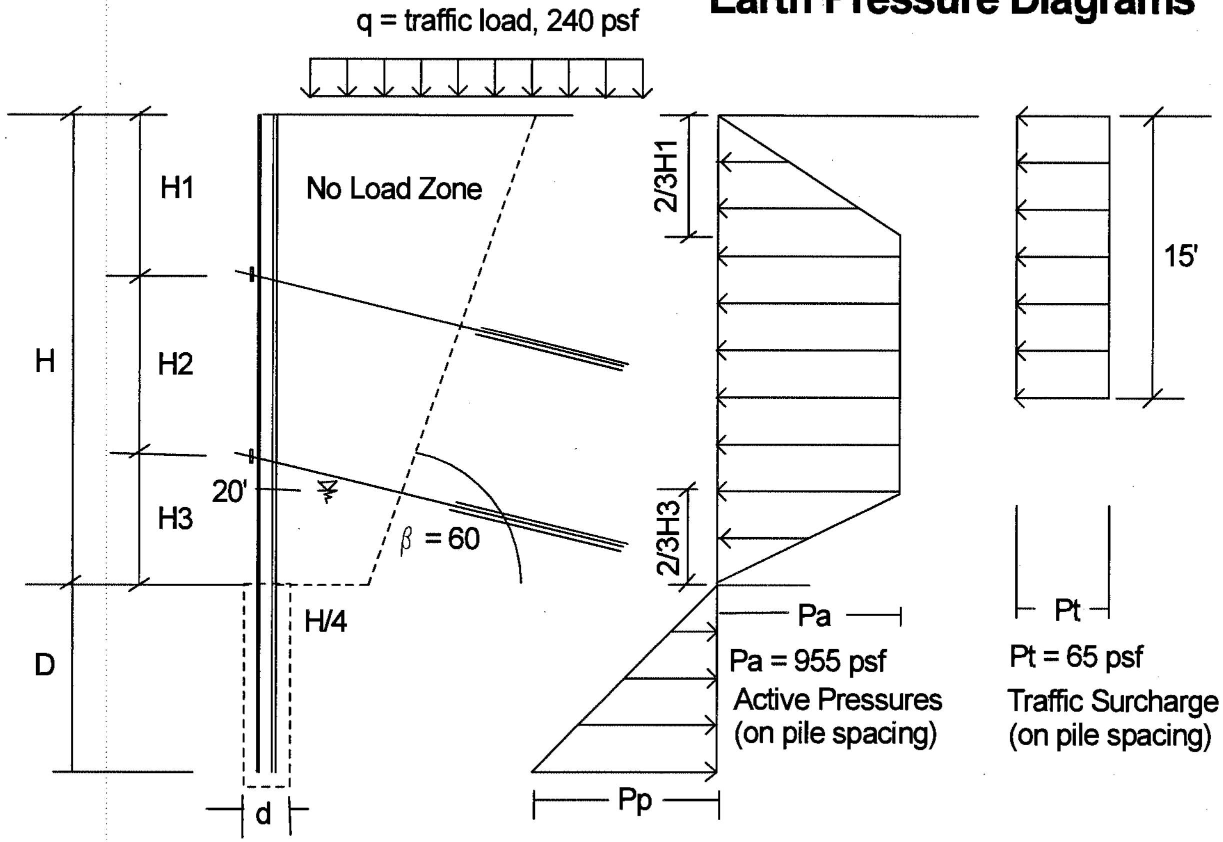
Figure 4

Soldier Pile Shoring with Tiebacks
Earth Pressure Diagrams



Pp = 250xD psf
Passive Resistance
(acts on 2 pile dia.)

Pressure Grouted Tieback Design Parameters

Ulitmate Capacity = 10 kips/ft bonded length
Unbonded length in no load zone
Number and spacing per structural engineer
Steel elements per structural engineer
Performance spec per structural engineer
Apply FOS = 2 for design values

Soil Design Parameters

ϕ = 35 degrees
Active Ka = 0.271
Passive Kp = 3.69
Unit weight, γ = 130 pcf
Active Pressure = 35 pcf (EFW)
Passive Pressure = 420 pcf (EFW) x 2d
Passive Effective = 250 pcf (EFW)

Soil Parameters are design values. Apply Factor of Safety of 1.5 for overturning and kick out.

