amec

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March 22, 2001
1-61M-09568-1

Mr. Bill Hardt
Weston Holding Company
2154 NE Broadway, Suite 200
Portland, Oregon 97232

Dear Mr. Hardt:

RE: REVIEW LETTER
RIVER PARK PLAZA
6530 SW MACADAM AVENUE
PORTLAND, OREGON

This letter summarizes AMEC Earth & Environmental, Inc. (AMEC) review of the new development at the above referenced site. We understand that the project consists of constructing a building with two levels of below grade parking and three above grade stories for office space.

AMEC completed a geotechnical engineering report for this site titled "Geotechnical Investigation, River Park Plaza, Portland, Oregon" dated October 1997 (File No. 7-61M-09568-0). Based on our review of the report and current drawings, the design recommendations included in the report can be used for the building that is presently proposed. AMEC conducted additional explorations and design to address additional geotechnical engineering topics. The report will be available on March 23, 2001.

We appreciate this opportunity to be of assistance to you. If you have any questions or require further information, please feel free to contact us at (503) 639-3400.

Sincerely,

AMEC Earth & Environmental, Inc.

Marcella M. Boyer
Project Geotechnical Engineer

c Doug Greenwalt, Howard S. Wright Construction Co.

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AMEC Earth & Environmental, Inc.
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Tel +1 (503) 639-3400
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www.amec.com

01-129413 EXC 3/27/01

amec

Memo

To Troy Dickson
Howard S. Wright Construction Co.
888 SW 5th Avenue, Suite 415
Portland, Oregon, 97204

Date March 9, 2001

Tel (503) 220-0895

File No. 1-61M-09568-1

Fax (503) 220-0892

From: Marcy Boyer

Subject River Park Plaza
Portland, Oregon

The purpose of this memorandum is to provide you with our recommendations concerning the soldier pile shoring for the above referenced project. We understand that the shoring will be used as the basement wall when construction is complete.

For this project, we recommend the following.
Active Equivalent Unit Weight: 35 pounds per cubic foot (pcf)
Passive Equivalent Unit Weight: 450 pcf
Water Unit Weight: 62.4 pcf
Silt Moist Unit Weight: 105 pcf
Troutdale Wet Unit Weight: 125 pcf

Please call if we can provide additional information or services at this time.

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GEOTECHNICAL INVESTIGATION

RIVER PARK PLAZA
PORTLAND, OREGON

Submitted To:

Weston Holding Company
2154 NE Broadway, Suite 200
Portland, Oregon 97232

7-61M-09568-0

October 1997

HSWCC
RC #001
REVISION CONTROL NUMBER

AGRA Earth & Environmental

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

AGRA Earth &
Environmental, Inc.
7477 SW Tech Center Drive
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October 20, 1997
7-61M-09568-0

Mr. Joseph E. Weston
Weston Holding Company
2154 NE Broadway, Suite 200
Portland, Oregon 97232

Dear Mr. Weston:

RE: GEOTECHNICAL INVESTIGATION
RIVER PARK PLAZA
PORTLAND, OREGON

In accordance with your authorization and our proposal dated September 19, 1997, AGRA Earth & Environmental, Inc. (AEE), is pleased to present this geotechnical investigation report for the proposed four- to six- story office building with a one level basement at the southeast corner of SW Macadam and SW Nebraska in Portland, Oregon. We appreciate the opportunity to assist you and look forward to continued involvement on this and other projects.

If you have any questions regarding this report or desire further information, please feel free to contact the undersigned at (503) 639-3400.

Sincerely,

AGRA Earth & Environmental, Inc.

Heather Devine

Heather Devine
Geotechnical Engineering Staff

for

A. Wesley Spang, PhD. P.E.
Principal Geotechnical Engineer

HSWCC
RC #001
REVISION CONTROL NUMBER

HD/klp

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List of Appendices

SUMMARY

This report presents the findings, conclusions and recommendations of our Geotechnical Investigation performed for the proposed development of the property located on the southeast corner of SW Macadam Avenue and SW Nebraska Street. It is the opinion of AGRA Earth & Environmental, Inc. (AEE), based on the results of our investigation, that the site is geotechnically suitable for the proposed construction of the four- to six- story office building with basement subject to the recommendations in this report. Key geotechnical design criteria are summarized below, and are discussed in greater detail in the following sections of this report.

- Near-surface soil conditions disclosed by the site reconnaissance and subsurface explorations consist of asphalt pavement, underlain by soft clayey silts/silty clays to depths varying from 12.5 to 16 feet below the ground surface. This layer is underlain by a dense to very dense gravel and weathered rock layer (Section 4.0).
- Groundwater was encountered approximately 0.5 feet above the gravel layer in all of the borings at the time of drilling at depths of 12 to 15.5 feet below the ground surface.
- A granular working blanket may be necessary to minimize disturbance to subgrade soils, particularly during wet weather (Section 7.0).
- The proposed buildings can be supported on shallow foundations designed for a maximum allowable soil bearing pressure of 6,000 psf for foundations placed on undisturbed gravel (Section 7.1.2).
- Basement retaining wall design criteria and perimeter footing drainage systems are provided. If moisture control is of concern, underslab drainage systems should also be provided (Section 7.2).
- A shoring system will be required to support the proposed 10- to 15- foot-deep site excavations. If expected excavations exceed 15 feet, additional subsurface investigation is recommended to provide additional foundation and shoring design recommendations (Section 7.3).
- Pavement designs are provided for the near- surface silts and the dense gravels as subgrade (Section 7.5).
- The UBC spectra for soil S1 and a zone factor of 0.3 (Zone 3) is recommended for seismic design of the building (Section 7.6).

The preceding summary is intended for introductory and reference use only. Final design should be based on the information and recommendations discussed in this report.

1.0 INTRODUCTION

This report presents the results of subsurface exploration and geotechnical engineering analyses prepared by AGRA Earth & Environmental, Inc. (AEE) for the proposed four- to six-story office building with one level basement. The location of the site is shown on the attached Site Location Map, Figure 1. The approximate locations of the subsurface explorations performed at the site are shown on Figure 2, Site Plan.

The purpose of this work was to establish general subsurface conditions at the site on which to base our recommendations regarding site preparation, excavation, foundation design, drainage, pavement design, and other pertinent geotechnical design criteria and construction considerations.

The scope of work for this project consisted of surficial reconnaissance, review of general geologic and geotechnical literature for the project vicinity, subsurface explorations including four drilled borings, laboratory testing, geotechnical engineering analyses, and the preparation of this report. This study has been accomplished in accordance with generally accepted geotechnical engineering practices for the exclusive use of Weston Holding Company, L..L.C. and their agents for the specific application to the above described project.

This work has been completed in general accordance with AEE's proposal, P97-522, dated September 16, 1997, and entitled "Proposal for Geotechnical Exploration, Proposed Building, SW Macadam and SW Nebraska Avenue, Portland, Oregon." Authorization to proceed with this work was granted by Joseph E. Weston of Weston Holding Company, L..L..C. on September 19, 1997.

2.0 SITE AND PROJECT DESCRIPTION

The property is located at 6530 SW Macadam Avenue on the southeast corner of S.W. Macadam Avenue and S.W. Nebraska Street, Portland, Oregon. The property is primarily rectangular in shape bordered by the Southern Pacific Railroad lines and Willamete park to the east, S.W. Macadam Avenue to the west, S.W. Nebraska Street to the north and a one-story structure to the south. The site is relatively flat with an approximate elevation of 43 feet MSL (Sienna Architects). Existing improvements at the site include a paved access road and parking lot, landscaping areas, curbs, and a structure comprised of connected railroad cars.

The project, as we understand it, is to consist of a four- to six- story office building with a one level basement. Based on discussions with Sienna Architects, the column and perimeter footing loads are on the order of 1100 kips and 6 kips per lineal foot, respectively. It is understood that the building basement will be at approximate elevation 33 ft msl. Thus, the planned depth of excavation will be approximately 10 to 12 feet. Access drives and landscaping areas for the project are also anticipated.

3.0 GEOLOGIC AND SEISMIC SETTING

3.1 GEOLOGIC SETTING

The near-surface geology of the project vicinity consists of Quaternary age river and stream deposits of silt, sand, and gravel composed of mixed lithologies. These soils may include local lacustrine, paludal, and eolian deposits. This alluvial layer is underlain by Pleistocene age catastrophic flood deposits of boulders, gravels, sandy gravels, and sands containing high percentages of Columbia River basalt clasts. This deposit is a result of the high energy, subfluvial deposition that occurred during the catastrophic floods that were caused by the repeated failure of the glacial ice dam that impounded glacial Lake Missoula.

The last of these glacial floods, also thought to be one of the largest, occurred about 12,400 years ago, establishing the minimum age of the silt and sand deposit. The near-surface silts appear to have weathered during soil horizon development and contain minor proportions of clay.

Based on the boring logs, depth of auger refusal and published records, we estimate the top of basalt bedrock to be approximately 25 to 50 feet below the ground surface.

