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Attachment 6b:
Geotechnical Report

Geotechnical Engineering Design Study
Washington School for the Deaf
Proposed School Additions
Vancouver, Washington

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Prepared for
Washington School for the Deaf
Acting Through the Department of
General Administration, Division of
Engineering and Architectural Services

May 16, 2002
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Prepared by
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CONTENTS

CONTENTS (continued)

GEOTECHNICAL ENGINEERING DESIGN STUDY
WASHINGTON SCHOOL FOR THE DEAF
PROPOSED SCHOOL ADDITIONS
VANCOUVER, WASHINGTON

1.0 INTRODUCTION

This report presents Hart Crowser's geotechnical engineering evaluation and preliminary recommendations for the proposed reconstruction of the Washington School for the Deaf, in Vancouver, Washington. The project was still in the preliminary planning stage during the period over which this geotechnical site investigation was conducted. It has been assumed that the reconstruction project will include significant segments of demolition of existing school facilities. It has also been assumed that new construction may include building additions of up to two stories. Slab-on-grade construction with possible basement segments under some of the new structures is also possible. After this introduction, the report presents the following:

- Summary;
- Project understanding;
- Subsurface conditions;
- Engineering conclusions; and
- Appendix addressing field explorations and laboratory data.

1.2 Purpose of Work: Provide Geotechnical Engineering Recommendations

The purpose of our work was to provide geotechnical engineering recommendations for the design and construction of the proposed complex. Our recommendations include the following:

- Site preparation;
- Grading;
- Foundation design;
- Drainage;
- Estimated settlement for engineered fill and footings;
- Foundation design;

- Site-specific seismic design assessment; and
- Other pertinent geotechnical design criteria and construction considerations.

1.3 Scope of Work: Soil Explorations, Engineering Analyses, and Report Preparation

Our scope of work for this project included the following:

- A review of general geologic literature and previous geotechnical reports in the project vicinity;
- Surficial reconnaissance;
- Subsurface explorations;
- Laboratory testing;
- Geotechnical engineering analyses; and
- Preparation of this report.

1.4 Limitations of Our Work

Hart Crowser completed this work in general accordance with our proposal 02-53-1501. We performed this work for the exclusive use of the Division of Engineering and Architectural Services, their clients, and agents for specific geotechnically related applications to this project. This work was conducted in accordance with generally accepted professional practices in the same or similar localities, related to the nature of the work accomplished at the time the services were performed. No other warranty, express or implied, is made.

2.0 SUMMARY

Following is a summary of the findings in this report. Please refer to the full report for all of the assumptions and details regarding our findings.

2.1 Subsurface Conditions

Asphalt Concrete Pavement. The majority of our borings were advanced through 2 to 5 inches of asphalt concrete (AC).

Topsoil & Landscaping Fill. Several of the borings were advanced into courtyard areas surrounding the campus. Topsoil thickness and landscaping fills totaled approximately 2 to 3 feet. The fill was observed to be soft and moist, with a large amount of organic material visible in the soil matrix.

Coarse Sand and Gravel. The school site is underlain primarily by a medium dense to dense, coarse sand and gravel unit.

Groundwater. The static groundwater table was not observed up to the maximum depth explored in our borings, approximately 40 feet below the ground surface (bgs).

2.2 Site Preparation and Excavations

It is strongly recommended that site preparation, earthwork, and paving and utility work, be conducted during warm, dry summer months.

Site Preparation. Site preparation should consist of the removal of all topsoil, landscaping fills, and existing pavements or concrete slabs from new building footprint areas and new pavement areas. Asphalt removal will entail demolition of pavement sections ranging in thickness from 2 to 5 inches.

Wet Weather Earthwork. During wet weather grading a granular working blanket should be installed over the site to both protect fine-grained soil subgrades and allow vehicular construction traffic to traverse the site. Structural fills placed during wet weather should consist of clean crushed rock.

Compaction Standards. Recommended compaction specifications should be based upon ASTM D 1557 (Modified Proctor Test) or AASHTO T-180. Fine-grained soils should be compacted to at least 92 percent of the material's maximum dry density, and imported granular fill should be compacted to at least 95 percent of the material's maximum dry density.

Trench Backfill. Utility trench backfill that will be overlain by structural or pavement areas should consist of relatively clean, compacted granular fill.

Existing Fills. Shallow fill areas were observed within the courtyard areas on the site. When encountered, these fills should be removed and excavation areas returned to design via suitable structural fill.

2.3 Foundation Design and Pavements

Spread Foundations. The native soils underlying the site are capable of supporting one- to three-story, lightweight, structures (structures with maximum column loads not in excess of 200 kips [1 kip = 1,000 pounds] and wall loads not exceeding 5 to 6 kips per lineal foot) on spread footings. An allowable soil bearing pressure of 3,500 pounds per square inch (psf) can be utilized in foundation design.

Slabs on Grade. It is recommended that a 6-inch layer of compacted crushed rock be placed below slab-on-grade areas to serve as a capillary break. A modulus of subgrade reaction of 250 pounds per square inch (psi) can be used in slab-on-grade design.

Pavements. Pavement design based upon projected traffic loading conditions has been provided in the recommendation section of this report.

3.0 PROJECT UNDERSTANDING AND SITE DESCRIPTION

The project is located at the southeast corner of Grand Boulevard and East Evergreen Boulevard, in the City of Vancouver, Washington, and is presently occupied by the Washington School for the Deaf.

The site is relatively flat with the exception of the far southern portion of the campus. These steeper sloping areas are relatively far removed from proposed addition areas. The campus structures appear to have been constructed primarily of brick and mortar. Maximum height of present structures does not exceed two stories, with the exception of a tall brick chimney associated with the old boiler room. Several of the buildings have basement areas. Athletic fields and courtyard areas surround many of the buildings. Several large asphalt paved parking areas are also located on site.

It is Hart Crowser's understanding that the present design concept includes the following specifics:

- Demolition of several of the older buildings and filling in of old basement areas on the site;
- Construction of the new school buildings, regrading of the site, installation of new parking and access drives and construction of new underground utilities and services;
- New structures will not exceed one or two stories in height, with the potential of construction of a new high-wall gymnasium, and their average height will be approximately 28 feet;
- New construction will consist primarily of steel framed structures with composite slabs, with brick veneer and metal stud backup (some curtain walls may also be proposed); and
- Construction of new access drives and asphalt paved parking areas.

4.0 GEOLOGIC SETTING AND SITE SPECIFIC SEISMIC ASSESSMENT

Office review of the United States Geological Survey (USGS) Bulletin 1119, entitled Geology of Portland, Oregon and Adjacent Areas by Donald E. Trimble (1963), indicates the presence of one near-surface geologic unit underlying the project site. This geologic unit consists of coarse gravels and sands. This unit consists of Upper Pleistocene-aged lacustrine gravels and sands (Qlg). The Qlg unit typically extends to depths of approximately found below this site at about 40 to 50 feet bgs in the project vicinity. The Tertiary-age Troutdale Formation in the project vicinity typically underlies the gravel unit.

The Troutdale Formation is generally composed of partially cemented sands and gravels with a cap of fine-grained sands and silts. Typically, the upper surface of the Troutdale Formation consists of deeply weathered sands and silts that are relatively impermeable. Consequently, when water percolates through the upper Qlg unit, it "perches" on the Troutdale surface, then begins to migrate laterally until it emerges as springs on the south-facing slopes in this area.

4.1 Seismicity and Earthquake Sources

The seismicity of the Clark County area, and, hence, the potential for the project site ground shaking, is controlled by three separate fault mechanisms: the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The Cascadia Subduction Zone. The CSZ is located offshore and extends from Northern California to British Columbia. Within this zone, the oceanic Juan De Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers. The return interval for large subduction zone earthquakes is believed to be 300 to 500 years. Evidence suggests that the most recent subduction zone event took place approximately 300 years ago. Geomatrix's study (1995) suggests that the maximum earthquake associated with the CSZ is moment magnitude (M_W) 8 to 9. A subduction zone earthquake of magnitude 8.5 was assumed for the purposes of this report

The Intraplate Zone. The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 30 to 50 km below southwestern Washington. Very low levels of seismicity have been observed within the intraplate zone in Washington; however, much higher levels of seismicity within this zone have been recorded in Washington and California. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1, the 1965 Puget

Sound magnitude 6.5, and the 2001 Nisqually magnitude 6.8 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The recent (February 28, 2001) seismic event near the town of Nisqually, Washington (the epicenter of which was between Tacoma and Olympia, approximately 10 miles northeast of Olympia) has been classified as an intraplate type seismic event. The focus of the Nisqually Quake was approximately 30 miles deep, and its magnitude was determined to be 6.8. This quake was felt strongly in Portland and Vancouver, as well as in British Columbia.

Near-Surface Crustal Sources. The third source of seismicity that can result in ground shaking within the greater Portland area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in southwest Washington is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0) earthquakes were crustal earthquakes.

Site Soil Seismic Coefficient. Any foundation design for the project in which seismic design parameters are required should be based upon the following criteria: a site soil coefficient of S_D and a Zone Factor (Z) of 0.3.

Individual faults or fault zones that have been mapped in Washington State Earthquake Hazard Information Circular 85 (1988) and Geomatrix (1995) in the near-vicinity of the site include the following:

Faults From Geomatrix (1995) and Washington State Earthquake Hazard (1988)

Fault System	Approximate Closest Distance to Site (Km)	Probability of Activity
Lacamas Creek Fault	12	0.5
Saint Helens Fault Zone	45	*
Portland Hills Fault Zone	15	0.7
Bolton Fault	30	0.2
Grant Butte, Damascus-Tickle Creek Fault Zone	18	0.5
Helvatia Fault	25	0.2
Sandy River Fault	23	0.1
Mount Angel Fault	48	0.6
Newberg Fault	43	0.2
Gales Creek Fault	45	0.2

***Note:** Washington earthquakes recorded between 1982 and 1987 along the Saint Helens Fault Zone include at least one event in the range of magnitude 4 to 4.9. Two separate events of magnitude 3 to 3.9 were also recorded for this same time period (Ref. Washington State Earthquake Hazards Information Circular 85, 1988).