Note: Shoring system elements to be designed by structural engineer. Plans Should be reviewed by geotechnical engineer prior to submittal

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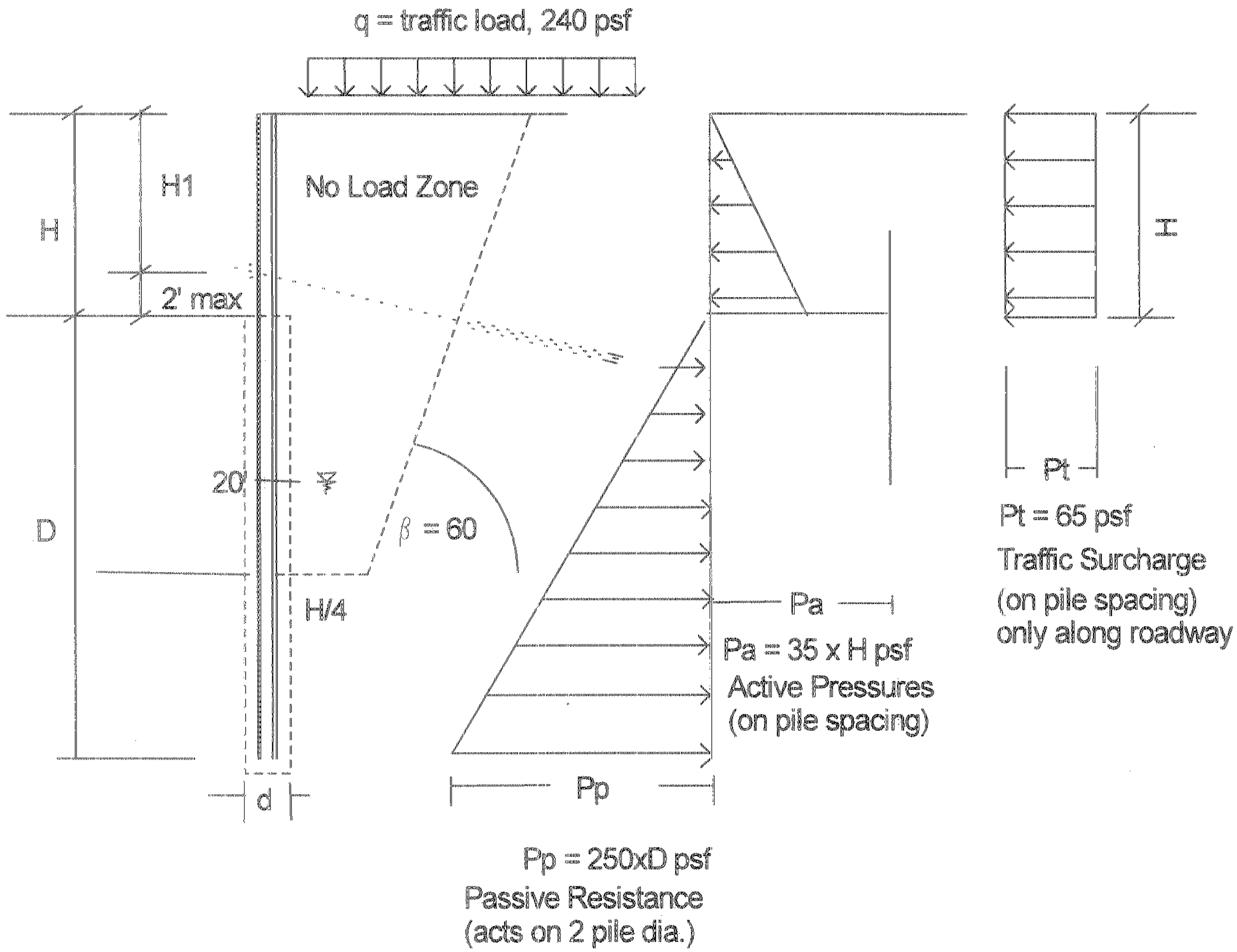
Figure 5

