Geotechnical and Environmental Engineering
Addendum Report
Snohomish County Campus
Administration Building
Everett, Washington

May 2003

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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SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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May 22, 2003

Mr. Larry Goetz
NBBJ
111 South Jackson Street
Seattle, WA 98104

RE: GEOTECHNICAL REPORT – SNOHOMISH COUNTY CAMPUS ADMINISTRATION BUILDING, EVERETT, WASHINGTON

Dear Mr. Goetz:

We are pleased to submit three copies of our geotechnical report for the proposed Snohomish County Administration Building.

We appreciate the opportunity to be of continuing service to you. Should you have any questions regarding this report, please contact us.

Sincerely,

SHANNON & WILSON, INC.

Thomas M. Gurtowski

Thomas M. Gurtowski, P.E.
Vice President

WLB:TMG/lkd

Enclosure: Geotechnical Report (3 copies)

c: Pat Harrigan, MKA (1 copy)

21-1-09644-005.R1Add-L1/wp/lkd

TABLE OF CONTENTS

LIST OF FIGURES

Figure No.

LIST OF APPENDICES

Appendix

GEOTECHNICAL AND ENVIRONMENTAL ENGINEERING ADDENDUM REPORT
SNOHOMISH COUNTY CAMPUS
ADMINISTRATION BUILDING
EVERETT, WASHINGTON

1.0 INTRODUCTION

This addendum report presents the results of subsurface explorations and our geotechnical engineering recommendations for the proposed Snohomish County (County) Campus Administration Building in Everett, Washington. The purpose of this study was to complete subsurface explorations at the project site and to provide geotechnical and environmental engineering conclusions and recommendations for the design and construction of the proposed facility. We prepared a geotechnical and environmental engineering Report, "Snohomish County Campus Administration Building and Garage", dated August 2002; at that time the Administration Building design was not yet fully underway and our recommendations were preliminary. This report has been prepared as an addendum to the aforementioned report to address the geotechnical and environmental issues regarding the Administration Building.

Our work was performed in general accordance with our proposals dated January 7 and February 20, 2002; the February proposal was later revised on May 3, 2002. Tasks included in these two proposals are field explorations and preparation of a memorandum summarizing field findings, and geotechnical design services. Mr. Mike Reyder of NBBJ authorized our initial scope of work for field explorations on January 30, 2002; on May 30, 2002, Mr. Larry Goetz of NBBJ authorized our design services proposal dated May 3, 2002 and on October 1, 2002 Mr. Larry Goetz authorized our proposal for additional two borings at the Snohomish County Administration Building Complex.

2.0 SITE AND PROJECT DESCRIPTION

The project location is shown on the Vicinity Map, Figure 1. The proposed Administration Building is bounded by the Administration Building to the west, Pacific Avenue to the south, Oakes Avenue to the east, and the parking garage excavation to the north. The proposed footprint of the structure is shown on the Site and Exploration Plan, Figure 2.

The construction of the Administration Building addition will require demolition of the existing three-story, concrete parking garage. The utility infrastructure for the existing County Administration Building is presently housed in the basement of the existing parking garage and will need to be relocated.

The ground floor of the new Administration addition will be connected to the existing Administration Building and will house offices and public hearing and meeting rooms. Currently, the Administration Building addition finish floor elevation is anticipated to range from approximately 132 to 145 feet.

3.0 FIELD EXPLORATIONS

3.1 Borings

Along with the applicable borings from our August 2002 report, the field explorations for this study included drilling two additional borings, designated B-19 and B-20. These borings were drilled on October 28, 2002. The borings were advanced to depths of approximately 30 feet, which correspond to approximate bottom elevations of 119 and 102 feet. No samples were collected from the two borings for analytical testing.

The field explorations completed for our August 2002 report included drilling fifteen borings, designated B-7 through B-12, B-12A, B-13 through B-16, B-16A, B-16B, B-17, and B-18. Boring logs and analytical test results for selected soil samples are provided in our August 2002 report.

The approximate locations of the subsurface explorations are shown on Figure 2. The locations of the new explorations were established by measuring from existing site features shown on the site map. The locations of all the borings should be considered approximate. Appendix A, Field Explorations, discusses drilling and sampling techniques and presents detailed logs for borings B-19 and B-20. The boring logs for the 15 original borings are included in our August 2002 report.

3.2 Observation Wells

No observation wells were installed in the two additional borings. For information pertaining to the wells that were installed in the previous borings that were, please refer to our August 2002 report.

4.0 GEOLOGY

The geologic deposits present in the vicinity of the site were largely deposited during the last glacial advance, known as the Vashon Stade of the Frasier glaciation. As the Vashon ice sheet advanced from the north, drainage from Puget Sound was blocked, and glaciolacustrine silt and clay, with some sand seams, were deposited in a proglacial lake. As the glacial ice sheet advanced further, sand and some gravel (advance outwash) were deposited on top of the glaciolacustrine sediments as a broad outwash plain in front of the glacier. The advance outwash typically is gradational with the underlying glaciolacustrine deposits at the base (interbedded sand and silt) and coarsens upward to sand and then gravelly sand at the top. The glacial ice eventually overrode the area, compacting the underlying sediments and depositing lodgment till at the base of the glacier. The till is a non-sorted mixture of clay, silt, sand, and gravel with scattered cobbles and boulders.

Fill was encountered in the alleyway between the garage and the existing Administration Building. The fill was probably placed during construction of the existing Administration Building.

5.0 SUBSURFACE CONDITIONS

5.1 General

The subsurface conditions at the site were evaluated based on conditions encountered in borings B-7, B-8, B11, B-12, B-12A, and B-18 through B-20. A description of the soil and groundwater conditions disclosed by the borings is presented below. Figures 3, 4 and 5 present generalized subsurface profiles running north-south, and east-west in the south half of the proposed building. The locations of the borings and profiles are shown on Figure 2.

5.2 Soil

As shown on the attached subsurface profiles (Figures 3 through 5), and as described in the geology section above, the site's subsurface conditions generally consist of the following geologic layers starting from the ground surface: fill, glacial till, advance outwash, glaciolacustrine deposits, and glacial outwash. The fill generally consists of slightly gravelly, slightly silty to silty sand. The glacial till generally consists of very dense, slightly gravelly to gravelly, silty sand. Advance outwash consists of very dense, slightly silty to silty sand and slightly clayey, sandy silt. The advance outwash has localized areas that are slightly gravelly to gravelly. The glaciolacustrine deposits are hard and vary from silty clay/clayey silt to slightly sandy, slightly clayey silt. These fine-grained deposits have scattered to numerous silty, fine sand partings/seams and slickensides. In some areas, scattered seams of highly plastic clay were noted in the glaciolacustrine deposits.

Boring B-11, located between the proposed and existing Administration Buildings, encountered 28 feet of very loose fill, which was probably placed during construction of the existing Administration Building. Boring B-19 encountered approximately 16.5 feet of fill; however, no fill was encountered in boring B-20.

5.3 Groundwater

Groundwater conditions at the proposed Administration Building were evaluated by observations made during drilling and by installing an observation well in boring B-11. Wet, caving soils were encountered in boring B-12A.

Where observed, groundwater was noted during drilling and the during-drilling measurements of groundwater levels are noted on the boring logs. Groundwater was encountered at a depth of approximately 29 feet below ground surface (elevation 120 feet) in boring B-19. No groundwater was encountered in boring B-20 during drilling. More detailed information regarding the groundwater conditions at the campus site can be found in our August 2002 report.

6.0 ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

6.1 General

This report presents Administration Building recommendations regarding:

- Seismic design criteria
- Earthquake-induced geologic hazards
- Shallow Footing foundations
- Mat foundations
- Lateral earth pressures and resistance
- Floor slabs
- Temporary excavation shoring
- Temporary cut slopes
- Potentially contaminated soil excavation
- Removal of underground storage tanks (USTs)

- Temporary dewatering
- Permanent drainage
- Fill placement and compaction

6.2 Seismic Design Considerations

The project is located in a moderately active seismic region. While the region has historically experienced moderate to large earthquakes (such as the April 13, 1949, magnitude 7.1 Olympia Earthquake; April 29, 1965, magnitude 6.5 Seattle-Tacoma Earthquake; and February 28, 2001, magnitude 6.8 Nisqually Earthquake) geologic evidence suggests that larger earthquakes have occurred in the recent past and will continue to occur in the future (for example, magnitude 8½ to 9 Cascadia Subduction Zone Interplate events and/or magnitude 7½ Seattle Fault events). We understand that the seismic design of the Administration Building will be in accordance with the International Building Code (IBC) 2000. Computation of forces used for the code seismic design is based on seismological input and site soil response factors.

For the IBC 2000, the seismological inputs are short period spectral acceleration, S_S, and spectral acceleration at the 1-second period, S_1, shown on Figure 1615 in the code. S_S and S_1 are for a maximum considered earthquake, which correspond to ground motions with a 2 percent probability of exceedance in 50 years or about a 2,500 year return period (with a deterministic maximum cap in some regions). The mapped S_S and S_1 values in the vicinity of the project are 1.25g and 0.40g, respectively.

In addition to seismicity, the IBC also requires that the response of the subsurface soils at the site be considered in developing design earthquake ground motions. A site soil classification is used to represent the soil conditions at the site. Because the project site is generally underlain by dense to very dense and hard soils, which are anticipated to extend to a depth of several hundred feet, we recommend that the soils at this site be characterized as an IBC Site Class C. The corresponding seismic coefficients Fa and Fv have values of 1.0 and 1.4, respectively.

6.3 Earthquake-induced Geologic Hazards

In general, earthquake-induced geologic hazards may include liquefaction, lateral spreading, slope instability, and ground surface fault rupture. In our opinion, the potential for liquefaction and lateral spreading is not significant because of the dense/hard nature of the on-site soils. The ground surface at the site slopes gently down to the northeast, therefore, the potential for significant earthquake-induced slope instability is also low. In our opinion, the potential for ground surface fault rupture at the site is low because the nearest known fault is in the northwest-southeast trending Southern Whidbey Island Fault zone, located approximately 5 to 6 miles southwest of the site.

6.4 Shallow Footing Foundations

We recommend that spread footing foundations be used to support the proposed Administration Building. Based on our borings, native, very dense and hard, glacially overridden soil would likely be encountered at or near the lowest floor elevations (132 to 145 feet). For footings bearing in the very dense or hard, native soil, we recommend a maximum allowable bearing capacity of 12 kips per square foot (ksf); this allowable value corresponds to an ultimate bearing capacity of 24 ksf in glacial soil. The allowable value can be increased by one-third to account for wind and seismic loading conditions. The allowable bearing capacity is based on the assumption that the subgrade preparation recommendations, which are discussed in this report, are followed. We recommend a minimum footing embedment of 24 inches below the lowest adjacent grade. We also recommend a minimum width of 18 inches for continuous footings and 24 inches for column footings. In areas where overexcavation is required to reach the dense/hard native soils, the over excavation should be backfilled with lean mix or structural concrete. Overexcavations may be required at or near UST locations, or where unsuitable material is encountered.

Assuming compliance with the recommendations in our geotechnical report, we anticipate that static loading settlements would be ½ to 1 inch, with differential settlements between adjacent footings or over a 20-foot span of continuous footing equal to about half the total settlement.

6.5 Mat Foundations

Mat foundation will be used to support the core walls in the center of the building. We recommend using a bearing pressure of 8 ksf for a mat foundation bearing in dense to very dense sand or hard, clayey silt material. We do not recommend that the mat bear on existing or future fill material. In areas where overexcavation is required to reach the dense/hard native soils, the overexcavation should be backfilled with lean mix or structural concrete. Soil conditions were assumed to be uniform beneath the mat foundation and soil properties were estimated from borings B-11, B-19, and B-20.

In order to provide moduli of vertical subgrade reaction (k) in pounds per square inch per inch of deflection (pci) for the mat, we obtained static and dynamic loading information from the structural engineer, calculated estimated mat settlements, and then calculated the k values. The recommended k values are presented on Figure 6. The mat is being designed as relatively flexible and therefore would not settle the same amount over its entire area. Because the k values are calculated using deflection (settlement), they vary across the base of the mat area. The following paragraphs describe the assumptions made and calculations completed for the mat subgrade reaction.

We understand from Magnusson Klemencic Associates (MKA) that there are two proposed mat foundations. Both foundations would be 55 feet long by 55 feet wide by 6 feet thick. The Young's Modulus of the mat is 3.6 x 10^6 pounds per square inch (psi). One mat foundation is located in the southeast portion of the building (south of the E-line, east of line 4 according to the plan sheet, dated February 14, 2002); the top of the mat would be at approximately elevation 132 feet and the base would be at elevation 126 feet. The second mat foundation is located in the northwest portion of the building (south of the B-line, east of line 1 according to the plan sheet, dated February 14, 2002); the top of the mat would be at approximately elevation 145 feet and the base would be at elevation 139 feet. MKA indicated that the total static load on the mats, including dead load and live load from core walls and building columns and the weight of the mats themselves, would range from approximately 6,640 kips to 7250 kips. We analyzed the mat under static conditions with a bearing load of 2,500 pounds per square foot (psf). We also analyzed the mat with 8,000 psf bearing load to assess the potential impact from seismic loading conditions.