Distances indicated are approximate horizontal distance from the project site to the fault zone of concern. All of the above-listed faults can be classified under Table 16-U (Seismic Source Type) of the Uniform Building Code (UBC) as seismic source type C. These faults are not anticipated to be capable of producing large Mw earthquakes. Maximum Mw associated with the above fault zones are estimated to be less than 6.0 to 6.5.

The probability of activity for fault and fault zones, which ranges from 0 to 1, is based on such factors as association of the fault with historical seismicity, evidence for late Quaternary fault displacements, geomorphic evidence for geologically recent deformation, association with neighboring structures showing evidence of Quaternary activity, pre-Quaternary history of deformation, and orientation relative to the present stress field.

Seismic and geologic parameters such as slip rate, horizontal and vertical offset, rupture length, and geologic age have not been determined for the majority of the above faults. This is primarily due to the lack of surface expressions or exposures of faulting because of urban development and the presence of late Quaternary soil deposits that overlie the faults. The low level of historical seismicity (particularly for earthquakes greater than Mw 5) and lack of paleo-seismic data, results in large uncertainties when evaluating individual crustal fault maximum Mw earthquakes and recurrence intervals. Ongoing studies are currently being undertaken to attempt to determine the probability of activity for the Portland Hills Fault Zone. These studies are utilizing deep trenching and geophysical methods at a number of locations along the inferred fault zone. To date, ongoing studies have not identified quaternary age seismic activity for any of the near-vicinity faults for this project.

For the reasons described above, it is prudent to evaluate the potential for seismic shaking due to crustal earthquakes based on a regional, probabilistic approach. Based on data presented by Geomatrix (1995) and Department of Geology and Minerals Industries (1990), the seismic exposure at the site from crustal zone sources is represented by an earthquake of Mw 6.0 to 6.5.

5.0 SITE-SPECIFIC SEISMIC GEOLOGIC HAZARD CHECK LIST

5.1 Site Liquefaction Potential

Liquefaction potential of soil underlying the site was assessed using methods developed by Seed and Idris (1983). SPT blow count data obtained during our subsurface exploration were utilized for this purpose. Native dense gravels extending to the maximum depth of our explorations (approximately 41-1/2 feet bgs) were found to have high factors of safety against liquefaction during a near-surface crustal earthquake. The event modeled for a near-surface crustal earthquake was a Mw 6.5 earthquake with peak ground acceleration of 0.26g.

5.2 Tsunami or Seiche

Due to the site's proximity relative to major bodies of water Tsunami or Seiche are not deemed as potential site-specific seismic hazards.

5.3 Dynamic Slope Stability

Dynamic slope stability hazards associated with the project area are not expected. The ground along the south side's Administration Building and Watson, Roberts and Macdonald Cottages slopes very steeply, but this area is outside of the proposed redevelopment area. A separate assessment of this area can be performed upon request.

5.4 Ground Subsidence Due to Fault Rupture

Based upon literature review, there are no known active faults underlying the project site. It is therefore surmised that ground subsidence as a result of fault rupture is not deemed probable for the project site.

6.0 SUBSURFACE CONDITIONS

The field explorations for this project were conducted on March 14 and March 26, 2002. The exploration consisted of a surficial reconnaissance, advancing five solid stem augured borings to depths shown on the drilling logs. The maximum depth of any of Hart Crowser's subsurface explorations was approximately 41-1/2 bgs.

The approximate locations of these borings have been indicated on the Site Plan (Figure 2) and the logs for the borings are provided in Appendix A of this report. The locations of the Hart Crowser subsurface explorations were paced off in the field from prominent surface features, and the locations indicated on Figure 2 should be considered approximate.

Soil conditions encountered within borings were logged in the field by a representative of Hart Crowser's geotechnical engineering staff. Logs of all subsurface explorations have been included in Appendix A of this report. The attached boring logs describe soils and various engineering properties of soils encountered during exploration. Descriptions are based upon field classification of soil samples.

It should be emphasized that our exploration revealed subsurface conditions only at discrete locations on the project site and that actual conditions could vary at other locations. Furthermore, the nature and extent of any such variations would not become evident until construction activities have begun. If significant variations are observed at that time, we may need to modify our conclusions and recommendations to reflect actual conditions. For ease of outside interpretation, subsurface conditions have been generalized into the following major categories.

Topsoil. Landscaping areas of the school campus are currently mantled in several inches of silty topsoil or organic-rich landscaping fill. Anticipated topsoil stripping depths during construction are expected to be 4 to 6 inches, with some limited areas of deeper stripping required. Landscaping fills within courtyard areas were observed to extend to depths of 2 to 3 feet bgs. This material should not be considered suitable as bearing stratum and should be removed from proposed building or pavement areas. Topsoil and landscaping should not be reemployed as structural fill, but could potentially be reused as landscape fill or within low-lying landscape berms.

Asphalt Concrete Pavement. The majority of our borings were advanced through several inches of AC. Varying thickness of AC were encountered across the site and were noted to range from 2 to 5 inches.

Coarse Sand and Gravel. The school site appears to be underlain primarily by a medium-dense to dense, coarse sand and gravel unit. Trace amounts of silt were observed in soil samples obtained from the upper 8 to 10 feet. The sand and gravel unit appeared to become progressively cleaner with depth (i.e., minimal silt and clay soil was observed within soil samples obtained below approximately 5 to 10 feet bgs).

Groundwater. The static groundwater table was not observed up to the maximum depth explored in our borings (approximately 40 feet bgs).

7.0 GEOTECHNICAL ENGINEERING CONCLUSIONS AND RECOMMENDATIONS

Our recommendations are based on our current understanding of the project. If the nature or location of the planned construction changes, Hart Crowser should be contacted so that we can confirm or revise our recommendations.

7.1 Site Preparation

We have provided recommendations for wet weather and dry weather construction, as well as for other geotechnical concerns and issues relative to the project site. Because of the moisture-sensitive near-surface soils and the potential for encountering shallow perched groundwater, Hart Crowser strongly recommends that site grading and utility trenching be conducted during dry weather conditions. The optimal time for site grading is generally between early July and mid-October. If wet weather construction is attempted, development costs could be significantly higher due in part to the increased cost of imported granular fill, maintenance of soft subgrade generated as a result of construction activities, and installation of a granular working blanket over the site.

Stripping and Removal of Existing Improvements. Courtyard and playground areas on the campus are presently mantled in 4 to 6 inches of silty topsoil. As part of site preparation, building areas, pavement subgrade, fill subgrades, structural areas, etc., should be stripped of all organic soils, roots, brush, and trees. Tree and brush roots larger than 1/2 inch in diameter should be grubbed, and the excavation areas left from root balls should be backfilled with structural fill. Shallow landscape fills are also observed in some of these areas and will also require removal from building footprint and pavement/sidewalk areas.

Topsoil strippings generated during this process should not be re-employed as structural fill or trench backfill. Strippings may be employed in thin areal fills in landscape areas or be used in low-lying landscape berms.

Several of the existing structures will be demolished as part of the project. Basement area slabs, old foundations, old slabs, underground storage tanks, etc., should be removed in their entirety during site preparation. All old pavements and abandoned utilities should be removed from building pad areas and pavement subgrade areas.

We recommend that a representative of Hart Crowser's geotechnical engineering staff be retained to observe the stripping, earthwork, and geotechnical related segments of site demolition work.

Existing Fills. It is typical to encounter landscape and other minor fills on previously developed sites. Although the fills can vary widely in their suitability to function as foundation or pavement subgrades, it should be assumed the majority of these fill will require removal or reworking during site earthwork.

Dry Weather Construction. It is anticipated that significant recontouring of the site may be conducted. The on-site native soils should provide adequate structural fill material if placed and compacted during dry weather months. Proper moisture conditioning should be conducted prior to placement and compaction.

Once grading plans have progressed to a preliminary state, it is recommended that Hart Crowser be retained to review such plans to determine if any additional recommendations will be required for site development. It is strongly recommended that this review process be undertaken early in the planning process so potential long-term cut and fill related stability issues can be addressed long before final grading plans have been issued.

Minimum compaction for the 8 inches immediately underlying pavement sections and slabs should be 95 percent of the soil or gravel's maximum density. Even during dry weather it is possible that some areas of the subgrade will become soft or may "pump" (deflect under wheel load), particularly in deeper cuts, poorly drained areas, abandoned drainage ditches, swales, etc. Soft or wet areas that cannot be effectively dried and compacted should be prepared in accordance with the recommendations provided in the Wet Weather Construction section of this report.

Wet Weather Construction. If wet weather site grading proves to be necessary, structural fills should consist of clean, imported granular material such as crushed rock or pit run material containing less than 5 percent fines by weight.

During wet weather or when adequate moisture control is not possible, it may be necessary to install a granular working blanket to support construction equipment and provide a firm base on which to place subsequent fill and pavement, and to protect fine-grained subgrade soils from agitation due to the combination of construction traffic and water. The working blanket commonly consists of a bank run gravel or pit run quarry rock (6-inch maximum size with no more than 5 percent by weight passing a No. 200 sieve). We recommend that we be consulted to approve the material before installation.

The working blanket should be installed on a stripped subgrade in a single lift, with trucks end-dumping off an advancing pad of granular fill. It should be possible to strip most of the site with the careful operation of crawler-mounted equipment; however, during prolonged wet weather, operation of this type of equipment may cause excessive subgrade disturbance. In some areas, final stripping and/or cutting may have to be accomplished with a large, smooth bucket track hoe or similar equipment, working from the advancing blanket of granular fill. After installation, the working blanket should be compacted by a minimum of four complete passes with a moderately heavy (15,000 pounds) static steel drum or grid roller. We recommend that we be retained to observe the granular working blanket installation and compaction.

The working blanket must provide a firm base for subsequent fill installation and compaction. It has been our experience that a minimum of 12 to 18 inches of working blanket is normally required, depending on the gradation and angularity of the working blanket material. This assumes that the material is placed on a relatively undisturbed subgrade in accordance with the preceding recommendations, and that it is not subjected to frequent heavy construction traffic.

Portions of the site used as haul routes for heavy construction equipment will require a thicker working blanket in order to protect the fine-grained subgrade. If particularly soft areas are encountered, a non-woven filter fabric installed on the fine-grained subgrade may be helpful in preventing silt from contaminating and pumping the granular working blanket. If desired, we can provide sample specifications for this material.

