Pacific Geotechnical, LLC
Geotechnical Engineering and Engineering Geology with a Focus on Sustainability

April 24, 2008

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Walker Macy
111 SW Oak, Suite 200
Portland, Oregon 97204

Attention Mrs Chelsea McCann

Subject Revised Seismic Site Hazard Report
Portland Saturday Market Cover Structure
Portland, Oregon
Project No 1050-001-00

1 INTRODUCTION AND PROJECT UNDERSTANDING

Pacific Geotechnical, LLC (Pacific Geotechnical) is pleased to submit this revised seismic site hazard report (SSHR) for the Portland Saturday Market (PSM) cover structure for the above project in Portland, Oregon Pacific Geotechnical completed a geotechnical report for the project dated April 12, 2007, and follow up technical documents including a supplemental geotechnical report for pile foundations to support the PSM structure, dated February 18, 2008 This report has been revised to update the seismic design parameters and clarify and revise our site specific response analysis based on City of Portland review comments dated April 11, 2008 The location of the site is shown on Figure 1

The project consists of relocating the PSM into Waterfront Park Construction associated with this move will include a new cover structure and overlook deck, reconfiguration of some pedestrian ways, installation of an interactive water feature, and appurtenant facilities such as piping and benches The PSM cover structure will be located just south of the Burnside Bridge, as shown in Figure 2

According to the project architect, the PSM cover structure is classified as an A3 assembly space with a maximum occupancy load of 1,698 people Based on this description and a discussion with City of Portland (City) Bureau of Development (BDS) staff, an SSHR is required for the PSM cover structure

2 PURPOSE AND SCOPE OF SERVICES

The purpose of our work is to provide an SSHR for the project Our specific scope of work included the following

- Reviewed applicable codes to determine whether an SSHR is required
- Reviewed existing available information, including subsurface information collected for our supplemental geotechnical report
- Used the above collected information to complete an SSHR in accordance with Section 1802 of the 2007 Oregon Structural Specialty Code (OSSC)
- Provided a total of five copies of this report to you Four copies are provided for your submittal to the City and one copy is provided for your submittal to the Department of Geology and Mineral Industries (DOGAMI), as required by the 2007 OSSC

3 SITE DESCRIPTION

3 1 REGIONAL AND LOCAL GEOLOGY

The *Geotechnical Baseline Report, West Side CSO Tunnel, Shafts, Pump Station and Pipelines Project* (Parsons Brinckerhoff, 2002a) contains a well-crafted explanation of the regional geologic setting of the project area as well as the history of local faulting and geomorphology (landforms) The following paragraphs briefly summarize the information presented in that report

The project is located along the west edge of the Willamette River, a tributary of the Columbia River that flows through the Portland Basin This pull-apart basin, a product of regional tectonic stresses related to the subduction of the Juan de Fuca plate with the over-riding North American plate, has been subsiding since at least Miocene time Tens of hundreds to thousands of feet of alluvial clay, silt, sand and gravel have been deposited in the Portland Basin between intervals of uplift, scour and erosion that have created extensive topographic relief in the basin sediments Massive "Missoula" floods scoured and then deposited sediment ranging from clay to gravel in the Portland Basin during the latest Pliocene to latest Pleistocene (~1 million to 12,500 years ago) Post-Pleistocene geologic activity in the basin includes Willamette River fluvial erosion and deposition of sediment as well as the placement of large quantities of man-made fill along the Willamette riverfront and in adjacent sloughs, lakes and bottomlands

The regional Columbia River Basalt (CRB) bedrock that forms the Portland Basin and adjacent West Hills (Tualatin Mountains) was formed 5 to 15 million years ago by an immense outpouring of lava from vents along the Oregon/Washington-Idaho border that covered this portion of the Pacific Northwest to depths of thousands of feet with igneous rock As the basalt-floored Portland Basin subsided, sediment eroded both from the nearby Cascade Mountain chain as well as from continental interior sources was carried into the basin by the Columbia and other, local rivers and deposited as the Plio-Pleistocene Sandy River Mudstone (SRM) and overlying Troutdale Formation sandy to silty gravel Alluvial processes ranging from the catastrophic floods to everyday fluvial activity of local rivers and streams eroded channels in, as well as deposited additional sediment on, the SRM and the Troutdale The material related to the Missoula Flooding is generally referred to as "flood-deposits" and is further characterized by grain size as "coarse-grained" or "fine-grained" flood deposits A veneer of Willamette River alluvium derived from Willamette Valley sources to the south as well as reworked flood deposit sediments mantle the topography the Willamette has incised into the last flood-created surface Since at least the late 1800's human activity has extensively altered the Willamette River shore and created unique and variable shallow soil conditions in the Portland area, including the project area specifically

3 2 SURFACE CONDITIONS

The project site is located in Waterfront Park just south of the Burnside Bridge The surface conditions in the vicinity of the proposed PSM cover structure include landscape grasses at the north end and a

construction area related to the West Side CSO (WCSO) at the south end The construction area is fenced off and encloses the existing WCSO Ankeny Shaft and a vault structure

3 3 SUBSURFACE CONDITIONS

Subsurface conditions encountered by others in and around the project area are summarized in our original report The following paragraphs describe the subsurface conditions encountered in explorations we completed for pile design for the PSM cover structure and reference other nearby explorations as needed

We explored subsurface conditions at the PSM cover structure site by drilling three borings at the locations shown on Figure 2 The borings were completed to between 38 to 73 feet below the ground surface (bgs) to evaluate subsurface conditions for foundation analysis and design

A geologic cross-section through the proposed PSM cover structure location based on our borings is provided as Figure 3 Appendix A summarizes the exploration methods and presents the exploration logs Pacific Geotechnical completed for the pile design Appendix B presents an exploration log completed by others in the immediate vicinity of the proposed cover structure Results of our laboratory testing are indicated on the exploration logs in Appendix A and described in Appendix C

Subsurface conditions were found to be relatively uniform across the site In general, we encountered 1) silt, sand and gravel fill material, over 2) a thin layer of Fine-grained Flood Deposits, over 3) Coarse-grained Flood Deposits, over 4) Troutdale Formation gravels The conditions encountered in our explorations generally match the geologic mapping described in our original report and found by others in the area The following paragraphs summarize the above units

3 3 1 Fill

We encountered variable fill material in all of our borings The fill material generally included loose to dense sand and gravel and very soft to stiff silt We also encountered occasional crushed rock, asphalt, concrete, brick and wood debris The thickness of the fill material varied from about 27 feet in boring B-3 to about 60 feet in boring B-1, adjacent to the seawall structure Although the later thickness places fill below original grades and water levels, it is supported by a drawing for the seawall/cribbing structure that shows the base of the cribbing at an elevation of -30 5 feet (City of Portland datum), or about 63 feet bgs It appears the original soil was removed for construction of the crib wall and then backfilled during construction of the wall The moisture content of the fill varied between 8 to 39 percent The percent fines in silty fill was between 75 to 79 percent, based on two tests A dry density of 84 pounds per cubic foot (pcf) was measured on a single sample

3 3 2 Unconsolidated Silt and Sand (Alluvium/Fine-grained Flood Deposits)

We encountered a thin layer of silty fine sand in boring B-3 under the fill material We did not encounter this material in our other explorations, however, this unit is noted in boring C-1 completed by Fujitani Hilts & Associates (FHA) (PB 2002b, Appendix G) This material was medium dense with a moisture content between 38 and 43 percent, based on two tests

3 3 3 Unconsolidated Sand, Silt and Gravel (Alluvium/Coarse-grained Flood Deposits)

We encountered Coarse-grained Flood Deposits under the Fine-grained Flood Deposits in boring B-3 and below the fill material in boring B-2 We did not encounter this material in boring B-1, as the fill material extended to the top of the Troutdale Formation gravels (see below) This unit generally consisted of very dense gravel with variable amounts of sand and silt The moisture content was measured at 14 percent from one sample tested

3.3 4 Consolidated Sand and Gravel (Troutdale Formation)

We encountered gravels of the Troutdale Formation beneath the fill in boring B-1 at a depth of about 61 feet bgs This corresponds well with the design depth for the base of the cribbing structure supporting the adjacent seawall Boring C-1, completed by FHA just south of the proposed cover structure, encountered Troutdale Formation at a depth of 51 feet bgs This unit consists of very dense sand and gravel in a silt matrix

3 4 GROUNDWATER

Groundwater could not be determined in our explorations due to the drilling method used

Based on previous subsurface explorations completed by others, groundwater levels for the project area are commonly between 18 and 23 feet bgs This depth usually corresponds to an elevation within several feet of the mean river level, suggesting that this groundwater is connected to, and will reflect changes in, the water level of the Willamette River Our experience and the groundwater levels interpreted by others (PB, 2002a) suggest that the groundwater levels will tend to be a few feet higher than the river level

4 SEISMIC SETTING/EVALUATION

4 1 DESIGN EARTHQUAKES

Seismicity in the Willamette Valley is primarily driven by the Cascadia Subduction Zone (CSZ), which is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate The interaction of these two plates results in three potential seismic sources affecting the Willamette Valley area, shallow crustal earthquakes within the overriding North American Plate, interplate earthquakes which occur along the boundary between the Juan de Fuca and North American plates, and intraplate earthquakes which occur within the subducting Juan de Fuca plate Although capable of generating significant earthquakes, intraplate earthquakes, such as the February 28, 2001 Nisqually earthquake (M_w = 6 8) in Washington State were evaluated, but found not to control design for this area so are not further considered Crustal and interplate earthquakes are further discussed below

Figure 4 shows the size and epicentral locations of earthquakes that have occurred from 1841 through 2002 and the location of late Quaternary and Holocene age faults, within a 50-mile radius of the site

According to the USGS, the shallow crustal earthquake source contributes about 90% to the probabilistic ground motion and the interplate source zone contributes about 10% to the probabilistic ground motion for the range of periods of concern Therefore, additional data representative of each of the two source zones were gathered for use in assessing the site seismic response Again, the intraplate earthquake is not considered capable of controlling design in this area so was not considered Discussion of the shallow crustal and interplate sources are provided below

4 1 1 Shallow Crustal Earthquakes

A significant earthquake could occur on a local fault very near the site within the design life of the facility Such event would cause ground shaking at the site that could be more intense than the CSZ interplate event, though the duration would be shorter

Several large, northwest-trending faults are mapped within one mile of the project area, including the Portland Hills Fault and the East Bank Fault (Beeson and others, 1991) Random crustal faults may also occur on yet unmapped faults as well The Portland Hills Fault Zone is considered potentially active (Geomatrix, 1995 and USGS, 2007) and believed capable of generating a magnitude 6 9 or greater event In addition to these mapped faults the geotechnical data report for the West Side CSO project (Parsons

Brinckerhoff, 2002b) reports a disparity in geologic unit elevations from borings located across the Willamette River from each other that infers a displacement of the CRB and SRM units as much as 130 vertical feet over this distance This suggests a previously unmapped fault along the axis of the Willamette that may have undergone as much as 0 04mm/yr displacement over the past 1 million years Alternately, this could be from past differential erosion by the river, but this could not be resolved for this study

4 1 2 CSZ Interplate Earthquakes

The CSZ interplate source zone generally extends along the coast from northern California to British Columbia and is characterized by researchers as being capable of generating earthquakes of magnitude 8 to magnitude 9 Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 500 years, which is substantially less frequent than shallow crustal or CSZ intraplate earthquakes The most recent CSZ event is believed to have occurred in the year 1700, and paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3500 years There is no instrumentally recorded seismicity of large interplate earthquakes for the CSZ The inferred seismogenic portion of the CSZ plate interface is roughly 60 miles (100 kilometers) west of the site

4 2 SEISMIC HAZARDS

Seismic hazards for this area have been mapped by the Department of Geology and Mineral Industries (DOGAMI) in the *Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Quadrangles, Oregon and Clark County, Washington* (Mabey and others, 1993) This map series includes hazard maps for ground amplification, liquefaction, and lateral spreading/slope instability A Relative Earthquake Hazard Map, which combines the above three earthquake hazards is also included Based on this map, the project site is within Zone C, defined as "Low to intermediate hazard" where relative earthquake hazard is mapped between Zone A (highest hazard) to Zone D (lowest hazard)

4 2 1 Fault Surface Rupture

The closest local fault is the northwest-southeast trending Portland Hills fault shown by the United States Geologic Survey (USGS, 2007) less than 0 6 miles (1 kilometer) west of the site Although it is considered potentially active, displacement of Holocene materials has not been observed Explorations we completed and extensive explorations for the WCSO did not find evidence of Holocene displacement Although the fault is capable of generating significant shaking at the site, from the lack of Holocene displacement here and elsewhere on the Portland Hills fault, it is our opinion that the possibility of surface fault rupture and related displacement at this site is low

4 2 2 Ground Motion Amplification

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long period ground motions such as those associated with subduction zone earthquakes, whereas areas underlain by shallow soil profiles are not likely to amplify seismic waves

The ground motion amplification map shows the general area of the site to be in a hazard zone with a 1 to 1 4 amplification factor These are the lowest and second-lowest amplification factor zones based on a six zone rating system An amplification factor of 2 5 or greater is the highest relative hazard and 1 the lowest

Ground motions are considered specifically in Section 4 4

4 2 3 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking

The thickness of liquefiable sediment (liquefaction hazard) is ranked in the highest hazard level, at greater than 9 meters (about 30 feet) of liquefiable soil potentially present in the area

4 2 4 Lateral Spread

Lateral spread occurs when large blocks of ground are displaced down gentle slopes or towards stream channels as a result of liquefaction of subsurface soil during an earthquake Man-made seawalls and loose, saturated sandy fills can increase site vulnerability to this hazard, and the presence of these features is reflected in the classification of the bulk of the project area in the highest (estimated lateral ground displacement in excess of 1 2 meters, about 4 feet) and second highest (displacement of 0 9 to 4 meters, about 3 to 4 feet) hazard zones (Mabey and others, 1993)

Lateral spreading estimates by others have been reported to vary between about 2 inches to as much as 2 feet The subsurface conditions encountered for this report concur with those findings

4 2 5 Earthquake Induced Landsliding

Existing mapping (Mabey and others, 1993) places the site in the lowest category of landslide hazard Based on the flat site slopes, we concur that the landsliding hazard is low at the PSM cover structure

4 2 6 Seiche and Tsunami

The site is well away from tsunami inundation zones and away from large bodies of water that may develop seiches Seiche and tsunamis are not considered a hazard at this site

4.2 7 Settlement

Settlement due to earthquakes is most prevalent in relatively deep deposits of loose, dry, clean sand In our original report we stated that vertical settlements estimated by others at nearby locations have ranged from less than an inch to up to 5 inches Based on our explorations, these values still appear appropriate

4 3 BUILDING CODE PARAMETERS

We understand that the structure will be designed and constructed in accordance with the 2006 IBC and the 2007 OSSC The parameters provided in Table 1 are appropriate for code level seismic design Results from our site specific seismic study are discussed in Section 4 4

Table 1 Seismic Design Parameters (2006 IBC)

Seismic Design Parameters (2006 IBC)	
Building Use Category	III
Seismic Use Group	II
Seismic Importance Factor, I_E	1 25
Site Class	D
Spectral Response Acceleration S_s	0 98 g
Spectral Response Acceleration S_1	0 34 g
Site Coefficient, F_a	1 1
Site Coefficient, F_v	1 7
Spectral Response Acceleration (Short Period), S_{DS}	0 73 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0 39 g
Seismic Design Category	D

Note
[1] Site soil is classified as Site Class F based on the liquefaction hazard Site Coefficients are based on Site Class D per the 2006 International Building Code provided the period of the structure is less than 0 5 seconds

Based on the 2002 USGS National Seismic Hazard Mapping Project, the expected peak bedrock acceleration having a 10 percent probability of exceedance in 50 years (475 year return period) is 0 19g The expected peak bedrock acceleration having a 2 percent probability of exceedance in 50 years (2,475 year return period) is 0 41g These values represent the peak acceleration on bedrock beneath the site and do not account for any ground motion amplification due to site-specific effects

4 4 SITE SPECIFIC RESPONSE ANALYSIS

4 4 1 Ground Motion Selection

Earthquake acceleration time histories used to evaluate the site response were selected based on the fault mechanism of the earthquake source zones, the soil/rock profile type at the site where the earthquake time history was recorded, the distance from the epicenter to the recording site, the PGA of the record, the predominate period, and the earthquake magnitude and energy content

Based on the USGS 2002 Interactive Deaggragations web site, the local shallow crustal event and the CSZ Interplate event contribute approximately 90 and 10 percent, respectively, to the hazard at the site Therefore earthquake time histories were selected according to ASCE 7-05 based on the above hazards Table 2 presents the earthquake acceleration time histories used in our analysis

Table 2 Selected Earthquake Acceleration Time Histories

Earthquake, Year	Recording Station	Recorded Magnitude	Recorded Peak Horizontal Acceleration (g)	Fault Mechanism
Whittier Narrows, 1987	Mt Wilson	5 3	0 16	Crustal
Whittier Narrows, 1987	LA - Obregon Park (360 degree component)	5 3	0 40	Crustal
N Palm Springs, 1986	North Palm Springs (NPS 300)	6 2	0 23	Crustal
Imperial Valley, 1979	Superstition Mountain	6 5	0 20	Crustal
Nahanni, Canada, 1985	Site 2 (330 degree)	6 8	0 32	Crustal
Michoacan, Mexico, 1985	La Union (N90W)	8 1	0 17	Interface
Valparaiso, Chile, 1985	NA, 070 degree	7 5	0 15	Interface

4 4 2 Ground Motion Scaling

The above input time histories were scaled using the computer program Shake2000 Version 3 5 in accordance with ASCE 7-05 Each selected time history was scaled to the maximum considered earthquake (MCE) rock response spectrum assuming Site Class A such that "its response spectrum is, on average, approximately at the level of the MCE rock response spectrum over the period range of significance to structural response" (ASCE, 7-05) Based on a fundamental period for the structure of approximately 0 26 seconds, the period range of significance is approximately 0 05 to 1 3 seconds

In addition, the scaling factor utilized for each time history was varied to evaluate the effect on the output Based on our analysis, there was little change to the final site specific spectral acceleration by varying the scaling factor within a reasonable range The individual scaled time histories are shown in Figure 5

4 4 3 Soil Model

The input soil model used in our analysis is based on the findings of our subsurface exploration program A detailed description of site subsurface conditions is provided in Section 3 3 Shear wave velocities could not be directly measured as the subsurface contains various layers of gravel that could not be penetrated with a cone penetrometer The engineering behavior of the site soils was modeled using estimated shear wave velocities and unit weights for the anticipated soil profile along with damping and shear modulus reduction relationships typical for the soil types underlying the site

The individual soil parameters were varied within the expected range of values to estimate the effect on the output Further, due to the variable soils encountered in the fill material, several damping and shear modulus reduction relationships suitable for the encountered soils were utilized to evaluate their effect on the final results Based on our analysis, varying the soil parameters within the expected range of values had little effect on the output, however, utilizing different damping and shear modulus reduction relationships for the upper fill material had a relatively significant effect on the output Therefore, due to the variability of the soil in the upper forty feet, we utilized depth, rather than soil type, to choose the damping and shear modulus reduction relationships for this material, as recommended by Dr Jonathan D Bray (ASCE, 2007) This method resulted in spectral acceleration values within the middle of the range encountered using other relationships

Table 3 provides a summary of the soil model used in our analysis

Table 3 Soil Model

Depth Interval (feet)	Subsurface Unit	Shear Wave Velocity Range (feet per second)
0 to 40	Fill	300 to 1,000
40 to 155	Very Dense Sand and Gravel	1,000 to 1,400
> 155 feet	Basalt Bedrock	5,000

4 4 4 Design Response Spectra

As required by ASCE 7-05, Section 21 1 3 the site specific response on soil is calculated by multiplying the target rock response by the ratio of response spectra (RRS) calculated from the response analysis Figure 6 provides the ratio of response spectra versus period for each of the input time histories Figure 7 provides the mean RRS versus period for the crustal and CSZ interface events Figure 8 provides a plot of the spectral acceleration versus period for the proposed PSM cover structure These spectra are scaled to two-thirds of the maximum computed as prescribed by the IBC Additionally, the spectral acceleration at any period must be equal to or greater than the response spectrum determined as described in Section 1615 1 4 of the 2006 IBC

5 FOUNDATION DESIGN

We completed a supplemental geotechnical report for foundation support recommendations, dated February 18, 2008 In that report we recommended that the PSM cover structure be supported on driven or drilled steel piles bearing in the native gravels, or on an improved subgrade We understand that the PSM cover structure will be supported on a combination of drilled and driven piles Our fill recommendations are provided in the February 18 report

6 REFERENCES

ASCE 7-05, *Minimum Design Loads for Buildings and Other Structures,* American Society of Civil Engineers, 2006

ASCE, *Evaluation and Mitigation of Geotechnical Seismic Hazards – 3-Day Technical Short Course,* Olympia, Washington, July, 2007

Geotechnical Baseline Report, West Side CSO Tunnel, Shafts, Pump Station, & Pipelines, prepared by Parsons Brinkerhoff for the City of Portland Bureau of Environmental Services, dated May 28, 2002, (2002a)

Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon and Clark County, Washington, Beeson and others, 1991

Final Geotechnical Data Report, West Side CSO Tunnel, Shafts, Pump Station, & Pipelines, prepared by Parsons Brinkerhoff for the City of Portland Bureau of Environmental Services, dated May 28, 2002, (2002b and appendices)

Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Quadrangles, Oregon and Clark County, Washington, Mabey and others, 1993

International Code Council, 2006, 2006 International Building Code

OSSC, 2007 Oregon Structural Specialty Code, accessed March, 2008 from OSSC web site http //www2 iccsafe org/states/oregon/07_Structural/Building07_Frameset htm

Geomatrix Consultants, 1995, *Seismic design mapping, State of Oregon Final Report*, prepared for Oregon Department of Transportation, 252 p

U S Geologic Survey, 2002, National Seismic Hazards Mapping Project, Seismic Hazard Curves, Response Parameters and Design Parameters, Java application, version 5 0 8

U S Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed March, 2008, from USGS web site http //earthquake usgs gov/regional/qfaults/

7 LIMITATIONS

We have prepared this report for use by Walker Macy, the City of Portland Bureau of Parks and the design team for construction of the PSM cover structure in Waterfront Park in Portland, Oregon Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared No warranty, express or implied, should be understood

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document The original document is stored by Pacific Geotechnical and will serve as the official document of record

8 CLOSING

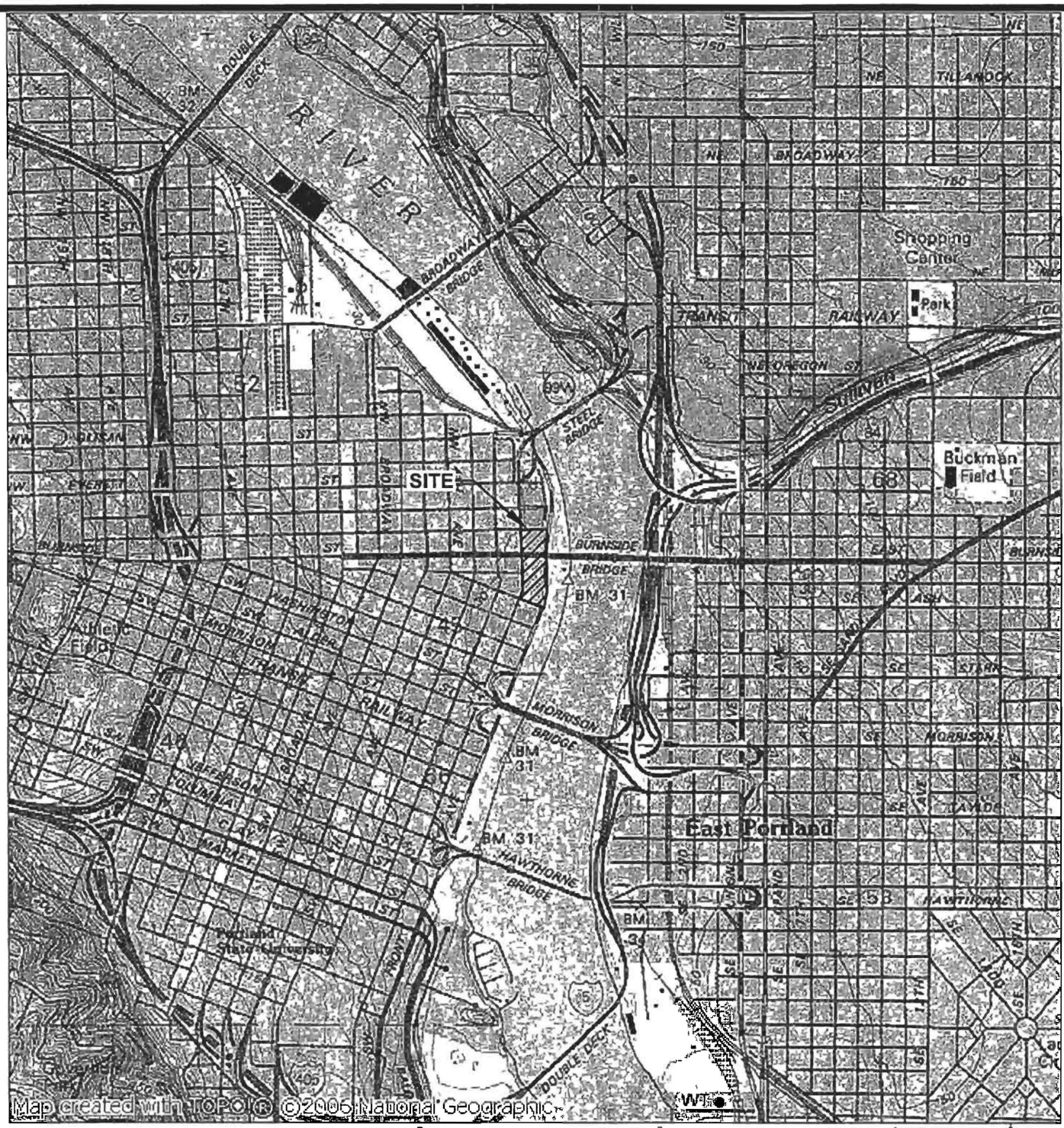
We appreciate the opportunity to submit this report to you Please contact us if you have any questions or need additional information

Sincerely,

Greg A Landau, PE
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
77490PE
OREGON
NOV. 8, 2005
GREG ALAN LANDAU
EXPIRES: 12 31 08

Attachments
Document ID 1050-001-00R3revised doc



NATIONAL GEOGRAPHIC

0 .5 1 MILES

1000 0 1000 2000 3000 4000 5000 FEET

1 .5 0 KILOMETERS 1

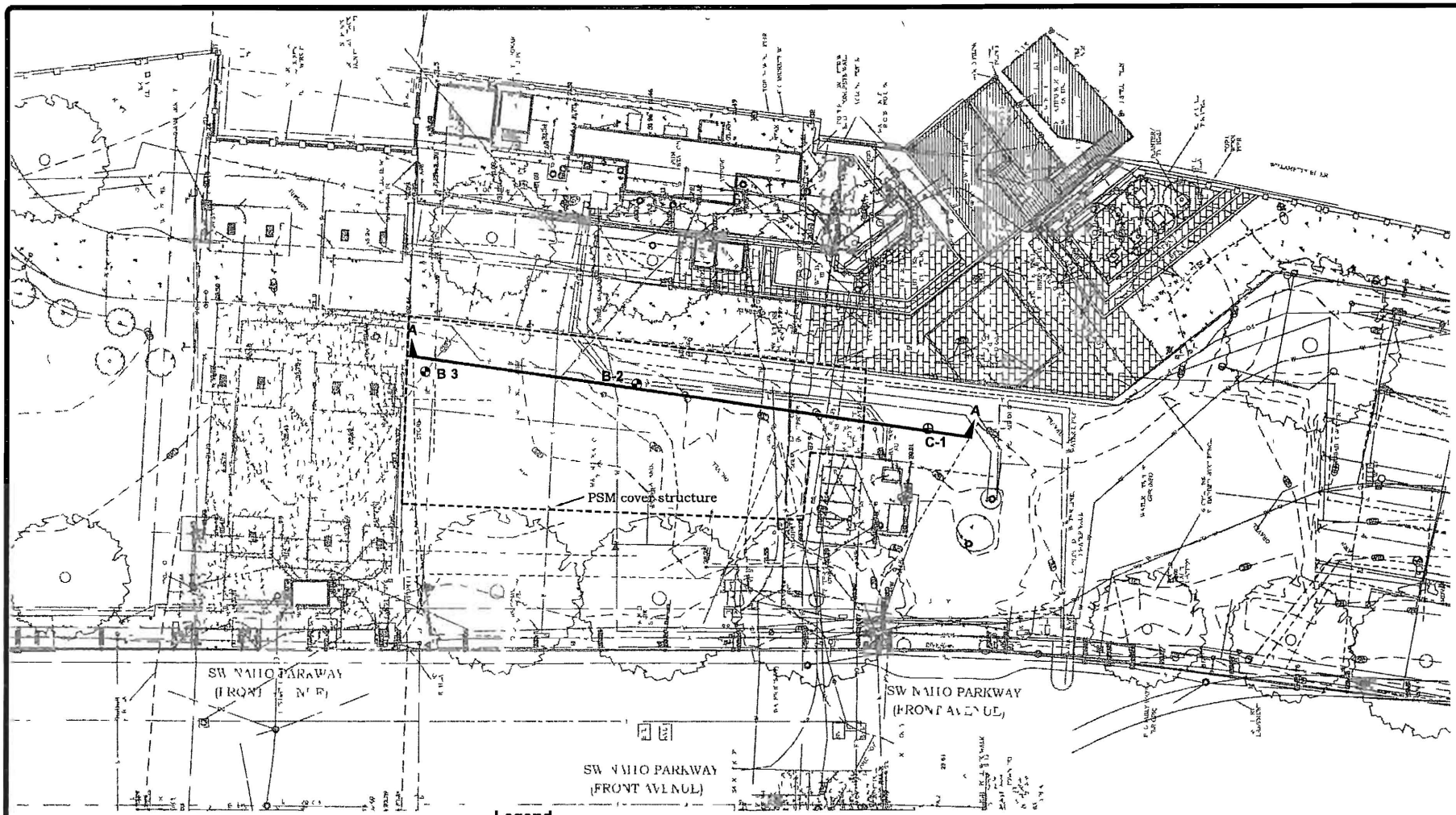
1000 0 METERS 1000

TN MN

16½

02/07/08

DWG NAME	Vicinity Map		
DESCRIPTION	Portland Waterfront Park Saturday Market Relocation		
JOB NO	1050-001-00	DATE	02/07/08
FILE NAME	105000100F1 dwg	FIGURE	1



Legend

B-1 ⊗ Explorations by Pacific Geotechnical

C 1 ⊕ Exploration by Others (See Appendix C)

Approximate Cross Section Location

Approximate Location proposed structure

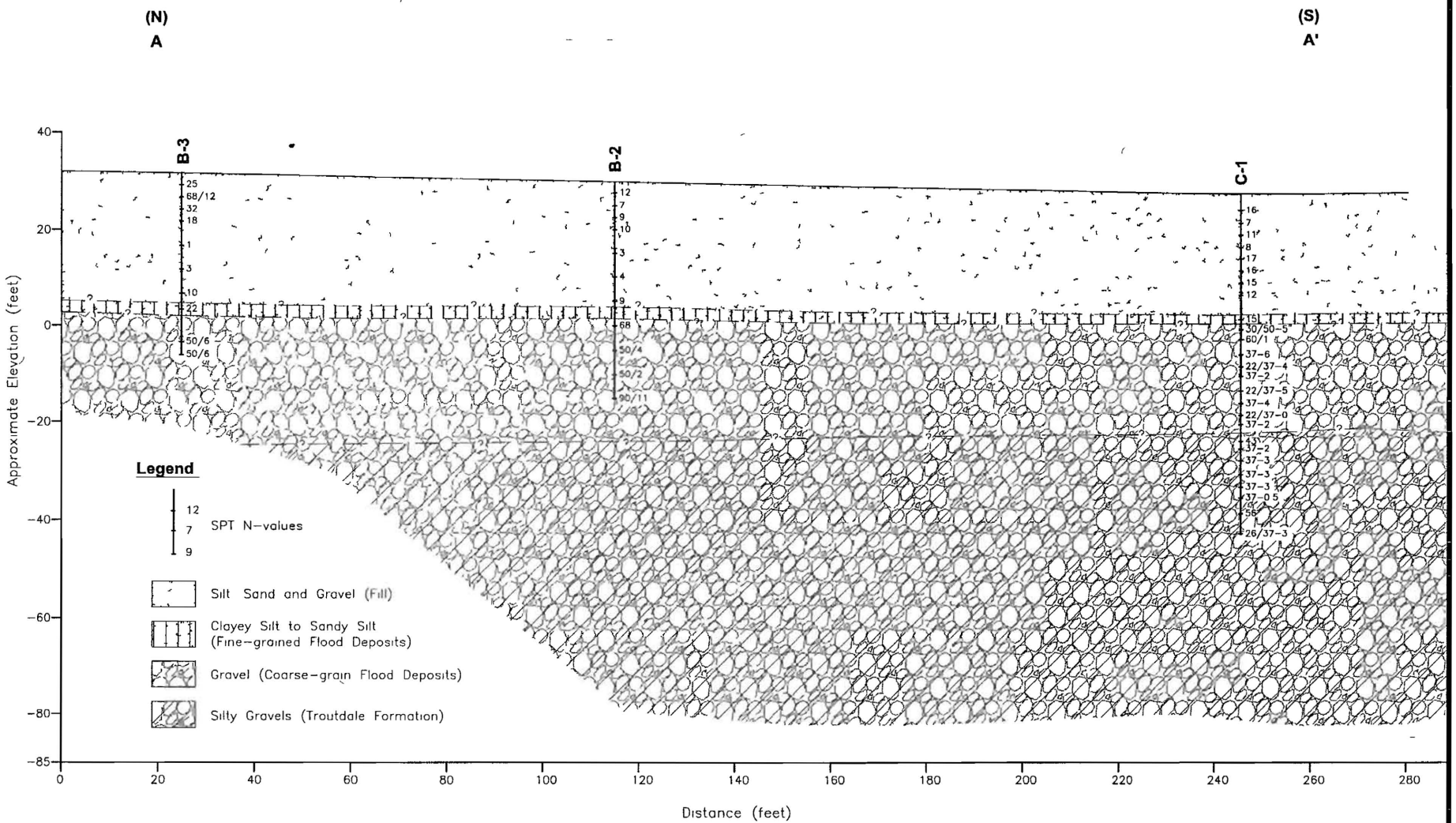
N

40 0 40

APPROXIMATE SCALE IN FEET

DWG NAME	Site and Exploration Plan
DESCRIPTION	Portland Waterfront Park Saturday Market Relocation
JOB NO 1050-001-00	DATE 03/04/08
FILE NAME 105000100F2 dwg	FIGURE 2

All Features shown are approximate
Base drawing was an AutoCAD file provided by Walker/Macy

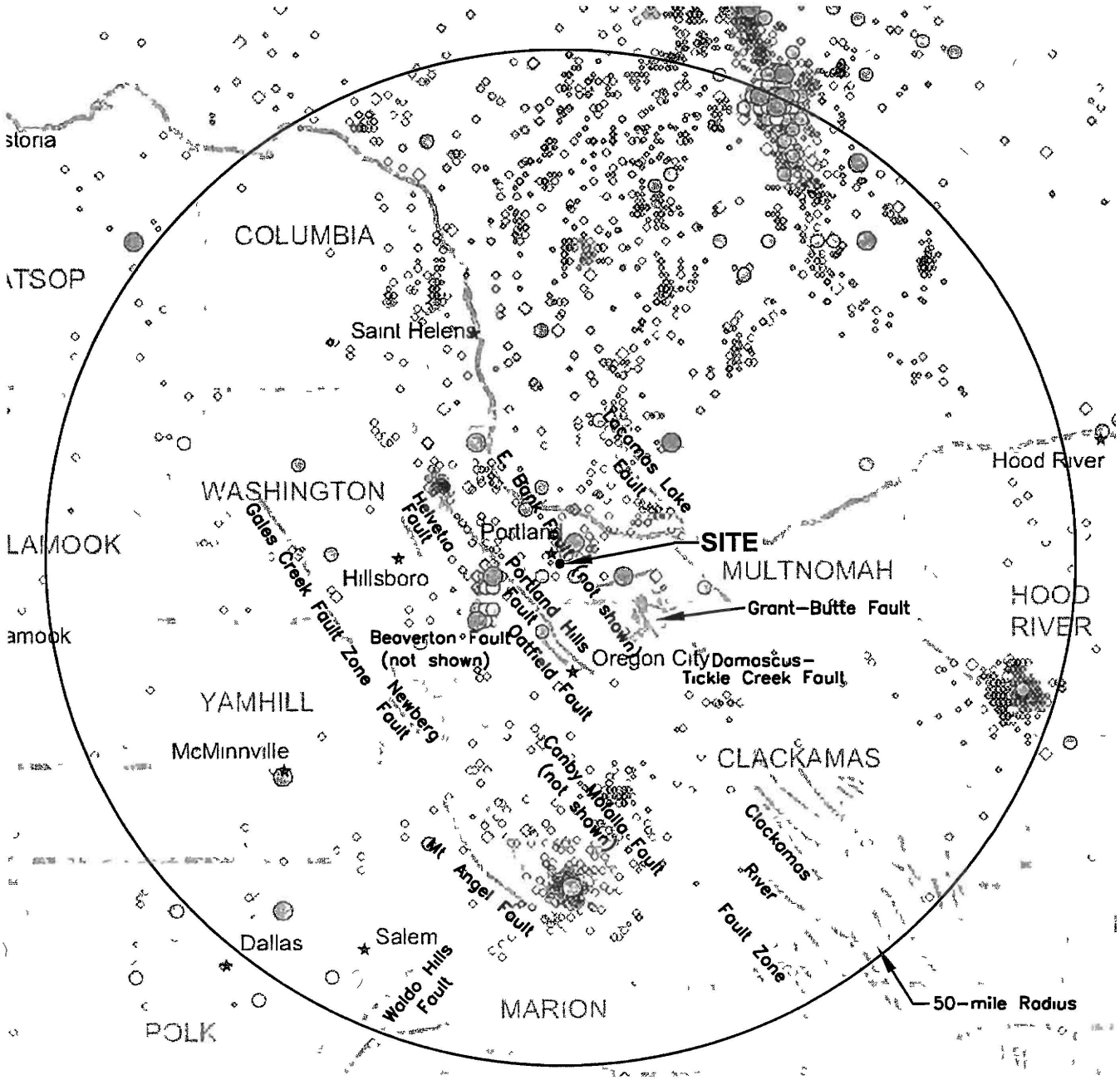


Notes
This figure is a general representation of soil conditions for informational purposes only and should not be used for bidding purposes Refer to attached Geotechnical Report and exploration logs for more detailed information

All locations and dimmensions are opproximate

Base drawing was a sketch provided by Pacific Geotechnical staff

DWG NAME	Cross Section A-A		
DESCRIPTION	Portland Waterfront Park Portland Oregon		
JOB NO	1050-001-00	DATE	03/04/08
FILE NAME	105000100F3 dwg	FIGURE	3