Following the method proposed by Brown (1969), we assumed a uniformly loaded circular mat 6 feet thick and with an area equal to that of a 55- by-55-foot equivalent square. This mat was also assumed to be on a subgrade material that extends down to a rigid base at or below a depth of 27.5 feet. A Young's modulus of the soil was estimated based on our borings and our experience in the Puget Sound area. The estimated deflection (settlement) at various points on the mat was then calculated. The settlement at the center and at the edge of the mat is estimated to be 0.25 and 0.1 inch, respectively under static conditions (2,500 psf bearing pressure). With a bearing pressure of 8,000 psf, the estimated settlements at the center of the mat are approximately ¾ to 1 inch and 0.3 to 0.5 inch at the edge. These deflections would occur essentially as the load is applied. This corresponds to an estimated differential settlement of approximately ½ inch from the center of the proposed mat. This does not represent the differential settlement between the mat/core area and the remainder of the building. Using the relative mat stiffness (which is based on the Young's Modulus of the soil and mat, the Poisson's ratio of the soil, the thickness of the mat, and the radius of the assumed circular mat) the ratio of the contact pressure on the soil to the uniform pressure on the mat was determined. Using the values of the contact pressure on the soil divided by the estimated settlements, k values were calculated. These calculations produced a higher k values at the edge than at the center of the shaft. For ease of design we have averaged the values and provided a simplified zonal pattern, which is presented on Figure 6.

6.6 Lateral Earth Pressures for Permanent Walls

Lateral earth pressures may act on buried portions of the building walls. For buried building walls that are allowed to move at least 0.001 times the wall height, we recommend that a static, active, lateral earth pressure be used. For buried building walls that are not allowed to move 0.001 times the wall height (a braced condition), static, at-rest, lateral earth pressures should be used. The equivalent fluid weights of native and imported fill for both the active and at-rest apparent earth pressure conditions are presented in the table below. All of the values in the table are based on the assumption that proper long-term drainage is provided, and that no buildup of hydrostatic pressure occurs.

The table below also presents active and at-rest equivalent fluid weights (triangular distributions); these values may be used for buried permanent walls on the site that do not have multiple braces (floors) but are single-braced or cantilevered, or where large distances occur between permanent wall supports.

Soil Type	Active Earth Pressure Equivalent Fluid Weight			At-Rest Earth Pressure Equivalent Fluid Weight		
	Static Case Triangular Distribution (pcf)	Static Apparent Earth Pressure	Seismic Increment (%)	Static Case Triangular Distribution (pcf)	Static Apparent Earth Pressure	Seismic Increment (%)
Native, glacially overridden soil	30	22H	35	50	32H	20
Compacted structural backfill	35	24H	30	55	36H	20

Notes:
H = wall height in feet
pcf = pounds per cubic foot

The total earth pressures should be analyzed for seismic loading conditions using a dynamic load increment equal to a percentage of the static, active or at-rest earth forces. The percentage increases for both the native and imported fill soil cases and for both the active and at-rest earth pressure conditions are presented in the table below. This percent load increment should be applied as a uniform load to the wall, with the resultant force acting at the midpoint of the wall height. A percentage load increase for seismic conditions is consistent with a pseudostatic analysis using the Mononobe-Okabe equation for lateral earth pressures and a horizontal seismic coefficient of 0.15g. The seismic coefficient is not necessarily equivalent to the site peak ground acceleration. The peak ground acceleration is experienced only a few times within the record of earthquake shaking, and the actual earthquake ground motion is cyclic in nature, not static. Values of the seismic coefficient are thus typically one-third to one-half the value of the peak ground acceleration that may be experienced at the site. These pressures assume drained conditions and a horizontal adjacent ground surface.

It may be found that the increase in lateral earth pressures during earthquake loading can be accommodated by the 33-percent capacity increase that is allowed in the strength of structural members by the IBC. Therefore, walls that are adequately designed for static loads may be capable of withstanding the combined effects of static and earthquake loading during the design earthquake. The structural engineer should consider whether or not the seismic earthquake increment is necessary or if the strength of the structural members accounts for the increase.

The equivalent fluid weights given above are based on the assumptions that the ground surface behind the wall is level and that proper drainage is installed to prevent water from building up behind the wall. For walls with a sloping backfill, add 1 pcf to the equivalent unit weights given above for each upward degree of backfill inclination. Recommendations for traffic and/or construction surcharge loads of 250 to 600 pounds per square foot (psf) are shown on Figure 7.

Lateral pressures due to adjacent footings are shown on Figure 9. The lateral pressures due to adjacent footings should be added to the recommended traffic and construction and lateral earth pressures.

6.7 Lateral Resistance for Permanent Walls

For structures founded on footings and mat foundations, lateral loads may be resisted by a combination of base friction and passive pressure against the footings/mats and buried portions of building walls. We recommend that the base sliding resistance be determined based on an allowable coefficient of friction of 0.45 for compacted structural fill and glacial till. We recommend an allowable passive pressure of 360 pcf and 260 pcf for native soil and compacted structural fill, respectively. These passive pressures are for soil above the groundwater table. Both the coefficient and passive pressure values above include factors-of-safety (FS) of 1.5.

6.8 Floor Slabs

In our opinion, floor slabs for the Administration Building could consist of slabs-on-grade. All fill placed under slabs-on-grade, including backfill for footing excavations, utilities, etc., should consist of properly compacted structural fill over dense/hard, native soil.

We estimate that a modulus of subgrade reaction equal to 250 pounds per cubic inch (pci) for densely compacted structural fill over properly prepared native soil and 300 pci for very dense/hard native soil could be used for design of slabs-on-grade. This recommendation assumes that proper drainage is provided beneath the floor slabs.

6.9 Temporary Excavation Shoring

6.9.1 Overview

In general, the proposed Administration Building is underlain by very dense/hard glacially overridden soils at relatively shallow depths except near the alleyway where loose fill was encountered. As shown on the attached generalized subsurface profiles, the water-bearing advance outwash unit is below the base elevation of the proposed Administration Building. However additional zones of water-bearing material may be encountered during construction. If shoring is required, either soldier pile/tieback or cantilever soldier pile shoring walls can be designed for temporary support. The existing basement walls could also be used as shoring with tiebacks connected to walers while new walls are constructed. Rakers supporting existing basement concrete walls may also be an alternative, depending on the alignment of new walls.

With every excavation in soil, both elastic and inelastic ground displacements will occur behind the earth support system as a result of the changes in stresses within the surrounding soil mass. The displacement magnitudes are dependent on the stress-deformation properties of the soil; design lateral earth pressures; the configuration, stages, and depth of excavation; wall stiffness; spacing of soldier piles and tiebacks (if used); groundwater conditions; and the care and skill with which the excavation work is accomplished. The following sections provide soldier pile and tieback design criteria.

6.9.2 Soldier Pile and Tieback Wall

General recommendations for soldier piles, lagging, and tiebacks are given in the following sections. Following the general recommendations are discussions of the temporary shoring design. Groundwater is anticipated to be below the proposed floor level; therefore, significant temporary dewatering measures would likely not be necessary.

6.9.2.1 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 Figures 7 and 8.

6.9.2.2 Lagging

We recommend that lagging be installed between soldier piles. Lagging should be installed as the excavation proceeds, and in general, not more than four 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, or locally with a weak control density fill (CDF).

Due to 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 Figures 7 and 8.

6.9.2.3 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 degrees from horizontal. The strands or bars are required to be in the center of the borehole so centralizers are spaced evenly along the length of the anchor prior to installation. The tieback is then grouted using either tremie methods or pressure grouting. 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 a series of test anchors. The following frictional values are only for planning and estimating anchor lengths.

Temporary tieback anchors installed by hollow-stem auger methods (that is, no pressure grouting) could be designed for an allowable frictional value of 1.5 kips per square foot (ksf) in dense to very dense, native granular (sand and gravel) soil. In hard silt/clay soil, we recommend an allowable frictional value of 1.0 ksf. Anchor grout should be placed by tremie method for open-hole anchor installations. Groundwater and caving soil conditions may prevent open-hole anchor installations. Allowable load transfer rates for a cased, pressure-grouted anchor of 4 kips per lineal foot (klf) and 2.0 klf are estimated for 3- to 6-inch-diameter boreholes in native, dense sand/gravel and hard silt/clay soils, respectively. A single-stage pressure-grouted anchor is defined as an anchor that undergoes high pressure grouting as the drill casing is removed. Otherwise, small-diameter, cased, tieback anchors, grouted using tremie methods, should be designed for 1.5 ksf or 1.0 ksf as described above. Later, post-grouting may result in a higher friction value. Where possible, we do not recommend that the anchor load zone be installed within the near-surface fill or within utility trench fill beneath the street. If the anchor load zone is installed within fill, we recommend using an allowable friction value of 0.5 ksf (with post-grouting). We recommend that tieback anchors be spaced a minimum of three diameters apart, measured center-to-center.

6.9.2.4 Temporary Shoring Wall

Recommended lateral earth pressures for temporary cantilever and single-row tieback walls are given on Figure 7. Figure 8 presents lateral earth pressures for a multiple-level tieback wall. Values shown on Figures 7 and 8 are for walls adjacent to a horizontal ground surface. If there is a cut slope behind the soldier pile wall, the earth pressures should be adjusted appropriately. We can provide cut slope surcharge pressures if the design team determines that cut slopes above soldier pile walls are necessary. Also included on Figure 7 are recommendations for traffic and/or construction surcharge loads of 250 to 600 pounds per square foot (psf).

The lateral earth pressures shown in Figures 7 and 8 are for active soil conditions. For active conditions lateral wall movements could range from 0.10 to 0.15 percent of the excavation depth. In general, settlements of the same order of magnitude could occur behind the wall for a distance of half the height of the excavation, decreasing linearly to zero at a distance of approximately 1.5 to 2 times the excavation height. The above-mentioned deflections and settlements are estimates only and are affected, in part, by the method and care used during excavation and shoring wall installation.

Note 12 on Figure 6 and Note 10 on Figure 8 include our recommendations for at-rest earth pressures should an alternative shoring system be used that limits deflections to less than 0.001 times the wall height.

6.9.2.5 Shoring Wall Drainage

Drainage recommendations for soldier pile shoring walls are presented in Figure 10. In our opinion, the care taken in construction of the weep drains at the base of the walls is critical. A positive connection with the drainage mat must be provided, and concrete contamination and plugging must be avoided. Close quality control during construction is very important.

6.10 Temporary Cut Slopes

If temporary open cut slopes are used on site, the "safe" temporary slope for excavations will depend on the following factors: (1) the amount of groundwater seepage, (2) the soils exposed in the excavation slope, (3) the depth of the excavation, (4) surcharge loads at the top of the excavation, (5) the geometry of the excavation, and (6) the time of construction. Construction slope values required for stability and safety depend on a careful evaluation of the above factors. Because of the many variables involved, cut slope inclinations can only be estimated prior to construction. For safe working conditions and prevention of ground loss, excavation slopes should be the responsibility of the contractor because he/she will be at the job site to observe and control the work. All current and applicable safety regulations regarding excavation slopes and shoring should be followed.

Excavations can be accomplished with conventional excavating equipment, such as a dozer, front-end loader, or backhoe. The glacially overridden material may be difficult to excavate. For planning purposes, we recommend that temporary, unsupported, open-cut slopes in the glacially overridden native soil be no steeper than 1 Horizontal to 1 Vertical (1H:1V), although localized steeper slopes may be possible in areas of stable soil. Where existing loose fill is encountered, we recommend that cut slopes be no steeper than 2H:1V. Flatter cut slopes may be required where loose soils or seepage zones are encountered during excavation. Exposed cut slopes may need to be protected with a waterproof covering during periods of wet weather to reduce sloughing and erosion.

The above recommendations are for temporary cut slopes in dry conditions. If wet conditions or uncontrolled groundwater flow is encountered, flatter slopes may be required. Based on our experience, in addition to the anticipated groundwater table, seeps and springs may be encountered, even in very dense, glacial till cut slopes. Care should be taken near the existing structures and facilities to make sure that the open cut does not undermine their integrity.

Also, all traffic and/or construction equipment loads should be set back from the edge of the cut slopes by a minimum of 2 feet. Excavated material, stockpiles of construction materials, and equipment should not be placed closer to the edge of any excavation than the depth of the excavation, unless the excavation is shored and such materials are accounted for as a surcharge load on the shoring system.

6.11 Excavation of Potentially Contaminated Soil

It is likely that petroleum contamination in soil and groundwater is present in the location of the proposed Administration Building. If soil encountered in the proposed excavation is affected by the contamination it should be handled and disposed of properly.

