Geotechnical Report
Proposed Stone Way Apartments
Seattle, Washington

December 9, 2004

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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GEOTECHNICAL REPORT
PROPOSED STONE WAY APARTMENTS
SEATTLE, WASHINGTON

1.0 INTRODUCTION

This report presents the results of subsurface explorations, laboratory tests, and geotechnical engineering studies for the proposed Housing Resources Group (HRG) Stone Way Apartments at 1205 North 45th Street in Seattle, Washington. Our scope of services included completing four exploratory borings at the site, performing laboratory tests and engineering analyses, and preparing this report. The purpose of the geotechnical studies was to develop recommendations for design and construction of the proposed structure. Our work was accomplished in general accordance with our proposal, which was dated October 19, 2004, and authorized by Vaughn McCleod of HRG on October 22, 2004.

2.0 SITE AND PROJECT DESCRIPTION

The project site is located on the north part of the block south of North 45th Street, and between Stone Way North and Midvale Avenue North, as shown on Figure 1. The site is approximately 325 feet long (parallel to North 45th Street) and 95 feet wide. The ground slopes gently to the south and west with approximately 8 feet of relief. Several one- and two-story buildings and associated parking areas occupy the site. An automobile parts store occupies the property to the south, on Stone Way North.

The parking area at the corner of Stone Way North and North 45th Street was formerly occupied by a Chevron gas station and in the 1930s by a service garage. Extensive subsurface explorations were made at this site, including at least 10 borings to depths up to 45 feet below ground surface. We understand contaminated soil is present beneath this former Chevron site, which will be cleaned up by others during the basement excavations. The cleanup will consist primarily of excavating and removing contaminated soil. Contaminated soil present beneath the foundation subgrade elevation will be overexcavated and replaced with structural fill material.

We understand you plan to develop the property with a four-story residential or mixed residential and commercial use building. The building would provide off-street parking with one level below grade. The bottom basement level finished floor would be about 5 to 12 feet below grade; therefore, the lowest slab foundation subgrade elevation would be about 15 feet below grade. Sumps for sewers, elevators, and other mechanical features may be about 5 feet below the slab subgrade. Deeper excavations likely will be made to remove contaminated soil beneath the former Chevron site. We understand these excavations may be about 6 feet below the proposed Stone Way Apartment subgrade elevation.

3.0 FIELD EXPLORATIONS

Four exploratory borings were drilled by Holocene Drilling on November 11, 2004, under subcontract to Shannon & Wilson, Inc. The approximate boring locations, determined by measuring distances from existing site features, are shown on Figure 2. The borings were drilled to depths of 25.0 to 31.5 feet below the ground surface, using hollow-stem auger (HSA) drilling techniques. A Shannon & Wilson, Inc. representative observed the drilling operations, collected soil samples, and prepared logs for the borings. A log key to the terms and symbols used in our classification of the soils is presented as Figure A-1 in Appendix A.

Disturbed samples from the borings were obtained in conjunction with the Standard Penetration Test (SPT). The tests were performed in general accordance with the American Society for Testing of Materials (ASTM) Designation: D 1586, Standard Method for Penetration Test and Split-Barrel Sampling of Soils. The SPT consists of driving a 2-inch, outside-diameter (O.D.), split-spoon sampler a total distance of 18 inches into the bottom of the drilled hole with a 140-pound hammer falling 30 inches. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). When penetration resistances exceeded 50 blows for 6 inches or less of penetration, the test was terminated and the number of blows and the corresponding penetration recorded. The N-values were recorded and plotted on a log of the boring. The completed logs for the borings are presented in Appendix A.

4.0 LABORATORY TESTING

Geotechnical laboratory tests were performed on selected samples retrieved from the borings to determine the index and engineering properties of the soils encountered at the site. The tests were performed at the Shannon & Wilson, Inc. laboratory and included visual classification, water content, and grain size analyses.

All soil samples recovered from the borings were visually reclassified in our laboratory using a system based on ASTM Designation: D 2487, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). This visual classification method allows for convenient and consistent comparison of soils from widespread geographic areas. Using this method, the soils can be classified by using the Unified Soil Classification System (USCS). The individual sample classifications have been incorporated into the boring logs shown in Appendix A.

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

Two grain size analyses were performed on selected samples of granular soil in general accordance with ASTM Designation: D 422, Standard Method for Particle-Size Analysis of Soils.

Grain size distribution is used to assist in classifying soils and to provide correlation with soil properties, including strength, permeability, and capillarity. The results of the grain size analyses are plotted on the grain size distribution curves presented on Figure B-1 in Appendix B.

5.0 SUBSURFACE CONDITIONS

Subsurface soil and groundwater conditions were interpreted from borings previously performed to evaluate contamination at the former Chevron gas station site and the subsurface explorations performed for this study. Figure 2 presents the locations of the existing and new borings. Figures A-2 through A-5 present the boring logs prepared for this report. The existing boring logs are presented in Appendix C. Figure 3 presents a generalized subsurface profile of the project site.

Based on the soils encountered in borings B-1 through B-4 and the Chevron site borings, we interpret that the site is generally underlain by fill overlying glacial till. Older non-glacial fluvial sand deposits were encountered below the glacial till in boring B-1 and other existing borings along the west margin of the project site. The fill, encountered in the upper 3.5 to 7 feet below the ground surface, consists of very loose to loose, gravelly, silty SAND. The glacial till consists of dense to very dense, slightly gravelly to gravelly, silty SAND. Scattered iron-oxide staining was present in the fill and glacial till soils. In boring B-2, B-3, and B-4, the glacial till was encountered to the bottom of the hole. In boring B-1, a dense, silty, fine sand was encountered below the glacial till at a depth of 20 feet and continued until the bottom of the hole at 31.5 feet below ground surface. Scattered ⅛-inch-thick organic-rich seams spaced roughly every 4 to 6 inches were observed in this soil unit.

The previous borings encountered similar dense to very dense, slightly gravelly, silty sand (glacial till) soils to a depth of 8 to 18 feet below ground surface. Below the glacial till soil, dense to very dense fine sand with 5 percent fines exists to the bottom of the borings. The sand unit described in previous borings located along the west margin of the site may be the same as encountered in B-1, although the described fines content in the boring log is less than we observed.

In boring B-1, a sample taken at 15 feet below ground surface exhibited a slight hydrocarbon odor. This finding suggests that the contamination plume from the Chevron station extends to at least approximately 80 feet west from the former gas station location. Further investigation may be warranted to determine the full extent of the contamination plume.

Groundwater was not detected in the borings performed for this study at the time of drilling. Observation wells were installed in the previous borings performed at the former Chevron gas station. All four existing monitoring wells recorded groundwater at 37 feet below ground surface. At MW-3, the boring log indicates that groundwater was also observed at 17 feet below ground surface, which may suggest the presence of a perched groundwater zone.

6.0 ENGINEERING STUDIES AND RECOMMENDATIONS

6.1 General

The following sections present our recommendations for design and construction of the proposed building. Our recommendations, which are based on the results of our subsurface explorations and laboratory tests, relate to seismic design, foundations, temporary shoring, earth pressures for walls and footings, floor slabs, drainage, earthwork, and construction considerations.

6.2 Seismic Design Recommendations

As of August 15, 2004, the International Building Code (IBC) 2003 is the official code used for seismic design of structures in the City of Seattle.

Characterization of soil profile type is required in the IBC 2003 to determine the site class definition. Based on the SPT values and soil classifications derived from previous explorations completed at the project site, it is our opinion that the project site could be classified as a Site Class C.

For design of seismic structures using the IBC 2003, mapped short-period and 1-second-period spectral accelerations, S_S and S_1, respectively, are required. S_S and S_1 are for a maximum considered earthquake, which corresponds 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 U.S. Geological Survey (USGS) completed probabilistic seismic hazard analyses (PSHA) for the entire country in November 1996, which were updated and republished in October 2003. The PSHA ground motion results can be obtained from the USGS website. The results of the updated USGS PSHA were referenced to determine S_S and S_1 for this site and are shown in the following table with the rest of the parameters necessary for IBC 2003 design.

INTERNATIONAL BUILDING CODE 2003
PARAMETERS FOR DESIGN OF SEISMIC STRUCTURES

Spectral Response Acceleration (SRA) and Site Coefficients	**Short Period**	**1-Second Period**
Mapped SRA	$S_s = 1.30$	$S_1 = 0.44$
Site Coefficients	$F_a = 1.00$	$F_v = 1.36$
Maximum Considered Earthquake SRA	$S_{MS} = 1.30$	$S_{M1} = 0.60$
Design SRA	$S_{DS} = 0.87$	$S_{D1} = 0.40$

Given the relatively dense nature of the soils underlying the site and that it is approximately 2.5 miles from the nearest known fault, the risks posed by earthquake-induced geologic hazards such as liquefaction, fault rupture, or lateral spreading at the project site should be considered relatively low.

6.3 Foundation Recommendations

In our opinion, the proposed structure could be supported on spread footings that bear on undisturbed dense to very dense soil that was encountered in the borings beneath the fill. The fill was approximately 3.5 to 7 feet thick. The following paragraphs provide a summary of our recommendations for spread footings.

For footings bearing on undisturbed dense to very dense, native soil, we recommend an allowable bearing pressure of 6,000 pounds per square foot (psf). In the area of the former Chevron site, contaminated native soil may be excavated and replaced with structural fill. For footings bearing on structural fill, we recommend an allowable bearing pressure of 4,000 psf. Greater allowable bearing pressures could be used if larger settlements are tolerable. We would review specific foundation designs for greater allowable bearing pressures, as needed.

Continuous footings should have a minimum width of 18 inches, and column footings should have a minimum width of 24 inches. The base of all footings should be at least 24 inches below the lowest adjacent exterior grade and at least 18 inches below the lowest adjacent interior grade.

Where adjacent individual footings are located at different elevations, we recommend that the horizontal distance between them be at least 1.5 times the elevation difference. Where adjoining continuous footings are at different elevations, we recommend that the upper footing be stepped down to the lower footing.

Spread footing foundations designed and constructed as recommended are estimated to undergo a total settlement of about ½ inch. Differential settlement between adjacent column footings or over a 20-foot span of continuous footing is estimated to be approximately one-half the total settlement between the columns. It is anticipated that estimated settlements would occur essentially simultaneously as the loads are applied.

6.4 Temporary Shoring

6.4.1 General

In our opinion, the site soils are suitable for a soldier pile and lagging wall system. Tieback anchors or internal bracing likely would be required for temporary shoring higher than about 15 feet. Alternatively, the excavation could be supported using a soil nail shoring system. Vertical elements would likely be required in fill soils to support soil nail shoring unless open cuts can be completed into sidewalks. For cuts less than 10 feet, particularly where the proposed building will be set back from the property line, unshored, temporary excavation slopes may be another option. The following sections provide preliminary design recommendations for these three alternatives.

6.4.2 Soldier Pile Tieback Anchor Walls

6.4.2.1 General

Recommended lateral earth pressures for design of a temporary soldier pile cantilever wall are presented in Figure 4. Recommended apparent lateral earth pressures for design of internally braced or tieback soldier pile walls are presented in Figure 5. Recommended surcharge loading for temporary and permanent walls is presented on Figure 6.

Recommendations assume active and at rest soil conditions and are shown in Figures 4 and 5. For active conditions, lateral wall movements could range from 0.10 to 0.15 percent of the excavation depth, or to approximately ½ inch for a 20-foot-deep excavation. 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 15 feet. At-rest conditions should be used where wall deflection is limited such that active pressures cannot develop, and in areas where deflections required to develop active earth pressures would not be desirable. For this site, these areas may include walls near existing buildings on adjacent properties, such as the automobile parts store on Stone Way.

The above-mentioned deflections and settlements are estimates only and are, in part, affected by the methods and care used during construction. Therefore, we recommend monitoring temporary shoring wall during construction. Refer to section 7.8 for monitoring recommendations.

6.4.2.2 Soldier Piles

Vertical members for a soldier pile shoring system typically consist of steel sections placed in predrilled holes and 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 be designed to resist the total vertical component of the tieback anchor forces. Temporary vertical soldier pile capacities below the bottom of the excavation can be evaluated from the skin friction and end-bearing pressures given in Figures 4 and 5.

6.4.2.3 Lagging

We recommend that lagging be installed between soldier piles. Lagging should be installed as the excavation proceeds, and, in general, not more than 4 feet (measured vertically) of unsupported excavation should be exposed at any one time. The actual height of vertical, unsupported excavation may be less than 4 feet depending on the soil encountered. Slightly silty to clean sand was reported in borings at the former Chevron gas station site. These soil types could run when exposed in an open excavation.

The Contractor should provide means, such as weep holes, to prevent the buildup of hydrostatic pressures behind shoring walls. Voids behind the lagging should be filled with concrete sand or drainage sand and gravel or locally with a controlled density fill (CDF).

Because of 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 4 and 5.

6.4.2.4 Tieback Anchors

Tieback anchors consist of steel strands or reinforcing bars placed into predrilled holes. The holes are typically drilled at an inclination of about 15 degrees below 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 the production anchors should be based on test anchors. The following frictional values are only for planning and estimating anchor lengths.

Temporary tiebacks installed by hollow-stem auger methods (that is, no pressure grouting) could be designed for an allowable frictional value of 2.0 kips per square foot (ksf). Anchor grout should be placed by tremie method for open-hole anchor installations. Perched groundwater and caving soils may prevent open-hole anchor installations; in which case, casing and/or pressure grouting may be used to install the anchor. Small-diameter, cased, tieback anchors, grouted using tremie methods, should be designed for 2.0 ksf as described above. For cased, pressure-grouted anchors in 3- to 6-inch-diameter boreholes, an allowable load transfer rate of 5 kips per lineal foot (klf) should be used for design. A single-stage, pressure-grouted anchor is defined as an anchor that undergoes high pressure grouting as the drill casing is removed. Higher friction values may be achieved if the anchors are post-grouted. A minimum anchor length of 15 feet is recommended.

We recommend that tieback anchors be spaced a minimum of three diameters apart, measured center-to-center.

6.4.3 Soil Nail Wall

6.4.3.1 General

Soil nailing consists of drilling and grouting a series of steel bars or "nails" behind the excavation face and then covering the face with reinforced shotcrete. The placement of relatively closely spaced steel nails in the retained soil mass increases the shear resistance of the soil against rotational sliding, increases the tensile strength of the soil behind potential slip surfaces, and moderately increases shear resistance at a potential slip surface due to the bending stiffness of the nails.

Soil nailing is typically effective in dense, granular soils with some apparent cohesion and stiff, low plasticity and fine-grained soils. It may not be cost-effective in loose granular soils, soft cohesive soils, highly plastic clays, or where uncontrolled groundwater exists above the bottom of the excavation. In general, excavation faces must be able to stand unsupported for 24 hours for soil nailing to be feasible. The borings performed for this report encountered dense gravelly, silty sand material (till) and silty, fine sand. The fines content of these sands would result in an apparent cohesion and therefore soil nailing could be a shoring option. However, previous borings performed at the former Chevron station suggest the presence of sand with 5 percent fines content (see Appendix C). This sand material might not have enough apparent cohesion, so that running sand and consequent ground loss could result during the excavation and result in low soil nail pullout resistance values. These problems may lead to excessive wall movements that can cause damage to neighboring building structures, such as the automobile parts store, and existing utilities.

Soil nail wall construction involves a top-down procedure that generally includes three steps for each horizontal row of nails: (1) staged excavation, (2) nail installation and select nail testing, and (3) drainage and facing construction. This sequence of staged excavation, nail installation, and drainage/facing construction in horizontal rows is repeated until the excavation and shoring is complete. Soil nails consist of steel bars (typically ¾ - to 1 ⅜-inch-diameter), which are installed by tremie grouting the nail into a predrilled hole. Soil nails are located in square or rectangular grid patterns (e.g., 5- to 8-foot grid) and are typically installed at an inclination angle of 15 degrees below horizontal. Drainage is provided behind the wall facing by placing vertical rows of geosynthetic drainage composites between the grids of soil nails and connecting these to weep holes in the bottom of the wall. The wall facing typically consists of shotcrete sprayed over a reinforcing steel mesh on the excavation face.

In general, the first row of nails is installed not more than 2 to 4 feet below the ground surface, and the bottom row of nails is installed less than 4 feet above the bottom of the cut or excavation. Soil nails lengths typically range from 0.7 to 1.0 times the wall height. For very dense glacial till soil and dense silty sand, the soil nail lengths likely would be about 0.75 to 0.8 times the wall height.

Design of a soil nail wall is beyond our current scope of services; however, as mentioned, we have in-house capability to prepare complete plans and specifications for soil nail walls. A separate proposal for this work can be provided at your request.