Construction practices can greatly affect the amount of working blanket necessary. By using tracked equipment and granular haul roads, the working blanket area can be minimized. If dump trucks, forklifts, and other rubber-tired equipment are allowed random access across the site, a thicker working blanket may be required. Normally, the design, installation, and maintenance of a granular working blanket are the responsibility of the earthwork contractor.

An alternative to the use of a granular working blanket would be cement treatment of native soils. The working blanket rock section can often be reduced if an equivalent section of cement treated subgrade is employed. This is accomplished using specialized spreaders and mixers and is often more cost-effective than imported rock working blankets. We would be happy to provide further recommendations for cement treated soils, if desired.

During wet weather site work, it is possible that groundwater levels will approach near-surface grade on the lower (southern) portion of the site. Utility trench work conducted during wet months is likely to encounter perched or static groundwater levels at shallow depth.

Proof-Rolling. Regardless of which method of subgrade preparation is used (i.e., wet weather or dry weather), we recommend that prior to fill placement or base course installation, the subgrade or granular working blanket be proof-rolled with a loaded 10- to 12-yard dump truck or other suitable equipment. This pertains to all pavement, structural fill, and floor slab areas. Any area of subgrade that pumps, weaves, or appears soft and muddy should be scarified, dried, and compacted, or over-excavated and backfilled with compacted granular fill. If a significant length of time passes between fill placement and commencement of construction operations, or if significant traffic has been routed over these areas, we recommend that the subgrade be similarly proof-rolled again before any foundation or pavement installation is allowed. We recommend that we be retained to observe this operation to evaluate preparation of structural grades.

7.2 Structural Fills

Structural fill should be installed on a subgrade that has been prepared in accordance with the above recommendations. Fills should be installed in horizontal lifts not exceeding 8 inches in thickness (loose - prior to compaction), and should be compacted to at least 92 percent of the maximum dry density for fine-grained native soils. The final 6 to 8 inches of fill immediately below pavements and building slabs should be compacted to at least 95 percent of the material's maximum dry density. The maximum dry densities should be determined in accordance with ASTM D 1557. The compaction criteria may be reduced to 85 percent in non-structural landscape or planter areas. A summary of recommended compaction specifications is provided in the table below.

Recommended Fill Compaction Specifications

Material	Percent of Maximum Dry Density ASTM D 1557
Fine-Grained Fill	92 percent
Landscaping Fill	85 percent
Granular Fill	95 percent
Pavement Subgrade	95 percent

During dry weather, structural fills may consist of virtually any relatively well-graded soil that is free of debris, organic matter, and high percentages of clay or clay lumps that can be compacted to the preceding specifications; however, if excess moisture causes the fill to pump or weave, those areas should be dried and recompacted, or removed and backfilled with compacted granular fill. In order to achieve adequate compaction during wet weather or if proper moisture content cannot be achieved by drying, we recommend that fills consist of well-graded granular soils (sand or sand and gravel) that do not contain more than 5 percent material by weight passing the No. 200 sieve. In addition, it is usually desirable to limit this material to a maximum 6 inches in diameter for ease of compaction and future installation of utilities.

Adequate compaction levels for structural fills can usually be obtained within fine-grained soils at +/-3 percent of the optimum moisture content. If excess soil moisture is present in potential fill soils, soil drying via aeration should be considered. Soils can commonly be dried by being turned with a disc in order to evaporate excess moisture. Soil drying in this manner is generally only possible during extended periods of warm dry weather. Optimal time for this type of operation is from early July through mid-September. During wetter weather, alternative methods of soil drying could include lime or Portland cement treatment of fill soils.

7.3 Suitable Fill Materials

Structural Fill Construction During Dry Weather. During dry weather, structural fills may consist of virtually any relatively well-graded soil that is free of debris, organic matter, and high percentages of clay or clay lumps that can be compacted to the preceding specifications; however, if excess moisture causes the fill to pump or weave, those areas should be dried and recompacted, or be removed and backfilled with compacted granular fill. In order to achieve adequate compaction during wet weather, or if proper moisture content cannot be achieved by drying, we recommend that fills consist of well-graded granular soils (sand or sand and gravel) that do not contain more than 5 percent material by weight passing the No. 200 sieve. In addition, it is usually desirable to limit this material to a maximum 6 inches in diameter for ease of compaction and future installation of utilities.

Slab on Grade and Pavement Base Rock. Crushed rock utilized in these areas should consist of clean 5/8- to 1-1/2-inch (minus) durable crushed rock. The material should contain less than 5 percent fines by weight passing a standard No. 200 sieve.

Trench Backfill. Utility conduits should be bedded in sand or 3/4-inch (minus) crushed rock within one conduit diameter. Bedding should surround the pipe in all directions. Trench backfill should be lightly compacted within two diameters or 18 inches, whichever is greater, above breakable conduits. Trench backfill underlying pavements, slabs or other settlement sensitive structures or features should consist of durable, clean crushed rock with nominal size from 5/8-inch (minus) to 1-1/2-inch (minus). This material should contain less than 5 percent fines by weight passing a standard No. 200 sieve.

Working Pad for Wet Weather Grading. The working pad for wet weather construction should consist of durable, clean crushed rock, bank run, or pit run material. Nominal size should be between 1-1/2-inch (minus) and 3-inch (minus). The material should contain less than 5 percent fines by weight passing a standard No. 200 sieve.

Quality Control During Fill Placement. To reduce the potential for long-term fill related settlement issues, all fill and backfill placed in building footprint areas, pavement areas, and areas that may function as subgrade for settlement sensitive features, should be observed and tested on a regular basis during construction. Observation and testing should be conducted to determine if compaction/density levels consistent with project plans and specifications are being achieved. Placement and compaction techniques and density testing should ideally be conducted on a lift-by-lift basis. This would usually entail at least one site visit per day by a soils inspector during rough grading operations.

7.4 Retaining Structures

Retaining wall geotechnical design parameters are presented in this section of the report. The following guidelines for restrained and non-restrained walls assume that the associated recommendations regarding drainage, compaction, and other issues will be implemented.

The design parameters in this section are for conventional retaining walls. If alternative retaining wall systems are proposed, we should be contacted for additional soil parameters.

Restrained Walls. Restrained walls are any walls that are prevented from rotation during backfilling. Walls with corners, basement walls, and those walls that are restrained by a floor slab or roof fall into the category of restrained walls. We recommend that restrained walls be designed for pressures developed from the equivalent fluid weights shown in the following table.

Restrained Wall Pressure Design Recommendations

Backfill Slope Horizontal/Vertical	Equivalent Fluid Weight (lb/ft^3)
Level	40
3H:1V	60
2H:1V	100

These pressures represent our best estimates of actual pressures that may develop and do not contain a factor of safety. These pressures are assumed to act horizontally (normal to the wall). This is based on the assumption that friction between the wall and backfill will be prevented by drainage membranes or impervious wall coatings. These pressures assume retaining wall backfill material is well drained. If traffic loads are expected within a horizontal distance from the top of the wall equal to the wall height, a uniform lateral earth pressure acting horizontally on restrained walls equal to 80 psf should be added to earth loads acting on the wall.

Non-Restrained Walls. Non-restrained walls have no restraint at the tops and are free to rotate about their bases. Most cantilever retaining walls fall into this category. We recommend that non-restrained walls be designed for pressures developed from the equivalent fluid weights shown in the following table.

Non-Restrained Retaining Wall Pressure Design Recommendations

Backfill Slope Horizontal/Vertical	Equivalent Fluid Weight (lb/ft^3)
Level	30
3H:1V	50
2H:1V	90

These pressures represent our best estimate of actual pressures that may develop and do not contain a factor of safety. These pressures assume retaining wall backfill material is well drained. If traffic loads are expected within a horizontal distance from the top of the wall equal to the wall height, a uniform lateral earth pressure acting horizontally on restrained walls equal to 60 psf should be added to earth loads acting on the wall.

Retaining Wall Backfill. Backfill behind retaining walls should consist of free-draining granular material. To minimize pressures on retaining walls, we recommend the use of well-graded crushed rock backfill with less than 5 percent by weight passing the No. 200 sieve. Use of other material could increase wall pressures. Over compaction of this fill can greatly increase lateral soil pressures. We recommend that this fill be compacted to between 90 and 92 percent of the maximum density determined in accordance with ASTM D 1557.

We recommend that foundations or major loads not be placed within the zone that extends back from the base of retaining walls at a 1H:1V slope.

Retaining Wall Drainage. Retaining walls will require drainage in order to alleviate lateral fluid forces on the walls. The drains should be protected by a filter fabric to prevent internal soil erosion and potential clogging.

7.5 Foundations

General Seismic Foundation Recommendations. We understand that the proposed building will be designed in general accordance with the 1997 Uniform Building Code. It is recommended that a soil profile type of S_D be utilized in earthquake design. A seismic zone factor of Z = 0.3 should also be utilized for design purposes.

7.6 Spread Footing Design Parameters

The native soils observed in our subsurface borings appeared dense and capable of supporting one- and two-story structures (two-story structures with maximum column loads not in excess of 200 kips) on spread footings. Based on the results of our subsurface exploration and office analysis, an allowable soil bearing pressure of 3,500 psf can be utilized in foundation design. This allowable bearing pressure assumes footings are designed to bear on native soils in excess of 18 inches below all adjacent grades.

Allowable bearing pressures for spread footings may be increased by one-third for short-term live loads such as wind or seismic loading. If foundation excavations are made during wet weather, or if foundation subgrades become disturbed, it may be necessary to remove disturbed soil and install a thin layer of crushed rock or lean concrete in foundation excavation bottoms.

For passive pressures in resistance to lateral loads, a 350-pcf equivalent fluid weight may be used for the soils. An ultimate base friction equal to 40 percent of the vertical load may also be used at the base of foundations as sliding resistance.

The estimated total settlements for conventional spread or continuous wall footings are anticipated to be 1 inch or less. Differential settlement between foundation elements is estimated to be 1/2 inch or less. These levels of settlement have been estimated based upon factored column footing loads which do not exceed approximately 200 kips and factored wall foundation loads of 4 to 6 kips per lineal foot. Settlement analyses have also been based upon a maximum allowable soil bearing pressure of 3,500 psf for shallow foundations. If factored loading conditions on foundation elements vary by more than 10 percent of the assumed loads detailed above for columns or wall footings, Hart Crowser should be informed in order to determine the validity of the above referenced settlement estimates, and provide additional recommendations if required. It should be understood that settlement estimates are inherently approximate, and actual settlements may vary from these estimates.