Shannon & Wilson, Inc. performed a Phase I Environmental Site Assessment in June 2002 for the proposed Administration Building site. The results of this study indicated that there have been, or currently are, as many as five underground or above ground storage tanks (USTs or ASTs) located on the property occupied by, or adjacent to, the existing garage. Approximate UST/AST locations are shown on Figure 2.

During utility installation contaminated soils were observed in a trench located at the northeast corner of the proposed Administration Building addition, near the entrance to the existing garage and in the vicinity of the two abandoned in-place gasoline USTs. According to Hos Brothers Excavating, and our observations, contaminated soil is present from approximately elevation 133 to 128 feet in the vicinity of the garage entrance. A soil sample was collected from the utility trench. Gasoline hydrocarbons were detected at 5,000 milligrams per kilogram; no benzene was detected.

Given the likely impacts to construction, we recommend that the prime contractor and excavation and shoring subcontractors be familiar with these site conditions (via meetings, plans, specifications, or other project documents) so they are prepared to address contamination in the field. This preparation includes using appropriately trained personnel; proper segregation, handling, and disposal of contaminated soil; and collection, possible treatment, and disposal of groundwater. Proper handling, screening, storing, testing, and disposal procedures should be included in the project specifications. We recommend that Shannon & Wilson prepare a Construction Contingency Plan to help field personnel be prepared for, identify, and properly handle contaminated soil and groundwater. The Construction Contingency Plan would also address proper equipment cleaning during and/or after work within the contaminated excavation zone.

We understand that where contaminated soil is encountered only the quantity required for construction of the addition will be removed. Therefore, we recommend that when excavation is occurring in the general areas of potential contamination, we be on site to field screen soil for the presence of contamination, to assist with soil segregation, and to document remaining conditions. Based on sampling and analyses during field investigations and Parking Garage sampling to date, contaminant levels in the soil do not exceed proposed cleanup levels at the footing elevations in the vicinity of the two 10,000-gallon USTs. However, the presence of small amounts of contamination can cause an odor. Our experience is that any soils with a detectable hydrocarbon odor will not be accepted as "clean fill." As a result, some excavated soils may need to be segregated for separate disposal even if they do not exceed cleanup levels. Field screening will assist in segregating "clean" soil from impacted soil for disposal purposes.

6.12 UST Removal

The current excavation plan includes removal of only what is required to accomplish construction, which may result in potentially contaminated soil an/or tanks remaining beneath the Administration Building addition. UST closure should be accomplished in accordance with Chapter 173-360 Washington Administrative Code (WAC) Underground Storage Tank Regulations. Closure may be performed in-place or by removal. Based on our experience with the Department of Ecology (Ecology), we recommend that any regulated USTs or ASTs discovered during demolition/excavation be removed and not closed in-place. By documenting removal of these tanks, it may be demonstrated to Ecology that the County made and effort to determine/remove potential contaminant sources, even if contamination remains on-site.

USTs/ASTs must be removed by a licensed UST contractor. Additionally, tank removal documentation (e.g., tank disposal, Site Assessment Check list, sampling results) will be required for submittal to Ecology.

6.13 Temporary Dewatering

The contractor should be responsible for the control of ground and surface water within the contract limits. In this regard, sloping, slope protection, ditching, sumps, dewatering, and other measures should direct water away from the structure to prevent ponding of water next to the facility.

We do not anticipate that the basement excavation would be below the groundwater table; however, perched groundwater may be encountered. Wet weather conditions may require the use of sumps or wells to control the surface and/or groundwater and allow for an accessible excavation.

6.14 Permanent Drainage

6.14.1 Backfilled Basement Walls

We recommend that free-draining, granular soils be used for any backfilled basement or buried walls. The free-draining backfill should be hydraulically connected to a subdrain system. Figure 10 presents typical recommendations for wall subdrainage and backfilling. This figure includes compaction criteria and gradation requirements for drainage materials.

6.14.2 Floor Slabs

The static groundwater table is expected to be below the proposed slab elevation; however, perched groundwater may be encountered. For long-term groundwater control we recommend the following.

We recommend a vapor barrier consisting of plastic sheeting and a clean granular material be placed under slab-on-grade floors. Figure 10 presents vapor barrier location recommendations. The plastic sheeting should be placed over a capillary break consisting of at least 6 inches of either washed pea gravel or properly compacted, 1.5-inch-minus crushed rock with less than 5 percent passing the No. 200 sieve, by weight, based on wet-sieving the minus ¾-inch fraction. The crushed rock would likely provide a firmer working surface for placing reinforcing steel.

A filter fabric or filter material may be required between the capillary break gravel and the native soil to reduce piping of fine-grained soil up into the gravel. We can determine if a filter is required once a proposed gravel gradation is provided, and once the floor slab subgrade has been exposed. Generally, a well-graded gravel would more likely be self-filtering than a poorly-graded (single size) gravel.

6.15 Fill Placement and Compaction

Construction of the Administration Building may require backfill to be placed around building walls and footing/mat excavations and possibly for sidewalks, retaining walls, and utilities. The on-site soils generally contain sufficient fines to make them moisture sensitive. In our opinion, the on-site soils may be difficult to place and compact to adequate relative compaction levels, especially during wet weather or in wet conditions. If the on-site soil becomes too difficult to compact or site space limitations prevent stockpiling, we recommend imported, granular structural backfill be used.

Imported, structural backfill should meet the gradation requirements of Section 9-03.14(1), Gravel Borrow, of the 2002 Washington State Department of Transportation (WSDOT) Standard Specifications. If fill is to be placed during periods of wet weather or under wet conditions, it should have the added requirement that the percentage of fines (material passing the No. 200 sieve based on wet-sieving the minus ¾-inch fraction) be limited to 5 percent. Any fines should be non-plastic.

Backfill should be placed in horizontal loose lifts not to exceed 4 inches for hand-operated compaction equipment and 8 inches for heavy compaction equipment. The fill should be compacted to at least 95 percent of its Modified Proctor maximum dry density as determined by American Society for Testing and Materials (ASTM) D 1557.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Footings

The recommended bearing pressures presented in this report require careful preparation of the footing subgrade. Footing excavations should be cleaned of all loose soil, leveled, and protected from water. We do not anticipate that the base of the footings will be below the groundwater level; however, if groundwater is encountered above the level of the proposed footings, temporary dewatering would be required to properly prepare footings. We recommend that temporary dewatering maintain the groundwater at least 2 feet below the level of the footing subgrade. The soils at the site contain sufficient fines to become soft and spongy when subjected to water and disturbance (from equipment or foot traffic). If construction is to take place during wet weather or under wet conditions, we recommend that the prepared footing subgrade be protected by placing a thin lean concrete "rat slab" immediately after excavation is completed.

Each footing subgrade on the project should be evaluated by a qualified geotechnical engineer to confirm suitable bearing conditions and to determine that all loose materials have been removed. The footing evaluation should be determined prior to placing the rat slab, if used.

If the footings are located on top of or near one of the USTs, the location of the footings will have to be moved or the UST removed and the subgrade excavated to very dense glacial soil. If contaminated soil is encountered it will have to be removed, segregated, and disposed of properly

7.2 Obstructions

Buildings currently and previously on site and their footings, floor slabs, basement walls, and USTs may still be partially or completely buried on site. Dealing with the existing foundations, walls, slabs, and USTs should be anticipated and may require special consideration during site excavation.

Although cobbles and boulders were not encountered in the explorations, it has been our experience that cobbles and boulders are commonly encountered in glacial soils. We recommend that contract specifications contain an advisory statement that cobbles, boulders, and other obstructions may be encountered in the mass excavation. The presence of these materials may require altering construction procedures. The contractor should be prepared to remove any cobbles, boulders, or other obstructions that protrude into the soil face of the excavation.

7.3 Wet Weather Earthwork

In the project area, wet weather generally begins about mid-October and continues through about May. The following recommendations apply for earthwork completed in wet weather or under wet conditions:

- The ground surface in the construction area should be sloped to promote the rapid runoff of precipitation and to prevent ponding of water.

- Fill material to be placed should consist of clean, granular soil of which not more than 5 percent by dry weight will pass the No. 200 sieve, based on wet-sieving the fraction passing the 3/4-inch sieve. The fines should be non-plastic.

- Earthwork should be accomplished in small sections to reduce exposure to wet weather. That is, the removal of unsuitable soil, and the placement and compaction of at least 12 inches of clean, imported fill, should be accomplished on the same day. The size of equipment may have to be limited to prevent soil disturbance. In some instances, it may be necessary to excavate soils with a backhoe or equivalent equipment outfitted with a flat plate on the bucket, to reduce subgrade disturbance caused by equipment traffic.

- No fill soil should be left uncompacted and exposed to water. A smooth-drum vibratory roller, or equivalent, should roll the fill surface to promote rapid runoff of surface water.

- Soils that become too wet for compaction should be removed and replaced with clean, imported structural fill material.

- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer or engineer's representative, experienced in wet weather earthwork, to determine that all work is being accomplished in accordance with the intent of the specifications.

The above recommendations for wet weather earthwork should be incorporated into the contract specifications.

7.4 Plans and Specifications Review and Construction Observation

We recommend that Shannon & Wilson be retained to review those portions of the plans and specifications that pertain to the items discussed in this report to determine if they are consistent with our recommendations. We are available to provide specification sections to address handling, screening, storing, testing, and disposal procedures of potentially contaminated soil and groundwater. We also recommend we be retained to observe the geotechnical and environmental aspects of construction. This observation 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.

7.5 Reporting

Because contaminated soil will likely remain in-place it is important to demonstrate that the County has made a good faith effort to remove known sources and document any cleanup action performed at the site. Therefore, following removal of contaminated soil, a report containing sample locations and analytical results should be prepared and submitted to the Department of Ecology. Further, regulated USTs/ASTs should also be removed and this action documented. Producing these reports will provide information for your records and future interaction with Ecology and likely result in reducing the County's future interaction with Ecology.

8.0 LIMITATIONS

The analyses, conclusions, and recommendations contained in this addendum report are based on site conditions as they presently exist and further assume that the field explorations are representative of the subsurface conditions at the site, that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

Within the limitation of scope, schedule, and budget, the conclusions and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical and environmental engineering principles and practices in the area at the time this report was prepared. We make no other warranty, either expressed or implied. The analyses, conclusions, and recommendations contained in this report are based on our understanding of the project and site conditions as described in the report. If, during construction, subsurface conditions different from those encountered in the field explorations are observed or appear to be present during excavations, we should be advised at once so 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, we recommend that this report be reviewed to determine the applicability of our conclusions and recommendations considering the changed conditions and time lapse.

We should be retained to review those portions of the plans and specifications that pertain to site preparation, earthwork, temporary shoring, temporary dewatering, footings, mat foundations, and permanent drainage installation to determine if they are consistent with our recommendations. In addition, we should also be retained to monitor these tasks during construction.

This report was prepared for the exclusive use of NBBJ, Snohomish County, and members of the design team for the proposed Administration Building. It should be made available to prospective contractors 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.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples or making test borings. 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.

We have prepared Appendix B, "Important Information About Your Geotechnical/ Environmental Report" to assist you in understanding the use and limitations of this report.

SHANNON & WILSON, INC.

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5/23/03

EXPIRES 10/08/03

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WLB:ACT:CLBM:SWG:TMG/wlb

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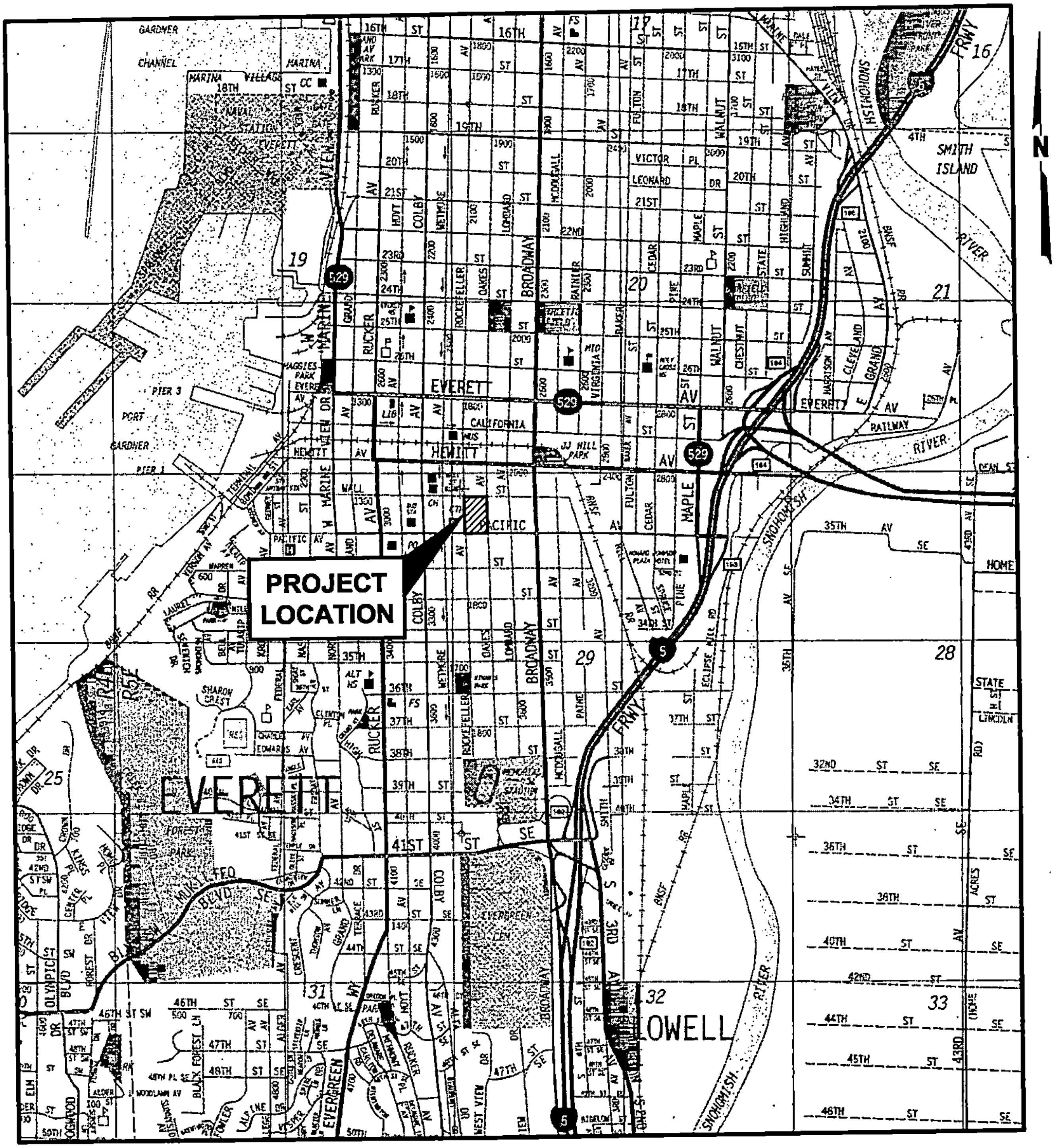

0 1/4 1/2 1

Scale in Miles

NOTE

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VICINITY MAP

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FIG. 1

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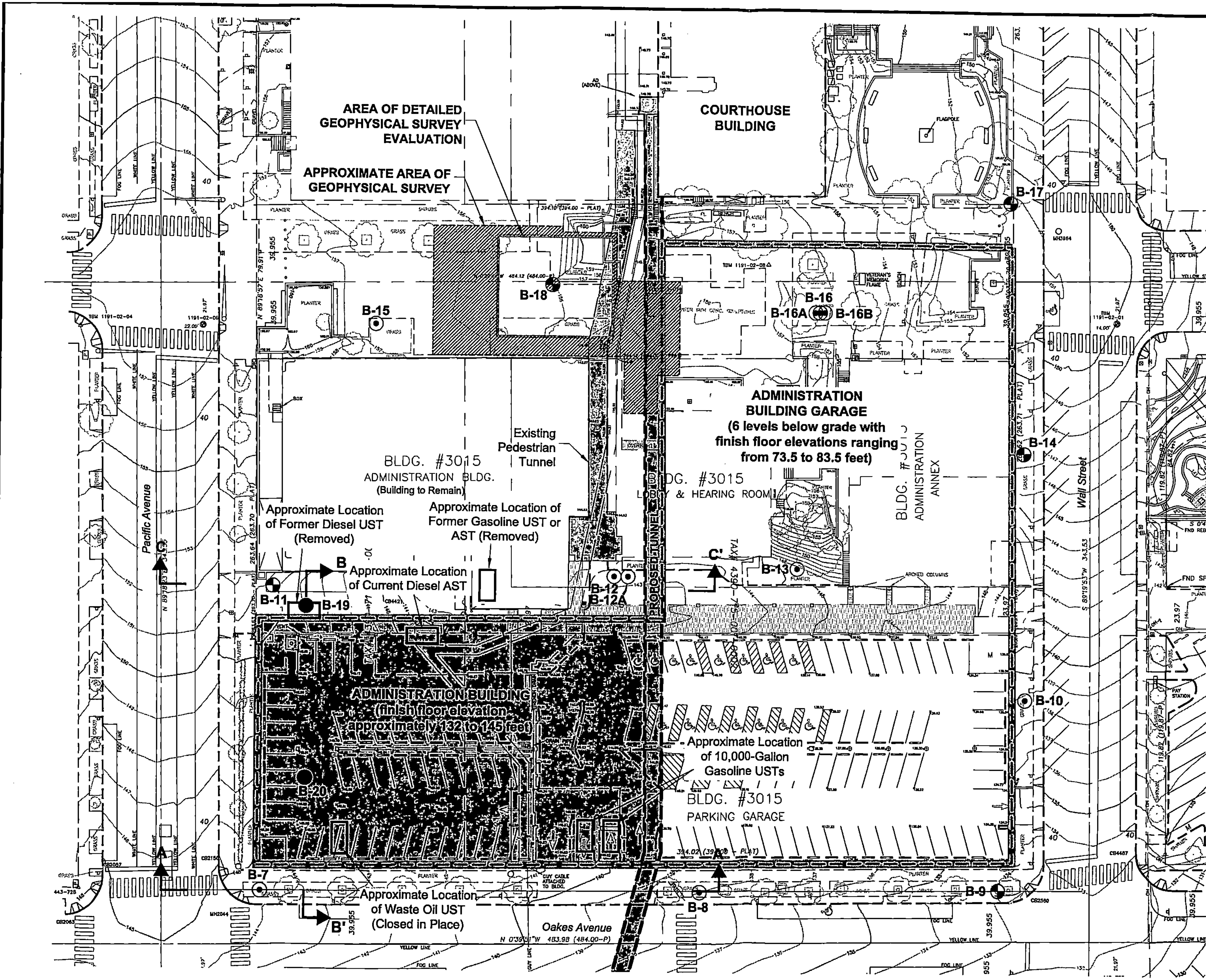

LEGEND

Approximate Extent of Proposed Excavation

B-19 Current Boring Designation and Approximate Location

B-7 Previous Boring Designation and Approximate Location

B-9 Previous Boring Designation and Approximate Location, Completed as a Monitoring Well

A Generalized Subsurface Profile Designation and Approximate Location

NOTES

1. This figure adapted from site topographic survey by Perteet Engineering, Inc., dated 6-2001. Elevation datum NAVD 88.
2. Boring locations are based on tape measurements from existing site features and should be considered approximate.
3. Contamination has been encountered in Oakes Avenue near the two 10,000-gallon gasoline USTs. Detected contamination concentrations in this area were:
   Gasoline: 19-5,000 mg/Kg
   Toluene: 140 mg/Kg
   Ethylbenzene: 95 mg/Kg
   Xylene: 560 mg/Kg
   Benzene was not detected in this area.

0 50 100

Scale in Feet

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SITE AND EXPLORATION PLAN

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FIG. 2

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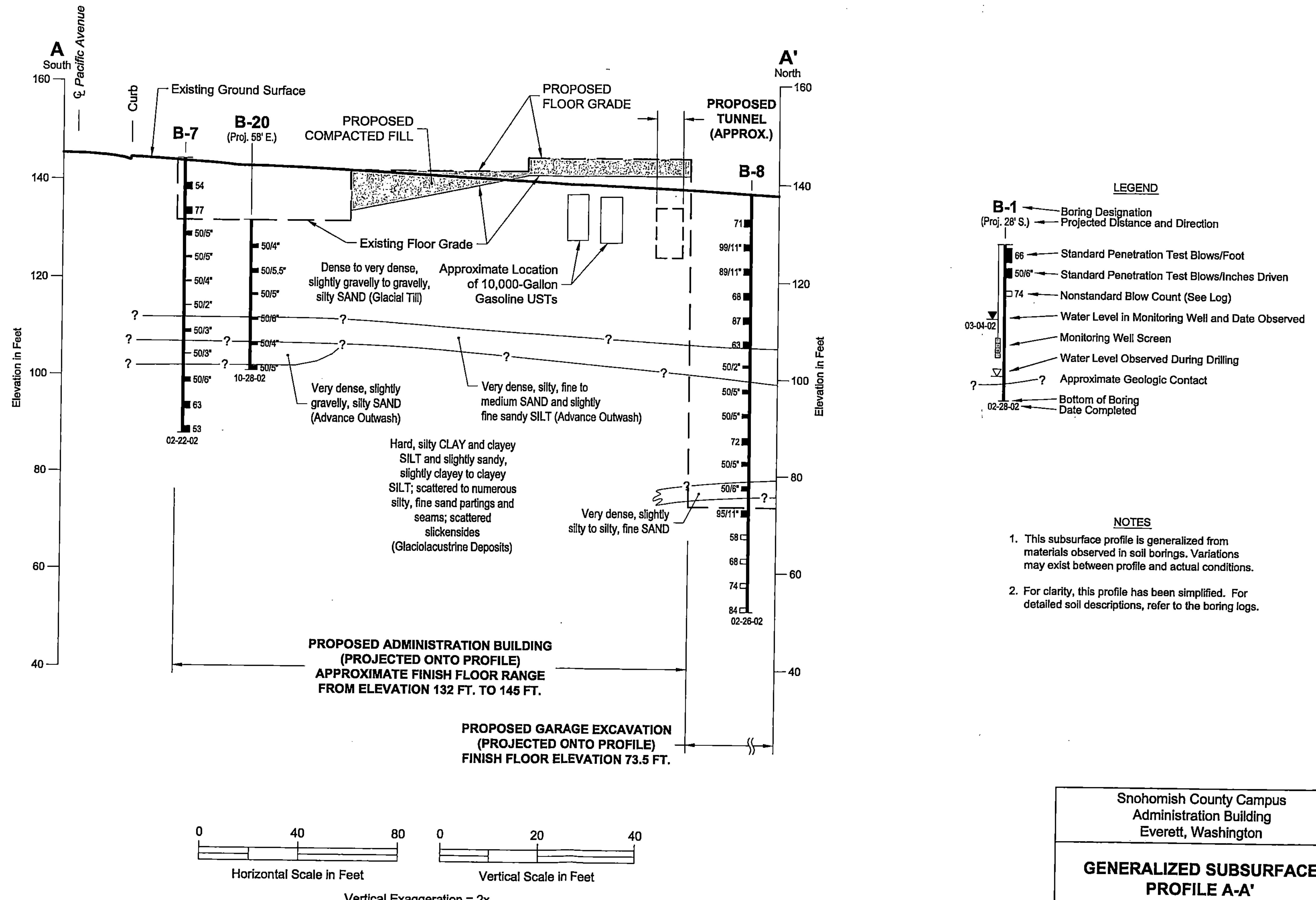

NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. For clarity, this profile has been simplified. For detailed soil descriptions, refer to the boring logs.

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GENERALIZED SUBSURFACE PROFILE A-A'

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FIG. 3

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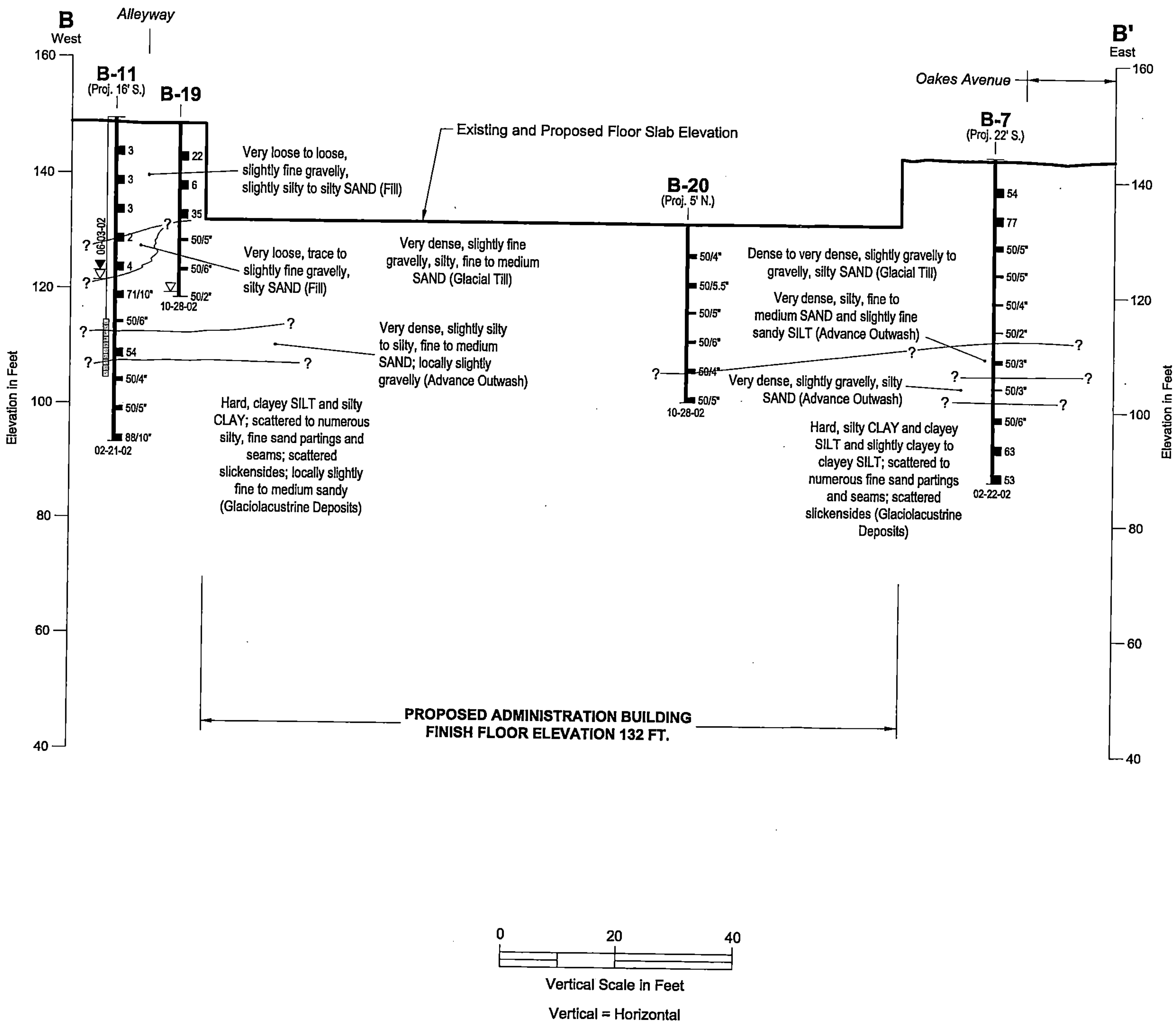

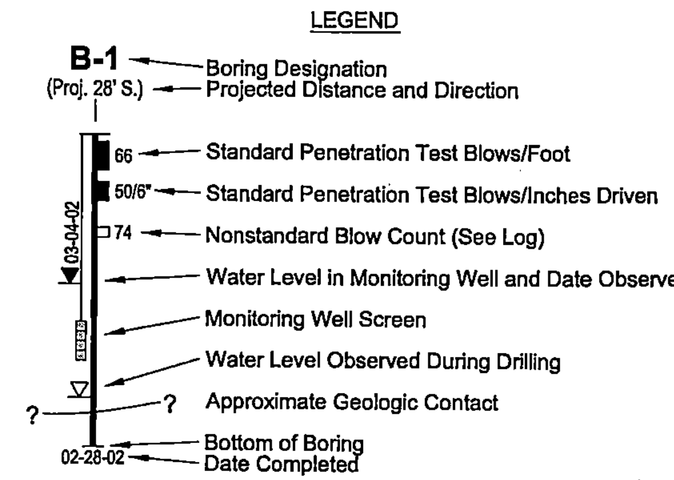

NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. For clarity, this profile has been simplified. For detailed soil descriptions, refer to the boring logs.

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GENERALIZED SUBSURFACE PROFILE B-B'

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FIG. 4

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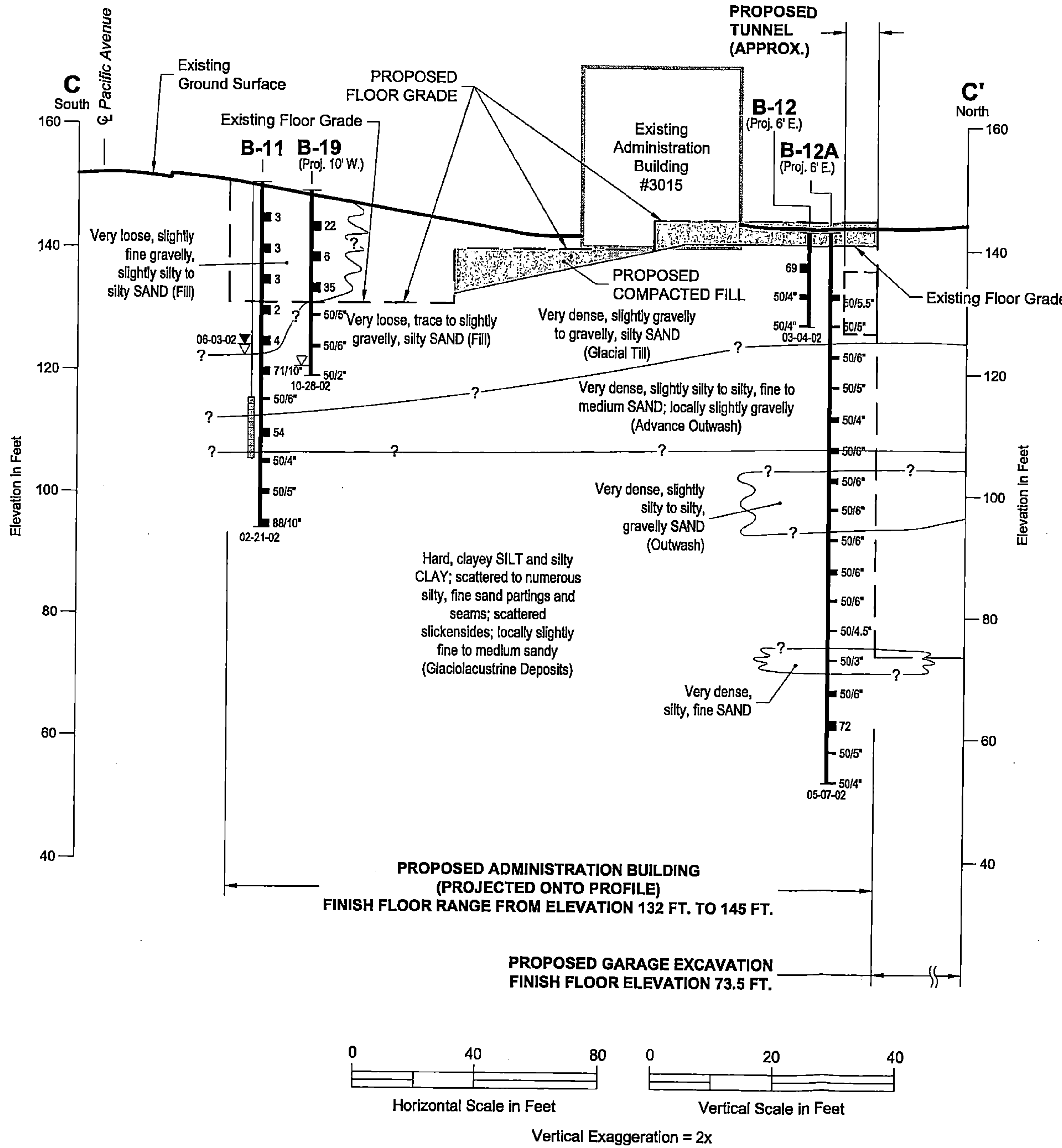

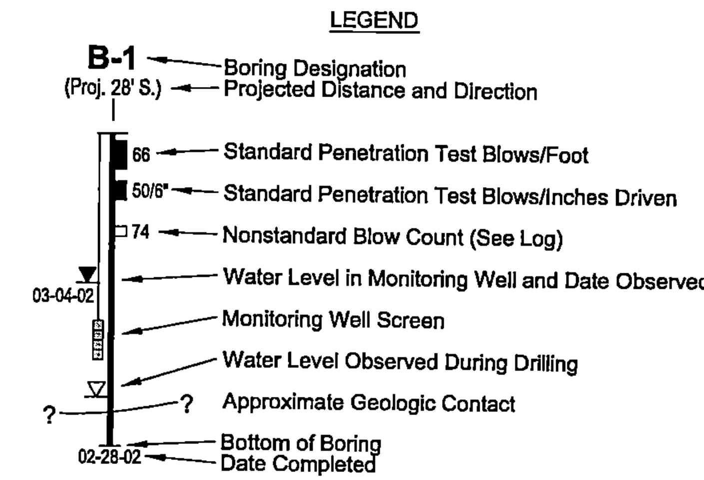

NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

2. For clarity, this profile has been simplified. For detailed soil descriptions, refer to the boring logs.

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GENERALIZED SUBSURFACE PROFILE C-C'

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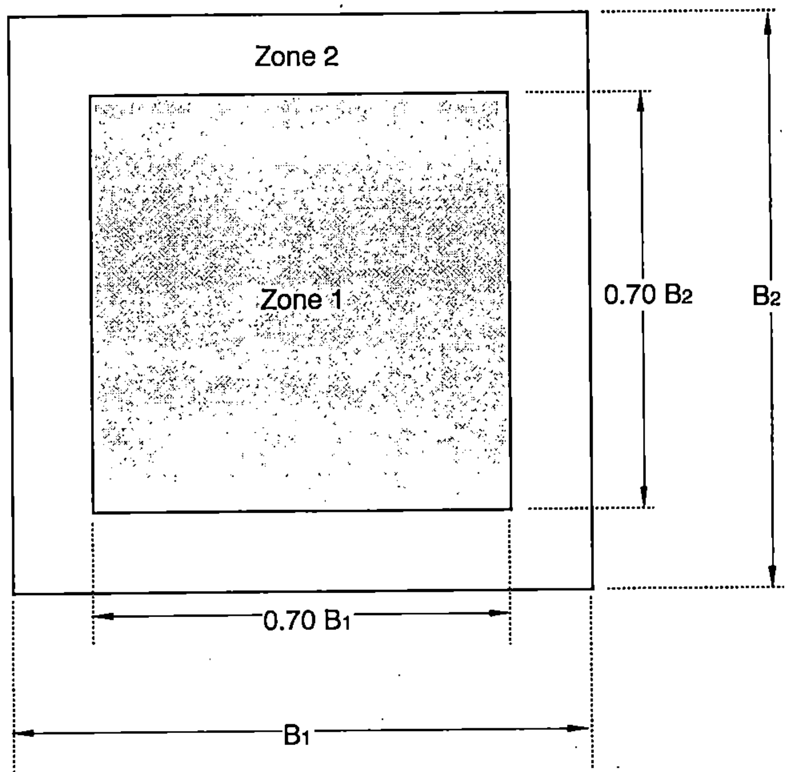
FIG. 5



Zone	Modulus of Vertical Subgrade Reaction, k (psi per inch)
1	100
2	180

MAT ASSUMPTIONS

B_1 = 55 feet
B_2 = 55 feet

Mat Thickness = 6 Feet
$E_{mat} = 3.6 \times 10^6$ psi

NOTE

Please refer to report text for a discussion of the calculation method used and the results.

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RECOMMENDED MODULI OF VERTICAL SUBGRADE REACTION FOR MAT FOUNDATION

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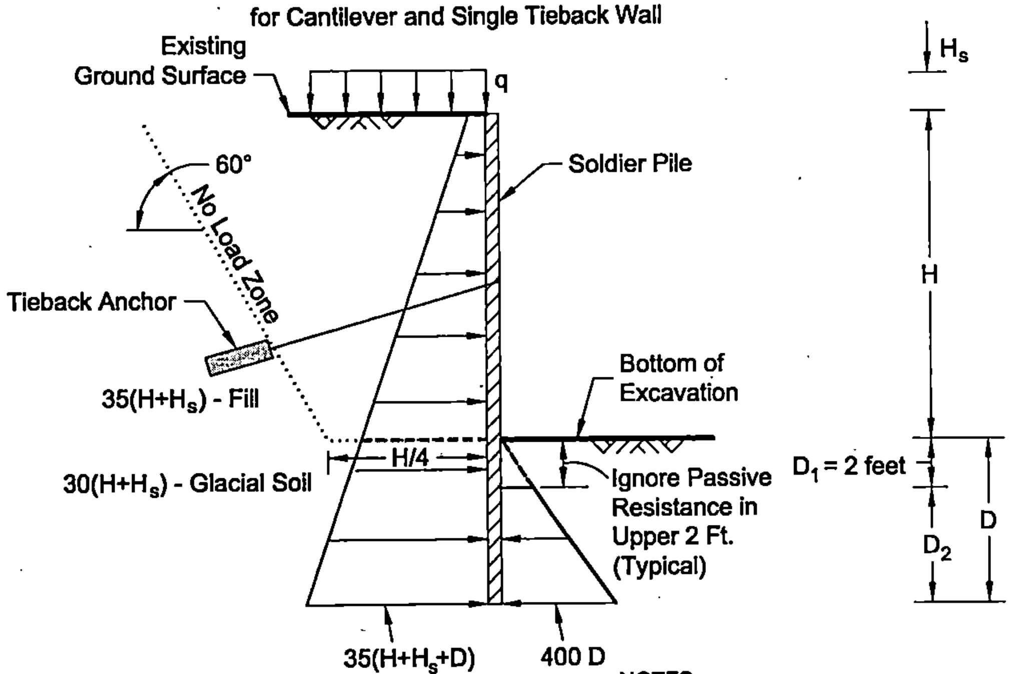
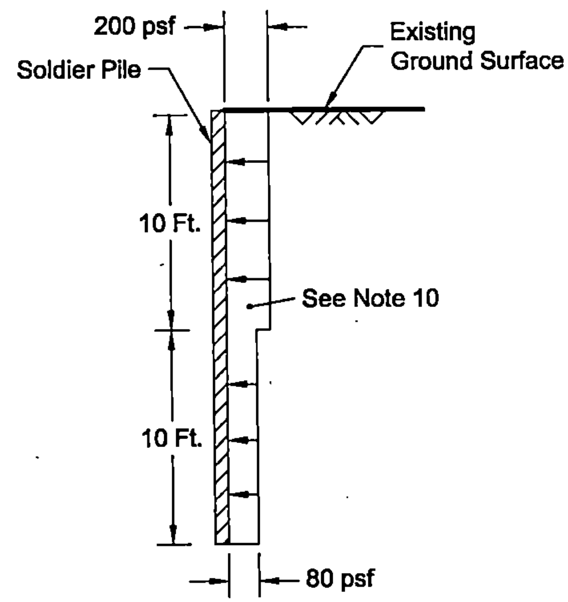
FIG. 6

I:/Drafting/Mac/21-1/21-1-09644-005 Fig. 6

Sketch A. Recommended Earth Pressures for Cantilever and Single Tieback Wall



Sketch B. Heavy Traffic & Construction Equipment Surcharge



NOTES

1. All Earth Pressures are in units of pounds per square foot.
2. Wall Embedment (D) should consider kickout resistance. Embedment should be determined by satisfying horizontal static equilibrium about the bottom of the pile. Minimum recommended embedment is 8 feet.
3. If loadings greater than 240 psf are anticipated, use Sketch B above for construction materials, crane outriggers, equipment, etc. Use 120 pcf to convert from equivalent uniform surcharge to equivalent height of soil.
4. Total design pressure should be the sum of the above active earth pressures and the traffic/construction surcharge. If cut slopes are made above the soldier pile walls, a slope surcharge should be added to the total design pressure.
5. 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 resistances over twice the width of the piles or the spacing of the piles, whichever is smaller.
6. Free drainage assumed behind the wall.
7. Use 80% of the design pressures for computing moment in piles.
8. For temporary lagging design, use 30 % of the design pressures.
9. Allowable vertical pile capacity:
   Temporary Skin Friction = 1 ksf
   Temporary End Bearing = 8 ksf
10. Lateral pressures in Sketch B are based on an assumed traffic surface surcharge of 600 psf acting over a limited influence area. More severe construction equipment loading requires special analysis.
11. If a sloping ground surface exists, the earth pressures should be adjusted.
12. If an alternative cantilever/single-brace system were used that limited deflections and thereby required at-rest earth pressures to complete the design, we recommend using 55(H+H$_s$+D) in Sketch A.

LEGEND

H = Wall Height (Ft.)

H$_s$ = Equivalent Soil Height of Surcharge Loading (Ft.)

D, D$_1$, D$_2$ = Embedment Depths (Ft.)

q = Uniform Vertical Load

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CANTILEVER AND SINGLE TIEBACK SOLDIER PILE WALL DESIGN CRITERIA

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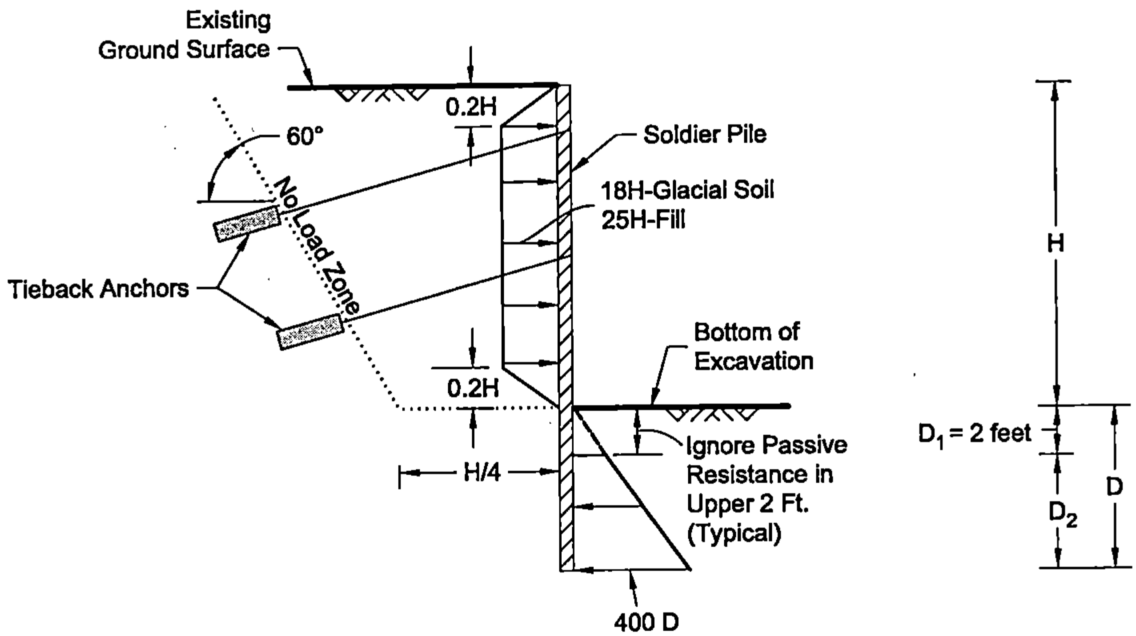
FIG. 7

FIG. 7



NOTES

1. All Earth Pressures are in units of pounds per square foot. The active earth pressure diagram applies to a multiple-level tieback wall.
2. Wall Embedment (D) should consider kickout resistance. Embedment should be determined by satisfying horizontal static equilibrium about the bottom of the pile. Minimum recommended embedment is 8 feet.
3. Lateral pressures for traffic surface surcharges are given on Figure 6.
4. If a sloping grounds surface exists, the earth pressures should be adjusted.
5. 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 resistances over twice the width of the piles or the spacing of the piles, whichever is smaller.
6. Free drainage assumed behind the wall.
7. Use 80% of the design pressures for computing moment in piles.
8. For temporary lagging design, use 30 % of the design pressures.
9. Allowable vertical pile capacity:
   Temporary Skin Friction = 1 ksf
   Temporary End Bearing = 8 ksf
10. If an alternative multiple-brace system were used that limited deflections and thereby required at-rest earth pressures to complete the design, we recommend using 36H in the figure.

LEGEND

H = Wall Height (Ft.)

D, D_1, D_2 = Embedment Depths (Ft.)

18H = Apparent Earth Pressure for Native Glacial Soil

25H = Apparent Earth Pressure for Fill

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MULTIPLE-LEVEL TIEBACK SOLDIER PILE WALL DESIGN CRITERIA

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FIG. 8

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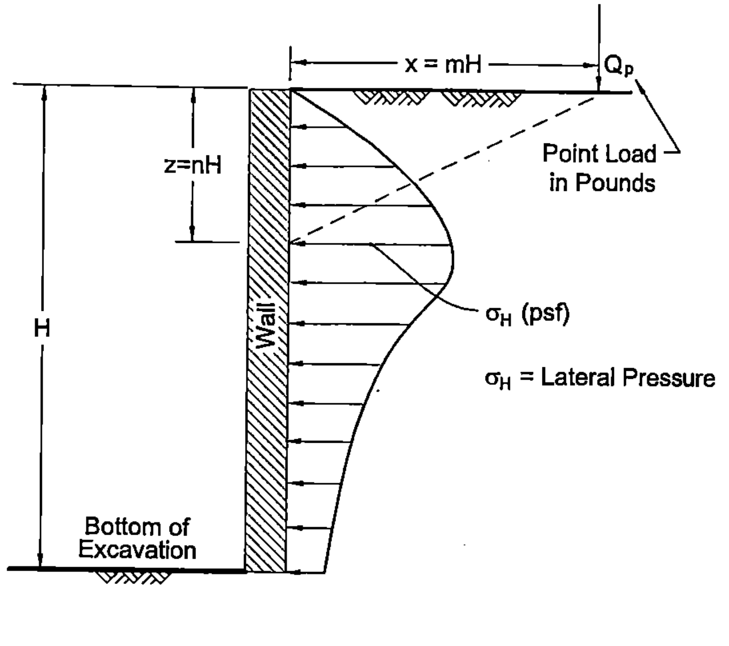
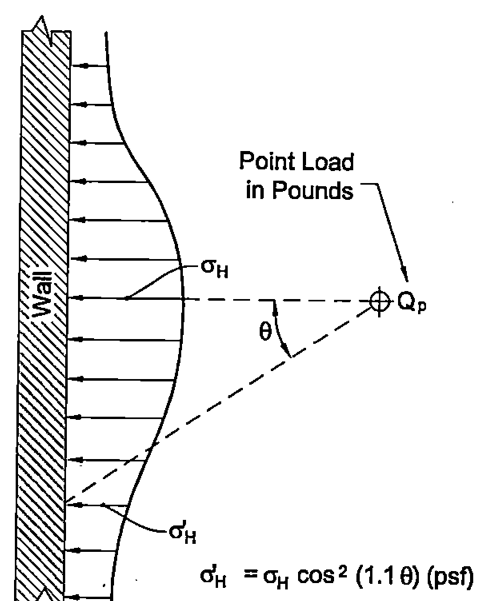


ELEVATION VIEW

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

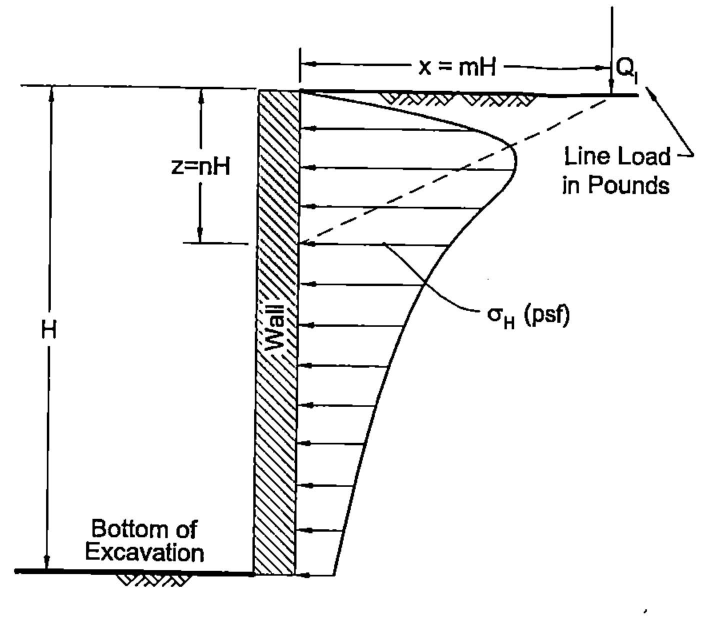


$\sigma'_H = \sigma_H \cos^2(1.1\,\theta)$ (psf)

PLAN VIEW

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

(NAVFAC DM 7.2, 1986)



