GRI Geotechnical & Environmental Consultants
9725 SW Beaverton Hillsdale Hwy, Ste 140
Beaverton, Oregon 97005-3364
PHONE 503/641/3478 FAX 503/644/8034

08-122284EXC 01

August 2, 2007

4666 GEOTECHNICAL RPT

6/24/08

Shriners Hospitals for Children
c/o SRG Partnership
621 SW Morrison Street, Suite 200
Portland, OR 97205

Attention: Tom Ochab

SUBJECT: Geotechnical Investigation
Addition to Shriners Hospital for Children
Portland, Oregon

At your request, GRI has conducted a geotechnical investigation for the proposed addition to Shriners Hospital for Children on Marquam Hill in Portland, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of the investigation was to evaluate subsurface conditions at the site and develop geotechnical recommendations for use in design and construction of the new structure. The investigation included a review of available geotechnical information for the site and vicinity, subsurface explorations, geophysical testing, laboratory testing, and engineering and seismic analyses. This report describes the work accomplished and provides our conclusions and recommendations for earthwork, temporary shoring, and design and construction of foundations for the project.

PROJECT DESCRIPTION

Based on our discussions with you and the structural engineer for this project, Catena Consulting Engineers (Catena), we understand the project will include partial renovation of the existing hospital and construction of three levels of offices and patient care areas that will span an existing parking structure. The location of the existing hospital and parking structure are shown on the Site Plan, Figure 2. We anticipate the new structure will be supported on drilled shafts or pin piles drilled into the underlying basalt. The maximum column load for the project will likely be on the order of 2,500 kips.

SITE DESCRIPTION

Geology

Our review of available geologic information for the site, our experience at the Oregon Health & Science University (OHSU) campus, and our review of materials and conditions disclosed by the subsurface explorations for this project indicate the site is mantled by a 20- to 30-ft-thick layer of brown silt, commonly referred to as Portland Hills Silt. The silt is mantled with relatively thin zones of silt fill in areas and is underlain by Columbia River Basalt across the site. The surface of the basalt is typically decomposed to the consistency of a hard, silty soil. The basalt tends to become less decomposed and harder with depth. The hard basalt is generally highly to moderately fractured and slightly to moderately weathered. The composition of the bedrock is characteristically somewhat variable, with the rock containing randomly occurring zones of highly fractured, more-weathered material.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were investigated on May 4, 2007, with one boring, designated B-1, and on June 29, 2007, with a second boring, designated B-2, and on April 19, 2007, with three seismic refraction lines. The borings were advanced to a depth of 53 and 62.5 ft, respectively, at the locations shown on Figure 2. A detailed discussion of the field exploration and laboratory testing programs conducted for this investigation is provided in Appendix A. GRI selected the locations for the borings and seismic lines and determined ground surface elevations based on the topographic information shown on Figure 2. Logs of borings B-1 and B-2 are provided on Figures 1A and 2A. The terms used to describe the soils and rock encountered in the borings are defined in Tables 1A and 2A. The locations and results of the seismic refraction testing are provided in Appendix B. The soil and groundwater conditions at the site are described below.

As a part of this study, we reviewed the August 1993 geotechnical report completed by Dames & Moore for the parking garage project entitled, "Geotechnical Investigation, Planned Parking Garage, Shriners Hospital for Crippled Children, Portland, Oregon." The original boring logs and a site plan showing the locations of the borings are provided in Appendix C. The Dames & Moore figures are provided for informational purposes only and are not intended for definition of soil layers.

The subsurface materials and conditions disclosed by the borings made for this investigation are generally similar to conditions encountered during previous investigations and excavations made for nearby and adjacent structures. However, it should be noted that significant variations in hard rock elevations were observed between the Dames & Moore boring B-1 and GRI boring B-1and Dames & Moore boring B-3 and GRI boring B-2. In general, the Dames & Moore borings appear to classify portions of the severely weathered basalt as gray basalt and provide a potentially non-conservative estimate for the top of "hard rock."

Soils

For the purpose of discussion, the materials disclosed by the borings have been grouped into the following categories, based on their physical characteristics and engineering properties.

1. **FILL**
2. **SILT**
3. **SILT (Residual Soil/Severely Weathered Basalt)**
4. **BASALT**

1. FILL. Boring B-2 encountered fill below a 1-ft-thick pavement section. The fill extends to a depth of 13 ft and consists of dark brown to gray silt. The relative consistency of the silt fill is medium stiff based on N-values of 5 and 6 blows/ft. The natural moisture content of the fill ranges from about 28 to 33%. The fill contained zones with some organic material, and a 1-ft-thick layer of wood debris was encountered at a depth of 8.5 ft.

2. SILT. Silt was encountered below a 1-ft-thick pavement section in boring B-1 and below the fill at a depth of 13 ft in boring B-2. The silt extends to a depth of 7 ft in boring B-1 and 29.5 ft in boring B-2 and is typically brown and contains a trace to some fine-grained sand and a trace of clay.. The relative consistency of the silt is medium stiff to stiff based on N-values of 6 to 13 blows/ft. The natural moisture content of the silt varies from about 28 to 36%.

3. SILT (Residual Soil/Severely Weathered Basalt. The silt is underlain by a zone of residual soil/severely weathered (decomposed) basalt. The severely weathered rock consists of fragments of highly weathered, friable basalt in a matrix of clay, silt, and sand, and has the consistency of stiff to hard soil. The material is typically gray with black, yellow, rust, and brown mottling. Original rock features, such as vesicles and fractures, are preserved as secondary mineral deposits. The severely weathered zone was encountered between a depth of 7 and 22.5 ft in boring B-1 and 29.5 and 41 ft in boring B-2. The decomposed basalt tends to become harder and less weathered with increasing depth. As the effects of weathering decrease with depth, the proportion of silt and clay decreases, and the basalt fragments become larger and less weathered. N-values in the severely weathered basalt ranged from 13 blows/ft to over 50 blows for 2 in. of sampler penetration. The natural moisture content of the residual soil/severely weathered basalt ranges from about 25 to 52%.

4. BASALT. Basalt was encountered beneath the severely weathered basalt and became sufficiently hard to permit coring at depths of 25 and 43 ft below the ground surface in borings B-1 and B-2, respectively. The quality of the basalt, as measured by hardness and weathering, was highly variable, with the majority of the cored basalt ranging from medium hard to very hard (RH-2 to RH-4). Core recovery ranged from 95 to 100%. The basalt typically contains close to very close joints and fractures, resulting in typical rock quality designations (RQD) of 0 to 55%. The joints and fractures were generally very close to close. Staining and occasional clay and other secondary mineral deposits were observed on some fracture surfaces. Unconfined compressive strengths varied widely from 390 to 7,410 psi.

Groundwater

We anticipate the static groundwater level at this site occurs at depth in the highly fractured, hard basalt; however, information developed during geotechnical investigations for other structures on the campus indicate perched groundwater can occur in the silt soils that mantle the site, particularly following intense and/or sustained precipitation. Deep excavations made for construction projects on the campus have encountered somewhat randomly occurring, localized zones of seepage. The recently completed Kohler Pavilion across the street from Shriners Hospital encountered large quantities of water in the fractured, hard basalt.

CONCLUSIONS AND RECOMMENDATIONS

General

The subsurface conditions disclosed by the borings and geophysical investigation for this study are similar to the conditions encountered by GRI and others during investigations and excavations for the adjacent and nearby structures. The proposed hospital expansion site is mantled by fill material and silt soils that are underlain by basalt rock. The surface of the basalt is severely weathered, or decomposed, and has a soil-like consistency. The borings and geophysical testing indicate the top of hard rock is about 20 to 30 ft below the existing site grades on the downhill side of the structure and about 40 ft below the ground surface near the southwest corner of the proposed structure. However, based on our experience at the OHSU campus, the depth to hard rock can vary significantly over short distances due primarily to non-uniform weathering of the basalt and various basalt flows. As noted previously, the Dames & Moore borings performed for the existing structure likely provide a non-conservative elevation of the top of hard rock relative to the definitions provided in this report.

Although detailed as-built information is not available, we understand the existing parking garage is supported on spread footings. Review of available subsurface information indicates the spread footings for at least the downhill portion of the parking garage are founded on residual soil or severely weathered basalt. Based on the estimated magnitude of the proposed foundation loads and proximity to the existing footings, it is our opinion it will be necessary to transfer structural loads to the underlying medium hard to very hard (RH-2 to RH-4) basalt to limit total and differential settlement to allowable values. Feasible foundation types include drilled piers or shafts and pin piles. Both drilled shafts and pin piles were successfully used to support the recently completed Kohler Pavilion and the Biomedical Research Building on the OHSU campus.

Site Preparation and Grading

Preparation of the site for construction will include removing existing trees and surficial organic matter within the project limits. All excavations required to remove existing root clumps should be shaped with 1H:1V side slopes and backfilled with compacted structural fill as recommended below.

The existing ground surface within the planned building area generally slopes from about 2H:1V to as steep as 1H:1V. Placing significant quantities of structural fill on the sloping site could adversely affect the stability of the slopes. For this reason, we anticipate filling will be limited to the backfilling of excavations made to remove existing features and utility trench backfill. Other potential areas of significant new fills should be reviewed by GRI on a case-by-case basis as the project design is developed.

In our opinion, imported granular material, such as sand, sandy gravel, or fragmental rock, should be used to construct structural fills for the project. The fill material should have a maximum size of about 4 in. and not more than about 5% passing the No. 200 sieve (washed analysis). Lifts should be placed 12 in. thick (loose) and compacted with a medium-weight (48-in.-diameter drum), smooth, steel-wheeled, vibratory roller until well-keyed and to not less than 95% of the maximum dry density as determined by ASTM D 698. A minimum of four passes with the roller is generally required to achieve compaction. Hand-operated compaction equipment should be used within 5 ft of any building walls or retaining walls.

In our opinion, permanent cut and fill slopes should be made no steeper than 2H:1V. Structural fill constructed on slopes steeper than 5H:1V should be benched into the existing grade. Since it is difficult to compact the surface of fill slopes, we recommend the slopes be overbuilt by 2 ft and trimmed back after construction to provide a surface that is more resistant to localized sloughing.

All backfill placed in utility trench excavations within the limits of the project should consist of sand, sand and gravel, or crushed rock with a maximum size of 1 1/2 in., with not more than about 5% passing the No. 200 sieve (washed analysis). The granular backfill should be placed in lifts and compacted using vibratory plate compactors or tamping units to at least 95% of the maximum dry density as determined by ASTM D 698. The thickness of the lifts will depend on the type of backfill material and the size and type of compaction equipment. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

To reduce surface flow from entering the utility trenches, we recommend the upper 1 to 2 ft of backfill in all utility trenches in landscape areas and on the steep slopes adjacent to the structure consist of the on-site, fine-grained, relatively impermeable material compacted to about 90% of the maximum dry density as determined by ASTM D 698. To reduce potential for hydrostatic pressure in the trenches, which would increase the risk of instability on the slopes, all utility trenches crossing the slopes should be drained with a 4-in.-diameter, geotextile-wrapped, perforated pipe placed at the bottom of the trench. The outlet of the pipes should deposit the accumulated water to a suitable storm sewer or drainage area.

Seismic Considerations

General. We understand the project will be designed using the 2006 International Building Code (IBC) with 2007 Oregon Structural Specialty Code (OSSC) modifications. Based on the subsurface conditions disclosed by the recent borings, and the proposed foundation elevations, the site is classified as IBC Site Class C. The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 seconds, to develop the design earthquake spectrum. The S_S and S_1 coefficients identified for the site are 0.99 and 0.35 g, respectively.

GRI completed a site-specific seismic hazard study for this project, which is discussed in Appendix D. Results of our seismic study indicate the response spectrum developed using the 2002 U.S. Geological Survey (USGS) probabilistic seismic hazard study is most appropriate for the conditions at this site. Based on review of the USGS study and comparison with the 2006 IBC normalized spectrum for Site Class C, we recommend using the IBC design spectrum for seismic design.

Based on the location of the site, grain size and stiffness of the soil beneath the site, and depth to groundwater, it is our opinion the risk for liquefaction and liquefaction-induced lateral spreading, settlement, and subsidence is low. Based on the depth to bedrock and groundwater at the site, it is our opinion the risk for earthquake-induced slope instability is low. Based on the elevation and location of the site, the risk of damage by tsunamis and/or seiches at the site is absent.

Based on our review of geologic maps and available subsurface information, several faults are located within about 5 km of the site. However, the USGS does not consider the closest mapped fault to the site, the Portland Hills Fault, active in their probabilistic seismic hazard analysis. Based on the location of the site and the published activity rates for the Portland Hills Fault, it is our opinion the potential for fault rupture at the site is low.

Foundations

Based on our review of the subsurface information, proximity of the existing parking garage spread footings, and discussions with Catena , our studies considered the use of drilled cast-in-place piers and pin piles for support of the structure. We anticipate site access will be a significant consideration in selection of a deep foundation option. The drill rigs necessary to install the drilled cast-in-place piers generally create a larger shaft and are installed by a much larger machine.

Drilled Cast-in-Place Piers. Drilled cast-in-place concrete piers can be constructed by drilling a shaft or socket into the underlying medium hard to very hard (RH-2 to RH-4) basalt, placing the steel reinforcement, and filling the shaft with concrete. Based on our experience on the OHSU campus and discussions with local contractors, we know equipment is available that is capable of drilling a 24-in.-

diameter rock socket into hard basalt. We recommended an ultimate compressive capacity of 120 kips/ft for a 24-in.-diameter rock socket embedded into the medium hard to very hard (RH-2 to RH-4) rock with a minimum and maximum embedment into the rock of 10 and 25 ft, respectively. Ultimate uplift resistances can be assumed to be two-thirds of the ultimate compressive capacity.