6.4.3.2 Anticipated Movement

Soil nails develop capacity when the shoring wall deflects toward the excavation and the nails are in tension. In other words, it is a passive shoring system. Excessive deflection could result in damage to structures and utilities adjacent to the excavation. Our experience with soil nail walls of similar height and in similar soil types as those anticipated is that horizontal deflection at the top of the wall is typically ¼ to ½ inch. Vertical settlements of the same magnitude are expected to occur at the face of the wall. Vertical settlements will decrease with distance from the wall and should be negligible beyond a distance of about the wall height. Wall monitoring is recommended in Section 7.9.

6.4.4 Unshored Excavation Slopes

Unshored, temporary excavation slopes may be used where planned excavation limits will not undermine existing structures, interfere with other construction, or extend beyond construction limits. Suitable temporary excavation slopes will depend on: (1) the presence of locally perched groundwater; (2) the type and density of the soils; (3) the depth of excavation; (4) surcharge loading adjacent to the excavation such as that from excavated material, existing structures, or construction equipment; and (5) the time of construction. For planning purposes, we recommend assuming that temporary slopes less than 10 feet high would be excavated at no steeper than 1 horizontal to 1 vertical (1H:1V) in the dense to very dense soils, and 1.5H:1V in loose to medium dense, near-surface, native soils and fill. If wetted by surface water, temporary slopes may be subject to erosion. Slope protection, such as a plastic covering weighted down with sand bags, should be employed as appropriate during construction to reduce the erosion.

Consistent with conventional construction practice, temporary excavation slopes should be made the responsibility of the Contractor. The Contractor is continually at the site and is able to observe the nature and conditions of the subsurface materials encountered, including groundwater, and has responsibility for the methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. Regardless of the construction method used, all excavation work should be accomplished in compliance with applicable local, state, and federal safety codes.

6.5 Lateral Earth Pressures

The lateral pressures against permanent walls are dependent upon many factors, including method of backfill placement and degree of compaction, backfill slope, surcharges, the type of backfill soil and/or adjacent native soils, drainage provisions, and whether or not the wall can yield laterally after or during placement of backfill. If the wall is free to yield at the top an amount equal to approximately 0.001 times the height of the wall, then active earth pressures should be mobilized. If movement is not allowed because of stiffness or resistance of the wall, the wall should be designed for at-rest earth pressures.

Walls allowed to deflect laterally or rotate at the top should be designed using an active lateral pressure equivalent to a fluid unit weight of 26 pounds per cubic foot (pcf). Walls that are not allowed to yield or deflect 0.001 times the wall height should be designed using an at-rest lateral pressure equivalent to a fluid unit weight of 43 pcf. These values should be increased by 1 pcf for each degree of upward inclination of back slope above any of the walls. These pressures assume that imported granular structural fill is used as wall backfill and that proper drainage is provided behind the walls so that there is no buildup of hydrostatic pressures. Below grade basement walls could be designed for the active apparent earth pressure (17H psf) plus surcharge loads, as the soil load from the shoring system would be transferred to the braced walls of the underground garage.

The total active earth pressure should be analyzed for seismic loading conditions using a dynamic load increment equal to 6 psf/ft. This increment should be applied as a uniform load to the wall and is consistent with a pseudostatic analysis using the Mononobe-Okabe equation for lateral earth pressures for a horizontal seismic coefficient of 0.17g. The horizontal seismic coefficient is not necessarily equivalent to the design peak ground acceleration at the site. The magnitude of this coefficient accounts for the fact that the peak ground acceleration is experienced only a few times within the record of earthquake shaking, and that the actual earthquake ground motion is cyclic in nature, as opposed to a static force. Values of the horizontal seismic coefficient are typically one-third to one-half the value of design peak ground acceleration of 0.58g that may be experienced at the site. Those pressures assume drained conditions behind the wall and a horizontal backfill surface.

6.6 Lateral Resistance

Resistance to lateral forces caused by wind, seismic, unbalanced earth pressures, and/or other forces can be provided by both passive earth pressures acting against the embedded portion of foundations and frictional resistance against base of foundations. In our opinion, passive earth pressures developed from the very dense, native soils could be based on an equivalent fluid density of 450 pounds pcf. The passive earth pressure developed from structural fill could be based on an equivalent fluid density of 200 pcf. These values are based on the assumptions that the footings extend at least 2 feet below the lowest adjacent exterior grade, they are properly drained, and that the backfill around the structure is properly compacted. The above equivalent fluid unit weights include a factor-of-safety (FS) of 1.5 to reduce wall movement required to develop passive resistance. We recommend that a coefficient of friction of 0.45 be used between cast-in-place concrete and soil.

6.7 Floor Slabs

The very dense, native sand or compacted structural fill should provide a suitable subgrade at locations where the slab is constructed on grade. We recommend that a vertical modulus of subgrade reaction equal to 300 pounds per cubic inch (pci) be used in the design of the floor slab-on-grade.

As a capillary break, we recommend that a minimum 6-inch-thick layer of washed pea gravel (⅜ inch to No. 8 sieve size) or clean, ⅝-inch minus crushed rock (less than 2 percent passing the No. 200 sieve), and a vapor barrier consisting of plastic sheeting, be placed beneath floor slabs, as shown on Figure 6. If pea gravel is used, a 2-inch layer of clean crushed rock can be placed over a 4-inch minimum layer of washed pea gravel to provide a more firm working surface on which to place the reinforcement. Alternatively, a 4-inch layer of ⅝-inch minus crushed gravel could be used as a capillary break. The vapor barrier should be placed on top of the capillary break materials. Prior to placing pea gravel and/or crushed rock, the exposed subgrade surface should be evaluated by a representative of our firm and compacted as needed to achieve a dense, unyielding condition. If used, the crushed rock should be compacted with at least three complete coverages of a vibrating plate compactor.

6.8 Drainage

In our opinion, suitable long-term performance of the proposed structure will require that subdrains be installed. We recommend that a foundation subdrain system be installed around the perimeter of the proposed structure.

Recommendations for a footing or wall subdrain system are presented in Figure 7. The perforated subdrain pipe invert should be at least as low as the top of the footing, and it should be sloped to drain. The ground surface should be sloped away from the building to prevent water ponding. All surface water that may come from driveway runoff, downspouts, and catch basins, and all subsurface water coming from footing and wall subdrains should be collected in drain tightlines and carried to a suitable discharge point. Surface water should not be allowed to discharge into the foundation drains. Where a perforated or slotted drain pipe discharges to a tightline drain pipe, we recommend that an impervious, concrete or low permeability soil collar be constructed around the first 2 feet of tightline pipe to force all water into the drainage system. Cleanouts should be provided at several convenient locations along all drain lines. The integrity of this piping system should be monitored on an annual basis.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Site Preparation and Grading

We recommend that trees and brush be cleared and that roots, stumps, construction debris, existing slabs, footings, and utility lines be removed from beneath proposed structure foundations and all areas to be graded. Any grass and topsoil that covers the site is loose and organic and should be removed from the structure site except in landscape areas where settlements would not be objectionable. Topsoil is not considered suitable for reuse as structural fill and should be removed from the site or stockpiled for reuse in landscape areas. We anticipate that excavations can be accomplished with conventional excavating equipment, such as a dozer, front-end-loader, or backhoe.

In areas to be filled, the exposed soil surface (after clearing and stripping and prior to any fill placement or foundation construction) should be compacted using a vibratory roller or Hoepac. Native subgrade soils should be proof-rolled and, if necessary, compacted to achieve at least 95 percent of the Modified Proctor maximum dry density and to a dense and unyielding condition. Areas that are wet, soft, loose, or yielding under the compaction process should be further compacted, removed and reconditioned, or replaced with compacted structural fill so that a dense and unyielding condition is achieved.

7.2 Fill Placement, Compaction, and Use of On-site Soils

All fill soil placed beneath areas where settlements are to be minimized should be structural fill. Structural fill soil should consist of a well-graded mixture of sand and gravel, free of organics, debris, and rubbish. If imported, it should contain not more than 15 percent fines (material passing the No. 200 mesh sieve, based on the minus ¾-inch fraction); the fines should be nonplastic; and the moisture content of the soil should be within ±2 percent of its optimum. The gravel content should range between 25 and 50 percent retained on a No. 4 sieve. Imported structural fill should be at a moisture content near optimum to allow proper compaction. All structural fill should have a maximum particle size of 3 inches. During wet weather or in wet conditions, structural fill material should consist of clean granular soil with not more than 5 percent by weight passing the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch sieve. Much of the on-site soil would not be suitable in wet weather or in wet conditions.

Prior to the placement of structural fill, any ponding water should be drained from the area. Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition, to at least 95 percent of the Modified Proctor maximum dry density (ASTM D 1557). The thickness of loose lifts should not exceed 8 inches for heavy equipment compactors or 4 inches for hand-operated mechanical compactors. Backfill compaction adjacent to walls should be performed using hand-operated compactors. In areas where some settlements could be tolerated, such as landscaped areas, the density requirement for fill could be reduced to 90 percent of the Modified Proctor maximum dry density.

In our opinion, the existing fill and till soils may be used as structural fill if they are suitably moisture conditioned and compacted to the desired density and unyielding condition. However, these soils may not be suitable for compaction during wet weather, as they may be or may become soft, unstable, and difficult to work.

7.3 Soil Nails

Soil nails should be installed in a horizontal sequence with the base of the staged excavation extending a maximum of 2 to 3 feet below the level of the nail to be installed. If new utilities are installed along or near the base of the wall, the full depth of the excavation (including utility trench) should be included in the design. Any utilities to be installed behind temporary shoring walls should be installed before excavation begins or after the permanent basement walls are capable of supporting the design lateral pressures.

Based on our experience, we anticipate that little sloughing will occur in the glacial till if the soil is dry and unsupported heights do not exceed 6 feet. However, if the soil does not contain sufficient fines to provide binder material, it may slough. No test cuts were completed for this study, therefore, the binding nature of the soil is unknown. If groundwater seepage is encountered from locally perched zones, flowing ground conditions and/or sloughing could occur. In the event that this occurs, dewatering will be necessary. To reduce ground loss at the excavation face, it may be necessary to leave a shallow stabilizing berm in front of the wall. Soil nails would then be installed through this berm. The berm must lie below the previous shotcrete lift and be constructed at a stable slope. After nails are installed, the berm would be excavated around the nails and reinforced shotcrete would be applied. In areas were sloughing is not severe enough to warrant a berm, it may be possible to shotcrete the excavation face first and drill soil nails through the shotcrete.

Approximately 5 percent of temporary soil nails, randomly selected, should be performance tested by loading in 25 percent (0.25P) increments to 150 percent of design capacity (1.5P), where P is the design capacity. The 150 percent load should be held constant for a minimum period of at least 60 minutes. Acceptance criteria and additional testing requirements would be provided in the shoring plan notes.

7.4 Soldier Pile and Tieback Installation and Testing

Tieback anchor holes should be drilled in a manner that will minimize loss of ground and not endanger previously installed anchors or undermine existing pavement or utilities. The contractor should be prepared to drill through and install anchors in very dense and hard soil and through any potential obstructions. Different drilling techniques such as casing may be required for tiebacks located below groundwater or running sand, if encountered. We recommend that tiebacks located below utilities be drilled, grouted, and installed using casing.

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

All temporary anchors should be proof-tested in 25 percent (0.25P) increments to 133 percent of their design capacity (1.33P). Each load increment should be held until the deformation stabilizes (normally about 1 minute) and the load and corresponding deformation are recorded. After reaching 1.33P, the load should be held for at least 10 minutes to evaluate creep and then be reduced to the lock-off load.

Prior to installing production anchors within a particular soil stratum, performance tests should be accomplished for each anchor type and/or installation method that will be used. The number of tendons in the selected anchors should be increased as required to complete the performance tests. Approximately 3 to 5 percent of temporary production anchors, randomly selected, should be performance tested by loading in 25 percent (0.25P) increments to 200 percent of design capacity (2.0P). At lease one performance test should be performed in each soil type expected. The 200 percent load should be held constant for a minimum period of at least 60 minutes.

We recommend that all temporary anchors be locked off at 80 to 90 percent of the design load to provide some wall flexibility. Anchors that do not meet the testing acceptance criteria should be locked off at 50 percent of the failure load and replaced with additional anchors, as required.

Load testing and acceptance criteria for all tieback anchors should be as recommended by the Post-Tensioning Institute (PTI) Manual, Chapter 4, Recommendations for Pre-Stressed Rock and Soil Anchors. As described above in the manual, the following tests should be accomplished.

Initial Lift-off Readings: After transferring the load to the stress anchorage and prior to removing the jack, a lift-off reading should be made. The load determined from the lift-off reading should be within 5 percent of the 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 anchors, both during and after construction, to check the magnitude of seating and transfer load losses and to determine whether long-term losses are occurring.

Acceptance Criteria: The results of each anchor test should be evaluated in order to determine anchor acceptability. An anchor would be acceptable provided:

- The total movement obtained from performance and proof tests exceeds 80 percent of the theoretical elastic elongation of the design free stressing length.
- The creep rate during the final test load does not exceed 0.08 inch per log cycle of time and is a linear or 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.

7.5 Footings

The recommended allowable bearing capacities presented previously in this report are contingent upon the following construction considerations:

- Footing subgrade excavations should be cleaned of all fill, debris, and loose, soft, wet, or disturbed soil prior to placing the reinforced concrete.
- If construction is to take place in wet weather, we recommend that a thin layer (2 to 4 inches thick) of lean concrete, also known as a "rat slab" or "mud slab," be placed immediately after excavating to serve as a working surface. Footing excavations should be kept free of water at all times. If groundwater is encountered, it should be lowered to at least 2 feet below the bottom of footing excavations.
- All excavations for spread footing foundations should be observed by a geotechnical engineer to evaluate the adequacy of the bearing stratum and to confirm that subsurface conditions at and below the bearing elevation are suitable for the design bearing values provided.

7.6 Wet Weather and Wet Condition Considerations

In the Seattle area, wet weather generally begins about mid-October and continues through about May, although rainy periods could occur at any time of year. Therefore, it would be advisable to schedule earthwork during the dry weather months of June through September. Most of the soil at the site likely contains sufficient silt/clay fines to produce an unstable mixture when wet. Such soil is highly susceptible to changes in water content and tends to become unstable and difficult or impossible to proof-roll and compact if the moisture content significantly exceeds the optimum. In addition, during wet weather months, the groundwater levels could increase, resulting in seepage into site excavations. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following recommendations are provided:

- The ground surface in and surrounding the construction area should be sloped as much as possible to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Work areas or slopes should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.

- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill could be accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.
- Fill material should consist of clean, well-graded, pit-run sand and gravel soils, of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve. The gravel content should range from between 20 and 50 percent retained on a No. 4 mesh sieve. The fines should be nonplastic.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see gradation requirements above).
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

We recommend that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

7.7 Obstructions

Buildings previously on site, such as the former Chevron gas station, and their footings, floor slabs, and basement walls may still be partially or completely buried on site. The existing foundations, walls, and slabs should be anticipated and may require special consideration during soil nail installation and site excavation.

Although 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 documents contain an advisory statement that cobbles, boulders, and other obstructions may be encountered in the mass excavation and soldier pile, tieback anchor, or soil nail drill holes. The presence of these materials may require installation of additional tiebacks or soil nails or 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. The void produced by removing face obstructions should be backfilled with shotcrete for a soil nail wall or sand for lagging walls.

7.8 Instrumentation

We recommend that an instrumentation program be implemented to monitor movements during excavation and shoring installation. A preconstruction crack survey of adjacent streets, buildings, and facilities should be completed prior to any excavation or shoring, and monitoring baseline readings should be established before excavation begins. We recommend that optical survey points be established on existing structures located within a distance equal to the depth of the excavation from the top of the excavation.

For soil nail walls, we recommend that optical survey points be established no more than 2 feet behind the walls at a spacing of 20 feet along the soil nail wall alignments and that a second set of survey points be established 20 feet behind the soil nail wall face. We recommend that optical survey points be established on the shoring wall as excavation progresses. Monitoring points should be evaluated on a weekly basis during construction or as excavation progress dictates. If horizontal movements are observed to be in excess of ½ inch between successive readings, construction of the soil nail wall should be stopped to determine the cause of the movement and to establish the type and extent of remedial construction. The survey points should be monitored until lateral loads are fully supported by the permanent structure. The top of the soldier piles should be recorded for vertical and horizontal movement. As with soil nails, survey points 20 feet behind the soldier pile wall should also be installed and monitored similarly, as recommended above.

