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Geotechnical Report
Systems Retail Building
Seattle, Washington

November 1995

Pine Street Associates
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November 6, 1995

Pine Street Associates
107 Spring Street, Suite 500
Seattle, Washington 98104-6541

Attn: Mr. Matt Griffin

RE: FINAL GEOTECHNICAL REPORT, PROPOSED SYSTEMS RETAIL BUILDING, SEATTLE, WASHINGTON

We are pleased to submit our final geotechnical report for the proposed Systems Retail Building. This report presents the results of subsurface explorations at the project site and our conclusions and recommendations for design of the proposed structure, including recommendations for shoring and foundation design. This final report incorporates revisions and additional recommendations to address comments from Skilling Ward Magnusson Barkshire, Inc.

We appreciate the opportunity to work with you on this project. Please call if you have any questions.

Sincerely,

SHANNON & WILSON, INC.

Martin W. Page

Martin W. Page, P.E.
Senior Engineer

MWP:TMG:WPG/dgw

Enclosure: Geotechnical Report

cc: Rick Fria, Seneca Group
Ken Dahl, SWMB (2 copies)
David Yuan, NBBJ (3 copies)
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SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of geotechnical studies for the proposed Systems Retail Building located in downtown Seattle, Washington. The purpose of this study is to evaluate the subsurface conditions at the proposed building site and to provide geotechnical engineering recommendations for the design and construction of the proposed structure. Our work included subsurface explorations, laboratory testing, and engineering studies to develop the recommendations presented in this report. Our work was authorized on December 22, 1994, by Mr. Matt Griffin of Pine Street Associates. This report includes revisions resulting from comments by Skilling Ward Magnusson Barkshire, Inc. on our previous draft report.

2.0 SITE AND PROJECT DESCRIPTION

The project location is shown on the attached Vicinity Map, Figure 1. The proposed building site consists of the city block bounded by Pine Street on the south, Olive Way on the north. Sixth Avenue on the west and Seventh Avenue on the east. The site presently contains commercial space and a bank building occupying the south half of the block, and a parking garage occupying the north half. The project site is shown on the Site and Exploration Plan, Figure 2.

The ground surface surrounding the site consists of paved streets and sidewalks. The surrounding ground surface gently slopes down to the northwest from elevation 121.5 feet at the southeast corner to elevation 99.9 at the northwest corner of the block. The lowest elevation within the proposed building site is at the northeast corner of the site, within the existing open-air parking area, where the ground surface is at elevation 86 feet.

The blocks surrounding the subject property are all occupied by existing buildings. The streets surrounding the subject property contain numerous buried utilities including abandoned or deactivated lines. Pine Street, adjacent to the south side of the subject property, contains the Metro transit system bus tunnel. This structure was constructed by cut- and-cover methods within a braced excavation. The side walls on the north side of the bus tunnel are located approximately 20 to 25 feet away from the south side property line of the subject property. The bottom of the bus tunnel is at approximate elevation 71 feet.

The proposed building will occupy the entire block and will have the same building footprint as the existing structures. The areaways that presently exist below most of the city sidewalks surrounding the existing buildings will not be used as part of the proposed structure and will be backfilled. The proposed building will consist of a six-story building aboveground with six levels of underground parking. The structure will contain 420,390 square feet of retail space, 540,588 square feet of parking and 77,271 square feet of theater space. The proposed structure will have lower parking levels at elevation 35 feet at the north end of the site and at elevation 45 feet at the south end of the site. The upper levels of parking will be at elevation 85 feet. The proposed depth of the parking garage will require vertical excavations of approximately 65 to 85 feet below adjacent street grade.

We understand based on our discussions with structural engineers, Skilling Ward Magnusson Barkshire Inc. (SWMB), that the proposed structure will have interior columns spaced at approximately 35 feet on-centers in one direction and 52 feet on-centers in the other direction. Interior columns will have dead loads of 2,000 kips and live loads of 900 kips. Perimeter wall loads will include 50 kips per lineal foot (klf) dead load and 12 klf live load.

3.0 EXPLORATIONS AND LABORATORY TESTING

Previous subsurface explorations were completed in the vicinity of the project site by Shannon & Wilson in 1986 (TB-90 and TB-19) and 1994 (B-4 and B-5), Dames & Moore in 1970 (DM-2, DM-3, DM-5, and DM-8), and the Seattle Engineering Department in 1970 (SED-4 and SED-5). Logs of the previous borings are presented in Appendix A as Figures A-4 through A-13. Shannon & Wilson recently completed three additional borings, designated B-1 through B-3, to supplement the existing subsurface information for the project site along Sixth Avenue and Olive Way. Logs of these recent borings are presented in Appendix A as Figures A-1, A-2 and A-3. The locations of the previous and recent borings are shown on the Site and Exploration Plan, Figure 2.

The three recently completed soil borings were each drilled to depths of 91.5 feet (elevations 9.9 to 19.5) on February 2, 3, and 6, 1995 by Associated Drilling Co. of Seattle under subcontract to Shannon & Wilson, Inc. A truck-mounted B-61 drill rig equipped with a 3 3/8-inch inside diameter (I.D.) hollow-stem auger was used.

Standard Penetration Tests (SPTs) were performed at 5-foot intervals in each of the recent borings. The SPT consists of driving a 2-inch outside diameter (O.D.) split-spoon sampler a distance of 18 inches into the bottom of the borehole with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler each of three 6-inch increments was recorded, and the number of blows required to cause the last 12 inches of penetration was termed the Standard Penetration Resistance (N-value). This value is an indicator of the relative density and consistency of the soils.

Samples obtained in the field during recent explorations were classified by a technician from our firm, sealed in jars, and returned to our laboratory where each was visually classified and moisture contents were determined. The results of the SPTs, moisture contents and soil classifications are summarized on the boring logs, Figures A-1 through A-3. Atterberg limits tests (ASTM D 4318) were conducted on two samples of cohesive soils from borings B-1 and B-2. The results of the Atterberg limits tests are presented graphically on the Plasticity Chart, Figure B-1. Grain size distribution analyses (ASTM D 422) were performed on several soil samples from borings B-1 through B-3 to assist in classifying and evaluating engineering properties of the soils. The results of the grain size analyses are presented as Grain Size Distributions on Figures B-2, B-3, and B-4.

4.0 SUBSURFACE CONDITIONS

4.1 Soils

The results of our recent subsurface explorations and previous explorations at the project site are compiled as Generalized Cross-Section Profiles A-A', B-B', C-C', and D-D', presented on Figures 3 through 6. Generalized subsurface profiles through Pine Street were prepared for our 1985 geotechnical report for the Metro Downtown Seattle Transit Project. Subsurface profiles P4 and P5 are presented in Appendix C as Figures C-1 and C-2, respectively. The locations of the profiles are shown on the Site and Exploration Plan, Figure 2.

The project site and the immediate vicinity is mantled with 20 to 30 feet of fill material that consists of very loose to medium dense sand and silt with variable gravel and clay content. Fill was encountered in all borings around the project site down to elevations 74 to 88 feet over the approximate north half of the site, and down to elevations 89 to 105 feet over the south half of the site (reference Figures 3 through 6).

Below the fill material the borings generally encountered 25 to 30 feet of loose to very dense, native, granular soils. Loose soils were encountered locally in the southeast corner of the site (TB-90 and DM-5) to elevations 75 feet and 90 feet. The granular soils generally consisted of layers of silty to clean sand and gravel (glacial outwash), gravelly, sandy clay (glacio-marine drift), and gravelly silty sand (glacio-marine drift). Glacially-derived granular soils were generally dense to very dense with SPT values frequently exceeding 100 blows per foot.

Below the fill and granular native soils, between depths of 25 to 60 feet (elevations 56 to 76 feet), the borings encountered a fairly thick and continuous layer of very stiff to hard silty clay or clayey silt. This layer appeared to be present below the majority of the project site except for the northwest corner. Based on information from the soil borings, the top of the layer appears to slope down slightly to the northwest and eventually disappears before it reaches the northwest corner of the project site. Where the silt/clay layer is not present (northwest corner of the site), the soils consist of very dense granular soils, as described in the previous paragraph.

Layers of very dense sand, gravelly sand, and sandy silt were present below the hard silt and clay layer. Additionally, relatively thin, discontinuous layers of stiff and hard clay were encountered within the very dense granular soils in the borings along 7th Avenue (DM-2, DM-3, and DM-5)

4.2 Groundwater

Groundwater was encountered during drilling of nearly all borings with the exception of SED-4 and SED-5. Groundwater was observed during drilling of our recent borings (B-1, B-2, and B-3) at approximate depths of 55 to 62 feet (elevations 38 to 53 feet). Wet sandy soils were present at these depths. Evidence of groundwater (wet soils) was also observed at a depth of 28 feet during drilling of boring B-2. Piezometers for measuring groundwater levels were installed in borings B-1 and B-2. Subsequent readings of the groundwater levels indicated that groundwater is present at the contact between the very dense gravelly sand and the hard clayey silt, around a depth of 55.5 feet (elevation 55.5 feet in boring B-1), as illustrated on the boring logs and subsurface profiles. Additional groundwater levels observed in previous soil borings by Shannon & Wilson and others are presented on the boring logs and Figures 3 through 6.

Based on the groundwater observations during drilling and information from piezometers, it appears that groundwater or wet soils are present at several different depths across the project site. Due to the complexity of the geology below the site, it is likely that groundwater will be encountered at several different locations and elevations where a more permeable soil layer overlies a less permeable soil, such as gravelly sands overlying silty sands or clay. It is likely that the quantity of groundwater flow will be relatively small because it is derived from locally perched groundwater tables and not from laterally extensive aquifers. It should be noted, however, that groundwater levels at this site will fluctuate seasonally and, at the time of construction, could be higher or lower than indicated by the borings.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General Conclusions

In general, the soils that are present at the proposed excavation depths consist of very dense gravelly, silty sand, sandy silt or hard, silt and clay. Based on the proposed building loads, it is our opinion that the structure can be supported on conventional spread footings bearing in the very dense to hard native soils at the proposed excavation elevations.

The proposed excavation will require use of temporary shoring to support vertical excavation sidewalls. Based on our discussions with the project structural engineers, SWMB, and general contractor, Lease Crutcher Lewis, we recommend that the shoring system consist of soldier piles and lagging rather than a soil nailing system due to the mixed soil layers and high potential for encountering difficult soil and groundwater conditions. In our opinion, the potential cost savings that can generally be realized through use of soil nailing instead of soldier piles and lagging would quickly disappear if difficult soil conditions, such as caving soils and running sand, are encountered.

The following paragraphs present our recommendations for other geotechnical design issues for the project including lateral resistance and lateral earth pressures, seismic design, drainage, floor slabs, fill placement, excavations, temporary shoring, and instrumentation.

5.2 Foundation Design

The results of our studies indicate that the proposed building's columns and perimeter walls can be supported on conventional spread footings or continuous wall footings bearing in very dense to hard native soils. We expect that very dense granular soils will be encountered at the proposed excavation elevation (approximately elevations 35 to 45 feet) over the majority of the building footprint with the exception of the southwest quadrant of the site where hard silt and clay are anticipated (refer to borings B-1 and TB-19).

Footings bearing in the very dense granular soils at this site could be designed for an allowable bearing pressure of 14 kips per square foot (ksf). We anticipate the proposed interior columns will require footings 10 to 14 feet wide and perimeter wall footings up to approximately 4 feet wide. Where hard clay is encountered at footing grade, such as at the southwest quadrant of the site, we recommend that footings be designed for an allowable bearing pressure of 10 ksf. Allowable soil bearing capacities may be increased by 33 percent for temporary earthquake and wind loading as allowed for in the Uniform Building Code (UBC).

The above recommended soil bearing pressures are based on the elastic settlements that will occur upon loading. We estimate that the total settlements of individual spread footings will be less than 1 inch and wall footings will settle less than 3/4-inch with the above recommended allowable bearing pressures. Differential settlements between individual spread footings and within a 25-foot span of wall footing will generally be less than 0.5-inch. It is anticipated that these settlements will occur nearly simultaneously with vertical loading.

Footings should bear at least 3 feet below the lowest adjacent finished grade. If adjacent individual footings are located at different elevations, it is recommended that the horizontal distance between them be at least 1.5 times the elevation difference. Where adjoining continuous footings are at different elevations, it is recommended that the upper footing be stepped down to the lower footing.

All foundation bearing soils must be evaluated by an experienced geotechnical engineer during construction to verify the type of soil present and the adequacy of the footing design. The contract plans and specifications should provide for increasing or decreasing the footing dimensions as appropriate to accommodate changes in soils encountered during excavation, i.e., hard clay vs. dense granular soil.

All loose or soft soil, and all soils disturbed by construction activity, should be removed from footing excavations prior to placing steel or concrete. Footing subgrades should be evaluated during construction to verify the presence of competent bearing soil, and to determine that all soft or loosened, disturbed soils and all existing fill have been removed.

Loose backfill sand as at pressure of 40 pcf over depth of any wall.

5.3 Lateral Earth Pressures For Permanent Structures

Permanent, below-ground-level rigid walls of the proposed structure should be designed for at-rest lateral earth pressures estimated using the earth pressure diagrams provided for shoring design on Figure 7. Cantilevered retaining walls or semi-rigid basement walls which are capable of displacing at the top a distance equal to 0.001 times the wall height could be designed for an equivalent fluid density of 35 pcf. The above pressures are based on the assumptions that the walls are drained so that hydrostatic pressures cannot develop and the ground surface behind the wall is level, as presently proposed. The earth pressure contribution from surcharge loads, such as adjacent footings, floor slabs or other loads, should be added to the lateral wall pressures. Surcharge pressures can be estimated using Figure 8, Lateral Earth Pressures From Surcharge Loading.

5.4 Lateral Resistance

Lateral forces due to earth pressures will be resisted by passive earth pressure against the buried portions of the structure, and by frictional resistance against the bottom. In our opinion, passive earth pressures in very dense sands or silts, or hard silty clay, around buried foundations could be estimated using an equivalent fluid weight of 375 pounds per cubic foot (pcf). This value assumes that the foundations extend at least 3 feet below the lowest adjacent final grade. Lateral resistance for portions of the structure that have densely compacted fill behind them can be estimated using an equivalent fluid pressure of 280 pcf. The above values include a factor-of-safety of 1.5.

Base sliding resistance between mass concrete and very dense native soil can be estimated using an ultimate coefficient of friction of 0.5. An appropriate factor-of-safety should be used to calculate the resistance to sliding.

5.5 Seismic Design Considerations

5.5.1 Ground Motion

We understand that the seismic design of the proposed structure will be in accordance with the 1994 UBC which is implicitly based on ground motion levels anticipated for an event with a 500-year recurrence interval. The UBC places Seattle within seismic zone 3. This designation implies that a peak ground acceleration of 0.3g corresponds to a 500-year event. This peak ground acceleration is consistent with the results of recent seismicity studies performed by Shannon & Wilson, Inc. for other projects in the Seattle area and is an appropriate level of ground shaking for the seismic design of the proposed building.

The subsurface conditions encountered in the boring logs are consistent with a UBC S2 soil type and the corresponding site coefficient of 1.2. Recommended response spectra for 2, 5, and 10 percent damping are presented on Figure 9. The spectral shapes are based on a UBC S2 soil profile and are scaled to be consistent with a 0.30g peak ground acceleration. These empirically derived spectra are consistent with site-specific ground motion studies conducted for other projects in the area with similar subsurface conditions.

5.5.2 Earthquake Induced Geologic Hazards

Earthquake hazards include fault-related ground rupture, liquefaction, settlement, and landsliding. Based on the relatively dense nature of the foundation soils at the site, the mild topography and deep depth to the groundwater table, it is our opinion that the risk to the structure from liquefaction, settlement, and landsliding is minimal. This conclusion is also substantiated by the absence of historical reports of instability in the immediate vicinity of the site.

It is also our opinion that there is a low risk of fault-related ground rupture affecting the site. The relatively low risk is based on the assessment that the closest, presumably active fault is the Seattle Fault, located about one mile to the south. While there is evidence that this fault may have moved about 1,100 years ago, no surface rupture has been detected from this movement; rather, broad areal uplift and subsidence is thought to have occurred. In addition, it appears that the recurrence interval on this fault is on the order of thousands of years, much longer than the 500-year event implicit in the UBC.

5.6 Temporary and Permanent Drainage

The groundwater level was encountered at various depths in the borings around the site. It is anticipated that seepage from water-bearing sand or silt layers will be encountered at numerous locations within the excavation face. Seepage water is expected to emerge through cracks between lagging during construction. The Contractor should provide means to prevent the buildup of hydrostatic pressures behind shoring walls, such as weep holes or cutouts in the lagging, as necessary. Any voids that are created behind the lagging of the temporary shoring walls should be backfilled with free-draining granular soils, such as clean concrete sand or sand and gravel held in place with geotextiles designed for drainage. They should be backfilled as they are created or as soon as they are observed.

Permanent drainage between the temporary shoring wall and the permanent concrete wall of the structure should consist of a pre-fabricated, geocomposite material known as a wall drain or sheet drain. This material consists of a geotextile bonded to a plastic core. The material is nailed to the excavation side of the wood lagging as excavation proceeds down. An illustration of this system is presented as Shoring Wall Drainage System, Figure 10. Wall drains are manufactured by over a dozen different geosynthetic manufacturers under numerous different trade names. Typical products that could be used for this project include Miradrain 6000, Battledrain II, Terradrain 203, or an equivalent. We recommend that the wall drain material be used continuously from top to bottom of the shoring wall and that it be installed between all soldier piles. Water emerging at the bottom of the shoring wall should be collected in perforated plastic pipes or a prefabricated collection system compatible with the geocomposite wall drain. Drainage from the walls should be routed in tightlines to a storage tank where it can be pumped. Additional recommendations for drainage between permanent walls and shoring are presented on Figure 10.

The excavation contractor should anticipate encountering groundwater seepage and/or standing water at footing excavations. Sump pumps and ditching should be employed as necessary to allow footing excavations and placement of steel to proceed under dry conditions.

5.7 Floor Slab Design

All soft, loose, or disturbed soils should be removed from areas to receive floor slabs on grade. Floor slabs may be founded on dense to very dense or hard native soils, or on compacted structural fill placed on these competent native soils. To lessen the potential for differential settlements beneath floor slabs, all backfills for footing excavations, utilities, etc., should be densely compacted structural fill.

We recommend that a capillary break consisting of a 6-inch (minimum) thick layer of washed pea gravel, and an overlying vapor barrier consisting of plastic sheeting, be placed beneath interior concrete floor slabs. Alternatively, the upper 2 inches of pea gravel could be replaced with 2 inches of clean, crushed rock placed and compacted over 4 inches of pea gravel to provide a more firm working surface on which to place reinforcing steel and concrete. The gravel should be hydraulically connected to drain into a storm sewer system or sump. We also recommend a floor slab subdrain system, consisting of 4-inch-diameter perforated pipe bedded in the pea gravel, be installed on a grid pattern approximately 20 feet apart.

Alt 7/18/94: allow 1½" min Cr Rk. w/ <2% fines compact to 95%

5.8 Fill Placement and Compaction

All backfill which must develop passive resistance and all fill placed where settlements are to be minimized, should be structural fill. Structural fill material should consist of a reasonably well-graded (from fine to coarse) sand, sand and gravel or crushed rock, free of organics and debris, with a maximum particle size of about 3 inches. It should contain not more than about 20 percent fines (material passing the No. 200 mesh sieve), by weight, based upon wet sieving of the minus 3/4-inch fraction. Most of the native soils that will be encountered during excavations will consist of sands and gravels which would be suitable for use as structural fill provided they are not excessively wet. The moisture content must be at or near the optimum to achieve adequate compaction. Some soils may require drying to allow proper compaction. The clays and silts that will be encountered during excavations will not be suitable for reuse as structural fill.