There are no landslide or slope stability hazards at the site due to the relatively flat topography of the site and surrounding areas.

3.2 SEISMIC SETTING

The seismicity of the Portland Metropolitan area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include 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 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 seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Oregon coast. Sequences of interlayered peats and sands have been interpreted to be the result of large subduction zone earthquakes occurring at intervals on the order of 300 to 500 years with the most recent event taking place approximately 300 years ago. A recent study by Geomatrix (1995) suggests that the maximum earthquake associated with the CSZ is moment magnitude 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes which have occurred within subduction zones in other parts of the world.

The intraplate zone encompasses the portion of the subducting Juan De Fuca Plate located at a depth of approximately 30 to 50 km below western Oregon. Very low levels of seismicity have been observed within the intraplate zone in Oregon. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of subduction between Oregon and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone include the 1949 Olympia (Magnitude 7.1) and the 1965 Puget Sound (Magnitude 6.5) earthquakes.

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 western Oregon 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.

This section has been provided for introductory purposes only. A site specific seismic study is beyond the scope of this work. UBC seismic design criteria is recommended in Section 7.6.

4.0 SITE EXPLORATION SUBSURFACE CONDITIONS

4.1 SITE EXPLORATION

The field investigation for this project was conducted on September 29, 1997. The investigation consisted of surficial geotechnical reconnaissance of the project area and subsurface exploration of the parcel. The subsurface exploration consisted of 4 drilled borings. The borings were completed by subcontracted drilling services. Subsurface exploration locations were determined by pacing from identifiable topographic and structural features shown on the furnished site plan and should be considered approximate.

Subsurface materials were sampled at selected intervals and classified and logged in the field by a member of AEE's geotechnical engineering staff. Samples were returned to the soils laboratory for further examination and testing. Boring logs are included in Appendix A, at the end of this report. The following descriptions of the site soils are based on the results of the borings as well as a review of previous exploratory borings performed by our firm for nearby properties.

4.2 SUBSURFACE CONDITIONS

The subsurface exploration borings depict subsurface conditions only at the specific locations at the time of exploration. The borings are spaced widely across the site and it is possible that some local variations and possibly unanticipated subsurface conditions exist. The passage of time may also result in changes in the conditions interpreted to exist at the locations where sampling was conducted. Based on conditions observed during reconnaissance and on the exploratory borings we have interpreted the subsurface conditions encountered at the site.

ASPHALT PAVEMENT: The site is covered with approximately 2 inches of asphalt concrete pavement underlain by approximately 6 inches base rock. Railroad tracks and landscaping material are also located at the surface. Rubble and loose fill was encountered to a depth of approximately 5.5 feet in B-4. The asphalt, railroad tracks, landscaping fill, rubble and loose fill material is not suitable for the placement of foundations, slabs, pavement or structural fill.

SILT: Brown, moist to wet, very soft to medium stiff silt is present below the pavement layer over the majority of the site to a depth varying from 12.5 to 16 feet below ground surface (bgs). These silty soils will be difficult to work due to their high moisture contents. Additionally, the silts are anticipated to be compressible under the proposed building loads.

DENSE GRAVEL: Dense to very-dense, poorly-graded gravel of the Troutdale Formation was encountered at depths varying from 12.5 to 16 feet bgs. All four exploratory borings encountered refusal within these gravels. Since little to no samples were recovered during split-spoon sampling in the gravel, it is difficult to identify the matrix soil. However, the drill cuttings indicate that the matrix material consists of weathered rock, silt and sand of the Troutdale Formation. The action of the drill rig suggests that the gravels may be on the order of 2 to 12 inches in size. A review of the "Geologic Map of the Lake Oswego Quadrangle" prepared by the Oregon Department of Geology and Mineral Industries (DOGAMI) indicates that the Troutdale Formation is underlain at depth by Columbia River basalt.

Groundwater was encountered in all of the borings at the time of drilling at depths ranging from 12.0 to 15.5 feet below the ground surface. Even during dry weather excavations within the site may be expected to encounter groundwater. Dewatering may be required for excavations that must remain open for a significant length of time.

5.0 LABORATORY TESTING

Laboratory testing was conducted on selected soil samples from the borings to aid in classification and evaluation of engineering properties of soils. Laboratory testing included in-situ moisture content, Atterberg Limits, California Bearing Ratio Tests (CBR), and maximum density curves. Laboratory testing was conducted in accordance with ASTM or other accepted testing standards.

6.0 DISCUSSION

Based on our investigation, the site appears to be geotechnically suitable for the proposed four- to six- story structure with a one level basement. The subsurface exploration conducted at this site encountered dense to very dense gravel underlying the 12.5 to 16 foot layer of soft clayey silt. Approximately 5 feet of rubble was also encountered at the southeast corner of the site. Based on the preliminary building section provided by Sienna Architects, it is our understanding that 10 to 12 feet of the existing soil will be excavated to accommodate the one level basement. Thus, approximately three to five feet of clayey silt will be left in place. It is our recommendation that the soil exposed at the basement subgrade be overexcavated by 18 inches. A geotextile fabric should then be installed on the soil and the excavation backfilled with 12 inches of pit run rock (6 inches maximum size). The basement slab base rock consisting of a minimum of six inches of one-inch minus rock should be placed above the pit run rock. The building foundations should bear on undisturbed gravels. This can be done by overexcavating the soils down to rock and backfilling with crushed rock or lean concrete. Figure 3 shows a typical detail for this type of construction.

As an alternate, the soft clayey silts can be completely overexcavated and the basement floor slab and footings founded on the dense gravel layer or on compacted granular fill.

The subsurface explorations conducted for this project encountered static groundwater at depths of approximately 12.0 to 15.5 feet below the ground surface. Excavation for the basement should be expected to encounter groundwater requiring construction dewatering. Dewatering systems are typically designed and constructed by the contractor. Perimeter footing drains are recommended around all exterior footings. Underslab drains are recommended if floor moisture is a concern.

Due to the proximity of existing structures, high traffic volume on adjacent streets and the expected depth of excavation, it is recommended that a soldier pile and wood lagging shoring system be implemented during construction. Difficult installation conditions may be encountered during the placement of the soldier piles within the dense gravels encountered at 12.5 to 16 feet below the existing ground surface.

The subsurface investigation completed for this site was based on an expected depth of excavation of 10 to 15 feet bgs for a single level basement. If anticipated excavation depths should exceed 15 feet an additional subsurface investigation is recommended to evaluate the soil and rock conditions within the depth of influence of the foundation loads.

7.0 GEOTECHNICAL RECOMMENDATIONS

7.1 SITE PREPARATION

Prior to beginning construction, all areas of the site should be cleared of previous improvements, including pavement, curbs, landscaping fills, abandoned utilities, etc. Site preparation would include excavation operations for the building basement. Excavation depths are anticipated to range from approximately 10 to 15 feet. These depths of excavation are expected to remove most of the soft clayey silt and fill material from the building footprint. However, it is possible that utilities, septic tanks, or previous building remnants might be encountered during construction. If encountered the improvements should be removed and the resulting excavation backfilled using compacted granular fill. Surface and subsurface water should be controlled through drainage structures.

It is recommended that site excavation and foundation construction operations occur during dry weather to minimize disturbance to exposed soils. The on-site soils are highly moisture-sensitive and upon saturation become difficult to use as a firm, stable working surface. Also, the groundwater table may rise during wet months complicating dewatering operations.

During wet weather or when adequate moisture control can not be maintained for the fine-grained soils, it may be necessary to install a granular working blanket to support construction equipment and personnel. The working blanket for this project should consist of crushed rock or gravel. It is recommended that AEE be consulted to approve this material before installation.

7.1.1 Basement Slab

The current site elevation is approximately 43 feet above msl. The top of basement slab will be at 33 feet above msl (Sienna Architects). Thus, approximately five feet of soft to medium stiff clayey silt will be left in place. It is our recommendation that the subgrade soil be overexcavated by 18 inches, and backfilled with 12 inches of pit run rock (6- inch maximum size). A geotextile separator should be installed between the silt and rock. The basement slab base rock consisting of a minimum of six inches of one-inch minus rock should be placed above the pit run as slab base rock. Figure 3 shows a typical detail for this type of construction.

As an alternate to the above, the silt soil may be excavated down to dense gravel. The resulting excavation should be backfilled using pit run rock or other suitable granular fill material. A minimum six-inch-thick compacted crushed rock or gravel layer (1 inch minus) should be installed over the fill as slab base rock. The crushed rock or gravel should be poorly-graded, angular, and contain no more that 5% fines passing a #200 sieve.