Construction Stage Earth Pressure Diagrams
First Row of Tieback Installation



Pressure Grouted Tieback Design Parameters

Ulitmate Capacity = 10 kips/ft bonded length
Unbonded length in no load zone
Number and spacing per structural engineer
Steel elements per structural engineer
Performance spec per structural engineer
Apply FOS = 2 for design values

Soil Design Parameters

ϕ = 35 degrees
Active Ka = 0.271
Passive Kp = 3.69
Unit weight, γ = 130 pcf
Active Pressure = 35 pcf (EFW)
Passive Pressure = 420 pcf (EFW) x 2d
Passive Effective = 250 pcf (EFW)

Soil Parameters are design values. Apply Factor of Safety of 1.5 for overturning and kick out.

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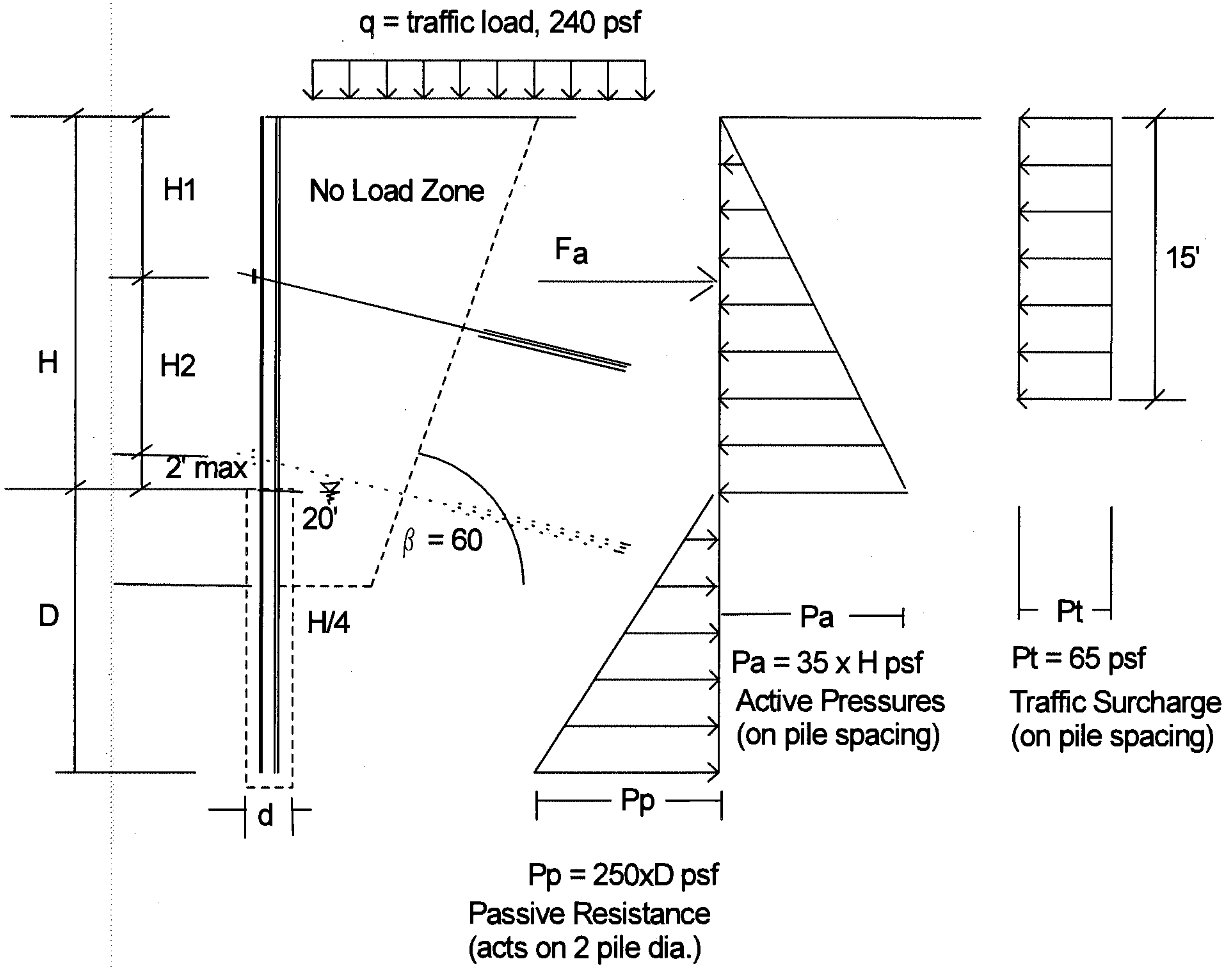
Figure 6