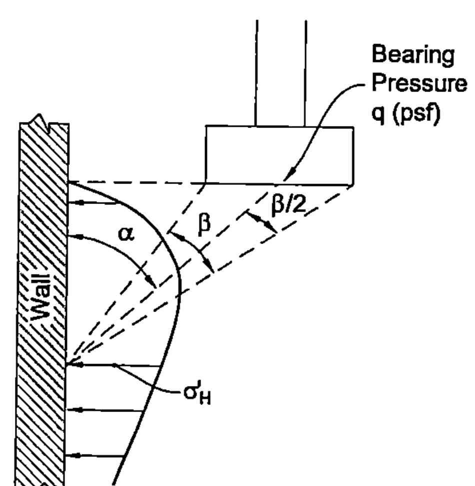
ELEVATION VIEW

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

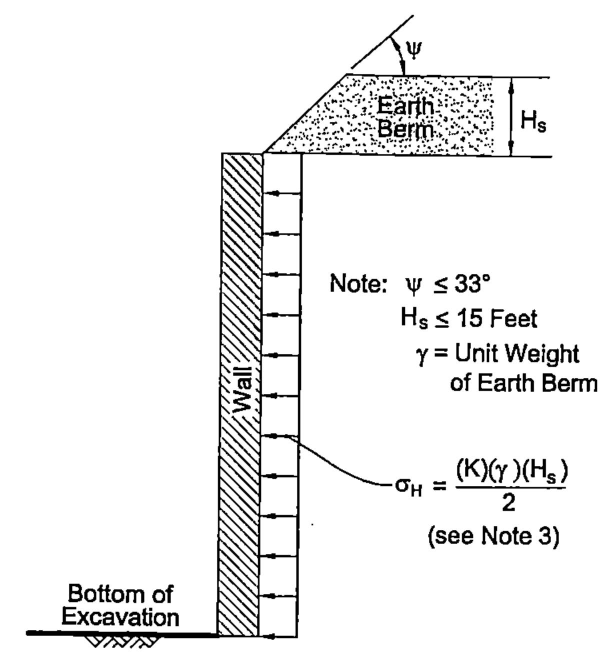


in radians

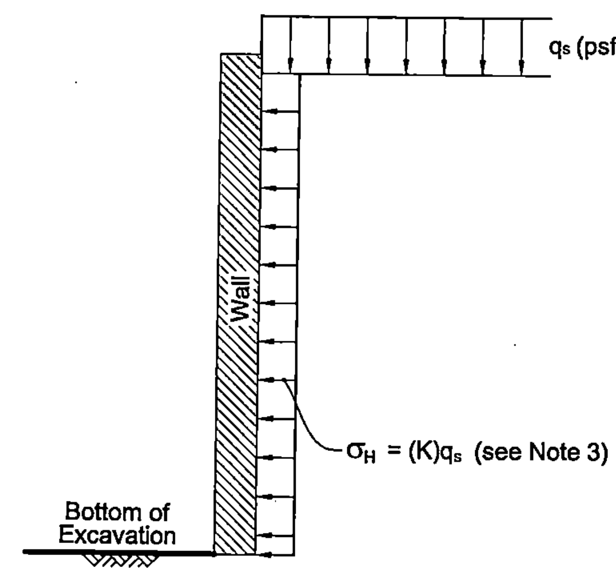
$\sigma_H = \frac{2q}{\pi}(\beta - \sin\beta \cos 2\alpha)$ (psf)

C) LATERAL PRESSURE DUE TO STRIP LOAD

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



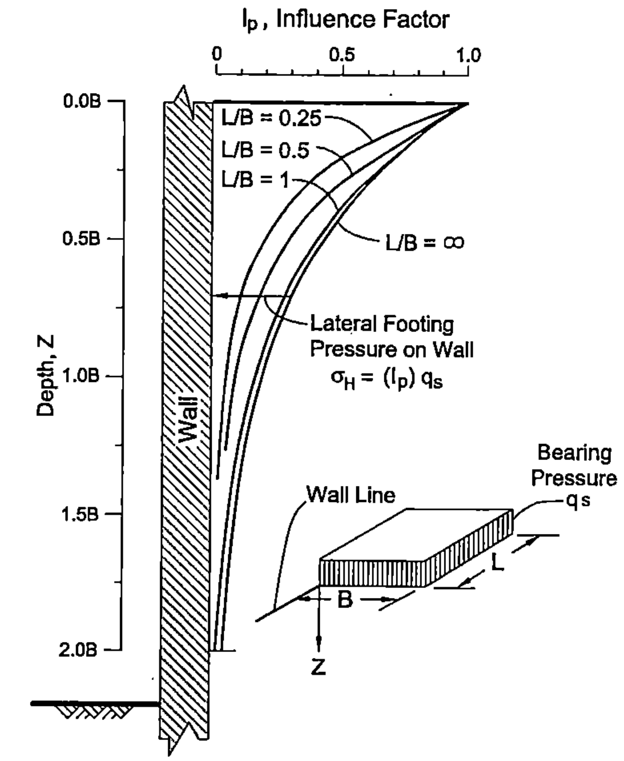
EARTH BERM



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. 9

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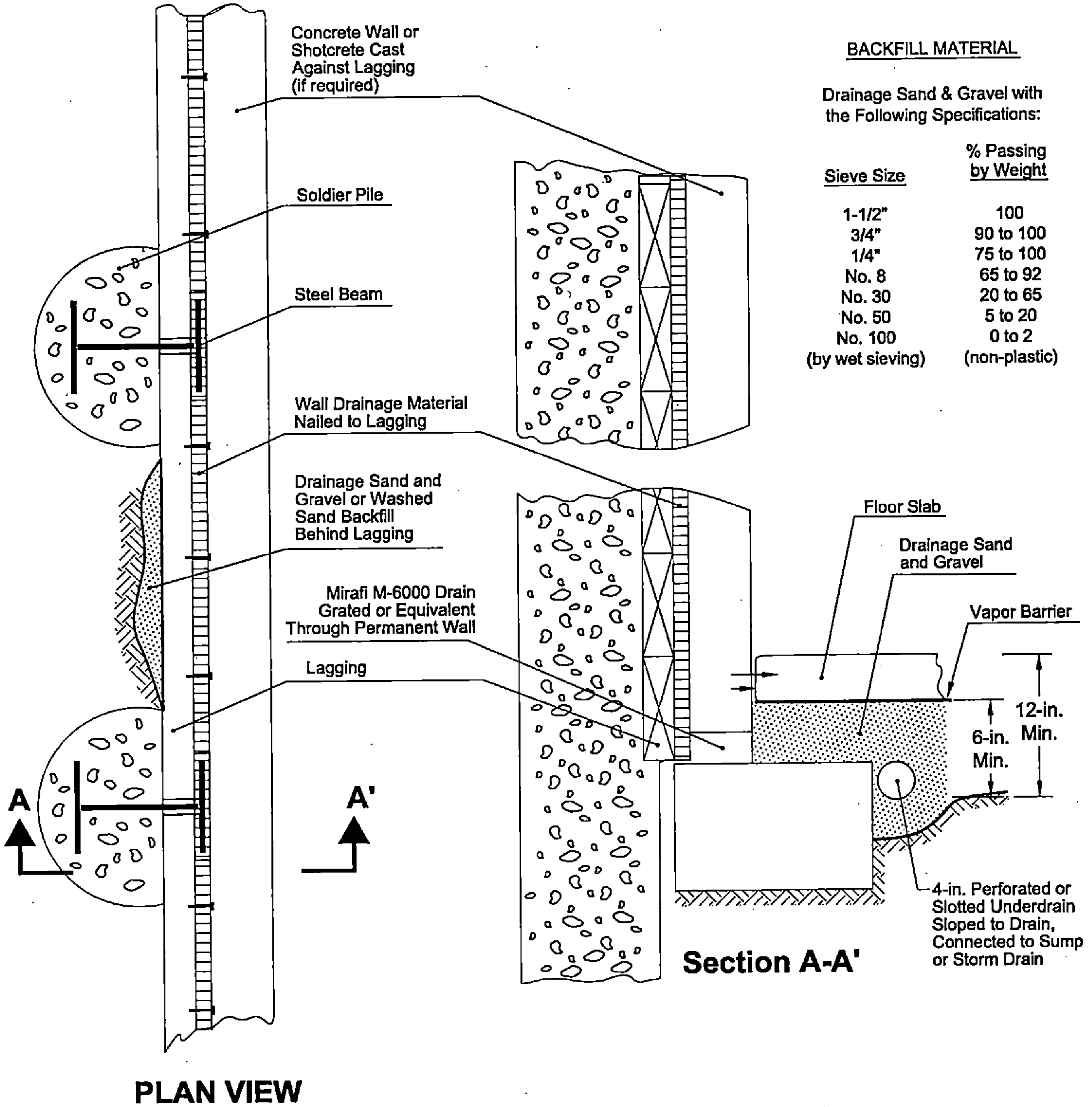

BACKFILL MATERIAL

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 (by wet sieving)	0 to 2 (non-plastic)

Not to Scale

NOTES

1. Wall drainage material: Miradrain 6000 or equivalent.
2. Clean-outs should be provided in the underdrain system.
3. Slotted pipe to have 1/8-inch maximum width slots.
4. Weep holes should be provided through lagging if concrete or steel panels are used.

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SHORING WALL AND FLOOR SLAB DRAINAGE RECOMMENDATIONS

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FIG. 10

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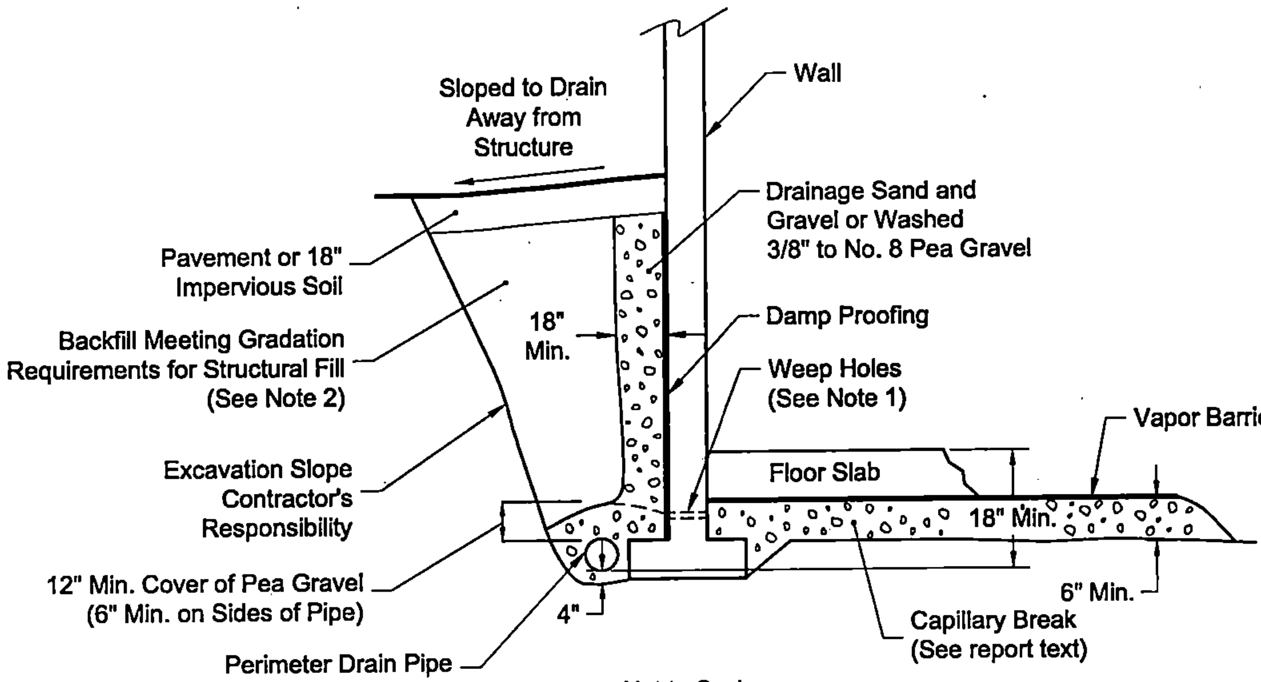

Not to Scale

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 (by wet sieving)	0 to 2 (non-plastic)

NOTES

1. Capillary break beneath floor slab should be hydraulically connected to perimeter drain pipe. Use of 1-inch diameter weep holes as shown is one applicable method.

2. Structural fill should meet WSDOT Gravel Borrow Specification 9-03.14(1) but should have a maximum size of 3 inches, and should not have more than 5% fines (by weight based on minus 3/4" portion) passing No. 200 sieve (by wet sieving) with no plastic fines during wet conditions or wet weather.

3. Backfill within 18" of wall should be compacted with hand-operated equipment. Heavy equipment should not be used for backfill, as such equipment operated near the wall could increase lateral earth pressures and possibly damage the wall.

4. All backfill should be placed in layers not exceeding 4" loose thickness for light equipment and 8" for heavy equipment and densely compacted. Beneath paved or sidewalk areas, compact to at least 95% Modified Proctor maximum dry density (ASTM: D1557, Method C or D). Otherwise compact to 90% minimum.

PERIMETER DRAIN PIPE

4" minimum diameter perforated or slotted pipe; tight joints; sloped to drain (6"/100' min. slope); provide clean-outs.

Perforated pipe holes (3/16" to 3/8" dia.) to be in lower half of the pipe with lower quarter segment unperforated for water flow.

Slotted pipe to have 1/8" maximum width slots.

