Geotechnical Engineering Re
Public Safety Building Demolition
Shoring Pro
Seattle, Washin

November 24, 2

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

At Shannon & Wilson, our mission is to be a progressive, well-managed professional consulting firm in the fields of engineering and applied earth sciences. Our goal is to perform our services with the highest degree of professionalism with due consideration to the best interests of the public, our clients, and our employees.

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21-1-20116-0

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GEOTECHNICAL ENGINEERING REPORT
PUBLIC SAFETY BUILDING DEMOLITION AND SHORING PROJECT
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of our subsurface explorations, geotechnical laboratory testing, and geotechnical engineering studies for the Seattle Public Safety Building (PSB) Demolition and Shoring Project in Seattle, Washington. The purpose of this study is to evaluate subsurface conditions at the project site and provide geotechnical engineering recommendations for the design of a shoring system that will be installed during the demolition of the PSB.

Our work was performed in general accordance with our proposal dated April 26, 2004.

2.0 SITE AND PROJECT DESCRIPTION

The project location is shown on the Vicinity Map, Figure 1. The existing PSB is on the city block bounded by Cherry Street to the north, James Street to the south, Fourth Avenue to the east, and Third Avenue to the west. The project site is shown on the Site and Exploration Plan, Figure 2.

The ground surface surrounding the site is paved with streets and sidewalks. The surrounding ground surface slopes down to the west from elevation 111 feet on the northeast corner of the building to elevation 76 feet on the southwest corner. The existing PSB is supported by spread footings, which reportedly bear at elevations between approximately 54 feet in the southwest corner of the building and 68 feet at the northeast corner of the block. The foundation subgrade is about 20 to 45 feet below the street level, with the deepest foundations in the northeast corner of the building.

Existing buildings occupy the blocks surrounding the PSB:

- King County Courthouse is south of project site.
- Seattle City Hall Building is east of project site.
- Arctic Club Building and Grand Central Garage are north of project site.
- St. Charles Hotel and Lyon Building are west of project site.

The street rights-of-way surrounding the project site contain numerous buried utilities. We understand a skid road may be buried under James Street. Logs and wood debris were encountered during construction of the tunnel between the City of Seattle Justice Center and the King County Correction Facility. In our review of historical records and previous borings, we did not find evidence of the skid road near the PSB.

The Burlington Northern Santa Fe (BNSF) railroad tunnel is under Fourth Avenue and between approximate elevations 12 and 54 feet. A zone of soil outside of this railroad tunnel likely has been disturbed from the construction of the tunnel. The zone of disturbed soil may include areas where tiebacks are installed. The Downtown Seattle Transit Project (DSTP) bus tunnel and Pioneer Station is under Third Avenue, adjacent to the PSB. The crown of the bus tunnel is between approximate elevations 36 feet at Cherry Street and elevation 32 feet at James Street.

We understand that the plans for the project site have not been completed but will likely include a multi-level building and a large plaza with parking below the plaza. The existing building probably will be demolished before plans for the site are completed. We understand the planned demolition will extend to the basement slab-on-grade but not deeper. However, deeper excavations could be made at a later date.

A temporary support system of tieback anchors installed through existing basement walls will be installed and will remain in place following demolition until the development plans are completed and the new building and plaza are constructed. The temporary shoring system may be in use for several years until the development plans are made and the permanent lateral bracing system is constructed.

3.0 SUBSURFACE EXPLORATIONS

Subsurface explorations for the project included two borings completed at the project site, designated B-1 and B-2. Subsurface information from previous studies that have been performed by Shannon & Wilson, Inc. and others dating back to 1948 was compiled. The approximate locations of the recent and previous borings are shown on the Site and Exploration Plan, Figure 2.

Methods and procedures used for drilling and sampling of the borings are presented in Appendix A. Logs for borings B-1 and B-2 are presented in Appendix A as Figures A-2 and A-3. Logs of selected previous borings are shown in Appendix C. A guide to the soil classification terms used in the recent boring and in this report by Shannon & Wilson, Inc. is included as Figure A-1 (2 sheets).

4.0 GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory tests were performed on selected samples retrieved from current subsurface explorations. The testing included visual classification, moisture content, grain size analyses, and Atterberg Limits determinations. Laboratory testing was performed to aid in classifying the soil and to determine soil index and engineering properties. The laboratory test results are incorporated into the borings logs presented in Appendix A. Descriptions of laboratory test procedures and the results are presented in Appendix B, Laboratory Test Results.

5.0 GEOLOGY AND GEOLOGIC HAZARDS

The project site is within the Puget Lowland, a structural trough between the Cascade Range and the Olympic Mountains. This trough was subjected to several major glaciations during the Pleistocene Epoch. As a result of these glaciations, the Puget Lowland was filled to significant depths with glacial and nonglacial sediments. Many of these glacial and nonglacial sediments have been glacially overridden and consolidated to dense or hard conditions. The last glaciation experienced by the Puget Lowland, the Vashon Stade, occurred approximately 13,000 years ago. The native soils at the project site predominantly consist of pre-Vashon nonglacial soil layers, which are primarily lacustrine fine sandy silt, silty fine sand, and clayey silt (Qpnl). Interbeds of nonglacial fluvial fine to medium sand (Qpnf) exist within the nonglacial lacustrine silt and fine sand. Several thin, less than ½-inch-thick, hard peat seams exist within the nonglacial deposits. Relatively thin, discontinuous layers of pre-Vashon glacial till (Qpgt) and glacial marine drift (Qpgm) overlie the nonglacial soils at the site. These soil layers have been overridden by glacial ice and, consequently, have high strength and low compressibility.

Earthquake hazards in the Puget Sound region can include fault-related ground rupture, liquefaction, settlement, and landsliding. Based on the dense nature of the glacially overridden soils at the project site, the topography, and the estimated depth to groundwater, it is our opinion that the risk of liquefaction, settlement, and landsliding at the site is low. In our opinion, the potential for fault-related ground rupture affecting the site is low. This opinion is based on published and unpublished reports that show the closest, identified, potentially active fault is the Seattle Fault, which is located about ½ mile to the south. While there is evidence that this fault may have moved about 1,100 years ago, no conclusive evidence of surface rupture in Seattle has been detected. It is generally believed that the recurrence interval for this fault is on the order of thousands of years.

6.0 SUBSURFACE CONDITIONS

The project site is underlain by glacially overridden soil layers that have been glacially consolidated to a hard or very dense condition. Our interpretation of the subsurface conditions at the site are summarized on the Generalized Subsurface Profiles A-A', and B-B', presented on Figures 3 and 4. Approximate locations of the subsurface profiles are shown on the Site and Exploration Plan, Figure 2. The approximate elevations of the base of the building foundation for the Public Safety Building have been projected onto these profiles.

Historical photographs show the basement walls for the PSB were constructed in an open excavation. Therefore, fill material is present between the basement wall and the old cut slope. From our study of the photographs, we estimate that the cut slopes were made at 1 Horizontal to 1 Vertical (1H to 1V) or steeper.

Fill deposits encountered in the borings were variable and generally very loose to loose or soft to medium stiff. The fill material included slightly silty to silty, gravelly sand; silty, sandy, clayey gravel; and sandy, silty clay. Debris encountered within the fill soil included brick fragments, wood debris, and chunks of silty clay soil intermixed within the sandy fill soils. Boring B-1 was terminated at 37.0 feet after refusal on concrete. The boring was drilled approximately 2.5 feet away from the retaining wall; therefore, we assumed that the concrete obstruction encountered was the footing for the existing PSB.

Below the footing elevations, the subsurface conditions consist primarily of interbedded, pre-Vashon nonglacial lacustrine (Qpnl) and fluvial (Qpnf) soils. A thin discontinuous layer of pre-Vashon glaciomarine drift (Qpgm) overlies the nonglacial deposits. Pre-Vashon nonglacial lacustrine deposits consist of very dense, massive to laminated, silty fine sand and fine sandy silt with scattered, thin, silty clay seams; peat seams; and fine gravel. The pre-Vashon nonglacial fluvial (Qpnf) soils consist of very dense, fine to medium sand with various amounts of gravel. Abundant fine organic fragments exist within all the pre-Vashon nonglacial soils.

Underground structures within the project area include the DSTP tunnel and Pioneer Square Station under 3rd Avenue, and the BNSF railroad tunnel under 4th Avenue, between the PSB and the City Hall. Please refer to Section 7.5 for additional descriptions of the DSTP structures.

7.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

7.1 General

The proposed demolition and excavation will require temporary lateral restraint to support the existing basement-level walls as the interior of the structure is removed. We recommend using post-grouted tieback anchors to provide lateral restraint. These tiebacks could be installed through the existing basement wall and post-tensioned to reduce wall deflection as the demolition progresses. If future excavations are performed below the existing basement slab-on-grade, we recommend underpinning the existing basement walls with soldier piles and installing a soldier pile with lagging wall with tieback anchors to support the excavation.

The following sections present our recommendations for shoring and other pertinent geotechnical design issues such as lateral resistance and lateral earth pressures, drainage, and construction considerations.

7.2 Foundation Design

Based on previous and recent explorations, we interpret the soil underlying the building site at or just below the existing basement slab is heterogeneous and likely consists of dense to very dense, granular soil and hard, cohesive soil. Plans for the existing PSB show that the existing spread footings bear at elevations ranging between approximately 54 feet in the southwest building corner and 68 feet at the northeast corner of the block. It is likely that all existing footings bear in very dense sand and gravel or hard, silty clay.

Because we do not know if a specific footing is underlain by hard clay or very dense sand, we recommend that an allowable bearing pressure of 8 kips per square foot (ksf) be used to analyze the capacity of existing footings. Greater allowable bearing pressures could be used if larger settlements are tolerable. This recommended allowable bearing pressures may be increased by one-third for short-term wind or seismic loading.

Our scope of services includes shoring design for the existing PSB and potential future excavation below the existing basement slab-on-grade. Our scope of services does not include foundation design for any new buildings.

7.3 Existing Floor Slab Drainage

Stormwater drainage should be provided by either tying into the existing drainage system or installing a new system. We do not have information about the existing floor slab drainage system. If plans showing the existing drainage system are not available, coring through the existing basement floor slab could provide information regarding the existing drainage system.

We understand that you plan to break up the existing floor slab and allow the stormwater to infiltrate into the ground. The existing subsurface soil types are heterogeneous, ranging from hard clay to very dense sand and gravel with silt lenses. The expected hydraulic conductivity of these soil types varies from approximately $1x10^{-6}$ centimeters per second (cm/sec) to $1x10^{-8}$ cm/sec. In some areas, sand with hydraulic conductivity on the order of $1x10^{-5}$ cm/sec may be encountered; however, the overall hydraulic conductivity will likely be between $1x10^{-6}$ cm/sec and $1x10^{-8}$ cm/sec.

7.4 Lateral Resistance

Lateral loads due to wind or seismic forces, or any unbalanced earth pressures on the sides of the structure, may be resisted by the passive earth pressure resistance of the in situ soils acting against the footings and buried portions of the basement walls. A coefficient of friction of 0.5 may be used for resisting lateral loads against the bottoms of existing and new footings. An appropriate factor-of-safety (FS) should be used in evaluating the resistance to base sliding. We recommend that an equivalent fluid density of 350 pounds per cubic foot (pcf) be used for computing passive soil resistance against the existing footings and basement walls. This value includes a FS of 1.5 to limit lateral deflections and assumes that the foundations extend at least 24 inches below the lowest adjacent grade.

7.5 Temporary Restraint of Basement Walls

The PSB basement walls may be laterally restrained with tieback anchors installed through the walls during demolition of the existing building. Where existing structures preclude tieback anchors rakers, struts or corner braces could be used for temporary bracing. The BNSF railroad tunnel below Fourth Avenue is below the PSB basement levels and probably would not be encountered; however, tieback anchors could extend into soil disturbed during the tunnel construction.

The DSTP bus tunnel and Pioneer Square Station are under Third Avenue. The crown of the bus tunnel is approximately 40 to 45 feet below ground surface, which is likely deep enough that the tunnel would not be encountered during tieback installation. Pioneer Square Station is along the southern 170 feet of the PSB. According to the plans, dated July 15, 1986, the Pioneer Square Station cofferdam was built about 15 feet west of the PSB building. Due to the limited space between the existing PSB basement walls and Pioneer Square Station, external bracing (i.e. tieback anchors) would not be feasible for shoring in this section of the wall. Internal bracing methods such as rakers, or struts could be used. The information provided in this report is based on July 15, 1986 Contract Drawings for Tunnels and Pioneer Square Station Excavation (Parsons Brinkerhoff Quade & Douglas, Inc. 1986). We recommend verifying the location of the Pioneer Square Station and bus tunnel with as-built drawings.

Recommended earth pressures for the design of temporary tied-back shoring walls with single and multiple tieback loads are presented in Figures 5 through 7. Figures 5 and 6 present earth pressures assuming no excavation is made below the existing basement slab. Figure 7 shows recommended earth pressures assuming future excavations are made below the basement slabs. These recommended earth pressures are based on the assumptions that the groundwater level is below the base of the excavation (as observed in the borings) and hydrostatic pressure will not occur on the basement walls.

Lateral earth pressures from general construction traffic and equipment surcharge loading should be added to the earth pressures and may be estimated by assuming an additional 2 feet of soil above the top of the excavation. The design of the tieback system should take into account any surcharges caused by adjacent buildings, heavy construction equipment, or other loads. Figure 8 shows surcharge loadings that can be used as appropriate. For other surcharge loads each case should be analyzed individually. We can provide appropriate recommendations as necessary.

7.5.1 Internal Bracing

Bracing systems fall under two main categories, internal and external. Internal bracing is typically provided by struts (horizontal braces) or rakers (inclined braces). External bracing is usually provided by tieback anchors. Because of the DSTP bus tunnel and Pioneer Square Station on Third Avenue, the installation of tieback anchors may not be practical in some locations.

Typical practice is to use a continuous or discontinuous horizontal waler to transfer loads from the ground support wall to the bracing. Walers are normally set about 10 to 15 feet apart vertically and braces are spaced about 15 to 20 feet apart horizontally.

It should be noted that the connection details are important in an internally braced excavation. Improper connections between struts and walers, or between the waler and the support wall can lead to twisting, buckling, and rotation of members, resulting in increased deflections and ground settlements.

Prestressed bracing produces a stiffer support system than one that is not prestressed, or one that has a large cantilevered section above the first brace, or one that has widely spaced braces. To reduce or lessen the potential for wall movement and ground settlement, we recommend that the struts be pre-loaded during installation to at least 80 percent of the design load. Prestressing could be accomplished by jacking and welding, or by jacking and driving steel plates and wedges. Temperature effects, i.e., thermal expansion and contraction, should be considered in the design.

A source of wall displacement with internally braced systems is the removal of braces, which is often accompanied by rebracing, associated with construction of the structure within the excavation or for the PSB, possible repositioning of braces as demolition progresses. Factors controlling the amount of displacement are the wall stiffness, the properties of the retained soil, the span distance between remaining braces, and the care and skill with which the work is accomplished and the quality and the compaction of the backfill between the structure and the shoring wall.

Horizontal bracing may be impractical due to the width of the basement. Use of rakers reacting to kicker blocks, reaction piles, or foundations constructed within the basement appears to be feasible.

Recommended allowable bearing pressures for horizontal and inclined footing or kicker blocks are presented in Section 7.2, Foundation Design. The allowable bearing pressures assume that inclined brace footings will bear completely within undisturbed, dense to very dense sand or hard clay or silt. Brace footing should not be supported in structural fill. All disturbed material should be removed and all footing subgrades should be evaluated by an experienced geotechnical engineer prior to concrete placement.

7.5.2 External Bracing (Tieback Anchors)

Tieback anchors consist of steel strands or a reinforcing bar placed into predrilled holes. The holes are typically drilled at an inclination of about 15 to 45 degrees from horizontal. The strands or bars are required to be in the center of the borehole so centralizers are spaced evenly along the tieback length. The frictional resistance of an anchor is dependent on many factors, including the Contractor's method and care of installation. Consequently, the length of production anchors should be based on test anchors. The following frictional values are only for planning and estimating anchor lengths.

In the anchor no-load zone a bond breaker should be used around the tieback tendons. The length of the bond breaker should be determined based on the elevation and inclination of each specific tieback anchor. Some anchors may need to be steeper or flatter than the inclination shown on Figures 5 through 7 to avoid obstructions.

We recommend that tieback anchors be installed into the stiff to hard or very dense native soil that underlies the streets (see Generalized Soil Profiles). The length of production anchors should be based on the results of a series of tieback anchor performance tests; however, we recommend a minimum bond length of 15 feet. For construction planning and estimating purposes, we recommend that a load transfer value on the order of 3.5 kips per lineal foot of embedment be used for small-diameter (6-inch), post-grouted anchors installed in the stiff to hard, silty clay. Anchors installed in very dense sand and gravel may be able to achieve load transfer values on the order of 8 kips per lineal foot. We recommend a load transfer value of 3.5 kips per lineal foot in disturbed soil above the BNSF and DSTP tunnels.

Tieback reinforcement steel should be sized for 80 percent of the guaranteed ultimate strength of the steel under seismic and field verification test loading conditions.

All anchors should be designed and installed to achieve twice the design capacity, 200 percent of the design working load. All temporary anchors should be proof tested by loading in 25 percent (0.25P) increments to 133 percent of their design capacity (1.33P), where P is the design capacity. Prior to installing production anchors within a particular soil stratum, performance tests should be accomplished for each anchor type and/or installation method that will be used. The number of tendons in the selected anchors should be increased as required to complete the performance tests. Approximately 5 percent of production anchors, randomly selected, should be performance tested by loading in 25 percent (0.25P) increments to 200 percent of design capacity (2.0P). We recommend a minimum of five performance tests at locations representing the different geologic conditions around the site. Performance tests should be evaluated by a geotechnical engineer. Production anchors should be installed using the same installation procedures as satisfactorily tested performance test anchors.

We recommend that all temporary anchors be locked off at 80 to 90 percent of the design load to provide some wall flexibility. Anchors that do not meet the acceptance criteria should be locked off at one-half the failure load and replaced with additional anchors as required.

Load testing for all tieback anchors and acceptability should be as recommended by the Post-Tensioning Institute manual, Chapter 4, Recommendations for Prestressed Rock and Soil Anchors, 1996. As described in this manual, the following tests should be accomplished:

- Initial Lift-Off Readings: After transferring the load to the stress anchorage and prior to removing the jack, a lift-off reading should be made. The load determined from the lift-off reading should be within 5 percent of the specified lock-off load. If the load is not within 5 percent of the lock-off load, the end anchorage should be reset and another lift-off reading should be made.
- Lift-Off Test: Lift-off tests may be conducted on selected tiebacks both during and after construction to check the magnitude of seating and transfer load losses and to determine if long-term losses are occurring.
- Acceptance Criteria: The results of each anchor test should be evaluated in order to determine anchor acceptability. An anchor would be acceptable provided:
  - The total movement obtained from a performance and proof test exceeds 80 percent of the theoretical elastic elongation of the design free stressing length.
  - The total anchor movement measured from a proof test does not exceed the calculated elastic movement measured from the jack to the middle of the grouted zone.

- The creep rate during the final test load does not exceed 0.080 inch per log cycle of time and is linear or exhibiting decreasing creep rate regardless of tendon length and load. Otherwise the anchor should be held for an additional 60 minutes at the required test load.
- The initial lift-off readings indicate that an anchor load has been locked-off within 5 percent of specified load.
- The lift-off tests, if required, show an anchor load within 5 percent of the specified transfer load.

7.5.3 Soldier Pile and Tieback Wall

If future excavations are made below the existing basement of the PSB, we recommend underpinning the existing basement walls with soldier piles and installing a soldier pile and lagging wall with tiebacks. General recommendations for soldier piles and lagging are given in the following sections.

7.5.3.1 Soldier Piles

Vertical members for the soldier pile shoring system consist of steel sections placed into predrilled holes and typically backfilled with lean mix concrete. Penetration depth below the final excavation level should be adequate for kick-out resistance. We recommend soldier piles penetrate at least 8 feet below the bottom of the excavation. Soldier piles should also be designed to resist the total vertical component of the tieback anchor forces. Vertical soldier pile capacities below the bottom of the excavation can be evaluated using the skin friction and end bearing recommendations presented in Figure 7.

7.5.3.2 Lagging

We recommend that lagging be installed between soldier piles. Lagging should be installed as the excavation proceeds, and in general, not more than 4 feet (measured vertically) of unsupported excavation should be exposed at any one time. The actual height of vertical, unsupported excavation may vary depending on the soils encountered.

Care should be taken to prevent the buildup of hydrostatic pressures behind shoring walls. Voids behind the lagging should be filled with permeable materials, such as concrete sand and drainage sand and gravel. Weak controlled density fill (CDF) could be used to backfill small areas of voids, but should not be used over large areas where groundwater seepage is encountered.

Because of soil arching between soldier piles, a reduced lateral earth pressure is recommended for design of lagging. Recommended pressures for temporary lagging design are presented in Figure 7.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 Fill Material

We understand that it may be necessary to place fill at the project site. Soil used for fill material found in western Washington is generally inert and is not reactive with normal cements.

8.2 Tieback Installation

The shoring contractor should anticipate drilling boreholes for the installation of tieback anchors through the concrete basement wall, loose silty sand, pea gravel and soft sandy clay, and native soil consisting or hard clayey silt and very dense sand and gravel. Pea gravel and loose cohesionless sand will flow through open holes made in the basement walls.

Tieback anchor holes should be drilled in a manner that will minimize loss of ground and not endanger previously installed anchors or undermine existing pavement or utilities. Different drilling techniques such as casing may be required for tiebacks located below perched groundwater. We recommend that tiebacks located below utilities be drilled, grouted, and installed using casing.

In the anchor no-load zone, tieback holes should be filled with a material such as a sand pozzolan mixture that will not adhere to the tieback rod and will prevent caving. We do not recommend that no-load zone lengths be left open overnight. Alternatively, a bond breaker could be used around the tiebacks in the no-load zone, and the zone filled with concrete or lean concrete backfill.

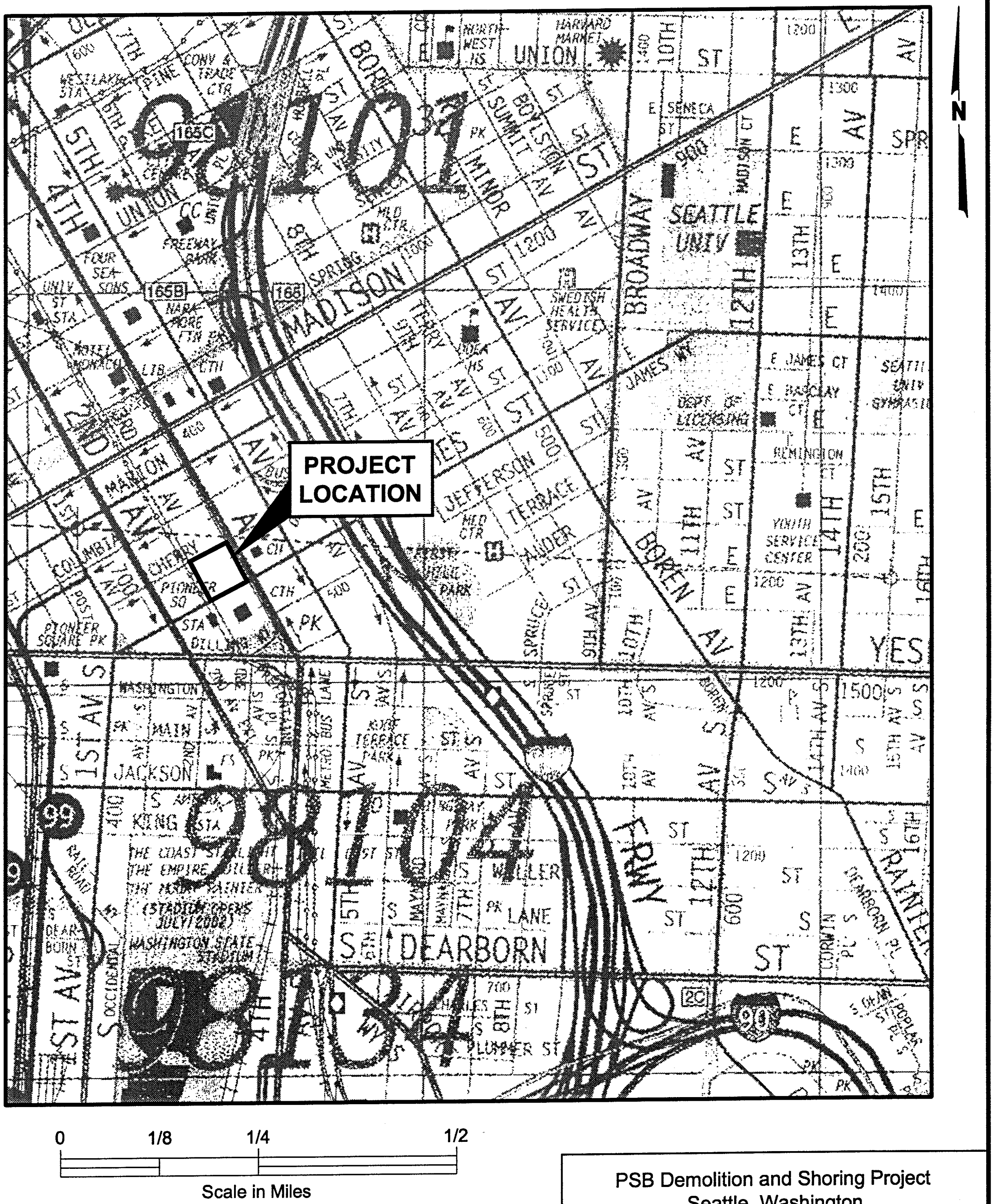
8.3 Obstructions

Although not encountered in the explorations, cobbles and boulders are commonly found in soils in the area and should be anticipated at the site. The Contractor should be prepared to encounter cobbles and boulders during shoring and excavation.

11.0 REFERENCES

American Society for Testing and Materials (ASTM), 2003 annual book of standards, construction, volume 04.08, soil and rock: West Conshocken, Pa., ASTM.

Parsons Brinkerhoff Quade & Douglas, Inc., 1986 Downtown Seattle transit project, contract drawings, tunnels and Pioneer Square Station excavation, part c, volume 2 of 2, Seattle, Washington, July 15.



PSB Demolition and Shoring Project
Seattle, Washington

VICINITY MAP

November 2004 21-1-20116-002

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FIG. 1

NOTE

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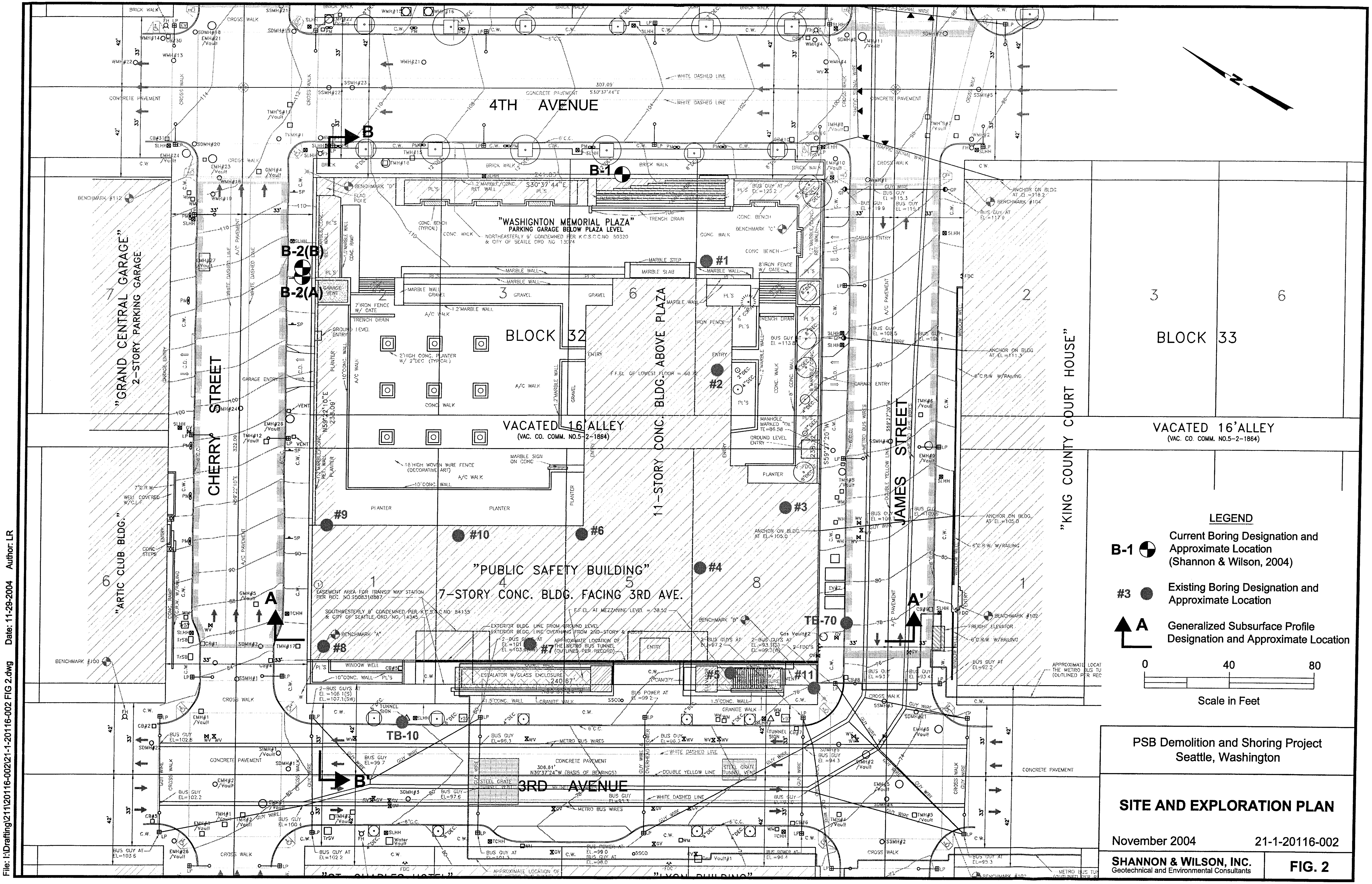