7.9 Plans Review and Construction Observation

We recommend that Shannon & Wilson, Inc., be retained to review those portions of the plans and specifications that pertain to foundations and earthwork to determine whether they are consistent with the recommendations in this report. We also recommend that we be retained to provide geotechnical construction observation services during construction activities. Our services would include observing earthwork construction, shoring wall installation, placement and compaction of structural fill, preparation of the footing subgrade and drainage layout, and accomplishing other geotechnically related earthwork activities.

8.0 LIMITATIONS

This report was prepared for the exclusive use of HRG and their design done for specific application to this project. The data and report should be provided to prospective contractors (or the Contractor) for information on factual data only. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the exploratory borings are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the field explorations. If subsurface conditions different from those encountered in the explorations are encountered or appear to be present during construction, 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 construction at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations concerning the changed conditions or time lapse.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from 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.

The scope of our present work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater, should any be encountered. Shannon & Wilson, Inc. has qualified personnel to assist you with these services should they be necessary.

Shannon & Wilson, Inc. has prepared and included Appendix D, "Important Information About Your Geotechnical Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.

Claire L Roggero

Claire L Roggero
Engineer

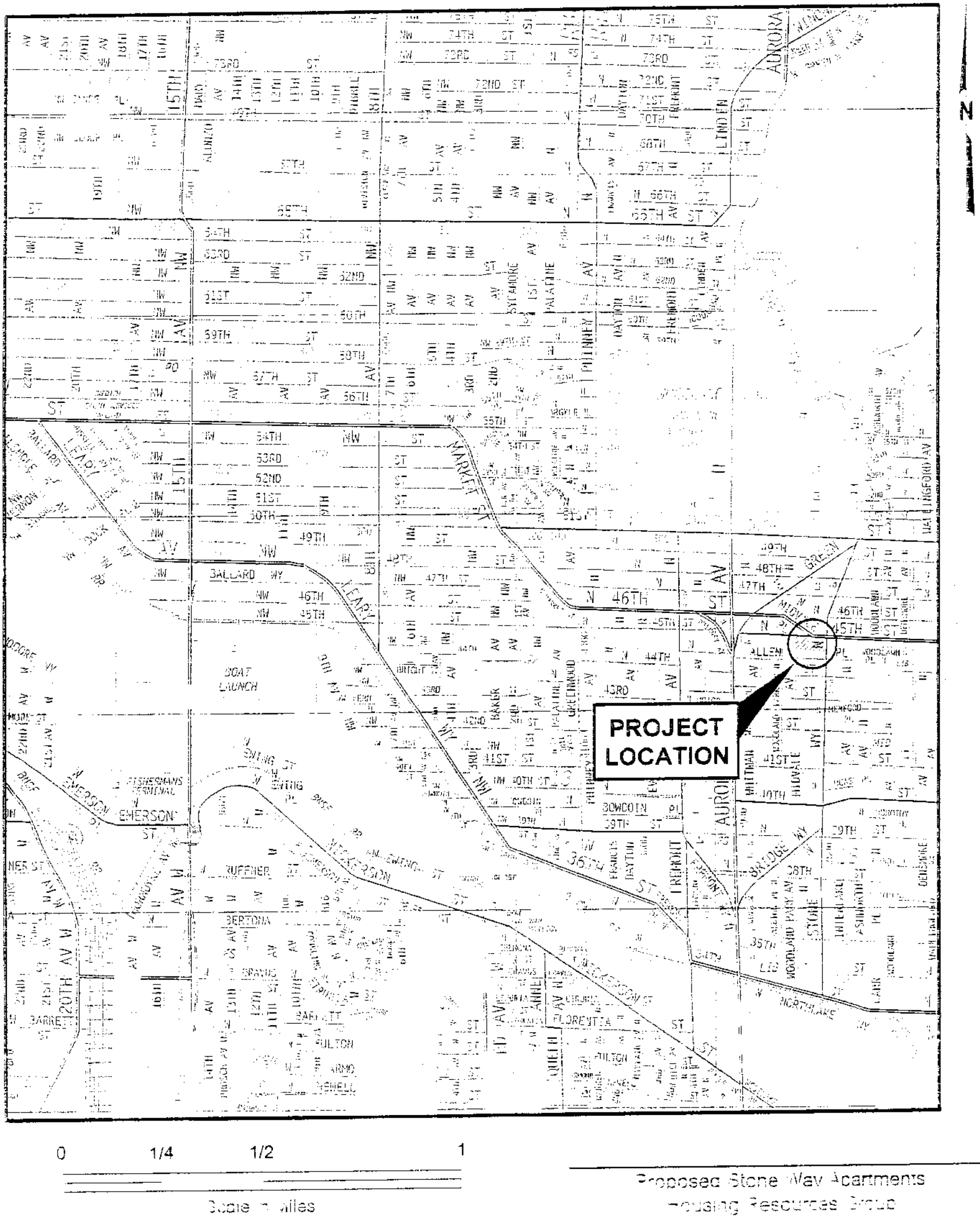
CHRISTOPHER A. ROBERTSON
STATE OF WASHINGTON
30636
REGISTERED PROFESSIONAL ENGINEER

12/9/04

EXPIRES 11/28/06

Christopher A. Robertson, P.E., L.E.G.
Senior Associate

CLR:CAR:TMG/clr



Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

NOTE

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VICINITY MAP

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FIG. 1

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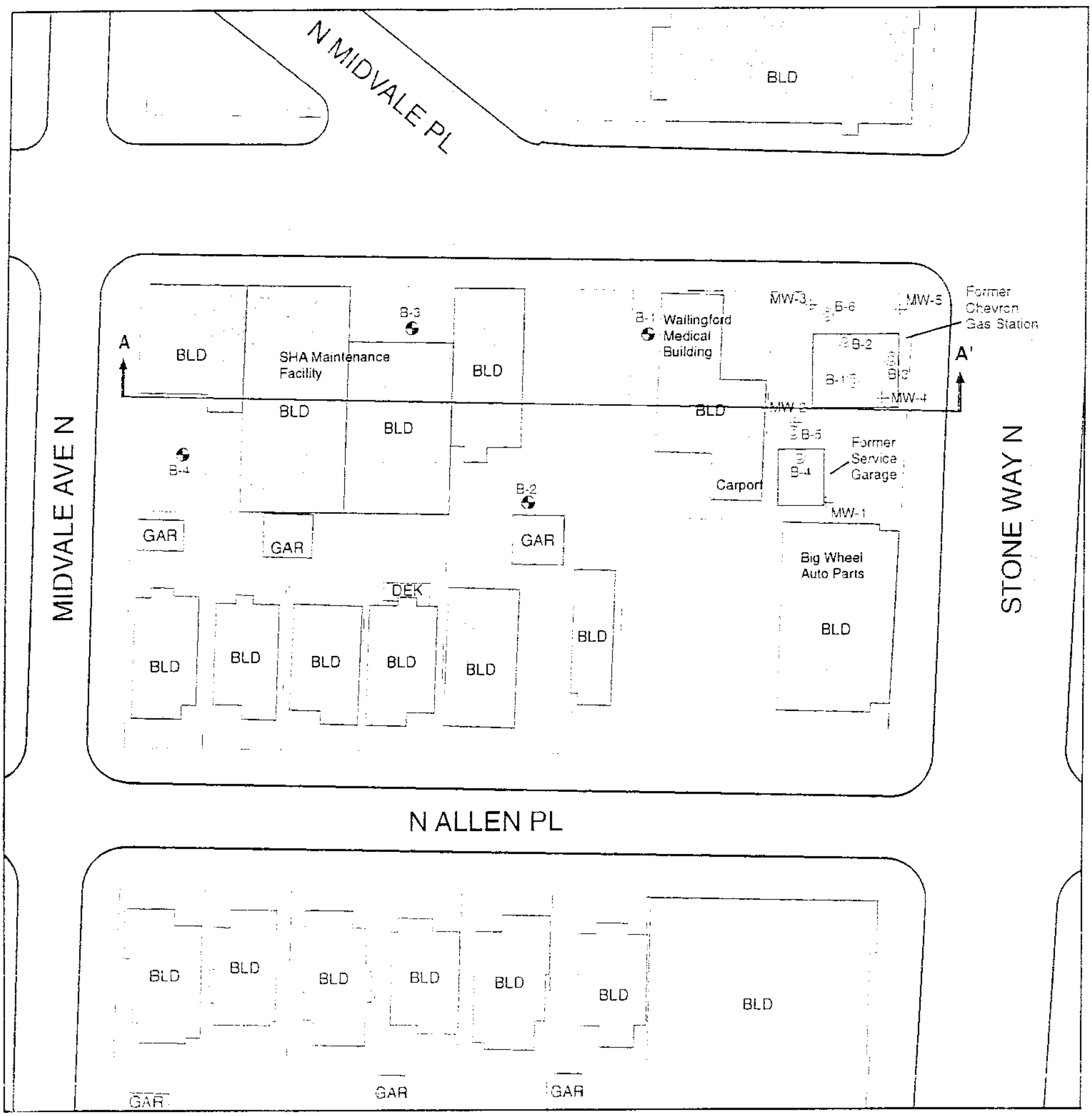

NOTES

1. Base data provided by City of Seattle.

2. Locations of borings are approximate.

LEGEND

- Existing Borings
- Monitoring Well
- Borings Performed for this Report

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SITE AND EXPLORATION PLAN

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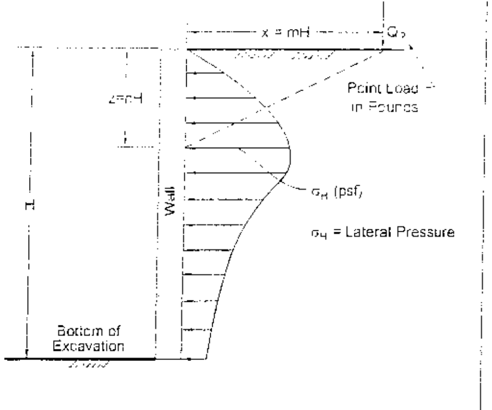
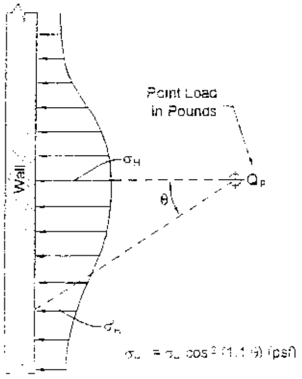
FIG. 2



ELEVATION VIEW

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

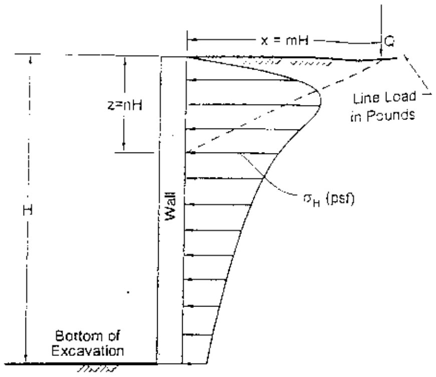
For m > 0.4: $\sigma_H = 1.77 \frac{Q_P}{H^2} \frac{m^2 n^2}{(m^2 + n^2)^3}$ (psf)



PLAN VIEW

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

(NAVFAC DM 7.2, 1986)



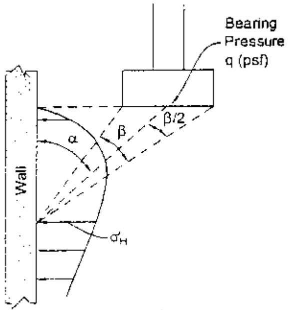
ELEVATION VIEW

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

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

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

(NAVFAC DM 7.2, 1986)

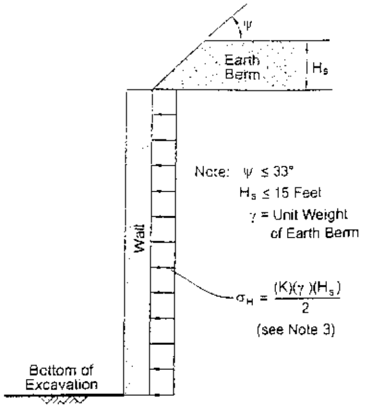


$\sigma_H = \frac{2q}{\pi} (\beta - \sin\beta \cos 2\alpha)$ (psf)

in radians

C) LATERAL PRESSURE DUE TO STRIP LOAD

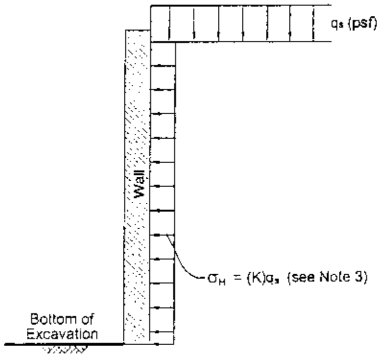
(derived from Fang, *Foundation Engineering Handbook*, 1991)



Note: ψ ≤ 33°
H_s ≤ 15 Feet
γ = Unit Weight of Earth Berm

$\sigma_H = \frac{(K)(\gamma)(H_s)}{2}$
(see Note 3)

EARTH BERM

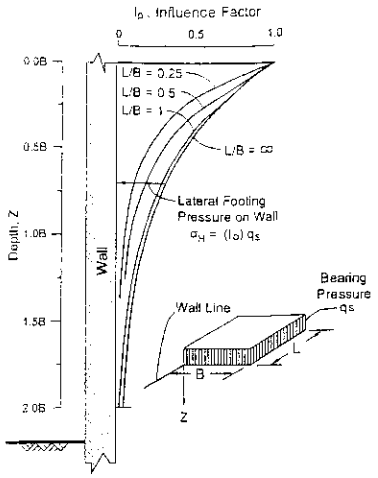


$\sigma_H = (K)q_s$ (see Note 3)

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)



Lateral Footing Pressure on Wall
$\sigma_H = (I_o)\,q_s$

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. 6

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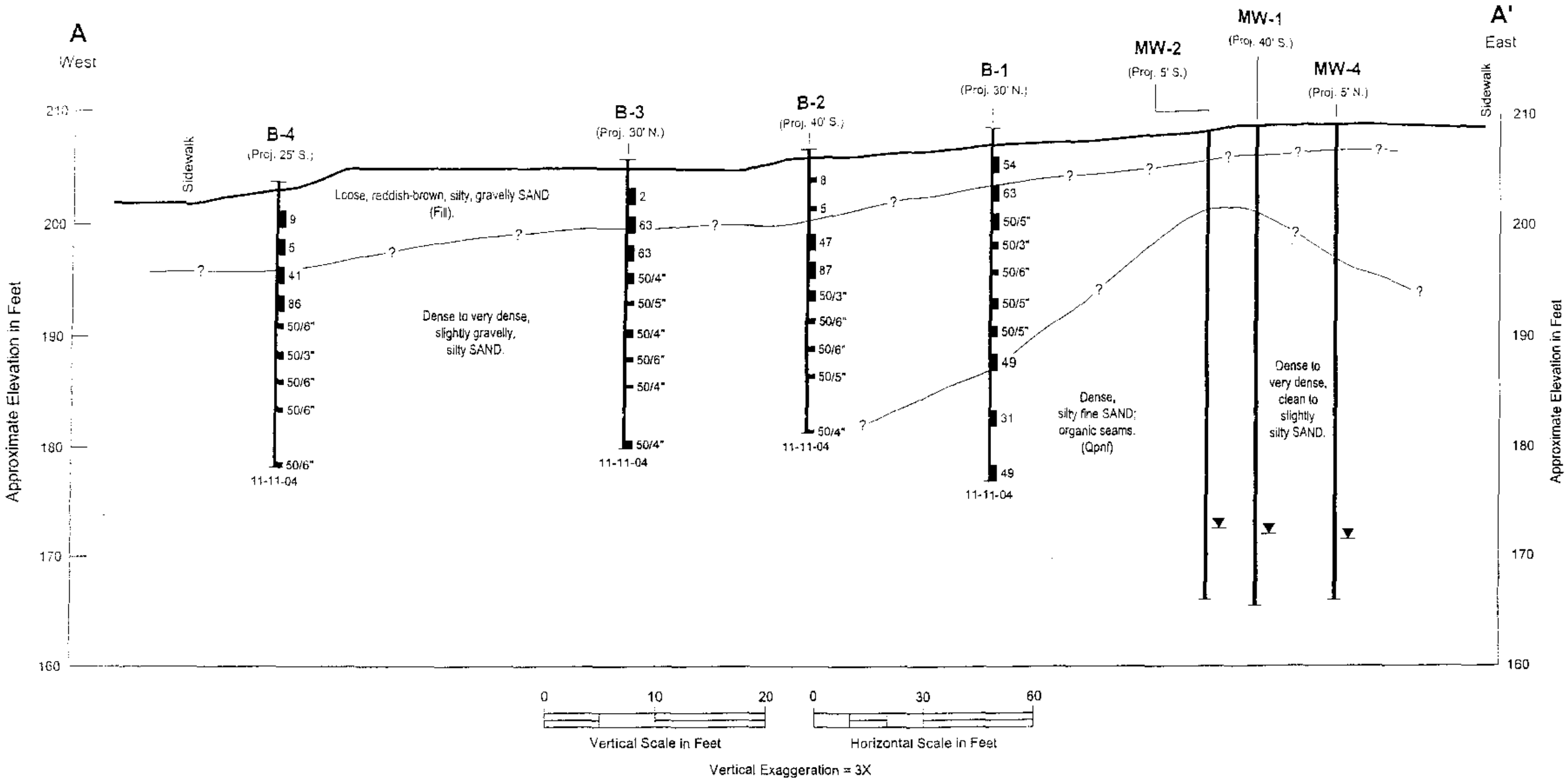