7.1.2 Foundations

Building foundations should be placed on undisturbed gravels. This can be done by overexcavating the soils down to rock and backfilling with crushed rock or lean concrete. Figure 3 shows a typical detail for this type of construction. Foundations for the office building and basement parking garage should have a minimum width of 18 inches and a minimum depth of embedment of 18 inches below the lowest adjacent pad grade. Foundations having these minimum dimensions and placed on dense, natural gravels or compacted granular fill may be designed for an allowable soil bearing pressure of 6,000 psf. The bearing pressure may be increased by one-third for short term transient loading due to wind or seismic forces.

Settlement analyses indicates that the total settlements for the building footings designed in accordance with the above recommendations will be less than one inch. The majority of the foundation settlements are expected to occur concurrently with the application of the structural loads. Differential settlements between adjacent footings are anticipated to be less than 1/2 inch.

For passive pressures used to resist lateral loads a 400 pcf equivalent fluid unit weight may be used for the site soils. A base friction equal to 40% of the vertical load may be used along the bottom of foundations as sliding resistance.

7.1.3 Vapor Retarders

Ground moisture may be abundant under the basement slab during the life of the project. The difference in moisture content between the air in the subgrade materials and the air in the finished building will cause water vapor to travel upward. The resultant water vapor pressure will force migration of moisture through the slab. This migration can result in the loosening of flooring materials attached with mastic, the warping of wood flooring, and, in extreme cases, mildewing of carpets and building contents. For most finished buildings, the presence of floor moisture would be considered a significant detriment to the tenants. Parking garages are less susceptible to floor moisture due to higher rates of evaporation.

If moisture control is a concern we recommend installing a membrane between the crushed rock and the basement slab to retard the migration of moisture into the slab. This function is analogous to the use of insulation to retard heat flow through exterior walls. Vapor retarders will frequently need to be perforated 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.

To maximize its effectiveness, the membrane must be installed in accordance with the manufacturer's recommendations. A 6 mil polyethylene retarder is suitable if the contractor takes care not to damage or tear the material during installation. Normally, a thin sand layer is placed both above and below 6 mil membranes to protect the retarder from excessive punctures during construction. An alternative to this detail is to increase the polyethylene thickness to 10 mils to improve puncture resistance.

Modern design has resulted in the creation of cost-effective concrete mixes. Such mixes are susceptible to slab curl and cracking. Both are caused by differential moisture loss in the concrete gel. A layer of sand placed above the membrane, below the slab can allow moisture to dissipate from the bottom of the slab. Alternately, it is possible to design concrete mixes that are not particularly susceptible to these problems. The use of such mixes may allow the slab to be poured directly on top of the vapor retarder.

7.1.4 Underslab and Perimeter Foundation Drains

The basement slab will likely encounter wet conditions at subgrade elevations. An underslab drainage system will assist in reducing the potential for high water to result in hydrostatic pressures against the bottom of the floor slabs. Typical underslab drains are constructed with 4 inch perforated PVC pipe embedded in a 12 inch layer of open graded crushed rock. The PVC pipes can also be installed in the pit run rock. A typical detail of an underslab drain is presented in Figure 3a.

Perimeter foundation drains are recommended around the building perimeter adjacent to the footing base. The purpose of perimeter drains is to protect against lateral migration of groundwater. AEE recommends perimeter drains to ensure that the soil surrounding the foundation can drain rapidly whenever necessary.

Perimeter foundation drains are usually constructed at, or slightly below, the base elevation of the footings using four-inch diameter perforated PVC pipe bedded in drain rock and sloped to drain by gravity. A geotextile separator is generally placed between fine-grained soils and the drain rock to minimize infiltration of fines into the rock. A typical detail of a perimeter drain is presented in Figure 3b.

In addition, positive surface drainage should be maintained away from the building foundations during construction. The finish grading should also provide for permanent, positive surface drainage away from the building. Surface water sources such as roof drains and parking lot runoff should be routed independently through non-perforated drain lines to a storm water collection system. Surface water should not be allowed to enter subsurface drainage systems.

7.2 BASEMENT RETAINING WALLS

7.2.1 Restrained Walls

Restrained walls are any walls that are prevented from rotation. Most basement or below-grade walls are restrained by a floor slab or roof and fall into the category of restrained walls.

AEE recommends that restrained walls be designed for the equivalent fluid unit weights shown below.

Backfill Slope Horizontal:Vertical	Equivalent Fluid Unit Weight (lbs./cu. ft.)
Level	55
3H:1V	65
3H:1V	75

These values represent AEE's 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. It is recommended that a uniform lateral surcharge pressure of 100 psf be applied to basement walls adjacent to the streets to account for traffic surcharge. If any foundations or major loads will be located adjacent to basement walls, AEE should be contacted to evaluate the resulting lateral surcharge pressures.

7.2.2 Non-Restrained Walls

Non-restrained walls have no restraint at the top and are free to rotate about their base. Most cantilever retaining walls fall into this category. It is anticipated that minor cantilever walls may be constructed for incidental and landscaping purposes. AEE recommends that non-restrained walls be designed using the equivalent fluid unit weight shown herein.

Backfill Slope Horizontal:Vertical	Equivalent Fluid Unit Weight (lbs./cu. ft.)
Level	35
3H:1V	45
3H:1V	55

This represents AEE's best estimate of actual pressures that may develop and do not contain a factor of safety.

If backfill is in direct contact with the wall, lateral forces can be assumed to act at a downward inclination of 20 degrees from horizontal, which will increase the wall stability. If friction is prevented by drainage membranes or water proofing membranes, then the forces should be assumed to act horizontally.

7.2.3 Retaining Wall Backfill

Restrained walls are anticipated to be constructed against the shoring system and thus do not require backfill. Backfill behind non-restrained retaining walls should consist of free-draining granular material. Overcompaction of this fill can greatly increase lateral soil pressures. We recommend that this fill be compacted to between 90% and 92% maximum dry density as determined by ASTM D-1557. In addition, AEE recommends that all fill within about five feet of non-restrained retaining walls be compacted with lightweight, hand-operated equipment.

7.2.4 Retaining Wall Drainage

The groundwater surface was encountered at 12 to 15.5 feet below the ground surface. It is also possible that near-surface seams of perched water may develop due to the layering of silt and clay soils. We recommend that basement and retaining walls be provided with adequate drainage. Drains should be protected by a filter fabric to prevent internal soil erosion and potential clogging. It is anticipated that basement wall drains will consist of manufactured drainage panels placed directly against the excavation support system.

Drains should be sloped to drain by gravity or should be collected in a sump and pumped to a storm sewer or other positive outlet. Surface water should be independently collected and routed to a storm sewer. This water must not be allowed to enter the subsurface drainage system.

7.3 EXCAVATION SHORING

Excavations on the order of 10 to 15 feet will be required to establish the below-grade levels of the proposed structures. It is recommended that the excavation shoring system consist of either soldier piles with wood lagging or a soil nail structure with shotcrete facing.

A typical soldier pile shoring system consists of excavating 24- to 30-inch-diameter drilled piers at spacings of 6 to 10 feet and installing a steel H-pile within the drilled pier excavation. The portion of the pier excavation below final grade is filled with structural concrete while the remaining portion is filled with lean concrete. As site excavation proceeds, wood lagging is continuously installed between the H-piles to laterally support the sides of the excavation. Tiebacks which extend behind the excavation face are used if additional lateral support of the excavation is required. The very dense weathered Troutdale Formation encountered at depths of 12 to 16 feet below the existing ground surface consists of gravels and cobbles within a weathered rock matrix. It should be anticipated that difficult excavation will be encountered during drilling within these dense gravels and cobbles for the construction of the piers.

Soil nailing is one alternative to soldier piles where the excavated soils are relatively competent. Soil nail systems are constructed during site excavation by placing 1- to 2-inch-diameter steel reinforcing bars ("soil nails") on a uniform horizontal and vertical pattern. A shotcrete facing is applied to the exposed soil surface to prevent sloughing and raveling. The soil nails can be installed pneumatically or placed and grouted within 6- to 10-inch-diameter borings excavated with a continuous hollow-stem auger.

AEE can provide geotechnical design criteria for soldier piles or soil nail excavation support systems when project plans become more definite.

It is recommended that the following criteria be included in the project excavation specifications:

1) Horizontal movement of the shoring system, and vertical settlement of the adjacent streets and buildings should be monitored during site excavation. Horizontal movements should be determined with inclinometers placed behind the shoring system at a minimum of two locations. Street and building settlements should be surveyed at a minimum spacing of 50 feet around the perimeter of the excavation.

2) The design of the shoring system should be sufficiently rigid so that horizontal movements do not result in distress to adjacent improvements such as streets, buildings, utilities, etc.

3) The height of unsupported vertical excavations should be limited to 4 feet.

4) Positive drainage should be provided away from the top and bottom of the excavation system.

5) Any voids that develop between the shoring facing and the excavated soil should be backfilled with lean concrete or clean gravel as soon as possible.

In addition the owner and the contractor should make themselves aware of and become familiar with applicable local, state, and federal safety regulations, including current OSHA excavation and trench safety standards. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. AEE is providing this information solely as a service to the Weston Holding Company. Under no circumstances should the information provided be interpreted to mean that AEE is assuming responsibility for construction site safety or the contractor's activities.