Structural fill should be placed in uniform lifts and each lift should be compacted to a dense and unyielding surface and to at least 95 percent of the Modified Proctor maximum dry density (ASTM Designation D 1557-70). Prior to the placement of structural fill, water (if present) should be drained or pumped from the area. The thickness of soil layers before compaction should not exceed 8 inches for heavy equipment compactors and 4 inches for hand-operated mechanical compactors.

If earthwork takes place in wet weather or wet conditions where the control of soil moisture is not possible, no matter what time of the year, the fines content of structural fill mate rial should be no more than 5 percent. Any fines should be non-plastic.

5.9 Excavations

5.9.1 Temporary Shoring

It is anticipated that the majority of the excavations for the proposed building will be vertical cuts retained with soldier pile and lagging as temporary shoring. Due to the depth of the excavation, tieback anchors will be required for lateral support.

Recommended earth pressures for the design of temporary tied-back shoring walls are presented on Figure 7, Design Criteria For Shoring Walls. These pressures assume the groundwater level is below the base of the excavation and do not consider hydrostatic pressures. Groundwater seepage at the face of the excavation is assumed to be drained through the lagging so that localized hydrostatic forces do not develop. Lateral pressures from surcharge loadings should be added to the earth pressures provided in Figure 7 and may be estimated using the relationships illustrated on Figure 8.

Due to the sensitivity of the adjacent structures, we recommend an instrumentation program be implemented for the selected temporary shoring systems. The purpose of the instrumentation will be to evaluate design assumptions and monitor ground movements at and behind the excavation surfaces. The ground movements can be evaluated to determine whether steps should be taken to revise the shoring design or construction sequence to limit movements. The following paragraphs present additional discussions and recommendations for shoring design and implementation.

5.9.2 Soldier Piles

Soldier piles generally consist of steel beam sections embedded in vertical, pre-drilled, grout-filled holes. They are installed on 6- to 12-foot spacings (typically) around the perimeter of the proposed excavation. As the excavation proceeds from the top down, wooden or concrete lagging is placed to retain the soil between soldier piles, and tiebacks are installed to provide lateral support.

In addition to supporting lateral earth pressures, they should also be designed for the vertical component of tieback anchor forces. Vertical soldier pile capacities below the bottom of the excavation can be estimated using an allowable skin friction value of 1.5 ksf and an allowable end-bearing pressure of 10 ksf. For this allowable end bearing the bottom of the soldier pile hole must be cleaned out to remove loose, disturbed soil. The bottom of the excavation should be considered the elevation of the bottom of the new wall foundation. If necessary, skin friction values for the portion of the piles above the bottom of the excavation can be provided.

In addition to vertical load capacity, penetration depth below the final excavation level should also be adequate for kick-out resistance. Recommendations for determining pile embedment are included on Figure 7. We recommend that soldier piles penetrate at least 8 feet below the bottom of the excavation. The shoring Contractor should anticipate drilling boreholes for the installation of soldier piles through sand, silt and clay layers containing gravel and occasional cobbles. Use of casing may be necessary to prevent caving through loose soils or soils that contain groundwater.

5.9.3 Lagging

We recommend that lagging be installed between soldier piles. Lagging should be installed as the excavation proceeds and not more than 5 feet (measured vertically) of unsupported excavation should be exposed at any one time. The height of the unsupported excavation could be less than 5 feet and should be determined based on the soil conditions present during construction.

Due to soil arching between soldier piles, a reduced lateral earth pressure may be used for design of lagging. At this site we recommend that the design for lagging be based on 30 percent of the lateral soil pressure recommended for shoring. This reduced soil pressure may be uniformly distributed over the length of the lagging. Generally, 4-inch-thick treated timber is sufficient to provide adequate support between soldier piles; however, high surcharge loads and/or earth pressures may necessitate thicker or stiffer lagging material.

5.9.4 Tieback Anchors

Anchor holes should be drilled in a manner that will minimize loss of ground and not endanger previously installed anchors or undermine existing pavement, foundations, or utilities. The Contractor should be prepared to drill through and install anchors in very dense granular soil with gravel and hard silty clay; wet soil layers with possible seepage; occasional cobbles and boulders may also be encountered.

In the anchor no-load zone, tieback holes could be filled with a material such as a sand-pozzolan mixture that will prevent caving and will not adhere to the tieback rod. We recommend that no-load zone lengths not be left open over night. Alternatively, a bond breaker could be used around the ties in the no-load zone and the zone could be filled with concrete or lean concrete backfill. However, a minimum 12-inch buffer zone of sand is required directly behind the soldier pile.

The length of production anchors should be based on a series of test anchors; however, the following frictional values are provided for construction planning and estimating purposes. Temporary tieback anchors installed in dense to very dense sand and gravel by hollow-stem auger methods (i.e., tremie grouting) could be designed for an allowable frictional value of 1,500 pounds per square foot (psf). Tiebacks installed in hard silty clay or clayey silt should be designed for an allowable frictional value of 1,000 psf. We recommend anchor grout be placed by tremie method for open-hole anchor installations.

All temporary anchors should be performance tested by loading in 25 percent (0.25P) increments to 133 percent of their design capacity (1.33P), where P is the design capacity. Approximately 5 percent of production anchors, randomly selected, should be performance tested by loading in 25 percent (0.25 P) increments to 200 percent of design capacity (2.0 P). Performance tests should be evaluated by a geotechnical engineer or her representative.

We recommend that all temporary anchors be locked off at 90 percent of the design load. Anchors which do not meet the acceptance criteria should be locked off at one-half the failure load and replaced with additional anchors as required. Soldier pile wall deflections should be monitored during excavation and the installation of tiebacks.

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, 1985. 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 reaching 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:

1) The total movement obtained from a performance and proof test exceeds 80 percent of the theoretical elastic elongation of the design free stressing length.

2) 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.

3) 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.

4) The initial lift-off readings indicate that an anchor load has been locked-off within 5 percent of specified load.

5) The lift-off tests, if required, show an anchor load within 5 percent of the specified transfer load.

We do not recommend using tiebacks within the existing fill around the site to provide lateral restraint to shoring walls or the existing areaway walls due to the presence of utilities and variable loose fill material. Additionally, use of conventional tiebacks adjacent to the Metro bus tunnel in Pine Street may not be possible. It may be possible to install horizontal tiebacks or deadmen anchored to existing structures within or along the south side of Pine Street to support the upper portion of the shoring system. We understand that these options are being evaluated by the structural engineer. Additional recommendations for excavations and shoring adjacent to the areaways and the Metro bus tunnel are presented in the following sections.

5.9.5 Areaway Wall Support

An areaway is present below the sidewalk surrounding the majority of the proposed building, except adjacent to the existing bank building at the southeast corner of the site. We understand that the areaway is to be abandoned and will be filled. In order to proceed with demolition of the existing buildings and installation of soldier piles for the temporary shoring system, it will be necessary to provide lateral support for the areaway walls. Additional studies by others will be necessary to determine the strength characteristics of the retaining walls and what form of lateral support is required within the areaways to prevent excessive wall movement once demolition begins. Internal lateral support could consist of steel braces bearing on the existing concrete floor slab or a concrete buttress cast against the inside of the retaining walls.

If the latter system is to be considered, we suggest using lean-mix concrete or control density fill (CDF) as the buttress material. The forms to contain the lean-mix could be conventional wood forms or a series of stacked concrete "ecology blocks." Ecology blocks are cast from waste concrete at the batch plant and are shaped to form interlocking blocks that are commonly used as gravity retaining walls. Ecology block walls used to retain the lean-mix can be designed as part of the permanent buttress or as temporary forms for the lean-mix concrete. In the event a concrete buttress system is utilized it will likely result in high toe pressures. We understand the pressure could be as high as 20 ksf along the inside edge of the buttress. This pressure could result in 1 to 2 inches of settlement of the buttress edge if the soils below consist of loose fill or loose native soil. The type of soil existing below the areaway slab has not been verified through subsurface explorations; however, borings located in the streets around the perimeter of the building indicate that soils may consist of very loose to dense silt and sand or very stiff clay. The amount of settlement resulting from pressures induced by the buttresses will depend on the condition of the underlying soil. We recommend that the condition of the soil at each buttress location be determined by coring through the concrete slab and extending a soil probe 5 to 10 feet into the soil.

In the event loose, compressible soils are encountered below buttress locations, it may be necessary to improve the condition of the soil, support the buttress on an underpinning structure, or provide a means of structurally distributing the buttress loads over a wider area, thereby reducing the bearing pressure along the toe. Improving the condition of the soil may be accomplished through compaction grouting by a specialty contractor. We recommend that a compaction grouting contractor be consulted to evaluate the feasibility of this alternative. Underpinning the toe of the buttress may be accomplished by installing vertical helical piers. Helical piers can be designed and installed by specialty contractors to support compression loads of up to 100 kips.

A third alternative for areaway wall support could consist of installing helical anchors through the wall to serve as tiebacks. Depending on the condition of the fill material behind the walls, helical anchors can probably be installed at this site to develop tension capacities ranging from 15 to 30 kips. We recommend installing at least one test anchor to verify that adequate pullout resistance can be developed. The test anchor should be installed at a location where the soils are known to represent average conditions and existing utilities can be avoided. Based on our discussions with local contractors, we understand that a test anchor could be installed for around $1,500.

5.9.6 Pine Street Excavations

We understand that the south foundation wall for the garage levels along Pine street will be located 25 feet north of the property line. Where areaways exist, filling them with CDF or lean-mix concrete will be utilized to retain the walls. Where areaways are not present, such as at the east end of the structure along Pine Street, a cantilevered pile shoring system has been proposed. The lateral earth pressure diagrams presented on Figures 7 and 8 should be used to design the cantilevered soldier pile walls. The cantilevered piles will be used in conjunction with a soil berm to increase the lateral resistance of the piles. The soil berms would extend a short distance northward from the inside face of the shoring and then slope down to the overall excavation grade. In order for the berms to develop full passive resistance we recommend that the width of the berm (north-south) be no less than 1.6 times the height of the berm, where the height of the berm is measured vertically from the top of the berm to the toe of the berm slope.

Based on the borings completed in Pine Street as part of the Downtown Seattle Transit Project, the soils that will be exposed during excavations at the south side of the project site would consist of medium dense sand, very stiff silty clay, or very dense sand and gravel. For planning purposes, we recommend temporary berm slopes in these soils be no steeper than 1.5 horizontal to 1 vertical. Locally flatter slopes may be necessary if loose soils or groundwater seepage zones are encountered. Slope protection consisting of a plastic covering or similar protective measures should be employed in order to reduce the erosional effects of rain and surface water on the exposed slopes. In addition, the Contractor should be made responsible for the control of any ground or surface water, wherever encountered.

5.10 Instrumentation and Settlement Surveys

Instrumentation and settlement surveys are recommended to monitor and document the wall's performance, evaluate design assumptions, determine if temporary retaining systems are functioning satisfactorily, provide an early warning of problems, and to allow assessment of the need for mitigating measures.

Surveys from stable benchmarks should be used to measure horizontal and vertical movements of the excavation support system and displacement of adjacent structures. The required measurements can be obtained through the establishment of settlement and horizontal offset survey markers and by installing slope inclinometer tubes on selected soldier piles. We recommend slope inclinometer tubes be installed on at least one soldier pile along each side of the excavation (four total). Optical measurements should be to an accuracy of 0.01-foot. We recommend that survey readings be obtained on a daily basis during excavation and construction of the shoring system.

Pre-construction surveys should be conducted to document the condition of the existing structures surrounding the project site immediately prior to start of construction or demolition. Any visible signs of distress, including cracks in walls, floor slabs, or sidewalks should be noted, photographed, and documented in a report.

5.11 Construction Monitoring and Plans Review

We recommend that we be retained to review and assist in developing the portions of the plans and specifications which pertain to earthwork, shoring and foundations to determine if they are consistent with our recommendations. We also recommend that we be retained to monitor excavations, shoring installations, drainage installations, instrumentation, structural fill placement and compaction, and footing subgrade preparation. This monitoring would allow us to verify the subsurface conditions as they are exposed during construction and to determine that the work is accomplished in accordance with our recommendations.

5.12 Additional Recommendations

Recommendations contained in this report are based on site conditions as they presently exist and further assume that the exploratory holes are representative of the subsurface conditions throughout the site. If, during construction, subsurface conditions different from those encountered in the exploratory holes are observed or appear to be present beneath excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, the recommendations presented herein should be reviewed.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or accomplishing explorations. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

6.0 LIMITATIONS

This report was prepared for the use of Pine Street Associates, NBBJ, and SWMB in the planning and design of the project. With respect to construction, it should be made available for information on factual data only and not as a warranty of subsurface conditions, such as those interpreted from the boring logs and discussions of subsurface conditions included in this report. Shannon & Wilson has prepared the attachment, "Important Information About

Your Geotechnical Report" to assist you and others in understanding the use and limitations of our reports. The scope of our services did not include any environmental assessment or evaluation regarding the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air at the subject site.

SHANNON & WILSON, INC.

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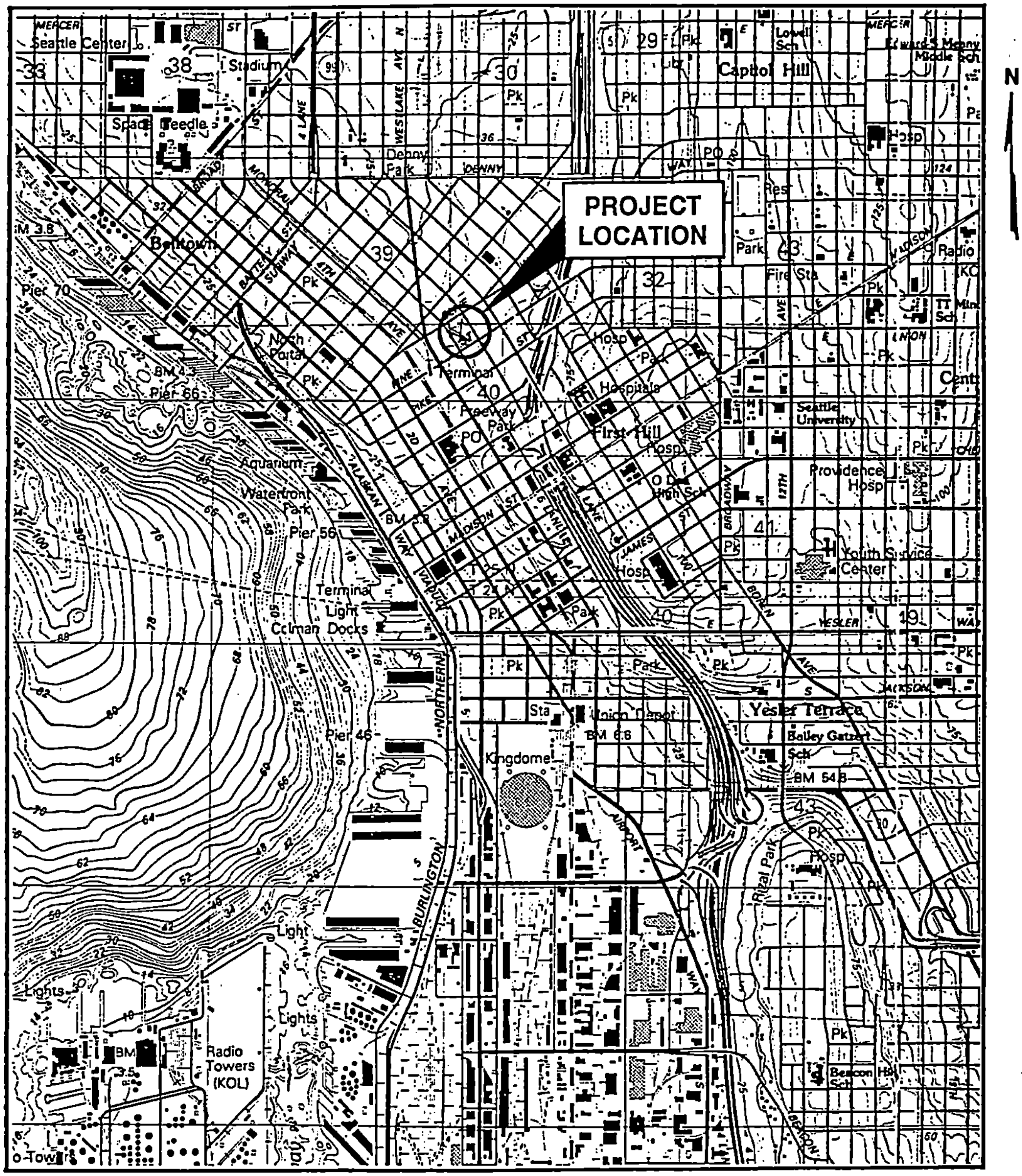
11-3-95/W6913-01.RPT/W6913-lkd/dgw



0 1/4 1/2 1

Scale in Miles

NOTE

Map adapted from USGS metric topographic map of Seattle South, WA quadrangle, dated 1983.

Systems Retail Building
Seattle, Washington

VICINITY MAP

March 1995 W-6913-01

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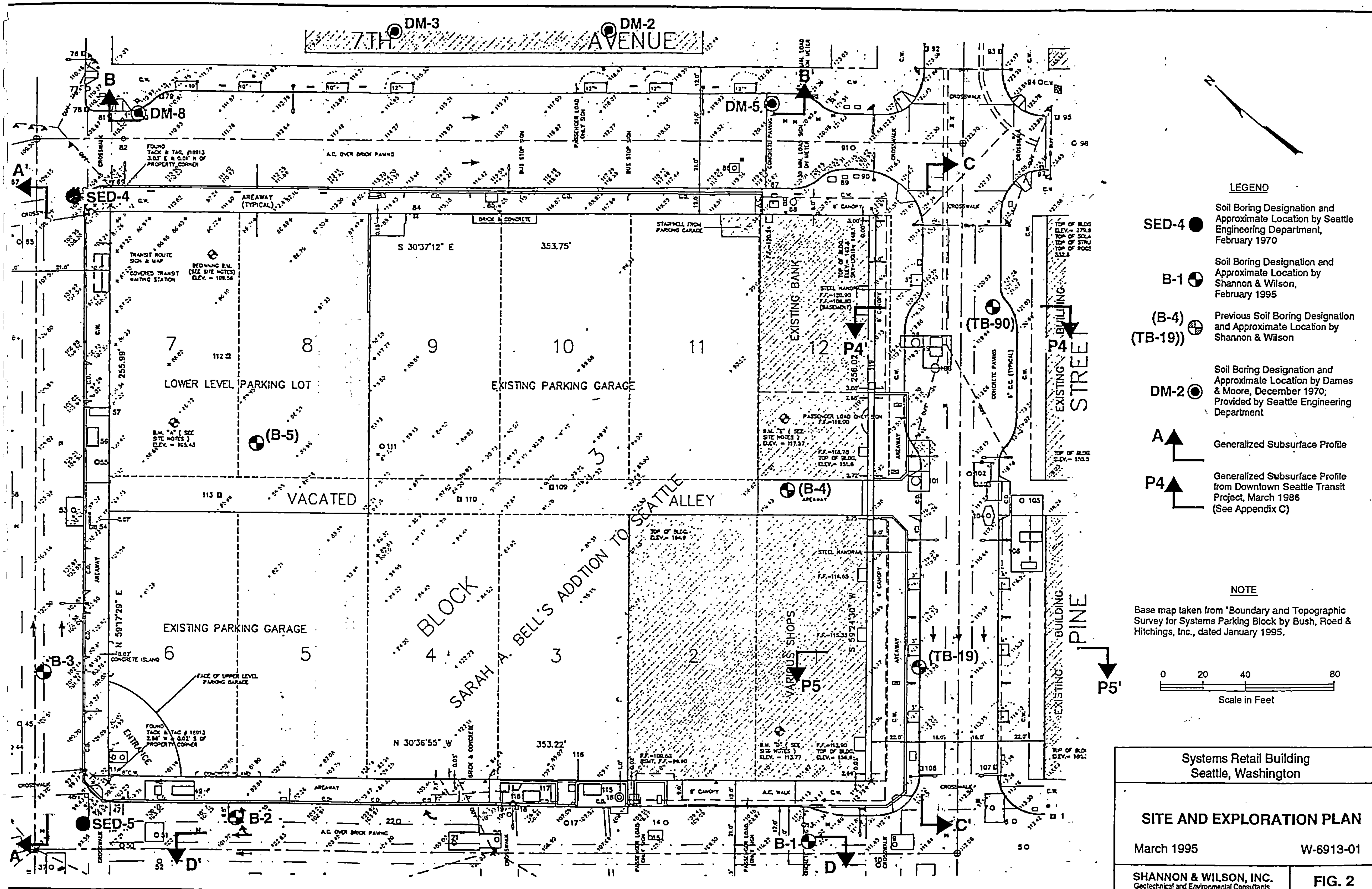
FIG. 1



Systems Retail Building
Seattle, Washington

SITE AND EXPLORATION PLAN

March 1995 W-6913-01

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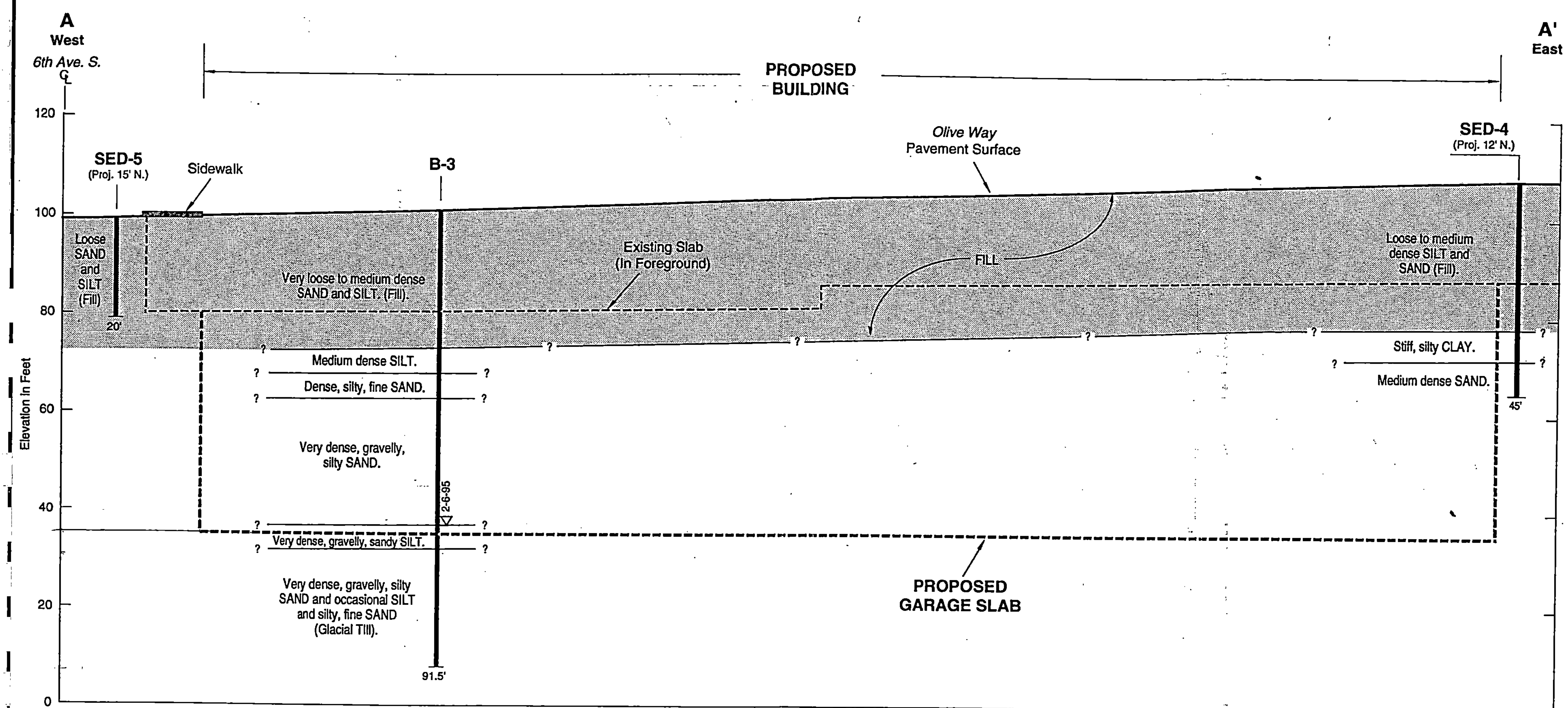
FIG. 2



LEGEND

B-2 ← Soil Boring Designation and Approximate Location
(Proj. 4' N.) ← Offset Distance

? — ? Approximate Geologic Contact

2-6-95 ← Water Level and Date Recorded

← Depth to Bottom of Boring

0 20 40

Scale in Feet
Horizontal = Vertical

NOTES

1. Approximate ground surface elevations based on topographic Survey by Bush, Roed & Hitchings, Inc., dated January 1995.
2. This profile is generalized based on subsurface conditions encountered in field explorations. Variations between this profile and actual conditions may exist.

Systems Retail Building
Seattle, Washington

GENERALIZED SUBSURFACE PROFILE A-A' OLIVE WAY

March 1995 W-6913-01

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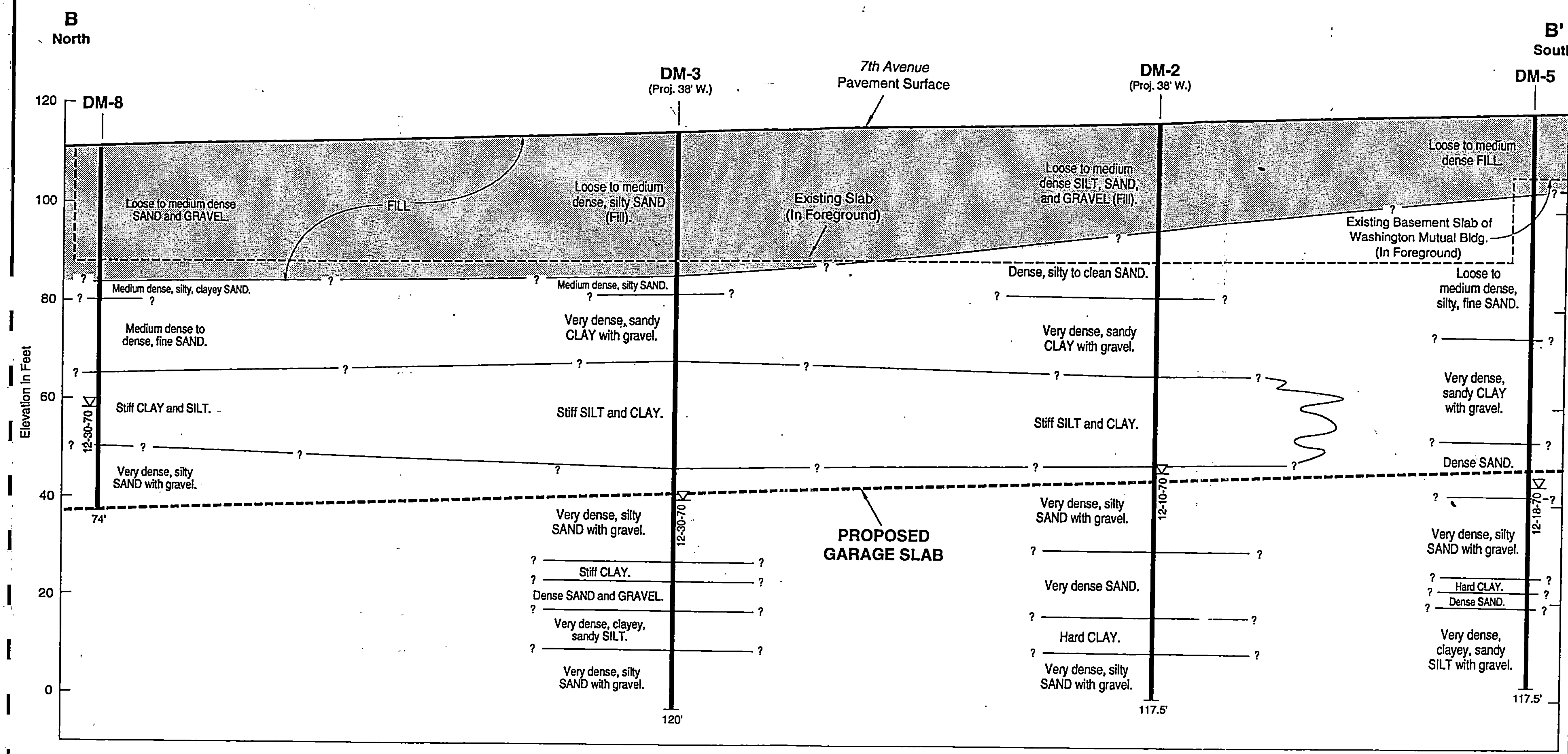
FIG. 3



0 20 40

Scale in Feet
Horizontal = Vertical

LEGEND

DM-2 ← Soil Boring Designation and Approximate Location
(Proj. 38' W.) ← Offset Distance

Water Level and Date Recorded →

12-10-70

? — ? Approximate Geologic Contact

← Depth to Bottom of Boring

NOTES

1. Approximate ground surface elevations based on topographic Survey by Bush, Roed & Hitchings, Inc., dated January 1995.

2. This profile is generalized based on subsurface conditions encountered in field explorations. Variations between this profile and actual conditions may exist.

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Seattle, Washington

GENERALIZED SUBSURFACE PROFILE B-B' 7TH AVENUE

March 1995 W-6913-01

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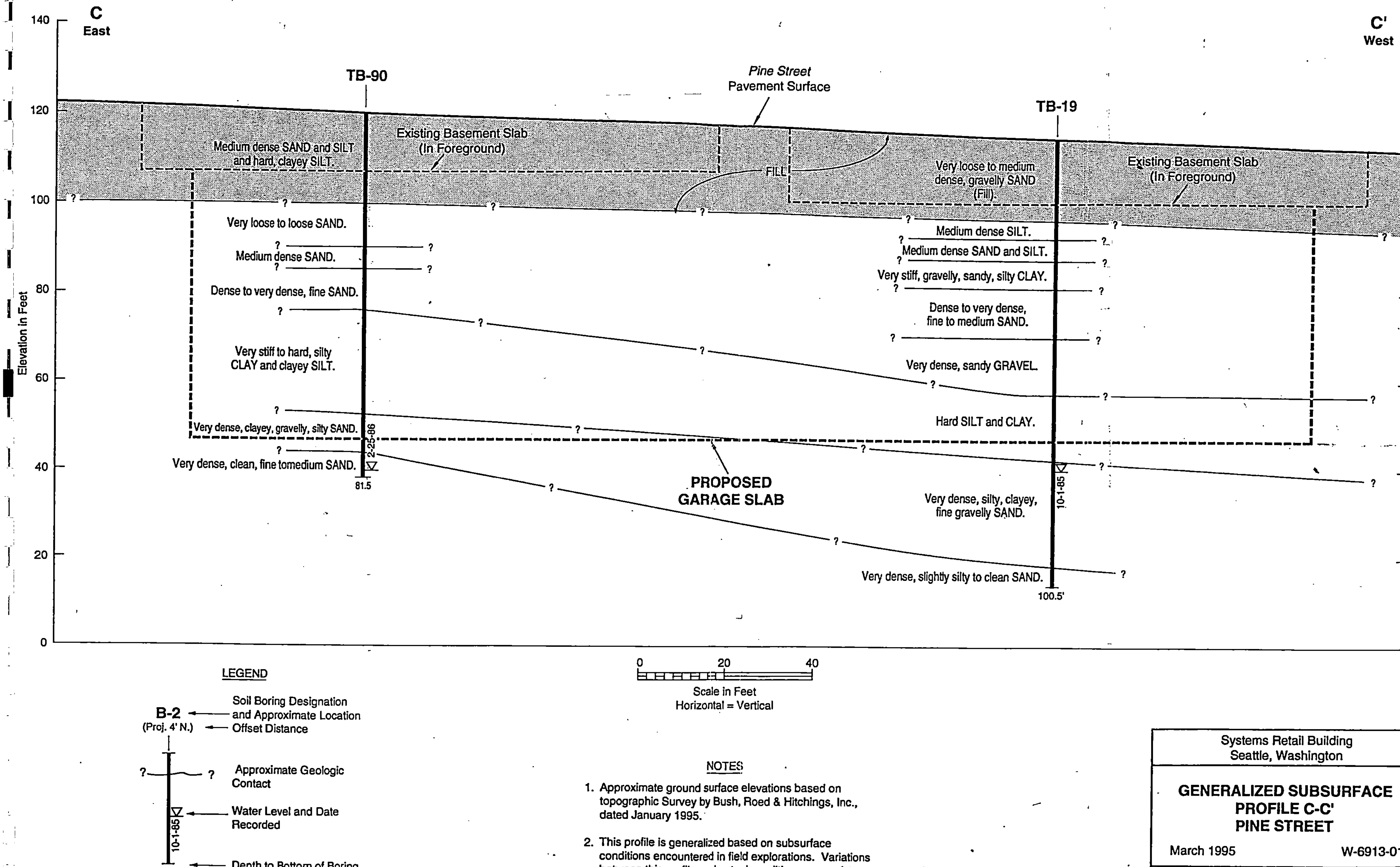
FIG. 4



NOTES

1. Approximate ground surface elevations based on topographic Survey by Bush, Roed & Hitchings, Inc., dated January 1995.
2. This profile is generalized based on subsurface conditions encountered in field explorations. Variations between this profile and actual conditions may exist.

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Seattle, Washington

GENERALIZED SUBSURFACE PROFILE C-C' PINE STREET

March 1995 | W-6913-01

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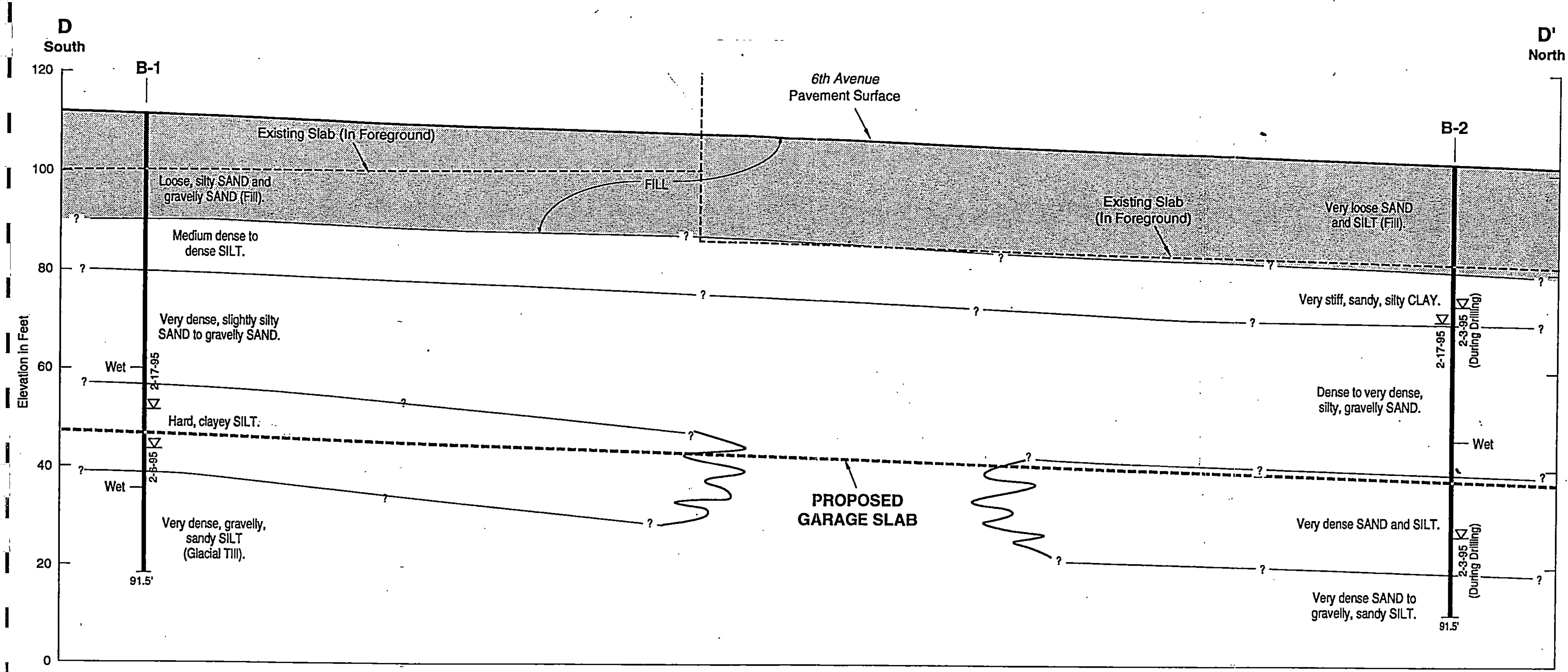
FIG. 5



0 20 40

Scale in Feet
Horizontal = Vertical

LEGEND

B-1 — Soil Boring Designation and Approximate Location

? — ? Approximate Geologic Contact

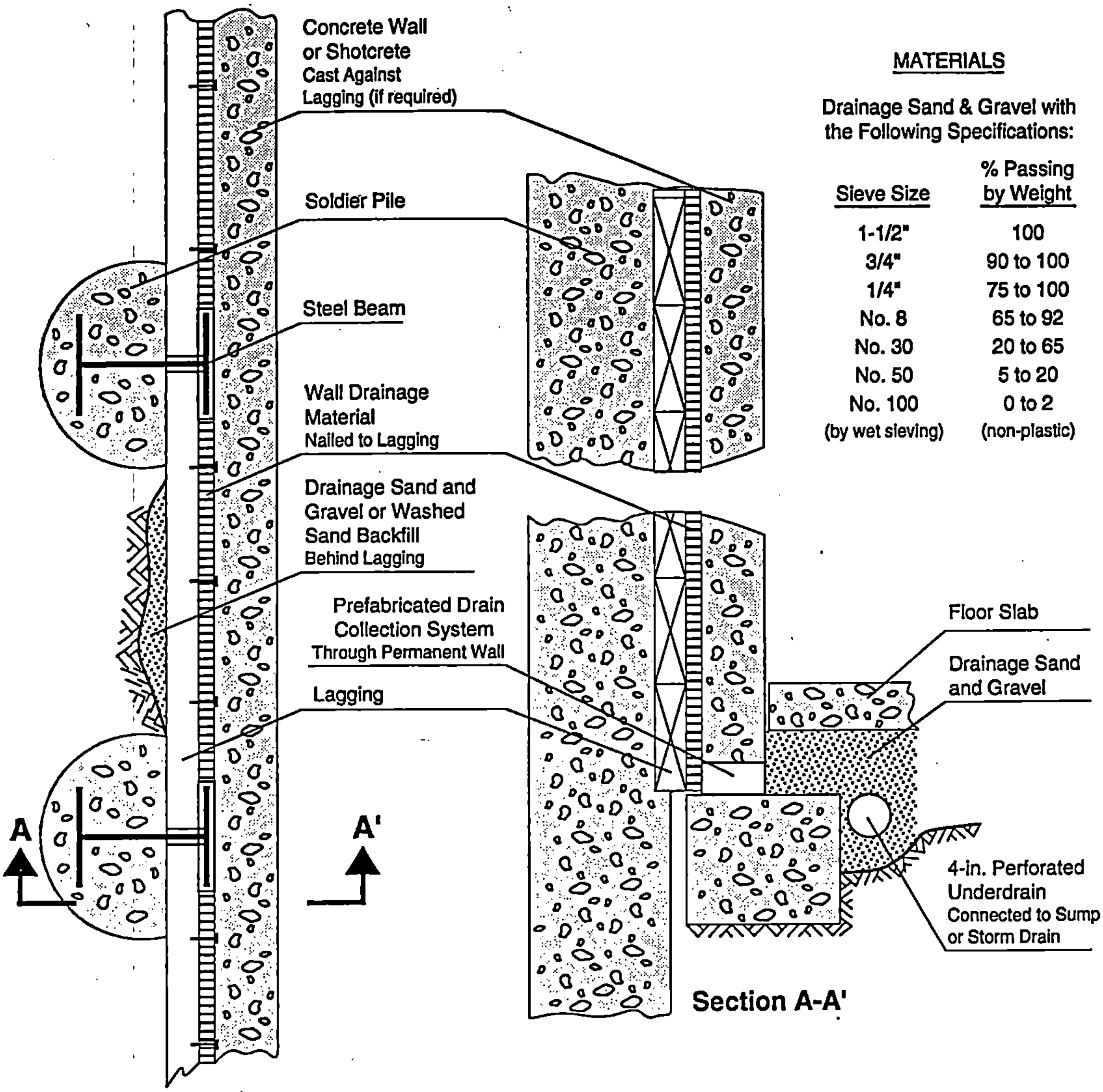
Water Level and Date Recorded (2-6-95)

Depth to Bottom of Boring

NOTES

1. Approximate ground surface elevations based on topographic Survey by Bush, Roed & Hitchings, Inc., dated January 1995.
2. This profile is generalized based on subsurface conditions encountered in field explorations. Variations between this profile and actual conditions may exist.