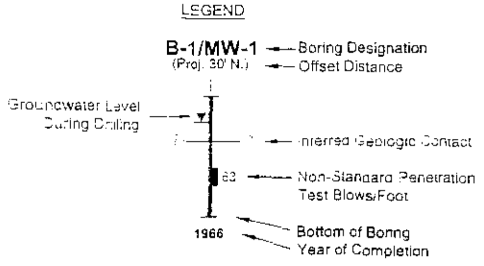

NOTE

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

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GENERALIZED SUBSURFACE PROFILE A-A'

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FIG. 3

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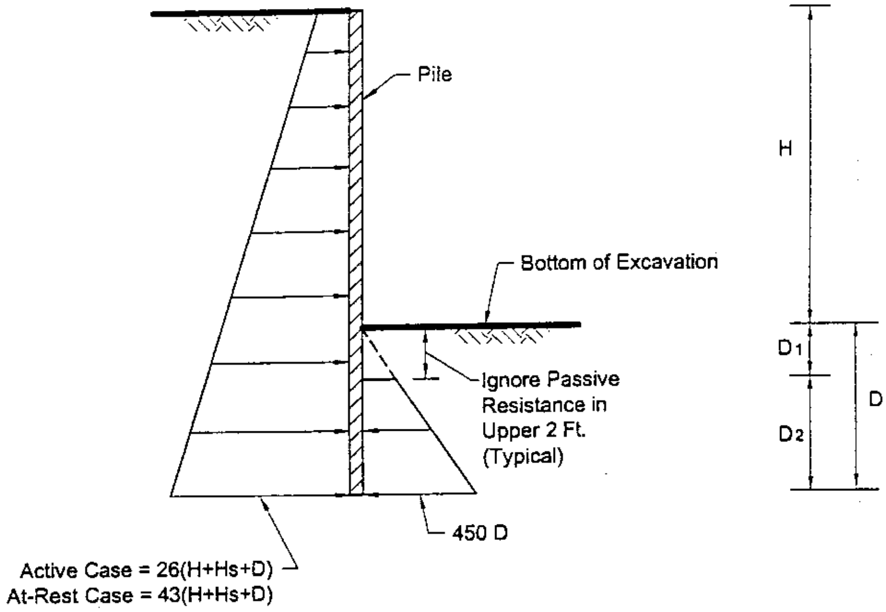
Recommended Earth Pressures
for Cantilever Wall



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. Passive pressures include FS = 1.5
4. If a sloping ground surface exists, the earth pressures should be adjusted. (Refer to text.)
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 bottom of excavation and apply passive resistance over twice the width of the piles or the spacing of the piles, whichever is smaller.
6. Free drainage assumed behind the wall.
7. Design lagging for 30% of lateral earth pressure.
8. Allowable vertical soldier pile capacity:

Skin friction = 1 ksf
End bearing = 6 ksf (After loose/disturbed soil
at bottom of hole is removed);
For Native Soil
= 4 ksf for Structural Fill

LEGEND

H	Excavation Height (Ft.)
Hs	Equiv. Surcharge Height (Ft.)
D, D1, D2	Embedment Depths (Ft.)
Ka, Ko, Kp	Active, At-Rest, and Passive Earth Pressure Coefficients, (See Table)

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CANTILEVERED PILE WALL DESIGN CRITERIA

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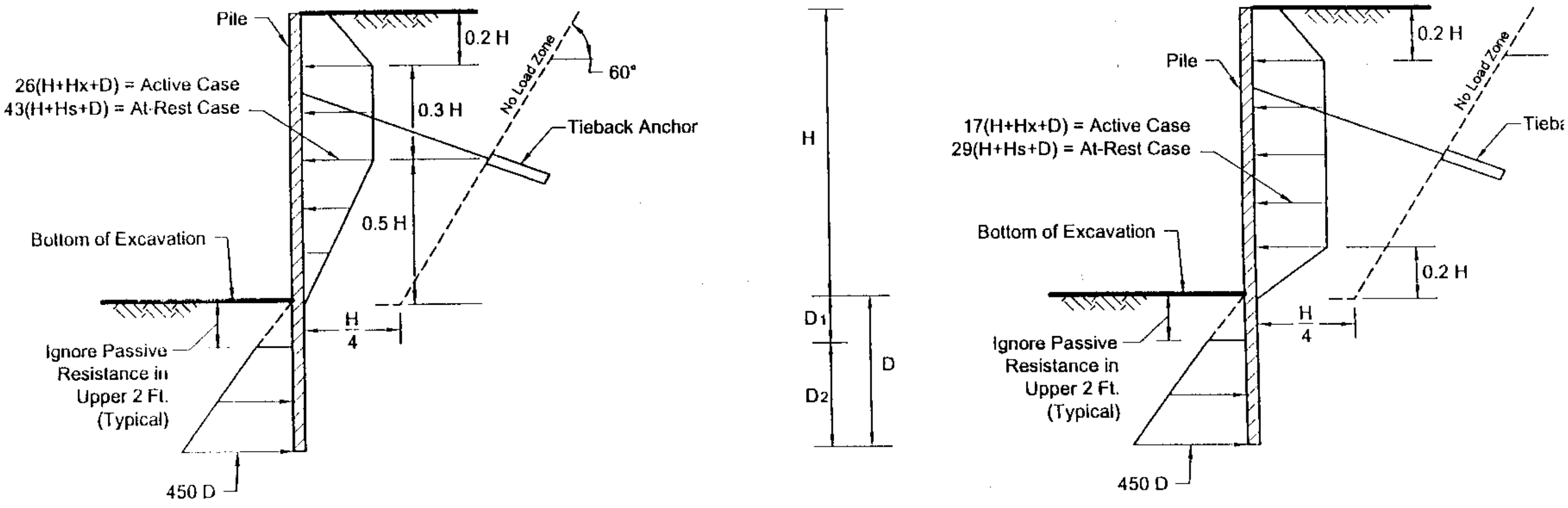
FIG. 4

File: I:\Drafting\211\20193-001\21-1-20193-001 Fig 5.dwg Date: 12-06-2004 Author: DRAFTEMP

Recommended Earth Pressures
for Single Tieback Wall

Recommended Earth Pressures
for Multiple Tieback Wall



LEGEND

H	Excavation Height (Ft.)
Hs	Equiv. Surcharge Height (F
D, D_1, D_2	Embedment Depths (Ft.)
Ka, Ko, Kp	Active, At-Rest, and Passi Pressure Coefficients, (Se

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. Passive pressures include FS = 1.5
4. If a sloping ground surface exists, the earth pressures should be adjusted. (Refer to text.)
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 bottom of excavation and apply passive resistance over twice the width of the piles or the spacing of the piles, whichever is smaller.
6. Free drainage assumed behind the wall.
7. Design lagging for 30% of lateral earth pressure.
8. Allowable vertical soldier pile capacity:

Skin friction = 1 ksf
End bearing = 6 ksf (After loose/disturbed soil
at bottom of hole is removed);
For Native Soil
= 4 ksf for Structural Fill

FIG. 5

Proposed Stone Way Apartme
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Seattle, Washington

SINGLE AND MULTIPLE TI
PILE WALL DESIGN CRIT

December 2004 21-1-20

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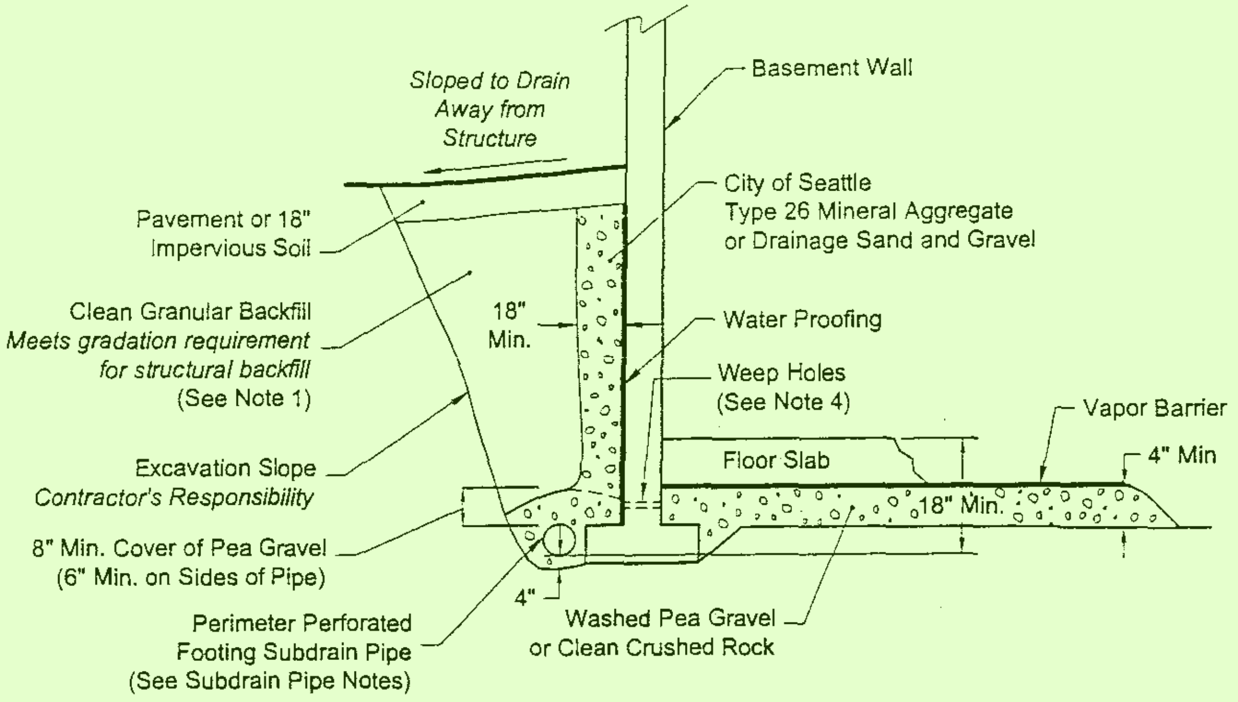

Not to Scale

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

SUBDRAIN PIPE NOTES

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.

Pipe invert should be at least as low as the top of the footing.

NOTES

1. Structural fill should consist of free-draining granular soil with no more than 15% fines, by weight, passing No. 200 sieve based on wet sieving the minus 3/4" fraction, fines to be non-plastic.

2. Backfill behind wall should be compacted with hand-operated vibrating plate compactor (not a tamper). 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.

3. All backfill should be placed in layers not exceeding 8" loose thickness and densely compacted. Beneath paved or sidewalk areas, compact to at least 95% Modified Proctor maximum density (ASTM: D1557). Otherwise compact to 92% minimum.

4. Drainage gravel beneath floor slab could be hydraulically connected to subdrain pipe. Use of 2" diameter weep holes as shown is one applicable method.

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TYPICAL WALL SUBDRAINAGE AND BACKFILLING

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FIG. 7

File: I:\Drafting\21\1\20193 001\21-1-20193-001 Fig 7.dwg Date: 12-09-2004 Author: DRAFTEMP

APPENDIX A

SUBSURFACE EXPLORATIONS

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

S&W CLASSIFICATION OF SOIL CONSTITUENTS

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

MOISTURE CONTENT DEFINITIONS

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

ABBREVIATIONS

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

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 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		

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

SOIL CLASSIFICATION AND LOG KEY

December 2004 21-1-20193-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

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.

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

SOIL CLASSIFICATION AND LOG KEY

December 2004 21-1-20193-001

SHANNON & WILSON, INC.
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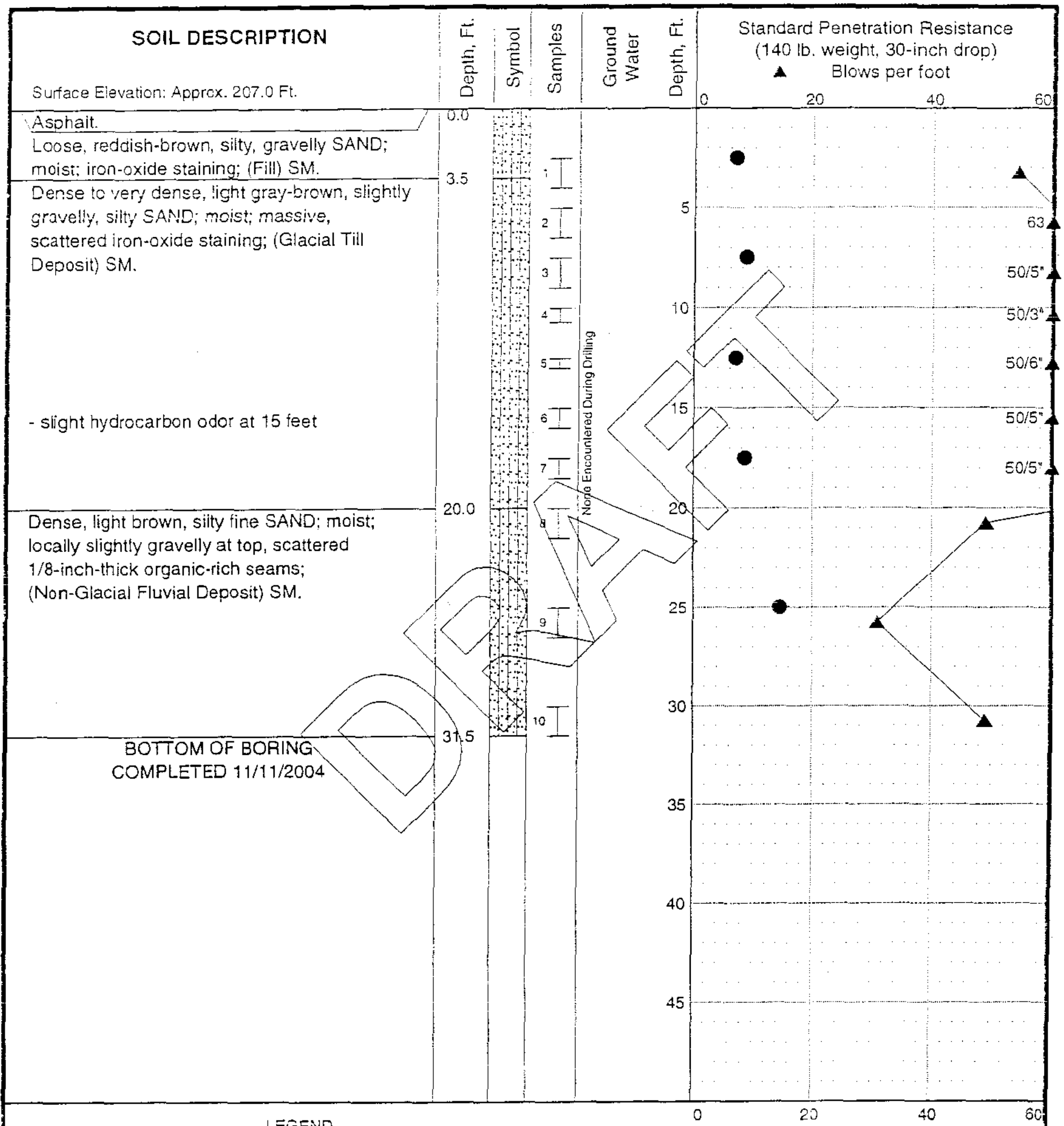
FIG. A-1
Sheet 2 of 2



SOIL DESCRIPTION

Surface Elevation: Approx. 207.0 Ft.