7.4 EROSION CONTROL

The near surface silty soils at this site are moderate to highly erodible, and any exposed soil may be subject to erosion by running water. AEE 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 a storm water disposal system. Silt fences should be established and maintained throughout the construction period down slope from all construction areas to protect the natural drainage channels from erosion and/or siltation.

7.5 PAVEMENT DESIGN

Pavement designs for both the near surface silts and the dense gravels at subgrade level are presented in the following tables.

Alternate pavement designs for both asphalt and Portland Cement Concrete (PCC) are presented in the tables below. All designs have been prepared in accordance with accepted AASHTO design methods. We have provided a range of pavement designs for various traffic conditions. These pavement sections are provided in Table 7.5, Pavement Design for Near-surface Silts. The designs assume that the top eight inches of pavement subgrade will be compacted to 95% ASTM D-1557. Specifications for pavements and base course should conform to current Oregon State Department of Transportation specifications, with the addition that the base rock should contain no more that 5% passing the #200 sieve, and that asphaltic concrete should be compacted to a minimum of 91% ASTM D-2041.

Near surface silt samples were analyzed in the laboratory to determine pavement design parameters. We recommend using the following values for native or engineered silt fill:

Relative Compaction	CBR	Resilient Modulus (psi)
95%	4	6,000

Table 7.5.1a: ASPHALT CONCRETE PAVEMENT DESIGN FOR SILT SUBGRADE

Approx. Number of Trucks per Day (each way)	Approx. Number of 18 Kip Design Axle Load (1000)	Asphalt Concrete Thickness (inches)	Crushed Rock Base Thickness (inches)
Auto Parking	10	2.0	7
5	22	2.5	8
10	44	2.5	9
15	66	2.5	10
25	110	3.0	10
50	220	3.5	11
100	440	4.0	12
150	660	4.0	13

Table 7.5.1b: PORTLAND CEMENT CONCRETE PAVEMENT DESIGN FOR SILT SUBGRADE

Approx. Number of Trucks per Day (each way)	Approx. Number of 18 Kip Design Axle Load (1000)	P.C.C. Thickness (inches)	Crushed Rock Base Thickness (inches)
5	22	5.0	0
10	44	5.5	0
15	66	6.0	0
25	110	6.5	0
50	220	6.0	6
100	440	7.0	6
150	660	7.5	6

7.6 UBC SEISMIC DESIGN CRITERIA

Western Oregon is in UBC zone 3, where a Z factor (developed to be equivalent to peak effective horizontal ground acceleration in g's) of 0.30 may be used. A site soil coefficient S_1 with an S factor of 1.0 may be used in base shear calculations. Development of site specific response spectra is beyond the scope of our work and we recommend designing the proposed project in accordance with UBC design criteria and local building codes.

8.0 CONSTRUCTION OBSERVATIONS

The analysis, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and on the assumption that the boring logs are representative of the subsurface conditions throughout the site. It is the nature of geotechnical work for soil conditions to vary from the conditions identified during the geotechnical investigation, even when a normally acceptable program of exploration has been implemented.

While some variations may appear slight, their impact on the performance of structures and other improvements can be significant. An example of unanticipated conditions for this site could include fills or materials from previous buildings or structures, requiring over-excavation and compaction with imported granular fill. It is therefore recommended that AEE be retained to observe the portions of this project relating to geotechnical engineering, particularly the construction of the excavation shoring system, basement excavations and foundations. This will allow AEE to correlate observations and findings with actual soil conditions encountered during construction and to evaluate construction conformance with respect to the recommendations in this report.

Unanticipated soil conditions frequently require additional expenditures to attain a properly constructed project. It is therefore prudent to allow for such unforeseen conditions in both the project schedule and construction budget.

9.0 LIMITATIONS

The recommendations in this report are based on information gathered in our office review and on site conditions observed at the time of the field exploration. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at, or adjacent to the site, it is recommended that AEE be requested to review this report to evaluate the conclusions and recommendations considering the lapse of time or changed conditions.

Conditions beneath individual structures could vary from those presented in this report. AEE requests that a copy of the plans and specifications be forwarded to AEE for review, so that we may evaluate any specific conceptual, architectural, or construction details which might affect the validity of AEE's recommendations, and ensure that AEE's recommendations have been appropriately interpreted.

If you have any questions regarding this report or desire further information, please feel free to contact the undersigned at (503) 639-3400 at your convenience.

AGRA Earth & Environmental, Inc.

Heather Devine

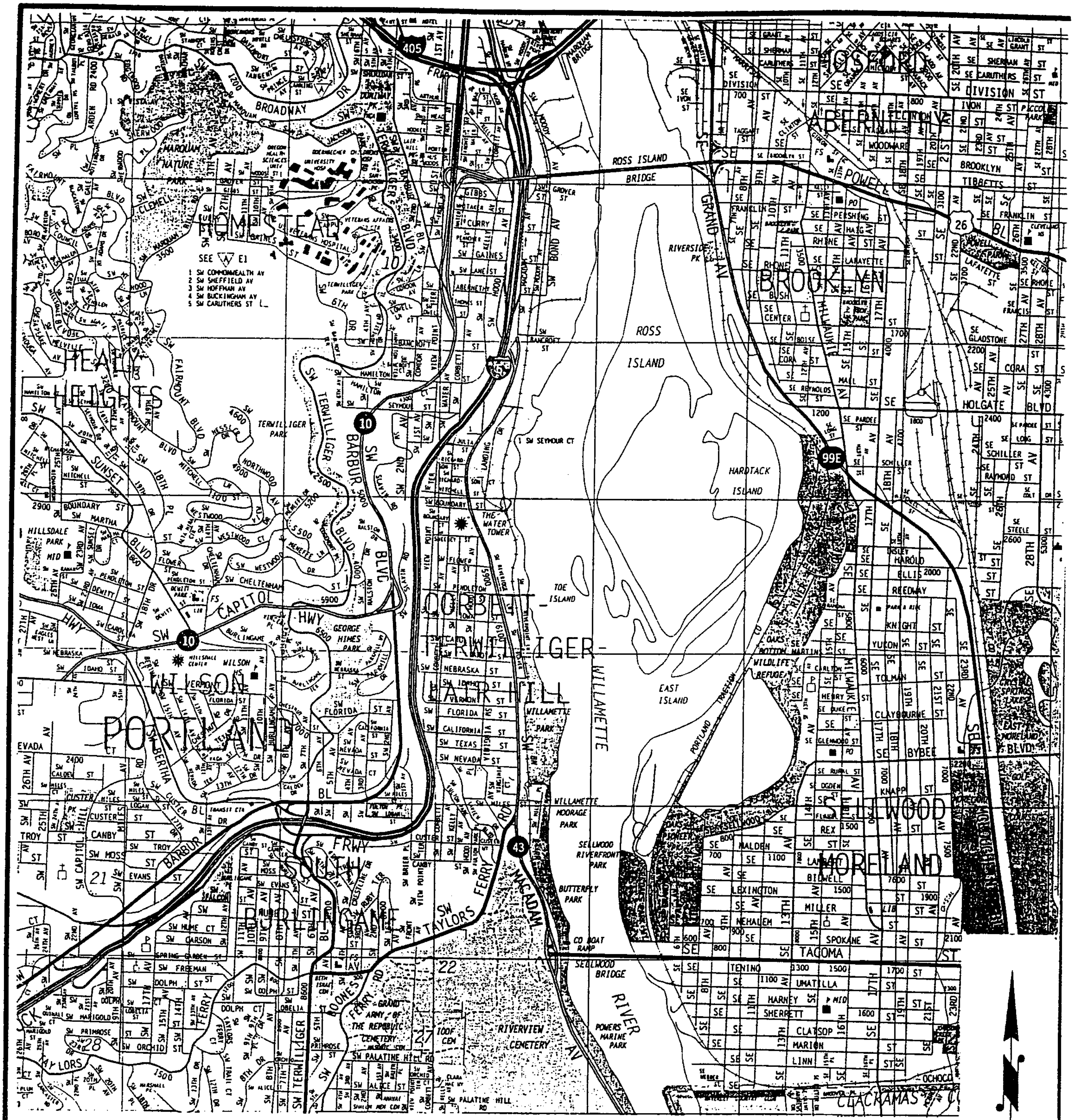
Heather Devine
Geotechnical Engineering Staff

/For

A. Wesley Spang, PhD., P.E.
Principal Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
18,281
OREGON
JANUARY 16, 1996
A. WESLEY SPANG
EXPIRES 12/31/97