7.7 Floor Slabs

After preparation, in accordance with the following sections, Hart Crowser recommends that any floor slab areas be proof-rolled with a loaded dump truck. Any areas that pump, weave, or appear soft and muddy should be over-excavated and stabilized with compacted fill. A minimum 6-inch thick compacted crushed rock layer should be installed over the prepared subgrade to minimize subgrade disturbance during construction. Base rock in slab areas should be compacted to

95 percent of the materials maximum dry density as determined by ASTM D 1557. This crushed rock material should be well graded, angular, and contain no more than 5 percent by weight passing a No. 200 sieve.

Slab Moisture. The difference in moisture content between the air in the subgrade soil and the air in the finished building can cause water vapor to migrate through slab-on-grade areas. The resultant water vapor pressure will force migration of moisture through slabs. This migration can result in the loosening of flooring materials attached with mastic, the warping of wood flooring, and in extreme cases the mildewing of carpets and building contents. For most finished buildings, the presence of floor moisture would be considered a significant detriment to tenants and occupants.

Specific conditions at this site contribute to the potential for water vapor migration through slab-on-grade areas. These conditions are the naturally high moisture content of near-surface soils, the fine-grained nature of the near-surface soils (indicating a low permeability), the tendency of near-surface soils to hold moisture, and the potential presence of shallow perched groundwater. It should be anticipated that ground moisture levels and groundwater levels will rise during particularly wet periods.

If floor moisture is a concern for this project, we recommend that measures be taken to reduce vapor transmission. Numerous methods are available for minimizing the impact of water vapor migration on floor coverings. These include a variety of proprietary products. One possible mitigation options involving conventional materials has been outlined below.

Vapor Retarders. Vapor retarders are commonly employed to reduce water vapor migration through slab-on-grade areas. Installation of vapor retarders generally consists of installing a membrane between the crushed rock and the floor slab. A minimum slab base rock section of 6 inches should be employed in combination with the vapor retarder to provide a system against capillary rise of ground moisture. The base rock should be 3/4- or 1-inch (minus) material with a fines content of 5 percent or less of the material by weight.

The function of a vapor retarder/crushed rock section is analogous to the use of insulation to reduce heat flow through exterior walls. Vapor retarders will frequently need to be perforated in order to install utility services. In spite of planned perforations and others that may occur inadvertently, vapor retarders will still perform their intended function of slowing the transfer of water vapor, although their effectiveness will be reduced by significant perforations.

To maximize its effectiveness, the membrane must be installed in accordance with the manufacturer's recommendations. A polyethylene thickness of 10 mils should be employed to minimize the potential for puncture damage. If a less durable membrane is used, a thin sand layer should be placed below the membrane to protect the retarder from excessive punctures during construction. To reduce the possibility of slab curl and cracking due to differential moisture loss caused by the vapor retarder, a layer of sand can be placed on top of the membrane. The sand will allow even moisture reduction on each side of the slab. Alternatively, it is possible to design concrete mixes, which are not particularly susceptible to these problems.

Slab Moisture and Floor Coverings. Capillary rise of water has often been correlated to de-coupling (or loss of bond) problems with slab-on-grade floor coverings and their adhesive. This sometimes occurs with floor coverings such as vinyl tile and seamless flooring. Other causes of de-coupling or bond loss phenomena can be evaporation of excess concrete mix water. In general, a great deal more water is utilized in most concrete mixes used for slab-on-grade applications. This extra water is employed to make the concrete workable during placement. Some of this water is trapped within the concrete matrix and the remaining excess water evaporates at the two slab surfaces. During this hydration/evaporation period the surface moisture levels of concrete can be quite high. This high surface moisture level can create de-bonding problems with water based flooring adhesives. It is recommended that calcium chloride testing be performed on slabs prior to the placement of floor coverings. Manufacturer recommendations for maximum slab moisture levels should be adhered to prior to the application of flooring adhesives or leveling compounds.

7.8 Drainage

Surface Drainage. Positive surface drainage should be maintained away from all building foundations during construction. The finish grading should also provide for permanent, positive surface drainage away from the building. Positive surface drainage can best be achieved by maintaining finished floor elevations at least 1 foot above all grades immediately adjacent to a building pad. Surface water sources such as roof drains and parking lot runoff should be routed independently through non-perforated drain lines to a stormwater collection system. Surface water should not be allowed to enter subsurface drainage systems discussed below.

Foundation Drains. Due to the fine-grained nature of the near-surface soils and soil moisture levels during late fall through late spring months, Hart Crowser recommends that perimeter foundation subdrains be installed on this project. Foundation perimeter subdrains are installed at or below the base elevation of the footing to reduce the likelihood of saturation and softening of foundation subgrades. Foundation drains typically consist of 4- to 6-inch perforated drainpipe. Drains should be bedded in at least 2 inches of clean crushed rock or drain rock (rock should contain less than 5 percent fines passing a No. 200 sieve) and topped with an additional 6 to 8 inches of clean rock. Usually the drain rock surrounding foundation drains is wrapped in a geotextile filter fabric to prevent long-term clogging by siltation. Drains should be sloped so as to drain by gravity. In addition, drains should be routed to storm sewers independently of any other drainage systems.

7.9 Excavations and Utilities

Subsurface conditions encountered during the site investigation indicate that precautions in utility excavations will be required due to the potential for caving/sloughing within native soils underlying the site. Any excavations deeper than 4 feet should be sloped or shored in accordance with OSHA regulations. Normally, shoring systems (for excavations less than 20 feet in depth) are contractor designed and installed items. Dewatering of perched groundwater and/or rainwater within trenches and excavations may also be required.

Utilities. Utilities sensitive to moisture should be placed in watertight conduits. Utility conduits should be bedded in sand or 3/4-inch (minus) crushed rock within one conduit diameter. Bedding should surround the pipe in all directions. Trench backfill should be lightly compacted within two diameters or 18 inches; whichever is greater, above breakable conduits. The remaining backfill should be compacted to 95 percent of the maximum dry density of the material as determined by ASTM D 1557 for granular/crushed rock backfill.

7.10 Erosion Control

Hart Crowser recommends that finished cut and fill slopes be protected immediately following grading with vegetation, gravel, or other approved erosion control methods. Water should not be allowed to flow over slope faces or drop from outfalls, but should be collected and routed to stormwater disposal systems. Riprap, gabion baskets, or similar erosion control methods may be necessary at stormwater outfalls or to reduce water velocity in ditches. Silt fences should be established and maintained throughout the construction period. Silt fence barriers should be established down slope from all construction areas to protect natural drainage channels from erosion and/or siltation. In order to decrease erosion potential, care should be taken to maintain native vegetation and organic soil cover in as much of the site as possible.

7.11 Pavement Design

The following pavement design recommendations are based upon near-surface soil samples collected from the project site, *in situ* soil strength testing, AASHTO pavement design guidelines, and Hart Crowser's experience with similar soil types in the vicinity of the project parcel. It is Hart Crowser's recommendation that pavement design for this site be based upon relevant soil properties and constraints outlined in the following table.

Soil Related Pavement Design Parameters

Relative Subgrade and Base Rock Compaction (ASTM D 1557)	CBR	Resilient Modulus (psi)
95 percent	10	15,000

The following table summarizes our recommendations regarding AC pavement design and crushed rock base section for pavement areas constructed for the proposed project.

Asphalt Pavement Design Recommendations

Approx. Number of Trucks per Day (each way)	Approx. Number of 18-Kip Design Axles (1000)	AC Thickness (inches)	Crushed Rock Base Thickness (inches)
Auto Parking	10	2	6
5	22	2	8
10	44	2.5	8
15	66	2.5	10
25	110	3	10
50	220	3.5	11
100	440	4	12

The above pavement section is designed using AASHTO design methods and assumes an AASHTO reliability level (R) of 90 percent, with a terminal serviceability of 2.0 for AC. The 18-kip design axle loads are estimated from the number of trucks per day using State of Washington typical axle distributions for truck traffic and AASHTO load equivalency factors, and assuming a 20-year design life.

Our design assumes that the subgrade will be prepared in accordance with Hart Crowser's previous recommendation sections on Site Preparation and Fill Placement and Compaction. The top 8 inches, immediately below paved areas, should be compacted to 95 percent relative compaction, utilizing AASHTO T 180 (or ASTM D 1557) as a standard. Specifications for pavements should conform to current Oregon Department of Transportation specifications with the addition that the base rock should contain no more than 5 percent fines by weight passing a No. 200 sieve. In addition, AC should be compacted to a minimum of 92 percent maximum density as determined by ASTM D 2041.

This design is intended for use on public streets. If possible, construction traffic should be limited to unpaved and untreated roadways, or specially constructed haul roads. If this is not possible, the pavement design selected from the pavement design table should include an allowance for construction traffic. If wet weather grading is to be conducted, and construction traffic is allowed free access over prepared parking and access drive areas, and subgrade areas, it is probable that thicker rock sections will be required in order to prevent subgrade agitation from heavy wheeled traffic.

8.0 INFILTRATION TESTING

Although beyond Hart Crowser's original scope of services, an Infiltration test was conducted in Boring B-1. The test was conducted at approximately 11 feet bgs.

The test was conducted in accordance with standardized testing guidelines. A field infiltration test is similar to a "Falling Head Permeability Test." Hollow stem augering was conducted in boring B-1, and the auger was employed as an 8-inch-outside-diameter infiltrometer. The infiltrometer was filled with 3 feet of water. The water within the infiltrometer was maintained at a constant head for approximately one hour to saturate underlying soils. Following soil saturation, an infiltration test was performed, which entails measuring the amount of time required to lower the standpipe head 6 inches.

Test Results. The tests results indicated a percolation rate of approximately 240 inches per hour. This infiltration rate should be considered an ultimate field rate and, therefore, contains no factor of safety.

It has been observed that the permeability of undisturbed native soils such as those found on this site can be substantially different than soils that have been disturbed by construction activities. Ideally, infiltration systems should penetrate the upper 5 to 10 feet of silty gravel, and rely on the cleaner coarse sands and rounded gravels located below these depths. Hart Crowser recommends that the infiltration system installation either be field tested during construction to confirm adequate capacity is available, or Hart Crowser should be retained during installation in order to observe that infiltration systems are installed into clean native gravels or clean coarse sand.

9.0 RECOMMENDATIONS FOR ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, we recommend Hart Crowser review the final design plans and specifications. This review will allow us to evaluate whether any change in concept may affect the validity of our recommendations, and whether our recommendations have been correctly interpreted. In order to correlate preliminary soil data with the actual soil conditions encountered during construction, and to assess construction conformance to our report, we recommend that we be retained for construction observation of the following:

- Site preparation activities including stripping and fill placement and compaction;
- Footing excavations to verify suitability of bearing soils;
- Subgrades beneath slabs on grade and pavements; and
- Other geotechnical considerations that may arise during the course of construction.

10.0 CLOSING

This report presented Hart Crowser's geotechnical engineering evaluation and recommendations for the proposed project. We trust that this report meets your needs. If you have any questions, or if we can be of further assistance, please call. We look forward to working with you in the future.

11.0 REFERENCES

Geomatrix Consultants, 1995. Seismic Design Mapping State of Oregon. Prepared for The Oregon Department of Transportation, January 1995.

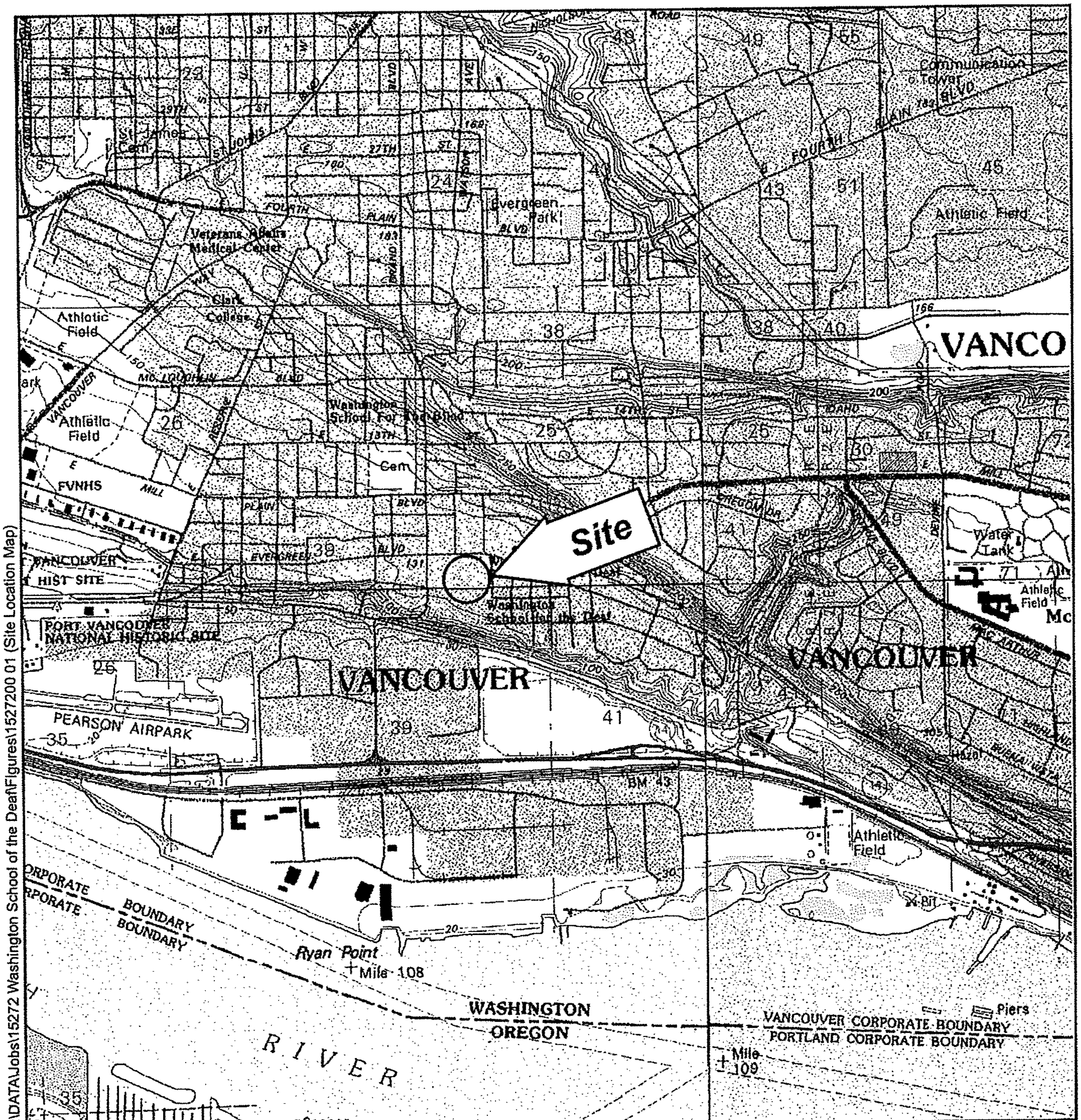
Nosan, L., et al, 1988. Washington State Earthquake Hazards. Washington Division of Geology and Earth Resources, Information Circular 85.

Trimble, D. E., 1963. Geology of Portland, Oregon and Adjacent Areas. United States Geological Survey (USGS) Bulletin 1119.

Walsh, T., et al, 1987. Geologic Map of Washington - Southwest Quadrant. Washington Division of Geology and Earth Resources, GM-34.

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY DATA

Site Location Map
Washington School for the Deaf
Vancouver, Washington



Note: Base map prepared from the USGS 7.5-minute quadrangles of Vancouver, Portland, and Mt. Tabor, OR-WA, and Orchards, WA, dated 1990.

Scale in Feet
Contour Interval 10 Feet

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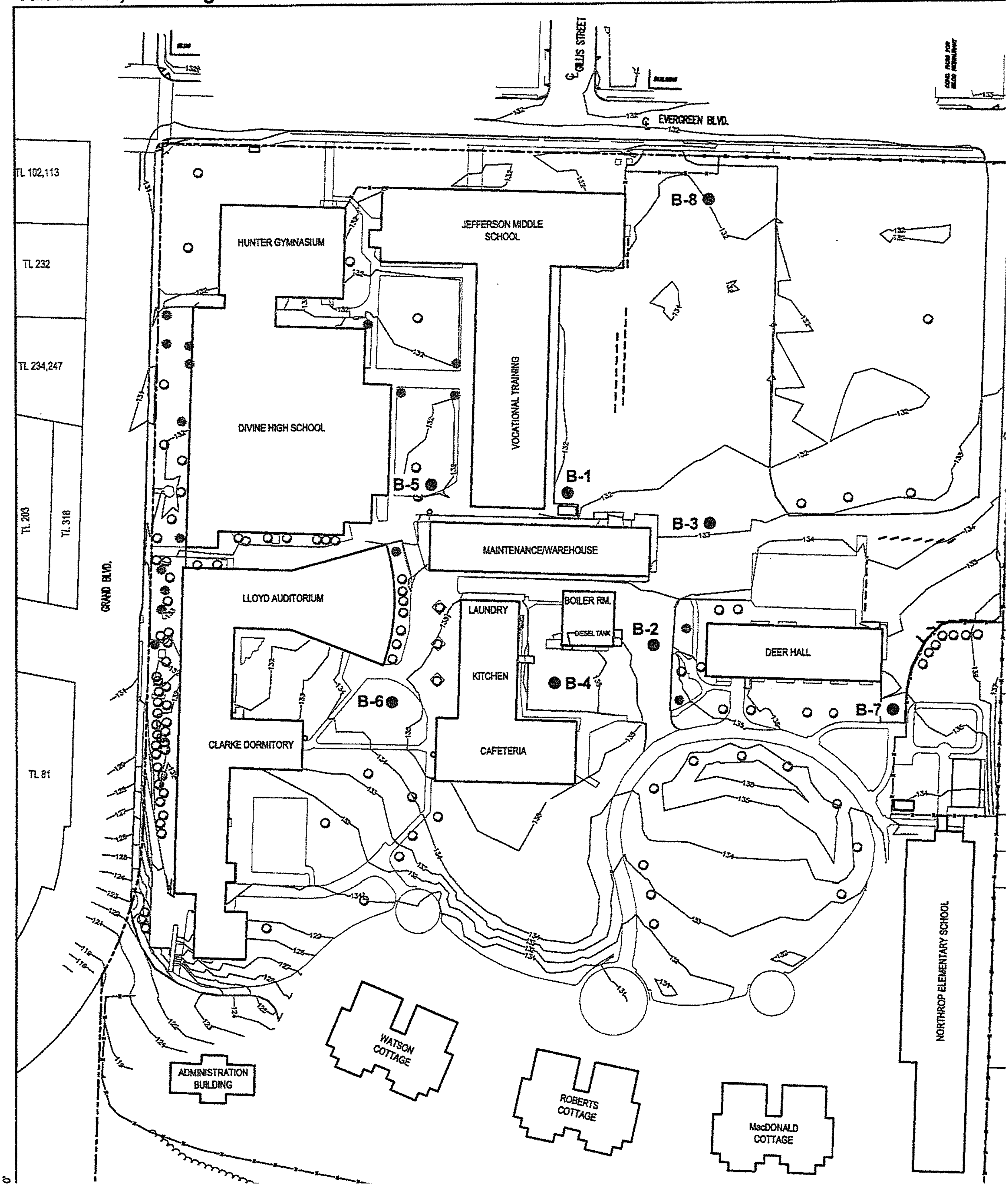
Figure 1

F:\DATA\Jobs\15272 Washington School of the Deaf\Figures\1527200 01 (Site Location Map)

Site Plan
Washington School for the Deaf
Vancouver, Washington



Key to Exploration Logs

Sample Descriptions

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, and grain size, and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
Density/consistency, moisture, color, minor constituents, MAJOR CONSTITUENT with additional remarks.

Density/Consistency

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and push probe explorations is estimated based on visual observation and is presented parenthetically on test pit and push probe exploration logs.