Snohomish County Campus
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Everett, Washington

TYPICAL BACKFILLED WALL PERIMETER DRAIN AND BACKFILL

May 2003 | 21-1-09644-005

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FIG. 11

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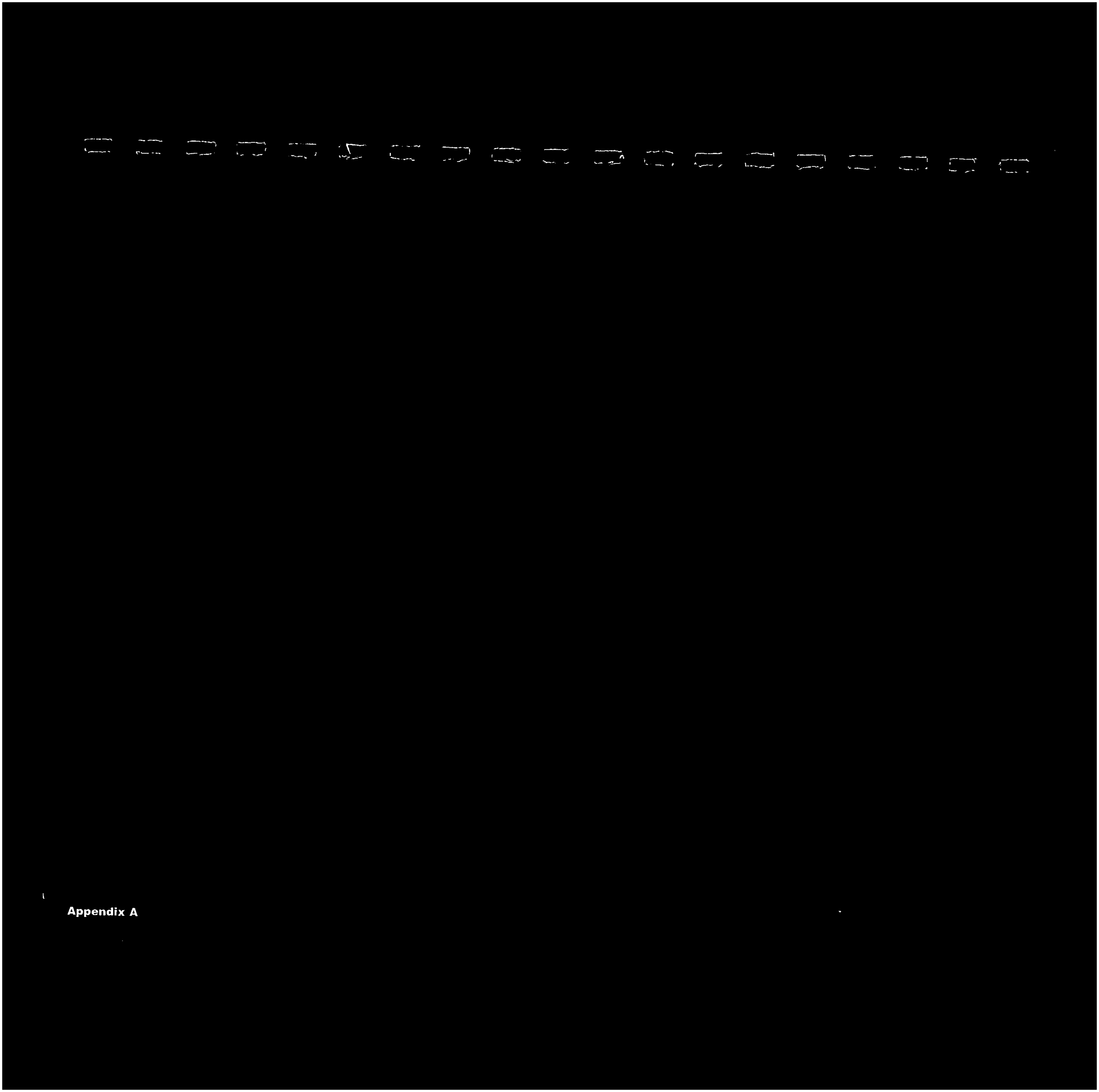
Appendix A


Appendix A

APPENDIX A

FIELD EXPLORATIONS

APPENDIX A

ADDITIONAL FIELD EXPLORATIONS

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

ADDITIONAL FIELD EXPLORATIONS

A.1 GENERAL

The additional field exploration program for the Snohomish County Campus Administration Building consisted of drilling and sampling two borings designated B-19 and B-20. The approximate exploration locations are shown on the Site and Exploration Plan (Figure 2) in the main text of the report. The locations of our borings were determined by measuring from site features. The elevations of the borings were determined by plotting the boring locations on the site topographic survey. All the boring locations and elevations should be considered accurate to the degree implied by the method used. The logs of our previous borings are included in our August 2002 report entitled, "Snohomish County Campus, Administration Building and Garage."

A representative from Shannon & Wilson, Inc. was present throughout the field exploration period to observe the drilling and sampling operations, retrieve representative soil samples, and to prepare descriptive field logs of the explorations. Soils were classified in general accordance with the American Society for Testing and Materials (ASTM) Designation: D 2488 Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). The current exploration logs presented in Figures A-2 and A-3 represent our interpretation of the contents of the field logs. Figure A-1 presents a key to our classification of the materials encountered.

A.2 BORINGS

The two borings were advanced at selected locations in and near the existing garage where access was available. Both borings were drilled with a track-mounted drill rig. The borings were advanced to depths of approximately 30 feet.

A.2.1 Drilling

Two borings (B-19 and B-20) were completed by Holt Drilling, of Puyallup, Washington, under subcontract to Shannon & Wilson, Inc., on October 28, 2002, using a track-mounted CME-45C drill rig. Drilling was accomplished using hollow-stem auger drilling techniques. These hollow-stem auger borings were drilled using a 3.25-inch or 4.25-inch inside-diameter continuous flight auger. Samples were retrieved from within the hollow-stem. After completion of drilling and sampling, the borings were sealed with bentonite grout and chips. No observation wells were installed.

A.2.2 Soil Testing and Sampling

Disturbed samples were obtained in conjunction with the Standard Penetration Test (SPT). SPTs were performed in general accordance with ASTM Designation D 1586, Standard Method for Penetration Testing and Split-Barrel Sampling of Soils. SPTs were generally performed at 5-foot intervals starting at 5 feet below ground surface. The SPT consists of driving a 2-inch outside-diameter, 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 for the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). This value is an empirical parameter that provides a means for evaluating the relative density, or compactness, of granular soils and the consistency, or stiffness, of cohesive soils. These values are plotted at the appropriate depths on the boring logs included in this appendix. Generally, whenever 50 or more blows were required to cause 6 inches or less of penetration, the test was terminated, and the number of blows and the corresponding penetration was recorded. The N-values are plotted on the boring logs presented on Figures A-2 and A-3.

A.2.3 Groundwater Observations

Where observed, groundwater was noted during drilling. No groundwater was encountered in boring B-20. Groundwater was observed in boring B-19 approximately 29 feet below the existing ground surface.

A.3 FIELD SCREENING FOR CONTAMINATION

Selected soil samples were retrieved and field screened for the potential presence of contamination. Field screening methods included photoionization detector (PID) measurements, visual observations, and olfactory observations.

A.3.1 PID Measurements

PID measurements were made to screen for volatile organic vapors such as gasoline and solvents. PID measurements were obtained by passing the instrument directly over the soil sample or by performing a headspace measurement. Readings of 2 parts per million (ppm) or more above background were considered suspect; however, no readings greater than zero were measured.

A.3.2 Visual Observations

Visual observations (such as sheen, or gray or black discoloration) of soil samples and groundwater were recorded on the boring logs.

A.3.3 Olfactory Observations

Olfactory observations were recorded on the boring logs when noted. Soil was not intentionally smelled for contamination.

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 40 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 (0.8 mm)
SAND* - Fine	#200 to #40 (0.8 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL* - Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	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

May 2003 21-1-09644-005

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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

May 2003 21-1-09644-005

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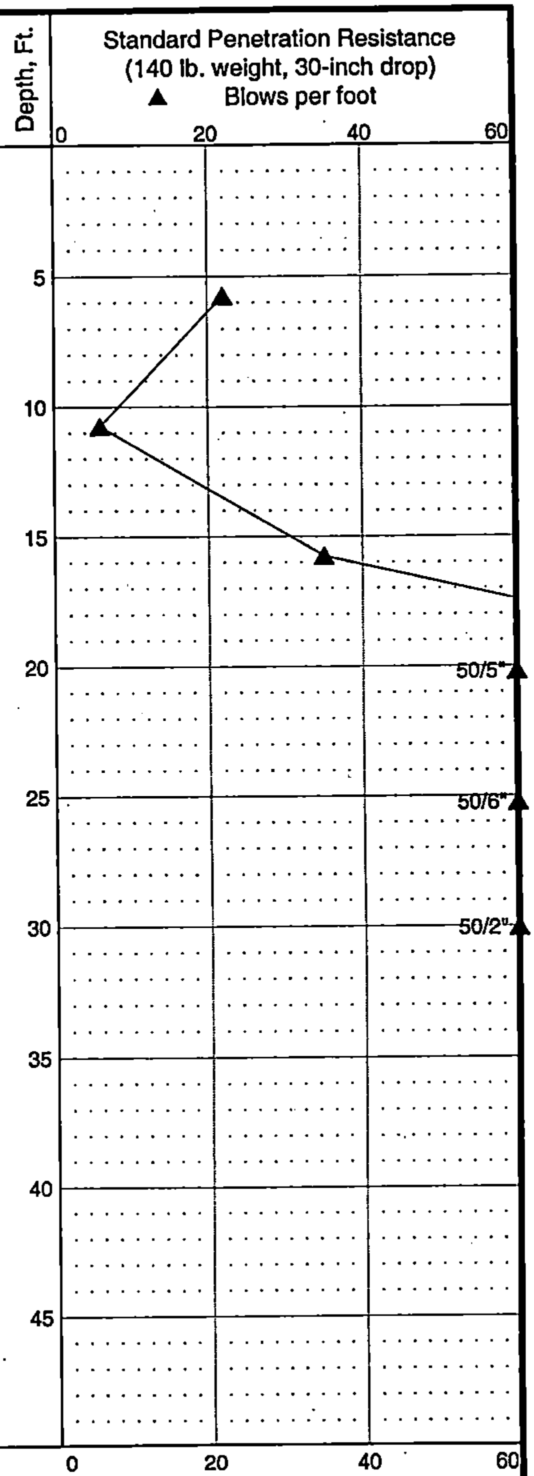
FIG. A-1
Sheet 2 of 2

BORING CLASS2 21-09644.GPJ SWNEW.GDT 5/14/03

SOIL DESCRIPTION Surface Elev.: Approx. 150 Ft. Datum:	Depth, Ft.	Symbol	Samples	Ground Water
Asphalt.	0.5			
Loose to dense, light brown, slightly gravelly, slightly silty to silty SAND; moist; (Fill) SM/SP-SM.			1, 2, 3	
Very dense, gray-brown, slightly fine gravelly, silty, fine to medium SAND; moist to wet; (Glacial Till) SM.	17.0		4, 5, 6	During Drilling
BOTTOM OF BORING COMPLETED 10/28/2002	30.1			

Note: PID measurements were made on all samples; however, no readings were greater than 0 parts per million.



LEGEND

* Sample Not Recovered

Ⅰ Standard Penetration Test

⊻ Ground Water Level ATD

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 the subsurface maerials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of symbols, codes, and definitions.
5. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING B-19

May 2003 21-1-09644-005

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FIG. A-2

MASTER_LOG_FINE 21-09644.GPJ SHAN_WIL.GDT 5/14/03 Log: WRL Rev: ACT Typ: LKD

SOIL DESCRIPTION Surface Elev.: Approx. 150 Ft. Datum:	Depth, Ft.	Symbol	Samples	Ground Water	Standard Penetration Resistance (140 lb. weight, 30-inch drop) ▲ Blows per foot
Concrete.	0.5				
Hard, light brown, slightly clayey, sandy SILT, trace of fine gravel interbedded with layers of silty, fine to medium SAND; moist; (Weathered Till) ML/SM.	6.0		1		5 ft: 50/4"
Very dense, brown-gray, silty SAND, trace of gravel; moist to wet; (Glacial Till) SM.	26.0		2, 3, 4, 5	None Observed During Drilling	10 ft: 50/5.5"; 15 ft: 50/5"; 20 ft: 50/6"; 25 ft: 50/4"
Vey dense, gray to dark gray, silty, fine to medium SAND, trace of fine gravel interbedded with seams of fine sandy SILT; moist; (Advance Outwash) SM/ML.	30.9		6		30 ft: 50/5"

BOTTOM OF BORING
COMPLETED 10/28/2002

Note: PID measurements were made on all samples; however, no readings were greater than 0 parts per million.

Depth, Ft.: 5, 10, 15, 20, 25, 30, 35, 40, 45

0 20 40 60

LEGEND

- * Sample Not Recovered
- Ɪ Standard Penetration Test

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 the subsurface maerials.
3. Groundwater level, if indicated above, is for the date specified and may vary.
4. Refer to KEY for explanation of symbols, codes, and definitions.
5. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING B-20

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FIG. A-3

MASTER_LOG_FINE 21-09644.GPJ SHAN_WIL.GDT 5/14/03 Log: WRL Rev: ACT Typ: LKD

APPENDIX B

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-09644-005

Date: May 22, 2003
To: Mr. Larry Goetz
NBBJ

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