HD/klp



HSWCO

RC#001

REVISION CONTROL NUMBER

FIGURE 1

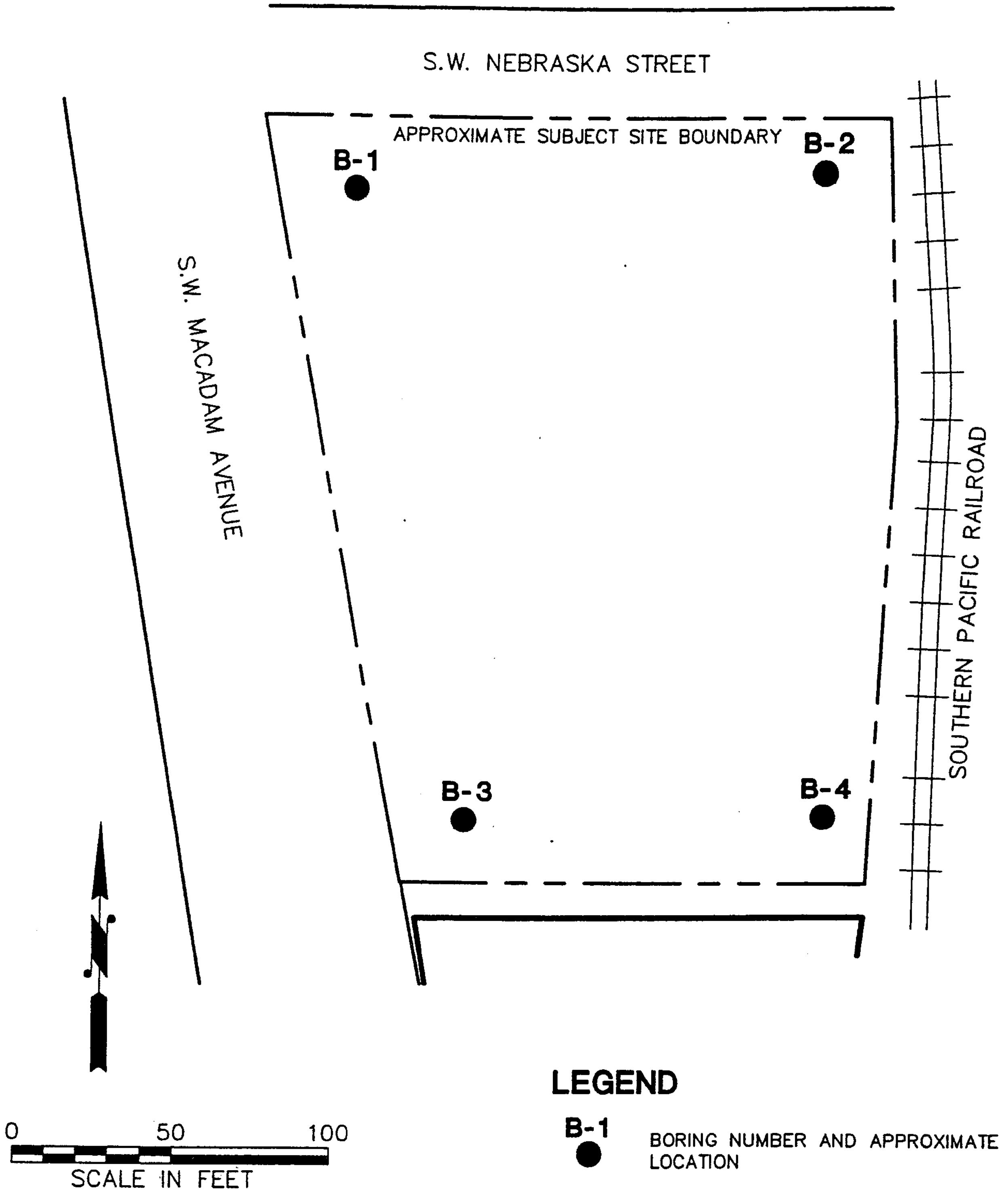
AGRA
Earth & Environmental
7477 S.W. Tech Center Drive
Portland, OR, U.S.A. 97223-8025

W.O.	7-61M-9568-0
DESIGN	HD
DRAWN	DRF
DATE	OCT. 1997
SCALE	NTS

RIVER PARK PLAZA
6530 S.W. MACADAM AVENUE
PORTLAND, OR

SITE LOCATION MAP

AGRA EARTH & ENVIRONMENTAL, INC. DRAWING NO. \PROJECTS\21\09568\SITELOC.DWG



NOTE: EXISTING SITE FEATURES FROM FIELD MEASUREMENTS BY AEE EMPLOYEES. LOCATION OF THESE FEATURES ARE NOT FROM DATA GATHERED BY A REGISTERED LAND SURVEYOR AND SHOULD BE CONSIDERED APPROXIMATE.

FIGURE 2

AGRA
Earth & Environmental
7477 S.W. Tech Center Drive
Portland, OR, U.S.A. 97223-8025

W.O. 7-61M-9568-0
DESIGN HD
DRAWN DRF
DATE OCT. 1997
SCALE 1"=50'

HSWC

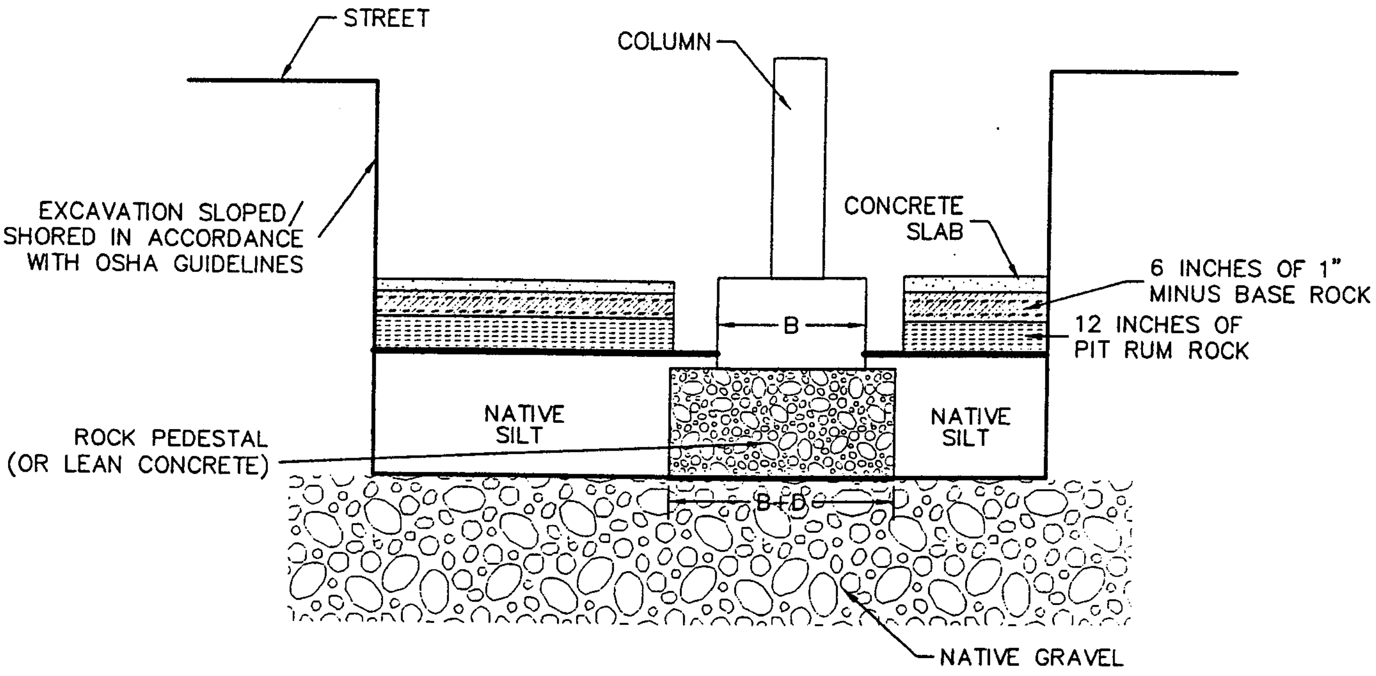
RIVER PARK PLAZA
6530 S.W. MACADAM AVENUE
PORTLAND, OREGON