Construction Stage Earth Pressure Diagrams
Second Row of Tieback Installation



Pressure Grouted Tieback Design Parameters

Ulitmate Capacity = 10 kips/ft bonded length
Unbonded length in no load zone
Number and spacing per structural engineer
Steel elements per structural engineer
Performance spec per structural engineer
Apply FOS = 2 for design values

Soil Design Parameters

ϕ = 35 degrees
Active Ka = 0.271
Passive Kp = 3.69
Unit weight, γ = 130 pcf
Active Pressure = 35 pcf (EFW)
Passive Pressure = 420 pcf (EFW) x 2d
Passive Effective = 250 pcf (EFW)

Soil Parameters are design values. Apply Factor of Safety of 1.5 for overturning and kick out.

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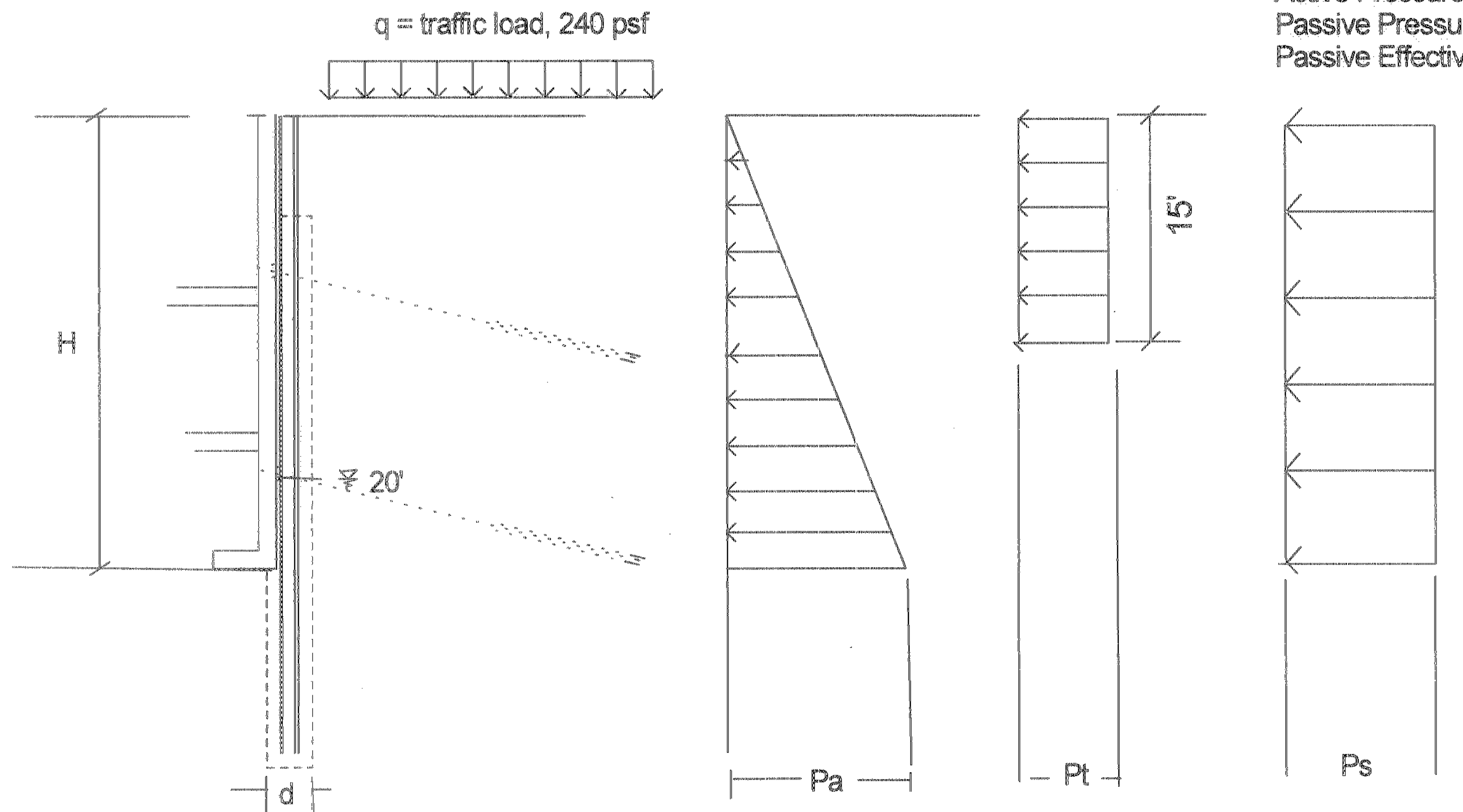
Figure 7