Systems Retail Building Seattle, Washington	
GENERALIZED SUBSURFACE PROFILE D-D' 6TH AVENUE	
March 1995	W-6913-01
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 6



MATERIALS

Drainage Sand & Gravel with the Following Specifications:

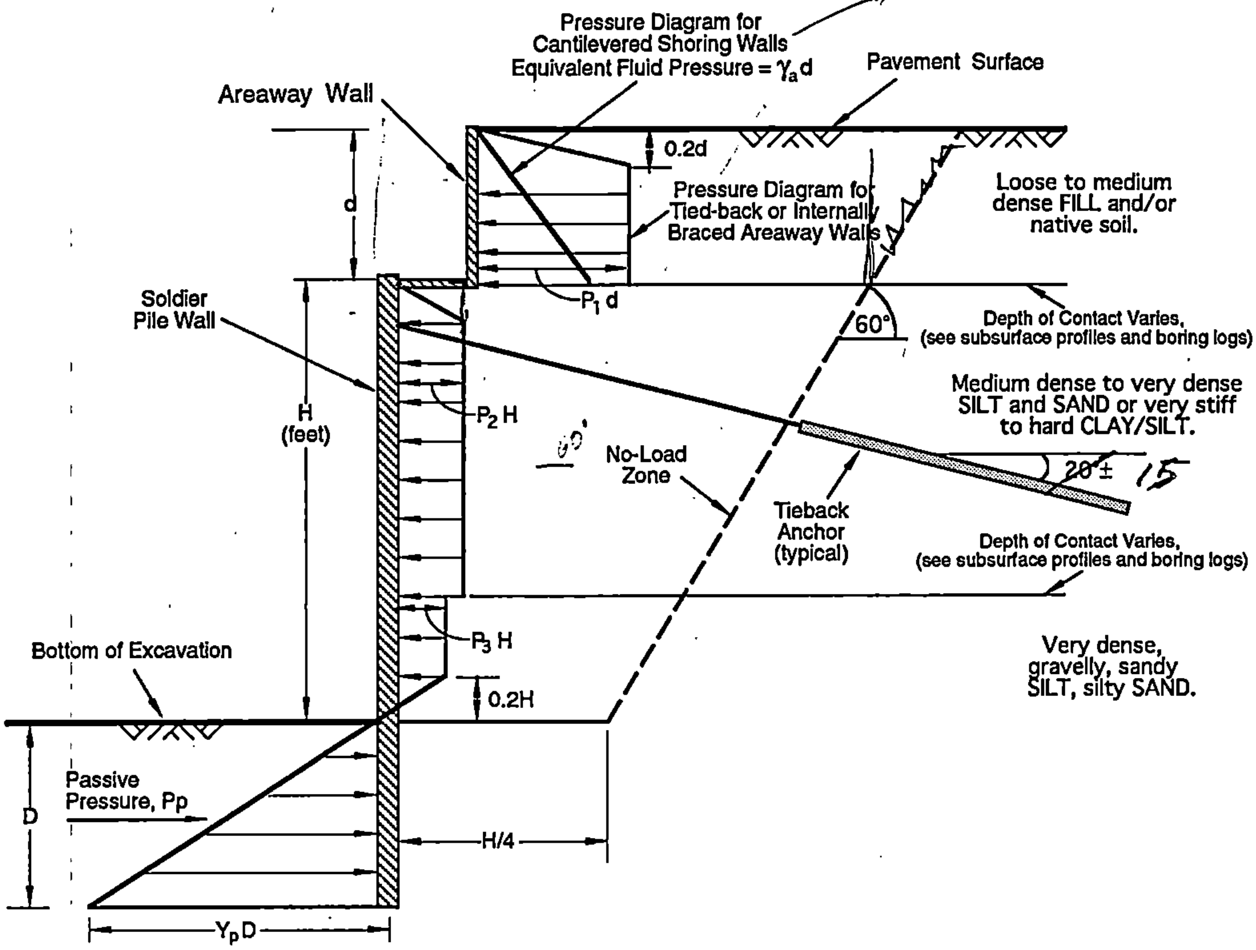
Sieve Size	% Passing by Weight
1-1/2"	100
3/4"	90 to 100
1/4"	75 to 100
No. 8	65 to 92
No. 30	20 to 65
No. 50	5 to 20
No. 100	0 to 2
(by wet sieving)	(non-plastic)

Not to Scale

NOTES

1. Wall drainage material: See report text.
2. Clean-outs should be provided in the underdrain system.
3. Weep holes should be provided through lagging.

Systems Retail Building Seattle, Washington	
SHORING WALL DRAINAGE RECOMMENDATIONS	
March 1995	W-6913-01
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 10



Not to Scale

NOTES

1. Use triangular pressure distribution, as shown, for cantilever walls and use rectangular distribution for tied-back or internally braced walls.
2. Pile embedment should be determined by moment equilibrium at lowest tieback level and must be sufficient to resist the vertical component of tieback loads.
3. Use 80% of the above pressure for computing moments in soldier piles.
4. Vertical working pile capacity may be estimated by assuming 1 ksf side friction below the bottom of the excavation (including footing excavation), and 10 ksf end bearing.
5. Passive pressure, Pp, should be computed as acting over twice the pile diameter or pile spacing whichever is less.
6. Groundwater level is assumed to be below the bottom of the excavation and free-drainage is assumed behind the wall. Fill all voids behind lagging with clean sand or sand and gravel.
7. Driving earth pressures should be computed as acting over the pile spacing above and pile width below bottom of excavation.
8. Lateral pressures from surcharge loads, such as areaways on the soldier pile wall, should be added to the appropriate earth pressures - see Figure 8.
9. Locate tieback anchors behind no-load zone.

LEGEND

H = Height of Excavation in Feet

D = Embedment Depth in Feet

P_1 = 30 pcf for Fill and Loose Native Soil

P_2 = 25 pcf for Medium Dense to Very Dense Sand and Silt, and Stiff to Hard Clay/Silt

P_3 = 20 pcf for Very Dense, Gravelly, Sandy Silt and Silty Sand

γ_p = 375 pcf = Equivalent Fluid Weight for Passive Pressure (includes FS of 2)

γ_a = 35 pcf = Active Pressure for Fill Material

Systems Retail Building
Seattle, Washington

DESIGN CRITERIA FOR SHORING WALLS

November 1995 W-6913-01

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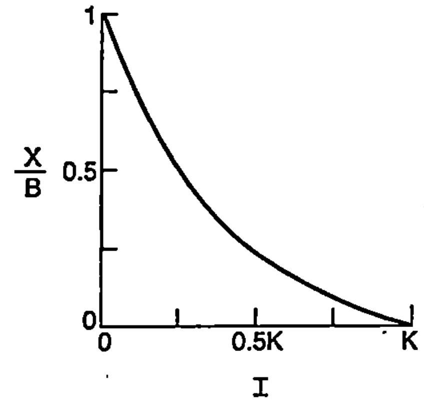
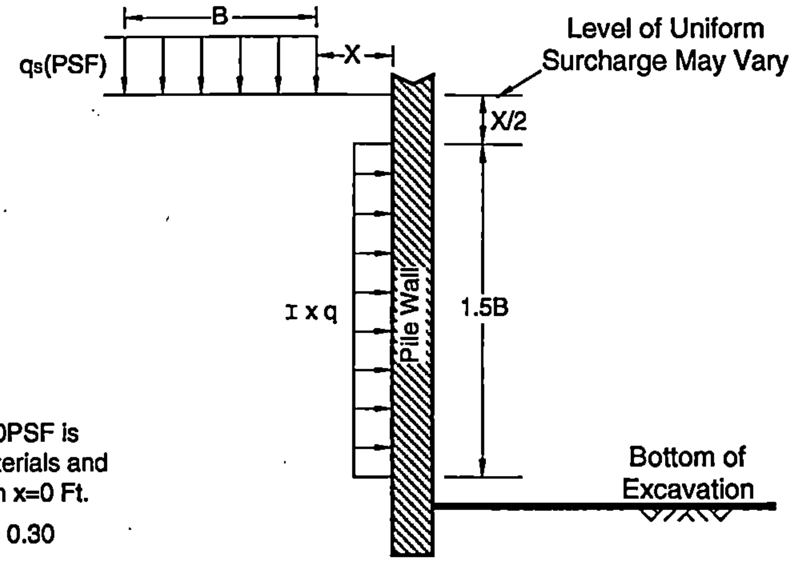
FIG. 7

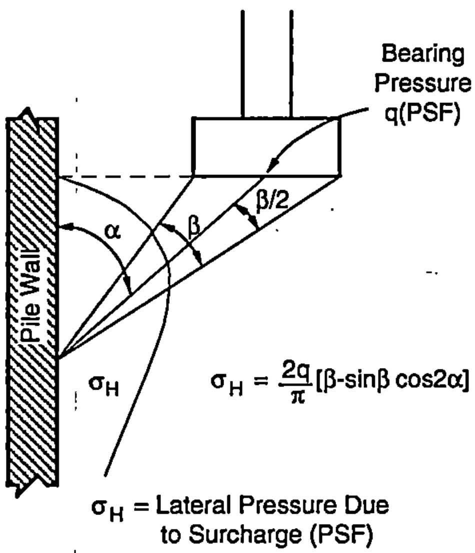


A minimum lateral pressure (I x q) of 120PSF is recommended for traffic, construction materials and equipment for a depth of 1.5B=20 Ft., with x=0 Ft.

For glacially-overidden soils use K = Ka = 0.30



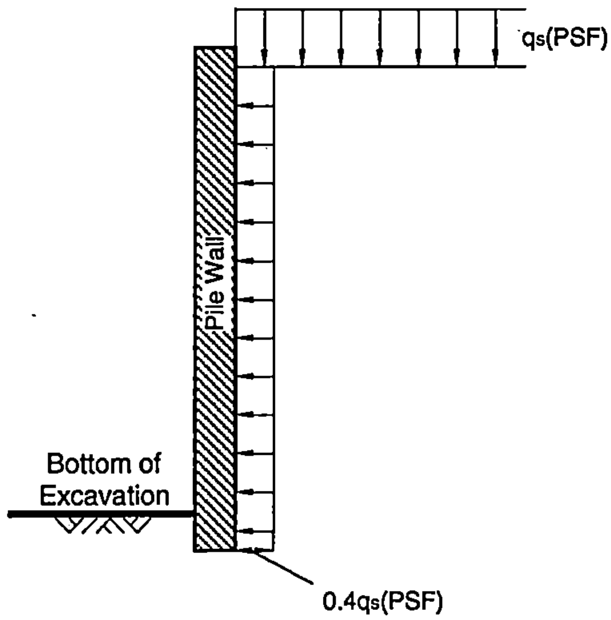
A) Recommended Lateral Surcharge Due to Buildings, Traffic, Construction Materials, Equipment, etc., with Net Bearing Pressures Less Than 3 TSF



$$\sigma_H = \frac{2q}{\pi}[\beta - \sin\beta \cos 2\alpha]$$

σ_H = Lateral Pressure Due to Surcharge (PSF)

β, α = Angles as Indicated



B) Surcharge Due to Building Wall Footings and Retaining Wall Footings with Net Bearing Pressures Greater Than 3 TSF

C) Surcharge Due to Uniform Loads, i.e., Mat Foundations and Spread Footings Wider Than 20 Feet

Not to Scale

<u>NOTE</u>

For other surcharge loading conditions, i.e. point loads, refer to <u>USS Steel Sheet Piling Design Manual</u>, ADUSS 25-3848-05, July 1975.

Systems Retail Building
Seattle, Washington

LATERAL EARTH PRESSURES FROM SURCHARGE LOADING

November 1995 W-6913-01

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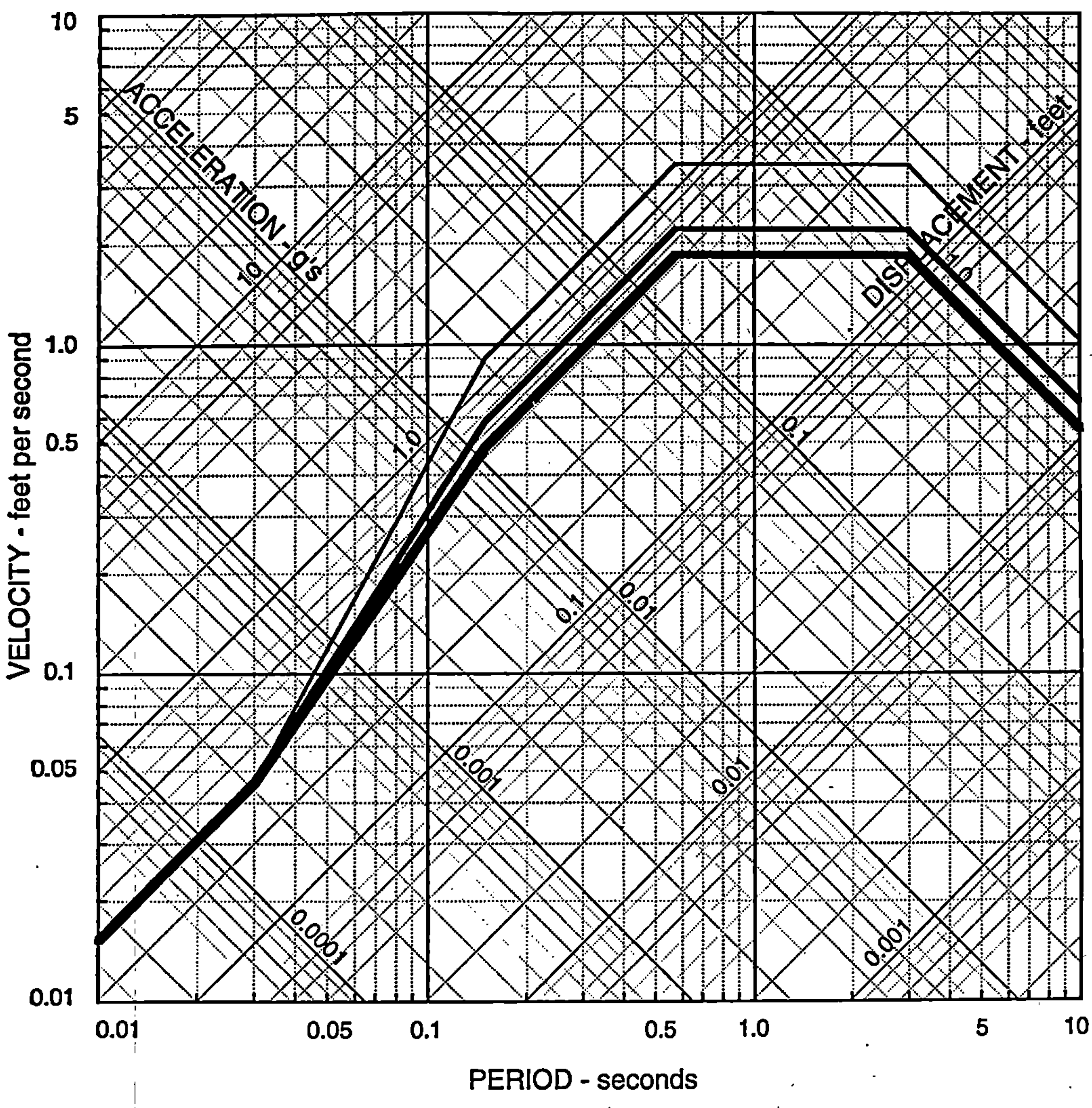
FIG. 8



LEGEND

- 2% Damping
- 5% Damping
- 10% Damping

NOTES

1. Recommended Response Spectra correspond to UBC Soil Type S_2 and a 0.30g PGA.
2. Spectra represent free-field, horizontal motions at the ground surface.
3. Vertical spectra correspond to two-thirds of the above horizontal values.

Systems Retail Building
Seattle, Washington

RECOMMENDED RESPONSE SPECTRA

March 1995 W-6913-01

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FIG. 9

APPENDIX A

EXPLORATION LOGS

APPENDIX A

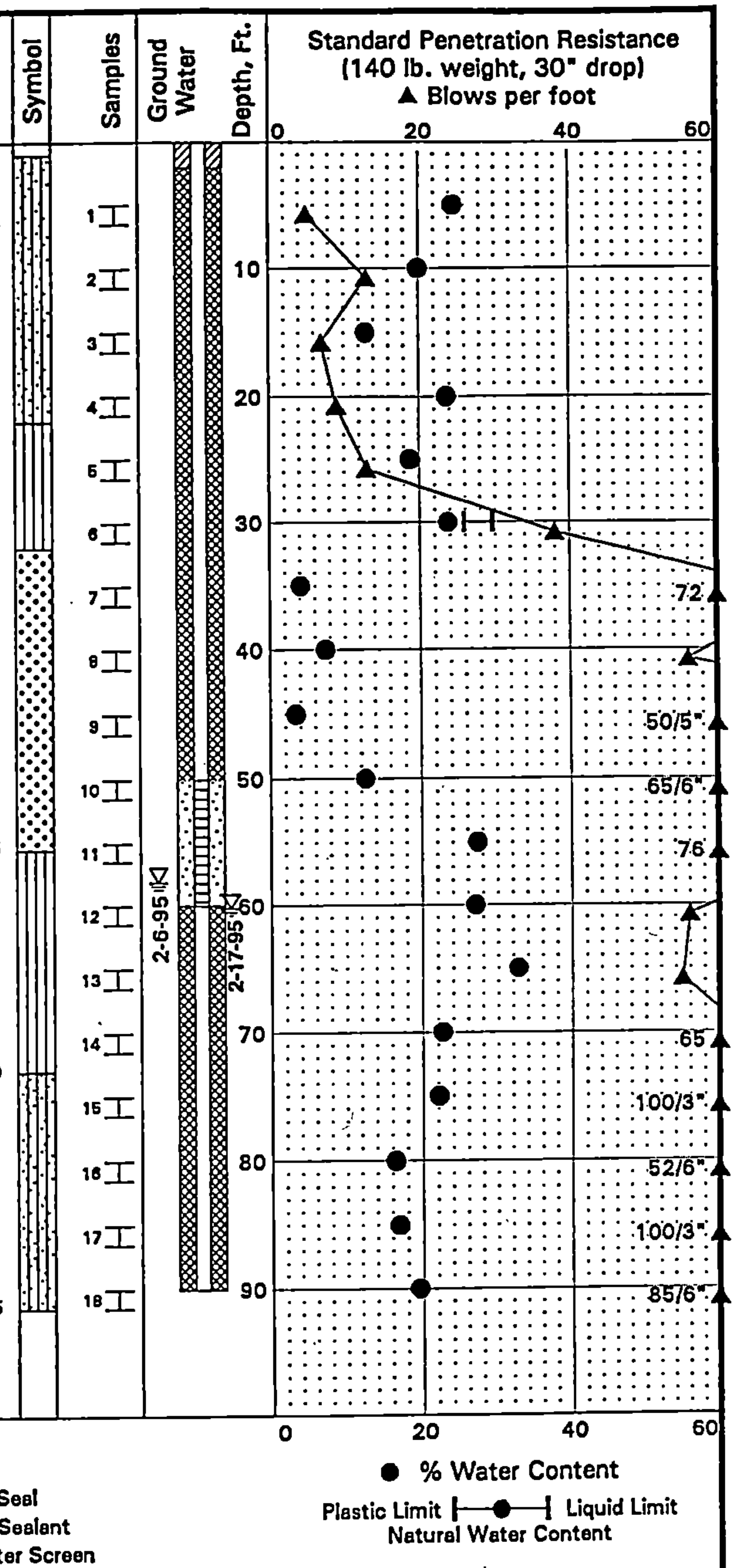
EXPLORATION LOGS

TABLE OF CONTENTS

LIST OF FIGURES

MASTERLG 3/3/95

MATERIAL DESCRIPTION Surface Elevation: Approx. 111 Feet	Depth, Ft.
Concrete.	1.0
Loose to medium dense, gray, silty, fine SAND; gravelly SAND; scattered layers of fine, sandy, clayey SILT; moist to dry; (Fill) SM-ML.	22.0
Medium dense to dense, gray to brown, slightly clayey SILT; moist; trace of sand; ML.	32.0
Very dense, brown, clean to slightly silty, fine to medium SAND becoming gravelly SAND at 44 feet; moist to wet at 51 feet; SW-SP.	55.5
Hard, gray, clayey SILT becoming fine sandy SILT at 70 feet; dry to moist; ML.	73.0
Very dense, gray, slightly gravelly to gravelly, fine to medium sandy SILT; moist with wet layer at 75 feet; (Glacial Till) SM.	91.5
BOTTOM OF BORING COMPLETED 2/2/95	



LEGEND

- • Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

Systems Retail Building
Seattle, Washington

LOG OF BORING B-1

February 1995 W-6913-01

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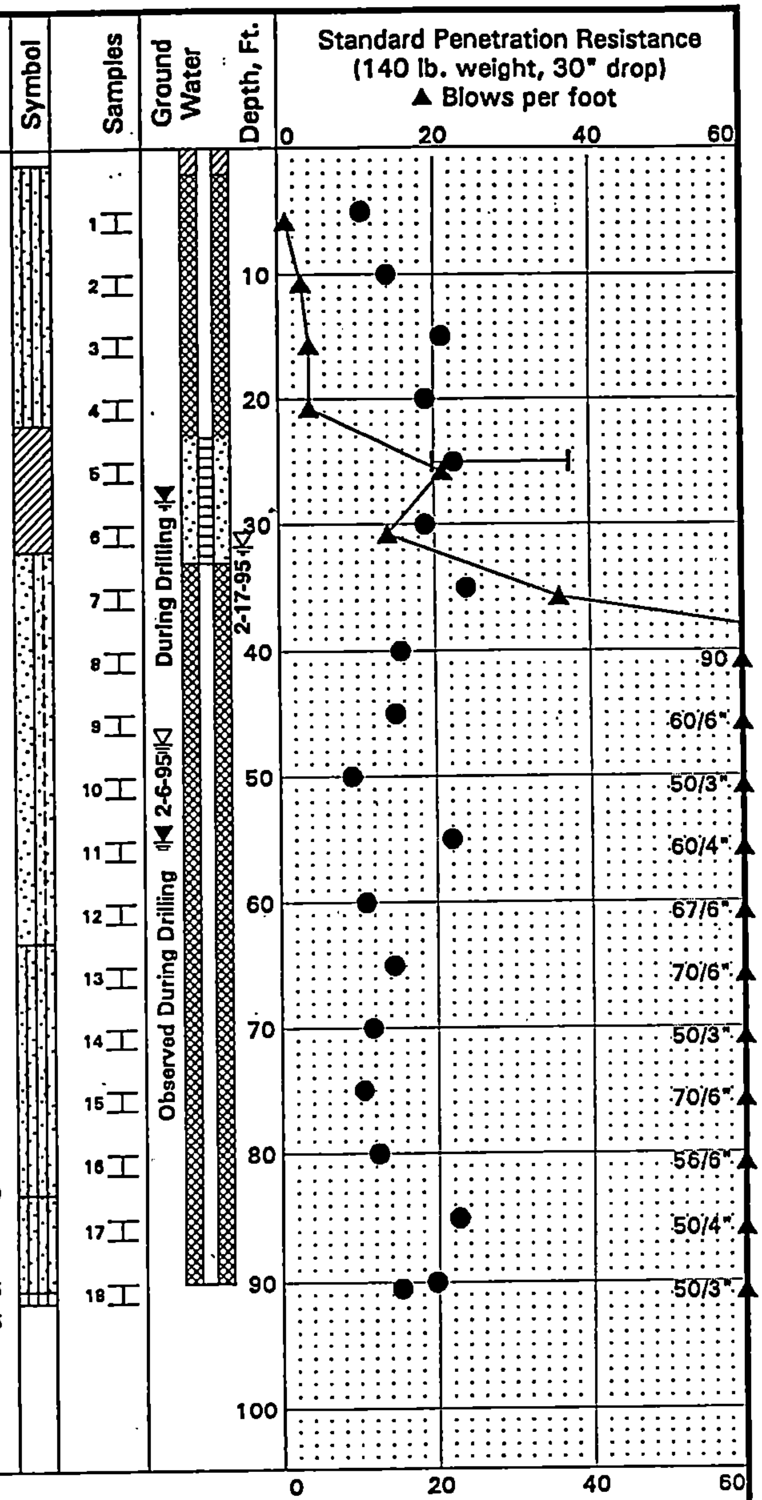
FIG. A-1

MASTERLG 3/3/95

MATERIAL DESCRIPTION Surface Elevation: Approx. 102 Feet	Depth, Ft.
Asaphlt, brick, and concrete layers.	1.2
Very loose, brown to gray, slightly gravelly, silty, fine to medium SAND; occasional clayey SILT layers; moist to wet at 21 feet; (Fill) SM.	22.0
Very stiff, gray to brown with iron-oxide staining, sandy, silty CLAY; moist; CL.	32.0
Dense to very dense, silty to slightly silty, slightly gravelly, fine to medium SAND becoming fine to coarse SAND at 50 feet; moist to wet at 55 feet; SP-SM.	63.0
Very dense, gray, gravelly, silty SAND and sandy SILT; moist; SM.	83.0
Very dense, brown, silty, fine to medium SAND; moist; SM.	90.5
Very dense, gray, gravelly, sandy SILT; moist; ML.	91.5

BOTTOM OF BORING
COMPLETED 2/3/95



LEGEND

- • Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Water Level in Piezometer
- Water Level Observed During Drilling
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout

● % Water Content

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

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

Systems Retail Building
Seattle, Washington

LOG OF BORING B-2

February 1995 W-6913-01

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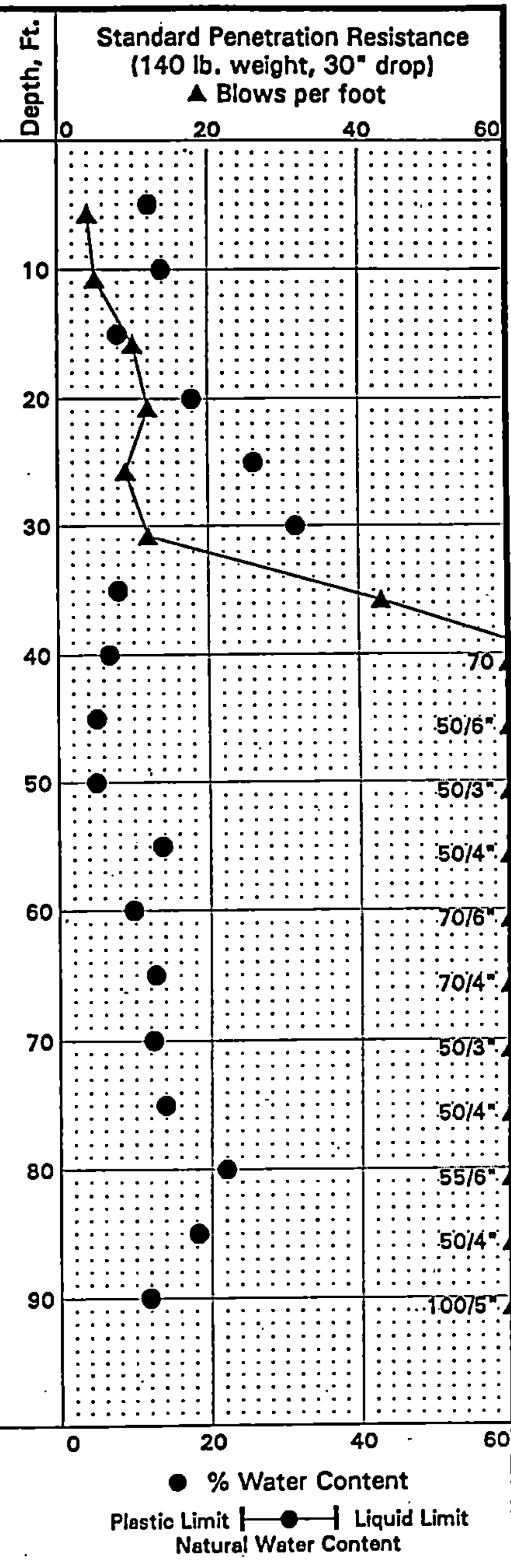
FIG. A-2

MASTERLG 3/3/95

MATERIAL DESCRIPTION Surface Elevation: Approx. 101 Feet	Depth, Ft.	Samples	Ground Water
Asphalt, brick, and concrete layers.	2.5		
Very loose to medium dense, brown, silty, fine to medium SAND, slightly silty, gravelly SAND, and gravelly, sandy SILT; moist; (Fill) SM.	27.0	1, 2, 3, 4, 5	
Medium dense, light brown SILT; moist; ML.	32.0	6	
Dense, brown to gray, silty, fine SAND; moist; SM.	37.0	7	
Very dense, gray to brown, slightly gravelly to gravelly, slightly silty SAND; moist; SM-SP.	63.0	8, 9, 10, 11, 12	
Very dense, gray, slightly gravelly, sandy SILT; moist; ML.	68.0	13	Observed During Drilling ⊻
Very dense, gray, gravelly, silty SAND; moist; occasional SILT and silty, fine SAND layers; (Glacial Till) SM.	91.5	14, 15, 16, 17, 18	
BOTTOM OF BORING COMPLETED 2/6/95			



LEGEND

- • Sample Not Recovered
- 2" O.D. Split Spoon Sample
- 3" O.D. Shelby Tube Sample
- Surface Seal
- Annular Sealant
- Piezometer Screen
- Grout
- ⊻ Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of 'Symbols' and definitions.
5. USC letter symbol based on visual classification.

Systems Retail Building
Seattle, Washington

LOG OF BORING B-3

February 1995 W-6913-01

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FIG. A-3

ENVLOG2 6/30/94

ENVIRONMENTAL BOREHOLE LOG

Date Started 5/26/94	Location T25N R4E Sec. 40	Depth Water First Encountered (Ft)
Date Completed 5/26/94	Drilling Company Pacific Testing Lab.	Drilling Method Hollow-Stem Auger
Total Depth (Ft) 36.5	Sampling Method 2" O.D. Split-Spoon	Hammer: Weight (lbs) 140 Drop (In) 30
Borehole Diam. (In) 7	Ground Elev. NA / Monument Elev. NA	PVC Elev. NA

Sample Number	Interval	Blow Counts/6 In	Recovery(%)	PID (ppm)	Time	Depth (Ft)	Lithologic Description	USCS* Symbol
							Ground Surface	
						0.3	Asphalt and corrugated 1/4-inch steel decking supported by I-beams.	
							Air space below alley decking to a depth of 17.5 feet.	
						17.5	Brown, fine SAND, trace of silt, common debris (concrete, metal, wood, etc.); dry to moist; (Fill).	SP
B4-1-20		6/6/4	20	1.4	1858	20.0	Loose, interbedded gray, gravelly, fine sandy SILT and black, silty, fine SAND; moist; sooty; common debris; (Fill).	SM/ML
B4-2-25		4/4/3	13	1.4	1908	25.0	Black, loose cinders, fine to coarse sand-sized; dry; (Fill).	SW
B4-3-30		9/8/6	87	2.4	1921	30.0	Medium dense, mottled brown and gray, slightly silty to silty, fine SAND; moist to wet; iron-stained; (Native).	SP-SM
						30.8	Medium dense, mottled gray and brown, fine sandy SILT; moist to wet; iron-stained. - Brown and unstained below 35 feet.	ML
B-4-4-35		10/11/11	80	3.4	1936	35.5	Brown, fine SAND, trace of silt; dry.	SP
						36.5	BOTTOM OF BORING 36.5 FEET	

Remarks: Refer to key for explanation of terminology and symbols.

* USC soil descriptions are based on visual classification, unless otherwise noted. Contacts between soil layers are approximate and may be gradual.

LEGEND

- 2" O.D. Split-Spoon Sample
- 3" O.D. Split-Spoon Sample
- Water Level and Date Measured
- Water Level at Time of Drilling

Systems Parking Garage
Seattle, Washington

LOG OF BORING B4

June 1994 W-6752-01

Logged By	Reviewed By		
PVH	KAT	SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. A-4

ENVLOG2 6/30/94

ENVIRONMENTAL BOREHOLE LOG

Date Started 5/26/94	Location T25N R4E Sec. 40	Depth Water First Encountered (Ft)
Date Completed 5/26/94	Drilling Company Pacific Testing Lab.	Drilling Method Hollow-Stem Auger
Total Depth (Ft) 15.0	Sampling Method 2" O.D. Split-Spoon	Hammer: Weight (lbs) 140 Drop (In) 30
Borehole Diam. (In) 7	Ground Elev. NA	Monument Elev. NA / PVC Elev. NA

Depth (Ft)	Sample Number	Interval	Blow Counts/6 In	Recovery(%)	PID (ppm)	Time	Depth (Ft)	Lithologic Description	USCS* Symbol
								Ground Surface	
							0.2	ASPHALT.	
								Black, slightly silty, gravelly fine SAND; sooty; (Fill).	SP-SM
							3.5	Loose, brown, slightly silty, fine SAND with wood from 3.8 to 4 feet; (Fill).	SP-SM
5	B5-1-5		3/2/2	100	2.7	1518	4.0	Loose to medium dense, mottled brown and gray, clayey, fine sandy SILT; moist; occasional thin wood and charcoal lenses; (Fill).	ML
10	B5-2-10		6/6/10	100	3.4	1525	8.5	Medium dense, interbedded brown, slightly silty, fine SAND, iron-stained and mottled gray and brown, silty, fine SAND; moist; (Native).	SP-SM
15	B5-3-15		2/8/8	100	2.4	1534	13.0	Medium dense, mottled gray and brown, slightly clayey, slightly fine sandy SILT with brown, slightly silty, fine SAND lenses from 14.1 to 14.4 feet; moist to wet.	ML
							15.0	BOTTOM OF BORING 15 FEET	

Remarks: Refer to key for explanation of terminology and symbols.

* USC soil descriptions are based on visual classification, unless otherwise noted. Contacts between soil layers are approximate and may be gradual.