RC #001

SITE PLAN

REVISION CONTROL NUMBER

AGRA EARTH & ENVIRONMENTAL, INC. DRAWING NO. \PROJECTS\21\09568\SITE.D



HSWCC

RC#001

REVISION CONTROL NUMBER

FIGURE 3

AGRA
Earth & Environmental
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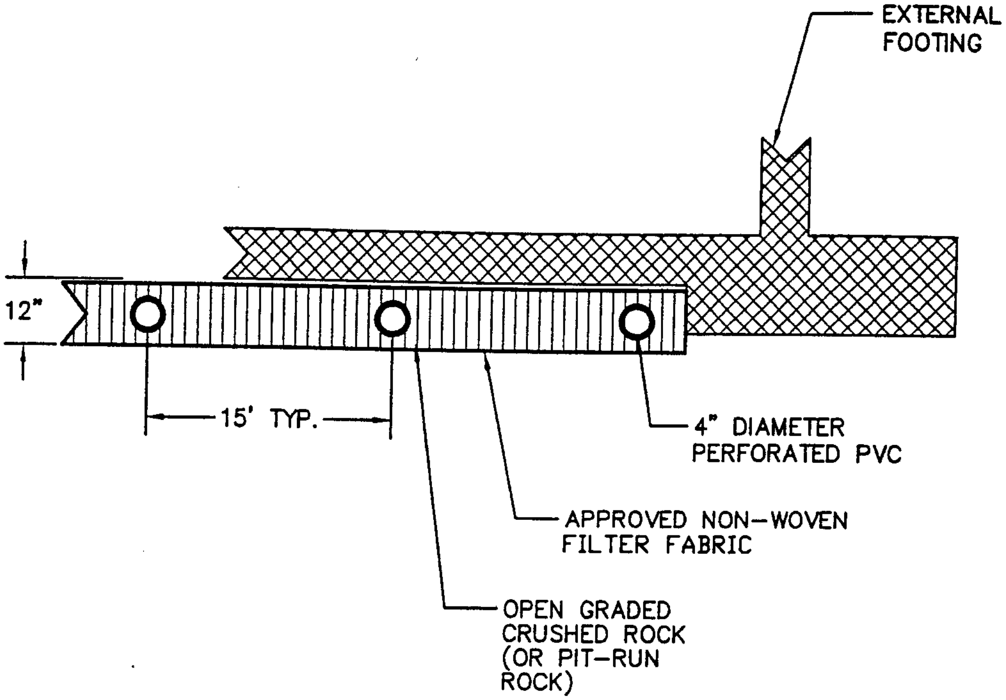
W.O. 7-61M-9568-0
DESIGN RA
DRAWN DRF
DATE OCT. 1997
SCALE NTS

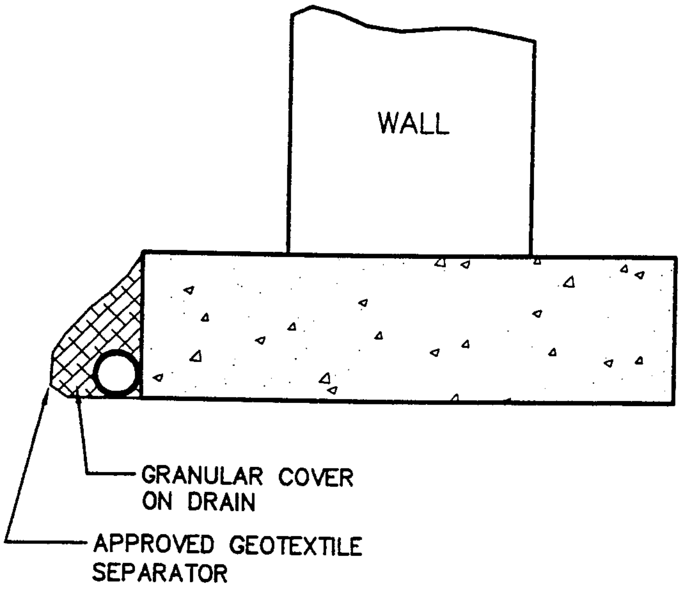
RIVER PARK PLAZA
6530 S.W. MACADAM AVENUE
PORTLAND, OREGON

TYPICAL ROCK PEDESTAL
(SLAB SUB-BASE DETAIL)

AGRA EARTH & ENVIRONMENTAL, INC. DRAWING NO. \PROJECTS\21\09568\RK-PED.DWG



A) UNDERSLAB DRAINS



B) PERIMETER DRAIN

FIGURE 4

AGRA
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Portland, OR, U.S.A. 97223-8025

W.O. 7-61M-9568-0
DESIGN HD
DRAWN DRF
DATE OCT. 1997
SCALE NTS

RIVER PARK PLAZA
6530 S.W. MACADAM AVENUE
PORTLAND, OREGON

TYPICAL PERIMETER AND UNDERSLAB DRAINAGE SYSTEMS

REFERENCES

Geomatrix, "Seismic Design Mapping, State of Oregon", prepared for the Oregon Department of Transportation, 1995.

Hart, D.H., Newcomb, R. C., "Geology and Groundwater of the Tualatin Valley Oregon," Geological Survey Water Supply Paper 1697, United States Department of the Interior, 1965

Kramer, S.L., Geotechnical Earthquake Engineering, Prentice Hall, 1996

Oregon Department of Geology and Mineral Industries, "Geologic Map of The Portland Quadrangle, Multnomah and Washington Counties, Oregon and Clark County, Washington", 1991.

Uniform Building Code, 1994 Edition, International Conference of Building Officials, Whittier, Calif., 1994

APPENDIX A

Field Investigation

APPENDIX A

FIELD INVESTIGATION

The field investigation was performed on September 29, 1997 and consisted of the excavation of 4 exploratory borings at the approximate locations shown in Figure 2. The borings were advanced to depths of 17.5 to 24.75 feet below the ground surface. The borings were advanced by subcontracted drillers using a mobile B-59 drill rig and hollow stem auger. All borings encountered refusal within the dense gravels and cobbles of the Troutdale Formation. Samples were obtained at selected depths for classification and laboratory testing. Logs of the borings are presented on the following pages.

HSWCC

RC #001

REVISION CONTROL NUMBER

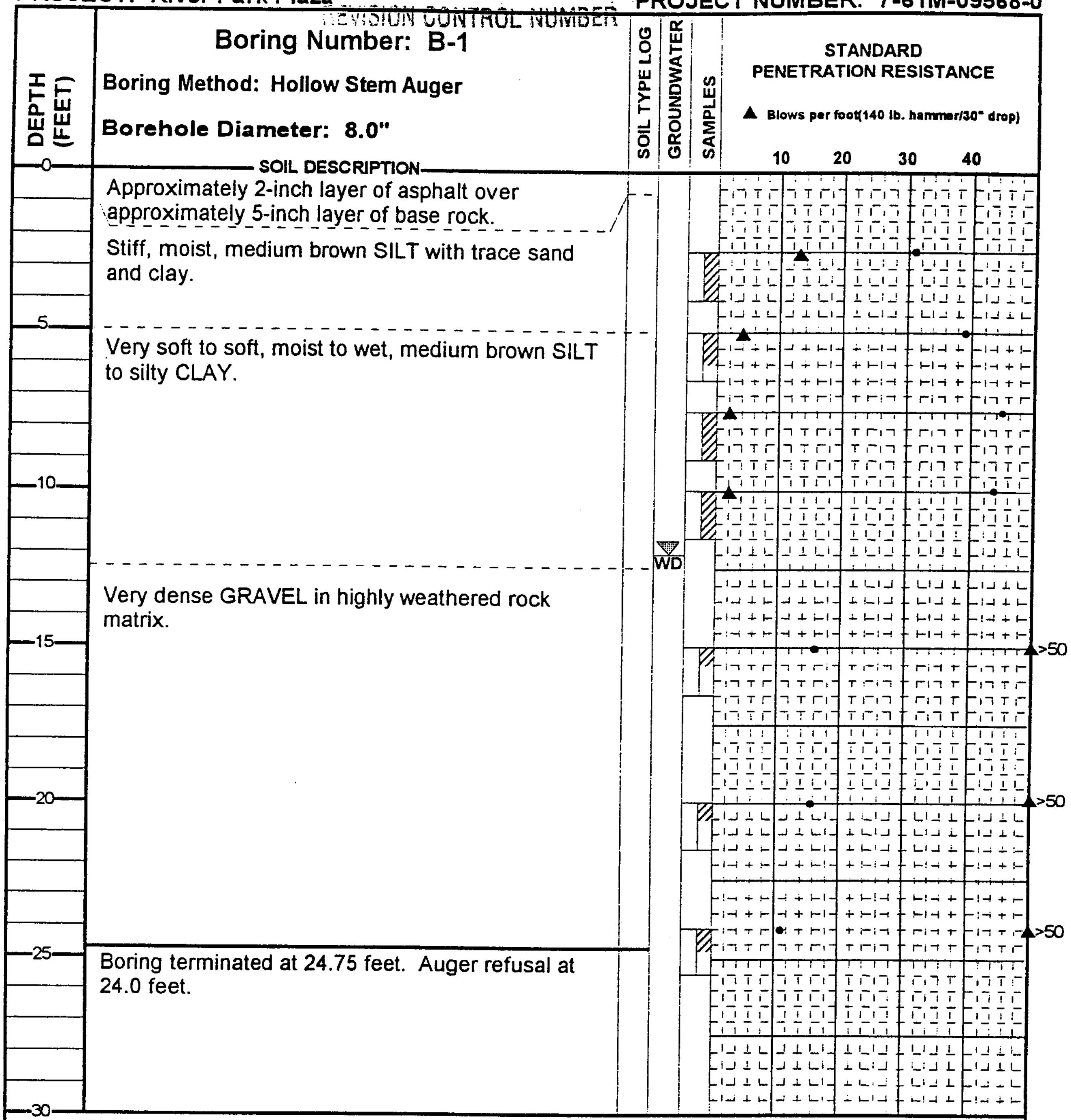
PROJECT: River Park Plaza

PROJECT NUMBER: 7-61M-09568-0

Boring Number: B-1

Boring Method: Hollow Stem Auger

Borehole Diameter: 8.0"



Approximately 2-inch layer of asphalt over approximately 5-inch layer of base rock.