PSB Demolition and Shoring Project
Seattle, Washington

SITE AND EXPLORATION PLAN

November 2004 21-1-20116-002

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FIG. 2

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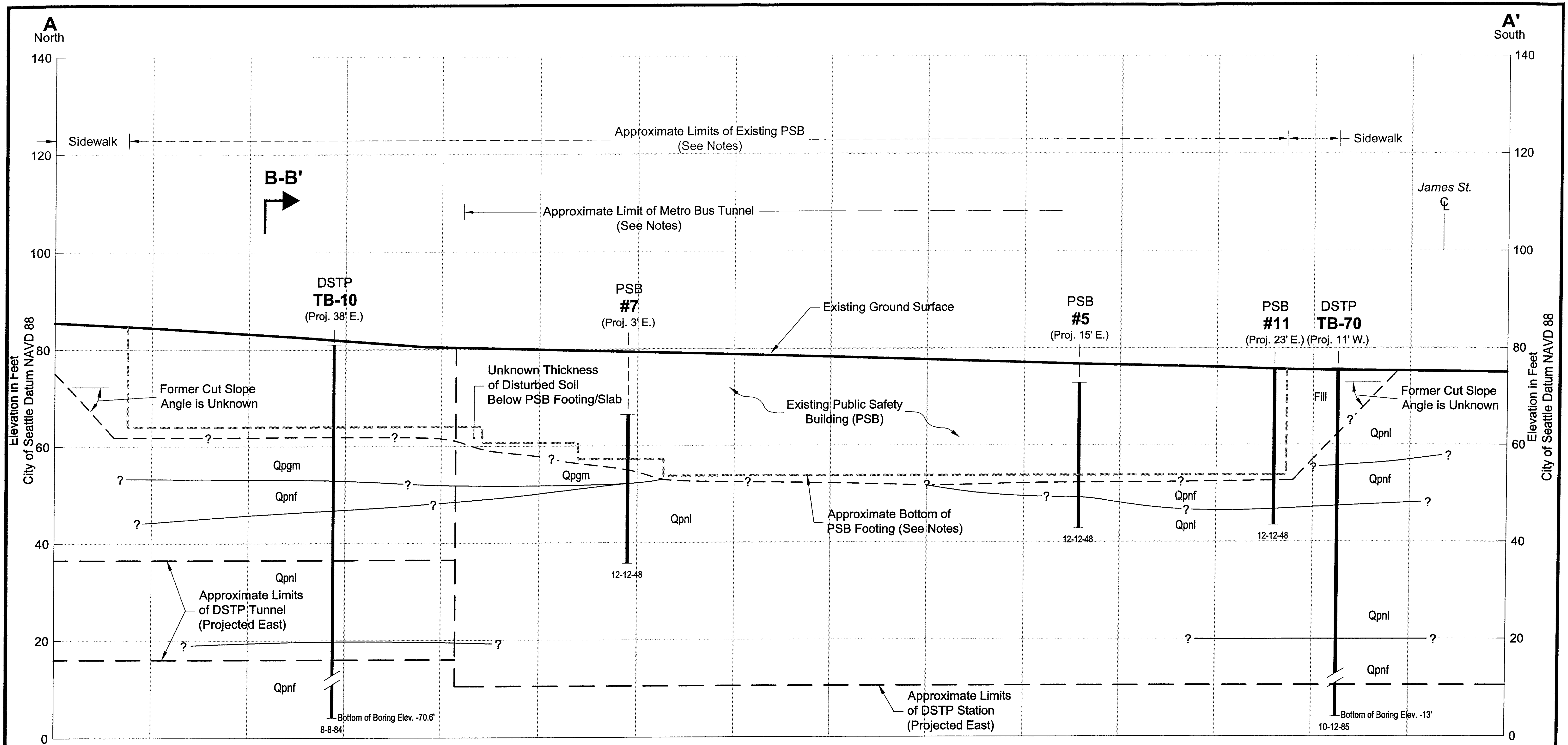

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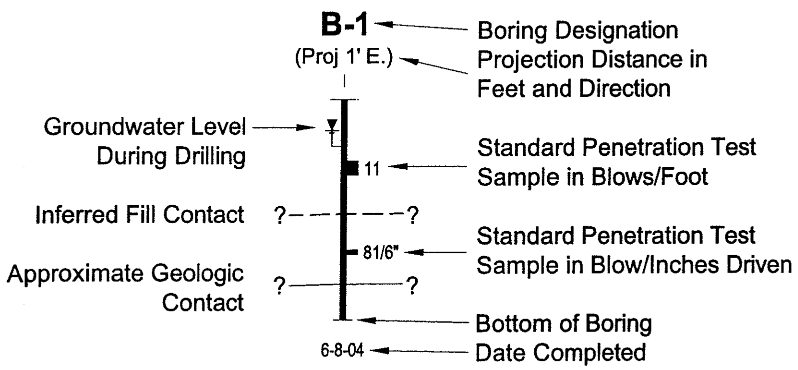

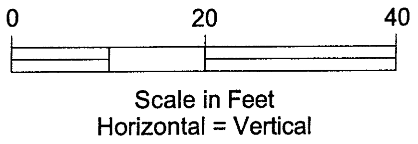

NOTES

1. This subsurface profile is generalized and interpreted from materials observed in soil borings and reported on borings by others. Variations between the profile and actual conditions may exist.
2. Existing limits of PSB taken from "Public Safety Building, City of Seattle" dated 5-20-1948.
3. Existing limits of the Metro Bus Tunnel taken from "Contract No. T/F16-86, Contract Documents Vol. 3 of 3 Part D: Design Summary Report, Tunnels and 3rd Ave. South Station Excavation," by Parsons Brinkerhoff Quade & Douglas, Inc., dated 3-3-1986.

PSB Demolition and Shoring Project
Seattle, Washington

GENERALIZED SUBSURFACE PROFILE A-A'

November 2004
21-1-20116-002

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FIG. 3

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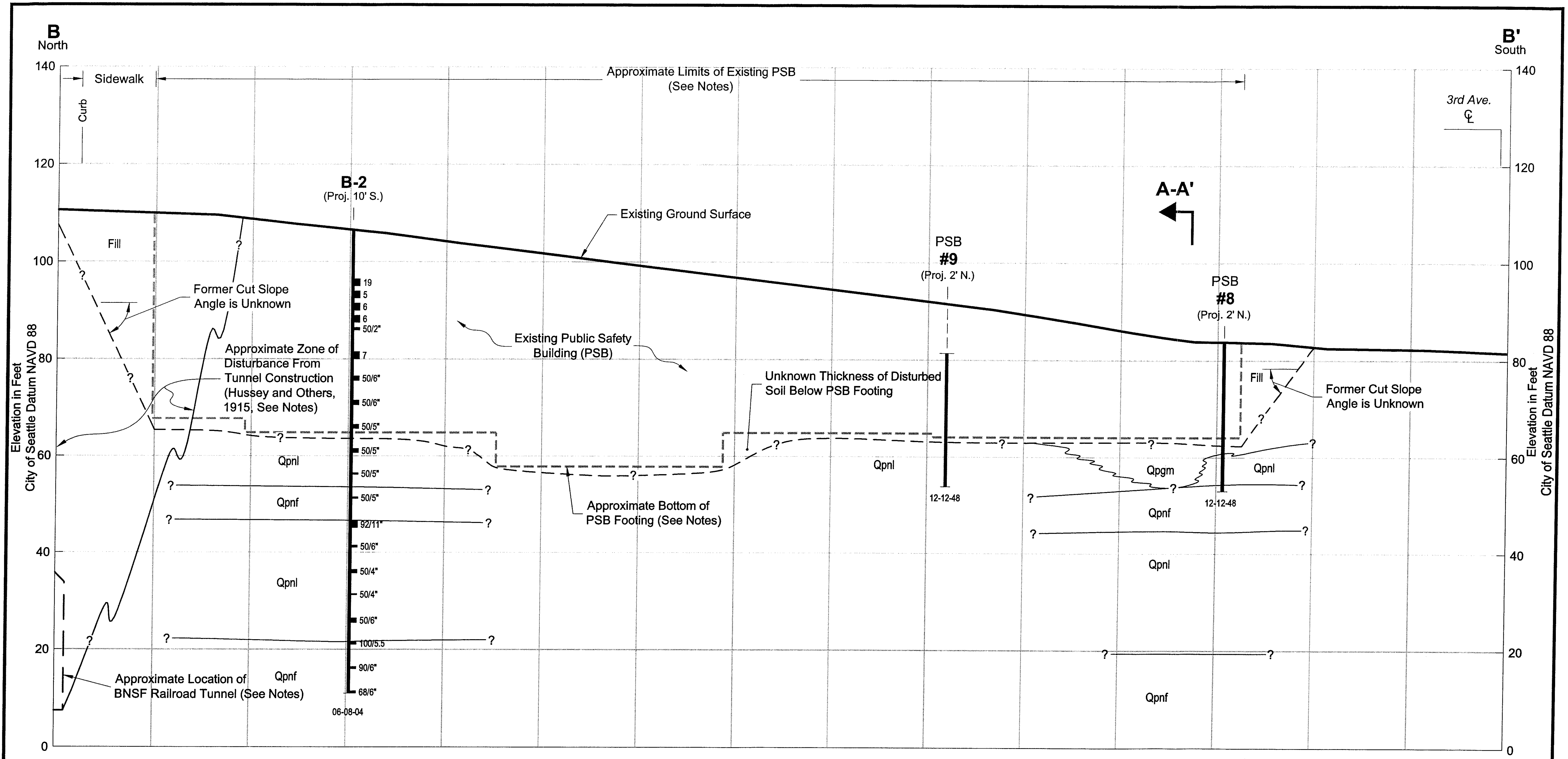

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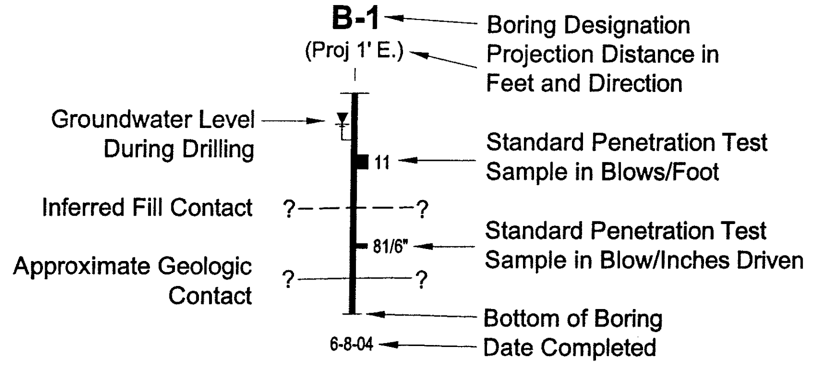

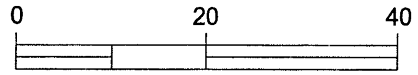

Scale in Feet
Horizontal = Vertical

NOTES

1. This subsurface profile is generalized and interpreted from materials observed in soil borings and reported on borings by others. Variations between the profile and actual conditions may exist.
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4. Approximate location of BNSF Railroad Tunnel taken from "Columbia Center" report by Shannon & Wilson dated 1982. Approximate Zone of Disturbance from Tunnel Construction (Hussey and Others, 1915) taken from Subsurface Profile C-C' in the above referenced report.

PSB Demolition and Shoring Project
Seattle, Washington

GENERALIZED
SUBSURFACE PROFILE B-B'

November 2004
21-1-20116-002

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FIG. 4

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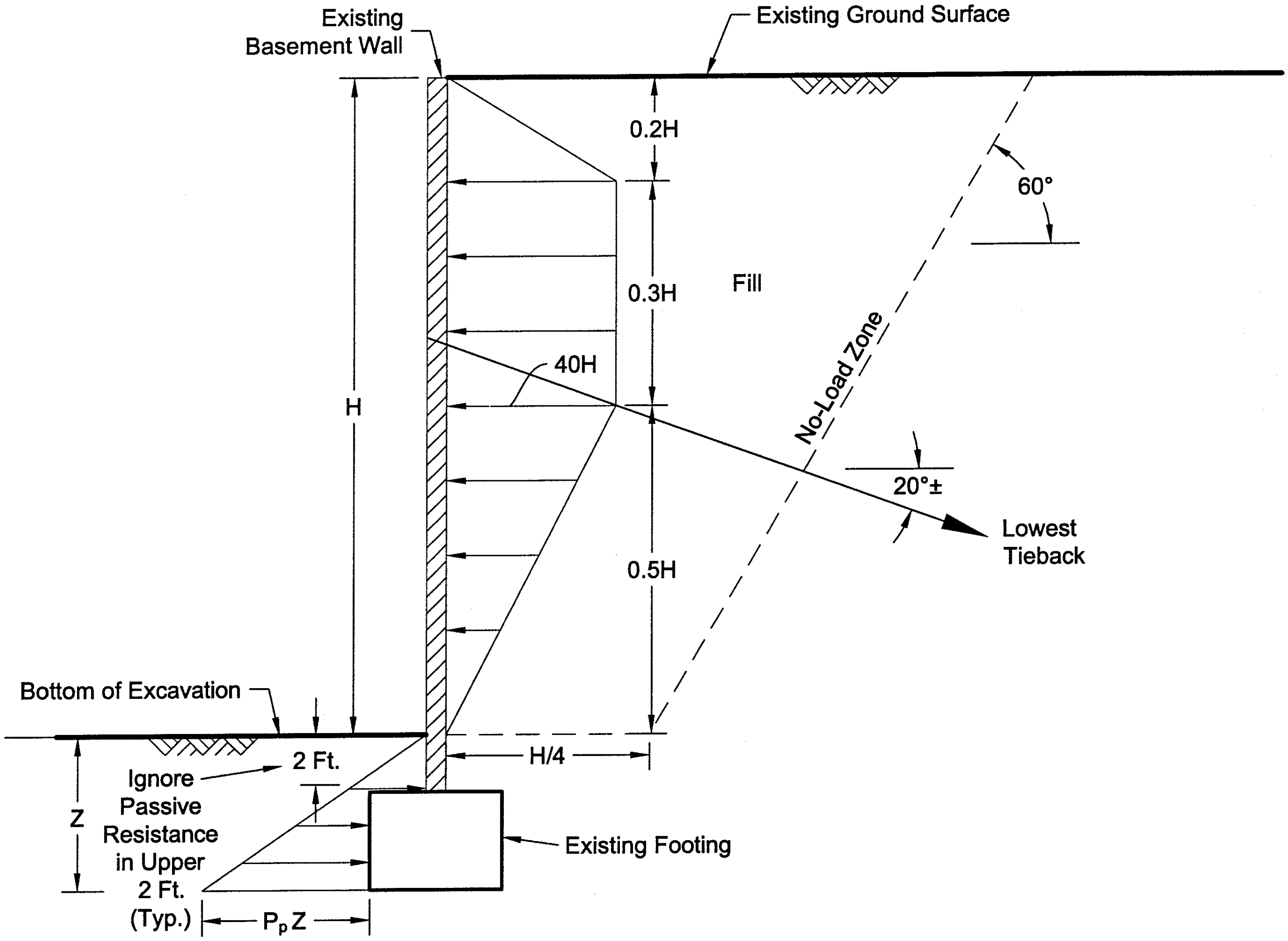

Not to Scale

NOTES

1. P_p computed as equivalent fluid density of 350 pcf.
2. Above pressures assume free draining behind shoring wall.
3. Locate anchors behind no-load zone.
4. Groundwater level assumed to be below the bottom of the excavation.
5. If the ground surface slopes more than 1H to 1.5V, the earth pressure should be adjusted.
6. All earth pressures are in units of pounds per square foot. The active earth pressure diagram applies to a single level wall.
7. Lateral earth pressures for traffic surcharge are shown on Figure 10.