SAND and GRAVEL Density	Standard Penetration Resistance in Blows/Foot
Very loose	0 - 4
Loose	4 - 10
Medium dense	10 - 30
Dense	30 - 50
Very dense	>50

SILT or CLAY Density	Standard Penetration Resistance in Blows/Foot	Approximate Shear Strength in TSF
Very soft	0 - 2	<0.125
Soft	2 - 4	0.125 - 0.25
Medium stiff	4 - 8	0.25 - 0.5
Stiff	8 - 15	0.5 - 1.0
Very Stiff	15 - 30	1.0 - 2.0
Hard	>30	>2.0

Moisture

Dry	Little perceptible moisture.
Damp	Some perceptible moisture, probably below optimum.
Moist	Probably near optimum moisture content.
Wet	Much perceptible moisture, probably above optimum.

Minor Constituents

Minor Constituents	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Legends

Sampling Symbols

BORING SYMBOLS

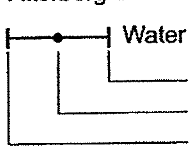
- Split Spoon
- Tube (Shelby, Push Probe)
- Cuttings
- Core Run
- * No Sample Recovery

TEST PIT SOIL SAMPLES

- Grab (Jar)
- Bag
- Shelby Tube

Test Symbols

GS	Grain Size Classification
K	Permeability
AL	Atterberg Limits



Groundwater Observations and Monitoring Well Construction

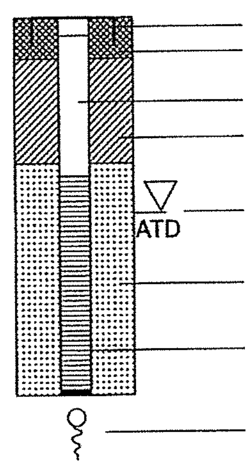


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Figure A-1

Boring Log B-1

Soil Descriptions

1" Asphalt concrete over 1' of soft SILT (FILL).

Very dense, wet, brown, silty GRAVEL.

Medium dense, damp, brown, coarse SAND with some rounded gravel.

Becomes gray with silt lenses.

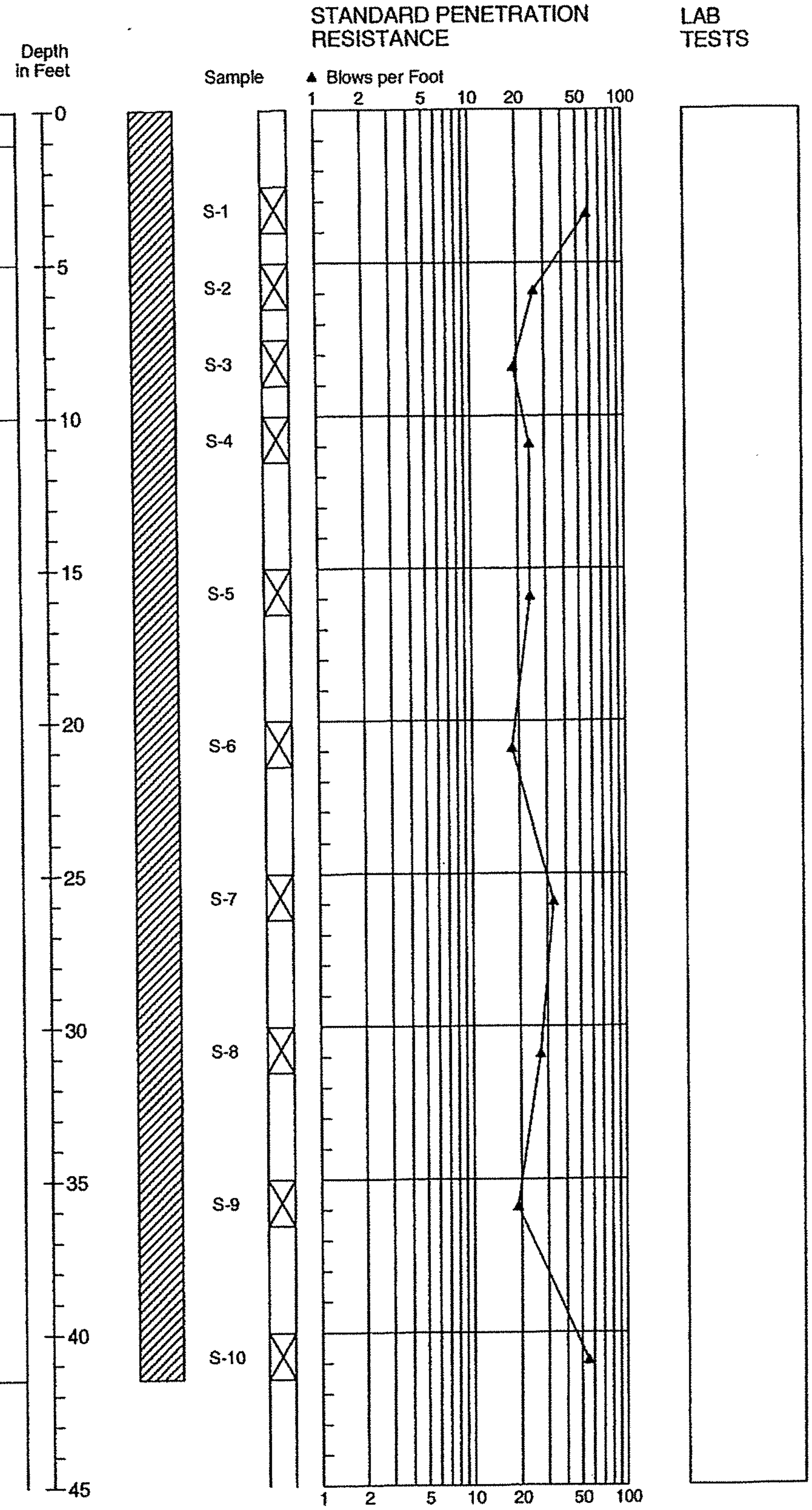
Medium dense, damp, gray, coarse SAND and rounded gravel.

Infiltration test conducted at 11' bgs.

Becomes very dense.

Bottom of Boring at 41.5 Feet.
Completed 03/26/02.

No Seepage or Groundwater Noted.



BORING LOG WITH BLOW COUNTS B_15272.GPJ HC_CORP.GDT 4/16/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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15272 3/02

Figure A-2

Boring Log B-2

Soil Descriptions

Depth in Feet

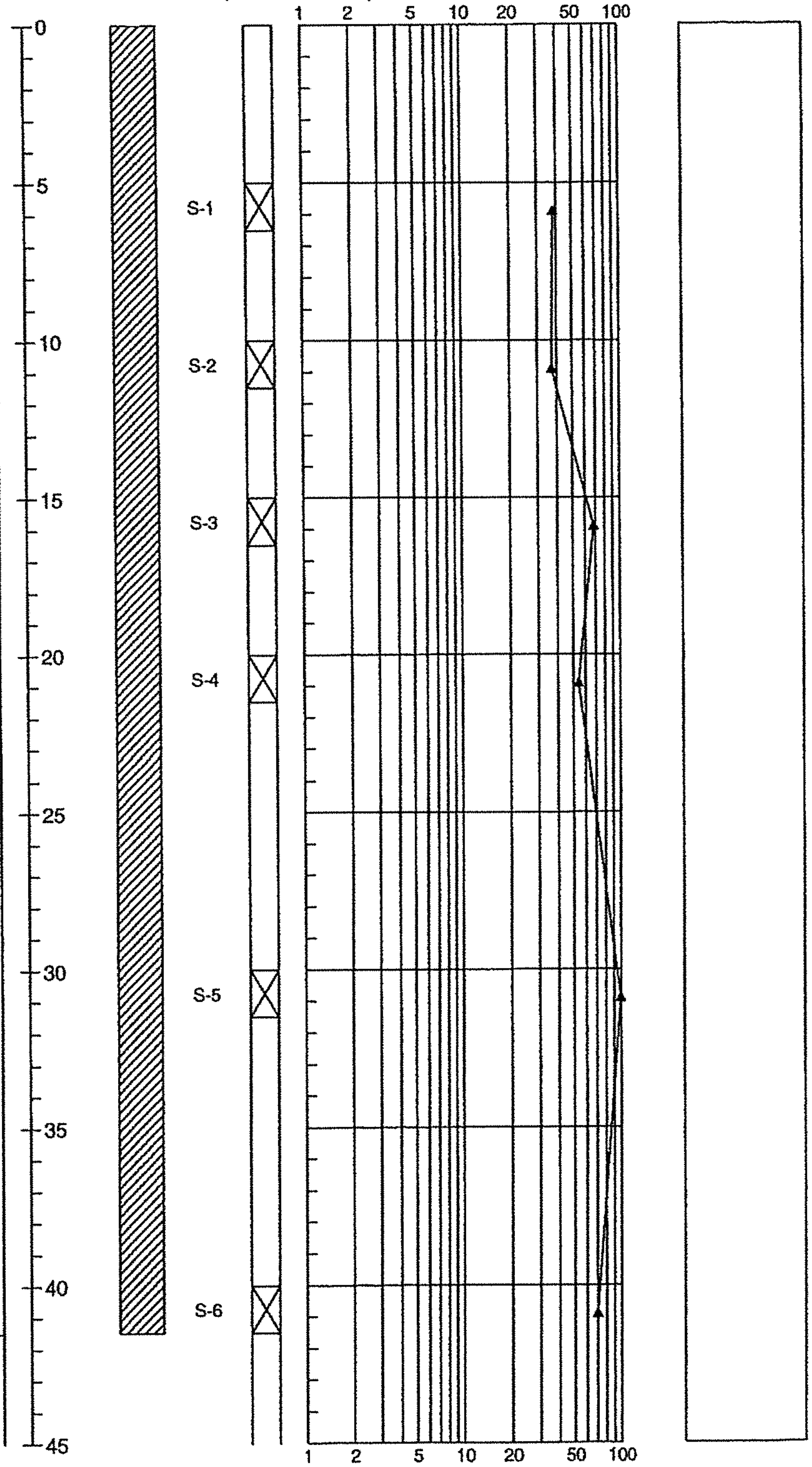
3" Asphalt concrete.