LEGEND

- 2" O.D. Split-Spoon Sample
- 3" O.D. Split-Spoon Sample
- Water Level and Date Measured
- Water Level at Time of Drilling

Systems Parking Garage
Seattle, Washington

LOG OF BORING B5

June 1994 W-6752-01

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

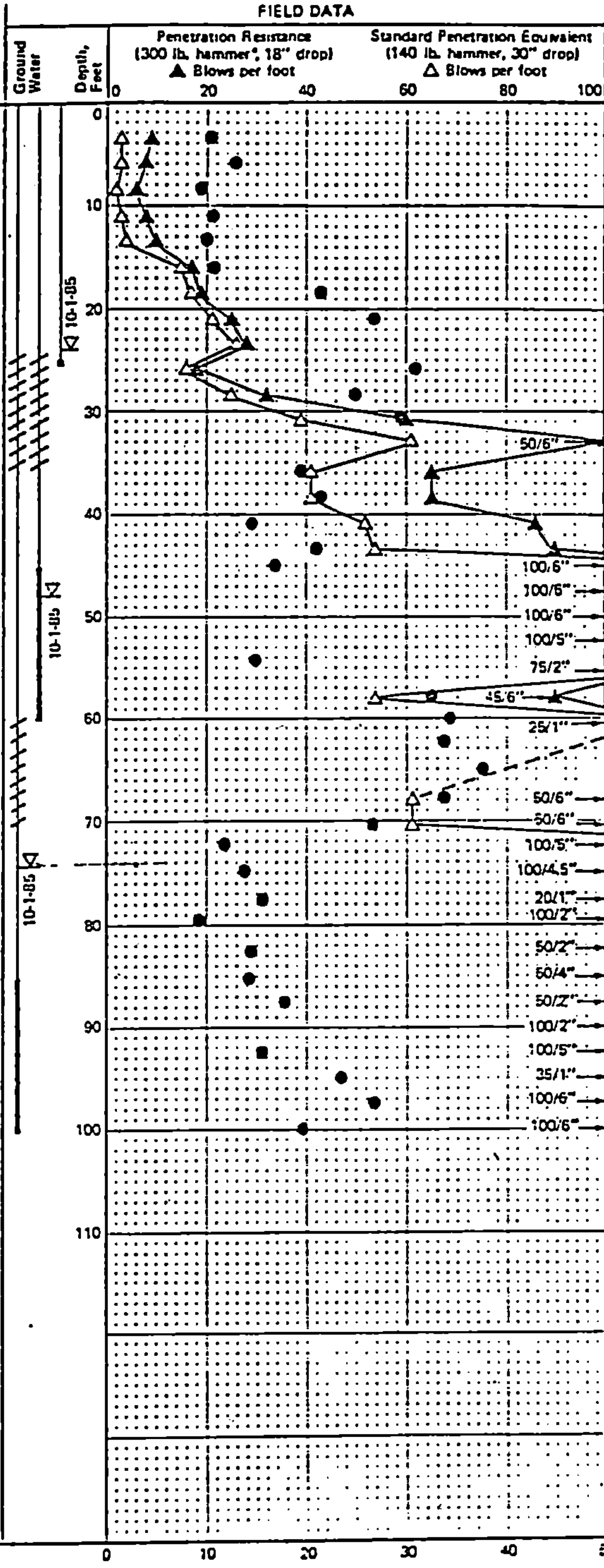
FIG. A-5

Logged By PVH

Reviewed By KAT

ELEV., FEET	SOIL DESCRIPTION (Surface Elevation: 113.8 feet)	DEPTH, FEET	SAMPLES
	Very loose to medium dense, gray-brown, silty, fine gravelly SAND; scattered layers of fine sandy silt and silty clay (FILL)	0	1, 2, 3, 4, 5
99.8	Medium dense, gray, gravelly, clayey SAND (FILL)	14	6, 7
94.8	Medium dense, brown, fine sandy SILT	19	8
91.8	Medium dense, tan, silty SAND and fine sandy SILT (TILL-LIKE)	22	9 N, 10
86.8	Very stiff, brown, slightly fine gravelly, sandy, silty CLAY (TILL-LIKE)	27	11, 12
80.8	Dense to very dense, gray, clean, fine to medium SAND; occasional gravel with silt chunks	33	13 N, 14, 15, 16, 17
69.8	Very dense, gray, silty, sandy GRAVEL; occasional cobbles (TILL-LIKE)	44	18, 19 N, 20 N, 21 N, 22 G, 23 N
56.8	Hard, gray to gray-brown, clayey SILT and silty CLAY; fractured and iron-oxide stained	57	24, 25, 26 P, 27 P
45.8	Hard, gray, gravelly, sandy, silty CLAY and clayey SILT (TILL-LIKE)	68	28, 29
41.8	Very dense, gray, silty, clayey, fine gravelly SAND; locally grades to gravel (TILL-LIKE)	72	30, 31, 32, 33, 34, 35, 36, 37 N, 38, 39
17.8	Very dense, gray, slightly silty to clean, SAND; occasional volcanic ash fragments and fine gravel	96	40
13.3	BOTTOM OF BORING COMPLETED 9-8-84	100.5	41

ROTARY DRILLED WITH REVERT DRILLING FLUID



LABORATORY TESTS (depth)	
GSA	~29 ft
GSA, P-16	~36 ft
GSA	~72 ft

*Downhole Hammer

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

LEGEND

- 2.25" 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-19

STATION 27+40, 12 FT. (R)

JANUARY 1986 W-4265-00

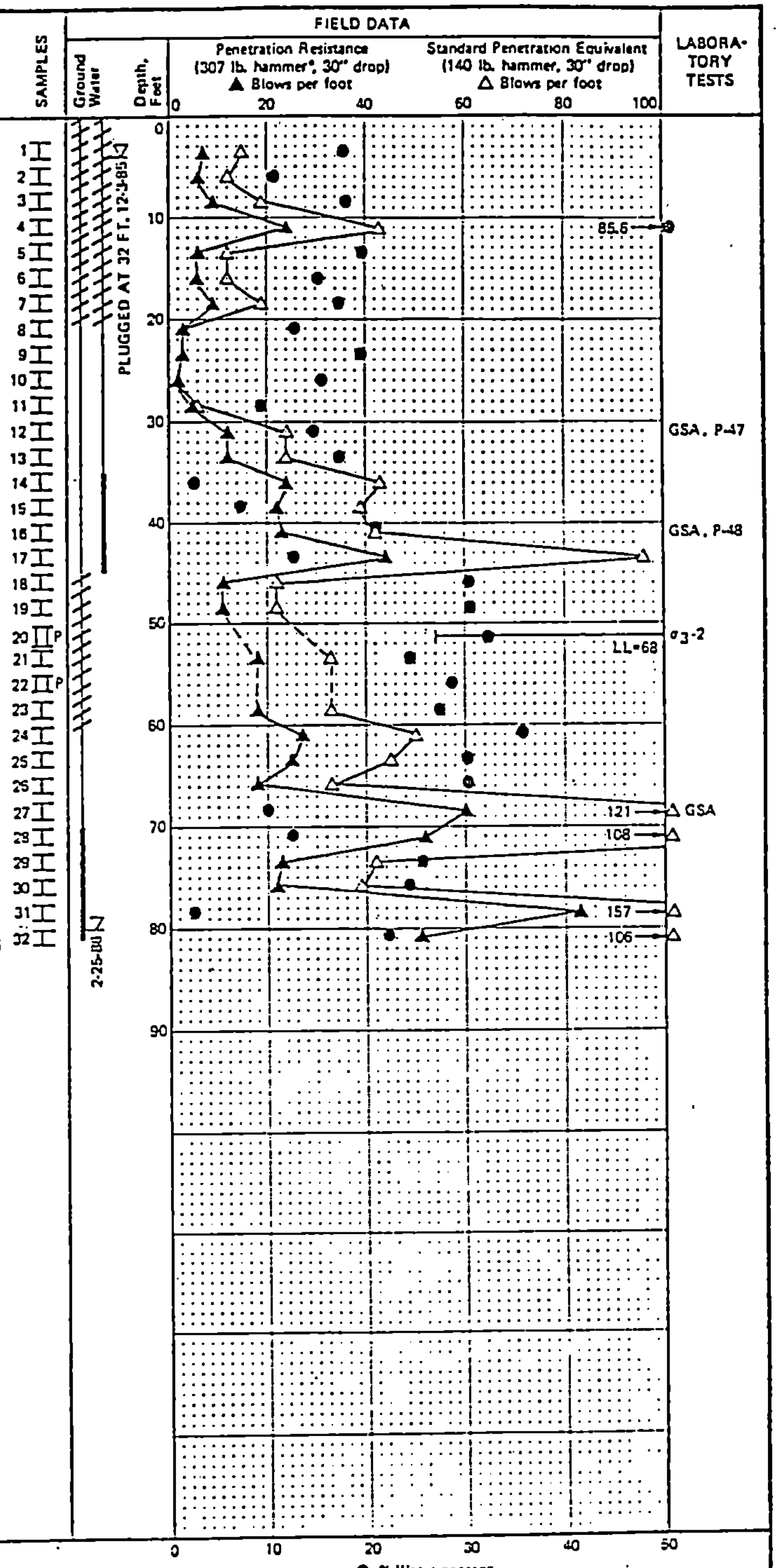
SHANNON & WILSON, INC. Geotechnical Consultants

FIG. A-6

ELEV., FEET	SOIL DESCRIPTION Surface Elevation: ± 121.2 feet	DEPTH, FEET
	Interbedded, medium dense, brown, clean to silty SAND, sandy SILT and hard, gray, clayey SILT; organic debris, compressed peat, disturbed appearance (FILL)	0
101.2	Very loose to loose, brown, silty, fine SAND; trace of organics, silt lenses, scattered angular gravel at 21 feet	20
91.2	Medium dense, brown, slightly silty to silty, fine SAND; trace organics, silt lenses	30
86.7	Dense to very dense, gray-brown, clean to slightly silty, fine SAND; trace of coarse sand and fine gravel, lenses of silt	34.5
76.7	Very stiff to hard, gray, silty CLAY and clayey SILT; laminated to massive, locally blocky, diced and slightly distorted	44.5
54.2	Very dense, gray, slightly clayey to clayey, gravelly, silty SAND and sandy SILT; lenses of clayey SILT (TILL-LIKE)	67
45.2	Very dense, gray-brown, clean, fine to medium SAND; locally gravelly	76
39.7	BOTTOM OF BORING COMPLETED 10-23-85	81.5

HOLLOW STEM AUGER DRILLED WITH WATER

NOTE: SHALLOW PIEZOMETER NOT FUNCTIONAL



*Uphole Hammer

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

LEGEND

- 3" 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-90
STATION 26+77, 23 FT. (L)
JANUARY 1986 W-4265-00

SHANNON & WILSON, INC.
Geotechnical Consultants

FIG. A-7

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE Feb 4, 1970 HOLE NO. 4

PROJECT Downtown Sewer Repl. WEATHER

LOCATION 7th & Olive

WATER TABLE

GRD.EL.	SAMPLE NO.	BLOW COUNT			STD. PEN.	BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
	A	2	4	5	9	SAND	fine w/chunks of SILT	loose	moist	brn
5										
	B	2	4	4	8	SAND	fine w/ silt chunks	loose	moist	brn
10										
	C	2	5	5	10	Mixture	fine SAND & SILT	loose	moist	brn & gr
15										
	D	3	6	7	13	SAND	fine w/silt chunks	firm	moist	brn & gr
20										
	E	4	7	6	13	Layers	fine sandy SILT, SILT & fine SAND	firm	moist	gray
25										
	F	3	4	5	9	SAND	silty fine	loose	moist	brn
30										

INSPECTOR:

FIG. A-8
Sheet 1 of 2

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 2/4/70 HOLE NO. 4

PROJECT Downtown Sewer WEATHER

LOCATION 7th & Olive

WATER TABLE

GRD. EL.	SAMPLE NO.	BLOW COUNT			STD. PEN.	BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
	G	2	3	4	7	SAND	fine, silty Top 3" organics	loose	moist	brn
	H	3	7	13	20	CLAY	sandy silty	stiff	moist	mottled gr & brn
35	I	4	10	13	23	CLAY	silty (6" wet) 1/4" fine SAND layer	stiff	wet	mottled gr & brn
	J	6	12	16	28	SAND	fine silty w/thin silt layer	firm	wet	brn
40	K	4	13	20	33	SAND	fine	Compact	moist	brn
	L	3	16	24	40	SAND	fine	Compact	moist	brn
45										

Bott. Hole

INSPECTOR:

FIG. A-8

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

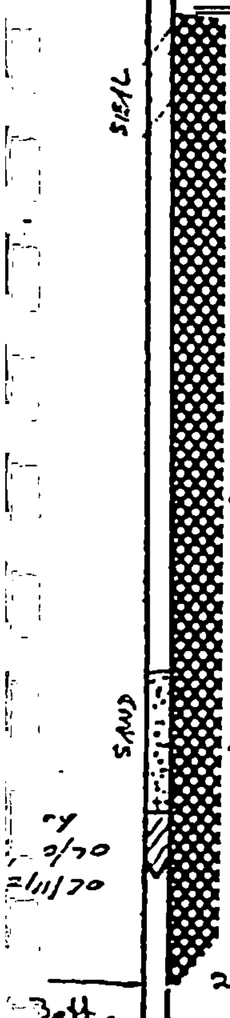
LOG OF TEST BORING

DATE 2/18/70 HOLE NO. 5

PROJECT Downtown Sewer Repl. WEATHER

LOCATION 6th & Olive

WATER TABLE



GRD. EL.	SAMPLE NO.	BLOW COUNT			STD. PEN.	BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
	A	2	2	2	4	SAND-SILT	mixture Fill	Loose	moist	gr-br
5										
	B	–	–	1	1	SAND-SILT	mixture Fill	Loose	moist	gr-brn
10										
	C	–	–	2	2	6" SILT 6" SAND	w/gravel	Loose	moist	gray brn
15										
	D	0	–	1	1	SILT	fine sandy	Very Soft	wet	gray
							HARD DRILLING			
20							REFUSAL			

INSPECTOR: ACR

FIG. A-9

Duplicate

SEATTLE ENGINEERING DEPARTMENT
MATERIALS LABORATORY

LOG OF TEST BORING

DATE 2/18/70 HOLE NO. 5-A

PROJECT Downtown Sewer WEATHER

LOCATION 3' South of 5

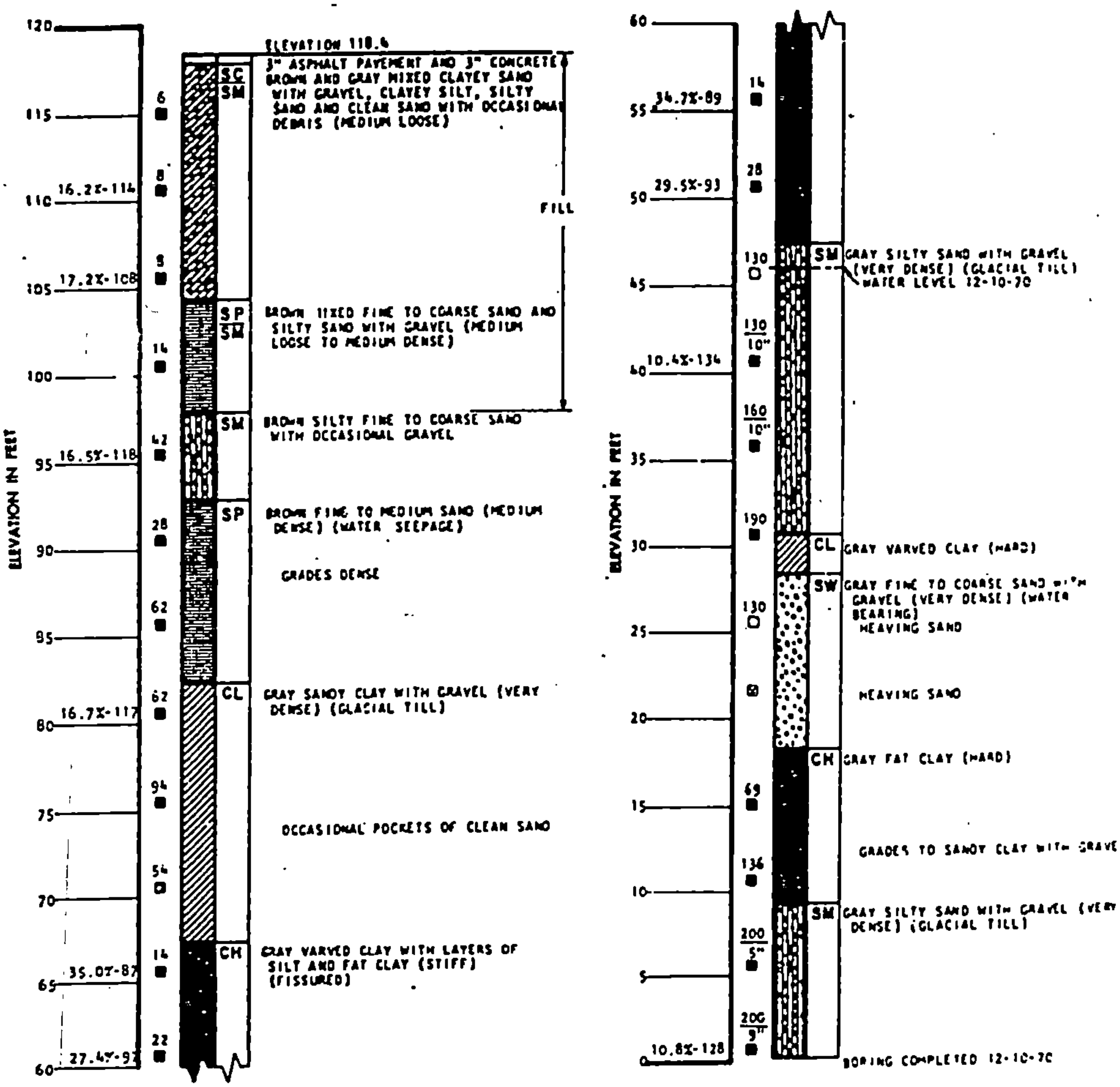
WATER TABLE

GRD.EL.	SAMPLE NO.	BLOW COUNT			STD. PEN.	DESCRIPTION OF MATERIAL: BASE MAT'L.	AUXILIARY MATERIAL	CONSISTENCY	MOISTURE	COLOR
5										
10										
	A	0	1	1	2	SILT	mixed with fine SAND (FILL)	very soft	moist	gray
15										
	B	1	1	1	2	SILT	mixed with fine SAND & gravel (FILL)	Very soft	moist	gray
						2" SILT	Organic			Dark brn
	C	—	1	50/3½	50+	4" SAND	fine w/silt & gravel	Very Compact	wet	brn
						4" GRAVEL	(Oil)			gray
20						REFUSAL				