LEGEND

H = Wall Height (feet)

Z = Footing Depth (feet)

40H = Apparent Earth Pressure for Fill (Loose silty Sand and soft sandy, silty Clay)

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RECOMMENDED SHORING DESIGN CRITERIA TO EXISTING BASEMENT LEVEL: SINGLE-LEVEL TIEBACK WALL

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21-1-20116-002

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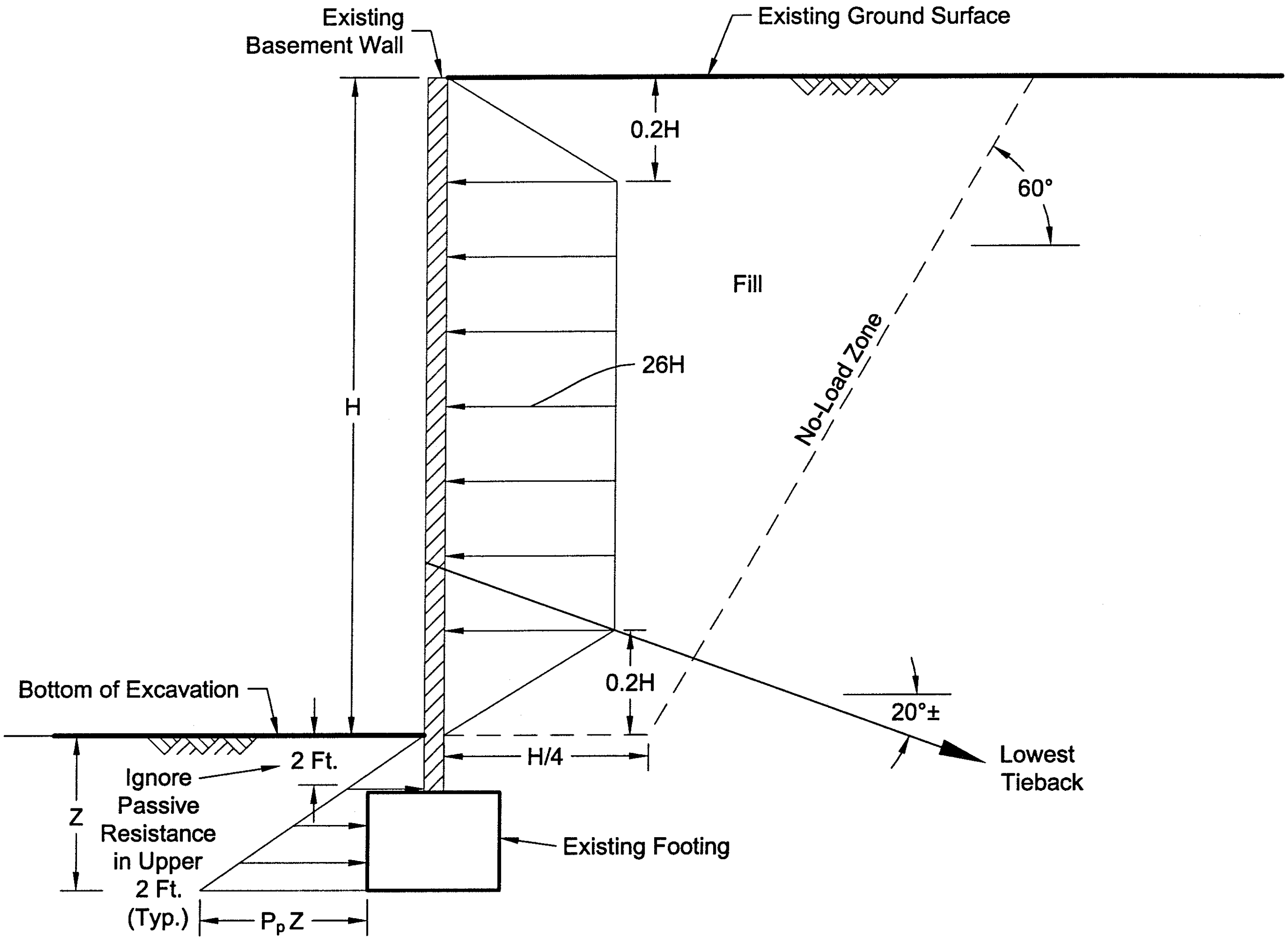
FIG. 5

File: I:\Drafting\21\20116-002\21-1-20116-002 Fig 5.dwg Date: 11-29-2004 Author: LR



Not to Scale

NOTES

1. P_p computed as equivalent fluid density of 350 pcf.
2. Above pressures assume free draining behind shoring wall.
3. Locate anchors behind no-load zone.
4. Groundwater level assumed to be below the bottom of the excavation.
5. If the ground surface slopes more than 1H to 1.5V, the earth pressure should be adjusted.
6. All earth pressures are in units of pounds per square foot. The active earth pressure diagram applies to a multiple level tieback wall.
7. Lateral earth pressures for traffic surcharge are shown on Figure 10.

LEGEND

H = Wall Height (feet)

Z = Footing Depth (feet)

26H = Apparent Earth Pressure for Fill (Loose silty Sand and soft sandy, silty Clay)

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RECOMMENDED SHORING DESIGN CRITERIA TO EXISTING BASEMENT LEVEL: MULTIPLE-LEVEL TIEBACK WALL

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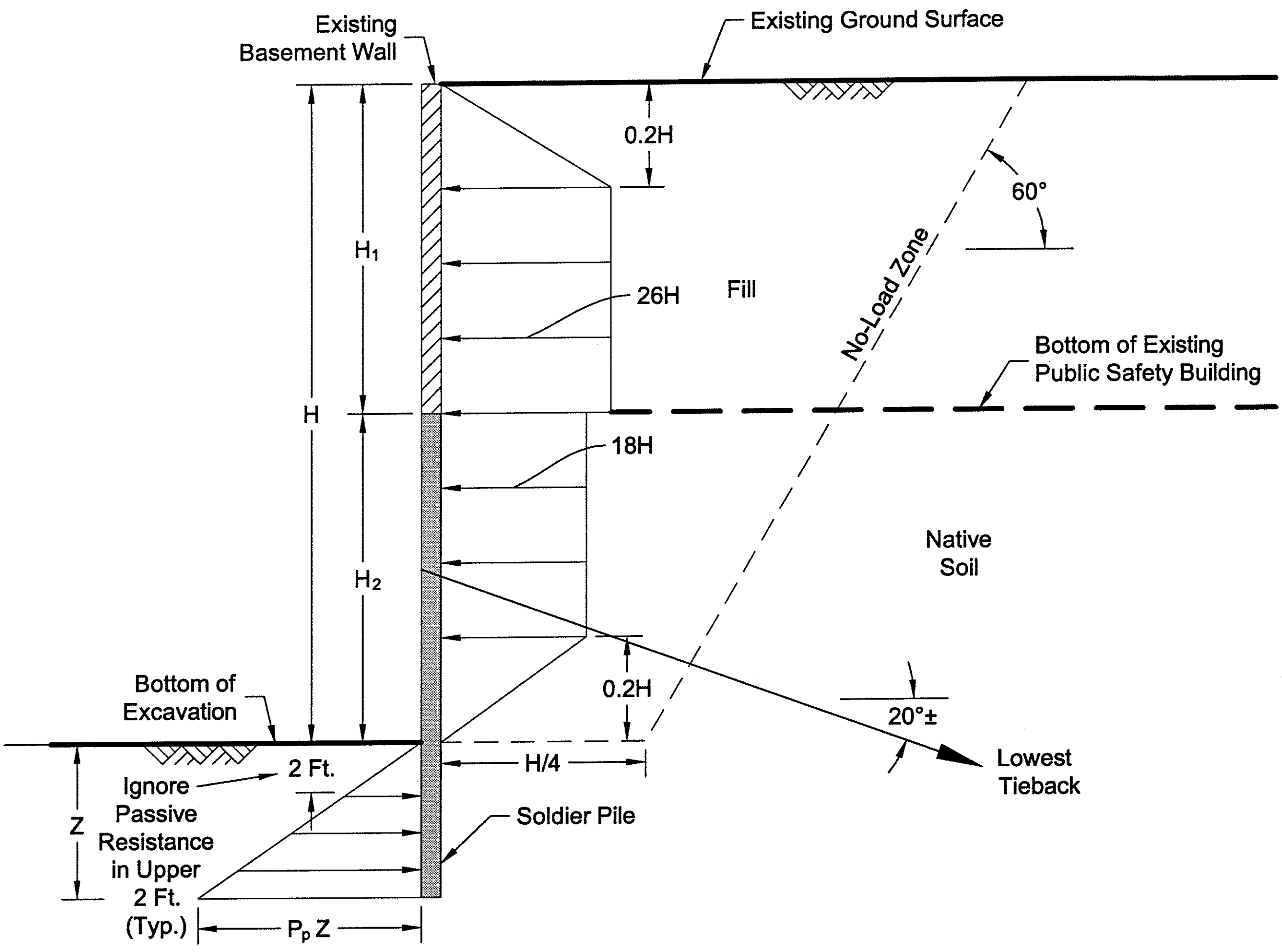
FIG. 6

File: I:\Drafting\21\20116-002\21-1-20116-002 Fig 6.dwg Date: 11-29-2004 Author: LR



Not to Scale

NOTES

1. P_p computed as equivalent fluid density of 450 pcf.
2. Above pressures assume free draining behind shoring wall.
3. Locate anchors behind no-load zone.
4. Groundwater level assumed to be below the bottom of the excavation.
5. If a sloping ground surface exists, the earth pressure should be adjusted.
6. All earth pressures are in units of pounds per square foot. The active earth pressure diagram applies to a multiple level tieback wall.
7. Wall embedment (Z) should consider kickout resistance. Embedment should be determined by satisfying horizontal static equilibrium at the bottom of the pile. Minimum recommended embedment is 8 feet.
8. Lateral pressures for traffic surcharge are shown on Figure 10.
9. The recommended pressure diagrams are based on a continuous wall system. If soldier piles with laggings are used, apply active pressure over the width of the soldier piles below the bottom of the excavation and apply passive resistance over twice the width of the piles or the spacing of the piles, whichever is smaller.
10. Use 80 percent of the design for computing moment in piles.
11. Fot temporary lagging design use 30 percent of the design pressures.
12. Allowable vertical pile capacity:
    Temporary Skin Friction = 1 ksf; Temporary End bearing = 25 ksf

LEGEND

H = Wall Height (feet)

Z = Embedment Depth (feet)

H_1 = Height of Fill Layer (feet)

H_2 = Height of Native Soil Layer (feet)

26H = Apparent Earth Pressure for Fill (Loose silty Sand and soft sandy, silty Clay)

18H = Apparent Earth Pressure for Native Soil

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RECOMMENDED SHORING DESIGN CRITERIA FUTURE EXCAVATIONS: MULTIPLE-LEVEL TIEBACK SOLDIER PILE WALL

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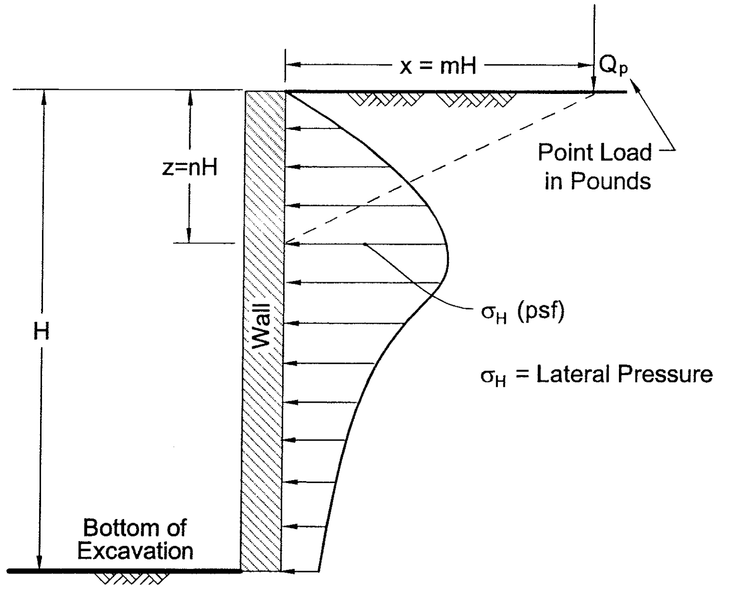
FIG. 7

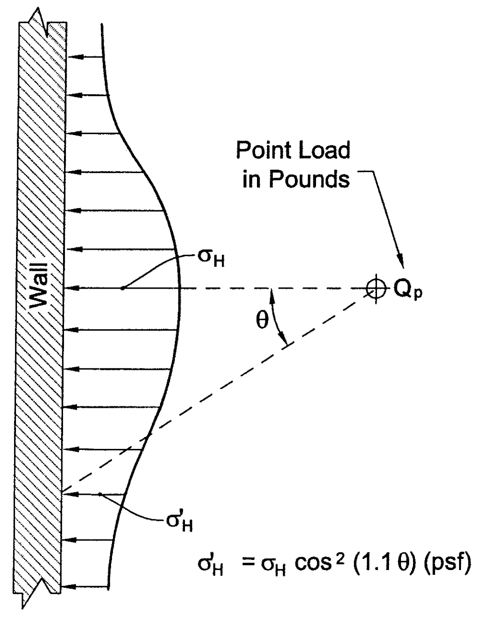
File: I:\Drafting\211\20116-002\21-1-20116-002 Fig 7.dwg Date: 11-29-2004 Author: LR



ELEVATION VIEW

For m ≤ 0.4: $\sigma_H = 0.28 \frac{Q_p}{H^2} \frac{n^2}{(0.16 + n^2)^3}$ (psf)

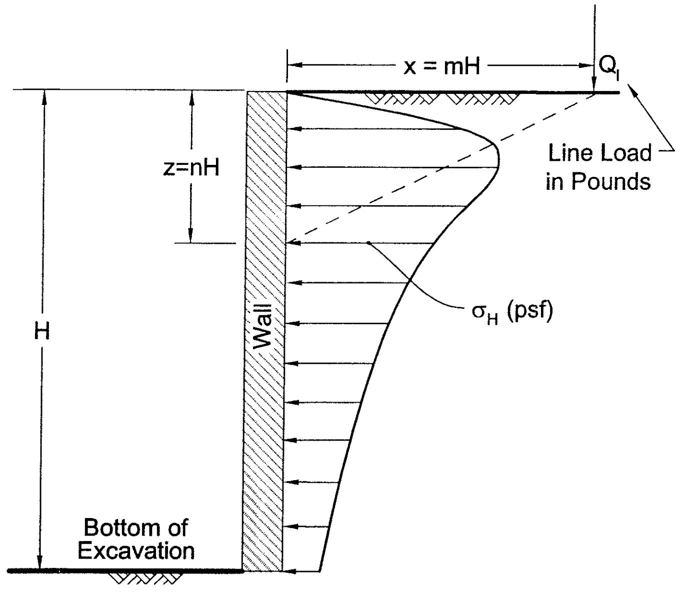
For m > 0.4: $\sigma_H = 1.77 \frac{Q_p}{H^2} \frac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)



PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.2, 1986)



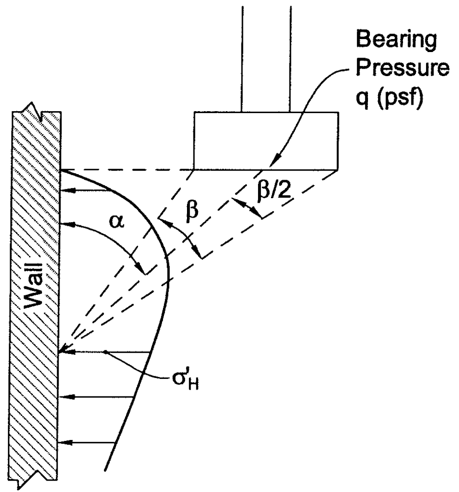
ELEVATION VIEW

For m ≤ 0.4: $\sigma_H = 0.20 \frac{Q_l}{H} \frac{n}{(0.16 + n^2)^2}$ (psf)

For m > 0.4: $\sigma_H = 1.28 \frac{Q_l}{H} \frac{m^2 n}{(m^2 + n^2)^2}$ (psf)

B) LATERAL PRESSURE DUE TO LINE LOAD i.e. NARROW CONTINUOUS FOOTING PARALLEL TO WALL

(NAVFAC DM 7.2, 1986)

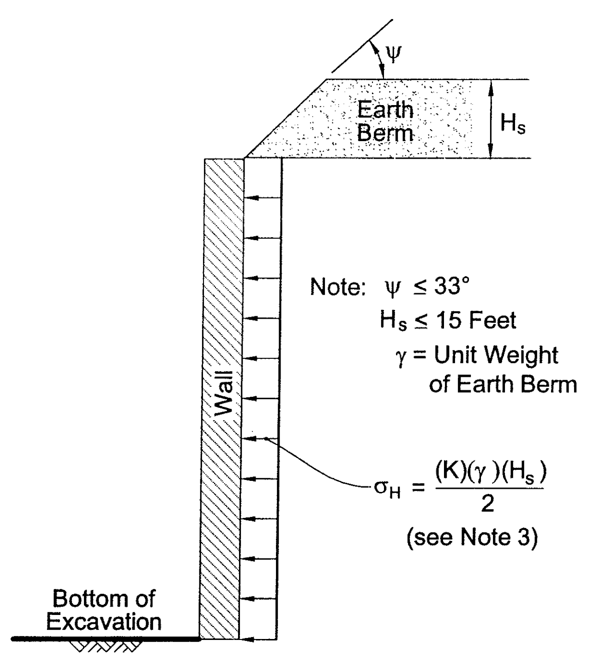


in radians

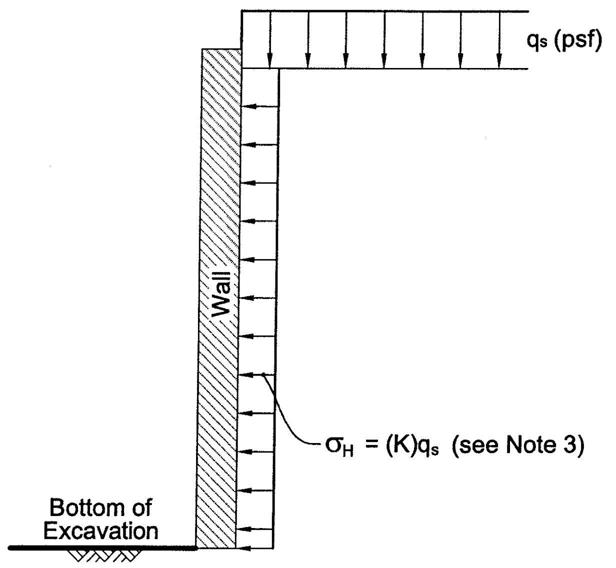
$\sigma_H = \frac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$ (psf)

C) LATERAL PRESSURE DUE TO STRIP LOAD

(derived from Fang, *Foundation Engineering Handbook*, 1991)



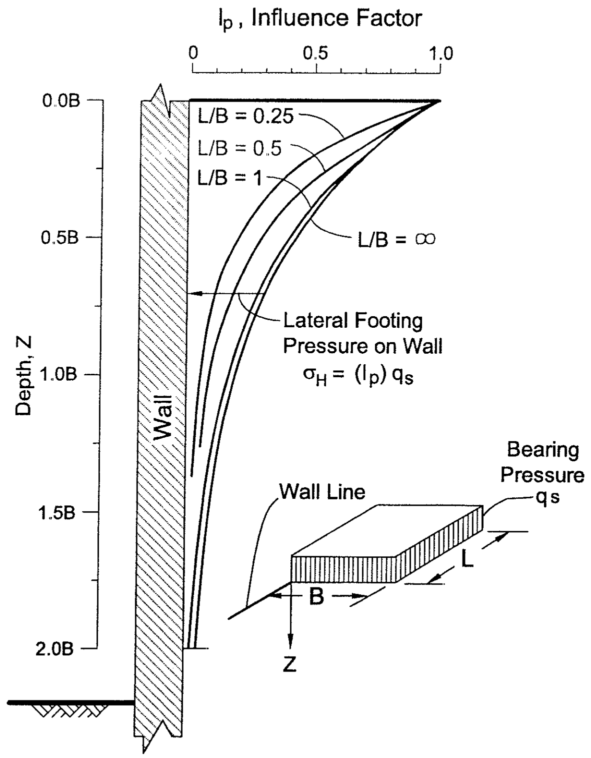
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM OR UNIFORM SURCHARGE

(derived from Poulous and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE TO ADJACENT FOOTING

(derived from NAVFAC DM 7.2, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. See text for recommended K values.

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RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

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FIG. 8

File: I:\Drafting\211\20116-002\21-1-20116-002 Fig 8.dwg Date: 11-29-2004 Author: LR

APPENDIX A

SUBSURFACE EXPLORATIONS

APPENDIX A

SUBSURFACE EXPLORATIONS

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Figure No.

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 INTRODUCTION

Field explorations performed for this project consisted of drilling two soil borings designated borings B-1 and B-2. The approximate locations of the explorations are shown on Figure 2. Shannon & Wilson, Inc. determined the boring locations by measuring from existing site features with a tape measure.

A.2 DRILLING PROCEDURES

The borings were drilled to depths ranging from 37.0 to 95.5 feet on June 3 through 8, 2004. Boart Longyear, formerly Geo-Tech Explorations, Inc., of Kent, Washington, under subcontract to Shannon & Wilson, Inc., drilled the borings using a track-mounted drill-rig. The upper 6 to 7.5 feet were excavated using an air lance and vactor truck to reduce the potential for damaging utilities. Hollow-stem auger methods were used to drill to a depth of approximately 15 feet in boring B-1 and a depth of 26.5 feet in boring B-2. The mud rotary method was used to drill the rest of the boring. The open-hole mud-rotary method consists of drilling subsurface soils and removing the cuttings by circulation of drilling mud. The drilling mud is a mixture of bentonite and water. Cuttings from the boring are deposited in a settling tank at the ground surface and the mud is recirculated into the boring. Steel casing below the hollow-stem auger was not required to advance the borings.

A.3 SOIL SAMPLING

A geologist from our firm was present throughout the boring program to observe drilling, collect representative samples for subsequent laboratory testing, and prepare descriptive field logs of the borings. Disturbed samples were taken at approximately 2.5-foot depth intervals in the upper 20 feet and at 5-foot depth intervals at depths greater than 20 feet. Sampling was performed in conjunction with Standard Penetration Tests (SPTs). All samples retrieved were classified by our field representative, placed in airtight containers, and transported to the Shannon & Wilson, Inc. laboratory in Seattle for further classification and testing. Each soil sample was classified according to a modified version of the Unified Soil Classification System (USCS), which is presented in the Soil Classification and Log Key (Figure A-1). Sample classification was based on American Society for Testing and Materials (ASTM) D 2487-98, Standard Test Method for Classification of Soil for Engineering Purposes, or ASTM D 2488-93, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).

SPTs were performed in general accordance with ASTM Designation: D 1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils. The SPT consists of driving a 2-inch outside-diameter (O.D.), 1.375-inch inside-diameter (I.D.), split-spoon sampler 18 inches into the bottom of the borehole with a 140-pound hammer falling 30 inches. The number of blows required to achieve each of three 6-inch increments of sampler penetration is recorded. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value), or blow count. This value is an indicator of the relative density or consistency of the soils. Whenever 50 or more blows are required to cause 6 inches of penetration, driving is generally stopped and the number of blows and corresponding penetration recorded. Samples recovered from the split-spoon sampler are disturbed but are generally representative of the soils encountered. The results of the SPTs are plotted on the boring logs in this Appendix.

A.4 BORING LOGS

Boring logs for borings B-1 and B-2 are presented as Figures A-2 and A-3. A boring log is a written record of the subsurface conditions encountered. It describes the geologic units (layers) encountered in the boring and the USCS symbol of each geologic layer. It also includes the water content (where tested) and blow counts. Other information shown on the boring logs includes groundwater level observations made during drilling, approximate surface elevation, types and depths of sampling, and Atterberg Limits (where tested). No groundwater monitoring wells were installed in borings B-1 and B-2.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND* - Fine - Medium - Coarse	 #200 to #40 (0.08 to 0.4 mm) #40 to #10 (0.4 to 2 mm) #10 to #4 (2 to 5 mm)
GRAVEL* - Fine - Coarse	 #4 to 3/4 inch (5 to 19 mm) 3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

▧	Bent. Cement Grout	░	Surface Cement Seal
▨	Bentonite Grout	■	Asphalt or Cap
▩	Bentonite Chips	▤	Slough
□	Silica Sand	▩	Bedrock
⊟	PVC Screen		
◫	Vibrating Wire		

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SOIL CLASSIFICATION AND LOG KEY

August 2004 21-1-20116-002

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FIG. A-1
Sheet 1 of 2

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

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SOIL CLASSIFICATION AND LOG KEY

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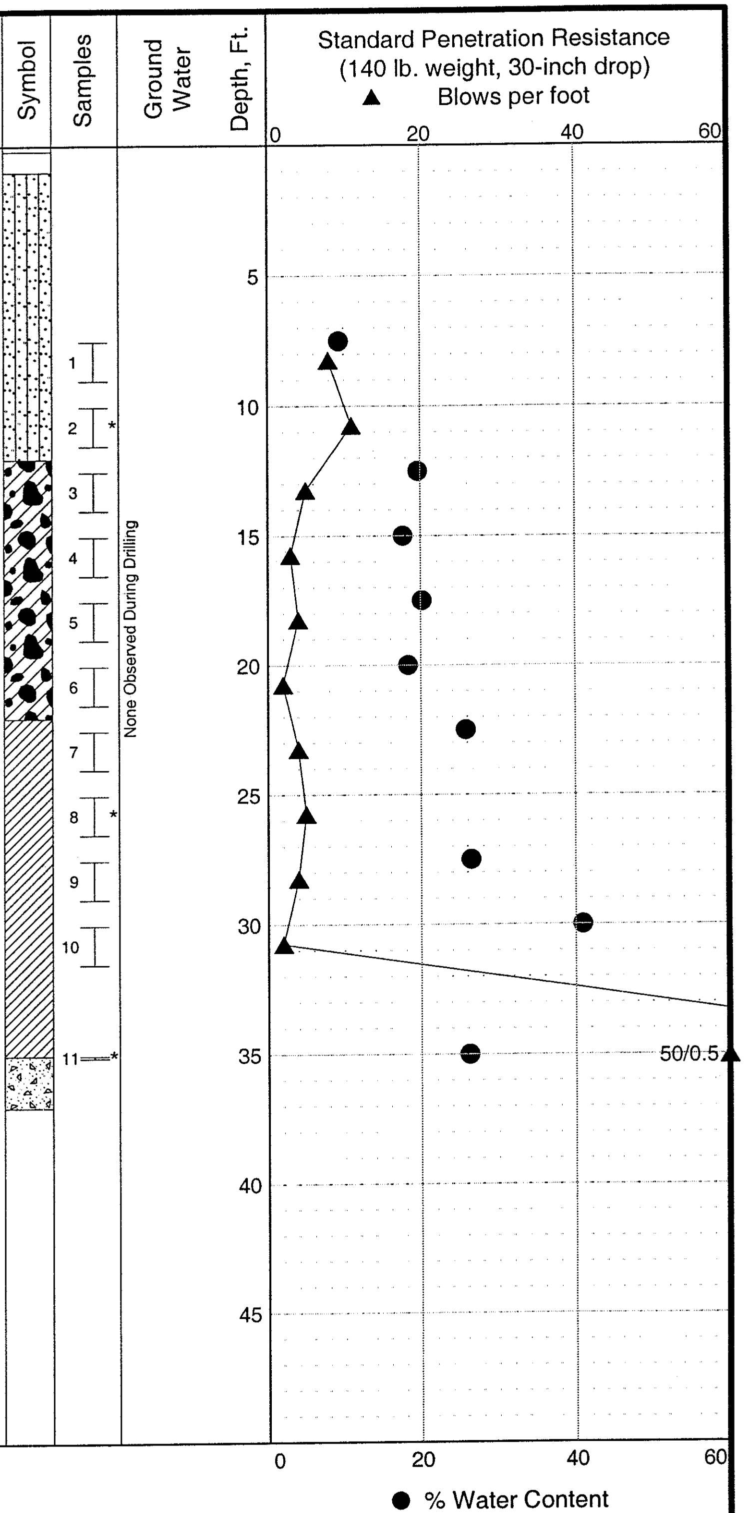
FIG. A-1
Sheet 2 of 2

SOIL DESCRIPTION Surface Elevation: Approx. 104.5 Ft.	Depth, Ft.
Brick pavement.	0.2
Crushed rock subbase.	1.0
Loose, brown, gravelly, silty SAND; moist; scattered wood debris and roots in upper 7 feet; (Fill) SM. Note: Boring was vacuum excavated to 7.5 feet. Soil descriptions above 7.5 feet are inferred from field observations.	12.0
Very loose to loose, gray, silty, sandy, clayey GRAVEL; moist; scattered brick debris; (Fill) GC/CL.	22.0
Medium stiff to soft, gray, trace of sand to sandy, silty CLAY, trace of gravel; moist to wet; disturbed texture, scattered brick fragments; (Fill) CL.	35.0
Concrete.	37.0

BOTTOM OF BORING
COMPLETED 6/3/2004

Note: Boring terminated at 37.0 feet on concrete obstruction (footing).



LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. The boring was performed using Mud Rotary and hollow-stem auger drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING B-1

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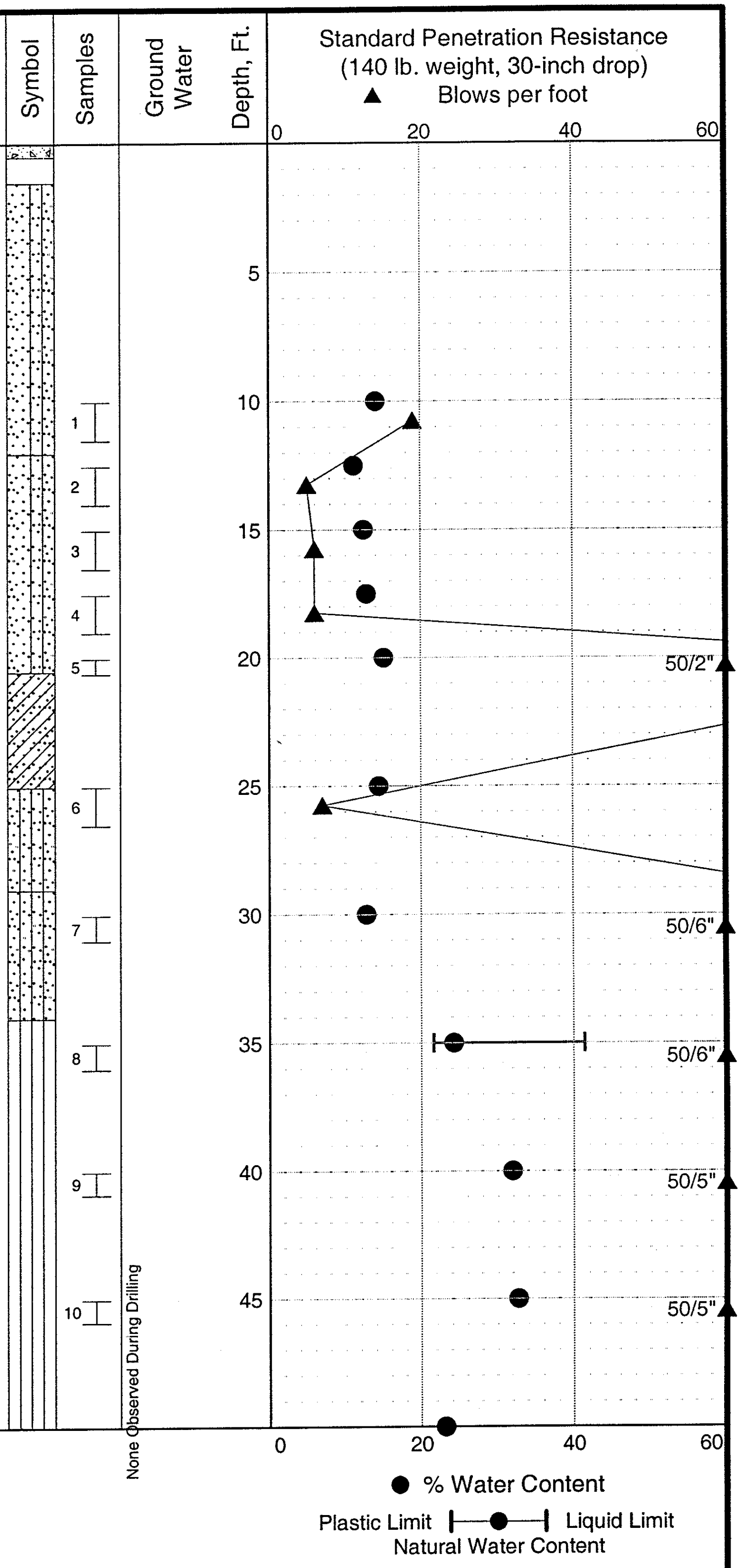
FIG. A-2

SOIL DESCRIPTION Surface Elevation: Approx. 106.5 Ft.	Depth, Ft.
Concrete.	0.5
Crushed rock.	1.5
Loose to medium dense, brown, slightly silty, gravelly SAND; moist; scattered layers of sandy gravel, abundant brick debris, locally sandy clay at bottom; (Fill) SP-SM. - Brick debris between 4.5 and 6.5 feet. Note: Boring was vacuum excavated to 6 feet. Soil descriptions above 6 feet are inferred from field observations.	12.0
Loose, brown, slightly silty to silty, fine to medium SAND, trace of gravel; moist; scattered clayey silt clasts, scattered iron-oxide staining; (Fill) SP-SM/SM.	20.5
Very dense, gray-brown, slightly gravelly, silty, clayey SAND; moist; intermixed with silty clay pockets; (Fill) SC.	25.0
Loose, brown, silty, fine SAND, trace of coarse sand and fine gravel; moist; abundant slightly clayey silt clasts; (Fill) SM.	29.0
Very dense, gray, gravelly, silty SAND; moist; scattered iron-oxide staining; (Qpgt) SM.	34.0
Very dense, gray-brown, slightly fine sandy SILT to silty, fine SAND; moist; locally slightly clayey at top, abundant fine organics; (Qpnl) ML/SM. - Abundant thin peat seams below 45.0 feet.	