Stiff, moist, medium brown SILT with trace sand and clay.

Very soft to soft, moist to wet, medium brown SILT to silty CLAY.

Very dense GRAVEL in highly weathered rock matrix.

Boring terminated at 24.75 feet. Auger refusal at 24.0 feet.

LEGEND

▨	2.0" O.D. split spoon sampler with percent recovered	P	Sampler pushed
▨	3.0" O.D. undisturbed sampler with percent recovered	●	% moisture content
☒	3.0" I.D. Universal sampler	*	Sample not recovered
☐	3.0" I.D. Ring sampler	▽	Water level fluctuation
G	Grab sample interval	▽	Static water level
L(C)	Laboratory/chemical analysis	▼ WD	Groundwater level at time of drilling
¡	Piezometer tip	▼ AB	Groundwater level after boring

AEE Project Number: 7-61M-09568-0

River Park Plaza
6530 SW Macadam
Portland, Oregon

AGRA EARTH & ENVIRONMENTAL, INC.

7477 S.W. Tech Center Drive
Portland, Oregon 97223-8025
Phone: (503) 639-3400 Fax: (503) 620-7892

Drilling Started: 9/29/97 Drilling Completed: 9/29/97 Logged By: HD a:\9568\9568B1.DRW

RC #001

PROJECT: River Park Plaza REVISION CONTROL NUMBER **PROJECT NUMBER:** 7-61M-09568-0

Boring Number: B-2

Boring Method: Hollow Stem Auger

Borehole Diameter: 8.0"

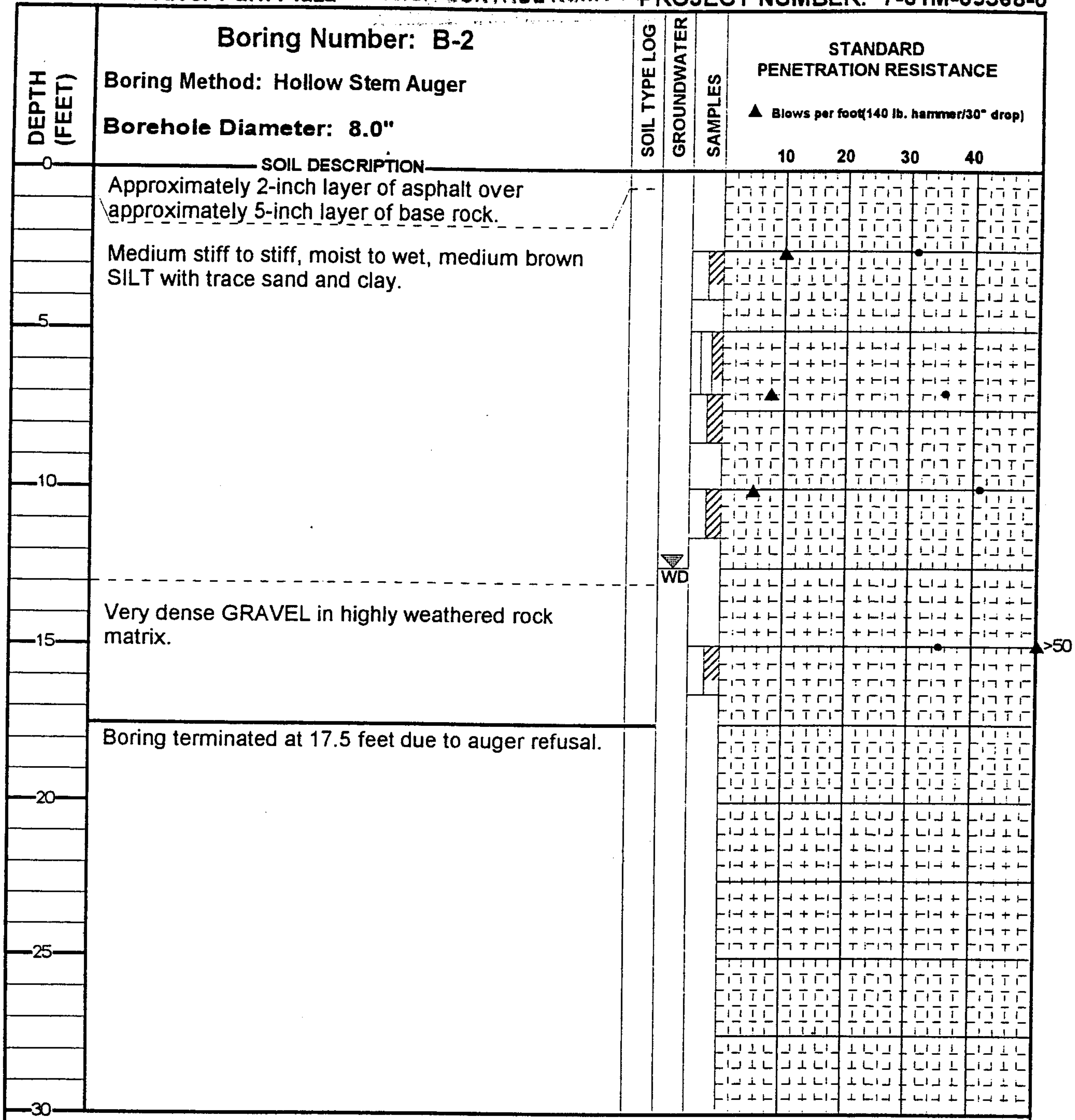
SOIL DESCRIPTION

Approximately 2-inch layer of asphalt over approximately 5-inch layer of base rock.

Medium stiff to stiff, moist to wet, medium brown SILT with trace sand and clay.

Very dense GRAVEL in highly weathered rock matrix.

Boring terminated at 17.5 feet due to auger refusal.



LEGEND

	2.0" O.D. split spoon sampler with percent recovered	P	Sampler pushed
	3.0" O.D. undisturbed sampler with percent recovered	●	% moisture content
☒	3.0" I.D. Universal sampler	*	Sample not recovered
▢	3.0" I.D. Ring sampler	▽	Water level fluctuation
G	Grab sample interval	▽	Static water level
L(C)	Laboratory/chemical analysis	▼ WD	Groundwater level at time of drilling
i	Piezometer tip	▼ AB	Groundwater level after boring

AEE Project Number: 7-61M-09568-0

River Park Plaza
6530 SW Macadam
Portland, Oregon

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Drilling Started: 9/29/97 Drilling Completed: 9/29/97 Logged By: HD a:\9568\9568B2.DRW

HSWCC

RC #001

REVISION CONTROL NUMBER

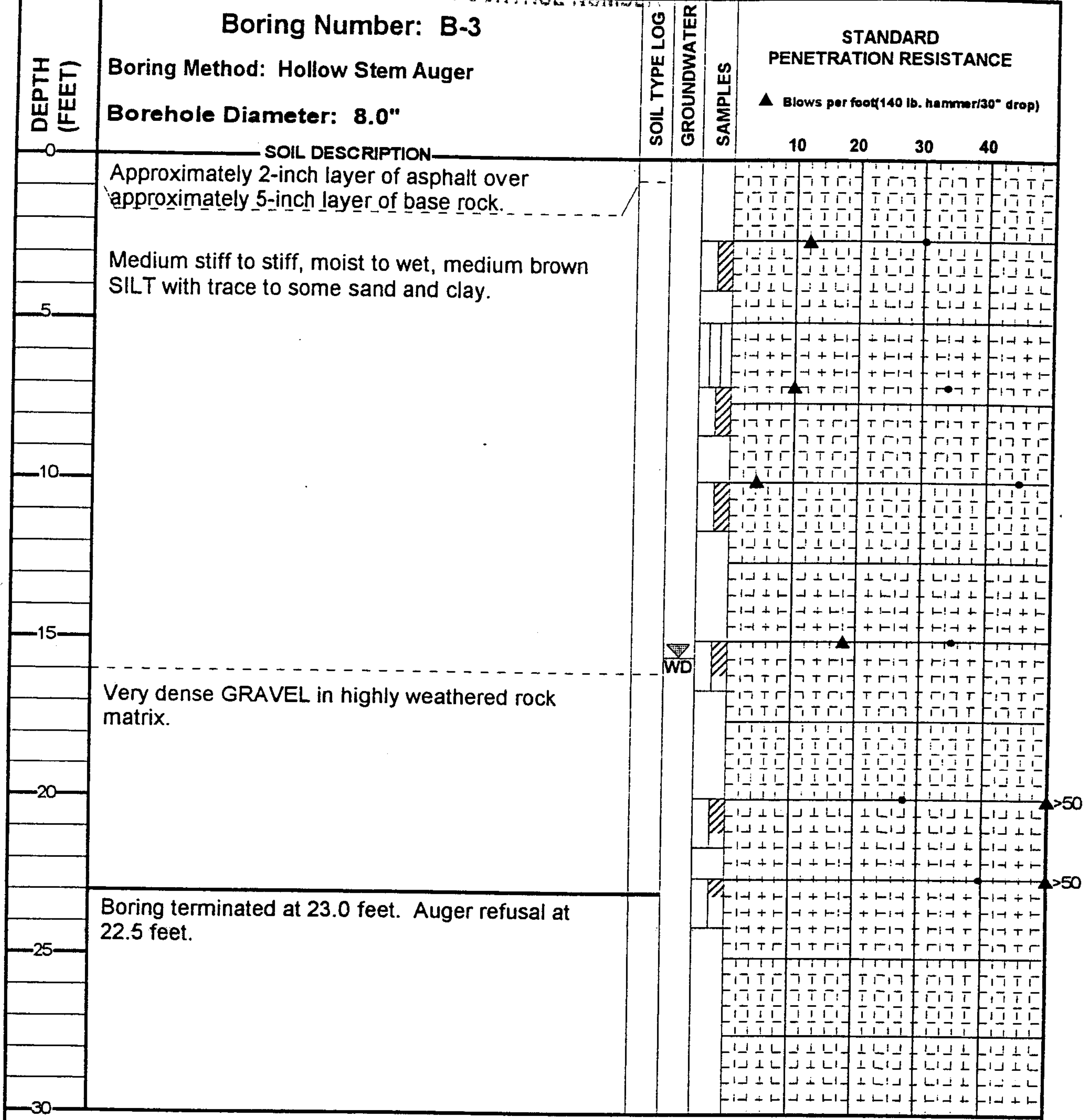
PROJECT: River Park Plaza **PROJECT NUMBER:** 7-61M-09568-0

Boring Number: B-3

Boring Method: Hollow Stem Auger

Borehole Diameter: 8.0"

SOIL DESCRIPTION



Approximately 2-inch layer of asphalt over approximately 5-inch layer of base rock.

Medium stiff to stiff, moist to wet, medium brown SILT with trace to some sand and clay.

Very dense GRAVEL in highly weathered rock matrix.

Boring terminated at 23.0 feet. Auger refusal at 22.5 feet.

LEGEND

▨	2.0" O.D. split spoon sampler with percent recovered	P	Sampler pushed
▨	3.0" O.D. undisturbed sampler with percent recovered	●	% moisture content
☒	3.0" I.D. Universal sampler	*	Sample not recovered
☐	3.0" I.D. Ring sampler	▽	Water level fluctuation
G	Grab sample interval	▽	Static water level
L(C)	Laboratory/chemical analysis	▽ WD	Groundwater level at time of drilling
i	Piezometer tip	▽ AB	Groundwater level after boring

AEE Project Number: 7-61M-09568-0

River Park Plaza
6530 SW Macadam
Portland, Oregon

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Drilling Started: 9/29/97 **Drilling Completed:** 9/29/97 **Logged By:** HD a:\9568\9568B3.DRW

RC#001

PROJECT: River Park Plaza REVISION CONTROL NUMBER **PROJECT NUMBER:** 7-61M-09568-0

Boring Number: B-4

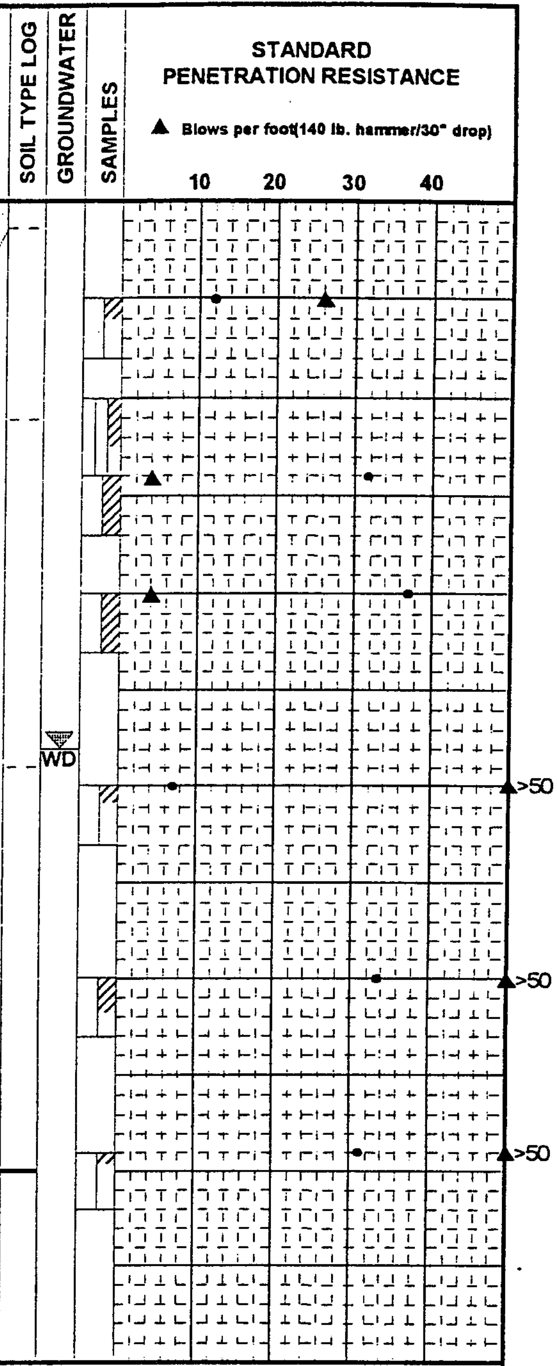
Boring Method: Hollow Stem Auger

Borehole Diameter: 8.0"

DEPTH (FEET)

SOIL DESCRIPTION

DEPTH (FEET)	SOIL DESCRIPTION
0	Approximately 2-inch layer of asphalt over approximately 5-inch layer of base rock.
	Medium stiff, moist, medium brown SILT with organics, gravel, and weathered rock (FILL).
5	Soft to medium stiff, moist to wet, medium brown with gray, clayey SILT with trace sand.
10	
15	Very dense GRAVEL in highly weathered rock matrix.
20	
25	Boring terminated at 25.0 feet. Auger refusal at 24.5 feet.
30	



LEGEND

	2.0" O.D. split spoon sampler with percent recovered	P	Sampler pushed
	3.0" O.D. undisturbed sampler with percent recovered	●	% moisture content
☒	3.0" I.D. Universal sampler	*	Sample not recovered
☐	3.0" I.D. Ring sampler		Water level fluctuation
G	Grab sample interval		Static water level
L(C)	Laboratory/chemical analysis	WD	Groundwater level at time of drilling
i	Piezometer tip	AB	Groundwater level after boring

AEE Project Number: 7-61M-09568-0

River Park Plaza
6530 SW Macadam
Portland, Oregon

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Phone: (503) 639-3400 Fax: (503) 620-7892

Drilling Started: 9/29/97 **Drilling Completed: 9/29/97** **Logged By: HD** a:\9568\9568B4.DRW

APPENDIX B

Laboratory Testing

HSWCC

RC #001

REVISION CONTROL NUMBER

AGRA Earth & Environmental
ENGINEERING GLOBAL SOLUTIONS

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in substantial accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for their moisture content, Atterberg Limits, grain size characteristics, compaction characteristics and CBR value. These results are presented in the following pages.

MOISTURE CONTENTS

The moisture contents of the soil samples were determined according to ASTM D 2216-90. The moisture contents of the soil samples are indicated on the boring logs.

ATTERBERG LIMITS

Laboratory testing was performed on two soil samples to determine their Atterberg Limits according to ASTM D 4318. The Atterberg Limits of the two samples are given below:

Sample No.	Depth (ft.)	Liquid Limit	Plastic Limit	Plasticity Index
B3-S1	2.5 to 4.0	45.7	24.7	21.0
B1-S3	7.5 to 9.0	41.0	16.7	24.3

GRAIN SIZE ANALYSIS

Grain size analysis were performed according to ASTM D 421 and D 422. The results of these tests are indicated below:

Sample No.	Depth (ft.)	% Sand	% Fines
B2-S1	**2.5 to 4.0**	**12.4**	**87.6**

COMPACTION CHARACTERISTICS

A compaction test was conducted on a selected soil sample in accordance with ASTM D 1557 to determine its maximum dry density and optimum moisture content. The maximum dry density of the soil sample tested was found to be 124.0 pcf at an optimum moisture content of 10.5 %.

CALIFORNIA BEARING RATIO

A California Bearing Ration (CBR) test was conducted on a selected soil sample in accordance with ASTM D 1883. The CBR value was found to be 4 for the selected soil sample compacted at 95 % of maximum dry density.