We recommend applying a factor of safety of 3 for dead load and permanently applied live loads. For transient loading conditions such as seismic conditions, we recommend utilizing a factor of safety of 1.5 and 2 for compressive and uplift resistances, respectively. The concrete and steel design may control the allowable capacities for the cast-in-place piers. The ultimate capacities assume the piers will be installed with a center-to-center spacing of at least three pier diameters, 3D, where D is the pier diameter.

The silt soils overlying the severely weathered and hard basalt are fine-grained and somewhat cohesive, and we do not anticipate the shaft walls will cave significantly in these materials during drilling. It should be anticipated the drilled pile shafts may encounter perched water seeping through the more permeable zones within the overburden soils and severely weathered basalt. As a result, it would be prudent for the piling contractor to assume that casing may be necessary to retain the overburden soils in at least some of the drilled shafts. For other projects on the surrounding OHSU campus, foundation contractors have used down-the-hole hammers and core barrels to drill the rock sockets for drilled piers.

All water and loose material should be removed from the bottom of each drilled shaft before placing the concrete. If the inflow of water is sufficiently high to prevent removal of the water, the concrete should be placed using tremie methods. The bottom of the tremie pipe should be maintained at least 4 ft below the surface of the concrete.

It is important to note the pier design criteria discussed in this section are based on the assumption that the high-capacity piers will be installed in good-quality, medium hard to very hard basalt. The drilling and installation of the concrete cast-in-place piers should be observed on a full-time basis by the geotechnical engineer to evaluate the rock quality and the contractor's procedures. In this regard, the recommended depths of penetration into hard rock should be considered a minimum depth. Based on our experience, the quality of the basalt can vary significantly over relatively short distances, and it should be anticipated it will be necessary to deepen some of the piles to accommodate zones of poorer-quality rock.

To reduce the risk of loss of support for the existing spread footings, we recommend new cast-in-place piers be offset a minimum of 5 ft from the edge of the adjacent footings.

Pin Piles. Due to the limited working space along the west side of the structure, we anticipate support for the west side of the addition will consist of pin piles drilled into the underlying medium hard to very hard rock located approximately 30 to 40 ft below the existing ground surface elevation.

Based on our experience, we anticipate pin piles with an allowable compressive and tensile capacities on the order of about 100 to 150 tons can be achieved with a minimum of 15 and 25 ft of embedment, respectively, in the underlying medium hard to very hard basalt. Pin piles consist of high-capacity, small-diameter (typically 5- to 12-in.), drilled and grouted, steel-cased piles. A pin pile is typically constructed by drilling a cased hole to the desired depth into the bearing layer, placing a reinforcing bar to the bottom of the hole, and pumping grout under pressure to form a bond zone as the casing is withdrawn. The steel casing typically extends from the pile cap connection to slightly below the top of the bond zone to provide load transfer into the bond zone and structural rigidity within the upper portion of the pile. The structural engineer should determine the amount of allowable deflection in the pin piles at the anticipated working load, which typically determines the required thickness of the pin pile casing.

The allowable capacities assume the pin piles will be installed with a center-to-center spacing of at least three pile diameters, 3D, where D is the pier or pile diameter. We recommend verification testing at least one pin pile to 200% of the design load. Proof testing to 150% of the design load should be performed on at least 10% of the production anchors. The proof and verification testing is commonly performed in tension.

Permanent Rock Anchors. Based on discussions with Catena, permanent rock anchors may be needed to resist lateral and/or uplift loads. In our opinion, individual small-diameter anchors with a minimum grouted length of 12 ft into medium soft to very hard basalt and a minimum overall length of 20 ft can be designed to develop a maximum capacity of 60 tons. Anchors with a minimum diameter of 6 in., a minimum grouted length of 15 ft into medium soft to very hard basalt, and a minimum length of 25 ft could develop a capacity of at least 100 tons. All permanent rock anchors should be protected against corrosion for their anticipated design life. The suitability of the installation of each anchor should be verified by proof-loading to 150% of the design load. The allowable capacities assume the anchors will be installed with a minimum center-to-center spacing of 4 ft.

Lateral Forces and Resistance. Cast-in-place concrete piers will resist lateral loads by the structural strength of the pile in bending. We estimate the depth of fixity of a 24-in.-diameter pier will be about 3 ft below the surface of the medium hard to very hard basalt. Passive resistance from the silt soils and severely weathered basalt can be evaluated using an equivalent fluid pressure of 300 pcf applied over the width of the pier or pile. Passive resistance from the upper 3 ft of rock can be evaluated using an equivalent fluid pressure of 1,000 pcf. If necessary, additional lateral shaft resistances for varying soil conditions can be developed using L-Pile 5.0 when more information is available.

Passive soil resistance against pile caps and grade beams established in the silt soils or severely weathered basalt can also be used to resist lateral loads. For design purposes, passive resistance can be evaluated on the basis of an equivalent fluid having a unit weight of 300 pcf for pile caps poured neat against undisturbed soil, or for pile cap excavations backfilled with well-compacted granular fill. Lateral resistance due to passive earth pressures against the embedded footings in medium soft to very hard basalt can be computed on the basis of an equivalent fluid having a unit weight of 500 and 1,000 pcf for lateral movements of 1/4 and 1/2 in., respectively. In areas where drilled piers are located on slopes of 2H:1V or steeper, the passive resistance from the upper 10 ft of near-surface soils should be neglected due to the marginal stability and downward creep characteristics of the upper silt.

We do not recommend using frictional forces on the base of pile-supported grade beams or pile caps.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to aid in design and construction of the proposed project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to design and construction of earthwork and foundations. In the event that any changes in the design and location of the project elements as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm our conclusions and recommendations in writing.

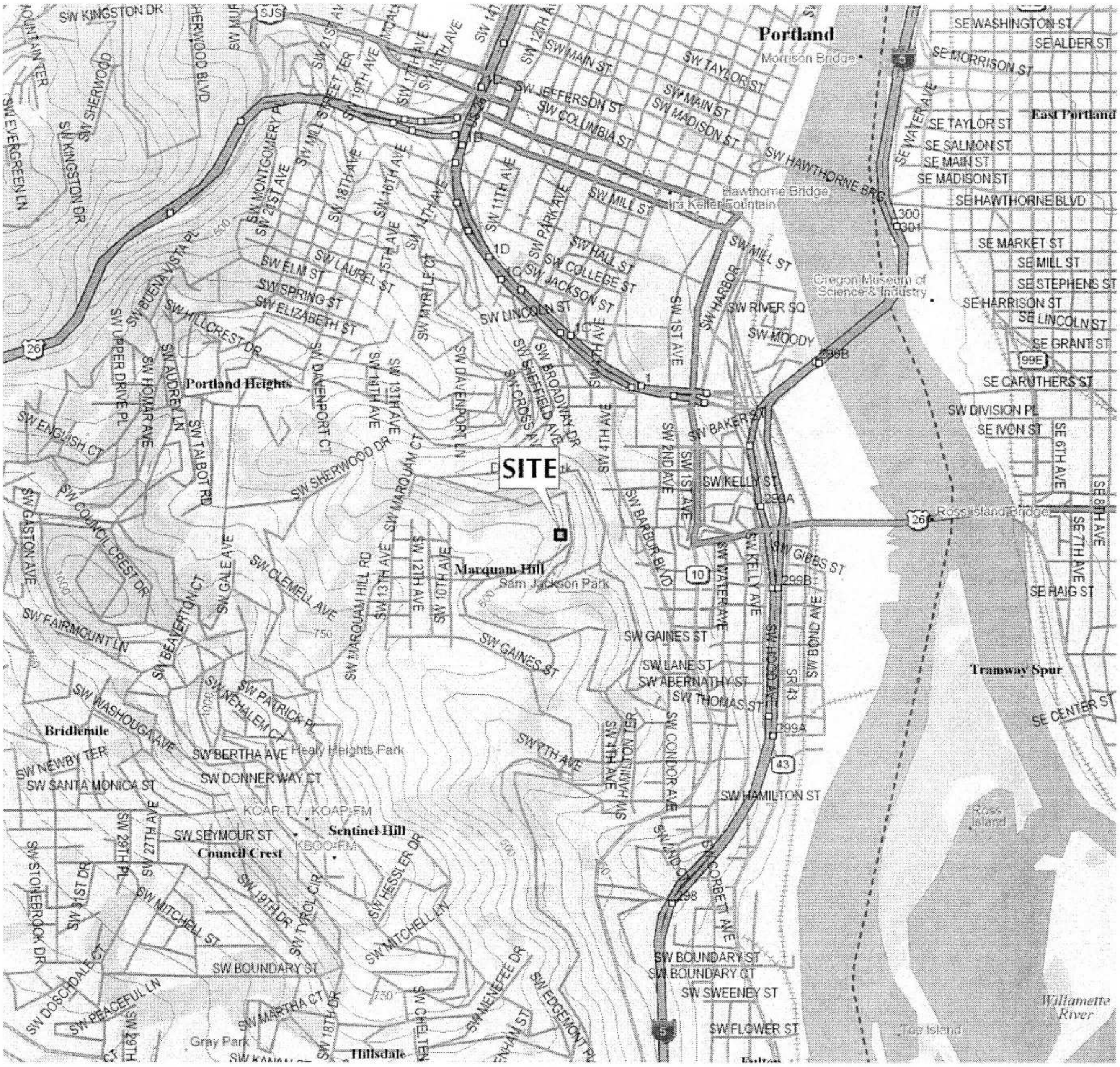
The conclusions and recommendations submitted in this report are based on the data obtained from the borings and geophysical testing at the locations indicated on Figure 2 and Appendix B, respectively, and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil and groundwater conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
13,930
OREGON
MARCH 16, 1988
MICHAEL W. REED
Exp. 12/08

Michael W. Reed, PE
Principal

Scott M. Schlechter, PE
Project Engineer



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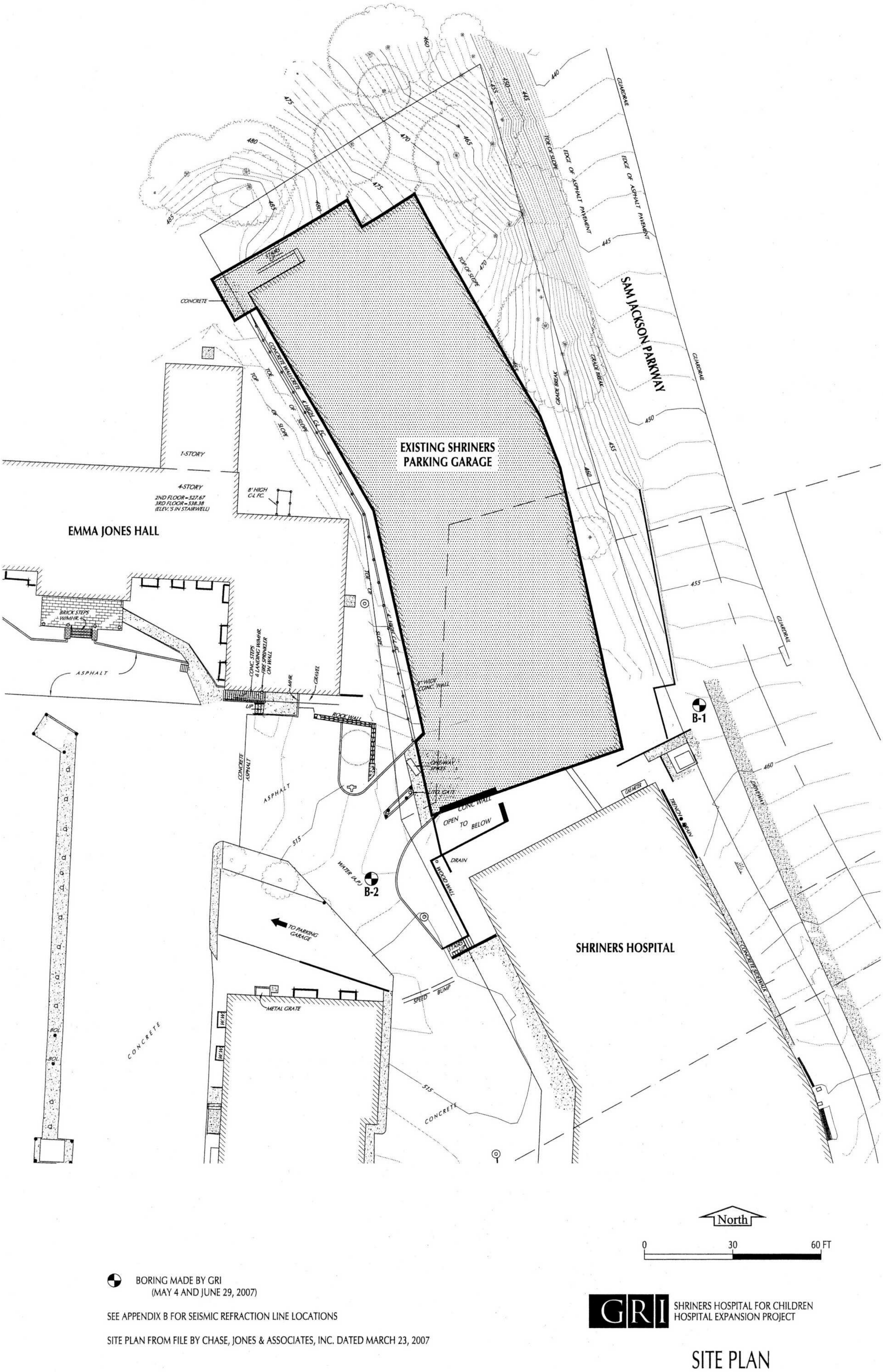
GRI

SHRINERS HOSPITAL FOR CHILDREN
HOSPITAL EXPANSION PROJECT

VICINITY MAP

AUG. 2007 JOB NO. 4666 FIG. 1



BORING MADE BY GRI
(MAY 4 AND JUNE 29, 2007)

SEE APPENDIX B FOR SEISMIC REFRACTION LINE LOCATIONS

SITE PLAN FROM FILE BY CHASE, JONES & ASSOCIATES, INC. DATED MARCH 23, 2007

GRI SHRINERS HOSPITAL FOR CHILDREN
HOSPITAL EXPANSION PROJECT

SITE PLAN

AUG. 2007 JOB NO. 4666 FIG. 2

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions at the site were investigated on May 4, 2007, with one boring, designated B-1, on June 28, 2007, with a second boring, designated B-2, and on April 19, 2007, with three seismic refraction lines completed by Earth Dynamics. The locations of the borings and seismic refraction lines are shown on the Site Plan, Figure 2, and Appendix B, respectively. The ground surface elevation at each location was obtained from the topographic information provided on Figure 2. An engineer provided from GRI observed the drilling, maintained a detailed log of the materials and conditions encountered in the borings, and collected representative soil samples. The terms used to describe the soil and rock encountered in the borings are defined in Tables 1A and 2A. A detailed description of the field explorations completed for this project is provided below.

Borings

Borings B-1 and B-2 were advanced to depths of 53 and 62.5 ft, respectively, with mud-rotary and coring methods using a CME 75 drill rig provided and operated by Western States Soil Conservation of Aurora, Oregon. Disturbed and undisturbed samples were obtained from the borings at about 5-ft intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the Standard Penetration Resistance, or N-value. The N-values provide a measure of the relative density of granular soils and the relative consistency of cohesive soils. The soil samples obtained in the split-spoon sampler were carefully examined in the field, and representative portions were saved in airtight jars for further examination and physical testing in our laboratory. In addition, relatively undisturbed Shelby tube samples were collected and returned to our laboratory for further testing.

Wireline drilling techniques were used to obtain core samples of the bedrock from borings B-1 and B-2 below depths of 25 and 42.5 ft, respectively. All core samples were placed in core boxes and returned to our laboratory for further examination and testing.

Logs of the borings are provided on Figures 1A and 2A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents. Also shown on the log are the core run locations, lengths, percent core recovery, and rock quality designations (RQD). The RQD is an index for determining the relative number of fractures and the amount of softening or alteration or the rock mass.

Geophysical Testing

A geophysical investigation of the site was completed by Earth Dynamics as a subconsultant to GRI. This investigation included three seismic refraction lines to estimate the depth to the top of rock in areas inaccessible by the truck-mounted drill rig. The Earth Dynamics report, which describes the methods used to conduct the seismic refraction surveys and summarizes the survey results, is provided in Appendix B.

LABORATORY TESTING

The samples obtained from the borings were examined in our laboratory, where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was determined in conformance with ASTM D 2216. The results are summarized on Figure 1A.

Three unconfined compressive strength tests were conducted in accordance with ASTM D 2938 to obtain data on the strength characteristics of representative intact rock core samples. ACS Testing Inc. of Tigard, Oregon, performed the tests as a subcontractor to GRI. The results of the compressive strength tests are tabulated below.

SUMMARY OF UNCONFINED COMPRESSIVE ROCK STRENGTH

Boring	Sample	Depth, ft	Compressive Strength, psi	Rock Description
B-1	Run 1	27	4,960	BASALT, gray, moderately to highly weathered, hard (RH-3)
	Run 3	34	390*	BASALT, gray, highly weathered, highly vesicular, medium hard (RH-2) to hard (RH-3)
B-2	Run-6	61	7,410	BASALT, gray, slightly to moderately weathered, hard (RH-3) to very hard (RH-4)

* Numerous fractures prior to testing and sample height-to-width ratio less than 2.0.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 - 2
trace	2 - 10
some	10 - 30
sandy, silty, clayey, etc.	30 - 50

Table 2A

GUIDELINES FOR CLASSIFICATION OF ROCK

Relation of RQD and Rock Quality

RQD (Rock Quality Designation), %	(Description of Rock Quality)
0-25	Very Poor
25-50	Poor
50-75	Fair
75-90	Good
90-100	Excellent

Descriptive Terminology for Joint Spacing

Spacing of Joints	Descriptive Term
< 2 in.	Very Close
2 in. - 1 ft	Close
1 ft - 3 ft	Moderately Close
3 ft - 10 ft	Wide
> 10 ft	Very Wide

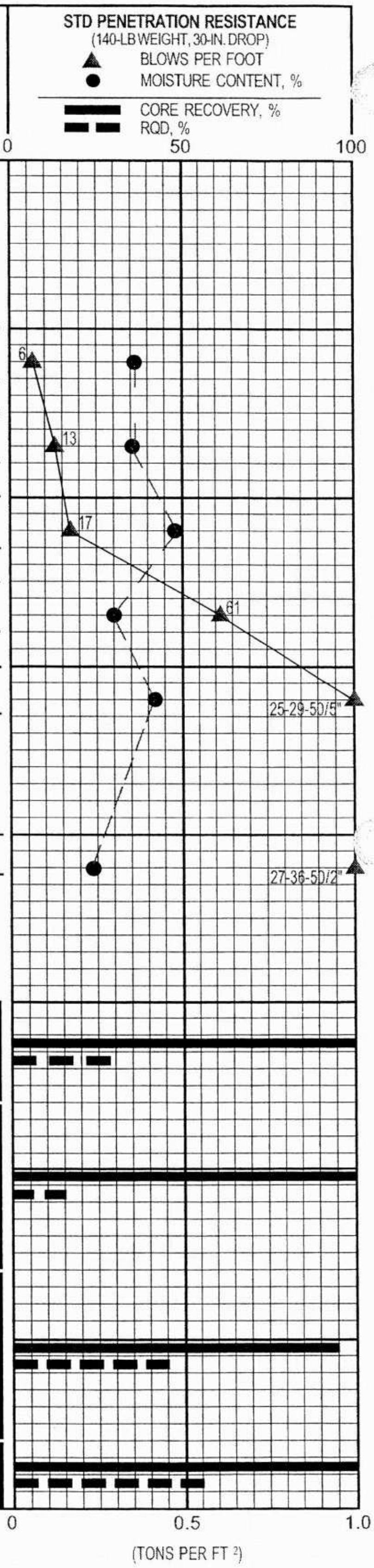
Scale of Rock Hardness (After Panama Canal Company, 1959)

RH-1	Soft	Slightly harder than hard overburden soil, rock-like structure, but crumbles or breaks easily by hand
RH-1	Medium Soft	Cannot be crumbled between fingers, but can be easily picked with light blows of the geology hammer.
RH-2	Medium Hard	Can be picked with moderate blows of geology hammer. Can be cut with knife.
RH-3	Hard	Cannot be picked with geology hammer, but can be chipped with moderate blows of the hammer.
RH-4	Very Hard	Chips can be broken off only with heavy blows of the geology hammer.

Terms Used to Describe the Degree of Weathering

Descriptive Term	Defining Characteristics
Fresh	Rock is unstained. May be fractured, but discontinuities are not stained.
Slight	Rock is unstained. Discontinuities show some staining on their surfaces, but discoloration does not penetrate rock mass.
Moderate	Discontinuity surfaces are stained. Discoloration may extend into rock along discontinuity surfaces.
High	Individual rock fragments are thoroughly stained and can be crushed with pressure hammer. Discontinuity surfaces are thoroughly stained and may be crumbly.
Severe	Rock appears to consist of gravel-sized fragments in a "soil" matrix. Individual fragments are thoroughly discolored and can be broken with fingers.

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES
		SURFACE ELEVATION 458 ft (±)			
		Asphaltic-concrete PAVEMENT (6 in.) over crushed rock BASE COURSE (6 in.)	1.0		
		Medium stiff, brown SILT; trace clay and fine-grained sand			
5					S-1
		Stiff, brown mottled red and yellow SILT; some clay (Residual Soil / Severely Weathered Basalt)	7.0		S-2
10		---------very stiff, contains scattered fragments of soft to medium soft (RH-1) basalt below 10 ft			S-3
		---------hard below 12.5 ft			S-4
15		---------contains scattered medium hard (RH-2) fragments of basalt below 15 ft			S-5
		---------interbedded medium hard (RH-2) zones below 17.5 ft			
20		---------contains scattered fragments of hard (RH-3) basalt below 20 ft			S-6
		Hard (RH-3), gray BASALT; moderately to highly weathered, very close to close joints	22.5		
25					RUN 1
30					RUN 2
		---------medium hard to hard (RH-2 to RH-3), some vesicles to highly vesicular below 32.5 ft			
35					RUN 3
40					RUN 4



2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
NX CORE RUN
SLOTTED PVC PIPE
Water Level (date)

TORVANE SHEAR STRENGTH, TSF
UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1

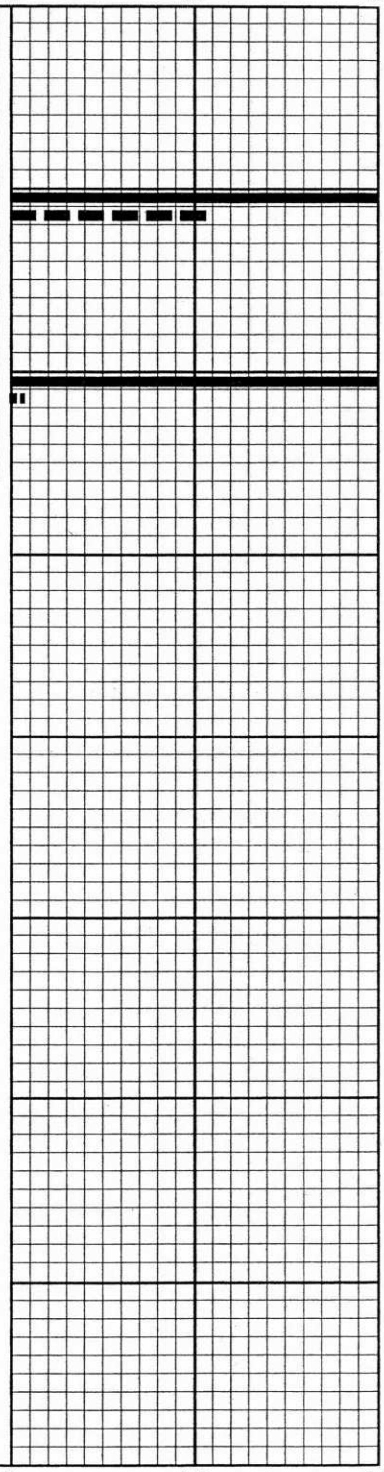
AUG. 2007 JOB. NO. 4666 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 458 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
40	+ + + +	Hard to very hard (RH-3 to RH-4), gray BASALT; moderately to highly weathered, very close to moderately close joints, some vesicles			
45	+ + + +				RUN 5
50	+ + + +				RUN 6
		(5/4/2007)	53.0		
55					
60					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
CORE RECOVERY, %
RQD, %

0 50 100



0 0.5 1.0
(TONS PER FT ^{2})

I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
|— SLOTTED PVC PIPE
▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

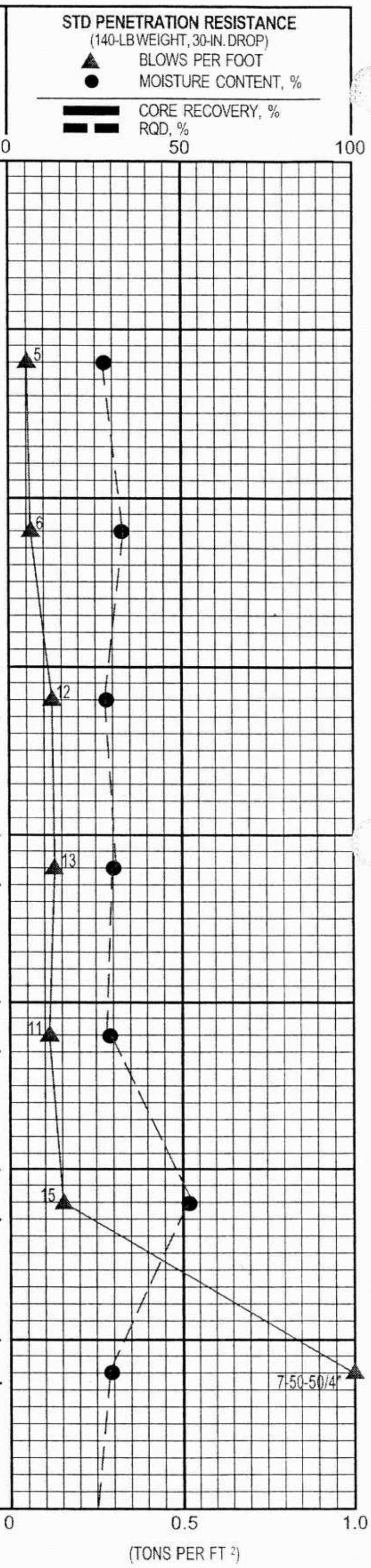
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

AUG. 2007 JOB. NO. 4666 FIG. 1A

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLES
	SURFACE ELEVATION 513 ft (±)		
	Asphaltic-concrete PAVEMENT (3 in.) over crushed rock BASE COURSE (9 in.)	1.0	
	FILL: Medium stiff, brown SILT; trace clay		
5	---------dark brown, significant organics below 4.5 ft		S-1
	---------gray, scattered organics below 8 ft		
10	-------1-ft-thick layer of wood debris at 8.5 ft		S-2
15	Stiff, brown SILT; trace clay	13.0	S-3
20			S-4
25			S-5
30	Very stiff, brown SILT; some clay to clayey, trace coarse sand-size particles (Residual Soil/Severely Weathered Basalt)	29.5	S-6
35			S-7
40	---------interbedded zones of medium soft to medium hard (RH-1 to RH-2) basalt below 38 ft		



GROUNDWATER

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

NX CORE RUN

SLOTTED PVC PIPE

Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

BORING B-2

AUG. 2007 JOB. NO. 4666 FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 513 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP)
40		Very stiff, brown SILT; some clay to clayey, interbedded zones of medium hard (RH-2) basalt (Residual Soil/Severely Weathered Basalt)	41.0		S-8	50/4"
					S-9 *	50/1"
		Hard (RH-3), gray BASALT, close to very close joints			RUN 1	
45					RUN 2	
		---------vesicular, secondary mineralization in vesicles and on fracture surfaces, clay infilling below 47 ft				
50					RUN 3	RQD 0%
		---------hard to very hard (RH-3 to RH-4) some vesicles, some secondary mineralization, clay infilling below 53.5 ft				
55					RUN 4	
					RUN 5	
60		---------non-vesicular below 60.5 ft			RUN 6	
			62.5			
		(6/28/2007)				
65						

STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
CORE RECOVERY, %
RQD, %

0 50 100

0 0.5 1.0
(TONS PER FT ^{2})

I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▌ NX CORE RUN
SLOTTED PVC PIPE
▼ Water Level (date)
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY
Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-2 (cont.)

AUG. 2007 JOB. NO. 4666 FIG. 2A

Report on Geophysical Exploration
Shriner's Hospital Parking Garage
Portland, Oregon

Prepared for:
Geotechnical Resources, Inc.
9725 SW Beaverton-Hillsdale Hwy, Suite 140
Portland, OR 97005-3364

Prepared by:
EARTH DYNAMICS
2284 NW Thurman Street
Portland, OR 97210
(503) 227-7659

May 10, 2007

1.0 INTRODUCTION

This report presents the results of geophysical explorations near the Shriner's Hospital parking garage in Portland, Oregon. The work was requested and authorized by Mr. Mike Reed of GRI. Seismic refraction data were acquired on April 19, 2007 by Dr. Michael Feves and Mr. Daniel Lauer. This report describes the methodology and results of the investigation.

2.0 SCOPE OF WORK

The primary purpose of this study was to help determine the subsurface geology in the vicinity of the parking garage for Shriner's Hospital. These data are needed for geotechnical design of a proposed expansion of the existing garage. Three, 138 foot long seismic refraction profiles were completed at the site. Each seismic line consisted of a 24 channel array with a geophone spacing of 6 feet. Seismic data were collected from five shot points on each line.

3.0 METHOD

3.1 Seismic Refraction

A seismic refraction exploration consists of measuring the time required for a seismic wave to travel from a seismic source to a receiving transducer. A sledgehammer, large weight dropped, or explosive device is typically used for the seismic source and vertical geophones are used as receiving transducers. A seismograph records signals from the geophones. By analyzing the arrival time of the seismic wave as a function of distance from the seismic source, the seismic velocities of the underlying soil/rock units and the depth to geologic contacts can be determined. The seismic refraction method requires that seismic sources be placed at each end of the geophone array. Intermediate and off end sources are also often used to increase resolution and penetration. Application of the method is limited to areas where seismic velocity increases or is constant with depth. The depth of penetration is typically one-quarter to one-third of the geophone array length, and lateral resolution is typically one-half of the geophone spacing.

The seismic refraction survey for this study was conducted using a Seismic Source 24-channel DAQ Link II seismograph. A sledgehammer was used as a seismic source. Several impacts were stacked to enhance the first arrivals. Additionally, all shots were timed to minimize noise from traffic on nearby roadways.

The seismic data were analyzed using SeisOpt@2D Ver. 5.0 by Optim Software. SeisOpt@2D uses a forward modeling global optimization technique. The technique consists of creating a velocity model through which travel times are computed. The computed times are compared with the observed data. Thousands of iterations are completed to find the velocity model with the minimum travel time error. Comparison of the computed travel times to the measured values provides an indication of the validity of the model. Several velocity models are run using different grid resolution and depth values to obtain the best result for each data set. The SeisOpt modeling technique is superior to discrete layer modeling because lateral, as well as vertical variations can be resolved. SeisOpt generates xyz data files that are input to Surfer® 8 for contouring, scaling, and data presentation.

3.2 Location and Elevation Survey

Horizontal locations of the seismic lines were surveyed from the existing garage using fiberglass tape measures. Wooden stakes were installed at the beginning and end of each seismic line. Elevation data were acquired along each line using a level and stadia-rod. The elevations were calculated using the elevation of 471.4 feet at "CJZ Control #102". The elevation for this control point was provided by GRI. Geophone and shot point positions were determined with a fiberglass tape laid out on the ground surface. The modeling software corrects for ground slope to provide actual horizontal position of the geophones.

4.0 RESULTS

The locations of the geophysical exploration profiles are shown in Figure 1. The seismic velocity profile models generated by SeisOpt2D for Seismic Line 1, 2 and 3 are contained in Figures 2, 3, and 4 respectively. The interpreted geologic interfaces are shown in Figures 2, 3, and 4.

5.0 DISCUSSION

The seismic velocity of soil and rock is a function of the density and elastic properties of the material. Therefore, variations in subsurface materials can be determined from analysis of the seismic velocity. Typical seismic velocities for various soil and rock types are listed in Table 1.

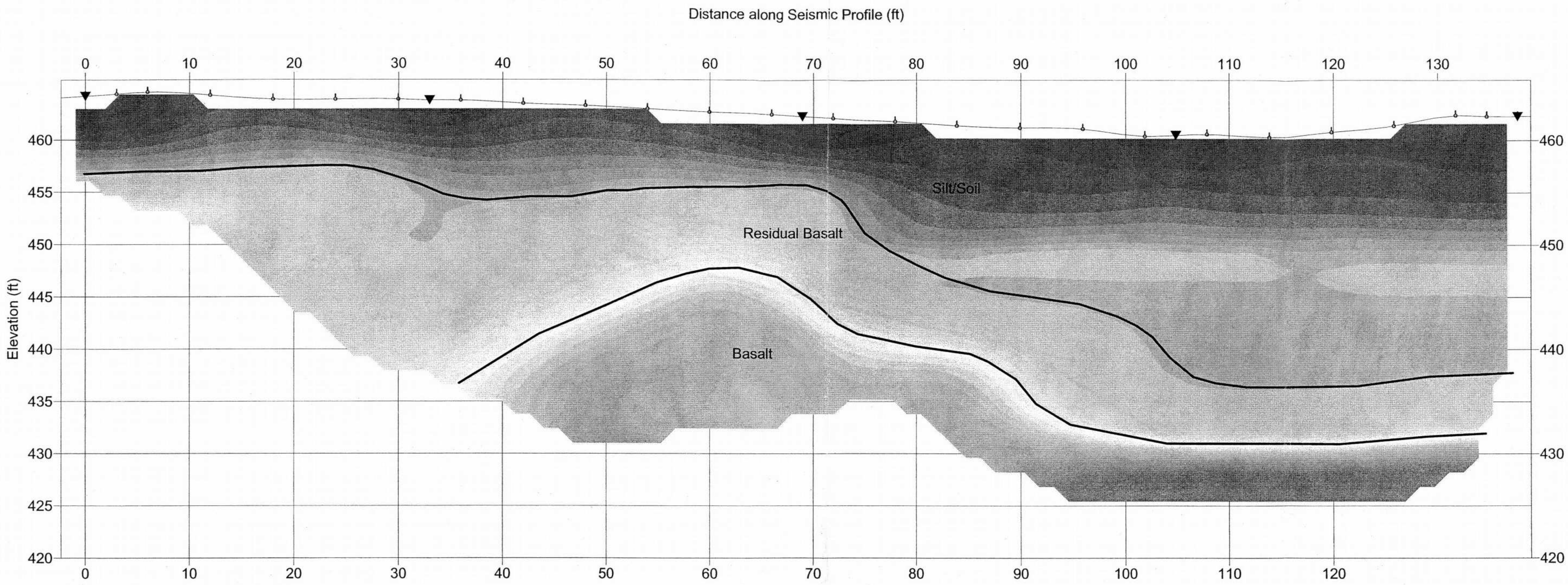
At the Shriner's parking garage site, material with a seismic velocity less than 2,500 ft/s is interpreted to be silt. Material with a seismic velocity ranging from 2,500 to 5,000 ft/s is interpreted to be severely weathered basalt or "residual basalt". Material with a seismic velocity greater than 5,000 ft/s is interpreted to be basalt bedrock. Increasing velocity indicates decreasing weathering and fracturing of the rock.

6.0 LIMITATIONS

The geophysical methods used in this study involve the inversion of measured data. Theoretically, the inversion process yields an infinite number of models which will fit the data. Further, many geologic materials have the same seismic velocity. We have presented models and interpretations which we believe to be the best fit given the geology and known conditions at the site. However, no warranty is made or intended by this report or by oral or written presentation of this work.

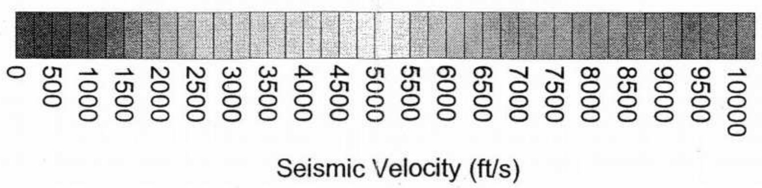
Table 1. Typical seismic velocities for geologic materials.

Description	Velocity (ft/sec)
Top Soils: Loose and dry Moist loamy or silty Clayey Wet Loam	 <1,000 1,000 – 1,300 1,300 – 2,000 2,400 – 2,600
Clay	2,900 – 8,800
Loose rock, talus (dry)	1,100 – 3,000
Sand (wet)	2,000 – 8,200
Gravel and cobble alluvium	4,900 – 7,400
Sandstone	4,600 – 14,000
Weathered and fractured rock	1,500 – 8,000
Intact Basalt or Diabase	7,000 – 16,000



Horizontal Scale: 1" = 10'
Vertical Scale: 1" = 10'

Elevations surveyed with level and rod.
Tied to CJZ Control # 102 Elevation 471.4'



▼ Shot point

ȏ Geophone

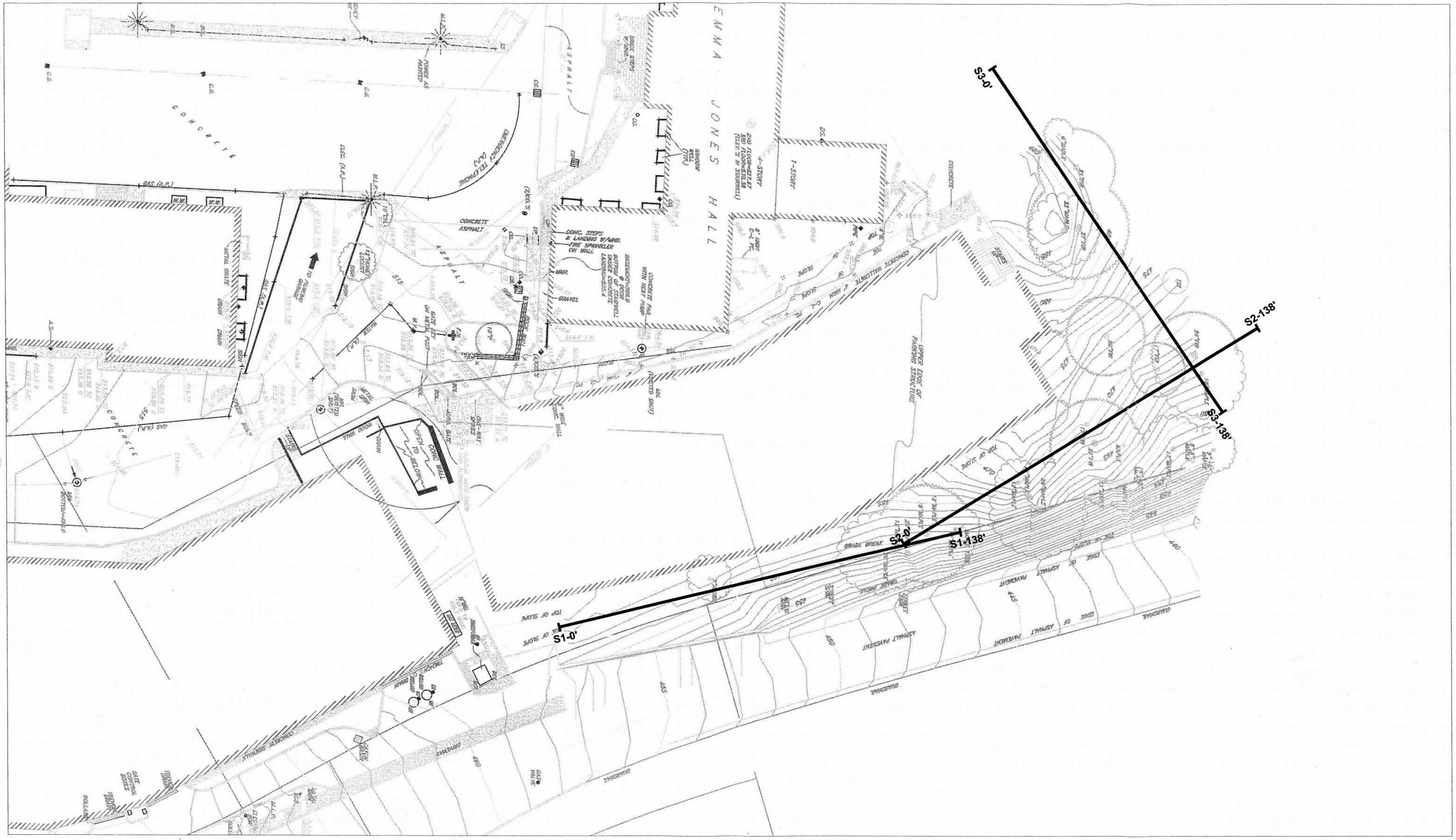
EARTH DYNAMICS
2284 NW Thurman St.
Portland, OR 97210
(503) 227-7659
Email: MFeves@earthdyn.com

Shriner's Hospital Parking Garage

Seismic Line 1

Job #: 07205	Date: 4/20/07	Figure: 2

Highest Nz = 47



Earth Dynamics seismic lines added to AutoCad file produced by Chase, Jones & Associates, Inc.

LEGEND

▬ Black lines indicate seismic refraction line locations

N

Scale: 1" = 30'

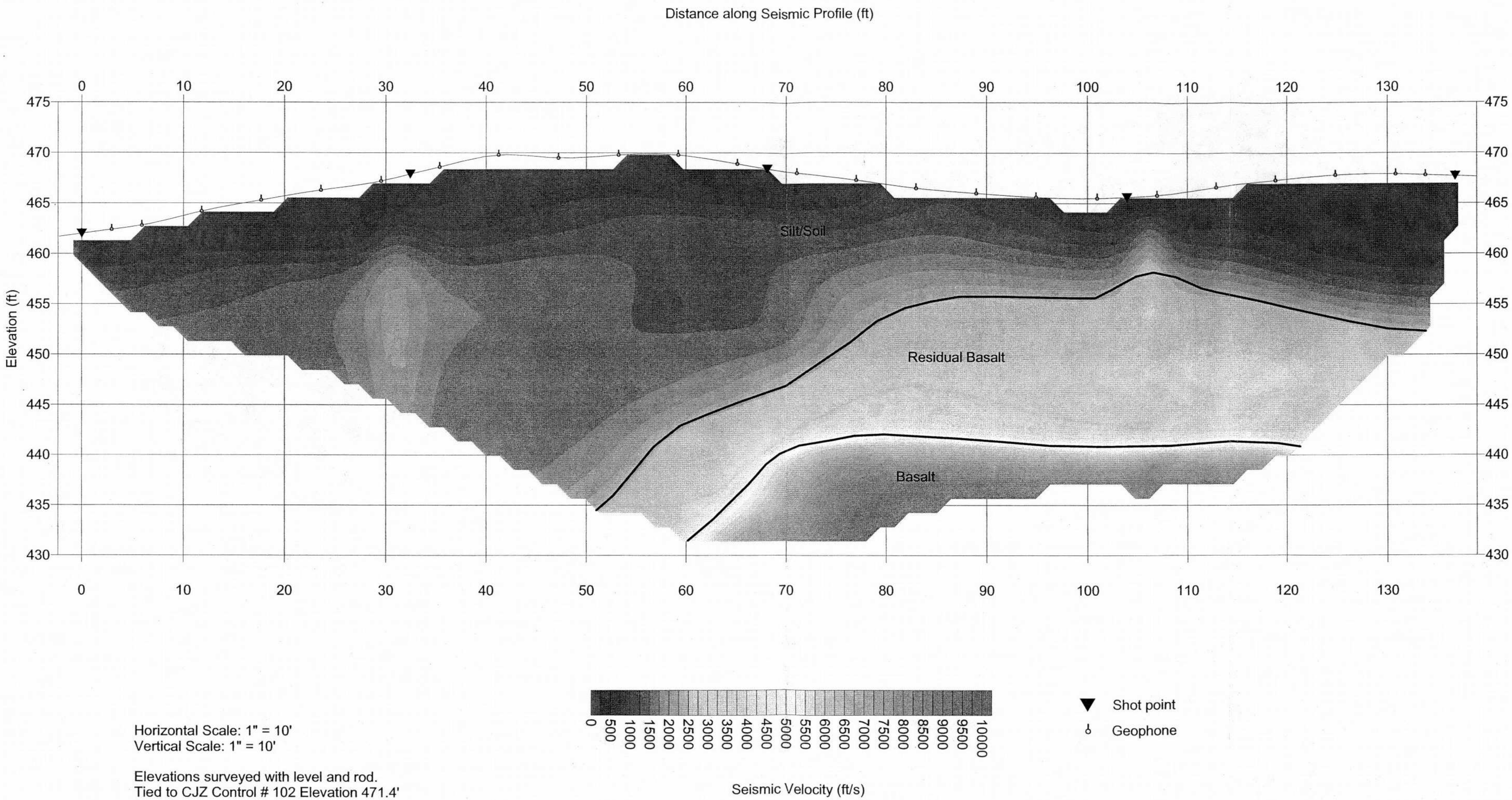
0 10 20 30

EARTH DYNAMICS
2284 NW Thurman Street
Portland, Oregon 97210
(503) 227-7659 (Office)
(503) 227-1074 (FAX)
Email: MFeves@earthdyn.com

Shriners Hospital Parking Garage
Seismic Refraction Study

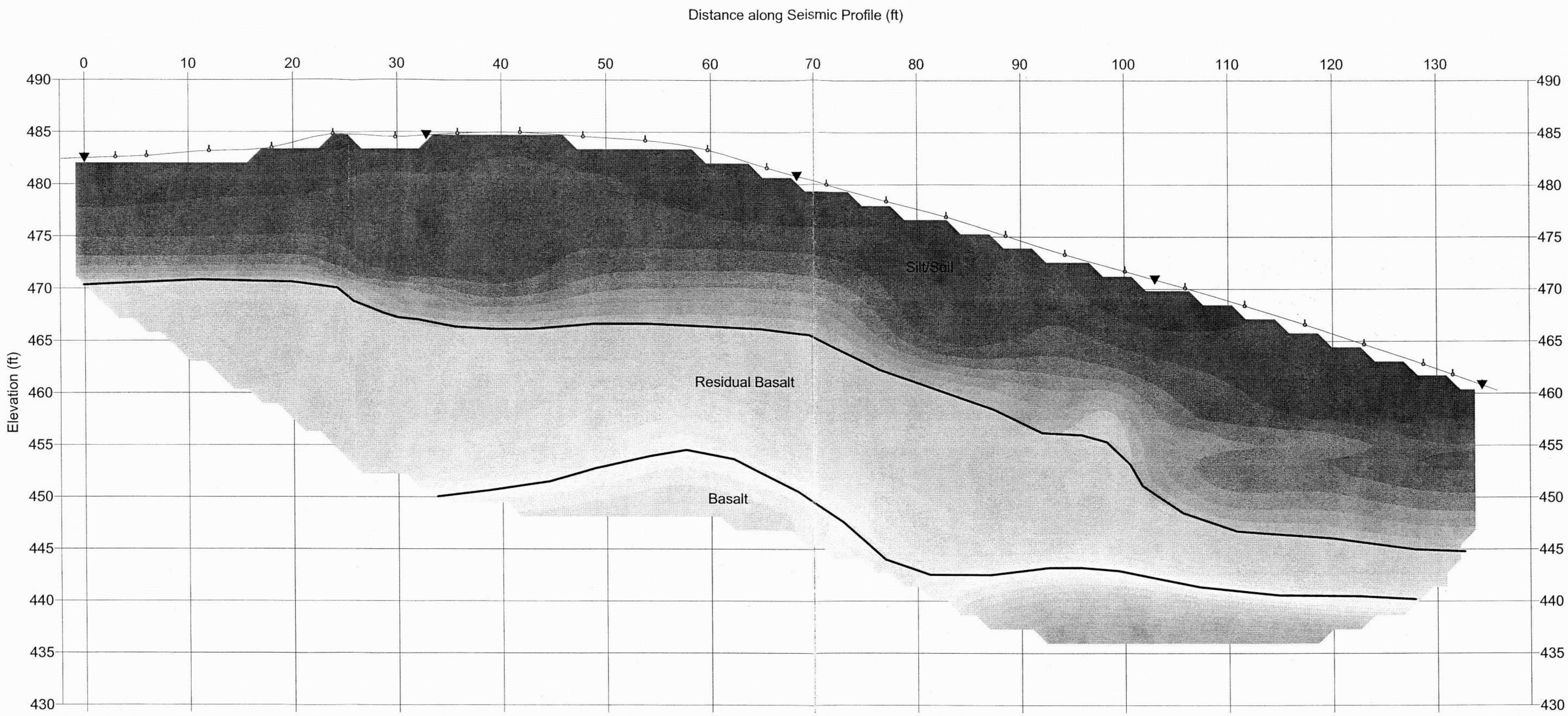
Site Layout with
Seismic Refration Profile Locations

Job #: 07205	Date: 05/10/07	Figure: 1



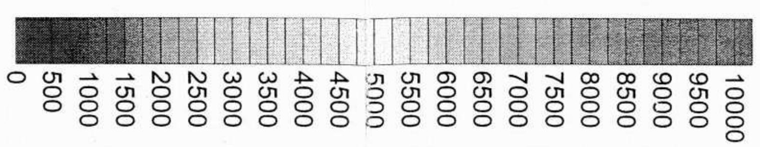
EARTH DYNAMICS 2284 NW Thurman St. Portland, OR 97210 (503) 227-7659 Email: MFeves@earthdyn.com	Shriner's Hospital Parking Garage Seismic Line 2		
	Job #: 07205	Date: 4/20/07	Figure: 3

Highest Nz = 52



Horizontal Scale: 1" = 10'
Vertical Scale: 1" = 10'

Elevations surveyed with level and rod.
Tied to CJZ Control # 102 Elevation 471.4'



Seismic Velocity (ft/s)

▼ Shot point
Geophone

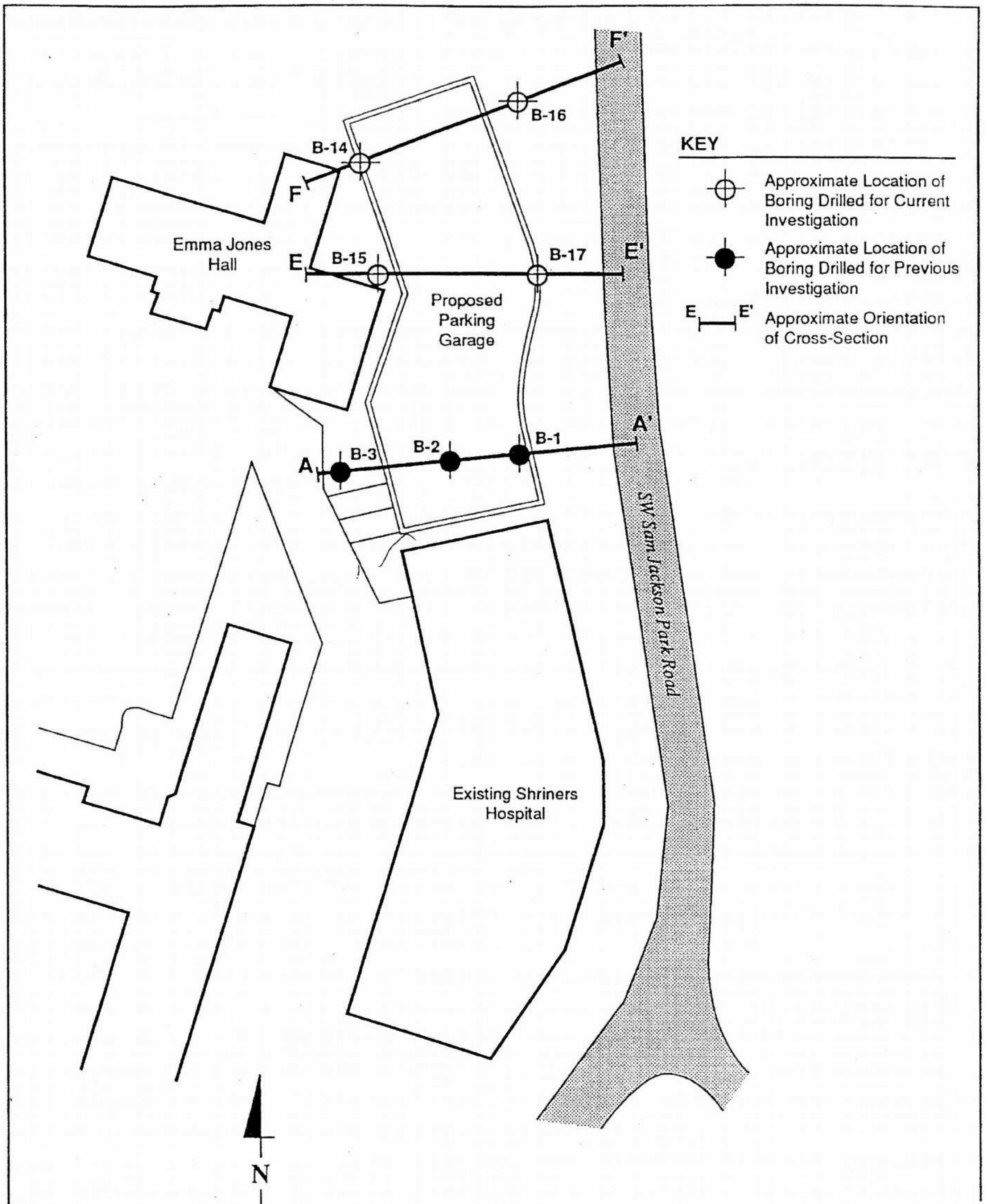
EARTH DYNAMICS
2284 NW Thurman St.
Portland, OR 97210
(503) 227-7659
Email: MFeves@earthdyn.com

Shriner's Hospital Parking Garage

Seismic Line 3

Job #: 07205	Date: 4/20/07	Figure: 4

Highest Nz = 62



SITE PLAN

Shriners Hospital
Parking Garage
Portland, Oregon

August 1993
796-009-041

DAMES & MOORE

FIGURE 2

BORING B-1

Date Drilled: January 19, 1979

Surface Elevation: 472.4 feet

Depth in Feet	Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type		Description
0						ML	Brown SILT, loose, moist, with abundant roots and fine organics [TOPSOIL].
							Brown SILT, medium stiff, moist to wet, with small roots.
						ML	Light brown clayey SILT, stiff, moist.
5						ML	Light brown SILT, stiff, moist.
10							
15							Small weathered rock fragments.

LABORATORY TEST DATA

NOTE: Auger refusal encountered at a depth of 15 feet on basalt.
Groundwater not encountered during drilling.

KEY:

■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.

□ Sampling attempt with no recovery.

◪ Standard Penetration Test sample.

☒ Disturbed sample.

▽ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

BORING B-2

Date Drilled: January 22, 1979

Surface Elevation: 490.9 feet

Depth in Feet	LABORATORY TEST DATA: Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type		Description
0						ML	Brown SILT, soft, wet, with abundant roots and fine organics [TOPSOIL].
							Brown SILT, soft to medium stiff, wet to moist, small roots in upper 2 feet.
5							Scattered weathered rock fragments.
10							
15							

NOTE: Auger refusal encountered at a depth of 19 feet on basalt.
Groundwater not encountered during drilling.

KEY:

■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.

□ Sampling attempt with no recovery.

◪ Standard Penetration Test sample.

⊠ Disturbed sample.

∇ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

BORING B-3

Date Drilled: January 18, 1979

Surface Elevation: 516.1 feet

Depth in Feet	LABORATORY TEST DATA: Test Reported Elsewhere	LABORATORY TEST DATA: Moisture Content (%)	LABORATORY TEST DATA: Dry Density (PCF)	Blows/Foot	Sample Type		Description
0							Asphaltic concrete approximately 3" thick.
						ML	Aggregate base approximately 9" thick.
							Brown sandy SILT, medium stiff, moist.
5		28	91	4	■		
10		27	90	14	■		
15		19	90	17	■		Consistency grades to stiff.
20		30	90	10	■		
25		34	89	8	■		
30				19	■	ML	Brown clayey SILT, very stiff, wet.
							Consistency increases to hard.
						CL	Red-green silty CLAY, hard, wet, (very intensely weathered basalt).
35		49	81	100	■		Power auger boring terminated. Diamond core drilling begins.
							Gray BASALT, slightly to moderately weathered, moderately to highly fractured, slightly vesicular, scattered seams of reddish-brown clayey sandy silt.
40							

Continued Next Page

KEY:

- ■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.
- ☐ Sampling attempt with no recovery.
- ◪ Standard Penetration Test sample.
- ☒ Disturbed sample.
- ▽ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

Figure A-3a

BORING B-3

Date Drilled: January 18, 1979

Surface Elevation: 516.1 feet

Depth in Feet	Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type	Description
40						Gray basalt, slightly weathered, moderately fractured.
						Gray basalt, slightly weathered, moderately to highly fractured.
45						
50						Gray basalt, moderately weathered, highly fractured, slightly vesicular.
						2' thick layer of black crumbly basalt, moderately to highly weathered, highly vesicular.
55						Dark gray basalt, moderately weathered, highly fractured, highly vesicular.
60						Gray basalt, moderately weathered, highly fractured, moderately vesicular.
						Gray basalt, slightly weathered, moderately fractured, slightly vesicular.
65						

NOTE: Boring completed at a depth of 67 feet. Groundwater not encountered during drilling.

KEY:

- ■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.
- □ Sampling attempt with no recovery.
- ◪ Standard Penetration Test sample.
- ⊠ Disturbed sample.
- ▽ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

Figure A-3b

BORING B-14

Date Drilled: June 28, 1993

Surface Elevation: 502.7 feet

Depth in Feet	Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type		Description
0						ML	Brown SILT with trace of clay, stiff to very stiff, wet, slightly micaceous. Possible fill in upper portion.
5		25.3	93	32	■		
							Color grades lighter brown.
10	DS	33.6	86	32	■		
15	DS	23.8	90	43	■		Material grades to silt with trace of sand.
				23	◩		
20							
				13	◩		
25							
				17	◩		
30							Minor gravel encountered.
				190/9"	◩	CL	Green silty CLAY, stiff to hard, wet (intensely weathered basalt).
							Basalt, hard, wet, moderately to highly fractured, lightly weathered.
35							
				50/1"	—		

NOTE: Boring completed in basalt at 37.5 feet on June 28, 1993.
Groundwater not encountered during drilling.

KEY:

- ■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.
- □ Sampling attempt with no recovery.
- ◩ Standard Penetration Test sample.
- ⊠ Disturbed sample.
- ▽ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

BORING B-15

Date Drilled: June 29, 1993

Surface Elevation: 504.5 feet

Depth in Feet	Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type
0					
		30.3	88	16	■
5					
	DS	23.5	96	17	■
10					
				11	◪
15					
				14	◪
20					
				9	◪
25					
				18	◪
30					
				50/6"	◪
					⊠

LABORATORY TEST DATA

ML	Brown SILT with trace of clay, stiff to very stiff, moist to wet, slighty micaceous. Possible fill in upper portion. Color grades lighter.
ML	Light brown SILT with trace of sand, stiff to very stiff, wet, slightly micaceous.
CL	Green-gray silty CLAY, very stiff, wet (intensely weathered basalt).
	Basalt, hard, wet, moderately to highly fractured, lightly weathered, brown silty clay infill in fractures.

NOTE: Boring completed in basalt at 34 feet on June 29, 1993.
Groundwater not encountered during drilling.

KEY:

- ■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.
- ☐ Sampling attempt with no recovery.
- ◪ Standard Penetration Test sample.
- ⊠ Disturbed sample.
- ▽ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

BORING B-16

Date Drilled: July 1, 1993

Surface Elevation: 471.0 feet

LABORATORY TEST DATA

Depth in Feet	Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type
0–5				23	■
5–10		10.6	93	62	■
10–15	DS			95/10"	■
15–20		29.3	91	51/6"	■
20–25	DS			70/6"	■
25–30				50/1"	■
30–					⊠

OL — Brown sandy SILT, soft, wet, abundant roots and fine organics [TOPSOIL].

ML — Brown SILT with trace sand, stiff, wet, slightly micaceous

Consistency increases to hard.

Color grades lighter, moisture content decreases to moist.

Medium brown silt lense approximately 6" thick.

Thin layer of green very intensely weathered basalt.

Brown BASALT, hard, wet, vesicular, highly fractured, moderately weathered.

Color grades to green-brown.

Gray and green BASALT, hard, wet, vesicular, moderately fractured, slightly weathered.

NOTE: Boring completed in basalt at 33.5 feet on July 1, 1993.
Groundwater not encountered during drilling.

KEY:

- ■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.
- □ Sampling attempt with no recovery.
- ◪ Standard Penetration Test sample.
- ⊠ Disturbed sample.
- ∇ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

BORING B-17

Date Drilled: July 1, 1993

Surface Elevation: 463.5 feet

LABORATORY TEST DATA							
Depth in Feet	Test Reported Elsewhere	Moisture Content (%)	Dry Density (PCF)	Blows/Foot	Sample Type		
0						OL	Brown sandy SILT, soft, wet, abundant rootlets and fine organics [TOPSOIL].
						ML	Light brown to brown SILT with trace of sand, stiff to very stiff, moist to wet, slightly micaceous.
5				32	■		
10		28.0	91	30	■		Red-brown clayey SILT with sand, very stiff, wet, slightly micaceous.
							Red-brown BASALT, hard, wet, very intensely weathered to soil-like consistency.
15				50/3"	■		Weathering decreases to moderate, highly fractured.
20				75/6" 50/2"	◪		

NOTE: Boring completed in basalt at 20 feet on July 1, 1993.
Groundwater not encountered during drilling.

KEY:

- ■ Relatively undisturbed sample obtained with a Dames & Moore Type-U sampler driven with a 140 pound hammer falling 30 inches.
- □ Sampling attempt with no recovery.
- ◪ Standard Penetration Test sample.
- ⊠ Disturbed sample.
- ▽ Groundwater depth at time of drilling.

BORING LOG

DAMES & MOORE

Figure A-7

APPENDIX D

SITE-SPECIFIC SEISMIC HAZARD STUDY

General

GRI has completed a site-specific seismic hazard study for the proposed expansion of the Shriners Hospital for Children in Portland, Oregon. The purpose of our study was to evaluate the potential seismic hazards associated with regional and local seismicity. Our work was based on the potential for regional and local seismic activity as described in the existing scientific literature and on the subsurface conditions at the site interpreted from subsurface explorations. Specifically, our work included the following tasks:

1) A detailed review of the literature, including published papers, maps, open-file reports, seismic histories and catalogs, works in progress, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

2) Compilation, examination, and evaluation of existing subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the site.

3) Identification of the potential seismic events (earthquakes) appropriate for the site and characterization of those events in terms of a series of generalized design events.

4) Office studies, based on the generalized subsurface profile and the generalized design earthquakes, resulting in conclusions and recommendations concerning:

   a) specific seismic events that might have a significant effect on the site,

   b) the potential for seismic energy amplification at the site, and

   c) site-specific acceleration response spectra for each of the design earthquakes.

This appendix describes the work accomplished and summarizes our conclusions.

Geologic Setting

The general area occupied by the subject site is underlain by up to approximately 30 ft of unconsolidated loess of Quaternary age. Locally, these deposits consist of well-sorted silt and fine-grained sand that, on exposed surfaces, weathers to clayey silt. These deposits are underlain by deposits of the Columbia River Basalt Group. The boundary between the overlying sedimentary materials and the underlying basalt is unconformable, indicating that a considerable period of time elapsed between the solidification of the last of the basalt flows and the deposition of the overlying sedimentary materials. During this period of time, the upper portion of the basalt was subjected to surficial processes, including erosion, mass wasting, and chemical and physical degradation. These processes resulted in severe weathering of the upper portions of the basalt. The thickness of this weathered zone is quite variable from place to place, but is approximately

20 ft thick at the site. This zone of severely weathered basalt is underlain by a zone of fresh basalt that is extensively fractured.

On a regional scale, the site lies at the northern end of the Willamette Valley, a broad, gently deformed, north-south-trending topographic feature separating the Coast Range to the west from the Cascade Mountains to the east. The valley lies approximately 120 km inland from the surface expression of the Cascadia Subduction Zone (CSZ), an active plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca and Explorer plates) are being subducted beneath the western edge of the North American continent. The configuration of these plates and the location, extent, and geometry of the surface expression of the subduction zone are shown schematically on the Tectonic Setting Summary, Figure 1B(a). The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the over-riding North American Plate, as shown schematically on Figure 1B(b).

On a local scale, the site lies in the west-central portion of the Portland Basin, a well-defined structural basin bounded by high-angle, northwest-trending, right lateral strike slip faults. The distribution of these faults relative to the site is shown on the Regional and Local Geologic Maps, Figures 2B and 3B. Within the basin, information regarding the precise location and extent of discrete faults is lacking, although a limited number of intrabasin faults have been mapped on the basis of stratigraphic offsets and geophysical evidence. The relationship between specific earthquakes and individual faults in the area is not well understood, since few of the faults in the area are expressed clearly at the ground surface. Other faults may be present within the basin, but clear stratigraphic evidence regarding their location and extent is not presently available.

Because of the proximity of the site to the CSZ and its location within the fault-bounded Portland Basin, three distinctly different sources of seismic activity contribute to the potential for the occurrence of damaging earthquakes at the site. Each of these sources is considered capable of producing damaging earthquakes. Two of these sources are associated with deep-seated tectonic activity related to the subduction zone; the third is associated with movement on the local, relatively shallow structures within and adjacent to the Portland Basin.

Seismicity

The geologic and seismic information available for identifying the potential seismicity at the site is incomplete, and large uncertainties are associated with any estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The information that is available indicates that the potential seismic sources that may affect the site can be grouped into three independent categories: *subduction zone events* related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, *subcrustal events* related to deformation and volume changes within the subducted mass of the Juan de Fuca plate, and *local crustal events* associated with movement on shallow, local faults within and adjacent to the Portland Basin. Based on our review of currently available information, we have developed generalized design earthquakes for each of these categories. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, size is expressed in Richter (local) magnitude (M_L), surface wave magnitude (M_S), Japanese Meteorological Association magnitude (M_{JMA}), or moment magnitude (M_W); location is expressed as epicentral or focal distance, measured radially from the subject site in kilometers; and peak horizontal bedrock accelerations are expressed in gravities (1 g = 32.2 ft/sec^2 = 980.6 cm/sec^2).

Subduction Zone Event. There have not been any interplate earthquakes on the CSZ in the historical past; however, geological studies show that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater and others, 1995; Clague and others, 1997; Goldfinger, 2003; and Kelsey and others, 2005), and geodetic studies (Hyndman and Wang, 1995; Savage and others, 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck and others, 1997; Wang and others, 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and others, 1994; Mitchell and others, 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single M_w 9 earthquake (Satake and others, 1996; Atwater and Hemphill-Haley, 1997; Clague and others., 2000). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey, 2005, and Goldfinger, 2003). Published estimates of the probable maximum size of subduction zone events range from moment magnitude M_w 8.3 to >9.0. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague, et al., 2000; Goldfinger and others, 2003; Kelsey and others, 2002; Kelsey and others, 2005; Witter and others, 1997). We have chosen to represent the subduction zone event by a design earthquake of M_w 9.0 at a focal depth of 15 km and an epicentral distance of 100 km. This corresponds to a sudden rupture of the whole length of the Juan de Fuca-North American plate interface, placed at the closest approach of the interface, due west of Portland. It should be noted that this choice of a design earthquake is based primarily on an estimate of the capability of the subduction zone to produce a large earthquake, not on a probabilistic analysis of a demonstrated seismic history. Based on the attenuation relationship published by Youngs and others (1997), a subduction zone event of this size and location would result in a peak horizontal bedrock acceleration of approximately 0.14 g at the site.

Subcrustal Event. There is no historic earthquake record of subcrustal, intraslab earthquakes in Oregon. Although both the Puget Sound and northern California region have experienced many of these earthquakes in historic times, Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions and local geology, Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. Offshore, along the northern California coast, the earthquakes are shallower (up to 40 km) and located along the deformation front. Estimates of the probable size, location, and frequency of subcrustal events in Oregon are generally based on comparisons of the CSZ with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from moment magnitude M_w 7.0 to 7.5. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to M_w 7.1, 6.5, and 6.8, respectively. Published information regarding the location and geometry of the subducting zone indicates that a focal depth of 50 km and an epicentral distance of 40 km from the site are probable (Weaver and Shedlock, 1989). We have chosen to represent the subcrustal event by a design earthquake of magnitude M_w 7.0 at focal depth of 50 km and an epicentral distance of 40 km. As with the subduction zone event, this choice is based on an estimate of the capability of the source region rather than on a probabilistic analysis of a historical record of events of this type. Based on an average of attenuation relationships published by Youngs and others (1997), and Atkinson and Boore (1997), a subcrustal event of this size would result in a peak horizontal bedrock acceleration of approximately 0.15 g at the site.

Local Crustal Event. Sudden crustal movements along relatively shallow, local faults in the Portland area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1841), it can serve as a guide for estimating the potential for seismic activity in the area.

Another method for estimating the magnitude to be expected of local crustal events involves an analysis of the lengths of local faults. The empirical relationship between fault rupture length and magnitude of the resulting earthquake has been studied extensively (Matthiesen, 1984; Wells and Coppersmith, 1994). Based on fault mapping conducted by Geomatrix for the Oregon Department of Transportation (1995), Wong, et al. (2000), and the U.S. Geological Survey (USGS, 1996, 2002), the closest mapped faults to the site are the following:

Fault	Distance From Site, km	Mapped Length, km	Characteristic Earthquake Magnitude, M_w
Portland Hills	<1	50	6.8 to 7.1
East Bank	3.5	29	6.6 to 7.1
Oatfield	4	29	6.5 to 6.9

Of these three faults, the USGS only considers the East Bank Fault active in their probabilistic seismic hazard study. The range of characteristic earthquake magnitudes depends on the geometry of the faults at depth and the degree to which the faults are segmented, neither of which is well understood. Based on the attenuation relationships of Boore, et al. (1997), Sadigh, et al. (1997), and Abrahamson and Silva (1997) for a design earthquake of magnitude M_w 7.0 at distance of 3 km, the estimated peak horizontal bedrock accelerations at the site would be approximately 0.56 g. The latter relationships both include site effects and are typically biased toward larger magnitude earthquakes.

Summary of Deterministic Earthquake Parameters

In summary, we have chosen earthquakes of three distinctly different types to represent the seismicity of the project area. Published attenuation relationships were used to estimate the peak bedrock accelerations at the site. The basic parameters of the deterministic earthquakes are as follows:

Earthquake Source	Magnitude, M_w	Epicentral Distance, km	Focal Depth, km	Peak Bedrock Acceleration, g
Subduction Zone	9.0	100	15	0.14
Subcrustal	7.0	40	50	0.15
Local Crustal	7.0	3	N/A	0.56

Probabilistic Considerations and Code Spectra

The probability of an earthquake of a specific magnitude occurring at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake is calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. These expected earthquake recurrences are expressed as a probability of exceedance during a given time period or design life. Historically, building codes have adopted an acceptable risk level by identifying ground acceleration values that meet or exceed a 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years. The International Building Code (IBC, 2006) develops a design spectrum by using two-thirds of the Maximum Considered Earthquake (MCE) ground motion. The MCE earthquake is generally defined as a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years) except where subject to deterministic limitations (Leyendecker, et al., 2000). The change to a MCE was an effort to reduce the risk of building collapse in portions of the country where the earthquake with a 2,500-year recurrence interval is significantly larger than the previous code recurrence interval of 475 years. The IBC design response spectrum is two-thirds of the MCE ground motion, which adjusts the design spectrum to a more traditional "life safety" level, previously represented by the 475-year recurrence interval earthquake (Holmes, 2000). The intent of the IBC is to insure structural design that will prevent collapse of a building subjected to 1.5 times the design acceleration response spectrum, which generally means that a structure designed and constructed in accordance with the design spectrum will not collapse during the MCE.

The 2006 IBC design methodology uses spectral response accelerations, S_S and S_1, corresponding to periods of 0.2 and 0.1 seconds, and the Site Class coefficients to develop the design response spectrum. The S_S and S_1 values for the site are 0.99 and 0.35 g, respectively. The spectral accelerations are based on the 2002 U.S. Geological Survey (USGS) probabilistic mapping effort. The USGS mapping identifies the likelihood of movement for all of the seismic sources (i.e., local crustal, subcrustal, and subduction zone earthquakes). In addition, the USGS mapping probabilistically determines a single acceleration response spectrum curve for bedrock.

Recommended Design Spectrum

The majority of the silt profile has been removed from the site during construction of the existing parking garage and only a relatively thin layer of residual soil or severely weathered basalt remains in place over the underlying relatively hard basalt. Therefore, based on the subsurface conditions encountered at the site and our understanding of the foundation grades, the code-based IBC Site Class C spectrum is appropriate to evaluate the site response. In accordance with the IBC, the design earthquake is two-thirds of the MCE. The corresponding IBC Site Class C response spectrum developed from the probabilistic coefficients described above is shown on Figure 4B.

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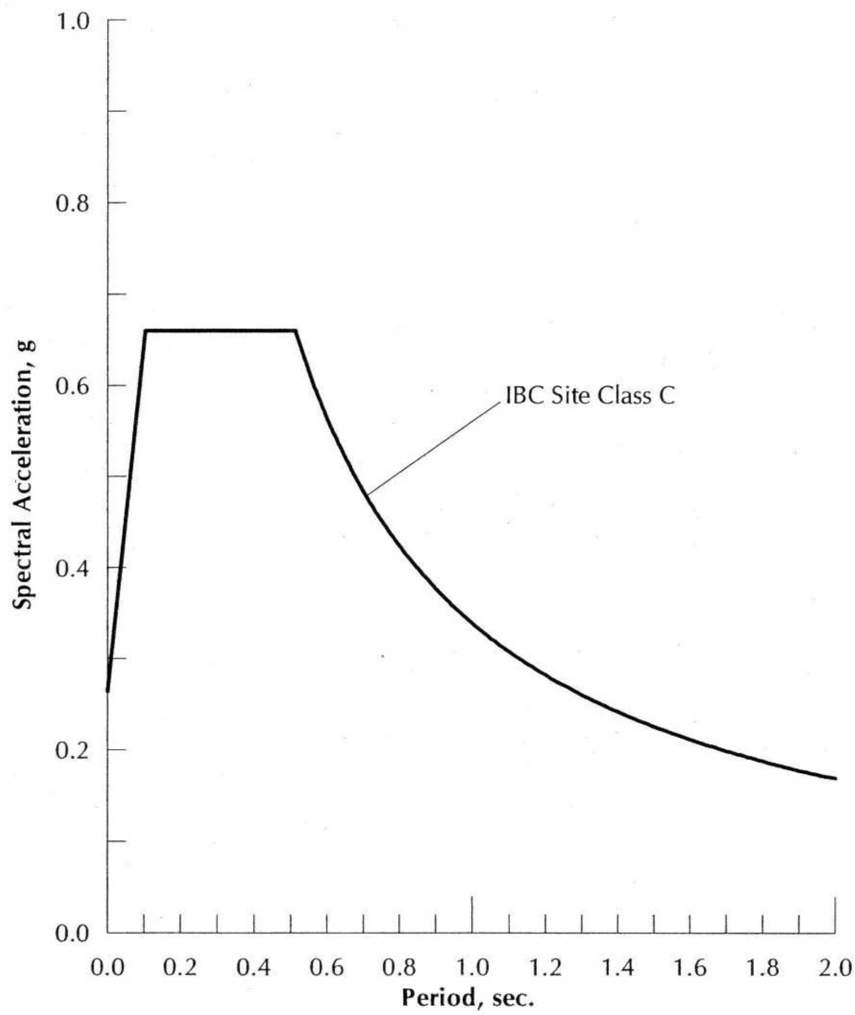
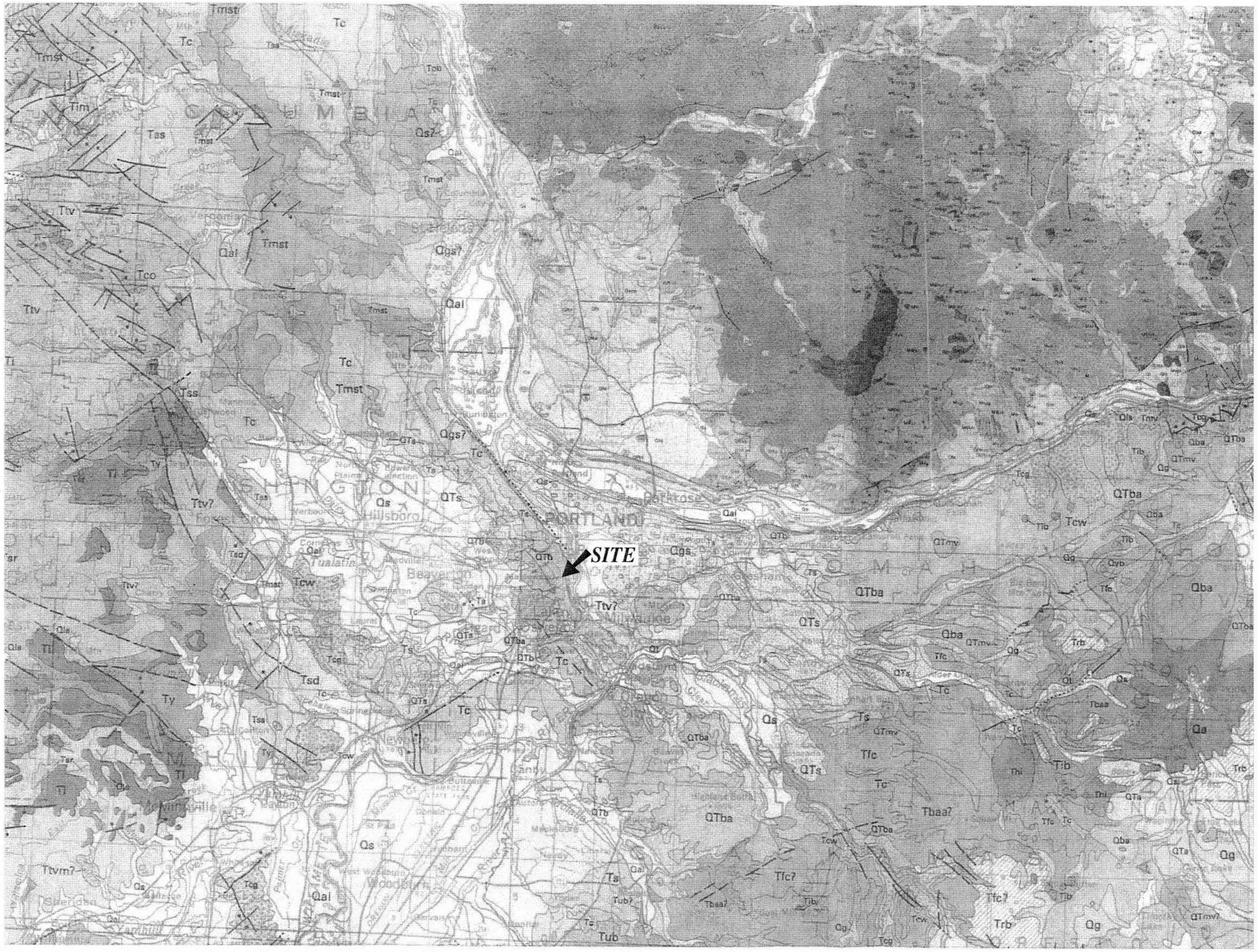


GRI

DESIGN SPECTRUM

(5% DAMPING)

AUG. 2007 | JOB NO. 4666 | FIG. 4D



——— **Contact** — Approximately located

—•?—••• **Fault** — Dashed where inferred; dotted where concealed; queried where doubtful; ball and bar on downthrown side

—▲?▲•▲• **Thrust fault** — Dashed where inferred; dotted where concealed; queried where doubtful; sawteeth on upper plate

FROM:

WALSH, T.J., KOROSEC, M.A., PHILLIPS, W.M., LOGAN, R.L., AND SCHASSE, H.W., 1987, GEOLOGIC MAP OF WASHINGTON-SOUTHWEST QUADRANT; 1:250,000: WASHINGTON DIVISION OF GEOLOGY AND EARTH RESOURCES, GM-34

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North

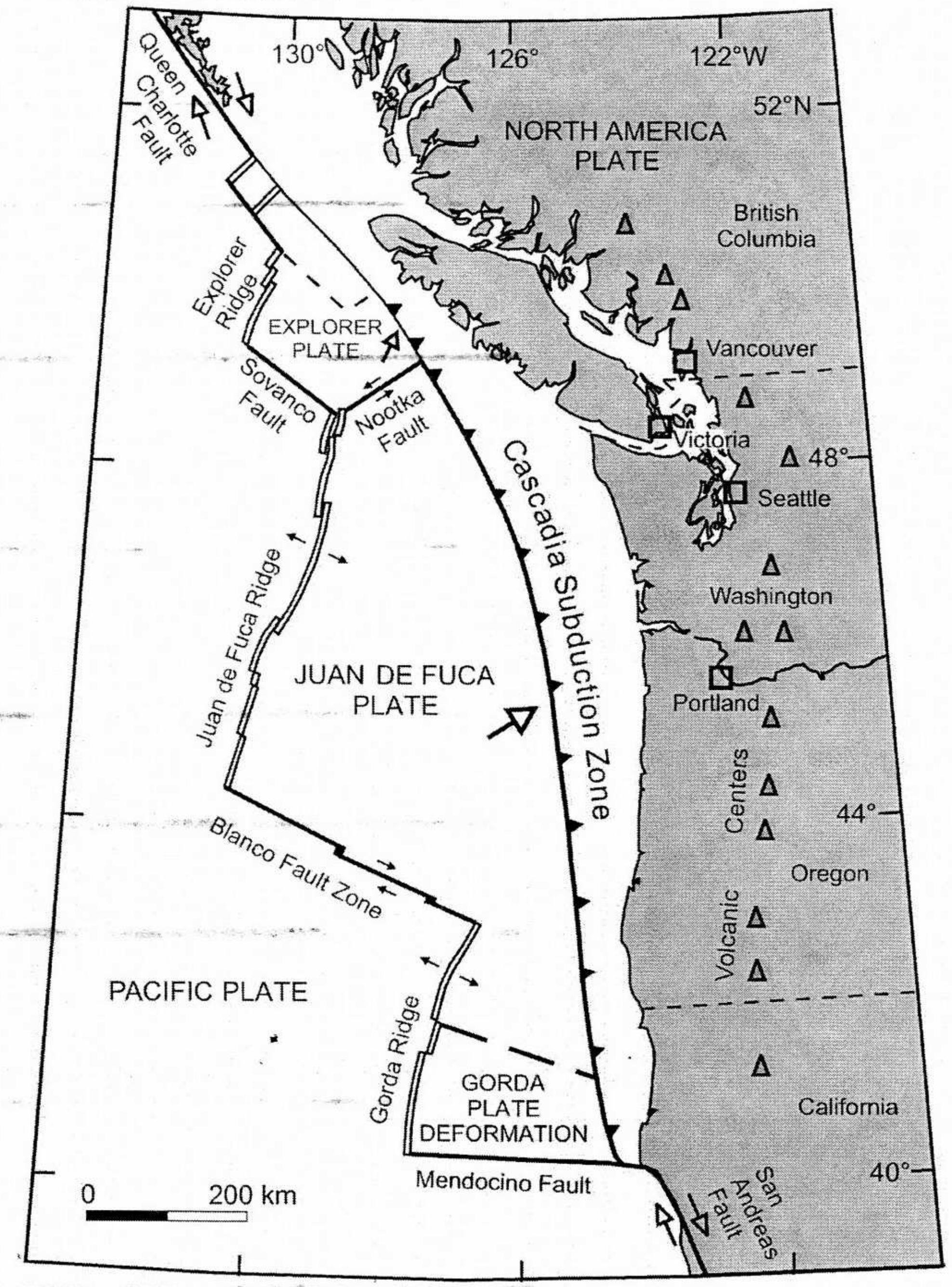
0 10 20 MILES

0 10 20 KILOMETERS

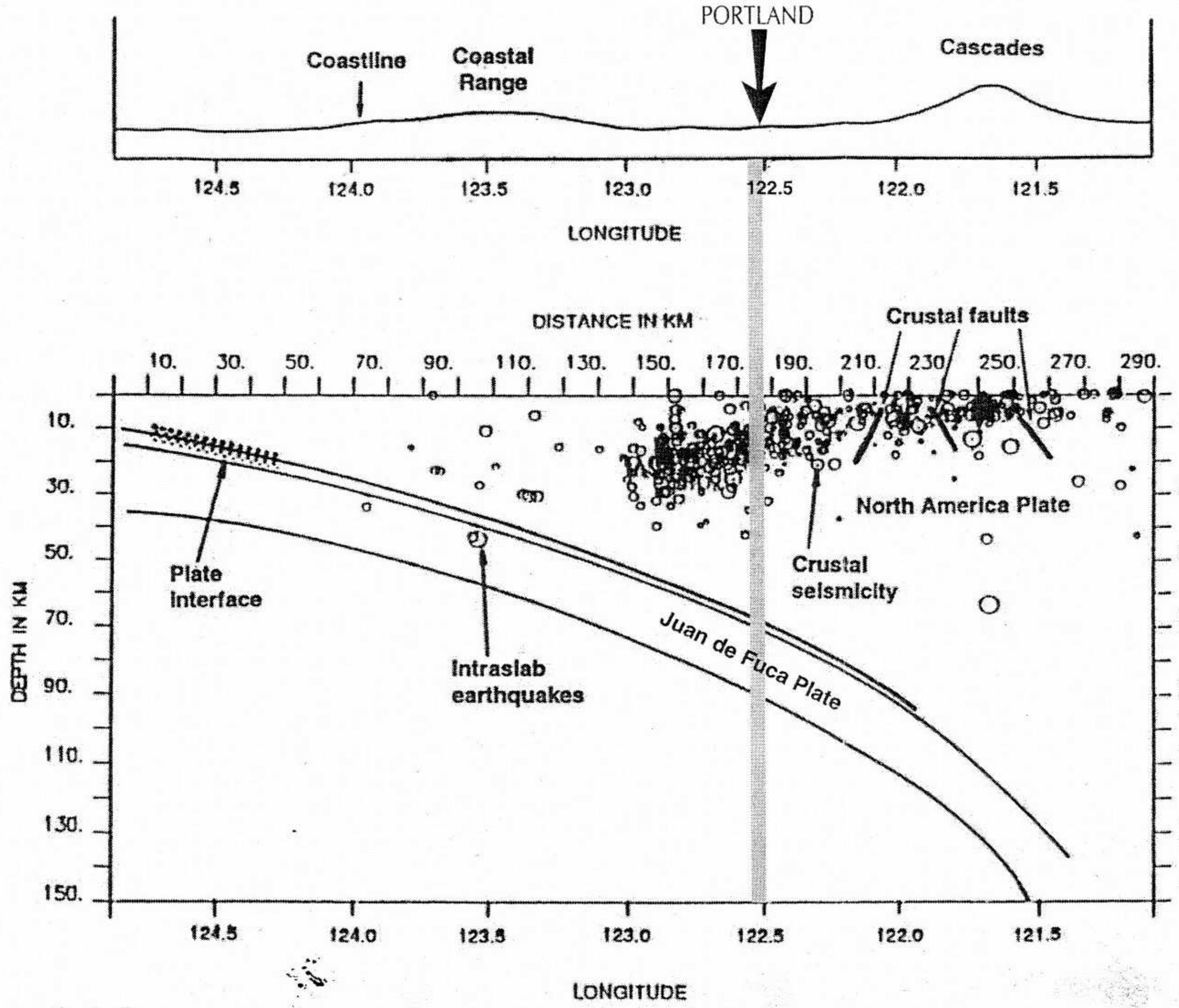
GRI

REGIONAL GEOLOGIC MAP

JUNE 2007 JOB NO. 4666 FIG. 2D



A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

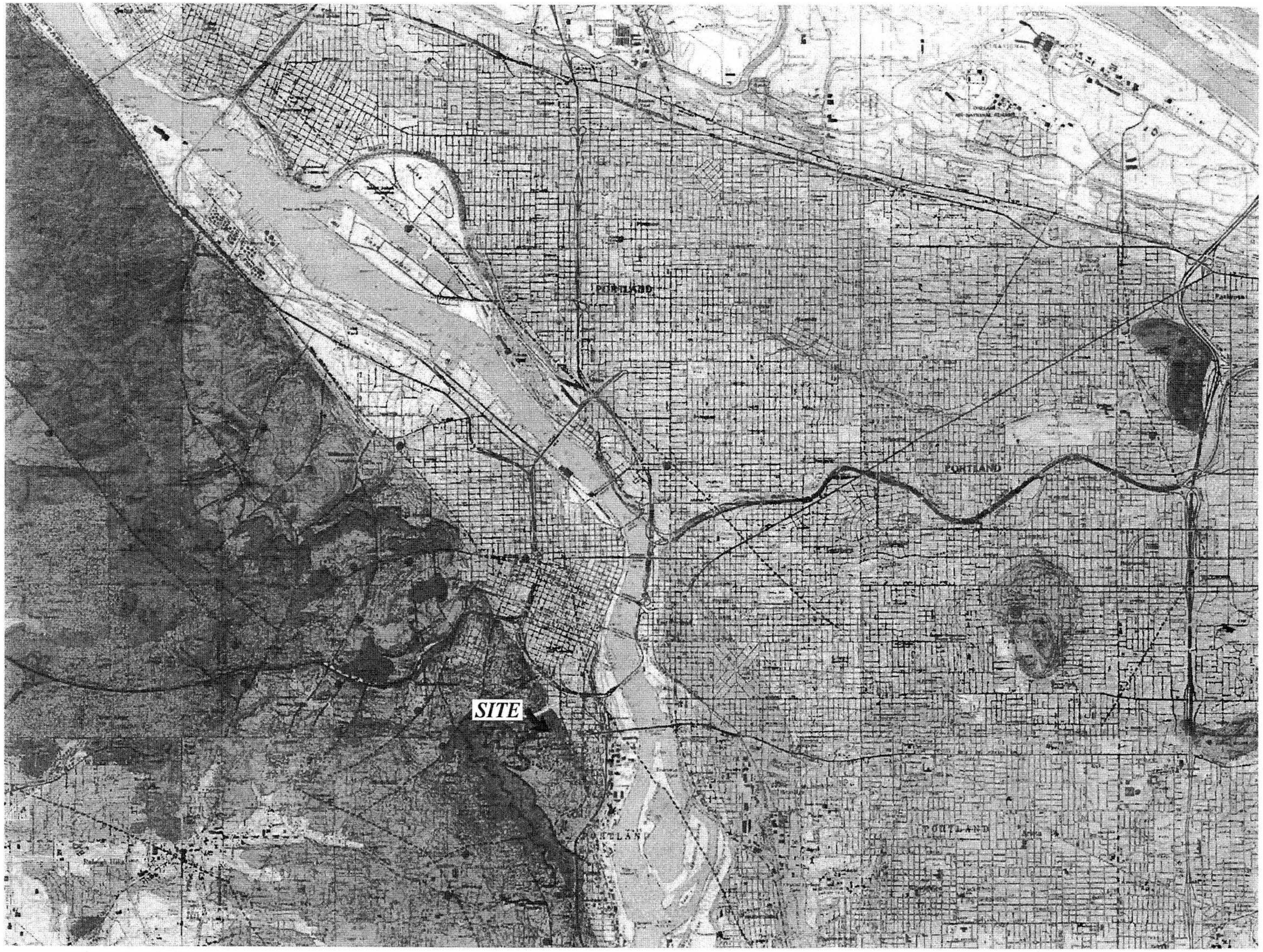


B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF PORTLAND, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)

GRI

TECTONIC SETTING OF THE PACIFIC NORTHWEST

JUNE 2007 | JOB NO. 4666 | FIG. 1D



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North

0 $2^1/2$ 5 KILOMETERS

GRI

LOCAL GEOLOGIC MAP

AUG. 2007 JOB NO. 4666 FIG. 3D