CONTINUED NEXT SHEET



LEGEND

* Sample Not Recovered

Ɪ Standard Penetration Test

NOTES

1. The boring was performed using Mud Rotary and hollow-stem auger drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING B-2

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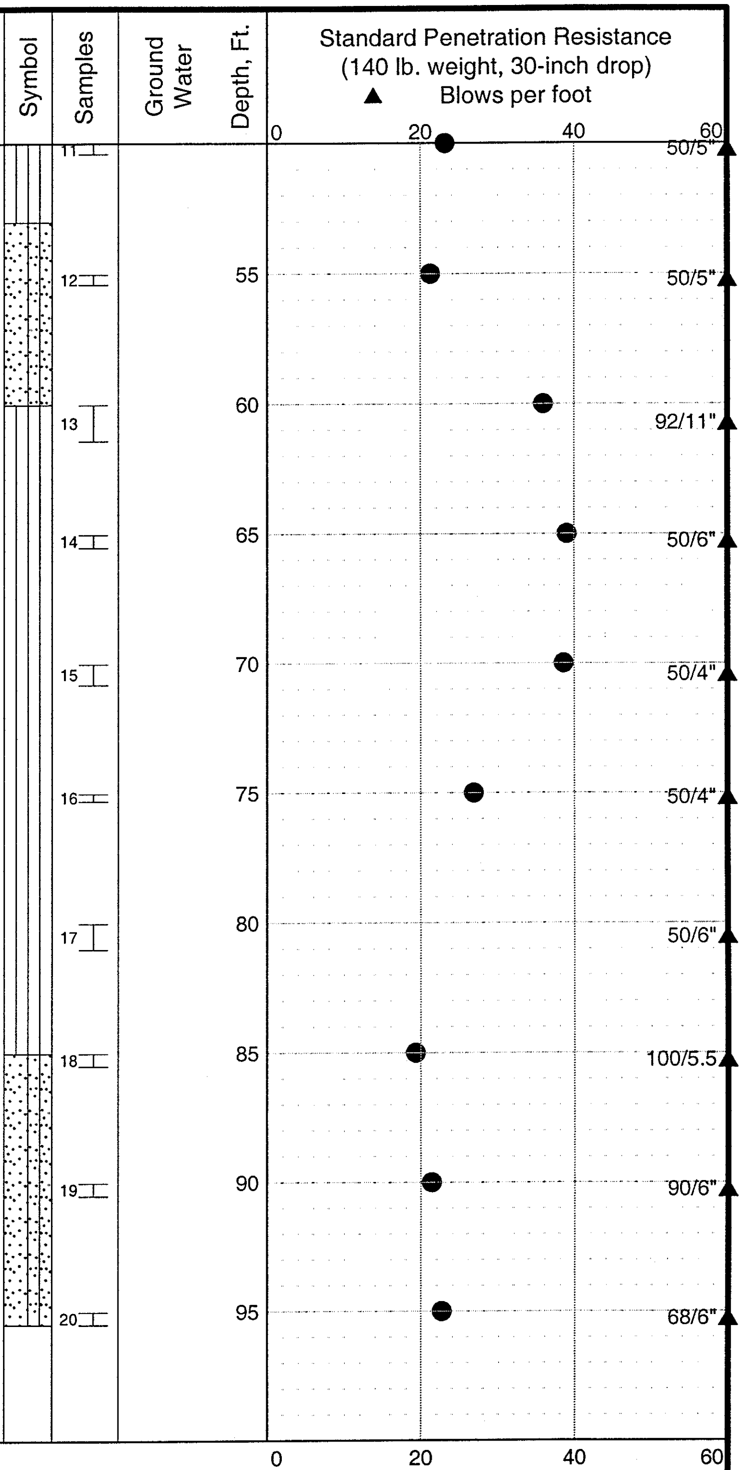
FIG. A-3
Sheet 1 of 2

SOIL DESCRIPTION Surface Elevation: Approx. 106.5 Ft.	Depth, Ft.
Very dense, gray-brown, slightly silty, fine to medium SAND; wet; abundant pumice; (Qpnf) SP-SM.	53.0
Very dense, gray-brown, fine sandy SILT, hard slightly clayey SILT and silty CLAY, trace of fine sand; moist; interbedded, abundant very fine organic fragments, peat seams, and pumice fragments; (Qpnl) ML.	60.0
Very dense, dark gray-brown, slightly silty to silty SAND; moist to wet; abundant fine sand and silt seams, abundant pumice; (Qpnf) SP-SM/SM.	85.0
BOTTOM OF BORING COMPLETED 6/8/2004	95.5



LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. The boring was performed using Mud Rotary and hollow-stem auger drilling methods.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. Refer to KEY for explanation of symbols, codes and definitions.
6. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING B-2

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FIG. A-3
Sheet 2 of 2

MASTER_LOG2 21-20116.GPJ SHAN_WIL.GDT 11/29/04

Log: XHL Rev: WLM Typ: LKD

APPENDIX B

LABORATORY TEST RESULTS

APPENDIX B

LABORATORY TEST RESULTS

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APPENDIX B

LABORATORY TEST RESULTS

B.1 INTRODUCTION

This appendix presents laboratory test procedures and results for tests performed on soil samples obtained from our field explorations. Selected samples recovered from the borings were tested to determine index properties and engineering characteristics of the subsurface soils.

Laboratory testing was performed in general accordance with the American Society for Testing and Materials (ASTM) standard test procedures at the Shannon & Wilson, Inc. laboratory in Seattle, Washington, during June 2004. Tests included water content determinations, Atterberg Limits, and grain size analyses.

B.2 VISUAL CLASSIFICATION

Each soil sample recovered from the borings was visually reclassified in our laboratory using a system based on the ASTM Designation: D 2487, Standard Test Method for Classification of Soil for Engineering Purposes, or ASTM Designation: D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). These ASTM standards use the Unified Soil Classification System (USCS). The USCS is described in Figure A-1 in Appendix A. The visual classification made using this system allows for convenient and relatively consistent comparison of soils logged by different people and from widespread geographic areas.

Soil classifications have been incorporated into the descriptions on the boring logs presented in Appendix A, Figures A-2 and A-3.

B.3 WATER CONTENT DETERMINATION

The water content of soil samples recovered from the field explorations was determined in general accordance with ASTM Designation: D 2216, Standard Method of Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Comparison of natural water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. Water content is plotted on the boring logs presented in Appendix A.

B.4 GRAIN SIZE ANALYSIS

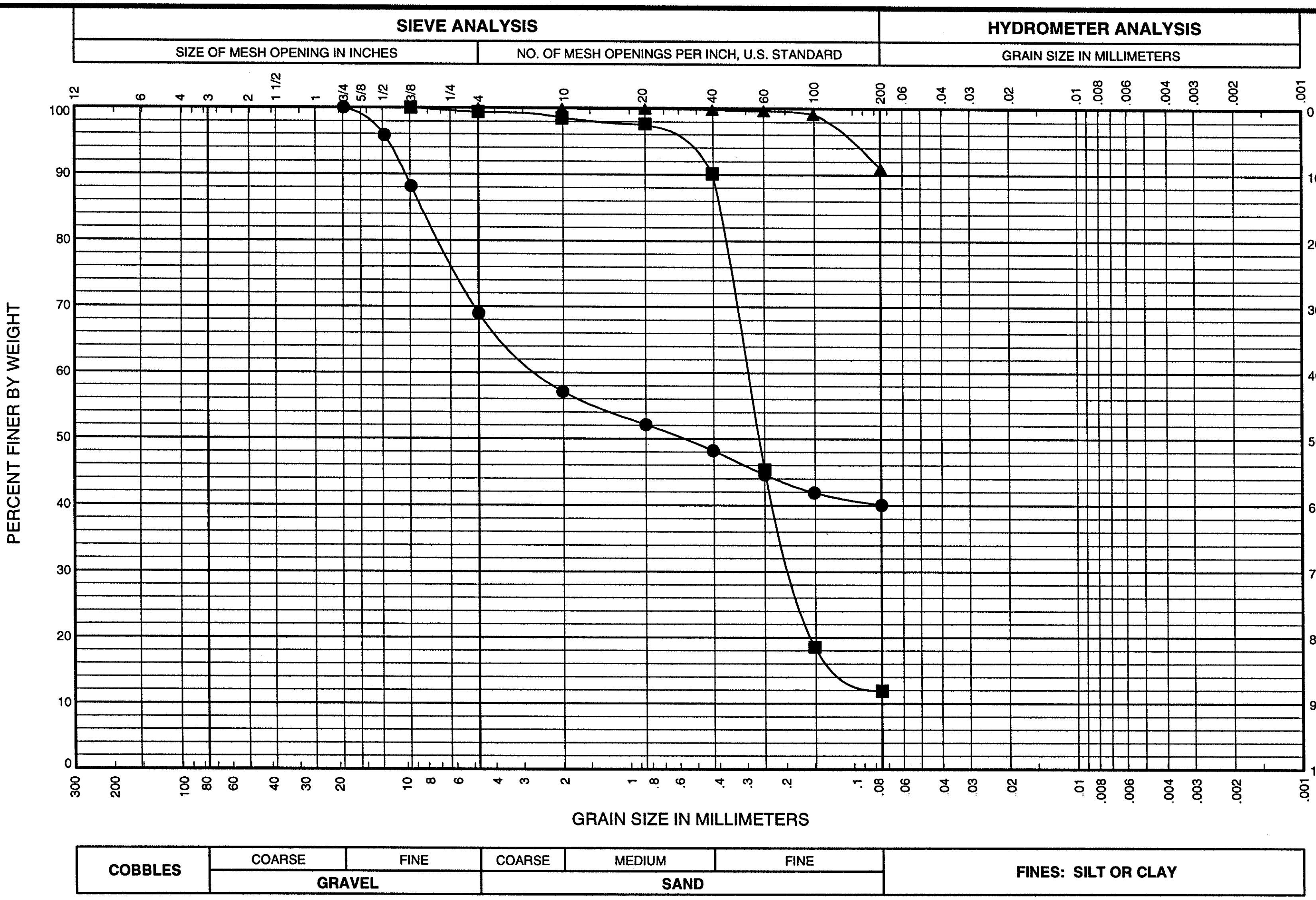
Grain size analyses were performed on selected samples in general accordance with ASTM Designation: D 422, Standard Method for Particle-Size Analysis of Soils. Results of these analyses are presented in Figure B-1 in this appendix. Along with each grain size distribution is a tabulated summary containing the sample description, USCS symbol for the soil group, percentage of fines passing the No. 200 sieve, and the natural water content.

Grain size distribution is used to assist in classifying soils and to providing correlation with soil properties, including permeability, capillary action, and sensitivity to moisture.

B.5 ATTERBERG LIMITS TEST

An Atterberg Limits test was performed on a selected fine-grained sample to determine soil plasticity. The test was performed in general accordance with ASTM Designation: D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg Limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). The results of the Atterberg Limit determination are presented graphically in Figure B-2 and on the boring logs presented in Appendix A.

Atterberg Limits can be used to assist in classification of soils, to evaluate soil consistency (when compared with natural water content), and to provide correlation to soil properties including compressibility and strength. Atterberg Limits are also useful for evaluating the ability to treat the soil using certain ground modification techniques.



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-1, S-6	20.0	GC	Gray, silty, sandy, clayey GRAVEL	40.1	18.3			
■ B-2, S-3	15.0	SP-SM	Brown, slightly silty, fine to medium SAND	11.9	12.5			
▲ B-2, S-9	40.0	ML	Gray, slightly fine sandy SILT	91.0	32.0			

Public Safety Building Demolition
Seattle, Washington

GRAIN SIZE DISTRIBUTION

November 2004
21-1-20116-002

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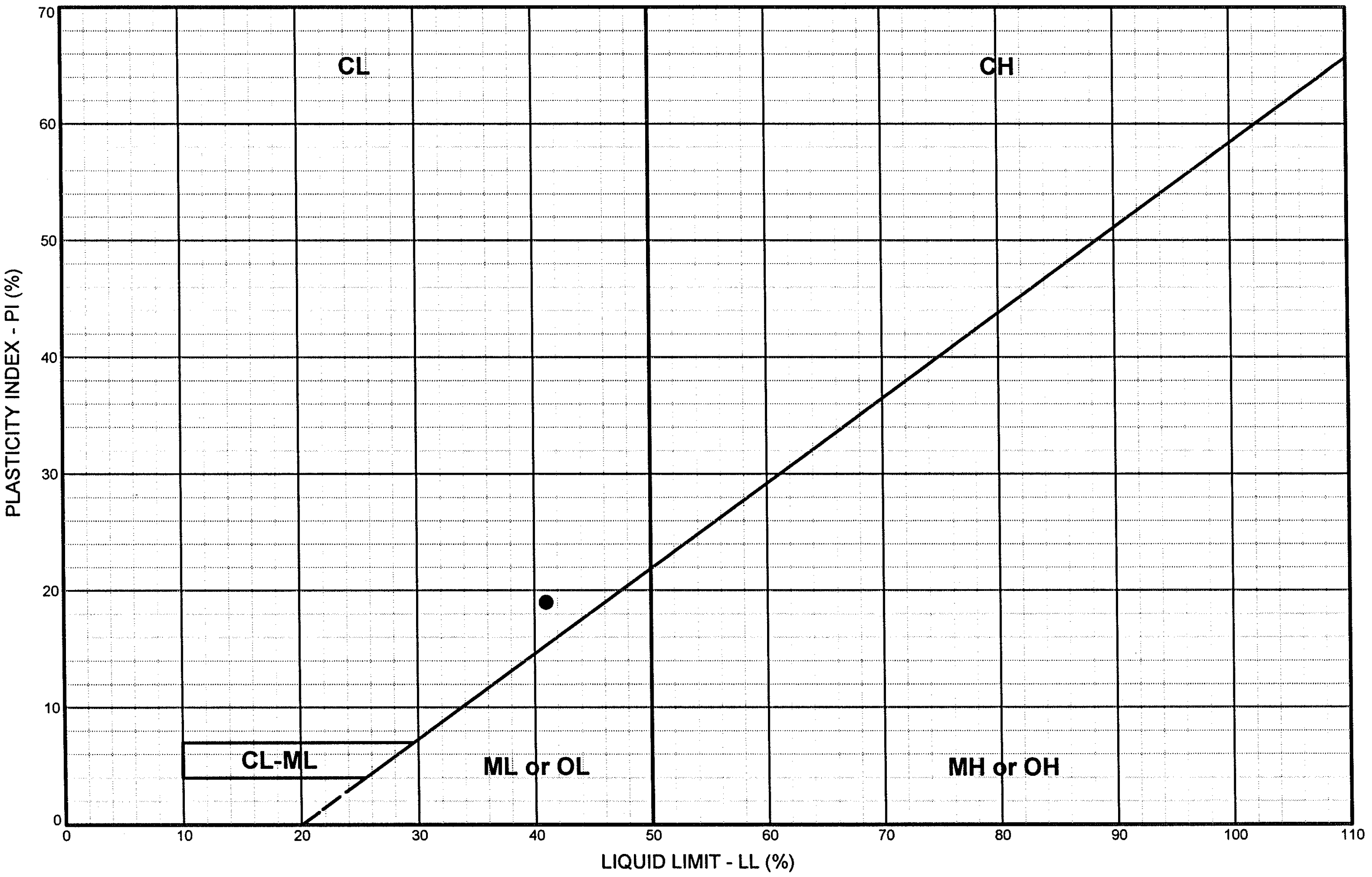
FIG. B-1

FIG. B-1

ATTMAIN1 21-20116.GPJ SHAN WIL.GDT 11/29/04



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● B-2, S-8	35.0	CL	Gray, silty CLAY	41	22	19	24.2	

Public Safety Building Demolition
Seattle, Washington

PLASTICITY CHART

November 2004 21-1-20116-002

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FIG. B-2

FIG. B-2

APPENDIX C

PREVIOUS EXPLORATIONS

APPENDIX C

PREVIOUS EXPLORATIONS

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LIST OF FIGURES

Figure No.

APPENDIX C

PREVIOUS EXPLORATIONS

Explorations were conducted in the project area for the Public Safety Building in 1948 and for the Downtown Seattle Transit Project (DSTP) in 1986. A total of 11 borings were drilled during construction of the Public Safety Building at the approximate locations shown on Figure 2. Our records search of the Department of Planning and Development (DPD) did not produce subsurface information for the adjacent Arctic building and the King County Courthouse. We did, however, obtain subsurface information for the adjacent City Hall, Administration building, Columbia Center, and the DSTP. Two proximal DSTP borings were incorporated into the generalized subsurface profiles, Figures 3 and 5. Logs of the 1948 and the 1986 borings are presented in Figures C-1 through C-5.

Form 30021—100 Pads—Sets of 3—25 Sets to Pad—5-47

TE' ~ BORING REPORT

Raymond Concrete Pile Co.

GOW DIVISION

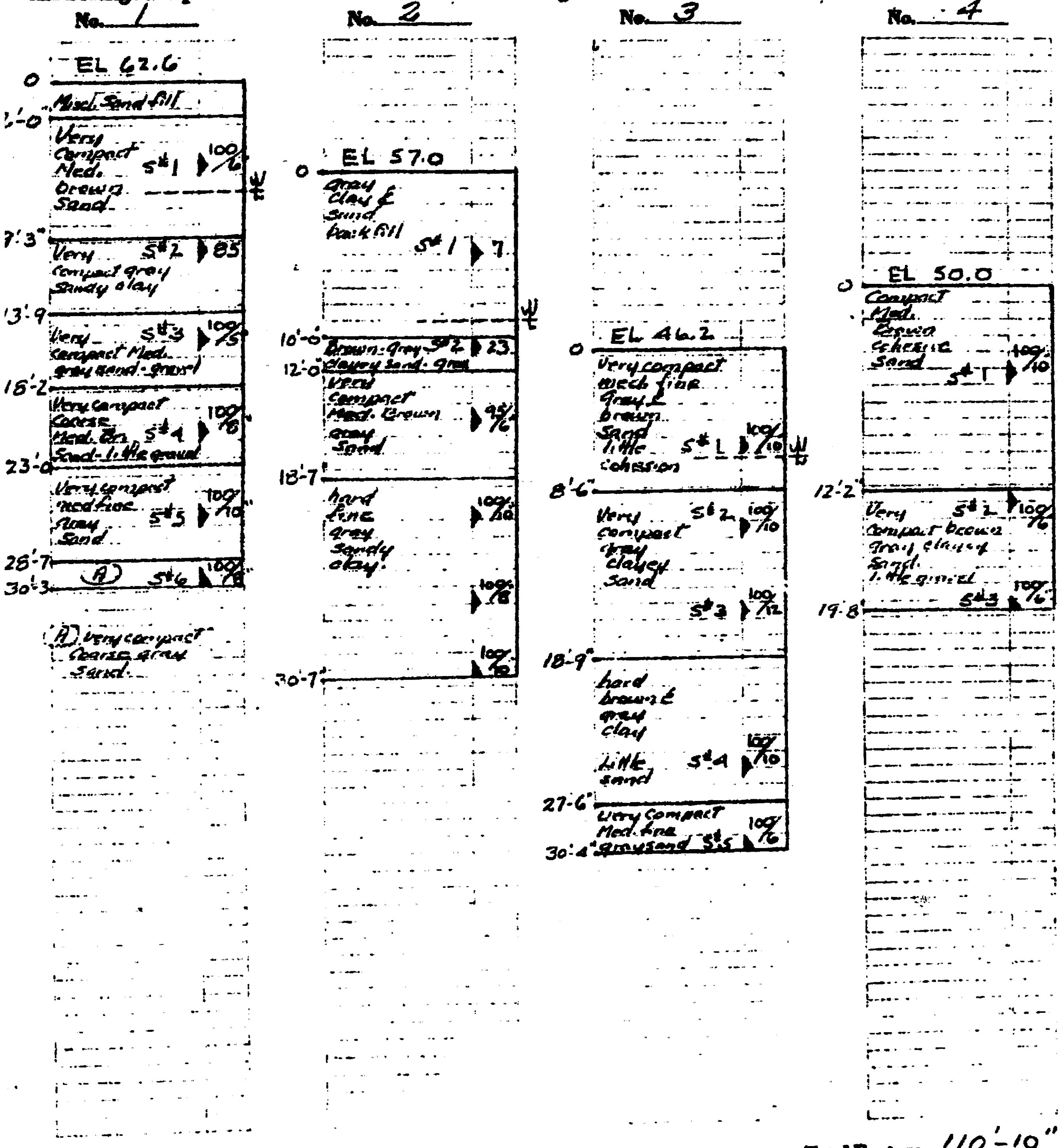
NEW YORK BOSTON

②

To Engineering Dept. City of Seattle Date Dec 12th 1948 Job No. B5358 (old)

Location of Borings Public Safety Bldg. Site

All borings are plotted to a scale of 1" = 8 ft. using City as a fixed datum.



Total Footage 110'-10"

Foreman A. Thomas

Classification by "

Sheet 1 of 3

Figures in right hand column indicate number of blows required to drive sampling pipe one foot, using 140-lb. weight falling 30 inches.

File: I:\Drafting\211\20116-002\21-1-20116-002 Appendix C Logs.dwg Date: 08-03-2004 Author: CNT

Source: Raymond Concrete Pile Co. - GOW Division

FIG. C-1

Form 208B1—160 Pads—Sets of 2—25 Sets to Pad—5-47

TE T BORING REPORT

Raymond Concrete Pile Co.

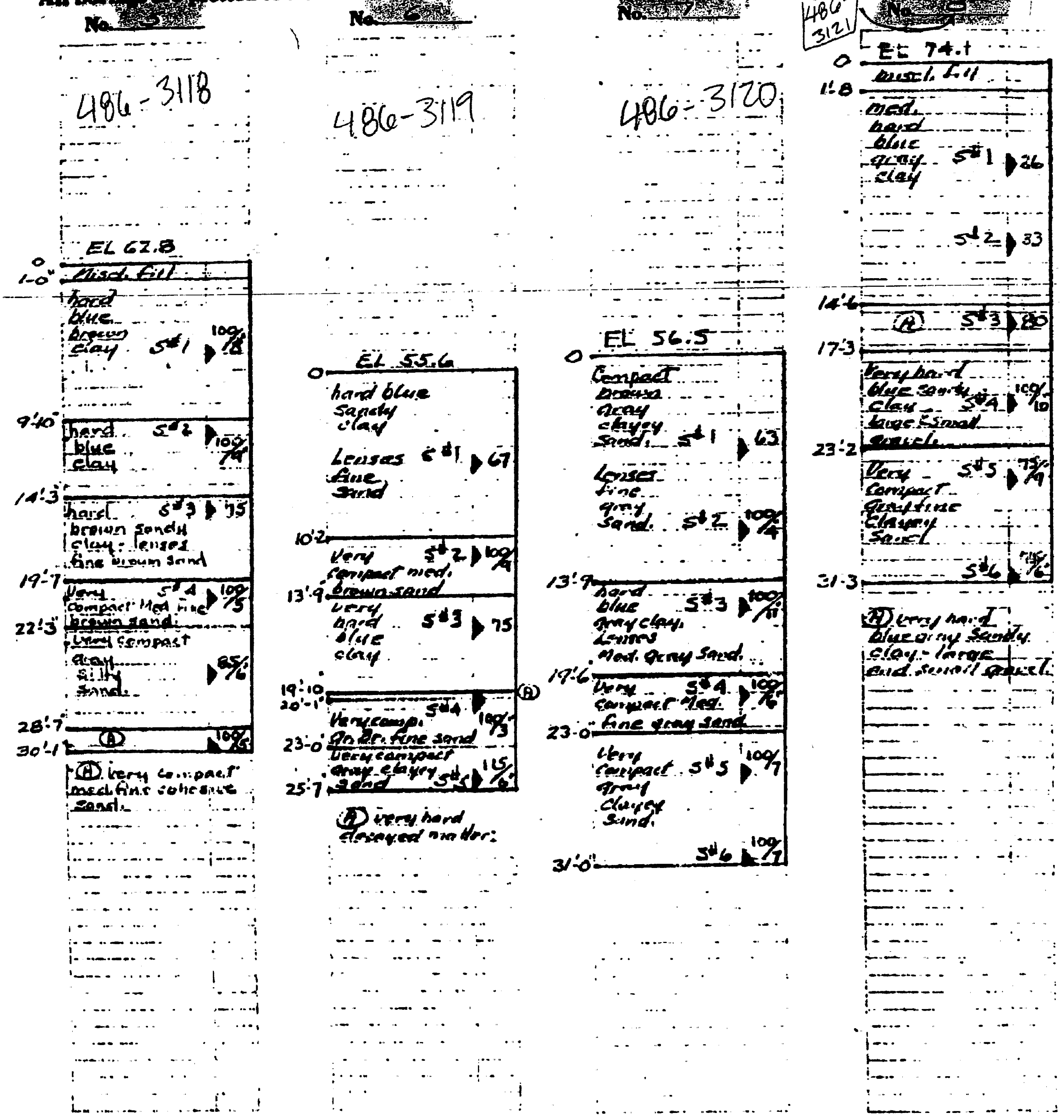
GOW DIVISION

NEW YORK BOSTON

To Engineering Dept. City of Seattle **Date** Dec 12th **194**5 **Job No.** B 5358 (odd)

Location of Borings Public Safety Bldg, Site

All borings are plotted to a scale of 1" = 8 **ft. using** City **as a fixed datum.**



Total Footage 117'11"

Foreman A. Thomas

Classification by "

Sheet 2 **of** 3

Figures in right hand column indicate number of blows required to drive sampling pipe one foot, using 140-lb. weight falling 30 inches.

File: I:\Drafting\211\20116-002\21-1-20116-002 Appendix C Logs.dwg Date: 08-03-2004 Author: CNT

Source: Raymond Concrete Pile Co. - GOW Division

FIG. C-2

Form 209B1—160 Pads—Sets of 3—25 Sets to Pad—5-47

4

TEST BORING REPORT

Raymond Concrete Pile Co.

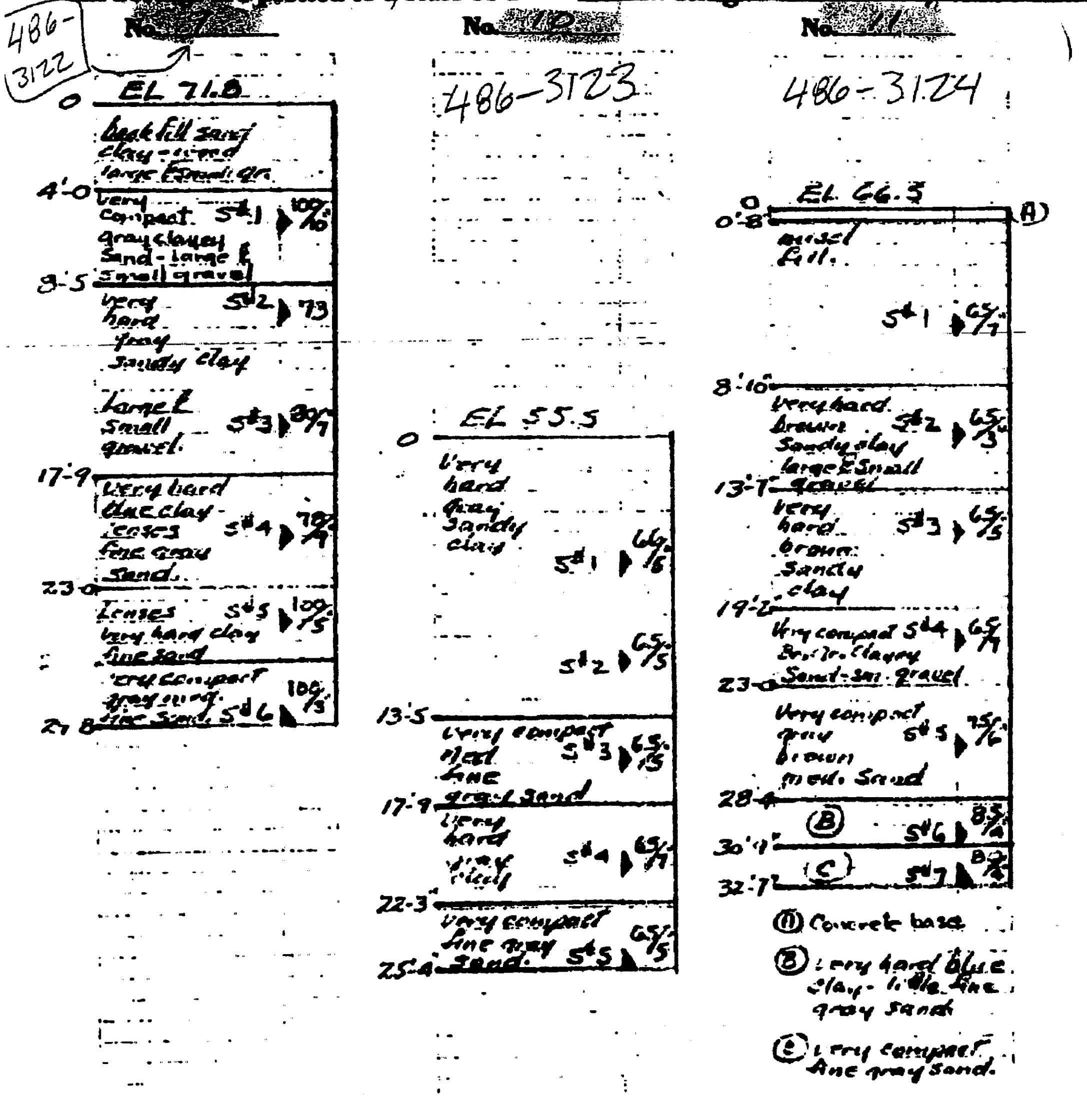
NEW YORK **GOW DIVISION** BOSTON

To Engineering Dept - City of Seattle Date Dec. 12th 1948 Job No. 25358

Location of Borings Public Safety Bldg. Site

All borings are plotted to a scale of 1" = 8 ft. using City as a fixed datum.

No. 7 No. 10 No. 11 No.



Total Footage 87'-7"

Foreman A Thomas

Classification by

Figures in right hand column indicate number of blows required to drive sampling pipe one foot, using 140-lb. weight falling 30 inches.

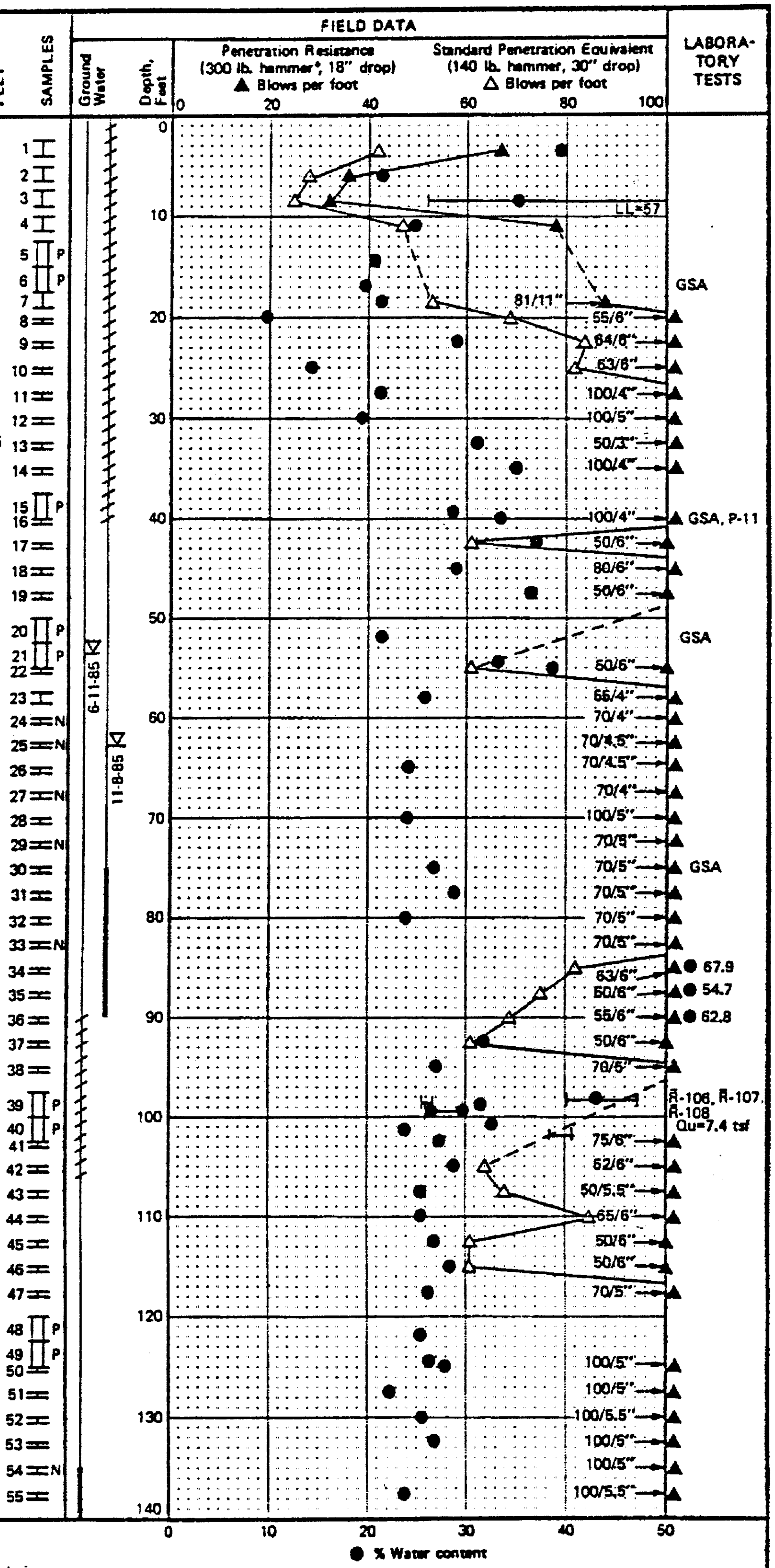
File: I:\Drafting\21\20116-002\21-1-20116-002 Appendix C Logs.dwg Date: 08-03-2004 Author: CNT

Source: Raymond Concrete Pile Co. - GOW Division

FIG. C-3

ELEV., FEET	SOIL DESCRIPTION Surface Elevation: ± 70.6 feet	DEPTH, FEET
	Hard to very stiff, gray, clayey SILT and silty CLAY	0
61.6	Very dense, gray, clayey, silty SAND and GRAVEL; with layers of silt and clay (TILL-LIKE)	9
47.6	Hard, gray, clayey SILT and sandy SILT	23
44.6	Very dense, gray, slightly silty, fine gravelly, fine to coarse SAND	26
38.1	Interbedded, very dense, gray, clean to silty, fine SAND and gray, fine sandy SILT; with occasional clayey silt layers, scattered organics	32.5
28.6	Hard, gray, mottled, slightly clayey to clayey SILT; with organics	42
21.6	Interbedded, very dense, gray, fine sandy SILT and dark gray, clean to silty, fine to medium SAND	49
8.6	Very dense, dark gray, clean, fine to medium SAND; abundant fine to coarse pumice sand; locally silty	62
-13.4	Very dense, brown, fine sandy SILT; thin peat layer	84
-15.4	Very dense, gray, slightly silty, fine to coarse SAND; occasional layers of hard peat and silt	86
-20.4	Hard, gray-brown, clayey SILT; with layers of silty, fine sand and sandy silt; occasional layers of hard peat	91
-45.4	Very dense, dark gray, clean, fine SAND grading to fine to medium SAND at 140 feet; locally slightly silty	116
	(CONTINUED)	

Non-Glacial



*Downhole Hammer

NOTE: The stratification lines represent the approximate boundaries between soil types and the actual transition may be gradual.

LEGEND

⌶	3.25" O.D. split spoon sample
Ⅱ	3" O.D. thin wall sample
G	Grab sample
N	Sample not recovered
⫲	Impervious seal
▽	Water level
▮	Piezometer tip
P	Sample pushed

Atterberg limits:
Liquid limit
Natural water content
Plastic limit

DOWNTOWN SEATTLE TRANSIT PROJECT

LOG OF BORING TB-10

STATION 70+00, 33 FT. (L)
JANUARY 1986 W-4265-00

SHANNON & WILSON, INC.
Geotechnical Consultants

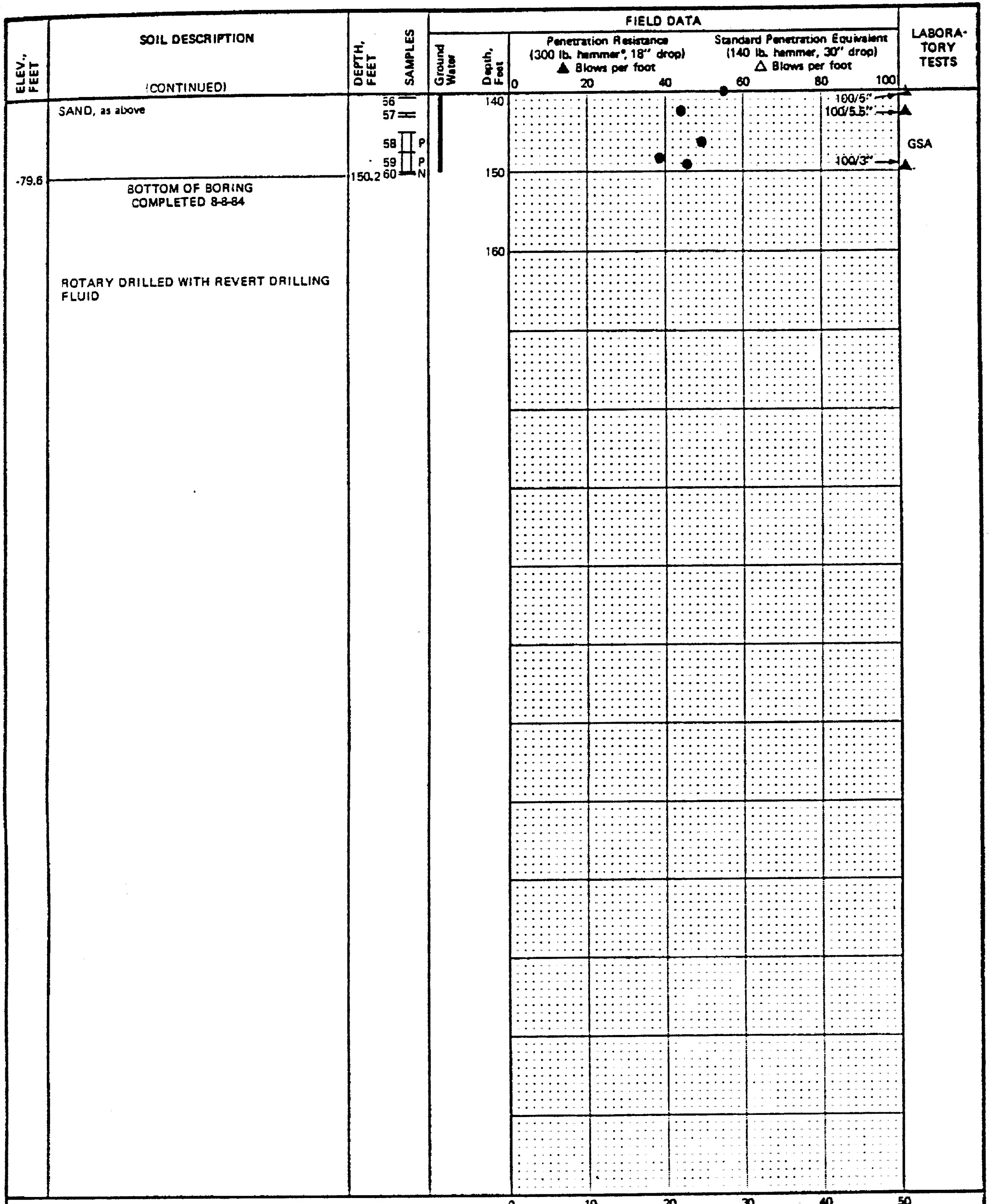
FIG. A-10

File: I:\Drafting\211\20116-002\21-1-20116-002 Appendix C Logs.dwg Date: 08-03-2004 Author: CNT

Source: Shannon & Wilson, Inc.

FIG. C-4
Sheet 1 of 2

DSTP-PB Catalog No. 806.15



SAND, as above

BOTTOM OF BORING
COMPLETED 8-8-84

ROTARY DRILLED WITH REVERT DRILLING FLUID

*Downhole Hammer

NOTE: The stratification lines represent the approximate boundaries between soil types and the actual transition may be gradual.

LEGEND

- 3.25" O.D. split spoon sample
- 3" O.D. thin wall sample
- G Grab sample
- N Sample not recovered
- Impervious seal
- Water level
- Piezometer tip
- P Sample pushed

Atterberg limits:
- Liquid limit
- Natural water content
- Plastic limit

DOWNTOWN SEATTLE TRANSIT PROJECT

LOG OF BORING TB-10

STATION 70+00, 33 FT. (L)

JANUARY 1986 W-4265-00

SHANNON & WILSON, INC.
Geotechnical Consultants

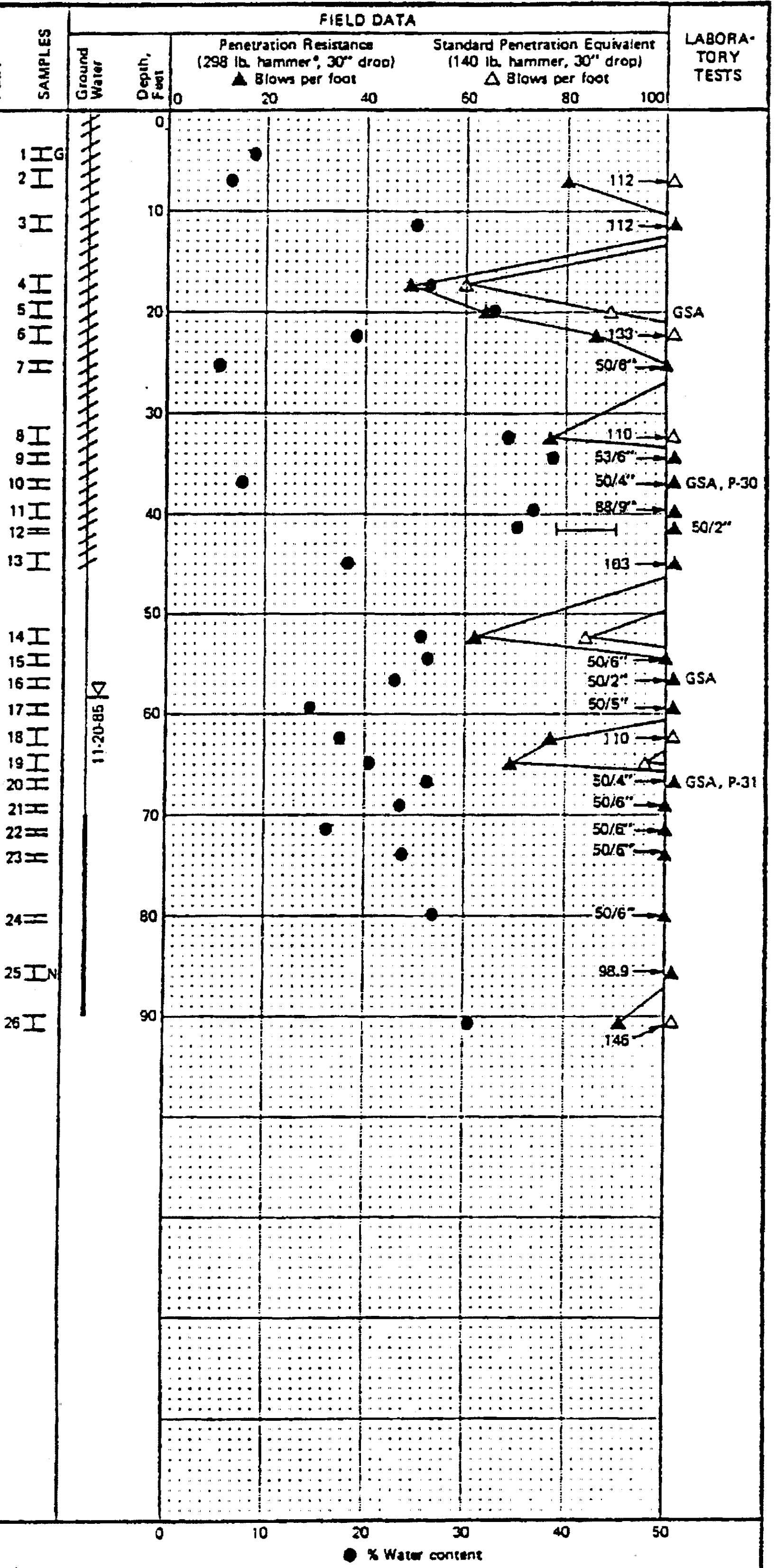
FIG. A-10

Source: Shannon & Wilson, Inc.

FIG. C-4
Sheet 2 of 2

File: I:\Drafting\211\20116-002\21-1-20116-002 Appendix C Logs.dwg Date: 08-03-2004 Author: CNT

ELEV., FEET	SOIL DESCRIPTION Surface Elevation: ± 68.4 feet	DEPTH, FEET
	Brown, mottled, gravelly, silty SAND; scattered cobbles (TILL-LIKE)	0
62.4	Very dense, brown, clean, fine gravelly, fine to coarse SAND	6
59.9	Hard, gray, clayey SILT grading to fine sandy SILT; sand partings	8.5
	Sand lenses and fine gravelly	21
45.9	Very dense, brown, clean, fine to medium SAND	22.5
39.4	Interbedded very dense, gray, fine sandy SILT and hard, clayey SILT; numerous compressed peat and organic lenses, pumice fragments and fine sand layers	29
15.4	Very dense, gray, silty, fine SAND to fine sandy SILT	53
11.4	Very dense, black, clean to slightly silty, fine to medium SAND; abundant pumice fragments, local silty lenses	57
-22.9	BOTTOM OF BORING COMPLETED 10-12-85 HOLLOW STEM AUGER DRILLED WITH WATER	91.3



*Downhole Hammer

NOTE: The stratification lines represent the approximate boundaries between soil types and the actual transition may be gradual.

LEGEND

I 3.25" O.D. split spoon sample
II 3" O.D. thin wall sample
G Grab sample
N Sample not recovered

≠ Impervious seal
▽ Water level
▌ Piezometer tip
P Sample pushed

Atterberg limits:
Liquid limit
Natural water content
Plastic limit

DOWNTOWN SEATTLE TRANSIT PROJECT

LOG OF BORING TB-70
STATION 72+03, 88 FT. (L)
JANUARY 1986 W-4265-00

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-70

File: I:\Drafting\211\20116-002\21-1-20116-002 Appendix C Logs.dwg Date: 08-03-2004 Author: CNT

Source: Shannon & Wilson, Inc.

FIG. C-5

APPENDIX D

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-20116-002

Date: November 24, 2004
To: City of Seattle
c/o Mr. Brad Tong, Shiels Obletz Johnsen

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.