Reference Base map from DOGAMI Open File Report 03–02 (Niewendorp and Neuhaus 2003)

DWG NAME	Fault and Seismicity Map		
DESCRIPTION	Portland Waterfront Park Saturday Market Relocation		
JOB NO	1050–001–00	DATE	3/4/08
FILE NAME	105000100F5 dwg	FIGURE	4

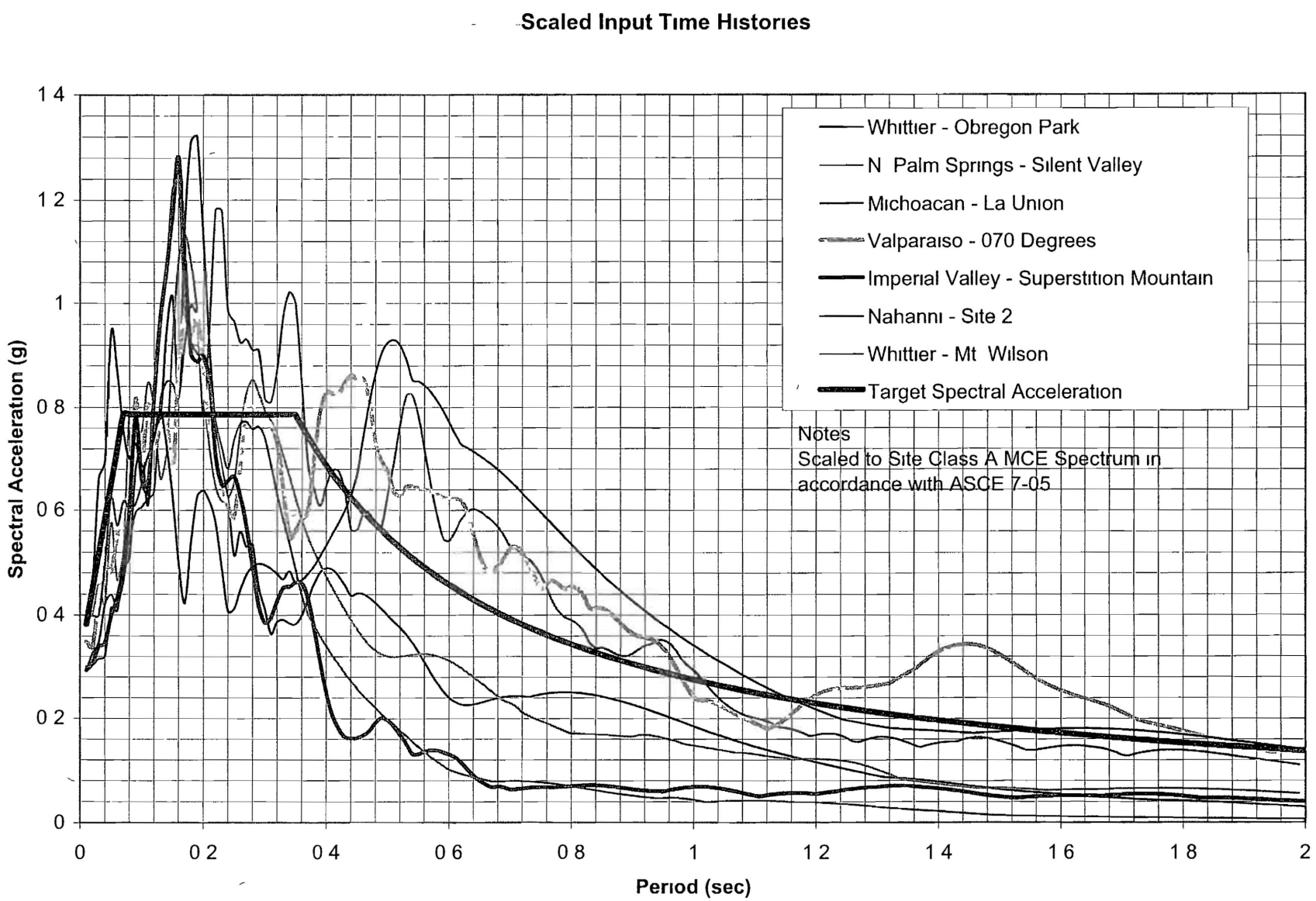

Scaled Input Time Histories
Spectral Acceleration (g)
Period (sec)
0
0 2
0 4
0 6
0 8
1
1 2
1 4
1 6
1 8
2
Whittier - Obregon Park
N Palm Springs - Silent Valley
Michoacan - La Union
Valparaiso - 070 Degrees
Imperial Valley - Superstition Mountain
Nahanni - Site 2
Whittier - Mt Wilson
Target Spectral Acceleration
Notes
Scaled to Site Class A MCE Spectrum in accordance with ASCE 7-05

Figure 5

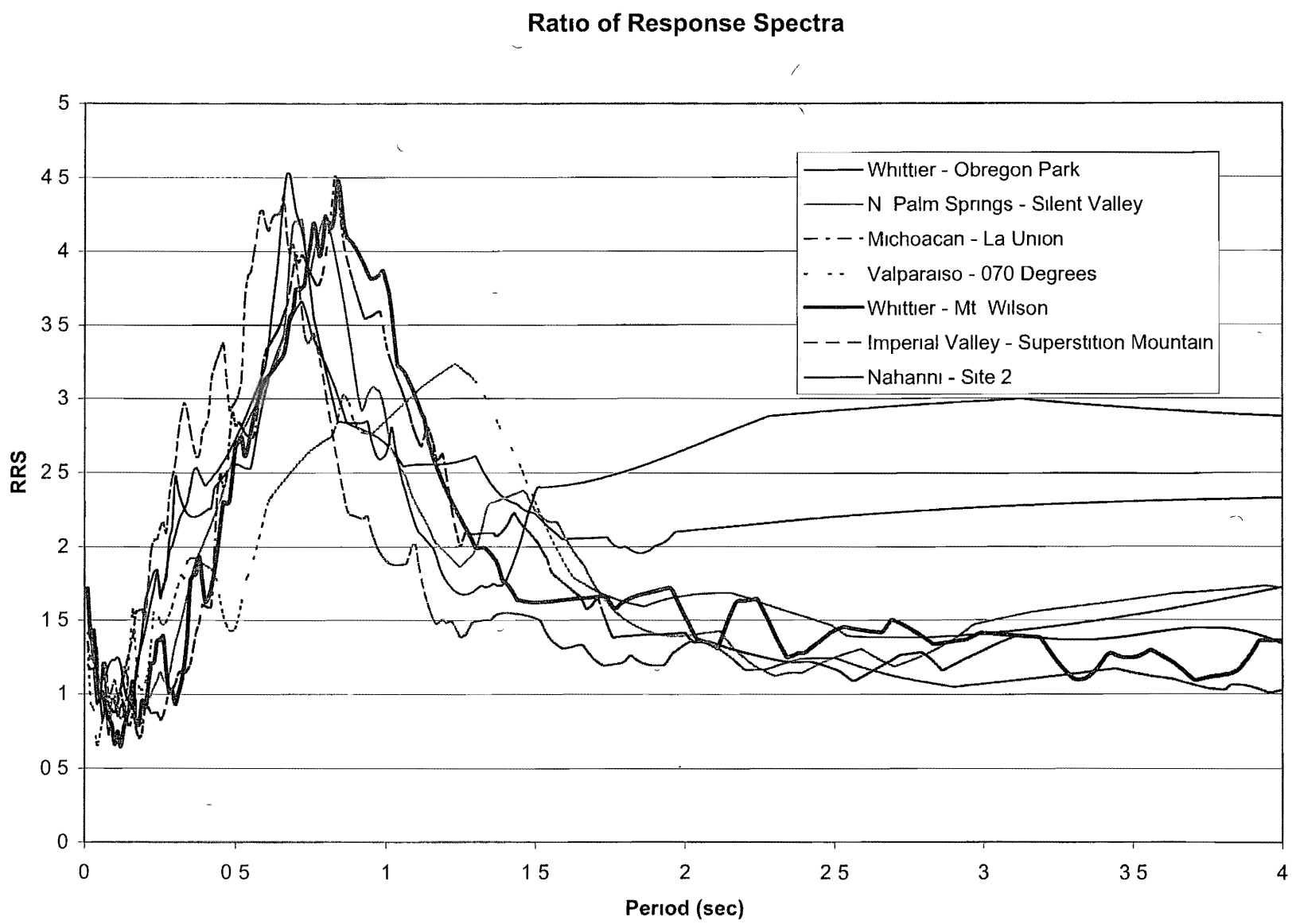

Ratio of Response Spectra
RRS
Period (sec)
Whittier - Obregon Park
N Palm Springs - Silent Valley
Michoacan - La Union
Valparaiso - 070 Degrees
Whittier - Mt Wilson
Imperial Valley - Superstition Mountain
Nahanni - Site 2
0
0 5
1
1 5
2
2 5
3
3 5
4
4 5
5

Figure 6

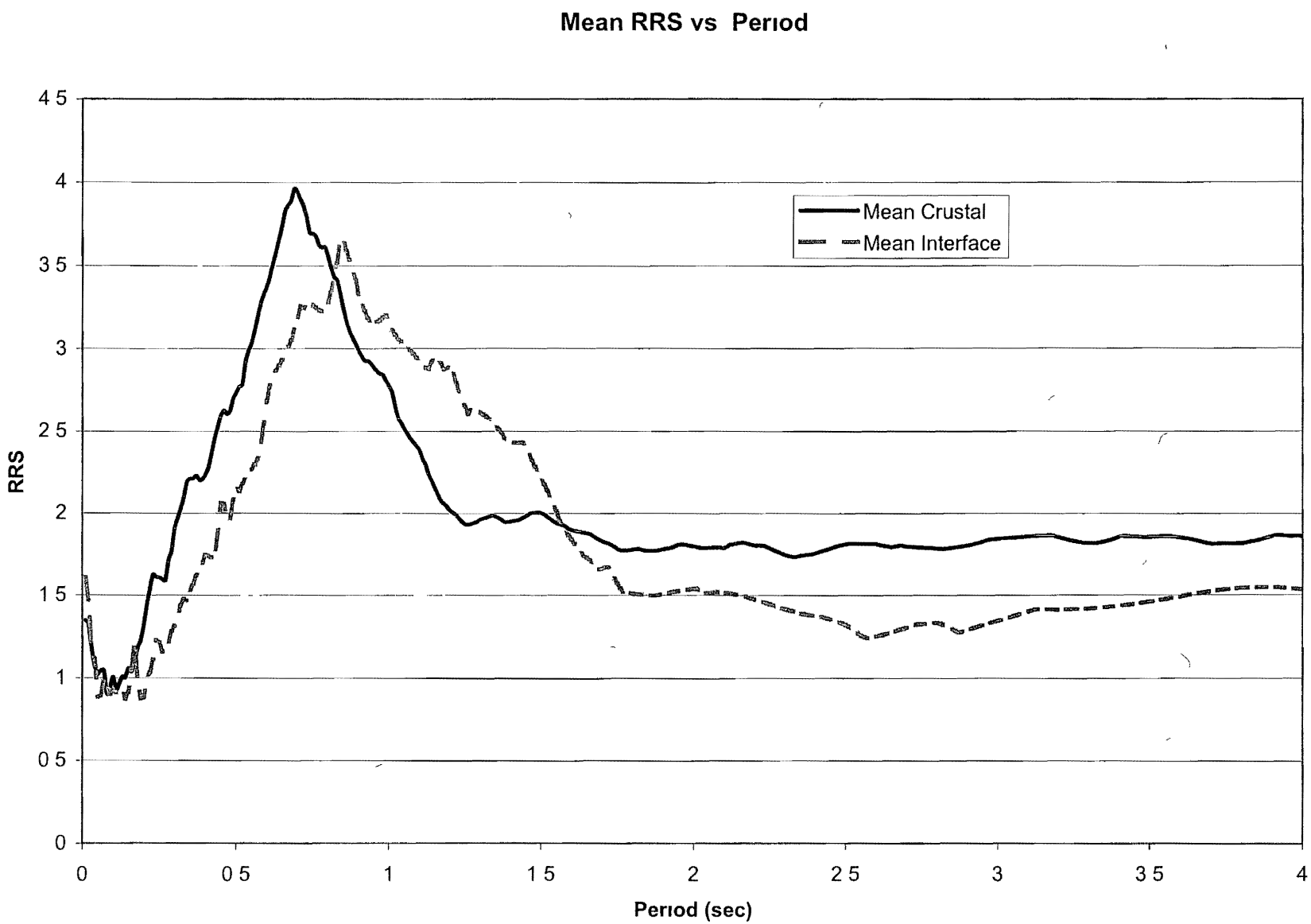

Mean RRS vs Period
Mean Crustal
Mean Interface
RRS
Period (sec)
4 5
4
3 5
3
2 5
2
1 5
1
0 5
0
0
0 5
1
1 5
2
2 5
3
3 5
4

Figure 7

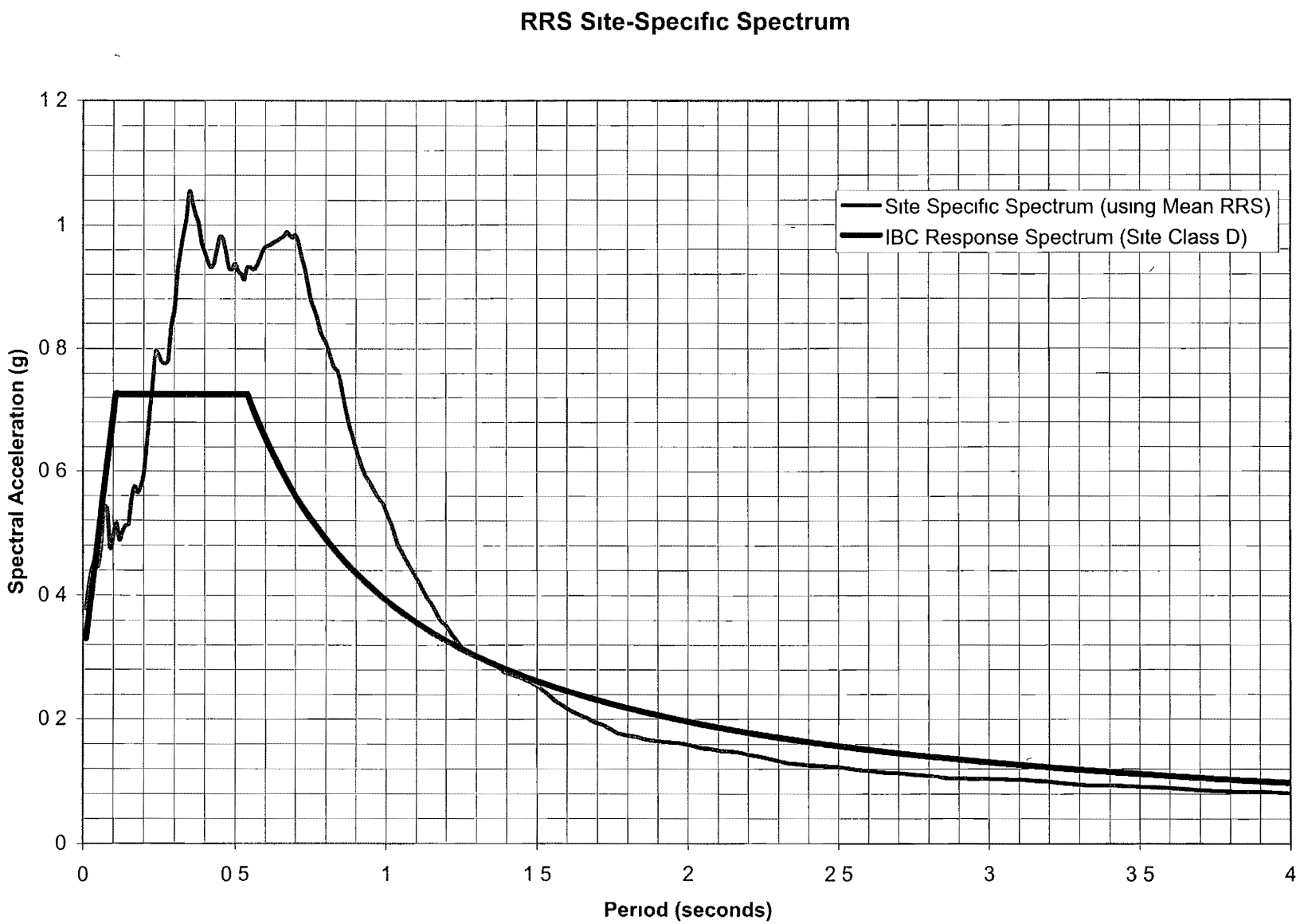

RRS Site-Specific Spectrum
Site Specific Spectrum (using Mean RRS)
IBC Response Spectrum (Site Class D)
Spectral Acceleration (g)
Period (seconds)
1 2
1
0 8
0 6
0 4
0 2
0
0
0 5
1
1 5
2
2 5
3
3 5
4

Figure 8

APPENDIX A
SUBSURFACE EXPLORATIONS BY PACIFIC GEOTECHNICAL

APPENDIX A
SUBSURFACE EXPLORATIONS

We evaluated subsurface soil and groundwater conditions by completing three borings at the approximate locations shown on Figure 2 of the report text The boring locations were approximately located by pacing from existing site facilities Exploration locations should be considered accurate only to the degree implied by the methods used

The borings were drilled using a truck-mounted drill rig provided by Subsurface Technologies of Banks, Oregon Soil samples were obtained from the explorations using one of the following methods

1. Standard Penetration Tests (SPT) were completed in general conformance with ASTM Test Method D1586, "Standard Method for Penetration Test and Split-Barrel Sampling of Soils" The sampler was driven with a 140-pound auto-trip hammer falling 30 inches The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the boring logs Disturbed samples were obtained from the split barrel for subsequent classification and index testing
2. Relatively undisturbed samples were obtained using a Dames & Moore Type-U sampler The sampler was driven using a 140-pound hammer falling 30 inches, just as with the SPT samples The penetration resistance recorded on the boring logs have been reduced by 50 percent for general correlation with the SPT blow counts Samples retained from the split barrel consist of up to six, 1-inch-high by 2 48-inch-diameter brass rings Disturbed rings were generally not retained

Materials encountered in the explorations were classified in the field in general accordance with ASTM Standard Practice D2488, "Standard Practice for the Classification of Soils (Visual-Manual Procedure)" Soil classifications and sampling intervals are shown in the exploration logs in this appendix

The field explorations were coordinated by an engineering geologist from our staff, who located the explorations, classified the various soil units encountered, obtained representative soil samples for geotechnical testing, observed and recorded groundwater conditions, and maintained a detailed log of each exploration Exploration logs are included in this appendix Results of the laboratory testing are indicated on the exploration logs and are discussed in Appendix C

KEY TO EXPLORATION LOGS

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
1419 Washington Street, Suite 101
Oregon City, Oregon 9704

SOILS CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS GRAVEL SAND MIXTURES
			GP	POORLY GRADED GRAVELS GRAVEL SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS GRAVELS SAND SILT MIXTURES
			GC	CLAYEY GRAVELS GRAVEL SAND CLAY MIXTURES
MORE THAN 50% RETAINED ON NO 200 SIEVE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)	SW	WELL GRADED SANDS GRAVELLY SANDS
			SP	POORLY-GRADED SANDS GRAVELLY SANDS
	MORE THAN 50% OF COARSE FRACTION PASSING NO 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SITLY SANDS SAND SILT MIXTURES
			SC	CLAYEY SANDS SAND CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY GRAVELLY CLAYS SANDY CLAYS SILTY CLAYS LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% PASSING NO 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS MICACEOUS OR DIATOMACEOUS SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT HUMUS SWAMP SOILS WITH HIGH ORGANIC CONTENTS

Note Multiple symbols are used to indicate borderline or dual soil classifications

SYMBOLS LETTER	DESCRIPTIONS
CC	CEMENT CONCRETE
AC	ASPHALT CONCRETE
TS	TOPSOIL/SOD FORREST DUFF

Stratigraphic Contact

——— Distinct contact between soil strata or geologic units

----- Gradual or approximate change between soil strata or geological units

Moisture Modifiers

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below the water table

Seepage Modifiers

None

Slow - < 1 gpm

Moderate - 1- 3 gpm

Heavy - > 3 gpm

Caving Modifiers

None

Minor - isolated

Moderate - frequent

Severe - general

Minor Constituents

Trace < 5% (silt/clay)

Occasional < 15% (sand/gravel)

With 5-15% (silt/clay) in sand or gravel

15-30% (sand/gravel) in silt or clay

Sampler Symbol Descriptions

2 4-inch I D split barrel

Standard Penetration Test (SPT)

Shelby tube

Piston

G Bulk or grab

Laboratory / Field Tests

%F Percent fines

AL Atterberg Limits

CP Laboratory compaction test

CS Consolidation test

DS Direct shear

HA Hydrometer analysis

Laboratory / Field Tests

DD Dry density

OC Organic content

PP Pocket penetrometer

SA Sieve analysis

TV Torvane shear

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586 See exploration log for hammer weight and drop

A "P" indicates sampler pushed using the weight of the drill rig

(2 4-inch) sampler N approximately corrected to equivalent SPT N by 50% reduction in N - modified California

Note Refer to the report text and exploration logs for a proper understanding of subsurface conditions Descriptions on the logs apply only at the exploration locations at the time the explorations were made The logs are not warranted to be representative of the subsurface conditions at other locations or times

Pacific Geotechnical, LLC

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
				20 40 60 80		10 30 50 70 90	
					SW Brown gravelly SAND, fine to coarse sand, fine rounded gravel (moist, loose) [Fill]		
	10	1	SPT	9		18	
5	8	2	SPT	7	Grades to with layers of SILTY SAND with gravel	18	
	5	3	SPT	4	Grades with trace silt		
10	6	4	SPT	4		16	
15	4	5	SPT	3	Grades with trace to some gravel (very loose)		
20	2	6	SPT	32	GP Brown POORLY GRADED GRAVEL with trace fine sand Rounded gravel to 1 maximum dimension (moist to wet medium dense to dense)[Fill]	8	Interpreted as fill based on depth
25	3	7	SPT	30		20	
30							

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	5	8	SPT	10	Grades to loose		Lost circulation and mud Sampler possibly driven through slough May be denser than blow counts suggest
35	6	9	SPT	10			
40	10	10	SPT	24	Grades with fine to coarse sand (medium dense to dense)	13	
							Regain circulation at 43ft
45	0	11	SPT	47			No recovery
50	9	12	SPT	21	GP-SP Sandy GRAVEL to gravelly SAND Fine to medium sand Fine rounded gravel (wet, dense)[Fill]	15	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
55	6	13	SPT	28	SM Gray SAND with gravel and silt Fine to medium sand Fine rounded gravels (wet, medium dense to dense) [Fill]	20	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
60							

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
1419 Washington Street Suite 101
Oregon City Oregon 97045

LOG OF BORING NO. B-1

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	1	14	SPT	42		SP Brown SAND with occasional fine rounded gravels (wet, very dense) [Troutdale Formation]		Hard drilling at 61ft
65	4	15	SPT	100				50 blows for 5
70	15	16	SPT	56		Becomes dark brown with some yellow		
	6	17	SPT	100		GP Brown POORLY-GRADED GRAVEL with fine to medium sand (wet, very dense)		50 blows for 4 5
75						Boring completed at 73ft bgs Groundwater not determined due to drilling method used		
80								
85								
90								

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

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LOG OF BORING NO. B-2

Project Name Ankeny Plaza

Location Waterfront Park, Portland OR

Number 1050 001-00

Depth feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
				20 40 60 80		10 30 50 70 90	
					GP-GM Dark gray POORLY-GRADED GRAVEL with sand and silt to SILTY GRAVEL Coarse angular basalt crushed rock (wet, medium dense)[Fill]		Boring relocated 10 feet west due to hitting abandoned sewer pipe
	10	1	SPT	12		15	
5	4	2	SPT	7	Grades to loose		
	10	3	SPT	9	SP-SM Brown to gray-brown POORLY GRADED SAND with silt to SILTY SAND with gravel and wood debris (moist, loose)[Fill]	20	
10	12	4	SPT	10	SM Dark gray-brown SILTY SAND with gravel (wet, loose to medium dense) [Fill]		
15	4	5	D+M	4	ML Gray brown and mottled brown SILT with fine sand Low to medium plasticity (moist, soft)[Fill]	37	DD= 84 pcf
20	16	6	SPT	4	Grades to dark gray with medium plastcity and occasional bricks	34	%F= 75
25	12	7	SPT	9	Grades to SILT with fine sand to fine sandy SILT (stiff)[Fill or Fine grained Flood Deposits?]	37	
30						24	

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
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Oregon City Oregon 97045

LOG OF BORING NO. B-2

Project Name Ankeny Plaza

Location Waterfront Park, Portland OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	14	8	SPT	68	GM Dark gray SILTY GRAVEL with trace coarse sand and with angular rock (wet, very dense)[Coarse-grained Flood Deposits]		Very slow drilling at 30ft to 35ft
35	4	9	SPT	100			50 blows for 4"
					Grades to dark gray-brown SILTY GRAVEL with fine to medium coarse sand and rounded basalt gravels to 1 in maximum dimension		
40	3	10	SPT	100			50 blows for 2
45	6	11	SPT	100		14	90 blows for 11"
					Boring completed at 46 ft bgs Groundwater not determined due to drilling method used		
50							
55							
60							

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

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LOG OF BORING NO. B-3

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
					SP Brown gravelly SAND to SAND with gravel Fine to medium sand Fine rounded gravel (moist, medium dense) [Fill]		
	13	1	SPT	25		13	
5	14	2	SPT	100	Grades with brick and concrete debris	20	50 blows for 6' due to concrete debris
	5	3	SPT	32	GM Gray SILTY GRAVEL with sand and occasional wood debris Fine to medium sand Fine to coarse rounded to subangular gravel (moist to wet, dense)[Fill]		
10	6	4	SPT	18		35	
15	4	5	SPT	1	ML Gray SILT with fine sand, clay and wood debris Low to medium plasticity (moist, very soft to medium stiff)[Fill]		%F= 79
20	0	6	D+M	3		39	No recovery due to wood in sampler shoe
25	0	7	D+M	10		43	No recovery
	26	8	ST		SM Gray silty fine SAND (moist, medium dense)[Fine-grained Flood Deposits]	33	
30	9	9	SPT	22			

Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
1419 Washington Street Suite 101
Oregon City Oregon 97045

LOG OF BORING NO. B-3

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						GP Gray GRAVEL with sand and trace silt Fine to coarse sand Fine to coarse rounded gravel (moist, dense) [Coarse-grained Flood Deposits]		
35	10	10	SPT	100		Grades to very dense		50 blows for 6
	8	11	SPT	100				50 blows for 6
						Boring completed at 38 ft bgs Groundwater not determined due to drilling method used		
40								
45								
50								
55								
60								

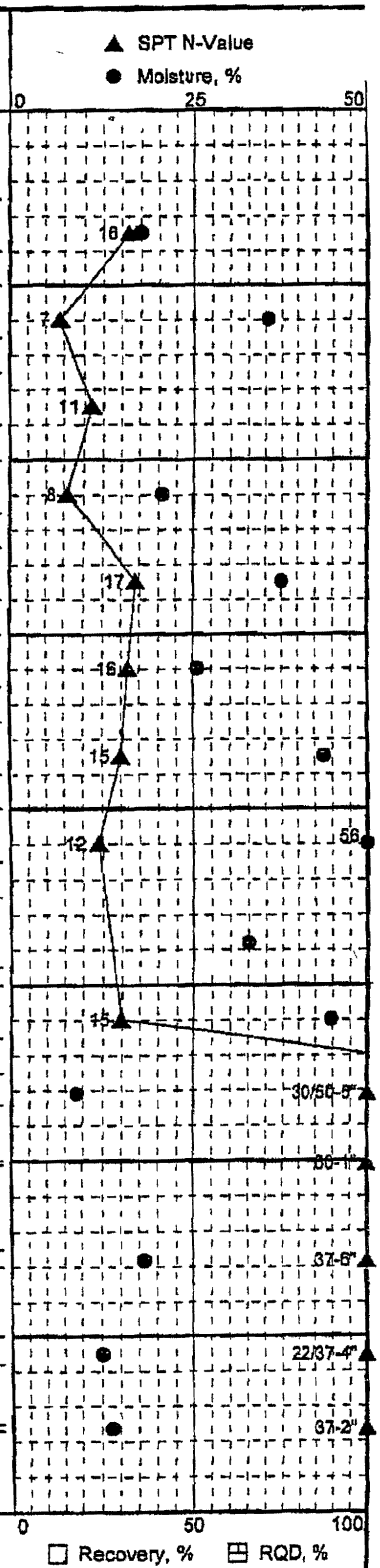
Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

APPENDIX B
SUBSURFACE EXPLORATIONS BY OTHERS

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth in Feet	Samples
		30 0				
		29 2	**PAVEMENT**, 2 5 inches of brick over 10 inches of steel reinforced concrete (Pavement)		0	
		0 9	**SAND FILL**, medium dense, gray, moist, fine-grained, trace fine rounded gravel and crushed rock (Fill)			S-1
		26 0				
		4 0	**Sandy SILT FILL**, slightly clayey, loose, light brown, moist, no to low plasticity, scattered fine rounded gravel (Dredged sand fill)		5	S-2
						S-3
		20 0			10	
		10 0	**SAND FILL**, loose, gray brown, moist, fine-grained, scattered fine rounded gravel (Dredged sand fill)			S-4
			Tree fragment, medium dense			S-5
		16 0				
		14 0	**Silty SAND FILL**, slightly clayey, medium dense, gray, moist to wet, no to low plasticity, fine-grained, scattered to abundant wood fragments and red crushed rock (Fill)		15	S-6
	-200 = 33%					S-7
▼	08/28/2000 -200 = 35%				20	S-8
	-200 = 16%					S-9
		5 0			25	
	-200 = 73%	25 0	**Clayey SILT**, stiff, gray, wet, low plasticity, micaceous (Recent Alluvium)			S-10
		3 0				
		27 0	**Sandy GRAVEL**, very dense, brown and gray, wet, fine- to coarse-grained, subrounded gravel, sand is fine- to coarse-grained, scattered cobbles (Pleistocene Flood Deposits - Qfch)			S-11
					30	S-12*
						S-13
					35	S-14
						S-15



LEGEND

- ⌶ = 2 0" O D Split Spoon Sample
- ⌶ = 3 0" O D Thin-Walled Sample
- * = Sample Not Recovered
- ▩ = Grab Sample Drill Cuttings
- ▮ = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1
page 1 of 2

December 2000 *F-3112 01*

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland, Oregon

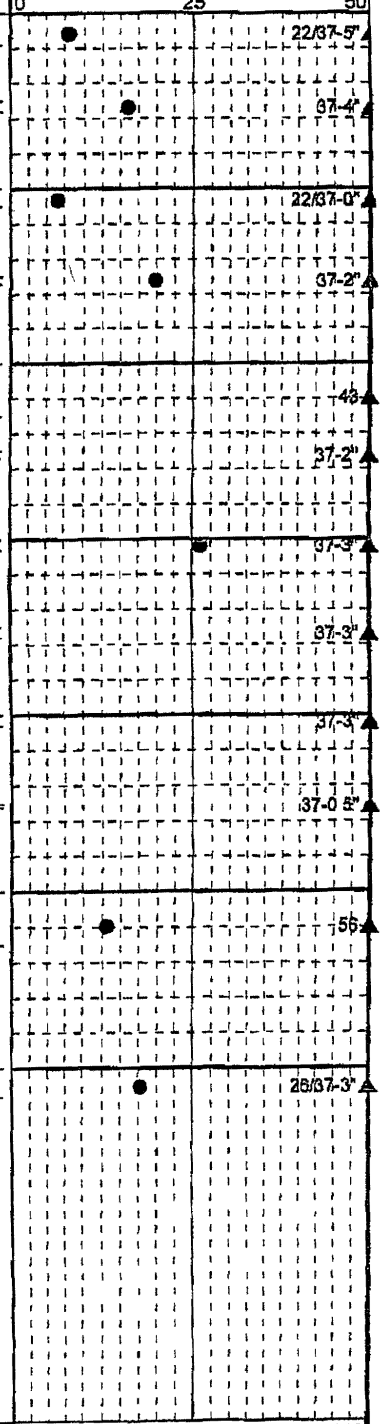
FIG.

FHA

WLG ANKPS.GPJ 12/6/00

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth in Feet	Samples
			Sandy GRAVEL continued		45	S-16, S-17, S-18, S-19
		-18 5 48 5	SAND, very dense, no sample return (Pleistocene Flood Deposits - Qfch)		50	S-20*, S-21*
		-21 0 51 0	Clayey Gravelly SAND, very dense, yellow-brown, moist, low plasticity, sand is fine-grained, fine- to coarse-grained, rounded gravel, weathered, overconsolidated, poorly graded, gradational transition to underlying material (Troutdale Formation Tt)		55 60	S-22, S-23*, S-24*, S-25*
		-35 0 65 0	Gravelly SAND, very dense, light brown, wet, non-plastic sand is fine-grained, fine- to coarse-grained, rounded gravel, overconsolidated, poorly graded (Troutdale Formation Tt)		65	S-26
		-40 8 70 8	Bottom of Boring, Completed 8/29/00		70	S-27

▲ SPT N-Value
● Moisture, %



□ Recovery, % ⊟ RQD, %

LEGEND

- I = 2.0" O.D. Split Spoon Sample
- II = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- ▩ = Grab Sample Drill Cuttings
- ▮ = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

FHA

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1
page 2 of 2

December 2000 *F-3112.01*

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland, Oregon

FIG

WLG ANKPS.GPJ 12/5/00

APPENDIX C
LABORATORY TESTING

APPENDIX C
LABORATORY TESTING

GENERAL

Soil samples obtained from the explorations were transported to our laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils encountered Representative samples were selected for laboratory testing The tests were performed in general accordance with the test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures

VISUAL CLASSIFICATIONS

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the Unified Soil Classification System (USCS) and ASTM classification methods ASTM Test Method D2488 was used to classify soils using visual and manual methods ASTM Test Method D2487 was used to classify soils based on laboratory test results

Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D2216 The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in Appendix A

Dry Density

Dry density tests were completed on low-disturbance samples obtained with the Dames & Moore (D&M) sampler The tests were conducted in general accordance with ASTM Test Method D2937 Dry density test results are presented on the boring logs in Appendix A

Percent Fines

Fines content analyses were performed to determine the percentage of soils finer than the No 200 sieve - the boundary between sand size particles and silt size particles The tests were performed in general accordance with ASTM D1140 The test results are indicated on the individual boring logs in the column labeled "Other Tests and Notes"

KEY TO EXPLORATION LOGS

Pacific Geotechnical, LLC

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SOILS CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL GRADED GRAVELS GRAVEL SAND MIXTURES
			GP	POORLY GRADED GRAVELS GRAVEL SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS GRAVELS SAND SILT MIXTURES
			GC	CLAYEY GRAVELS GRAVEL SAND CLAY MIXTURES
MORE THAN 50% RETAINED ON NO 200 SIEVE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)	SW	WELL GRADED SANDS GRAVELLY SANDS
			SP	POORLY-GRADED SANDS GRAVELLY SANDS
	MORE THAN 50% OF COARSE FRACTION PASSING NO 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SITLY SANDS SAND SILT MIXTURES
			SC	CLAYEY SANDS SAND CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY GRAVELLY CLAYS SANDY CLAYS SILTY CLAYS LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% PASSING NO 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS MICACEOUS OR DIATOMACEOUS SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT HUMUS SWAMP SOILS WITH HIGH ORGANIC CONTENTS

Note Multiple symbols are used to indicate borderline or dual soil classifications

SYMBOLS LETTER	DESCRIPTIONS
CC	CEMENT CONCRETE
AC	ASPHALT CONCRETE
TS	TOPSOIL/SOD FORREST DUFF

Stratigraphic Contact

—— Distinct contact between soil strata or geologic units

----- Gradual or approximate change between soil strata or geological units

Moisture Modifiers

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below the water table

Seepage Modifiers

None

Slow - < 1 gpm

Moderate - 1- 3 gpm

Heavy - > 3 gpm

Caving Modifiers

None

Minor - isolated

Moderate - frequent

Severe - general

Minor Constituents

Trace < 5% (silt/clay)

Occasional < 15% (sand/gravel)

With 5-15% (silt/clay) in sand or gravel

15-30% (sand/gravel) in silt or clay

Sampler Symbol Descriptions

- 2 4-inch I D split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- G Bulk or grab

Laboratory / Field Tests

%F Percent fines

AL Atterberg Limits

CP Laboratory compaction test

CS Consolidation test

DS Direct shear

HA Hydrometer analysis

Laboratory / Field Tests

DD Dry density

OC Organic content

PP Pocket penetrometer

SA Sieve analysis

TV Torvane shear

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586 See exploration log for hammer weight and drop

A "P" indicates sampler pushed using the weight of the drill rig

(2 4-inch) sampler N approximately corrected to equivalent SPT N by 50% reduction in N - modified California

Note Refer to the report text and exploration logs for a proper understanding of subsurface conditions Descriptions on the logs apply only at the exploration locations at the time the explorations were made The logs are not warranted to be representative of the subsurface conditions at other locations or times

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						SW Brown gravelly SAND, fine to coarse sand, fine rounded gravel (moist, loose) [Fill]	18	
	10	1	SPT	9				
5	8	2	SPT	7		Grades to with layers of SILTY SAND with gravel	18	
	5	3	SPT	4		Grades with trace silt		
10	6	4	SPT	4			16	
15	4	5	SPT	3		Grades with trace to some gravel (very loose)		
20	2	6	SPT	32		GP Brown POORLY-GRADED GRAVEL with trace fine sand Rounded gravel to 1 maximum dimension (moist to wet, medium dense to dense)[Fill]	8	Interpreted as fill based on depth
25	3	7	SPT	30				
30							20	

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	5	8	SPT	10	Grades to loose		Lost circulation and mud Sampler possibly driven through slough May be denser than blow counts suggest
35	6	9	SPT	10			
40	10	10	SPT	24	Grades with fine to coarse sand (medium dense to dense)	13	
							Regain circulation at 43ft
45	0	11	SPT	47			No recovery
50	9	12	SPT	21	GP-SP Sandy GRAVEL to gravelly SAND Fine to medium sand Fine rounded gravel (wet, dense)[Fill]	15	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
55	6	13	SPT	28	SM Gray SAND with gravel and silt Fine to medium sand Fine rounded gravels (wet, medium dense to dense) [Fill]	20	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
60							

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	1	14	SPT	42	SP Brown SAND with occasional fine rounded gravels (wet, very dense) [Troutdale Formation]		Hard drilling at 61ft
65	4	15	SPT	100			50 blows for 5
70	15	16	SPT	56	Becomes dark brown with some yellow		
	6	17	SPT	100	GP Brown POORLY-GRADED GRAVEL with fine to medium sand (wet, very dense)		50 blows for 4 5
75					Boring completed at 73ft bgs Groundwater not determined due to drilling method used		
80							
85							
90							

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-2

Project Name Ankeny Plaza

Location Waterfront Park, Portland OR

Number 1050-001-00

Depth feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
				20 40 60 80		10 30 50 70 90	
					GP-GM Dark gray POORLY-GRADED GRAVEL with sand and silt to SILTY GRAVEL Coarse angular basalt crushed rock (wet, medium dense)[Fill]		Boring relocated 10 feet west due to hitting abandoned sewer pipe
	10	1	SPT	12		15	
5	4	2	SPT	7	Grades to loose		
	10	3	SPT	9	SP-SM Brown to gray-brown POORLY GRADED SAND with silt to SILTY SAND with gravel and wood debris (moist, loose)[Fill]	20	
10	12	4	SPT	10	SM Dark gray-brown SILTY SAND with gravel (wet, loose to medium dense) [Fill]		
15	4	5	D+M	4	ML Gray brown and mottled brown SILT with fine sand Low to medium plasticity (moist, soft)[Fill]	37	DD= 84 pcf
20	16	6	SPT	4	Grades to dark gray with medium plastcity and occasional bricks	34	%F= 75
25	12	7	SPT	9	Grades to SILT with fine sand to fine sandy SILT (stiff)[Fill or Fine grained Flood Deposits?]	37	
30						24	

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-2

Project Name Ankeny Plaza

Location Waterfront Park, Portland OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	14	8	SPT	68		GM Dark gray SILTY GRAVEL with trace coarse sand and with angular rock (wet, very dense)[Coarse-grained Flood Deposits]		Very slow drilling at 30ft to 35ft
35	4	9	SPT	100				50 blows for 4"
						Grades to dark gray-brown SILTY GRAVEL with fine to medium coarse sand and rounded basalt gravels to 1 in maximum dimension		
40	3	10	SPT	100				50 blows for 2
45	6	11	SPT	100			14	90 blows for 11
						Boring completed at 46 ft bgs Groundwater not determined due to drilling method used		
50								
55								
60								

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-3

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050 001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
				20 40 60 80		10 30 50 70 90	
	13	1	SPT	25	SP Brown gravelly SAND to SAND with gravel Fine to medium sand Fine rounded gravel (moist, medium dense) [Fill]	13	
5	14	2	SPT	100	Grades with brick and concrete debris	20	50 blows for 6 ' due to concrete debris
	5	3	SPT	32	GM Gray SILTY GRAVEL with sand and occasional wood debris Fine to medium sand Fine to coarse rounded to subangular gravel (moist to wet, dense)[Fill]	35	
10	6	4	SPT	18			
15	4	5	SPT	1	ML Gray SILT with fine sand, clay and wood debris Low to medium plasticity (moist, very soft to medium stiff)[Fill]		%F= 79
20	0	6	D+M	3		39	No recovery due to wood in sampler shoe
25	0	7	D+M	10		43	No recovery
	26	8	ST		SM Gray silty fine SAND (moist, medium dense)[Fine grained Flood Deposits]	33	
30	9	9	SPT	22			

Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-3

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						GP Gray GRAVEL with sand and trace silt Fine to coarse sand Fine to coarse rounded gravel (moist, dense) [Coarse-grained Flood Deposits]		
35	10	10	SPT	100		Grades to very dense		50 blows for 6
	8	11	SPT	100				50 blows for 6
40						Boring completed at 38 ft bgs Groundwater not determined due to drilling method used		
45								
50								
55								
60								

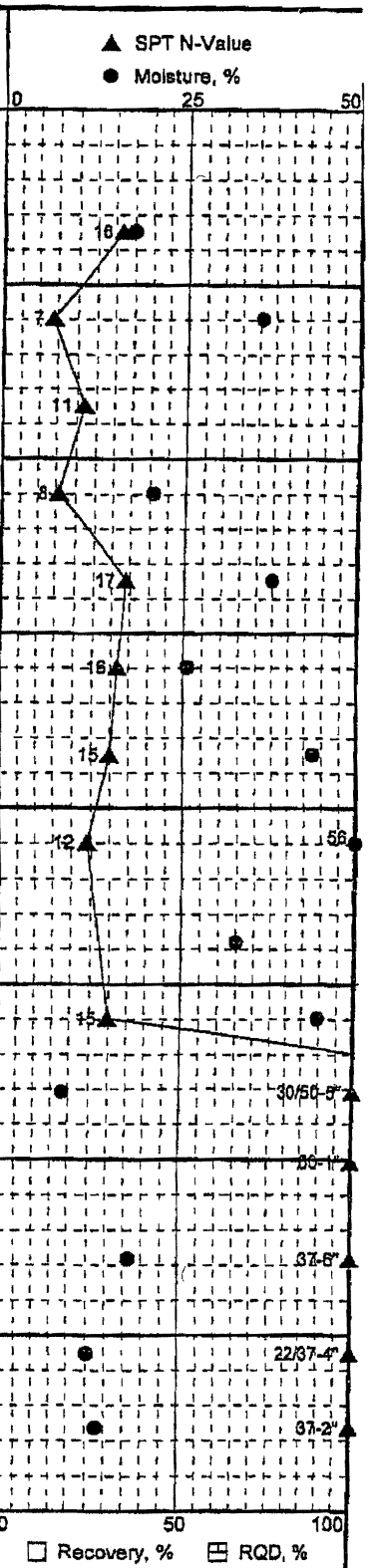
Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

APPENDIX B
SUBSURFACE EXPLORATIONS BY OTHERS

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Depth in Feet	Samples
		30 0			
		29 2	**PAVEMENT**, 2 5 inches of brick over 10 inches of steel reinforced concrete (Pavement)	0	
		0 9	**SAND FILL**, medium dense, gray, moist, fine-grained, trace fine rounded gravel and crushed rock (Fill)		S-1
		26 0			
		4 0	**Sandy SILT FILL**, slightly clayey, loose, light brown, moist, no to low plasticity, scattered fine rounded gravel (Dredged sand fill)	5	S-2
					S-3
		20 0		10	
		10 0	**SAND FILL**, loose, gray brown, moist, fine-grained, scattered fine rounded gravel (Dredged sand fill)		S-4
			Tree fragment, medium dense		S-5
		16 0			
		14 0	**Silty SAND FILL**, slightly clayey, medium dense, gray, moist to wet, no to low plasticity, fine-grained, scattered to abundant wood fragments and red crushed rock (Fill)	15	S-6
	-200 = 33%				S-7
▼	08/28/2000 -200 = 35%			20	S-8
	-200 = 16%				S-9
		5 0		25	
	-200 = 73%	25 0	**Clayey SILT**, stiff, gray, wet, low plasticity, micaceous (Recent Alluvium)		S-10
		3 0			
		27 0	**Sandy GRAVEL**, very dense, brown and gray, wet, fine- to coarse-grained, subrounded gravel, sand is fine- to coarse-grained, scattered cobbles (Pleistocene Flood Deposits - Qfch)		S-11
				30	S-12*
					S-13
				35	S-14
					S-15



LEGEND

- = 2 0" O D Split Spoon Sample
- = 3 0" O D Thin-Walled Sample
- * = Sample Not Recovered
- = Grab Sample Drill Cuttings
- = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1
page 1 of 2

December 2000 *F-3112 01*

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Geotechnical Consultants
Portland Oregon

FIG

FHA

WLG ANKPS GPJ 12/6/00

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	▲ SPT N-Value ● Moisture, % (0 – 25 – 50)
			Sandy GRAVEL continued			S-16	22/37-5"
						S-17	37-4"
					45	S-18	22/37-0"
						S-19	37-2"
		-18 5 48 5	SAND, very dense, no sample return (Pleistocene Flood Deposits - Qfch)		50	S-20*	48
		-21 0 51 0	Clayey Gravelly SAND, very dense, yellow-brown, moist, low plasticity, sand is fine-grained, fine- to coarse-grained, rounded gravel, weathered, overconsolidated, poorly graded, gradational transition to underlying material (Troutdale Formation Tt)			S-21*	37-2"
					55	S-22	37-3"
						S-23*	37-3"
					60	S-24*	37-3"
						S-25*	37-0.5"
		-35 0 65 0	Gravelly SAND, very dense, light brown, wet, non-plastic sand is fine-grained, fine- to coarse-grained, rounded gravel, overconsolidated, poorly graded (Troutdale Formation Tt)		65	S-26	56
					70	S-27	26/37-3"
		-40 8 70 8	Bottom of Boring, Completed 8/29/00				

0 50 100
□ Recovery, % ⊞ RQD, %

LEGEND

- I = 2 0" O D Split Spoon Sample
- II = 3 0" O D Thin-Walled Sample
- • = Sample Not Recovered
- ▩ = Grab Sample Drill Cuttings
- ▮ = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

FHA

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1
page 2 of 2

December 2000 *F-3112 01*

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FIG

WLG ANKPS GPJ 12/5/00

APPENDIX C
LABORATORY TESTING

WATERFRONT PARK IMPROVEMENTS AT SW NAITO AND BURNSIDE BRIDGE

SPECIFICATION INDEX

DIVISION 1 – GENERAL REQUIREMENTS

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Section 01251 CONTRACT MODIFICATION
Section 01290 PAYMENT PROCEDURES
Section 01300 PROJECT MANAGEMENT
Section 01320 PROJECT SCHEDULE AND COORDINATION
Section 01321 SPECIAL PROJECT SCHEDULE SUBMITTAL
Section 01330 SUBMITTALS AND SHOP DRAWINGS
Section 01400 QUALITY CONTROL
Section 01460 LEAD-BASED PAINT NOTIFICATION
Section 01500 TEMPORARY FACILITIES AND CONTROLS
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Section 01715 CONSTRUCTION WASTE MANAGEMENT
Section 01732 SELECTIVE DEMOLITION
Section 01748 WATER FEATURE COMMISSIONING
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DIVISION 2 – SITE CONSTRUCTION

Section 02221 BUILDING DEMOLITION
Section 02230 SITE CLEARING
Section 02231 TREE PROTECTION AND TRIMMING
Section 02270 EROSION CONTROL
Section 02300 EARTHWORK
Section 02458 STEEL H PILES – Added Addendum 3, April 11, 2008
Section 02665 WATER SUPPLY
Section 02466 DRILLED PIERS – Added Addendum 3, April 11, 2008
Section 02682 NATURAL GAS SERVICE
Section 02710 SUBDRAINS
Section 02715 STORM SEWERS
Section 02732 SANITARY SEWERS
Section 02741 HOT MIX ASPHALT PAVING
Section 02751 CEMENT CONCRETE PAVING
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Section 02810 IRRIGATION SYSTEMS
Section 02870 SITE FURNISHINGS
Section 02910 SOIL PREPARATION
Section 02920 LAWNS AND GRASSES
Section 02930 EXTERIOR PLANTS
Section 02950 ESTABLISHMENT MAINTENANCE

DIVISION 3 – CONCRETE

Section 03300 CAST-IN-PLACE CONCRETE
Section 03450 ARCHITECTURAL PRECAST CONCRETE

DIVISION 4 – MASONRY

Section 04810 UNIT MASONRY ASSEMBLIES – To be issued by Addendum Anticipated issue date, March 7, 2008

DIVISION 5 – METALS

Section 05120 STRUCTURAL STEEL – To be issued by Addendum Anticipated issue date, March 7, 2008
Section 05310 STEEL DECK – To be issued by Addendum Anticipated issue date, March 7, 2008
Section 05500 METAL FABRICATIONS
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DIVISION 6 - WOOD AND PLASTICS

Section 06100 ROUGH CARPENTRY AND SHEATHING
Section 06200 EXTERIOR FINISH CARPENTRY

DIVISION 7 - THERMAL AND MOISTURE PROTECTION

Section 07410 METAL ROOF PANELS
Section 07530 SINGLE-PLY MEMBRANE ROOFING
Section 07620 SHEET METAL FLASHING AND TRIM
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DIVISION 8 - DOORS AND WINDOWS

Section 08630 METAL-FRAMED SKYLIGHTS
Section 08800 GLASS AND GLAZING

DIVISION 9 – FINISHES

Section 09910 PAINTING
Section 09930 WOOD STAINS AND TRANSPARENT FINISHES
Section 09960 HIGH-PERFORMANCE COATINGS

DIVISION 10 – SPECIALTIES

NOT USED

DIVISION 11 – EQUIPMENT

NOT USED

DIVISION 12 – FURNISHINGS

NOT USED

DIVISION 13 – SPECIAL CONSTRUCTION

Section 13060 BIRD DETERRENT SYSTEM
Section 13150 WATER FEATURES

DIVISION 14 - CONVEYING SYSTEMS

NOT USED

DIVISION 15 – MECHANICAL

NOT USED

DIVISION 16 – ELECTRICAL

Section 16010 ELECTRICAL GENERAL REQUIREMENTS
Section 16012 ELECTRICAL SPECIAL REQUIREMENTS FOR WATER FEATURES
Section 16100 BASIC MATERIALS AND METHODS
Section 16110 RACEWAYS AND FITTINGS
Section 16130 CONDUCTORS
Section 16134 SPLICING AND TERMINATION
Section 16400 SERVICE AND DISTRIBUTION
Section 16450 GROUNDING
Section 16500 LIGHTING

End of Index

Bid Set

DRILLED PIERS

PART 1 - GENERAL

1 1 RELATED DOCUMENTS

A Drawings and general provisions of the Contract, including General and Supplemental Conditions and Division 1 Specification Sections, apply to this Section

1 2 SUMMARY

A This Section includes the following
1 Dry-installed or slurry displacement installed drilled piers, at Contractor's choice

1 3 DEFINITION

A Drilled Shafts - Reinforced concrete sections, cast-in-place against in situ soil, rock, or a casing

B Permanent Casing - Casing designed as part of the drilled shaft and intended to remain in place after concrete placement is completed

C Temporary Casing - Casing installed to facilitate drilled shaft construction only and removed during or after concrete placement

1 4 SUBMITTALS

A Drilled Shaft Installation Plan - At least 21 calendar days before beginning shaft construction, submit the following
1 The sequence of drilled shaft construction as it relates to the overall construction plan
2 A review of equipment suitability based on the Contractor's understanding of the site subsurface conditions Include a project history of the drilling equipment that demonstrates the successful use of the equipment for drilled shafts of equal or greater size in similar subsurface conditions
3 Details of shaft excavation methods, including proposed drilling methods and a disposal plan for excavated material Include details of methods used to perform final cleaning of the excavation Include details of the methods and materials used to fill or eliminate all voids between the plan shaft diameter and excavated shaft diameter, or between the casing and surrounding soil, if permanent casing is specified Include a disposal plan for any water or contaminated concrete expelled from the top of the shaft (if applicable)
4 Details of the proposed method(s) for ensuring drilled shaft stability during excavation and concrete placement

5 Details for the use of drilling slurry including mix design, slurry head requirements, mixing methods, maintaining, and disposing of the slurry (if applicable) Include a discussion of the suitability of the proposed drilling slurry in relation to the anticipated subsurface conditions

6 A plan for quality control of all drilling slurries, if their use is proposed In the quality control plan, include property requirements, required tests and test methods to ensure the slurry performs as intended Submit to the Engineer the name and current phone number of synthetic slurry manufacturer's representatives who will provide technical assistance during construction Provide the name(s) of the Contractor's personnel assigned to the Project and trained by the synthetic slurry manufacturer in the proper use of synthetics slurries

7 Unstamped shop drawings and details of H-pile placement, including splicing and centering Include details on the type, number, and placement of spacers and other devices for ensuring the H-pile position is maintained during construction

8 Evidence that the proposed materials and concrete mix design conform to all applicable Specifications

9 Details of concrete placement, including proposed operational procedures for pumping and tremie methods

10 Detailed procedures for permanent casing installation and temporary casing installation and removal Include casing diameters, dimensions, and depths and the methods and equipment for casing installation and removal

11 Confinement methods required to contain drilling fluids, spoils, waste concrete and other products from contacting sensitive environmental areas according to City of Portland Section 00290 and all applicable regulatory permits

12 Methods for protecting existing structures according to City of Portland Section 00170 82

13 The Engineer will approve or reject the drilled shaft installation plan within 21 calendar days after receipt of all submissions Provide any additional information and submit a revised plan, if requested, for review and approval All procedural approvals given by the Engineer will be subject to trial in the field and will not relieve the Contractor of the responsibility to satisfactorily complete the work Submit requests for modification of adopted procedures to the Engineer Allow 21 calendar days for approval of modifications Do not begin drilled shaft construction work until all drilled shaft submittals have been approved

B Drilled Shaft Repair Plans - For any shaft determined to be unacceptable, submit a repair plan to the Engineer for approval Furnish all materials and work, including engineering analysis and design, needed to correct unacceptable drilled shafts, at no additional cost to the City Do not begin repair operations before remedial procedures or designs are approved Any modifications to the dimensions or material of the drilled shafts shown on the plans that are proposed in the repair plan will require stamped calculations and working drawings according to City of Portland Section 00150 35

C Drilled Shaft Inspection Reports - Provide the Engineer with a completed Drilled Shaft Inspection Report for each drilled shaft, detailing the actual location, alignment, elevations, dimensions, and quantities of the shafts Submit the report within 21 calendar days after the completion and acceptance of each shaft

D Concrete Placement Logs and Volume Curves - Measure and record all concrete placed into drilled shafts using forms designated for this purpose Provide the Engineer with a completed Drilled Shaft Concrete Placement Log and Concrete Volume Curve Form for each drilled shaft within 24 hours after completion of shaft concrete placement

1 5 QUALITY CONTROL –

A Maintain and be responsible for quality control of the drilled shaft work throughout the construction operation The Engineer will inspect all drilling operations and verify the suitability of all drilled shaft construction procedures Provide lights, mirrors, weighted tape, weighted probe, personnel, and all assistance required for the Engineer to perform inspection during drilled shaft construction

1 6 QUALITY ASSURANCE

A DRILLED SHAFT COORDINATION MEETING

1 Hold a drilled shaft coordination meeting at least seven calendar days before beginning any shaft construction work at the site to discuss construction procedures, schedules, staging, personnel, equipment to be used, and other elements of the approved shaft installation plan as specified in City of Portland Section 00512 40 If synthetic slurry is used to construct the shafts, the frequency of scheduled site visits to the project site by the synthetic slurry manufacturer's representative will be discussed Those attending the meeting include

a Representing the Contractor - The superintendent, on-site supervisors, and all supervisors in charge of excavating the shaft, placing the casing, mixing and installing slurry (as applicable), placing the H-piles, and placing the concrete If synthetic slurry is used to construct the shafts, the slurry manufacturer's representative and a Contractor's employee trained in the use of the synthetic slurry shall also attend

b Representing the City - The Project Engineer, key inspection personnel, and designers of record or their appointed representatives

2 If the Contractor's key personnel change, or if the Contractor proposes a significant revision of the approved shaft installation plan, hold an additional meeting before any additional shaft construction operations are performed

PART 2 - PRODUCTS

2 1 General –

A Furnish materials meeting the following requirements

1 Piles - Use steel H-piles as shown on the plans and described in City of Portland Section 02520 10

2 Concrete - Use Class 4000 structural concrete according to City of Portland Section 02001, except as modified in this Section

2 2 Concrete Mix Design –

A Design the drilled shaft concrete mix for minimum segregation Use a pre-approved mix design that meets the slump requirements of City of Portland Section 02001

B Mix a trial batch and take test cylinders for one seven-day and two 28-day test breaks prior to placing concrete in the completed shaft excavations

C Water may be added at the Project Site only if allowed by the mix design and if approved Accurately measure water added at the site by water meters, buckets or other approved devices and limit it to 1 0 gallon per cubic yard Retarding or water-reducing agents may be used to maintain specified slump ranges and to facilitate temporary casing extraction To allow for adequate concrete workability throughout the duration of concrete placement and prior to temporary casing extraction, provide a concrete mix having a slump loss characteristic such that a minimum slump of 4 inches is maintained for the entire duration of the pour Admixtures may be used if tested and certified in the mix design and if approved

2 3 Steel Casing –

A Furnish temporary casing meeting the requirements of ASTM A 252 or ASTM A 36 Furnish permanent casing meeting the requirements of ASTM A 36 with the application of supplemental requirement S5 Test each heat of steel at 40 °F with a minimum absorbed energy requirement of 15 foot pounds Do not use previously used casing for permanent casing Use casing of sufficient strength to resist handling, transportation and installation stresses and the external stresses of the subsurface materials Ensure that the casing is clean and watertight prior to placement in the drilled shaft excavation Use casing with an outside diameter not less than the specified drilled shaft diameter

2 4 Drilling Slurry –

A Furnish drilling slurry meeting one of the following requirements

1 Mineral Slurry - Use mineral slurry conforming to the following requirements

Property	**Test**	**Requirement**
Density	Mud Density API * 13B-1, Section 1	64 - 75 lb /cu ft
Viscosity	Marsh Funnel and Cup API * 13B-1, Section 2 2	26 - 50 sec /qt
pH	Glass Electrode, pH Meter, or pH Paper	8 – 11
Sand Content	Sand API * 13B-1, Section 5	4 0 % max

Maintain slurry temperature at 40 °F or more during testing

*American Petroleum Institute

2 Synthetic Slurries - Select synthetic slurries from the QPL Use synthetic slurries according to the manufacturer's recommendations and the Contractor's quality control plan The sand content of synthetic slurry shall be less than 2 0 percent (API 13B-1, Section 5) prior to final cleaning and immediately prior to concrete placement

3 Water Slurry - Water may be used as slurry when casing is used for the entire length of the drilled shaft Use of water slurry without full-length casing will only be allowed with the Engineer's approval Use water slurry conforming to the following requirements

Property	**Test**	**Requirement (Maximum)**
Density	Mud Weight (Density) API 13B-1, Section 1	70 lb /cu ft
Sand Content	Sand API 13B-1, Section 5	2 0 %

Do not use blended slurries

2 5 Grout –

A Furnish portland cement grout meeting the requirements of City of Portland Section 02080

PART 3 - EXECUTION

3 1 CONSTRUCTION TOLERANCES

A Excavate drilled shafts as accurately as possible at the locations shown and within the specified tolerances listed below Determine the drilled shaft dimensions and alignment with approved methods The following construction tolerances apply to drilled shafts unless otherwise stated

1 Horizontal Position (At the Plan Elevation of the Top of Shaft)
   a Shaft Diameter Less Than or Equal to 6 Feet - 3 inch horizontal tolerance from the location shown
2 Top Elevation of Shaft Concrete
   a Top Elevation Above Water - Minus 3 inches to plus 1 inch from the plan top of shaft elevation
3 Vertical Alignment in Soil - May not vary from the plan alignment by more than 1 5% of the shaft length
4 Top of Pile - Plus or minus 6 inches from the plan top of pile elevation

B Frequently check the plumbness, alignment, and dimensions of the shaft during construction Correct all out-of-tolerance shaft excavations and completed shafts to the satisfaction of the Engineer Materials and work necessary to complete corrections for out-of-tolerance drilled shafts will be at the Contractor's expense, and no extension of the Project completion date will be granted Materials and work necessary to complete corrections for out-of-tolerance drilled shafts resulting from the removal of unexpected drilled shaft obstructions will be paid for as Extra Work

3 2 DRILLED SHAFT EXCAVATION

A General - Excavate drilled shafts to the dimensions and elevations shown or as directed Provide and maintain stabilized drilled shaft sidewalls and bottoms for the full depth of the excavation, using approved materials, equipment and methods If caving or other unstable conditions occur during any construction procedure, stop further construction, notify the Engineer, and stabilize the shaft excavation by approved methods and submit a revised installation plan which addresses the problem and prevents further instability Do not continue with shaft construction until any damage which occurred has been repaired according to the Specifications and until receiving the Engineer's approval of the revised shaft installation plan

1 If the Engineer has reason to believe that the drilled shaft excavation techniques or workmanship have been deficient, so that the integrity of any excavation is in question, work on that drilled shaft may be stopped Drilled shaft excavation will not be allowed to resume until the deficient excavation techniques or workmanship have been changed to the Engineer's satisfaction

2 Dispose of materials removed from the shaft excavations according to City of Portland Section 00290 20

3 Do not leave partially completed shaft excavations open overnight unless they are cased full depth or otherwise stabilized with approved methods If approved by the Engineer, a partially excavated shaft may be left open overnight, provided that the excavation

a Is stabilized at the bottom, sides and surface to prevent soil caving or swelling or a reduction of soil strength, and

b Is covered at the surface to protect the public

4 Extend the drilled shaft excavation if the Engineer determines that the subsurface materials encountered are not capable of providing the required bearing resistance or differ from those anticipated in the design of the drilled shafts

B Protection of Existing Structures - Control shaft construction operations to prevent damage to existing structures and utilities Preventive measures include, but are not limited to, selecting construction methods and procedures that will prevent caving of the shaft excavation and monitoring and controlling the vibrations from construction activities such as the driving and vibrating of casing or sheeting or drilling of the shaft Repair all damage caused to existing structures, utilities or other facilities, resulting from drilled shaft construction activities, at no additional cost to the Owner

C Temporary Casing - Provide temporary casing according to the approved installation plan and of sufficient quantities to meet the needs of the anticipated construction method

D Unexpected Drilled Shaft Obstructions - Remove any natural or manmade object encountered that was not revealed by the Owner's site investigation, and that would cause a significant decrease in the rate of advancement if removed using the techniques and equipment used successfully to excavate the shaft The Engineer will be the sole judge of the significance of any reduced rate of shaft advancement and the classification of any unexpected obstructions Removal of unexpected obstructions from the shaft excavation will be paid as Extra Work

E Lost Tools - Promptly remove drilling tools lost in the excavation Lost tools will not be considered unexpected obstructions and shall be removed without additional compensation Drilling tools lost during the course of removing unexpected drilled shaft obstructions will be paid as Extra Work

F Drilling Slurry Installation - If synthetic drilling slurry is selected, provide a manufacturer's representative to provide technical assistance at the site prior to use of the slurry, who shall remain at the site during construction and completion of a minimum of one drilled shaft to adjust the slurry mix for the specific site subsurface conditions After the manufacturer's representative is no longer at the site, provide the approved personnel trained in the use of the synthetic slurry for the remainder of the shaft slurry operations to supervise the proper slurry mix design and quality control procedures

1 All in-hole drilling slurry shall meet the required slurry specifications during excavation and prior to concrete placement Clean, recirculate, de-sand or replace the slurry to maintain the required slurry properties
2 Unless otherwise approved, maintain the level of slurry in the excavation at not less than 5 feet above the groundwater level for mineral slurries or 10 feet above the groundwater level for synthetic or water slurries Maintain the slurry level a sufficient distance above all unstable zones to prevent bottom heave, caving or sloughing
3 Maintain the required slurry properties and levels at all times during shaft construction, including work stoppages, unless other approved stabilization methods are applied
4 Feed slurry continuously into the shaft excavation as drilling progresses so that a stable excavation is maintained Use a self-priming pump to reclaim the slurry Keep a standby pump available during the drilling operation

G Drilling Slurry Inspection and Testing - Mix and thoroughly hydrate all drilling slurries in an appropriate storage facility Collect sample sets from the storage facility and perform tests to ensure the slurry conforms to the specified material properties before introduction into the drilled shaft excavation A sample set shall be composed of samples taken at mid-depth and within 24 inches of the bottom of the storage facility

1 Sample and test all slurry in the presence of the Engineer, unless otherwise directed The sample sets of slurry within the excavation shall consist of samples taken at mid-depth of the excavation and within 24 inches of the bottom of the excavation Collect and test sample sets during the drilling operation as necessary to ensure the specified properties of the slurry are maintained Clean, recirculate, de-sand, or replace the slurry as necessary to maintain the specified slurry properties Final cleaning of the excavation and placement of concrete will not be allowed until the test results indicate the slurry properties are as specified

2 Perform a minimum of two sets of slurry tests per eight-hour work shift, the first test being done at the beginning of the shift Field conditions may require more frequent testing to ensure acceptable slurry properties
3 Make copies of all slurry test results available to the Engineer on request

H Clean Out - Use appropriate means, such as a cleanout bucket, pump or air lift, to clean the bottom of the drilled shaft excavations No more than 2 inches of loose or disturbed material will be allowed at the bottom of the excavation for end-bearing drilled shafts Shaft cleanliness will be determined by the Engineer
1 Notify the Engineer of completion of each drilled shaft excavation to permit inspection before proceeding with construction Measure final shaft depths with a suitable weighted tape or other approved method after final cleaning to determine that the shaft bottom meets the requirements in the Contract Do not proceed with shaft construction until the bottom cleanliness requirements have been met and the bottom (shaft tip) elevation is approved

3 3 PILES

A Placement - Do not place H-pile in the shaft excavation until the Engineer has approved the final elevation of the bottom of the shaft
1 In each shaft, place H-pile extending from 6 inches above the bottom of the shaft excavation to the elevation shown The H-pile may be supported on the bottom of the shaft excavation if approved Support the H-pile to prevent distortion or settlement during concrete placement If concrete placement does not immediately follow H-pile placement, remove the H-pile from the excavation and rectify the integrity of the excavation prior to reinstallation of the H-pile

B Splicing - Where splices are unavoidable, submit for approval their number, location and details per City of Portland Section 00520 43

C Concrete Cover - Maintain a minimum of 2 inches of concrete cover by placing concentric spacer bars or other approved devices around the H-pile Provide details of the proposed centering method on the shop drawings submitted according to City of Portland Section 00512 40

3 4 CONCRETE

A Concrete Placement - Place concrete immediately after completion of the shaft excavation and with the approval of the Engineer Prior to concrete placement, ensure the shaft clean-out requirements are met according to City of Portland Section 00512 43(h) and the properties of the slurry (if used) conform to specifications Shaft concrete may be placed without mechanical vibration in those areas of the drilled shaft that are not formed or are below the ground line or the water surface

Place concrete continuously until concrete at the top of the shaft is free of water, soil, and debris, and uncontaminated concrete extends to the plan top-of-shaft elevation Dispose of all contaminated concrete expelled from the top of the shaft in an approved manner Remove waste concrete from the site If a delay in concrete placement occurs because of a

delay in concrete delivery or other factors, reduce the placement rate to maintain a flow of fresh concrete into the shaft excavation

Unless otherwise approved by mix design, allow a maximum of 60 minutes between concrete placements and use no concrete older than 90 minutes from batch time Use procedures for concrete placement which ensure that the concrete within the shaft becomes a monolithic, homogeneous unit

Place concrete using hoses or pipes having watertight joints For concrete placement by gravity tremie, use hose or pipe having an inside diameter of at least 8 inches For placement by concrete pump, use hose with inside diameter of at least 4 inches Provide an alternate delivery system that can be used in case of failure of the primary delivery system Place concrete only against the bottom of the drilled shaft or into fresh concrete

If caving occurs during concrete placement, the shaft may be rejected

B Wet Shaft Concrete Placement - If the drilled shaft excavation does not meet the requirements for dry concrete placement, stabilize water inflow and place the concrete under water or slurry with a tremie pipe or pump hose according to City of Portland Section 00540 48(e) Place concrete continuously from the bottom of the shaft to the top-of-shaft elevation shown Use a plug in the tremie pipe or pump hose to force water or slurry ahead of the advancing flow of fresh concrete Dispose of all displaced water, slurry, or waste concrete according to City of Portland Section 00290 20 When groundwater, the drilling water or slurry in the shaft excavation is to be removed by pumping during concrete placement, have a standby pump available

Place concrete in a continuous operation so that the concrete always flows upward within the shaft Withdraw the delivery hose or pipe slowly as the elevation of the fresh concrete rises in the shaft Keep the discharge end of the pipe or hose at least 5 feet below the surface of the concrete after the concrete has reached a depth of 5 feet Maintain sufficient concrete inside the hose or pipe to prevent drilling fluid from entering During concrete placement, provide and maintain markings on the tremie pipe or pump hose, or a sounding device or other appropriate method to determine the relative elevations of the fresh concrete surface and the bottom end of the pipe or hose Raise the bottom end of the pipe or hose only when the pipe or hose has a sufficient head of fresh concrete to prevent the formation of a void at the bottom

C Concrete Curing and Cleaning - Allow the exposed top of concrete to cure a minimum of seven calendar days by covering with wet burlap overlain with plastic sheets or by keeping top of concrete under water Keep the burlap wet during the concrete cure

Prior to placing any fresh concrete on top of a completed shaft, clean the upper surface of the concrete by removing all scum, laitance, loose gravel, and sediment and chip off any high spots on the upper surface that would prevent the H-pile from being properly placed in the position shown on the plans

D Casing Removal - Remove all temporary casing during or after completion of concrete placement Do not start temporary casing removal until the level of fresh concrete within

the casing has reached a depth of at least 10 feet or the level necessary to adequately counteract the external hydrostatic pressure head As the temporary casing is withdrawn, maintain a minimum 5 feet head of concrete above the bottom of the casing A slight downward movement of the casing while exerting downward pressure, or hammering or vibrating the casing will be allowed to facilitate extraction Extract the casing so that concrete is cast directly against the surrounding in-situ material Check the elevation of the top of the H-pile before and after temporary casing extraction for conformance with the construction tolerance criteria Casing that cannot be extracted during, or immediately after, the concrete placement operation may be cause for rejection of the shaft

Remove the tops of permanent casing to the top of the drilled shaft or the finished groundline, whichever is lower, unless otherwise shown or directed

3 5 DRILLED SHAFT TESTING AND ACCEPTANCE

A Acceptance of drilled shafts will be based on the Engineer's review of field inspection reports and visual observations during drilled shaft construction The Engineer has final authority on the approval of drilled shafts For shafts that are integrity tested, the Engineer will determine final acceptance of each tested shaft, based on the integrity test results and inspection reports and will provide a response to the Contractor within five calendar days

B Additional Testing and Investigation - Conduct additional testing or investigation necessary to identify the location, extent and condition of possible shaft defects if requested by the Engineer Additional testing and investigation may include, but is not limited to, excavation work or core drilling

If requested by the Engineer, drill a core hole in any questionable quality shaft to explore the shaft condition The number, location and depths of the core holes will be determined by the Engineer Submit the method and equipment used to drill and remove cores from the shaft to the Engineer for review and approval prior to drilling Use a coring method that provides complete core recovery and minimizes abrasion and erosion of the core If a defect is confirmed, as determined by the Engineer, all investigation costs associated with identifying the defect will be at no additional cost to the City and no extension of the Project completion date will be granted, regardless of whether the identified defect is repaired or not

If no defect is identified in the investigation work, the City will pay for all coring and excavation costs associated with the additional investigation and grant an appropriate time extension, if required, according to City of Portland Section 00190 and City of Portland Section 00195

Fill all core holes with grout only after the evaluation process is completed and the shaft is accepted and approved

C Drilled Shaft Repair - Repair all defects and rejected shafts according to City of Portland Section 00512 40(b) Perform investigation required, as directed by the Engineer, to confirm the quality of the completed shaft repair work at no additional cost to the City with no time extension granted *For temporary casing not extracted from the shaft

excavation, submit a repair plan or a structural evaluation to the Engineer for approval according to City of Portland Section 00512 40 If caving occurs during concrete placement submit a repair plan to the Engineer for approval

3 6 SCHEDULING AND RESTRICTIONS

A Do not proceed with construction of subsequent shafts until the first drilled shaft has been approved by the Engineer Approval to proceed with the construction of subsequent shafts will be based on the Engineer's observations of the Contractor's workmanship during construction of the first shaft and the Engineer's review and assessment of the following

1 The Contractor's conformance with the approved shaft installation plan
2 The Contractor's daily reports and inspector's daily logs of excavation, and concrete placement
3 The concrete placement logs and volume curves

B Written notification will be provided to the Contractor on whether or not to proceed with subsequent shaft construction within 24 hours after completion of the first shaft

C Denial of permission to proceed with subsequent shaft construction will not be cause for contract time extension

D After the first drilled shaft on the Project has been accepted, make no significant changes in construction methods, equipment, or materials used to construct subsequent shafts, unless otherwise approved

END OF SECTION 02466

Pacific Geotechnical, LLC
Geotechnical Engineering and Engineering Geology with a Focus on Sustainability

February 18, 2008

Walker Macy
111 SW Oak, Suite 200
Portland, Oregon 97204

Attention Mrs Chelsea McCann

Subject Supplemental Geotechnical Report for Foundation Support Recommendations
Portland Waterfront Park, Ankeny Plaza & Street Improvements
Portland, Oregon
Project No 1050-001-00

1 INTRODUCTION

Pacific Geotechnical, LLC (Pacific Geotechnical) is pleased to submit this supplemental geotechnical report for the Portland Waterfront Park, Ankeny Plaza & Street Improvements project located in Portland, Oregon The location of the site is shown on Figure 1

We completed a preliminary geotechnical report for the project dated April 12, 2007 that included a document review of available subsurface information completed by others, but did not include any site specific subsurface explorations In our report we provided general recommendations for earthwork and foundation support but recommended that site specific explorations be completed for important structures once final design locations were selected The purpose of this report is to provide foundation support recommendations for the proposed Portland Saturday Market (PSM) cover structure and the cantilever overlook structure Our work for this supplemental report was completed in general accordance with Change Order No 2, dated December 28, 2007

2 PROJECT UNDERSTANDING

The PSM cover structure and cantilever overlook structure will be located in Waterfront Park just south of the Burnside Bridge, as shown in Figure 2 The PSM cover structure will be located above the existing combined sewer overflow (CSO) pipe and immediately adjacent to an existing concrete vault Structural loads are expected to be up to about 200 kips Shallow foundations are not considered appropriate at this location due to the anticipated structural loads and the thickness and variability of the underlying fill soils

The appropriate foundation type will need to maintain a safe distance from the CSO pipe and prevent damage to the existing concrete vault during installation

The proposed overlook deck is expected to be constructed just south of the existing pump station The existing wooden dock structure in this area will be removed and the existing concrete seawall will be reconstructed in places The overlook deck will be supported on the seawall with possible additional support provided by new piles Structural loads for the overlook deck are about10 kips per lineal foot (klf) along the seawall and about 40 kips for the piles

Additional related civil improvements are expected to include site utilities, pavements, a stormwater infiltration garden and a water feature

3 PURPOSE AND SCOPE OF SERVICES

The purpose of our work is to provide geotechnical engineering services for design and construction of the project Our specific scope of work included the following

- Located proposed exploration locations in the field based on existing conditions, proposed structure locations and available survey drawings
- Coordinated clearance of existing site utilities via the required One-Call Service
- Explored subsurface soil and groundwater conditions by completing five borings using hollow stem and mud rotary equipment
- Obtained samples at representative intervals from the explorations, observed groundwater conditions, and maintained detailed logs in general accordance with the American Society for Testing and Materials (ASTM) Test Method D2488
- Performed two falling head infiltration tests
- Conducted laboratory testing on representative collected samples
- Provided a geotechnical evaluation and analysis of the site and design recommendations in this geotechnical report

4 SITE DESCRIPTION

4 1 REGIONAL AND LOCAL GEOLOGY

Regional and local geology are summarized in our original report and discussed in detail in the geotechnical baseline report for the West Side CSO Tunnel, Shafts, Pump Station and Pipelines Project (Parsons Brinckerhoff, 2002a)

4 2 SURFACE CONDITIONS

The project site is located in Waterfront Park just south of the Burnside Bridge The surface conditions in the vicinity of the proposed PSM cover structure include landscape grasses at the north end and a construction area at the south end The construction area is fenced off and encloses the existing Ankeny Shaft and a vault structure In the vicinity of the overlook structure surface conditions include brick pavement adjacent to the existing pump station and seawall A dock structure is located in the Willamette River adjacent to the seawall in this area

4 3 SUBSURFACE CONDITIONS

Subsurface conditions encountered by others in and around the project area are summarized in our original report The following paragraphs describe the subsurface conditions encountered in our explorations for this project and reference other nearby explorations as needed

We explored subsurface conditions at the site by drilling five borings at the locations shown on Figure 2 Three borings (B-1, B-2 and B-3) were completed to between 38 to 73 feet below the ground surface (bgs) to evaluate subsurface conditions for foundation analysis and design Two borings (B-4 and B-5) were completed to shallow depths to complete infiltration testing

A geologic cross-section through the proposed PSM cover structure location is provided as Figure 3 Appendix A summarizes the exploration methods and presents the exploration logs completed under this contract Appendix B presents the exploration logs completed by others in the immediate vicinity of the proposed improvements Results of our laboratory testing are indicated on the exploration logs in Appendix A and described in Appendix C

Subsurface conditions were found to be relatively uniform across the site In general, we encountered 1) silt, sand and gravel fill material, over 2) a thin layer of Fine-grained Flood Deposits, over 3) Coarse-grained Flood Deposits, over 4) Troutdale Formation gravels The conditions encountered in our explorations generally match the geologic mapping described in our original report The following paragraphs summarize the above units

4.3.1. Fill

We encountered variable fill material in all of our borings The fill material generally included loose to dense sand and gravel and very soft to stiff silt We also encountered occasional crushed rock, asphalt, concrete, brick and wood debris The thickness of the fill material varied from about 27 feet in boring B-3 to about 60 feet in boring B-1, adjacent to the seawall structure The later thickness is based on a drawing for the seawall/cribbing structure that shows the base of the cribbing at an elevation of -30 5 feet (City of Portland datum), or about 63 feet bgs Boring B-4 and B-5 did not penetrate the fill material The moisture content of this material varied between 8 to 39 percent The percent fines in the silt material was between 75 to 79 percent, based on two tests A dry density of 84 pounds per cubic foot (pcf) was measured on a single sample

4 3 2 Unconsolidated Silt and Sand (Alluvium/Fine-grained Flood Deposits)

We encountered a thin layer of silty fine sand in boring B-3 under the fill material We did not encounter this material in our other explorations, however, this unit is noted in boring C-1 completed by Fujitani Hilts & Associates (FHA) (PB 2002b, Appendix G) This material was medium dense with a moisture content between 38 and 43 percent, based on two tests

4 3.3 Unconsolidated Sand, Silt and Gravel (Alluvium/Coarse-grained Flood Deposits)

We encountered Coarse-grained Flood Deposits under the Fine-grained Flood Deposits in boring B-3 and below the fill material in boring B-2 We did not encounter this material in boring B-1, as the fill material extended to the top of the Troutdale Formation gravels (see below) This unit generally consisted of very dense gravel with variable amounts of sand and silt The moisture content measured 14 percent based on one test

4 3 4 *Consolidated Sand and Gravel (Troutdale Formation)*

We encountered gravels of the Troutdale Formation beneath the fill in boring B-1 at a depth of about 61 feet bgs This corresponds well with the design depth for the base of the cribbing structure supporting the adjacent seawall This unit consists of very dense sand and gravel in a silt matrix

4 4 GROUNDWATER

Groundwater could not be determined in our explorations due to the drilling method used

Based on previous subsurface explorations completed by others, groundwater levels for the project area are commonly between 18 and 23 feet bgs This depth usually corresponds to an elevation within several feet of the mean river level, suggesting that this groundwater is connected to, and will reflect changes in, the water level of the Willamette River Our experience and the groundwater levels interpreted by others (PB, 2002a) suggest that the groundwater levels will tend to be a few feet higher than the river level

5 INFILTRATION TESTING

Two infiltration tests were completed at the exploration locations and depths shown in Table 1 The tests were completed by placing a 5-inch diameter pipe into the borehole at the selected depth The pipe was embedded several inches into the underlying soil to prevent water loss around the annulus The pipe was filled with water to a level of 36 inches The water was allowed to infiltrate overnight We then refilled the pipe with water to 36 inches above the bottom of the pipe and began the infiltration test In accordance with City of Portland standards, we performed four infiltration test iterations by refilling the casing to 36 inches and measuring the drop in head during one-hour periods The following table provides a summary of the test results

Table 1 Infiltration Results

Test Location	Depth	Soil Description	Test Period	Infiltration Rate
B-4/IT-1	5 feet	SAND with gravel and silt (SM) [Fill]	4 hours	0 5 in/hr
B-5/IT-2	10 feet	SILT with gravel and sand (ML) [Fill]	4 hours	0 5 in/hr

The above results are similar to the 1 0 inches/hour that we measured in HA-2, nearby, during previous work Based on our earlier work, the infiltration rate is expected to vary widely across the site due to the variable fill material encountered near the surface

6. CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed project provided the recommendations in this report are included in design and construction We offer the following primary conclusions

1 The project lies in an area underlain by heterogeneous fill to depths varying from about 27 feet under the PSM cover structure to about 60 feet near the seawall The fill consists primarily of silt, sand and gravel, but has areas of rock, wood debris and other obstructions

2 Liquefaction and related effects are expected at the project site during the design earthquake

3 Due to the presence of the fill material and the potential for liquefaction during the design earthquake, structures should be supported on intermediate or deep foundations For intermediate

foundations, rammed aggregate piers were considered For deep foundations, steel H-piles were considered

4 Proposed piles near existing structures should be drilled H-piles, rather than driven to reduce the potential for vibrations to damage the structures

5 The CSO pipe traverses underneath the proposed PSM cover structure Based on previous work completed over the CSO pipe, a minimum clearance of at least one pipe diameter above and one half pipe diameter on both sides should be maintained between the CSO pipe and any foundation elements The required clearance should be verified with the City of Portland

6 Based on available drawings, the existing seawall is supported on timber cribbing bearing on native Troutdale Formation gravels From a geotechnical standpoint, this structure can adequately support the proposed light loads from the overlook deck, provided the structure is sound

7 STRUCTURAL DESIGN RECOMMENDATIONS

Due to the presence of variable fill material near the ground surface, potentially liquefiable soils and moderate structural loads, shallow foundations cannot adequately support the anticipated structural loads without ground improvements

For the PSM cover structure, we evaluated both intermediate (rammed aggregate piers) and deep (driven or drilled H-piles) foundation options For the overlook deck we evaluated deep foundations and supporting the structure on the existing/reconstructed seawall The following sections summarize our recommendations for each of the support options described above

7 1 INTERMEDIATE FOUNDATIONS

Rammed aggregate piers are a common ground improvement method and have been used successfully in supporting structures in these conditions

7 1 1. GeoPiers©

GeoPier© is a proprietary rammed aggregate pier system designed and constructed by GeoPier© Foundation Company, a specialty contractor, to a performance specification We anticipate that GeoPiers© should extend to a depth of approximately 15 feet or more below the bottom of the footings, depending on location GeoPiers© designers would provide final design for the subgrade improvement We recommend that Pacific Geotechnical provide construction quality assurance for the owner during the construction process GeoPier© would provide in-house quality control as well

7.1.2. Bearing Capacity

We typically would expect a bearing capacity of about 4,000 pounds per square foot (psf) in soils similar to those at the site that have been improved with GeoPiers© The final design bearing capacity would be provided by GeoPier©

For this option we recommend that footings have a minimum width of 24 inches The bottom of the footings should be founded at least 18 inches below the lowest adjacent grade The recommended minimum footing depth is greater than the anticipated frost depth

7 1.3. Footing Subgrade Preparation

After installation of the GeoPiers©, the subgrade should be prepared for the footings We recommend that loose or disturbed soils be compacted in-place or removed before placing reinforcing steel and concrete

Compaction should be performed as described in Sections 6 15 and 6 16 of our original report Foundation bearing surfaces should not be exposed to standing water Should water infiltrate and pool in the excavation, the water along with any disturbed soil should be removed before placing reinforcing steel A 6-inch thick layer of crushed rock may be used to provide protection of the subgrade from weather and light foot traffic

We recommend that Pacific Geotechnical observe all foundation excavations before placing reinforcing steel in order to determine if bearing surfaces have been adequately prepared and that the soil conditions are consistent with those observed during our explorations

7 1 4 Foundation Settlement

Settlement for shallow foundations supported on a GeoPier© improved subgrade as described above would depend on the GeoPiers© design Typically the systems are designed to a performance specification which is normally on the order of about 1 inch Differential settlements of up to the total settlement magnitude can be expected between individual footings

7 2 DEEP FOUNDATIONS

We evaluated both driven and drilled steel H-piles for support of the proposed PSM cover structure and overlook deck Based on information provided by the structural engineer, we understand that HP8x36 piles are the preferred size

Driven piles are expected to be suitable everywhere except adjacent to the existing vault structure Damage to nearby structures due to pile driving is a complex problem involving a number of factors including pile driving energy and frequency, distance between the pile and the structure, the structure's sensitivity to vibration and subsurface conditions It is difficult to predict during design and requires considerable monitoring during construction For these reasons, we recommend that proposed piles near the existing vault structure be drilled rather than driven We have assumed a 12-inch diameter drilled hole for design of the drilled pile option The hole would likely require casing to reduce soil sloughing The hole would be backfilled around the H-pile with Portland Cement Concrete grout Alternately, augercast drilling methods may be used to achieve similar capacities

7 2 1. Pile Capacities

The axial and lateral capacity of individual piles were evaluated with the computer program AllPile Version 7 8g, available from CivilTech Software

7 2 1 1 Axial Capacity and Uplift

The piles will achieve their capacity primarily from end bearing Pile foundations to support the PSM cover structure should be embedded a minimum of 5 feet into the native Coarse-grained Flood Deposits Static pile capacity recommendations for the minimum recommended embedment depth are provided in Table 2 For greater axial loads, an estimated allowable axial load versus depth plot is provided as Figure 4 Based on one pile diameter clearance above the CSO pipe, piles should not extend more than about 70 feet bgs where they are located above the CSO pipe This depth should be verified by the City of Portland

Pile foundations to support the overlook structure may require greater embedment than for the PSM cover structure due to the thick fill zone adjacent to the seawall, despite the relatively light structural loads Static pile capacity recommendations for the minimum recommended embedment depth are provided in Table 2 A greater embedment depth may be required depending on the suitability of the fill material at

the location of the proposed piles The terminal pile depth should be determined based on driving performance We expect a maximum depth of 70 feet may be required in this area

The capacities do not include the affects of seismic or liquefaction induced forces For driven piles, the final pile penetration depths should be determined in the field based on depth of embedment and terminal blow count criteria

The allowable pile capacities include a factor of safety of 3 0 for end bearing and 2 0 for friction The structural engineer should verify that the pile is structurally capable of supporting the expected loads The pile capacities shown do not include a group reduction factor

7 2 1 2 Lateral Capacity

The individual piles were modeled under fixed head conditions at the ground surface, based on our understanding that the pile will be connected with a pile cap at this point We used the calculated axial downward capacity of the pile in calculating the lateral capacity The static lateral capacities provided in Table 2 are based on an approximate 0 5 inch of horizontal deflection at the ground surface and do not include the affects of seismic forces or liquefaction induced forces

Table 2 Summary of Pile Capacities for HP8x36 Pile

Location	Minimum Embedment[2] (feet)	Axial Downward Capacity (kips)	Axial Upward Capacity (kips)	Lateral Capacity for <0 5 inches Deflection (kips)
PSM cover structure	40	100 (driven)/80 (drilled[1])	25	20 (strong axis)/12 (weak axis)
Overlook Deck	40	40	15	14 (strong axis)/ 8 (weak axis)

[1] Assumes a 12 inch diameter augered hole filled with structural concrete
[2] Or 5 feet into gravel, whichever is greater

7 2 1 3 Downdrag

Downdrag loads occur when the soil adjacent to the pile settles relative to the pile and results in decreased allowable axial capacity for the pile Downdrag usually occurs where piles are driven through soft, compressible materials or recent fill embankments The soils at the project site are generally not susceptible to downdrag forces under static conditions

7 2 1 4 Pile Group Reduction Factors

Based on anticipated structural loads, large pile groups are not expected for this project We recommend that driven piles be spaced at least 3 pile diameters apart (center-to-center) to minimize installation problems Driven piles spaced more than 3 pile diameters apart will not require a group reduction factor for calculating group axial capacity Drilled piles spaced closer than 5 pile diameters apart should not be constructed until adjacent piles have completely cured

For lateral capacity, group reduction factors are not required provided piles are spaced at least 5 pile diameters apart (center-to-center) For closer spacing, the reduction factors provided in Table 3 should be used Linear interpolation should be used for piles spaced between the values provided

Table 3 Group Lateral Pile Capacity Reduction Factors

Center-to-Center Pile Spacing (in pile diameters)	Pile Reduction Factors		
	Leading Row	Second Row	Third Row
5	1 0	0 85	0 70
3	0 80	0 40	0 30

7 2.2 Foundation Settlement

Settlement for pile foundations as described above are expected to be less than 1 inch Differential settlements of up to the total settlement magnitude can be expected between individual pile groups

7 2.3 Driving Criteria

Driving criteria for driven piles, including selection of hammer size and driving cushion, should be determined when the foundation design is completed The wave equation program WEAP should be used for establishing the criteria We can provide driving criteria for the piles when the foundation design has been finalized and a driving system has been selected by the contractor

7 3 EXISTING CRIBBING/SEAWALL STRUCTURE

We reviewed construction drawings for the harbor wall along Waterfront Park provided by Walker Macy Based on the sheet titled "Details of Harbor Wall for the Construction of Front St Interceptor Sewer" dated 1928, the harbor wall includes a stepped timber cribbing structure bearing at an elevation of approximately -30 5 feet (City datum), or a depth of approximately 63 feet bgs The cribbing structure is about 42 feet wide at its base and includes sheet piling extending to an unstated depth at the waterside base of the structure A concrete seawall is constructed on top of the cribbing structure

Based on the base depth from the drawing, the cribbing structure is likely bearing on very dense Troutdale Formation gravels This unit will provide a very high bearing capacity Provided the integrity of the cribbing and seawall is sound, the structure should adequately support the relatively light increased load from the overlook deck

8. SEISMIC DESIGN

Seismic design parameters based on the International Building Code (IBC) and Oregon Structural Specialty Code (OSSC) were provided in our original report Liquefaction and related effects, including lateral spread, were also discussed In our report we stated that vertical settlements estimated by others at nearby locations have ranged from less than an inch to up to 5 inches Lateral spreading estimates by others have been reported to vary between about 2 inches to as much as 2 feet The subsurface conditions encountered for this report concur with those findings Pile supported structures designed as described above are expected to exhibit minimal deflections due to liquefaction and lateral spread

9 OTHER CONSIDERATIONS

9 1 CORROSION POTENTIAL

We did not complete laboratory tests to evaluate the corrosion potential of the site soils Based on the soil type and fluctuating water table, we expect that there is a moderate risk of corrosion at this site to steel piles used for foundation support

10 CONSTRUCTION OBSERVATIONS

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration Recognition of changed conditions often requires experience, therefore, the project geotechnical engineer or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated

We recommend that the project geotechnical engineer or their representative be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork and foundation construction are being met. In particular, we recommend that site stripping, overexcavation and foundation subgrades be observed by Pacific Geotechnical prior to placing any fill or backfill Compaction of all structural backfill and slab and pavement subgrades should be tested by Pacific Geotechnical to confirm that the specified compaction is met

11. REFERENCES

Details of Harbor Wall for the Construction of Front Street Interceptor Sewer, prepared for the City of Portland, final revision dated September 24, 1928

Geotechnical Baseline Report, West Side CSO Tunnel, Shafts, Pump Station, & Pipelines, prepared by Parsons Brinkerhoff for the City of Portland Bureau of Environmental Services, dated May 28, 2002, (2002a)

Final Geotechnical Data Report, West Side CSO Tunnel, Shafts, Pump Station, & Pipelines, prepared by Parsons Brinkerhoff for the City of Portland Bureau of Environmental Services, dated May 28, 2002, (2002b and appendices)

12. LIMITATIONS

We have prepared this report for use by Walker Macy, the City of Portland Bureau of Parks and the design team for construction of the PSM cover structure and overlook deck in Waterfront Park in Portland, Oregon Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared No warranty, express or implied, should be understood

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document The original document is stored by Pacific Geotechnical and will serve as the official document of record

13 CLOSING

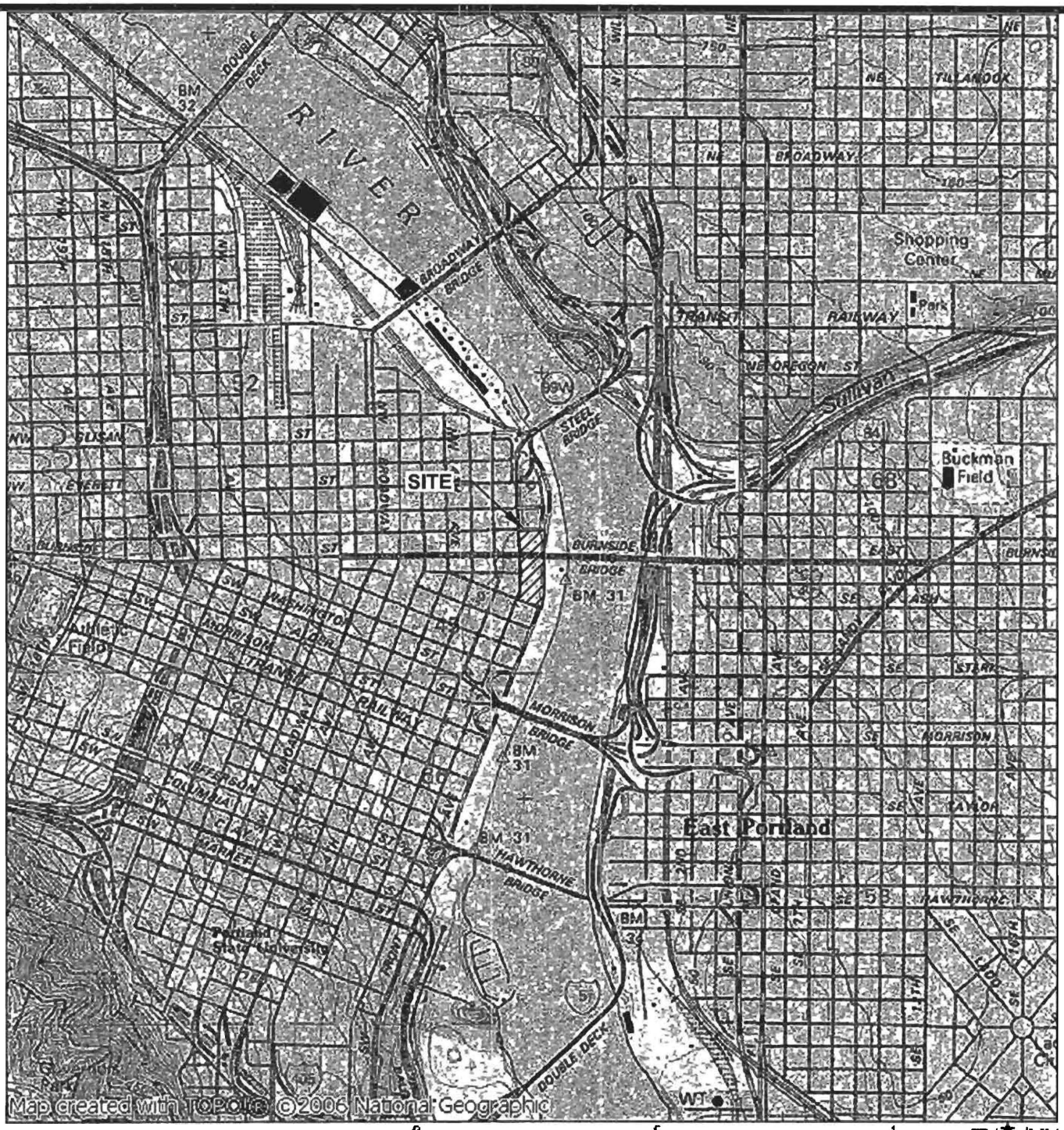
We appreciate the opportunity to submit this report to you Please contact us if you have any questions or need additional information

Sincerely,

Greg A Landau, PE
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
77490PE
OREGON
NOV. 8, 2005
GREG ALAN LANDAU
EXPIRES: 12 31 08

Attachments
Document ID 1050-001-00R2 doc



NATIONAL GEOGRAPHIC

0 5 1 MILES

1000 0 1000 2000 3000 4000 5000 FEET

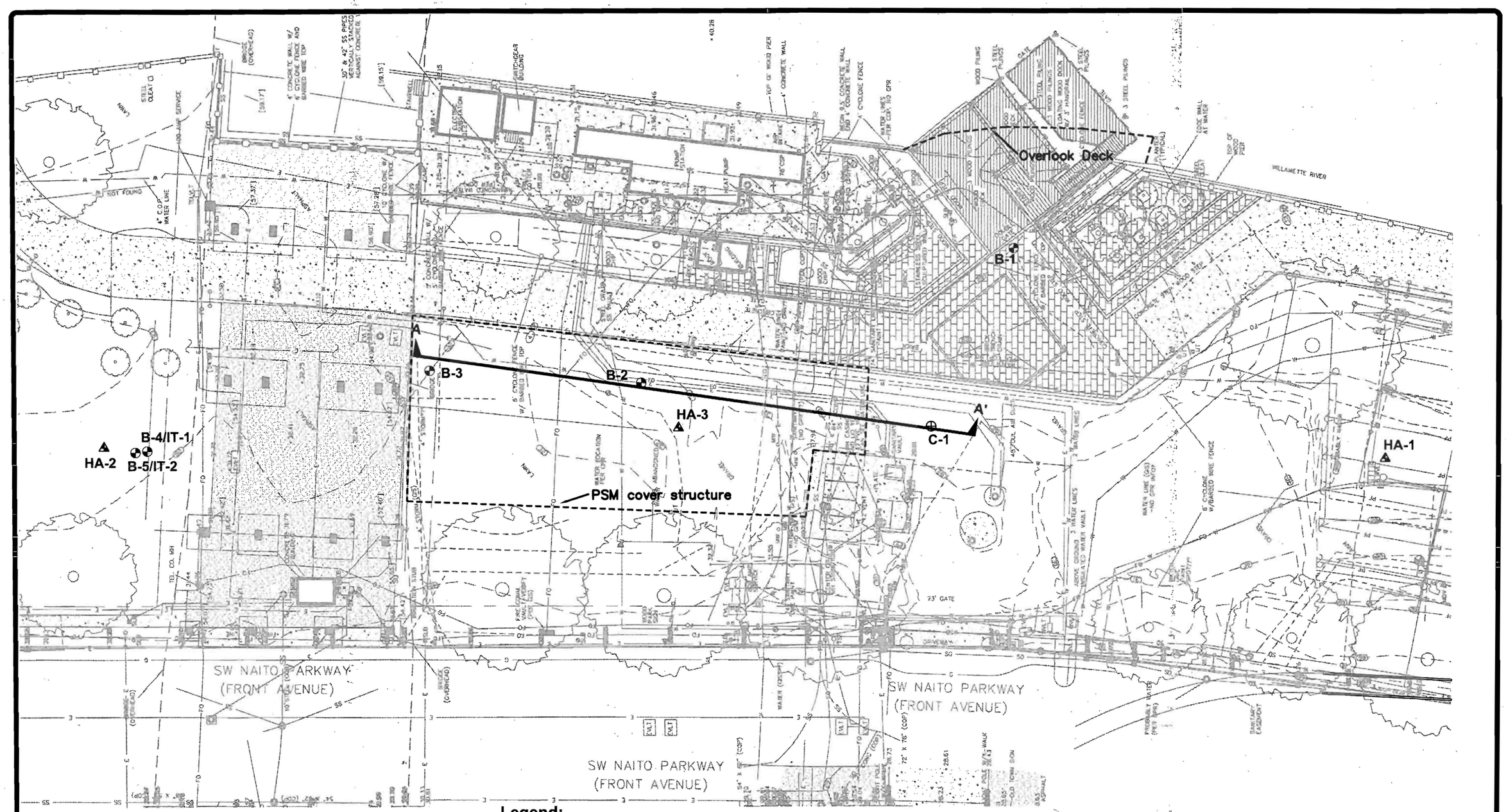
1 5 0 KILOMETERS 1

1000 0 METERS 1000

TN MN 16½

02/07/08

DWG NAME	Vicinity Map		
DESCRIPTION	Portland Waterfront Park Saturday Market Relocation		
JOB NO	1050-001-00	DATE	02/07/08
FILE NAME	105000100F1 dwg	FIGURE	1



Legend:

B-1	Explorations by Pacific Geotechnical
C-1	Exploration by Others (See Appendix C)
HA-1	Approximate Hand Auger Location (previous work by Pacific Geotechnical)
A — A'	Approximate Cross Section Location
- - - -	Approximate Location proposed structures

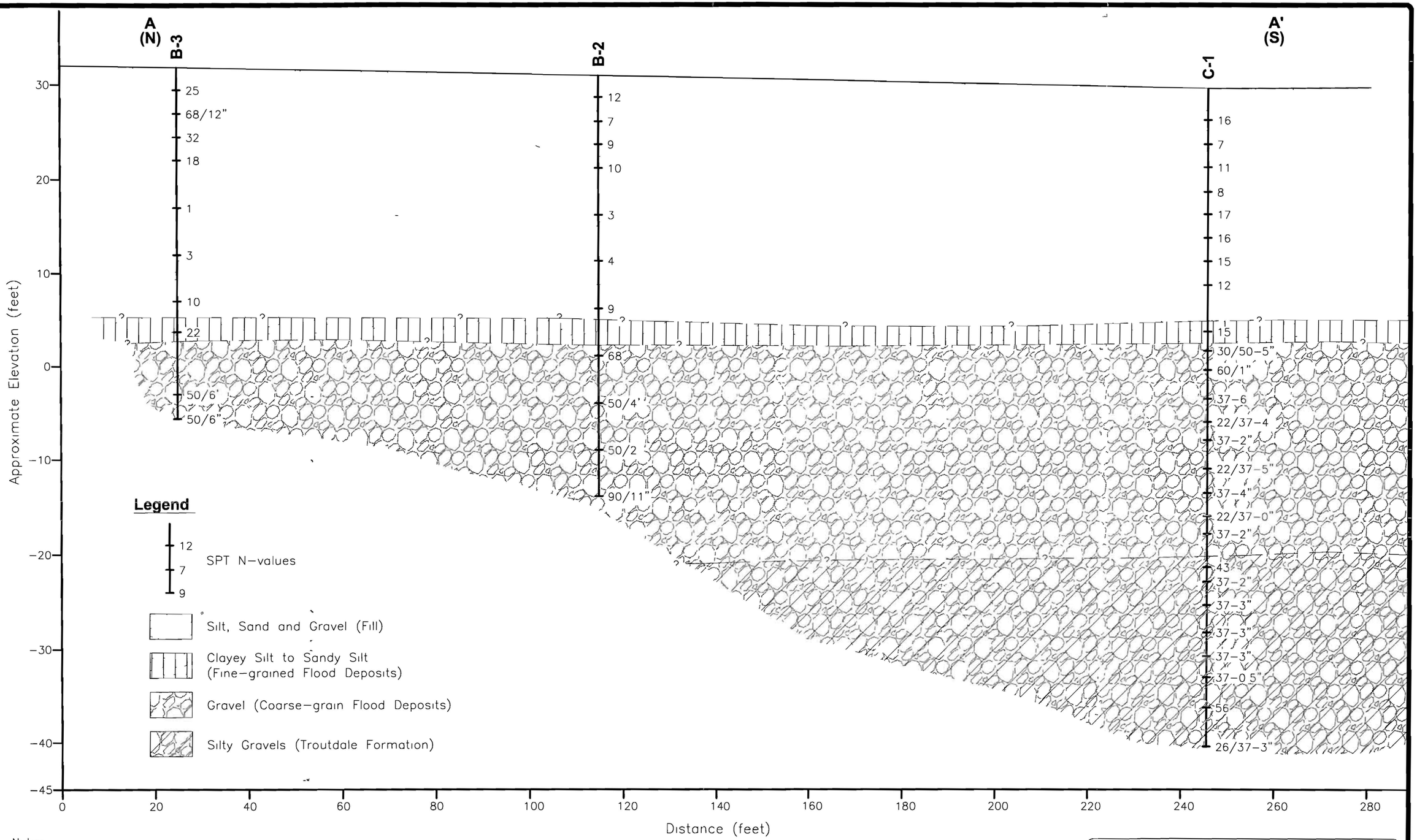
N

40 0 40

APPROXIMATE SCALE IN FEET

All Features shown are approximate.
Base drawing was an AutoCAD file provided by Walker/Macy.

DWG NAME:	Site and Exploration Plan		
DESCRIPTION:	Portland Waterfront Park Saturday Market Relocation		
JOB NO.:	1050-001-00	DATE:	02/13/08
FILE NAME:	105000100F2.dwg	FIGURE	2



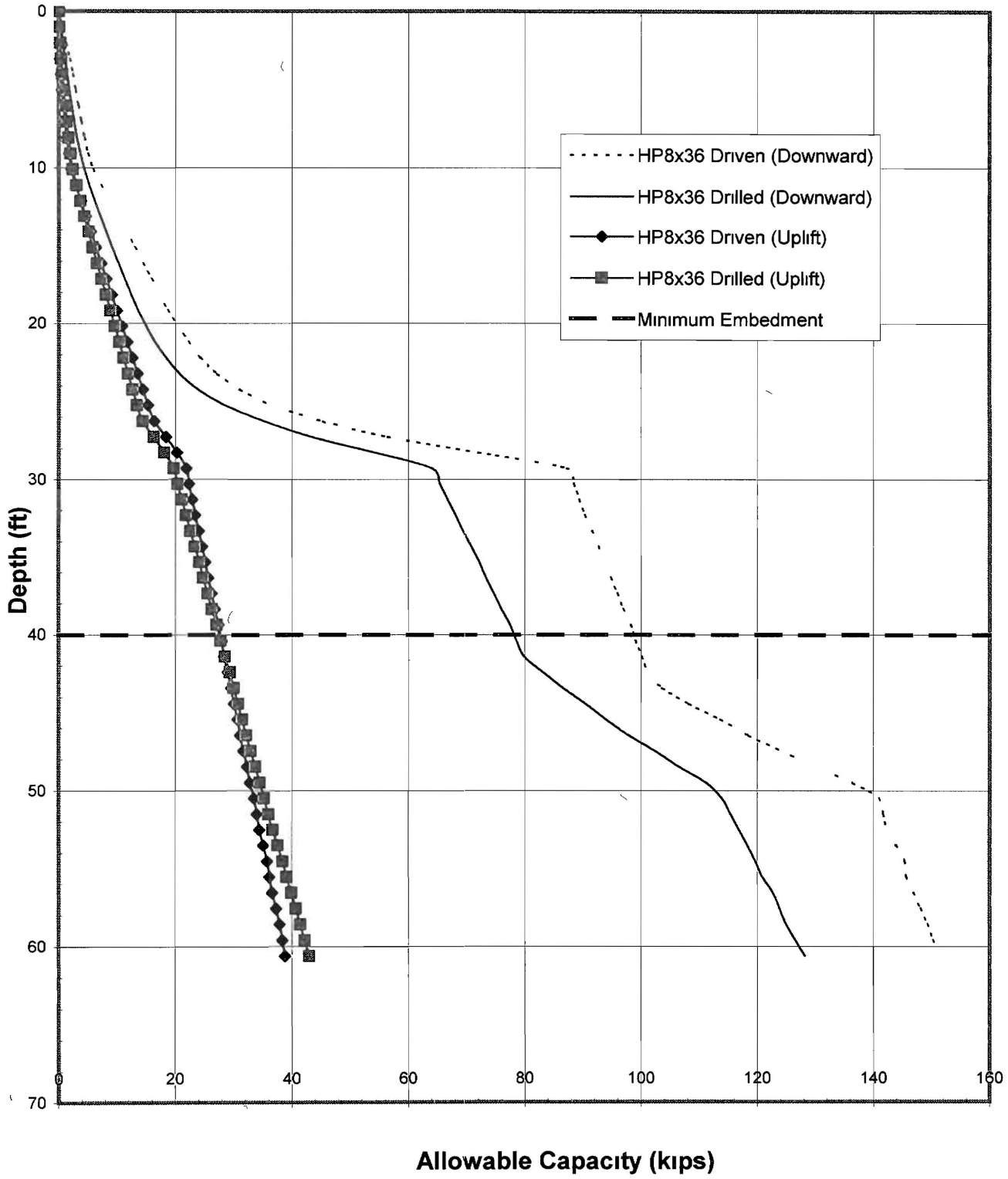
Notes
This figure is a general representation of soil conditions for informational purposes only, and should not be used for bidding purposes Refer to attached Geotechnical Report and exploration logs for more detailed information

All locations and dimmensions are approximate

Base drawing was a sketch provided by Pacific Geotechnical staff

DWG NAME	Cross Section A-A'		
DESCRIPTION	Portland Waterfront Park Portland Oregon		
JOB NO	1050-001-00	DATE	02/13/08
FILE NAME	105000100F3 dwg	FIGURE	3

Allowable Pile Capacity vs Depth



Project No 1050-001-00

Figure 4
Axial Load vs Depth

APPENDIX A
SUBSURFACE EXPLORATIONS BY PACIFIC GEOTECHNICAL

APPENDIX A
SUBSURFACE EXPLORATIONS

We evaluated subsurface soil and groundwater conditions by completing five borings at the approximate locations shown on Figure 2 of the report text The boring locations were approximately located by pacing from existing site facilities Exploration locations should be considered accurate only to the degree implied by the methods used

The borings were drilled using a truck-mounted drill rig provided by Subsurface Technologies of Banks, Oregon Soil samples were obtained from the explorations using one of the following methods

1. Standard Penetration Tests (SPT) were completed in general conformance with ASTM Test Method D1586, "Standard Method for Penetration Test and Split-Barrel Sampling of Soils" The sampler was driven with a 140-pound auto-trip hammer falling 30 inches The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the boring logs Disturbed samples were obtained from the split barrel for subsequent classification and index testing
2. Relatively undisturbed samples were obtained using a Dames & Moore Type-U sampler The sampler was driven using a 140-pound hammer falling 30 inches, just as with the SPT samples The penetration resistance recorded on the boring logs have been reduced by 50 percent for general correlation with the SPT blow counts Samples retained from the split barrel consist of up to six, 1-inch-high by 2 48-inch-diameter brass rings Disturbed rings were generally not retained

Materials encountered in the explorations were classified in the field in general accordance with ASTM Standard Practice D2488, "Standard Practice for the Classification of Soils (Visual-Manual Procedure)" Soil classifications and sampling intervals are shown in the exploration logs in this appendix

The field explorations were coordinated by an engineering geologist from our staff, who located the explorations, classified the various soil units encountered, obtained representative soil samples for geotechnical testing, observed and recorded groundwater conditions, and maintained a detailed log of each exploration Exploration logs are included in this appendix Results of the laboratory testing are indicated on the exploration logs and are discussed in Appendix C

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
1419 Washington Street Suite 101
Oregon City Oregon 97045

LOG OF BORING NO. B-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Graphic Log	Material Description	Water Content, %	Other Tests and Notes
				20 40 60 80			10 30 50 70 90	
						SW Brown gravelly SAND, fine to coarse sand, fine rounded gravel (moist, loose) [Fill]	18	
	10	1	SPT	9				
5	8	2	SPT	7		Grades to with layers of SILTY SAND with gravel	18	
	5	3	SPT	4		Grades with trace silt		
10	6	4	SPT	4			16	
15	4	5	SPT	3		Grades with trace to some gravel (very loose)		
20	2	6	SPT	32		GP Brown POORLY-GRADED GRAVEL with trace fine sand Rounded gravel to 1" maximum dimension (moist to wet, medium dense to dense)[Fill]	8	Interpreted as fill based on depth
25	3	7	SPT	30				
30							20	

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001 00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	5	8	SPT	10	Grades to loose		Lost circulation and mud Sampler possibly driven through slough May be denser than blow counts suggest
35	6	9	SPT	10			
40	10	10	SPT	24	Grades with fine to coarse sand (medium dense to dense)	13	
							Regain circulation at 43ft
45	0	11	SPT	47			No recovery
50	9	12	SPT	21	GP SP Sandy GRAVEL to gravelly SAND Fine to medium sand Fine rounded gravel (wet, dense)[Fill]	15	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
55	6	13	SPT	28	SM Gray SAND with gravel and silt Fine to medium sand Fine rounded gravels (wet, medium dense to dense)[Fill]	20	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
60							

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050 001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	1	14	SPT	42		SP Brown SAND with occasional fine rounded gravels (wet, very dense) [Troutdale Formation]		Hard drilling at 61ft
65	4	15	SPT	100				50 blows for 5
70	15	16	SPT	56		Becomes dark brown with some yellow		
	6	17	SPT	100		GP Brown POORLY-GRADED GRAVEL with fine to medium sand (wet, very dense)		50 blows for 4 5
75						Boring completed at 73ft bgs Groundwater not determined due to drilling method used		
80								
85								
90								

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

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LOG OF BORING NO. B-2

Project Name Ankeny Plaza
Location Waterfront Park, Portland OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
					GP-GM Dark gray POORLY-GRADED GRAVEL with sand and silt to SILTY GRAVEL Coarse angular basalt crushed rock (wet, medium dense)[Fill]		Boring relocated 10 feet west due to hitting abandoned sewer pipe
	10	1	SPT	12		15	
5	4	2	SPT	7	Grades to loose		
	10	3	SPT	9	SP-SM Brown to gray-brown POORLY-GRADED SAND with silt to SILTY SAND with gravel and wood debris (moist, loose)[Fill]	20	
10	12	4	SPT	10	SM Dark gray-brown SILTY SAND with gravel (wet, loose to medium dense) [Fill]		
15	4	5	D+M	4	ML Gray-brown and mottled brown SILT with fine sand Low to medium plasticity (moist, soft)[Fill]	37	DD= 84 pcf
20	16	6	SPT	4	Grades to dark gray with medium plasticity and occasional bricks	34	%F= 75
25	12	7	SPT	9	Grades to SILT with fine sand to fine sandy SILT (stiff)[Fill or Fine-grained Flood Deposits?]	37	
30						24	

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-2

Project Name Ankeny Plaza

Location Waterfront Park, Portland OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	14	8	SPT	68		GM Dark gray SILTY GRAVEL with trace coarse sand and with angular rock (wet, very dense)[Coarse-grained Flood Deposits]		Very slow drilling at 30ft to 35ft
35	4	9	SPT	100				50 blows for 4"
						Grades to dark gray brown SILTY GRAVEL with fine to medium coarse sand and rounded basalt gravels to 1" in maximum dimension		
40	3	10	SPT	100				50 blows for 2'
45	6	11	SPT	100			14	90 blows for 11
						Boring completed at 46 ft bgs Groundwater not determined due to drilling method used		
50								
55								
60								

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-3

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Other Tests and Notes
	13	1	SPT	25	SP Brown gravelly SAND to SAND with gravel Fine to medium sand Fine rounded gravel (moist, medium dense) [Fill]	
5	14	2	SPT	100	Grades with brick and concrete debris	50 blows for 6" due to concrete debris
	5	3	SPT	32	GM Gray SILTY GRAVEL with sand and occasional wood debris Fine to medium sand Fine to coarse rounded to subangular gravel (moist to wet, dense)[Fill]	
10	6	4	SPT	18		
15	4	5	SPT	1	ML Gray SILT with fine sand, clay and wood debris Low to medium plasticity (moist, very soft to medium stiff)[Fill]	%F= 79
20	0	6	D+M	3		No recovery due to wood in sampler shoe
25	0	7	D+M	10		No recovery
	26	8	ST		SM Gray silty fine SAND (moist, medium dense)[Fine-grained Flood Deposits]	
30	9	9	SPT	22		

Water Content, % (10 30 50 70 90): 13, 20, 35, 39, 43, 38

Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
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Oregon City Oregon 97045

LOG OF BORING NO. B-3

Project Name Portland Waterfront Park

Location Portland, OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						GP Gray GRAVEL with sand and trace silt Fine to coarse sand Fine to coarse rounded gravel (moist, dense) [Coarse-grained Flood Deposits]		
35	10	10	SPT	100		Grades to very dense		50 blows for 6
	8	11	SPT	100				50 blows for 6 '
						Boring completed at 38 ft bgs Groundwater not determined due to drilling method used		
40								
45								
50								
55								
60								

Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
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Oregon City Oregon 97045

LOG OF BORING NO. B-4/ IT-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						ML Brown SILT (moist, soft) [Fill]		
						GM Dark gray silty GRAVEL with much rounded gravel and trace to some asphalt		
		1	Grab					
5						Boring completed at 5 ft bgs Infiltration test performed at 5 5 ft bgs		
10								
15								
20								
25								
30								

Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 5ft
Drilled By G Sandstrom
Logged By G Sandstrom

Drilling Method Hand auger
Sampling Method SPT
Auger Data 6" HSA
Hammer Data
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
1419 Washington Street Suite 101
Oregon City Oregon 97045

LOG OF BORING NO. B-5/ IT-2

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						ML Brown SILT (moist, soft) [Fill]		
						GM Dark gray SILTY GRAVEL with much rounded gravel and trace to some asphalt [Fill]		
5						ML Gray SILT with gravel and occasional wood debris [Fill]		
10		1	SPT	18				Wood debris in sampler shoe
						Boring completed at 9 5 ft bgs Infiltration test performed at 9 75ft bgs		
15								
20								
25								
30								

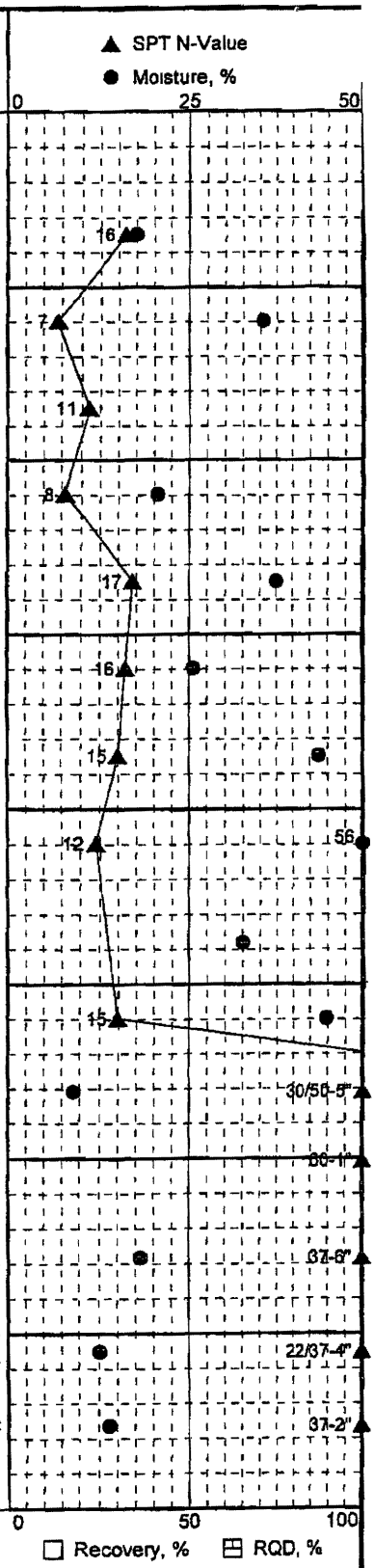
Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 10 ft
Drilled By J Lawes
Logged By J Lawes

Drilling Method Hand auger
Sampling Method SPT
Auger Data 10" HSA
Hammer Data
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

APPENDIX B
SUBSURFACE EXPLORATIONS BY OTHERS

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth in Feet	Samples
		30 0				
		29 2	**PAVEMENT**, 2 5 inches of brick over 10 inches of steel reinforced concrete (Pavement)		0	
		0 9	**SAND FILL**, medium dense, gray, moist, fine-grained, trace fine rounded gravel and crushed rock (Fill)			S-1
		26 0				
		4 0	**Sandy SILT FILL**, slightly clayey, loose, light brown, moist, no to low plasticity, scattered fine rounded gravel (Dredged sand fill)		5	S-2
						S-3
		20 0			10	
		10 0	**SAND FILL**, loose, gray brown moist, fine-grained, scattered fine rounded gravel (Dredged sand fill)			S-4
			Tree fragment, medium dense			S-5
		16 0				
		14 0	**Silty SAND FILL**, slightly clayey, medium dense, gray, moist to wet, no to low plasticity, fine-grained, scattered to abundant wood fragments and red crushed rock (Fill)		15	S-6
	-200 = 33%					S-7
▼	08/28/2000 -200 = 35%				20	S-8
	-200 = 16%					S-9
		5 0			25	
	-200 = 73%	25 0	**Clayey SILT** stiff, gray, wet, low plasticity, micaceous (Recent Alluvium)			S-10
		3 0				
		27 0	**Sandy GRAVEL**, very dense, brown and gray wet, fine- to coarse-grained, subrounded gravel, sand is fine- to coarse-grained, scattered cobbles (Pleistocene Flood Deposits - Qfch)			S-11
					30	S-12*
						S-13
					35	S-14
						S-15



LEGEND

- ⌶ = 2 0" O D Split Spoon Sample
- ⌶ = 3 0" O D Thin-Walled Sample
- * = Sample Not Recovered
- ▩ = Grab Sample Drill Cuttings
- ▮ = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- ▽ Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

FHA

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1

page 1 of 2

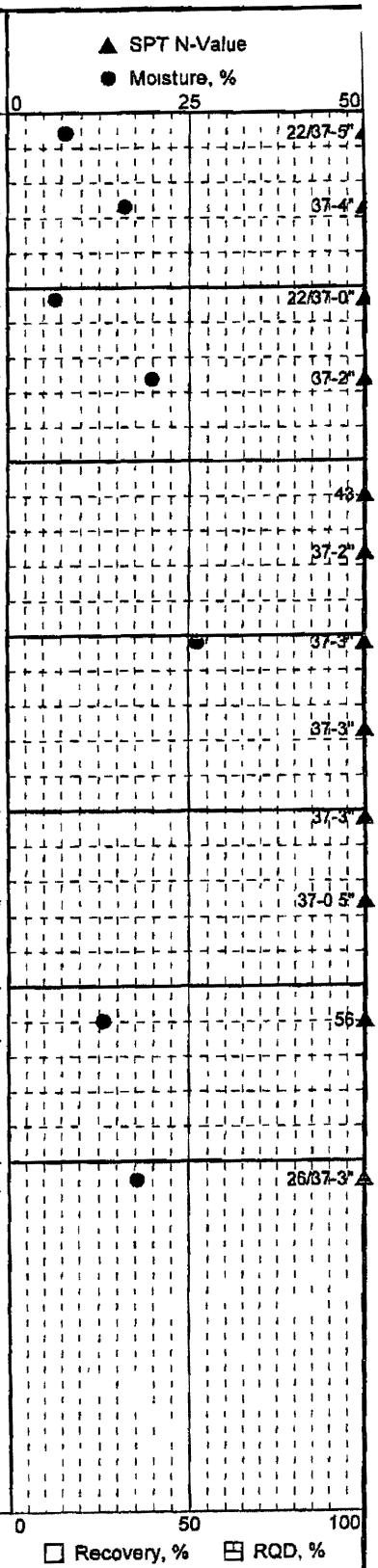
December 2000 *F-3112 01*

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland Oregon

FIG

WLG ANKPS GPJ 12/6/00

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Sandy GRAVEL continued			S-16, S-17
					45	S-18, S-19
		-18 5 48 5	**SAND**, very dense, no sample return (Pleistocene Flood Deposits - Qfch)			
					50	S-20*
		-21 0 51 0	**Clayey Gravelly SAND**, very dense, yellow-brown, moist, low plasticity, sand is fine-grained, fine- to coarse-grained, rounded gravel, weathered, overconsolidated, poorly graded, gradational transition to underlying material (Troutdale Formation Tt)			S-21*
					55	S-22, S-23*
					60	S-24*, S-25*
		-35 0 65 0	**Gravelly SAND**, very dense, light brown, wet, non-plastic, sand is fine-grained, fine- to coarse-grained, rounded gravel, overconsolidated, poorly graded (Troutdale Formation Tt)		65	S-26
		-40 8 70 8	Bottom of Boring, Completed 8/29/00		70	S-27



LEGEND

- I = 2 0" O D Split Spoon Sample
- II = 3 0" O D Thin-Walled Sample
- * = Sample Not Recovered
- = Grab Sample Drill Cuttings
- = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1
page 2 of 2

December 2000 *F-3112 01*

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland Oregon

FIG

FHA

WLG ANKPS GPJ 12/6/00

APPENDIX C
LABORATORY TESTING

APPENDIX C
LABORATORY TESTING

GENERAL

Soil samples obtained from the explorations were transported to our laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils encountered Representative samples were selected for laboratory testing The tests were performed in general accordance with the test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures

VISUAL CLASSIFICATIONS

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the Unified Soil Classification System (USCS) and ASTM classification methods ASTM Test Method D2488 was used to classify soils using visual and manual methods ASTM Test Method D2487 was used to classify soils based on laboratory test results

Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D2216 The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in Appendix A

Dry Density

Dry density tests were completed on low-disturbance samples obtained with the Dames & Moore (D&M) sampler The tests were conducted in general accordance with ASTM Test Method D2937 Dry density test results are presented on the boring logs in Appendix A

Percent Fines

Fines content analyses were performed to determine the percentage of soils finer than the No 200 sieve - the boundary between sand size particles and silt size particles The tests were performed in general accordance with ASTM D1140 The test results are indicated on the individual boring logs in the column labeled "Other Tests and Notes"

Pacific Geotechnical, LLC
Geotechnical Engineering and Engineering Geology with a Focus on Sustainability

March 6, 2008

Walker Macy
111 SW Oak, Suite 200
Portland, Oregon 97204

Attention Mrs Chelsea McCann

Subject Seismic Site Hazard Report
Portland Saturday Market Cover Structure
Portland, Oregon
Project No 1050-001-00

08-113505-00

1 INTRODUCTION AND PROJECT UNDERSTANDING

Pacific Geotechnical, LLC (Pacific Geotechnical) is pleased to submit this seismic site hazard report (SSHR) for the Portland Saturday Market (PSM) cover structure for the above project in Portland, Oregon Pacific Geotechnical completed a geotechnical report for the project dated April 12, 2007, and follow up technical documents including a supplemental geotechnical report for pile foundations to support the PSM structure, dated February 18, 2008 The location of the site is shown on Figure 1

The project consists of relocating the PSM into Waterfront Park Construction associated with this move will include a new cover structure and overlook deck, reconfiguration of some pedestrian ways, installation of an interactive water feature, and appurtenant facilities such as piping and benches The PSM cover structure will be located just south of the Burnside Bridge, as shown in Figure 2

According to the project architect, the PSM cover structure is classified as an A3 assembly space with a maximum occupancy load of 1,698 people Based on this description and a discussion with City of Portland (City) Bureau of Development (BDS) staff, an SSHR is required for the PSM cover structure

2 PURPOSE AND SCOPE OF SERVICES

The purpose of our work is to provide an SSHR for the project Our specific scope of work included the following

- Reviewed applicable codes to determine whether an SSHR is required

- Reviewed existing available information, including subsurface information collected for our supplemental geotechnical report
- Used the above collected information to complete an SSHR in accordance with Section 1802 of the 2007 Oregon Structural Specialty Code (OSSC)
- Provided a total of five copies of this report to you Four copies are provided for your submittal to the City and one copy is provided for your submittal to the Department of Geology and Mineral Industries (DOGAMI), as required by the 2007 OSSC

3. SITE DESCRIPTION

3 1 REGIONAL AND LOCAL GEOLOGY

The *Geotechnical Baseline Report, West Side CSO Tunnel, Shafts, Pump Station and Pipelines Project* (Parsons Brinckerhoff, 2002a) contains a well-crafted explanation of the regional geologic setting of the project area as well as the history of local faulting and geomorphology (landforms) The following paragraphs briefly summarize the information presented in that report

The project is located along the west edge of the Willamette River, a tributary of the Columbia River that flows through the Portland Basin This pull-apart basin, a product of regional tectonic stresses related to the subduction of the Juan de Fuca plate with the over-riding North American plate, has been subsiding since at least Miocene time Tens of hundreds to thousands of feet of alluvial clay, silt, sand and gravel have been deposited in the Portland Basin between intervals of uplift, scour and erosion that have created extensive topographic relief in the basin sediments Massive "Missoula" floods scoured and then deposited sediment ranging from clay to gravel in the Portland Basin during the latest Pliocene to latest Pleistocene (~1 million to 12,500 years ago) Post-Pleistocene geologic activity in the basin includes Willamette River fluvial erosion and deposition of sediment as well as the placement of large quantities of man-made fill along the Willamette riverfront and in adjacent sloughs, lakes and bottomlands

The regional Columbia River Basalt (CRB) bedrock that forms the Portland Basin and adjacent West Hills (Tualatin Mountains) was formed 5 to 15 million years ago by an immense outpouring of lava from vents along the Oregon/Washington-Idaho border that covered this portion of the Pacific Northwest to depths of thousands of feet with igneous rock As the basalt-floored Portland Basin subsided, sediment eroded both from the nearby Cascade Mountain chain as well as from continental interior sources was carried into the basin by the Columbia and other, local rivers and deposited as the Plio-Pleistocene Sandy River Mudstone (SRM) and overlying Troutdale Formation sandy to silty gravel Alluvial processes ranging from the catastrophic floods to everyday fluvial activity of local rivers and streams eroded channels in, as well as deposited additional sediment on, the SRM and the Troutdale The material related to the Missoula Flooding is generally referred to as "flood-deposits" and is further characterized by grain size as "coarse-grained" or "fine-grained" flood deposits A veneer of Willamette River alluvium derived from Willamette Valley sources to the south as well as reworked flood deposit sediments mantle the topography the Willamette has incised into the last flood-created surface Since at least the late 1800's human activity has extensively altered the Willamette River shore and created unique and variable shallow soil conditions in the Portland area, including the project area specifically

3 2 SURFACE CONDITIONS

The project site is located in Waterfront Park just south of the Burnside Bridge The surface conditions in the vicinity of the proposed PSM cover structure include landscape grasses at the north end and a construction area related to the West Side CSO (WCSO) at the south end The construction area is fenced off and encloses the existing WCSO Ankeny Shaft and a vault structure

3 3 SUBSURFACE CONDITIONS

Subsurface conditions encountered by others in and around the project area are summarized in our original report The following paragraphs describe the subsurface conditions encountered in explorations we completed for pile design for the PSM cover structure and reference other nearby explorations as needed

We explored subsurface conditions at the PSM cover structure site by drilling three borings at the locations shown on Figure 2 The borings were completed to between 38 to 73 feet below the ground surface (bgs) to evaluate subsurface conditions for foundation analysis and design

A geologic cross-section through the proposed PSM cover structure location based on our borings is provided as Figure 3 Appendix A summarizes the exploration methods and presents the exploration logs Pacific Geotechnical completed for the pile design Appendix B presents an exploration log completed by others in the immediate vicinity of the proposed cover structure Results of our laboratory testing are indicated on the exploration logs in Appendix A and described in Appendix C

Subsurface conditions were found to be relatively uniform across the site In general, we encountered 1) silt, sand and gravel fill material, over 2) a thin layer of Fine-grained Flood Deposits, over 3) Coarse-grained Flood Deposits, over 4) Troutdale Formation gravels The conditions encountered in our explorations generally match the geologic mapping described in our original report and found by others in the area The following paragraphs summarize the above units

3 3 1 Fill

We encountered variable fill material in all of our borings The fill material generally included loose to dense sand and gravel and very soft to stiff silt We also encountered occasional crushed rock, asphalt, concrete, brick and wood debris The thickness of the fill material varied from about 27 feet in boring B-3 to about 60 feet in boring B-1, adjacent to the seawall structure Although the later thickness places fill below original grades and water levels, it is supported by a drawing for the seawall/cribbing structure that shows the base of the cribbing at an elevation of -30 5 feet (City of Portland datum), or about 63 feet bgs It appears the original soil was removed for construction of the crib wall and then backfilled during construction of the wall The moisture content of the fill varied between 8 to 39 percent The percent fines in silty fill was between 75 to 79 percent, based on two tests A dry density of 84 pounds per cubic foot (pcf) was measured on a single sample

3 3 2 Unconsolidated Silt and Sand (Alluvium/Fine-grained Flood Deposits)

We encountered a thin layer of silty fine sand in boring B-3 under the fill material We did not encounter this material in our other explorations, however, this unit is noted in boring C-1 completed by Fujitani Hilts & Associates (FHA) (PB 2002b, Appendix G) This material was medium dense with a moisture content between 38 and 43 percent, based on two tests

3.3 3 Unconsolidated Sand, Silt and Gravel (Alluvium/Coarse-grained Flood Deposits)

We encountered Coarse-grained Flood Deposits under the Fine-grained Flood Deposits in boring B-3 and below the fill material in boring B-2 We did not encounter this material in boring B-1, as the fill material extended to the top of the Troutdale Formation gravels (see below) This unit generally consisted of very dense gravel with variable amounts of sand and silt The moisture content was measured at 14 percent from one sample tested

3 3 4 Consolidated Sand and Gravel (Troutdale Formation)

We encountered gravels of the Troutdale Formation beneath the fill in boring B-1 at a depth of about 61 feet bgs This corresponds well with the design depth for the base of the cribbing structure supporting the adjacent seawall Boring C-1, completed by FHA just south of the proposed cover structure, encountered Troutdale Formation at a depth of 51 feet bgs This unit consists of very dense sand and gravel in a silt matrix

3 4 GROUNDWATER

Groundwater could not be determined in our explorations due to the drilling method used

Based on previous subsurface explorations completed by others, groundwater levels for the project area are commonly between 18 and 23 feet bgs This depth usually corresponds to an elevation within several feet of the mean river level, suggesting that this groundwater is connected to, and will reflect changes in, the water level of the Willamette River Our experience and the groundwater levels interpreted by others (PB, 2002a) suggest that the groundwater levels will tend to be a few feet higher than the river level

4. SEISMIC SETTING/EVALUATION

4 1 DESIGN EARTHQUAKES

Seismicity in the Willamette Valley is primarily driven by the Cascadia Subduction Zone (CSZ), which is the zone where the westward advancing North American Plate is overriding the subducting Juan de Fuca Plate The interaction of these two plates results in three potential seismic sources affecting the Willamette Valley area, shallow crustal earthquakes within the overriding North American Plate, interplate earthquakes which occur along the boundary between the Juan de Fuca and North American plates, and intraplate earthquakes which occur within the subducting Juan de Fuca plate Although capable of generating significant earthquakes, intraplate earthquakes, such as the February 28, 2001 Nisqually earthquake (M_w = 6 8) in Washington State were evaluated, but found not to control design for this area so are not further considered Crustal and interplate earthquakes are further discussed below

Figure 4 shows the size and epicentral locations of earthquakes that have occurred from 1841 through 2002 and the location of late Quaternary and Holocene age faults, within a 50-mile radius of the site

According to the USGS, the shallow crustal earthquake source contributes about 90% to the probabilistic ground motion and the interplate source zone contributes about 10% to the probabilistic ground motion for the range of periods of concern Therefore, additional data representative of each of the two source zones were gathered for use in assessing the site seismic response Again, the intraplate earthquake is not considered capable of controlling design in this area so was not considered Discussion of the shallow crustal and interplate sources are provided below

4 1 1 Shallow Crustal Earthquakes

A significant earthquake could occur on a local fault very near the site within the design life of the facility Such event would cause ground shaking at the site that could be more intense than the CSZ interplate event, though the duration would be shorter

Several large, northwest-trending faults are mapped within one mile of the project area, including the Portland Hills Fault and the East Bank Fault (Beeson and others, 1991) Random crustal faults may also occur on yet unmapped faults as well The Portland Hills Fault Zone is considered potentially active (Geomatrix, 1995 and USGS, 2007) and believed capable of generating a magnitude 6 9 or greater event In addition to these mapped faults the geotechnical data report for the West Side CSO project (Parsons

Brinckerhoff, 2002b) reports a disparity in geologic unit elevations from borings located across the Willamette River from each other that infers a displacement of the CRB and SRM units as much as 130 vertical feet over this distance This suggests a previously unmapped fault along the axis of the Willamette that may have undergone as much as 0 04mm/yr displacement over the past 1 million years Alternately, this could be from past differential erosion by the river, but this could not be resolved for this study

4 1 2 CSZ Interplate Earthquakes

The CSZ interplate source zone generally extends along the coast from northern California to British Columbia and is characterized by researchers as being capable of generating earthquakes of magnitude 8 to magnitude 9 Recurrence intervals for CSZ interplate earthquakes are thought to be on the order of 500 years, which is substantially less frequent than shallow crustal or CSZ intraplate earthquakes The most recent CSZ event is believed to have occurred in the year 1700, and paleogeologic evidence suggests five to seven interplate earthquakes may have been generated along the CSZ over the last 3500 years There is no instrumentally recorded seismicity of large interplate earthquakes for the CSZ The inferred seismogenic portion of the CSZ plate interface is roughly 120 miles (190 kilometers) west of the site

4 2 SEISMIC HAZARDS

Seismic hazards for this area have been mapped by the Department of Geology and Mineral Industries (DOGAMI) in the *Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Quadrangles, Oregon and Clark County, Washington* (Mabey and others, 1993) This map series includes hazard maps for ground amplification, liquefaction, and lateral spreading/slope instability A Relative Earthquake Hazard Map, which combines the above three earthquake hazards is also included Based on this map, the project site is within Zone C, defined as "Low to intermediate hazard" where relative earthquake hazard is mapped between Zone A (highest hazard) to Zone D (lowest hazard)

4 2 1 Fault Surface Rupture

The closest local fault is the northwest-southeast trending Portland Hills fault shown by the United States Geologic Survey (USGS, 2007) less than 0 6 miles (1 kilometer) west of the site Although it is considered potentially active, displacement of Holocene materials has not been observed Explorations we completed and extensive explorations for the WCSO did not find evidence of Holocene displacement Although the fault is capable of generating significant shaking at the site, from the lack of Holocene displacement here and elsewhere on the Portland Hills fault, it is our opinion that the possibility of surface fault rupture and related displacement at this site is low

4 2 2 Ground Motion Amplification

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long period ground motions such as those associated with subduction zone earthquakes, whereas areas underlain by shallow soil profiles are not likely to amplify seismic waves

The ground motion amplification map shows the general area of the site to be in a hazard zone with a 1 to 1 4 amplification factor These are the lowest and second-lowest amplification factor zones based on a six zone rating system An amplification factor of 2 5 or greater is the highest relative hazard and 1 the lowest

Ground motions are considered specifically in Section 4 4

4 2 3 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking

The thickness of liquefiable sediment (liquefaction hazard) is ranked in the highest hazard level, at greater than 9 meters (about 30 feet) of liquefiable soil potentially present in the area Our explorations confirm that 9 meters or more of liquefiable soils are present at the site

4 2 4 Lateral Spread

Lateral spread occurs when large blocks of ground are displaced down gentle slopes or towards stream channels as a result of liquefaction of subsurface soil during an earthquake Man-made seawalls and loose, saturated sandy fills can increase site vulnerability to this hazard, and the presence of these features is reflected in the classification of the bulk of the project area in the highest (estimated lateral ground displacement in excess of 1 2 meters, about 4 feet) and second highest (displacement of 0 9 to 4 meters, about 3 to 4 feet) hazard zones (Mabey and others, 1993)

Lateral spreading estimates by others have been reported to vary between about 2 inches to as much as 2 feet The subsurface conditions encountered for this report concur with those findings

4 2 5 Earthquake Induced Landsliding

Existing mapping (Mabey and others, 1993) places the site in the lowest category of landslide hazard Based on the flat site slopes, we concur that the landsliding hazard is low at the PSM cover structure

4 2 6 Seiche and Tsunami

The site is well away from tsunami inundation zones and away from large bodies of water that may develop seiches Seiche and tsunamis are not considered a hazard at this site

4 2 7 Settlement

Settlement due to earthquakes is most prevalent in relatively deep deposits of loose, dry, clean sand In our original report we stated that vertical settlements estimated by others at nearby locations have ranged from less than an inch to up to 5 inches Based on our explorations, these values still appear appropriate

4 3 BUILDING CODE PARAMETERS

We understand that the structure will be designed and constructed in accordance with the 2006 IBC and the 2007 OSSC The parameters provided in Table 1 are appropriate for code level seismic design Results from our site specific seismic study are discussed in Section 4 4

Table 2 Selected Earthquake Acceleration Time Histories

Earthquake, Year	Recording Station	Recorded Magnitude	Recorded Peak Horizontal Acceleration (g)	Fault Mechanism
Whittier Narrows, 1987	Mt Wilson	5 3	0 16	Crustal
North Palm Springs, 1986	Silent Valley	6 2	0 11	Crustal
Imperial Valley, 1979	Superstition Mountain	6 5	0 20	Crustal

4 4 2 Ground Motion Scaling

The response spectra analyses were performed with the computer program Shake2000 Version 3 4 using the above selected earthquake acceleration time histories These time histories were scaled to the design bedrock PGA value of 0 41g Table 2 summarizes the scaled peak ground accelerations

4 4 3 Soil Model

The input soil model used in our analysis is based on the findings of our subsurface exploration program A detailed description of site subsurface conditions is provided in Section 3 3 Shear wave velocities could not be directly measured as the subsurface contains various layers of gravel that could not be penetrated with a cone penetrometer The engineering behavior of the site soils was modeled using estimated shear wave velocities and unit weights for the anticipated soil profile along with damping and shear modulus reduction relationships typical for the soil types underlying the site Table 3 provides a summary of the soil model used in our analysis

Table 3 Soil Model

Depth Interval (feet)	Subsurface Unit	Shear Wave Velocity Range (feet per second)
0 to 40	Fill	400 to 650
40 to 155	Very Dense Sand and Gravel	650 to 1,000
> 155 feet	Basalt Bedrock	2,500

4 4.4 Design Response Spectra

Figure 5 presents the computed design acceleration response spectra appropriate for the proposed PSM cover structure These spectra are scaled to two-thirds of the maximum computed as prescribed by the IBC Additionally, the spectral acceleration at any period must be equal to or greater than the response spectrum determined as described in Section 1615 1 4 of the IBC

5 FOUNDATION DESIGN

We completed a supplemental geotechnical report for foundation support recommendations, dated February 18, 2008 In that report we recommended that the PSM cover structure be supported on driven or drilled steel piles bearing in the native gravels, or on an improved subgrade We understand that the

Table 1 Seismic Design Parameters (2006 IBC)

Seismic Design Parameters (2006 IBC)	
Building Use Category	III
Seismic Use Group	II
Seismic Importance Factor, I_E	1 25
Site Class	E
Spectral Response Acceleration S_s	0 98 g
Spectral Response Acceleration S_1	0 34 g
Site Coefficient, F_a	0 9
Site Coefficient, F_v	2 6
Spectral Response Acceleration (Short Period), S_{DS}	0 60 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0 60 g
Seismic Design Category	D

Note
[1] Site soil is classified as Site Class F based on the liquefaction hazard Site Coefficients are based on Site Class E per the 2006 International Building Code, provided the period of the structure is less than 0 5 seconds

Based on the 2002 USGS National Seismic Hazard Mapping Project, the expected peak bedrock acceleration having a 10 percent probability of exceedance in 50 years (475 year return period) is 0 19g The expected peak bedrock acceleration having a 2 percent probability of exceedance in 50 years (2,475 year return period) is 0 41g These values represent the peak acceleration on bedrock beneath the site and do not account for any ground motion amplification due to site-specific effects

4 4 SITE SPECIFIC RESPONSE ANALYSIS

4 4 1. Ground Motion Selection

Earthquake acceleration time histories used to evaluate the site response were selected based on the fault mechanism of the earthquake source zones, the soil/rock profile type at the site where the earthquake time history was recorded, the distance from the epicenter to the recording site, the PGA of the record, the predominate period, and the earthquake magnitude and energy content

Based on the proximity (and magnitude) of the Portland Hills fault and a review of literature comparing the potential impact of shallow crustal faults and the CSZ Interplate fault in the Portland metropolitan area, it is our opinion that the ground motions at the site are controlled by a crustal event For this reason, we selected three acceleration time histories for this event as input for the seismic response analysis Table 2 presents the earthquake acceleration time histories used in our analysis These recordings were determined, based on the screening criteria described above, to be compatible with the project site conditions

PSM cover structure will be supported on a combination of drilled and driven piles Our fill recommendations are provided in the February 18 report

6. REFERENCES

Geotechnical Baseline Report, West Side CSO Tunnel, Shafts, Pump Station, & Pipelines, prepared by Parsons Brinkerhoff for the City of Portland Bureau of Environmental Services, dated May 28, 2002, (2002a)

Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon and Clark County, Washington, Beeson and others, 1991

Final Geotechnical Data Report, West Side CSO Tunnel, Shafts, Pump Station, & Pipelines, prepared by Parsons Brinkerhoff for the City of Portland Bureau of Environmental Services, dated May 28, 2002, (2002b and appendices)

Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Quadrangles, Oregon and Clark County, Washington, Mabey and others, 1993

International Code Council, 2006, 2006 International Building Code

OSSC, 2007 Oregon Structural Specialty Code, accessed March, 2008 from OSSC web site http //www2 iccsafe org/states/oregon/07_Structural/Building07_Frameset htm

Geomatrix Consultants, 1995, *Seismic design mapping, State of Oregon Final Report*, prepared for Oregon Department of Transportation, 252 p

U S Geologic Survey, 2002, National Seismic Hazards Mapping Project, Seismic Hazard Curves, Response Parameters and Design Parameters, Java application, version 5 0 8

U S Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed March, 2008, from USGS web site http //earthquake usgs gov/regional/qfaults/

7 LIMITATIONS

We have prepared this report for use by Walker Macy, the City of Portland Bureau of Parks and the design team for construction of the PSM cover structure in Waterfront Park in Portland, Oregon Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance

Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared No warranty, express or implied, should be understood

Any electronic form, facsimile or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document The original document is stored by Pacific Geotechnical and will serve as the official document of record

8. CLOSING

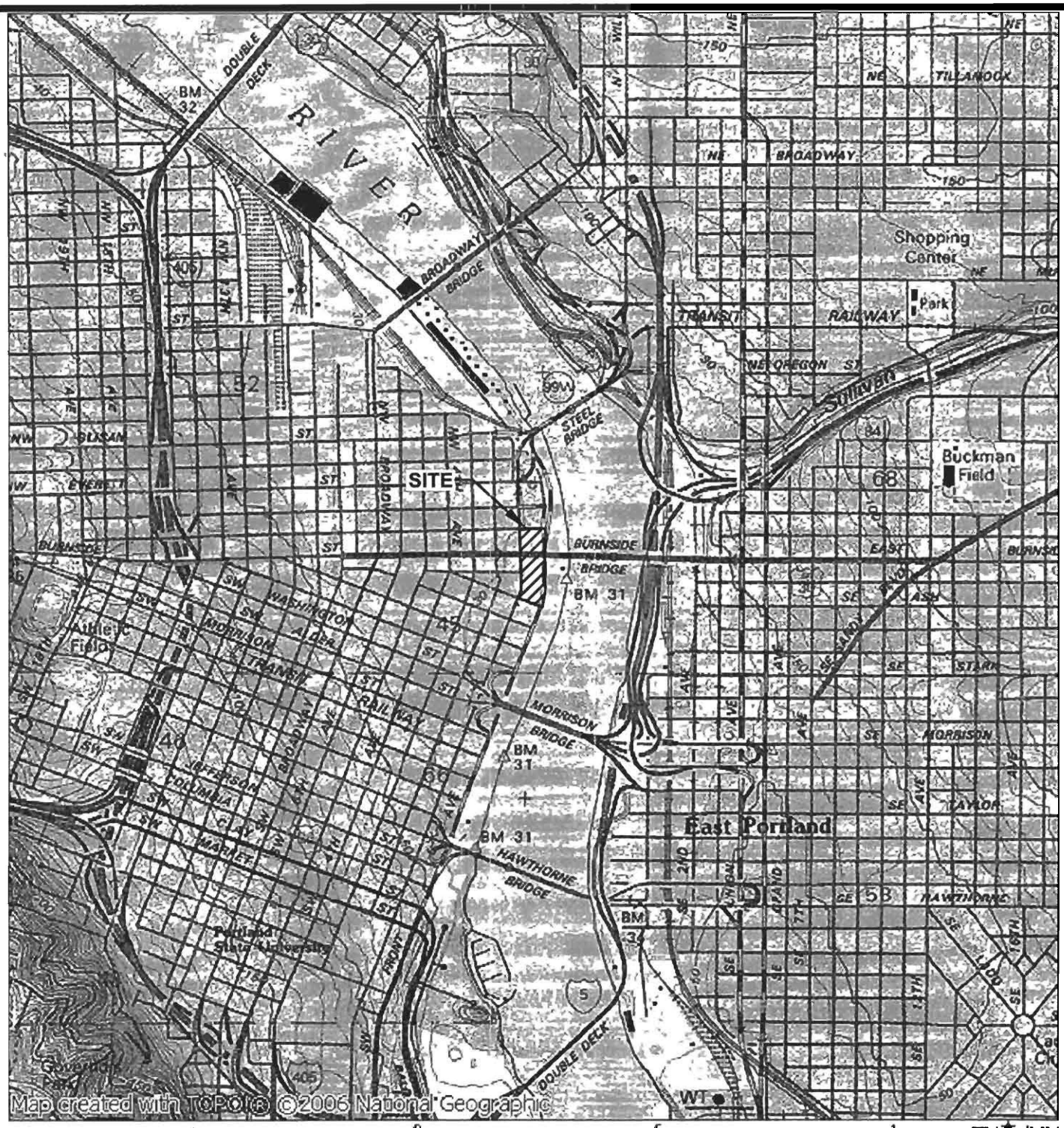
We appreciate the opportunity to submit this report to you Please contact us if you have any questions or need additional information

Sincerely,

Greg A Landau, PE
Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
77490PE
OREGON
NOV. 8, 2005
GREG ALAN LANDAU
EXPIRES: 12-31-08

Attachments
Document ID 1050-001-00R3 doc



NATIONAL GEOGRAPHIC

0 .5 1 MILES

1000 0 1000 2000 3000 4000 5000 FEET

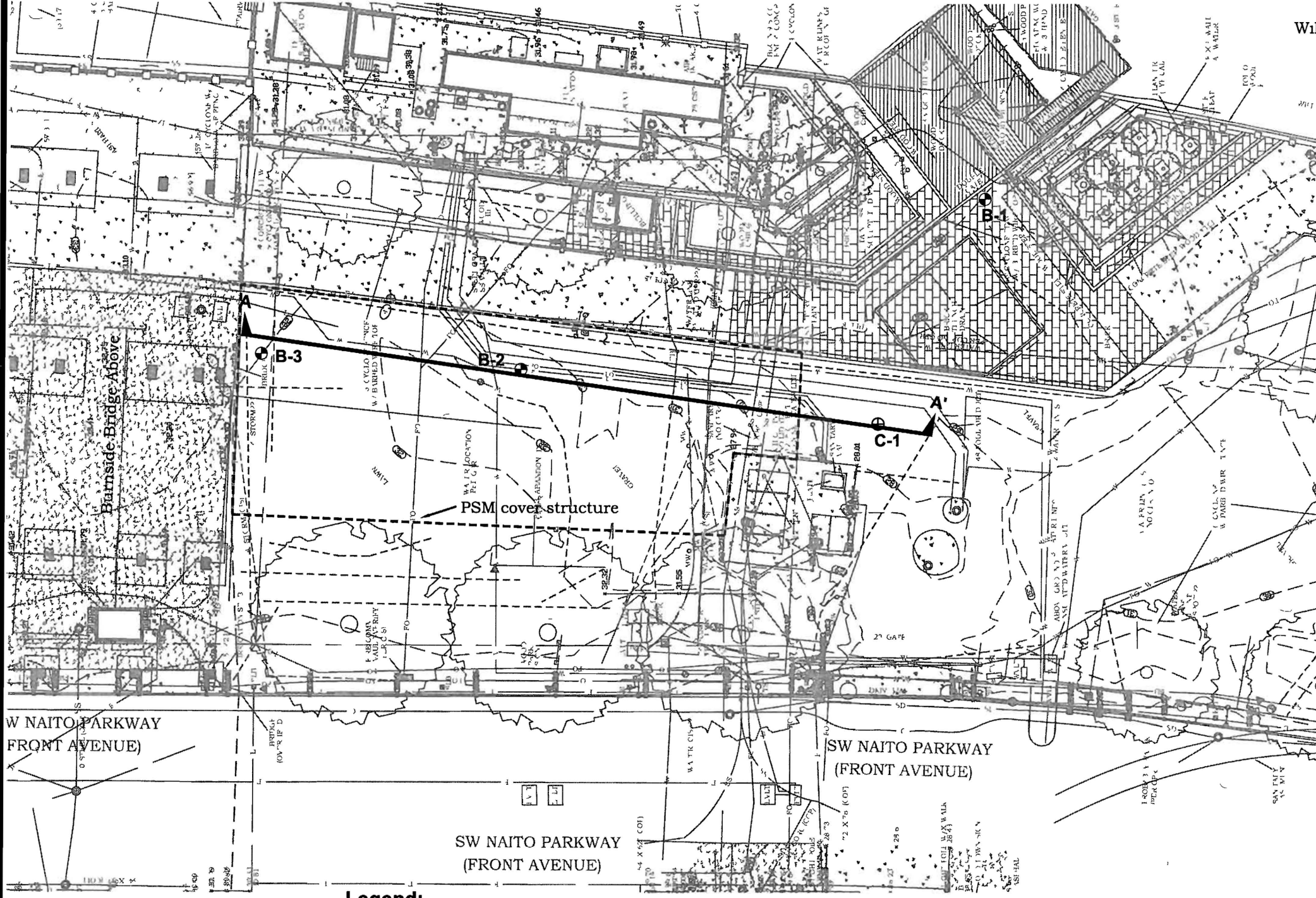
1 .5 0 KILOMETERS 1

1000 0 METERS 1000

TN MN 16½

02/07/08

DWG NAME	Vicinity Map		
DESCRIPTION	Portland Waterfront Park Saturday Market Relocation		
JOB NO	1050-001-00	DATE	02/07/08
FILE NAME	105000100F1 dwg	FIGURE	1



Legend:

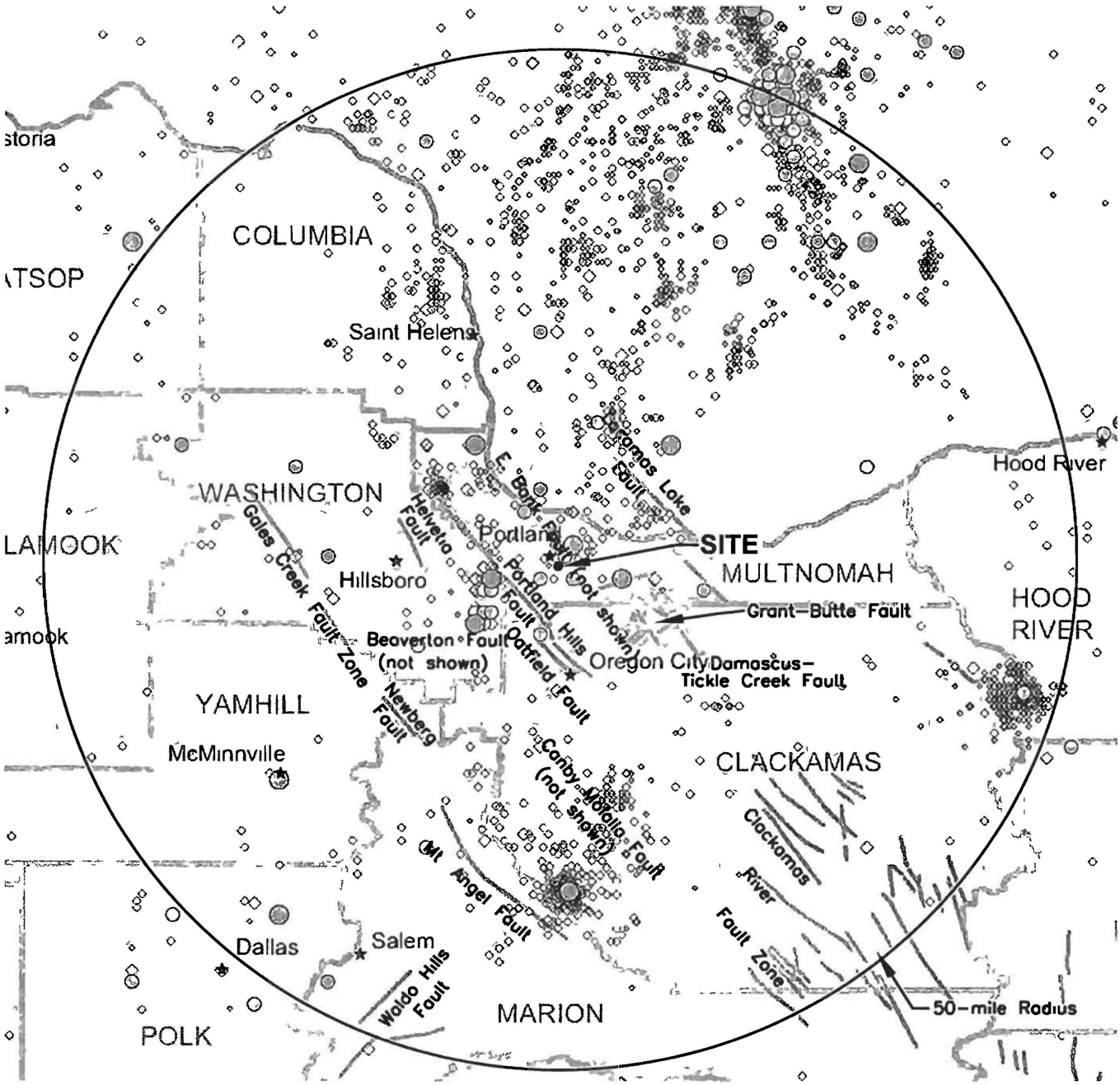
B-1 ◔ Explorations by Pacific Geotechnical

C-1 ⊕ Exploration by Others
(See Report Appendices and Text)

A A Approximate Cross Section Location

N

DWG NAME	Site an
DESCRIPTION	



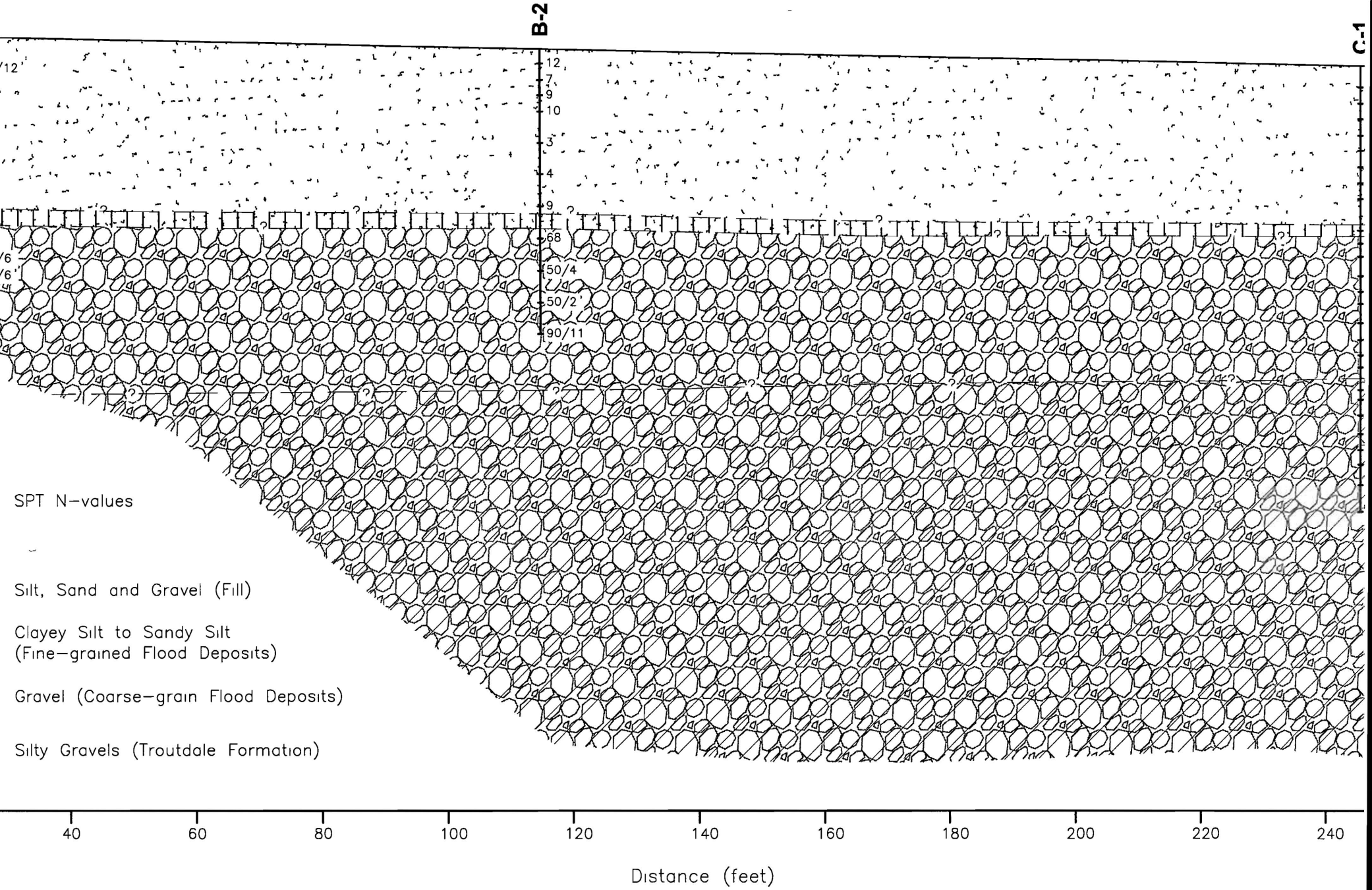
Reference Base map from DOGAMI Open File Report 03–02 (Niewendorp and Neuhaus 2003)

Legend

- Magnitude 7 and higher
- Magnitude 6.0 6.9
- Magnitude 5.0 5.9
- Magnitude 4.0 4.9
- Magnitude 3.0 3.9
- Magnitude 1.0 2.9
- Magnitude 0.0 0.9
- Fault - Holocene
- Fault Late quaternary
- State line
- County line
- County seat

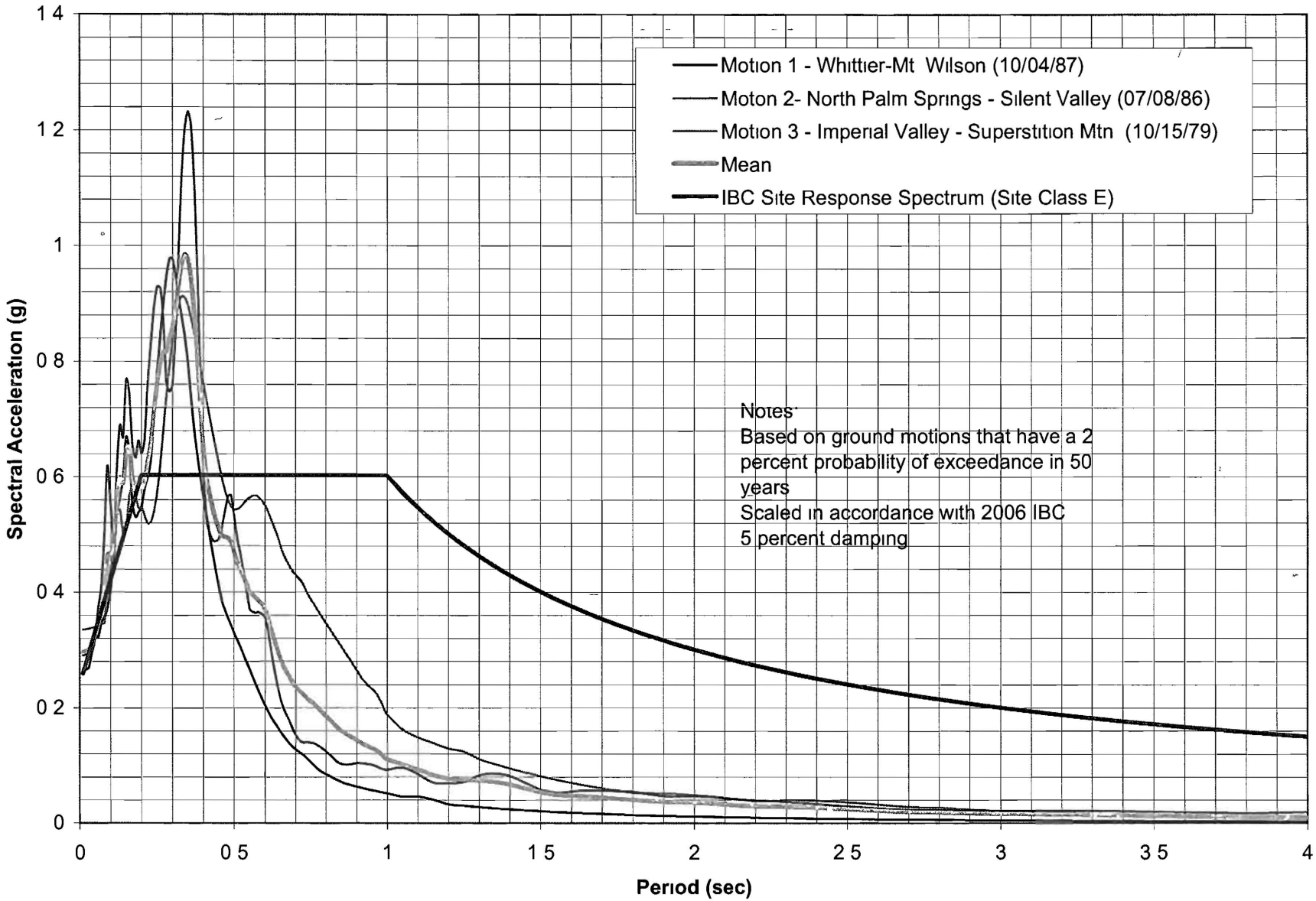
N

DWG NAME	Fault and Seismicity Map		
DESCRIPTION	Portland Waterfront Park Saturday Market Relocation		
JOB NO	1050–001–00	DATE	3/4/08
FILE NAME	105000100F5 dwg	FIGURE	4



entation of soil conditions for informational purposes
for bidding purposes. Refer to attached Geotechnical Report

DWG NAME	Cros
DESCRIPTION	



Portland Waterfront Park
Saturday Market Relocation
Figure 5

APPENDIX A
SUBSURFACE EXPLORATIONS BY PACIFIC GEOTECHNICAL

APPENDIX A
SUBSURFACE EXPLORATIONS

We evaluated subsurface soil conditions by completing three borings at the approximate locations shown on Figure 2 of the report text The boring locations were approximately located by pacing from existing site facilities Exploration locations should be considered accurate only to the degree implied by the methods used

The borings were drilled utilizing mud rotary drilling techniques using a truck-mounted drill rig provided by Subsurface Technologies of Banks, Oregon Soil samples were obtained from the explorations using one of the following methods

1 Standard Penetration Tests (SPT) were completed in general conformance with ASTM Test Method D1586, "Standard Method for Penetration Test and Split-Barrel Sampling of Soils" The sampler was driven with a 140-pound auto-trip hammer falling 30 inches The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the boring logs Disturbed samples were obtained from the split barrel for subsequent classification and index testing

2 Relatively undisturbed samples were obtained using a Dames & Moore Type-U sampler The sampler was driven using a 140-pound hammer falling 30 inches, just as with the SPT samples The penetration resistance recorded on the boring logs have been reduced by 50 percent for general correlation with the SPT blow counts Samples retained from the split barrel consist of up to six, 1-inch-high by 2 48-inch-diameter brass rings Disturbed rings were generally not retained

Materials encountered in the explorations were classified in the field in general accordance with ASTM Standard Practice D2488, "Standard Practice for the Classification of Soils (Visual-Manual Procedure)" Soil classifications and sampling intervals are shown in the exploration logs in this appendix

The field explorations were coordinated by an engineering geologist from our staff, who located the explorations, classified the various soil units encountered, obtained representative soil samples for geotechnical testing and maintained a detailed log of each exploration Exploration logs are included in this appendix Results of the laboratory testing are indicated on the exploration logs and are discussed in Appendix C

KEY TO EXPLORATION LOGS

Pacific Geotechnical, LLC

Pacific Geotechnical, LLC
1419 Washington Street, Suite 101
Oregon City, Oregon 9704

SOILS CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)	GW	WELL-GRADED GRAVELS GRAVEL - SAND MIXTURES
			GP	POORLY GRADED GRAVELS GRAVEL - SAND MIXTURES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	GM	SILTY GRAVELS GRAVELS SAND SILT MIXTURES
			GC	CLAYEY GRAVELS GRAVEL SAND CLAY MIXTURES
MORE THAN 50% RETAINED ON NO 200 SIEVE	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)	SW	WELL-GRADED SANDS GRAVELLY SANDS
			SP	POORLY-GRADED SANDS GRAVELLY SANDS
	MORE THAN 50% OF COARSE FRACTION PASSING NO 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	SM	SITLY SANDS SAND SILT MIXTURES
			SC	CLAYEY SANDS SAND CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	ML	INORGANIC SILTS ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY GRAVELLY CLAYS SANDY CLAYS SILTY CLAYS LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% PASSING NO 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	MH	INORGANIC SILTS MICACEOUS OR DIATOMACEOUS SILTY SOILS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY
			OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT-HUMUS SWAMP SOILS WITH HIGH-ORGANIC CONTENTS

Note Multiple symbols are used to indicate borderline or dual soil classifications

SYMBOLS LETTER	DESCRIPTIONS
CC	CEMENT CONCRETE
AC	ASPHALT CONCRETE
TS	TOPSOIL/SOD FORREST DUFF

Stratigraphic Contact

—— Distinct contact between soil strata or geologic units

----- Gradual or approximate change between soil strata or geological units

Moisture Modifiers

Dry - Absence of moisture, dusty, dry to the touch

Moist - Damp, but no visible water

Wet - Visible free water or saturated, usually soil is obtained from below the water table

Seepage Modifiers

None

Slow - < 1 gpm

Moderate - 1- 3 gpm

Heavy - > 3 gpm

Caving Modifiers

None

Minor - isolated

Moderate - frequent

Severe - general

Minor Constituents

Trace < 5% (silt/clay)

Occasional < 15% (sand/gravel)

With 5-15% (silt/clay) in sand or gravel

15-30% (sand/gravel) in silt or clay

Sampler Symbol Descriptions

- 2 4-inch I D split barrel
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- G Bulk or grab

Laboratory / Field Tests

%F Percent fines

AL Atterberg Limits

CP Laboratory compaction test

CS Consolidation test

DS Direct shear

HA Hydrometer analysis

Laboratory / Field Tests

DD Dry density

OC Organic content

PP Pocket penetrometer

SA Sieve analysis

TV Torvane shear

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586 See exploration log for hammer weight and drop

A "P" indicates sampler pushed using the weight of the drill rig

(2 4-inch) sampler N approximately corrected to equivalent SPT N by 50% reduction in N - modified California

Note Refer to the report text and exploration logs for a proper understanding of subsurface conditions Descriptions on the logs apply only at the exploration locations at the time the explorations were made The logs are not warranted to be representative of the subsurface conditions at other locations or times

Pacific Geotechnical, LLC

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Oregon City Oregon 97045

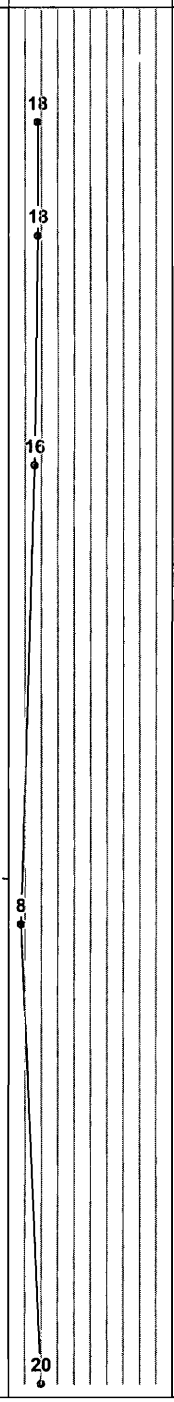
LOG OF BORING NO. B-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050 001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Other Tests and Notes
					SW Brown gravelly SAND, fine to coarse sand, fine rounded gravel (moist, loose) [Fill]	
	10	1	SPT	9		
5	8	2	SPT	7	Grades to with layers of SILTY SAND with gravel	
	5	3	SPT	4	Grades with trace silt	
10	6	4	SPT	4		
15	4	5	SPT	3	Grades with trace to some gravel (very loose)	
20	2	6	SPT	32	GP Brown POORLY-GRADED GRAVEL with trace fine sand Rounded gravel to 1 maximum dimension (moist to wet, medium dense to dense)[Fill]	Interpreted as fill based on depth
25	3	7	SPT	30		
30						

Graphic Log

Water Content, %



Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
	5	8	SPT	10	Grades to loose		Lost circulation and mud Sampler possibly driven through slough May be denser than blow counts suggest
35	6	9	SPT	10			
40	10	10	SPT	24	Grades with fine to coarse sand (medium dense to dense)	13	
							Regain circulation at 43ft
45	0	11	SPT	47			No recovery
50	9	12	SPT	21	GP-SP Sandy GRAVEL to gravelly SAND Fine to medium sand Fine rounded gravel (wet, dense)[Fill]	15	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
55	6	13	SPT	28	SM Gray SAND with gravel and silt Fine to medium sand Fine rounded gravels (wet, medium dense to dense) [Fill]	20	Interpreted as fill based on blow counts and drawings of adjacent cribbing/seawall
60							

Graphic Log

Water Content, % scale: 10 30 50 70 90

*Corrected Blows/foot scale: 20 40 60 80

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC
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Oregon City Oregon 97045

LOG OF BORING NO. B-2

Project Name Ankeny Plaza
Location Waterfront Park, Portland OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
					GP-GM Dark gray POORLY-GRADED GRAVEL with sand and silt to SILTY GRAVEL Coarse angular basalt crushed rock (wet, medium dense)[Fill]		Boring relocated 10 feet west due to hitting abandoned sewer pipe
	10	1	SPT	12		15	
5	4	2	SPT	7	Grades to loose		
	10	3	SPT	9	SP-SM Brown to gray-brown POORLY-GRADED SAND with silt to SILTY SAND with gravel and wood debris (moist, loose)[Fill]	20	
10	12	4	SPT	10	SM Dark gray-brown SILTY SAND with gravel (wet, loose to medium dense) [Fill]		
15	4	5	D+M	4	ML Gray-brown and mottled brown SILT with fine sand Low to medium plasticity (moist, soft)[Fill]	37	DD= 84 pcf
20	16	6	SPT	4	Grades to dark gray with medium plasticity and occasional bricks	34	%F= 75
25	12	7	SPT	9	Grades to SILT with fine sand to fine sandy SILT (stiff)[Fill or Fine-grained Flood Deposits?]	37	
30						24	

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-1

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	1	14	SPT	42	SP Brown SAND with occasional fine rounded gravels (wet, very dense) [Troutdale Formation]		Hard drilling at 61ft
65	4	15	SPT	100			50 blows for 5 '
70	15	16	SPT	56	Becomes dark brown with some yellow		
	6	17	SPT	100	GP Brown POORLY-GRADED GRAVEL with fine to medium sand (wet, very dense)		50 blows for 4 5
75					Boring completed at 73ft bgs Groundwater not determined due to drilling method used		
80							
85							
90							

Date Started 1/30/08
Date Completed 1/30/08
Completion Depth 73ft 4"
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT
Auger Data NA
Hammer Data 140lb Autohammer 30 inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-2

Project Name Ankeny Plaza

Location Waterfront Park, Portland OR

Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
	14	8	SPT	68		GM Dark gray SILTY GRAVEL with trace coarse sand and with angular rock (wet, very dense)[Coarse-grained Flood Deposits]		Very slow drilling at 30ft to 35ft
35	4	9	SPT	100				50 blows for 4"
40	3	10	SPT	100		Grades to dark gray-brown SILTY GRAVEL with fine to medium coarse sand and rounded basalt gravels to 1 in maximum dimension		50 blows for 2 '
45	6	11	SPT	100			14	90 blows for 11
50						Boring completed at 46 ft bgs Groundwater not determined due to drilling method used		
55								
60								

Date Started 1/31/08
Date Completed 1/31/08
Completion Depth 46 ft
Drilled By Subsurface Techologies
Logged By J Lawes

Drilling Method Mud rotary
Sampling Method SPT, D&M
Auger Data NA
Hammer Data 140 lb Auto-trip falling 30 inches
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

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LOG OF BORING NO. B-3

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot	Material Description	Water Content, %	Other Tests and Notes
					SP Brown gravelly SAND to SAND with gravel Fine to medium sand Fine rounded gravel (moist, medium dense) [Fill]		
	13	1	SPT	25		13	
5	14	2	SPT	100	Grades with brick and concrete debris	20	50 blows for 6" due to concrete debris
	5	3	SPT	32	**GM** Gray SILTY GRAVEL with sand and occasional wood debris Fine to medium sand Fine to coarse rounded to subangular gravel (moist to wet, dense)[Fill]	35	
10	6	4	SPT	18			
15	4	5	SPT	1	**ML** Gray SILT with fine sand, clay and wood debris Low to medium plasticity (moist, very soft to medium stiff)[Fill]		%F= 79
20	0	6	D+M	3		39	No recovery due to wood in sampler shoe
25	0	7	D+M	10		43	No recovery
	26	8	ST		**SM** Gray silty fine SAND (moist, medium dense)[Fine-grained Flood Deposits]	33	
30	9	9	SPT	22			

Graphic Log

Corrected Blows/foot: 20 40 60 80

Water Content, %: 10 30 50 70 90

Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

Pacific Geotechnical, LLC
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LOG OF BORING NO. B-3

Project Name Portland Waterfront Park
Location Portland, OR
Number 1050-001-00

Depth, feet	Recovery (in)	Sample Number	Sample Type	*Corrected Blows/foot (20 40 60 80)	Graphic Log	Material Description	Water Content, % (10 30 50 70 90)	Other Tests and Notes
						GP Gray GRAVEL with sand and trace silt Fine to coarse sand Fine to coarse rounded gravel (moist, dense) [Coarse-grained Flood Deposits]		
35	10	10	SPT	100		Grades to very dense		50 blows for 6
	8	11	SPT	100				50 blows for 6"
						Boring completed at 38 ft bgs Groundwater not determined due to drilling method used		
40								
45								
50								
55								
60								

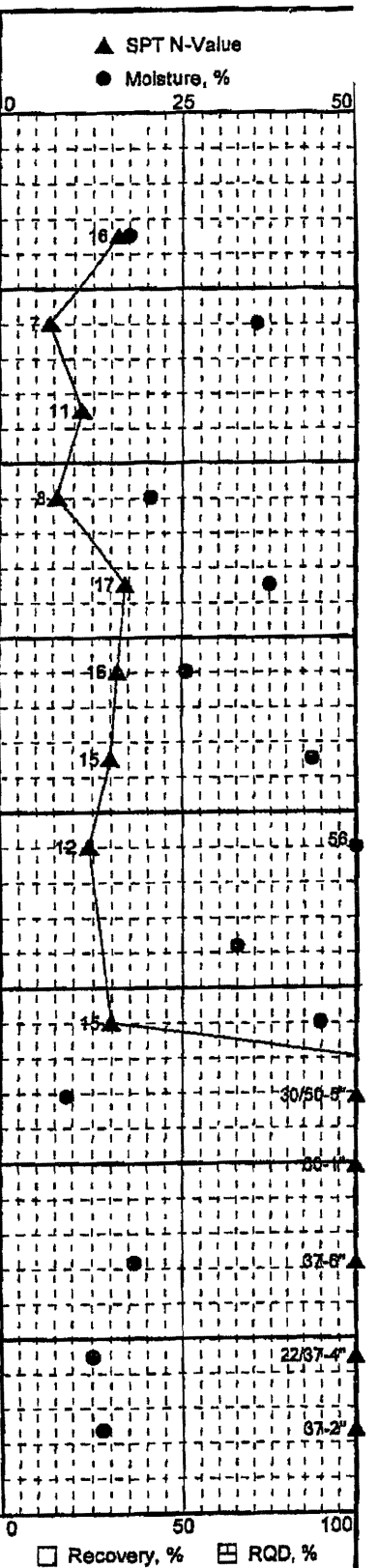
Date Started 1/29/08
Date Completed 1/29/08
Completion Depth 38ft
Drilled By Subsurface Technologies
Logged By G Sandstrom

Drilling Method Mud rotary
Sampling Method SPT, D&M, Shelby Tube
Auger Data NA
Hammer Data 140lb Autohammer 30-inch drop
Drilling Equipment

* D&M N values reduced by 50% to correlate with SPT

APPENDIX B
SUBSURFACE EXPLORATIONS BY OTHERS

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth in Feet	Samples
		30 0				
		29 2 0 9	**PAVEMENT**, 2 5 inches of brick over 10 inches of steel reinforced concrete (Pavement)		0	
			SAND FILL, medium dense, gray, moist, fine-grained, trace fine rounded gravel and crushed rock (Fill)			S-1
		26 0 4 0	**Sandy SILT FILL**, slightly clayey, loose, light brown, moist, no to low plasticity, scattered fine rounded gravel (Dredged sand fill)		5	S-2 S-3
		20 0 10 0	**SAND FILL**, loose, gray brown, moist, fine-grained, scattered fine rounded gravel (Dredged sand fill)		10	S-4
			Tree fragment, medium dense			S-5
		16 0 14 0	**Silty SAND FILL**, slightly clayey, medium dense, gray, moist to wet, no to low plasticity, fine-grained, scattered to abundant wood fragments and red crushed rock (Fill)		15	S-6
	-200 = 33%					S-7
▼	08/28/2000 -200 = 35%				20	S-8
	-200 = 16%					S-9
		5 0			25	
	-200 = 73%	25 0	**Clayey SILT**, stiff, gray, wet, low plasticity, micaceous (Recent Alluvium)			S-10
		3 0 27 0	**Sandy GRAVEL**, very dense, brown and gray, wet, fine- to coarse-grained, subrounded gravel, sand is fine- to coarse-grained, scattered cobbles (Pleistocene Flood Deposits - Qfch)			S-11
					30	S-12*
						S-13
					35	S-14
						S-15



LEGEND

- I = 2 0" O D Split Spoon Sample
- II = 3 0" O D Thin-Walled Sample
- • = Sample Not Recovered
- = Grab Sample Drill Cuttings
- = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1

page 1 of 2

December 2000 *F-3112 01*

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland, Oregon

FIG.

FHA

WLG ANKPS.GPJ 12/6/00

Ground Water	Remarks	Elev Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	SPT N-Value / Moisture, % (0 – 25 – 50)
			Sandy GRAVEL continued			S-16	22/37-5"
						S-17	37-4"
					45	S-18	22/37-0"
						S-19	37-2"
		-18 5 48 5	**SAND**, very dense, no sample return (Pleistocene Flood Deposits - Qfch)		50	S-20*	43
		-21 0 51 0	**Clayey Gravelly SAND**, very dense, yellow-brown, moist, low plasticity, sand is fine-grained, fine- to coarse-grained, rounded gravel, weathered, overconsolidated, poorly graded, gradational transition to underlying material (Troutdale Formation Tt)			S-21*	37-2"
					55	S-22	37-3"
						S-23*	37-3"
					60	S-24*	37-3"
						S-25*	37-0 5"
		-35 0 65 0	**Gravelly SAND**, very dense, light brown, wet, non-plastic, sand is fine-grained, fine- to coarse-grained, rounded gravel, overconsolidated, poorly graded (Troutdale Formation Tt)		65	S-26	56
					70	S-27	26/37-3"
		-40 8 70 8	Bottom of Boring, Completed 8/29/00				

0 50 100
□ Recovery, % ⊞ RQD, %

LEGEND

- I = 2 0" O D Split Spoon Sample
- II = 3 0" O D Thin-Walled Sample
- * = Sample Not Recovered
- ▩ = Grab Sample Drill Cuttings
- ▮ = Core Rock Sample

NOTE
Lines between soil/rock units are approximate and transition may be gradual

- Impervious Seal (Bentonite)
- Cement Grout
- Random Backfill
- Granular Backfill
- Ground Water Level on Date Shown
- Piezometer/Inclinometer Tubing
- Perforated Zone

ATTERBERG LIMITS
- Liquid Limit
- Natural Water Content
- Plastic Limit

FHA

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1
page 2 of 2

December 2000 *F-3112 01*

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland Oregon

FIG.

WLG ANKPS.GPJ 12/6/00

APPENDIX C
LABORATORY TESTING

APPENDIX C
LABORATORY TESTING

GENERAL

Soil samples obtained from the explorations were transported to our laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils encountered Representative samples were selected for laboratory testing The tests were performed in general accordance with the test methods of the American Society for Testing and Materials (ASTM) or other applicable procedures

VISUAL CLASSIFICATIONS

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the Unified Soil Classification System (USCS) and ASTM classification methods ASTM Test Method D2488 was used to classify soils using visual and manual methods ASTM Test Method D2487 was used to classify soils based on laboratory test results

Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D2216 The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in Appendix A

Dry Density

Dry density tests were completed on low-disturbance samples obtained with the Dames & Moore (D&M) sampler The tests were conducted in general accordance with ASTM Test Method D2937 Dry density test results are presented on the boring logs in Appendix A

Percent Fines

Fines content analyses were performed to determine the percentage of soils finer than the No 200 sieve - the boundary between sand size particles and silt size particles The tests were performed in general accordance with ASTM D1140 The test results are indicated on the individual boring logs in the column labeled "Other Tests and Notes"

Bid Set

Section 00520 - Driven Piles

Description

00520 00 Scope - This work consists of furnishing and driving piles of the type and dimensions shown or specified, including cutting off or building up piles when required

Materials

00520 10 General - Furnish materials meeting the following requirements

Reinforced Pile Tip	02520 10
Steel Piles	02520 10

00520 11 Engineer's Estimated Length List - Furnish steel piles of sufficient length to attain the penetration and bearing values specified, and to extend into the cap or footing as shown The Contractor may, at no cost to the City, drive test piles, make borings, or perform other investigations the Contractor considers necessary The "Engineer's Estimated Length" listed in the Special Provisions will be used only for compensation of bids

00520 14 Unused Piles - Acceptable full length piles furnished according to the estimated length list, but not incorporated in the work, will be handled according to one of the following

- Mark and identify piles for the Contractor's own use
- Return piles to the supplier with the City paying transportation and restocking charges
- The City will purchase from the Contractor piles that are stockpiled at a location on the Project selected by the Engineer according to 00195 80

Equipment

00520 20 Equipment for Driving Piles - Provide pile driving equipment meeting the following requirements

(a) Impact Pile Hammers - Provide a striking part of the hammer not less than one-third the weight of the helmet and pile being driven, but never less than 2,750 pounds

(2) Open-End Diesel Hammers - Provide open-end (single-acting) diesel hammers equipped with a device which allows the Engineer to visually determine hammer stroke at all times during pile driving operations Provide the Engineer with the hammer manufacturer's chart equating stroke and blows per minute

(3) Closed-End Diesel Hammers - Provide closed-end (double-acting) diesel hammers equipped with a bounce chamber pressure gauge, mounted near ground level so the Engineer can easily read it Before driving, provide the Engineer a chart calibrated within six months before first use on the Project to actual hammer performance, equating bounce chamber pressure to either equivalent energy or stroke

(c) Driving Components

(2) Helmet - Equip piles driven with impact hammers with an adequate metal helmet The helmet shall

- Fit around the pile top
- Be axially aligned with the hammer and pile
- Distribute the hammer energy to the total cross section of the pile head
- Be guided by leads

(3) Hammer Cushion - Equip impact pile driving equipment with a suitable thickness of hammer cushion material to prevent damage to the hammer or pile and to ensure uniform driving performance Provide hammer cushions of durable manufactured materials according to the hammer manufacturer's guidelines Do not use wood, wire rope, or asbestos hammer cushions Place a striker plate, as recommended by the hammer manufacturer, on the hammer cushion to ensure uniform compression of the cushion material

Inspect the hammer cushion in the presence of the Engineer at the beginning of pile driving at each structure or after each 100 hours of use during pile driving, whichever is less Replace the hammer cushion when its thickness becomes less than 75% of its original thickness

(5) Leads - Support piles in line and position while driving Construct pile hammer leads to give the hammer freedom of movement while maintaining alignment of the hammer and the pile to ensure concentric impact for each blow Leads shall be fixed unless the Engineer approves the use of swinging leads Fit swinging leads, when used, with a pile gate at the bottom of the leads To maintain alignment of batter piles, use horizontally braced swinging leads, adequately embedded in the ground, or rigidly attached to prevent movement during pile driving

(d) Approval of Pile-Driving Equipment

(1) General - Before beginning production pile driving, obtain approval in writing of pile driving equipment

To obtain approval, complete and submit the City's "Pile and Driving Equipment Data" form at least 14 calendar days before pile driving begins blank form is included in the Special Provisions or is available from the Engineer Within 14 calendar days of receiving the form, the Engineer will notify the Contractor of approval or rejection of the pile-driving equipment

During pile-driving operations, no changes to the approved equipment will be allowed without the Engineer's written permission Submit a request for change on a "Pile and Driving Equipment Data" form The Engineer will give notification of approval or rejection within seven calendar days of receiving the form Time required for resubmission and review of a Contractor's equipment change request is not a basis for a Contract Time extension request unless the Engineer does not respond in seven calendar days

(3) Wave Equation Method - The method used to evaluate the driving equipment will be based on wave equation analysis The criteria will be the required number of hammer blows per inch and the pile stresses at the required ultimate pile bearing capacity

The energy of the submitted hammer shall produce a wave equation predicted blow count between 3 and 15 blows per inch at the ultimate pile bearing capacity

If the wave equation analysis shows an inability to drive the pile(s) to the required ultimate pile bearing capacity with an acceptable blow count or that pile damage will occur, change the proposed driving equipment until the wave equation analysis indicates the piles can be driven as specified Submit changes proposed for review according to 00520 20(d)(1) Make approved changes at no additional compensation

The pile stresses indicated by the wave equation at the ultimate pile bearing capacity shall not be greater than the stress at the point of impending damage to the pile as follows

- **Steel Piles** - Tensile and compressive stresses in the pile of 90% of the pile material's yield strength for the grade of steel specified at any time during the pile installation

Hammers not meeting these requirements will be rejected Replace rejected hammers with suitable hammers

Construction

00520 40 Preparation for Driving

Bid Set

(a) Excavation - Unless otherwise provided or authorized, do not drive piles until after excavation is complete Remove to the correct elevation any material forced up by pile driving at the Contractor's expense before concrete for the foundation is placed

00520 41 Driving

(a) General - Drive piles as specified with approved pile driving equipment to the required penetration depth and to the required ultimate pile bearing capacity as shown or specified

(b) Installation Sequence - Unless otherwise shown or specified, install individual piles in pile groups starting from the center of the group and proceeding outward in either direction, or as approved

(c) Minimum Penetration - Unless otherwise specified or approved, drive piles at least 12 feet below the footing or pile cap When shown or specified drive piles to a greater minimum penetration If the required penetration cannot be attained with a hammer complying with 00520 20(d), provide a larger hammer, prebore or jet holes, or use other approved methods as necessary to attain the required penetration

(d) Preboring - Use augering, wet-rotary drilling or other methods of preboring only when specified or with written approval When permitted, prebore holes at pile locations and to the depths shown or directed Make prebored holes smaller than the diameter or diagonal of the pile cross section, but sufficient to allow penetration of the pile to the specified depth If subsurface obstructions, such as cobbles, boulders or rock layers are encountered, the hole diameter may be increased to the least dimension which is adequate for pile installation The use of a reinforced section (spud) to loosen the subsurface material at pile locations will not be allowed unless otherwise approved

Perform preboring in a manner that will not impair the bearing or lateral capacity of the piles already in place or the safety of existing adjacent structures When it is determined that preboring has disturbed the load bearing capacities of previously installed piles, restore those piles that have been disturbed to conditions meeting the requirements of this Specification by redriving or by other acceptable methods The Contractor shall be responsible for the costs of any necessary remedial measures unless the preboring method was specifically included in the Contract Documents and properly executed by the Contractor

> **(1) End-Bearing Piles** - For end-bearing pile as classified by the Engineer, preboring may be carried to the surface of the end-bearing foundation material Following that, drive pile with an approved impact pile hammer to the specified blow count
>
> **(2) Other Piles** - For other piles, extend preboring to the minimum pile penetration depth and then drive pile with an approved impact pile hammer to the specified blow count

After completion of driving, fill any void space remaining around the pile with sand or other approved material

(e) Jetting - Jetting may only be used when allowed in the Contract Documents or if approved in writing When jetting is not required in the Contract Documents, but approved at the Contractor's request, determine and submit for review the number of jets and the volume and pressure of water at the jet nozzles necessary to freely erode the material adjacent to the pile without affecting the lateral stability of the final in-place pile The Contractor shall be responsible for all damage caused by unapproved or improper jetting operations, unless the jetting method was specifically included in the Contract Documents and properly executed by the Contractor Control, treat if necessary, and dispose of all jet water in a satisfactory manner Drive all jetted pile with an approved impact hammer

(f) Location and Alignment Tolerance - Place the tops of piles at plan cutoff elevation and horizontally within 6 inches of plan locations No pile shall be nearer than 4 inches from any edge of the cap Any increase in cap size to meet this edge distance requirement will be at the Contractor's expense

Install piles so the axial alignment of the top 10 feet of the pile is within 5 inches of the specified alignment For piles that cannot be inspected after installation, make an alignment check before installing the last 5 feet of pile The Engineer may require that driving be stopped to check the pile alignment Pulling laterally on piles to correct misalignment or splicing a properly aligned section onto a misaligned section will not be allowed

If the specified location or alignment tolerances are exceeded, the effect of the pile misalignment on the substructure design will be investigated If the Engineer determines corrective measures are necessary, implement suitable measures and pay all costs and delays associated with the corrective action

(g) Heaved Piles - Make elevation readings on piles during pile driving operations to check on pile heave Take elevation readings after each pile has been driven and again after piles within a radius of 15 feet have been driven Redrive to the required penetration and resistance, at Contractor's cost, all piles that have risen more than 1/2 inch Continue readings until the Engineer determines that such checking is no longer required If pipe piles which have been filled with concrete subsequently heave, redrive them to original position, after the concrete has attained specified strength, with an approved hammer-pile cushion system

00520 42 Ultimate Pile Bearing Capacity

(a) General - Drive piles with approved pile driving equipment to the lengths necessary to attain the required penetration and ultimate pile bearing capacity Adequate pile penetration will be considered reached when the piles are driven to or below the minimum penetration depth and the specified equation resistance value is achieved If piles do not achieve the specified resistance when driven to order length or estimated length, splice and drive them to penetrations established by the Engineer The required number of hammer blows per inch at final penetration shall be maintained for 3 consecutive inches unless "refusal" driving is first obtained "Refusal" driving is defined as 20 blows per 1 inch or as determined by the Engineer

If water jets are used with the driving, the bearing value shall be determined by the specified equation from the results of driving after the jetting has been completed according to 00520 42(e)

(c) Wave Equation Analysis - The Engineer will determine ultimate pile bearing capacity based on wave equation analysis

(d) Set Period and Re-driving - If piles do not attain the required bearing capacity when driven to the specified length, and if permitted or required, allow the piles to stand for a "set period" without driving The "set period" shall be a minimum of 24 hours unless otherwise approved by the Engineer After the set period, perform check driving on either two piles in each bent or on one pile in every 10 piles, whichever is more The Engineer will designate the piles on which check driving is to be performed Do not use a cold hammer for redriving Warm up the hammer before redriving begins by applying at least 20 blows to another pile Redriving shall consist of driving the pile to the required bearing with a maximum of 15 blows If the specified hammer blow count is not attained on redriving, the Engineer may direct the Contractor to drive all of the remaining pile length and repeat the set period and redriving procedure Splice those piles driven to plan grade that do not attain the hammer blow count required, and drive until the required bearing is attained If the required bearing capacity is attained for each pile that is redriven, then the remaining piles in that bent will be considered satisfactory when driven to at least the same penetration and resistance as the redriven piles

(e) Jetted Piles - The ultimate pile bearing capacity of jetted piles will be based on impact driving blow count after jetting has been completed Jet pipes may be removed when the pile tip is at the required minimum pile tip elevation and before the pile is driven to the required bearing capacity For piles that are jetted at the Contractor's request and do not attain the required ultimate bearing capacity at the ordered length, splice, as required, and drive with a specified impact pile hammer until the required ultimate pile bearing capacity is achieved according to appropriate criteria in 00520 42 Regardless of City approval, the Contractor shall pay all costs of splicing and driving piles beyond the order length if jetting is requested by the Contractor

00520 43 Steel Piles

(a) General - Unless otherwise specified, furnish standard steel piles in the longest practical lengths

(b) Storage and Handling - Store and handle steel piles in ways that protect them from damage Bent or kinked piles will be rejected

(c) End Treatment - Cut pile ends square

(d) Reinforced Pile Tips - Install pile points, shoes, or other tip reinforcement according to the manufacturer's recommendations and Section 02520

(e) Driving - During driving, protect the pile head with a fitted metal helmet

Bid Set

(f) Splices - Where splices are unavoidable, submit for approval their number, location and details

(1) Welded Splices - Make welded splices using a full penetration butt weld, as shown Comply with the welding procedures of AWS D1 1

(2) Mechanical Splices - Mechanical splices may be used if the splice transfers the full pile strength in compression, tension, and bending, according to working drawingssubmitted according to 00150 35(m)(2) and approved by the Engineer

(g) Welding - Weld pile splices, pile tips, pile anchors, and other welded attachments to steel piles according to AWS D1 1

(1) Splices - Splice joints for round piles shall conform to Joint B-U4a or C-U4a-GF (Single-Bevel Groove Weld) in D1 1 Figure 3 4 Weld back-up rings with a full penetration groove weld

Splice joints for H-piles shall conform to Joint B-U3b or B-U3-GF (Double V-Groove Weld) in D1 1 Figure 3 4 for both the web and flange sections Joint B-U4a or C-U4a-GF may be substituted on the flange weld Provide access holes at the ends of the web according to D1 1 Section 5 17

(2) Submittals - Prior to welding, submit the following for approval

- A Welding Procedure Specification (WPS) for all pile welds, conforming to the limitations of D1 1 Table 4 5 Both ASTM A 36 and ASTM A 252 Grade 1 and 2 may be treated as prequalified base metals under Group 1 ASTM A 252 Grade 3 will not be considered a prequalified base metal unless the steel has a Carbon Equivalent (CE) of 0 30% or less Develop a Procedure Qualification Record (PQR) for all welding using Grade 3 steel or present proof that the chemistry of the steel meets the CE requirements

- Qualification documents for each welder Use welders qualified according to D1 1 Section 4 for the position, process and pile diameter used on the job

Do not begin welding without approval

Following completion of all welding, submit the following

- An inspection report stating that the welding under the Contract was performed according to D1 1 The report shall include a review of the WPS, a review of welder qualifications and a report on visual inspection of the welds on the job site The inspection shall be signed by a Certified Welding Inspector (CWI) holding QC1 certification as defined in D1 1 Section 6

- If the plans or Specifications call for additional inspection other than visual, include reports in the submittal

(3) Additional Testing - The Engineer may request additional nondestructive testing (NDT), such as radiography or ultrasonic testing of any or all welds If the additional testing identifies defects warranting rejection, perform repair and additional inspection at no additional cost to the Department If the additional NDT does not identify defects warranting rejection, the Department will pay the cost of the additional testing Radiographic and ultrasonic defect indications will be evaluated according to the statically loaded criteria of D1 1

(h) Cutoff Lengths - Cut off the tops of all permanent piles square and smooth at the elevation shown or as directed All cut-off pile becomes the property of the Contractor Dispose of according to 00290 20 With approval, undamaged cutoffs may be used as pile extensions or welded together to form full length piles Steel pile cutoffs welded together, whether pile extensions or full length piles, shall not vary from a straight line more than 1/4 inch in 20 feet measured along any edge of the pile

All acceptable cutoffs and unused pile lengths remaining at completion of pile driving will be marked for identification by the Engineer as acceptable for use on other or future City projects if requested by the Contractor

(i) Capping - If required by the plans, cap steel piles with a steel plate of the size and shape shown Connect this cap to the pile according to the details shown

00520 46 Damaged or Defective Piles - In addition to other specified requirements at Contractor's expense

- Approval of a pile hammer shall not relieve the Contractor of responsibility for piles damaged from misalignment of the leads, failure of capblock or cushion materials, failure of splices, malfunctioning of the pile hammer or other improper construction methods
- Piles damaged during installation will be considered unsatisfactory unless the bearing capacity is proved by load tests performed by the Contractor If such tests indicate inadequate capacity, take corrective measures, such as the use of damaged piles at reduced capacity, installation of additional piles, strengthening of damaged piles, or replacement of damaged piles
- Do not place footing concrete until all piles within a footing are inspected by the Engineer

Measurement

00520 80 Measurement - The quantities of work performed under this Section will be measured according to the following

(a) Furnish Equipment for Driving Piles - No measurement of quantities will be made for furnishing equipment for driving piles

(b) Furnish Piles - The quantities of furnishing steel, prestressed concrete, timber, and test piles will be measured on the length basis, to the nearest foot, as follows

> **(1) Steel Piles** - Steel piles will be the length of each pile remaining in the completed work, from the pile tip to the cutoff plane

No allowance will be made for that length of pieces furnished by the Contractor to replace piles previously accepted by the Engineer, but that are subsequently damaged before completion of the Project

(c) Drive Piles - The quantities of driving steel, prestressed concrete, timber, and test piles will be measured on the unit basis Driving test piles includes test piles remaining in the completed work

Preboring will be measured on the length basis, to the nearest foot

Jetting will be measured on the unit basis, for each pile driven with the aid of jetting

(e) Reinforced Pile Tips - The quantities of reinforced pile tips will be measured on the unit basis

(f) Pile Splices - Pile splices will be determined as follows

> **(1) Steel Piles** - Splices incorporated in the finished structure that were made to increase the length of the pile 5 feet or more for estimated pile lengths of 60 feet or less and 10 feet or more for estimated pile lengths of over 60 feet beyond the estimated pile length will be measured on the unit basis Only one splice will be measured per pile No measurement will be made for splices to steel piles within the estimated lengths listed in 00520 11 of the Special Provisions

Payment

00520 90 Payment - The accepted quantities of work performed under this Section will be paid for at the Contract unit price, per unit of measurement, for the following items

	Pay Item	Unit of Measurement
(a)	Furnish Pile Driving Equipment	Lump Sum
(b)	Furnish _____ Piles	Foot
(c)	Furnish _____ Test Piles	Foot

(d)	Drive _____ Piles	Each
(e)	Drive Test Piles	Each
(f)	Preboring Piles	Foot
(g)	Jetting Piles	Each
(h)	Pile Load Test (static)	Each
(i)	Pile Load Test (dynamic)	Each
(j)	Reinforced Pile Tips	Each
(k)	__ Steel Pile Splices	Each

Partial payments for Item (a) will be made as follows

- When equipment for driving piles is furnished and is satisfactorily driving piles 75%
- When driving piles is complete and equipment has been removed from site 25%

Item (a) includes

- furnishing all materials, equipment, and labor necessary for transporting, erecting, maintaining, replacing any ordered equipment, dismantling and removing the entire pile driving equipment
- resubmittal of wave equation analysis data if original data is rejected
- replacing previously approved hammers if hammer operates improperly
- all considerations when selecting the pile hammer size

The cost of all materials and labor, including the manipulation of the pile driving equipment in connection with driving piles will be included in the unit price each for driven piles Furnishing equipment for driving sheet piling is not included in this work

In items (b),(c), (d) and (k) the type and size of pile will be inserted in the blank

Item (d) includes cutting off piles, treating and capping pile heads, attaching anchor brackets, lugs or other attachments, and finishing concrete piles

Items (d) and (e) include all expenses involved in driving piles which have not attained the required bearing capacity and are required to stand for a "set period"

Item (j) includes attaching the tips to the piles

Item (k) includes steel pile splices required to increase pile length beyond the estimated length listed in 00520 11 of the Special Provisions No payment will be made for splices to steel piles that are within the estimated lengths listed in 00520 11 of the Special Provisions

Payment will be payment in full for furnishing and placing all materials, and for furnishing all equipment, labor, and incidentals necessary to complete the work as specified

No separate or additional payment will be made for work needed to drive piles to minimum tip elevation as shown or specified

No separate or additional payment will be made for welding inspection performed according to 00520 43(g-2)

No separate or additional payment will be made for preboring and jetting of piles if not included in the Contract Schedule of Items but requested by the Contractor

Preboring and jetting, if not included in the Contract Schedule of Items, larger hammers, and construction of concrete pile extensions, build-ups, and splices ordered by the Engineer, as a result of differing site conditions (see 00140 40) will be made on an Extra Work basis according to Section 00196

Pacific Geotechnical, LLC
Geotechnical Engineering and Engineering Geology
with a Focus on Sustainability

April 12, 2007

Walker Macy
111 SW Oak, Suite 200
Portland, Oregon 97204

Attention Mr Mauricio Villarreal

Subject Report of Geotechnical Engineering Services
Portland Waterfront Park, Ankeny Plaza & Street Improvements
Portland, Oregon
Project No 1050-001-00

08-113505-00

1. INTRODUCTION

Pacific Geotechnical, LLC (Pacific Geotechnical) is pleased to submit our geotechnical engineering report for the proposed Waterfront Park and Ankeny Plaza redevelopment project The project is located in and adjacent to Waterfront Park in the city of Portland, Oregon The location of the site is shown on Figure 1 Figure 2 shows the approximate project boundaries This report was completed in general accordance with our proposal of January 17, 2007

2. PROJECT DESCRIPTION

The project consists of relocation of the Portland Saturday Market (PSM) west into existing Waterfront Park Associated with this move will be reconfiguration of some pedestrian ways, construction of a restroom and cover structure for the PSM, installation of an interactive water feature, and appurtenant facilities such as piping and benches Structural loading information was not available at the time of completing this report, but loads on structures are expected to be very light The exception to this might be the PSM cover structure which could impose some significant moments on the foundations, depending on the final design Cuts and fills are expected to be minimal, probably less than about 3 feet

3. SITE CONDITIONS

The project area is located in the existing Waterfront Park in downtown Portland, Oregon Construction associated with this project is anticipated to extend to the Ankeny Plaza area and nearby portions of Burnside and Ankeny Streets and First Avenue, as shown on Figure 2 This site is within the densely urbanized Portland metropolitan core, and is characterized by complex geologic and soil conditions related to the regional geologic setting as well as the extensive alteration of the western bank of the Willamette River that has occurred since European settlement

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3 1. REGIONAL GEOLOGIC AND TECTONIC SETTING

The *Geotechnical Baseline Report, West Side CSO Tunnel, Shafts, Pump Station and Pipelines Project* (Parsons Brinckerhoff, 2002a) contains a well crafted explanation of the regional geologic setting of the project area as well as the history of local faulting and geomorphology (landform) The following paragraphs briefly summarize the information presented in that report

The project is located along the west edge of the Willamette River, a tributary of the Columbia River that flows through the Portland Basin This pull-apart basin, a product of regional tectonic stresses related to the subduction collision of the Juan de Fuca plate with the over-riding North American plate, has been subsiding since at least Miocene time Tens of hundreds to thousands of feet of alluvial clay, silt, sand and gravel have been deposited in the Portland Basin between intervals of uplift, scour and erosion that have created extensive topographic relief in the basin sediments Massive "Missoula" floods scoured and then deposited sediment ranging from clay to gravel in the Portland Basin during the latest Pliocene to latest Pleistocene (~1 million to 12,500 years ago) Post-Pleistocene geologic activity in the basin includes Willamette River fluvial erosion and deposition of sediment as well as the placement of large quantities of man-made fill along the Willamette riverfront and in adjacent sloughs, lakes and bottomlands

The regional Columbia River Basalt (CRB) bedrock that forms the Portland Basin and adjacent West Hills (Tualatin Mountains) was formed 5 to 15 million years ago by an immense outpouring of lava from vents along the Oregon-Idaho border that covered this portion of the Pacific Northwest to depths of thousands of feet with igneous rock As the basalt-floored Portland Basin subsided, sediment eroded both from the nearby Cascade Mountain chain as well as from continental interior sources were carried into the basin by the Columbia and other, local, rivers and deposited as the Plio-Pleistocene Sandy River Mudstone (SRM) and overlying Troutdale Formation sandy to silty gravel Alluvial processes ranging from the catastrophic floods to everyday fluvial activity of local rivers and streams eroded channels in, as well as deposited additional sediment on, the SRM and the Troutdale The material related to the Missoula Flooding is generally referred to as "flood-deposits" and is further characterized by grain size as "coarse-grained" or "fine-grained" flood deposits A veneer of Willamette River alluvium derived from Willamette Valley sources to the south as well as reworked flood deposit sediments mantle the topography the Willamette has incised into the last flood-created surface Since at least the late 1800's human activity has extensively altered the Willamette River shore and created unique and variable shallow soil conditions in the project area

Regional stresses have also produced pervasive northwest-southeast and southwest-northeast faulting in the Portland region Several large, northwest-trending faults are mapped within one mile of the project area, including the Portland Hills Fault and the East Bank Fault (*Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon and Clark County, Washington,* Beeson and others, 1991 The Portland Hills Fault Zone is considered potentially active (*Seismic Design Mapping, State of Oregon* (Geomatrix, 1995) and *U S Geologic Survey Quaternary Fault and Fold Database* (USGS, 2007) We noted during our document review that in addition to the mapped faults the geotechnical data report for the West Side CSO project (Parsons Brinckerhoff, 2002b) reports a disparity in geologic unit elevations from borings located across the Willamette River from each other that infers a displacement of the CRB and SRM units as much as 130 vertical feet over this distance This suggests a previously unmapped fault along the axis of the Willamette that may have undergone as much as 0 04mm/yr displacement over the past 1 million years

Seismic hazard mapping for the project area is presented in the *Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Quadrangles, Oregon and Clark County, Washington* (Mabey and others, 1993) These maps estimate relative hazard from ground motion amplification, liquefaction and lateral spreading/slope instability The ground motion amplification map shows the site is in the 1 to 1 4 amplification factor hazard zones, the lowest and second-lowest zones based on a six zone rating system An amplification factor of 2 5 or greater is the highest relative hazard and 1 the lowest The thickness of liquefiable sediment (liquefaction hazard) is ranked in the highest hazard level, at greater than 9 meters (about 30 feet) of liquefiable soil potentially present in the area

A seismic hazard particular to the project area is lateral spreading As described in *Liquefaction of soils during earthquakes* (NRC, 1985), this ground movement occurs when large blocks of ground are displaced down gentle slopes or towards stream channels as a result of liquefaction of subsurface soil during an earthquake Man-made seawalls and loose, saturated sandy fills can increase site vulnerability to this hazard, and the presence of these features is reflected in the classification of the bulk of the project area in the highest (estimated lateral ground displacement in excess of 1 2 meters, about 4 feet) and second highest (displacement of 0 9 to 4 meters, about 3 to 4 feet) hazard zones (Mabey and others, 1993)

3.2. SITE HISTORY

Prior to European settlement site geomorphology was dominated by the Willamette River meandering through what is now the project area across a broad, low alluvial floodplain Tributary streams drained the West Hills and adjacent lowlands to join the Willamette through extensive bankside wetlands, sloughs and lakes The project area would have been characterized by a gradual transition from elevated upland through riparian border to open river, extensive swamp and slough areas adjacent to the Willamette as well as impermanent riverbanks prone to relocation by erosion and deposition during floods

Economic development of Portland, as well as Euro-American building requirements, were not compatible with this transient, riparian riverside landscape Hardening of the west bank of the Willamette began as early as the middle 1800's, with the construction of timber and stone bank armor, piling for wharves and timber seawalls, filling of wetlands and pipeline-enclosure of tributary streams Man-made alterations in the project area since that time include, burial of the earlier structures during construction of the timber-crib-and-backfill Portland Seawall in the 1920's, concurrent construction of a brick-walled sewer as much as 7 feet in diameter, as well as widespread shallow soil alteration effected by construction and reconstruction of local pavements, building and the Burnside Bridge foundation elements between the 1920s and today (Parsons Brinckerhoff 2002a)

The portion of the project area west of Front Avenue (Naito Parkway) has been urbanized almost since the earliest settlement of Portland This has included the construction, demolition and reconstruction of generations of structures ranging from wooden temporary sheds to modern steel-reinforced concrete buildings Many of the earlier building materials were not removed from the site after demolition and should be anticipated within the shallow fill layer Along with this demolition debris, earlier human occupation has left behind a mixture of metals, organic compounds including petroleum hydrocarbons, polycyclic aromatic hydrocarbons, volatile and semi-volatile organic compounds (VOCs and SVOCs), pesticides, cyanide, and polychlorinated biphenyls (PCBs) The possible presence of these contaminants in the project area is discussed in the *Environmental Data Report, West Side CSO Tunnel, Shafts, Pump Station and Pipelines Project* (Parsons Brinckerhoff, 2002c) as well as summarized in the "Fill" section, below

3.3 SITE CONDITIONS

We conducted a site reconnaissance to observe existing conditions We focused our observations on paved areas and other surface conditions within the project area Key areas include

- Southwest Ankeny Street west of First Avenue which is a narrow asphaltic concrete (AC) paved street
- Southwest First Avenue in the area of the max light rail station and the station area which are paved with cobblestone
- Southwest First Avenue north of the Burnside Bridge which is paved with AC pavement
- The Ankeny Plaza
- Waterfront Park within the project area

Most pavements and hard surfacing we observed in and adjacent to streets within the project area are in good condition The AC pavements and cobblestone surfacing show occasional indications of local distress mostly related to edge effects/transitions between differing material types, areas of past repairs, and tree root displacement Areas of broad failure from an inadequate pavement section or poor subgrade conditions were not apparent, although apparent fatigue cracking of pavements was observed at the intersection of Ankeny and Second Photos 1 through 3 in Appendix A show typical paved surfaces in the project area with either no distress or only localized distress Photo 4, Appendix A shows pavement cracking at Ankeny and Second, the most significant damage observed in the project area

The paved surfacing of Ankeny Plaza shows the most distress The surfacing consists of square stone pavers The surfacing has numerous areas where pavers exhibit cracking We observed that most of the cracking occurrs linearly and at right angles to site features and other distressed areas Most of the linear cracking terminates at site features such as trees, poles and buildings These features suggest that the cracking is most likely due to inadequate trench compaction where power, water, or other utilities are run between the noted elements

Hard surfaces within Waterfront Park are in good condition No indications of settlement, cracking or other distress were observed in pavements along the seawall or walkways between Naito Parkway and the Seawall Photo 5 in Appendix A shows typical hard surfaces within Waterfront Park

4. GEOTECHNICAL DOCUMENT REVIEW

We reviewed a total of thirteen geotechnical documents prepared by others for the project area These reports are listed below and the approximate locations of the investigations are shown on Figure 2

Six geotechnical reports prepared for sites located within or less than two city blocks from the project area These are

1 Northwest Testing Laboratory (NTL) 1979 report for an office building located at 50 S W Pine Street,
2 Geotechnical Resources Incorporated (GRI) 1986 report prepared for the parking/helistop building located at 33 N W Davis Street,
3 Geocon 2001 report prepared for the Union Gospel Mission located at 3 N W 3rd Avenue,
4 Parsons Brinkerhoff West Side CSO Project geotechnical baseline report (2002a),

5 Parsons Brinkerhoff West Side CSO Project final geotechnical data report (2002b and appendices), and
6 Parsons Brinkerhoff West Side CSO Project final environmental data report (2002c)

Four geotechnical reports prepared for sites located more than two but less than four blocks from the project area These are

7 Shannon and Wilson (S&W) 1961 report prepared for the Northwest Natural Gas service building located at 123 N W Flanders Street,
8 AGRA Earth & Environmental (AGRA) 1997 report for the Port of Portland Building located at 123 N W Everett Street,
9 GeoEngineers, Inc (GEI) 1998 report prepared for the Classical Chinese Garden located at 239 N W Everett Street, and
10 GRI 1999 report prepared for Pacific House located at 333 N W 4th Avenue

Two geotechnical reports prepared for sites located more than four blocks from the project area These are

11 GRI 1985 report prepared for the Old Imperial Hotel located at 407 S W Broadway Avenue, and
12 GRI 1997 report prepared for the Kalberer Apartments located at 221 N W 5th Avenue

One pavement-specific report near the project site

13 Pavement Services, Inc (PSI) 2005 pavement investigation report prepared for Burnside and Couch Streets between N W 14th and N E 3rd Avenues

The primary findings of these documents are summarized in Table 1 below

Table 1 Ankeny Project Area Selected Geotechnical Report Data 1961-2002

Report No	Location	Prepared by, date	Subsurface investigation	General soil description	Depth encountered (ft below surface)	Blow count range (average)	Groundwater level (ft. below surface)
1	50 S W Pine St	NTL, 1979	1 boring to 22 5 ft max 2 test pits to 12 ft max	Debris fill	0-1 5 to 4 ft	11	Not encountered
				Silty alluvium	1 5 to 4 – 18 5 ft	9-18	
				Gravel	Below 18 5 ft	30/3"	
2	33 N W Davis St	GRI, 1986	4 borings to 111 ft max	Silty fill with debris and contaminants	0 – 5 to 20 ft	2 – 60/4"	22 5
				Silty and sandy alluvium	5 to 20 – 25 to 101 ft	3-34	
				Gravel	Below 25 to 101 ft	43-60/2"	
3	3 N W 3rd Ave	Geocon, 2001	1 boring to 41 5 ft max	Silty-clayey fill with debris	0-7 5 ft	4-16	Not determined
				Silty to clayey alluvium	7 5-25 ft	4-12	
				Silty flood deposits	25-33 ft	3-6	
				Gravel	Below 33 ft	47-50/4"	
4	West side of the Willamette River	PB, 2002(a) (Baseline report)	See PB (2002c)	Generally as described for PB (2002c)			
5	West side of the Willamette River	PB, 2002(b) (Geotech data report)	See Appendix G and PB (2002c)	Generally as described for PB (2002c)			
6	West side of the Willamette River	PB, 2002(c) (Enviro data report)	11 borings to 166 ft max	Sand/silt with debris fill	0-3 to 31 ft	3-13	23 ft
				Silty/sandy alluvium (Borings A-1 and B-1)	3 to 22 – 30 ft	6-10	
				Unconsolidated Sand and Gravel	22 to 31-46 to 65 ft	19 to 50/0"	
				Consolidated Gravel	Below 46 to 65 ft	50/3"	

7	123 N W Flanders St	Shannon & Wilson, 1961	6 borings to 141 5 ft max	Silty to clayey fill with debris	0 – 20 ft	None	10-20'
				Silty, organic silt and peaty alluvium	20-90 ft	None	
				Silty and sandy alluvium	Below 90 ft		
8	121 N W Everett St	AGRA, 1997	2 borings to 110 ft max 3 CPT to 113 ft max	Silty fill with debris	0-15 to 23 ft	5-13	~20 (estimated)
				Silty alluvium	15 to 23 ft – 30 to 140 ft	4-34	
				Gravel	Below 30 to 140 ft	None	
9	239 N W Everett St	GEI, 1998	9 borings to 21 5 ft max	Silty debris fill	0-8 to 10 ft	4-54	Not encountered
				Silty and sandy alluvium	8 to 10 – 21 5 ft	6-18	
10	333 N W 4th Ave	GRI, 1999	3 borings to 65 ft max	Debris fill	0-8 to 10 ft	2-21	19-23'
				Silty/Sandy alluvium	8 to 10 ft -30 ft	12-16	
				Gravel	Below 30 ft	8-50/6"	
11	407 S W Broadway Ave	GRI, 1985	3 borings to 29 5 ft max 4 hand augers to 14 ft max	Silty fill (street side only)	0-9 to10 ft	5-30	Not encountered
				Silty and sandy alluvium	9-10 to 22 ft	9-33	
				Gravel	Below 9 to 22 ft	50/2"	
12	221 N W 5th Ave	GRI, 1997	3 borings to 65 ft max 6 test pits to 11 5 ft max	Debris fill	0-7 to 10 ft	9-26	~20 (estimated)
				Silty/sandy alluvium	7 to 10 – 30 to 37 ft	4-15	
				Gravel	Below 30 to 37 ft		
13	Burnside-Couch Couplet	PSI, 2005	3 pavement and shallow soil cores to 5 ft max	Silty and sandy gravel	Below pavement to 32 in max	N/A	Not encountered
				Silt (C-4 only)	Below gravel to 5 ft	N/A	Not encountered

4.1. SUMMARY OF REPORTED SUBSURFACE CONDITIONS

The above referenced geotechnical investigations reported a wide variation of soils underlying the project area However, the majority of the reports show a soil profile across the area that can be generalized as a four layer sequence, consisting of the following units in order from the surface to the deepest

1 A surface layer of fill, consisting predominately of silty and clayey soil with minor man-made debris, but ranging to debris fill composed of man-made materials such as brick, concrete, wood and metal

2 A layer of alluvial soil ranging from silt and clay to lesser sand with gravel

3 A layer of unconsolidated sand and gravel

4 A layer of consolidated sand and gravel

Each of the reported characteristics of these soil groups is discussed in the paragraphs below in order of youngest to oldest

4.1.1. Fill

4 1 1 1 General Occurrence

The widest variety of thickness and material types is reported from this uppermost layer of man-made fill In general, all the investigations in and around the project area found some thickness of fill at the ground surface and extending to depths ranging from as shallow as 1½ feet below ground surface (bgs) (NTL, 1979) to as deep 31 feet bgs (PB 2002c) This material was used to backfill the seawall forming the east edge of the property and is generally deepest along the seawall, and shallows to the west Descriptions of this material ranged from a deposit composed largely of soil with a trace of rock fragments and debris to that composed entirely of artificial materials Silt predominated in the descriptions of the soil fraction of the fill, with clay, sand and gravel mentioned as well Brick was the most commonly mentioned artificial material, along with rock, concrete, organic material including sawdust and wood, metal, glass and ash

4 1 1 2 Near Site

The most comprehensive information near the site reported from shallow soil investigations is contained in the various Ankeny Shaft investigations performed for the CSO Project (PB 2002a, PB 2002b and appendices, PB 2002c) A total of five borings were drilled in and around the location of the proposed Ankeny Shaft, in Waterfront Park approximately 120 feet east of the intersection of S W Ankeny and Naito Parkway Three borings, PB-306R, AB-1, and AB-3 are reported by PB as having been drilled in Waterfront Park (PB 2002c), Fujitani Hilts & Associates (FHA) reports drilling two borings, A-1 and A-1a, in the lot southwest of the intersection of S W Naito Parkway and Ankeny, and three more, B-1, C-1 and D-1 in Waterfront Park around the proposed Ankeny Shaft (PB 2002b, Appendix G) The boring reported in PB (2002c) as AB-1 is the same boring as B-1 reported in PB (2002b Appendix G), the boring reported in PB (2002c) as AB-3 is the same boring as D-1 reported in PB (2002b Appendix G) For clarity, these borings will be identified by the FHA designations, B-1 and D-1

Borings A-1 and A-1a encountered a three-foot thick veneer of silty fill, the thinnest fill material section reported The fill encountered in the remaining five borings ranged from loose to medium dense sand and silty sand and soft to stiff silt and clayey silt to depths ranging between 22 and 31 8 feet bgs in B-1 and D-1, respectively The remaining FHA boring (C-1) reports a similar section 25 feet of loose to medium dense sand, silty sand and sandy silt All of these borings report a variety of man-made components including brick, gravel, concrete, wood, organics and metal

PB-306R encountered a very different fill section, consisting of clay, silt sand and gravel to approximately 5 feet bgs overlying a cobble- and boulder-sized rock rip-rap fill from 5 to 21 feet bgs This in turn was underlain by what PB (2002c) reports as a "timber pile" from 21 to 29 feet bgs The lowermost three feet of fill reported in this boring consisted of gravel with silt, wood and sand

No moisture data is reported from fill encountered in PB-306R, and the rotosonic drill used for this boring is not equipped to drive a soil sampler The remaining borings reported that Standard Penetration Test (SPT) blow counts from samples driven in sandy and silty fill materials range from 3 blows per foot (bpf) to 20 bpf, averaging 8 bpf

Laboratory test results from fill soils included moisture content, mechanical grain size determination, and washed sieve (percent fines) tests Reported moisture contents ranged from 6 percent in sandy fill samples to 38 percent in silty fill samples (PB, 2002b, PB 2002c) Mechanical grain size determinations and washed sieve/fines content tests were reported only from B-1 Reported grain size test results ranged from gravelly silty sand, Unified Soil Classification System (USCS) soil class SM at 2 5-12 5 feet bgs, to silty sand (USCS class SM) and sandy silt (USCS class ML) to 25 feet bgs Washed sieve/ fines content ranged from 3 percent finer than the #200 sieve (silt and clay) at 7 5 feet bgs to 94 and 95 percent silt and clay at 15 and 17 5 feet bgs, respectively (PB, 2002b, Appendix G)

As part of a pavement investigation, two samples of silty fill subgrade material from street locations immediately northwest of the project area were tested to provide a modulus of subgrade reaction (M_r) The reported test results yielded a M_r of 3,400 pounds per square inch (psi) for a sample taken from boring B-5 located at W Burnside Street and 3rd Avenue, and a M_r of 8,400 to 10,300 psi for a sample taken from boring C-4 located at N W Couch Street between 4th and 5th Streets (PSI, 2005)

Difficult drilling conditions reported from borings in the fill suggest that planned excavations might encounter potential difficult or complicated subsurface conditions Examples of drilling difficulties reported from the project area include loss of drilling fluid at 17 feet bgs and 34 5 feet bgs reported from boring PB-305A, drilled in Waterfront Park about 50 feet southeast of the intersection of S W Ash Street and Naito Parkway, and hard drilling from the surface to 29 feet bgs reported from PB-306R

Chemical and metal contaminants have been reported from the fill layer, both within and nearby the Waterfront Park portion of the project area "Chemical odors" and an "oil-like material" are reported from the surface and soils above 11½ feet bgs during an investigation at 33 N W Davis Street (GRI, 1986) Contaminants reported from borings AB 1 and AB-3 include detectable levels of arsenic, barium, chromium, lead, mercury, lead TCLP and petroleum hydrocarbon (PB, 2002c)

4.1.2. Unconsolidated Silt and Sand (Alluvium/Fine-grained Flood Deposits)

4 1 2 1 General Occurrence

All the investigations reported that native alluvial soils underlie the fill material These alluvial deposits also vary considerably in soil type and thickness Silt is the most commonly reported soil type, with clay and fine sand usually mentioned, either as distinct layers within the alluvial silts or as sandy silt to silty sand soil types Organic silt and peat were reported from an investigation north of N W Flanders (S&W, 1961) as well as traces of gravel or gravel layers throughout the project area Consistency of the alluvial materials is reported to range from soft to very stiff and density from medium dense to dense Reported thickness of these alluvial materials varies as well, from as little as 10 feet of interbedded silt and sand (GRI, 1999) to as much as 120 feet of clayey silt, organic silt, peat, silt and fine sand (S&W, 1961)

4 1 2 2 Near Site

Near the project site, a significant thickness of fine-grained alluvium is reported from FHA boring A-1 southwest of the intersection of S W Naito Parkway and Ankeny This boring penetrated 27 feet of soft to stiff silt and lean clay and sandy silt and very loose to loose silty sand from 3 feet bgs to 30 5 feet bgs

The alluvium reported in the borings within the Waterfront Park area is predominantly coarse-grained flood deposits The only fine-grained alluvium reported in these borings is from the log of B-1, where a 5-foot thick layer of loose to medium dense sandy silt is reported underlying the fill at a depth of 22 feet bgs Standard Penetration Test (SPT) blow counts from samples driven in this layer range from 2 bpf to 11 bpf, averaging 7 bpf (PB, 2002b, Appendix G, PB 2002c)

Laboratory test results from fine-grained alluvial soils included moisture content, mechanical grain size determination, washed sieve (percent fines) tests, dry density, torvane, and Atterberg Limits tests Reported moisture contents ranged from 7 percent to 41 percent (PB, 2002b, PB 2002c) Reported grain size test results ranged from clayey silt to sandy clayey silt (USCS class ML) at 5 and 10 feet bgs in A-1a Washed sieve/ fines content ranged from 42 percent finer than the #200 sieve (silt and clay) at 25 feet bgs in B-1 to 95 percent silt and clay at 15 feet bgs in A-1 Dry density results reported from A-1a ranged from 83 7 pounds per cubic foot (pcf) at 5 feet bgs to 84 8 pcf at 10 feet bgs Torvane readings from the same samples ranged from 0 84 tons per square foot (tsf) at 5 feet bgs to 0 63 tsf at 10 feet bgs A single Atterberg Limits test reported from the 10 feet bgs sample taken in A-1a produced a liquid limit of 49, a plastic limit of 26 and a plasticity index of 23, indicating that the soil should be classified as a lean clay (USCS class CL) (PB 2002b, Appendix G)

4 1 3. Unconsolidated Sand, Silt and Gravel (Alluvium/Coarse-grained Flood Deposits)

4 1 3 1 General Occurrence

A heterogeneous zone of cobbles, gravels, silt and sand was generally reported below fine-grained alluvium This sediment is typically described as a massive deposit of rounded to subrounded gravels to cobble and occasionally boulder size supported in a sand- to clay-size matrix, but some sandy and silty interbeds are reported in Boring AB-3 (PB, 2002c) Elevation of the top of this gravel ranges from approximately 18 feet bgs to 65 feet bgs across much of the project area, and the thickness of the deposit is difficult to determine from most published reports, since most explorations terminate in the upper portions of this unit

We noted that the top of this deposit is reported to drop off steeply north of N W Davis Street and east of N W 1st to 2nd Avenue top of gravel is reported at 30 feet bgs at the corner of N W 1st and Davis, the southwest corner of the block, but at 101 feet bgs at the corner of N W Naito Parkway and Everett, the northeast corner (GRI, 1986), similarly the depth to the top of gravel is reported at 30 feet bgs at the corner of N W 2nd and Everett and at 140 feet bgs at N W 1st and Flanders (AGRA, 1997) North of N W Flanders the gravel was not encountered to a depth of 140 feet bgs (S&W, 1961) This abrupt decline in the depth to the top of gravel is probably related to a paleo-Willamette River channel incised into the gravels, and may be encountered at depth under the farthest easternmost portions of the project area

4 1 3 2 Near Site

Near the site, the three PB borings do penetrate the unconsolidated gravel underlying Waterfront Park Boring PB-306R reports the thickness of the unit as 14 feet, AB-1 reports 24 feet and AB-3 34 feet All the investigations we reviewed report this material as dense to very dense, SPT blow counts from the gravel portions of the deposit in borings AB-1 and AB-3 range from 19 to denser than 50 blows for no penetration (50/0"), averaging 57 bpf Measured moisture contents range from 10 percent to 20 percent

4.1.4. Consolidated Sand and Gravel (Troutdale Formation)

Typically the oldest material reported from the project area is the Plio-Pleistocene Troutdale Formation gravel Where distinguished from the unconsolidated alluvial gravel above, this deposit is characterized as a very dense, yellow to green and gray sandy gravel in a partially cemented, silty, sandy and clayey matrix Although usually described as gravel, the log of Boring AB-3 reports that the top 20 feet of this deposit are "slightly clayey, very dense sand" (PB, 2002c) Troutdale sands and gravels are typically

described as overconsolidated Although mapped underlying the entire project area (Beeson and others, 1991), the Troutdale Formation is identified by name in only three locations at depths ranging from 30 feet bgs to 140 feet bgs underlying the Port of Portland Building at 1st and Everett (AGRA, 1997), at a depth of 50 feet bgs in A-1 southwest of the intersection of S W Naito Parkway and Ankeny, and in the boring logs from Waterfront Park, at 46 feet bgs in PB-306R, 51 3 feet bgs in B-1 and 65 feet bgs in D-1 The Troutdale Formation is fully penetrated in only one location in our project area, the log of PB-306R reports the thickness of the Troutdale as 93 feet Reported blow counts in Troutdale Formation deposits (both sand and gravel) exceed 50 blows for 6 inches

4.1.5. Groundwater

Groundwater levels reported for the project area range between as high as 10 feet bgs to as low as 25 feet bgs, however, the average depth to groundwater is more commonly between 18 and 23 feet bgs When the elevation of the site under investigation is compared to the level of the Willamette River this depth usually corresponds to an elevation within several feet of the mean river level, suggesting that this groundwater is connected to, and will reflect changes in, the level of the waters of the Willamette River Our experience and the groundwater levels interpreted by others (PB, 2002a) suggest that the groundwater levels will tend to be a few feet higher than the river level

5. CONCLUSIONS

Based on our review of the work completed by others, the site is suitable for the proposed development The following factors will most influence the project design

Heterogeneous soils - the project lies in an area underlain by heterogeneous fill to depths varying from a few feet to tens of feet The fill is mostly silty, but has areas of rock, wood debris and obstructions Conditions encountered within the project area could consist of any of these types of materials, but are most likely to be silty Because of the highly variable nature of the fill materials and lack of site-specific explorations, reliable soil properties could not be determined for design, so the following approach was used

- Conservative soil parameters were selected for geotechnical design
- Site specific explorations should be completed for foundations and any other critical areas, once selected, including the cover structure, restroom, and water feature, to confirm soil conditions, as discussed in Section 6 2 1
- The potential for changed conditions will be high for this project due to the highly variable fill present The project budget should include such consideration
- Significant geotechnical oversight should be anticipated to evaluate actual subsurface conditions encountered compared to design and facilitate field changes as necessary

Pavements – The extent or scope of expected pavement activities have not been determined at the time this report was prepared We anticipate that pavement activities will limited to replacement or repair of existing asphalt paving on local arterial streets and construction of new asphalt concrete (AC), Portland cement concrete (PCC) or pavers for pedestrian areas The suitability of the proposed pavement subgrades should be verified during construction by proofrolling which should be observed by a qualified individual Unsuitable soils should be scarified and recompacted or removed and replaced with structural fill The actual depth and extent of overexcavation will need to be addressed on an individual basis

Shallow Foundations – Shallow foundations should be founded in areas with site specific investigations as noted above Where suitable soils are present, lightly loaded foundations are appropriate Total and differential settlements may be high and should be accounted for in design This could be achieved with

flexible buildings, ground improvements, or some other means of support depending on actual site conditions

Deep Foundations – Deep foundations are not generally anticipated for the lightly loaded structures for the project unless very poor conditions are encountered, or the cover structures create heavy foundation loads In these instances, deep foundations may be necessary Deep foundations can achieve high axial bearing in the gravels present below the site While investigations related to the Ankeny Shaft suggest that these materials should be encountered within 40 to 50 feet of the existing ground surface across most of the portion of the Park proposed for development, investigations performed immediately north and east of the Waterfront Park area suggest that some undetermined distance east of Boring AB-3, these deposits may drop off steeply to as much as 140 feet bgs or more Lateral capacity in the upper fill soils, however, will be relatively low

Excavations - The heterogeneous fill may contain abandoned pipelines, stone rip-rap, timber piles and cribbing Borings drilled through these materials reported problems ranging from drill fluid loss to difficult drilling Excavations that extend a significant distance into the fill may encounter difficult excavation due to these materials

Seismic Hazards – The site is subject to seismic hazards typical to the region In particular, horizontal displacement due to lateral spread and vertical settlement from liquefaction are expected Structures should be designed to prevent collapse at a minimum in the event of these hazards

Soil Contaminants - Some areas of the project may encounter fill that contains contaminants including metals, organics and petroleum hydrocarbons Excavations which extend into such soils should be performed with the expectation of encountering such contaminants

Groundwater - Groundwater levels should be anticipated within several feet above the level of the adjacent Willamette River Excavations below these levels should anticipate encountering groundwater Dewatering may be required, and will vary depending on the type of soil present

6. RECOMMENDATIONS

The following sections provide our geotechnical recommendations for pavements and structural elements of the project As noted in Section 5 our approach for this project was to assume conservative soil parameters for design These will presumably be confirmed prior to or during construction and design changes will be made as necessary to account for the differing conditions, if present

6 1 EARTHWORKS RECOMMENDATIONS

6 1.1. Demolition Recommendations

Demolition of remaining existing structures will be necessary in new structure areas We recommend that existing structures, foundations, concrete slabs and pavements within new structure areas be completely removed Debris generated from demolition should be transported off-site for disposal Existing asphaltic and cement concrete and base rock may be segregated, processed and reused as structural fill provided it is processed to a well-graded material with a maximum particle size less than 4 inches and mixed with suitable material so that voids do not occur in the fill Underground utilities in new facility areas should be identified and located prior to construction Existing utility lines that will be beneath new structures should be relocated or evaluated by Pacific Geotechnical if they are to be left in place Abandoned utility lines beneath structural components should be completely removed or grouted full if left in place to reduce the potential for pipe collapse and eventual settlement of new structures

Excavations resulting from removal of existing foundations, utilities or other subsurface elements should be backfilled with structural fill The bottoms of the excavations should be carried to firm subgrade soils or to the depth recommended by a qualified geotechnical engineer

Existing voids and new depressions created during site preparation and demolition should be cleaned of loose soil or debris down to firm soil and backfilled with structural fill Also, soft or loose backfill soils encountered in existing utility trenches should be removed and replaced with structural fill where they are located within new structure areas

Existing site vegetation should be removed from all proposed building and pavement areas and at least 5 feet beyond these areas Stripping depths in areas covered with vegetation will vary depending on vegetation type but could be up to 4 to 6 inches Greater stripping depths may be required to remove localized zones of soft or organic soil

Trafficability of the on-site soils, where exposed by removal of existing pavement or in existing unpaved areas, will generally be difficult during the normally wet season of the year (typically early October through late May) or when the moisture content is more than about 3 percent above optimum During such conditions, measures such as those described in Section 6 1 3 would help protect site soils

Pacific Geotechnical did not sample the existing structures for lead based paint, asbestos insulation or tile backing, or any other material that would require special handling during demolition

6.1 2. Subgrade Preparation

Existing fill at the site is highly variable Subgrade soil conditions should be determined in foundation areas and other critical areas per Sections 5 and 6 2 1 If unsuitable fill is encountered in these areas, it should be removed and the excavation backfilled with structural fill as described in Section 6 2 Fill beneath other structural elements (pavement and walkway areas, for example) should be evaluated by proofrolling as described below If proofrolling verifies it is suitably dense, it can be left in place Otherwise, it should be scarified and recompacted or removed and replaced with structural fill

If the subgrade soils deflect under an initial proofroll, the exposed subgrade should be scarified and compacted such that the top 1 foot of subgrade soil is compacted to at least 92 percent of the maximum dry density as determined in accordance with ASTM Test Method D 1557 The moisture content of the subgrade soils should be at or near optimum for compaction, so may need to be moisture-conditioned Scarification and compaction of existing soils should not be attempted during wet weather

After subgrade preparation has been completed, the suitability of new subgrade areas should be reevaluated by proofrolling with a fully loaded dump truck or similar heavy rubber-tire construction equipment Areas of excessive yielding identified during proofrolling should be recompacted, if practical, or the soft, loose or otherwise unsuitable soils removed and replaced with compacted structural fill We recommend that a qualified geotechnical engineer be present during proofrolling to aid in identifying areas that may need additional compaction or other remedial work During wet weather, the subgrade should be evaluated by probing rather than proofrolling, as proofrolling could cause excessive disturbance of the subgrade soils This should be completed by a qualified geotechnical engineer

6.1.3. Wet Weather Earthworks

When site soils are wet, they will be easily softened by equipment to the point that they may not adequately support structures or construction equipment In such conditions, wet weather earthwork procedures may be necessary to protect the subgrade

Haul roads will likely be needed during persistent wet weather They should consist of imported crushed rock compacted to a dense state The initial lift of fill over the exposed native soil subgrade should be placed in a single lift of at least 12 inches thick and compacted without vibration using a smooth-drum roller to reduce potential disturbance of the subgrade Additional rock may be needed to protect the subgrade and support construction equipment, depending on traffic

Placement of geotextile over the exposed subgrade may be appropriate in construction traffic or staging areas before crushed rock is placed If geotextile is used, it should have a Mullen burst strength of at least 500 pounds per square inch (psi) and an apparent opening size (AOS) between the U S No 70 and No 100 sieve to reduce migration of fines into the structural fill soil Propex 2019 and Mirafi 1100N are two geotextiles that meet these criteria

Where concurrent with final pavement areas, the haul road section can serve as the aggregate base for final pavements provided the base section is not contaminated with fine-grained soils It may be prudent to leave the haul roads at least four inches below paving grades to allow a final layer of fresh base immediately before placing pavement

6.1.4. Excavation Stability

The on-site soils are generally OSHA Class C for excavation purposes It is unlikely that groundwater will be encountered in excavations expected for this project It groundwater is encountered, however, dewatering or shoring will be necessary

Permanent slopes higher than 4 feet are not planned as a part of this project If such slopes become necessary, we recommend that they be inclined no steeper than 2H 1V (horizontal to vertical) Slopes that will be mowed should be 3H 1V or flatter

We recommend that temporary cut slopes in the fill soils be inclined no steeper than 1 5H 1V (horizontal to vertical) provided they are not subjected to seepage Surface loads should be kept a minimum distance of at least one- half the height of the cut away from the top of the temporary slope

Trenches may not stand vertically where soft fill soils are encountered Standard trench boxes or shoring should be suitable to keep shallow trenches open provided groundwater is not encountered and structures are not near excavations

Temporary cut slopes and any trench boxes or shoring must comply with applicable state and local regulations The contractor completing the earthwork should have the primary responsibility for establishing safe inclinations for temporary cut slopes, shoring requirements, protection of site personnel, and protection of adjacent structures and facilities

Temporary cut slopes should be protected from erosion and raveling Surface water should be prevented from flowing over the top of cut slopes

6.1.5. Structural Fill

6 1 5 1 General

Fill beneath building foundations and floor slabs, pavements, sidewalks and other settlement-sensitive structures should be placed and compacted as structural fill

Structural fill soils should be free of debris, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches, and other deleterious materials The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in

moisture content and achieving the required degree of compaction becomes more difficult or impossible Structural fill for specific applications are discussed below

6 1 5 2 On-Site Soils

The on-site soils are moisture sensitive, highly variable and may contain environmental contaminants We understand, however, that it may be desirable to reuse site soils as structural fill for sustainability purposes Provided the soils are not contaminated, the on-site soils can be used as structural fill during dry weather If used as structural fill, any deleterious materials should be removed from the soils as described above, including organic matter, debris, or rock particles large than 4 inches If construction occurs during wet weather, imported granular material should be used as structural fill

6 1 5 3 Recycled Materials

Asphalt concrete, cement concrete or over-sized rock can be used as structural fill provided they are processed by crushing and screening, grinding in place, or other methods to meet the structural fill recommendations for gradation and maximum particle size in this report They may be used as structural fill in all areas except within 3 feet of the perimeter of buildings

6 1 5 4 Imported Fill Soils

General

Requirements for imported fill will vary with the specific use Fill for various uses described later in this report are provided in the following sections We recommend that samples of all import fill soils be submitted to Pacific Geotechnical for review and approval at least 7 days prior to beginning earthwork

Granular Embankment Fill

Imported fill for raising site grades should consist of pit run sand and gravel, quarry rock, crushed rock, or crushed gravel containing less than 5 percent fines (particles passing the No 200 sieve) by weight relative to the fraction passing the ¾ inch sieve

Granular Subbase Replacement Fill

Fill beneath foundations and slabs-on-grade to replace overexcavated subgrade soils should consist of imported crushed rock The gradation of the crushed rock should conform to Oregon State Department of Transportation (ODOT) Standard Specifications for Highway Construction, Section 02630 – Base Aggregate, Size 1-1/2" – 0

Pavement Subbase Fill

Aggregate base course for asphalt concrete and Portland cement concrete pavements should conform to ODOT Section 02630, Base Aggregate, Size 1-1/2" - 0, with the additional requirement that the base course contain less than 5 percent passing the No 200 sieve

Retaining Wall Drainage Zone Fill

Fill placed to provide a drainage zone behind retaining walls and for footing drains should consist of free-draining sand and gravel or crushed rock The material should conform to ODOT Section 00510 12 Granular Wall Backfill

Trench Backfill

Trench backfill for pipe base and bedding should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U S No 200 sieve The bedding soil should be placed around the sides of the pipe and a minimum of one-half the pipe diameter or 6 inches above the top of the pipe or in accordance with the manufacturer's recommendations The remainder of

the trench backfill should consist of either on-site or imported fill soils in accordance with the criteria included in this report

6.1 6. *Structural Fill Placement and Compaction*

Structural fill should be placed and compacted in accordance with the following

- Place all fill and backfill on a prepared subgrade that consists of firm, inorganic soils or approved structural fill
- Place all fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment Table 2 provides general guidance for lift thicknesses

Table 2 Guidelines for Uncompacted Lift Thickness

	Guidelines for Uncompacted Fill Thickness (inches)	
Compaction Equipment	**Granular and Crushed Rock Maximum Particle Size ≤ 1 1/2 inch**	**Crushed Rock Maximum Particle Size > 1 1/2 inch**
Hand Tools Plate Compactors and Jumping Jacks	4 – 8	Not Recommended
Rubber-tire Equipment	10 – 12	6 – 8
Light Roller	10 – 12	8 – 10
Heavy Roller	12 – 18	12 – 16
Hoe Pack Equipment	18 – 24	12 – 16

This table is intended to serve as a guideline and should not be included in the project specifications

- Use appropriate operating procedures to attain uniform coverage of the area being compacted
- Place fill at a moisture content within 3 percent of optimum as determined in accordance with ASTM Test Method D 1557 Moisture condition fill soil to achieve a uniform moisture content within the specified range before compacting
- Do not place, spread or compact fill soils during freezing or unfavorable weather conditions Frozen or disturbed lifts should be removed or properly recompacted prior to placement of subsequent lifts of fill
- Do not place fill and backfill until tests and evaluation of the underlying materials have been made and the appropriate approvals have been obtained
- Do not damage or displace underground utilities or adjacent structures during backfilling and compaction
- Grade the surface of the fill at the end of each working shift so that surface water can drain readily
- Compact fill soils to the percentages of maximum dry density as shown in Table 3

Table 3 Fill Compaction Criteria

	Percent of Maximum Dry Density Determined in Accordance with ASTM D 1557	
Fill Type	0 – 2 Feet Below Subgrade	>2 Feet Below Subgrade
Aggregate Bases	95	95
Utility Trench Backfill[1]	95	92
Nonstructural Trench Backfill	88	88
Retaining Wall Backfill[2]	95	95
Nonstructural Zones	88	88

Notes
[1]Compact to 90 percent of ASTM D 1557 for pipe bedding and in pipe zone, or as recommended by the pipe manufacturer
[2] Within 3 feet of the back of retaining walls, compact to a lower percent density of 92 to limit potential wall damage from high horizontal stresses

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by a qualified geotechnical engineer to verify that the specified degree of compaction is being achieved

6.1 7. PAVEMENTS

The extent and scope of expected pavement activities have not been determined at the time this report was prepared We anticipate that pavement activities will include replacement or repair of existing asphalt paving on local arterial streets and construction of new asphalt concrete (AC), Portland cement concrete (PCC) or pavers for pedestrian areas

Pavement subgrade areas prepared in accordance with the above recommendations should provide a California Bearing Ratio (CBR) of 3 This assumes the subgrade is constructed in dry weather If constructed during wet weather, special subgrade treatment or thicker base rock may be needed

The aggregate base course should conform to Section 6 1 5 4 and be compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM Test Method D-1557

Pavement design for roadways should be completed based on site specific traffic data once it has been determined Most pavement sections have suitably supported existing traffic loads, based on our reconnaissance, so matching existing sections would be a reasonable approach if traffic loads will be similar to past loads

6 2. STRUCTURAL DESIGN

As noted in Section 5, soil properties can be highly variable due to the uncontrolled fill present under most if not all of the site Although existing site facilities do not exhibit significant evidence of distress, new structures will impose new loading that could result in settlement or other soil-related problems The following sections provide our recommendations for structural components of the project assuming that subsurface conditions will be equivalent to, or better than, medium stiff silt soil, as noted in Section 5 A discussion of how differing soil conditions could affect the recommendations for significant site improvements are provided below where appropriate

6.2.1. Shallow Foundations

Shallow foundations bearing on medium stiff or better silt soil or on structural fill overlying such soil can be used to support lightly loaded structures and appurtenances Structural design can use the preliminary design recommendations in Table 4 below Either prior to or during construction, however, foundation conditions should be verified In general verification should consist of explorations via test pits or similar means to a depth of 10 feet or at least four times the greatest footing width of the specific structure, whichever is greater If unsuitable soils are encountered within this depth they should be removed and replaced with granular fill, as described in Section 6 2 1 2 If unsuitable soils continue below the exploration depths, additional geotechnical evaluations or alternate foundation support as described later in this report may be required The following paragraphs provide additional discussion

6 2 1 1 Bearing Capacity

Recommended preliminary bearing pressures are provided in Table 4 These are net bearing pressures, the weight of the footing and overlying backfill can be ignored in calculating footing sizes These bearing capacities can be used for wall loads up to 2 kips per linear foot and 20 kips per column When final loads are determined, we should be consulted to review the loads if they exceed these values

6 2 1 2 Bearing Surface Preparation

The site has highly variable fill, creating the potential for differential bearing performance and resulting differential settlement It is, therefore, likely that unsuitable soils will be encountered in footing excavations If present, such soils should be removed to the depth they occur, but not greater than four times the width of the largest footing of the structure The overexcavation can be backfilled to bearing grade with compacted granular fill as described in Section 6 1 5 The backfill should extend one foot wider than the supported footing for each foot of overexcavation depth Where such excavations transition to soil subgrade, we recommend that the based of the overexcavation slope at a 2H 1V gradient to meet the soil subgrade

Foundation bearing surfaces should not be exposed to standing water Should water infiltrate and pool in the excavation, it should be removed before placing crushed rock fill or reinforcing steel Loose or disturbed materials should be removed from bearing surfaces before placing reinforcing steel, and concrete

We recommend that an experienced geotechnical engineer observe all foundation excavations before placing reinforcing steel in order to confirm that adequate bearing surfaces have been achieved and that the soil conditions are as anticipated

6 2 1 3 Foundation Settlement

Shallow foundations constructed in accordance with our recommendations should experience settlements of up to about 1 inch Differential settlements of up to one-half of the total settlement magnitude can be expected between adjacent footings with similar loads

6 2 1 4 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of footings and by friction on the bearing surface using the values in Table 4 The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total

The passive earth pressure value is based on the assumptions that the adjacent grade is level and the static groundwater is below the base of the footing throughout the year The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the foundation area is covered with pavement or is inside a building

Table 4 Geotechnical Parameters for Structural Design

Bearing Stratum	Medium stiff or better silt or structural fill over such soil that is confirmed per report			
Bearing Capacity	Dead+Live Loads	1,000 psf	Wind/Seismic Loads	2,000 psf
Safety Factor	Dead+Live Loads	3 0	Wind/Seismic Loads	2 0
Minimum Depth	Exterior	18 inches	Interior	12 inches
Minimum Width	Strip Footing	18 inches	Column Footing	24 inches
Settlement	Total	1 inch	Differential	½ inch
Lateral Resistance	Application Shallow foundations, retaining walls			
Passive Earth Pressure [1]	200 pcf			
Friction Coefficient	Concrete/crushed rock	0 4	Concrete/silt fill	0 3

Note
[1] Does not include safety factor

6 2 1 5 Alternate Foundations

It is possible that soil conditions will be encountered that will be worse than the design assumptions above For example areas of rip rap/rock fill were encountered, as were areas of organic debris Occurrence of overly soft soils could lead to bearing failure or excessive settlement of site structures Combinations of such conditions within the building footprint could lead to potentially large differential settlements Such conditions could preclude the use of standard foundations as noted above In such cases, alternate foundations would be required Alternate foundations would most likely consist of subgrade improvement or deep foundations

Subgrade improvements would most likely consist of rock replacement elements within the building footprint such as stone columns or rammed aggregate piers The depth, spacing and final design parameters of such elements cannot be determined until specific conditions are known Typically, however, such elements would be completed through the unsuitable soil, or to some lesser depth that limited settlement to an acceptable level Spacing of such elements normally is in the range of six to eight feet on center These systems are commonly designed and installed by a specialty contractor to meet a performance specification Once the locations and loading of specific structures are known, explorations should be completed as recommended above to determine the need for such systems

Deep foundations, such as small diameter steel or grouted piles may also be used to support site structures Such piling would most likely be completed through the fill and soft alluvium and into the dense gravels Minimum depths of about 30 to 40 feet are expected, but could be much greater as noted in Section 5 Final design of such elements would depend on the specific conditions at the structure locations

6.2.2. Retaining Structures

Retaining structures are not anticipated in the preliminary design However, any necessary walls should be designed using the earth pressures in Table 5 These recommendations are based on the following assumptions

1 The backfill is level

2 The backfill consists of free-draining granular materials

3 Weep holes or a back drain will control hydrostatic pressures For embedded walls, either drainage will be assured or hydrostatic pressures will be balanced on both sides of the wall

4 The walls are less than 5 feet high

Surcharge loads applied closer than one-half of the wall height should be considered as uniformly distributed horizontal pressures equal to one-third of the distributed vertical surcharge pressure Footings for retaining walls should be designed as recommended for shallow foundations Subgrade conditions can affect retaining wall design recommendations for walls similar to their potential affects on footings as noted earlier

Table 5 Retaining Wall Design Parameters

Equivalent Fluid Unit Weight [1]	Free to rotate (active)	40 pcf	Restrained (at-rest)	60 pcf
Passive Resistance Equivalent [1]	200 pcf			

Note [1] Does not include safety factor

6.2.3. Slabs-on-Grade

Slabs-on-grade will generally consist of sidewalks and pedestrian areas with transitory heavier loading from activities associated with the PSM Such loads are expected to still be very light The subgrade reaction modulus provided in Table 6 can be used for design of such slabs This value assumes that the subgrade is proofrolled and prepared in accordance with Section 6 1 2 Slab settlement is estimated to be less than 1 inch for a load of up to about 150 pounds per square foot (psf) or less The interactive water feature can also be designed with this value, provided loading does not exceed the above assumptions If loading exceeds this value, we should be contacted to provide additional recommendations

The subgrade soils are non-expansive, heave is not expected beneath slabs-on-grade All slabs should be underlain by at least 6 inches of imported crushed rock material to aid as a capillary break and for subgrade leveling and support The crushed rock should conform to Section 6 1 5 4

Table 6 Slab-on-Grade Design Parameters

Application	Lightly Loaded Slabs-on-Grade
Aggregate Base Layer	6 inches of crushed rock
Modulus of Subgrade Reaction	100 pci

6.3. SEISMIC DESIGN PARAMETERS

The 2003 International Building Code (IBC) with 2004 State of Oregon Structural Specialty Code Amendments provides seismic design parameters for code-regulated structures in the state Based on subsurface conditions to depths of 100 feet bgs, the site is given a "Site Class" and the various coefficients determined based, in part, on the Site Class Although dense soils occur within about 30 to 40 feet of the ground surface, the upper fill and alluvial silts are expected to have 10 feet or more of very weak soils We, therefore, categorized the site as Site Class E Due to liquefiable soils as noted in the following section, the site could be classified as Class F, however, since structures are expected to have periods of vibration of equal to or less than 0 5 seconds, the values of Site Class E are permitted, so we utilized these values The following parameters provided in Table 7 are applicable to this site for the project based on Site Class E and provided that no structures have periods of vibration greater than 0 5 seconds

Table 7 Seismic Design Parameters (2003 IBC)

Site Class	E
Spectral Response Acceleration S_s	1 0 g
Spectral Response Acceleration S_1	0 35 g
Site Coefficient, F_a	0 9
Site Coefficient, F_v	2 6
Spectral Response Acceleration (Short Period), S_{DS}	0 90 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0 91 g

6 4 LIQUEFACTION POTENTIAL

Site subsurface conditions that we reviewed and analyses completed by others indicate that the site is susceptible to liquefaction and its related affects These affects include principally lateral spreading and vertical settlement Vertical settlements estimated by others at nearby locations have ranged from less than an inch to up to 5 inches Lateral spreading estimates by others have been reported to vary between about 2 inches to as much as 2 feet Borings at the site found areas of predominately loose sand and other areas of predominately silt Areas of sandy soils within the project area will experience greater displacements than those of predominately silt

The structural engineer should evaluate the potential for structural collapse under such conditions once the structural system has been selected Precast concrete would probably not meet code performance requirements under this loading condition, however, more ductile or redundant systems could If proposed structures will not perform acceptably under such conditions, a ground improvement program or deep foundations would be necessary We can assist with this aspect of structural design, if requested

7. CONSTRUCTION OBSERVATIONS

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration Recognition of changed conditions often requires experience, therefore, the project geotechnical engineer or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated

We recommend that the project geotechnical engineer or their representative be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork and foundation construction are being met In particular, we recommend that exploratory test pits as recommended earlier in this report, overexcavation of soft soils and suitability of foundation subgrades be observed by Pacific Geotechnical, LLC, prior to placing any fill or backfill Compaction of all structural backfill should be tested to confirm that the specified compaction is met Slabs and pavement subgrades should be observed and tested for compaction Full time observation of piling installation and subgrade improvements are also recommended, if they are completed

8. LIMITATIONS

We have prepared this report for use by Walker Macy, the City of Portland Bureau of Parks and the design team for this project Our evaluation was based on a surface reconnaissance and review of existing information only, as requested We did not complete subsurface explorations for this project Although the documents we reviewed are believed to be reliable, we cannot assure their accuracy

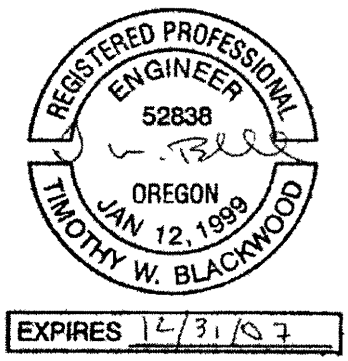
The opinions and recommendations contained within the report are based on work completed by others and limited site information and should not be construed as a warranty of subsurface conditions Within the limitations of scope, schedule and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared No warranty or other conditions, express or implied, should be understood

If the project information is incorrect or materially changes, please contact us so that our recommendations can be reviewed The discovery of any site and/or subsurface conditions during construction that deviate from that presented in this investigation should also be reported to us for review

9. CLOSING

We appreciate the opportunity to submit this report for your project Please contact us if you have any questions or need additional information

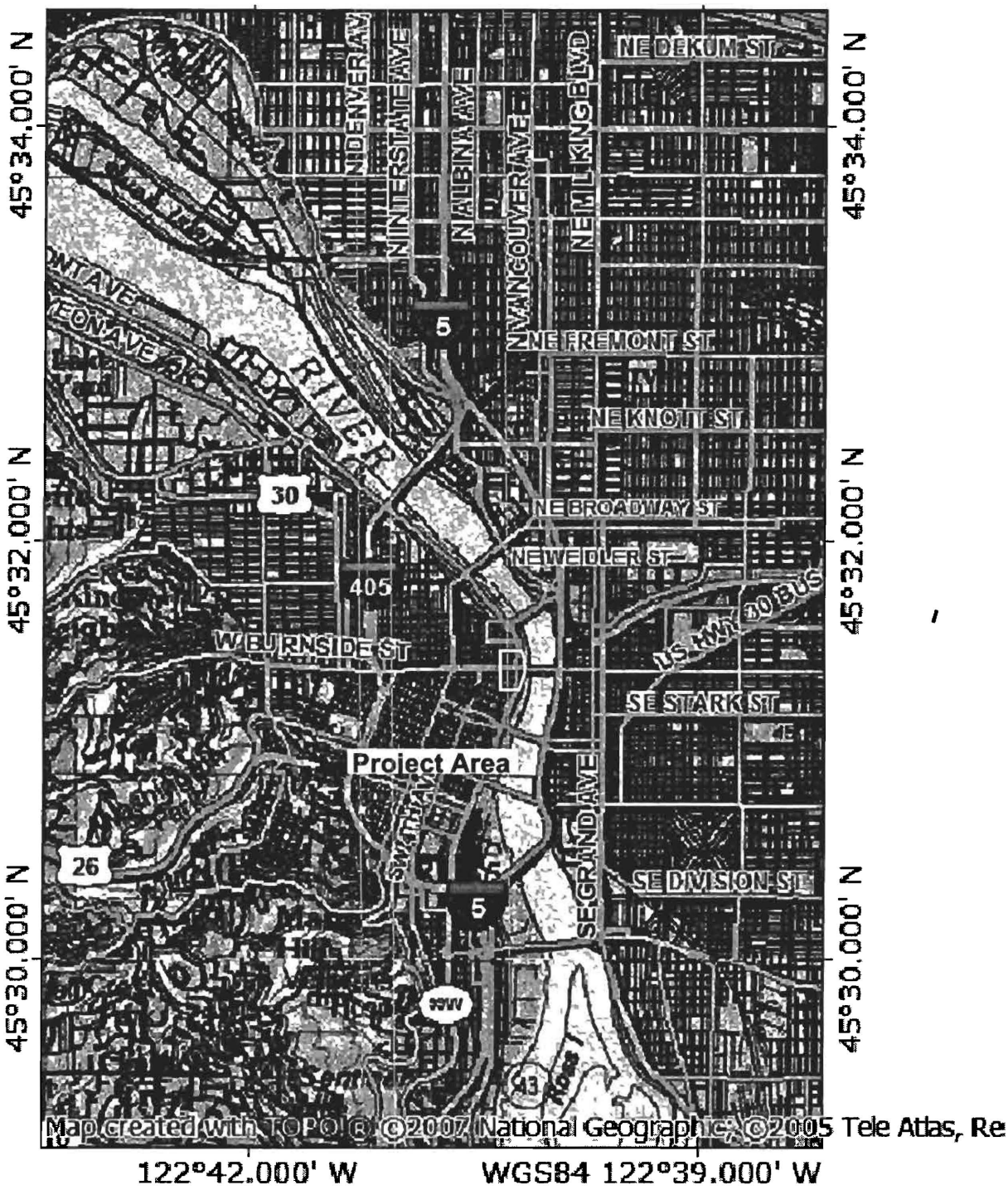
Sincerely,



Timothy W Blackwood, PE, CEG
President

Attachments

Document ID 1050-001-00R doc



NATIONAL GEOGRAPHIC

0.0 0.5 1.0 1.5 2.0 2.5 3.0 3.5 miles

0 1 2 3 4 5 km

N

DWG NAME	Vicinity Map
DESCRIPTION	PDX Ankeny Redevelopment Plan Portland, Oregon
JOB NO 1050-001-00	DATE 04/11/2007
FILE NAME Vicinity Map dwg	FIGURE 1

0 1000 2000

Scale in Feet

N

Legend

13 Approx location of report reviewed for this study with report number

For report numbers, see Table 1 in this data report

DWG NAME	Site Plan		
DESCRIPTION	PDX Ankeny Redevelopment Plan Portland, Oregon		
JOB NO	1050-001-00	DATE	04/11/2007
FILE NAME	Site Plan dwg	FIGURE	2

APPENDIX A – SITE PHOTOGRAPHS

Photo 1 Ankeny Plaza looking southeast from S W First Avenue

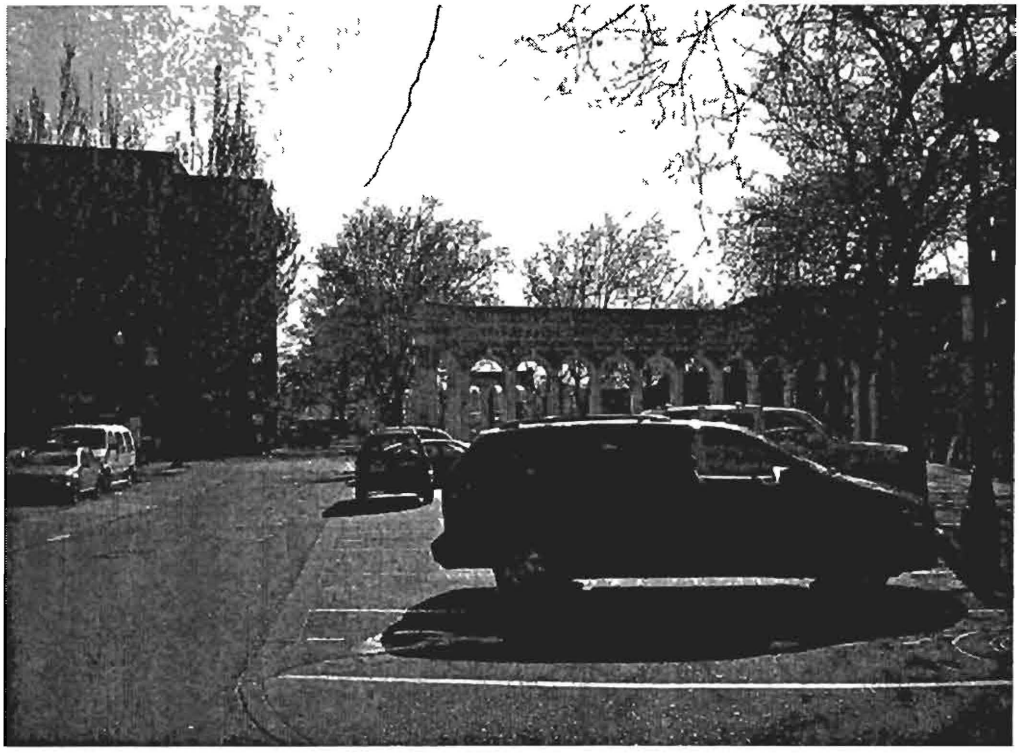


Photo 2 Ankeny Plaza looking east towards S W First from S W Second Avenue

SITE PHOTOGRAPHS	
Waterfront Park, Ankeny Plaza & street improvements	
Page 1 of 3	Appendix A

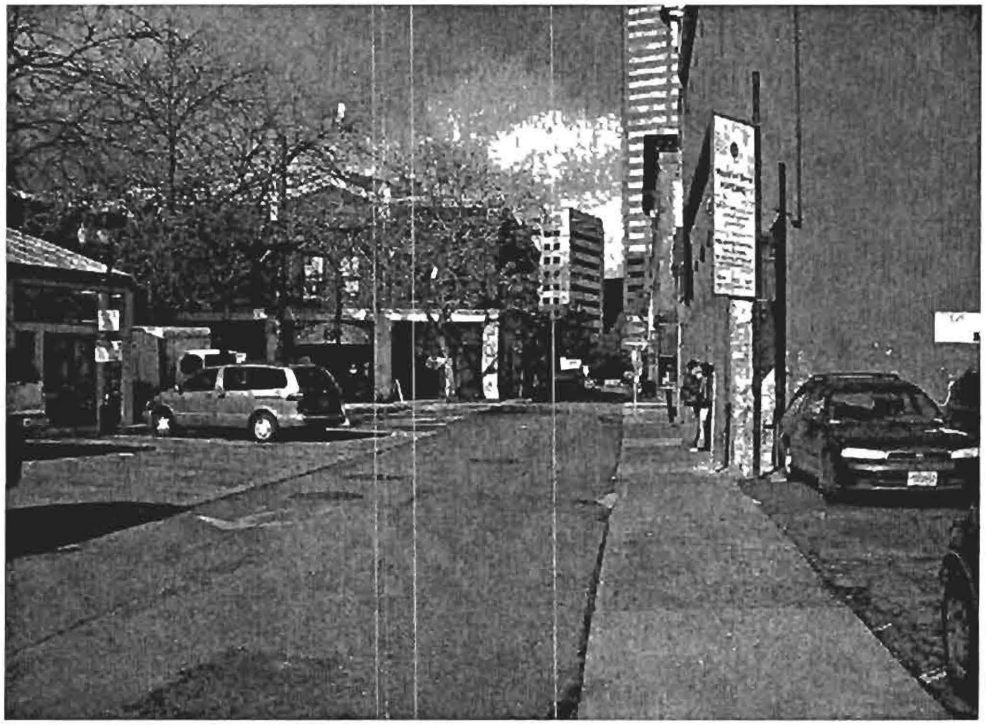


Photo 3 West side of Ankeny St. between S W First and S W Second Avenues
Note minor damage/fatigue at repaired locations

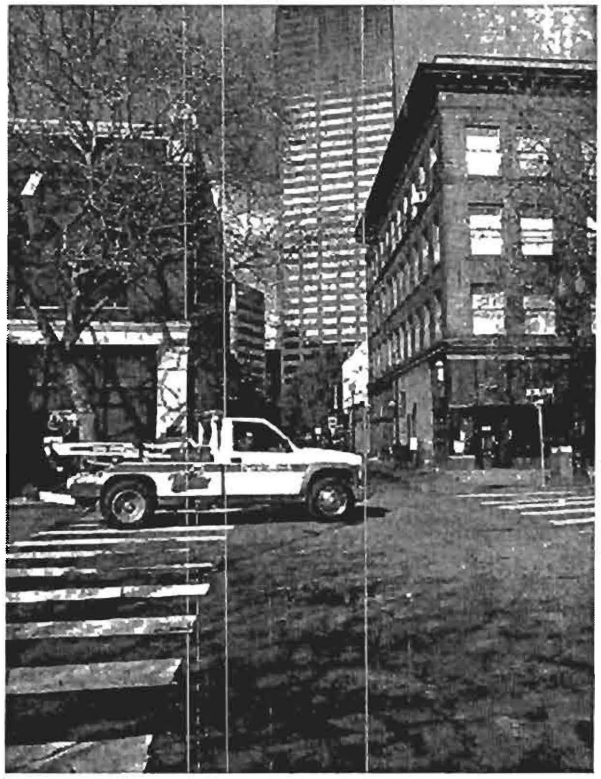


Photo 4 Intersection of S W Second Avenue and S W Ankeny Street looking west Note local damage and cracking (most significant observed in project area)

SITE PHOTOGRAPHS	
Waterfront Park, Ankeny Plaza & street improvements	
Page 2 of 3	Appendix A

Photo 5 Waterfront Park looking south from underneath Burnside Bridge

SITE PHOTOGRAPHS	
Waterfront Park, Ankeny Plaza & Street Improvements	
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