Earth Pressure Diagrams
Permanent Retaining Walls

Soil Design Parameters

ϕ = 35 degrees
Active Ka = 0.271
Passive Kp = 3.69
Unit weight, γ = 130 pcf
Active Pressure = 35 pcf (EFW)
Passive Pressure = 420 pcf (EFW) x 2d
Passive Effective = 250 pcf (EFW)



Pa = 55 x H psf
At-rest Pressures
(on pile spacing)

Pt = 65 psf
Traffic Surcharge
(on pile spacing)
only along roadway

Ps = 10H psf
Seismic Force

Noe: Assumes permanent dewatering

Soil Parameters are design values. Apply Factor of Safety of 1.5 for overturning and kick out.

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Figure 8

Project 1466 - 01

Boring Log B-1

5-14-2007

Elevation = ± 39'

Soil Description	Depth	Sample	SPT	
Med. Dense, brown, silty SAND with organics; contains broken glass, brick (FILL)	5'	S-1	11-15-15	
Brown, SAND with trace silt; trace gravel; moist	10'	S-2	16-21-26	m.c. = 10.1%
Gray-brown, SAND with silt; trace gravel; contains occ. silt seam(SM, 22.6% silt)	15'	S-3	24-32-37	▽ 15' a.t.d. m.c. = 11.5%
Gray-brown, SAND with silt; trace gravel; contains silty SAND seams; wet	20'	S-4	19-26-33	m.c. = 9.8%
Gray, silty SAND with trace gravel; occ. silt seams	25'	S-5	21-50/6"	m.c. = 16.1%
Very dense, gray, poorly graded SAND with silt and silty SAND; wet -pebbly gravel at 32'	30'	S-6	50/6"	m.c. = 13.1% ▽ 31' a.t.d. estimated
Poorly graded SAND and pebbly gravel; wet; -driller added water after about 6" heave	35'	S-7	12-34-50/4"	
8" coarse SAND; 6" silty SAND; wet	40'	S-8	2-19-17	
	45'			

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Figure A-2

Boring Log B-1

Log	Soil Description	Depth	Sample	SPT
	NOTE: Driller is washing out some of the samples too much in dealing with heave; blows might be understated 40 to 50 feet depth	40'		
	Poorly graded SAND and silty SAND; layered	45'	S-9	4-16-26
	Very dense, gray, SAND with silt; silty SAND in tip	50'	S-10	3-20-34
	Very dense, gray, SAND withsilt; occ. gravel and poorly graded SAND layers	55'	S-11	14-38-39
	Bottom of Boring at 56.5 feet Water encountered at ± 15 feet during drilling			
	Driller filled hole with bentonite chips	60'		
		65'		
		70'		
		75'		
		80'		
		85'		

Project 1466 - 01

Boring Log B-2

5-14-2007

Elevation = ± 39'

Depth	Soil Description	Sample	SPT	Well Const.
5'	Med. Dense, brown, silty SAND with organics; weathered to 6' depth (FILL)	S-1	2-10-12	Bentonite
	Brown, dense to very dense, silty SAND with trace gravel; (est. 15% silt)			
10'	Brown, SAND with trace silt; trace gravel; moist	S-2	10-12-14 w.c. = 12.5%	10.5' 5-21-07 Piezo
15'	Brown, SAND with silt; trace gravel; moist	S-3	18-50/6" w.c. = 11.3 %	
20'	Gray-brown, SAND with silt; trace gravel; contains silty SAND seams; wet	S-4	26-50/4" w.c. = 15.6 %	SAND
25'	9" v.silty fine SAND; 12" fine SAND with silt NOTE: Driller added water to prevent anticipated heave	S-5	w.c. = 12.3 % 22-17-32 26' a.t.d. (estimated)	
30'	Med. dense, gray, med. SAND with silt ; one piece gravel in sampler; wet (possible understated SPT)	S-6	1-9-14	
35'	Dense, gray, fine to medium SAND with trace silt	S-7	11-24-30	Bentonite
40'	6" Silty fine SAND and 10" fine sandy SILT	S-8	11-13-22	
45'				

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Figure A-4

Boring Log B-2

Log	Soil Description	Depth	Sample	SPT
		40'		
	Poorly graded SANDwith interbedded SILT; ± 6" beds	45'	S-9	7-25-29
	Dense, poorly graded SAND with occ. gravel	50'	S-10	3-15-23
	Poorly grade SAND	55'	S-11	3-12-15
	Bottom of Boring at 56.5 feet			
	Constructed well 0-20' 2" sollid PVC pipe 20' - 30' slotted PVC pipe 0' - 18' Bentonite 18' - 32' SAND 32' - 44' Bentonite plug	60' 65'		
		70'		
		75'		
		80'		
		85'		

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Boring Log B-3

5-15-2007

Elevation = ± 37'

Soil Description	Depth	Sample	SPT	
Organic rich topsoil and gray clay in tailings (FILL)				
No Recovery	5'		0-0-2	
Med. dense, gray-brown, silty SAND with trace clay; wet	10'	S-2	7-10-9	w.c. = 17.4%
Med. dense, gray-brown silty SAND with SAND beds	15'	S-3	7-11-9	w.c. = 17.9%
Dense, gray brown, SAND with trace silt; some silt seams	20'	S-4	19-26-33	w.c. = 12.5%
				⊻ 23' a.t.d.
SAND with silt -driller added water	25'	S-5	21-50/6"	
SAND with silt (SM, 24% silt)	30'	S-6	50/6"	w.c. = 9.9%
SAND with silt	35'	S-7	12-34-50/4"	
Poorly graded SAND (8' of heave after sampling)	40'	S-8	2-19-17	
	45'			

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Figure A-6

Boring Log B-3

(Sheet 2 of 2)

Log	Soil Description	Depth	Sample	SPT	
	NOTE: after sampling sand heaved in auger and plugged rods (8' heave); blows likely understated at 40 depth.	40'			
	Poorly graded SANDwith silt; occ. gravel	45'	S-9	4-16-26	
	Bottom of Boring at 46.5 feet Water encountered at ± 23 feet during drilling				
	Driller filled hole with bentonite chips	50'			
		55'			
		60'			
		65'			
		70'			
		75'			
		80'			
		85'			

Project 1466 - 01

Boring Log B-4

5-21-2007

Elevation = ± 38'

Log	Soil Description	Depth	Sample	SPT	
	Loose, brown, silty SAND with organics; contains debris (FILL)				
		5'			
	Med. dense, brown, silty SAND		S-1	5-6-5	
	Med. dense, gray-brown, silty SAND with mottled streaks; moist		S-2	10-12-12	
	V. dense, gray brown, silty fine SAND with trace gravel	10'	S-3	13-23-34	
	Dense, SAND with silt; wet -driller added water	15'	S-4	12-16-16	
	-added drilling mud				
	12" poorly graded SAND 3" gray silty SAND in tip	20'	S-5	10-50/6"	20' a.t.d. (estimated)
	Poorly graded SAND; 8" silty SAND in tip		S-6	21-18-20	
	Bottom of Boring at 24 feet depth	25'			
		30'			
		35'			
		40'	S-8		
		45'			

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Figure A-8