Depth, Ft.	
0.0	Asphalt.
	Loose, reddish-brown, silty, gravelly SAND; moist; iron-oxide staining; (Fill) SM.
3.5	Dense to very dense, light gray-brown, slightly gravelly, silty SAND; moist; massive, scattered iron-oxide staining; (Glacial Till Deposit) SM. - slight hydrocarbon odor at 15 feet
20.0	Dense, light brown, silty fine SAND; moist; locally slightly gravelly at top, scattered 1/8-inch-thick organic-rich seams; (Non-Glacial Fluvial Deposit) SM.
31.5	BOTTOM OF BORING COMPLETED 11/11/2004

LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

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

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

LOG OF BORING B-1

December 2004 21-1-20193-001

SHANNON & WILSON, INC.
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FIG. A-2

MASTER LOG2 21-20193-001.GPJ SHAN_WIL.GDT 12/8/04 Log: CLR Rev: CLR Typ: LKD

SOIL DESCRIPTION

Surface Elevation: Approx. 205.5 Ft.

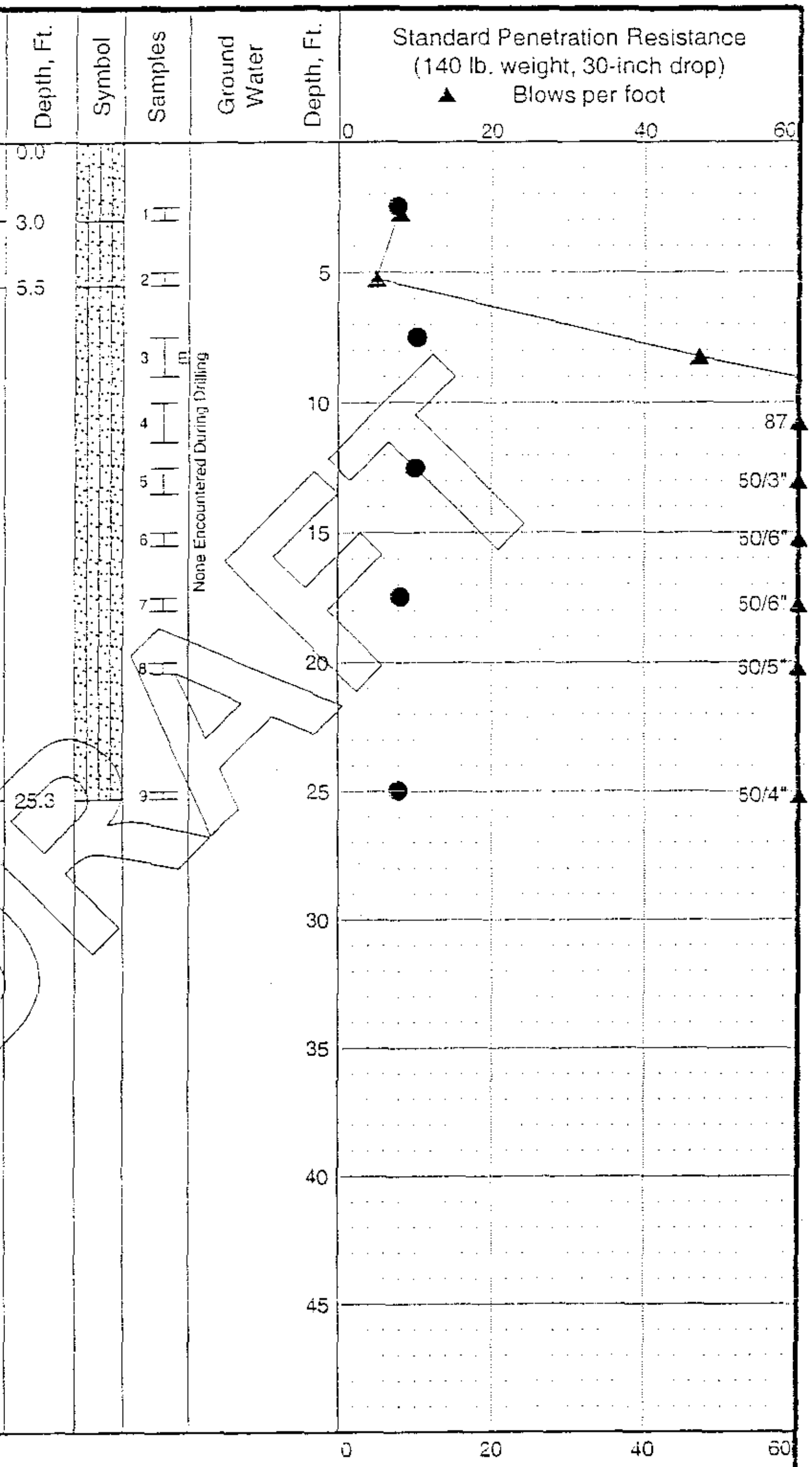
Asphalt.

Loose, brown, gravelly, silty SAND; moist; (Fill) SM.

Loose, reddish-brown, silty, gravelly SAND; moist; (Fill) SM.

Dense to very dense, light gray-brown, slightly gravelly to gravelly, silty SAND; moist; massive, scattered iron-oxide staining; SM.

BOTTOM OF BORING
COMPLETED 11/11/2004



LEGEND
- `*` Sample Not Recovered
- E Environmental Sample Obtained
- ⊥ Standard Penetration Test

● % Water Content

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

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

LOG OF BORING B-2

December 2004 21-1-20193-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-3

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

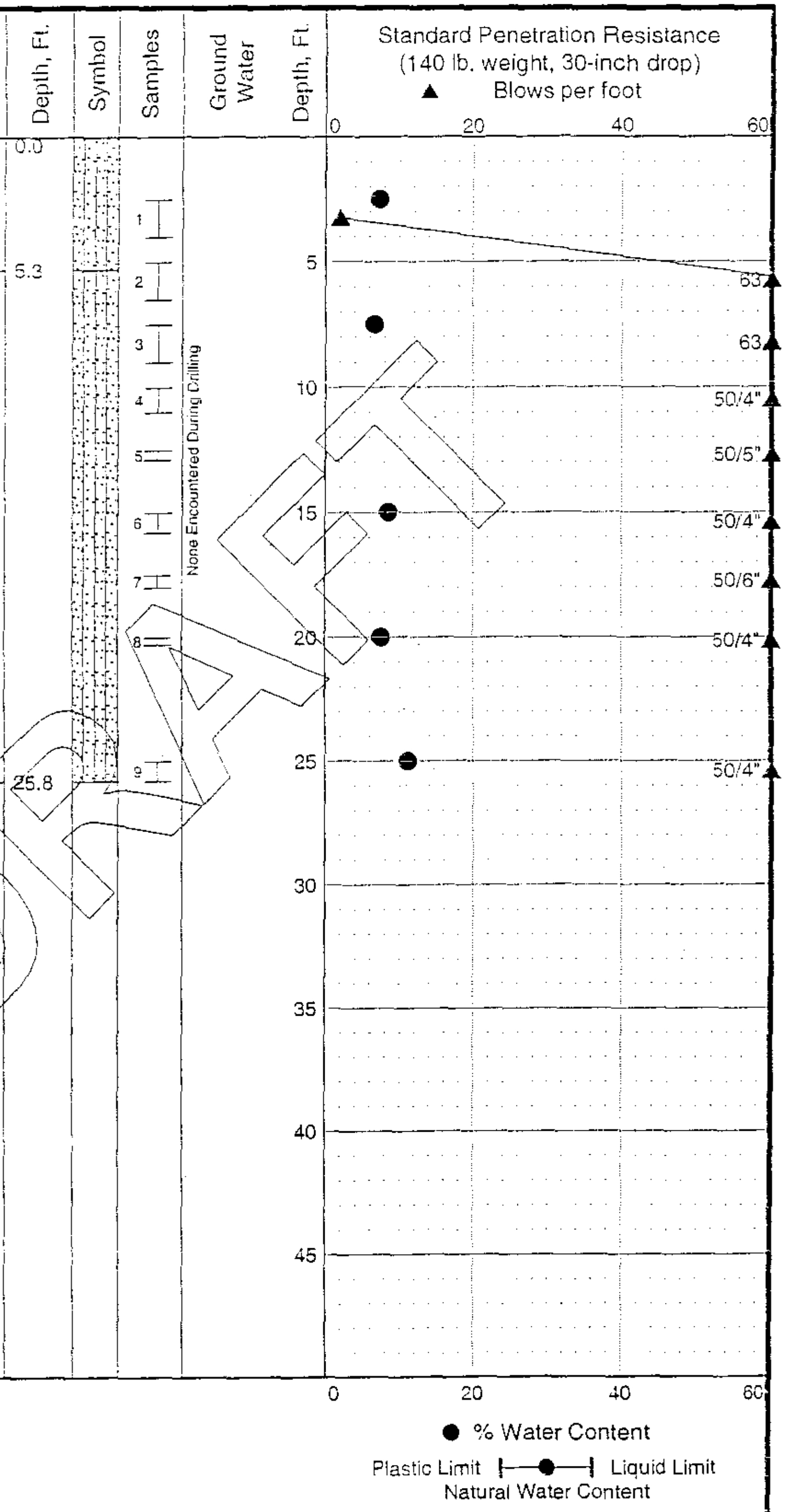
Log: CLR Rev: CLR Typ: LKD

MASTER LOG2 21-20193.GPJ SHAN_WIL.GDT 12/6/04

SOIL DESCRIPTION

Surface Elevation: Approx. 204.7 Ft.

Depth, Ft.	Soil Description
0.0	Asphalt. Very loose, reddish-brown, silty, gravelly SAND; moist; (Fill) SM.
5.3	Very dense, light gray-brown, slightly gravelly to gravelly, silty SAND; moist; massive, locally trace of clay, scattered iron-oxide staining; SM.
25.8	BOTTOM OF BORING COMPLETED 11/11/2004



LEGEND

* Sample Not Recovered

⌶ Standard Penetration Test

NOTES

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

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

LOG OF BORING B-3

December 2004 21-1-20193-001

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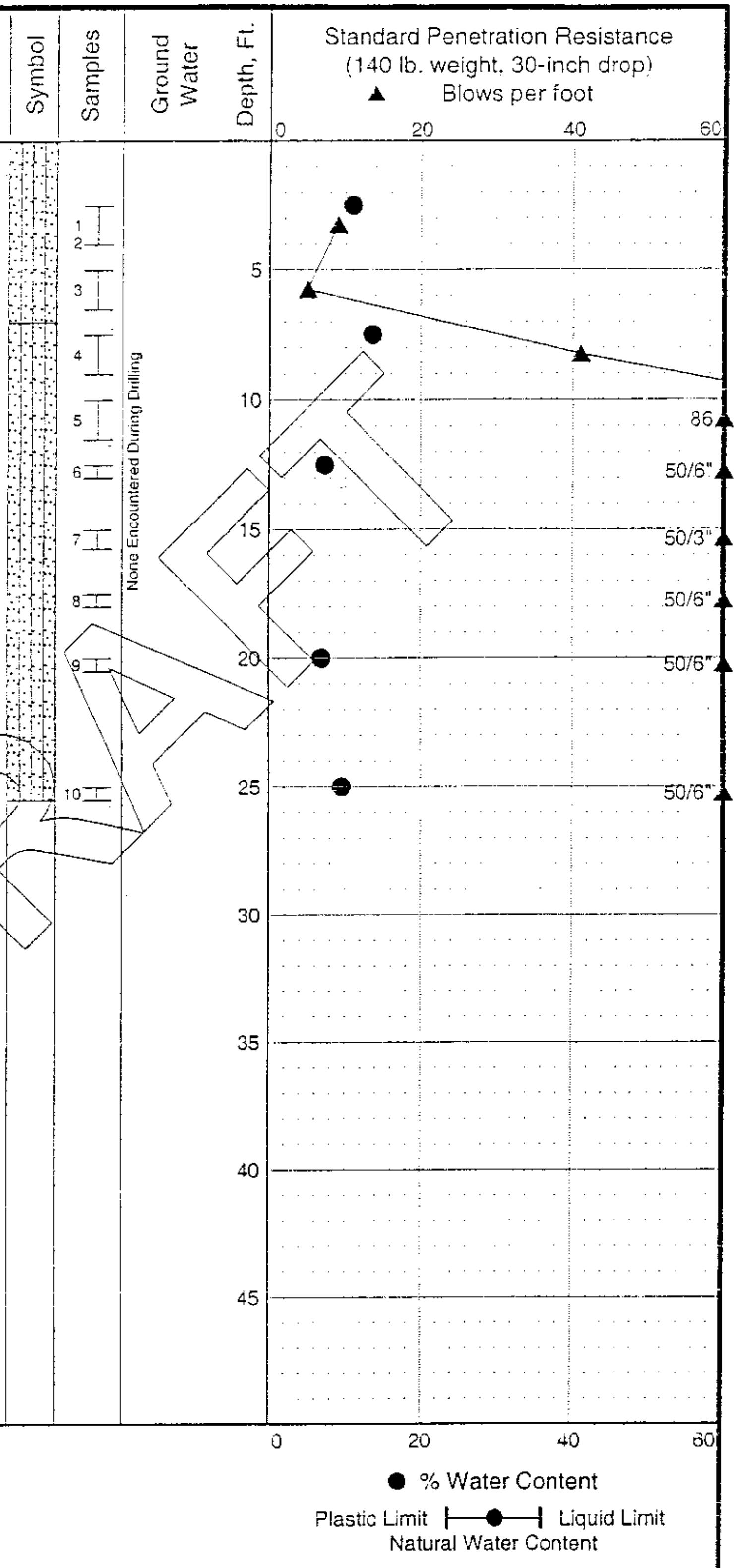
FIG. A-4

SOIL DESCRIPTION Surface Elevation: Approx. 203.0 Ft.	Depth, Ft.
Asphalt.	0.0
Loose, brown, gravelly, silty SAND; moist; scattered organics; (Fill) SM.	
Dense to very dense, light gray-brown, slightly gravelly to gravelly, silty SAND; moist; scattered, abundant iron-oxide staining at top; SM.	7.0
BOTTOM OF BORING COMPLETED 11/11/2004	25.5



LEGEND

* Sample Not Recovered

I Standard Penetration Test

NOTES

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

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

LOG OF BORING B-4

December 2004 21-1-20193-001

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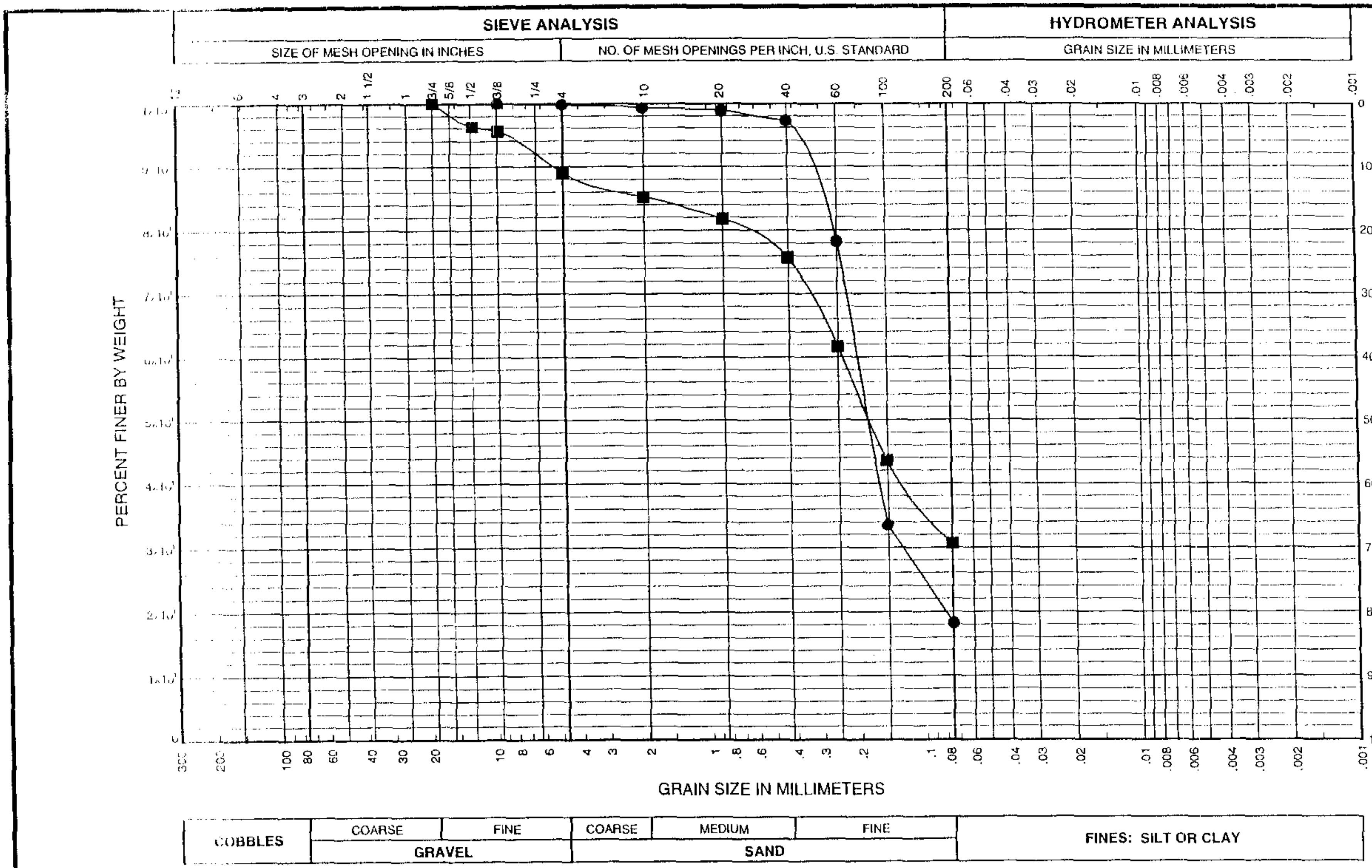
FIG. A-5

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-1, S-9	25.0	SM	Light brown, silty, fine SAND, trace of fine gravel	18.3	14.5			
■ B-3, S-9	25.0	SM	Light gray-brown, slightly gravelly, silty SAND, trace of clay	30.7	11.1			

Proposed Stone Way Apartments
Housing Resources Group
Seattle, Washington

GRAIN SIZE DISTRIBUTION

December 2004 21-1-20193-001

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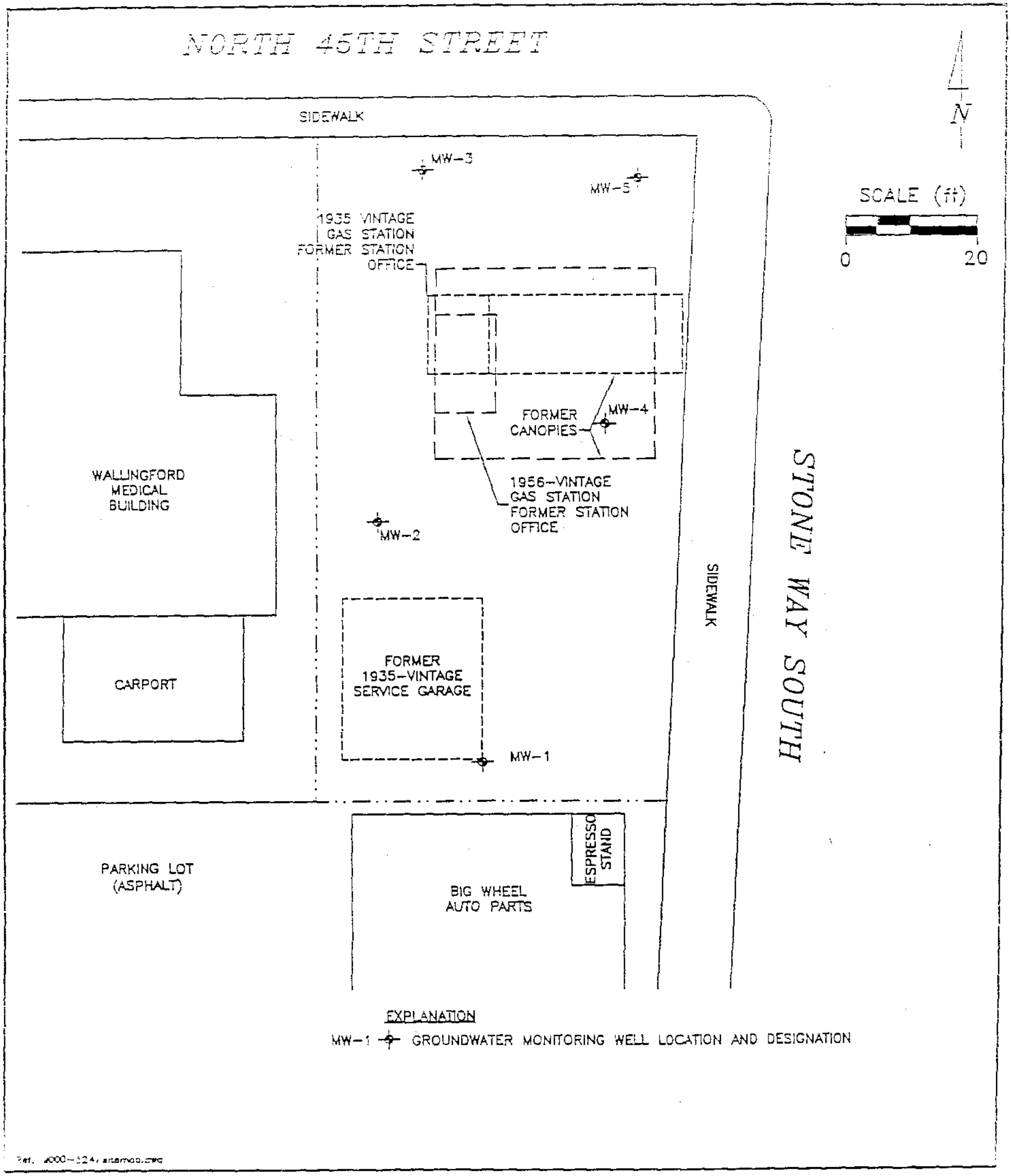
FIG. B-1

FIG. B-1

GSAMAIN1 21-20193.GPJ SHAN_WIL.GDT 12/8/04

APPENDIX C

PREVIOUS BORING LOGS



Environmental Consultants, Inc.

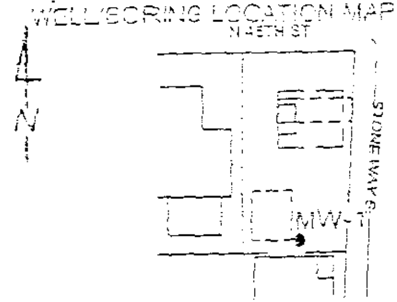
SITE MAP

Former Chevron Service Station #209335
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

FIGURE:
1

PROJECT:
M000-524

WELL/BORING LOCATION MAP



Delta Environmental Consultants, Inc.

WELL/BORING: MW-1

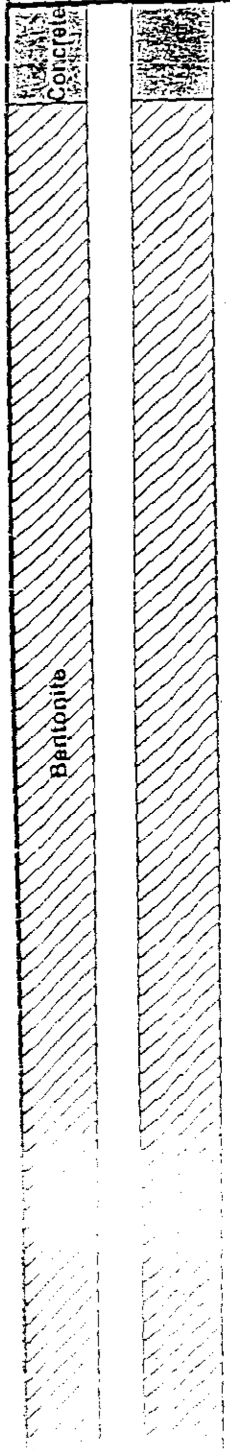
DATE: 10/10/00
PROJECT: M000-524
CLIENT: Chevron 209335
LOCATION: E. of 1225 N. 45th St.
CITY: Wallingford
CO./STATE: WA.
DRILLER: Cascade

DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: DM Split Spoon
BORING DIAMETER: 8"
BORING DEPTH: 42'
WELL CASING: 2" SCH 40 PVC
WELL SCREEN: 32-42' (0.020")
SAND PACK: 30-42' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

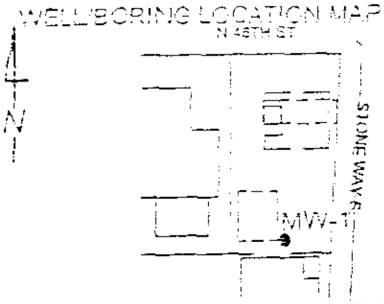
DESCRIPTION/LOGGED BY: Shawn M.

WELL/BORING COMPLETION



▽ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6".	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION
		Dp	9 50	0.0		5	SM	SILTY SAND: brown; 30% fines; 20% very fine to fine sand; medium sand; 5% gravel; medium dense; no TPH odor.
		Dp	70	0.0		10	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
		Dp	60	0.0		15	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
		Dp	32 50	0.0		20	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.

WELL/BORING LOCATION MAP



Delta Environmental Consultants, Inc.

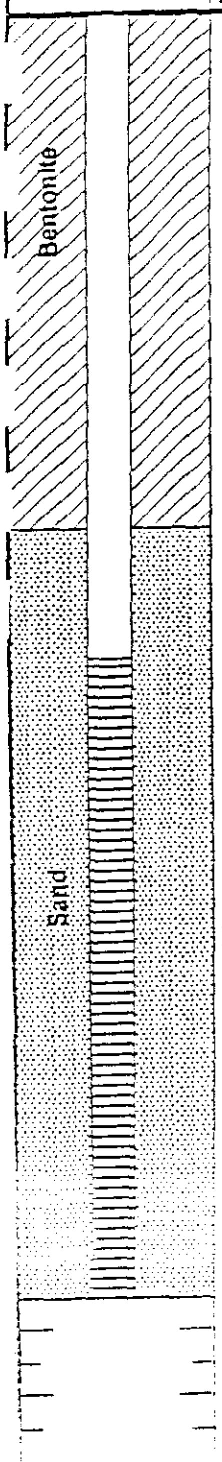
WELL/BORING: MW-1

DATE: 10/10/00
PROJECT: M000-524
CLIENT: Chevron 209335
LOCATION: E. of 1225 N. 45th St.
CITY: Wallingford
CO./STATE: WA.
DRILLER: Cascade

DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: DM Split Spoon
BORING DIAMETER: 8"
BORING DEPTH: 42'
WELL CASING: 2" SCH 40 PVC
WELL SCREEN: 32-42' (0.020")
SAND PACK: 30-42' (#10/20)

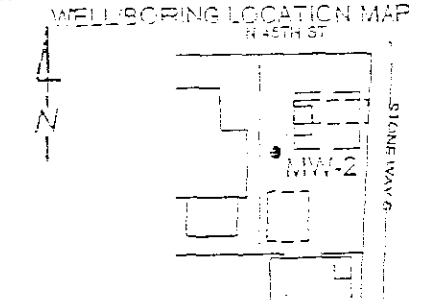
WATER LEVEL:			
TIME:			
DATE:			

WELL/BORING COMPLETION



▽ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6".	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION/LOGGED BY: Shawn M.
		M	50	0.0		25	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
		M	32 50	0.0		30	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
		W	35 50	0.0		35	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
▽						36		
			[illegible] 50	0.0		40	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						42		Bottom of boring @ 42'.

WELL/BORING LOCATION MAP



Delta Environmental Consultants, inc.

WELL/BORING: MW-2

DATE: 10/11/00

DRILLING METHOD: Hollow Stem Auger

PROJECT: M000-524

SAMPLING METHOD: DM Split Spoon

CLIENT: Chevron 209335

BORING DIAMETER: 8"

LOCATION: E. of 1225 N. 45th St.

BORING DEPTH: 43'

CITY: Wallingford

WELL CASING: 2" SCH 40 PVC

CO./STATE: WA.

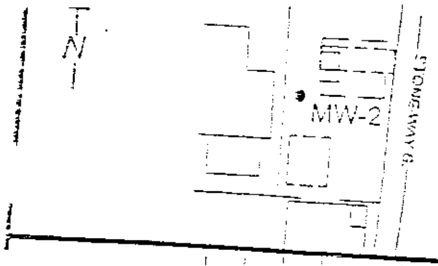
WELL SCREEN: 32-42' (0.010")

DRILLER: Cascade

SAND PACK: 30-43' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

WELL/BORING COMPLETION	▽ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	RECOVERY	SAMPLE INTERVAL	GRAPHIC	USCS SYMBOL	DESCRIPTION/LOGGED BY: Shawn M.
Concrete							1					
Bentonite							2					
							3					
							4					
			Dp	8 16 16	0.0		5				SM	SILTY SAND: reddish brown; 25% fines; very fine to fine sand; 20% medium to coarse sand; 5% gravel; iron oxide staining; dense; no TPH odor.
							6					
							7					
							8					
							9					
			Dp	21 34 34	0.0		10				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; medium dense; no TPH odor.
							11					
							12					
							13					
							14					
			Dp	24 47 31	0.0		15				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; medium dense; no TPH odor.
							16					
							17					
							18					
							19					
				27 50	0.0		20				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; medium dense; no TPH odor.
							21					
							22					



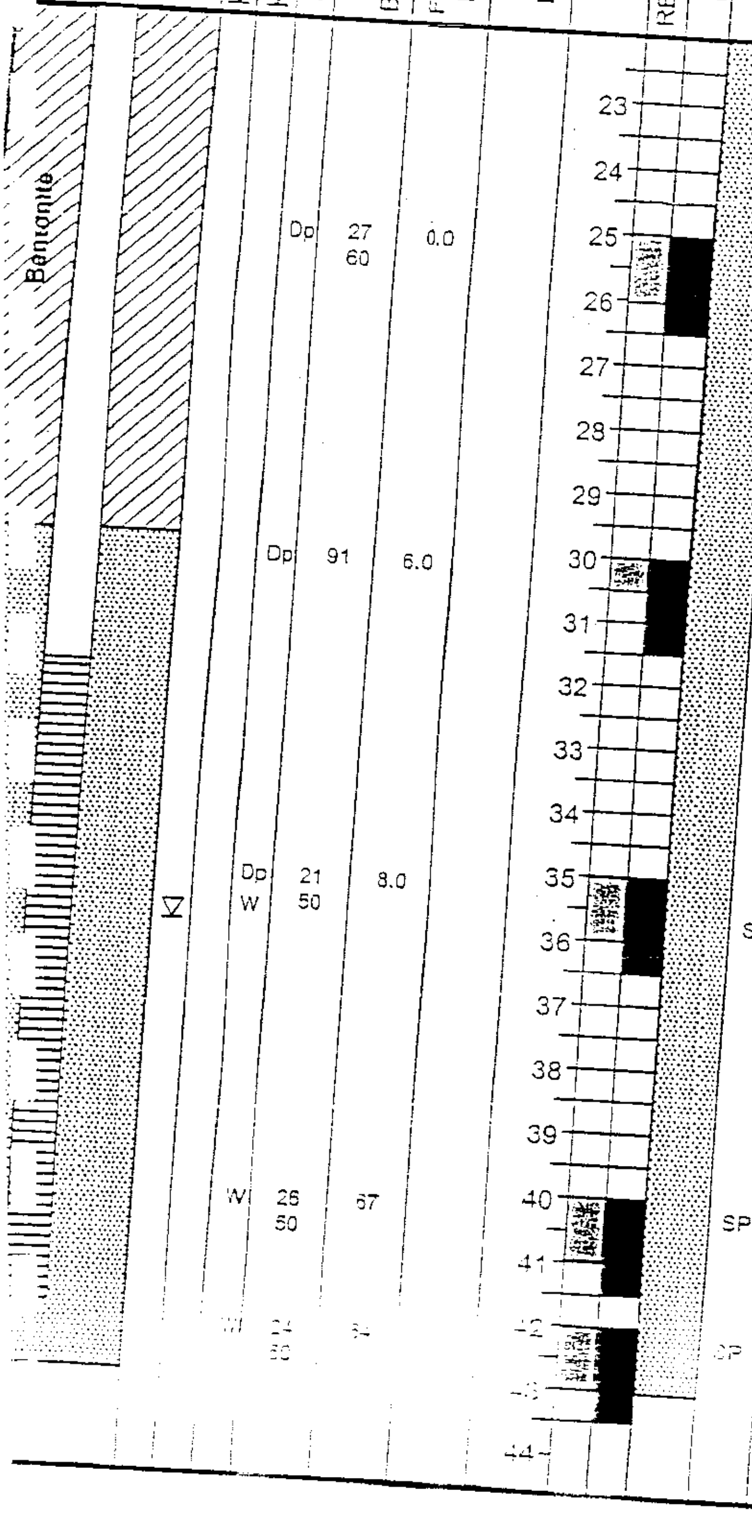
ants, Inc.

DATE: 10/11/00
PROJECT: MC00-524
CLIENT: Chevron 209335
LOCATION: E. of 1225 N. 45th St.
CITY: Wallingford
CO./STATE: WA.
DRILLER: Cascade

WELL/BORING: MW-2
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: DM Split Spoon
BORING DIAMETER: 8"
BORING DEPTH: 43'
WELL CASING: 2" SCH 40 PVC
WELL SCREEN: 32-42' (0.010")
SAND PACK: 30-43' (#10/20)

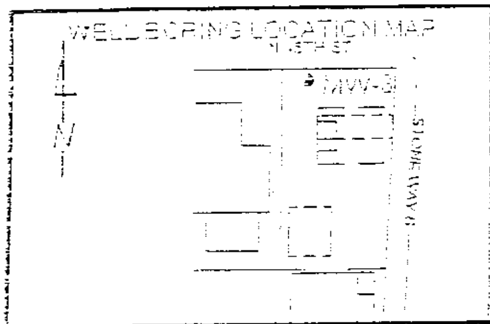
WATER LEVEL:			
TIME:			
DATE:			

DESCRIPTION/LOGGED BY: Shawn M.



∇ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION
						23		
						24		
		Dp	27 60	0.0		25	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; medium dense; no TPH odor.
						26		
						27		
						28		
						29		
		Dp	91	6.0		30	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						31		
						32		
						33		
						34		
∇		Dp W	21 50	8.0		35	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						36		
						37		
						38		
						39		
		W	26 50	67		40	SP	SAND: gray; 5% fines; 95% very fine to fine sand; dense; strong TPH odor.
						41		
		W	24 50	64		42	SP	SAND: gray; 5% fines; 95% very fine to fine sand; dense; strong TPH odor.
						43		
						44		Bottom of boring @ 43'.

WELL/BORING LOCATION MAP



Delta Environmental Consultants, Inc.

WELL/BORING: MW-3

DATE: 10/11/00	DRILLING METHOD: Hollow Stem Auger
PROJECT: M000-524	SAMPLING METHOD: DM Split Spoon
CLIENT: Chevron 209335	BORING DIAMETER: 8"
LOCATION: E. of 1225 N. 45th St.	BORING DEPTH: 45.5'
CITY: Wallingford	WELL CASING: 2" SCH 40 PVC
CO./STATE: WA.	WELL SCREEN: 35-45' (0.010")
DRILLER: Cascade	SAND PACK: 33-45.5' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

DESCRIPTION/LOGGED BY: Shawn M.

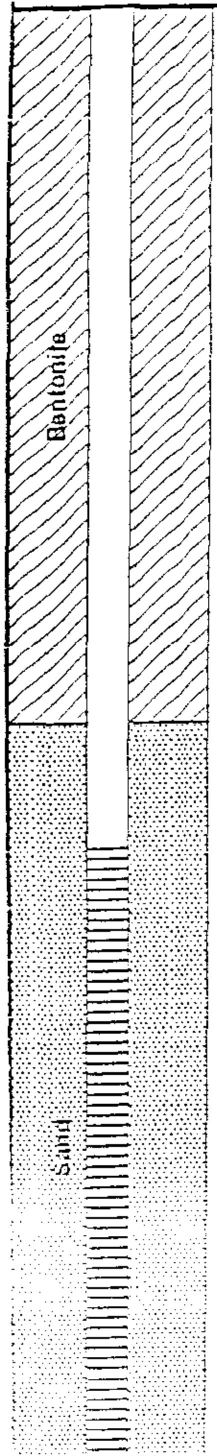
WELL/BORING COMPLETION	▽ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	RECOVERY	SAMPLE INTERVAL	GRAPHIC	USCS SYMBOL	DESCRIPTION
Concrete							1					
Bentonite							2					
							3					
							4					
			Dp	8 10 30	0.0		5 6				SM	SILTY SAND: brown; 35% fines; 30% very fine to fine sand; medium sand; 5% gravel; loose to medium dense; no TPH odor.
							7					
							8					
							9					
			Dp	100/3	0.0		10 11				SM	SILTY SAND: brown; 35% fines; 30% very fine to fine sand; medium sand; 5% gravel; very dense; no TPH odor.
							12					
							13					
							14					
			Dp	35 50	0.0		15 16				SM	SILTY SAND: brown; 35% fines; 30% very fine to fine sand; medium sand; 5% gravel; very dense; no TPH odor.
	▽						17					
							18					
							19					
			Dp	30 50	0.0		20 21				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.
							22					

WELL/BORING LOCATION MAP	Delta Environmental Consultants, Inc.	WELL/BORING: MW-3
	DATE: 10/11/00	DRILLING METHOD: Hollow Stem Auger
	PROJECT: M000-524	SAMPLING METHOD: DM Split Spoon
	CLIENT: Chevron 209335	BORING DIAMETER: 8"
	LOCATION: E. of 1226 N. 45th St.	BORING DEPTH: 45.5'
	CITY: Wallingford	WELL CASING: 2" SCH 40 PVC
	CO./STATE: WA.	WELL SCREEN: 35-45' (0.010")
	DRILLER: Cascade	SAND PACK: 33-45.5' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

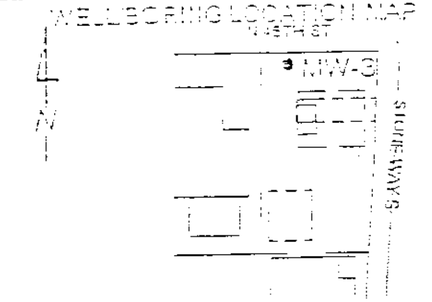
DESCRIPTION/LOGGED BY: Shawn M.

WELL/BORING COMPLETION



▽ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION
		Dp	50	0.0		25	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.
		Dp	40 50	0.0		30	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.
		Dp	50	0.0		35	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.
▽						39		
		W	40 [illegible] [illegible]	0.0		40	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.

WELL/BORING LOCATION MAP



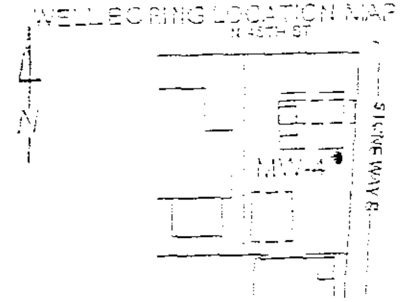
Delta Environmental Consultants, Inc.	WELL/BORING: MW-3
DATE: 10/11/00	DRILLING METHOD: Hollow Stem Auger
PROJECT: M000-524	SAMPLING METHOD: 2" Split Spoon
CLIENT: Chevron 209335	BORING DIAMETER: 8"
LOCATION: E. of 1225 N. 45th St.	BORING DEPTH: 45.5'
CITY: Wallingford	WELL CASING: 2" SCH 40 PVC
CO./STATE: WA.	WELL SCREEN: 35-45' (0.010")
DRILLER: Cascade	SAND PACK: 33-45.5' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

DESCRIPTION/LOGGED BY: Shawn M.

WELL/BORING COMPLETION	FIRST	STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	RECOVERY	SAMPLE INTERVAL	GRAPHIC	USCS SYMBOL	DESCRIPTION
			W	60	45		45				SP	SAND: grayish brown: 5% fines: 95% very fine to fine sand: no TPH odor.
							46					Bottom of boring @ 45.5'.
							47					
							48					
							49					
							50					
							51					
							52					
							53					
							54					
							55					
							56					
							57					
							58					
							59					
							60					
							61					
							62					
							63					
							64					
							65					
							66					

WELL/BORING LOCATION MAP



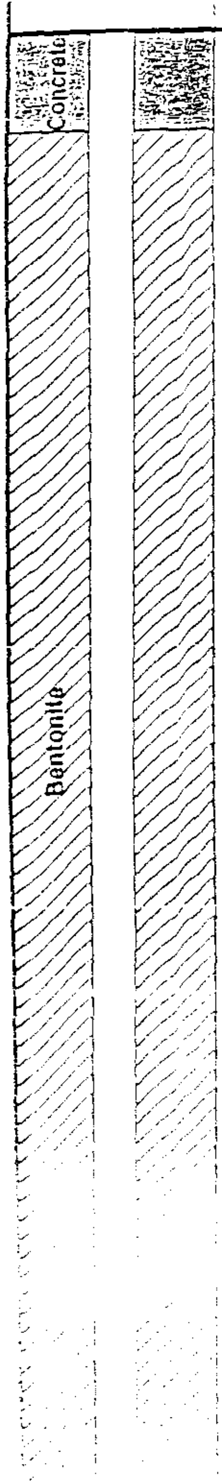
Delta Environmental Consultants, Inc.

WELL/BORING: MW-4

DATE: 10/10/00	DRILLING METHOD: Hollow Stem Auger
PROJECT: M000-524	SAMPLING METHOD: DM Split Spoon
CLIENT: Chevron 209335	BORING DIAMETER: 8"
LOCATION: E. of 1225 N. 45th St.	BORING DEPTH: 43'
CITY: Wallingford	WELL CASING: 2" SCH 40 PVC
CO./STATE: WA.	WELL SCREEN: 32-42' (0.020")
DRILLER: Cascade	SAND PACK: 30-43' (#10/20)

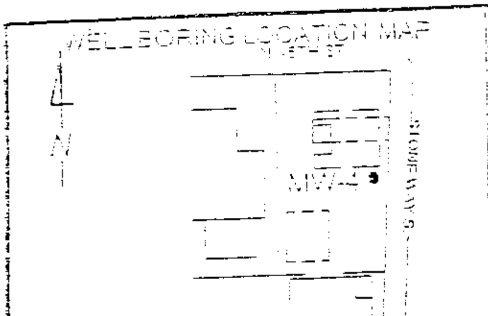
WATER LEVEL:			
TIME:			
DATE:			

DESCRIPTION/LOGGED BY: Shawn M.



FIRST	STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION
						1		
						2		
						3		
						4		
		Dp	55	0.0		5	SM	SILTY SAND: brown; 30% fines; very fine to fine sand; 20% medium to coarse sand; 5% gravel; dense; no TPH odor.
						6		
						7		
						8		
						9		
		Dp	60	0.0		10	SM	SILTY SAND: brownish gray; 15% fines; 85% very fine to fine sand; dense; no TPH odor.
						11		
						12		
						13		
						14		
		Dp	32 50	2.0		15	SP	SAND: brownish gray; 5% fines; 95% very fine to fine sand; medium dense; no TPH odor.
						16		
						17		
						18		
						19		
			32 50			20	SP	SAND: brownish gray; 5% fines; 95% very fine to fine sand; medium dense; no TPH odor.
						21		
						22		

WELL/BORING LOCATION MAP



Delta Environmental Consultants, Inc.

WELL/BORING: MW-4

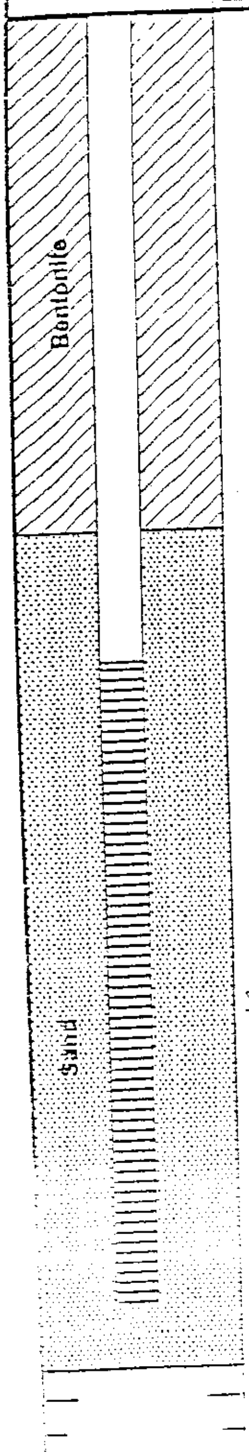
DATE: 10/10/00
PROJECT: M000-524
CLIENT: Chevron 209335
LOCATION: E. of 1225 N. 45th St.
CITY: Wallingford
CO./STATE: WA.
DRILLER: Cascade

DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: DM Split Spoon
BORING DIAMETER: 8"
BORING DEPTH: 43'
WELL CASING: 2" SCH 40 PVC
WELL SCREEN: 32-42' (0.020")
SAND PACK: 30-43' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

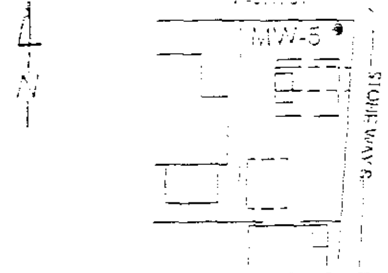
DESCRIPTION/LOGGED BY: Shawn M.

WELL/BORING COMPLETION



∇ FIRST	▼ STABILIZED	MOISTURE	DENSITY BLOWS / 6"	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	RECOVERY	SAMPLE INTERVAL	GRAPHIC	USCS SYMBOL	DESCRIPTION/LOGGED BY: Shawn M.
						23					
						24					
		Dp	50	0.0		25				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						26					
						27					
						28					
						29					
		Dp	50	0.0		30				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						31					
						32					
						33					
						34					
		Dp	30 50	5.0		35				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						36					
						37					
∇											
						38					
						39					
		[illegible]	[illegible]	[illegible]		40				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						41					
		[illegible]	40 50	0.0		42				SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; dense; no TPH odor.
						43					Bottom of boring @ 43'.
						44					

WELL/BORING LOCATION MAP



Delta Environmental Consultants, Inc.	WELL/BORING: MW-5
DATE: 10/11/00	DRILLING METHOD: Hollow Stem Auger
PROJECT: M000-524	SAMPLING METHOD: DM Split Spoon
CLIENT: Chevron 209335	BORING DIAMETER: 8"
LOCATION: E. of 1225 N. 45th St.	BORING DEPTH: 43'
CITY: Wallingford	WELL CASING: 2" SCH 40 PVC
CO./STATE: WA.	WELL SCREEN: 32-42' (0.010")
DRILLER: Cascade	SAND PACK: 30-43' (#10/20)

WATER LEVEL:			
TIME:			
DATE:			

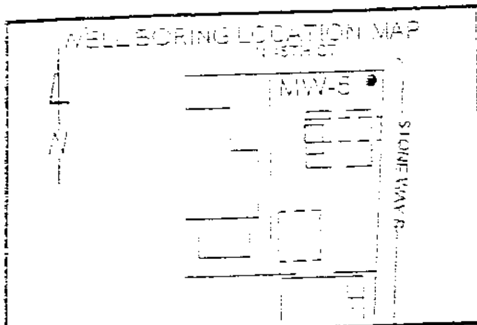
DESCRIPTION/LOGGED BY: Shawn M.

WELL/BORING COMPLETION	FIRST	STABILIZED	MOISTURE	DENSITY BLOWS / 6".	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION
Concrete							1		
							2		
							3		
							4		
			D	16 21 21	24		5	SM	SILTY SAND: brown; 20% fines; very fine to fine sand;15% medium to coarse sand; 5% cobble; medium dense; no TPH odor; trace iron oxide staining.
							6		
							7		
							8		
							9		
			Dp	16 19 20	67		10	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; slight TPH odor.
Bentonite							11		
							12		
							13		
							14		
				50	83		15	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; slight TPH odor.
							16		
							17		
							18		
							19		
				50	[illegible]		20	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; slight TPH odor.
							21		
							22		

Delta Environmental Consultants, Inc.

WELL/BORING: MW-5

WELL/BORING LOCATION MAP



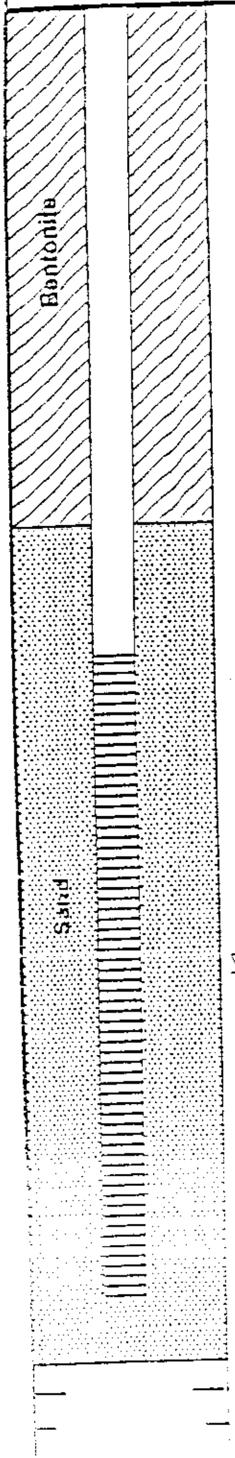
DATE: 10/11/00
PROJECT: M000-524
CLIENT: Chevron 209335
LOCATION: E. of 1225 N. 45th St.
CITY: Wallingford
CO./STATE: WA.
DRILLER: Cascade

DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: DM Split Spoon
BORING DIAMETER: 8"
BORING DEPTH: 43'
WELL CASING: 2" SCH 40 PVC
WELL SCREEN: 32-42' (0.010")
SAND PACK: 30-43' (#10/20)

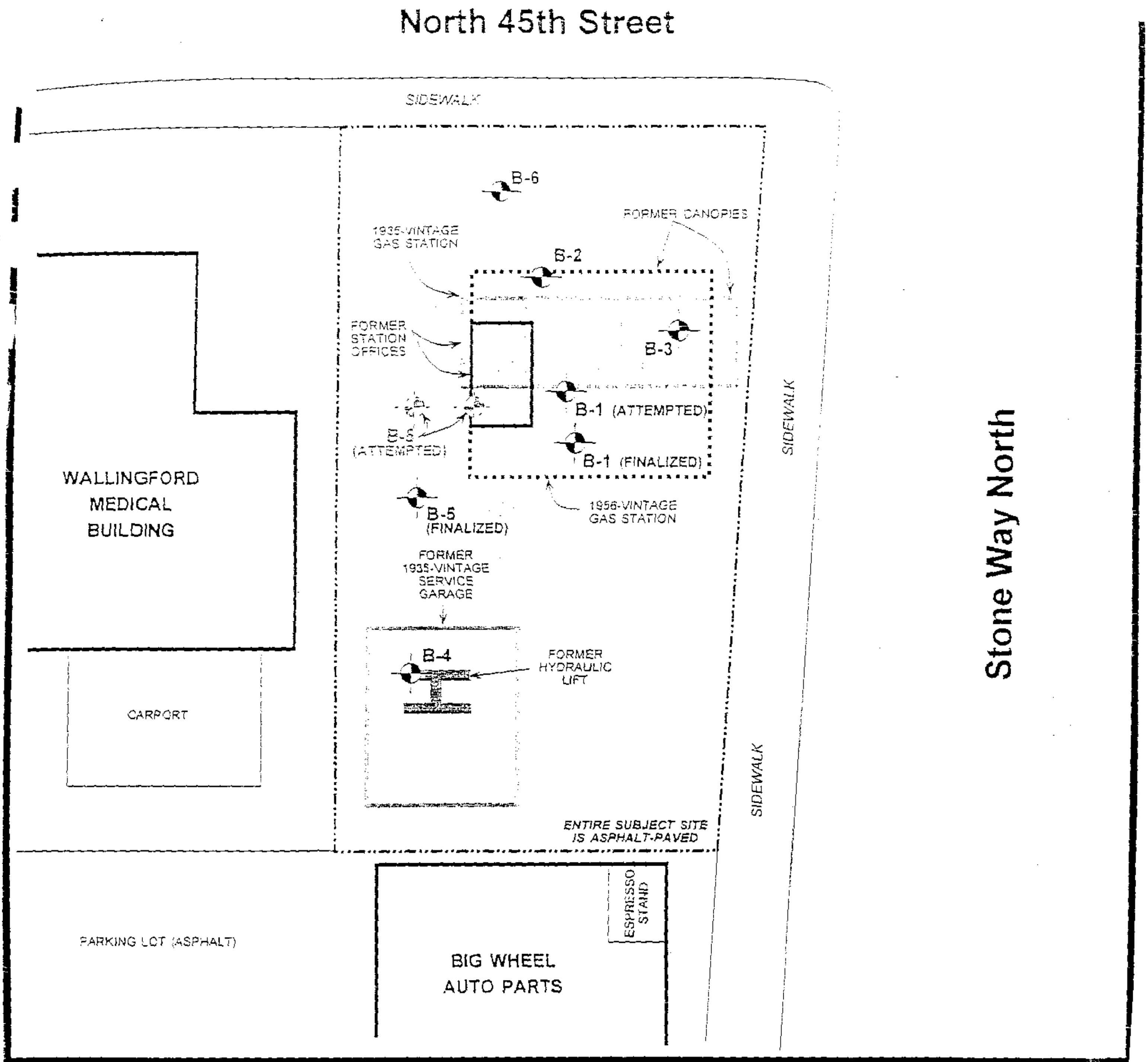
WATER LEVEL:			
TIME:			
DATE:			

DESCRIPTION/LOGGED BY: Shawn M.

WELL/BORING COMPLETION



FIRST	STABILIZED	MOISTURE	DENSITY BLOWS / 6".	FIELD TEST PID (ppm)	SAMPLE NUMBER	DEPTH (FEET)	USCS SYMBOL	DESCRIPTION/LOGGED BY: Shawn M.
						23		
						24		
		Dp	30 50	73		25	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; slight TPH odor.
						26		
						27		
						28		
						29		
		Dp	50	64		30	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; slight TPH odor.
						31		
						32		
						33		
						34		
		Dp	50	59		35	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; slight TPH odor.
						36		
⊻						37		
						38		
						39		
			[illegible] 55			40	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.
						41		
			[illegible] [illegible]			42	SP	SAND: grayish brown; 5% fines; 95% very fine to fine sand; no TPH odor.
						43		Bottom of boring @ 43'.
						44		



Legend

☐ Current buildings - former buildings shown in gray shades (approximate locations)

B-1 Approximate location of EAI borings completed August 29, 1999

Subject property boundary

Other property line

B-5 Approximate location of EAI borings completed September 21, 1999 (B-5, B-6)

Probable direction of shallow-seated groundwater flow

N

Approximate Scale
0 20 ft.

ENVIRONMENTAL ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

SITE PLAN

Former Gasoline Station
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

Job Number:	Date:		Plate:
JN 9301-2	September 1999		2

BORING B-1

Depth/ Sample	Well Design	Moisture Content (%) Water Table	Blows/ Foot	USCS	DESCRIPTION
0					Surface: 2" asphalt over reddish brown gravelly sand (FILL), medium grained, gravel to 1.5".
1	MONITORING WELL NOT INSTALLED	GROUNDWATER NOT ENCOUNTERED	32	F (SW)	1- As above at first attempt, very little recovery due to clast in sampler (9:35 - 8/24). Sample B-1-1A: As above at second attempt, dense, damp. (8:50 - 8/29) @ 3.5' on first attempt, <u>very</u> large concrete clast encountered.
2			>50	SP	2- As above at first attempt, moderate odor of hydrocarbons, (10:15 - 8/24). Second attempt: Upper half: As above. Lower half: SAND, brownish gray, medium grained, minor silt, damp, very dense, no odors. (9:02 - 8/29)
3			>50		3- As lower part of sample 2 above during second attempt at drilling, minor gravel to 3/4", damp, very dense, faint odor of hydrocarbons. (9:15 - 8/29)
4			>50		4- As sample 3 above, medium-fine grained, damp, very dense, no discernible odors. (9:30 - 8/29)
5			>50		5- As sample 4 above, extremely faint odor of hydrocarbons, damp, very dense. (9:40 - 8/29)

* Boring initially located at magnetometer survey coordinates 35 east, 58 north on August 24, 1999. Boring located south of the former gasoline stations in an area exhibiting a magnetic anomaly (see Appendix A). Encountered a very large gravel clast at 3.5 feet which pushed auger to the site. Continued drilling and obtained the sample 7.5 feet. At 10 feet, a very large gravel clast or concrete rubble was encountered which broke the universal joint connecting the drive mechanism to the auger string. We returned to the site on August 29, 1999 to complete the drilling of B-1 at magnetometer coordinates 36 east, 51 north. First two samples (2.5 and 7.5 feet in depth) on this date were labeled B-1-1A and B-1-2A. Later samples designated B-1-3, B-1-4, and B-1-5.

* Boring terminated at 24.0 feet on August 29, 1999.

* Monitoring well not installed, groundwater not encountered

* No visual indications of contamination. Olfactory indications of contamination noted in samples B-1-2, B-1-3, and B-1-5.

* Numbers in parentheses indicate the time the sample was obtained.

ENVIRONMENTAL
ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

TEST BORING LOG: B-1

Former Gasoline Station
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

Job Number:	Date:	Logged by:	Plate:
JN 9301-2	September 1999	D. Holmes	3

BORING B-2

Depth/ Sample	Well Design	Moisture Content (%) Water Table	Blows/ Foot	USCS	DESCRIPTION
0					Surface: 2" asphalt over reddish brown gravelly sand (FILL), medium grained, gravel to 1.5".
1	MONITORING WELL NOT INSTALLED	GROUNDWATER NOT ENCOUNTERED	34	F (SW)	1- As above, with burnt wood fragments noted, damp, dense, no odors noted. (10:15)
5					
2			>50		2- As above, no wood fragments noted, damp, very dense, odor of hydrocarbons noted. (10:20)
10					
					@ 11' - gray medium fine sand encountered in cuttings.
3			>50		3- SAND, gray, medium-fine grained, damp, very dense, noticeable odor of hydrocarbons. (10:30)
15				SP	
4			>50		4- As sample 3 above, damp, very dense, odor of hydrocarbons noted. (10:45)
20					
5			>50		5- As samples 3 and 4 above, damp, very dense, odor of hydrocarbons noted. (10:50)

- * Boring located at magnetometer survey coordinates 32 east, 73 north on August 29, 1999. Boring located north of the former gasoline stations in an area exhibiting a magnetic anomaly.
- * Boring terminated at 24.0 feet on August 29, 1999.
- * Monitoring well not installed, groundwater not encountered
- * No visual indications of contamination. Olfactory indications of contamination noted in all samples obtained.

ENVIRONMENTAL
ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

TEST BORING LOG: B-2

Former Gasoline Station
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

Job Number:	Date:	Logged by:	Plate:
JN 9301-2	September 1999	D. Holmes	4

BORING B-3

	Depth/ Sample	Well Design	Moisture Content (%) Water Table	Blows/ Foot	USCS	DESCRIPTION
0		MONITORING WELL NOT INSTALLED	GROUNDWATER NOT ENCOUNTERED		F (SW)	Surface: 2" asphalt over reddish brown gravelly sand (FILL), medium grained, gravel to 1.5".
	1			24		1- As above, faint odor of hydrocarbons, damp, medium dense. (11:15)
5						
	2			50		2- Upper 9": As above, no wood fragments noted, damp, very dense, faint odor of hydrocarbons.
10					SP	Lower 9": SAND, brownish gray, medium-fine grained, damp, very dense, no odors noted. (11:25)
	3			>50		3- As above, faint bedding visible, damp, very dense, no odors noted. (11:30)

- Boring located at magnetometer survey coordinates 51 east, 66 north on August 29, 1999. Boring located in an area near a possible former pump island.
- Boring terminated at 14.0 feet on August 29, 1999.
- Monitoring well not installed, groundwater not encountered
- No visual indications of contamination. Olfactory indications of contamination noted only in sample B-3-1 and upper portion of B-3-2.
- Numbers in parentheses indicate the time the sample was obtained.

ENVIRONMENTAL
ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

TEST BORING LOG: B-3

Former Gasoline Station
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

Job Number:	*Date:*	*Logged by:*	*Plate:*
JN 9301-2	September 1999	D. Holmes	5

BORING B-4

Depth/ Sample	Well Design	Moisture Content (%) Water Table	Blows/ Foot	USCS	DESCRIPTION
0					Surface: 2" asphalt over reddish brown gravelly sand (FILL), medium grained, gravel to 1.5".
1	MONITORING WELL NOT INSTALLED	GROUNDWATER NOT ENCOUNTERED	41	F (SW)	1- As above, damp, dense, no odors noted. (12:45) @ 5' - very gravelly interval with metal fragments in cuttings, "easy" drilling once through this thin interval.
5					
2			40	SP	2- SAND, grayish brown, medium-fine grained, very minor pea-sized gravel, minor horizontal oxidized layers (weathered till?), damp, very dense, no odors noted. (12:55)

- * Boring located at magnetometer survey coordinates 14 east, 19 north. Boring located beneath the former service garage.
- * Boring terminated at 9.0 feet on August 29, 1999.
- * Monitoring well not installed, groundwater not encountered
- * No visual or olfactory indications of contamination.
- * Numbers in parentheses indicate the time the sample was obtained.

ENVIRONMENTAL
ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

TEST BORING LOG: B-4

Former Gasoline Station
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

Job Number:	Date:	Logged by:	Plate:
JN 9301-2	September 1999	D. Holmes	6

Boring B-5

Depth/ Sample	Well Design	Moisture Content (%)/ Water Table	Blows/ Foot	USCS	DESCRIPTION
0	MONITORING WELL NOT INSTALLED				Surface: 2" asphalt over reddish brown gravelly sand (FILL) to 8" at 1st attempt & 3' at 2nd attempt, where concrete slabs were encountered. Fill to 5' at 3rd attempt
1			15	F	1- FILL, as above with asphalt clasts, medium grained, gravel to 5/8", damp, medium dense. (11:35).
2			>50	SP	2- SAND, grayish brown, medium fine grained, sparse pea gravel, no odors, damp, very dense. (11:45)
3			49		3- SAND, as above, slightly grayer, no gravel, no odors noted, damp, dense to very dense. (11:50)
4			>50		4- SAND, as above, no gravel, no odors, damp, very dense. (11:55)
5			>50	SP	5- SAND, as above, gray-brown, damp, very dense. (12:20)
6			>50		6- SAND, as above, extremely faint ("sweet") odor dissimilar to hydrocarbons noted, damp, very dense. (12:25)
7			>50		7- SAND, as above, extremely faint ("sweet") odor dissimilar to hydrocarbons noted, damp, very dense. (12:30)
8		▼	38	SP	8- SAND, as above, dark gray stained upper 2" of sample less stained, extremely strong odor of hydrocarbons, wet, dense. (12:40)
9			>50		9- SAND, similar to sample 8, gray at top, brown at base, slightly siltier at base, strong of odor of hydrocarbons in the upper few inches, lower part very faint odor of hydrocarbons, wet, very dense. (12:50)

- Boring attempted at magnetometer coordinates 22 east, 55 north (refusal on concrete slab at 10"), and at 15 east, 55 north (concrete slab at three feet). Completed at coordinates 15 east, 45 north.
- Boring terminated at 44 feet on September 21, 1999.
- Groundwater encountered at a depth of approximately 37.5 feet. Monitoring well not installed. Groundwater sample B-5-W obtained at 13:10.
- Indications of hydrocarbon contamination (very strong odor and pronounced staining) noted in samples 8 and 9 at depths between 37.5 feet and 44 feet.
- Numbers in parentheses represent the time sample was obtained.

ENVIRONMENTAL
ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

TEST BORING LOG: B-5

Former Gasoline Station
Vacant Lot East of 1225 North 45th Street
Seattle (Wallingford), Washington

Job Number:	Date:	Logged by:	Plate:
JN 9301-2	September 1999	D. Holmes	3

BORING B-6

Depth/ Sample	Well Design	Moisture Content (%) Water Table	Blows/ Foot	USCS	DESCRIPTION
0					Surface: 2" asphalt over reddish brown gravelly sand (FILL), medium grained, gravel to 1.5", damp, fill to approximately 11'.
1	MONITORING WELL NOT INSTALLED	GROUNDWATER NOT ENCOUNTERED	>50	F (SW)	1- As above, with wood (cedar) in sample tip, very little recovery, no sample obtained, 3 inches penetration, dense.
5					
2			>50		2- As sample 1 above, gravel to 2", damp, very dense, odor of cedar, 12" penetration. (14:35)
10					@ 11' - brown medium fine sand encountered in cuttings.
3			48		3- SAND, brown, medium-fine grained, damp, dense, "sweet" odor not similar to hydrocarbons noted. (14:45)
15				SP	@ 15' - rig bounce, gravel in cuttings (gravelly sand)
4			>50		4- SAND, with pea gravel, brown, medium fine grained, damp to slightly moist, very dense, odor similar to sample 3 above noted, 11" penetration. (14:55)
20					
5			>50		5- SAND, grayish brown, medium fine grained, damp, very dense, no odors noted. (15:00)
25				SP	
6			>50		6- As sample 5 above, extremely faint odor similar to samples 3 and 4 noted, damp, very dense, 9" penetration. (15:10)

- * Boring located at magnetometer survey coordinates 26 east, 85 north.
- * Boring terminated at 28.25 feet on September 21, 1999.
- * Monitoring well not installed, groundwater not encountered
- * No visual indications of contamination. Olfactory indications of contamination noted in samples 3 and 4 (12.5 feet and 17.5 feet). Odors not similar to hydrocarbons, but could possibly be very aged gasoline.
- * Numbers in parentheses indicate the time the sample was obtained.

ENVIRONMENTAL
ASSOCIATES, INC.

2122 - 112th Avenue N.E., Ste. B-100
Bellevue, Washington 98004

TEST BORING LOG: B-6

Former Gasoline Station
Vacant Lot East of 1226 North 45th Street
Seattle (Wallingford), Washington

Job Number:	Date:	Logged by:	Plate:
JN 9301-2	September 1999	D. Holmes	4

APPENDIX D

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Attachment to and part of Report 21-1-20193-001

Date: December 9, 2004
To: Mr. Vaughn McCleod
Housing Resources Group

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