Very dense, damp, brown, coarse SAND and rounded GRAVEL.

Dense to very dense, damp, gray, coarse SAND and rounded gravel.

Bottom of Boring at 41.5 Feet.
Completed 03/14/02.

No Seepage or Groundwater Noted.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-3

Boring Log B-3

Soil Descriptions

Depth in Feet

Sample

STANDARD PENETRATION RESISTANCE

▲ Blows per Foot

1 2 5 10 20 50 100

LAB TESTS

4" Asphalt concrete.

(Dense), damp, brown, coarse SAND and rounded GRAVEL with some silt.

Bottom of Boring at 10.0 Feet.
Completed 03/14/02.

No Seepage or Groundwater Noted.
Boring Started as a Pre-Drill Location for CPT.

0 5 10 15 20 25 30 35 40 45

1 2 5 10 20 50 100

BORING LOG WITH BLOW COUNTS B_15272.GPJ HC_CORP.GDT 4/16/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-4

Boring Log B-4

Soil Descriptions

Depth in Feet

Sample

STANDARD PENETRATION RESISTANCE

LAB TESTS

4" Asphalt concrete over 4" crushed rock.

(Dense), damp, brown, coarse SAND and rounded GRAVEL.

S-1

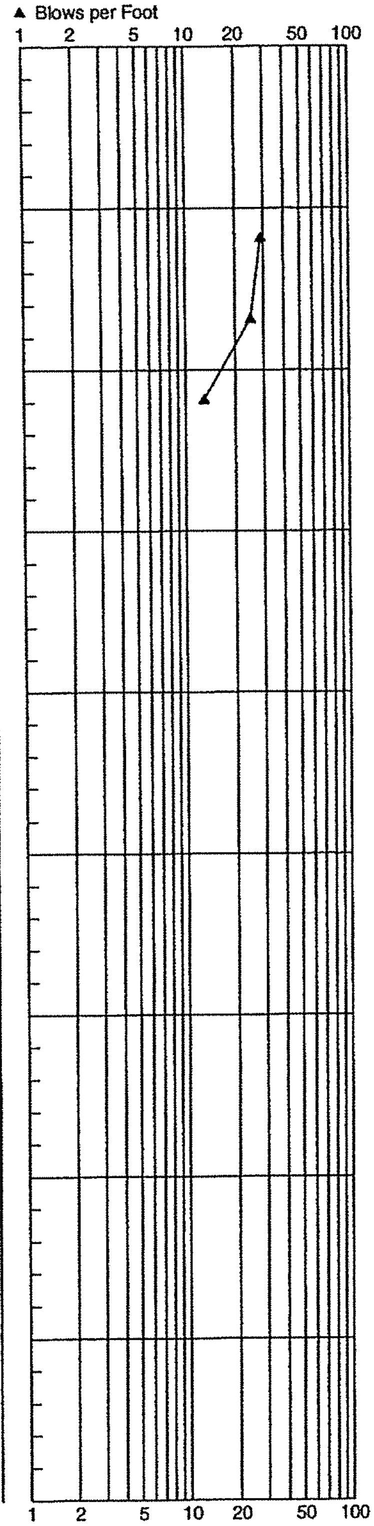
S-2

S-3

Bottom of Boring at 11.5 Feet.
Completed 03/14/02.

No Seepage or Groundwater Noted.

0 5 10 15 20 25 30 35 40 45



BORING LOG WITH BLOW COUNTS B_15272.GPJ HC_CORP.GDT 4/16/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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15272 *3/02*

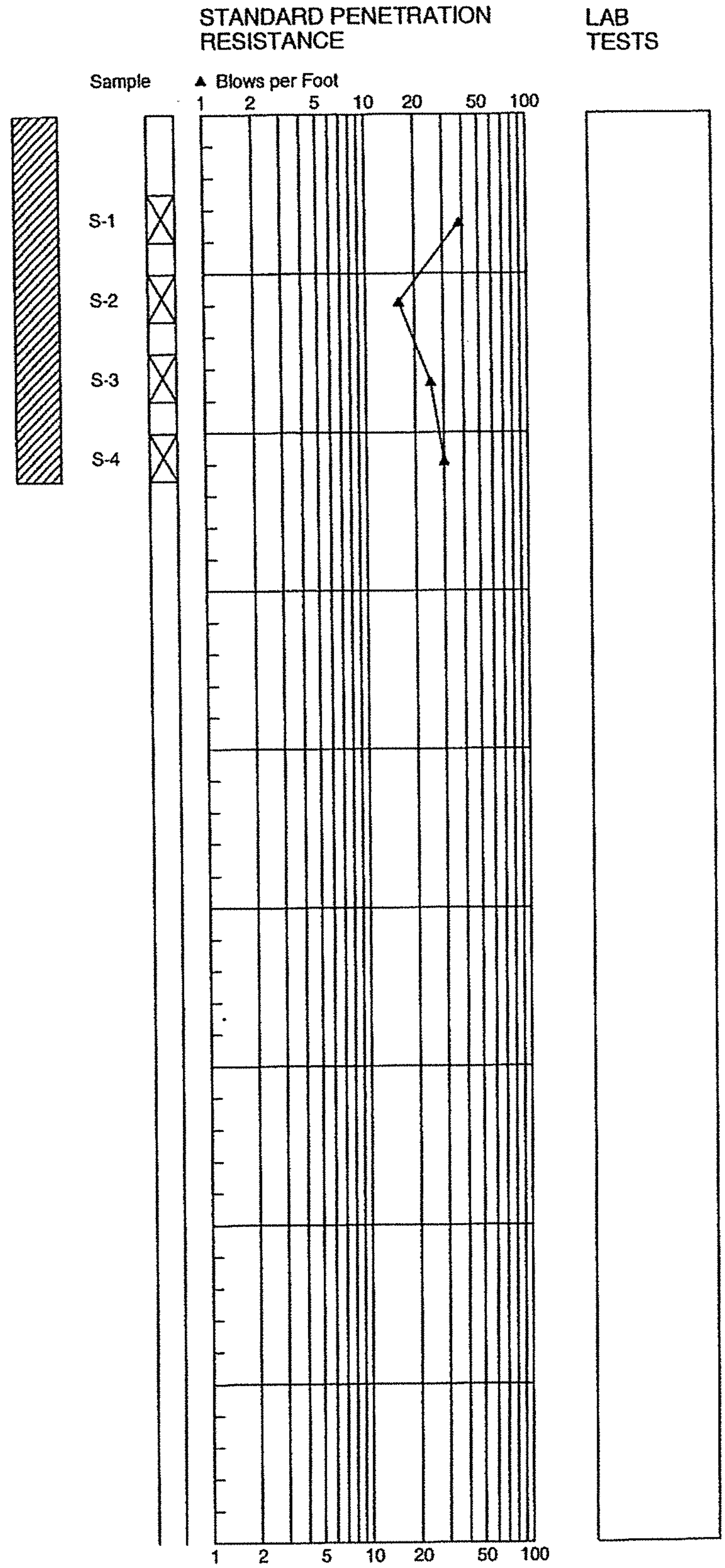
Figure A-5

Boring Log B-5

Soil Descriptions

Depth in Feet

Soil Descriptions	Depth in Feet
6" (Soft), damp to moist, dark brown TOPSOIL over SILT (FILL).	0
Dense, moist, brown, silty, coarse SAND and rounded GRAVEL.	
Becomes medium dense.	5
Becomes dense, damp, and gray.	
	10
Bottom of Boring at 11.5 Feet. Completed 03/14/02.	
No Seepage or Groundwater Noted.	15



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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15272 3/02

Figure A-6

Boring Log B-6

Soil Descriptions

Depth in Feet

6" Silty, sandy TOPSOIL over approximately 2' landscaping FILL.

Stiff, damp to moist, brown, coarse, sandy SILT with some gravel.

Very dense, damp, brown, silty, coarse SAND and rounded GRAVEL.

Becomes gray.

Trace quartzite.

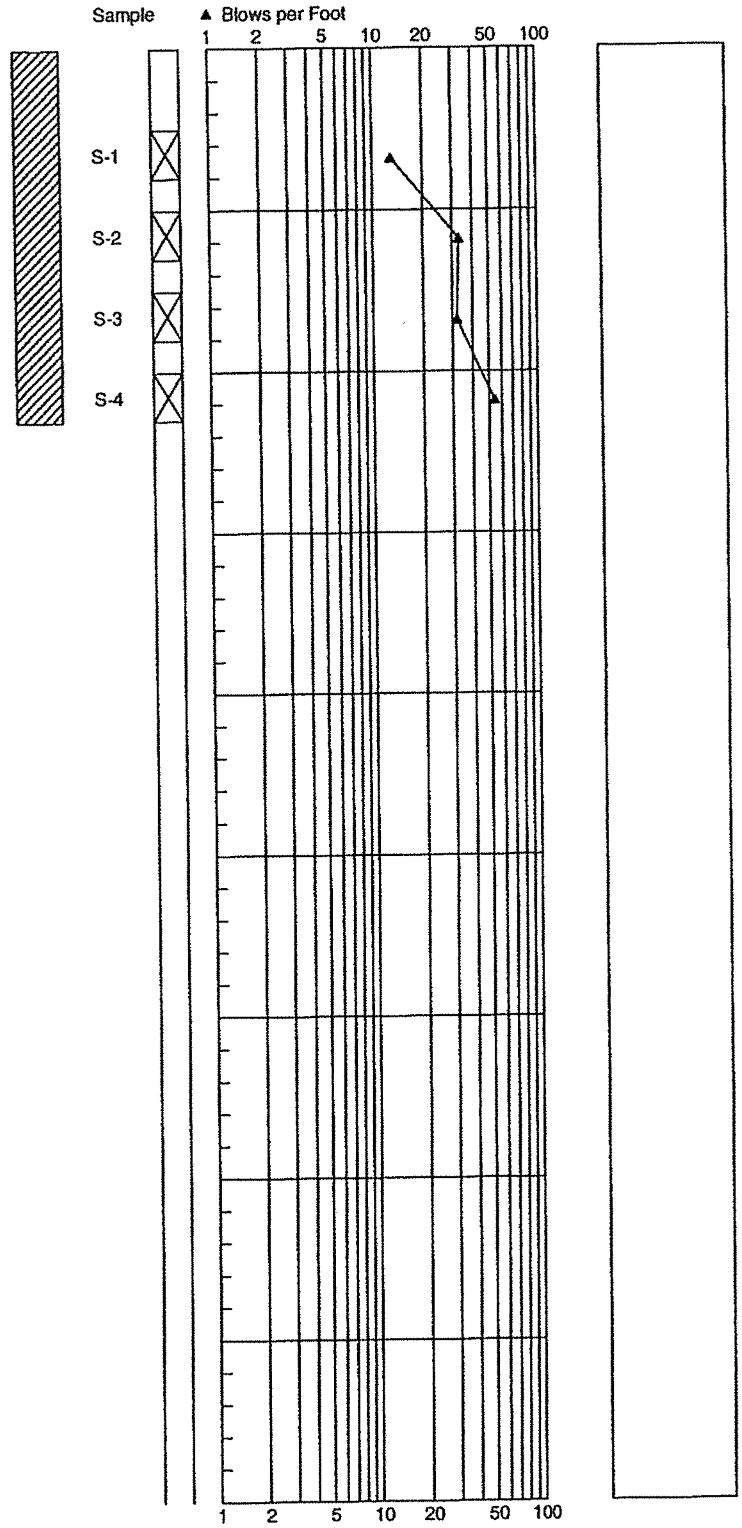
Bottom of Boring at 11.5 Feet.
Completed 03/14/02.

No Seepage or Groundwater Noted.

0 5 10 15 20 25 30 35 40 45

STANDARD PENETRATION RESISTANCE

LAB TESTS



BORING LOG WITH BLOW COUNTS B_15272.GPJ HC_CORP.GDT 4/16/02

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15272 3/02

Figure A-7

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Boring Log B-7

Soil Descriptions

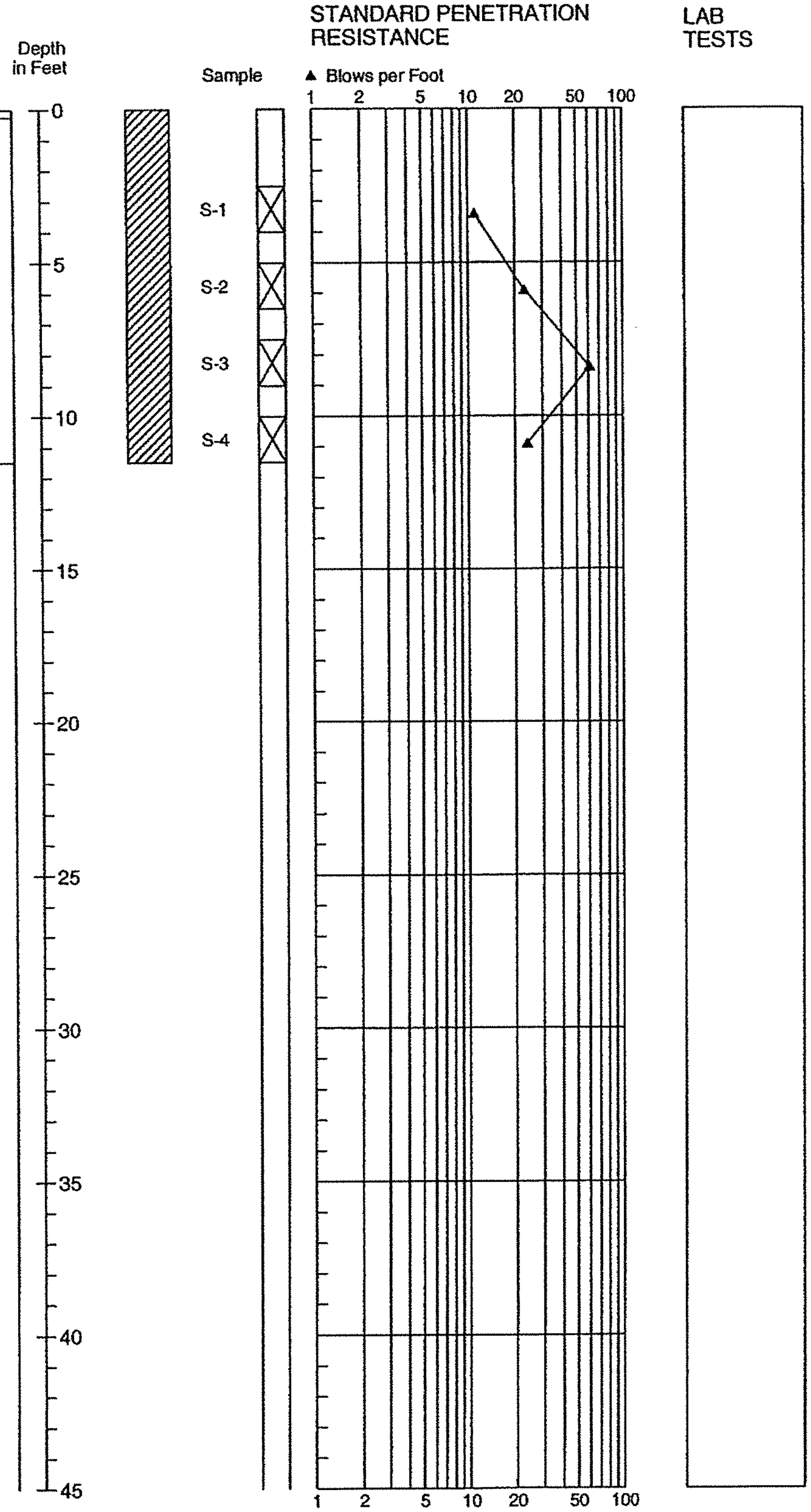
3" Asphalt concrete.
Medium dense, damp, brown, coarse SAND and rounded GRAVEL.

Becomes dense and gray.

Becomes very dense.

Bottom of Boring at 11.5 Feet.
Completed 03/14/02.

No Seepage or Groundwater Noted.



BORING LOG WITH BLOW COUNTS B_15272.GPJ HC_CORP.GDT 4/16/02

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-8

Boring Log B-8

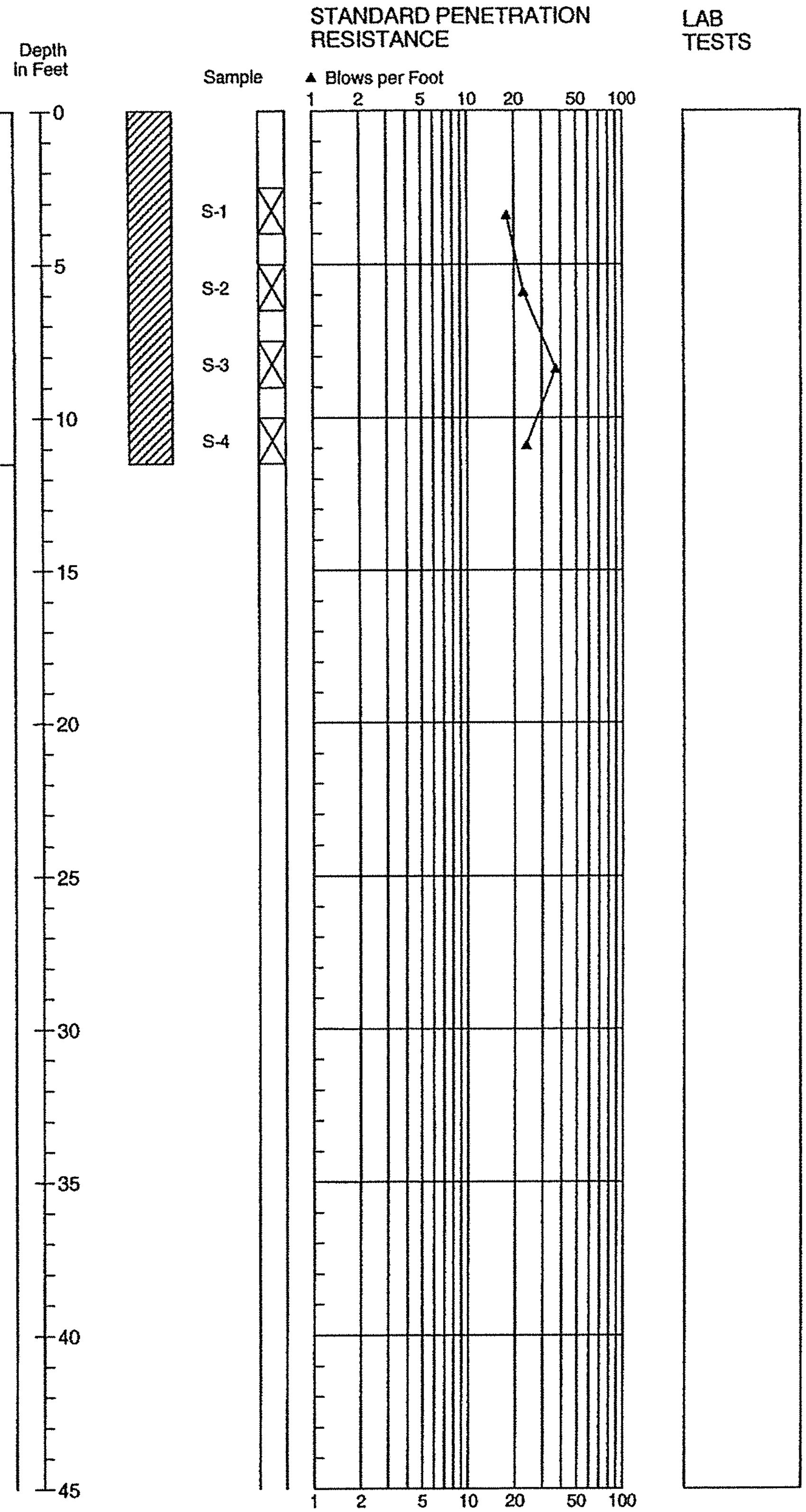
Soil Descriptions

4.5" Asphalt concrete over medium dense, damp, brown, silty, coarse SAND and rounded GRAVEL.

Becomes dense and gray.

Bottom of Boring at 11.5 Feet.
Completed 03/14/02.

No Seepage or Groundwater Noted.



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

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Figure A-9