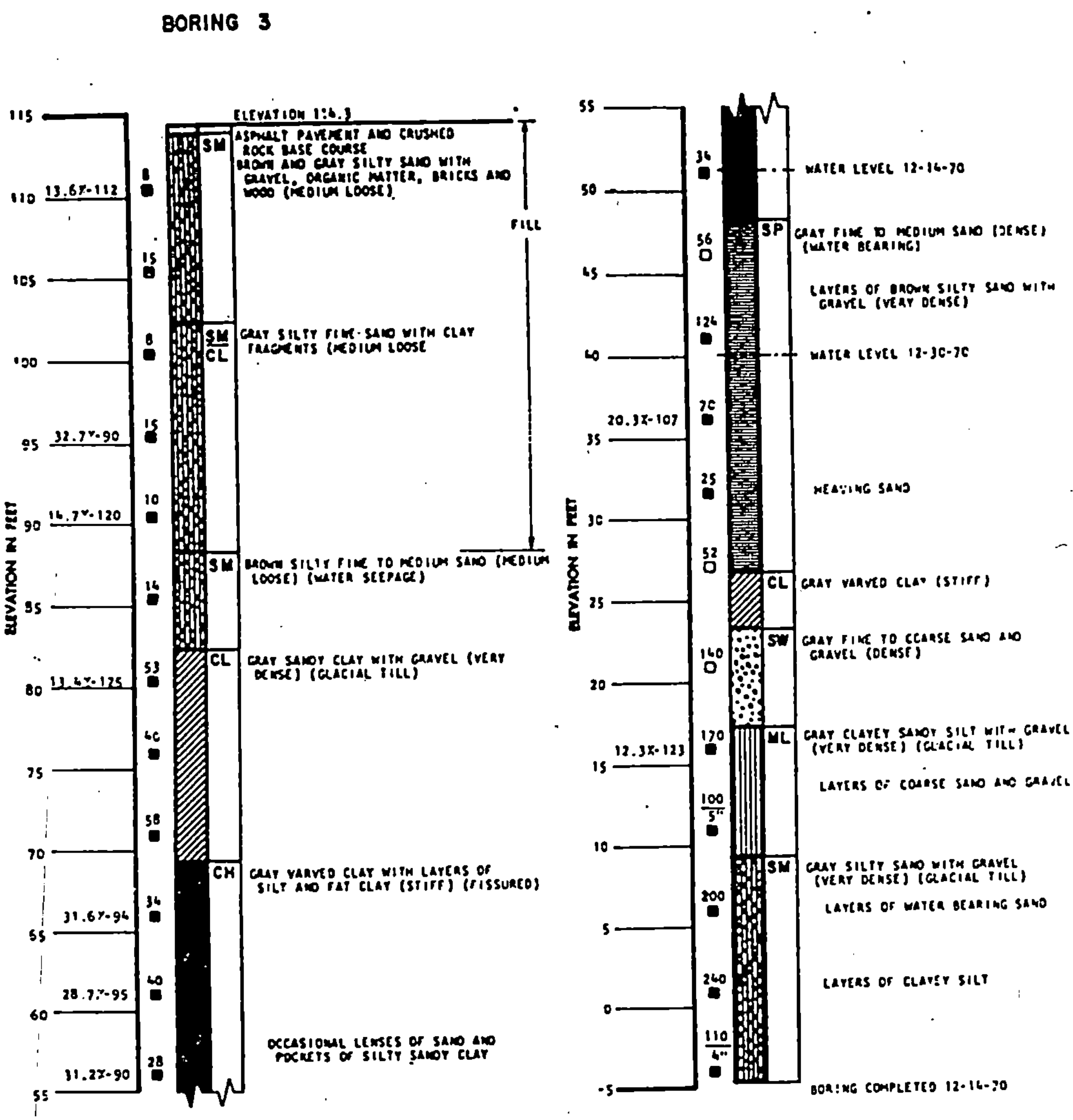
INSPECTOR: ACR

FIG A-9

BORING 2



LOG OF BORINGS

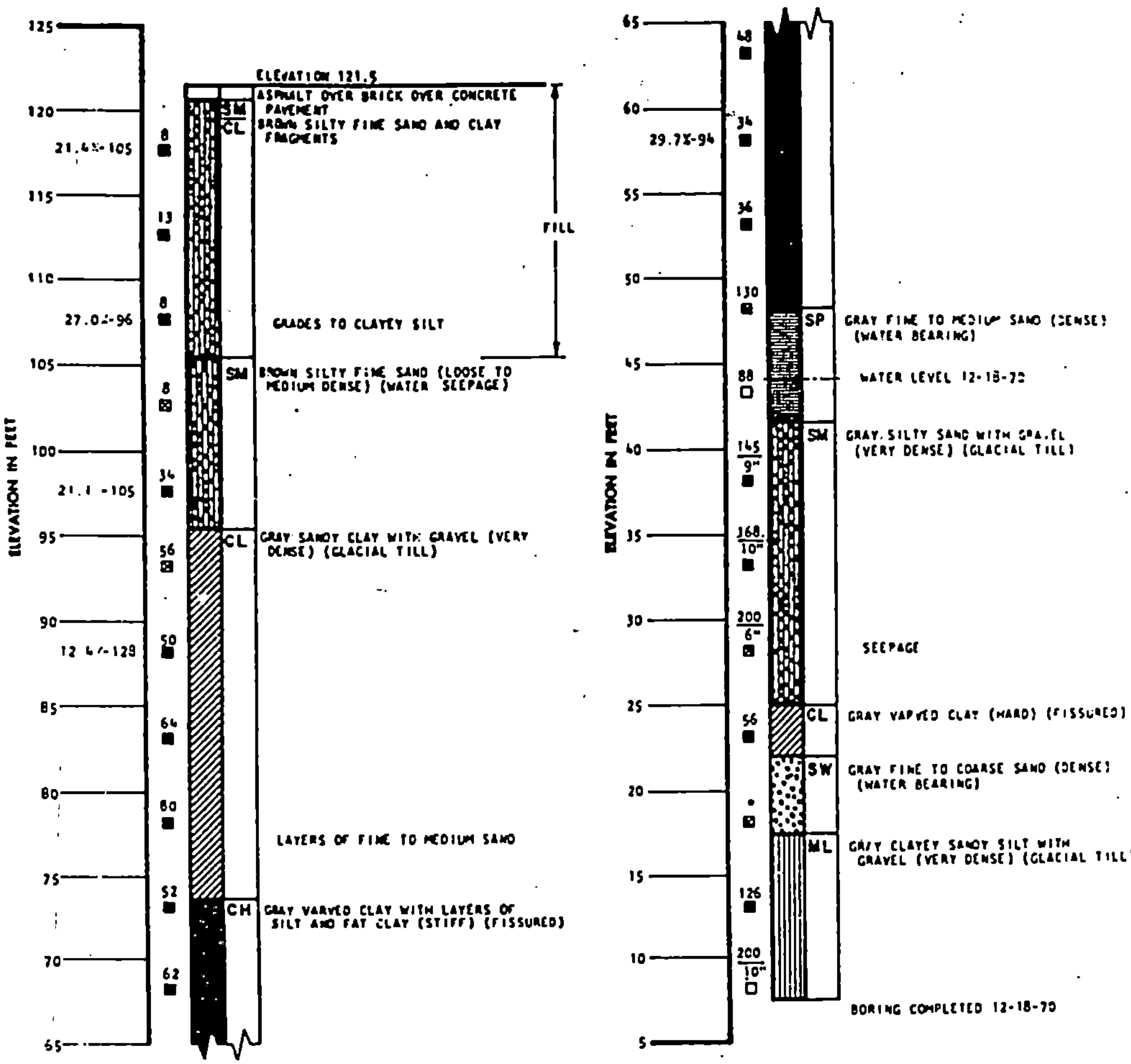


LOG OF BORINGS

DAMES & MOORE

FIG. A-11

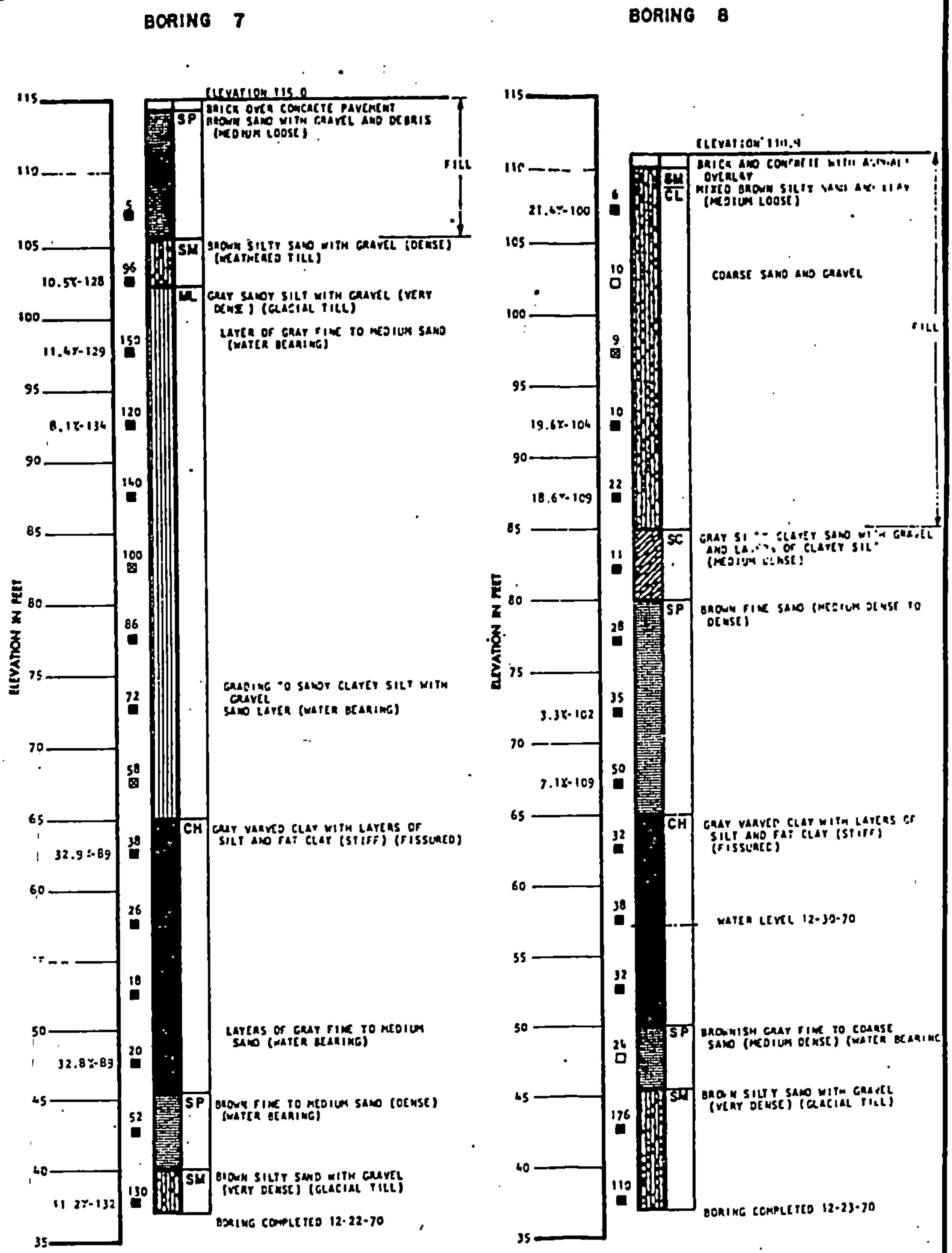
BORING 5



LOG OF BORINGS

DAMES & MOORE

FIG. A-12



LOG OF BORINGS

DAMES & MOORE

FIG. A-13

APPENDIX B

LABORATORY TEST RESULTS

APPENDIX B

LABORATORY TEST RESULTS

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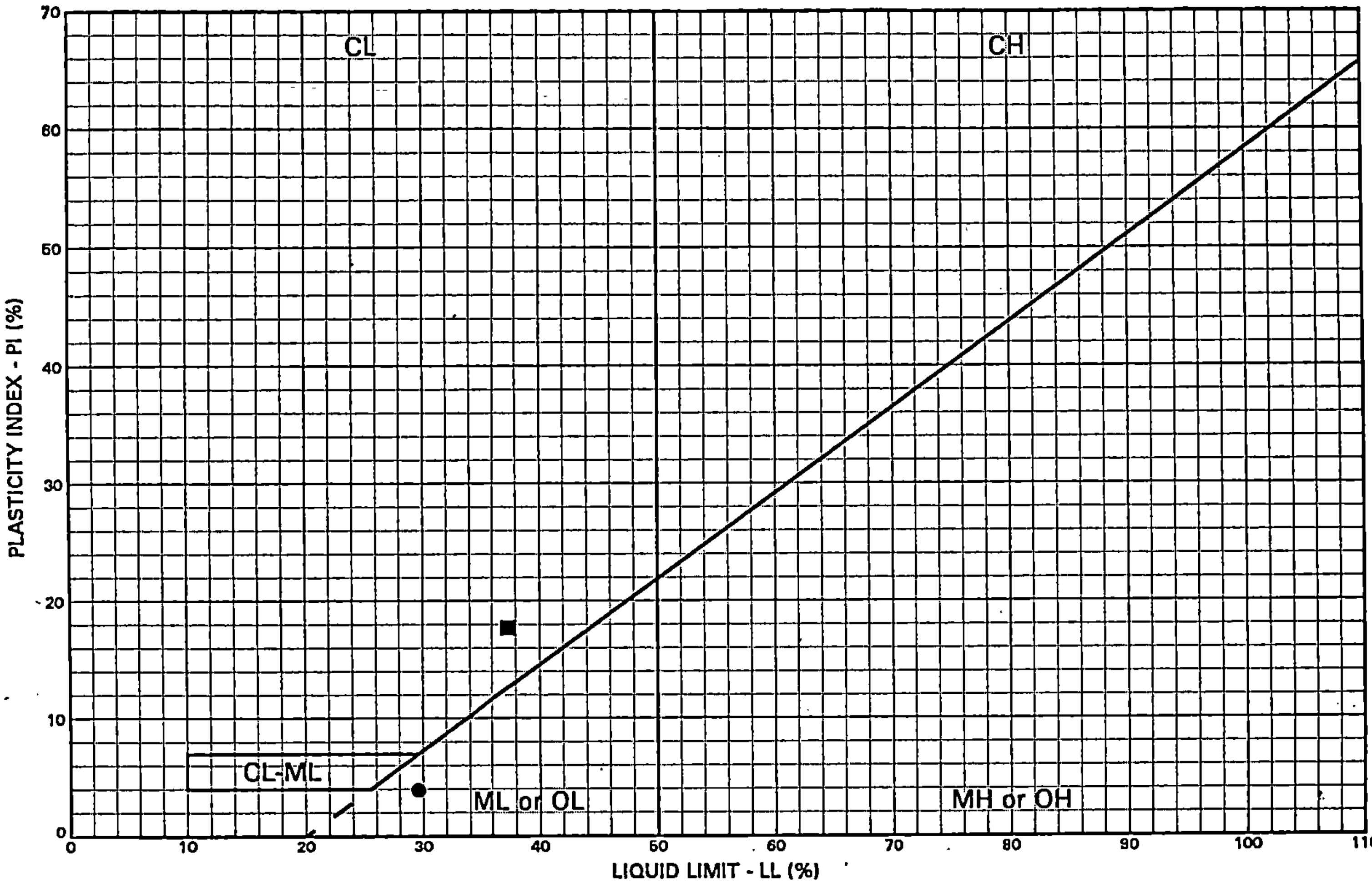
LIST OF FIGURES

Figure No.



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, FT.	U.S.C.	CLASSIFICATION	LL, %	PL, %	PI, %	NAT. W.C. %	PASS. #200, %
● B-1, S-6	30.0	ML	Gray-brown, slightly clayey SILT; trace of sand.	30	26	4	23.7	
■ B-2, S-5	25.0	CL	Gray and brown, sandy, silty CLAY; trace of gravel.	37	20	18	22.5	

Systems Retail Building
Seattle, Washington

PLASTICITY CHART
BORINGS B-1 AND B-2

February 1995 W-6913-01

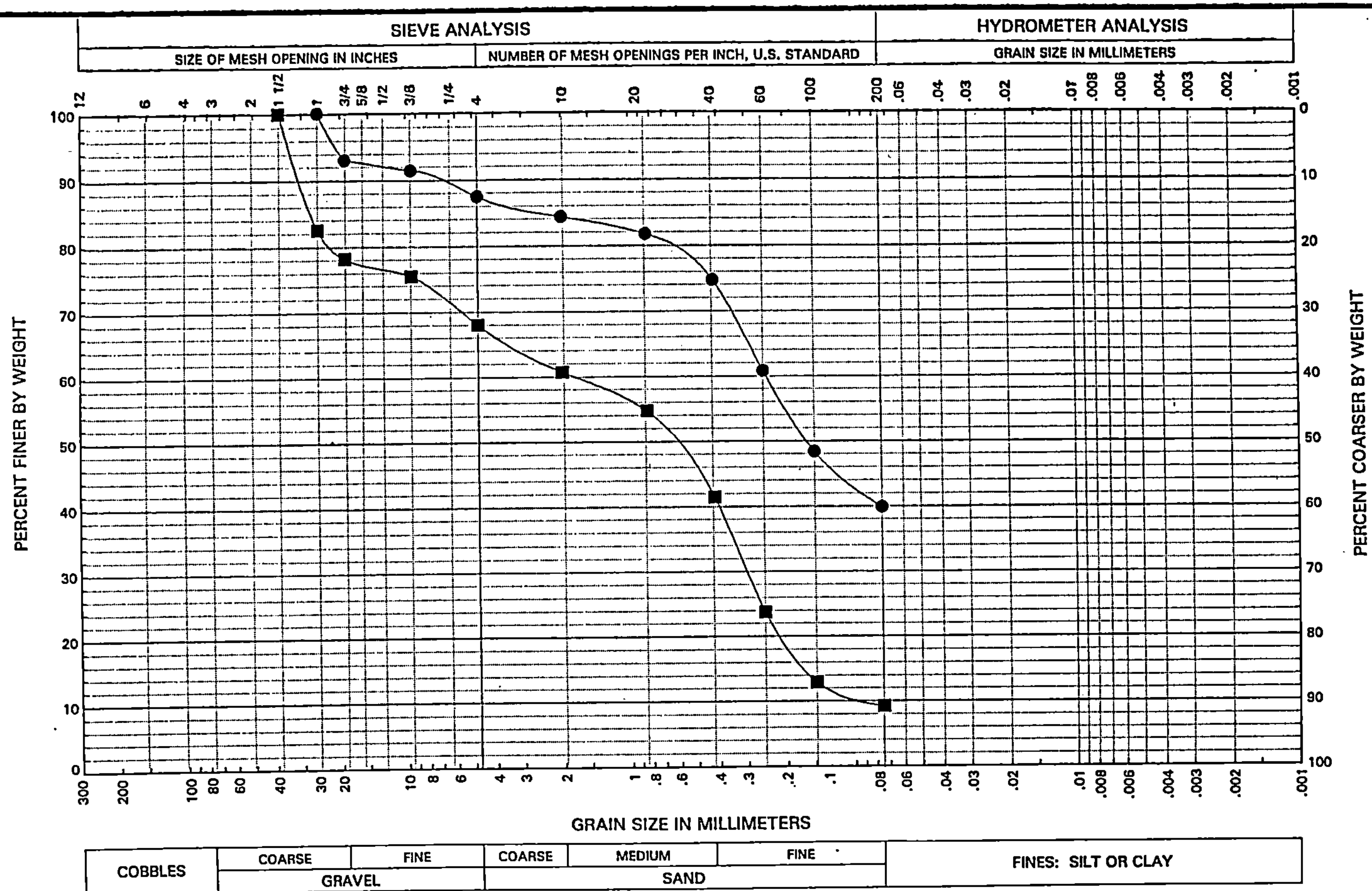
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-1

PLASTCTY 2/13/95

FIG. B-1

NCDGRAIN 2/10/95



BORING AND SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	% FINES	NAT. W.C. %	LL	PL	PI
● B-1, S-3	15.0	SM	Gray-brown, gravelly, silty SAND.	39.8	12.9			
■ B-1, S-10	50.0	SP-SM	Gray-brown, slightly silty, gravelly SAND.	9.2	12.6			

Systems Retail Building
Seattle, Washington

GRAIN SIZE DISTRIBUTION
BORING B-1

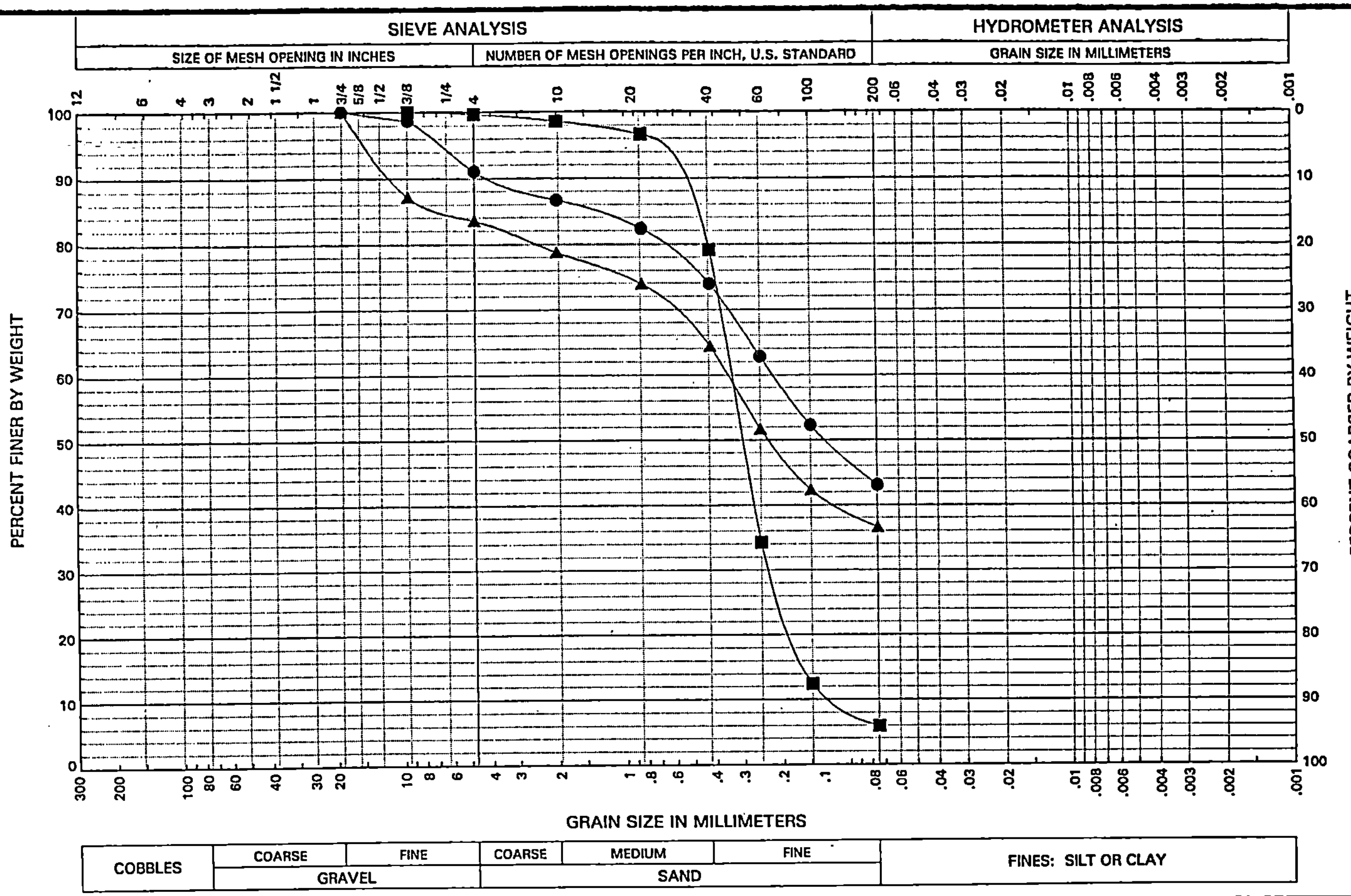
February 1995 W-6913-01

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-2

FIG. B-2

NCDGRAIN 2/10/95



BORING AND SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	% FINES	NAT. W.C. %	LL	PL	PI
● B-2, S-4	20.0	SM	Gray-brown, slightly gravelly, clayey, silty SAND.	43.0	18.9			
■ B-2, S-8	40.0	SP-SM	Gray-brown, slightly silty SAND.	5.9	15.6			
▲ B-2, S-15	75.0	SM	Gray, gravelly, silty SAND.	36.5	10.5			

Systems Retail Building
Seattle, Washington

GRAIN SIZE DISTRIBUTION
BORING B-2

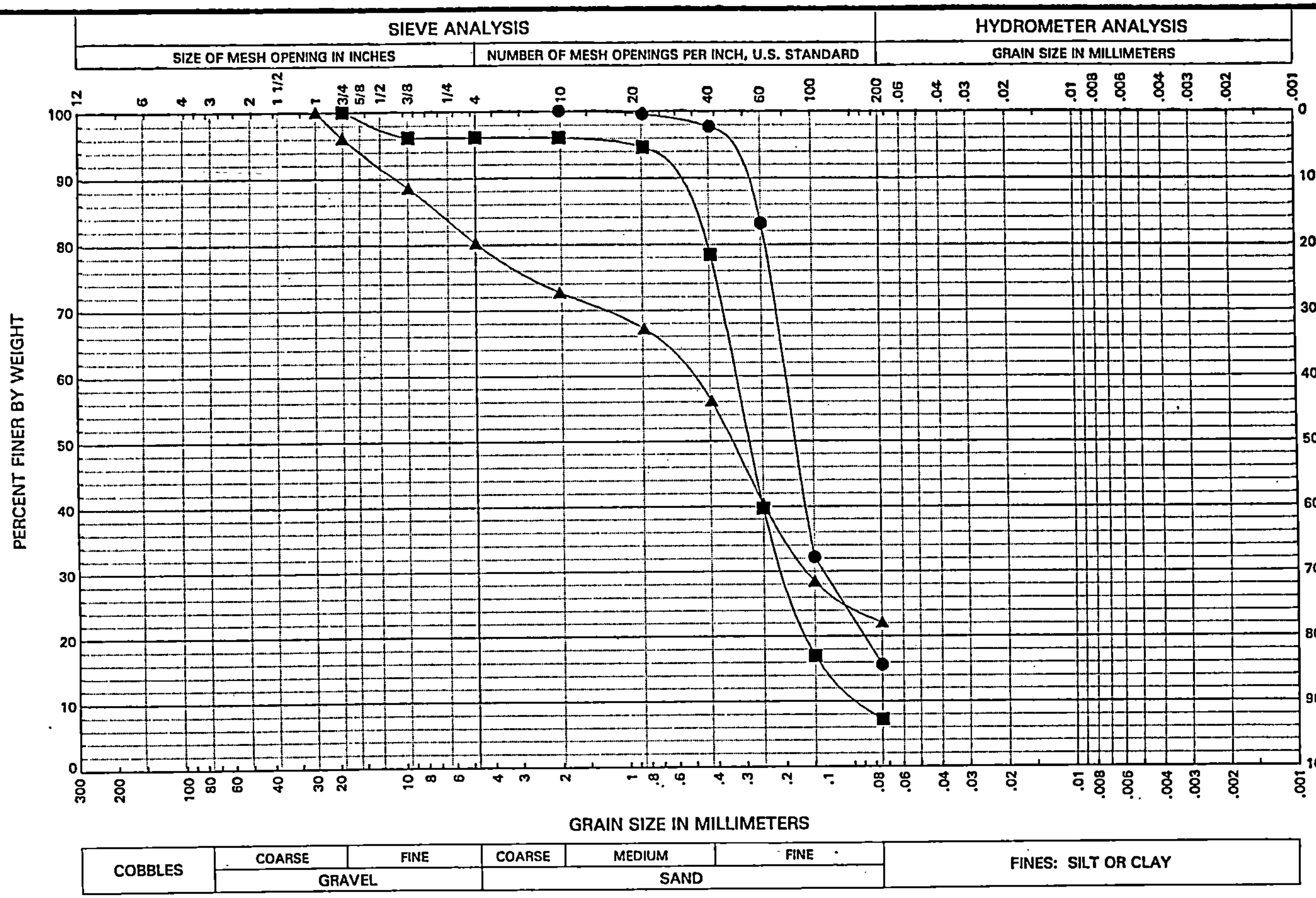
February 1995 W-6913-01

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-3

FIG. B-3

NCDGRAIN 2/10/95



BORING AND SAMPLE NO.	DEPTH, FT.	U.S.C.	CLASSIFICATION	% FINES	NAT. W.C. %	LL	PL	PI
● B-3, S-7	35.0	SM	Brown and gray, silty SAND.	15.6	8.0			
■ B-3, S-9	45.0	SP-SM	Gray, slightly silty SAND.	7.2	5.1			
▲ B-3, S-14	70.0	SM	Gray, gravelly, silty SAND.	22.1	12.4			

Systems Retail Building
Seattle, Washington

GRAIN SIZE DISTRIBUTION
BORING B-3

February 1995 W-6913-01

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-4

FIG. B-4

APPENDIX C

GENERALIZED SUBSURFACE PROFILES FROM METRO DOWNTOWN SEATTLE TRANSIT PROJECT

APPENDIX C

GENERALIZED SUBSURFACE PROFILES FROM METRO DOWNTOWN SEATTLE TRANSIT PROJECT

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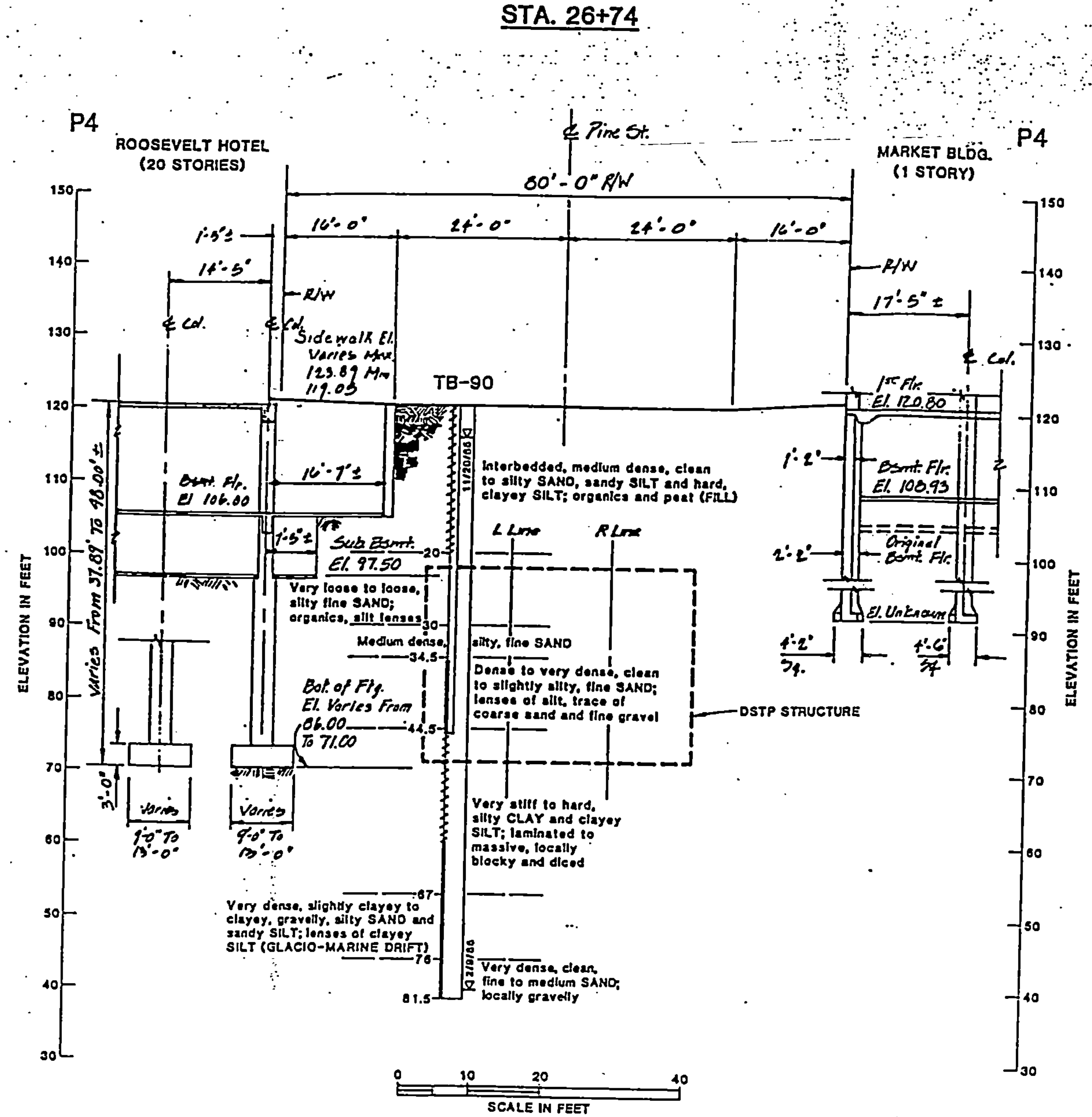
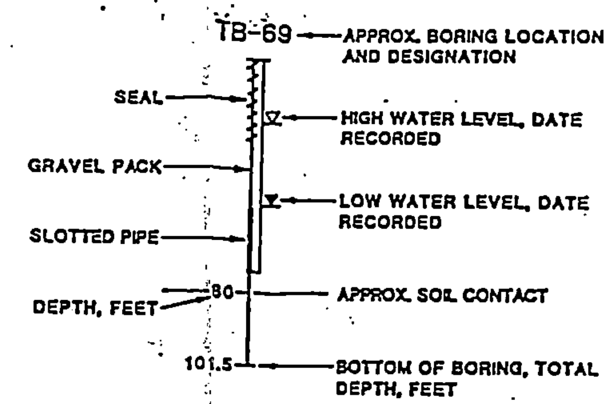


LEGEND



NOTES

1. THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS ENCOUNTERED IN THE BORINGS. VARIATIONS BETWEEN THIS SUBSURFACE PROFILE AND ACTUAL CONDITIONS MAY EXIST.
2. BASE DRAWINGS SHOWING FOUNDATION CONDITIONS ARE ADAPTED FROM ABKJ, FILE NO. CP 201, DRAWING NO. PS-888, DATED AUGUST 30, 1985.
3. APPROXIMATE BORING ELEVATIONS ARE COMPILED FROM "BASE TOPOGRAPHIC MAPS," DELIVERABLE NO. 5, DSTP-PB CATALOG NO. B03.03, BY HORTON DENNIS & ASSOCIATES, INC., DATED FEBRUARY 1, 1985.
4. EXISTING STRUCTURE INFORMATION SHOWN ON THESE DRAWINGS IS COMPILED FROM INFORMATION FURNISHED BY VARIOUS SOURCES AND IS NOT FIELD VERIFIED. THE ACCURACY AND COMPLETENESS OF THE INFORMATION SHOWN IS NOT GUARANTEED.

NO	REVISION	BY	DATE

DOWNTOWN SEATTLE TRANSIT PROJECT

The preparation of this document was partially financed through a grant from the U.S. Department of Transportation, Urban Mass Transportation Administration under the provisions of the Urban Mass Transportation Act of 1964, as amended.

SHANNON & WILSON, INC.
Geotechnical Consultants
W-4385

Parsons Brinckerhoff
Parsons Brinckerhoff Quade & Douglas, Inc. • Seattle, Washington 98104
SUBMITTED

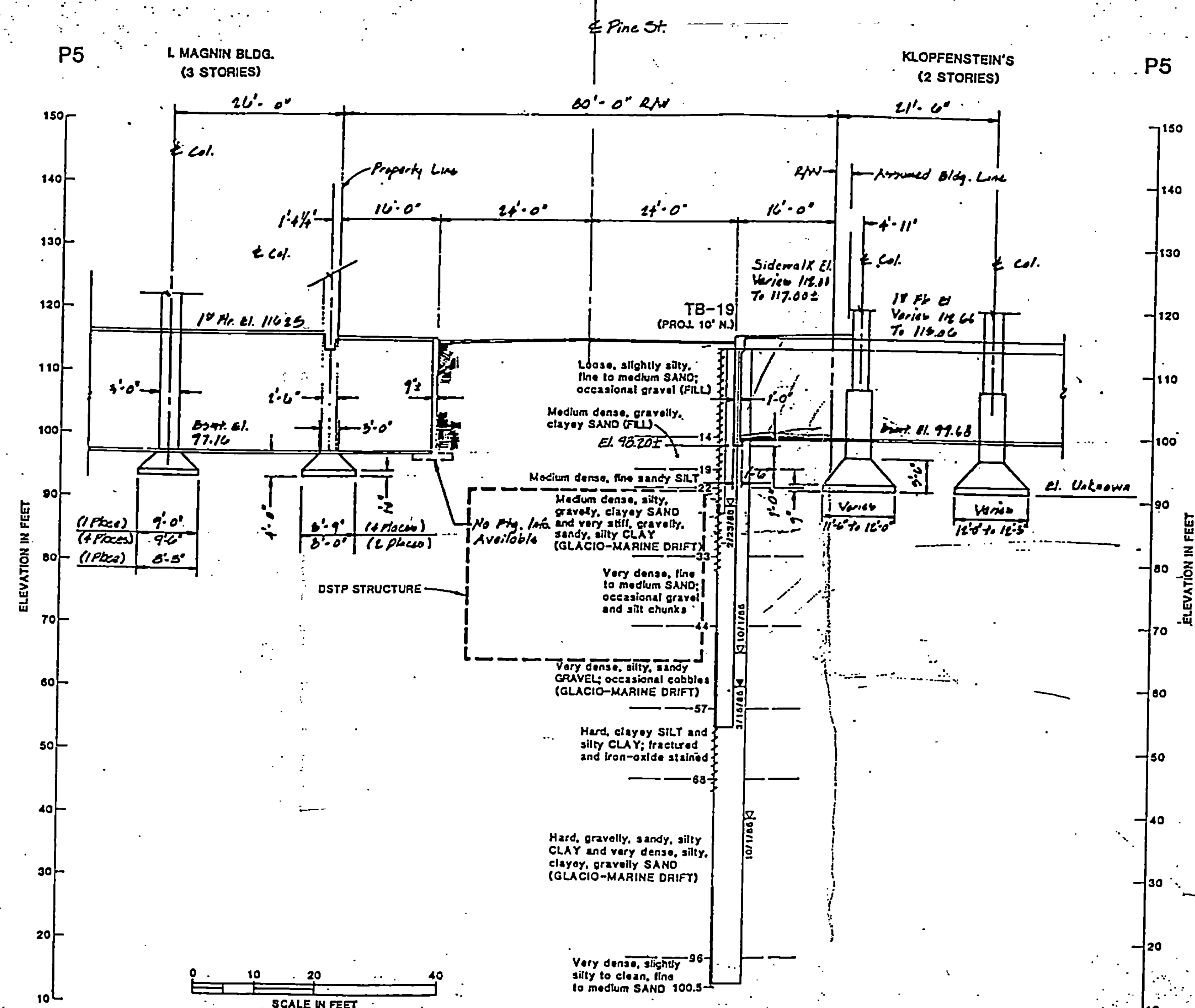
DESIGNED
DRAWN
CHECKED
IN CHARGE
SCALE
APPROVED

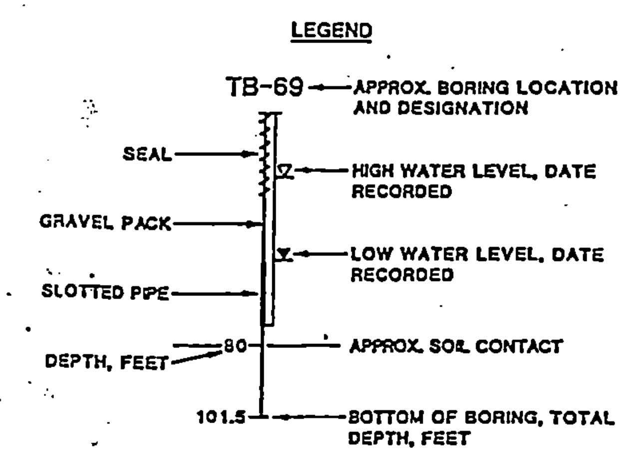
METRO Municipality of Metropolitan Seattle

GENERALIZED SUBSURFACE PROFILE
PINE ST. LINE STRUCTURE
AT STA. 26+74 (P4)

DATE MARCH 1986
FILE NO.
DWG NO.
DRAWING NO. FIG. C-1

STA. 28+30





NOTES

1. THIS SUBSURFACE PROFILE IS GENERALIZED FROM MATERIALS ENCOUNTERED IN THE BORINGS. VARIATIONS BETWEEN THIS SUBSURFACE PROFILE AND ACTUAL CONDITIONS MAY EXIST.
2. BASE DRAWINGS SHOWING FOUNDATION CONDITIONS ARE ADAPTED FROM ABKJ, FILE NO. CP 201, DRAWING NO. PS-867, DATED AUGUST 30, 1985.
3. APPROXIMATE BORING ELEVATIONS ARE COMPILED FROM "BASE TOPOGRAPHIC MAPS," DELIVERABLE NO. 5, DSTP-PB CATALOG NO. B03.03, BY HORTON DENNIS & ASSOCIATES, INC., DATED FEBRUARY 1, 1985.
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DOWNTOWN SEATTLE TRANSIT PROJECT

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SHANNON & WILSON, INC.
Geotechnical Consultants

Parsons Brinckerhoff

METRO Municipality of Metropolitan Seattle

DESIGNED | DRAWN | CHECKED | SCALE | APPROVED

DATE MARCH 1986

GENERALIZED SUBSURFACE PROFILE
PINE ST. LINE STRUCTURE
AT STA. 28+30 (P5)

FIG. C-2

APPENDIX D

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Dated: November 3, 1995

To: Pine Street Associates
Attn: Mr. Matt Griffin

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.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland