Geotechnical Engineering Report
City of Vancouver Water Station No. 1
Detailed Seismic Analysis Project
Vancouver, Washington

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

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VANCOUVER, WASHINGTON

1.0 INTRODUCTION

1.1 General

This report presents the results of our geotechnical evaluation to support the Vancouver Reservoir Seismic Retrofit Project in Vancouver, Washington. The Vicinity Map, Figure 1, shows the location of the project site and surrounding geographical features. Degenkolb Engineers is leading the design of the project and has subcontracted Shannon & Wilson, Inc. to provide geotechnical support to the design team.

1.2 Scope of Work

The scope of work for this geotechnical evaluation included the following:

- Review of the published and available geologic and geotechnical subsurface information (relevant previous boring locations shown on Figure 2 and boring logs attached to Appendix B).
- Exploring the subsurface conditions, ground water levels, and collecting soil samples from four (4) soil borings at the site (shown on Figure 2 and detailed in Appendix A).
- Conducting laboratory testing on selected samples to characterize the subsurface soils (details and results included in Appendix C).
- Conducting a Site-Specific Seismic Hazards Evaluation, including site-specific ground motion analysis to assist the seismic slope displacement evaluation, seismic slope stability, fault rupture, soil liquefaction, and settlement.
- Discussing the seismic slope instability hazard mitigation strategies, and recommended approaches to improve the seismic slope stability;
- Conducting a siting assessment for a 90-foot tall Stand Pipe reservoir, and developing conceptual recommendations for the foundation design;
- Providing recommendations for site earthwork including site preparation, excavation, groundwater control, structural fill, and compaction criteria; and
- Providing a written report summarizing our geotechnical analysis, conclusions, and recommendations.

2.0 BACKGROUND INFORMATION

2.1 Site Description

The reservoirs subjected for this seismic retrofit study are located in the City of Vancouver Water Station No. 1 located at the north end of East Reserve Street and adjacent to the campus of Clark College. It is situated on a high ground at about 0.5-mile northeast of the city center and on the east side of Interstate Highway 5 (see Figure 1, Vicinity Map).

Water Station No. 1 contains one 1 million gallon (MG) concrete reservoir, one 4 MG concrete reservoir, a 250,000 gallon elevated steel water tank, five stripping towers, groundwater production wells, pump stations and various support infrastructures. The 1 MG reservoir is located at the east portion of the site, and has a footprint of 88 feet by 140 feet. The north side of the 1 MG reservoir is adjacent to a relatively steep, north facing slope with steepness ranging from approximately 3 horizontal to 1 vertical (3H:1V) to 2H:1V. The closest portion of the reservoir to the slope is its northeast corner with less than 10 feet of horizontal distance from the reservoir wall to the edge of the slope. The slope is well vegetated and has a vertical height of about 40 feet.

The 4 MG reservoir is located at the west portion of the site, and has a footprint of 170 feet by 170 feet. The reservoir's west wall is very close to a steep but short slope with a masonry retaining wall at the toe. The height of the retaining wall ranges from 3 feet to a maximum of 8 feet. The slope above the retaining wall to the reservoir west wall has a steepness of 1.5H to 2H:1V, and has a vertical height of approximately 10 feet. The locations of the reservoirs, masonry retaining wall and site slopes are shown in Figure 2, Site and Exploration Plan.

Based on the original design drawings, we understand that these two reservoirs are partially below ground structures, and was designed and constructed in approximately the late 1930's. At both reservoirs, the reservoir sidewalls are approximately 13 feet high with the lower 7 foot portion buried below ground surface. The 1 MG reservoir has a flat bottom. The 4 MG reservoir has a hopper shaped bottom with the base slab sloping down inward from the sidewalls to the center. A copy of the original design drawings for these two reservoirs is included in Appendix B, Existing Reservoir and Site Information, for reference.

No information is available for the masonry retaining wall at the toe of the slope west of the 4 MG reservoir. However, it appears to be an unreinforced gravity wall constructed with the original construction of the Water Station.

2.2 Project Understanding

We understand that this project consists of a seismic retrofit study for the existing 1 MG and 4 MG reservoirs and a siting evaluation for a possible new stand pipe reservoir at the site. For the geotechnical engineering evaluation portion of the seismic retrofit study, the focuses are the seismic stability assessment for the steep slopes adjacent to the existing reservoirs and the auger-cast pile capacities evaluation for the potential new roof supports as part of the structural mitigation schemes. For the potential new reservoir, it is proposed to be a circular, stand pipe steel tank with a rough diameter of 35 feet and a height of approximately 90 feet. The location of the proposed stand pipe reservoir will be to the east of the existing 1MG reservoir, as shown on Figure 2.

2.3 Regional Geology

The general geology of the project area has been mapped and described by Trimble (1963) and ground water conditions in the area have been described by Mundorff, (1964). The project area is underlain at depth by flow rocks of the middle Miocene Columbia River Basalt Group. The basalts are complexly folded and faulted downward forming the Portland Basin. A sequence of Pliocene age alluvial sedimentary rocks known as the Troutdale Formation overlies the basalts. The Troutdale consists of two members. The lower member (commonly referred to as the Sandy River Mudstone) consists primarily of lacustrine deposits of fine-grained sandstone and siltstone. The upper member (or “Troutdale Formation”) consists of fluvial sandstone and pebble-cobble conglomerate.

Up to 250 feet or more of unconsolidated Quaternary silt, sand, and gravel overlie the Troutdale Formation in the project area. Catastrophic flooding during the late Pleistocene epoch deposited these sediments rapidly as floodwaters inundated the area many times between about 19,000 and 12,000 years ago. Borings and water wells drilled in vicinity of the project site indicate that brown to dark gray, medium dense sand with fine gravel is present from the surface to depths between 180 and 240 feet. Dense to very dense, sandy coarse gravel and cobbles continued to depths between 250 and 280 feet or more where yellow-brown cemented sand and gravel (Troutdale Formation) was encountered.

3.0 GEOTECHNICAL DATA

3.1 Field Explorations

The field exploration for this project was conducted on November 14 and 15, 2012, and consisted of four borings (B-1-12 through B-4-12). The approximate locations of these borings are shown on Figure 2, Site and Exploration Plan. These locations were measured from the known site features in the field. The borings were advanced to a depth of between approximately 30 to 50 feet below the existing grade. After the completion of the borings the holes were backfilled in accordance with the State of Washington's Department of Ecology regulation.

A Shannon & Wilson geotechnical engineering representative was present during the field exploration program to locate the borings, log materials encountered in the borings, collect soil samples, and measure water levels. The ground surface elevations of the borings were estimated from site topographic information provided by the City of Vancouver. These elevations should be considered approximate. Details about the drilling and sampling procedures, and exploration boring logs are presented in Appendix A, Subsurface Explorations.

In addition to the boring field explorations, one geophysical testing for shear wave velocity measurement was performed using refraction microtremor methods (ReMi) deployed from the ground surface. The ReMi test was conducted on December 3, 2012 from the southeast corner of the 4MG reservoir to the southeast corner of the 1MG reservoir, as shown on Figure 1 of Attachment 1 in Appendix A. A report of the ReMi test is attached to the end of Appendix A for reference.

3.2 Past Geotechnical Explorations

Shannon & Wilson completed a previous Geotechnical Investigation near the existing reservoirs in October 2005. The investigation included three (3) borings west of the 4 MG reservoir. The approximate locations of these previous borings are shown in Figure 2. Copies of the boring logs are presented in Appendix B, Existing Reservoir and Site Information.

3.3 Laboratory Testing

Selected samples from the borings were chosen for laboratory testing. The laboratory testing program consisted of visual classifications, natural moisture contents, and grain size analyses. The tests were used to verify field descriptions of the soils and provide material characterization to support evaluation of soil behavior. The details of the laboratory testing program and the laboratory testing results are presented in Appendix C, Laboratory Testing Program.

3.4 Discussion of Subsurface Condition

3.4.1 Subsurface Soils

The soils encountered in the borings for this project generally consist of two soil units: Fill and Catastrophic Flood Deposits. Details of the subsurface conditions encountered in the boring are presented on the Log of Boring sheets in Appendices A and B. The interpreted subsurface conditions are illustrated in the generalized subsurface profiles A-A’ and B-B’ included as Figures 3 and 4. The locations of the profile lines are shown on Figure 2. The following paragraphs summarize the conditions of these two soil units:

Fill: Fill materials consisting of very loose to medium dense silty sand and gravelly silt were encountered to a depth of 9.5 feet in boring B-1-12 and to a depth of 7 feet in borings B-2-12, B-3-12, and B-4-12. Relative densities of the fill vary from very loose to medium dense with Standard Penetration Test (SPT) N-values ranging from 2 to 24 blows per foot (bpf).

Catastrophic Flood Deposits (CFD): Below the fill, fine to coarse sand deposits of CFD was encountered to the bottom of the borings (approximately 50 feet at B-1-12, 40 feet at B-2-12 and B-3-12, and 30 feet at B-4-12.) The upper 10 to 20 feet of sand deposit in CFD generally contains some gravel, and below that more fines (silt size particles) were found in the sand deposit. The SPT N-values in CFD unit range from 8 to 41 bpf, with a majority between 11 to 22 bpf, indicating a generally medium dense condition.

3.4.2 Groundwater

No standing groundwater was encountered in any of the 4 borings conducted for this study. Water well reports filed with the State of Washington in 1997 indicate that water levels were recorded approximately 200 feet below the ground surface, or between elevations -10 to 10 feet.

4.0 SITE SEISMIC HAZARD EVALUATION

4.1 General

The seismic hazard evaluation for this project is conducted in accordance with geotechnical investigation requirements for Seismic Design Categories D through F in International Building Code, 2012 Edition (IBC 2012). IBC 2012 refers to ASCE’s Minimum Design Loads for Buildings and Other structures, 2010 Edition (ASCE 7-10) for determination of Site Class. According to the building codes, the earthquake event for the seismic hazards evaluation is the Maximum Considered Earthquake (MCE) having a 2-percent probability of exceedance in a 50-year period, or a 2,475-year return period.

4.2 Seismic Setting

This project is subject to seismic events from three major sources: (1) Cascadia Subduction Zone (CSZ) Megathrust earthquakes at the interface of the Juan de Fuca and North American Plates; (2) deep-focus, CSZ intraplate earthquakes (within the Juan de Fuca and North American Plates);

and (3) shallow-focus earthquakes in local and regional continental crustal faults. The maximum magnitude for a CSZ Megathrust event is expected to be in the range of Moment Magnitude (M_w) 8 to 9 with a possible reoccurrence interval of 500 to 600 years (Barnett and others, 2004). Intraslab events have occurred on a frequent basis in the Puget Sound area, but there is no strong historical evidence for such events in Oregon and southern Wasington. Known and suspected crustal faults in the region have been characterized by the United States Geological Survey (USGS) and the Oregon Department of Geology and Mineral Industries (DOGAMI).

According to the USGS Quaternary Fault and Fold Database of the United States (Personius, S.F., 2002), the nearest mapped Quaternary fault is the East Bank Fault approximately 6.0 miles (mi) southwest of the site. Other nearby faults, listed below in Table 2, with evidences of movement during the Quaternary Period have been mapped within an approximate 20-miles radius of the project site. Each of the faults in Table 2 is defined as a “Class A” fault by the USGS. Class A faults are those for which there is demonstrable evidence of tectonic movement during the Quaternary Period that are known or presumed to be associated with relatively large magnitude earthquakes of M_w of 6 to 7.

TABLE 1: QUATERNARY FAULTS WITHIN A 20-MILE RADIUS OF THE VANCOUVER RESERVOIR SITE

Name	Distance and Direction from Site	Most Recent Deformation*	Slip rate
East Bank Fault	6.0 mi Southwest	<15 ka	<0.2 mm/yr
Portland Hills Fault	7.0 mi Southwest	<1.6 Ma	<0.2 mm/yr
Lacamas Lake Fault	7.3 mi Northeast	<750 ka	<0.2 mm/yr
Oatfield Fault	9.0 mi Southwest	<1.6 Ma	<0.2 mm/yr
Grant Butte Fault	11.8 mi Southeast	<750 ka	<0.2 mm/yr
Damascus – Tickle Creek Fault	12.0 mi Southeast	<750 ka	<0.2 mm/yr
Beaverton Fault Zone	13.2 mi Southwest	<750 ka	<0.2 mm/yr
Helvetia Fault	14.7 mi Southwest	<1.6 Ma	<0.2 mm/yr
Canby-Molalla Fault	15.6 mi Southwest	<15 ka	<0.2 mm/yr

*ka= “kilo-annum,” or 1,000 years; Ma=”Mega-annum”, or 1,000,000 years

4.3 Earthquake Hazard Contributions and MCE

The contribution of earthquake hazard from various seismogenic sources was analyzed using the 2008 Interactive Deaggregations tool from the USGS website for the project location (Longitude = 122.648 W, and Latitude = 45.636 N). Table 2 shows the percent of earthquake hazard contribution due to the three principle sources discussed in Section 4.2.

TABLE 2: EARTHQUAKE HAZARD CONTRIBUTION

Return Period (years)	Exceedance Probability in 50 yrs	Spectral Acceleration Period	CSZ Megathrust Earthquake	Shallow Crustal Earthquake	50-km Deep Intraplate
2,475	2%	0 sec (PGA)	38.5	37.5	22.4
		0.2 sec	36.2	44.2	17.5
		1 sec	71.7	13.3	14.9

As shown on this table, CSZ (Subduction Zone) earthquakes contribute significantly to the seismic hazard at the site for the 2 percent probability of exceedance in 50 years (2,475-year return period) MCE event.

4.4 Seismic Site Classification

Below the surficial fill unit the site is underlain by medium dense sand with silt and gravel in the CFD unit. Based on measured Standard Penetration Test (SPT) results, and the relatively deep ground water, the sand in the CFD unit is not susceptible to liquefaction during the design earthquake event. Thus, in accordance with ASCE 7-10, the site should be classified as Site Class D, according to SPT N-values at the site.

4.5 Code-Based Seismic Site Response

As stated previously, IBC 2012 and ASCE 7-10 define the MCE earthquake in structural design as an event having a 2-percent probability of exceedance in 50 years. The design spectral accelerations were obtained from the 2008 USGS National Seismic Hazard Mapping Program probabilistic seismic hazard analyses (PSHA).

The seismically induced acceleration values at the rock interface for the MCE at the site and the coefficients used to estimate ground surface response adjusted for Site Class D are presented in Table 3.

TABLE 3: USGS CODE-BASED MCE AND DESIGN SEISMIC PARAMETERS

Seismic Parameters	Value
MCE Peak Bedrock Acceleration (PBA)	0.40g
MCE Bedrock Spectral Acceleration, 0.2 second period (S_S)	0.94g
MCE Bedrock Spectral Acceleration, 1.0 second period (S_1)	0.36g
Site Coefficient, F_{PGA}	1.1
Short-Period Site Factor, F_a	1.12
Long-Period Site Factor, F_v	1.68
Soil MCE Peak Ground Acceleration (MCE PGA)	0.44g
Soil MCE Spectral Acceleration, 0.2 second period, Site Class D (S_{MS})	1.06g
Soil MCE Spectral Acceleration, 1.0 second period, Site Class D (S_{M1})	0.60g
Soil Design Peak Ground Acceleration (Design PGA, ⅔ Soil MCE PGA)	0.29g
Soil Design Spectral Acceleration, 0.2 second period, Site Class D (S_{DS}, ⅔ Soil MCE S_{MS})	0.71g
Soil Design Spectral Acceleration, 1.0 second period, Site Class D (S_{D1}, ⅔ Soil MCE S_{M1})	0.40g

Note: PGA stands for Peak Ground Acceleration which corresponds to spectral acceleration at zero period.

4.6 Time History Selection and Scaling for Displacement Analyses

We understand that the code-based design site response parameters of ⅔ of the Maximum Considered Earthquake will be used to determine the structural demand at the reservoirs site. However, the seismic slope displacement analyses discussed in Section 5.1.3 were conducted using a suite of recorded earthquake acceleration time histories that were scaled to match the code-based MCE PGA. Using these scaled ground motions, surface time histories were attained through a 1D Site Response Analysis conducted in SHAKE2000, Version 8.9.0, published in September 2011 by GeoMotions, Inc. These surface time histories were used in the Newmark Displacement analyses to estimate slope displacements.

To select appropriate time histories, we searched publically-available ground motion databases for previously recorded earthquake motions that were compatible with the MCE PGA and shape of the code-based response spectrum. Twelve acceleration time history records were taken from six separate seismogenic events. Eight shallow crustal records and four subduction zone records were selected and scaled to match or slightly exceed the bedrock PGA from the USGS PSHA using a single scale factor applied to the whole time history. The selected input ground motion records and their scaling factors are presented in Table 4, Selected Earthquake Time Histories, below. Detailed information about each recorded and scaled time history is provided in Appendix E, Ground Motion Time Histories and Slope Displacement Analyses.

TABLE 4: SELECTED EARTHQUAKE TIME HISTORIES FOR NEWMARK DISPLACEMENT ANALYSES

Record Name	Type	Earthquake	Date	Station	Comp.	Mag.	Dist. (km)	Duration (sec)	Scale Factor
KOB_nis000	Crust.	Kobe	1/16/95	Nishi-Akashi	0°	6.9	11.1	23.98	0.8627
KOB_nis090	Crust.	Kobe	1/16/95	Nishi-Akashi	90°	6.9	11.1	23.98	0.8800
LOM_gav067	Crust.	Loma-Prieta	10/18/89	Gilroy Gavilan College	67°	6.9	11.6	39.96	1.2222
LOM_gav337	Crust.	Loma-Prieta	10/18/89	Gilroy Gavilan College	337°	6.9	11.6	39.96	1.3400
NOR_how060	Crust.	Northridge	1/17/94	Howard Rd.	60°	6.7	20.0	29.99	3.6667
NOR_how330	Crust.	Northridge	1/17/94	Howard Rd.	330°	6.7	20.0	29.99	2.7000
NOR_mul035	Crust.	Northridge	1/17/94	Mulholland Dr.	35°	6.7	20.8	23.98	0.7097
NOR_mul125	Crust.	Northridge	1/17/94	Mulholland Dr.	125°	6.7	20.8	23.98	1.0000
MCH_uni090	Sub.	Michoacán	9/19/85	La Unión	90°	8.1	83.9	65.66	2.9333
MCH_uni360	Sub.	Michoacán	9/19/85	La Unión	360°	8.1	83.9	65.66	2.5882
PER_arq008	Sub.	Perú Coast	10/17/66	Arequipa	8°	8.0	237	62.93	1.6200
PER_arq278	Sub.	Perú Coast	10/17/66	Arequipa	278°	8.0	237	62.93	2.4000

4.7 Site Seismic Hazards

4.7.1 Seismic Slope Stability Concerns

Considering the relatively steep slopes on the west side of the 4 MG reservoir and on the north side of the 1 MG reservoir, we performed slope stability evaluations for the MCE earthquake loading scenario. Details of these evaluations and results are presented in Section 5.1. In summary, the concern for seismic slope stability affecting the existing reservoirs is not a wide spread issue, but rather at limited locations such as the northwest corner of the 4 MG reservoir where lack of horizontal setback, steep slope, and high unreinforced retaining wall are present.

4.7.2 Liquefaction Hazard

Liquefaction involves the substantial loss of shear strength in saturated soil caused by the rapid increase in pore water pressure when subjected to impact by seismic or dynamic loading. The soils at the subject site are not considered to be liquefiable due to the deep groundwater table and the absence of saturated soils above the groundwater table.

4.7.3 Seismically-Induced Settlements

We evaluated the seismically-induced ground surface settlement based on the SPT blow counts in borings B-1-12 through B-4-12. As discussed above, the site is not considered to have a liquefaction issue, and the seismic settlement is expected to be caused by the cyclic compaction of the unsaturated sandy soils. Using Pradel's method for unsaturated granular soil seismic settlement, we estimated less than 1-inch of vertical settlement under the MCE earthquake scenario.

4.7.4 Lateral Spread Hazard

Lateral spreading occurs when saturated soil underlying gently-sloping ground experiences soil strength loss from soil liquefaction. As mentioned above, due to the absence of saturated soils, we do not anticipate that subsurface soils at the site will experience liquefaction and strength loss. As a result, lateral spreading is not a risk for this site.

4.7.5 Fault Rupture Hazard

The nearest Quaternary fault mapped by the USGS is the East Bank fault, which is located approximately 6.0 miles to the southwest of the project area. In our opinion, this fault does not represent a fault rupture hazard to the project.

4.7.6 Tsunami and Seiche Hazard

Due to the absence of a large water body in the vicinity of the site, the site has no risk of tsunami or seiche.

5.0 ENGINEERING EVALUATIONS AND RECOMMENDATIONS

5.1 Seismic Slope Stability Evaluations

5.1.1 Slope Surface Condition Assessment

During the field exploration at the site, we also conducted a surface reconnaissance on the slopes adjacent to the reservoirs. In general, the surfaces of the slopes adjacent to the reservoirs are well vegetated or landscaped. No erosion gullies or distresses were observed on the slopes, and no cracks were observed on the reservoir walls facing slopes and on the masonry retaining wall. Additionally, no bending tree trunks near the ground are observed to indicate slow movement of the slope. These generally imply that the current conditions of the slopes are stable.

However, two areas were concerned during our initial slope condition assessment. One is at the northwest corner of the 4 MG reservoir, where the unreinforced masonry retaining wall at the bottom of the slope is approximately 8 feet in height and the slope above is relatively steep (slightly less than 2H:1V). Due to the age and that it was likely not designed to resist the seismic load, the failure of the retaining wall and slope under the MCE earthquake may have a negative effect on the 4 MG reservoir.

The other area is at the northeast corner of the 1 MG reservoir, close to the north slope. During our initial site visit, moss marks on the concrete wall buttress were observed which could indicate previous ground settlement (Figure D9 in Appendix D shows moss marks on the concrete wall buttress approximately 12-inches above the current grade). After discussing this concern with the City's Engineering Program Manager, Tyler Clary, we understand that the City of Vancouver is not aware of any settlement and leaking problem at this reservoir. Tyler suggested that the marks on the concrete buttress were likely caused by the rain water from the reservoir roof drip line which is directly above the moss marks. In a later site visit, similar moss marks were observed on the some wall buttress at the other potion of the reservoir, which supported Tyler's explanation. Additionally, considering that soft and weak soils were not observed in the adjacent boring, this area is not considered to have long-term settlement and slope instability issues.

5.1.2 Slope Stability Analysis and Factors of Safety

Based on our subsurface explorations and the topographic information provided by the City of Vancouver, we evaluated the stability of the slopes in 5 places (Cross Sections A-A' through E-E', see Figure 2 for locations) near the existing 1 MG and 4 MG reservoir facilities. Slope stability analyses were performed using the Spencer method, which satisfies both moment and force equilibrium. Pseudo-static analyses were conducted using seismic forces represented by a horizontal seismic force coefficient, k_h, equal to one-half of the MCE PGA. All stability analyses were conducted with the aid of the computer program SLOPE/W, a component of the GeoStudio 2007 (Version 7.19 Build 5027) analysis suite published by Geo-Slope International, of Calgary, Canada.

The stability analysis result is expressed in terms of factor of safety (FS), which is defined as the ratio of resistance (i.e. soil shear strength) to driving loads (i.e. gravity loads). Theoretically, a FS of 1.0 indicates the driving loads are balanced by the soil resistances, and a FS below 1.0 indicates a pending or imminent failure condition. FS greater than 1.0 indicates the soil resistance is greater than the driving loads; however, due to various factors (i.e. variations in soil strengths, subsurface geometry, failure surface location and orientation, groundwater levels, and other factors), FS for stable condition is typically defined as a value higher than 1.0 to account for these factors and variations. The generally accepted minimum FS values are 1.5 for static loading conditions and 1.1 for seismic loading conditions.

The soil parameters selected for slope stability analyses were based on current and existing soil boring and laboratory testing data, and are presented in Table 5. Groundwater levels are assumed to be below the depth of the analyses. Seismic forces were represented by conducting pseudo-static analyses using one-half of the MCE PGA as the horizontal seismic force coefficient, k_h.

TABLE 5: SLOPE STABILITY MATERIAL PARAMETERS

Soil Unit	Unit Weight, γ (pcf)	Friction Angle, ϕ (deg)	Cohesion, c (psf)
Fill	120	30	0
Sand with Gravel	125	36	0
Sand with Silt	105	34	0

For the 4 MG reservoir, the analysis is concentrated on the northwest corner of the reservoir (Cross Section A-A’). For the 1 MG reservoir, analyses are conducted at and near the northeast corner of the reservoir for three cross sections (Cross Sections B-B’ through D-D’) to cover various slope steepness (ranging from slightly less than 2H:1V to 3H:1V) and setback distances. For both reservoirs, we assume that the reservoir is not leaking and will not leak during and after the MCE earthquake event to cause saturation of the slope below the reservoirs.

Cross Section E-E’ is for the evaluation of stability at the proposed new stand pipe tank site. The FS results of the slope stability analyses for all five cross sections are presented in Table 6. The slope stability models and analysis results for some cross sections are included in Appendix D.

TABLE 6: SLOPE STABILITY ANALYSIS RESULTS

Cross Section	Facility	Factors of Safety	
		Static	Seismic (k_h = 0.22g)
A-A’	4 MG Reservoir	1.90	0.84
B-B’	1 MG Reservoir	1.84	1.23
C-C’	1 MG Reservoir	2.32	1.27
D-D’	1 MG Reservoir	2.45	1.31
E-E’	New Stand Pipe Tank	2.03	1.21

As can be seen, the FS values under static loading condition all exceed the accepted minimum FS of 1.5, indicating stable condition of the slopes. This result generally conforms to the relatively good conditions of the slope observed during this evaluation. Under seismic loading condition of the MCE earthquake and no leaking assumption, except for Cross Section A-A’, analyses at other cross sections yielded FS values greater than the accepted minimum FS of 1.1, indicating stable slope performance during an earthquake. The low seismic FS of 0.84 at Cross Section A-A’ suggests potential failure of slope causing soil movement from the northwest corner of the 4 MG reservoir wall to the toe of the unreinforced masonry wall (see Figure D2 in Appendix D). To estimate the potential movement of the slope at this location, we conducted seismic slope displacement analysis using Newmark method. Details of the analysis procedure and results are presented in the next section (Section 5.1.3).

At the proposed new stand pipe tank area, both static and seismic FS values exceed the accepted minimum FS criteria (Figures D6 and D7 in Appendix D). In the seismic stability analysis, a setback distance of approximately 20 feet is assumed between the edge of the tank foundation and the edge of the slope.

5.1.3 Seismic Slope Displacement

Pseudo-static slope stability analyses provide an index of stability but no information on deformations associated with slope failure. We analyzed the potential for slope displacement using a Newmark Displacement analysis. Newmark displacement analyses consider the behavior of a slope subjected to inertial forces that vary with any given acceleration time history. Displacement is calculated for the slide mass each time an acceleration history exceeds the yield acceleration determined for each geologic cross section. The yield acceleration is defined as the horizontal component of seismic force that is required to reduce the static factor of safety to 1.0.

In our analysis, the yield acceleration was estimated using in a pseudo-static slope stability analysis using the computer program SLOPE/W.

Newmark displacement values were then calculated using the computer program SHAKE2000 with a suite of 12 surface ground motion time histories. The surface ground motion was resulted from inputting scaled bedrock time histories into the base of a soil column using 1D linear site response analysis approach. The input ground motion time history suite selection, scaling and site response analysis are discussed in Section 4.6, and are presented in Appendix E for reference.

The Newmark analyses were conducted at Cross Section A-A', where low FS under seismic loading condition is obtained at previous section. At the Cross Section A-A', the yield acceleration was estimated as 0.26g for failure surfaces affecting the reservoir base. The estimated Newmark displacement at the base of the reservoir is less than 1-inch. This result suggests that although the FS values at Cross Section A-A' is less than the minimum seismic FS criterion, the risk of large magnitude of slope movement affecting the existing 4 MG reservoir foundation is relatively low.

5.1.4 Seismic Slope Stability Evaluation Conclusion

4 MG Reservoir

Based on the FS values from the pseudo-static seismic slope stability analyses and the Newmark seismic displacement analysis, it is our opinion that, although the steep slope and masonry retaining are likely to fail in the MCE earthquake, the risk of seismic slope failure affecting the existing 4 MG reservoir foundation is relatively low. However, the failure will likely cause separation of soil from the lower portion of the reservoir west wall and even expose the reservoir footing. This will cause drastic reduction of overburden pressure at the foundation level, and will have an adverse effect on the seismic bearing capacity of the foundation soil.

To reduce the risk of this "uncontrolled" overburden pressure reduction, we recommend unloading the seismically unstable slope to a level of 2 feet above the base of the reservoir footing. This will increase the FS from 0.84 to 1.2, and therefore, reduce the risk of seismic slope failure approaching or encroaching reservoir foundation. From a slope stability improvement point of view, the slope unloading will need to start from the northwest corner of the reservoir to about 60 feet southerly where the existing masonry wall height is less than 5 feet. For a slope section with masonry wall less than 5 feet in height, the seismic FS is more than 1.1; therefore, mitigation by unloading is not considered necessary.

For this recommendation, we understand from Degenkolb Engineers that soil confinement from the existing slope is not needed for the structural integrity and serviceability of the reservoir wall under the static and earthquake loading conditions. Also, as an added benefit, the unloading of the slope will create a construction access to the west side of the reservoir for the installation of new auger-cast piles which is proposed as part of the structural mitigation to the reservoir roof.

1 MG Reservoir

Under a non-leaking circumstance of the reservoir, the FS values from the pseudo-static seismic slope stability analyses exceed the minimum FS criterion and the risk of seismic slope failure affecting existing reservoir foundation and base is considered to be low. Therefore, under non-leaking scenario, seismic slope mitigation is not considered necessary for this reservoir.

New Stand Pipe Tank

At the proposed new stand pipe tank area, with a setback distance of 20 feet from the edge of the slope, both static and seismic FS values exceed the accepted minimum FS criteria. Therefore, the site for the new stand pipe tank is considered suitable from a slope stability point of view as long as at least 20 feet setback can be maintained for the new tank development.

5.2 Seismic Soil Bearing Capacity Assessment

We evaluated the static and seismic bearing capacity of the soils underlying the 4 MG & 1 MG reservoirs, assuming any site grading or improvement activities will maintain a minimum 2 feet of soil cover above the base of the perimeter footing foundation. Based on our subsurface explorations, our engineering evaluation and the current performance of the existing reservoir, we believe an allowable static bearing capacity of 3,000 pounds per square foot (psf) can be used for the underlying native sandy soils. Under seismic loading condition, we estimate the bearing capacity can be increased to 4,000-psf, considering the absence of soil liquefaction and strength loss as well as the short-term loading condition.

5.3 Auger-Cast Pile Recommendations for the New Roof Support

We understand that as one of the seismic retrofit scheme, new roof supports for the 1 MG and 4 MG reservoirs are proposed and will be founded on auger-cast piles. An auger-cast pile is constructed by drilling down to the prescribed bearing stratum with a hollow-stem, continuous-flight auger. The auger is left in place to support the walls of the borehole. A high-strength grout mix is then pumped through the hollow stem under pressure while the auger is slowly withdrawn from the hole. Care is required to coordinate the rate of grout placement and grout level with the rate of auger withdrawal to prevent the sides of the hole from sloughing in and necking, thereby reducing the pile cross section area. Immediately after grout placement, a rebar cage with or without a center single rebar (depending on the depth of the cage) is lowered into the grouted shaft. This type of pile requires installation by an experienced and competent foundation contractor, as well as full-time construction observation and QA/QC documentation under the supervision of an experienced geotechnical engineer to assure satisfactory installation.

We recommend 18-inch-diameter auger-cast piles to optimize the pile length and provide a pile diameter conducive to installation of the reinforcing cage through the in-place grout. Considering the depth of the estimated pile lateral deflection (detailed in pile lateral capacity discuss in the section), we recommend a pile depth of 40 feet which corresponds to a pile tip elevation of approximately 185 feet.

The static allowable axial compressive capacity is estimated to be approximately 120 kips, and the allowable uplift capacity to be approximately 100 kips for the 18-inch-diameter pile. The static allowable axial capacity has a factor of safety of 3. Under the seismic loading condition, the factor of safety can be reduced to 2 and the estimated seismic axial and uplift capacities are 180 kips and 150 kips for the compression and uplift condition, respectively.

Minor pile settlement will result from the proposed structural loads. Based upon our experience and engineering analyses, we anticipate that the maximum static or seismic total settlements for the auger-cast piles should be less than 1 inch, and the maximum differential settlements should be less than 50 percent of the total settlement.

The lateral capacities of the recommended auger-cast piles were calculated using the computer program LPILE. Lateral loads imposed by seismic forces are resisted primarily by the stiffness of the soil adjacent to the pile shafts. The lateral capacity of a pile depends on its diameter, length, stiffness in the direction of loading, proximity to other piles, and degree of fixity at the head of the piles (at bottom of pile cap), as well as the engineering properties in the soil, especially within the upper portion of the pile. The analysis results are presented in Table 7, below, for free-head and fixed-head pile conditions with ½-inch pile top (head) deflections. Diagrams of LPILE analysis results are included in Appendix F, Auger-Cast Pile Lateral Capacity Analyses.

TABLE 7: UNFACTORED LATERAL LOAD INFORMATION OF AUGER-CAST PILES

Pile Type and Diameter (in.)	Loading Condition	Deflection (in)	Unfactored Lateral Resistance (kips)	Maximum Bending Moment (in-kips)	Depth of Maximum Moment (ft.)	Depth to Points of Fixity (ft.)
18”-Diameter Auger-Cast Pile	Free	0.5	15	1,000	7	22
	Fixed	0.5	45	3,000	0	24

Note: Maximum moment depth zero means maximum moment is at top (head) of pile. The point of fixity is defined as near zero pile lateral deflection.

The lateral resistance values presented in Table 7 are unfactored and preliminary in nature. We anticipate that the lateral load resistance of the piles will be limited by the cross-sectional ratio of the reinforcing steel cage to grout; therefore, based on structural evaluations the actual pile lateral load capacity used may be different from the values provided above. Additionally, the project structural engineer should verify that the piles have sufficient internal strength to accommodate the lateral loads and determine the depth of the reinforcing cage. We assume a steel cage with close spiral will be used as reinforcement, and a single bar will be installed to the full-depth of the pile.

The above-mentioned values for compressive, uplift, and lateral capacity refer to single piles unaffected by group interactions. To reduce or eliminate group effects, we recommend that the pile spacing never be less than three pile diameters center-to-center. If piles are at least three diameters apart, group effects can be neglected for compressive and uplift. However, for lateral loads, group effects reduce the lateral load capacity of the pile at a pile spacing of less than five diameters. If the pile spacing is less than five times the pile diameters, the pile group reduction factors in Table 8, below, should be applied to the above unfactored pile lateral capacities.

TABLE 8: REDUCTION FACTORS FOR IN-LINE LATERALLY LOADED PILES

Pile Spacing	In-line Load Reduction Factor
5 pile widths	1.0
3 pile widths	0.75

Note: Widths are measured center-to-center of the piles.

5.4 Proposed New Stand Pipe Tank

We understand that the City would like to consider conceptual design for a proposed vertical stand pipe. The proposed stand pipe will be approximately 90 feet tall with a 35-foot diameter. As discussed in the previous sections, the static and seismic slope stability are not considered to be issues at the proposed new tank location, as long as a setback distance of at least 20 feet from the edge of the slope can be maintained.

We understand that due to the heavy vertical loads and overturning moment under seismic condition, the new stand pipe will likely have a mat foundation at the base which will be supported on pile foundations. Based on the conceptual information provided by Degenkolb, the piles will likely be 24-inch in diameter and have a center-to-center spacing of 6 feet. The maximum compressive load on each pile will be approximately 240 kips under static gravity loading condition and 470 kips under seismic loading condition (gravity plus seismic load combination). The maximum uplift load on the pile is estimated to be on the order of 90 kips.

Considering the relatively high vertical loads, we recommend 24-inch diameter auger-cast piles with a minimum length of 70-feet to sustain the required compressive and uplift load. In the auger-cast pile capacity/length analyses, a factor of safety of 3 is used in the static loading condition, and a factor of safety of 2 is used in the seismic loading condition.

5.5 Construction Considerations

5.5.1 Site Preparation

All subgrade areas should be carefully stripped, excavated, prepared or compacted. Considering the relatively shallow site grading and excavation will be required, common earth excavation and grading machines can be used for this project. However, to minimize disturbance to the subgrade, we recommend the excavation equipment be positioned on the material to be excavated, not on the final subgrade level, and the grading equipment not directly driving on the final subgrade level.

Depending on the construction schedule/method and the moisture sensitive nature of the silty subgrade soils, the site stripping and excavation may cause considerable disturbance and loosening of the subgrade. We recommend that the disturbed soils be overexcavated to expose the undisturbed competent subgrade. A representative of the geotechnical engineer should determine the depth of overexcavation at the time of construction.

After stripping and excavating to the proposed subgrade levels, as required, the subgrade should be proof-rolled with a self-propelled, smooth drum compaction equipment with a static weight of at least 8 tons (dead weight). The approved equipment should make a sufficient number of passes to obtain complete coverage of the subgrade. Any areas that pump, weave, or appear soft should be removed by overexcavation, backfilled with structural fill material, and compacted to structural fill standards. Sand should not be used under any foundation. The proof-rolling, subgrade compaction, and overexcavation activities should be witnessed by a representative of the geotechnical engineer and should be performed during a period of dry weather.

For the case of grading operations in wet weather, all excavations should be performed using a smooth-bladed tracked backhoe working from areas where material has yet to be removed, or from the already placed structural fill. Subgrade areas should be cleanly cut to firm undisturbed soil. Proof-rolling of the subgrade is likely not appropriate during wet weather grading in order to avoid disturbance of moisture-sensitive soils. Should construction take place during wet weather, we recommend that a representative of the geotechnical engineer be present to observe the subgrade in order to evaluate whether additional preparation is required.

5.5.2 Excavation

For planning purposes, the temporary excavation slope can be cut at slopes of 1.5H:1V. The actual temporary slopes and slope protections should be designed by the contractor. Permanent earth slopes (if any) should be constructed to 2H:1V or flatter and protected from erosion.

All excavations should be completed in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. While we have described certain approaches for excavations in the foregoing discussions, the contractor should be responsible for selecting the excavation and groundwater or perched water control methods, monitoring the excavations for safety, and providing shoring, as required, to protect personnel, adjacent improvements, and existing facilities, such as known buried pipelines adjacent to the foundation excavation.

5.5.3 Structure Fill

We recommend imported clean crushed rock as structural Fill. The imported crushed rock should be maximum 1½-inch particle size and contain less than 5 percent passing the No. 200 sieve based on a washed sieve analysis, ASTM D 1140.

Unless otherwise specified, the crushed rock fill should be compacted to not less than 92 percent of the modified Proctor maximum dry density (ASTM 1557). For the crushed rock fill pad below the new stand pipe tank foundation, it should be compacted to 95 percent of the modified proctor. The crushed rock structural fill should be placed in maximum lifts of 10 inches of loose material. Each lift of compacted engineered fill should be tested by a qualified representative of a qualified testing agency and results reviewed by the geotechnical engineer.

5.5.4 Auger-Cast Pile Installation

For the auger-cast pile foundation, the piles should be installed within a tolerance of 3 inches of the locations shown on the plans. The completed piles should be plumbed to within 2 percent from vertical. We also recommend that the pile construction specification and construction procedures should follow most recent edition of “Augered Cast-in-Place Piles Manual,” developed by Deep Foundation Institute (DFI). Further, we recommend the full-time inspection of the pile foundation installation by a qualified geotechnical field representative from Shannon & Wilson.

6.0 LIMITATIONS

This report was prepared for the exclusive use of Degenkolb Engineers for specific application to the seismic retrofit design and construction of the Vancouver Reservoirs facility as they relate to the geotechnical aspects discussed herein. This report is interpretive in content and should not be made available to prospective bidders, contractors and/or subcontractors as a basis for bidding.

The analysis, conclusions, and recommendations contained in this report are based on site conditions as they currently exist. Our conclusions and recommendations are based on our understanding of the project as described in this report and the site conditions as interpreted from the explorations. We have assumed that the explorations are representative of the subsurface conditions at the site of the proposed improvements and that subsurface conditions everywhere are not significantly different from those disclosed by the explorations. Within the limitations of the scope, schedule, and budget, the analyses, conclusions and recommendations presented in this report, this report was prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no warranty, either express or implied.

If, during construction, subsurface conditions different from those encountered in the field explorations are observed or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations concerning the changed conditions and/or the time lapse.

This report is not a warranty of subsurface conditions, such as those interpreted from the exploration logs including conclusions of subsurface conditions in other interpretive sections of the report. Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples from borings. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects. Therefore, some contingency fund is recommended to accommodate the potential for extra costs.

The scope of our geotechnical services did not include any environmental assessment or evaluation regarding the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below the site, or for evaluation of disposal of contaminated soils or groundwater, should any be encountered, except as noted in this report.

Shannon & Wilson, Inc. has prepared a document, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our report. This document is included at the end of this report in Appendix I.

Sincerely,

SHANNON & WILSON, INC.

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JASON KEITH GARNER
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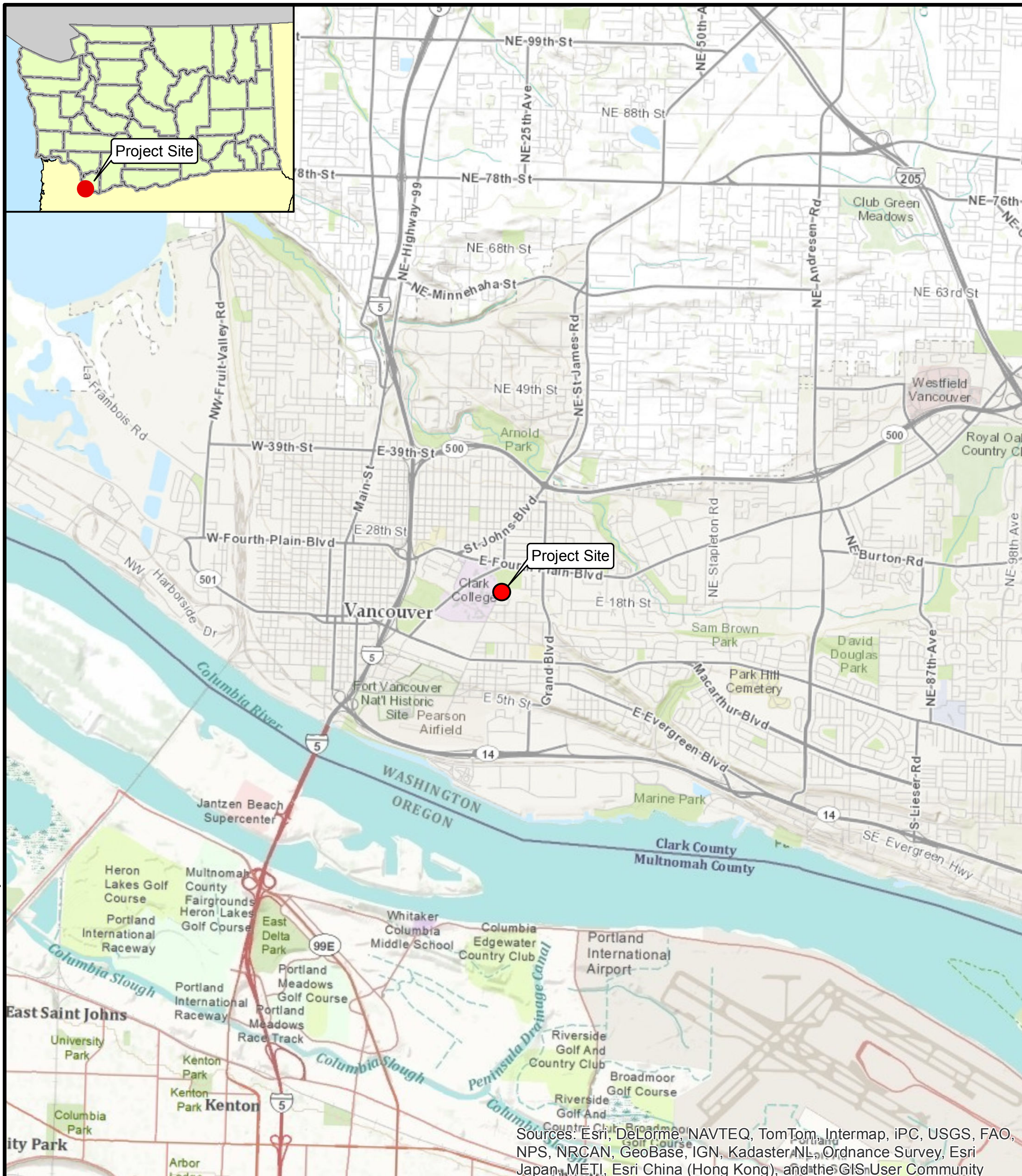

N
W E
S

0 0.5 1 2
Miles

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

VICINITY MAP

April 2013 24-1-03744-001

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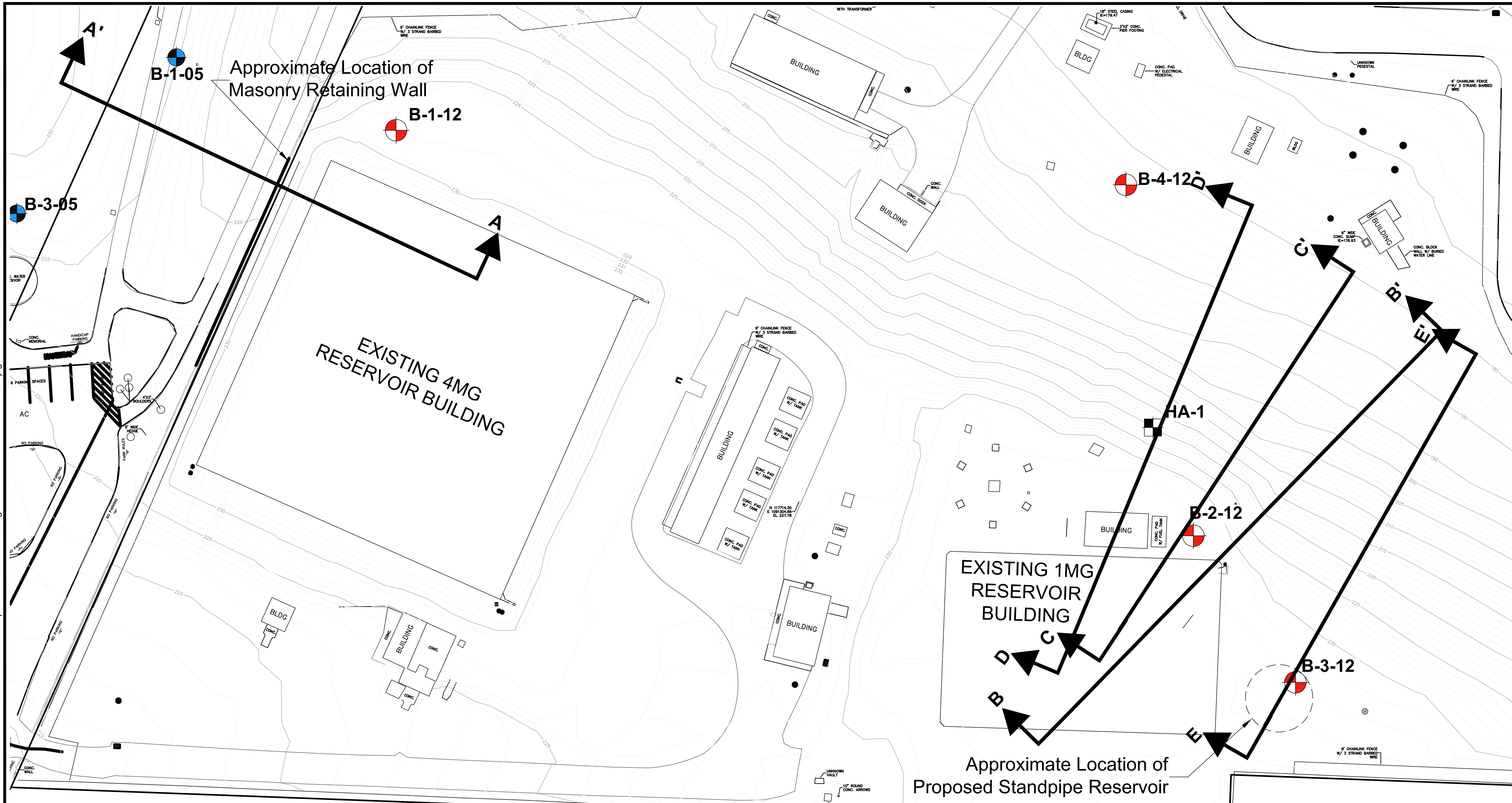
FIG. 1

Path: T:\Projects\24-1\3744_Vancouver Reservoir Seismic Retrofit\Avmxd\Vicinity.mxd 1/17/2013 ath



EXPLANATION

B-1-12 Approximate Location and Designation of 2012 Boring

B-1 Approximate Location and Designation of Boring from Shannon & Wilson's 2005 Investigation

HA-1 Approximate Location and Designation of Hand Auger

A — A' Location and Designation of Generalized Subsurface Cross Section

0 50 100

Scale in Feet

N

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

SLOPE STABILITY
CROSS SECTION LOCATIONS

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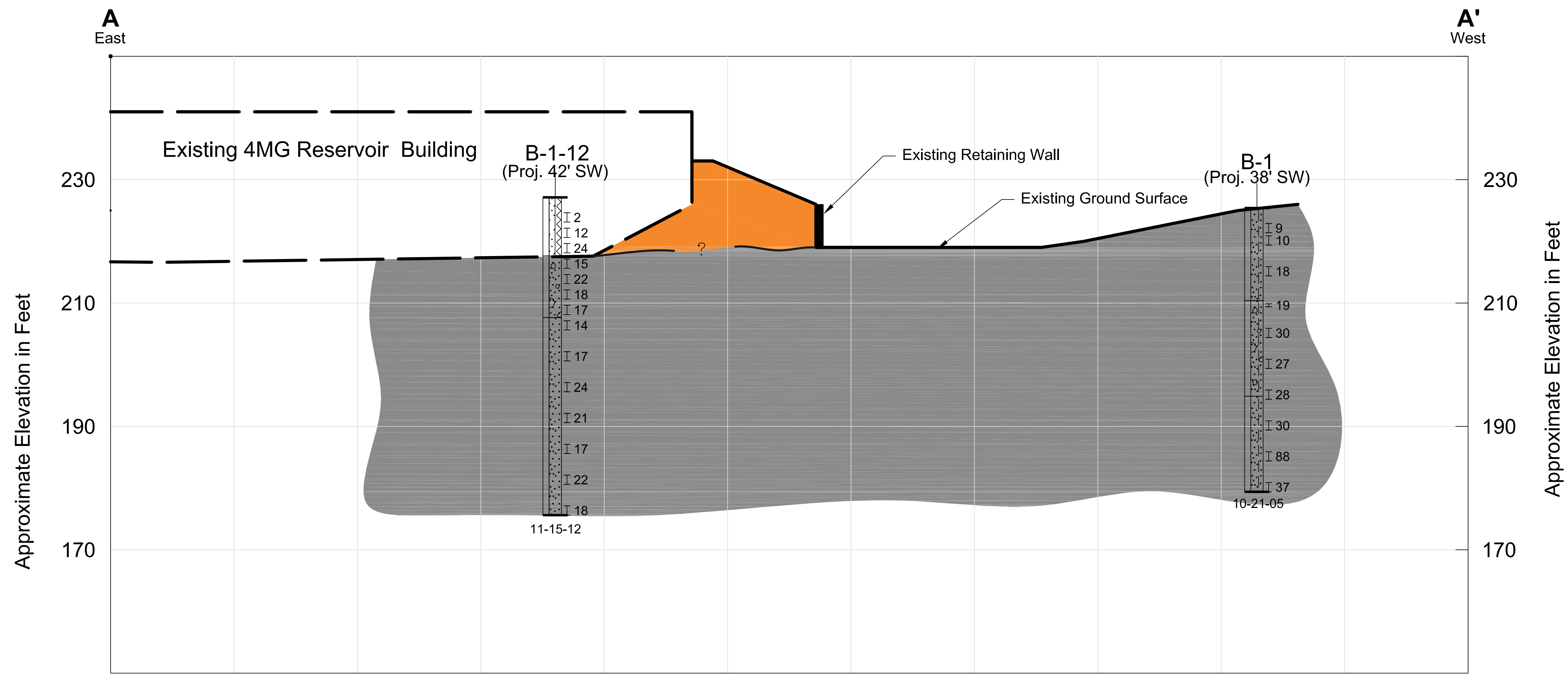
FIG. 2



LEGEND

B- 1 — Boring Designation, Projected Distance

- Standard Penetration Test Blows/Foot or Blows/Inches Driven (4; 50/3")
- Groundwater under pressure
- USCS Symbol
- Water level measured during drilling
- Approximate Geologic Contact
- Bottom of Boring
- Date of Completion (12-12-10)

SUBSURFACE MATERIAL LEGEND

- FILL
- Catastrophic Flood Deposits

NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. The base of the existing 4MG reservoir base is based on the plans provided to Shannon & Wilson on 11/28/2012.

0 20 40
Scale in Feet
Horizontal = Vertical

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

GEOLOGIC SUBSURFACE PROFILE A-A'

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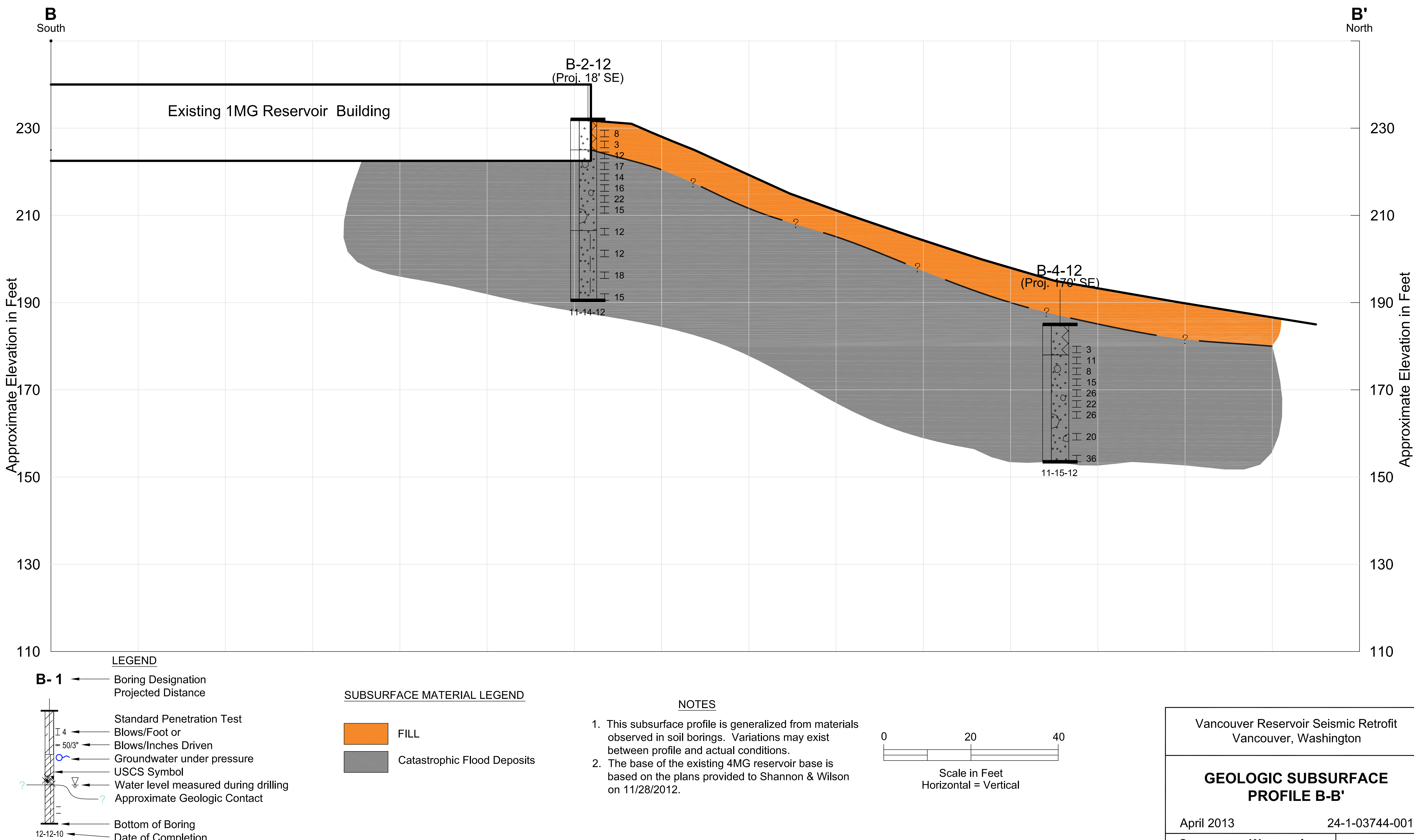
FIG. 3



NOTES

1. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.
2. The base of the existing 4MG reservoir base is based on the plans provided to Shannon & Wilson on 11/28/2012.

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

GEOLOGIC SUBSURFACE PROFILE B-B'

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FIG. 4

APPENDIX A

SUBSURFACE EXPLORATIONS

TABLE OF CONTENTS

FIGURES

ATTACHMENT

APPENDIX A

SUBSURFACE EXPLORATIONS

A.1 GENERAL

The field exploration program for the Vancouver Reservoir Seismic Retrofit project includes four borings. This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered during drilling. The locations of the completed borings were measured from existing site features and their approximate locations are shown on the Site and Exploration Plan, Figure 2.

A.2 DRILLING

Drilling operations were accomplished between November 14, 2012, and November 15, 2012. Subsurface Technologies, Inc., of North Plains, Oregon, provided and operated a truck-mounted Diedrich D50 Turbo rotary drill rig. The boreholes were advanced using open-hole mud-rotary tri-cone drilling techniques. A Shannon & Wilson engineering geologist supervised the field investigation effort, and geology and engineering staff members located the borings, observed the exploratory drilling, collected samples, and logged the borings.

A.3 SAMPLING

A.3.1 Disturbed Sampling

Disturbed samples were collected in the borings, typically at 2.5- to 5-foot depth intervals, using a standard 2-inch outside diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing. In a Standard Penetration Test (SPT), ASTM D1586, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. The SPT N-value provides a measure of in situ relative density of granular soils such as sand and gravel, and the consistency of cohesive soils such as silt and clay. All disturbed samples were visually classified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination and testing.

N-values are significantly affected by several variables, including the efficiency of the hammer used. One automatic hammer system was used to drive all SPT samples performed in the borings. We obtained a hammer calibration report for this hammer that indicated an average energy transfer ratio (hammer efficiency) of 71 percent. *The N values reported on the logs are in blows per foot, as counted in the field, and no corrections of any kind have been applied to them.*

Refusal is reached in an SPT when more than 50 blows are required to drive the sampler through a 6-inch interval. If refusal was encountered in the first 6-inch interval (for example 50 for 1.5”), the count is reported as 50/1st 1.5”. If refusal was encountered in the second 6-inch interval (for example 48, 50 for 1.5”), the count is reported as 50/1.5”. If refusal was encountered in the last 6-inch interval (for example 39, 48, 50 for 1.5”), the count is reported as 98/7.5”.

A.4 BOREHOLE ABANDONMENT

The boreholes were backfilled in accordance with Washington Water Resource Department regulations. Each hole was backfilled with a bentonite chips.

A.5 MATERIAL DESCRIPTIONS

In the field, soil and rock samples were classified visually in general accordance with the ASTM standards, following ASTM D2487. Once returned to the laboratory, soil and rock samples were re-examined, various standard classification tests were conducted, and field classifications were modified as necessary. Refer to the ASTM D2487-11 for terminology used in the soil and rock classifications.

A.6 LOGS OF BORINGS

Summary logs of the borings are presented in Figures A1 to A4. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the boring logs gives our description, classification, and geotechnical unit designation of the soil and rock encountered in the boring. The right-hand portion of the boring logs shows a graphic log, sample locations and designations, groundwater information, and a graphical representation of N-values, natural water contents, sample recovery, Atterberg limits, and fines content.

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

S&W CLASSIFICATION OF SOIL CONSTITUENTS

Major constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).

Modifying (secondary) constituents precede the major constituents (i.e., silty SAND) and compose 15 to 45 percent, by weight, for fine-grained soils and 30 to 45 percent, by weight, for coarse-grained soils.

Minor constituents follow major and modifying constituents (i.e., silty SAND with gravel) and compose 10 percent, by weight, for fine-grained soils and 10 to 25 percent, by weight for coarse-grained soils.

Trace constituents follow all other constituents and are labeled "trace" (i.e., silty SAND with trace gravel). Trace constituents comprise 5 percent, by weight of coarse-grained soils and 5 to 10 percent, by weight of fine-grained soils.

Percentages are based on estimating amounts to the nearest 5 percent.

MOISTURE CONTENT DEFINITIONS

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

ABBREVIATIONS

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

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	Fill
Vibrating Wire	

PLASTICITY

PLASTICITY ADJECTIVE	PLASTICTY INDEX (PI) RANGE
Nonplastic	0 - 4
Low Plasticity	>4 - 10
Medium Plasticity	>10 - 20
High Plasticity	>20 - 40
Very High Plasticity	>40

Vancouver Reservoir Seismic Retrofit
Vancouver, WA

SOIL CLASSIFICATION AND LOG KEY

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FIG. A1
Sheet 1 of 2

POST_07_BORING_CLASS1 24-1-03744-001.GPJ SWNEW.GDT 1/17/13

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOIL *(more than 50% retained on No. 200 sieve)*	Gravel *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravel *(less than 5% fines)*	GW	Well-graded gravel, gravel, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravel, gravel-sand mixtures, little or no fines
		Gravel with Fines *(more than 10% fines)*	GM	Silty gravel, gravel-sand-silt mixtures
			GC	Clayey gravel, gravel-sand-clay mixtures
	Sand *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sand *(less than 5% fines)*	SW	Well-graded sand, gravelly sand, little or no fines
			SP	Poorly graded sand, gravelly sand, little or no fines
		Sand with Fines *(more than 10% fines)*	SM	Silty sand, sand-silt mixtures
			SC	Clayey sand, sand-clay mixtures
FINE-GRAINED SOIL *(50% or more passes the No. 200 sieve)*	Silt and Clay *(liquid limit less than 50)*	Inorganic	ML	Inorganic silt of low to medium plasticity, rock flour, sandy silt, gravelly silt, or clayey silt with slight plasticity
			CL	Inorganic clay of low to medium plasticity, gravelly clay, sandy clay, silty clay
		Organic	OL	Organic silt and organic silty clay of low plasticity
	Silt and Clay *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silt, micaceous or diatomaceous fine sand or silty soils, elastic silt
			CH	Inorganic clay or medium to high plasticity
		Organic	OH	Organic clay of medium to high plasticity, organic silt
HIGHLY-ORGANIC SOIL	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Solid lines on the logs indicate contacts between major units. Dashed lines indicate contacts between different material types within the same unit. Dotted lines indicate subtle or uncertain contacts within a unit. The contacts shown are an interpretation of the condition encountered and actual contacts may be more gradational than shown.

2. Dual symbols (symbols separated by a hyphen, i.e., SP-SM, SAND with silt) are used for coarse-grained soils with 10 percent fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

3. Borderline symbols (symbols separated by a slash, i.e., CL/ML and GW/SW) indicate that the soil may fall into one of two possible basic groups.

4. The soil graphics above represent the various USCS designations (i.e., GP, SM, etc.) and may be augmented with additional symbology to represent differences within USCS designations. Sandy SILT (ML), for example, may be accompanied by the ML soil graphic with sand grains added.

Vancouver Reservoir Seismic Retrofit
Vancouver, WA

SOIL CLASSIFICATION AND LOG KEY

April 2013 | 24-1-03744-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 2 of 2

Total Depth:	51.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger
Top Elevation:	~ 227 ft.	Easting:	~	Drilling Company:	Subsurface Technologies
Vert. Datum:		Station:	~	Drill Rig Equipment:	Diedrich D50 Turbo
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.:	8 in.
Rod Type:	NWJ
Hammer Type:	Automatic

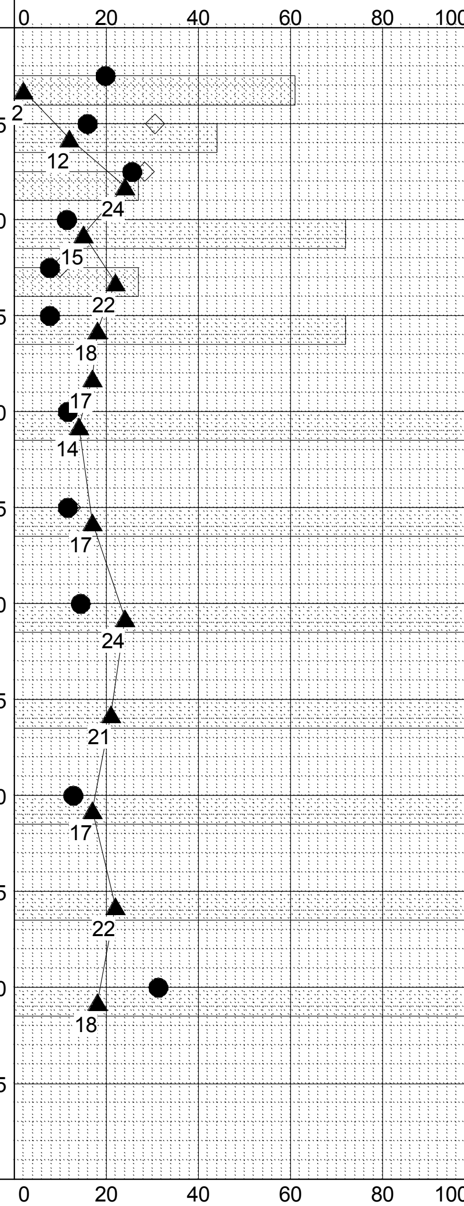
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

SOIL DESCRIPTION	Elev. Depth (ft.)	Samples	Depth, ft.
Very loose to dense silty SAND, trace gravel; moist; fine to coarse sand; fine gravel; occasional organics. (SM) **FILL**		S-1, S-2, S-3	5
Medium dense brown-gray SAND with gravel and silt; moist; fine to coarse sand; fine subrounded gravel; micaceous. (SP)	217.5 / 9.5	S-4, S-5, S-6, S-7	10, 15
Medium dense brown SAND with silt; moist; fine sand; micaceous; grades to coarser sand with depth. (SP-SM) **CATASTROPHIC FLOOD DEPOSITS**	207.5 / 19.5	S-8, S-9, S-10, S-11, S-12, S-13	20, 25, 30, 35, 40, 45
Silty SAND at 50.0 feet		S-14	50
Coarse SAND with gravel at 51.0 feet	175.5 / 51.5		55
Completed - November 14, 2012			

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

* Sample Not Recovered

Standard Penetration Test

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

LOG OF BORING B-1-12

April 2013 24-1-03744-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A2

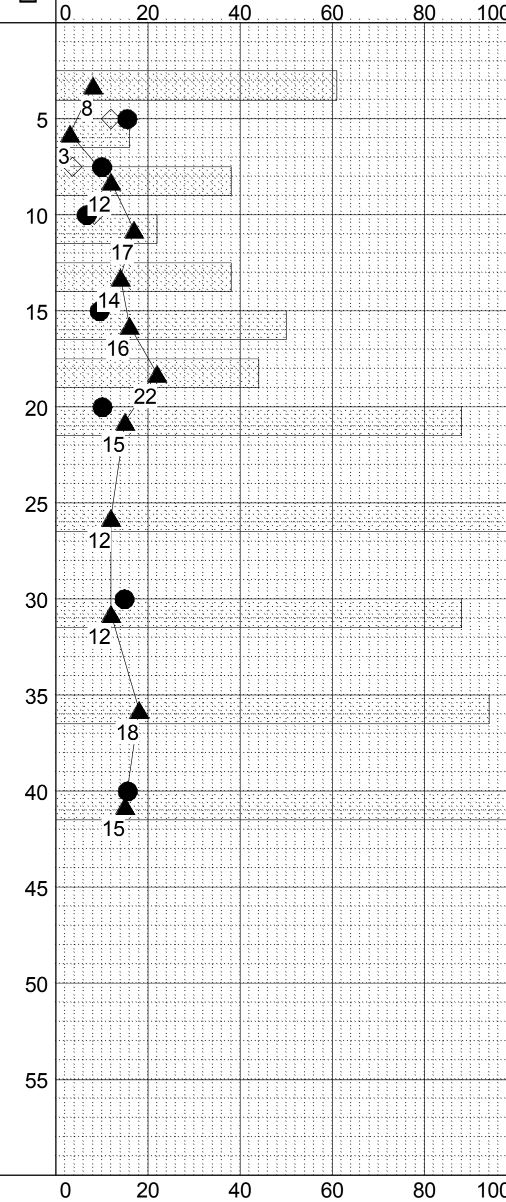
REV 3

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	8 in.
Top Elevation:	~ 232 ft.	Easting:	~	Drilling Company:	*Subsurface Technologies*	Rod Type:	NWJ
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Diedrich D50 Turbo*	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Samples
Very loose to loose brown silty SAND with gravel; moist; nonplastic fines; fine to coarse sand; fine gravel. (SP-SM) **FILL**		S-1, S-2
Medium dense dark gray-brown SAND with gravel and silt; moist; fine to coarse sand; fine subrounded gravel; micaceous. (SP) **CATASTROPHIC FLOOD DEPOSITS**	225.0 7.0	S-3, S-4, S-5, S-6, S-7, S-8
Medium dense brown silty SAND to SAND with silt; moist; nonplastic fines; fine sand; micaceous; grades coarser with depth. (SM)	206.5 25.5	S-9, S-10, S-11, S-12
Completed - November 14, 2012	190.5 41.5	

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

* Sample Not Recovered

Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

LOG OF BORING B-2-12

April 2013 24-1-03744-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

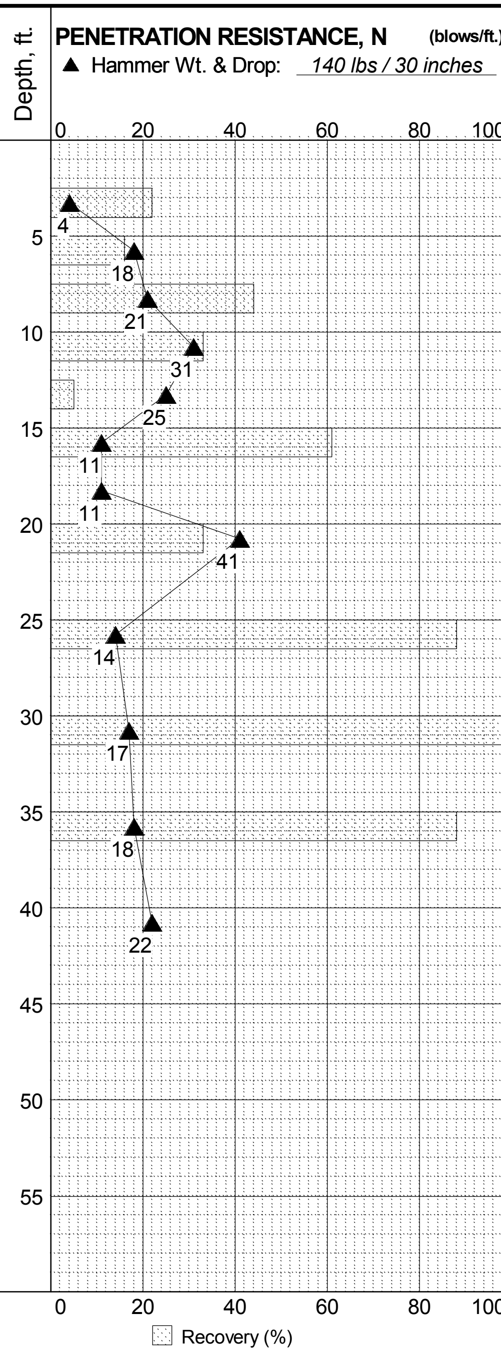
FIG. A3

REV 3

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger
Top Elevation:	~ 232 ft.	Easting:	~	Drilling Company:	Subsurface Technologies
Vert. Datum:		Station:	~	Drill Rig Equipment:	Diedrich D50 Turbo
Horiz. Datum:		Offset:	~	Other Comments:	

Hole Diam.: 8 in.
Rod Type: NWJ
Hammer Type: Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water
Loose to medium dense brown silty SAND; moist; fine sand. (SM) **FILL**			S-1, S-2	
Medium dense to dense dark gray-brown SAND with gravel and silt; moist; fine to coarse sand; fine subrounded gravel; micaceous; grades coarser with depth. (SP) **CATASTROPHIC FLOOD DEPOSITS**	*225.0* 7.0		S-3, S-4, S-5, S-6, S-7, S-8, S-9	
Medium dense light brown SAND with silt; moist; fine sand; micaceous; grades to coarser sand and less fines with depth. (SM)	*205.0* 27.0		S-10, S-11, S-12	
Completed - November 15, 2012	*190.5* 41.5			



LEGEND

* Sample Not Recovered
Standard Penetration Test

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

LOG OF BORING B-3-12

April 2013 24-1-03744-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A4

REV 3

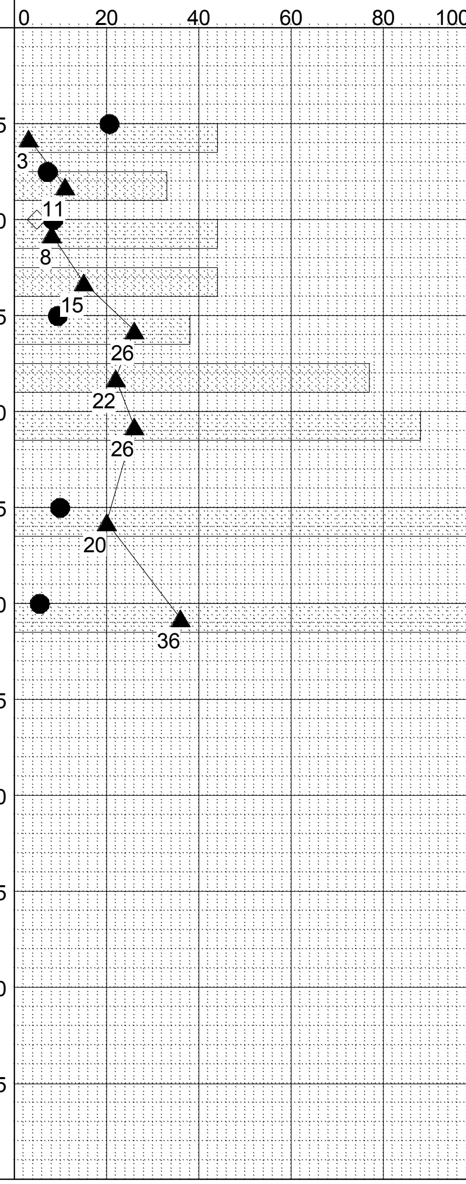
MASTER_LOG_E 24-1-03744-001.GPJ SHAN_WIL.GDT 4/25/13 Log: JKG Rev: Typ: ATH

Total Depth:	*31.5 ft.*	Northing:	~	Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation:	*~ 185 ft.*	Easting:	~	Drilling Company:	*Subsurface Technologies*	Rod Type:	*NWJ*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Diedrich D50 Turbo*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Symbol	Samples	Ground Water	Depth, ft.
Very loose orange and brown SAND with silt; moist; nonplastic fines; fine to medium sand. (SM) **FILL**			S-1		5
Medium dense dark brown-gray SAND with gravel, trace silt; moist; fine to coarse sand; fine to coarse subrounded gravel; micaceous. (SP)	*178.0* 7.0		S-2 S-3 S-4		10
Interbedded layer of brown fine SAND with silt at 15.0 feet Grades to SAND with gravel at 17.5 feet.			S-5 S-6 S-7		15 20
CATASTROPHIC FLOOD DEPOSITS			S-8 S-9		25 30
Completed - November 15, 2012	*153.5* 31.5				35 40 45 50 55

PENETRATION RESISTANCE, N (blows/ft.)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



LEGEND

* Sample Not Recovered

Standard Penetration Test

Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

LOG OF BORING B-4-12

April 2013 24-1-03744-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A5

MASTER_LOG_E 24-1-03744-001.GPJ SHAN_WIL.GDT 4/25/13 Log: JKG Rev: Typ: ATH

REV 3

APPENDIX A ATTACHMENT

Shear Wave Refraction Microtremor (ReMi) Analysis Report

Report on Shear Wave Refraction Microtremor Analysis (ReMi)
City of Vancouver Reservoir
Vancouver, Washington

Report Date: December 4, 2012

Prepared for:

Shannon & Wilson, Inc.
3990 Collins Way, Suite 100
Lake Oswego, OR 97035

Prepared by:

EARTH DYNAMICS
2284 N.W. Thurman St.
Portland, OR 97210
(503) 227-7659
Project No. 12203

1.0 INTRODUCTION

This report presents the results of shear wave seismic explorations at the City of Vancouver Reservoir located at 4203 E. Reserve Street, Vancouver, Washington. The work was requested and authorized by Mr. Wolfe Lang of Shannon & Wilson, Inc. Mr. Daniel Lauer of Earth Dynamics conducted the fieldwork on December 3, 2012. This report describes the methodology and results of the investigation.

2.0 SCOPE OF WORK

The primary purpose of this study is to determine the subsurface shear wave velocity at the site. These data are needed as part of a seismic retrofit to help determine the seismic response of the site to earthquake loading. The exploration consisted of one twenty-four channel refraction microtremor (ReMi) array. The total length of the ReMi array is 345 feet. The location of the ReMi line is shown in Figure 1.

3.0 METHOD

The ReMi technique provides a simplified characterization of relatively large volumes of the subsurface. The method can be used to estimate one-dimensional shear wave velocity profiles and provide site-specific soil classification data as described by the National Earthquake Hazards Reduction Program (NEHRP, 1997). In a ReMi survey, geophones are deployed at designated intervals along a linear array. The resolution and depth of investigation depends upon the cut-off frequency and spacing of the geophones and the total array length. The depth of investigation is approximately one-third of the geophone array length. For this project, twenty-four 4.5 Hz geophones spaced fifteen feet apart were deployed for the ReMi array. All but one of the geophones was installed with a spike in the soil. One geophone was mounted on a tripod on an asphalt driveway. Several 30-second long seismic records of ambient seismic noise were recorded.

The theoretical basis of the ReMi method is the same as Spectral Analysis of Surface Waves (SASW) and Multi-channel Analysis of Surface Waves (MASW) as first described to the earthquake engineering community by Nazarian and Stokoe (1984). However, ReMi does not require a frequency controlled source and the field equipment is much more compact and economical. A complete description of the theoretical basis for ReMi is described by Louie (2001). In ReMi analysis all interpretation is done in the frequency domain, and the method assumes that the most energetic arrivals recorded are Rayleigh waves. By applying a time-domain velocity analysis, Rayleigh waves can be separated from body waves, air waves, and other coherent noise. Transforming the time-domain velocity results into the frequency domain allows combination of many arrivals over a long time period, and yields easy recognition of dispersive surface waves.

Data reduction is completed in two steps. First, the time versus amplitude seismic records are transformed into spectral energy shear wave frequency versus shear wave velocity (or slowness). The data are graphically presented in what is commonly termed a p-f plot. The interpreter determines a dispersion curve from the p-f plot by selecting the lower bound of the spectral energy shear wave velocity versus frequency trend. The second phase of the analysis consists of fitting the measured dispersion curve with a theoretical dispersion curve that is based upon a model of multiple layers with various shear wave velocities. The model velocities and layer thicknesses are adjusted until a 'best fit' to the measured data is obtained. This type of interpretation does not provide a unique model. Interpreter experience and knowledge of the existing geology is important to provide a realistic solution. The data are presented as one-dimensional velocity profiles that represent the average shear wave velocities of the subsurface layers over the length of the geophone array.

4.0 RESULTS

The location of the ReMi array is shown in Figure 1. The results of ReMi analysis is summarized in Figure 2. Figure 2 contains the p-f plot, the dispersion curve and the derived velocity versus depth model that best fits the geology of the site and the dispersion curve for the array.

5.0 DISCUSSION

Prior to the ReMi survey, borings were drilled near the ReMi line. The logs from these borings indicate that the site is underlain by medium dense sand and gravel. Borings B2 and B3 encountered medium dense fine sand at a depth of 25 to 27 feet below the ground surface. The shear wave velocity model indicates that there is an increase in shear wave velocity at approximately 30 feet below the ground surface. It is likely that this increase in velocity is associated with the dense find sand layer.

NEHRP (1997) defines five site classes based upon the average shear-wave velocity of the soil to a depth of 100 feet. The NEHRP classification is summarized in Table 1. The classifications in Table 1 are incorporated into the International Building Code (IBC 2006) Earthquake shaking is expected to be stronger where shear-wave velocity is lower. The average shear wave velocity of the model derived from this study is 1,003 ft/s. Therefore, NEHRP seismic design classification "D" is appropriate for this site.

6.0 LIMITATIONS

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

7.0 REFERENCES

Louie, J.N. (2001). "Faster, better: shear-wave velocity to 100 meters depth from refraction microtremor arrays", Bull. Seism. Soc. Am., 91, 347-364.

Nazarian, S., and Stokoe II, K.H., (1984), "In situ shear-wave velocities from spectral analysis of surface waves", Proceedings for the World Conference on Earthquake Engineering Vol. 8, San Francisco, Calif., July 21-28, v.3, 31-38.

NEHRP (1997). NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, Part 1: Provisions (FEMA 302), Building Seismic Safety Council, Washington, D.C., 290 p.

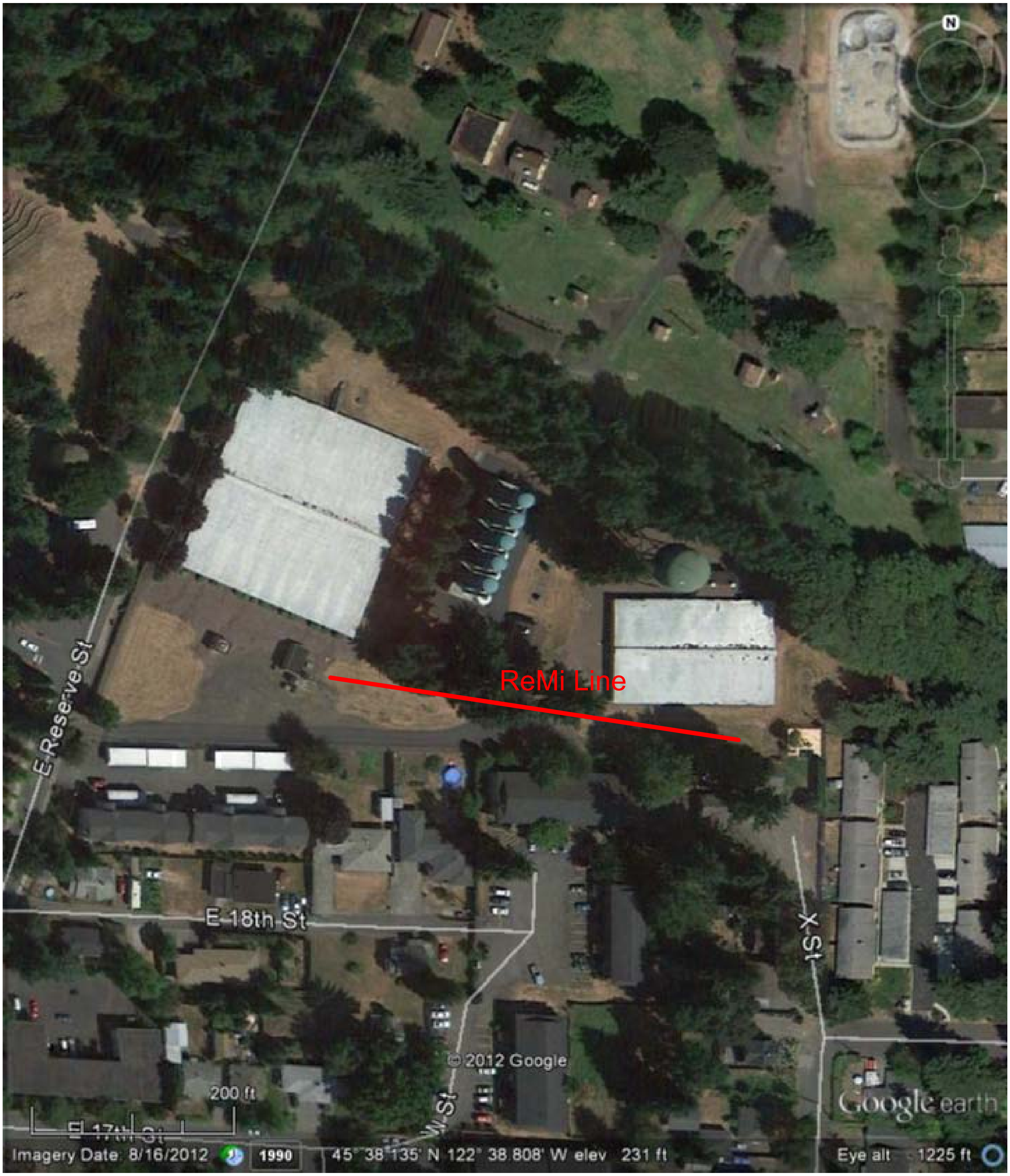
IBC (2006) 2006 International Building Code , International Code Council, Washington D.C.

RESPECTFULLY SUBMITTED
EARTH DYNAMICS

Daniel Lauer
Senior Geophysicist

Table 1. Summary of NEHRP soil classification.

Class	Average S-wave Velocity (ft/sec)	Description
A	> 5,000	Hard rock
B	2,500 – 5,000	Rock
C	1,200 – 2,500	Very dense soil and soft rock
D	600 – 1,200	Stiff soil
E	<600	Soil



LEGEND

ReMi Line Location

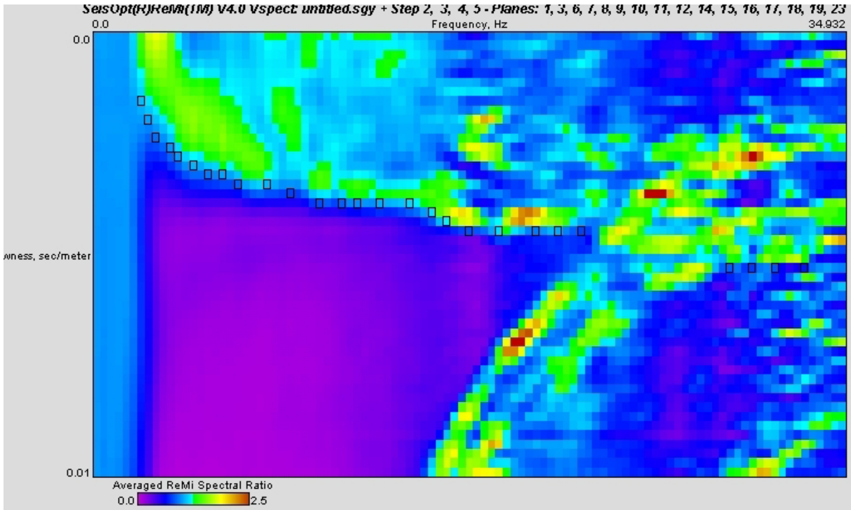
EARTH DYNAMICS
2284 NW Thurman St.
Portland, OR 97210
(503) 227-7659
Email: MFeves@earthdyn.com

City of Vancouver Resrvoir
Vancouver, WA

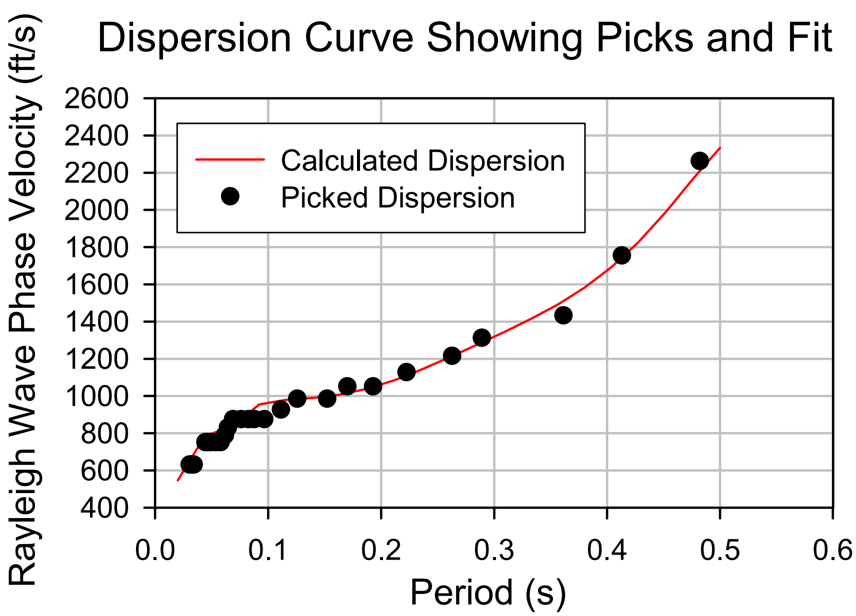
Site Plan

Job #:	12203	Date:	12/03/12	Figure:	1

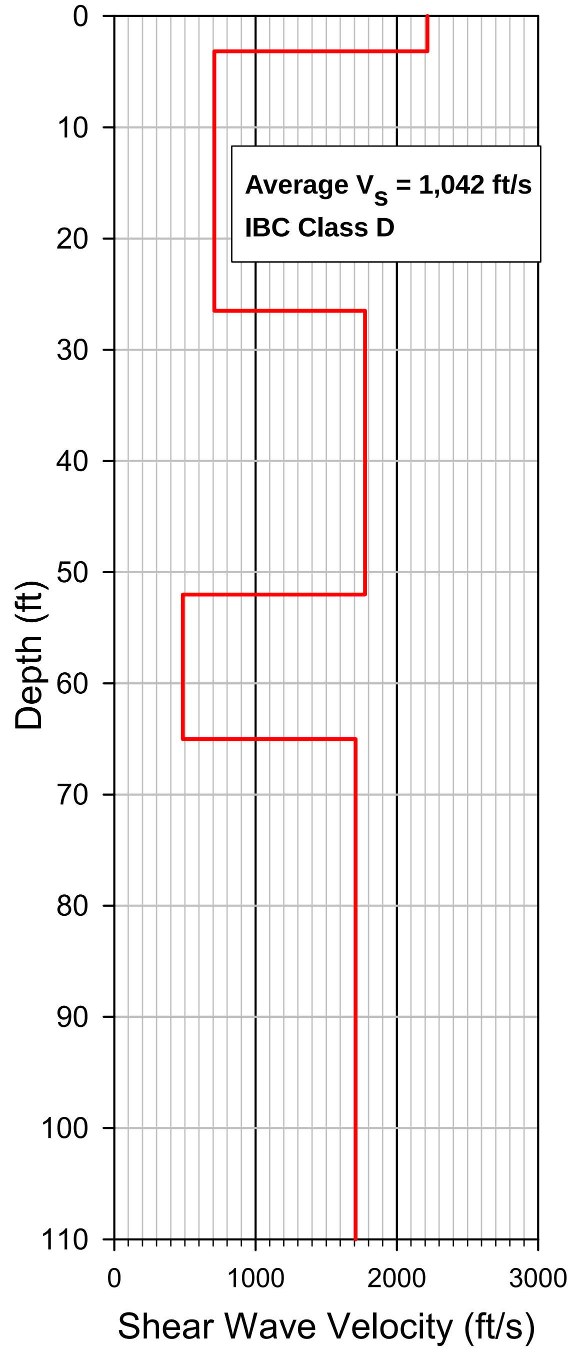
p-f Image with Dispersion Modeling Picks



Dispersion Curve Showing Picks and Fit



V_s Model



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ReMi Study
City of Vancouver Reservoir

Line 1 ReMi Data

Job #:	Date:	Fig:
12203	12/03/12	2

APPENDIX B

EXISTING RESERVOIR AND SITE INFORMATION

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

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 40 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

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

ABBREVIATIONS

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

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.8 mm)
SAND*	
- Fine	#200 to #40 (0.8 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sands and gravels, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Cement/Concrete	Asphalt or Cap
Bentonite Grout	Slough
Bentonite Seal	Ash
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

3 MG Reservoir, Water Works Park
Vancouver, Washington

SOIL CLASSIFICATION AND LOG KEY

November 2005 *24-1-03307-002*

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants
Portland, Oregon

FIG. 3
Sheet 1 of 2

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

3 MG Reservoir, Water Works Park
Vancouver, Washington

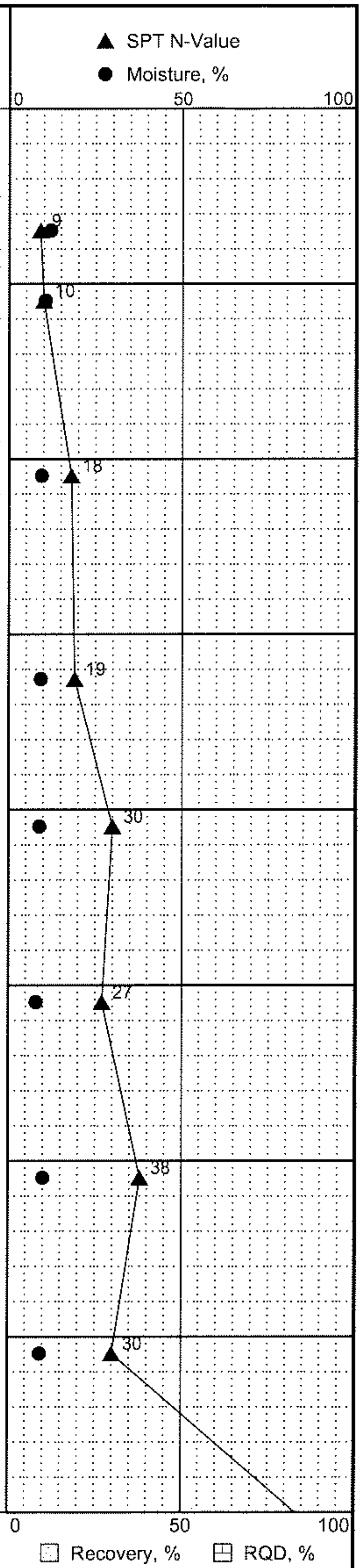
SOIL CLASSIFICATION AND LOG KEY

November 2005 *24-1-03307-002*

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants
Portland, Oregon

FIG. 3
Sheet 2 of 2

Ground Water	Remarks	Elev. Depth Feet 224.5	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed	Drilled with 4" OD solid stem flight augers.		**SILTY SAND** with occasional fine gravel, loose grading to medium dense, brown, moist, non-plastic, medium- to coarse-grained. Consists of stratified and graded beds: silty coarse sand and fine gravel, grading upward to clean medium to coarse sand and fine gravel, grading upward to clean medium to coarse sand. (Catastrophic Flood Deposits Channel Facies) Sample S-4 Wet Density = 102.05 pcf Dry Density = 94.88 pcf		0 5 10	S-1 S-2 S-3
		209.5 15.0	**SILTY SAND with Gravel**, medium dense grading to dense, brown, moist, non-plastic, medium- to coarse-grained, fine gravel. Consists of stratified and graded beds, typical sequence: silty medium to coarse sand and fine gravel, grading upward to clean medium to coarse sand with fine gravel; silty layers are dense and consolidated. (Catastrophic Flood Deposits, Channel Facies) Hole caving between 23 and 25 feet. Plugged hole with bentonite chips, then drilled through.		15 20 25	S-4 S-5 S-6 S-7
		194.0 30.5	**SILTY SAND**, dense, brown, moist, non-plastic; stratified beds consisting of nearly dry consolidated silty coarse sand grading upward to silty fine laminated micaceous sand. (Catastrophic Flood Deposits, Channel Facies)		30 35	S-8 S-9



LEGEND

= 2.0" O.D. Split Spoon Sample
= 3.0" O.D. Thin-Walled Sample
* = Sample Not Recovered
= 3.0" O.D. Split Spoon Sample
= 3.25" O.D. Split Spoon Sample
= Core Rock Sample

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

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

3 MG Reservoir, Water Works Park
Vancouver, Washington

LOG OF BORING B-1

Page 1 of 2

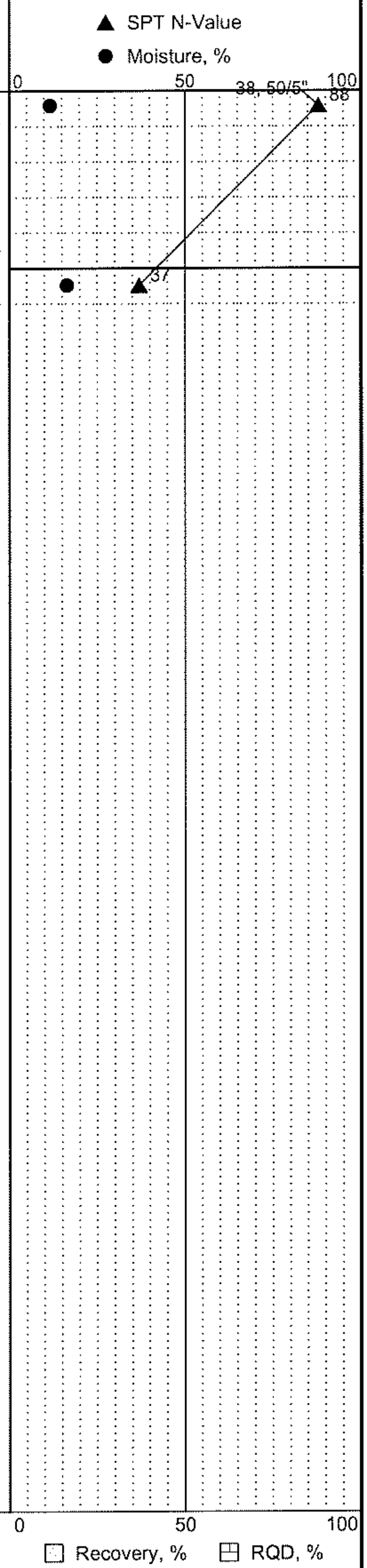
November 2005 *24-1-03307-002*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 4

WLG WWPNR.GPJ 11/3/05 Log: kee Typ: kee Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed			**Silty SAND**, continued.			S-10
					45	S-11
		178.5 46.0	Bottom of Boring - Completed 21 October 2005			



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = 3.25" O.D. Split Spoon Sample
- = 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

3 MG Reservoir, Water Works Park
Vancouver, Washington

LOG OF BORING B-1

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November 2005 *24-1-03307-002*

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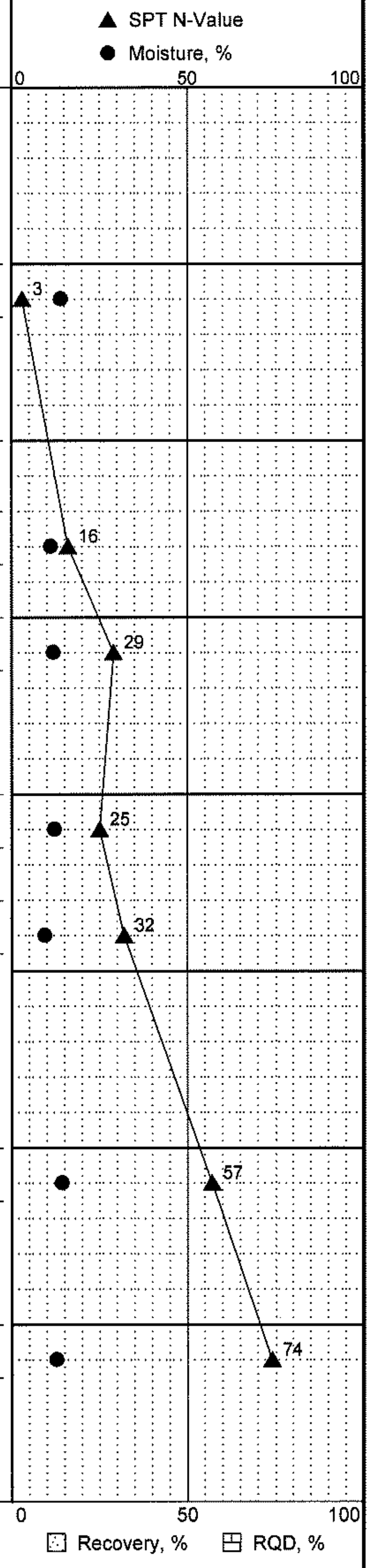
Portland, Oregon

FIG. 4

Ground Water	Remarks	Elev. Depth Feet 227.2	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed	Drilled with 4" OD solid stem flight augers.	226.9 0.3	Sod and top soil. **SILTY SAND** with occasional fine gravel, loose grading to medium dense, brown, moist, non-plastic, medium- to coarse-grained sand with fine gravel stratified with clean medium- to coarse-grained sand. (Catastrophic Flood Deposits, Channel Facies Facies)		0	
					5	S-1
			Sample S-2 Wet Density = 112.35 pcf Dry Density = 97.75 pcf PPP = 0.25 to 0.75 tons/sq. ft Torvane = 0.2 kg/sq. cm		10	S-2 S-3
					15	S-4
		206.2 21.0	**SILTY SAND with Gravel**, medium dense grading to very dense, brown, moist, non-plastic, medium- to coarse-grained; stratified silt, silty sand, sand, and sandy fine to coarse gravel. (Catastrophic Flood Deposits, Channel Facies)		20	S-5 S-6
					25	
					30	S-7
		190.7 36.5	Bottom of Boring - Completed 21 October 2005		35	S-8



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = 3.25" O.D. Split Spoon Sample
- = Core Rock Sample

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

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

3 MG Reservoir, Water Works Park
Vancouver, Washington

LOG OF BORING B-2

Page 1 of 1

November 2005 *24-1-03307-002*

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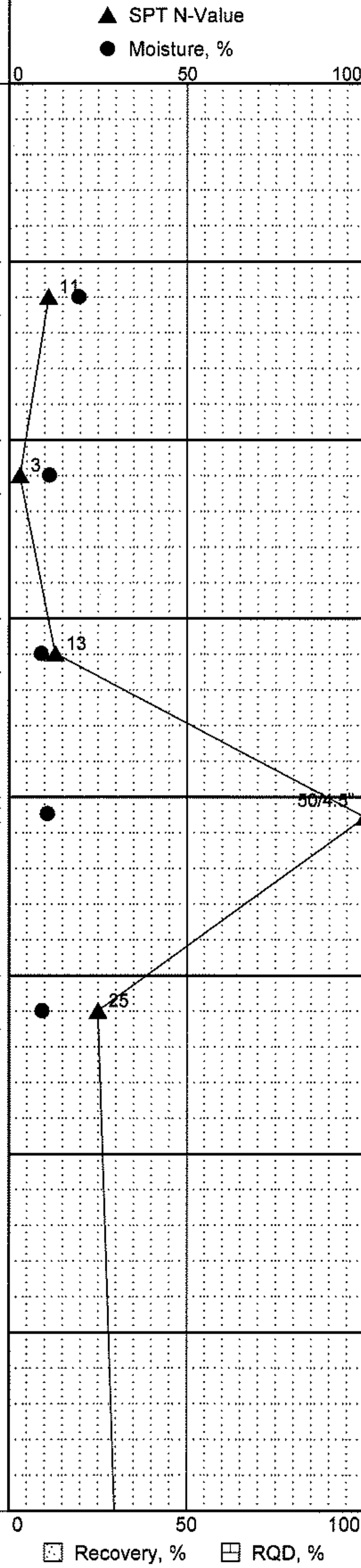
Geotechnical Consultants
Portland, Oregon

FIG. 5

WLG WWPNR.GPJ 10/31/05 Log: kee Typ: kee Rev:

Ground Water	Remarks	Elev. Depth Feet 226.8	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
None Observed	Drilled with 4" OD solid stem flight augers.	226.5 0.3	Sod and Top Soil. **SILTY SAND** with occasional fine gravel, loose, brown, moist, non-plastic, medium- to coarse-grained, micaceous. Consists of stratified and graded beds: coarse sand, medium sand, silty sand. (Catastrophic Flood Deposits, Channel Facies)		0, 5	S-1
					10	S-2
			Sample S-4 Wet Density = 98.20 pcf Dry Density = 89.79 pcf PPP = 0 to 0.5 tons/sq. ft Torvane = 0.1 kg/sq. cm		15	S-3
		207.8 19.0	**SILTY SANDY GRAVEL**, very dense, moist, non-plastic, medium- to coarse-grained sand with fine to coarse gravel. (Catastrophic Flood Deposits, Channel Facies)		20	S-4 S-5
		203.8 23.0	**SILTY GRAVELLY SAND**, medium dense, brown, moist, non-plastic; stratified and graded beds: silty sand with fine gravel, fine to medium sand, coarse sand and fine gravel. (Catastrophic Flood Deposits, Channel Facies)		25	S-6
		198.8 28.0	**Hole caving, no recovery**: Loose medium- to coarse-grained sand is inferred.		30, 35	
		188.8 38.0	**GRAVELLY SAND** with trace silt, medium dense, brown, moist, non-plastic, medium- to coarse-grained sand with estimated 15% fine gravel. (Catastrophic			



LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = 3.25" O.D. Split Spoon Sample
- = Core Rock Sample

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

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

3 MG Reservoir, Water Works Park Vancouver, Washington

LOG OF BORING B-3

November 2005 *24-1-03307-002*

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Portland, Oregon

FIG. 6

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	▲ SPT N-Value ● Moisture, % (0 – 50 – 100)
None Observed		185.3	Flood Deposits, Channel Facies)			S-7	30
		41.5	Bottom of Boring - Completed 21 october 2005				

0 50 100

Recovery, % RQD, %

LEGEND

- = 2.0" O.D. Split Spoon Sample
- = 3.0" O.D. Thin-Walled Sample
- * = Sample Not Recovered
- = 3.0" O.D. Split Spoon Sample
- = 3.25" O.D. Split Spoon Sample
- = 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

3 MG Reservoir, Water Works Park
Vancouver, Washington

LOG OF BORING B-3

Page 2 of 2

November 2005 *24-1-03307-002*

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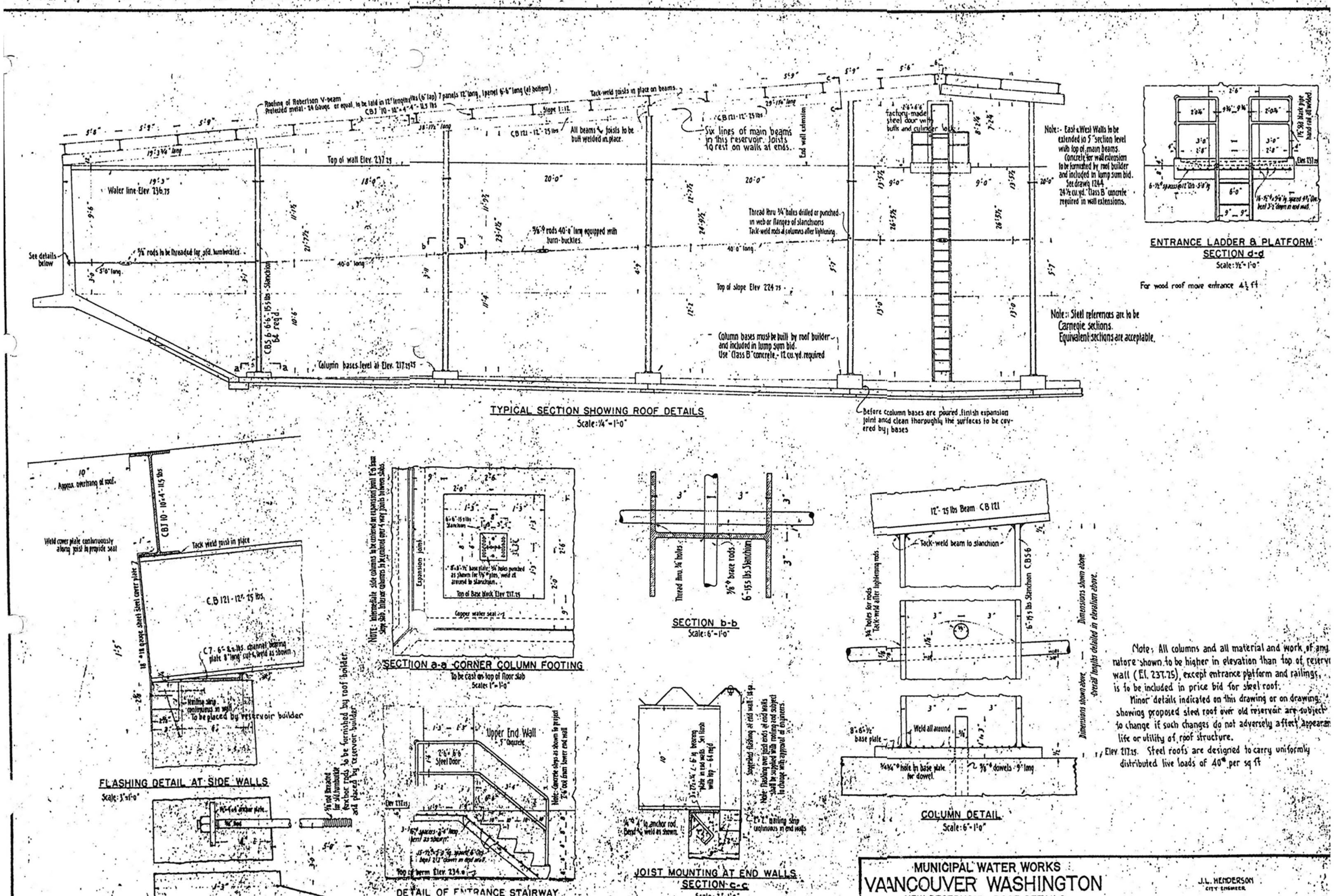
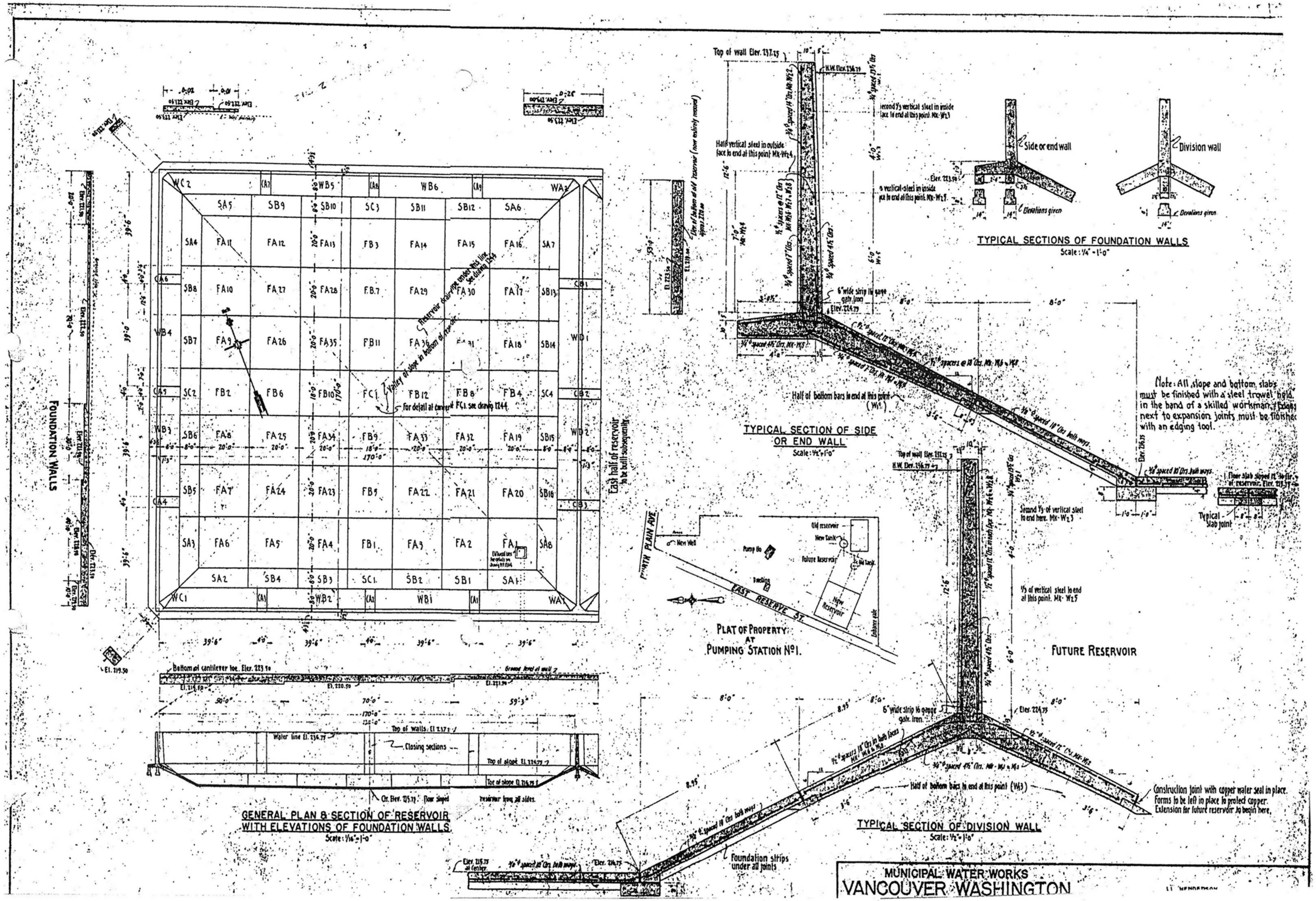
Portland, Oregon

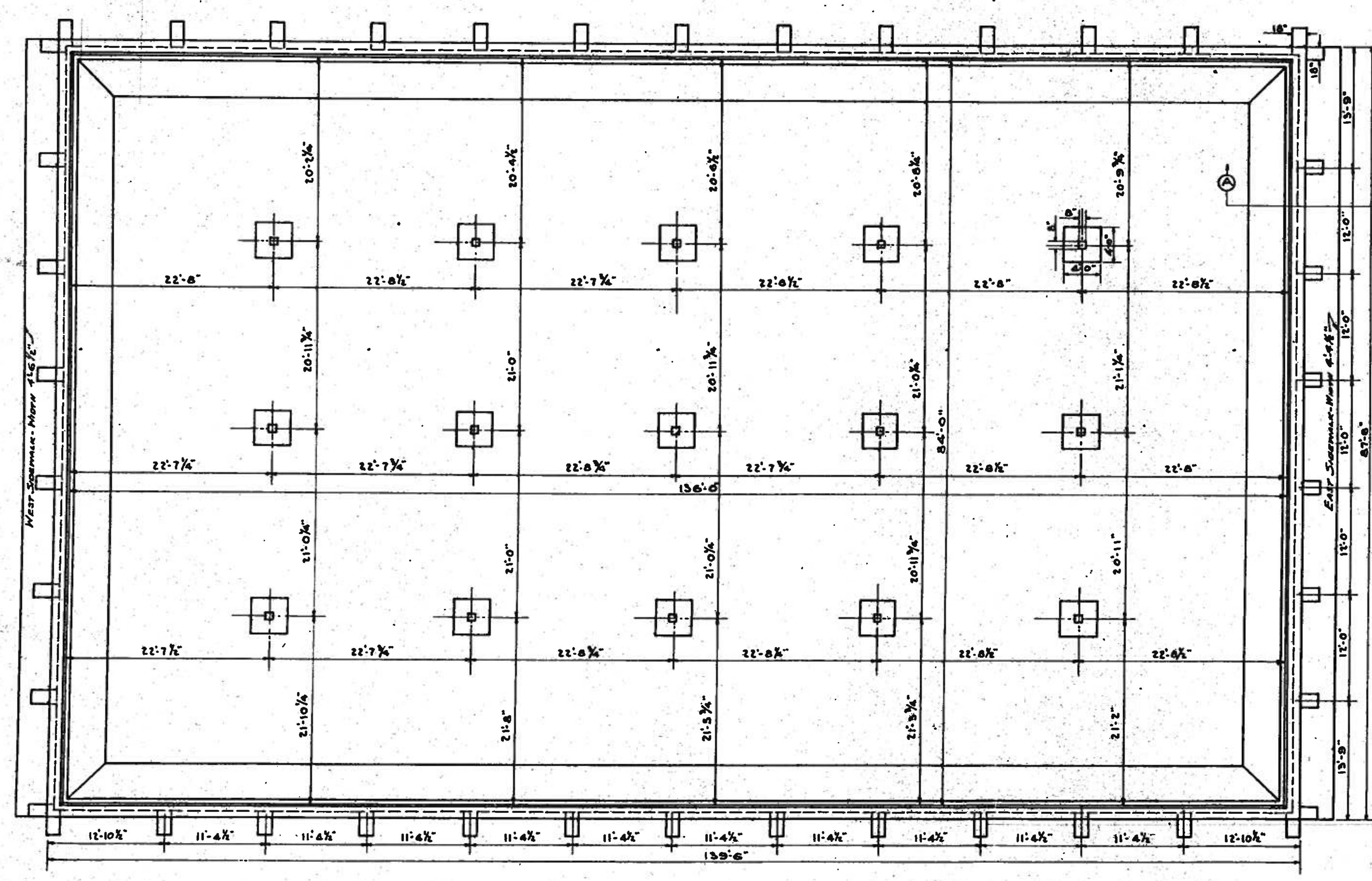
FIG. 6

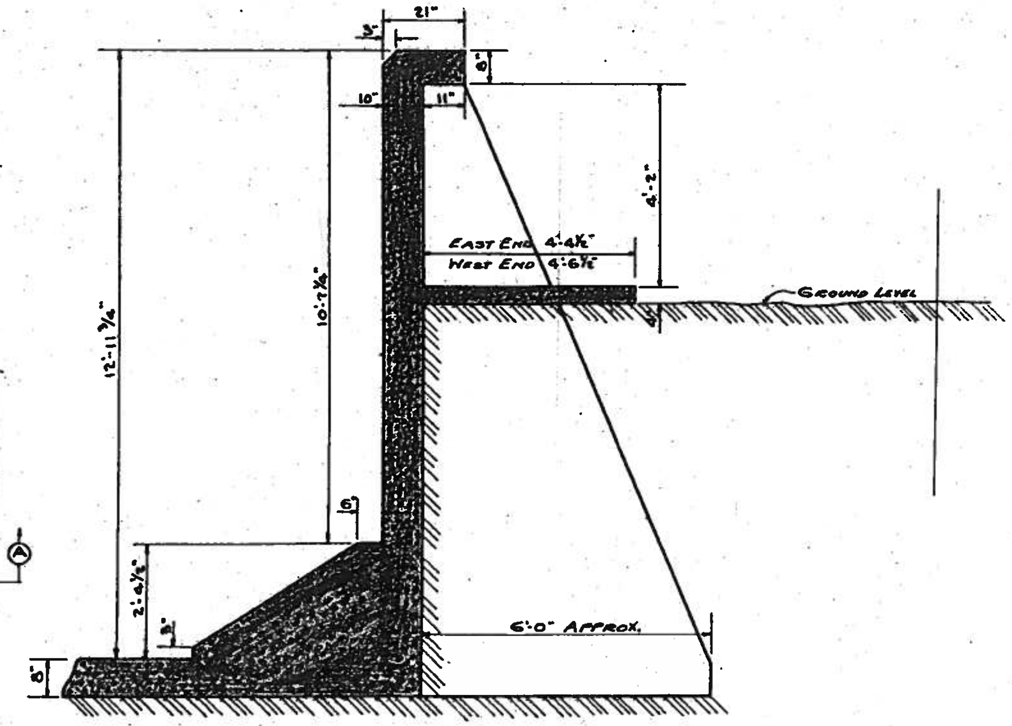
WLG WWPNR.GPJ 10/31/05 Log: kee Typ: kee Rev:


Rooling of Robertson V-beam Protected metal - 24 Gauge or equal, to be laid in 12' lengths (6" lap) 7 panels 12' long, 1 panel 6'-6" long (at bottom)
Tack-weld joists in place on beams
Top of wall Elev. 237.25
Water line Elev. 236.75
Slope 1:12
All beams & joists to be butt welded in place.
Six lines of main beams in this reservoir. Joists to rest on walls at ends.
End wall extension
factory-made steel door with butts and cylinder lock
3/8" rods 40'-0" long equipped with turn-buckles.
Thread thru 3/4" holes drilled or punched in web or flanges of stanchions
Tack-weld rods at columns after tightening.
See details below
Top of slope Elev. 224.75
Column bases must be built by roof builder and included in lump sum bid. Use Class B concrete, 12 cu.yd. required
Column bases level at Elev. 217.25
Before column bases are poured, finish expansion joint and clean thoroughly the surfaces to be covered by bases
TYPICAL SECTION SHOWING ROOF DETAILS
Scale: 1/4" = 1'-0"
Note: East & West Walls to be extended to 5" section level with top of main beams. Concrete for wall extension to be furnished by roof builder and included in lump sum bid. See drawing 1244. 24 1/2 cu.yd. Class B concrete required in wall extensions.
Note: Steel references are to be Carnegie sections. Equivalent sections are acceptable.
ENTRANCE LADDER & PLATFORM
SECTION d-d
Scale: 1/2" = 1'-0"
For wood roof move entrance 4 1/2 ft
Approx. overhang of roof.
Weld cover plate continuously along joist to provide seat
Tack weld joist in place
C.B 121 - 12" - 25 lbs.
C7 - 6" - 8.2 lbs. channel bearing plate 8" long cut & weld as shown
18" #18 gauge sheet steel cover plate
To be placed by reservoir builder
FLASHING DETAIL AT SIDE WALLS
Scale: 3" = 1'-0"
Expansion joint
Top of Base block Elev. 217.25
Copper water seal
SECTION a-a CORNER COLUMN FOOTING
To be cast on top of floor slab
Scale: 1" = 1'-0"
Upper End Wall
2'-6" x 6'-6" Steel Door
Top of berm Elev. 234.0
DETAIL OF ENTRANCE STAIRWAY
SECTION b-b
Scale: 6" = 1'-0"
JOINT MOUNTING AT END WALLS
SECTION c-c
12" - 25 lbs Beam CB 121
Tack-weld beam to stanchion
6" - 15.5 lbs Stanchion CB 6
Weld all around
base plate
dowels - 9" long
COLUMN DETAIL
Scale: 6" = 1'-0"
Dimensions shown above
Note: All columns and all material and work of any nature shown to be higher in elevation than top of reservoir wall (El. 237.25), except entrance platform and railings, is to be included in price bid for steel roof.
Minor details indicated on this drawing or on drawing showing proposed steel roof over old reservoir are subject to change if such changes do not adversely affect appearance, life or utility of roof structure.
Steel roofs are designed to carry uniformly distributed live loads of 40# per sq ft
MUNICIPAL WATER WORKS
VANCOUVER WASHINGTON
J.L. HENDERSON
CITY ENGINEER

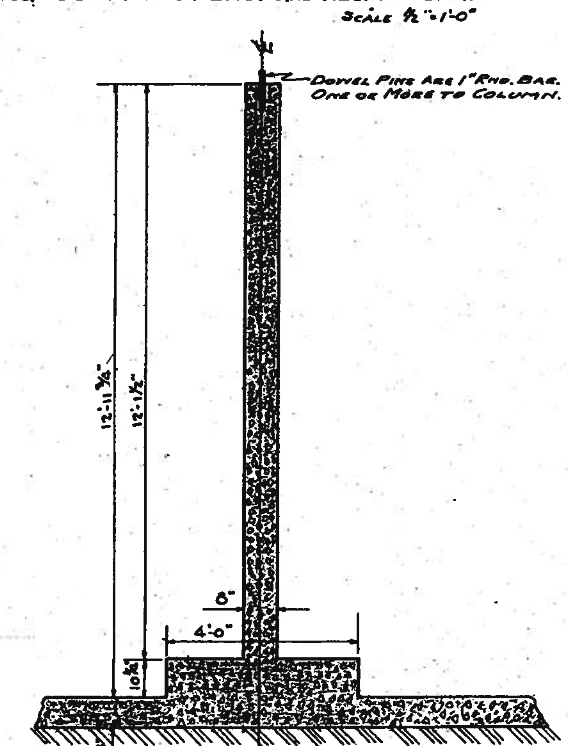

GENERAL PLAN & SECTION OF RESERVOIR WITH ELEVATIONS OF FOUNDATION WALLS
Scale: 1/16"=1'-0"
FOUNDATION WALLS
East half of reservoir to be built subsequently
PLAT OF PROPERTY AT PUMPING STATION No1.
EAST RESERVE ST.
TYPICAL SECTION OF SIDE OR END WALL
Scale: 1/2"=1'-0"
TYPICAL SECTIONS OF FOUNDATION WALLS
Scale: 1/4"=1'-0"
Side or end wall
Division wall
Note: All slope and bottom slabs must be finished with a steel trowel held in the hand of a skilled workman. Edges next to expansion joints must be finished with an edging tool.
FUTURE RESERVOIR
TYPICAL SECTION OF DIVISION WALL
Scale: 1/2"=1'-0"
Construction joint with copper water seal in place. Forms to be left in place to protect copper. Extension for future reservoir to begin here.
Foundation strips under all joints
MUNICIPAL WATER WORKS
VANCOUVER WASHINGTON

STA #1 SMALL





DETAIL A-A, TYPICAL CROSS SECTION OF WALL, WALK & BUTTRESS. SIDEWALKS ON EAST AND WEST END ONLY.
SCALE ½"=1'-0"



TYPICAL CROSS SECTION OF ALL SLABS AND COLUMNS.
SCALE ½"=1'-0"

242/5

Revisions		
Date	By	App'd

PEOPLES WATER AND GAS CO.
ENGINEERING DEPARTMENT
VANCOUVER DISTRICT

PLAN SHOWING POSITIONS OF SLABS & COLUMNS AT RESERVOIR, STA. No 1.

Drawn By W.E.W.	Date 12-28-34	Approved By
Traced By W.E.W.		Auth. No
Checked By	Scale 1"=8'-0"	Dwg No. VA-526

APPENDIX C

LABORATORY TESTING

TABLE OF CONTENTS

FIGURES

APPENDIX C

LABORATORY TESTING

C.1 GENERAL

Soil samples obtained during field explorations were examined in the laboratory. Physical characteristics of the samples were noted and field classifications were modified as necessary in accordance with the terminology presented in Appendix A, Figure A1. During the course of the examination, representative samples were selected for further testing. The soil-testing program included index property tests such as moisture content analyses, mechanical gradations, as well as Atterberg limits. These tests are described in the following paragraphs. All test procedures were performed in general accordance to applicable ASTM International standards. The term "general accordance" means that certain local and common descriptive practices and methodologies have been followed.

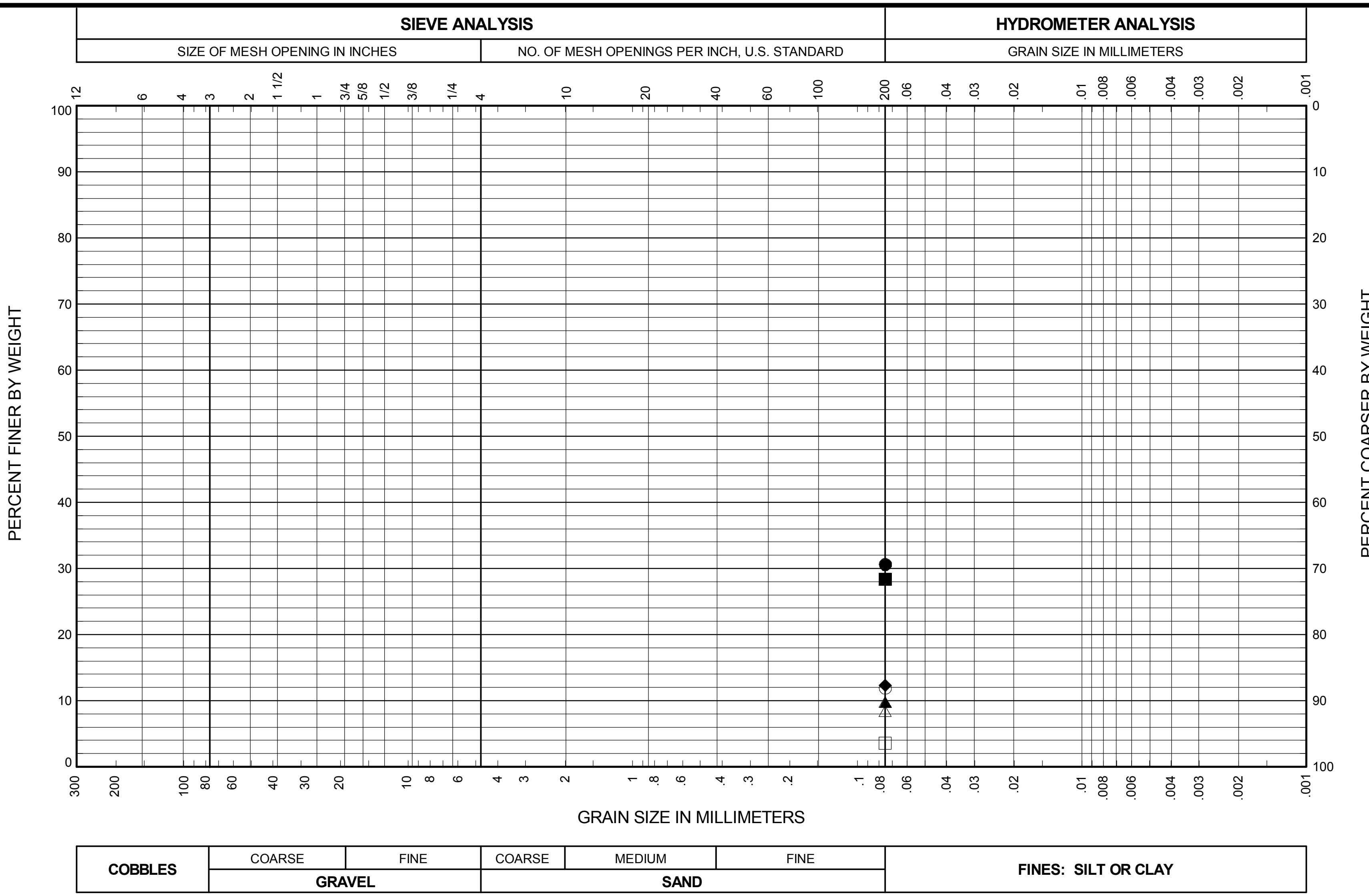
C.2 SOIL TESTING

C.2.1 Moisture (Natural Water) Content

Natural moisture content determinations were performed, in accordance with ASTM D2216, on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time of exploration. It is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of moisture content determinations are presented in the boring logs in Appendix A.

C.2.2 Mechanical Particle-Size Analysis

Particle-size analyses were conducted on samples to determine the grain size distribution. Grain size distributions were determined by sieve analysis in accordance with ASTM D422 and ASTM D1140. Results of the particle-size analyses are presented in Figure C1, Grain Size Distribution. For all particle-size analyses, the percentage of material passing the No. 200 sieve is also shown graphically in the boring logs in Appendix A.



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1-12, S-2	5.0	SM	Silty SAND	-	-	31	16	
■ B-1-12, S-3	7.5	SM	Silty SAND	-	-	28	26	
▲ B-1-12, S-5	12.5	SP-SM	SAND with silt	-	-	10	8	
◆ B-1-12, S-9	25.0	SP-SM	SAND with silt	-	-	12	12	
○ B-2-12, S-2	5.0	SP-SM	SAND with silt	-	-	12	16	
□ B-2-12, S-3	7.5	SP	SAND	-	-	4	10	
△ B-2-12, S-4	10.0	SP-SM	SAND with silt	-	-	8	7	

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GRAIN SIZE DISTRIBUTION

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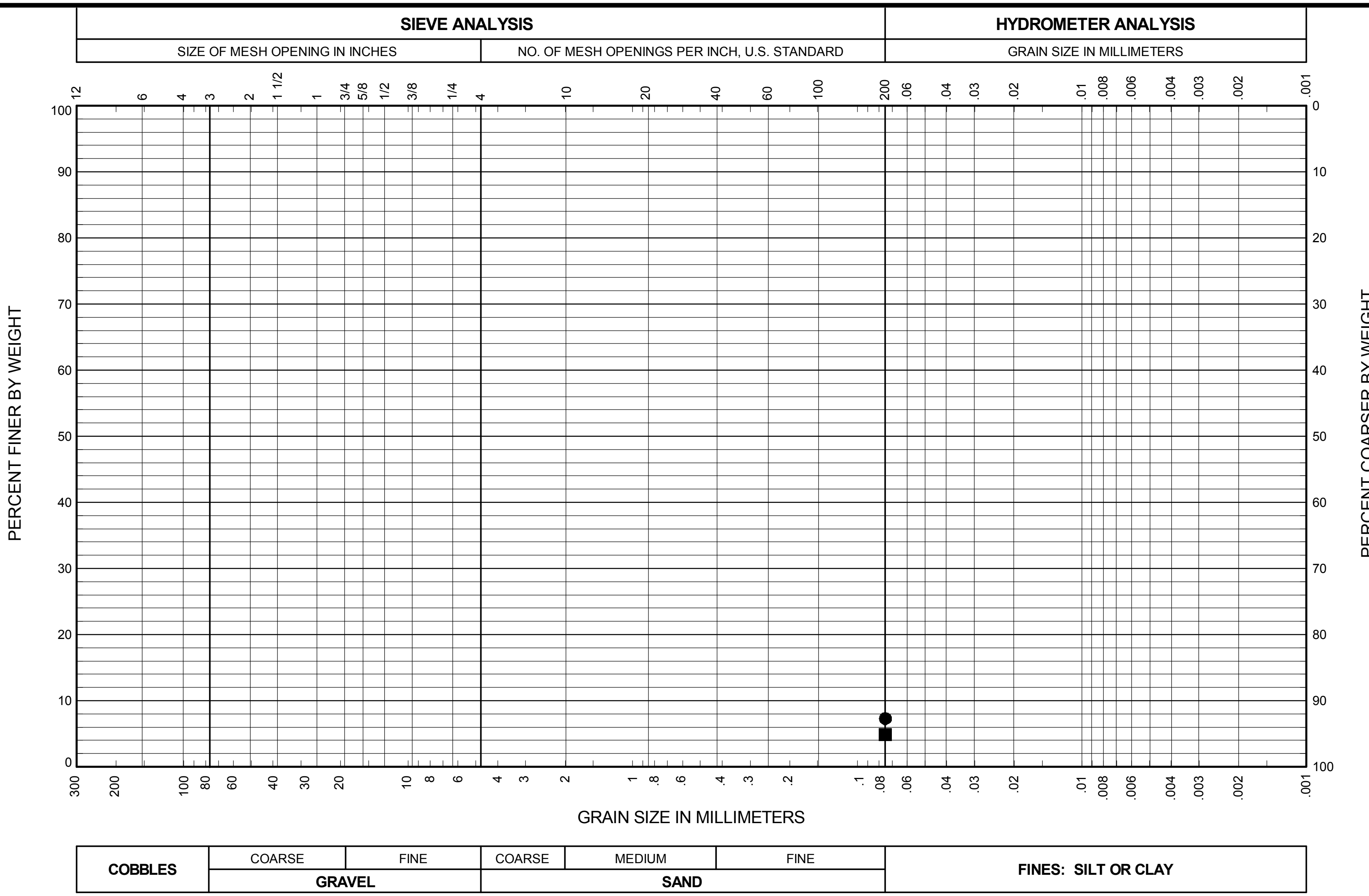
FIG. C1
Sheet 1 of 2

GSA_MAIN 24-1-03744-001.GPJ SHAN_WIL.GDT 4/25/13

GSA_MAIN 24-1-03744-001.GPJ SHAN_WIL.GDT 4/25/13



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-4-12, S-2	7.5	SP	SAND, trace silt	-	-	7	7	
■ B-4-12, S-3	10.0	SP	SAND, trace silt	-	-	5	8	

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GRAIN SIZE DISTRIBUTION

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FIG. C1
Sheet 2 of 2

FIG. C1

APPENDIX D

SLOPE STABILITY ANALYSES

FIGURES

SITE PHOTOGRAPHS

Cross Section A-A'
4MG Reservoir Facility
Static Conditions

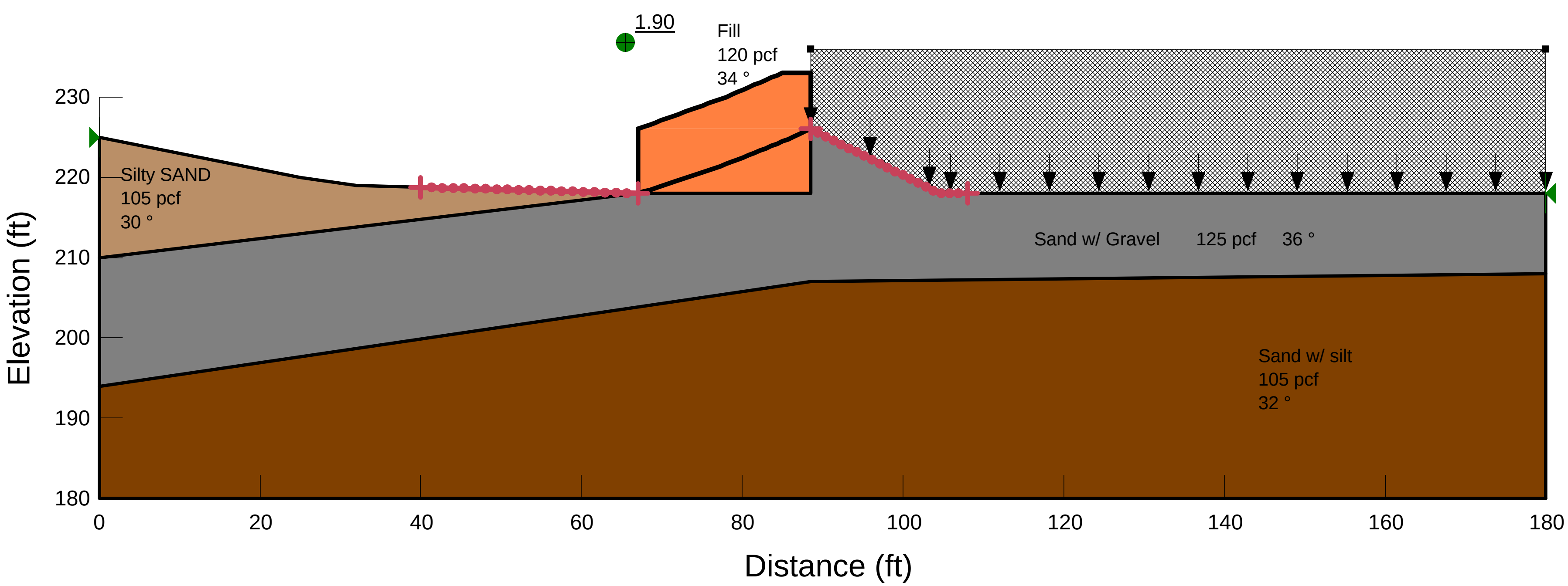


Figure D1

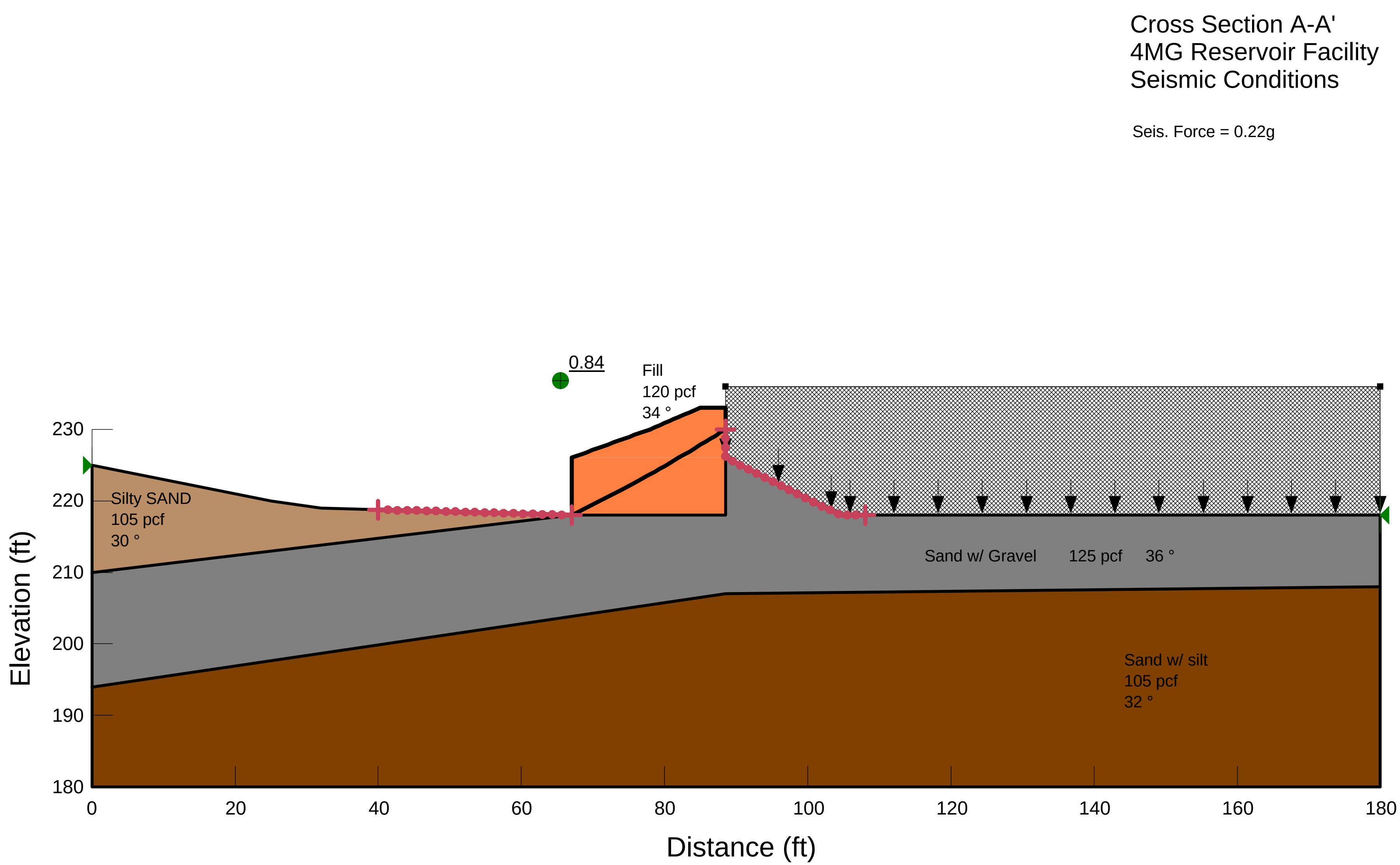

Cross Section A-A'
4MG Reservoir Facility
Seismic Conditions
Seis. Force = 0.22g
0.84
Fill
120 pcf
34 °
Silty SAND
105 pcf
30 °
Sand w/ Gravel 125 pcf 36 °
Sand w/ silt
105 pcf
32 °
Elevation (ft)
Distance (ft)
230
220
210
200
190
180
0
20
40
60
80
100
120
140
160
180

Figure D2

Cross Section A-A'
4MG Reservoir Facility
Seismic Conditions w/Fill Removed

Seis. Force = 0.22g

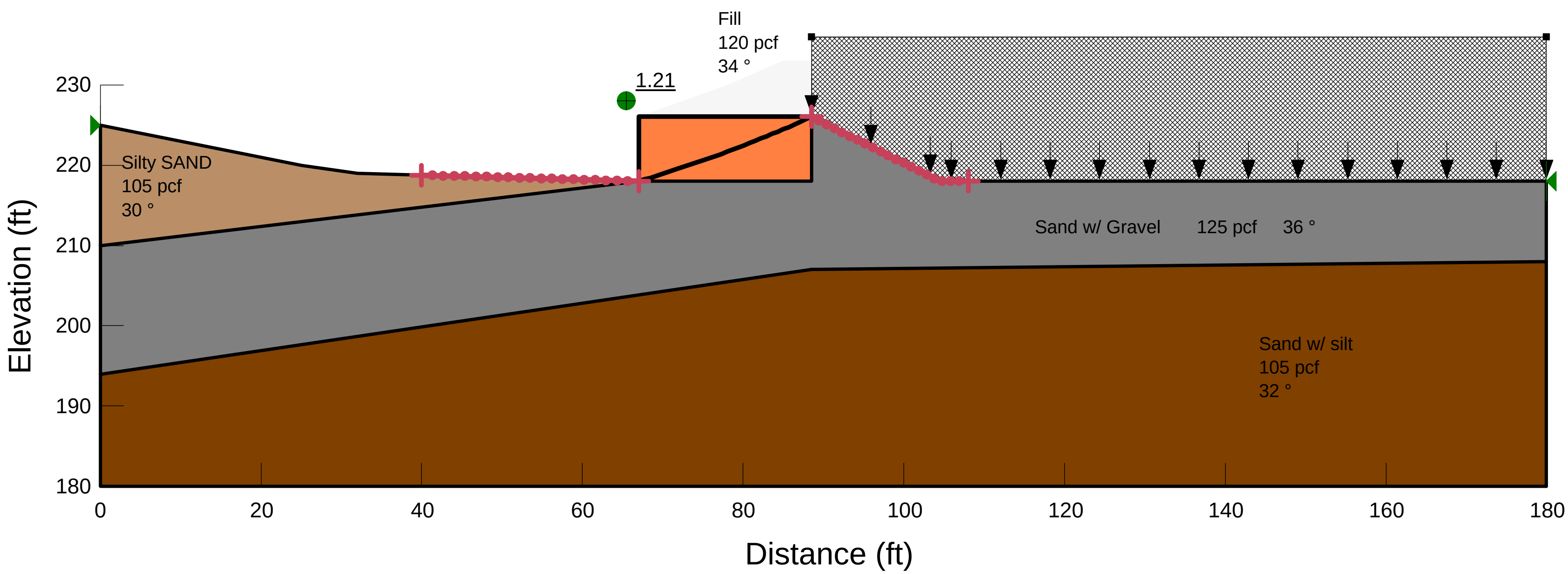


Figure D3

Cross Section B-B'
1MG Reservoir, No Leak
Static Conditions

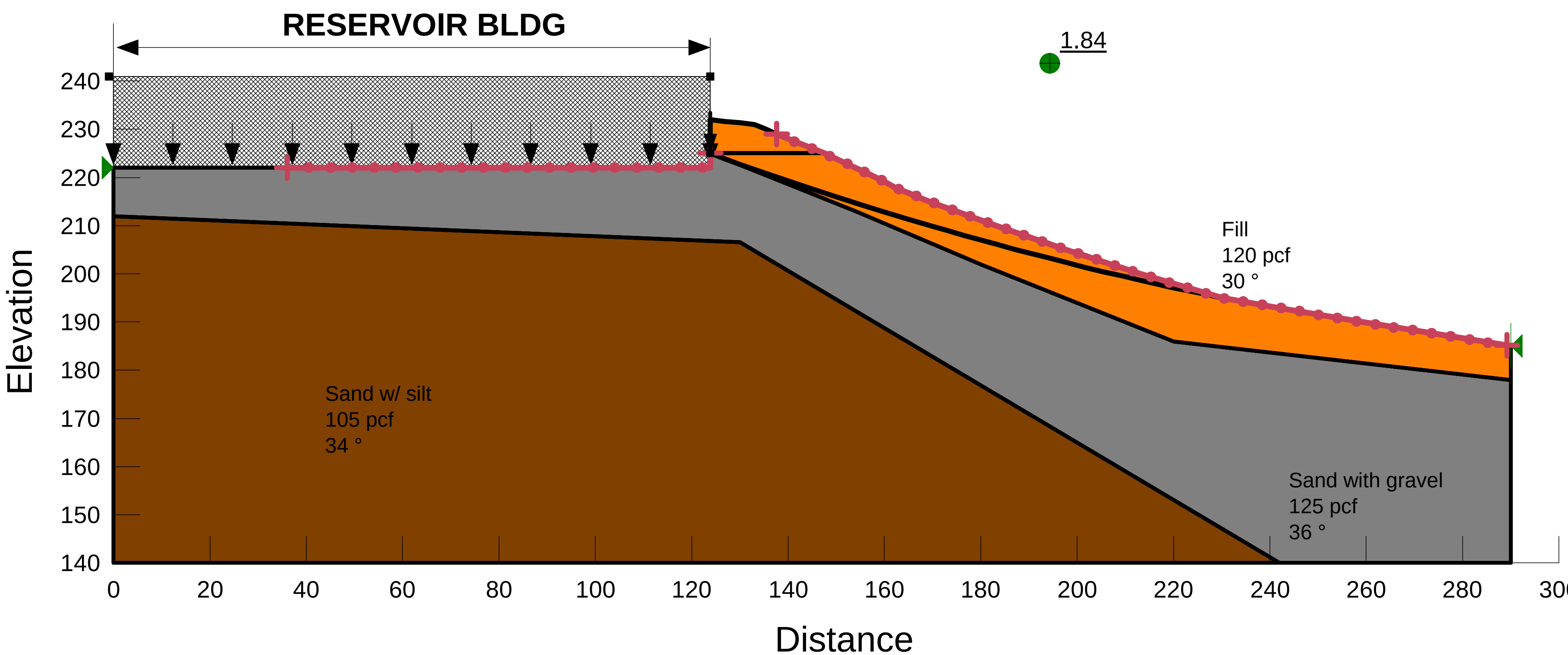


Figure D4

Cross Section B-B'
1MG Reservoir, No Leak
Seismic Conditions

Seis. Force = 0.22g

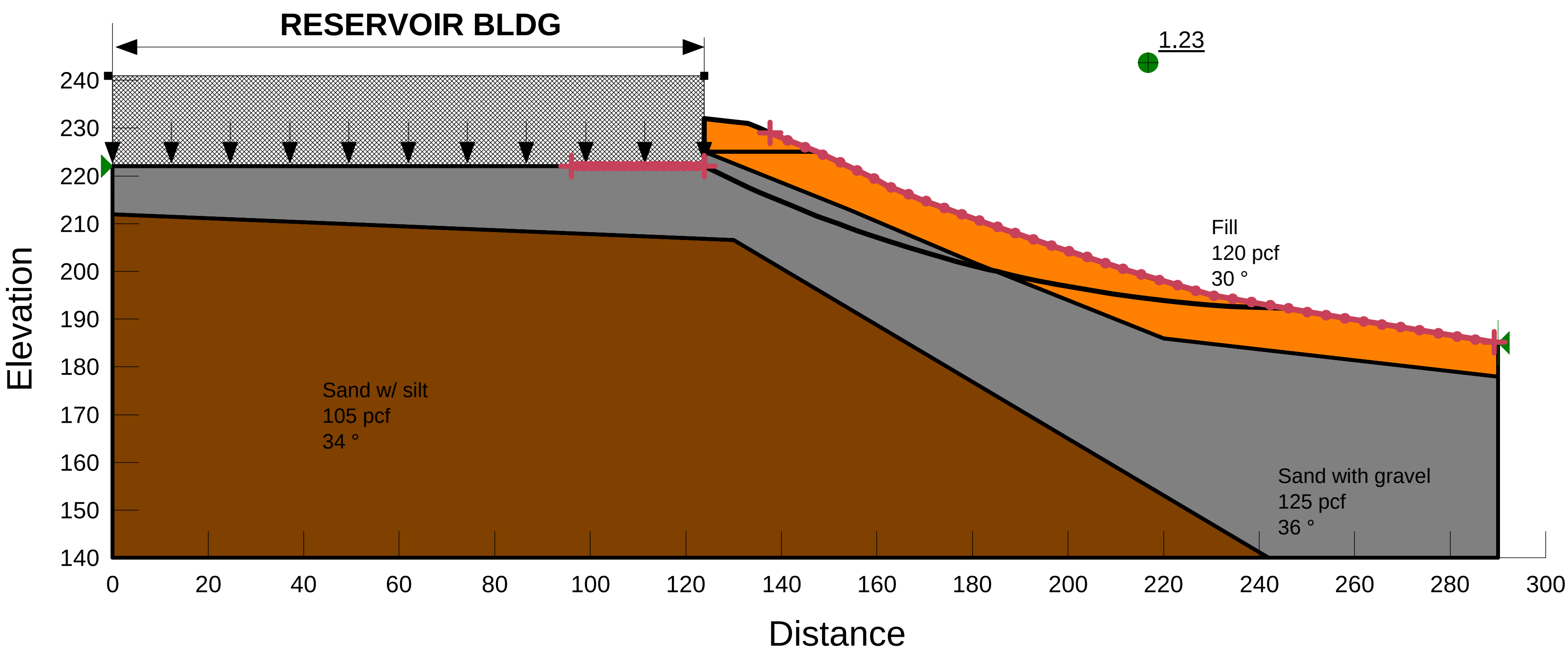
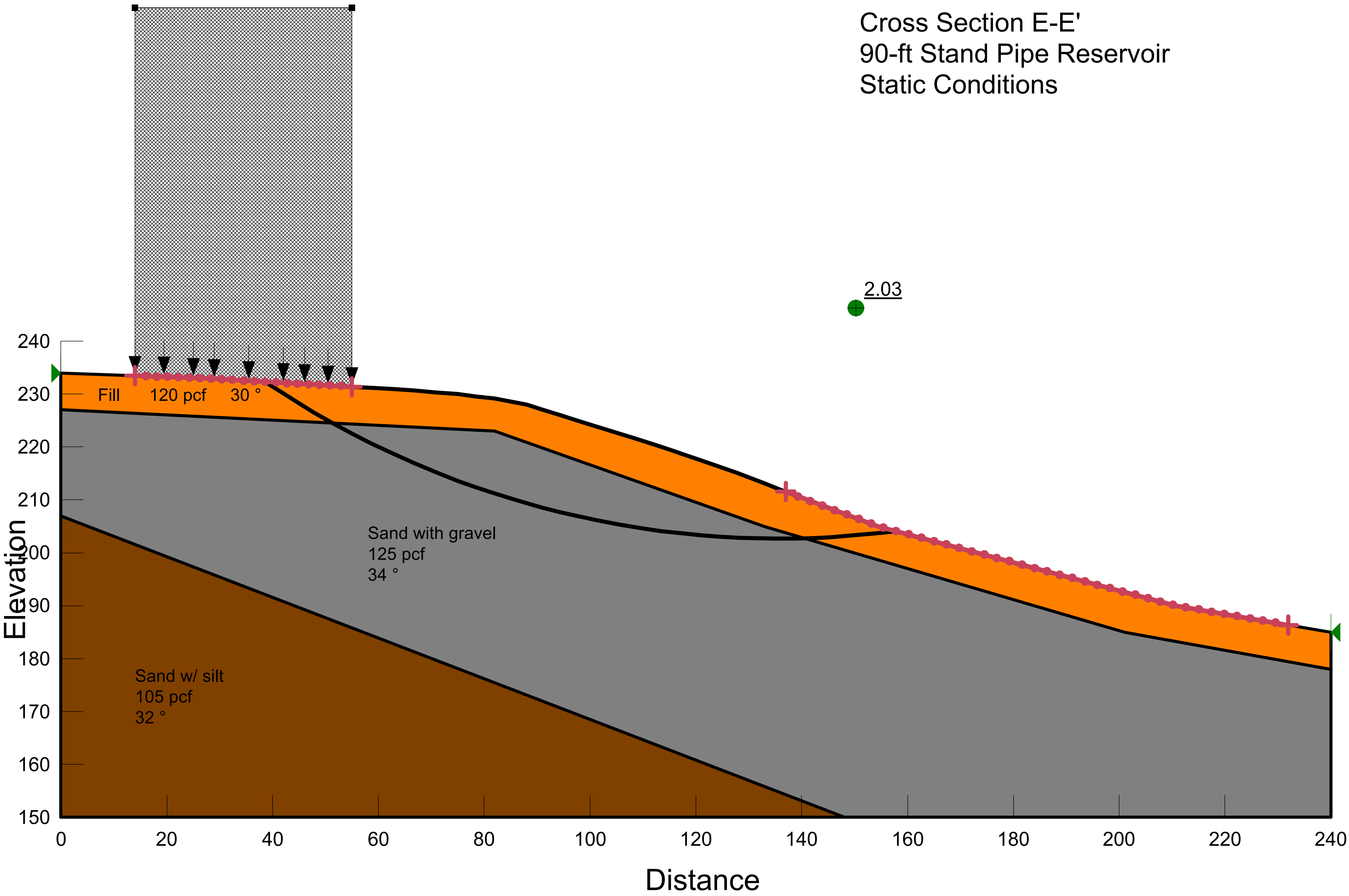
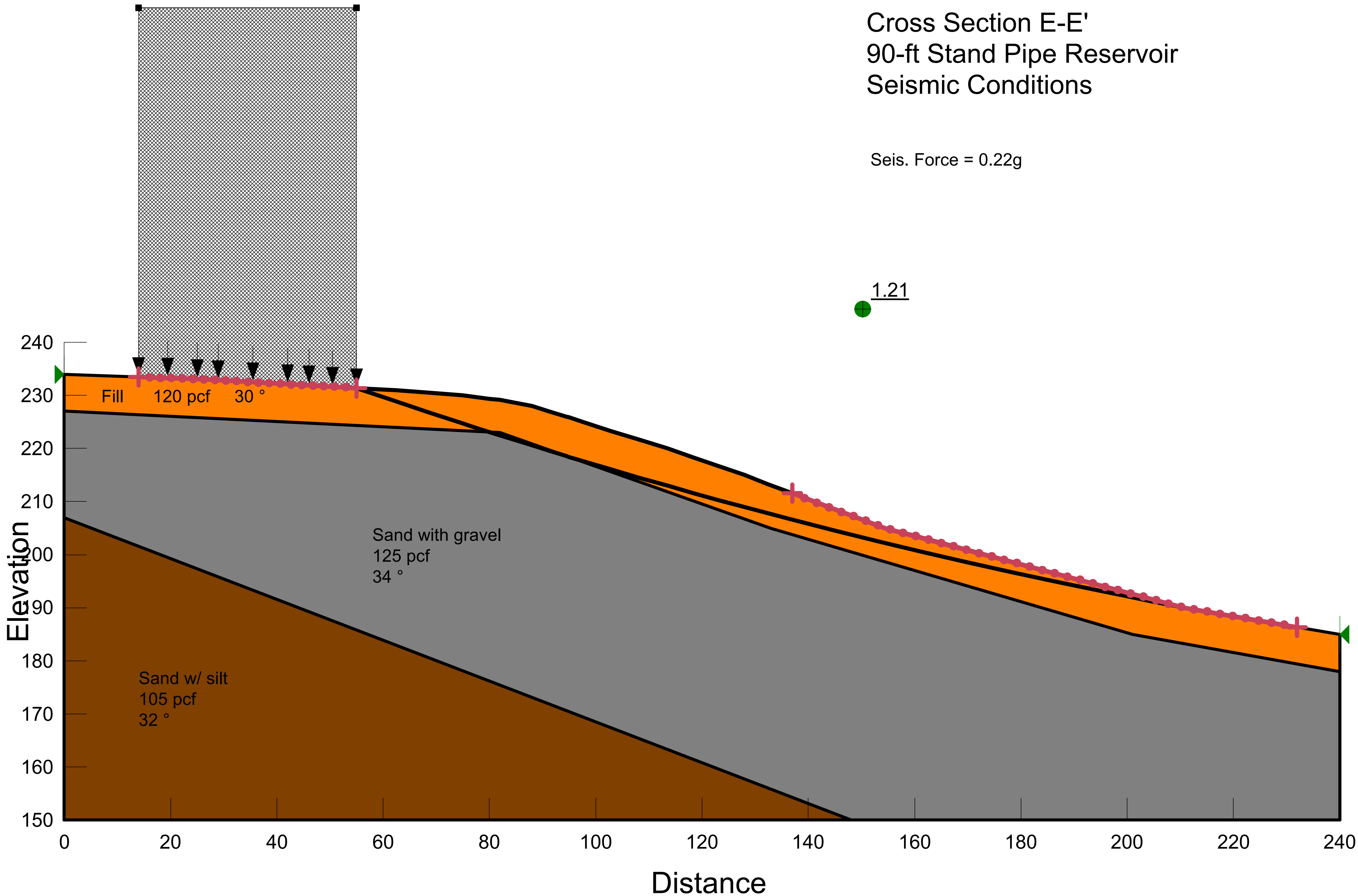


Figure D5



Cross Section E-E'
90-ft Stand Pipe Reservoir
Static Conditions
2.03
Fill
120 pcf
30 °
Sand with gravel
125 pcf
34 °
Sand w/ silt
105 pcf
32 °
Elevation
Distance
240
230
220
210
200
190
180
170
160
150
0
20
40
60
80
100
120
140
160
180
200
220
240

Figure D6



Cross Section E-E'
90-ft Stand Pipe Reservoir
Seismic Conditions
Seis. Force = 0.22g
1.21
Fill
120 pcf
30 °
Sand with gravel
125 pcf
34 °
Sand w/ silt
105 pcf
32 °
Elevation
240
230
220
210
200
190
180
170
160
150
0
20
40
60
80
100
120
140
160
180
200
220
240
Distance

Figure D7

Cross Section A-A'
4MG Reservoir Facility
Minimum Yield Acceleration Determination

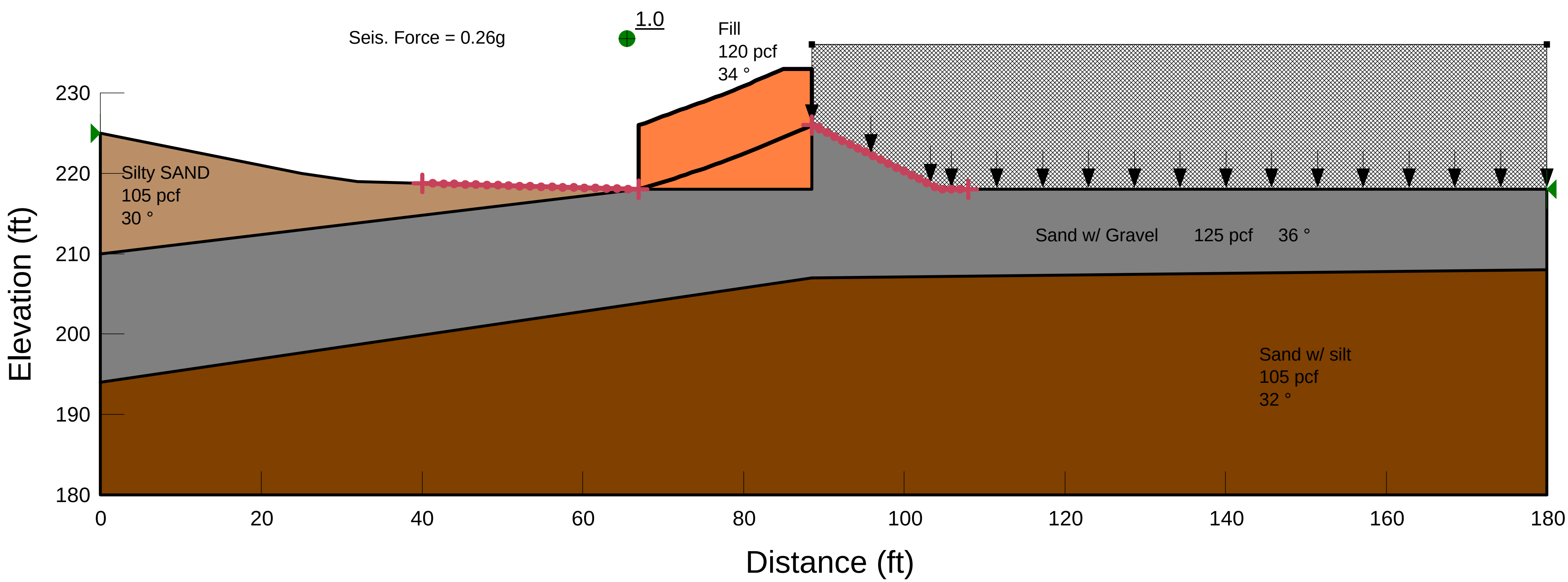


Figure D8

Photo 1: 1MG Reservoir, Northeast Corner

Vancouver Reservoirs Seismic Retrofit
Vancouver, Washington

SITE PHOTOGRAPHS

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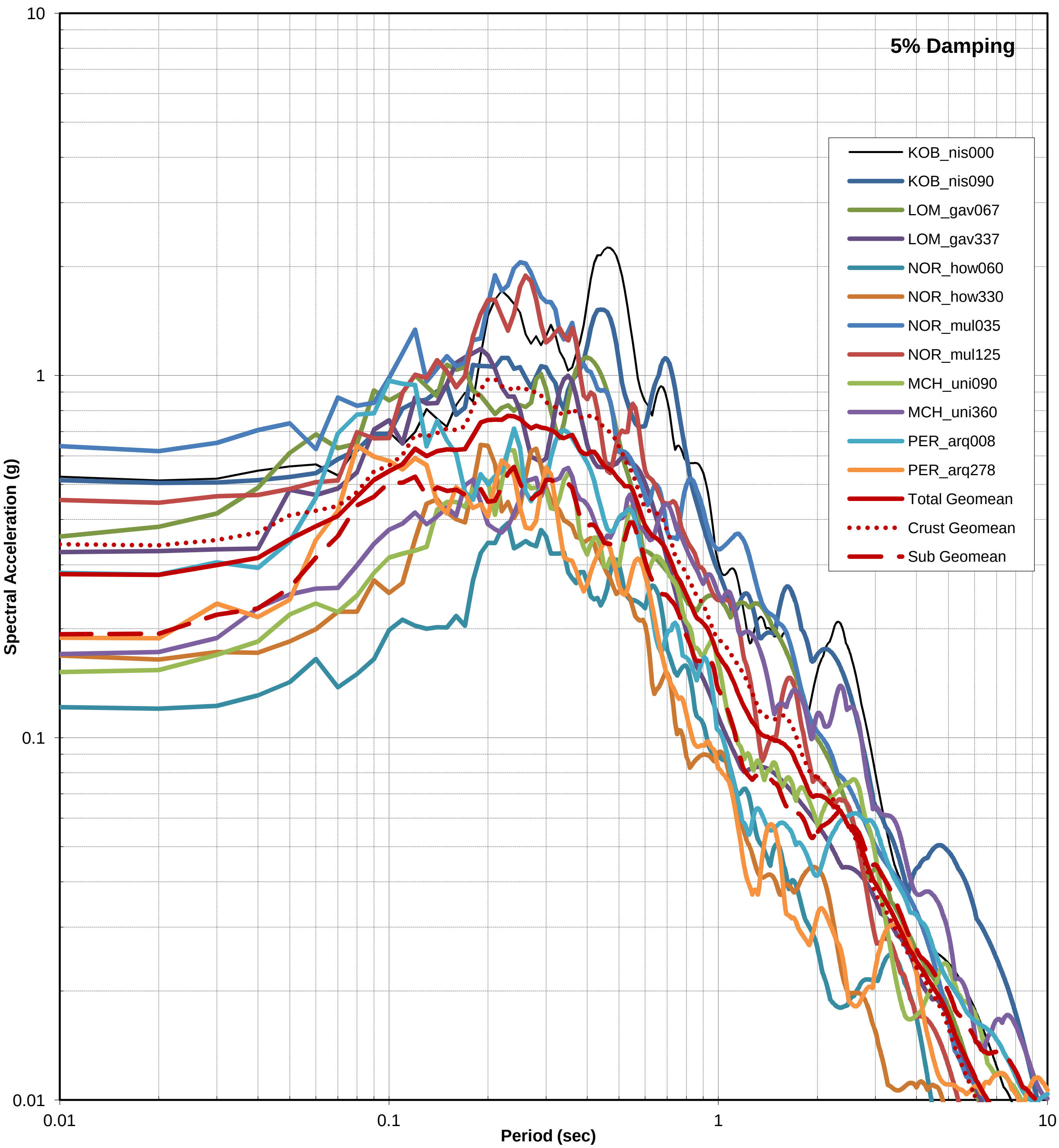
FIG. D9

APPENDIX E

GROUND MOTION TIME HISTORIES AND SLOPE DISPLACEMENT ANALYSES

FIGURES

ATTACHMENT



Vancouver Reservoir Seismic Retrofit
Vancouver, Washington

SPECTRAL ACCELERATION AS-RECORDED BEDROCK MOTIONS ALL TIME HISTORIES

January 2013 | 24-1-03744-001

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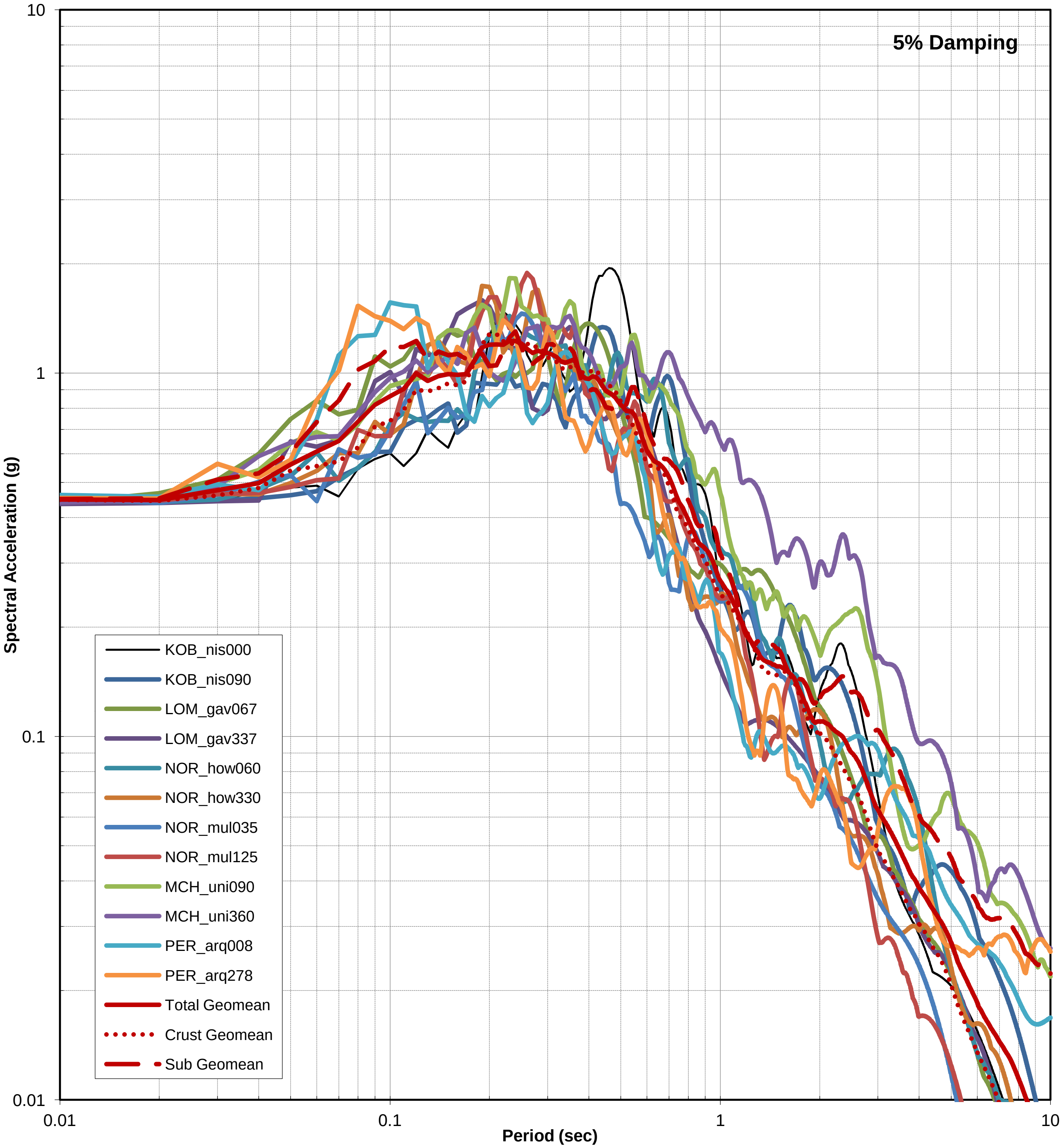
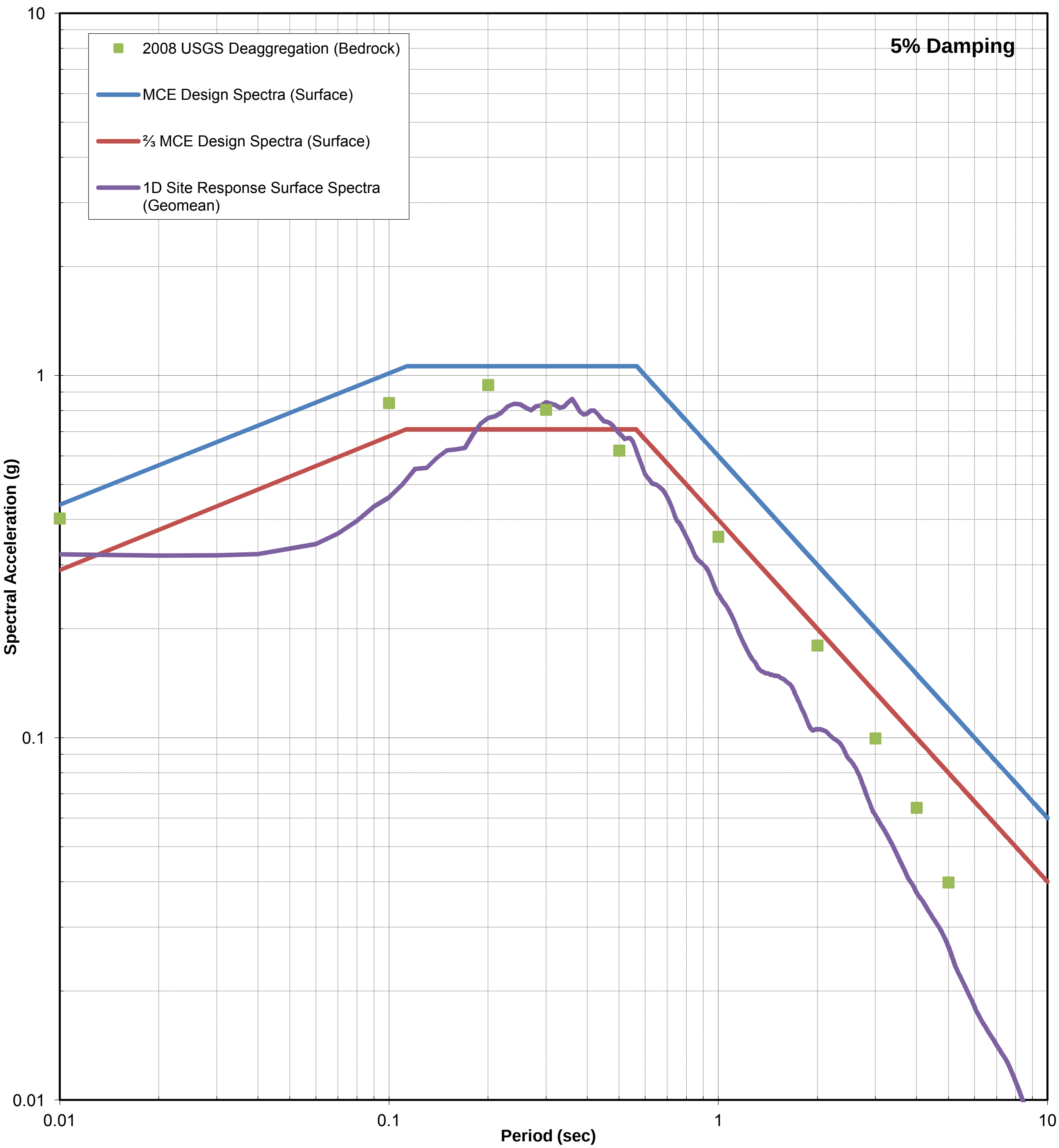
FIG. E1



Vancouver Reservoir Seismic Retrofit Vancouver, Washington	
SPECTRAL ACCELERATION SCALED BEDROCK MOTIONS ALL TIME HISTORIES	
January 2013	24-1-03744-001
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. E2**



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Vancouver, Washington

CODE-BASED DESIGN SURFACE ACCELERATION RESPONSE SPECTRA

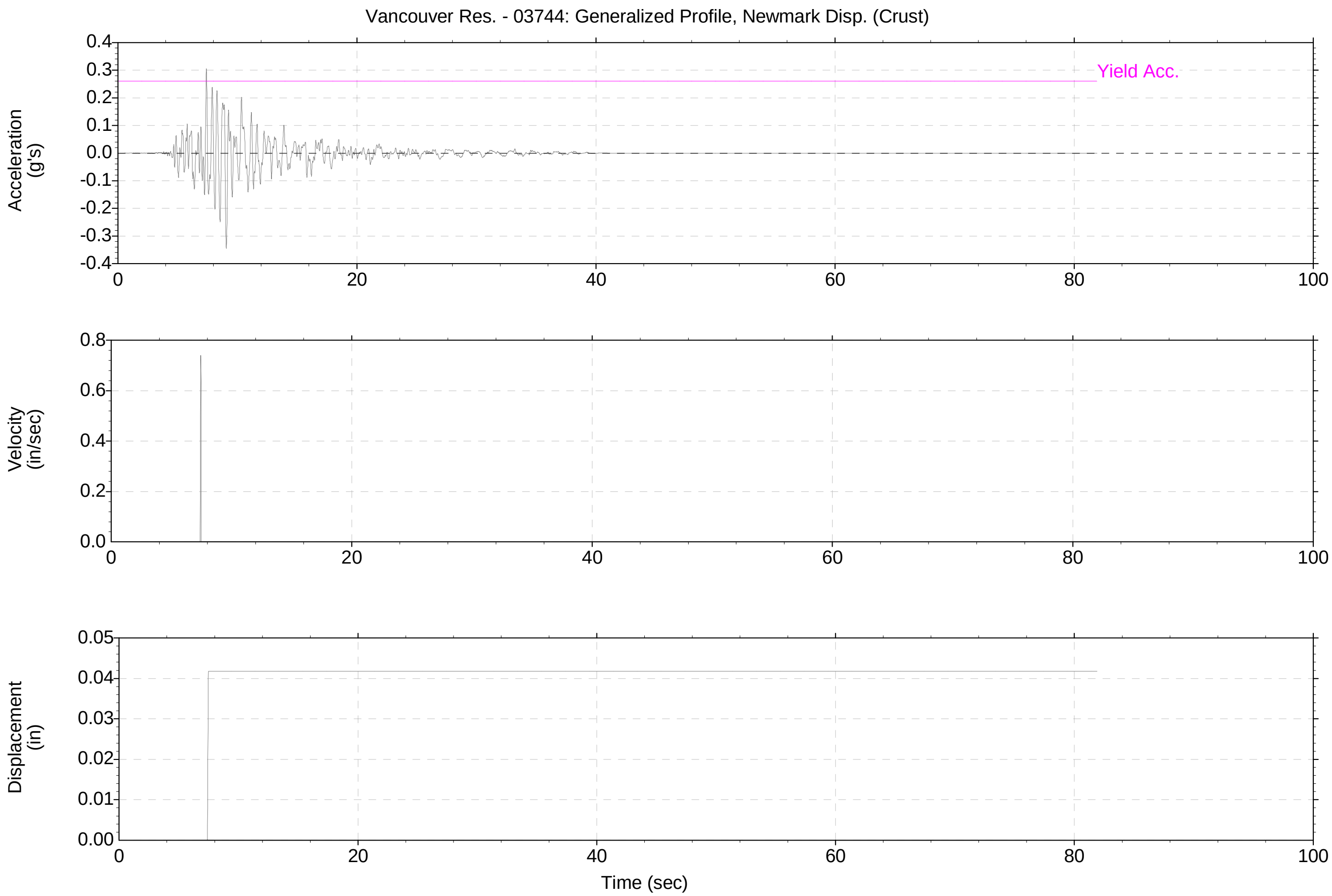
January 2013 | 24-1-03744-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E3

APPENDIX E ATTACHMENT

Newmark Displacement Analysis Results

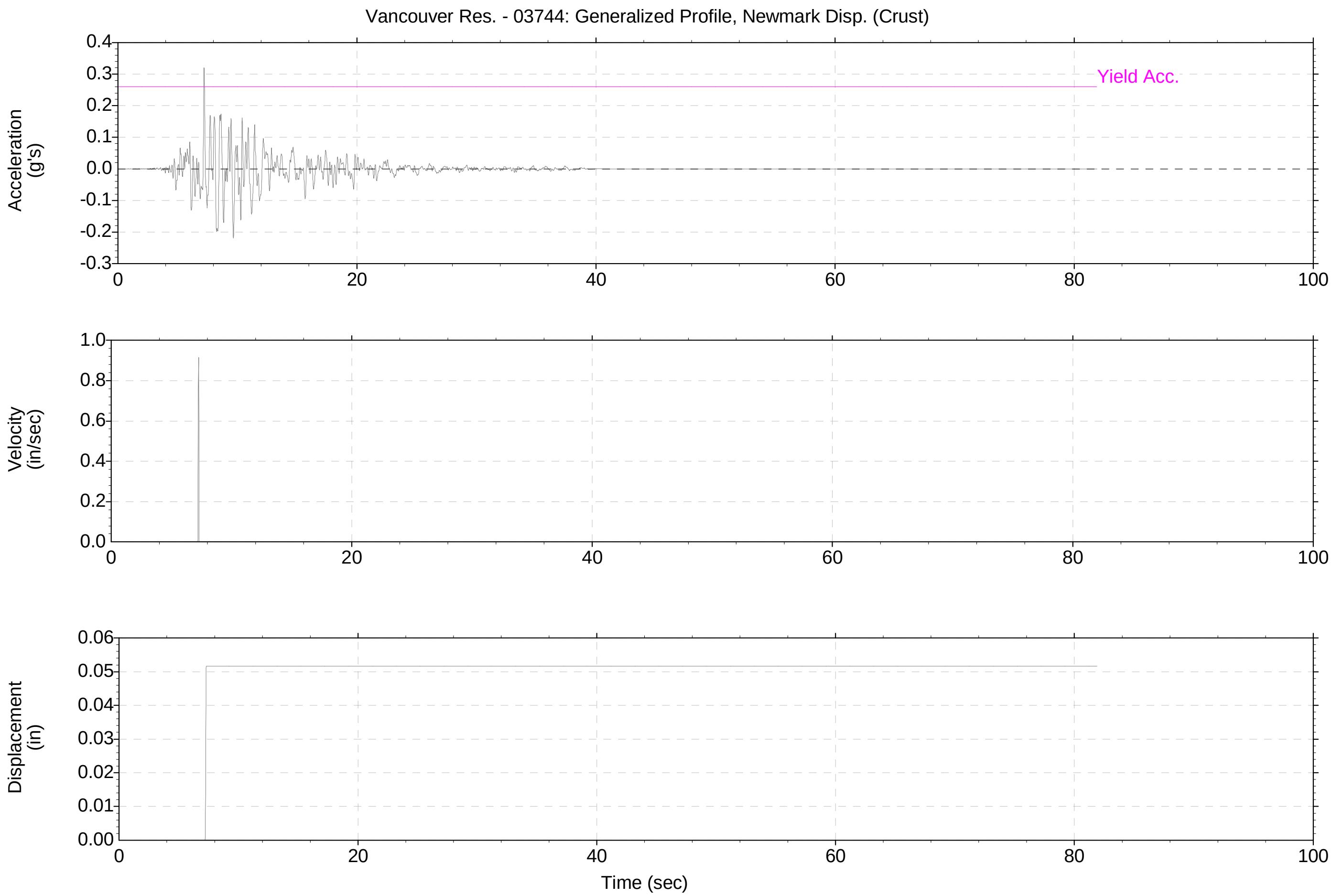


Notes:

Displacement Analysis - Newmark Method
Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

Newmark Method by Houston et.al. (1987)
Constant Yield Acceleration: .26 (g)
Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                           Analysis: 1 - Soil Deposit: 4
Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A1D4-1-Generali-nis000.ahl
Peak Acceleration Value: .344892 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

Displacement computed: 4.179921E-02 in

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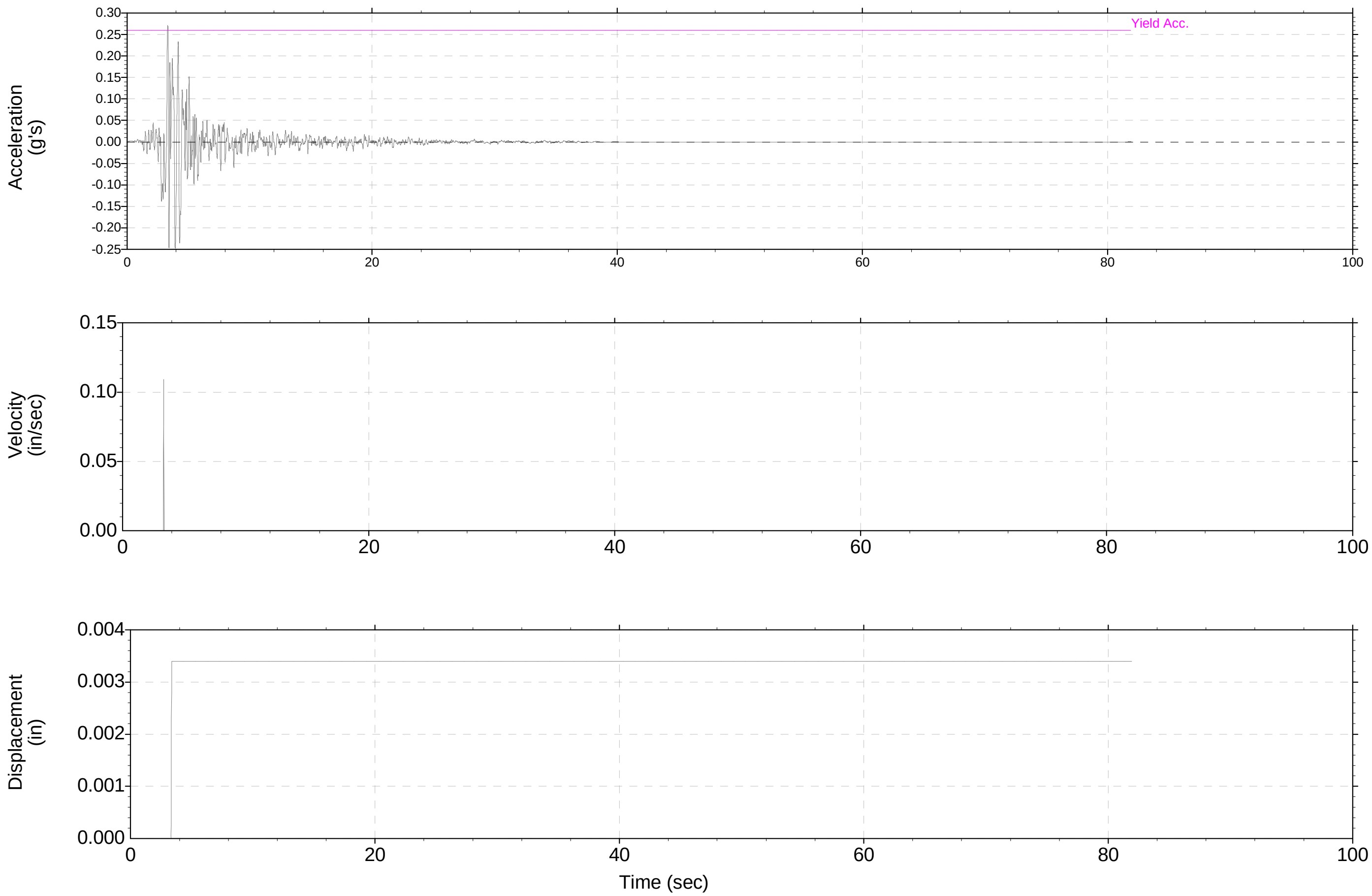
Notes:
      Displacement Analysis - Newmark Method
      Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

      Newmark Method by Houston et.al. (1987)
      Constant Yield Acceleration:      .26 (g)
      Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                                 Analysis: 2 - Soil Deposit: 4
      Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A2D4-2-Generali-nis090.ahl
      Peak Acceleration Value:  .319856 (g)
      Upslope Movement not Included in Analysis
      Acceleration due to gravity:   386.4 (in/sec^2)

      Displacement computed: 5.165429E-02 in
```

Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

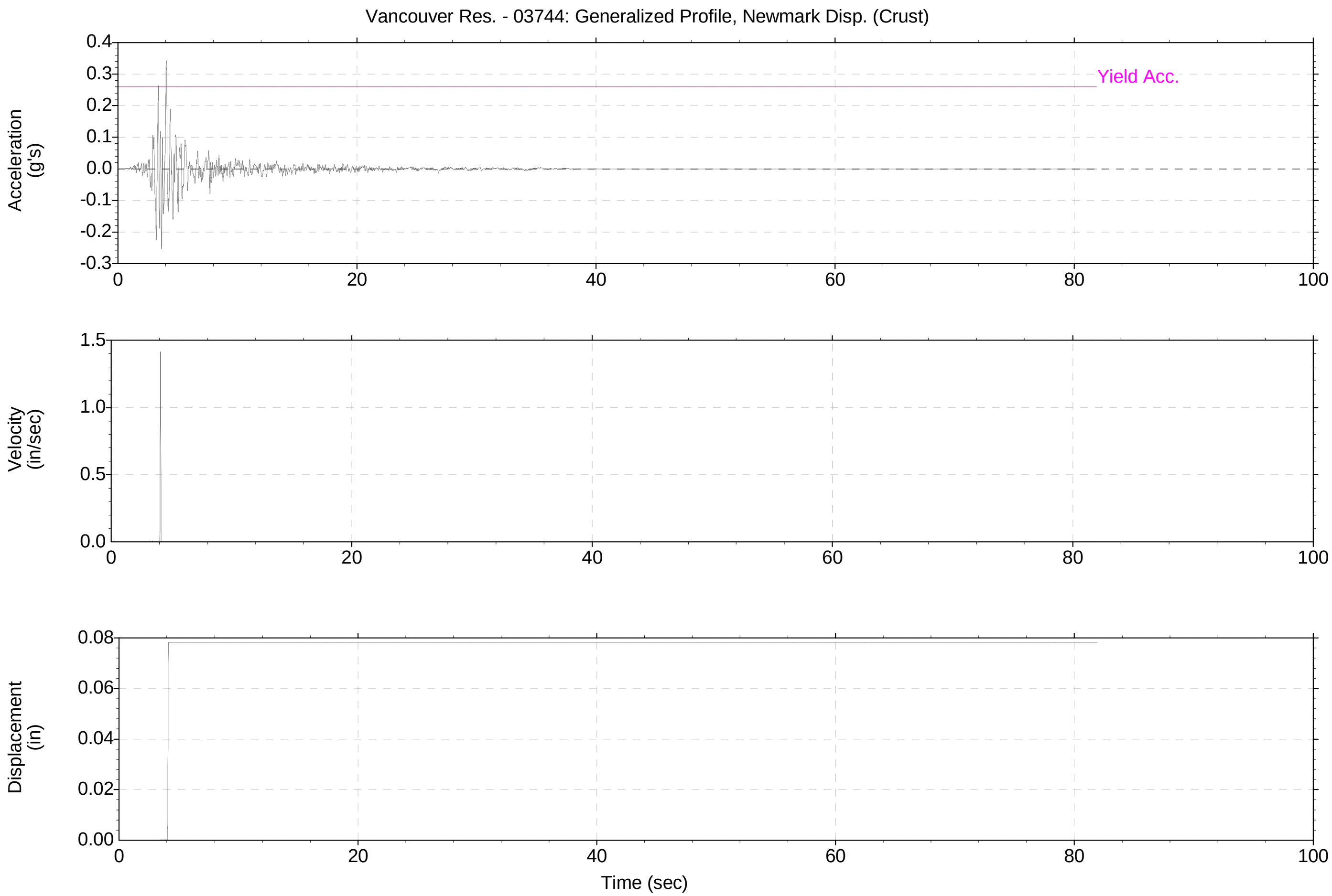


Notes:

Displacement Analysis - Newmark Method
Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

Newmark Method by Houston et.al. (1987)
Constant Yield Acceleration: .26 (g)
Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 - Analysis: 3 - Soil Deposit: 4
Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A3D4-3-Generali-gav067.ahl
Peak Acceleration Value: .270792 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

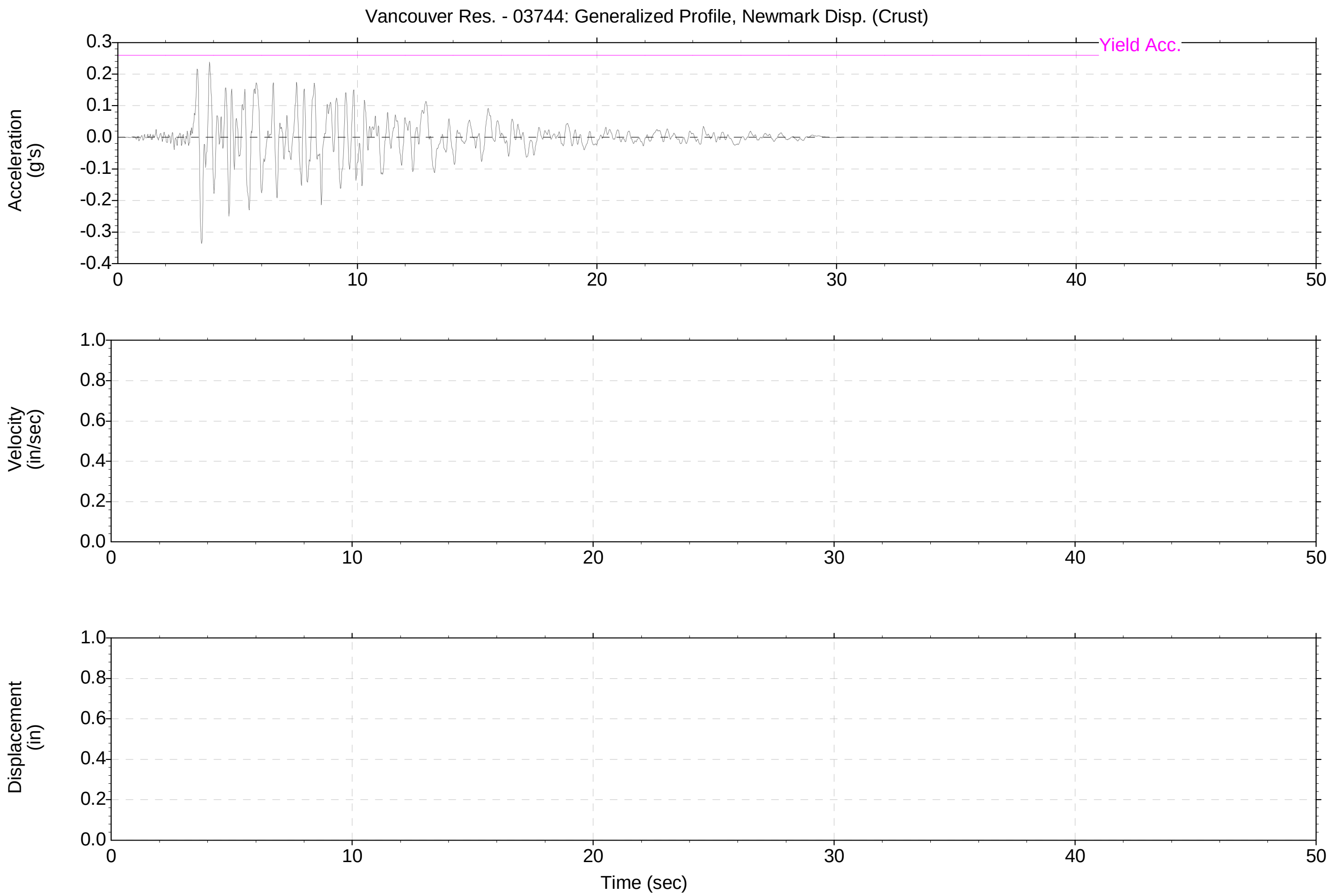
Displacement computed: 3.396416E-03 in




```
Notes:
      Displacement Analysis - Newmark Method
      Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

      Newmark Method by Houston et.al. (1987)
      Constant Yield Acceleration:      .26 (g)
      Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                                 Analysis: 4 - Soil Deposit: 4
      Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A4D4-4-Generali-gav337.ahl
      Peak Acceleration Value:  .341649 (g)
      Upslope Movement not Included in Analysis
      Acceleration due to gravity:   386.4 (in/sec^2)

      Displacement computed: 7.834553E-02 in
```

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Notes:
     Displacement Analysis - Newmark Method
     Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

     Newmark Method by Houston et.al. (1987)
     Constant Yield Acceleration:      .26 (g)
     Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                           Analysis: 5 - Soil Deposit: 4
     Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A5D4-5-Generali-how060.ahl
     Peak Acceleration Value:   .33619 (g)
     Upslope Movement not Included in Analysis
     Acceleration due to gravity:   386.4 (in/sec^2)

     Displacement computed:           0 in
```
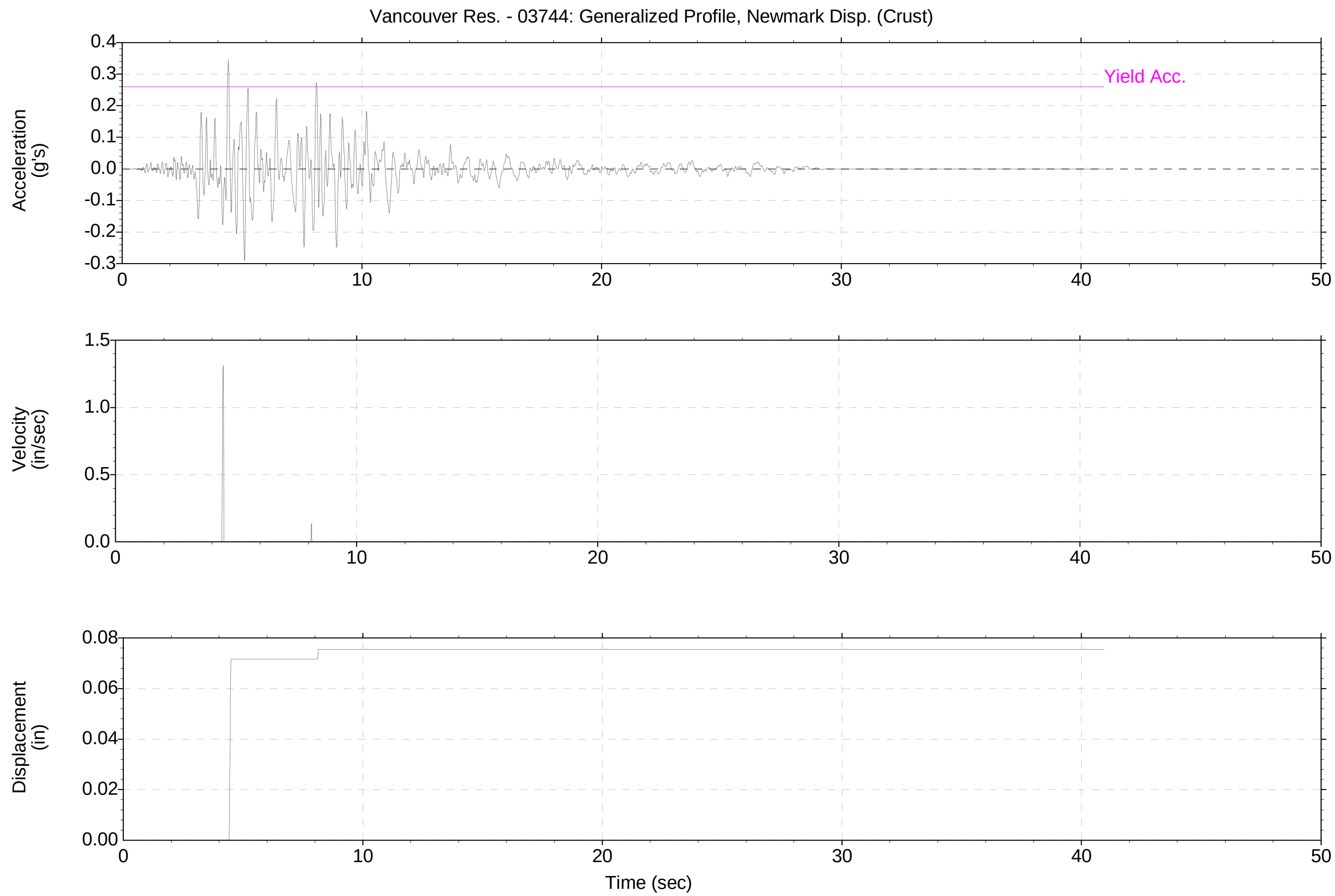
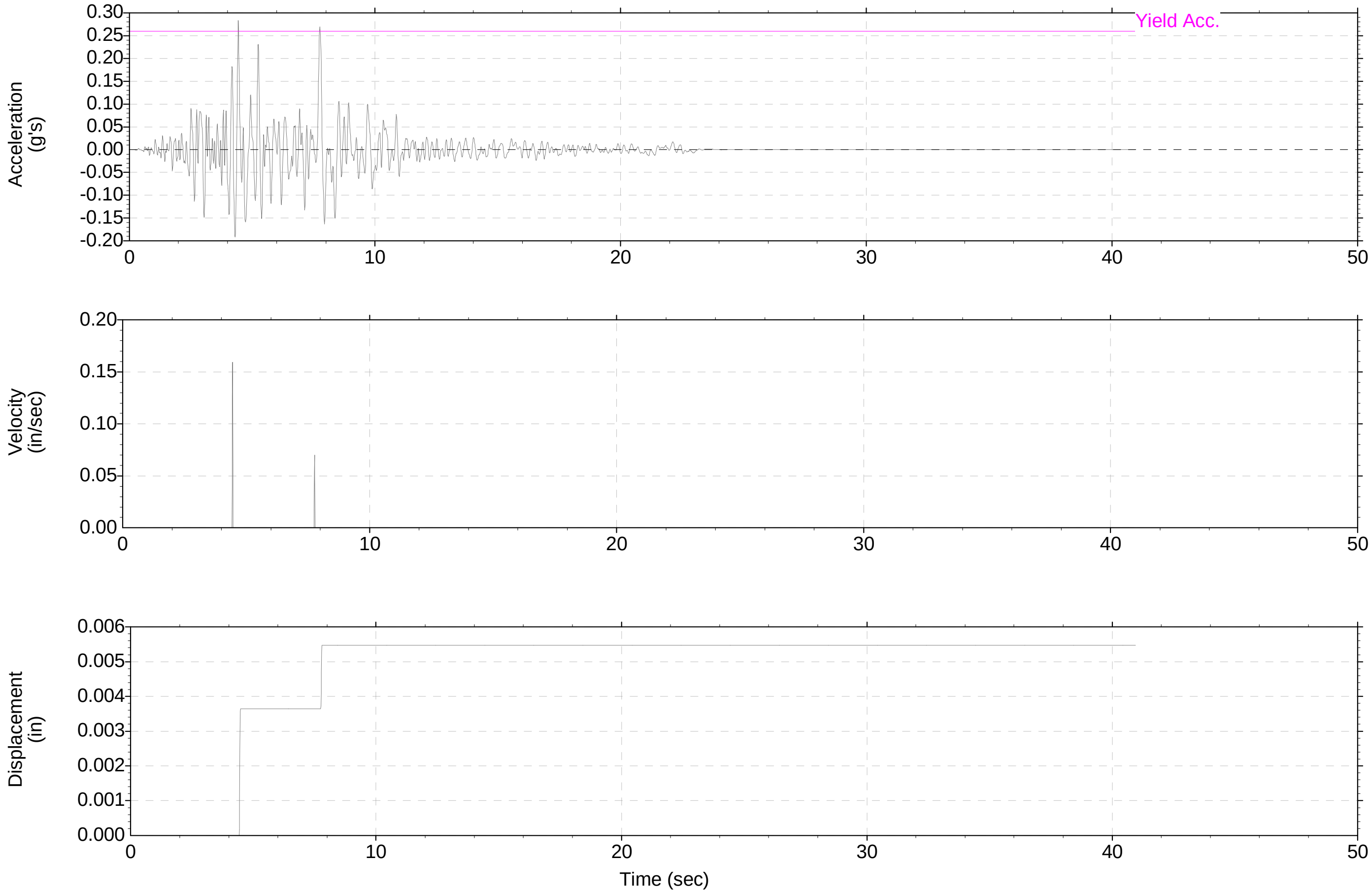



Notes:
Displacement Analysis - Newmark Method
Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

Newmark Method by Houston et.al. (1987)
Constant Yield Acceleration: .26 (g)
Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
Analysis: 6 - Soil Deposit: 4
Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A6D4-6-Generali-how330.ahl
Peak Acceleration Value: .345707 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

Displacement computed: 7.553837E-02 in

Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

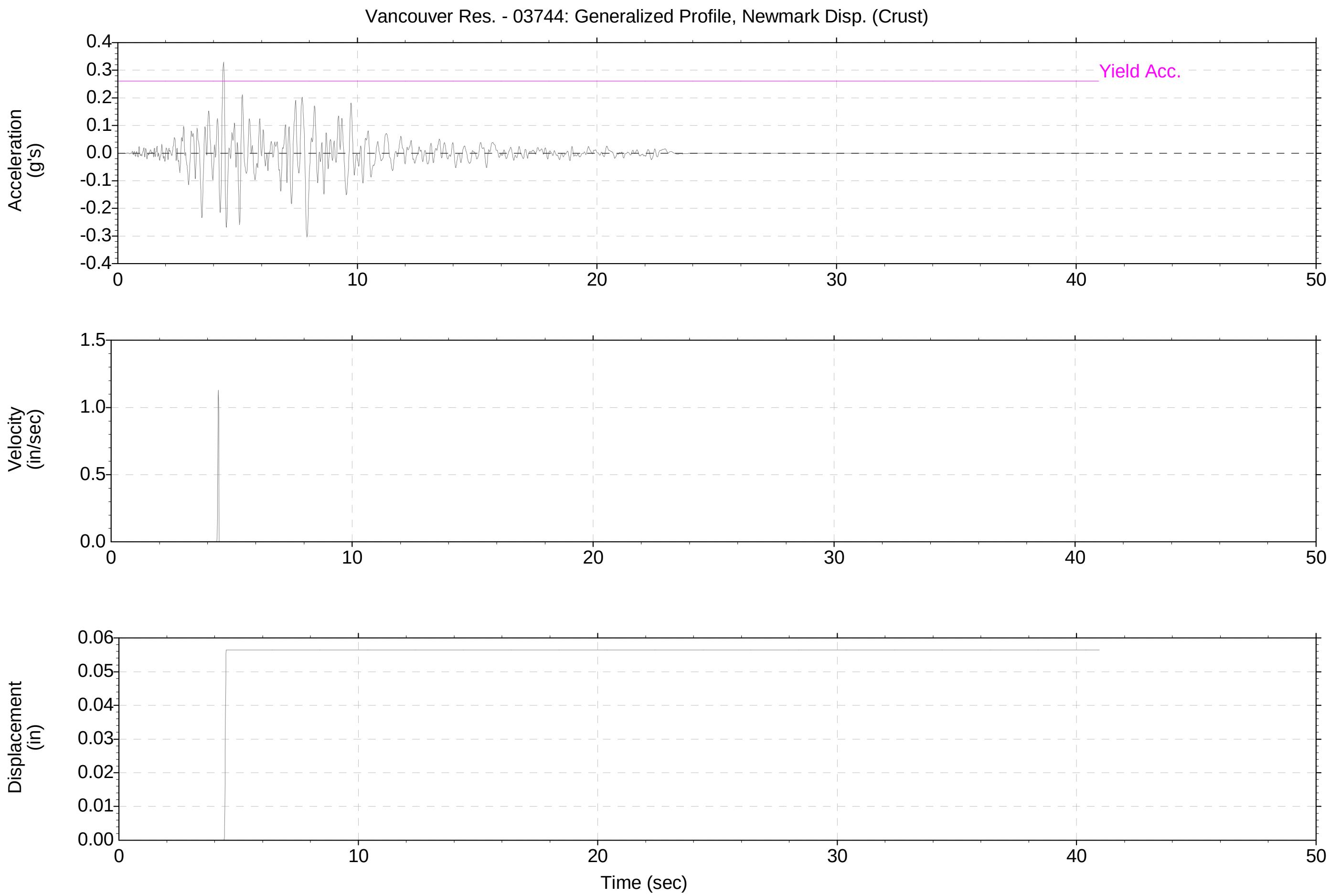


Notes:

Displacement Analysis - Newmark Method
Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

Newmark Method by Houston et.al. (1987)
Constant Yield Acceleration: .26 (g)
Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
Analysis: 7 - Soil Deposit: 4
Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A7D4-7-Generali-mul035.ahl
Peak Acceleration Value: .283499 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

Displacement computed: 5.477956E-03 in

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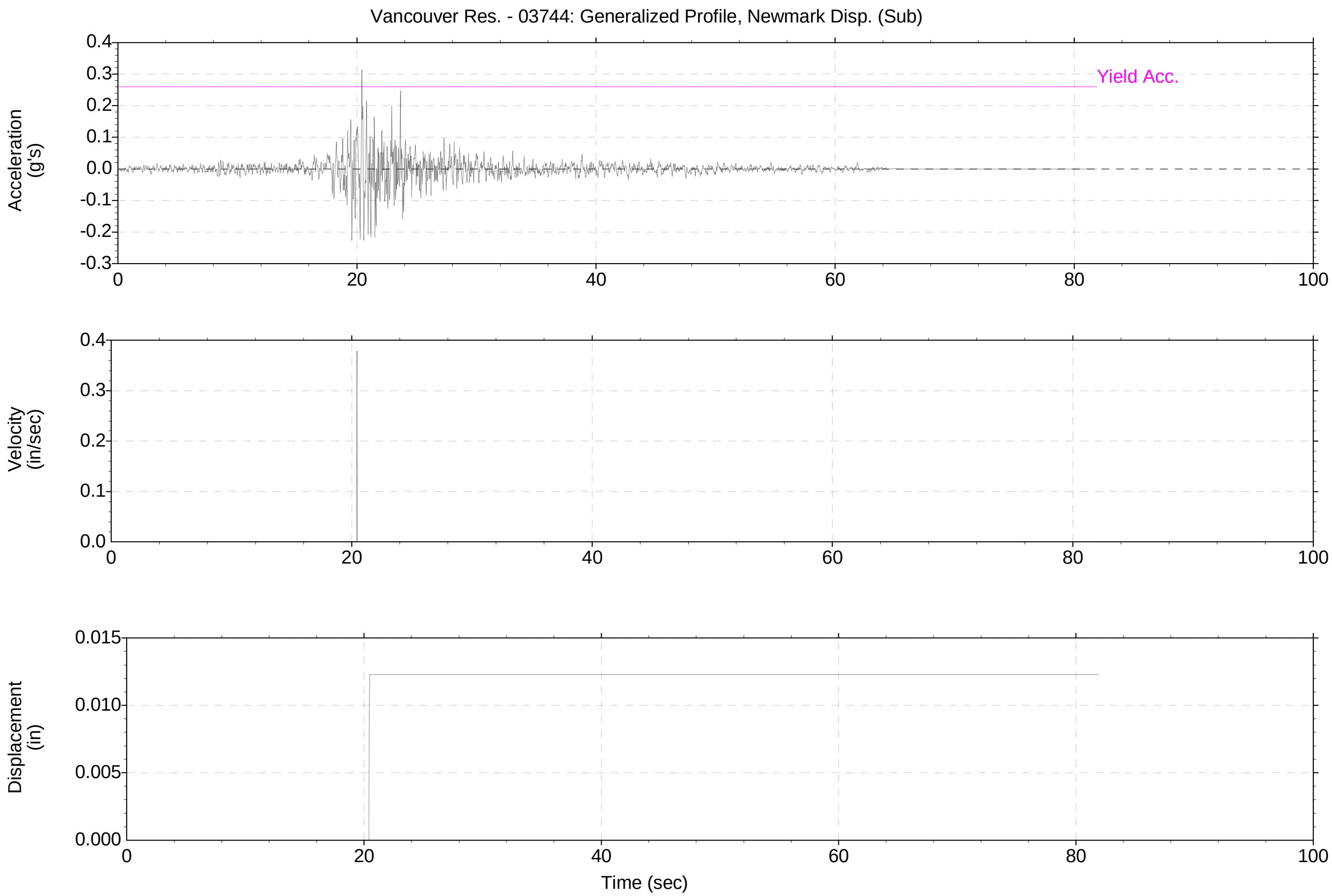
Notes:
     Displacement Analysis - Newmark Method
     Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Crust)

     Newmark Method by Houston et.al. (1987)
     Constant Yield Acceleration:      .26 (g)
     Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                           Analysis: 8 - Soil Deposit: 4
     Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A8D4-8-Generali-mul125.ahl
     Peak Acceleration Value:  .329091 (g)
     Upslope Movement not Included in Analysis
     Acceleration due to gravity:   386.4 (in/sec^2)

     Displacement computed: 5.642563E-02 in
```

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Notes:
     Displacement Analysis - Newmark Method
     Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)

     Newmark Method by Houston et.al. (1987)
     Constant Yield Acceleration:      .26 (g)
     Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                           Analysis: 11 - Soil Deposit: 4
     Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A11D4-11-Generali-arq008.ahl
     Peak Acceleration Value:  .313579 (g)
     Upslope Movement not Included in Analysis
     Acceleration due to gravity:   386.4 (in/sec^2)

     Displacement computed: 1.227993E-02 in
```
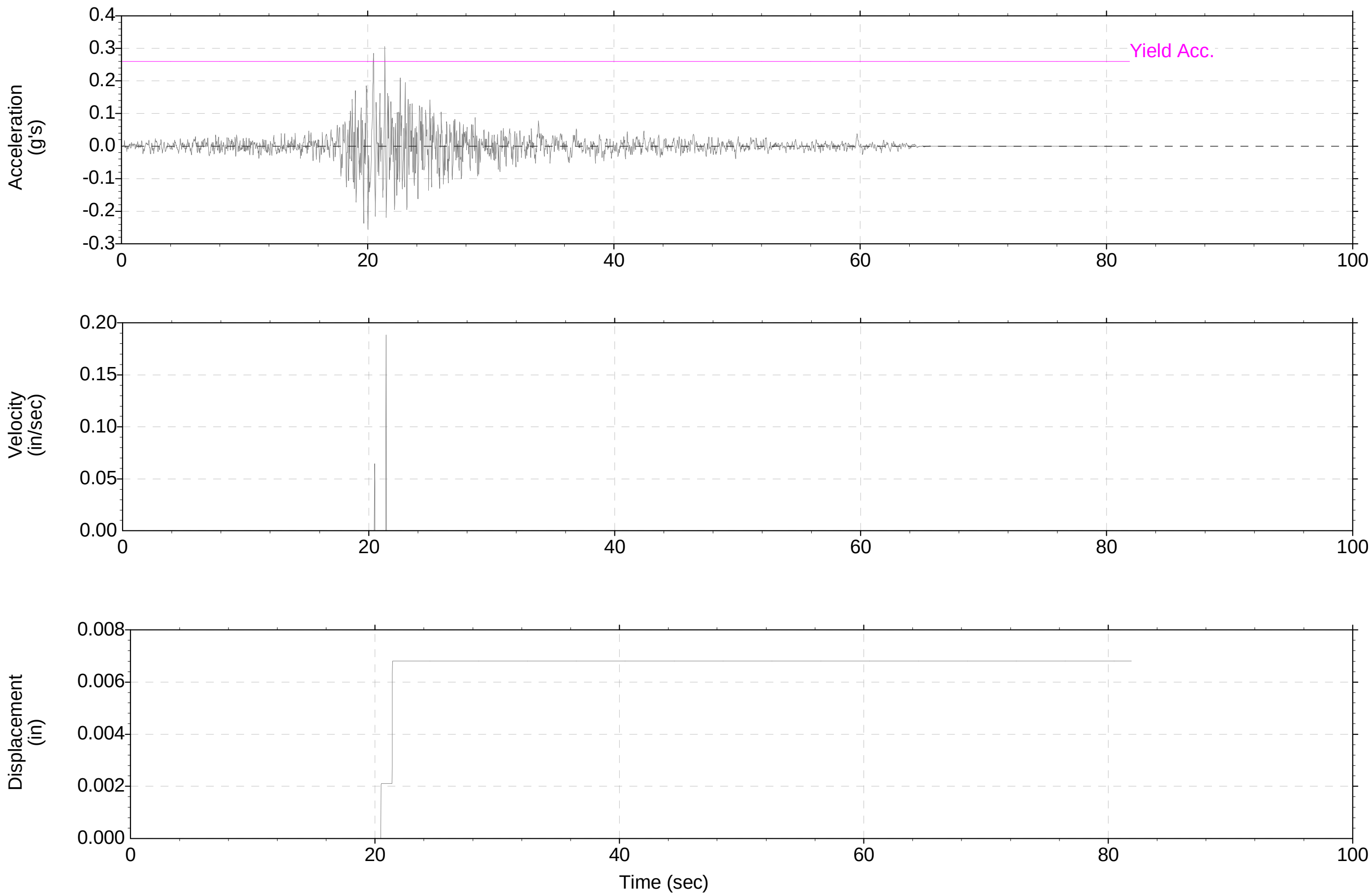

Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)



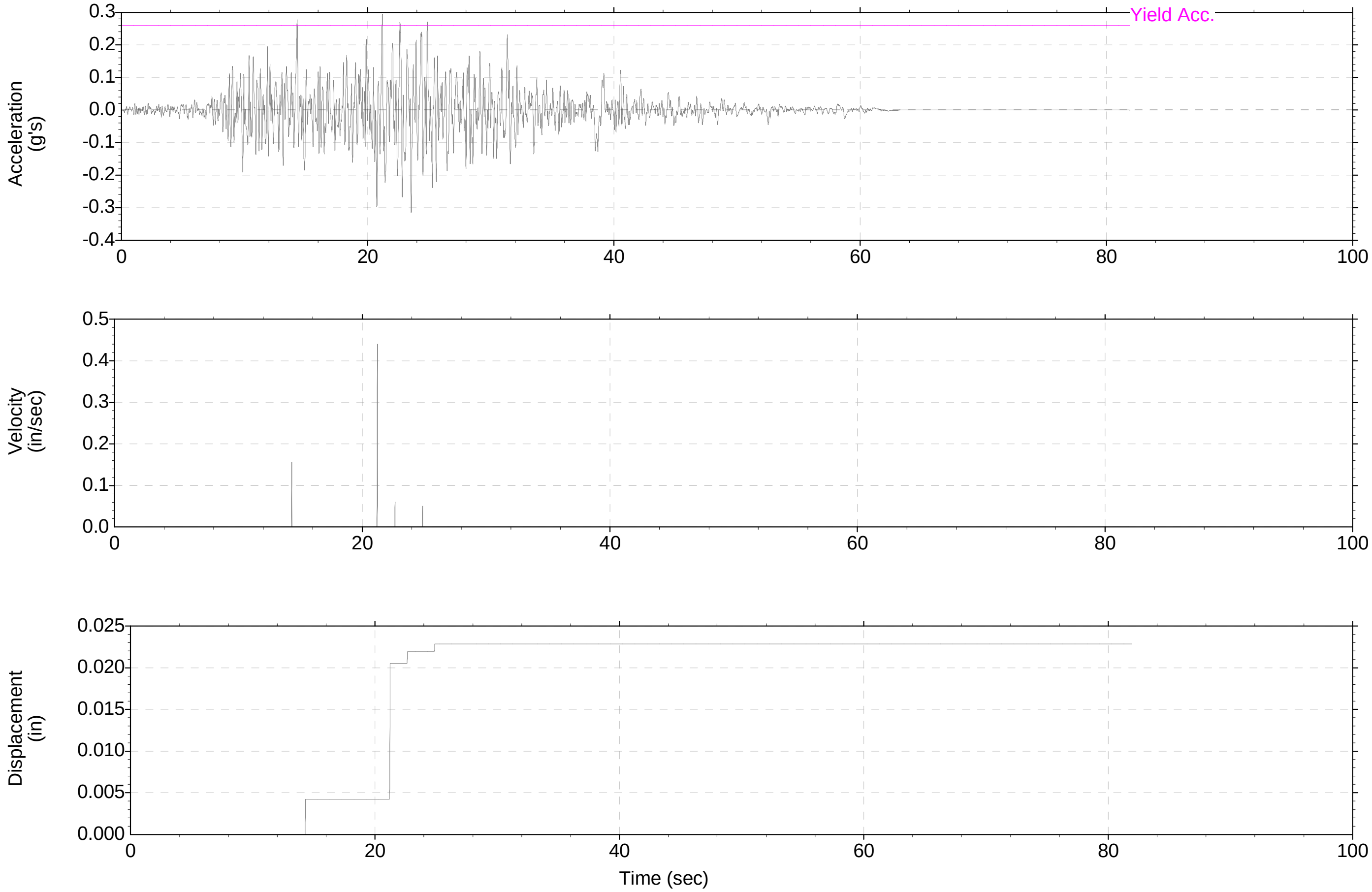
Notes:

Displacement Analysis - Newmark Method
Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)

Newmark Method by Houston et.al. (1987)
Constant Yield Acceleration: .26 (g)
Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 - Analysis: 12 - Soil Deposit: 4
Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A12D4-12-Generali-arq278.ahl
Peak Acceleration Value: .305744 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

Displacement computed: 6.802941E-03 in

Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)

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Notes:
     Displacement Analysis - Newmark Method
     Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)

     Newmark Method by Houston et.al. (1987)
     Constant Yield Acceleration:      .26 (g)
     Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 -
                               Analysis: 9 - Soil Deposit: 4
     Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A9D4-9-Generali-uni090.ahl
     Peak Acceleration Value:  .315126 (g)
     Upslope Movement not Included in Analysis
     Acceleration due to gravity:   386.4 (in/sec^2)

     Displacement computed: 2.283089E-02 in
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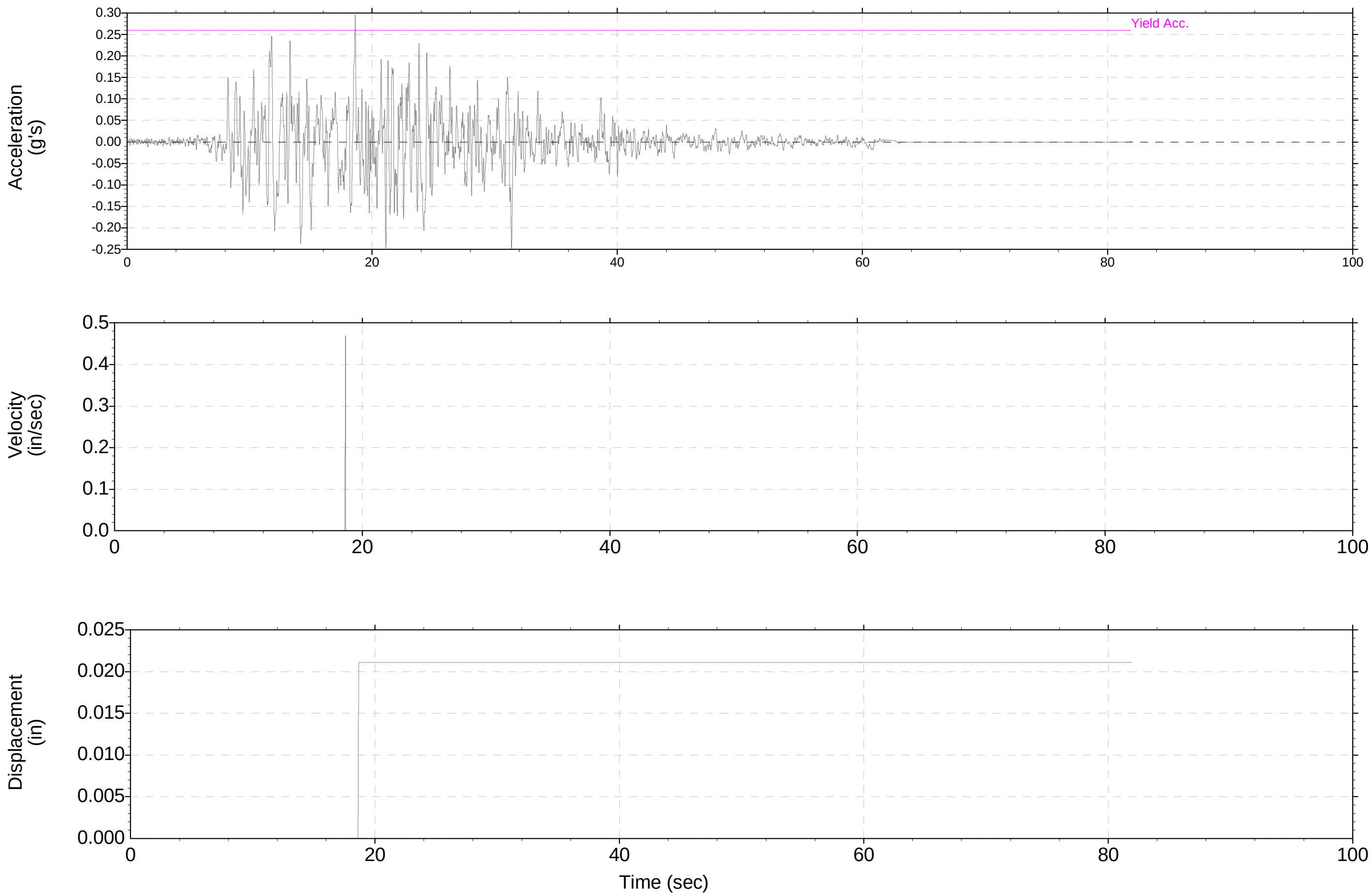
Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)



Notes:

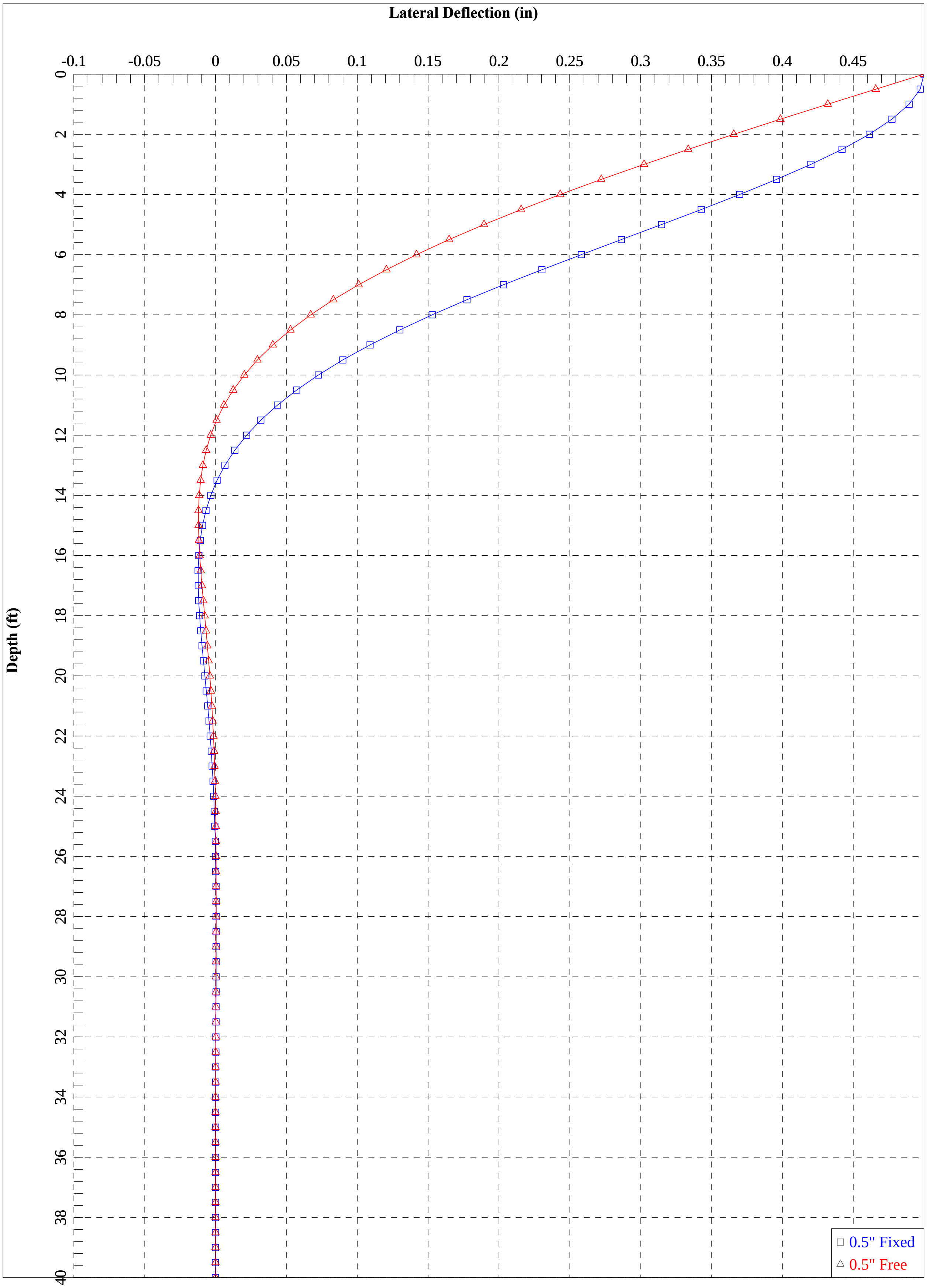
Displacement Analysis - Newmark Method
Project: Vancouver Res. - 03744: Generalized Profile, Newmark Disp. (Sub)

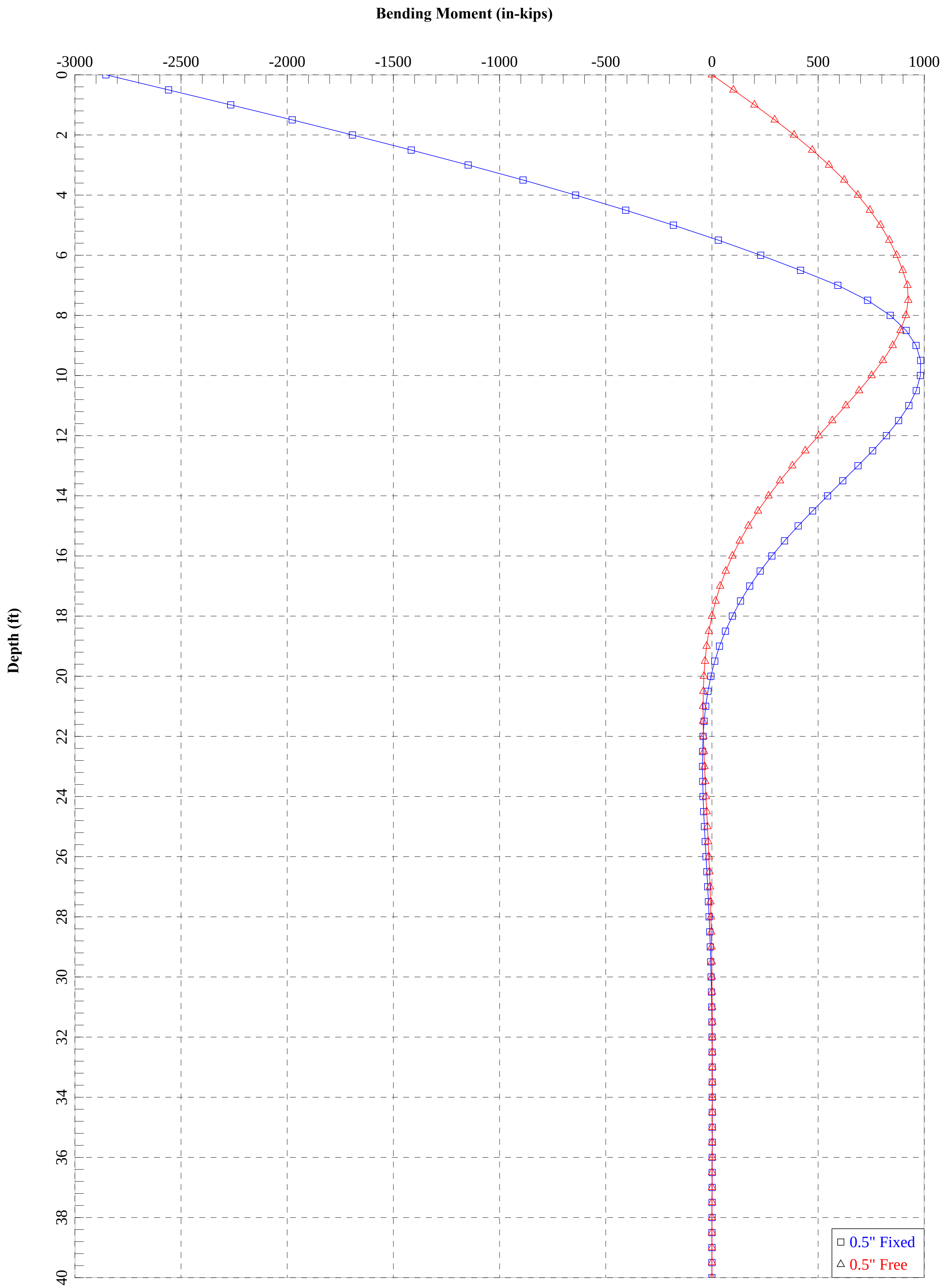
Newmark Method by Houston et.al. (1987)
Constant Yield Acceleration: .26 (g)
Acceleration Time History: Outcrop - Generalized - AHL - Layer: 1 - Analysis: 10 - Soil Deposit: 4
Acceleration Time History File: C:\Vanc\SHAKE\RESULTS\Generalized\Scaled\SHAKEout-L1A10D4-10-Generali-uni360.ahl
Peak Acceleration Value: .295892 (g)
Upslope Movement not Included in Analysis
Acceleration due to gravity: 386.4 (in/sec^2)

Displacement computed: 2.112842E-02 in

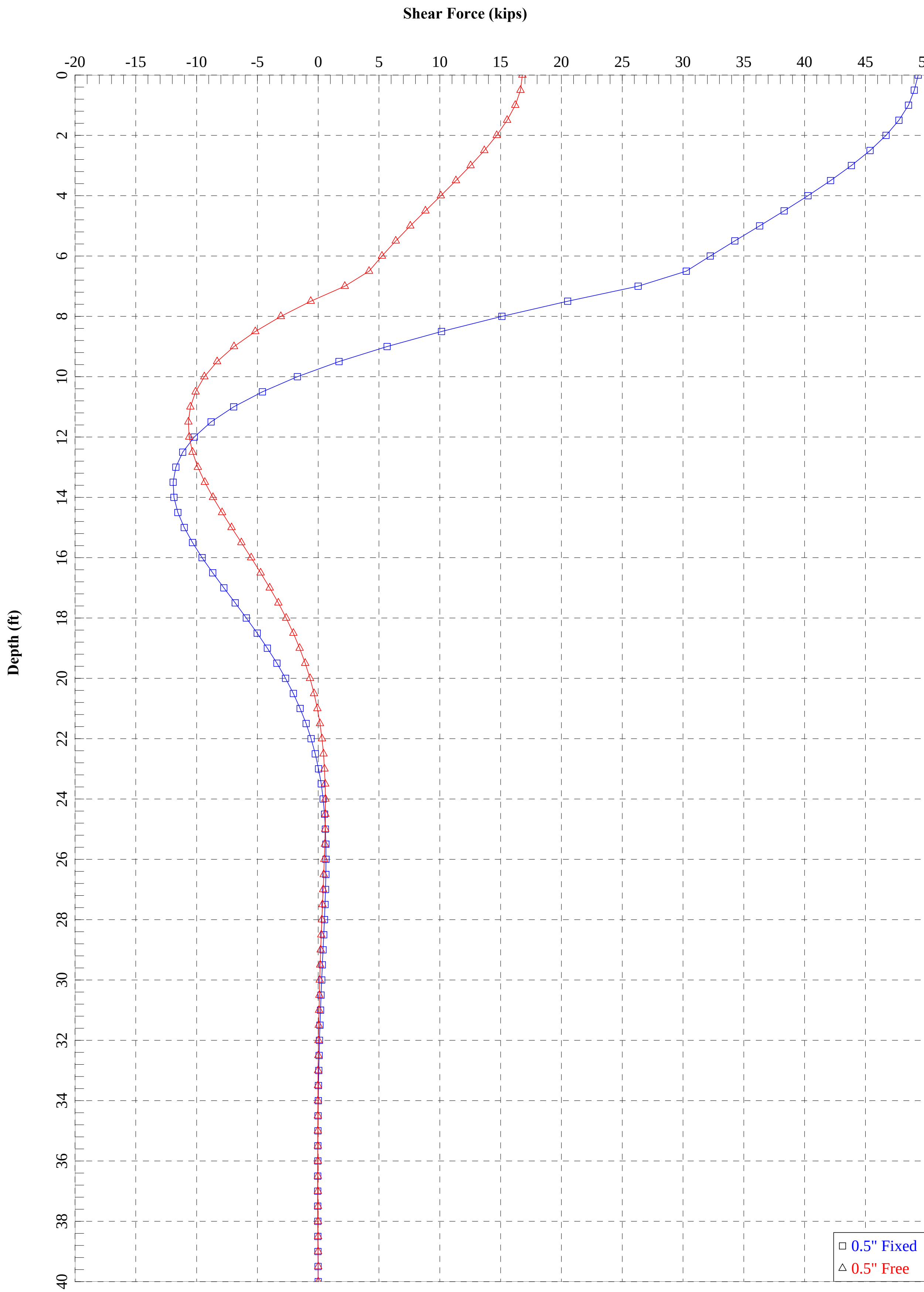
APPENDIX F

AUGER-CAST PILE LATERAL CAPACITY ANALYSES


Lateral Deflection (in)
-0.1
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
0.5" Fixed
0.5" Free



Bending Moment (in-kips)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
0.5" Fixed
0.5" Free



Shear Force (kips)
-20
-15
-10
-5
0
5
10
15
20
25
30
35
40
45
50
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
0.5" Fixed
0.5" Free

APPENDIX G

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Important Information About Your Geotechnical Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors, which were considered in the development of the report, have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based on interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland