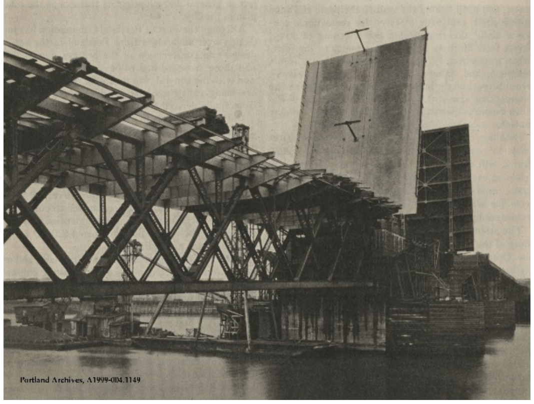

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GEOTECHNICAL REPORT

Burnside Bridge Environmental Impact Study

PORTLAND, OREGON

February 2021

Shannon & Wilson No: 102636-001

SHANNON & WILSON

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Submitted To: HDR, Inc.
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Attn: Steve Drahota, PE

Subject: GEOTECHNICAL REPORT, BURNSIDE BRIDGE ENVIRONMENTAL IMPACT STUDY, PORTLAND, OREGON

Shannon & Wilson, Inc. (Shannon & Wilson) prepared this report and participated in this project as a subconsultant to HDR Engineering, Inc (HDR). Our scope of services was specified in the Geotech Subconsultant Agreement with HDR dated January 24, 2019. This report presents geotechnical analysis and design services to support the Burnside Bridge National Environmental Policy Act (NEPA) and Type Selection Phase and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

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1 INTRODUCTION

1.1 Project Overview

This report presents the results of our geotechnical research, field explorations, laboratory testing, analyses, and design recommendations for the Multnomah County Burnside Bridge NEPA and Type Selection Phase in Portland, Oregon. The project is part of Multnomah County's larger effort to address the condition of its critical transportation infrastructure. After a review of the County's four downtown Portland bridges, it was determined the Burnside Bridge was a top priority due to its designation as the only Priority 1 lifeline route across the Willamette River in downtown Portland. The location of the bridge site is shown on the Vicinity Map, Figure 1.

As currently built, the bridge is not expected to withstand a major seismic event. Therefore, the County has taken on the responsibility to seek ways to improve the bridge in order to meet the region's needs for seismic resiliency. As part of the Burnside Bridge NEPA and Type Selection Phase, the County and their consulting team, led by HDR, will perform an environmental review in compliance with the NEPA of the alternatives presented in the Earthquake Ready Burnside Bridge Project (EQRB) Feasibility Study. The preferred alternative, as identified through the NEPA process, will be further developed to result in the bridge type selection. Shannon & Wilson, as a subconsultant to HDR, is providing geotechnical services to support the project.

We have prepared this geotechnical report in accordance with our scope of services for the project. We understand that the bridge will be evaluated in accordance with the following guidance documents:

- Burnside Bridge Earthquake Readiness Seismic Design Criteria – May 2017
- AASHTO LRFD Movable Highway Bridge Design Specifications – Second Edition (with Interim Revisions, 2015)
- AASHTO Guide Specifications for LRFD Seismic Bridge Design – Second Edition (with Interim Revisions, 2015)
- AASHTO LRFD Bridge Design Specifications – Seventh Edition, 2014 (with Interim Revisions, 2016)
- AASHTO LRFD Bridge Design Specifications – Eighth Edition, 2017
- ODOT Bridge Design Manual (BDM) – May 2019
- ODOT Geotechnical Design Manual (GDM) – December 2018

- FHWA-HRT-06-032 ~ Seismic Retrofitting Manual for Highway Structures: Part 1 – Bridges – January 2006

Geotechnical analyses and recommendations presented in this report expand on the preliminary geotechnical work performed during the EQRB Feasibility Study. The recommendations herein are based on the explored subsurface conditions and substructure components as depicted in the as-constructed plans provided by HDR, existing geotechnical borings at the site, and as encountered in the three borings we previously drilled at the site for this project.

1.2 Scope of Services

Shannon & Wilson's services were conducted in accordance with the Scope of Work defined in the Geotech Subconsultant Agreement with HDR, dated January 24, 2019, and our Master Subconsultant Agreement with HDR, dated October 20, 2014. The completed geotechnical design services for the project consisted of the following tasks:

- Provided a summary of existing geotechnical information to support roadway and bridge design tasks;
- Refine the geologic profile at the west abutment as needed;
- Evaluated foundation alternatives and proposed conceptual mitigation measures for geotechnical hazard impacts from each alternative;
- Developed conceptual mitigation alternatives for geotechnical hazards on both sides of the Willamette River;
- Performed refined analyses to update seismic and acceleration response spectrum (ARS) curves for the bridge seismic evaluations;
- Assessed geologic implications or impacts to each alternative; and
- Developed a revised Geotechnical Report based on our refined analyses.

2 PROJECT UNDERSTANDING

2.1 Site Description

The Burnside Bridge is located in the Portland central business district as shown on the Vicinity Map, Figure 1, and the Site and Exploration Plan, Figure 2. The bridge conveys Burnside Street across the Willamette River and connects 2nd Avenue on the west side of the river to Martin Luther King Jr. Boulevard (Highway 99E) on the east side of the river. The bridge consists of three major structures: the West Approach Bridge (ODOT Bridge No. 00511A), the Main Span River Bridge (ODOT Bridge No. 00511), and the East Approach

Bridge (ODOT Bridge No. 00511B). The West Approach consists of 19 reinforced concrete spans ranging in length from 22 to 62 feet with an overall bridge length of 604 feet and spans 1st Avenue, the TriMet MAX Blue/Red lines, Naito Parkway, and Tom McCall Waterfront Park. The Main Span consists of two 268-foot-long fixed steel spans flanking a 252-foot-long double leaf bascule draw span with an overall bridge length of 856 feet that spans the Willamette River and the Eastbank Esplanade. The East Approach consists of eight steel plate girder spans ranging in length from 75 to 106 feet and seven reinforced concrete spans ranging in length from 22 to 40 feet, with an overall bridge length of 849 feet. The East Approach spans Interstate 5 (I-5) and its associated ramps, the Union Pacific Railroad (UPRR), 2nd Avenue, and 3rd Avenue. The overall bridge structure is approximately 86 feet wide, aligned in a west-east direction, and accommodates five travel lanes (two westbound and three eastbound).

Embankment fills for both the west and east approaches are approximately 15 feet high and are retained by abutment walls at each approach. The Willamette River runs within a wide channel about 60 feet below the bridge in the vicinity of the Main Span Bridge crossing. The section of the riverbed beneath the bridge is typically at an elevation of about -40 to -60 feet (North American Vertical Datum of 1988 [NAVD88]). The west riverbank is retained by a pile-supported concrete retaining wall with a level fill surface at about elevation 35 feet behind the wall (Tom McCall Waterfront Park). The east riverbank slopes up at about 2 horizontal to 1 vertical (2H:1V) to an elevation of about 10 feet, east of which the topography has a gentle uphill slope.

2.2 Project Description

The purpose of the Burnside Bridge NEPA and Type Selection Phase is to perform an environmental review of the seismic retrofit and bridge replacement alternatives developed during the EQRB Feasibility Study, in accordance with the NEPA. We understand that a preferred alternative will be identified through the NEPA process. The preferred alternative will be further developed to result in the bridge type selection.

We understand the following four alternatives are being considered for bridge type selection:

1. Enhanced Seismic Retrofit (aka, Retrofit);
2. Replacement Alternative with Short-span Approach (aka, Short-span Alternative);
3. Replacement Alternative with Long-span Approach (aka, Long-span Alternative); and
4. Replacement Alternative with Couch Extension (aka, Couch Extension).

Based on current design plans, the Short-span Alternative and Couch Extension will each include 14 bents along the existing Burnside Street alignment. We understand that the proposed span lengths of the two short-span approach replacement alternatives are the same along E Burnside Street; however, the east approach of the Couch Extension splits into one-way connections on E Burnside Street and NE Couch Street. The north branch of the Couch Extension will include an additional six bents along the connection to NE Couch Street. The Long-span Alternative includes 10 bents along the existing bridge alignment. We further understand that each of the three bridge replacement alternatives will be supported on a drilled shaft foundation system.

Conceptual seismic ground improvement design recommendations for the retrofit and replacement options are presented in Section 10. Foundation resistance and stiffness parameters for the preferred retrofit alternative are presented in Section 11, and design parameters for the replacement alternatives are presented in Section 12.

The project scope of services specifies two earthquake ground motion performance levels for evaluation and retrofit or replacement of the bridge: a “Full Operation” Performance Level (referred to as "Operational" in the ODOT BDM and GDM) for CSZ event ground motions and a “Limited Operation” Performance Level (ground motion level referred to as "Life Safety" in the ODOT BDM and GMD and referred to as "Limited Operation") for probabilistic 1,000-year return period ground motions.

3 EXISTING FOUNDATION SYSTEM

Based on As-Constructed Drawing No. T2, the existing bridge was originally constructed in the mid-1920s, replacing an earlier bridge built in 1894. This drawing is included in Appendix A, Existing Information. Preliminary ground surface and subsurface information was taken from the As-Constructed Record of Borings, dated 1924 (drawing included in Appendix A). Foundation configurations were taken from As-Constructed Drawing Nos. 7, T8, T10, T16, 18, and 48, dated February 1924, As-Constructed Drawing No. L-75 dated April 1925, and the Foundation Piling Summary (all drawings and piling summary included in Appendix A). All as-constructed drawings were prepared by Hedrick & Kremers Consulting Engineers.

According to the drawings provided by HDR, the Burnside Bridge has 37 spans supported by 34 bents and four piers. The bents supporting the West Approach Bridge are designated Bent 1 through Bent 19, the piers supporting the Main Span Bridge are designated Pier 1 through Pier 4, and the bents supporting the East Approach Bridge are designated Bent 21 through Bent 35. The west abutment of the West Approach Bridge is designated Bent 1, and the east abutment of the East Approach Bridge is designated Bent 35. The west abutment of the Main Span Bridge is designated Pier 1, and the east abutment of the Main Span Bridge is designated Pier 4. The overcrossing configuration is shown on As-Constructed Drawing No. T2.

Bents 1 and 35 are supported on abutment walls with a continuous footing. Bents 2 through 17 and Bents 28 through 34 are supported on spread footings. Based on our review of the provided drawings, we developed Exhibit 3-1, which provides a summary of the existing footing dimensions, number of footings at each bent, footing embedment and elevations, and bearing material. The design bearing pressures for the footings are not indicated on the plans. The spread footing foundation configurations are also shown on the drawings included in Appendix A.

Bents 18 and 19, Piers 1 through 4, and Bents 21 through 27 are supported on driven timber piles. Based on our review of the provided drawings and foundation piling summary, we developed Exhibit 3-2, which provides a summary of the existing pile cap dimensions, number of piles at each bent or pier, pile type and section, pile length and tip elevations, and bearing material. The required pile bearing capacities and pile diameters are not indicated on the plans. A 16-inch pile diameter (butt diameter) is assumed based on typical timber pile sections available at the time the bridge was constructed. The driven pile foundation configurations are also shown on the drawings included in Appendix A.

The bearing materials for the spread footings and driven piles are not clearly defined in the as-constructed drawings and are interpreted based on information in the drawings and existing subsurface explorations at the site, as well as our subsurface explorations. In addition, elevations obtained from the as-constructed drawings were converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the drawings.

Exhibit 3-1: As-Constructed Foundation Summary of Spread Footings

	Number of Footings	Footing Dimensions (W x L x H) (ft)	[1]Approximate Bottom of Footing Elevation (ft)	Approximate Footing Embedment (ft)	[2]Bearing Material
Bent 1	1	10’ x 110’	24.5	5	Fine-Grained Alluvium
Bent 2	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 3	4	Exterior: 6.5’x 6.5’ x 3’ Int. North: 8’ x 8’ x 8’ Int. South: 7.5’ x 7.5’ x 3’	Exterior: 22 Interior North: 17 Interior South: 22	7	Fine-Grained Alluvium
Bent 4	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 5	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 6	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 7	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 8	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 9	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 10	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 11	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 12	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 13	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	22	7	Fine-Grained Alluvium
Bent 14	4	Exterior: 8’ x 8’ x 3’ Interior: 11.5’ x 11.5’ x 4.5’	22	9	Fine-Grained Alluvium
Bent 15	4	Exterior: 8’ x 8’ x 3’ Interior: 11.5’ x 11.5’ x 4.5’	22	9	Fine-Grained Alluvium
Bent 16	4	Exterior: 8’ x 8’ x 3’ Interior: 11.5’ x 11.5’ x 4.5’	22	9	Fill
Bent 17	4	Exterior: 14’x 14’ x 5’ Interior: 16.5’ x 16.5’ x 5’	Exterior: 12 Interior North: 14 Interior South: 12	18	Fill
Bent 28	3	16’ x 16’ x 4’	22	27	Fine-Grained Alluvium
Bent 29	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	40	10	Fill
Bent 30	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	40	10	Fill
Bent 31	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	40	10	Fill
Bent 32	4	Exterior: 6.5’ x 6.5’ x 3’ Interior: 7.5’ x 7.5’ x 3’	40	10	Fill
Bent 33	4	Exterior: 8’ x 8’ x 3’ Interior: 11.5’ x 11.5’ x 4.5’	37	12	Fill
Bent 34	4	Exterior: 8’ x 8’ x 3’ Interior: 11.5’ x 11.5’ x 4.5’	37	12	Fill
Bent 35	1	9.25’ x 110’	41	9	CFD – Channel Facies

NOTES:

1 Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set.

2 Bearing material is interpreted from the information in the plan set, existing borings, and current borings.

Exhibit 3-2: As-Constructed Foundation Summary for Driven Piles

Location	Number of Piles	[a]Pile Cap Dimensions (W x L x H) (ft)	[b]Pile Type and Section	[c]Approximate Bottom Pile Cap Elevation (ft)	[c]Approximate Pile Tip Elevation (ft)	Approximate Pile Length (ft)	[d]Bearing Material
Bent 18N	68	19’ x 28’ x 6’	16-inch dia. Timber	9	-2.8	11.8	Sand/Silt Alluvium
Bent 18S	71	19’ x 28’ x 6’	16-inch dia. Timber	9	-1.7	10.7	Sand/Silt Alluvium
Bent 19N	59	19’ x 28’ x 6’	16-inch dia. Timber	7	-35.5	42.5	Sand Alluvium
Bent 19S	50	19’ x 28’ x 6’	16-inch dia. Timber	7	-22.6	29.6	Sand Alluvium
Pier 1	276	33’ x 71’ x 21.7’	16-inch dia. Timber	-41.6	-72.4	30.8	Sand Alluvium
Pier 2	382	68’ x 78’ x 37’	16-inch dia. Timber	-70	-94.2	24.2	Sand/Silt Alluvium
Pier 3	392	68’ x 78’ x 37’	16-inch dia. Timber	-68.6	-92.6	24	Sand Alluvium
Pier 4	277	36’ x 68’ x 21.5’	16-inch dia. Timber	-40.3	-70.7	30.4	Sand Alluvium
Bent 21N	63	24’ x 24’ x 10.5’	16-inch dia. Timber	2	-67.2	69.2	Fine-Grained Alluvium
Bent 21S	63	24’ x 24’ x 10.5’	16-inch dia. Timber	2	-76.4	78.4	Fine-Grained Alluvium
Bent 22N	61	24’ x 24’ x 10.5’	16-inch dia. Timber	2	-58.8	60.8	Fine-Grained Alluvium
Bent 22S	63	24’ x 24’ x 10.5’	16-inch dia. Timber	2	-59.2	61.2	Fine-Grained Alluvium
Bent 23N	62	24’ x 24’ x 10.5’	16-inch dia. Timber	2	-54.5	56.5	Sand/Silt Alluvium
Bent 23S	64	24’ x 24’ x 10.5’	16-inch dia. Timber	2	-58.7	60.7	Sand/Silt Alluvium
Bent 24N	72	24’ x 27’ x 10.5’	16-inch dia. Timber	7	-53.2	60.2	Sand/Silt Alluvium
Bent 24S	72	24’ x 27’ x 10.5’	16-inch dia. Timber	7	-51.7	58.7	Sand/Silt Alluvium
Bent 25N	77	27’ x 27’ x 10.5’	16-inch dia. Timber	10	-57.7	67.7	Sand/Silt Alluvium
Bent 25S	79	27’ x 27’ x 10.5’	16-inch dia. Timber	10	-54.7	64.7	Sand/Silt Alluvium
Bent 26N	70	24’ x 27’ x 10.5’	16-inch dia. Timber	10	-59	69	Sand/Silt Alluvium
Bent 26S	68	24’ x 27’ x 10.5’	16-inch dia. Timber	10	-54.3	64.3	Sand/Silt Alluvium
Bent 27N	63	24’ x 24’ x 10.5’	16-inch dia. Timber	10	-49.5	59.5	Sand/Silt Alluvium
Bent 27C	25	15’ x 15’ x 8’	16-inch dia. Timber	12.6	-47.4	60	Sand/Silt Alluvium
Bent 27S	64	24’ x 24’ x 10.5’	16-inch dia. Timber	10	-50.9	60.9	Sand/Silt Alluvium

Notes:

a. W = Pile cap dimension in longitudinal direction (perpendicular to bent/pier centerline), L = Pile cap dimension in transverse direction (parallel to bent/pier centerline)

b. Pile type and section are not shown in the plans, therefore pile type and section is assumed.

c. Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set.

d. Bearing material is interpreted from the information in the plan set, existing borings, and current borings.

4 REGIONAL GEOLOGY AND SEISMIC SETTING

4.1 Regional Geology

The greater Portland metropolitan area lies within the Portland Basin, a structural depression created by complex folding and faulting of the basement rocks. This Portland Basin is approximately 40 miles long and 20 miles wide, with the long axis trending to the northwest. The most prevalent basement rock of the Portland Basin is a sequence of lava flows of the Columbia River Basalt Group (CRBG), which flowed into the area between about 17 million and 6 million years ago (Beeson and others, 1991).

The Columbia and Willamette Rivers converge within the Portland Basin and, with their tributaries, have contributed to extensive sedimentary deposits which overly the basement rock formations. The Burnside Bridge lies within the Portland Quadrangle, where Beeson and others (1991) have mapped the Portland Basin sediments as Sandy River Mudstone (SRM), overlain by Troutdale Formation. According to Beeson and others (1991), the SRM locally consists of between 200 to 300 feet of claystone, siltstone, and sandstone beds deposited in the Miocene to Pliocene epochs (about 10 million to 3.5 million years ago), and the Troutdale Formation locally consists of about 100 to 400 feet of well-consolidated friable to moderately well-cemented conglomerate and sandstone, also deposited in the Miocene to Pliocene epochs (about 12.5 million to 1.6 million years ago).

The SRM and Troutdale Formation are locally overlain in places by a sequence of catastrophic flood deposits. During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which then formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, which allowed the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley and the lake refilled, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). These repeated floods are collectively referred to as the Missoula Floods.

During each short-lived Missoula Flood episode, floodwaters washed across the Idaho panhandle, through eastern Washington's scablands, and through the Columbia River Gorge. When the floodwater emerged from the western end of the gorge, it spread out over the Portland Basin and pooled to elevations of about 400 feet, depositing a tremendous load of sediment. Boulders, cobbles, and gravel were deposited nearest the mouth of the gorge and along the main channel of the Columbia River. Cobble-gravel bars reached westward across the basin, grading to thick blankets of micaceous sand and silt (Allen and others, 2009). Beeson and others (1991) divided the flood deposits into three facies: Fine-grained facies, Coarse-grained facies, and Channel facies. The Fine-grained facies consists of coarse sand to silt. The Coarse-grained facies consists of gravel, cobbles, and boulders in a sand and silt matrix. The Channel facies consists of complexly interlayered fine and coarse-grained material formed by channeling of flood deposits into earlier and/or contemporaneous deposits.

Irregular post-flood surfaces were filled in locally by pond or bog deposits and overbank alluvium. In historic times, many areas have also been altered by grading, cuts, and fills made by humans. Generalized surficial geology along the project alignment, as compiled from multiple sources by the Oregon Department of Geology and Mineral Industries (DOGAMI), is shown in Published Geologic Mapping, Figure 3.

4.2 Seismic Setting

The contemporary tectonics and seismicity of the region are the result of oblique, northeastward subduction at a rate of about 37 millimeters per year (mm/yr) (DeMets and others, 2010) of the Juan de Fuca oceanic plate beneath the North American continental plate (e.g., Wells and others, 1998; Wells and Simpson, 2001). This complex tectonic setting produces east-west compressive strain along the Cascadia Subduction Zone (CSZ), as well as northward translation and rotation of the mobile, crustal, Cascadia forearc blocks that span the leading edge of the North America plate (Wells and others, 1998; McCaffrey and others, 2007, 2013). Rotation of the Sierra-Nevada block and expansion of the Basin and Range drive the northward migration and clockwise rotation of the Cascadia forearc blocks (e.g., Pezzopane and Weldon, 1993; Wells and others, 1998; Wells and Simpson, 2001). As a result, the southern portion of the forearc, the Oregon Coast block, is impinging on western Washington at a rate of about 8 to 12 mm/yr causing crustal shortening in northwest Oregon and western Washington (Wells and others, 1998; Wells and Simpson, 2001; Mazzotti and others, 2002).

The combined effect of margin-normal subduction and margin-parallel shortening produces complex and diverse deformation within the northern edge of the Cascadia forearc and triggers large (greater than magnitude [M] 6), damaging earthquakes from three seismogenic source zones:

- The locked zone of the CSZ fault interface, which produces great mega-thrust earthquakes;
- The deep intraslab portion of the CSZ (i.e., the subducted portion of the Juan de Fuca Plate), the source off Wadati-Benioff zone earthquakes; and
- The overriding North American Plate, where shallow crustal faults rupture.

All three sources potentially produce earthquakes that impact the ground motion hazards at the project site. Offshore, elastic release of strain accumulated in the locked plate interface of the CSZ produces great megathrust earthquakes (greater than M 8.0) about every 500 years (Atwater and Hemphill-Haley, 1997; Clague, 1997; Goldfinger and others, 2003 and 2012); the most recent rupture occurred in A.D. 1700 (Satake and others, 1996; Atwater and Hemphill-Haley, 1997; Clague, 1997; Yamaguchi and others, 1997; Goldfinger and others, 2003 and 2012). Onshore, migration and rotation of tectonic blocks produce deformation along shallow faults within the upper part of the crust. At depth, rupture within the subducting slab, referred to as the intraslab, has produced some of the largest recorded earthquakes (M 6.5 to 7) to strike the Pacific Northwest in the northern California Coast region and Western Washington. However, over the past century, intraslab earthquakes have been markedly infrequent in Oregon. The following sections briefly describe the location, characteristics, and seismicity of each of the sources.

4.2.1 Cascadia Subduction Zone: Mega-Thrust Interface Source

CSZ mega-thrust earthquakes originate along the interface between the subducting oceanic plates and the North American plate. Because of the significant uncertainty of the landward extent of a potential rupture surface, estimates of the closest distance between the project and potential rupture surface range from about 65 to 140 horizontal miles. Focal depths for mega-thrust earthquakes are commonly on the order of about 15 to 25 miles. Rupture of the interface could result in earthquakes with moment magnitudes on the order of 8.5 to over 9.0, with strong shaking that lasts for several minutes. No large earthquakes have occurred in this zone during historic times (the last 170 years). However, geologic evidence suggests that coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years (e.g., Atwater, 1987; Atwater and Hemphill-Haley, 1997) as a result of tectonic movement associated with mega-thrust earthquakes on the CSZ. It appears that ruptures of this zone have occurred at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997). Based on historical tsunami records in Japan (Satake and others, 1996) the most recent interplate event on the CSZ was a moment magnitude (Mw) 9 event on January 26, 1700.

4.2.2 Cascadia Subduction Zone: Intraslab Source

CSZ intraslab earthquakes originate from within the subducting oceanic plates as a result of down-dip tensional forces and bending caused by mineralogical and density changes in the plates at depth. These earthquakes typically occur 28 to 37 miles beneath the surface. The nearest seismogenic intraslab portion of the Juan de Fuca plate is approximately 30 to 60 miles below the Portland area. Ludwin and others (1991) estimate that the maximum Mw from this source zone would be about 7.5. Ground shaking produced by intraplate earthquakes would be less intense and less prolonged in the Portland area than ground motions generated by large subduction zone interface earthquake events. Historic seismicity from this source zone includes the 1949 Mw 6.7 Olympia earthquake, the 1965 Mw 6.7 earthquake between Tacoma and Seattle, and the 2001 Mw 6.8 Nisqually earthquake. While intraslab events have occurred frequently in the Puget Sound area, they are historically rare in Oregon.

4.2.3 Shallow Crustal Source

Shallow crustal earthquakes within the North American Plate have historically occurred in a diffuse pattern within Pacific Northwest, typically within the upper 4 to 19 miles of the continental crust. Mabey and others (1993) concluded from their analysis of local geologic features that a crustal earthquake of up to Mw 6.5 could occur virtually anywhere in the Portland area. Based on their fault model, Wong and others (2000) determined that an earthquake of up to Mw 6.8 is possible on the Portland Hills Fault, which is mapped within about one half-mile of the project site. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake at approximate Mw 6.5 to 7.0. Other examples include the 1993 Mw 5.6 Scotts Mill earthquake and the 1993 Mw 6.0 Klamath Falls earthquake.

Shallow crustal faults and folds throughout Oregon and Washington have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database. The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold database (USGS, 2017), there are 12 Class A features within approximately 30 miles of the project site. Their names, general locations relative to the site, and the time since their most recent deformation are summarized in Exhibit 4-1. The CSZ itself is approximately 135 miles west of the project site, with an average slip rate of approximately 40 millimeters (1.5 inches) per year and the most recent deformation occurring about 300 years ago (Personius and Nelson, 2006).

Exhibit 4-1: USGS Class A Faults Within an Approximate 30-Mile Radius of the Project Site

Fault Name	USGS Fault Number	Approximate Length	Approximate Distance and Direction from Project Site[1]	Slip Rate Category[2]	Time Since Last Deformation[3]
Portland Hills Fault	877	30.4 miles	0.5 miles W	< 0.2 mm/yr	< 15 ka
East Bank Fault	876	18.0 miles	0.6 miles NE	< 0.2 mm/yr	< 15 ka
Oatfield Fault	875	18.0 miles	3.1 miles SW	< 0.2 mm/yr	< 1.6 Ma
Grant Butte Fault	878	6.2 miles	6.1 miles SE	< 0.2 mm/yr	< 750 ka
Damascus-Tickle Creek Fault	879	9.9 miles	6.3 miles SE	< 0.2 mm/yr	< 750 ka
Beaverton Fault Zone	715	9.3 miles	7.0 miles SW	< 0.2 mm/yr	< 750 ka
Canby-Molalla Fault	716	31.1 miles	8.5 miles SW	< 0.2 mm/yr	< 15 ka
Helvetia Fault	714	4.3 miles	12.0 miles NW	< 0.2 mm/yr	< 1.6 Ma
Lacamas Lake Fault	880	14.9 miles	12.9 miles NE	< 0.2 mm/yr	< 750 ka
Newberg Fault	717	3.1 miles	21.3 miles SW	< 0.2 mm/yr	< 1.6 Ma
Gales Creek Fault Zone	718	45.4 miles	22.5 miles W-SW	< 0.2 mm/yr	< 1.6 Ma
Mount Angel Fault	873	18.6 miles	26.8 miles SW	< 0.2 mm/yr	< 15 ka

NOTES:

1 Approximate distance between project site and nearest extent of fault mapped at the ground surface.
2 mm = millimeters; yr = year.
3 Ma = “Mega-annum” or million years ago; ka = “Kilo-annum” or one thousand years ago.

5 RECENT FIELD EXPLORATIONS

5.1 Historic Geotechnical Data

Numerous geotechnical borings were previously drilled at and around the project site by other geotechnical firms or agencies, both for the Burnside Bridge and for various unrelated projects including the Banfield Access Ramp, Ankeny Pump Station, West and East Side Combined Sewer Overflow (CSO) Projects, and borings for the Portland Development Commission. Approximate locations of the relevant historic borings are shown on the Site and Exploration Plan, Figure 2. Logs of the relevant historic borings are provided in Appendix A, Historic Information. While the borings performed by Shannon & Wilson for this project were logged in accordance with the ODOT Soil and Rock Classification Manual, the borings presented in Appendix A, which were logged by others, may use other descriptive methodologies.

5.2 Recent Geotechnical Explorations

Shannon & Wilson did not perform field explorations during the NEPA phase of the project. In the previous phase of the project, Shannon & Wilson performed subsurface explored subsurface conditions at project site with three geotechnical borings, designated B-1 through B-3. Borings B-1 and B-3 were drilled on land and were advanced to depths of 221.5 and 230.3 feet below the existing ground surface, respectively. Boring B-2 was drilled in the Willamette River from a floating barge and was advanced to a depth of 148.2 feet below mudline. The borings were drilled between September 19, and October 25, 2016. Completed borehole locations were measured in the field relative to existing site features and with a hand-held GPS unit (Geo 7X H-Star) capable of decimeter-level accuracy. Approximate borehole locations are shown graphically on the Site and Exploration Plan, Figure 2. At the initial location of boring B-2, designated on Figure 2 as B-2A, we encountered concrete and metal debris that resulted in extreme mud loss and practical drilling refusal at a depth of approximately 8 feet below the mudline. Boring B-2 was then moved approximately 28 feet south and 7 feet west of B-2A, where it was drilled to its ultimate depth of 148.2 feet below mudline. Details of drilling, sampling procedures, and our logs of the materials encountered in the explorations are presented in Appendix B, Drilling Explorations. All borings included in situ geophysical testing (OYO Suspension Logging), which is discussed and presented in Appendix C, In Situ Geophysical Tests.

6 RECENT LABORATORY TESTING

In the previous phase of the project, Shannon & Wilson performed laboratory tests for the soil samples obtained in the recent geotechnical explorations. The testing program included Atterberg limits tests and particle-size analyses. Atterberg limits tests and particle size analyses were completed by Northwest Testing, Inc., of Wilsonville, Oregon, and all test procedures were performed in accordance with applicable ASTM International standards. Results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix D, Laboratory Test Results.

7 SUMMARY OF SUBSURFACE CONDITIONS

7.1 Geotechnical Soil Units

We grouped the materials encountered in our field explorations and in the historic borings into 10 geotechnical units. Our interpretation of the subsurface conditions is based on the explorations and regional geologic information from published sources. The geotechnical units are as follows:

- **Fill:** highly variable mixtures of gravel, sand, silt, and clay that may include wood debris, concrete debris, brick fragments, glass, and other man-made materials;
- **Fine-Grained Alluvium:** very soft to medium stiff (less commonly stiff to very stiff) Silt and Clay with varying amounts of sand, typically less than 30 percent (ML and CL);
- **Sand/Silt Alluvium:** very loose grading with depth to dense/very soft grading with depth to stiff, Silty Sand (SM) and Sandy Silt (ML); trace gravel, trace silt/clay interbeds, and trace organics;
- **Sand Alluvium:** loose to medium dense, occasionally dense to very dense, Sand to Gravelly Sand with varying amounts of silt (SP, SP-SM); lesser amounts of Silty Sand (SM); some zones contain organics and wood debris;
- **Gravel Alluvium:** medium dense to very dense Gravel with varying amounts of sand and fines (GP, GW, GP-GM, GW-GM, and GM); includes zones with cobbles and possible boulders; trace lenses of sand and silt;
- **Catastrophic Flood Deposits - Fine-Grained Facies:** stiff to very stiff Silt (ML);
- **Catastrophic Flood Deposits - Channel Facies:** dense to very dense interbedded Sand and Gravel deposits with varying amounts of fines (GW, GW-GM, GP-GM, SW-SM, SP, and SP-SM); lesser layers of stiff Sandy Silt (ML); includes zones with cobbles and possible boulders;
- **Upper Troutdale Formation:** dense to very dense Sand and Gravel deposits with varying fines content, interbedded with hard Silt and Clay deposits containing varying amounts of sand (GP, GW, GP-GC, SP, SC, ML, MH, CL, and CH); some zones of cementation;
- **Lower Troutdale Formation:** very dense Gravel with varying amounts of sand and fines (GP, GW, GP-GM, GW-GM, GP-GC, GM, and GC); trace sand and fine-grained layers were also encountered (SP, SP-SM, SM, CL, CH); some zones of cementation; cobbles are likely present in some areas;
- **Sandy River Mudstone:** hard Clay with varying amounts of sand interbedded with very dense Sand that contains varying amounts of fines (CL, CH, CL-ML, SM, and, to a lesser extent, ML, SP-SM, and SP).

These geotechnical units were grouped based on their engineering properties, geologic origins, and distribution in the subsurface. Our interpretation of the unit distributions within the subsurface is presented on the Interpretive Subsurface Profile A-A’, Figure 4. The location of the interpretive profile is shown on the Site and Exploration Plan, Figure 2. Our interpretation emphasized some data points more than others, considering factors such as relative distance to the alignment and quality of the data source. Contacts between the units may be more gradational than shown in the profile and boring logs, and subsurface conditions may vary between explorations differently from what is shown on Figure 4.

Standard Penetration Test (SPT) N-values presented on the Shannon & Wilson drill logs in Appendix B and on Figure 4 are in blows per foot (bpf) as counted in the field (i.e. no corrections have been applied). The historic borings contain some logs where the SPT N-values are similarly presented "as counted in the field" and some where it is not specified if the N-values are corrected or not. Discussions of SPT N-values that follow in this report are based on SPT N-values as reported on the logs (current and historic). The sections below describe the geotechnical unit characteristics in greater detail.

7.1.1 Fill

Based on the available subsurface information, it appears that varying thicknesses of Fill are present at the ground surface on both the west and east banks of the Willamette River in the project area. Fill thickness is up to 25 feet or more. Fill composition is variable across the site and includes mixtures of gravel, sand, silt, and clay that may include wood debris, concrete debris, brick fragments, glass, and other man-made materials. Refer to the boring logs in Appendix A and Appendix B for greater details of Fill composition in specific areas. Concrete and metal debris were encountered approximately 8 feet below the mudline at the initial location of Shannon & Wilson Boring B-2 (designated Boring B-2A). Two out of 96 SPTs attempted in the Fill met refusal, where more than 50 blows were required to drive the sampler through a 6-inch interval. Non-refusal SPT N-values ranged from 1 to 67 bpf. Natural moisture contents of tested specimens ranged from 7 to 62 percent. Sieve analyses indicated fines contents that ranged from 2 to 95 percent by dry weight.

7.1.2 Fine-Grained Alluvium

Fine-Grained Alluvium was encountered in explorations on both sides of the river. The unit is intermittently present below the Fill and as interbeds within and between other alluvial units. The thickest accumulations exist on the east side of the river, near Burnside Bridge Bent 21, and near Parsons Brinckerhoff Boring ES-2003A, where thicknesses are up to 110 feet and 45 feet, respectively. The Fine-Grained Alluvium consists of very soft to medium stiff (less commonly stiff to very stiff) Silt and Clay with varying amounts of sand, typically less than 30 percent. The unit includes USCS group designations ML and CL. Several samples from the unit were reported to contain organic material. SPT N-values in the unit ranged from 0 to 20 bpf. Natural moisture contents of tested specimens ranged from 22 to 63 percent. Dry unit weights of tested specimens ranged from 84 to 85 pounds per cubic foot (pcf). Sieve analyses indicated fines contents that ranged from 72 to 99 percent by dry weight. Atterberg limits tests indicated plasticity indices that ranged from 9 to 23 percent.

7.1.3 Sand/Silt Alluvium

Sand/Silt Alluvium was encountered intermittently throughout the project area, interbedded with the other alluvial units. The unit is most prevalent on the east side of the Willamette River, where thicknesses in the vicinity of Shannon & Wilson Boring B-3 are on the order of 110 feet. In the western and central portions of the site, thicknesses range from about 5 to 20 feet. The Sand/Silt Alluvium consists of Sandy Silt (ML) and Silty Sand (SM). Some samples contain trace interbeds of silt or clay, organics, or trace gravel. SPT N-values in the unit range from 1 to 48 bpf, and typically increase with depth below the ground surface. Natural moisture contents of tested specimens ranged from 30 to 47 percent. Sieve analyses indicated fines contents that ranged from 14 to 89 percent by dry weight. Atterberg limits tests indicated plasticity indices that ranged from 4 to 9 percent.

7.1.4 Sand Alluvium

Based on the available subsurface information, including older borings for the Burnside Bridge and Shannon & Wilson's current in-water boring B-2, we interpret an approximately 25- to 50-foot-thick layer of Sand Alluvium at the bottom of the modern-day Willamette River. Lesser layers, about 5 to 10 feet thick, were also encountered in the subsurface below the banks of the river in Shannon & Wilson Borings B-1 and B-3, and in Fujitani Hilts & Associates Boring D-1. The Sand Alluvium consists of loose to medium dense, occasionally dense to very dense, Sand to Gravelly Sand with varying amounts of silt including USCS group designations SP, SP-SM, and, to a lesser extent, SM. Some zones within the unit contain organics and wood debris. SPT N-values in the unit ranged from 9 to 51 bpf. The natural moisture content of one specimen was 21 percent. Sieve analyses indicated fines contents that ranged from 1 to 9 percent by dry weight.

7.1.5 Gravel Alluvium

We interpret a layer of Gravel Alluvium, ranging from about 10 to 40 feet thick, underlying the Sand Alluvium below the Willamette River, and underlying other alluvial deposits on the adjacent banks. As encountered in many explorations by Shannon & Wilson and others, the Gravel Alluvium consists of medium dense to very dense Gravel with varying amounts of sand and fines including USCS group designations GP, GW, GP-GM, GW-GM, and GM. Portions of the unit contain cobbles and possible boulders. Trace lenses of sand and silt may also be present. For the purposes of our interpretation, the Gravel Alluvium may include both coarse-grained Willamette River alluvium and coarse-grained Catastrophic (Missoula) Flood Deposits. The Gravel Alluvium is differentiated from the Catastrophic Flood Deposits – Channel Facies because it has a more consistent composition and contains fewer interbeds of silt and sand. During drilling in the gravel alluvium, mud loss and hole-caving were frequently noted. Forty-nine out of 78 SPTs attempted in the Gravel Alluvium met refusal. Non-refusal SPT N-values ranged from 19 to 95 bpf. Natural moisture contents of tested specimens ranged from 6 to 22 percent. Sieve analyses indicated fines contents that ranged from 2 to 33 percent by dry weight.

7.1.6 Catastrophic Flood Deposits – Fine-Grained Facies

Catastrophic Flood Deposits – Fine-Grained Facies sediments were encountered on the east side of the Burnside Bridge in borings made by GeoEngineers for the Portland Development Commission. In Borings GEI-8 and GEI-9, the unit was encountered directly underneath the Fill and extended to depths of 13 to 15 feet below the ground surface, respectively. In the vicinity of the Burnside Bridge, encountered portions of the unit were reported to consist of stiff to very stiff, brown Silt (ML). Two SPT N-values in the unit were 32 and 38 bpf. Natural moisture contents of tested specimens ranged from 23 to 41 percent. Dry unit weights of tested specimens ranged from 72 to 87 pcf.

7.1.7 Catastrophic Flood Deposits – Channel Facies

An approximately 20-foot-thick layer of Catastrophic Flood Deposits – Channel Facies sediments were encountered below the Catastrophic Flood Deposits – Fine-Grained Facies on the east side of the Burnside Bridge in borings made by GeoEngineers for the Portland Development Commission. In the vicinity of the Burnside Bridge, in Borings GEI-8 and GEI-9, encountered portions of the unit were reported to consist of dense to very dense interbedded sand and gravel deposits with varying amounts of fines, including USCS group designations GW, GW-GM, GP-GM, SW-SM, SP, and SP-SM. Lesser layers of stiff Sandy Silt (ML) were also reported in the unit. Portions of the unit contain cobbles and possible boulders. Three out of 11 SPTs attempted in the Catastrophic Flood Deposits – Channel Facies met refusal. Non-refusal SPT N-values ranged from 32 to 85 bpf. Natural moisture contents of tested specimens ranged from 6 to 38 percent.

7.1.8 Upper Troutdale Formation

Based on the available information, Troutdale Formation appears to underlie the entire project site, beneath the overlying alluvial and fill units. In our interpretation of the existing information, we identified both an Upper and Lower Troutdale Formation. The Upper Troutdale formation is approximately 15 to 30 feet thick and was encountered in the western portion of the project area. The unit includes dense to very dense Sand and Gravel deposits with varying fines content interbedded with hard Silt and Clay deposits containing varying amounts of sand. The unit includes USCS group designations GP, GW, GP-GC, SP, SC, ML, MH, CL, and CH. Some cementation was reported in portions of the unit.

The Upper Troutdale Formation contains more prevalent, lower-strength sand and fine-grained layers, compared to the underlying Lower Troutdale Formation. It also has relatively lower shear wave velocities. The upper unit may reflect Troutdale Formation that has weathered in place or that has been reworked by the Willamette River to include Pleistocene alluvium. Twenty-one out of 31 SPTs attempted in the Upper Troutdale Formation met refusal. Non-refusal SPT N-values ranged from 26 to 80 bpf and were associated with layers with greater sand and fines content. Natural moisture contents of tested specimens ranged from 2 to 33 percent. Sieve analyses indicated fines contents that ranged from 6 to 77 percent, with most tested samples being between 6 and 11 percent. Atterberg limits tests from samples in fine-grained layers indicated plasticity indices that ranged from 24 to 30 percent, with USCS designations of MH and CH.

7.1.9 Lower Troutdale Formation

Lower Troutdale Formation was encountered below the Upper Troutdale Formation on the west side of the project site, and directly below the Gravel Alluvium or Catastrophic Flood Deposits – Channel Facies on the east side of the project site. Thickness of the unit is on the order of 80 feet on the west side of the river and about 10 to 30 feet beneath the river. On the east side of the river, none of the borings fully penetrated the Lower Troutdale Formation and it appears to be over 100 feet thick. The unit typically consists of very dense Gravel with varying amounts of sand and fines, including USCS group designations GP, GW, GP-GM, GW-GM, GP-GC, GM, and GC. Zones of cementation are noted throughout the unit, and cobbles may be present in some areas. Some sand and fine-grained layers were also encountered (SP, SP-SM, SM, CL, CH). All but two of the 129 SPTs attempted in the Lower Troutdale Formation met refusal, most within the first 6 inches of penetration. The non-refusal SPT N-values were 76 and 79 bpf and came from sand layers within the unit. Natural moisture contents of tested specimens ranged from 7 to 43 percent. Sieve analyses indicated fines contents that ranged from 4 to 67 percent, with most tested samples being between 4 and 31 percent. An Atterberg limits test of one sample from a finer-grained layer indicated a plasticity index of 25 percent and a USCS designation of CH.

7.1.10 Sandy River Mudstone

We interpret that Sandy River Mudstone was encountered below the Lower Troutdale Formation in four borings along the western side of the project. These borings include the historic Burnside Bridge Boring for Pier 1; Parsons Brinckerhoff Boring PB-306R, performed for the West Side CSO; and recent Shannon & Wilson Borings B-1 and B-2. The Sandy River Mudstone may have also been encountered in the historic Burnside Bridge Boring for Pier 2, about 25 feet higher in elevation than it was encountered in the nearby Shannon & Wilson Boring B-2. This suggests possible variability in the elevation of the unit's surface in a north-south direction. Encountered portions of the unit include hard Clay with varying amounts of sand interbedded with very dense Sand that contains varying amounts of fines. The unit includes USCS group designations CL, CH, CL-ML, SM, and, to a lesser extent, ML, SP-SM, and SP. Trace gravel was observed in some samples and, in some areas, the sand constituent could be remolded to clay under finger pressure. Two out of 10 SPTs attempted in the Sandy River Mudstone met refusal. Non-refusal SPT N-values ranged from 35 to 93 bpf. The natural moisture contents of two tested specimens were both 25 percent. Sieve analyses of two specimens indicated fines contents of 70 and 93 percent. An Atterberg limits test of one fine-grained sample indicated a plasticity index of 46 percent and a USCS designation of CH.

7.2 Groundwater

The geotechnical borings performed by Shannon & Wilson for this study were drilled using mud rotary techniques, which make it difficult to discern the depth to groundwater, if it is encountered, due to the use of artificial drilling fluids in the boreholes. Logs of historic borings on the west side of the Willamette River, performed for the Ankeny Pump Station and the West Side CSO, report groundwater elevations that range from approximately 6 to 10 feet (NAVD 88). The log of ES-2005C, a historic boring performed for the East Side CSO on the east side of the Willamette River, reports a groundwater elevation of approximately 14.8 feet. Subsurface profiles associated with the GeoEngineers borings performed for the Portland Development Commission indicate a groundwater elevation of 25 feet. One of the GeoEngineers borings, GEI-7, encountered a layer of perched water at an elevation of approximately 50 feet. These groundwater level measurements were made during various seasons.

Over the course of a year, water levels in the Willamette River typically fluctuate between elevations of approximately 6 and 20 feet. The Willamette River Ordinary High Water (OHW) level is at elevation 20 feet and the annual high-water level (defined here as the average water level of the wettest six-month period) is at an approximate elevation of 10 feet. We based the annual high-water level on the design river elevation used for the nearby Tilikum Crossing project. This is comparable to the groundwater elevations reported in the historic on-land borings, with the exception of the perched groundwater reported in GEI-7. Based on the materials present in the subsurface at the site, it is reasonable to assume that there is hydraulic connectivity between the Willamette River and groundwater in the adjacent banks. We used the OHW elevation for our ground surface response analyses, and the annual high-water level for evaluation of our recommended ground improvement configuration. The annual high-water level was used for ground improvement design to limit conservatism and reduce the total volume of required ground improvements, and is consistent with recommendations in the ODOT Geotechnical Design Manual.

Groundwater levels throughout the site should be expected to vary seasonally and with changes in topography, precipitation, and the level of the Willamette River. Zones of perched water are likely to be encountered above fine-grained layers. Locally, groundwater highs typically occur in the late fall to spring and groundwater lows typically occur in the late summer and early fall.

8 SEISMIC GROUND MOTIONS AND HAZARD EVALUATIONS

Seismic hazard evaluations and soil ground motion responses for the Burnside Bridge Seismic Feasibility Study is performed following guidelines presented in the ODOT GDM (ODOT, 2018), ODOT BDM (ODOT, 2019), AASHTO LRFD Bridge Design Specifications (AASHTO, 2017), and the Earthquake Ready Burnside Bridge (EQRB) Seismic Design Criteria. In accordance with the project Seismic Design Criteria, the full-rupture CSZ event (Full Operation) and 1,000-year ground motion levels (Limited Operation) are considered for the seismic design.

We performed dynamic soil-structure interaction (DSSI) analyses to develop site-specific design ground motions and evaluate ground deformations from seismic shaking. DSSI analyses estimate the seismic response of a site based on earthquake time histories applied to the base of the model.

8.1 DSSI Analysis

We performed the DSSI analyses using the numerical modelling suite FLAC (Fast Lagrangian Analysis of Continua, Itasca, 2016). FLAC uses the finite difference method to model the behavior of continuous materials such as soil. We constructed the DSSI model based on limited subsurface explorations, in situ testing, and laboratory testing. In our opinion, the level of subsurface information available is acceptable for a NEPA-level evaluation. However, additional significant field explorations and testing program must be planned and performed for final design.

We developed a finite difference mesh based on the subsurface profile shown in Figure 4. We excluded bridge structural elements from the model, since they would not materially impact the behavior of the soil mass. We assigned engineering parameters such as density, stiffness, and strength to the various geologic units along the bridge alignment. We fixed the sides and base of the model against movement and allowed the model to come to equilibrium under gravity loads.

Next, we prepared the model for application of dynamic earthquake loads. We applied free-field boundary conditions to the edges of the model and quiet boundary conditions to the base of the model. These boundary conditions absorb earthquake waves to act as an infinite boundary. We also applied dynamic constitutive models to the various geologic units.

For non-liquefiable geologic units, we applied FLAC's hysteretic damping constitutive model. This model degrades the unit's shear modulus under shear strains using a calibrated backbone curve to model material damping. For potentially liquefiable soil units, we used the PM4SAND model (Boulanger and Ziotopoulou, 2015). PM4SAND models soil liquefaction behavior by generating excess pore water pressures in soil subjected to cyclic loading. We calibrated the PM4SAND behavior based on liquefaction triggering charts in Boulanger and Idriss (2014).

8.2 Base Ground Motions

We developed a suite of seven earthquake time histories for the Full Operation performance level and a suite of nine earthquake time histories for the Limited Operation performance level for use in the DSSI analyses. The time histories were selected to match target spectra for Site Class B/C boundary soil conditions that correspond to the soil conditions at the base of the soil model. The target spectra for the Limited Operation ground motion level were developed for Conditional Mean Spectra (CMS) conditioned at periods of 0.2 seconds and 1.0 second. A total of six earthquake time histories were selected to match the 0.2-second CMS, and three time histories were selected to match the 1.0-second CMS. Of the six time histories selected for the 0.2-second CMS, three were chosen from crustal earthquakes and three were chosen from subduction zone earthquakes. All three time histories selected for the 1.0-second CMS were selected from subduction zone earthquakes.

Exhibit 8-1 contains a summary of the earthquake time histories selected to model the Full Operation (CSZ event) and Limited Operation (1,000-year return period) ground motion levels.

Exhibit 8-1: Summary of Selected Earthquake Time Histories

Earthquake Name	Magnitude, M_W	Station, Component	Source-to-Site Distance (km)	Target Response Spectra	Designation
Tohoku (2011)	9.0	AKT023-Tsubakidai, EW	105	Operation (CSZ event)	O-AKT023EW
Tohoku (2011)	9.0	FKSH05-Shimogou, EW	126	Operation (CSZ event)	O-FKSH05EW
Tohoku (2011)	9.0	FKSH08-Naganuma, EW	100	Operation (CSZ event)	O-FKSH08EW
Tohoku (2011)	9.0	IWT011-Mizusawa, NS	75	Operation (CSZ event)	O-IWT011NS
Tohoku (2011)	9.0	TCGH12-Ujiie, NS	104	Operation (CSZ event)	O-TCGH12NS
Maule (2010)	8.8	ANTU-Cien Agronomicas, UC, La Plantina, 90°	73	Operation (CSZ event)	O-ANTU90
Maule (2010)	8.8	ROC1-Recinto d. SHOA, Cerro El Roble, 90°	93	Operation (CSZ event)	O-ROC190
L'Aquila, Italy (2009)	6.3	L'Aquila Parking, NS	5	Limited Operation CMS @ 0.2 seconds (crustal)	LS-AM043YLN
Northridge (1994)	6.7	LA 00, 270°	19	Limited Operation CMS @ 0.2 seconds (crustal)	LS-LA0270
Northridge (1994)	6.7	Santa Susana Ground, 0°	17	Limited Operation CMS @ 0.2 seconds (crustal)	LS-SSU000
Tohoku (2011)	9.0	FKS014-Yamatsuri, EW	76	Limited Operation CMS @ 0.2 seconds (subduction zone)	LS-FKS014EW
Tohoku (2011)	9.0	GNM010-Tatebayashi, NS	143	Limited Operation CMS @ 0.2 seconds (subduction zone)	LS-GNM010NS
Maule (2010)	8.8	CCSP97-Concepcion San Pedro, 97°	36	Limited Operation CMS @ 0.2 seconds (subduction zone)	LS-CCSP97
Tohoku (2011)	9.0	TCG012-Oyama, NS	119	Limited Operation CMS @ 1.0 second (subduction zone)	LS-TCG012NS
Tohoku (2011)	9.0	FKSH10-Nishigou, EW	106	Limited Operation CMS @ 1.0 second (subduction zone)	LS-FKSH10EW
Maule (2010)	8.8	ANTU-Cien Agronomicas, UC, La Plantina, 90°	73	Limited Operation CMS @ 1.0 second (subduction zone)	LS-ANTU90

8.3 Ground Surface Response Spectra

For each earthquake time history, we calculated ground surface response spectra at each of the existing bridge bents/piers. Based on the spectral response, we grouped the ground surface response spectra into two groups: Bents 1 through 27 (including Piers 1 through 4), and Bents 28 through 35. The individual site-specific response spectra at each bent are presented in Figures E1 through E38 in Appendix E for Full Operation and Figures E39 through E152 for Limited Operation performance levels.

To inform the development of our recommended Full Operation design response spectra, we generated the ODOT code-based design response spectra using the web-based application maintained by Portland State University (PSU) (ODOT 2019). The PSU application requires an input value of Vs30 to calculate an acceleration response spectrum. We used estimated values of 200 meters/second to approximate Site Class E conditions, and 274 meters/second to approximate Site Class D conditions. The response spectra generated by the PSU application are shown on Figures 5 and 6. Similarly, we generated ODOT code-based design response spectra to inform our development of our recommended Limited Operation design response spectra using the Microsoft Excel-based ODOT Design Response Spectrum Program available on the ODOT Bridge Section website. The response spectra generated using the ODOT program are shown on Figures 7 and 8.

8.4 Recommended Seismic Design Ground Motions

We developed the smoothed design response spectra for the bent groups by approximating or enveloping the hazard-consistent geometric means of the ground surface response spectra. Figures 5 and 6 show that the Operation (CSZ event) response spectra derived from the ODOT web-based application (ODOT 2019) for Site Class E and D are lower than our calculated geometric mean ground surface response spectra for periods less than about 0.5 seconds and are greater than or equal to the geometric mean ground surface response spectra for longer periods. The smoothed, Full Operation design spectra also shown on Figures 5 and 6 are equal to or greater than the ODOT web-based spectra at all periods.

Similarly, Figures 7 and 8 show that our calculated geometric mean ground surface response spectra for the Limited Operation (1,000-year ground motion) are typically equal to or higher than the ODOT code-based response spectra for periods less than about 0.5 to 1.0 seconds. The smoothed, Limited Operation spectra also shown on Figure 7 and 8 are greater than or equal to the ODOT web-based spectra for periods greater than about 0.6 to 0.8 seconds and follow the higher ODOT code-based spectra at longer periods.

Exhibit 8-2 provides the recommended site-specific smoothed ground surface design response spectra for Operation and Limited Operation performance levels.

Exhibit 8-2: Recommended Seismic Design Spectral Accelerations at Existing Bent Groups*

	"Full Operation" Performance Level (CSZ Event)		"Limited Operation" Performance Level (1,000-Year Return Period)	
Period (seconds)	Bents 1 through 27 (g)	Bents 28 through 35 (g)	Bents 1 through 27 (g)	Bents 28 through 35 (g)
0	0.293	0.201	0.457	0.361
0.02	0.336	0.255	0.552	0.509
0.03	0.362	0.298	0.599	0.583
0.05	0.414	0.382	0.694	0.731
0.075	0.479	0.488	0.812	0.917
0.1	0.544	0.594	0.930	1.10
0.2	0.75	0.86	1.17	1.24
0.3	0.75	0.86	1.17	1.24
0.5	0.75	0.65	1.17	0.934
0.75	0.717	0.434	0.925	0.623
1	0.538	0.325	0.694	0.467
1.5	0.359	0.217	0.463	0.311
2	0.269	0.163	0.347	0.234
3	0.179	0.108	0.231	0.156
5	0.065	0.039	0.083	0.056
7.5	0.029	0.017	0.037	0.025
10	0.016	0.010	0.021	0.014

NOTES:

* Response spectrum analyses at proposed Retrofit and Replacement Alternative bents should use the response spectra corresponding to the existing bent groups provided.

8.5 Seismic Hazard Evaluation

Seismic hazards considered in the evaluation include ground shaking, liquefaction and associated effects (e.g., flow failure, lateral spreading, and settlement), ground surface fault rupture, tsunami, and seiche. In our opinion, the potential for fault rupture is low; while there are potentially active faults with approximately 1/2 mile of the bridge site, the recurrence interval for movement on these faults appear to be on the order of several thousand years and much longer than the return period for the for the "Limited Operation" Performance Level. The risk of seismically induced tsunami and seiche is also very low at the site given the location of the site is over 60 miles inland from the Pacific Ocean (where a tsunami wave would initially reach landfall), and that the Willamette River is not a closed water body that is typically required for the occurrence of seismic seiche. The primary hazards at this site are ground shaking, liquefaction, and liquefaction-related effects. Liquefaction is a phenomenon in which excess pore pressure of loose to medium dense, saturated, granular soils increases during ground shaking to a level near the initial effective stress. The increase in excess pore pressure results in a reduction of soil shear strength and a potential quicksand-like condition. The effects of liquefaction may include lateral spreading, flow failure, and ground surface settlement. Liquefaction impacts to foundations may also include reduction or loss of axial and lateral resistance and downdrag forces on deep foundations.

Liquefaction in gently sloping ground or ground adjacent to a free face can result in permanent lateral ground displacement in phenomena known as lateral spreading and flow failure. Lateral spreading ground movement occurs toward a free face or down slope during seismic shaking; flow failure may occur after ground shaking has ended. Similarly, steeper slopes may become unstable during seismic shaking or due to the associated strength loss caused by excess pore pressure development. The permanent ground displacement may result in additional lateral forces acting on deep foundations that extend through liquefiable layers and may also result in moderate to severe damage to the existing structure, up to and including collapse of the bridge foundations.

Settlement may occur in cohesionless soil that undergoes liquefaction and pore pressure development during ground shaking. The settlement is related to densification and rearrangement of particles during ground shaking, as well as volume change as the excess pore pressure dissipates after ground shaking. Seismic ground settlement may not occur uniformly over an area, and differential settlement could impact structures supported by liquefied soil. Seismic settlement may also result in downdrag forces on foundations if the soil settlement is greater than the foundation settlement.

Liquefaction, excess pore pressure development, and lateral movement can be evaluated directly using nonlinear effective stress numerical analysis. The results of an effective stress analysis provide estimates of excess pore pressure and lateral movement during ground shaking. Liquefaction and associated soil shear strength loss may be estimated to occur where excess pore pressures exceed a certain threshold. Soil strength reductions may also be estimated when excess pore pressure development occurs but is less than the liquefaction threshold. Liquefaction-induced settlement and lateral soil movement can also be estimated from the nonlinear effective stress analysis. The nonlinear effective stress analyses performed for this study were utilized to evaluate liquefaction and its associated impacts. A brief summary of the analyses and results is presented in the following sections.

8.5.1 Liquefaction-Induced Excess Pore Pressure Development and Residual Soil Strength

Figures E153 through E168, Appendix E, presents contour plots of the excess pore pressure ratio based on the DSSI analyses for each input ground motion. Liquefaction is considered to occur when the excess pore pressure ratio exceeds 0.9 (i.e. liquefaction is considered to occur when the factor of safety (FS) against liquefaction is less than 1.1; the excess pore pressure ratio criteria is the inverse of the FS, equal to the ratio of 1:1.1).

When the excess pore pressure ratio exceeds 0.9, residual shear strengths are considered in the nonlinear effective stress analyses. We estimated the shear strength of the liquefied soil using methods recommended in the ODOT GDM and other standard methods. These methods include Seed and Harder (1990), Olson and Stark (2002), Idriss and Boulanger (2007), and Kramer (2008). These methods base the liquefied soil shear strength on (N1)60 or (N1)60-cs values. For our analysis, we estimated the residual shear strength by taking the average of the residual shear strengths determined using the four recommended methods.

Please see Section 9 for information on how liquefaction will affect the seismic resistance of the foundations. Conceptual options to mitigate liquefaction effects are presented in Section 10 of this report.

8.5.2 Liquefaction-Induced Lateral Spreading and Flow Failure

Figures E153 through E168, Appendix E, present contour plots of estimated permanent horizontal deformation based on the nonlinear effective stress model for each input ground motion.

The figures indicate that liquefaction-induced permanent ground deformation will occur at the west and east approaches to varying displacements and elevations for the ground motion levels considered. For the 1,000-year "Limited Operation" ground motion level, ground surface movements up to 14 feet are calculated for the west riverbank. Permanent displacements greater than one foot are typically located within 100 feet inland of the west seawall. Flow failure with displacements in excess of approximately 25 feet is anticipated at the east riverbank. Lateral spreading displacements of approximately 3 feet or greater are anticipated at distances up to around 600 feet inland from the east riverbank.

For the CSZ "Full Operation" event ground motion level, ground surface movements up to 4 and 25 feet are anticipated at the west and east riverbanks, respectively. Permanent displacements greater than one foot are typically located within 100 feet inland of the west seawall. Lateral spreading displacements of approximately 2 feet or greater are anticipated at distances up to around 400 feet inland from the east riverbank.

The effects of permanent ground displacement on the existing foundations are presented in Section 9 of this report. Conceptual options to mitigate permanent ground displacement are presented in Section 10 of this report.

8.5.3 Liquefaction-Induced Settlement

We estimated post-liquefaction reconsolidation settlement using the average of the maximum shear strains from the input ground motions for each ground motion level, determined in the DSSI analyses. We used the relationship between shear strain and volumetric strain by Idriss and Boulanger (2008) to estimate settlement.

The maximum shear strains and estimated settlements from the models are influenced by shear stains caused by permanent lateral displacement of the west and east riverbanks. In our opinion, the estimated settlement from the models may overestimate actual ground settlement at the west and east riverbanks. Therefore, we used the average of the maximum shear strains to provide an approximation for this report.

Exhibit 8-3 presents the estimated liquefaction-induced settlement at the existing spread footing foundations. The effects of liquefaction and associated settlement on the existing spread footing foundations are presented in Section 9.1.1 of this report.

Exhibit 8-3: Estimated Liquefaction-Induced Settlement at Existing Spread Footing Foundations

	Liquefaction-Induced Settlement at Bottom of Footing (in)	
Location	Full Operation	Limited Operation
Bent 1	1	2
Bent 2	1	3
Bent 3	1	2
Bent 4	2	3
Bent 5	2	3
Bent 6	2	3
Bent 7	2	3
Bent 8	2	3
Bent 9	2	4
Bent 10	2	4
Bent 11	2	3
Bent 12	2	3
Bent 13	2	2
Bent 14	2	2
Bent 15	1	2

Location	Liquefaction-Induced Settlement at Bottom of Footing (in)	
	Full Operation	Limited Operation
Bent 16	1	2
Bent 17	1	2
Bent 28	0	0
Bent 29	0	0
Bent 30	0	0
Bent 31	0	0
Bent 32	0	0
Bent 33	0	0
Bent 34	0	0
Bent 35	0	0

Exhibit 8-4 presents the estimated liquefaction-induced settlement at the existing pile group foundations. The effects of liquefaction and associated settlement on the existing pile group foundations are presented in Section 9.2.1 of this report.

Exhibit 8-4: Estimated Liquefaction-Induced Settlement at Existing Pile Group Foundations

Location	Liquefaction-Induced Settlement at Bottom of Pile Cap Elevation (in)		Liquefaction-Induced Settlement at Average Pile Tip Elevation (in)	
	CSZ Event	1,000-Year Return Period	CSZ Event	1,000-Year Return Period
Bent 18	1	2	0	0
Bent 19	3	5	0	0
Pier 1	2	4	0	0
Pier 2	1	2	0	0
Pier 3	5	9	0	0
Pier 4	24	32	13	19
Bent 21	43	51	13	20
Bent 22	26	46	8	22
Bent 23	16	38	5	17
Bent 24	10	28	3	9
Bent 25	4	25	1	3
Bent 26	3	17	0	1
Bent 27	1	6	0	0

9 EXISTING FOUNDATION RESISTANCE AND STIFFNESS

9.1 Spread Footings

Based on the bottom of footing elevations provided in the as-constructed drawings and the available subsurface information, the spread footings at Bents 1 through 15 and Bent 28 were likely founded in the Fine-Grained Alluvium, spread footings at Bents 16, 17, and 29 through 34 were likely founded in Fill, and the spread footing at Bent 35 was likely founded in the Catastrophic Flood Deposits – Channel Facies. The existing spread footing foundations and anticipated bearing material are shown on the Interpretive Subsurface Profile A-A’, Figure 4.

9.1.1 Liquefaction Effects

Based on our seismic hazard evaluation and as-constructed information, the spread footings at Bents 1 through 17 are founded within or above potentially liquefiable Fine-Grained Alluvium, Fill, and Sand/Silt Alluvium. No liquefaction effects are anticipated at Bents 28 through 35.

Liquefaction-related risks to the spread footing foundations at Bents 1 through 17 include ground surface disruption, liquefaction-induced settlement, and bearing capacity reduction. The liquefaction-induced settlement at Bents 1 through 17 presented in Exhibit 8-3 should be considered in the seismic performance evaluation of the bridge.

Based on discussions with HDR, we understand the seismic performance of the existing spread footing foundations is inadequate. Therefore, we only performed evaluation of the existing spread footings for the static and seismic (pseudo-static) conditions; we did not estimate a post-seismic/reduced strength bearing resistance for the liquefied soil conditions. A discussion of conceptual options to mitigate the liquefaction-induced loss in bearing resistance and liquefaction-induced settlement of the existing spread footing foundations is presented in Section 10, and foundation modeling parameters for the post-seismic/reduced strength condition for the preferred retrofit and replacement alternatives are presented in Section 11 and Section 12, respectively.

9.1.2 Bearing Resistance

We estimated the nominal static and seismic bearing resistance for existing spread footings by evaluating the strength parameters from the available subsurface information and performing a conventional spread footing evaluation. The nominal bearing resistance is provided in Exhibit 9-1. The bearing resistances reported in the table are nominal geotechnical resistances and should be reduced by resistance factors of 1.0, 0.45, and 0.9 for service, strength, and extreme event limit states, respectively.

9.1.3 Subgrade Stiffness

We understand that the seismic performance of the footings will be modeled using equivalent six degree of freedom springs. The spring constants will be developed using the recommended procedures in the ODOT BDM or the FHWA Seismic Retrofitting Manual for Highway Structures. Exhibit 9-1 presents the recommended values for the required information to fully describe spring stiffness, including bearing material shear modulus, Poisson's ratio, and nominal bearing resistance. In Exhibit 9-1, we have provided bearing material initial shear modulus (maximum modulus) for static and seismic conditions. We understand that the structural engineer will develop the necessary large strain shear modulus values based on the ODOT BDM or the FHWA Seismic Retrofit Manual. In general, we recommend that the strain calculated in the structural analyses be checked against the strain assumed in selecting the shear modulus. The structural engineer may need to iterate their analyses using a different strain-compatible shear modulus. The Poisson's ratio is constant for the purposes of the evaluation.

9.1.4 Sliding Resistance

Sliding resistance for a spread footing may be developed through friction on the base of the footing and passive earth pressures on the face of the footing. The nominal friction resistance is expressed as the vertical load (i.e., actual footing pressure) multiplied by a coefficient of friction (tan δ). Sliding resistance generated by the lateral passive earth pressure acting on the face of the footing is assumed to develop if the footing is free to translate horizontally. If horizontal movement of the footing is limited, the earth pressure resistance values should be reduced to reflect the reduced footing movement based on the FHWA Seismic Retrofit Manual.

We estimated the nominal static and seismic frictional sliding coefficient for the existing footings; the results are presented in Exhibit 9-1 in terms of tan δ. Sliding resistance factors of 0.8 and 1.0 should be used for the strength and extreme event limit states, respectively.

The passive earth pressures we developed for the static and seismic conditions are also presented in Exhibit 9-1 in terms of equivalent fluid pressure and depth of footing (D, in feet). These earth pressure values may be used to estimate the lateral resistance of footings. Alternatively, for abutments, the ODOT BDM Section 1.10.4.2 allows the use of a wall height-adjusted pressure value of 5 ksf for calculating seismic translational horizontal resistance of an abutment. We present the equivalent fluid pressure for both static and seismic cases; the passive earth pressures are not additive, i.e., use only the seismic passive earth pressure (EFPpE) for seismic cases. Passive pressure resistance factors of 0.5 and 1.0 should be used for strength and extreme event limit design cases, respectively.

Exhibit 9-1: Recommended Unfactored Static and Seismic Soil Parameters for Existing Spread Footings and Pile Caps

	Location	[a]Approx. Footing Elev. (ft) (depth below ground surface, ft)	[b]Soil Type	Total Unit Weight, γ (pcf)	Friction Angle,Φ (degrees)	Cohesion, c (psf)	Q_{nom} (ksf)	Nominal Sliding Coeff., tanδ	[e]Bearing Material Initial Shear Modulus, (ksi)	Poisson's Ratio	[f]Lateral Earth Coefficients						[f,g,m]Lateral Earth Pressures (psf)					
											Ko	Ka	Kp	[h]ΔKoE	[h]ΔKaE	[h]KpE	[i]EFPo	[i]EFPa	[i]EFPp	[j,k]ΔEFPoE	[j,k]ΔEFPaE	[k,l]EFPpE
Abutments	Bent 1	24.5 (5)	Fine-Grained Alluvium	110	29	----	3	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57H 57D	39H 39D	317H 317D	12H [31H] 12D [31D]	6H [13H] 6D [13D]	300H [282H] 300D [282D]
	Bent 35	41 (9)	CFD Channel Facies	130	36	----	9	0.58	18	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57H 57D	39H 39D	317H 317D	12H [31H] 12D [31D]	6H [13H] 6D [13D]	300H [282H] 300D [282D]
Footings	Bents 2 through 15	22 (7)	Fine-Grained Alluvium	110	29	----	4	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
	Bent 16	22 (9)	Fill	110	29	----	4	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
	Bent 17	12 (18)	Fill	110	29	----	4	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
	Bent 28	22 (27)	Gravel Alluvium (assumed)	110	29	----	8	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
	Bents 29 through 32	40 (10)	Gravel Alluvium (assumed)	110	29	----	8	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
	Bents 33 and 34	37 (12)	Gravel Alluvium (assumed)	110	29	----	8	0.44	7	0.35	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
Pile Caps	Bents 18 and 19	7 – 9 (24)	Fill	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]
	Pier 1 [n]	-41.6 (17)	Fill / Fine-Grained Alluvium	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	0.11 [0.28]	0.05 [0.12]	2.73 [2.56]	57D	39D	317D	12D [31D]	6D [13D]	300D [282D]

Exhibit 9-1 (cont'd): Recommended Unfactored Static and Seismic Soil Parameters for Existing Spread Footings and Pile Caps

											[f]Lateral Earth Coefficients						[f,g,m]Lateral Earth Pressures (psf)					
	Location	[a]Approx. Footing Elev. (ft) (depth below ground surface, ft)	[b]Soil Type	Total Unit Weight, γ (pcf)	Friction Angle, Φ (degrees)	Cohesion, c (psf)	Qnom (ksf)	Nominal Sliding Coeff., tanδ	[e]Bearing Material Initial Shear Modulus, (ksi)	Poisson's Ratio	Ko	Ka	Kp	[h]ΔKoE	[h]ΔKaE	[h]KpE	[i]EFPo	[i]EFPa	[i]EFPp	[j,k]ΔEFPoE	[j,k]ΔEFPaE	[k,l]EFPpE
Pile Caps	Piers 2 and 3	-70 (16)	Sand Alluvium	125	35	---	---[c]	---[d]	---	---	0.43	0.27	3.69	0.11 [0.27]	0.05 [0.11]	3.52 [3.33]	27D	17D	231D	7D [17D]	3D [7D]	220D [208D]
	Pier 4 [o]	-40.3 (48)	Fine-Grained Alluvium	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	0.13 [0.39]	0.06 [0.17]	2.72 [2.46]	25D	17D	137D	6D [19D]	3D [8D]	130D [117D]
	Bents 21 and 22	2 (14)	Fine-Grained Alluvium	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	0.13 [0.39]	0.06 [0.17]	2.72 [2.46]	57D	39D	317D	14D [43D]	7D [19D]	299D [271D]
	Bents 23 and 24	2 – 7 (22)	Fine-Grained Alluvium	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	0.13 [0.39]	0.06 [0.17]	2.72 [2.46]	57D	39D	317D	14D [43D]	7D [19D]	299D [271D]
	Bents 25 through 27	10 (25)	Fill	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	0.13 [0.39]	0.06 [0.17]	2.72 [2.46]	57D	39D	317D	14D [43D]	7D [19D]	299D [271D]

NOTES:

* Groundwater is assumed to be at an elevation of 20 feet based on existing borings and Willamette River Ordinary High Water level.

a. Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set. Indicates bottom of pile cap elevation for Bents 18 and 19, Piers 1 through 4, and Bents 21 through 27.

b. Soil type refers to bearing material for abutments and footings, and retained soil for pile caps.

c. Pile caps should not be assumed to provide bearing resistance.

d. Pile caps should not be assumed to develop lateral resistance from base friction.

e. Initial shear modulus values are estimated from shear wave velocity measurements and ODOT GDM Table 6-2 (Seed, et al.).

f. Bracketed seismic values represent the 1,000-year event and unbracketed values represent the CSZ event.

g. For abutments, D is the minimum embedment of the abutment wall measured from the ground surface to the bottom of the wall footing and H is the height of the retained soil behind the abutment wall; D or H will be used to determine lateral earth pressures on the abutment wall depending on the direction of loading. For footings and pile caps, D is the minimum embedment of the footing or pile cap measured from the ground surface to the bottom of the footing or pile cap.

h. Seismic lateral earth coefficients for active and at-rest cases are incremental values and should be added to static values to estimate total lateral earth pressures. Passive seismic lateral earth coefficients are given as total lateral earth pressures.

i. Static lateral equivalent fluid pressures - Assume a triangular pressure distribution.

j. Incremental seismic equivalent earth pressures for active and at-rest cases - Assume an inverted triangular pressure distribution.

k. Seismic lateral equivalent fluid pressures for active and at-rest cases are incremental values and should be added to static values to estimate total seismic pressures. Passive seismic lateral equivalent fluid pressure is given as a total pressure.

l. Seismic passive lateral equivalent fluid pressure - Assume a triangular pressure distribution.

m. For abutments, ODOT BDM Section 1.1.4.2 allows the use of a wall height-adjusted pressure value of 5.0 ksf for calculating seismic translational resistance. Refer to BDM for additional application details.

n. For Pier 1, due to unbalanced retained soil height in the longitudinal direction, add 55 feet to pile cap embedment (D) when calculating lateral earth pressures against the west (upslope) side of the pile cap.

o. For Pier 4, due to sloping ground in front of pile cap in the longitudinal direction, ignore lateral earth pressures against the west (downslope) side of the pile cap.

9.2 Piles

Based on the pile tip elevations provided in the as-constructed drawings, foundation piling summary, and available subsurface information, the timber piles at Bents 18, 19, and Piers 1 through 3 were likely driven into the Sand/Silt Alluvium and/or Sand Alluvium, and founded on the top of the Gravel Alluvium. The timber piles at Pier 4 were likely driven into the Sand/Silt Alluvium and founded on the top of the Sand Alluvium, and timber piles at Bents 21 through 27 were likely driven into the Sand/Silt Alluvium and/or Fine-Grained Alluvium. The existing timber pile foundations and anticipated bearing material are shown on the Interpretive Subsurface Profile A-A’, Figure 4.

9.2.1 Liquefaction Effects

Based on our seismic hazard evaluation and as-constructed information, the piles at Bents 18, 19, and Piers 1 through 3 extend through potentially liquefiable Sand/Silt Alluvium and/or Sand Alluvium and bear on the top of the Gravel Alluvium, and the piles at Pier 4 and Bents 21 through 27 bear within potentially liquefiable Sand Alluvium, Sand/Silt Alluvium, and Fine-Grained Alluvium.

The liquefaction-related risks to the pile foundations are different depending on the location of the liquefiable soil in relation to the pile. At Bents 18, 19, and Piers 1 through 3, liquefaction-induced settlement of the liquefiable layer and overlying soil will induce downdrag loads on the piles that bear in the Gravel Alluvium below the liquefiable layer, resulting in potential pile overstressing. Additionally, due to the minimal pile embedment below the liquefiable layer, lateral stability of the pile foundations is also a potential concern. Permanent ground displacement at the west riverbank (Bents 18, 19, and Pier 1) may also result in collapse of the existing bridge foundations.

The primary concern at Pier 4 and Bents 21 through 27 is permanent ground displacement at the east riverbank that may result in collapse of the existing bridge foundations. Additionally, liquefaction-induced settlement will result in settlement of the pile caps, downdrag loads on the piles, and reduction in axial pile resistance.

The liquefaction-induced settlement at the existing pile group foundations presented in Exhibit 8-4 should be considered in the seismic performance evaluation of the bridge. Based on discussions with HDR, we understand the seismic performance of the existing pile group foundations is inadequate. Therefore, we only performed evaluation of these existing pile group foundations for the static and seismic (pseudo-static) conditions; we did not estimate a post-seismic/reduced strength resistance for the liquefied soil conditions. A discussion of conceptual seismic mitigation alternatives for the existing pile group foundations is presented in Section 10, and foundation modeling parameters for the post-seismic/reduced strength condition for the preferred retrofit and replacement alternatives are presented in Section 11 and Section 12, respectively.

9.2.2 Single Pile Axial and Uplift Resistance

We estimated the nominal axial and uplift resistance of individual piles using the computer program APILE v2015 (Ensoft, 2015). We developed engineering parameters for the pile resistance evaluation based on our characterization of subsurface materials, subsurface explorations, and our interpretation of the available subsurface information. We performed the pile resistance evaluation in general accordance with the FHWA (Norlund-Thurman) methodology. For preliminary evaluation purposes, we assumed a single value for the resistance of all piles at each pile group. The results of the single pile axial and uplift resistance evaluation for the static and seismic conditions are shown on Exhibit 9-2. The axial resistances reported in the table are nominal geotechnical resistances and should be reduced by resistance factors of 1.0, 0.45, and 1.0 for service, strength, and extreme event limit states, respectively. The uplift resistances should be reduced by resistance factors of 1.0, 0.35, and 0.8 for service, strength, and extreme event limit states, respectively.

Exhibit 9-2: Recommended Nominal Static and Seismic Axial and Uplift Resistance for Existing Piles

Location	Nominal Single Pile Axial Resistance (kips)	Nominal Single Pile Uplift Resistance (kips)
Bent 18	30	5
Bent 19	60	40
Pier 1	155	115
Pier 2	65	50
Pier 3	80	50
Pier 4	45	15
Bent 21	100	95
Bent 22	65	60
Bent 23	65	60
Bent 24	70	65
Bent 25	95	90
Bent 26	90	85
Bent 27	65	60

9.2.3 Pile Group Evaluation

We recommend the nominal axial and uplift resistance of pile groups be considered as the sum of the axial or uplift resistance of all the piles included in the pile group.

During the previous phase of the project, we evaluated the pile cap response of the existing pile group foundations to axial loading and lateral loading in the longitudinal and transverse orientations for the static and seismic conditions. We completed the analysis using the computer program GROUP v2016, (Ensoft, 2016). We modeled the pile group axial and lateral efficiency and overall stiffness of the piers considering pile geometry and lateral and axial pile resistance only (i.e. the earth pressures on the embedded portion of the pile cap and footing column were not considered). Passive earth pressures that may be induced by relative movement between the pile caps and the surrounding soil may also provide resistance to lateral forces and movement. Earth pressures on embedded pile caps are discussed in Section 9.3. Based on the results of our analyses, we have developed axial and lateral load-displacement curves at the bottom of the pile cap for each existing pile group for the static and seismic conditions. It was assumed the pile cap is rigid and that the pile head connection to the pile cap is fixed. The results of the evaluation are shown in Appendix F, Load-Displacement Curves for Existing Pile Groups.

9.3 Earth Pressure on Abutment Walls and Embedded Pile Caps

The lateral earth pressures on a retaining wall, including capacity/stiffness and seismically induced loads, are a function of relative wall and soil displacement. If the wall is allowed to displace (typically 2 percent of the wall height), the lateral pressures may be developed assuming active pressures as a load and full passive pressure as a resistance. If the wall is restrained from moving, seismically induced loads increase and the passive resistances decrease. If a wall is allowed to displace less than 2 percent, the active earth pressures should be calculated using the full seismic acceleration coefficient (as opposed to one-half of the acceleration coefficient used for walls that are allowed to freely displace), and passive resistance should be taken as a portion of the full value.

We assume that the soil surrounding the various abutment walls and pile caps will be allowed to displace at least 2 percent of the wall height and therefore mobilize full active and passive lateral earth pressures. The earth pressure parameters we developed for the static and seismic conditions for existing abutment walls and pile caps are presented in Exhibit 9-1.

10 CONCEPTUAL SEISMIC MITIGATION GROUND IMPROVEMENT DESIGN

10.1 Enhanced Seismic Retrofit

We understand the seismic performance of the existing bridge foundations is inadequate. Based on our seismic hazard evaluation and HDR's evaluation of the seismic performance of the existing bridge foundations, seismic ground improvement and foundation retrofit will be required at the following existing bridge bents and piers:

- Spread footings at Bents 1 through 17 due to liquefaction-induced settlement, bearing capacity reduction, and inadequate footing size and strength;
- Pile groups at Bents 18, 19, and Pier 1 due to liquefaction-induced settlement, permanent ground displacement of the west riverbank, and inadequate pile lateral strength and uplift resistance;
- Pile groups at Piers 2 and 3 due to liquefaction-induced settlement, permanent ground displacement, and inadequate pile uplift resistance;
- Pile groups at Pier 4 and Bents 21 through 27 due to liquefaction-induced settlement, permanent ground displacement of the east riverbank, and inadequate pile lateral strength and uplift resistance; and
- Spread footings at Bents 28 through 35 due to inadequate footing size and strength.

Based on our discussion with the design team, we understand the existing spread footings (except Bent 17) will be enlarged to address inadequate footing size and strength, and the spread footings at Bent 17 and all existing pile group foundations may be retrofitted with drilled shafts to address inadequate pile lateral strength and uplift resistance. Therefore, seismic mitigation may be required to mitigate liquefaction-induced settlement and bearing capacity reduction at Bents 1 through 16, permanent ground displacement of the west riverbank at Bents 18, 19, and Pier 1, and permanent ground displacement of the east riverbank at Pier 4 and Bents 21 through 27. The effects of liquefaction-induced settlement at Bents 17 through 19, 21 through 27, and Piers 1 through 4 will be mitigated through the use of drilled shafts founded below the liquefiable layers. Large lateral soil displacements are also anticipated within the river channel. However, because the sand alluvium is expected to liquefy during the seismic event, we understand the shafts at Piers 2 and 3 can be designed to resist the lateral loads caused by the laterally displaced soil. Therefore, we assumed that ground improvements will not be required at Piers 2 and 3. The proposed Enhanced Retrofit alternative described in this report is shown on Figure 9.

10.1.1 Seismic Mitigation and Ground Improvement Strategy

Ground improvement methods include excavation and replacement, soil densification (e.g., vibro-compaction, deep dynamic compaction), drainage (e.g., EQ Drain), soil cementation (e.g., jet grouting, deep soil mixing) or a combination of methods such as soil densification and drainage (e.g., stone columns) or soil densification and cementation (e.g., compaction grouting). The selection of an appropriate mitigation method(s) for a particular site depends on factors such as soil type (fines content, organic content, pH, etc.), site access, right-of-way constraints, cost, environmental concerns, and vibration impacts on existing facilities, among others.

In our opinion, the critical factors for developing a ground improvement strategy at the site include:

1. The anticipated depth of potentially liquefiable soils;
2. The engineering properties required from the improved soil mass;
3. Low-overhead clearance issues for performing work below the bridge deck; and
4. Existing obstructions, such as timber pile groups located within anticipated extents of ground improvement.

In our opinion, a soil cementation strategy is required to mitigate the potentially liquefiable soils below the bridge alignment and create an improved soil mass capable of resisting lateral spreading forces. We evaluated the advantages and disadvantages for deep soil mixing and jet grouting strategies for the Retrofit alternative. A summary of our evaluation is presented in Exhibit 10-1.

Exhibit 10-1: Comparison Between Viable Ground Improvement Strategies for Retrofit Alternative

Alternative	Advantages	Disadvantages
Deep Soil Mixing Consists of mechanical blending of grout and in situ soil using a soil mixing tool such as an auger.	- Lower relative cost; - Lower environmental impacts (no chance of fracking out, surface spoils containment etc.); - More competitive bidding	- Requires relatively high overhead clearance for equipment; - Becomes very difficult in areas with underground obstructions, such as existing timber piles; - Cannot be performed at an inclination away from vertical; - Performed from ground surface to depth using a top to bottom approach;
Jet Grouting Uses high velocity jets of slurry grout to erode and mix in situ soils.	- Effective in almost all soil types; - Low headroom and highly mobile equipment available; - Can be performed at inclinations away from vertical and through existing footings/pile caps/seals etc.; - Can be performed within specific isolated soil layers using a bottom-top approach; - Can improve soil mass around existing timber piles.	- Higher relative cost; - Generates a relatively large volume of construction spoils which require removal and disposal.

Due to the overhead clearance limitations and anticipated obstructions, such as the existing timber piles, we believe that using jet grouting with low headroom equipment may be the most feasible strategy for the Retrofit alternative.

We developed our recommended ground improvement strategy using the 2D FLAC model described in Section 8. Ground improvement zones were added to the model and dimensions were iterated to determine the minimum anticipated ground improvements required to achieve tolerable displacements at each bent. We applied the ground motion identified to produce the largest lateral soil displacements for this analysis. Recommended soil improvements at existing spread footings and existing foundation elements were not included in the model since they would not materially impact the behavior of the soil mass.

The following sections present our conceptual-level design recommendations for seismic mitigation consistent with the proposed bridge retrofit and widening strategies as we understand them at the time of this report. Figures E169 and E170 show the results of the of the 2D FLAC model with our recommended ground improvements as contours of deformation, excess pore pressure, and time to liquefaction along the entire bridge alignment for the “worst-case” ground motions. Figures E171 through E206 present profile results of the 2D FLAC model at each bent location.

10.1.2 West Approach (Bents 1-19 Retrofit)

At the time of this report, we understand the existing spread footings at Bents 1 through 16 will be enlarged, and the existing spread footings at Bent 17 and existing pile group foundations at Bents 18 and 19 will be retrofitted by constructing a “superbent” supported by two drilled shafts at each bent. This superbent would also be used to support the bridge widening. Each superbent will consist of two 8-foot diameter drilled shafts adjacent to the spread footings or pile caps, connected by a grade beam that is also tied into the existing spread footings or pile caps.

Seismic mitigation may be required to mitigate liquefaction-induced settlement and bearing capacity reduction at Bents 1 through 16 and permanent ground displacement of the west riverbank. Conceptual seismic mitigation alternatives at Bents 1 through 16 may include supporting the enlarged footings on micropiles or ground improvement. Ground improvement may be required at the west riverbank to mitigate the potential permanent ground displacement hazard. Based on the site conditions and limited overhead clearance to work under the existing bridge, ground improvement using jet grouting may be the preferred seismic mitigation alternative at the west approach. In our opinion, supporting the enlarged footings at Bents 1 through 16 using micropiles with no ground improvement is not preferred due to potential lateral stability issues (i.e. buckling of the micropiles) within the liquefied soils.

We recommend that ground improvement at Bents 1 through 16 be performed underneath the enlarged portion of the spread footings and around the retrofitted footings with low-overhead jet grouting equipment to form a cellular soil-cement ground improvement zone. The cellular soil-cement ground improvement zone at each bent would consist of longitudinal “panels” in front and behind the bent that are connected by transverse “struts” between the footings. We assumed that ground improvement at the west riverbank would be performed from the west side of Bent 19 to the east side of Pier 1 with low-overhead jet grouting equipment to form a soil-cement ground improvement zone. We understand removal of the existing seawall will be performed under the bridge and extend to approximately 10 feet on either side of the bridge. The excavation to remove the existing seawall could be made with an open cut or a temporary shoring wall may be constructed if an open cut is not feasible due to existing utilities or other issues. Temporary shoring on the riverside of the seawall excavation will be provided by a cofferdam constructed in front of Pier 1. The existing seawall is supported on vertical and battered timber piles as shown on the Burnside Bridge Sketch showing Harbor Wall west of Pier No. 1, dated July 1925 and included in Appendix A. The existing timber piles would remain in place and be encapsulated within the cellular soil-cement panels and struts.

To develop conceptual-level cost estimate information, we estimated the lateral and vertical extents of potential cellular soil-cement ground improvement at the west approach. For the purpose of the conceptual level cost estimate, we used liquefiable layer thicknesses of 25 feet under Bents 1 through 16, and 60 feet at the west riverbank. We assumed a cellular soil-cement ground improvement width of 25 feet and length of 120 feet at each bent location (Bents 1 through 16), not including the area under the existing spread footings. We estimated a cellular soil-cement ground improvement width of 90 feet and length of 100 feet at the west riverbank. The estimated extents of cellular soil-cement ground improvement at the west riverbank are shown on Figure 10, Conceptual Ground Improvement Extents for Lateral Spread Mitigation.

10.1.3 Main Span (Piers 1-4 Retrofit)

At the time of this report, we understand the existing pile caps at Piers 1 through 3 will be enlarged and retrofitted with drilled shafts. Pier 1 will be supported by six 7-foot diameter drilled shafts, and Piers 2 and 3 will be supported by 24 12-foot diameter drilled shafts. The current preferred option for Pier 4 is to construct a new pier supported on two 10-foot diameter drilled shafts. We understand the new Pier 4 will be located approximately 30 feet west of the existing location. Seismic mitigation may be required at the west and east riverbanks to mitigate the potential permanent ground displacement hazard at Piers 1 and 4, respectively. Conceptual seismic mitigation alternatives to mitigate the potential permanent ground displacement hazard at Piers 1 and 4 are discussed in Sections 10.1.2 and 10.1.4, respectively.

10.1.4 East Approach (Bents 23-35 Retrofit)

At the time of this report, we understand the existing spread footings at Bents 28 through 35 will be enlarged, and the existing pile group foundations at Bents 25 through 27 will be retrofitted by constructing a “superbent” supported by two 8-foot diameter drilled shafts at each bent. These superbents would also be used to support the bridge widening. We also understand Bents 21 through 24 will be removed entirely and replaced with a three-span structure between Pier 4 and Bent 25. The two new bents between Pier 4 and Bent 25 are currently designated Bent 23 and 24. Both new bents will be supported on four 10-foot diameter drilled shafts.

Seismic mitigation will be required to mitigate permanent ground displacement of the east riverbank and at the approach spans further inland up to Bent 26. Based on the site conditions and limited overhead clearance, ground improvement using jet grouting may be the preferred seismic mitigation alternative at the east riverbank.

We assumed that ground improvement at the east riverbank and approach spans would be performed using low-overhead jet grouting equipment to form four cellular soil-cement ground improvement zones:

1. **At the east riverbank, from Pier 4 extending approximately 120 feet west into the river.** We assumed a liquefiable layer/soil strength reduction layer thickness of 120 feet adjacent to Pier 4 and 55 feet at the west side of the zone, and a length of 120 feet. The cellular soil-cement ground improvement in front of Pier 4 would be performed from a floating barge which would require removal of a portion of the Eastbank Esplanade for equipment access and construction of a temporary sheet pile cofferdam to prevent grout seepage into the river.
2. **Between existing Bents 22 and 23, in the area of an ODOT-owned access road.** We assumed a liquefiable/soil strength reduction layer thickness of 140 feet, a length of 110 feet, and a width ranging from 50 feet at the ground surface to 120 feet at depth.
3. **Between existing Bents 24 and 25, in an area between two existing commercial buildings.** We assumed a liquefiable/soil strength reduction layer thickness of 140 feet, a length of 110 feet, and a width ranging from 45 feet at the ground surface to 85 feet at depth.
4. **At existing Bent 26, in an area between two existing commercial buildings**. We assumed a liquefiable/soil strength reduction layer thickness of 130 feet, a length of 110 feet, and a width ranging from 45 feet at the ground surface to 85 feet at depth.

The estimated extents of cellular soil-cement ground improvement at the east riverbank are shown on Figure 10, Conceptual Ground Improvement Extents for Lateral Spread Mitigation.

10.2 Short-Span and Couch Extension Replacement Alternative

At the time of this report, we understand that the current Short-span Alternative and Couch Extension plans includes supporting the main bridge structure on a drilled shaft foundation system distributed over 14 bents, including two bascule or lift piers in the river. The north branch of the Couch Extension will be supported on six additional bents, designated Bent N10 through N15, also supported on drilled shafts. We assume that the drilled shafts will extend through the potentially liquefiable layers and be founded in the competent Troutdale Formation. The drilled shafts would be required to accommodate downdrag loads caused by liquefaction-induced settlements and provide adequate uplift resistance. Additionally, our analyses indicate potential flow failures at the west and east banks and large permanent ground displacements further inland that could cause significant damage to drilled shafts of any practical dimension. Therefore, seismic mitigation may be required to mitigate the large-scale lateral ground displacement hazards anticipated at the west and east riverbanks, and within the thick potentially liquefiable deposits between the east riverbank and existing

Bent 27 (approximately 85 feet east of proposed Bent 12). Large lateral soil displacements are also anticipated within the river channel. However, because the sand alluvium is expected to liquefy during the seismic event, we understand the shafts at proposed Bents 7 and 8 can be designed to resist the lateral loads caused by the laterally displaced soil. Therefore, we assumed that ground improvements will not be required at proposed Bents 7 and 8. The proposed Short-span Alternative and Couch Extension described in this report are shown on Figure 11. Note that the profile in Figure 11 does not show the north branch of the Couch Extension (i.e. Bents N10 through N15).

10.2.1 Seismic Mitigation and Ground Improvement Strategy

As discussed above, ground improvement methods include excavation and replacement, soil densification (e.g., vibro-compaction, deep dynamic compaction), drainage (e.g., EQ Drain), soil cementation (e.g., jet grouting, deep soil mixing) or a combination of methods such as soil densification and drainage (e.g., stone columns) or soil densification and cementation (e.g., compaction grouting). The selection of an appropriate mitigation method(s) for a particular site depends on factors such as soil type (fines content, organic content, pH, etc.), site access, right-of-way constraints, cost, environmental concerns, and vibration impacts on existing facilities, among others.

In our opinion, the critical factors for developing a ground improvement strategy at the site include:

1. The anticipated depth of potentially liquefiable soils;
2. The engineering properties required from the improved soil mass; and
3. Existing timber pile groups located within anticipated extents of ground improvement.

In our opinion, a soil cementation strategy is required to mitigate the potentially liquefiable soils below the bridge alignment and create an improved soil mass capable of resisting lateral spreading forces. We evaluated the advantages and disadvantages for deep soil mixing and jet grouting strategies for the Short-span Alternative and Couch Extension. A summary of our evaluation is presented in Exhibit 10-2.

Exhibit 10-2: Comparison Between Viable Ground Improvement Strategies for Short-span Alternative and Couch Extension

Alternative	Advantages	Disadvantages
Deep Soil Mixing Consists of mechanical blending of grout and in situ soil using a soil mixing tool such as an auger.	- Lower relative cost; - Lower environmental impacts (no chance of fracking out, surface spoils containment etc.); - More competitive bidding.	- Requires relatively high overhead clearance for equipment; - Becomes difficult in areas with underground obstructions, such as existing timber piles; - Cannot be performed at an inclination away from vertical; - Performed from ground surface to depth using a top to bottom approach.
Jet Grouting Uses high velocity jets of slurry grout to erode and mix in situ soils.	- Effective in almost all soil types; - Low headroom and highly mobile equipment available; - Can be performed at inclinations away from vertical; - Can be performed within specific isolated soil layers using a bottom-top approach; - Can improve soil mass around existing foundation elements.	- Higher relative cost; - Generates a relatively large volume of construction spoils which require removal and disposal.

In general, we believe that jet grouting is the single most viable ground improvement strategy for the entire proposed Short-span and Couch Extension alignments. Jet grouting can be performed between existing foundation elements that would be left in place after removal of the existing structure. Furthermore, jet grouting is likely highly effective in all the soil types we anticipate along the bridge alignment and may be more feasible to perform within spatially constrained areas, especially along the east approach. However, if the ground improvement area is unlikely to encounter subsurface obstructions and additional subsurface explorations indicate suitable soil types, deep soil mixing may be a viable strategy for some areas along the proposed alignment.

We developed our recommended ground improvement strategy using the 2D FLAC model described in Section 8. Ground improvement zones were added to the model and dimensions were iterated to determine the minimum anticipated ground improvements required to achieve tolerable displacements at each bent (except proposed Bents 7 and 8). We developed our improved soil mass material parameters assuming all ground improvements will be completed using jet grouting. However, the material properties are similar to those that could likely be achieved using deep soil mixing methods. For this analysis, we applied the ground motion identified to produce the largest lateral soil displacements. Existing foundation elements were not included in the model since they would not materially impact the behavior of the soil mass.

The following sections present our conceptual-level design recommendations for seismic mitigation consistent with the proposed bridge replacement strategy as we understand it at the time of this report. Figures E169 and E170 show the results of the of the 2D FLAC model with our recommended ground improvements as contours of deformation, excess pore pressure, and time to liquefaction along the entire bridge alignment for the “worst-case” ground motions. Figures E207 through E220 present profile results of the 2D FLAC model at each bent location.

10.2.2 West Approach (Proposed Bents 1-6)

We understand Bents 1 through 6 will be supported on drilled shafts founded below the potentially liquefiable layers. We understand that Bents 1 through 5 will be designed to accommodate anticipated downdrag loads. Seismic mitigation will be required at the west riverbank from proposed Bent 6 to the east side of existing Pier 1.

For the purpose of the conceptual level cost estimate and our design recommendations, we assumed a cellular soil-cement ground improvement zone with a width of 90 feet, a length of 100 feet, and a maximum thickness of 70 feet at the west riverbank. These are the same dimensions as our recommended ground improvement zone for the Retrofit strategy. The estimated extents of cellular soil-cement ground improvement at the west riverbank are shown on Figure 10, Conceptual Ground Improvement Extents for Lateral Spread Mitigation.

10.2.3 Main Span (Proposed Bents 7 and 8)

At the time of this report, we understand Bents 7 and 8 will each be supported on 18 12-foot diameter drilled shafts. Based on conversations with HDR, we understand the drilled shafts will be designed to accommodate lateral soil displacements and downdrag loads caused by liquefaction-induced settlement. Therefore, we assume that ground improvement will not be necessary at Bents 7 and 8.

10.2.4 East Approach (Proposed Bents 9-14/S14 and N10-N15)

We understand Bents 9 through 14 (Short-span Alternative), Bents 9 though S14 (south branch of the Couch Extension along E Burnside Street), and Bents N10 through N15 (north branch of the Couch Extension) are supported on drilled shafts founded below potentially liquefiable layers.

Seismic mitigation will be required to mitigate permanent ground displacements at Bents 9 through 12/S12 and N10 through N12. Based on the site conditions and limited overhead clearance, ground improvement using jet grouting may be the preferred seismic mitigation alternative at the east riverbank. We assumed that ground improvement at the east riverbank and approach spans would be performed using low-overhead jet grouting equipment to form cellular soil-cement ground improvement zones:

1. **At the east riverbank, from Bent 9 extending approximately 110 feet west into the river.** We assumed a liquefiable layer/soil strength reduction layer thickness of 120 feet adjacent to Bent 9 and 55 feet at the west side of the zone, and a length of 120 feet. The cellular soil-cement ground improvement in front of Bent 9 would be performed from a floating barge which would require removal of a portion of the Eastbank Esplanade for equipment access and construction of a temporary sheet pile cofferdam to prevent grout seepage into the river.
2. **At Bent 10/S10 and Bent N10, in the area of an ODOT-owned access road.** We assumed a liquefiable/soil strength reduction layer thickness of 140 feet, a length of 110 feet, and a width ranging from 45 feet at the ground surface to 85 feet at depth.
3. **At Bent 11/S11, in an area between two existing commercial buildings; at Bent N11, in the footprint of an existing building.** We assumed a liquefiable/soil strength reduction layer thickness of 140 feet, a length of 110 feet, and a width ranging from 45 feet at the ground surface to 85 feet at depth.
4. **At Bent 12/S12, in an area between two existing commercial buildings; at Bent N12, in the footprint of an existing building.** We assumed a liquefiable/soil strength reduction layer thickness of 130 feet, a length of 110 feet, and a width ranging from 45 feet at the ground surface to 85 feet at depth.

The dimensions and locations of the recommended ground improvement zones for the Short-span Alternative are the same as those for the Enhanced Retrofit strategy. For the Couch Extension we recommend the additional zones at N10 through N12, as noted above. The estimated extents of cellular soil-cement ground improvements at the east approach are shown on Figure 10, Conceptual Ground Improvement Extents for Lateral Spread Mitigation. The three additional ground improvement zones at Bents N10 through N12 are not shown but are assumed to be located at the respective proposed bent locations.

10.3 Long-span Replacement Alternative

At the time of this report, we understand that the current Long-span Alternative plans includes supporting the bridge structure on a drilled shaft foundation system distributed over 10 bents, including two bascule piers in the river. We assume that the drilled shafts will extend through the potentially liquefiable layers and be founded in the competent Troutdale Formation. The drilled shafts would be required to accommodate downdrag loads caused by liquefaction-induced settlements and provide adequate uplift resistance. Our analyses indicate potential flow failures at the west and east banks and large permanent ground displacements further inland that could cause significant driving forces on the proposed drilled shafts. We understand a goal of the Long-span Alternative is to bridge over the potential ground displacements at the west and east approaches by incorporating an approximately 490-foot span between proposed Bent 5 and proposed Bent 6 (near existing Bent 17 to Pier 2) and an approximately 775-foot span between proposed Bent 7 and proposed Bent 8 (existing Pier 2 to Bent 26). Therefore, this alternative may significantly reduce the amount of ground improvement required along the proposed bridge alignment as compared with the other alternatives. However, our analyses indicate that ground improvements may still be required to mitigate large permanent ground displacements at proposed Bent 8. Large lateral soil displacements are also anticipated within the river channel. However, because the sand alluvium is expected to liquefy during the seismic event, we understand the shafts at proposed Bents 6 and 7 can be designed to resist the lateral loads caused by the laterally displaced soil. Therefore, we assumed that ground improvements will not be required at proposed Bents 6 and 7. The proposed Long-span Alternative described in this report is shown on Figure 12.

10.3.1 Seismic Mitigation and Ground Improvement Strategy

As discussed above, ground improvement methods include excavation and replacement, soil densification (e.g., vibro-compaction, deep dynamic compaction), drainage (e.g., EQ Drain), soil cementation (e.g., jet grouting, deep soil mixing) or a combination of methods such as soil densification and drainage (e.g., stone columns) or soil densification and cementation (e.g., compaction grouting). The selection of an appropriate mitigation method(s) for a particular site depends on factors such as soil type (fines content, organic content, pH, etc.), site access, right-of-way constraints, cost, environmental concerns, and vibration impacts on existing facilities, among others.

In our opinion, the critical factors for developing a ground improvement strategy at Bent 8 include:

1. The anticipated depth of potentially liquefiable soils;
2. The engineering properties required from the improved soil mass; and
3. Existing timber pile groups located within anticipated extents of ground improvement.

In our opinion, a soil cementation strategy is required to mitigate the potentially liquefiable soils below the bridge alignment and create an improved soil mass capable of resisting lateral spreading forces. We evaluated the advantages and disadvantages for deep soil mixing and jet grouting strategies for the Long-span Alternative. A summary of our evaluation is presented in Exhibit 10-3.

Exhibit 10-3: Comparison Between Viable Ground Improvement Strategies for Long-span Alternative

Alternative	Advantages	Disadvantages
Deep Soil Mixing Consists of mechanical blending of grout and in situ soil using a soil mixing tool such as an auger.	- Lower relative cost; - Lower environmental impacts (no chance of fracking out, surface spoils containment etc.); - More competitive bidding.	- Very difficult in areas with underground obstructions, such as existing timber piles; - Cannot be performed at an inclination away from vertical; - Performed from ground surface to depth using a top to bottom approach.
Jet Grouting Uses high velocity jets of slurry grout to erode and mix in situ soils.	- Effective in almost all soil types; - Highly mobile equipment available; - Can be performed at inclinations away from vertical; - Can be performed within specific isolated soil layers using a bottom-top approach; - Can improve soil mass around existing foundation elements.	- Higher relative cost; - Generates a relatively large volume of construction spoils which require removal and disposal.

In general, we believe that jet grouting is the most viable ground improvement strategy for the Long-span Alternative. Jet grouting can be performed between existing foundation elements and around other facilities that may be left in place after removal of the existing bridge structure. Furthermore, jet grouting is likely highly effective in the soil types we anticipate at Bent 8. We will further evaluate the ground improvement alternatives after we complete the field explorations during the final design phase.

We developed our recommended ground improvement strategy using the 2D FLAC model described in Section 8. A ground improvement zone was added to the model and dimensions were iterated to determine the minimum anticipated ground improvements required to achieve tolerable displacements at Bent 8. We developed our improved soil mass material parameters assuming all ground improvements will be completed using jet grouting. However, the material properties are similar to those that could likely be achieved using deep soil mixing methods. For this analysis, we applied the ground motions identified to produce the largest lateral soil displacements. Existing foundation elements were not included in the model since they would not materially impact the behavior of the soil mass.

The following sections present our conceptual-level design recommendations for seismic mitigation consistent with the proposed bridge replacement strategy as we understand it at the time of this report. Figures E221 and E222 show the results of the of the 2D FLAC model with our recommended ground improvements as contours of deformation, excess pore pressure, and time to liquefaction along the entire bridge alignment for the “worst-case” ground motions. Figures E223 through E232 present profile results of the 2D FLAC model at each bent location.

10.3.2 West Approach (Proposed Bents 1-5)

We understand Bents 1 through 5 will be supported on drilled shafts founded below the potentially liquefiable layers and will be designed to accommodate anticipated downdrag loads. Since the intent of the Long-span Alternative is to bridge over the anticipated soil displacements at the west riverbank, we assume seismic mitigation will not be required at the west approach.

10.3.3 Main Span (Proposed Bents 6 and 7)

At the time of this report, we understand Bents 6 and 7 will each be supported on 18 12-foot diameter drilled shafts. Based on conversations with HDR, we understand the drilled shafts will be designed to accommodate lateral soil displacements and downdrag loads caused by liquefaction-induced settlement. Therefore, we assume that ground improvement will not be necessary at Bents 6 and 7.

10.3.4 East Approach (Proposed Bents 8-10)

We understand Bents 8 through 10 are supported on drilled shafts founded below potentially liquefiable layers. Seismic mitigation will be required to mitigate permanent ground displacements at Bent 8. Based on the site conditions, ground improvement using jet grouting may be the preferred seismic mitigation alternative at the east approach. We assumed that ground improvement would be performed using jet grouting methods to form an “island” of cellular soil-cement ground improvement around Bent 8. We assumed a liquefiable/soil strength reduction layer thickness of 130 feet, a length of 110 feet, and a width of 100 feet. The estimated extents of cellular soil-cement ground improvements at the east approach are shown on Figure 13, Conceptual Ground Improvement Extents for Lateral Spread Mitigation.

11 FOUNDATION RESISTANCE FOR BRIDGE ENHANCED RETROFIT ALTERNATIVE

We developed foundation modeling parameters for the preferred retrofit and seismic mitigation alternatives presented in Section 10. The post-seismic/reduced strength foundation modeling parameters consider soil strength reductions assuming full liquefaction of potentially liquefiable layers as determined from our FLAC analysis.

11.1 Spread Footings

As discussed in Section 10, the preferred retrofit and seismic mitigation alternative for the existing spread footings (except Bent 17) is to enlarge all the footings and perform cellular soil-cement ground improvement at Bents 1 through 16. No ground improvements are anticipated below the foundations at Bents 28 through 35. Exhibit 11-1 provides a summary of the proposed retrofitted footing dimensions, footing embedment and elevations, and bearing material based on the preferred retrofit and seismic mitigation alternative.

Exhibit 11-1: Summary of Spread Footing Foundations for Preferred Retrofit and Seismic Mitigation Alternative

Location	Number of Footings	Footing Dimensions (W x L x H) (ft)	[a]Approximate Bottom of Footing Elevation (ft)	Approximate Footing Embedment (ft)	[b]Bearing Material
Bent 1	1	10’ x 110’	24.5	5	Soil-Cement / Fine-Grained Alluvium
Bent 2	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 3	4	Exterior: 12.5’x 12.5’ x 4’ Interior North: 13.5’ x 13.5’ x 8’ Interior South: 13.5’ x 13.5’ x 4’	Exterior: 22 Interior North: 17 Interior South: 22	7	Soil-Cement / Fine-Grained Alluvium
Bent 4	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 5	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 6	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 7	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 8	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 9	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 10	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 11	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 12	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 13	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	22	7	Soil-Cement / Fine-Grained Alluvium
Bent 14	4	Exterior: 14’ x 14’ x 4’ Interior: 17.5’ x 17.5’ x 6’	22	9	Soil-Cement / Fine-Grained Alluvium
Bent 15	4	Exterior: 14’ x 14’ x 4’ Interior: 17.5’ x 17.5’ x 6’	22	9	Soil-Cement / Fine-Grained Alluvium
Bent 16	4	Exterior: 14’ x 14’ x 4’ Interior: 17.5’ x 17.5’ x 6’	22	9	Soil-Cement / Fill
Bent 28	3	16’ x 16’ x 4’	22	27	Fine-Grained Alluvium
Bent 29	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	40	10	Fill
Bent 30	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	40	10	Fill
Bent 31	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	40	10	Fill
Bent 32	4	Exterior: 12.5’ x 12.5’ x 4’ Interior: 13.5’ x 13.5’ x 4’	40	10	Fill
Bent 33	4	Exterior: 14’ x 14’ x 4’ Interior: 17.5’ x 17.5’ x 6’	37	12	Fill
Bent 34	4	Exterior: 14’ x 14’ x 4’ Interior: 17.5’ x 17.5’ x 6’	37	12	Fill
Bent 35	1	9.25’ x 110’	41	9	CFD – Channel Facies

NOTES:

a. Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set.

b. Bearing material is interpreted from the information in the plan set, existing borings, current borings, and the preferred seismic mitigation alternative.

11.1.1 Bearing Resistance

We estimated the nominal post-seismic/reduced strength bearing resistance for the retrofitted spread footings by performing a conventional spread footing evaluation. For this evaluation, the enlarged portions of the footings at Bents 1 through 16 are assumed to be founded on cellular soil-cement columns. The nominal bearing resistance is provided in Exhibit 11-2. The bearing resistances reported in the table are nominal geotechnical resistances and should be reduced by a resistance factor of 1.0 for the extreme event limit state.

11.1.2 Subgrade Stiffness

We understand that the seismic performance of the retrofitted footings will be modeled using equivalent six degree of freedom springs. The spring constants will be developed using the recommended procedures in the ODOT BDM or the FHWA Seismic Retrofitting Manual for Highway Structures. Exhibit 11-2 presents the recommended values for the required information to fully describe spring stiffness, including bearing material shear modulus, Poisson's ratio, and nominal bearing resistance. In Exhibit 11-2, we have provided bearing material initial shear modulus (maximum modulus) for the post-seismic condition. We understand that the structural engineer will develop the necessary large strain shear modulus values based on the ODOT BDM or the FHWA Seismic Retrofit Manual. In general, we recommend that the strain calculated in the structural analyses be checked against the strain assumed in selecting the shear modulus. The structural engineer may need to iterate their analyses using a different strain-compatible shear modulus. The Poisson's ratio is constant for the purposes of the evaluation.

11.1.3 Sliding Resistance

Sliding resistance for a spread footing may be developed through friction on the base of the footing and passive earth pressures on the face of the footing. The nominal friction resistance can be expressed as the vertical load (i.e., actual footing pressure) multiplied by a coefficient of friction (tan δ). Sliding resistance generated by the lateral passive earth pressure acting on the face of the footing can be assumed to be developed if the footing is free to translate horizontally. If movement of the footing is limited, the earth pressure resistance values should be reduced to reflect the reduced footing movement based on the FHWA Seismic Retrofit Manual.

We estimated the nominal post-seismic/reduced strength frictional sliding coefficient for the retrofitted footings; the results are presented in Exhibit 11-2 in terms of tan δ. A sliding resistance factor of 1.0 should be used for the extreme event limit state.

The passive earth pressures we developed for the post-seismic/reduced strength condition are also presented in Exhibit 11-2 in terms of equivalent fluid pressure and depth of footing (D, in feet). These earth pressure values may be used to estimate the lateral resistance of footings. A passive pressure resistance factor of 1.0 should be used for the extreme event limit design case.

Exhibit 11-2: Recommended Unfactored Post-Seismic/Reduced Strength Soil Parameters for Spread Footings and Pile Caps for Preferred Retrofit and Seismic Mitigation Alternative

	Location	[a]Approx. Footing Elev. (ft) (depth below ground surface, ft)	[b]Soil Type	Total Unit Weight, γ (pcf)	Friction Angle, Φ (degrees)	Cohesion, c (psf)	Q_{nom} (ksf)	Nominal Sliding Coeff, **tan δ**	[e]Bearing Material Initial Shear Modulus, (ksi)	Poisson's Ratio	Lateral Earth Coefficients			[h]Lateral Earth Pressures (psf)		
											[f]K_o	[f]K_a	K_p	[i]EFP_o	[i]EFP_a	[i]EFP_p
Abutments	Bent 1	24.5 (5)	Soil-Cement / Fine-Grained Alluvium	120	---	6,500	8	0.44	11	0.3	0.52	0.35	2.88	57H 57D	39H 39D	317H 317D
	Bent 35	41 (9)	CFD Channel Facies	130	36	---	9	0.58	18	0.35	0.52	0.35	2.88	57H 57D	39H 39D	317H 317D
Footings	Bents 2 through 13	22 (7)	Soil-Cement / Fine-Grained Alluvium	120	---	6,500	15	0.44	11	0.3	0.52	0.35	2.88	57D	39D	317D
	Bents 14 and 15	22 (7)	Soil-Cement / Fine-Grained Alluvium	120	---	6,500	11	0.44	11	0.3	0.52	0.35	2.88	57D	39D	317D
	Bent 16	22 (9)	Soil-Cement / Fill	120	---	6,500	11	0.44	11	0.3	0.52	0.35	2.88	57D	39D	317D
	Bent 28	22 (27)	Gravel Alluvium (assumed)	110	29	---	8	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
	Bents 29 through 32	40 (10)	Gravel Alluvium (assumed)	110	29	---	8	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
	Bents 33 and 34	37 (12)	Gravel Alluvium (assumed)	110	29	---	8	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
Pile Caps	Bent 17	17 (13)	Fill	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	57D	39D	317D
	Bents 18 and 19	13 – 15 (18)	Fill	110	29	---	--[-c]	---[d]	---	---	0.52	0.35	2.88	57D	39D	317D
	Pier 1[j]	-41.6 (17)	Fill / Fine-Grained Alluvium	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	57D	39D	317D
	Piers 2 and 3[k]	-70 (16)	Sand Alluvium	125	10	---	---[c]	---[d]	---	---	0.3	0.3	---[g]	19D	19D	---[g]
	Bents 25 through 27	20.5 (14.5)	Fill	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	57D	39D	317D

NOTES:

* Groundwater is assumed to be at an elevation of 20 feet based on existing borings and Willamette River OHW level.

a. Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set. Indicates proposed bottom of pile cap elevation for Bents 17 through 19, Piers 1 through 4, and Bents 21 through 27.

b. Soil type refers to bearing material for abutments and footings, and retained soil for pile caps.

c. Pile caps should not be assumed to provide bearing resistance.

d. Pile caps should not be assumed to develop lateral resistance from base friction.

e. Initial shear modulus values are estimated from shear wave velocity measurements, ODOT GDM Table 6-2 (Seed, et al.), and typical values for soil-cement.

f. For liquefied soil, active and at-rest lateral earth coefficient of 0.3 is estimated in accordance with ODOT GDM.

g. Liquefied soil is not assumed to provide passive resistance.

h. For abutments, D is the minimum embedment of the abutment wall measured from the ground surface to the bottom of the wall footing and H is the height of the retained soil behind the abutment wall; D or H will be used to determine lateral earth pressures on the abutment wall depending on the direction of loading. For footings and pile caps, D is the minimum embedment of the footing or pile cap measured from the ground surface to the bottom of the footing or pile cap.

i. Post-seismic/reduced strength lateral equivalent fluid pressures - Assume a triangular pressure distribution.

j. For Pier 1, due to unbalanced retained soil height in the longitudinal direction, add 55 feet to pile cap embedment (D) when calculating lateral earth pressures against the west (upslope) side of the pile cap.

k. When considering seismically-induced lateral soil displacements at Piers 2 and 3, apply a lateral earth pressure distribution as shown on Figure 14.

11.2 Drilled Shafts

As discussed in Section 10, the preferred retrofit and seismic mitigation alternative for the existing pile group foundations and the spread footing foundations at Bent 17 is to retrofit the foundations with drilled shafts and perform cellular soil-cement ground improvement at the west and east approaches. We understand Bents 17 through 19 and 25 through 27 may be retrofitted by constructing a "superbent" supported by two drilled shafts at each bent that are connected by a grade beam or infill wall that is also tied into the existing spread footings or pile caps. The existing pile caps at Piers 1 through 3 will be enlarged and retrofitted with drilled shafts. Pier 1 will be supported by six drilled shafts, and Piers 2 and 3 will be supported by 24 drilled shafts. We understand the current preferred retrofit option for Pier 4 involves the construction of a new pier to the west of the existing Pier 4 location which will be supported by two drilled shafts. As described in Section 10, we understand that existing Bents 21 through 24 will be demolished and replaced by two new Bents designated Bents 23 and 24. The new Bents 23 and 24 will each be supported on four drilled shafts. Exhibit 11-3 provides a summary of the proposed number of shafts, shaft diameter, and pile cap/grade beam elevation at each bent/pier location based on the preferred retrofit alternative.

Exhibit 11-3: Summary of Drilled Shaft Group Foundations and Estimated Downdrag Loads for Preferred Retrofit and Seismic Mitigation Alternative

Location	Number of Shafts	Shaft Diameter (ft)	[a]Assumed Bottom Pile Cap/Grade Beam Elevation (ft)	Estimated Post-seismic/reduced strength Downdrag Load (kips/shaft)
Bent 17	2	8	19	130
Bent 18	2	8	14	120
Bent 19 [b]	2	8	13	0
Pier 1 [b]	6	7	-20	0
Pier 2	24	12	-66	90
Pier 3	24	12	-66	90
Pier 4 [b]	2	10	--	0
Bent 23 [b]	4	10	--	0
Bent 24 [b]	4	10	--	0
Bent 25	2	8	21	1590
Bent 26 [b]	2	8	22	0
Bent 27	2	8	22	710

NOTES:

a. Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set.

b. Foundations located in proposed ground improvement zone.

11.2.1 Single Shaft Axial and Uplift Resistance

We developed estimates of axial resistance with depth for individual shafts at Bents 17 through 19 and Pier 4 through Bent 27 in general accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD). Engineering parameters for the shaft resistance evaluation were based on our characterization of subsurface materials, subsurface explorations, our interpretation of the available subsurface information, and FLAC analysis.

Plots of nominal side resistance, nominal base (tip) resistance, and factored total compressive capacities are provided for the AASHTO LRFD Strength Limit and Extreme Event Limit states in Appendix G. For the Extreme Event Limit state, we provided resistances for the non-liquefied, liquefied/reduced strength, and post-seismic "downdrag" cases. The factored compression total capacities shown on the plots have incorporated the applicable Limit State resistance factors specified by AASHTO LRFD (see Note 2 on the figures). However, they do not include Group Reduction Factors that may be required for bearing resistance based on shaft center-to-center spacing. Group reduction factors should be applied to the factored compression total values based on AASHTO LRFD Table 10.8.3.6.3-1.

Uplift resistance can be determined by multiplying the nominal side resistance by a factor of 0.45 or 0.8 for the Strength Limit and Extreme Event Limit, respectively. For shaft groups, the nominal side resistance is equal to the lesser of the sum of the individual nominal side resistance or, the uplift resistance of the shaft group considered as a block, per AASHTO LRFD Section 10.7.3.11. The weight of the block that will be uplifted is determined using a spread of load of 1H in 4V from the base of the shaft group, as illustrated in AASHTO LRFD Figure 10.7.3.11-1.

Drilled shafts located within liquefiable soils outside of the ground improvement zones will experience post-seismic downdrag loads due to liquefaction-induced settlement. We have estimated the unfactored single shaft post-seismic downdrag loads and provided them in Exhibit 11-3. Estimated downdrag loads are also included in Note 6 of the axial resistance figures. A load factor of 1.0 is recommended to be applied to this post-seismic downdrag load. Downdrag loads should be applied using the Extreme Event post-seismic downdrag resistance curves.

11.2.2 Single Shaft Lateral Resistance

We understand the design team is using the computer program LPILE, developed by Ensoft, Inc., to perform lateral resistance analyses of the proposed shafts at Bents 17 through 19 and Pier 4 through Bent 27. Lateral soil parameters for static and post-seismic/reduced strength cases are included in Appendix G. Lateral resistance reduction factors (P-multipliers) to account for multiple rows with a center-to-center shaft spacing of five times the shaft diameter or less should be applied to the generated P-Y springs per AAHSTO LRFD Table 10.7.2.4-1. Note that the provided lateral soil resistance parameters are independent of shaft diameter.

We also typically provide lateral soil displacement profiles which can be directly entered into LPILE analyses to model earth pressures caused by laterally displaced ground. However, the results of our preliminary ground improvement models indicate that there are negligible lateral displacements at each bent/pier location, except at Piers 2 and 3. See Section 11.2.3 for our recommendations for modeling lateral soil displacements at Piers 2 and 3.

11.2.3 Piers 1, 2, and 3 Drilled Shaft Group Evaluation

We understand the design team is using the computer program FB-Multipier (developed by Bridge Software Institute) to perform analyses of the proposed shaft groups at Piers 1 through 3. We developed soil resistance input parameters for FB-Multipier analyses at Piers 1 through 3, including soil springs for the shaft base (tip) resistance (Q-Z springs), shaft side resistance (T-Z springs), and parameters used by the program to generate lateral resistance springs (P-Y springs). The values provided for Piers 1 through 3 are nominal, unfactored values for the Extreme Event Limit liquefied case. Per AASHTO LRFD, Q-Z spring values should use a resistance factor of 1.0 for the Extreme Event Limit. Group reduction factors for bearing resistance should be applied to the factored Q-Z and T-Z spring values based on AASHTO LRFD Table 10.8.3.6.3-1. Lateral resistance reduction factors (P-multipliers) to account for multiple rows with a center-to-center shaft spacing of five times the shaft diameter or less should be applied to the generated P-Y springs per AAHSTO LRFD Table 10.7.2.4-1. Plots of the soil springs are presented in Figures G10 and G11 in Appendix G. Lateral soil (P-Y) parameters are included in Appendix G in Tables G7 through G9.

For Piers 2 and 3, we recommend modeling the anticipated lateral soil displacements by applying a lateral stress distribution according to the following method:

1. The lateral pressure distribution should start at mudline and extend to the bottom of the Sand Alluvium layer.
2. The pressure at the bottom of the concrete seal is equal to $35H_s$, where H_s = the vertical distance from mudline to the bottom of the concrete seal.
3. The pressure at the bottom of the Sand Alluvium layer is equal to 35H, where H = the vertical distance from mudline to the bottom of the Sand Alluvium layer.
4. The pressure distribution is assumed to act over the entire width of the pile cap/concrete seal and should also be applied to each shaft in the pile group.

The method described above is presented graphically on Figure 14.

11.3 Earth Pressure on Abutment Walls and Embedded Pile Caps

The lateral earth pressures on a retaining wall, including capacity/stiffness and seismically induced loads, are a function of relative wall and soil displacement. If the wall is allowed to displace (typically 2 percent of the wall height), the static lateral pressures may be developed, assuming active pressures as a load and full passive pressure as a resistance. For seismic lateral pressures, active pressures increase and passive resistances decrease due to inertial effects. If the wall is restrained from moving, seismically induced loads increase and the passive resistances decrease further. If a wall is allowed to displace less than 2 percent, the active earth pressures should be calculated using the full seismic acceleration coefficient (as opposed to one-half of the acceleration coefficient used for walls that are allowed to freely displace), and passive resistance should be taken as a portion of the full seismic value.

We assume that the soil surrounding the various retrofitted abutment walls and pile caps will be allowed to displace at least 2 percent of the wall height and therefore will mobilize full active and passive lateral earth pressures. Liquefied soil should be assumed not to provide any passive resistance. The earth pressure parameters we developed for the post-seismic/reduced strength condition for the retrofitted abutment walls and pile caps are presented in Exhibit 11-2.

12 FOUNDATION RESISTANCE FOR SHORT-SPAN AND COUCH EXTENSION REPLACEMENT ALTERNATIVES

We developed foundation modeling parameters for the Short-span Alternative and Couch Extension considering the seismic mitigation alternatives presented in Section 10. Foundation modeling parameters for the Long-span Alternative are discussed in Section 13. For this phase of the project, we did not perform any subsurface explorations along the north branch of the Couch Extension east approach (Bents N10 through N15). We assumed the subsurface profile along Bents N10 through N15 matched a projection of the subsurface profile along the southern branch of the alignment (Bents S10 through S14). The post-seismic/reduced strength foundation modeling parameters consider soil strength reductions assuming full liquefaction of potentially liquefiable soil layers, based on the results of our FLAC analysis.

12.1 Drilled Shafts

As discussed in Section 10, the current Short-span Alternative and Couch Extension plans includes supporting the main bridge structure along Burnside Street on a drilled shaft foundation system distributed over 14 bents, including two bascule or lift piers in the river. The north branch of the Couch Extension will be supported on six additional bents, designated N10 through N15. The preferred seismic mitigation strategy includes performing ground improvements at the west and east approach. Exhibit 12-1 provides a summary of the proposed number of shafts and shaft diameters at each bent location for the proposed Short-span Alternative and Couch Extension at the time of this report.

Exhibit 12-1: Summary of Drilled Shaft Group Foundations and Estimated Downdrag Loads for Short-span Alternative and Couch Extension with Seismic Mitigation

Location	Number of Shafts	Shaft Diameter (ft)	Estimated Post-Seismic Downdrag Load (kips/shaft)
Bent 1	10	3	70
Bent 2	4	7	100
Bent 3	4	7	100
Bent 4	4	8	170
Bent 5	4	10	180
Bent 6[a]	4	10	0
Bent 7	18	12	90
Bent 8	18	12	90
Bent 9[a]	4	12	0
Bent 10/S10[a]	4	10	0
Bent 11/S11[a]	4	10	0
Bent 12/S12[a]	4	10	0
Bent 13/S13	4	7	0
Bent 14/S14	13	3	0
Bent N10	2	10	0
Bent N11	2	10	0
Bent N12	2	8	0
Bent N13	2	8	0
Bent N14	2	6	0
Bent N15	6	3	0

NOTES:

a. Foundations located in proposed ground improvement zone.

Drilled shafts for the replacement alternative are assumed to extend to existing ground surface.

12.1.1 Single Shaft Axial and Uplift Resistance

We developed estimates of axial resistance with depth for individual shafts at Bents 1 through 6, 9 through 14/S14, and N10 through N15 in general accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD). Engineering parameters for the shaft resistance evaluation were based on our characterization of subsurface materials, subsurface explorations, our interpretation of the available subsurface information, and FLAC analysis.

Plots of nominal side resistance, nominal base (tip) resistance, and factored total compressive capacities are provided for the AASHTO LRFD Strength Limit and Extreme Event Limit states in Appendix H. For the Extreme Event Limit state, we provided resistances for the non-liquefied, liquefied/reduced strength, and post-seismic "downdrag" cases. The factored compression total capacities shown on the plots have incorporated the applicable Limit State resistance factors specified by AASHTO LRFD (see Note 2 on the figures). However, they do not include Group Reduction Factors that may be required for bearing resistance based on shaft center-to-center spacing. Group reduction factors should be applied to the factored compression total values based on AASHTO LRFD Table 10.8.3.6.3-1.

Uplift resistance can be determined by multiplying the nominal side resistance by a factor of 0.45 or 0.8 for the Strength Limit and Extreme Event Limit, respectively. For shaft groups, the nominal side resistance is equal to the lesser of the sum of the individual nominal side resistance or, the uplift resistance of the shaft group considered as a block, per AASHTO LRFD Section 10.7.3.11. The weight of the block that will be uplifted is determined using a spread of load of 1H in 4V from the base of the shaft group, as illustrated in AASHTO LRFD Figure 10.7.3.11-1.

Drilled shafts located within liquefiable soils outside of the ground improvement zones will experience post-seismic downdrag loads due to liquefaction-induced settlement. We have estimated the unfactored single shaft post-seismic downdrag loads and provided them in Exhibit 12-1. Estimated downdrag loads are also included in Note 6 of the axial resistance plots. A load factor of 1.0 is recommended to be applied to this post-seismic downdrag load. Downdrag loads should be applied using the Extreme Event post-seismic downdrag resistance curves.

12.1.2 Single Shaft Lateral Resistance

We understand the design team is using the computer program LPILE, developed by Ensoft, Inc., to perform lateral resistance analyses of the proposed shafts at Bents 1 through

6, 9 through 14/S14, and N10 through N15. Lateral soil parameters for static and post-seismic/reduced strength cases are included in Appendix H. Lateral resistance reduction factors (P-multipliers) to account for multiple rows with a center-to-center shaft spacing of five times the shaft diameter or less should be applied to the generated P-Y springs per AAHSTO LRFD Table 10.7.2.4-1. Note that the provided lateral soil resistance parameters are independent of shaft diameter.

We also typically provide lateral soil displacement profiles which can be directly entered into LPILE analyses to model earth pressures caused by laterally displaced ground. However, the results of our preliminary ground improvement models indicate that there are negligible lateral displacements at each bent location, except at Bents 7 and 8. See Section 12.1.3 for our recommendations for modeling lateral soil displacements at Bents 7 and 8.

12.1.3 Bents 7 and 8 Drilled Shaft Group Evaluation

We understand the design team is using the computer program FB-Multipier (developed by Bridge Software Institute) to perform analyses of the proposed shaft groups at Bents 7 and 8. We developed soil resistance input parameters for FB-Multipier analyses at Bents 7 and 8, including soil springs for the shaft base (tip) resistance (Q-Z springs), shaft side resistance (T-Z springs), and parameters used by the program to generate lateral resistance springs (P-Y springs). The values provided for Bents 7 and 8 are nominal, unfactored values for the Extreme Event Limit liquefied case. Per AASHTO LRFD, Q-Z spring values should use a resistance factor of 1.0 for the Extreme Event Limit. Group reduction factors for bearing resistance should be applied to the factored Q-Z and T-Z spring values based on AASHTO LRFD Table 10.8.3.6.3-1. Lateral resistance reduction factors (P-multipliers) to account for multiple rows with a center-to-center shaft spacing of five times the shaft diameter or less should be applied to the generated P-Y springs per AAHSTO LRFD Table 10.7.2.4-1. Plots of the soil springs are presented in Figure H18 in Appendix H. Lateral soil (P-Y) parameters are included in Appendix H.

For Bents 7 and 8, we recommend modeling the anticipated lateral soil displacements by applying a lateral stress distribution according to the following method:

1. The lateral pressure distribution should start at mudline and extend to the bottom of the Sand Alluvium layer.
2. The pressure at the bottom of the concrete seal is equal to $35H_s$, where H_s = the vertical distance from mudline to the bottom of the concrete seal.
3. The pressure at the bottom of the Sand Alluvium layer is equal to 35H, where H = the vertical distance from mudline to the bottom of the Sand Alluvium layer.

4. The pressure distribution is assumed to act over the entire width of the pile cap/concrete seal and should also be applied to each shaft in the pile group.

The method described above is presented graphically on Figure 14.

12.2 Earth Pressure on Abutment Walls

The lateral earth pressures on a retaining wall, including capacity/stiffness and seismically induced loads, are a function of relative wall and soil displacement. If the wall is allowed to displace (typically 2 percent of the wall height), the static lateral pressures may be developed assuming active pressures as a load and full passive pressure as a resistance. For seismic lateral pressures, active pressures increase and passive resistances decrease due to inertial effects. If the wall is restrained from moving, seismically induced loads increase and the passive resistances decrease further. If a wall is allowed to displace less than 2 percent, the active earth pressures should be calculated using the full seismic acceleration coefficient (as opposed to one-half of the acceleration coefficient used for walls that are allowed to freely displace), and passive resistance should be taken as a portion of the full seismic value.

We assume that the soil surrounding the abutment walls will be allowed to displace at least 2 percent of the wall height and therefore will mobilize full active and passive lateral earth pressures. Liquefied soil should be assumed not to provide any passive resistance. The earth pressure parameters we developed for the post-seismic/reduced strength condition for the abutment walls are presented in Exhibit 12-2.

Exhibit 12-2: Recommended Unfactored Post-Seismic/Reduced Strength Soil Parameters for Abutment Walls

									Lateral Earth Coefficients			[c]Lateral Earth Pressures (psf)		
Location	[a]Soil Type	Total Unit Weight, γ (pcf)	Friction Angle, Φ (degrees)	Cohesion, c (psf)	Q_{nom} (ksf)	Nominal Sliding Coeff. **tan δ**	[b]Bearing Material Initial Shear Modulus, (ksi)	Poisson's Ratio	K_o	K_a	K_p	EFP_o	EFP_a	EFP_p
Bent 1	Fine-Grained Alluvium	110	29	---	4	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
Bent 14/S14/N15	CFD Channel Facies	130	36	---	9	0.58	18	0.35	0.52	0.35	2.88	57H 57D	39H 39D	317H 317D

NOTES:

* Groundwater is assumed to be at an elevation of 20 feet based on existing borings and Willamette River mean water level.

a. Soil type refers to bearing material for abutments.

b. Initial shear modulus values are estimated from shear wave velocity measurements, ODOT GDM Table 6-2 (Seed, et al.).

c. D is the minimum embedment of the abutment wall measured from the ground surface to the bottom of the wall footing and H is the height of the retained soil behind the abutment wall; D or H will be used to determine lateral earth pressures on the abutment wall depending on the direction of loading.

13 FOUNDATION RESISTANCE FOR LONG-SPAN ALTERNATIVE

We developed foundation modeling parameters for the Long-span Alternative considering the seismic mitigation alternatives presented in Section 10. The post-seismic/reduced strength foundation modeling parameters consider soil strength reductions assuming full liquefaction of potentially liquefiable soil layers, based on the results of our FLAC analysis.

13.1 Drilled Shafts

As discussed in Section 10, the current Long-span Alternative plans includes supporting the bridge structure on a drilled shaft foundation system distributed over 10 bents, including two bascule piers in the river. The preferred seismic mitigation strategy includes performing ground improvements near proposed Bent 8 at the east approach. Exhibit 13-1 provides a summary of the proposed number of shafts and shaft diameters at each bent location for the proposed Long-span Alternative at the time of this report.

Exhibit 13-1: Summary of Drilled Shaft Group Foundations and Estimated Downdrag Loads for Long-span Alternative with Seismic Mitigation

Location	Number of Shafts	Shaft Diameter (ft)	Estimated Post-Seismic Downdrag Load (kips/shaft)
Bent 1	10	3	70
Bent 2	4	7	100
Bent 3	4	7	100
Bent 4	4	8	170
Bent 5	8	10	180
Bent 6	18	12	90
Bent 7	18	12	90
Bent 8[a]	8	10	0
Bent 9	4	7	0
Bent 10	13	3	0

NOTES:

a. Foundations located in proposed ground improvement zone.

Drilled shafts for the replacement alternatives are assumed to extend to existing ground surface.

13.1.1 Single Shaft Axial and Uplift Resistance

We developed estimates of axial resistance with depth for individual shafts at Bents 1 through 5 and Bents 8 through 10 in general accordance with the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD). Engineering parameters for the shaft resistance evaluation were based on our characterization of subsurface materials, subsurface explorations, our interpretation of the available subsurface information, and FLAC analysis.

Plots of nominal side resistance, nominal base (tip) resistance, and factored total compressive capacities are provided for the AASHTO LRFD Strength Limit and Extreme Event Limit states in Appendix I. For the Extreme Event Limit state, we provided resistances for the non-liquefied, liquefied/reduced strength, and post-seismic "downdrag" cases. The factored compression total resistances shown on the plots have incorporated the applicable Limit State resistance factors specified by AASHTO LRFD (see Note 2 on the figures). However, they do not include Group Reduction Factors that may be required for bearing resistance based on shaft center-to-center spacing. Group reduction factors should be applied to the factored compression total values based on AASHTO LRFD Table 10.8.3.6.3-1.

Uplift resistance can be determined by multiplying the nominal side resistance by a factor of 0.45 or 0.8 for the Strength Limit and Extreme Event Limit, respectively. For shaft groups, the nominal side resistance is equal to the lesser of the sum of the individual nominal side resistance or, the uplift resistance of the shaft group considered as a block, per AASHTO LRFD Section 10.7.3.11. The weight of the block that will be uplifted is determined using a spread of load of 1H in 4V from the base of the shaft group, as illustrated in AASHTO LRFD Figure 10.7.3.11-1.

Drilled shafts located within liquefiable soils outside of the ground improvement zones will experience post-seismic downdrag loads due to liquefaction-induced settlement. We have estimated the unfactored single shaft post-seismic downdrag loads and provided them in Exhibit 13-1. Estimated downdrag loads are also included in Note 6 of the axial resistance plots. A load factor of 1.0 is recommended to be applied to this post-seismic downdrag load. Downdrag loads should be applied using the Extreme Event post-seismic downdrag resistance curves.

13.1.2 Single Shaft Lateral Resistance

We understand the design team is using the computer program LPILE, developed by Ensoft, Inc., to perform lateral resistance analyses of the proposed shafts at Bents 1 through 5 and Bents 8 through 10. Lateral soil parameters for static and post-seismic/reduced strength cases are included in Appendix I. Lateral resistance reduction factors (P-multipliers) to account for multiple rows with a center-to-center shaft spacing of five times the shaft diameter or less should be applied to the generated P-Y springs per AAHSTO LRFD Table 10.7.2.4-1. Note that the provided lateral soil resistance parameters are independent of shaft diameter.

We also typically provide lateral soil displacement profiles which can be directly entered into LPILE analyses to model earth pressures caused by laterally displaced ground. The results of our preliminary ground improvement models indicate up to six inches of lateral displacement at proposed Bent 5, and significant lateral displacements at Bents 6 and 7. See Table I6 for our recommended displacement profile at proposed Bent 5 and Section 13.1.3 for our recommendations for modeling lateral soil displacements at Bents 6 and 7.

13.1.3 Bents 6 and 7 Drilled Shaft Group Evaluation

We understand the design team is using the computer program FB-Multipier (developed by Bridge Software Institute) to perform analyses of the proposed shaft groups at Bents 6 and 7. We developed soil resistance input parameters for FB-Multipier analyses at Bents 6 and 7, including soil springs for the shaft base (tip) resistance (Q-Z springs), shaft side resistance (T-Z springs), and parameters used by the program to generate lateral resistance springs (P-Y springs). The values provided for Bents 6 and 7 are nominal, unfactored values for the Extreme Event Limit liquefied case. Per AASHTO LRFD, Q-Z spring values should use a resistance factor of 1.0 for the Extreme Event Limit. Group reduction factors for bearing resistance should be applied to the factored Q-Z and T-Z spring values based on AASHTO LRFD Table 10.8.3.6.3-1. Lateral resistance reduction factors (P-multipliers) to account for multiple rows with a center-to-center shaft spacing of five times the shaft diameter or less should be applied to the generated P-Y springs per AAHSTO LRFD Table 10.7.2.4-1. Plots of the soil springs are presented in Figure I8 in Appendix I. Lateral soil (P-Y) parameters are included in Table I7.

For Bents 6 and 7, we recommend modeling the anticipated lateral soil displacements by applying a lateral stress distribution according to the following method:

1. The lateral pressure distribution should start at mudline and extend to the bottom of the Sand Alluvium layer.
2. The pressure at the bottom of the concrete seal is equal to $35H_s$, where H_s = the vertical distance from mudline to the bottom of the concrete seal.
3. The pressure at the bottom of the Sand Alluvium layer is equal to 35H, where H = the vertical distance from mudline to the bottom of the Sand Alluvium layer.
4. The pressure distribution is assumed to act over the entire width of the pile cap/concrete seal and should also be applied to each shaft in the pile group.

The method described above is presented graphically on Figure 14.

13.2 Large-Diameter Caisson Foundation Alternative

Based on the results of our FLAC analysis for the Long-span Alternative ground improvements, in our opinion, the proposed drilled shaft group of eight, 10-foot diameter shafts at proposed Bent 8 could potentially be replaced by a single, large-diameter caisson foundation. In our experience, a large-diameter caisson at proposed Bent 8 may be stiff enough to eliminate the need for ground improvements anywhere along the Long-span Alternative in its current configuration. Additionally, a caisson may be easier to construct than a large drilled shaft group. However, large-diameter caissons must be founded on very stiff, uniform material to avoid differential settlements or bearing capacity failure. Therefore, additional geotechnical explorations and numerical modeling analysis are required before a caisson alternative can be evaluated.

13.3 Earth Pressure on Abutment Walls

The lateral earth pressures on a retaining wall, including capacity/stiffness and seismically induced loads, are a function of relative wall and soil displacement. If the wall is allowed to displace (typically 2 percent of the wall height), the static lateral pressures may be developed assuming active pressures as a load and full passive pressure as a resistance. For seismic lateral pressures, active pressures increase and passive resistances decrease due to inertial effects. If the wall is restrained from moving, seismically induced loads increase and the passive resistances decrease further. If a wall is allowed to displace less than 2 percent, the active earth pressures should be calculated using the full seismic acceleration coefficient (as opposed to one-half of the acceleration coefficient used for walls that are allowed to freely displace), and passive resistance should be taken as a portion of the full seismic value.

We assume that the soil surrounding the abutment walls will be allowed to displace at least 2 percent of the wall height and therefore will mobilize full active and passive lateral earth pressures. Liquefied soil should be assumed not to provide any passive resistance. The earth pressure parameters we developed for the post-seismic/reduced strength condition for the abutment walls are presented in Exhibit 13-2.

Exhibit 13-2: Recommended Unfactored Post-Seismic/Reduced Strength Soil Parameters for Abutment Walls

Location	[a]Soil Type	Total Unit Weight, γ (pcf)	Friction Angle, Φ (degrees)	Cohesion, c (psf)	Q_{nom} (ksf)	Nominal Sliding Coeff. tan δ	[b]Bearing Material Initial Shear Modulus, (ksi)	Poisson's Ratio	Lateral Earth Coefficients			[c]Lateral Earth Pressures (psf)		
									K_o	K_a	K_p	EFP_o	EFP_a	EFP_p
Bent 1	Fine-Grained Alluvium	110	29	---	4	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
Bent 14/S14/N15	CFD Channel Facies	130	36	---	9	0.58	18	0.35	0.52	0.35	2.88	57H 57D	39H 39D	317H 317D

NOTES:

* Groundwater is assumed to be at an elevation of 20 feet based on existing borings and Willamette River mean water level.

a. Soil type refers to bearing material for abutments.

b. Initial shear modulus values are estimated from shear wave velocity measurements, ODOT GDM Table 6-2 (Seed, et al.).

c. D is the minimum embedment of the abutment wall measured from the ground surface to the bottom of the wall footing and H is the height of the retained soil behind the abutment wall; D or H will be used to determine lateral earth pressures on the abutment wall depending on the direction of loading.

14 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If subsurface conditions different from those encountered in the explorations are encountered in future explorations or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations. Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

We developed our opinions of probable construction costs based on our experience with similar projects. The costs include several assumptions, including:

- The subsurface conditions that will be encountered,
- Decisions of other design professionals and government agency personnel,
- The means and methods of construction the Contractor will employ,
- The Contractor’s techniques in determining price and market conditions at the time of construction, and
- Other factors over which we have no control.

Given the assumptions that must be made, Shannon & Wilson cannot guarantee the accuracy of the opinion of probable construction costs. Shannon & Wilson is not a construction cost estimator or construction contractor, nor should our rendering of an opinion of probable construction costs be considered equivalent to the nature and extent of services a construction cost estimator or contractor would provide.

This report was prepared for the exclusive use of HDR Engineering, Inc., and Multnomah County for use in the Burnside Bridge NEPA and Type Selection Phase. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions included in this report.

The scope of our present work did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson, Inc., has prepared and included the attached “Important Information About Your Geotechnical/Environmental Report,” to assist you and others in understanding the use and limitations of our reports.

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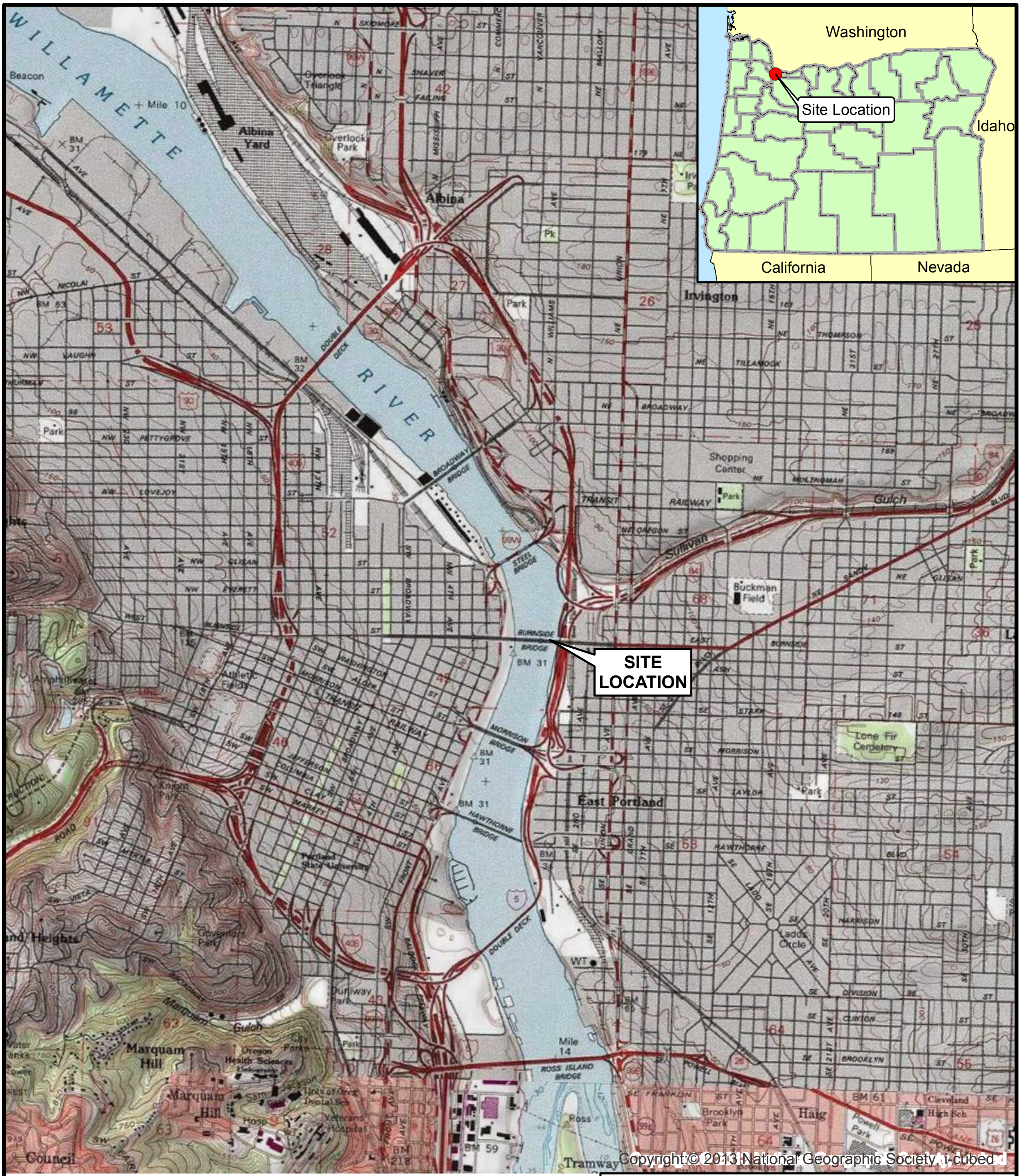

Burnside Bridge Environmental Impact Study
Portland, Oregon

VICINITY MAP

September 2019

102636-001

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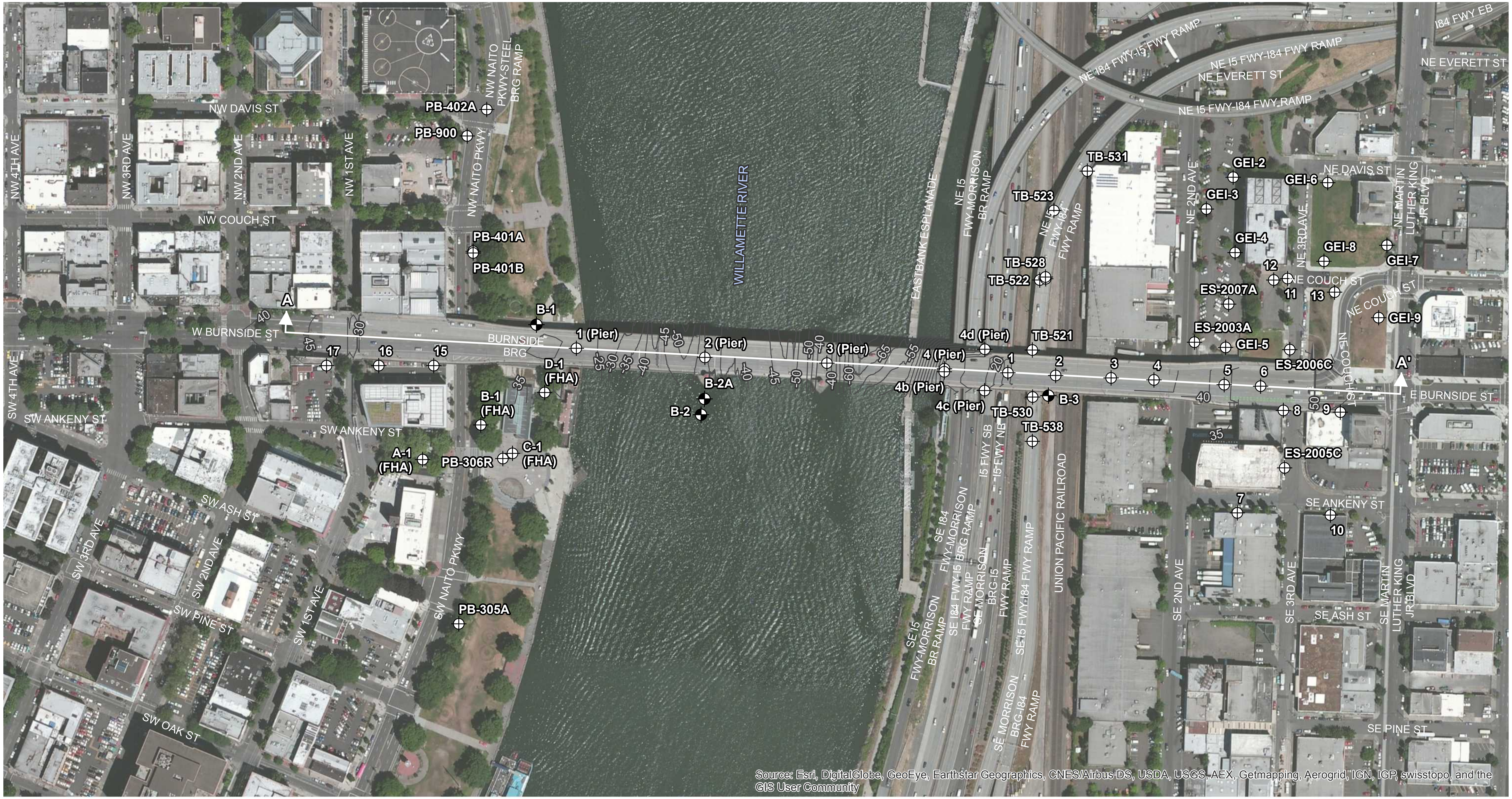
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

Filename: T:\Projects\24-1\4065_Burnside Bridge Seismic Feasibility Study\Avmxd\BurnsideVicinityMap.mxd Date: 1/31/2017 Login: aeh



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

LEGEND

B-1 ⊕	Designation and Approximate Location of Boring (Shannon & Wilson, 2016)
TB-530 ⊕	Designation and Approximate Location of Previous Boring
A A'	Location and Designation of Interpretive Subsurface Profile

0 100 200 400

Scale in Feet

NOTES

1. See Appendix A for locations and logs of previous borings.
2. Existing contours and bathymetry adapted from files dtm.dwg and DEA Point Data Hydro cross sections..asc, provided by HDR, Inc., on November 21, 2016.

Burnside Bridge Environmental Impact Study
Portland, Oregon

SITE AND EXPLORATION PLAN

September 2019

102636-001

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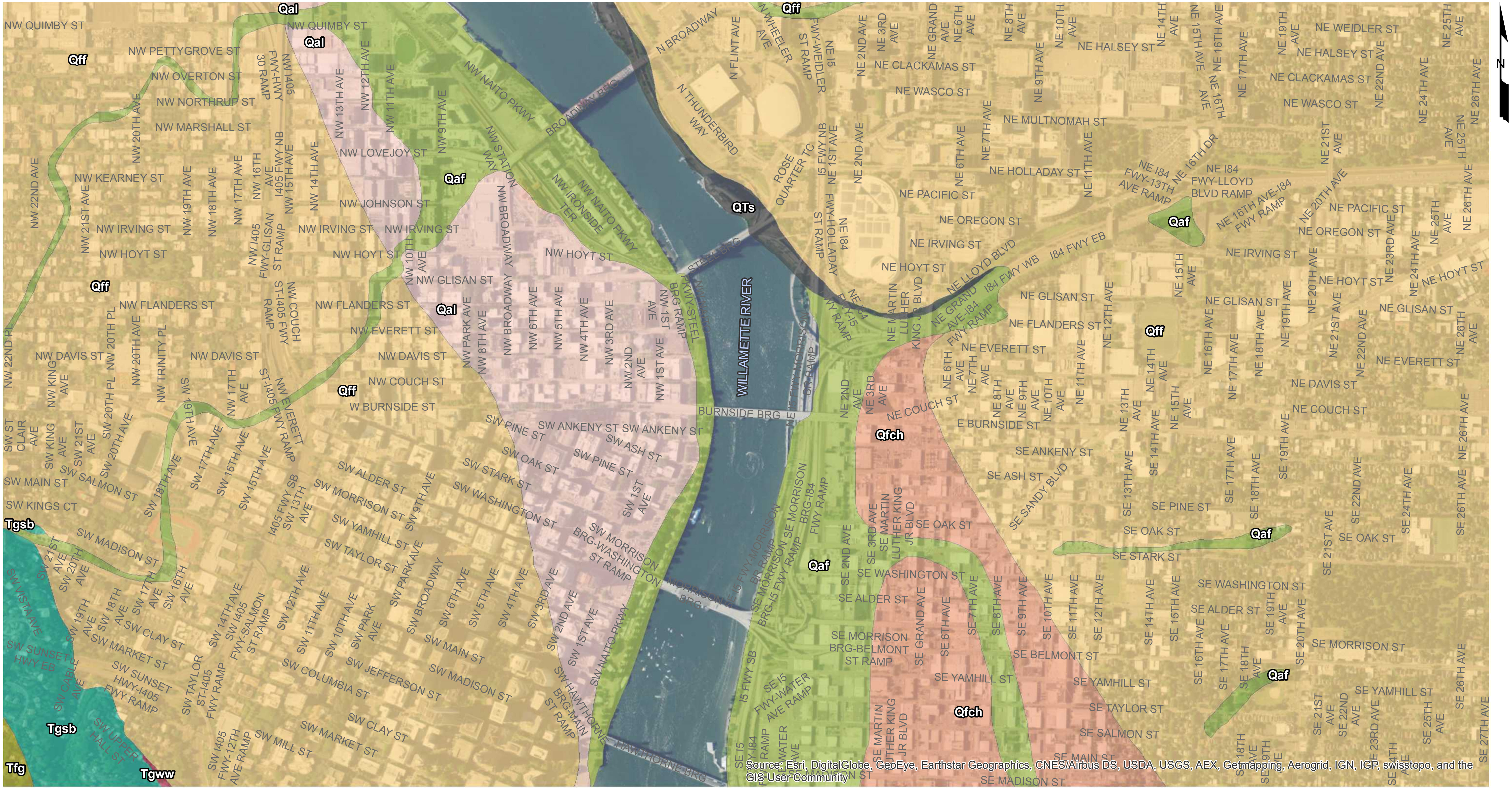
FIG. 2

Filename: T:\Projects\24-1\4065_Burnside Bridge Seismic Feasibility Study\Avmxd\BurnsideSitePlan.mxd Date: 6/14/2017 Login: aeh



Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AEX, Getmapping, Aerogrid, IGN, IGP, swisstopo, and the GIS User Community

LEGEND

Qaf - Artificial Fill

Qal - Alluvium

Qfch - Catastrophic Flood Deposits, Channel Facies

Qff - Catastrophic Flood Deposits, Fine-grained Facies

QTs - Troutdale Formation

Tfg - Basalt of Ginko

Tgsb - Columbia River Basalt Group, Sentinel Bluffs Member

Tgww - Columbia River Basalt Group, Winter Water Member

0 500 1,000 2,000

Scale in Feet

NOTES

1. Geologic mapping from Oregon Geologic Data Compilation, Release 6 (OGDC-6) by DOGAMI.

Burnside Bridge Environmental Impact Study
Portland, Oregon

PUBLISHED GEOLOGIC MAPPING

September 2019 102636-001

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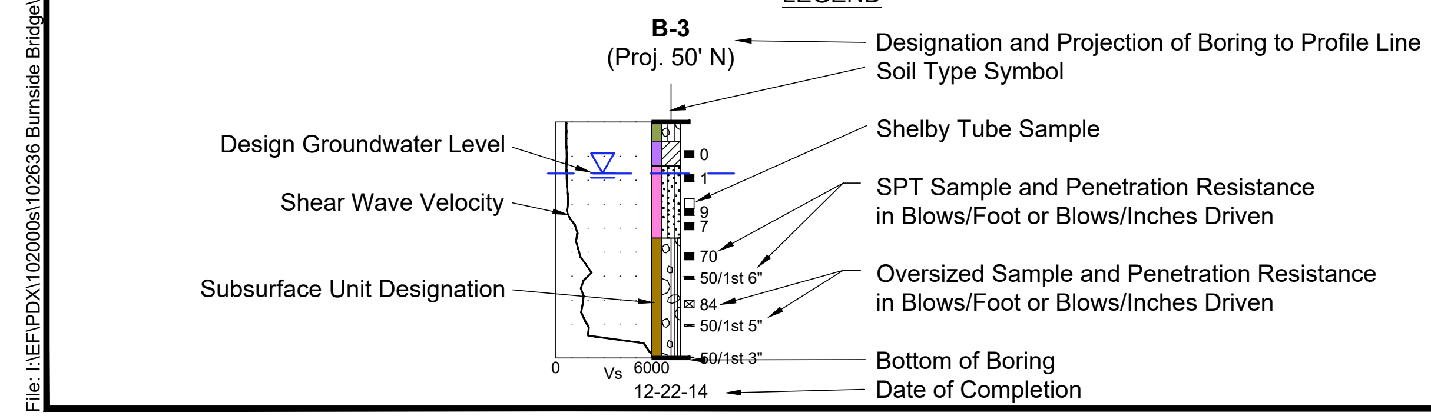
FIG. 3

Filename: T:\Projects\24-14065_Burnside Bridge Seismic Feasibility Study\Avmxd\BurnsideGeology.mxd Date: 4/6/2017 Login: aeh

LEGEND



0 40 80

Scale in Feet

No Vertical Exaggeration

NOTES

1. Ground surface generated from files dtm.dwg and DEA Point Data Hydro cross sections..asc, provided by HDR, Inc., on November 21, 2016.
2. Ground surface from file BR00511_X_Plan_Elev.dwg, provided by HDR, Inc., on October 13, 2016.
3. This profile was generalized from materials as observed in current borings and reported in historic boring logs. Variations may exist between profile and actual conditions. See Appendices A and B for complete boring logs and explanations of symbols.
4. See Figure 2 for profile location.
5. Borings performed by Shannon & Wilson, Inc., in 2016 were located using a handheld GPS (B-1 through B-3). All other boring locations were approximated based on information available in historic reports and boring logs (some by others).
6. Bridge elevation view adapted from file BR00511_X_Plan_Elev.dwg, provided by HDR, Inc., on October 13, 2016. Foundations were modified based on available as-built drawings included in Appendix A.

Burnside Bridge Environmental Impact Study
Portland, Oregon

INTERPRETIVE SUBSURFACE PROFILE A-A'

September 2019 102636-001

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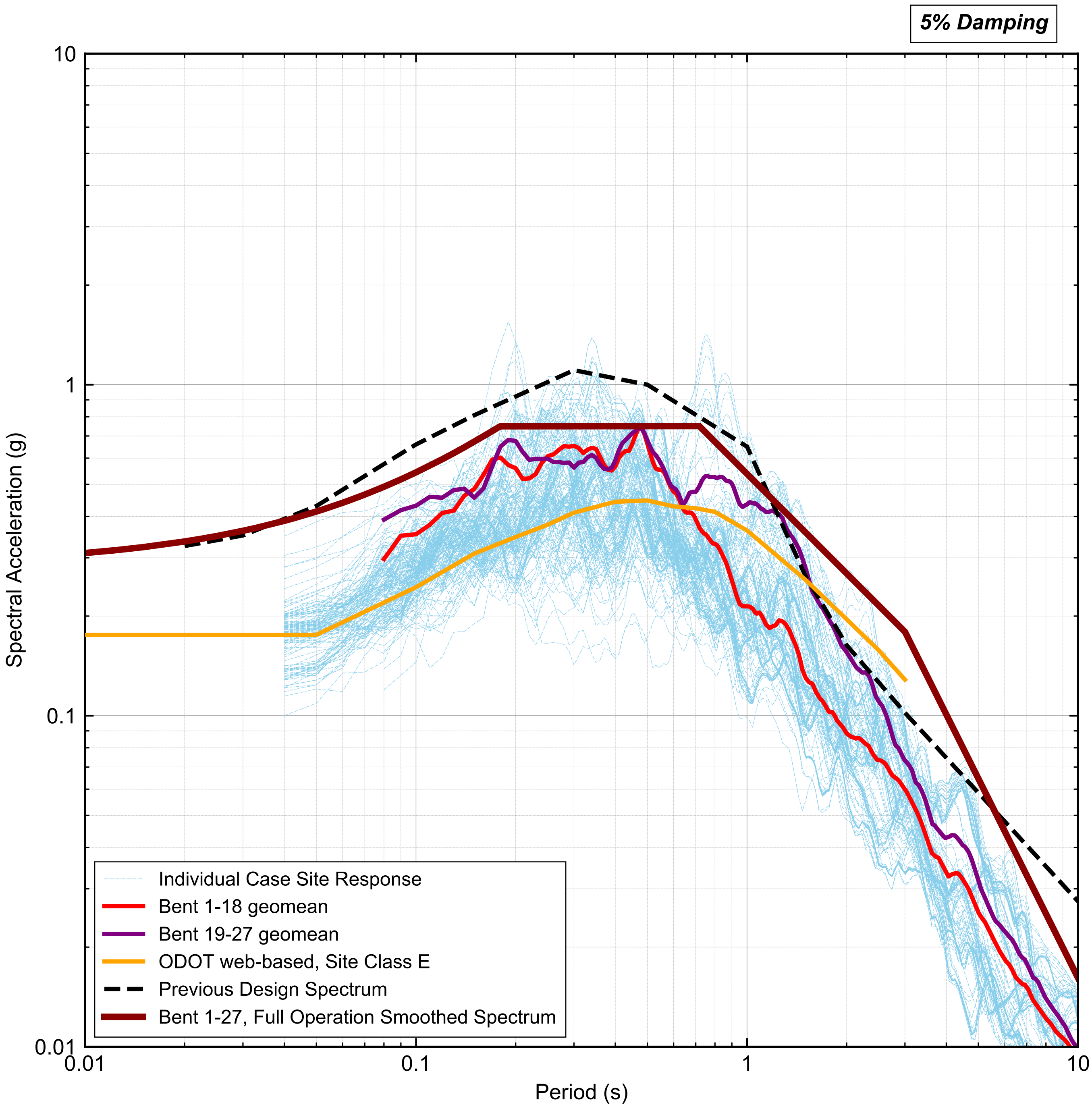
Geotechnical and Environmental Consultants

FIG. 4

ver:0.8.5.post0.dev0+g414b689, by:PTO



Notes:
1. g = gravitational acceleration
   ft = feet
   s = seconds
2. Response spectrum for "ODOT web-based" is derived from the Portland State University online application.

Burnside Bridge
Environmental Impact Study
Portland, Oregon

RESPONSE SPECTRA
FULL OPERATION
DESIGN LEVEL
EXISTING BENTS 1 TO 27

May 2020 102636-001

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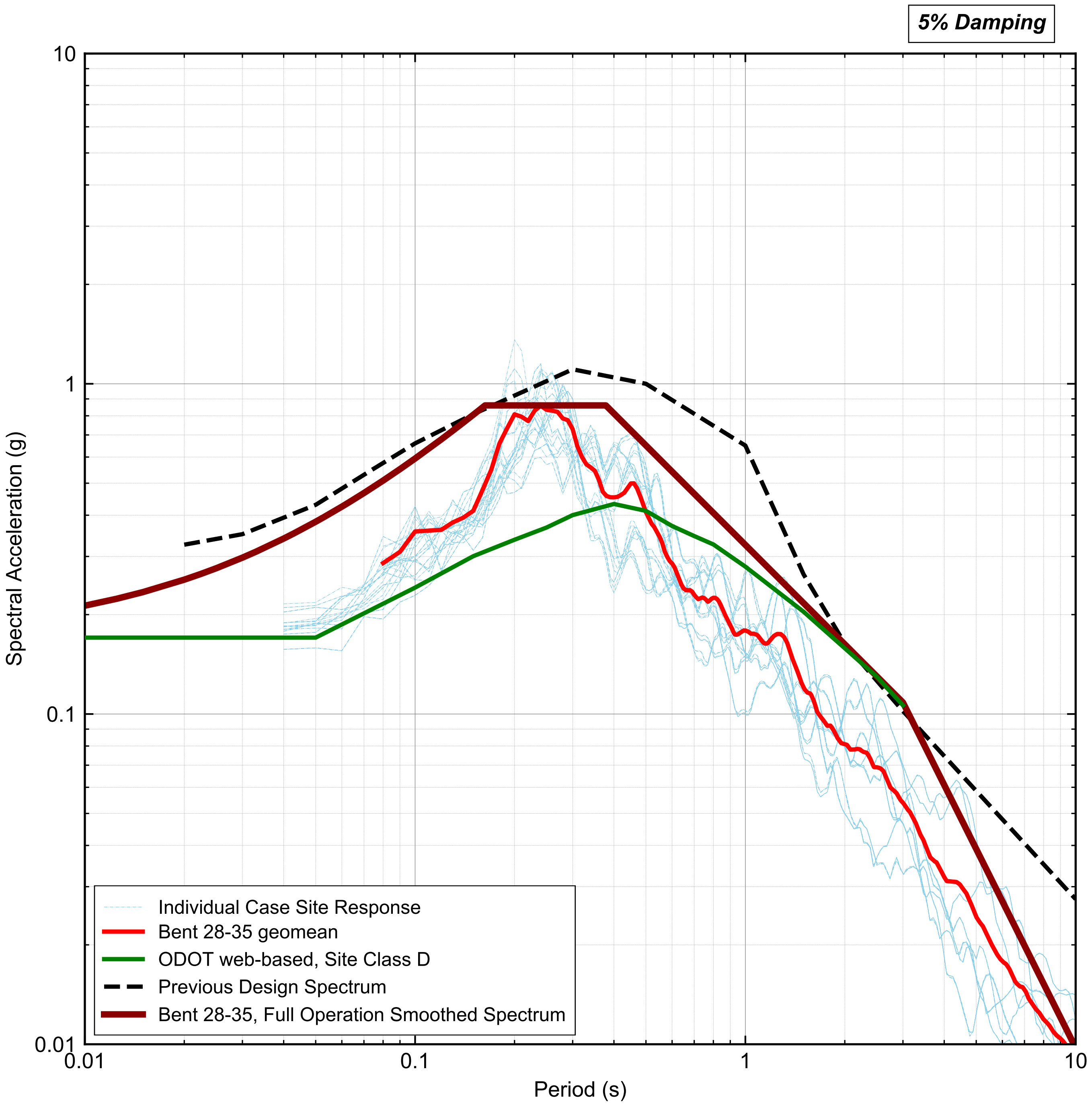
Geotechnical and Environmental Consultants

FIG. 5

C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB\tripartite.py

ver:0.8.5.post0.dev0+g414b689, by:PTO

C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB\tripartite.py



Notes:
1. g = gravitational acceleration
   ft = feet
   s = seconds
2. Response spectrum for "ODOT web-based" is derived from the Portland State University online application.

Burnside Bridge
Environmental Impact Study
Portland, Oregon

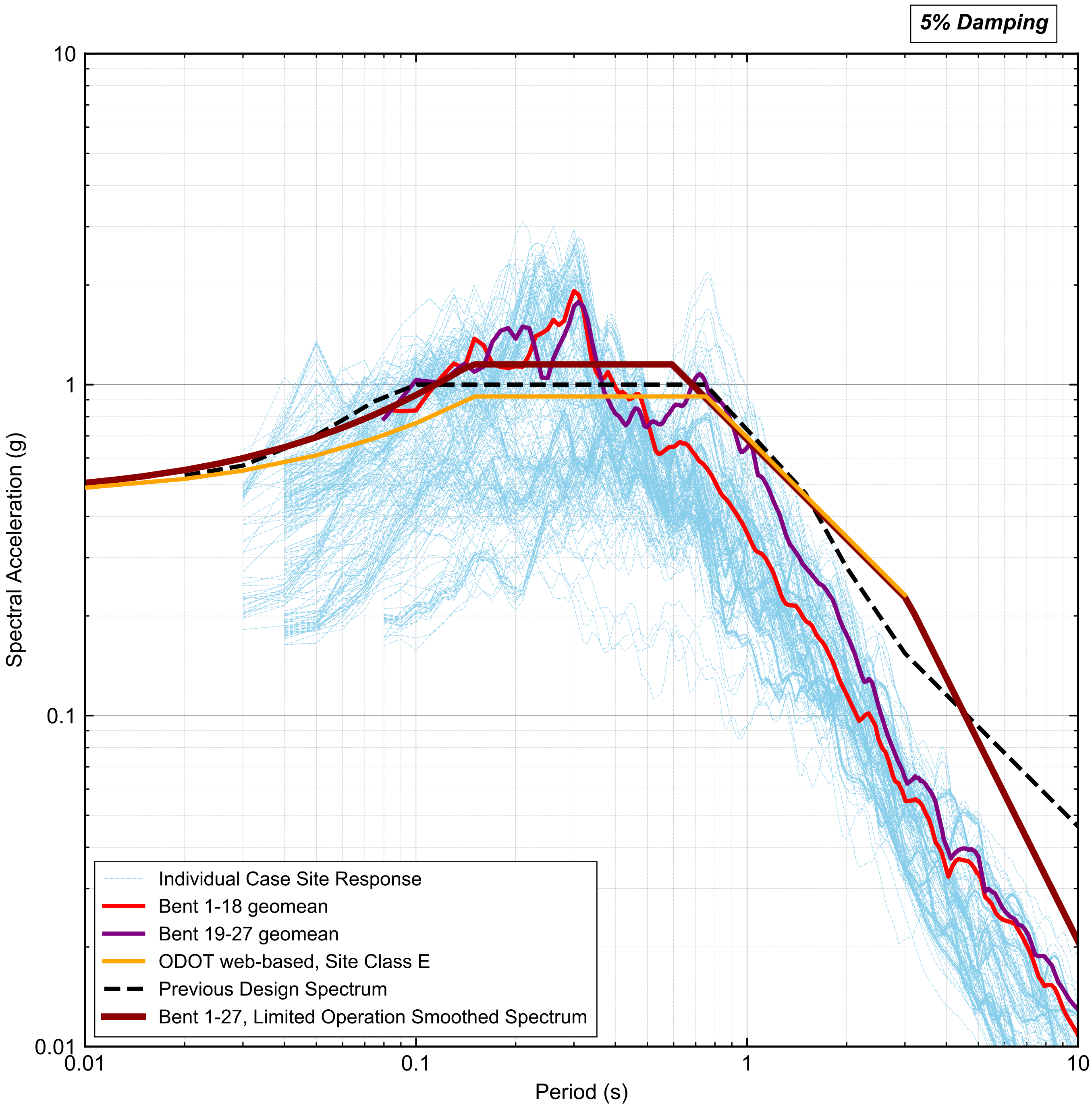
RESPONSE SPECTRA
FULL OPERATION
DESIGN LEVEL
EXISTING BENTS 28 TO 35

May 2020 102636-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6

ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. g = gravitational acceleration
   ft = feet
   s = seconds

C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB\triparite.py

Burnside Bridge Environmental Impact Study
Portland, Oregon

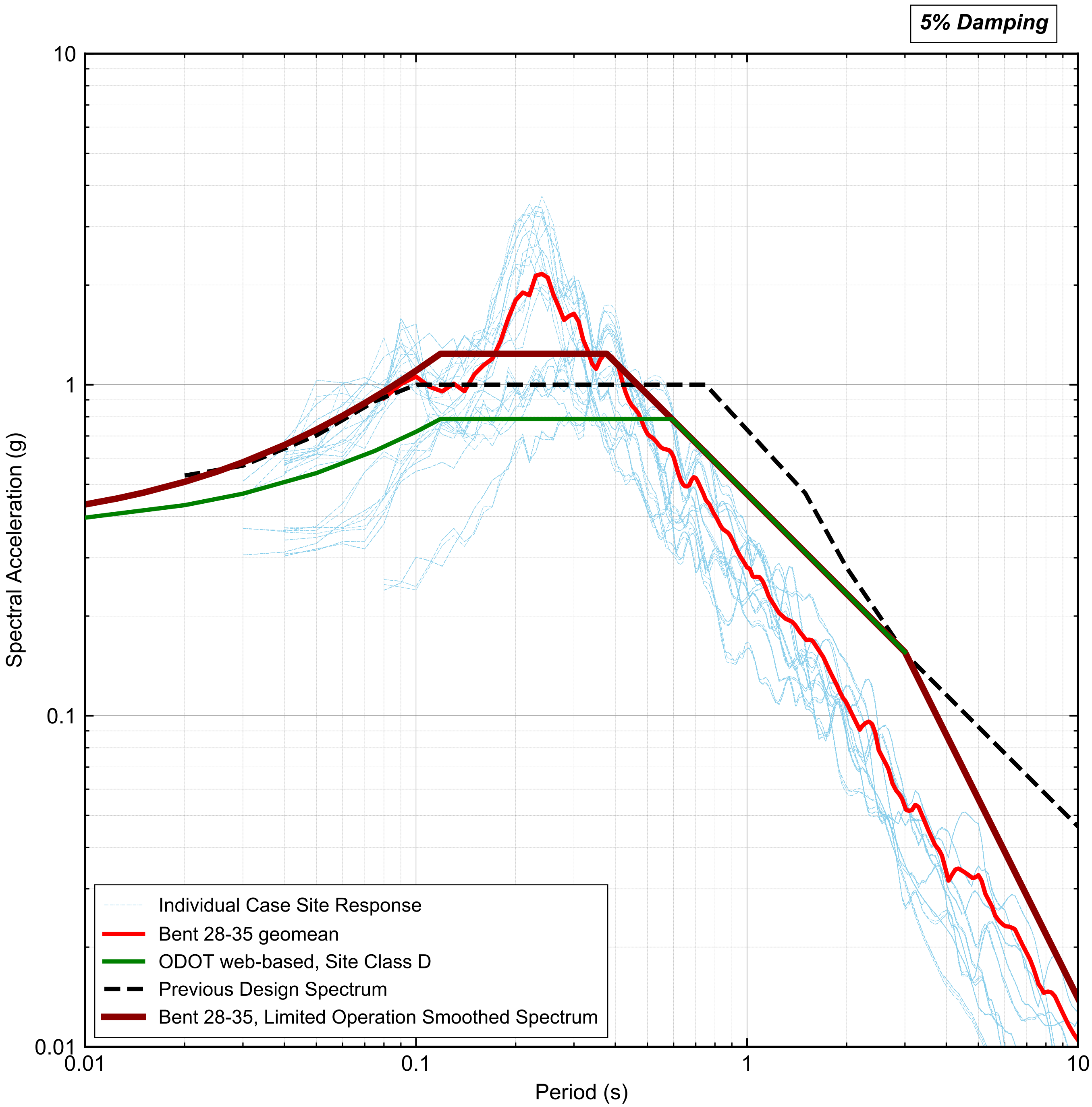
RESPONSE SPECTRA
LIMITED OPERATION
DESIGN LEVEL
EXISTING BENTS 1 TO 27

September 2019
102636-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 7

ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. g = gravitational acceleration
   ft = feet
   s = seconds

Burnside Bridge Environmental Impact Study
Portland, Oregon

RESPONSE SPECTRA
LIMITED OPERATION
DESIGN LEVEL
EXISTING BENTS 28 TO 35

September 2019
102636-001

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Geotechnical and Environmental Consultants

FIG. 8

C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB\tripartite.py

LEGEND

🟥 Approximate Cellular Soil-Cement Ground Improvement Zones (See Figure 10)

0 40 80

Scale in Feet

No Vertical Exaggeration

NOTES

1. See Figure 4 for complete interpretive subsurface profile and associated notes.
2. See Figure 10 for conceptual ground improvement zone details.
3. Existing bridge elevation view adapted from file BR00511_X_Plan_Elev.dwg, provided by HDR, Inc., on October 13, 2016. Foundations were modified based on available as-built drawings included in Appendix A.
4. Retrofit alternative bridge elevation view from file EQRB_HDR_ALT1_Elev_and_SoilProfile.dwg, provided by HDR, Inc., on August 2, 2019.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ENHANCED RETROFIT
ALTERNATIVE PROFILE
A-A'

September 2019 102636-001

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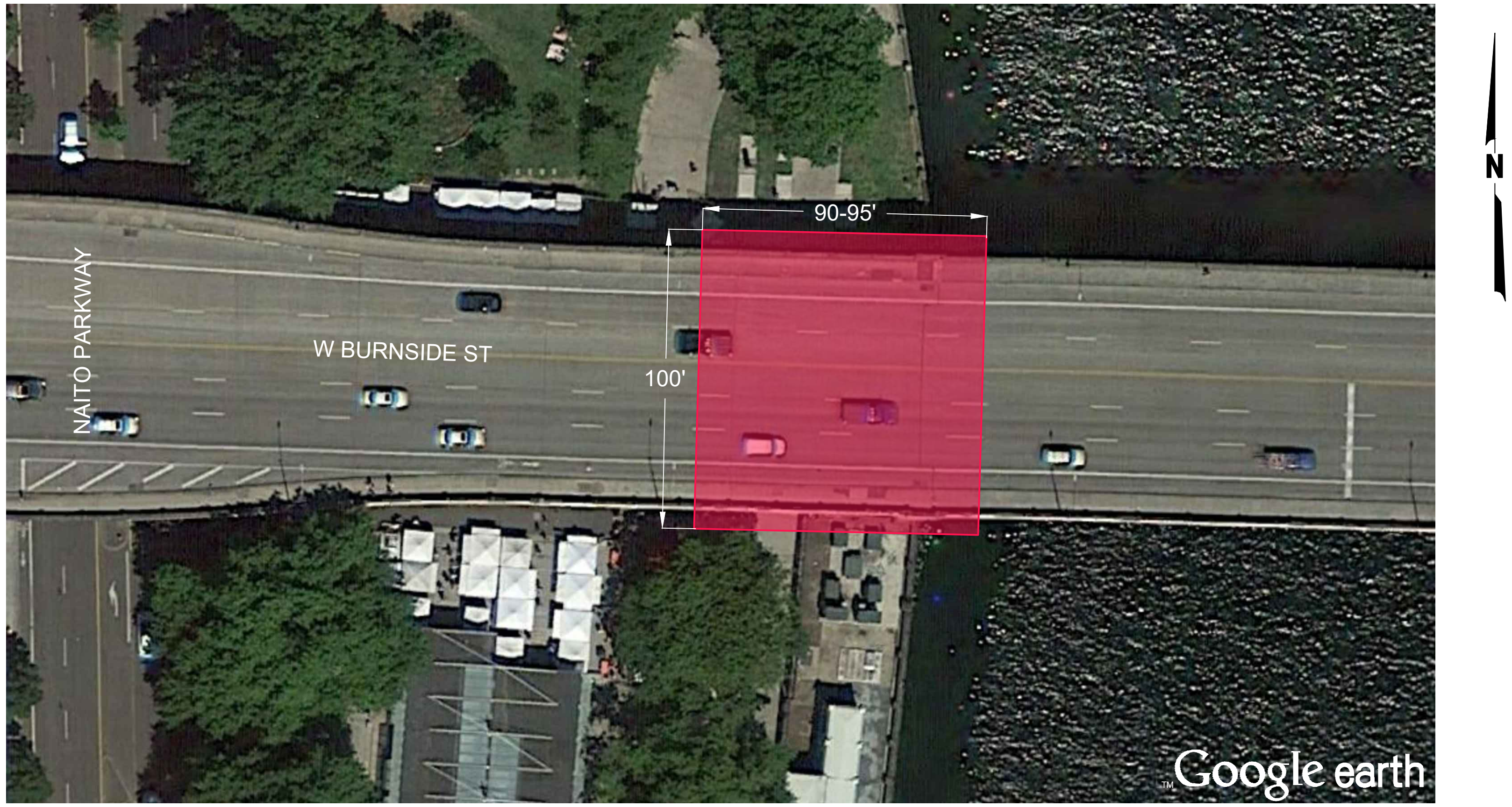
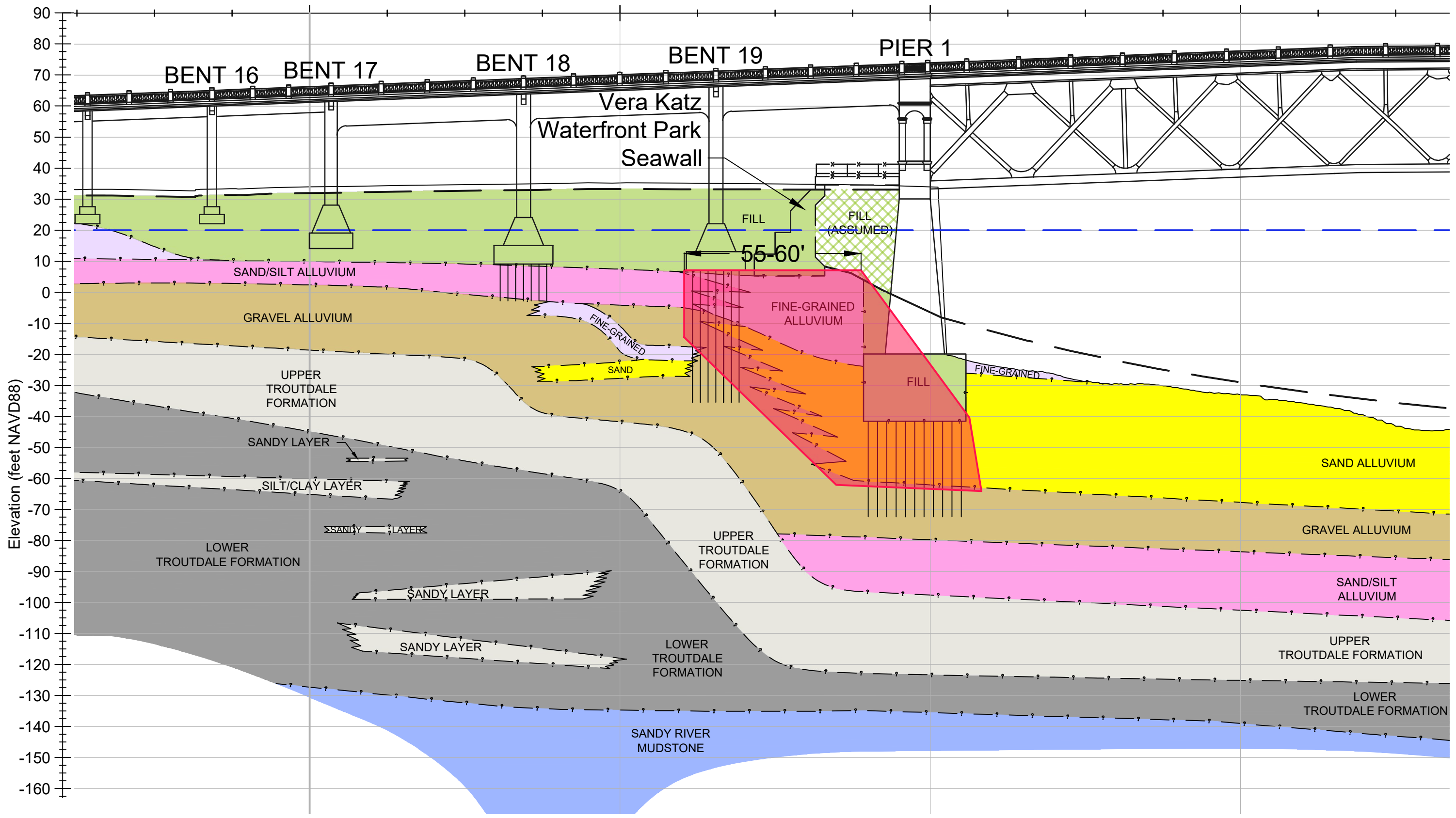
Geotechnical and Environmental Consultants

FIG. 9

File: I:\EF\PDX\102000s\102636 Burnside Bridge\DRAFTING\Profile_2018.dwg Date: 08-15-2019 Author: clv





LEGEND

🟥 Approximate Cellular Soil-Cement Ground Improvement Zones

NOTES

1. Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.
2. See Figure 4 for complete interpretive subsurface profile and associated notes.
3. Ground improvement zones at Bents 2 through 16 of the Retrofit Alternative are not included on this figure but are shown on Figure 9 and described in Section 10.1.2 of the report.

0 60 120

Approximate Scale in Feet

Burnside Bridge Environmental Impact Study
Portland, Oregon

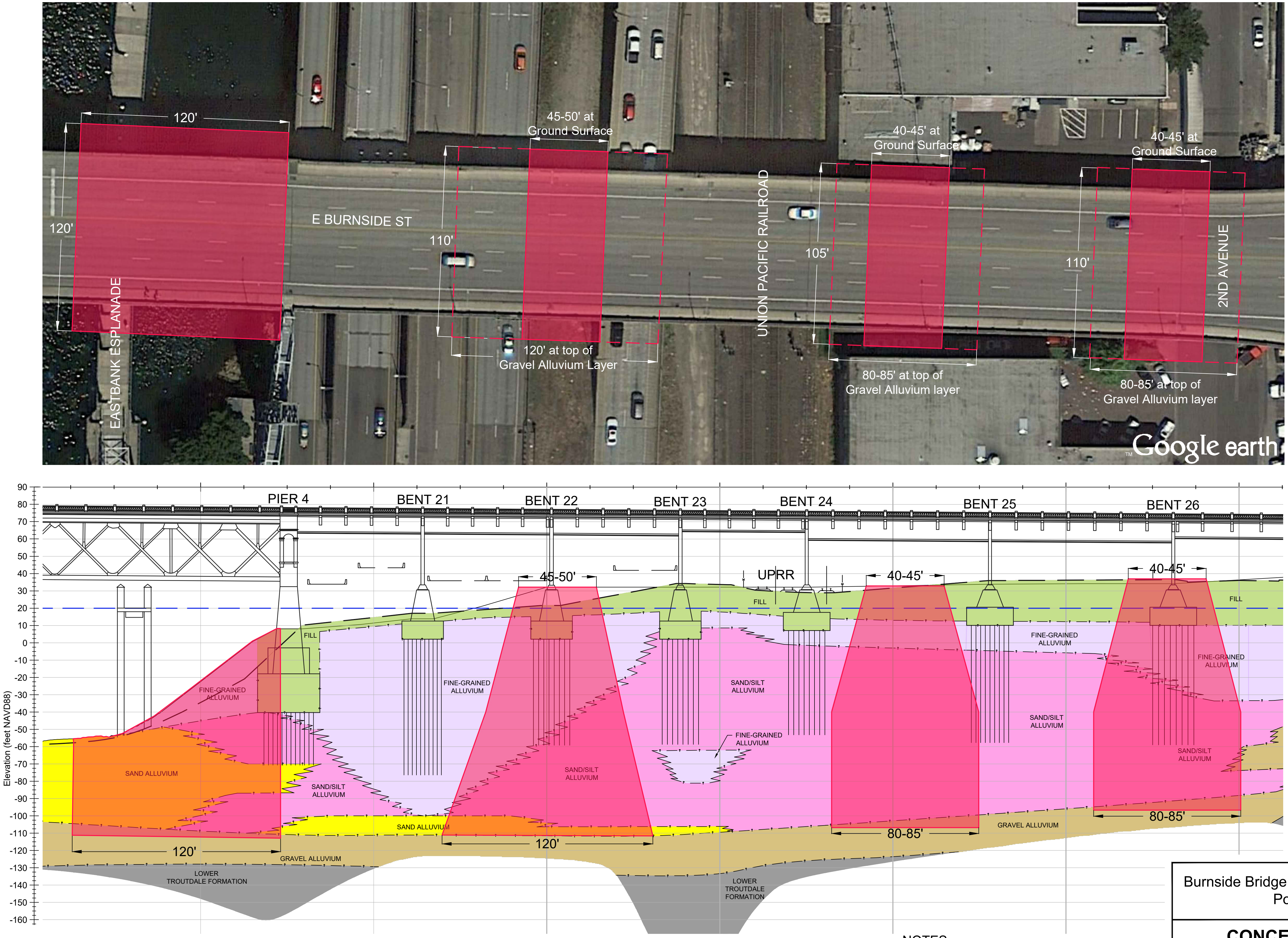
CONCEPTUAL GROUND IMPROVEMENT EXTENTS FOR LATERAL SPREAD MITIGATION

May 2020 102636-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 10
(Sheet 1 of 2)

File: I:\EF\PDX\102000s\102636 Burnside Bridge\DRAFTING\GroundImprovement.dwg Date: 05-13-2020 Author: AEH



LEGEND

🟥 Approximate Cellular Soil-Cement Ground Improvement Zones

0 60 120

Approximate Scale in Feet

NOTES

1. Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.
2. See Figure 4 for complete interpretive subsurface profile and associated notes.
3. Three additional ground improvement zones at Bents N10 through N12 of the Couch Extension (not shown) are assumed to be located at the respective proposed bent locations. See Section 10 of the report text for more information.

Burnside Bridge Environmental Impact Study
Portland, Oregon

CONCEPTUAL GROUND IMPROVEMENT EXTENTS FOR LATERAL SPREAD MITIGATION

May 2020 102636-001

SHANNON & WILSON, INC.
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FIG. 10
(Sheet 2 of 2)

LEGEND

🟥 Approximate Cellular Soil-Cement Ground Improvement Zones (See Figure 10)

0 40 80
Scale in Feet
No Vertical Exaggeration

NOTES
1. See Figure 4 for complete interpretive subsurface profile and associated notes.
2. See Figure 10 for conceptual ground improvement zone details.
3. Existing bridge elevation view adapted from file BR00511_X_Plan_Elev.dwg, provided by HDR, Inc., on October 13, 2016. Foundations were modified based on available as-built drawings included in Appendix A.
4. Replacement alternative bridge elevation view from file EQRB_HDR_ALT3_Elev_and_SoilProfile.dwg, downloaded from ProjectWise on May 5, 2020.
5. Figure shows profile along the south branch of the Couch Extension; Bents N10 through N15 of the north branch are not shown.

Burnside Bridge Environmental Impact Study
Portland, Oregon

SHORT-SPAN ALTERNATIVE AND COUCH EXTENSION PROFILE A-A'

May 2020 102636-001

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FIG. 11

\PDX\102000s\102636 Burnside Bridge\DRAFTING\Profile_2018.dwg Date: 05-13-2020 Author: AEH

File: I:\EF\PDX\102000s\102636 Burnside Bridge\DRAFTING\Profile_2018.dwg Date: 05-13-2020 Author: AEH

LEGEND

🟥 Approximate Cellular Soil-Cement Ground Improvement Zones (See Figure 13)

0 40 80
Scale in Feet
No Vertical Exaggeration

NOTES

1. See Figure 4 for complete interpretive subsurface profile and associated notes.
2. See Figure 13 for conceptual ground improvement zone details.
3. Existing bridge elevation view adapted from file BR00511_X_Plan_Elev.dwg, provided by HDR, Inc., on October 13, 2016. Foundations were modified based on available as-built drawings included in Appendix A.
4. Replacement alternative bridge elevation view from file EQRB_HDR_ALT3_Elev_and_SoilProfile.dwg, downloaded from ProjectWise on May 5, 2020.

Burnside Bridge Environmental Impact Study
Portland, Oregon

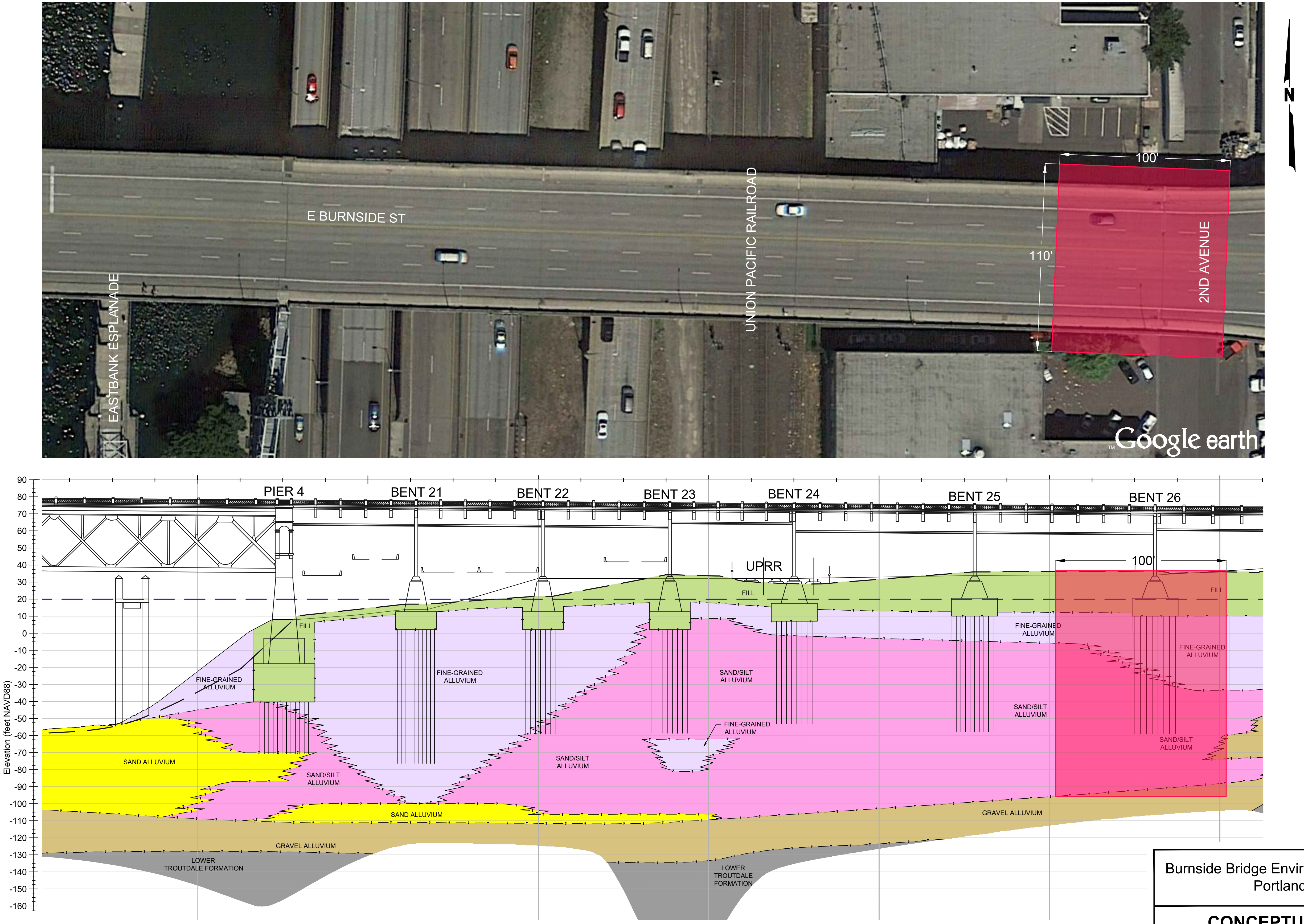
LONG-SPAN ALTERNATIVE
PROFILE A-A'

May 2020
102636-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 12

File: I:\EF\PDX\102000s\102636 Burnside Bridge\DRAFTING\GroundImprovement.dwg Date: 05-13-2020 Author: AEH



LEGEND

🟥 Approximate Cellular Soil-Cement Ground Improvement Zones

0 60 120

Approximate Scale in Feet

NOTES

1. Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth™ Mapping Service.
2. See Figure 4 for complete interpretive subsurface profile and associated notes.

Burnside Bridge Environmental Impact Study
Portland, Oregon

CONCEPTUAL GROUND IMPROVEMENT EXTENTS FOR LATERAL SPREAD MITIGATION

May 2020 102636-001

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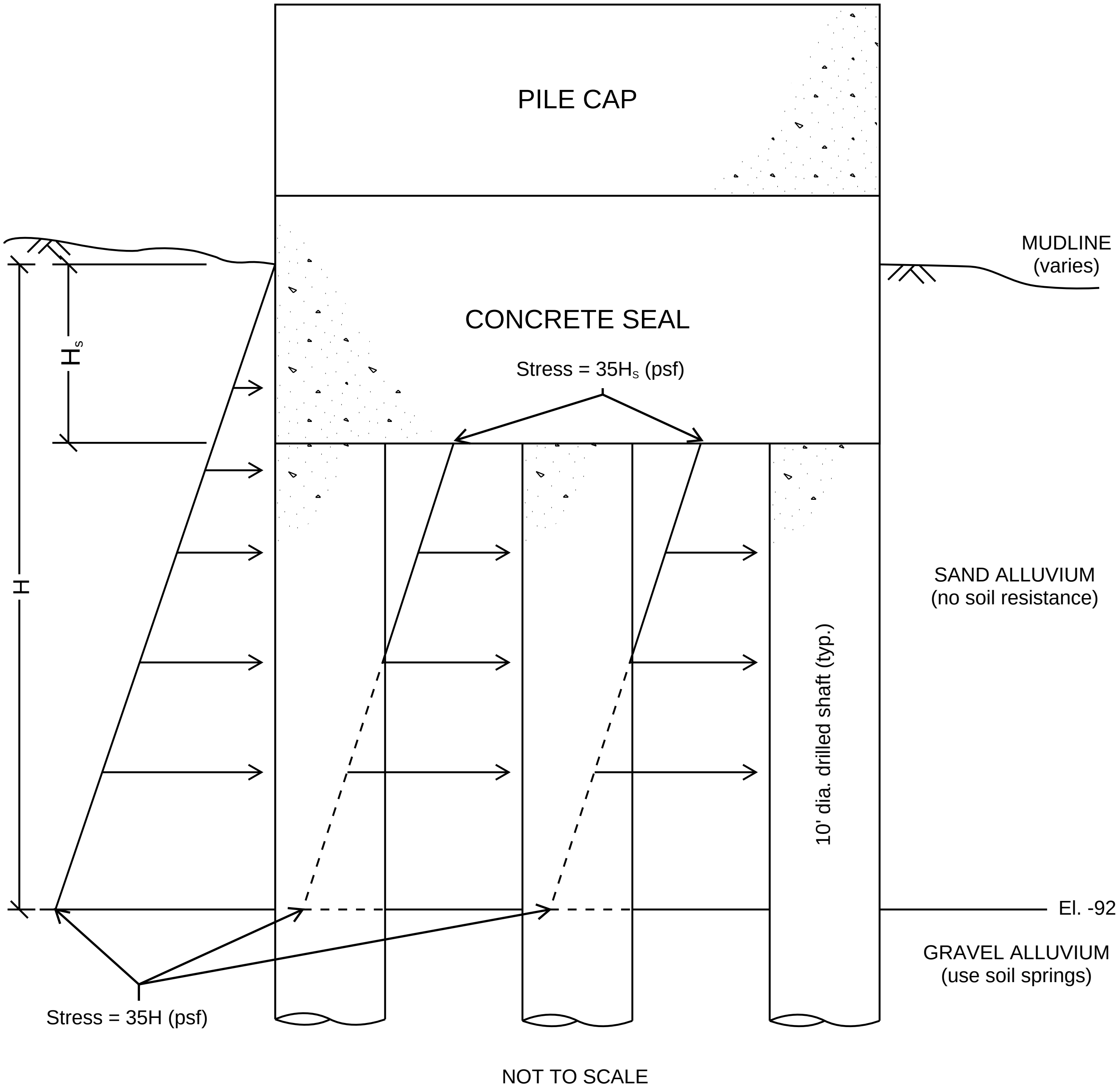
Geotechnical and Environmental Consultants

FIG. 13



NOTES

1. Lateral stress distribution should be applied with "post-seismic" Q-Z and T-Z springs and P-Y soil parameters

Burnside Bridge Environmental Impact Study
Portland, Oregon

PIERS 2 & 3 OR BENTS 6 & 7 / 7 & 8
LATERAL SPREADING
EARTH PRESSURE DISTRIBUTIONS

September 2019 102636-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 14

Appendix A

Existing Information

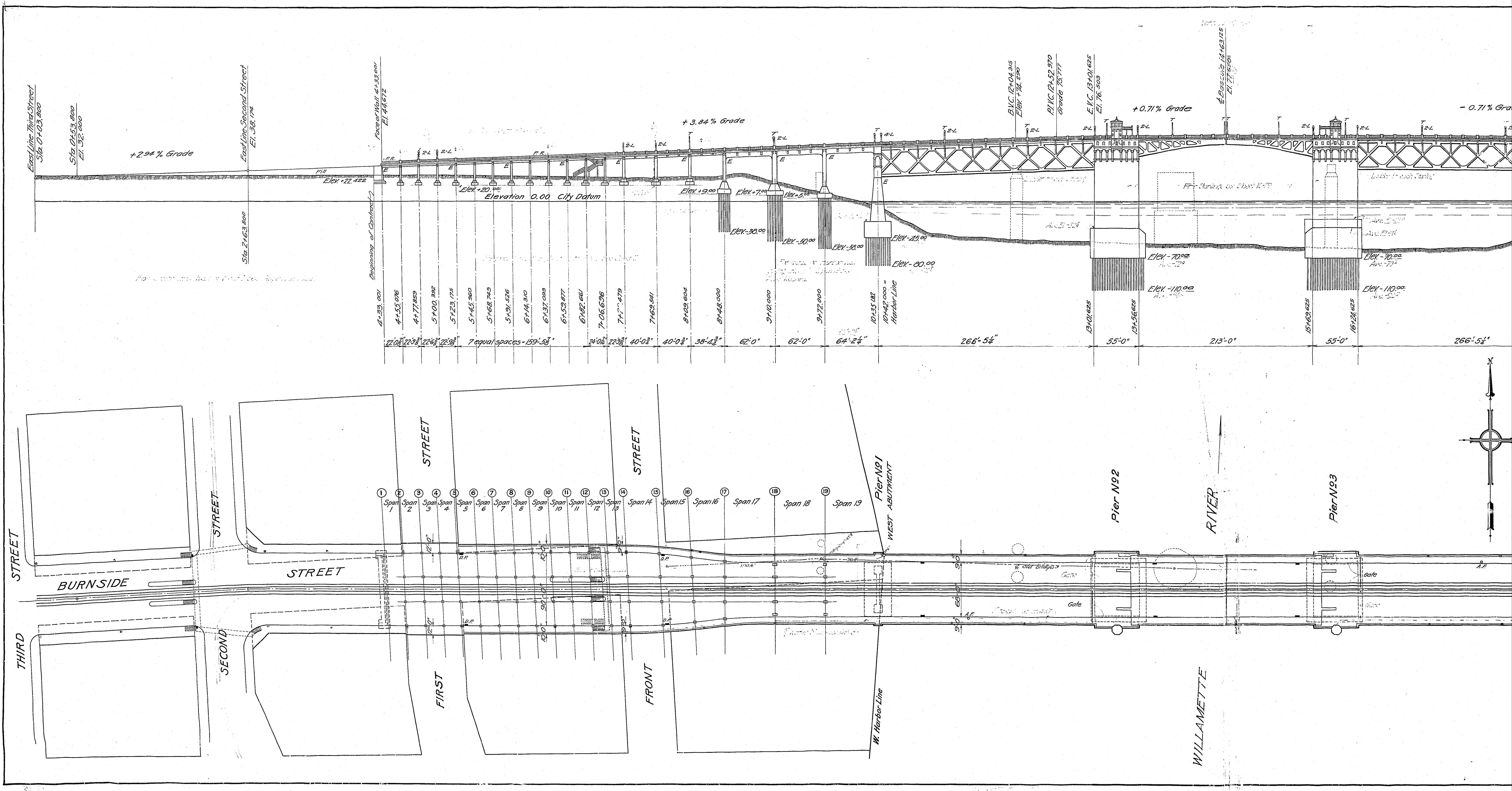
CONTENTS

- Plans for Burnside Bridge (Hedrick & Kremers Consulting Engineers, 1924)
    - Sheet No. T2
    - Sheet No. 7
    - Sheet No. T8
    - Sheet No. T10
    - Sheet No. T16
    - Sheet No. 18
    - Sheet No. 48
- Plans for Completing Approaches to Burnside Bridge (Hedrick & Kremers Consulting Engineers, 1925)
    - Sheet No. L-75
- Burnside Bridge Foundation Piling Summary
- Burnside Bridge Sketch Showing Harbor Wall West of Pier 1 (Gustav Lindenthal Consulting Engineers, 1925)
- Burnside Bridge Record of Borings (Hedrick & Kremers Consulting Engineers, 1924)
    - Includes boring 1 (pier), 2 (pier), 3 (pier), 4 (pier), 4b (pier), 4c (pier), 4d (pier), 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 15, 16, and 17
- Banfield Access Ramp Foundation Data (Oregon Department of Transportation, 1991)
    - Includes boring TB-521, TB-522, TB-523, TB-527, TB-528, TB-530, TB-531, and TB-538
- Ankeny Pump Station (Fujitani Hilts & Associates, 2000-2001)
    - Plan of Explorations
    - Log of Boring A-1
    - Log of Boring A-1a
    - Log of Boring B-1
    - Log of Boring C-1
    - Log of Boring D-1
    - Plasticity Chart
    - Grain Size Distribution Plots

CONTENTS, CONT.

- West Side CSO Project (Parsons Brinckerhoff, 2000-2001)
  - Borehole Location Plan
  - Boring Log PB-305A
  - Boring Log PB-306R
  - Boring Log PB-401A
  - Boring Log PB-401B
    West Side CSO Project (Parsons Brinckerhoff, 2000-2001)
  - Boring Log PB-402A
  - Boring Log PB-900
  - Grain Size Analysis Test Results
  - Atterberg Limits Test Results
  - Corrosivity Data
- Portland Development Commission (GeoEngineers, 2004)
  - Site Plan
  - Geologic Cross Section A-A′
  - Geologic Cross Section B-B′
  - Geologic Cross Section C-C′
  - Log of Boring GEI-2
  - Log of Boring GEI-3
  - Log of Boring GEI-4
  - Log of Boring GEI-5
  - Log of Boring GEI-6
  - Log of Boring GEI-7
  - Log of Boring GEI-8
  - Log of Boring GEI-9
- East Side CSO Project (Parsons Brinckerhoff, 2003-2005)
  - Borehole Location Plan (Figure 2-K)
  - Borehole Location Plan (Figure 2-L)
  - Boring Log ES-2003A
  - Boring Log ES-2005C
  - Boring Log ES-2006C
  - Boring Log ES-2007A

NOTE: Approximate locations of explorations contained in this appendix are shown on the Site and Exploration Plan, Figure 2.


East Line Third Street
Sta. 0+03.800
Sta. 0+53.800
El. 32.000
+2.94 % Grade
East Line Second Street
El. 38.174
Sta. 2+63.800
Fill
Elev. +22.422
Face of Wall 4+33.001
El. 44.672
Beginning of Contract
P.R.
2-L
E
T
4-L
Elev. +20.00
Elevation 0.00 City Datum
+ 3.84 % Grade
Elev. +9.00
Elev. +7.00
Elev. +5.00
Elev. -38.00
Elev. -50.00
Elev. -58.00
Elev. -25.00
Elev. -80.00
B.V.C. 12+04.315
Elev. +74.290
P.I.V.C. 12+52.970
Grade 75.777
E.V.C. 13+01.625
El. 76.503
+0.71 % Grade
℄ Bascule 14+63.125
El. 77.650
- 0.71 % Gra
Elev. -70.00
Elev. -110.00
4+33.001
4+55.076
4+77.859
5+00.392
5+23.175
5+45.960
5+68.743
5+91.526
6+14.310
6+37.093
6+59.877
6+82.661
7+06.696
7+29.479
7+69.541
8+09.604
8+48.000
9+10.000
9+72.000
10+35.183
10+42.000 = Harbor Line
13+01.625
13+56.625
15+69.625
16+24.625
22'-0 7/8"
22'-9 3/8"
22'-6 3/8"
22'-9 3/8"
7 equal spaces = 159'-5 7/8"
24'-0 7/16"
22'-9 3/8"
40'-0 3/4"
40'-0 3/4"
38'-4 3/4"
62'-0"
62'-0"
64'-2 1/4"
266'-5 1/4"
55'-0"
213'-0"
55'-0"
266'-5 1/4"
N
STREET
BURNSIDE
STREET
THIRD
STREET
SECOND
STREET
FIRST
STREET
FRONT
① ② ③ ④ ⑤ ⑥ ⑦ ⑧ ⑨ ⑩ ⑪ ⑫ ⑬ ⑭ ⑮ ⑯ ⑰ ⑱ ⑲
Span 1
Span 2
Span 3
Span 4
Span 5
Span 6
Span 7
Span 8
Span 9
Span 10
Span 11
Span 12
Span 13
Span 14
Span 15
Span 16
Span 17
Span 18
Span 19
Pier No 1
WEST ABUTMENT
W. Harbor Line
Pier No 2
Pier No 3
RIVER
WILLAMETTE
12'-0"
10'-0"
9'-0"
9'-0"
68'-0"
D.P.
A.F.
Gate

E D C B A A B C D E

LEGEND

T	= Trolley pole
•L	= Light post
—··—	= Existing tracks
▬	= Scupper
E	= Expansion Joint
A.F.	= Automatic Flagman
P.R.	= Pipe Railing (Not included in Contract)
▬ D.P.	= Inlet with Drain Pipe to Ground or Sewer

MULTNOMAH COUNTY OREGON

BURNSIDE BRIDGE
PORTLAND OREGON

GENERAL ELEVATION & PLAN
DESIGN NO. 2

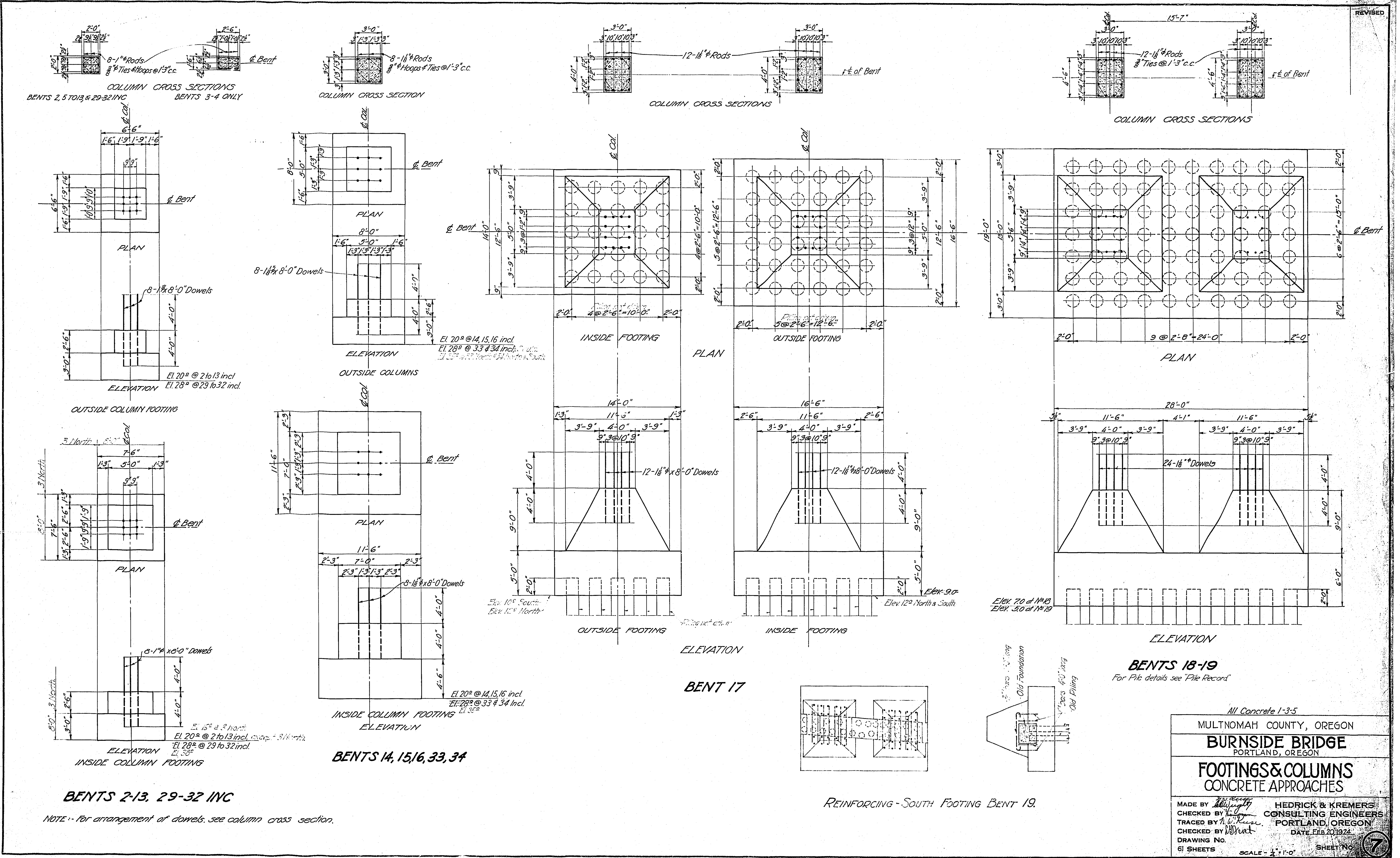
MADE BY John Goss.
CHECKED BY ..L.Wright
TRACED BY John Goss.
CHECKED BY
DRAWING NO.
SHEETS
SCALE - 1" = 40'-0"

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB. 21-1924
SHEET NO. (T-2)

G-14

FL-25

E D C B A B C D E



All Concrete 1-3-5

MULTNOMAH COUNTY, OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

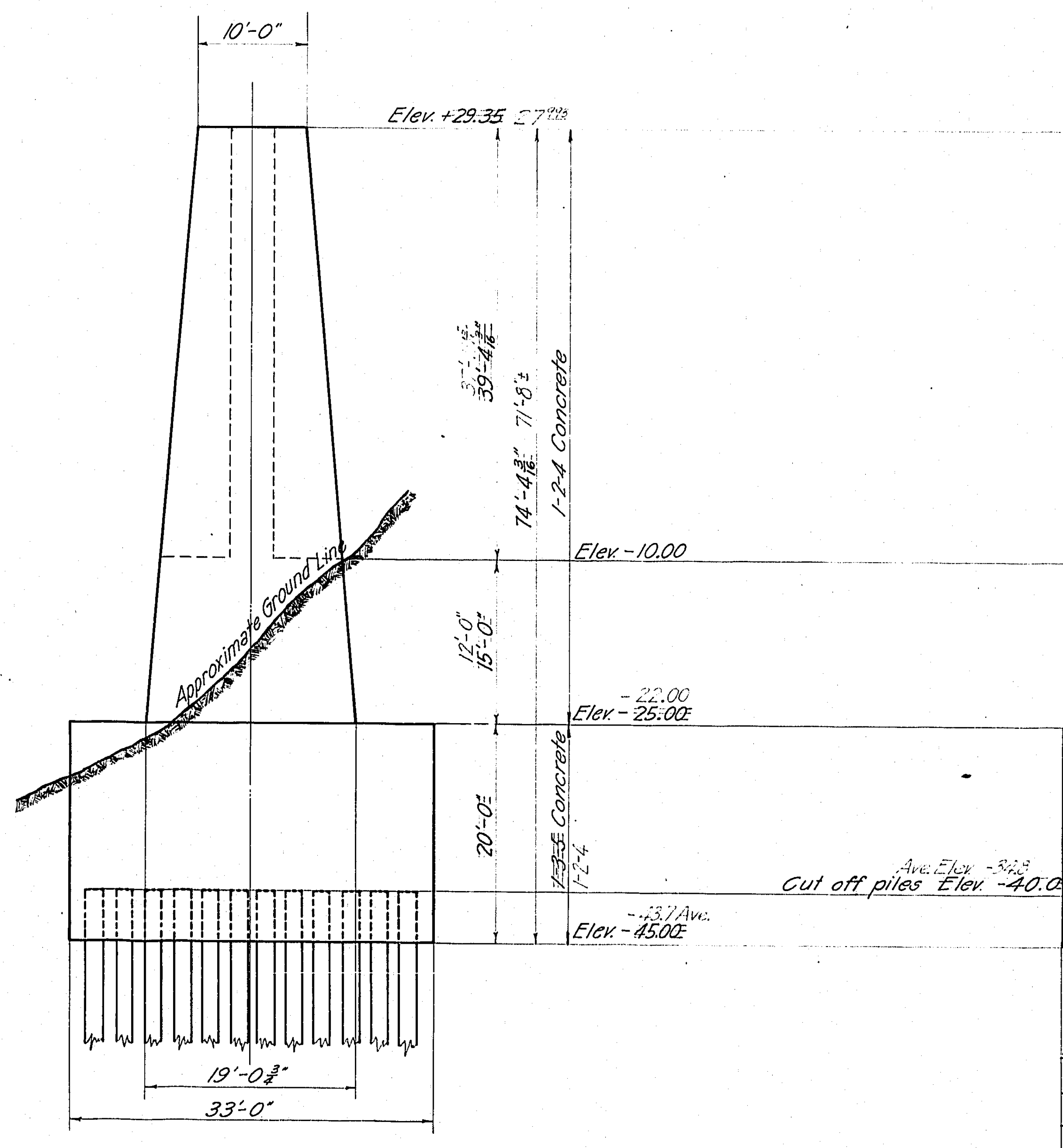
FOOTINGS & COLUMNS
CONCRETE APPROACHES

MADE BY ~~H.W. Leu~~ A.W. Wright
CHECKED BY Thompson
TRACED BY H. W. Ruse.
CHECKED BY W. Frost
DRAWING No.
61 SHEETS
SCALE - ¼" = 1'-0"

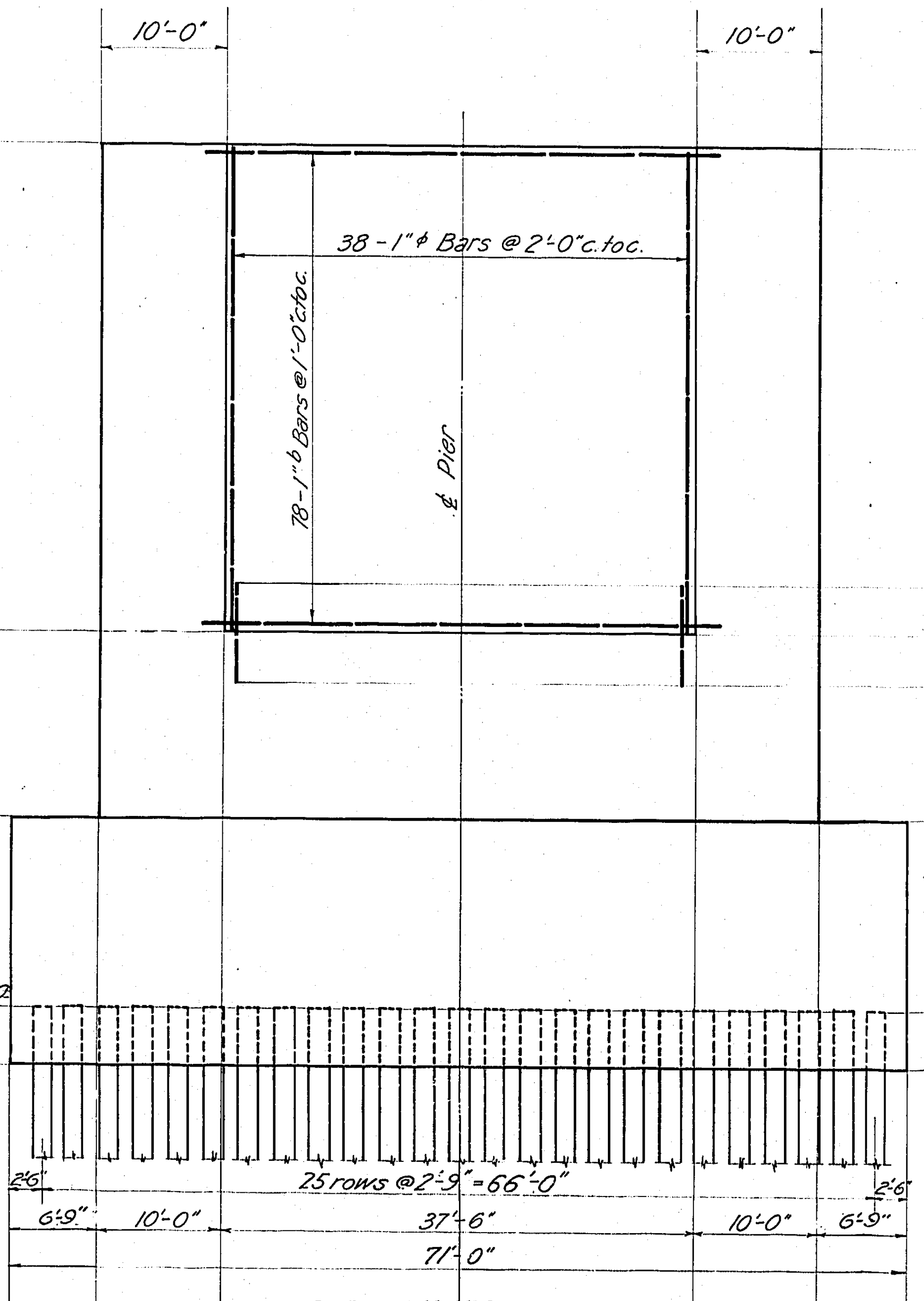
HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB 20, 1924
SHEET No. ⑦

AB1
8

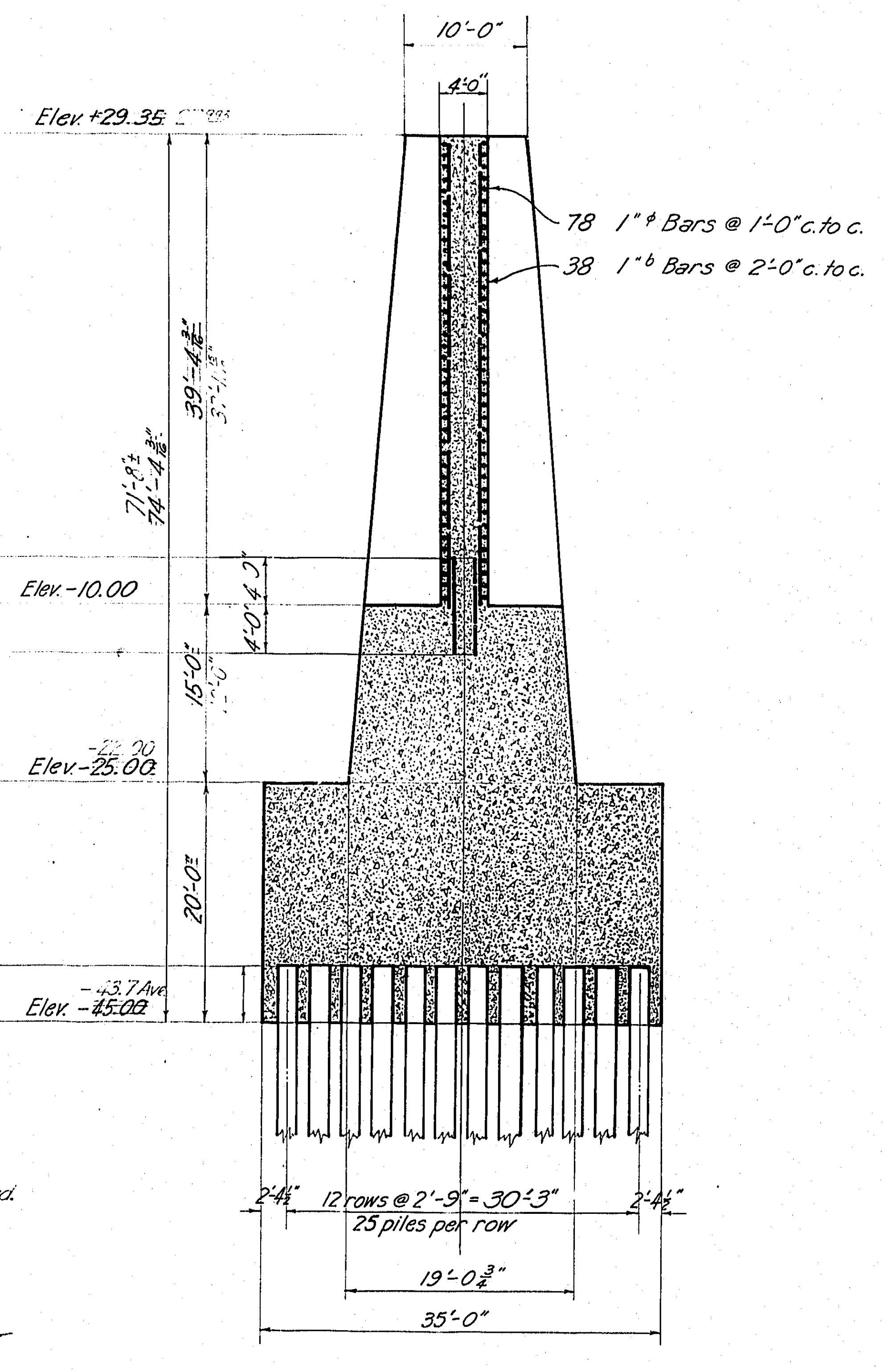
REVISED



SIDE ELEVATION

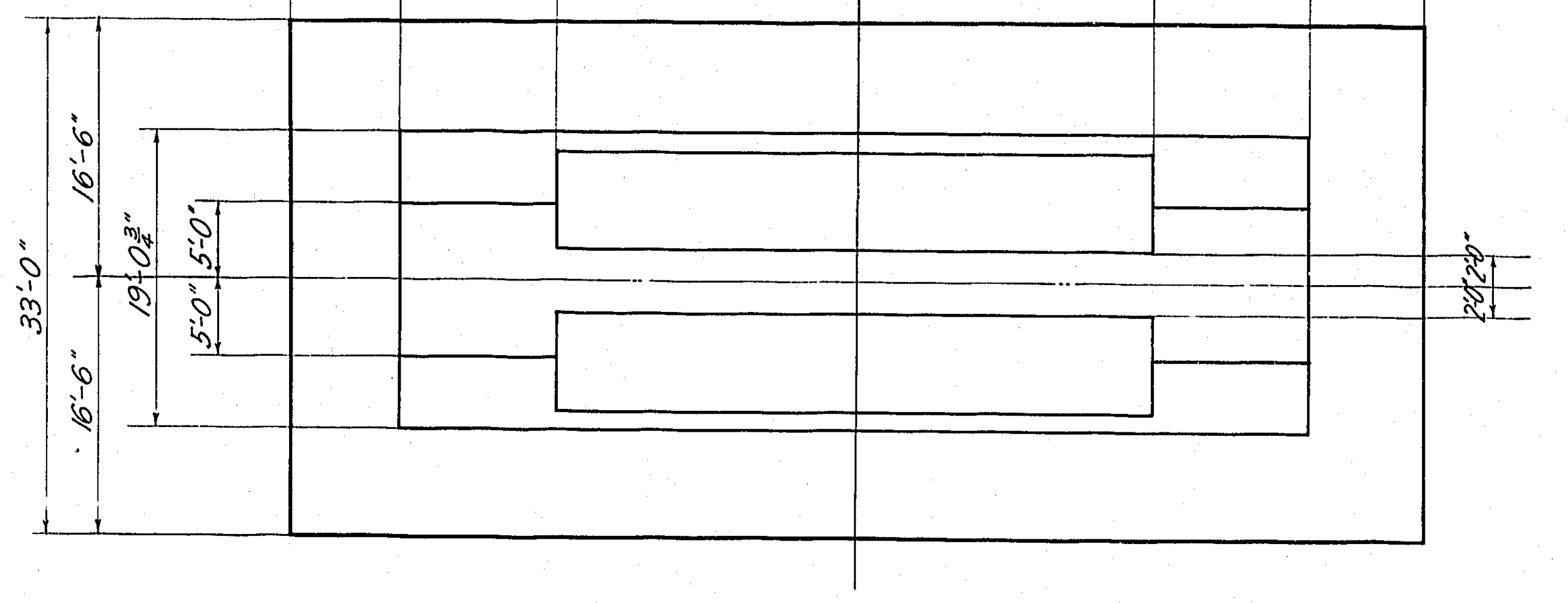


END ELEVATION



SECTION THROUGH CENTER

Piling:-
Number driven = 276
Ave. Elev. point = -74.5
See Pile Record.



PLAN

MULTNOMAH COUNTY — OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

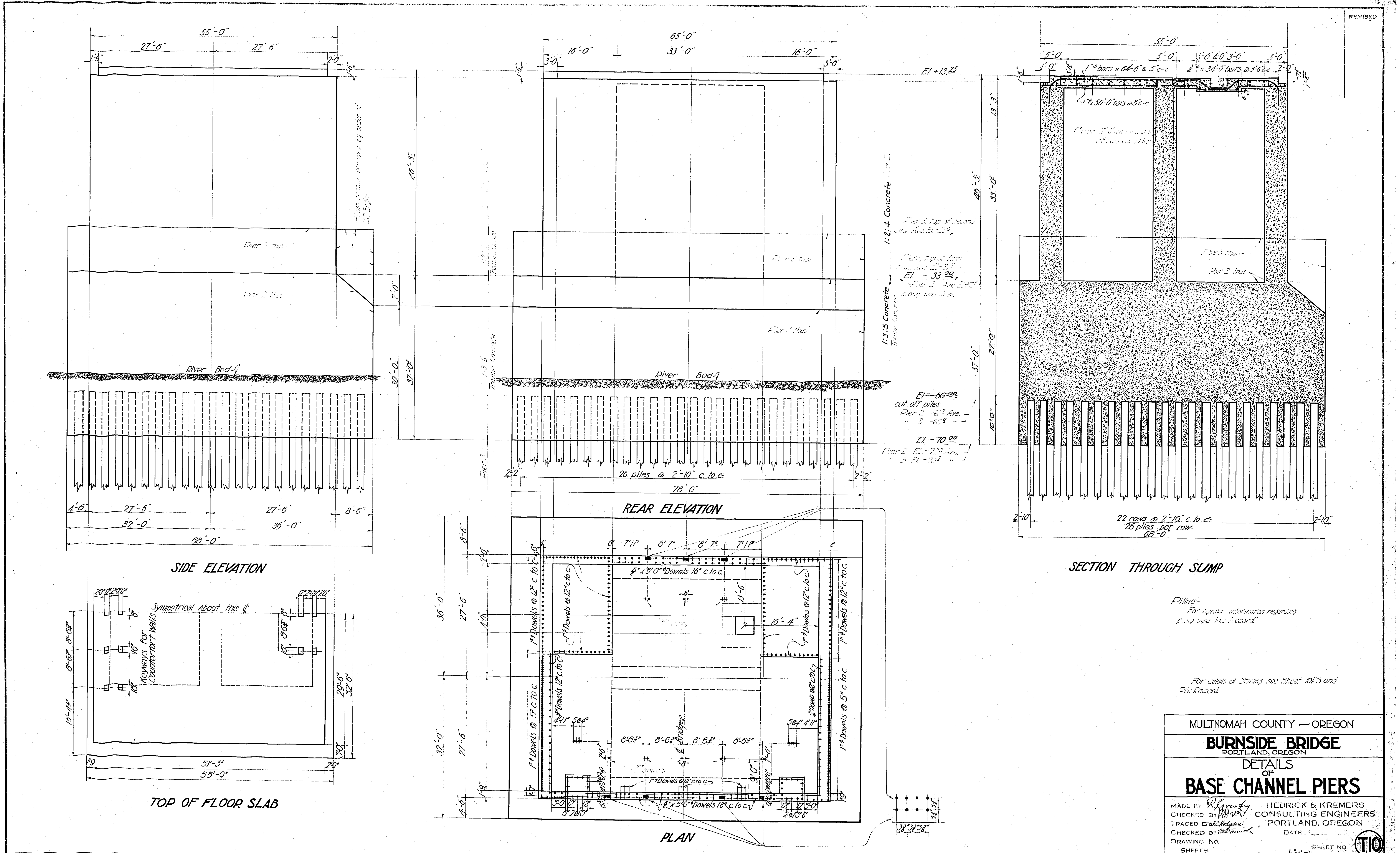
DETAILS
OF
WEST ABUTMENT

MADE BY R. Coventry
CHECKED BY
TRACED BY
CHECKED BY
DRAWING NO.
SHEETS

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE

SHEET NO. T ⑧

ABI
9



Piling:-
For further information regarding piling see "Pile Record"

For details of Staining see Sheet 1 of 3 and Pile Record.

MULTNOMAH COUNTY — OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

DETAILS
OF
BASE CHANNEL PIERS

MADE BY R. Coventry
CHECKED BY
TRACED BY F.L. Hodgdon.
CHECKED BY W. D. Smith
DRAWING NO.
SHEETS

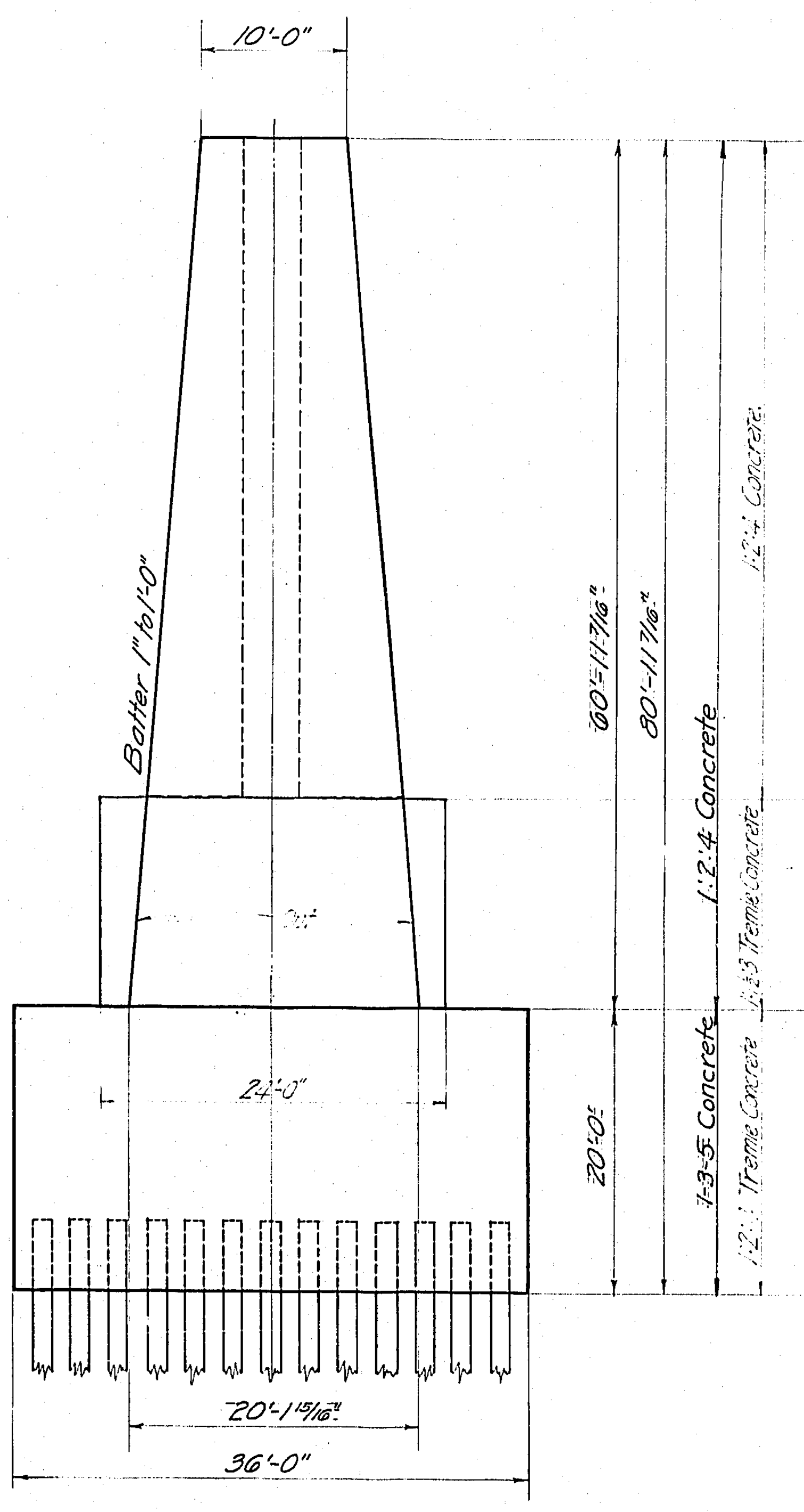
HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE

SHEET NO. T10
SCALE ⅛" = 1'-0"

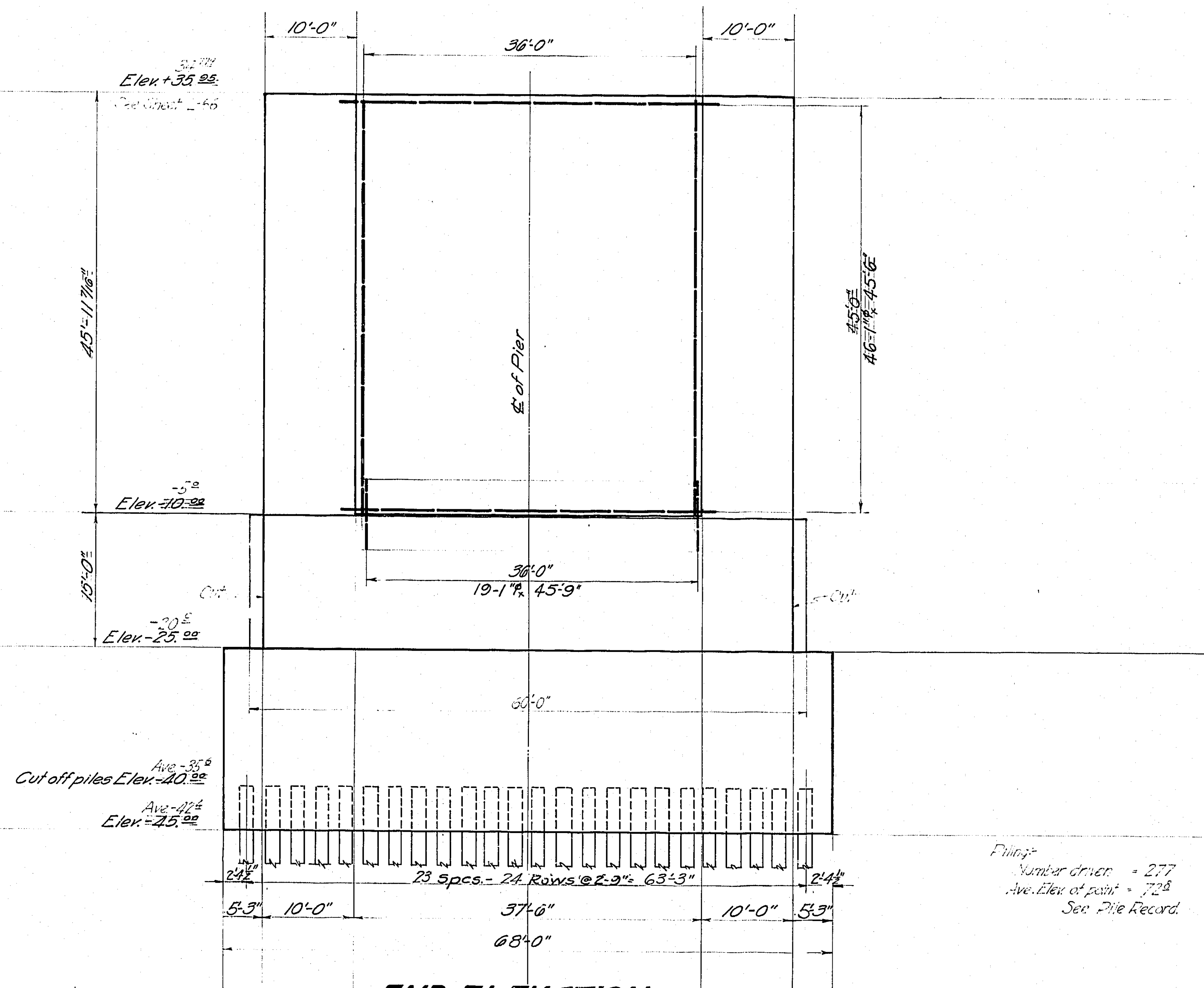
REVISED

AB1
T1

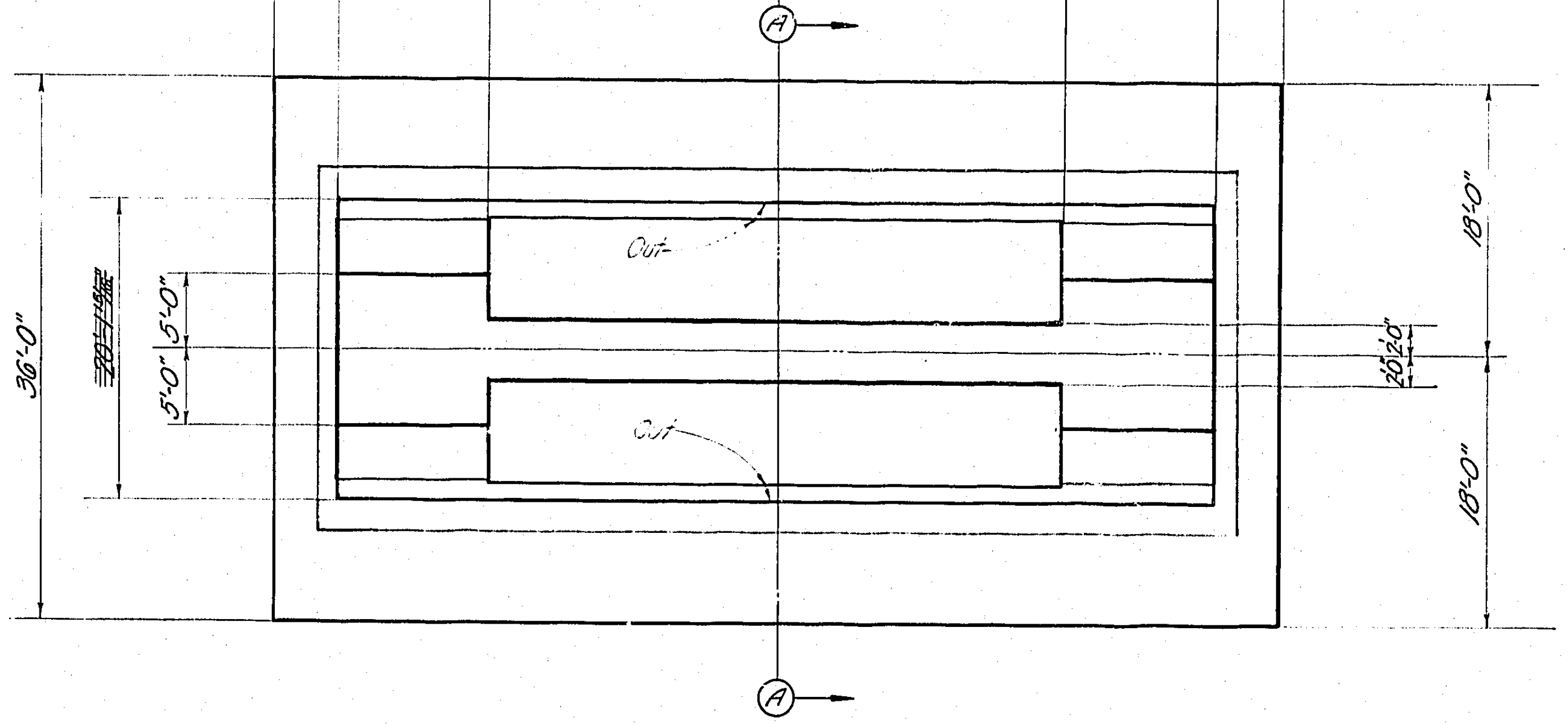
REVISED



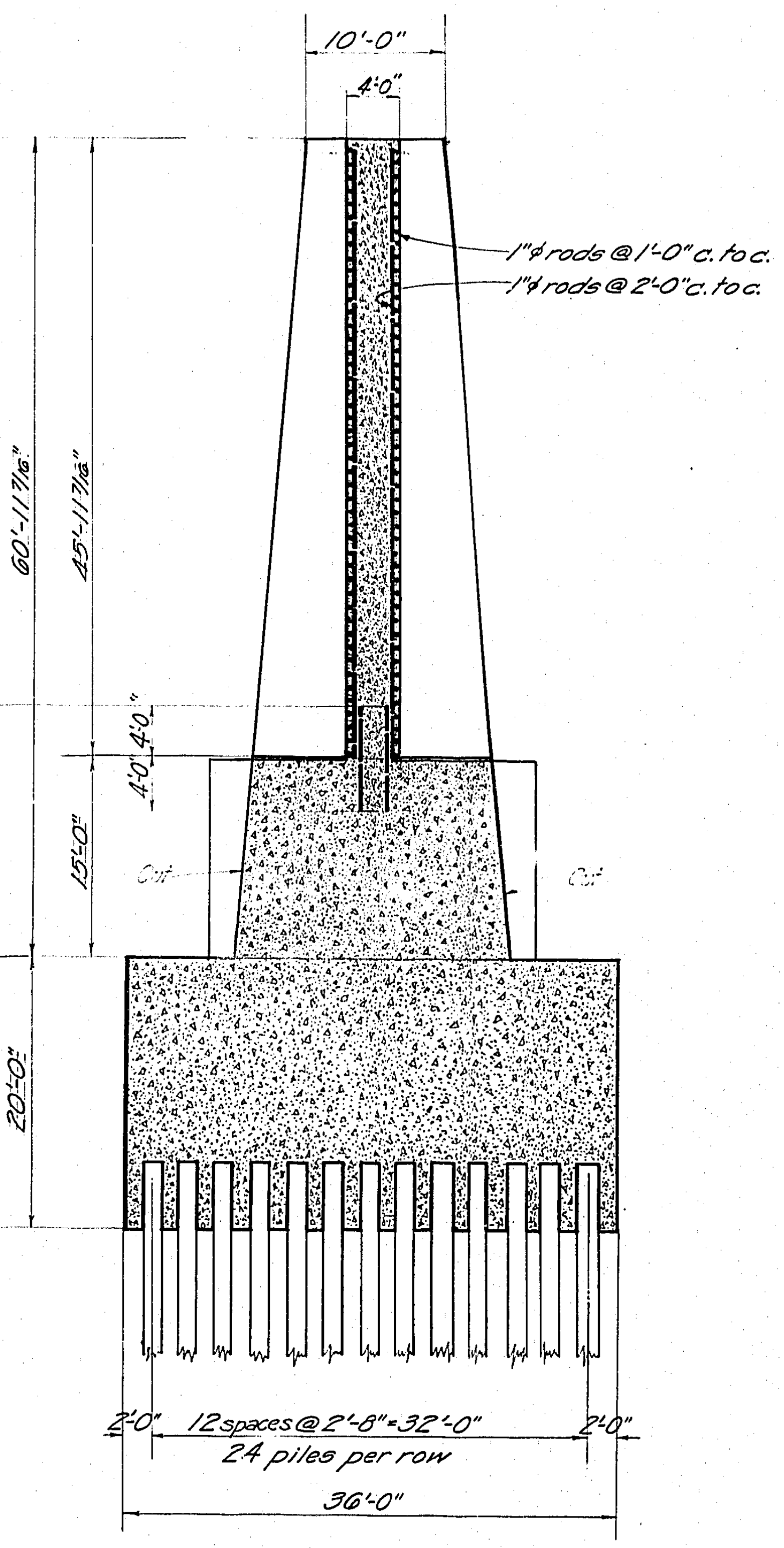
SIDE ELEVATION



END ELEVATION



PLAN



SECTION A-A

MULTNOMAH COUNTY – OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

DETAILS
OF
EAST ABUTMENT

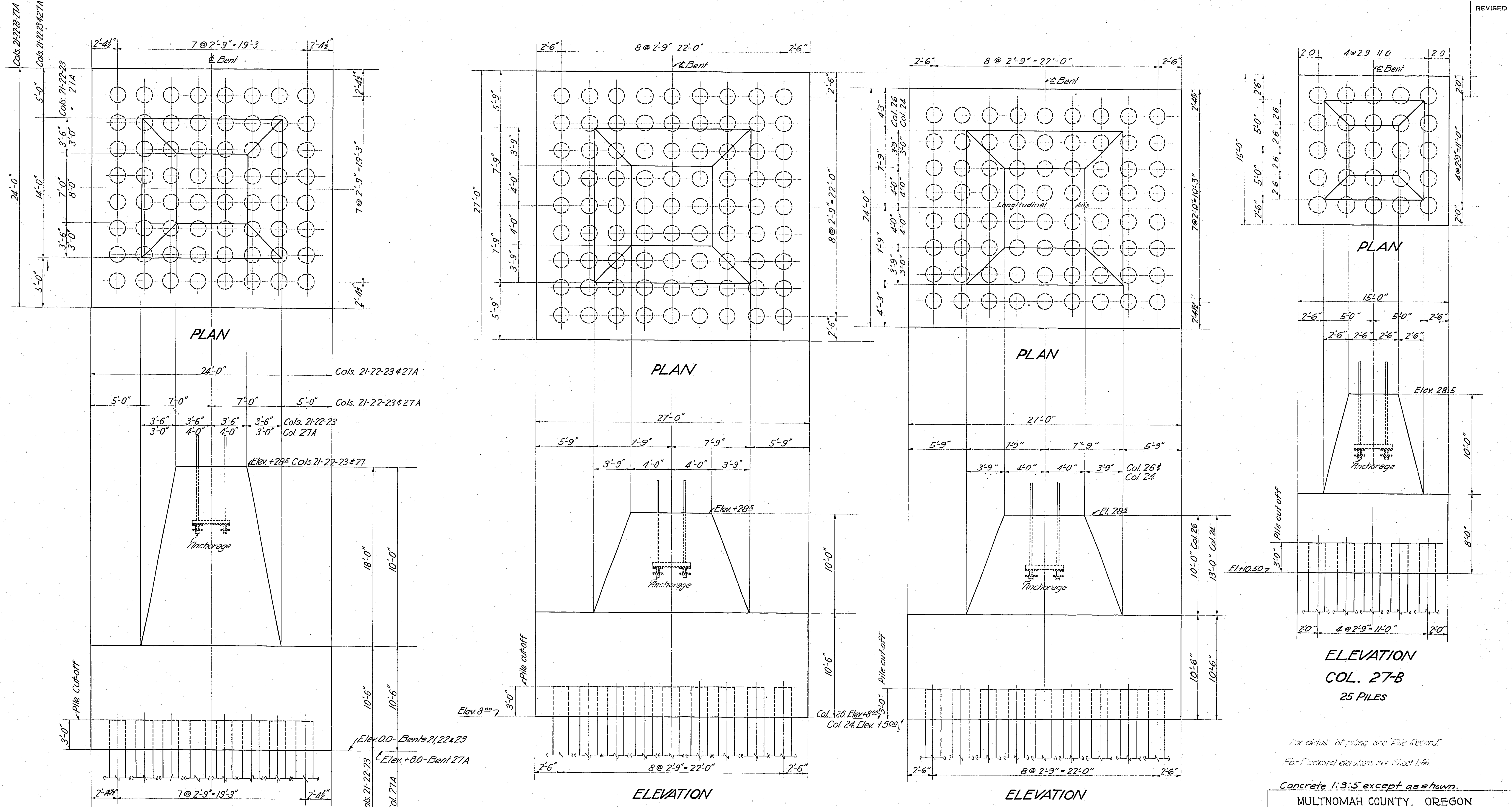
MADE BY R. County
CHECKED BY
TRACED BY F.C. Hodgson
CHECKED BY W.D. Smith
DRAWING NO.
SHEETS

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB. 14, 1924.

SHEET NO. T ⑯

AB1
17

F2-25



ELEVATION
COLS. 21-22-23-27A
64 PILES

ELEVATION
COL. 25
81 PILES

ELEVATION
COLS. 24&26
72 PILES

ELEVATION
COL. 27-B
25 PILES

For details of piling see "Pile Record."

For Recorded elevations see Sheet 56.

Concrete 1:3:5 except as shown.

General Notes:-
All piles are to be accurately spaced as indicated, and cut off to horizontal plane at correct elevation.
All anchorages and anchor bolts for columns - are to be set in exact position and held at top - by means of rigid wooden template.

Note: Elevations of all piers show sides parallel to longitudinal axis.

MULTNOMAH COUNTY, OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

DETAILS
OF
COLUMN FOOTINGS, PLATE GIRDER SPANS

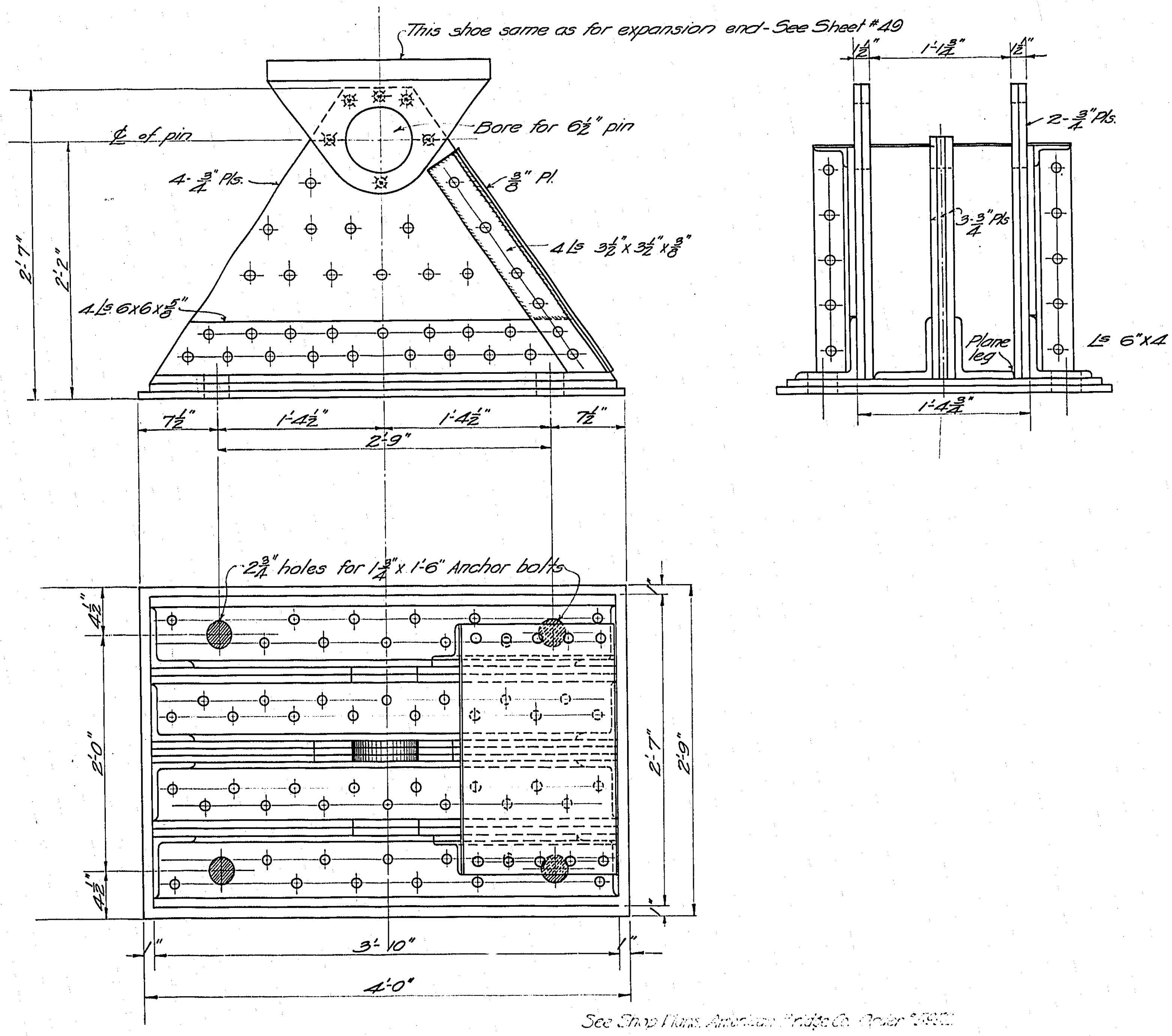
MADE BY ABWright
CHECKED BY
TRACED BY HRMorean
CHECKED BY
DRAWING NO.
SHEETS

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB. 18, 1924

Scale ¼" = 1'-0"
SHEET NO. ⑱

ABI/19 FL-25

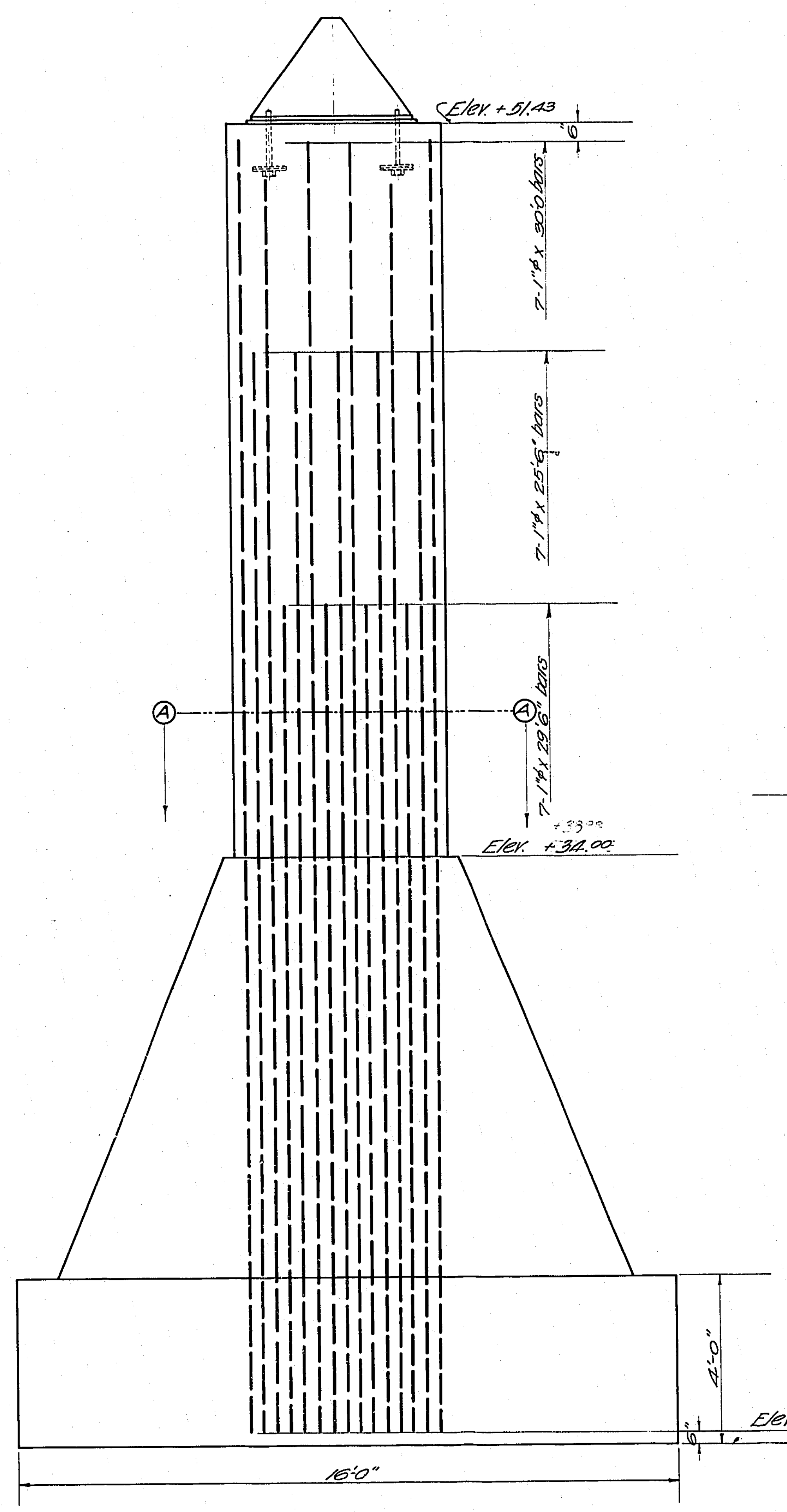
REVISED



See Shop Plans American Bridge Co. Order #[illegible]

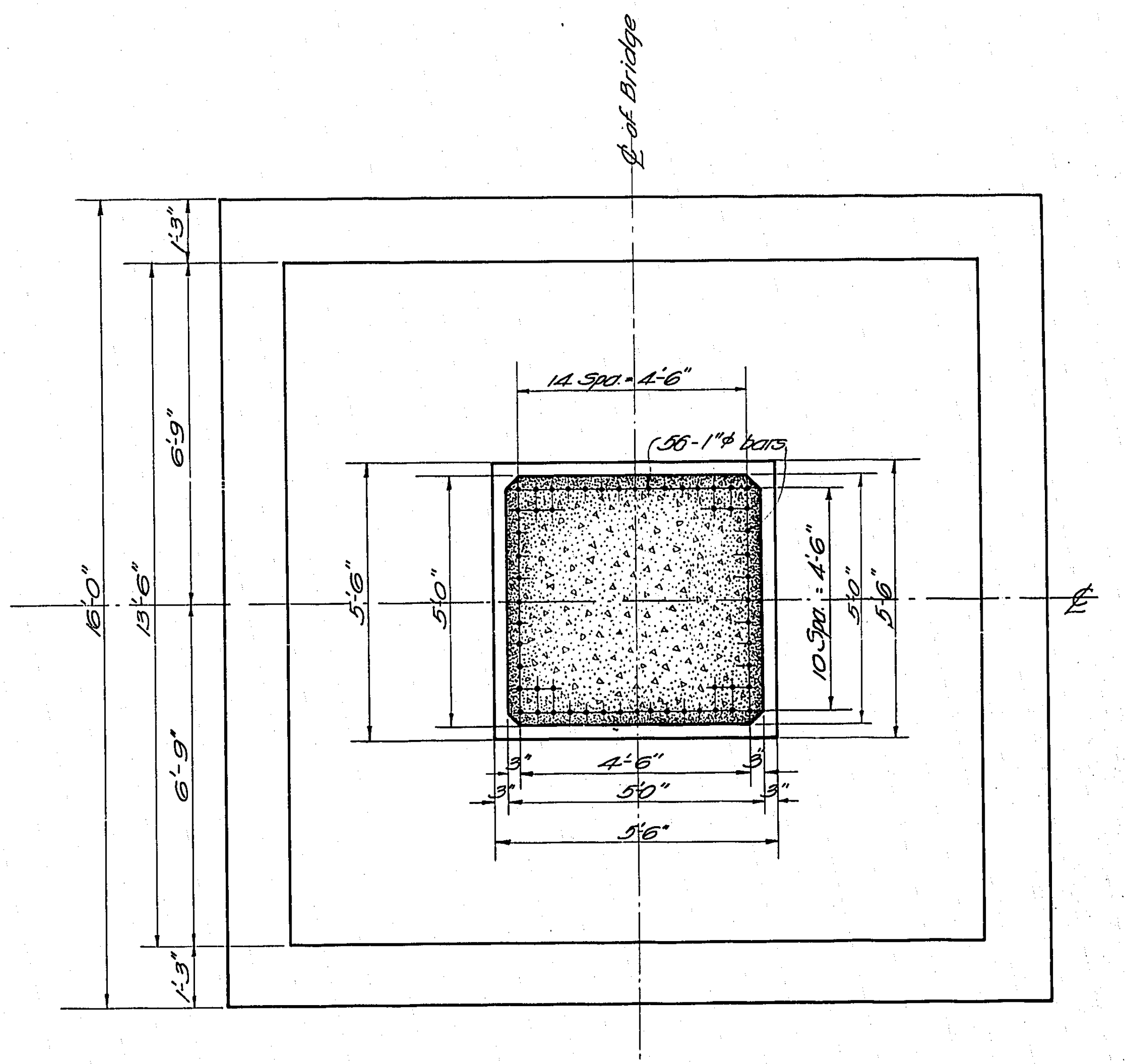
FIXED SHOE - COL. 28 3-Required

Scale - 1½" = 1'-0"



ELEVATION ON TRANSVERSE AXIS

Scale - ½" = 1'-0"



SECTION A-A

Scale ½" = 1'-0"

Concrete 1:2:4

MULTNOMAH COUNTY, OREGON

BURNSIDE BRIDGE

PORTLAND, OREGON

DETAILS
- OF -
BENT NO. 28

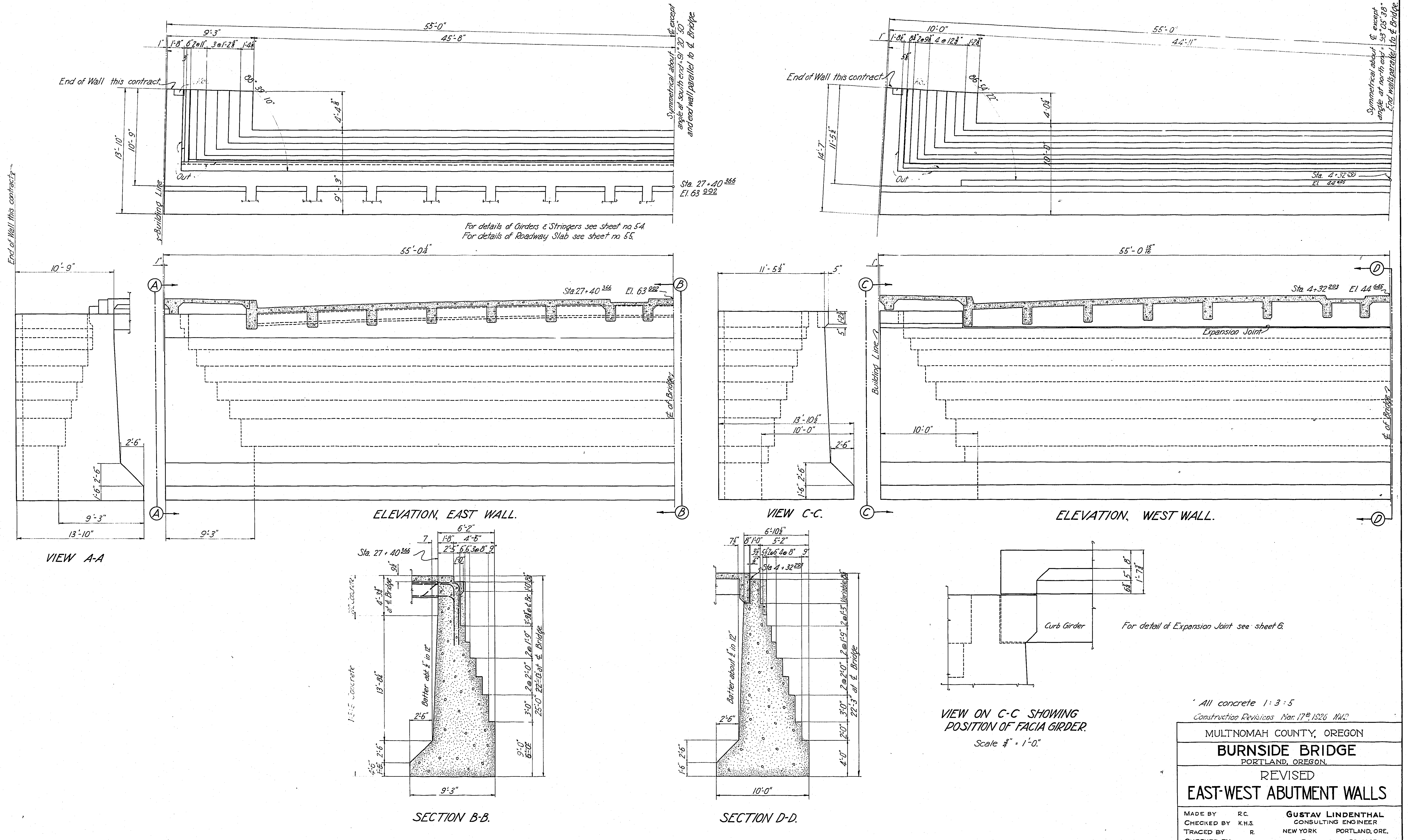
MADE BY Dont.
CHECKED BY
TRACED BY F.S. Hilsche
CHECKED BY
DRAWING NO.
SHEETS

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB. 20-24

SHEET NO. ㊽

SCALE AS SHOWN

~~Fees~~ A.B.



All concrete 1:3:5

Construction Revisions Mar. 17th 1926 N.W.R.

MULTNOMAH COUNTY, OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON.

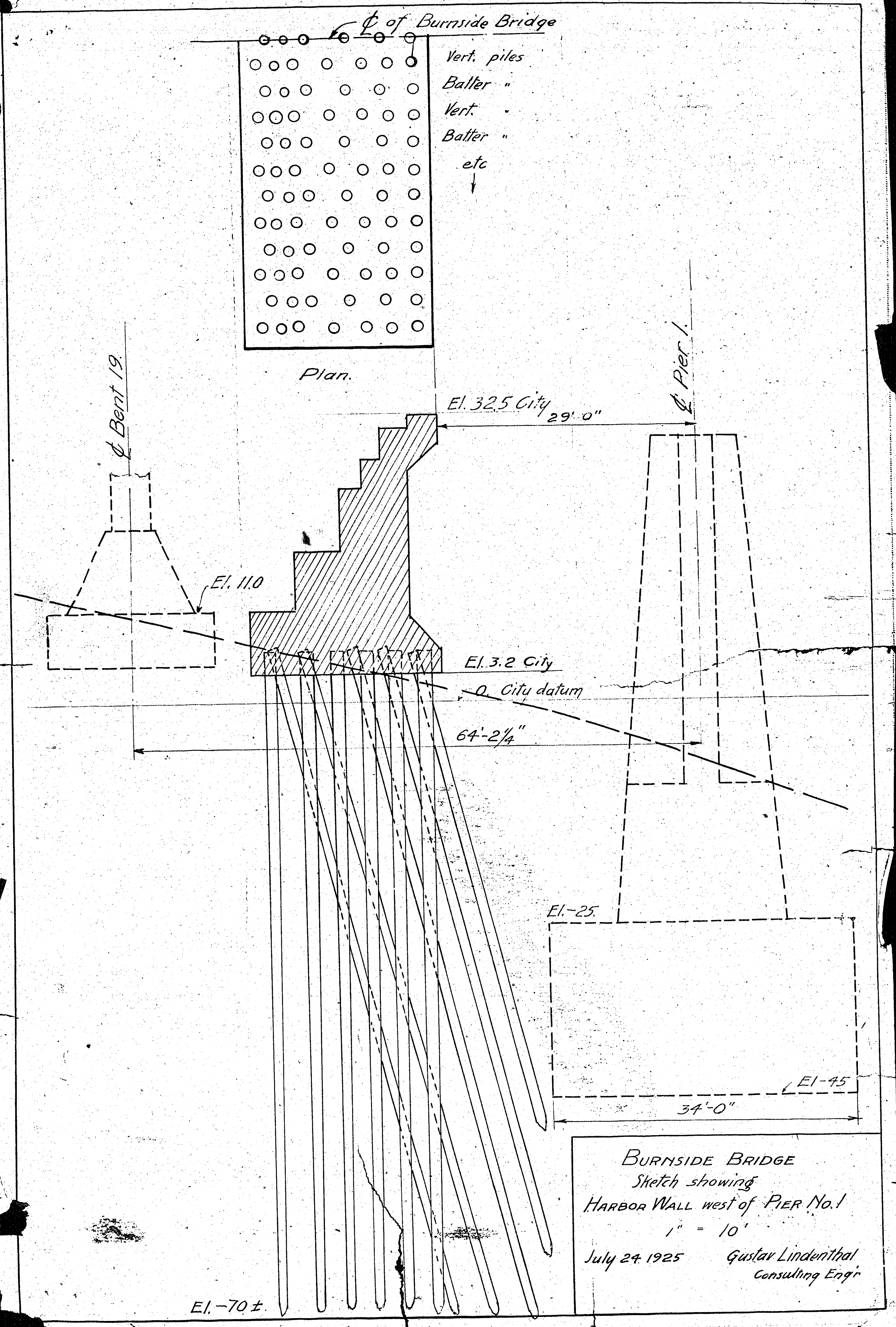
REVISED
EAST-WEST ABUTMENT WALLS

MADE BY R.C.	**GUSTAV LINDENTHAL**
CHECKED BY K.H.S.	CONSULTING ENGINEER
TRACED BY R.	NEW YORK PORTLAND, ORE.
CHECKED BY	DATE - APRIL 29, 1925.
DRAWING NO. GL 185-B. Scale - ¼" = 1'-0"	SHEET NO. L-75 PORTLAND OFFICE

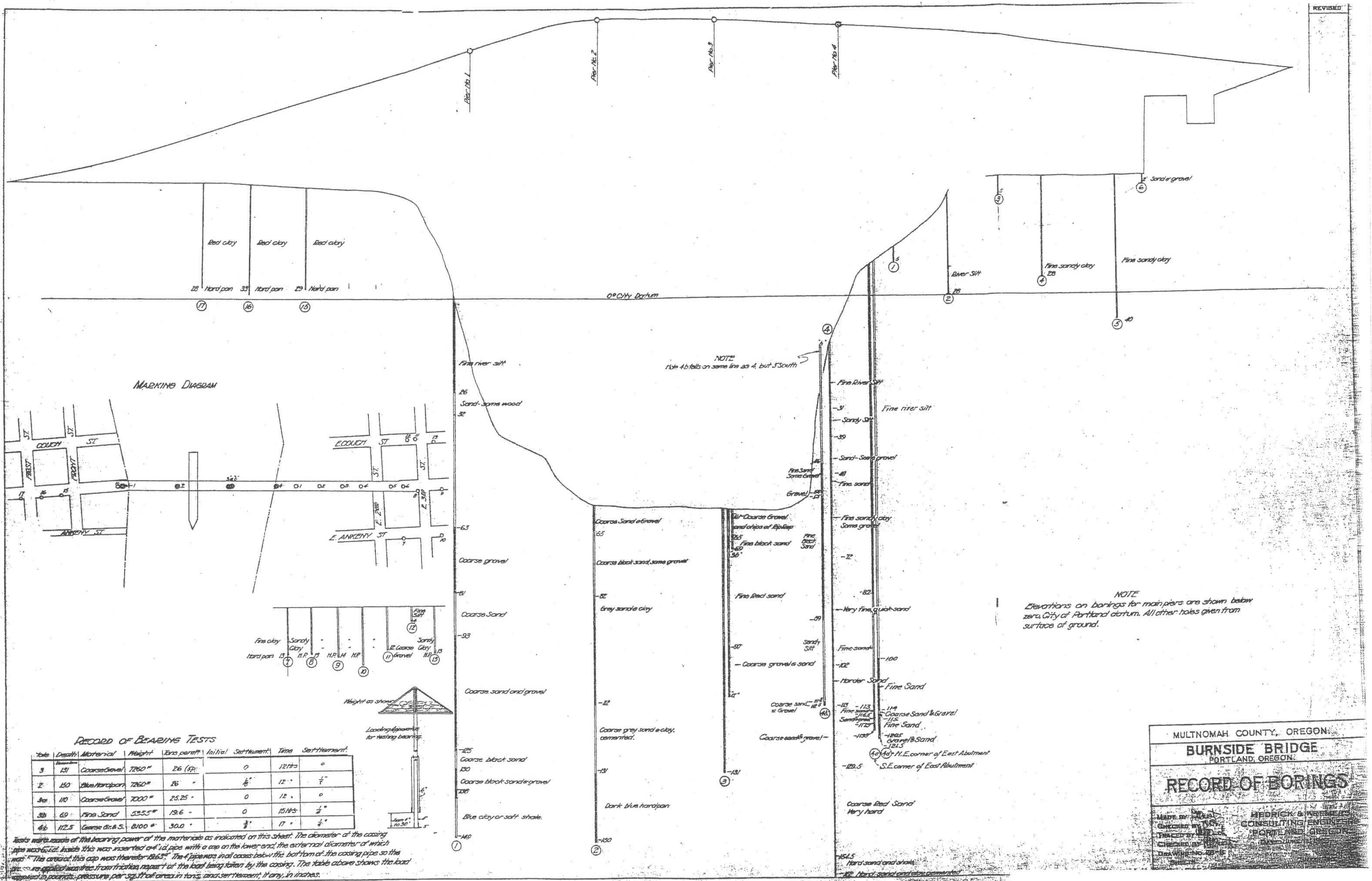
AB1
83 ~~F-24~~

BURNSIDE BRIDGE FOUNDATION PILING SUMMARY

Pier or Bent	Piling as per Plan			Piling as Driven			Deductions Total Length
	No.	Length each	Total Length	No.	Average Length	Total Length	
17N.	61	49	2,989	None			2,989
17S.	61	49	2,989	None			2,989
18N.	70	59	4,130	68	$13^{\underline{8}}$	937	3,193
18S.	70	59	4,130	71	$12^{\underline{7}}$	901	3,229
19N.	70	65	4,550	59	$44^{\underline{5}-}$	2,624	1,926
19S.	70	65	4,550	50	$31^{\underline{6}+}$	1,580	2,970
1	300	40	12,000	276	$39^{\underline{7}}$	10,957	1,043
2	572	50	28,600	382	$34^{\underline{7}-}$	13,249	15,351
3	572	50	28,600	392	$33^{\underline{8}}$	13,255	15,345
4	312	80	24,960	277	$37^{\underline{2}-}$	10,300	14,660
21N.	64	98	6,272	63	$72^{\underline{2}}$	4,549	1,723
21S.	64	98	6,272	63	$81^{\underline{4}}$	5,131	1,141
22N.	64	98	6,272	61	$63^{\underline{8}}$	3,891	2,381
22S.	64	98	6,272	63	$64^{\underline{2}}$	4,045	2,227
23N.	64	98	6,272	62	$59^{\underline{5}}$	3,690	2,582
23S.	64	98	6,272	64	$63^{\underline{7}}$	4,074	2,198
24N.	72	98	7,056	72	$63^{\underline{2}}$	4,548	2,508
24S.	72	98	7,056	72	$61^{\underline{7}}$	4,445	2,611
25N.	81	99	8,019	77	$70^{\underline{7}}$	5,446	2,573
25S.	81	99	8,019	79	$67^{\underline{7}}$	5,353	2,666
26N.	72	99	7,128	70	$72^{\underline{0}}$	5,039	2,089
26S.	72	99	7,128	68	$67^{\underline{3}}$	4,574	2,554
27N.	64	99	6,336	63	$62^{\underline{5}}$	3,940	2,396
27C.	25	$101^{\underline{5}}$	$2,537^{\underline{5}}$	25	$63^{\underline{0}}$	1,575	963
27S.	64	99	6,336	64	$63^{\underline{9}}$	4,090	2,246



BURNSIDE BRIDGE
Sketch showing
HARBOR WALL west of PIER No. 1
1" = 10'
July 24 1925
Gustav Lindenthal
Consulting Engr



RECORD OF BEARING TESTS

Hole	Depth Below datum	Material	Weight	Tons per ft²	Initial Settlement	Time	Settlement
3	131	Coarse Gravel	7260#	26 (sp.	0	12 Hrs	0
2	150	Blue Hardpan	7260#	26 "	1/16"	12 "	1/4"
3a	110	Coarse Gravel	7000#	25.25 "	0	12 "	0
3b	69	Fine Sand	5355#	19.6 "	0	15 Hrs	1/4"
4b	112.5	Coarse Gr. & S.	8100#	30.0 "	3/8"	17 "	1/4"

Tests were made of the bearing power of the materials as indicated on this sheet. The diameter of the casing pipe was 6" i.d. Inside this was inserted a 4" i.d. pipe with a cap on the lower end, the external diameter of which was 5" The area of this cap was therefor 19.63". The 4" pipe was in all cases below the bottom of the casing pipe so the pressure applied was free from friction, no part of the load being taken by the casing. The table above shows the load applied in pounds, pressure per sq. ft. of area in tons, and settlement, if any, in inches.

NOTE

Elevations on borings for main piers are shown below zero, City of Portland datum. All other holes given from surface of ground.

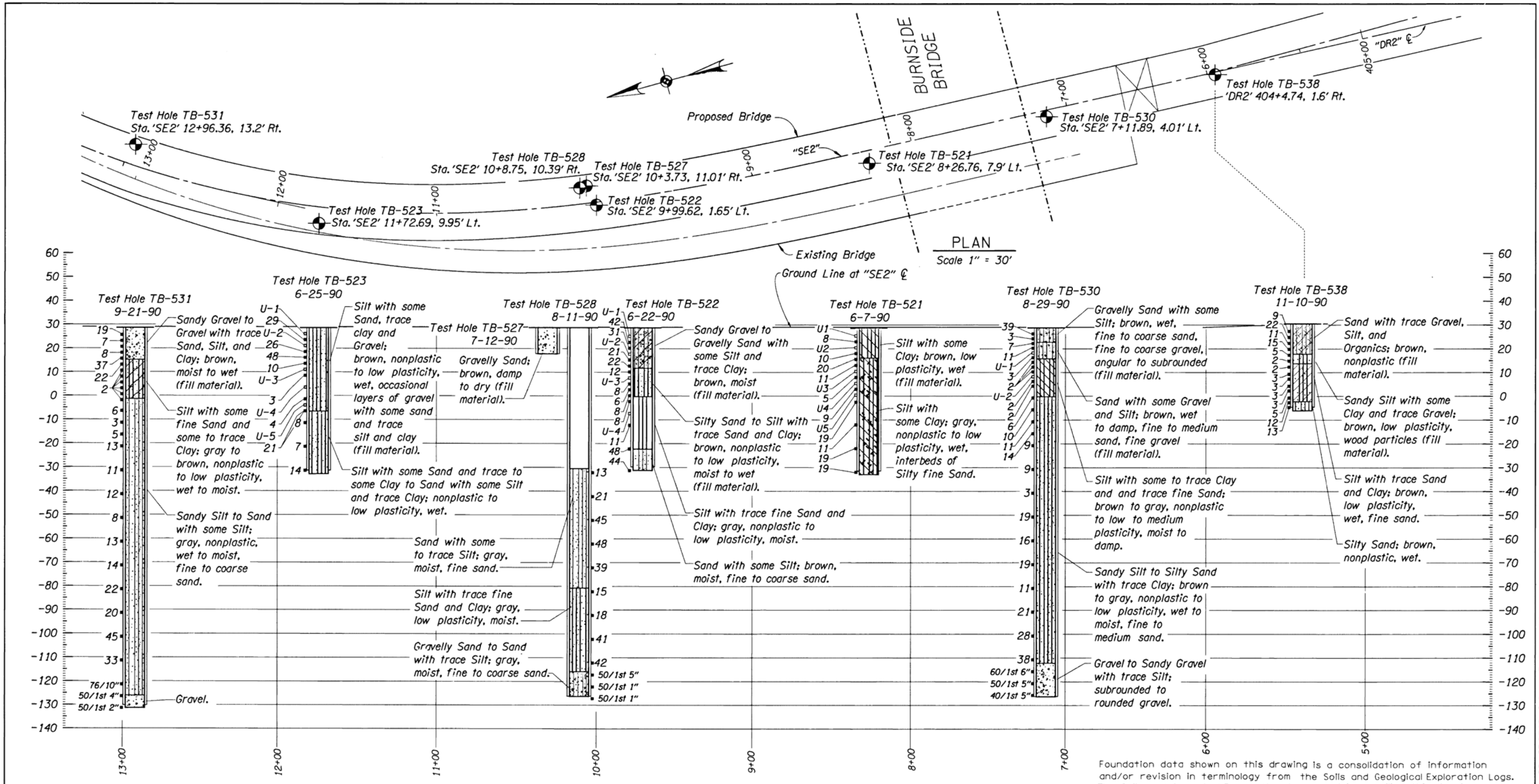
MULTNOMAH COUNTY, OREGON.

BURNSIDE BRIDGE
PORTLAND, OREGON.

RECORD OF BORINGS

MADE BY
CHECKED BY
TRACED BY
CHECKED BY
DRAWING NO. 28-B
SHEETS

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON



PLAN

Scale 1" = 30'

FOUNDATION DATA

Scale 1" = 30' (horiz.)
1" = 20' (vert.)

Foundation data shown on this drawing is a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Geological Exploration Logs used in compiling this drawing are available upon request.

LEGEND OF MATERIALS

▨	Gravel
▨	Sand.
▨	Silt.
▨	Clay.
▨	Silty gravel, gravel-silt mixture.
▨	Silty sand, sand-silt mixture.
▨	Clayey gravel, gravel-clay mixture.
▨	Clayey sand sand-clay mixture.
▨	Silty clay mixture.
▨	Sand-silt-clay mixture.
▨	Siltstone, claystone, mudstone or shale.
▨	Sandstone or conglomerate.
▨	Basalt
☐	

24■ Standard Penetration Test.
N value.

C = Core Sample.
U = Undistrubed Sample.
RQD = Rock Quality Designation.
≚ = Elevation ground water encountered.

Thomas D. Ulay
BRIDGE ENGINEER PE NO. 8491

J. Six
DESIGNED PE NO. 13717
K. Williams
DRAWN R. W. Burns
CHECKED
REVIEWED Marvin Goff

13717
OREGON
JULY 16, 1987
JAN LIEN SIX

DATE	REVISION	BY
11-96	As Constructed	PGA

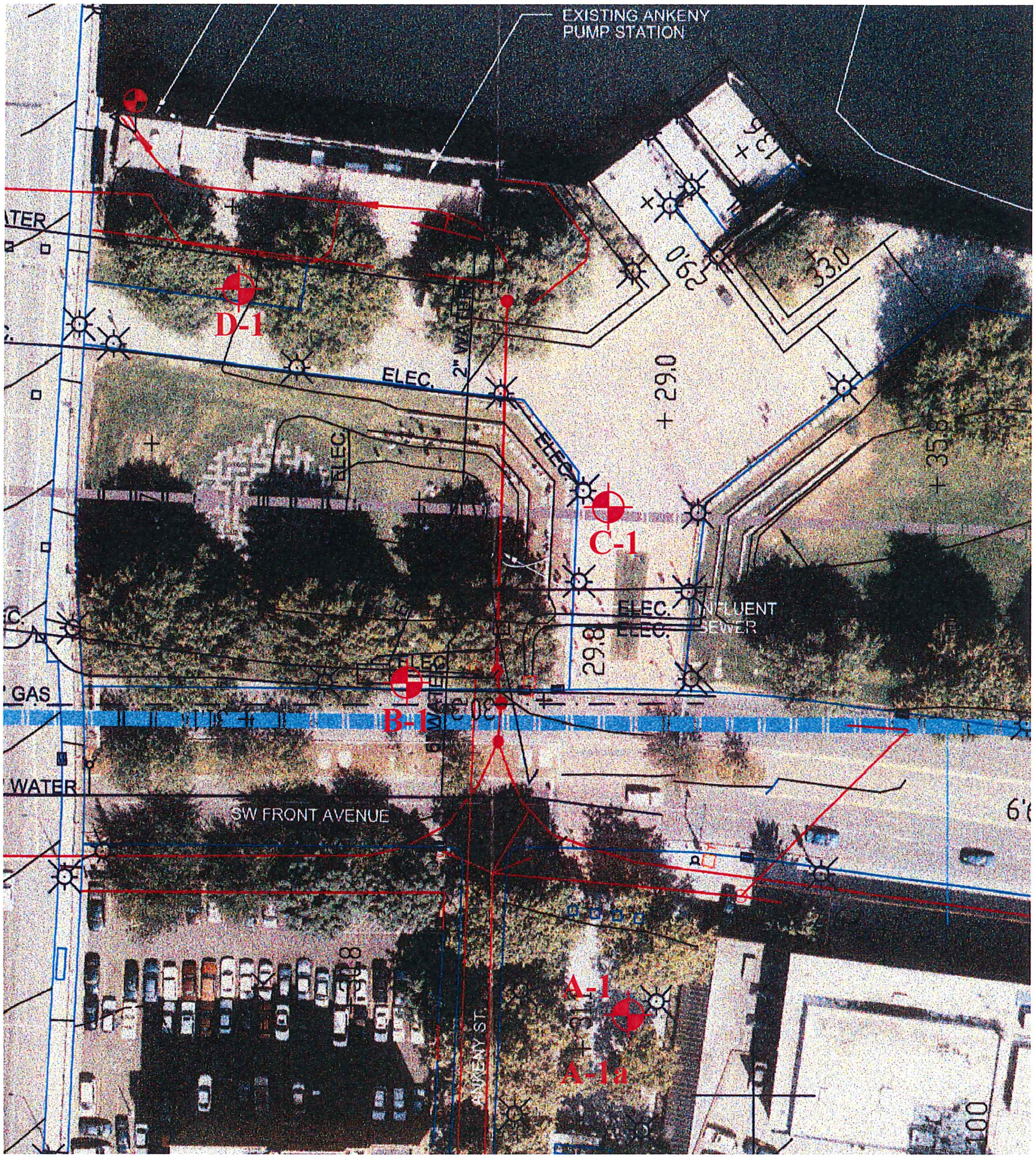
OREGON DEPARTMENT OF TRANSPORTATION
BRIDGE DESIGN SECTION

BANFIELD ACCESS RAMP

FOUNDATION DATA

DATE MAR. 1991 CALC.BOOK Fnd. File SHEET 3 OF 66
BRIDGE NO. 8588D-A DRAWING NO. 47977

ZG2:[200,203]KW8588.DGN [VIEW=2] [PGRID=C1] 15-Oct-1991 HERNANDEZ



Scale 1 Inch = 60 Feet

30 0 30 60

NORTH

Ankeny Pump Station
Portland, Oregon

PLAN OF EXPLORATIONS

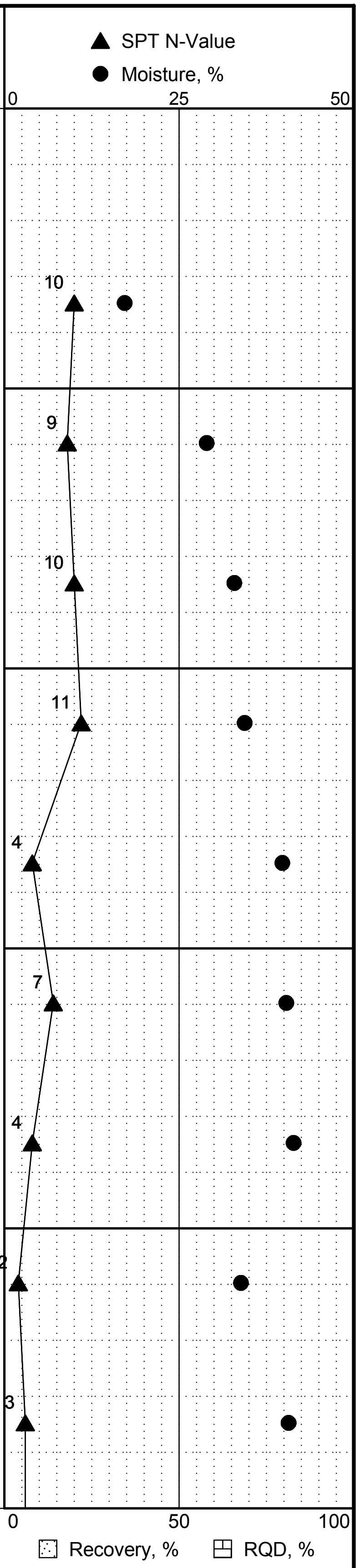
June 2001

F-3112.01

FUJITANI HILTS & ASSOCIATES
Geotechnical Consultants
Portland, Oregon

FIG. 1

Ground Water	Remarks	Elev. Depth Feet 31.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Artificial Granite PAVER (Pavement)		0	
		30.9 0.1	**CONCRETE** (Pavement)			
		30.5 0.5	**SILT FILL**, slightly clayey, stiff, dark gray, moist, low plasticity, scattered crushed rock and charcoal. (Fill)			
		28.0 3.0	**SILT**, slightly sandy, slightly clayey, stiff, light brown & rust w/ gray mottling, moist, low plasticity, micaceous, with scattered rock fragments. (Pleistocene Flood Deposits)			S-1
					5	S-2
						S-3
					10	S-4
		18.5 12.5	**Clayey SILT**, slightly sandy, soft to medium stiff, gray brown w/ rust, moist, low plasticity, micaceous. (Pleistocene Flood Deposits)			S-5
					15	
	-200 = 95%					S-6
		13.5 17.5	Sandy			
	-200 = 87%					S-7
		11.0 20.0	**Sandy SILT**, slightly clayey, soft, light brown, moist becoming wet, low plasticity, micaceous. (Pleistocene Flood Deposits)		20	
	-200 = 57%					S-8
▼	10/16/2000					
	-200 = 71%					S-9
		6.0				



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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING A-1

Page 1 of 3

December 2000 *F-3112.01*

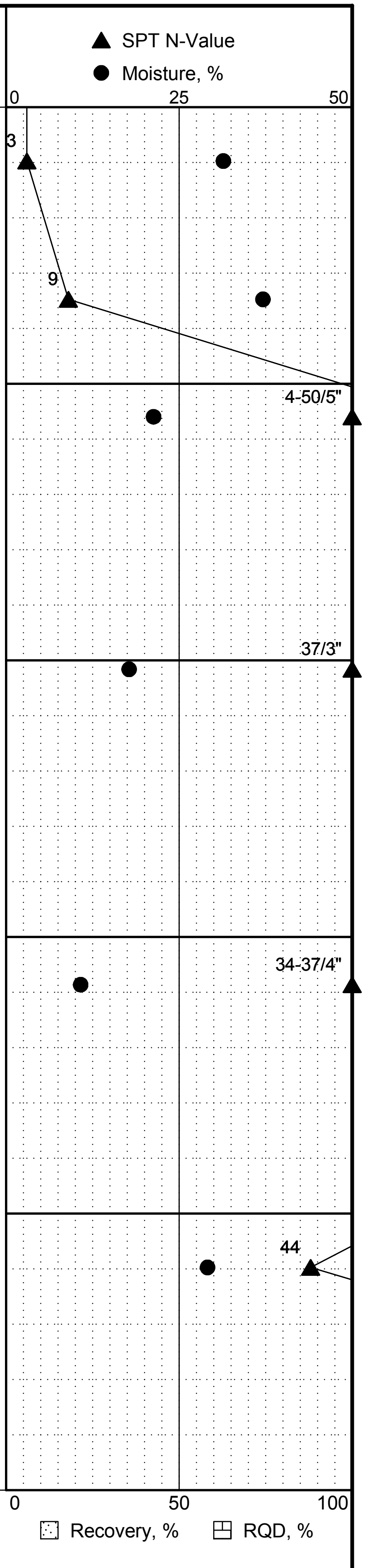
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

Log: Typ: Rev:

WLG ANKPS.GPJ 12/2/16

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	-200 = 25%	25.0	**Silty SAND**, very loose to loose, light brown, wet, low to no plasticity in matrix, fine-grained. (Pleistocene Flood Deposits)			S-10
	-200 = 74%	3.5 27.5	**Sandy SILT**, slightly clayey, stiff, light brown, wet, low to no plasticity, micaceous. (Pleistocene Flood Deposits)			S-11
	-200 = 89%	1.0 30.0	Slightly sandy		30	
		0.5 30.5	**Sandy GRAVEL**, very dense, gray, wet, non-plastic, fine- to coarse-grained sand, fine- to coarse-grained rounded gravel, well-graded, scattered cobbles. (Pleistocene Flood Deposits)		35 40	S-12 S-13 S-14
		-10.5 41.5	**SAND**			
		-11.5 42.5	**Clayey SAND**, dense, brown & orange, damp, low plasticity, micaceous, fine-grained, weathered, indurated. (Troutdale Formation)		45	S-15
		-19.0				



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.

Ankeny Pump Station
Portland, Oregon

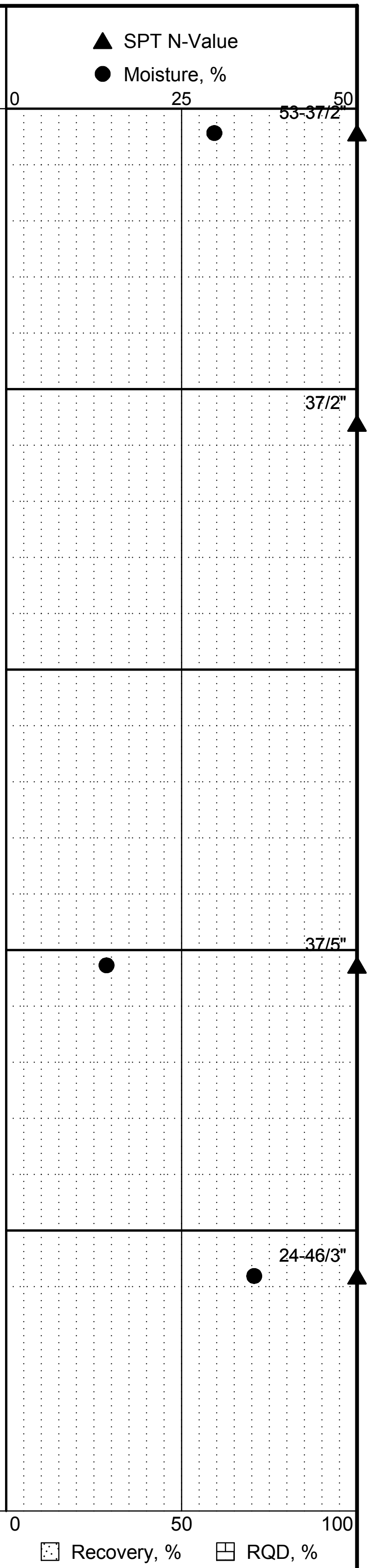
LOG OF BORING A-1
Page 2 of 3

December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
		50.0	**SAND**, slightly clayey, very dense, dark brown, damp, low plasticity in matrix, micaceous, fine-grained, weathered, indurated, scattered rounded cobbles. (Troutdale Formation)			S-16
					55	S-17
					60	
		-33.0 64.0	**Sandy GRAVEL & COBBLES**, very dense, gray and black, wet, fine-grained sand, fine- to coarse-grained, poorly graded, rounded gravel. (Troutdale Formation)		65	S-18
		-36.0 67.0	**SAND**, very dense, brown, moist, non-plastic, micaceous, fine-grained, indurated, trace of clay, scattered cobbles. (Troutdale Formation)			
	Dry density = 122.6 pcf				70	S-19
		-40.2 71.2	Bottom of Boring, Completed 10/17/00			



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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING A-1

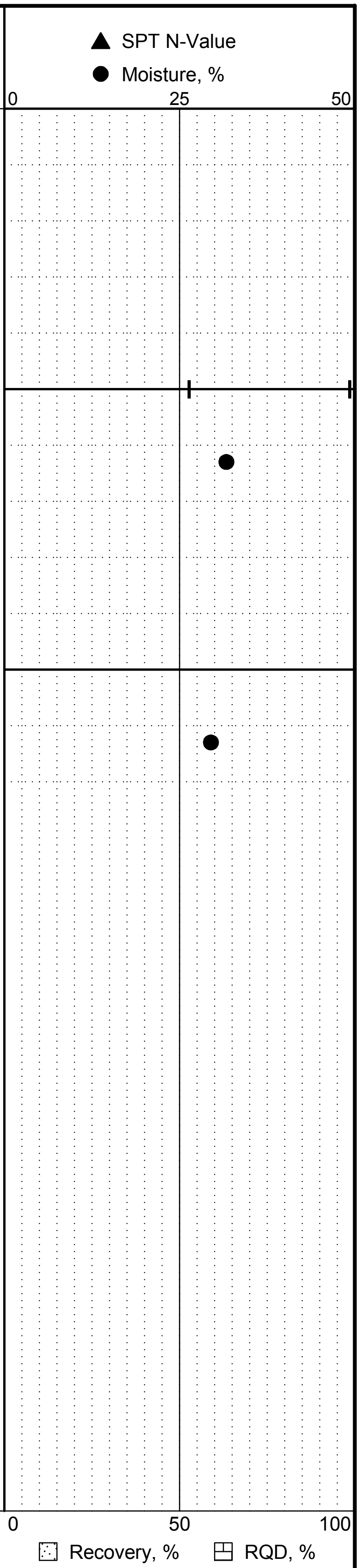
Page 3 of 3

December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

Ground Water	Remarks	Elev. Depth Feet 31.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
					0	
			SILT, slightly sandy, slightly clayey, light brown, moist, low to no plasticity, micaceous. (Pleistocene Flood Deposits)		5	S-1
			Sandy SILT,		10	S-2



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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING A-1a

Page 1 of 1

December 2000 *F-3112.01*

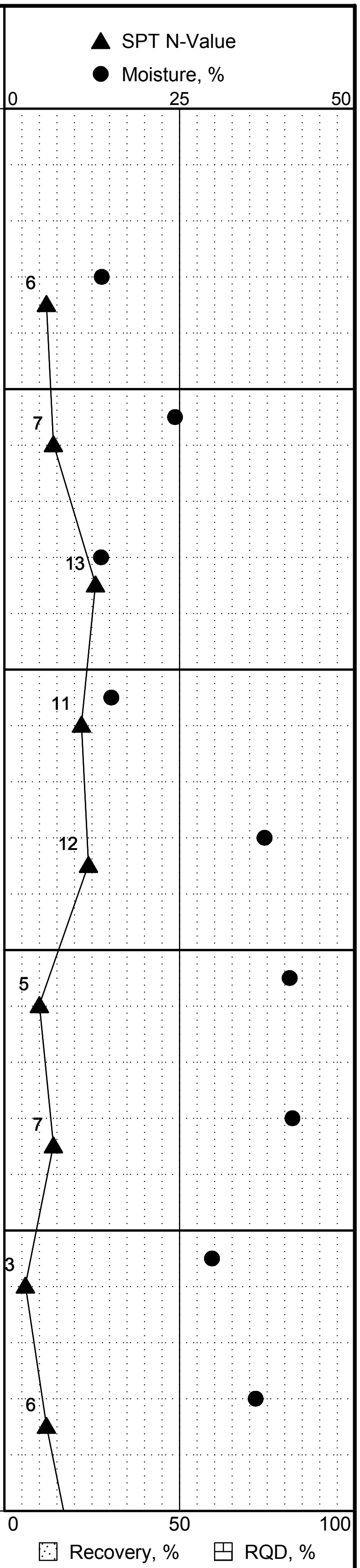
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

WLG ANKPS.GPJ 12/2/16

Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet 32.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			Silty SAND FILL, loose to medium dense, brown, dry, ranging to low plasticity slightly clayey SILT in places, fine-grained, scattered brick fragments to 9.5 feet. (Fill)		0	
						S-1
			Moist.		5	S-2
			Medium dense, scattered concrete fragments.			S-3
					10	S-4
			Scattered organics.			S-5
	-200 = 94%	17.0 15.0	**SILT FILL**, slightly sandy and clayey, medium stiff, light brown, moist, low plasticity, scattered brick fragments and organics. (Fill)		15	S-6
	-200 = 95%					S-7
	-200 = 75%		**Grading Sandy**, soft.		20	S-8
		10.0 22.0	**Sandy SILT**, loose, brown, wet, micaceous, sand is fine-grained. (Pleistocene Flood Deposit)			
▼	8/30/2000 -200 = 65%					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

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

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

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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING B-1

Page 1 of 3

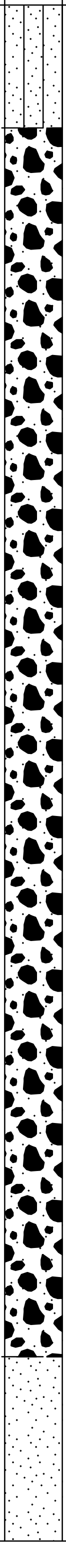
December 2000 *F-3112.01*

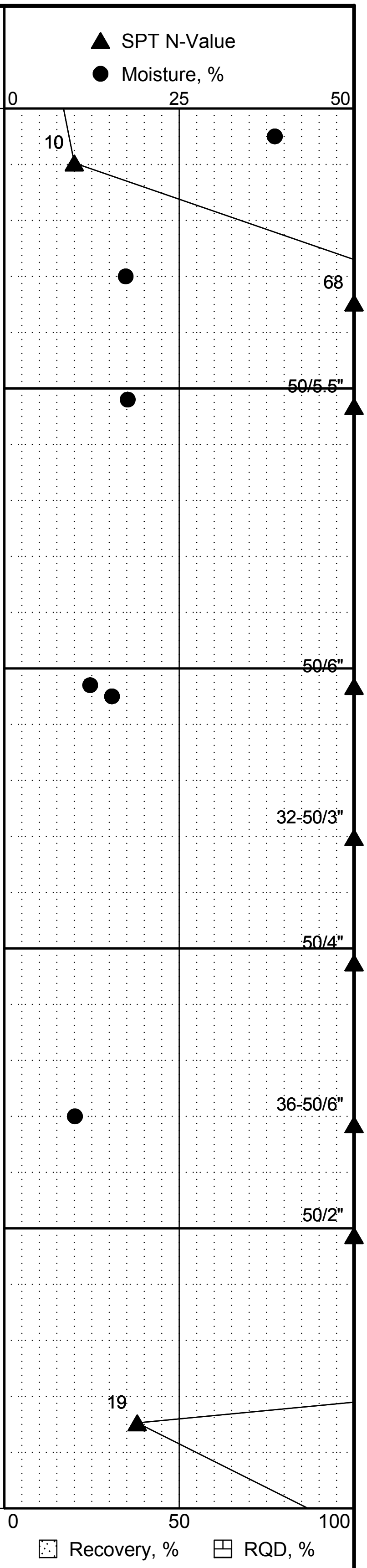
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Depth In Feet	Samples
	-200 = 41%		**Grading to Silty SAND**.		S-10
		5.0 27.0	**Cobbly Sandy GRAVEL**, very dense, brown and gray, wet, fine to coarse subrounded gravel with numerous cobbles, sand is fine- to medium-grained. (Pleistocene Flood Deposits)		S-11
				30	S-12
				35	S-13
					S-14
			Cobbly Sandy GRAVEL continued	40	S-15*
					S-16
				45	S-17*
		-15.0 47.0	**SAND**, medium dense, brown, wet, fine-grained. (Pleistocene Flood Deposits)		S-18

Log



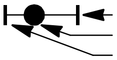


LEGEND

⌶ = 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.

Ankeny Pump Station
Portland, Oregon

LOG OF BORING B-1

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December 2000 *F-3112.01*

SHANNON & WILSON, INC.
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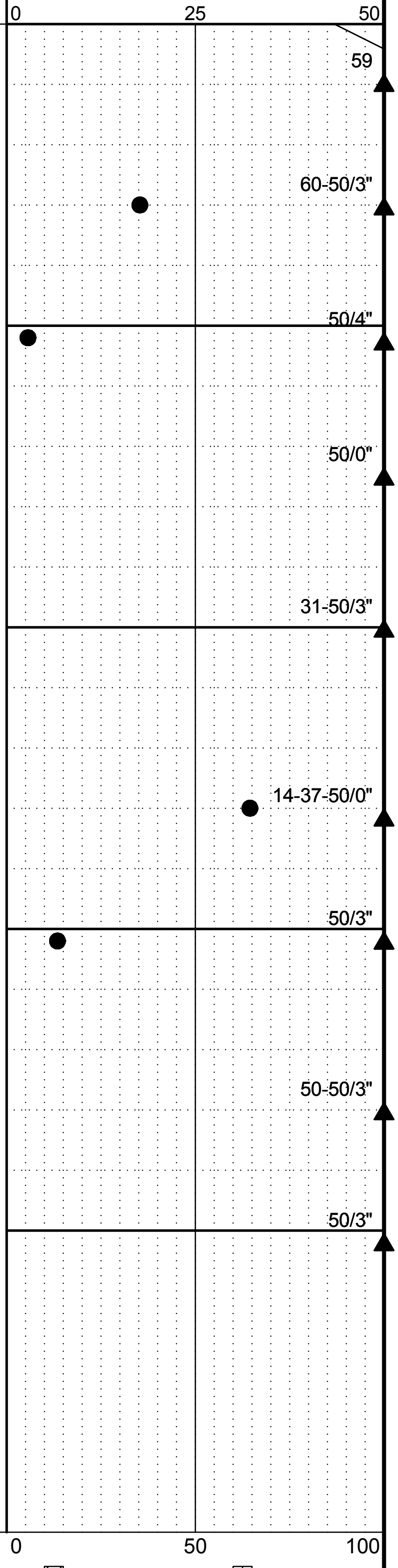
Portland, Oregon

FIG.

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Depth In Feet
		-19.3 51.3	**Sandy GRAVEL**, slightly clayey, very dense; gray, brown and yellow; wet, low plasticity matrix, fine to coarse rounded gravel with scattered cobbles, sand is fine-grained. (Pleistocene Flood Deposits with reworked Troutdale Formation material, possible Troutdale Formation))	55
		-27.5 59.5	**Clayey SAND**, very dense, yellow-brown, moist, low plasticity, fine-grained, scattered fine gravel, weathered, over-consolidated, poorly graded. (Troutdale Formation)	60
		-33.0 65.0	**Sandy GRAVEL**, slightly clayey, very dense; gray, brown and yellow; wet, low plasticity matrix, fine to coarse rounded gravel with scattered cobbles, sand is fine-grained, weathered, over-consolidated, poorly graded. (Troutdale Formation)	65
		-38.3 70.3	Bottom of Boring, Completed 8/31/00	70

Samples	SPT N-Value
S-19	59
S-20	60-50/3"
S-21	50/4"
S-22*	50/0"
S-23	31-50/3"
S-24	14-37-50/0"
S-25	50/3"
S-26	50-50/3"
S-27	50/3"

▲ SPT N-Value
● Moisture, %



⊡ 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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING B-1

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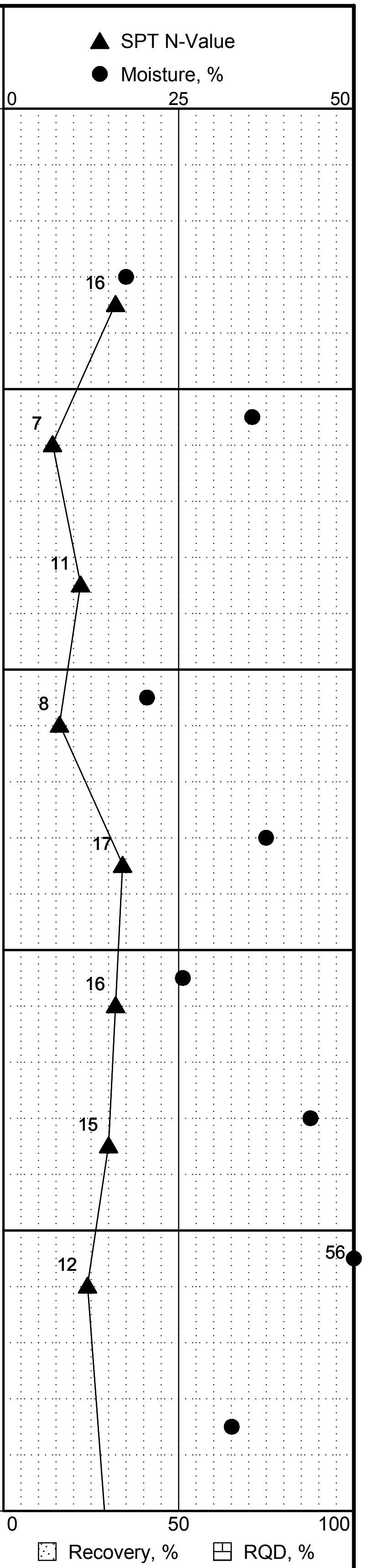
FIG.

WLG ANKPS.GPJ 12/2/16 Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet 30.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			BRICK and CONCRETE FILL; BRICK from 0 to 0.2, CONCRETE with rebar from 0.2 to 0.85. (Pavement)		0	
		29.2 0.9	**SAND FILL**, medium dense, gray, moist, fine-grained, trace fine rounded gravel and crushed rock. (Fill)	▨		S-1
		26.0 4.0	**Sandy SILT FILL**, slightly clayey, loose, light brown, moist, no to low plasticity, scattered fine rounded gravel. (Dredged sand fill)	▨	5	S-2 S-3
		20.0 10.0	**SAND FILL**, loose, gray brown, moist, fine-grained, scattered fine rounded gravel. (Dredged sand fill) Tree fragment, medium dense.	▨	10	S-4 S-5
		16.0 14.0	**Silty SAND FILL**, slightly clayey, medium dense, gray, moist to wet, no to low plasticity, fine-grained, scattered to abundant wood fragments and red crushed rock.	▨	15	S-6
	-200 = 33%			▨		S-7
▼	8/28/2000 -200 = 35%			▨	20	S-8
	-200 = 16%			▨		S-9
		5.0				



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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1

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December 2000 *F-3112.01*

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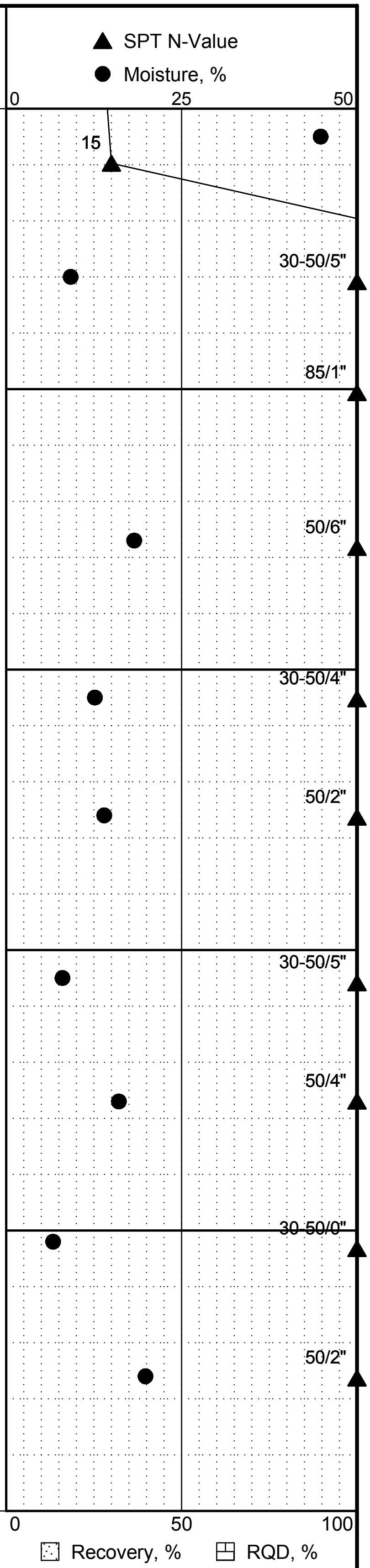
Portland, Oregon

FIG.

WLG ANKPS.GPJ 12/2/16 Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	-200 = 73%	25.0	**Clayey SILT**, stiff, gray, wet, low plasticity, micaceous. (Recent Alluvium)			S-10
		3.0 27.0	**Stratified Cobbly Sandy GRAVEL**, very dense, brown and gray, wet, fine to coarse subrounded gravel, sand is fine- to coarse-grained. (Pleistocene Flood Deposits)			S-11
					30	S-12*
						S-13
					35	S-14
						S-15
			Stratified Cobbly Sandy GRAVEL continued.		40	S-16
						S-17
					45	S-18
						S-19
	Wet density = 163.3 pcf	-18.5 48.5	**SAND,** very dense, no sample return. (Pleistocene Flood Deposits)			



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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING C-1

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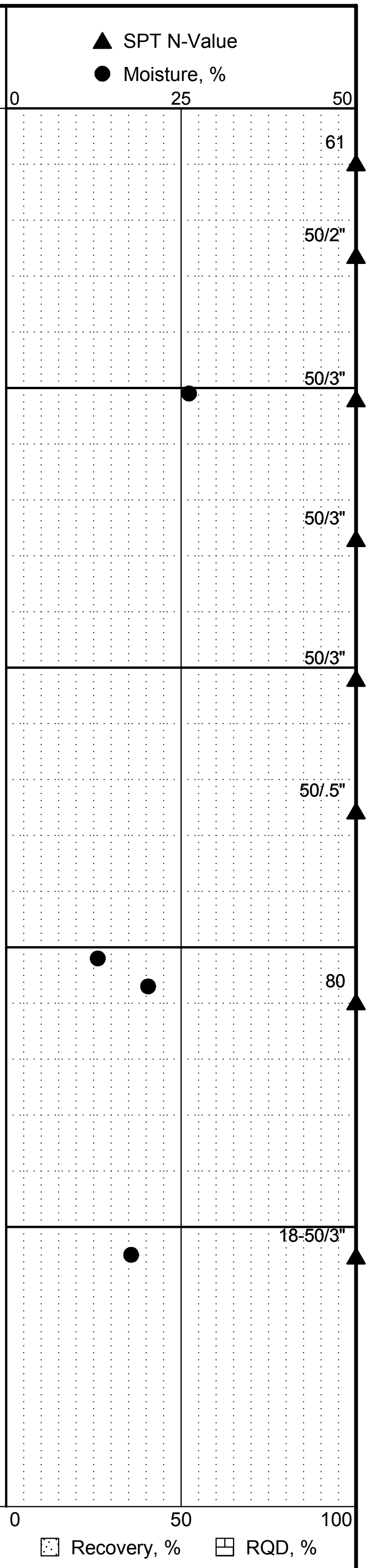
Portland, Oregon

FIG.

WLG ANKPS.GPJ 12/2/16 Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Depth In Feet	Samples
	Wet density = 165.4 pcf	-21.0 51.0	**Clayey Gravelly SAND**, very dense, yellow-brown, moist, low plasticity, sand is fine-grained, fine to coarse rounded gravel, weathered, over-consolidated, poorly graded, gradational transition to underlying material. (Troutdale Formation)		S-20* S-21*
				55	S-22
					S-23*
	Wet density = 170.0 pcf			60	S-24*
					S-25*
	Wet density = 161.4 pcf				
		-35.0 65.0	**Gravelly SAND**, very dense, light brown, wet, non-plastic, sand is fine-grained, fine to coarse rounded gravel, over-consolidated, poorly graded. (Troutdale Formation)	65	S-26
				70	S-27
		-40.8 70.8	Bottom of Boring, Completed 8/29/00		



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

Ankeny Pump Station
Portland, Oregon

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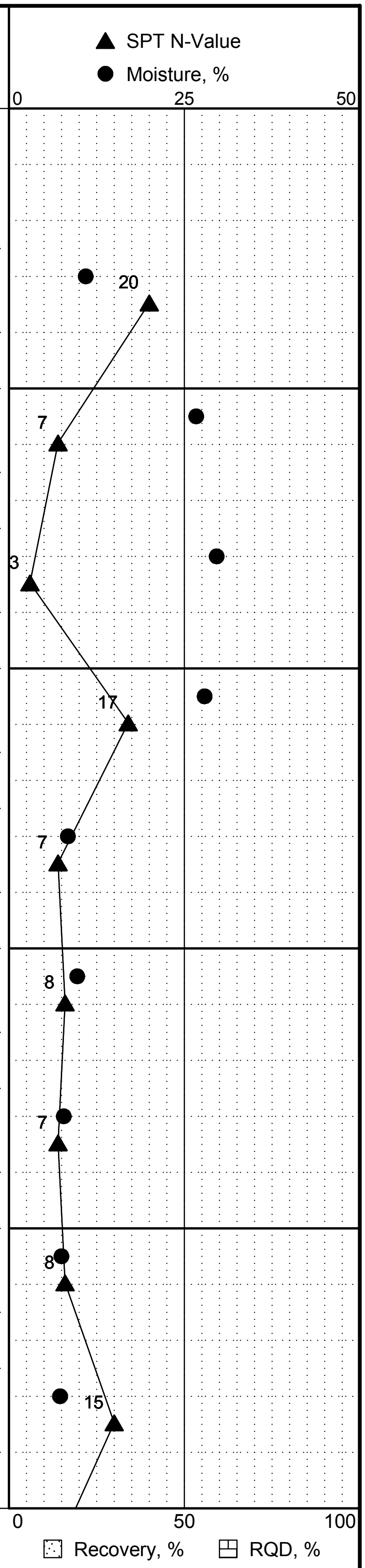
FIG.

WLG ANKPS.GPJ 12/2/16

Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet 32.0	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
			SAND FILL, medium dense, brown, dry, fine-grained, scattered gravel, stratified silt. (Fill)	▩	0	
				▩		S-1
	Samples S-2, S-5, S-8 and S-10 taken with 3" SS and 140# hammer, n-values normalized	27.0 5.0	**Clayey SILT FILL**, stiff to soft, gray, moist, low plasticity, micaceous, scattered organics, trace metal scraps. (Fill)	▩	5	S-2
				▩		S-3
			Very stiff, slightly sandy, scattered crushed rock.	▩	10	S-4
	-200 = 3%	19.5 12.5	**SAND FILL**, loose to medium dense, brown, moist, micaceous, fine-grained, scattered fine rounded gravel, trace clayey silt lenses. (Dredged sand fill)	▩		S-5
	-200 = 7%			▩	15	S-6
			2-inch Clayey SILT lens.	▩		
	-200 = 4%			▩		S-7
	-200 = 4%			▩	20	S-8
			Gray, wood fragments.	▩		
	-200 = 2%			▩		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

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

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

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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING D-1

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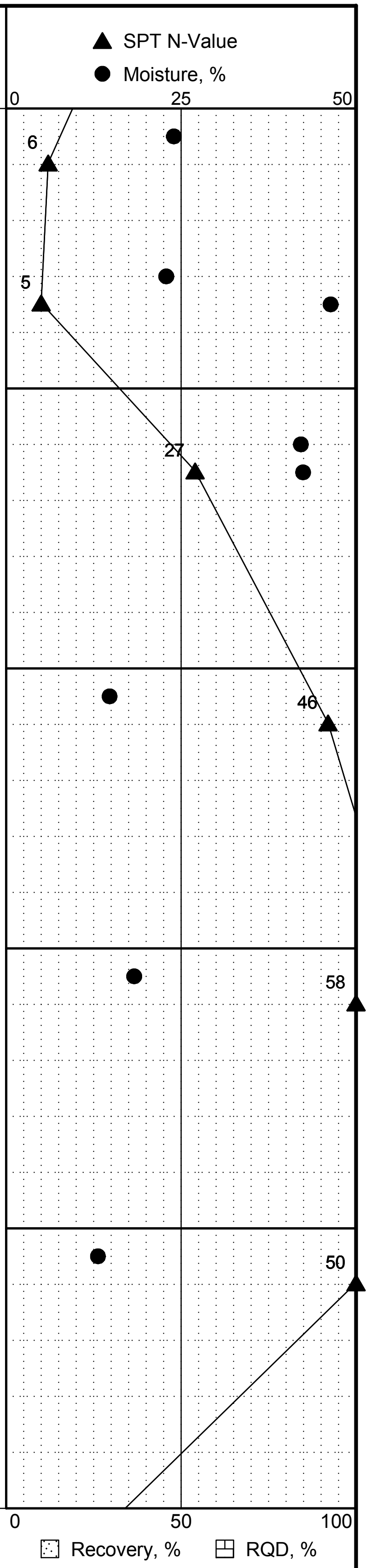
FIG.

WLG ANKPS.GPJ 12/2/16 *Log:* *Typ:* *Rev:*

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	-200 = 3%		Wet, some medium grained sand.			
▼	9/21/2000					S-10
			Grading slightly silty with interstratified Clayey SILT.			
	-200 = 13%					S-11
					30	
	-200 = 29%		3-inch layer of Clayey SILT at base of fill. (Recent Alluvium)			S-12
		0.2 31.8	**Cobbly Sandy GRAVEL**, dense to very dense, gray and brown, wet, fine to coarse subrounded gravel, sand is fine- to medium-grained. (Pleistocene Flood Deposits)			
					35	
	-200 = 9%					S-13
					40	
	-200 = 67%		**Cobbly Sandy GRAVEL** continued.			S-14
					45	
	-200 = 5%					S-15
		-17.5 49.5	**SILT**, slightly clayey, stiff, light brown, moist, low			



LEGEND

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

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Portland, Oregon

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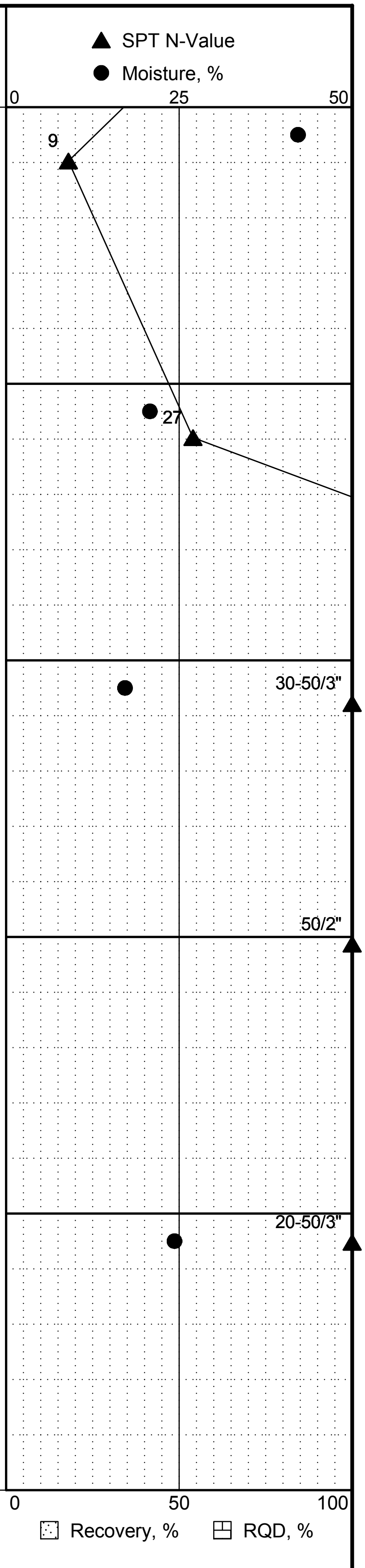
FIG.

WLG ANKPS.GPJ 12/2/16 Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	-200 = 99%		plasticity, micaceous. (Pleistocene Flood Deposits, silt interlayer)			S-16
	-200 = 9%	-22.0 54.0	**Gravelly SAND**, medium dense, gray, wet, sand is medium-grained, fine rounded gravel, gradational transition to underlying material. (Pleistocene Flood Deposits)		55	S-17
		-27.0 59.0	**Cobbly Sandy GRAVEL**, dense, gray and brown, wet, fine to coarse subrounded gravel, sand is fine- to medium-grained, poorly graded. (Pleistocene Flood Deposits)		60	S-18
		-33.0 65.0	**SAND**, slightly clayey, very dense, gray and brown, moist, fine-grained, micaceous, over-consolidated, weathered, gradational transition to underlying material. (Troutdale Formation)		65 70	S-19* S-20



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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING D-1

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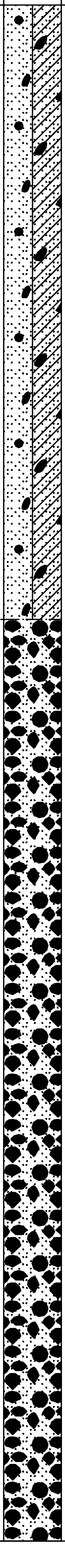
Portland, Oregon

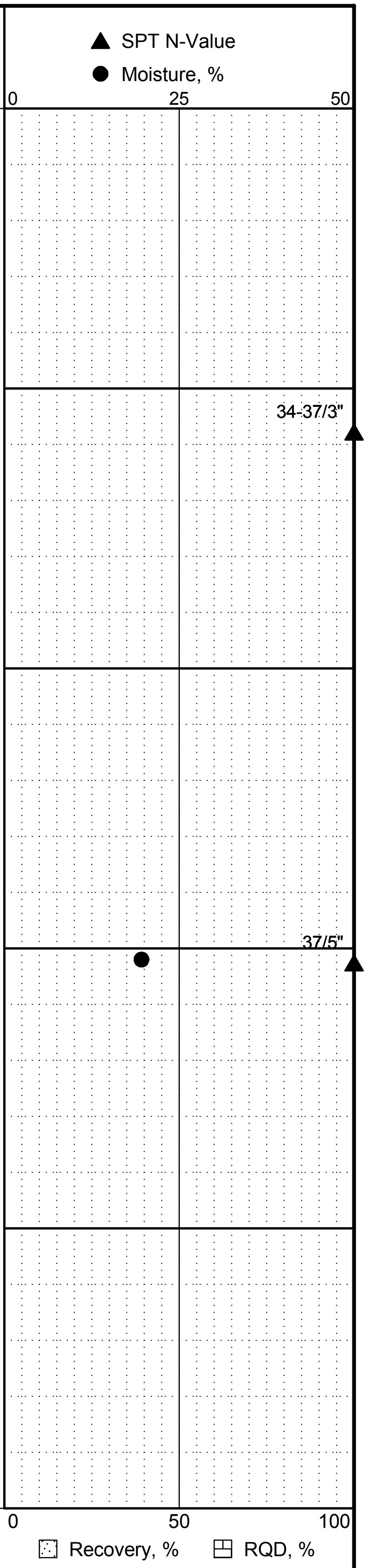
FIG.

WLG ANKPS.GPJ 12/2/16

Log: Typ: Rev:

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples
	Wet density = 171.6 pcf		**SAND**, slightly clayey continued.		80	S-21
		-53.0 85.0	**Sandy GRAVEL**, slightly clayey, very dense, brown, wet, low plasticity matrix, fine to coarse rounded gravel with scattered cobbles, sand is fine- to medium-grained, over-consolidated, poorly graded. (Troutdale Formation)		85	
					90	S-22
					95	
	Hole collapsed while attempting to sample at	-68.0				





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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING D-1

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FIG.

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	▲ SPT N-Value ● Moisture, % 0 25 50
	100 feet.	100.0	Bottom of Boring, Completed 9/21/00				

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

Ankeny Pump Station
Portland, Oregon

LOG OF BORING D-1
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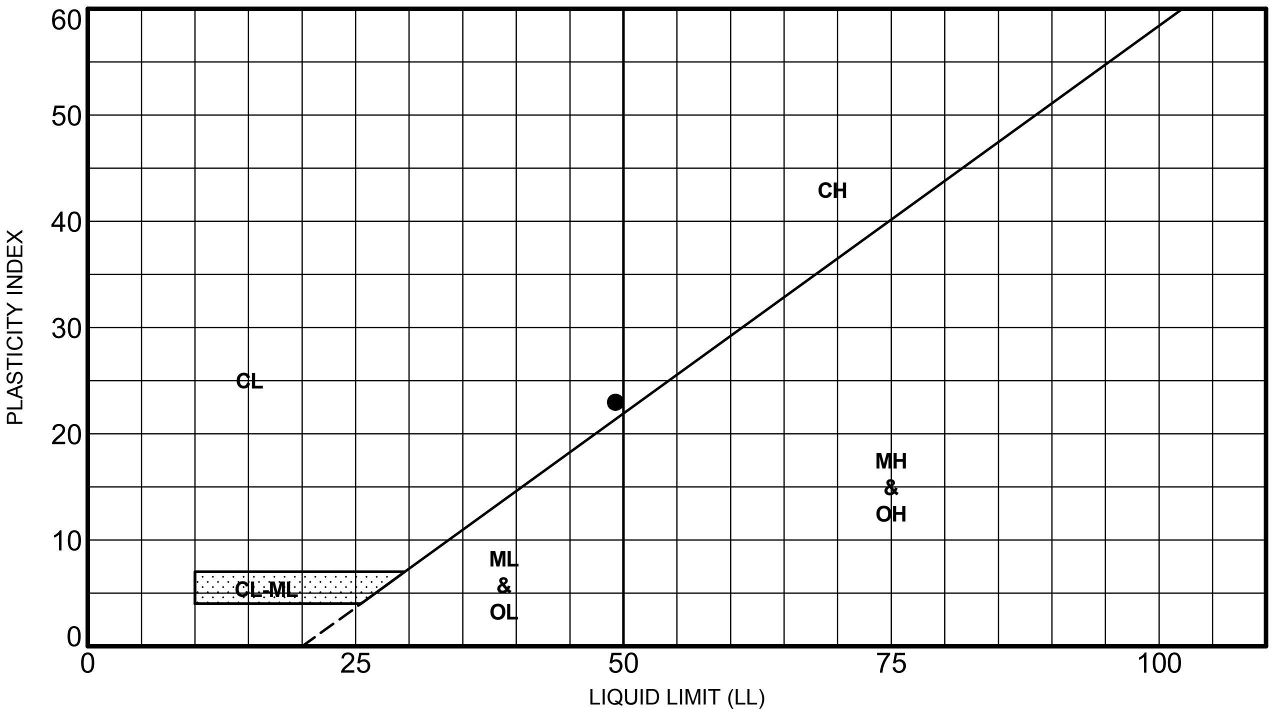
FIG.

WLG ANKPS.GPJ 12/2/16 *Log:* *Typ:* *Rev:*



	Hole	Depth	LL	PL	PI	wc %	Classification
●	A-1a	5.0	49	26	23		Clayey SILT, slightly sandy, medium plasticity. (S-1)

Ankeny Pump Station
Portland, Oregon

PLASTICITY CHART

December 2000

F-3112.01

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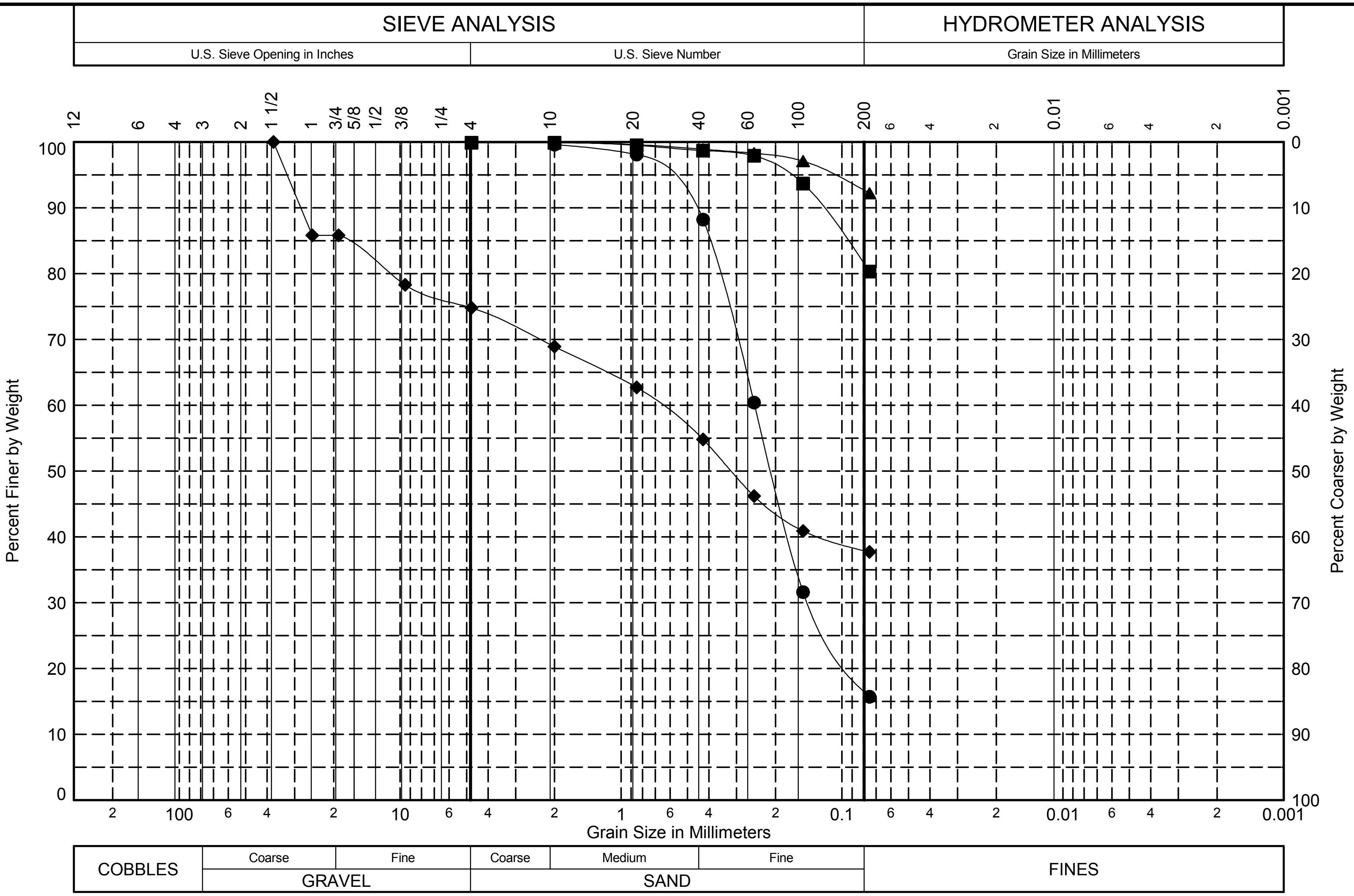
FIG. 1

ATTCHART ANKPS.GPJ 12/1/16



	Hole	Depth	USC	Classification	wc%	LL	PL	PI
●	A-1	70.0		Silty SAND, brown, trace clay. (S-19)				
▲	A-1a	5.0		Clayey SILT, slightly sandy, medium plasticity. (S-1)		49	26	23
■	A-1a	10.0		Sandy Clayey SILT, brown. (S-2)				
◆	B-1	7.5		Gravelly Silty SAND, brown. (Combined S-1 - S-5)				

Ankeny Pump Station
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GRAIN SIZE DISTRIBUTION

December 2000
F-3112.01

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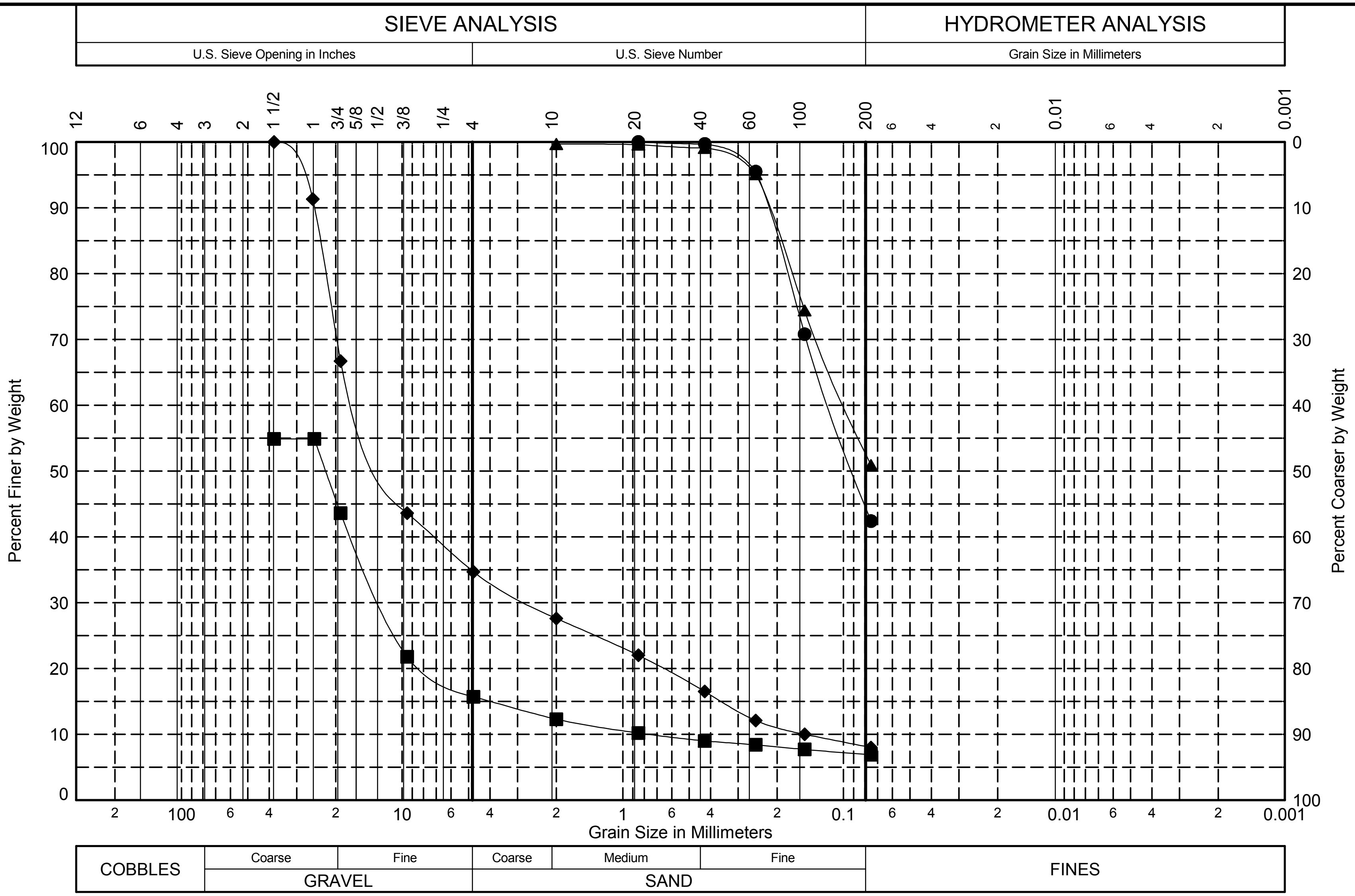
FIG. 1



Hole	Depth	USC	Classification	wc%	LL	PL	PI
● B-1	25.0		Silty SAND, brown. (S-10a)				
▲ B-1	25.5		Sandy SILT, brown. (S-10b)				
■ B-1	35.5		GRAVEL, slightly sandy, slightly silty, brown, scattered cobbles. (S-13)				
◆ B-1	38.0		Sandy GRAVEL, slightly silty, brown. (S-14)				

Ankeny Pump Station
Portland, Oregon

GRAIN SIZE DISTRIBUTION

December 2000
F-3112.01

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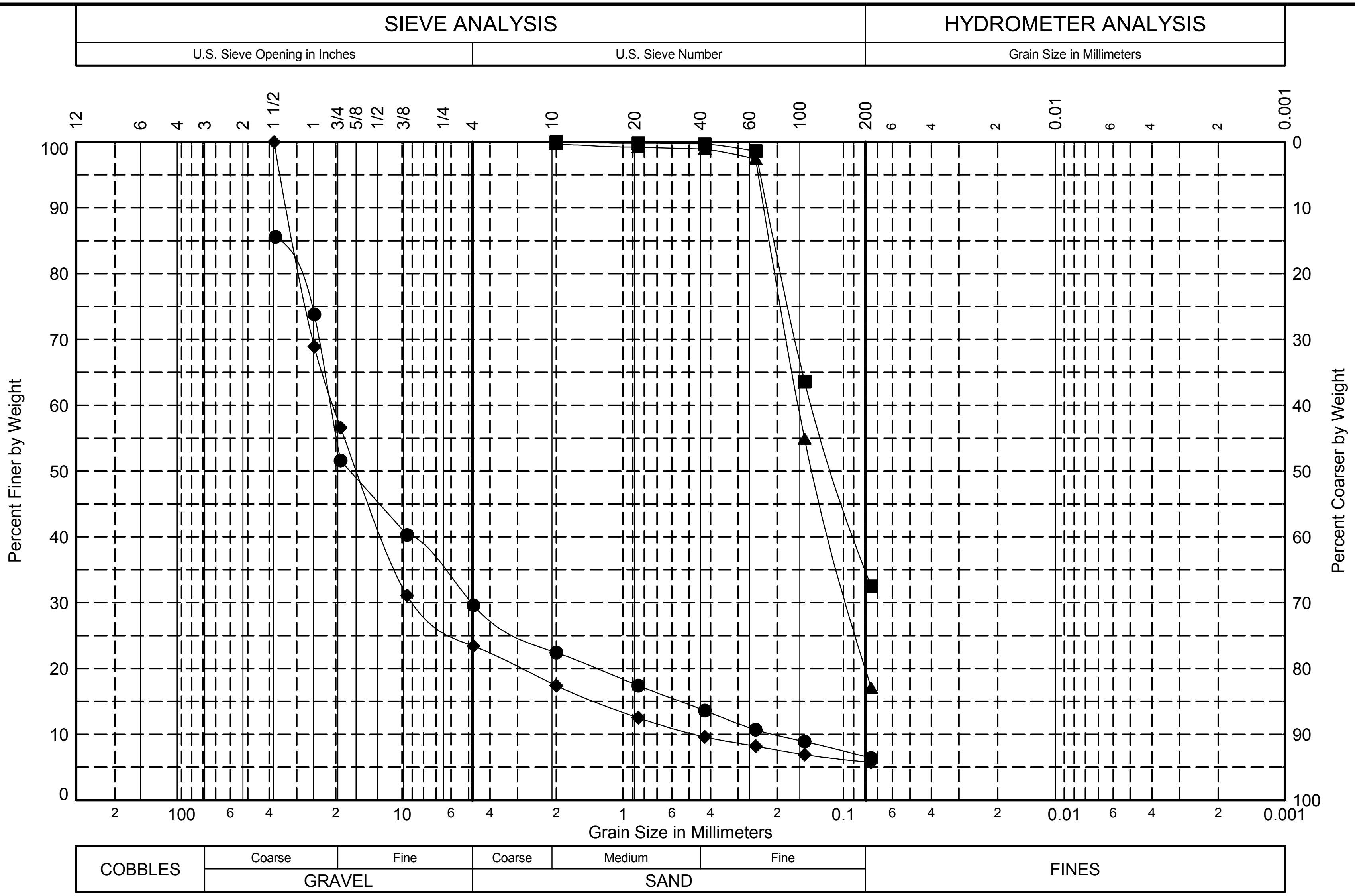
FIG. 2

GRNSIZE ANKPS.GPJ 12/1/16



Hole		Depth	USC	Classification	wc%	LL	PL	PI
●	B-1	43.0		Sandy GRAVEL, slightly silty, brown, scattered cobbles. (S-16)				
▲	B-1	47.5		Silty SAND, brown. (S-18)				
■	B-1	50.5		Silty SAND, brown, wet. (S-19)				
◆	B-1	53.0		Sandy GRAVEL, slightly silty, brown, trace clay. (S-20)				

Ankeny Pump Station
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GRAIN SIZE DISTRIBUTION

December 2000 *F-3112.01*

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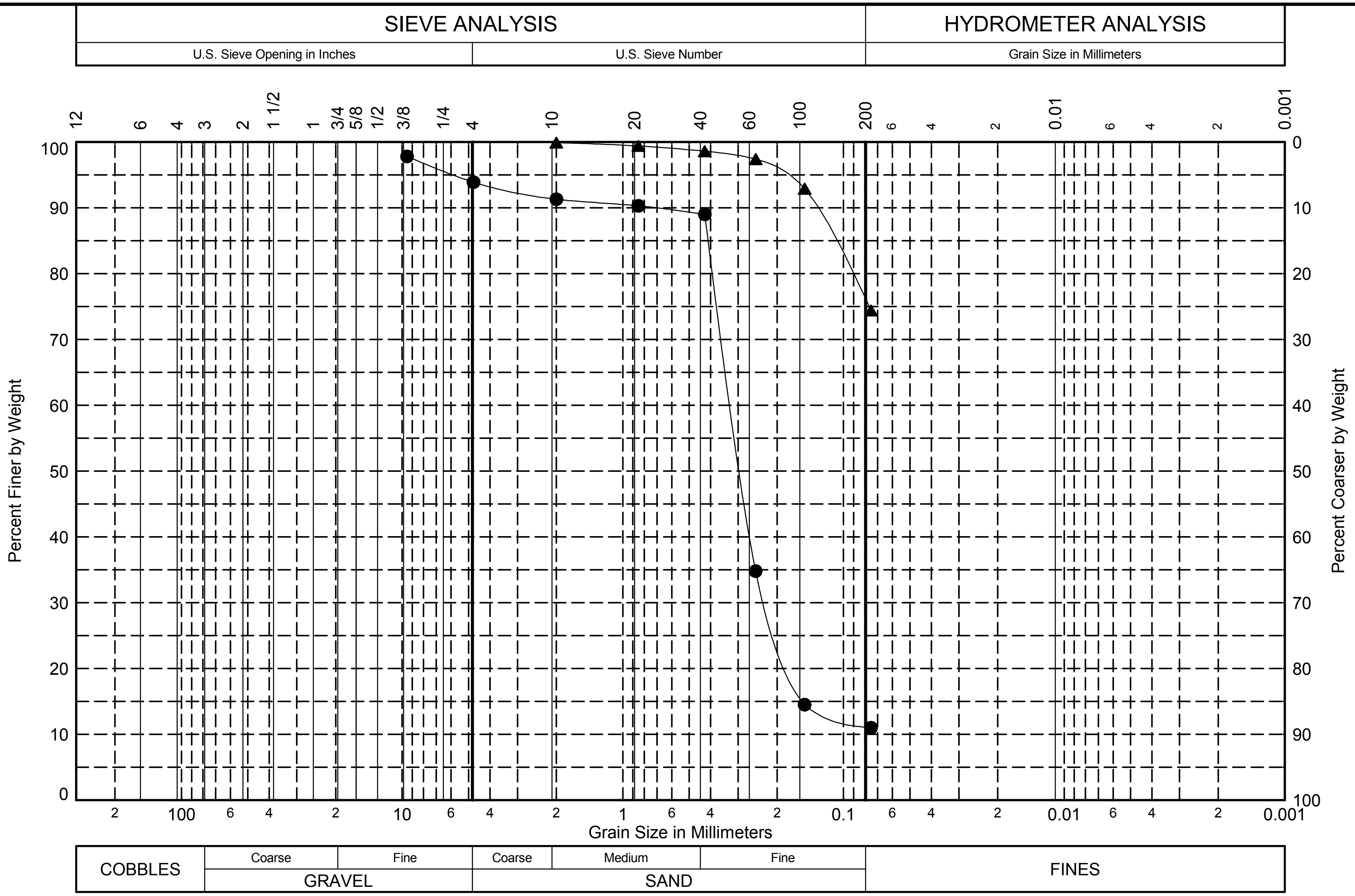
FIG. 3



Hole	Depth	USC	Classification	wc%	LL	PL	PI
● B-1	59.5		SAND, slightly silty, yellow-brown, trace clay, trace gravel. (S-23)				
▲ B-1	63.0		Sandy Clayey SILT, yellow brown. (S-24)				

Ankeny Pump Station
Portland, Oregon

GRAIN SIZE DISTRIBUTION

December 2000

F-3112.01

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

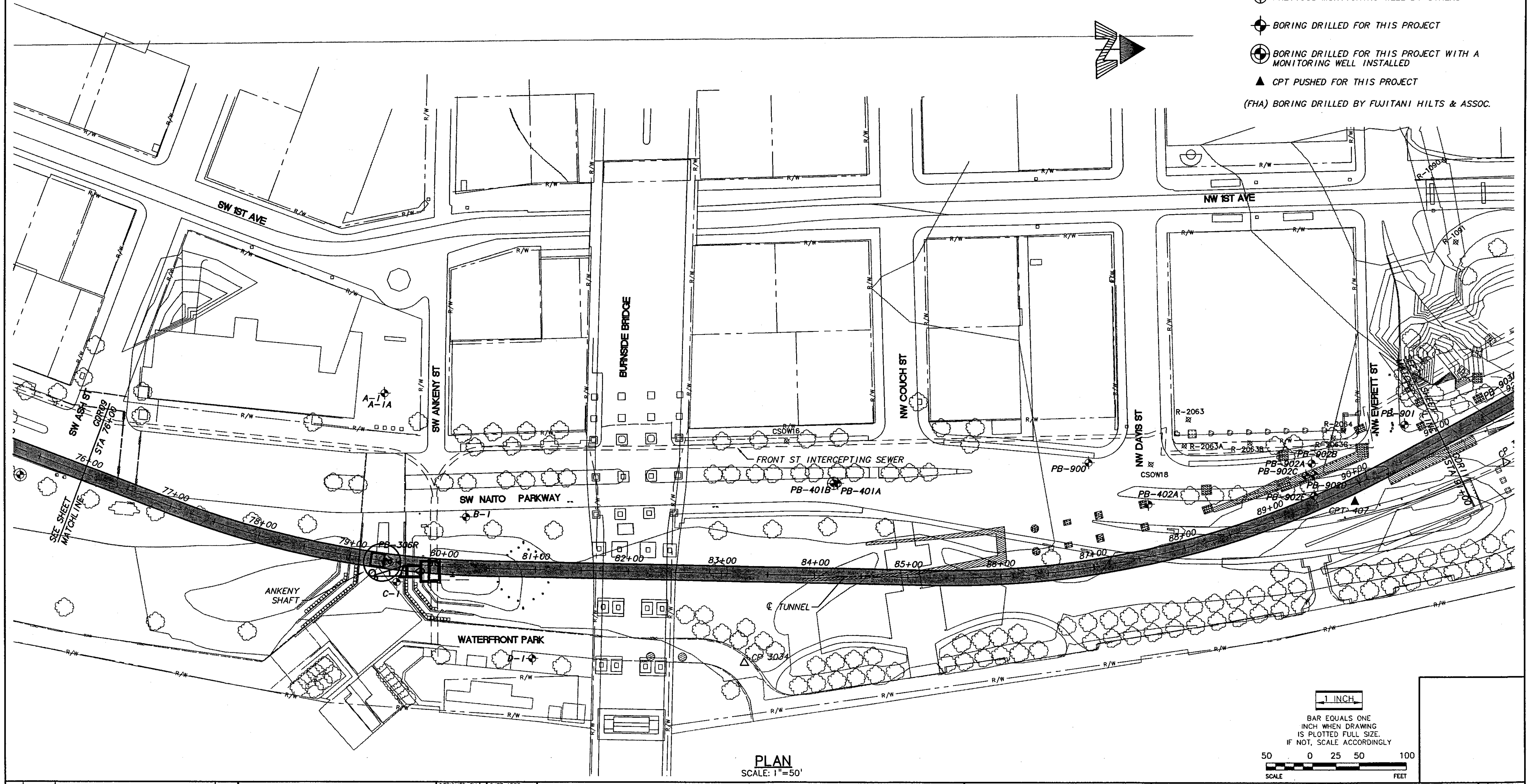
FIG. 4

NOTES:

1. ABANDONED FOOTINGS ARE SHOWN SHADED

LEGEND

- ⊕ PREVIOUS BORING BY OTHERS
- ● PREVIOUS MONITORING WELL BY OTHERS
- ⊕ BORING DRILLED FOR THIS PROJECT
- ⊕ BORING DRILLED FOR THIS PROJECT WITH A MONITORING WELL INSTALLED
- ▲ CPT PUSHED FOR THIS PROJECT
- (FHA) BORING DRILLED BY FUJITANI HILTS & ASSOC.



PLAN
SCALE: 1"=50'

NO.	DATE	DESCRIPTION	APPD.
REVISION			

XREF(S) USED: 6680X01,..X02,..X10,..X11,..X12,..X13
ROTATION ANGLE: 271.68°
CONSTRUCTED BY
PROJECT COMPLETED
MAP CORRECTED BY ______ CHECKED BY ______
FINAL MAP DATA
GDR10.dwg, 05/29/02 at 09:22

DESIGNED BY	SLB	DATE APPD.	
DRAWN BY	ARO	PROGRAM MGR	PTG
CHECKED BY	MLF	CONST. MGR.	PTG
DESIGN MGR.	GMB	CADD MGR.	

CITY OF PORTLAND
ENVIRONMENTAL SERVICES
WILLAMETTE RIVER CSO CONTROL PROJECT

PB PARSONS BRINCKERHOFF
400 S.W. Sixth Ave. Portland, OR 97204

WEST SIDE CSO PROJECT
BOREHOLE LOCATION PLAN
STA 76+00 TO STA 91+00
FIGURE 2.1

1/4 SECTION
VARIES
JOB NO.
6680
SHEET NO.
GDR10
9 OF 23

PARSONS BRINCKERHOFF

BORING LOG
PB-305A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 11/1/00 LOGGED BY AR
SHEET 1 OF 2
STATION NO. 75+37 (26 R)
SURFACE ELEV. 29.95 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 and 300 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"
25	5	◧	1	4 5 12
20	10	◧	2	23 31 19
15	15	◧	3	15 23 20
10	20	◧	4	5 3 1
5	25	◧	5	10 27 23
		☒	6	2 21 50
0	30	☒	7	34 35 31
		☒	8	18 29 30
-5	35	☒	9	22 28 25

△ PERCENT FINES △ 20 40 60 80
PLASTIC M.C. LIQUID 20 40 60 80

SOIL DESCRIPTION

SILT (ML); low plasticity, brown and grey, moist to wet, (Topsoil).

FILL consisting of layered bricks, gravel, and wood fragments in a silt matrix; medium dense to very dense.

Drilling mud loss (200 gallons) at 17 feet.

Fill becomes very loose at 20 feet.

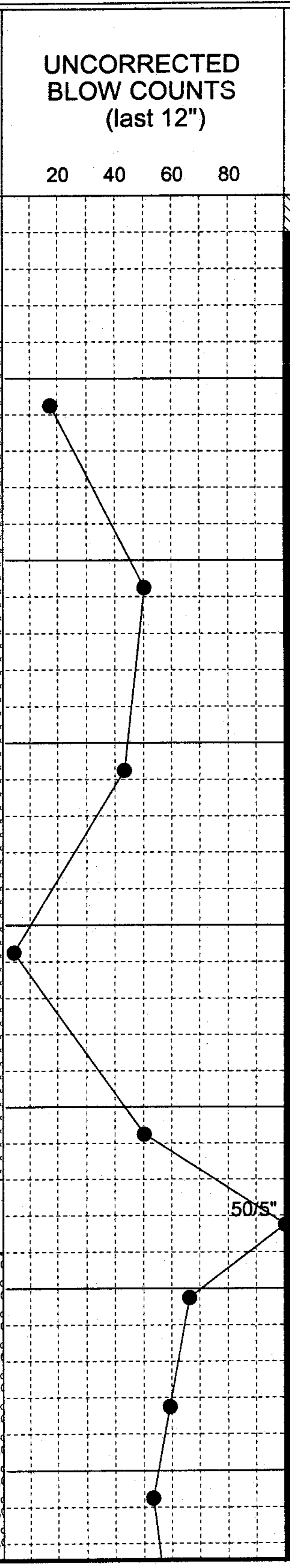
Fill becomes very dense at 25 feet.

SILTY SAND (SM); few gravel, trace wood fragments, non-plastic silt, grey-brown, wet, very dense, (Sand/silt alluvium).

POORLY GRADED GRAVEL WITH SAND (GP); fine to coarse grained gravel, subangular to rounded gravel, grey, wet, very dense, (Gravel alluvium), clasts consist of basalt and minor quartzite.

Drilling mud loss encountered at 34.5 feet.

LITHOLOGY



WELL

PARSONS BRINCKERHOFF

BORING LOG
PB-305A

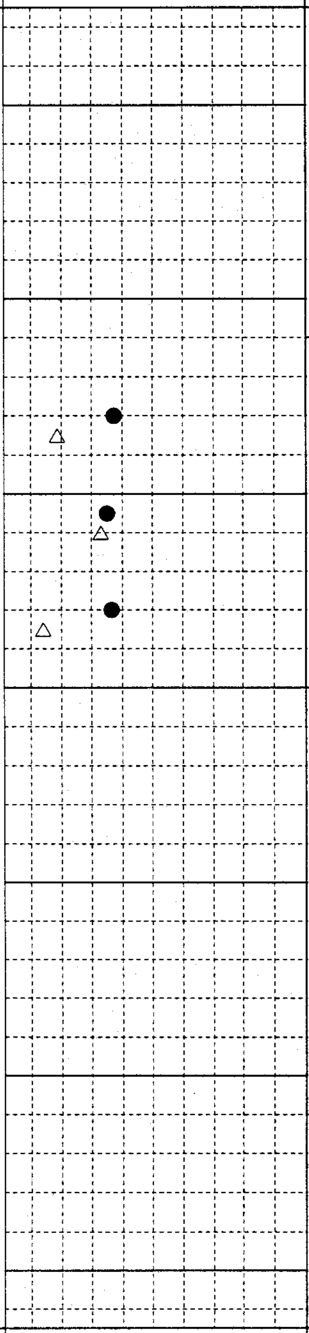
Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 11/1/00 LOGGED BY AR
SHEET 2 OF 2
STATION NO. 75+37 (26 R)
SURFACE ELEV. 29.95 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 and 300 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▮ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"
		☒	10	40 33 24
-10	40	☒	11	20 20 17
		☒	12	50
-15	45	☒	13	50
		☒	14	12 17 18
-20	50	☒	15	12 11 12
		☒	16	13 17 23
-25	55	☒	17	50
		☒	18	21 29 50
-30	60	☒	19	25 37 50
		☒	20	50
-35	65	☒	21	18 18 19
		☒	22	11 13 16
-40	70	☒	23	17 16 23

△ PERCENT FINES △ 20 40 60 80
PLASTIC M.C. LIQUID 20 40 60 80



SOIL DESCRIPTION

POORLY GRADED GRAVEL WITH SAND (GP); fine to coarse grained gravel, subangular to rounded gravel, grey, wet, very dense, (Gravel alluvium), clasts consist of basalt and minor quartzite.

SILTY SAND (SM); fine to medium grained, low plasticity silt, grey-brown, wet, medium dense to dense, (Sand/silt alluvium).

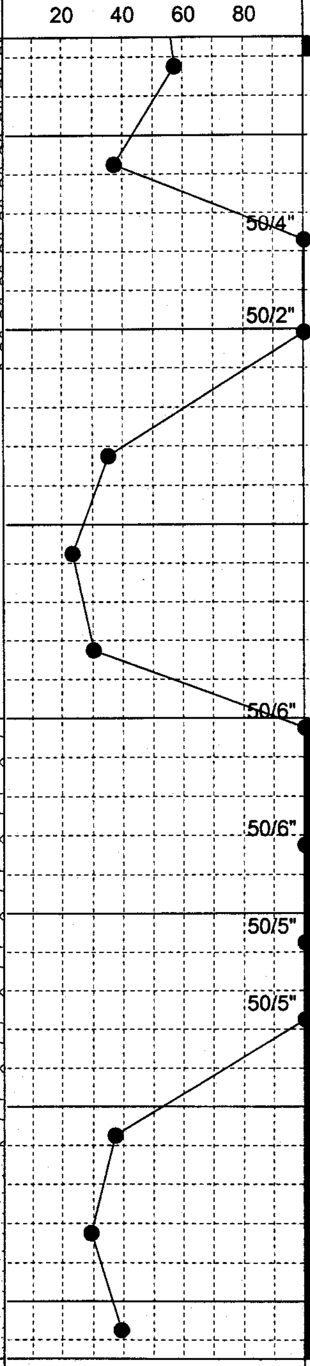
POORLY GRADED GRAVEL WITH COBBLES, SAND, AND SILT (GP-GM); fine to coarse grained gravel, subrounded, low plasticity silt, grey and brown, very dense, uncemented to moderately cemented, (Troutdale Formation).

POORLY GRADED SAND WITH SILT (SP-SM); fine to medium grained, low plasticity silt, grey-brown, wet, dense, (Troutdale Formation).

Bottom of Boring at 71.5 feet.

LITHOLOGY

UNCORRECTED BLOW COUNTS (last 12")



WELL

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

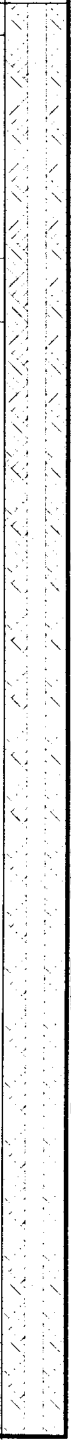
PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01
SHEET 1 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft
INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE G Grab Sample ○ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	SOIL DESCRIPTION
										Grass
				190	0					**Artificial Fill (Qaf)** POORLY GRADED SAND WITH GRAVEL (SP); 70-75% sand, fine to medium, subangular to rounded, hard, quartz and basalt, moist; 25-30% gravel, fine to coarse, subrounded to rounded.
									1	SILTY CLAY TO CLAYEY SAND WITH GRAVEL (CL-SC) and brick fragments; 45-50% silty clay, low to medium plasticity, no dilatancy; 35-40% sand, fine to medium; 10-15% gravel, brick fragments and glass shards; mottled gray and brown, moist.
25		G	1	200	300					POORLY GRADED SAND WITH GRAVEL, COBBLES AND BOULDERS (SP); 70-75% sand, fine to medium, hard; 25-30% gravel, fine to coarse, cobbles to boulders, angular, moist.
	5									BOULDERS. POORLY GRADED GRAVEL (GP); 90% cobbles and boulders. Rip Rap Fill -- cobbles and boulders.
					300					
									2	No recovery at 8 to 11 ft.
20										
	10									
									3	
15										
	15									
									4	
10										
	20									
										TIMBER PILE.

LITHOLOGY

WELL



PARSONS BRINCKERHOFF

BORING LOG
PB-306R

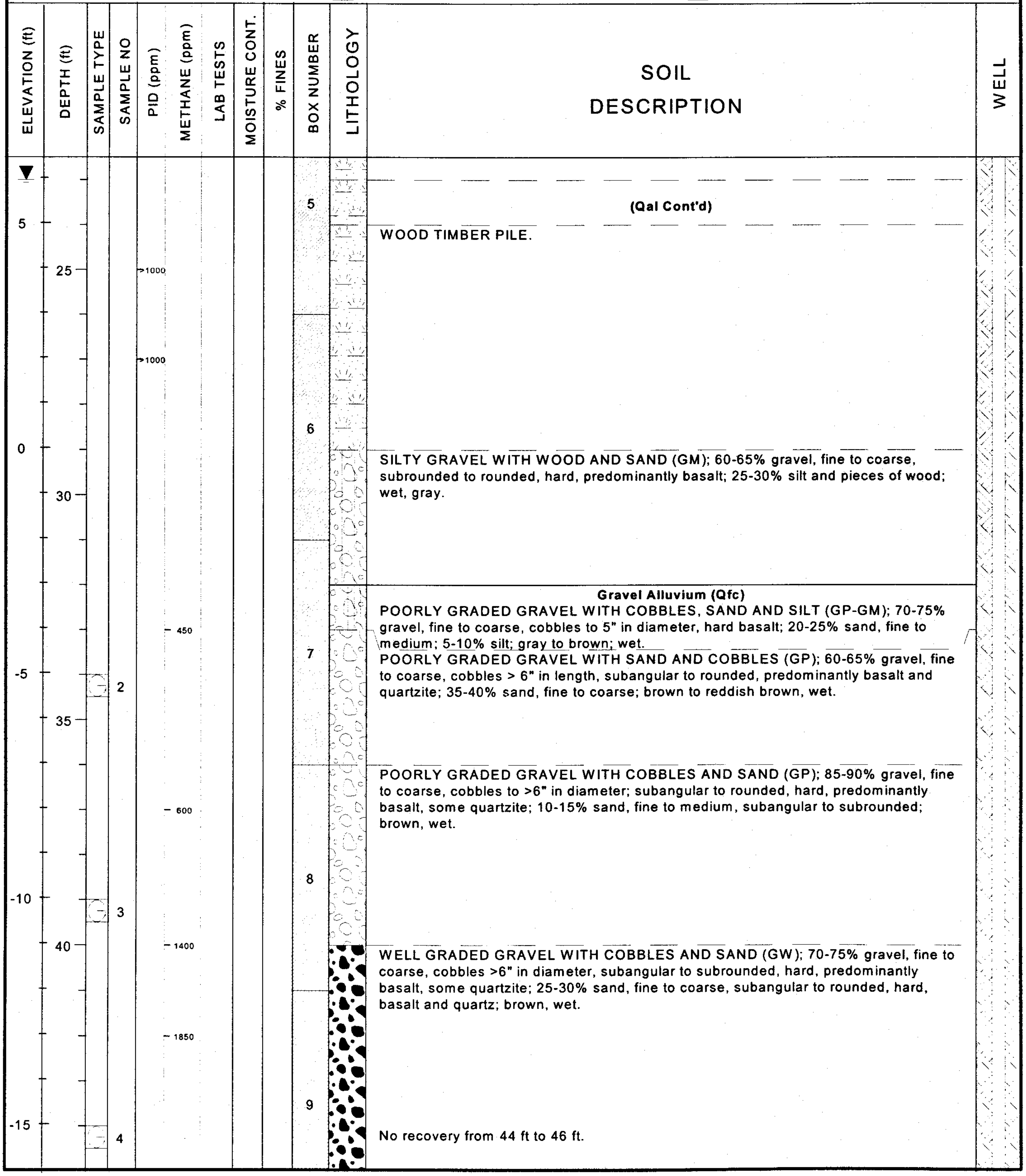
PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01

SHEET 2 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft

INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE Ⓖ Grab Sample ○ No Recovery



(Qal Cont'd)

WOOD TIMBER PILE.

SILTY GRAVEL WITH WOOD AND SAND (GM); 60-65% gravel, fine to coarse, subrounded to rounded, hard, predominantly basalt; 25-30% silt and pieces of wood; wet, gray.

Gravel Alluvium (Qfc)

POORLY GRADED GRAVEL WITH COBBLES, SAND AND SILT (GP-GM); 70-75% gravel, fine to coarse, cobbles to 5" in diameter, hard basalt; 20-25% sand, fine to medium; 5-10% silt; gray to brown; wet.

POORLY GRADED GRAVEL WITH SAND AND COBBLES (GP); 60-65% gravel, fine to coarse, cobbles > 6" in length, subangular to rounded, predominantly basalt and quartzite; 35-40% sand, fine to coarse; brown to reddish brown, wet.

POORLY GRADED GRAVEL WITH COBBLES AND SAND (GP); 85-90% gravel, fine to coarse, cobbles to >6" in diameter; subangular to rounded, hard, predominantly basalt, some quartzite; 10-15% sand, fine to medium, subangular to subrounded; brown, wet.

WELL GRADED GRAVEL WITH COBBLES AND SAND (GW); 70-75% gravel, fine to coarse, cobbles >6" in diameter, subangular to subrounded, hard, predominantly basalt, some quartzite; 25-30% sand, fine to coarse, subangular to rounded, hard, basalt and quartz; brown, wet.

No recovery from 44 ft to 46 ft.

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BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01

SHEET 3 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft

INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE G Grab Sample ○ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	LITHOLOGY	SOIL DESCRIPTION	WELL
											Troutdale (Tt) POORLY GRADED GRAVEL WITH COBBLES AND SAND (GP); 70-75% gravel, fine to coarse, cobbles to 5" in length, subangular to rounded, hard, predominantly basalt; 25-30% sand, fine to medium, subangular to subrounded, hard, predominantly quartz and some basalt; brown, wet, some iron staining and occasional spotty coating of fine to medium sand cemented to surface of gravel.	
					1400							
									10		POORLY GRADED SAND (SP); 100% sand, fine to medium, subangular to rounded, hard, predominantly quartz, some basalt and mica; yellowish brown, wet.	
-20		G	5									
	50				0							
					3500							
									11			
-25		G	6									
	55				>5000							
											POORLY GRADED GRAVEL WITH SILTY CLAY, SAND AND COBBLES (GP-GC); 65-70% gravel, fine to coarse, cobbles to 4" in diameter, subangular to rounded, hard, quartzite and basalt; 25-30% sand, fine to medium, subangular to subrounded, hard, predominantly quartz, mica and basalt; 5-10% silty clay and silt, low to medium plasticity, no dilatancy, mottled dark gray brown, rusty brown, wet; some iron staining on gravel and some have thin spotty coating of fine to medium sand cemented to surface.	
		G	7A			GSD	10	11				
									12			
-30		G	7									
	60											
									13			
-35		G	8								POORLY GRADED GRAVEL WITH SAND (GP); 75-80% gravel, fine to coarse, subrounded to rounded, hard, quartzite and basalt; 20-25% sand, subangular to subrounded; <5% silty clay.	
	65											
											Note: From 66' to 76', core fell out while being retrieved and was resampled out of 8" casing. It has been washed and is highly disturbed.	

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BORING LOG
PB-306R

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GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01
SHEET 4 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft
INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE G Grab Sample ○ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	SOIL DESCRIPTION
									14	**(Tt Cont'd)**
-40										
	70									POORLY GRADED GRAVEL WITH SILTY CLAY AND SAND (GP-GM); 60-65% gravel, fine to coarse, subangular to rounded, hard, quartzite and basalt; 25-30% sand, fine to medium; 5-15% silty clay, occurs as lenses, dark gray.
										WELL GRADED GRAVEL WITH SAND (GW); 70-75% gravel, fine to coarse, subangular to rounded, hard, quartzite and basalt; 20-25% sand, fine to coarse, subangular to rounded, hard, quartz and basalt; <5% silt, some gravel has thin spotty coating of fine to medium sand cemented to the surface.
									15	
-45		G	9							
	75									
					->5000					
										SILTY CLAYEY GRAVEL WITH SAND (GC-GM); 60-65% gravel, fine to coarse, subrounded to rounded, hard, quartzite and basalt; 20-35% sand, fine to coarse, hard, quartz, basalt and others; angular to subrounded; 10-20% silty clay, light gray to brown, wet; matrix composed of silty clayey sand with pockets/lenses of fine to coarse sand.
									16	
-50		G	10			GSD	10	11		
	80				->5000					
					->5000					
			TUNNEL CROWN							POORLY GRADED SAND WITH SCATTERED GRAVEL (SP); 90-95% sand, fine to medium, subangular to rounded, predominantly basalt; 5-10% gravel, fine to medium, subrounded, hard, basalt and quartzite; reddish black, wet.
									17	SILTY CLAYEY GRAVEL WITH SAND (GC-GM); 70-75% gravel, fine to coarse, subrounded to rounded, hard, basalt and quartzite; 15-20% sand, fine to medium, subangular to rounded, hard, quartz and basalt; 10-15% silty clay; matrix contains pockets of clean fine to medium sand.
-55		G	11							
	85				->5000					
										POORLY GRADED GRAVEL WITH SAND, SILTY CLAY AND COBBLES (GP-GM); 70-75% gravel, fine to coarse, cobbles to 4" in diameter, subangular to rounded, hard, quartzite and basalt; 15-20% sand, fine to coarse, subangular to rounded, basalt and quartz; 5-10% silty clay; matrix has pockets of fine to coarse sand, brown, wet.
					->5000					
									18	
-60		G	12							

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01
SHEET 5 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft
INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE G Grab Sample ○ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	LITHOLOGY	SOIL DESCRIPTION	WELL
					>5000						**(Tt Cont'd)**	
											SILTY CLAY (CL); 100% silty clay, medium plasticity, no dilatancy, medium dry strength, medium toughness, dark gray, moist.	
					400							
									19			
-65		G	13			AL	43					
	95				300						POORLY GRADED GRAVEL WITH SAND (GP), trace silty clay; 75-80% gravel, fine to coarse, cobbles <4" in diameter, subangular to rounded; 15-20% sand, fine to coarse, subangular to rounded, predominantly basalt; <5% silty clay, dark gray, wet.	
					2100						POORLY GRADED GRAVEL WITH COBBLES, SAND AND SILTY CLAY (GP-GC); 70-75% gravel, fine to coarse, cobbles to 4" in diameter, subangular to rounded, hard, with basalt and quartzite, some with spotty coating of fine to medium sand cemented to surface; 10-15% sand, fine to medium, subangular to rounded; 5-10% silty clay; dark gray, wet.	
			TUNNEL INVERT									
									20			
-70		G	14			FC	9	9				
	100				4600							
					>5000							
											CLAYEY GRAVEL WITH SAND (GC); 65-70% gravel, fine to coarse; 10-20% sand, fine to medium, subangular to rounded, predominantly basalt; 10-15% silty clay; dark gray.	
									21			
-75		G	15		>5000	GSD	11	15				
											POORLY GRADED GRAVEL WITH COBBLES (GP); 95-100% gravel, fine to coarse, with cobbles > 5" in diameter, subangular to rounded, predominantly basalt with some quartzite; dark gray.	
	105											
											POORLY GRADED SAND (SP); 100% sand, fine to medium, subangular, predominantly quartz, basalt and mica, grayish green/greenish gray, wet.	
					3700							
											POORLY GRADED GRAVEL WITH COBBLES, SAND AND SILTY CLAY (GP-GC); 70-75% gravel, fine to coarse with cobbles to 5" in length, subrounded to rounded, hard, basalt and quartzite; 15-20% sand, fine to medium, hard, quartz, basalt and mica; 5-10% silty clay; some gravel has spotty coating of fine to medium sand cemented to surface.	
									22			
-80		G	16									
	110				2300							
					>5000							

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO 027-003
DATE DRILLED 4/9/01 to 4/11/01
SHEET 6 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft
INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE G Grab Sample ○ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	LITHOLOGY	SOIL DESCRIPTION	WELL
											(Tt Cont'd)	
									23			
-85		G	17			GSD	7	8				
	115				>5000							
					>5000							
									24		POORLY GRADED GRAVEL WITH COBBLES, SAND AND SILTY CLAY (GP-GC); 70-75% gravel, fine to coarse with cobbles to 5" in length, subrounded to rounded, hard, basalt and quartzite; 15-20% sand, fine to medium, hard, quartz, basalt and mica; 5-10% silty clay; some gravel has spotty coating of fine to medium sand cemented to surface.	
-90		G	18									
	120				>5000							
					>5000							
									25			
-95		G	19									
	125				4000							
											Slough from 126 ft to 127.5 ft. POORLY GRADED SAND (SP); 100% sand, fine to medium, subangular to rounded, hard, quartz, basalt and others.	
					3000							
									26		WELL GRADED GRAVEL WITH SAND (GW), trace silt and cobbles; 75-80% fine to coarse gravel, with cobbles to 5" in length, subrounded to rounded; 15-20% sand, fine to coarse, subangular to rounded, hard, basalt, quartz and others; <5% silt; dark gray, wet.	
-100		G	20									
	130				>5000							
					>5000						POORLY GRADED GRAVEL WITH SAND, SILT AND COBBLES (GP-GM); 65-70% gravel, fine to coarse, with occasional cobbles to 4", subrounded to rounded, hard, predominantly basalt; 20-25% sand, fine to coarse, subangular to rounded, basalt and quartz; 5-10% silt to silty clay; dark gray, wet; some gravel has spotty coating of fine to medium sand cemented to surface.	
									27			
-105		G	21			GSD	7	7				

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

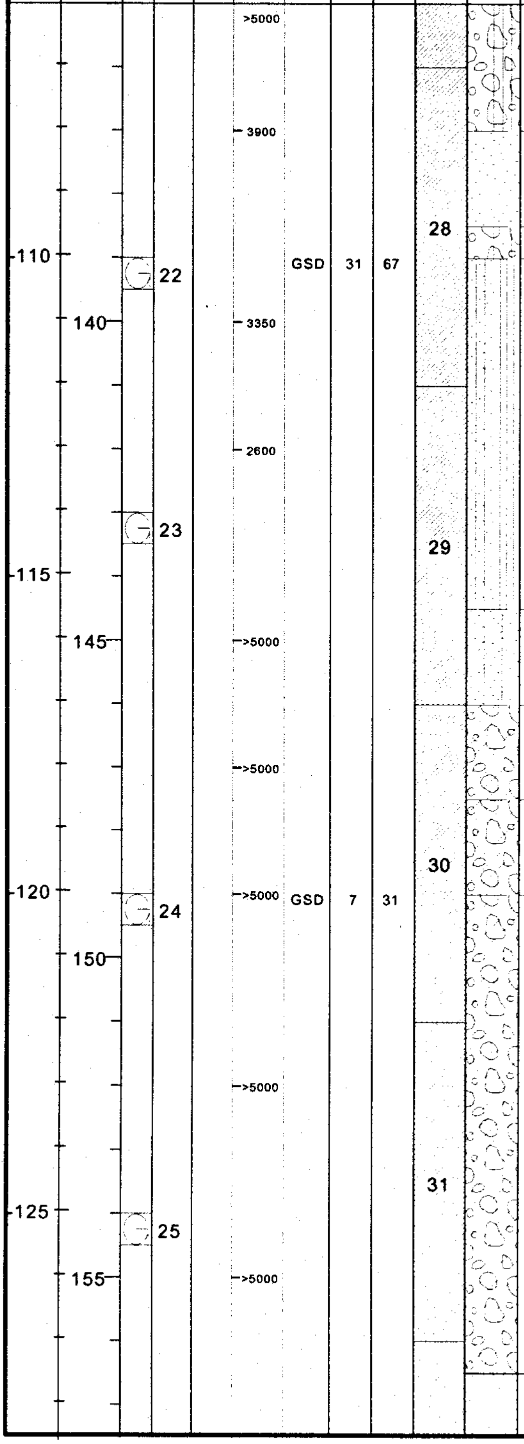
PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01

SHEET 7 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft

INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE Ⓖ Grab Sample ◯ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	LITHOLOGY	SOIL DESCRIPTION	WELL



(Tt Cont'd)

POORLY GRADED SAND (SP); with scattered gravel at top of layer; 95-100% sand, fine to medium, subangular to subrounded, hard basalt and quartz, mica; greenish gray to grayish green, wet.

Becomes dark gray poorly graded gravel with sand, silt and cobbles (GP-GM).

SILTY SAND (SM-ML); 45-55% sand, fine subangular to subrounded, quartz, basalt and others; 45-55% silt, non-plastic, moderately cemented sandstone/siltstone; very dense, dry to moist.

SILTY CLAYEY SAND WITH GRAVEL (SM); 55-60% sand, fine to medium; 15-20% gravel, fine to medium, subrounded to rounded; 25-30% silt and clay; greenish gray, very dense, weakly cemented conglomerate.

CONGLOMERATE (GC); weak to moderately indurated with angular clasts of green clayey moderately strong sandstone and matrix of gray silty clay with gravel from 146' to 147.5'.

CONGLOMERATE (GC); with mottled green, dark gray matrix of weakly indurated silty claystone; gravel, fine to coarse, subrounded to rounded.

CLAYEY GRAVEL TO WEAK CONGLOMERATE (GC); occasional zones of moderately indurated gravel conglomerate with claystone matrix, interbedded with layers of poorly indurated/weakly cemented clayey gravel; 60-65% gravel, fine to coarse subangular to rounded; 35-40% silty clay; mottled green, dark green, dark gray, wet and occasional cobbles to 4".

Sandy River Mudstone (Tsr)

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 027-003
DATE DRILLED 4/9/01 to 4/11/01
SHEET 8 OF 8
STATION NO. 79+38 (2' R)
SURFACE ELEV. 28.95 ft
INITIAL GWL@ 23.0 ft (4/10/01)
EQUIPMENT RotoSonic Drill
DRILLING METHOD Rotosonic - 6" OD Core Barrel
LOGGED BY KJL

SAMPLE TYPE G Grab Sample ○ No Recovery

ELEVATION (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	PID (ppm)	METHANE (ppm)	LAB TESTS	MOISTURE CONT.	% FINES	BOX NUMBER	LITHOLOGY	SOIL DESCRIPTION	WELL
									32		SILTY CLAY (CL-ML); medium plasticity, high dry strength, no dilatancy, medium toughness, mottled rusty brown, greenish gray, gray; dry to moist; with scattered gravel at 156.5 ft.	
130		G	26			AL, FC	25	93			**(Tsr Cont'd)** CLAYSTONE / SILTY CLAY (CL-ML); medium plasticity, medium toughness, medium to high dry strength, no dilatancy, greenish gray, dry to moist, very hard.	
	160											
					600							
									33			
135		G	27			FC	25	70			Occasional lense of silty sand/sandstone; 85-90% sand, fine.	
	165				0						SANDSTONE / POORLY GRADED SILTY SAND (SM); 85-90% sand, fine, subangular, predominantly quartz; 10-15% silt, non plastic. Sandstone is poorly indurated, easily carved with knife, greenish gray, very dense, dry, moderately to strongly cemented. Boring completed to a depth of 166 ft on 04/11/01.	
140												
	170											
145												
	175											
150												

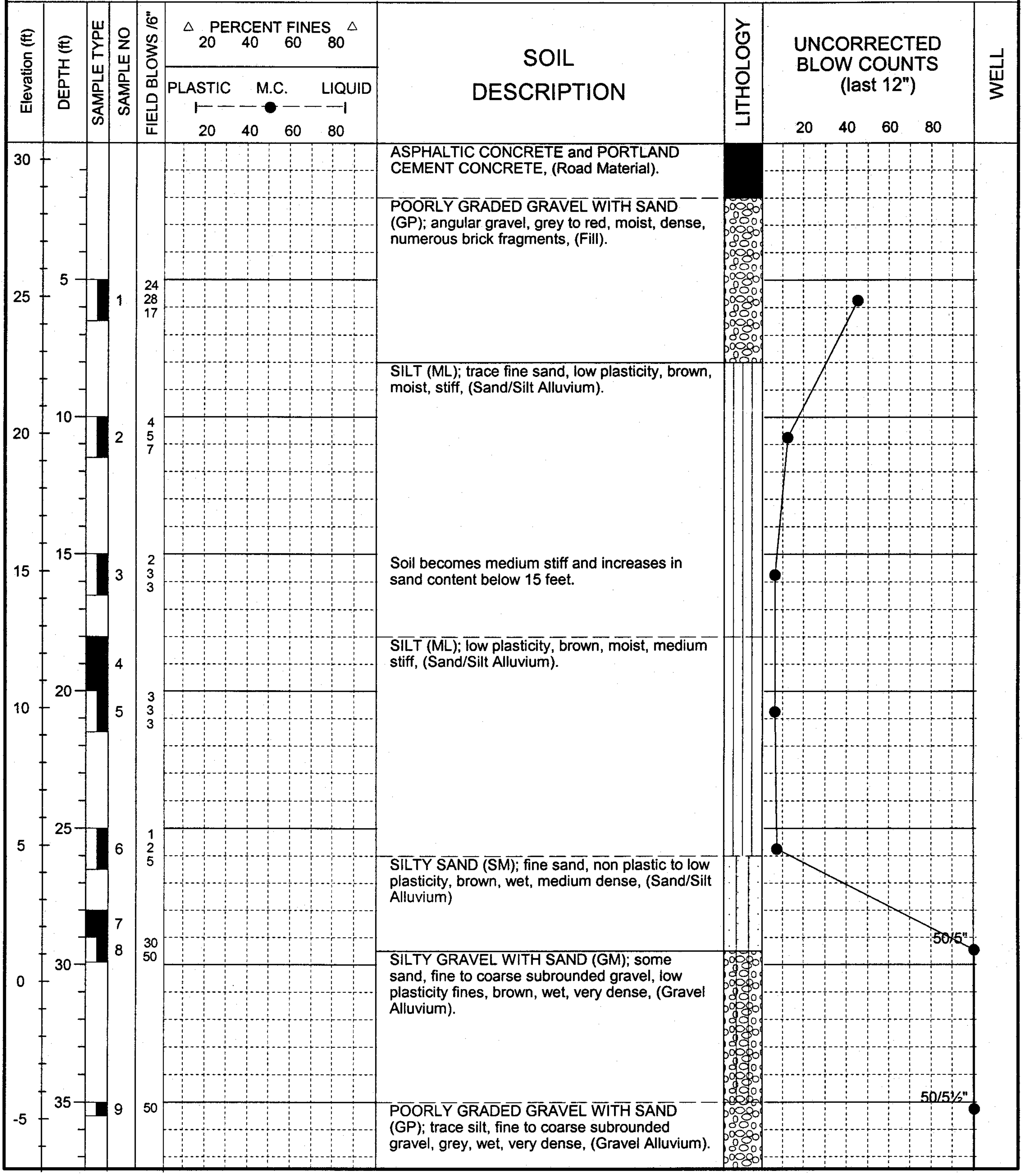
PARSONS BRINCKERHOFF

BORING LOG
PB-401A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 3/6/01 LOGGED BY AR
SHEET 1 OF 3
STATION NO. 84+19 (99L)
SURFACE ELEV. 30.62 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▮ Standard Penetration Test (2" OD) ■ Shelby Tube



SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"
▮	1	24 28 17
▮	2	4 5 7
▮	3	2 3 3
■	4	
▮	5	3 3 3
▮	6	1 2 5
■	7	
▮	8	30 50
▮	9	50

SOIL DESCRIPTION

ASPHALTIC CONCRETE and PORTLAND CEMENT CONCRETE, (Road Material).

POORLY GRADED GRAVEL WITH SAND (GP); angular gravel, grey to red, moist, dense, numerous brick fragments, (Fill).

SILT (ML); trace fine sand, low plasticity, brown, moist, stiff, (Sand/Silt Alluvium).

Soil becomes medium stiff and increases in sand content below 15 feet.

SILT (ML); low plasticity, brown, moist, medium stiff, (Sand/Silt Alluvium).

SILTY SAND (SM); fine sand, non plastic to low plasticity, brown, wet, medium dense, (Sand/Silt Alluvium)

SILTY GRAVEL WITH SAND (GM); some sand, fine to coarse subrounded gravel, low plasticity fines, brown, wet, very dense, (Gravel Alluvium).

POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse subrounded gravel, grey, wet, very dense, (Gravel Alluvium).

PB PARSONS BRINCKERHOFF

BORING LOG
PB-401A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 3/6/01 LOGGED BY AR
SHEET 2 OF 3
STATION NO. 84+19 (99L)
SURFACE ELEV. 30.62 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube

DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	UNCORRECTED BLOW COUNTS (last 12")
40	◧	10	50	50/4"
45	◧	11	41 50	50/3½"
50	◧	12	7 12 14	
	■	13		
	◧	14	6 12 21	
60	◧	15	40 37 37	
65	◧	16	50	50/2½"
70	◧	17	50	50/2"

Elevation (ft): -10, -15, -20, -25, -30, -35, -40
DEPTH (ft): 40, 45, 50, 55, 60, 65, 70

PERCENT FINES △ 20 40 60 80
PLASTIC M.C. LIQUID 20 40 60 80

SOIL DESCRIPTION

POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse subrounded gravel, grey, wet, very dense, (Gravel Alluvium).

POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse rounded gravel, grey-orange, wet, very dense, (Troutdale Formation).

SILT (ML); some clay, medium plasticity, brown, wet, stiff, (Troutdale Formation).

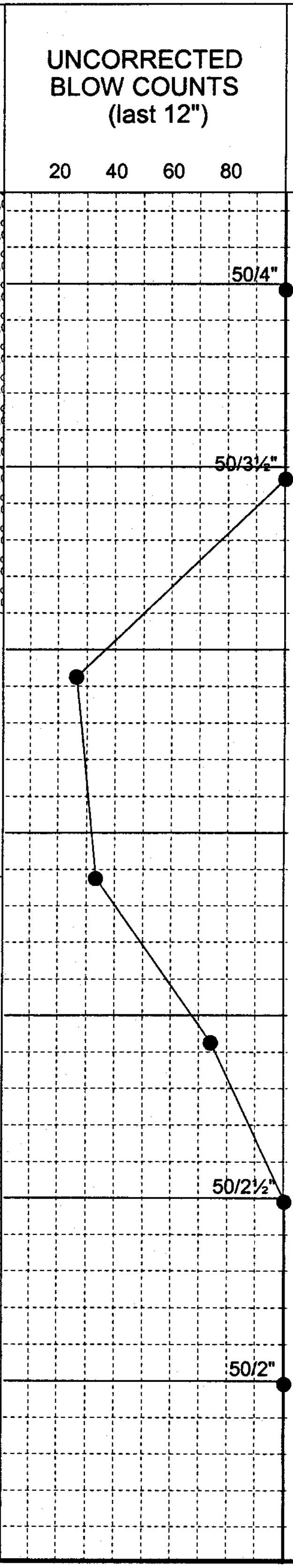
POORLY GRADED SAND (SP); trace silt, fine to medium sand, grey, wet, dense, (Troutdale Formation).

Little fine to coarse rounded gravel below 60 feet.

POORLY GRADED GRAVEL WITH SAND (GP); trace silt, some fine to coarse sand, fine to coarse rounded gravel, grey to brown, wet, very dense, (Troutdale Formation).

LITHOLOGY

WELL



PARSONS BRINCKERHOFF

BORING LOG
PB-401A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 3/6/01 LOGGED BY AR
SHEET 3 OF 3
STATION NO. 84+19 (99L)
SURFACE ELEV. 30.62 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
-45		◧	18	50		Lense of sand with gravel at 75 feet.		50/4½"	
						POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse rounded gravel, low plasticity fines, yellow-brown to grey, wet, very dense, weak cementation, (Troutdale Formation).			
-50	80	◧	19	50		Bottom of boring at 80.33 feet.		50/4½"	
-55	85				**TUNNEL CROWN**				
-60	90								
-65	95								
-70	100				**TUNNEL INVERT**				
-75	105								
-80	110								

PARSONS BRINCKERHOFF

BORING LOG
PB-401B

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 3/15/01 LOGGED BY AR
SHEET 1 OF 4
STATION NO. 84+23 (99L)
SURFACE ELEV. 30.66 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobile Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 and 300 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▮ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID ├──●──┤ 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
30						Drilled 0 to 85 feet without sampling. Material description between 0 and 80 feet is show on PB-401A.			
25	5								
20	10								
15	15								
10	20								
5	25								
0	30								
-5	35								

PARSONS BRINCKERHOFF 100 YEARS

BORING LOG
PB-401B

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 3/15/01 LOGGED BY AR
SHEET 2 OF 4
STATION NO. 84+23 (99L)
SURFACE ELEV. 30.66 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobile Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 and 300 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▮ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID ├ – – – ● – – – ┤ 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
						Drilled 0 to 85 feet without sampling. Material description between 0 and 80 feet is show on PB-401A.			
	40								
-10									
	45								
-15									
	50								
-20									
	55								
-25									
	60								
-30									
	65								
-35									
	70								
-40									

PARSONS BRINCKERHOFF

BORING LOG
PB-401B

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 3/15/01 LOGGED BY AR
SHEET 3 OF 4
STATION NO. 84+23 (99L)
SURFACE ELEV. 30.66 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobile Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 and 300 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
-45						Drilled 0 to 85 feet without sampling. Material description between 0 and 80 feet is show on PB-401A.			
-50	80					POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse rounded gravel, grey-brown, wet, very dense, weakly cemented, (Troutdale Formation).			
-55	85	◨	1	50	TUNNEL CROWN			50/2"	
		☒	2	50				50/4"	
						SILT (ML) some clay, low to medium plasticity, grey, wet, hard, (Troutdale Formation).			
-60	90	☒	3	17 50				50/4"	
						POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse rounded gravel, grey-brown, wet, very dense, weakly cemented, (Troutdale Formation).			
		◨	4	50				50/1"	
-65	95		5	50				50/0"	
			6	50				50/0"	
						Falling head permeability test conducted at 97.5 to 99.5 ft. Hydraulic conductivity is approx. 5E-06 cm/sec			
-70	100	◨	7	50	TUNNEL INVERT			50/2"	
		◨	8	50		Becomes dark green to grey below 102 feet.		50/3"	
-75	105	☒	9	50				50/2½"	
		☒	10	50				50/2"	
-80	110		11	50		Falling head permeability test conducted at 110 to 112 ft. Hydraulic conductivity is approx. 3E-06 cm/sec		50/1"	

PARSONS BRINCKERHOFF

BORING LOG
PB-401B

Foundation Engineering, Inc.

PROJECT West Side CSO Project

CITY Portland, Oregon

PROJECT NO. 2002013

DATE DRILLED 3/15/01 LOGGED BY AR

SHEET 4 OF 4

STATION NO. 84+23 (99L)

SURFACE ELEV. 30.66 ft

INITIAL GWL@ Not Available

EQUIPMENT Mobile Drill B-59

DRILLING METHOD Mud Rotary

HAMMER SYS. Manual 140 and 300 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube

DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	UNCORRECTED BLOW COUNTS (last 12")
	12	50	50/1"
115	13	50	50/2"
	14	50	50/2½"
120	15	50	50/1"
	16	50	50/2½"
125	17	50	50/1"
	18	50	50/3½"
130	19	50	50/3½"
135	20	50	50/4"
140	21	50	50/5¾"

Elevation (ft): -85, -90, -95, -100, -105, -110, -115

DEPTH (ft): 115, 120, 125, 130, 135, 140, 145

PERCENT FINES: 20 40 60 80

PLASTIC M.C. LIQUID: 20 40 60 80

SOIL DESCRIPTION

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse rounded gravel, dark grey to green, wet, very dense, weakly cemented, (Troutdale Formation).

Trace clay encountered below 130 feet.

Bottom of boring at 140.5 feet.

PARSONS BRINCKERHOFF

BORING LOG
PB-402A

Foundation Engineering, Inc

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 10/17/00 LOGGED BY AR
SHEET 1 OF 3
STATION NO. 87+72 (55 L)
SURFACE ELEV. 31.9 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▐ Standard Penetration Test (2" OD) ■ Shelby Tube

DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"
5	▐	1	6 4 3
10	▐	2	0 1 2
15	▐	3	9 8 8
20	▐	4	13 9 10
25	▐	5	3 4 3
30	▐	6	2 4 9
	▐	7	11 8 9
35	☒	8*	16 14 11

Elevation (ft): 30, 25, 20, 15, 10, 5, 0, -5

△ PERCENT FINES △ 20 40 60 80
PLASTIC M.C. LIQUID 20 40 60 80

SOIL DESCRIPTION

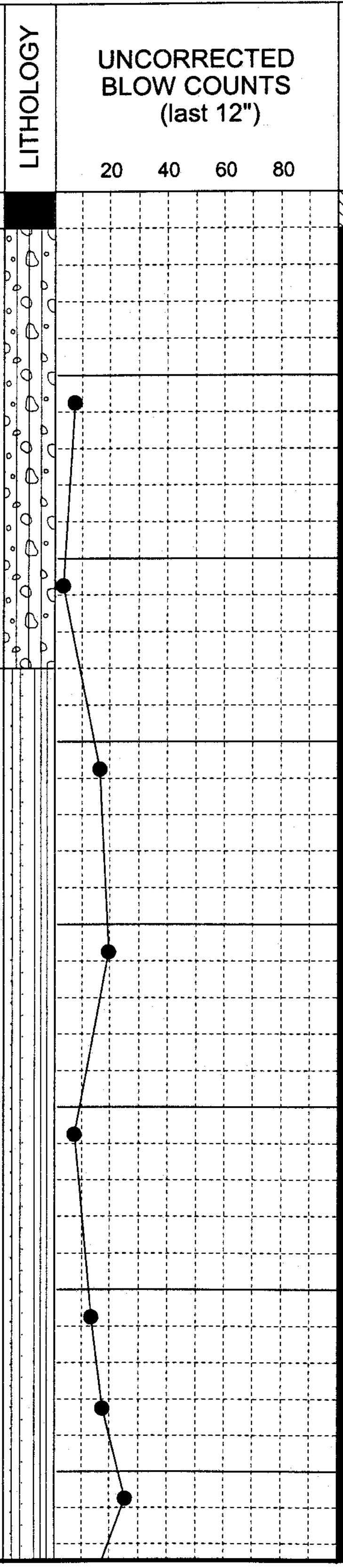
ASPHALTIC CONCRETE AND PORTLAND CEMENT CONCRETE (Road surface).

GRAVELLY SILT WITH SAND (ML); fine to coarse grained sand, angular gravel, low plasticity, grey-brown, moist, medium stiff, (Fill).

Scattered wood fragments encountered at 9 feet.
Fill becomes soft with few gravel below 10 feet.

Interbedded SANDY SILT and SILTY SAND (SM/ML); fine grained sand, low plasticity silt, grey and brown, wet, medium stiff/medium dense, stratified, (Sand/silt alluvium).

* 3 inch sampler/140 lb hammer



WELL

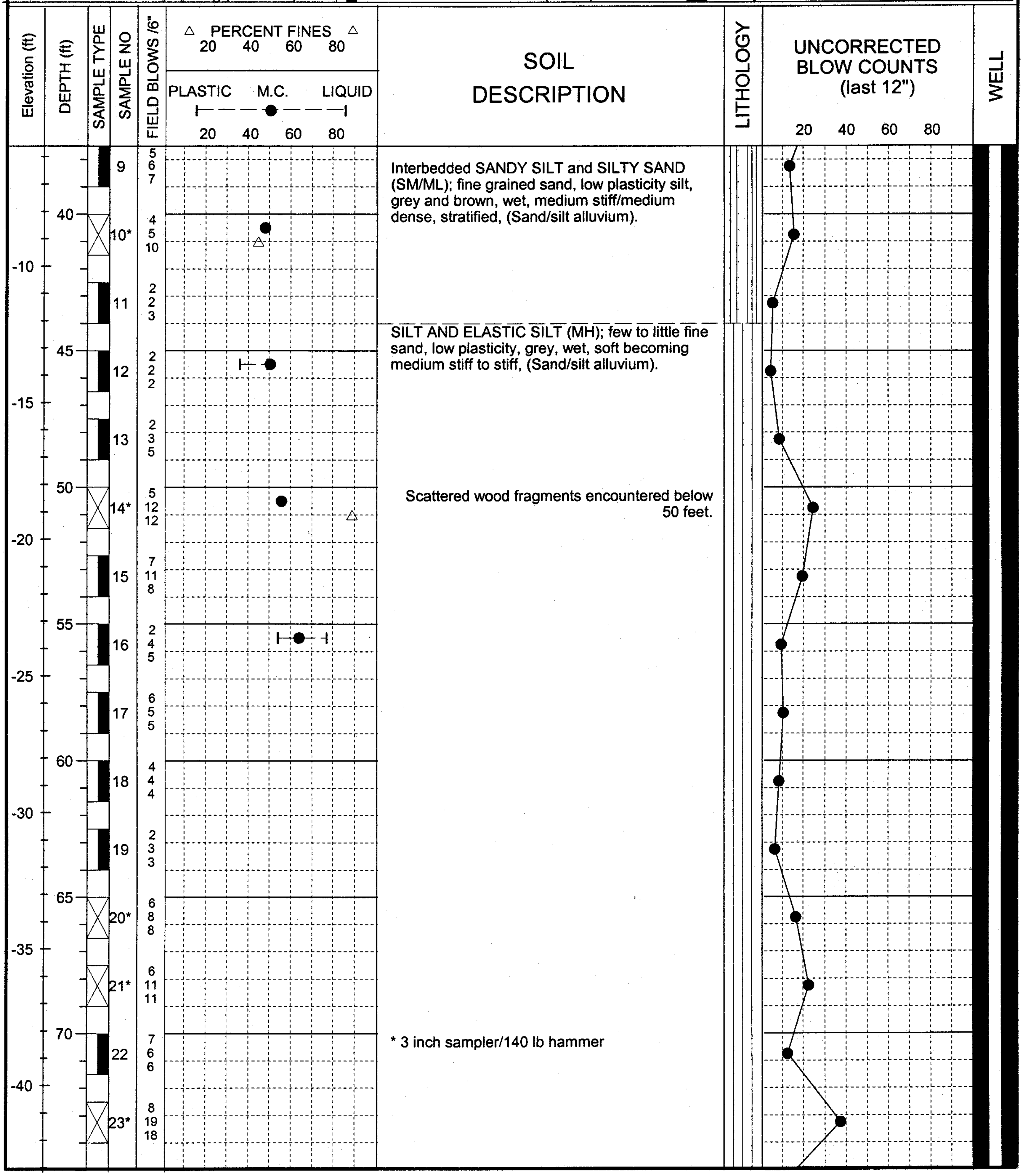
PB PARSONS BRINCKERHOFF

BORING LOG
PB-402A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 10/17/00 LOGGED BY AR
SHEET 2 OF 3
STATION NO. 87+72 (55 L)
SURFACE ELEV. 31.9 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▐ Standard Penetration Test (2" OD) ■ Shelby Tube



SAMPLE NO	FIELD BLOWS /6"
9	5 6 7
10*	4 5 10
11	2 2 3
12	2 2 2
13	2 3 5
14*	5 12 12
15	7 11 8
16	2 4 5
17	6 5 5
18	4 4 4
19	2 3 3
20*	6 8 8
21*	6 11 11
22	7 6 6
23*	8 19 18

Interbedded SANDY SILT and SILTY SAND (SM/ML); fine grained sand, low plasticity silt, grey and brown, wet, medium stiff/medium dense, stratified, (Sand/silt alluvium).

SILT AND ELASTIC SILT (MH); few to little fine sand, low plasticity, grey, wet, soft becoming medium stiff to stiff, (Sand/silt alluvium).

Scattered wood fragments encountered below 50 feet.

* 3 inch sampler/140 lb hammer

PARSONS BRINCKERHOFF

BORING LOG
PB-402A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 10/17/00 LOGGED BY AR
SHEET 3 OF 3
STATION NO. 87+72 (55 L)
SURFACE ELEV. 31.9 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. Manual 140 lb. drop

SAMPLE TYPE ☒ Ring (3.25" OD) ▌ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
		▌	24	3 3 4				●	
-45									
	80	▌	25	3 4 4				●	
						Bottom of Boring at 81.5 feet.			
-50									
	85								
-55					**TUNNEL CROWN**				
	90								
-60									
	95								
-65									
	100								
-70					**TUNNEL INVERT**				
	105								
-75									
	110								
-80									

PARSONS BRINCKERHOFF

BORING LOG
PB-900

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 9/20/00 LOGGED BY AR
SHEET 1 OF 3
STATION NO. 87+12 (111 L)
SURFACE ELEV. 30.2 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. None

SAMPLE TYPE ☒ Ring (3.25" OD) ▯ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
30						AC and PCC (Road surface).			
						BASEROCK (Road material). No sampling during drilling. Soil identification based on cuttings and drilling action.			
25	5					Interbedded SILTY SAND and SANDY SILT (ML/SM); fine grained sand, low plasticity silt, stratified, (Fill over alluvium).			
20	10								
15	15								
10	20								
5	25								
0	30								
-5	35					Wood chips encountered between 35 and 35.5 feet followed by a 3 inch gravel layer. Wood encountered between 36 and 39 feet.			

PB PARSONS BRINCKERHOFF

BORING LOG
PB-900

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 9/20/00 LOGGED BY AR
SHEET 2 OF 3
STATION NO. 87+12 (111 L)
SURFACE ELEV. 30.2 ft
INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. None

SAMPLE TYPE ☒ Ring (3.25" OD) ▯ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
-10	40					Interbedded SANDY SILT AND SILTY SAND (ML/SM), trace wood fragments; fine grained sand, low plasticity silt, stratified, (Sand/silt alluvium).			
-15	45								
-20	50								
-25	55								
-30	60					Few to little organics encountered below 60 feet.			
-35	65								
-40	70								
						Gravel lense encountered at 73 feet.			

PARSONS BRINCKERHOFF

BORING LOG
PB-900

Foundation Engineering, Inc.

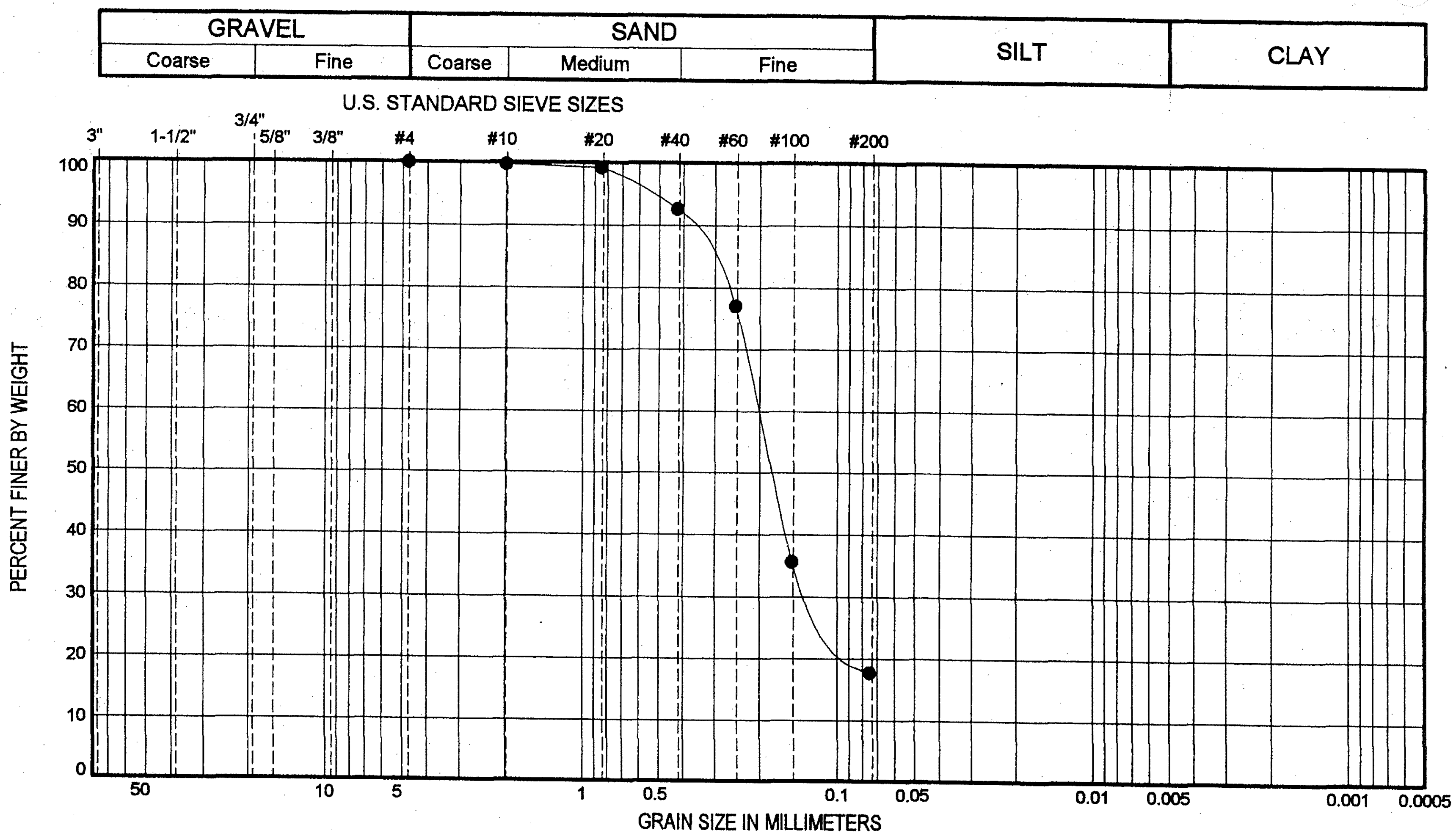
PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 9/20/00 LOGGED BY AR

SHEET 3 OF 3
STATION NO. 87+12 (111 L)
SURFACE ELEV. 30.2 ft

INITIAL GWL@ Not Available
EQUIPMENT Mobil Drill B-59
DRILLING METHOD Mud Rotary
HAMMER SYS. None

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	△ PERCENT FINES △ 20 40 60 80 PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
-45						Interbedded SANDY SILT AND SILTY SAND (ML/SM), trace wood fragments; fine grained sand, low plasticity silt, stratified, (Sand/silt alluvium).			
-50	80					Bottom of Boring at 80 feet.			
-55	85				TUNNEL CROWN				
-60	90								
-65	95								
-70	100				TUNNEL INVERT				
-75	105								
-80	110								



SYMBOL	SAMPLE		DEPTH	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	PB-305A	DM-305-14	47.5 - 49.0	SILTY SAND (SM)	36.3				0.0	82.3	17.7

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES

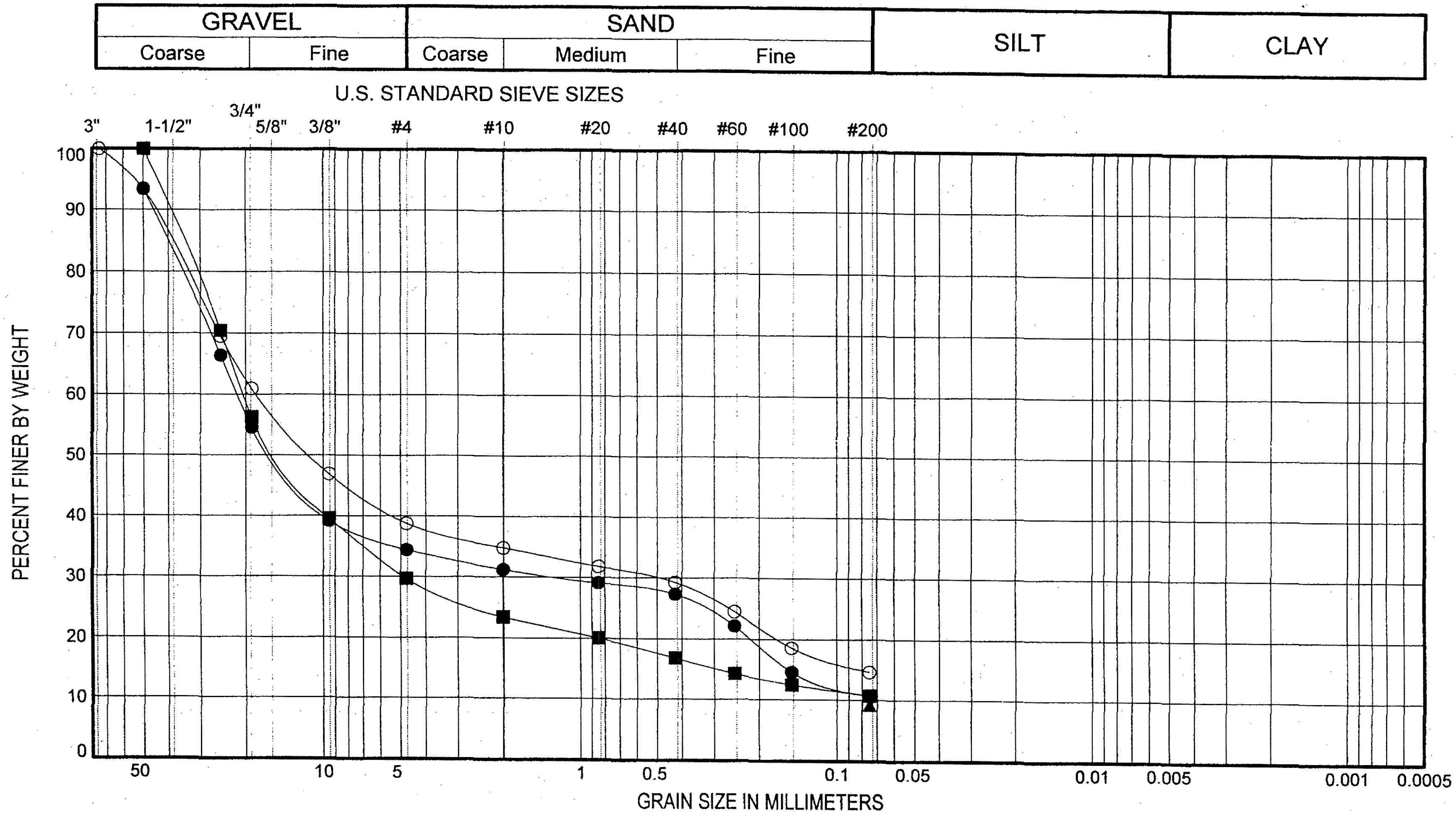
West Side CSO Project

Parsons Brinkerhoff

GRAIN SIZE ANALYSIS TEST RESULTS

Project No. 2002013

FIGURE



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	PB-306R	7A	58.0 - 58.5	Poorly graded GRAVEL with cobbles, sand, and clay (GP-GC) (Tt)	10				59	24	11
■	PB-306R	10	79.0 - 79.5	Poorly graded GRAVEL with sand and silt (GP-GM) (Tt)	10				70	19	11
▲	PB-306R	14	99.0 - 99.5	Poorly graded GRAVEL with sand and clay (GP-GC) (Tt)	9						9
○	PB-306R	15	104.0 - 104.5	Clayey GRAVEL (GC) (Tt)	11				61	24	15

PACRIM GEOTECHNICAL INC.

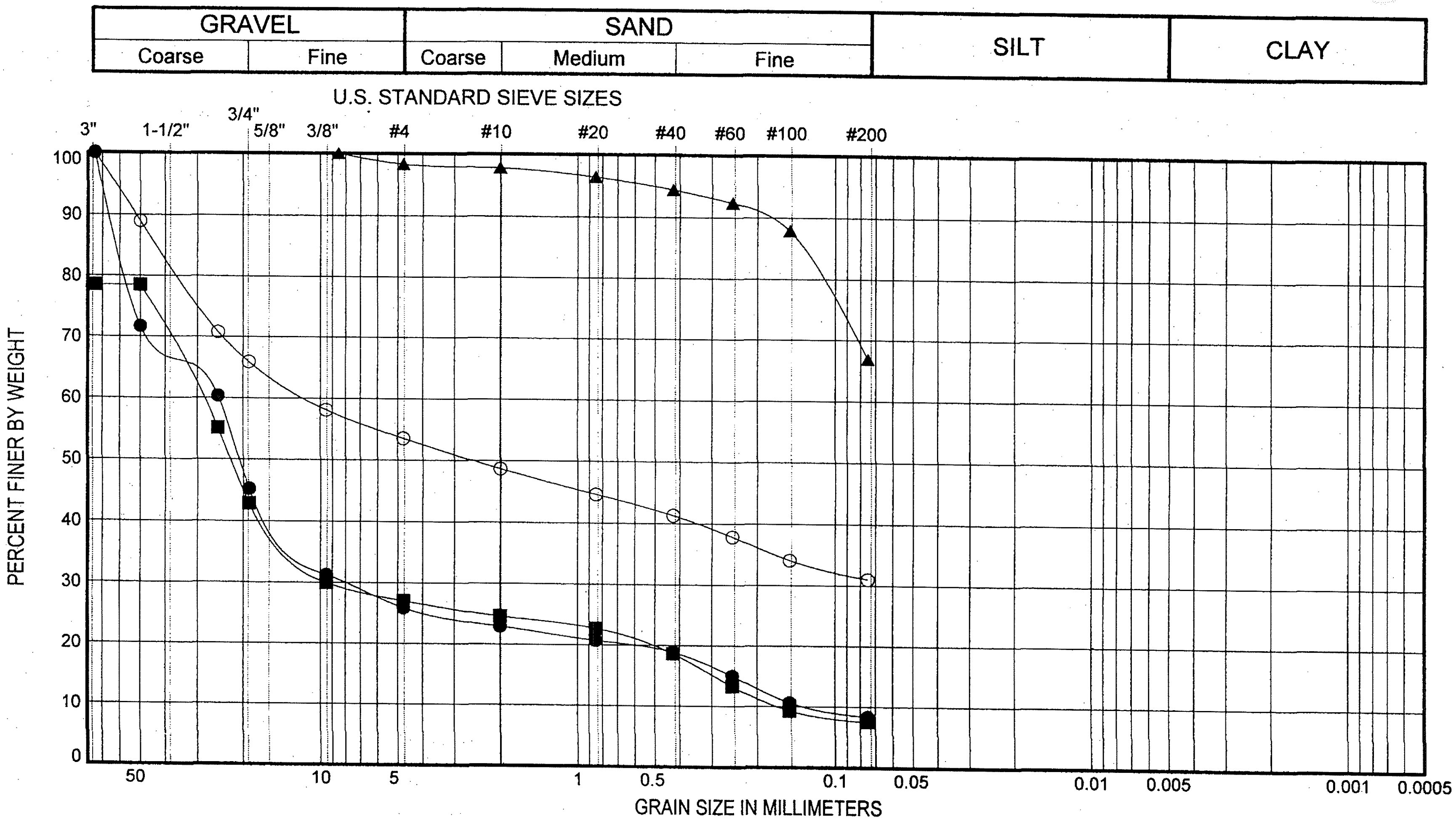
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

West Side CSO Project

Parsons Brinckerhoff

GRAIN SIZE ANALYSIS TEST RESULTS

Project No. 027-003



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	PB-306R	17	114.0 - 114.5	Poorly graded GRAVEL with sand and silt (GP-GM) (Tt)	7				74	18	8
■	PB-306R	21	134.0 - 134.5	Poorly graded GRAVEL with cobbles, sand, and silt (GP-GM) (Tt)	7				51	20	7
▲	PB-306R	22	139.0 - 139.5	Sandy SILT (ML) (SRM)	31				2	31	67
○	PB-306R	24	149.0 - 149.5	Clayey GRAVEL with sand (GC) (SRM)	7				46	23	31

PACRIM GEOTECHNICAL INC.

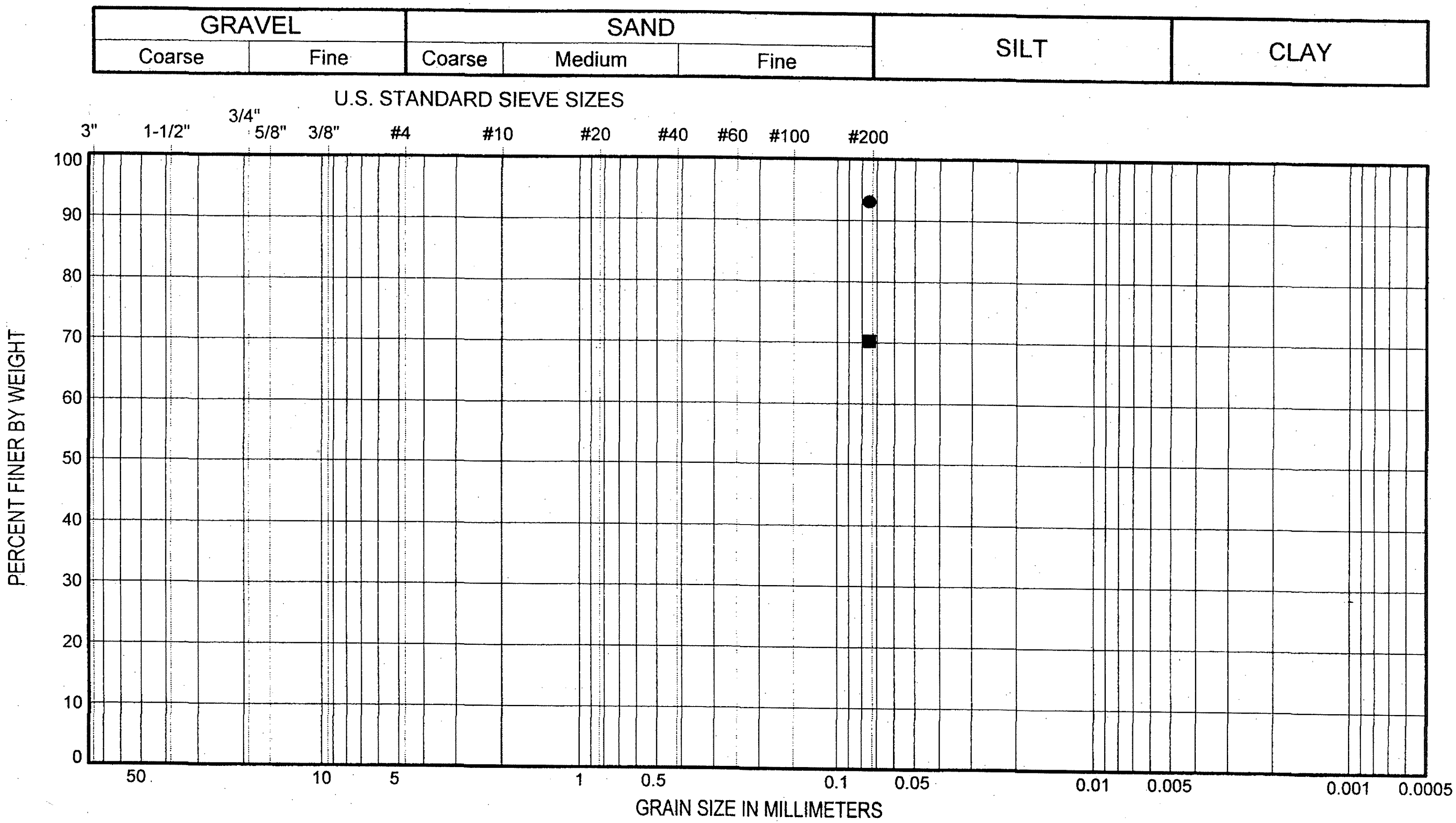
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

West Side CSO Project

Parsons Brinckerhoff

GRAIN SIZE ANALYSIS
TEST RESULTS

Project No. 027-003



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	PB-306R	26	159.0 - 159.5	Silty CLAY (CH) (SRM)	25	65	19	46			93
■	PB-306R	27	164.0 - 164.5	Claystone / Silty CLAY (CL-ML) (SRM)	25						70

PacRim Geotechnical Inc.

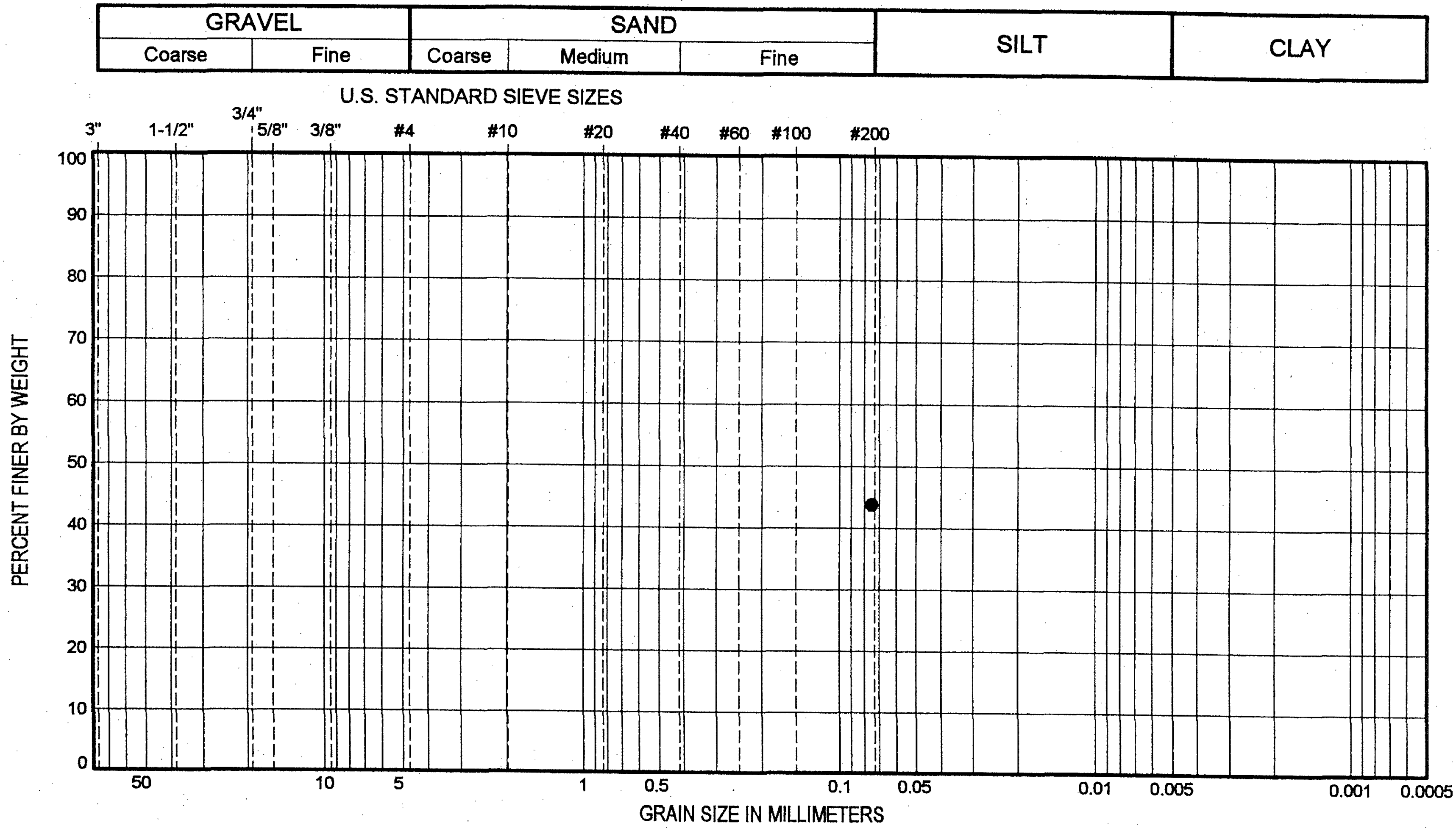
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

West Side CSO Project

Parsons Brinckerhoff

GRAIN SIZE ANALYSIS TEST RESULTS

Project No. 027-003



SYMBOL	SAMPLE		DEPTH	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	PB-402A	DM-402-10	40.0 - 41.5	SILTY SAND (SM)	47.0						43.8

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES

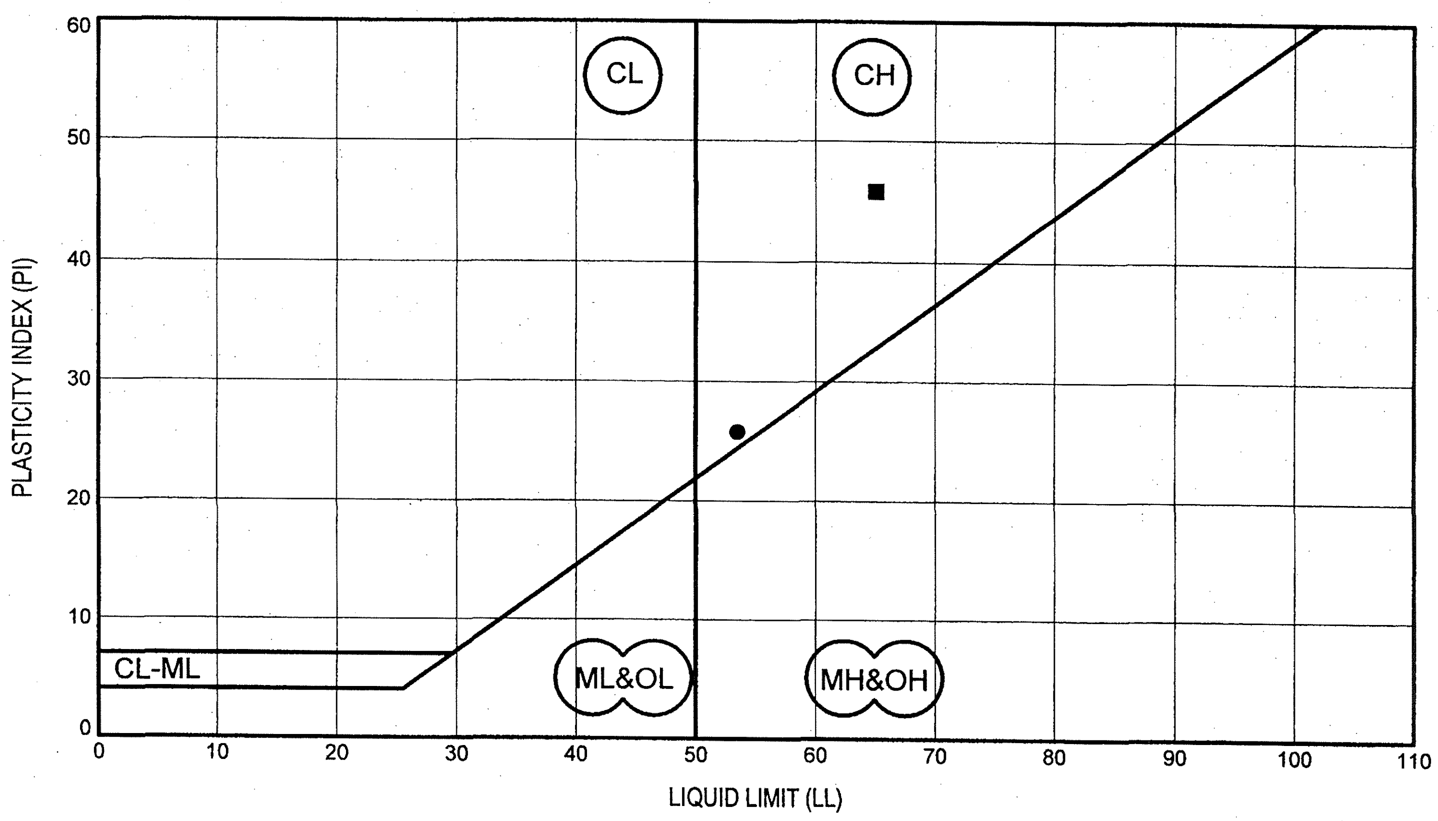
West Side CSO Project

Parsons Brinkerhoff

GRAIN SIZE ANALYSIS TEST RESULTS

Project No. 2002013

FIGURE



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	PB-306R	13	94.0 - 94.5	Silty CLAY (CH) (Tt)	43	53	28	25	
■	PB-306R	26	159.0 - 159.5	Silty CLAY (CH) (SRM)	24	65	19	46	93

PacRim Geotechnical Inc.

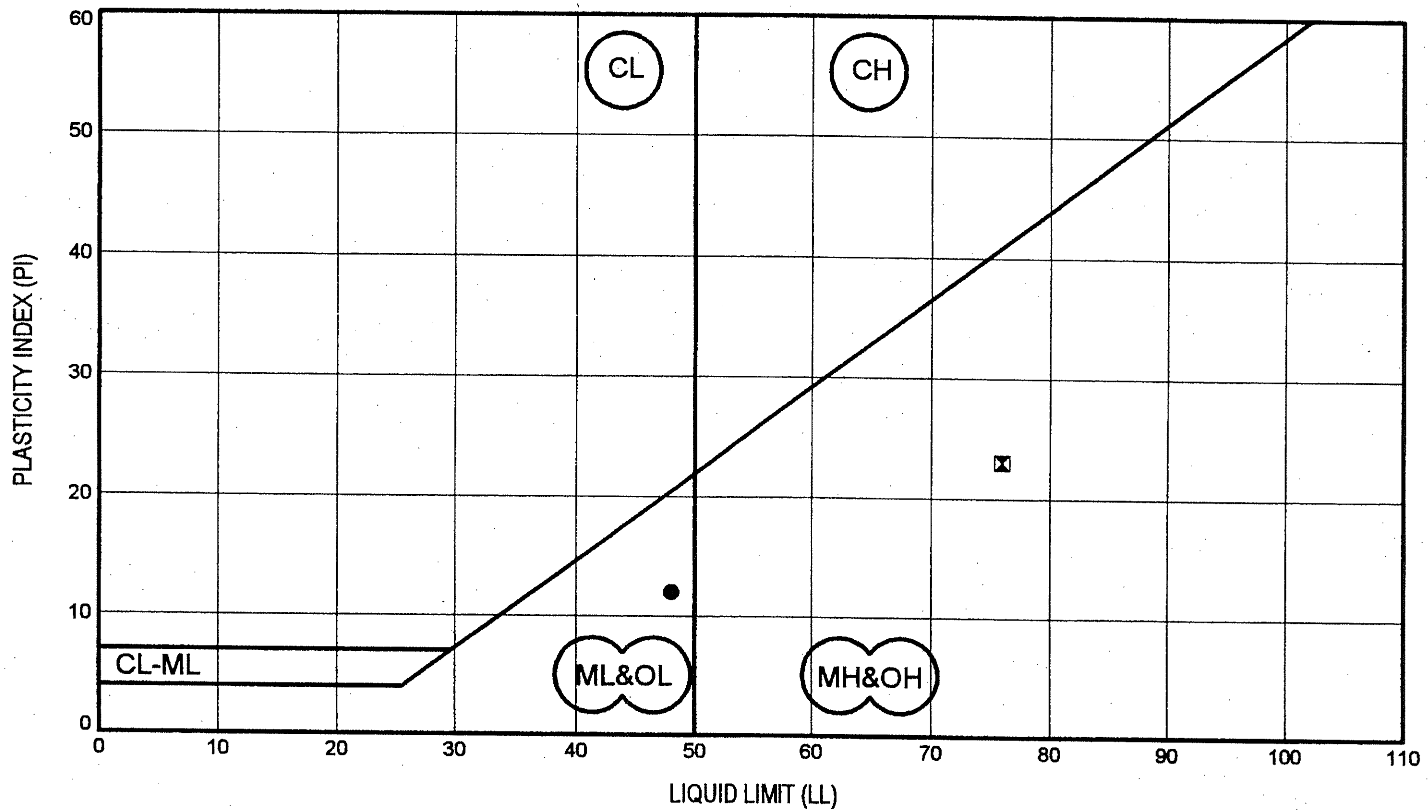
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

West Side CSO Project

Parsons Brinckerhoff

ATTERBERG LIMITS
TEST RESULTS

Project No. 027-003



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	PB-402A	SS-402-12	45.0 - 46.5	SILT (ML)	50	48	36	12	
☒	PB-402A	SS-402-16	55.0 - 56.5	ELASTIC SILT (MH)	63	76	53	23	

FOUNDATION ENGINEERING INC.
PROFESSIONAL GEOTECHNICAL SERVICES

West Side CSO Project

Parsons Brinkerhoff

ATTERBERG LIMITS TEST RESULTS

Project No. 2002013

FIGURE

APPENDIX E.2
CORROSIVITY DATA

GROUNDWATER

Shaft Locations	Boring No.	Depth (ft)	Date Sample Collected	Chlorides (mg/L)	Sulfate (mg/L)	Sulfides (mg/L)	Total Dissolved Solids (mg/L)	Total Solids (mg/L)	Total Suspended Solids (mg/L)	Conductivity (µmhos/cm)	pH	Nitrates (mg/L)
Min Values Detected (not incl non-detects)				5.5	2.04	0.22	240	238	0	330	6.6	1.7
Maximum Values Detected				34.1	60.7	0.22	410	902,000	548	670	8.3	4.6
Clay Street Shaft	PB-109R	74	06/15/01	14	60.7		400	398	1	530	7.2	
Ankeny Shaft	PB-306R		7/19/01	9.1	3.5	0.22	250	251	2.4	370	7.4	
Albers Mill Access Shaft	PB-602A	18	4/2/01									
	PB-602A	115-125	6/27/01	17	4.1		240	310	71	363	7.2	
Swan Island Pump Station	PB-1003R	20	6/20/01									
	PB-1003R	55	6/21/01									
	PB-1003R	142-194.5	6/14/01	14	2.04		240	238	1	340	7.8	
	PB-1005R	24	6/22/01									
	PB-1005R	55	6/22/01									

Field Data from Shallow Grab and Deep Groundwater Samples

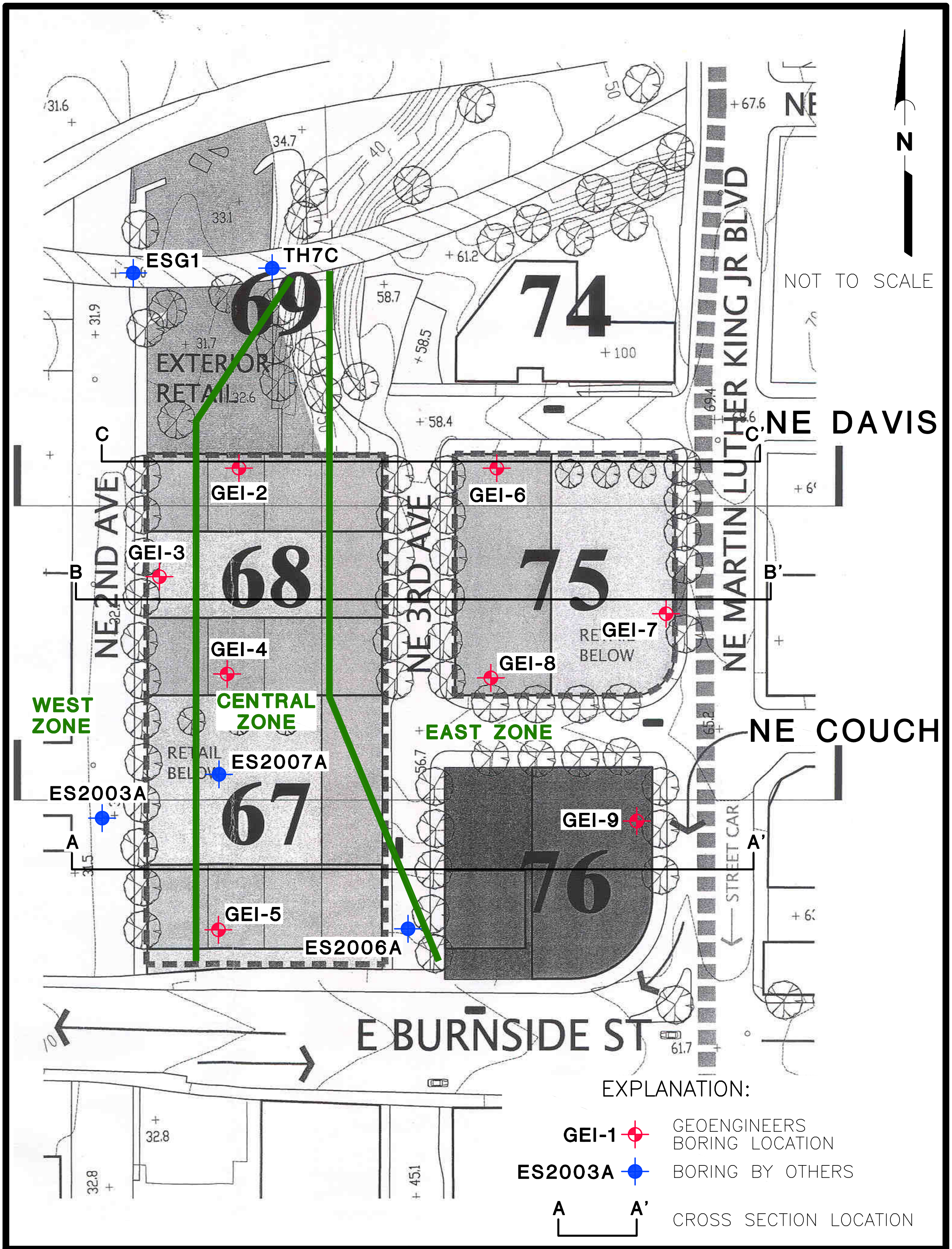
Specific Conductance (µSiemens/cm)	Temp (C)	pH	Eh (mV)	Turbidity Data from wells only (NTUs)
104.3	9.5	5.97	-505	0.67
1,814	26.2	7.83	71.4	8
492	16.8	6.71	-260	0.8
654.7	13.2	6.48	28	
353	14.7	6.7	-177	8
1,122	21.2	7.83	-258	---
995	21.6	6.07	-176.8	---
330.6	17.2	7.52	-306.2	2.81
1,622	15.4	6.57	-215.4	---
1,475	16	6.65	-193.5	---

Notes:
1. Data presented only at shaft locations. All other corrosivity data presented in the Environmental Data Report.
2. Data collected by CH2M Hill through October 30, 2001.

SOIL

Locations	Boring No.	Depth (ft)	Soil Description	Formation	pH	Chlorides* (ppm)	Sulfate* (ppm)	Redox Potential (millivolt)	Minimum Resistivity (ohm-cm)
				TEST METHOD	ASTM D4972	SM 4500 - Cl^- B	SM 4500 SO_4^{2-} E	ASTM D1498	Specific Conductance
Couch Lake	PB-504	90-100	Silty Sand to Sandy Silt	Qal	6.3	28	55	55	4,720
Peninsular FM	PB-1402A / PB-1404A	15-26	Poorly Graded Sand	Qaf	6.8	7	10	10	23,530
	PB-1404A / PB-1405A	12-16	Silty Sand	Qaf	6.7	14	21	21	21,980
Swan Island PS	PB-1202A	15.5-30.5	Silty Clay	Qaf	5.6	70	180	180	1,870

Notes:
* - 1:1 water extraction, 24 hours
1. Tests run on soil samples collected during Phase B and C geotechnical investigations.
2. Qaf - Fill; Qal - Sand/Silt Alluvium
3. SM - Standard Methods for the Examination of Water and Wastewater, 18th ed, 1992.
4. ASTM - American Standard for Testing and Materials



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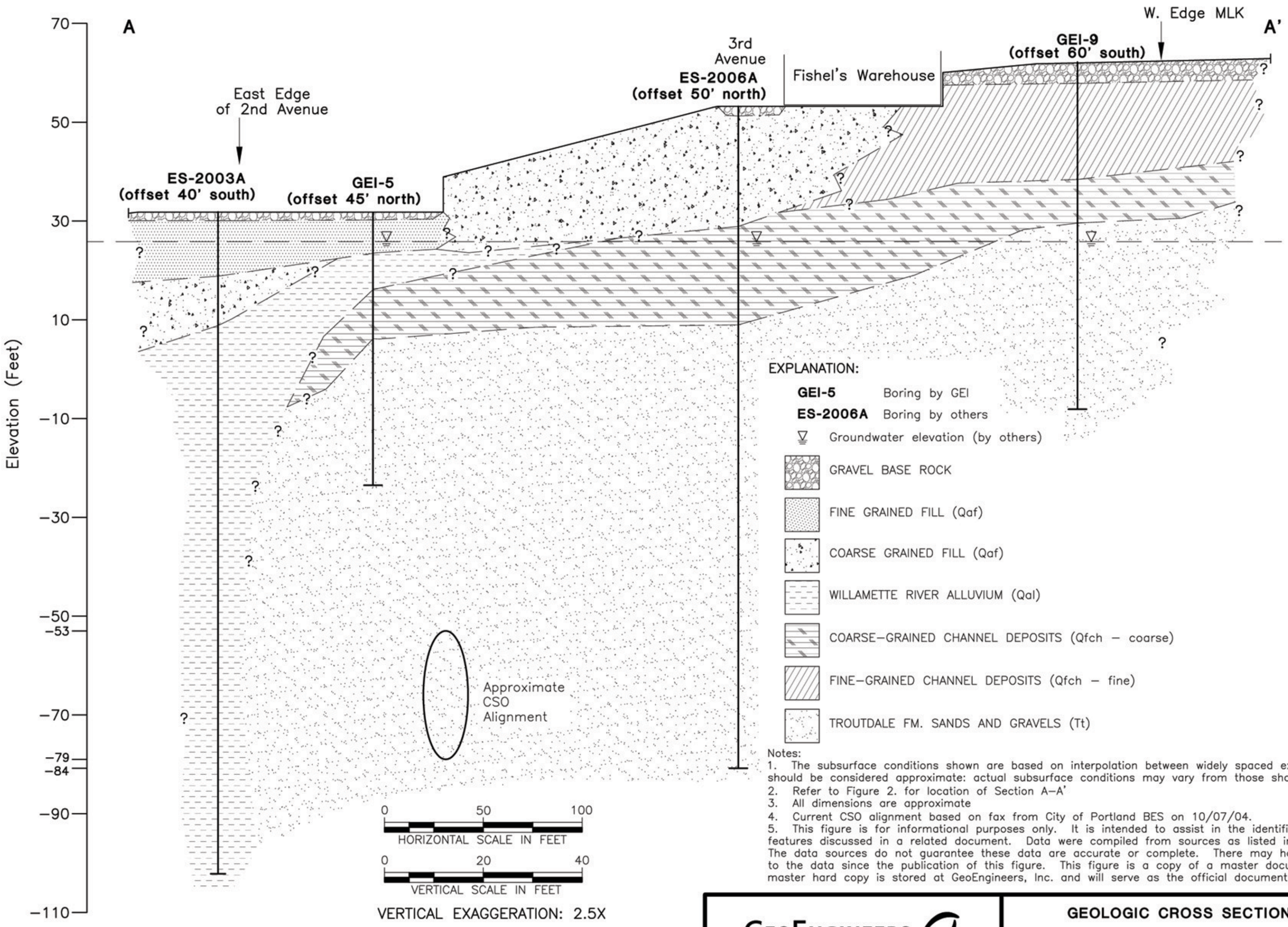
GEOENGINEERS

SITE PLAN

FIGURE 2



Notes:

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate: actual subsurface conditions may vary from those shown.
2. Refer to Figure 2. for location of Section A–A'
3. All dimensions are approximate
4. Current CSO alignment based on fax from City of Portland BES on 10/07/04.
5. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored at GeoEngineers, Inc. and will serve as the official document of record.

GEOENGINEERS

GEOLOGIC CROSS SECTION A-A'

FIGURE 4

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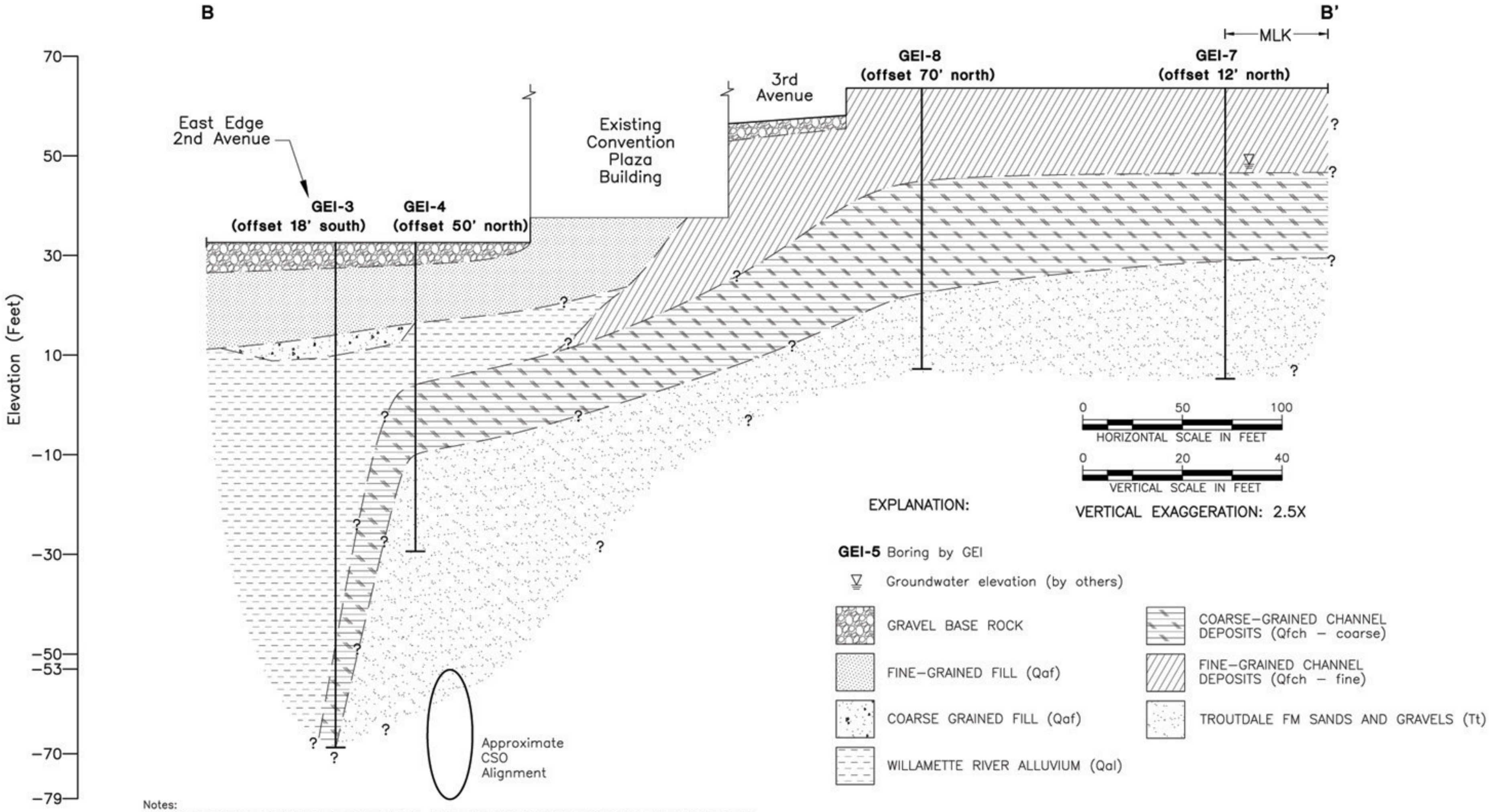


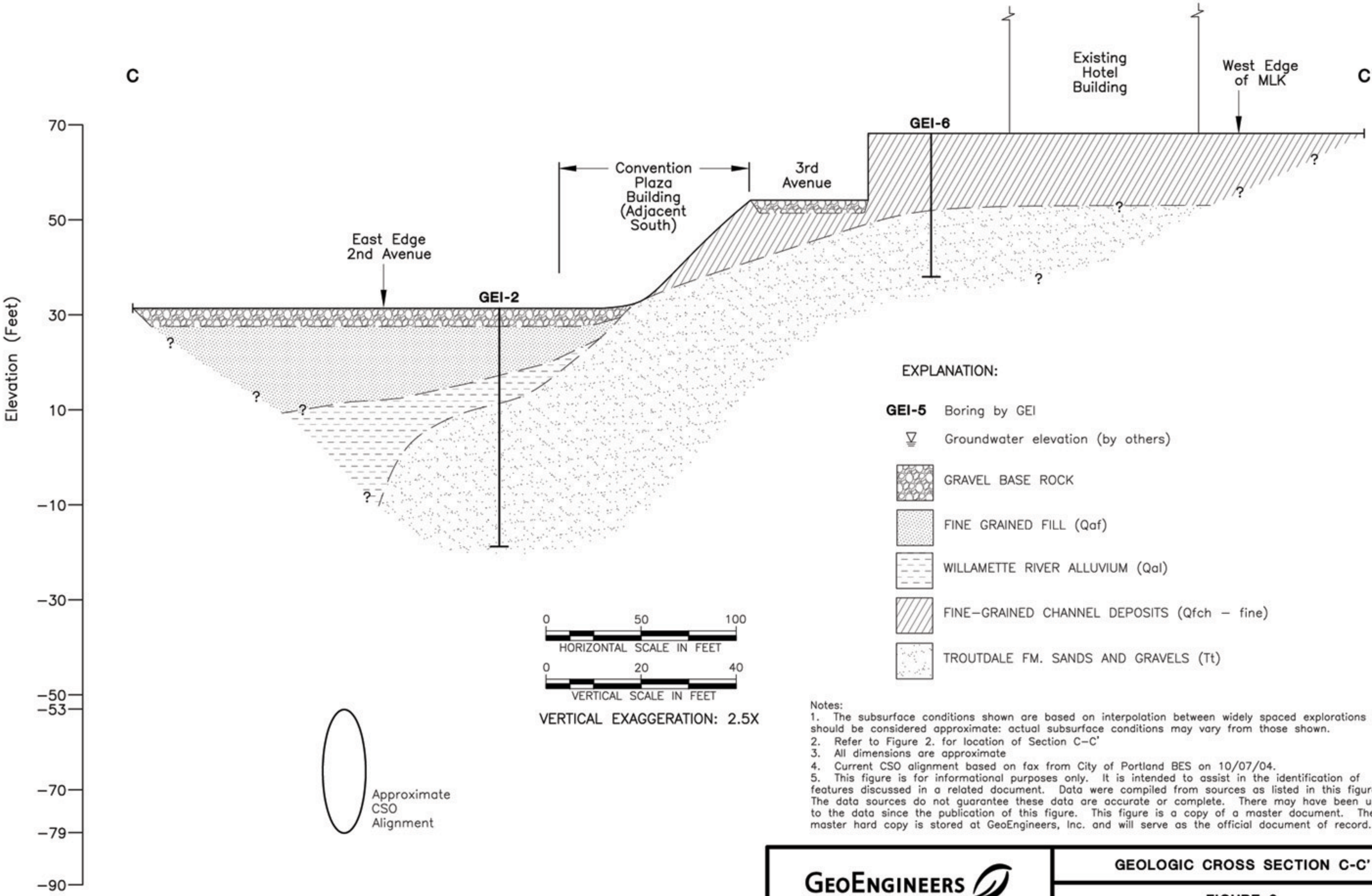
Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate: actual subsurface conditions may vary from those shown.
2. Refer to Figure 2. for location of Section B-B'
3. All dimensions are approximate
4. Current CSO alignment based on fax from City of Portland BES on 10/07/04.
5. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored at GeoEngineers, Inc. and will serve as the official document of record.

GEOENGINEERS

GEOLOGIC CROSS SECTION B-B'

FIGURE 5

Portland\P:\1\1792018\00\Cad\CrossBB.dwg TWB:MWJ 10/27/04



Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate: actual subsurface conditions may vary from those shown.
2. Refer to Figure 2. for location of Section C–C'
3. All dimensions are approximate
4. Current CSO alignment based on fax from City of Portland BES on 10/07/04.
5. This figure is for informational purposes only. It is intended to assist in the identification of features discussed in a related document. Data were compiled from sources as listed in this figure. The data sources do not guarantee these data are accurate or complete. There may have been updates to the data since the publication of this figure. This figure is a copy of a master document. The master hard copy is stored at GeoEngineers, Inc. and will serve as the official document of record.

GEOENGINEERS

GEOLOGIC CROSS SECTION C-C'

FIGURE 6

Portland\P:\1\1792018\00\Cad\CrossCC.dwg TWB:MWJ 10/27/04

Date(s) Drilled	07/07/04 - 07/08/04	Logged By	MR2	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Mud Rotary	Sampling Methods	SPT / D&M / Grab
Auger Data	Tri-Cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-57
Total Depth (ft)	50	Surface Elevation (ft)	32	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Recovered (in)	Blows/foot	Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0					GP	Pea gravel (fill)			Drill times in parentheses
	18	6	1		ML	Brown/green micaceous silt and sand, roots (medium stiff, moist)			
5	18	10	2			Increasing roots/organics becomes hard	33	89	(1350)
	18	4	3			Decreasing sand becomes medium stiff			(1400)
10	12	6	4			Brown/gray	38	87	(1410)
15	3	12	5			With occasional well rounded gravel becomes hard	26		(1420)
					GW-GM	Coarse round gravel with sand and silt (very dense, wet)			
20	6	50/6"	6				17		Boulders deflect rod (1435)
						Hole caves, lose mud at 24 feet			End of day, 7/7/04 at 1600. Start drilling 7/8/04 at 0800
25									

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-2

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 2
Sheet 1 of 2

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
25	■	6	50/6"	I	7				Gravels are 60% basalt and 40% metaphorphic quartzite, drilling hard	8		(0845)
30					8				Gravels are 60% basalt and 40% felsic, drilling hard			On boulders, no sample; Grab sample (0940)
35					9				Drilling easier			Grab sample (1000)
									Drilling hard, apparent boulders			
40					10							Grab sample (1035)
									Gravels are 50% basalt and 50% felsic, hard drilling			
45					11							Grab sample (1110)
									Gravels 40% basalt and 60% felsic			Grab sample (1145)
50									Boring terminated at 50 feet Groundwater depth not determined			
55												

LOG OF BORING GEI-2 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 2
Sheet 2 of 2

Date(s) Drilled	07/16/04 - 07/17/04	Logged By	DRL	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Mud Rotary	Sampling Methods	SPT / D&M / Shelby
Auger Data	Tri-cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-57
Total Depth (ft)	106.5	Surface Elevation (ft)	32	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (In)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
0								CR	2 inches asphalt concrete over base course (fill)			Drill times in parentheses
								GM	Brown silty fine gravel with sand (dense, moist) (fill)			
	■	7	65	I	1							(0922)
									Grades to silty fine to coarse gravel			
5	■	6	9	I	2			ML	Gray silt with fine gravel (medium stiff to stiff, moist) (fill)	33		(0955)
	■	5	7	I	3							
10	■	0	8	I	4				Becomes soft			(1005)
15	□	0		I	5					30		
								GM	Gray silty fine gravel with sand and wood/organics (medium dense, moist, mottling) (fill)			
	■	18	65	I	6							No recovery; buried log 17.5-21'
20	■	8	67	I	7							Lose mud
								ML	Gray silt with fine gravel and organic matter (wood) (soft, moist)			11:55 to 13:30 pull rods, set casing
25												

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-3

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 3
Sheet 1 of 4

	SAMPLES											
Depth feet	Interval	Recovered (In)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
25		7	7		8							(1445)
												Boring terminated at 26.5 feet due to refusal on wood and brick; started new boring 6 feet south
30		18	4		9				Gray silt (soft, wet)	57		(1650)
35		0	11		10							(1700) lose sample
40		0			11				Becomes very soft	56		Grab sample; most is lost
									Grades with occasional fine gravel			
45		18	4		12				Grades to very soft to soft			(1730) Driller reports small gravel
50		18	3		13					48		
55		0			14							

LOG OF BORING GEI-3 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 3
Sheet 2 of 4

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (In)	Blows/foot	Sub-Sample	Sample Number							
	□			[								
60	■	12	3	[	15				Grades to clayey with fine sand and occasional medium to coarse sand (soft, wet)	59		% Fines=81 (1840)
65	■	18	4	[	16					54		% Fines =89 End of day, 07/16/04 at 2000. Start drilling 07/17/04 at 0910
70	■	3	12	[	17				Becomes stiff			
75	□	0							Drive Shelby 12 inches and hit hard object; stopped			Let sit 10 minutes; no recovery
80	■	4	44	[	18			GW-GM	Rounded gravel with sandy silt (dense, wet)			(0955)
									Hard drilling			
85	■	4	58	[	19			SW-SM	Coarse sand with silt and gravel (dense, wet) (70% basalt and 30% felsic)	21		% Fines =7 Lose mud, hole caves (1100)

LOG OF BORING GEI-3 (continued)

GeoEngineers

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 3
Sheet 3 of 4

Depth feet	SAMPLES Interval	 Recovered (in)	 Blows/foot	 Sub-Sample	 Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
								GW	Increasing cobble size			
90		6	30		20							Lose mud; blow pressure relief valve (1230)
95					21				Drill past caving zone increasing gravel (60% basalt, 40% felsic) too much slough to do SPT's			Grab sample (1410)
100		12	98/11"		22			GW-GM	Sandy coarse gravel (dense, wet) drove 36 inches through slough (final blow counts suspect)	18		% Fines =11 (1450)
105		14	25		23			SP-SM	Orange-brown medium sand with gravel and silt; grades to coarse sand			Hole caves (1610)

Boring terminated due to caving
Groundwater not determined

110

115

LOG OF BORING GEI-3 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Date(s) Drilled	07/08/04 - 07/09/04	Logged By	MR2	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Mud Rotary	Sampling Methods	SPT / D&M / Grab
Auger Data	Tri-Cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-57
Total Depth (ft)	61	Surface Elevation (ft)	32	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
0								ML				Drilling times in parentheses
		12	6		1				Brown/gray silt occasional fine sand, brick, roots (medium stiff, moist) (fill)	21		
5		12	9		2				Increased sand and roots			
		18	5		3				Becomes blue/gray with roots and branches			
10		12	9		4				Increasing sand - wood particles (black) decayed with clay	33	90	
15		18	4		5			ML	Blue/gray mottled silt with sand and decayed organics (soft, moist)			
20		15	5		6				Increasing clay, occasional gravel, becomes medium stiff			
												Driller reports gravel, deflects rod
25												

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-4

GEOENGINEERS

Project:	Portland Development Commission
Project Location:	Portland, Oregon
Project Number:	1792-018-00

Figure: A- 4
Sheet 1 of 3

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
25		6	12		7			SM	Brown silty sand with gravel (medium dense, moist)	22		
					8			GW-GM	Gravels: 70% basalt and 30% felsic (occasional quartzite) with sand and silt			
30		18	42		9			GW	Rounded to sub-rounded coarse gravel with occasional cobbles (medium dense to dense) Hard drilling			
35		2	50/2"		10			GW-GM	Subrounded to sub angular coarse gravel with sand and silt (very dense) gravels are 70% basalt, 30% felsic			
	☒	0			11				Less silt, gravels 60% basalt, 40% felsic			
40		0	50/1"		12			GW	Well-graded gravel with sand (very dense). Gravels 50% basalt, 50% felsic. Hard drilling.			
45		0	50/0"		13				Gravels 40-50% basalt, 50-60% felsic. Hard drilling			Grab sample
50		3	50/3"		14				Increasing basalt fragments			
55		9	50/3"		15			SW	Orange/brown medium sand with silt, occasional coarse basalt black sand (very dense) Interpret as Troutdale			

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LOG OF BORING GEI-4 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 4
Sheet 2 of 3

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
							▨		Sandstone.			
60	■	0	50/0"	I	16		▨					(1510)
									Boring terminated at 61 feet Groundwater not determined			
65												
70												
75												
80												
85												

LOG OF BORING GEI-4 (continued)

GeoEngineers

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 4
Sheet 3 of 3

Date(s) Drilled	07/06/04 - 07/07/04	Logged By	MR2	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Mud Rotary	Sampling Methods	SPT / D&M
Auger Data	Tri-Cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-57
Total Depth (ft)	55	Surface Elevation (ft)	32	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
0								AC	6 inches asphalt concrete			Drilling times in parentheses
								GW	12 inches base rock (fill)			
								ML	Mottled gray green clayey silt with occasional gravel (stiff, moist) (fill)			
		18	16		1							
5		10	8		2					16	103	
									Becomes medium stiff with roots, brick fragments and some medium sand			
		12	7		3							
10		18	3		4			ML	Silt with clay and occasional coarse sand (soft, moist)			(0935)
15		0	39		5			GW-GM	Subangular to subrounded well graded gravel 80% basalt 20% felsic (medium dense, moist). Hard drilling			Grab sample at 16 feet. Lose mud
		6	22		7					18		(1000)
20		6	39		8					9		% Fines =2 (1145)
									Becomes more coarse with possible cobbles, and increasing sand and silt (dense)			
25												

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-5

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 5
Sheet 1 of 2

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
25		12	50/6"		9				Becomes very dense			(1240)
												(1400) Hole caved, redrill
									Rounded gravels and increasing sand			
30		4	50/4"		10							(1535)
35		3	50/3"		11							End of day, 7/6/04 at 1625. Start drilling 7/7/04 at 0900.
									Hard drilling			
40		6	50/6"		12			SW	Coarse sand to fine gravel (very dense, moist) Clasts are 80% basalt, 20% felsic			(1015)
45		4	50/4"		13				Gravels are 70% basalt, 30% felsic			(1105)
50		3	50/3"		14							(1205)
												(1300) Hole caves; no sample
55									Boring terminated due to caving Groundwater not determined			

LOG OF BORING GEI-5 (continued)

GeoEngineers

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 5
Sheet 2 of 2

Date(s) Drilled	06/28/04	Logged By	RNM	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Mud Rotary	Sampling Methods	SPT/ D&M / Grab
Auger Data	Tri-Cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	30.3	Surface Elevation (ft)	65	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
0								AC	3 inches asphalt concrete			
								ML	Brown silt (very stiff, moist)			
5		18	32		1				Becomes sandy	37	83	
10		18	38		2					32	87	CS
15		18	99/11"		3			GW	Gray to brown fine to coarse gravel with sand, trace silt (very dense, moist)			
								GW-GM	Brown fine to coarse gravel with sand and silt (very dense, moist)			
20		2	50/3"		4							
25	☒											

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-6

GeoEngineers

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 6
Sheet 1 of 2

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
25	☒		58	I	5					13		
								SP-SM	Gray fine to medium sand with silt, occasional gravel (very dense, moist)			
								GW-GM	Brown fine to coarse gravel with sand and silt (very dense, moist)			
30	■	2	50/4"	I	6							
									Boring terminated at 30.3 feet Groundwater not determined			
35												
40												
45												
50												
55												

V6 GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV6 1.GDT 11/4/04

LOG OF BORING GEI-6 (continued)

GEOENGINEERS

Project:	Portland Development Commission
Project Location:	Portland, Oregon
Project Number:	1792-018-00

Figure: A- 6
Sheet 2 of 2

Date(s) Drilled	06/29/04 - 06/30/04	Logged By	RMN	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Direct Push/Hollow Stem Auger	Sampling Methods	SPT/ D&M
Auger Data	Tri-cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-57
Total Depth (ft)	58	Surface Elevation (ft)	65	Groundwater Elevation (ft)	50
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
0							AC	3 inches asphalt concrete			
							ML	Brown silt (stiff to very stiff, moist)			
5	■	16	30	I	1				23	84	
10	□	24			2						DS
15	■	15	67	I	3	▽		Becomes sandy	29		Perched groundwater
							GW-GM	Gray fine to coarse gravel with sand and silt (very dense, moist)			
							SP	Gray fine to medium sand with gravel, trace silt (very dense, moist)			
20	■	14	65	I	4				6		
25											

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-7

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 7
Sheet 1 of 3

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
25		16	36		5			ML	Brown with orange mottling, silt with occasional fine gravel			End drilling 1650; start drilling at 0905 on 06/30/04
								SP	Gray fine to medium sand with occasional fine gravel, trace silt (dense, moist)			
								SW-SM	Gray fine to coarse sand with silt, occasional fine gravel (very dense, moist)			
30		8	84/11"		6					21		(1015)
								GW-GM	Gray to brown fine to coarse gravel with sand and silt (very dense, moist)			
35		8	91/10"		7							(1035)
40		8	63/9"		8							Stop drilling for groundwater; sample by others
									Hard drilling			
45		2	50/3"		9					11		(1220)
50		1	50/2"		10							Grab sample (1310)
55		0.5	50/1"		11							Grab sample (1410) Replace bit at 55'

LOG OF BORING GEI-7 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 7
Sheet 2 of 3

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
												End drilling at 1520

Boring completed at 58 feet
Perched groundwater encountered at 15 feet bgs

60

65

70

75

80

85

LOG OF BORING GEI-7 (continued)

GeoEngineers

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 7
Sheet 3 of 3

Date(s) Drilled	07/01/04 - 07/02/04	Logged By	RNM	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Direct Push	Sampling Methods	SPT/ D&M / Shelby
Auger Data	Tri-Cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	52	Surface Elevation (ft)	64	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
0							AC	2 inches asphalt concrete			Drilling times in parentheses
							GP	Gray fine gravel with occasional sand, trace silt (fill) (medium dense, moist)			
							ML	Brown silt (very stiff, moist)			
5		18	31	1					36	72	(1050)
10		16		2							(1115)
							GW	Gray to brown fine to coarse gravel with sand, trace silt (dense, moist)			Hard drilling at 13 feet; gravel cuttings
							ML	Brown sandy silt (stiff, moist)			
15		16	32	3					34		(1125)
							GP-GM	Gray to brown fine gravel with sand and silt (dense, moist)			
							GW-GM	Gray to brown fine to coarse gravel with sand and silt (very dense, moist)			Hard drilling
20		16	41	4			SP-SM	Gray to brown fine to medium sand with silt and occasional fine gravel (dense, moist)	17		(1150)
								Increasing gravel content			
							SW-SM	Gray to brown fine to coarse sand with gravel and silt (very dense, moist)			
25											

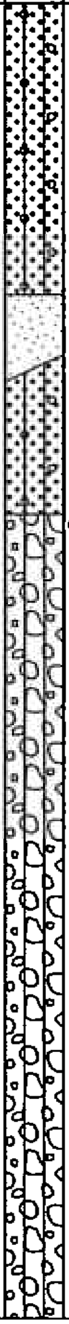
Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-8

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 8
Sheet 1 of 2



Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
25	▀	4	50/4"	5						(1210)
30	▀	17	81	6				18		(1240)
						SP	Gray fine to medium sand, trace silt (very dense, moist)			
						SW-SM	Gray fine to coarse sand with gravel and silt (very dense, moist)			
35	▀	4	50/5"	7						(1305) Hard drilling; Grab sample
						GW-GM	Gray to brown fine to coarse gravel with sand and silt (very dense, moist)			
40	▀	1	50/2"	8						(1410)
45	▀	0	50/1"	9						End of drilling 7/1/04 at 1610. Start drilling 07/2/04 at 0800
50	▀	0.5	50/0.5"	10						Hard drilling (0930) (1110)
55										

Boring completed at 52 feet
Groundwater not determined

LOG OF BORING GEI-8 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 8
Sheet 2 of 2

Date(s) Drilled	06/28/04 - 06/29/04	Logged By	RNM	Checked By	TWB
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Mud Rotary	Sampling Methods	SPT / D&M / Shelby
Auger Data	Tri-Cone	Hammer Data	140 lb hammer/ 30 in drop	Drilling Equipment	Mobile Drill B-53/B-57
Total Depth (ft)	70.1	Surface Elevation (ft)	63	Groundwater Elevation (ft)	Not Encountered
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES				Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number							
0							AC	2 inches asphalt concrete			Drilling times in parentheses (1325)
							GM	Brown silty gravel with sand (medium dense, moist) (fill)			
							ML	Brown silt (stiff, moist)			
5	■	18	21	1					41	80	
10	□	8		2							Pushed hard
15	■	9	66/9"	3			GW-GM	Gray to brown fine to coarse gravel with sand and silt (very dense, moist)	38	88	(1420)
20	■	12	41	4			SP-SM	Gray fine to medium sand with silt, occasional fine gravel (dense, moist)	17		(1500)
							SW-SM	Gray fine to coarse sand with silt, occasonal fine gravel (dense, moist)			
25											

Note: See Figure A-1 for explanation of symbols

LOG OF BORING GEI-9

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 9
Sheet 1 of 3

Depth feet	SAMPLES					Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
	Interval	Recovered (in)	Blows/foot	Sub-Sample	Sample Number							
25		7	83/11"		5					16		(1550)
30		12	85		6					17		End drilling 6/28/04 at 1635. Start drilling 6/29/04 at 0845
									Becomes very dense			
35		8	95/10"		7					18		
40		1	50/3"		8							(1000)
45		14	88/8"		9					21		(1025)
50		2	50/4"									
					10							(1050)
55		1	50/3"		11				Increasing gravel			(1130)

LOG OF BORING GEI-9 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 9
Sheet 2 of 3

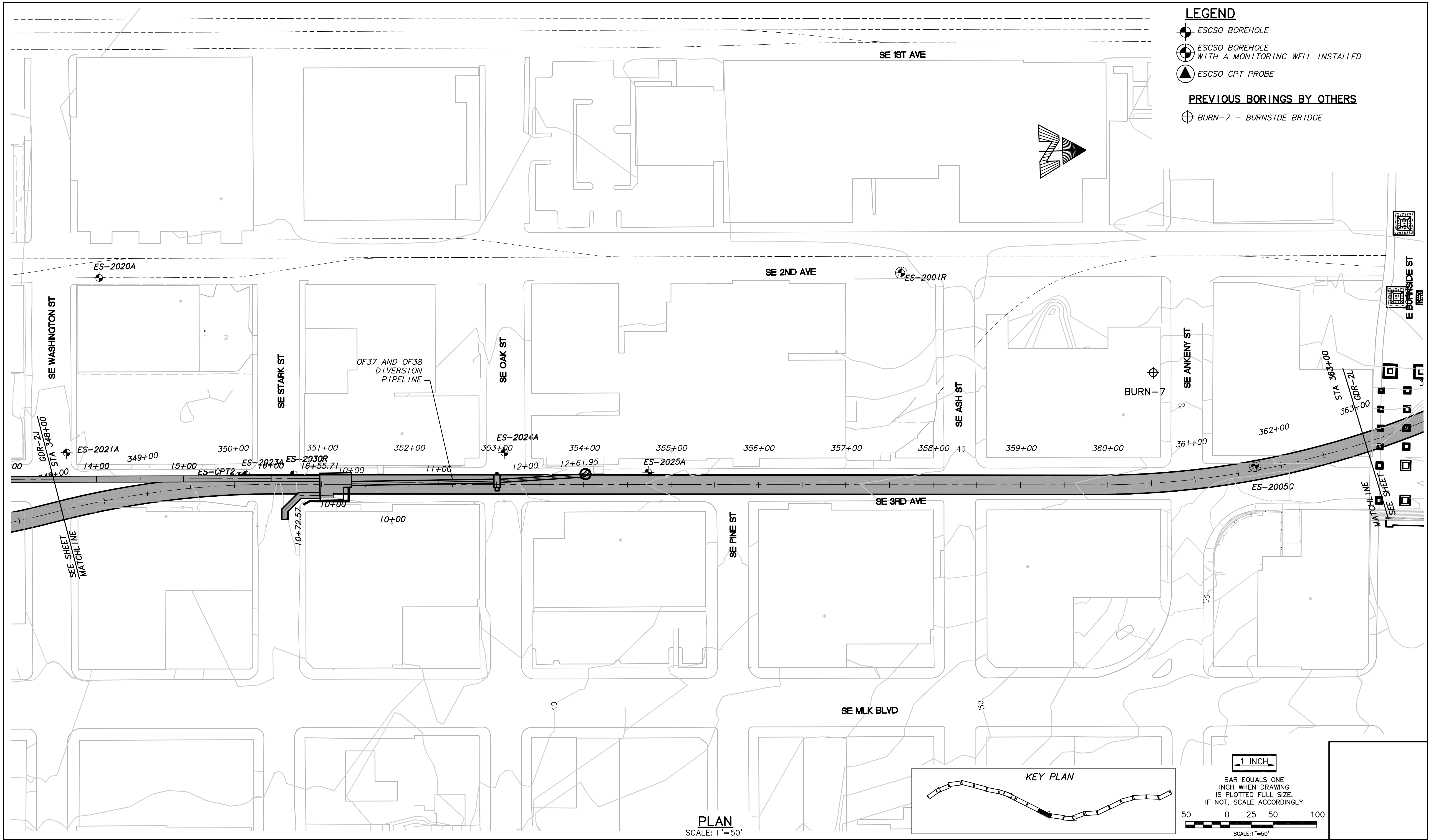
Depth feet	SAMPLES Interval	Recovered (In)	Blows/foot	Sub-Sample	Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft³	OTHER TESTS AND NOTES
60	▬	0										
			50/2"		12							(1220) Grab sample
								GM				
65	▬	0.5	50/1"	▬	13				Brown silty gravel with sand (very dense, moist)			(1310) Grab sample
												(1410) Grab sample
			50/1"		14							
70	▬	0.5							Boring completed at 70.1 feet Groundwater not determined			
75												
80												
85												

LOG OF BORING GEI-9 (continued)

GEOENGINEERS

Project: Portland Development Commission
Project Location: Portland, Oregon
Project Number: 1792-018-00

Figure: A- 9
Sheet 3 of 3



PLAN
SCALE: 1"=50'

NO.	DATE	DESCRIPTION	APPD.
	02/11/05	GDR	

REVISION

XREF(S) USED:
ROTATION ANGLE:
CONSTRUCTED BY
PROJECT COMPLETED
MAP CORRECTED BY CHECKED BY
FINAL MAP DATA

GDR-2K.DWG, 07/20/05 at 15:19

DESIGNED BY	TWP	DATE APPD.	
DRAWN BY	TAS	PROGRAM MGR	PTG
CHECKED BY	MLF	CONST. MGR.	GC
DESIGN MGR.	RFC	PROJECT MGR.	TRC

CITY OF PORTLAND
ENVIRONMENTAL SERVICES
WILLAMETTE RIVER CSO CONTROL PROJECT

PARSONS BRINCKERHOFF
400 S.W. Sixth Ave, Portland, OR 97204

EAST SIDE CSO PROJECT

FIGURE 2-K
BOREHOLE LOCATION PLAN

1/4 SECTION

JOB NO.
5516

SHEET NO.
GDR-2K
— OF —

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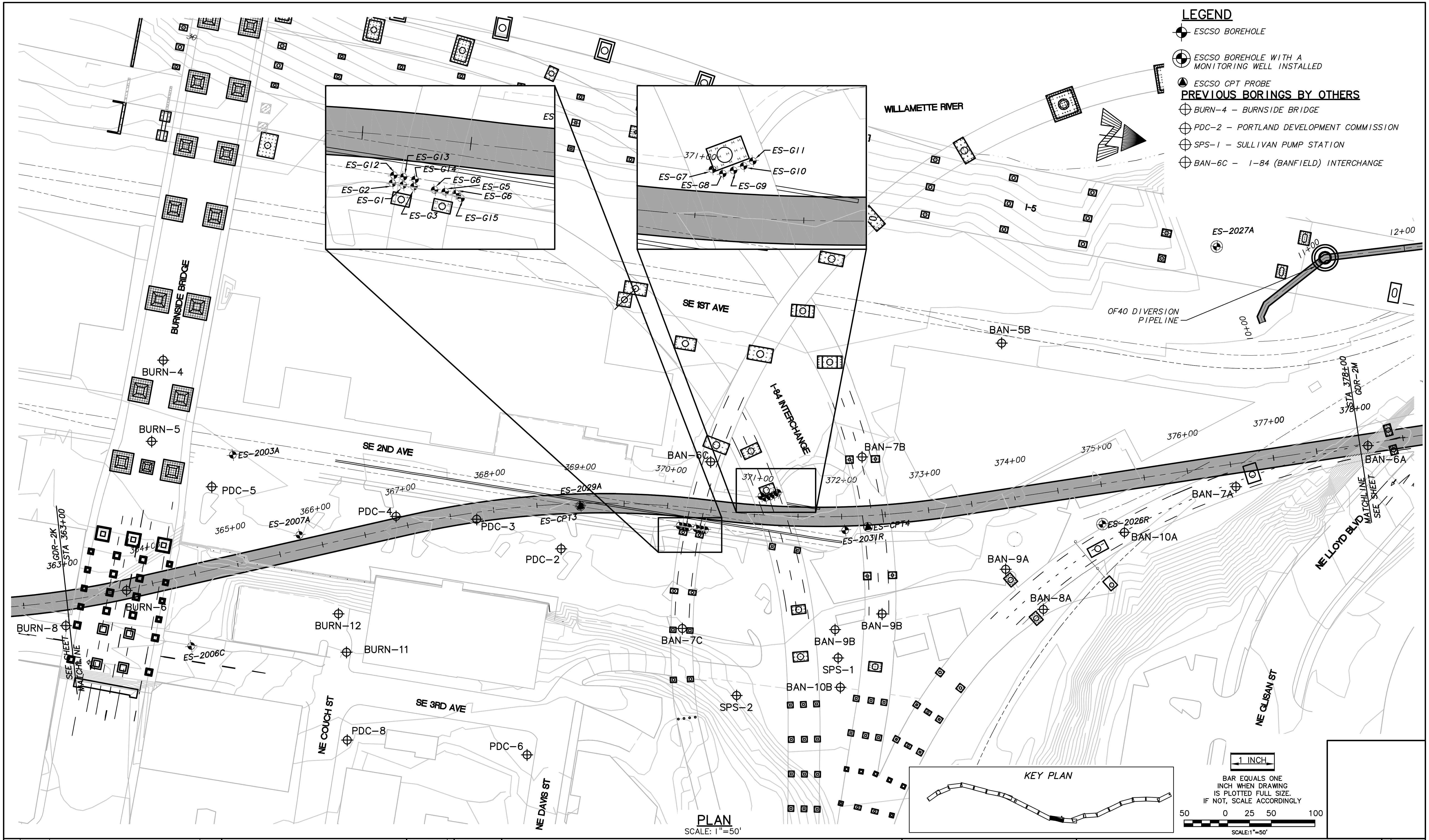


LEGEND

⊕ ESCSO BOREHOLE

⊕ ESCSO BOREHOLE WITH A MONITORING WELL INSTALLED

▲ ESCSO CPT PROBE

PREVIOUS BORINGS BY OTHERS

⊕ BURN-4 – BURNSIDE BRIDGE

⊕ PDC-2 – PORTLAND DEVELOPMENT COMMISSION

⊕ SPS-1 – SULLIVAN PUMP STATION

⊕ BAN-6C – I-84 (BANFIELD) INTERCHANGE

PLAN
SCALE: 1"=50'

1 INCH

BAR EQUALS ONE
INCH WHEN DRAWING
IS PLOTTED FULL SIZE.
IF NOT, SCALE ACCORDINGLY

50 0 25 50 100

SCALE:1"=50'

NO.	DATE	DESCRIPTION	APPD.
	07/15/05	GDR	

REVISION

XREF(S) USED:

ROTATION ANGLE:

CONSTRUCTED BY

PROJECT COMPLETED

MAP CORRECTED BY CHECKED BY

FINAL MAP DATA

GDR-2L.DWG, 07/20/05 at 15:20

DESIGNED BY	TWP	DATE APPD.	
DRAWN BY	TAS	PROGRAM MGR	PTG
CHECKED BY	MLF	CONST. MGR.	GC
DESIGN MGR.	RFC	PROJECT MGR.	TRC

CITY OF PORTLAND
ENVIRONMENTAL SERVICES
WILLAMETTE RIVER CSO TUNNEL PROGRAM

PARSONS BRINCKERHOFF
400 S.W. Sixth Ave, Portland, OR 97204

EAST SIDE CSO TUNNEL PROJECT

FIGURE 2-L
BOREHOLE LOCATION PLAN

1/4 SECTION

JOB NO.
5516

SHEET NO.
GDR-2L
— OF —

K:\CADD\MISC\TOM-P\GDR-011405\GDR-2L.DWG, Layout1, 7/20/2005 3:20:33 PM

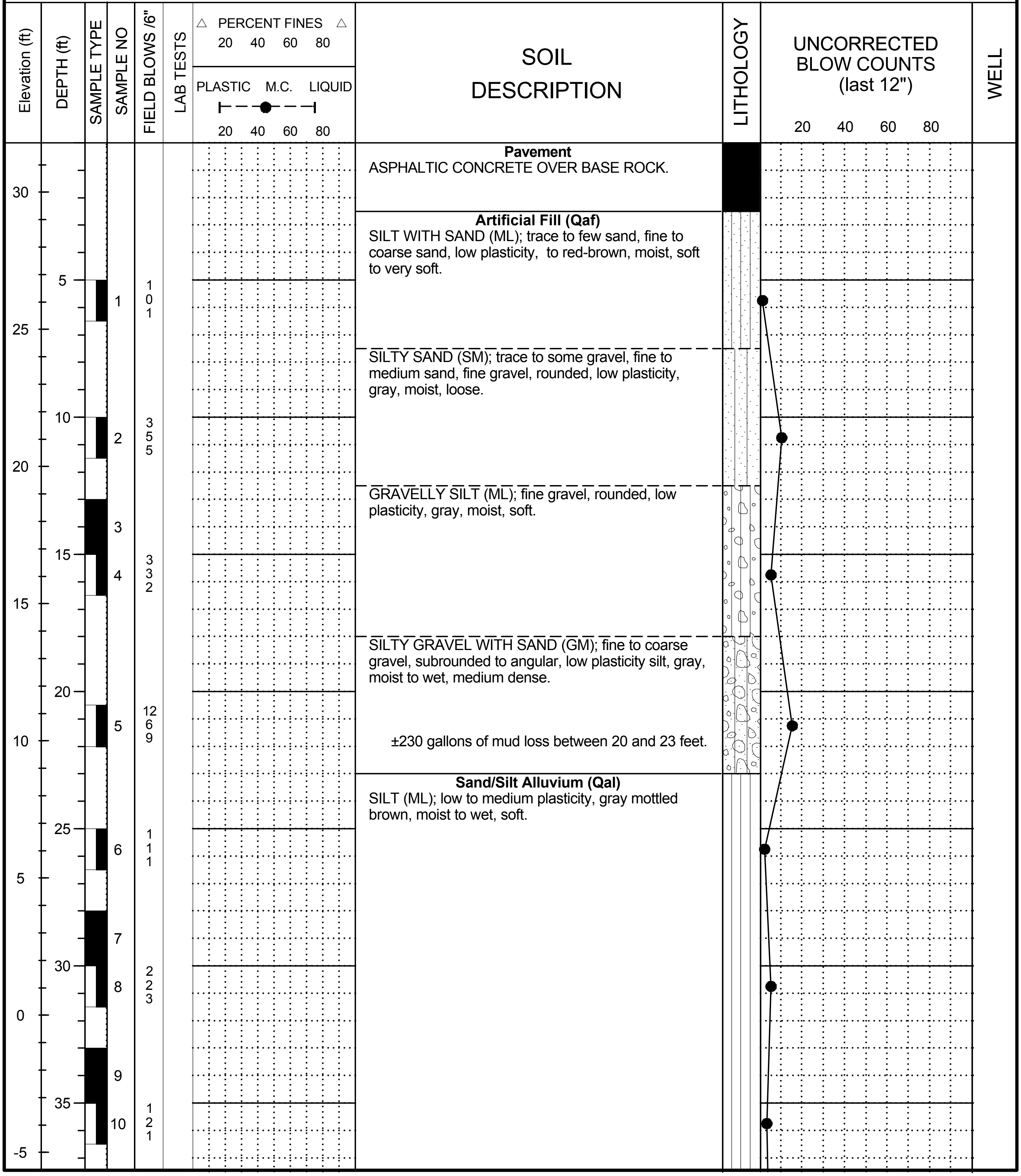
PARSONS BRINCKERHOFF

BORING LOG
ES-2003A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 1 OF 4
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 31.8 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684338.1 EASTING 7647623.2
DRILLING METHOD Mud Rotary
DATE DRILLED 9/9/03 LOGGED BY AR
LOCATION NE 2nd & Burnside
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample



SAMPLE NO	FIELD BLOWS /6"
1	1 0 1
2	3 5 5
3	
4	3 3 2
5	12 6 9
6	1 1 1
7	
8	2 2 3
9	
10	1 2 1

Pavement
ASPHALTIC CONCRETE OVER BASE ROCK.

Artificial Fill (Qaf)
SILT WITH SAND (ML); trace to few sand, fine to coarse sand, low plasticity, to red-brown, moist, soft to very soft.

SILTY SAND (SM); trace to some gravel, fine to medium sand, fine gravel, rounded, low plasticity, gray, moist, loose.

GRAVELLY SILT (ML); fine gravel, rounded, low plasticity, gray, moist, soft.

SILTY GRAVEL WITH SAND (GM); fine to coarse gravel, subrounded to angular, low plasticity silt, gray, moist to wet, medium dense.

±230 gallons of mud loss between 20 and 23 feet.

Sand/Silt Alluvium (Qal)
SILT (ML); low to medium plasticity, gray mottled brown, moist to wet, soft.

ESCSO MUD 2005 2032011.GPJ 2042034-REV1.GPJ 7/1/05

PARSONS BRINCKERHOFF

BORING LOG
ES-2003A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
CITY Portland, Oregon
PROJECT NO. 2032011
DATE DRILLED 9/9/03 LOGGED BY AR
SHEET 2 OF 4
SURF. EL. 31.8 ft STA. OFST.
NORTHING 684338.1 EASTING 7647623.2
LOCATION NE 2nd & Burnside
INITIAL GWL@ Not Available
EQUIPMENT CME 75
DRILLING METHOD Mud Rotary
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

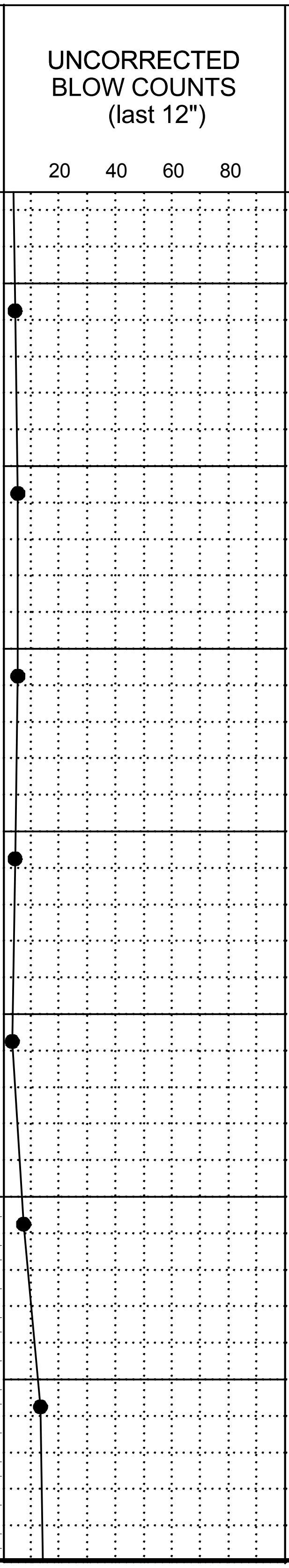
SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube G Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	SOIL DESCRIPTION
	37					**(Qal Cont'd)** SILT (ML); low to medium plasticity, gray mottled brown, moist to wet, soft.
	40	◨	11	2 2 2		
-10						
	45	◨	12	3 2 3		
-15						
	50	◨	13	10 2 3		
-20						
	55	◨	14	6 2 2		
-25						
		■	15			
	60	◨	16	1 0 3		
-30						
	65	◨	17	6 3 4		INTERBEDDED LAYERS OF SANDY SILT (ML) to SILT WITH SAND (ML); trace organics, fine sand, nonplastic to low plasticity silt, gray, wet, medium stiff to stiff.
-35						
	70	☒	18	7 6 7		
-40						

△ PERCENT FINES △
20 40 60 80

PLASTIC M.C. LIQUID
20 40 60 80

LITHOLOGY



WELL

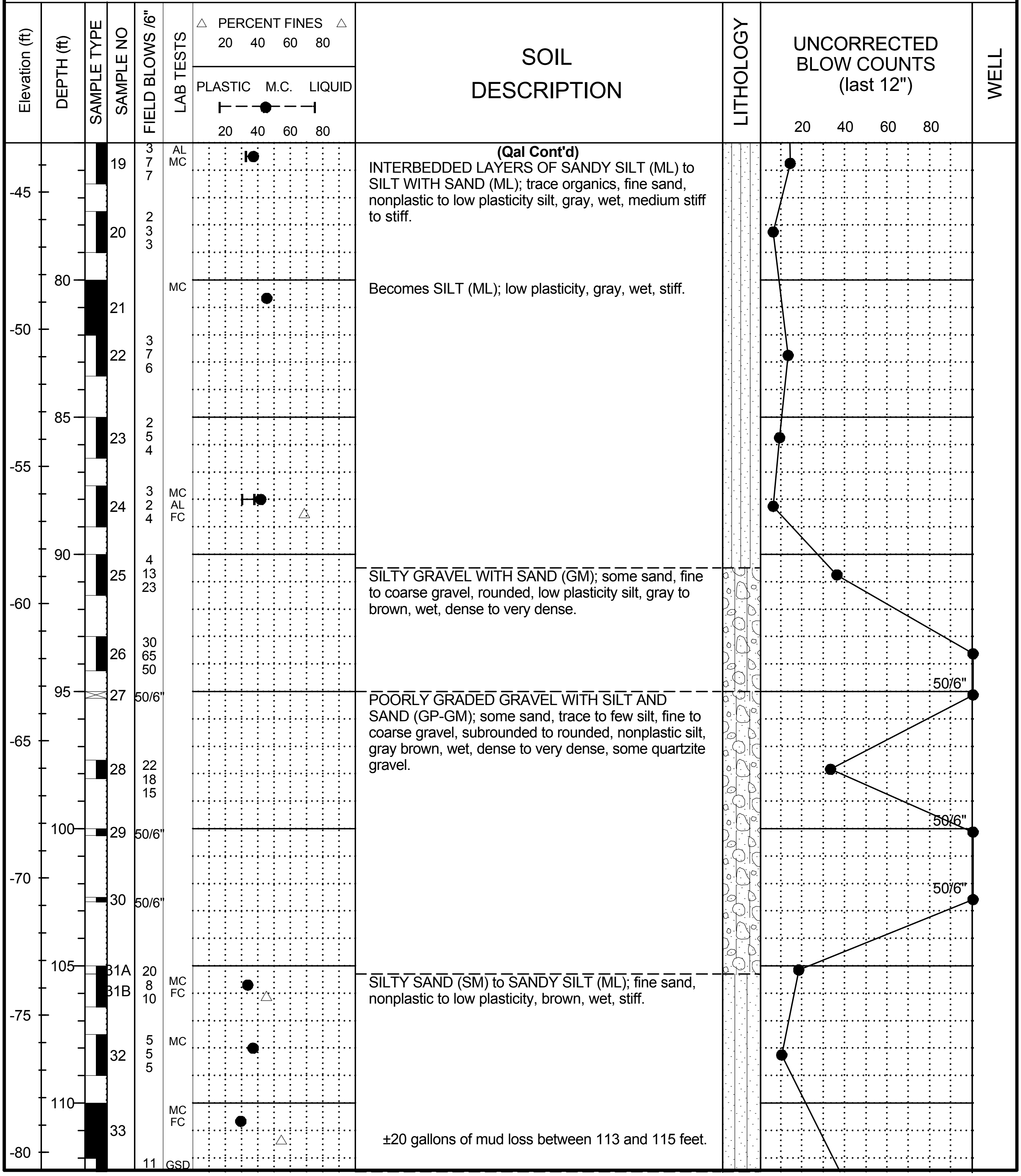
PARSONS BRINCKERHOFF 100 YEARS

BORING LOG
ES-2003A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 3 OF 4
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 31.8 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684338.1 EASTING 7647623.2
DRILLING METHOD Mud Rotary
DATE DRILLED 9/9/03 LOGGED BY AR
LOCATION NE 2nd & Burnside
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample



Sample No	Field Blows /6"	Lab Tests
19	3 7 7	AL MC
20	2 3 3	
21		MC
22	3 7 6	
23	2 5 4	
24	3 2 4	MC AL FC
25	4 13 23	
26	30 65 50	
27	50/6"	
28	22 18 15	
29	50/6"	
30	50/6"	
31A	20	
31B	8 10	MC FC
32	5 5 5	MC
33		MC FC
	11	GSD

(Qal Cont'd)

INTERBEDDED LAYERS OF SANDY SILT (ML) to SILT WITH SAND (ML); trace organics, fine sand, nonplastic to low plasticity silt, gray, wet, medium stiff to stiff.

Becomes SILT (ML); low plasticity, gray, wet, stiff.

SILTY GRAVEL WITH SAND (GM); some sand, fine to coarse gravel, rounded, low plasticity silt, gray to brown, wet, dense to very dense.

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); some sand, trace to few silt, fine to coarse gravel, subrounded to rounded, nonplastic silt, gray brown, wet, dense to very dense, some quartzite gravel.

SILTY SAND (SM) to SANDY SILT (ML); fine sand, nonplastic to low plasticity, brown, wet, stiff.

±20 gallons of mud loss between 113 and 115 feet.

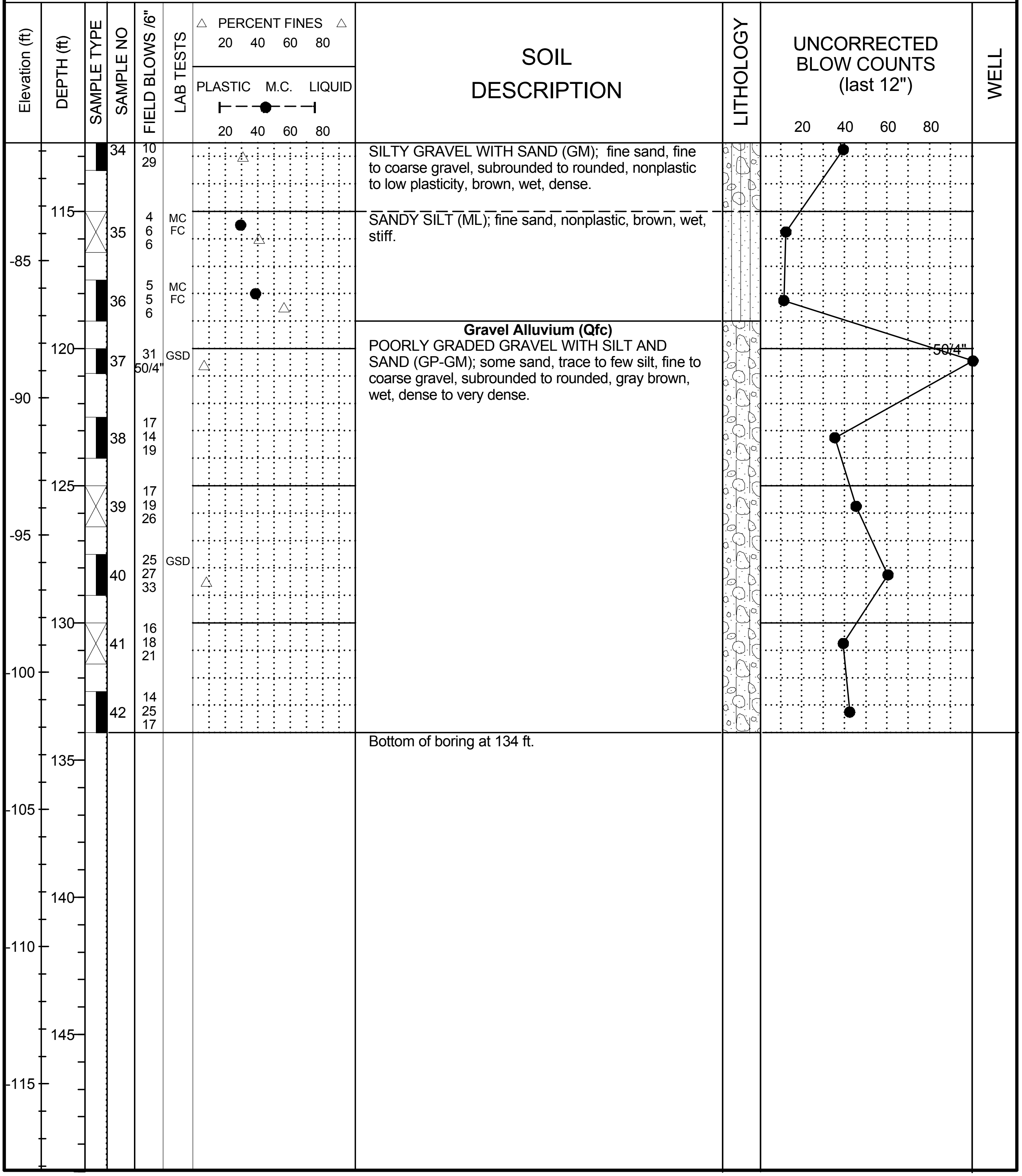
PARSONS BRINCKERHOFF

BORING LOG
ES-2003A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 4 OF 4
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 31.8 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684338.1 EASTING 7647623.2
DRILLING METHOD Mud Rotary
DATE DRILLED 9/9/03 LOGGED BY AR
LOCATION NE 2nd & Burnside
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample



SAMPLE NO	FIELD BLOWS /6"	LAB TESTS
34	10 29	
35	4 6 6	MC FC
36	5 5 6	MC FC
37	31 50/4"	GSD
38	17 14 19	
39	17 19 26	
40	25 27 33	GSD
41	16 18 21	
42	14 25 17	

SOIL DESCRIPTION

SILTY GRAVEL WITH SAND (GM); fine sand, fine to coarse gravel, subrounded to rounded, nonplastic to low plasticity, brown, wet, dense.

SANDY SILT (ML); fine sand, nonplastic, brown, wet, stiff.

Gravel Alluvium (Qfc)

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); some sand, trace to few silt, fine to coarse gravel, subrounded to rounded, gray brown, wet, dense to very dense.

Bottom of boring at 134 ft.

BORING LOG
ES-2005C

PARSONS BRINCKERHOFF 100 YEARS

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 1 OF 4
INITIAL GWL@ 28.8 ft ()
CITY Portland, Oregon
SURF. EL. 43.6 ft STA. OFST.
EQUIPMENT CME/B-59
PROJECT NO. 2032011
NORTHING 684044 EASTING 7648465.5
DRILLING METHOD Mud Rotary
DATE DRILLED 2/2/04 LOGGED BY AR/NMC
LOCATION SE 3rd & Ankeny
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS
40					
	5	◨	1	3 0 50	
35					
	10	◨	2	3 2 3	
30					
	15	◨	3	17 19 27	
25					
	20	◨	4	28 26 28	
20					
	25	◨	5	50	
15 ▼					
	30	☒	6	50	
10					
	35	◨	7	50	

△ PERCENT FINES △ 20 40 60 80

PLASTIC M.C. LIQUID 20 40 60 80

SOIL DESCRIPTION

Pavement
ASPHALTIC CONCRETE OVER CONCRETE.

Artificial Fill (Qaf)
POORLY GRADED GRAVEL (GP); loose.

SILTY GRAVEL (GM) to GRAVELLY SILT (ML); fine to coarse gravel, angular, low plasticity, gray to brown, moist, loose gravel, soft silt.

CONCRETE RUBBLE

GRAVELLY SILT (ML)

SILT (ML); low plasticity, light brown, moist, medium stiff.

SILTY GRAVEL (GM)

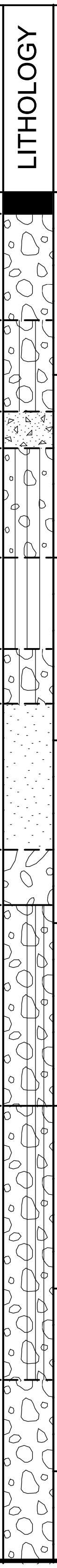
POORLY GRADED SAND (SP); trace to few gravel, trace silt, fine sand, fine to coarse gravel, angular, light brown, moist, dense.

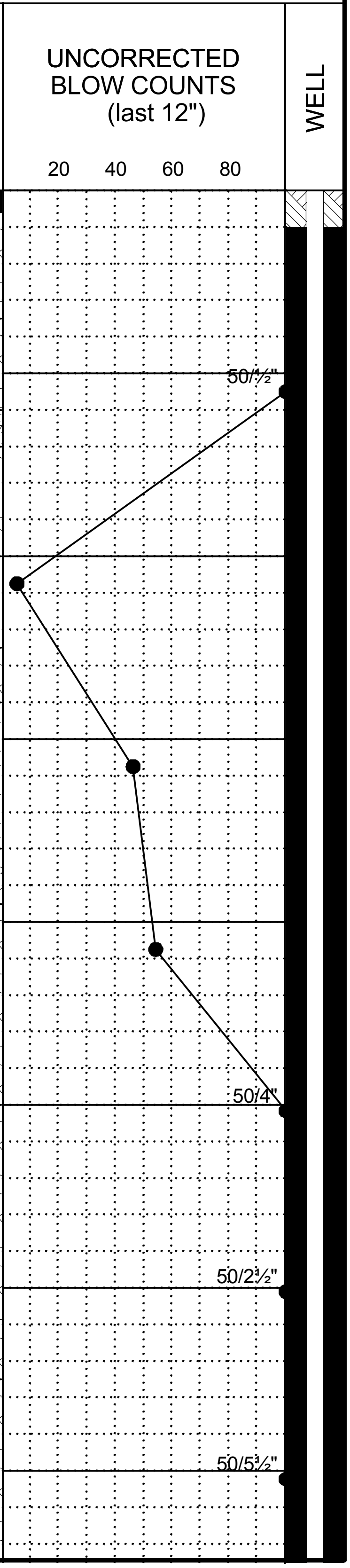
BOULDER at 18.0 to 19.5 feet.

Gravel Alluvium (Qfc)
POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity, gray brown, wet, very dense.

Troutdale Formation (Tt)
POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, gray brown, wet, very dense.

POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse gravel, subrounded, nonplastic, gray gravel, light brown sand, wet, very dense.





PROJECT East Side CSO - Final Design
SHEET 2 OF 4
INITIAL GWL@ 28.8 ft ()
CITY Portland, Oregon
SURF. EL. 43.6 ft STA. OFST.
EQUIPMENT CME/B-59
PROJECT NO. 2032011
NORTHING 684044 EASTING 7648465.5
DRILLING METHOD Mud Rotary
DATE DRILLED 2/2/04 LOGGED BY AR/NMC
LOCATION SE 3rd & Ankeny
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
5							**(Tt Cont'd)** POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, gray gravel, light brown sand, wet, very dense, some quartzite.			
	40	◧	8	50/3"					50/3"	
0										
	45	◧	9	50/3"					50/2½"	
-5										
	50	◧	10	50/3"			With interbedded layers of POORLY GRADED GRAVEL WITH SAND (GP)		50/3½"	
-10										
	55	☒	11	50/3"					50/3"	
-15										
	60		12	50/3"					50/½"	
-20										
	65		13	50/3"					50/½"	
-25										
	70		14	50/3"			Driller indicates formation becomes more cemented.		50/½"	
							Matrix becomes low to medium plasticity.			
-30										

PARSONS BRINCKERHOFF 100 YEARS

BORING LOG
ES-2005C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 3 OF 4
INITIAL GWL@ 28.8 ft ()
CITY Portland, Oregon
SURF. EL. 43.6 ft STA. OFST.
EQUIPMENT CME/B-59
PROJECT NO. 2032011
NORTHING 684044 EASTING 7648465.5
DRILLING METHOD Mud Rotary
DATE DRILLED 2/2/04 LOGGED BY AR/NMC
LOCATION SE 3rd & Ankeny
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dro

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

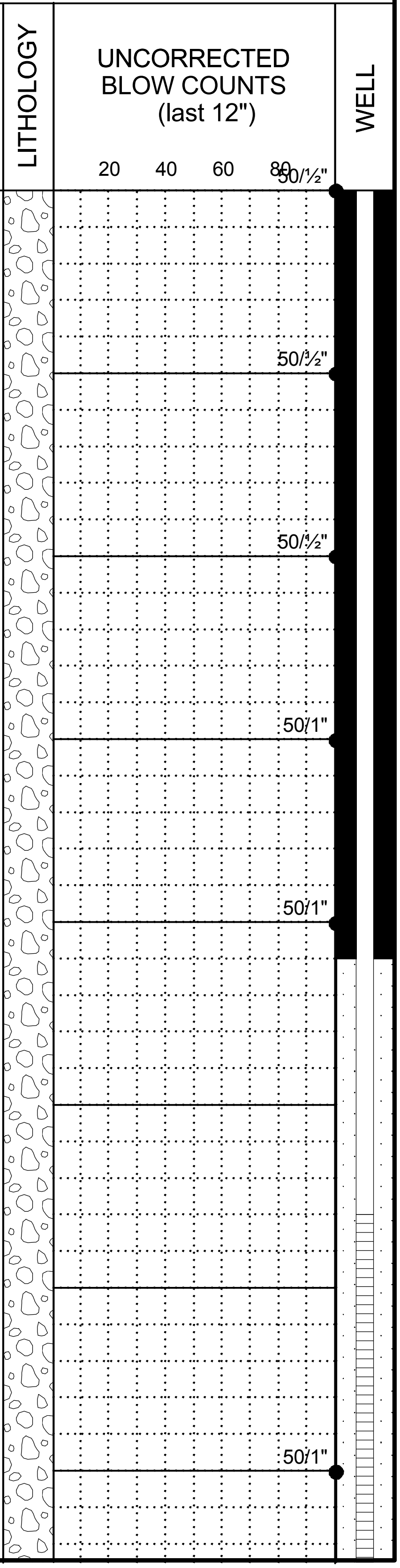
Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	UNCORRECTED BLOW COUNTS (last 12")
			15	50/3"		50/½"
-35						
	80		16	50/3"		50/½"
-40						
	85		17	50/3"		50/½"
-45						
	90		18	50/3"		50/1"
-50						
	95		19	50/3"		50/1"
-55						
	100					
-60						
	105					
-65						
	110		22	50/3"		50/1"

△ PERCENT FINES △ 20 40 60 80

PLASTIC M.C. LIQUID 20 40 60 80

SOIL DESCRIPTION

(Tt Cont'd)
POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, gray gravel, light brown sand, low to medium plasticity, wet, very dense.



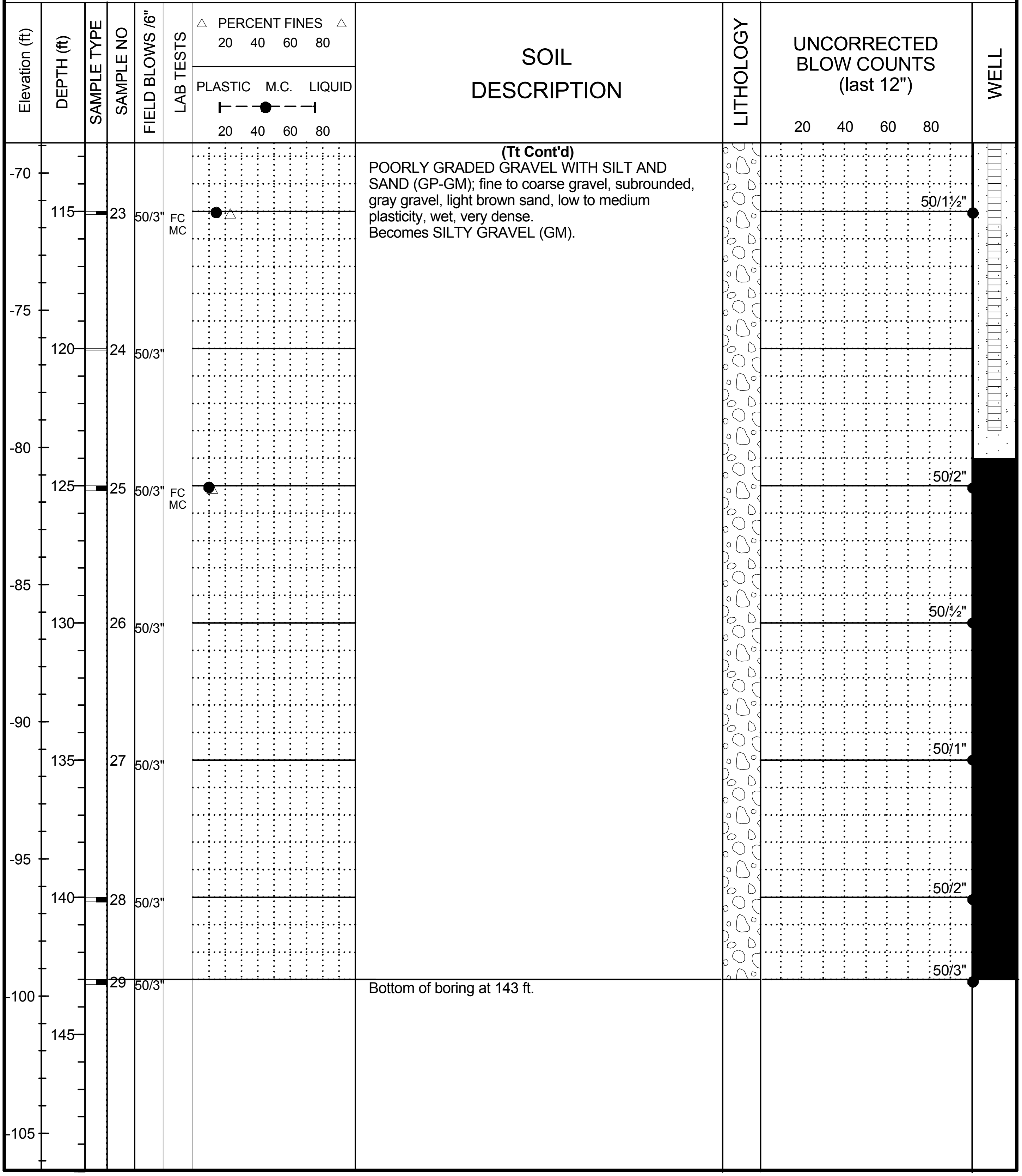
PARSONS BRINCKERHOFF

BORING LOG
ES-2005C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 4 OF 4
INITIAL GWL@ 28.8 ft ()
CITY Portland, Oregon
SURF. EL. 43.6 ft STA. OFST.
EQUIPMENT CME/B-59
PROJECT NO. 2032011
NORTHING 684044 EASTING 7648465.5
DRILLING METHOD Mud Rotary
DATE DRILLED 2/2/04 LOGGED BY AR/NMC
LOCATION SE 3rd & Ankeny
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample



DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	UNCORRECTED BLOW COUNTS (last 12")
115	23	50/3"	FC MC	50/1½"
120	24	50/3"		
125	25	50/3"	FC MC	50/2"
130	26	50/3"		50/½"
135	27	50/3"		50/1"
140	28	50/3"		50/2"
143	29	50/3"		50/3"

SOIL DESCRIPTION

(Tt Cont'd)

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, gray gravel, light brown sand, low to medium plasticity, wet, very dense.

Becomes SILTY GRAVEL (GM).

Bottom of boring at 143 ft.

PARSONS BRINCKERHOFF 100 YEARS

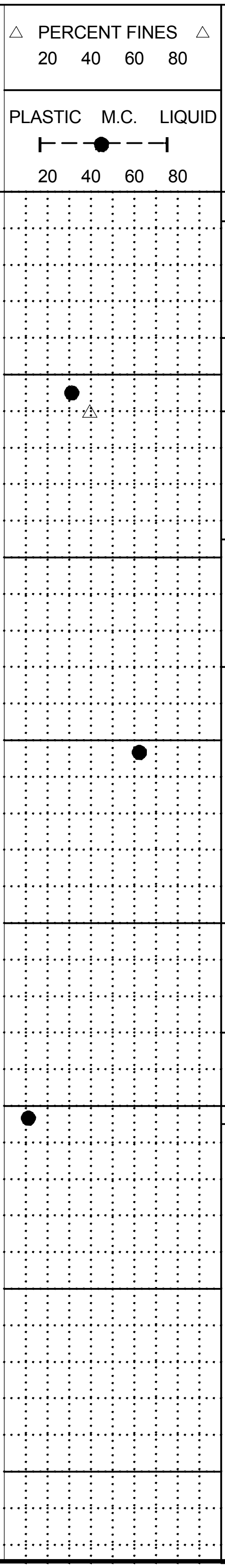
BORING LOG
ES-2006C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 1 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 47.7 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684319.2 EASTING 7648477
DRILLING METHOD Mud Rotary
DATE DRILLED 3/12/04 LOGGED BY AR
LOCATION SE 3rd & Burnside
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube G Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS
45					
	5	◧	1	2 4 14	FC MC
40					
	10	◧	2	50	MC
35					
	15	◧	3	12 12 16	MC
30					
	20	◧	4	50	
25					
	25	◧	5	50	MC
20					
	30	◧	6	50	
15					
	35	◧	7	50	



SOIL DESCRIPTION

Pavement

ASPHALTIC CONCRETE OVER BASE ROCK.

Artificial Fill (Qaf)

SILT WITH SAND AND COBBLES (ML); soft to medium stiff.

SILT (ML) to SILTY SAND (SM); fine sand, nonplastic, brown, loose.

SILTY GRAVEL WITH SAND (GM); fine to coarse gravel, angular, low plasticity silt, gray brown, moist, medium dense.

POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse gravel, angular to subrounded, nonplastic, gray, very dense.

POORLY GRADED SAND (SP); trace silt, fine sand, light brown, moist, medium dense.

Gravel Alluvium (Qfc)

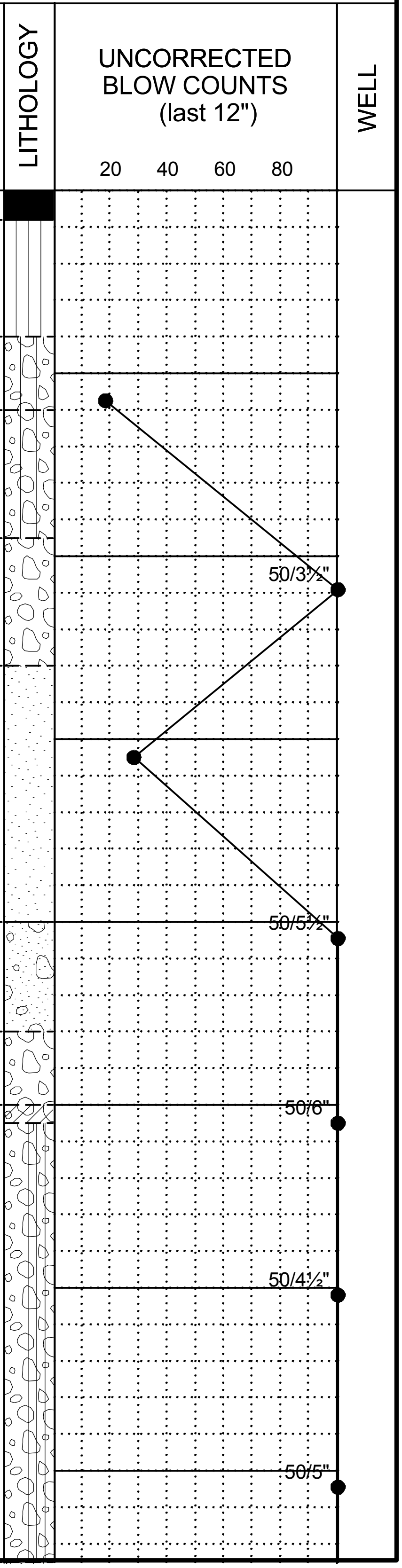
POORLY GRADED SAND WITH GRAVEL (SP); fine to coarse gravel, subrounded to subangular, light brown, very dense.

POORLY GRADED GRAVEL WITH SAND (GP); trace silt.

CLAYEY GRAVEL (GC).

Gravel Alluvium (Qfc)

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity to nonplastic silt, gray brown, moist to wet, very dense.



PARSONS BRINCKERHOFF

BORING LOG
ES-2006C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 2 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 47.7 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684319.2 EASTING 7648477
DRILLING METHOD Mud Rotary
DATE DRILLED 3/12/04 LOGGED BY AR
LOCATION SE 3rd & Burnside
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube G Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
10							**(Qfc Cont'd)** POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, nonplastic silt, gray to brown, wet, very dense.			
	40	◨	8	50					50/4"	
5										
	45	◨	9	50			**Troutdale Formation (Tt)** POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, nonplastic, gray to brown, wet, very dense.		50/3"	
0										
	50	◨	10	50					50/2½"	
-5										
	55	◨	11	50					50/3	
-10										
	60	◨	12	50			Becomes with some quartzite.		50/2½"	
-15										
	65	◨	13	50					50/½"	
-20										
	70	◨	14	50	GSD	△	Becomes with low plasticity fines, gray to yellow brown, some moderately weathered gravels.		50/3½"	
-25										

PARSONS BRINCKERHOFF

BORING LOG
ES-2006C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
CITY Portland, Oregon
PROJECT NO. 2032011
DATE DRILLED 3/12/04 LOGGED BY AR
SHEET 3 OF 5
SURF. EL. 47.7 ft STA. OFST.
NORTHING 684319.2 EASTING 7648477
LOCATION SE 3rd & Burnside
INITIAL GWL@ Not Available
EQUIPMENT CME 75
DRILLING METHOD Mud Rotary
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dro

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	UNCORRECTED BLOW COUNTS (last 12")
15	50		50/2"
16	50	GSD	50/5"
17	50		50/2"
18	50		50/3"
19	50	GSD	50/4"
20	50		50/1½
21	50	GSD	50/1"
22	50		50/2½"

Elevation (ft): -30, -35, -40, -45, -50, -55, -60
DEPTH (ft): 80, 85, 90, 95, 100, 105, 110

SOIL DESCRIPTION

(Tt Cont'd)
POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, nonplastic to low plasticity silt, yellow brown to gray, wet, very dense, some quartzite, some slightly to moderately weathered gravels.

Sand lens at 95.4 to 96.5 feet.

Sand lens at 97 to 97.5 feet.

Interbedded sand lenses from 105 to 110 feet.

±7-inch diameter cobble at 112 feet.

BORING LOG
ES-2006C

PROJECT East Side CSO - Final Design

SHEET 4 OF 5

INITIAL GWL@ Not Available

CITY Portland, Oregon

SURF. EL. 47.7 ft STA. OFST.

EQUIPMENT CME 75

PROJECT NO. 2032011

NORTHING 684319.2 EASTING 7648477

DRILLING METHOD Mud Rotary

DATE DRILLED 3/12/04 LOGGED BY AR

LOCATION SE 3rd & Burnside

HAMMER SYS. 140 + 300 lb. hammer, 30 inch dro

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
-65						**(Tt Cont'd)** POORLY GRADED GRAVEL WITH SILT, SAND AND COBBLES (GP-GM); trace to few cobbles, fine to coarse gravel, subrounded, low plasticity silt, olive gray to black, wet, very dense, some slightly weathered gravels.			
	115	23	50					50/4½"	
-70									
	120	24	50					50/3"	
-75									
	125	25	50					50/1"	
-80									
	130	26	50					50/1"	
-85									
	135	27	50					50/2"	
-90									
	140	28	50					50/2"	
-95									
	145	29	50					50/1"	
-100									

PARSONS BRINCKERHOFF

BORING LOG
ES-2006C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 5 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 47.7 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684319.2 EASTING 7648477
DRILLING METHOD Mud Rotary
DATE DRILLED 3/12/04 LOGGED BY AR
LOCATION SE 3rd & Burnside
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	△ PERCENT FINES △ 20 40 60 80 / PLASTIC M.C. LIQUID 20 40 60 80	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80	WELL
		■	30	50			Bottom of boring at 150.2 ft.		50/2½" ●	
-105										
	155									
-110										
	160									
-115										
	165									
-120										
	170									
-125										
	175									
-130										
	180									
-135										
	185									

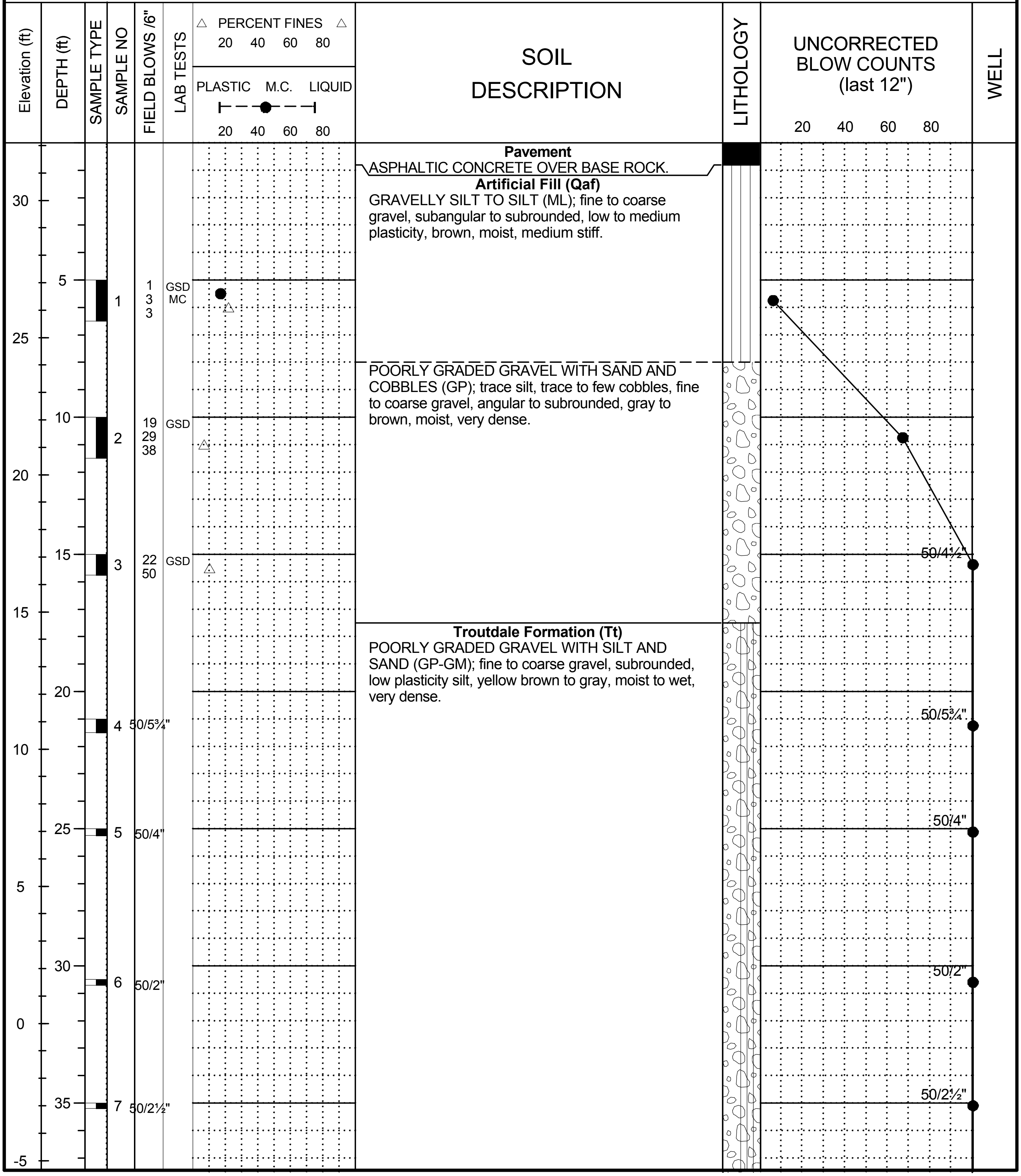
PARSONS BRINCKERHOFF

BORING LOG
ES-2007A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 1 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 32.1 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684425.8 EASTING 7648334.6
DRILLING METHOD Mud Rotary
DATE DRILLED 3/3/04 LOGGED BY AR
LOCATION NE 2nd & Burnside (Parking Lot)
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample



SAMPLE NO	FIELD BLOWS /6"	LAB TESTS
1	1 3 3	GSD MC
2	19 29 38	GSD
3	22 50	GSD
4	50/5¾"	
5	50/4"	
6	50/2"	
7	50/2½"	

SOIL DESCRIPTION

Pavement

ASPHALTIC CONCRETE OVER BASE ROCK.

Artificial Fill (Qaf)

GRAVELLY SILT TO SILT (ML); fine to coarse gravel, subangular to subrounded, low to medium plasticity, brown, moist, medium stiff.

POORLY GRADED GRAVEL WITH SAND AND COBBLES (GP); trace silt, trace to few cobbles, fine to coarse gravel, angular to subrounded, gray to brown, moist, very dense.

Troutdale Formation (Tt)

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, yellow brown to gray, moist to wet, very dense.

PARSONS BRINCKERHOFF

BORING LOG
ES-2007A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 2 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 32.1 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684425.8 EASTING 7648334.6
DRILLING METHOD Mud Rotary
DATE DRILLED 3/3/04 LOGGED BY AR
LOCATION NE 2nd & Burnside (Parking Lot)
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	SOIL DESCRIPTION	UNCORRECTED BLOW COUNTS (last 12")
						(Tt Cont'd) POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, yellow brown to gray, moist to wet, very dense.	
	40	◨	8	50/2"			50/2"
-10							
	45	■	9	50/5"	GSD MC	POORLY GRADED SAND WITH SILT (SP-SM); fine sand, yellow brown, wet, very dense.	50/5"
-15							
						POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, gray to yellow brown, wet, very dense.	
	50	◨	10	50/3"			50/3"
-20							
	55	◨	11	50/3½"			50/3½"
						Sand lens at 56 to 57 feet.	
-25							
						Sand lens at 58 to 59 feet.	
						Becomes with some slightly to moderately weathered gravel, some quartzite.	
	60	◨	12	50/5"	GSD		50/5"
-30							
						Slight caving from 63 to 65 feet.	
	65	☒	13	50/2"			50/2"
-35						Sand lens at 67.5 to 69.5 feet.	
	70	◨	14	50/3"			50/3"
-40							

PARSONS BRINCKERHOFF

BORING LOG
ES-2007A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 3 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 32.1 ft STA. ______ OFST. ______
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684425.8 EASTING 7648334.6
DRILLING METHOD Mud Rotary
DATE DRILLED 3/3/04 LOGGED BY AR
LOCATION NE 2nd & Burnside (Parking Lot)
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	UNCORRECTED BLOW COUNTS (last 12")
		15	50/2"		50/2"
-45					
	80	16	50/3"		50/3"
-50					
	85	17	50/½"		50/½"
-55					
	90	18	50/5"	GSD	50/5"
-60					
	95	19	50/4"	GSD	50/4"
-65					
	100	20	50/3½"		50/3½"
-70					
	105	21	50/2"		50/2"
-75					
	110	22	50/½"		50/½"
-80					

△ PERCENT FINES △ 20 40 60 80

PLASTIC M.C. LIQUID 20 40 60 80

UNCORRECTED BLOW COUNTS (last 12") 20 40 60 80

SOIL DESCRIPTION

(Tt Cont'd)

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, gray to yellow brown, wet, very dense, some slightly to moderately weathered gravel, some quartzite.

POORLY GRADED SAND WITH SILT (SP-SM); trace gravel, olive gray, wet, very dense.

POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, olive gray, wet, very dense, some quartzite.

PARSONS BRINCKERHOFF

BORING LOG
ES-2007A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 4 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 32.1 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684425.8 EASTING 7648334.6
DRILLING METHOD Mud Rotary
DATE DRILLED 3/3/04 LOGGED BY AR
LOCATION NE 2nd & Burnside (Parking Lot)
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◧ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	SOIL DESCRIPTION	UNCORRECTED BLOW COUNTS (last 12")
					(Tt Cont'd) POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, olive gray, wet, very dense, some quartzite.	
	115	23	50/2½"	GSD		50/2½"
-85						
	120	24	50/2"			50/2"
-90						
	125	25	50/2"			50/2"
-95						
	130	26	50/3"			50/3"
-100						
					POORLY GRADED SAND WITH SILT (SP-SM).	
	135	27	50/4"	GSD	POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, nonplastic silt, gray to yellow brown, wet, very dense.	50/4"
-105						
	140	28	50/2"			50/2"
-110						
	145	29	50/2"			50/2"
-115						

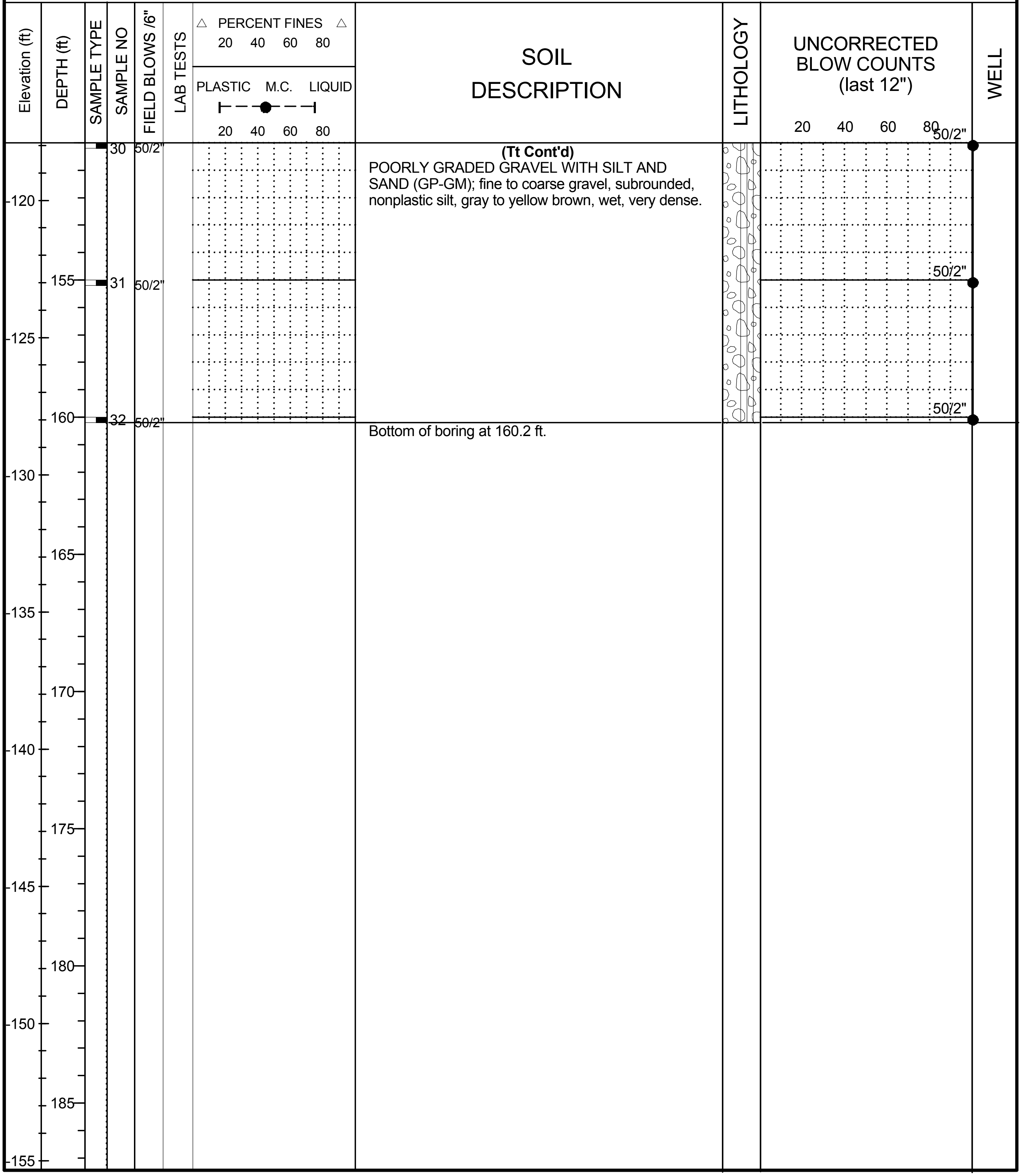
BORING LOG
ES-2007A

PARSONS BRINCKERHOFF 100 YEARS

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design
SHEET 5 OF 5
INITIAL GWL@ Not Available
CITY Portland, Oregon
SURF. EL. 32.1 ft STA. OFST.
EQUIPMENT CME 75
PROJECT NO. 2032011
NORTHING 684425.8 EASTING 7648334.6
DRILLING METHOD Mud Rotary
DATE DRILLED 3/3/04 LOGGED BY AR
LOCATION NE 2nd & Burnside (Parking Lot)
HAMMER SYS. 140 + 300 lb. hammer, 30 inch dr

SAMPLE TYPE ☒ Ring (3.25" OD) ◨ Standard Penetration Test (2" OD) ■ Shelby Tube Ⓖ Grab Sample



(Tt Cont'd)
POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, nonplastic silt, gray to yellow brown, wet, very dense.

Bottom of boring at 160.2 ft.

Appendix B

Previous Drilling Explorations

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B.1 General B-1
B.2 Drilling B-1
B.3 Sampling B-1
  B.3.1 Disturbed Sampling B-1
  B.3.2 Undisturbed Sampling B-2
B.4 Borehole Abandonment B-2
B.5 Materials Description B-3
B.6 Drill Logs B-3

Figures

Figure B-1: Drill Log, B-1
Figure B-2: Drill Log, B-2
Figure B-3: Drill Log, B-3

APPENDIX B: PREVIOUS DRILLING EXPLORATIONS

B.1 GENERAL

Shannon & Wilson, Inc., explored subsurface conditions at the project site during the previous phase of the project with a total of three geotechnical borings, designated B-1, B-2, and B-3. Borings B-1 and B-3 were drilled on land, and boring B-2 was drilled in the Willamette River from a floating barge. Completed borehole locations were measured in the field relative to existing site features and with a hand-held GPS unit (Geo 7X H-Star) capable of decimeter-level accuracy. Approximate borehole coordinates (OR83-NIF) and elevations (NAVD88) are presented on the drill logs in this appendix. Approximate borehole locations are also shown graphically on the Site and Exploration Plan, Figure 2. This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered during drilling.

B.2 DRILLING

The geotechnical borings were drilled between September 19, and October 25, 2016, using a truck-mounted CME-75 drill rig that was provided and operated by Western States Soil Conservation, Inc. (Western States), of Hubbard, Oregon. The on-land borings (B-1 and B-3) were advanced to depths of 221.5 and 230.3 feet below the existing ground surface using open-hole, mud rotary drilling techniques. The in-water boring (B-2) was drilled in the Willamette River to a depth of 148.2 feet below mudline using open-hole, mud rotary drilling techniques through a 5-inch diameter circulation casing. The in-water boring was drilled from a floating barge that was provided and operated by Mark Marine Service, Inc., of Washougal, Washington. At the initial location of boring B-2, designated on Figure 2 as B-2A, we encountered concrete and metal debris that resulted in extreme mud loss and practical drilling refusal at a depth of approximately 8 feet below the mudline. The final location of boring B-2 was moved approximately 28 feet south and 7 feet west of B-2A, where it was drilled to its ultimate depth of 148.2 feet below mudline. A Shannon & Wilson geologist was present during the explorations to locate the borings, observe the drilling, collect soil samples, and log the materials encountered.

B.3 SAMPLING

B.3.1 Disturbed Sampling

Disturbed samples were collected in the borings, typically at 5- to 10-foot depth intervals, using a standard 2-inch outside diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing standards. 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 cohesionless soils (silt, sand, and gravel), and the consistency of cohesive soils (silt and clay). All disturbed samples were visually identified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination and testing.

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. One automatic hammer was used throughout the exploration program. Automatic hammers generally have higher energy transfer efficiencies than cathead driven hammers. Based on information we received from Western States, the energy efficiency of their automatic hammer used on site averaged 92.6 percent when measured in May 2015. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied.

An SPT was considered to have met refusal where more than 50 blows were required to drive the sampler 6 inches. 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”.

B.3.2 Undisturbed Sampling

Undisturbed samples were collected in 3-inch O.D. thin-wall Shelby tubes, which were hydraulically pushed into the undisturbed soil at the bottoms of boreholes. The soils exposed at the ends of the tubes were examined and described in the field. After examination, the ends of the tubes were sealed to preserve the natural moisture of the samples. The sealed tubes were stored in the upright position, and care was taken to avoid shock and vibration during their transport and storage in our laboratory.

B.4 BOREHOLE ABANDONMENT

All borings were backfilled with bentonite cement grout or bentonite chips in accordance with Oregon Water Resource Department regulations. No wells or other instruments were installed in the boreholes. Backfill of boring B-1, which penetrated a paved surface, was finished at the surface with a matching section of ODOT-approved asphalt cold patch and nominally compacted gravel extending to a depth of at least 2 feet below the ground surface.

B.5 MATERIALS DESCRIPTION

In the field, soil samples were described and identified visually in accordance with the ODOT Soil and Rock Classification Manual (1987). The ASTM International (ASTM) D2488 Visual-Manual method was also used as a guide in determining the key diagnostic properties of soils. Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the samples were noted. Once returned to our laboratory, the samples were re-examined, various standard laboratory tests were conducted, and the field descriptions and identifications were modified where necessary. Please refer to the ODOT Soil and Rock Classification Manual (1987) for definitions of descriptive terminology used in the Drill Logs.

B.6 DRILL LOGS

Summary logs of the borings are presented in the Drill Logs, Figures B1 through B3. Soil descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the drill logs gives individual sample intervals, percent recovery, Standard Penetration Test data, and natural moisture content measurements. Material descriptions and geotechnical unit designations are shown in the center of the drill log, and the right-hand portion provides a graphic log, miscellaneous comments, and a graphic depicting hole backfill details.

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

Figure **B1**
Page **1** of **8**

		Hole No. **B-1**
Project **Burnside Bridge Seismic Feasiblity Study**	Purpose **Burnside Bridge**	E.A. No. **N/A**
Highway **Burnside Street**	County **Multnomah**	Key No. **N/A**
Hole Location Northing: **~ 684,323** Easting: **~ 7,646,091**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 92.6%)**	Driller **Western States/Brad**	Bridge No. **00511**
Project Geologist **Adrian A.J. Holmes**	Recorder **Elizabeth Barnett**	Ground Elev. **~ 35 ft.**
Start Date **October 3, 2016** End Date **October 7, 2016**	Total Depth **221.50 ft**	Tube Height **N/A**

Test Type
- "A" - Auger Core
- "GP" - GeoProbe®
- "X" - Auger
- "C" - Core, Barrel Type
- "N" - Standard Penetration
- "U" - Undisturbed Sample
- "T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 8.50 Sandy GRAVEL with some silt; GP-GM; Orange-brown; Nonplastic fines; Wet; Medium Dense; Fine to coarse, subrounded gravel; Mostly fine to medium sand, trace coarse sand; Trace brick fragments; Trace iron oxide staining; (Fill)**		Mud rotary drilling technique; 5-inch diameter borehole; OYO suspension logging performed between depths of 1.6 feet and 206.7 feet		
5	N1	13	7-6-6			N- 1 (5.00-6.50) Sandy GRAVEL with some silt; GP-GM; Orange-brown; Nonplastic fines; Wet; Medium Dense; Fine to coarse, subrounded gravel; Mostly fine to medium sand, trace coarse sand; Trace brick fragments; Trace iron oxide staining; (Fill)					
							8.50 - 12.00 Silty CLAY with trace sand; CL; Blue-gray; Medium plasticity; Moist; Medium Stiff; Fine to medium sand; Trace brick fragments; (Fill)		Wood fragments in cuttings between depths of 8.5 feet and 25.0 feet		
10	N2	20	5-2-3			N- 2 (10.00-11.50) Silty CLAY with trace sand; CL; Blue-gray; Medium plasticity; Moist; Medium Stiff; Fine to medium sand; Trace brick fragments; (Fill)					
							12.00 - 25.00 Clayey GRAVEL with some sand; GC; Gray to dark gray; Low to medium plasticity fines; Moist to wet; Loose; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Few to some wood and charcoal fragments; Trace brick fragments; (Fill)				
15	N3	13	4-3-6			N- 3 (15.00-16.50) Clayey GRAVEL with some sand; GC; Gray; Low to medium plasticity fines; Wet; Loose; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Few wood fragments; Trace brick fragments; (Fill)					
									Possible wood stump or log between depths of 17 feet and 19 feet		
20											

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
20	N4	13	8-3-3		N- 4 (20.00-21.50) Clayey GRAVEL with some sand; GC; Dark gray; Low to medium plasticity fines; Moist; Loose; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Some wood and charcoal fragments; Trace fine brick fragments; (Fill)			Wood fragment content decreases and includes small twigs at 22 feet		
25	N5	0	12-4-4		N- 5 (25.00-26.50) No Recovery	**25.00 - 38.25** **Sandy SILT to Sandy SILT with trace gravel; ML; Gray; Low plasticity; Moist to wet; Medium Stiff; Fine to coarse, subrounded gravel; Fine to medium sand; Trace organics and thin laminations of PEAT; (Sand/Silt Alluvium)**				
30	N6	13	3-2-3		N- 6 (30.00-31.50) Sandy SILT with trace gravel; ML; Gray; Low plasticity; Wet; Medium Stiff; Fine to coarse, subrounded gravel (clast stuck in split spoon tip); Fine to medium sand; (Sand/Silt Alluvium)					
35	N7	80	4-3-3		N- 7 (35.00-36.50) Sandy SILT; ML; Gray; Low plasticity; Moist to wet; Medium Stiff; Fine to medium sand; Micaceous; Trace organics and thin laminations of PEAT; (Sand/Silt Alluvium)					
						38.25 - 42.00 **Silty CLAY with trace sand; CL; Gray-green; Medium plasticity; Moist; Soft to Medium Stiff; Fine to coarse sand; Trace organics; (Fine-grained Alluvium)**				
40	N8	100	0-1-3		N- 8 (40.00-41.50) Silty CLAY with trace sand; CL; Gray-green; Medium plasticity; Moist; Soft to Medium Stiff; Fine to coarse sand; Trace organics; (Fine-grained Alluvium)					
						42.00 - 48.25 **GRAVEL with some clay and some sand; GP-GC; Gray; Low to medium plasticity fines; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Trace fine organics; (Gravel Alluvium)**				
45	N9	20	14-21-45		N- 9 (45.00-46.50) GRAVEL with some clay and some sand; GP-GC; Gray; Low to medium plasticity fines; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Trace fine organics; (Gravel Alluvium)					
						48.25 - 58.25 **Sandy GRAVEL with some silt to Gravelly SAND with some silt;**				
50										

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-1**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
50	N10	100	45-45-50		N- 10 (50.00-51.50) Sandy GRAVEL with some silt; GP-GM; Brown and gray; Nonplastic fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Trace layers of Silty SAND (SM); Some iron oxide staining; (Gravel Alluvium)	**GP-GM, SP-SM; Brown and gray; Nonplastic to low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse or medium to coarse sand; Trace layers of Silty SAND (SM); Some iron oxide staining; (Gravel Alluvium)**				
55	N11	100	27-36-26		N- 11 (55.00-56.50) Sandy GRAVEL with some silt to Gravelly SAND with some silt; GP-GM/SP-SM; Gray; Low plasticity fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Medium to coarse sand; (Gravel Alluvium)					
60	N12	67	18-18-17		N- 12 (60.00-61.50) SAND with some silt and trace gravel; SP-SM; Light gray-brown; Nonplastic fines; Wet; Dense; Fine to coarse, subrounded gravel; Mostly medium to coarse sand, trace fine sand; Some iron oxide staining; (Sand Alluvium)	**58.25 - 63.25 SAND with some silt and trace gravel; SP-SM; Light gray-brown; Nonplastic fines; Wet; Dense; Fine to coarse, subrounded gravel; Mostly medium to coarse sand, trace fine sand; Some iron oxide staining; (Sand Alluvium)**				
65	N13	0	50/1st 3"		N- 13 (65.00-65.25) GRAVEL with cobbles; GP; Dark gray; Wet; Very Dense; Single, broken basalt cobble retrieved from 3-inch sampler; (Gravel Alluvium)	**63.25 - 75.00 GRAVEL with cobbles; GP; Gray to dark gray; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; (Gravel Alluvium)**		Lost approximately 60 gallons of drilling mud between 65 feet and 80 feet; No recovery in sample N13, used 3-inch sampler after SPT to retrieve sample		
70	N14	59	50/1st 2"		N- 14 (70.00-70.17) GRAVEL with cobbles; GP; Gray; Wet; Very Dense; Recovered one fine gravel-sized fragment of andesite; (Gravel Alluvium)					
75	N15	75	43-50/2"		N- 15 (75.00-75.67) GRAVEL with some sand and trace silt; GP; Gray and brown; Nonplastic fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Upper Troutdale Formation)	**75.00 - 80.00 GRAVEL with some sand and trace silt; GP; Gray and brown; Nonplastic fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Upper Troutdale Formation)**				
80										

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance / Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
80	N16	93	35-50/3"		N- 16 (80.00-80.75) Clayey GRAVEL with some sand; GC; Yellow-brown; Medium plasticity fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Upper Troutdale Formation)	**80.00 - 88.00** **Clayey GRAVEL with some sand grading down to Sandy clayey GRAVEL; GC; Gray and yellow-brown; Medium plasticity fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse or fine to medium sand; Some iron oxide staining; (Upper Troutdale Formation)**				
85	N17	55	40-50/5"		N- 17 (85.00-85.92) Sandy clayey GRAVEL; GC; Gray and yellow-brown; Medium plasticity fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Mostly fine to medium sand, trace coarse sand; Some iron oxide staining; (Upper Troutdale Formation)					
90	N18	100	14-17-23		N- 18 (90.00-91.50) Clayey SILT with trace sand; MH; Gray; Medium plasticity; Moist; Hard; Fine sand; Trace organics; (Upper Troutdale Formation)	**88.00 - 94.00** **Clayey SILT with trace sand; MH; Gray; Medium plasticity; Moist; Hard; Fine sand; Trace organics; (Upper Troutdale Formation)**				
95	N19	100	50/1st 2"		N- 19 (95.00-95.17) Sandy clayey GRAVEL; GC; Dark gray; Low to medium plasticity fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Lower Troutdale Formation)	**94.00 - 98.25** **Sandy clayey GRAVEL; GC; Dark gray; Low to medium plasticity fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Lower Troutdale Formation)**				
100	N20	60	50/1st 2"		N- 20 (100.00-100.17) GRAVEL with some silt and some sand; GP-GM; Gray and yellow-brown; Low plasticity fines; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Lower Troutdale Formation)	**98.25 - 126.00** **GRAVEL with some sand to Gravel with some silt and some sand; GP, GP-GM; Gray to dark gray and yellow-brown; Nonplastic to low plasticity fines; Moist to wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Some micaceous zones; Some iron oxide staining; Some zones of weak cementation; (Lower Troutdale Formation)**				
105	N21	100	50/1st 1"		N- 21 (105.00-105.08) Silty SAND with some gravel; SM; Olive; Low plasticity fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Some iron oxide staining; Weak cementation; (Lower Troutdale Formation)					
110										

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-1**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
110										
115	N22	80	50/1st 3"		N- 22 (115.00-115.25) GRAVEL with some sand; GP; Dark gray; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Lower Troutdale Formation)					
120										
125	N23	0	50/1st 2"		N- 23 (125.00-125.17) No Recovery					
						126.00 - 133.00 SAND with some silt to Silty SAND; SP-SM/SM; Dark green-gray; Nonplastic fines; Moist; Very Dense; Fine to medium sand; Micaceous; (Lower Troutdale Formation)				
130	N24	100	32-35-41		N- 24 (130.00-131.50) SAND with some silt to Silty SAND; SP-SM/SM; Dark green-gray; Nonplastic fines; Moist; Very Dense; Fine to medium sand; Micaceous; (Lower Troutdale Formation)					
						133.00 - 150.00 Clayey GRAVEL with some sand; GC; Dark green-gray; Low to medium plasticity fines; Moist; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Trace iron oxide staining; (Lower Troutdale Formation)				
135										
140										

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-1**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
140	N25	100	49-50/5"		N- 25 (140.00-140.92) Clayey GRAVEL with some sand; GC; Dark green-gray; Low to medium plasticity fines; Moist; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Trace iron oxide staining; (Lower Troutdale Formation)					
145										
150	N26a	100	40-34-45		N- 26a (150.00-150.75) Silty SAND with some gravel; SM; Green-gray; Nonplastic to low plasticity fines; Moist; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Lower Troutdale Formation)	**150.00 - 155.00 Silty SAND with some gravel grading down to Sandy SILT with trace gravel; SM, ML; Green-gray; Nonplastic to low plasticity fines; Moist; Very Dense / Very Hard; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Lower Troutdale Formation)**				
	N26b	100			N- 26b (150.75-151.50) Sandy SILT with trace gravel; ML; Green-gray; Nonplastic to low plasticity; Moist; Very Hard; Fine subrounded gravel; Mostly fine sand, trace medium sand; Micaceous; (Lower Troutdale Formation)					
155						**155.00 - 169.00 GRAVEL to GRAVEL with some silt and some sand; GP, GP-GM; Very Dense; Inferred based on drill action and drill cuttings; (Lower Troutdale Formation)**		Lost approximately 20 gallons of drilling mud between 157 feet and 160 feet		
160	N27	0	50/1st 5"		N- 27 (160.00-160.42) No Recovery					
165										
						169.00 - 185.75 Silty CLAY to CLAY				
170										

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
170	N28	0	16-22-22		N- 28 (170.00-171.50) Silty CLAY to CLAY; CL/CH; Gray and green-mottled; Medium to high plasticity; Moist; Hard; Micaceous; (Sandy River Mudstone)	**with trace sand; CL/CH; Gray to gray and green-mottled; Medium to high plasticity; Moist; Hard; Fine sand; Micaceous; Trace organics; (Sandy River Mudstone)**		No recovery in sample N28, used 3-inch sampler after SPT to retrieve sample		
175										
180	N29	100	10-19-24		N- 29 (180.00-181.50) CLAY with trace sand; CH; Gray; Medium to high plasticity; Moist; Hard; Fine sand; Micaceous; Trace organics; (Sandy River Mudstone)					
185										
						185.75 - 215.75 Silty SAND to Silty SAND with trace gravel; SM; Gray, green-gray, and purple; Nonplastic to low plasticity fines; Moist; Very Dense; Fine, subrounded gravel; Fine to medium or fine to coarse sand; Some micaceous zones; Few 2- to 3-inch thick interbeds of Sandy SILT (ML) above 203 feet; Few gravelly lenses below 203 feet based on drill action; (Sandy River Mudstone)				
190	N30	100	20-33-34		N- 30 (190.00-191.50) Silty SAND and Sandy SILT; SM, ML; Green-gray; Low plasticity fines; Moist; Very Dense / Very Hard; Fine to medium sand; SM and ML interbedded in 2- to 3-inch-thick layers; (Sandy River Mudstone)					
195										
200										

ODOT DRILL LOG - FOR SW REVIEW 24-1-04065.GPJ ODOT_MANWITHSWLAB.GDT 1/23/17

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-1**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
200	N31	80	35-43-50		N- 31 (200.00-201.50) Silty SAND with trace gravel; SM; Gray; Nonplastic to low plasticity fines; Moist; Very Dense; Fine, subrounded gravel; Fine to coarse sand; 1- to 3-inch-thick layers with finer and coarser sand; (Sandy River Mudstone)					
								Intermittent drill chatter below 203 feet		
205										
210	N32	80	28-32-40		N- 32 (210.00-211.50) Silty SAND; SM; Purple and green-gray; Nonplastic to low plasticity fines; Moist; Very Dense; Fine to medium sand; Micaceous; (Sandy River Mudstone)					
215										
						215.75 - 221.50 SAND to SAND with some silt; SP/SP-SM; Purple and green-gray; Nonplastic fines; Moist; Very Dense; Mostly fine to medium sand, trace coarse sand; Trace 1- to 2-inch-thick interbeds of Sandy SILT (ML); (Sandy River Mudstone)				
220	N33	100	39-35-31		N- 33 (220.00-221.50) SAND to SAND with some silt; SP/SP-SM; Purple and green-gray; Nonplastic fines; Moist; Very Dense; Mostly fine to medium sand, trace coarse sand; Trace 1- to 2-inch thick interbeds of Sandy SILT (ML); (Sandy River Mudstone)					
						221.50 End of Hole				
225										
230										

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

Figure **B2**
Page **1** of **6**

		Hole No. **B-2**
Project **Burnside Bridge Seismic Feasiblity Study**	Purpose **Burnside Bridge**	E.A. No. **N/A**
Highway **Burnside Street**	County **Multnomah**	Key No. **N/A**
Hole Location Northing: **~ 684,114** Easting: **~ 7,646,475**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 92.6%)**	Driller **Western States/Brad**	Bridge No. **00511**
Project Geologist **Adrian A.J. Holmes**	Recorder **Elizabeth Barnett**	Ground Elev. **~ -38 ft.**
Start Date **October 17, 2016** End Date **October 25, 2016**	Total Depth **148.20 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"GP" - GeoProbe®
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 14.10 SAND with trace silt grading to SAND with trace silt and trace gravel; SP; Dark gray; Nonplastic fines; Wet; Very Dense; Fine, subrounded gravel; Fine to medium sand; Some possible wood debris; (Sand Alluvium)**		Boring drilled from barge using mud rotary drilling technique; 5-inch diameter borehole; all depths are below mudline; HWT casing advanced progressively after each sample, up to a depth of 41 feet, in order to maintain borehole stability; OYO suspension logging performed between depths of 41.0 feet and 134.5 feet		
5											
10									Wood fragments in cuttings at 10 feet; increased gravel content based on drill action; possible heave prior to sample N1		
	N1	100	6-22-29			N- 1 (10.70-12.20) SAND with trace silt and trace gravel; SP; Dark gray; Nonplastic fines; Wet; Very Dense; Fine, subrounded gravel; Fine to medium sand; (Sand Alluvium)					
							14.10 - 24.35 Gravelly SAND with trace silt; SP; Dark gray; Nonplastic fines; Wet; Loose; Fine to coarse, subangular to subrounded gravel; Fine to medium or fine to coarse sand; (Sand Alluvium)				
15											
	N2	67	6-4-5			N- 2 (16.00-17.50) Gravelly SAND with trace silt; SP; Dark gray; Nonplastic fines; Wet; Loose; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Sand Alluvium)			Lost approximately 40 gallons of drilling mud between 16.5 feet and 18 feet		
20											

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
20											
	N3	67	7-4-5			N- 3 (21.00-22.50) Gravelly SAND with trace silt; SP; Dark gray; Nonplastic fines; Wet; Loose; Fine to coarse, subrounded gravel; Mostly fine to medium sand, trace coarse sand; (Sand Alluvium)					
									Some sand heaving and mud loss at 22 feet; driller added Barite to mud		
25							**24.35 - 40.00 SAND with some gravel and trace silt to Silty SAND with some gravel; SP, SP-SM, SM; Dark gray; Nonplastic fines; Wet; Medium Dense; Fine to coarse, subrounded gravel; Mostly fine to medium sand, trace coarse sand; Some micaceous zones; Some zones with trace wood and twigs; (Sand Alluvium)**				
	N4	80	8-7-7			N- 4 (26.20-27.70) SAND with some gravel and trace silt; SP; Dark gray; Nonplastic fines; Wet; Medium Dense; Fine to coarse, subrounded gravel; Mostly fine to medium sand, trace coarse sand; (Sand Alluvium)					
30											
	N5	13	8-8-10			N- 5 (31.50-33.00) Silty SAND with some gravel; SM; Dark gray; Nonplastic fines; Wet; Medium Dense; Fine to coarse, subrounded gravel; Mostly fine to medium sand, trace coarse sand; Trace wood and twigs; (Sand Alluvium)					
35											
	N6	67	6-6-10			N- 6 (36.50-38.00) SAND with some silt; SP-SM; Dark gray; Nonplastic fines; Wet; Medium Dense; Fine to medium sand; Micaceous; (Sand Alluvium)			Lost approximately 80 gallons of drilling mud around 36 feet; some sand heaving; driller added Barite to mud		
40							**40.00 - 53.00 GRAVEL with trace sand to Sandy GRAVEL with trace silt; GP; Dark gray; Nonplastic fines; Wet; Medium Dense to Very Dense; Fine to coarse, subrounded to rounded gravel; Fine to coarse sand; Trace 0.25-inch-thick interbeds of SILT (ML) and 2-inch-thick interbeds of Silty SAND (SM); (Gravel Alluvium)**		Lost approximately 300 gallons of drilling mud between 40 feet and 47 feet; driller added N-Seal to borehole to mitigate mud loss		
	N7	33	32-31-33			N- 7 (41.00-42.50) GRAVEL with some sand and trace silt; GP; Dark gray; Nonplastic fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Trace 0.25-inch-thick interbeds of green-gray SILT (ML); (Gravel Alluvium)					
45											
	N8	33	17-14-14			N- 8 (45.70-47.20) GRAVEL with trace sand; GP; Dark gray; Wet; Medium Dense; Fine to coarse, subrounded gravel; Sample could be slough; (Gravel Alluvium)					
50											

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
50											
	N9	67	22-18-25			N- 9 (51.20-52.70) Sandy GRAVEL with trace silt; GP; Dark gray; Nonplastic fines; Wet; Dense; Fine to coarse, subrounded to rounded gravel; Mostly coarse sand, trace fine to medium sand; Trace 2-inch-thick interbeds of Silty SAND (SM); (Gravel Alluvium)					
							53.00 - 72.00 Sandy SILT to Sandy SILT with trace gravel; ML; Gray; Low plasticity; Moist; Very Stiff to Hard; Fine to coarse, subrounded to rounded gravel; Fine sand; Trace organics; Trace interbeds of Silty SAND (SM) with nonplastic fines; (Sand/Silt Alluvium)				
55											
	N10	33	28-20-13			N- 10 (56.90-58.40) Sandy SILT with trace gravel; ML; Gray; Low plasticity; Moist; Hard; Fine, subrounded gravel; Fine sand; Trace interbeds of Silty SAND (SM) with nonplastic fines; (Sand/Silt Alluvium)					
60											
	N11	7	10-12-14			N- 11 (62.00-63.50) Poor Recovery; One coarse, rounded gravel clast stuck in split spoon tip; (Sand/Silt Alluvium)					
65											
	N12	67	4-2-15			N- 12 (67.10-68.60) Sandy SILT; ML; Gray; Low plasticity; Moist; Very Stiff; Fine sand; Trace organics; (Sand/Silt Alluvium)					
70											
							72.00 - 73.50 GRAVEL; GP; Gray; Wet; Very Dense; Fine to coarse, subrounded gravel; Possible cobbles; (Gravel Alluvium)				
	N13	30	50/1st 4"			N- 13 (72.70-73.03) GRAVEL; GP; Gray; Wet; Very Dense; Fine to coarse, subrounded gravel; Possible cobbles based on drill action; (Gravel Alluvium)					
							73.50 - 80.00 CLAY with some sand; CH; Yellow-brown to green-gray with orange mottling; Medium to high plasticity; Moist; Hard; Fine sand; (Upper Troutdale Formation)				
75											
	N14	100	25-23-30			N- 14 (77.00-78.50) CLAY with some sand; CH; Yellow-brown to green-gray with orange mottling; Medium to high plasticity; Moist; Hard; Fine sand; (Upper Troutdale Formation)					
80											

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-2**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
80						**80.00 - 82.00** **Silty CLAY with trace gravel; CL; Gray-brown; Medium plasticity; Moist; Very Hard; Fine to coarse gravel; Micaceous; (Upper Troutdale Formation)**				
	N15a	100	15-26-42		N- 15a (81.50-82.00) Silty CLAY with trace gravel; CL; Gray-brown; Medium plasticity; Moist; Very Hard; Fine to coarse gravel; Micaceous; Orange-mottled in bottom 2 to 3 inches; (Upper Troutdale Formation)					
	N15b	100			N- 15b (82.00-83.00) Sandy SILT; ML; Light brown and orange-mottled; Nonplastic; Moist; Very Dense; Fine sand; Micaceous; (Upper Troutdale Formation)	**82.00 - 89.00** **Sandy SILT; ML; Brown to light brown and orange-mottled; Nonplastic; Moist; Dense to Very Dense; Fine sand; Micaceous; (Upper Troutdale Formation)**				
85										
	N16	100	14-20-28		N- 16 (86.30-87.80) Sandy SILT; ML; Brown; Nonplastic; Moist; Dense; Fine sand; Micaceous; (Upper Troutdale Formation)					
						89.00 - 116.00 **GRAVEL with some sand to GRAVEL with some silt and some sand; GP, GP-GM; Gray, yellow, and brown; Nonplastic to low plasticity fines; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Some iron oxide staining and zones of weak cementation; (Lower Troutdale Formation)**				
90										
	N17	60	50/1st 2"		N- 17 (91.50-91.67) GRAVEL with some sand; GP; Yellow and gray; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel with weathered surfaces and traces of cemented fine to medium sand; (Lower Troutdale Formation)					
95										
	N18	100	50/1st 1"		N- 18 (96.90-96.98) GRAVEL with some silt and some sand; GP-GM; Yellow and brown; Low plasticity fines; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Lower Troutdale Formation)					
100										
105										
	N19	100	50/1st 1"		N- 19 (107.40-107.48) GRAVEL with some sand and trace silt; GP; Gray; Nonplastic fines; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; Mostly fine sand, trace medium and coarse sand; Some iron oxide staining and weak cementation; (Lower Troutdale Formation)					
110										

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
110											
115											
							116.00 - 130.00 SAND with some silt to Silty SAND; SP-SM, SM; Green-gray to gray-brown; Nonplastic to low plasticity fines; Moist; Very Dense; Fine to medium sand; Some micaceous zones; Some zones with sand grains that can be reduced to Silty CLAY (CL) under finger pressure; (Sandy River Mudstone)				
	N20	75	49-50/2"			N- 20 (118.70-119.37) SAND with some silt to Silty SAND; SP-SM/SM; Green-gray; Nonplastic fines; Moist; Very Dense; Fine to medium sand; Micaceous; (Sandy River Mudstone)					
120											
125											
	N21	99	25-40-50/5"			N- 21 (128.20-129.62) Silty SAND; SM; Gray-brown; Nonplastic to low plasticity fines; Moist; Very Dense; Fine to medium sand; Some iron oxide staining; Sand grains can be reduced to clay under finger pressure; (Sandy River Mudstone)					
130							**130.00 - 141.95 Silty CLAY to CLAY; CL/CH; Blue-green and gray; Medium to high plasticity; Moist; Very Hard; Some mottled iron oxide staining; (Sandy River Mudstone)**				
135											
	N22	100	30-33-43			N- 22 (136.50-138.00) Silty CLAY to CLAY; CL/CH; Blue-green and gray; Medium to high plasticity; Moist; Very Hard; Some mottled iron oxide staining; (Sandy River Mudstone)					
140											

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
140								▨			▨
							141.95 - 148.20 **Silty CLAY with some sand; CL; Blue-green and gray with dark green mottling; Low to medium plasticity; Moist; Hard; Fine sand; (Sandy River Mudstone)**	▨			▨
145											
	N23	100	12-14-21			N- 23 (145.90-147.40) Silty CLAY with some sand; CL; Blue-green and gray with dark green mottling; Low to medium plasticity; Moist; Hard; Fine sand; (Sandy River Mudstone)					
							148.20 **End of Hole**		Boring B-2 was first attempted approximately 28 feet north and 7 feet east of its final location. At the northern location (B-2A), concrete and metal debris were encountered at a depth of approximately 8 feet below the mudline, causing drilling refusal.		
150											
155											
160											
165											
170											

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

Figure **B3**
Page **1** of **9**

			Hole No. **B-3**
Project **Burnside Bridge Seismic Feasiblity Study**		Purpose **Burnside Bridge**	E.A. No. **N/A**
Highway **Burnside Street**		County **Multnomah**	Key No. **N/A**
Hole Location Northing: **~ 684,158**	Easting: **~ 7,647,283**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 92.6%)**		Driller **Western States/Brad**	Bridge No. **00511**
Project Geologist **Adrian A.J. Holmes**		Recorder **Elizabeth Barnett**	Ground Elev. **~ 32 ft.**
Start Date **September 19, 2016**	End Date **September 22, 2016**	Total Depth **230.25 ft**	Tube Height **N/A**

Test Type

"A" - Auger Core
"GP" - GeoProbe®
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 4.00** **Silty GRAVEL with some sand; GM; Inferred from drill action and drill cuttings; (Fill)**		Mud rotary drilling technique; 5-inch diameter borehole; OYO suspension logging performed between depths of 6.6 feet and 216.5 feet		
							4.00 - 13.00 **SAND with some silt and some gravel; SP-SM; Brown; Nonplastic fines; Moist to wet; Loose; Fine, subrounded gravel; Fine to medium sand; Some iron oxide staining; (Fill)**				
5	N1	20	5-3-4			N- 1 (5.00-6.50) SAND with some silt and some gravel; SP-SM; Brown; Nonplastic fines; Wet; Loose; Fine, subrounded gravel; Fine to medium sand; (Fill)					
10	N2	33	3-3-3			N- 2 (10.00-11.50) SAND with some silt and some gravel; SP-SM; Brown with orange staining; Nonplastic fines; Moist; Loose; Fine, subrounded gravel; Fine to medium sand; (Fill)					
							13.00 - 18.25 **Silty CLAY; CL; Gray; Medium to high plasticity; Wet; Very Soft; Trace charcoal fragments; (Fine-grained Alluvium)**		Wood fragments in cuttings from 13 to 15 feet		
15	N3	53	0-0-0			N- 3 (15.00-16.50) Silty CLAY; CL; Gray; Medium to high plasticity; Wet; Very Soft; Trace charcoal fragments; (Fine-grained Alluvium)					
							18.25 - 23.25 **Sandy SILT; ML; Gray; Low plasticity; Moist to wet; Very**				
20											

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
20	N4	100	0-0-0		N- 4 (20.00-21.50) Sandy SILT; ML; Gray; Low plasticity; Moist to wet; Very Soft; Fine sand; Micaceous; (Fine-grained Alluvium)	**Soft; Fine sand; Micaceous; (Fine-grained Alluvium)**				
						23.25 - 38.25 **Silty SAND; SM; Brown to gray-brown; Nonplastic to low plasticity fines; Wet; Very Loose to Loose; Fine sand grading to fine to medium sand; Micaceous; Some iron oxide staining; Trace 1-inch-thick layers of Sandy silty CLAY (CL); (Sand/Silt Alluvium)**				
25	N5	67	1-1-0		N- 5 (25.00-26.50) Silty SAND; SM; Brown; Low plasticity fines; Wet; Very Loose; Fine sand; Micaceous; Some iron oxide staining; (Sand/Silt Alluvium)					
30	U1	100			U- 1 (30.00-32.00) Inferred Silty SAND; SM; (Sand/Silt Alluvium)					
	N6	100	2-3-6		N- 6 (32.00-33.50) Silty SAND; SM; Brown; Low plasticity fines; Wet; Loose; Fine sand; Micaceous; (Sand/Silt Alluvium)					
35	N7	100	3-5-2		N- 7 (35.00-36.50) Silty SAND; SM; Gray-brown; Nonplastic fines; Wet; Loose; Fine to medium sand; Micaceous; Some iron oxide staining; Trace 1-inch-thick layers of Sandy silty CLAY (CL); (Sand/Silt Alluvium)					
						38.25 - 43.25 **Sandy SILT; ML; Gray; Low plasticity; Wet; Medium Stiff; Fine to medium sand; Micaceous; (Sand/Silt Alluvium)**				
40	N8	100	3-2-4		N- 8 (40.00-41.50) Sandy SILT; ML; Gray; Low plasticity; Wet; Medium Stiff; Fine to medium sand; Micaceous; (Sand/Silt Alluvium)					
						43.25 - 48.25 **SAND with some silt to Silty SAND; SP-SM/SM; Dark gray; Wet; Medium Dense; Fine to medium sand; Micaceous; (Sand/Silt Alluvium)**				
45	N9	33	7-5-6		N- 9 (45.00-46.50) SAND with some silt to Silty SAND; SP-SM/SM; Dark gray; Wet; Medium Dense; Fine to medium sand; Micaceous; (Sand/Silt Alluvium)					
						48.25 - 63.25 **Silty SAND; SM; Gray to gray-brown; Nonplastic to low**				
50										

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-3**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
50	N10	0	4-12-12		N- 10 (50.00-51.50) Silty SAND; SM; Gray; Nonplastic to low plasticity fines; Wet; Medium Dense; Fine to medium sand; Micaceous; Stratified with 2- to 3-inch-thick layers of Silty CLAY (CL); (Sand/Silt Alluvium)	**plasticity fines; Wet; Loose to Medium Dense; Fine to medium sand; Micaceous; Stratified with 1- to 4-inch thick layers of Silty CLAY to Sandy Silty CLAY (CL); (Sand/Silt Alluvium)**		No recovery in sample N10, used 3-inch sampler after SPT to retrieve sample Drill chatter from 52 feet to 53 feet; possible gravel lens		
55	N11	67	5-5-8		N- 11 (55.00-56.50) Silty SAND; SM; Gray; Nonplastic to low plasticity fines; Wet; Medium Dense; Fine to medium sand; Micaceous; Stratified with 3- to 4-inch-thick layers of Silty CLAY (CL); (Sand/Silt Alluvium)					
60	N12	67	4-6-3		N- 12 (60.00-61.50) Silty SAND; SM; Gray-brown; Nonplastic to low plasticity fines; Wet; Loose; Fine to medium sand; Micaceous; Stratified with 1- to 2-inch-thick layers of Sandy silty CLAY (CL); (Sand/Silt Alluvium)					
						63.25 - 88.25 **Sandy SILT; ML; Gray; Low plasticity; Moist to wet; Medium Stiff to Stiff; Fine Sand; Micaceous; Trace roots and wood fragments; Stratified with 1- to 3-inch layers of Silty/Clayey SAND (SM/SC) with nonplastic to medium plasticity fines; (Sand/Silt Alluvium)**				
65	N13	80	5-3-2		N- 13 (65.00-66.50) Sandy SILT; ML; Gray; Low plasticity; Wet; Medium Stiff; Fine sand; Micaceous; Trace roots and wood fragments; Stratified with 1- to 2-inch layers of Silty SAND (SM); (Sand/Silt Alluvium)					
70	N14	100	0-1-12		N- 14 (70.00-71.50) Sandy SILT; ML; Gray; Low plasticity; Wet; Stiff; Fine sand; Micaceous; Trace wood fragments; Stratified with 2- to 3-inch layers of Silty/Clayey SAND (SM/SC); (Sand/Silt Alluvium)					
75	N15	100	9-5-3		N- 15 (75.00-76.50) Sandy SILT; ML; Gray; Low plasticity; Moist; Medium Stiff to Stiff; Fine sand; Micaceous; (Sand/Silt Alluvium)					
80										

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
80	N16	100	8-5-1		N- 16 (80.00-81.50) Sandy SILT; ML; Gray; Low plasticity; Moist; Medium Stiff; Fine to medium sand; Micaceous; Trace interbeds of Silty SAND (SM) with nonplastic fines; (Sand/Silt Alluvium)					
85	N17	100	3-2-7		N- 17 (85.00-86.50) SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Stiff; Fine sand; Micaceous; Trace rootlets; Trace 0.25- to 1-inch-thick layers of Silty SAND with nonplastic fines (SM); (Sand/Silt Alluvium)	**85.00** **Grades to SILT with some sand; ML**				
						88.25 - 93.25 **Silty SAND; SM; Gray; Nonplastic fines; Moist to wet; Medium Dense; Fine to medium sand; Micaceous; Stratified with 2-inch-thick layers of low plasticity SILT (ML); (Sand/Silt Alluvium)**				
90	N18	100	7-10-6		N- 18 (90.00-91.50) Silty SAND; SM; Gray; Nonplastic fines; Moist to wet; Medium Dense; Fine to medium sand; Micaceous; Stratified with 2-inch-thick layers of low plasticity SILT (ML); (Sand/Silt Alluvium)					
						93.25 - 113.25 **SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Soft to Medium Stiff; Fine to medium sand; Micaceous; Stratified with up to 2-inch-thick layers of Sandy SILT (ML) and Silty SAND (SM); (Fine-grained Alluvium)**				
95	N19	100	0-1-5		N- 19 (95.00-96.50) SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Medium Stiff; Fine sand; Micaceous; Trace organics; Stratified with 0.5- to 1-inch-thick layers of Silty SAND (SM); (Fine-grained Alluvium)					
100	N20	100	3-1-1		N- 20 (100.00-101.50) SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Soft; Fine sand; Micaceous; Stratified with up to 1-inch-thick layers of Sandy SILT (ML); (Fine-grained Alluvium)					
105	N21	100	8-5-1		N- 21 (105.00-106.50) SILT with some sand; ML; Gray; Low plasticity; Wet; Medium Stiff; Fine to medium sand; Micaceous; Stratified with 1- to 2-inch-thick layers of Silty SAND (SM); (Fine-grained Alluvium)					
110										

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance / Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
110	N22	100	0-1-3		N- 22 (110.00-111.50) SILT with some sand; ML; Gray; Low plasticity; Wet; Soft to Medium Stiff; Fine sand; Laminated with thin seams of Silty SAND (SM); (Fine-grained Alluvium)					
						113.25 - 118.25 **Silty SAND; SM; Gray-brown; Nonplastic fines; Moist to wet; Medium Dense; Fine to medium sand; Micaceous; (Sand/Silt Alluvium)**				
115	N23	80	13-12-9		N- 23 (115.00-116.50) Silty SAND; SM; Gray-brown; Nonplastic fines; Moist to wet; Medium Dense; Fine to medium sand; Micaceous; (Sand/Silt Alluvium)					
						118.25 - 138.25 **Sandy SILT grading to SILT with some sand; ML; Gray; Nonplastic to low plasticity; Moist to wet; Stiff; Fine sand; Micaceous; Stratified with thin seams to 2-inch-thick layers of Silty SAND (SM); (Sand/Silt Alluvium)**				
120	N24	100	0-8-6		N- 24 (120.00-121.50) Sandy SILT; ML; Gray; Nonplastic to low plasticity; Moist; Stiff; Fine sand; Laminated with thin seams of Silty SAND (SM); (Sand/Silt Alluvium)					
125	N25	0	5-8-9		N- 25 (125.00-126.50) No Recovery					
130	N26	80	5-1-8		N- 26 (130.00-131.50) SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Stiff; Fine sand; Micaceous; Stratified with 2-inch-thick layers of Silty SAND (SM); (Sand/Silt Alluvium)					
135	N27	67	8-1-0		N- 27 (135.00-136.50) SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Very Soft; Fine sand; Micaceous; Stratified with 1- to 2-inch-thick layers of Silty SAND (SM); (Sand/Silt Alluvium)	**135.00** **Grades to very soft**				
						138.25 - 142.00 **SAND with some silt to Silty SAND; SP-SM/SM; Dark**				
140										

ODOT DRILL LOG - FOR SW REVIEW 24-1-04065.GPJ ODOT_MANWITHSWLAB.GDT 1/23/17

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-3**

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance / Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
140	N28	33	11-14-10		N- 28 (140.00-141.50) SAND with some silt to Silty SAND; SP-SM/SM; Dark gray; Nonplastic fines; Wet; Medium Dense; Fine to medium sand; (Sand Alluvium)	**gray; Nonplastic fines; Wet; Medium Dense; Fine to medium sand; (Sand Alluvium)**				
						142.00 - 167.00 GRAVEL with some silt and some sand to Sandy GRAVEL with some silt; GP-GM; Dark gray to gray and brown; Nonplastic to low plasticity fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Gravel Alluvium)				
145	N29	33	46-30-40		N- 29 (145.00-146.50) GRAVEL with some silt and some sand; GP-GM; Dark gray; Nonplastic to low plasticity fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Slight iron oxide staining; (Gravel Alluvium)					
150	N30	60	50/1st 6"		N- 30 (150.00-150.50) GRAVEL with some silt and some sand; GP-GM; Brown to dark gray; Low plasticity fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Gravel Alluvium)					
155	N31	67	31-34-50		N- 31 (155.00-156.50) Sandy GRAVEL with some silt; GP-GM; Gray and brown; Nonplastic to low plasticity fines; Wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Gravel Alluvium)					
160	N32	98	50/1st 5"		N- 32 (160.00-160.42) GRAVEL with some silt and some sand; GP-GM; Gray and brown; Nonplastic fines; Moist to wet; Very Dense; Fine to coarse, subrounded gravel; Fine to coarse sand; Some iron oxide staining; (Gravel Alluvium)					
165										
						167.00 - 180.20 Sandy silty GRAVEL; GM; Dark green-gray; Low plasticity fines; Moist; Very Dense; Fine to coarse, subrounded to				
170										

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-3**

Depth (ft)	Test Type, No.	Percent Recovery	Soil Driving Resistance	Rock Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
170	N33	100	50/1st 3"			N- 33 (170.00-170.25) Sandy silty GRAVEL; GM; Dark green-gray; Low plasticity fines; Moist; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; Micaceous; Some iron oxide staining; (Lower Troutdale Formation)	**rounded gravel; Mostly fine to medium sand, trace coarse sand; Micaceous; Some iron oxide staining; (Lower Troutdale Formation)**				
175											
180	N34a N34b	100 100	50/1st 3"			N- 34a (180.10-180.20) Silty SAND; SM; Dark gray; Low to medium plasticity fines; Wet; Very Dense; Fine to medium sand; Trace thin laminations of Silty CLAY (CL); (Lower Troutdale Formation) N- 34b (180.20-180.35) GRAVEL with some silt and some sand; GP-GM; Dark green-gray; Low plasticity fines; Wet; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; (Lower Troutdale Formation)	**180.20 - 195.00 GRAVEL with some silt and some sand; GP-GM; Dark green-gray to gray and brown; Low plasticity fines; Moist to wet; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; Some iron oxide staining; (Lower Troutdale Formation)**				
185											
190	N35	100	50/1st 1"			N- 35 (190.00-190.08) GRAVEL with some silt and some sand; GP-GM; Gray and brown; Low plasticity fines; Moist to wet; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; Some iron oxide staining; (Lower Troutdale Formation)					
195							**195.00 - 230.25 Sandy silty GRAVEL; GM; Gray and yellow-brown; Low to medium plasticity fines; Moist; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand;**				
200											

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-3**

Depth (ft)	Test Type, No.	Percent Recovery	Soil / Rock: Driving Resistance / Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
200	N36	0	50/1st 3"		N- 36 (200.00-200.25) No Recovery	**Micaceous; Some iron oxide staining; Some cemented sand on surfaces of gravel clasts; (Lower Troutdale Formation)**				
205										
210	N37	60	50/1st 2"		N- 37 (210.00-210.17) Sandy silty GRAVEL; GM; Gray and yellow-brown; Low to medium plasticity fines; Moist; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; Micaceous; Some iron oxide staining; (Lower Troutdale Formation)					
215										
220	N38	100	50/1st 3"		N- 38 (220.00-220.25) Sandy silty GRAVEL; GM; Gray and yellow-brown; Low to medium plasticity fines; Moist; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; Micaceous; Some iron oxide staining; (Lower Troutdale Formation)			Drill advances quickly through inferred softer layer and increased silt/clay in cuttings between 221 feet and 224 feet		
225										
230										

Project Name **Burnside Bridge Seismic Feasiblity Study** Hole No. **B-3**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
230	N39	100	50/1st 3"		N- 39 (230.00-230.25) Sandy silty GRAVEL; GM; Gray and yellow-brown; Low to medium plasticity fines; Moist; Very Dense; Fine to coarse, subrounded to rounded gravel; Mostly fine to medium sand, trace coarse sand; Micaceous; Some iron oxide staining; Some evidence of cementation on surfaces of gravel clasts; (Lower Troutdale Formation)	**230.25** **End of Hole**				
235										
240										
245										
250										
255										
260										

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APPENDIX C: PREVIOUS IN SITU GEOPHYSICAL TESTS

Appendix C

Previous In Situ Geophysical Tests

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C.2 OYO Suspension Logging....C-1

Attachments

GEOVision report dated November 28, 2016: "Burnside Bridge Suspension PS Velocities; Boreholes B-1, B-2, and B-3"

C.1 GENERAL

The field exploration program performed during the previous phase of the project included geophysical measurements of compressional and shear wave velocities in all three borings performed for the project. Approximate locations of the tested boreholes are shown on the Site and Exploration Plan, Figure 2. The measurements were taken at regular depth intervals and used to generate profiles of compressional and shear wave velocities, the latter of which were used in this study to model the seismic response of the site to earthquake loading.

C.2 OYO SUSPENSION LOGGING

The measurements of compressional and shear wave velocities were made using OYO Suspension Logging techniques. The OYO Suspension Logging was performed by GEOVision Geophysical Services of Corona, California, using an OYO Model 170 Suspension Logging Recorder and Suspension Logging Probe. During suspension logging, measurements were taken at 1.6-foot depth intervals using a down-hole probe that contains a wave source and two geophones. The OYO Suspension Logging was performed in 5-inch diameter, open-hole, mud rotary borings that were drilled by Western States Soil Conservation, Inc., using a truck-mounted CME-75 drill rig. Borehole information, including the approximate ground surface elevation and encountered geotechnical units, are shown on the drill logs in Appendix B. A description of the OYO Suspension Logging procedures and logs of the recorded compressional wave and shear wave velocities are provided in a report prepared by GEOVision Geophysical Services which is attached to the end of this appendix.

BURNSIDE BRIDGE

SUSPENSION PS VELOCITIES

BOREHOLES B-1, B-2, AND B-3

PORTLAND, OREGON

November 28, 2016

Report 16361-01 rev 0

BURNSIDE BRIDGE
SUSPENSION PS VELOCITIES
BOREHOLES B-1, B-2, AND B-3
PORTLAND, OREGON

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November 28, 2016
Report 16361-01 rev 0

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APPENDICES

APPENDIX A **SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS**

APPENDIX B **GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS**

INTRODUCTION

GEO*Vision* acquired borehole geophysical data in three boreholes at the Burnside Bridge in Portland, Oregon. The work was performed for Shannon & Wilson, Inc. Fieldwork was performed by Jonathan Jordan and Glenn Goss. Analysis and report was completed by Emily Feldman, and reviewed by John Diehl, Professional Engineer.

SCOPE OF WORK

This report presents results of Suspension PS velocity data acquired in three boreholes between September 26th and October 23rd, 2016, as detailed in Table 1. The purpose of these measurements was to supplement stratigraphic information by acquiring shear wave and compressional wave velocities as a function of depth.

The OYO Suspension PS Logging System (Suspension System) was used to obtain in-situ horizontal shear (S_H) and compressional (P) wave velocity measurements in three uncased boreholes at 1.6 foot intervals. Measurements followed **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.5. Acquired data were analyzed and a profile of velocity versus depth was produced for both S_H and P waves. Borehole B-2 was logged offshore from a barge, while boreholes B-1 and B-3 were logged on land.

A detailed reference for the suspension PS velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

INSTRUMENTATION

Suspension Velocity Instrumentation

Suspension velocity measurements were performed using the suspension PS logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geologging. This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the borehole of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the borehole producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is approximately 25 feet, with the center point of the receiver pair 12.5 feet above the bottom end of the probe.

The probe receives control signals from, and sends the digitized receiver signals to, instrumentation on the surface via an armored multi-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data using a sheave of known circumference fitted with a digital rotary encoder.

The entire probe is suspended in the borehole by the cable, therefore, source motion is not coupled directly to the borehole walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the borehole and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it passes through the casing and grout annulus and impinges upon the wall of the borehole. These waves propagate through the soil and rock surrounding the borehole, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 6.3 foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H-wave signals.
4. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (feet versus inches scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.

3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The Suspension PS system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale. Data are stored on disk for further processing.

Review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), and sample rate to optimize the quality of the data before recording. Verification of the calibration of the Suspension PS digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

MEASUREMENT PROCEDURES

Suspension Velocity Measurement Procedures

Boreholes B-1, B-2, and B-3 were logged uncased and filled with fresh water mud. Measurements followed the **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.5. Prior to the logging run, the probe was positioned with the top of the probe even with a stationary reference point such as top of casing stick up. The electronic depth counter was set to the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point, if any. For borehole B-2, the probe was then lowered until the mid-point between receivers coincided with the mudline, recorded in the boring log, where the depth counter was reset to zero. Measurements were verified with a tape measure, and calculations recorded on a field log.

The probe was lowered to the bottom of the boreholes, stopping at 1.6 foot intervals to collect data, as summarized in Table 2. At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed. Gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and saved to disk before moving to the next depth.

Upon completion of the measurements, the probe was returned to the surface and the zero depth indication at the depth reference point was verified prior to removal from the borehole.

DATA ANALYSIS

Suspension Velocity Analysis

Using the proprietary OYO program PSLOG.EXE version 1.0, the recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 1.0 meter segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into a Microsoft Excel® template to complete the velocity calculations based on the arrival time picks made in PSLOG. The Microsoft Excel® analysis files accompany this report.

The P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked using PSLOG, and calculated and plotted in Microsoft Excel®, for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting the calculated and experimentally verified delay, in milliseconds, from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

As with the P-wave records, the recorded digital waveforms were analyzed to locate clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital Fast Fourier Transform – Inverse Fast Fourier Transform (FFT – IFFT) lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 4000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by borehole inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.33-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting the calculated and experimentally verified delay, in milliseconds, from the beginning of the record at the source trigger pulse to source impact.

Poisson’s Ratio, **ν**, was calculated in the Microsoft Excel® template using the following formula:

$$\boldsymbol{\nu} = \frac{\left(\frac{V_s}{V_p}\right)^2 - 0.5}{\left(\frac{V_s}{V_p}\right)^2 - 1.0}$$

Data and analyses were reviewed by a **GEO***Vision* Professional Geophysicist or Engineer as a component of the in-house data validation program.

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3 foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering of the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

RESULTS

Suspension Velocity Results

Suspension R1-R2 P- and S_H-wave velocities for boreholes B-1, B-2, and B-3 are plotted in Figures 4, 5, and 6, respectively. Suspension velocity data are also presented in Tables 3, 4, and 5, respectively. The Microsoft Excel® analysis files accompany this report.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figures A-1 through A-3 in Appendix A to aid in visual comparison. It should be noted that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.3 feet, creating a significant smoothing relative to the R1-R2 plots. The S-R1 velocity data are also presented in Tables A-1 through A-3 and included in the Microsoft Excel® analysis files, which also includes Poisson's Ratio calculations, tabulated data and plots.

SUMMARY

Discussion of Suspension Velocity Results

Suspension PS velocity data are ideally collected in uncased fluid filled boreholes drilled with rotary wash methods, as was the borehole for this project. Overall, Suspension PS velocity data quality is judged on 5 criteria, as summarized below.

	Criteria	B-1	B-2	B-3
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Yes.	Yes.	Yes.
2	Consistency between data from adjacent depth intervals.	Yes	Yes	Yes
3	Consistent relationship between P-wave and S_H - wave (excluding transition to saturated soils)	Yes Saturation occurs at about 40ft BGS	Yes All data is in saturated material (logged from a barge)	Yes Saturation occurs at about 25ft BGS
4	Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.	Overall, good data. Some sequences were difficult to interpret due to multiple arrivals in gravels or weathered rock	Excellent data set	Good data. Some sequences were difficult to interpret due to multiple arrivals in gravels or weathered rock
5	Consistency of profile between adjacent borings, if available.	Although the overall profiles are different, there are sequences that look very similar. The velocities in the soils are very similar, and the peak velocities in the rock are comparable.		

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analyses. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Velocity Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable with estimated precision of +/- 5%. Depth indications are very reliable with estimated precision of +/- 0.2 feet. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO***Vision* California Professional Geophysicist.

Prepared by

Emily

11/28/2016

Emily Feldman Date
Senior Staff Geophysicist
GEO*Vision* Geophysical Services

Reviewed and approved by

J. G. DIEHL
No. 30362

11/28/2016

John G. Diehl Date
California Professional Engineer 30362
GEO*Vision* Geophysical Services

∗ This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing, interpretation and reporting. All original field data files, field notes and observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations or ordinances.

Table 1. Borehole locations and logging dates

BOREHOLE DESIGNATION	DATES LOGGED	COORDINATES [(1)]		ELEVATION [(1)] (FEET)
		LATITUDE	LONGITUDE	
B-1	10/7/2016	684330.7	7646088.4	34.0
B-2	10/25/2016	684113.6	7646474.6	-37.7
B-3	9/23/2016	684157.8	7647283.1	32.0

(1) Survey locations State Plane North, Intl. Feet and NAVD88

Table 2. Logging dates and depth ranges

BOREHOLE NUMBER	TOOL AND RUN NUMBER	SURFACE CASING DEPTH (FEET)	DEPTH RANGE (FEET FROM SURFACE OR MUDLINE)	OPEN HOLE (FEET)	SAMPLE INTERVAL (FEET)	DATE LOGGED
B-1	SUSPENSION DOWN 01	N/A	1.64- 206.69	220	1.6	10/7/2016
B-2	SUSPENSION DOWN 01	41	41.01 – 134.51	148	1.6	10/25/2016
B-3	SUSPENSION DOWN 01	N/A	6.56 – 216.54	230	1.6	9/23/2016

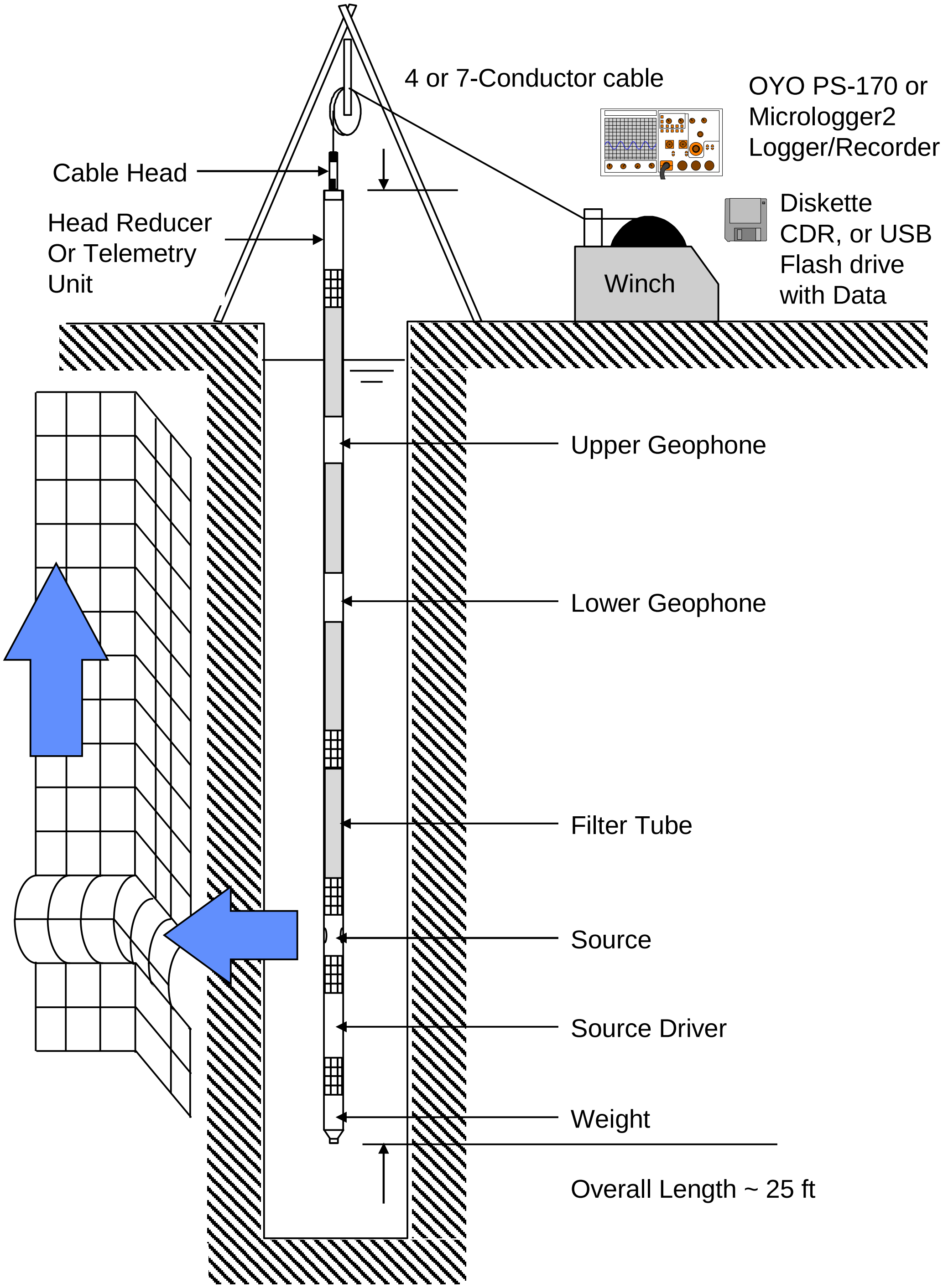

Figure 1: Concept illustration of P-S logging system

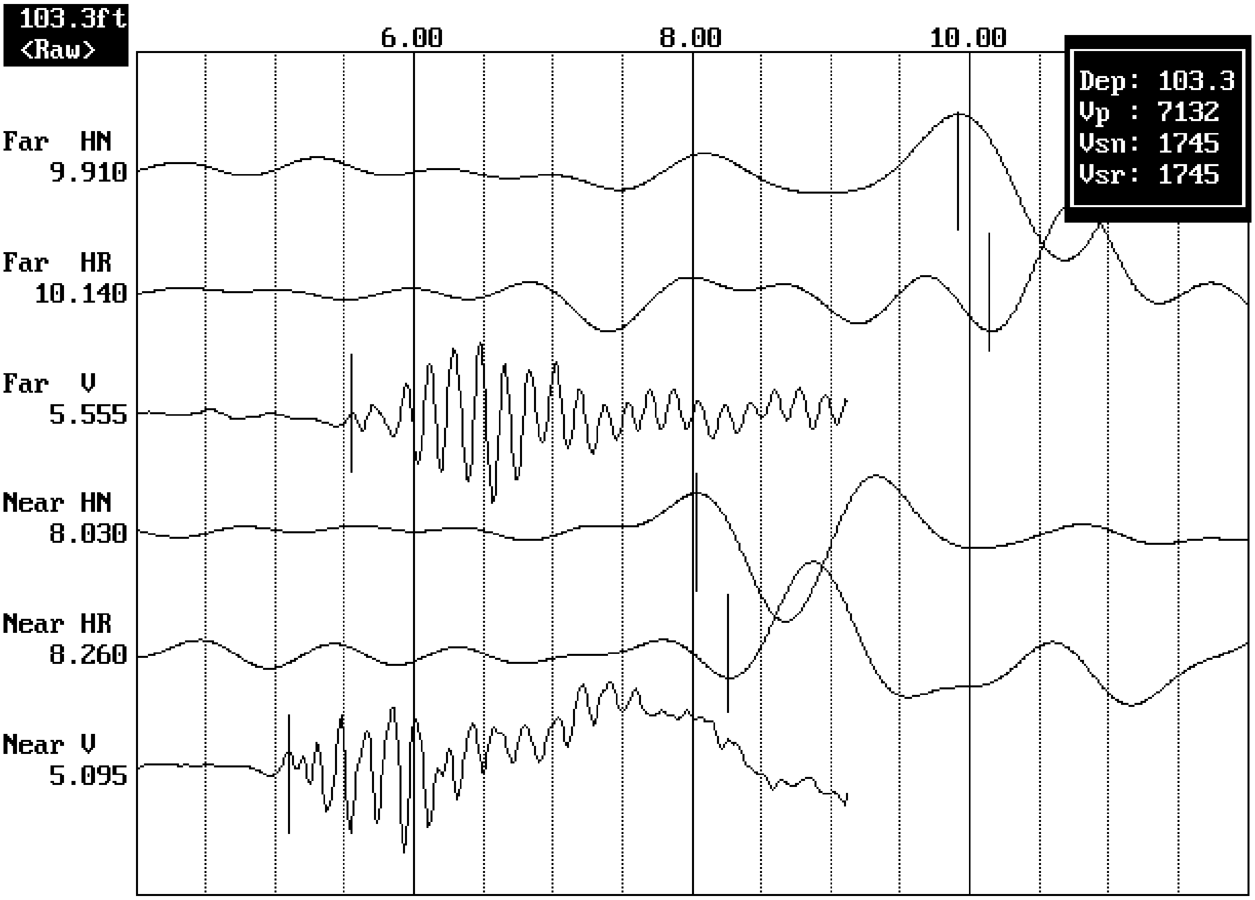

Figure 2: Example of filtered (1400 Hz lowpass) suspension record

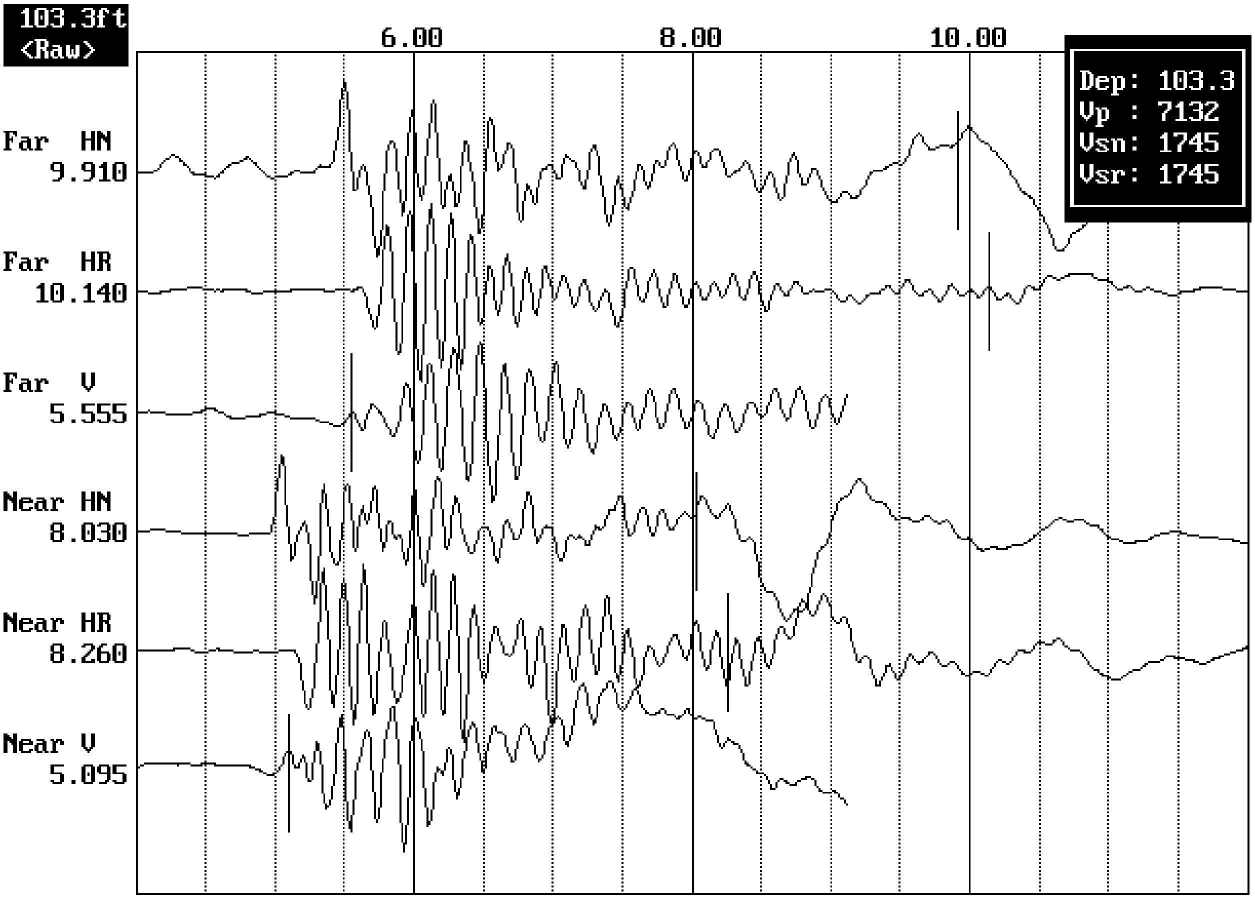

Figure 3. Example of unfiltered suspension record

BURNSIDE BRIDGE BOREHOLE B-1
Receiver to Receiver V_s and V_p Analysis

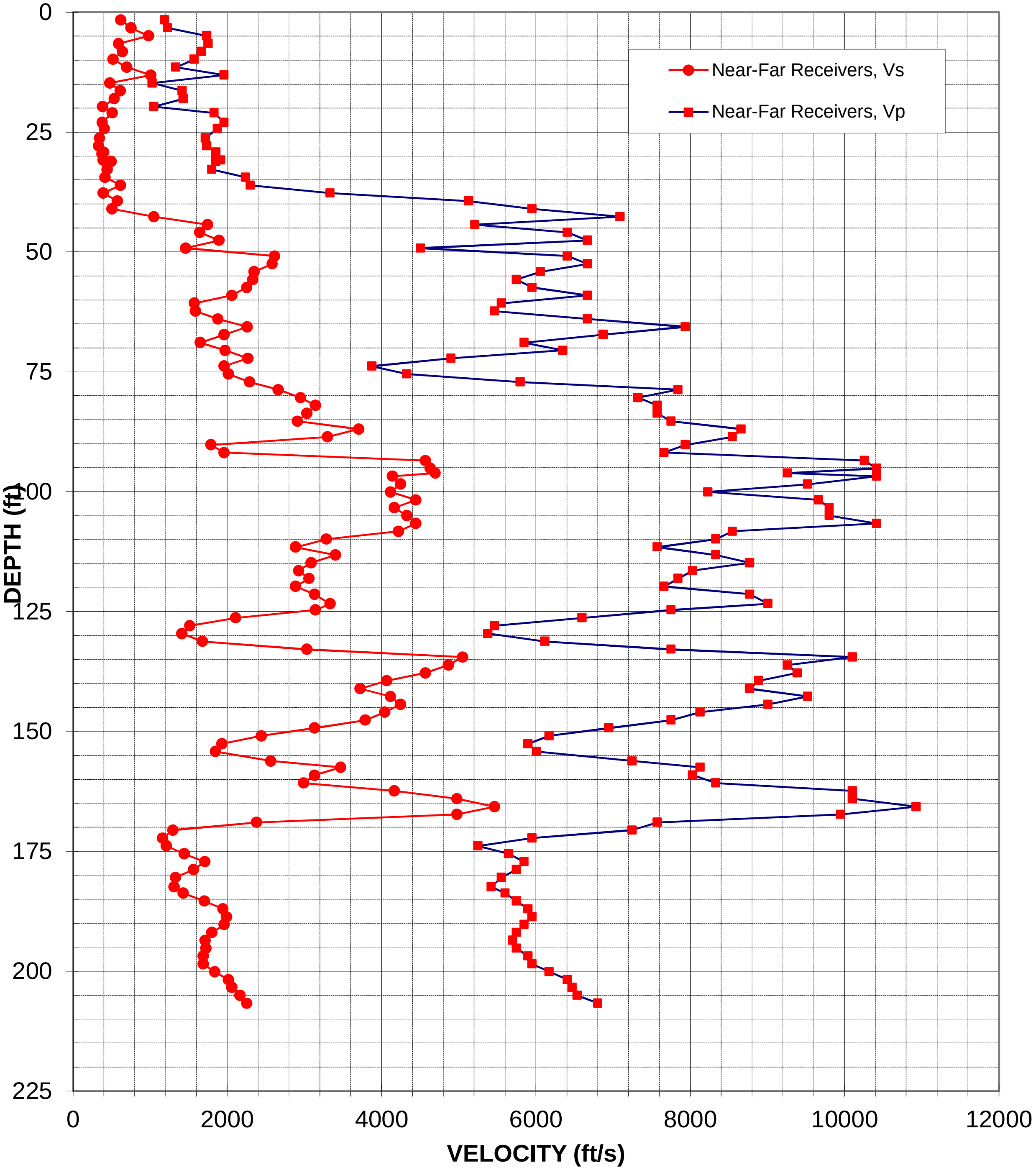

Figure 4: Borehole B-1, Suspension R1-R2 P- and S_H-wave velocities

Table 3. Borehole B-1, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
1.6	620	1190	0.31
3.3	750	1230	0.20
4.9	980	1740	0.27
6.6	590	1750	0.44
8.2	640	1670	0.41
9.8	520	1570	0.44
11.5	700	1330	0.31
13.1	1010	1960	0.32
14.8	480	1030	0.36
16.4	610	1420	0.39
18.0	540	1430	0.42
19.7	380	1050	0.42
21.0	510	1830	0.46
23.0	380	1960	0.48
24.3	410	1870	0.48
26.3	340	1720	0.48
27.9	330	1740	0.48
29.2	400	1850	0.48
29.5	380	1850	0.48
30.8	390	1920	0.48
31.2	500	1850	0.46
32.8	440	1800	0.47
34.5	420	2240	0.48
36.1	620	2300	0.46
37.7	390	3330	0.49
39.4	580	5130	0.49
41.0	510	5950	0.50
42.7	1050	7090	0.49
44.3	1750	5210	0.44
45.9	1650	6410	0.46
47.6	1890	6670	0.46
49.2	1460	4500	0.44
50.9	2610	6410	0.40
52.5	2580	6670	0.41
54.1	2350	6060	0.41
55.8	2330	5750	0.40
57.4	2250	5950	0.42
59.1	2060	6670	0.45

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
0.5	190	360	0.31
1.0	230	370	0.20
1.5	300	530	0.27
2.0	180	530	0.44
2.5	200	510	0.41
3.0	160	480	0.44
3.5	210	410	0.31
4.0	310	600	0.32
4.5	150	310	0.36
5.0	190	430	0.39
5.5	160	440	0.42
6.0	120	320	0.42
6.4	160	560	0.46
7.0	120	600	0.48
7.4	120	570	0.48
8.0	110	520	0.48
8.5	100	530	0.48
8.9	120	560	0.48
9.0	110	560	0.48
9.4	120	580	0.48
9.5	150	560	0.46
10.0	140	550	0.47
10.5	130	680	0.48
11.0	190	700	0.46
11.5	120	1020	0.49
12.0	180	1560	0.49
12.5	150	1810	0.50
13.0	320	2160	0.49
13.5	530	1590	0.44
14.0	500	1950	0.46
14.5	580	2030	0.46
15.0	450	1370	0.44
15.5	800	1950	0.40
16.0	790	2030	0.41
16.5	720	1850	0.41
17.0	710	1750	0.40
17.5	690	1810	0.42
18.0	630	2030	0.45

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
60.7	1570	5560	0.46
62.3	1590	5460	0.45
64.0	1880	6670	0.46
65.6	2260	7940	0.46
67.3	1960	6870	0.46
68.9	1650	5850	0.46
70.5	1970	6350	0.45
72.2	2270	4900	0.36
73.8	1960	3880	0.33
75.5	2010	4330	0.36
77.1	2290	5800	0.41
78.7	2660	7840	0.43
80.4	2950	7330	0.40
82.0	3140	7580	0.40
83.7	3030	7580	0.40
85.3	2910	7750	0.42
86.9	3700	8660	0.39
88.6	3300	8550	0.41
90.2	1790	7940	0.47
91.9	1960	7660	0.46
93.5	4570	10260	0.38
95.1	4630	10420	0.38
96.1	4690	9260	0.33
96.8	4140	10420	0.41
98.4	4250	9520	0.38
100.1	4120	8230	0.33
101.7	4440	9660	0.37
103.4	4170	9800	0.39
105.0	4330	9800	0.38
106.6	4440	10420	0.39
108.3	4220	8550	0.34
109.9	3280	8330	0.41
111.6	2890	7580	0.42
113.2	3400	8330	0.40
114.8	3090	8770	0.43
116.5	2920	8030	0.42
118.1	3060	7840	0.41
119.8	2890	7660	0.42
121.4	3130	8770	0.43
123.4	3330	9010	0.42

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
18.5	480	1690	0.46
19.0	480	1670	0.45
19.5	570	2030	0.46
20.0	690	2420	0.46
20.5	600	2090	0.46
21.0	500	1780	0.46
21.5	600	1940	0.45
22.0	690	1490	0.36
22.5	600	1180	0.33
23.0	610	1320	0.36
23.5	700	1770	0.41
24.0	810	2390	0.43
24.5	900	2230	0.40
25.0	960	2310	0.40
25.5	920	2310	0.40
26.0	890	2360	0.42
26.5	1130	2640	0.39
27.0	1010	2610	0.41
27.5	540	2420	0.47
28.0	600	2340	0.46
28.5	1390	3130	0.38
29.0	1410	3180	0.38
29.3	1430	2820	0.33
29.5	1260	3180	0.41
30.0	1290	2900	0.38
30.5	1250	2510	0.33
31.0	1350	2940	0.37
31.5	1270	2990	0.39
32.0	1320	2990	0.38
32.5	1350	3180	0.39
33.0	1290	2610	0.34
33.5	1000	2540	0.41
34.0	880	2310	0.42
34.5	1040	2540	0.40
35.0	940	2670	0.43
35.5	890	2450	0.42
36.0	930	2390	0.41
36.5	880	2340	0.42
37.0	950	2670	0.43
37.6	1020	2750	0.42

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
124.7	3140	7750	0.40
126.3	2110	6600	0.44
128.0	1520	5460	0.46
129.6	1410	5380	0.46
131.2	1680	6120	0.46
132.9	3030	7750	0.41
134.5	5050	10100	0.33
136.2	4870	9260	0.31
137.8	4570	9390	0.35
139.4	4070	8890	0.37
141.1	3720	8770	0.39
142.7	4120	9520	0.39
144.4	4250	9010	0.36
146.0	4040	8130	0.34
147.6	3790	7750	0.34
149.3	3130	6940	0.37
150.9	2440	6170	0.41
152.6	1930	5900	0.44
154.2	1850	6010	0.45
156.2	2560	7250	0.43
157.5	3470	8130	0.39
159.1	3130	8030	0.41
160.8	2990	8330	0.43
162.4	4170	10100	0.40
164.0	4980	10100	0.34
165.7	5460	10930	0.33
167.3	4980	9950	0.33
169.0	2380	7580	0.45
170.6	1290	7250	0.48
172.2	1160	5950	0.48
173.9	1210	5250	0.47
175.5	1440	5650	0.47
177.2	1710	5850	0.45
178.8	1560	5750	0.46
180.5	1330	5560	0.47
182.4	1310	5420	0.47
183.7	1430	5600	0.47
185.4	1710	5750	0.45
187.0	1940	5900	0.44
188.7	1990	5950	0.44

Metric Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
38.0	960	2360	0.40
38.5	640	2010	0.44
39.0	460	1670	0.46
39.5	430	1640	0.46
40.0	510	1860	0.46
40.5	920	2360	0.41
41.0	1540	3080	0.33
41.5	1480	2820	0.31
42.0	1390	2860	0.35
42.5	1240	2710	0.37
43.0	1140	2670	0.39
43.5	1250	2900	0.39
44.0	1290	2750	0.36
44.5	1230	2480	0.34
45.0	1150	2360	0.34
45.5	950	2120	0.37
46.0	740	1880	0.41
46.5	590	1800	0.44
47.0	560	1830	0.45
47.6	780	2210	0.43
48.0	1060	2480	0.39
48.5	950	2450	0.41
49.0	910	2540	0.43
49.5	1270	3080	0.40
50.0	1520	3080	0.34
50.5	1670	3330	0.33
51.0	1520	3030	0.33
51.5	730	2310	0.45
52.0	390	2210	0.48
52.5	350	1810	0.48
53.0	370	1600	0.47
53.5	440	1720	0.47
54.0	520	1780	0.45
54.5	480	1750	0.46
55.0	410	1690	0.47
55.6	400	1650	0.47
56.0	440	1710	0.47
56.5	520	1750	0.45
57.0	590	1800	0.44
57.5	610	1810	0.44

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
190.3	1960	5850	0.44
191.9	1800	5750	0.45
193.6	1710	5700	0.45
195.2	1720	5750	0.45
196.9	1690	5900	0.46
198.5	1690	5950	0.46
200.1	1840	6170	0.45
201.8	2010	6410	0.45
203.4	2060	6470	0.44
205.1	2160	6540	0.44
206.7	2250	6800	0.44

Metric Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
58.0	600	1780	0.44
58.5	550	1750	0.45
59.0	520	1740	0.45
59.5	530	1750	0.45
60.0	510	1800	0.46
60.5	510	1810	0.46
61.0	560	1880	0.45
61.5	610	1950	0.45
62.0	630	1970	0.44
62.5	660	1990	0.44
63.0	690	2070	0.44

BURNSIDE BRIDGE BORING B-2
Receiver to Receiver V_s and V_p Analysis

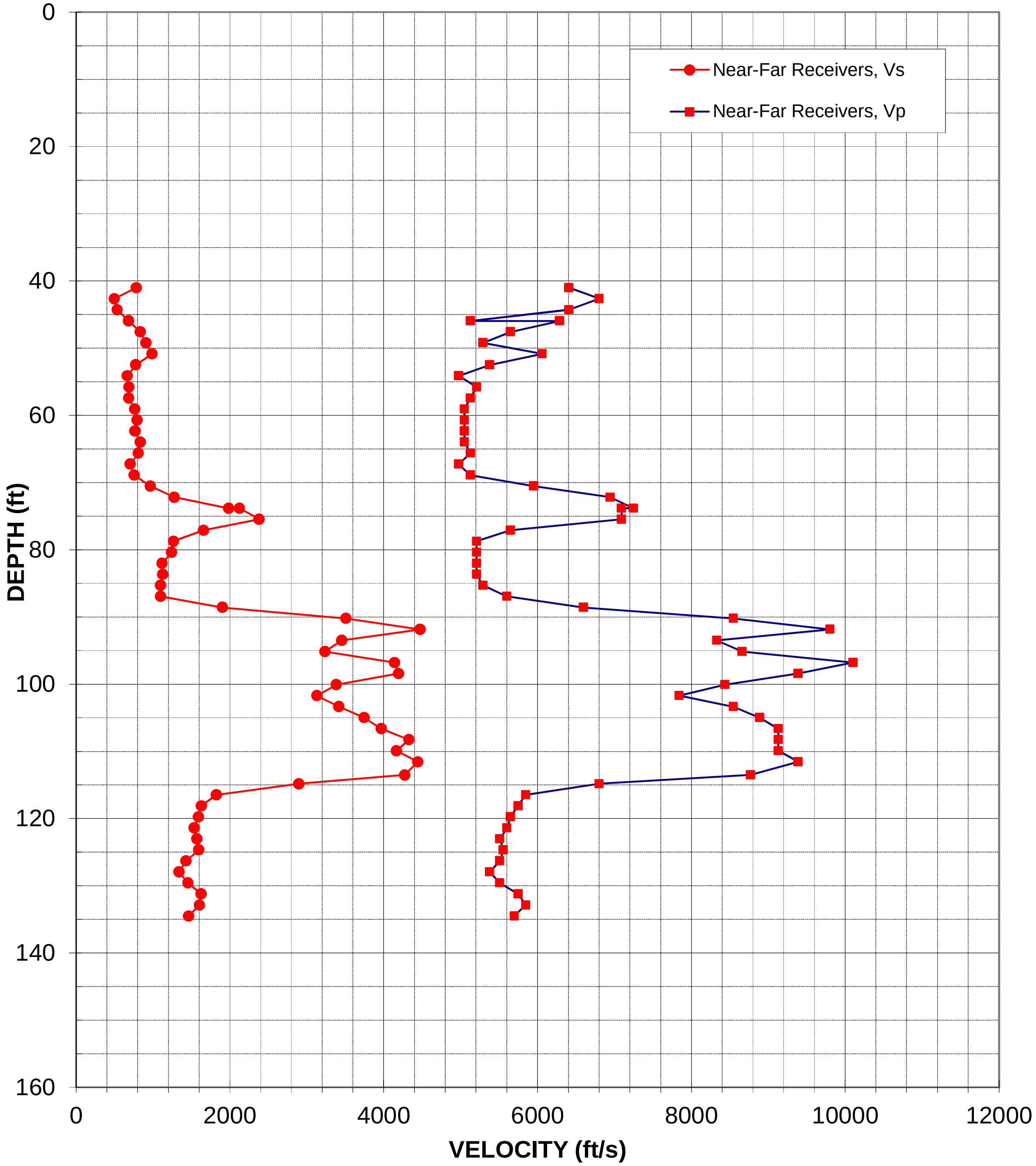

Figure 5: Borehole B-2, Suspension R1-R2 P- and S_H-wave velocities

Table 4. Borehole B-2, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole B-2

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
41.0	780	6410	0.49
42.7	500	6800	0.50
44.3	540	6410	0.50
45.9	680	5130	0.49
45.9	680	6290	0.49
47.6	840	5650	0.49
49.2	910	5290	0.48
50.9	990	6060	0.49
52.5	780	5380	0.49
54.1	670	4980	0.49
55.8	690	5210	0.49
57.4	680	5130	0.49
59.1	760	5050	0.49
60.7	790	5050	0.49
62.3	770	5050	0.49
62.3	770	5050	0.49
64.0	830	5050	0.49
65.6	810	5130	0.49
67.3	700	4980	0.49
68.9	760	5130	0.49
70.5	970	5950	0.49
72.2	1280	6940	0.48
73.8	1980	7250	0.46
73.8	2120	7090	0.45
75.5	2380	7090	0.44
77.1	1660	5650	0.45
78.7	1270	5210	0.47
80.4	1240	5210	0.47
82.0	1120	5210	0.48
83.7	1130	5210	0.48
85.3	1100	5290	0.48
86.9	1100	5600	0.48
88.6	1900	6600	0.45
90.2	3510	8550	0.40
91.9	4470	9800	0.37
93.5	3450	8330	0.40
95.1	3240	8660	0.42
96.8	4140	10100	0.40

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
12.5	240	1950	0.49
13.0	150	2070	0.50
13.5	160	1950	0.50
14.0	210	1560	0.49
14.0	210	1920	0.49
14.5	250	1720	0.49
15.0	280	1610	0.48
15.5	300	1850	0.49
16.0	240	1640	0.49
16.5	200	1520	0.49
17.0	210	1590	0.49
17.5	210	1560	0.49
18.0	230	1540	0.49
18.5	240	1540	0.49
19.0	230	1540	0.49
19.0	230	1540	0.49
19.5	250	1540	0.49
20.0	250	1560	0.49
20.5	210	1520	0.49
21.0	230	1560	0.49
21.5	290	1810	0.49
22.0	390	2120	0.48
22.5	600	2210	0.46
22.5	650	2160	0.45
23.0	730	2160	0.44
23.5	510	1720	0.45
24.0	390	1590	0.47
24.5	380	1590	0.47
25.0	340	1590	0.48
25.5	340	1590	0.48
26.0	340	1610	0.48
26.5	330	1710	0.48
27.0	580	2010	0.45
27.5	1070	2610	0.40
28.0	1360	2990	0.37
28.5	1050	2540	0.40
29.0	990	2640	0.42
29.5	1260	3080	0.40

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-2

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
98.4	4190	9390	0.38
100.1	3380	8440	0.40
101.7	3130	7840	0.41
103.4	3420	8550	0.40
105.0	3750	8890	0.39
106.6	3970	9130	0.38
108.3	4330	9130	0.36
109.9	4170	9130	0.37
111.6	4440	9390	0.36
113.5	4270	8770	0.34
114.8	2900	6800	0.39
116.5	1830	5850	0.45
118.1	1630	5750	0.46
119.8	1590	5650	0.46
121.4	1540	5600	0.46
123.0	1570	5510	0.46
124.7	1590	5560	0.46
126.3	1430	5510	0.46
128.0	1340	5380	0.47
129.6	1460	5510	0.46
131.2	1630	5750	0.46
132.9	1600	5850	0.46
134.5	1470	5700	0.46

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
30.0	1280	2860	0.38
30.5	1030	2570	0.40
31.0	950	2390	0.41
31.5	1040	2610	0.40
32.0	1140	2710	0.39
32.5	1210	2780	0.38
33.0	1320	2780	0.36
33.5	1270	2780	0.37
34.0	1350	2860	0.36
34.6	1300	2670	0.34
35.0	880	2070	0.39
35.5	560	1780	0.45
36.0	500	1750	0.46
36.5	480	1720	0.46
37.0	470	1710	0.46
37.5	480	1680	0.46
38.0	490	1690	0.46
38.5	440	1680	0.46
39.0	410	1640	0.47
39.5	440	1680	0.46
40.0	500	1750	0.46
40.5	490	1780	0.46
41.0	450	1740	0.46

BURNSIDE BRIDGE BOREHOLE B-3
Receiver to Receiver V_s and V_p Analysis

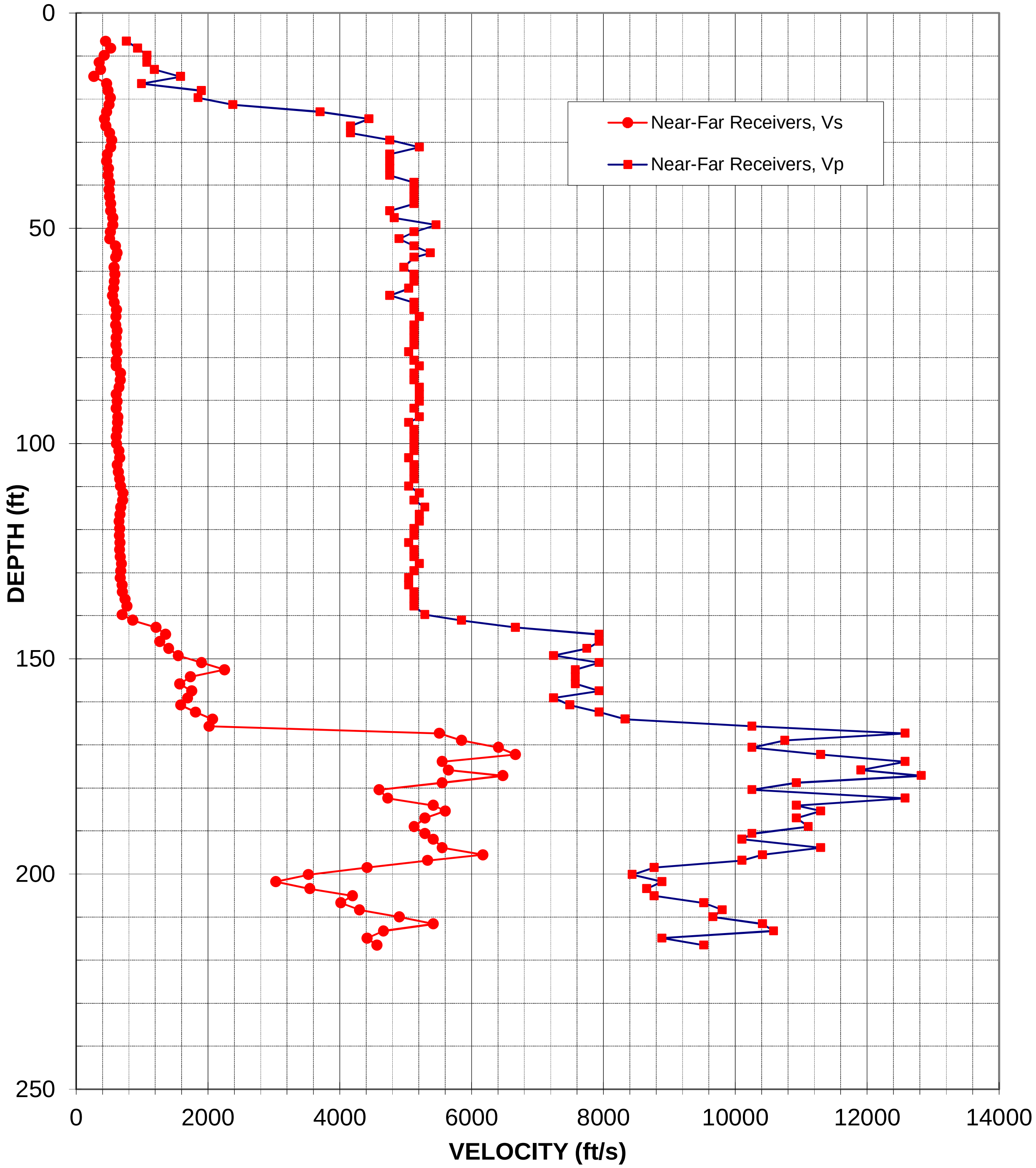

Figure 6: Borehole B-3, Suspension R1-R2 P- and SH-wave velocities

Table 5. Borehole B-3, Suspension R1-R2 depths and P- and S_H-wave velocities

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
6.6	450	760	0.23
8.2	520	940	0.27
9.8	430	1080	0.41
11.5	350	1080	0.44
13.1	370	1190	0.45
14.8	270	1590	0.49
16.4	460	1000	0.36
18.0	480	1900	0.47
19.7	520	1850	0.46
21.3	500	2380	0.48
23.0	460	3700	0.49
24.6	430	4440	0.50
26.3	450	4170	0.49
27.9	510	4170	0.49
29.5	540	4760	0.49
31.2	530	5210	0.49
32.8	470	4760	0.49
34.5	460	4760	0.50
36.1	490	4760	0.49
37.7	490	4760	0.49
39.4	510	5130	0.49
41.0	500	5130	0.50
42.7	510	5130	0.50
44.3	530	5130	0.49
45.9	520	4760	0.49
47.6	560	4830	0.49
49.2	560	5460	0.49
50.9	520	5130	0.49
52.5	510	4900	0.49
54.1	600	5130	0.49
55.8	630	5380	0.49
56.8	600	5130	0.49
59.1	580	4980	0.49
60.7	590	5130	0.49
62.3	580	5130	0.49
64.0	570	5050	0.49

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
2.0	140	230	0.23
2.5	160	290	0.27
3.0	130	330	0.41
3.5	110	330	0.44
4.0	110	360	0.45
4.5	80	480	0.49
5.0	140	300	0.36
5.5	150	580	0.47
6.0	160	560	0.46
6.5	150	730	0.48
7.0	140	1130	0.49
7.5	130	1350	0.50
8.0	140	1270	0.49
8.5	150	1270	0.49
9.0	160	1450	0.49
9.5	160	1590	0.49
10.0	140	1450	0.49
10.5	140	1450	0.50
11.0	150	1450	0.49
11.5	150	1450	0.49
12.0	160	1560	0.49
12.5	150	1560	0.50
13.0	160	1560	0.50
13.5	160	1560	0.49
14.0	160	1450	0.49
14.5	170	1470	0.49
15.0	170	1670	0.49
15.5	160	1560	0.49
16.0	160	1490	0.49
16.5	180	1560	0.49
17.0	190	1640	0.49
17.3	180	1560	0.49
18.0	180	1520	0.49
18.5	180	1560	0.49
19.0	180	1560	0.49
19.5	170	1540	0.49

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
65.6	560	4760	0.49
67.3	580	5130	0.49
68.9	610	5130	0.49
70.5	610	5210	0.49
72.5	600	5130	0.49
73.8	620	5130	0.49
75.5	610	5130	0.49
77.1	610	5130	0.49
78.7	620	5050	0.49
80.7	610	5130	0.49
82.0	610	5210	0.49
83.7	680	5130	0.49
85.3	670	5130	0.49
86.9	650	5210	0.49
88.6	610	5210	0.49
90.2	630	5210	0.49
91.9	610	5130	0.49
93.8	630	5210	0.49
95.1	630	5050	0.49
96.8	620	5130	0.49
98.4	610	5130	0.49
100.1	610	5130	0.49
101.7	650	5130	0.49
103.4	660	5050	0.49
105.0	630	5130	0.49
106.6	640	5130	0.49
108.3	660	5130	0.49
109.9	680	5050	0.49
111.6	710	5210	0.49
113.2	710	5130	0.49
114.8	680	5290	0.49
116.5	670	5210	0.49
118.1	650	5210	0.49
119.8	660	5130	0.49
121.4	660	5130	0.49
123.0	670	5050	0.49
124.7	660	5130	0.49
126.3	670	5130	0.49
128.0	690	5210	0.49
129.6	680	5130	0.49

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
20.0	170	1450	0.49
20.5	180	1560	0.49
21.0	190	1560	0.49
21.5	180	1590	0.49
22.1	180	1560	0.49
22.5	190	1560	0.49
23.0	190	1560	0.49
23.5	180	1560	0.49
24.0	190	1540	0.49
24.6	190	1560	0.49
25.0	190	1590	0.49
25.5	210	1560	0.49
26.0	210	1560	0.49
26.5	200	1590	0.49
27.0	190	1590	0.49
27.5	190	1590	0.49
28.0	190	1560	0.49
28.6	190	1590	0.49
29.0	190	1540	0.49
29.5	190	1560	0.49
30.0	190	1560	0.49
30.5	190	1560	0.49
31.0	200	1560	0.49
31.5	200	1540	0.49
32.0	190	1560	0.49
32.5	200	1560	0.49
33.0	200	1560	0.49
33.5	210	1540	0.49
34.0	220	1590	0.49
34.5	220	1560	0.49
35.0	210	1610	0.49
35.5	200	1590	0.49
36.0	200	1590	0.49
36.5	200	1560	0.49
37.0	200	1560	0.49
37.5	200	1540	0.49
38.0	200	1560	0.49
38.5	210	1560	0.49
39.0	210	1590	0.49
39.5	210	1560	0.49

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
131.2	670	5050	0.49
132.9	700	5050	0.49
134.5	710	5130	0.49
136.2	740	5130	0.49
137.8	780	5130	0.49
139.8	700	5290	0.49
141.1	860	5850	0.49
142.7	1210	6670	0.48
144.4	1360	7940	0.48
146.0	1270	7940	0.49
147.6	1410	7750	0.48
149.3	1550	7250	0.48
150.9	1900	7940	0.47
152.6	2250	7580	0.45
154.2	1740	7580	0.47
155.8	1570	7580	0.48
157.5	1750	7940	0.47
159.1	1690	7250	0.47
160.8	1590	7490	0.48
162.4	1810	7940	0.47
164.0	2070	8330	0.47
165.7	2020	10260	0.48
167.3	5510	12580	0.38
169.0	5850	10750	0.29
170.6	6410	10260	0.18
172.2	6670	11300	0.23
173.9	5560	12580	0.38
175.9	5650	11900	0.35
177.2	6470	12820	0.33
178.8	5560	10930	0.33
180.5	4600	10260	0.37
182.4	4730	12580	0.42
184.1	5420	10930	0.34
185.4	5600	11300	0.34
187.0	5290	10930	0.35
189.0	5130	11110	0.36
190.6	5290	10260	0.32
191.9	5420	10100	0.30
193.9	5560	11300	0.34
195.5	6170	10420	0.23

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
40.0	210	1540	0.49
40.5	210	1540	0.49
41.0	220	1560	0.49
41.5	230	1560	0.49
42.0	240	1560	0.49
42.6	210	1610	0.49
43.0	260	1780	0.49
43.5	370	2030	0.48
44.0	410	2420	0.48
44.5	390	2420	0.49
45.0	430	2360	0.48
45.5	470	2210	0.48
46.0	580	2420	0.47
46.5	690	2310	0.45
47.0	530	2310	0.47
47.5	480	2310	0.48
48.0	530	2420	0.47
48.5	520	2210	0.47
49.0	480	2280	0.48
49.5	550	2420	0.47
50.0	630	2540	0.47
50.5	620	3130	0.48
51.0	1680	3830	0.38
51.5	1780	3280	0.29
52.0	1950	3130	0.18
52.5	2030	3440	0.23
53.0	1690	3830	0.38
53.6	1720	3630	0.35
54.0	1970	3910	0.33
54.5	1690	3330	0.33
55.0	1400	3130	0.37
55.6	1440	3830	0.42
56.1	1650	3330	0.34
56.5	1710	3440	0.34
57.0	1610	3330	0.35
57.6	1560	3390	0.36
58.1	1610	3130	0.32
58.5	1650	3080	0.30
59.1	1690	3440	0.34
59.6	1880	3180	0.23

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Receiver-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
196.9	5330	10100	0.31
198.5	4420	8770	0.33
200.1	3530	8440	0.39
201.8	3030	8890	0.43
203.4	3550	8660	0.40
205.1	4190	8770	0.35
206.7	4020	9520	0.39
208.3	4300	9800	0.38
210.0	4900	9660	0.33
211.6	5420	10420	0.31
213.3	4660	10580	0.38
214.9	4420	8890	0.34
216.5	4570	9520	0.35

Metric Units			
Depth at Midpoint Between Receivers	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
60.0	1630	3080	0.31
60.5	1350	2670	0.33
61.0	1080	2570	0.39
61.5	920	2710	0.43
62.0	1080	2640	0.40
62.5	1280	2670	0.35
63.0	1220	2900	0.39
63.5	1310	2990	0.38
64.0	1490	2940	0.33
64.5	1650	3180	0.31
65.0	1420	3230	0.38
65.5	1350	2710	0.34
66.0	1390	2900	0.35

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

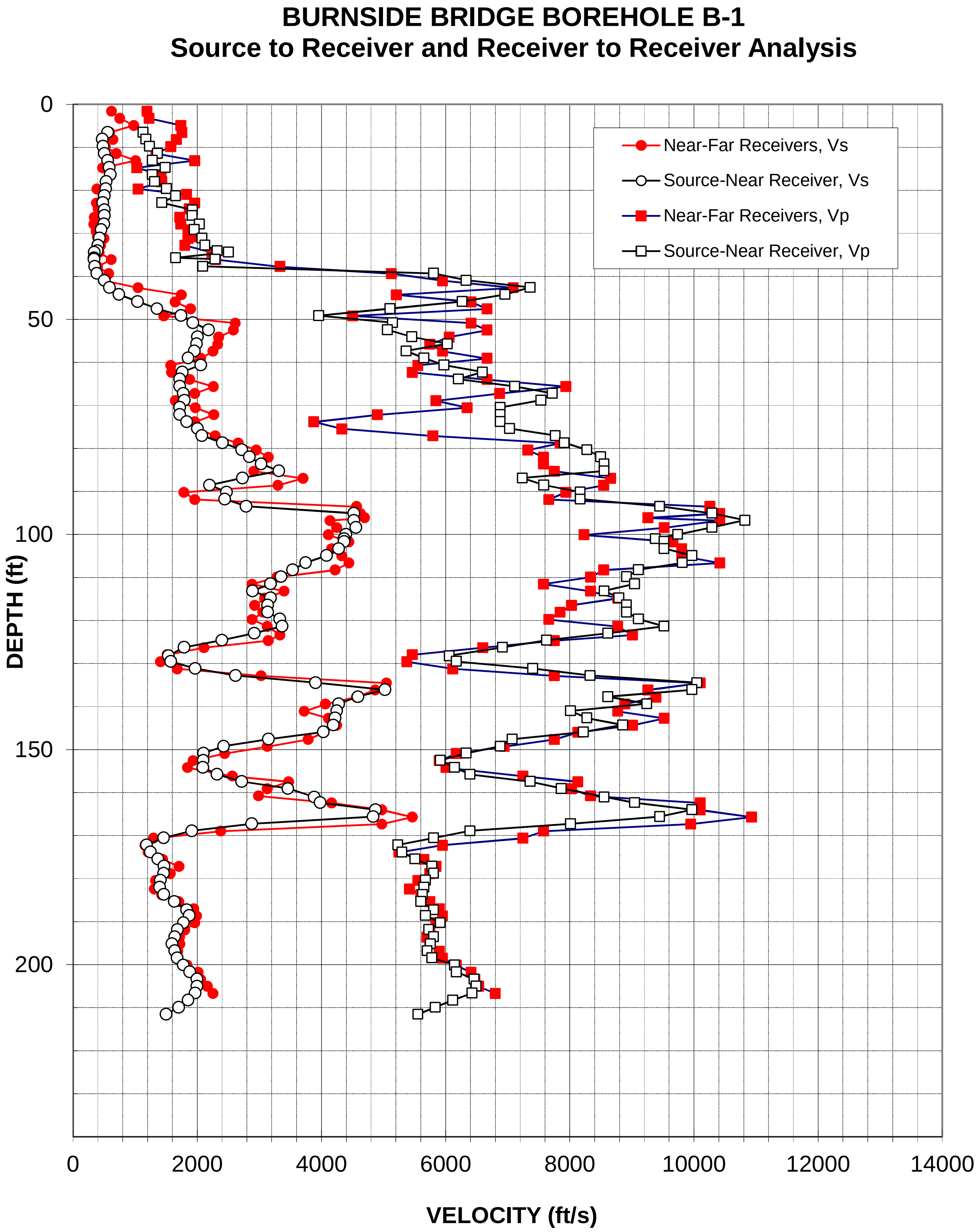

Figure A-1: Borehole B-1, Suspension S-R1 P- and S_H-wave velocities

Table A-1. Borehole B-1, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
6.5	560	1130	0.34
8.1	470	1170	0.40
9.8	480	1230	0.41
11.4	500	1360	0.42
13.0	560	1280	0.38
14.7	580	1490	0.41
16.3	600	1280	0.36
18.0	540	1320	0.40
19.6	530	1500	0.43
21.2	500	1650	0.45
22.9	480	1430	0.44
24.5	500	1920	0.46
25.8	510	1920	0.46
27.8	500	2040	0.47
29.1	450	1950	0.47
31.1	430	2080	0.48
32.7	400	2120	0.48
34.0	390	2320	0.49
34.4	340	2500	0.49
35.7	340	1650	0.48
36.0	340	2290	0.49
37.6	350	2090	0.49
39.3	380	5810	0.50
40.9	500	6330	0.50
42.6	590	7360	0.50
44.2	740	6960	0.49
45.8	1040	6270	0.49
47.5	1350	5100	0.46
49.1	1740	3960	0.38
50.8	1930	5150	0.42
52.4	2180	5060	0.39
54.0	2000	5460	0.42
55.7	1990	6030	0.44
57.3	1950	5360	0.42
59.0	1850	5650	0.44
60.6	2060	5970	0.43
62.2	1760	6590	0.46

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
2.0	170	340	0.34
2.5	140	360	0.40
3.0	150	380	0.41
3.5	150	410	0.42
4.0	170	390	0.38
4.5	180	450	0.41
5.0	180	390	0.36
5.5	160	400	0.40
6.0	160	460	0.43
6.5	150	500	0.45
7.0	150	440	0.44
7.5	150	590	0.46
7.9	150	580	0.46
8.5	150	620	0.47
8.9	140	600	0.47
9.5	130	630	0.48
10.0	120	650	0.48
10.4	120	710	0.49
10.5	100	760	0.49
10.9	100	500	0.48
11.0	100	700	0.49
11.5	110	640	0.49
12.0	120	1770	0.50
12.5	150	1930	0.50
13.0	180	2240	0.50
13.5	230	2120	0.49
14.0	320	1910	0.49
14.5	410	1560	0.46
15.0	530	1210	0.38
15.5	590	1570	0.42
16.0	670	1540	0.39
16.5	610	1660	0.42
17.0	610	1840	0.44
17.5	600	1640	0.42
18.0	560	1720	0.44
18.5	630	1820	0.43
19.0	540	2010	0.46

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
63.9	1720	6210	0.46
65.5	1720	7110	0.47
67.2	1770	7720	0.47
68.8	1790	7540	0.47
70.5	1720	6880	0.47
72.1	1720	6880	0.47
73.7	1830	6880	0.46
75.4	2000	7030	0.46
77.0	2080	7770	0.46
78.7	2410	7910	0.45
80.3	2720	8270	0.44
81.9	2840	8500	0.44
83.6	3030	8550	0.43
85.2	3310	8550	0.41
86.9	2730	7230	0.42
88.5	2200	7580	0.45
90.1	2470	8170	0.45
91.8	2440	8170	0.45
93.4	2790	9450	0.45
95.1	4520	10290	0.38
96.7	4520	10820	0.39
98.3	4550	10290	0.38
100.0	4400	9740	0.37
101.0	4370	9380	0.36
101.6	4370	9520	0.37
103.3	4280	9520	0.37
104.9	4080	9970	0.40
106.5	3750	9810	0.41
108.2	3540	9110	0.41
109.8	3350	8920	0.42
111.5	3180	9040	0.43
113.1	2890	8550	0.44
114.7	3180	8790	0.42
116.4	3130	8920	0.43
118.0	3130	8920	0.43
119.7	3330	9110	0.42
121.3	3370	9520	0.43
122.9	2920	8610	0.44
124.6	2400	7630	0.45
126.2	1790	6920	0.46

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
19.5	520	1890	0.46
20.0	520	2170	0.47
20.5	540	2350	0.47
21.0	550	2300	0.47
21.5	520	2100	0.47
22.0	520	2100	0.47
22.5	560	2100	0.46
23.0	610	2140	0.46
23.5	630	2370	0.46
24.0	730	2410	0.45
24.5	830	2520	0.44
25.0	870	2590	0.44
25.5	920	2610	0.43
26.0	1010	2610	0.41
26.5	830	2210	0.42
27.0	670	2310	0.45
27.5	750	2490	0.45
28.0	740	2490	0.45
28.5	850	2880	0.45
29.0	1380	3140	0.38
29.5	1380	3300	0.39
30.0	1390	3140	0.38
30.5	1340	2970	0.37
30.8	1330	2860	0.36
31.0	1330	2900	0.37
31.5	1300	2900	0.37
32.0	1240	3040	0.40
32.5	1140	2990	0.41
33.0	1080	2780	0.41
33.5	1020	2720	0.42
34.0	970	2760	0.43
34.5	880	2610	0.44
35.0	970	2680	0.42
35.5	960	2720	0.43
36.0	960	2720	0.43
36.5	1020	2780	0.42
37.0	1030	2900	0.43
37.5	890	2630	0.44
38.0	730	2320	0.45
38.5	550	2110	0.46

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
128.2	1540	6060	0.47
129.5	1570	6180	0.47
131.1	1970	7400	0.46
132.8	2620	8330	0.45
134.4	3910	10050	0.41
136.1	5020	9970	0.33
137.7	4590	8610	0.30
139.3	4280	9240	0.36
141.0	4250	8010	0.30
142.6	4220	8270	0.32
144.3	4190	8850	0.36
145.9	4030	8220	0.34
147.6	3150	7070	0.38
149.2	2430	6880	0.43
150.8	2100	6330	0.44
152.5	2100	5920	0.43
154.1	2090	6150	0.43
155.8	2320	6390	0.42
157.4	2720	7360	0.42
159.0	3460	7860	0.38
161.0	3880	8550	0.37
162.3	3980	9040	0.38
164.0	4870	9970	0.34
165.6	4830	9450	0.32
167.2	2880	8010	0.43
168.9	1910	6390	0.45
170.5	1460	5810	0.47
172.2	1190	5230	0.47
173.8	1250	5300	0.47
175.4	1370	5500	0.47
177.1	1470	5780	0.47
178.7	1460	5810	0.47
180.4	1410	5680	0.47
182.0	1390	5650	0.47
183.6	1460	5630	0.46
185.3	1630	5600	0.45
187.2	1830	5810	0.44
188.6	1870	5680	0.44
190.2	1780	5920	0.45
191.8	1680	5730	0.45

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
39.1	470	1850	0.47
39.5	480	1880	0.47
40.0	600	2260	0.46
40.5	800	2540	0.45
41.0	1190	3060	0.41
41.5	1530	3040	0.33
42.0	1400	2630	0.30
42.5	1300	2820	0.36
43.0	1290	2440	0.30
43.5	1290	2520	0.32
44.0	1280	2700	0.36
44.5	1230	2510	0.34
45.0	960	2160	0.38
45.5	740	2100	0.43
46.0	640	1930	0.44
46.5	640	1800	0.43
47.0	640	1870	0.43
47.5	710	1950	0.42
48.0	830	2240	0.42
48.5	1050	2400	0.38
49.1	1180	2610	0.37
49.5	1210	2760	0.38
50.0	1480	3040	0.34
50.5	1470	2880	0.32
51.0	880	2440	0.43
51.5	580	1950	0.45
52.0	440	1770	0.47
52.5	360	1590	0.47
53.0	380	1610	0.47
53.5	420	1680	0.47
54.0	450	1760	0.47
54.5	440	1770	0.47
55.0	430	1730	0.47
55.5	420	1720	0.47
56.0	450	1720	0.46
56.5	500	1710	0.45
57.1	560	1770	0.44
57.5	570	1730	0.44
58.0	540	1800	0.45
58.5	510	1750	0.45

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-1

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
193.5	1640	5810	0.46
195.1	1590	5750	0.46
196.8	1640	5700	0.46
198.4	1680	5780	0.45
200.0	1770	6150	0.45
201.7	1880	6180	0.45
203.3	2000	6460	0.45
205.0	2000	6490	0.45
206.6	1970	6430	0.45
208.2	1850	6120	0.45
209.9	1700	5830	0.45
211.5	1500	5550	0.46

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
59.0	500	1770	0.46
59.5	490	1750	0.46
60.0	500	1740	0.46
60.5	510	1760	0.45
61.0	540	1870	0.45
61.5	570	1880	0.45
62.0	610	1970	0.45
62.5	610	1980	0.45
63.0	600	1960	0.45
63.5	560	1860	0.45
64.0	520	1780	0.45
64.5	460	1690	0.46

BURNSIDE BRIDGE BORING B-2
Source to Receiver and Receiver to Receiver Analysis

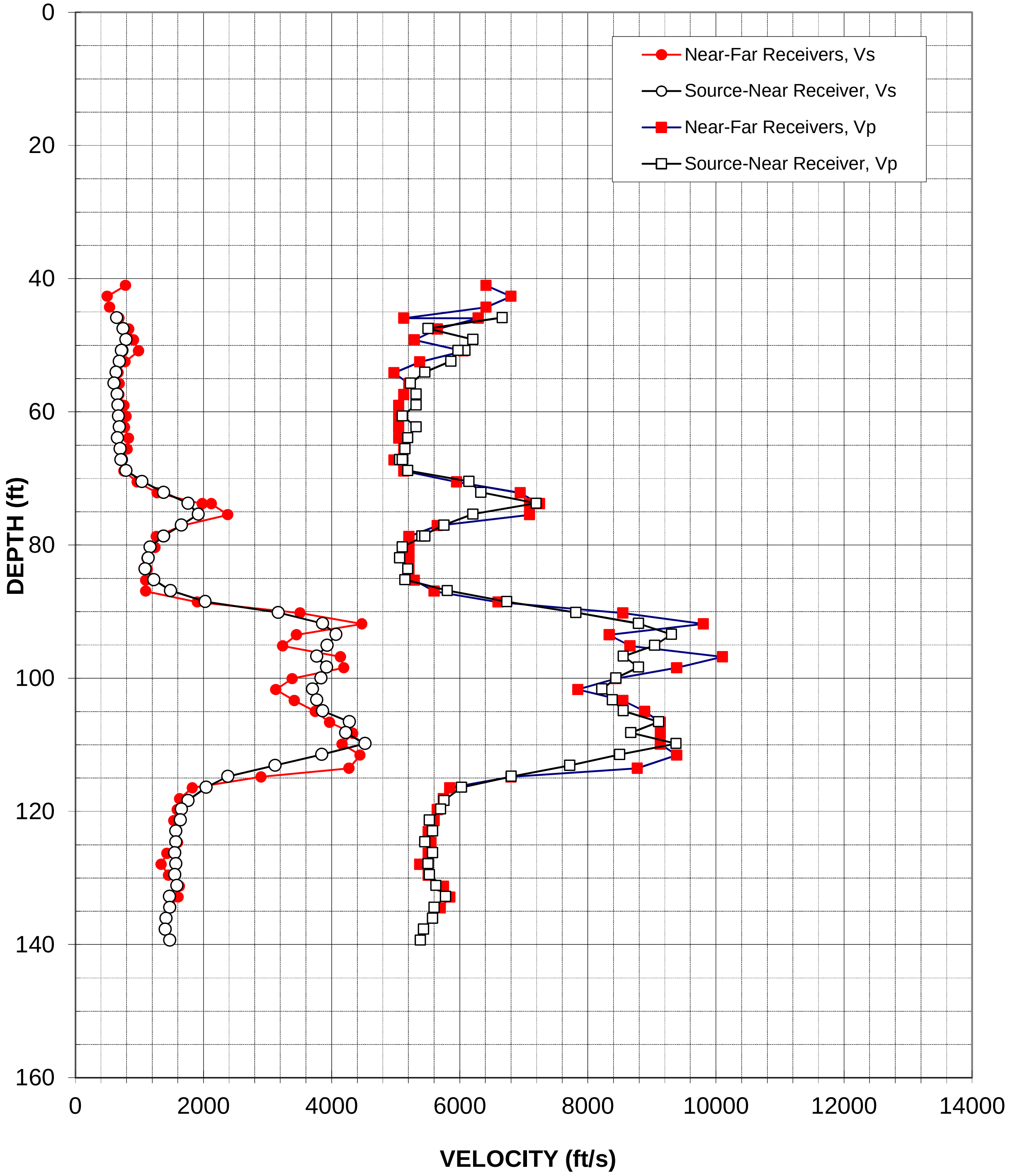

Figure A-2: Borehole B-2, Suspension S-R1 P- and S_H-wave velocities

Table A-2. Borehole B-2, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole B-2

American Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(ft)	(ft/s)	(ft/s)	
45.8	650	6660	0.50
47.5	740	5500	0.49
49.1	790	6210	0.49
50.8	730	6090	0.49
50.8	720	5970	0.49
52.4	690	5860	0.49
54.0	630	5460	0.49
55.7	610	5230	0.49
57.3	650	5320	0.49
59.0	660	5320	0.49
60.6	670	5100	0.49
62.2	690	5320	0.49
63.9	660	5190	0.49
65.5	700	5150	0.49
67.2	720	5060	0.49
67.2	710	5100	0.49
68.8	790	5190	0.49
70.5	1040	6150	0.49
72.1	1380	6330	0.48
73.7	1760	7190	0.47
75.4	1920	6210	0.45
77.0	1660	5750	0.45
78.7	1380	5410	0.47
78.7	1380	5460	0.47
80.3	1170	5100	0.47
81.9	1140	5060	0.47
83.6	1090	5190	0.48
85.2	1220	5150	0.47
86.9	1490	5810	0.46
88.5	2030	6730	0.45
90.1	3170	7810	0.40
91.8	3860	8790	0.38
93.4	4070	9310	0.38
95.1	3930	9040	0.38
96.7	3770	8550	0.38
98.3	3920	8790	0.38
100.0	3840	8440	0.37
101.6	3700	8220	0.37
103.3	3770	8380	0.37

Metric Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**	
(m)	(m/s)	(m/s)	
14.0	200	2030	0.50
14.5	230	1680	0.49
15.0	240	1890	0.49
15.5	220	1860	0.49
15.5	220	1820	0.49
16.0	210	1790	0.49
16.5	190	1660	0.49
17.0	180	1590	0.49
17.5	200	1620	0.49
18.0	200	1620	0.49
18.5	210	1560	0.49
19.0	210	1620	0.49
19.5	200	1580	0.49
20.0	210	1570	0.49
20.5	220	1540	0.49
20.5	220	1560	0.49
21.0	240	1580	0.49
21.5	320	1870	0.49
22.0	420	1930	0.48
22.5	540	2190	0.47
23.0	580	1890	0.45
23.5	510	1750	0.45
24.0	420	1650	0.47
24.0	420	1660	0.47
24.5	360	1560	0.47
25.0	350	1540	0.47
25.5	330	1580	0.48
26.0	370	1570	0.47
26.5	450	1770	0.46
27.0	620	2050	0.45
27.5	960	2380	0.40
28.0	1180	2680	0.38
28.5	1240	2840	0.38
29.0	1200	2760	0.38
29.5	1150	2610	0.38
30.0	1190	2680	0.38
30.5	1170	2570	0.37
31.0	1130	2510	0.37
31.5	1150	2560	0.37

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-2

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	**Velocity**		**Poisson's Ratio**	**Depth at Midpoint Between Source and Near Receiver**	**Velocity**		**Poisson's Ratio**
	V_s	**V_p**			**V_s**	**V_p**	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
104.9	3860	8550	0.37	32.0	1180	2610	0.37
106.5	4280	9110	0.36	32.5	1300	2780	0.36
108.2	4220	8670	0.34	33.0	1290	2640	0.34
109.8	4520	9380	0.35	33.5	1380	2860	0.35
111.5	3850	8500	0.37	34.0	1170	2590	0.37
113.1	3120	7720	0.40	34.5	950	2350	0.40
114.7	2380	6810	0.43	35.0	730	2070	0.43
116.4	2040	6030	0.44	35.5	620	1840	0.44
118.4	1760	5750	0.45	36.1	540	1750	0.45
119.7	1660	5700	0.45	36.5	510	1740	0.45
121.3	1640	5530	0.45	37.0	500	1690	0.45
122.9	1570	5580	0.46	37.5	480	1700	0.46
124.6	1570	5460	0.45	38.0	480	1660	0.45
126.2	1550	5580	0.46	38.5	470	1700	0.46
127.9	1570	5500	0.46	39.0	480	1680	0.46
129.5	1550	5530	0.46	39.5	470	1690	0.46
131.1	1590	5630	0.46	40.0	480	1720	0.46
132.8	1470	5780	0.47	40.5	450	1760	0.47
134.4	1470	5600	0.46	41.0	450	1710	0.46
136.1	1410	5580	0.47	41.5	430	1700	0.47
137.7	1400	5430	0.46	42.0	430	1660	0.46
139.3	1480	5390	0.46	42.5	450	1640	0.46

BURNSIDE BRIDGE BOREHOLE B-3
Source to Receiver and Receiver to Receiver Analysis

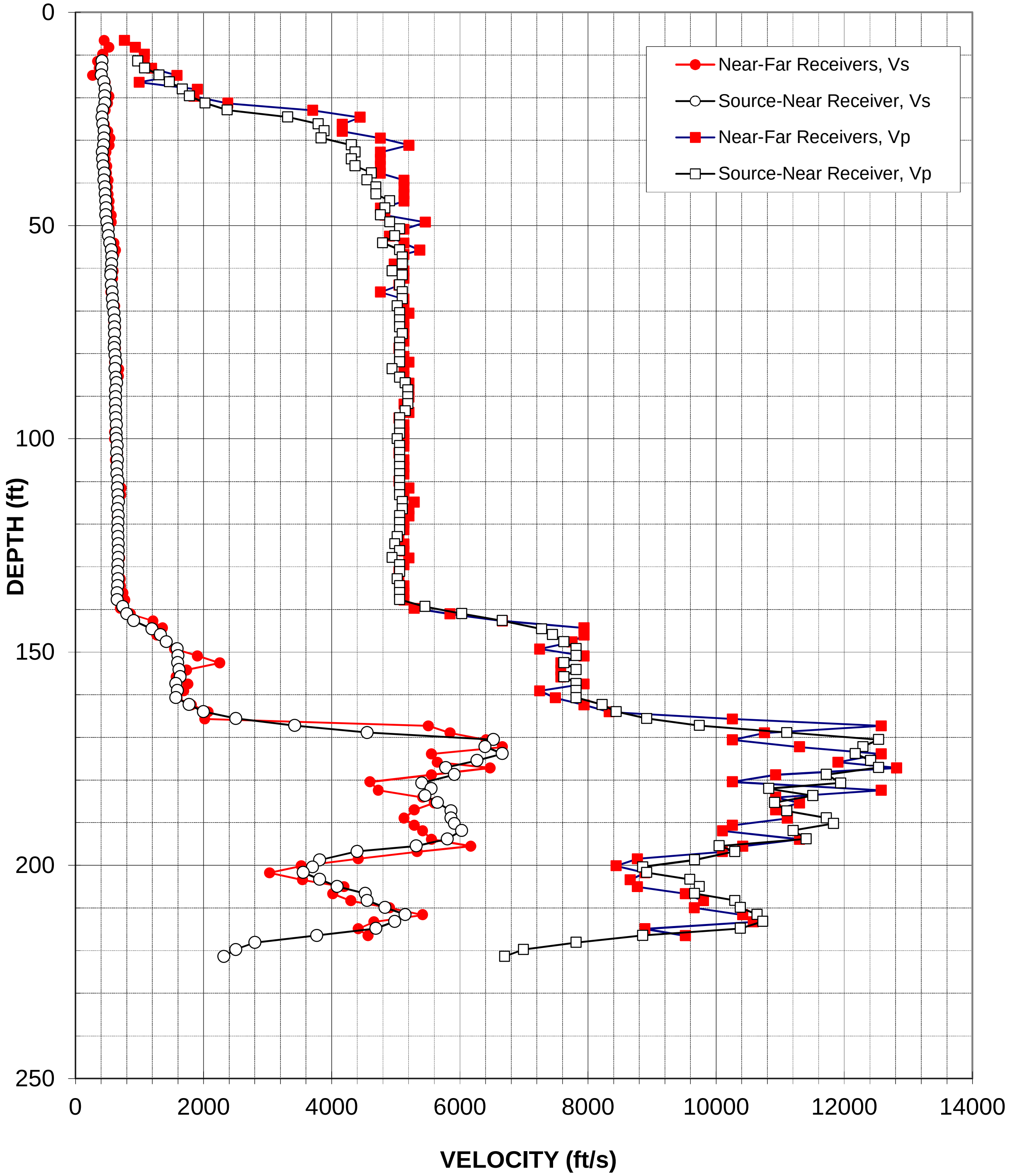

Figure A-3: Borehole B-3, Suspension S-R1 P- and S_H-wave velocities

Table A-3. Borehole B-3, S - R1 quality assurance analysis P- and S_H-wave data

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint	Velocity		Poisson's Ratio
Between Source and Near Receiver	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
11.4	420	980	0.39
13.0	410	1080	0.42
14.7	410	1310	0.45
16.3	440	1470	0.45
18.0	470	1670	0.46
19.6	460	1780	0.46
21.2	460	2020	0.47
22.9	430	2370	0.48
24.5	420	3310	0.49
26.2	430	3790	0.49
27.8	450	3880	0.49
29.4	450	3840	0.49
31.1	440	4310	0.49
32.7	420	4370	0.50
34.4	420	4310	0.50
36.0	440	4370	0.49
37.6	450	4620	0.50
39.3	440	4550	0.50
40.9	460	4690	0.50
42.6	460	4690	0.50
44.2	470	4910	0.50
45.8	480	4830	0.50
47.5	470	4760	0.49
49.1	490	4910	0.49
50.8	510	5060	0.49
52.4	520	4980	0.49
54.0	530	4800	0.49
55.7	560	5060	0.49
57.3	570	5100	0.49
59.0	570	5100	0.49
60.6	560	4950	0.49
61.6	550	5100	0.49
63.9	560	5060	0.49
65.5	580	5100	0.49
67.2	580	5100	0.49
68.8	590	5020	0.49
70.5	600	5060	0.49

Metric Units			
Depth at Midpoint	Velocity		Poisson's Ratio
Between Source and Near Receiver	V_s	V_p	
(m)	(m/s)	(m/s)	
3.5	130	300	0.39
4.0	130	330	0.42
4.5	120	400	0.45
5.0	140	450	0.45
5.5	140	510	0.46
6.0	140	540	0.46
6.5	140	620	0.47
7.0	130	720	0.48
7.5	130	1010	0.49
8.0	130	1160	0.49
8.5	140	1180	0.49
9.0	140	1170	0.49
9.5	130	1310	0.49
10.0	130	1330	0.50
10.5	130	1310	0.50
11.0	130	1330	0.49
11.5	140	1410	0.50
12.0	140	1390	0.50
12.5	140	1430	0.50
13.0	140	1430	0.50
13.5	140	1500	0.50
14.0	150	1470	0.50
14.5	140	1450	0.49
15.0	150	1500	0.49
15.5	160	1540	0.49
16.0	160	1520	0.49
16.5	160	1460	0.49
17.0	170	1540	0.49
17.5	170	1560	0.49
18.0	170	1560	0.49
18.5	170	1510	0.49
18.8	170	1560	0.49
19.5	170	1540	0.49
20.0	180	1560	0.49
20.5	180	1560	0.49
21.0	180	1530	0.49
21.5	180	1540	0.49

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
72.1	610	5060	0.49
73.7	610	5060	0.49
75.4	610	5100	0.49
77.3	610	5060	0.49
78.7	610	5060	0.49
80.3	620	5060	0.49
81.9	630	5060	0.49
83.6	620	4950	0.49
85.5	630	5060	0.49
86.9	640	5150	0.49
88.5	630	5190	0.49
90.1	630	5190	0.49
91.8	630	5190	0.49
93.4	630	5150	0.49
95.1	630	5060	0.49
96.7	640	5060	0.49
98.7	640	5060	0.49
100.0	640	5020	0.49
101.6	650	5060	0.49
103.3	640	5060	0.49
104.9	660	5060	0.49
106.5	650	5060	0.49
108.2	650	5060	0.49
109.8	670	5060	0.49
111.5	660	5060	0.49
113.1	670	5060	0.49
114.7	670	5100	0.49
116.4	660	5100	0.49
118.0	670	5060	0.49
119.7	670	5060	0.49
121.3	660	5060	0.49
122.9	670	5020	0.49
124.6	670	4980	0.49
126.2	670	5060	0.49
127.9	670	4950	0.49
129.5	670	5060	0.49
131.1	660	5060	0.49
132.8	670	5020	0.49
134.4	660	5060	0.49
136.1	650	5060	0.49

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
22.0	190	1540	0.49
22.5	190	1540	0.49
23.0	190	1560	0.49
23.6	190	1540	0.49
24.0	180	1540	0.49
24.5	190	1540	0.49
25.0	190	1540	0.49
25.5	190	1510	0.49
26.1	190	1540	0.49
26.5	200	1570	0.49
27.0	190	1580	0.49
27.5	190	1580	0.49
28.0	190	1580	0.49
28.5	190	1570	0.49
29.0	190	1540	0.49
29.5	190	1540	0.49
30.1	190	1540	0.49
30.5	190	1530	0.49
31.0	200	1540	0.49
31.5	200	1540	0.49
32.0	200	1540	0.49
32.5	200	1540	0.49
33.0	200	1540	0.49
33.5	200	1540	0.49
34.0	200	1540	0.49
34.5	200	1540	0.49
35.0	210	1560	0.49
35.5	200	1560	0.49
36.0	200	1540	0.49
36.5	200	1540	0.49
37.0	200	1540	0.49
37.5	200	1530	0.49
38.0	200	1520	0.49
38.5	200	1540	0.49
39.0	200	1510	0.49
39.5	200	1540	0.49
40.0	200	1540	0.49
40.5	200	1530	0.49
41.0	200	1540	0.49
41.5	200	1540	0.49

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
137.7	660	5060	0.49
139.3	740	5460	0.49
141.0	800	6030	0.49
142.6	910	6660	0.49
144.6	1190	7280	0.49
145.9	1330	7450	0.48
147.6	1420	7630	0.48
149.2	1590	7810	0.48
150.8	1600	7810	0.48
152.5	1590	7630	0.48
154.1	1620	7810	0.48
155.8	1640	7630	0.48
157.4	1570	7810	0.48
159.0	1590	7810	0.48
160.7	1570	7810	0.48
162.3	1770	8220	0.48
164.0	2000	8440	0.47
165.6	2500	8920	0.46
167.2	3420	9740	0.43
168.9	4550	11110	0.40
170.5	6530	12530	0.31
172.2	6390	12290	0.31
173.8	6660	12170	0.29
175.4	6270	12410	0.33
177.1	5780	12530	0.36
178.7	5920	11720	0.33
180.7	5410	11940	0.37
182.0	5550	10820	0.32
183.6	5460	11510	0.35
185.3	5650	10910	0.32
187.2	5860	11110	0.31
188.9	5860	11720	0.33
190.2	5920	11830	0.33
191.8	6030	11200	0.30
193.8	5810	11410	0.33
195.5	5320	10050	0.31
196.8	4400	10290	0.39
198.7	3810	9660	0.41
200.4	3700	8850	0.39
201.7	3560	8920	0.41

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
42.0	200	1540	0.49
42.5	230	1660	0.49
43.0	250	1840	0.49
43.5	280	2030	0.49
44.1	360	2220	0.49
44.5	400	2270	0.48
45.0	430	2320	0.48
45.5	480	2380	0.48
46.0	490	2380	0.48
46.5	490	2320	0.48
47.0	490	2380	0.48
47.5	500	2320	0.48
48.0	480	2380	0.48
48.5	480	2380	0.48
49.0	480	2380	0.48
49.5	540	2510	0.48
50.0	610	2570	0.47
50.5	760	2720	0.46
51.0	1040	2970	0.43
51.5	1390	3380	0.40
52.0	1990	3820	0.31
52.5	1950	3750	0.31
53.0	2030	3710	0.29
53.5	1910	3780	0.33
54.0	1760	3820	0.36
54.5	1800	3570	0.33
55.1	1650	3640	0.37
55.5	1690	3300	0.32
56.0	1660	3510	0.35
56.5	1720	3330	0.32
57.1	1790	3380	0.31
57.6	1790	3570	0.33
58.0	1800	3610	0.33
58.5	1840	3410	0.30
59.1	1770	3480	0.33
59.6	1620	3060	0.31
60.0	1340	3140	0.39
60.6	1160	2950	0.41
61.1	1130	2700	0.39
61.5	1080	2720	0.41

Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio Based on Source-to-Receiver Travel Time Data - Borehole B-3

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
203.3	3810	9590	0.41
205.0	4080	9740	0.39
206.6	4520	9660	0.36
208.2	4550	10290	0.38
209.9	4830	10380	0.36
211.5	5150	10640	0.35
213.2	4980	10730	0.36
214.8	4690	10380	0.37
216.4	3770	8850	0.39
218.1	2800	7810	0.43
219.7	2500	6990	0.43
221.4	2320	6700	0.43

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(m)	(m/s)	(m/s)	
62.0	1160	2920	0.41
62.5	1240	2970	0.39
63.0	1380	2950	0.36
63.5	1390	3140	0.38
64.0	1470	3160	0.36
64.5	1570	3240	0.35
65.0	1520	3270	0.36
65.5	1430	3160	0.37
66.0	1150	2700	0.39
66.5	850	2380	0.43
67.0	760	2130	0.43
67.5	710	2040	0.43

APPENDIX B

BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

Calibration CERT #
AC-1969

Certificate of Calibration

Date: Jul 14, 2016

Cert No. 222200812421146

Customer:

GEOVISION
1124 OLYMPIC DRIVE
CORONA CA 92881

		Work Order #:	N/A
MPC Control #:	AM6767	Serial Number:	160023
Asset ID:	160023	Department:	N/A
Gage Type:	LOGGER	Performed By:	TYLER MCKEEN
Manufacturer:	OYO	Received Condition:	IN TOLERANCE
Model Number:	3403	Returned Condition:	IN TOLERANCE
Size:	N/A	Cal. Date:	July 14, 2016
Temp/RH:	72.0°F / 54.0%	Cal. Interval:	12 MONTHS
		Cal. Due Date:	July 14, 2017

Calibration Notes:

See attached data sheet for calculations. (1 Page)

Calibrated IAW customer supplied data form Rev 2.1
Frequency measurement uncertainty = 0.0005 Hz
Unit calibrated with Laptop Panasonic Model CF-29,s/n: 4FKSA41798
Calibrated To 4:1 Accuracy Ratio

This Calibration has been performed in conformance with, and complies to all requirements as set forth in S&ME purchase order SCP-0022, Dated July 13, 2016

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
T1100	UNIVERSAL COUNTER	53131A	3546A09912	HEWLETT PACKARD	Feb 2, 2017	222008122827657
DB8748	GPS TIME AND FREQUENCY RECEIVER	58503A	3625A01225	HEWLETT PACKARD	Jun 17, 2017	222008122553843
AM4000	WAVEFORM GENERATOR	33250A	MY40000703	AGILENT	Jul 8, 2017	222200812420653

Calibrating Technician:

TYLER MCKEEN

QC Approval:

Jim Williams

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. This report may not be reproduced in part or in a whole without the prior written approval of the issuing MPC lab.

MICRO PRECISION CALIBRATION INC.

MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659

ilac-MRA ANAB
Calibration CERT #
AC-1969

Certificate of Calibration

Date: **Jul 14, 2016** **Cert No.** **222200812421146**

Procedures Used in this Event

Procedure Name	Description
GEOVISION SEISMIC	Suspension PS Seismic Logger/Recorder Calibration Procedure

Calibrating Technician: TYLER MCKEEN

QC Approval: Jim Williams

The reported expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for normal distribution corresponds to a coverage probability of approximately 95%. The standard uncertainty of measurement has been determined in accordance with EA's Publication and NIST Technical Note 1297, 1994 Edition. Services rendered comply with ISO 17025:2005, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. This report may not be reproduced in part or in a whole without the prior written approval of the issuing MPC lab.

GEOVision
geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.: OYO / Robertson Geologging — Model no.: 3403
Serial no.: 160023 — Calibration date: 7/14/16
By: Emily Feldman — Due date: 7/14/17

Counter mfg.: Hewlett Packard — Model no.: 53131A
Serial no.: 3546A09912 — Calibration date: 2/02/16
By: Micro Precision — Due date: 2/02/17

Signal generator mfg.: Agilent — Model no.: 33250A
Serial no.: MY40000703 — Calibration date: 4/08/16
By: Micro Precision — Due date: 7/08/17

Laptop controller mfg.: Panasonic — Model no.: CF29 Toughbook
Serial no.: 4FKSA41798 — Calibration date: N/A

SYSTEM SETTINGS:

Gain: ~~x10~~ EFF 7/14/16 2
Filter: Open (Low pass 10K)
Range: 200 to 50 µS
Delay: 0.3 msec
Stack (1 std): 1
System date = correct date and time: 7/14/16, 1300 hrs

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.22% As left 0.22%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name C001	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	001	180.2	50.06	179.8	50.06	180.4	49.89
100.0	100.0	100	002	90.0	100.0	90.0	100.0	89.9	100.1
200.0	200.0	50	003	45.0	200.0	45.05	199.8	44.95	200.2
500.0	500.0	20	004	18.02	499.5	18.00	500.0	18.00	~~200.~~ EF 7/14/16 500.0 EF 7/14/16
1000	1000	10	005	9.01	998.9	9.01	998.9	9.00	1000
2000	2000	5	006	4.50	2000	4.495	~~2002.2~~ ~~2000~~ EF 7/14/16 2002	4.5	2000

Calibrated by: Name — Date 7/14/16 — Signature Tyler McKeen

Witnessed by: Emily Feldman Name — 7/14/16 Date — Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012

Appendix D

Previous Laboratory Test Results

CONTENTS

D.1 General...D-1

D.2 Soil Testing...D-1

    D.2.1 Atterberg Limits...D-1

    D.2.2 Particle-Size Analysis..D-1

Figures

Figure D1: Atterberg Limits Results

Figure D2: Grain Size Distribution

Attachments

Northwest Testing, Inc. Technical Report, dated November 28, 2016

D.1 GENERAL

The soil samples obtained during the previous field explorations were described and identified in the field in accordance with the ODOT Soil and Rock Classification Manual (1987). The samples were then reviewed in the laboratory. Physical characteristics of the samples were noted, and field descriptions and identifications were modified as necessary. During the course of the examination, representative samples were selected for further testing. We refined our descriptions and identifications based on the results of the laboratory tests, in accordance with the ODOT Soil and Rock Classification Manual (1987).

The soil testing program included Atterberg limits determinations and particle-size analyses. All testing was completed by Northwest Testing, Inc. (NTI), of Wilsonville, Oregon. All test procedures were performed in accordance with applicable ASTM International standards. Tests procedures are summarized in the following paragraphs.

D.2 SOIL TESTING

D.2.1 Atterberg Limits

Atterberg limits were determined for selected samples in accordance with ASTM D4318. This analysis yields index parameters of the soil that are useful in soil identification, as well as in a number of engineering analyses, including liquefaction analysis. An Atterberg limits test determines a soil’s liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil’s plasticity index (PI) can be determined by subtracting PL from LL. The LL, PL, and PI of the tested samples are presented on the Atterberg Limits Results, Figure D1. They are also presented in the NTI report, dated November 28, 2016, which is attached to the end of this appendix. For the purposes of soil description, we use the term nonplastic to refer to soils with a PI less than 3, low plasticity for soils with a PI range of 3 to 15, medium plasticity for soils with a PI range of 15 to 30, and high plasticity for soils with a PI greater than 30.

D.2.2 Particle-Size Analysis

Particle-size analyses were conducted on select samples in accordance with ASTM C117 and C136. A wet sieve analysis was performed to determine a percentage (by weight) of the sample passing the No. 200 (0.075 mm) sieve (ASTM C117). The material retained on the No. 200 sieve was shaken through a series of sieves to determine the distribution of the plus No. 200 fraction (ASTM C136). Results of all particle-size analyses are plotted on Figure D2, Grain Size Distribution. The results are also shown in tabular format in the NTI report, dated November 28, 2016, which is attached to the end of this appendix.

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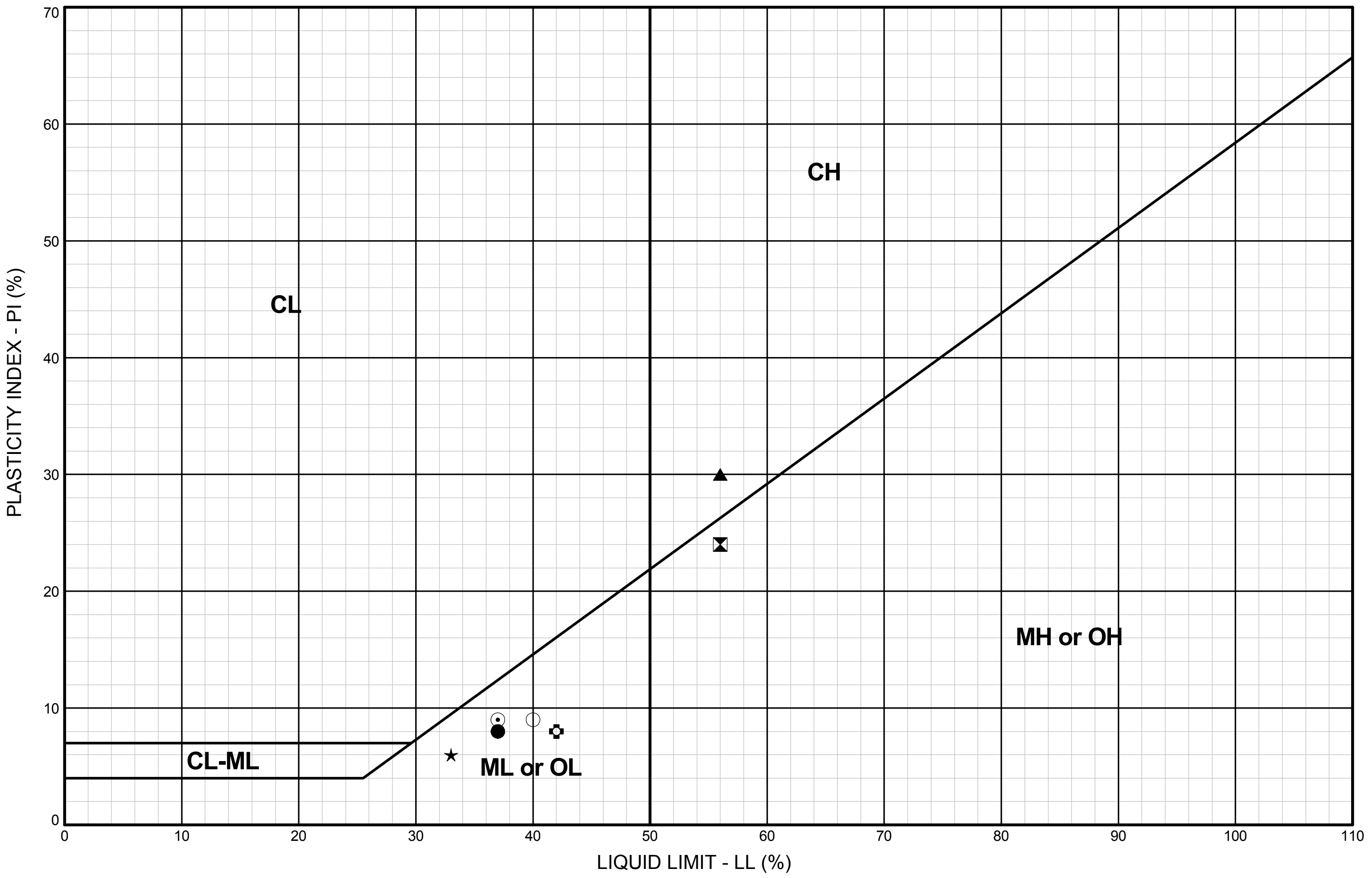

NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
  - Nonplastic (NP) (< 3%)
  - Low Plasticity (3 to 15%)
  - Medium Plasticity (15 to 30%)
  - High Plasticity (> 30%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-1, N7	35.0	*ML*	*Sandy SILT*	37	29	8		67
⧗ B-1, N18	90.0	*MH*	*Clayey SILT*	56	32	24		
▲ B-2, N14	77.0	*CH*	*CLAY with some sand*	56	26	30		76
★ B-3, N5	25.0	*SM*	*Silty SAND*	33	27	6		43
⊙ B-3, N14	70.0	*ML*	*Sandy SILT*	37	28	9		57
✤ B-3, N17	85.0	*ML*	*SILT with some sand*	42	34	8		77
○ B-3, N20	100.0	*ML*	*SILT with some sand*	40	31	9		73

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Portland, Oregon

ATTERBERG LIMITS RESULTS

February 2017 24-1-04065-005

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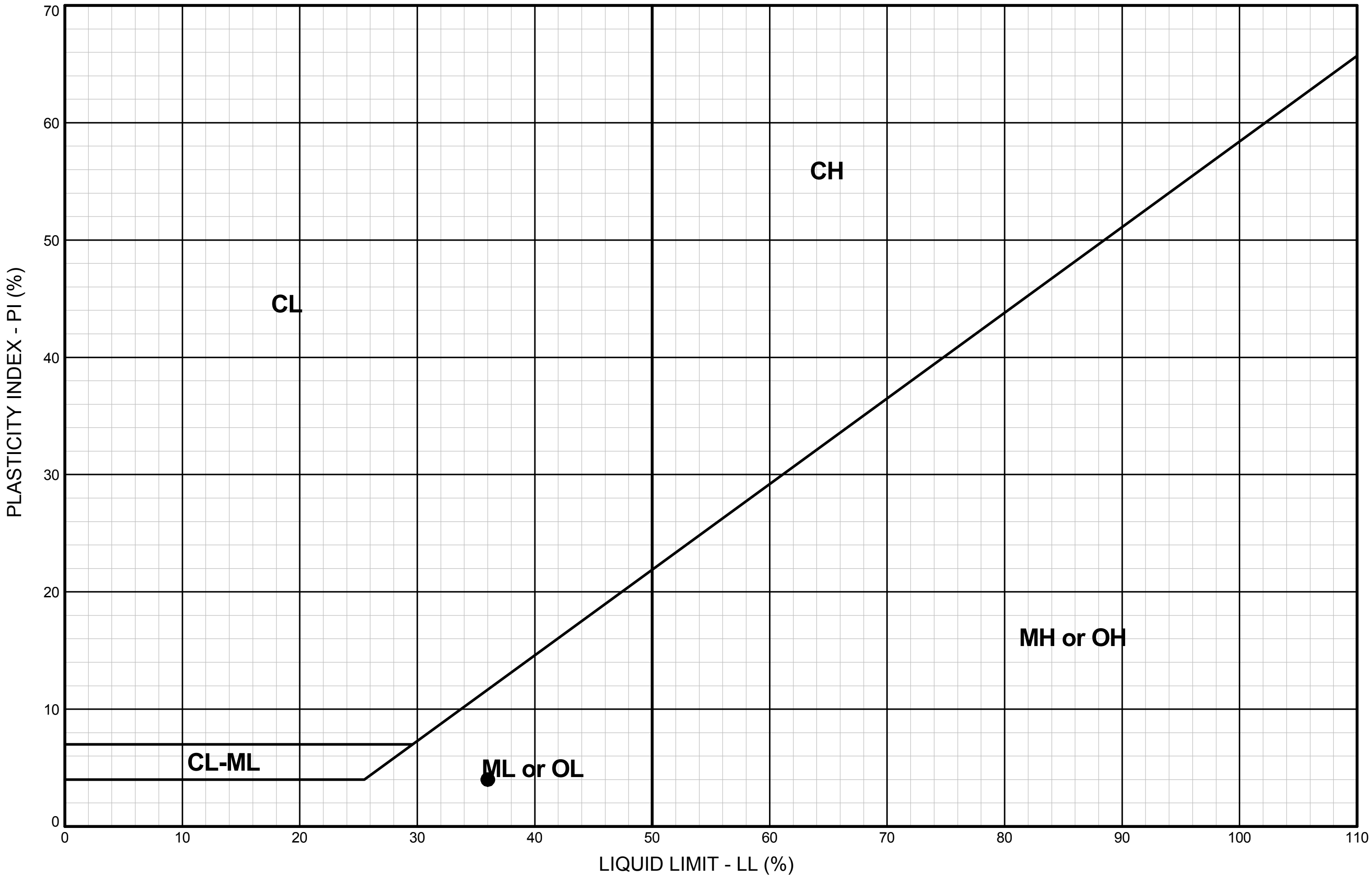
FIG. D1
Sheet 1 of 2

FIG. D1



NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 3%)
- Low Plasticity (3 to 15%)
- Medium Plasticity (15 to 30%)
- High Plasticity (> 30%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-3, N24	120.0	*ML*	*Sandy SILT*	36	32	4		68

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Portland, Oregon

ATTERBERG LIMITS RESULTS

February 2017 24-1-04065-005

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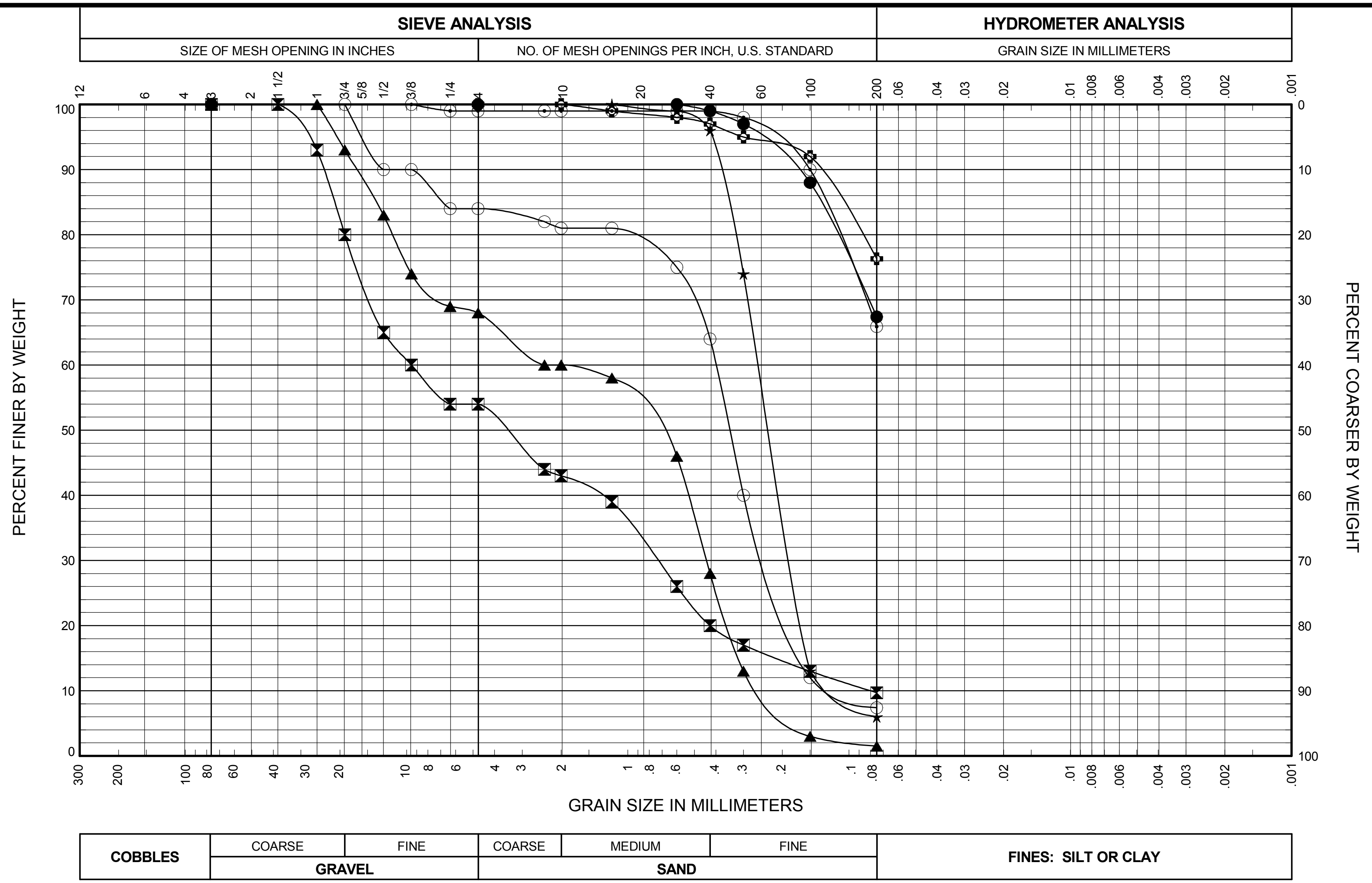
FIG. D1
Sheet 2 of 2

NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL^2	GROUP NAME^2	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, N7	35.0	*ML*	*Sandy SILT*	0	33	67		
⧗ B-1, N10	50.0	*GP-GM*	*Sandy GRAVEL with some silt*	46	44	10		
▲ B-2, N2	16.0	*SP*	*Gravelly SAND with trace silt*	32	67	2		
★ B-2, N6	36.5	*SP-SM*	*SAND with some silt*	0	94	6		
⊙ B-2, N12	67.1	*ML*	*Sandy SILT*	1	33	66		
✚ B-2, N14	77.0	*CH*	*CLAY with some sand*	0	24	76		
○ B-3, N2	10.0	*SP-SM*	*SAND with some silt and some gravel*	16	77	7		

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GRAIN SIZE DISTRIBUTION

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24-1-04065-005

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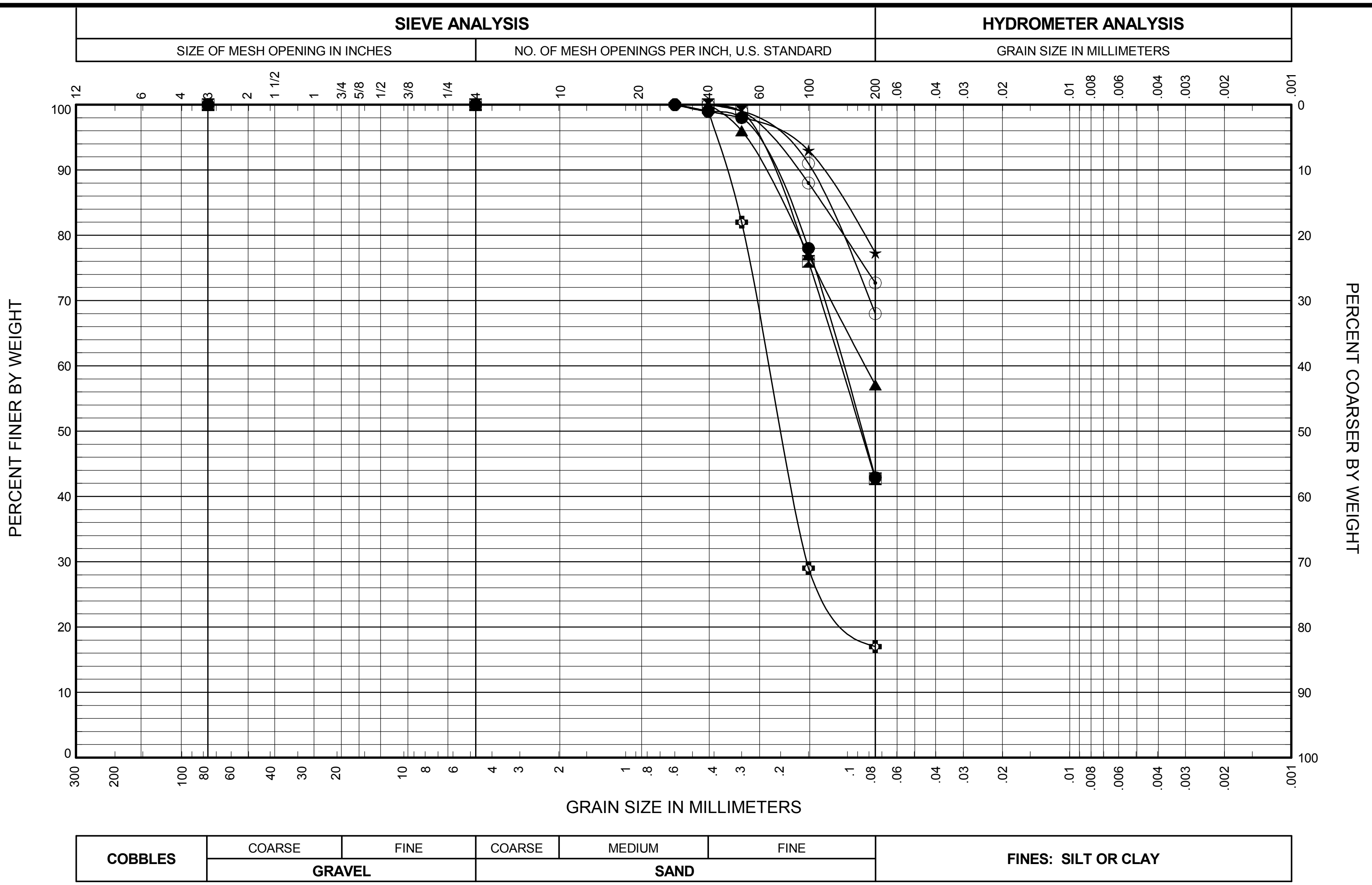
FIG. D2
Sheet 1 of 2

FIG. D2

NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-3, N5	25.0	*SM*	*Silty SAND*	0	57	43		
⧗ B-3, N6	32.0	*SM*	*Silty SAND*	0	57	43		
▲ B-3, N14	70.0	*ML*	*Sandy SILT*	0	43	57		
★ B-3, N17	85.0	*ML*	*SILT with some sand*	0	23	77		
⊙ B-3, N20	100.0	*ML*	*SILT with some sand*	0	27	73		
✚ B-3, N23	115.0	*SM*	*Silty SAND*	0	83	17		
○ B-3, N24	120.0	*ML*	*Sandy SILT*	0	32	68		

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GRAIN SIZE DISTRIBUTION

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FIG. D2
Sheet 2 of 2

FIG. D2

Northwest Testing, Inc.
A Division of Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 fax: 503.682.2753 | www.nwgeotech.com

TECHNICAL REPORT

Report To: Ms. Aimee Holmes, P.E., C.E.G.
Shannon & Wilson, Inc.
3990 S.W. Collins Way, Suite 203
Lake Oswego, Oregon 97035

Date: 11/28/16

Lab No.: 16-304

Project: Laboratory Testing – 24-1-04065-001

Project No.: 2966.1.1

Report of: Atterberg Limits and sieve analysis

Sample Identification

NTI completed Atterberg limits and sieve analysis testing on samples delivered to our laboratory on November 17, 2016. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following table and attached page.

Laboratory Testing

Atterberg Limits (ASTM D4318)			
Sample ID	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**
B-1 N-7 @ 35 – 36.5 ft.	37	29	8
B-1 N-18 @ 90 – 91.5 ft.	56	32	24
B-2 N-14 @ 77 – 78.5 ft.	56	26	30
B-3 N-5 @ 25 – 26.5 ft.	33	27	6
B-3 N-14 @ 70 – 71.5 ft.	37	28	9
B-3 N-17 @ 85 – 86.5 ft.	42	34	8
B-3 N-20 @ 100 – 101.5 ft.	40	31	9
B-3 N-24 @ 120 – 121.5 ft.	36	32	4

Copies: Laboratory Test Results

Copies: Addressee

TECHNICAL REPORT

Report To: Ms. Aimee Holmes, P.E., C.E.G.
Shannon & Wilson, Inc.
3990 S.W. Collins Way, Suite 203
Lake Oswego, Oregon 97035

Date: 11/28/16

Lab No.: 16-304

Project: Laboratory Testing – 24-1-04065-001

Project No.: 2966.1.1

Laboratory Testing

Sieve Analysis of Aggregate (ASTM C117/C136)

Sieve Size	B-1 N-7 @ 35 – 36.5 ft. Percent Passing	B-1 N-10 @ 50 – 51.5 ft. Percent Passing	B-2 N-2 @ 16 – 17.5 ft. Percent Passing	B-2 N-6 @ 36.5 – 38 ft. Percent Passing
1 ½”	--	100	--	--
1”	--	93	100	--
¾”	--	80	93	--
½”	--	65	83	--
⅜”	--	60	74	--
¼”	--	54	69	--
#4	--	54	68	--
#8	--	44	60	--
#10	--	43	60	--
#16	--	39	58	100
#30	100	26	46	99
#40	99	20	28	96
#50	97	17	13	74
#100	88	13	3	13
#200	67.4	9.7	1.5	6.0

Sieve Analysis of Aggregate (ASTM C117/C136)

Sieve Size	B-2 N-12 @ 67.1 – 68.6 ft. Percent Passing	B-2 N-14 @ 77 – 78.5 ft. Percent Passing	B-3 N-2 @ 10 – 11.5 ft. Percent Passing	B-3 N-5 @ 25 – 26.5 ft. Percent Passing
¾”	--	--	100	--
½”	--	--	90	--
⅜”	100	--	90	--
¼”	99	--	84	--
#4	99	--	84	--
#8	99	--	82	--
#10	99	100	81	--
#16	99	99	81	--
#30	99	98	75	100
#40	99	97	64	99
#50	98	95	40	98
#100	90	92	12	78
#200	65.9	76.3	7.4	42.9

Northwest Testing, Inc.
A Division of Northwest Geotech, Inc.
9120 SW Pioneer Court, Suite B, Wilsonville, Oregon 97070 | ph: 503.682.1880 fax: 503.682.2753 | www.nwgeotech.com

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3990 S.W. Collins Way, Suite 203
Lake Oswego, Oregon 97035

Date: 11/28/16

Lab No.: 16-304

Project: Laboratory Testing – 24-1-04065-001

Project No.: 2966.1.1

Laboratory Testing

Sieve Analysis of Aggregate (ASTM C117/C136)			
Sieve Size	**B-3 N-6 @ 32 – 33.5 ft. Percent Passing**	**B-3 N-14 @ 70 – 71.5 ft. Percent Passing**	**B-3 N-17 @ 85 – 86.5 ft. Percent Passing**
#30	--	--	100
#40	100	100	99
#50	99	96	98
#100	76	77	93
#200	42.7	57.1	77.3

Sieve Analysis of Aggregate (ASTM C117/C136)			
Sieve Size	**B-3 N-20 @ 100 – 101.5 ft. Percent Passing**	**B-3 N-23 @ 115 – 116.5 ft. Percent Passing**	**B-3 N-24 @ 120 – 121.5 ft. Percent Passing**
#30	--	100	--
#40	100	99	100
#50	99	82	99
#100	88	29	91
#200	72.7	17.0	68.0

APPENDIX E: FLAC RESULTS

Appendix E

FLAC Results

Figures

Figures E1 through E152: Ground Surface Response

Figures E153 through E168: Contour Plots of Results (Non-GI)

Figures E169 and E170: Contour Plots of Results (Enhanced Retrofit, Short-Span Alternative and Couch Extension GI)

Figures E171 through E206: Pier Response Profiles for Enhanced Retrofit Alternative

Figures E207 through E220: Pier Response Profiles for Short-Span Alternative and Couch Extension

Figures E221 and E222: Contour Plots of Results (Long-Span Alternative)

Figures E223 through E232: Pier Response Profiles for Long-Span Alternative

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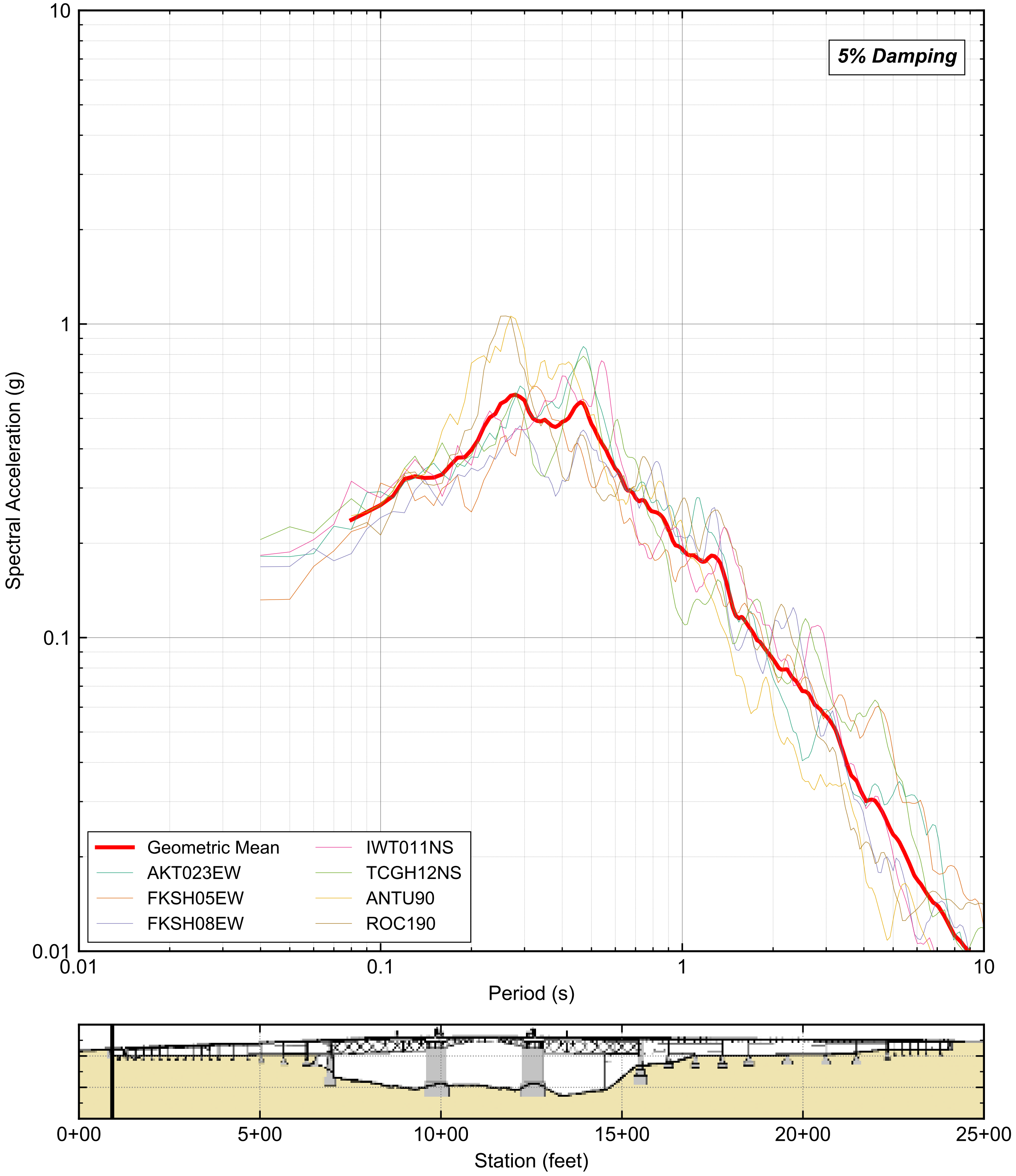

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 1

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FIG. E-1

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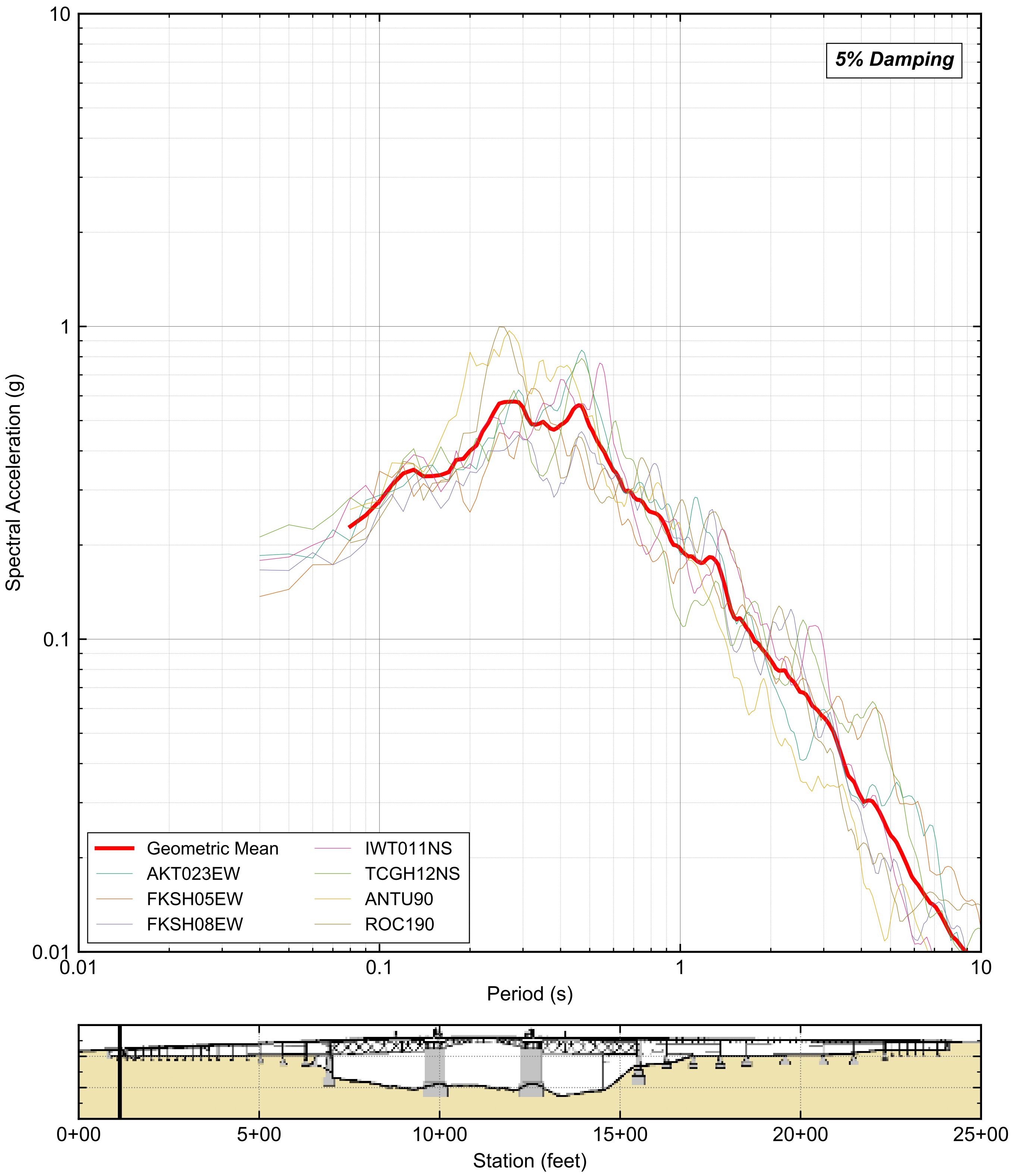

Notes:

1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
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FIG. E-2

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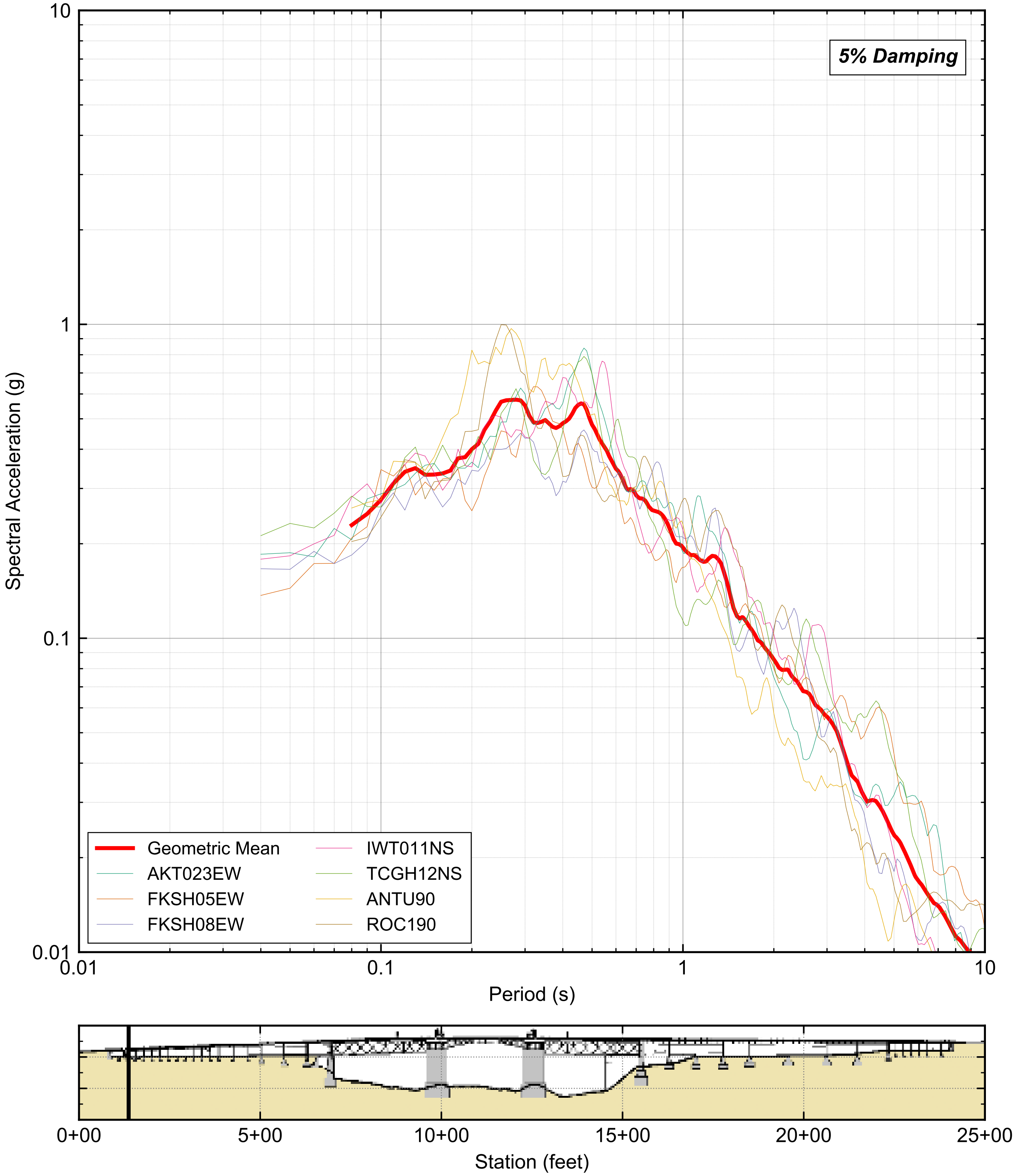

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
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FIG. E-3

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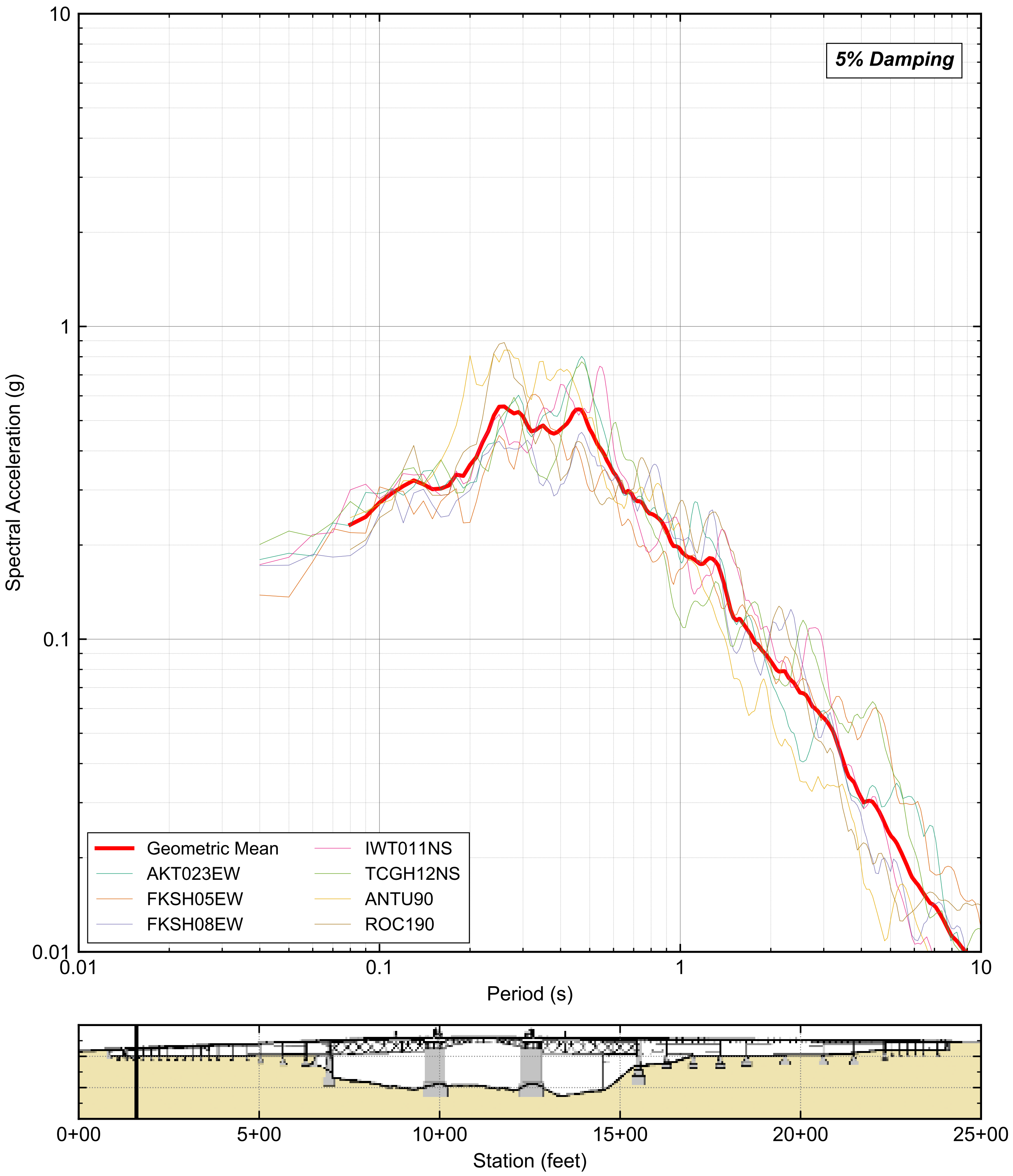

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 4

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FIG. E-4

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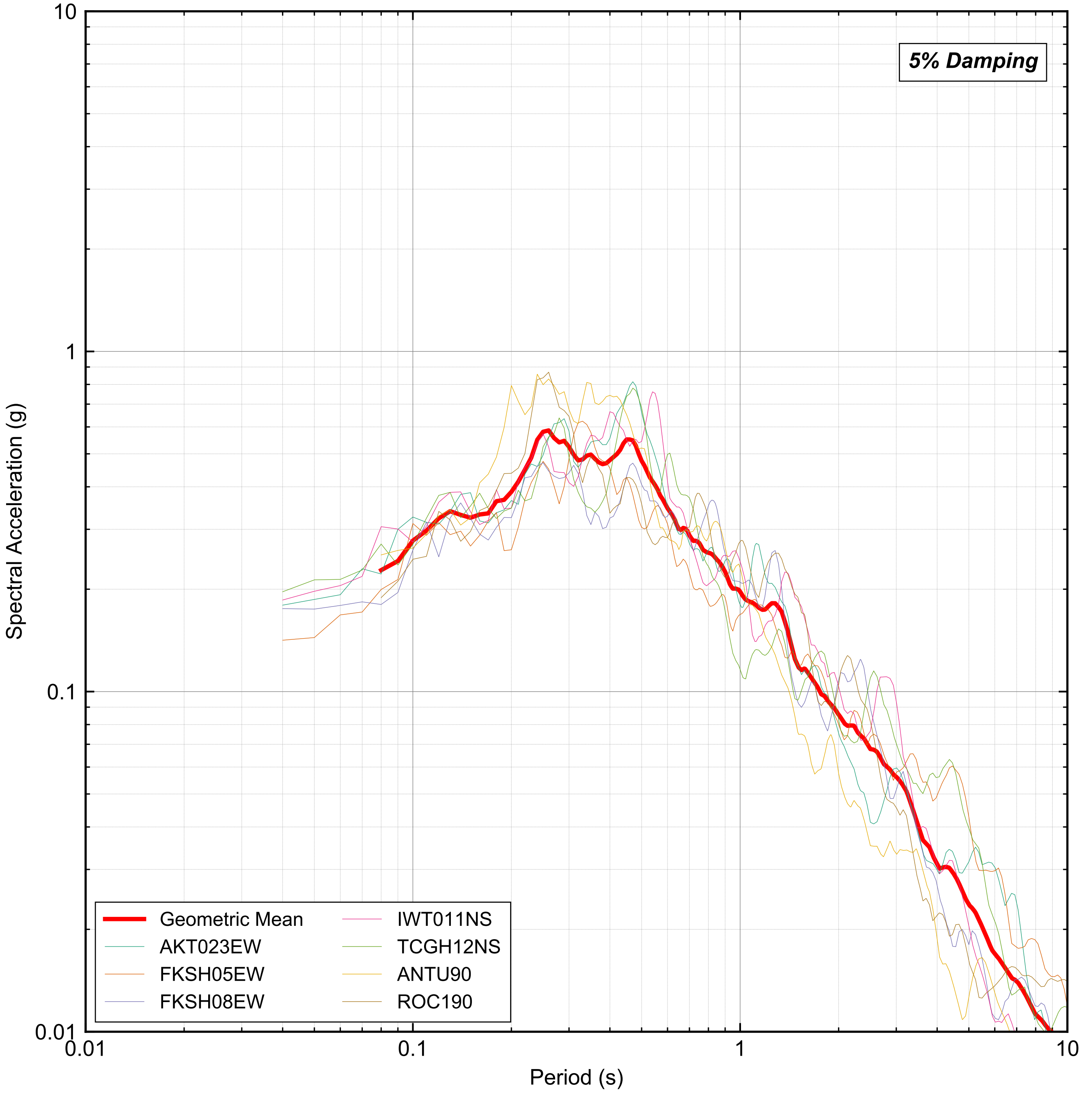

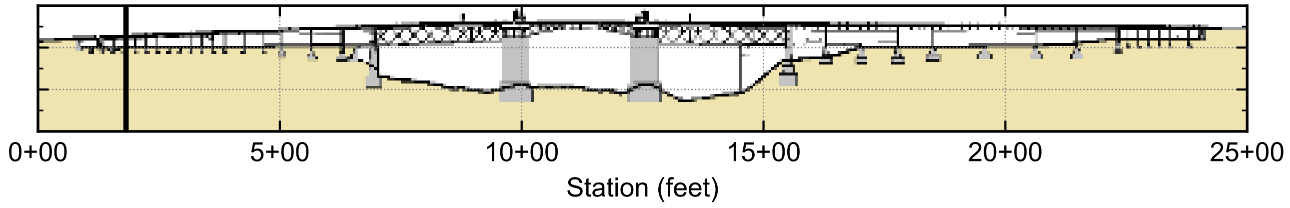

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 5

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FIG. E-5

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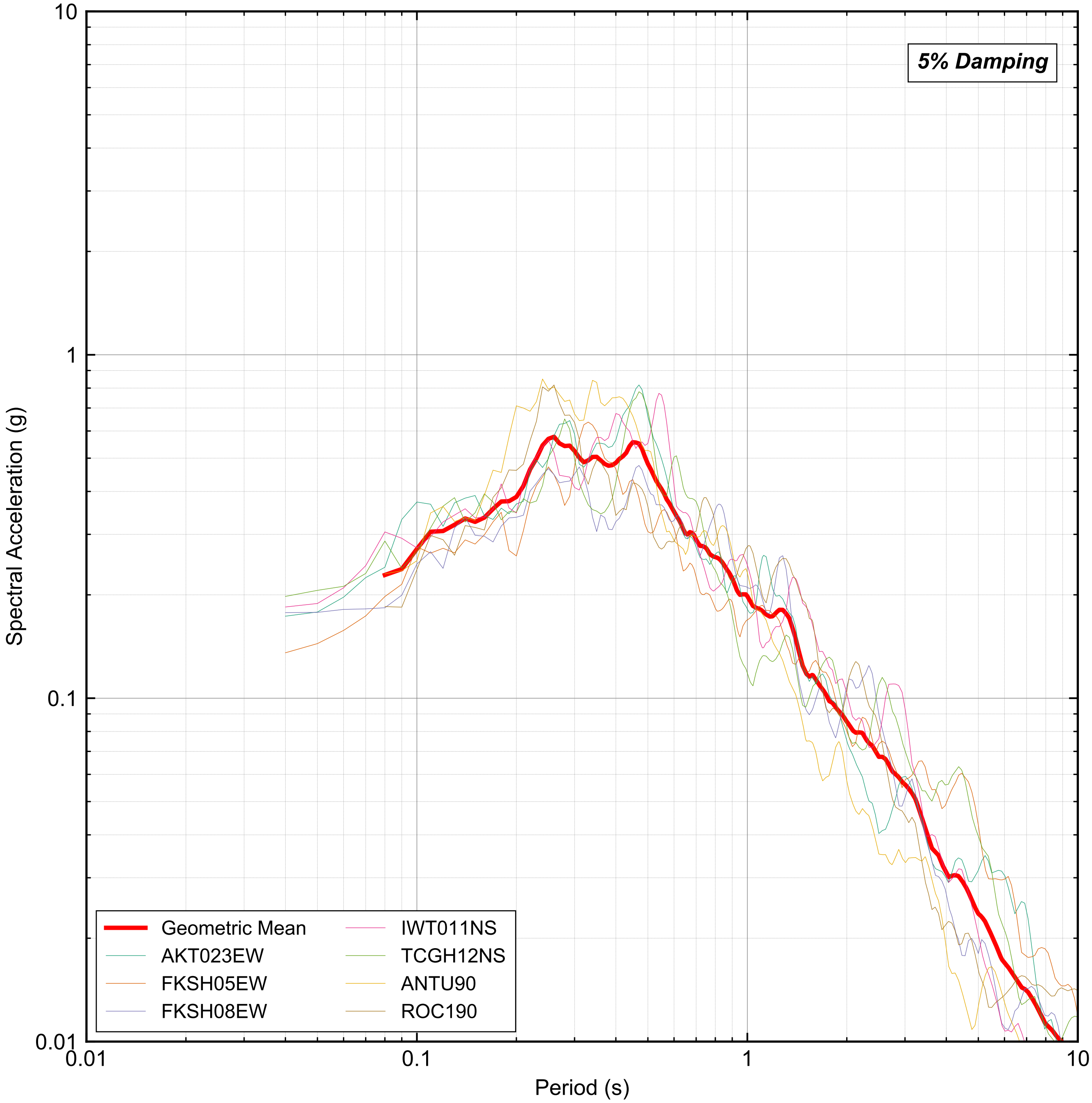

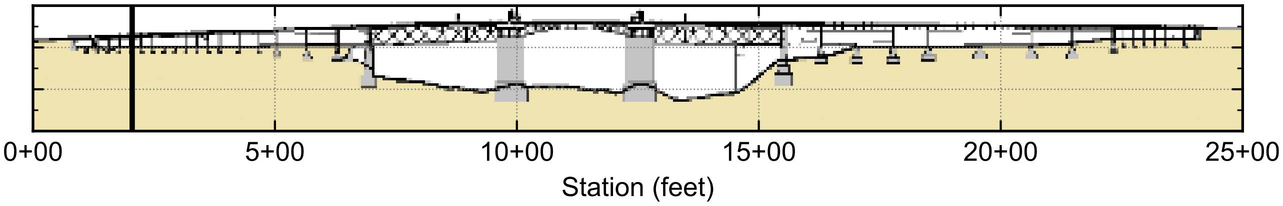

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 6

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FIG. E-6

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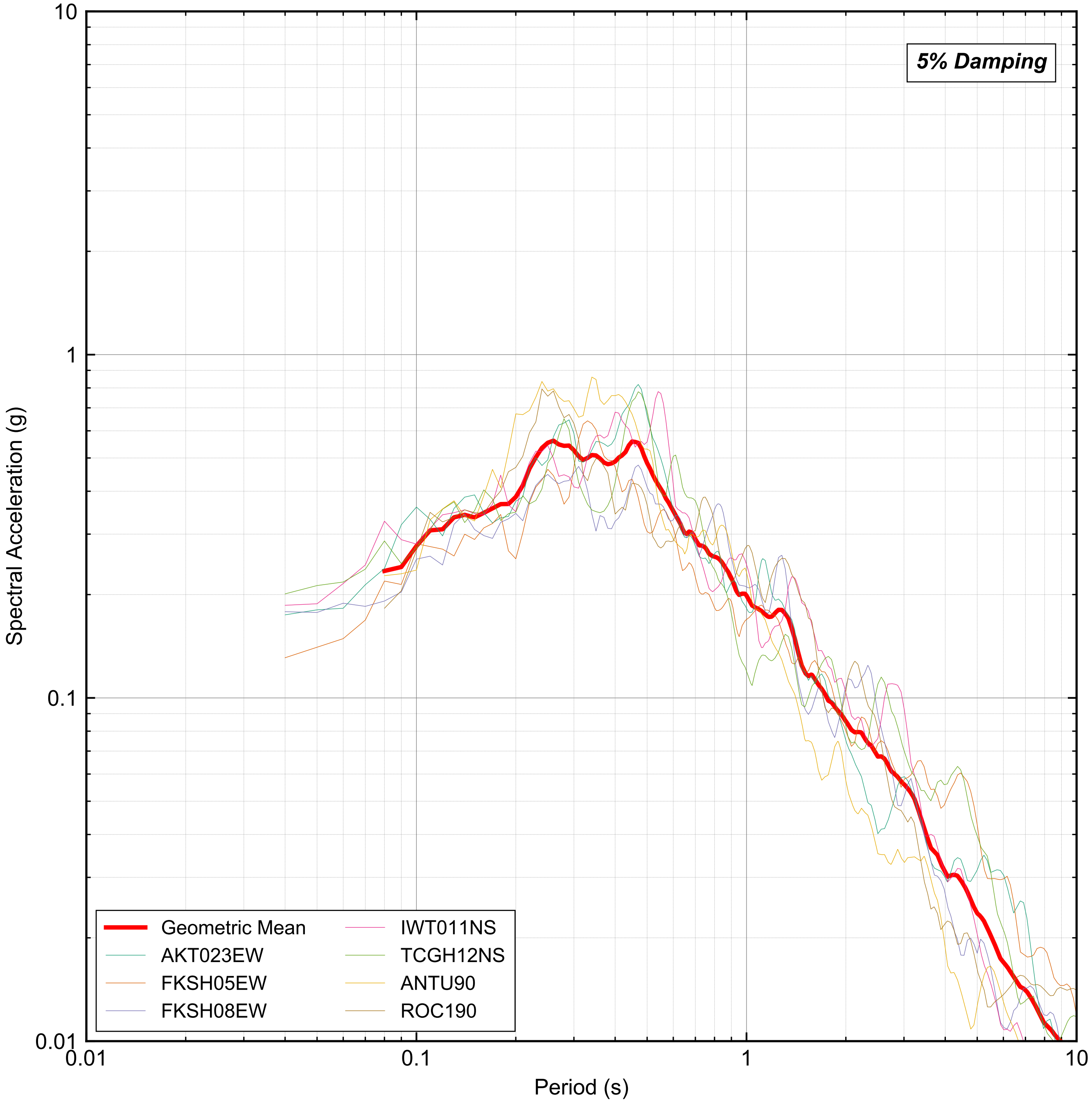

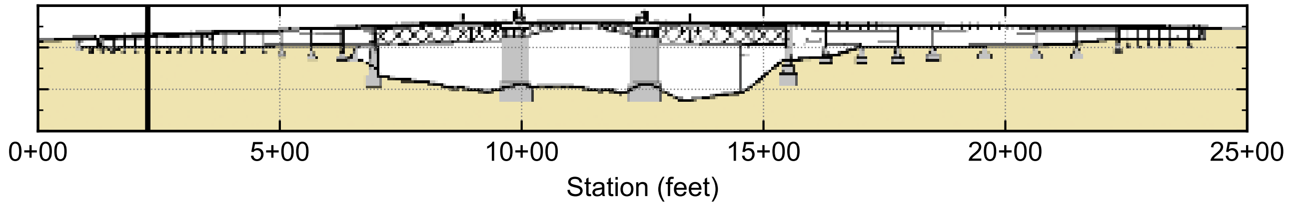

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 7

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FIG. E-7

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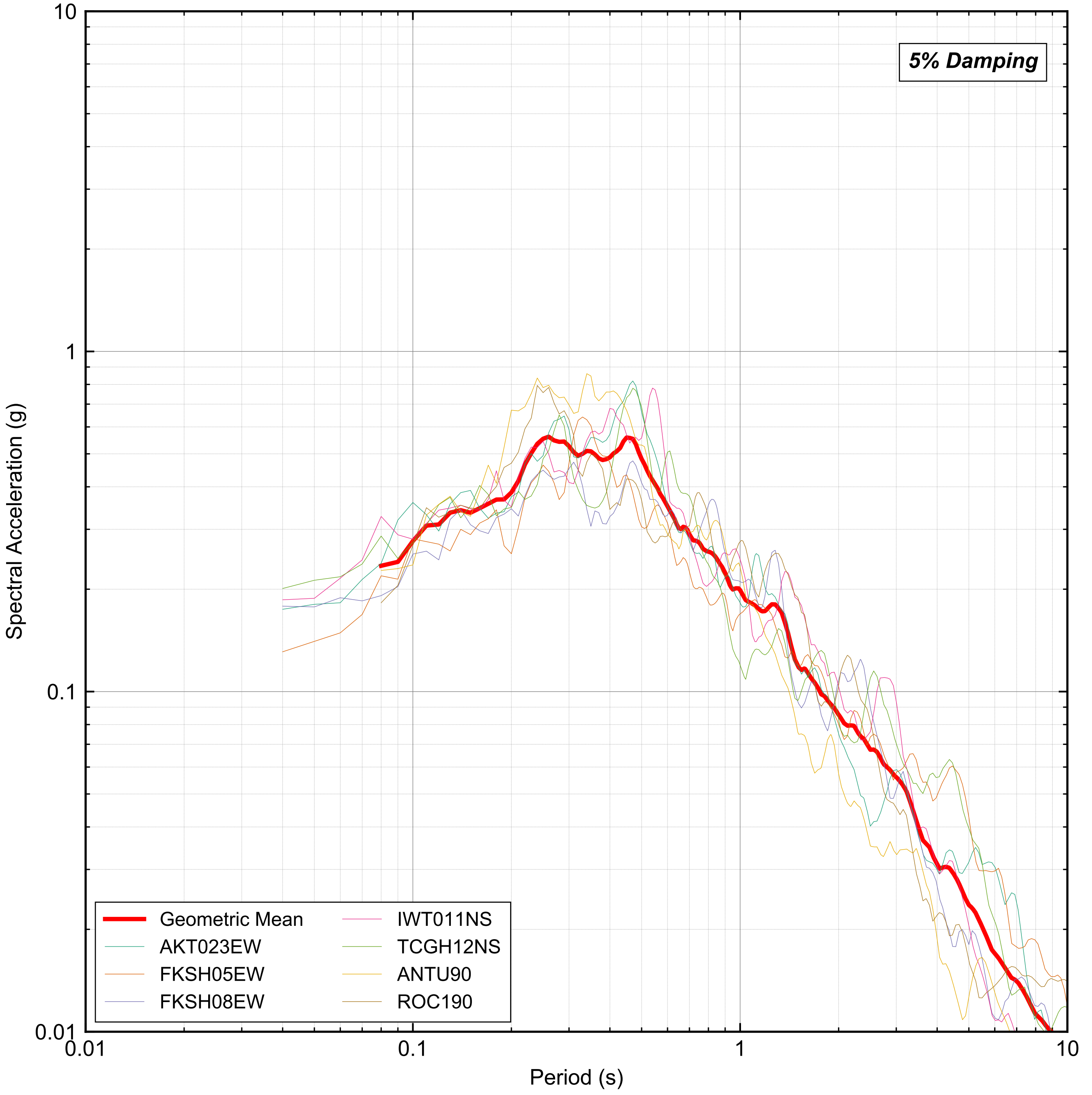

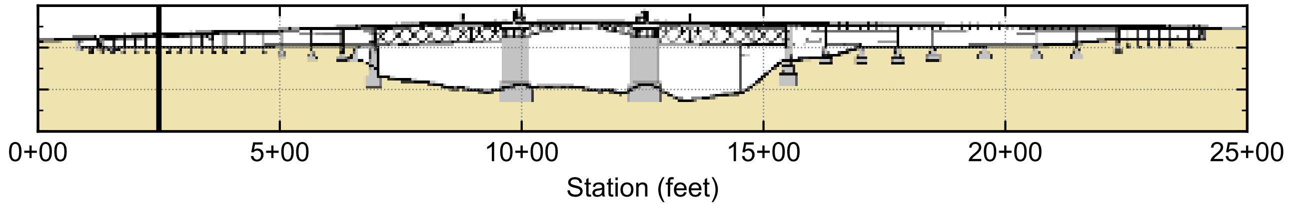

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 8

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FIG. E-8

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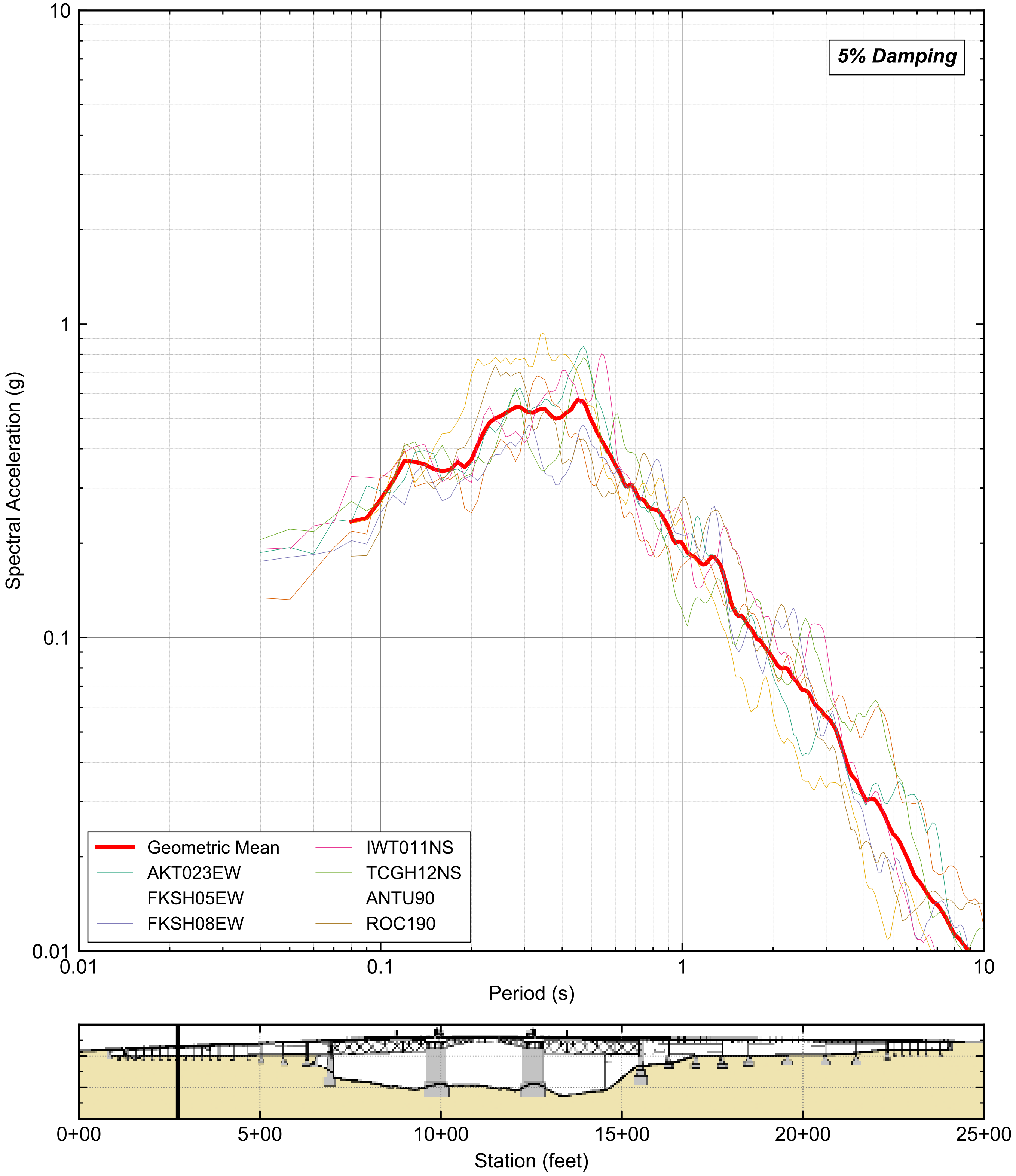

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 9

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FIG. E-9

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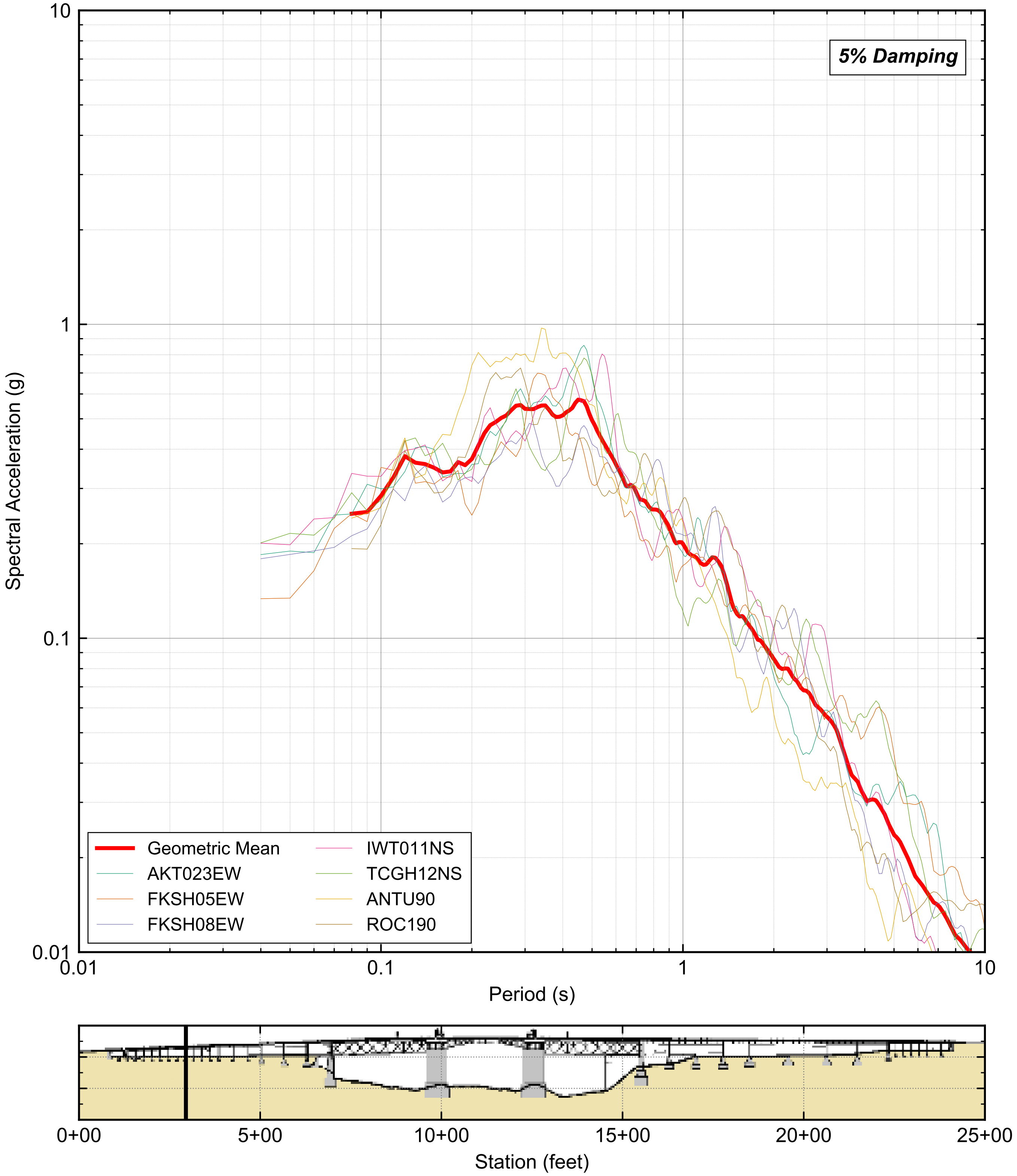

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 10

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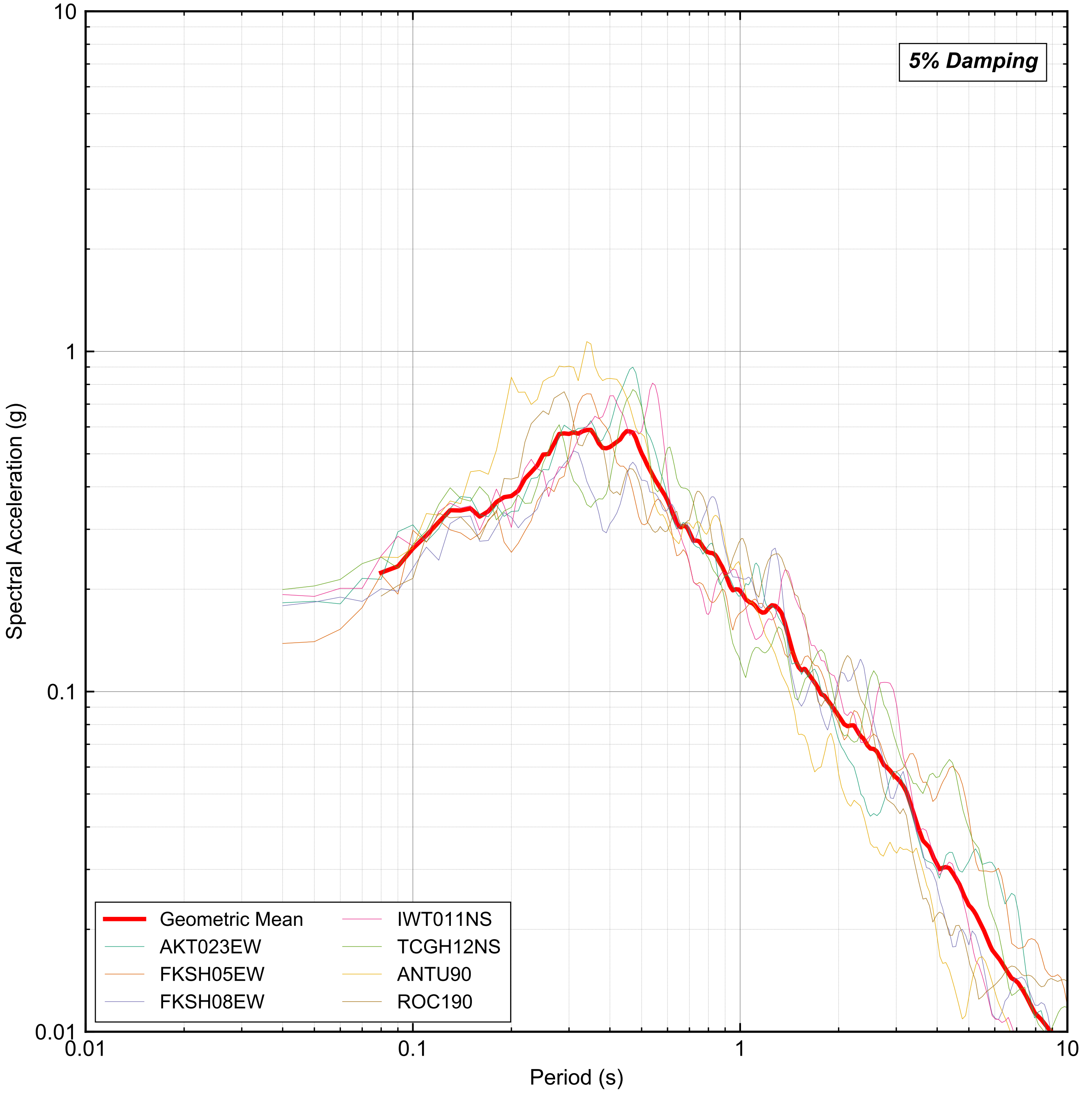
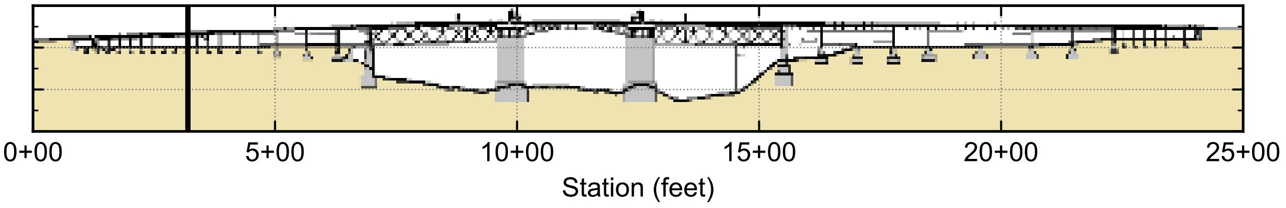
FIG. E-10





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 11

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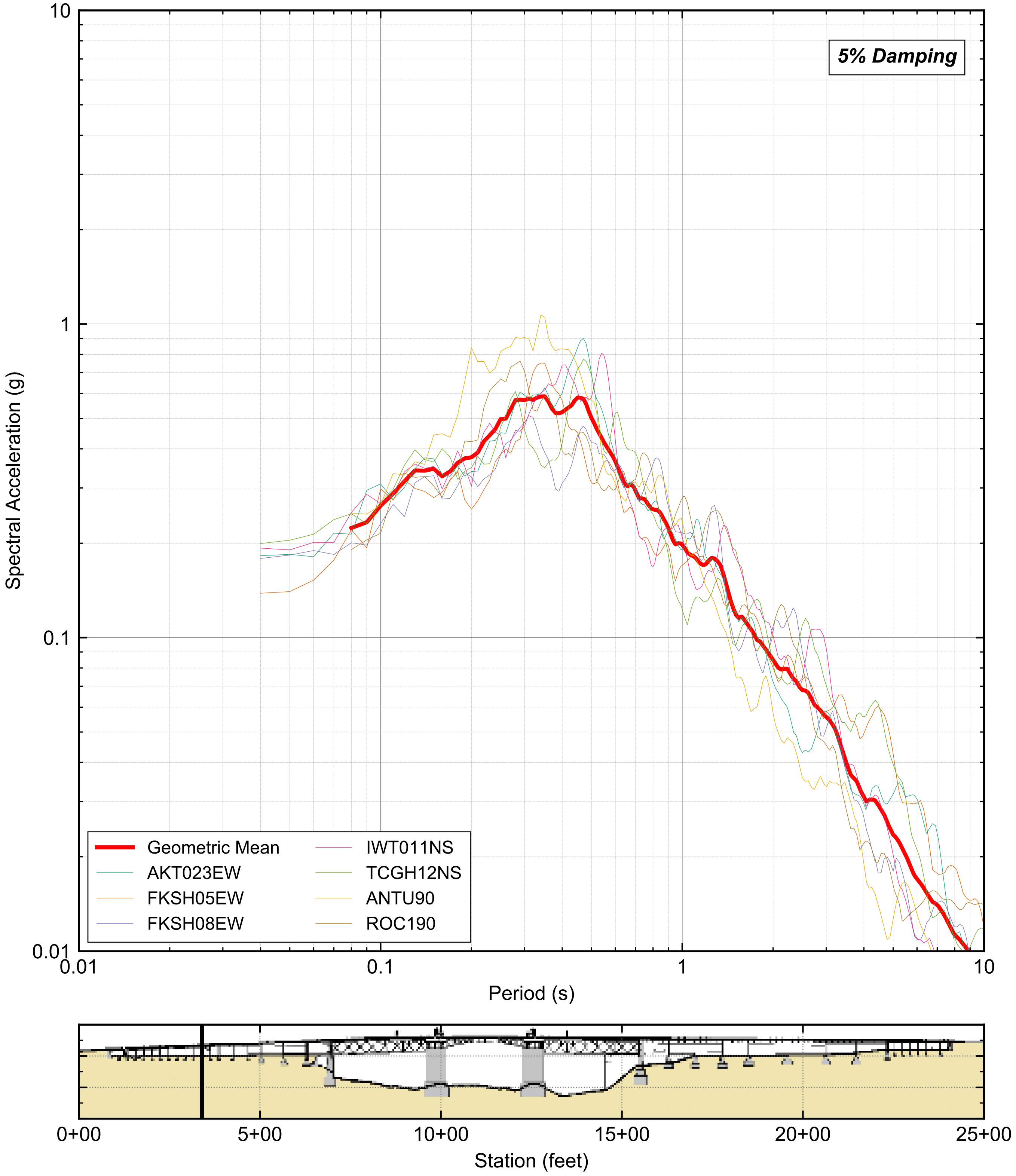
FIG. E-11

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 12

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FIG. E-12

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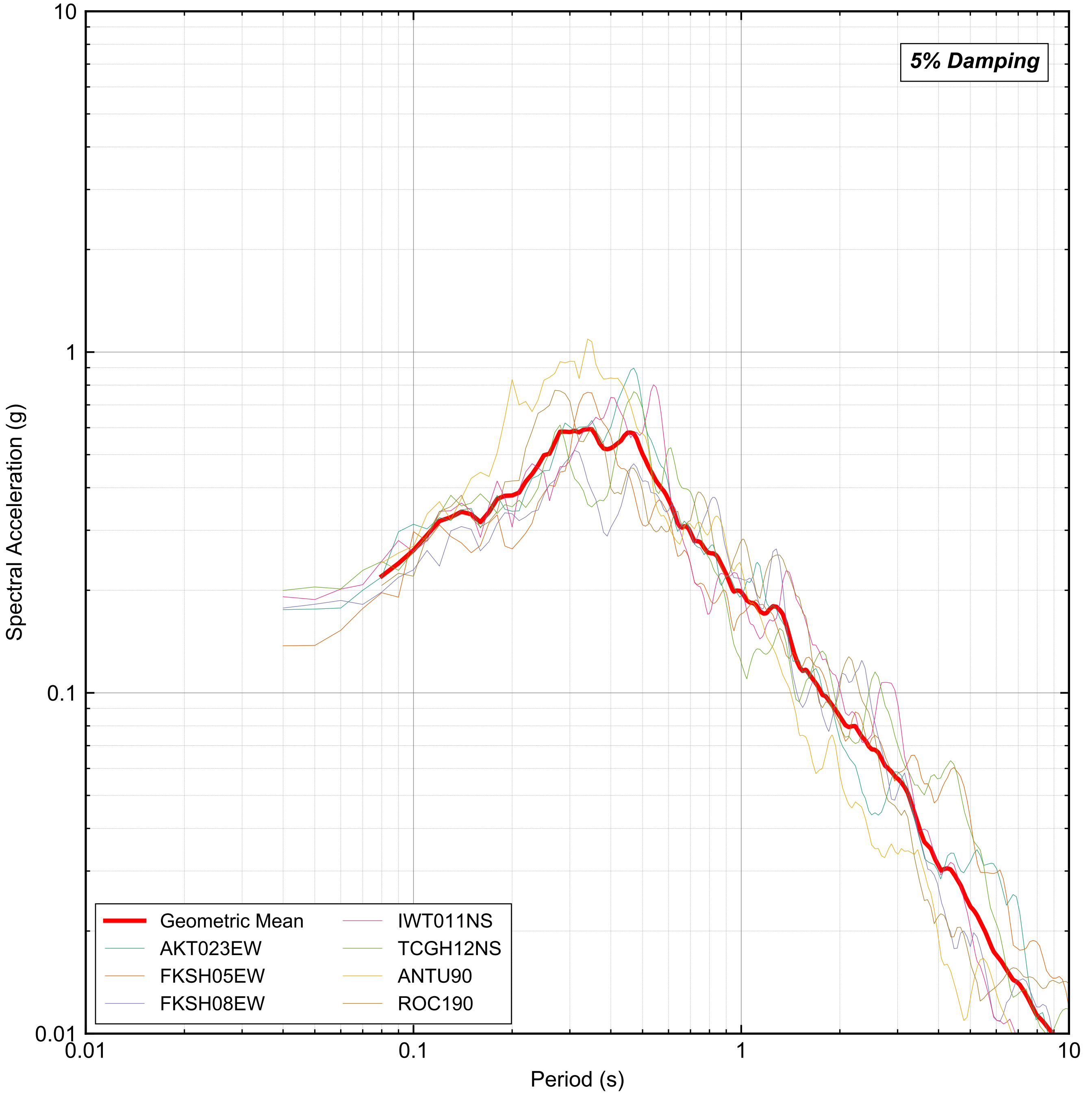

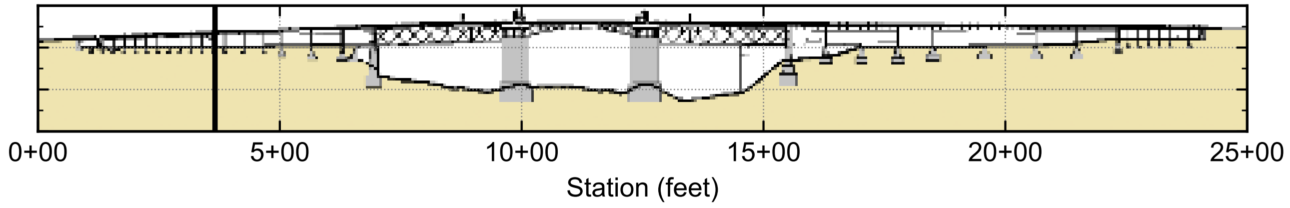

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 13

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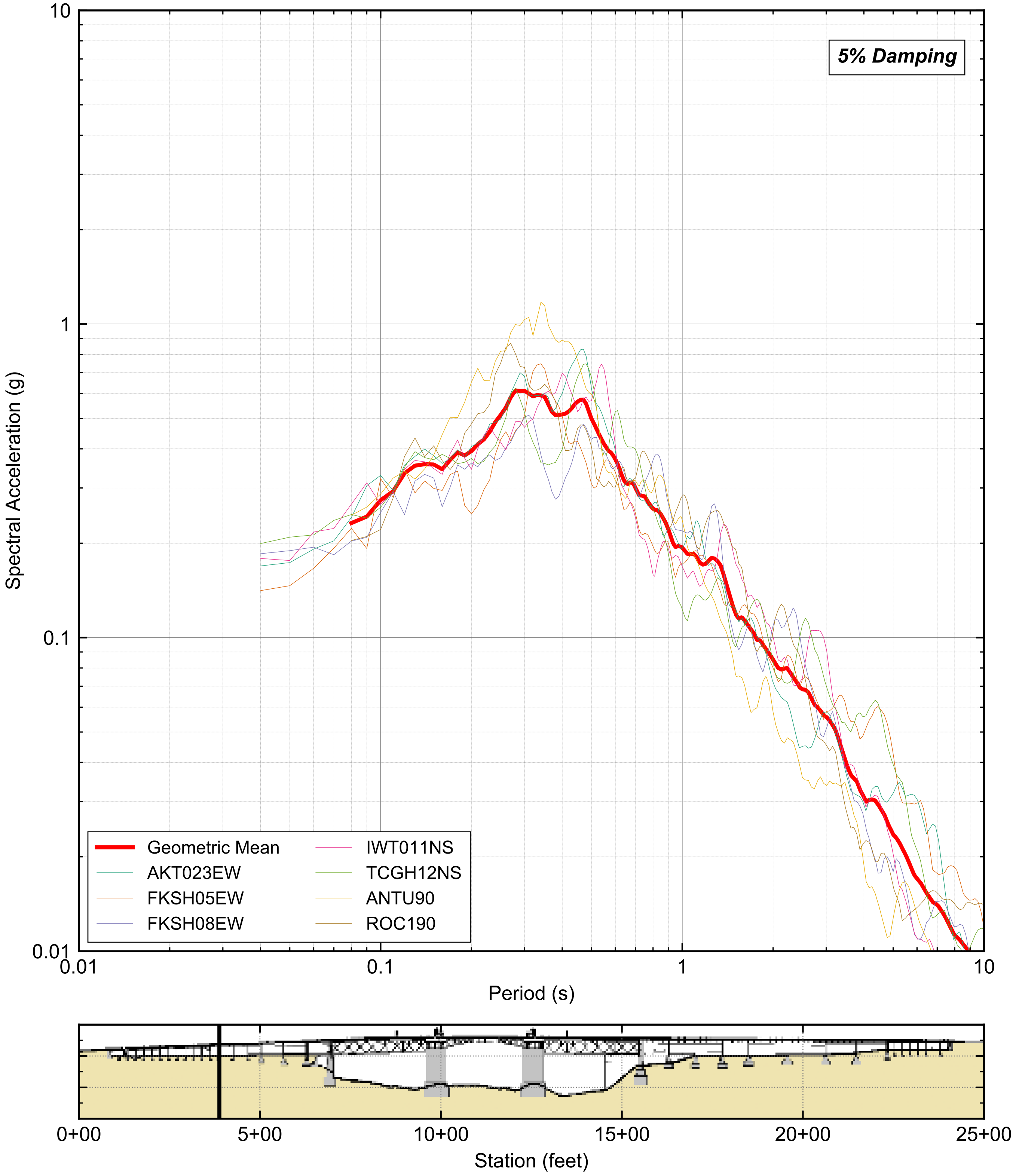
FIG. E-13

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:

1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 14

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FIG. E-14

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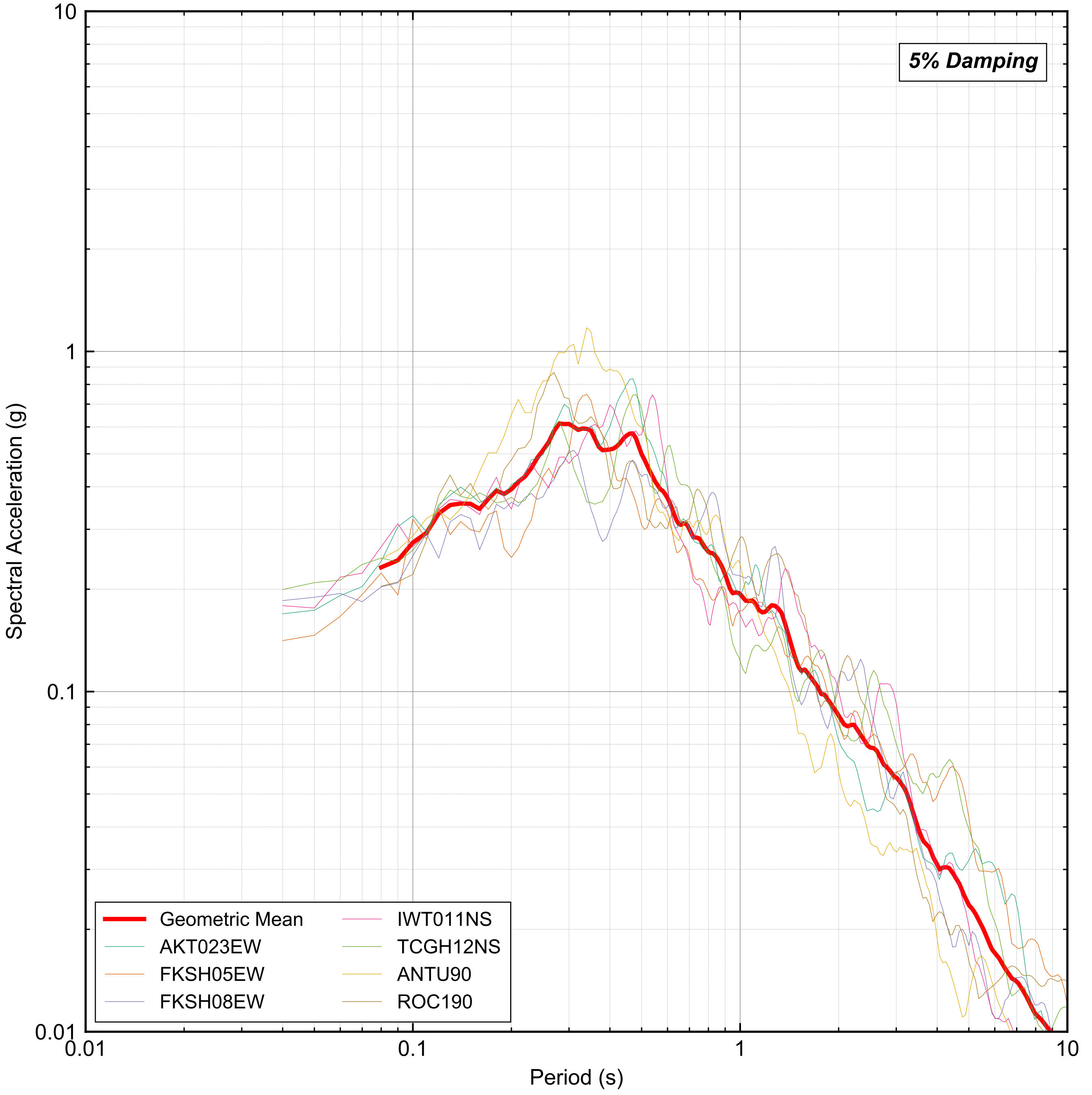

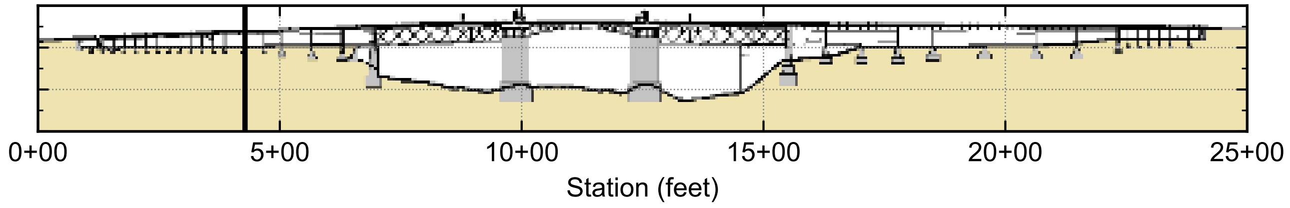

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 15

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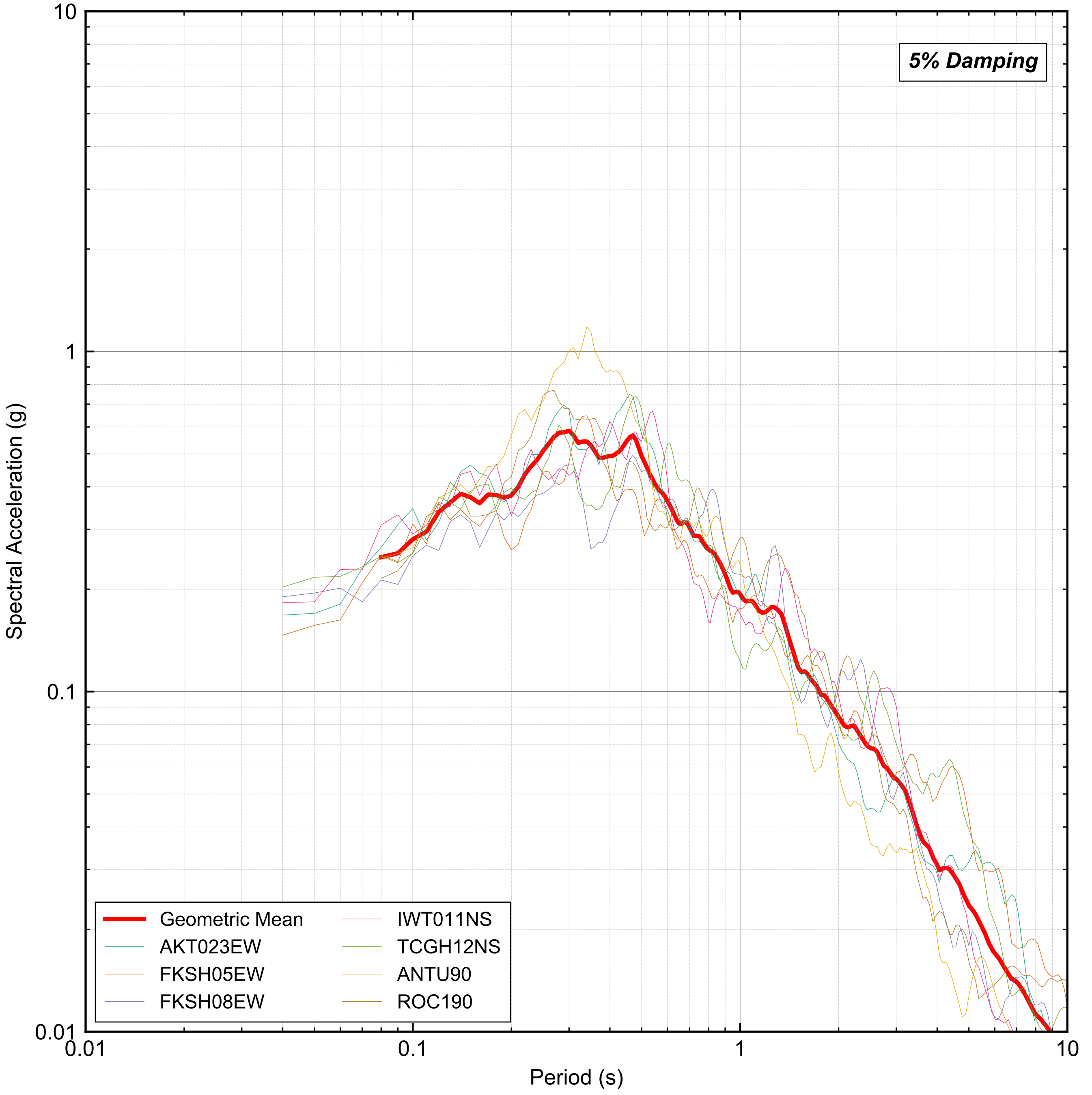
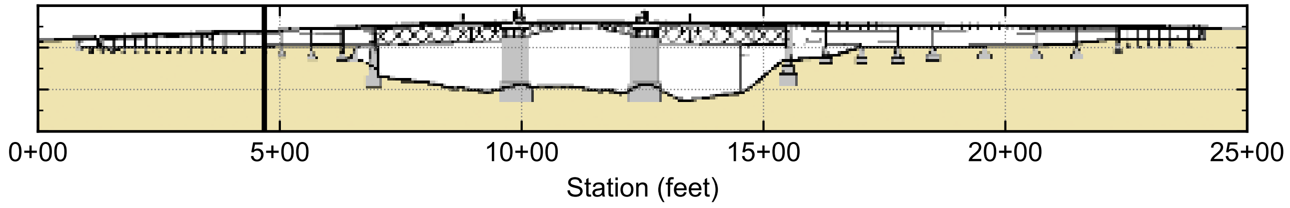
FIG. E-15





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 16

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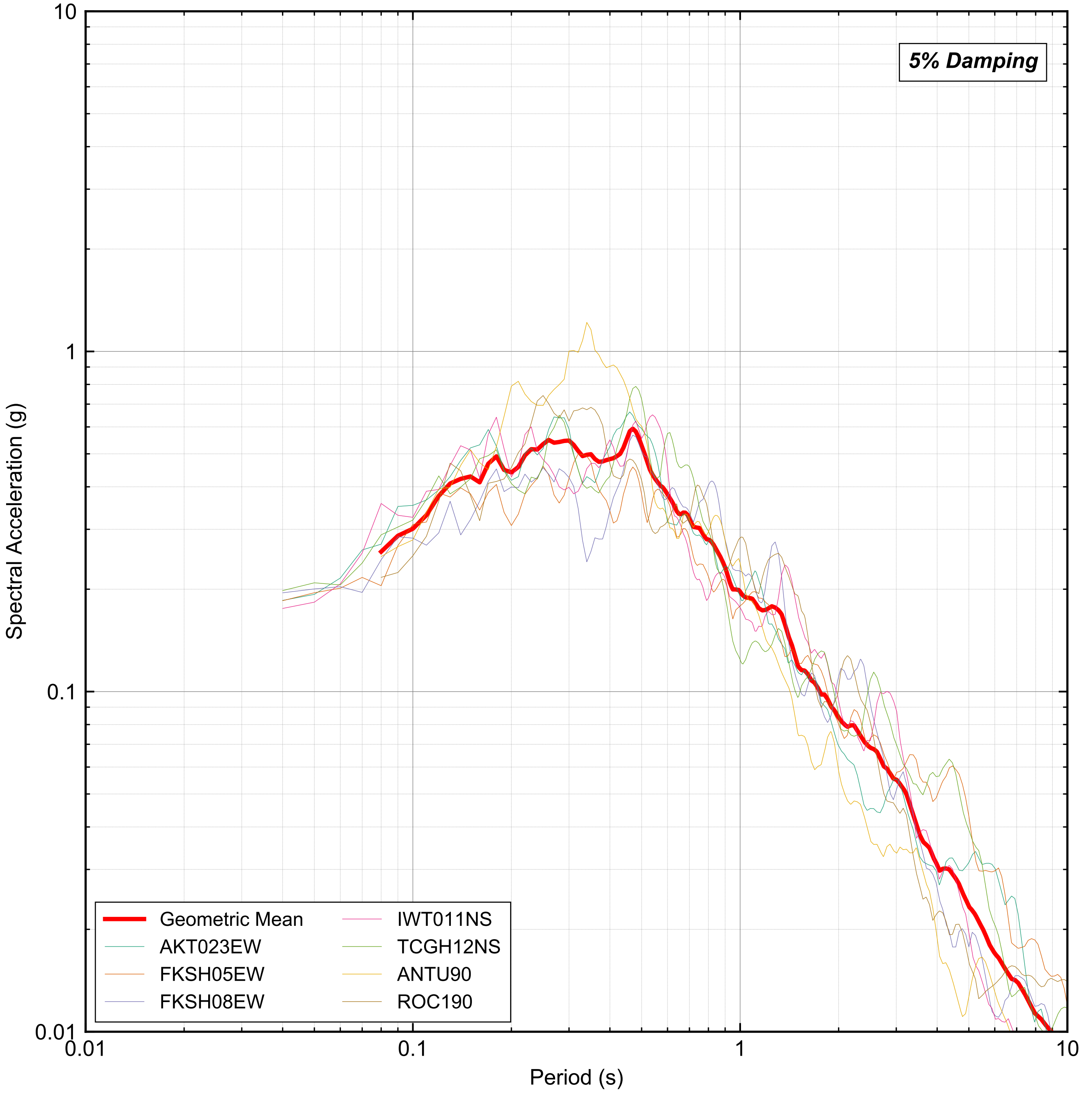
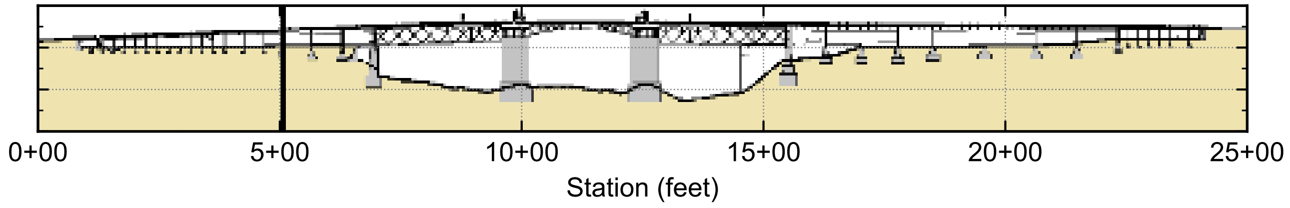
FIG. E-16





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 17

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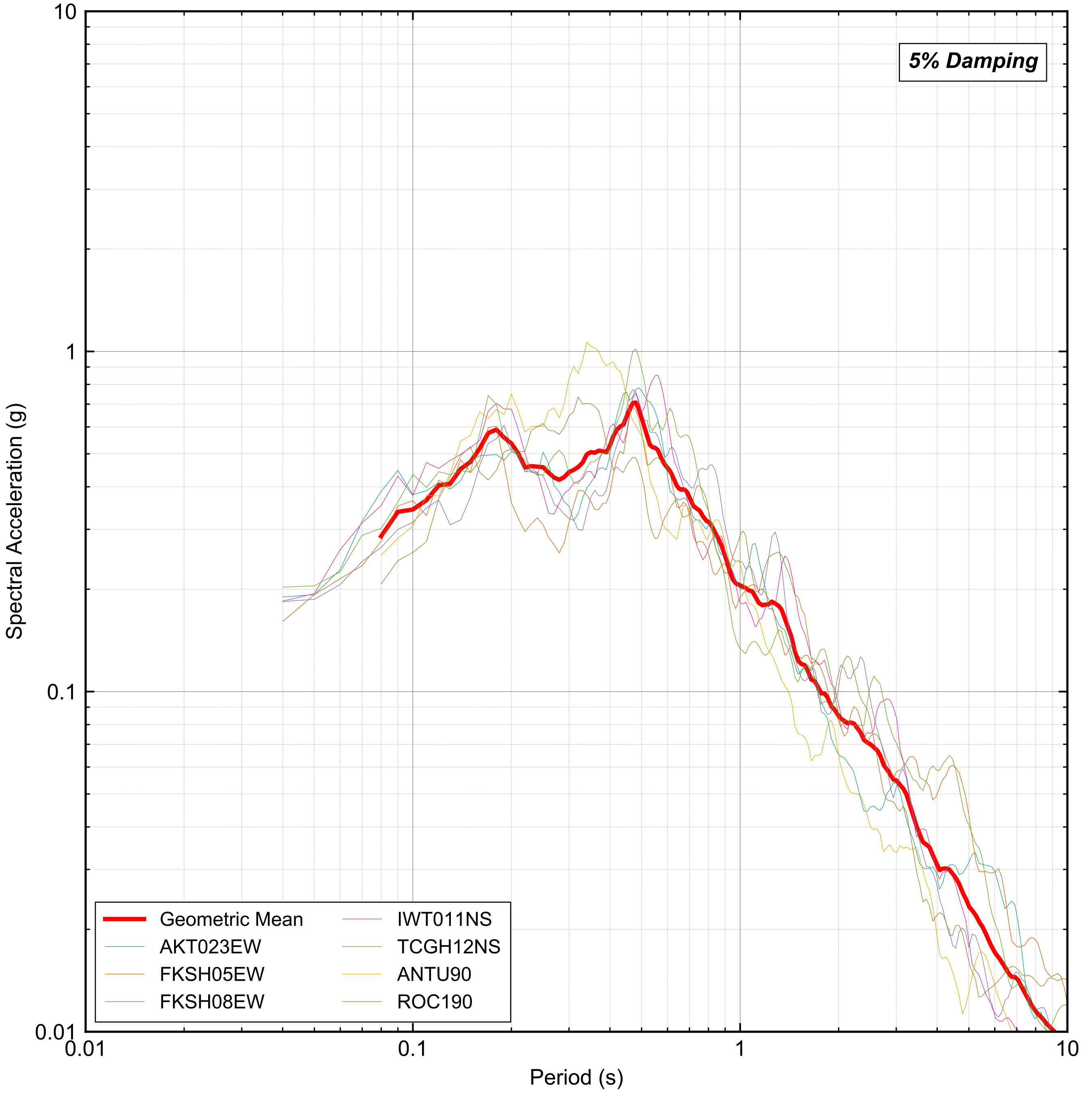
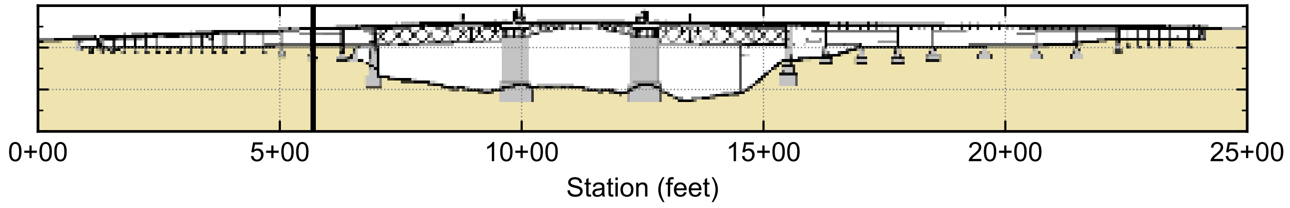
FIG. E-17





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 18

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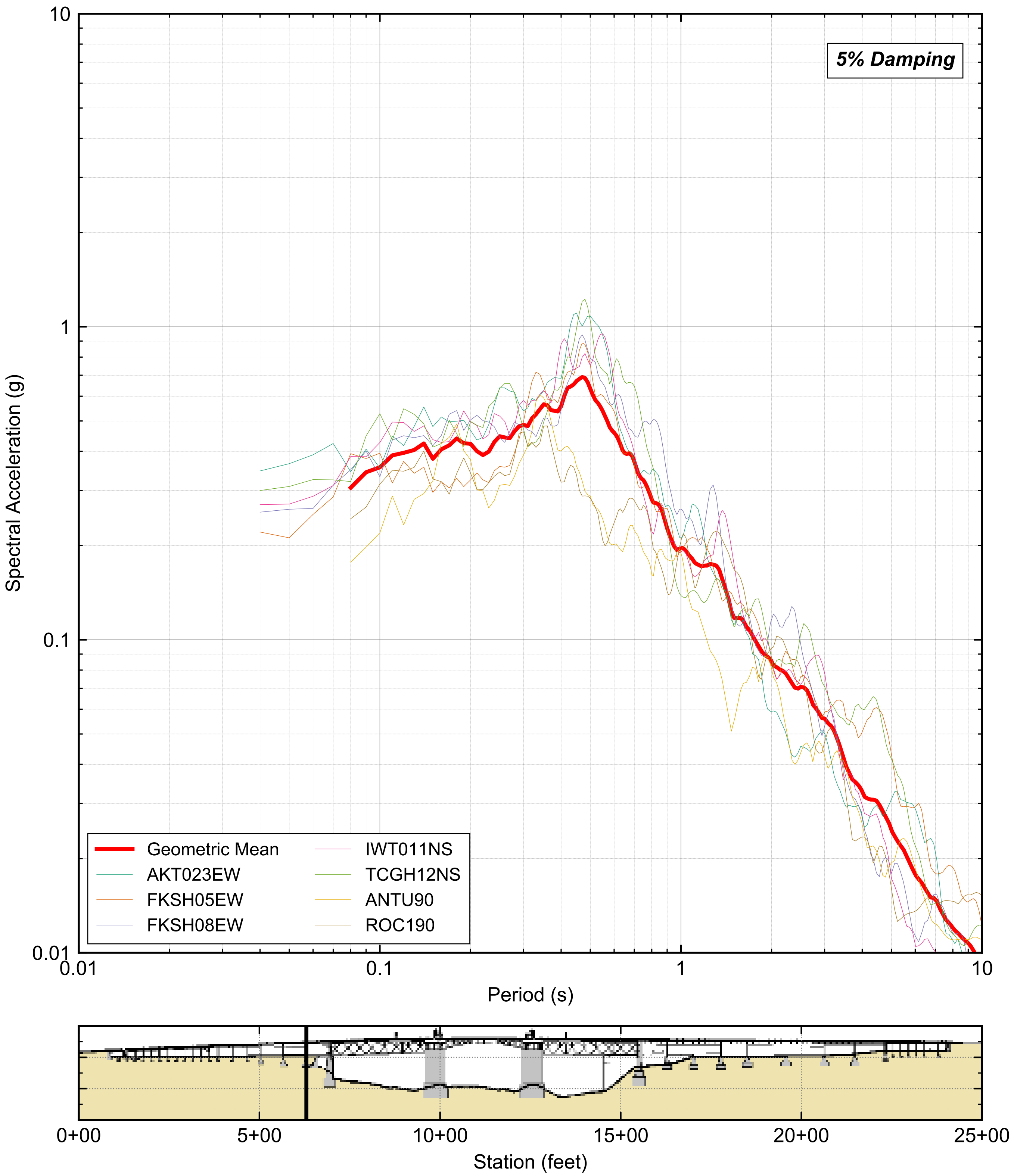
FIG. E-18

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 19

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FIG. E-19

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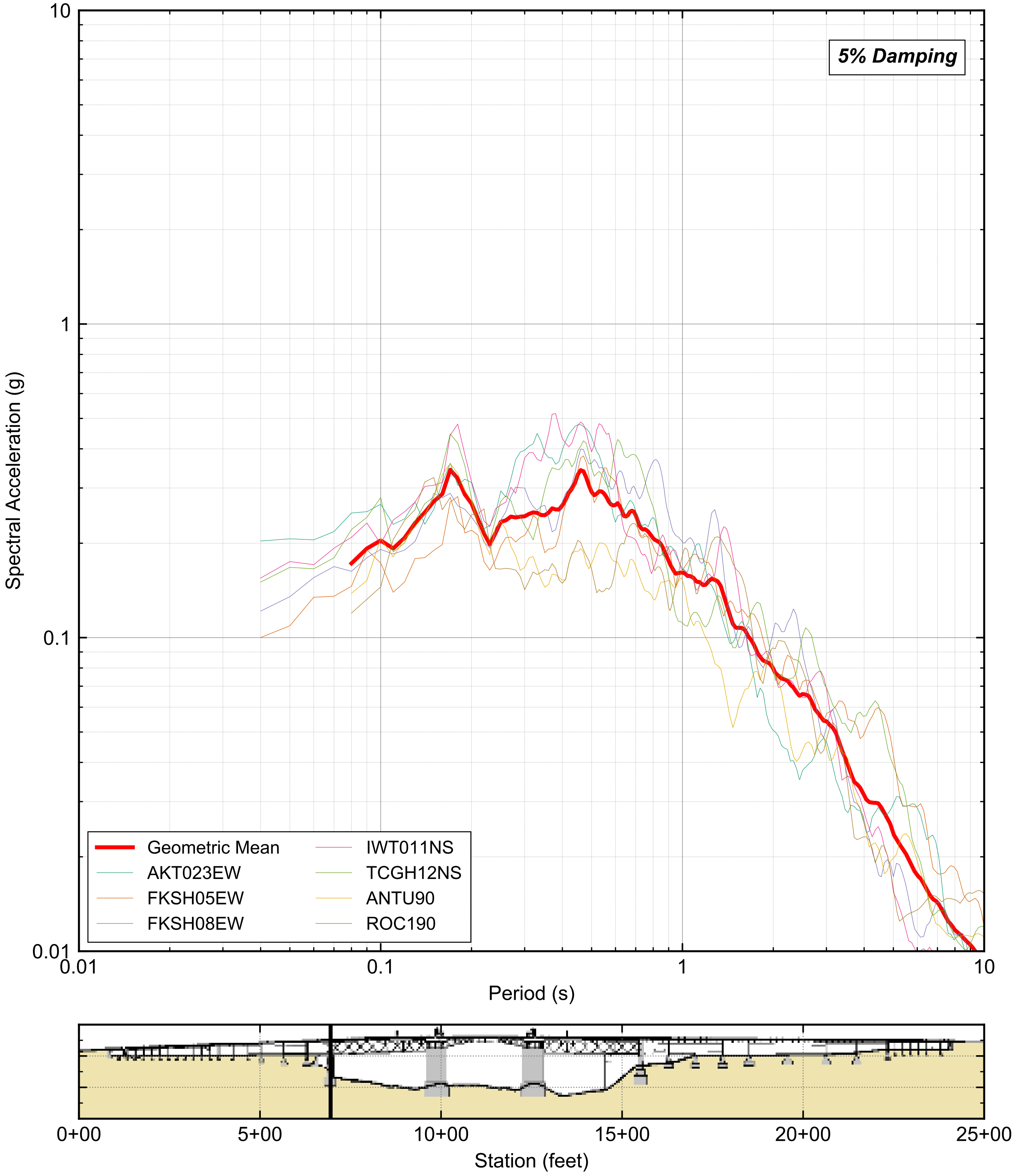
ver:0.8.5.post0.dev0+g414b689 by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING PIER 1

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FIG. E-20

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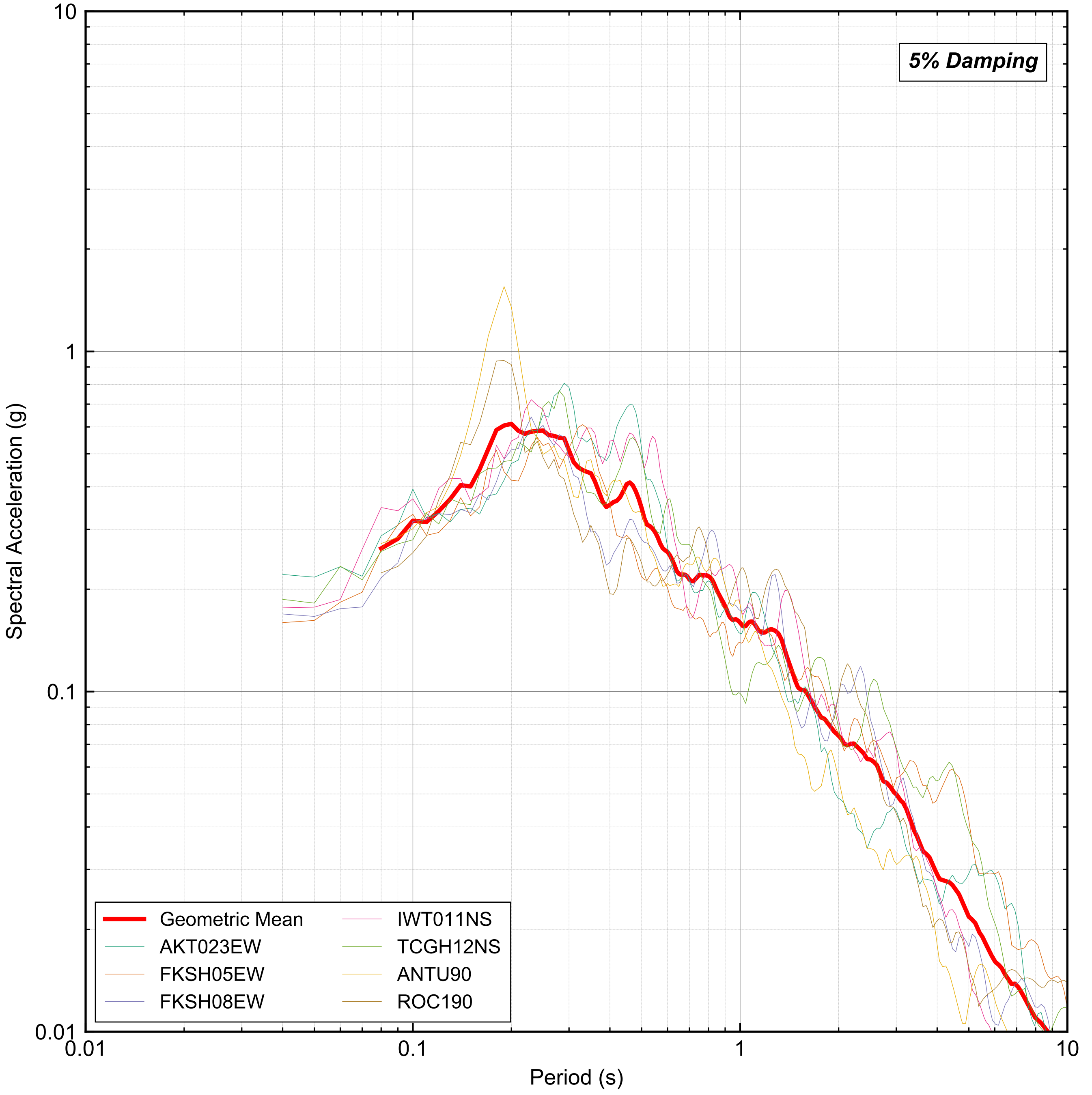

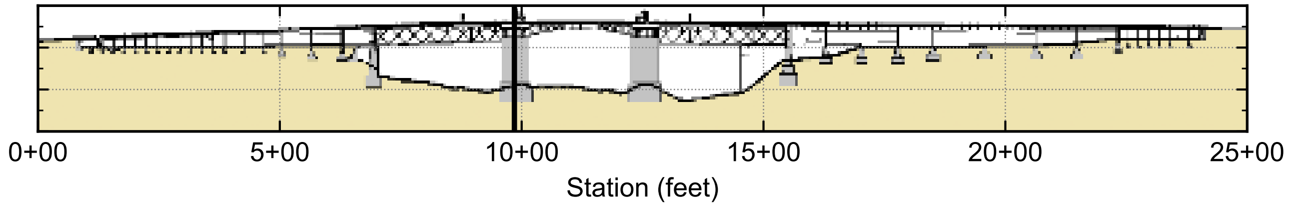

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING PIER 2

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FIG. E-21

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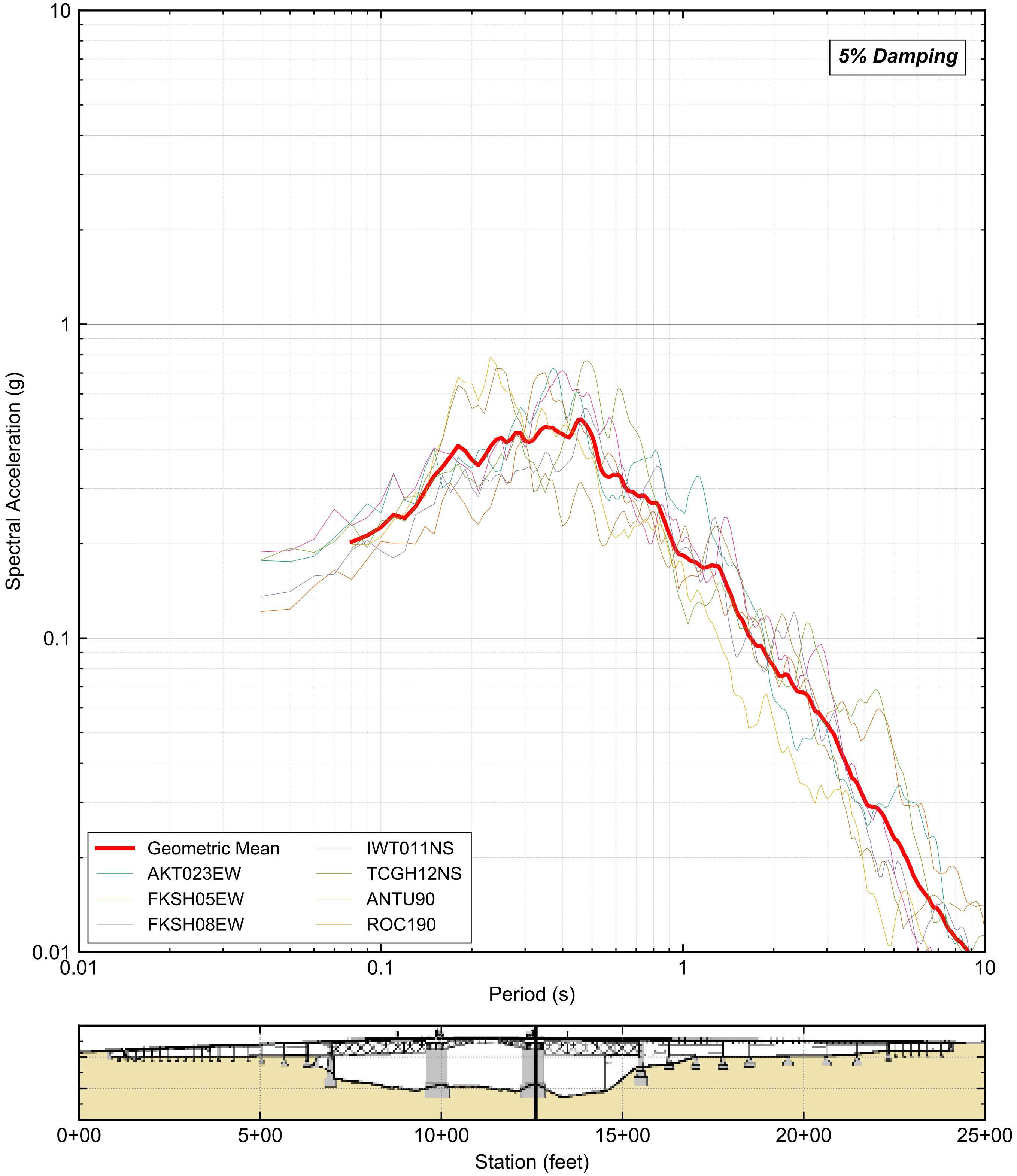
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING PIER 3

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FIG. E-22

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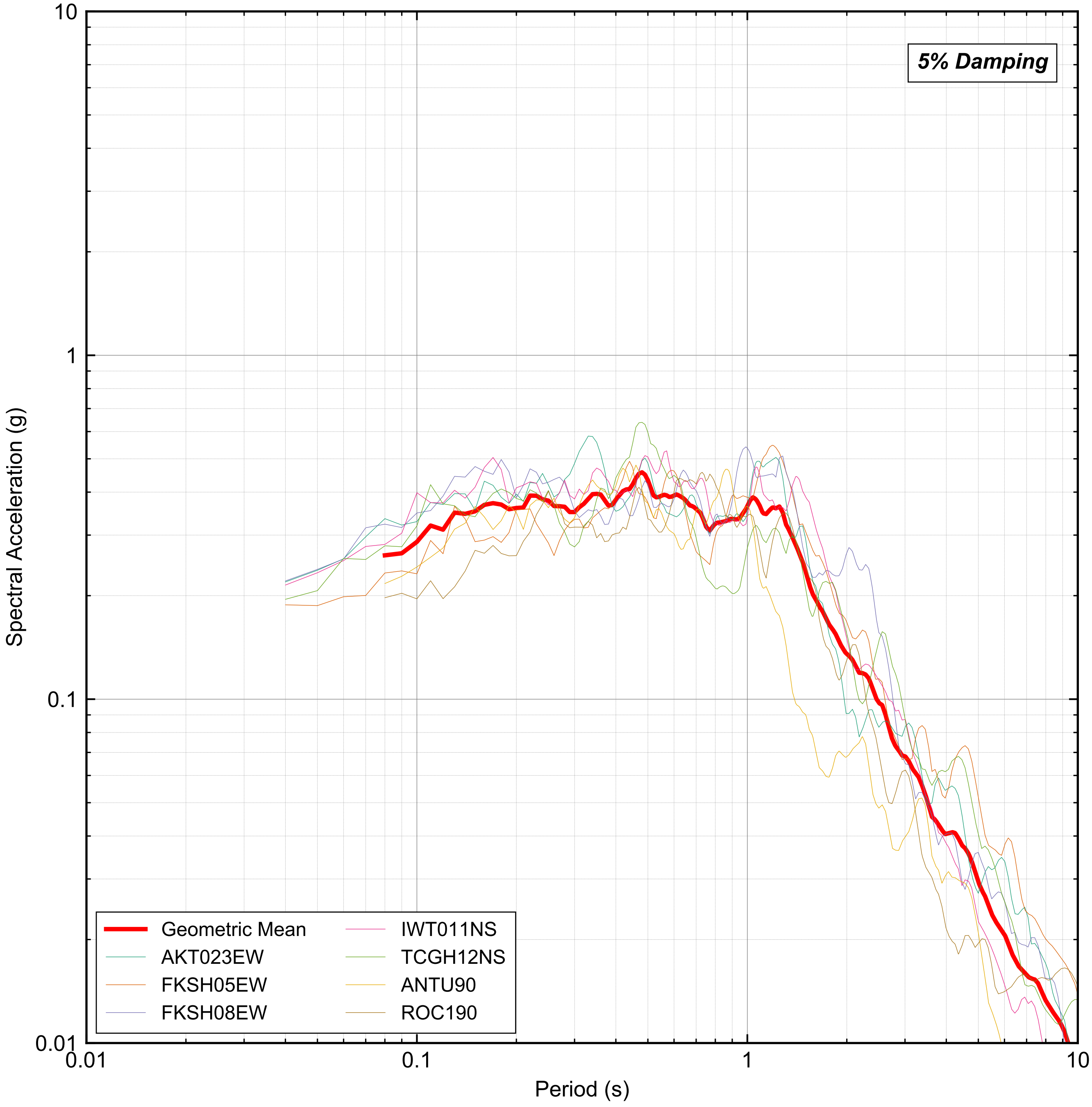

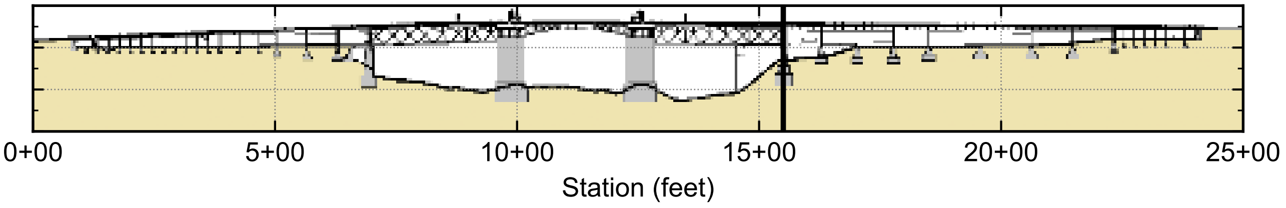

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING PIER 4

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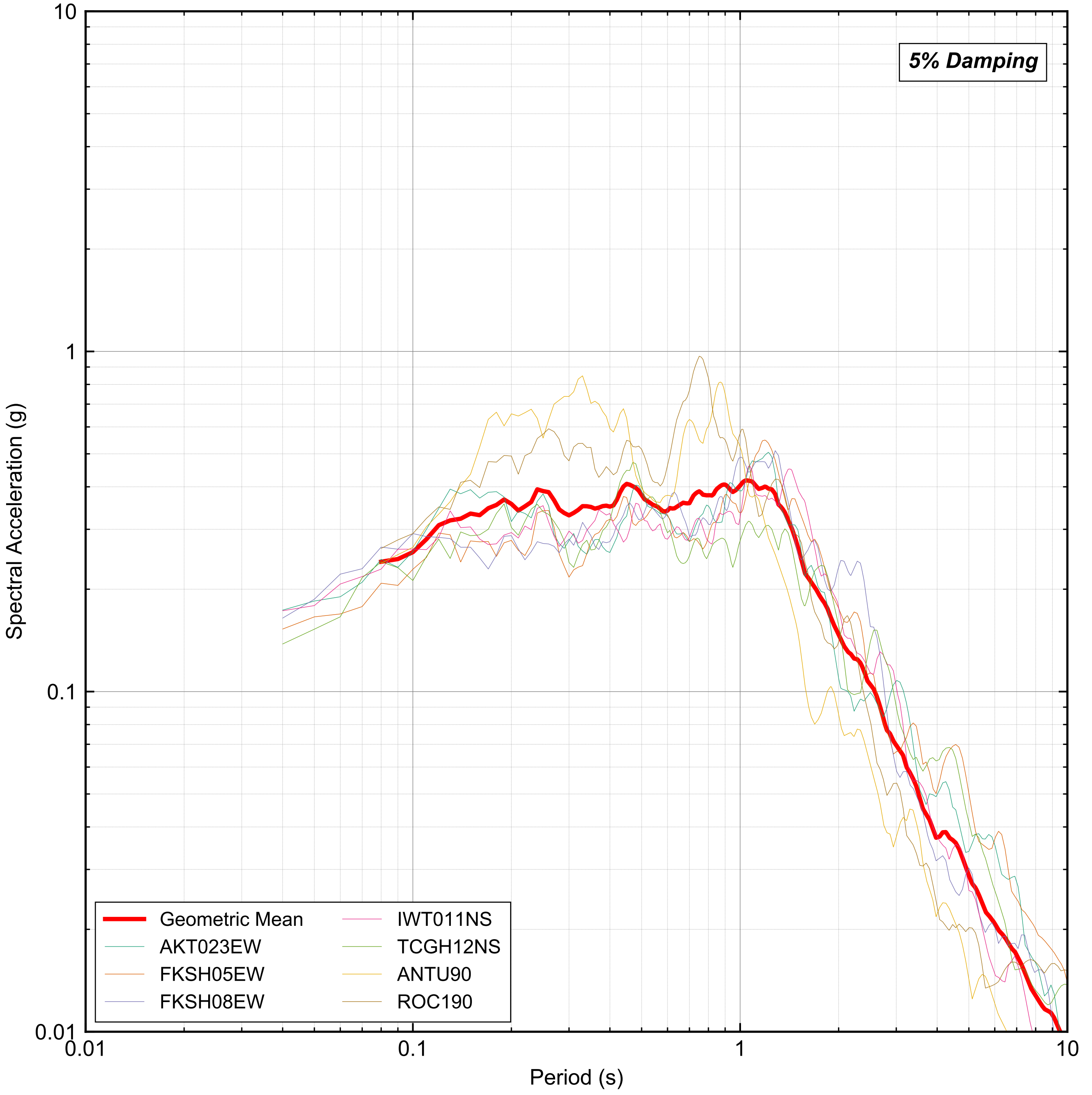
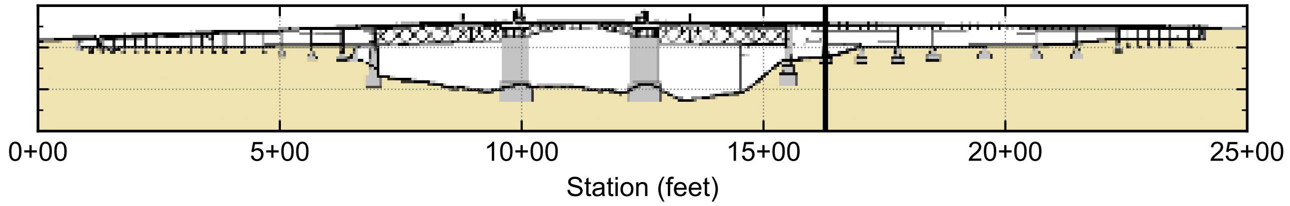
FIG. E-23

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 21

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FIG. E-24

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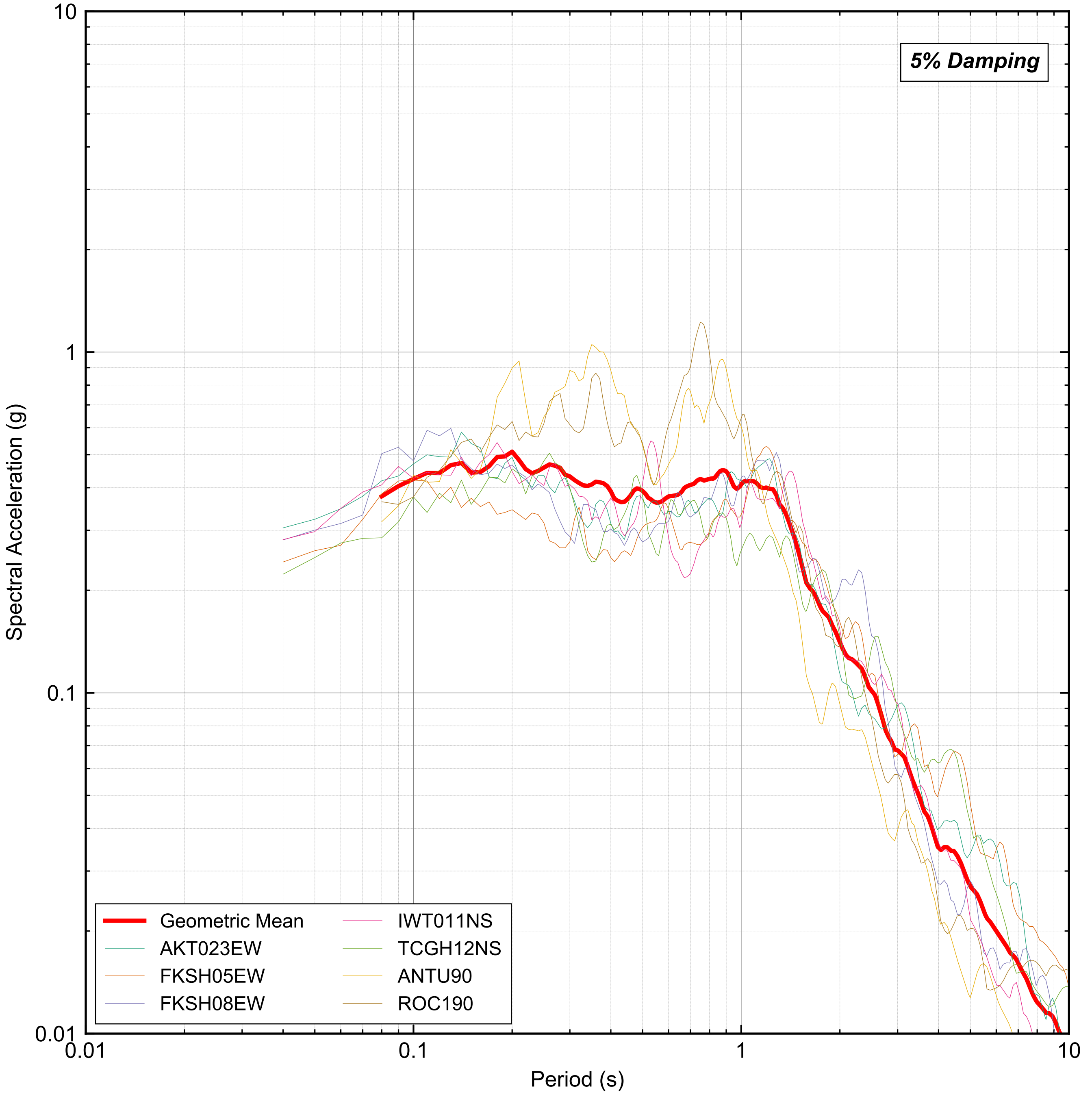

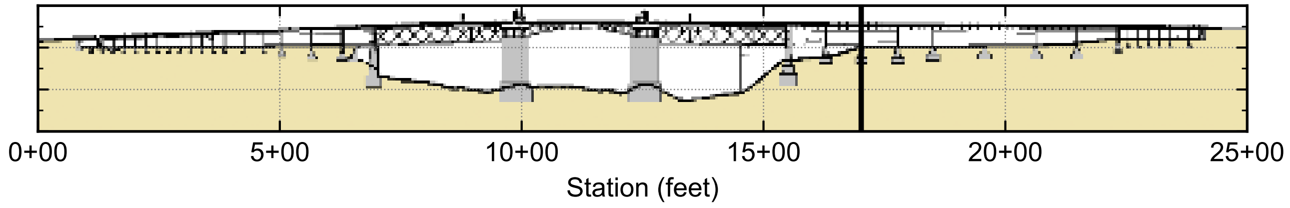

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 22

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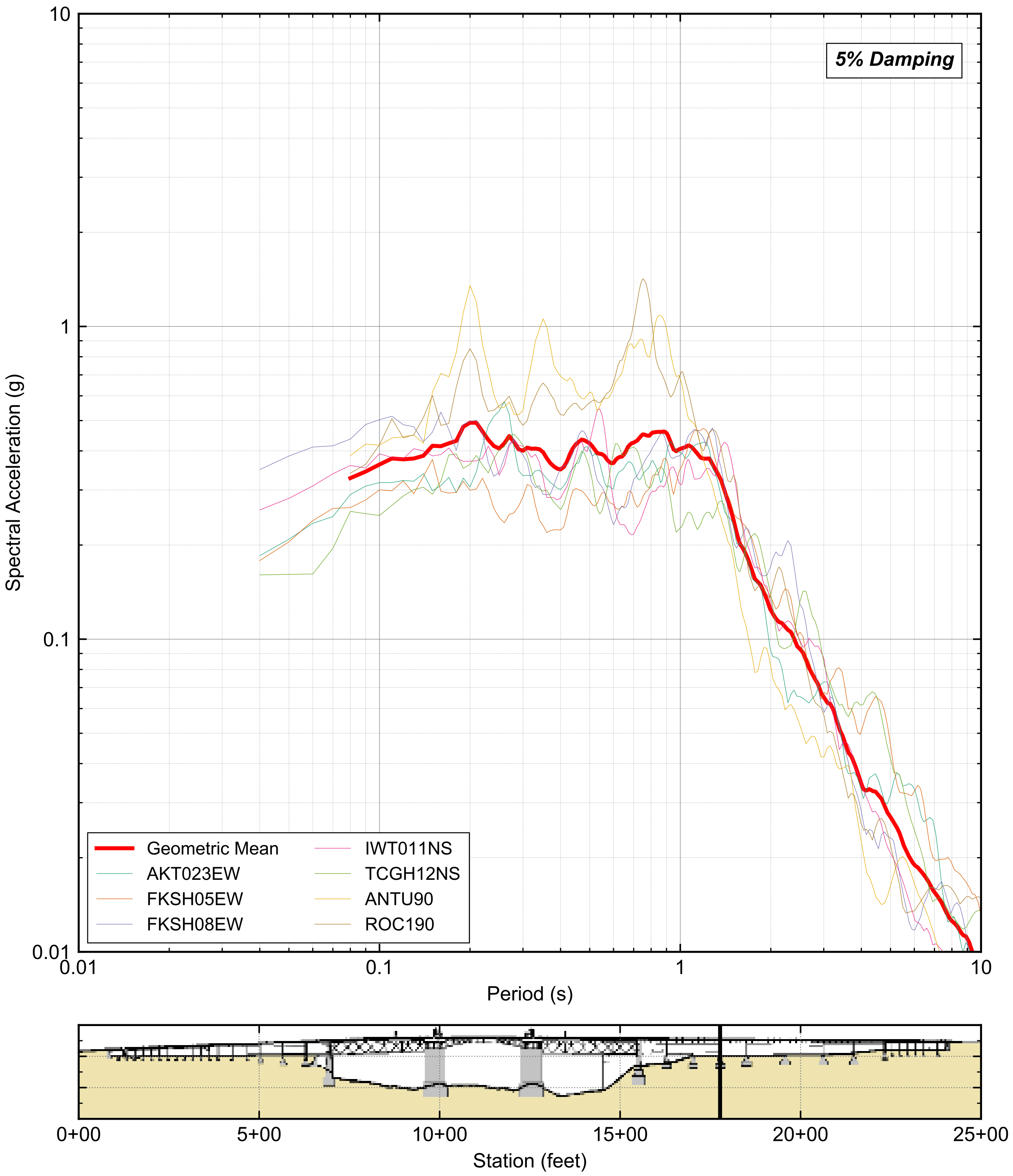
FIG. E-25

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 23

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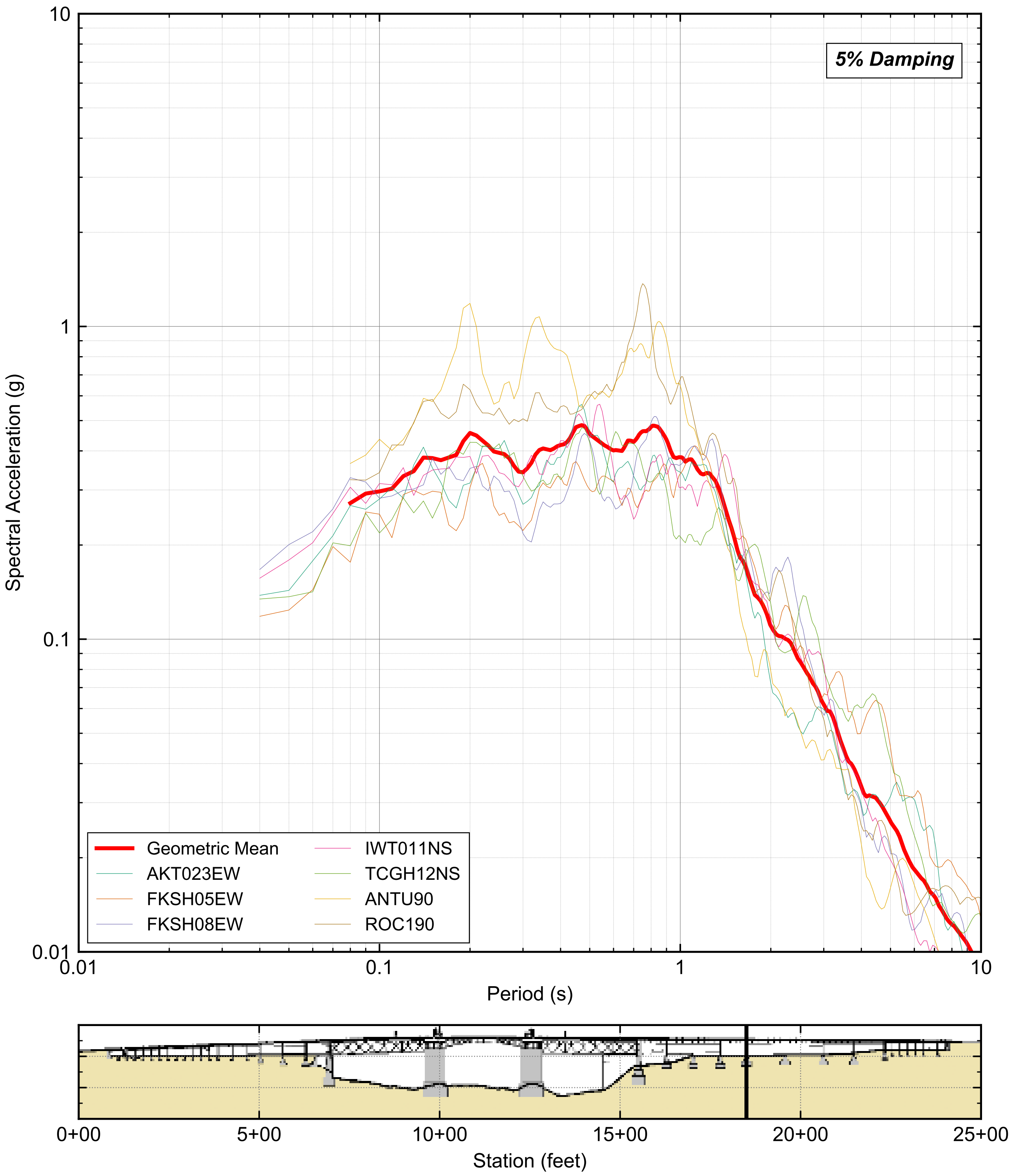
FIG. E-26

C:\Users\jnb\FLAC\Burnside\v6_test

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 24

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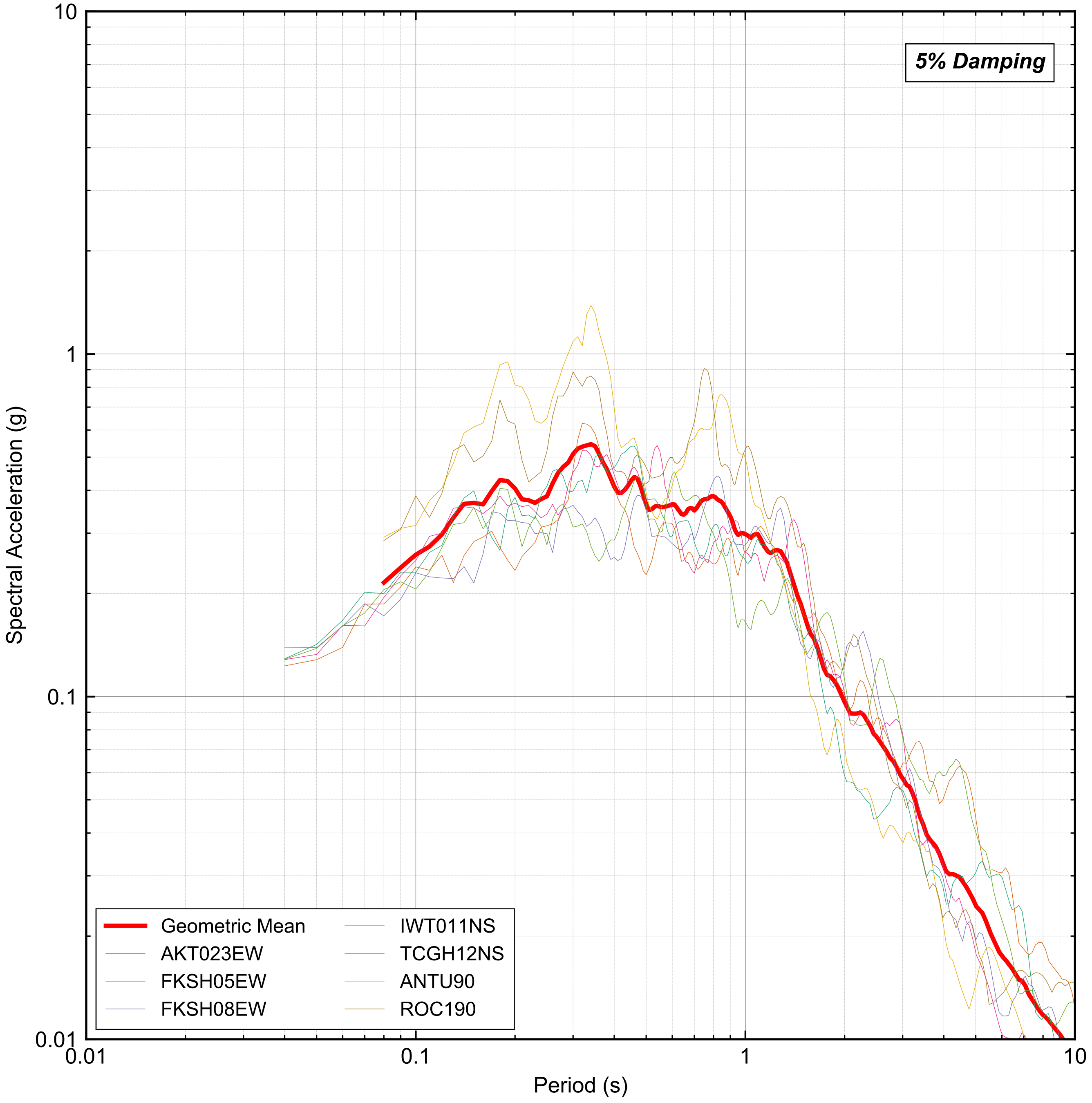
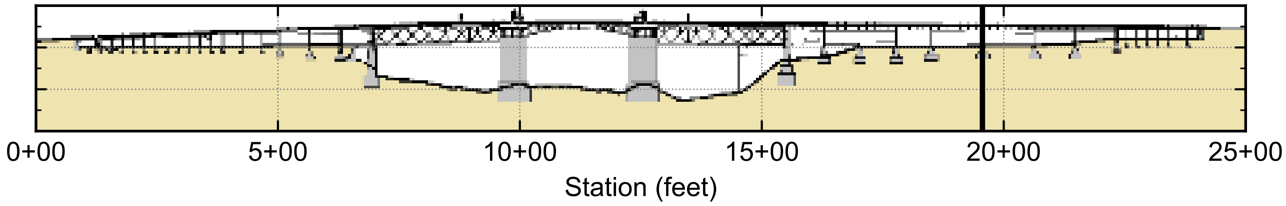
FIG. E-27

C:\Users\jnb\FLAC\Burnside\v6_test

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 25

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FIG. E-28

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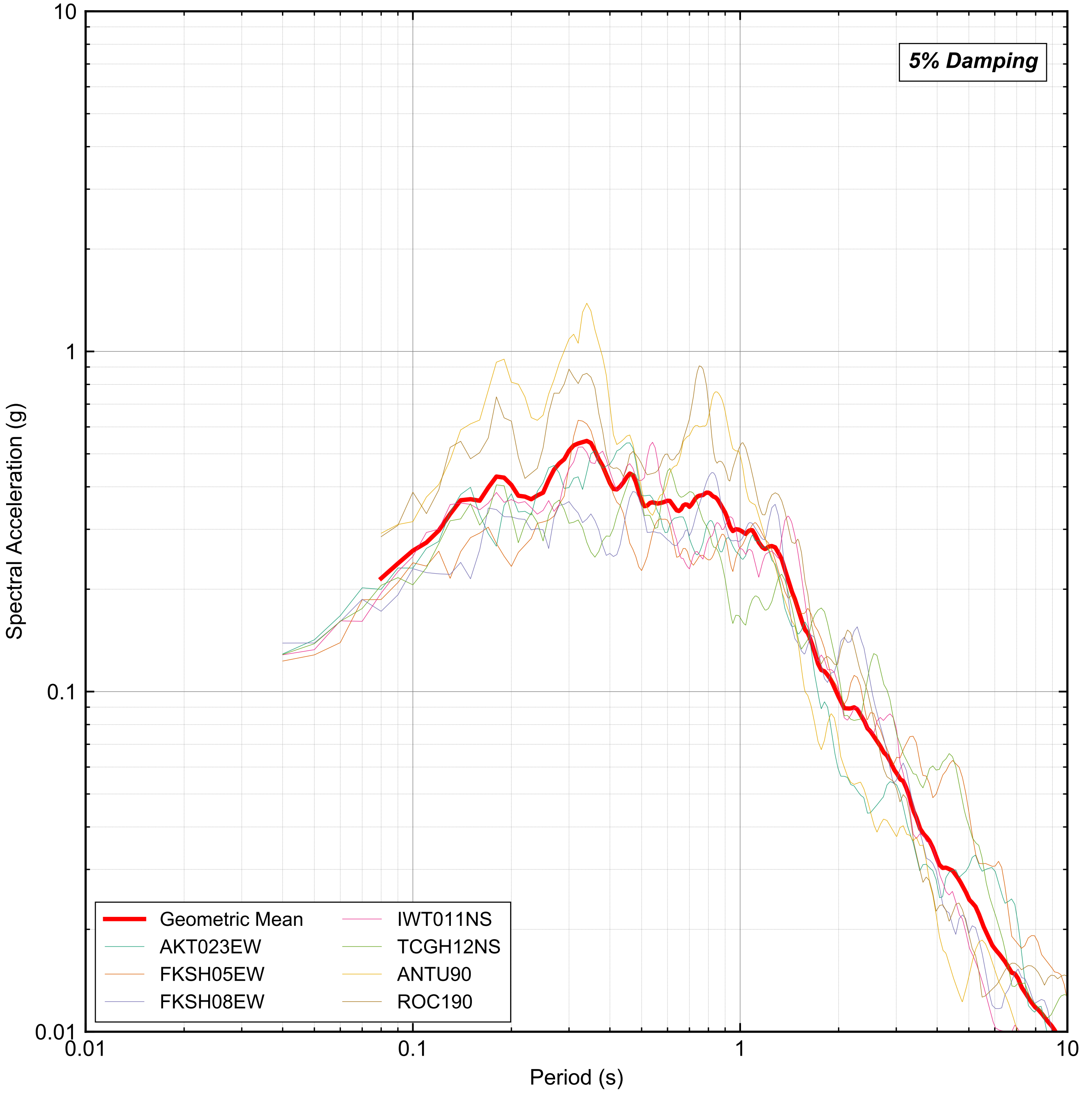

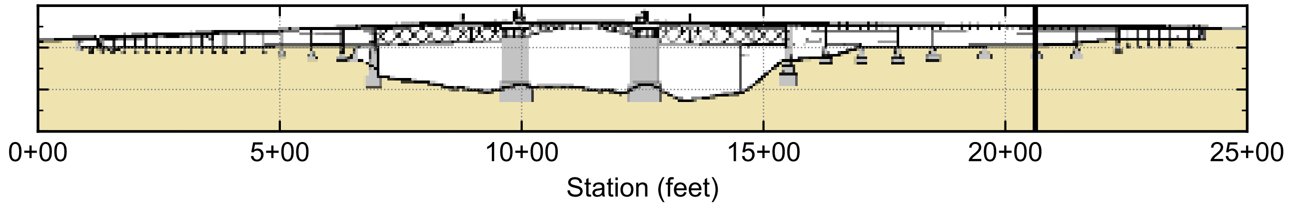

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 26

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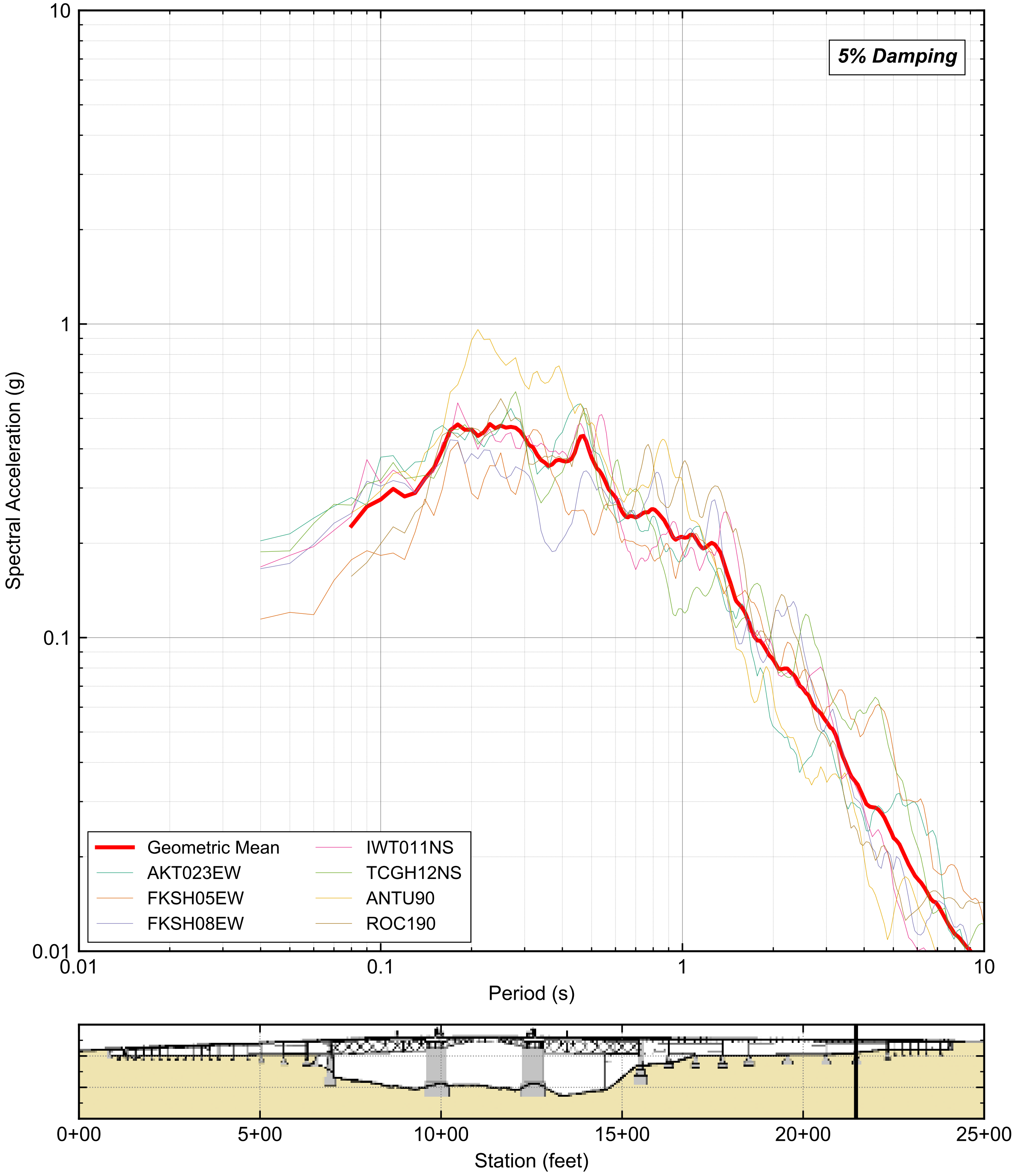
FIG. E-29



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 27

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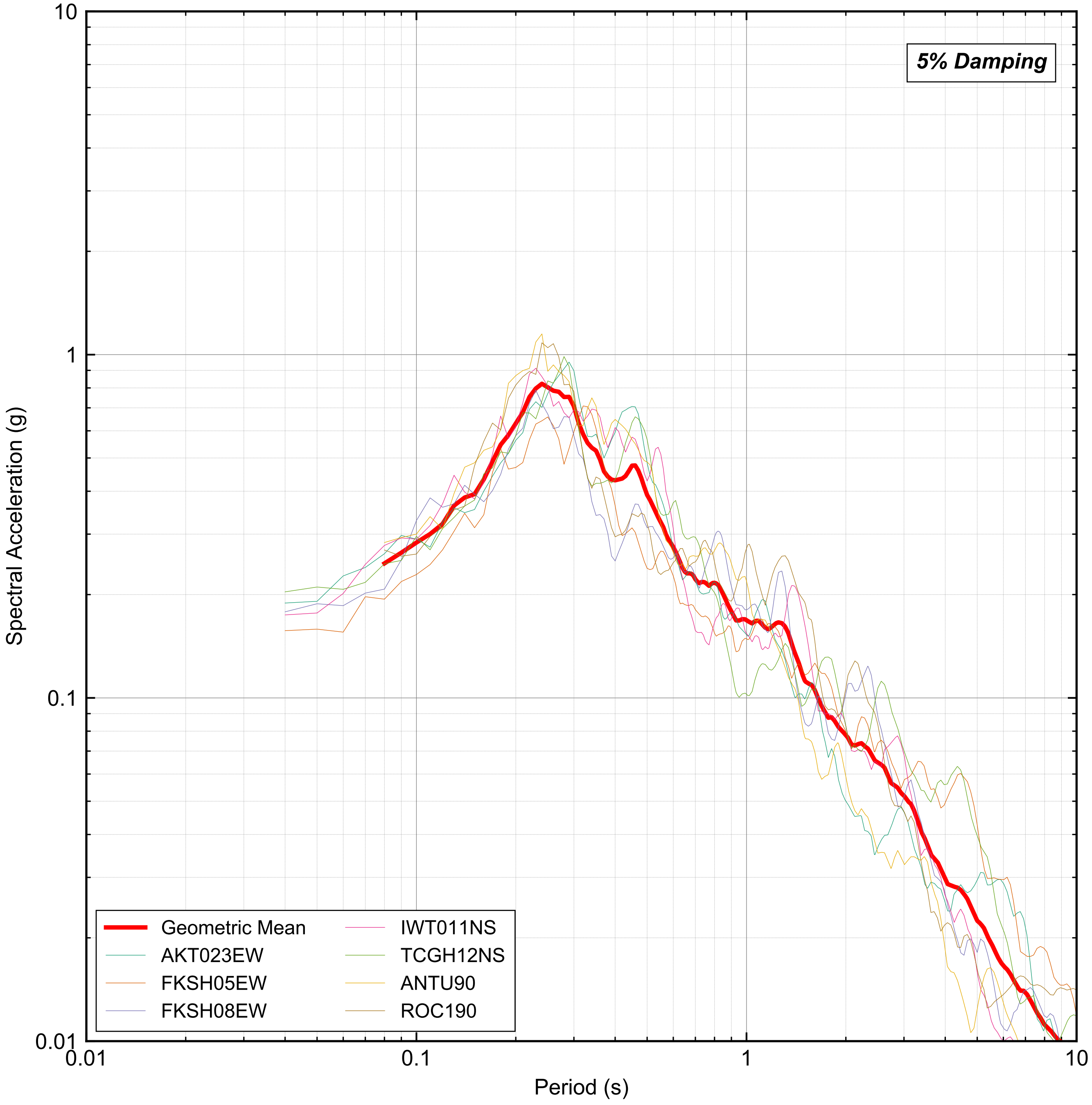
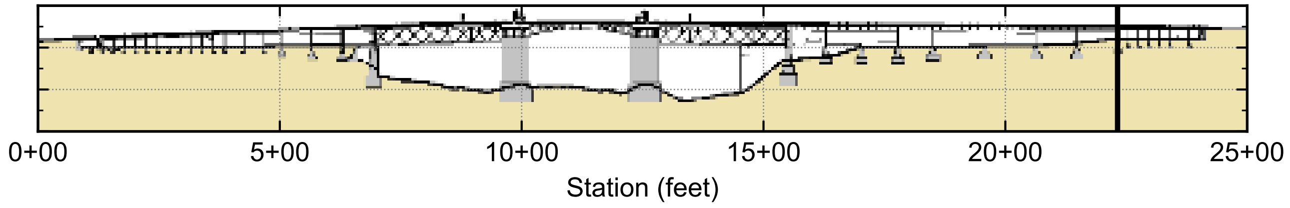
FIG. E-30





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 28

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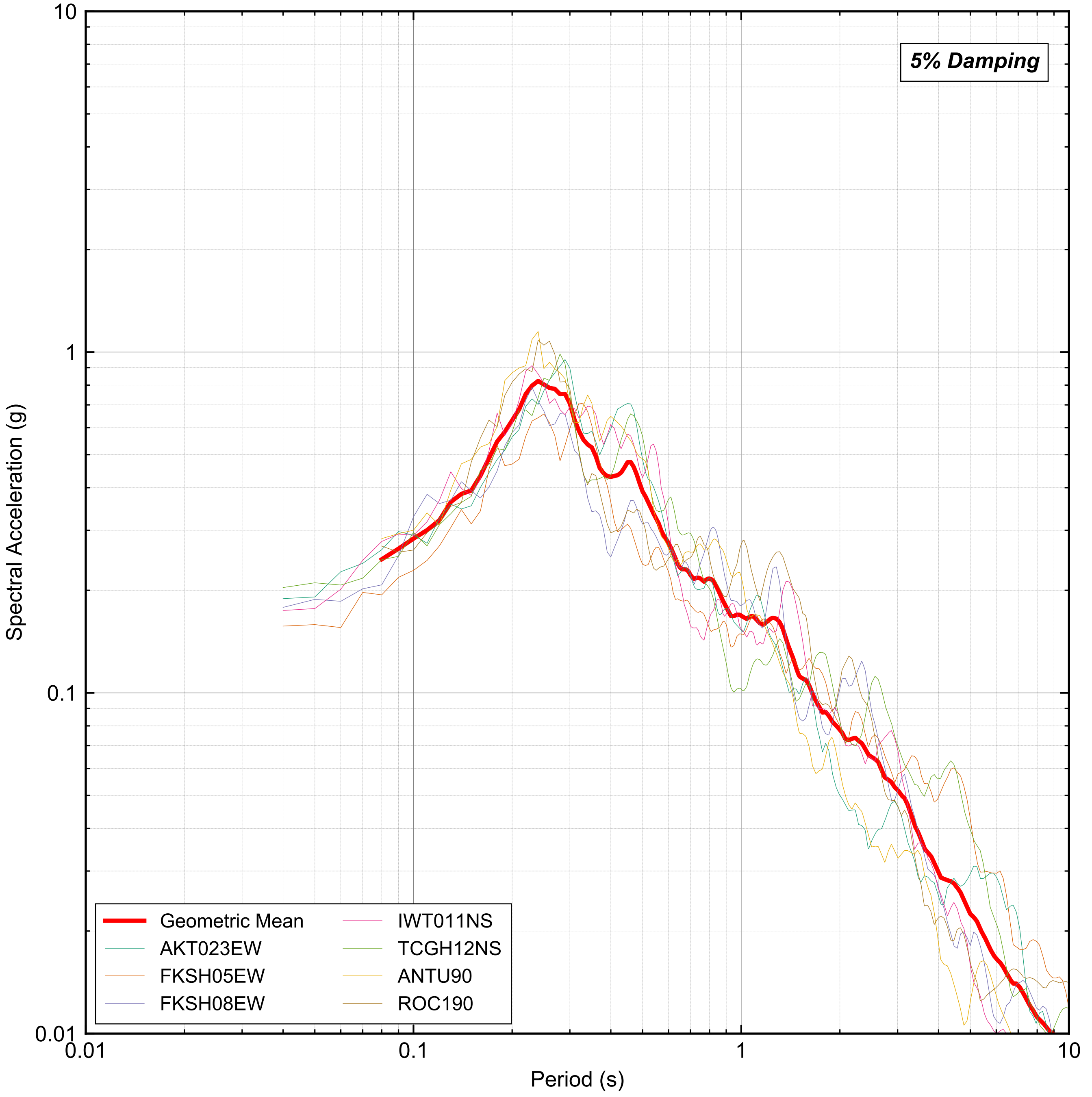
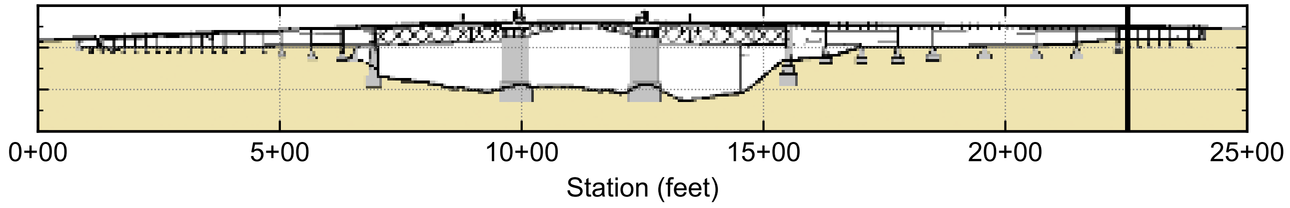
FIG. E-31





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 29

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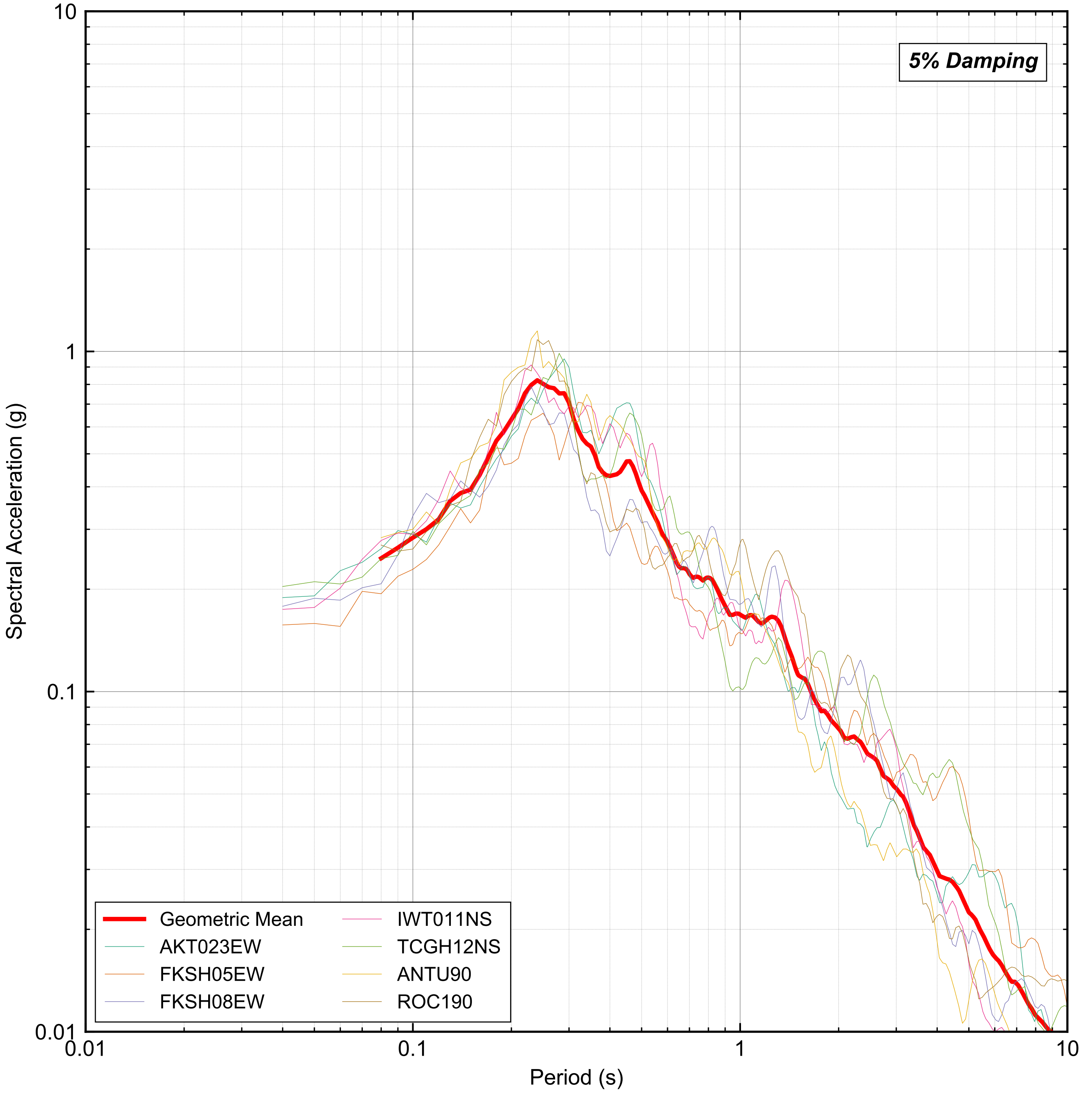
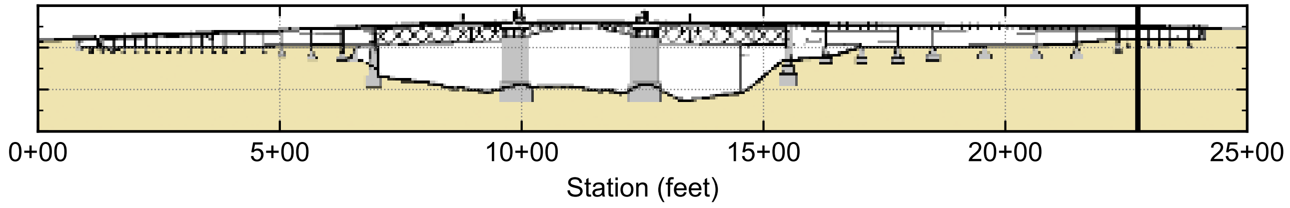
FIG. E-32





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 30

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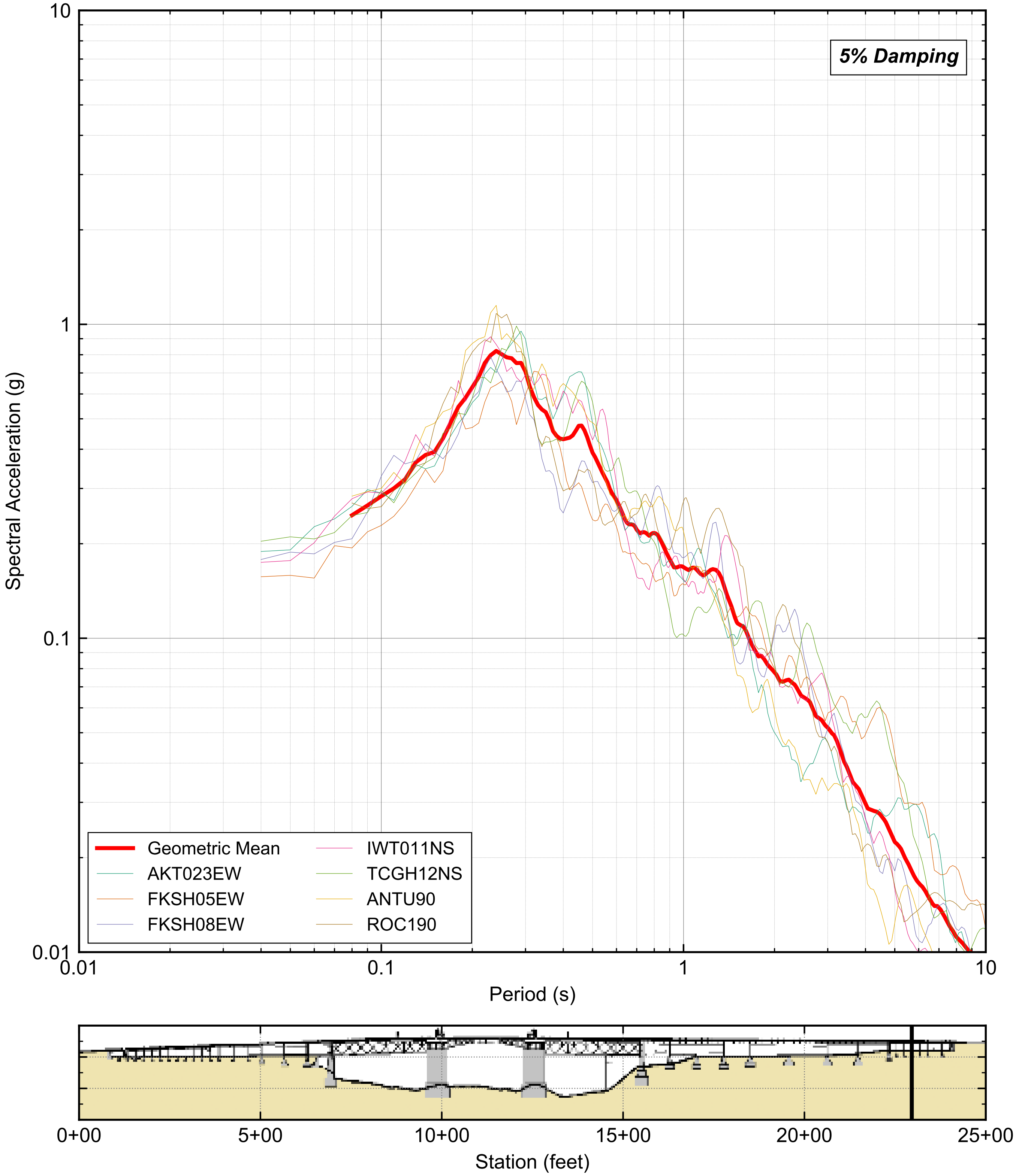
FIG. E-33

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:

1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 31

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FIG. E-34

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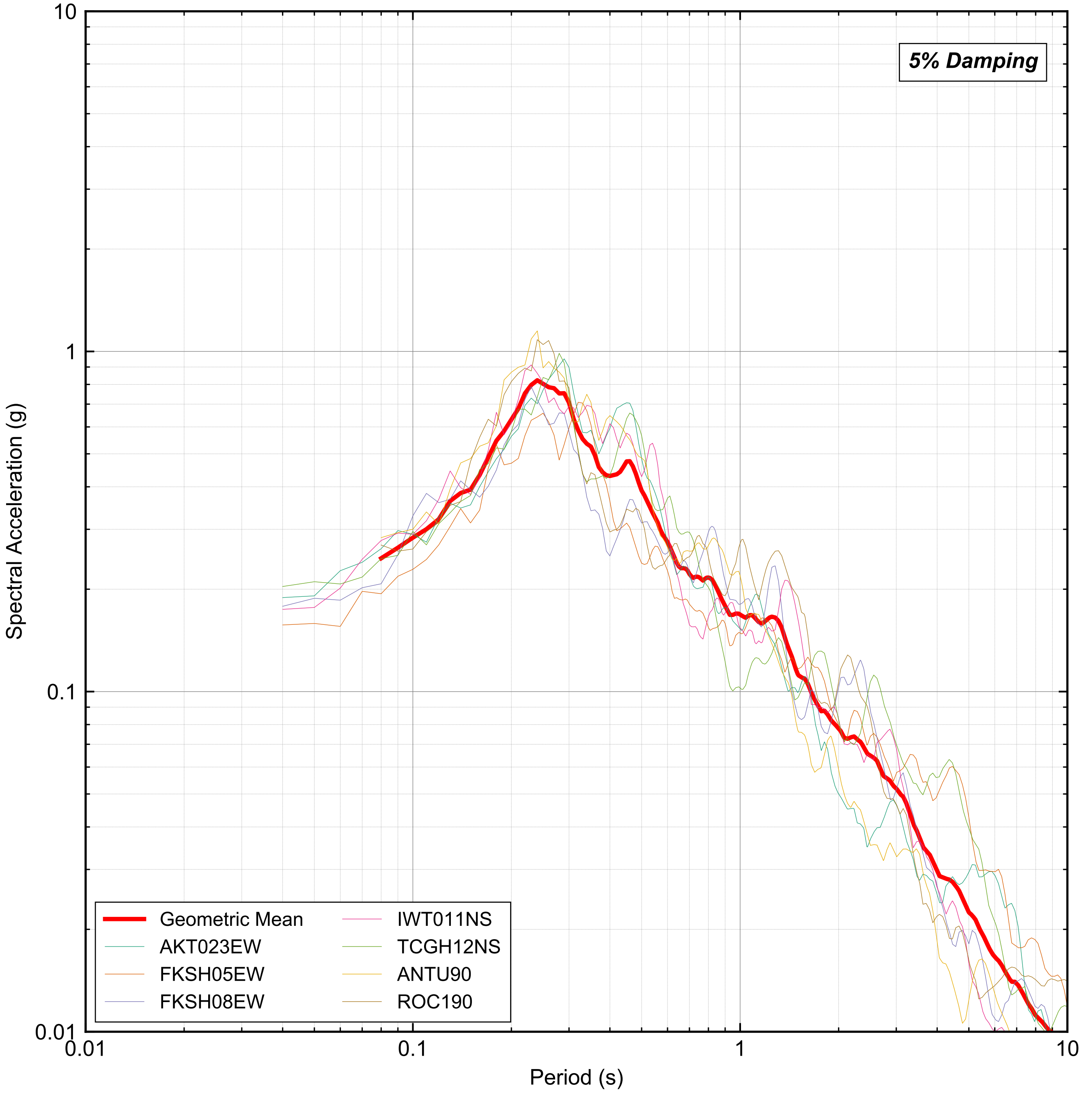
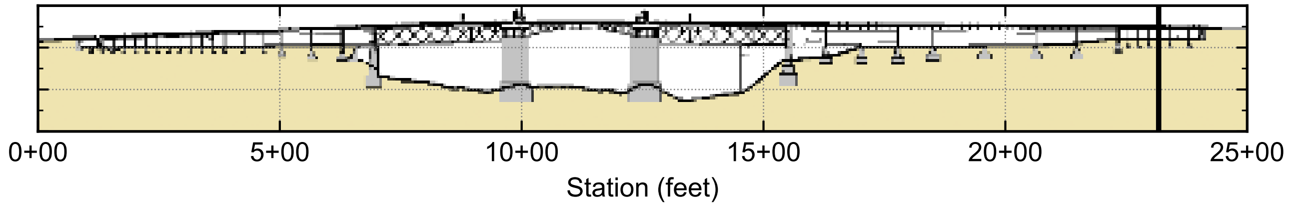
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 32

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FIG. E-35

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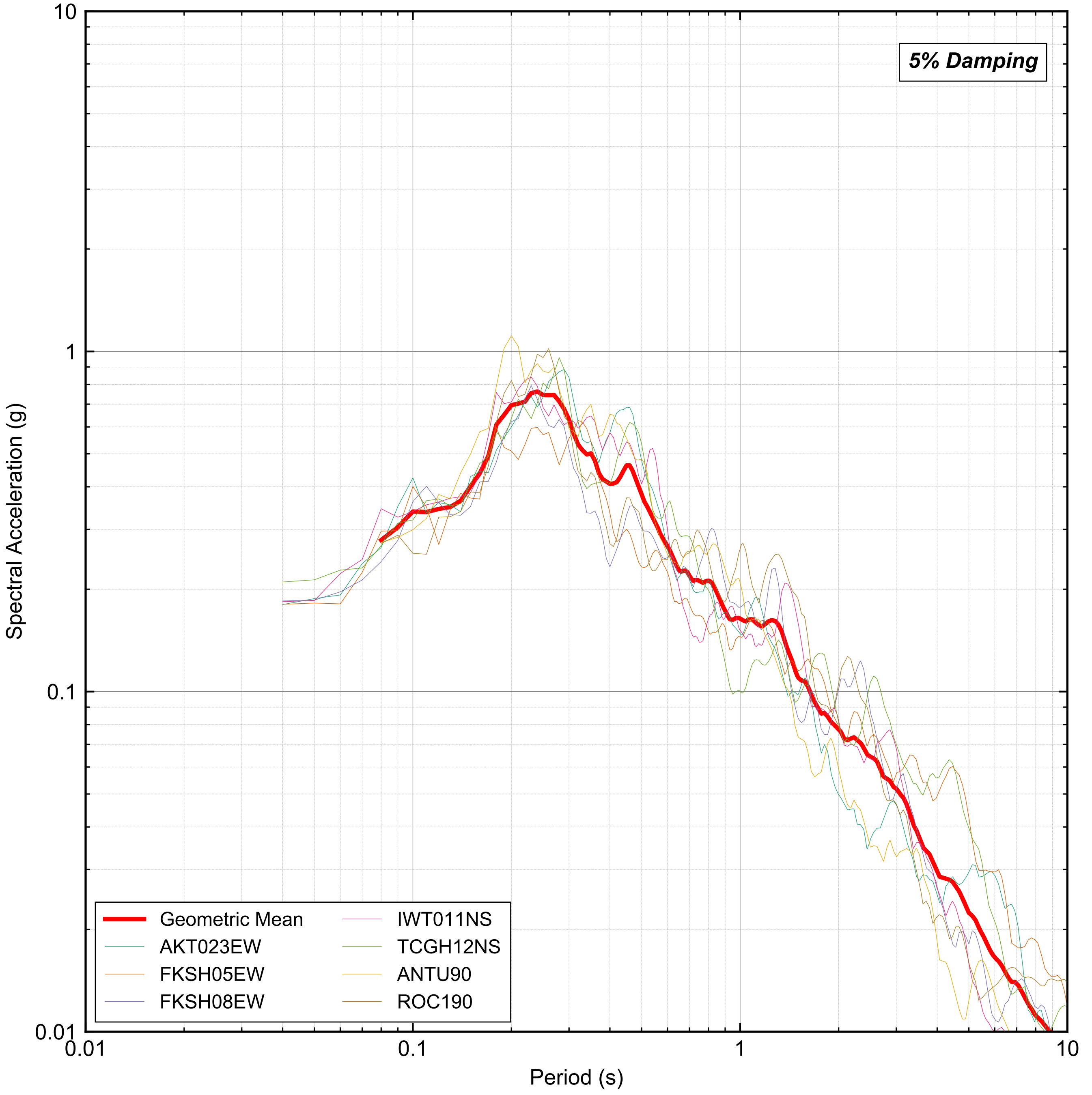

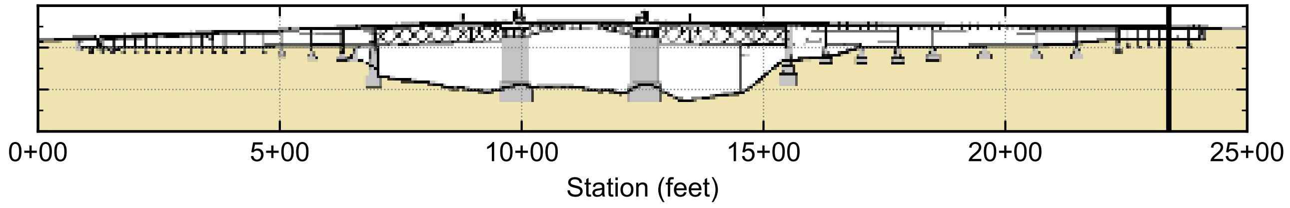

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 33

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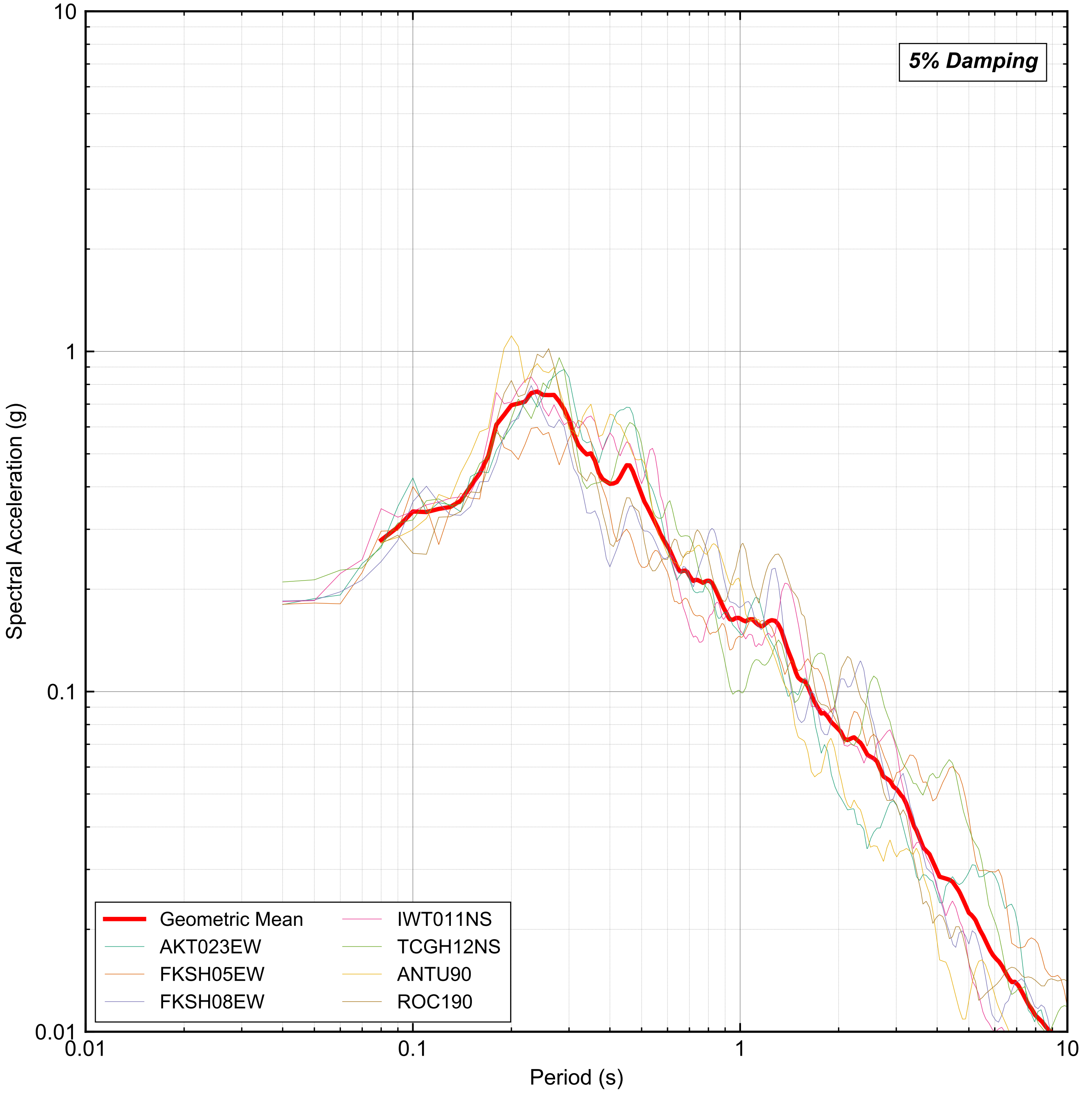
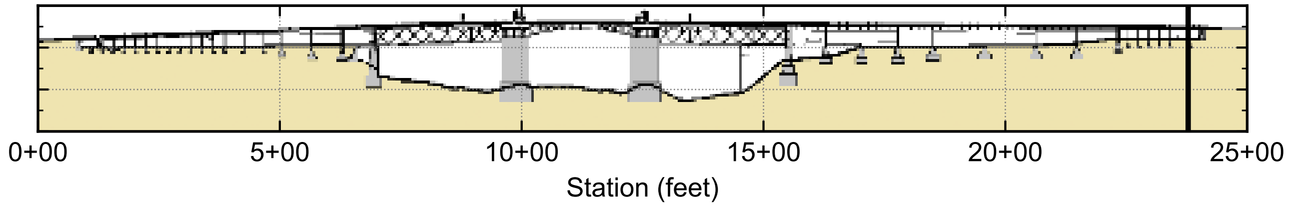
FIG. E-36





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 34

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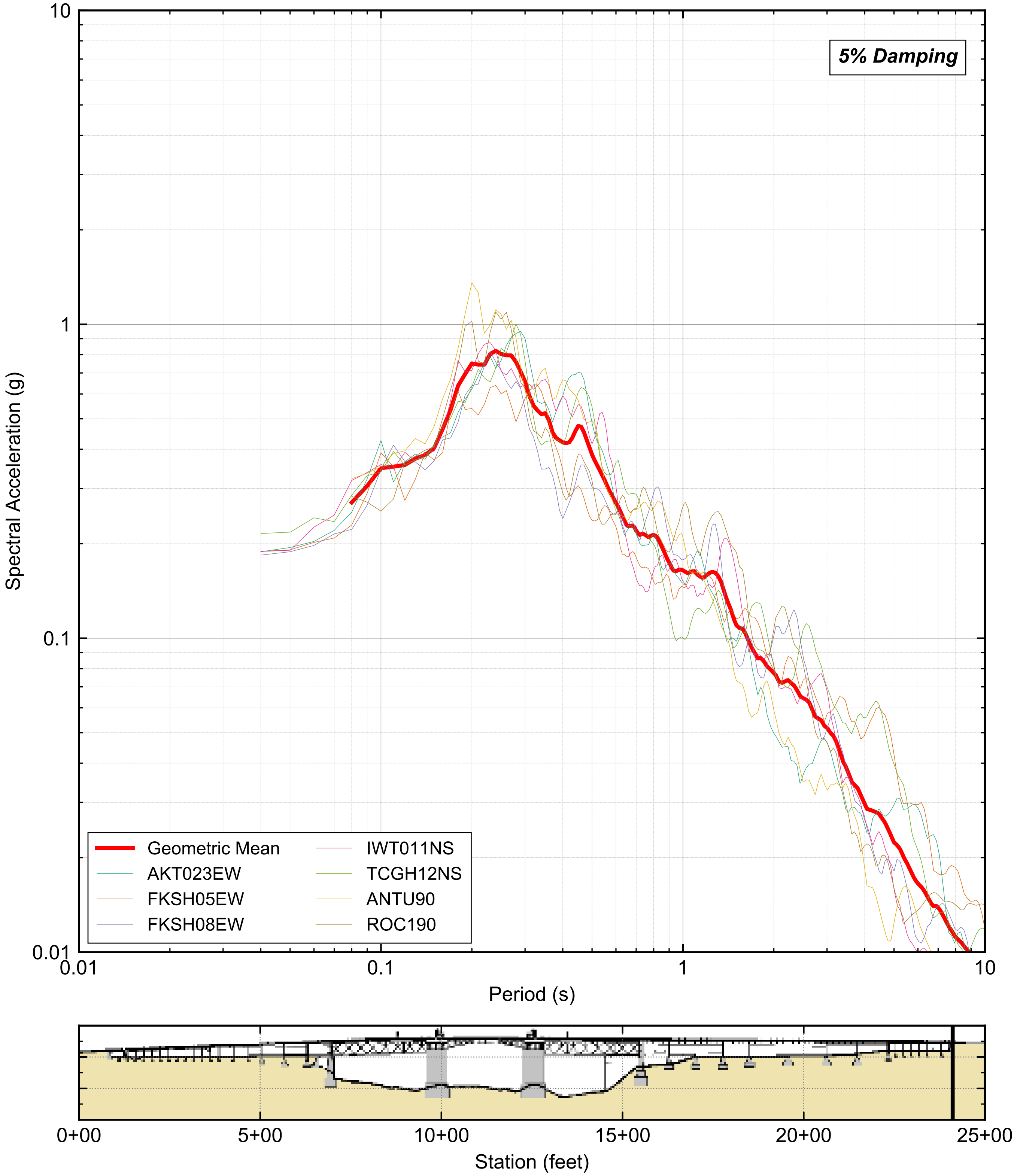
FIG. E-37

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
FULL OPERATION
EXISTING BENT 35

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FIG. E-38

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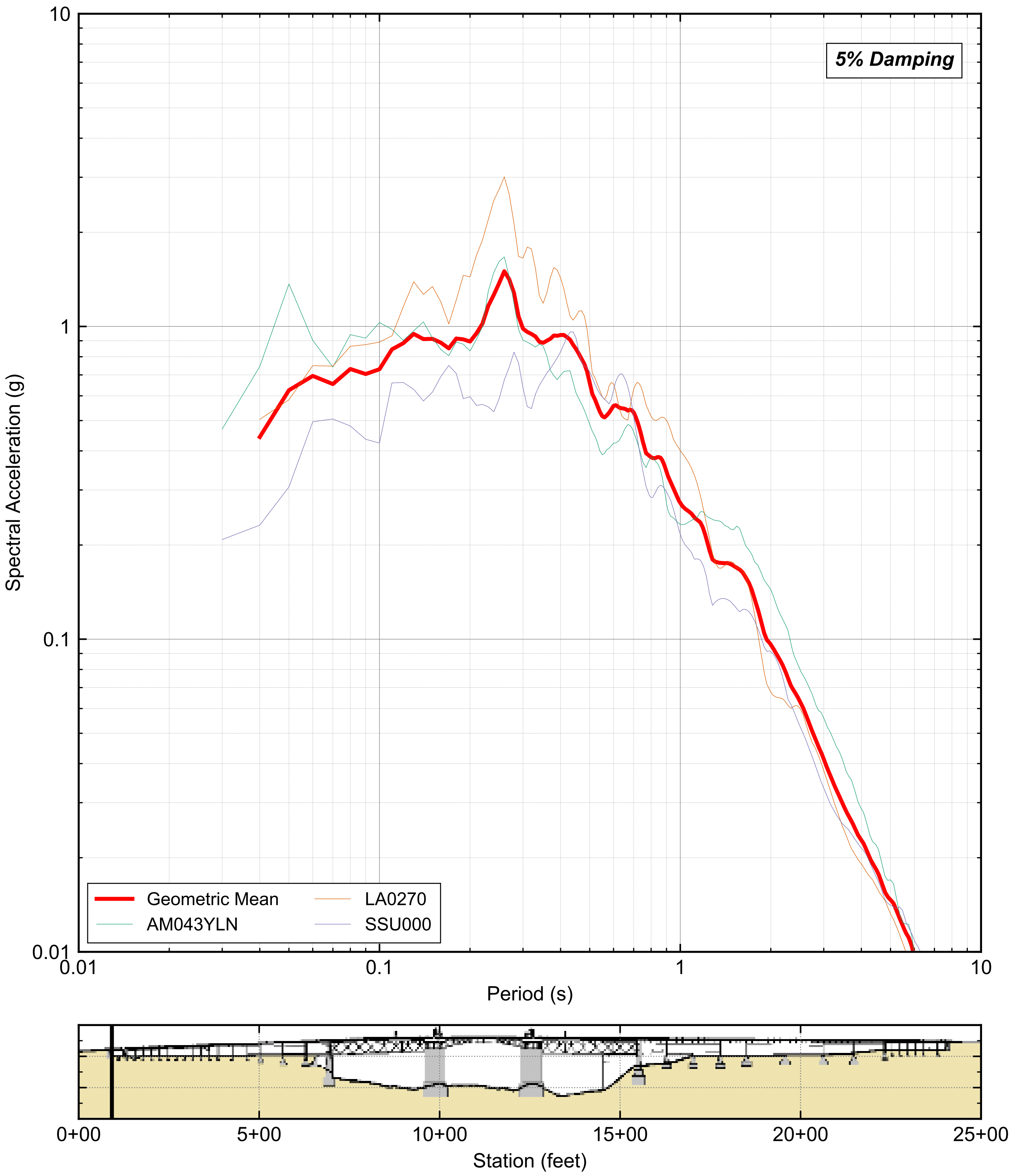
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 1

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FIG. E-39

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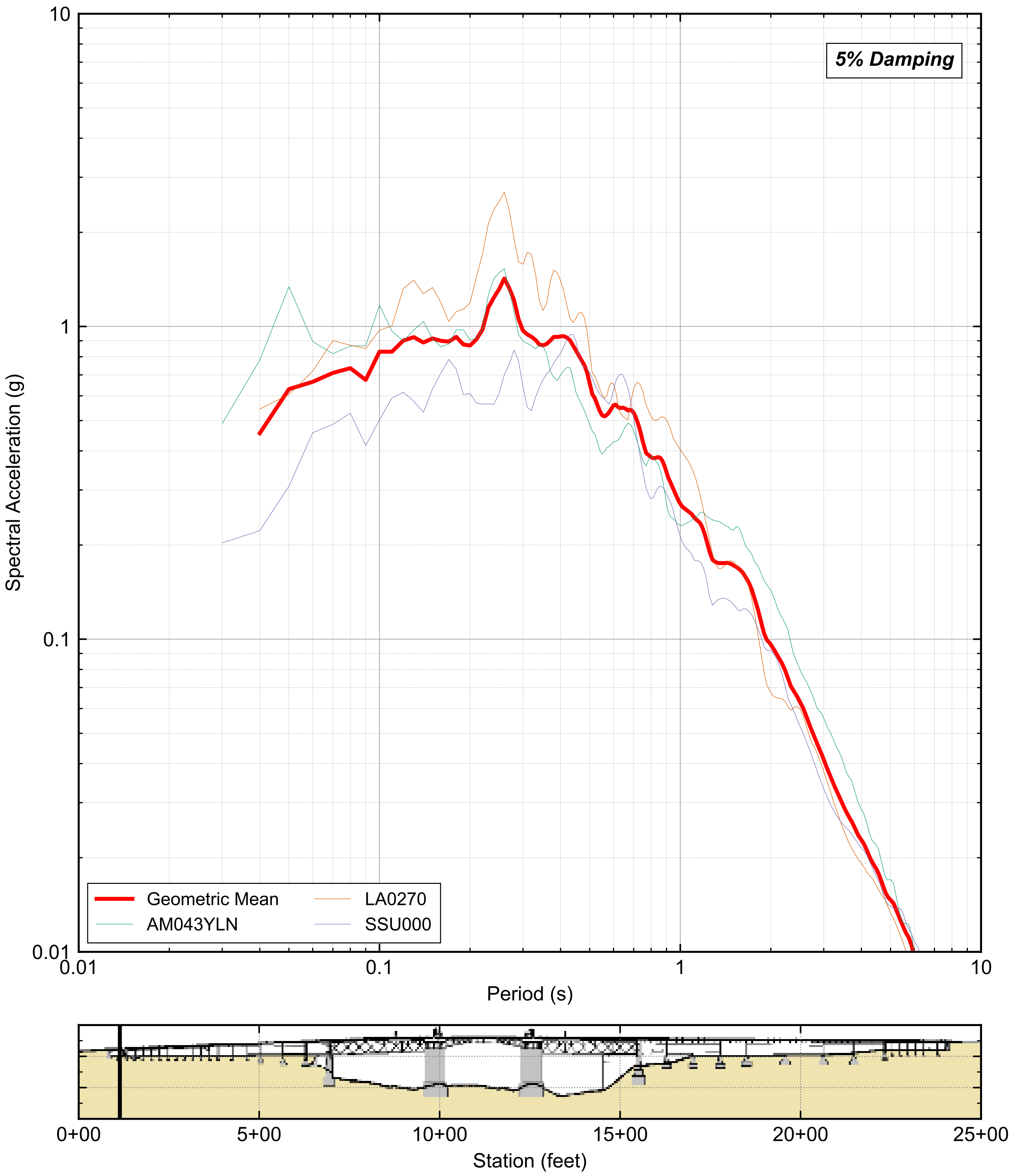
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CRUSTAL) EXISTING BENT 2

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FIG. E-40

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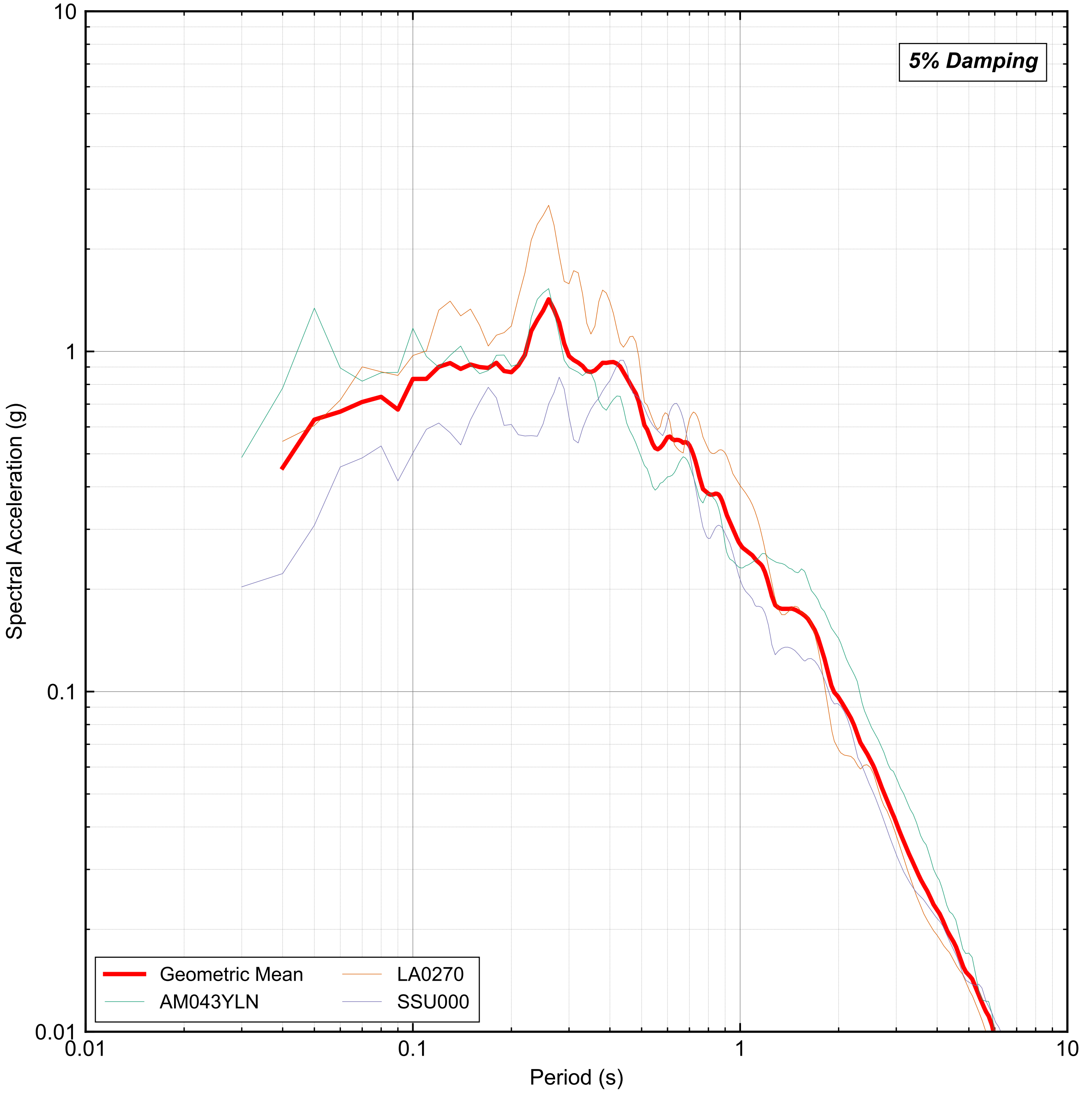
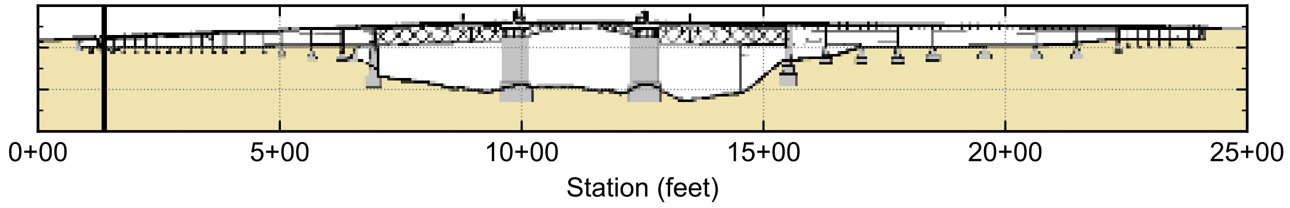
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 3

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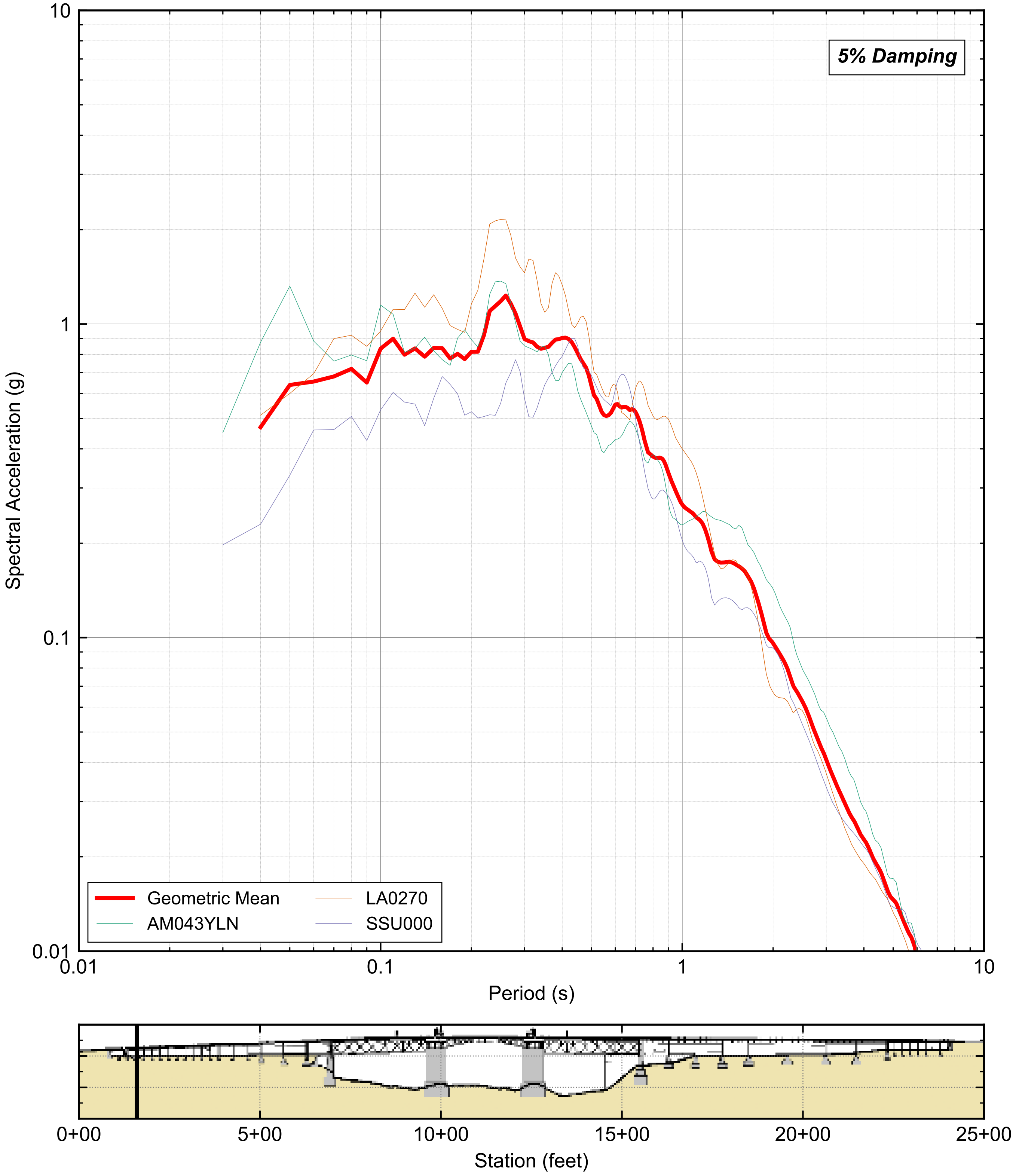
FIG. E-41

C:\Users\jnb\FLAC\Burnside\v6_test

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 4

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FIG. E-42

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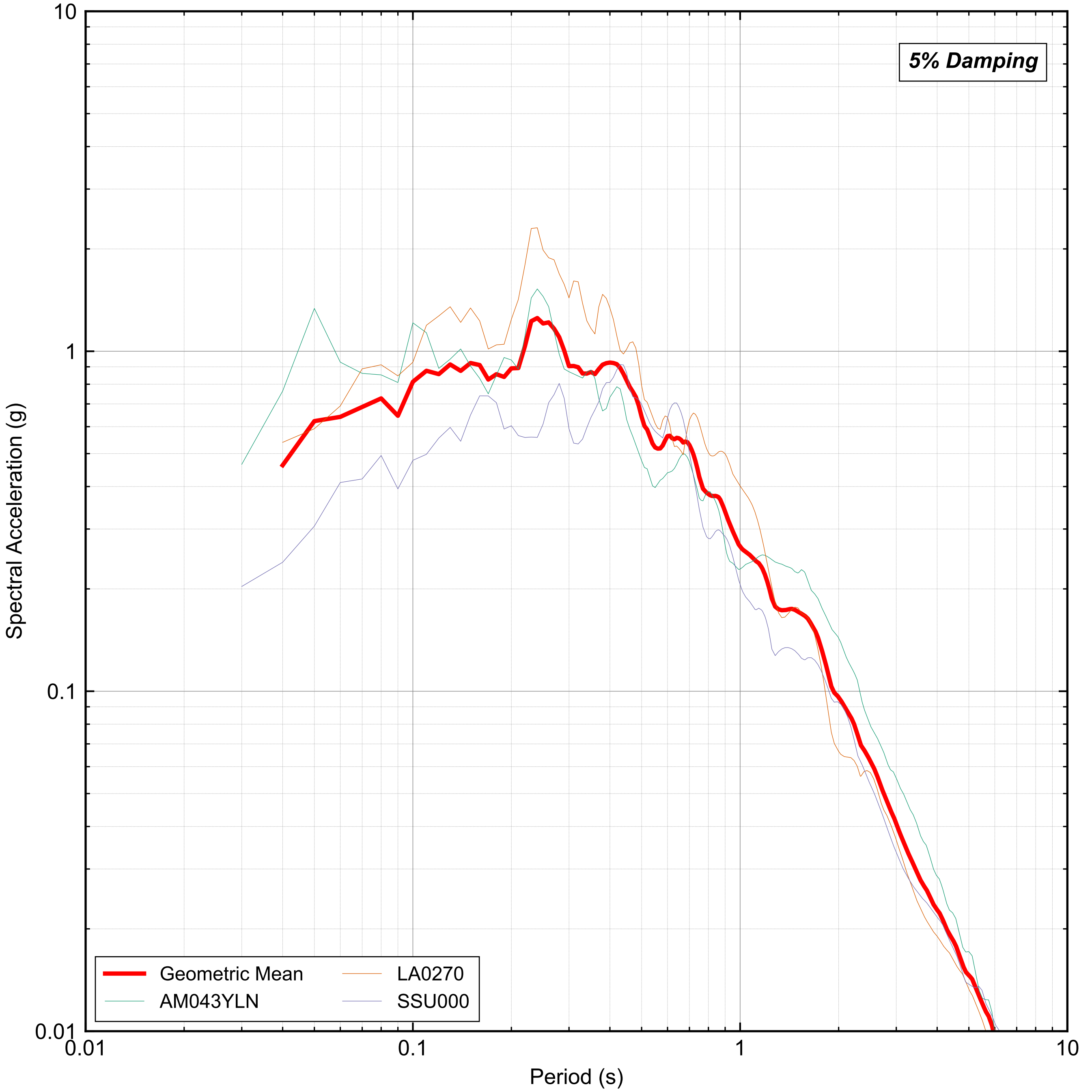

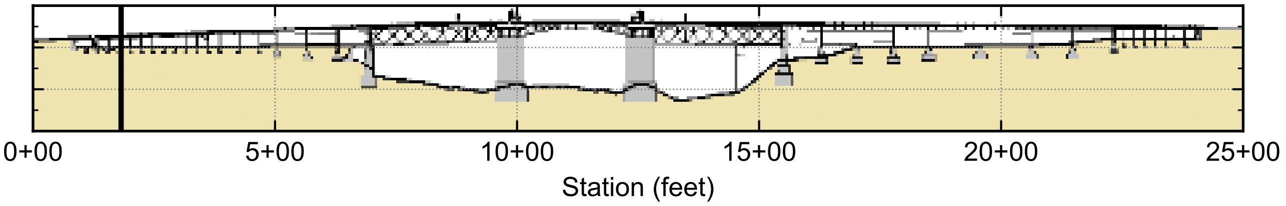

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 5

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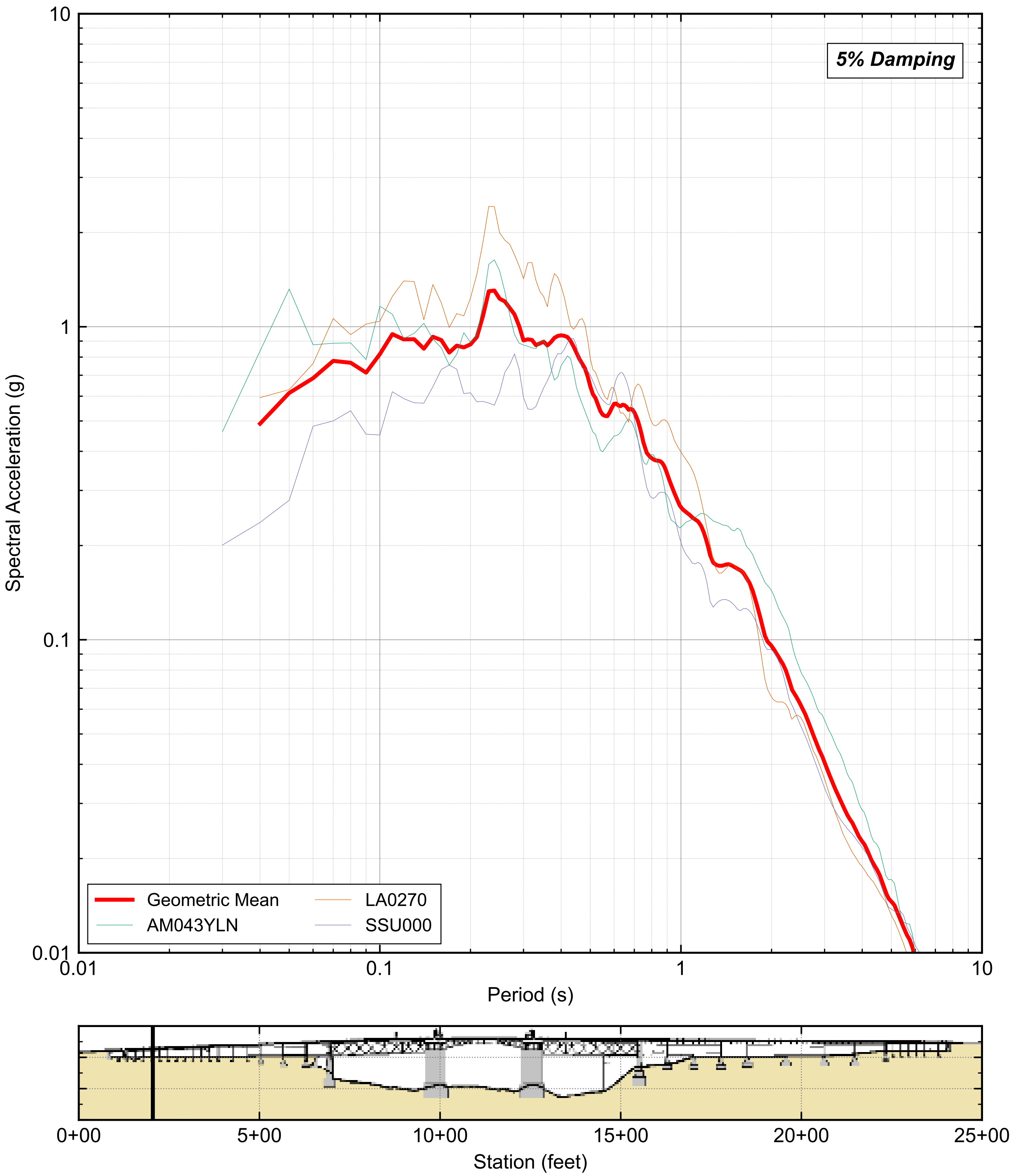
FIG. E-43

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 6

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FIG. E-44

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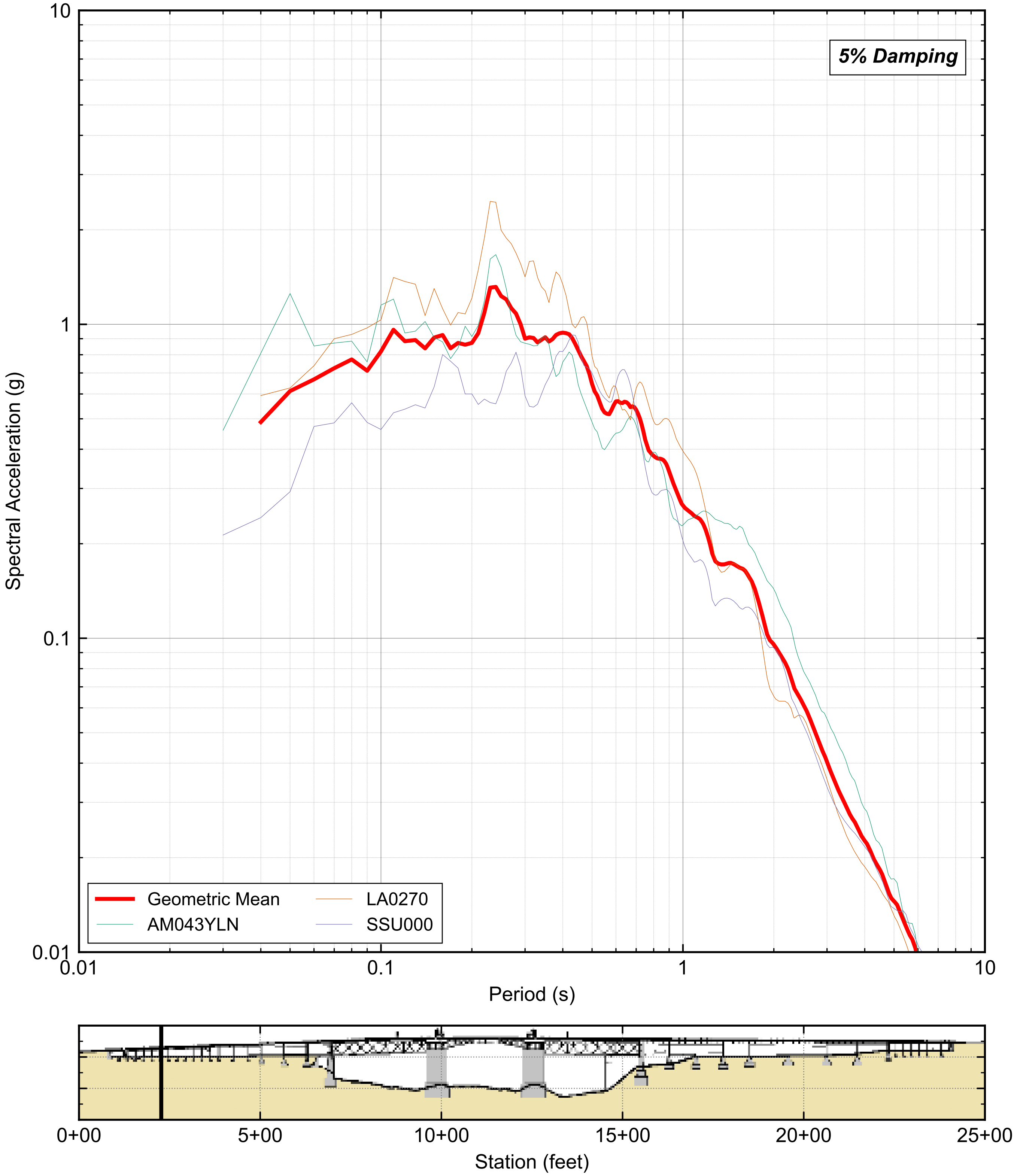
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 7

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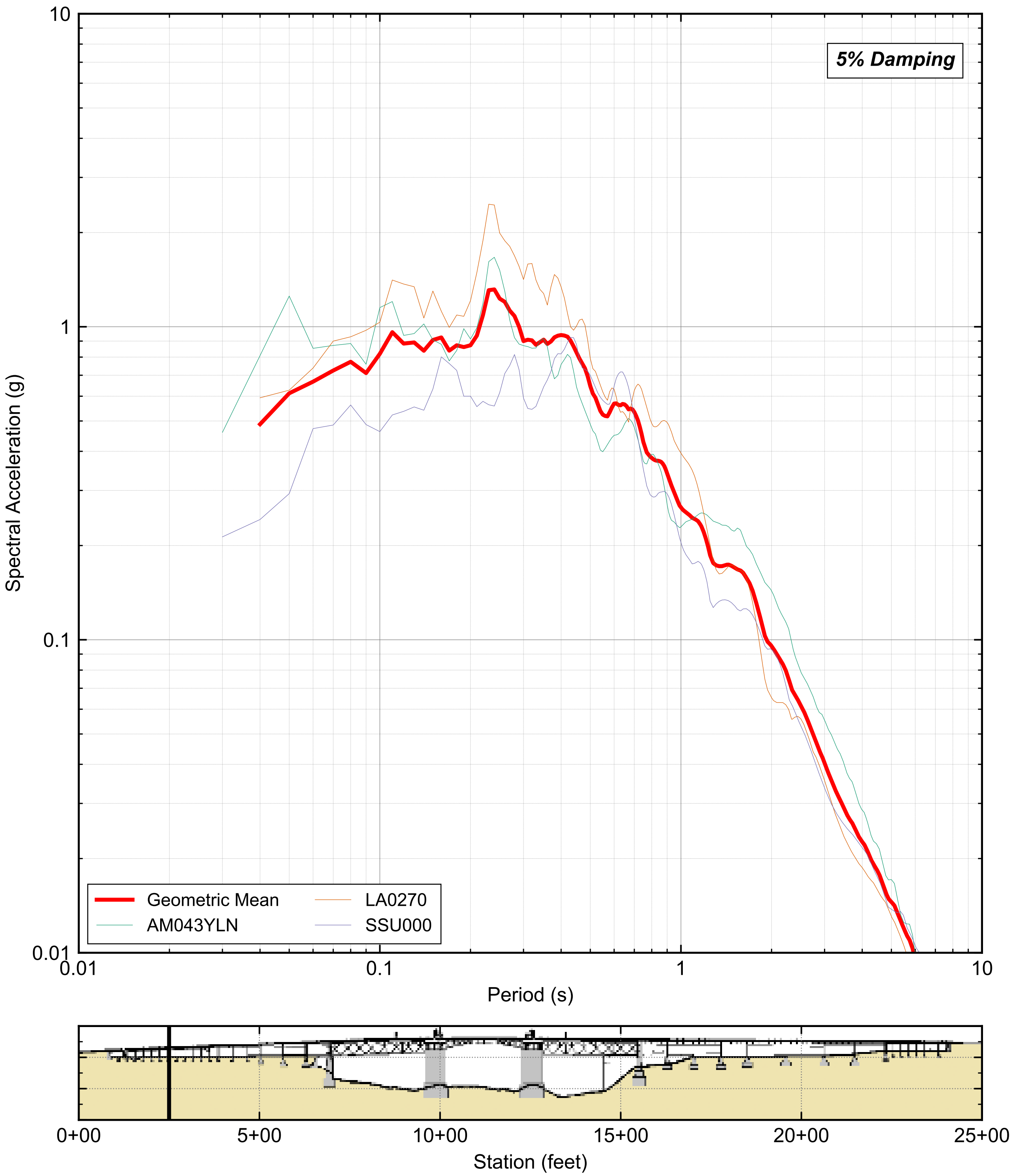
FIG. E-45

C:\Users\jnb\FLAC\Burnside\v6_test

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 8

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FIG. E-46

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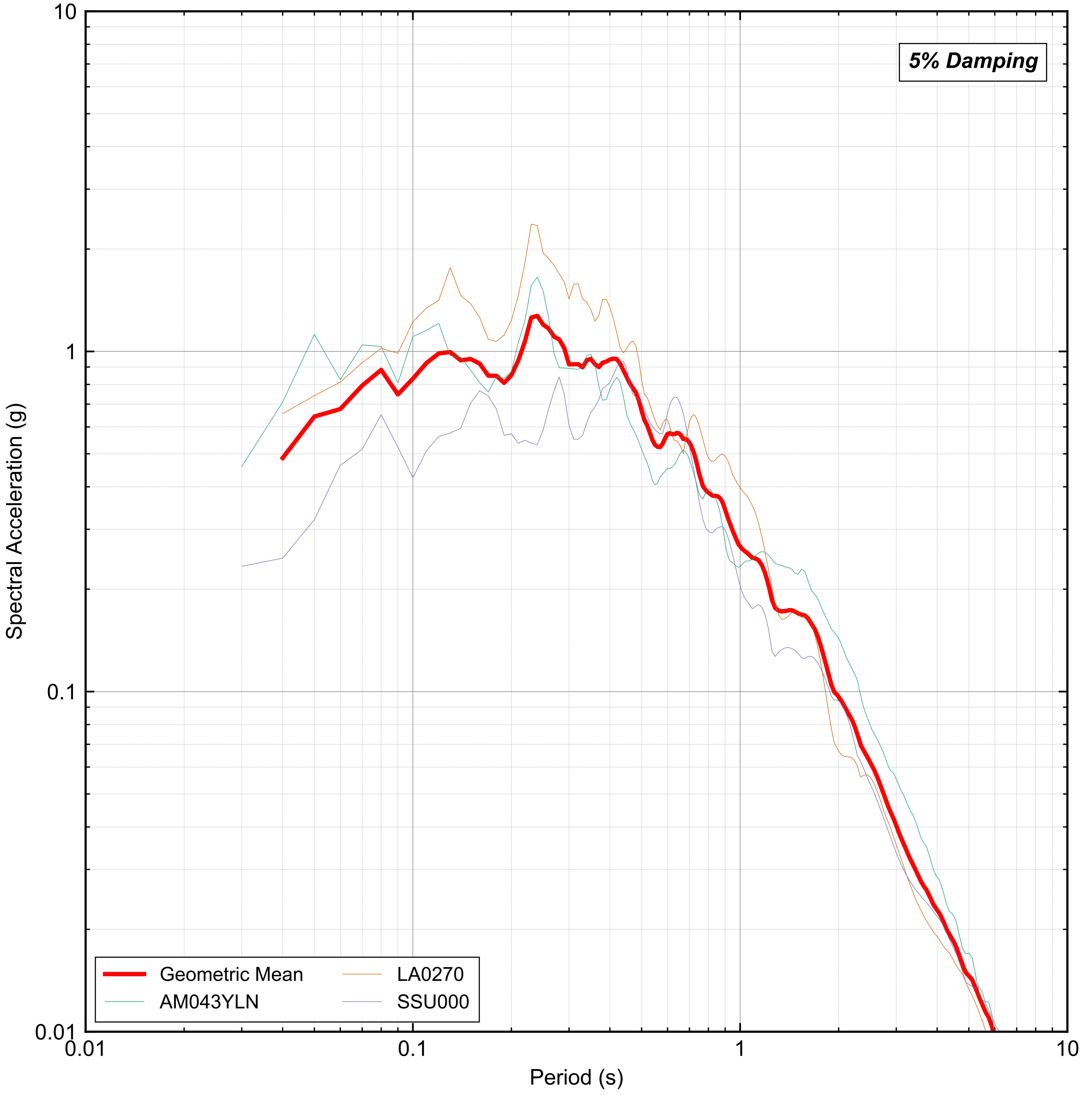

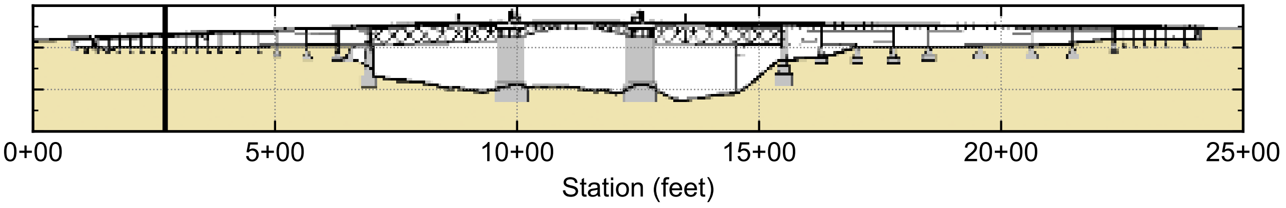

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 9

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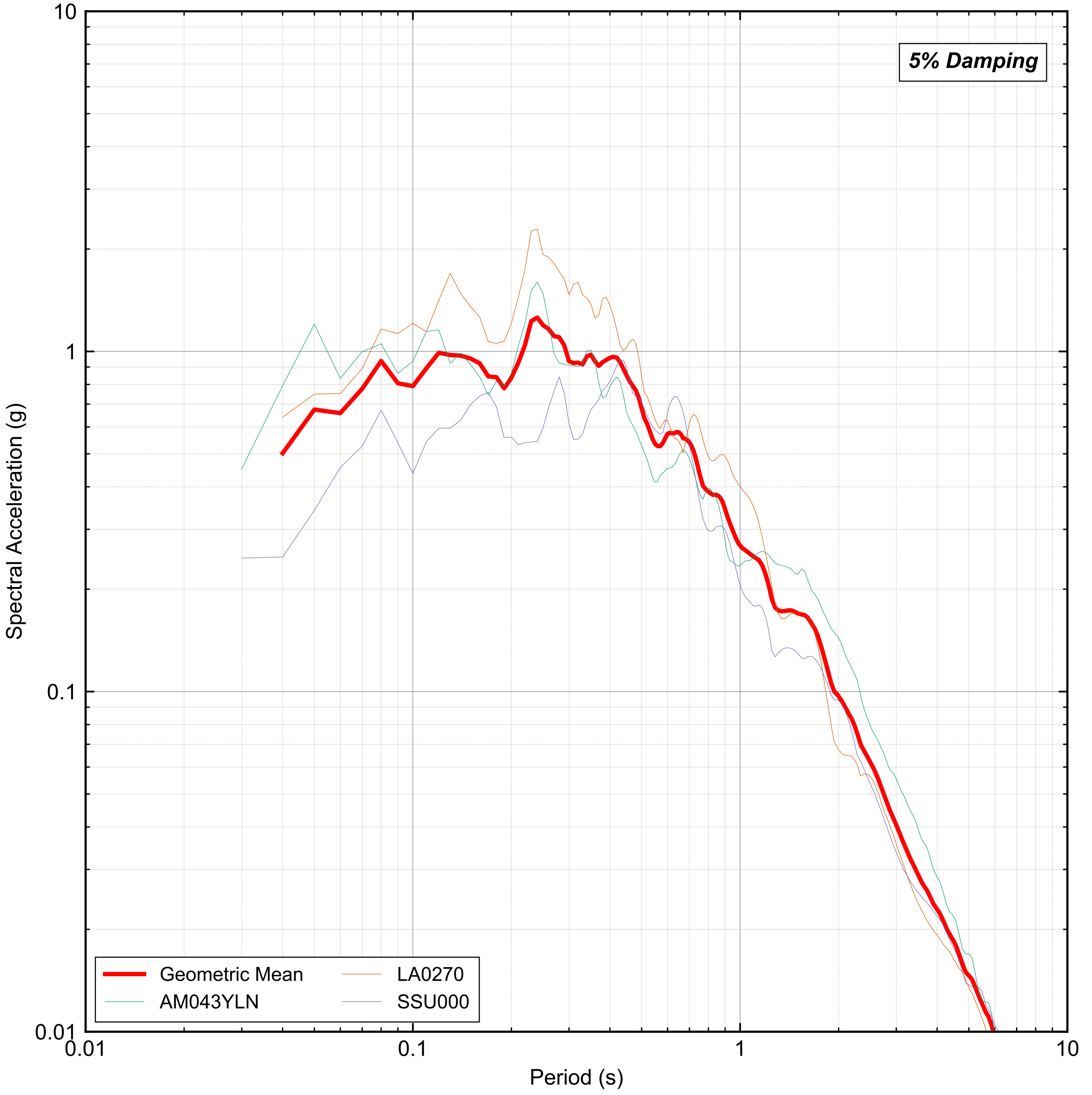
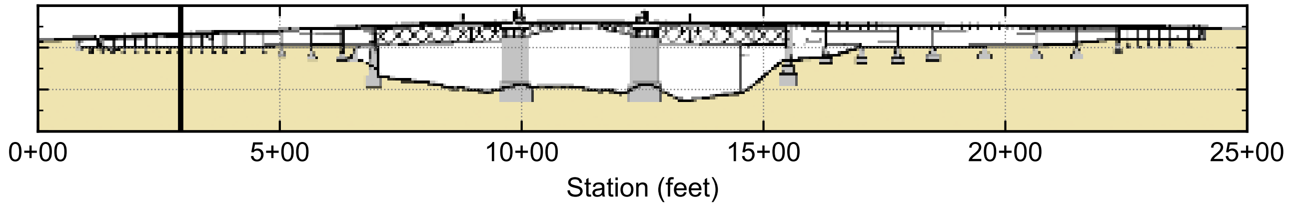
FIG. E-47





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CRUSTAL) EXISTING BENT 10

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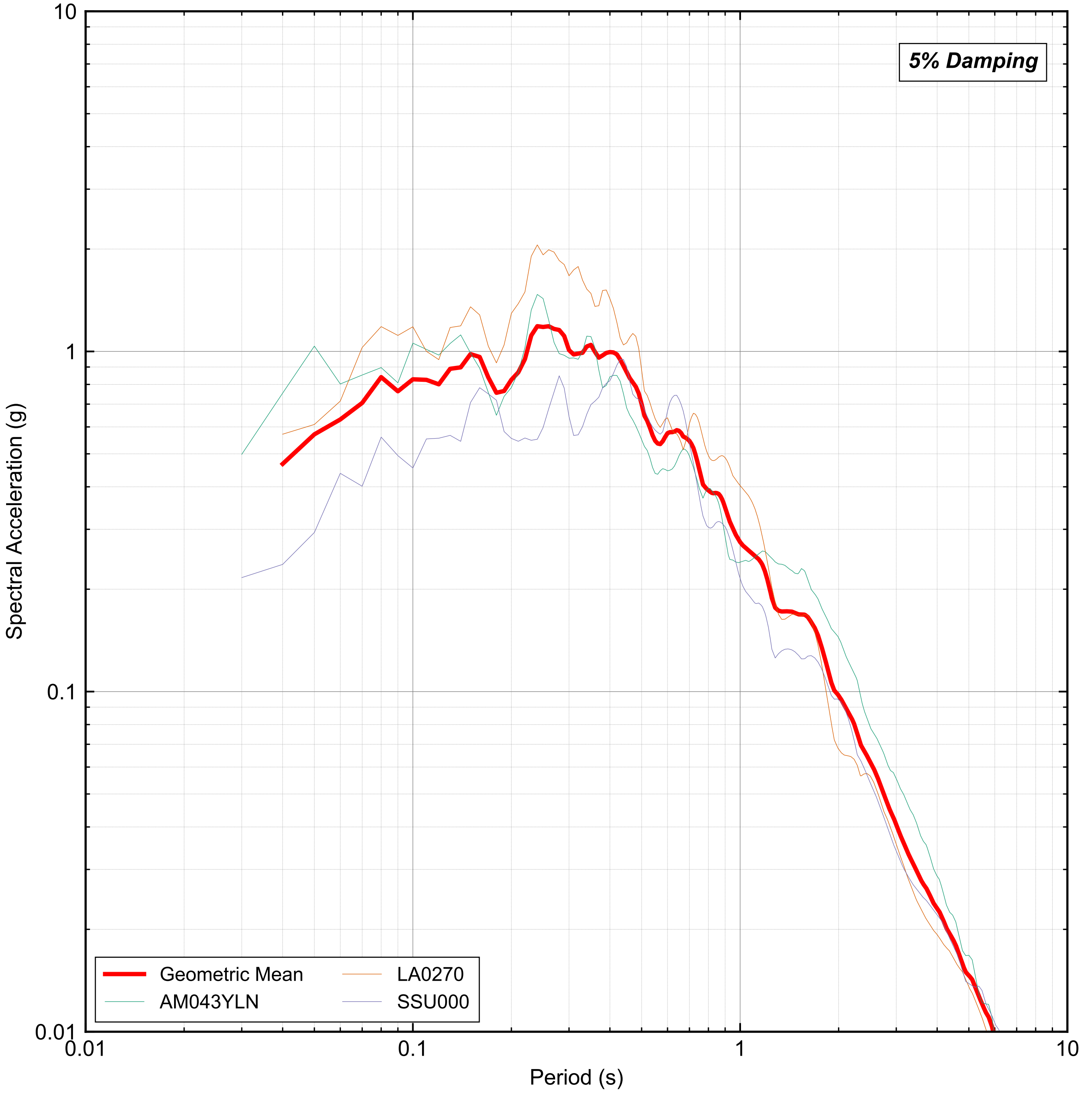
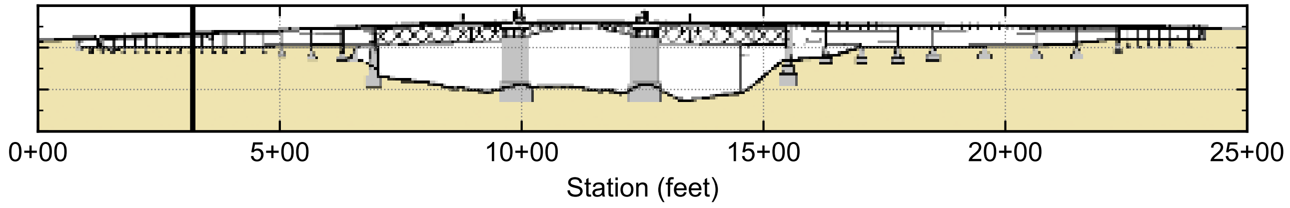
FIG. E-48





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 11

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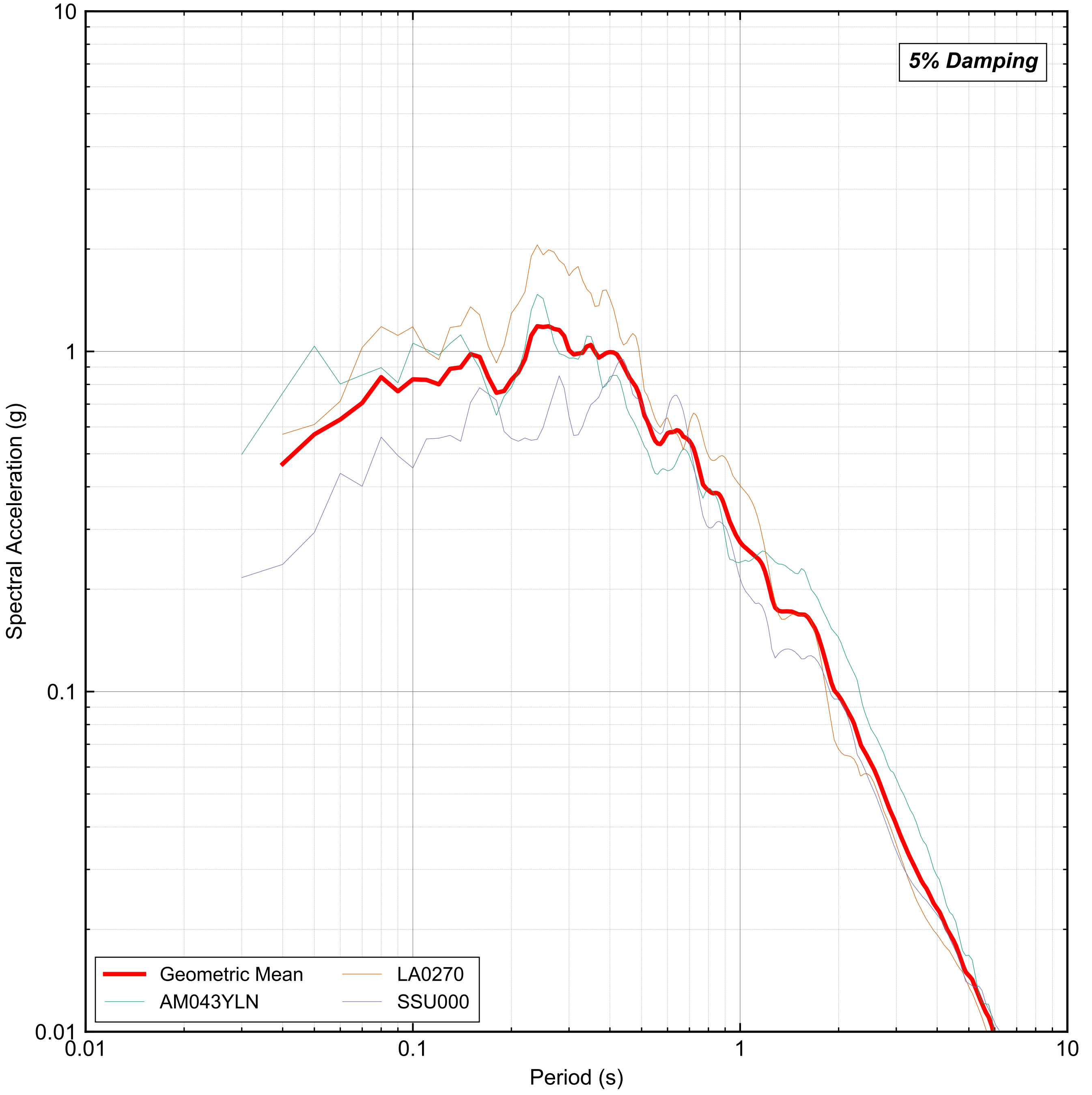
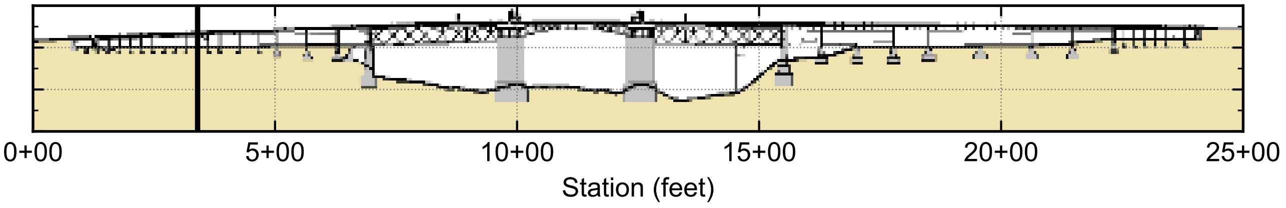
FIG. E-49





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CRUSTAL) EXISTING BENT 12

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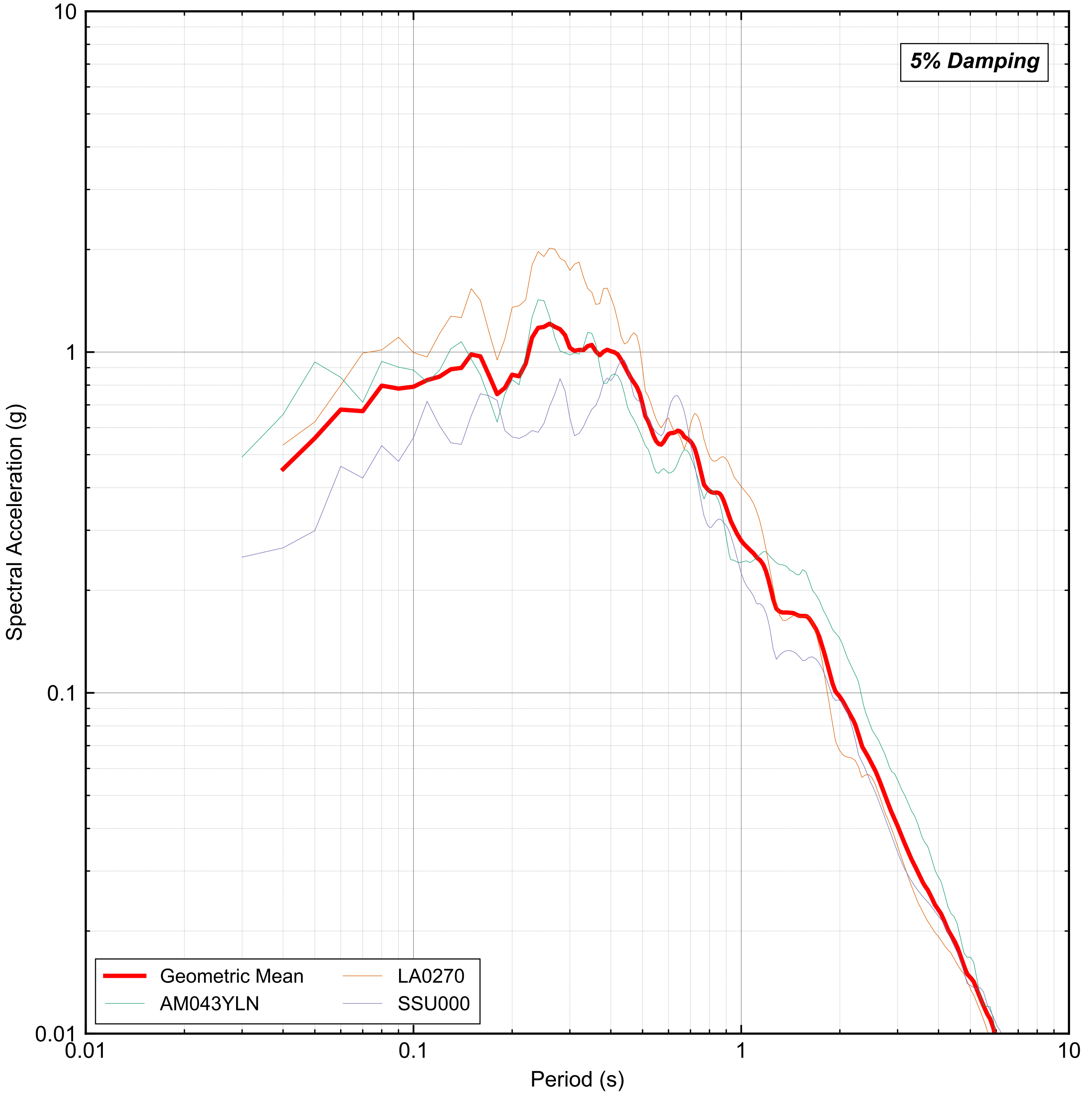
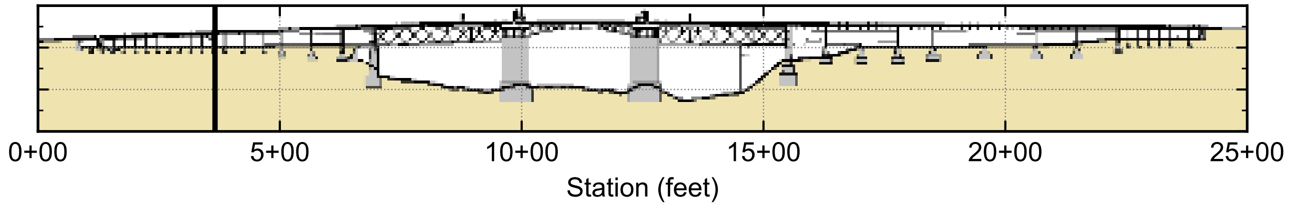
FIG. E-50

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 13

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FIG. E-51

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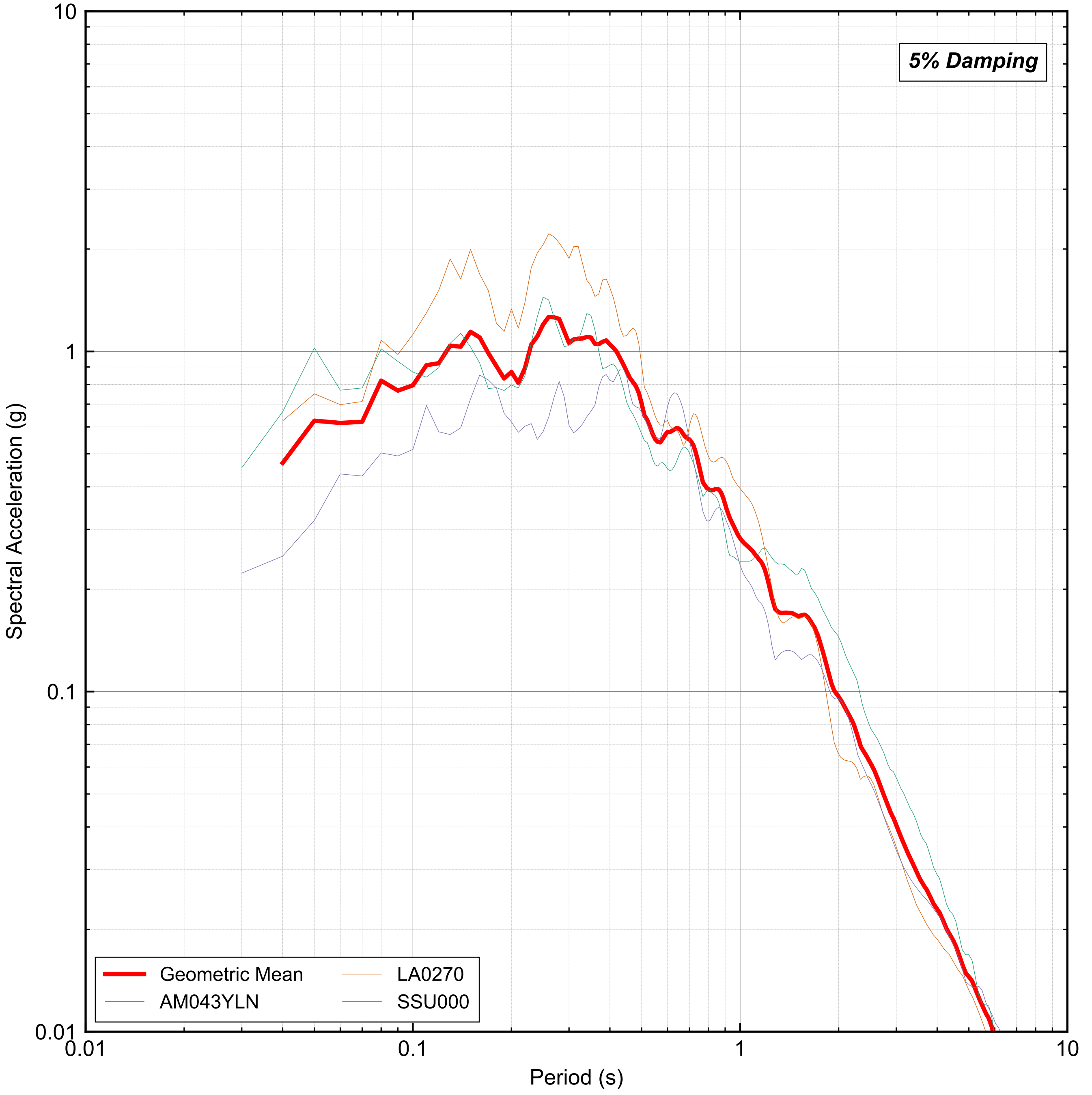
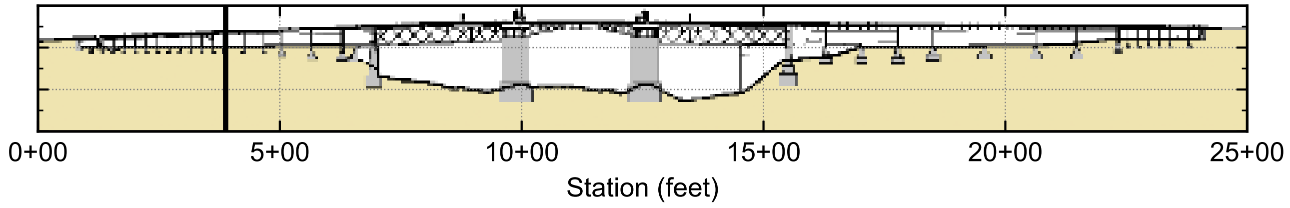
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 14

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FIG. E-52

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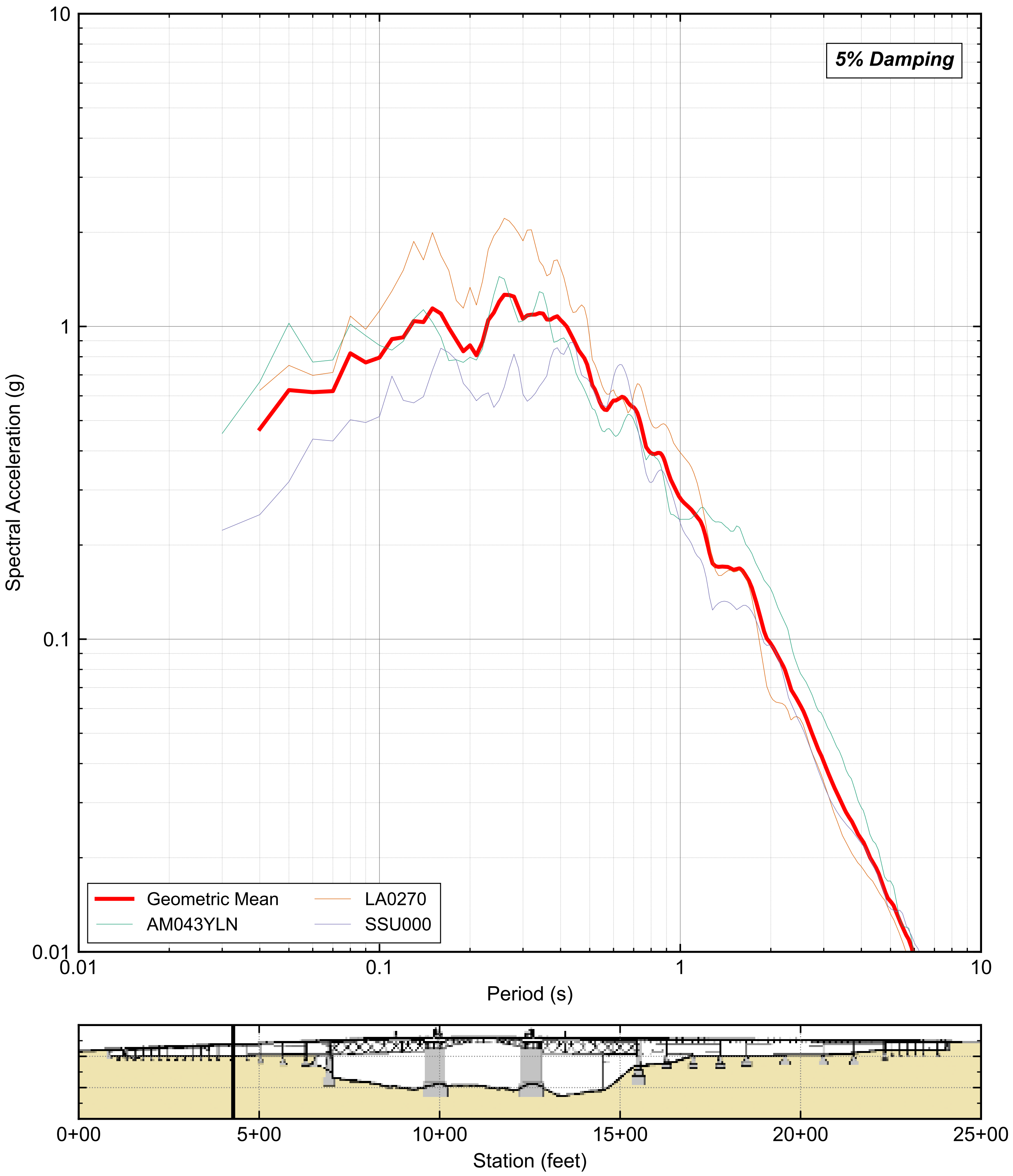

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 15

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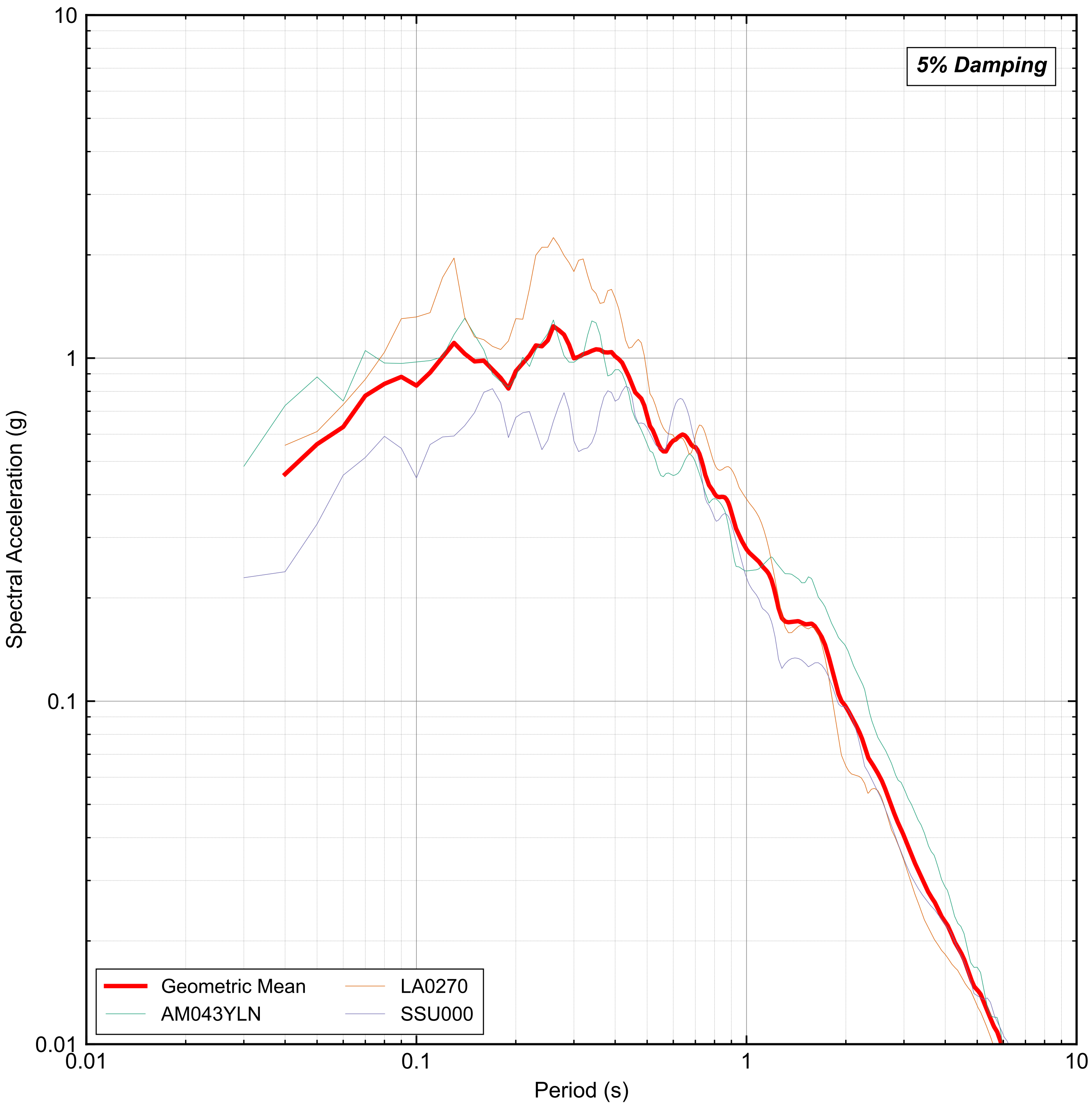
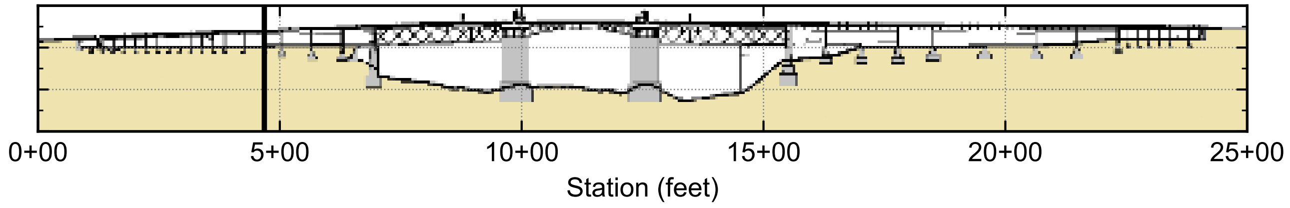
FIG. E-53

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 16

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FIG. E-54

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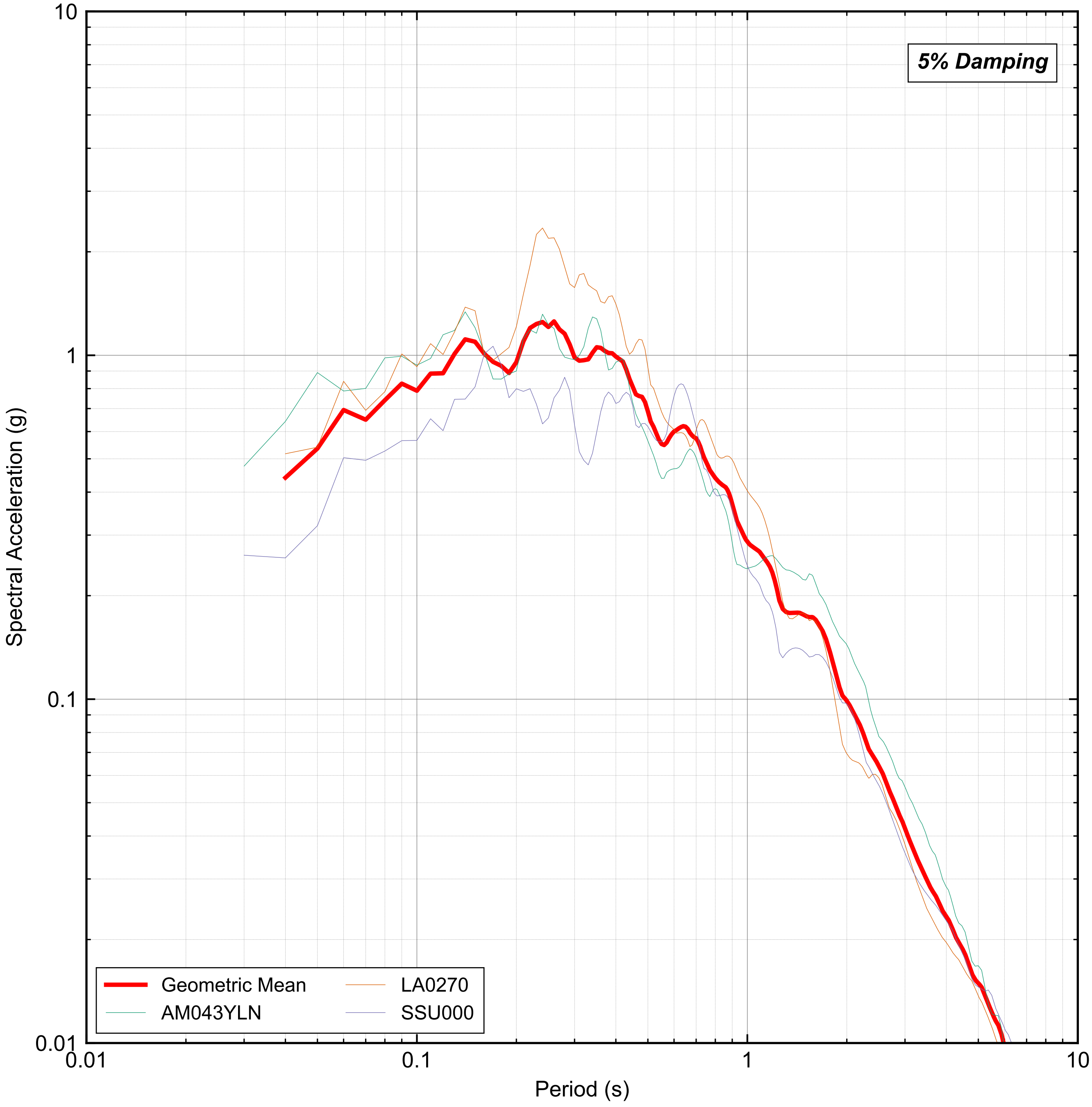
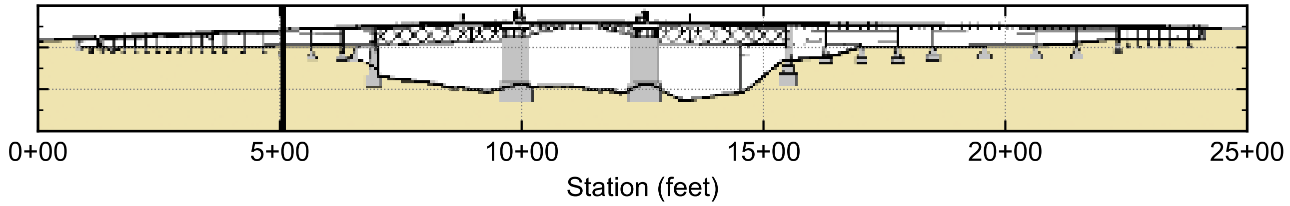
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 17

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FIG. E-55

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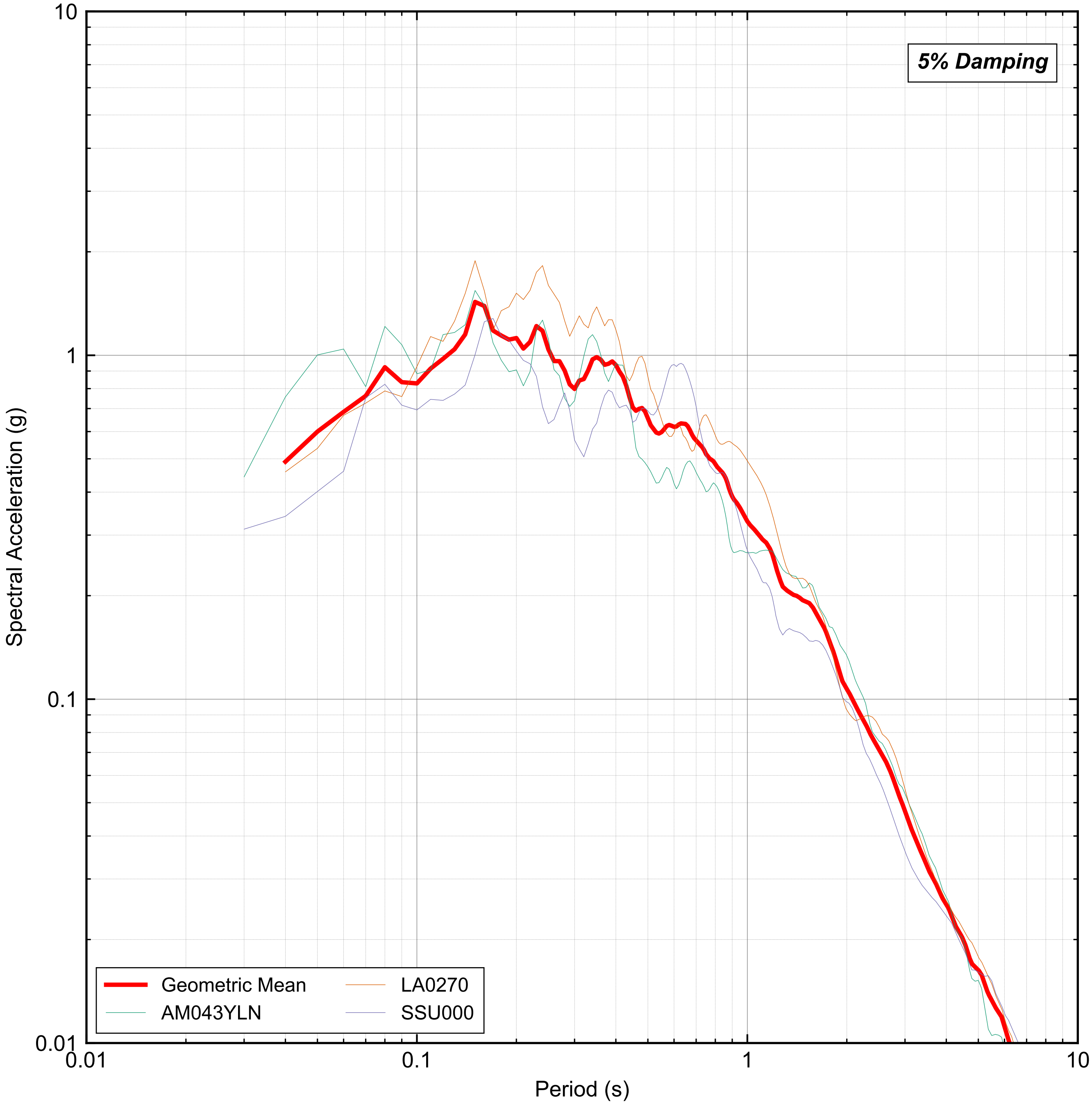
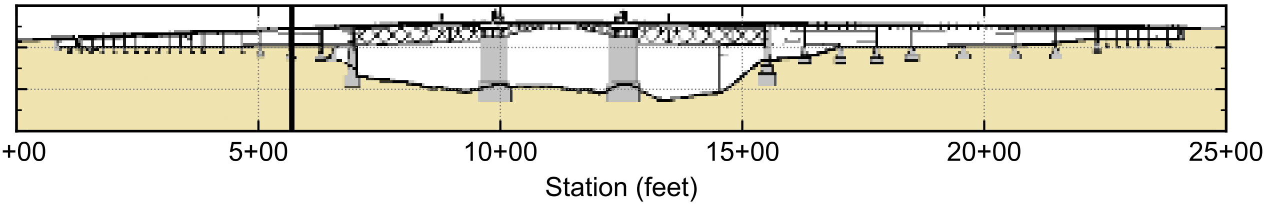
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 18

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FIG. E-56

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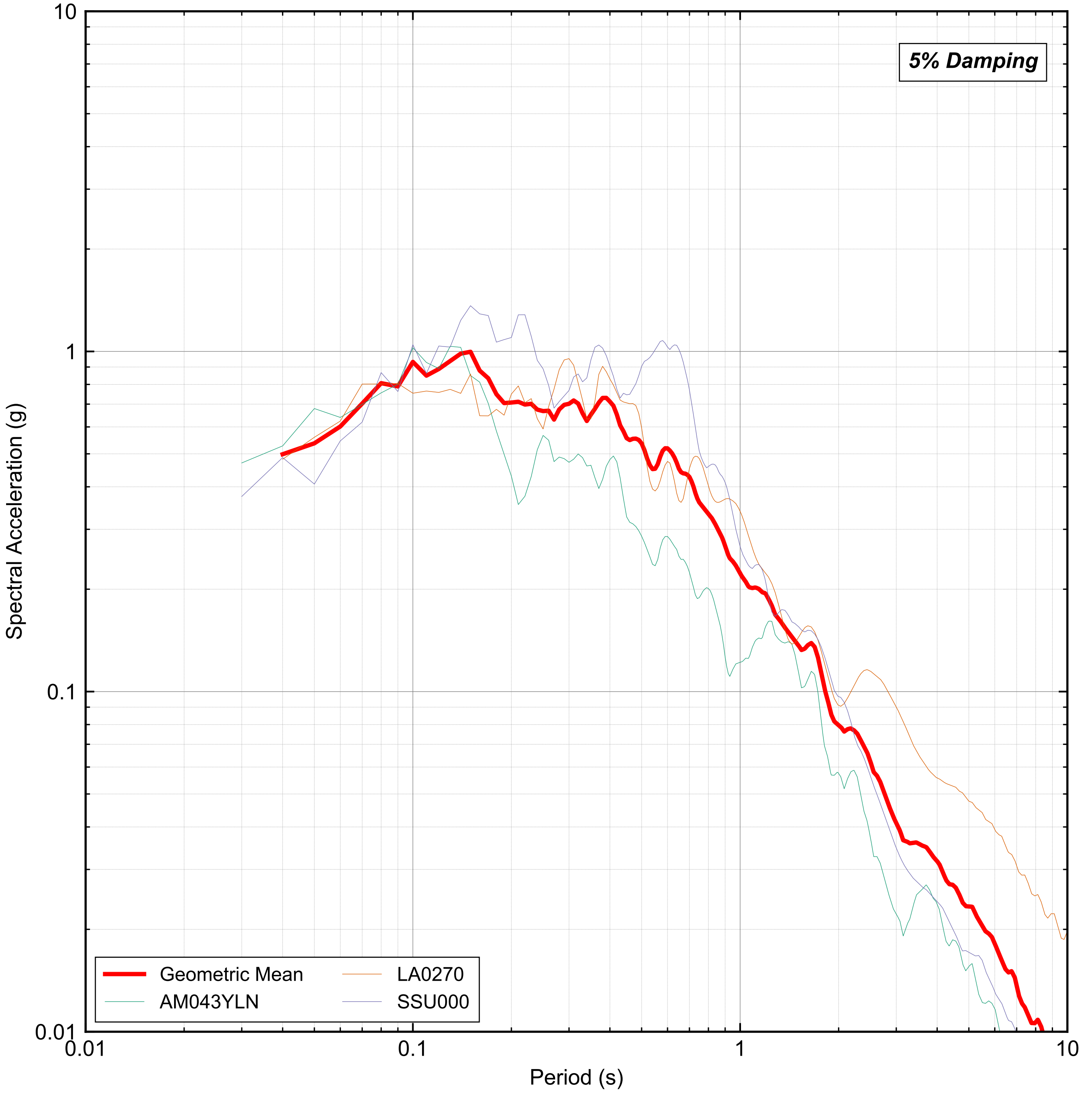

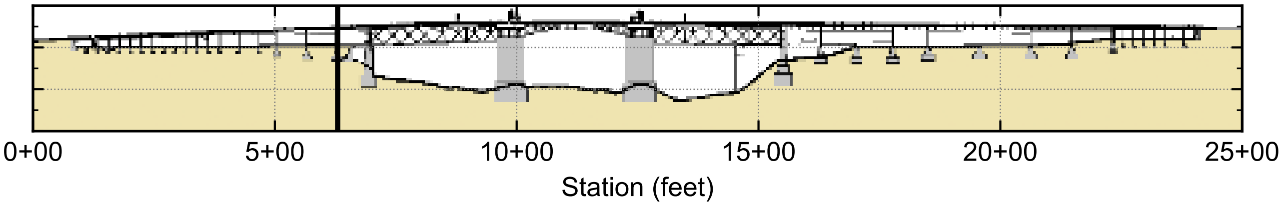

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 19

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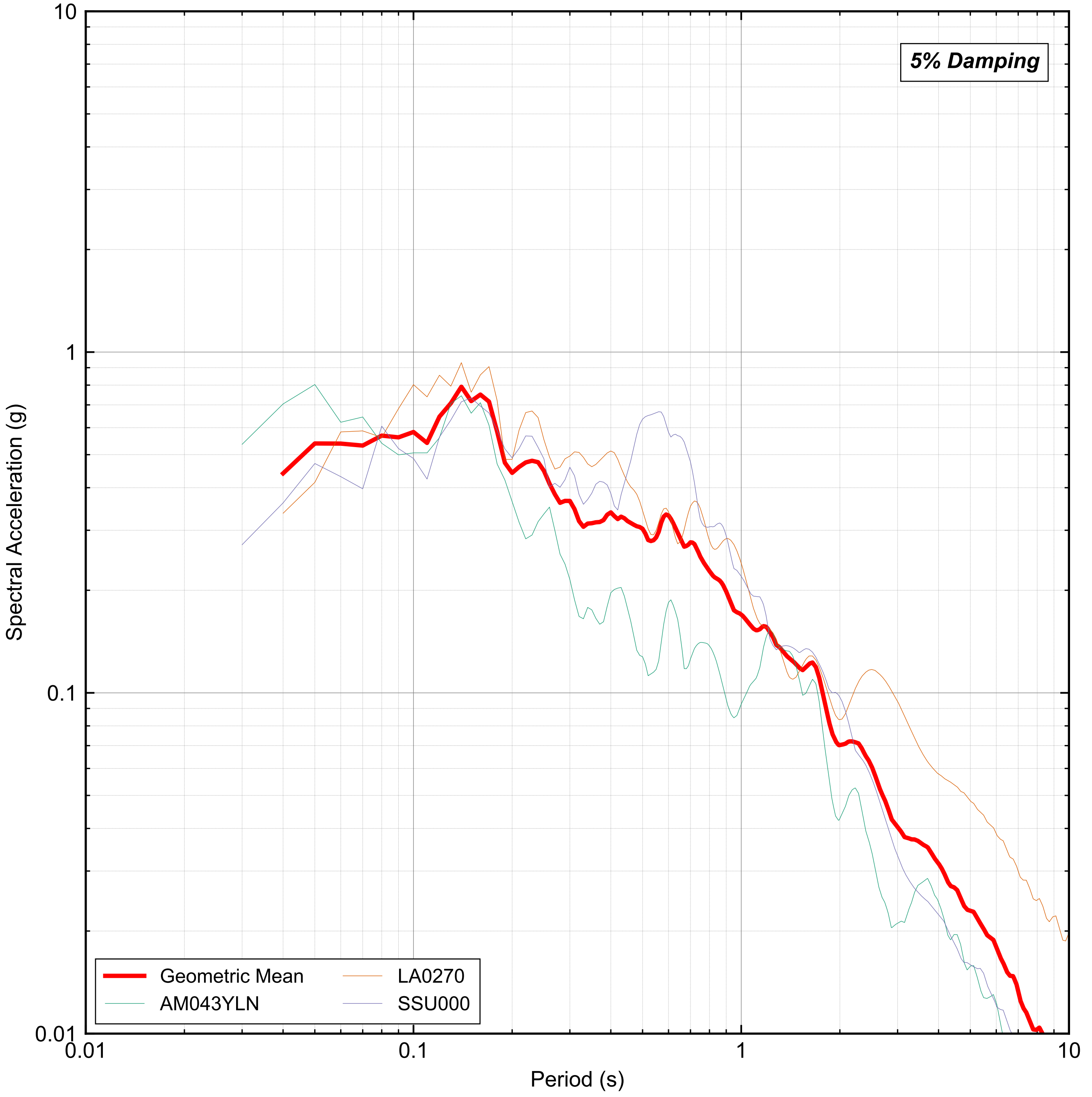
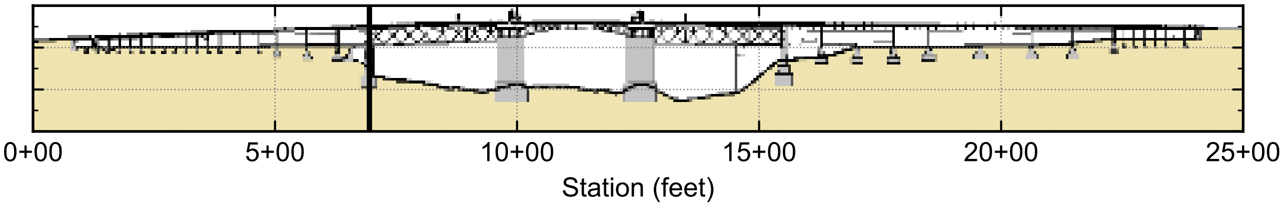
FIG. E-57





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CRUSTAL) EXISTING PIER 1

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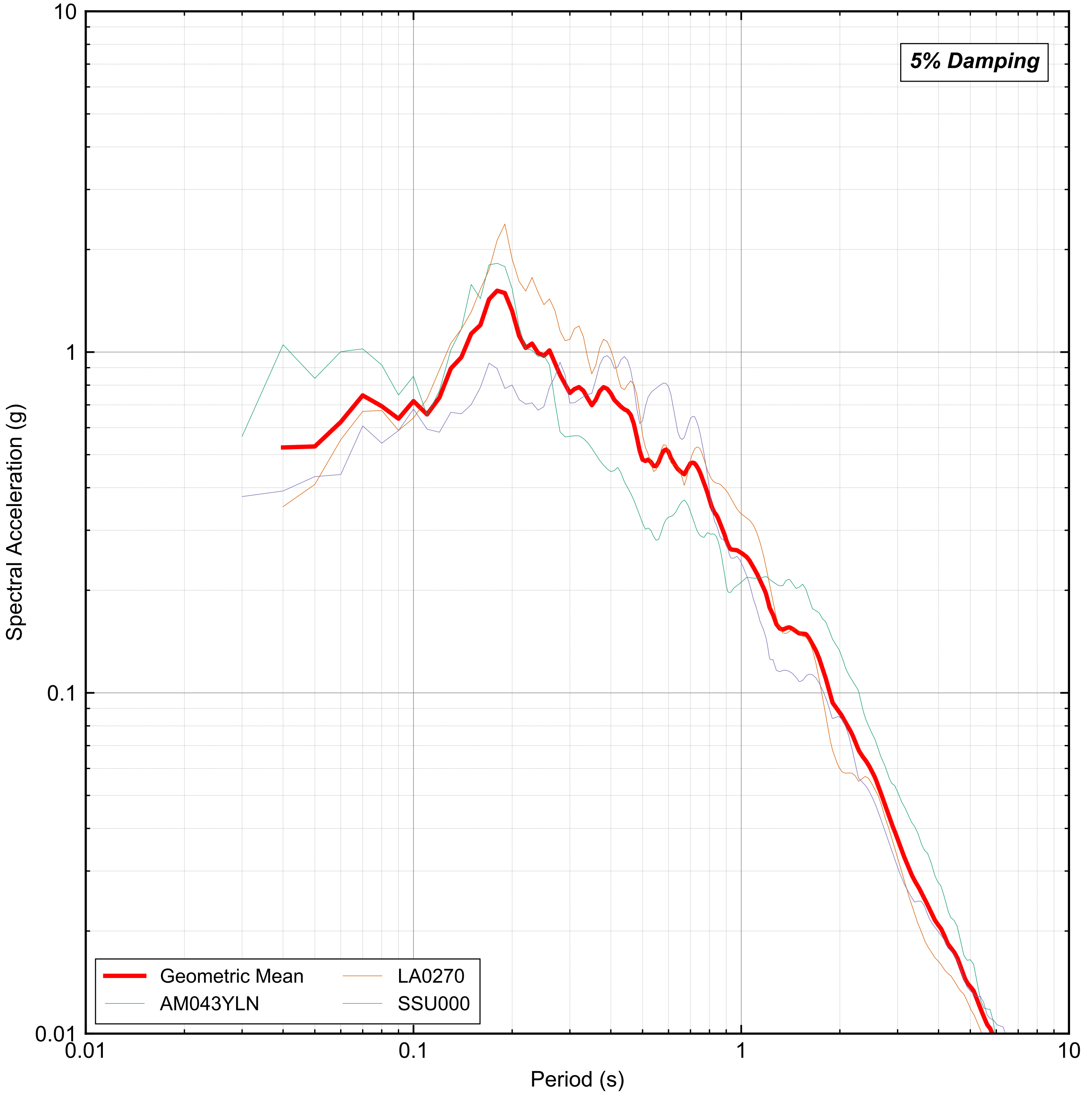
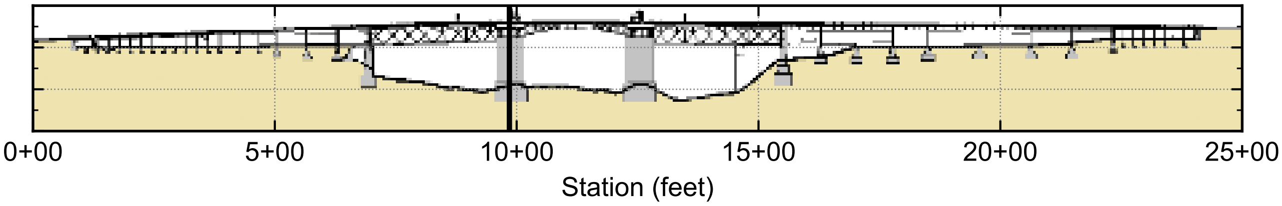
FIG. E-58





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING PIER 2

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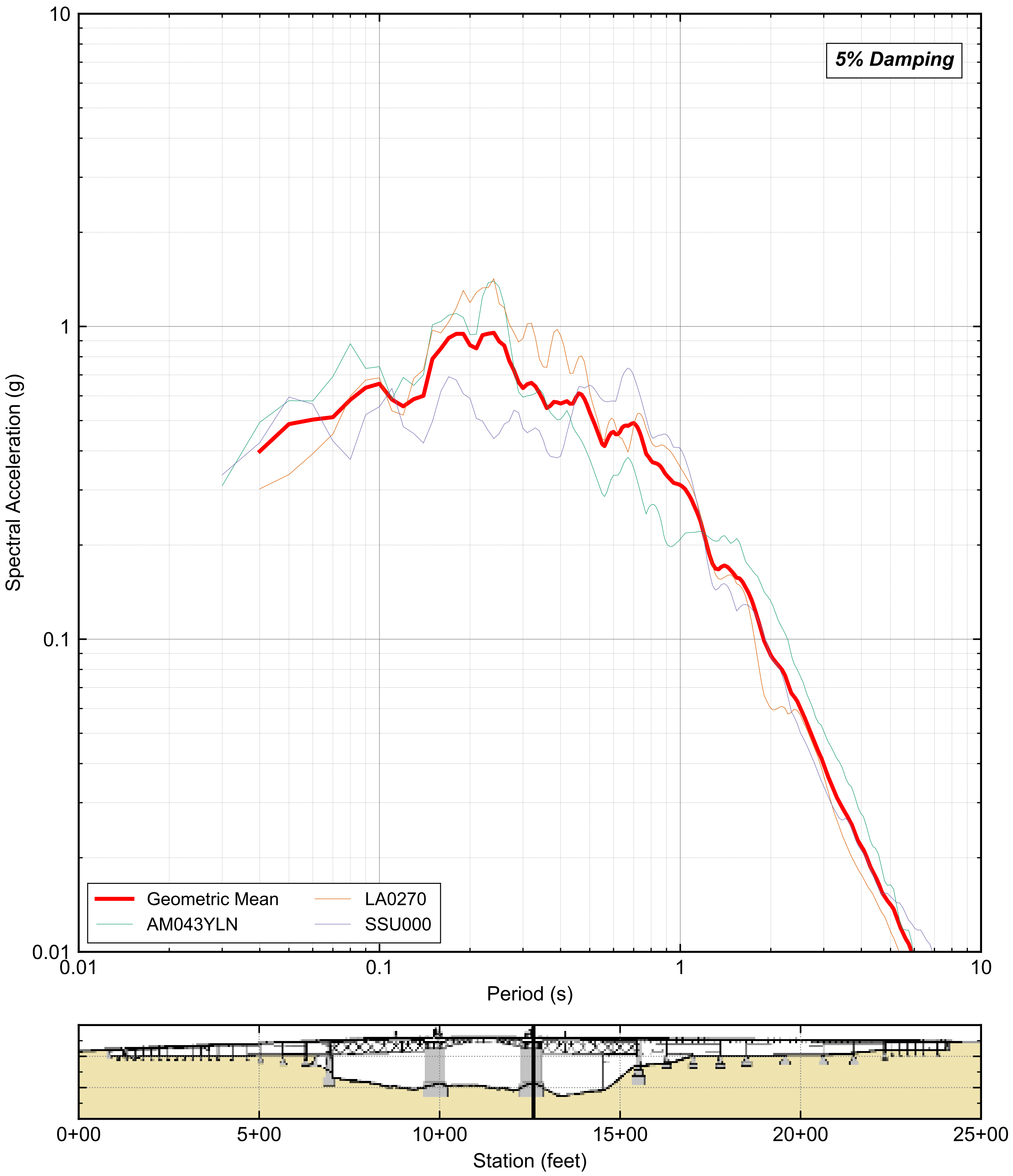
FIG. E-59

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING PIER 3

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FIG. E-60

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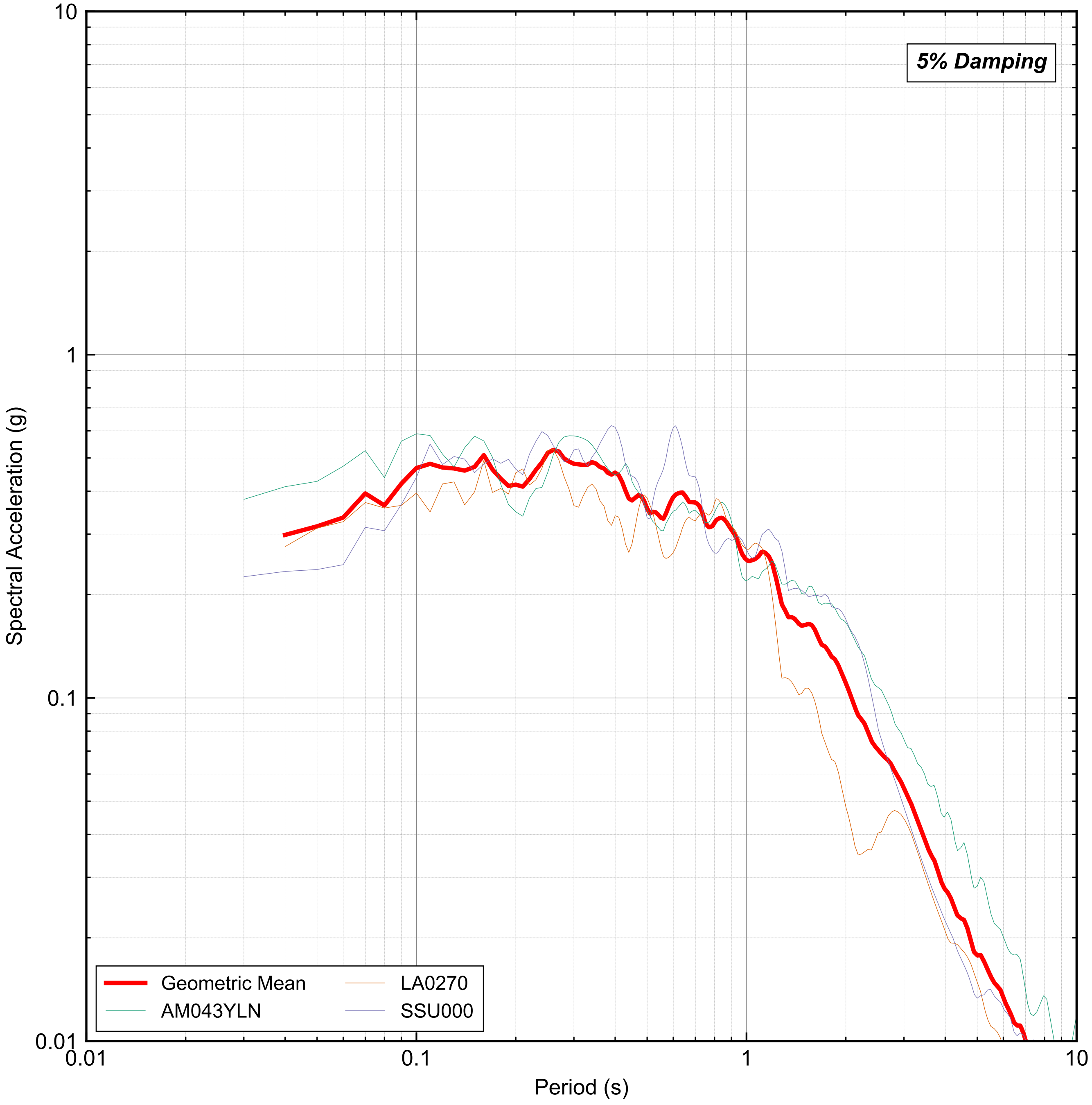

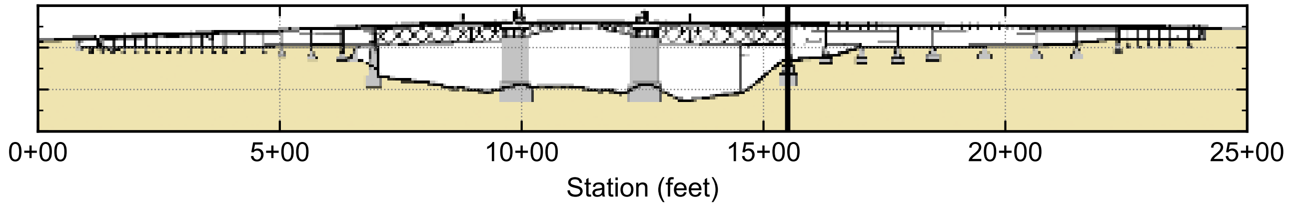

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING PIER 4

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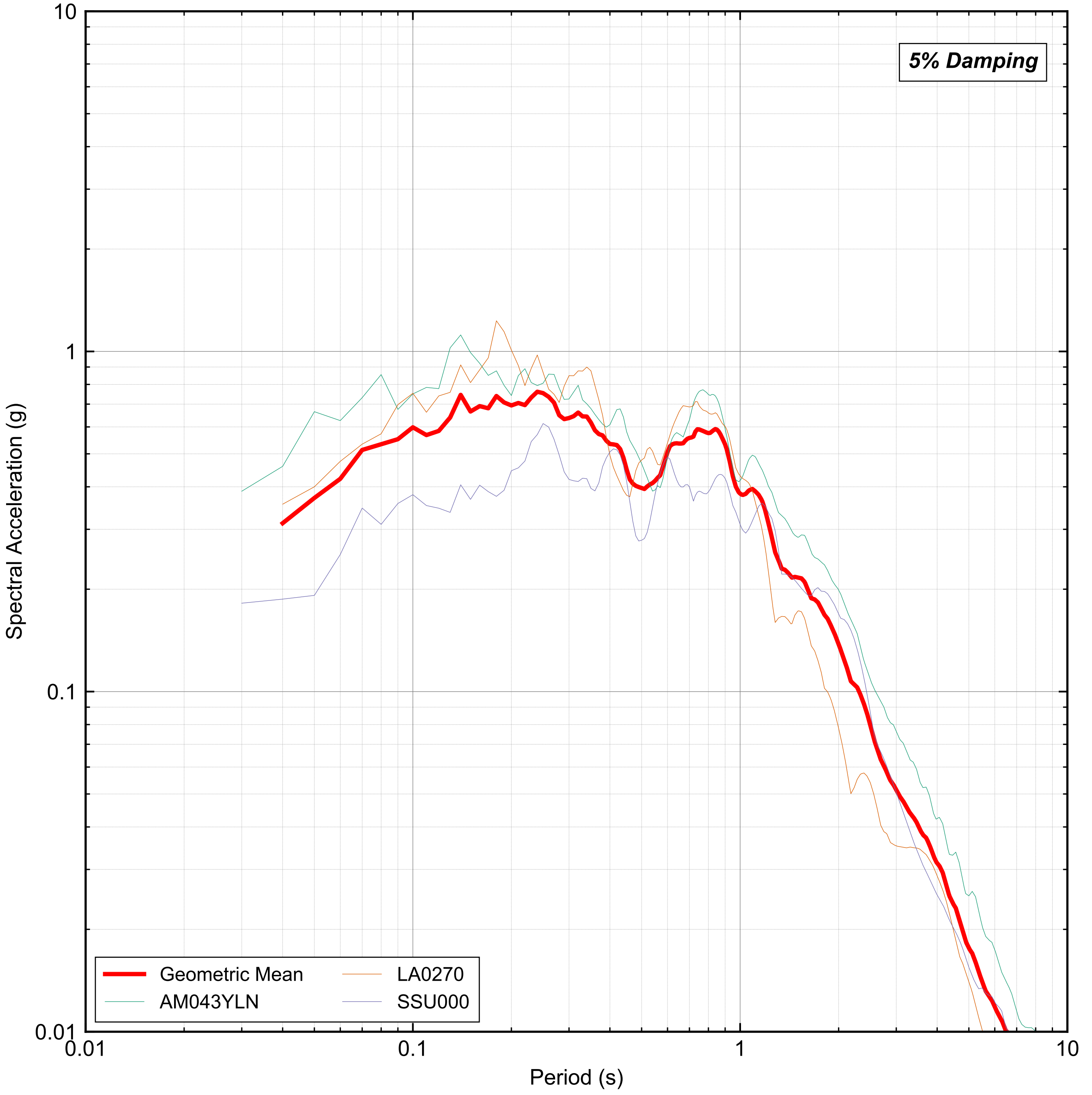
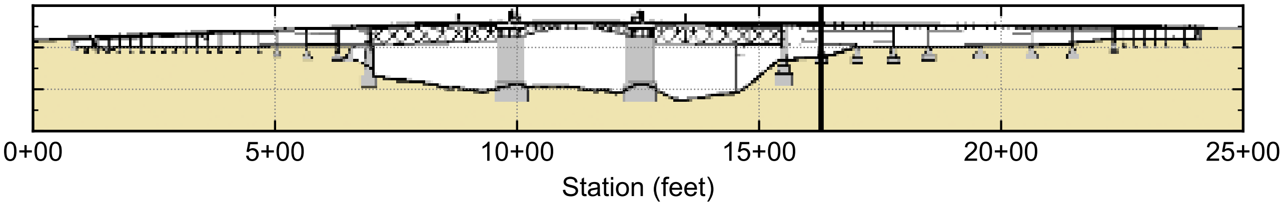
FIG. E-61

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 21

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FIG. E-62

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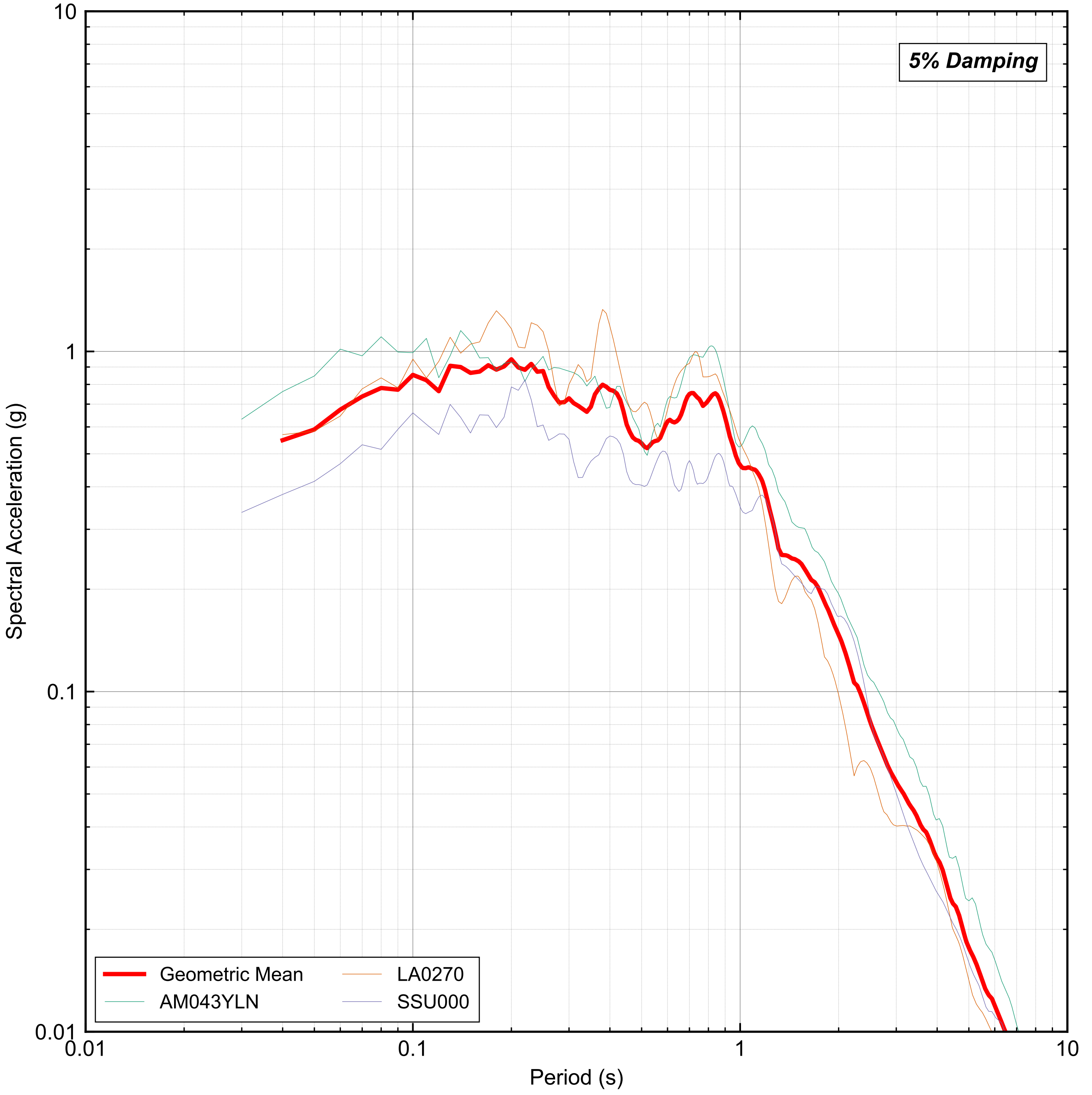

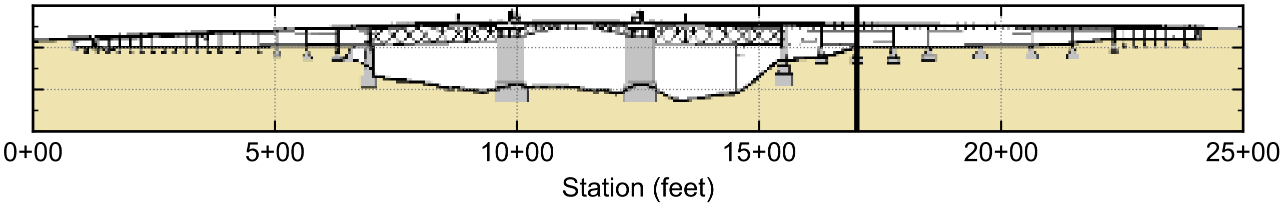

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
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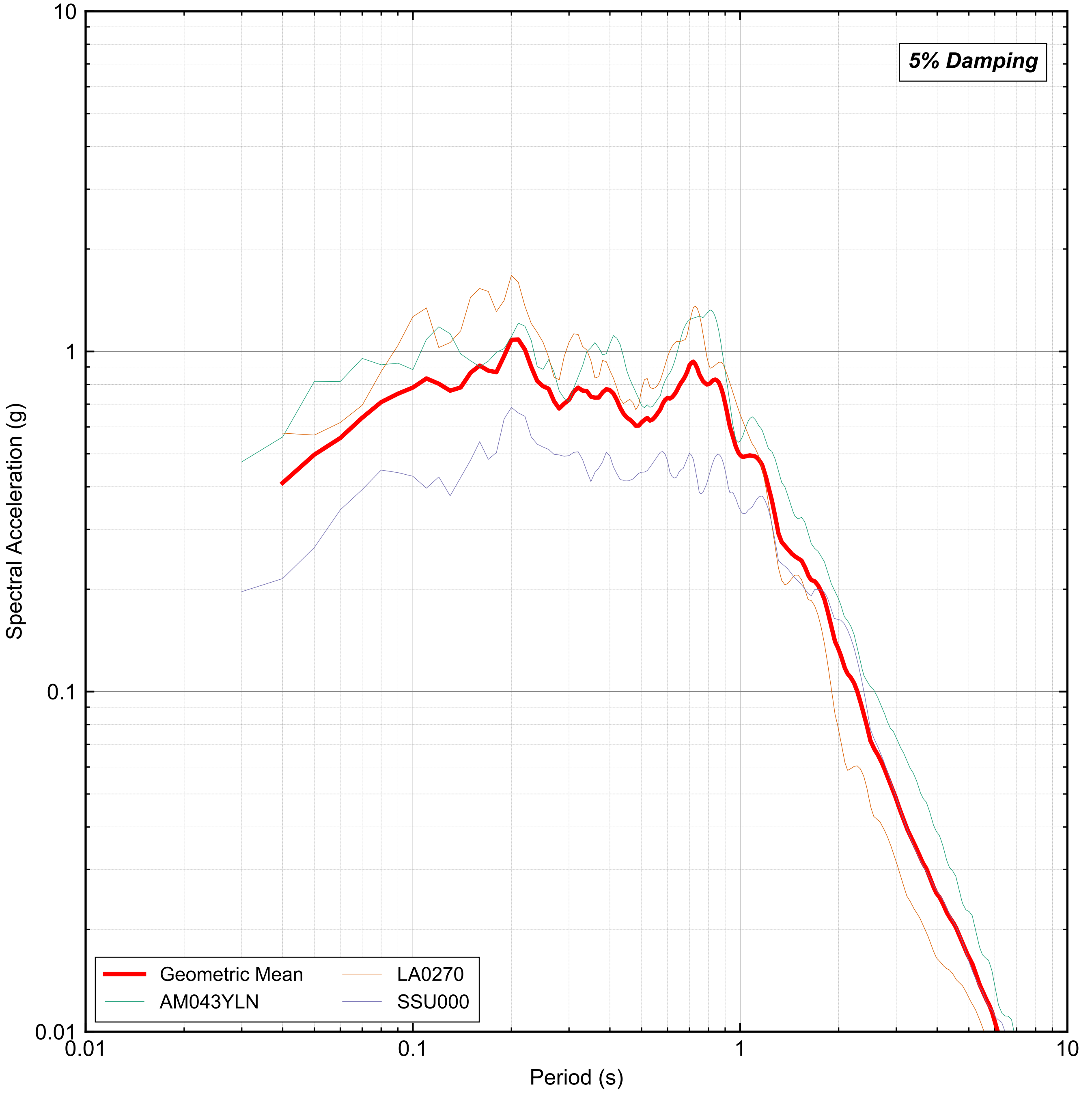
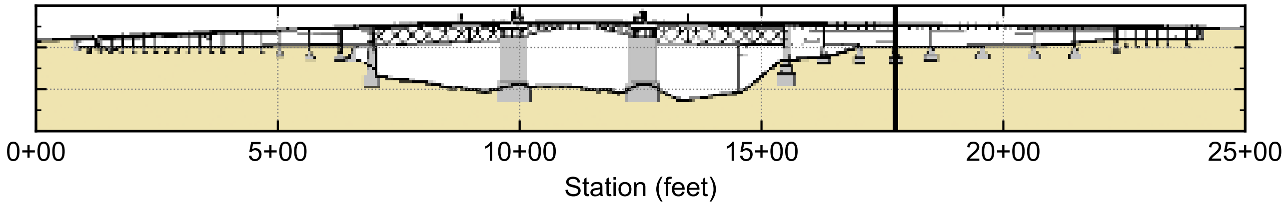
FIG. E-63

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 23

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FIG. E-64

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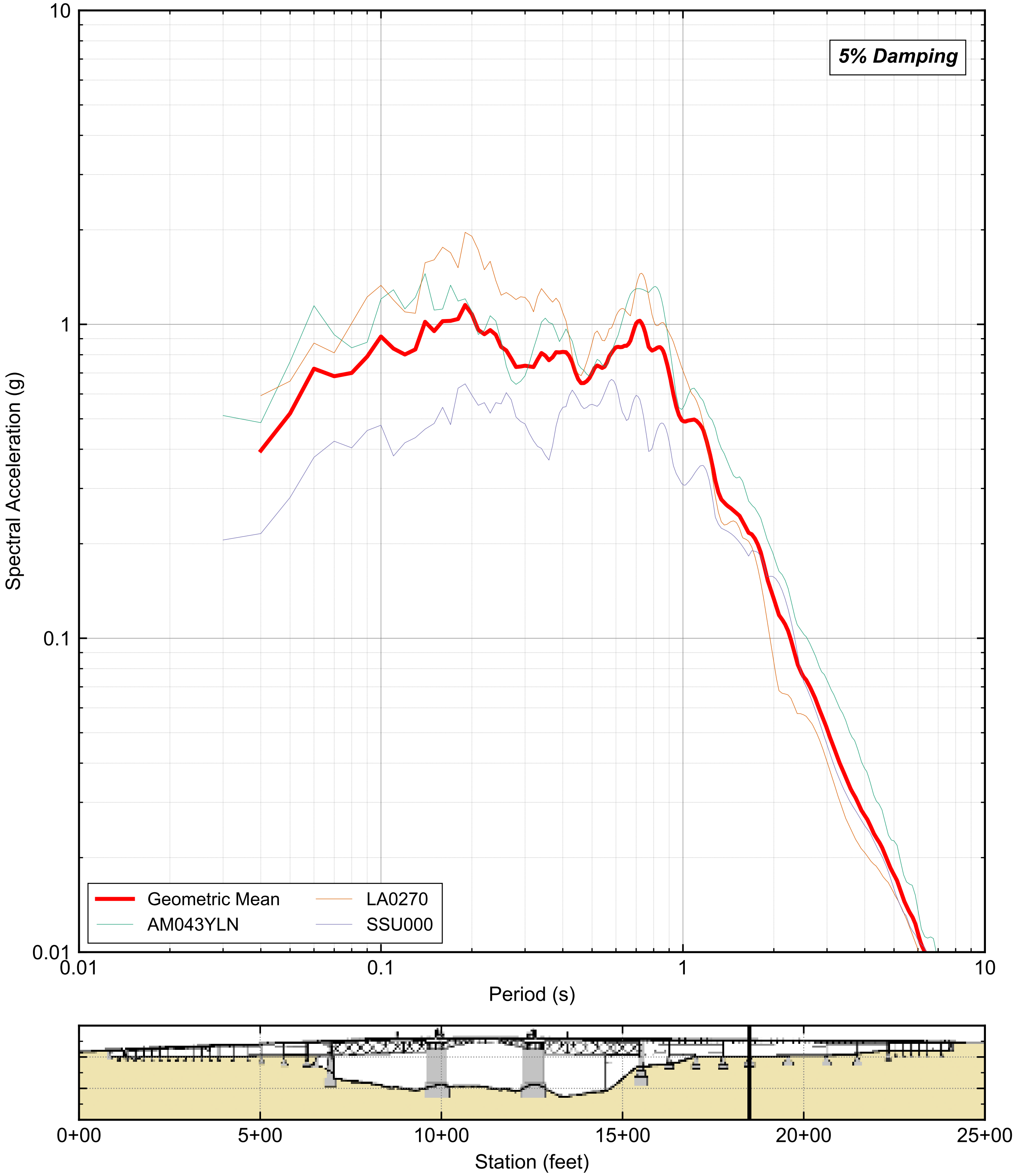
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 24

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FIG. E-65

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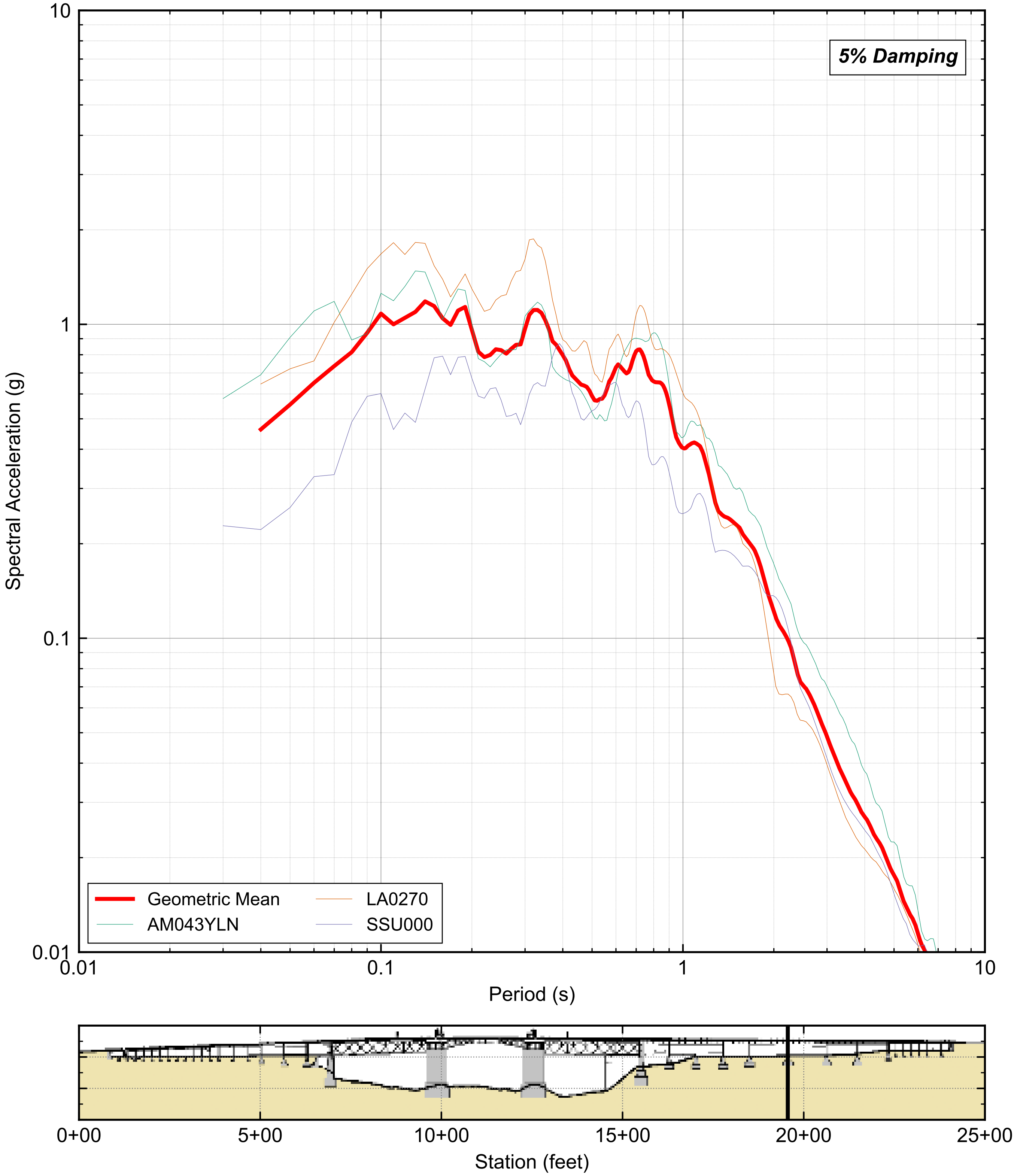
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CRUSTAL) EXISTING BENT 25

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FIG. E-66

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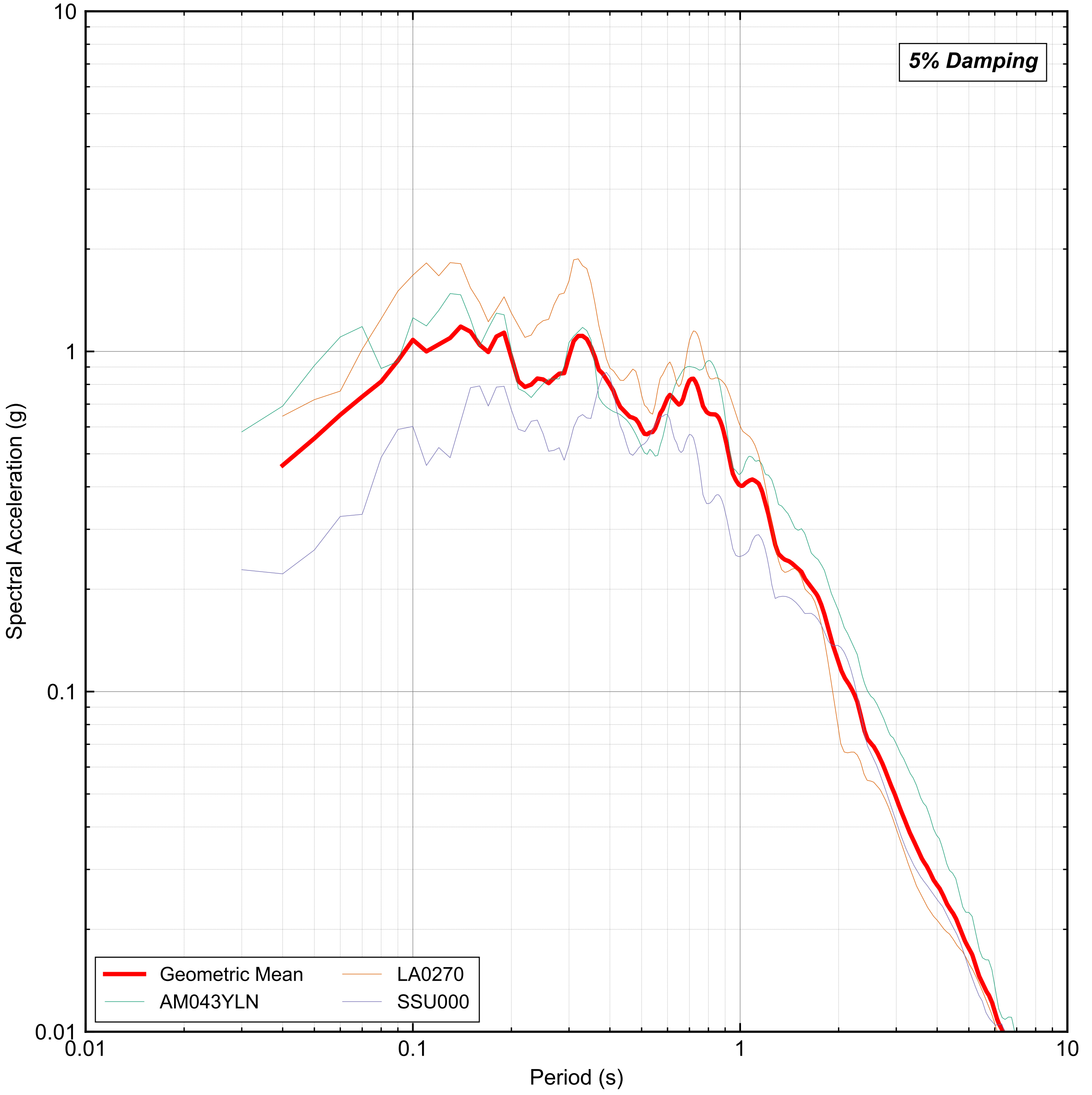

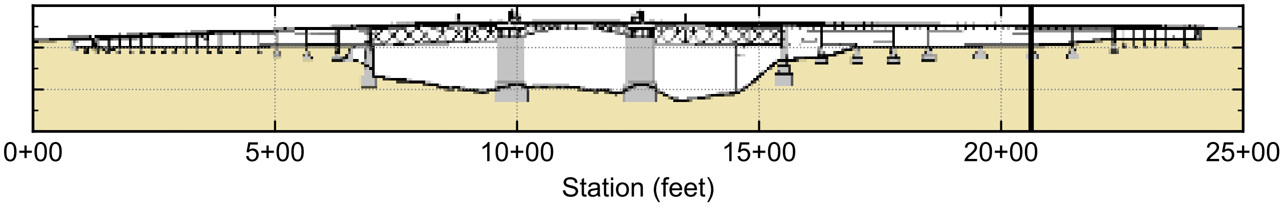

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CRUSTAL) EXISTING BENT 26

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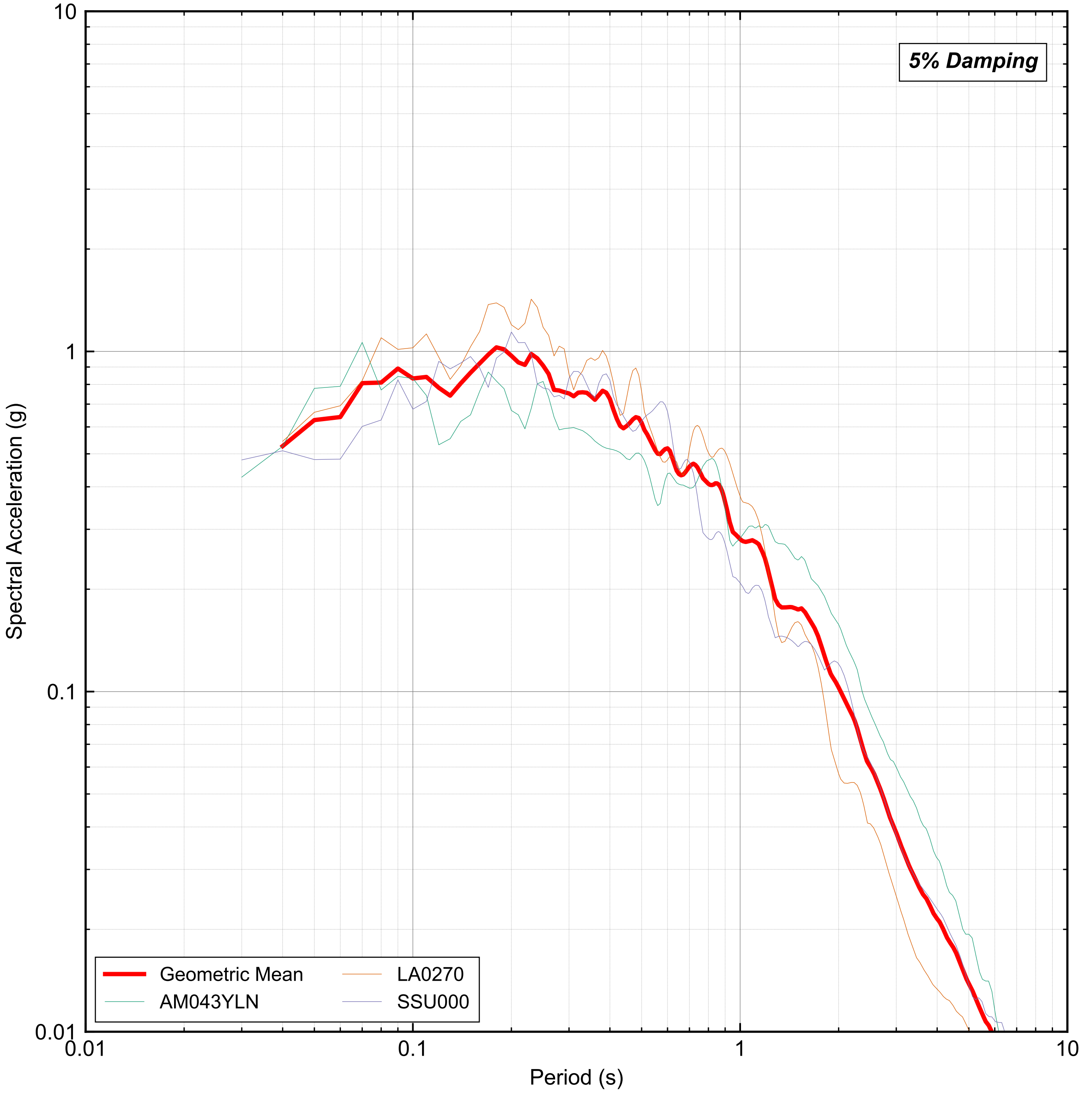
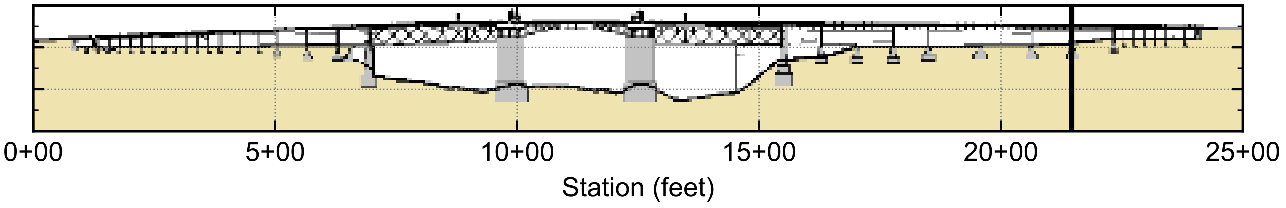
FIG. E-67





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 27

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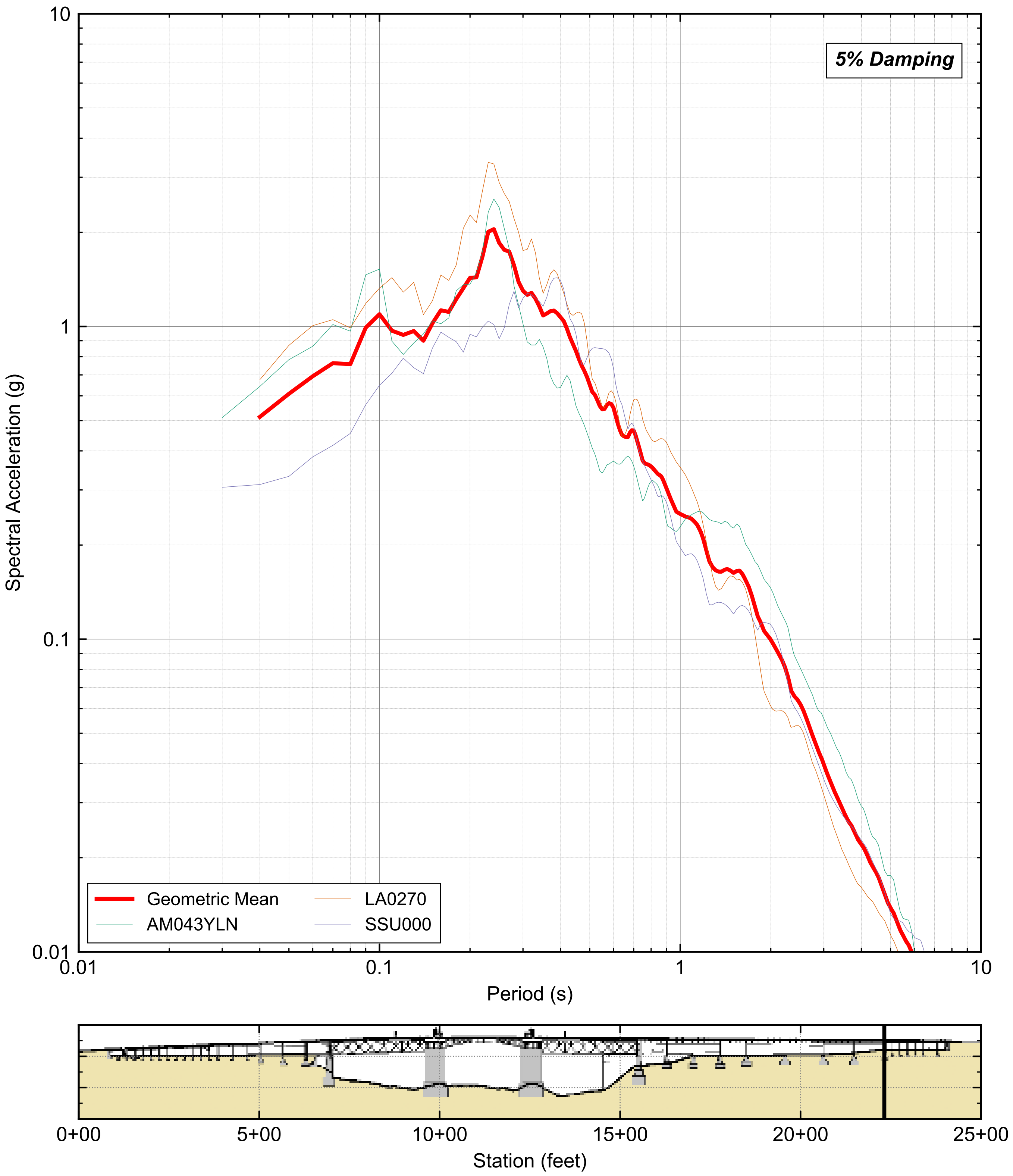
FIG. E-68

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 28

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FIG. E-69

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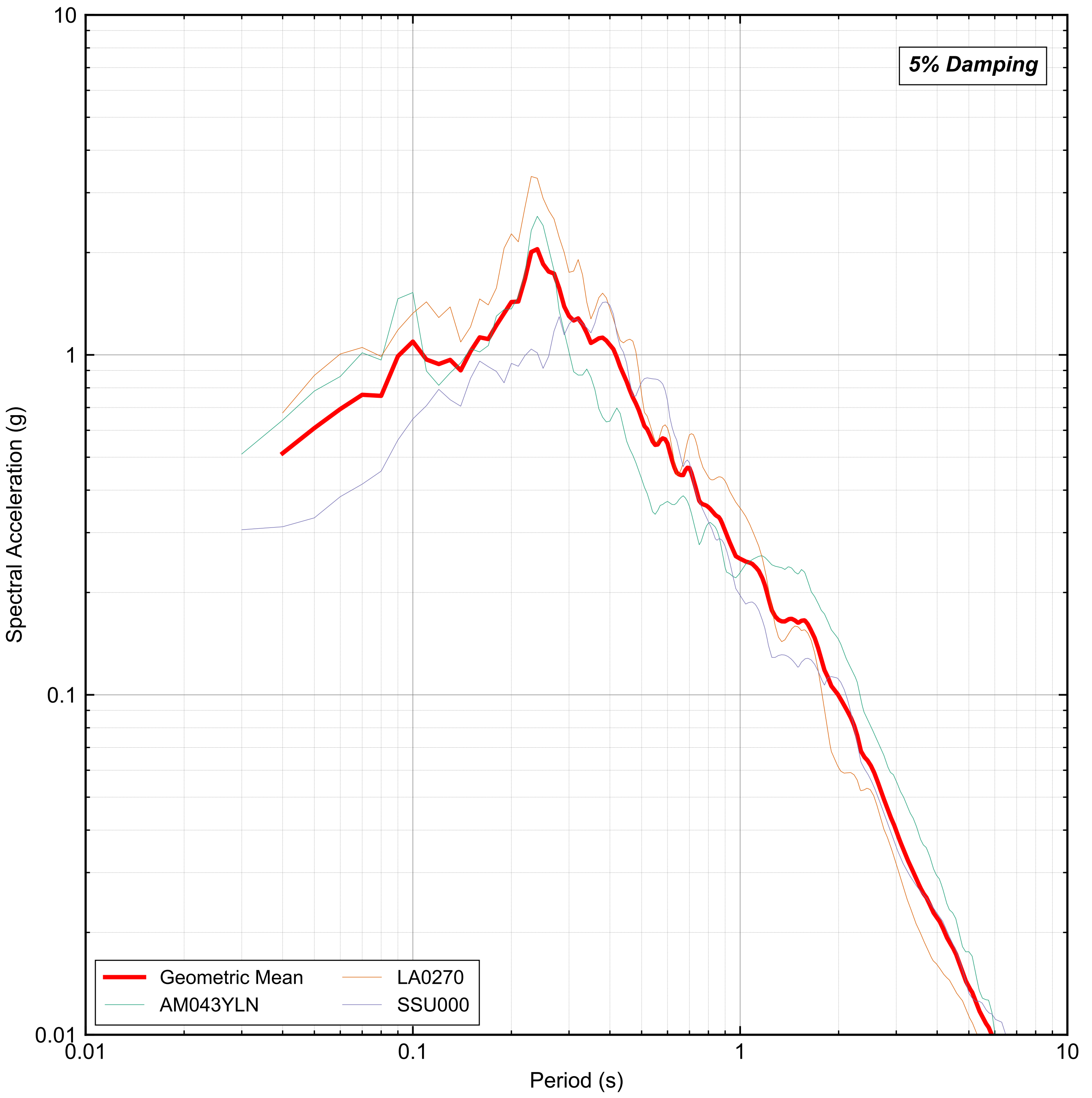

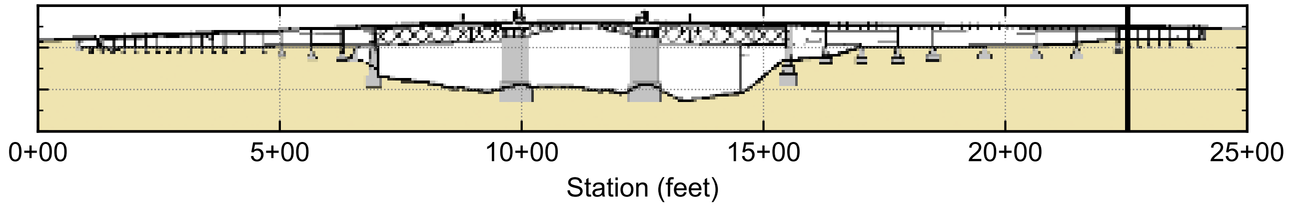

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 29

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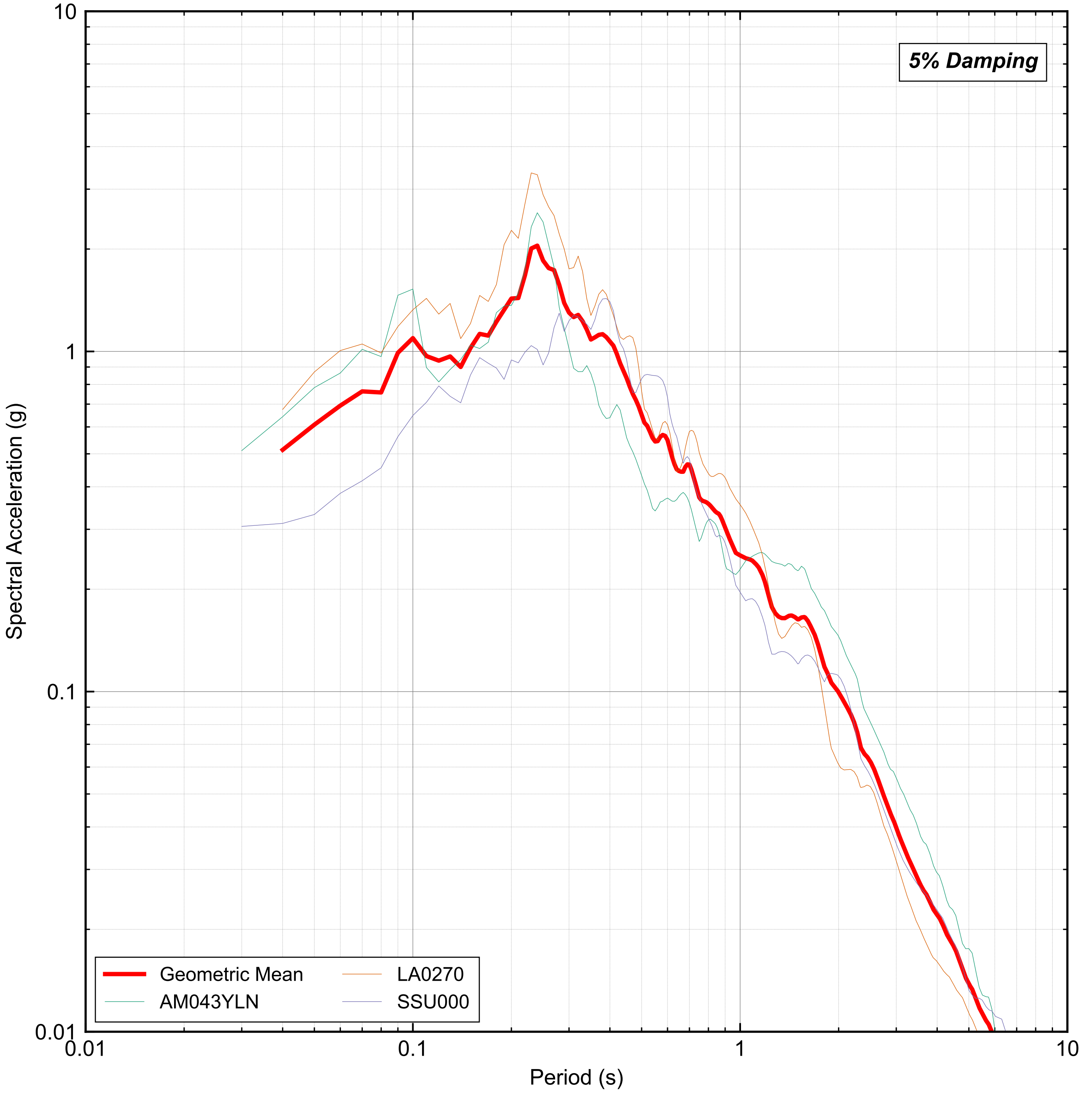
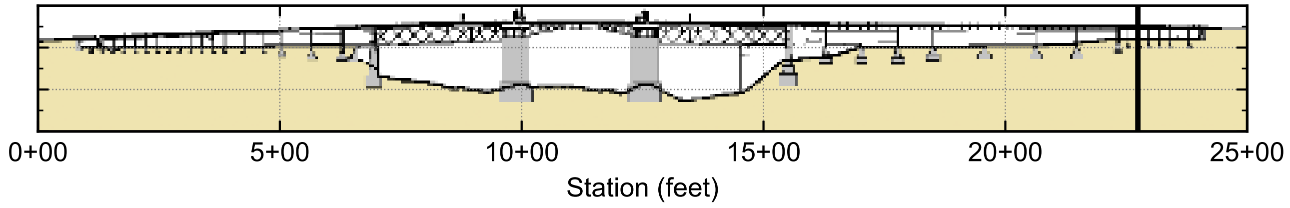
FIG. E-70





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 30

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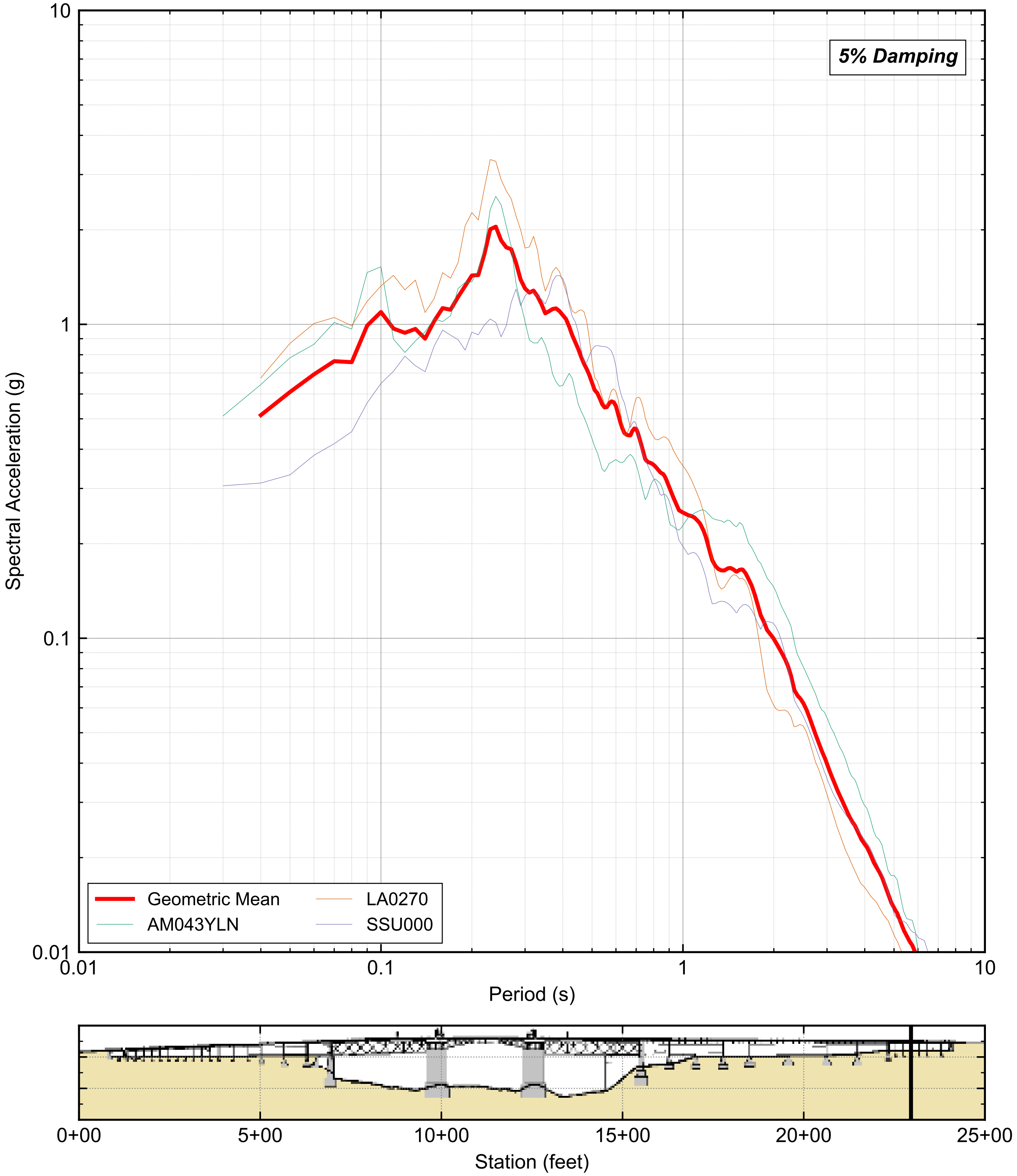
FIG. E-71

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 31

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FIG. E-72

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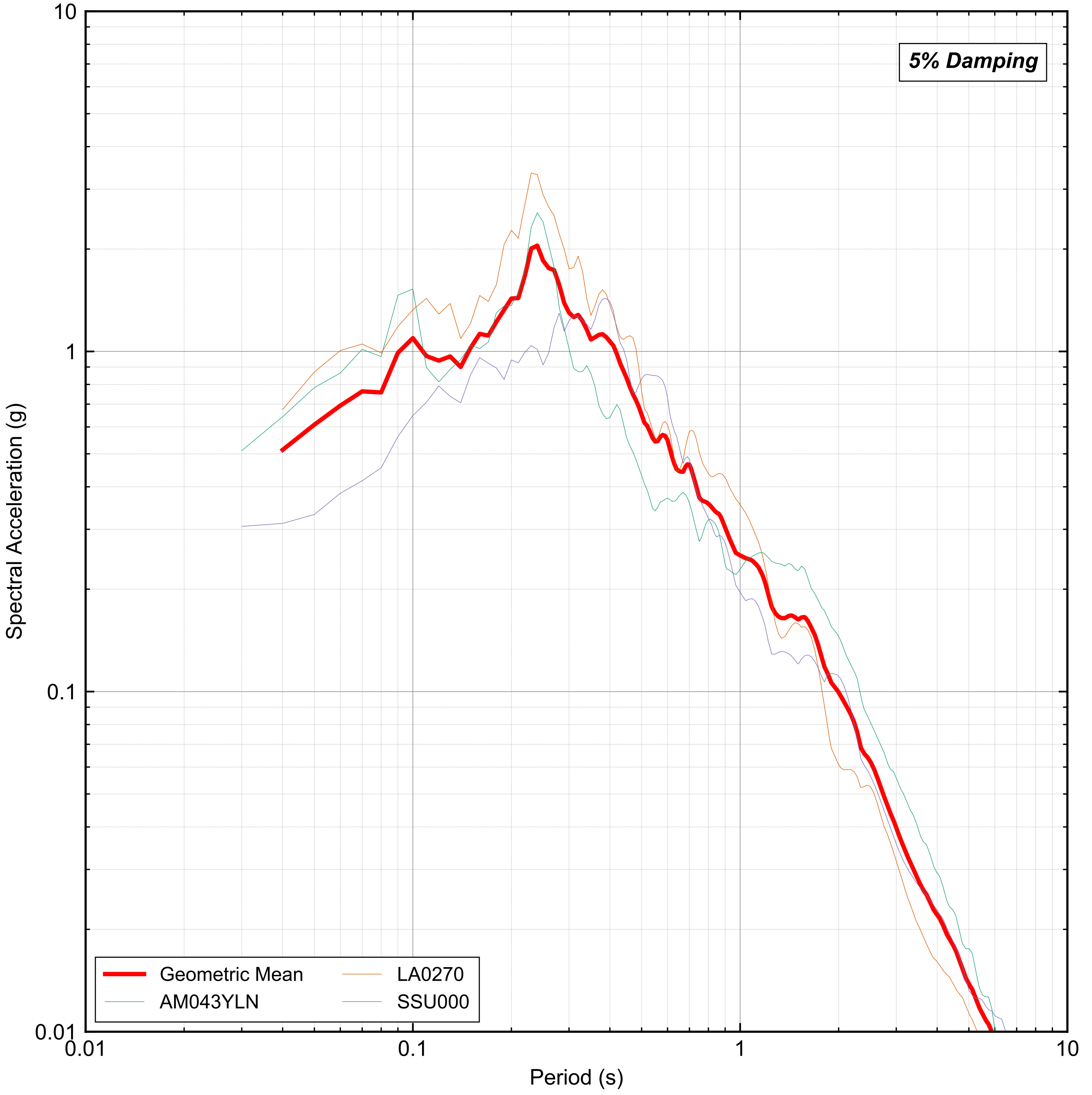

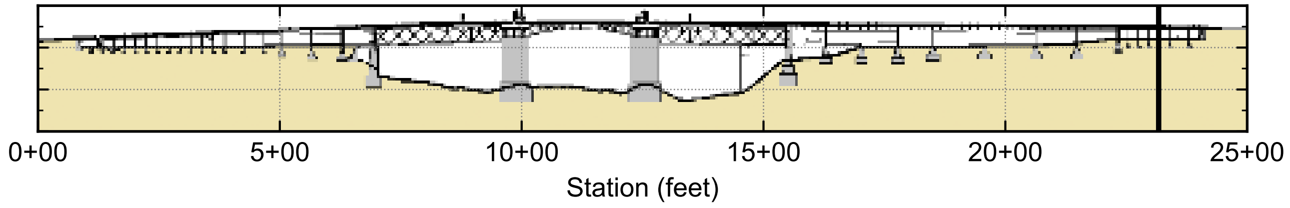

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 32

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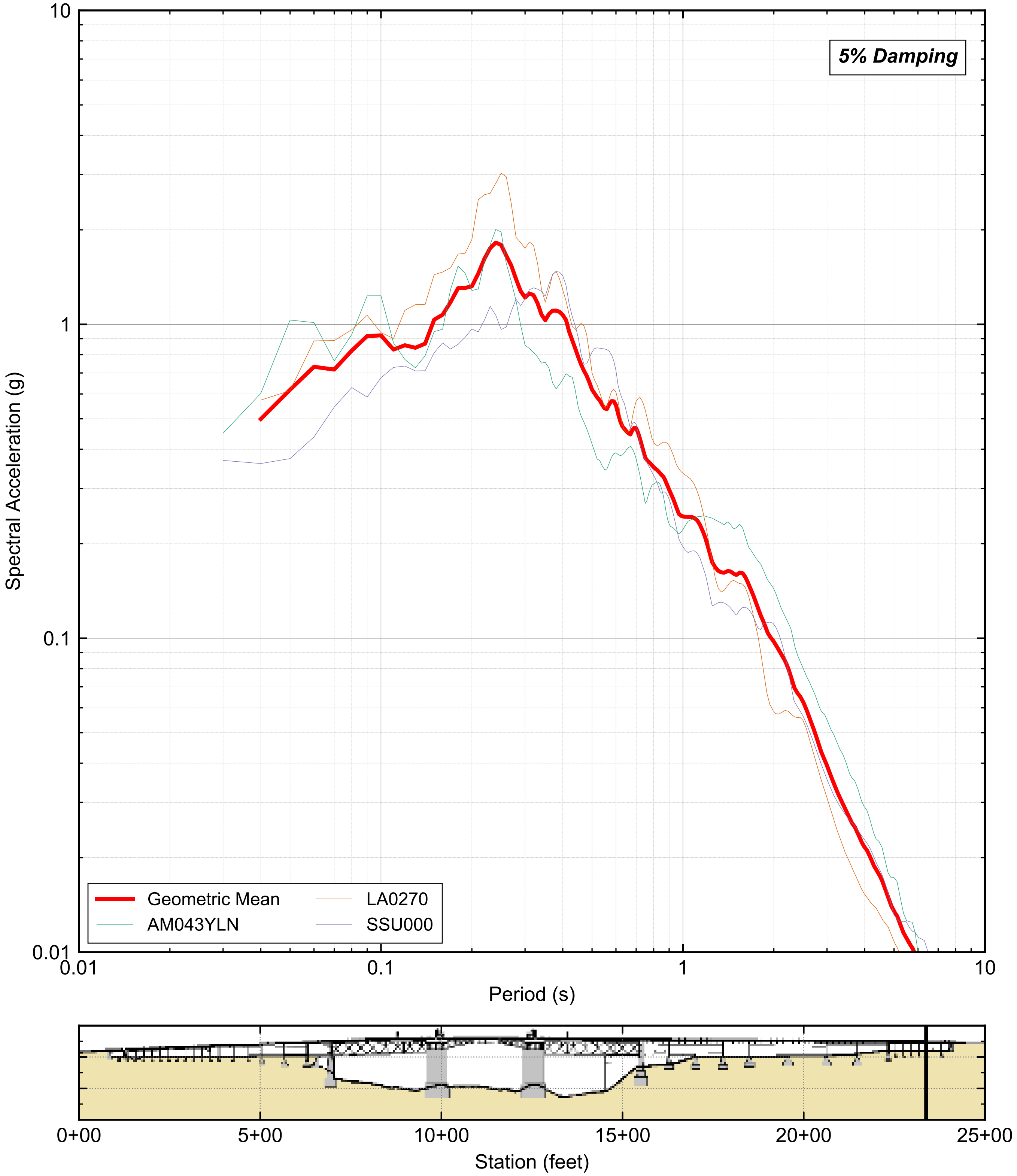
FIG. E-73

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 33

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FIG. E-74

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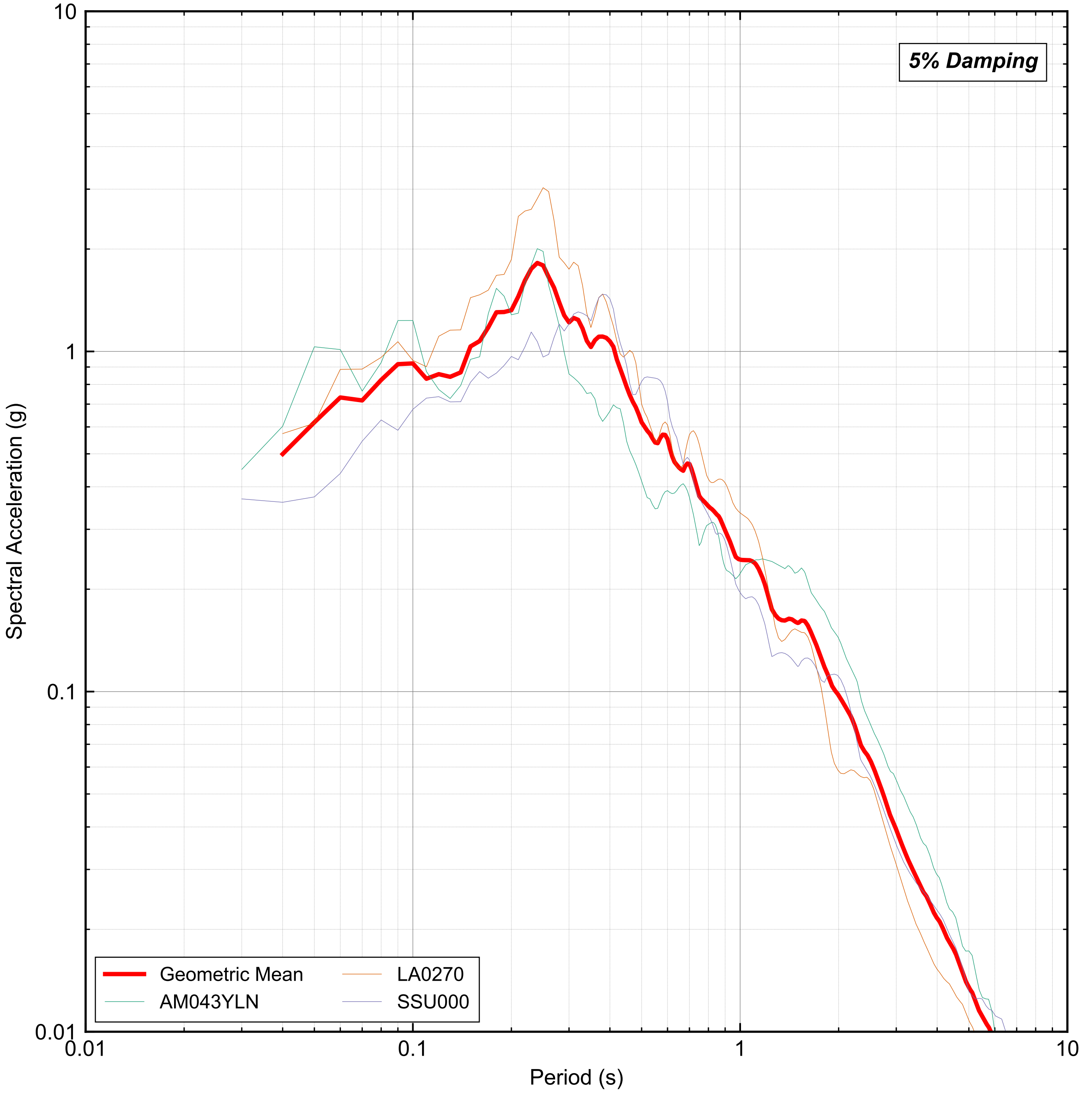

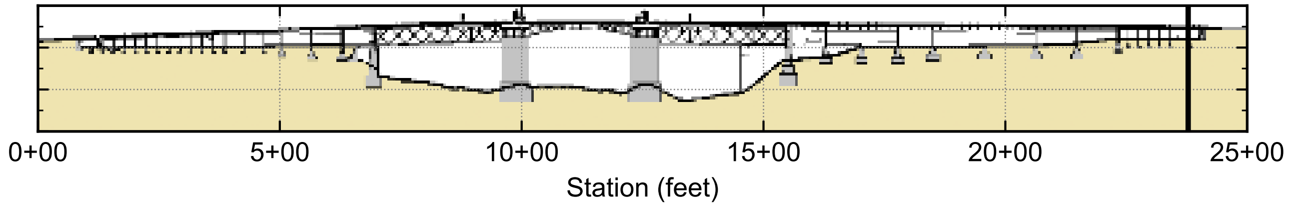

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 34

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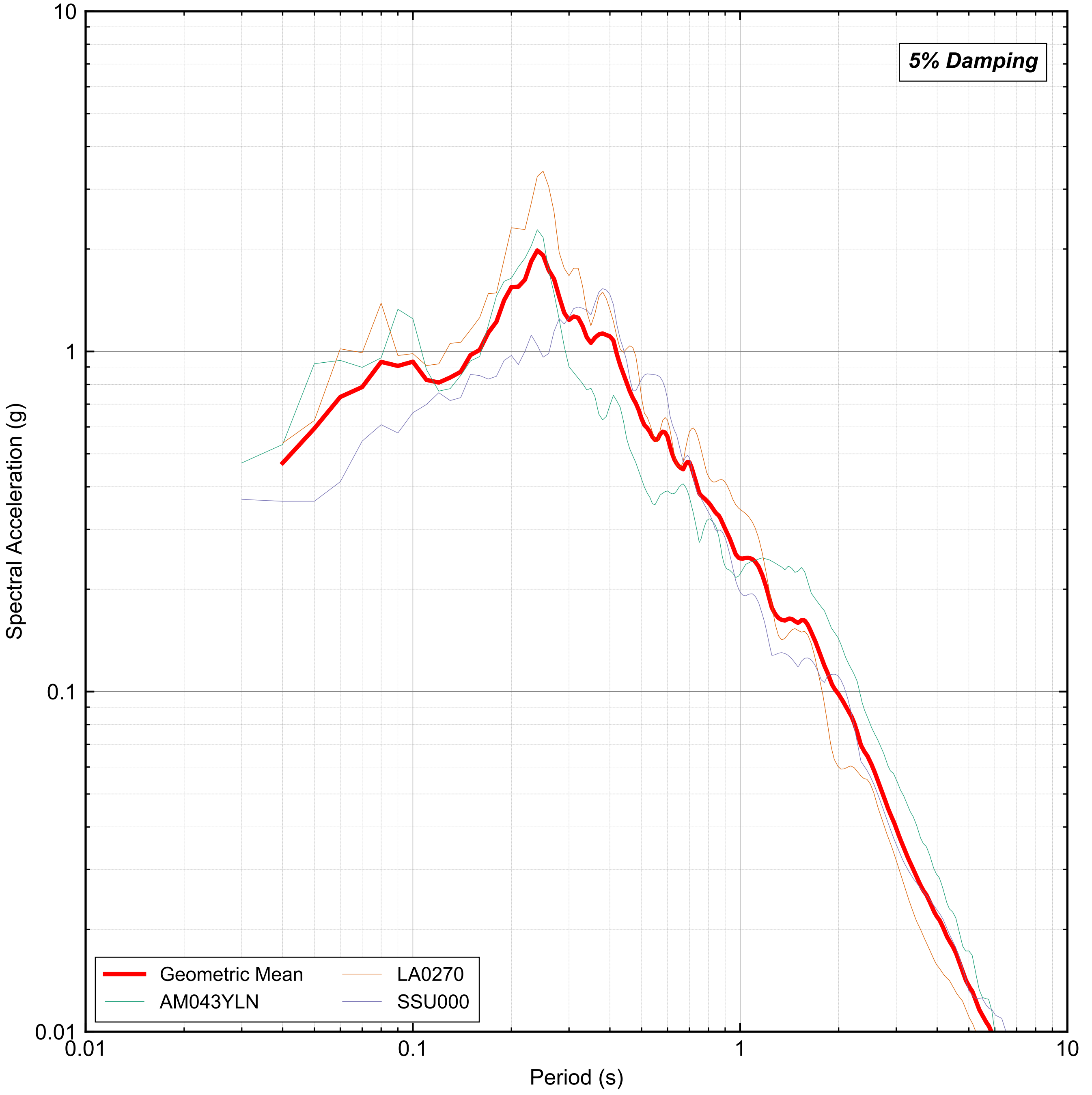
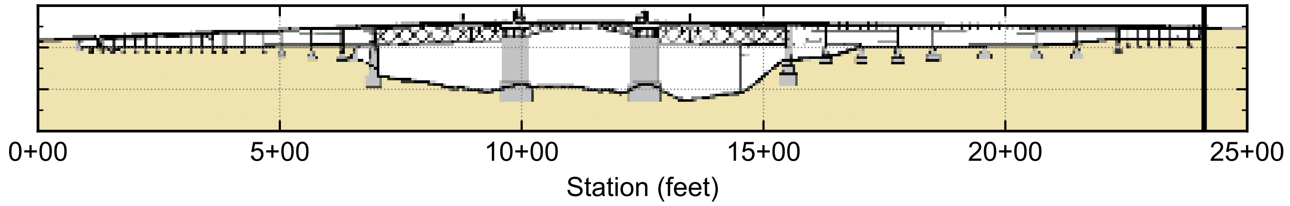
FIG. E-75





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CRUSTAL)
EXISTING BENT 35

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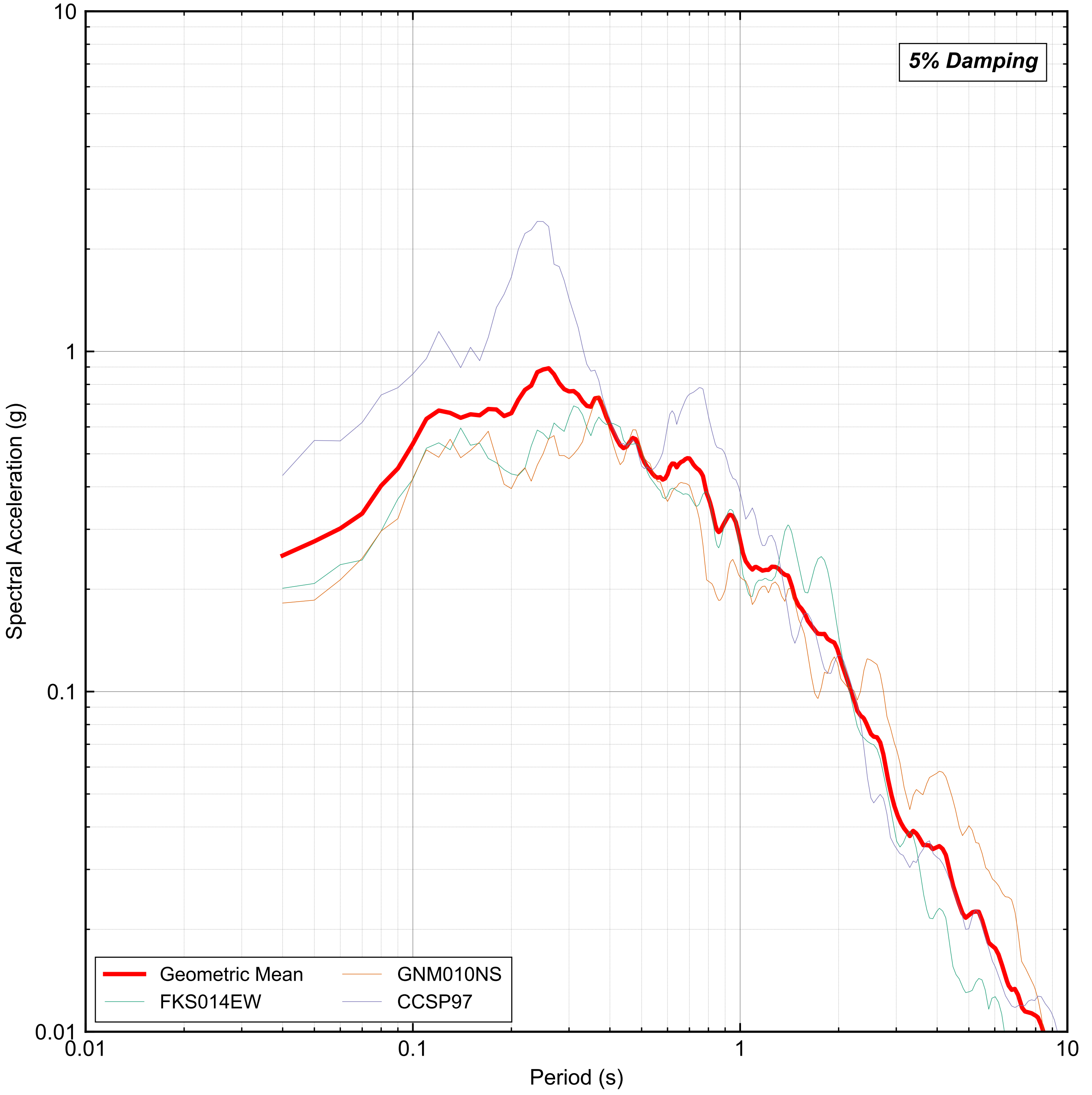
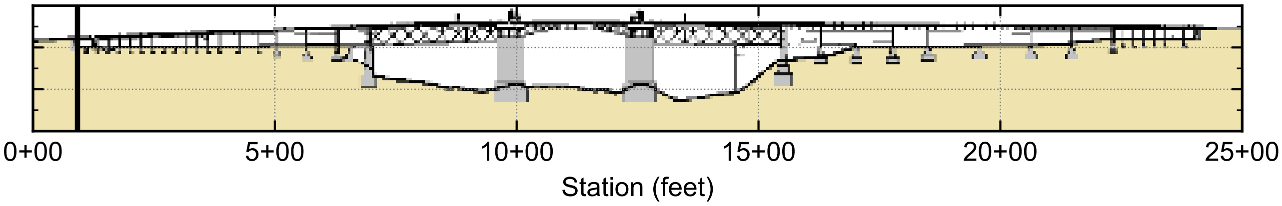
FIG. E-76

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 1

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FIG. E-77

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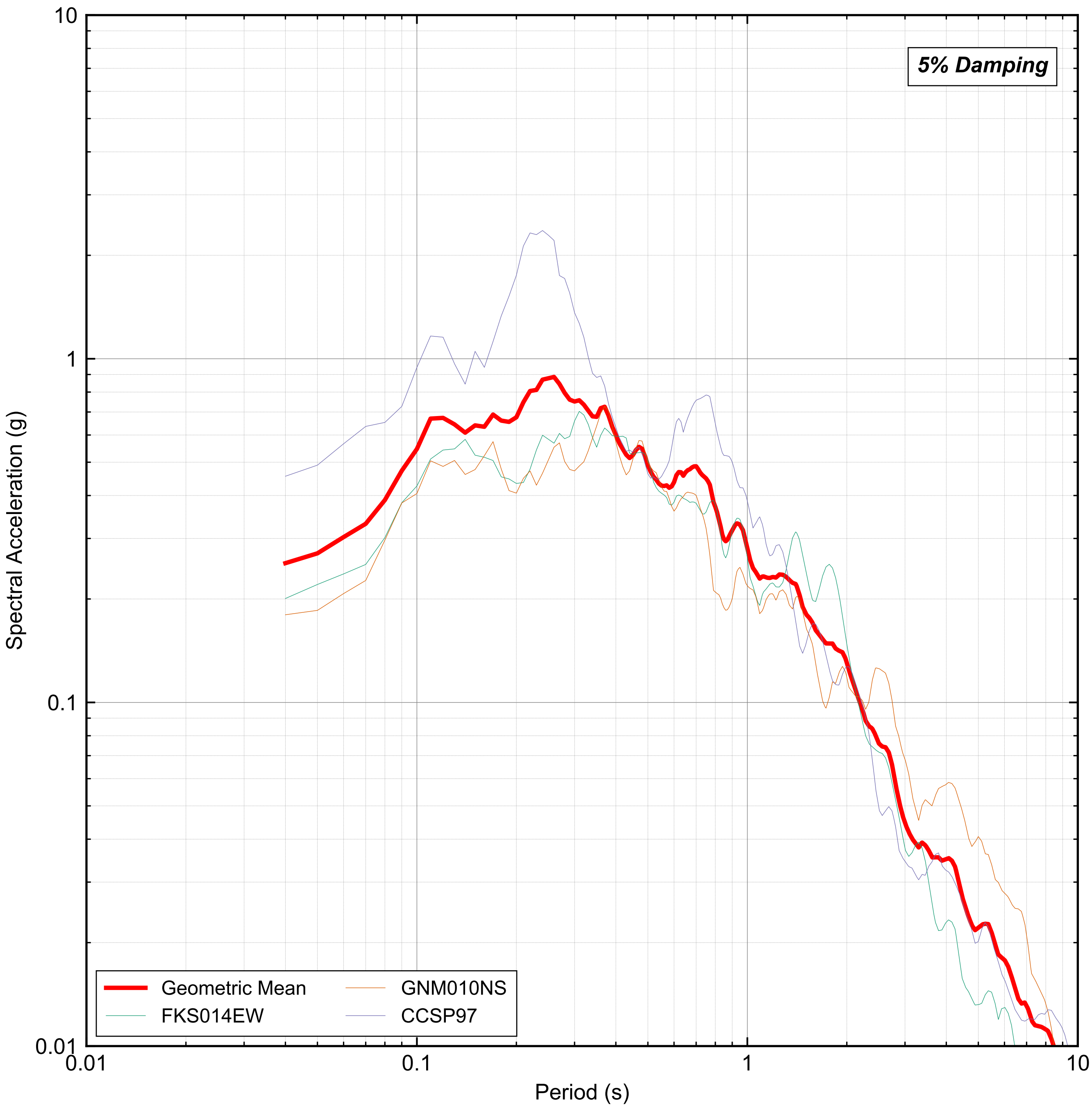

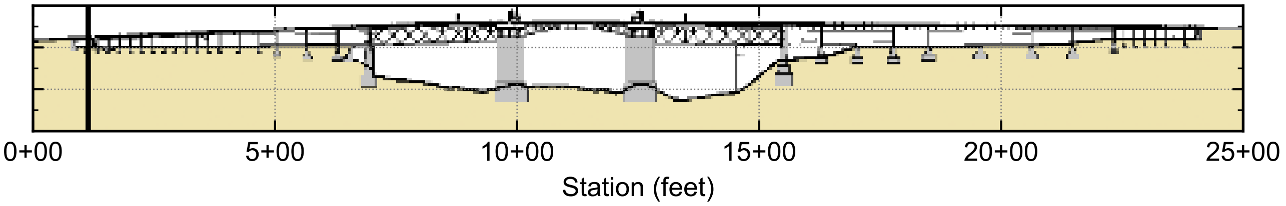

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 2

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FIG. E-78

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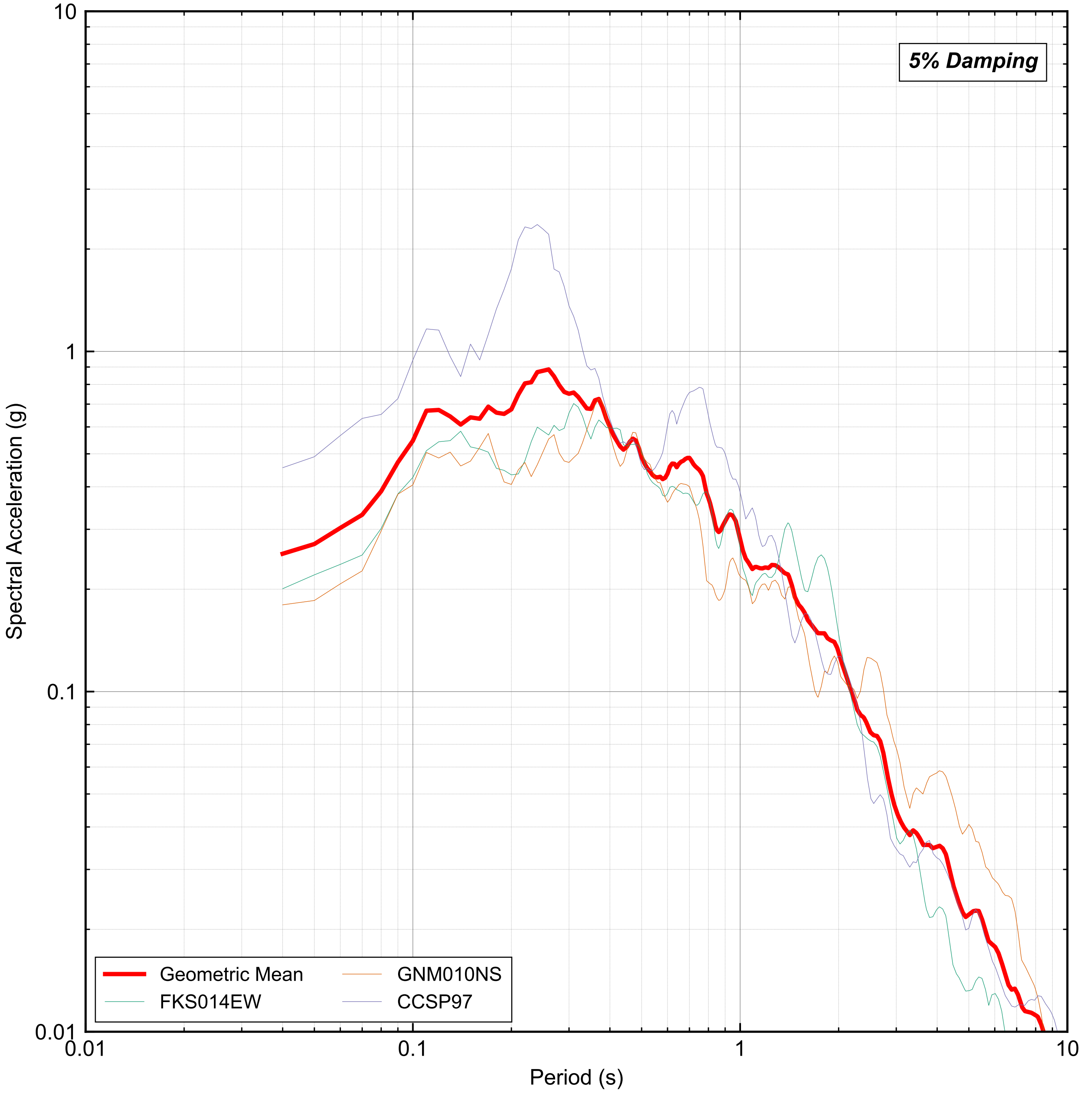

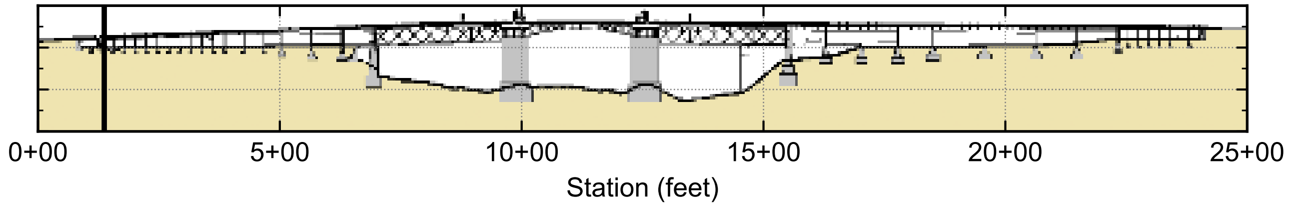

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CSZ 0.2S) EXISTING BENT 3

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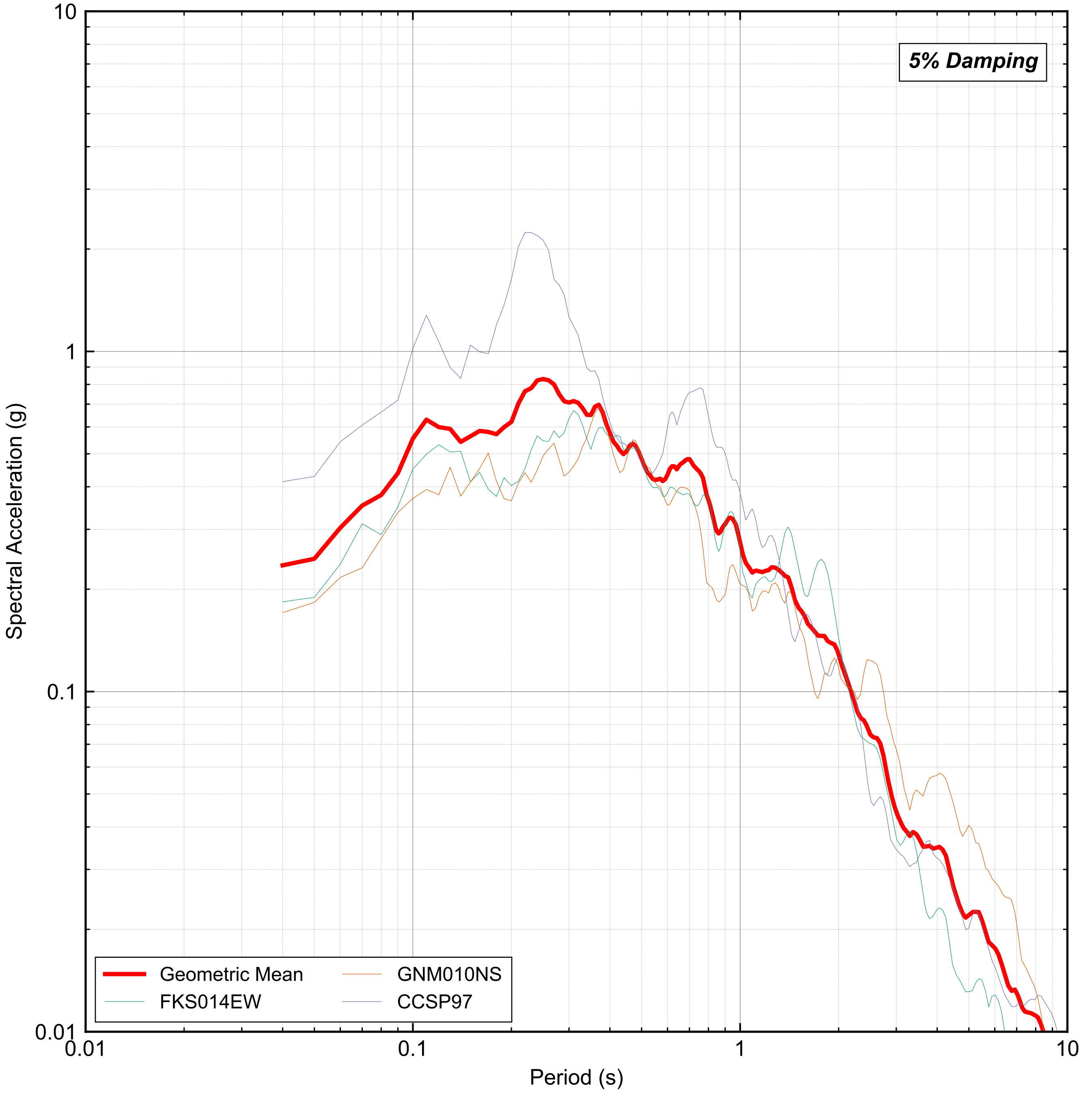
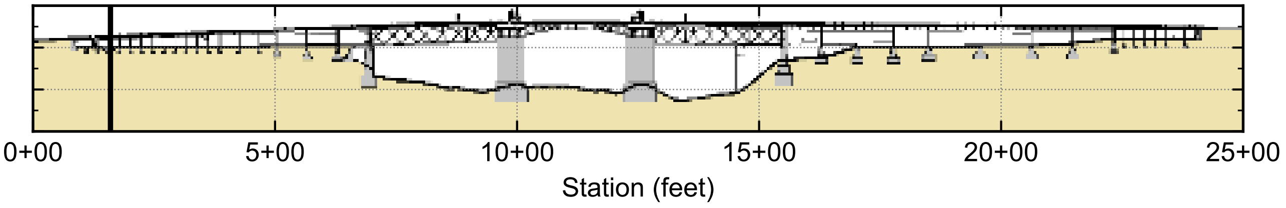
FIG. E-79





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 4

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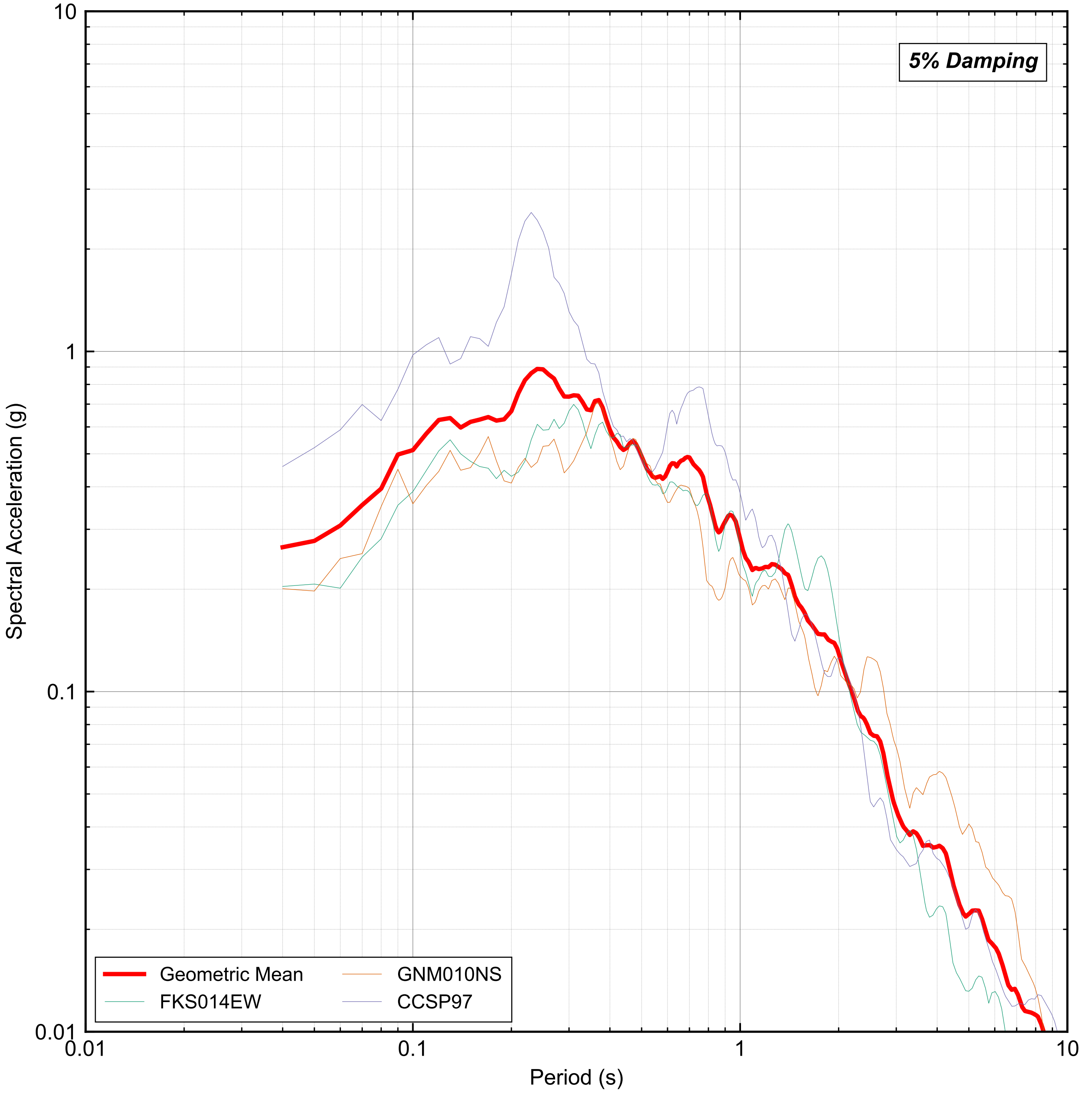
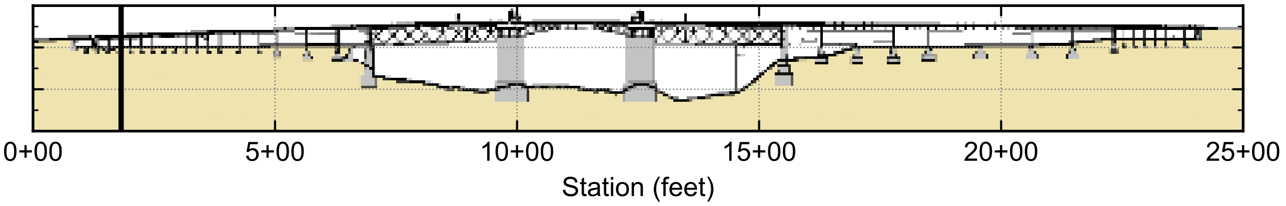
FIG. E-80





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 5

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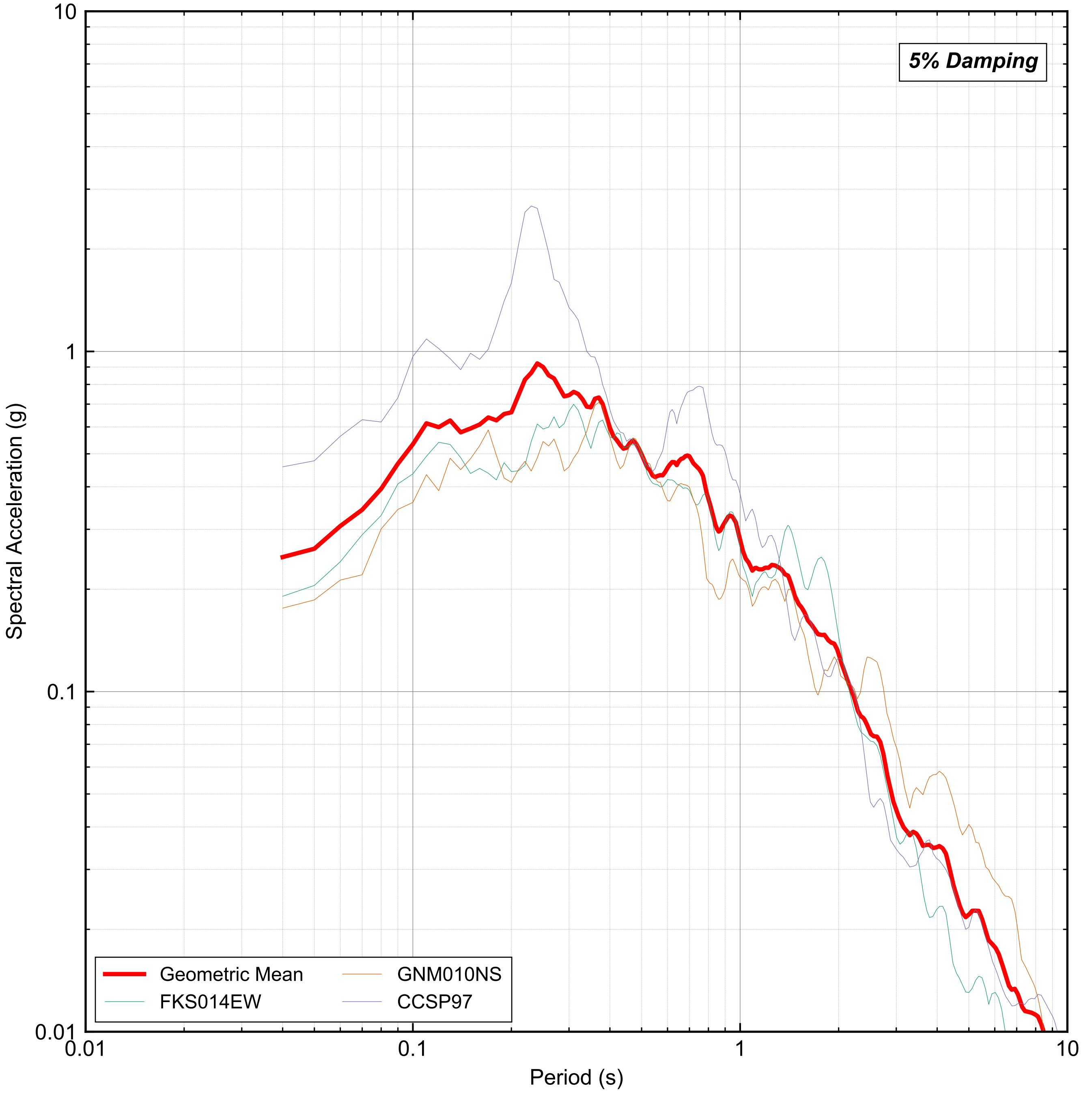
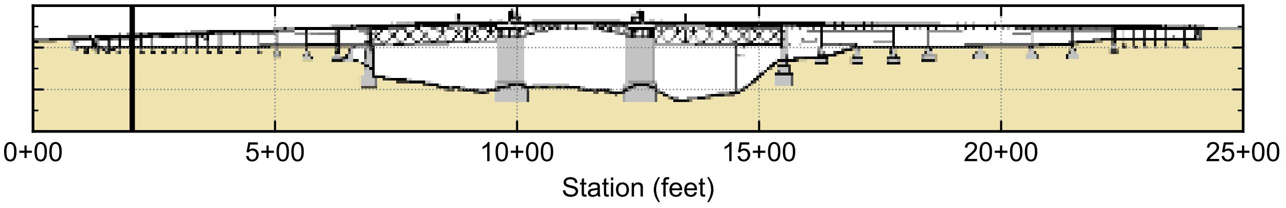
FIG. E-81





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CSZ 0.2S) EXISTING BENT 6

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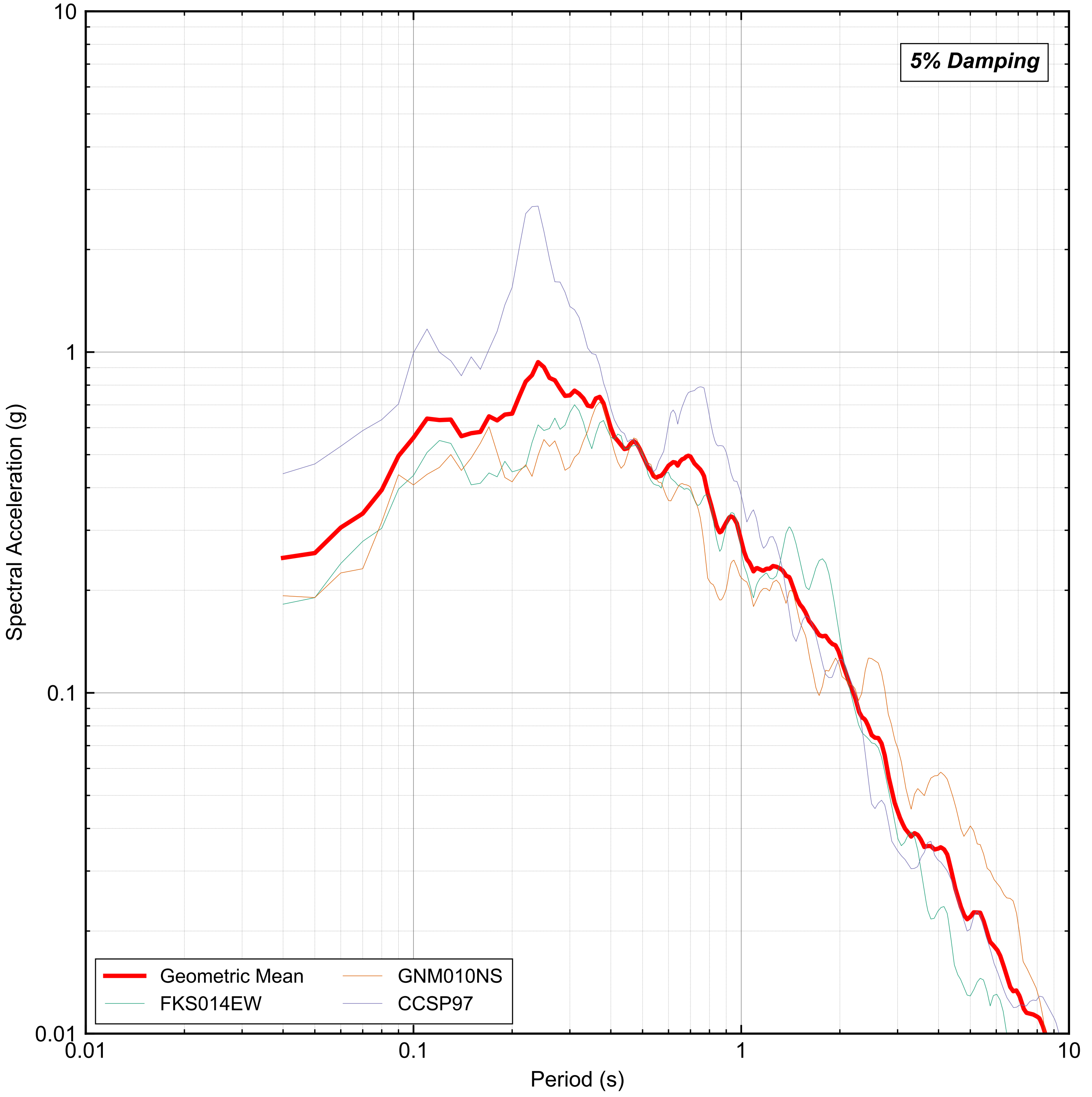
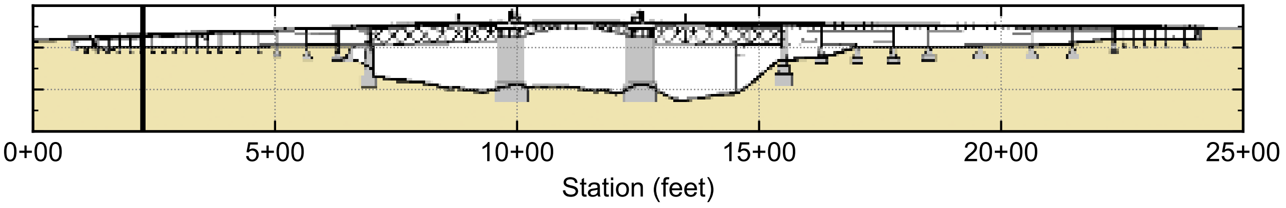
FIG. E-82





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 7

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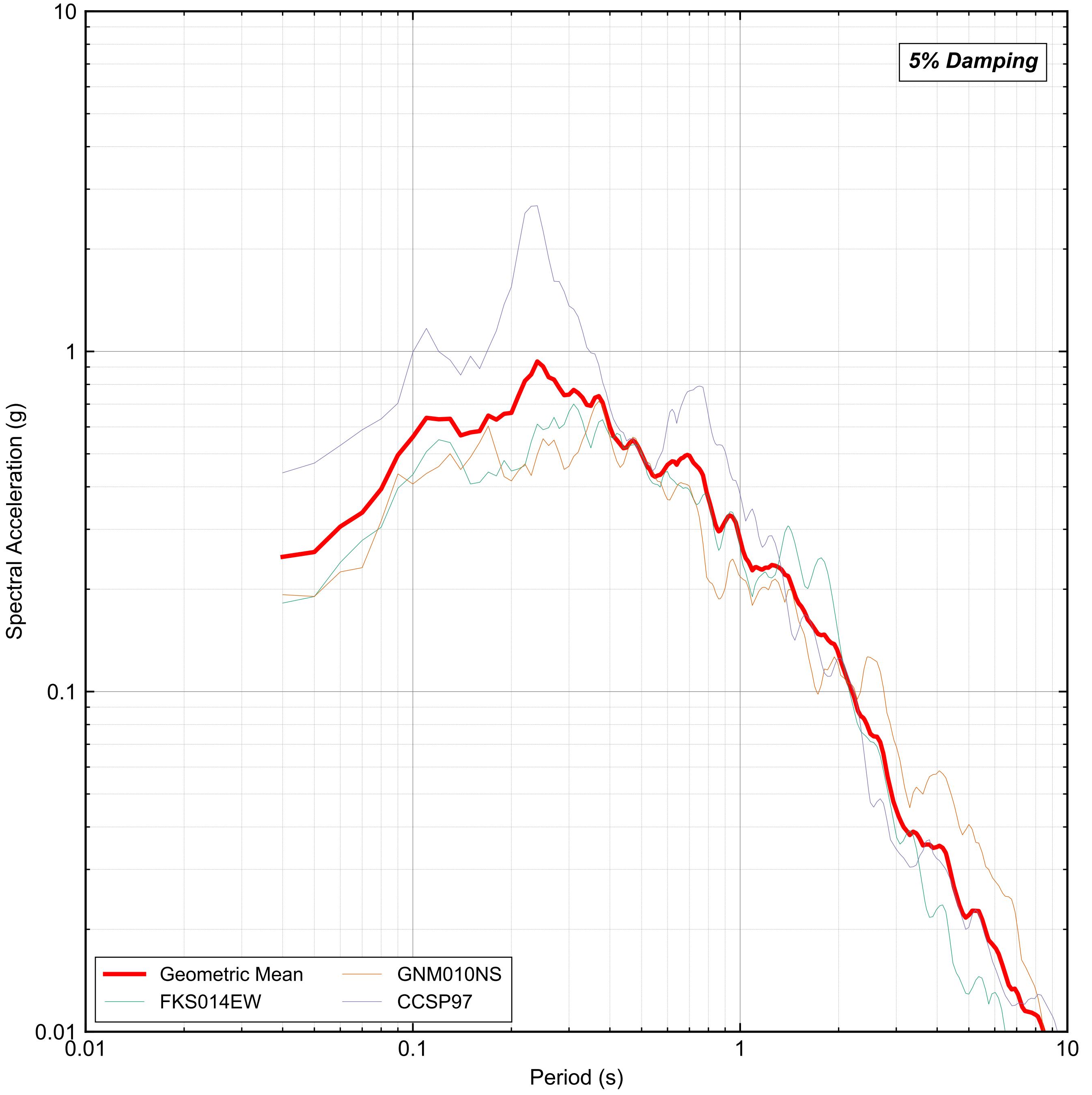
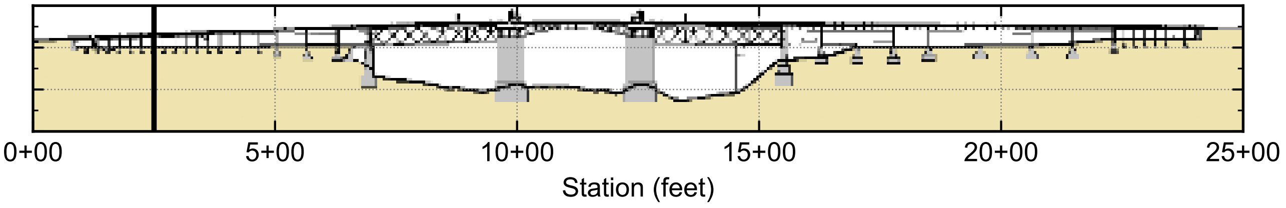
FIG. E-83





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 8

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FIG. E-84

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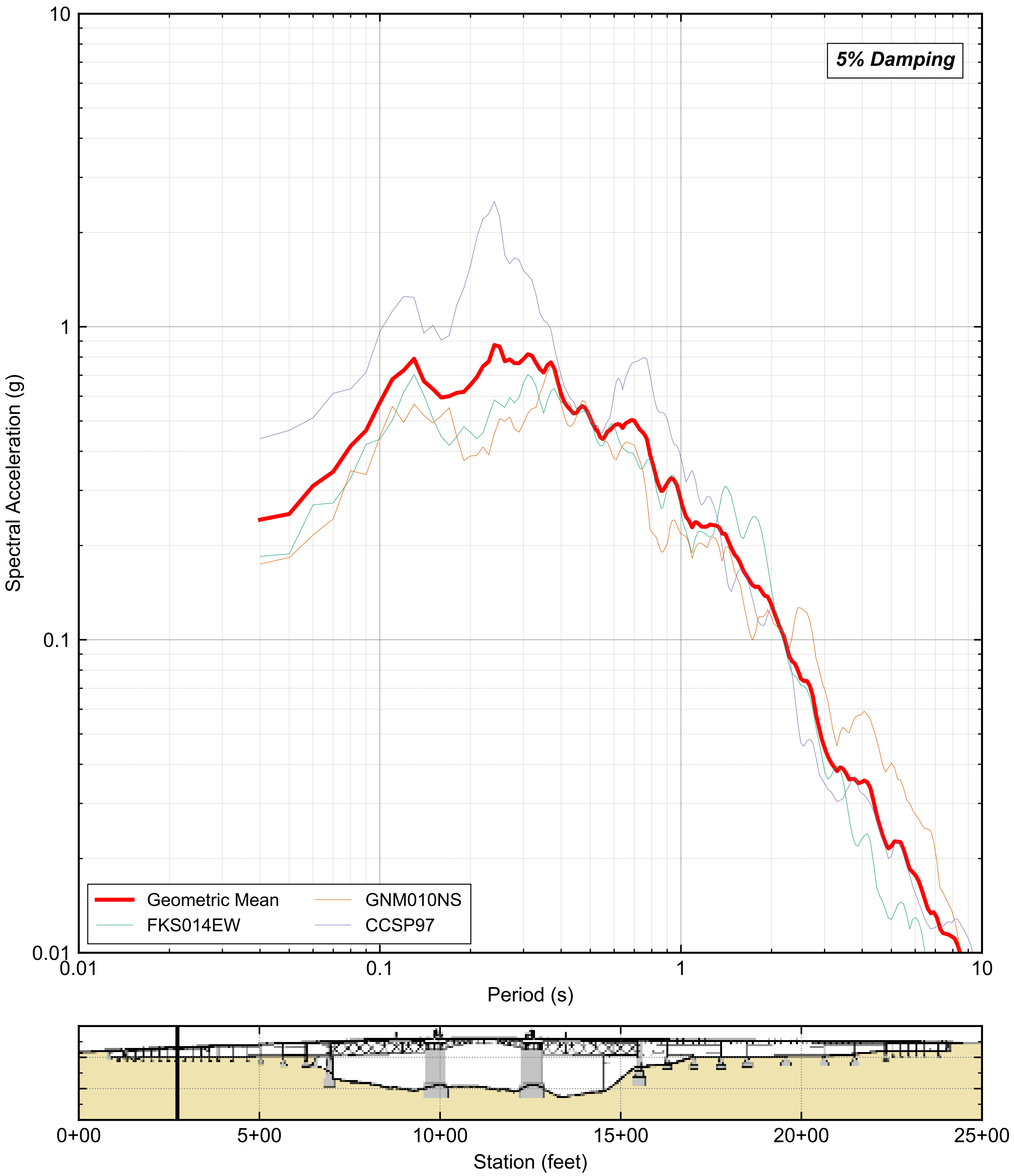

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 9

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FIG. E-85

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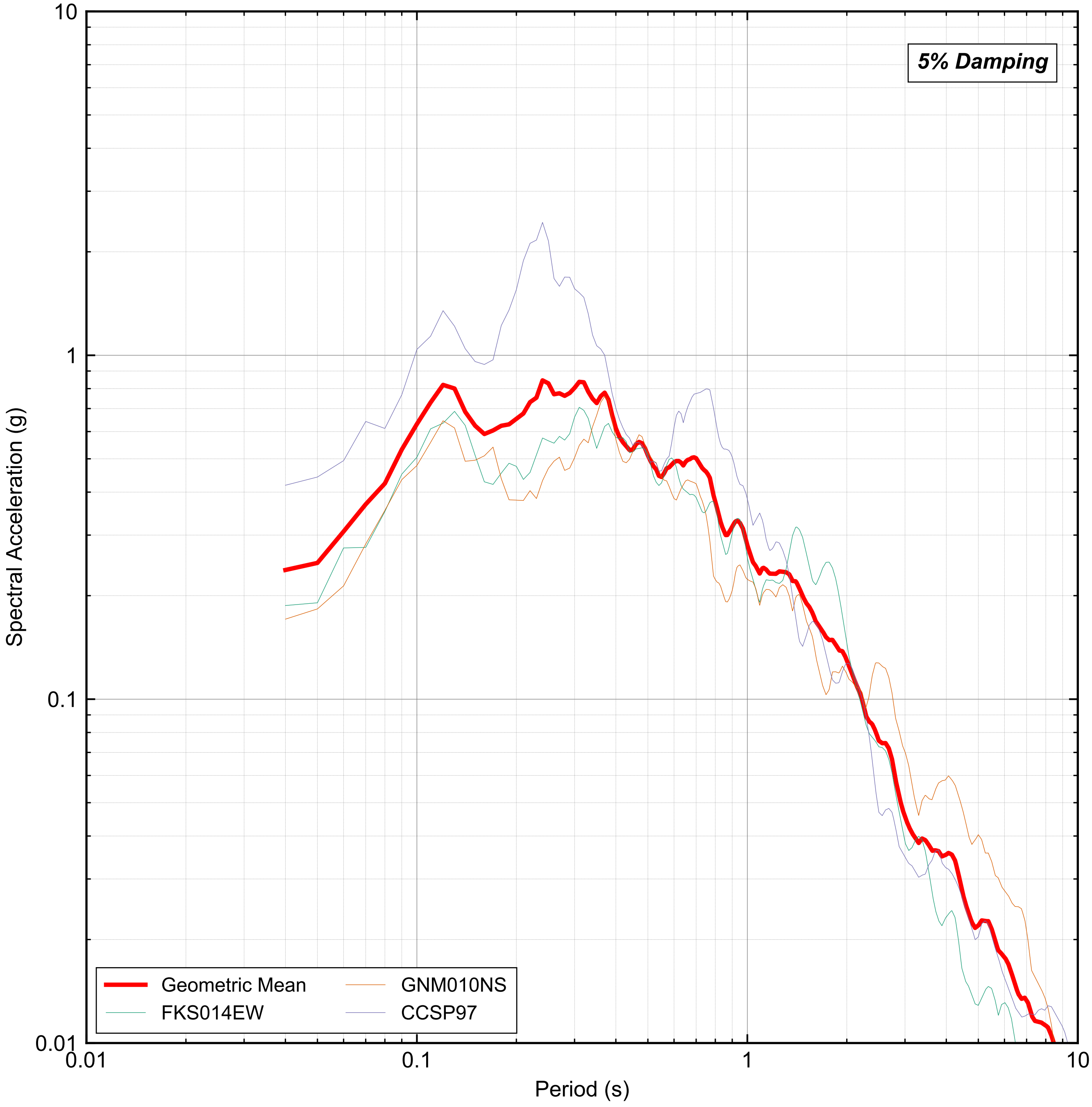

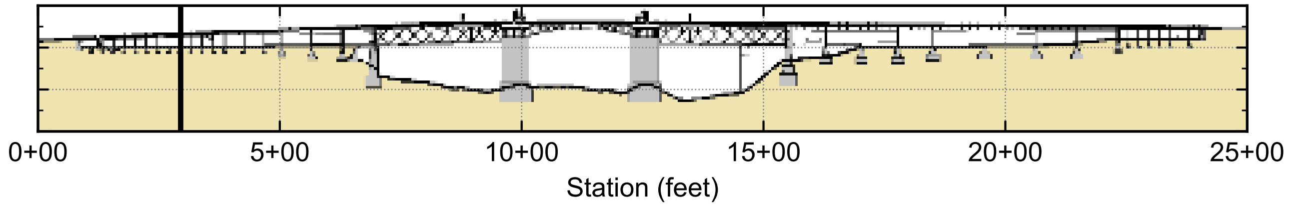

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CSZ 0.2S) EXISTING BENT 10

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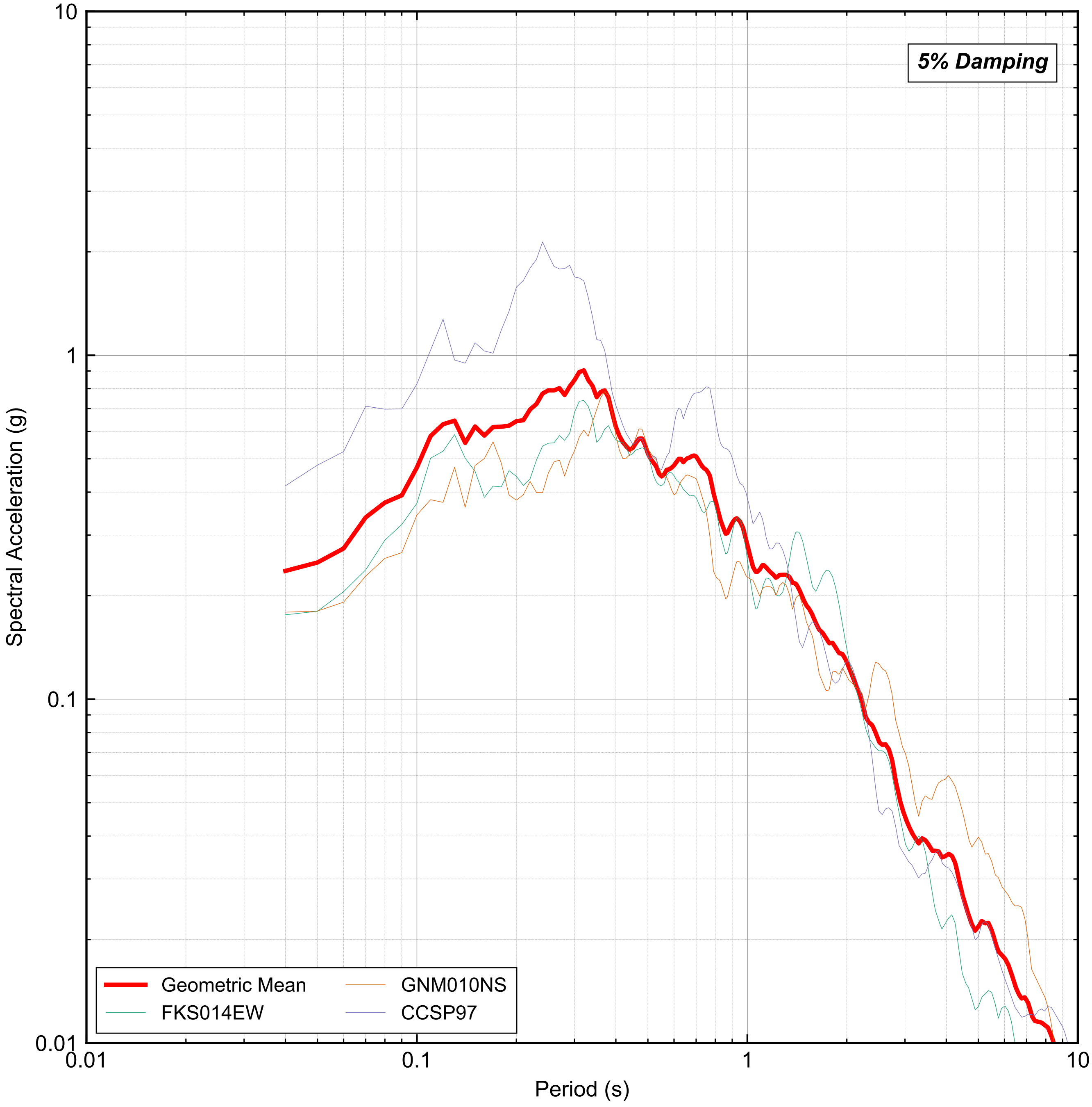
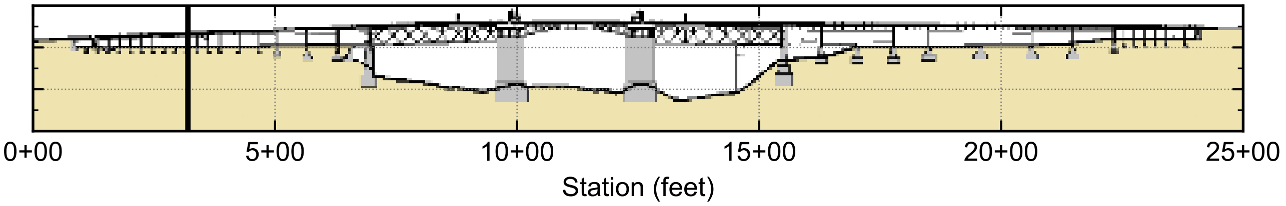
FIG. E-86





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 11

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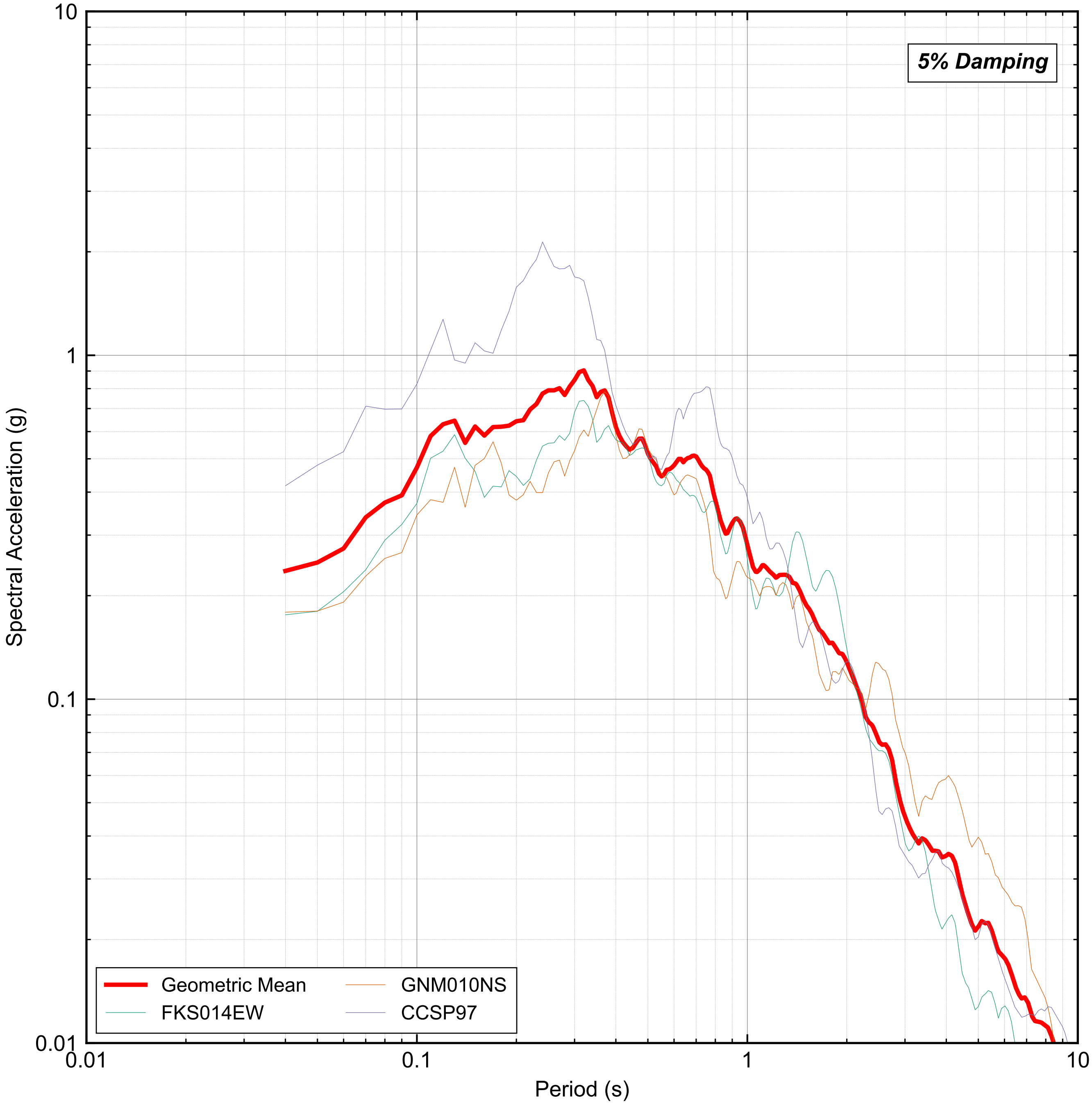
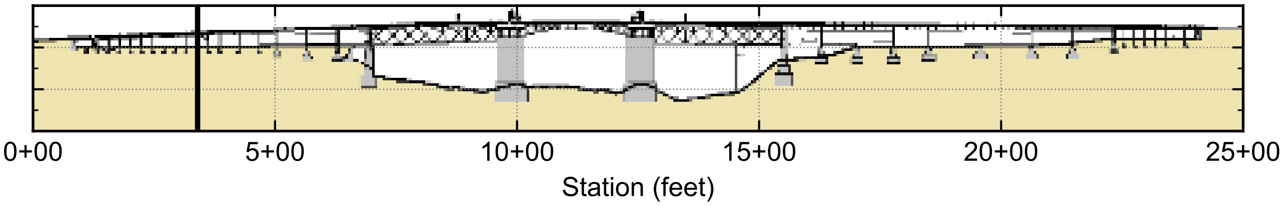
FIG. E-87

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 12

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FIG. E-88

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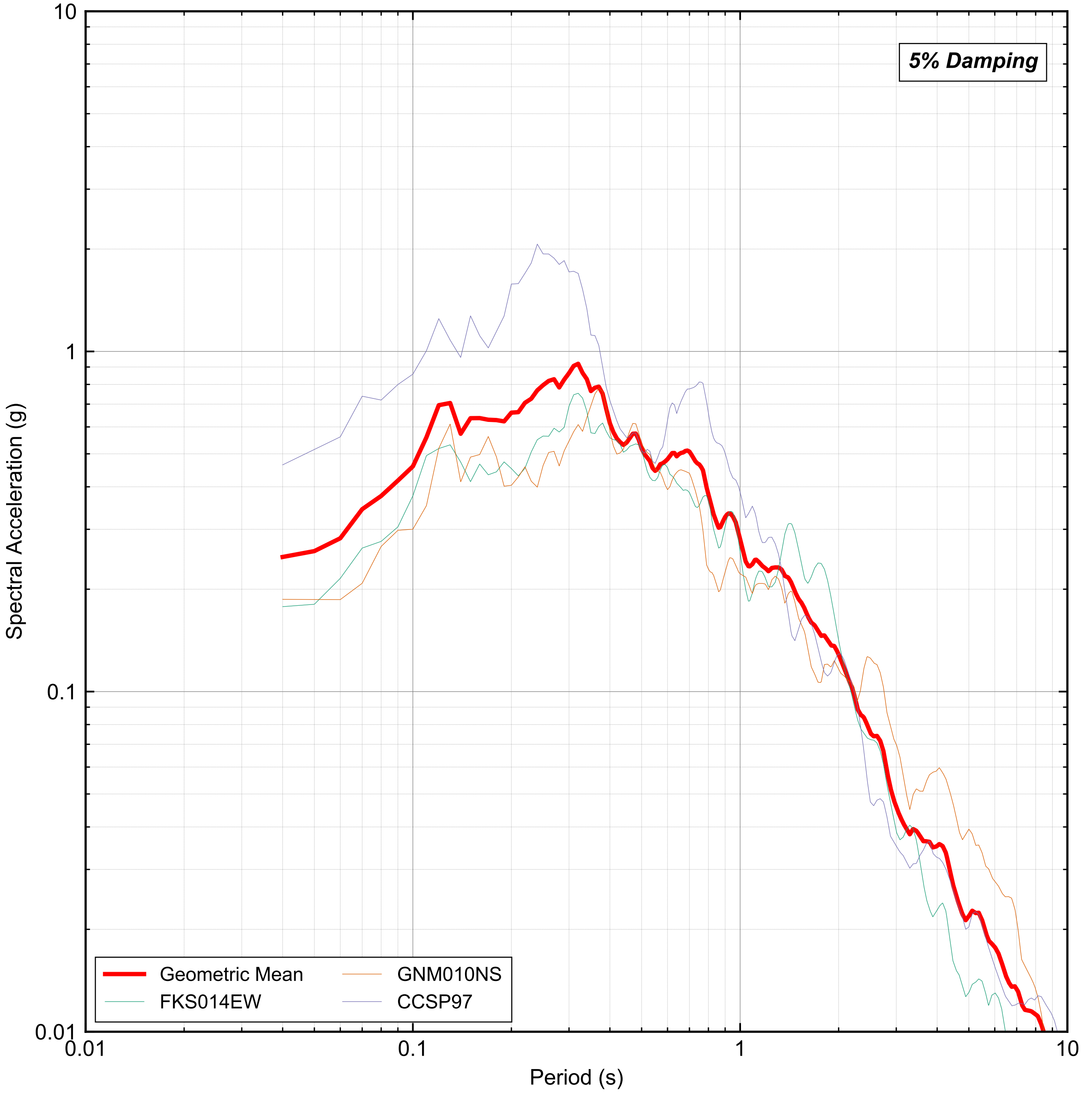

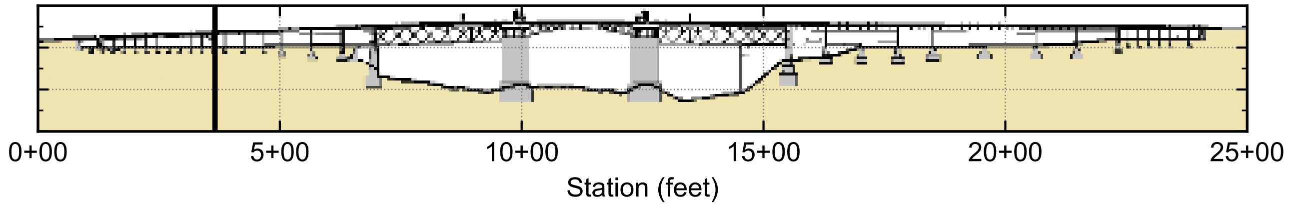

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 13

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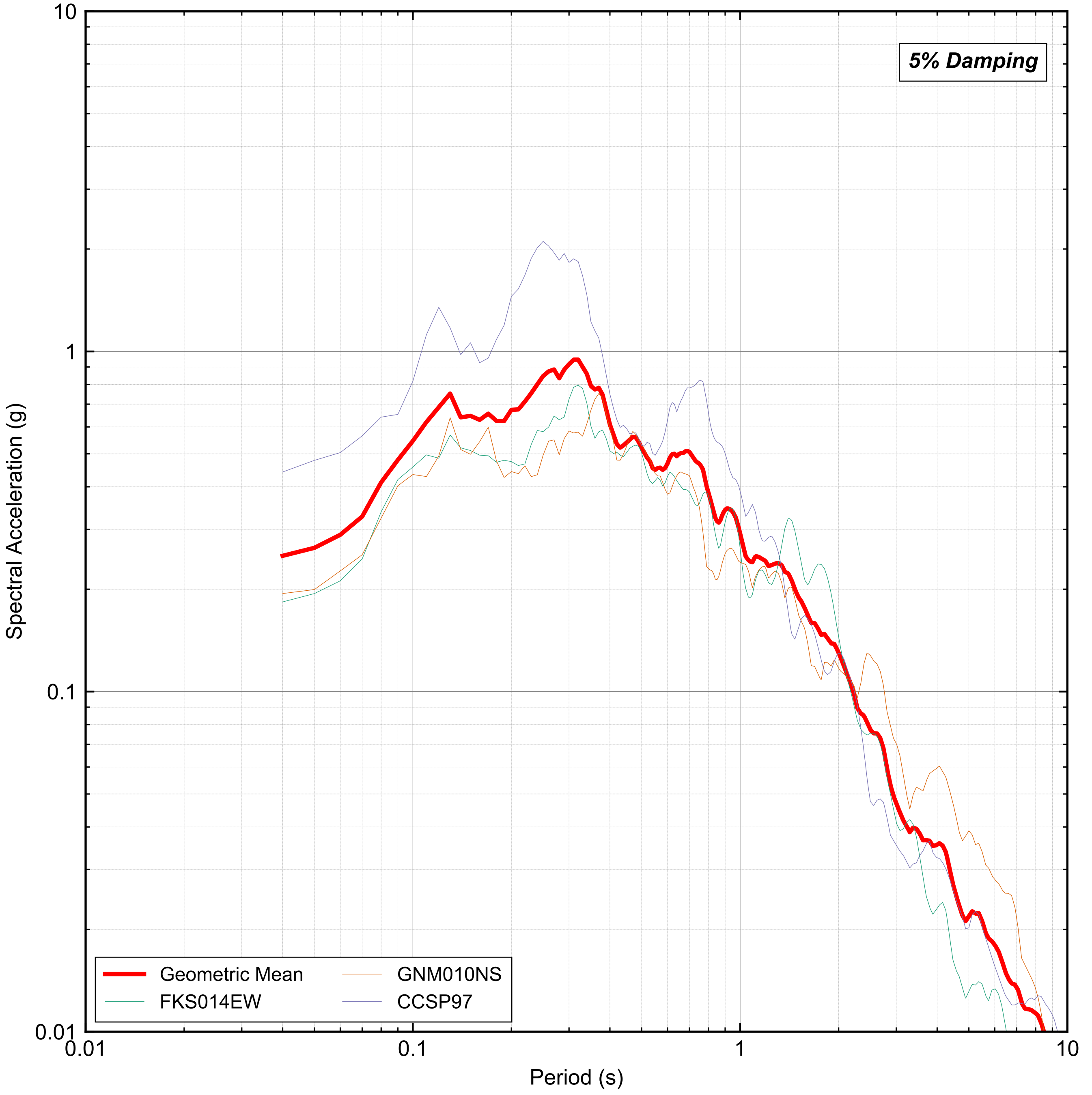
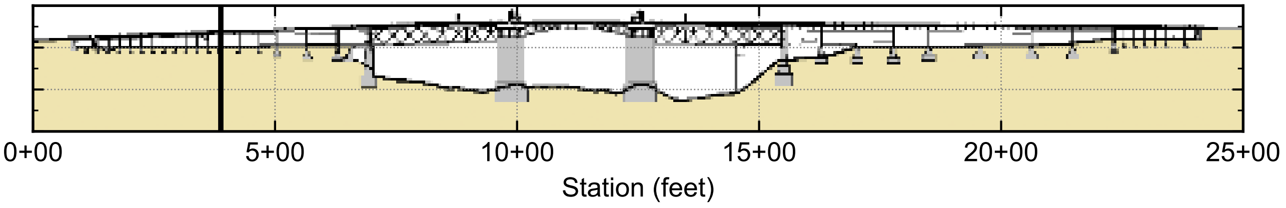
FIG. E-89





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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**GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 14**

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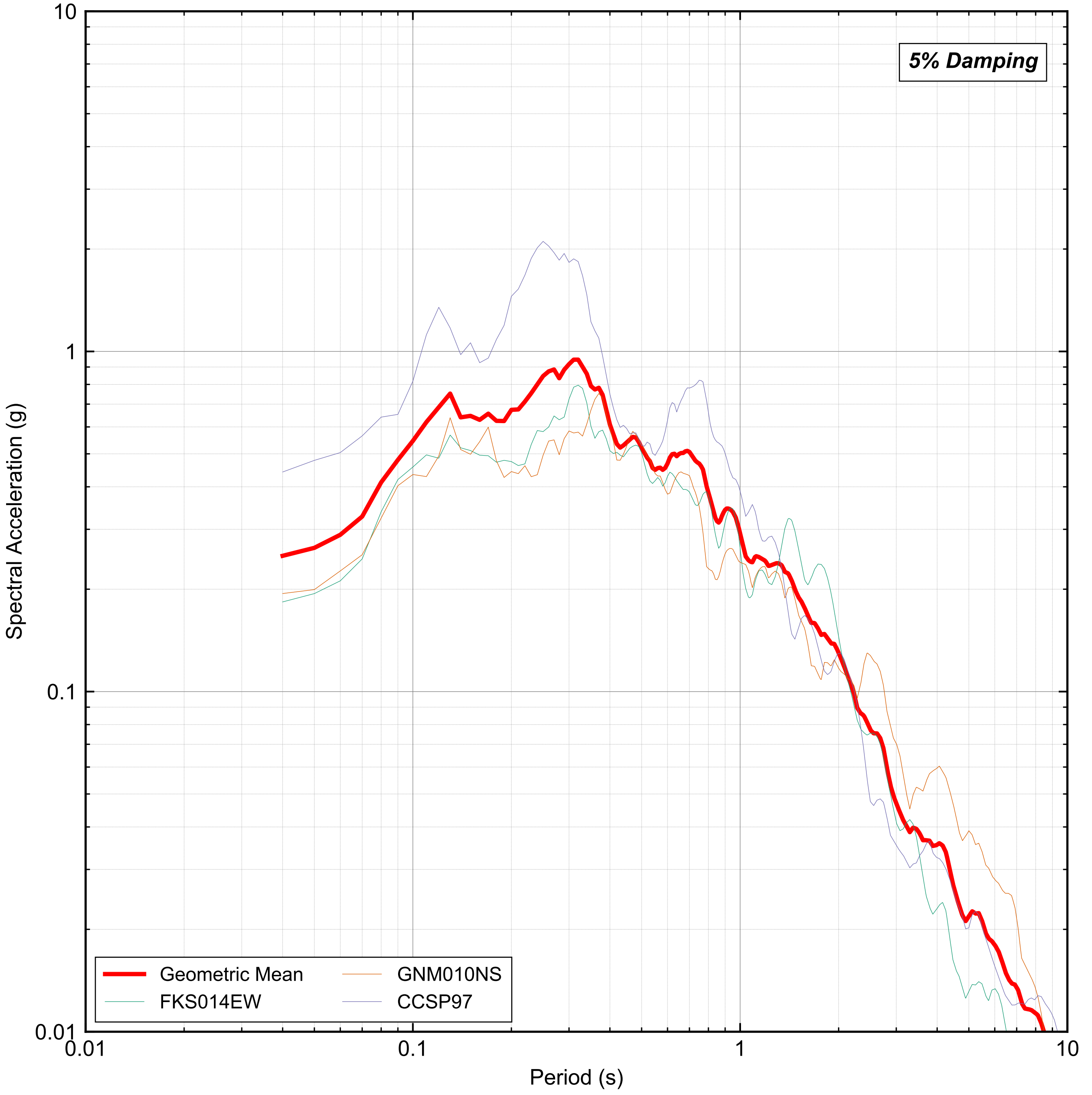
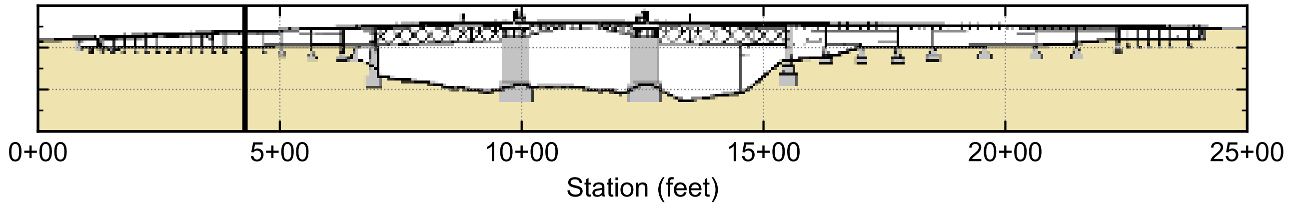
FIG. E-90

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 15

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FIG. E-91

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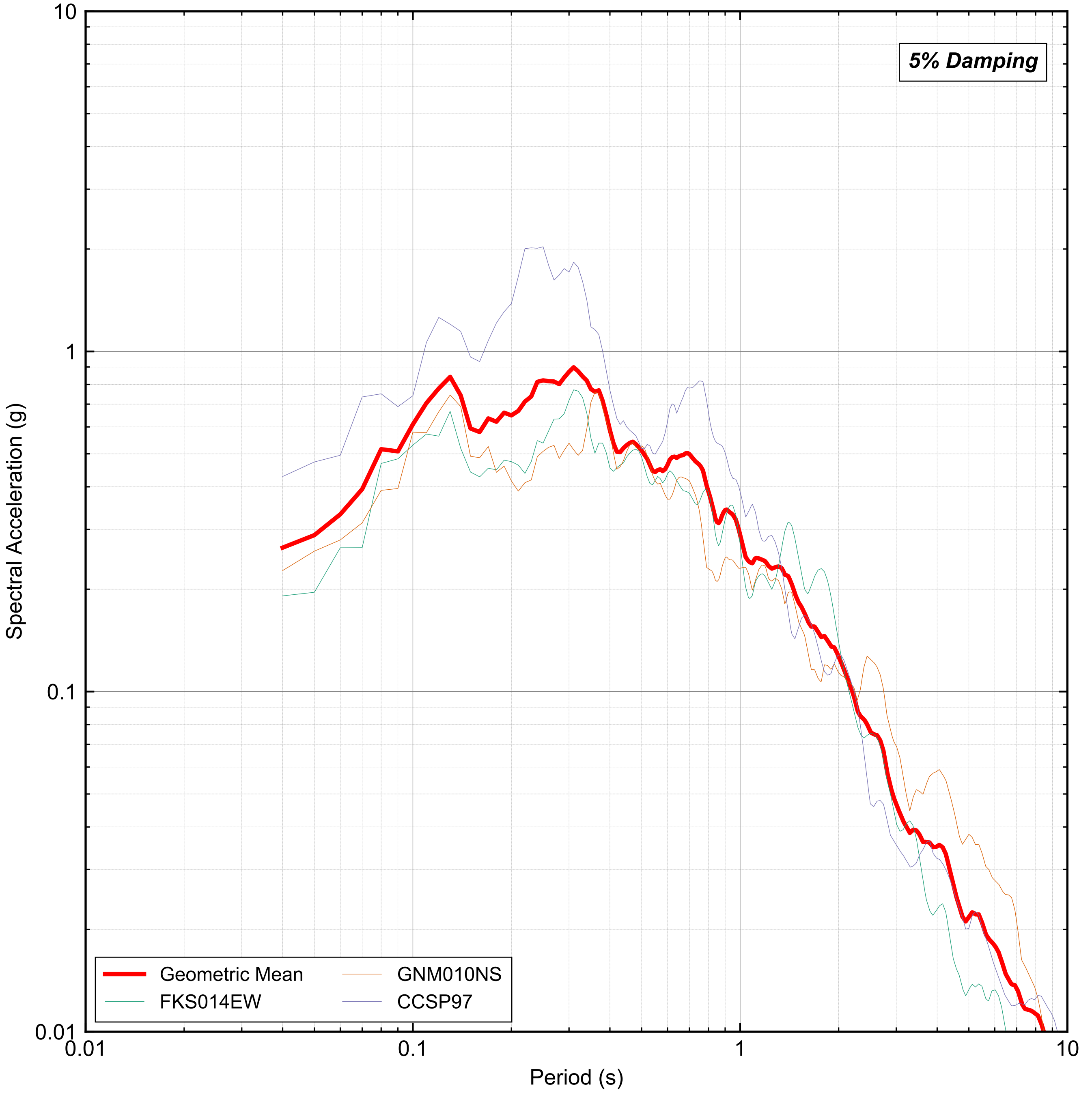

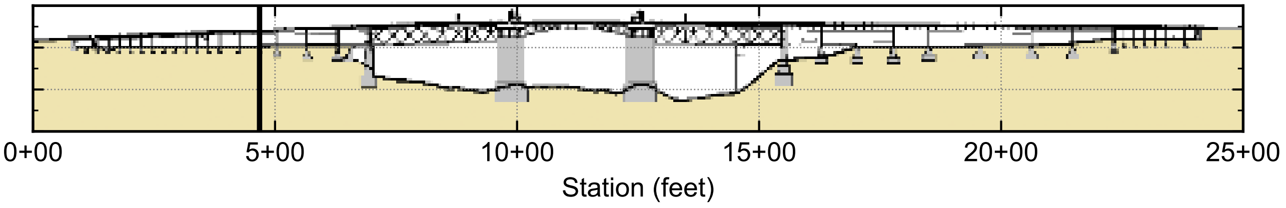

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 16

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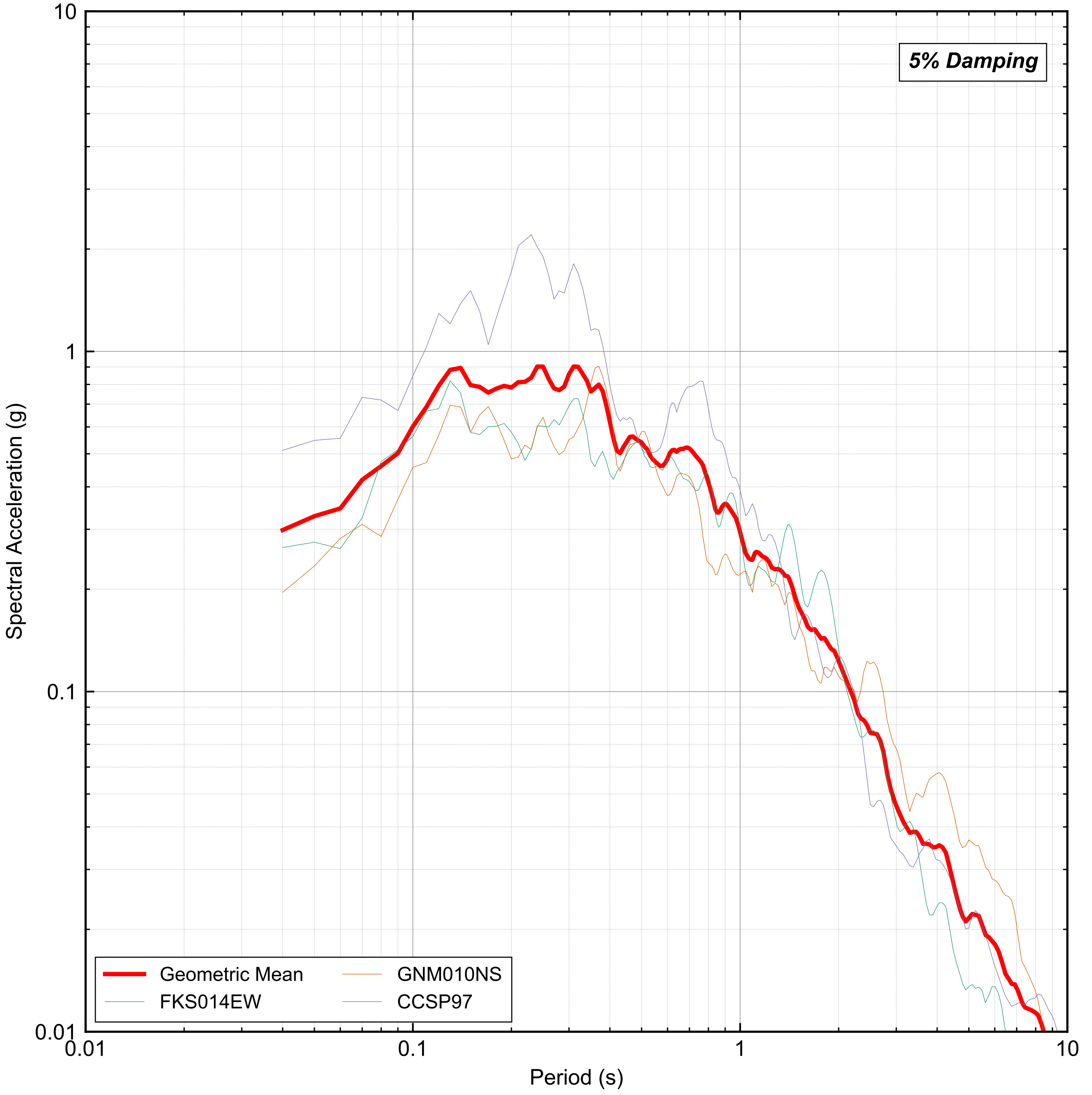
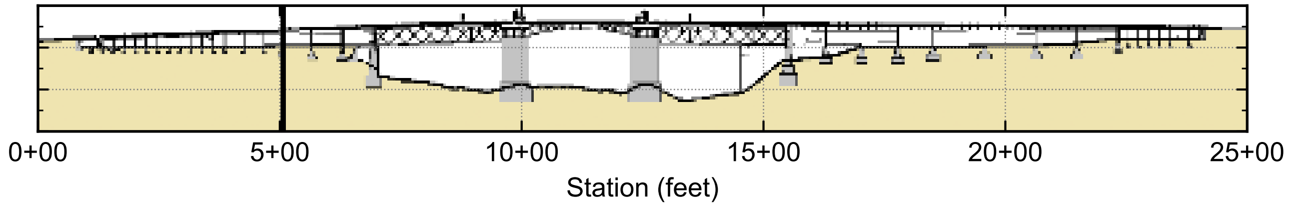
FIG. E-92





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 17

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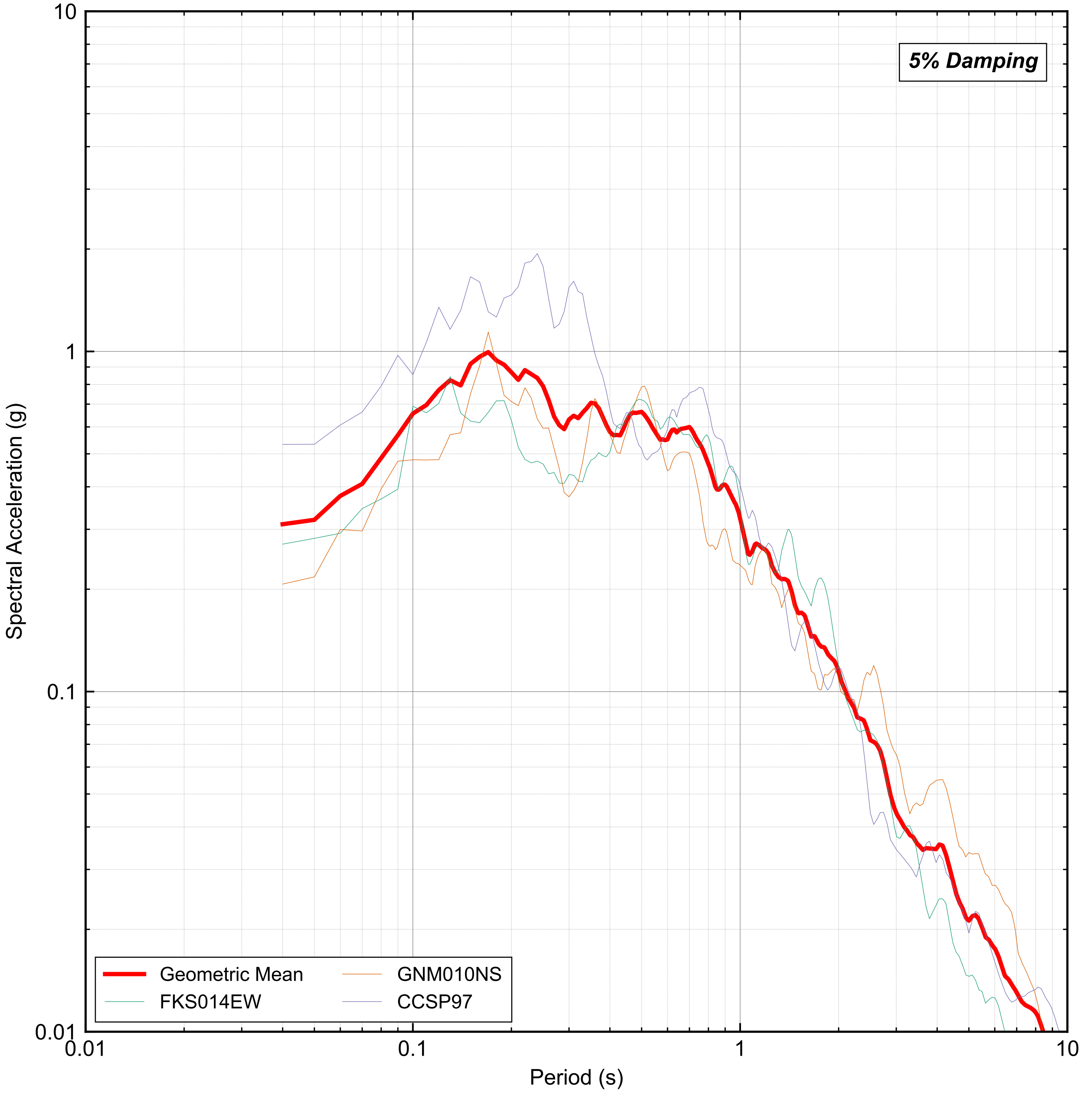
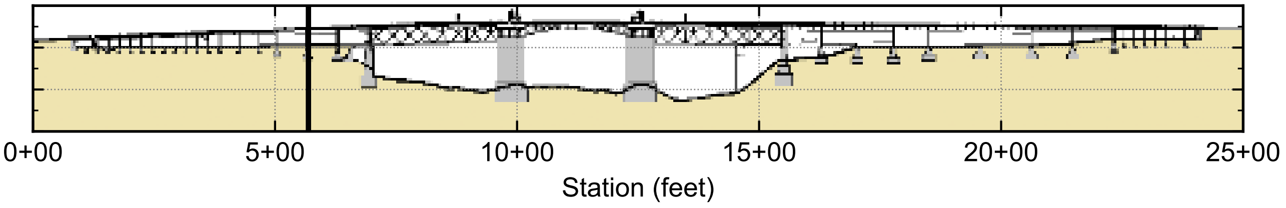
FIG. E-93

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 18

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FIG. E-94

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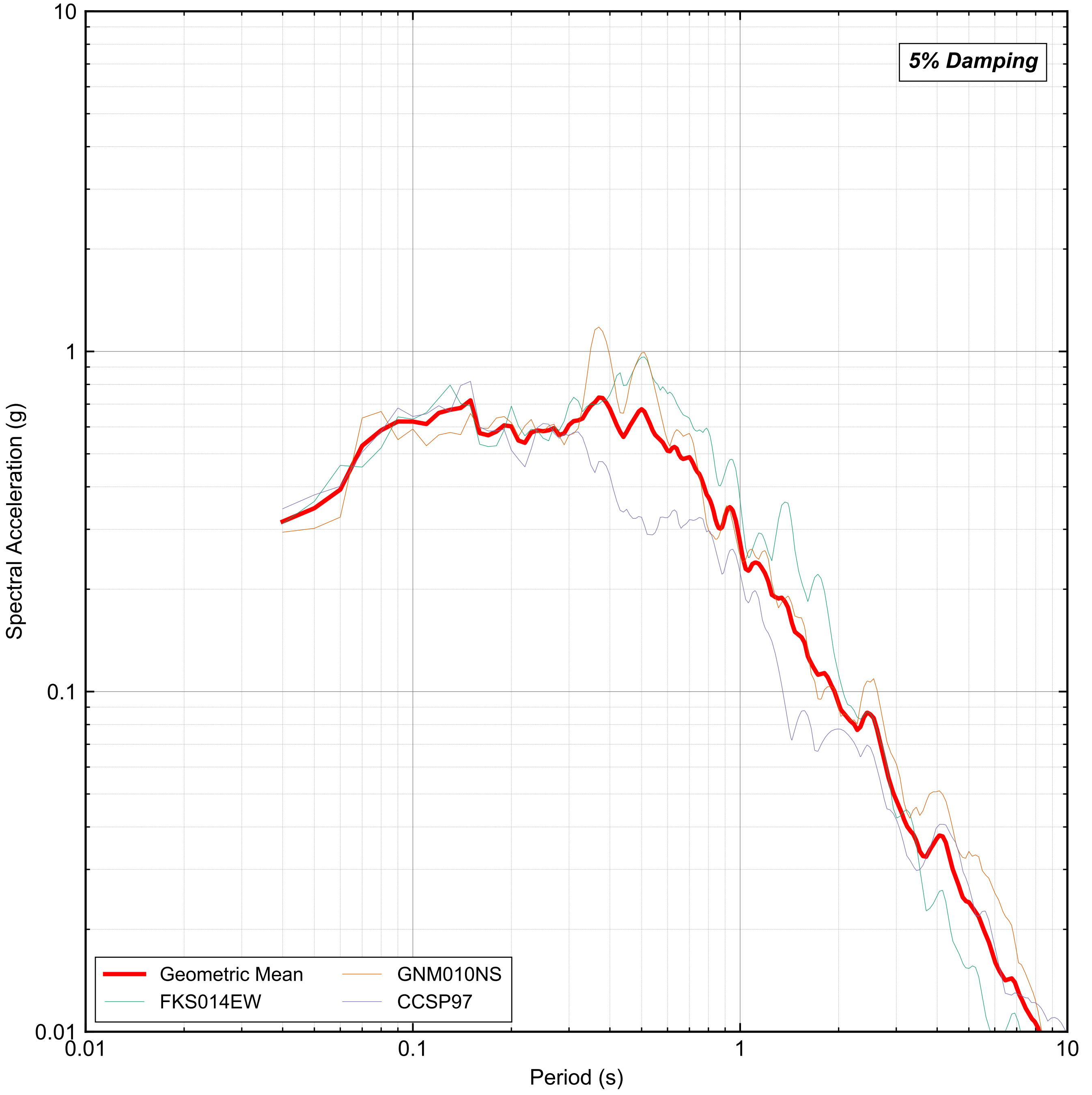
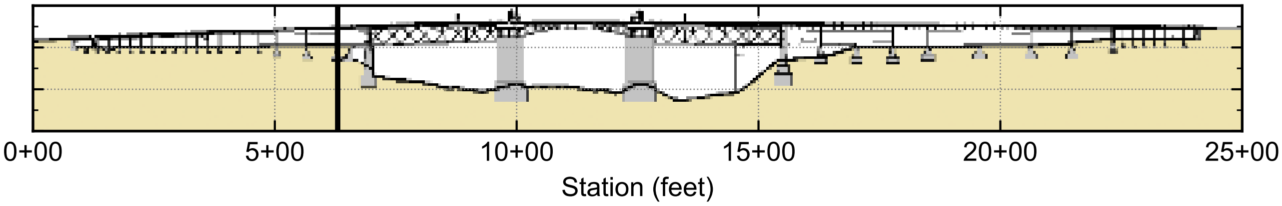
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 19

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FIG. E-95

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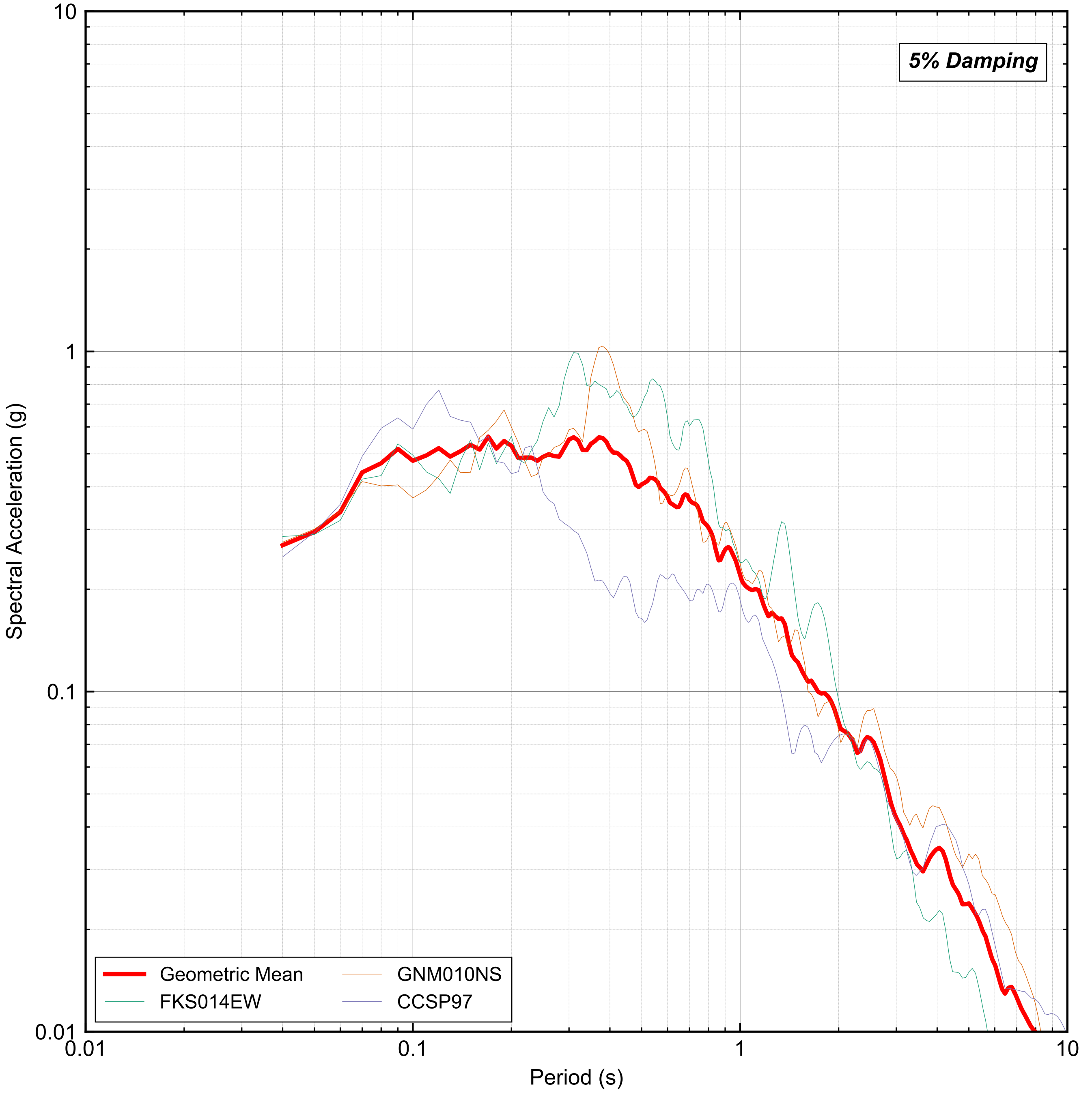

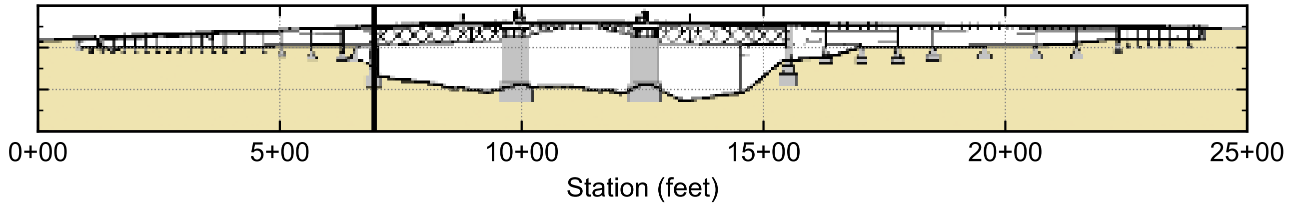

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CSZ 0.2S) EXISTING PIER 1

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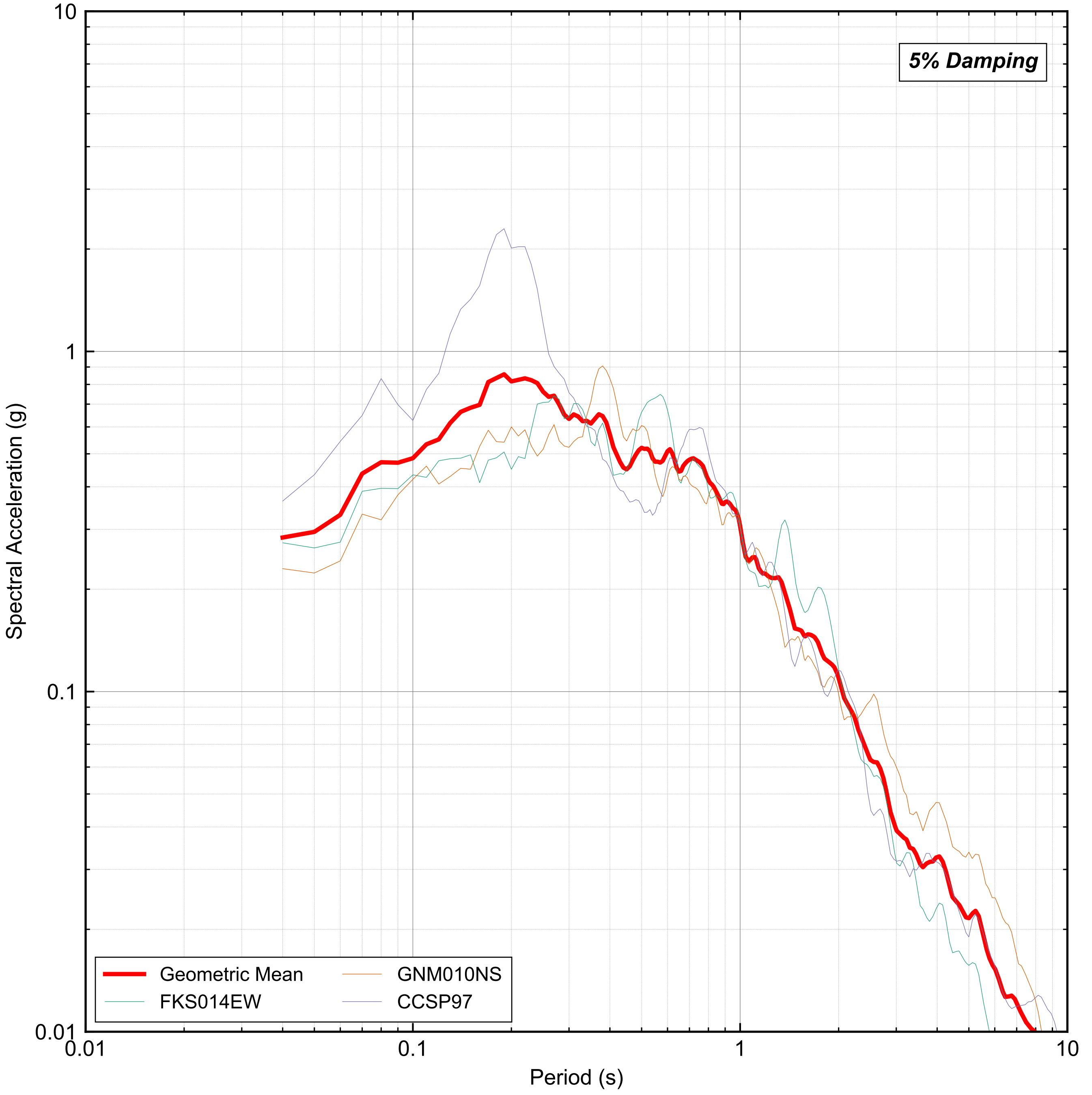
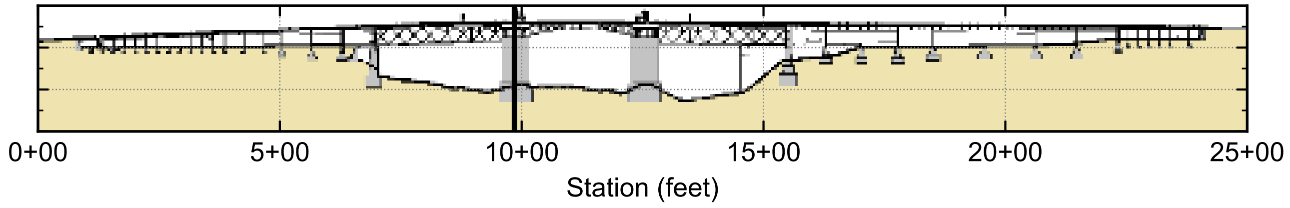
FIG. E-96

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING PIER 2

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FIG. E-97

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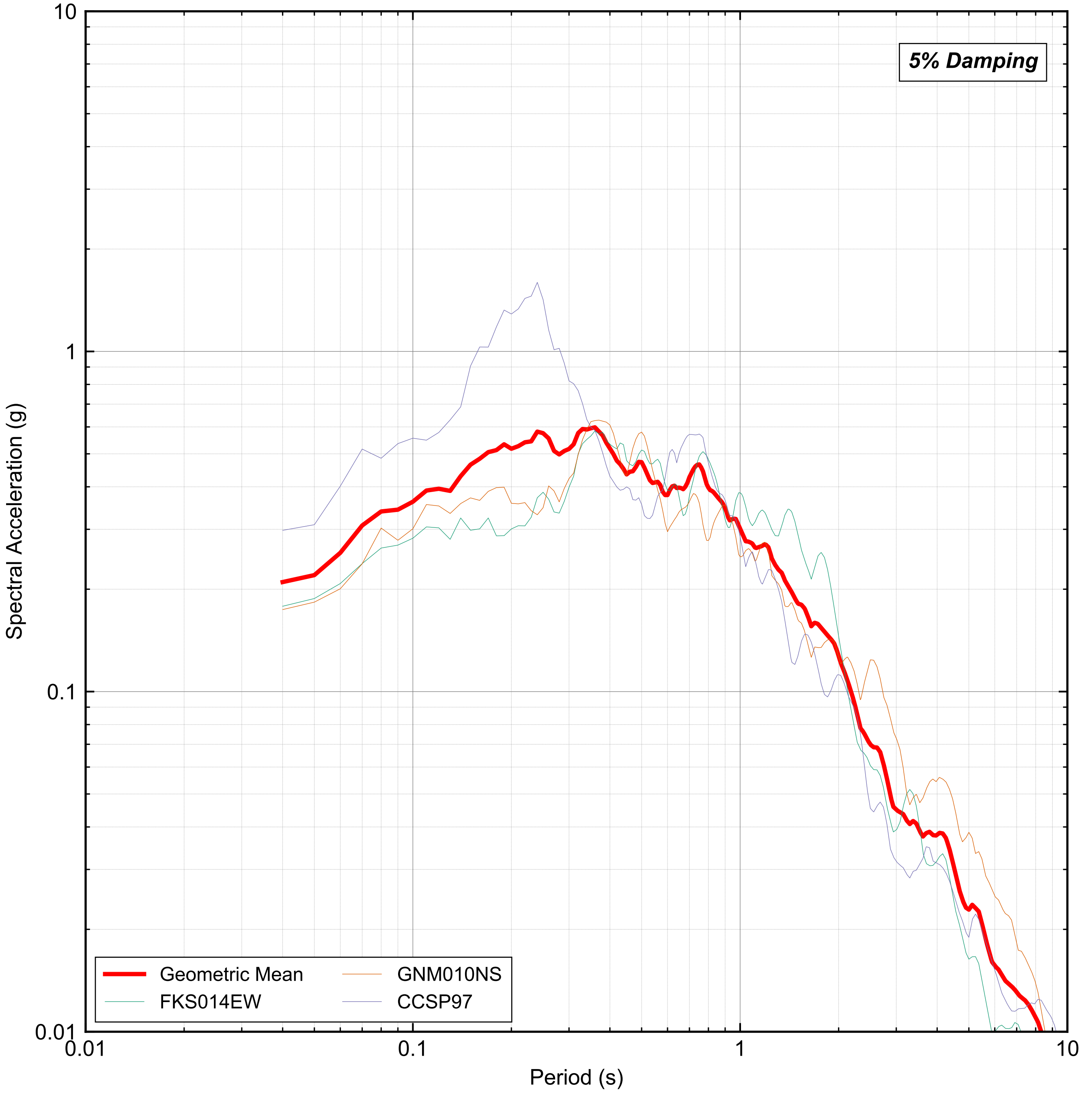
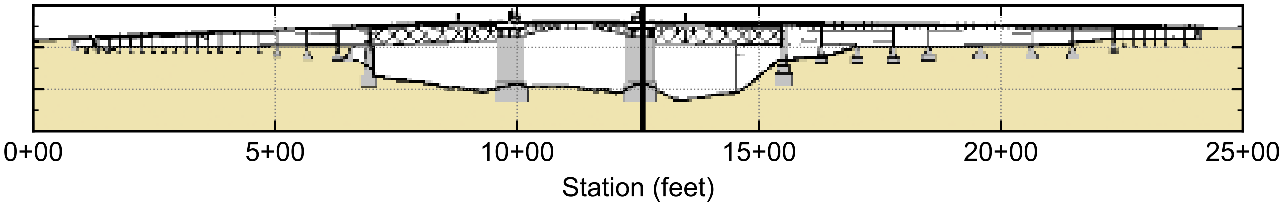
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING PIER 3

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FIG. E-98

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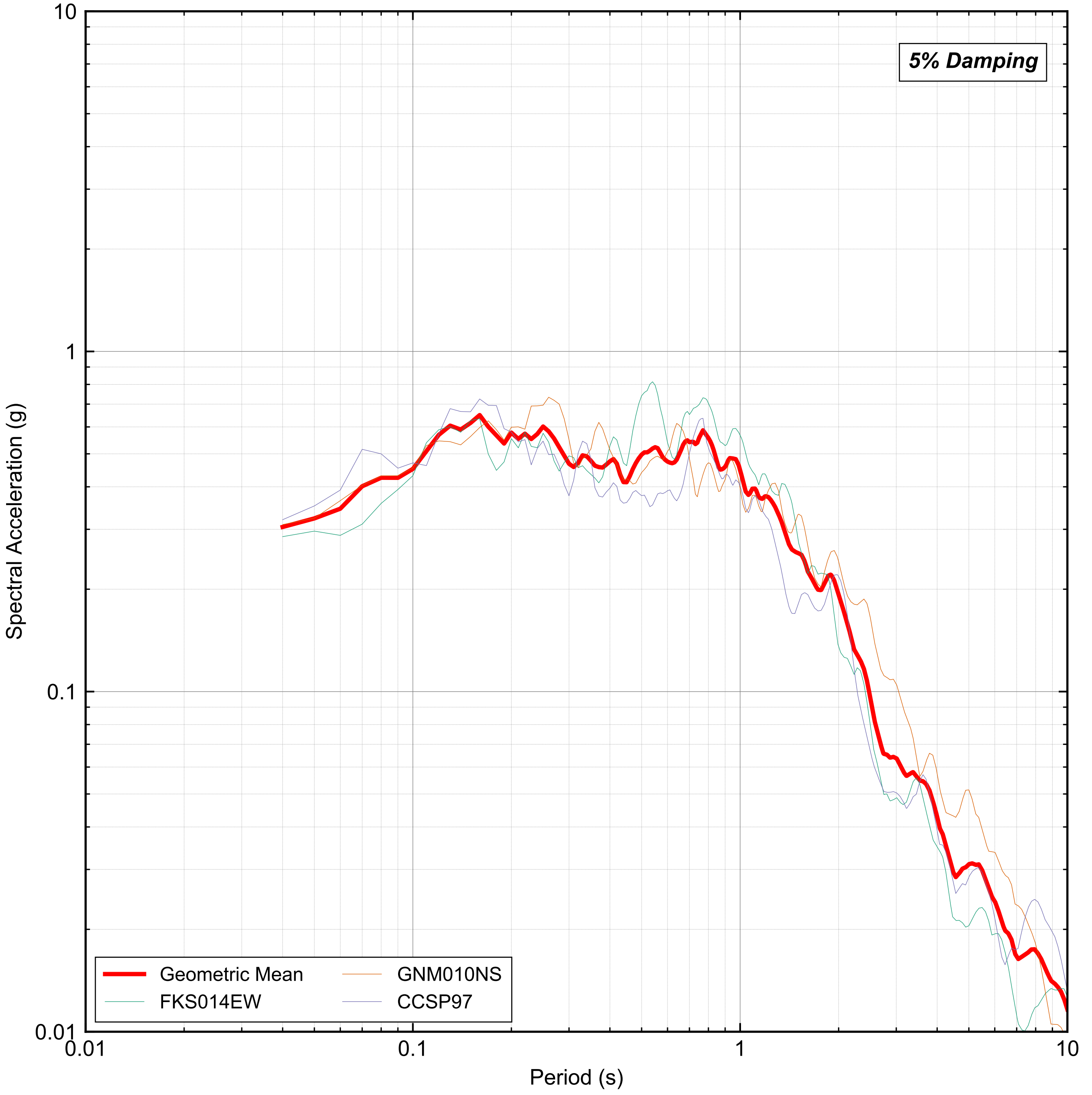
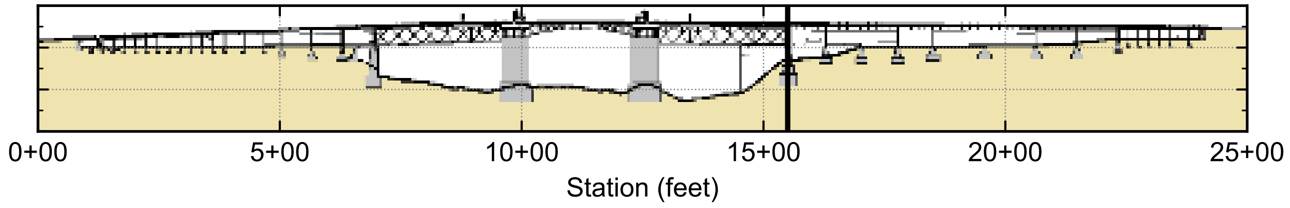
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING PIER 4

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FIG. E-99

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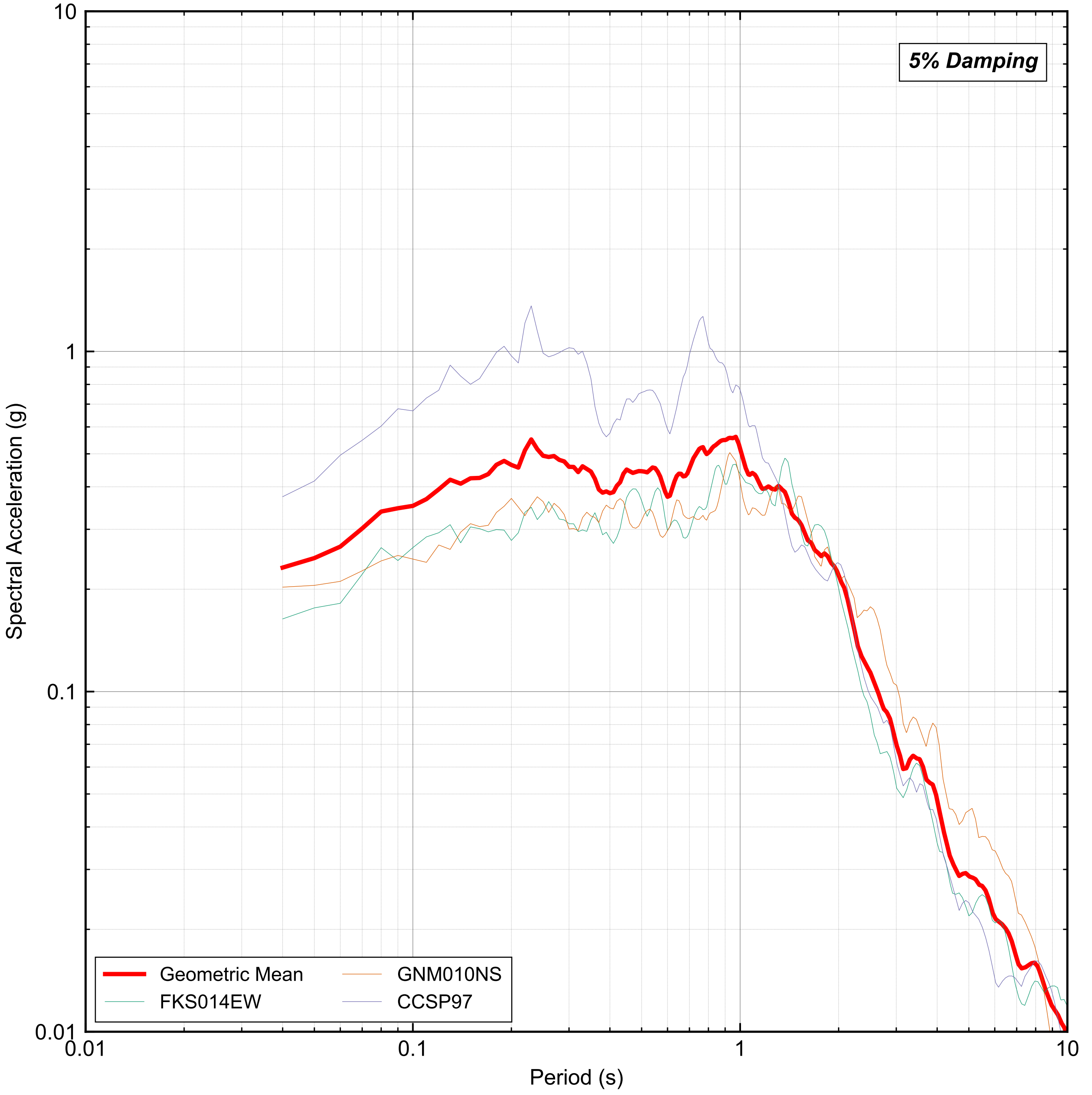
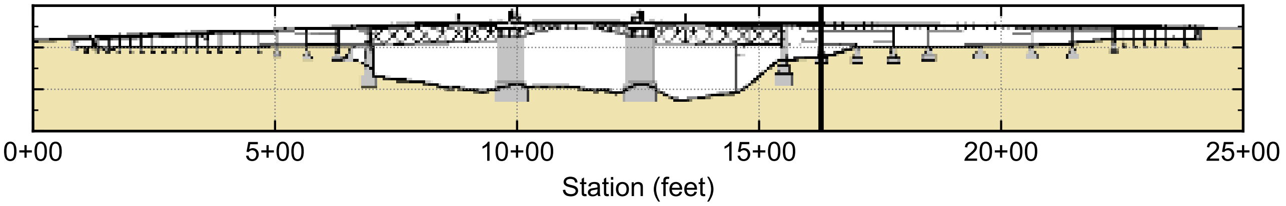
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 21

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FIG. E-100

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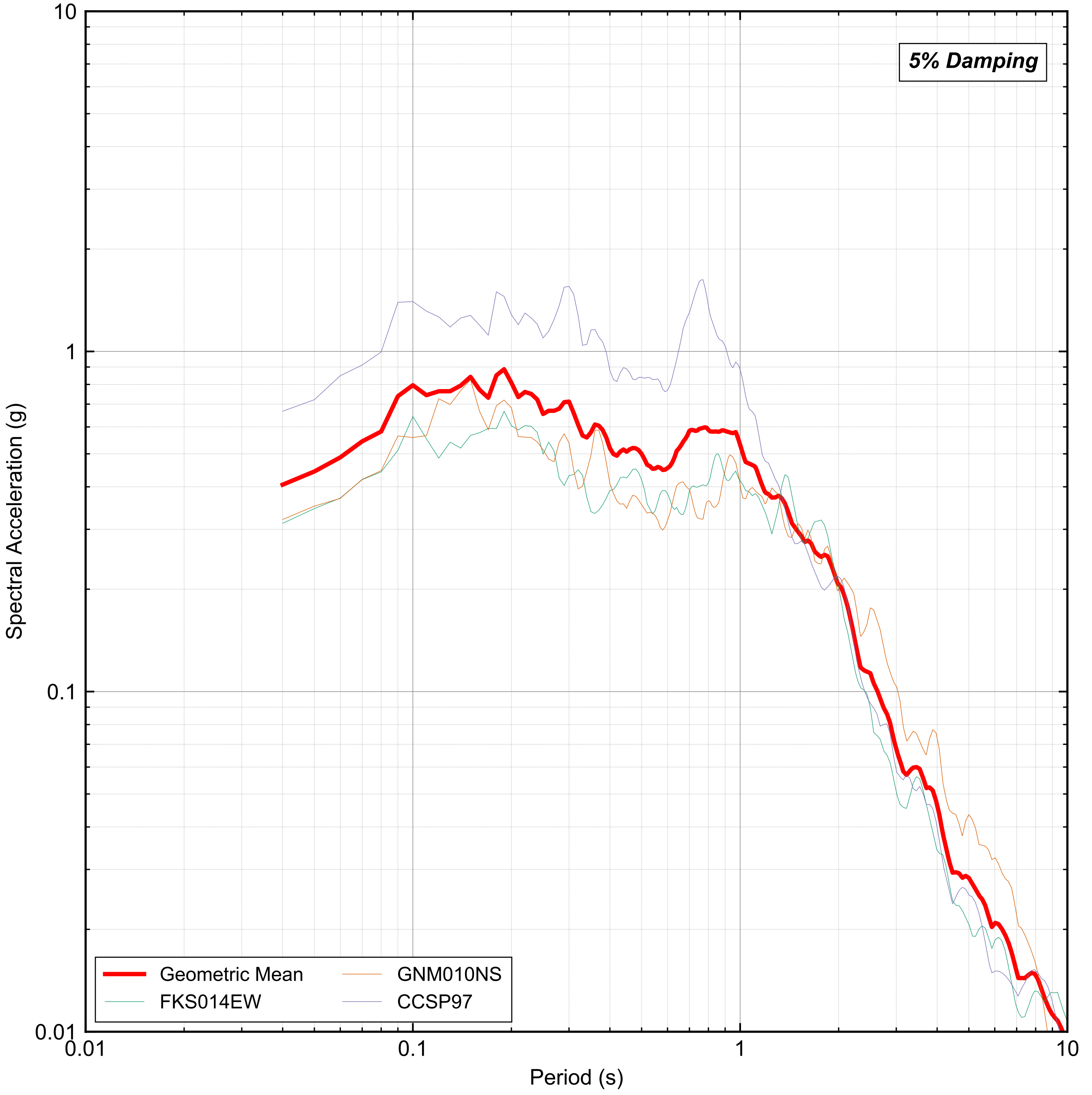

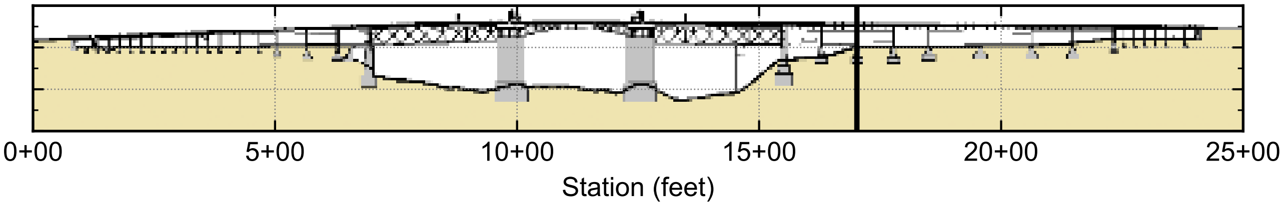

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 22

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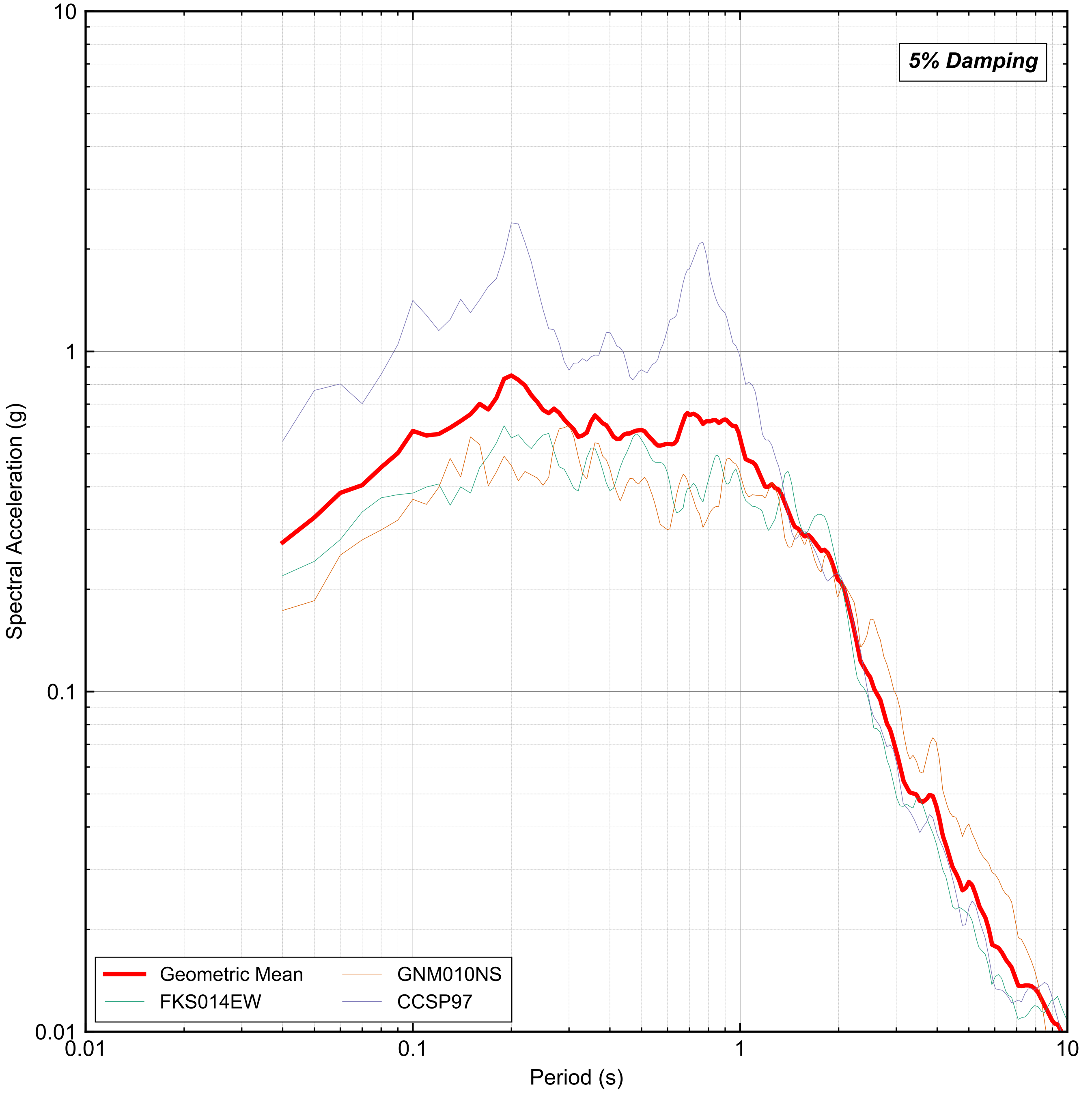
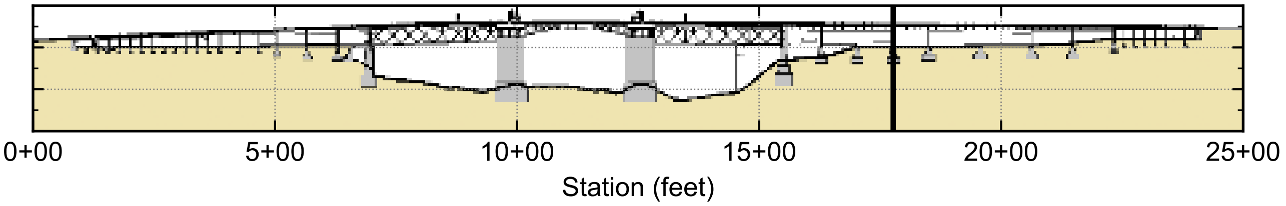
FIG. E-101





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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**GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 23**

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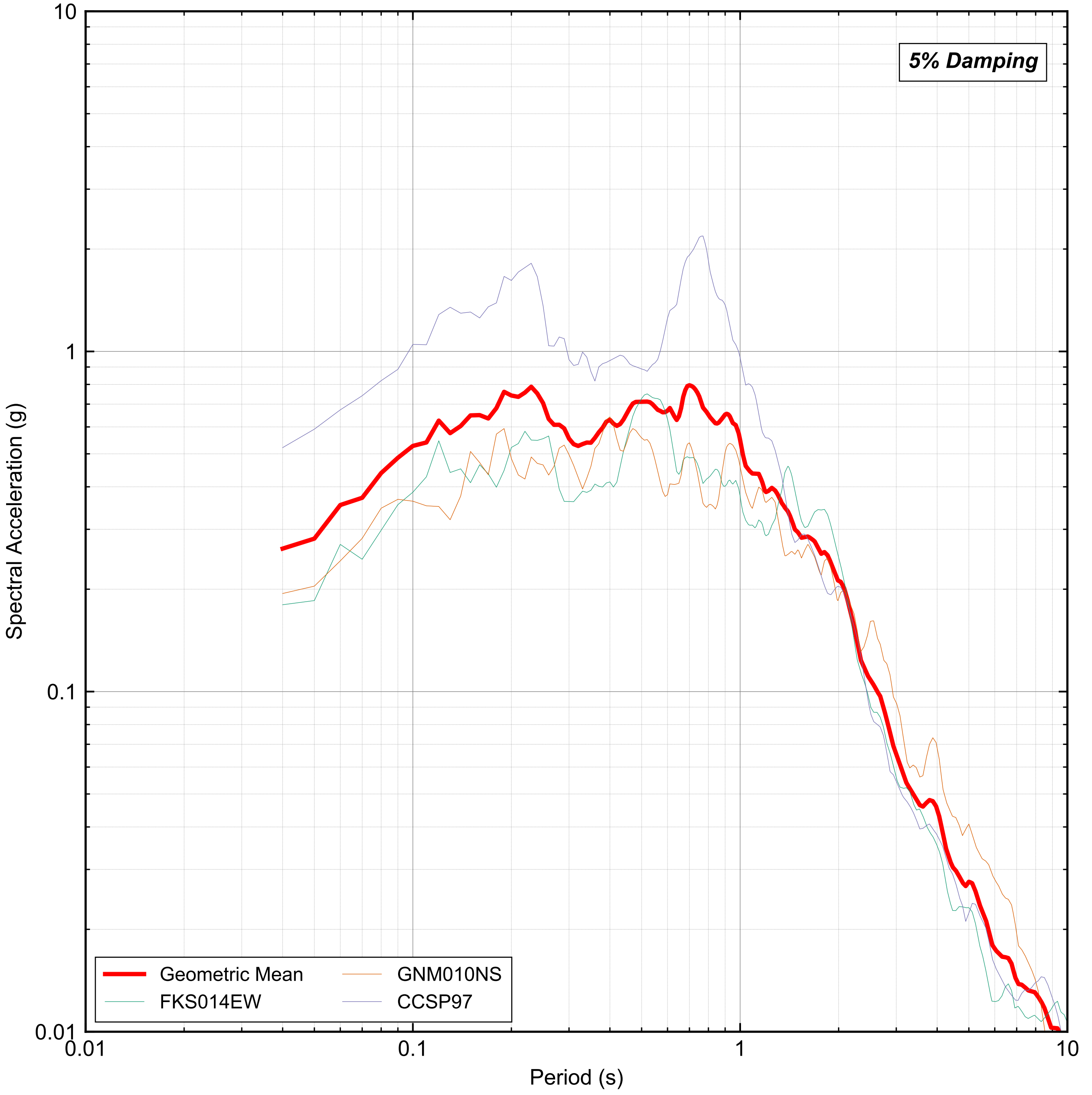
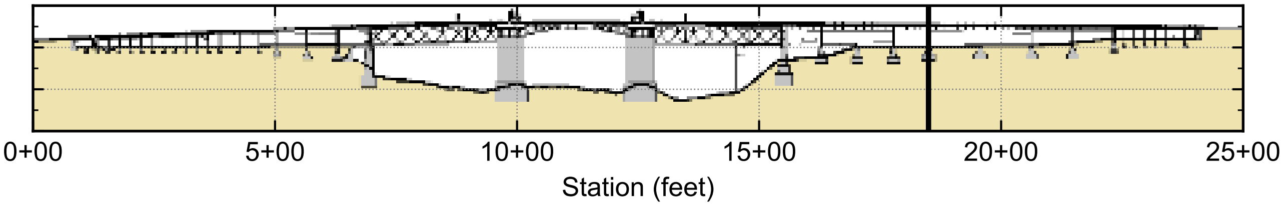
FIG. E-102

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 24

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FIG. E-103

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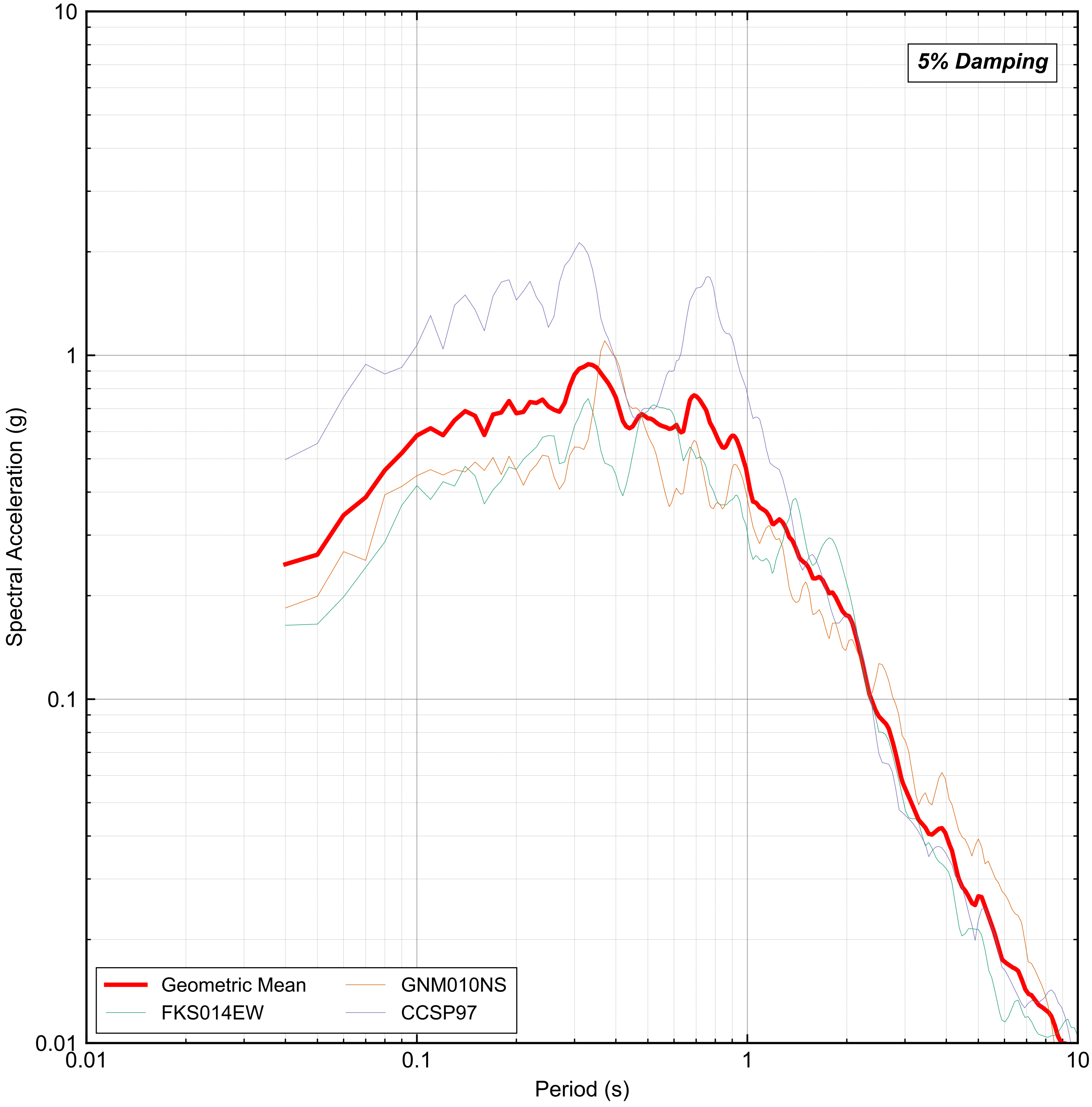

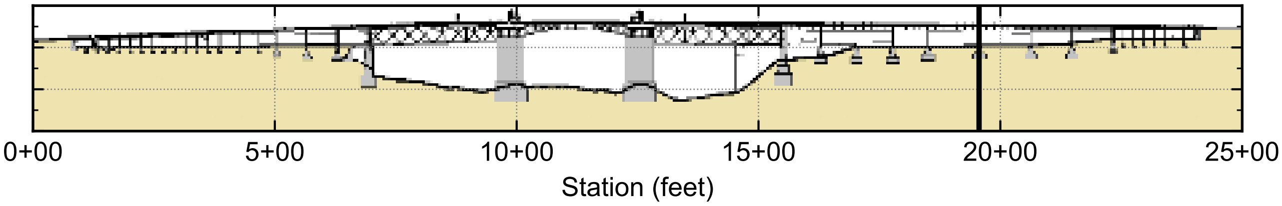

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 25

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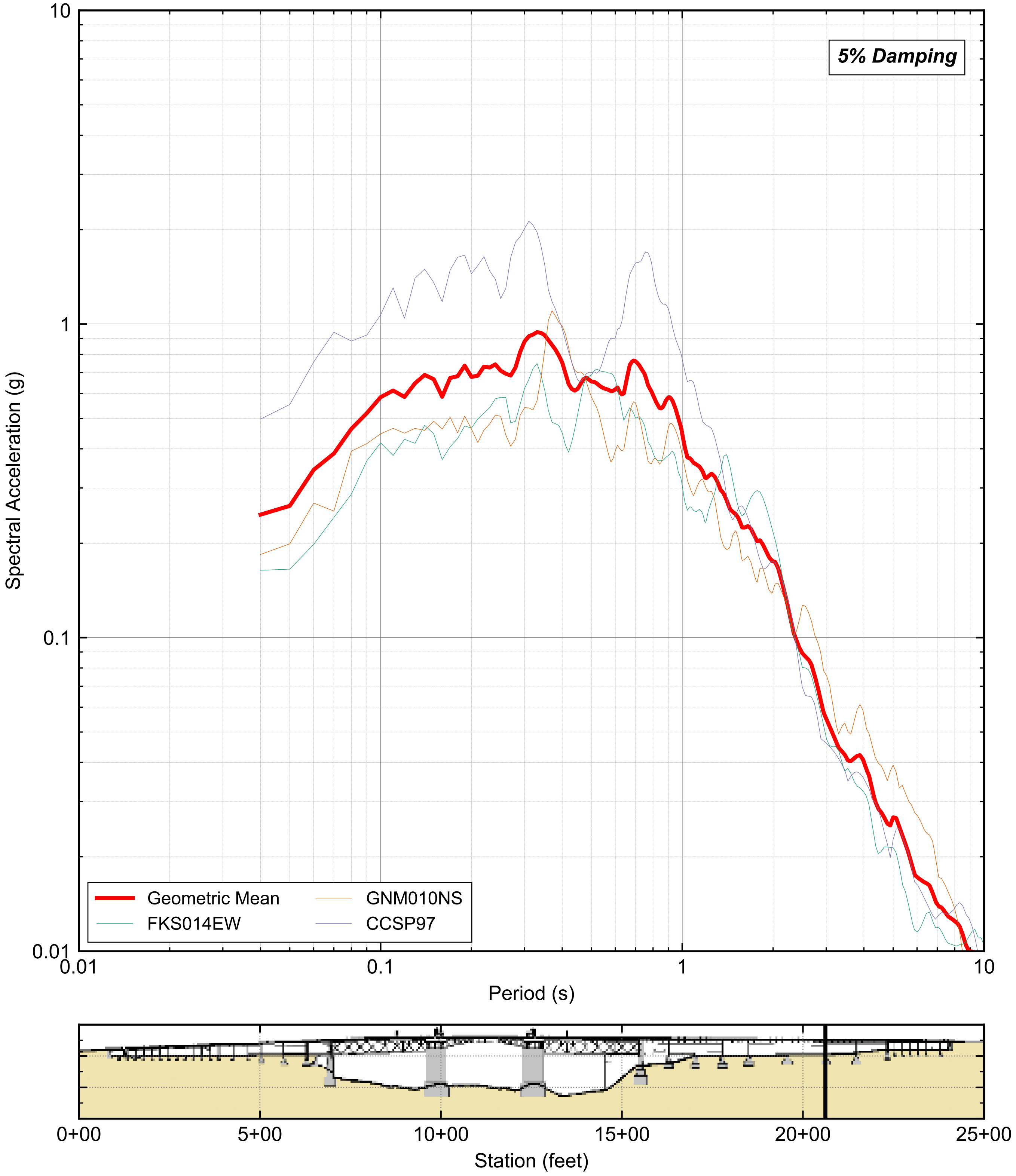
FIG. E-104

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 26

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FIG. E-105

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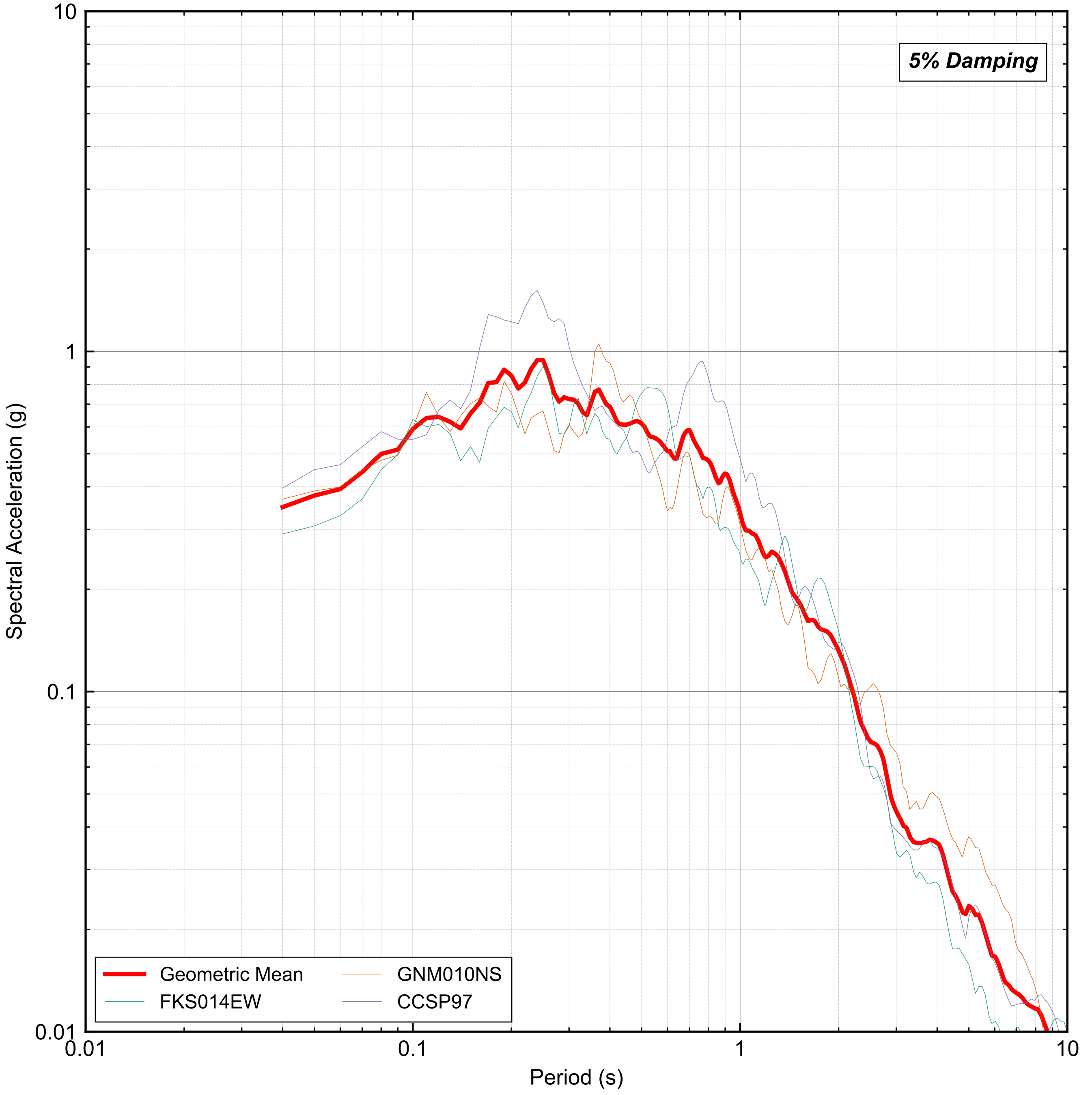

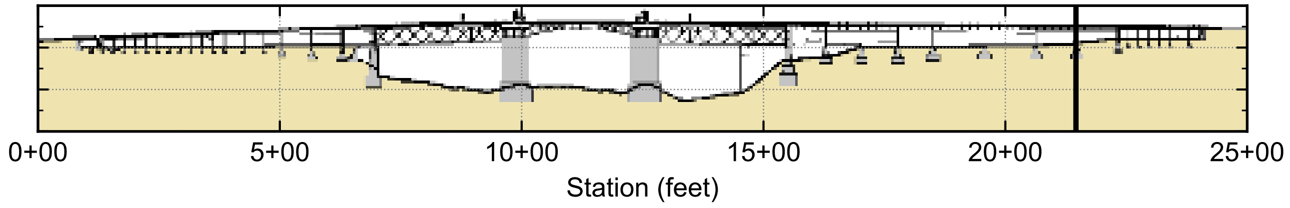

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 27

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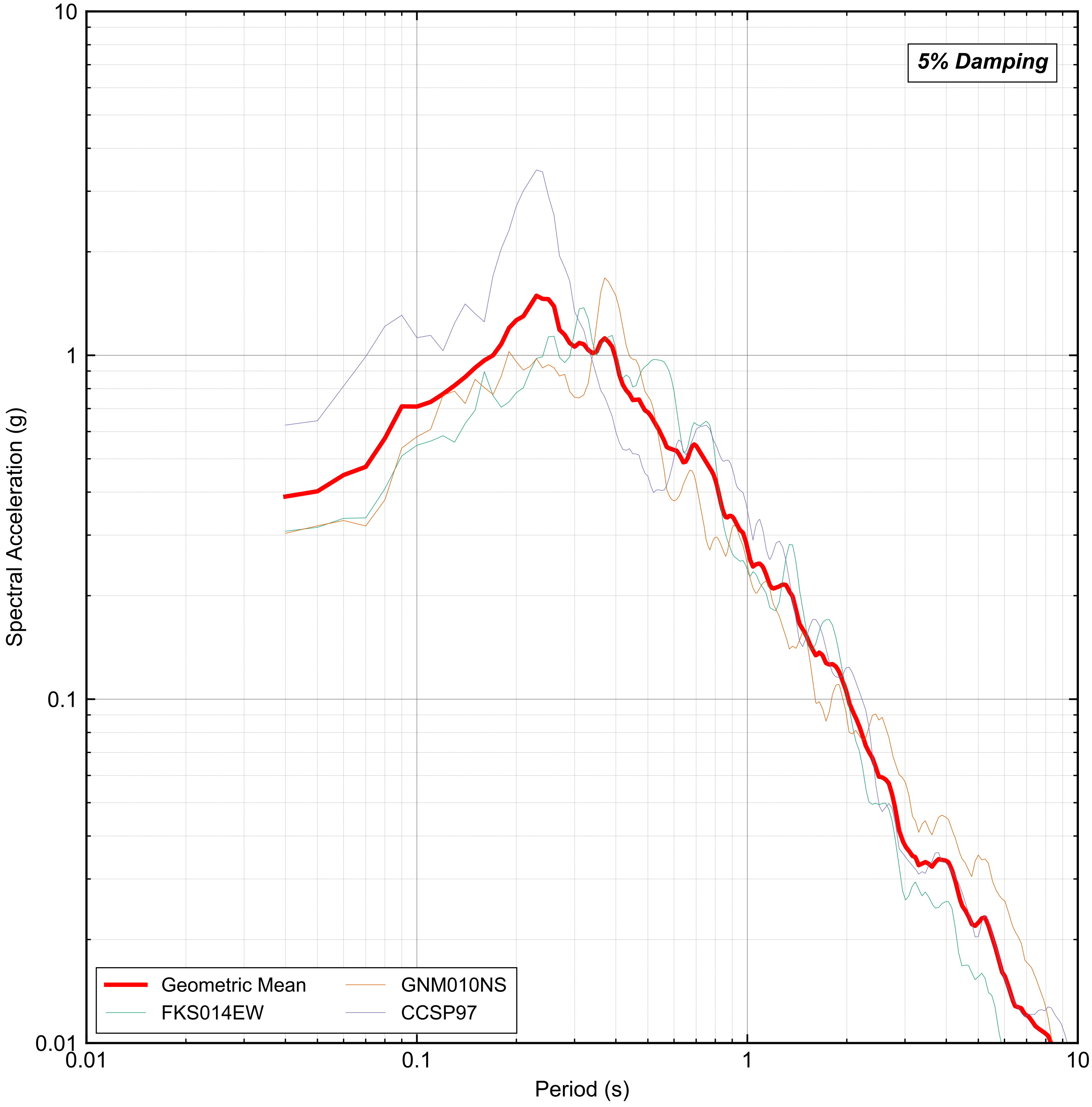
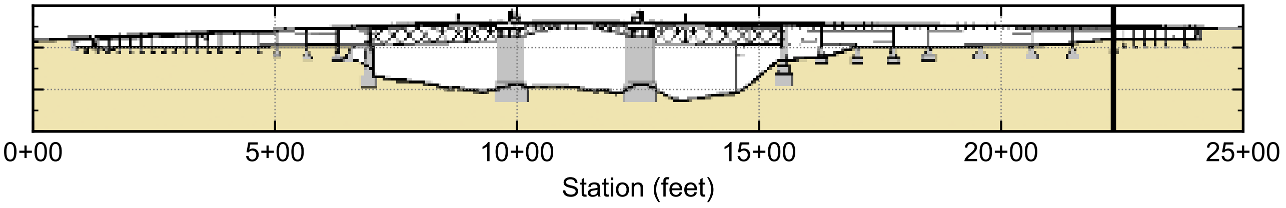
FIG. E-106





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 28

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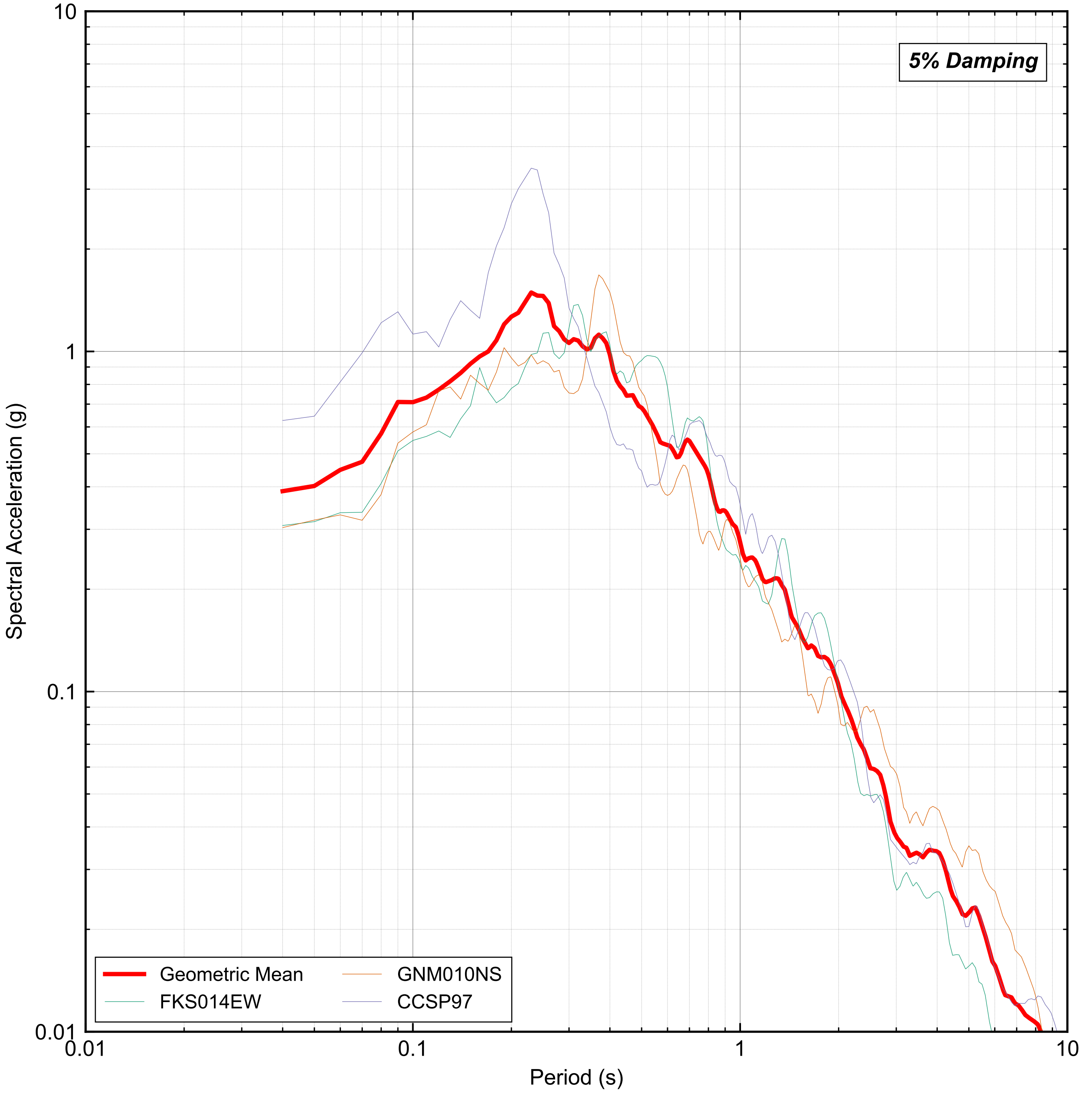
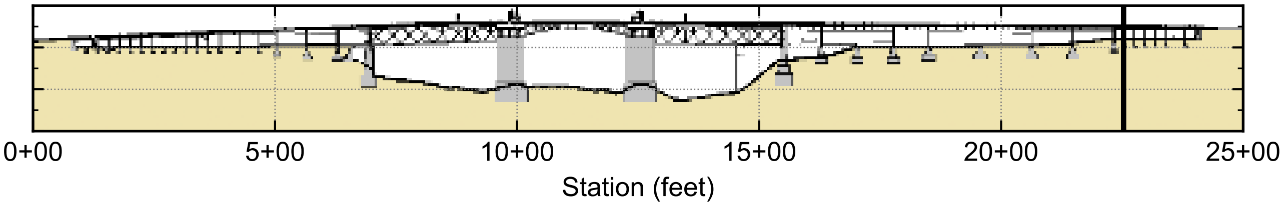
FIG. E-107





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 29

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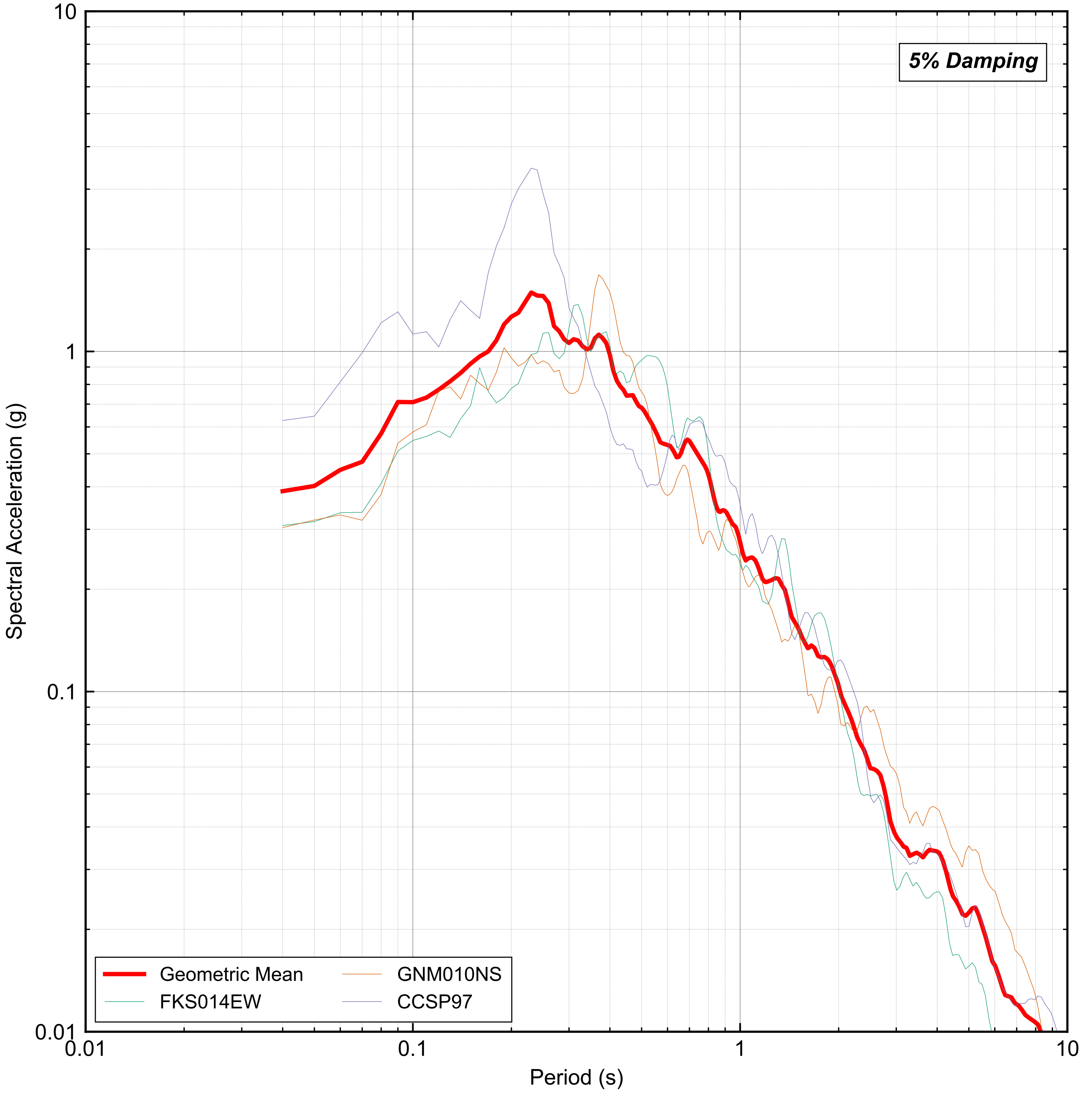
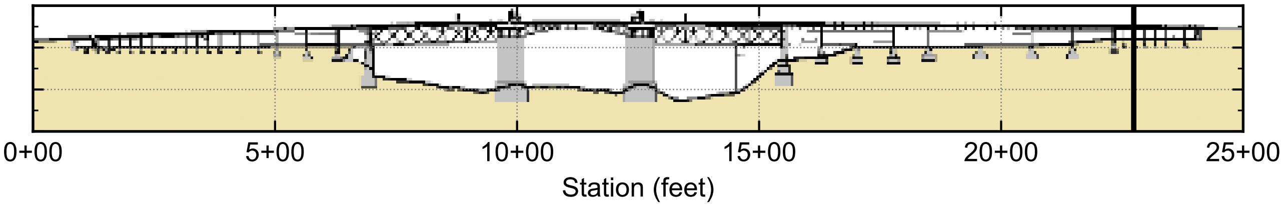
FIG. E-108





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 30

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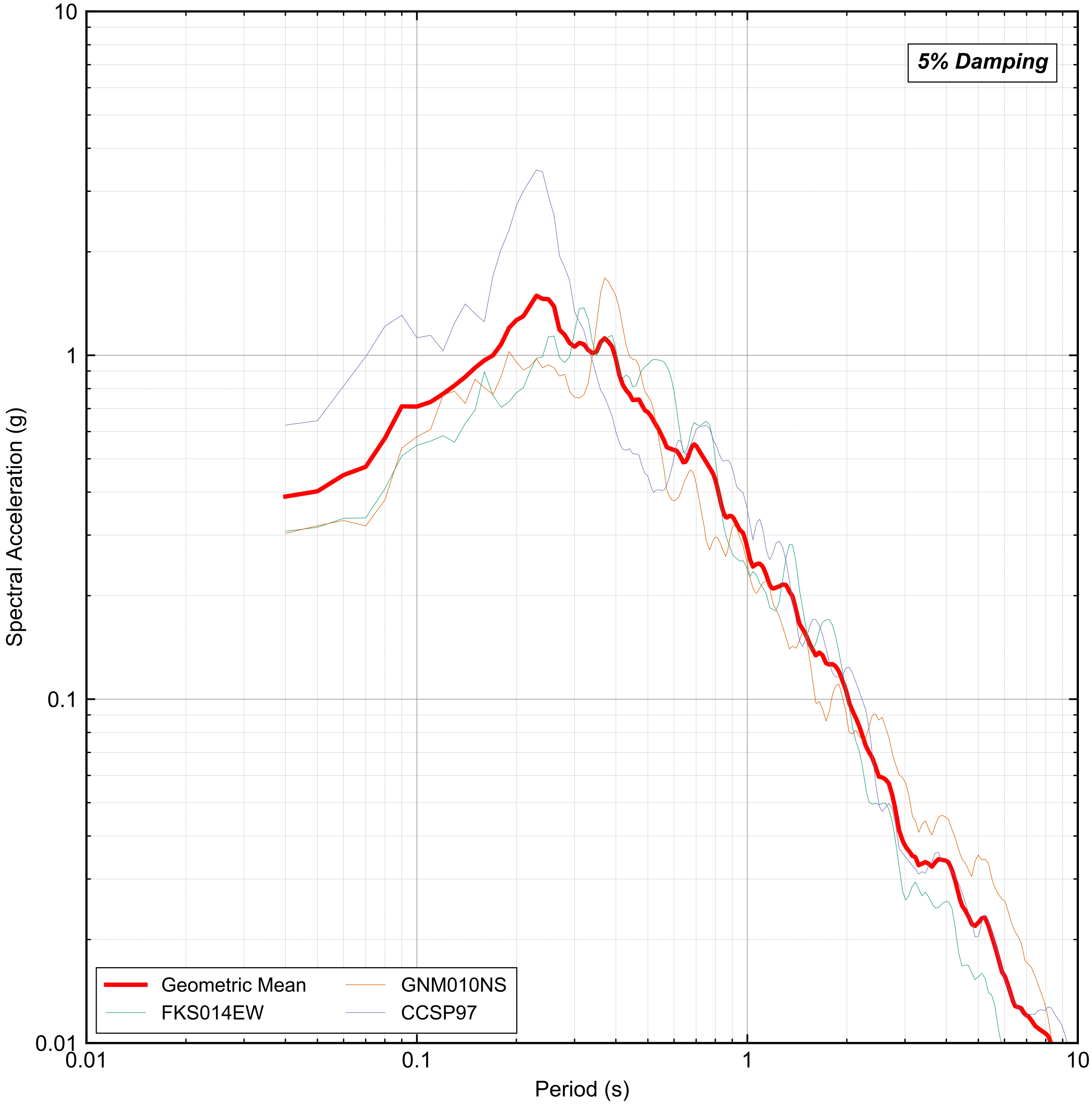
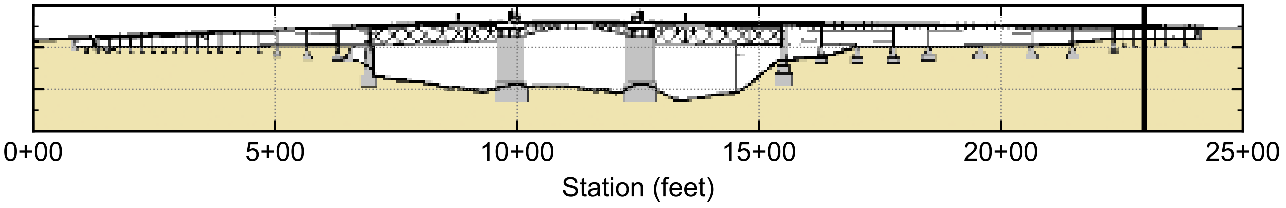
FIG. E-109





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 31

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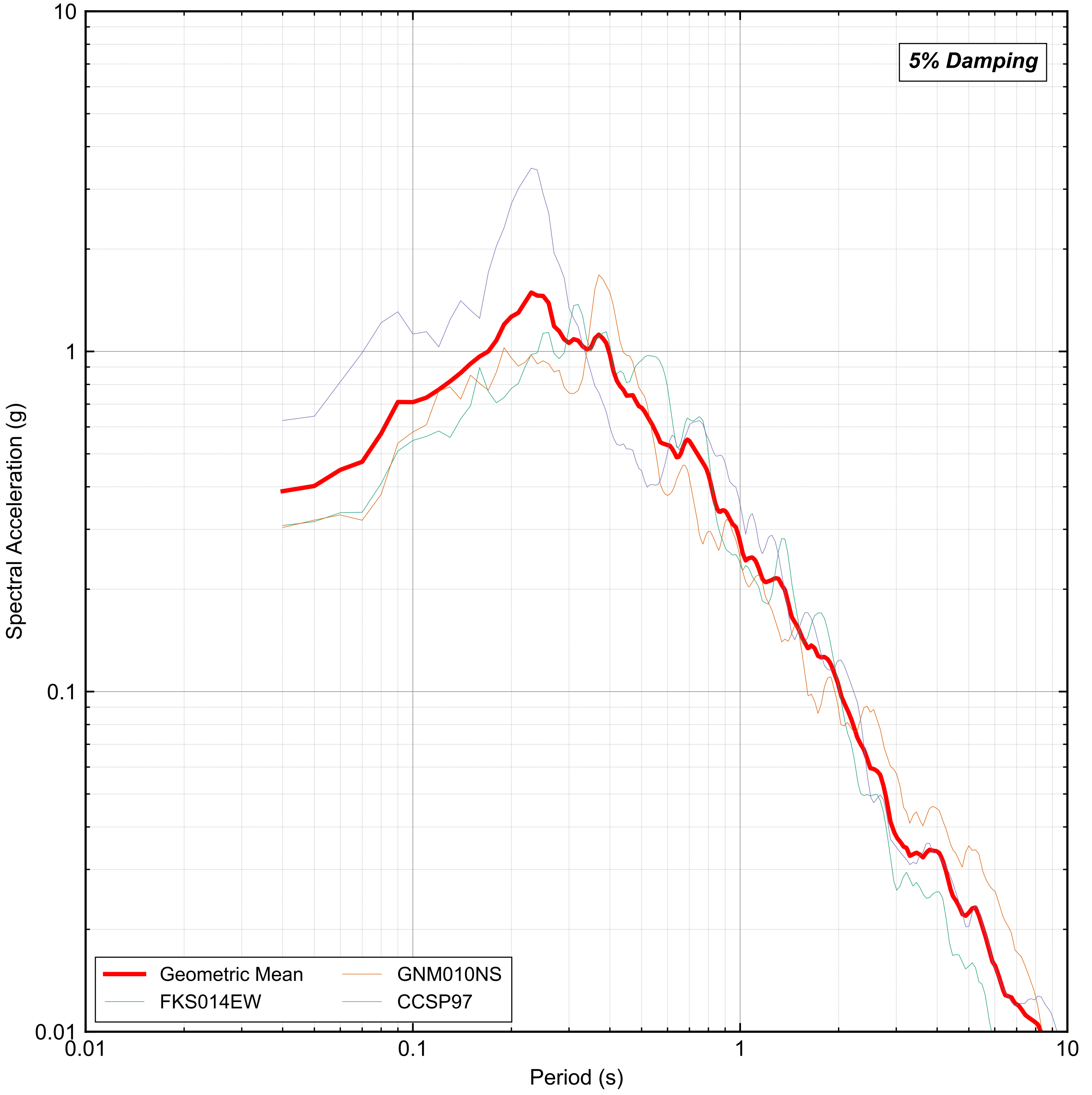
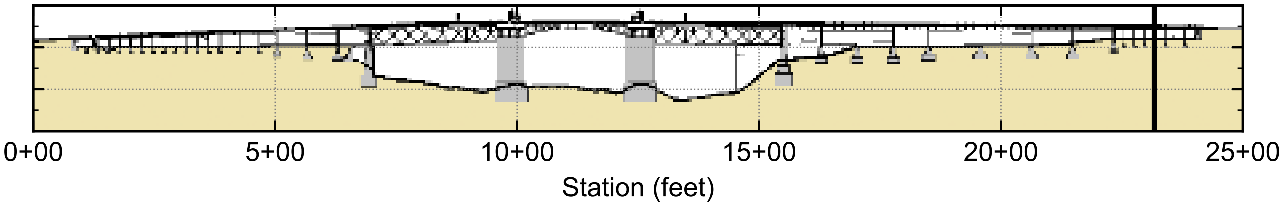
FIG. E-110

ver:0.8.5.post0.dev0+g414b689 by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CSZ 0.2S) EXISTING BENT 32

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FIG. E-111

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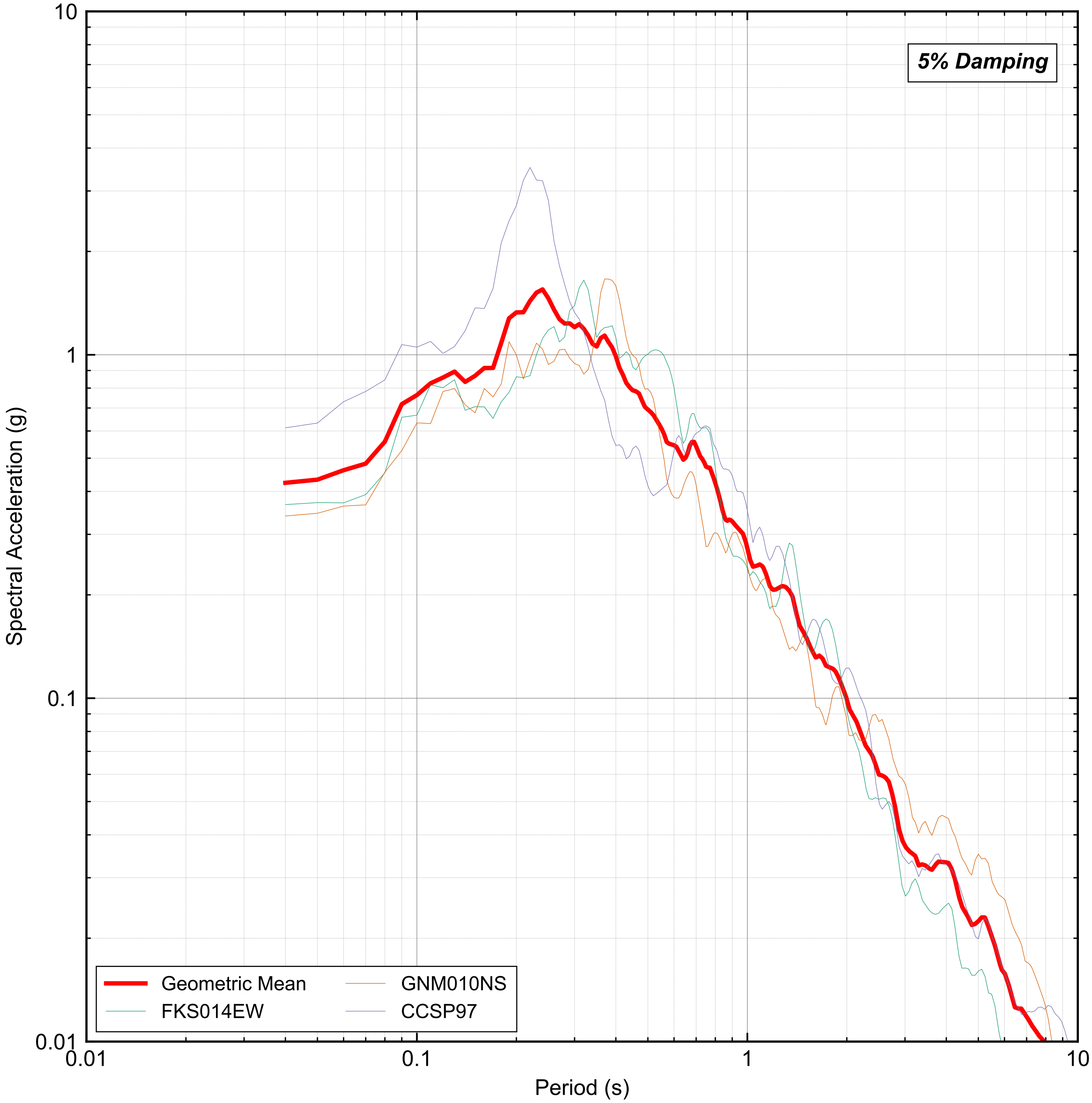
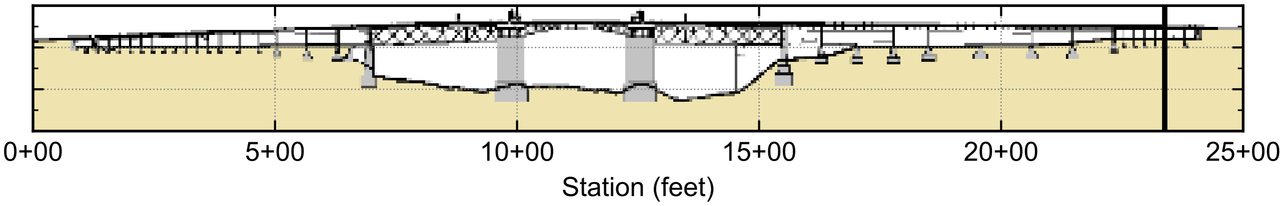
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 33

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FIG. E-112

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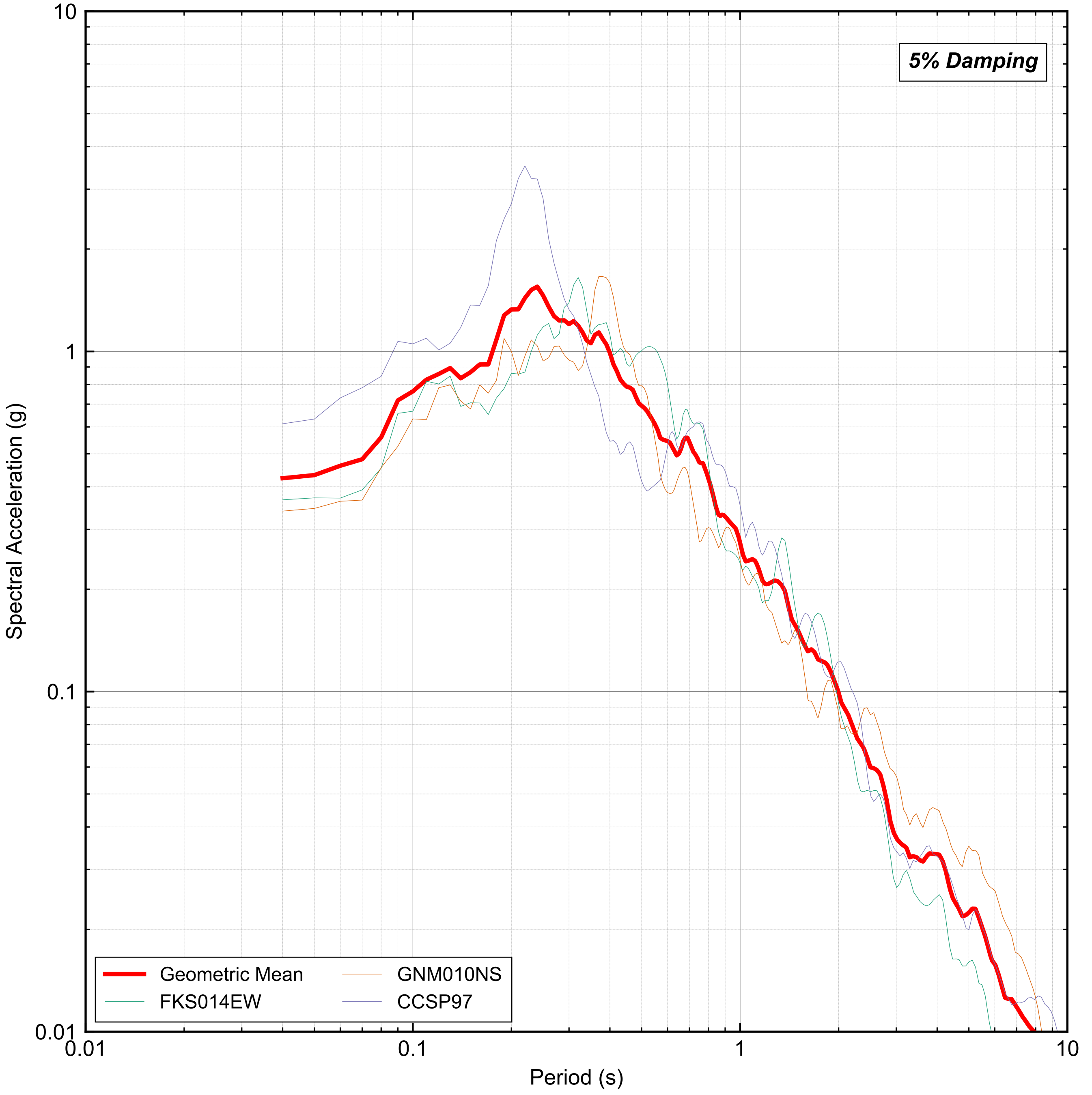

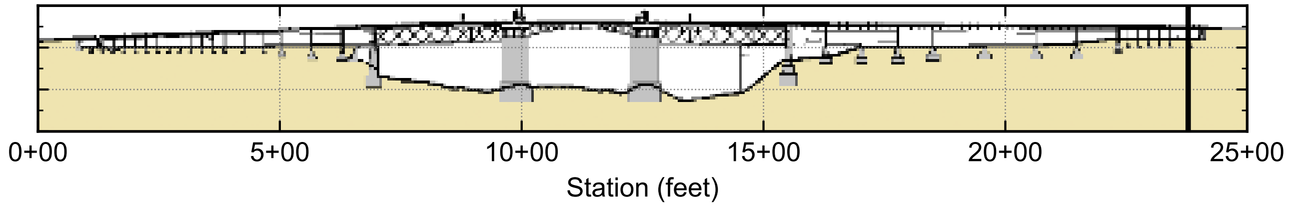

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE LIMITED OPERATION (CSZ 0.2S) EXISTING BENT 34

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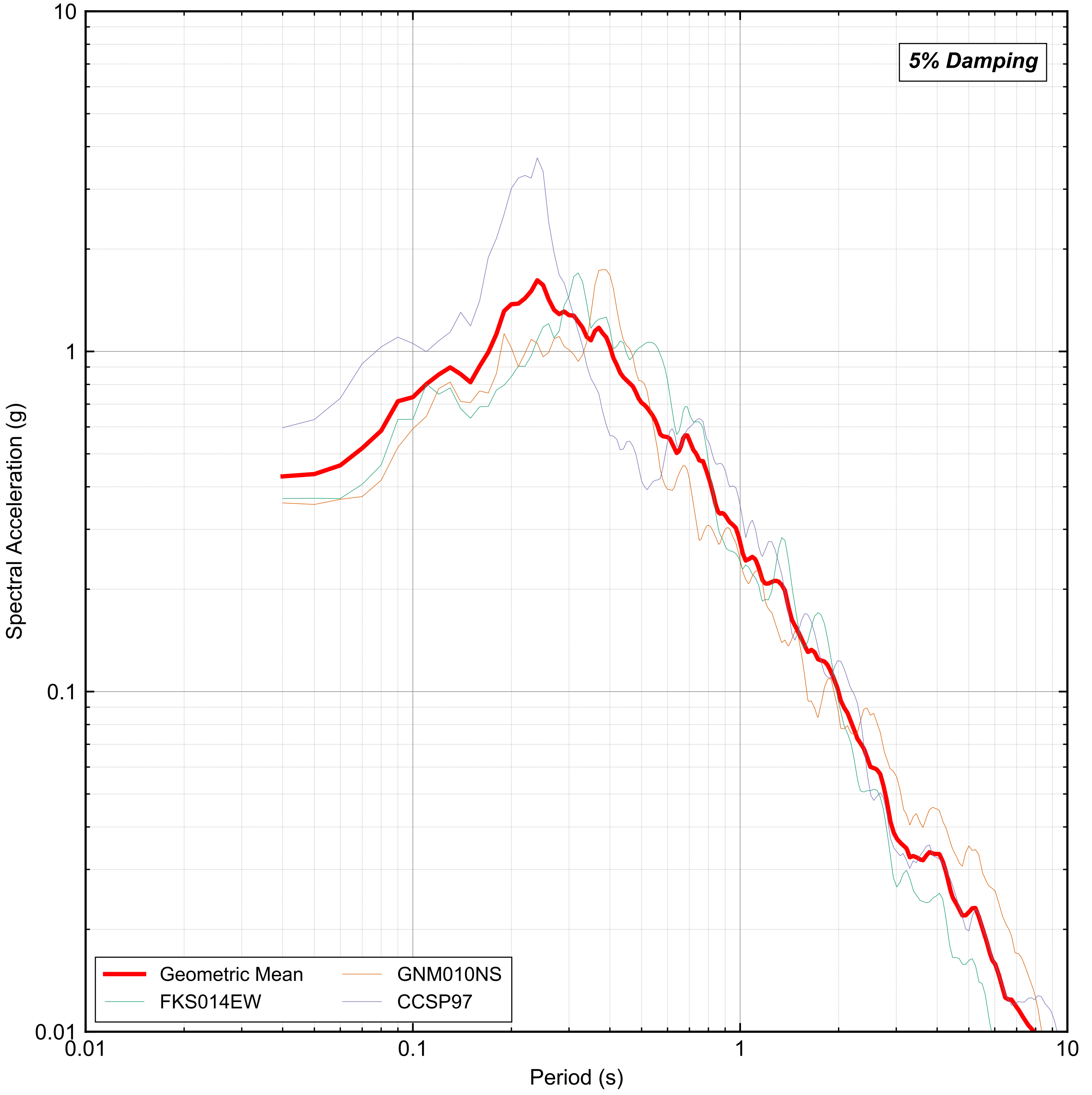
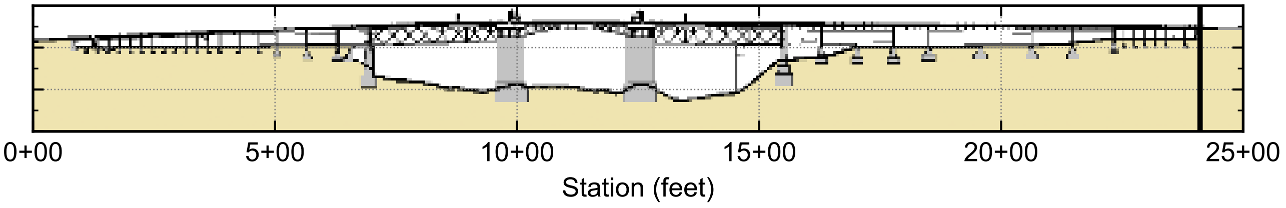
FIG. E-113





Notes:

1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 0.2S)
EXISTING BENT 35

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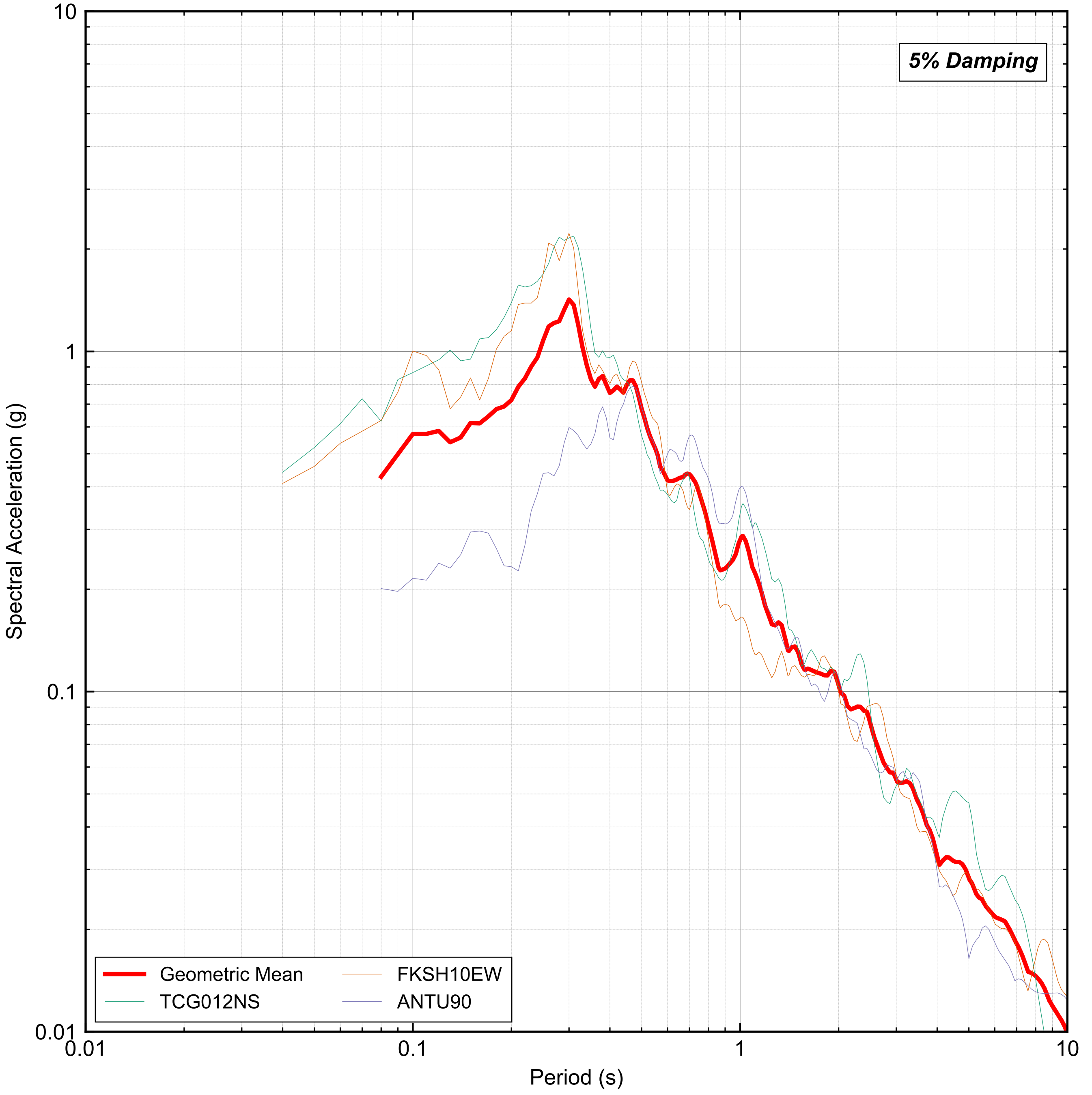
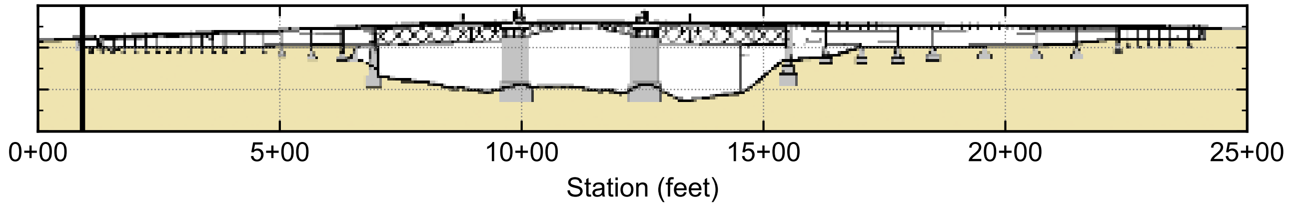
FIG. E-114





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 1

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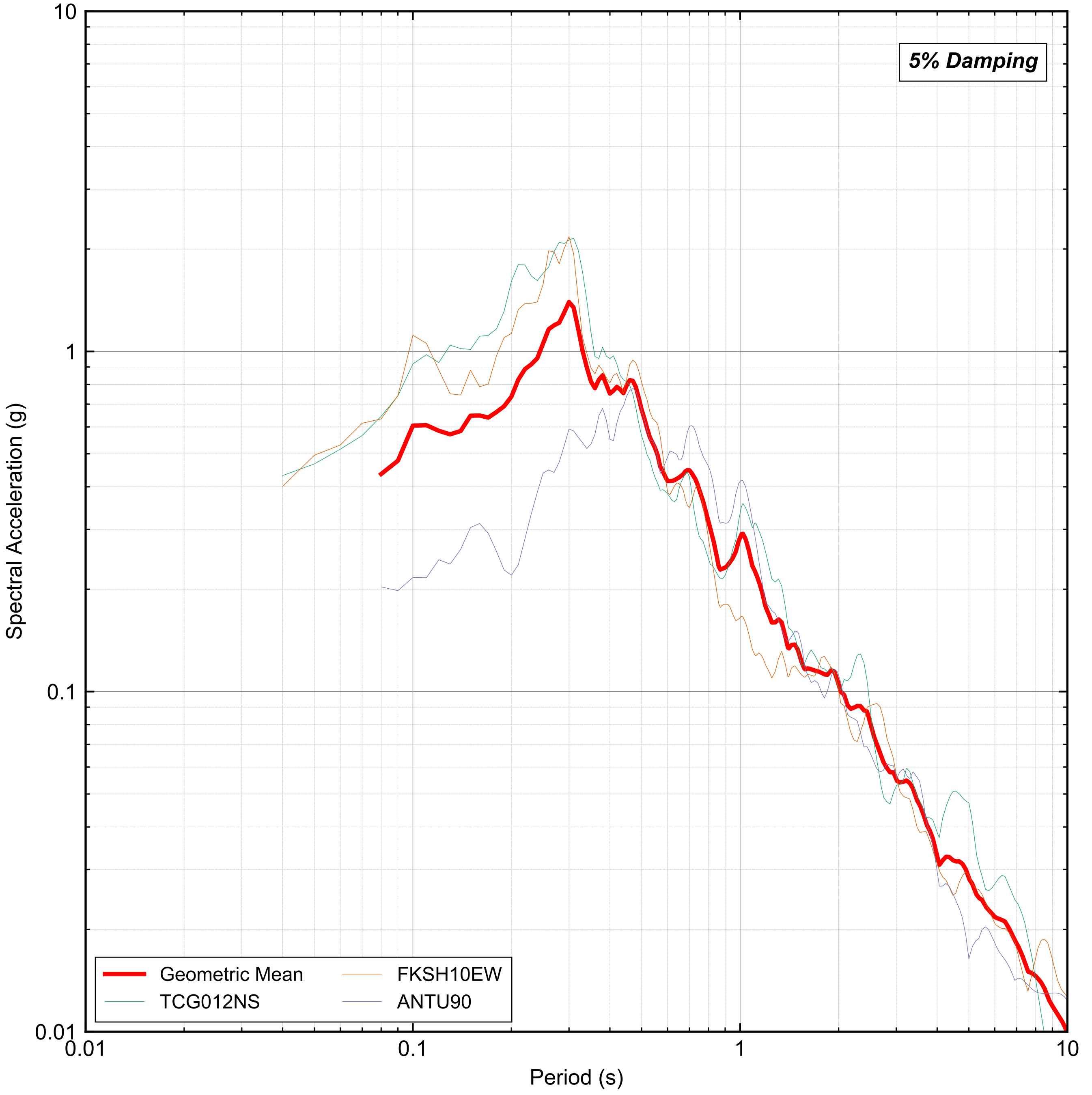
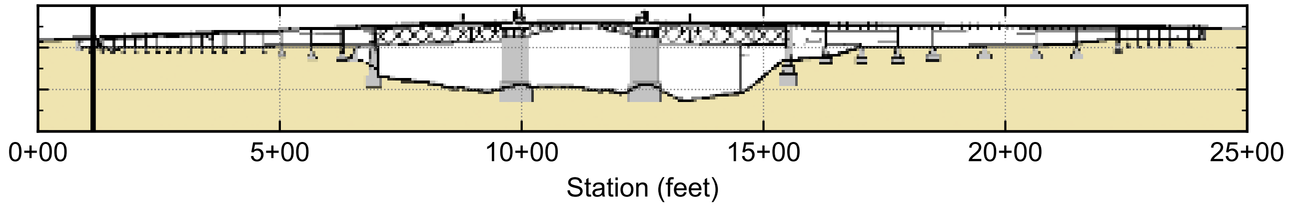
FIG. E-115





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 2

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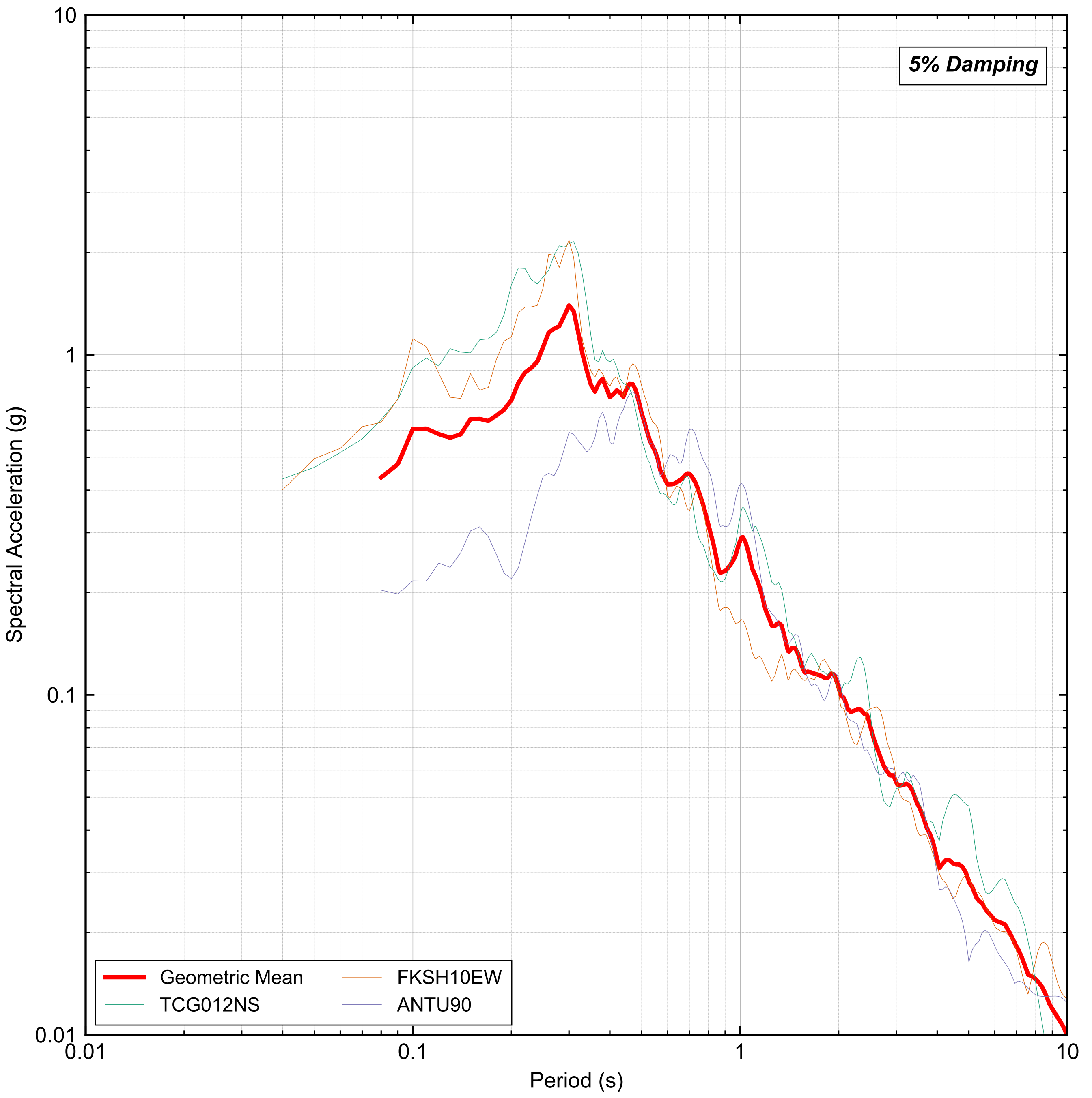
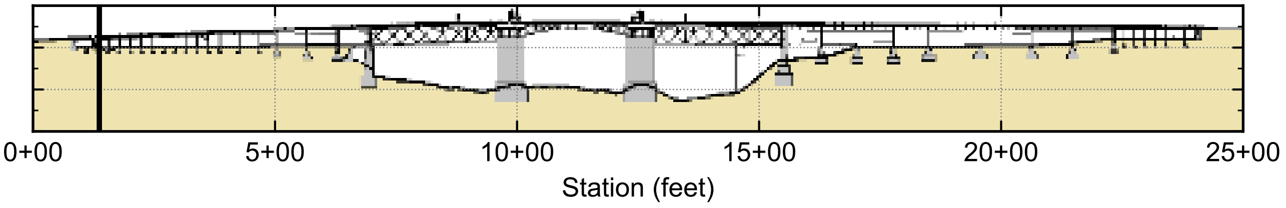
FIG. E-116

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 3

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FIG. E-117

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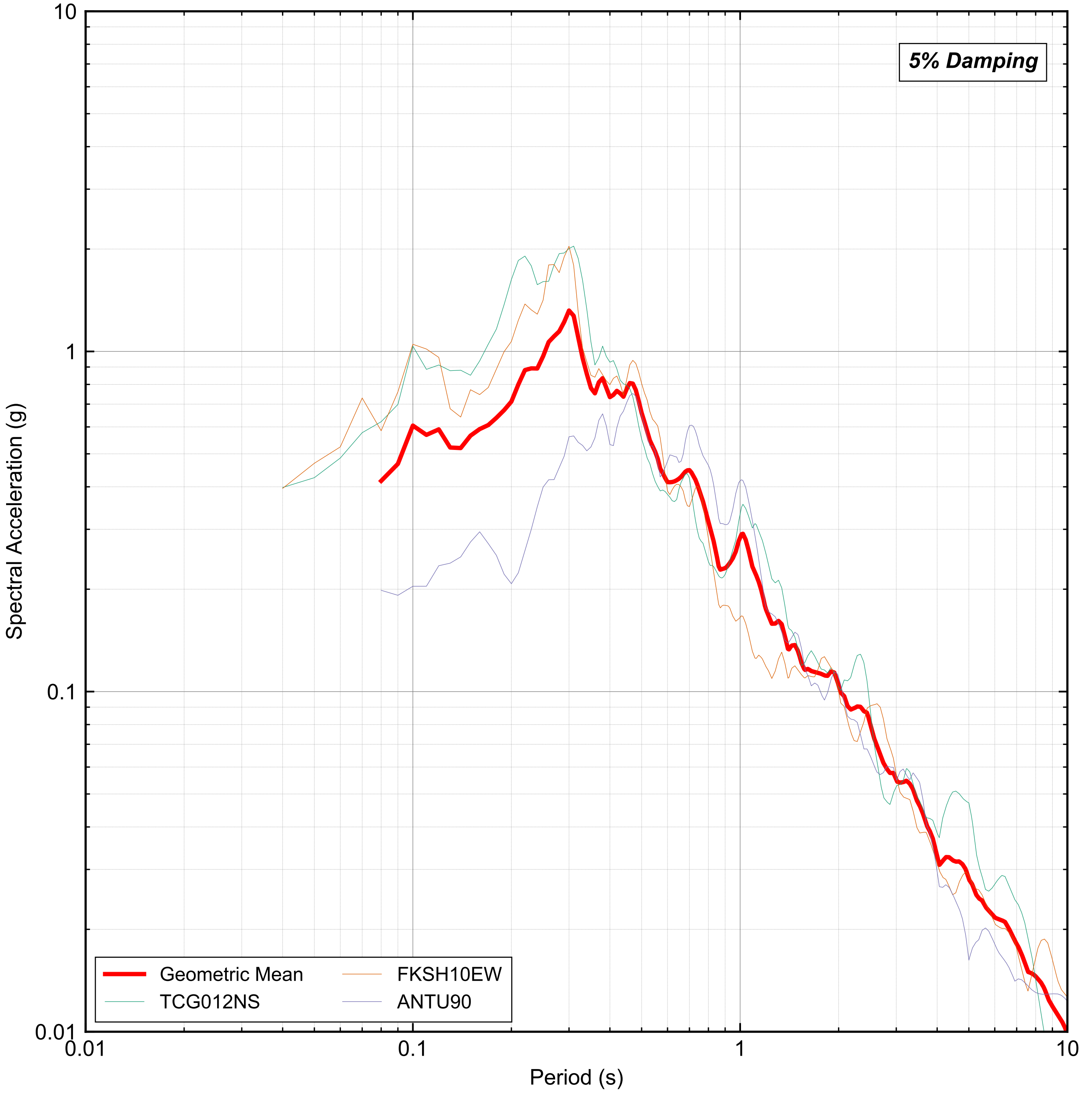
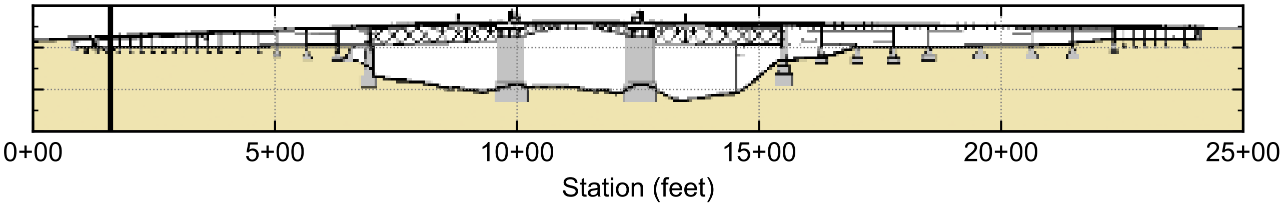
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 4

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FIG. E-118

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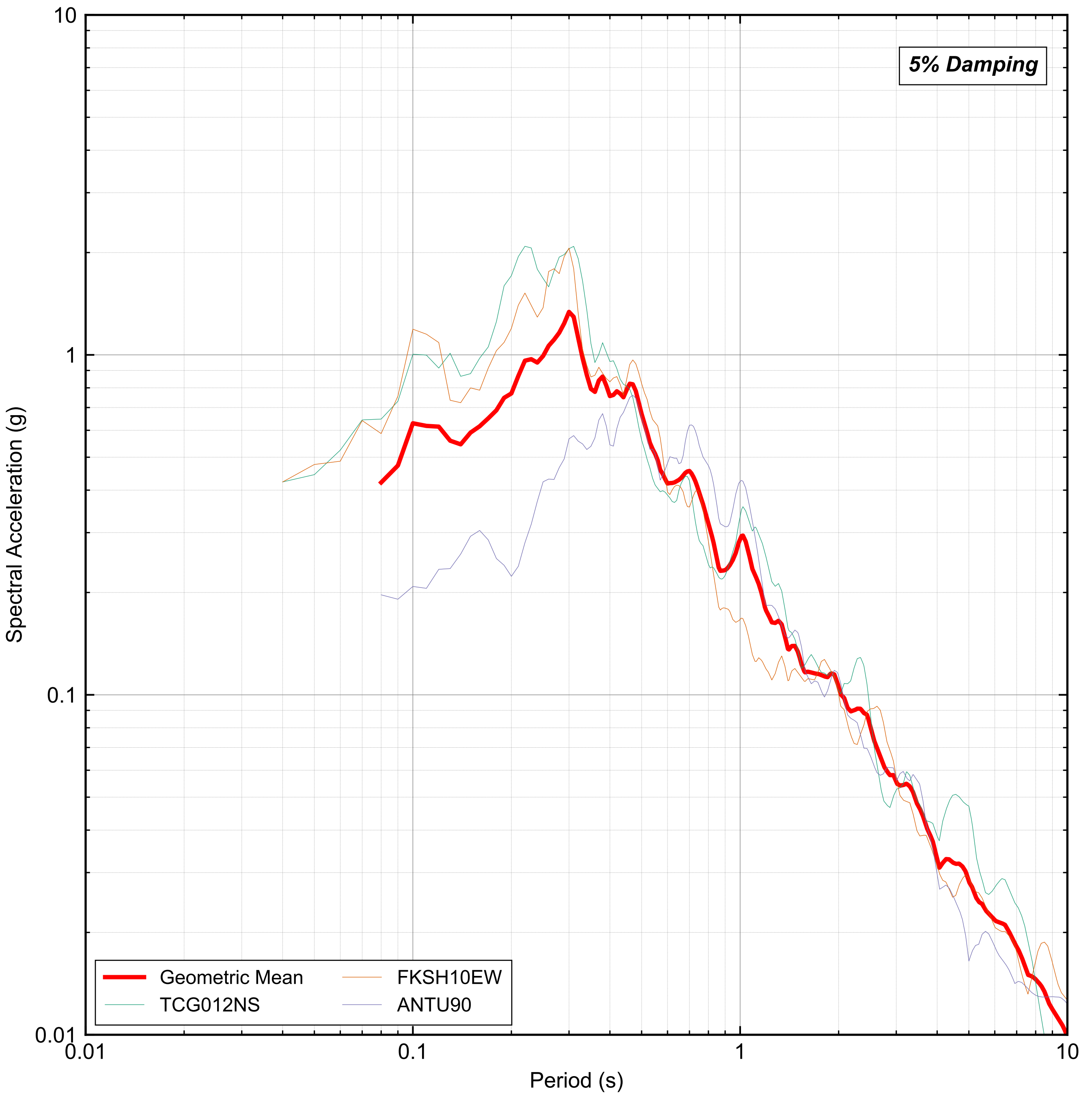
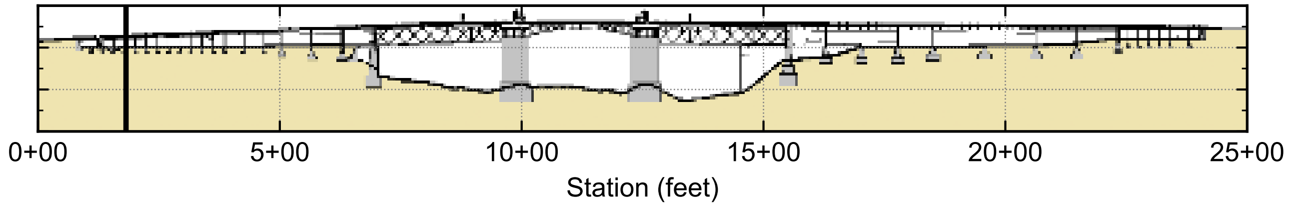
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 5

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FIG. E-119

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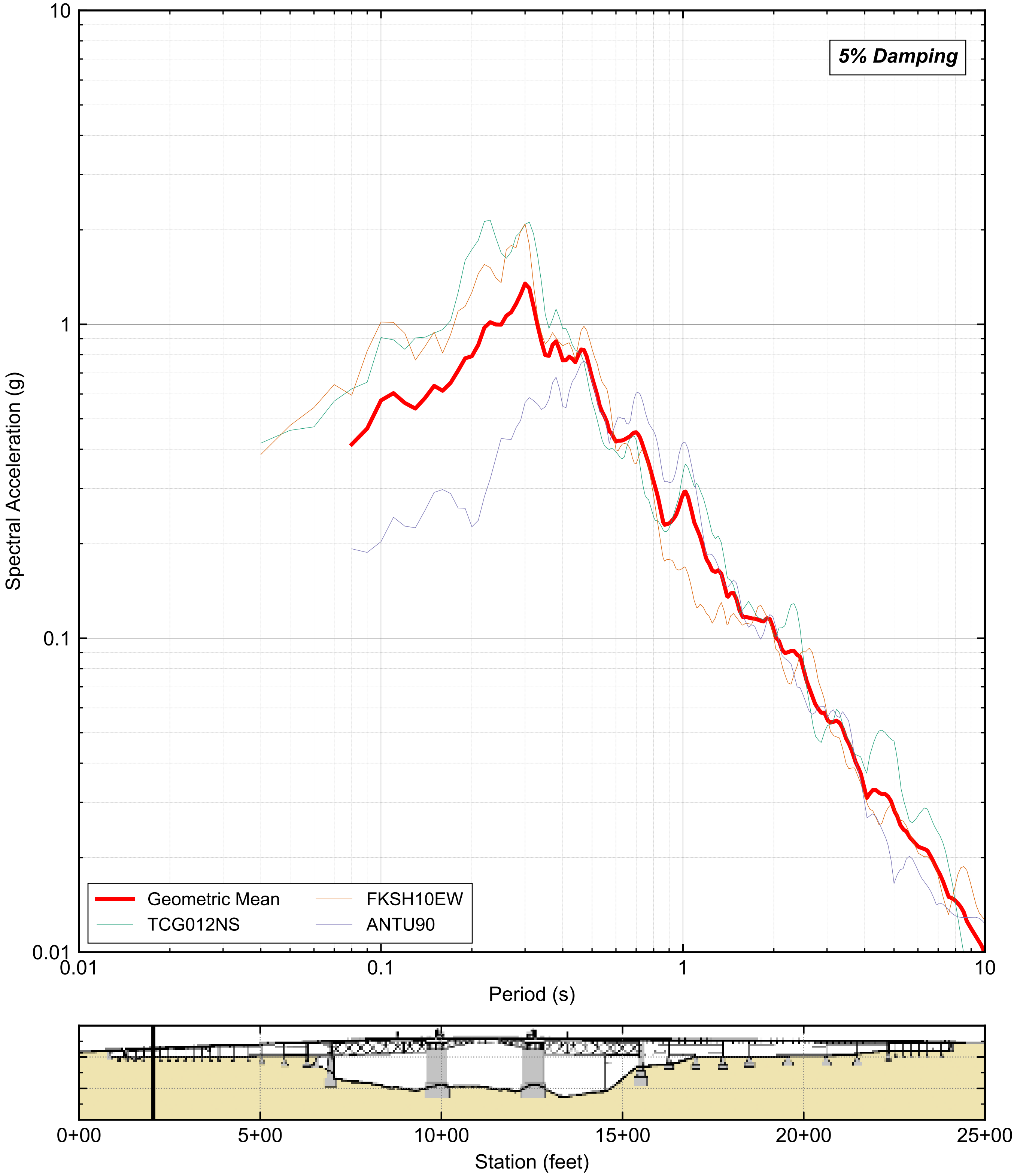
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 6

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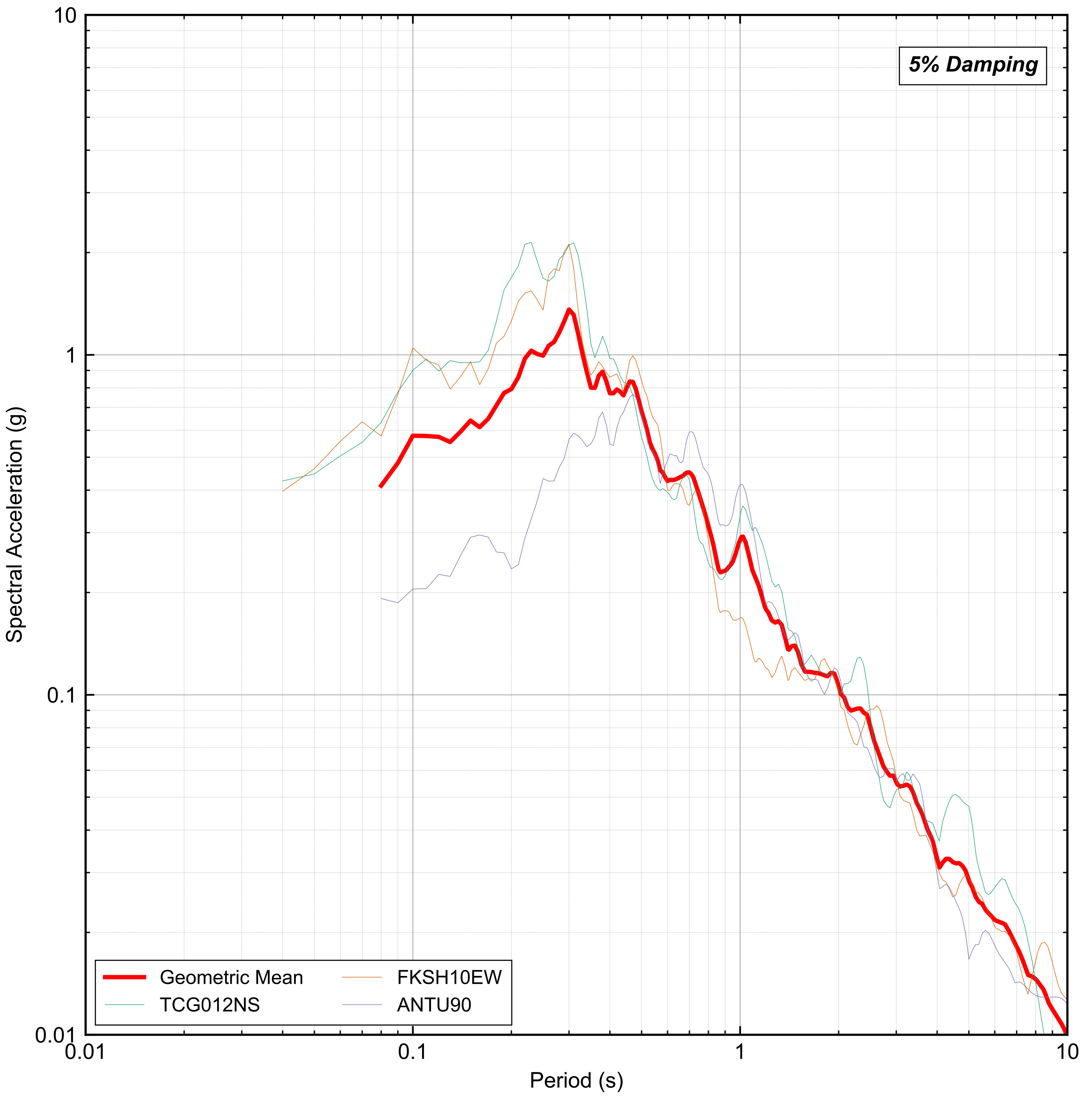
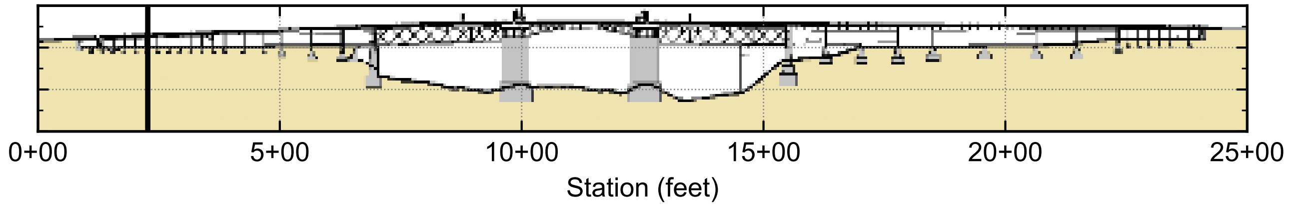
FIG. E-120

C:\Users\jnb\FLAC\Burnside\v6_test

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 7

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FIG. E-121

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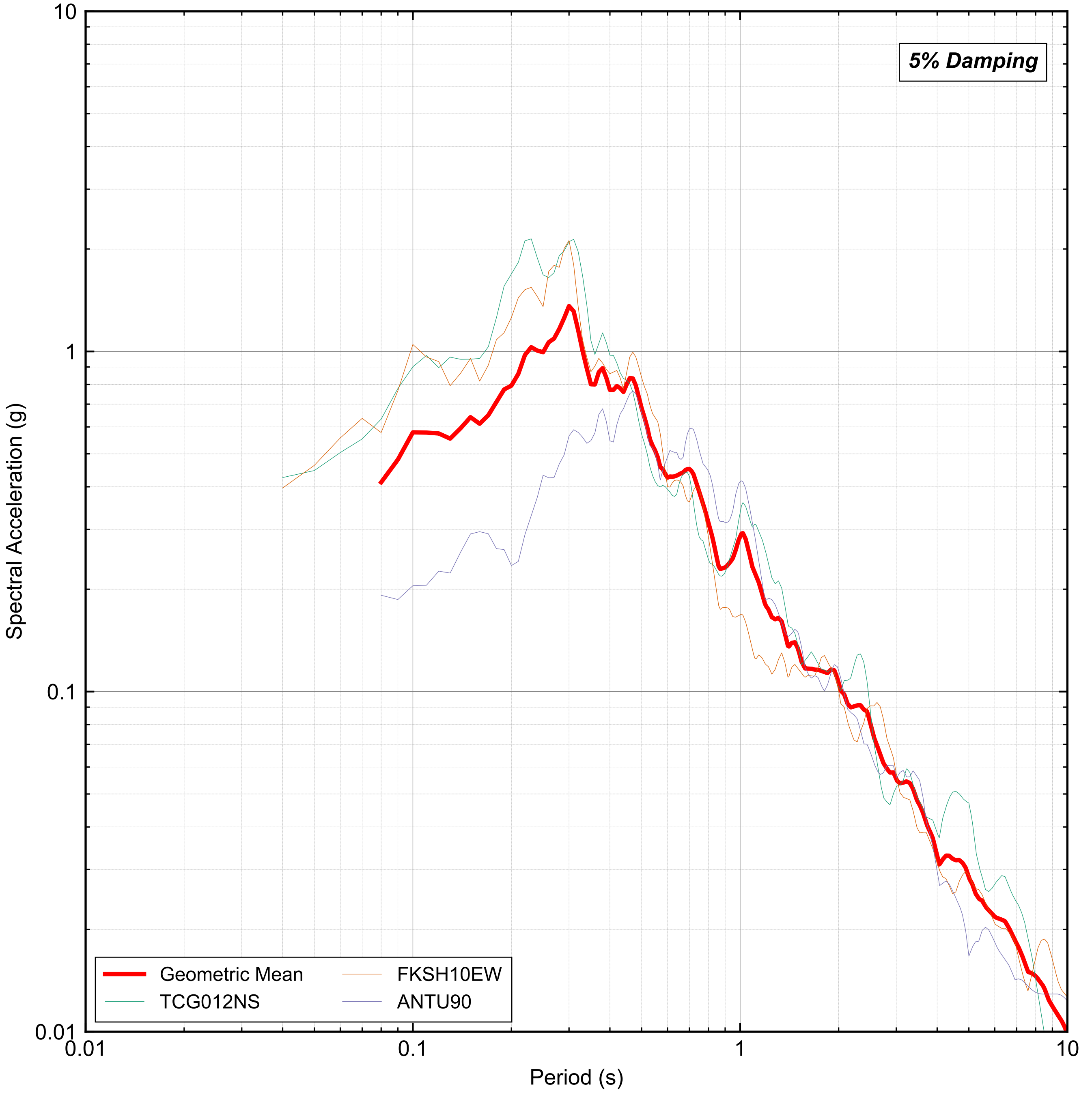

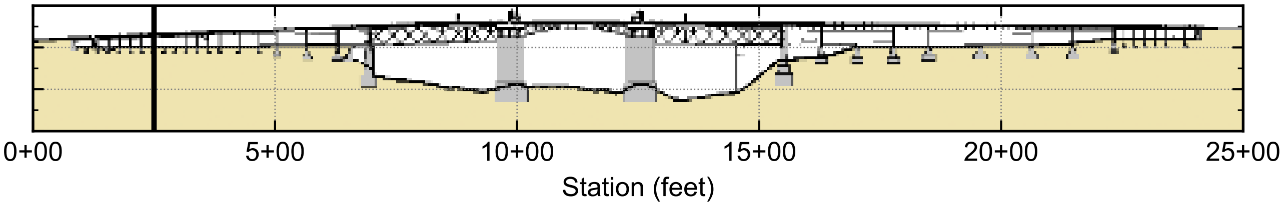

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 8

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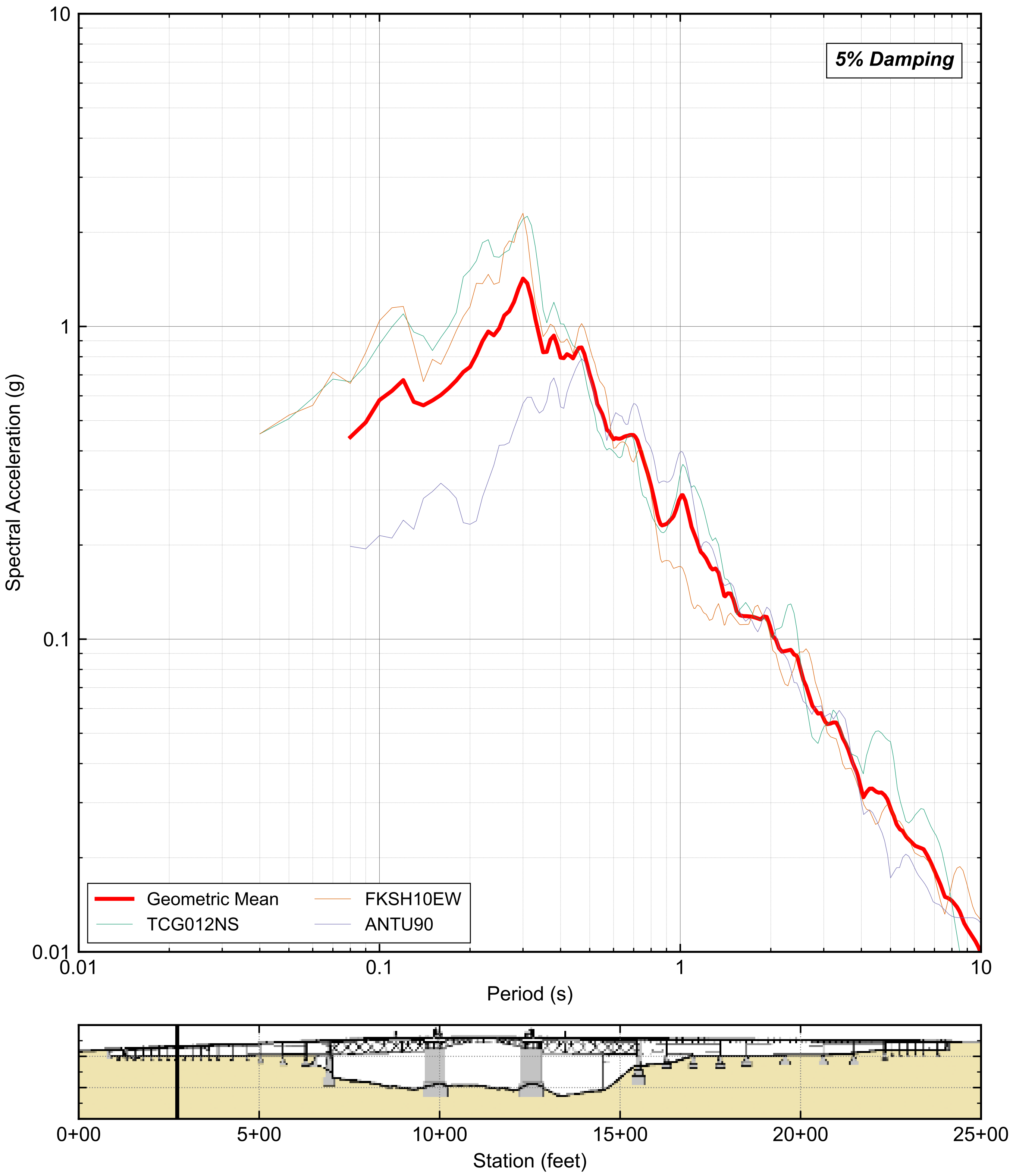
FIG. E-122

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 9

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FIG. E-123

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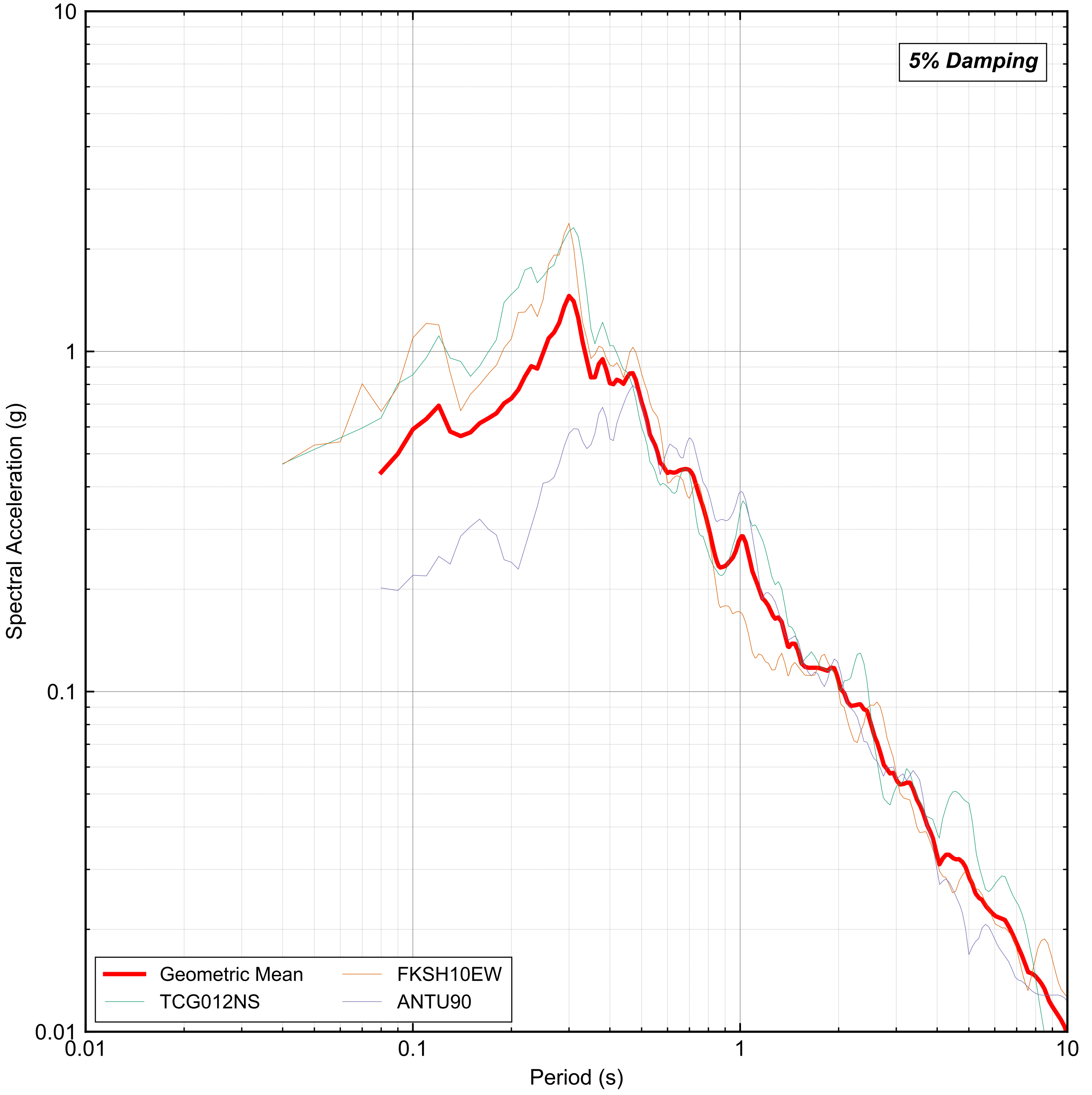
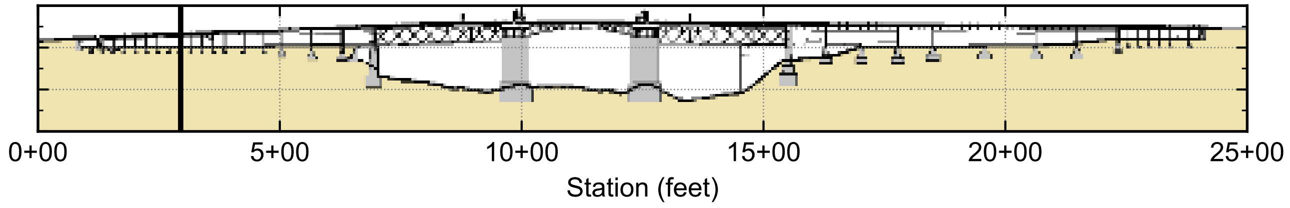
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 10

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FIG. E-124

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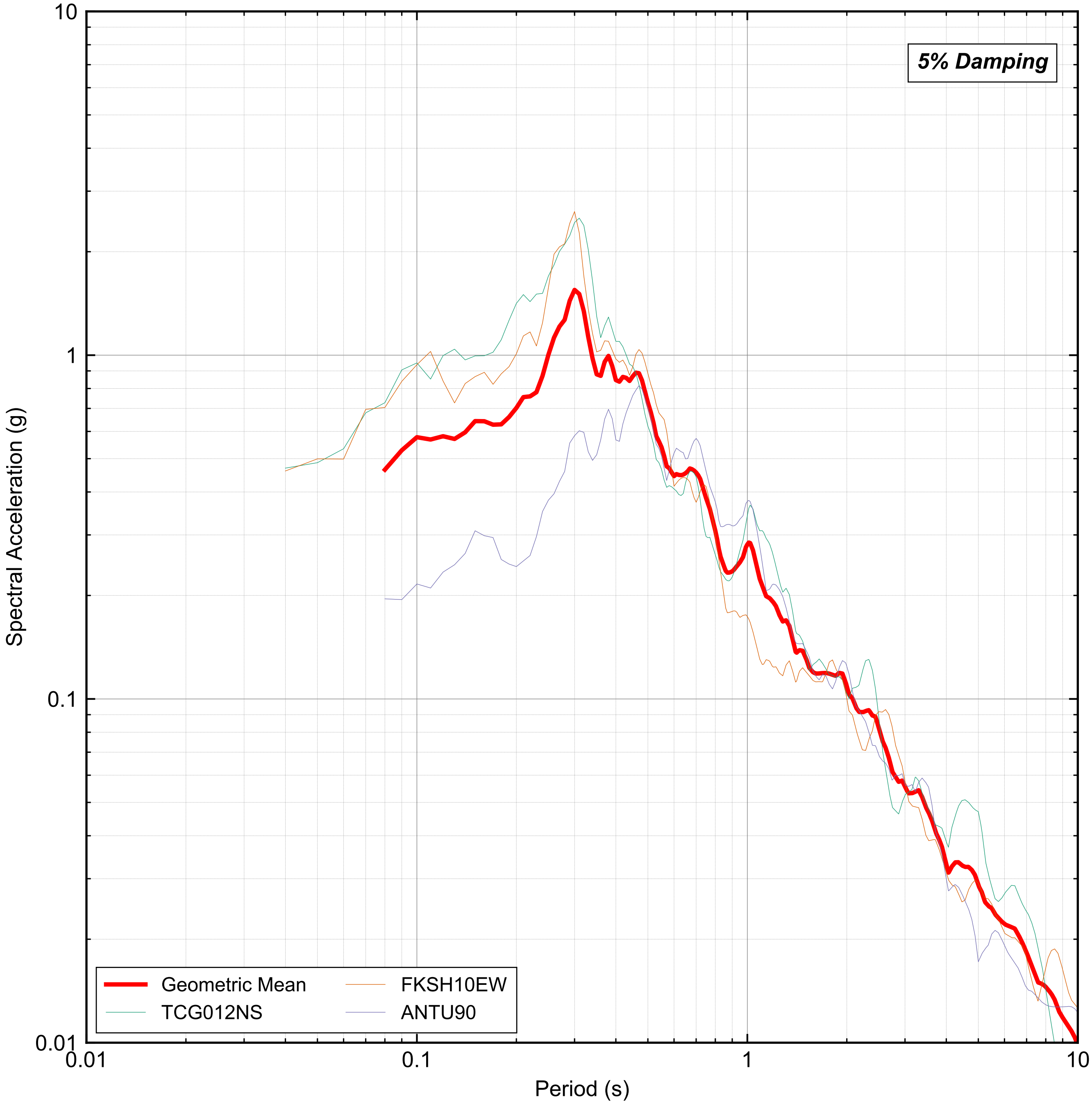
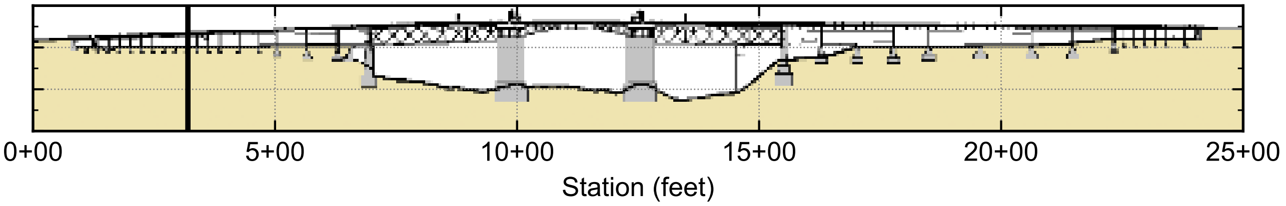
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 11

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FIG. E-125

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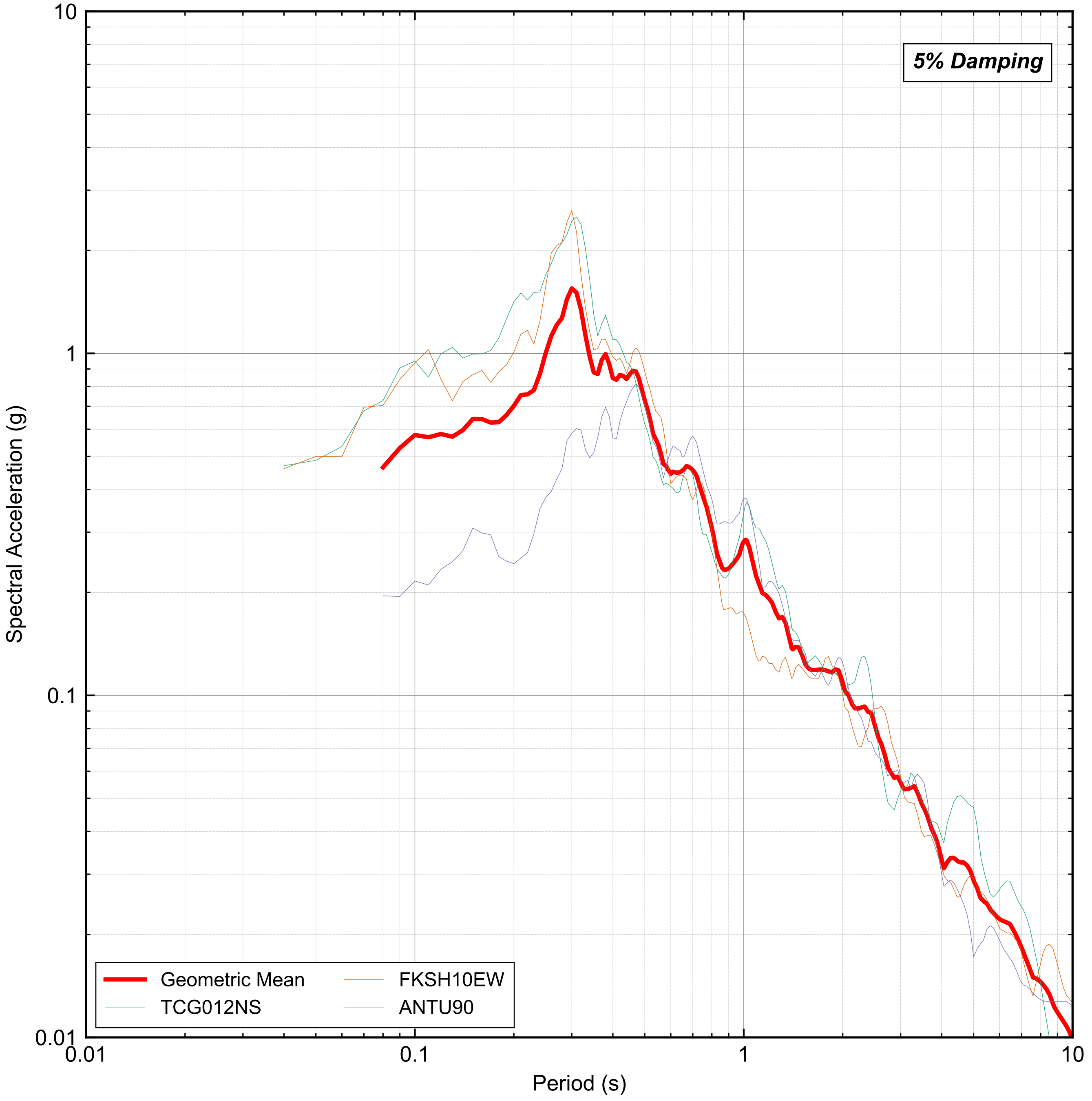

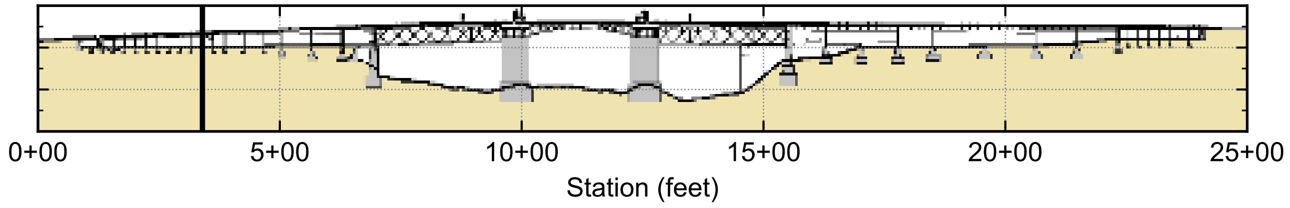

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 12

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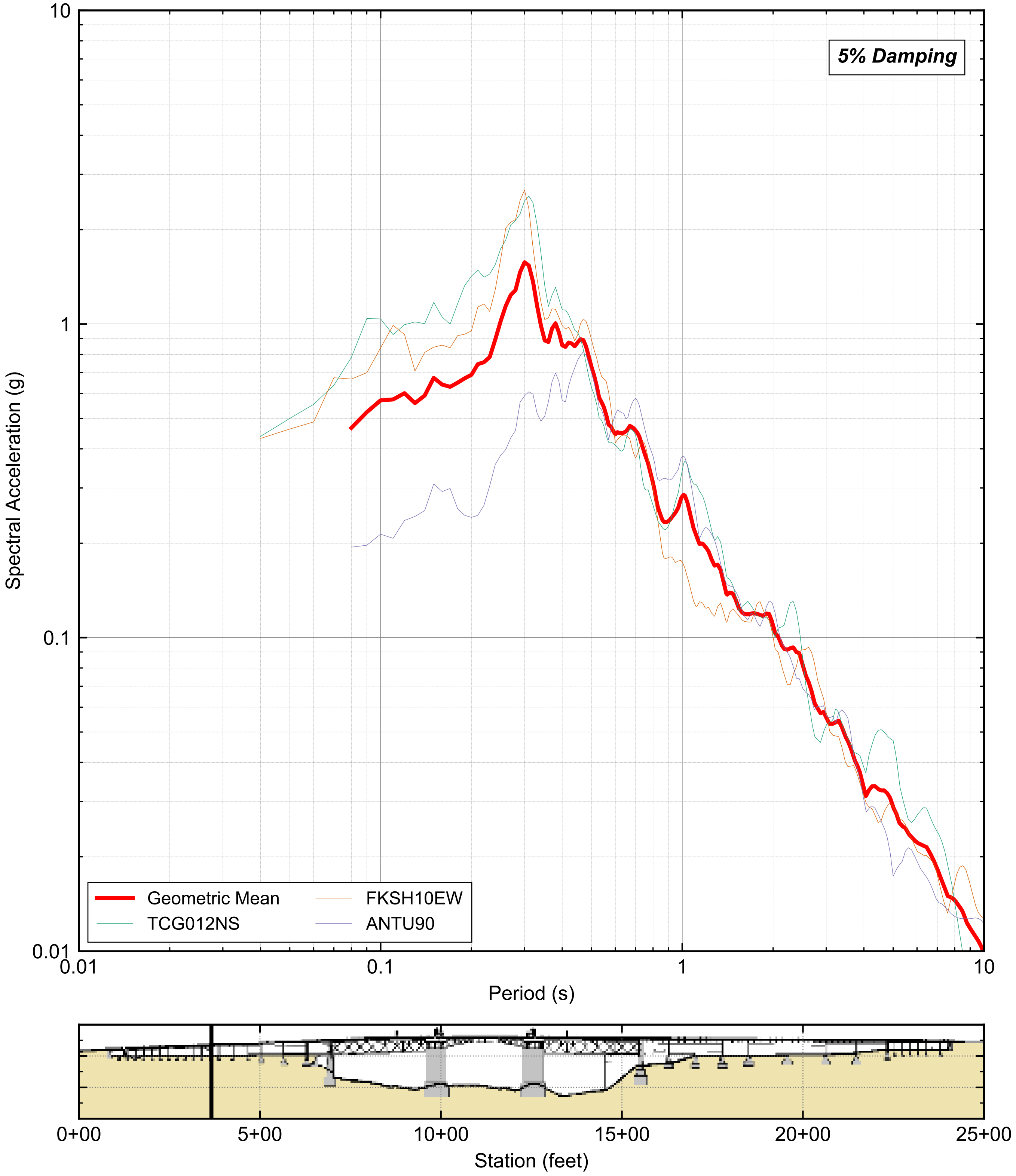
FIG. E-126

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 13

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FIG. E-127

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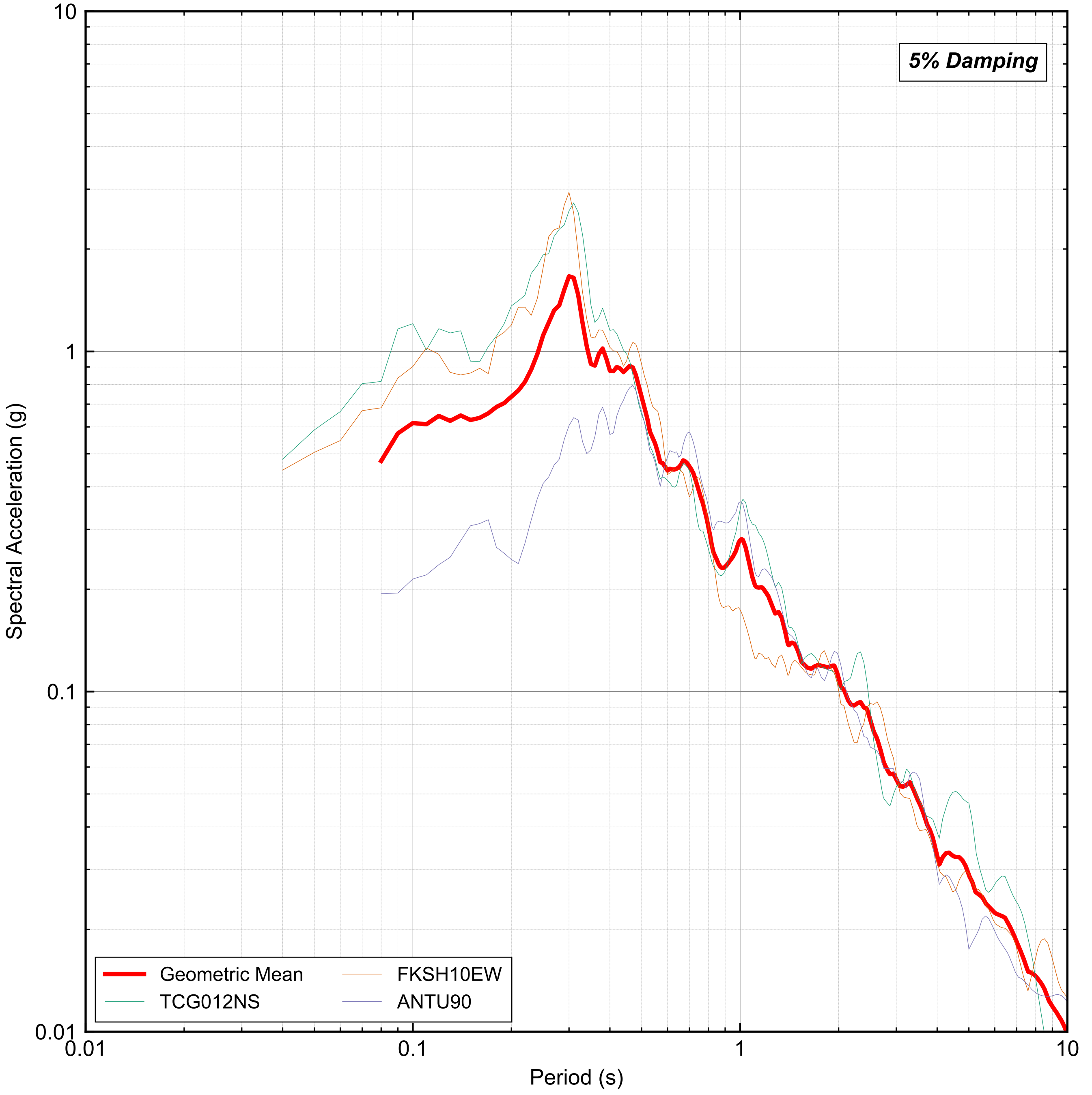

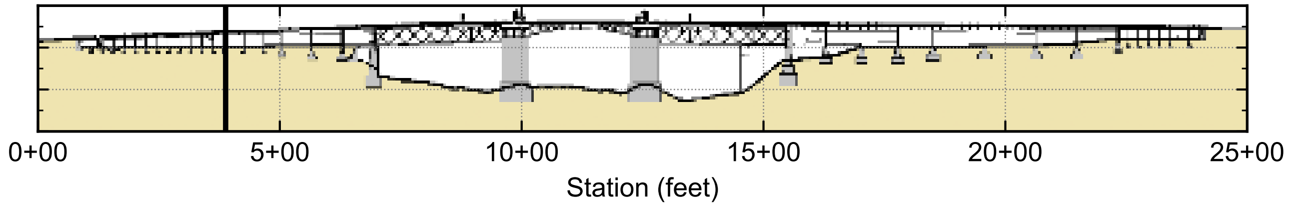

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 14

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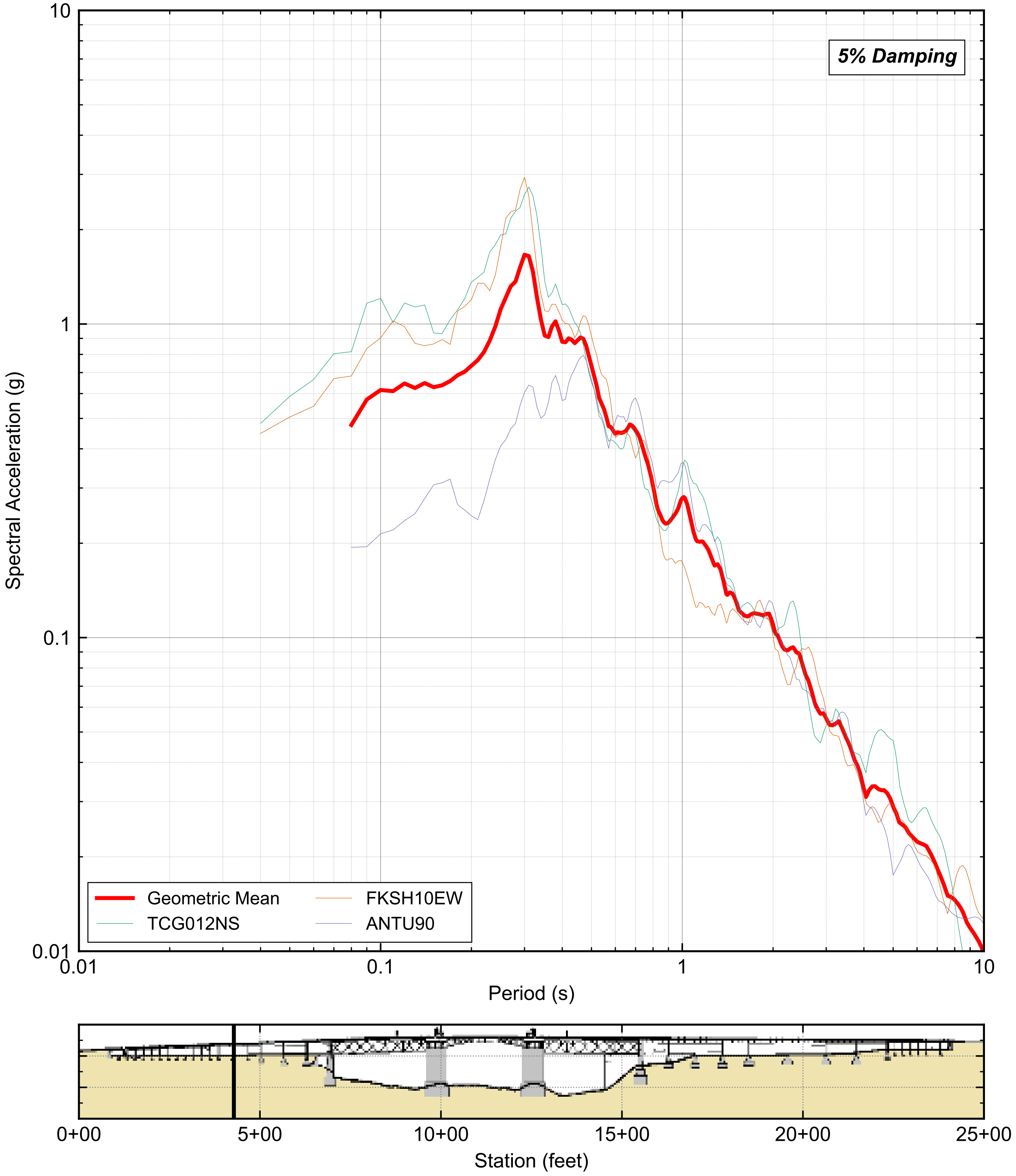
FIG. E-128



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 15

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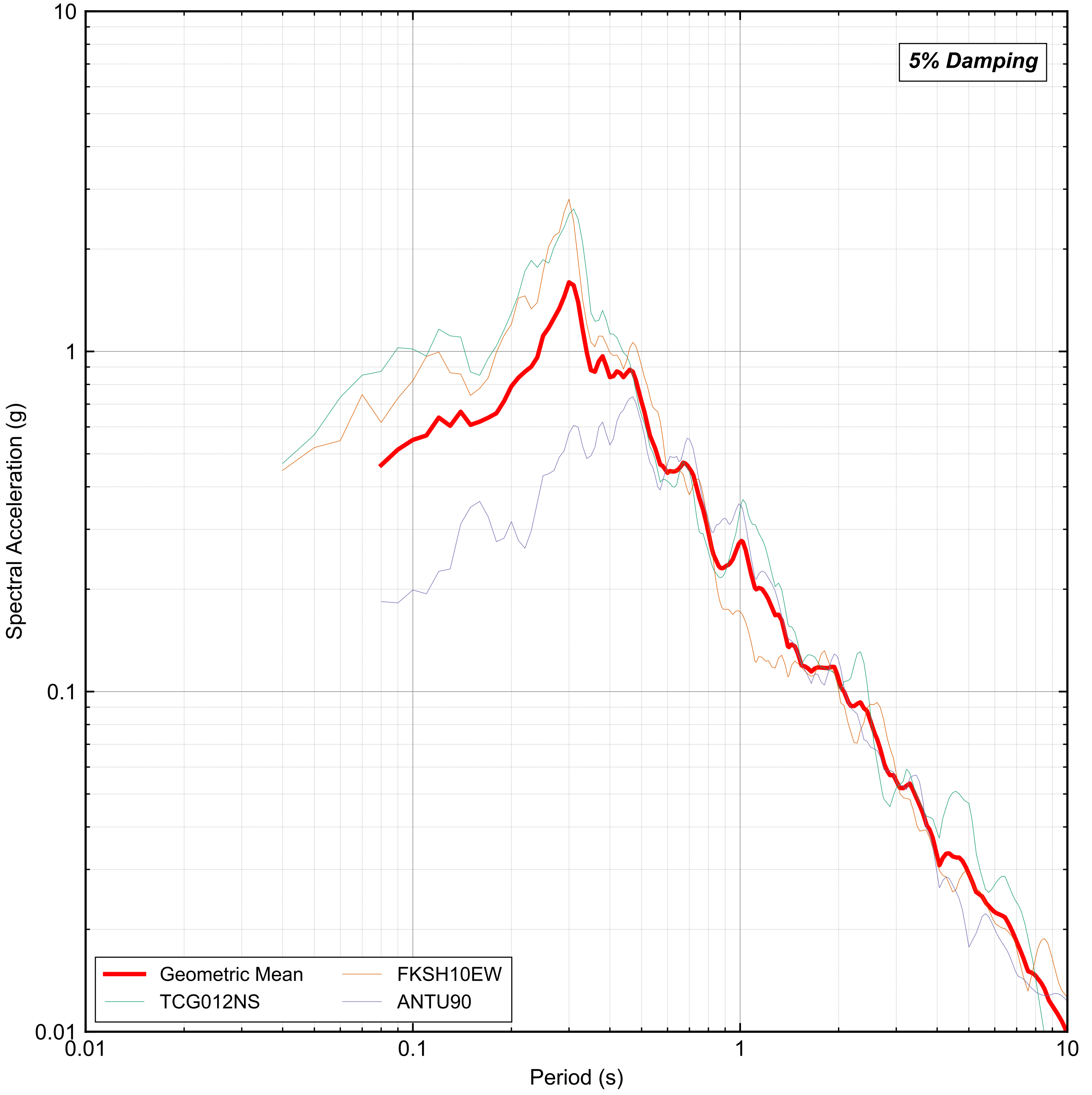
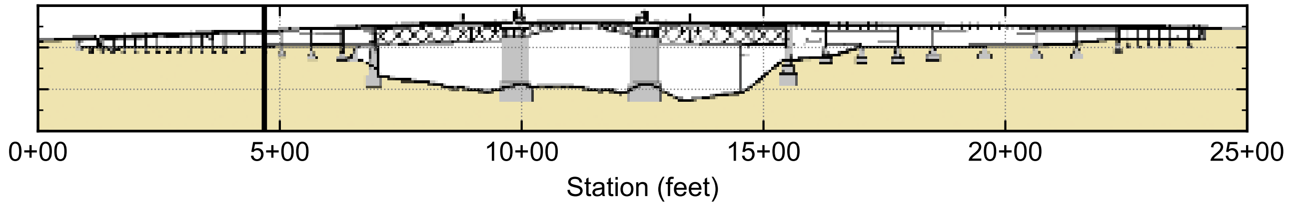
FIG. E-129

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 16

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FIG. E-130

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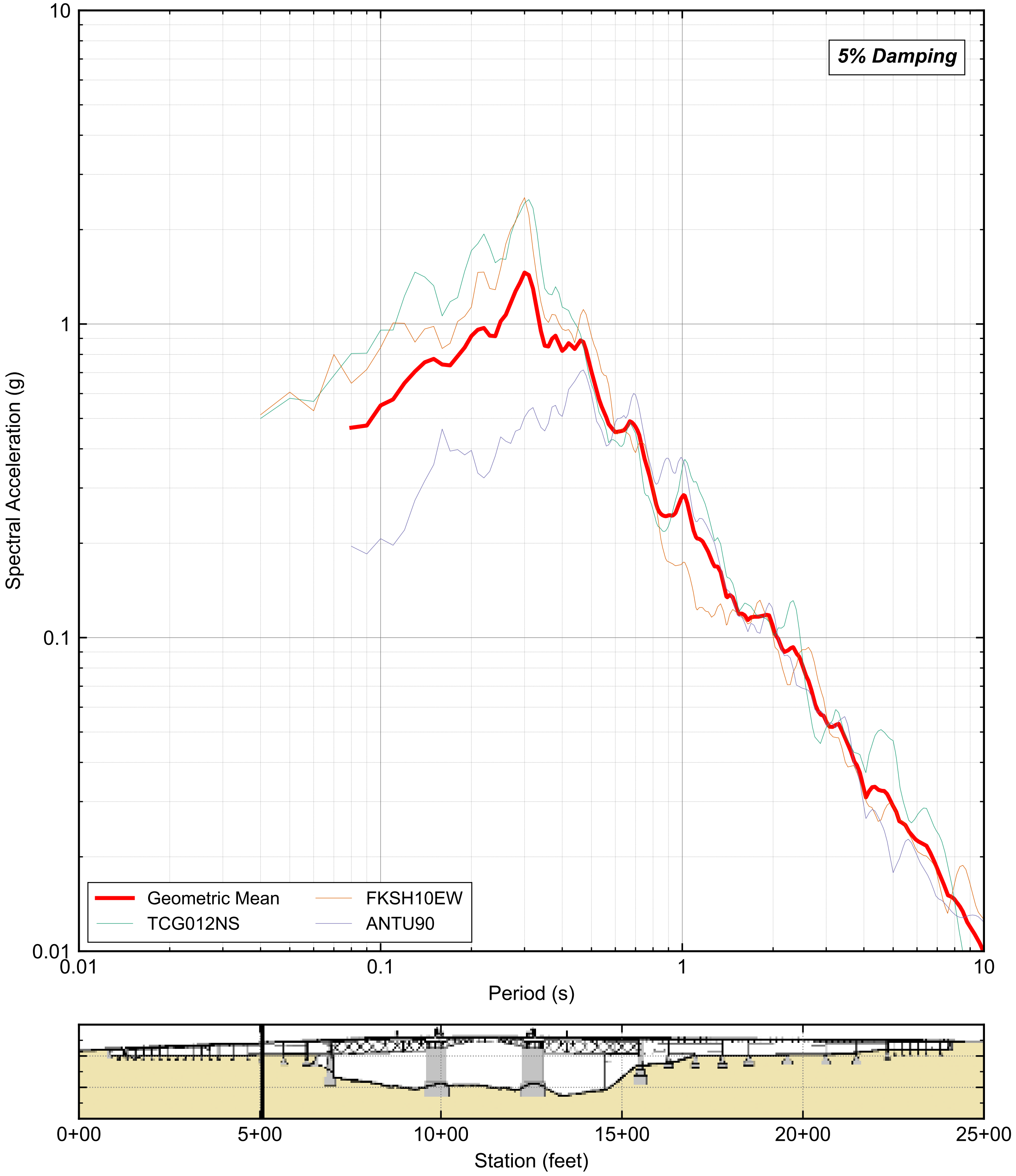
ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 17

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FIG. E-131

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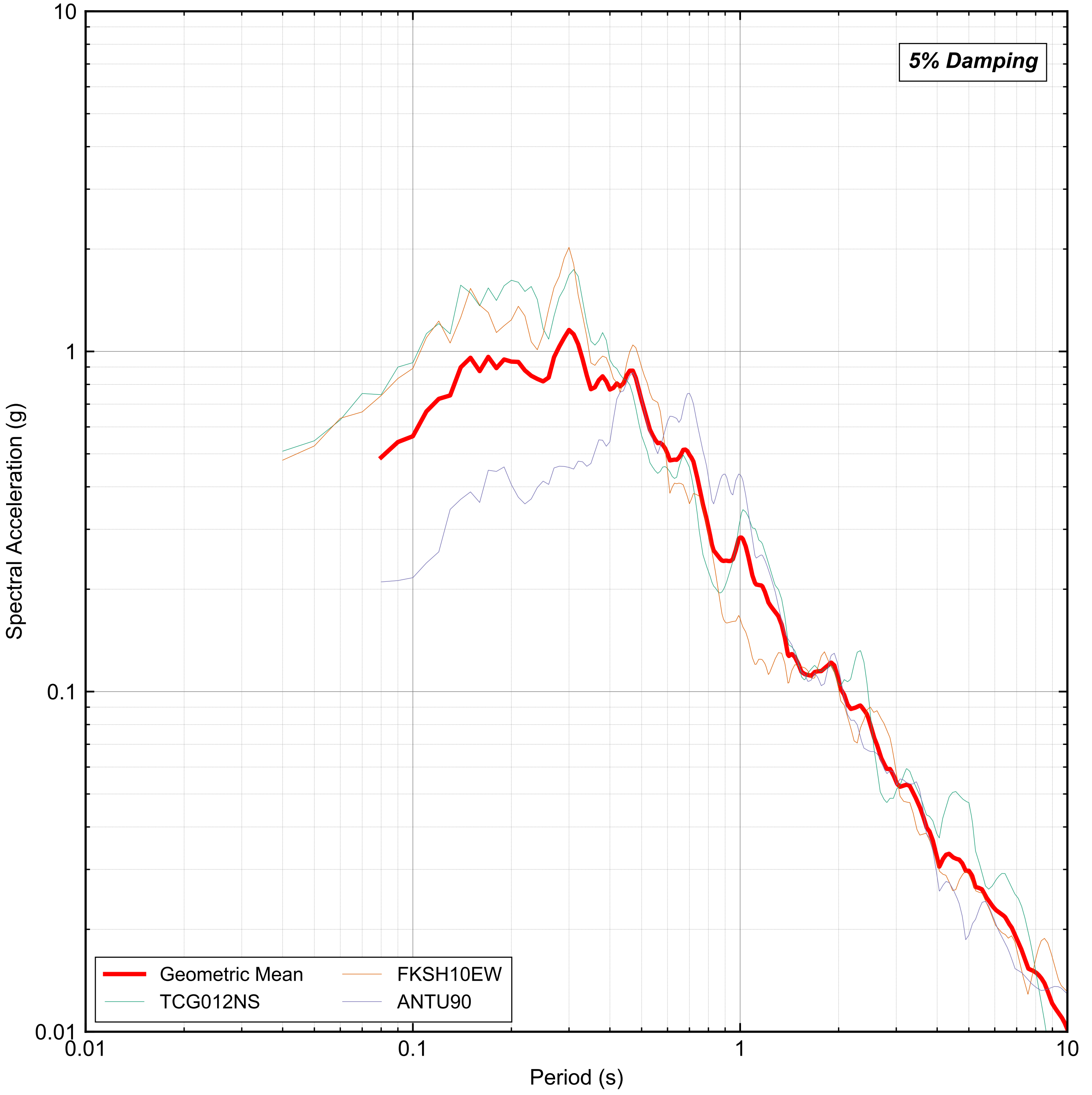
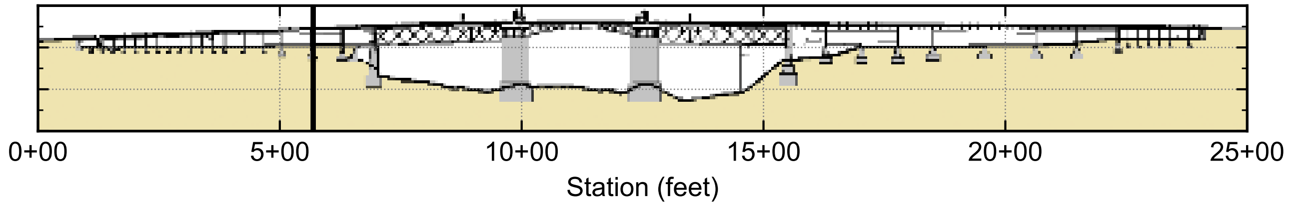
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 18

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FIG. E-132

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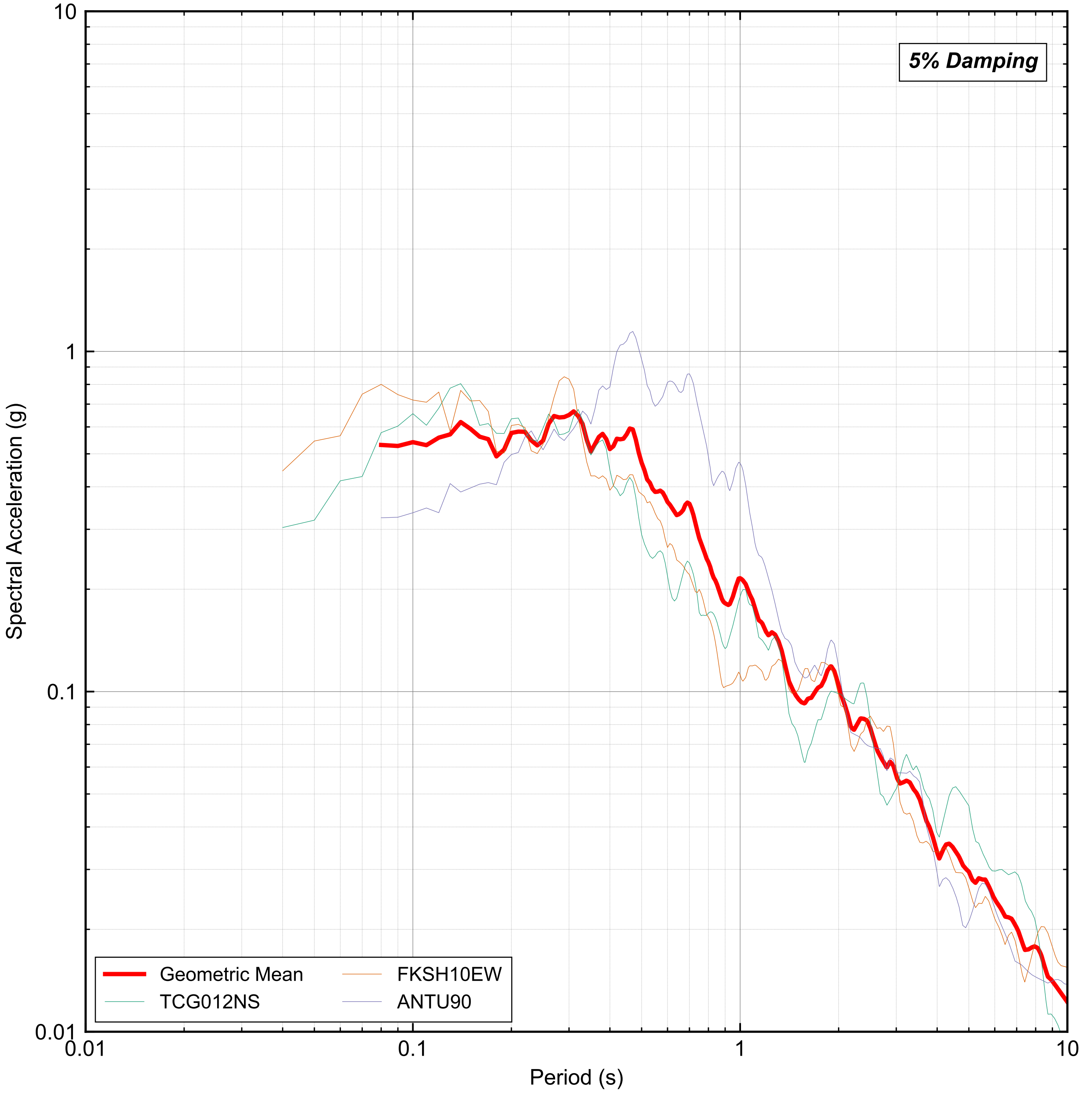

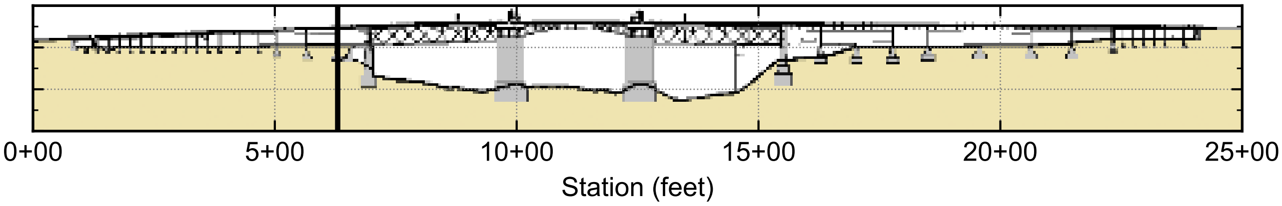

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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**GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 19**

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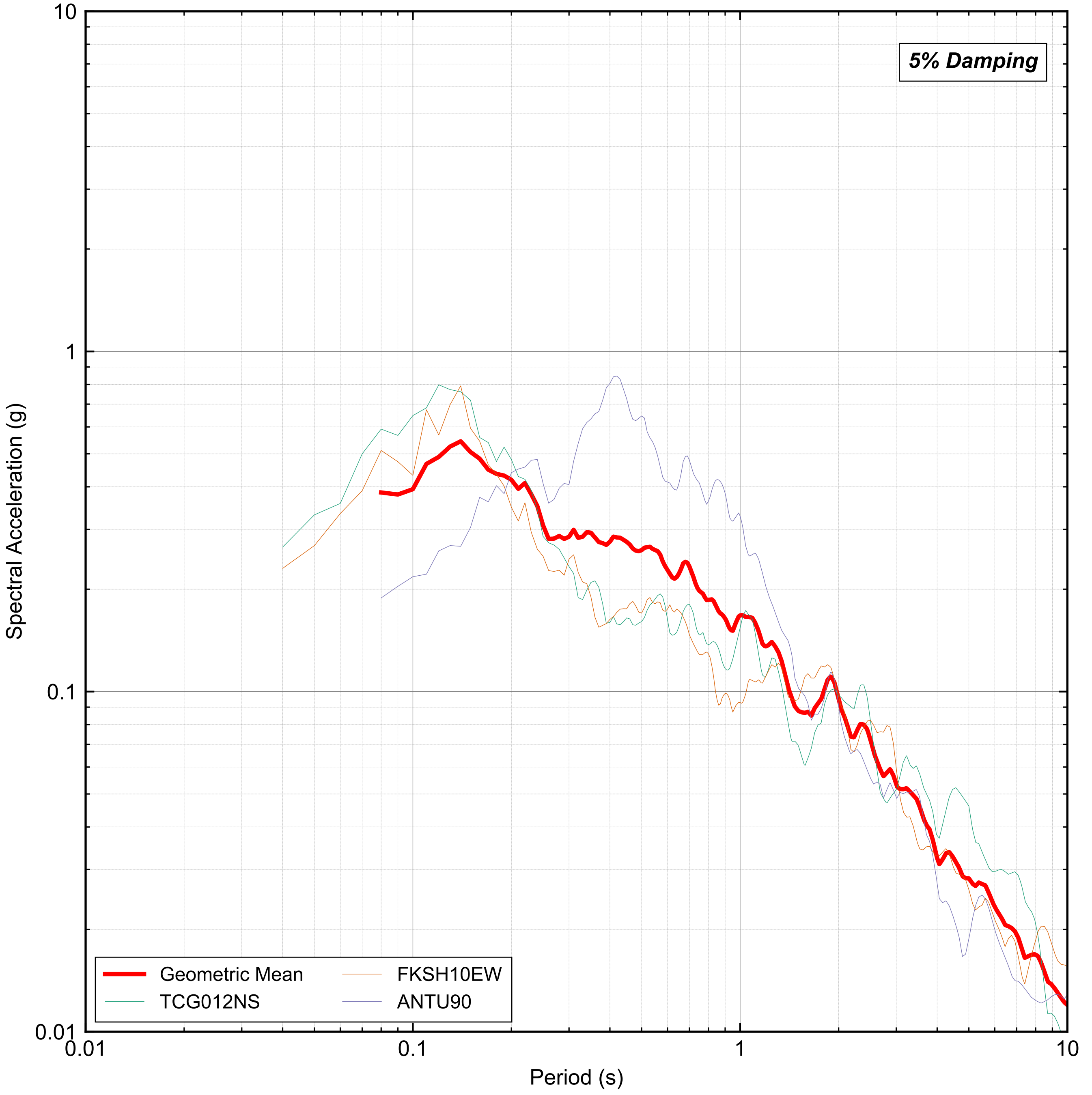
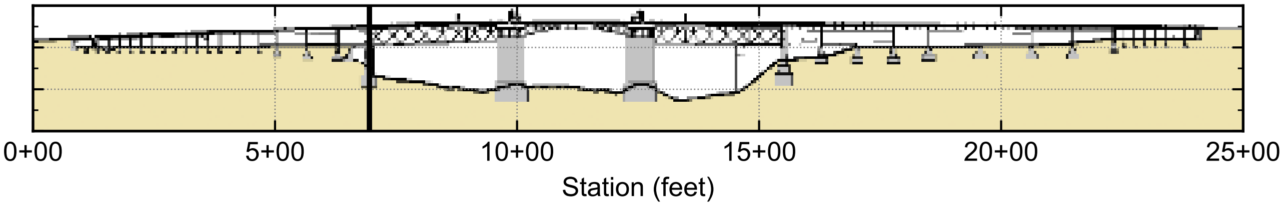
FIG. E-133





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING PIER 1

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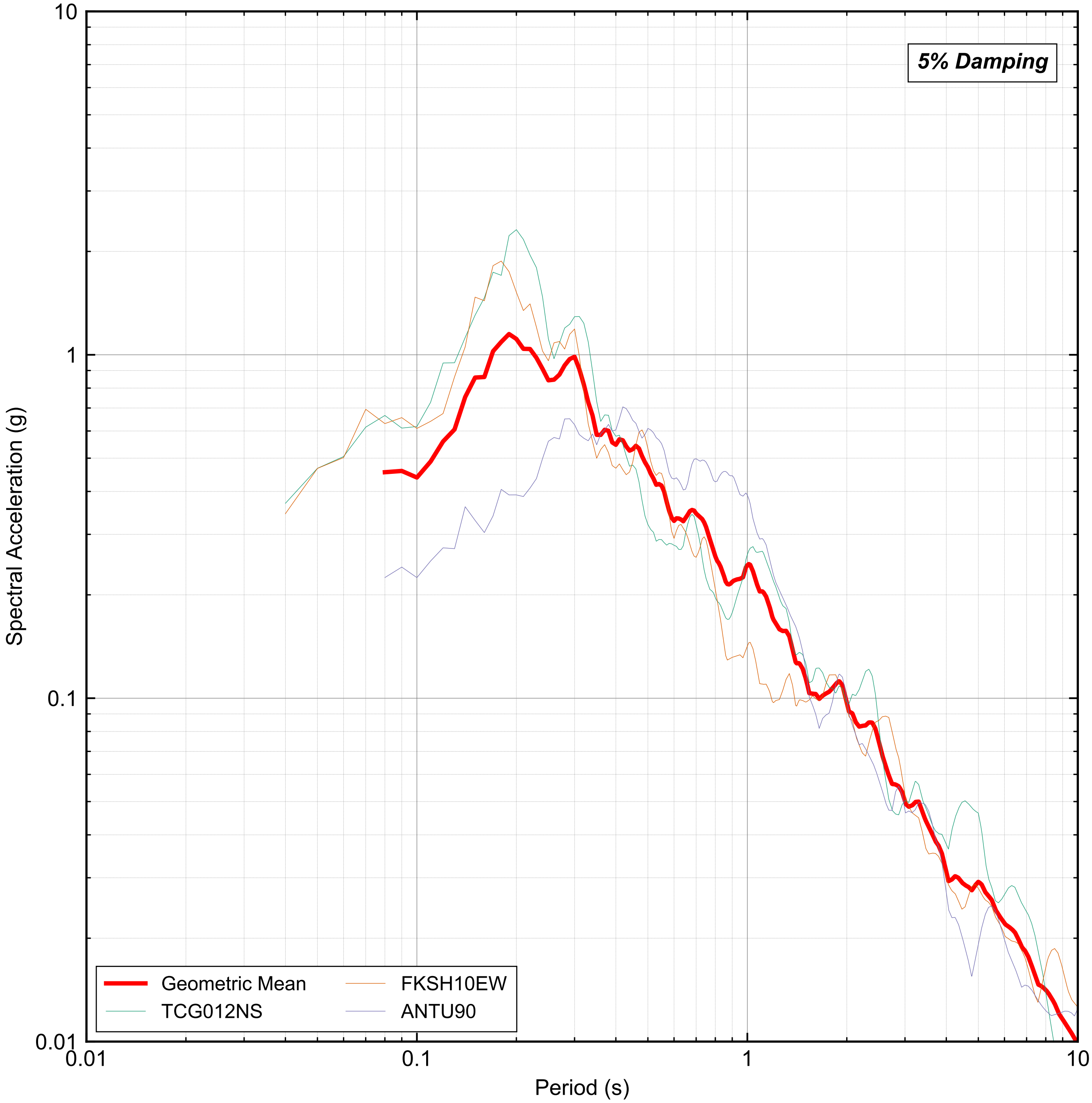
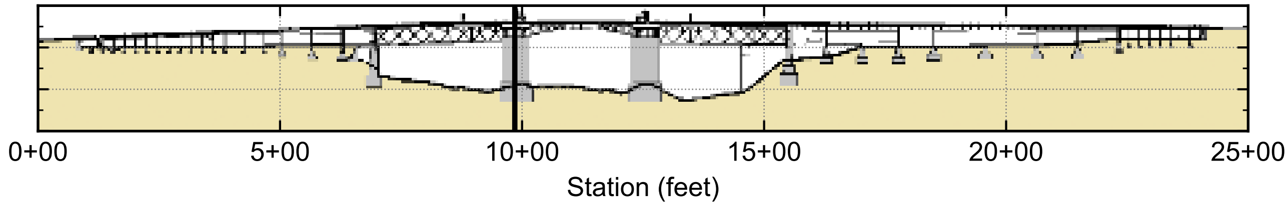
FIG. E-134

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:

1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING PIER 2

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FIG. E-135

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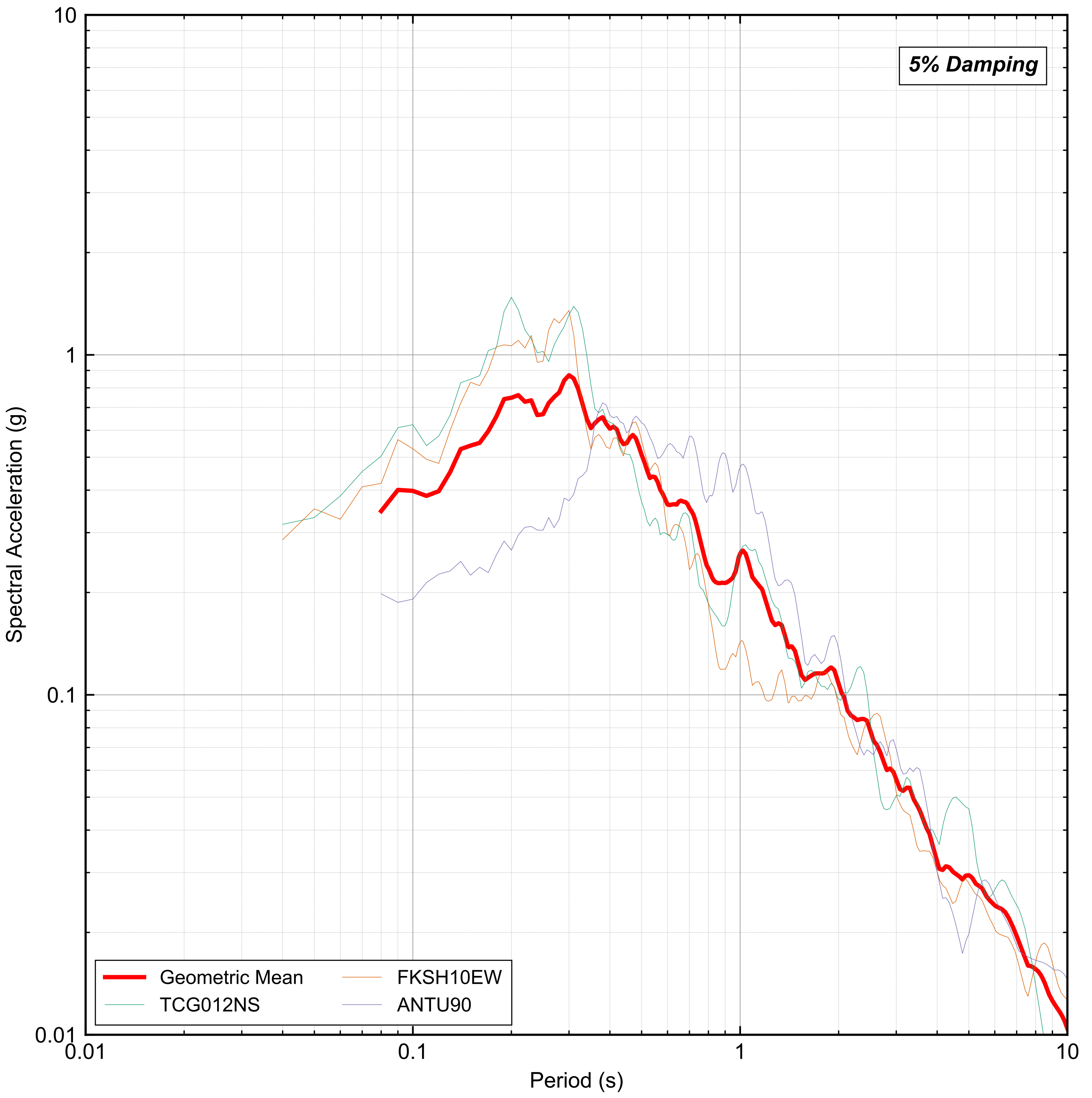

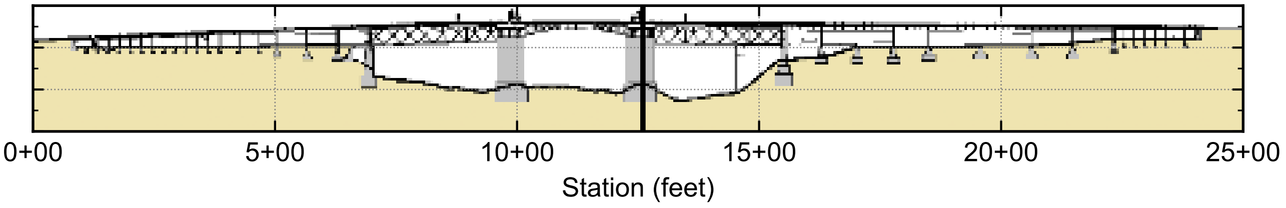

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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**GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING PIER 3**

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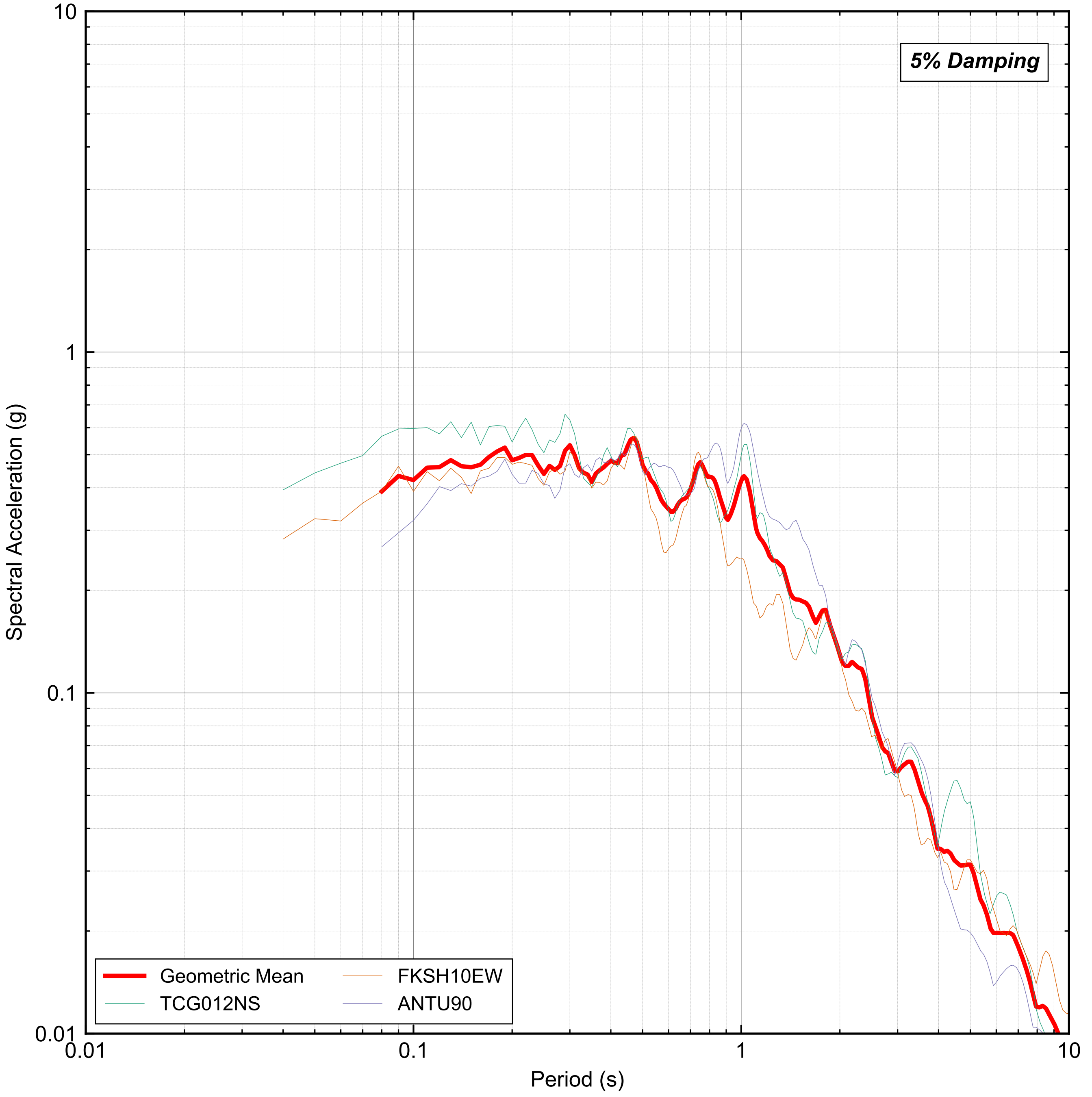
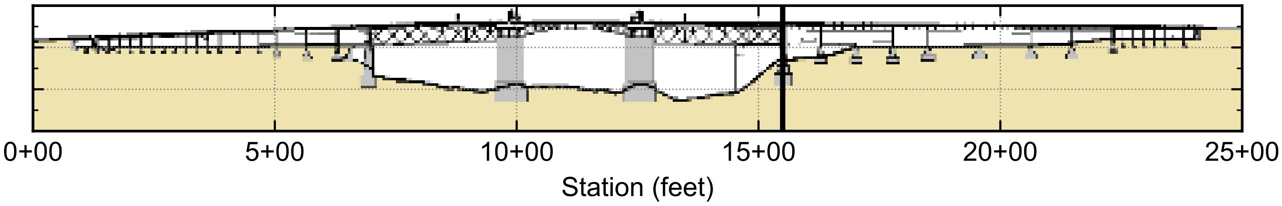
FIG. E-136

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING PIER 4

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FIG. E-137

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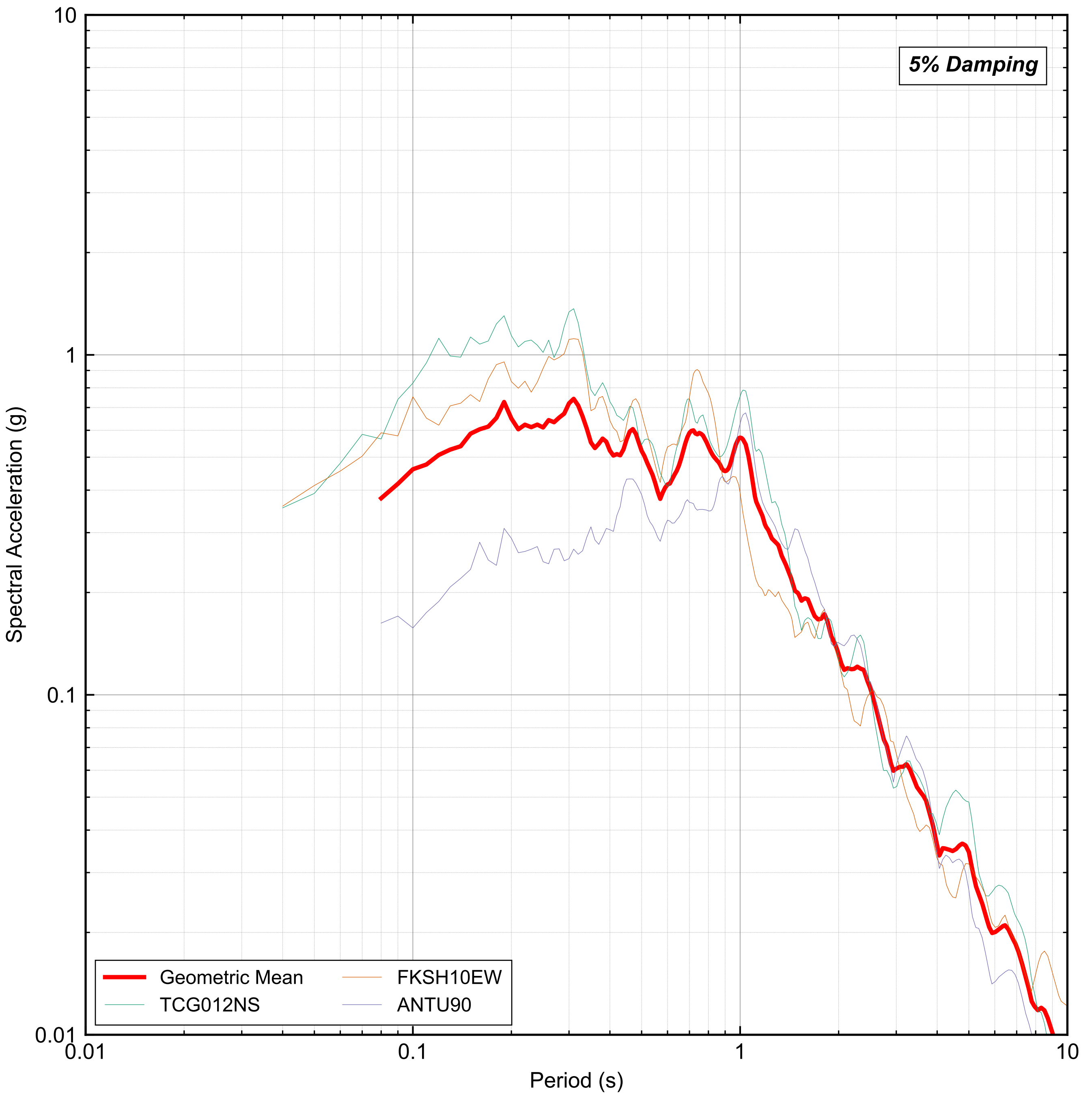
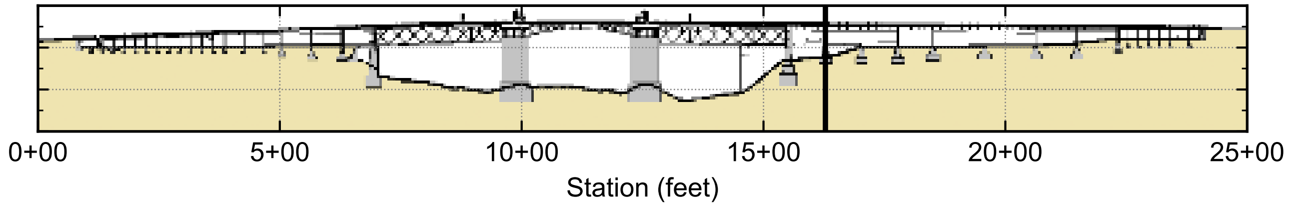
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 21

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FIG. E-138

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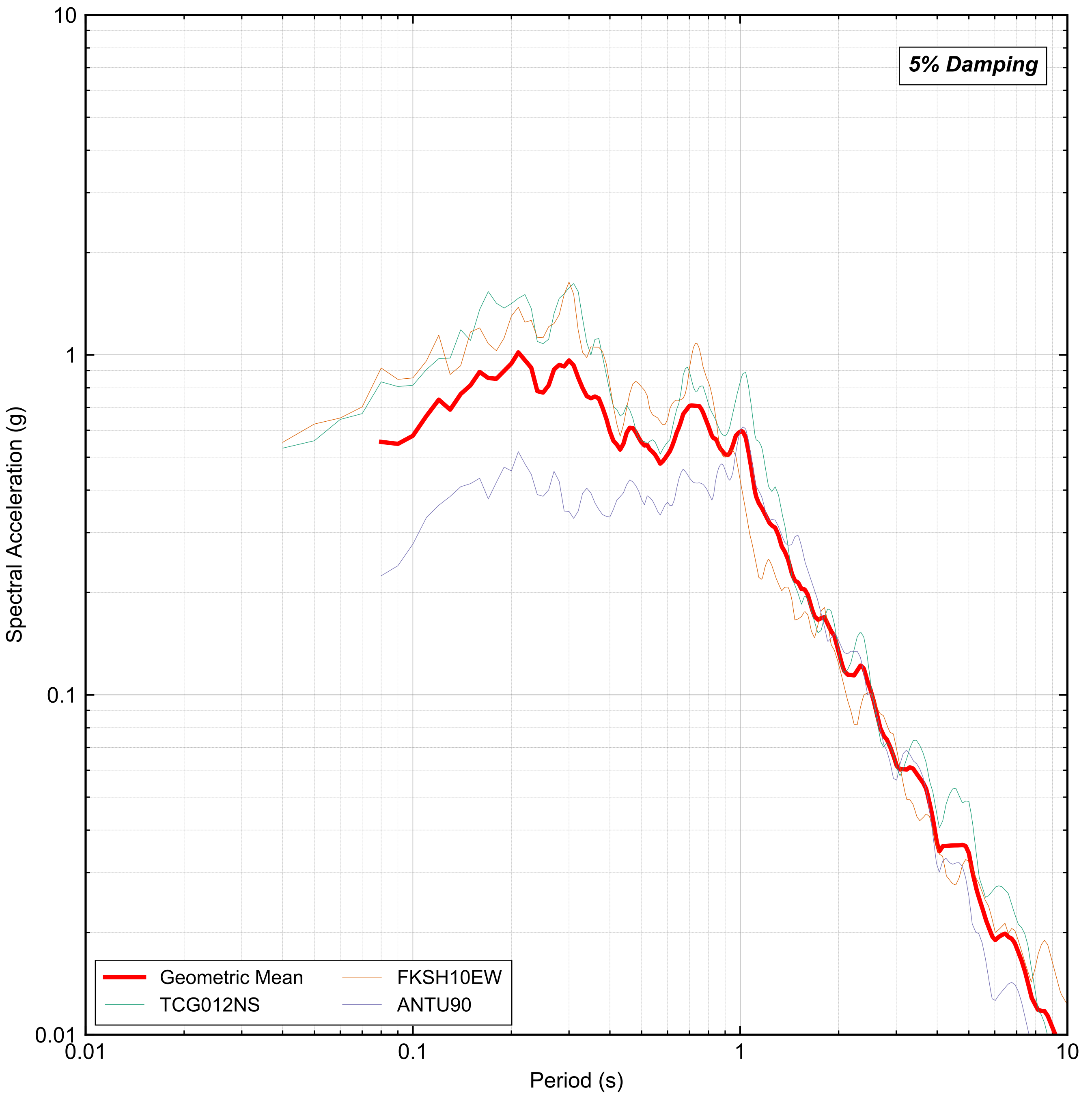

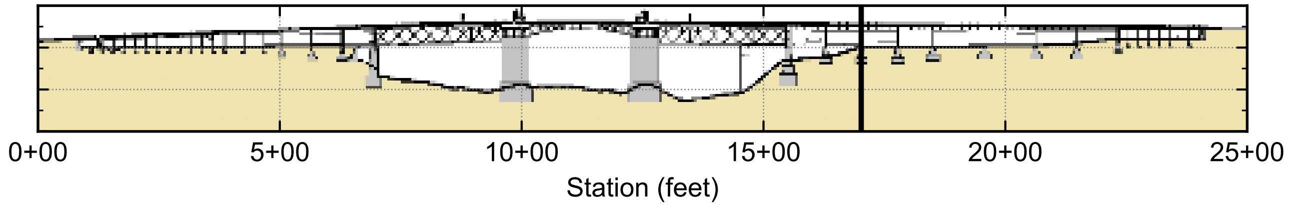

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

**GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 22**

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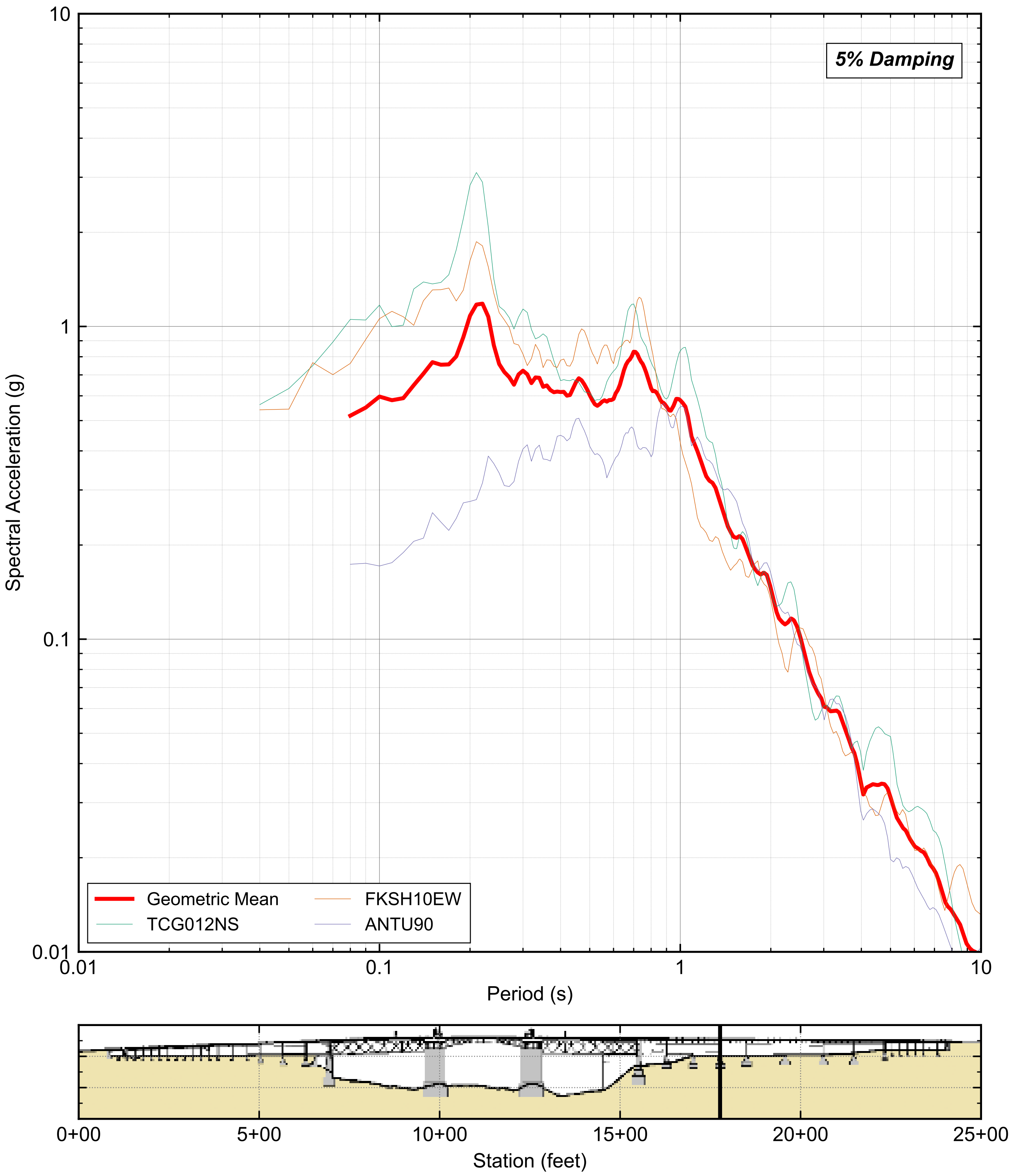
FIG. E-139

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 23

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FIG. E-140

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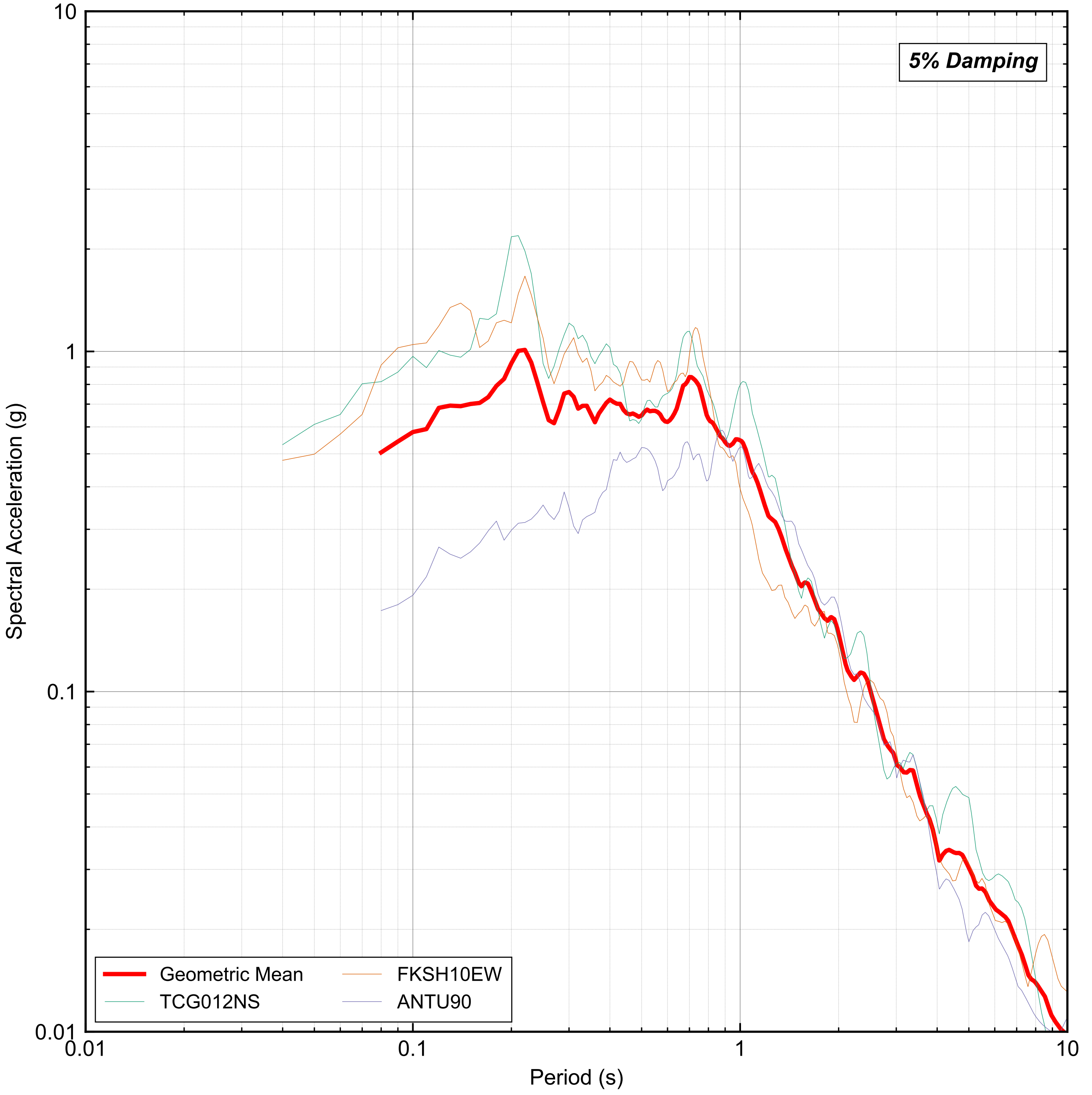

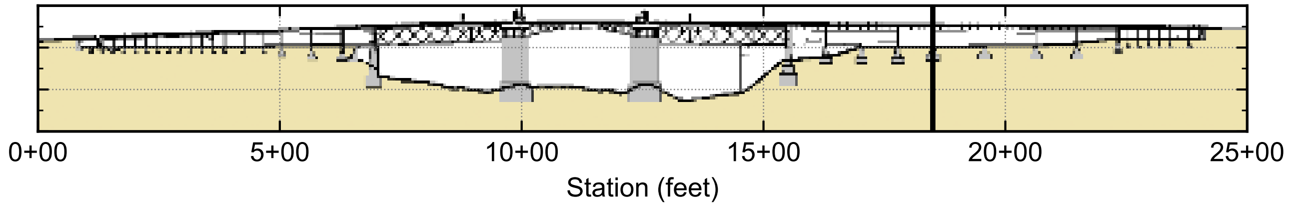

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 24

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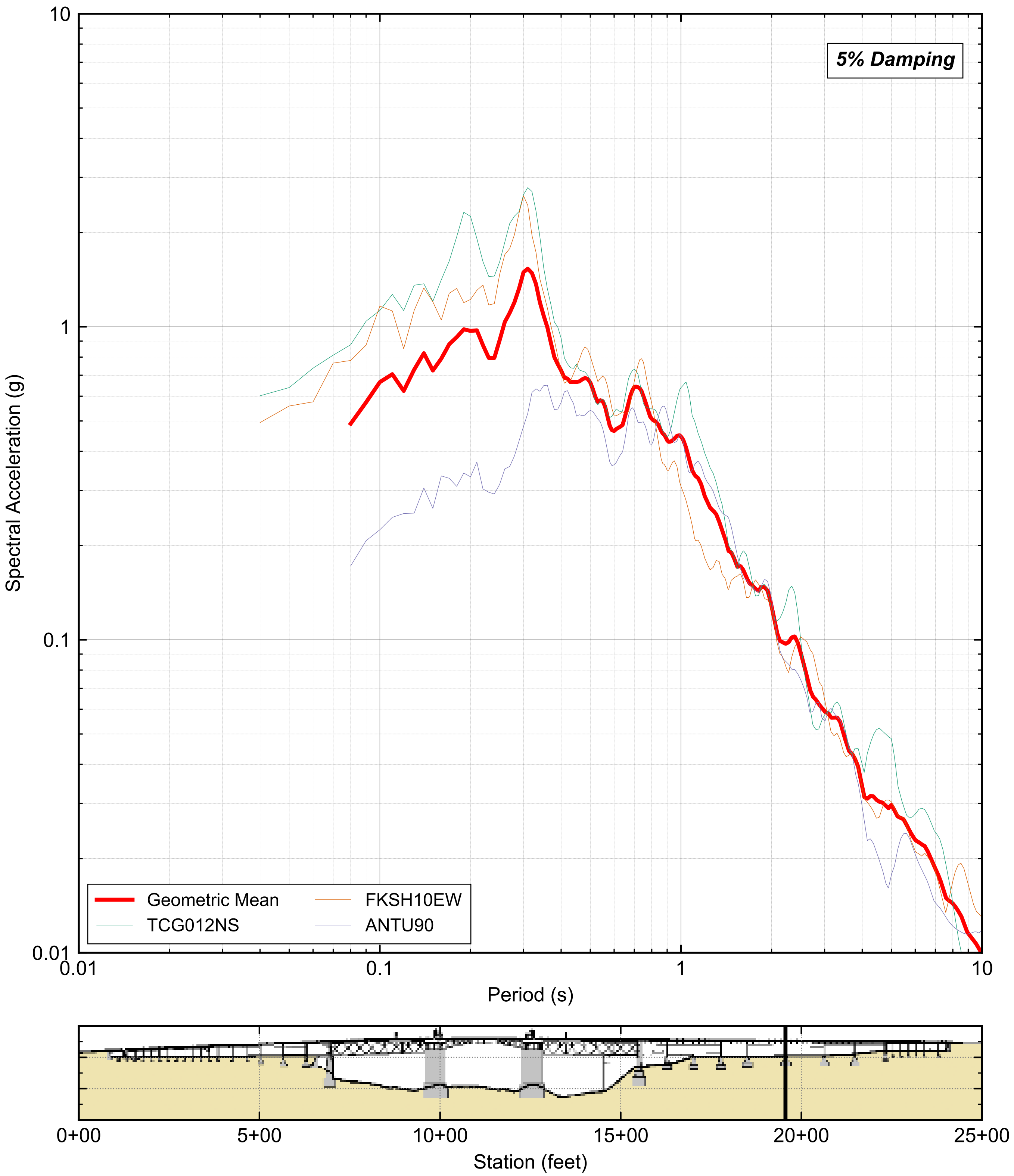
FIG. E-141

ver:0.8.5.post0.dev0+g414b689, by:JNB



Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 25

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FIG. E-142

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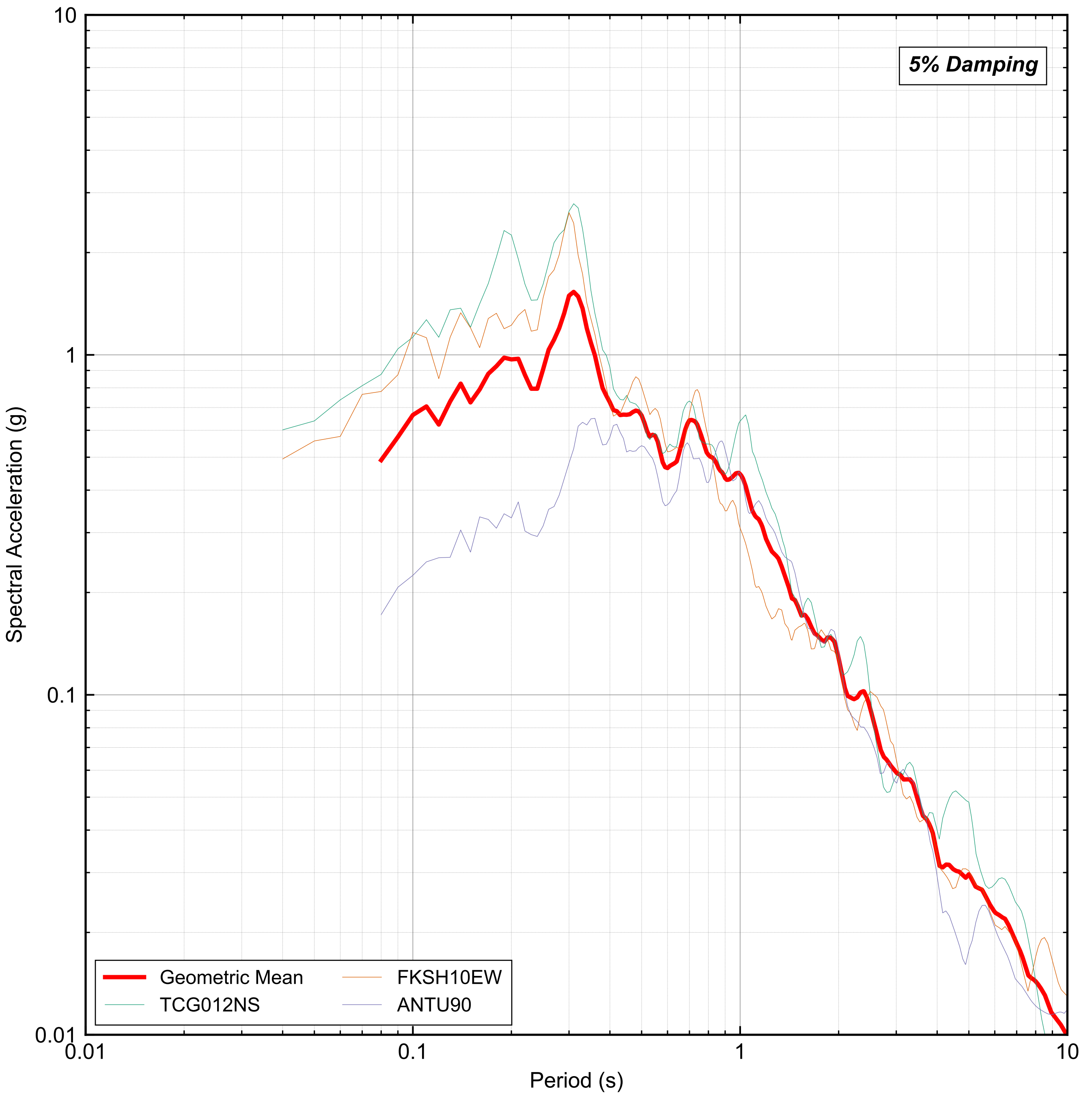

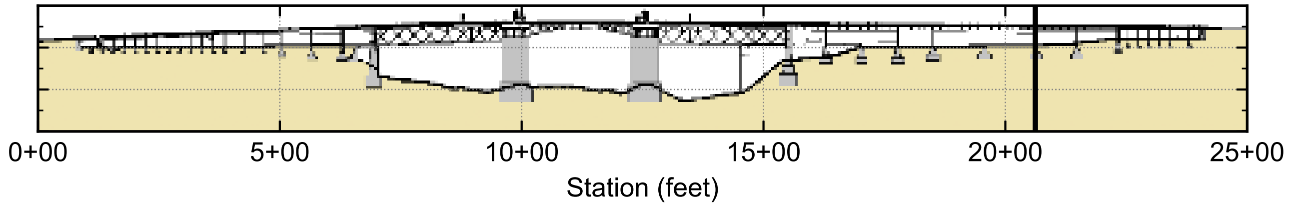

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 26

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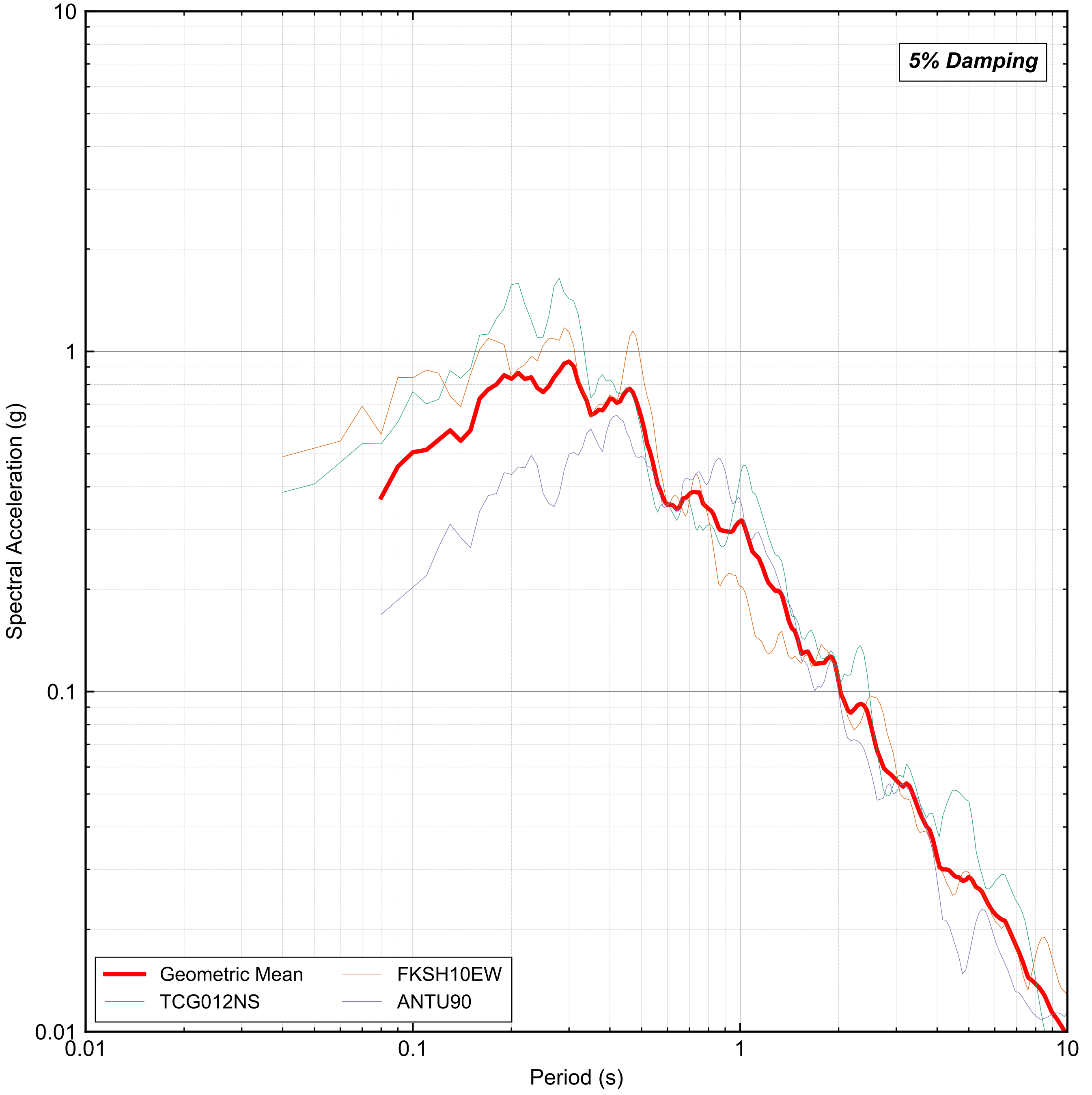
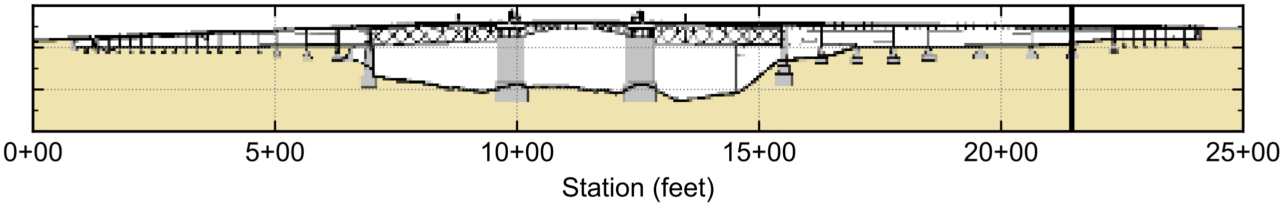
FIG. E-143

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 27

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FIG. E-144

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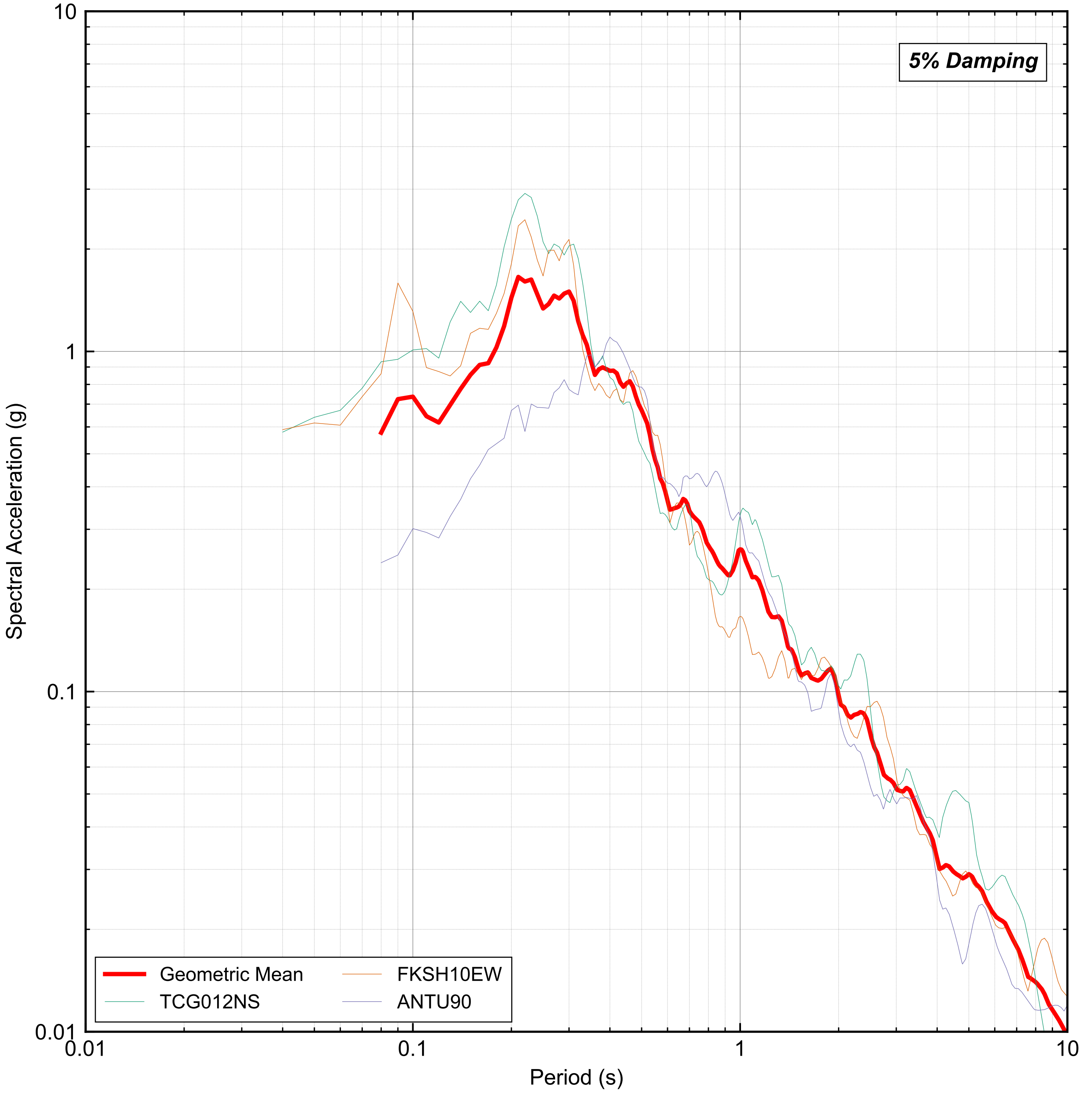
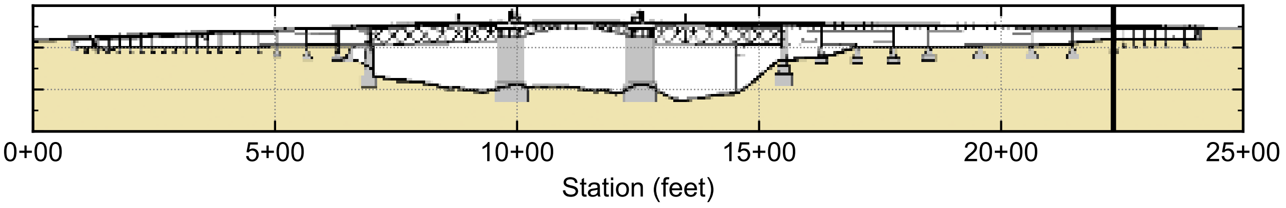
ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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**GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 28**

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FIG. E-145

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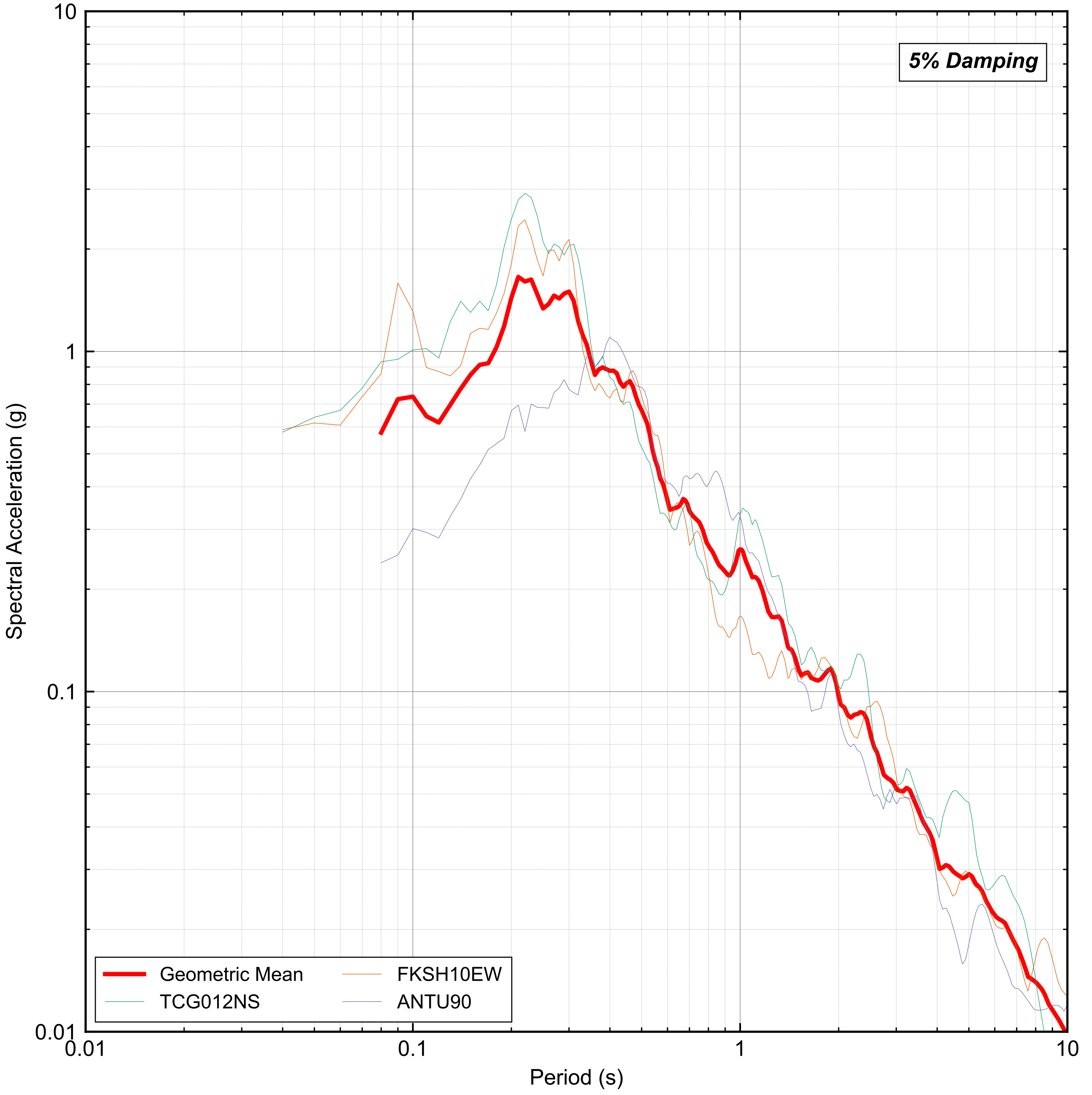

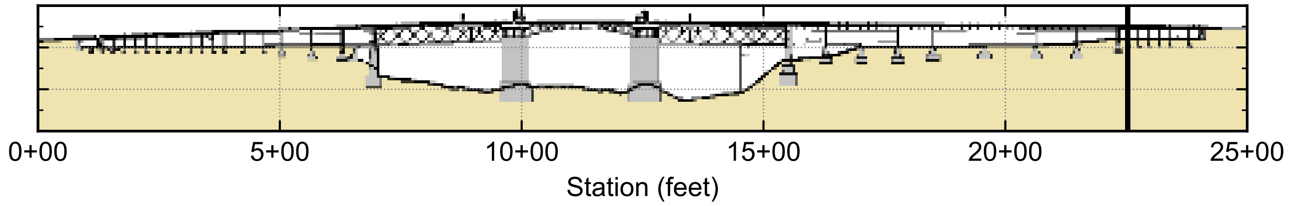

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 29

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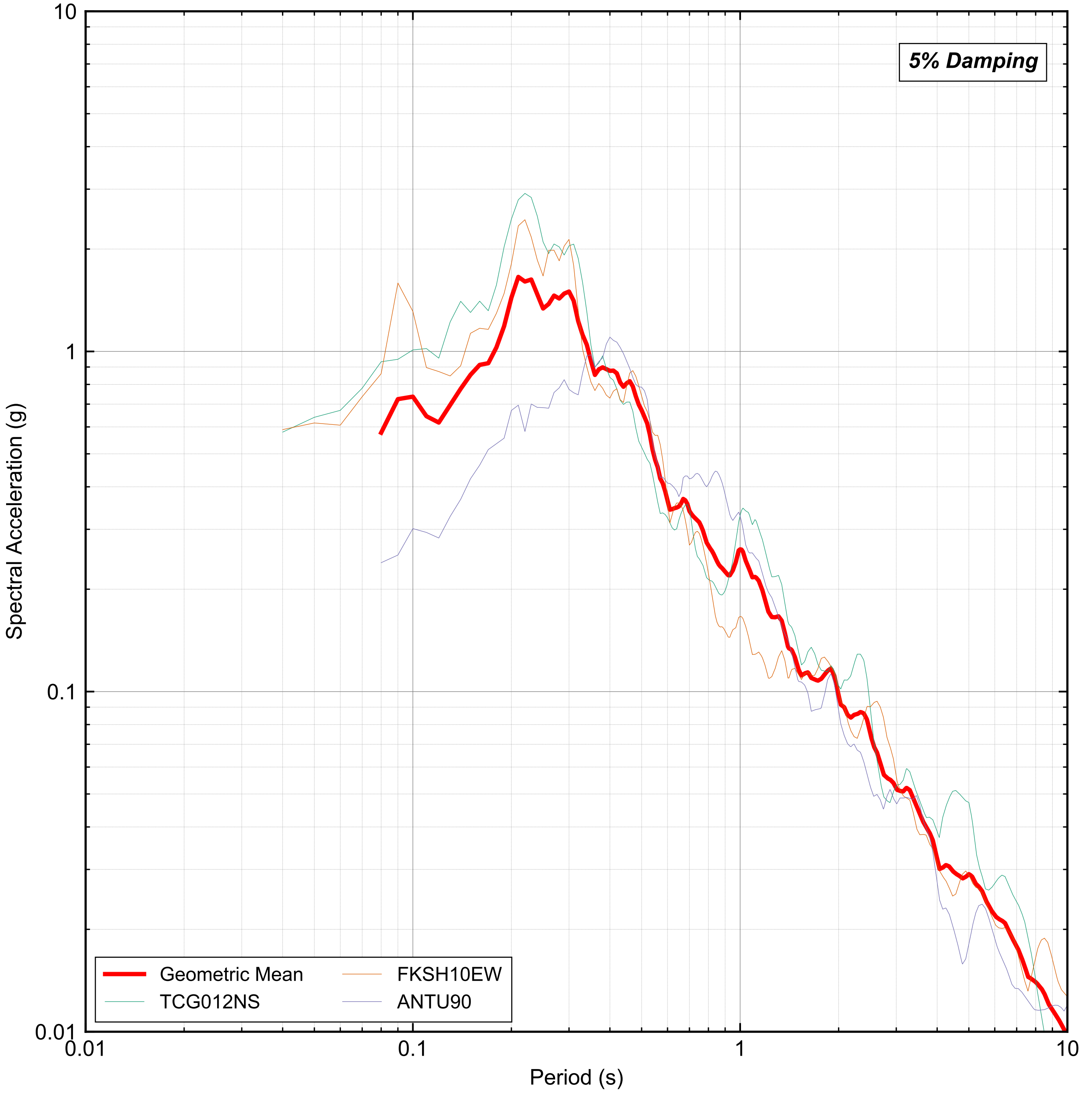
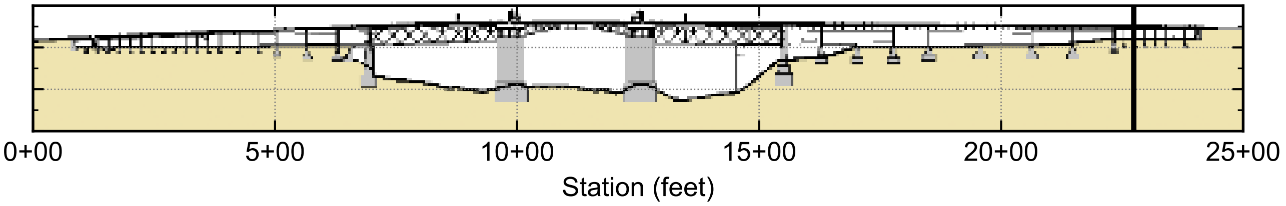
FIG. E-146





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 30

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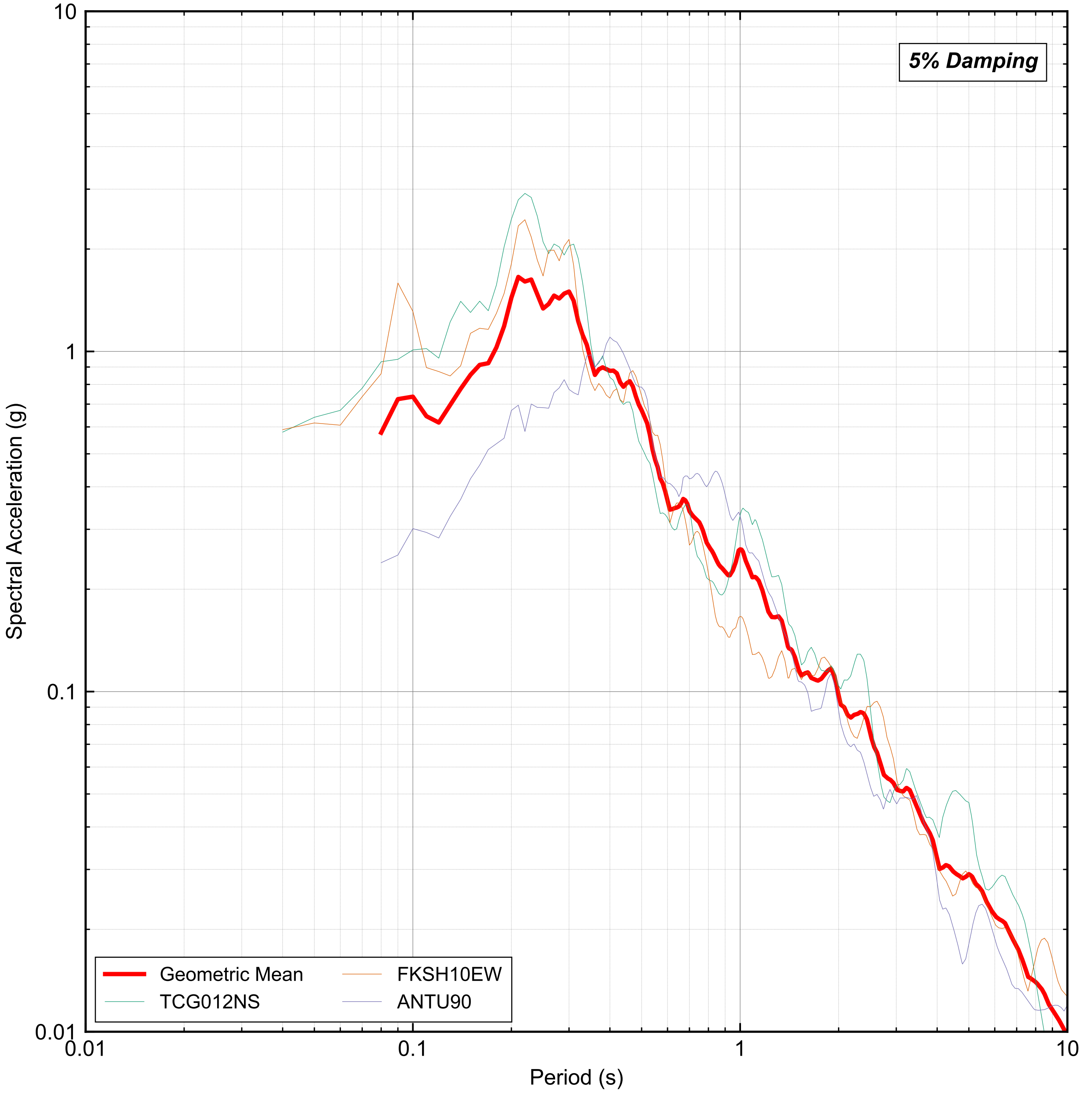
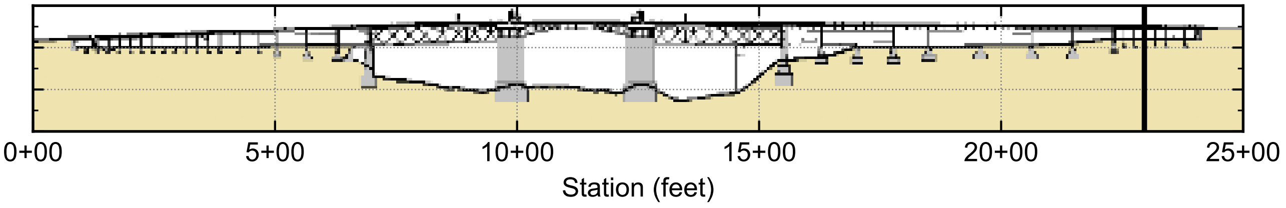
FIG. E-147





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 31

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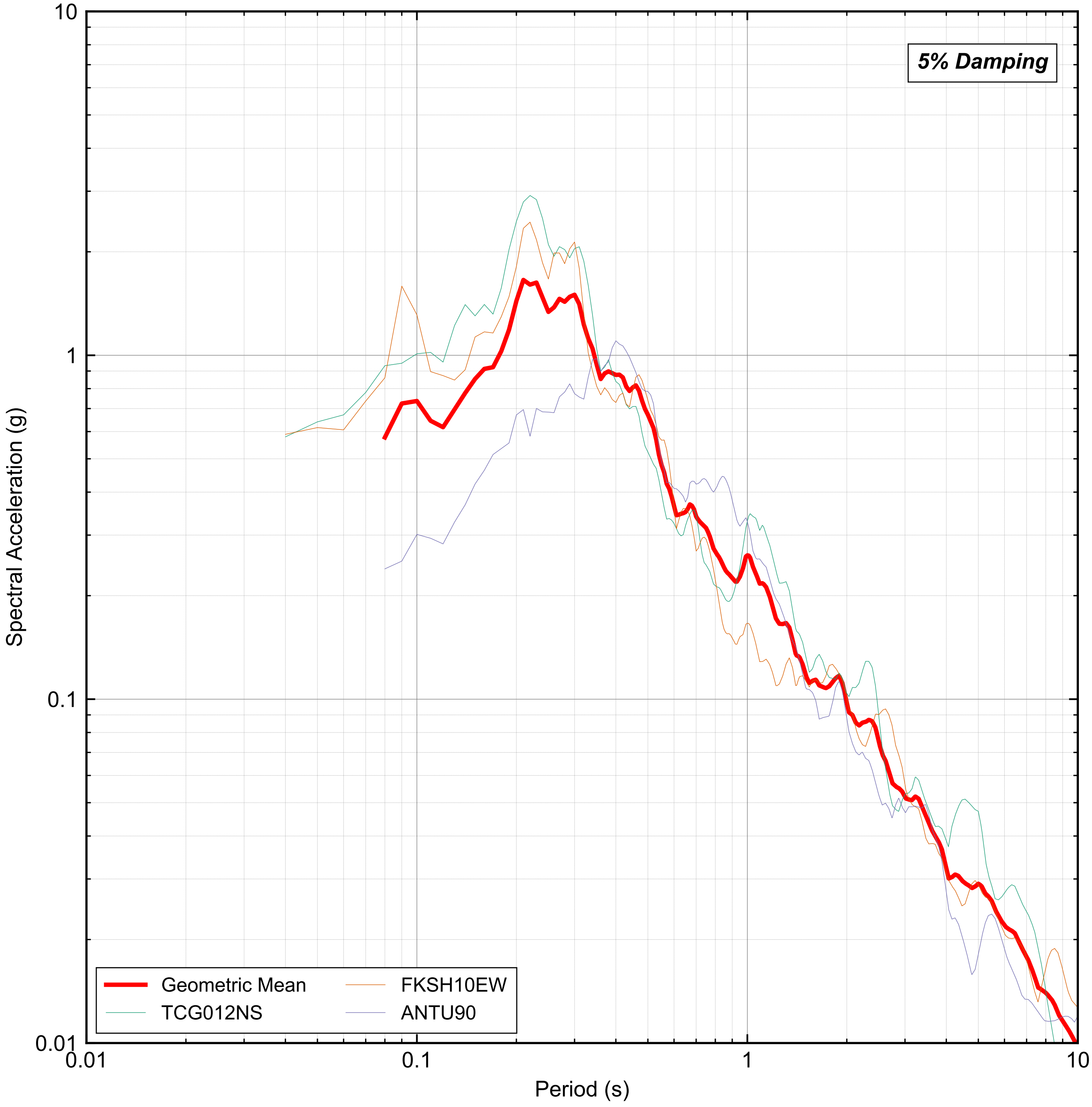
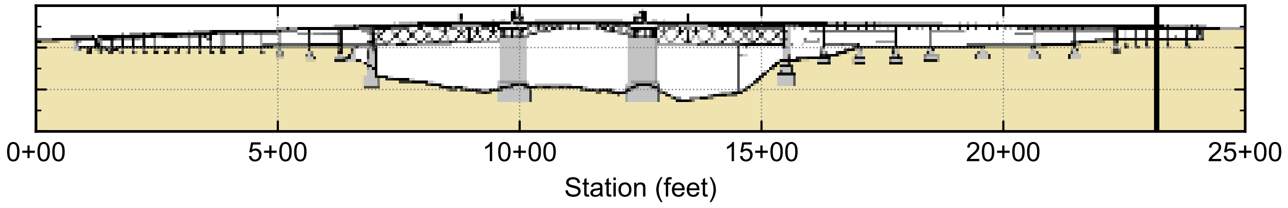
FIG. E-148

ver:0.8.5.post0.dev0+g414b689, by:JNB





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 32

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FIG. E-149

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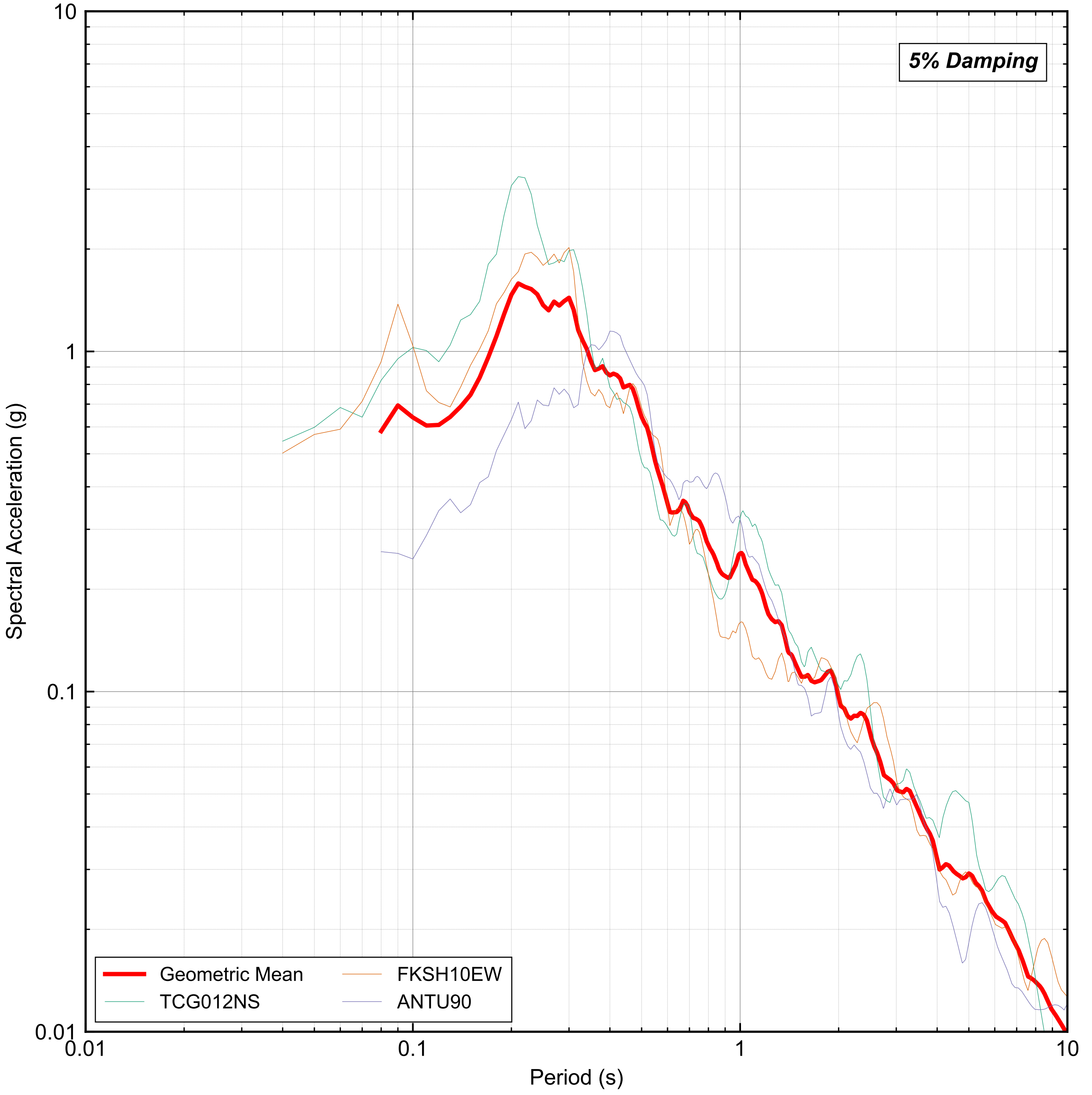

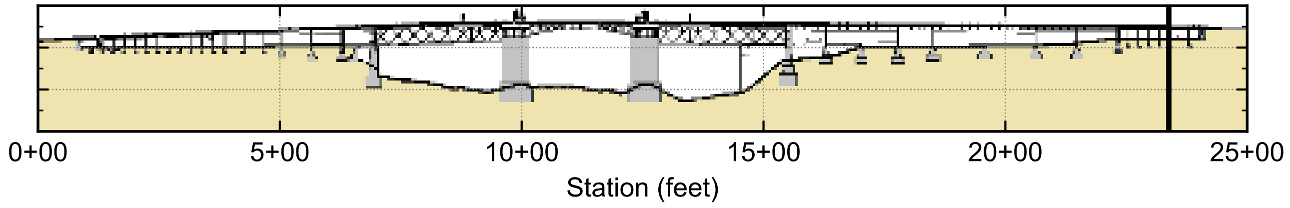

Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 33

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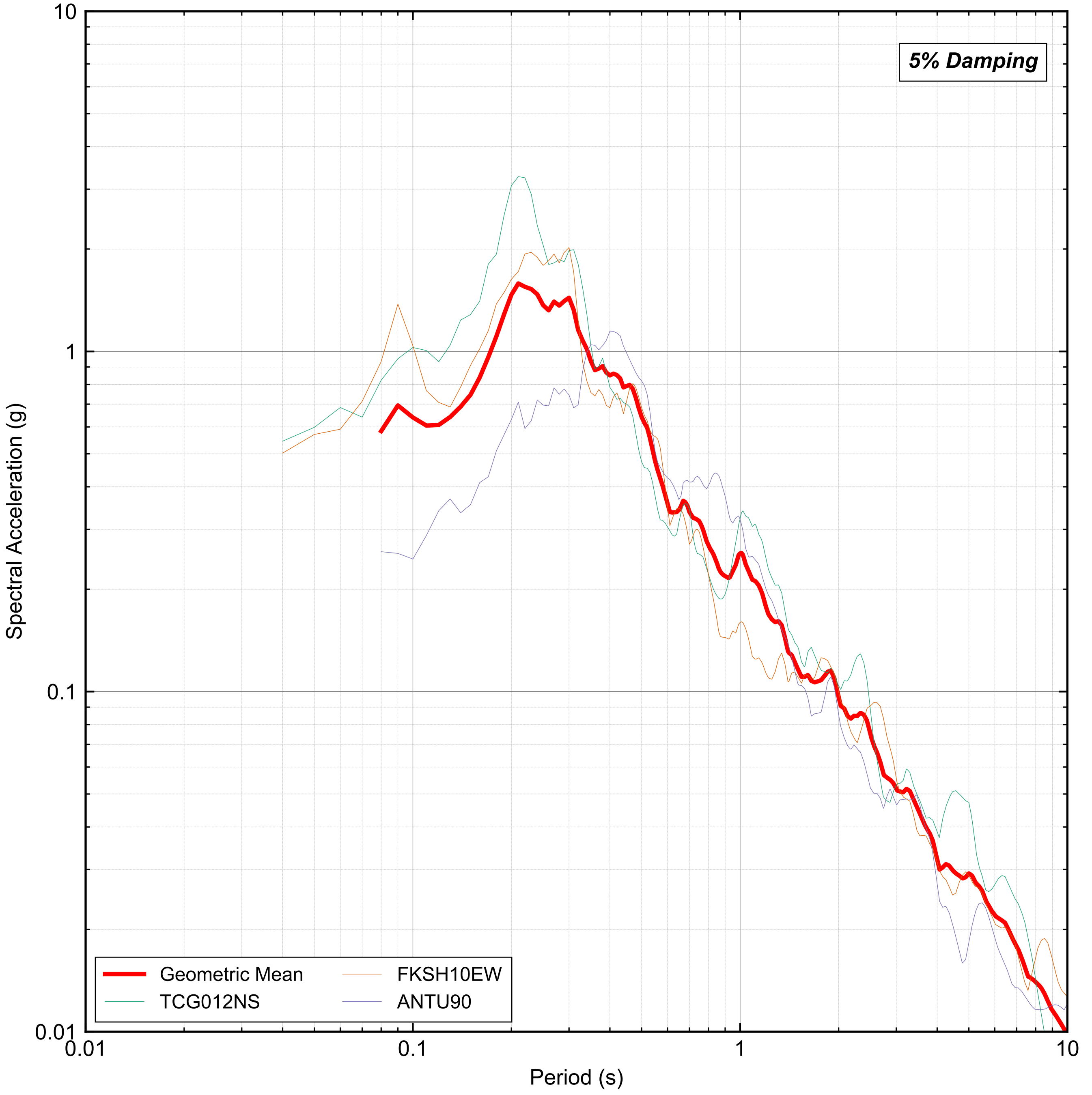
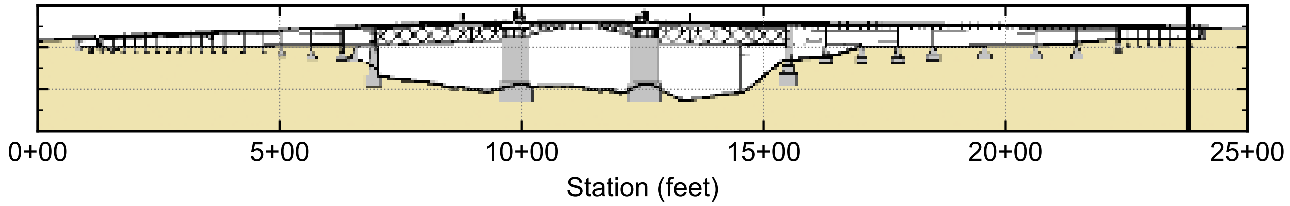
FIG. E-150





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 34

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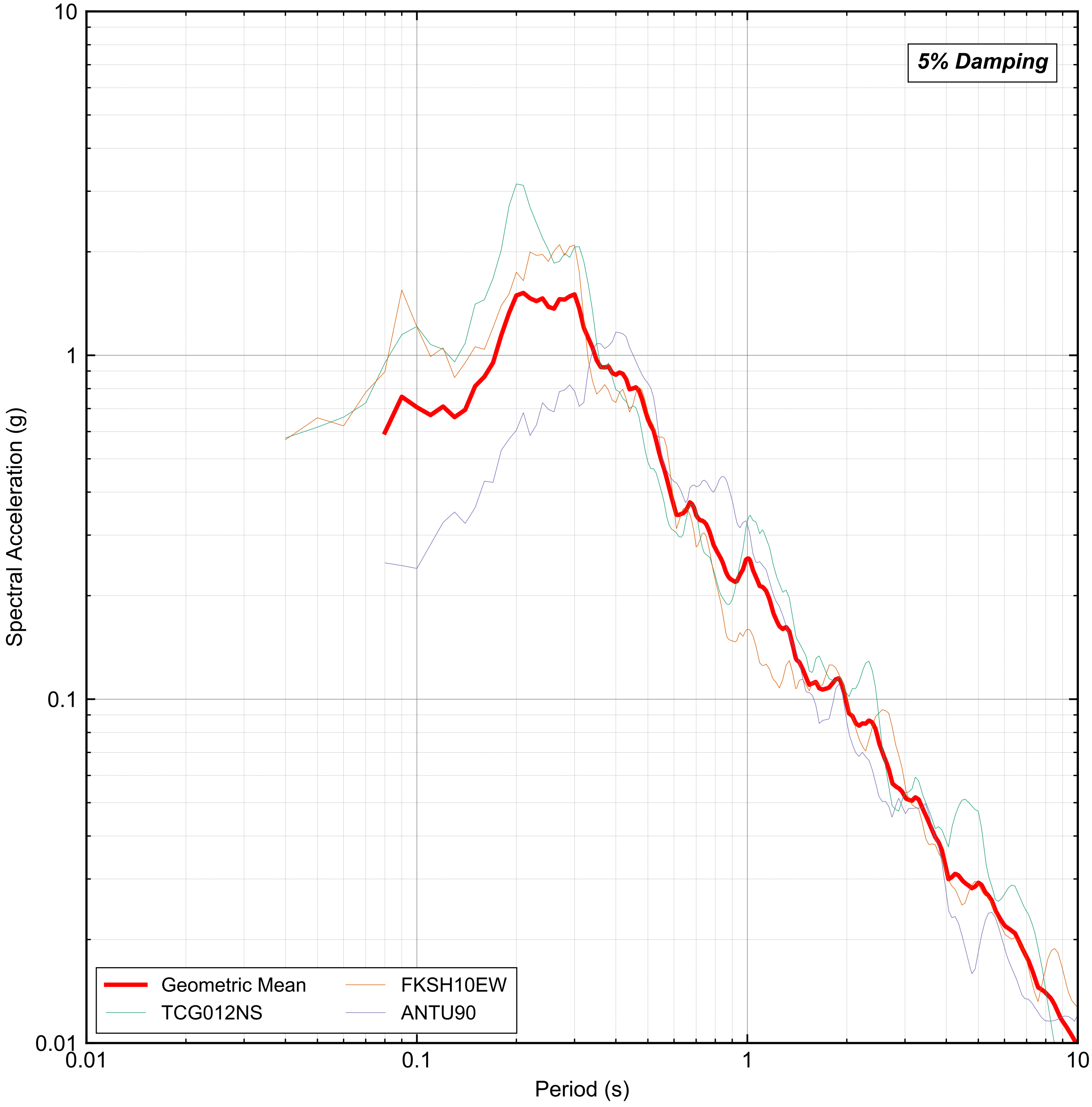
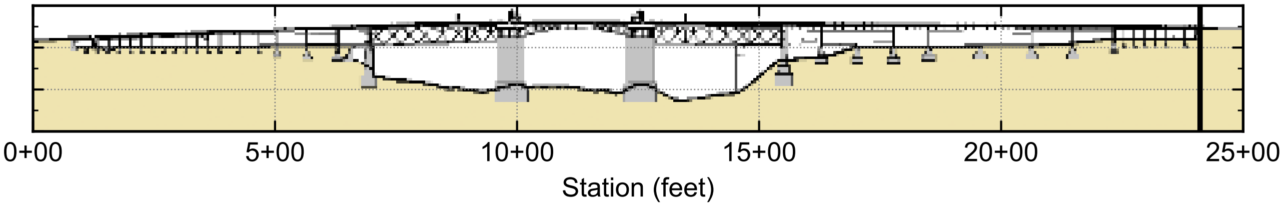
FIG. E-151





Notes:
1. See the main text for a description of the ground motion time histories.
2. g = gravitational acceleration
   s = seconds

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Portland, Oregon

GROUND SURFACE RESPONSE
LIMITED OPERATION (CSZ 1S)
EXISTING BENT 35

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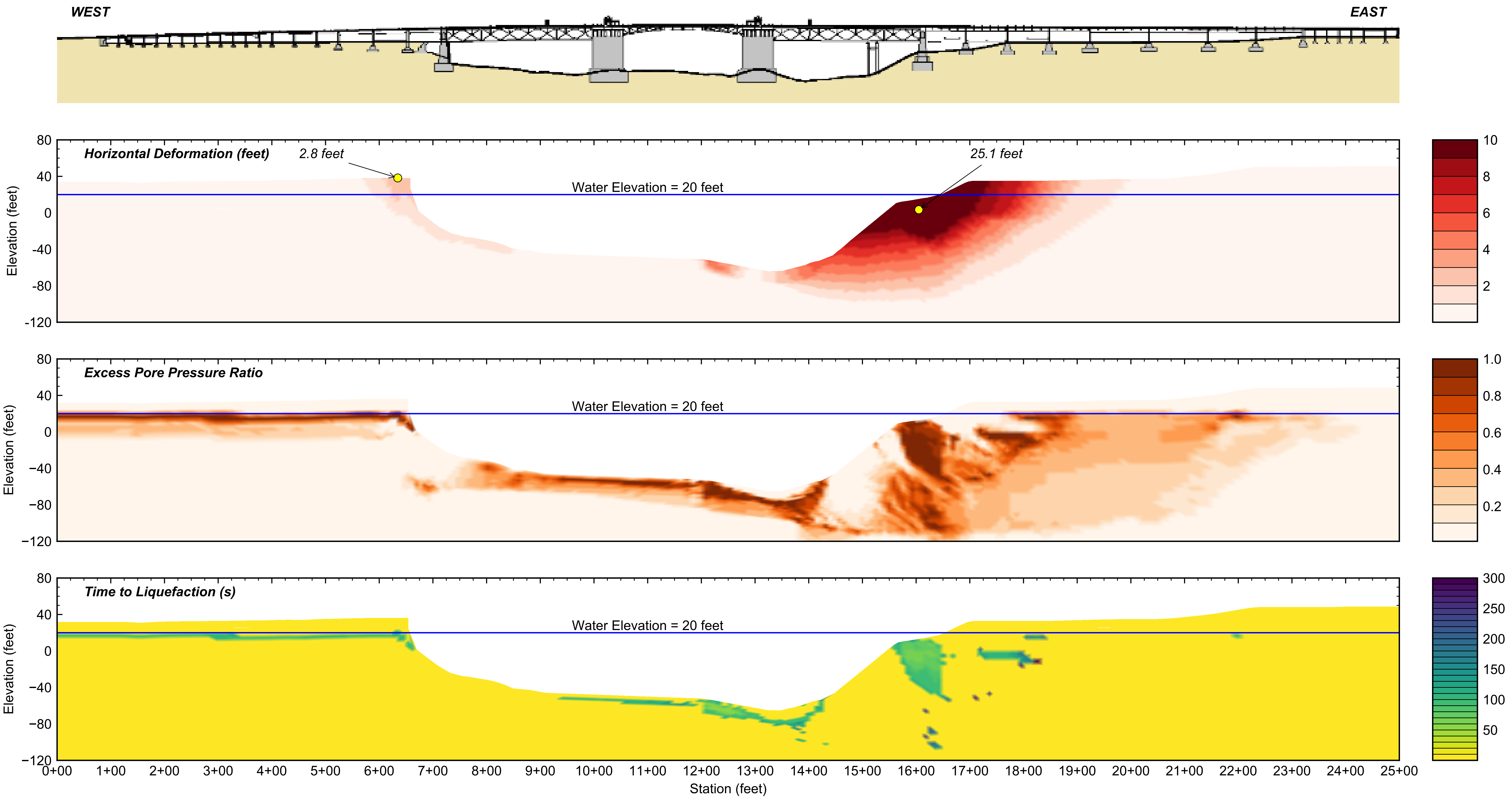
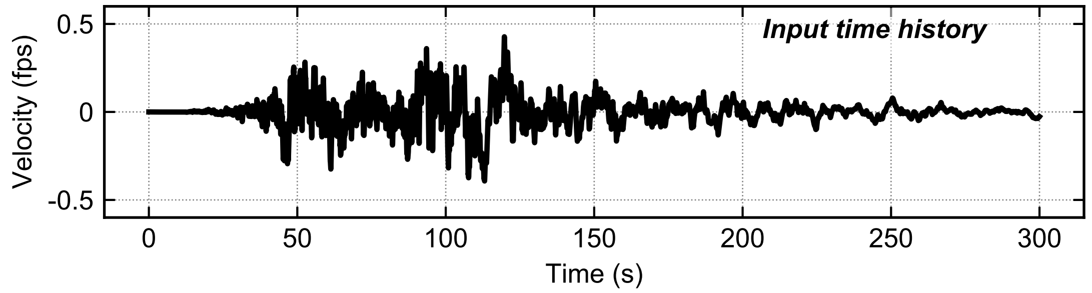
FIG. E-152

ver:0.8.5.post0.dev0+g414b689 by:JNB





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MODEL RESPONSE
FULL OPERATION
TOHOKU (AKT023EW)

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FIG. E-153

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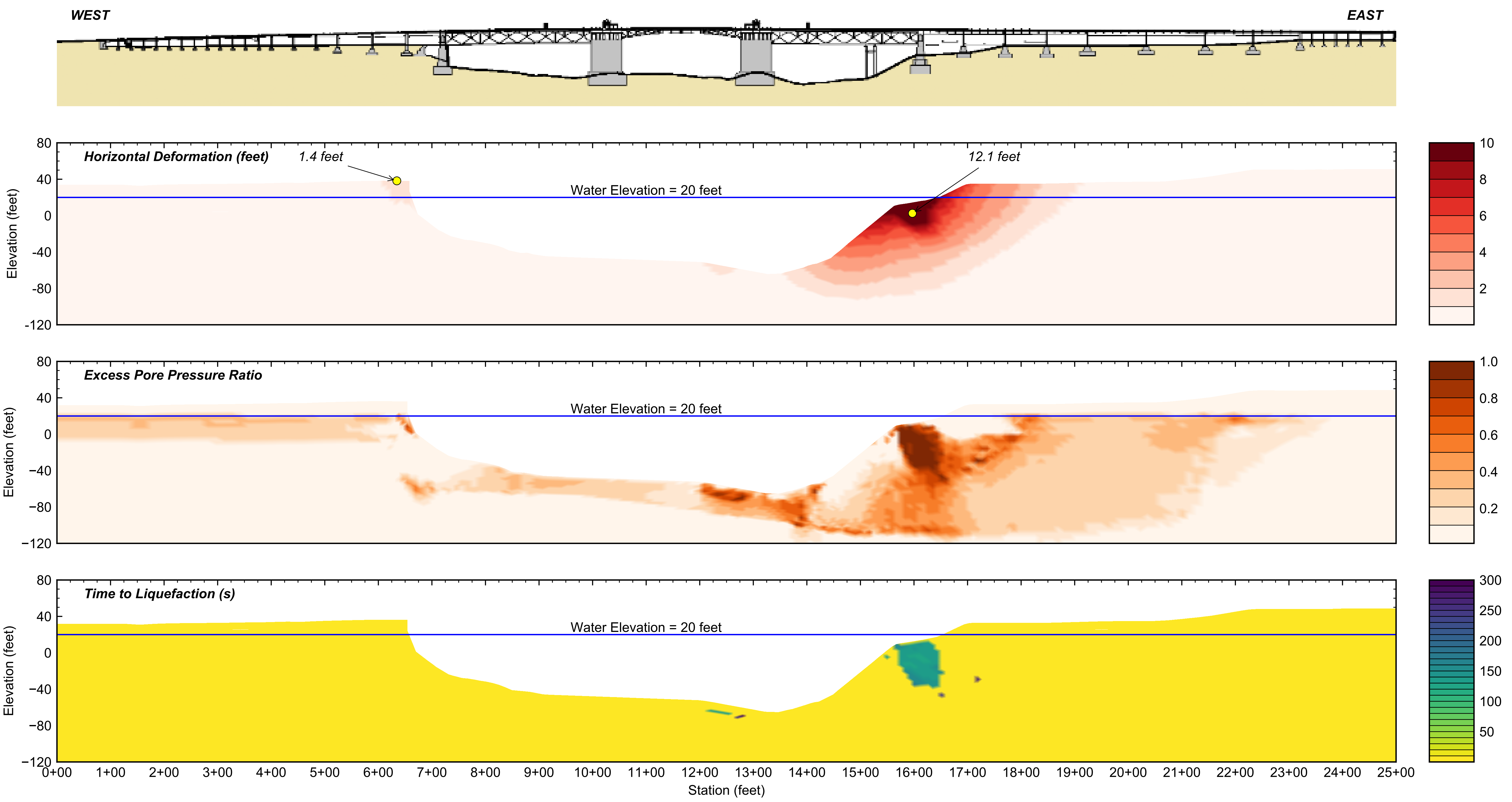
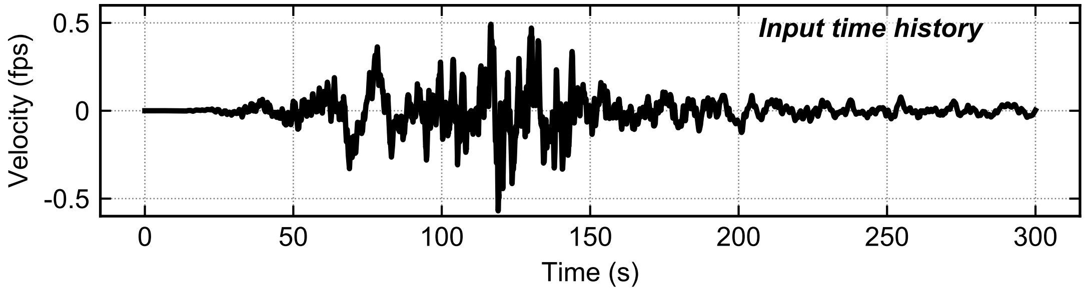
ver:0.8.5.post0.dev0+g414b689 by:JNB





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MODEL RESPONSE
FULL OPERATION
TOHOKU (FKSH05EW)

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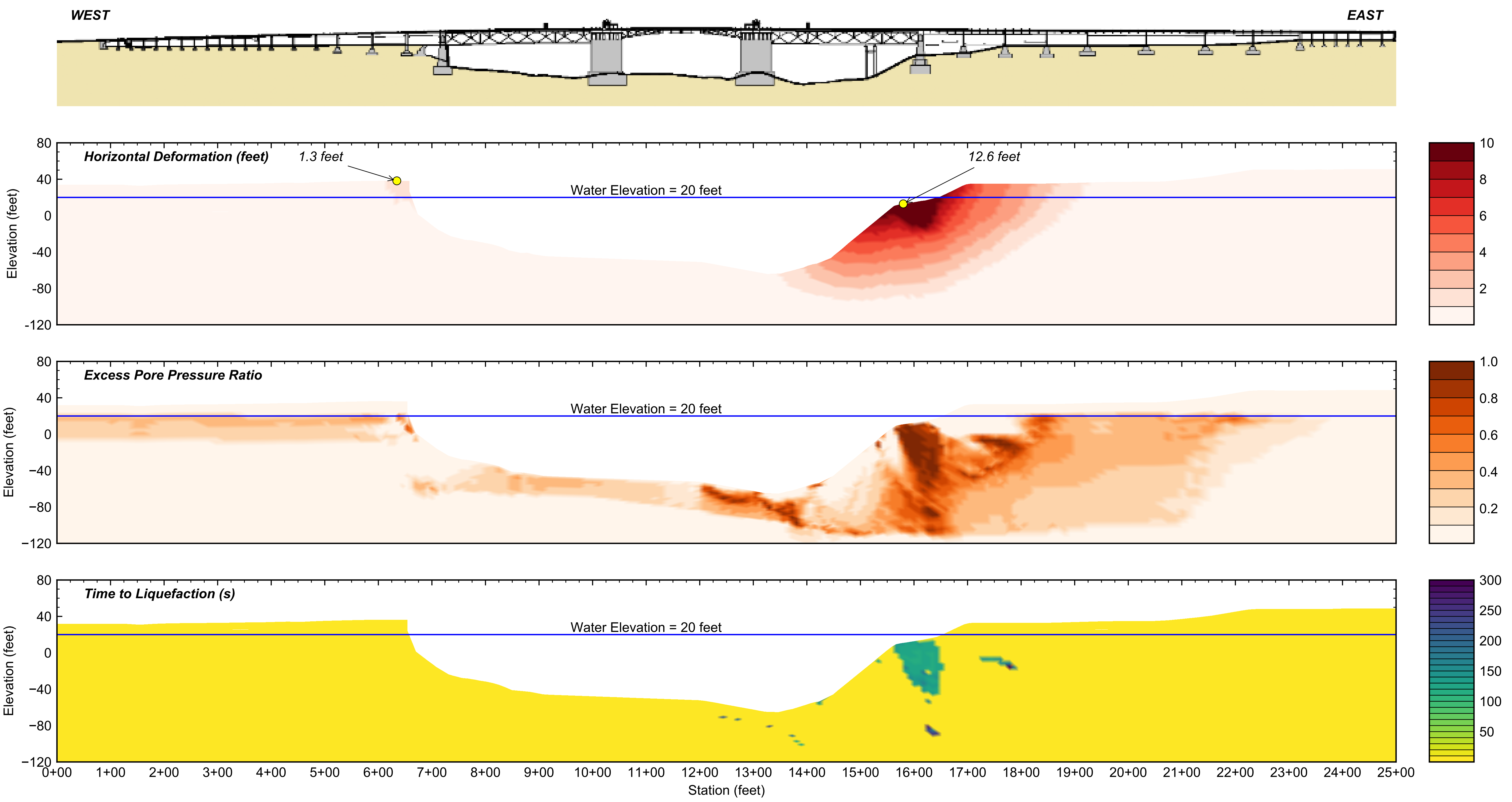
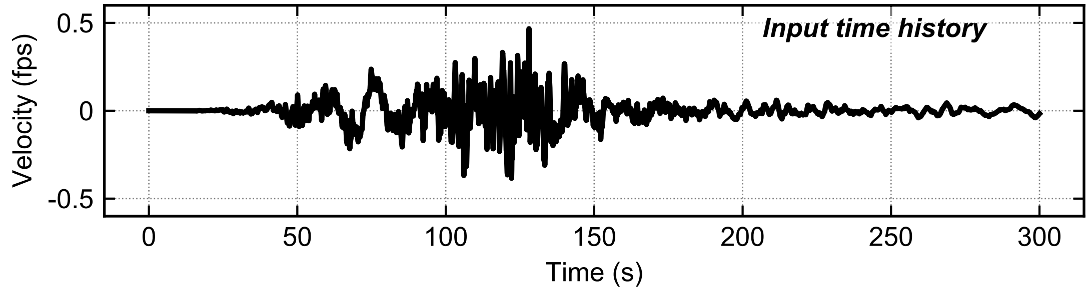
FIG. E-154

ver:0.8.5.post0.dev0+g414b689 by:JNB





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Portland, Oregon

MODEL RESPONSE
FULL OPERATION
TOHOKU (FKSH08EW)

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FIG. E-155

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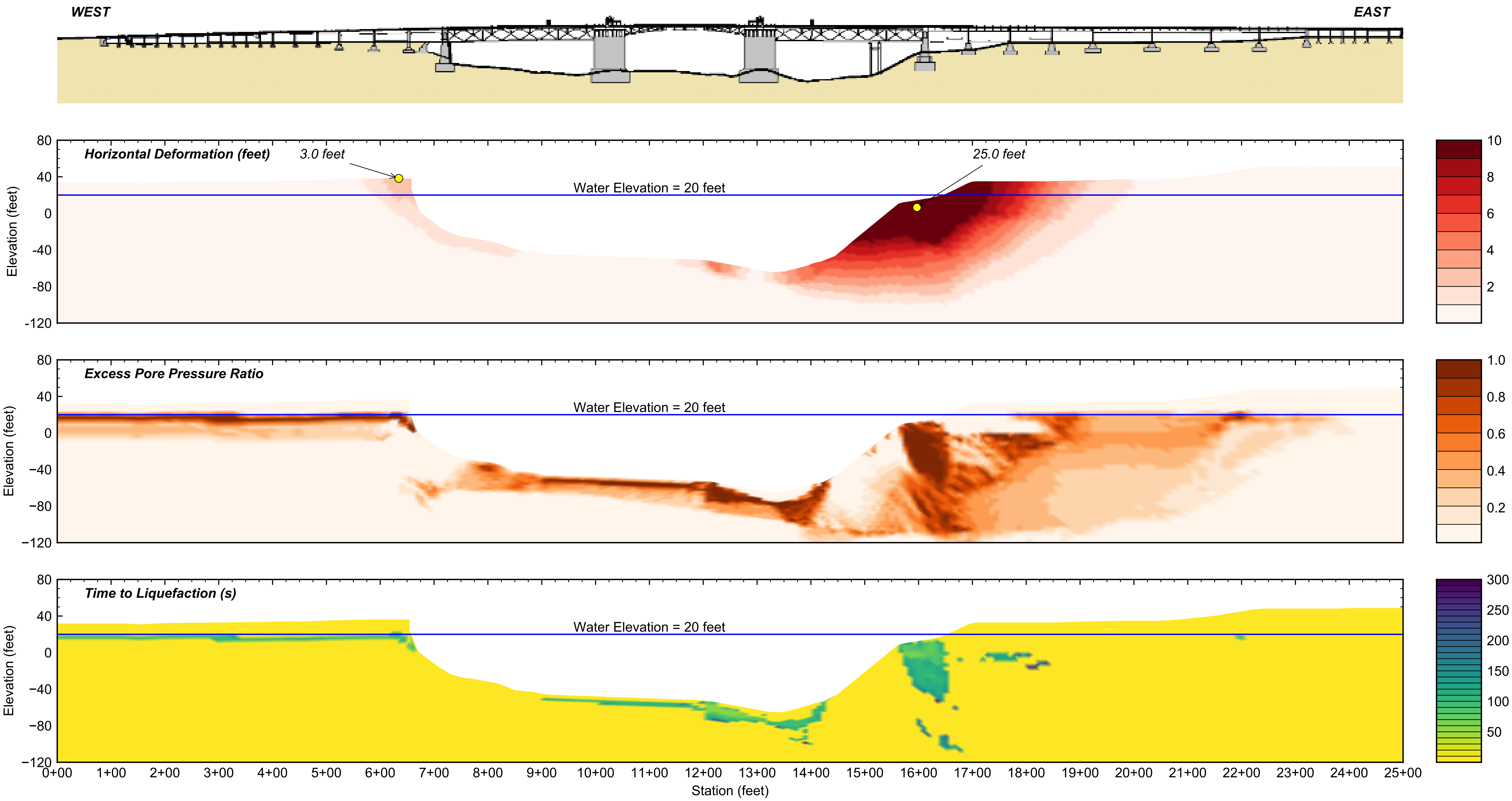
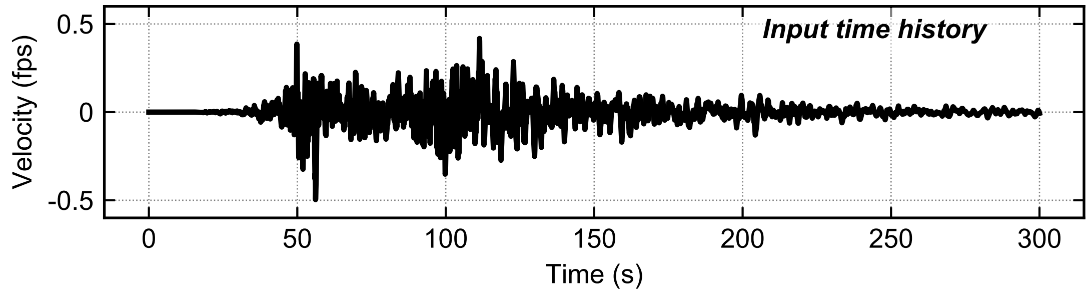
ver:0.8.5.post0.dev0+g414b689, by:JNB





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Portland, Oregon

MODEL RESPONSE
FULL OPERATION
TOHOKU (IWT011NS)

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FIG. E-156

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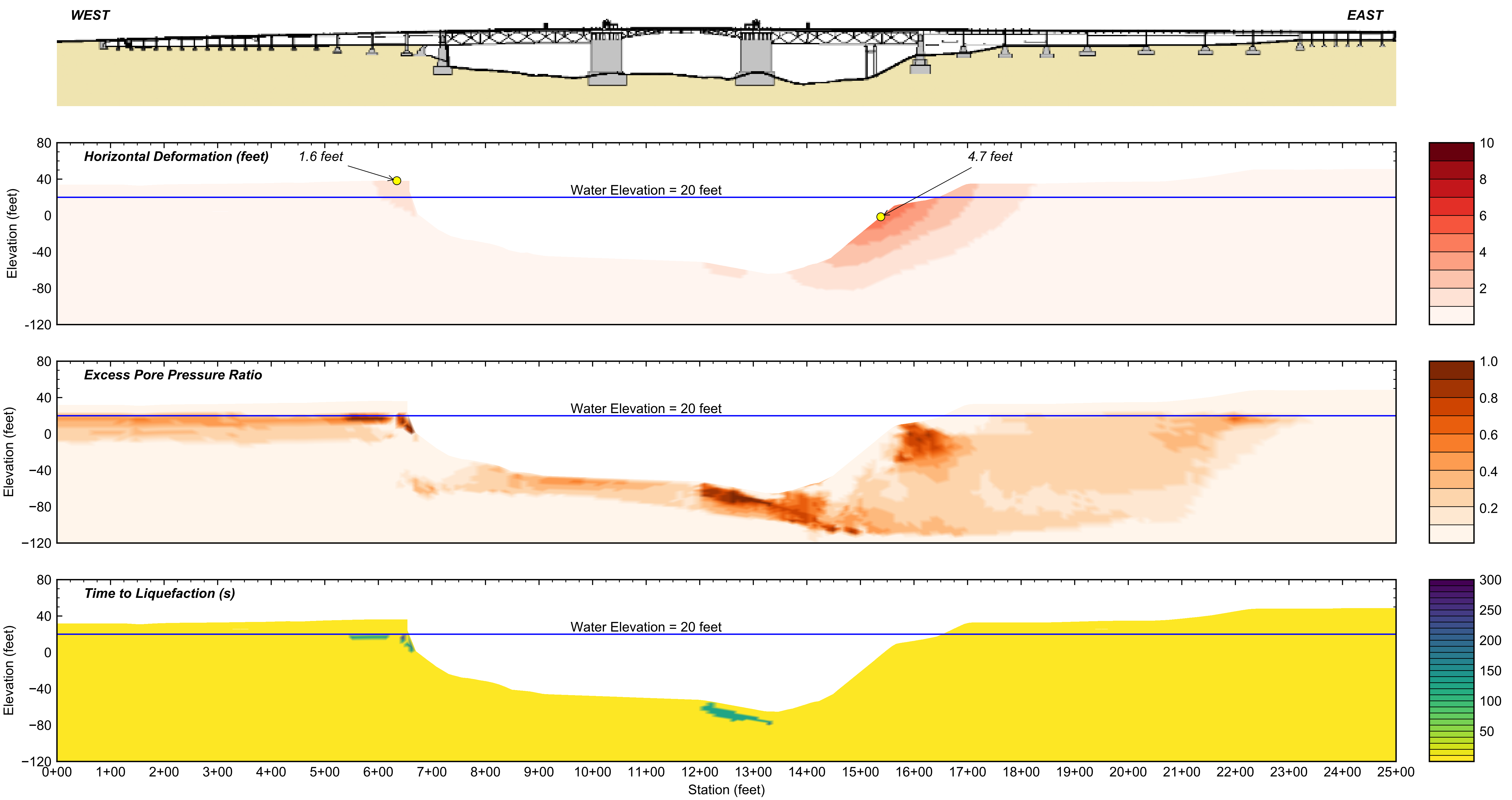
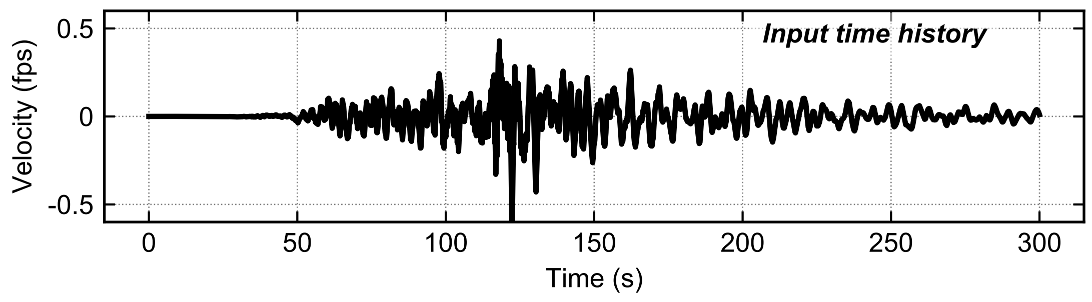
ver:0.8.5.post0.dev0+g414b689 by:JNB





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Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
FULL OPERATION
TOHOKU (TCGH12NS)

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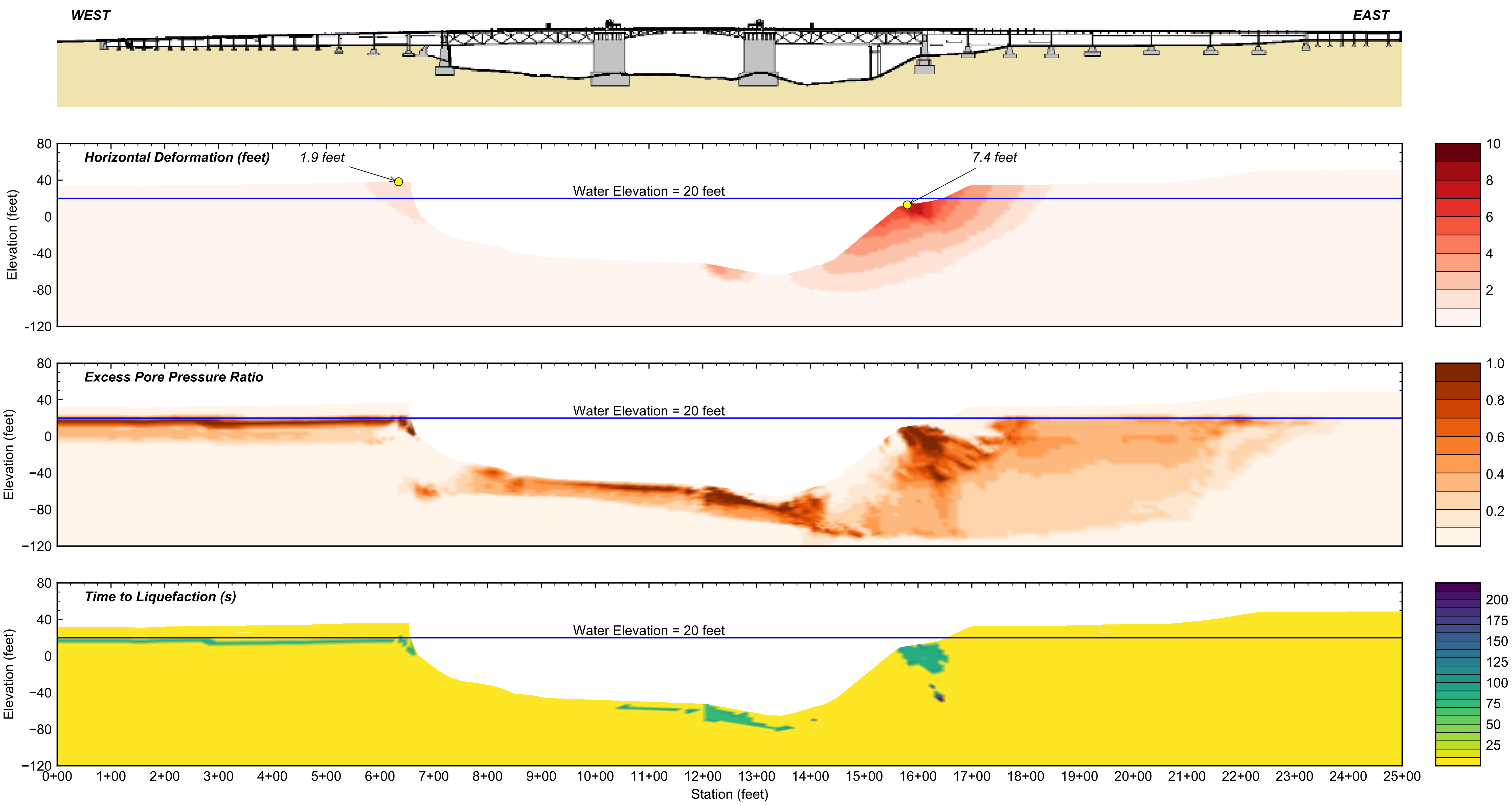
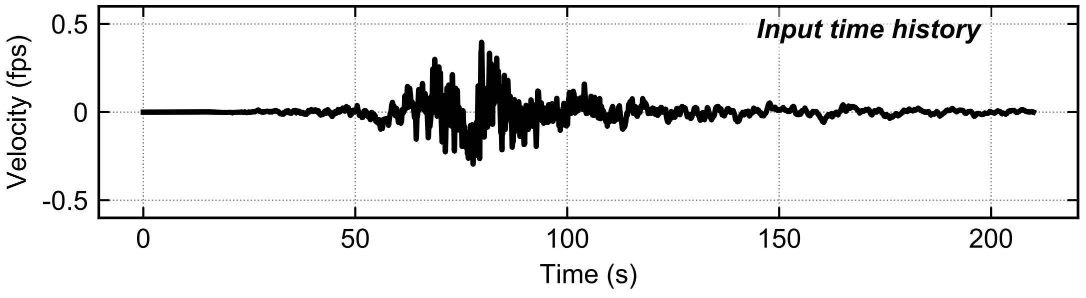
FIG. E-157

ver:0.8.5.post0.dev0+g414b689, by:pto





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MODEL RESPONSE
FULL OPERATION
MAULE (ANTU90)

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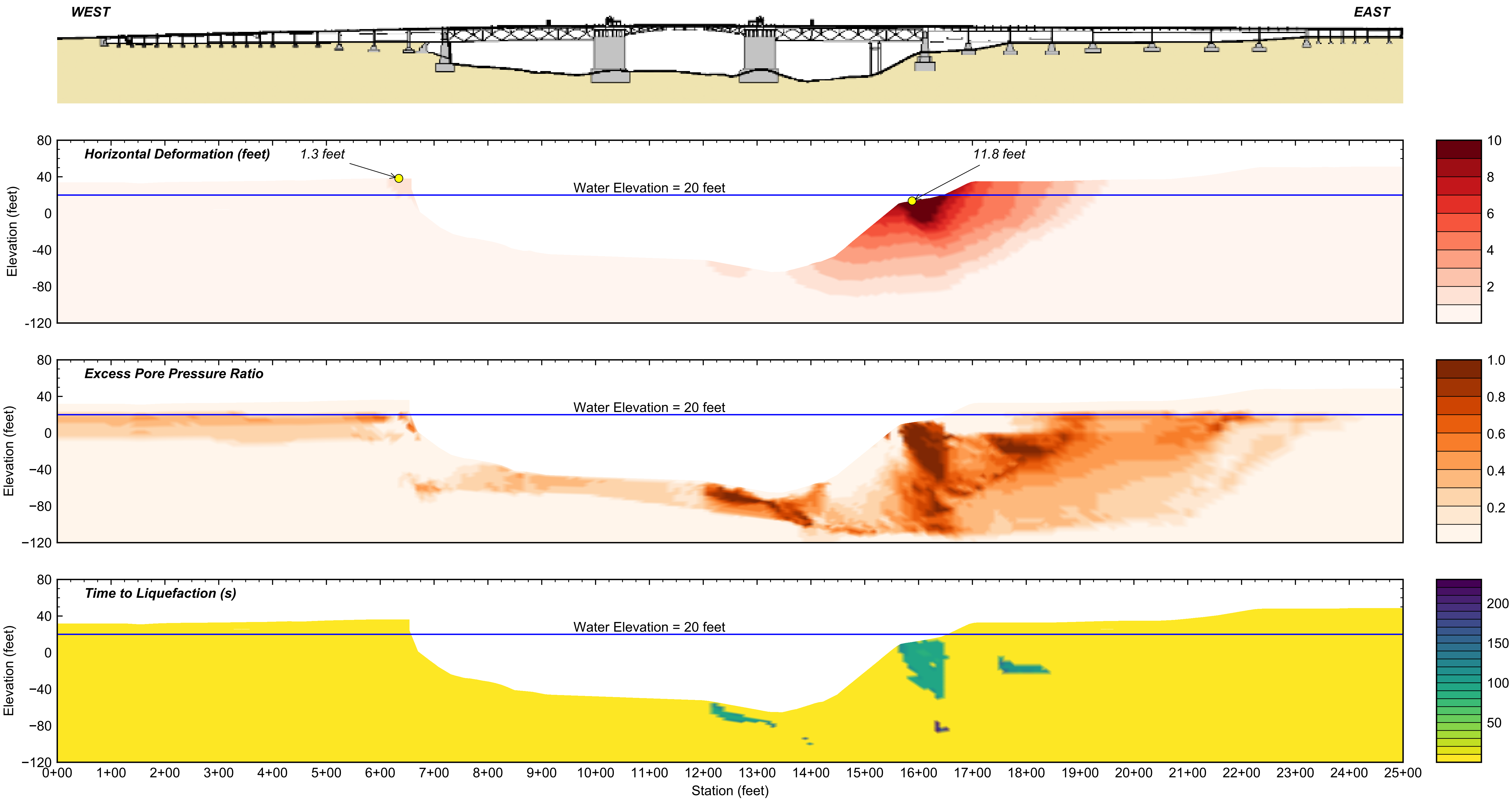
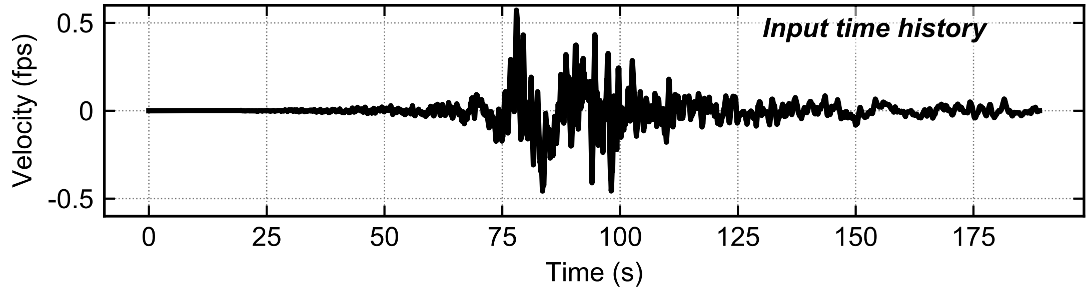
FIG. E-158

ver:0.8.5.post0.dev0+g414b689, by:pto





C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB

Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
FULL OPERATION
MAULE (ROC190)

August 2019
102636-001

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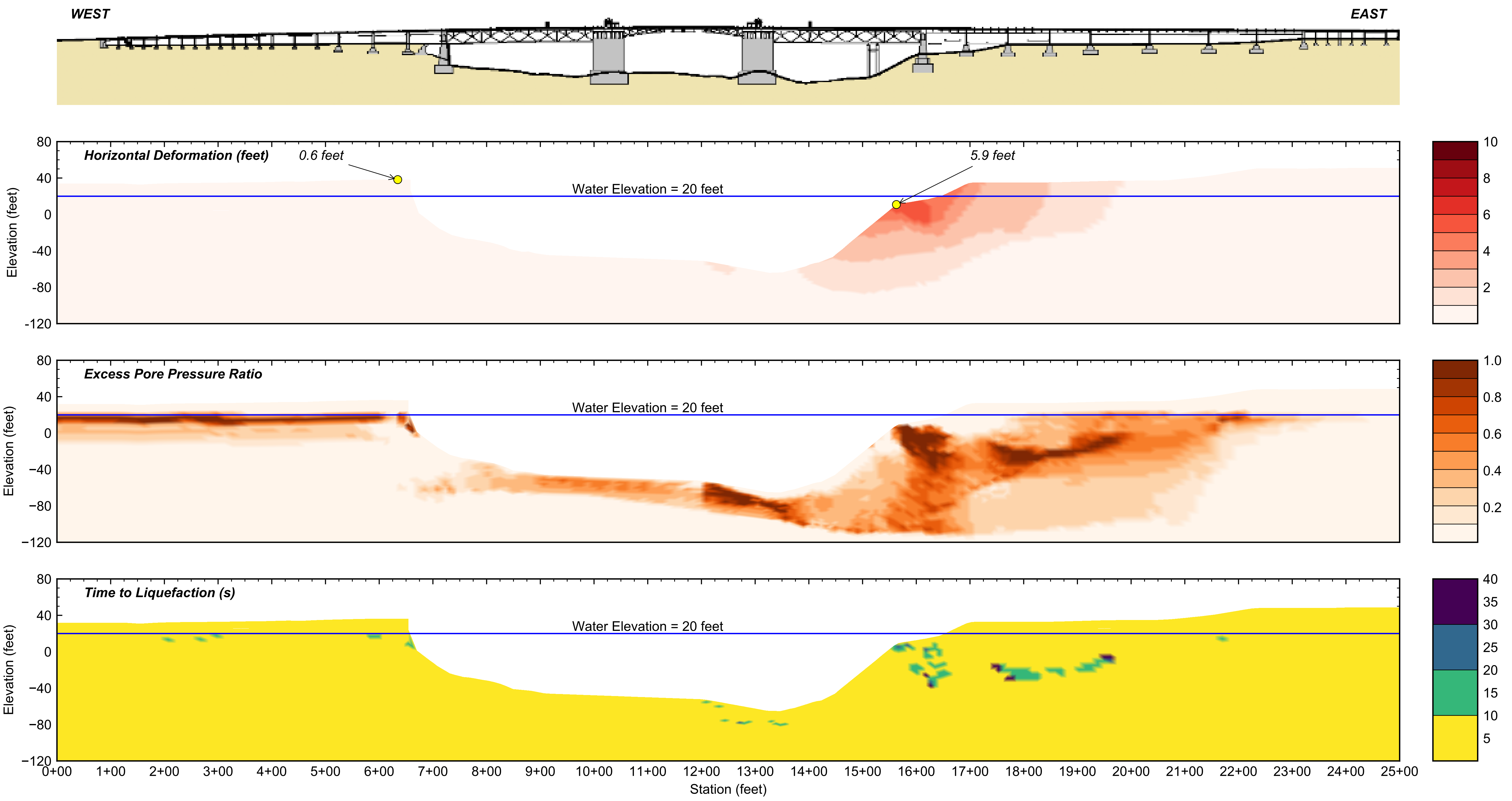
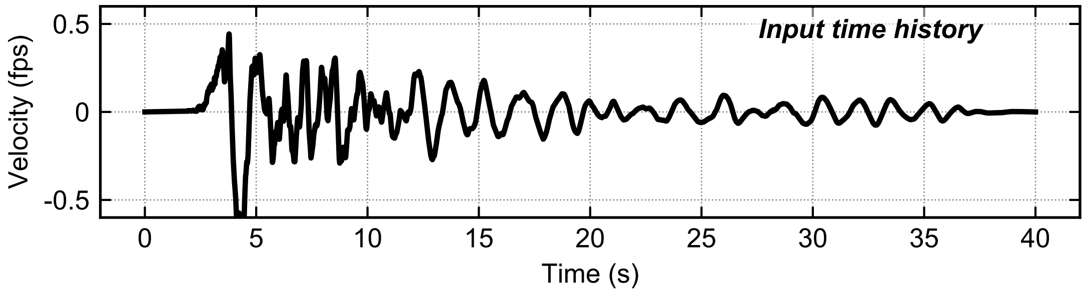
FIG. E-159

ver:0.8.5.post0.dev0+g414b689, by:pto





C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB

Burnside Bridge Environmental Impact Study
Portland, Oregon

**MODEL RESPONSE
LIMITED OPERATION
LAQUILA (AM043YLN)**

August 2019
102636-001

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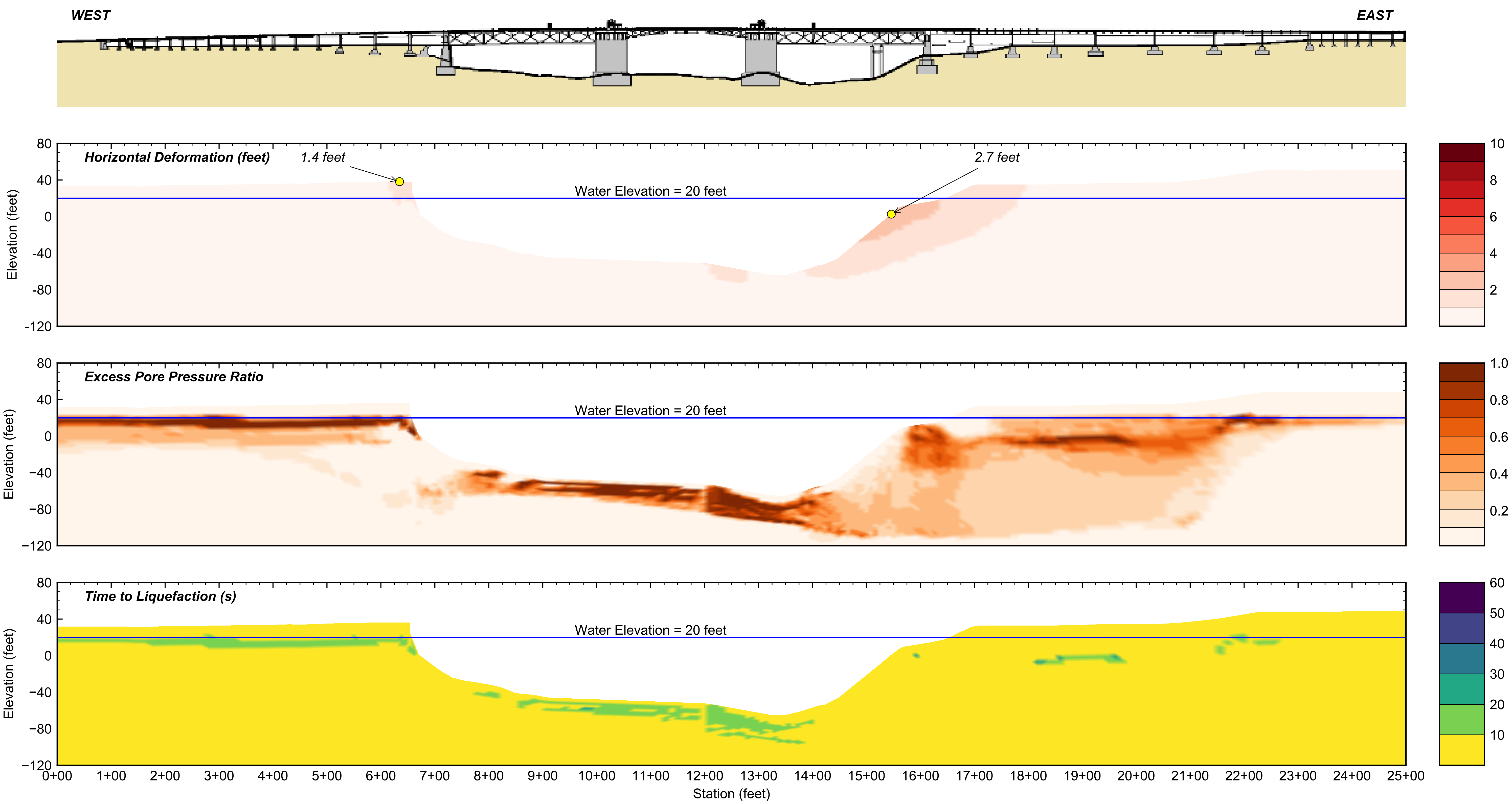
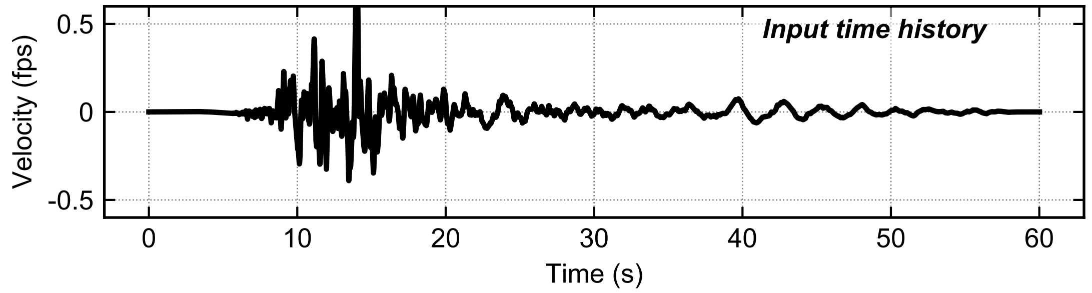
FIG. E-160





Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
NORTHRIDGE (LA0270)

August 2019 102636-001

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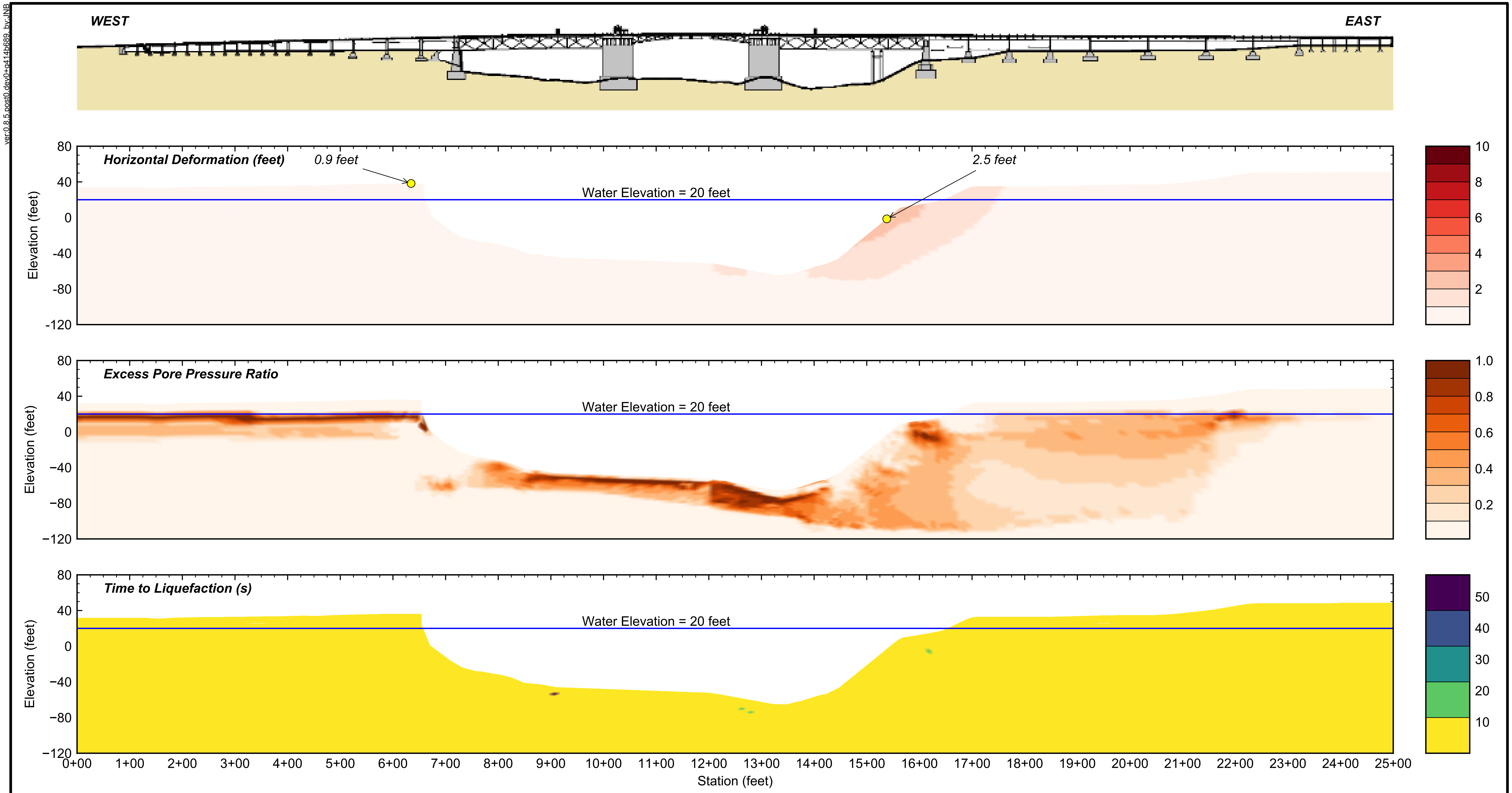
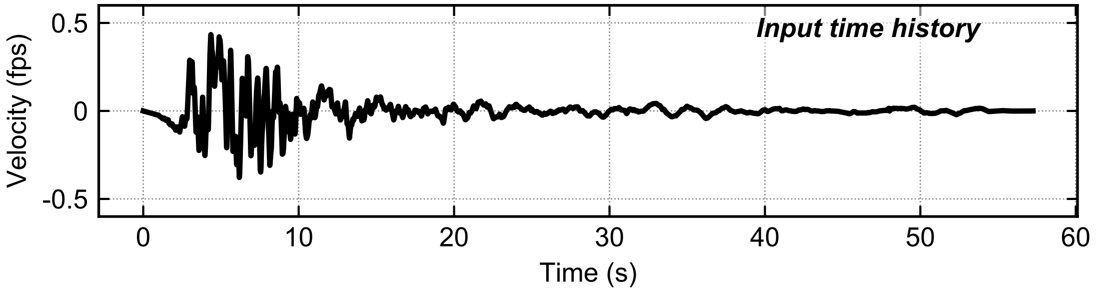
FIG. E-161

ver:0.8.5.post0.dev0+g414b689, by:pto

C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB

ver:0.8.5.post0.dev0+g414b689, by:JNB





Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
NORTHRIDGE (SSU000)

August 2019 102636-001

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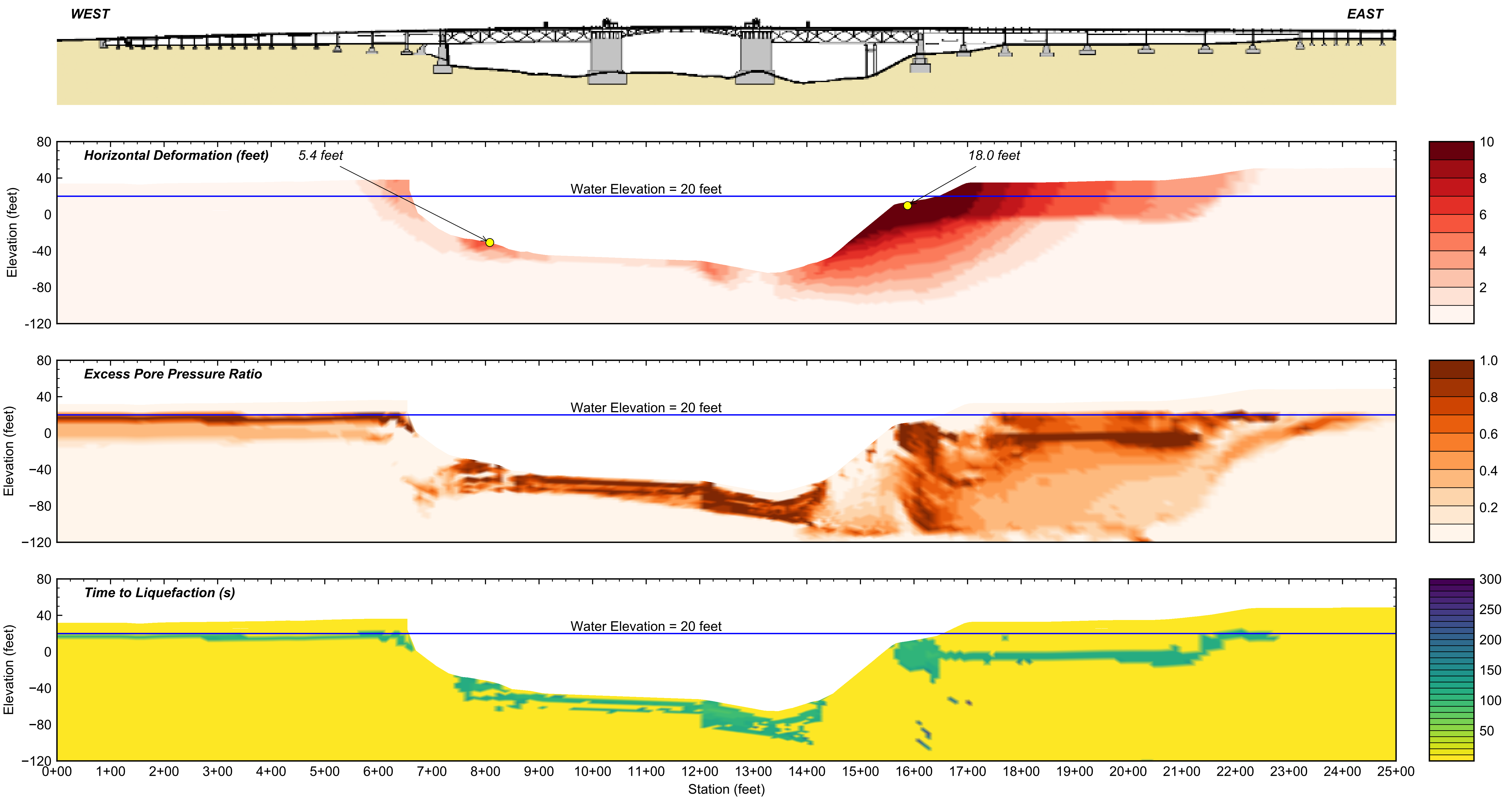
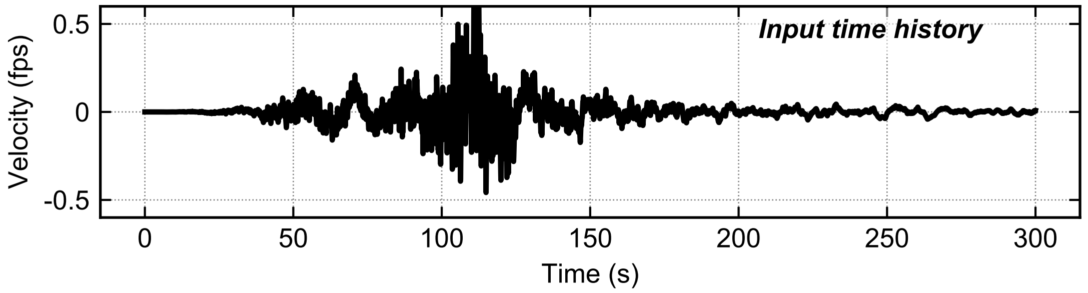
FIG. E-162

C:\Users\jnb\FLAC\Burnside\v6_test

ver:0.8.5.post0.dev0+g414b689 by:JNB





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Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
TOHOKU (FKS014EW)

August 2019 102636-001

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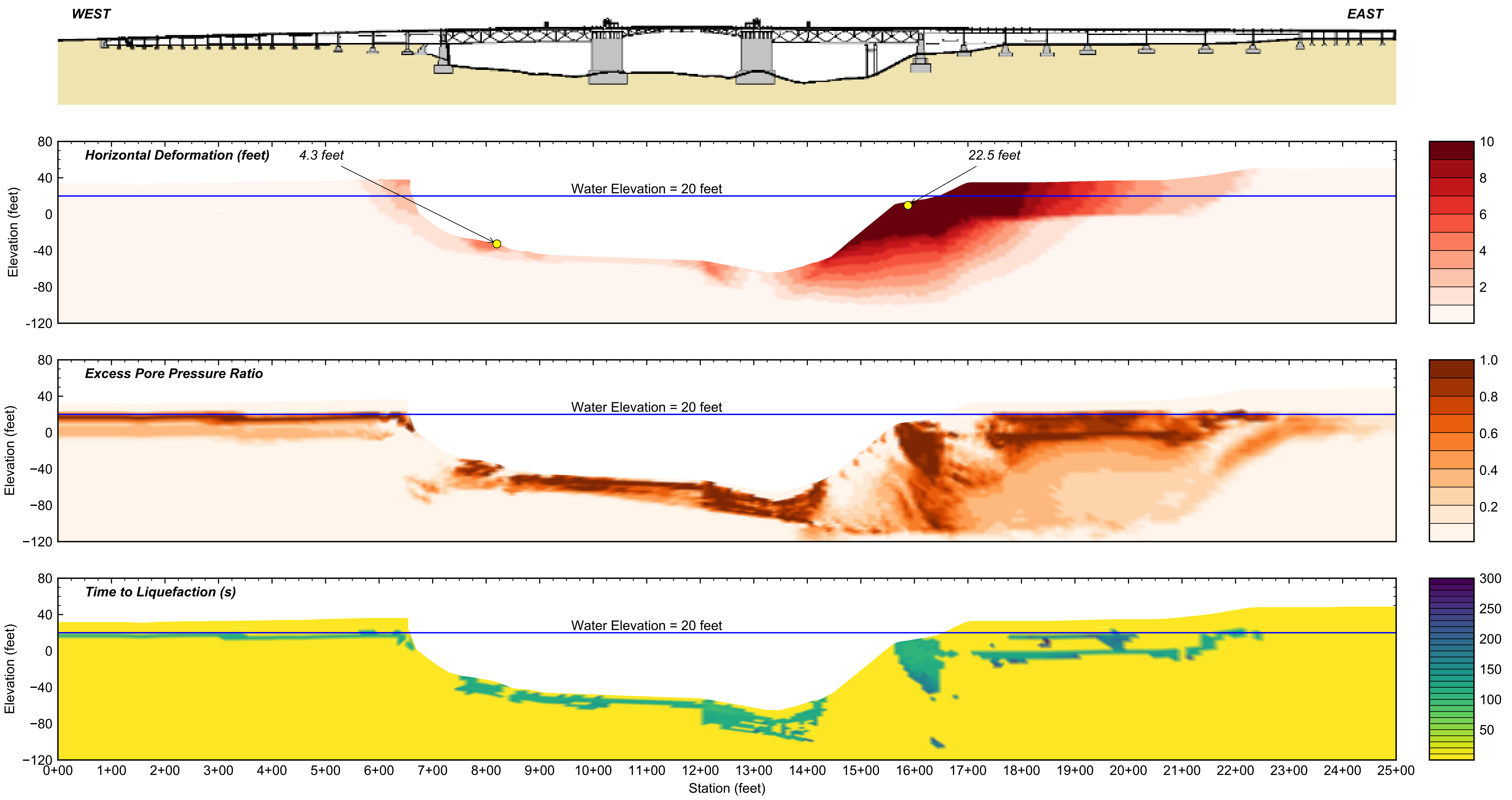
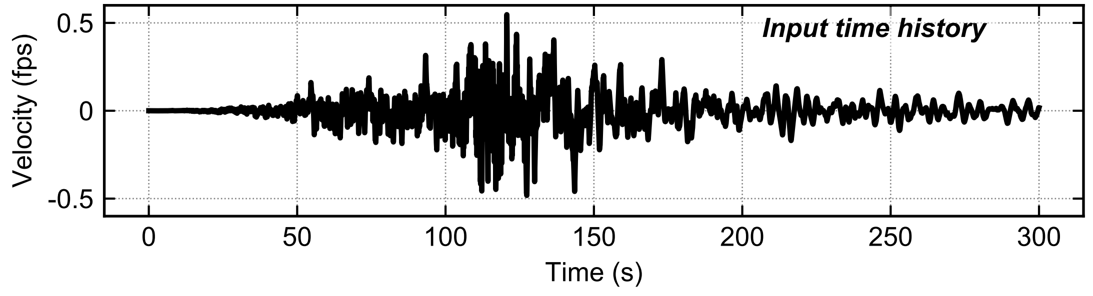
FIG. E-163

ver:0.8.5.post0.dev0+g414b689 by:JNB





Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
TOHOKU (GNM010NS)

August 2019
102636-001

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FIG. E-164

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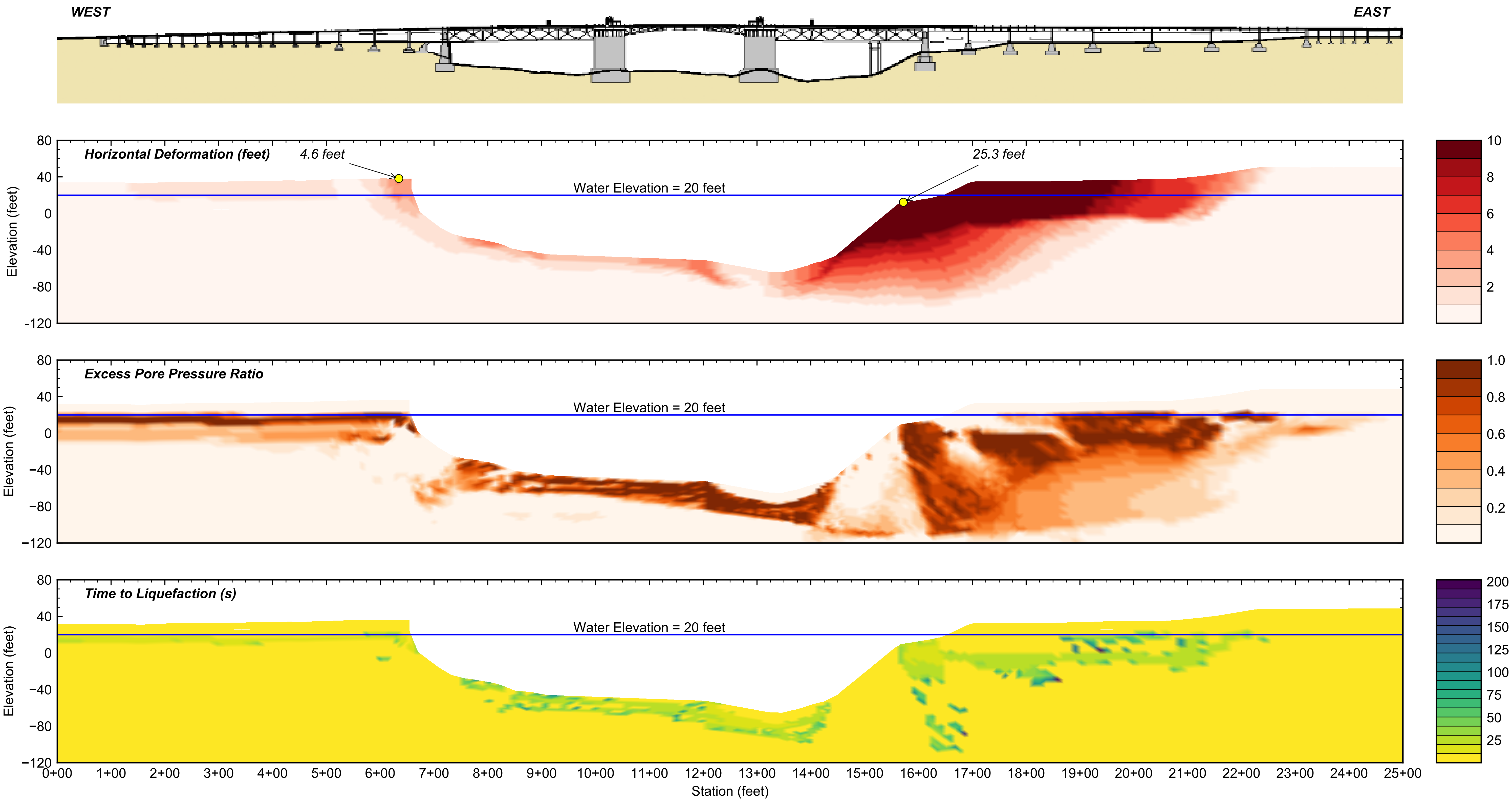
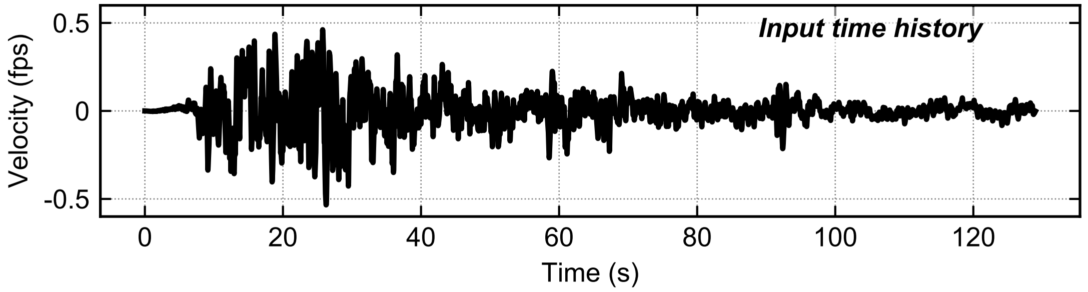
ver:0.8.5.post0.dev0+g414b689, by:pto





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Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
MAULE (CCSP97)

August 2019 102636-001

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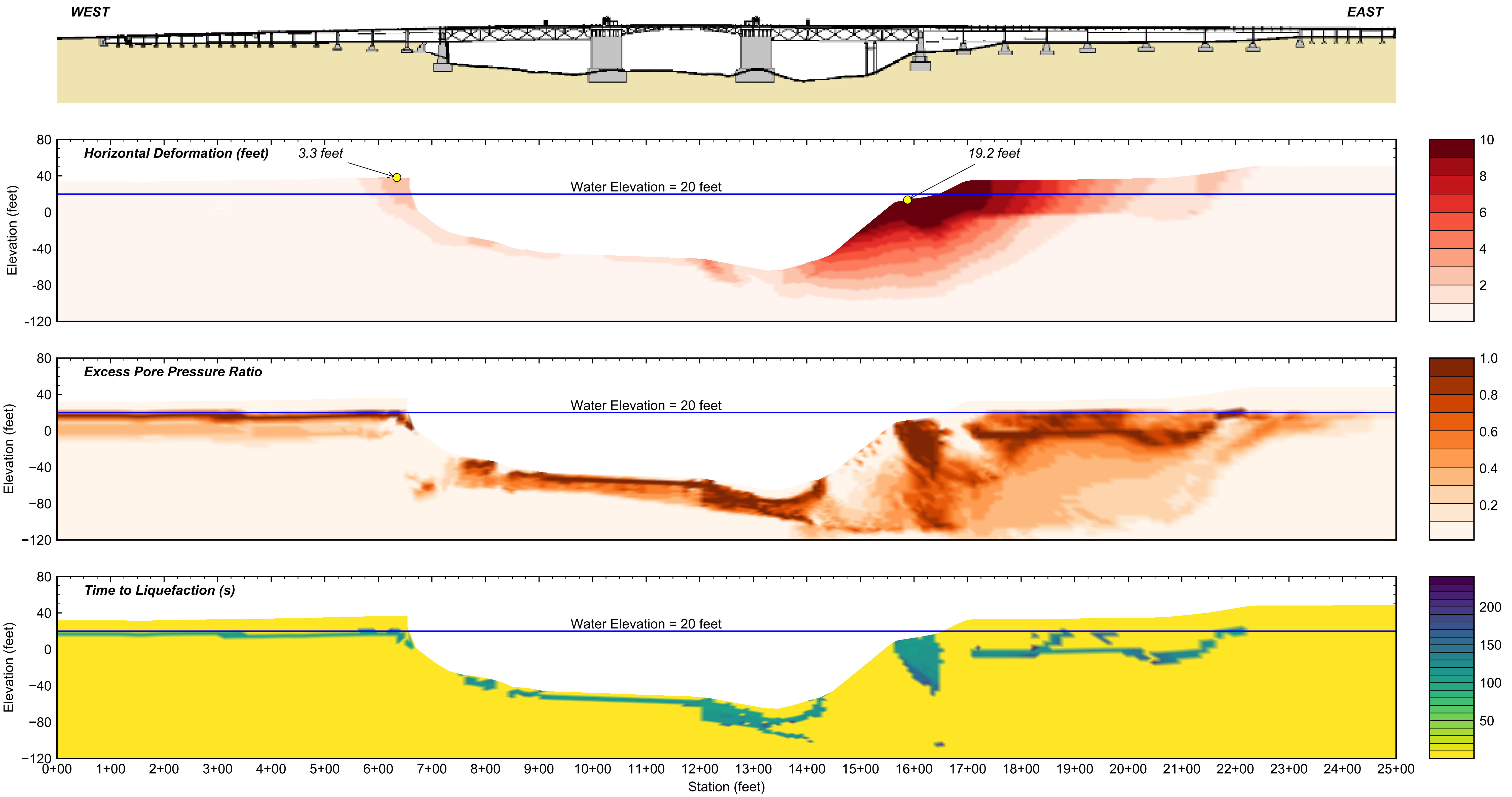
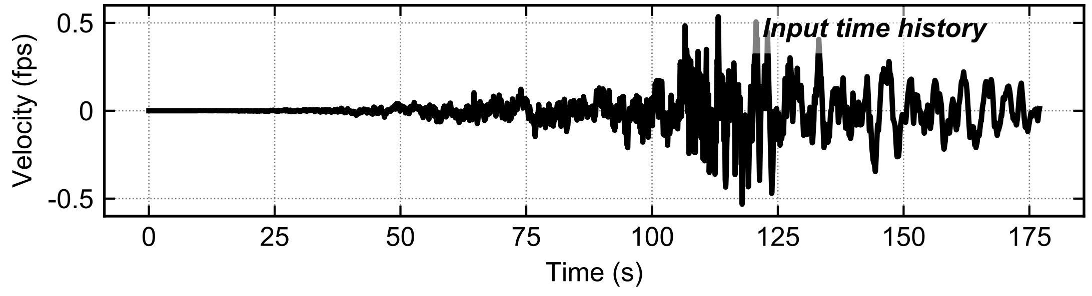
FIG. E-165

ver:0.8.5.post0.dev0+g414b689, by:pto





C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB

Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
TOHOKU (TCG012NS)

August 2019
102636-001

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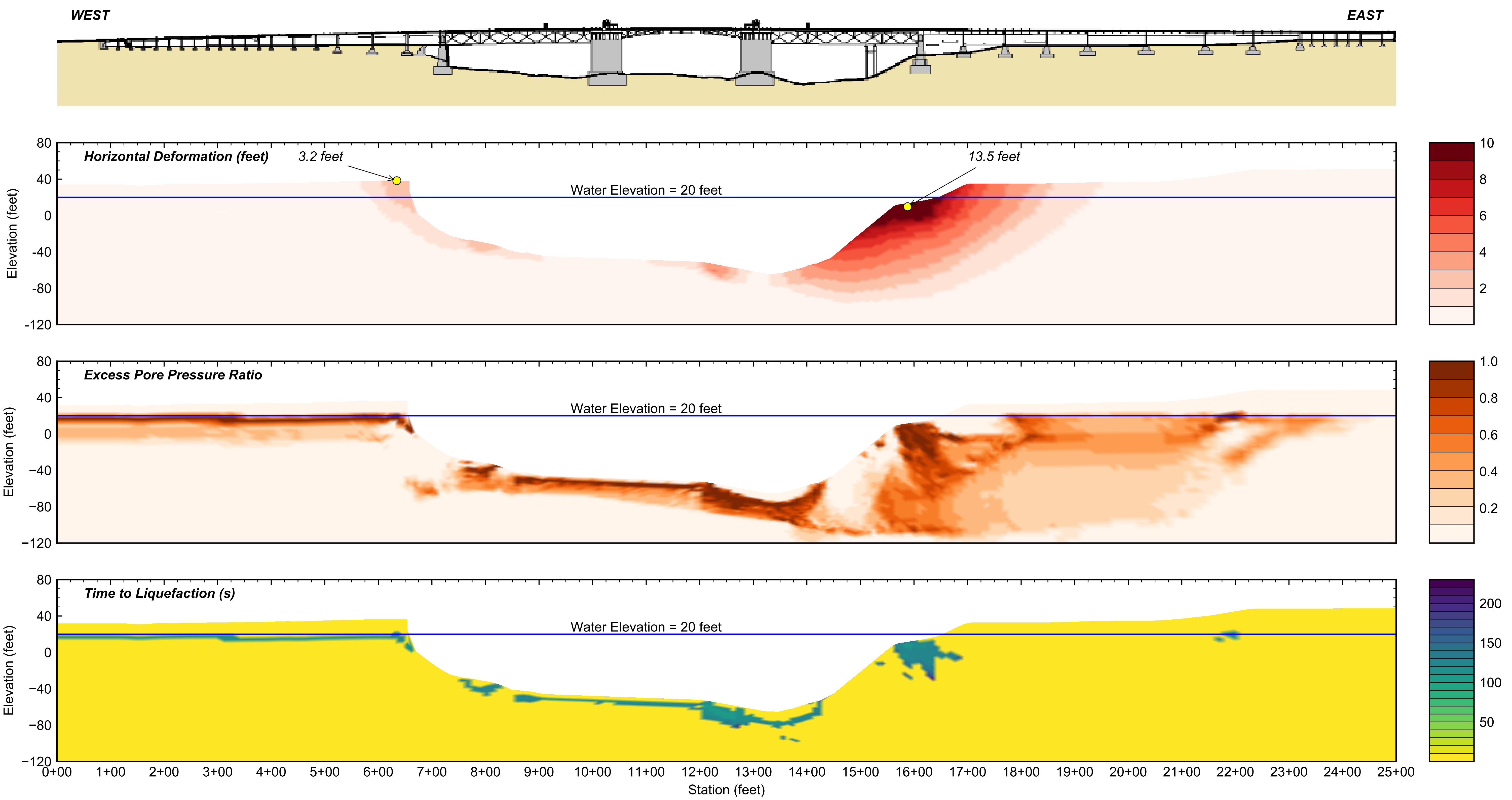
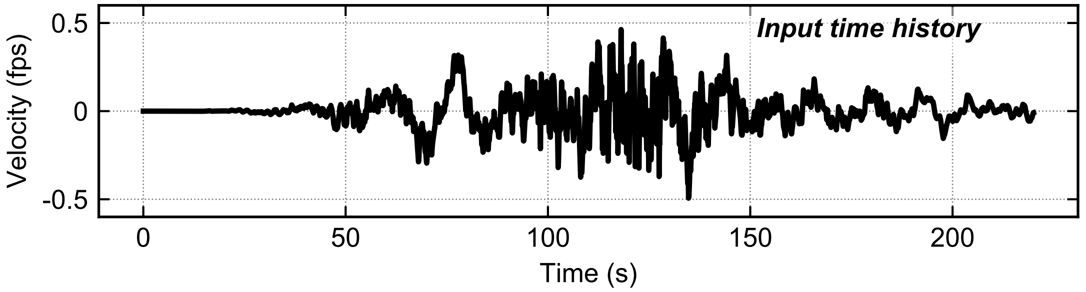
FIG. E-166

ver:0.8.5.post0.dev0+g414b689, by:pto





C:\Users\pto\Desktop\FLAC Processing\2019.05.14 python processing from JNB

Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
TOHOKU (FKSH10EW)

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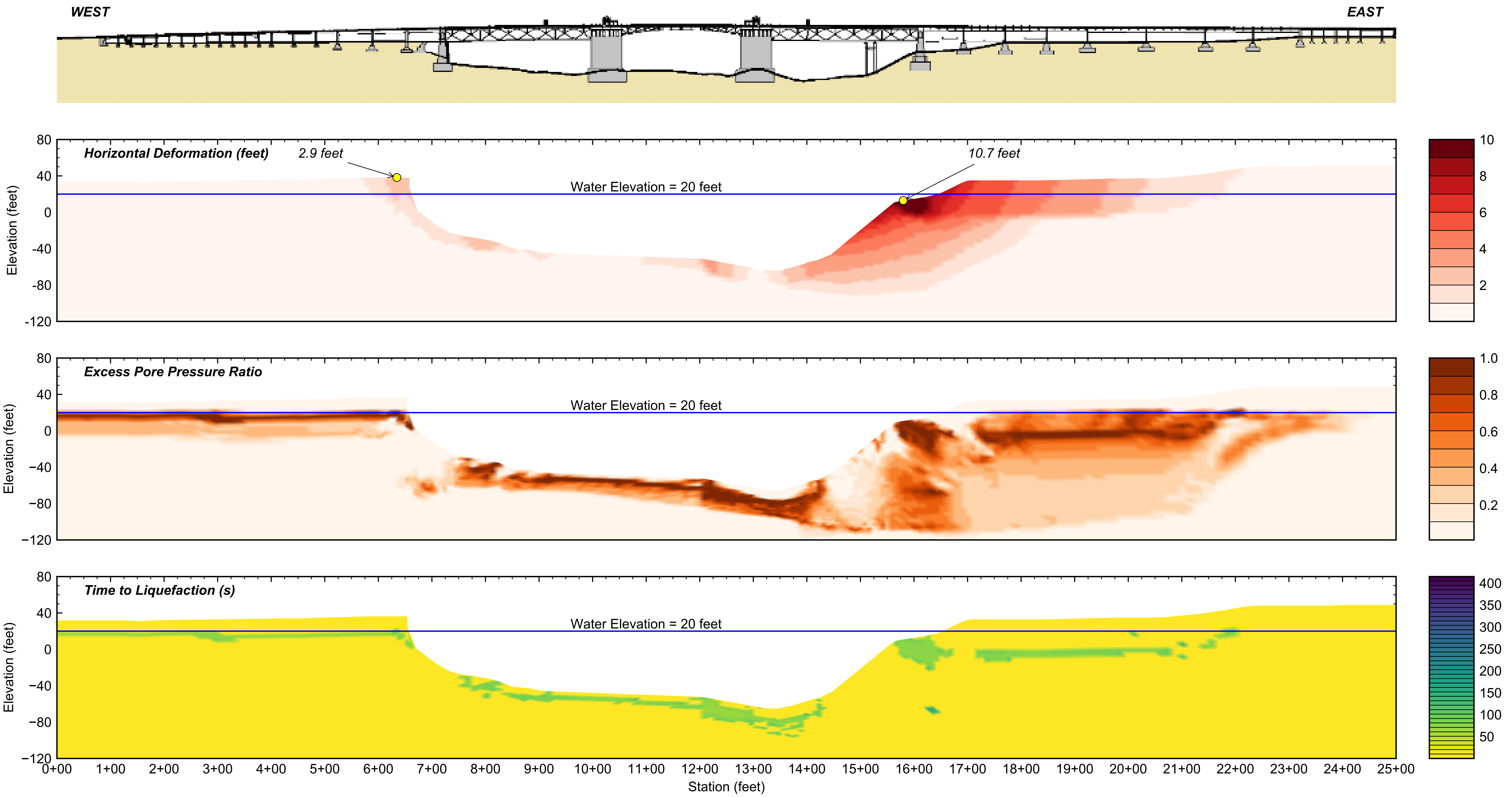
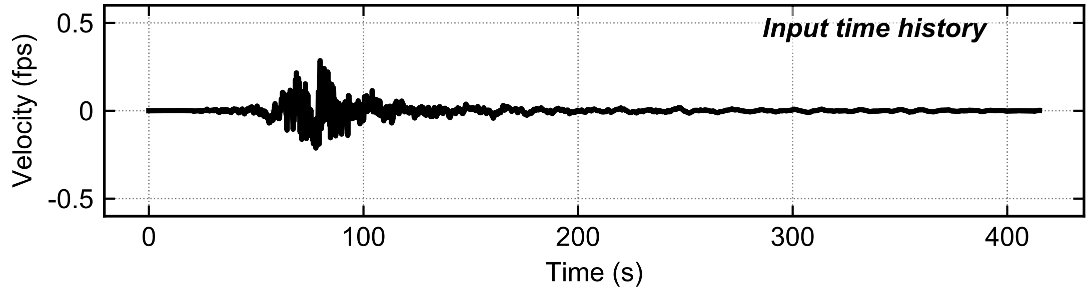
FIG. E-167

ver0.8.5.post0.dev0+g414b689 by:JNB





Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE
LIMITED OPERATION
MAULE (ANTU90)

August 2019
102636-001

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FIG. E-168

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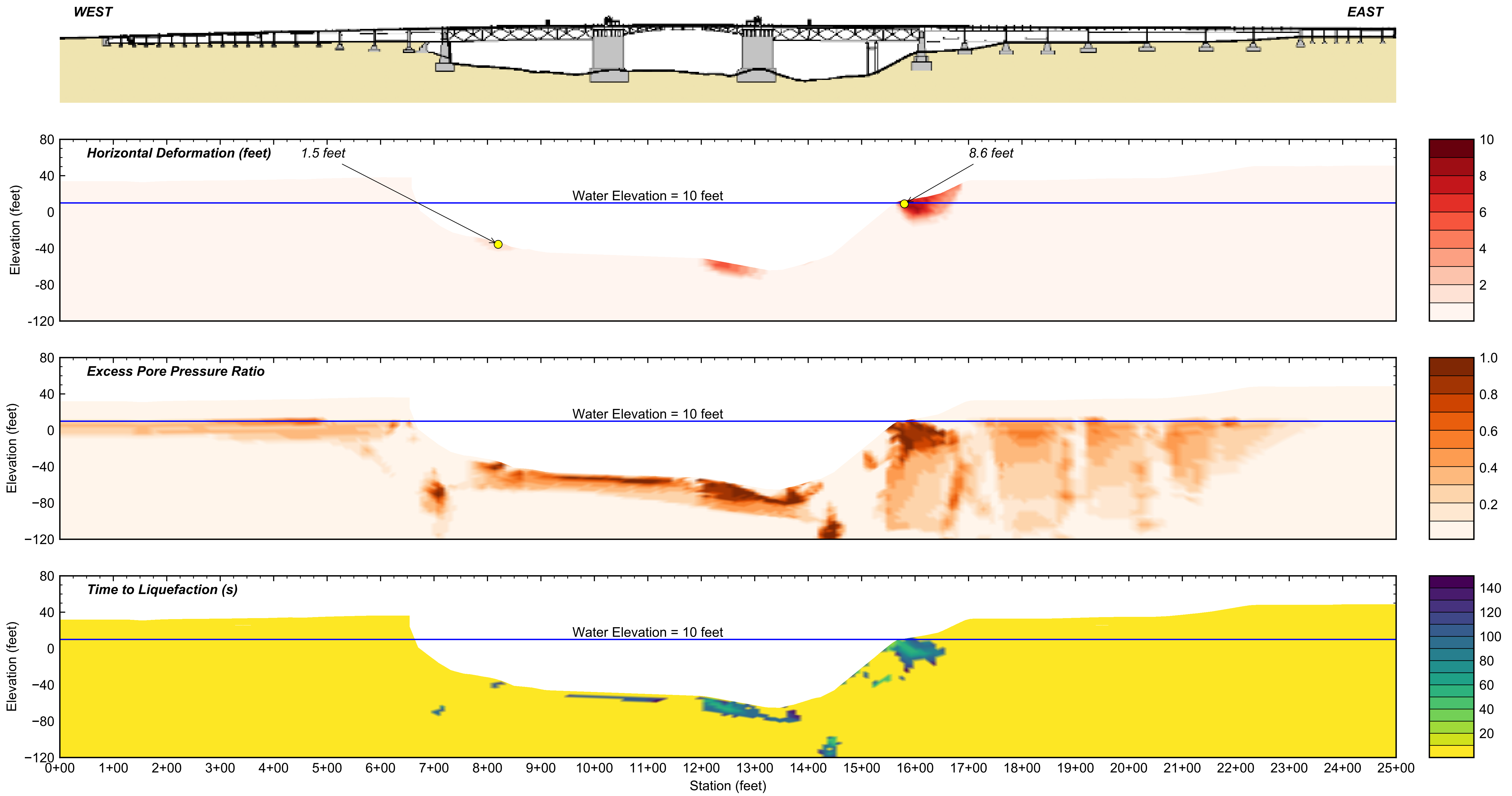
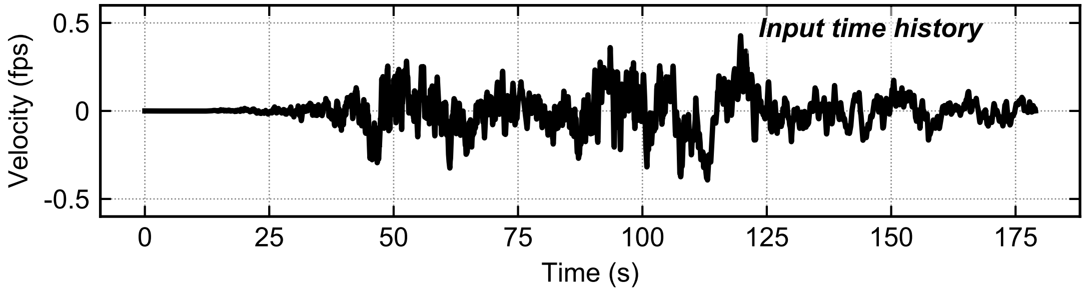
ver:0.8.5.post0.dev0+g414b689, by:pto





Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE (WITH GI)
FULL OPERATION MOTION
TOHOKU (AKT023EW)

August 2019 102636-001

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FIG. E-169

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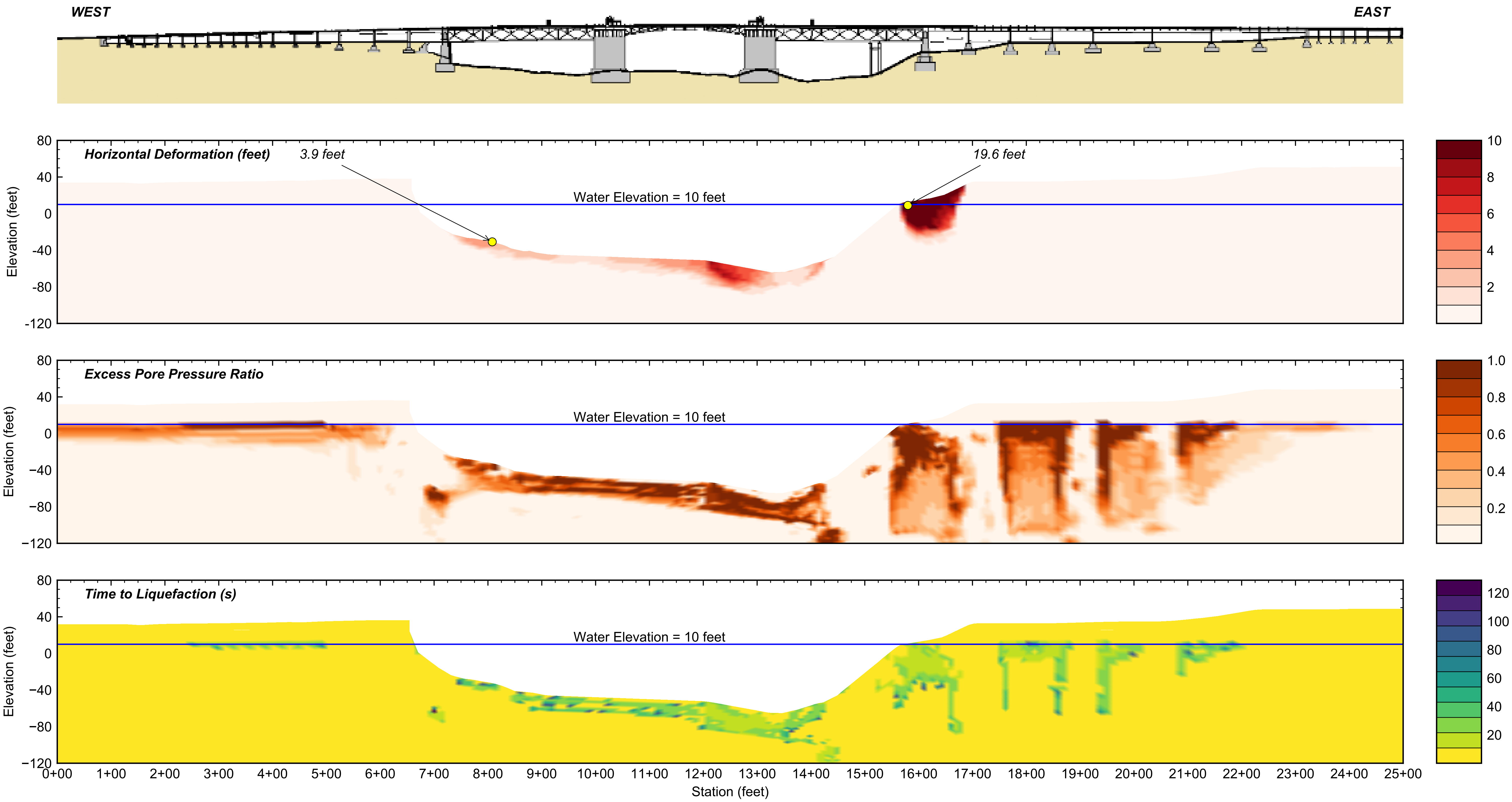
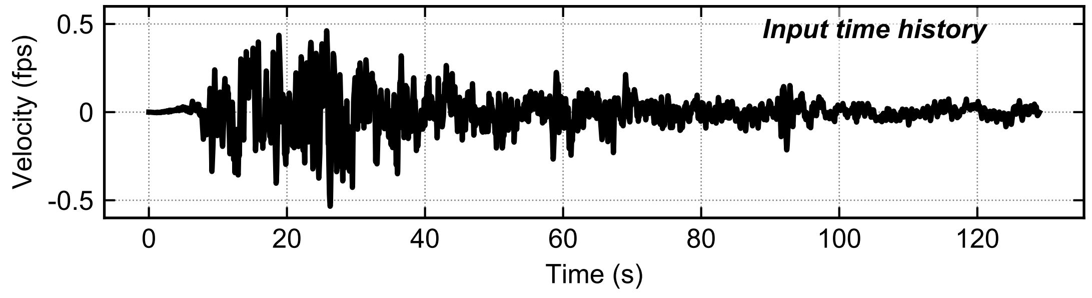
ver:0.8.5.post0.dev0+g414b689_by:pto





Burnside Bridge Environmental Impact Study
Portland, Oregon

MODEL RESPONSE (WITH GI)
LIMITED OPERATION MOTION
MAULE (CCSP97)

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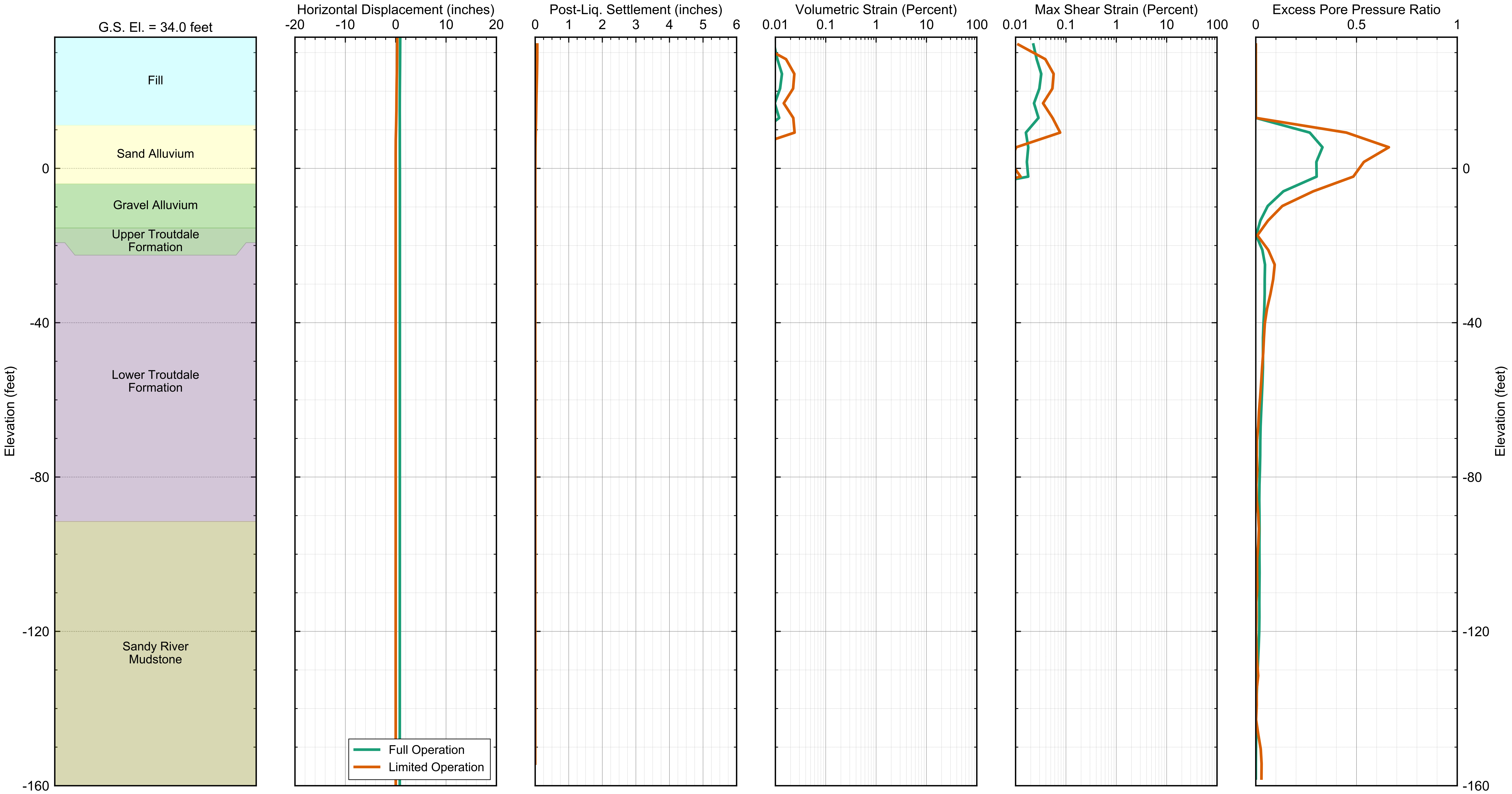
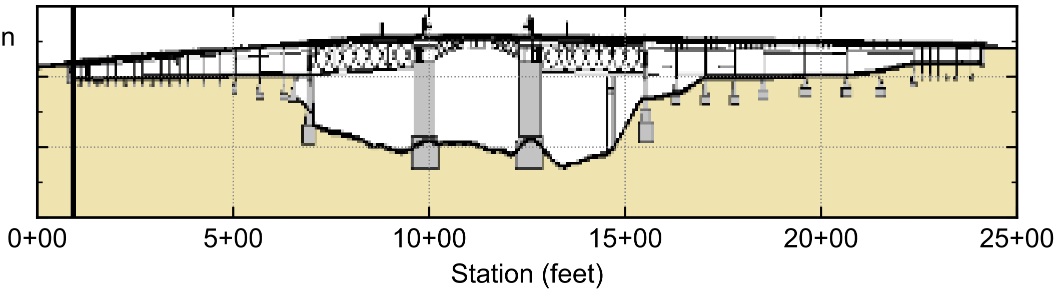
FIG. E-170

C:\Users\pto\Desktop\FLAC Processing\v6-GI-3-6 FLAC Processing



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 1 RETROFIT

August 2019
102636-001

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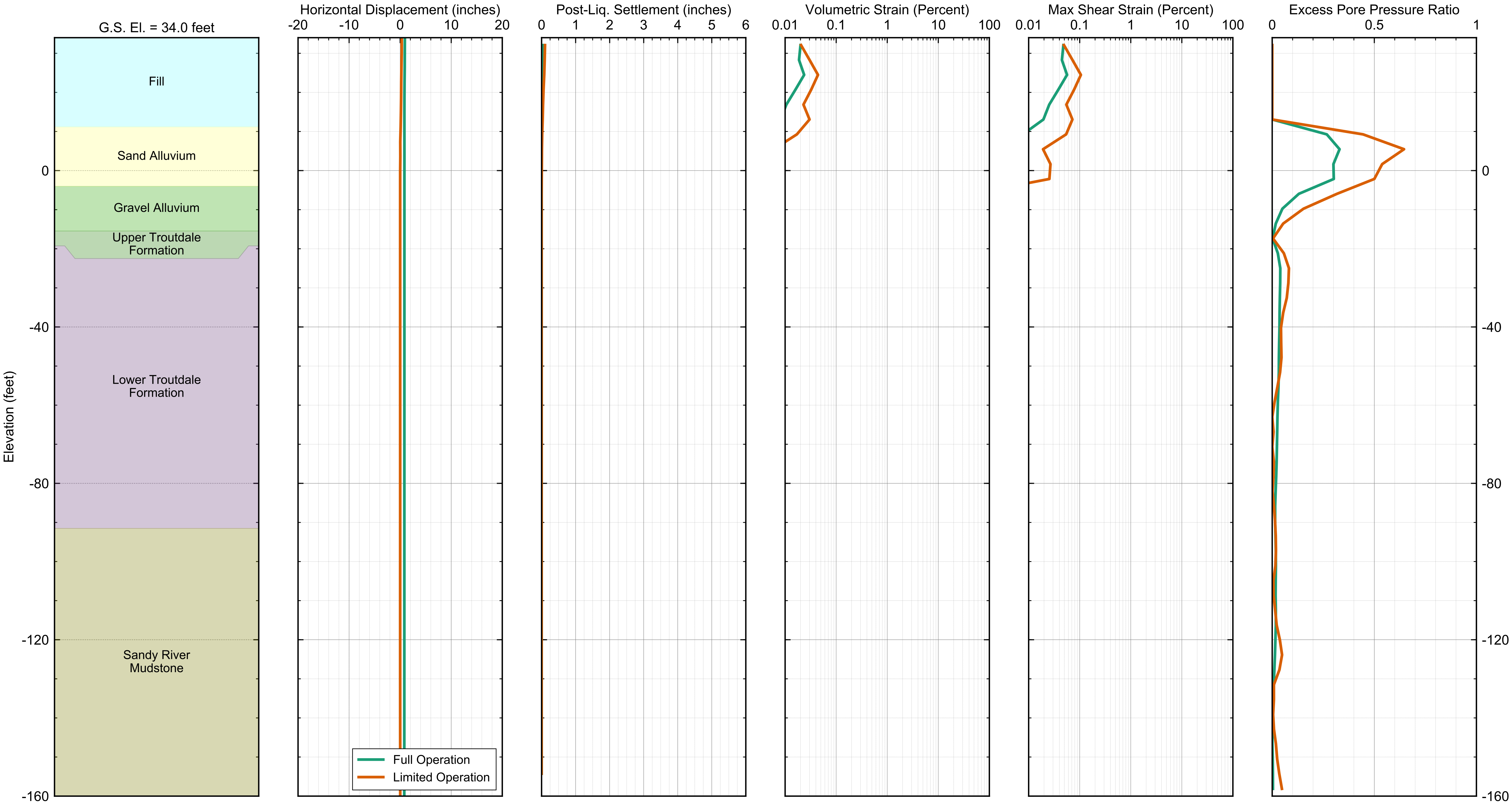
FIG. E-171

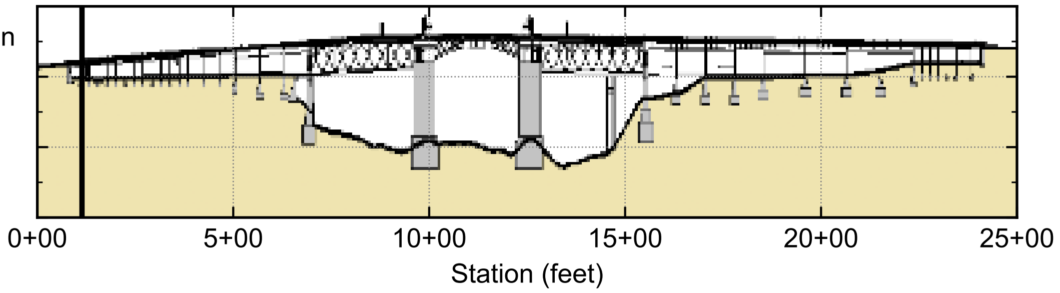
ver:0.8.5.post0.dev0+g414b689, by:pto

C:\Users\pto\Desktop\FLAC Processing\v6-GI-3-6 FLAC Processing



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 2 RETROFIT

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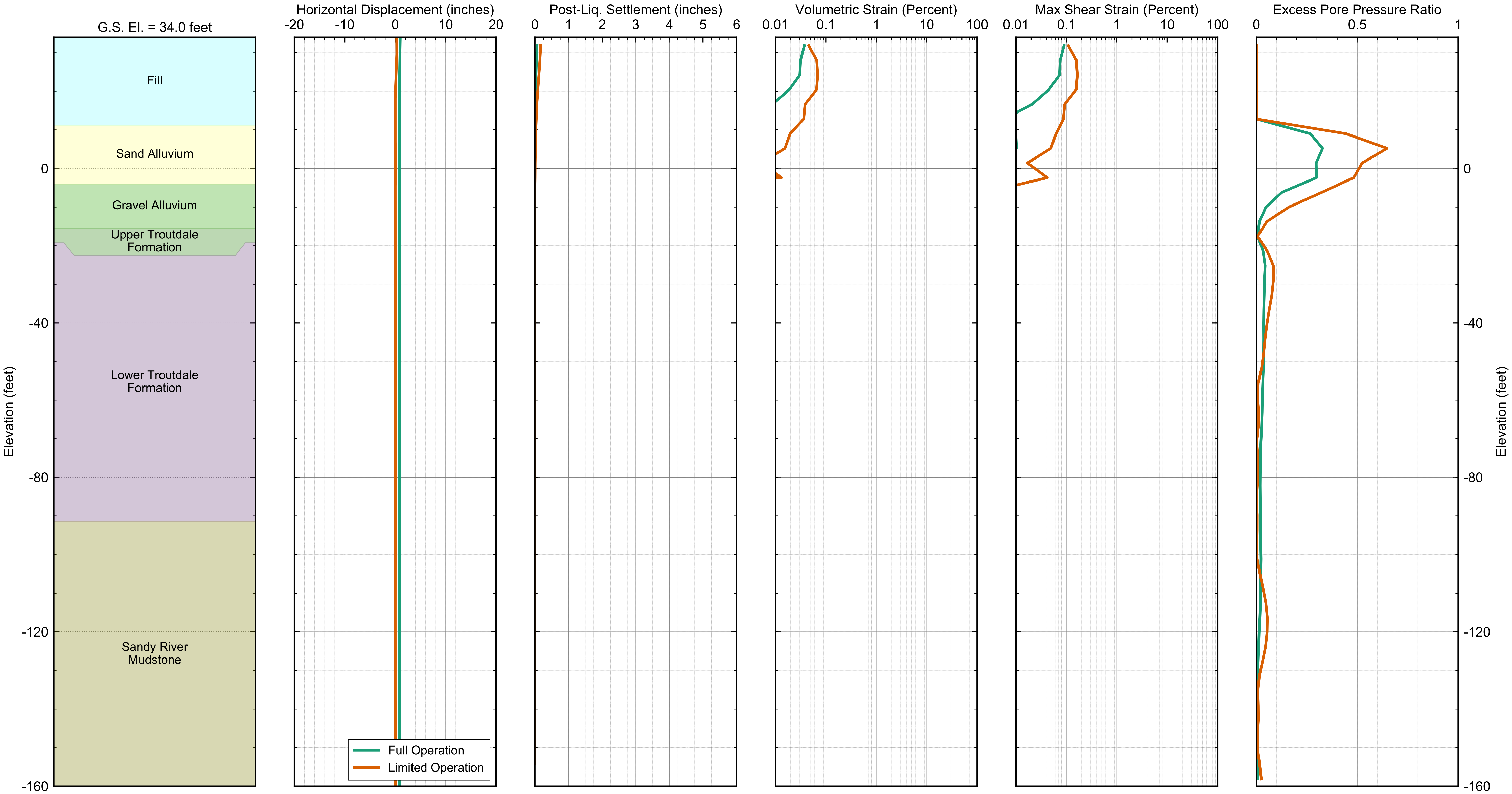
FIG. E-172

ver:0.8.5.post0.dev0+g414b689, by:pto

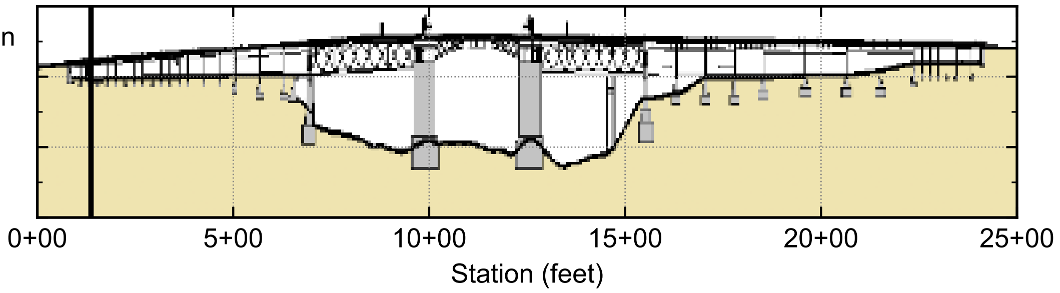
C:\Users\pto\Desktop\FLAC Processing\v6-GI-3-6 FLAC Processing

ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

**SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 3 RETROFIT**

August 2019
102636-001

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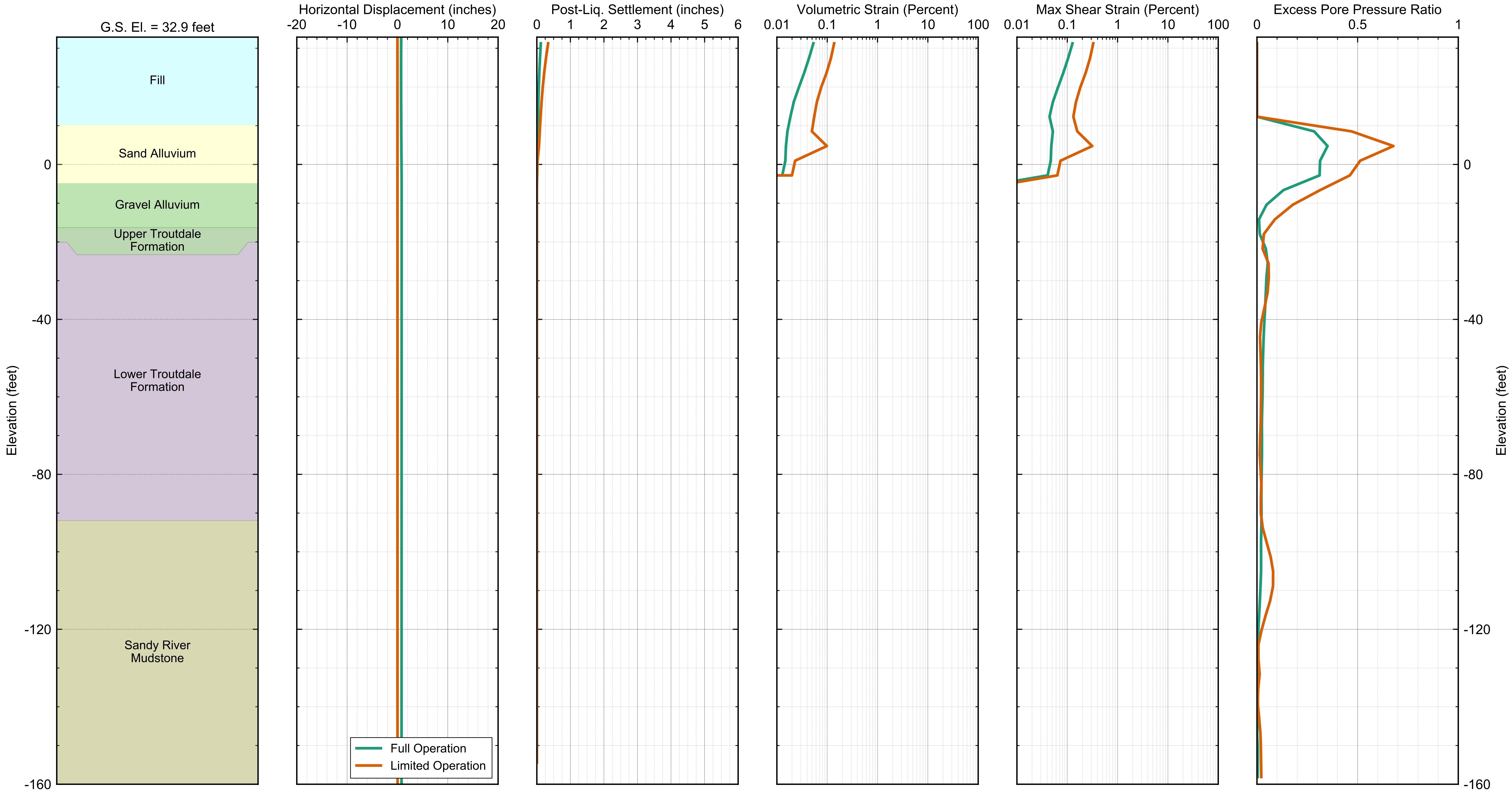
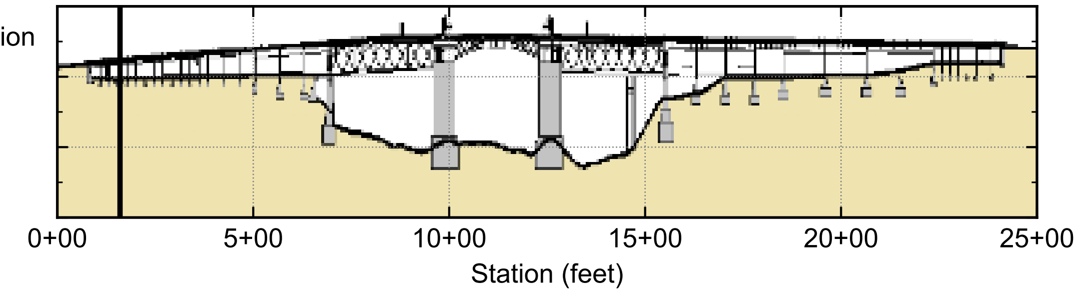
FIG. E-173

C:\Users\pto\Desktop\FLAC Processing\v6-GI-3-6 FLAC Processing



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 4 RETROFIT

August 2019
102636-001

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Geotechnical and Environmental Consultants

FIG. E-174

ver:0.8.5.post0.dev0+g414b689, by:pto

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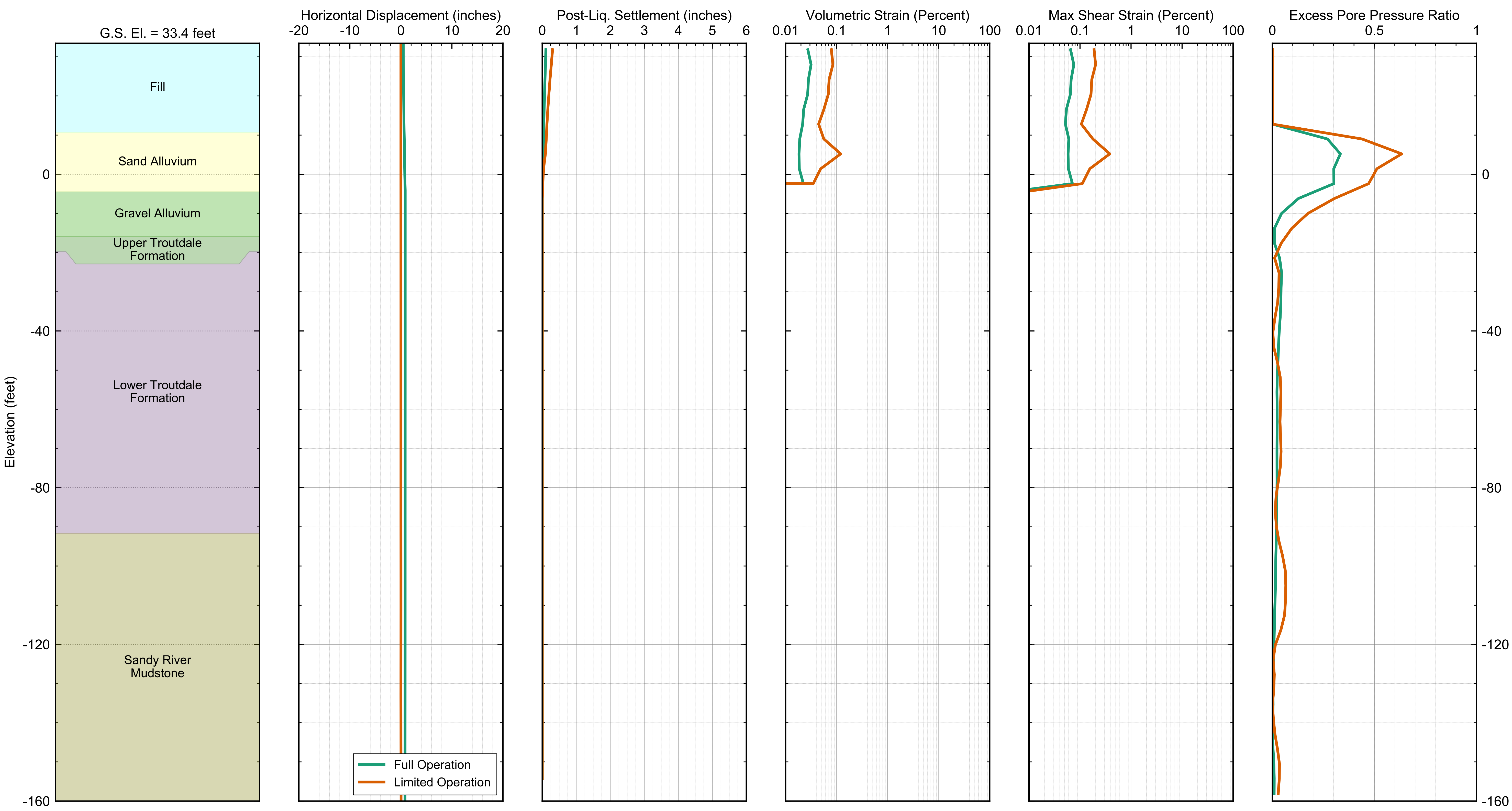
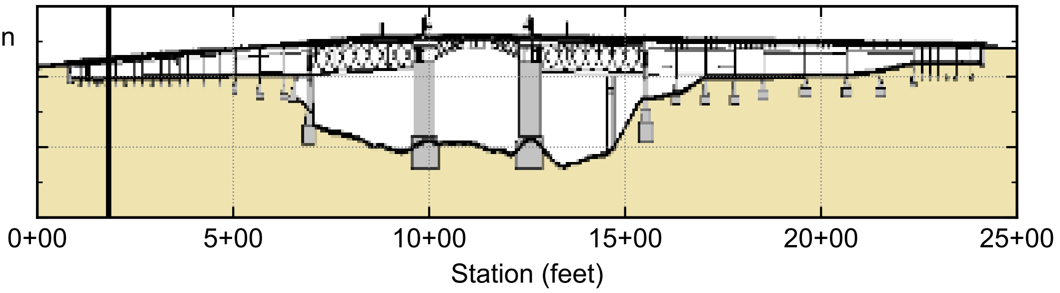
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 5 RETROFIT

August 2019
102636-001

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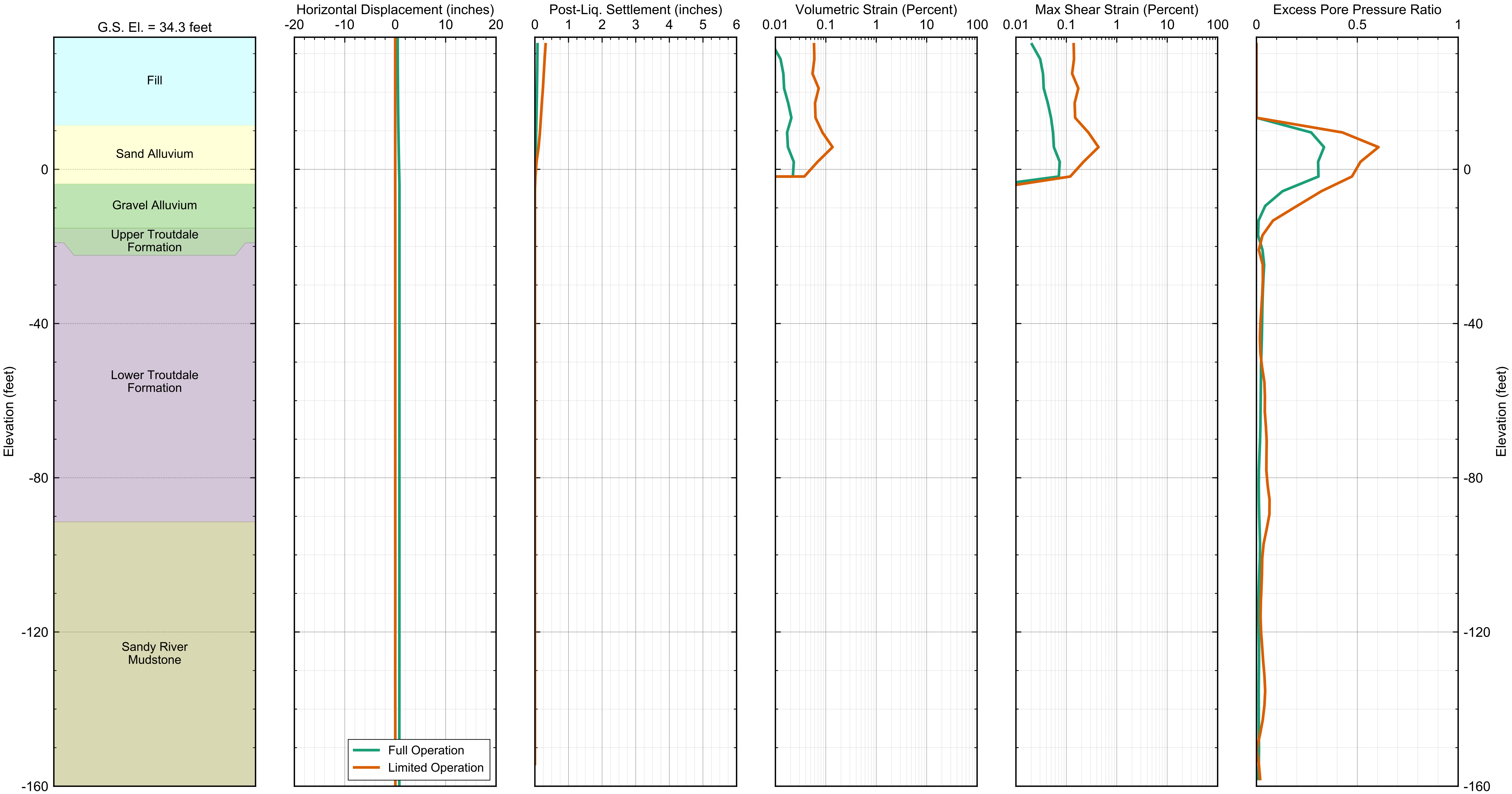
FIG. E-175

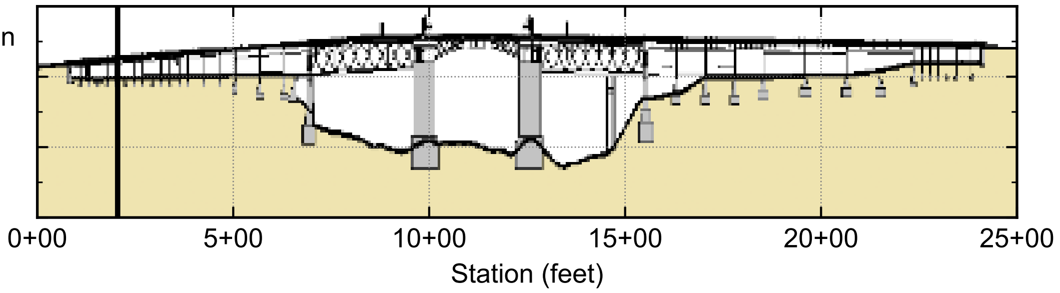
C:\Users\pto\Desktop\FLAC Processing\v6-GI-3-6 FLAC Processing

ver.0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 6 RETROFIT

August 2019
102636-001

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FIG. E-176

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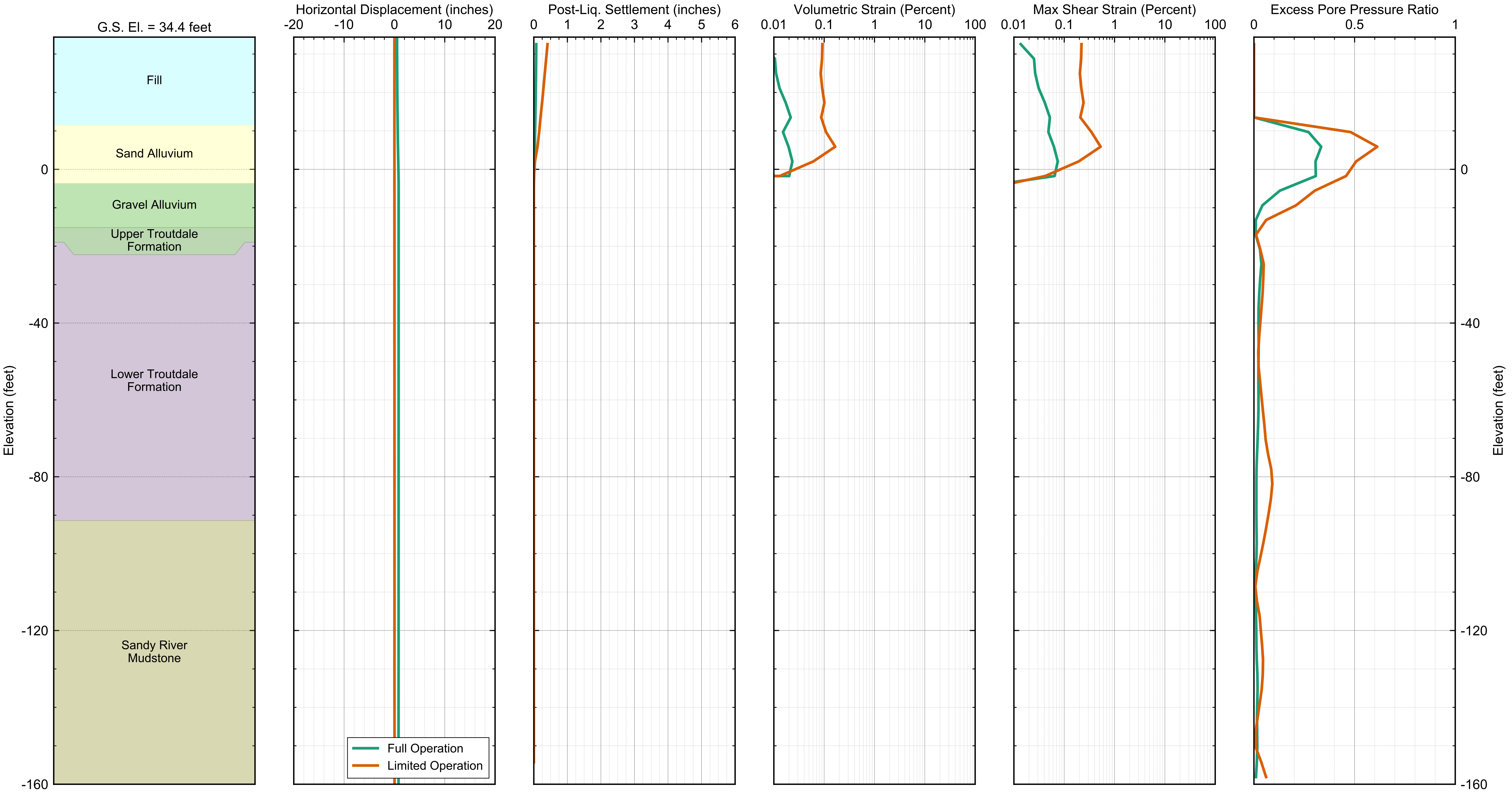
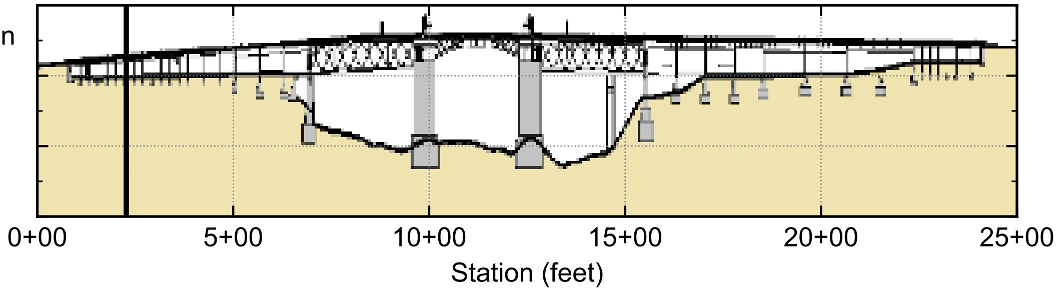


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 7 RETROFIT

August 2019
102636-001

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Geotechnical and Environmental Consultants

FIG. E-177

ver:0.8.5.post0.dev0+g414b689, by:pto

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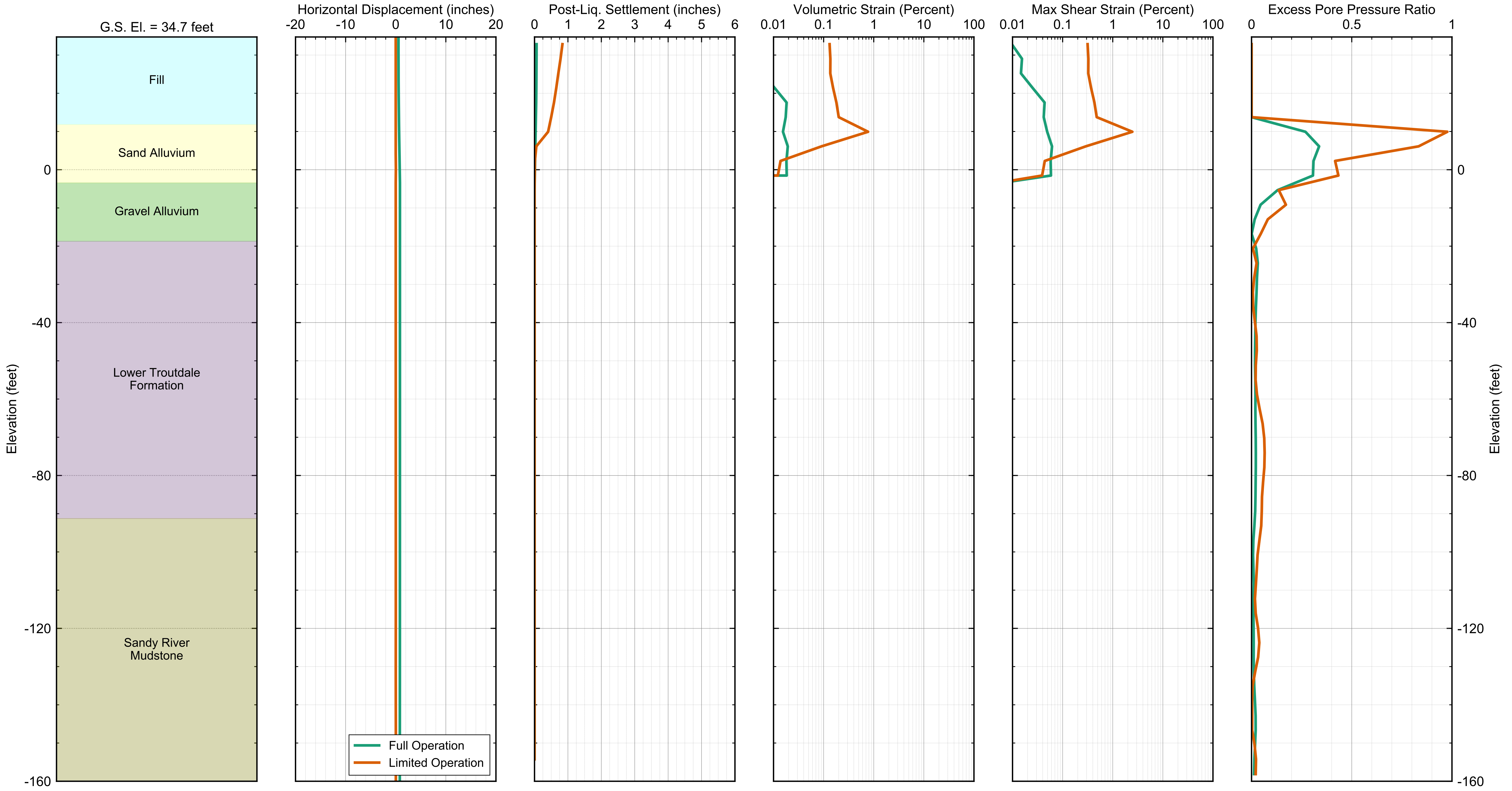
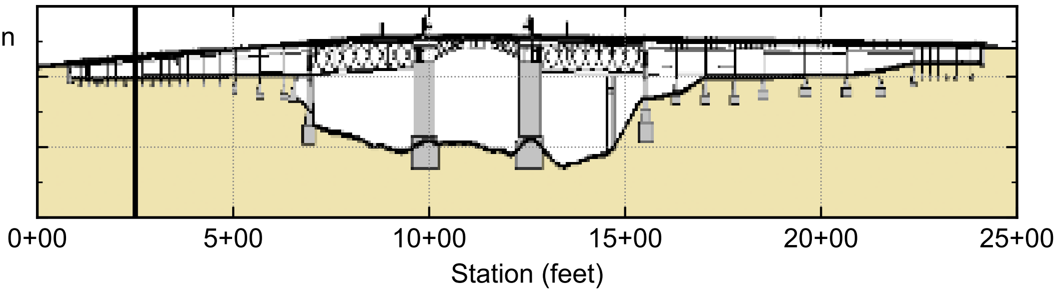
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 8 RETROFIT

August 2019
102636-001

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FIG. E-178

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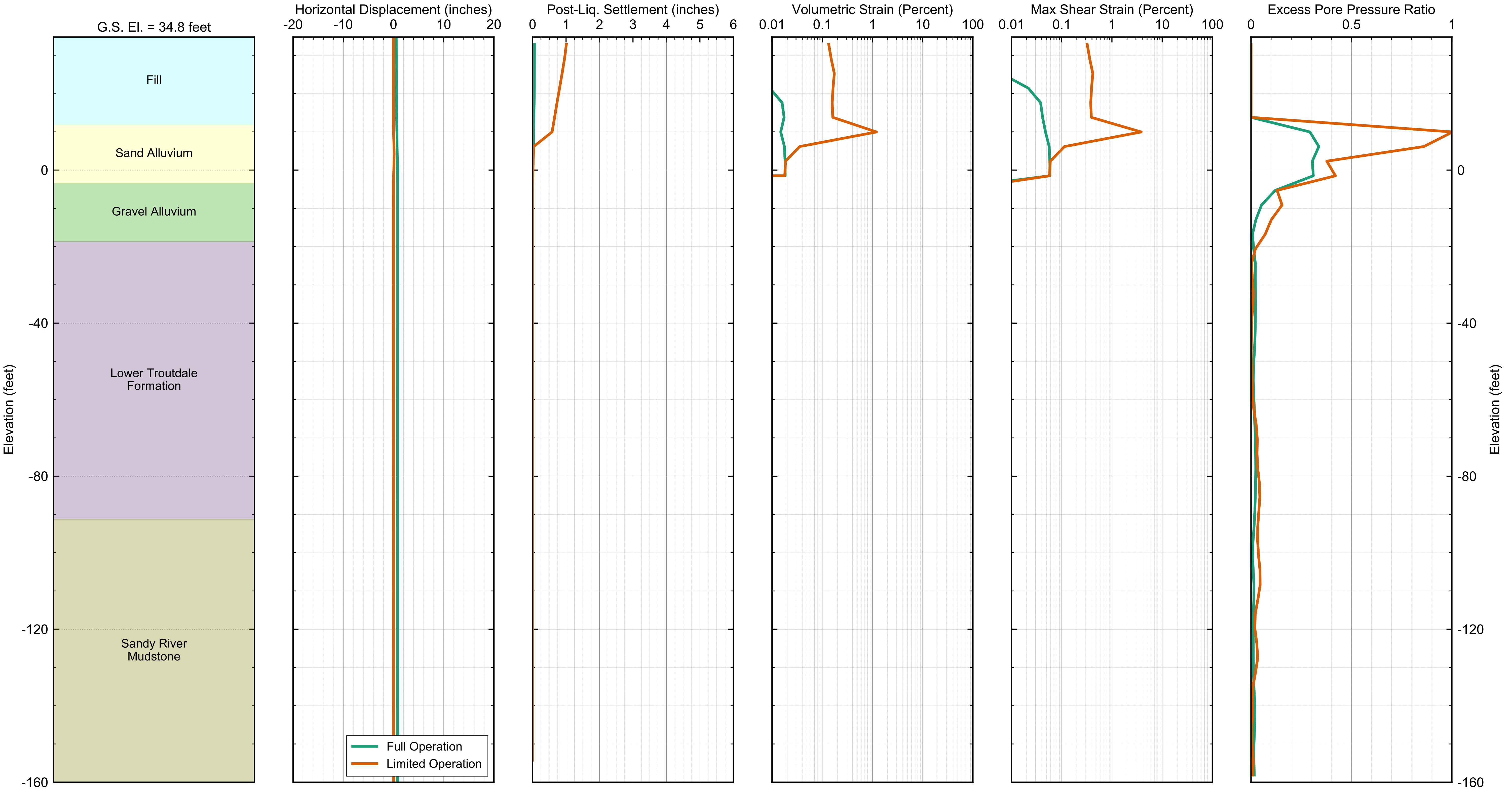
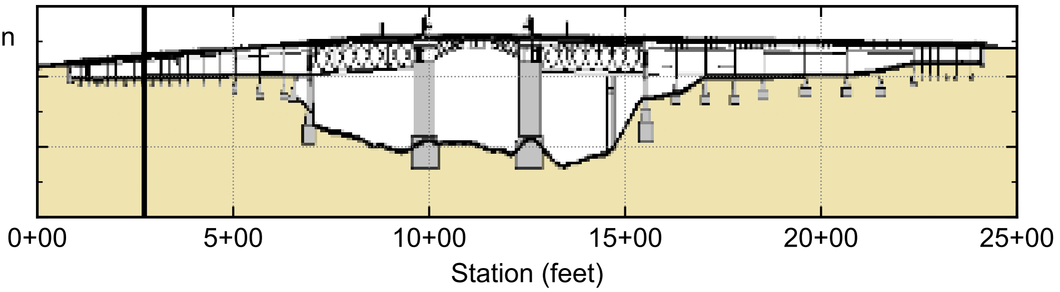
C:\Users\pto\Desktop\FLAC Processing\v6-GI-3-6 FLAC Processing



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 9 RETROFIT

August 2019 102636-001

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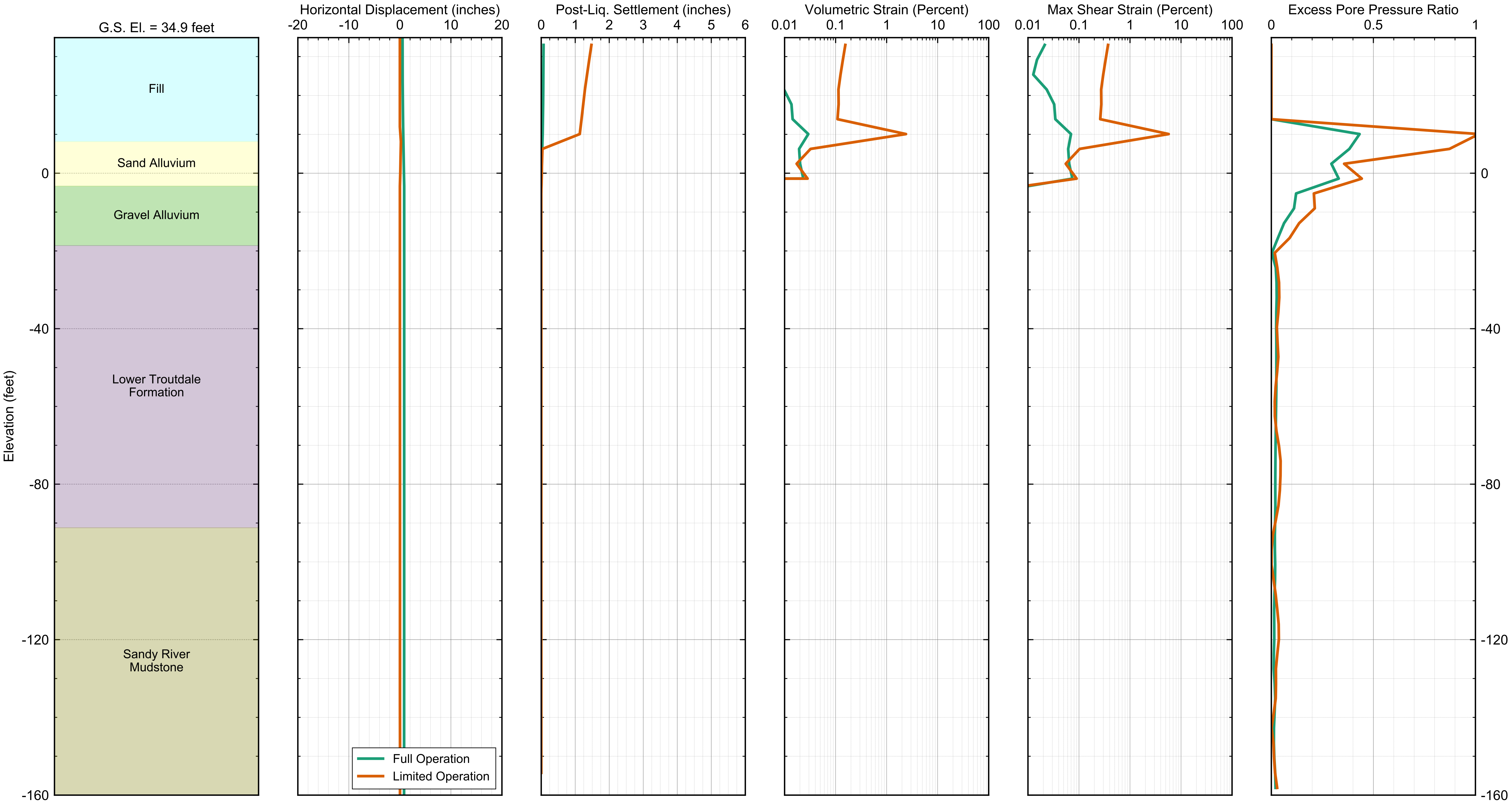
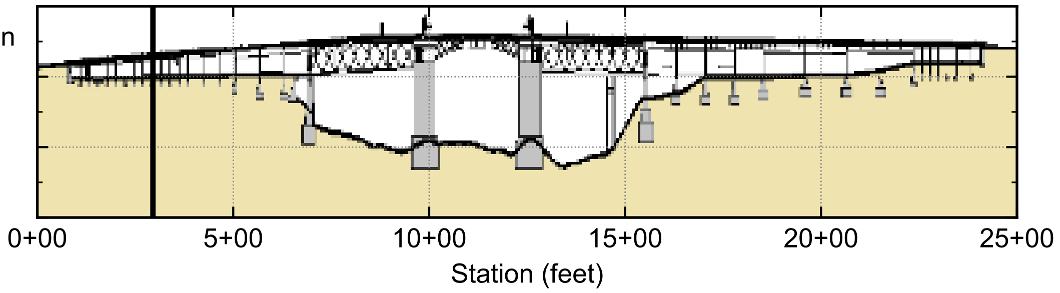
FIG. E-179

ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 10 RETROFIT

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FIG. E-180

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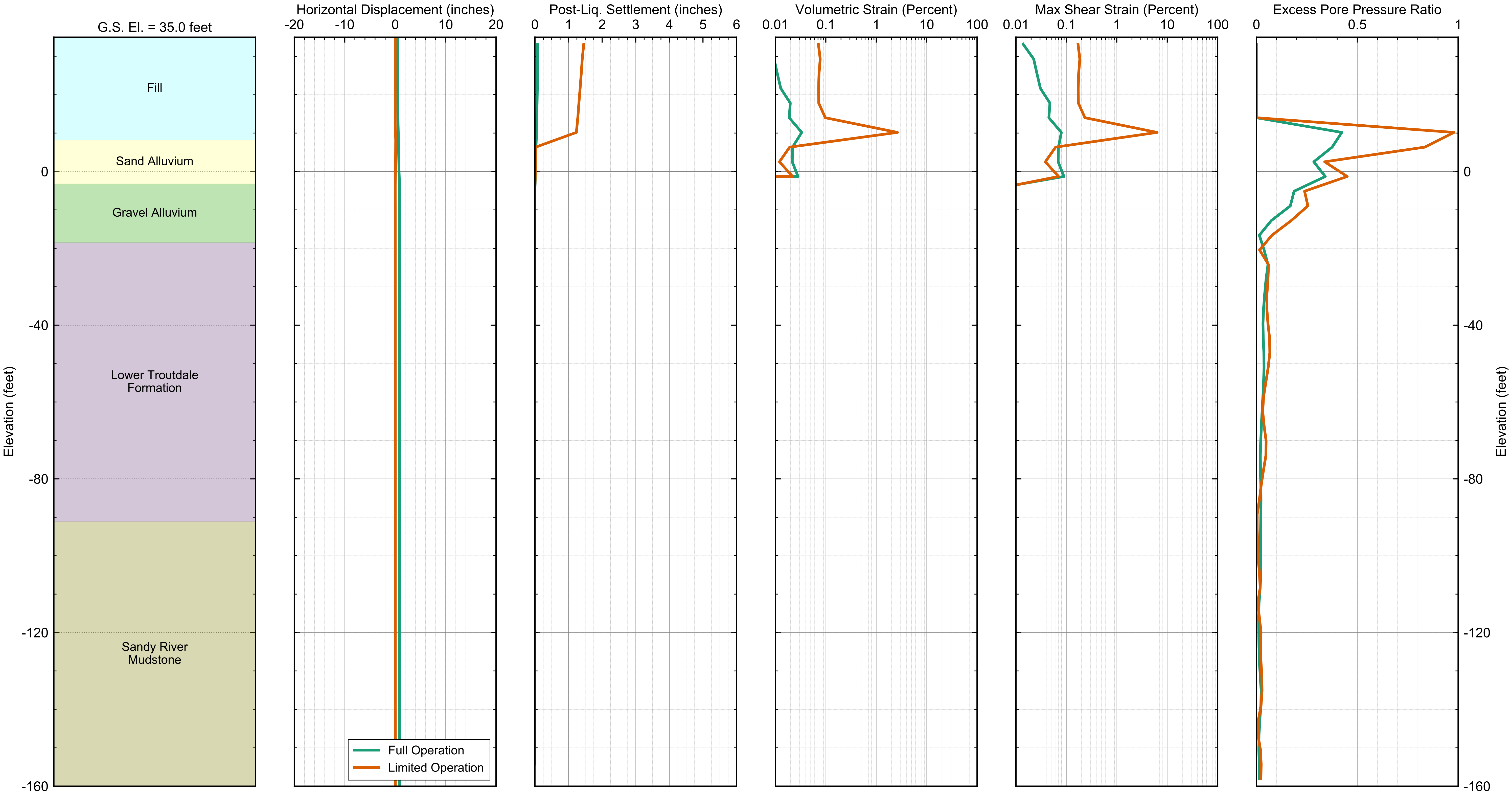
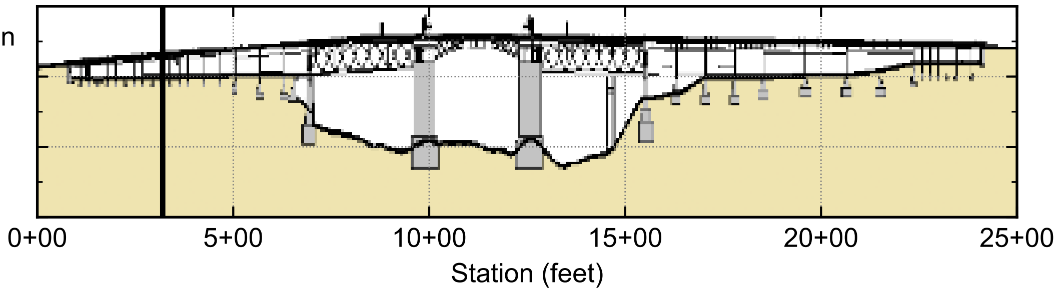


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 11 RETROFIT

August 2019 102636-001

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FIG. E-181

ver:0.8.5.post0.dev0+g414b689_by:pto

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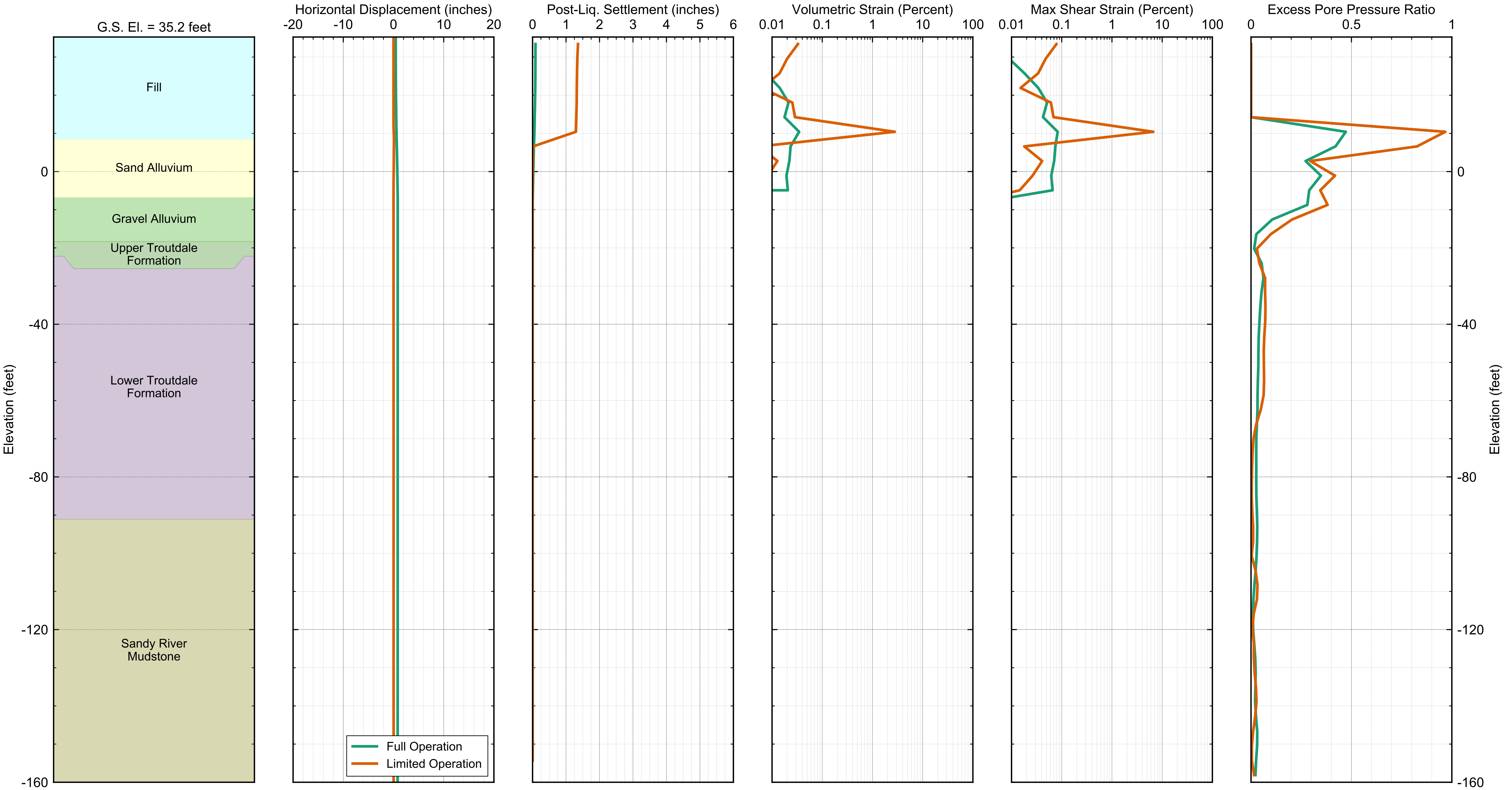
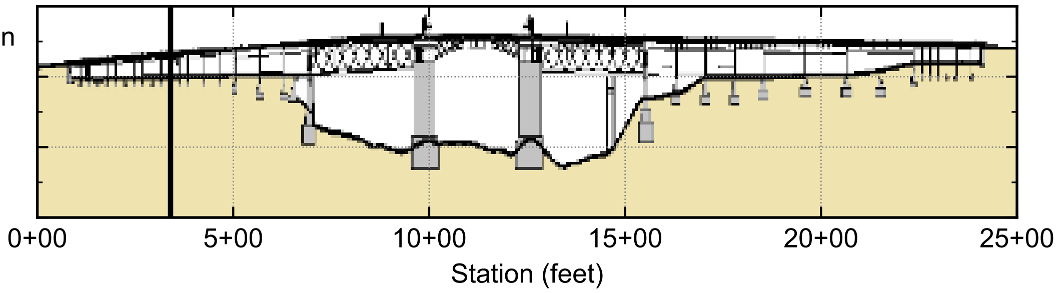


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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**SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 12 RETROFIT**

August 2019 102636-001

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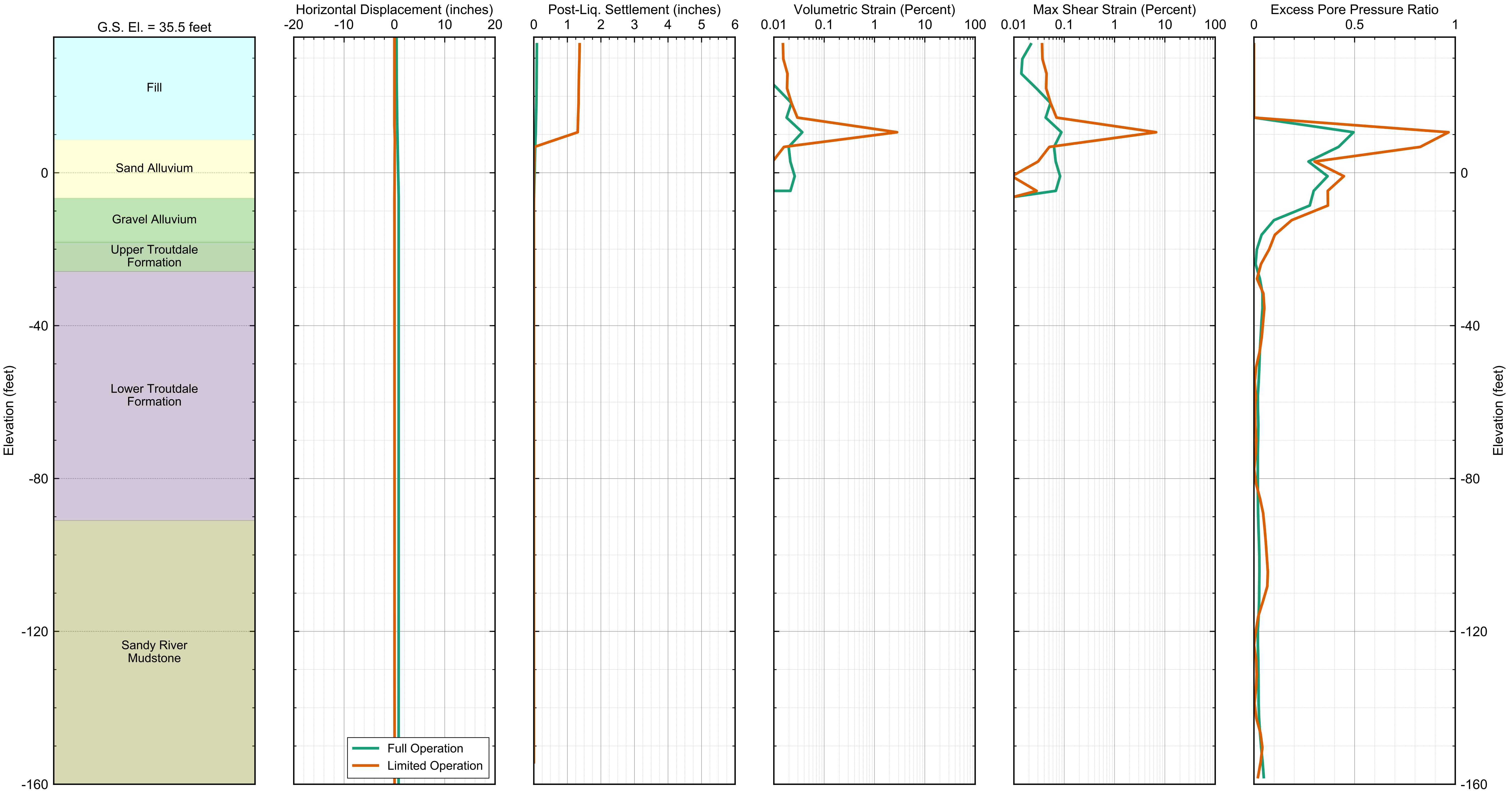
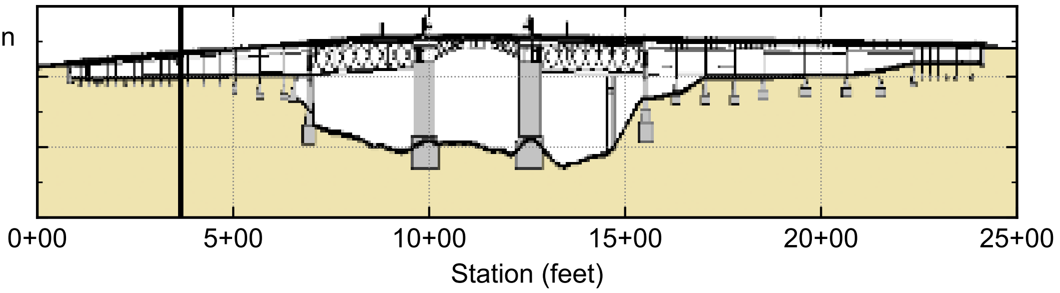
FIG. E-182

ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 13 RETROFIT

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FIG. E-183

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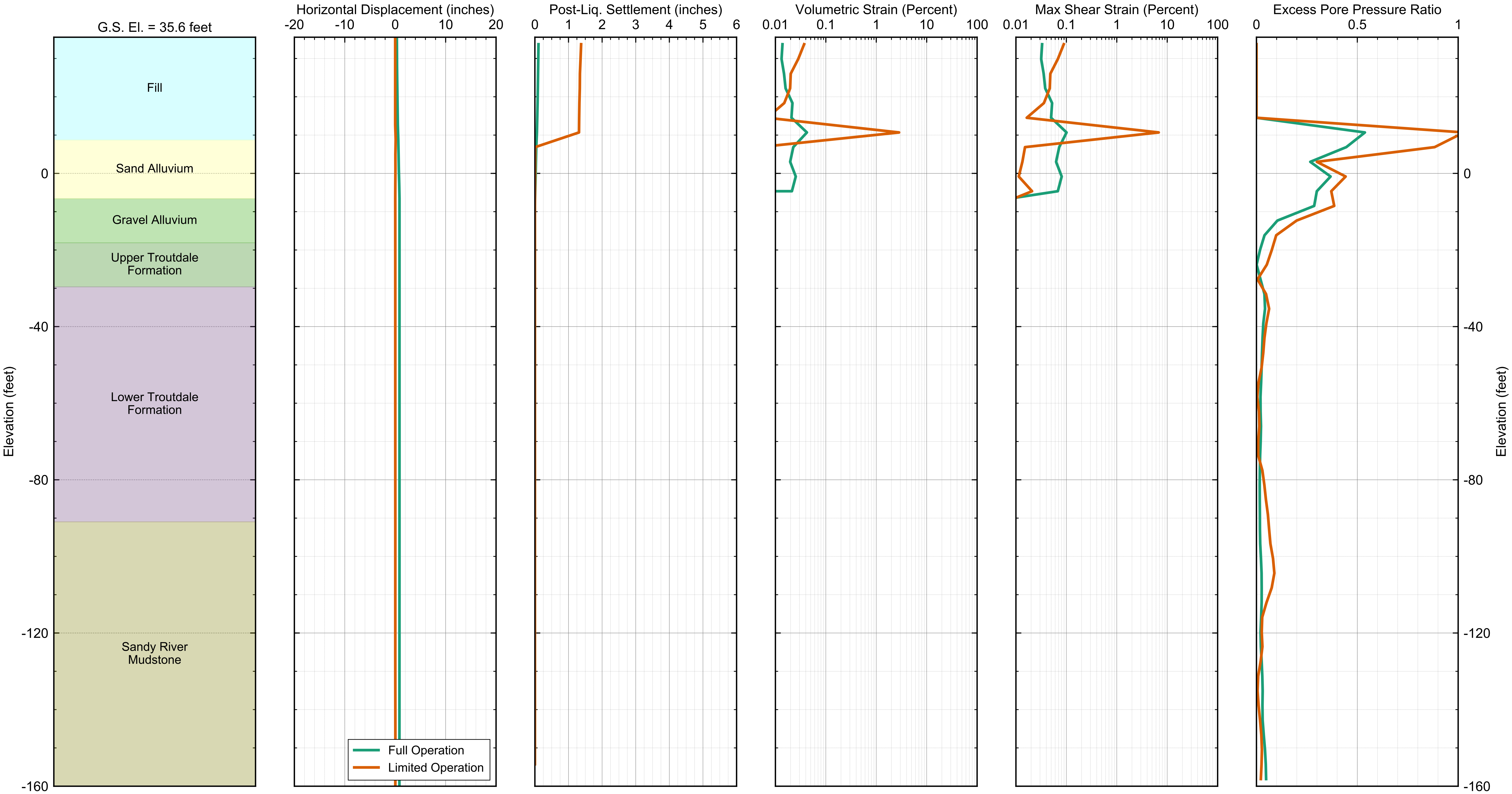
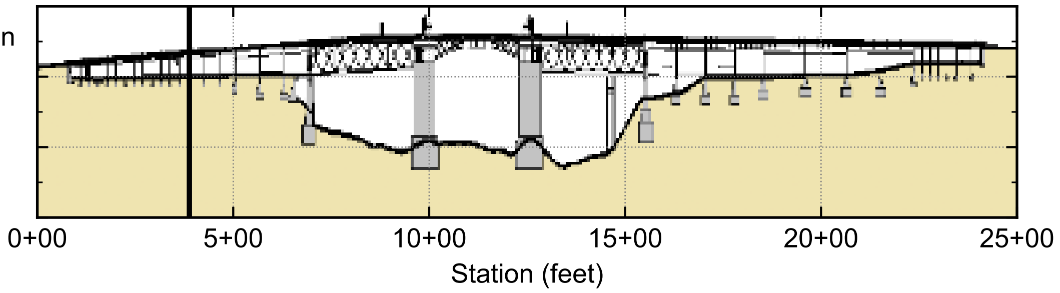
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 14 RETROFIT

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FIG. E-184

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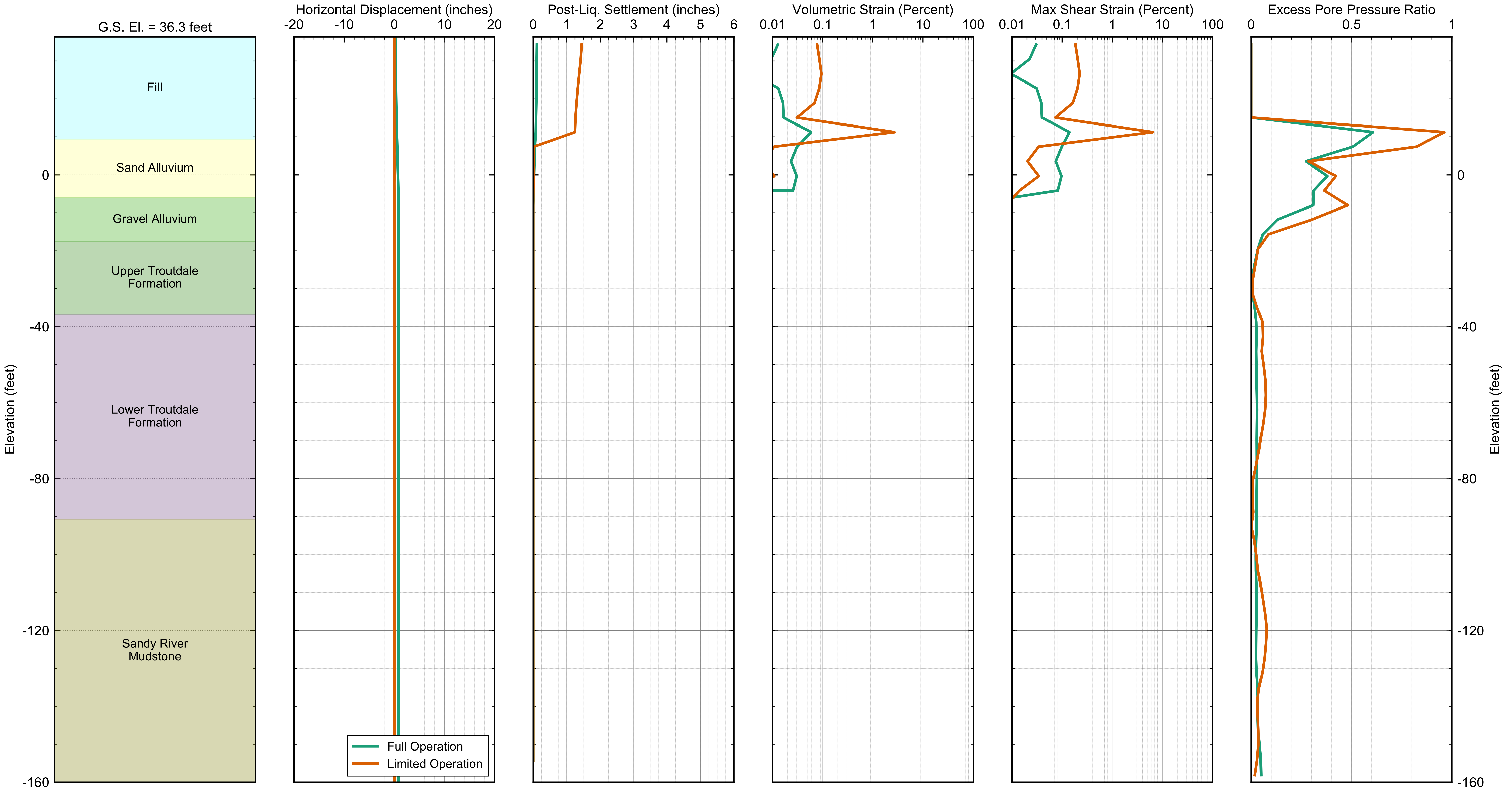
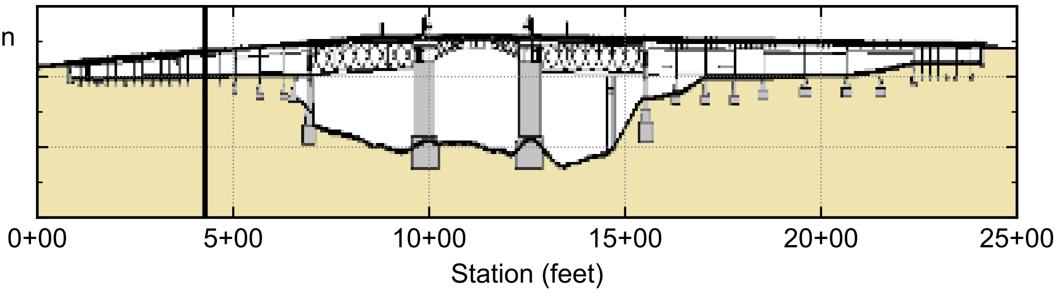


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 15 RETROFIT

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FIG. E-185

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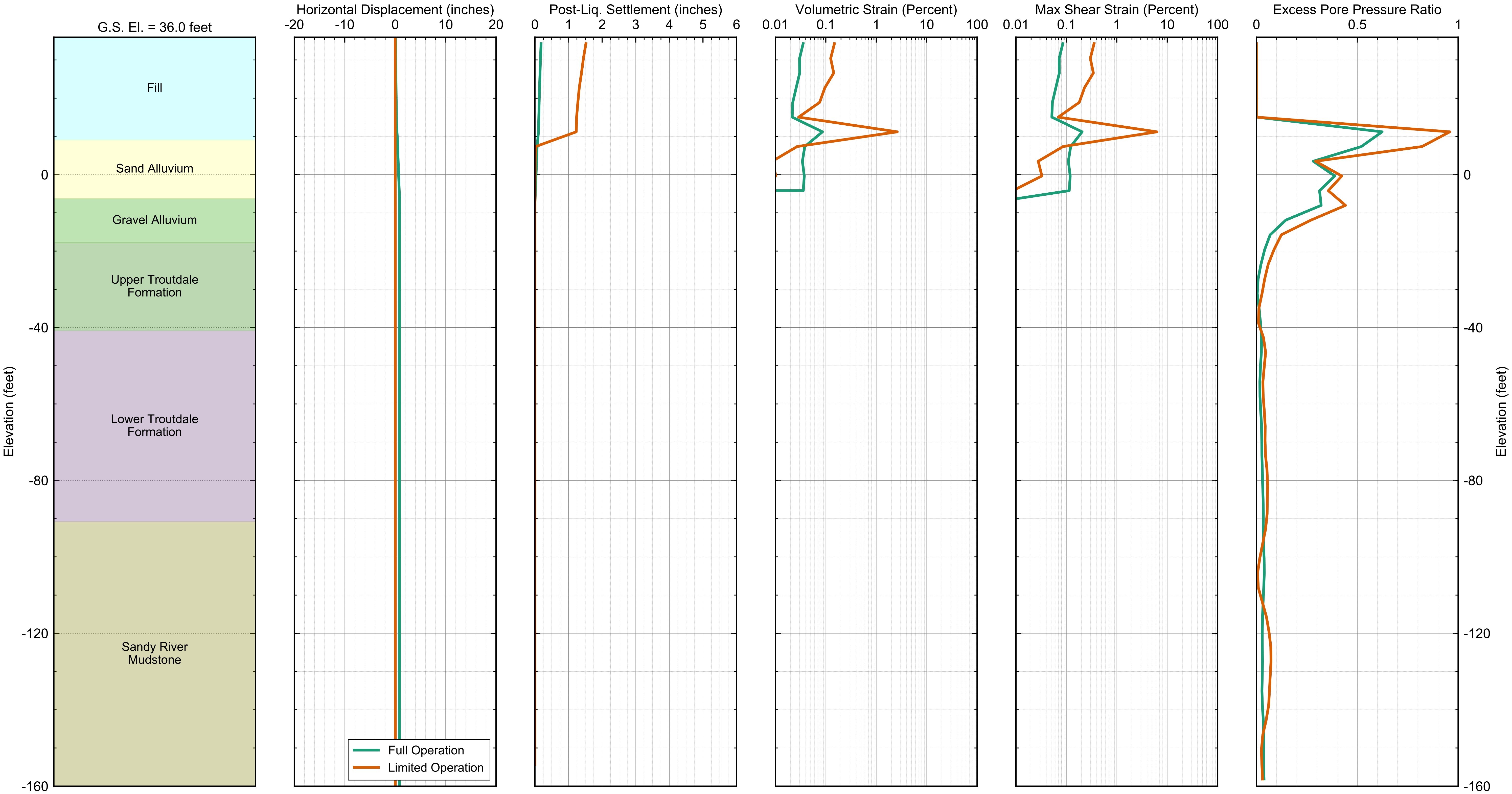
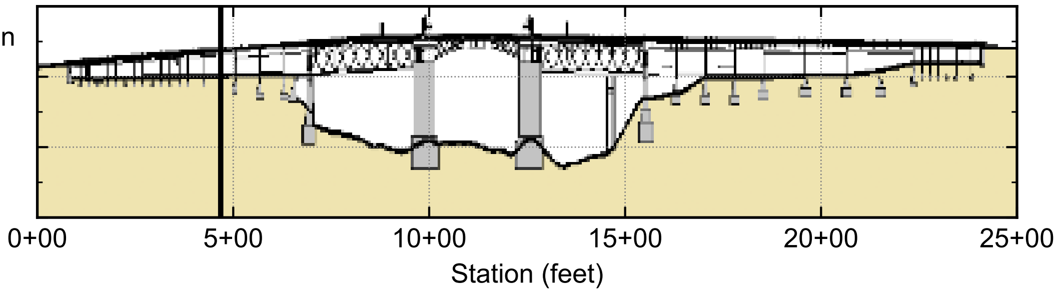
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 16 RETROFIT

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FIG. E-186

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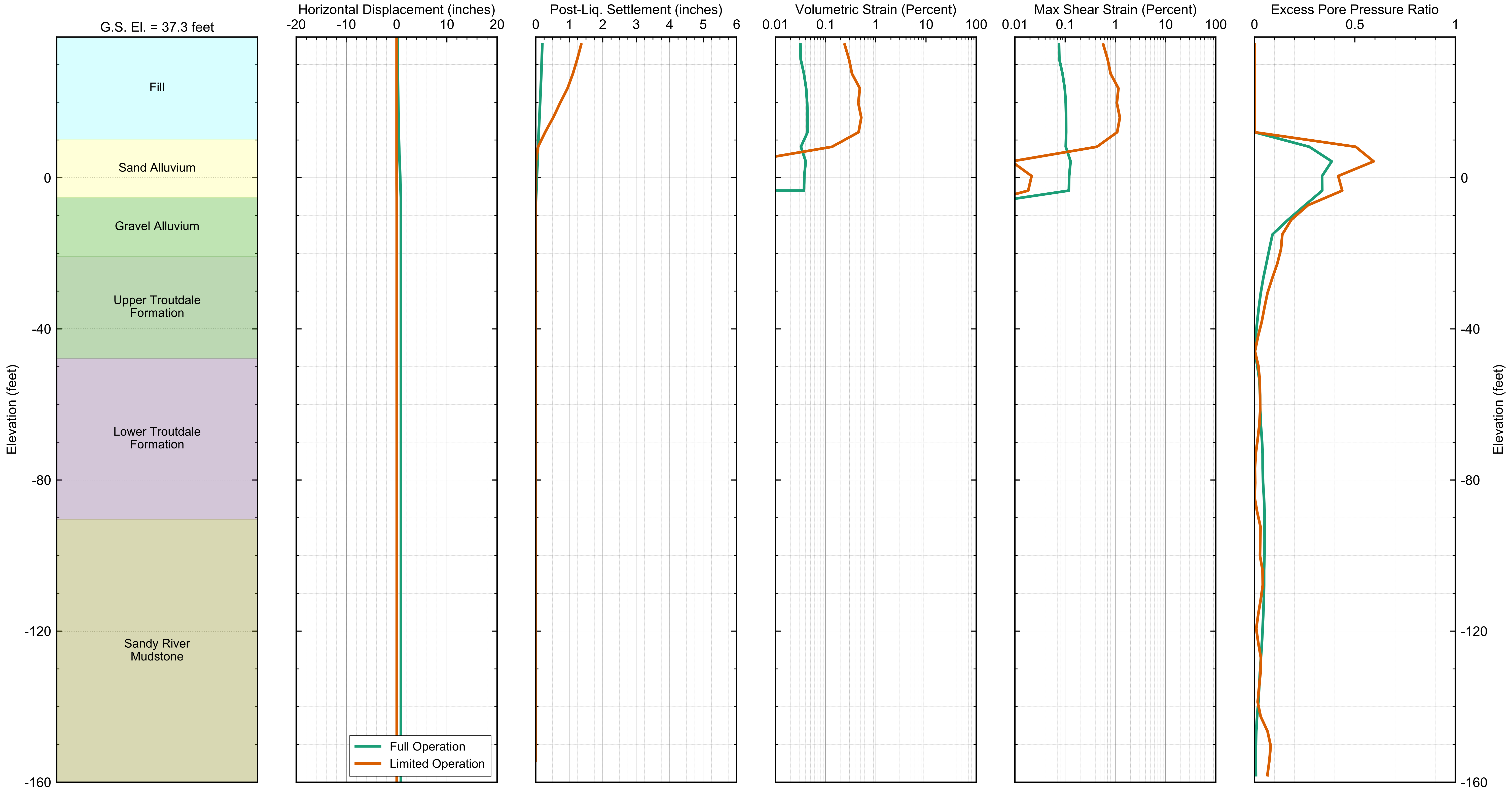
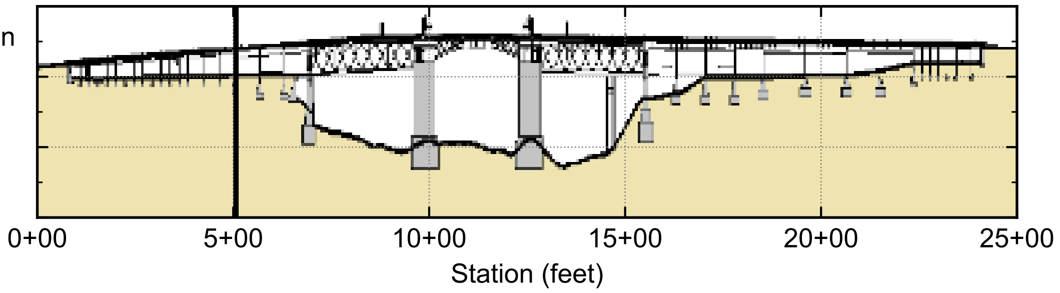


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 17 RETROFIT

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FIG. E-187

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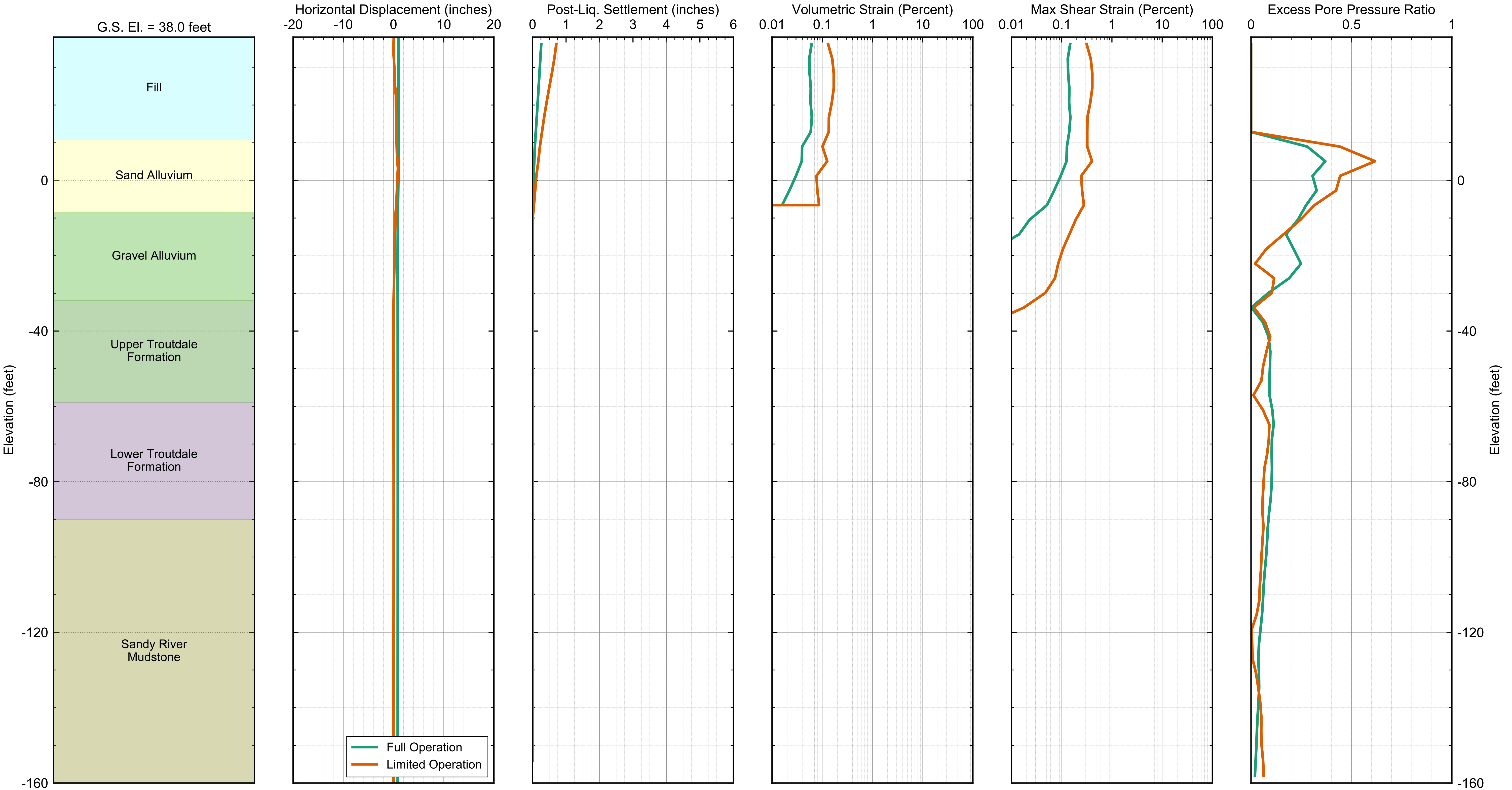
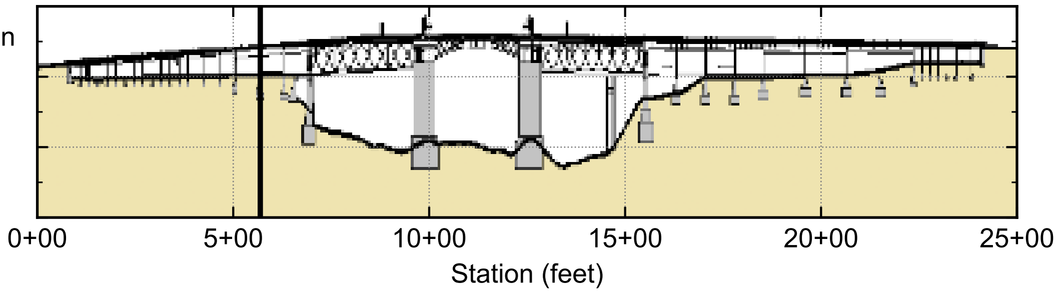
ver:0.8.5.post0.dev0+g414b689_bv:pto



Notes:

1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 18 RETROFIT

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FIG. E-188

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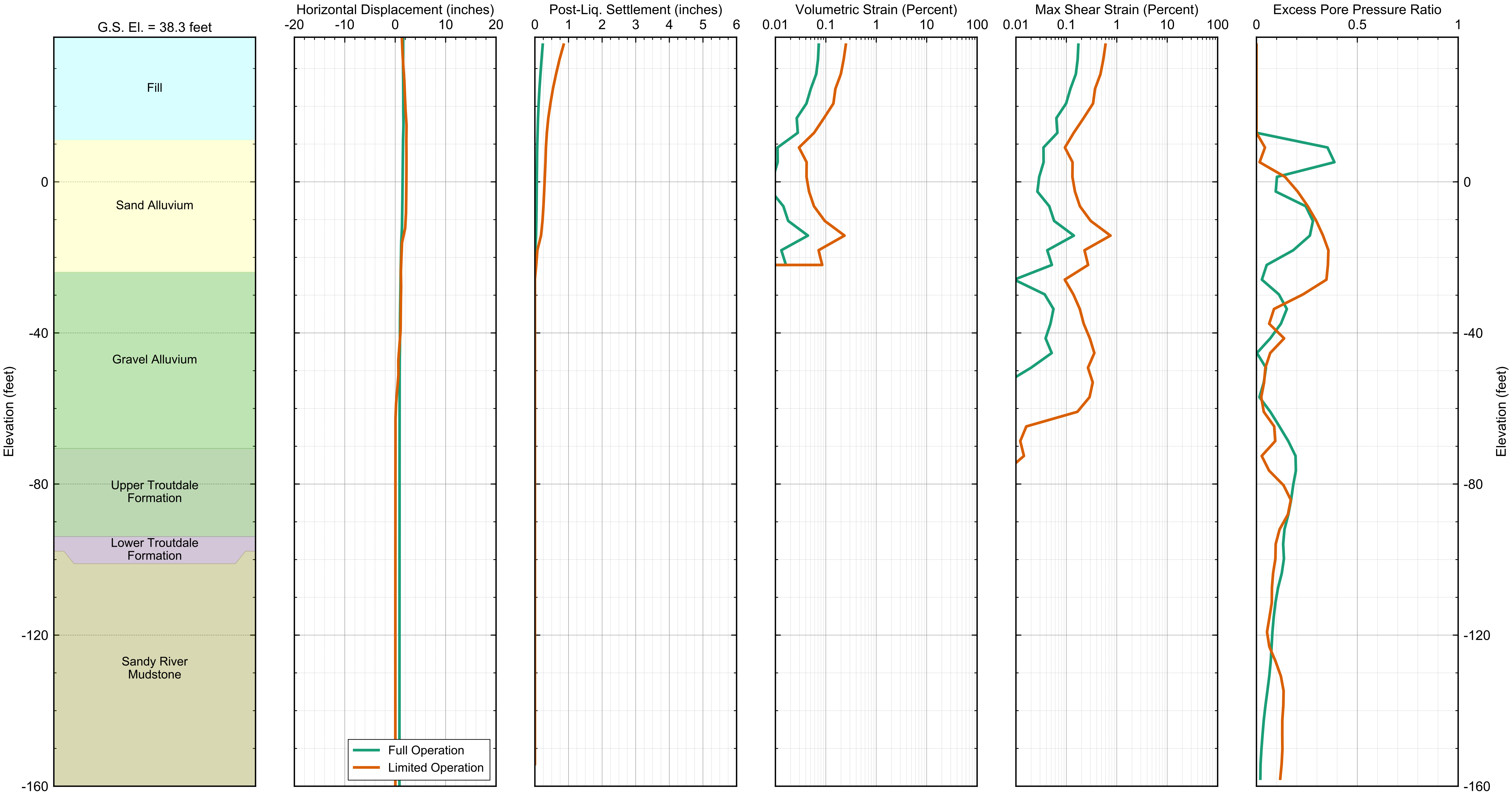
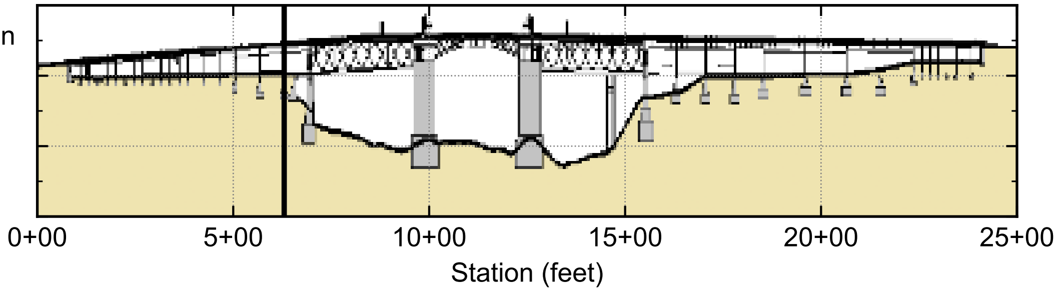


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 19 RETROFIT

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FIG. E-189

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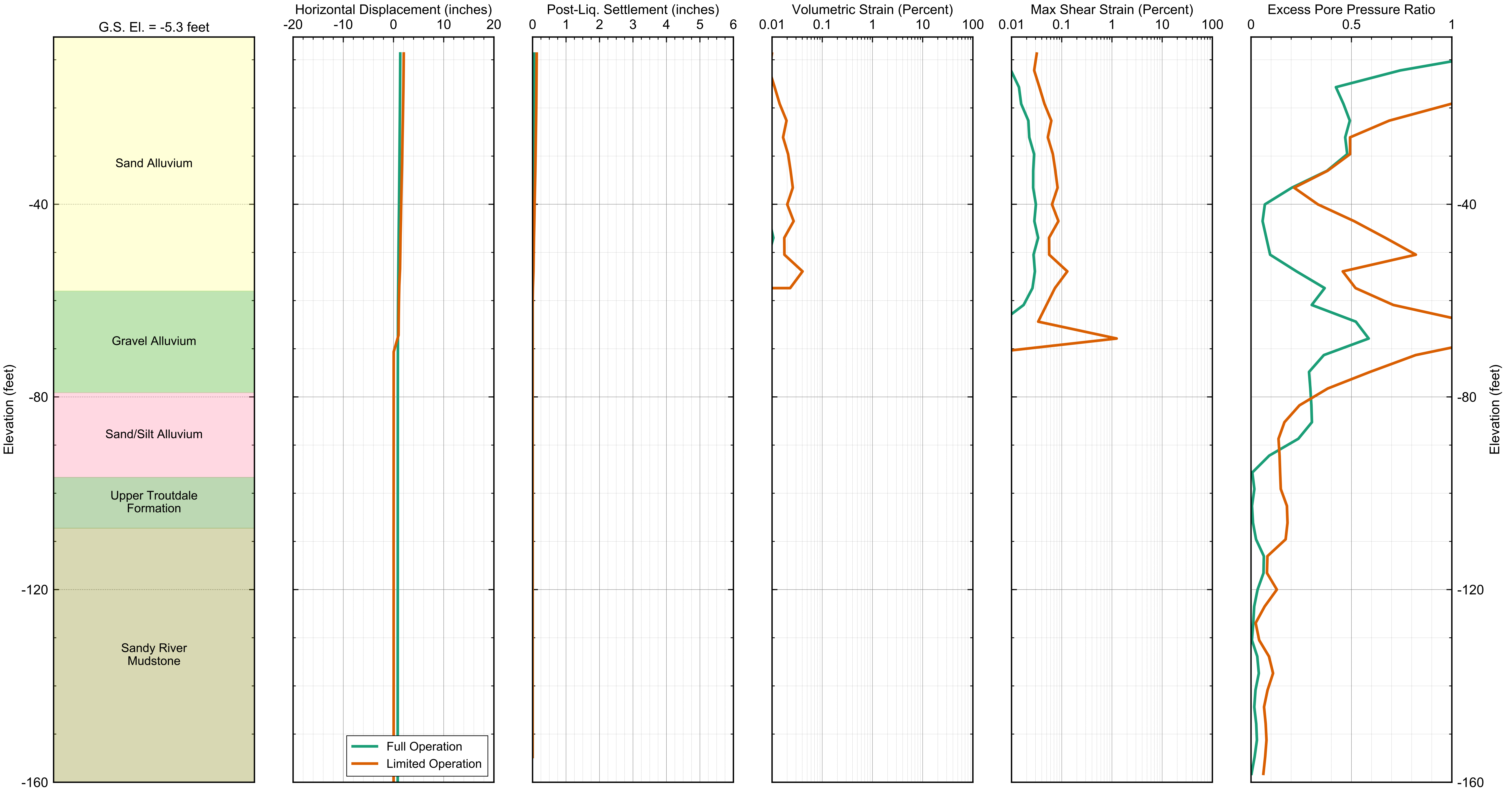
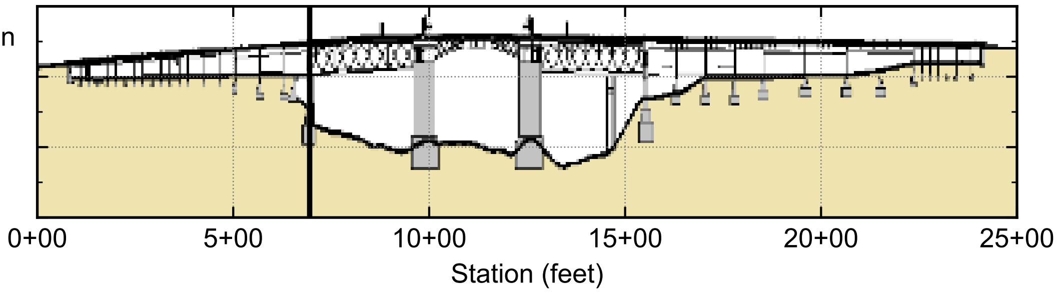


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT PIER 1 RETROFIT

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FIG. E-190

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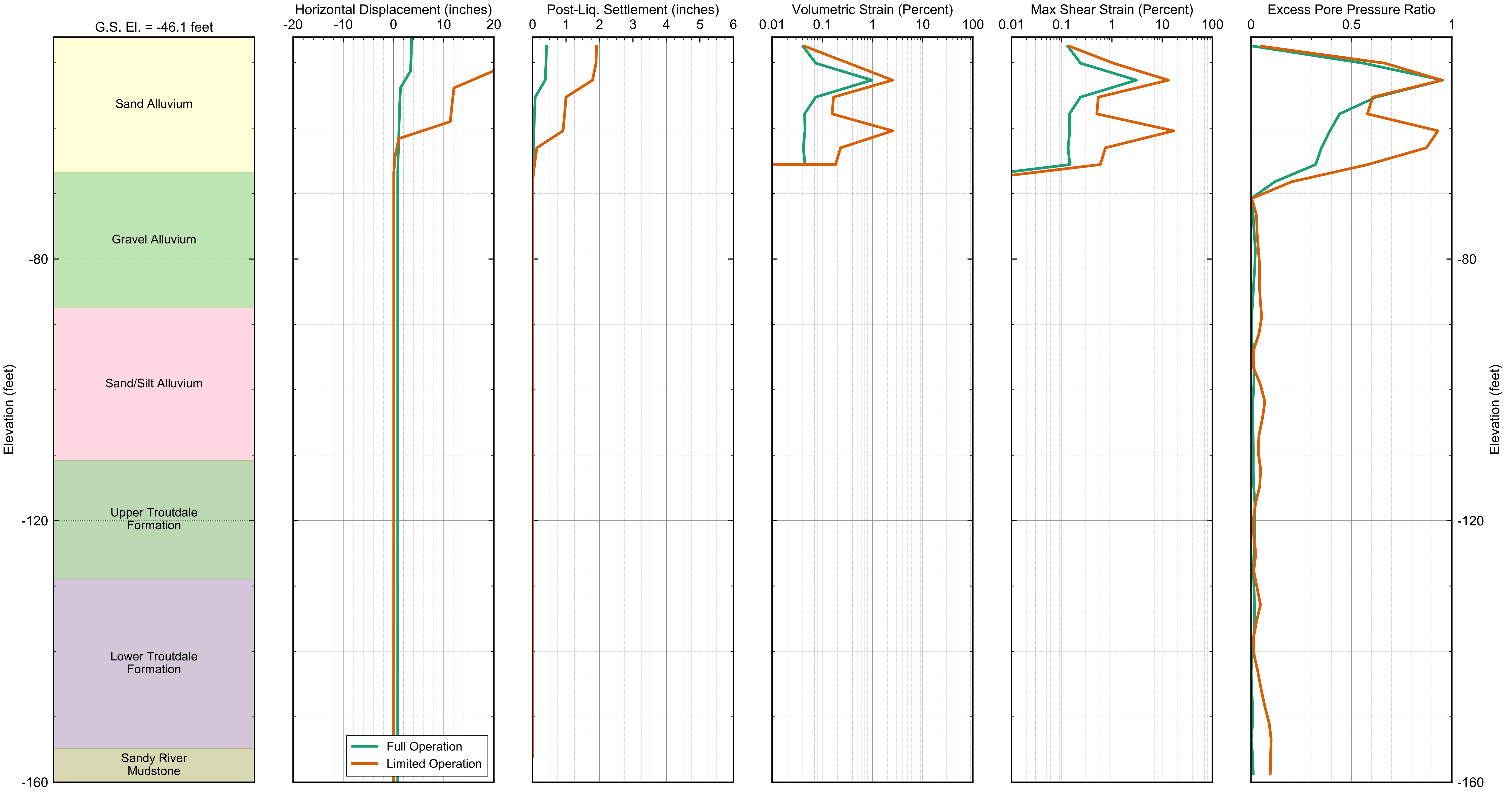
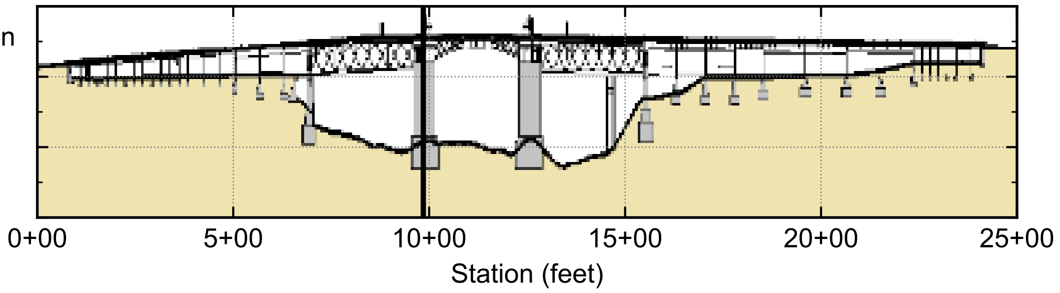


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT PIER 2 RETROFIT

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FIG. E-191

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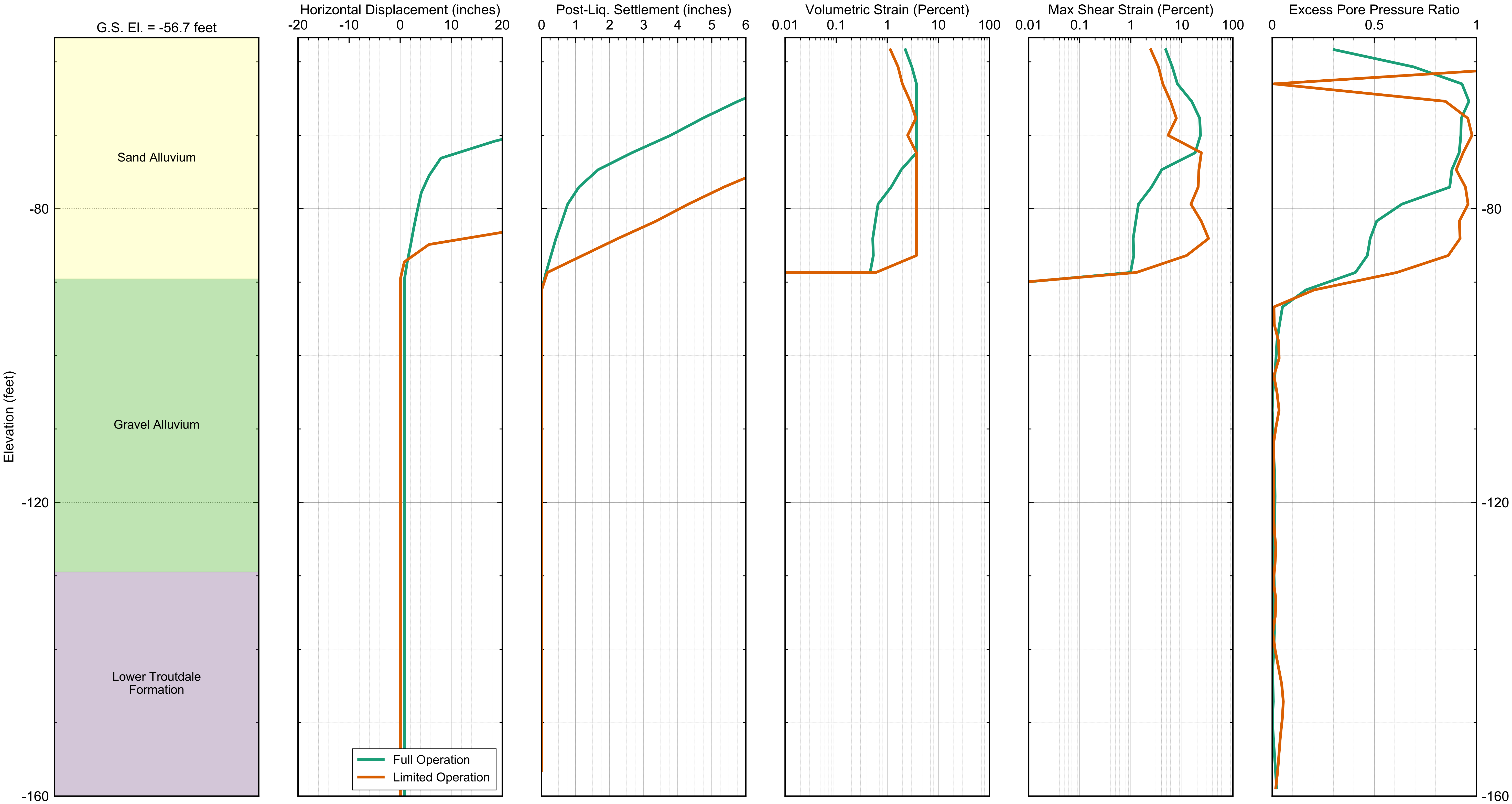
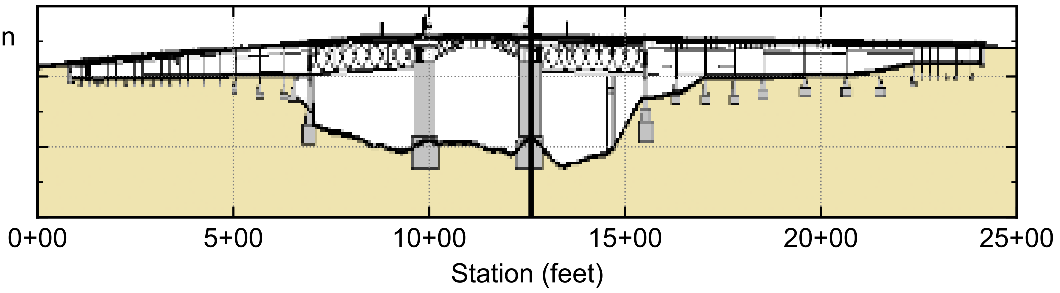
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
PIER 3 RETROFIT

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FIG. E-192

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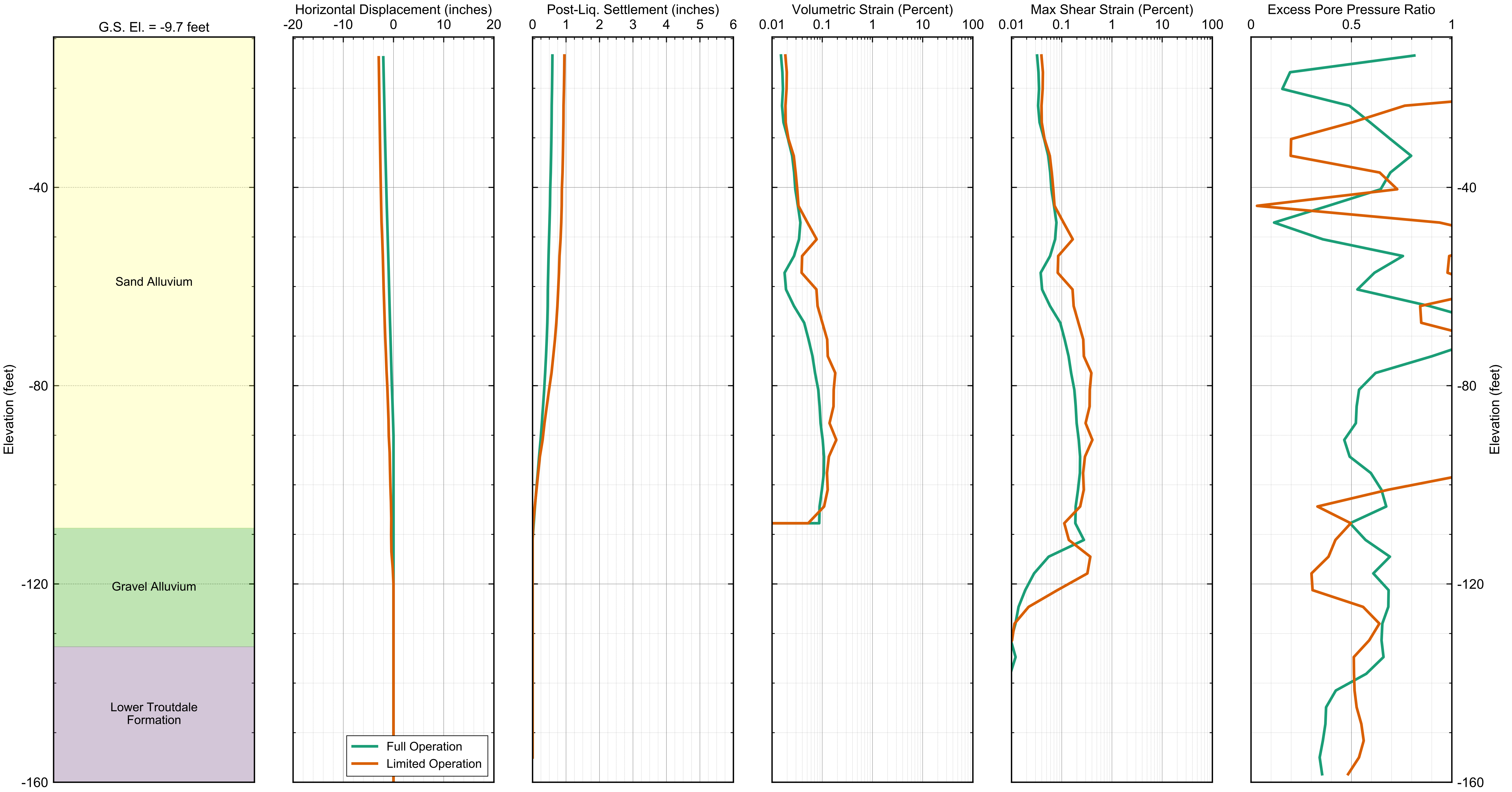
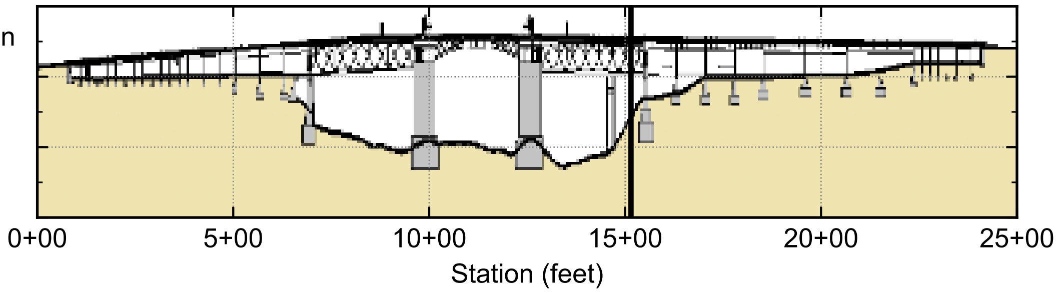
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT PIER 4 RETROFIT

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FIG. E-193

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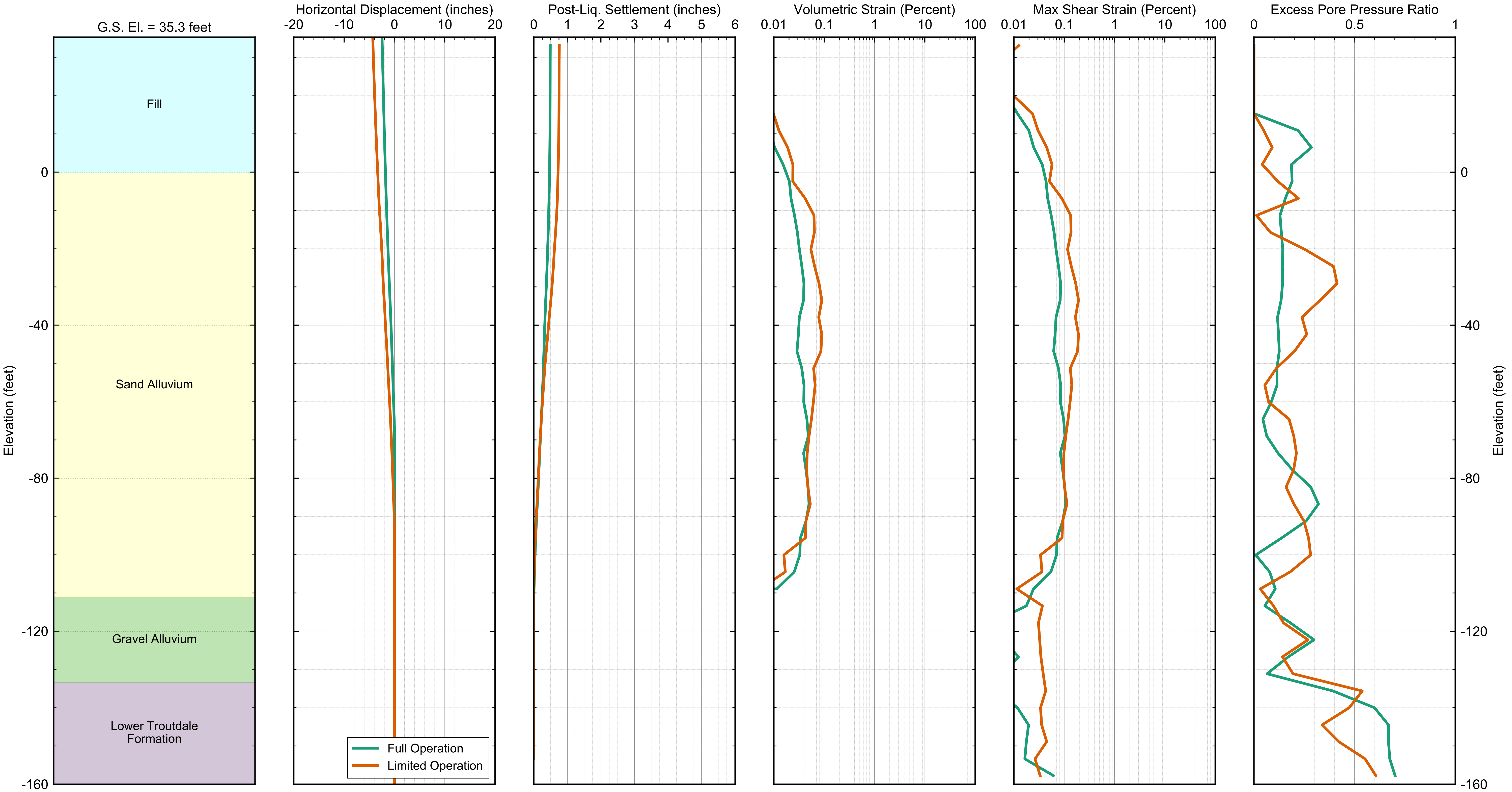
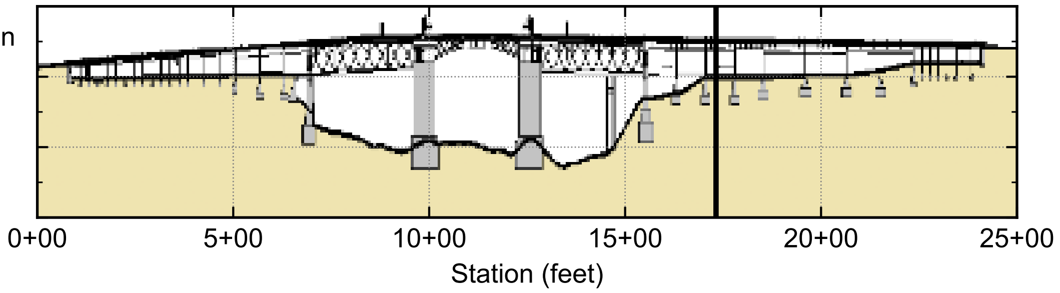
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 23 RETROFIT

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FIG. E-194

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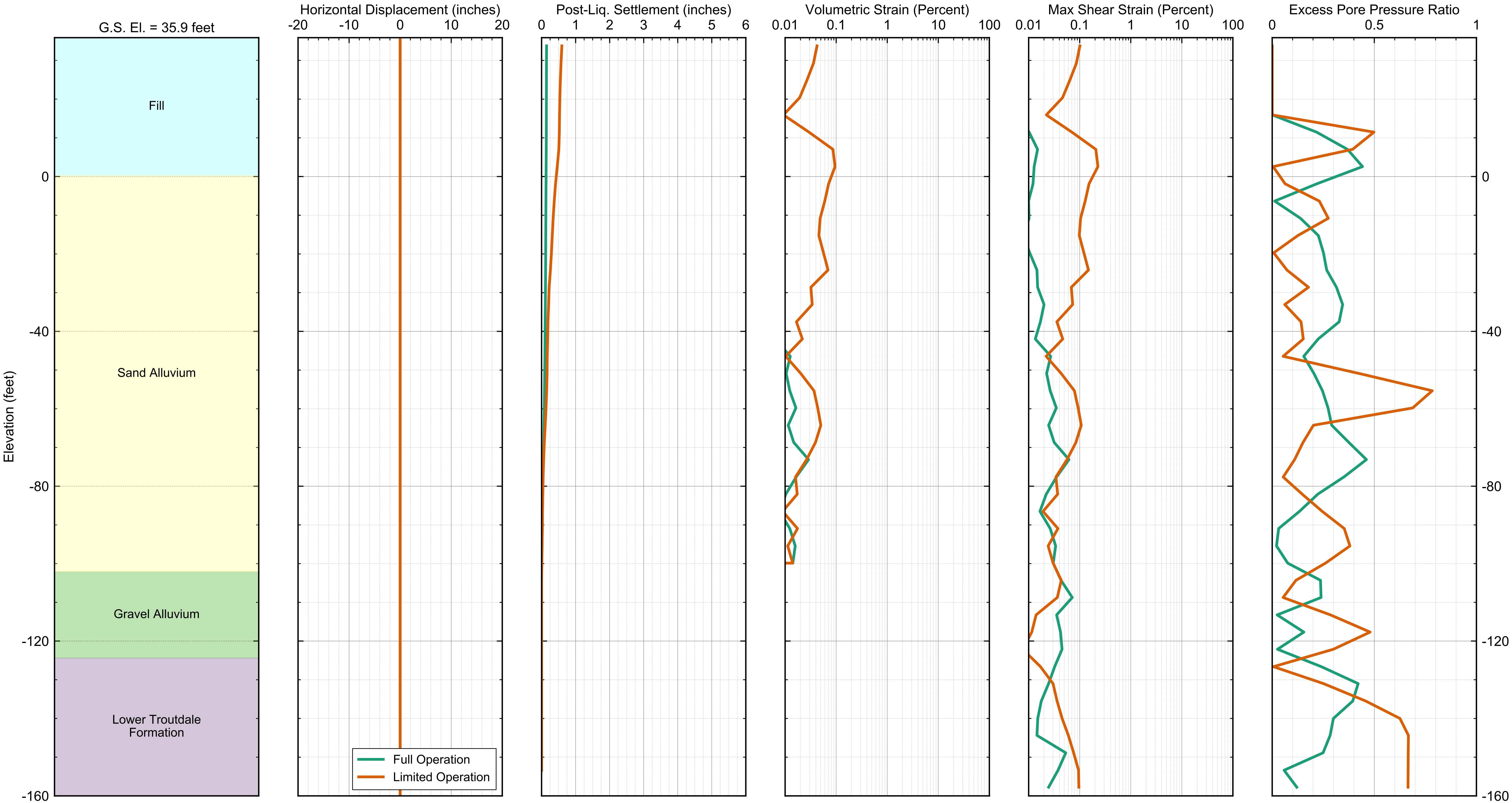
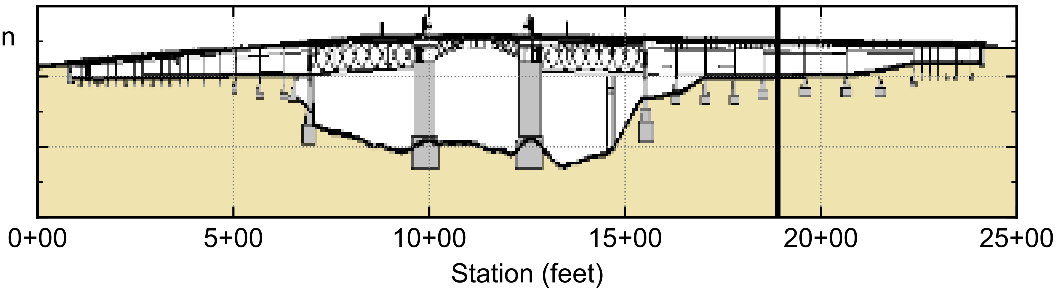
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES
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BENT 24 RETROFIT

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FIG. E-195

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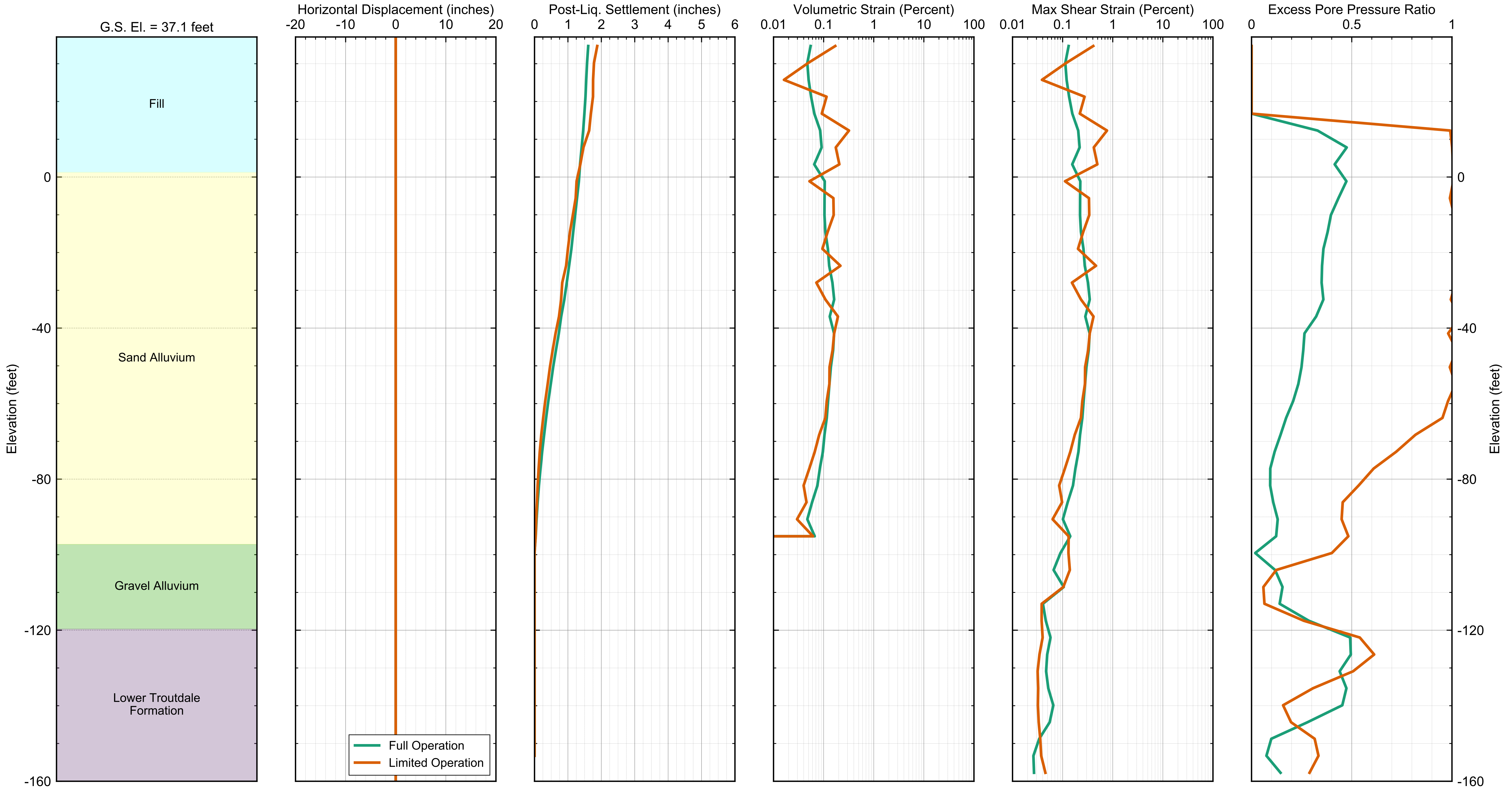
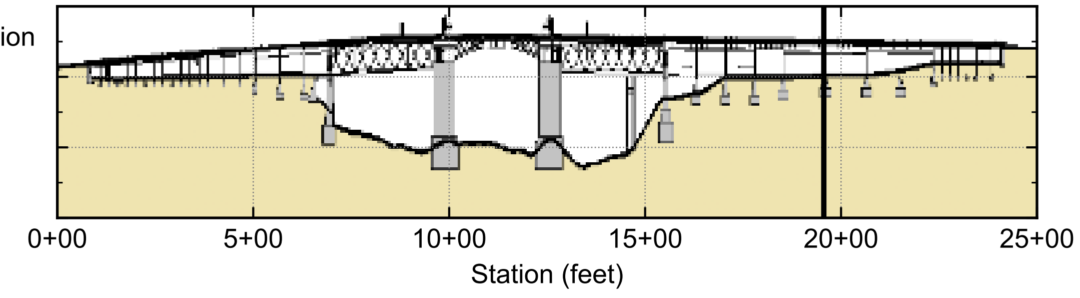
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 25 RETROFIT

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FIG. E-196

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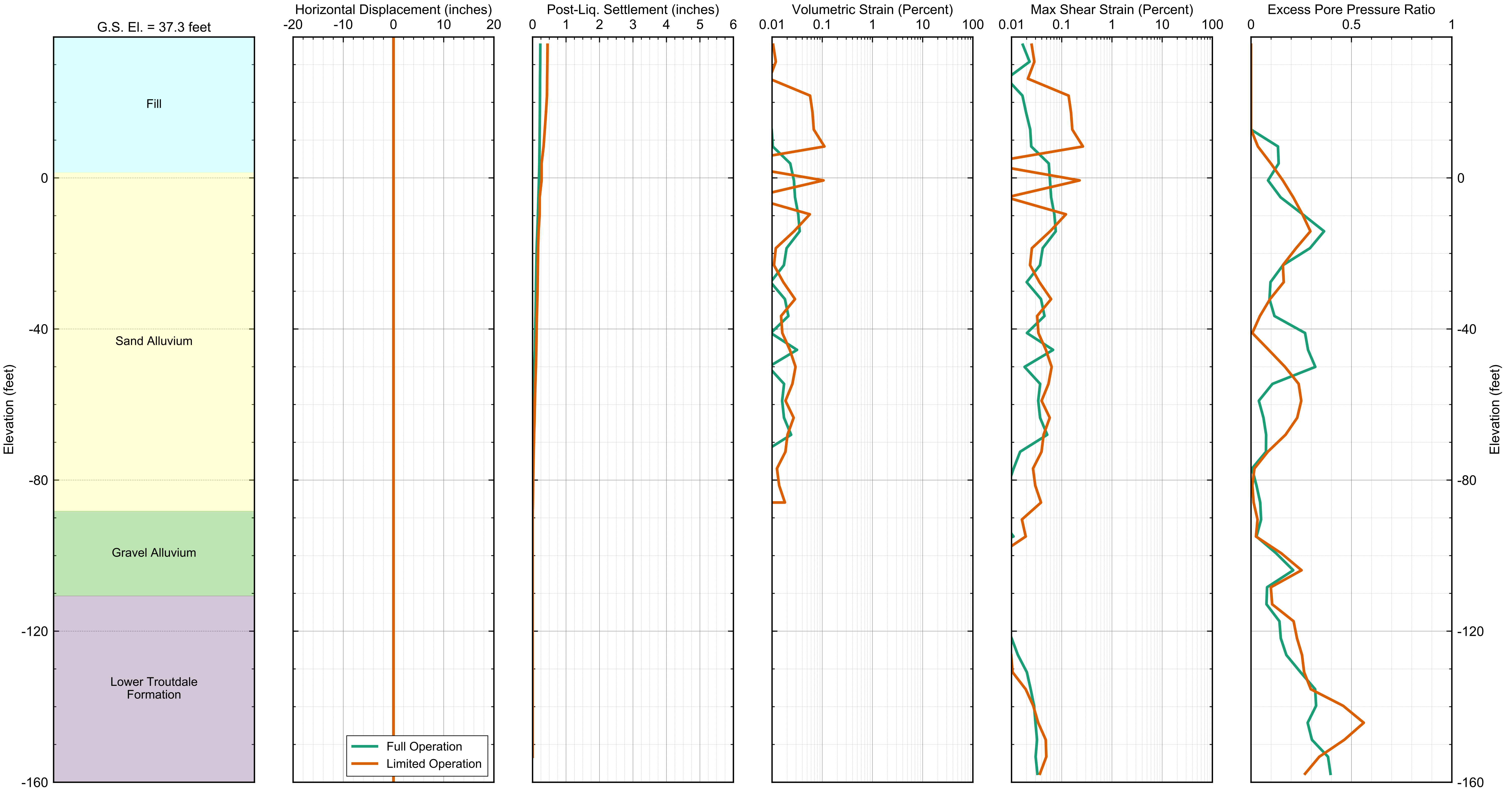
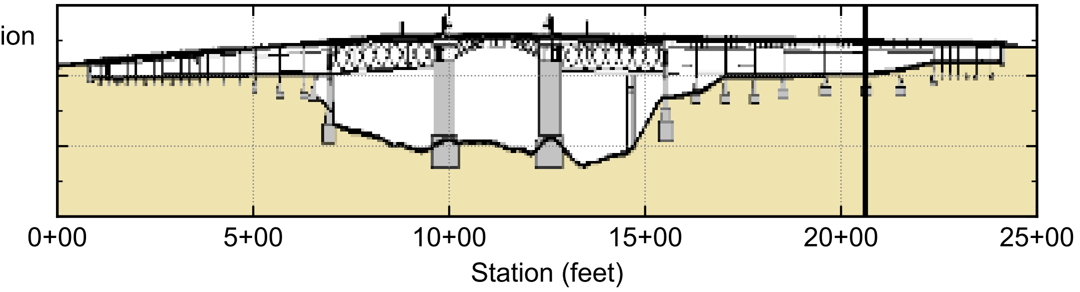


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 26 RETROFIT

August 2019
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FIG. E-197

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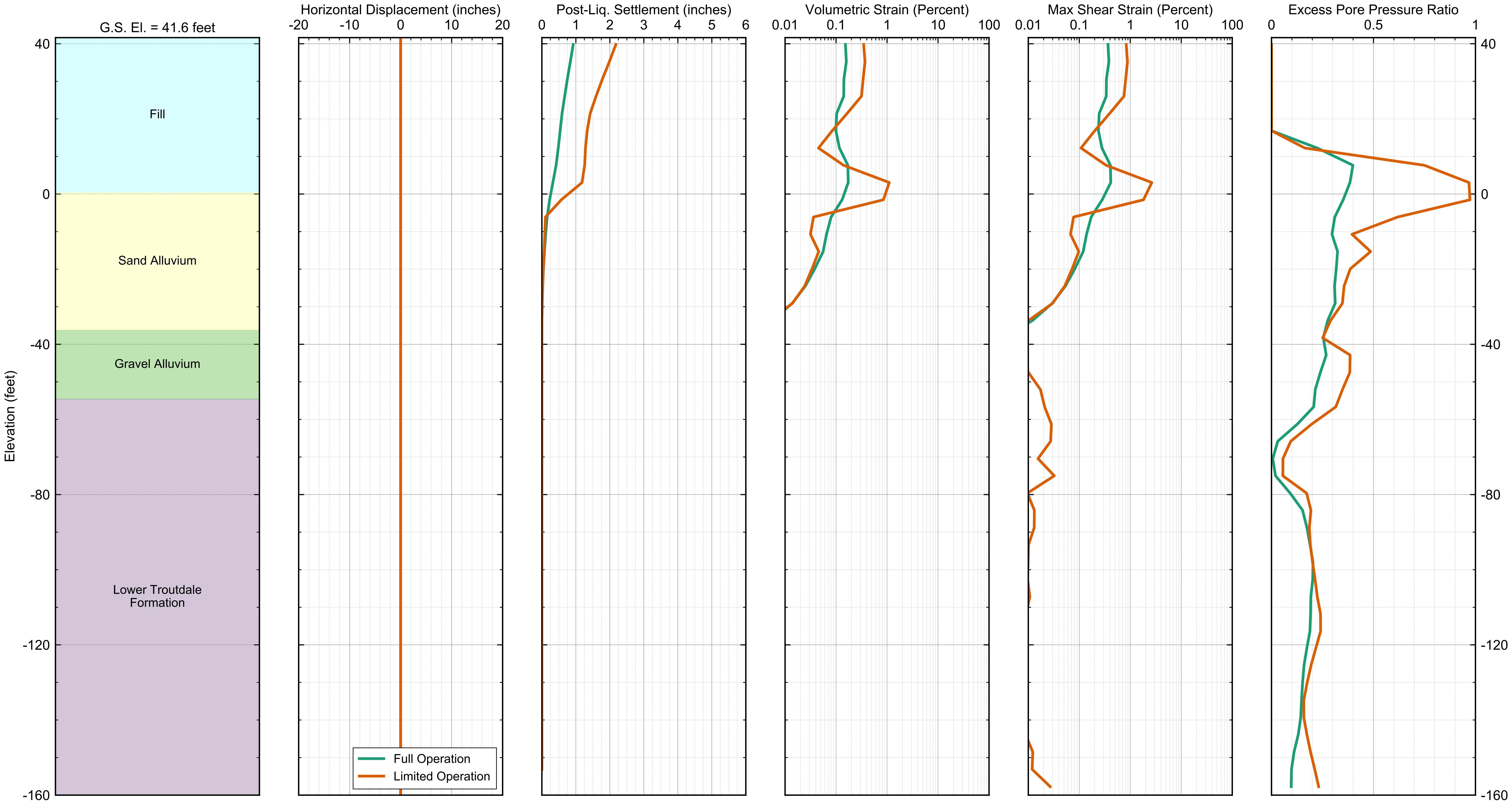
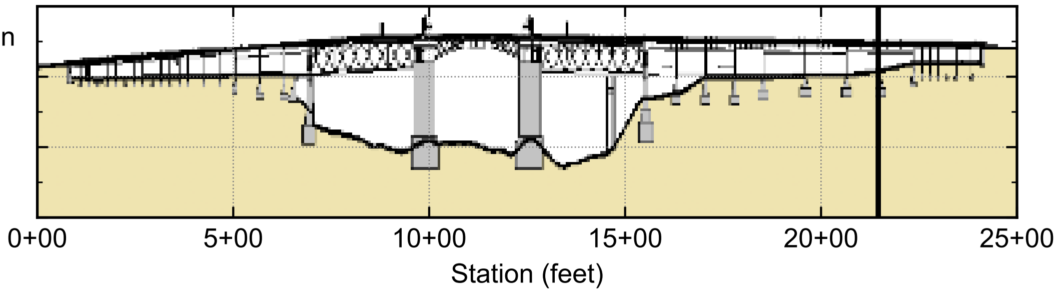


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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**SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 27 RETROFIT**

August 2019
102636-001

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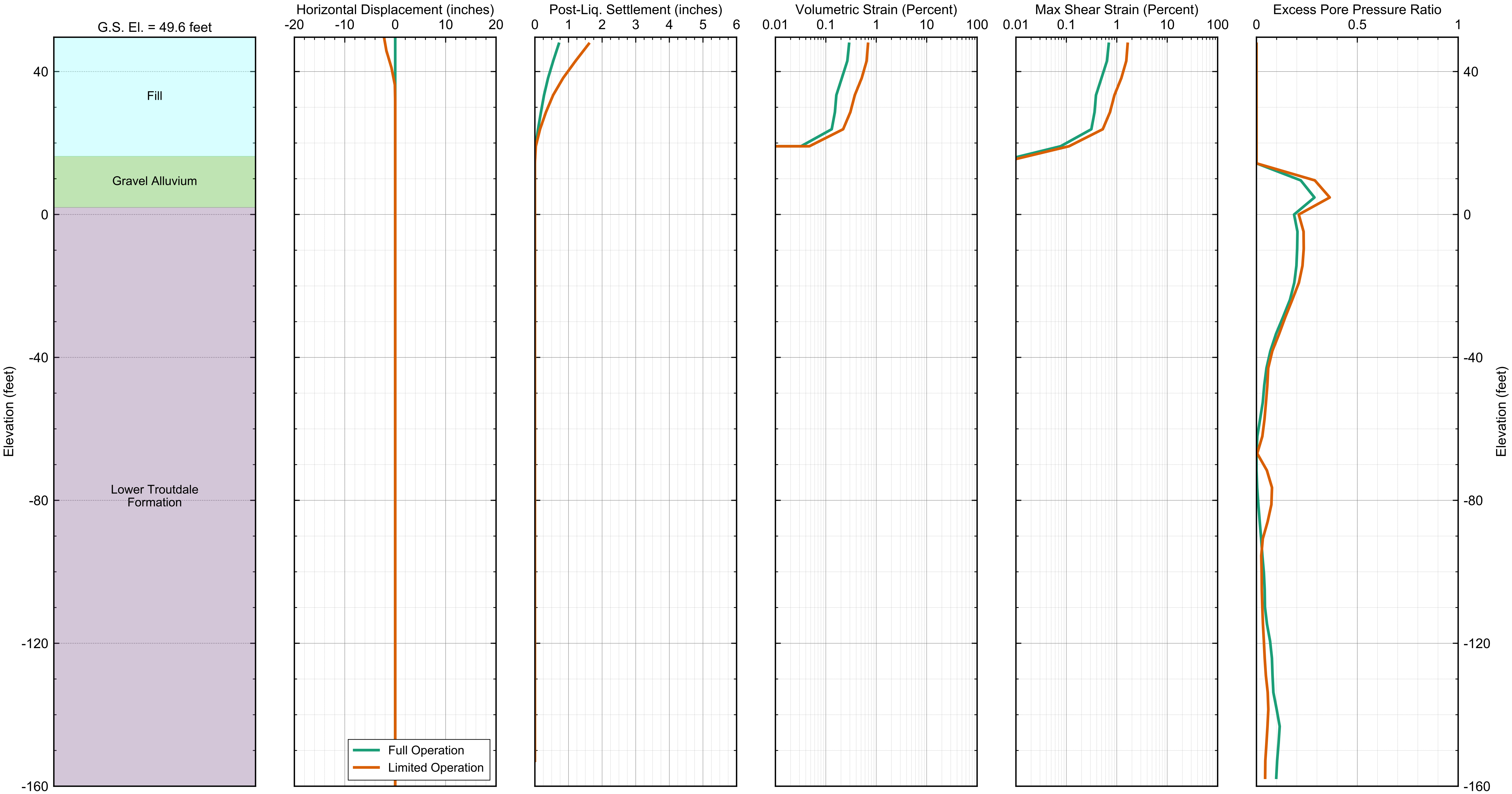
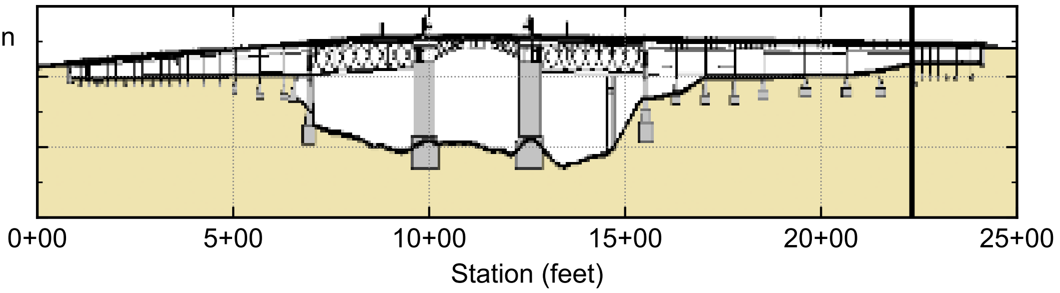
FIG. E-198

ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 28 RETROFIT

August 2019
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FIG. E-199

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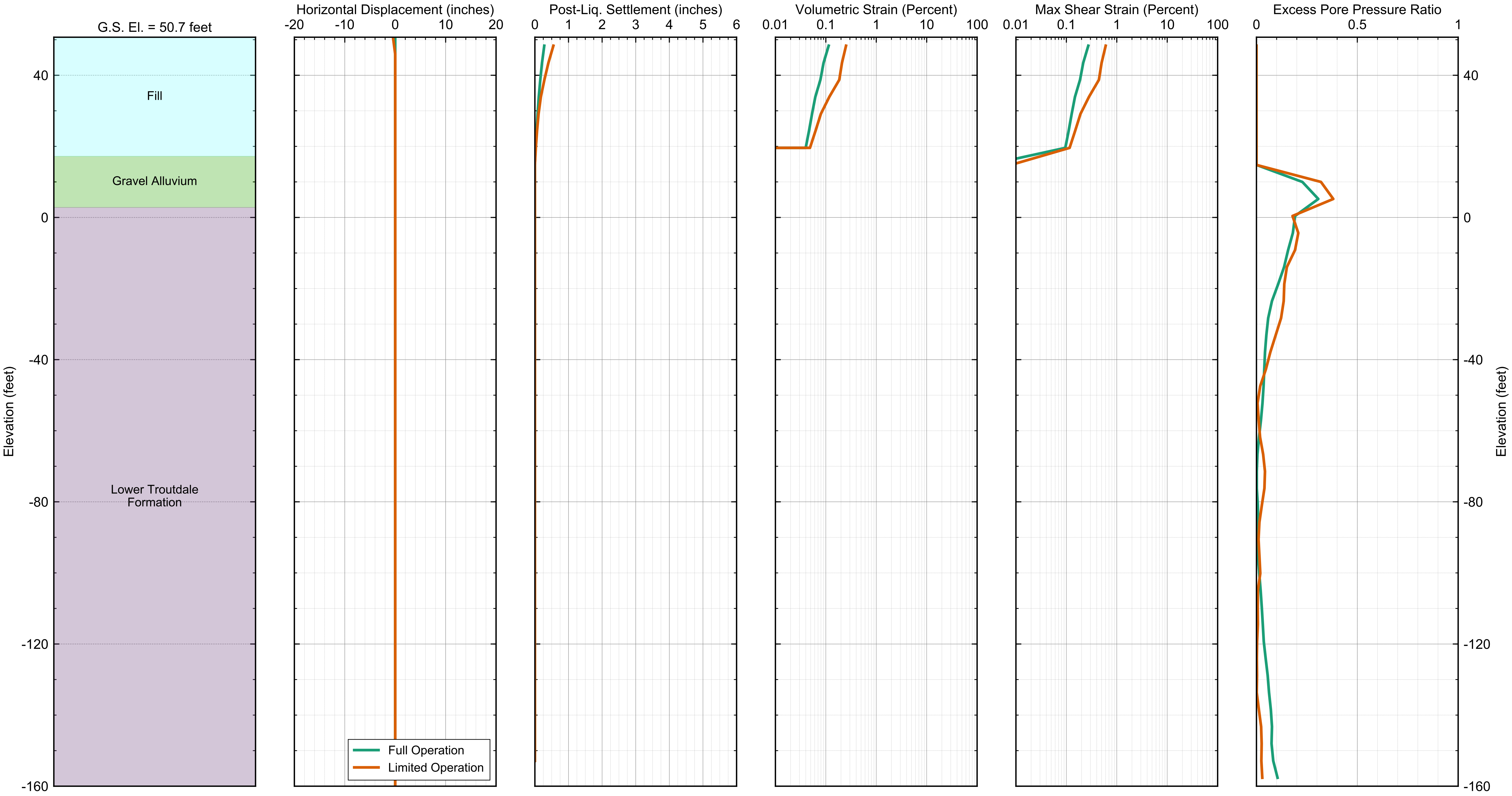
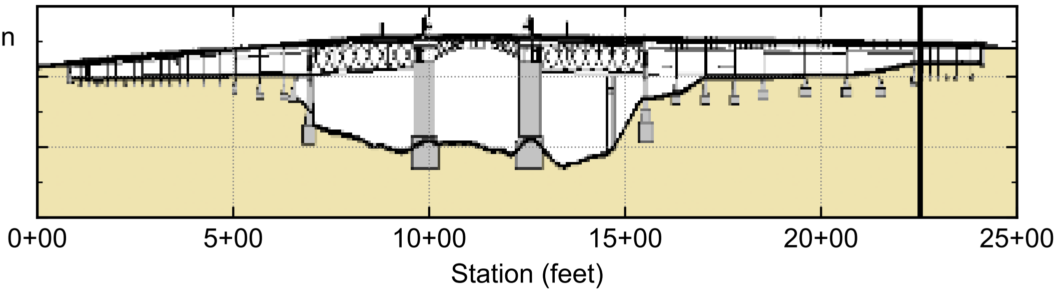


Notes:

1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 29 RETROFIT

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FIG. E-200

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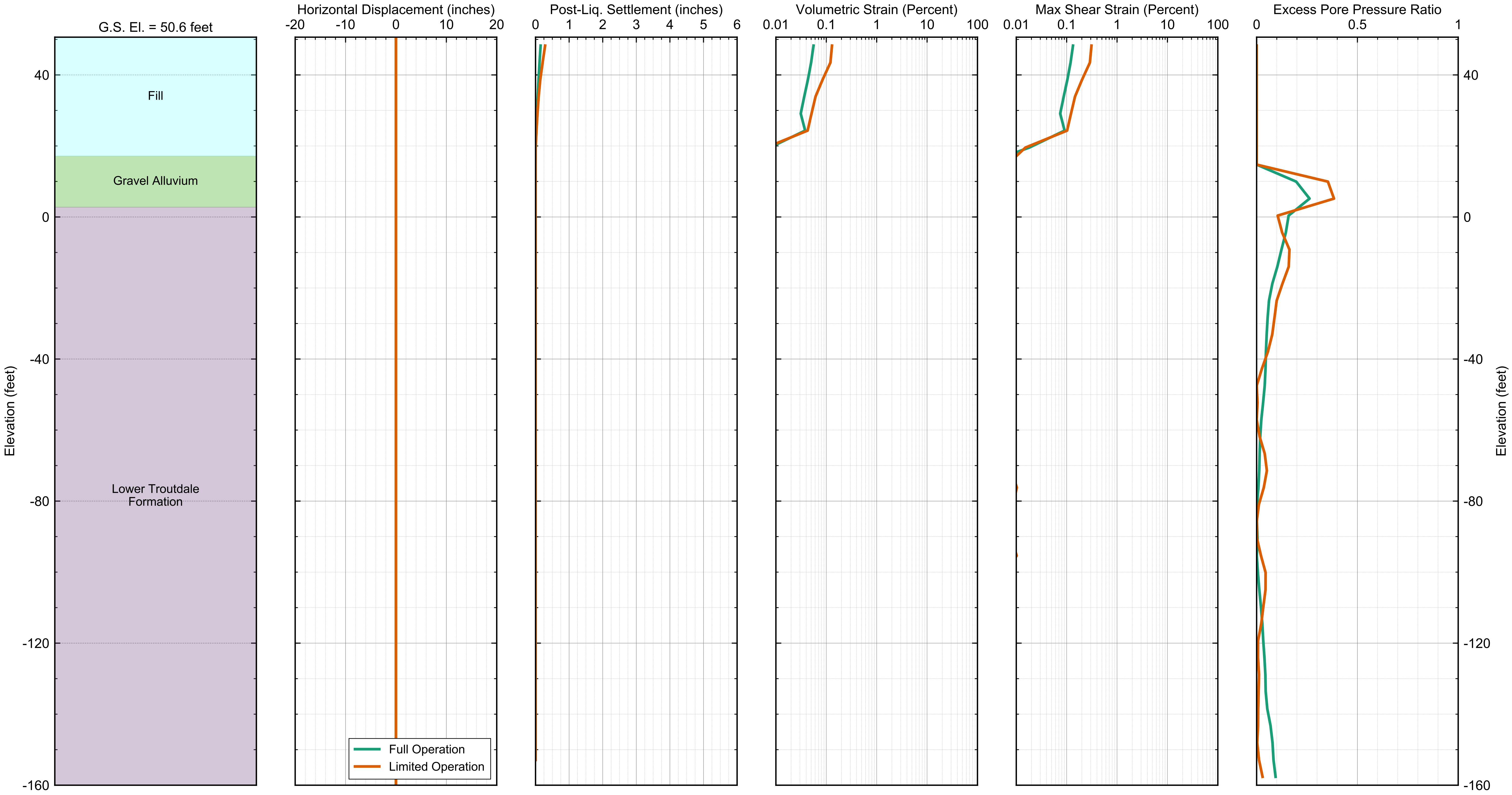
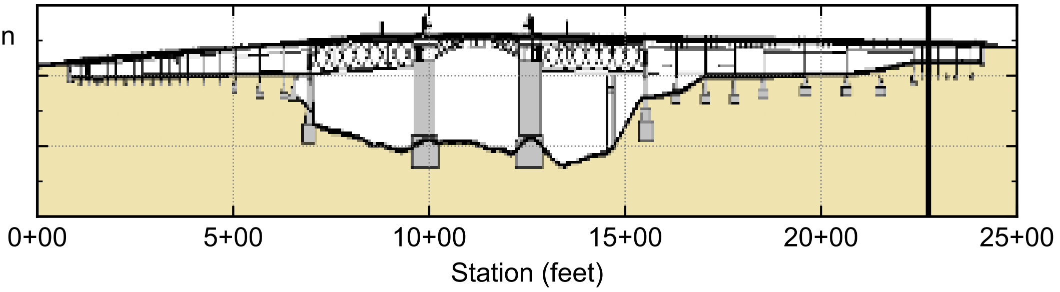


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 30 RETROFIT

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FIG. E-201

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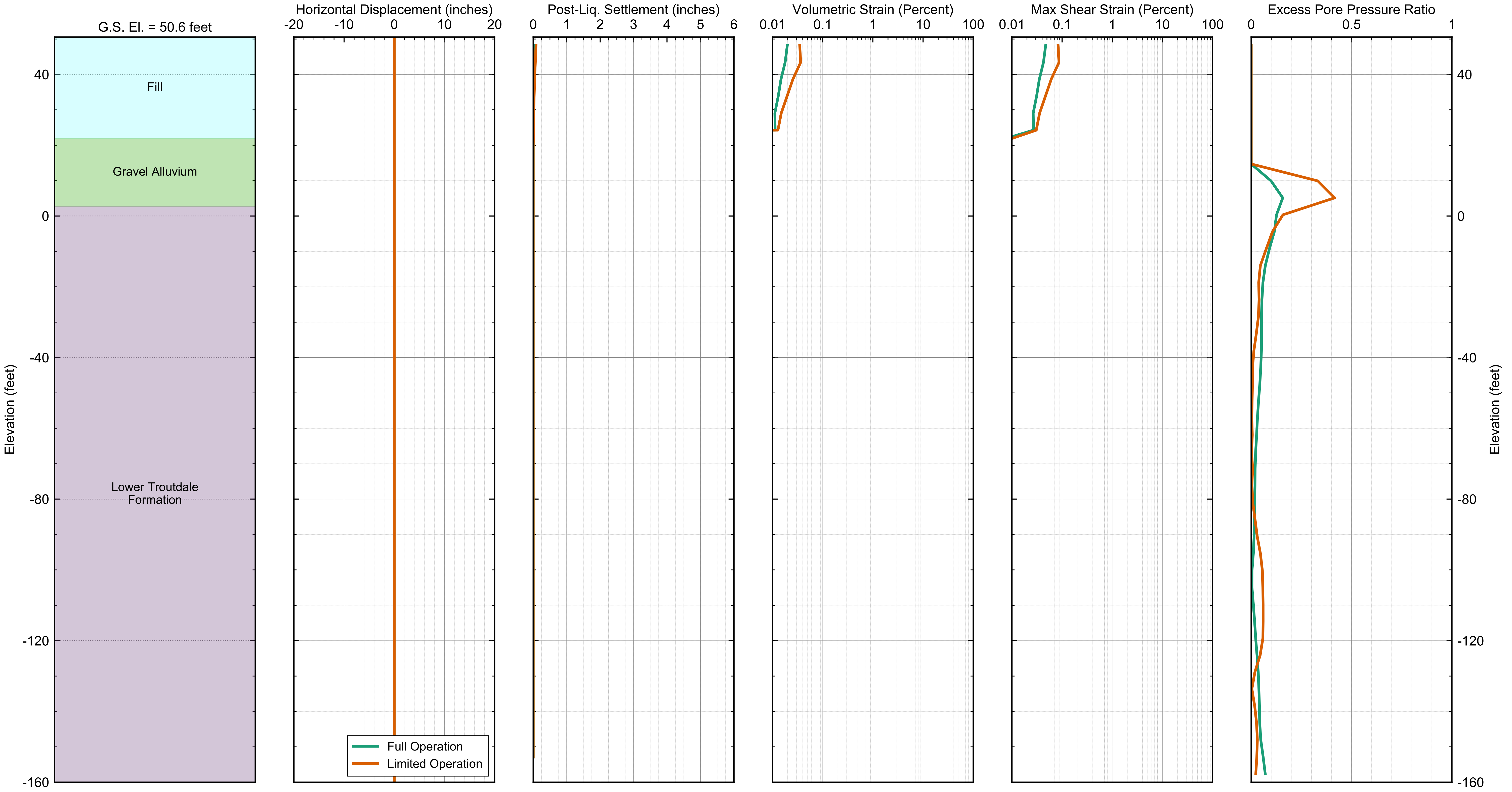
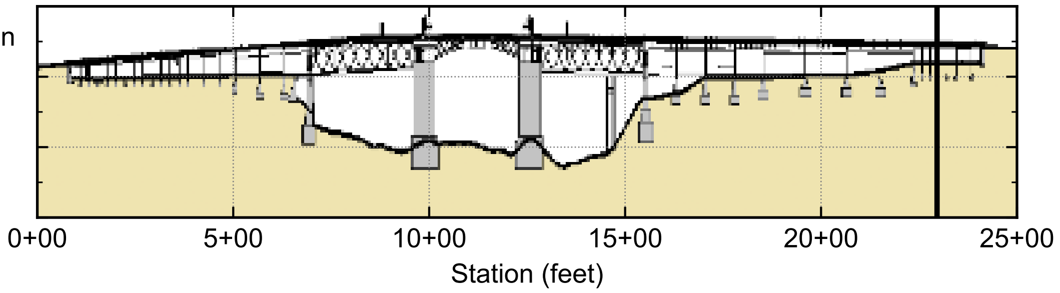


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 31 RETROFIT

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FIG. E-202

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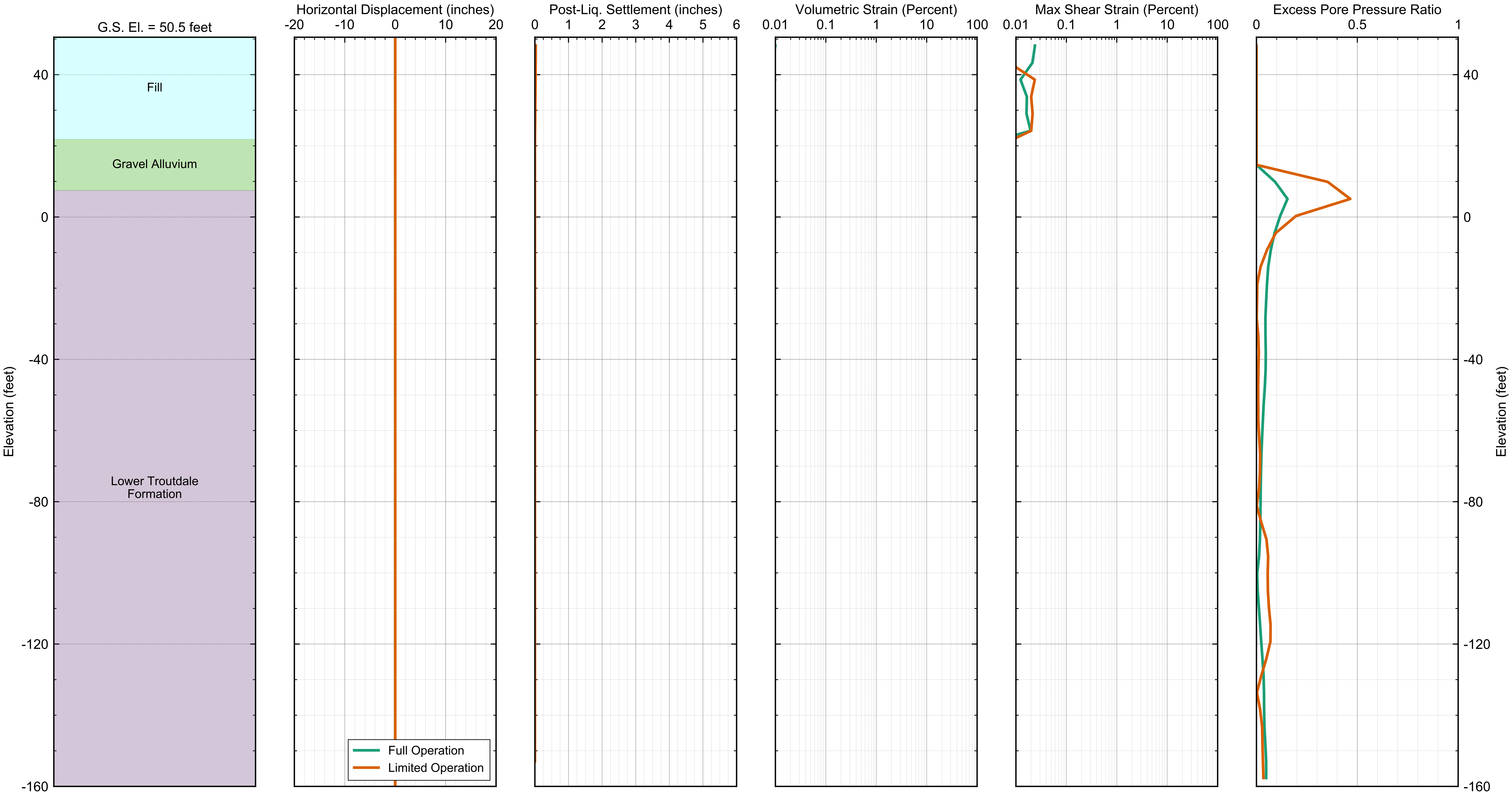
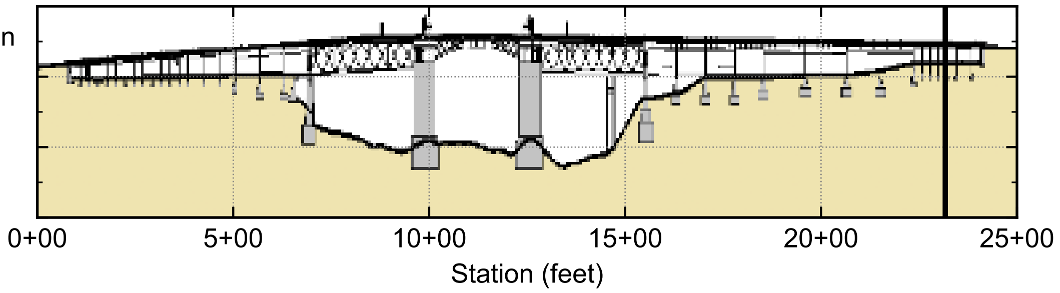
ver:0.8.5.post0.dev0+g414b689, by:pto



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 32 RETROFIT

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FIG. E-203

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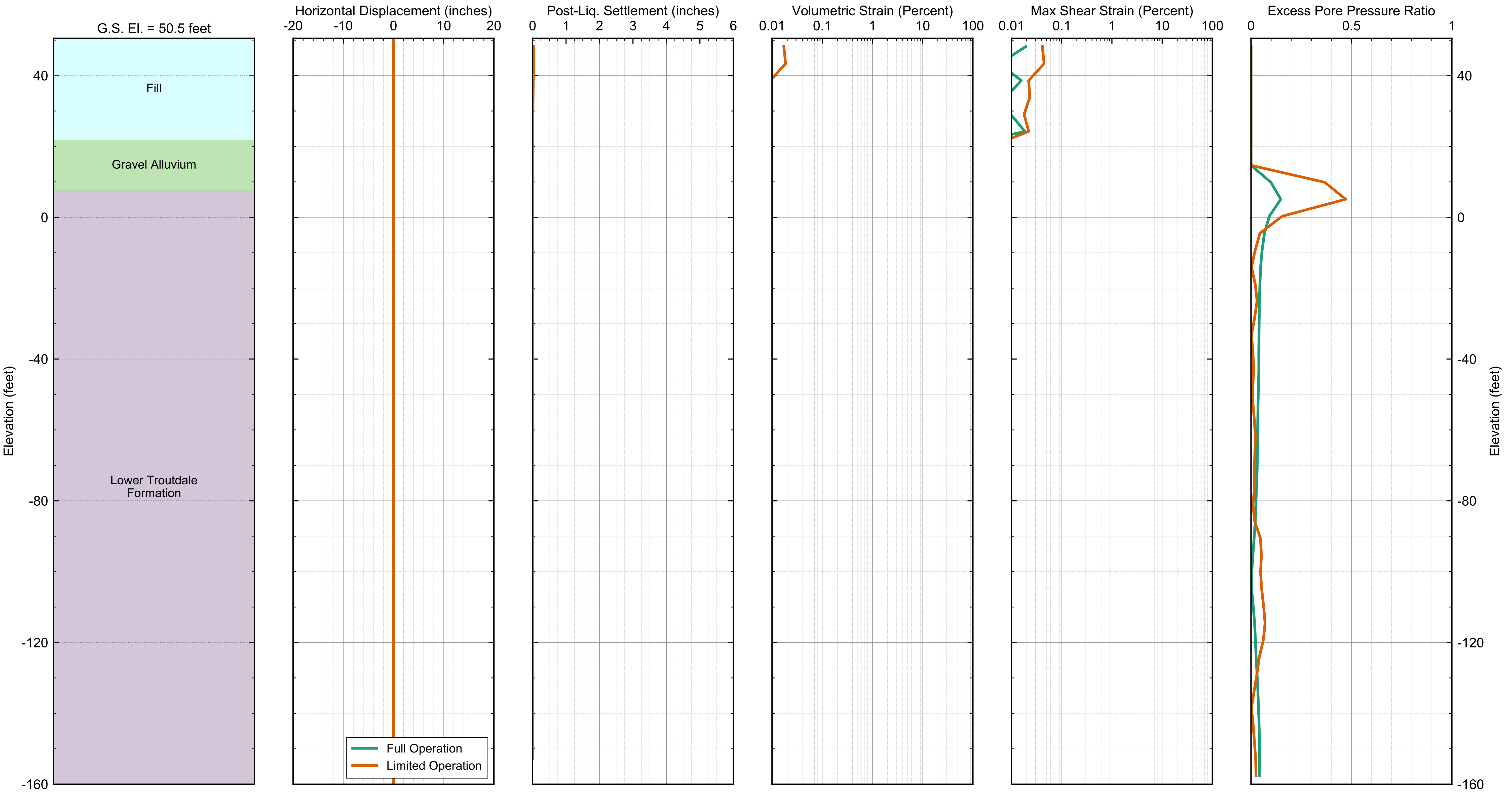
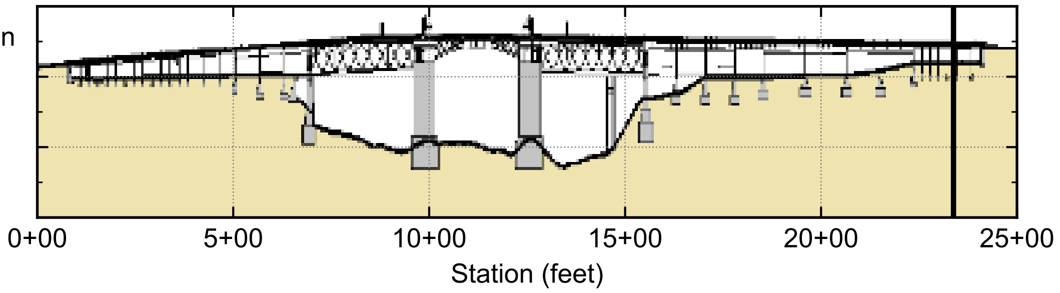


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 33 RETROFIT

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FIG. E-204

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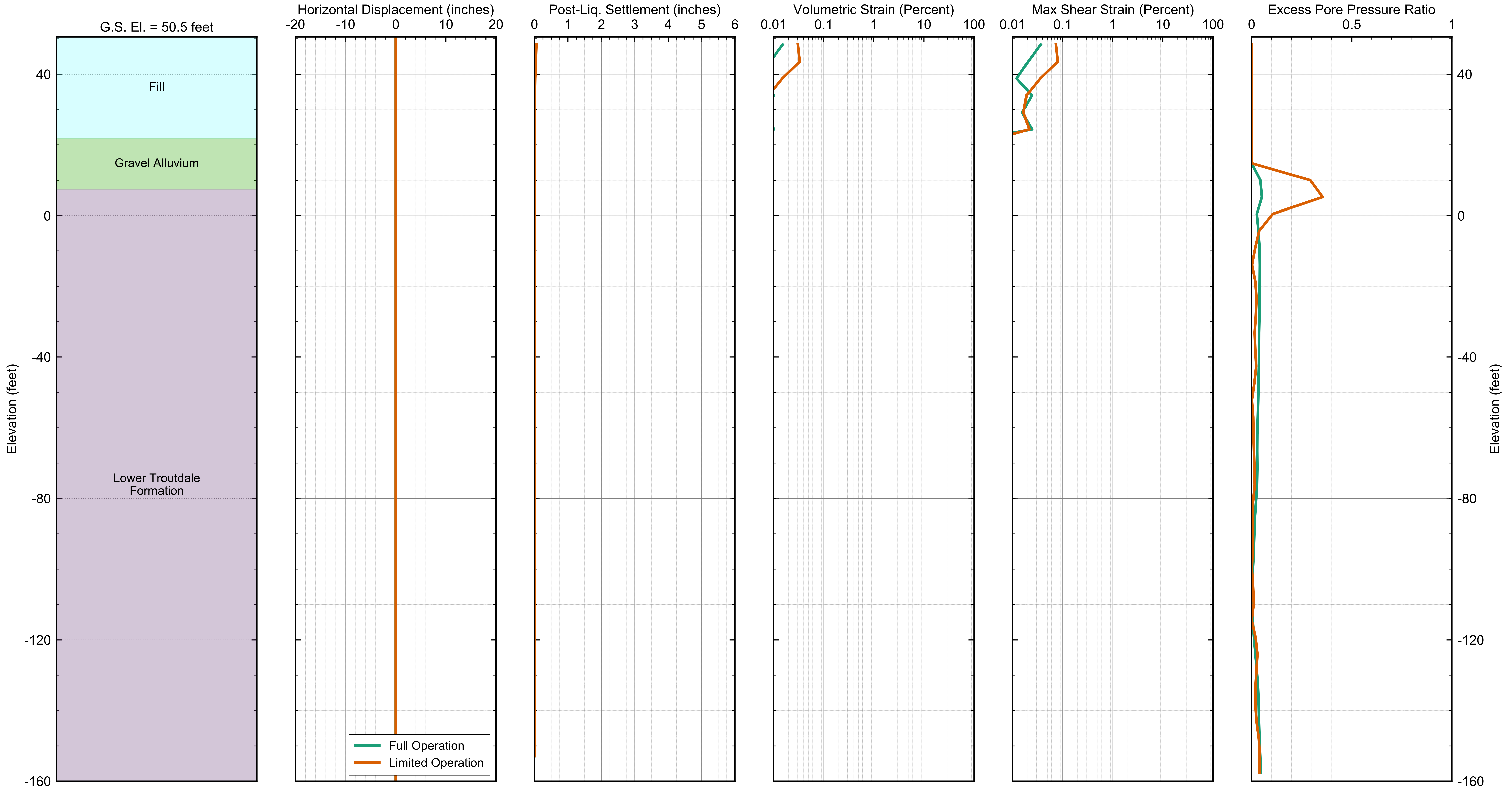
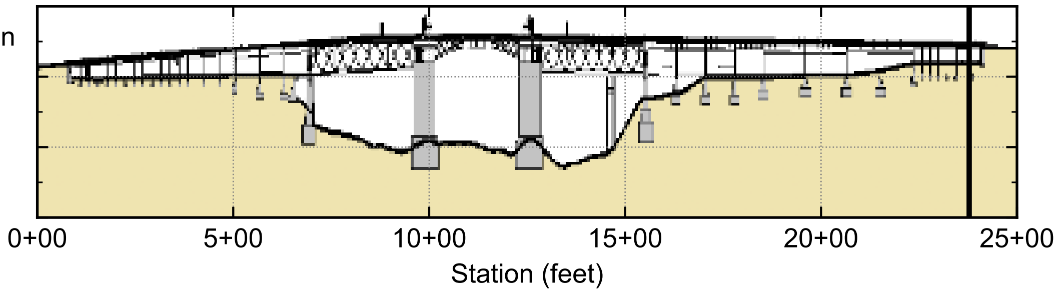


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
BENT 34 RETROFIT

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FIG. E-205

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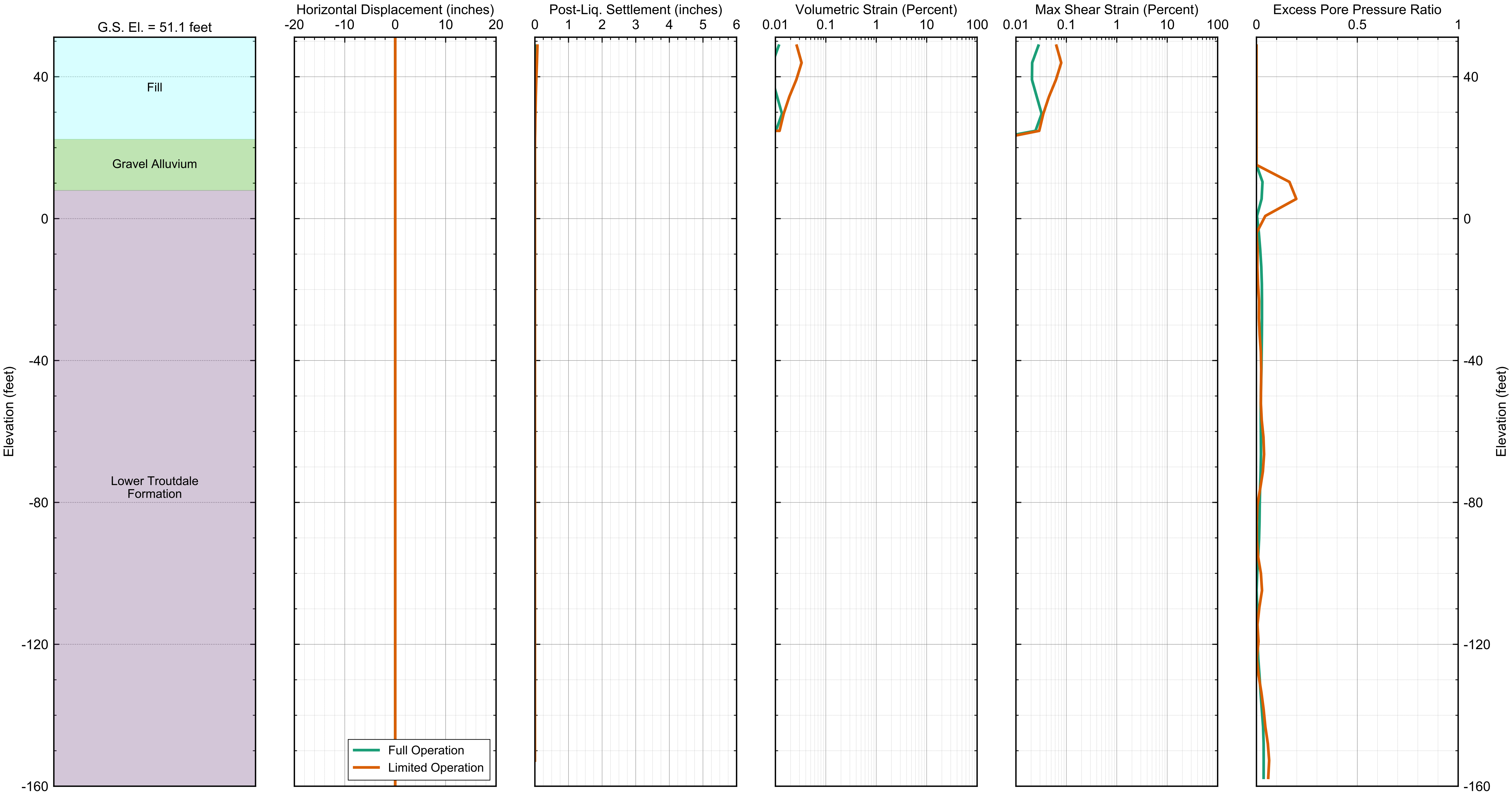
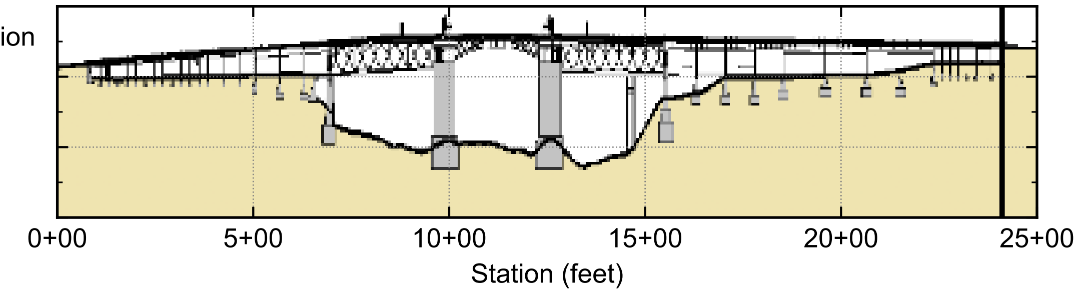


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT BENT 35 RETROFIT

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FIG. E-206

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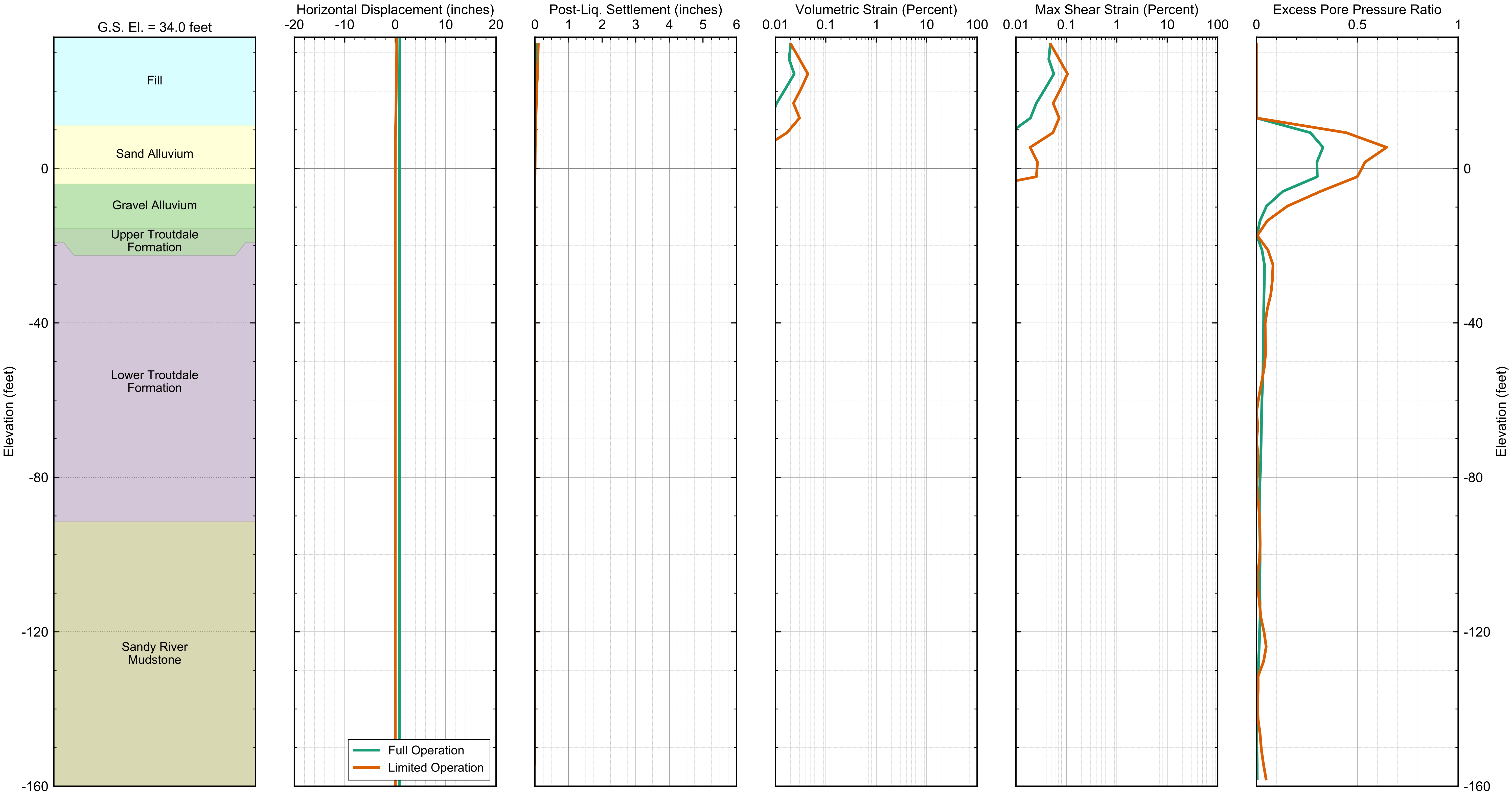
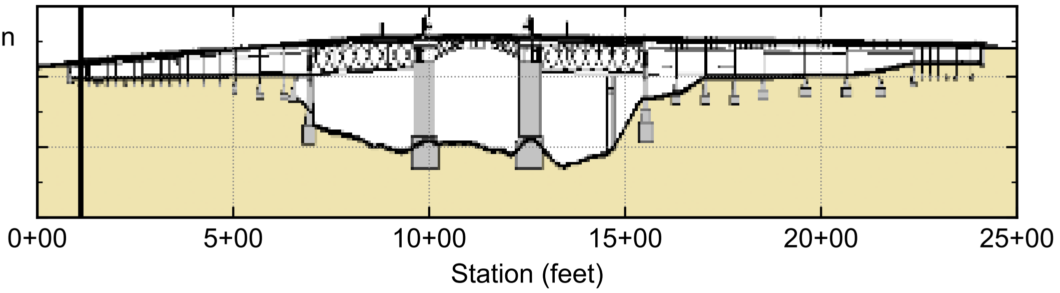


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 1

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FIG. E-207

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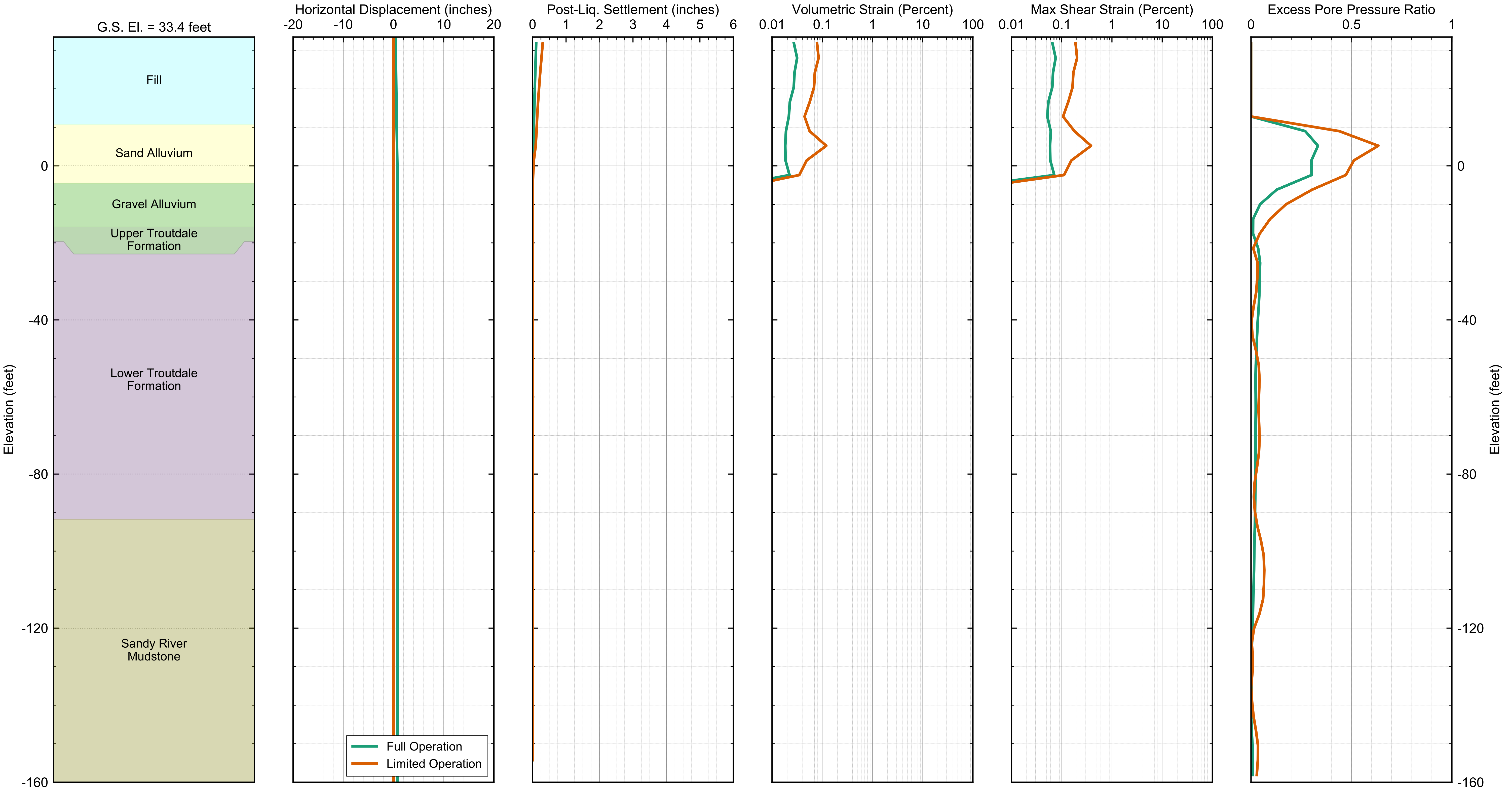
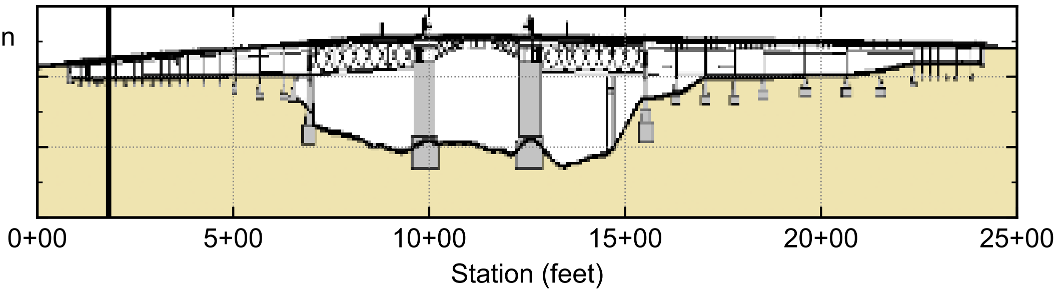


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 2

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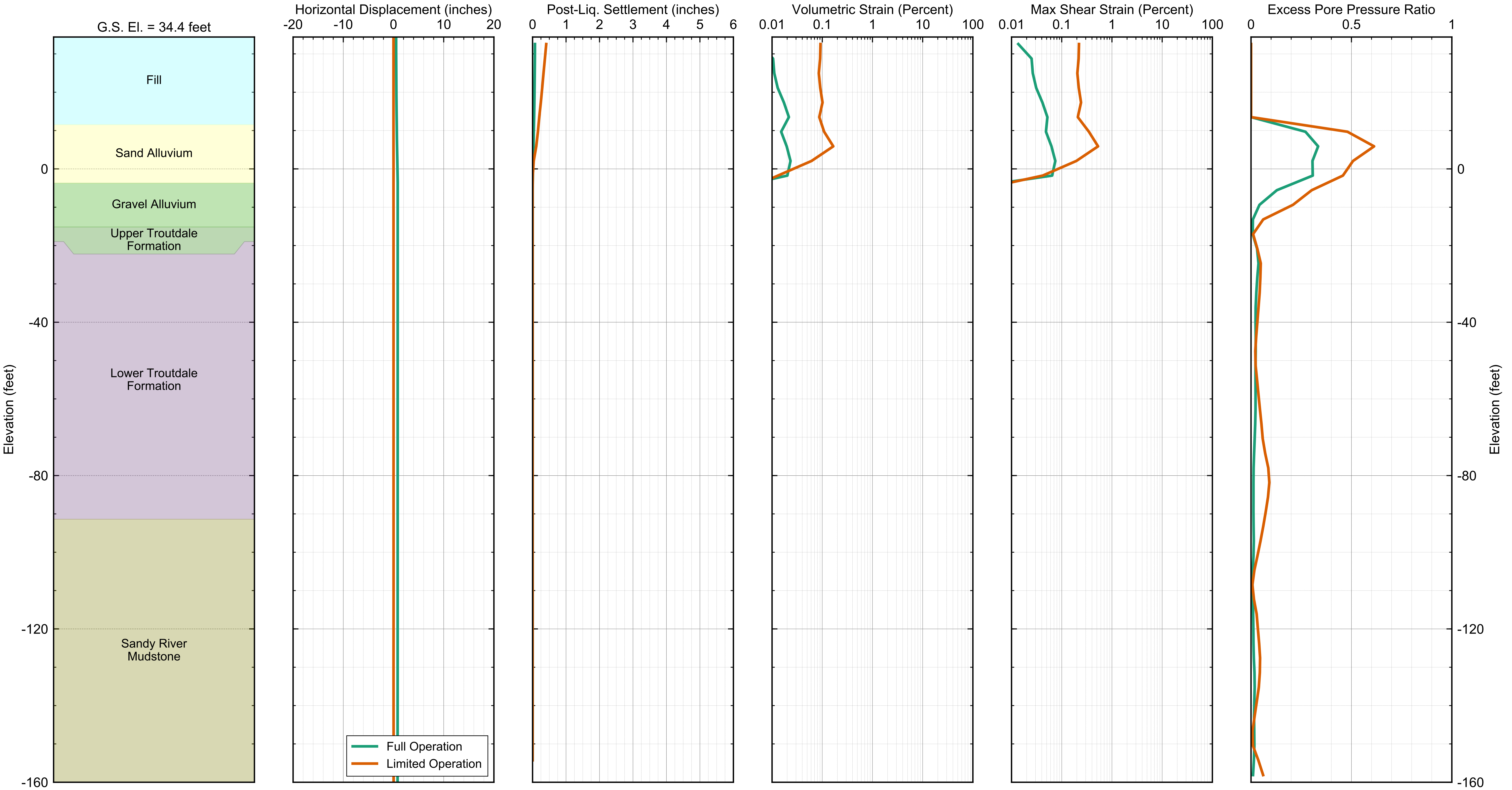
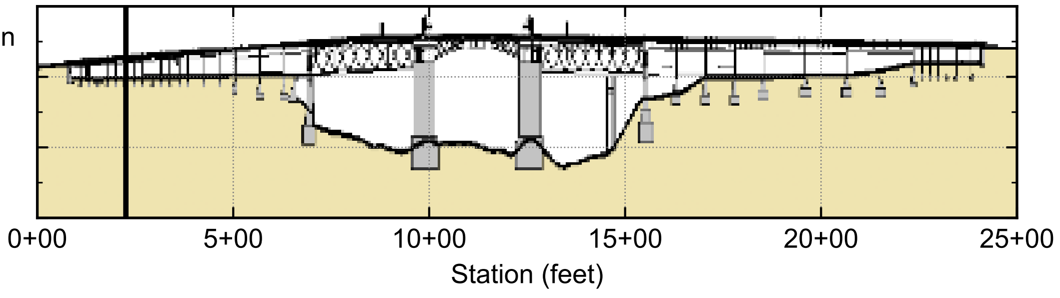
FIG. E-208

ver:0.8.5.post0.dev0+g414b689, by:PTO



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 3

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FIG. E-209

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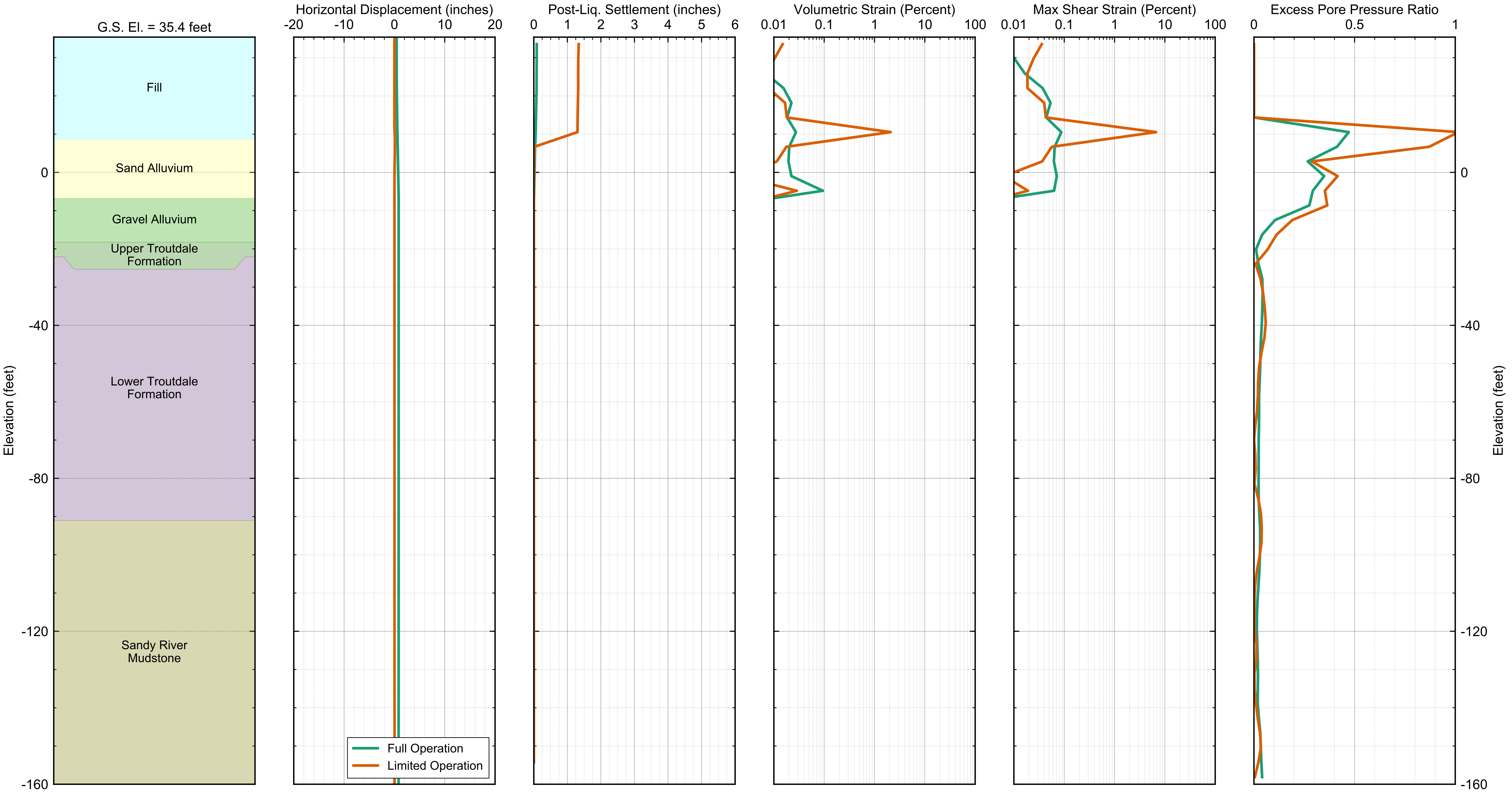
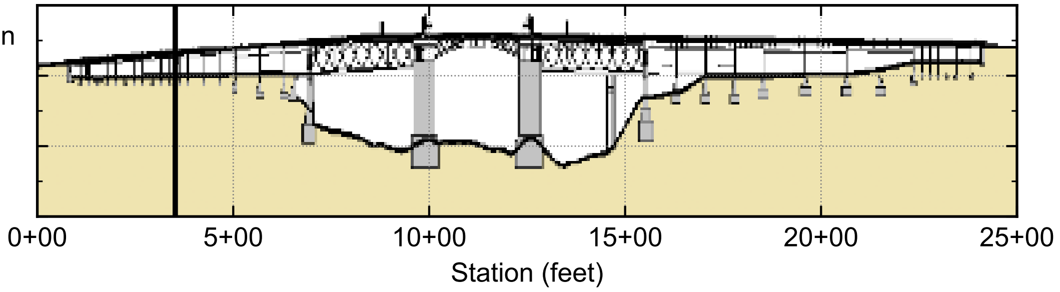


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
SHORT-SPAN BENT 4

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FIG. E-210

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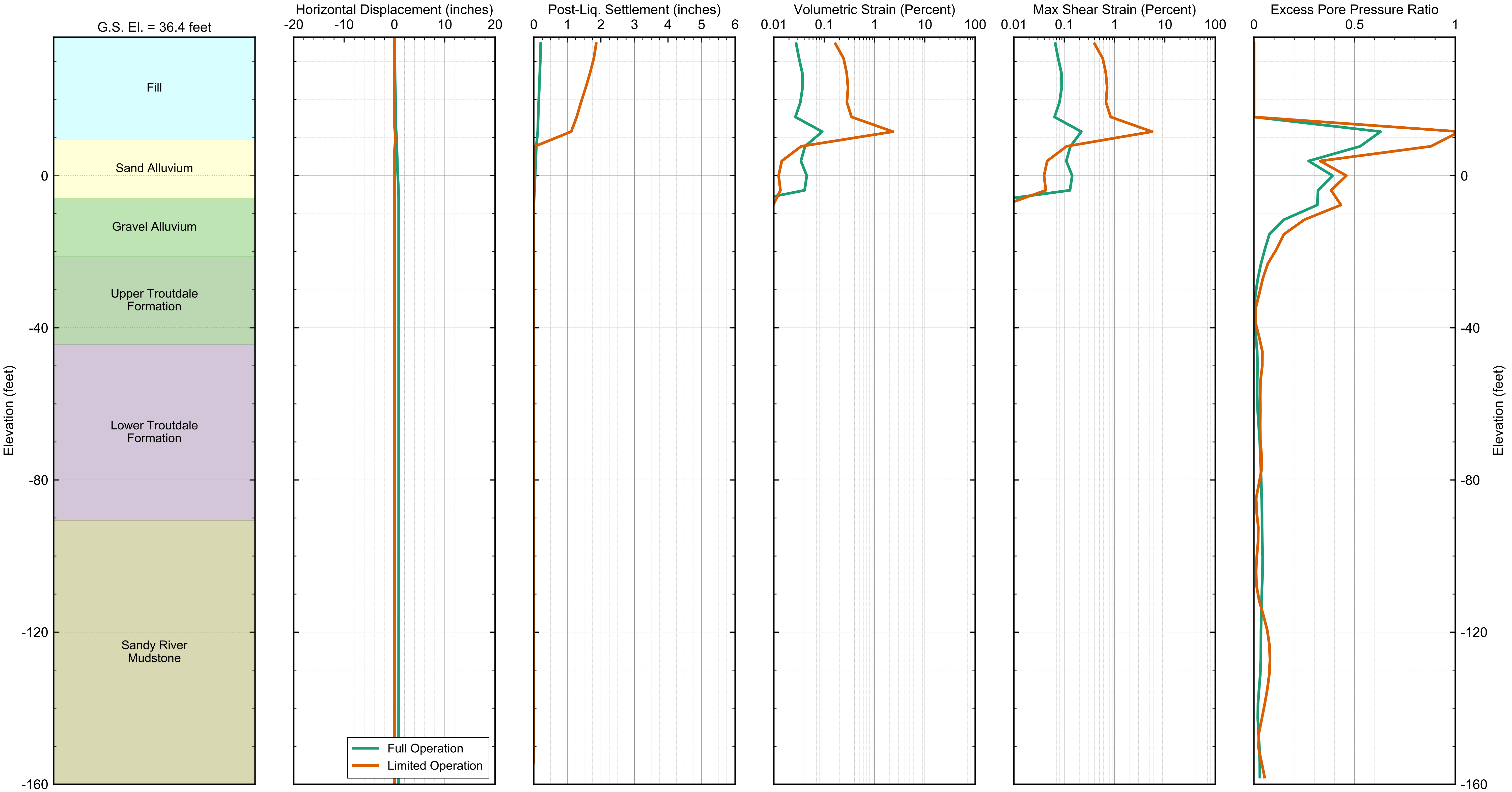
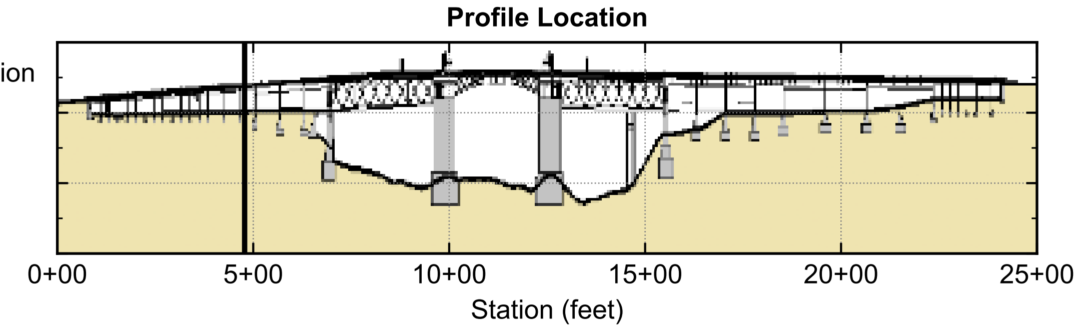


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 5

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FIG. E-211

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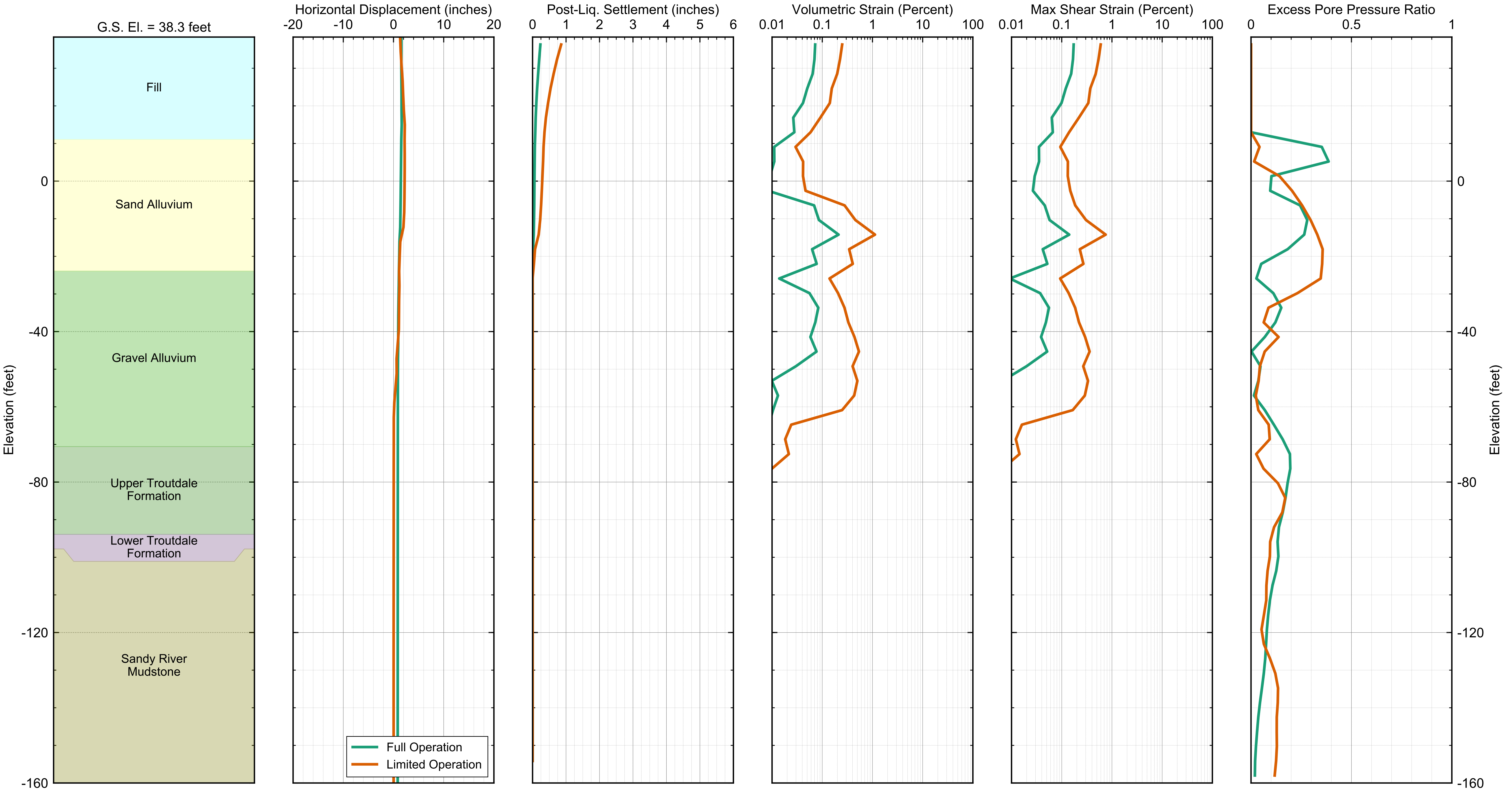
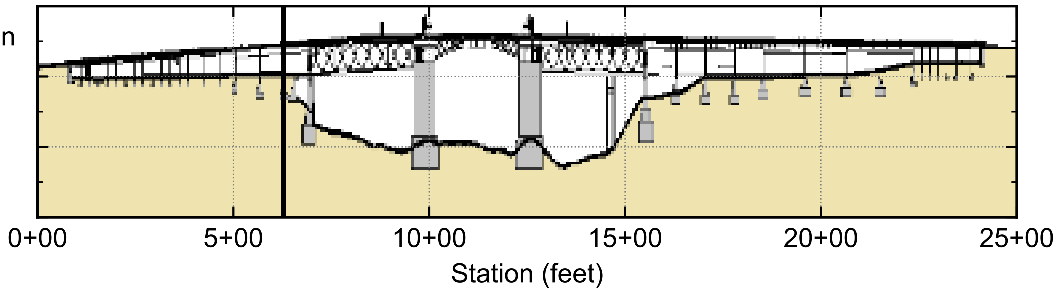


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 6

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FIG. E-212

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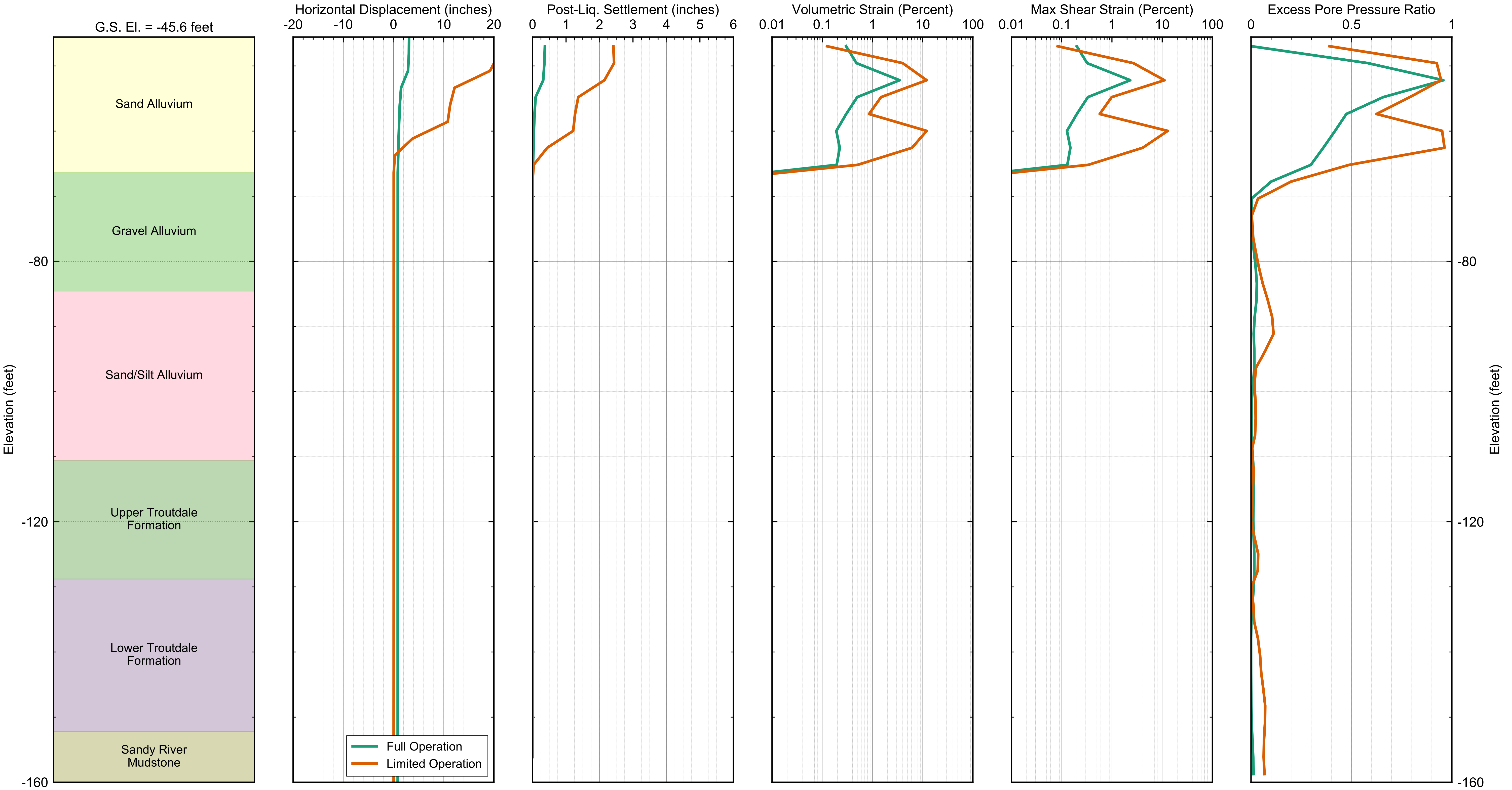
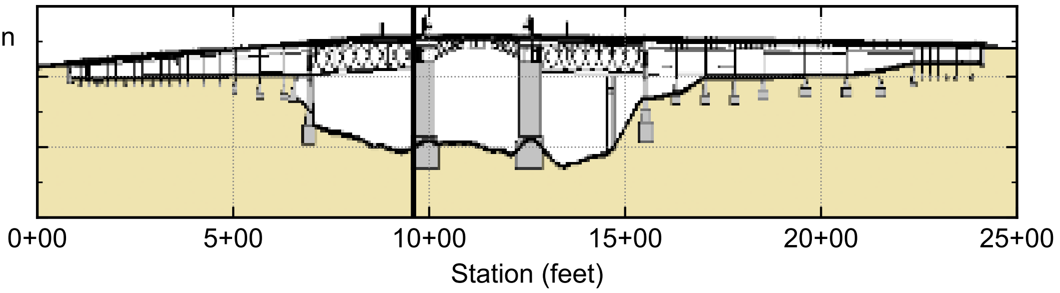


Notes:

1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
SHORT-SPAN BENT 7

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FIG. E-213

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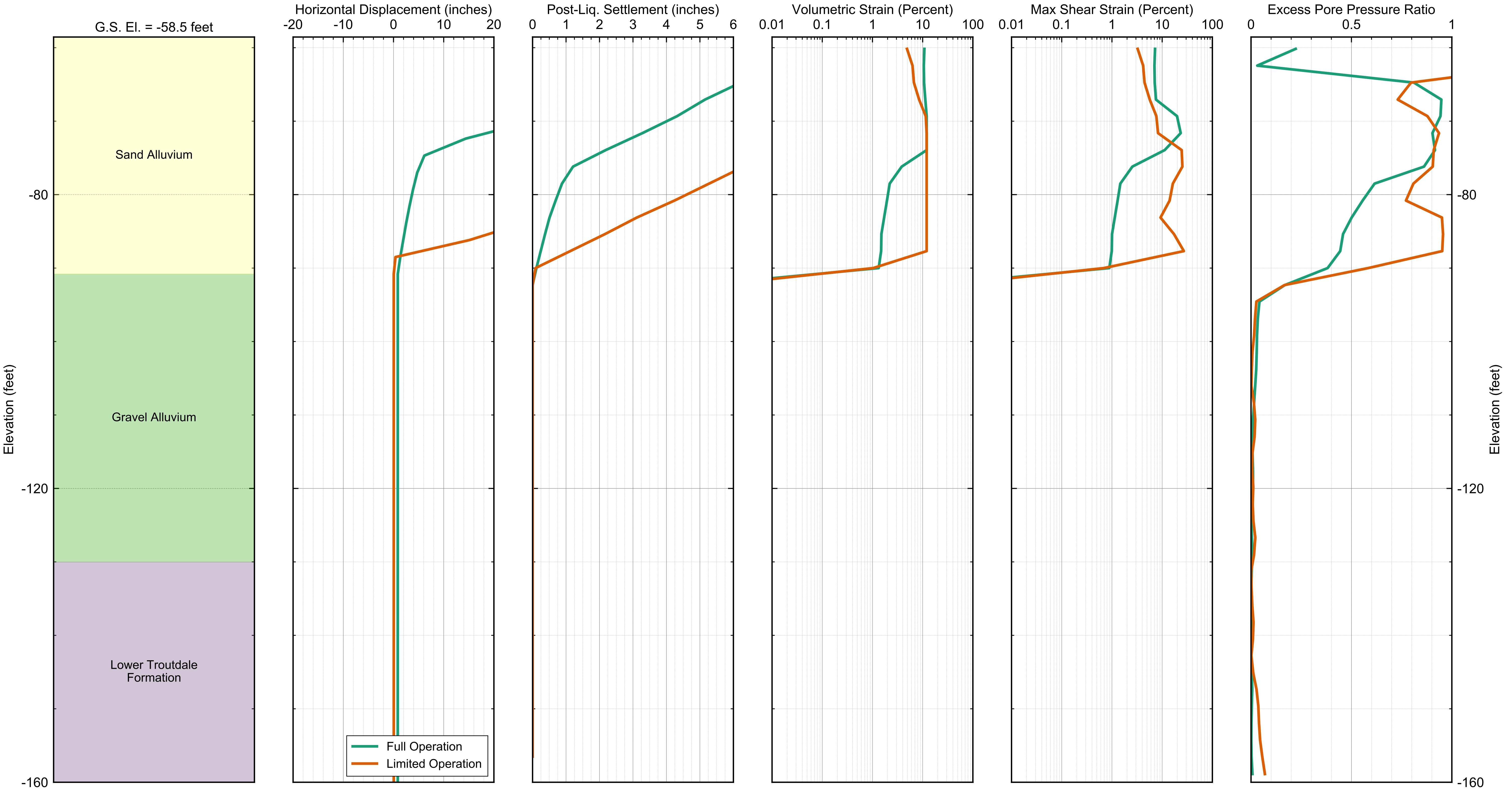
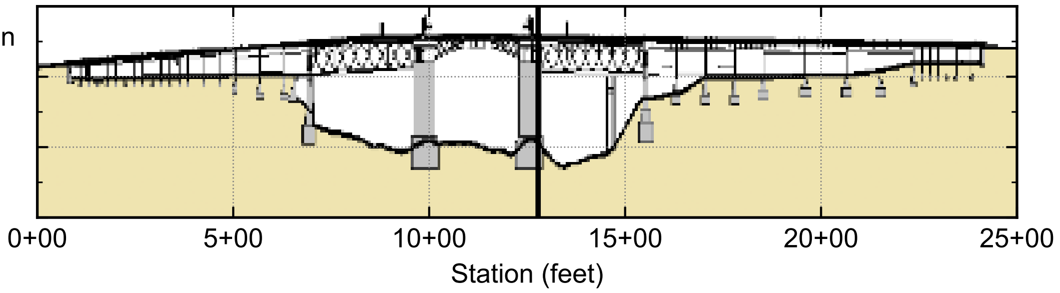


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 8

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102636-001

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FIG. E-214

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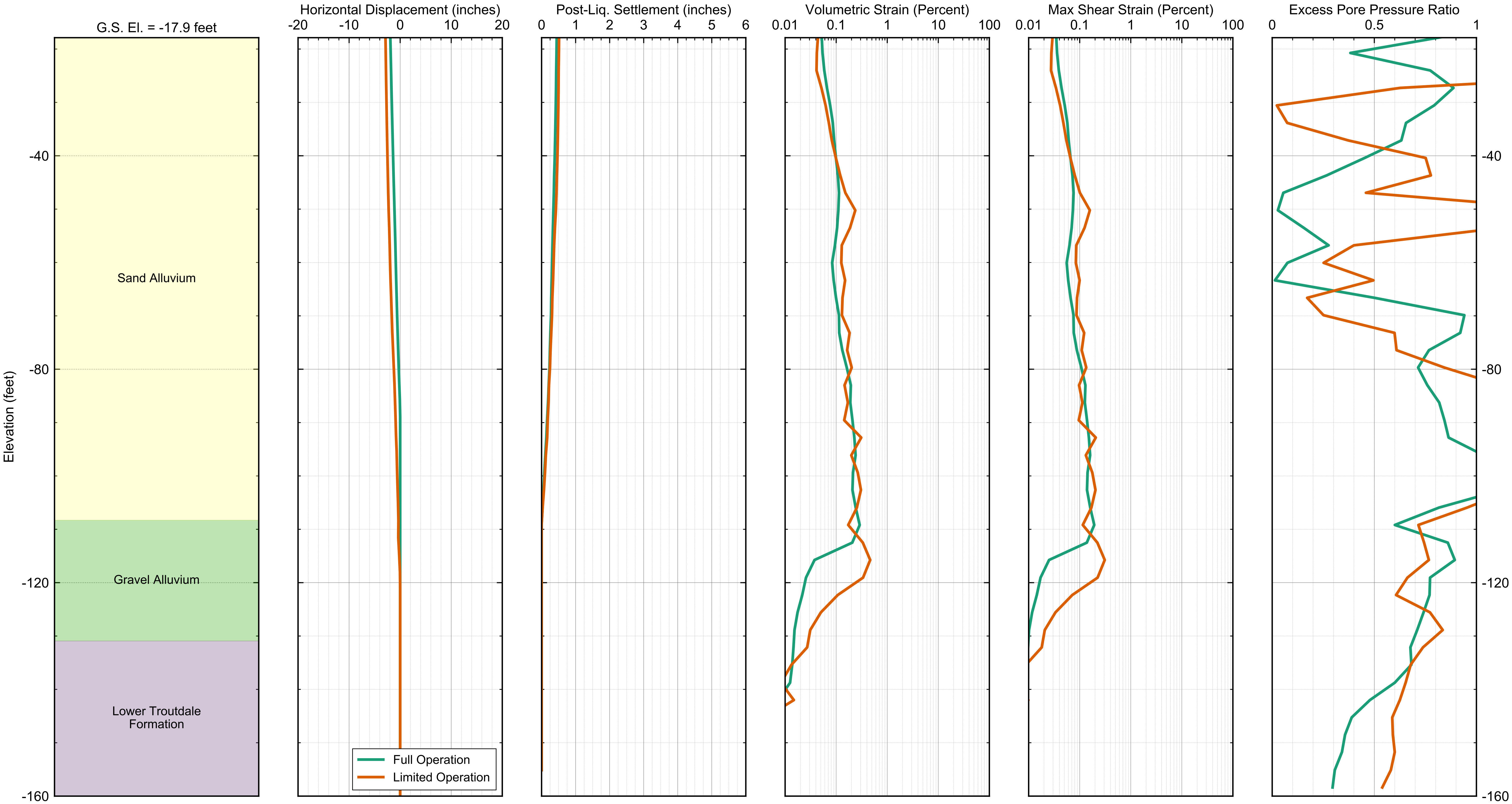
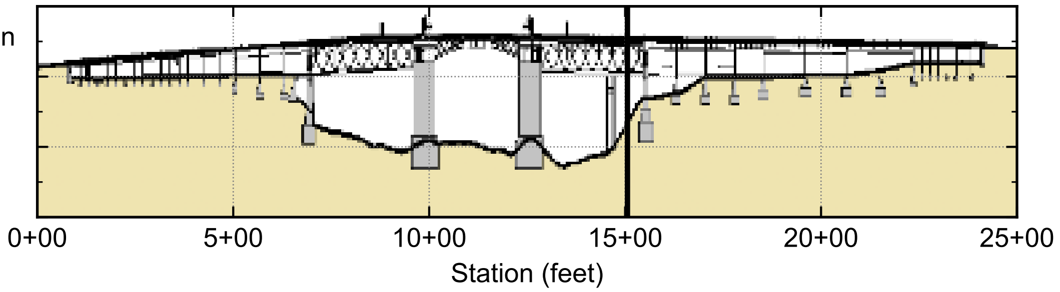


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

**SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
SHORT-SPAN BENT 9**

May 2020 102636-001

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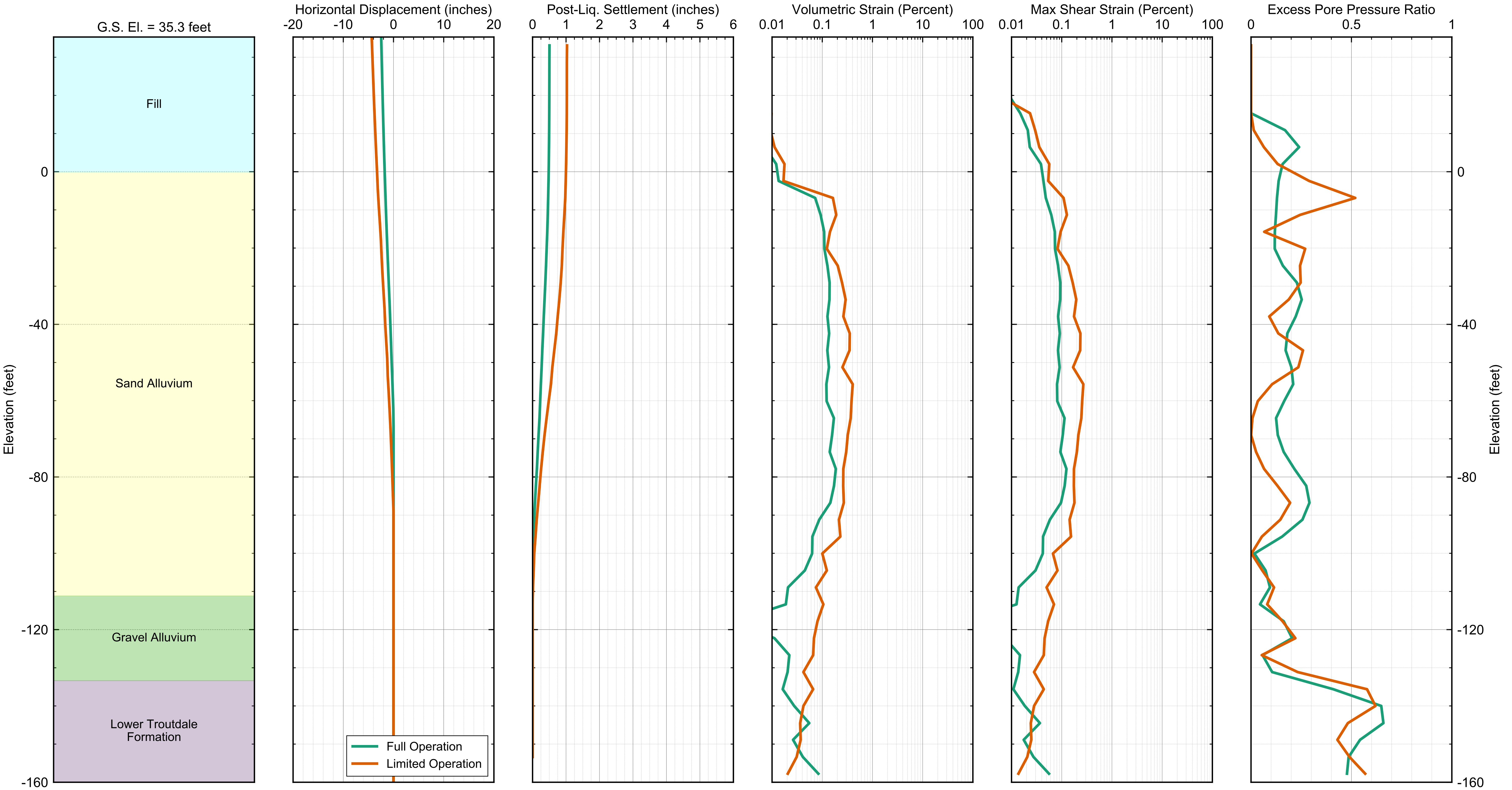
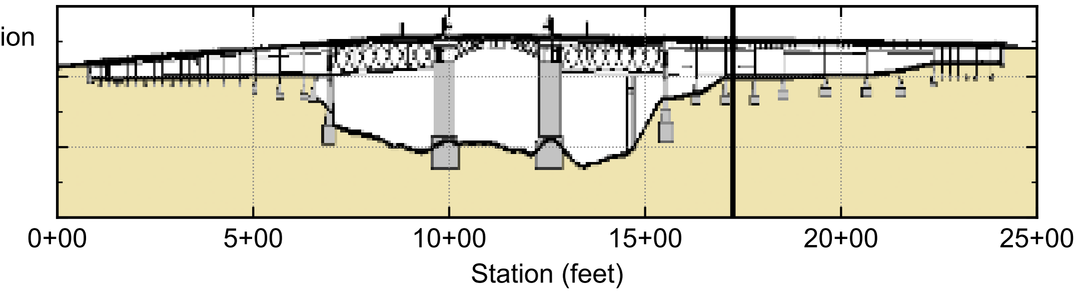
FIG. E-215

ver:0.8.5.post0.dev0+g414b689, by:PTO



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
SHORT-SPAN BENT 10/S10

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FIG. E-216

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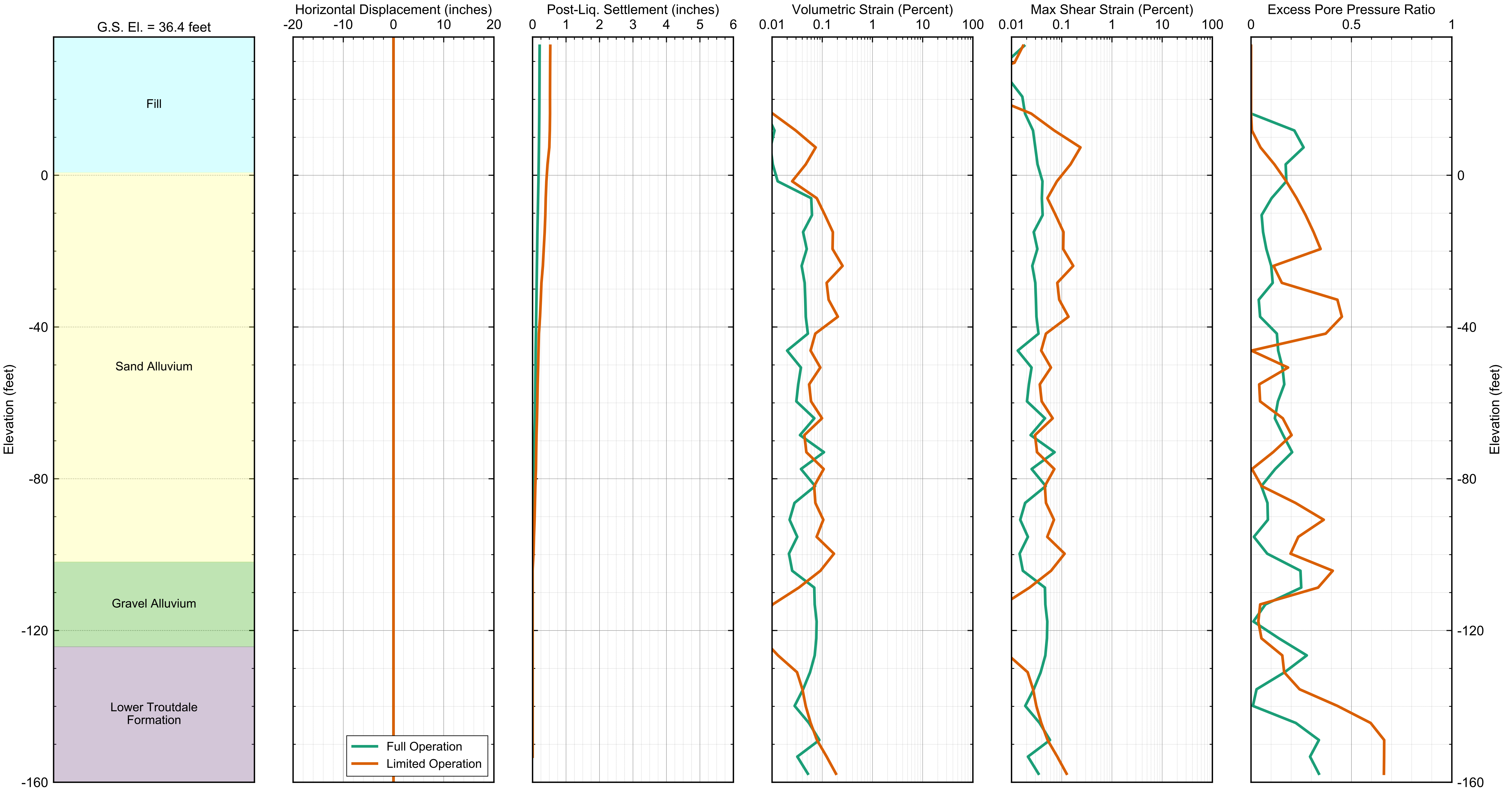
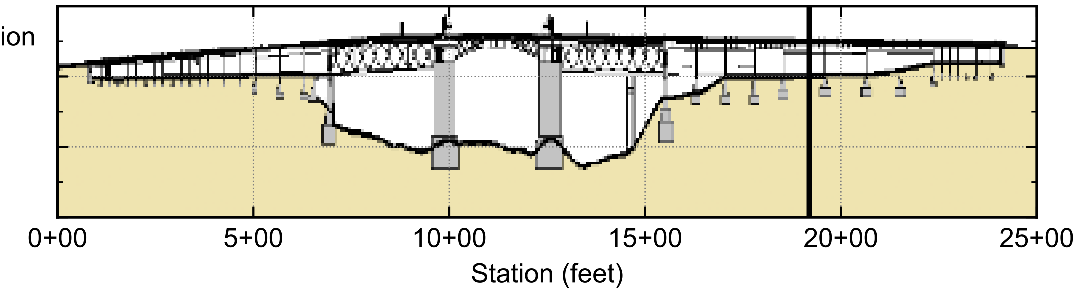


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 11/S11

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FIG. E-217

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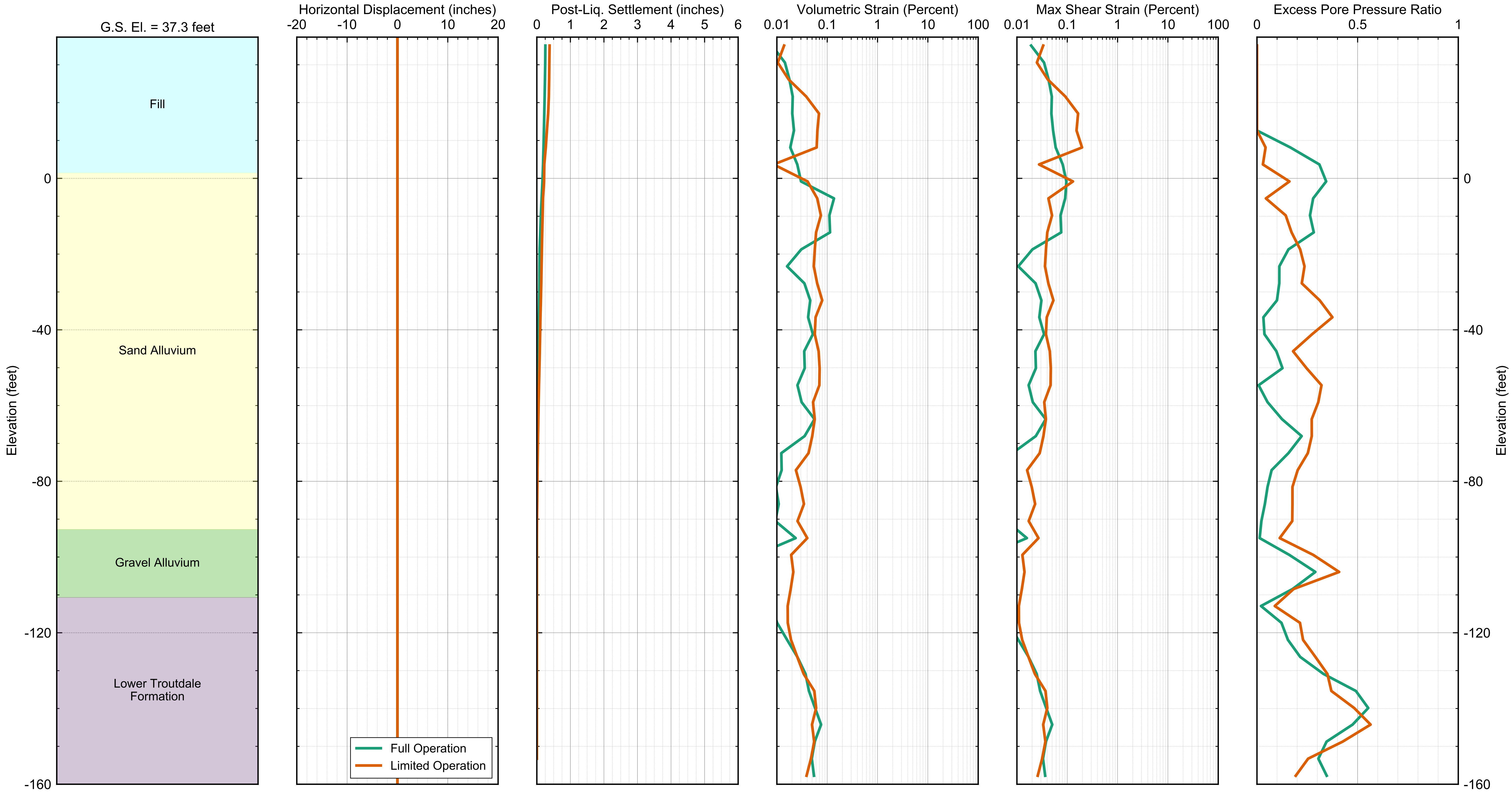
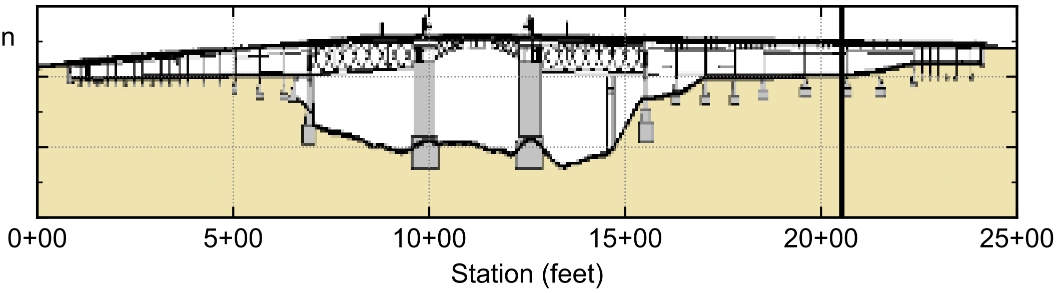


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
SHORT-SPAN BENT 12/S12

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FIG. E-218

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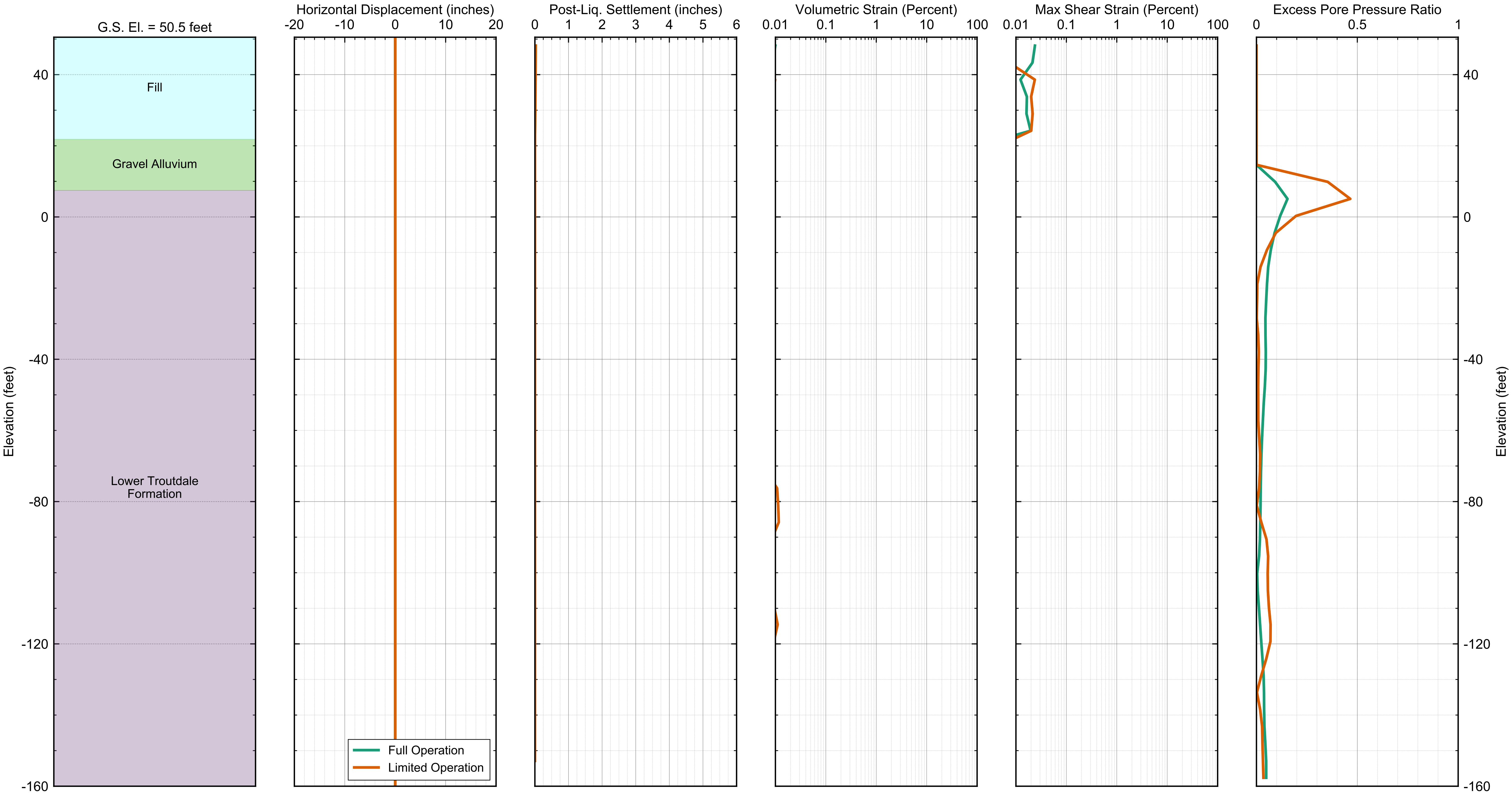
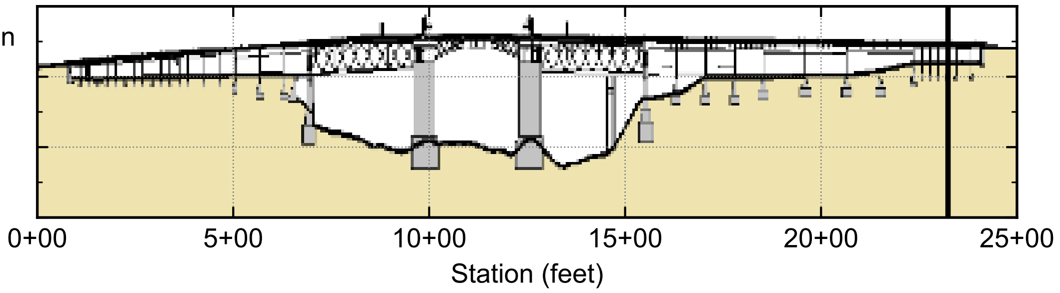


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT SHORT-SPAN BENT 13/S13

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102636-001

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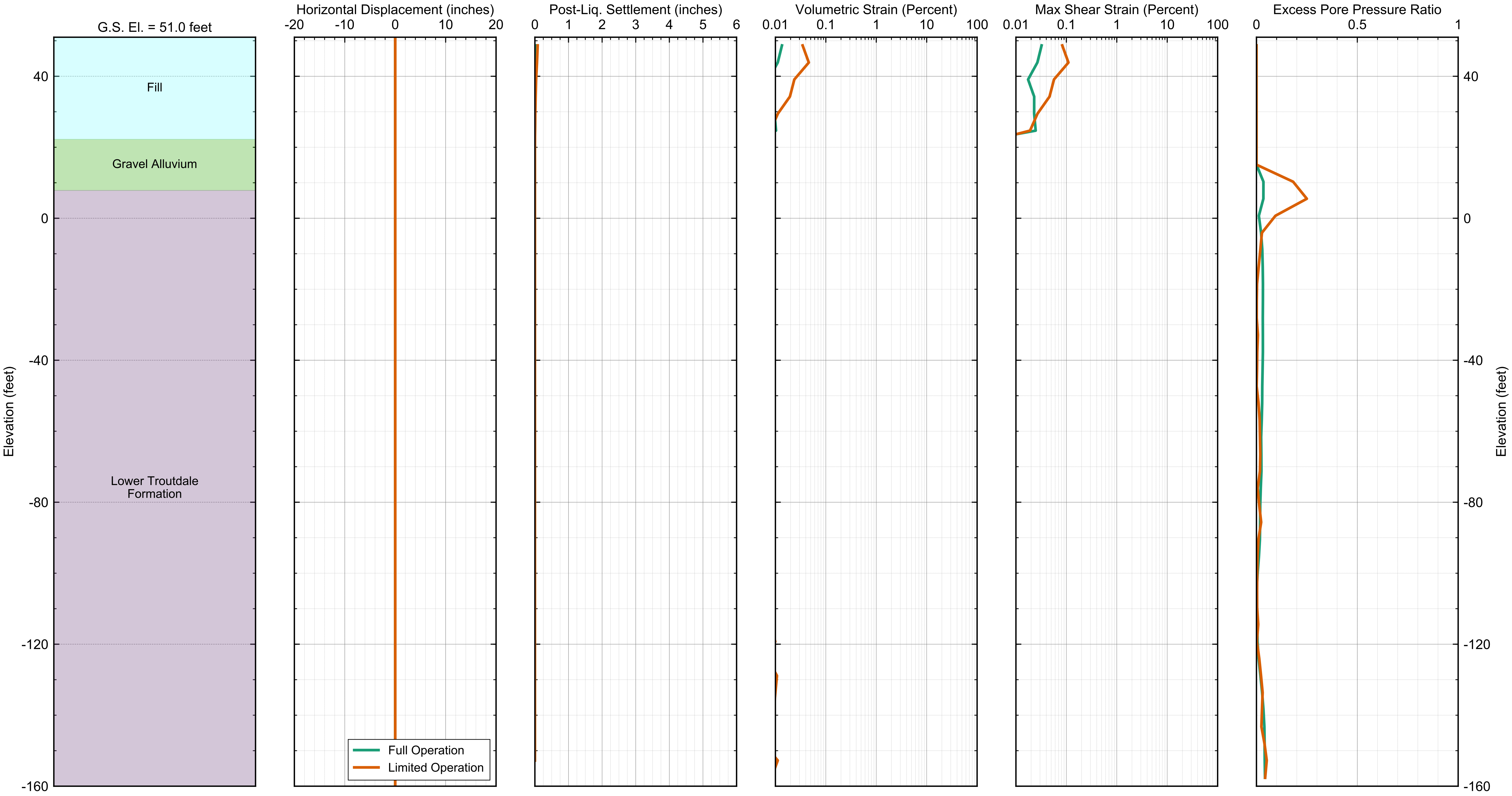
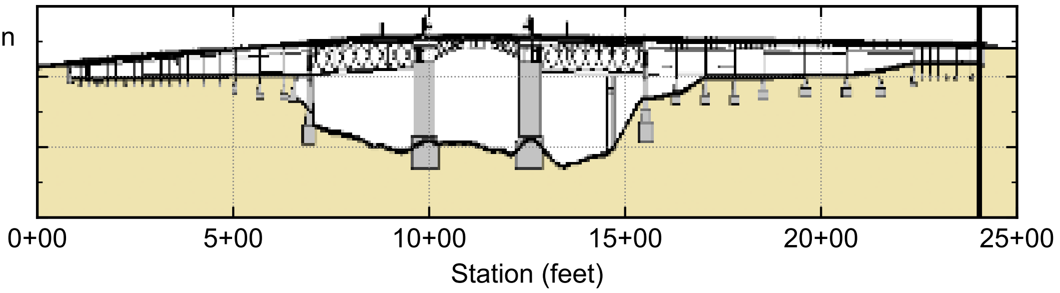
FIG. E-219



Notes:

1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
SHORT-SPAN BENT 14/S14

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FIG. E-220

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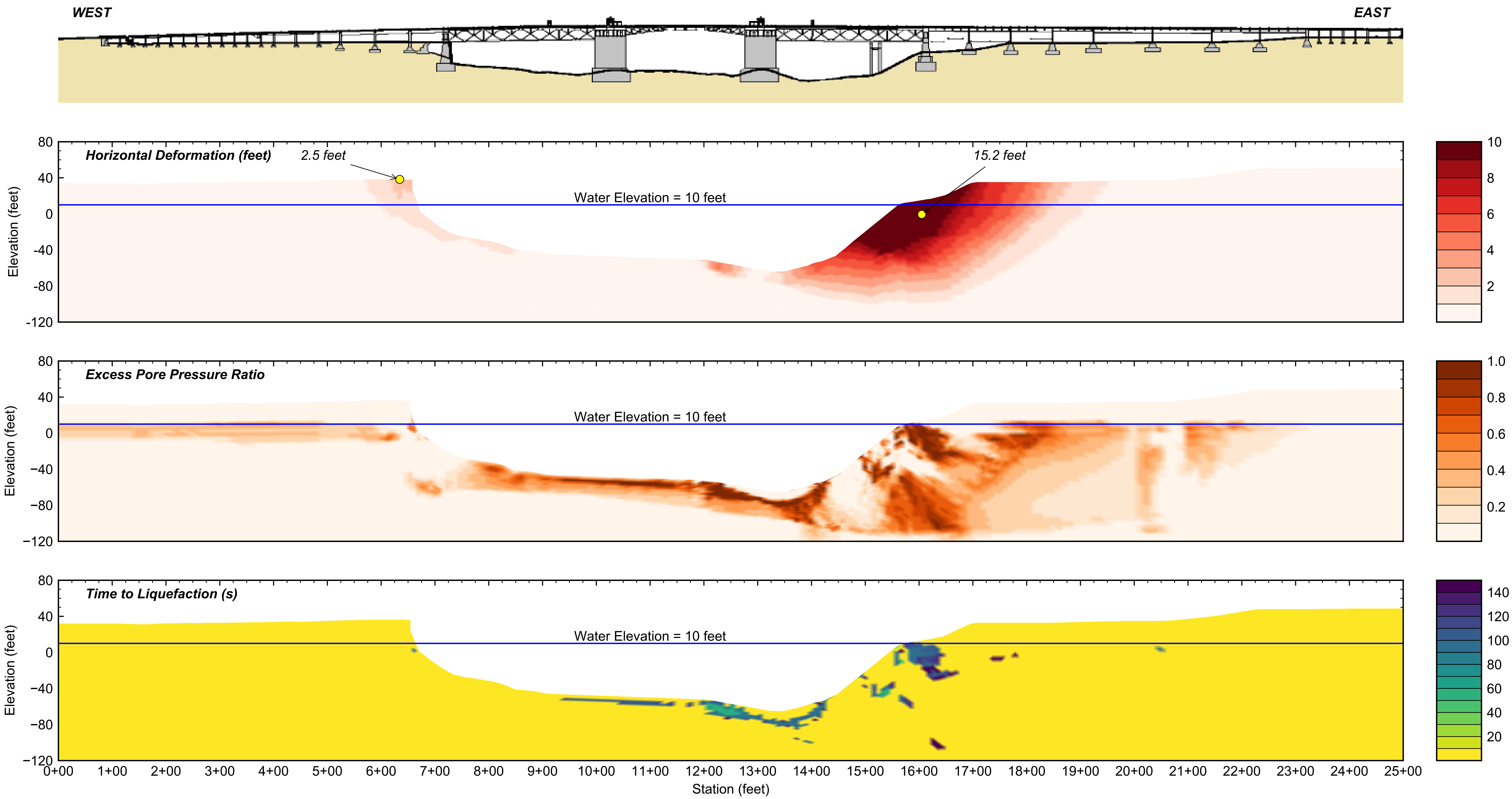

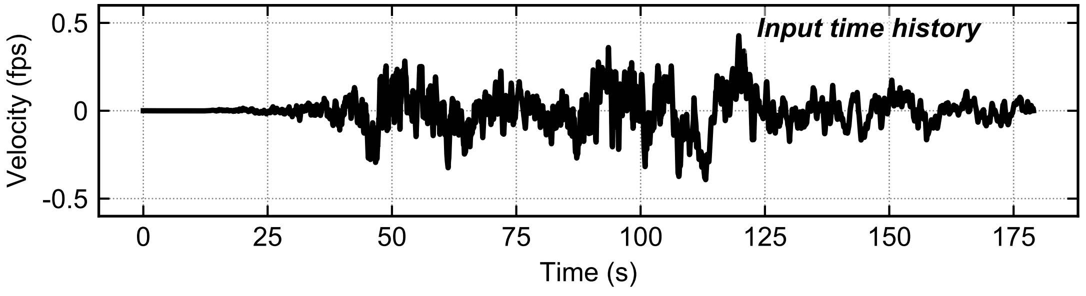

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Portland, Oregon

MODEL RESPONSE
LONG SPAN WITH GI
FULL OPERATION MOTION
TOHOKU (AKT023EW)

May 2020
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FIG. E-221

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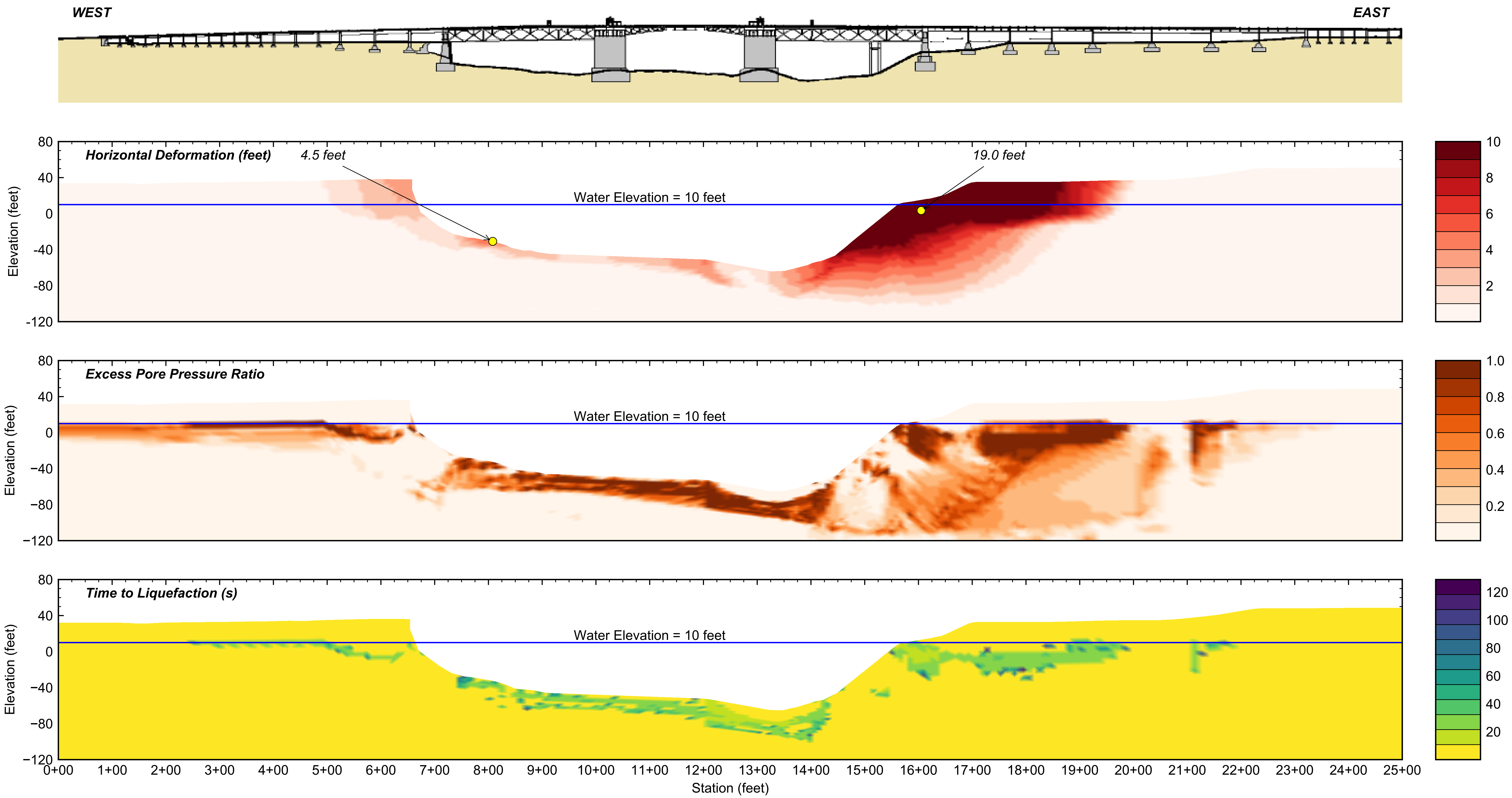

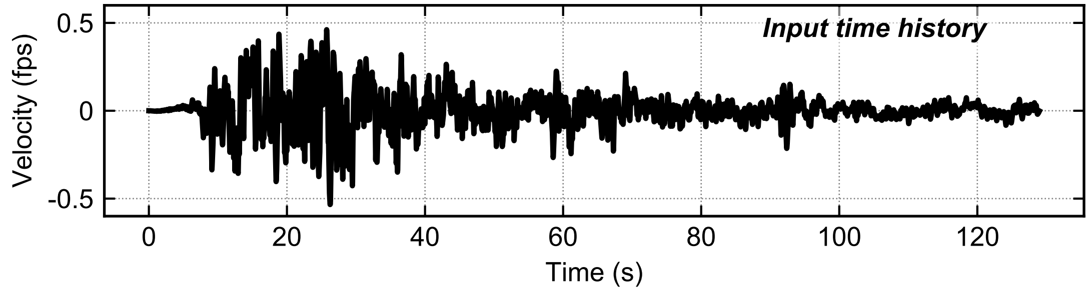

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Portland, Oregon

MODEL RESPONSE
LONG SPAN WITH GI
LIMITED OPERATION MOTION
MAULE (CCSP97)

May 2020
102636-001

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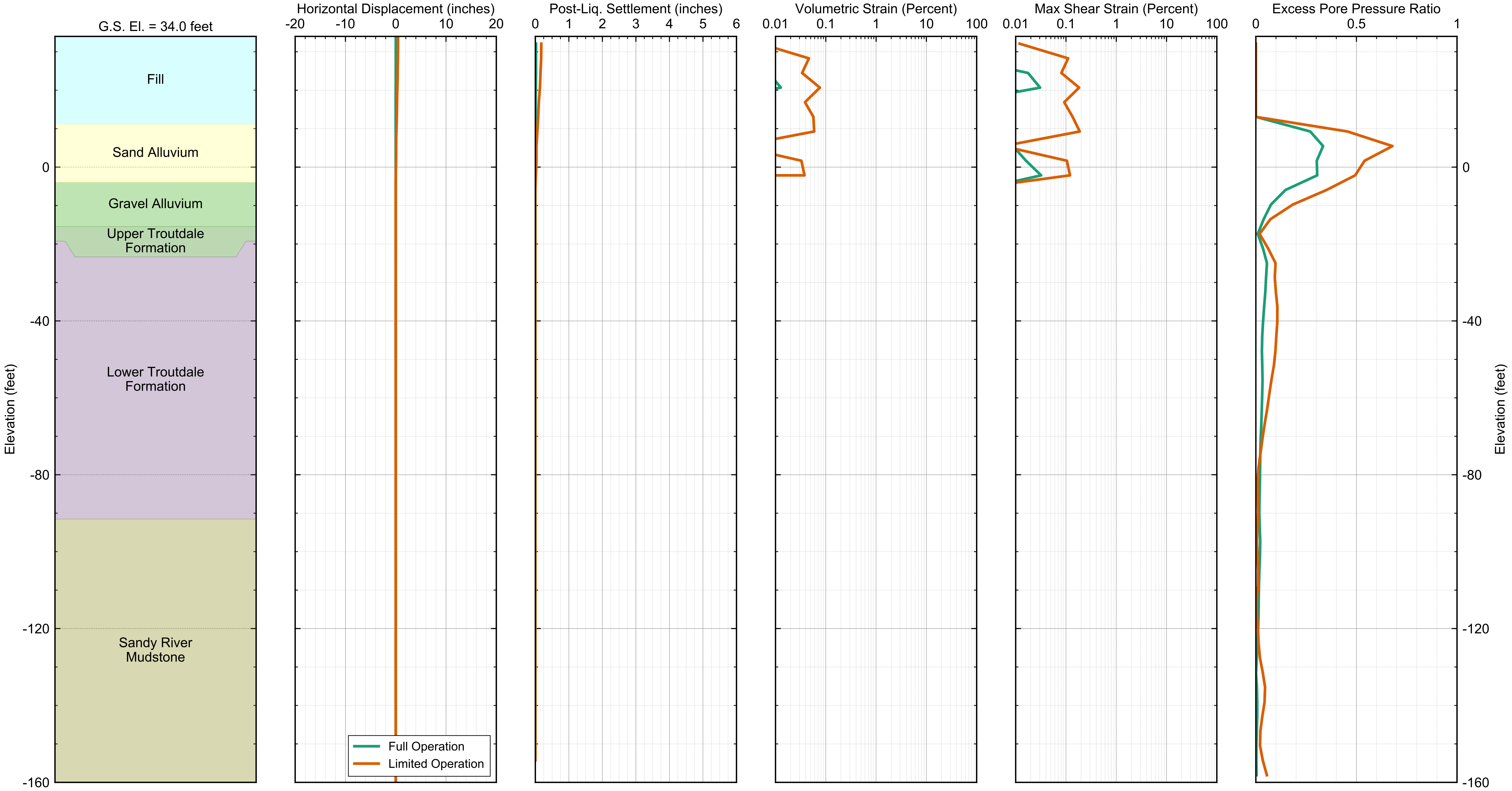
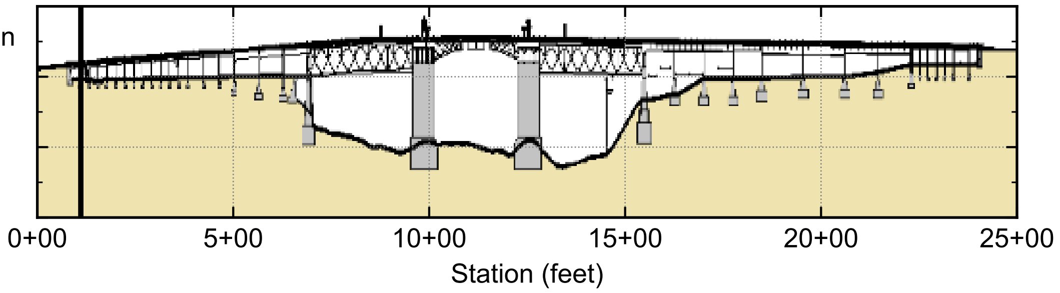
FIG. E-222



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN BENT 1

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102636-001

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FIG. E-223

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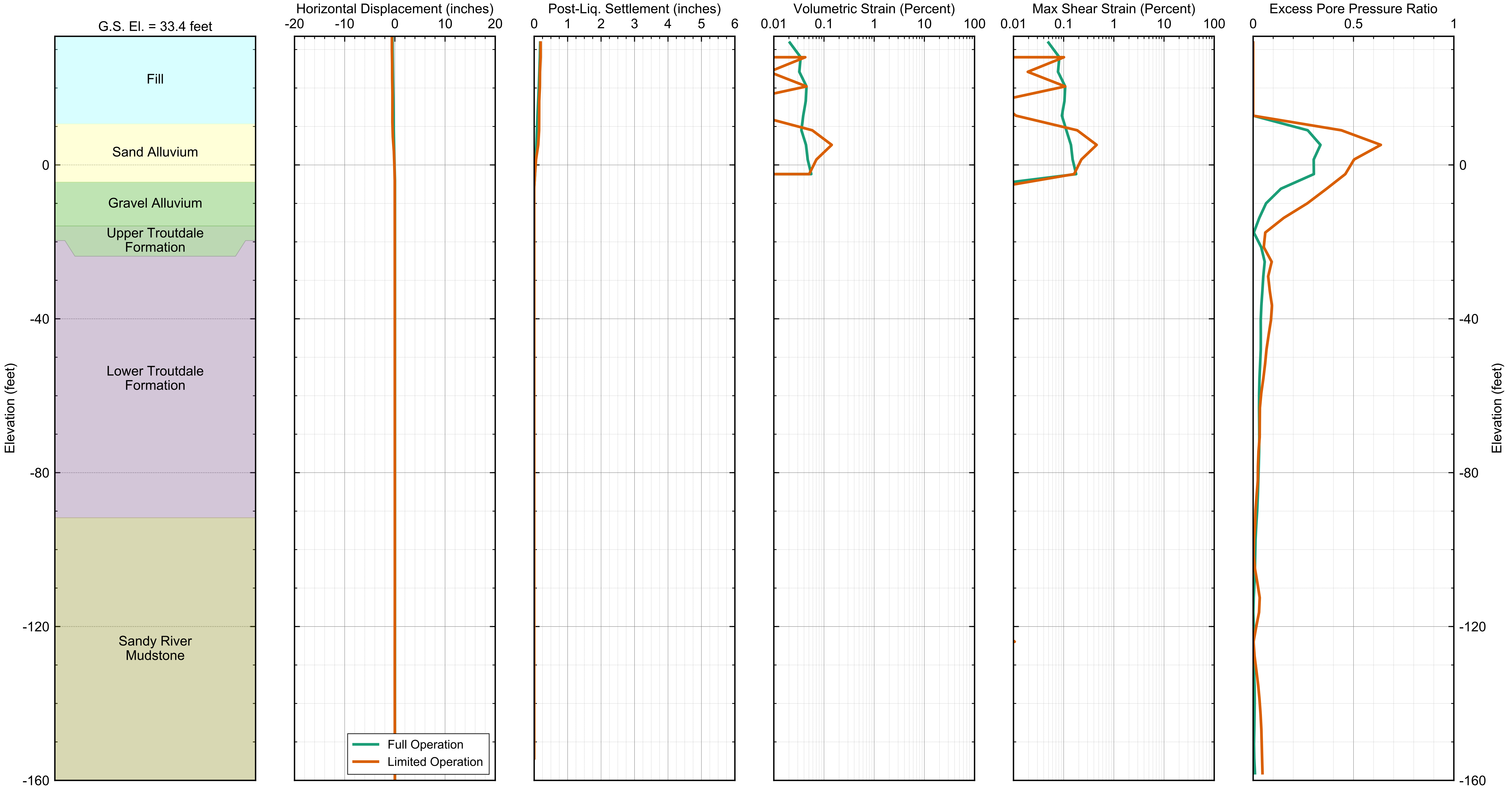
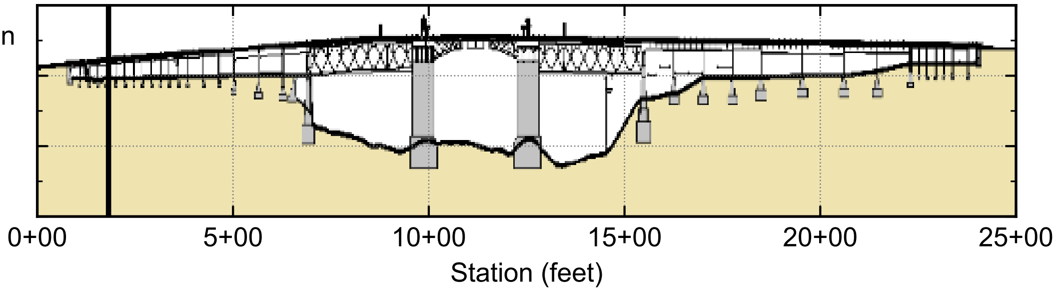
ver:0.7.17.post0.dev0+g54b3f9e, by:sss



Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN BENT 2

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FIG. E-224

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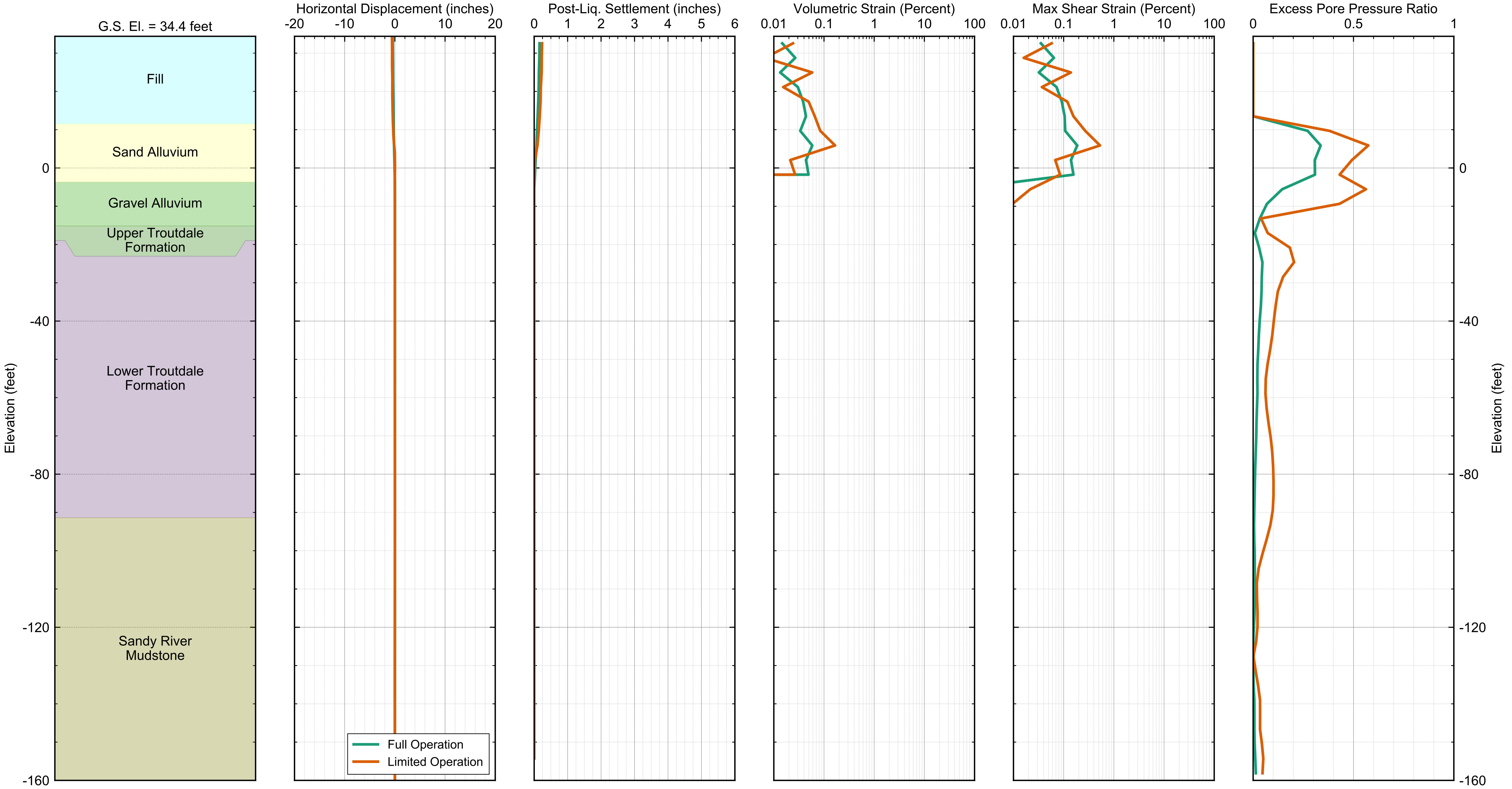
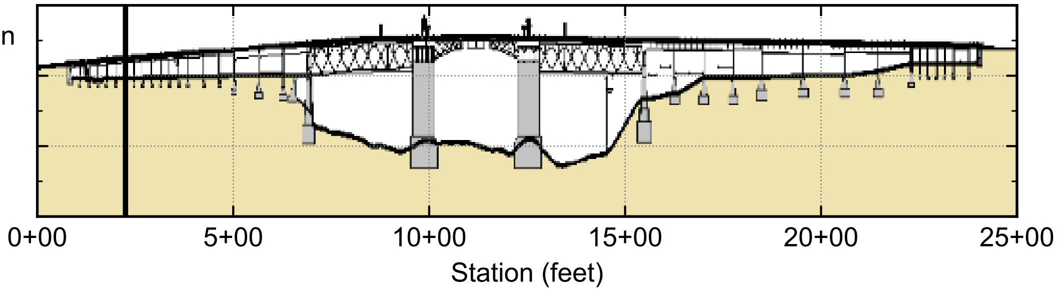


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

**SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
LONG SPAN BENT 3**

May 2020
102636-001

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FIG. E-225

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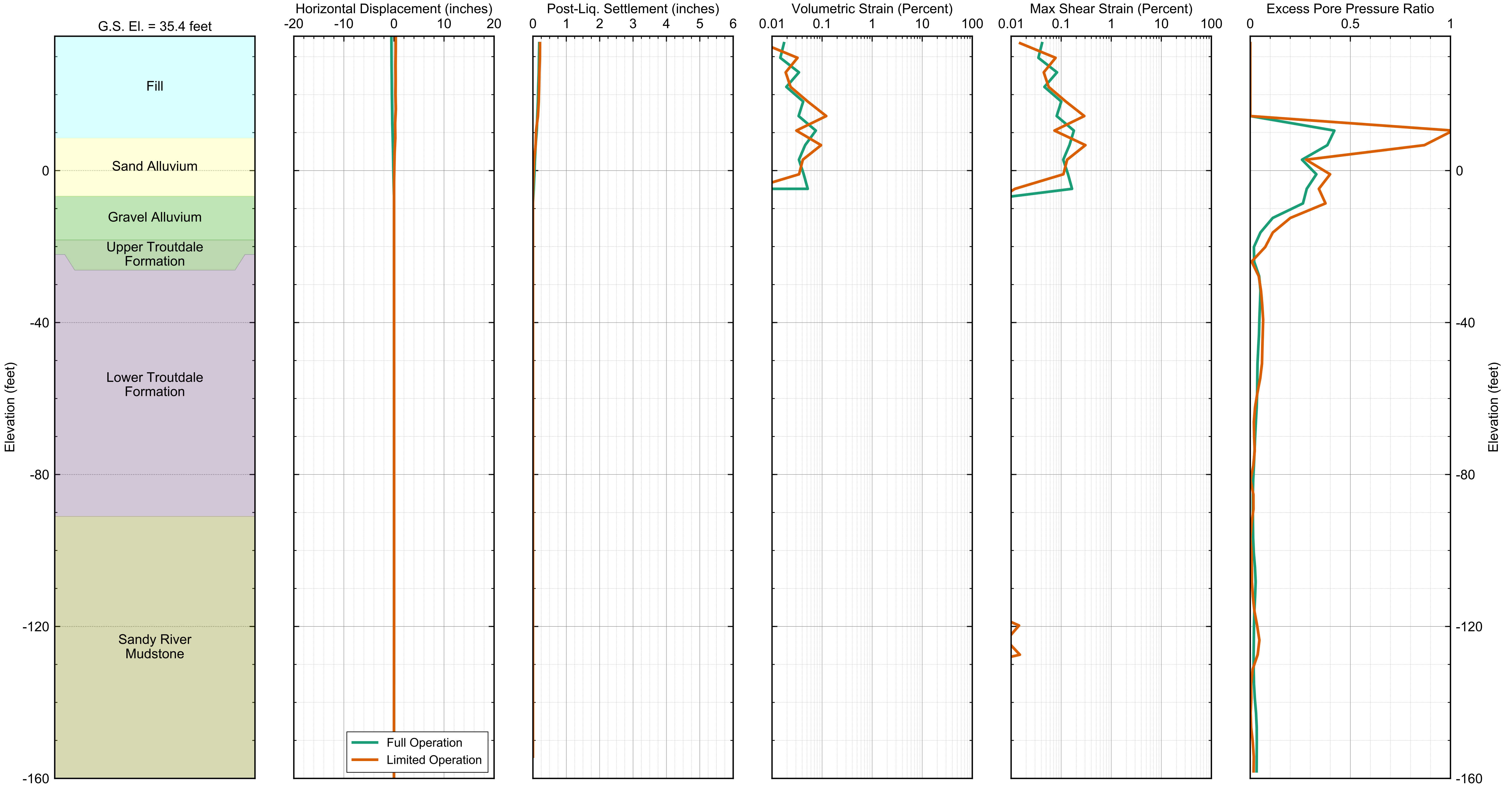
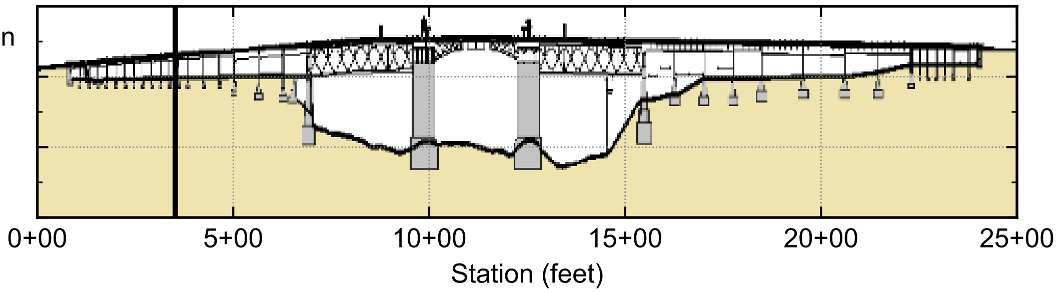


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

**SUBSURFACE PROFILES
WITH GROUND IMPROVEMENT
LONG SPAN BENT 4**

May 2020
102636-001

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FIG. E-226

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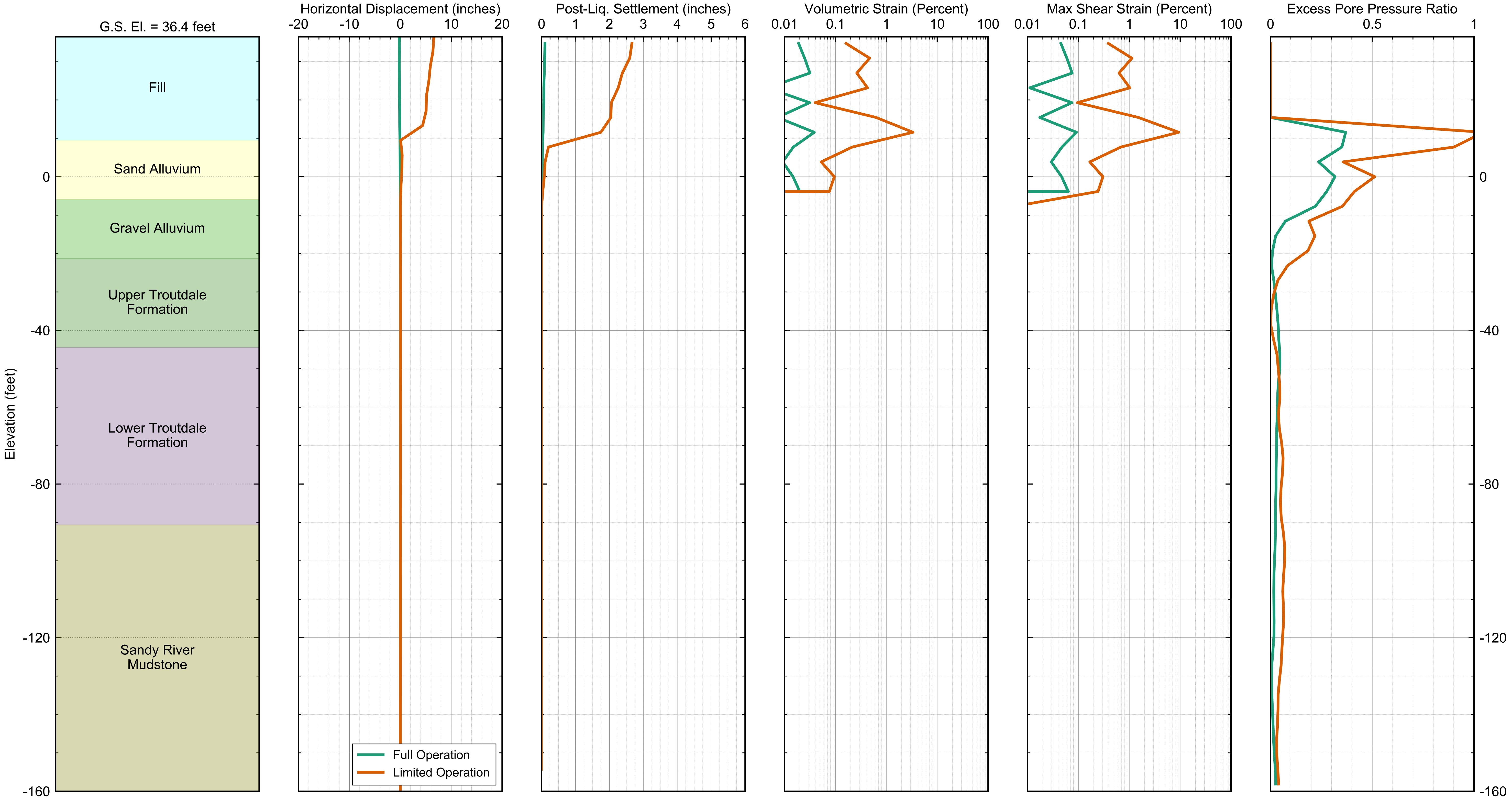
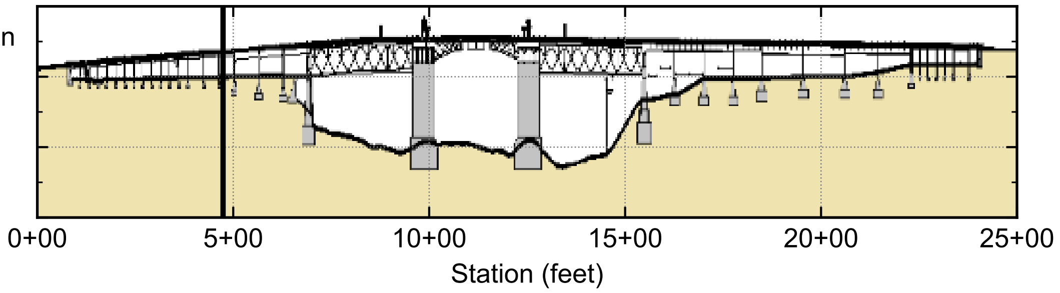


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN BENT 5

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FIG. E-227

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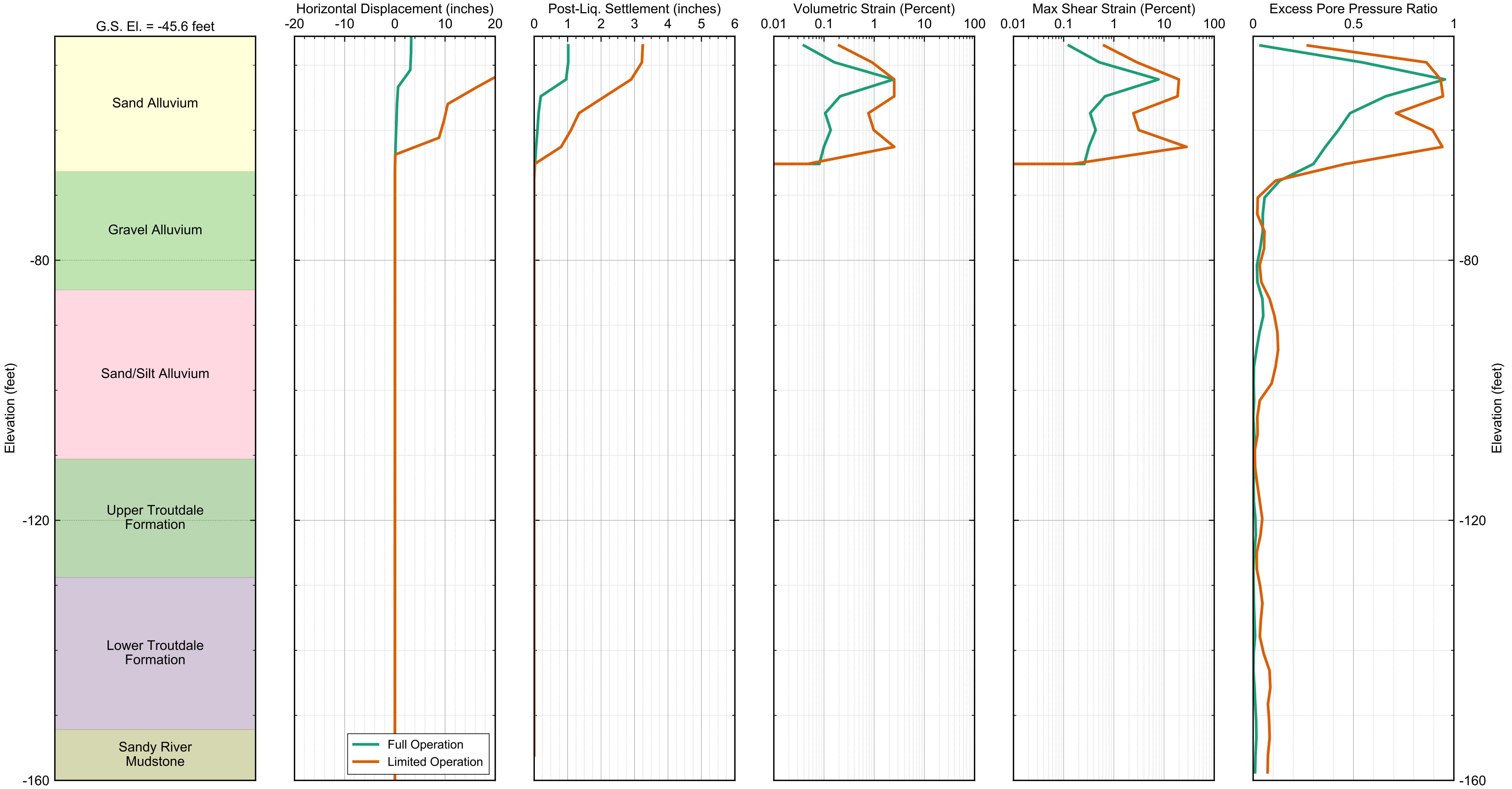
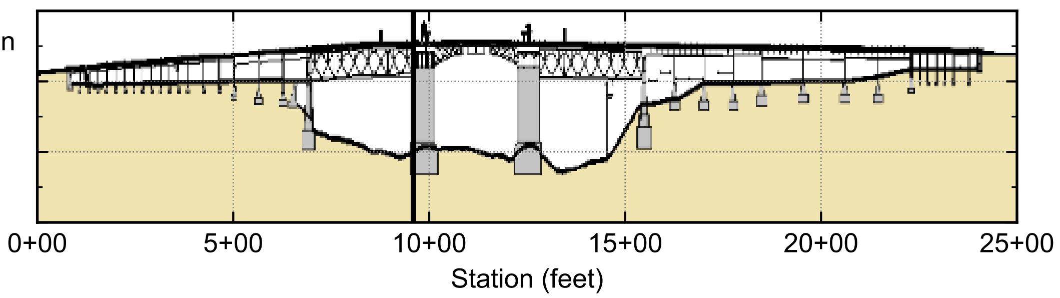


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN BENT 6

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FIG. E-228

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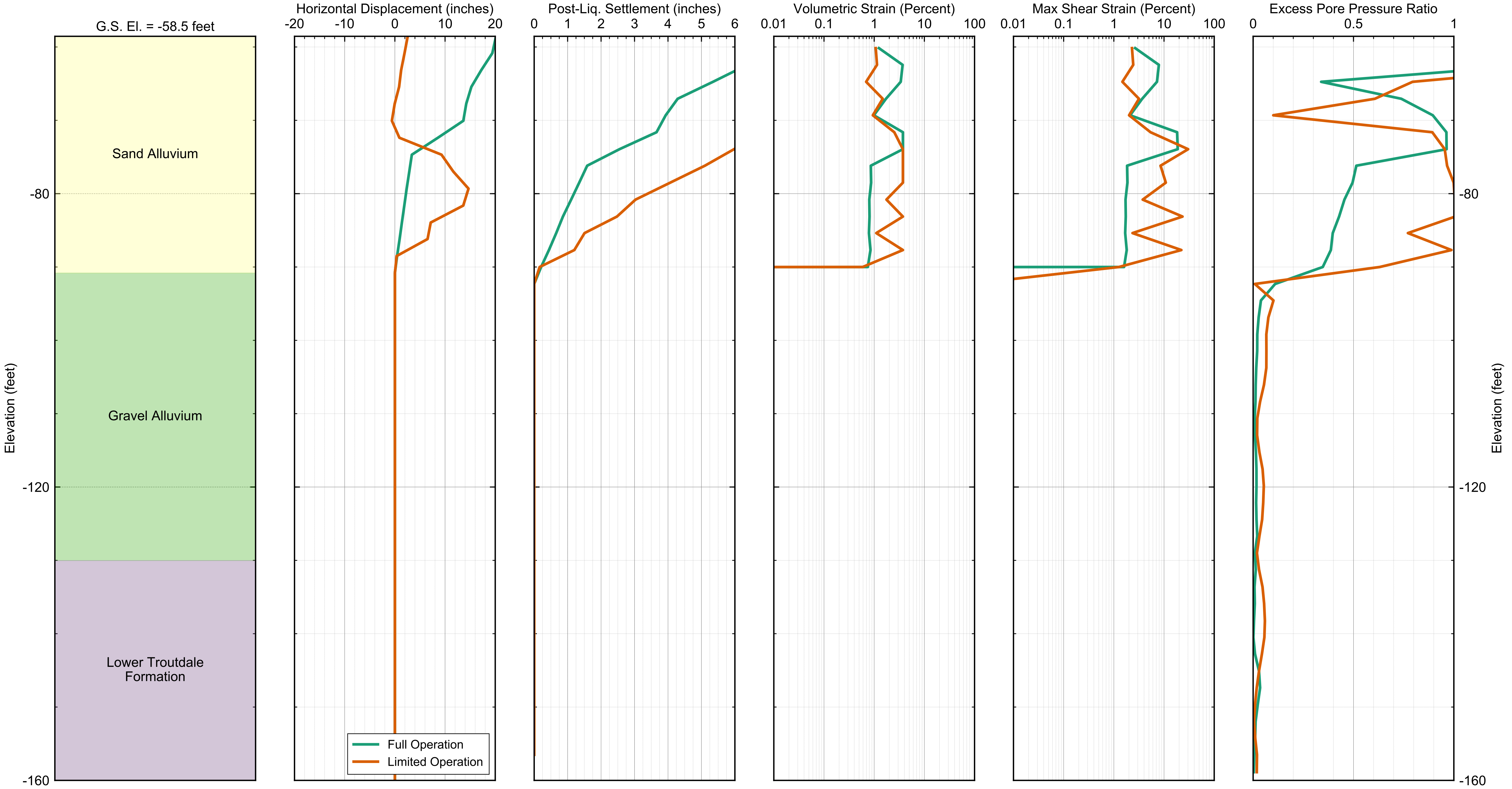
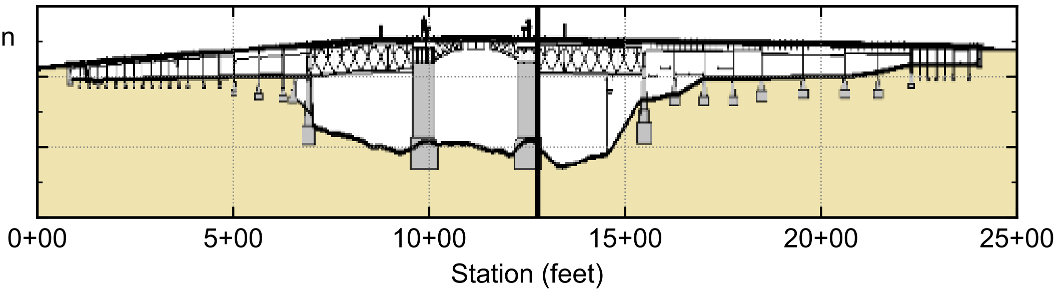


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN BENT 7

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102636-001

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FIG. E-229

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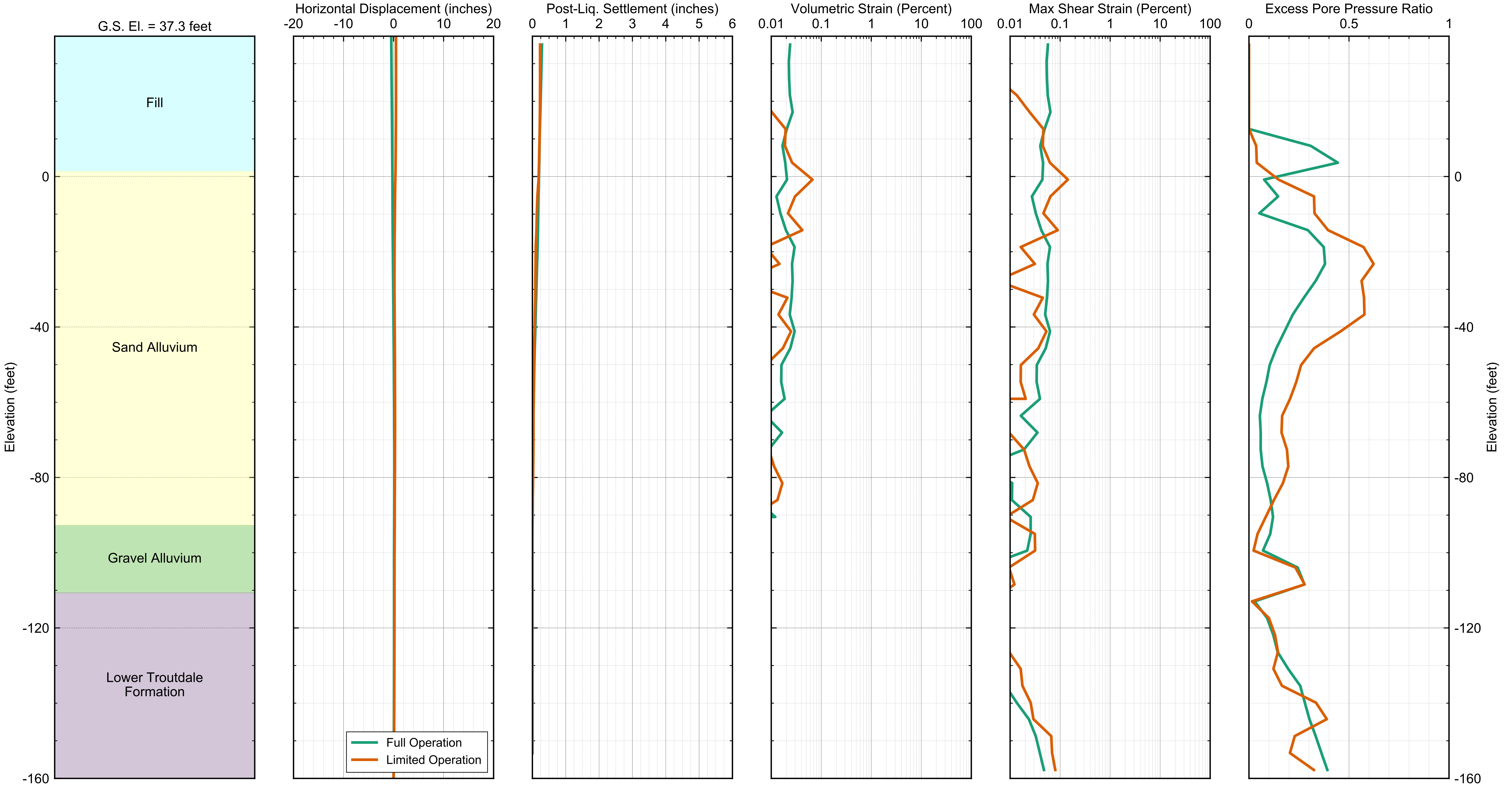
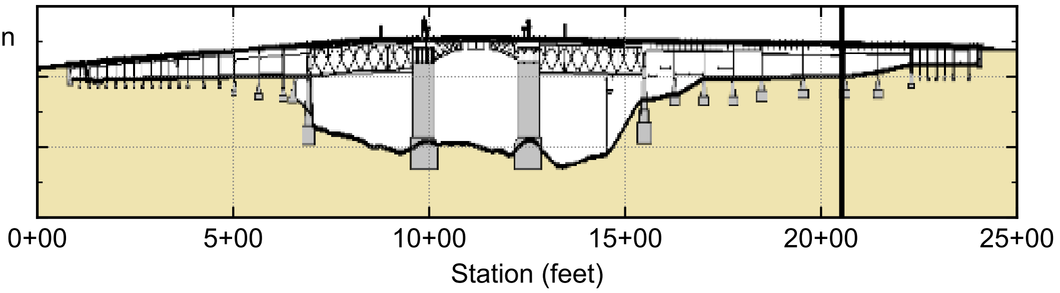


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



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SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN PIER 8

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102636-001

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FIG. E-230

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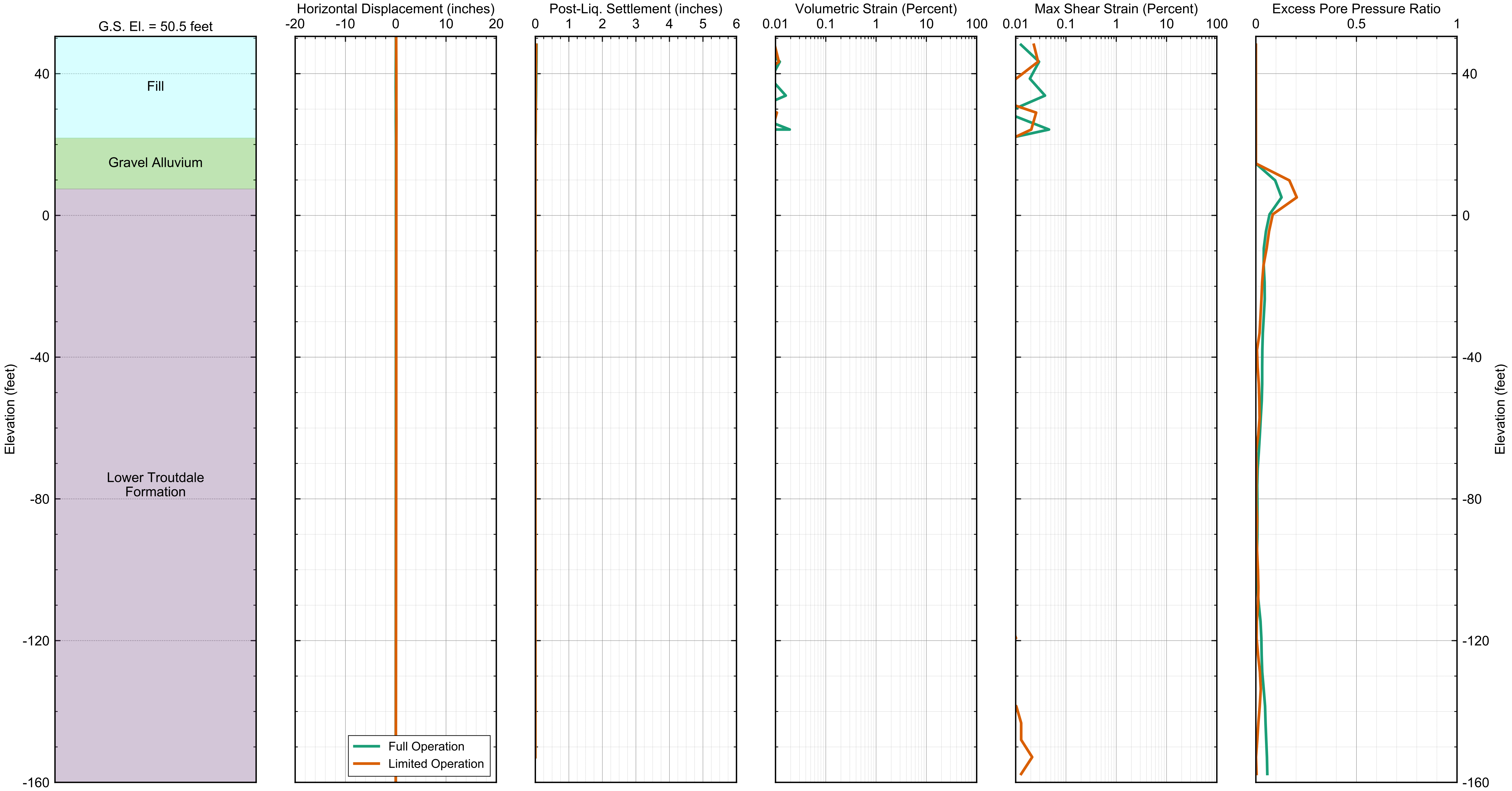
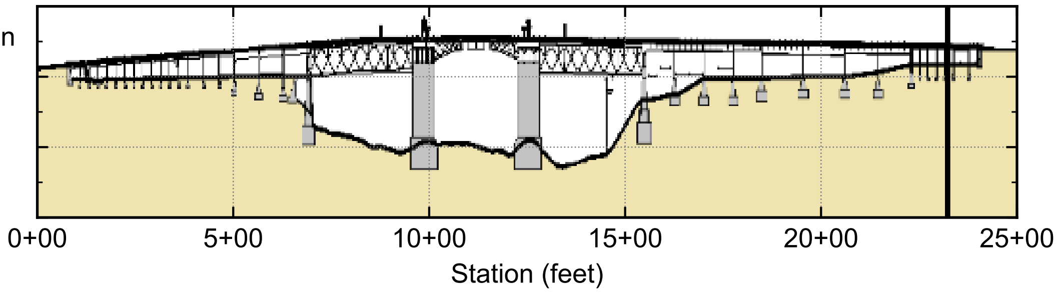


Notes:
1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN PIER 9

May 2020
102636-001

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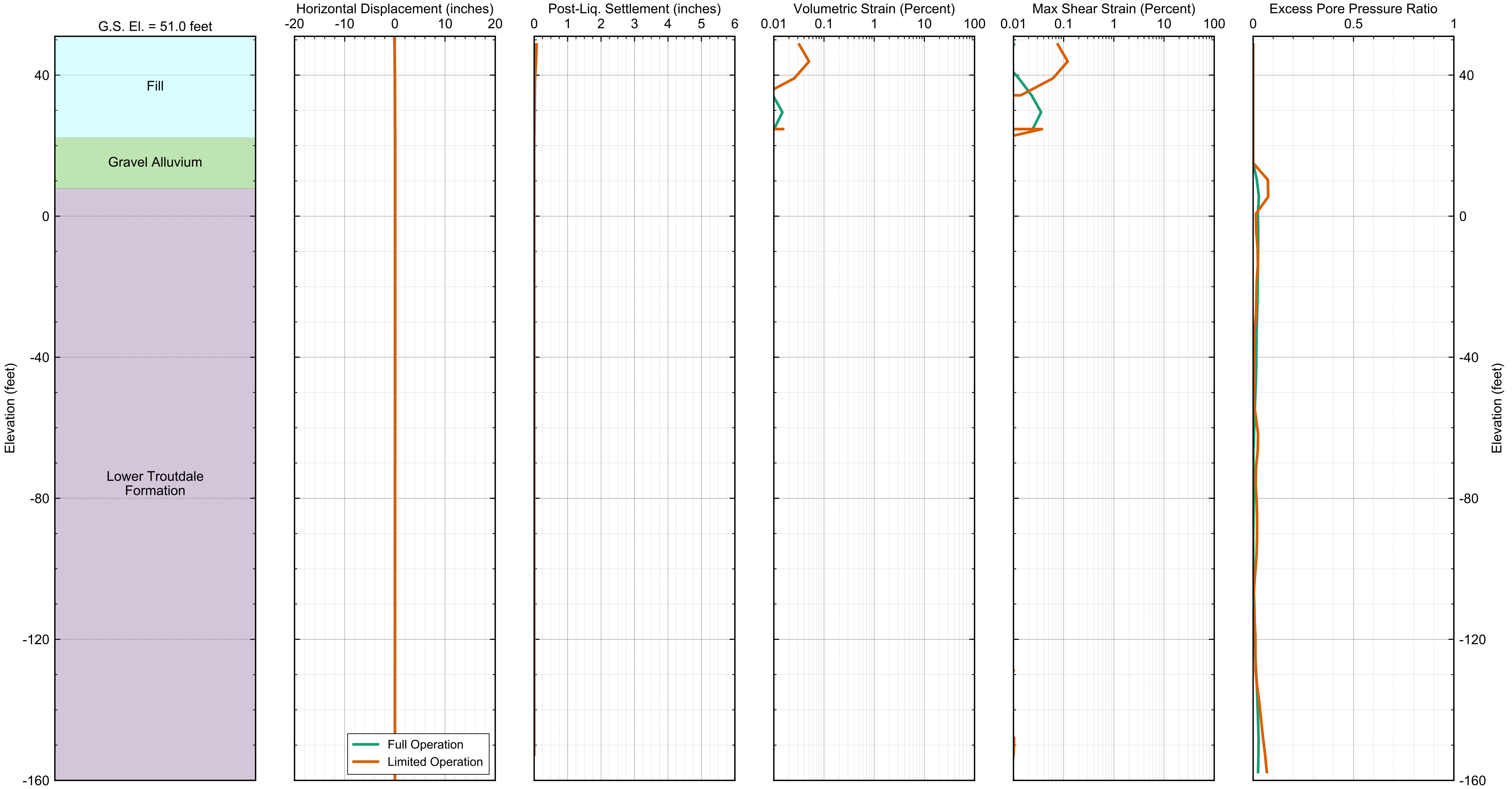
FIG. E-231

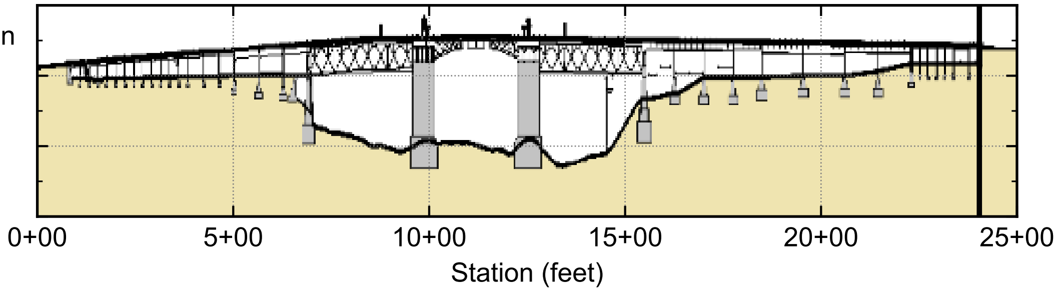
ver:0.7.17.post0.dev0+g54b3f9e, by:sss



Notes:

1. "Full Operation" response is based on the time history O-AKT023EW.
2. "Limited Operation" response is based on the time history LS-CCSP97. See main text for a description of the ground motion time histories.
3. Estimates of volumetric strain are based on correlations to shear strain presented in EERI MNO-12 (Idriss and Boulanger, 2008).
4. Post-liquefaction settlements were calculated by multiplying soil layer thickness by the estimated volumetric strains.
5. G.S. El. = Ground Surface Elevation

Profile Location



Burnside Bridge Environmental Impact Study
Portland, Oregon

SUBSURFACE PROFILES WITH GROUND IMPROVEMENT LONG SPAN BENT 10

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102636-001

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FIG. E-232

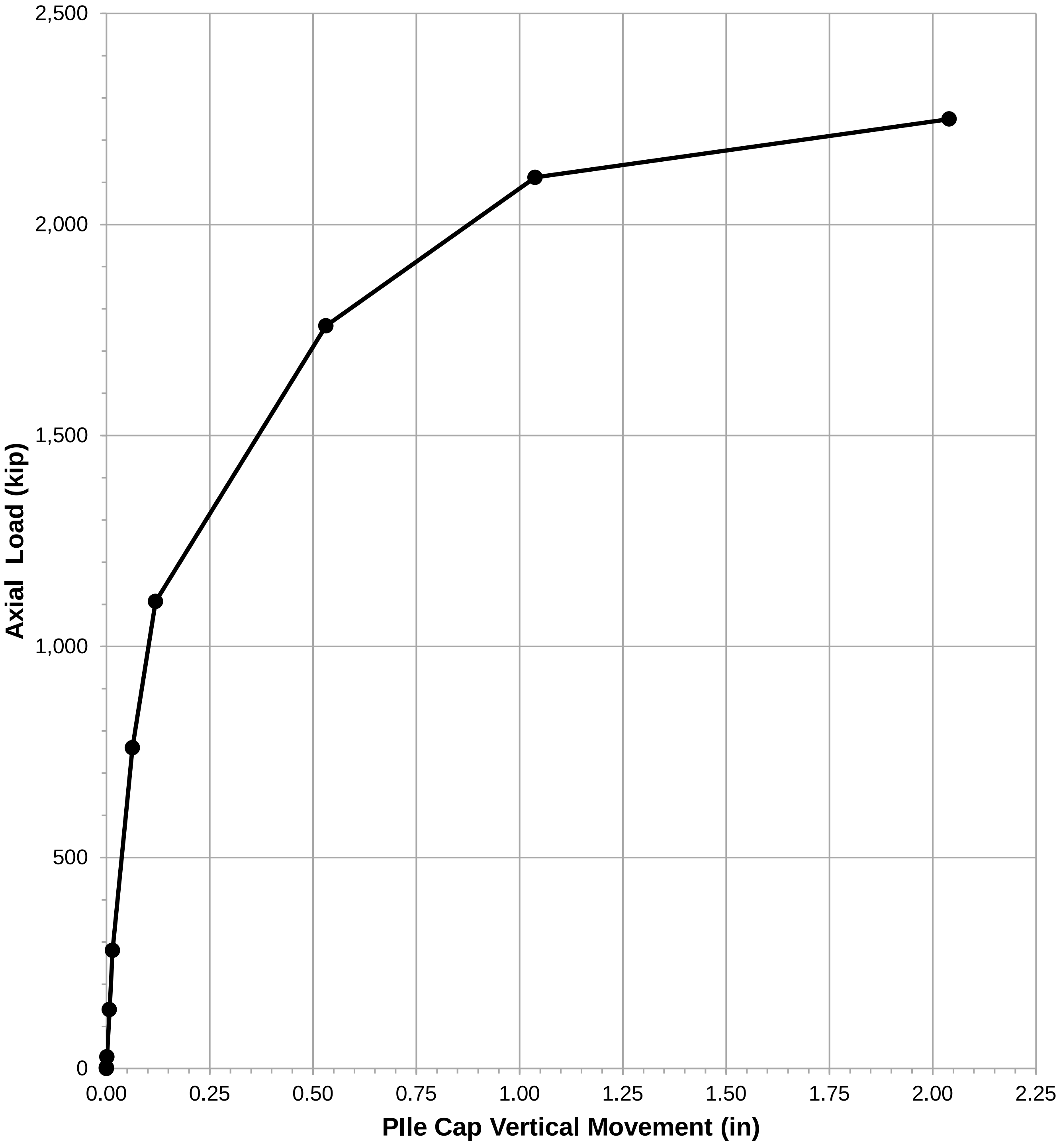
C:\ FLACPROCESS\burnside

Appendix F

Load-Displacement Curves for Existing Pile Groups

Figures

F1 to F26 Existing Pile Group Axial and Lateral Load-Displacement Relationships for Static and Seismic Conditions from Previous Phase



NOTES

1. See Table 2 for details of each pile group evaluated.

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Portland, Oregon

**PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 18**

May 2017 24-1-04065-005

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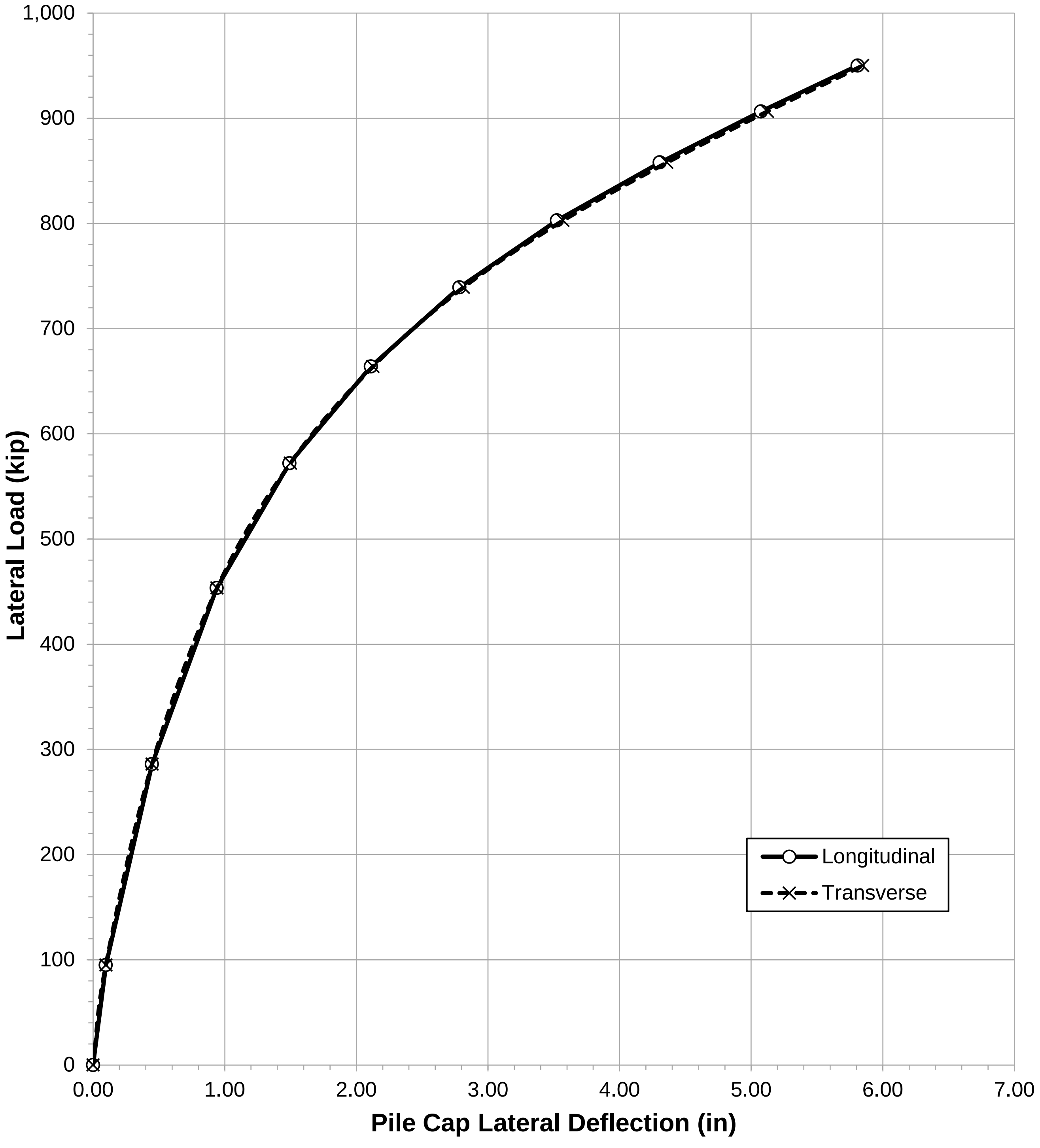
FIG. F1



NOTES

1. See Table 2 for details of each pile group evaluated.

2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

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PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 18

May 2017 24-1-04065-005

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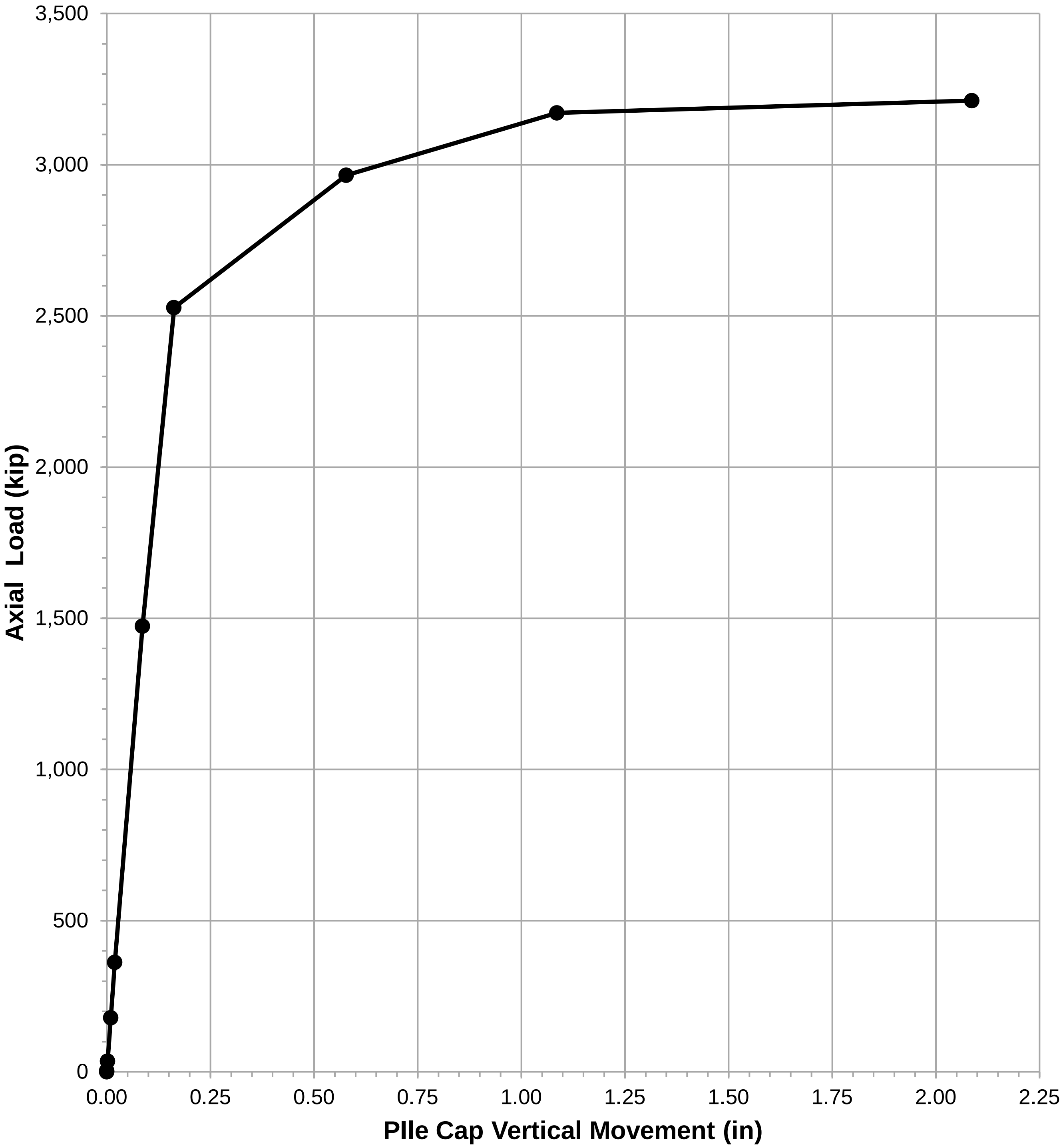
FIG. F2



NOTES

1. See Table 2 for details of each pile group evaluated.

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PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 19

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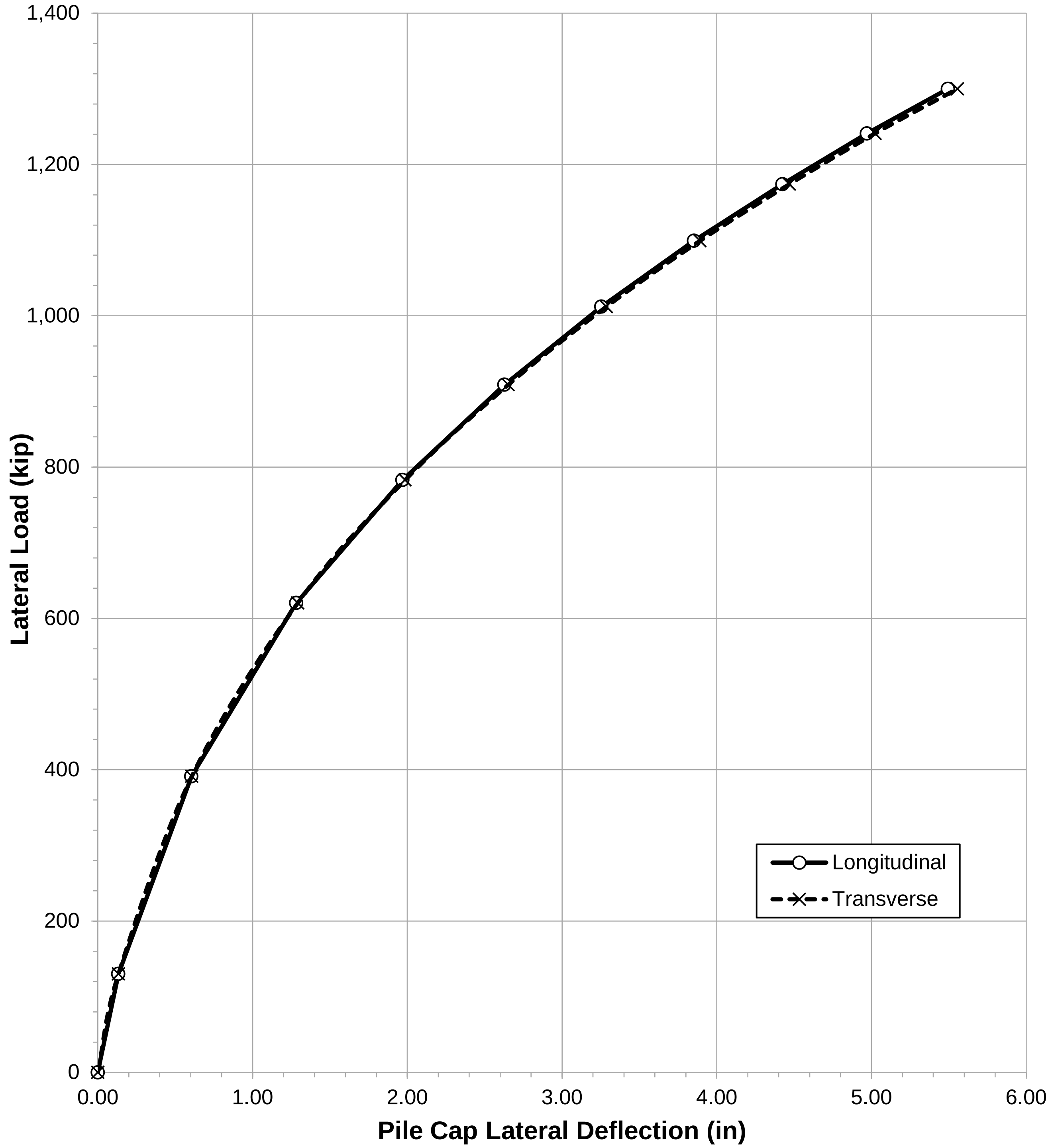
FIG. F3



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 19

May 2017 24-1-04065-005

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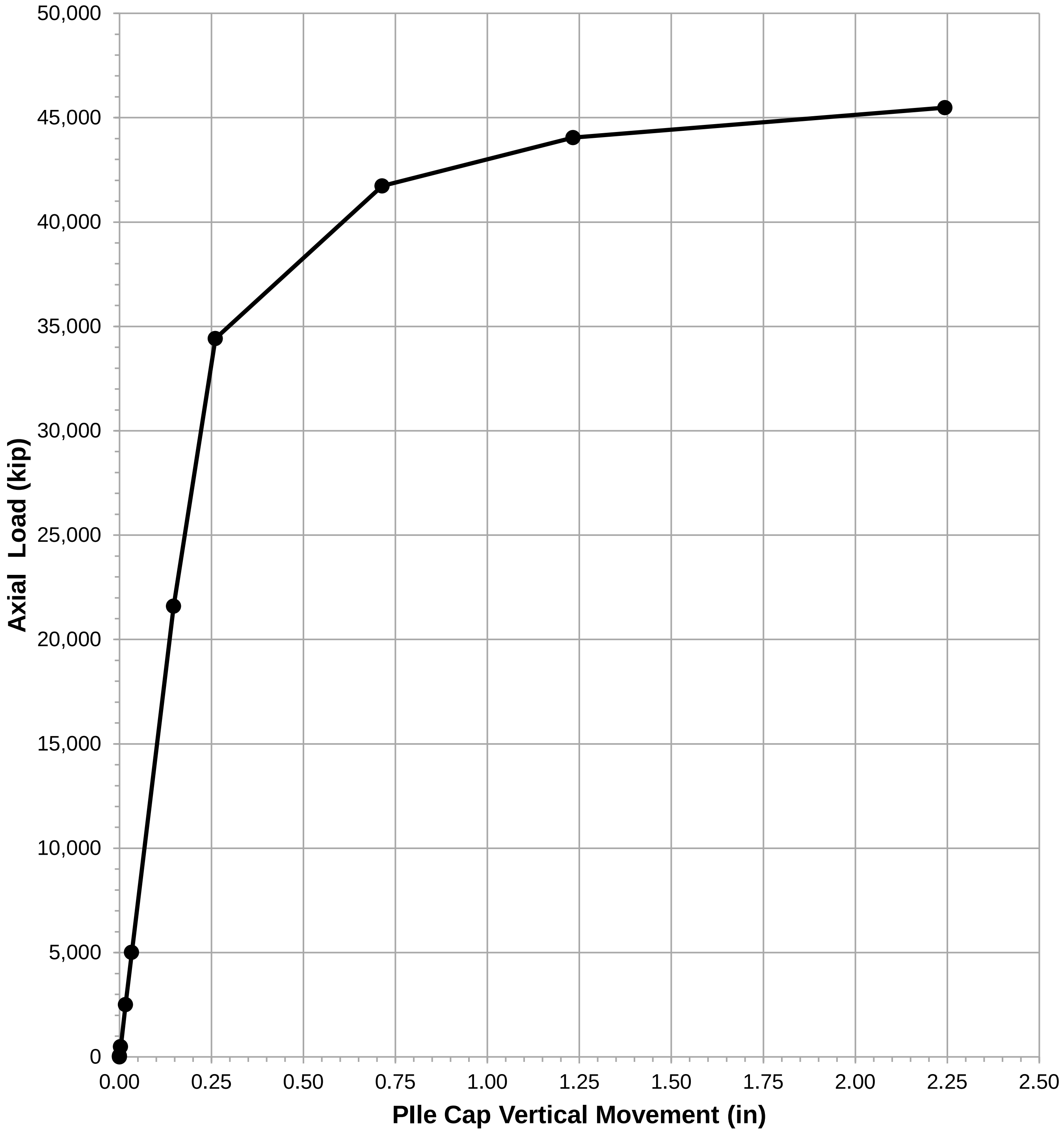
FIG. F4



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
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**PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 1**

May 2017 24-1-04065-005

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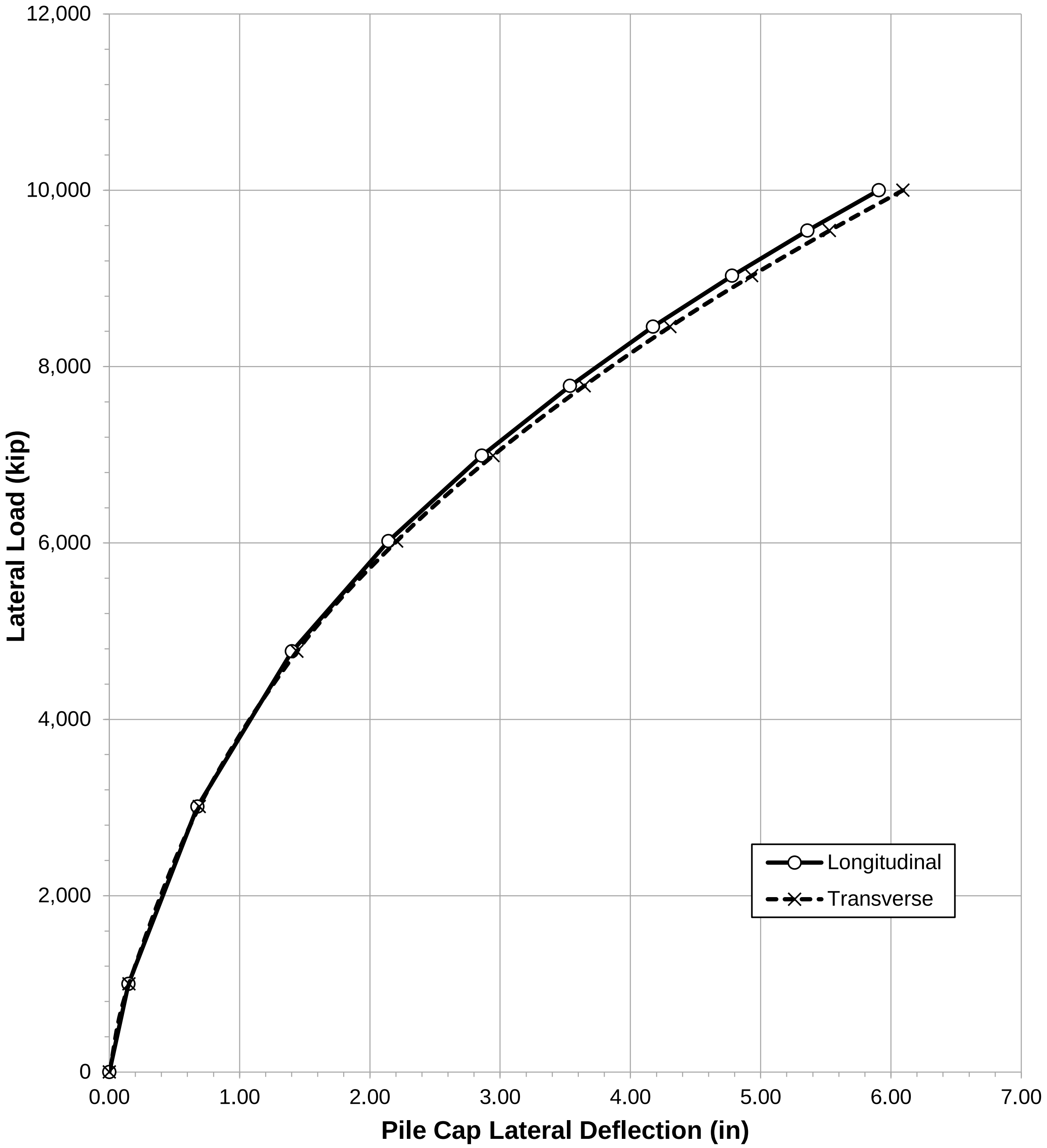
FIG. F5



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 1

May 2017
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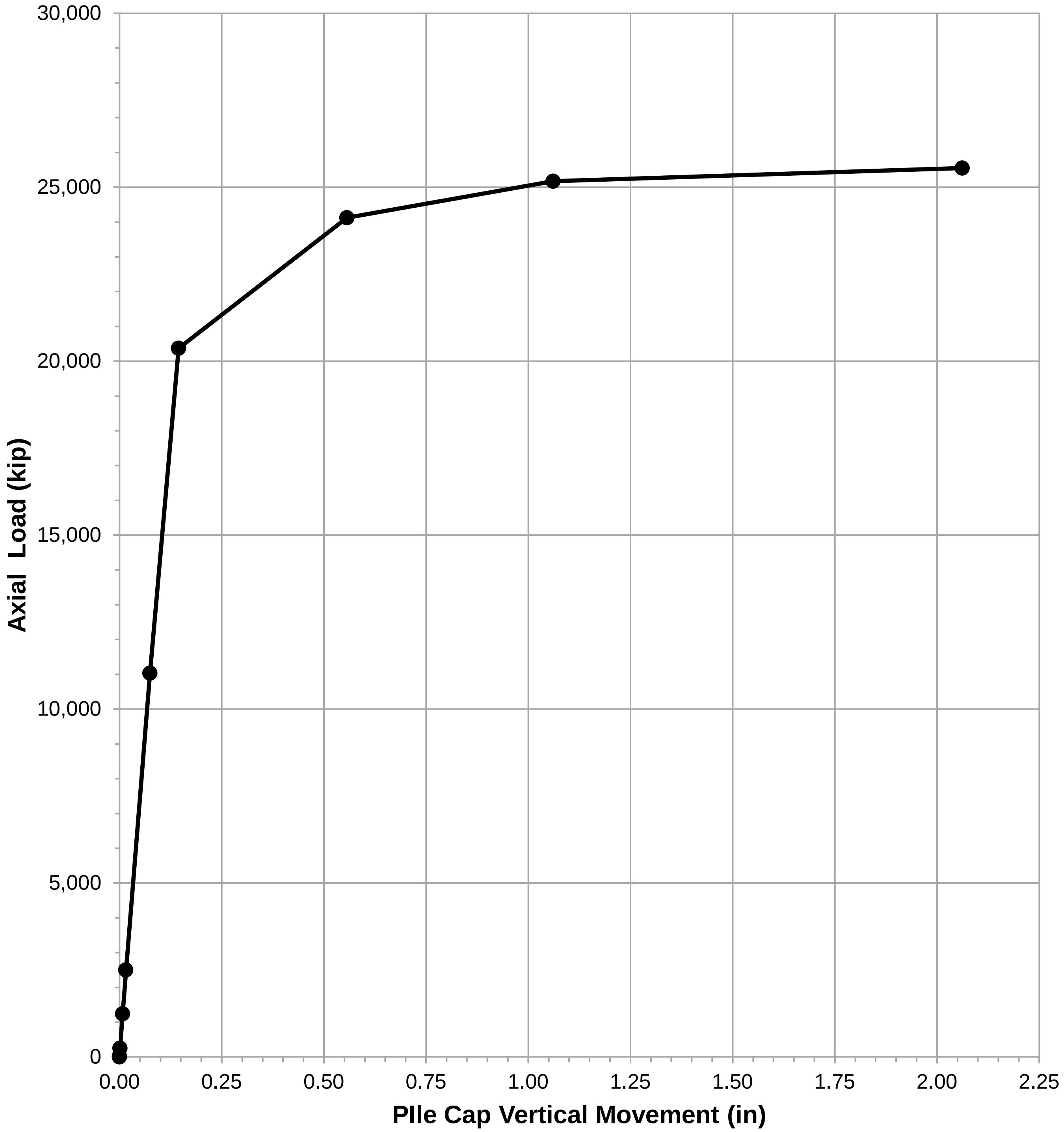
FIG. F6



NOTES

1. See Table 2 for details of each pile group evaluated.

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Portland, Oregon

PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 2

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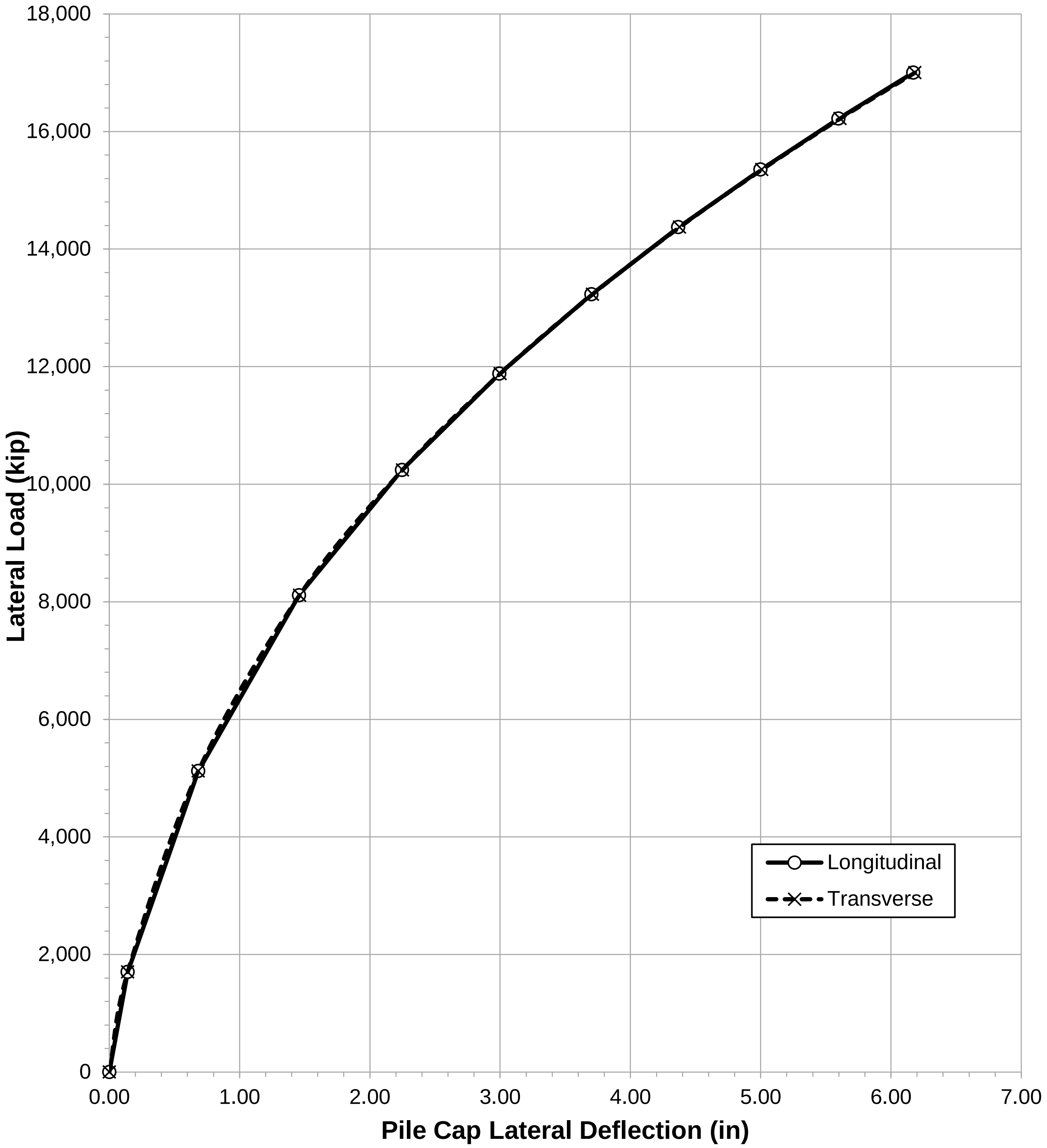
FIG. F7



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

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Portland, Oregon

**PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 2**

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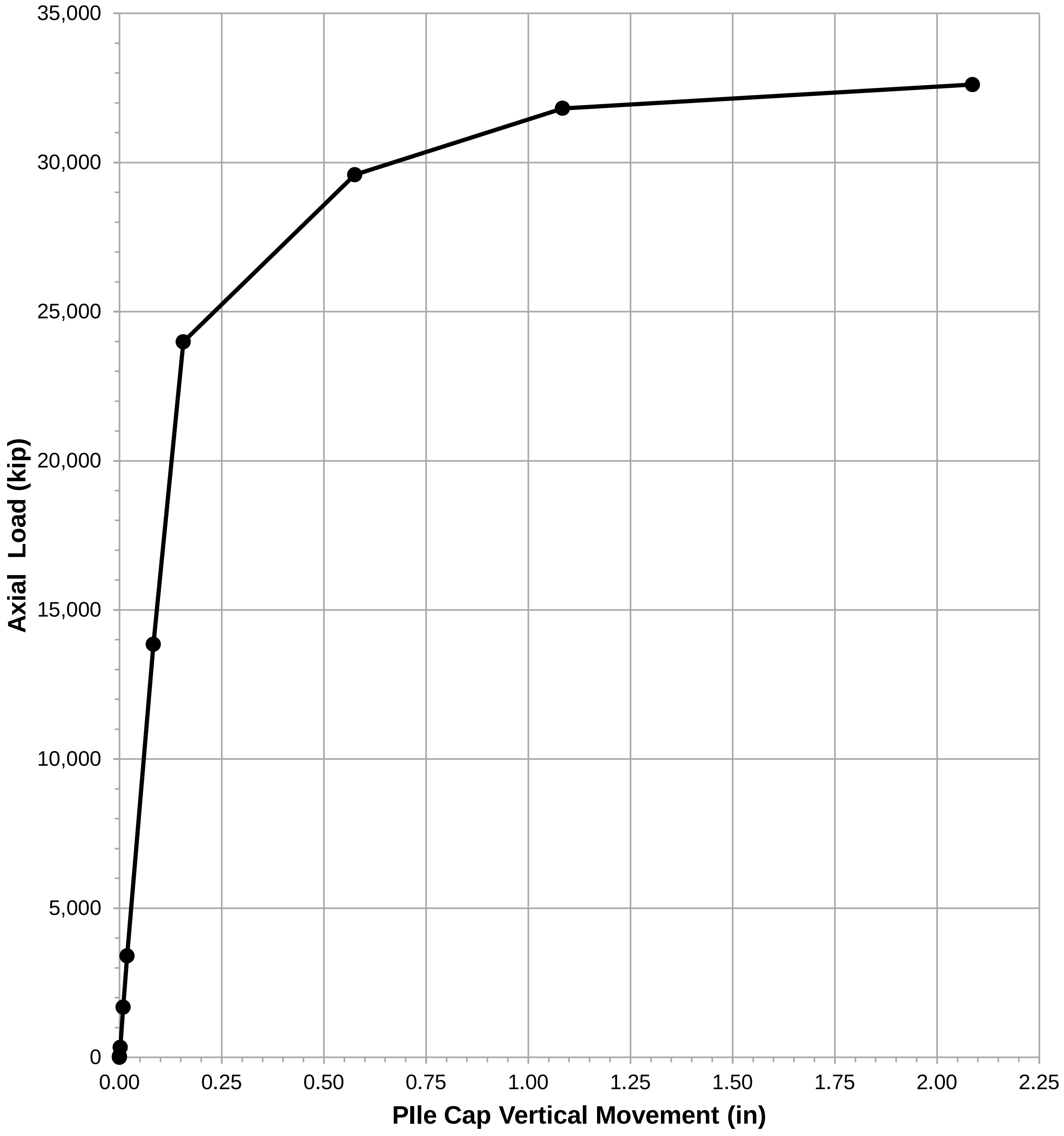
FIG. F8



NOTES

1. See Table 2 for details of each pile group evaluated.

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**PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 3**

May 2017 24-1-04065-005

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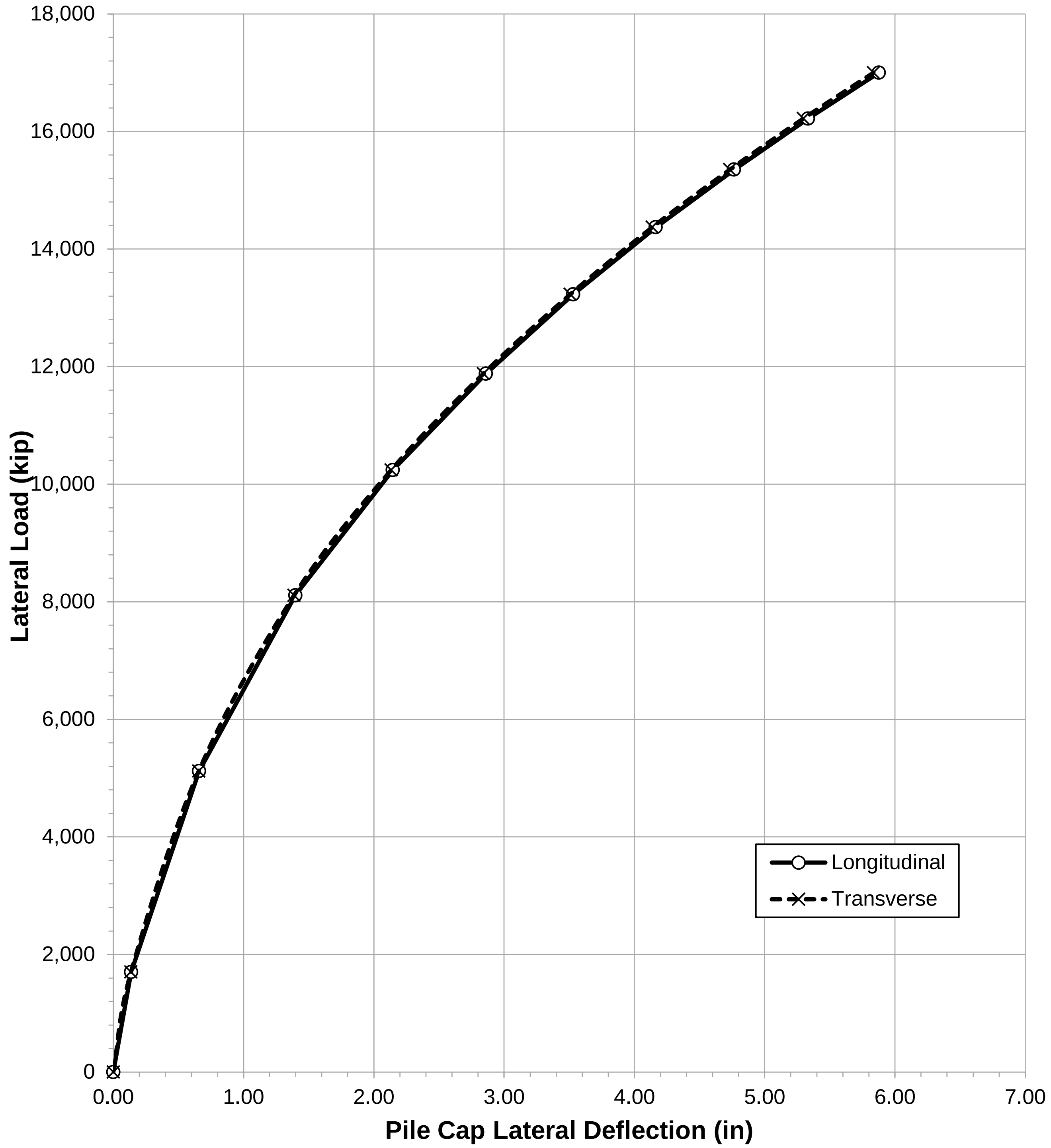
FIG. F9



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

**PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 3**

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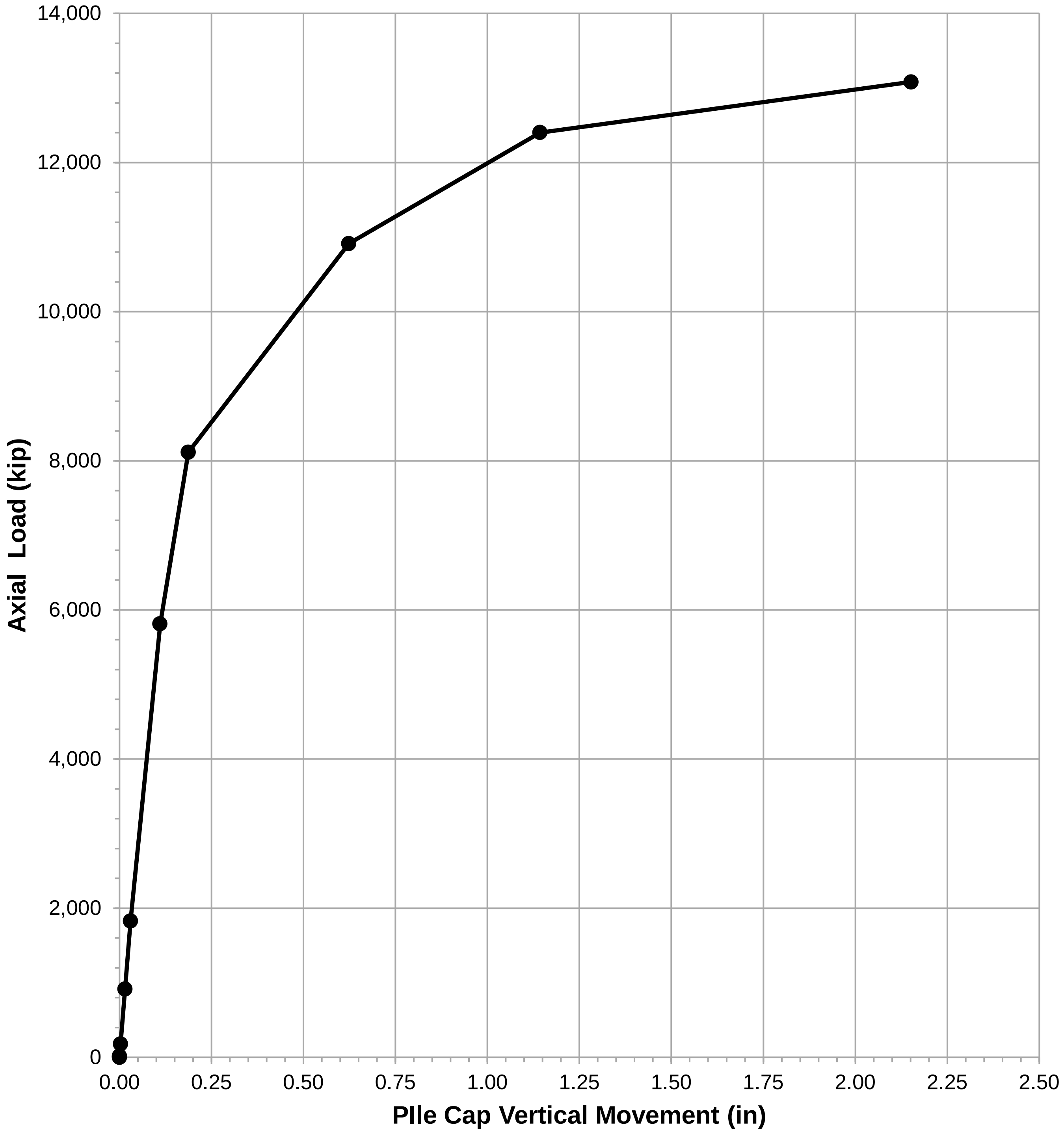
FIG. F10



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 4

May 2017
24-1-04065-005

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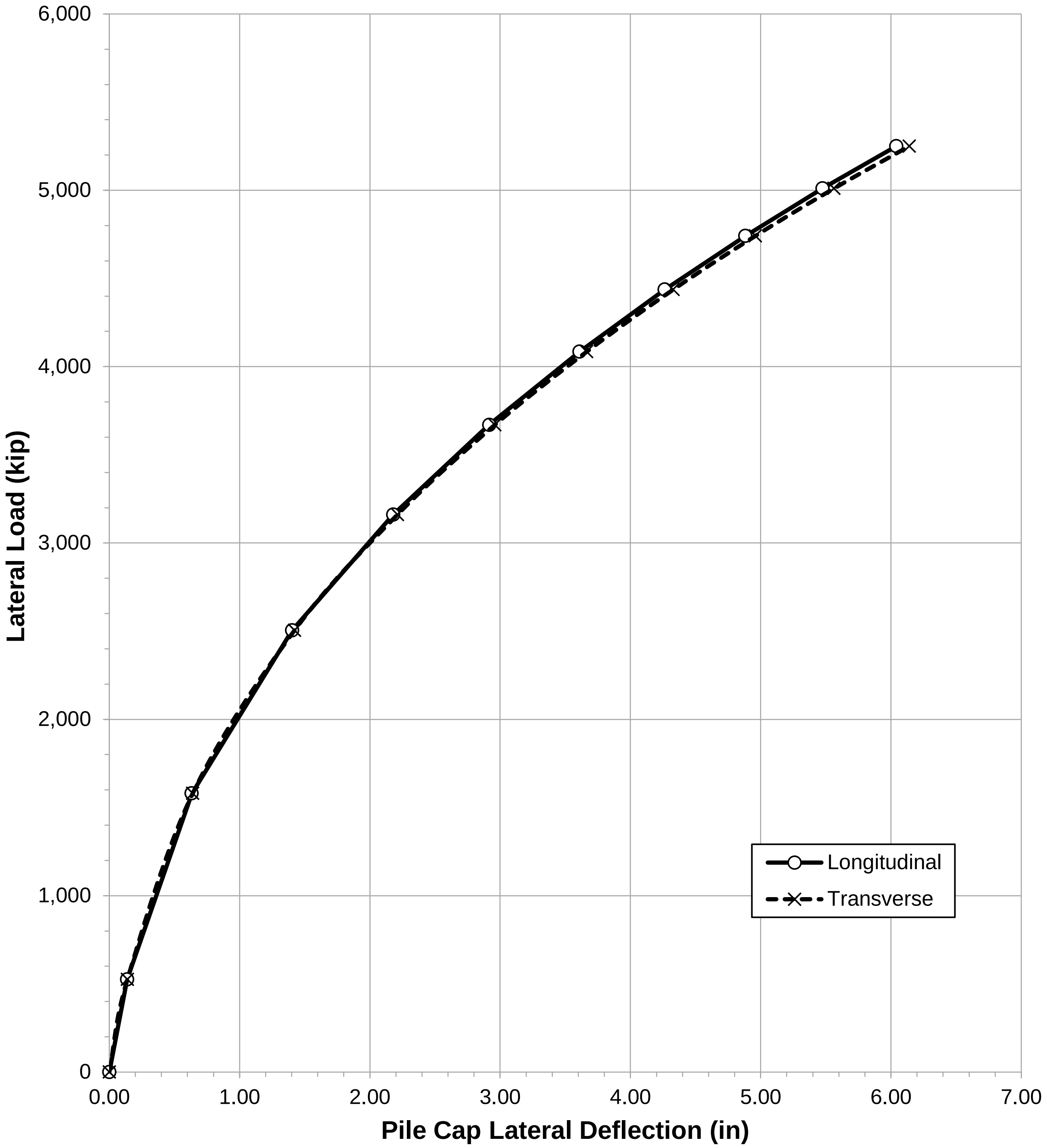
FIG. F11



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
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PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT PIER 4

May 2017 24-1-04065-005

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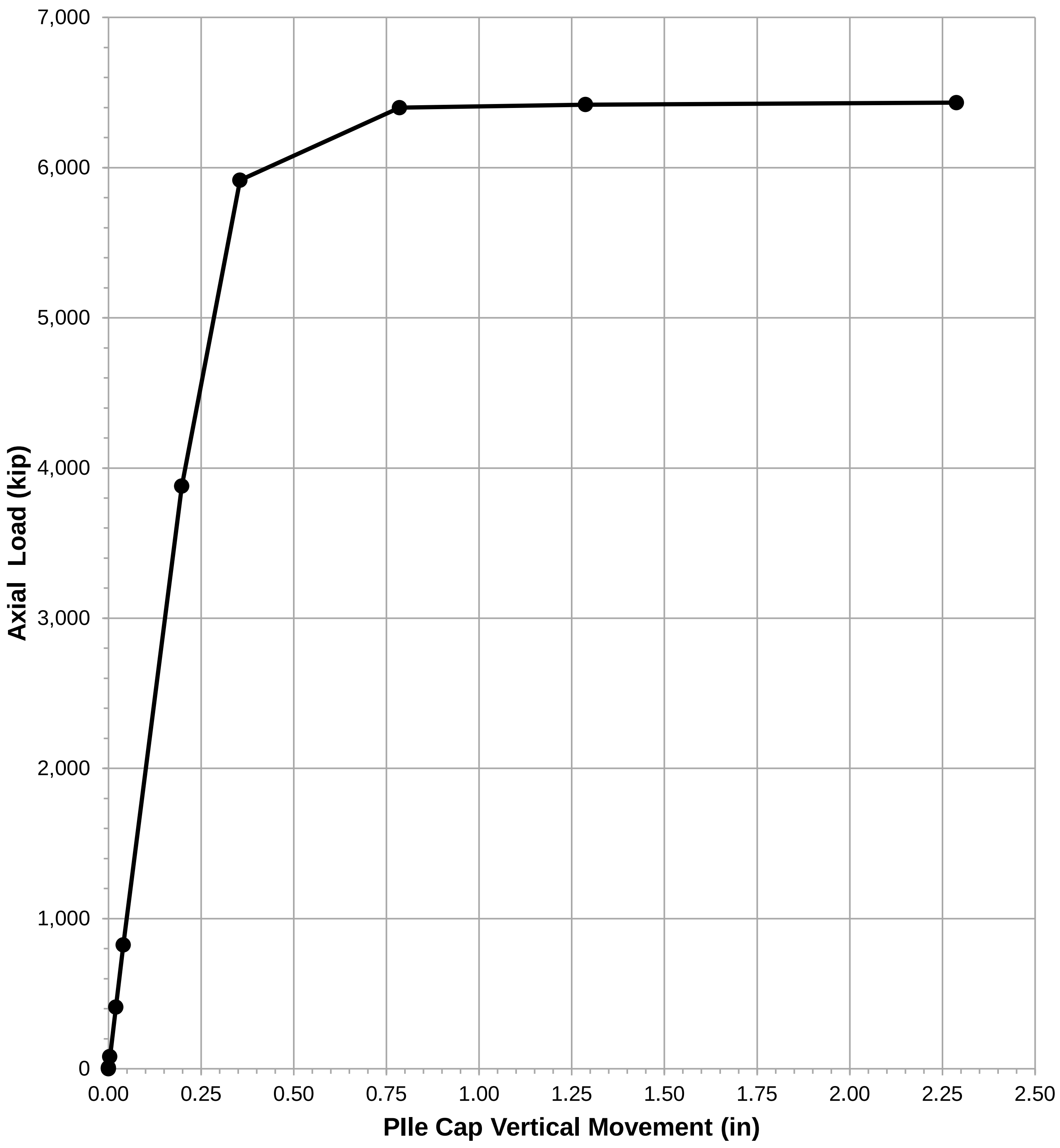
FIG. F12



NOTES

1. See Table 2 for details of each pile group evaluated.

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Portland, Oregon

PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 21

May 2017
24-1-04065-004

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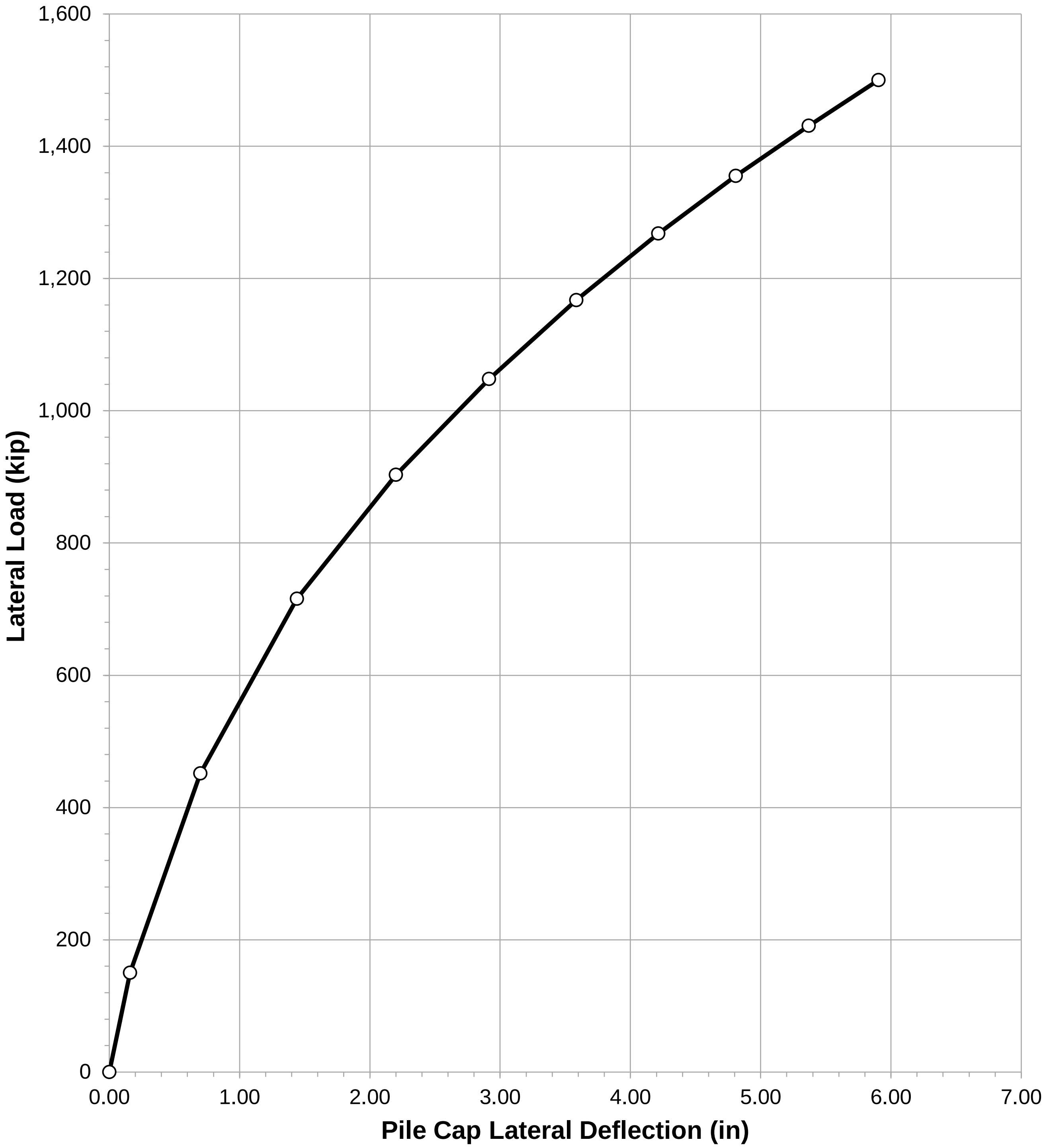
FIG. F13



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.
3. Recommended pile group lateral load-displacement curve can be used for both longitudinal and transverse loading directions.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 21

May 2017 24-1-04065-005

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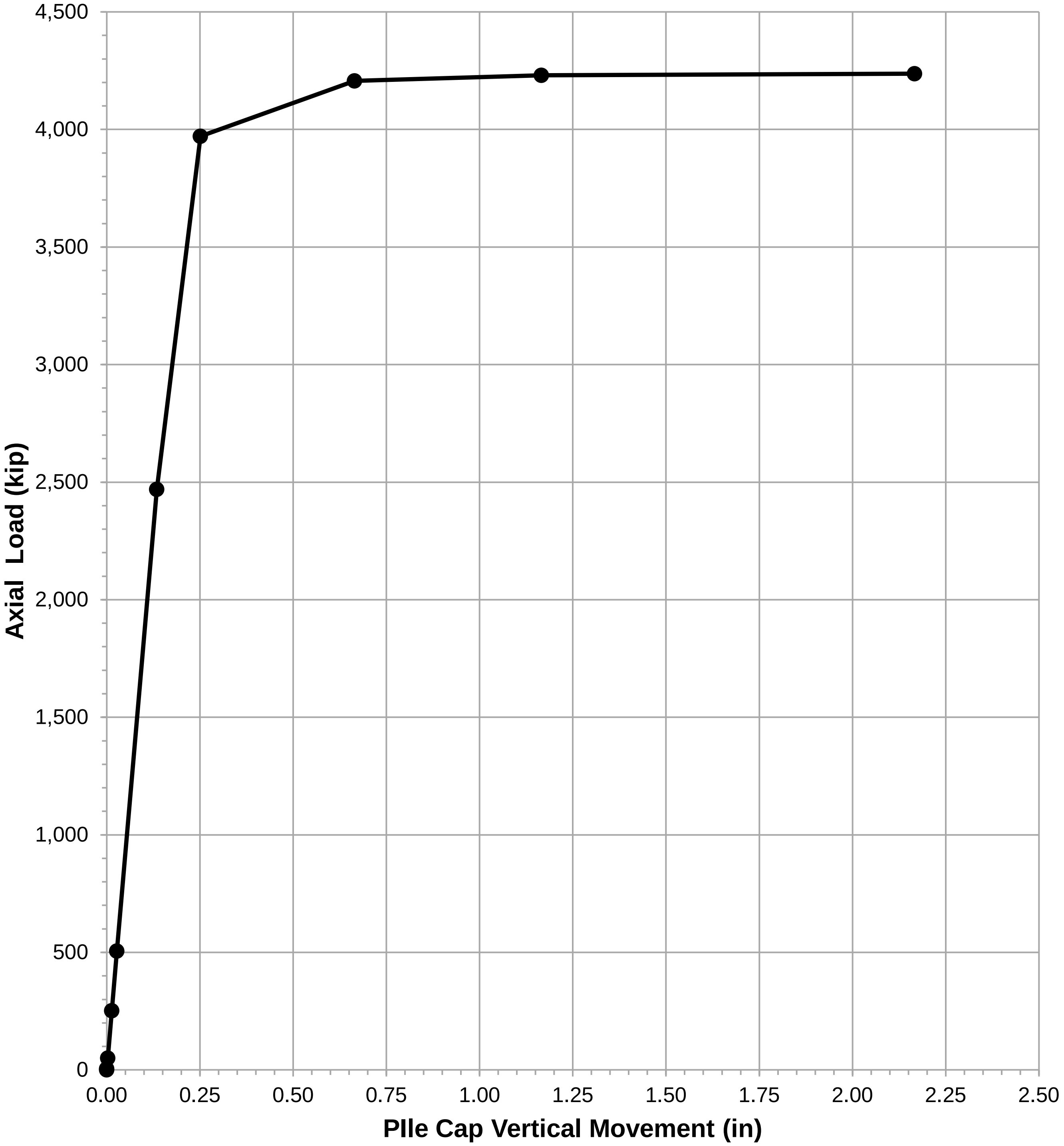
FIG. F14



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

**PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 22**

May 2017 24-1-04065-004

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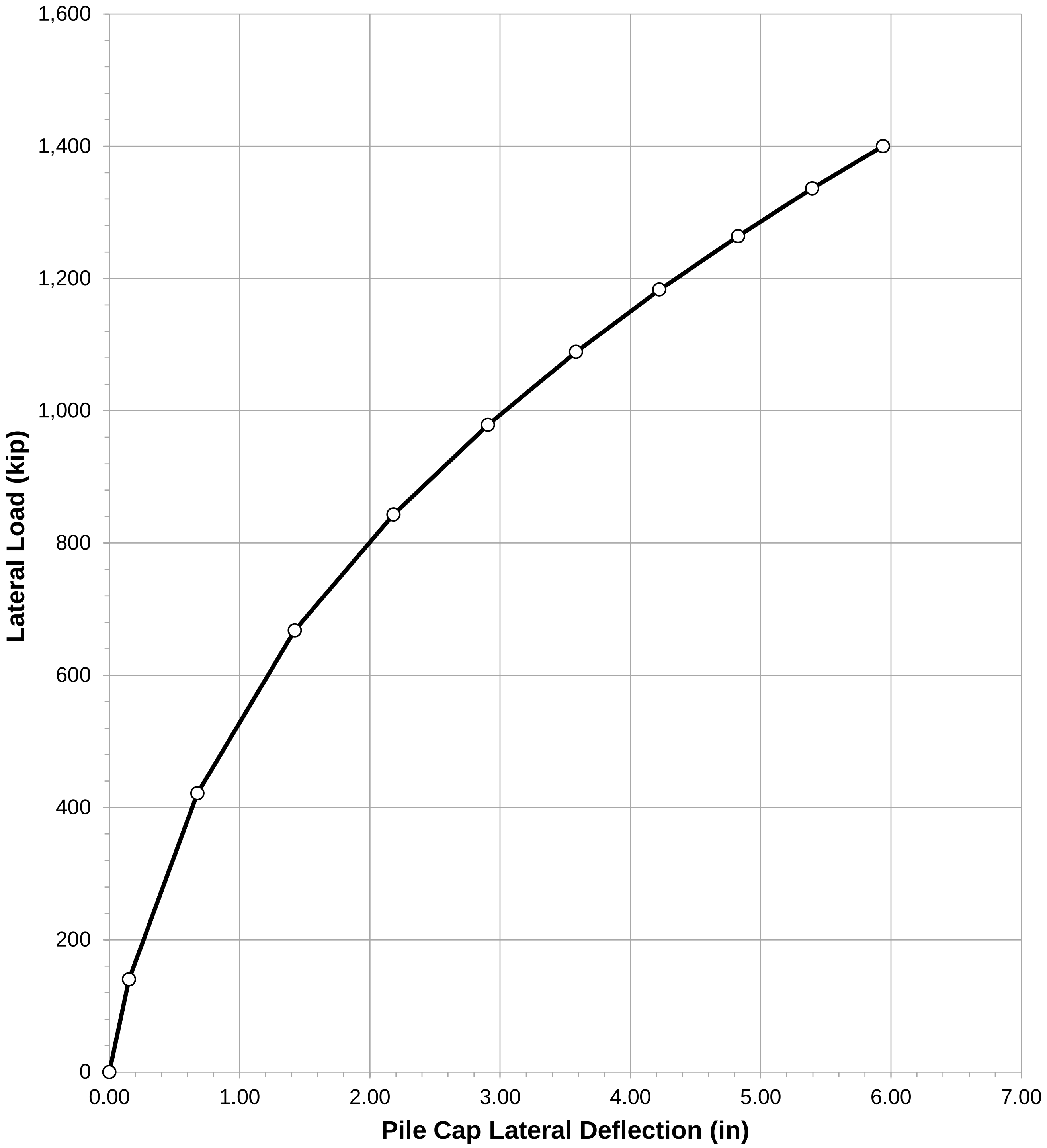
FIG. F15



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.
3. Recommended pile group lateral load-displacement curve can be used for both longitudinal and transverse loading directions.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

**PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 22**

May 2017 24-1-04065-005

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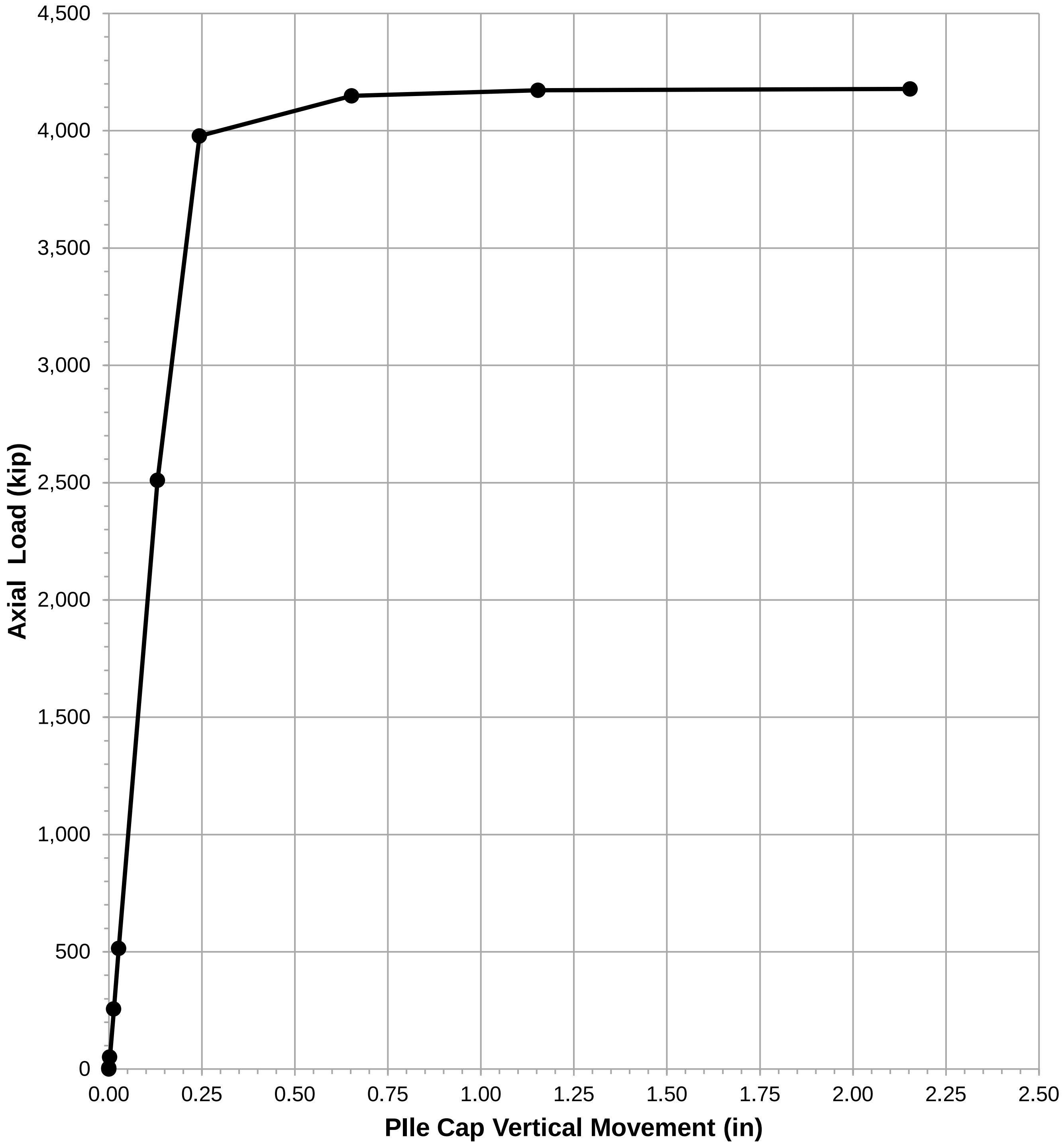
FIG. F16



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 23

May 2017 24-1-04065-004

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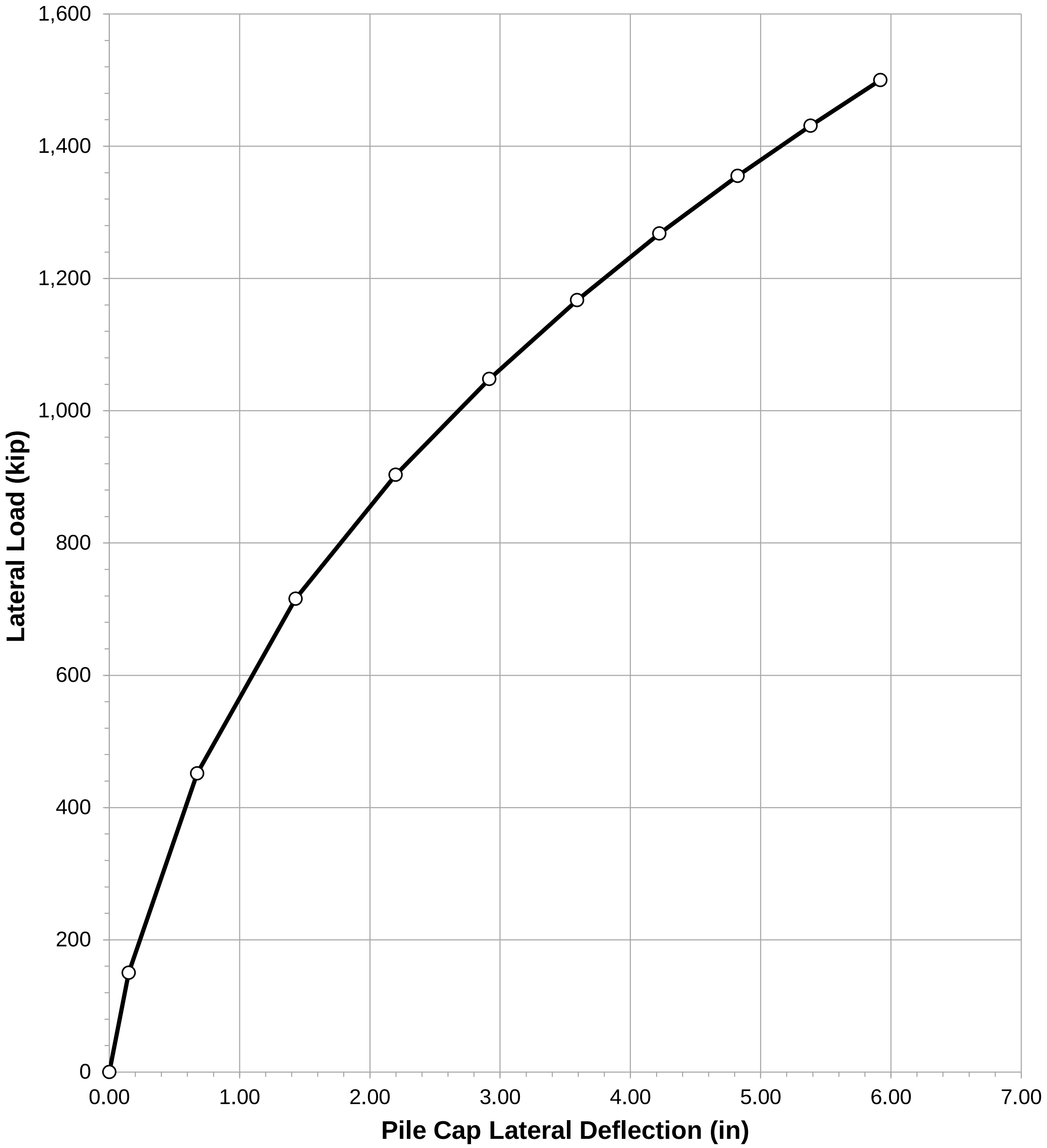
FIG. F17



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.
3. Recommended pile group lateral load-displacement curve can be used for both longitudinal and transverse loading directions.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 23

May 2017
24-1-04065-005

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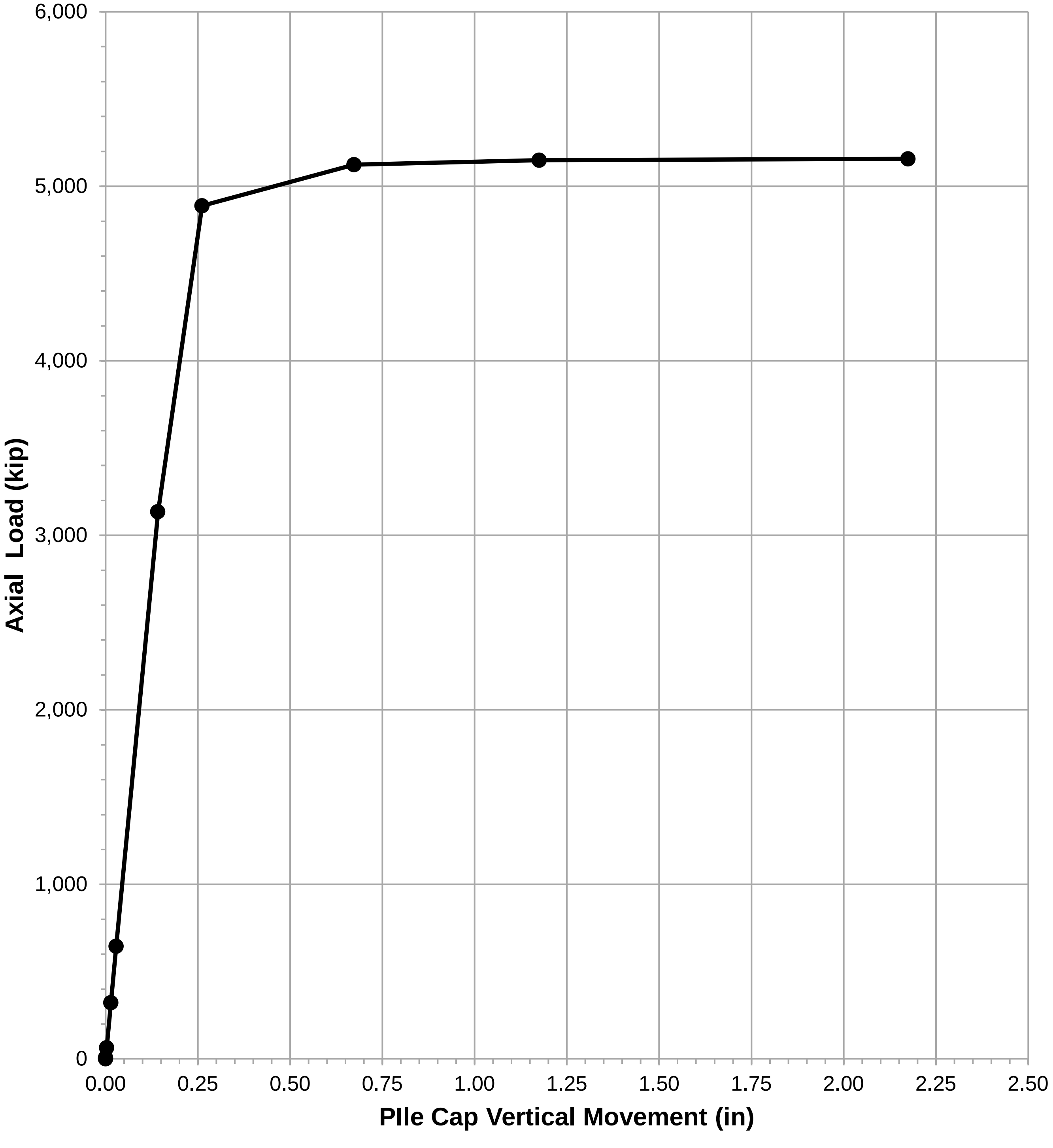
FIG. F18



NOTES
1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 24

May 2017
24-1-04065-004

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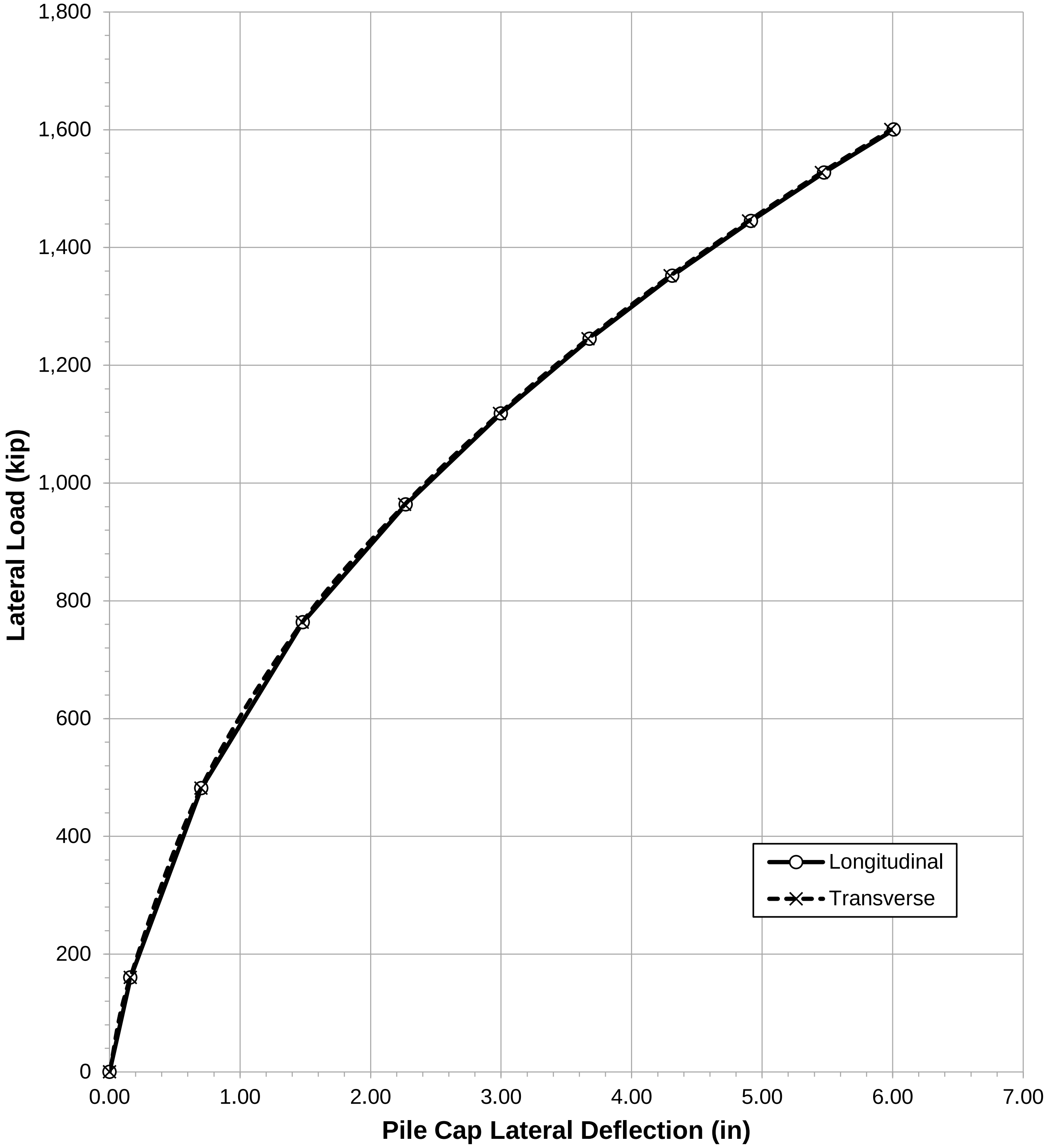
FIG. F19



NOTES

1. See Table 2 for details of each pile group evaluated.

2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 24

May 2017
24-1-04065-005

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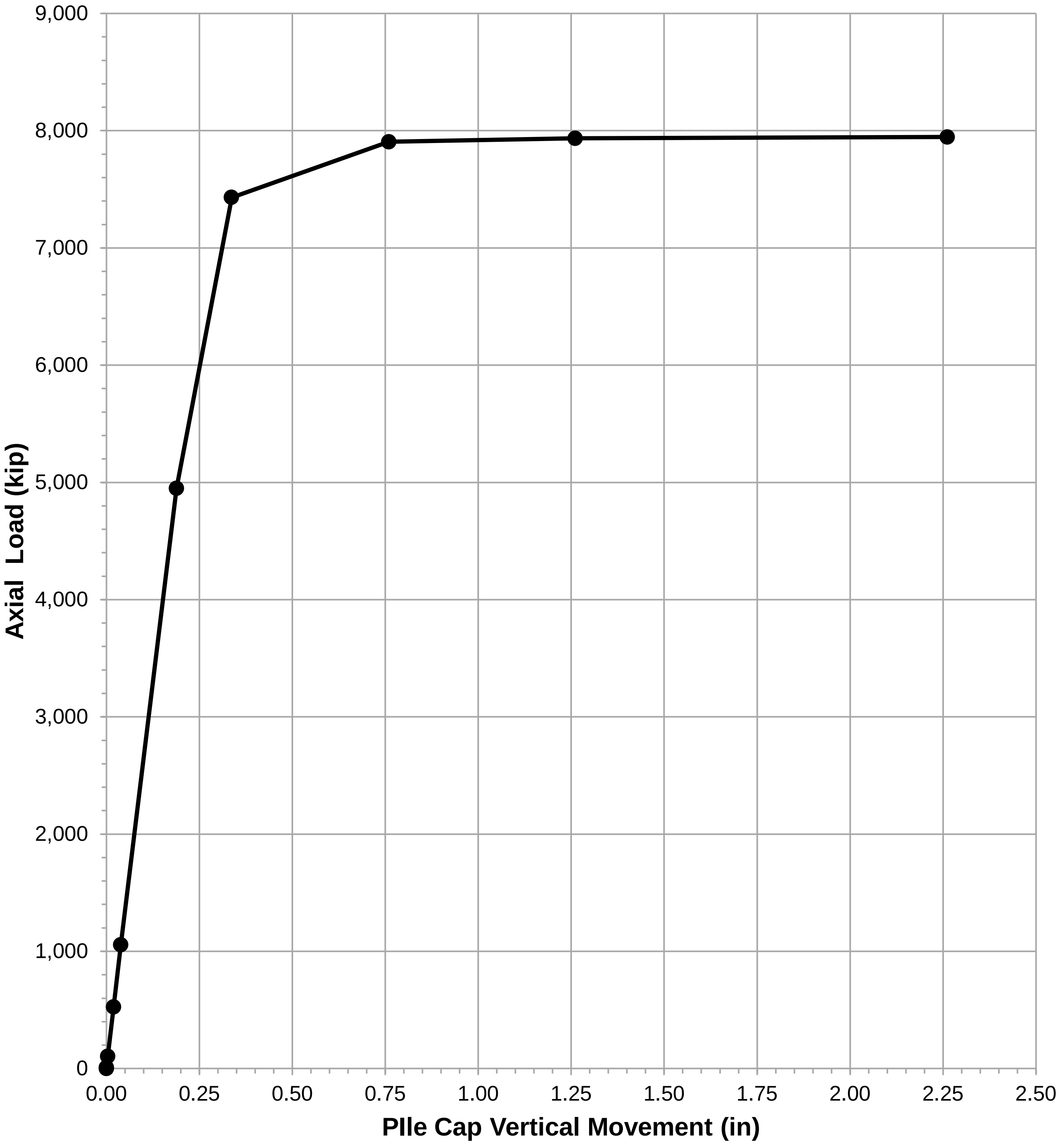
FIG. F20



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

**PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 25**

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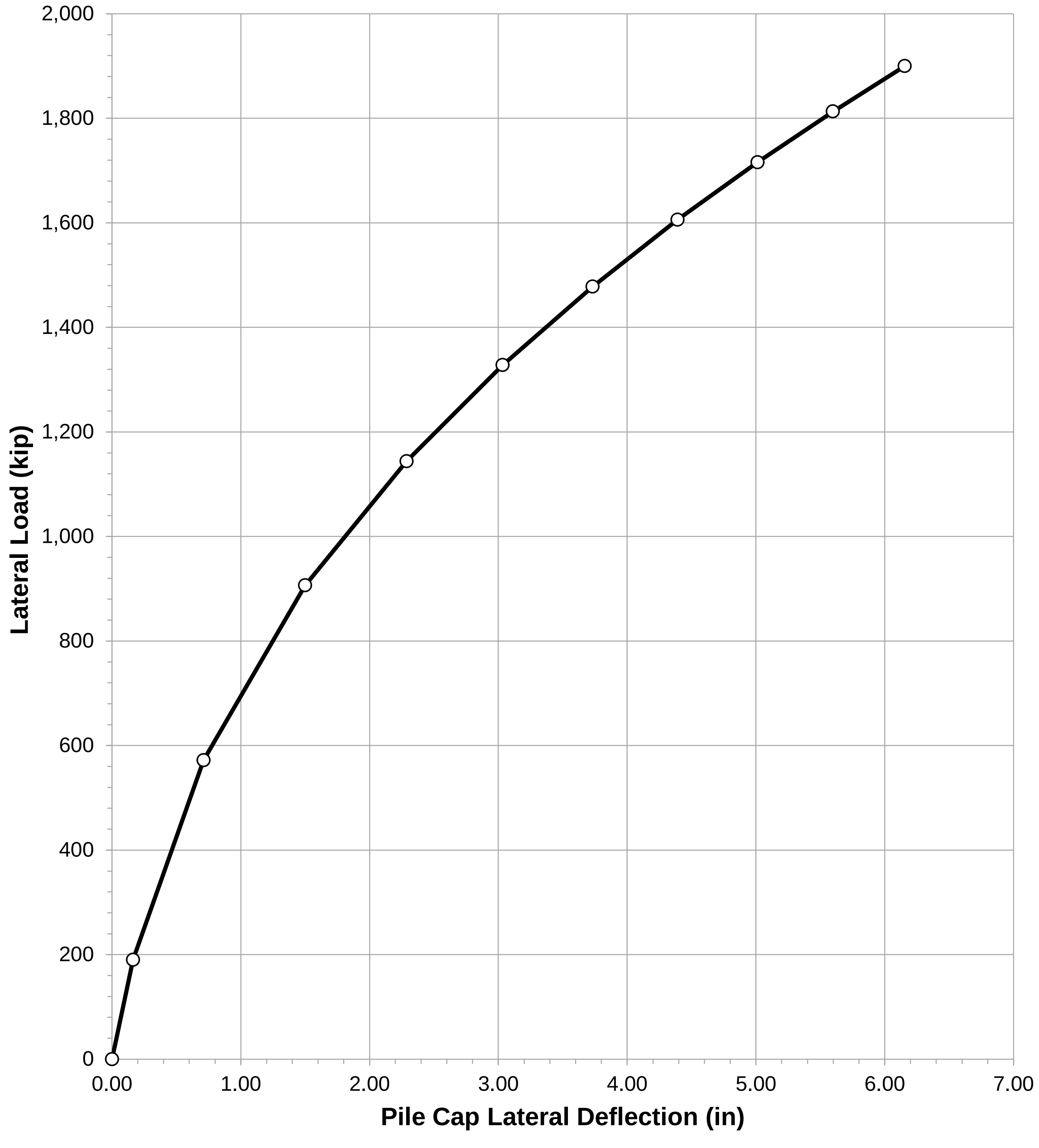
FIG. F21



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.
3. Recommended pile group lateral load-displacement curve can be used for both longitudinal and transverse loading directions.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 25

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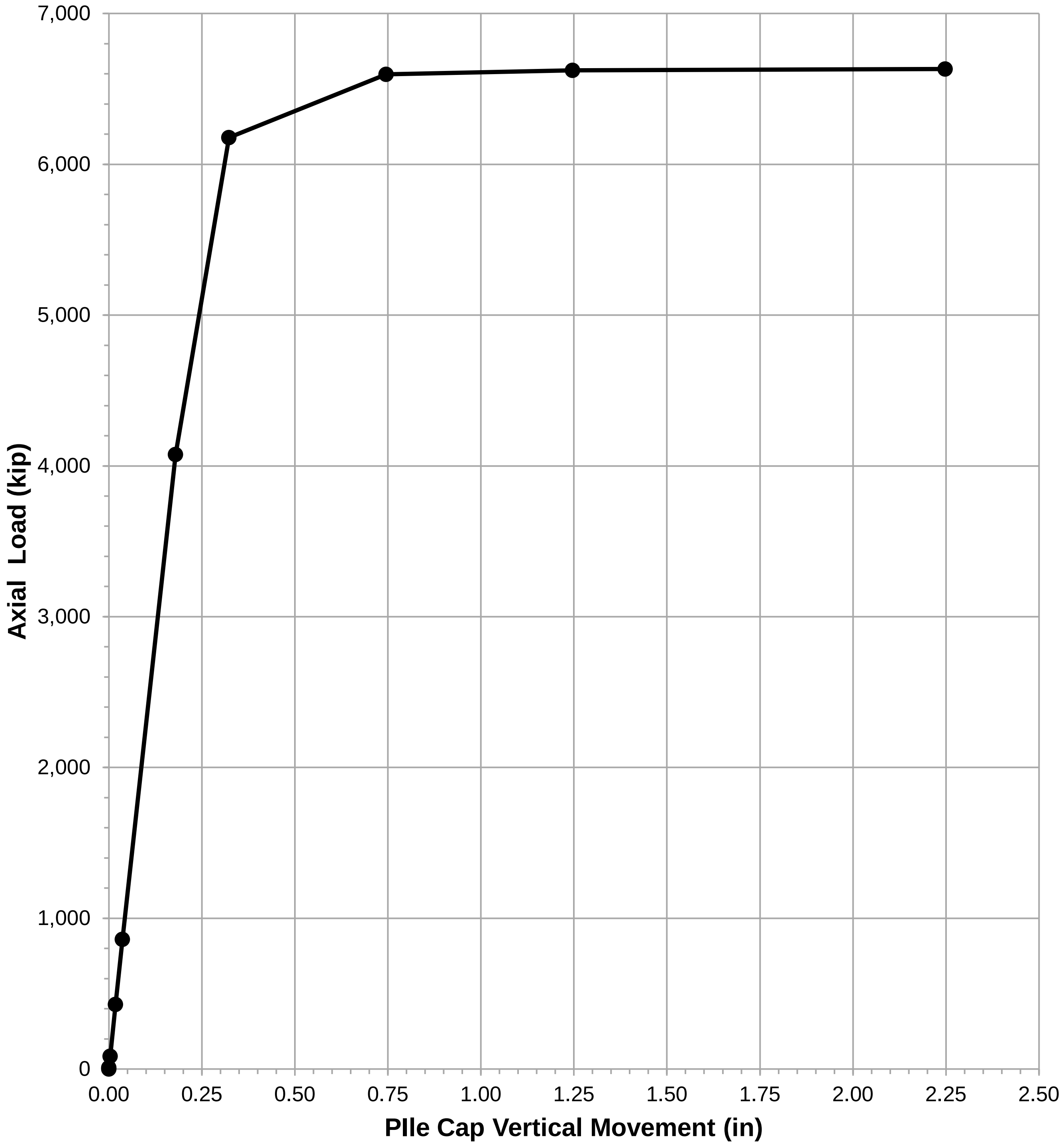
FIG. F22



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 26

May 2017 24-1-04065-004

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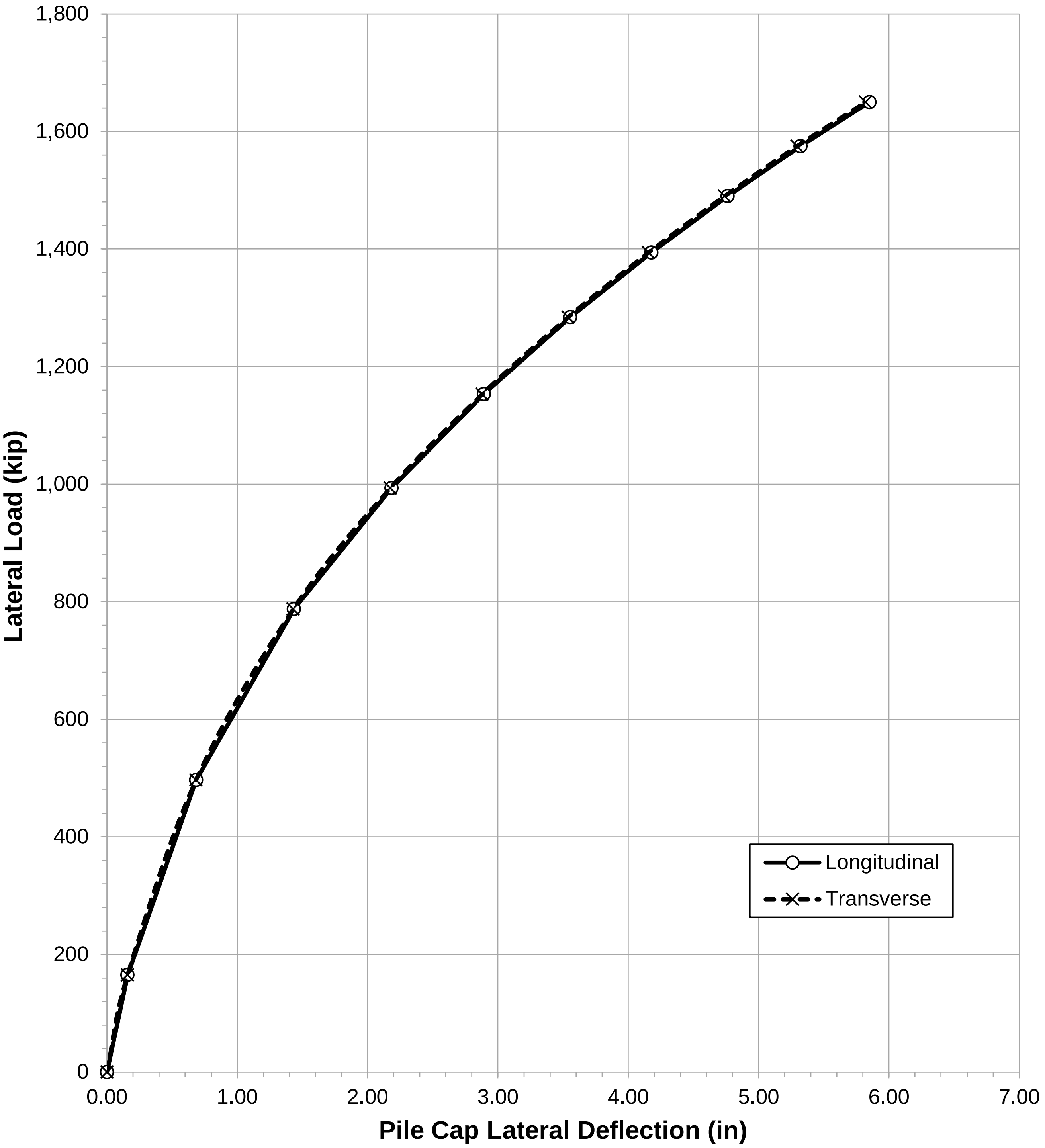
FIG. F23



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 26

May 2017
24-1-04065-005

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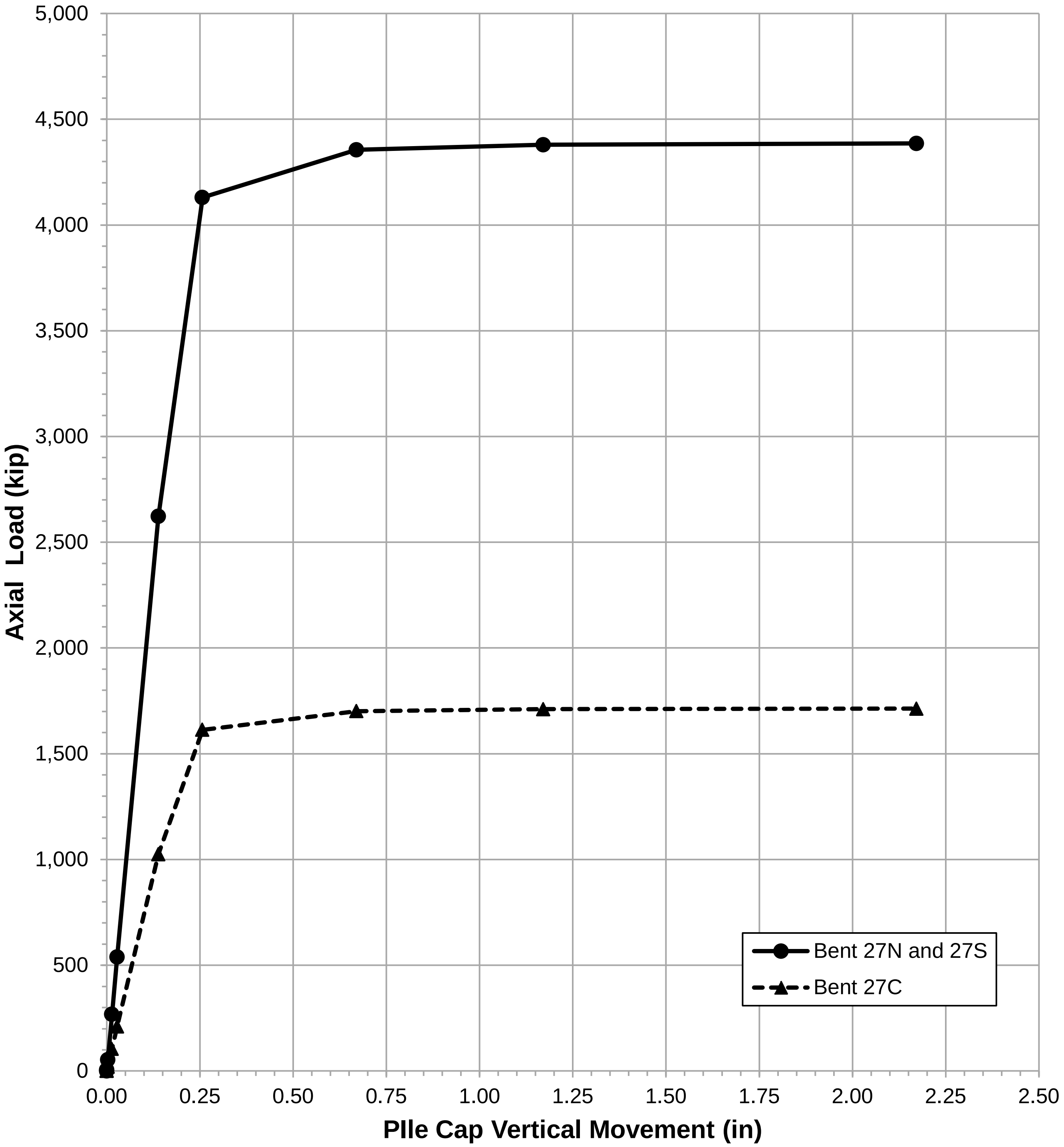
FIG. F24



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP AXIAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 27

May 2017 24-1-04065-004

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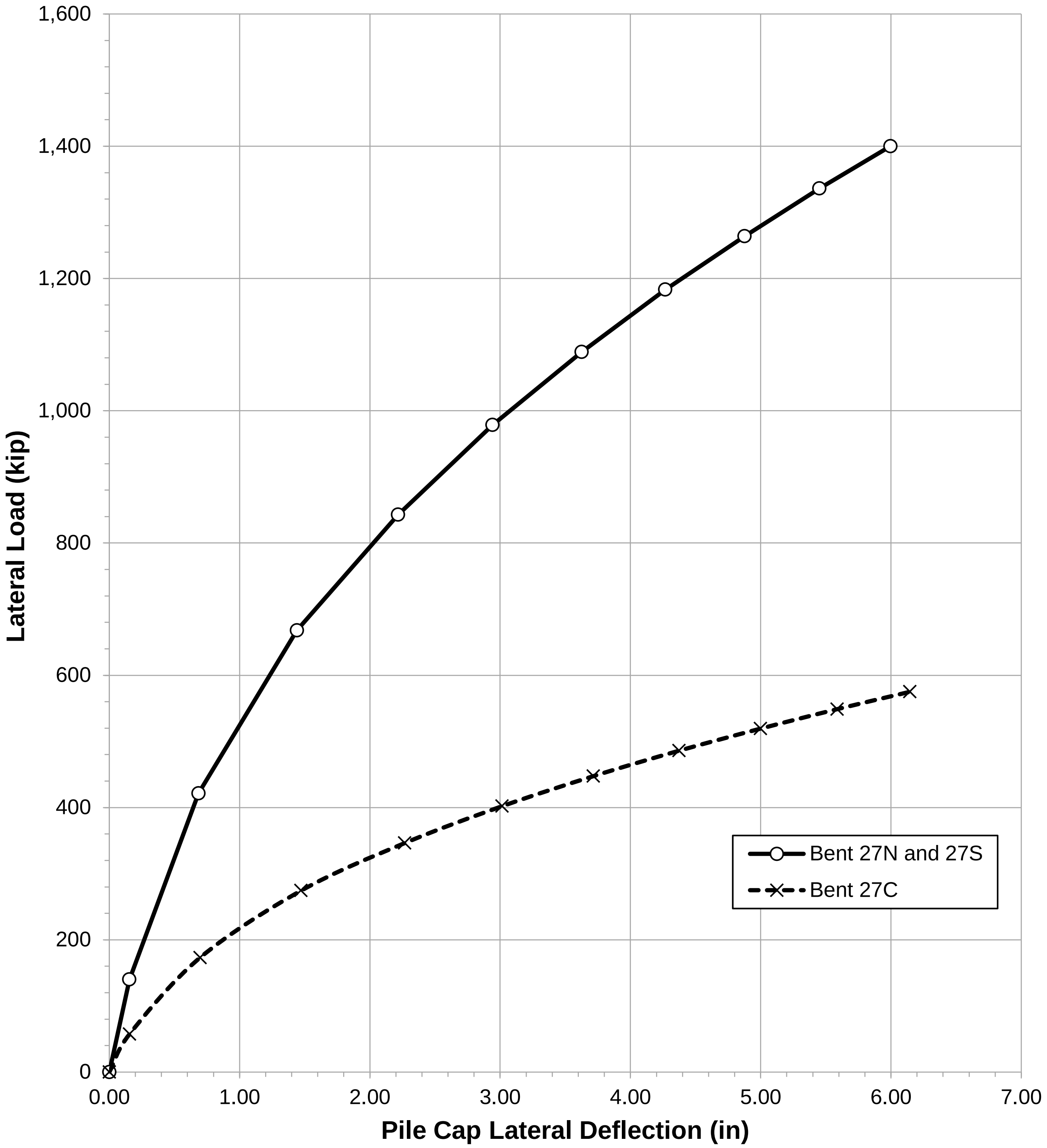
FIG. F25



NOTES

1. See Table 2 for details of each pile group evaluated.
2. Analysis assumes a fixed pile head condition and passive resistance from pile cap is not included. See Table 6 for recommended passive resistance on pile caps.
3. Recommended pile group lateral load-displacement curves can be used for both longitudinal and transverse loading directions.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP LATERAL
LOAD-DISPLACEMENT RELATIONSHIP
AT BENT 27

May 2017 24-1-04065-005

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FIG. F26

Appendix G

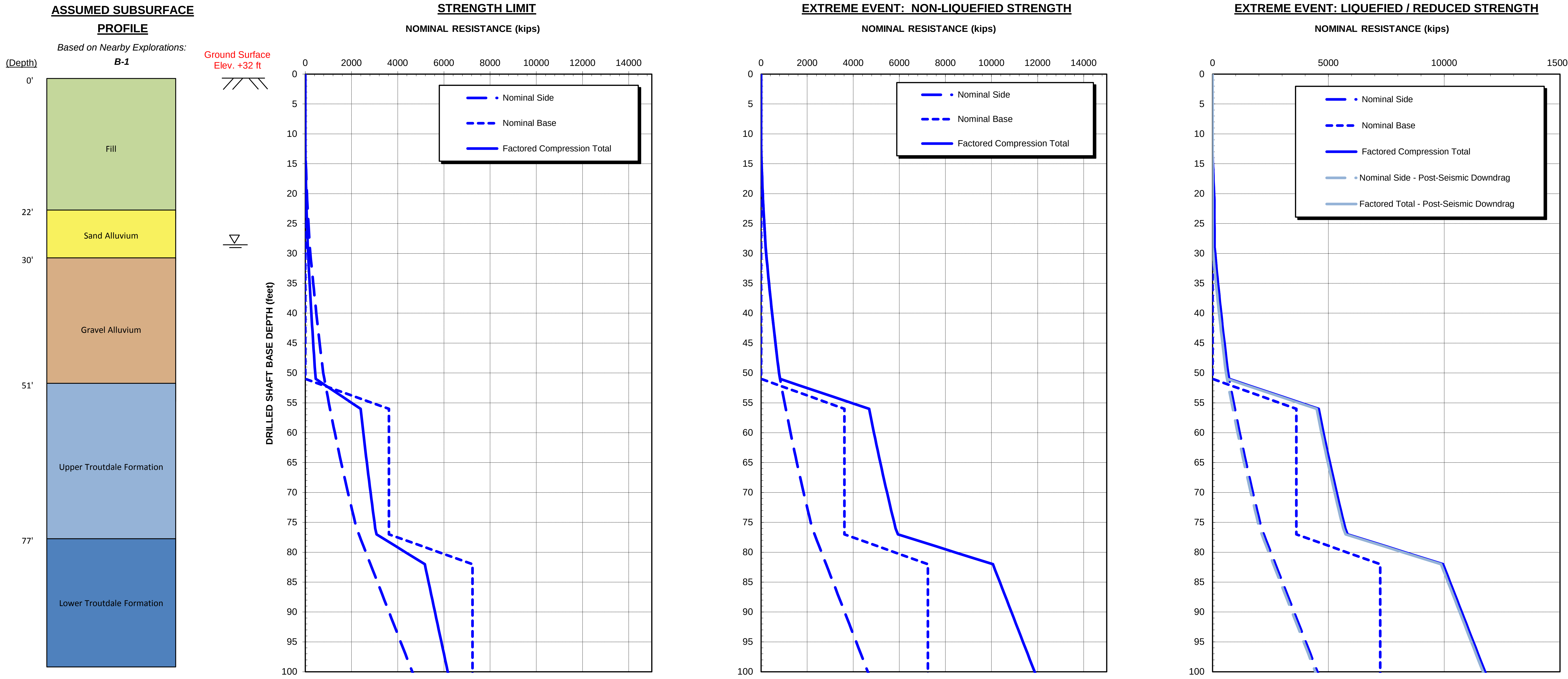
Drilled Shaft Parameters for Retrofit Alternative

Figures

Figures G1 through G9: Axial Resistance Curves for Bents 17 through 19 and Pier 4 through Bent 27

Figures G10 and G11: Summary of Soil Springs for Shafts at Piers 1 through 3

Tables G1 through G20: LPILE Parameters for Bents 17 through 27



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 130 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
BENT 17 RETROFIT
8-FOOT DIAMETER DRILLED SHAFT

August 2019 102636-001

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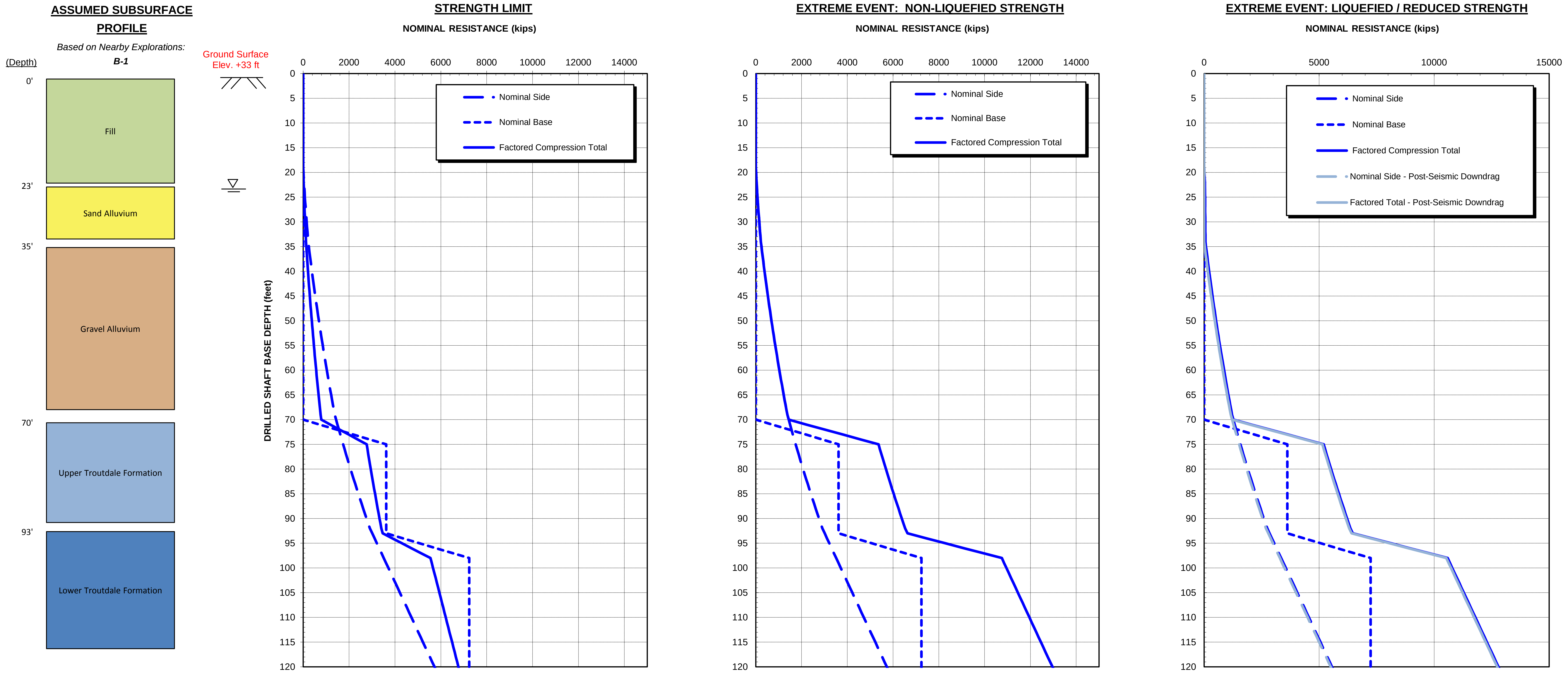
FIG. G1



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.

2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.

4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.

5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.

6. The estimated post-seismic downdrag force is 120 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 18 RETROFIT
8-FOOT DIAMETER DRILLED SHAFT

August 2019 102636-001

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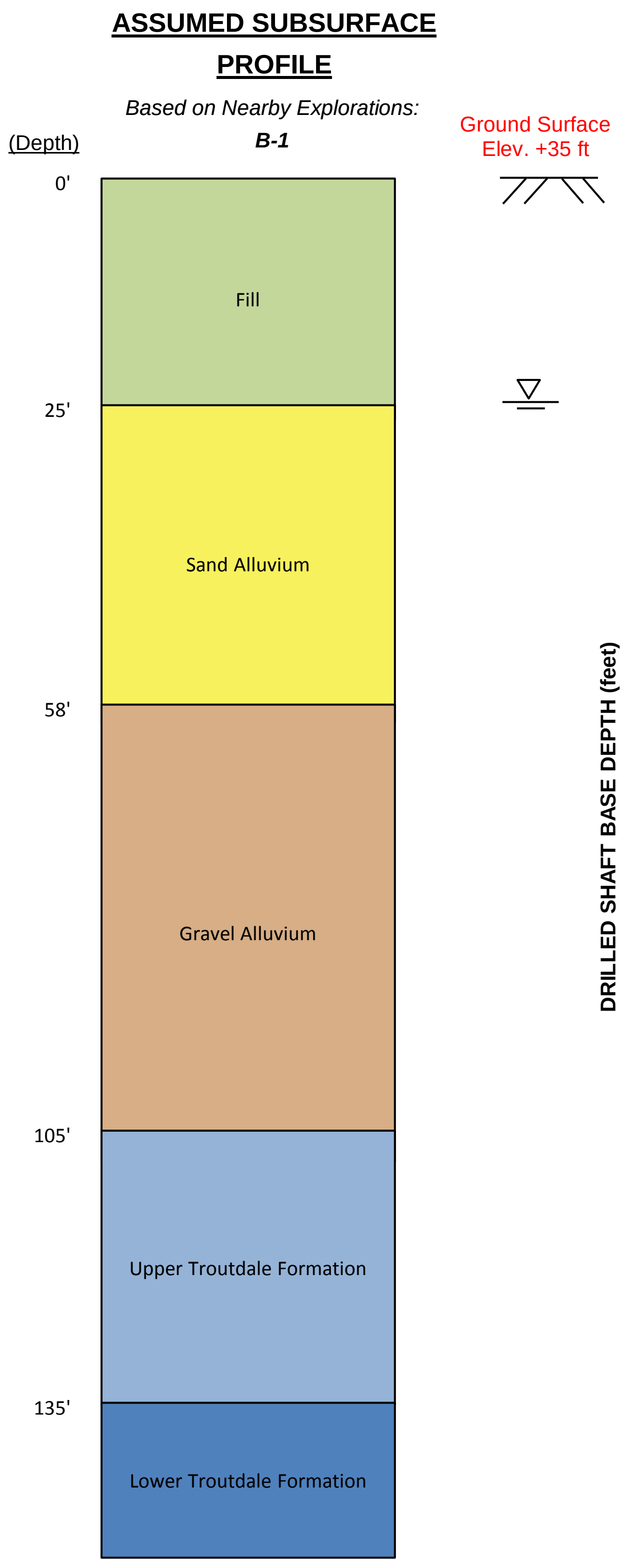
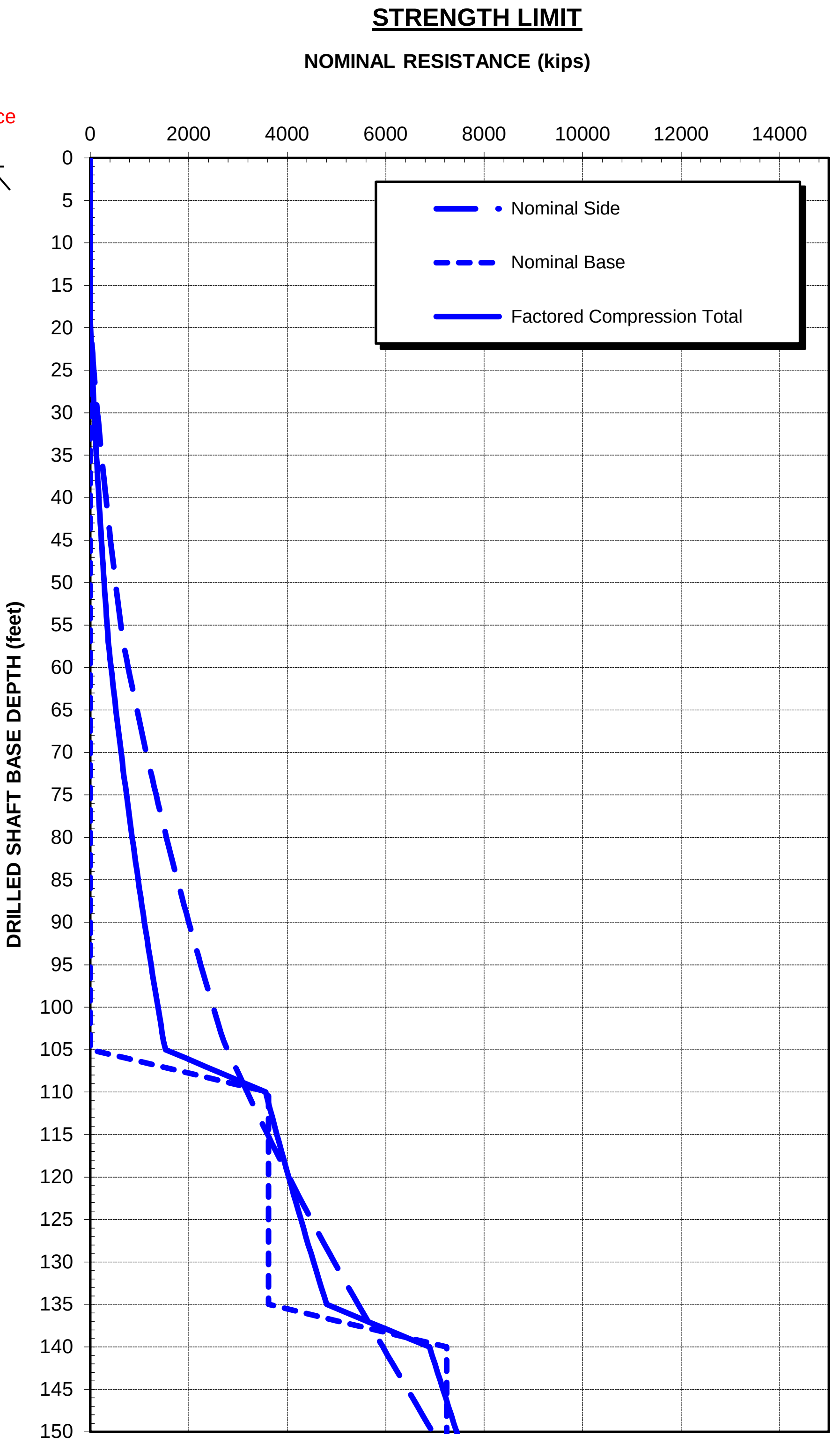
FIG. G2

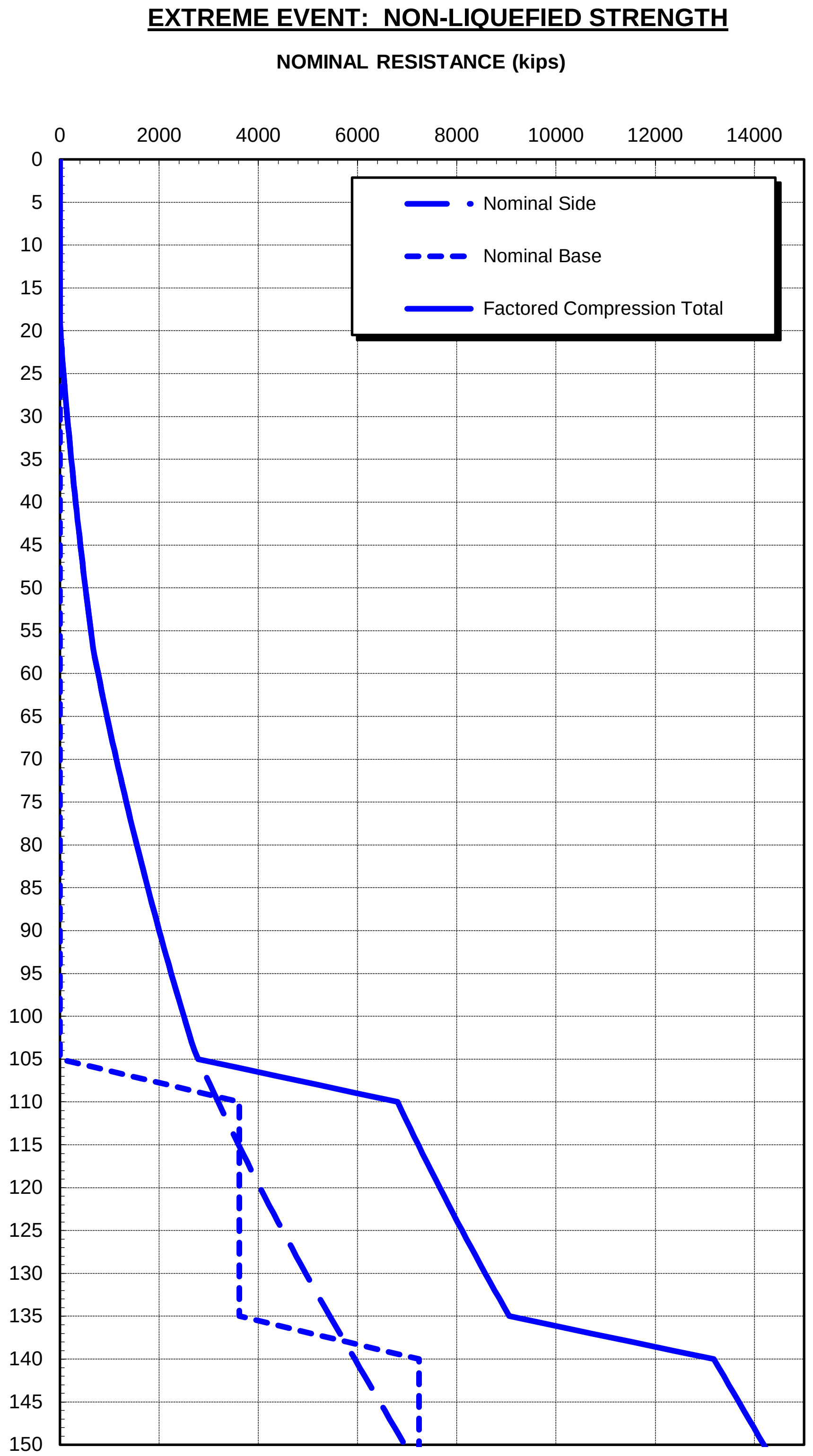


ASSUMED SUBSURFACE PROFILE

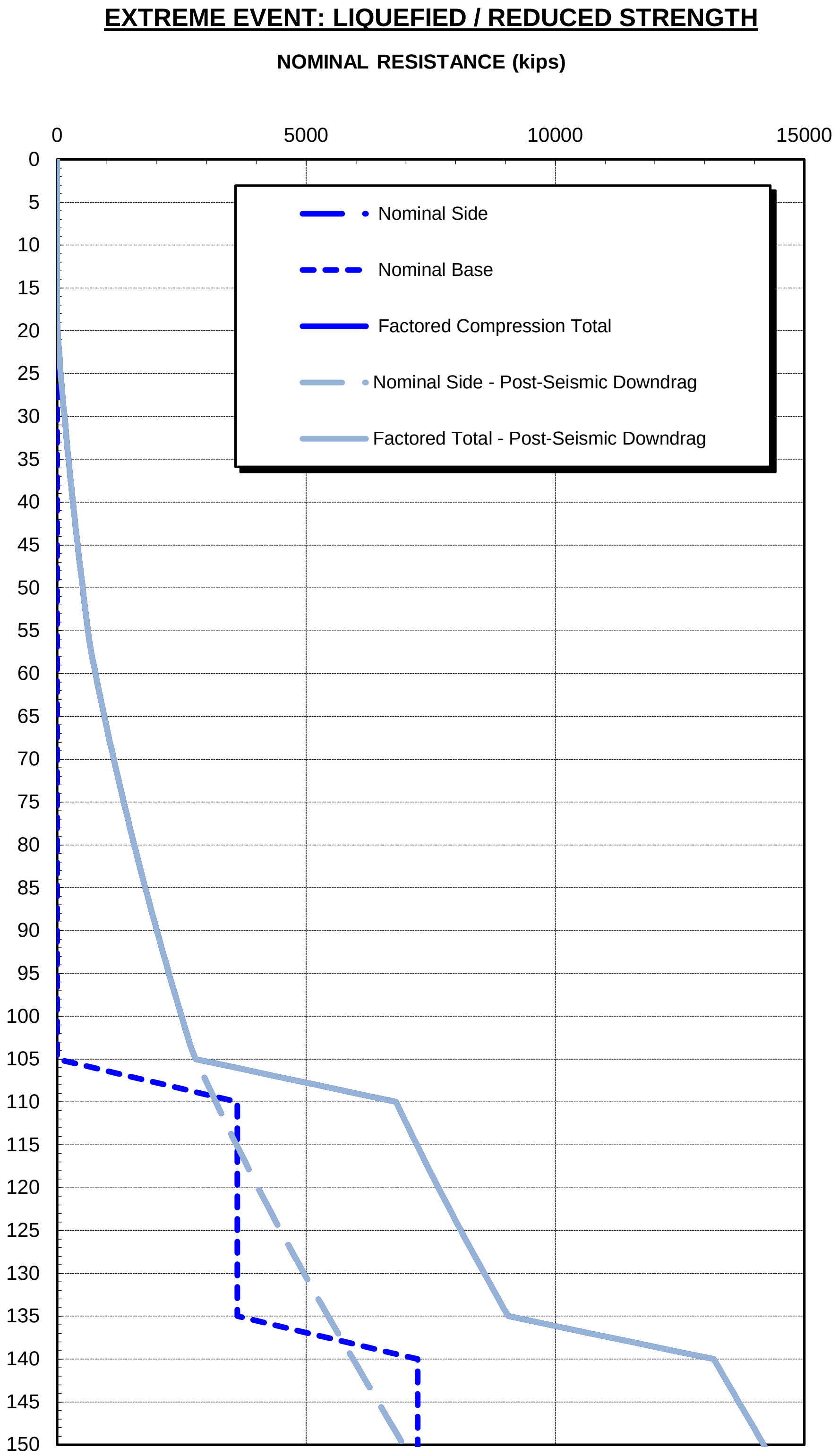
Based on Nearby Explorations:
B-1



STRENGTH LIMIT



EXTREME EVENT: NON-LIQUEFIED STRENGTH



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 19 RETROFIT
8-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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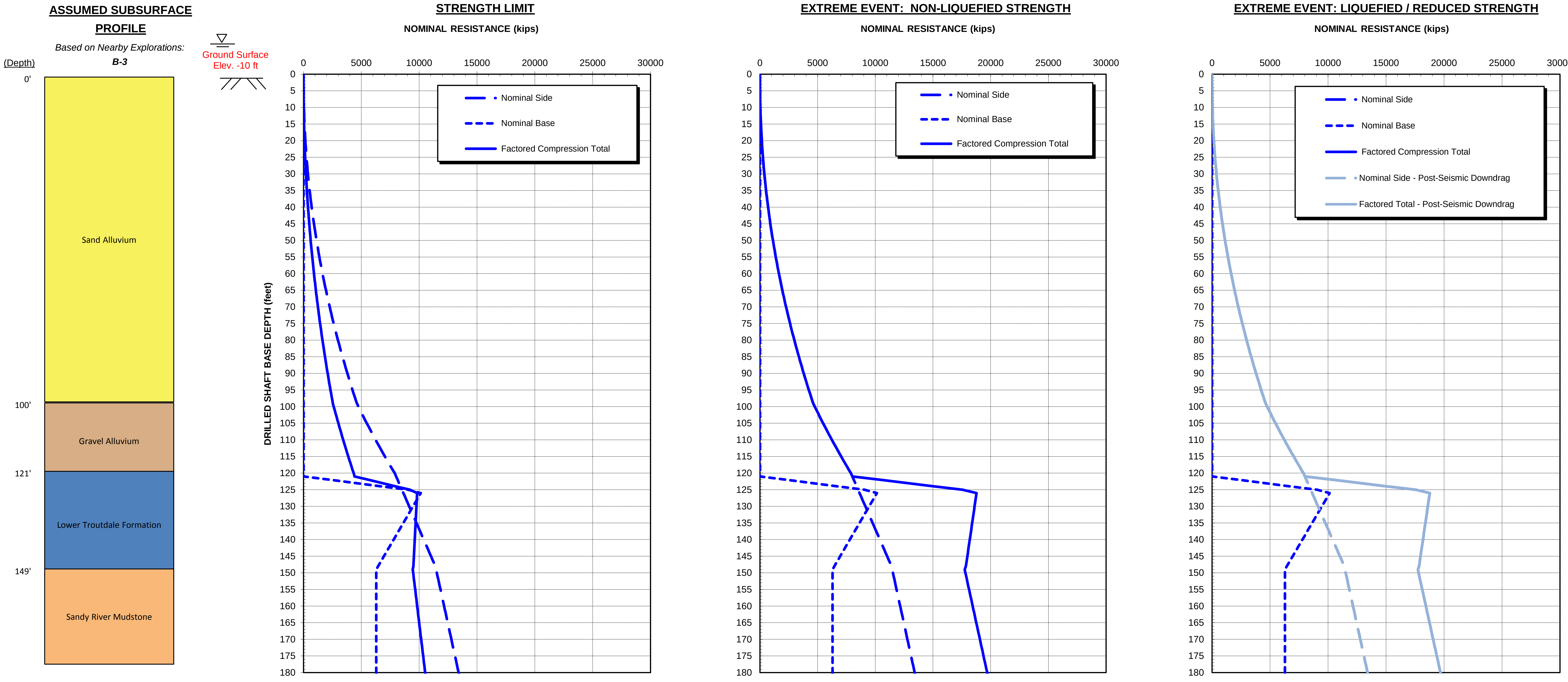
FIG. G3



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
PIER 4 RETROFIT
10-FOOT DIAMETER DRILLED SHAFT

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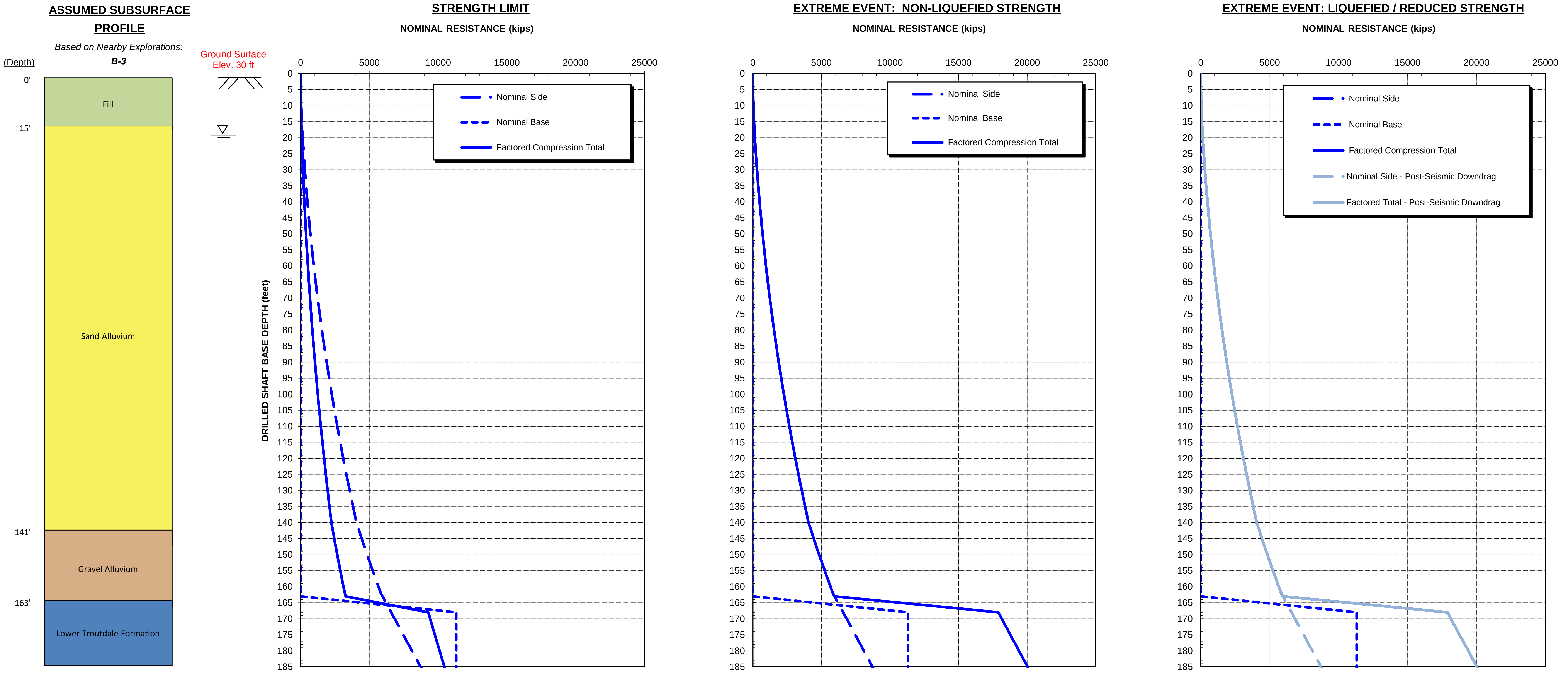
FIG. G4



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 23 RETROFIT
10-FOOT DIAMETER DRILLED SHAFT

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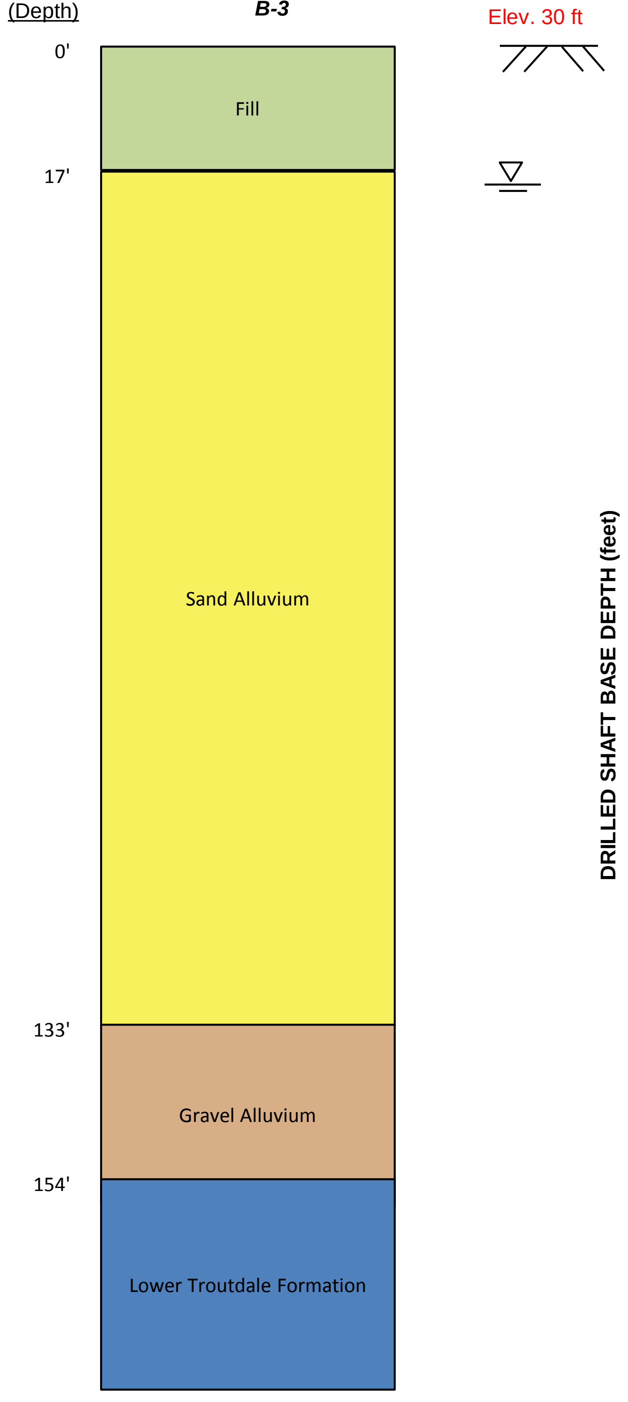
FIG. G5

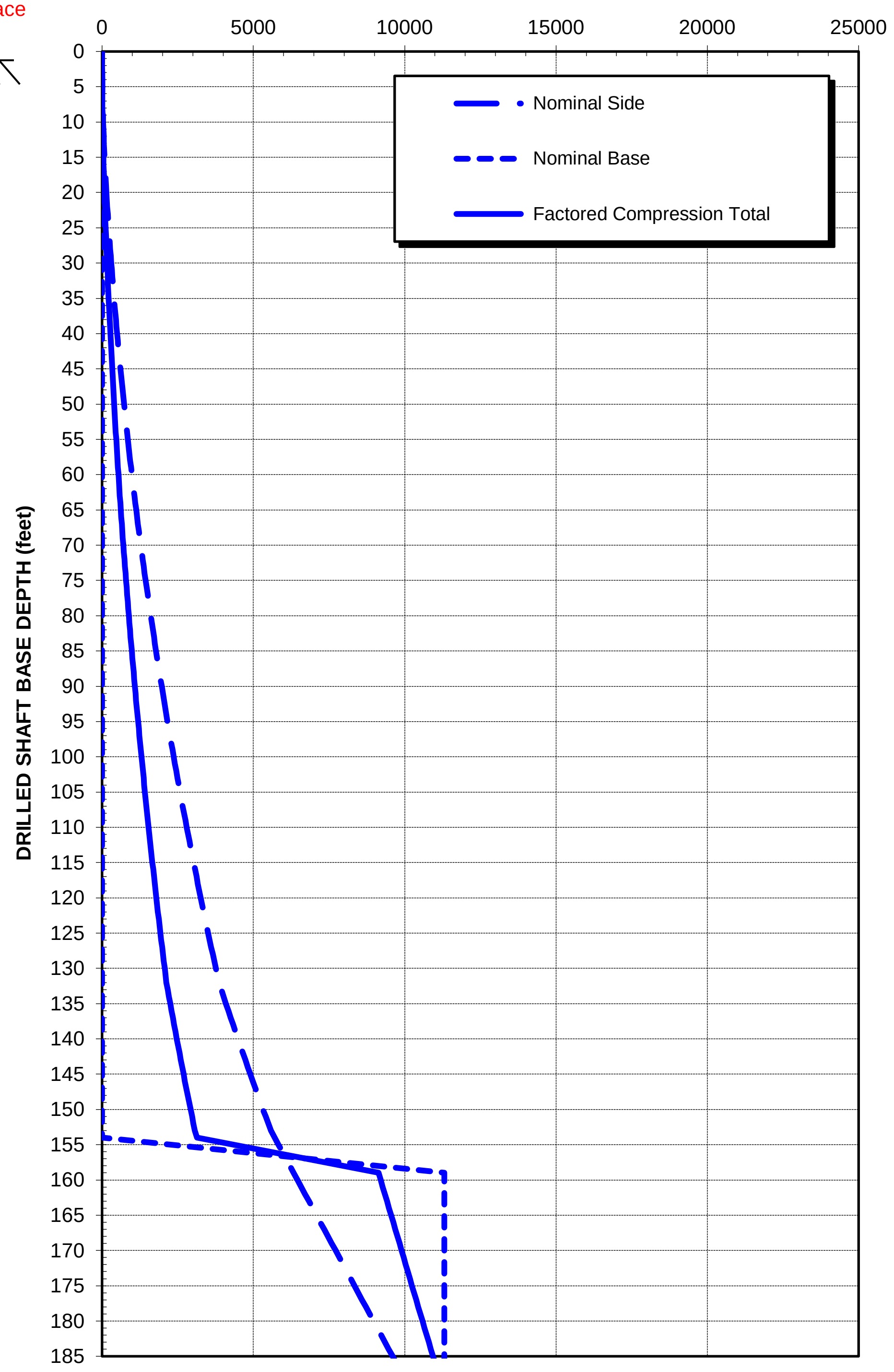
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-3



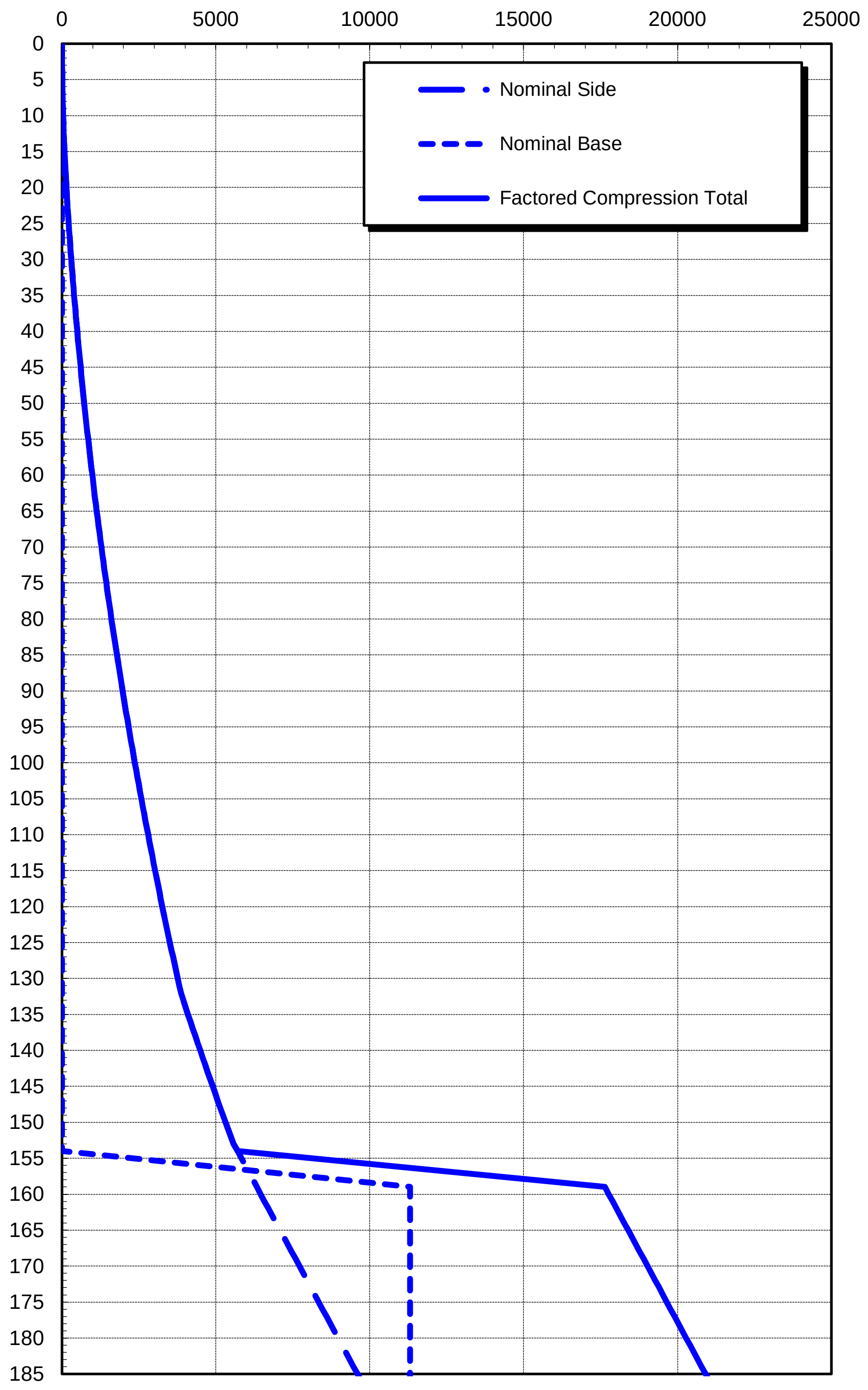
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



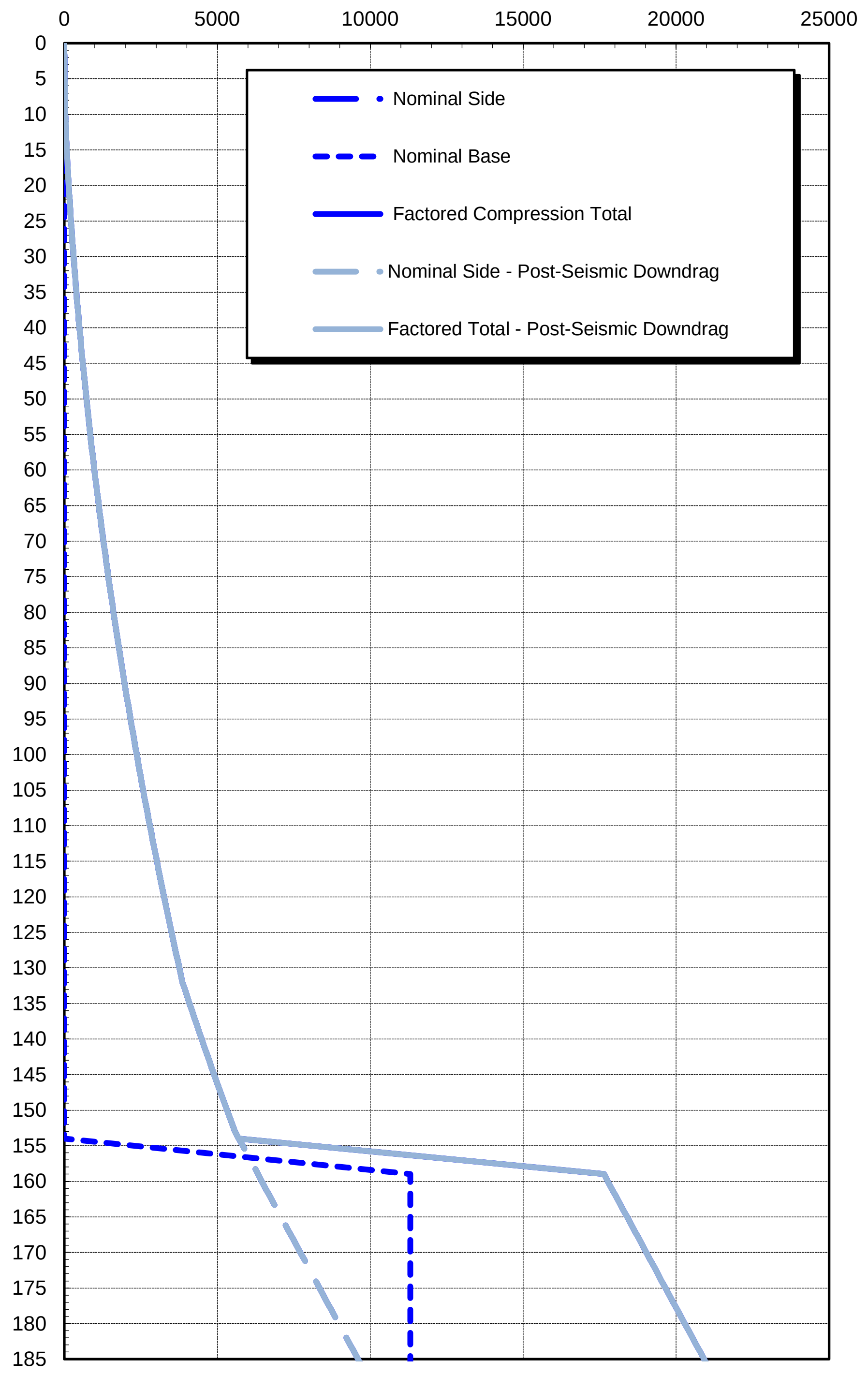
EXTREME EVENT: NON-LIQUEFIED STRENGTH

NOMINAL RESISTANCE (kips)



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOMINAL RESISTANCE (kips)



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 24 RETROFIT
10-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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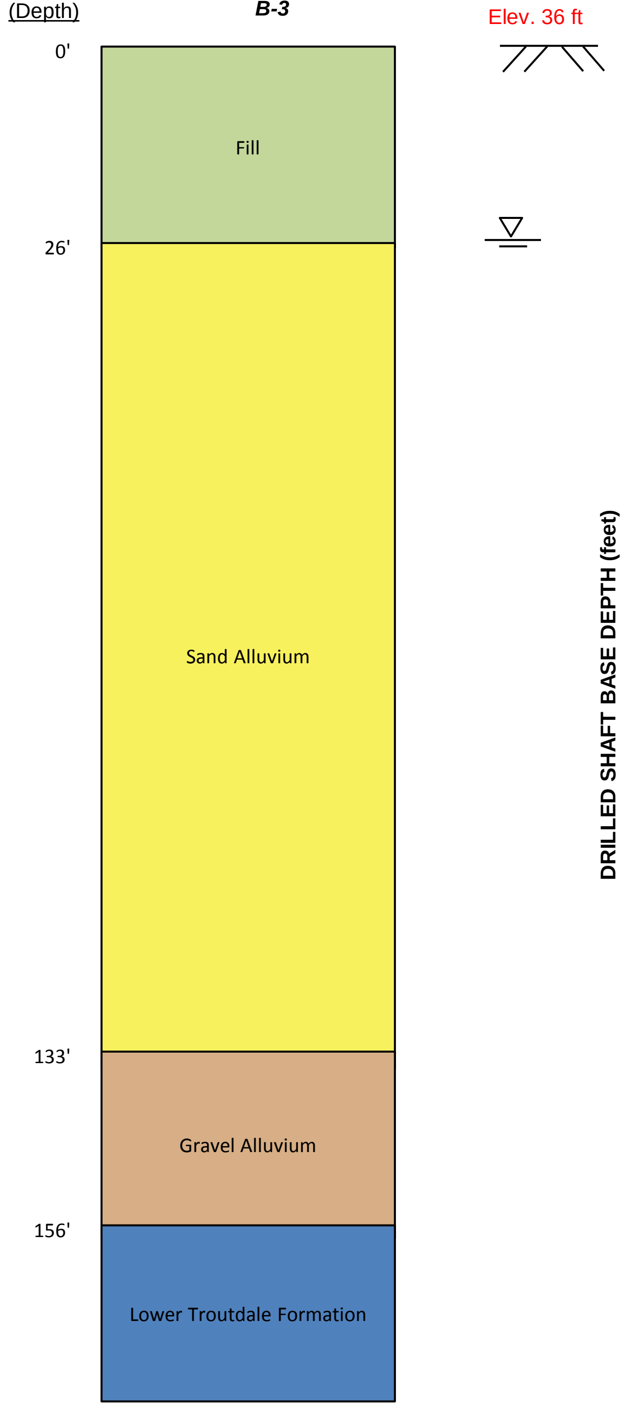
FIG. G6

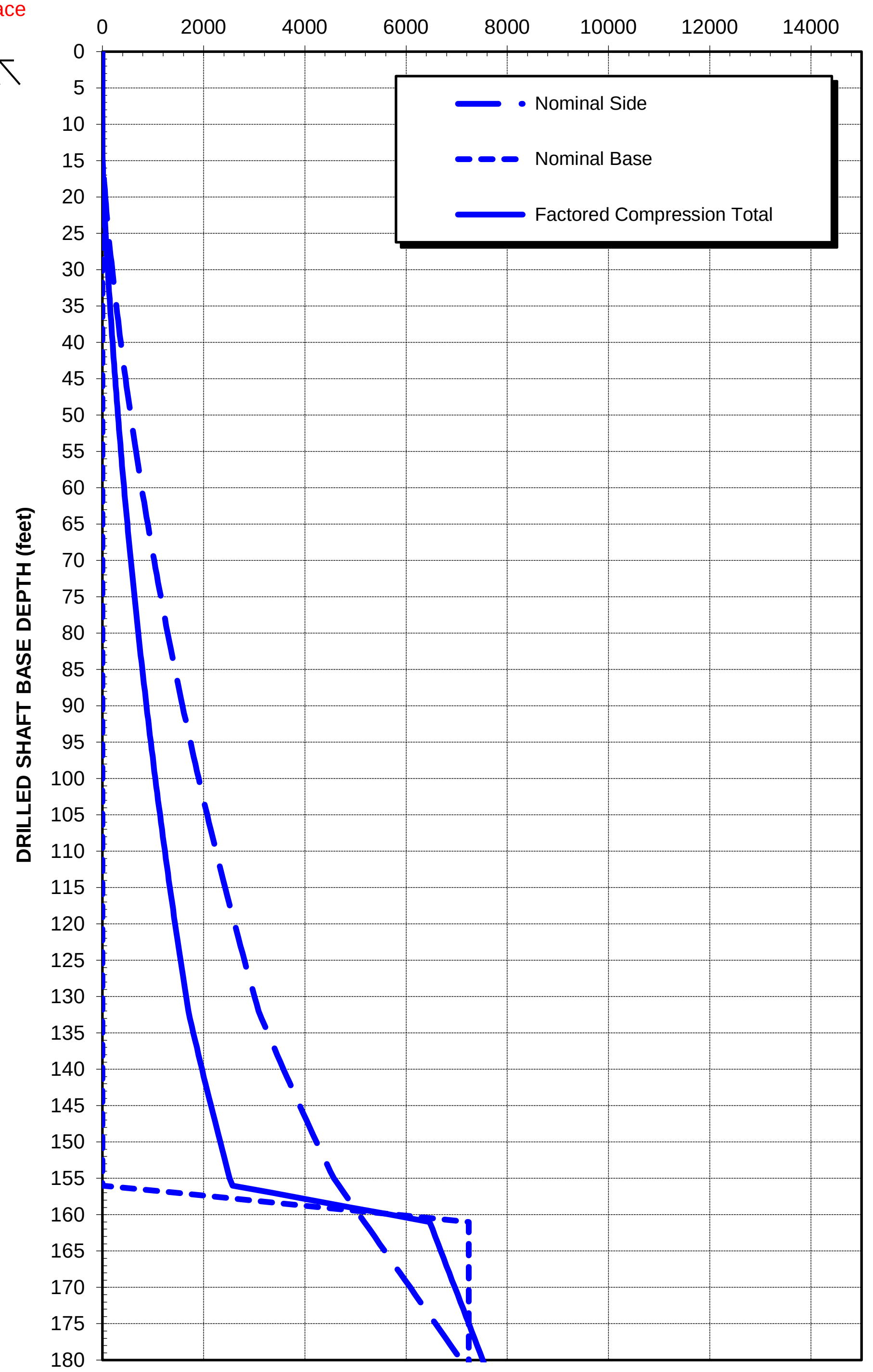
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:



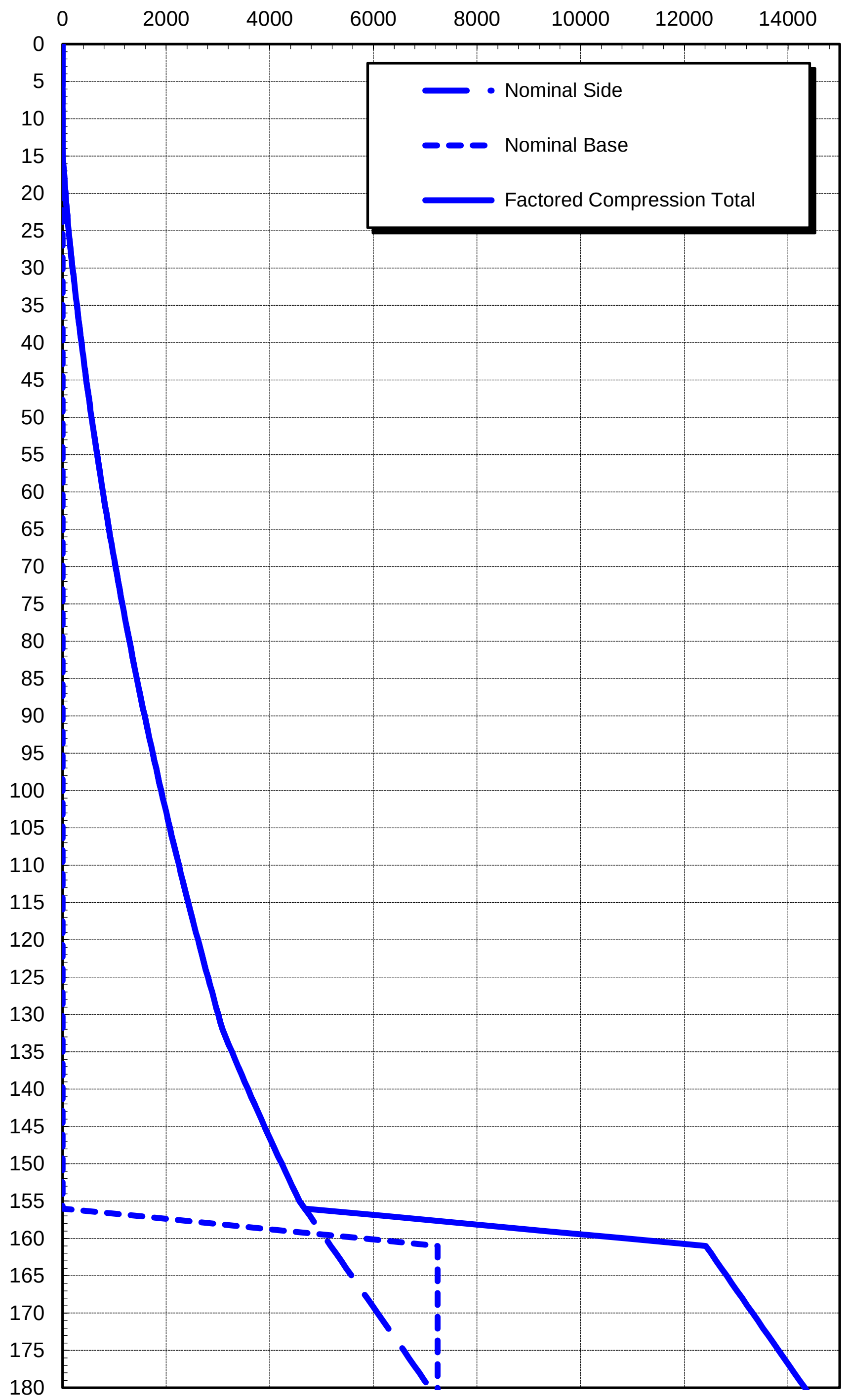
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



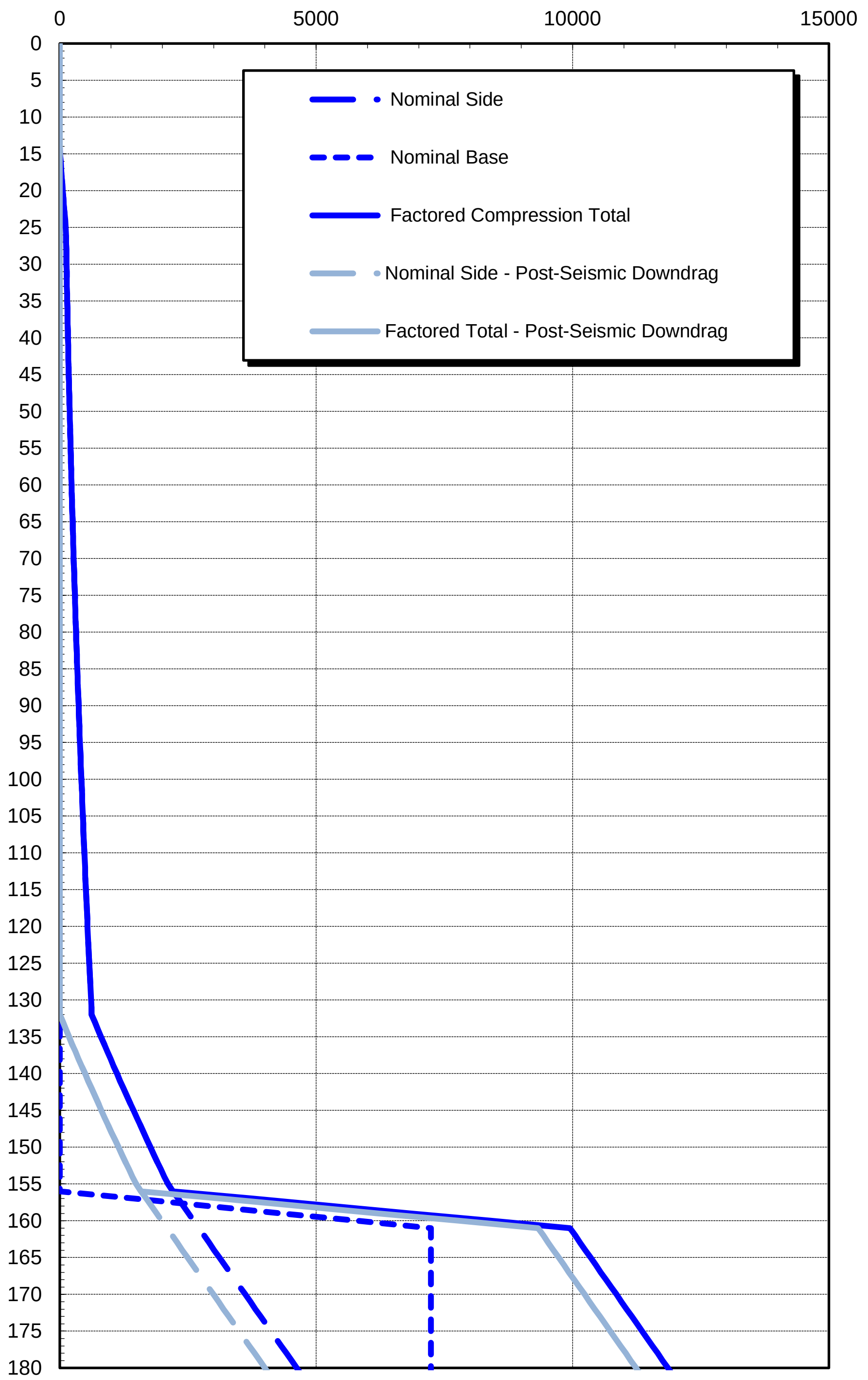
EXTREME EVENT: NON-LIQUEFIED STRENGTH

NOMINAL RESISTANCE (kips)



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOMINAL RESISTANCE (kips)



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 1590 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 25 RETROFIT
8-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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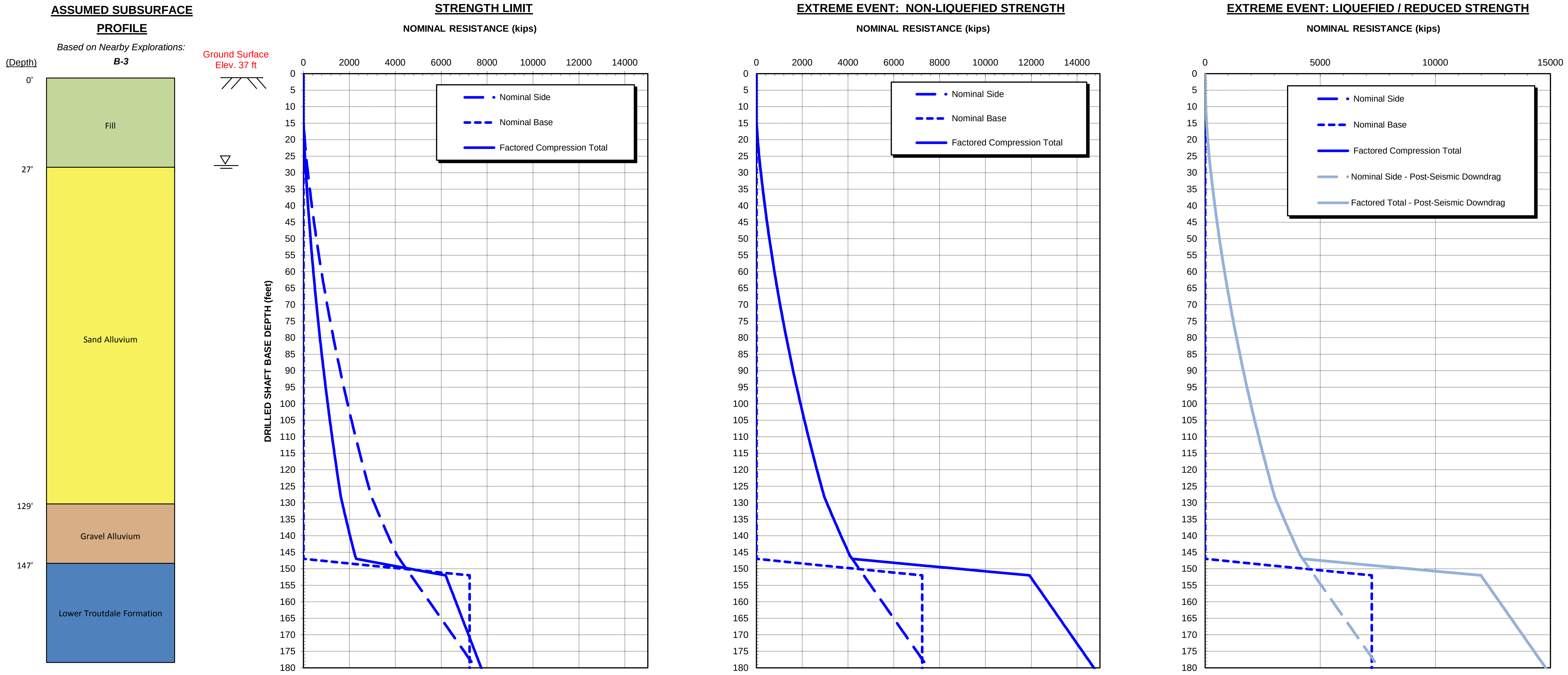
FIG. G7



ASSUMED SUBSURFACE PROFILE

STRENGTH LIMIT

EXTREME EVENT: NON-LIQUEFIED STRENGTH

EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 26 RETROFIT
8-FOOT DIAMETER DRILLED SHAFT

August 2019 102636-001

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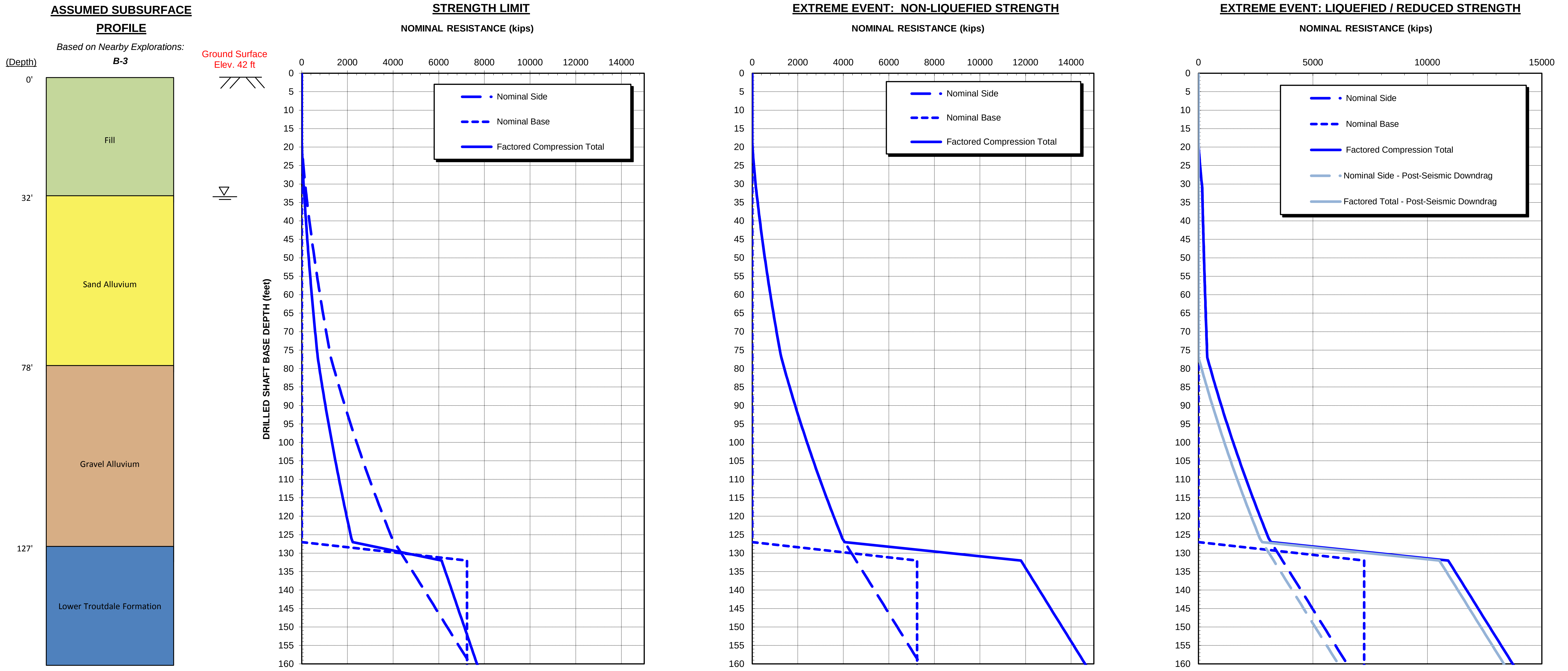
FIG. G8



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 710 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
BENT 27 RETROFIT
8-FOOT DIAMETER DRILLED SHAFT

August 2019 102636-001

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FIG. G9

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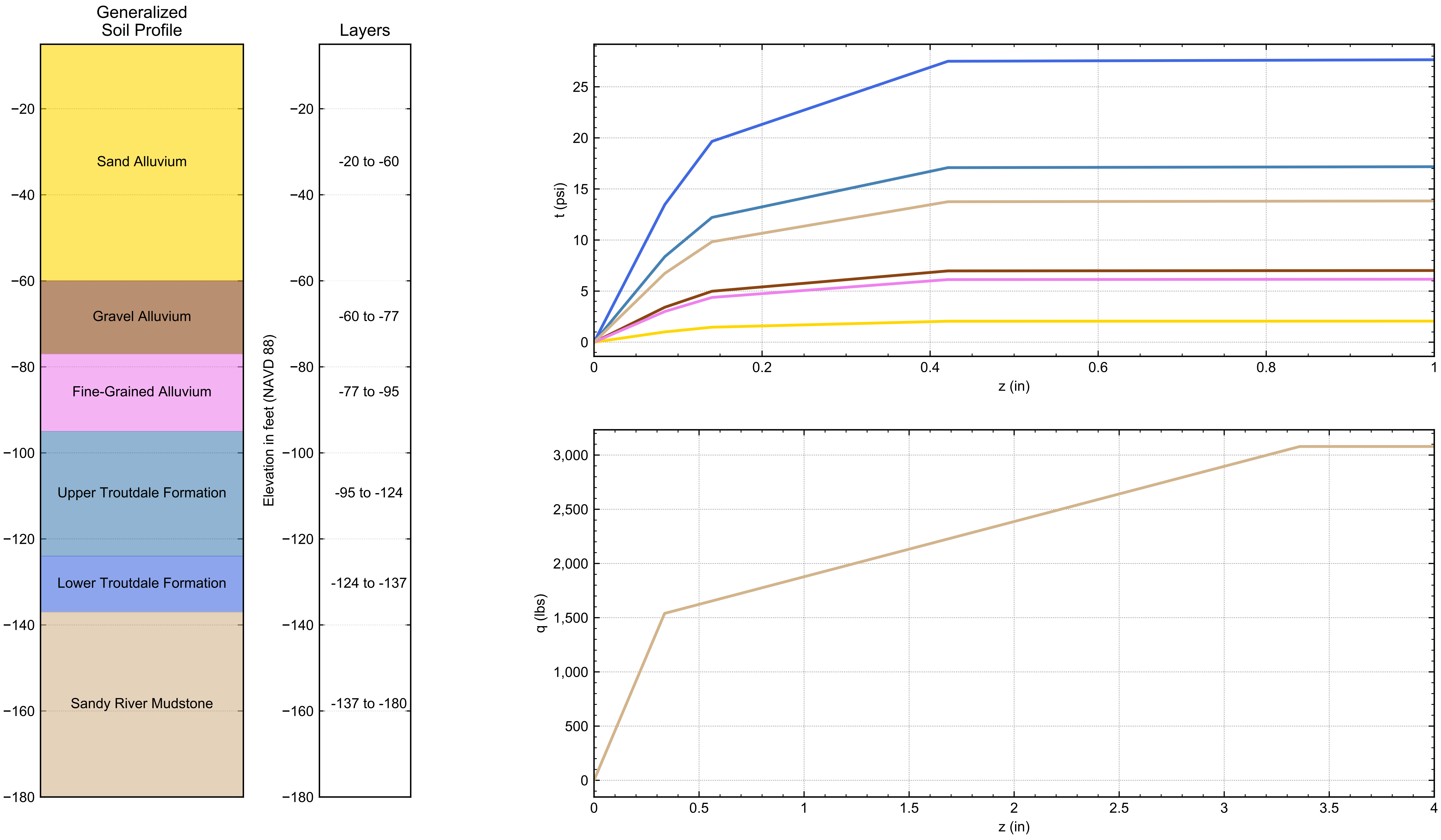

Notes:
1. Foundation type: 7-ft-diameter Drilled Shaft
2. Assigned end bearing at Node -137.0 to -180.0: 3 kips
3. Number of layers = 6
    Number of t-z files: 6
    Number of q-z files: 1
4. The provided q-z spring is based on nominal resistance values.
   Spring values should be reduced according to AASHTO (2017)
   Table 10.8.3.6.3-1 based on shaft center-to-center spacing.

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Burnside Bridge
Environmental Impact Study
Portland, Oregon

DRILLED SHAFT SPRINGS
EXTREME EVENT: LIQUEFIED CASE
PIER 1

January 2020 102636-001

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FIG. G10

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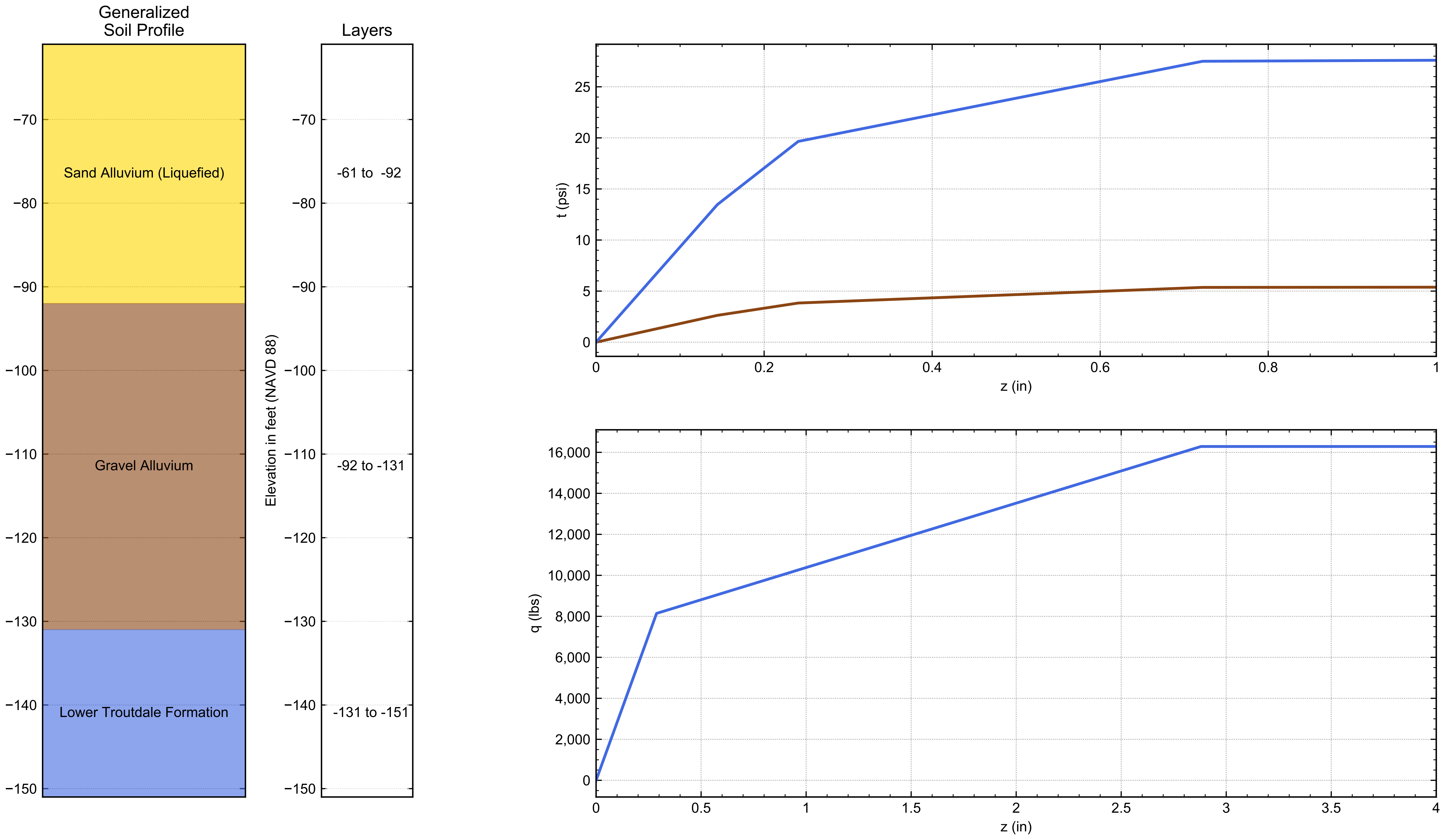

Notes:
1. Foundation type: 12-ft-diameter Drilled Shaft
2. Assigned end bearing at Node -131.0 to -151.0: 16 kips
3. Number of layers = 3
    Number of t-z files: 2
    Number of q-z files: 1
4. The provided q-z spring is based on nominal resistance values.
   Spring values should be reduced according to AASHTO (2017)
   Table 10.8.3.6.3-1 based on shaft center-to-center spacing.

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Burnside Bridge
Environmental Impact Study
Portland, Oregon

DRILLED SHAFT SPRINGS
EXTREME EVENT: LIQUEFIED CASE
PIERS 2 AND 3

January 2020 102636-001

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FIG. G11

Table G1 - L-Pile Parameters for Bent 17 Retrofit Profile

Elevation			Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To	Soil Type			Phi (deg)	k (pci)	Phi (deg)	k (pci)
32	10	Fill	Sand (Reese)	120	32	25	32	25
10	2	Sand Alluvium	Sand (Reese)	58	32	38	3	4
2	-19	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-19	-45	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-45	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G2 - Lateral Soil Displacement Profile at Bent 17 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G3 - L-Pile Parameters for Bent 18 Retrofit Profile

Elevation			Recommended p-y Curve	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
33	10	Fill	Sand (Reese)	120	32	25	32	25
10	-2	Sand Alluvium	Sand (Reese)	58	32	38	3	4
-2	-37	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-37	-60	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-60	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G4 - Lateral Soil Displacement Profile at Bent 18 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G5 - L-Pile Parameters for Bent 19 Retrofit Profile

Elevation			Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To	Soil Type			Phi (deg)	k (pci)	Phi (deg)	k (pci)
35	10	Fill	Sand (Reese)	120	32	25	32	25
10	-23	Sand Alluvium	Sand (Reese)	58	32	30	32	30
-23	-70	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-70	-100	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-100	-115	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G6 - Lateral Soil Displacement Profile at Bent 19

Depth (feet)	Displacement (inch)
0	0

Table G7 - L-Pile Parameters for Proposed Pier 1 Retrofit Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
-5	-60	Sand Alluvium	Sand (Reese)	58	32	38	32	38
-60	-77	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-77	-95	Fine-Grained Alluvium	Sand (Reese)	58	32	83	32	83
-95	-124	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-124	-137	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225
-137	-180	Sandy River Mudstone	Sand (Reese)	78	44	125	44	125

NOTES:

1 P-y springs generated in FB-Multipier should be reduced according to AASHTO (2017) Table 10.7.2.4-1 based on shaft center-to-center spacing where applicable.

Table G8 - Lateral Soil Displacement Profile at Pier 1 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G9 - L-Pile Parameters for Piers 2 and 3 Retrofit

Elevation		Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To				Phi (deg)	k (pci)	Phi (deg)	k (pci)
-61	-92	Sand Alluvium	Sand (Reese)	53	30	38	2	2
-92	-131	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-131	-160	Lower Troutdale Formation	Sand (Reese)	78	44	125	44	125

NOTES:

1 P-y springs generated in FB-Multipier should be reduced according to AASHTO (2017) Table 10.7.2.4-1 based on shaft center-to-center spacing where applicable.

Table G10 - L-Pile Parameters for Pier 4 Retrofit Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
-10	-110	Sand Alluvium	Sand (Reese)	53	30	38	30	38
-110	-129	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-129	-161	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225
-161	-180	Sandy River Mudstone	Sand (Reese)	78	44	125	44	125

Table G11 - Lateral Soil Displacement Profile at Pier 4 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G12 - L-Pile Parameters for Bent 23 Retrofit Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	15	Fill	Sand (Reese)	120	32	25	32	25
15	10	Sand Alluvium	Sand (Reese)	115	30	40	30	40
10	-111	Sand Alluvium; below groundwater table	Sand (Reese)	53	30	30	30	30
-111	-133	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-133	-150	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G13 - Lateral Soil Displacement Profile at Bent 23 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G14 - L-Pile Parameters for Bent 24 Retrofit Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	13	Fill	Sand (Reese)	120	32	25	32	25
13	10	Sand Alluvium	Sand (Reese)	115	30	40	30	40
10	-103	Sand Alluvium; below groundwater table	Sand (Reese)	53	30	30	30	30
-103	-124	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-124	-150	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G15 - Lateral Soil Displacement Profile at Bent 24 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G16 - L-Pile Parameters for Bent 25 Retrofit Profile

Elevation			Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To	Soil Type			Phi (deg)	k (pci)	Phi (deg)	k (pci)
36	10	Fill	Sand (Reese)	120	32	25	32	25
10	-97	Sand Alluvium	Sand (Reese)	53	30	30	5	9
-97	-120	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-120	-160	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G17 - Lateral Soil Displacement Profile at Bent 25 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G18 - L-Pile Parameters for Bent 26 Retrofit Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
37	10	Fill	Sand (Reese)	120	32	25	32	25
10	-88	Sand Alluvium	Sand (Reese)	53	30	30	30	30
-92	-110	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-110	-160	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G19 - Lateral Soil Displacement Profile at Bent 26 Retrofit

Depth (feet)	Displacement (inch)
0	0

Table G20 - L-Pile Parameters for Bent 27 Retrofit Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
42	10	Fill	Sand (Reese)	120	32	25	32	25
10	-36	Sand Alluvium	Sand (Reese)	53	30	30	5	9
-36	-85	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-85	-130	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table G21 - Lateral Soil Displacement Profile at Bent 27 Retrofit

Depth (feet)	Displacement (inch)
0	0

Appendix H

Drilled Shaft Parameters for Short-span Alternative & Couch Extension

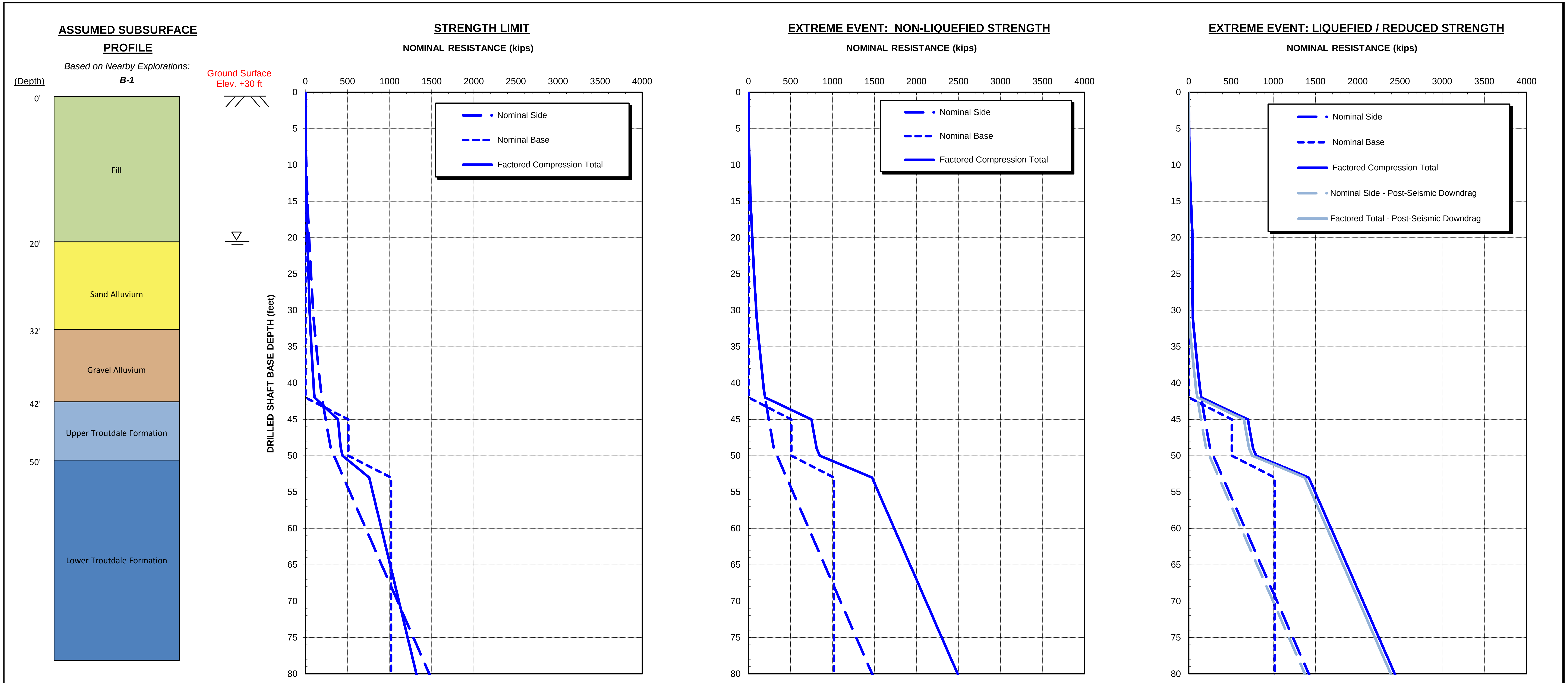
Figures

Figures H1 through H11: Axial Resistance Curves for Bents 1 through 6 and Bents 9 through 14/S14

Figures H12 through H17: Axial Resistance Curves for Bents N10 through N15

Figure H18: Summary of Soil Springs for Bents 7 and 8

Tables H1 through H21: LPILE Parameters for Bents 1 through 14/S14 and N10 through N15



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 70 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 1
3-FOOT DIAMETER DRILLED SHAFT

August 2019 102636-001

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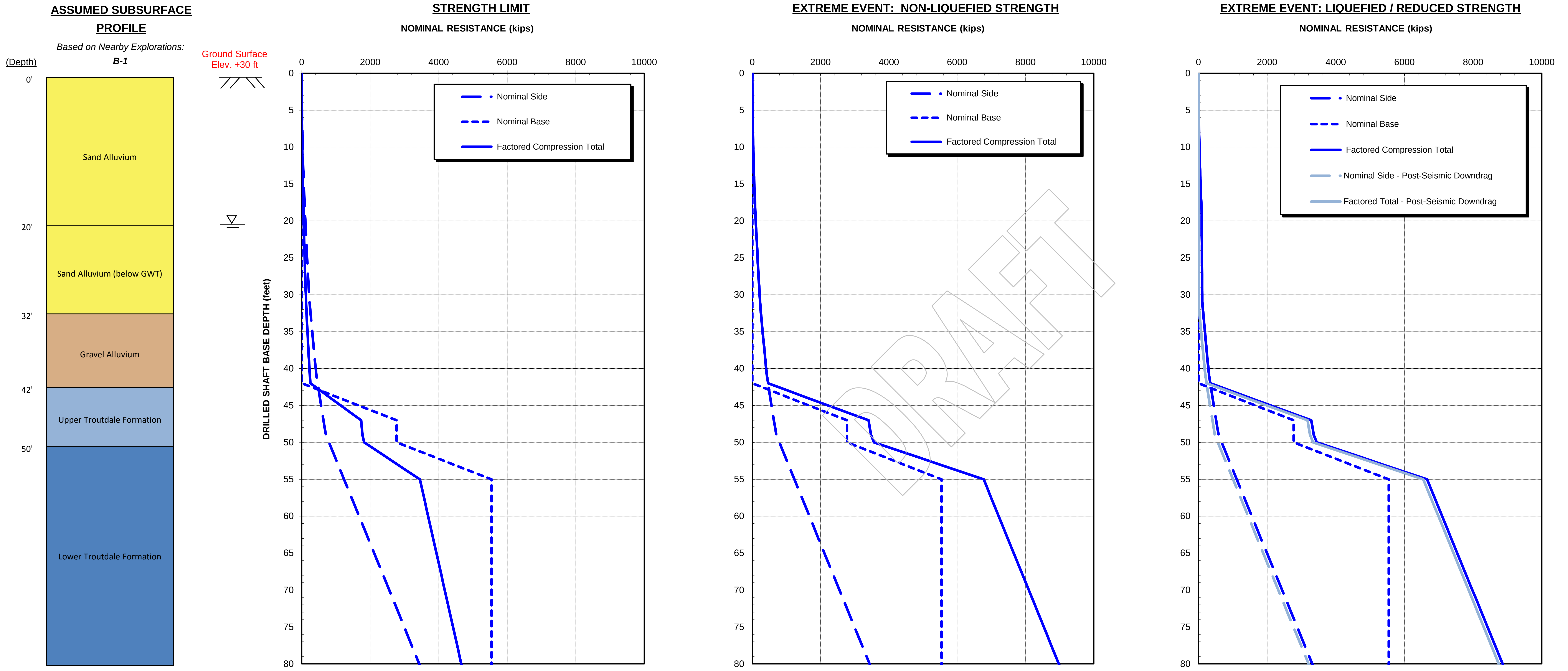
FIG. H1



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.

2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**

4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.

5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.

6. The estimated post-seismic downdrag force is 160 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENTS 2 AND 3
7-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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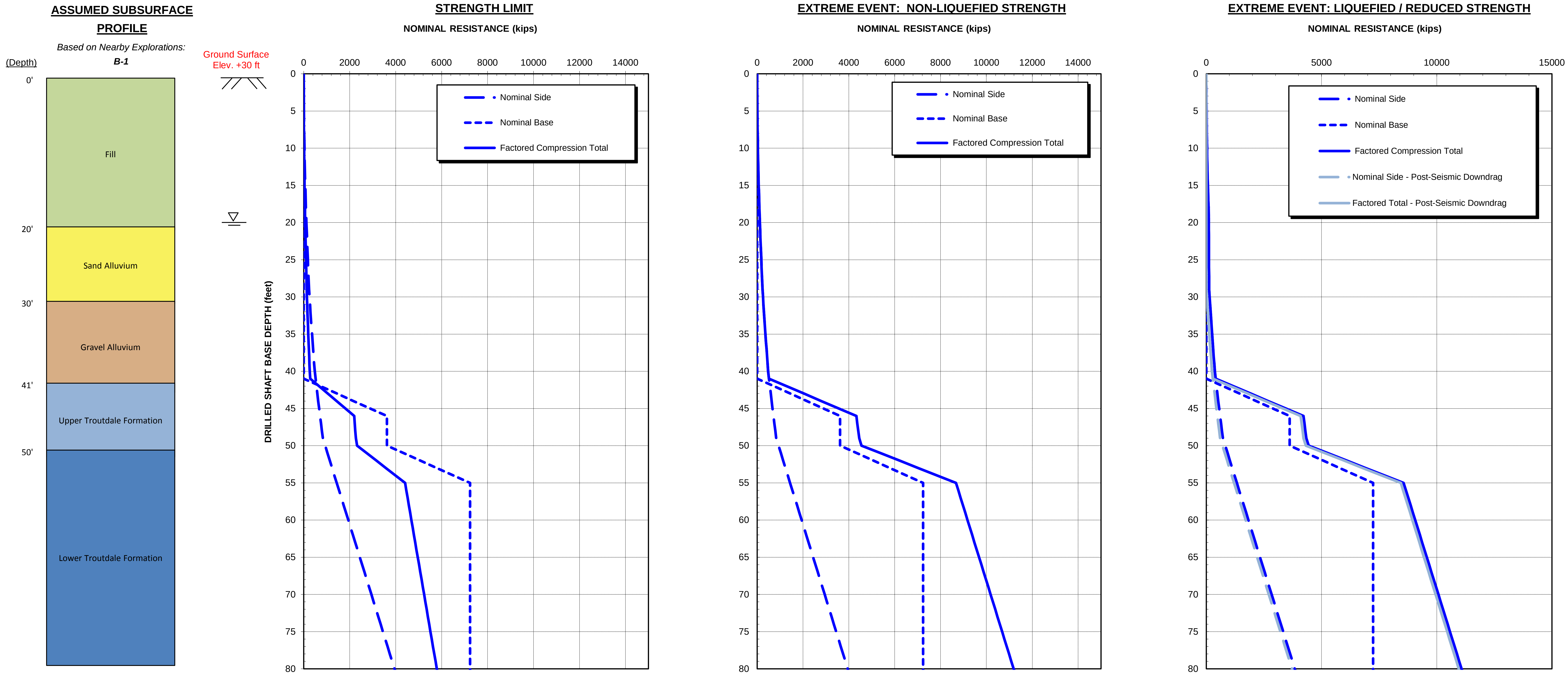
FIG. H2



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 170 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 4
8-FOOT DIAMETER DRILLED SHAFT

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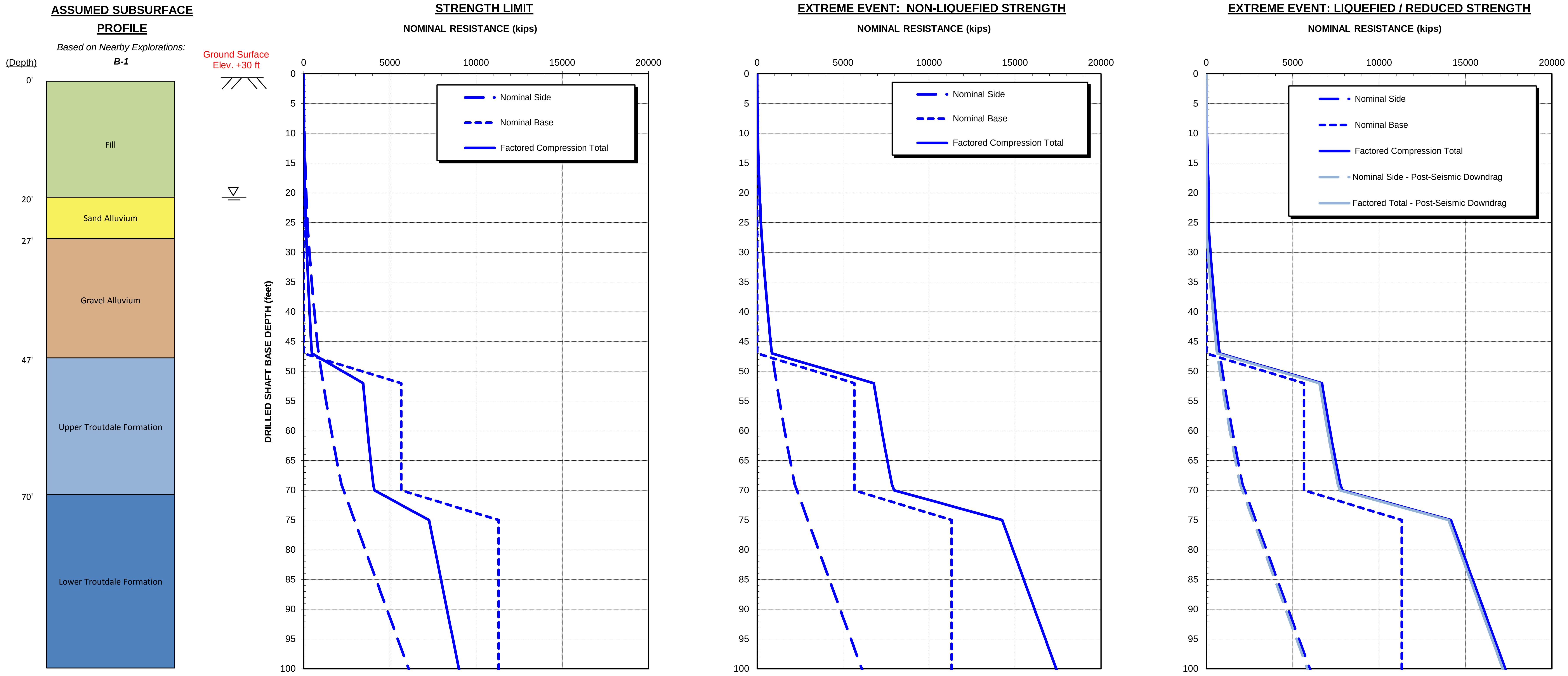
FIG. H3



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 180 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 5
10-FOOT DIAMETER DRILLED SHAFT

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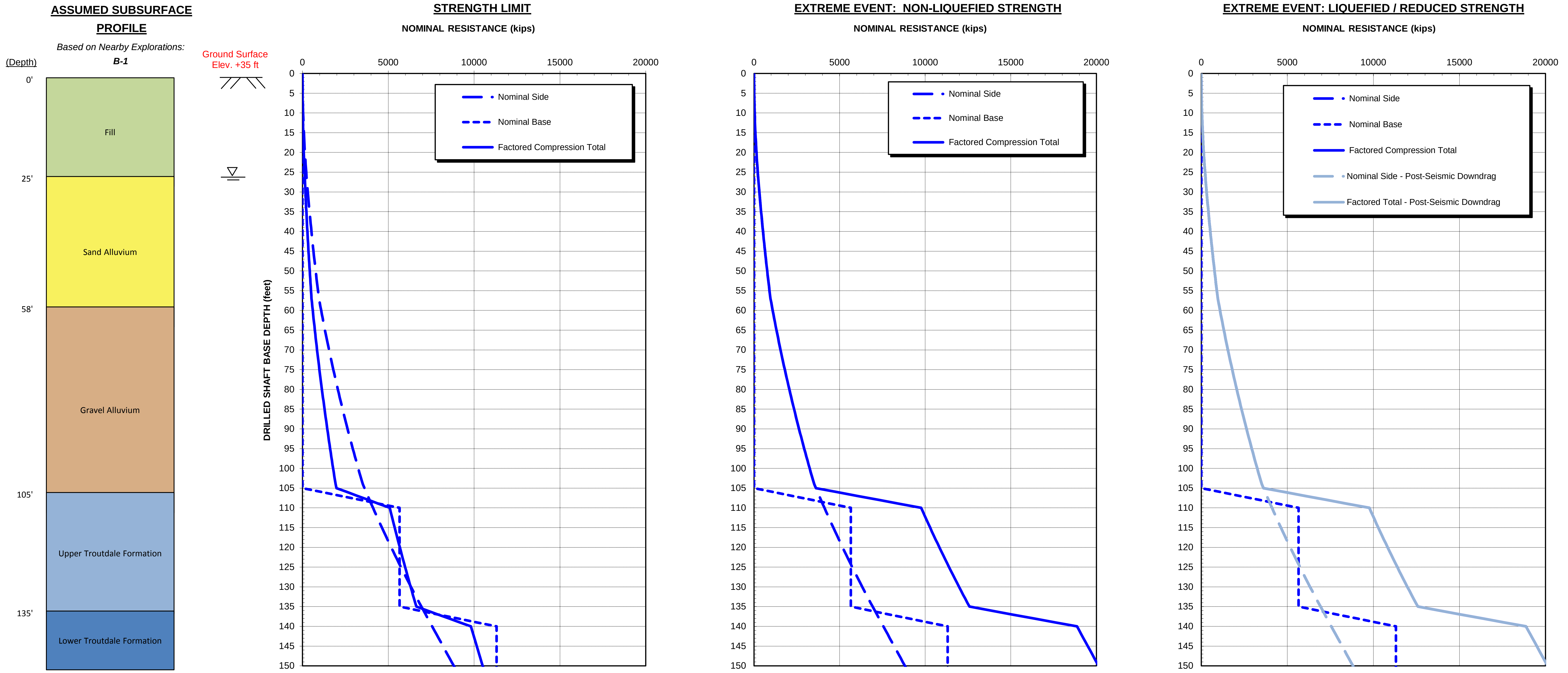
FIG. H4


ASSUMED SUBSURFACE PROFILE

STRENGTH LIMIT

EXTREME EVENT: NON-LIQUEFIED STRENGTH

EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 6
10-FOOT DIAMETER DRILLED SHAFT

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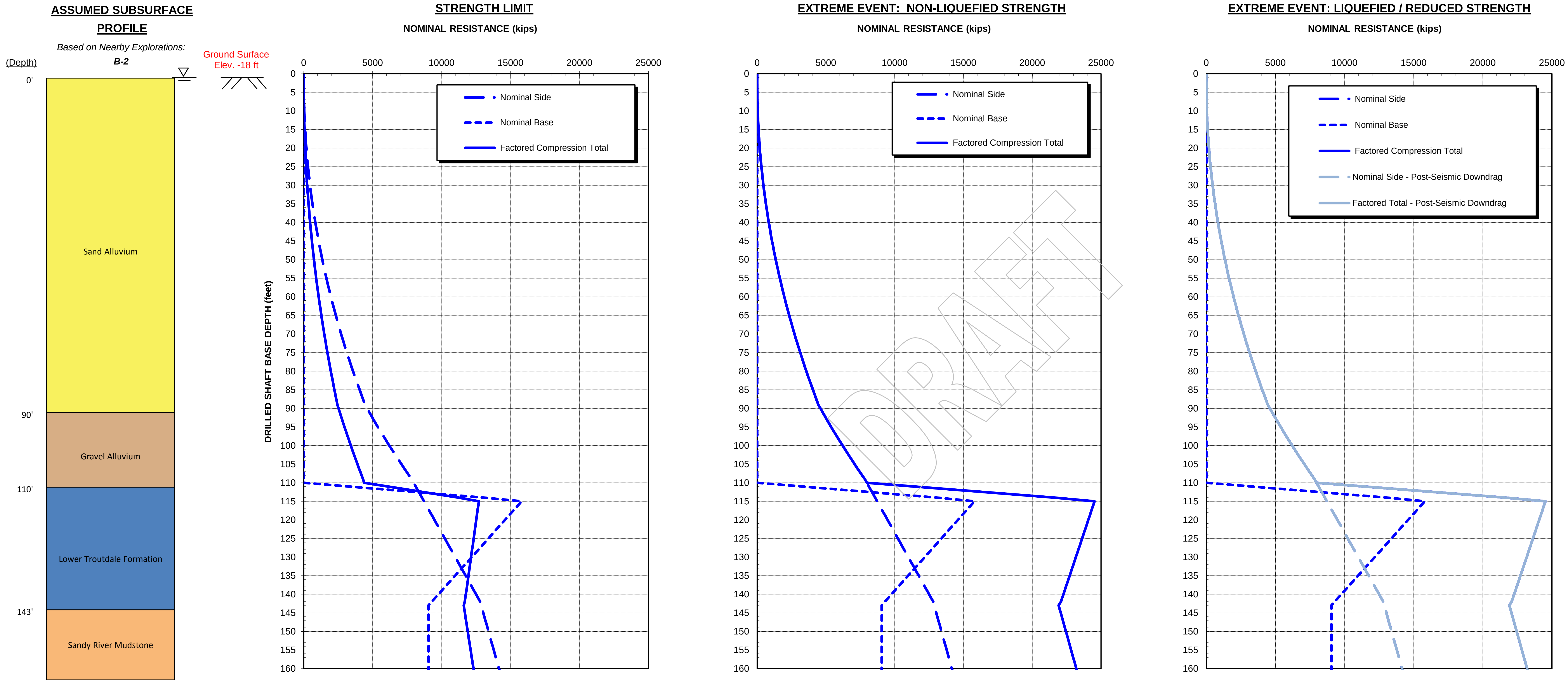
FIG. H5



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.

2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.

4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.

5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.

6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 9
12-FOOT DIAMETER DRILLED SHAFT

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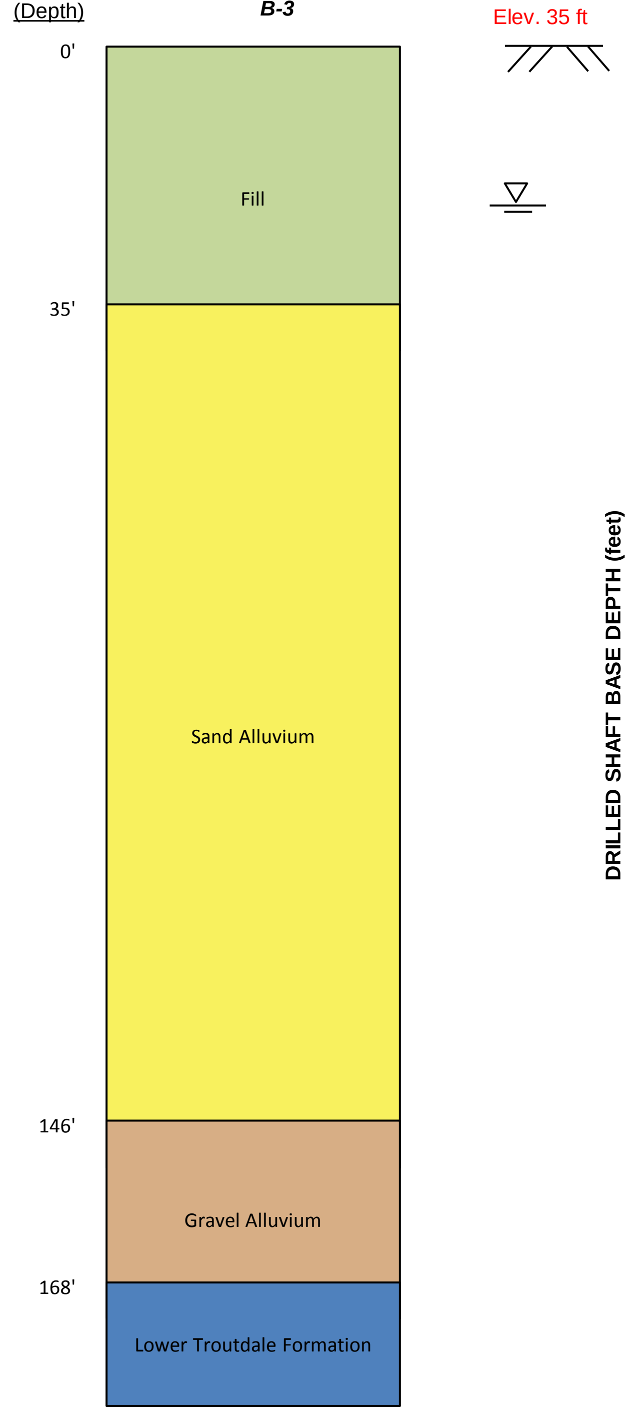
FIG. H6

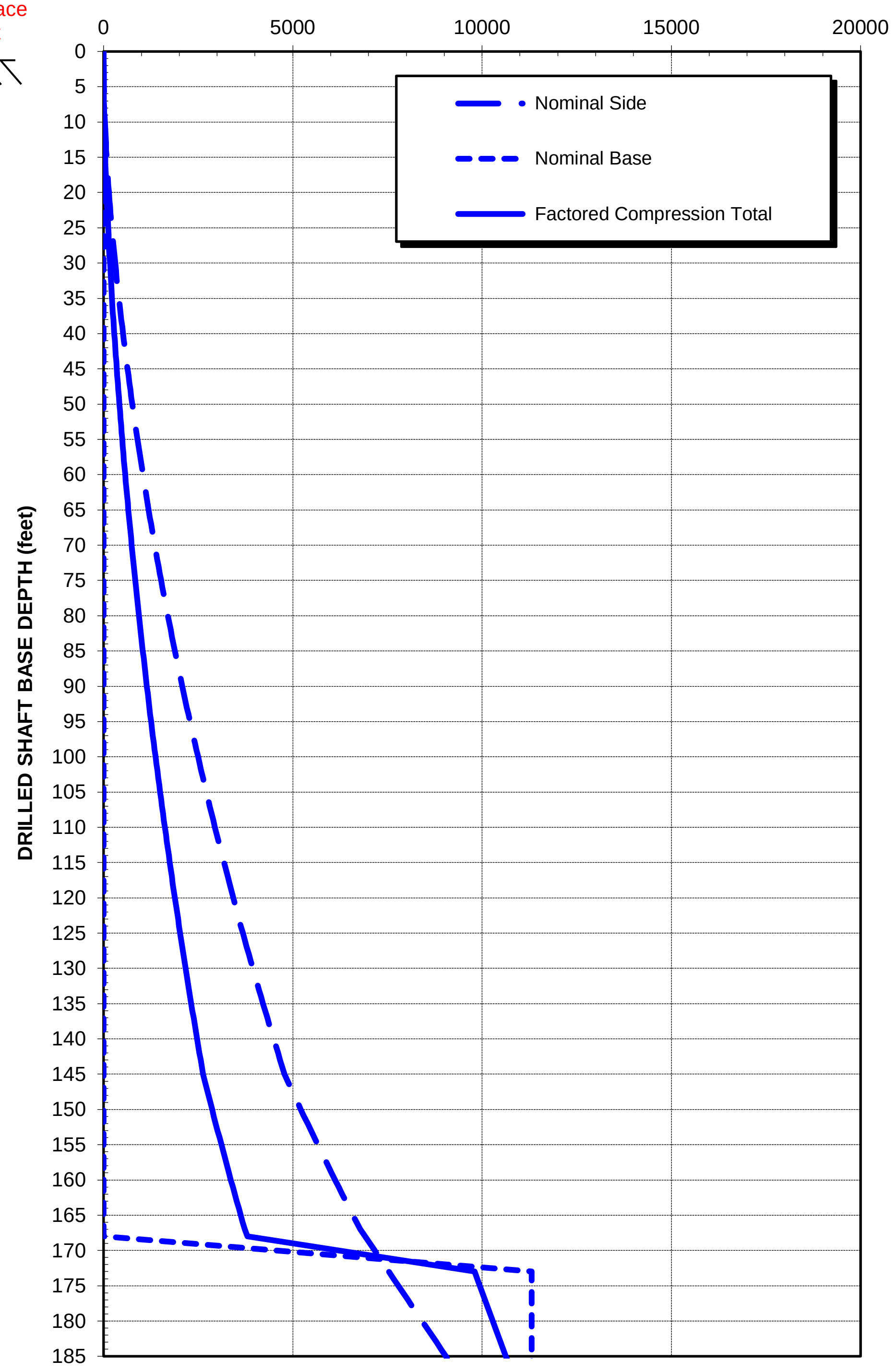
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations: ***B-3***



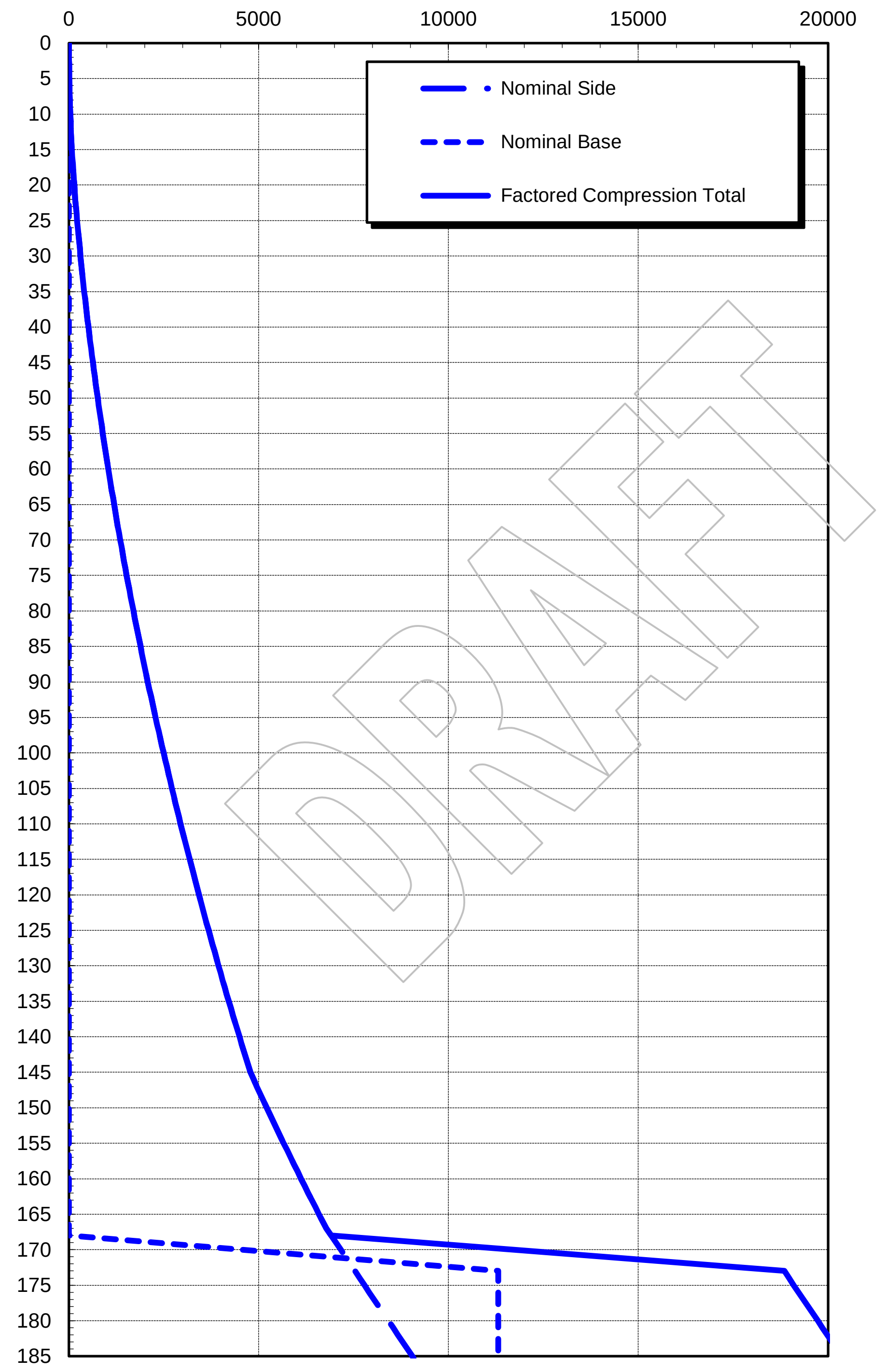
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



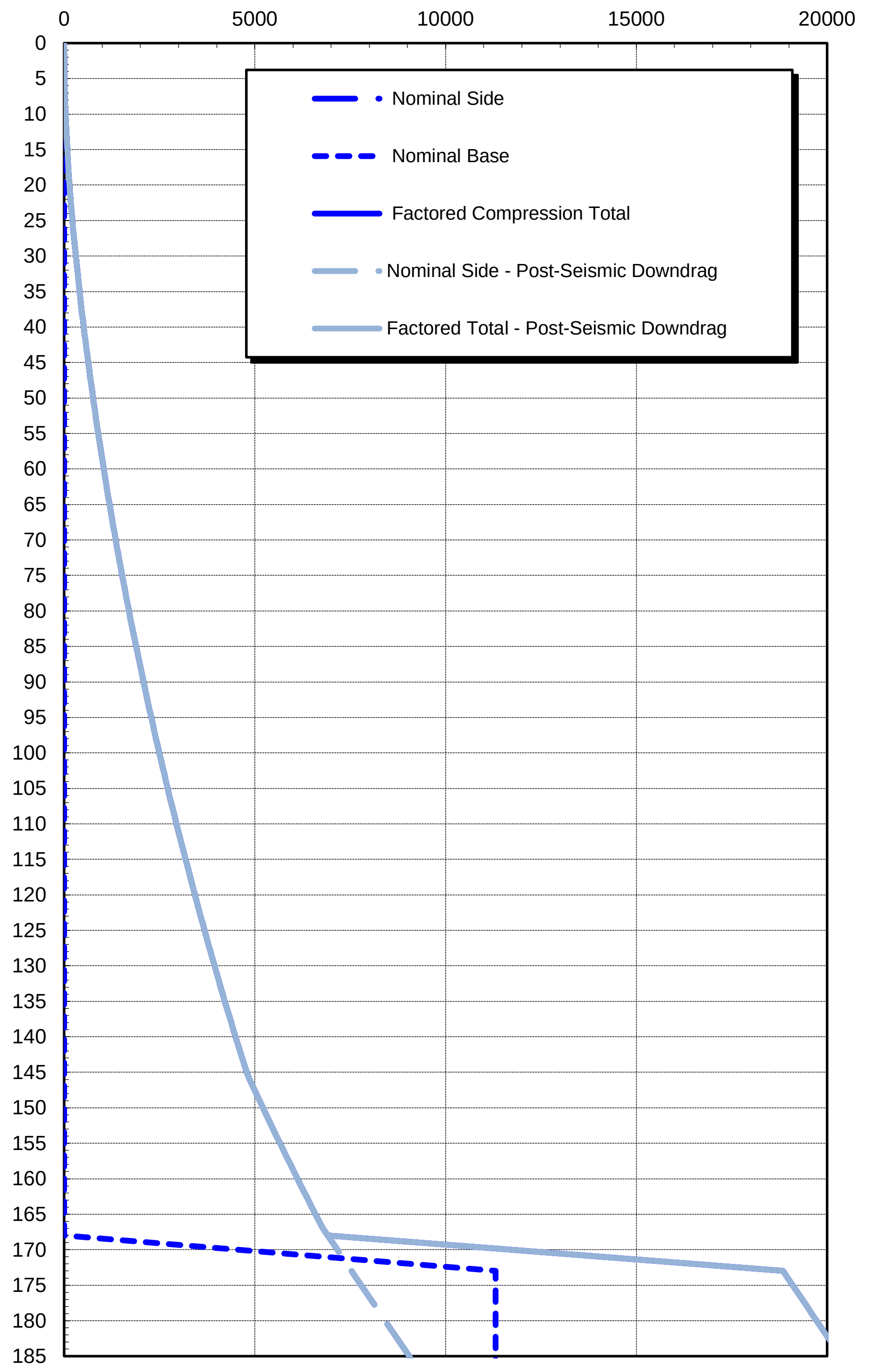
EXTREME EVENT: NON-LIQUEFIED STRENGTH

NOMINAL RESISTANCE (kips)



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOMINAL RESISTANCE (kips)



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
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ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 10/S10
10-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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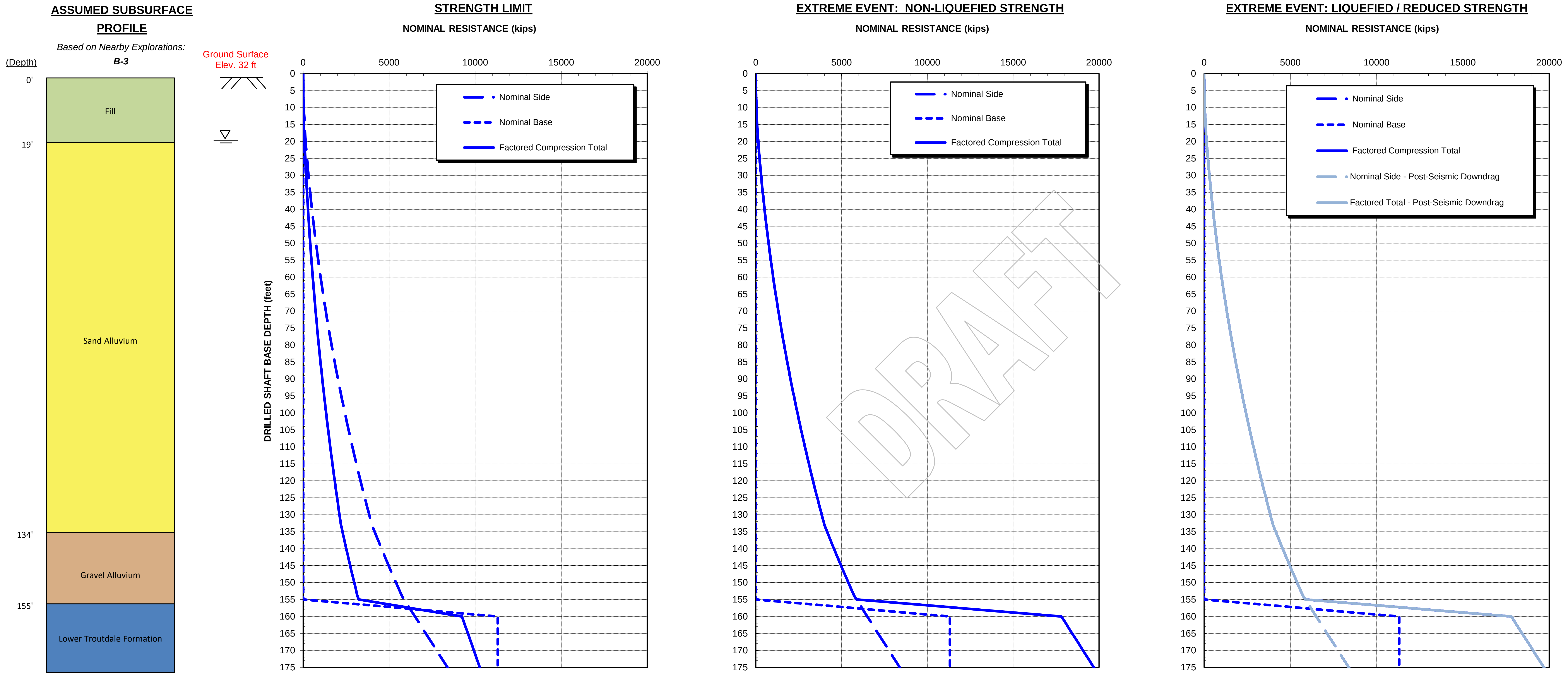
FIG. H7



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.

2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.

4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.

5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.

6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 11/S11
10-FOOT DIAMETER DRILLED SHAFT

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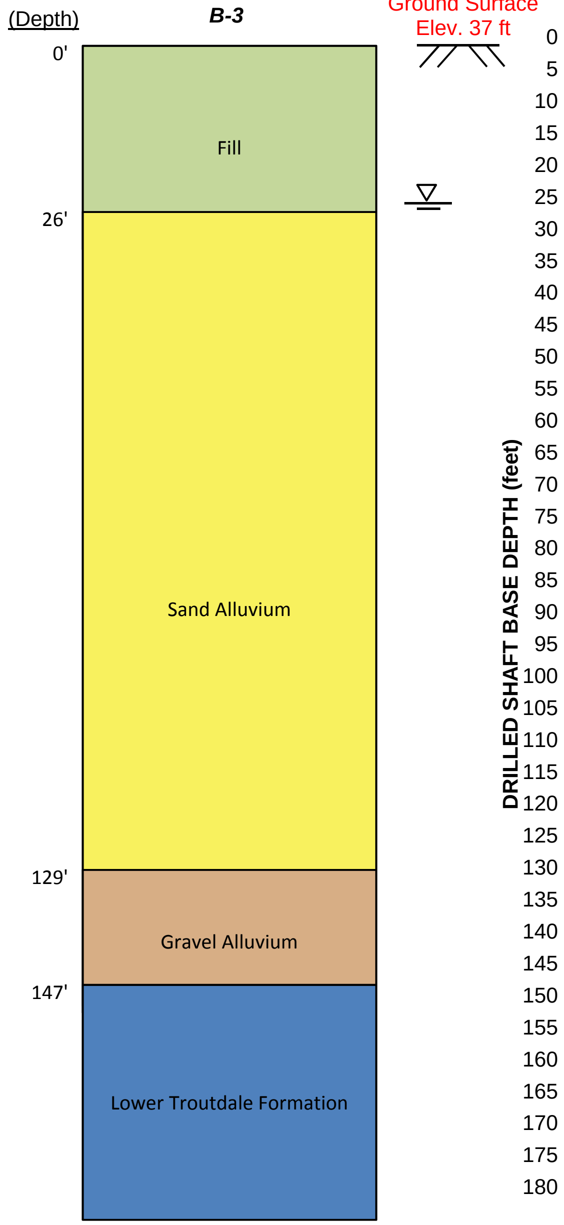
FIG. H8

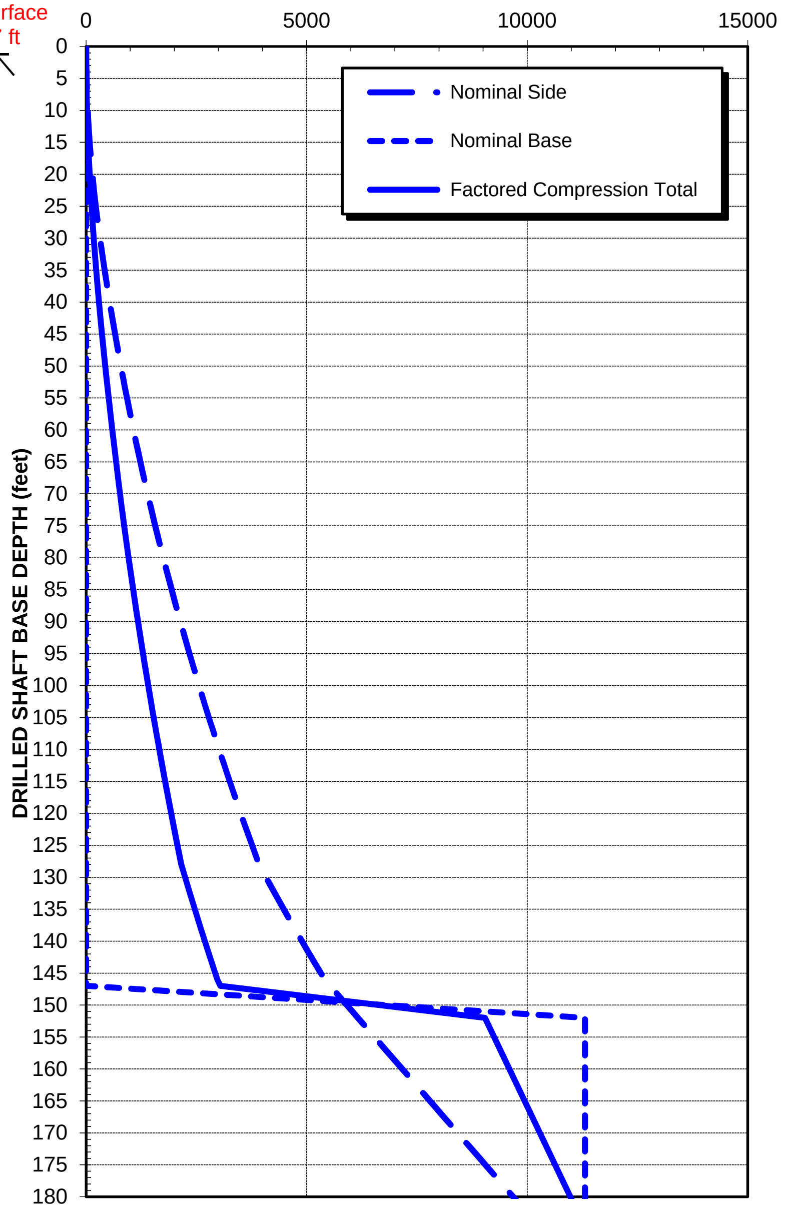
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:



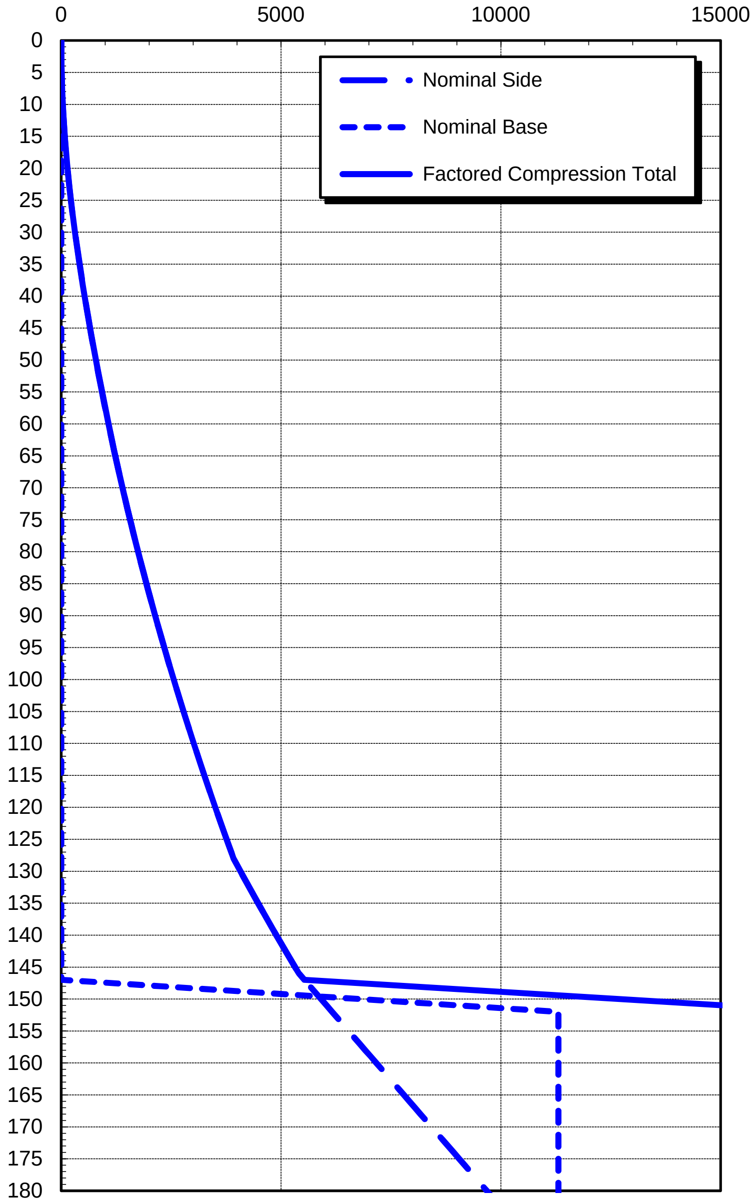
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



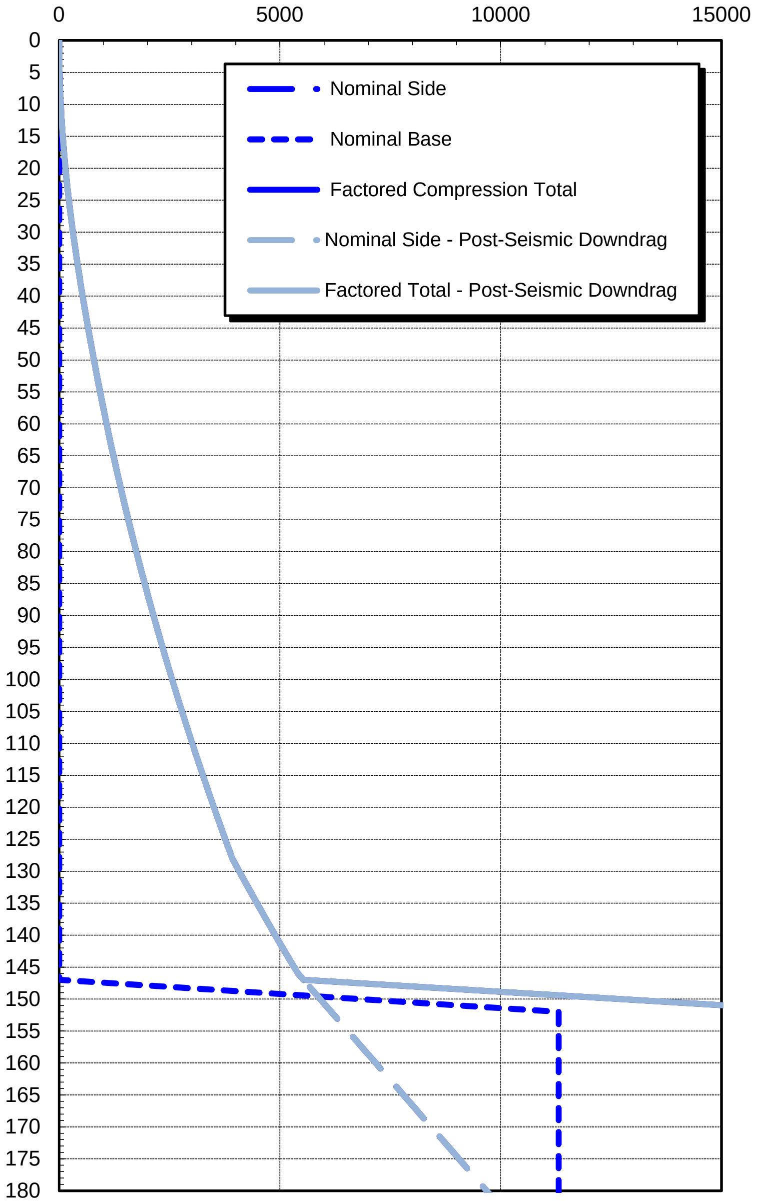
EXTREME EVENT: NON-LIQUEFIED STRENGTH

NOMINAL RESISTANCE (kips)



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOMINAL RESISTANCE (kips)



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 12/S12
10-FOOT DIAMETER DRILLED SHAFT

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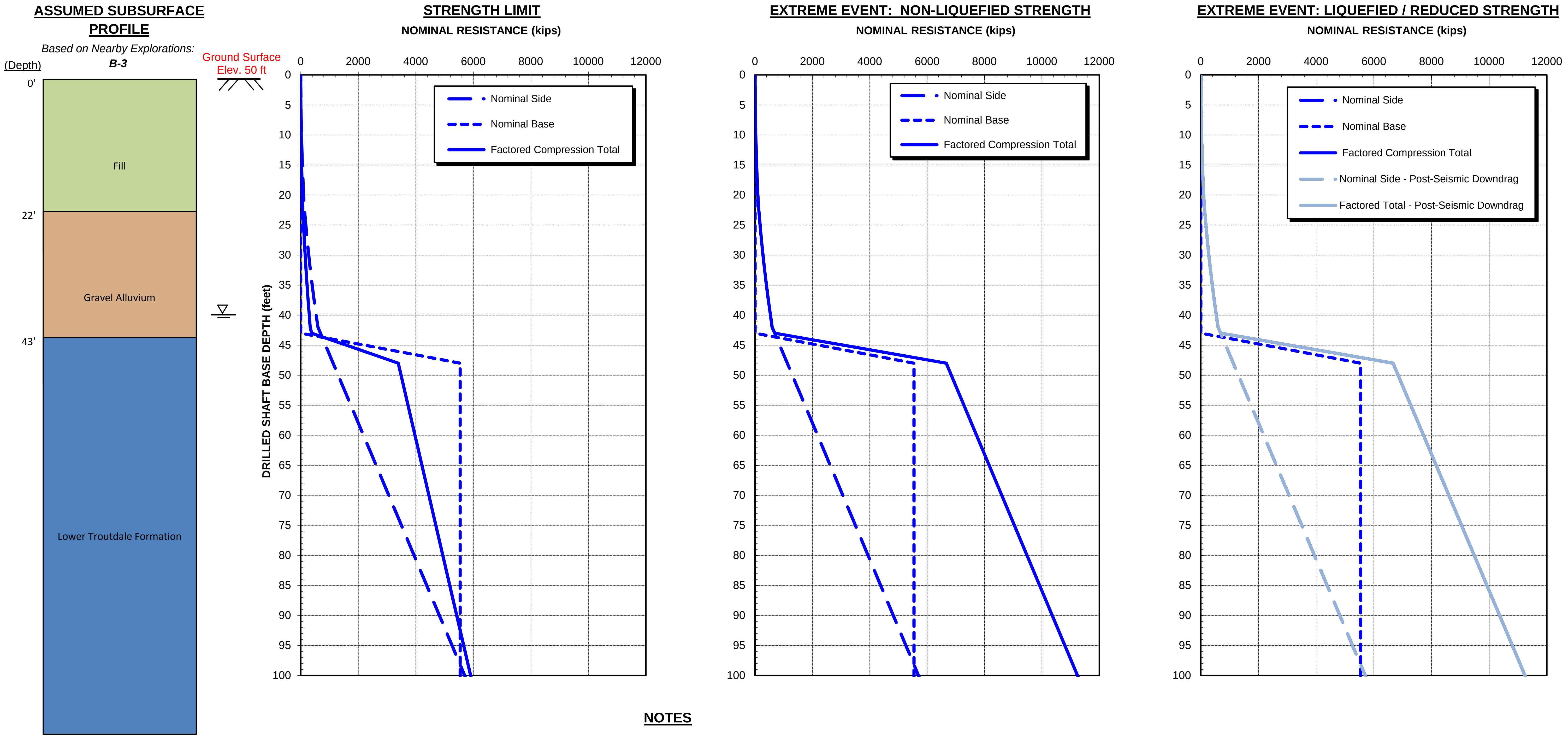
FIG. H9



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 13/S13
7-FOOT DIAMETER DRILLED SHAFT

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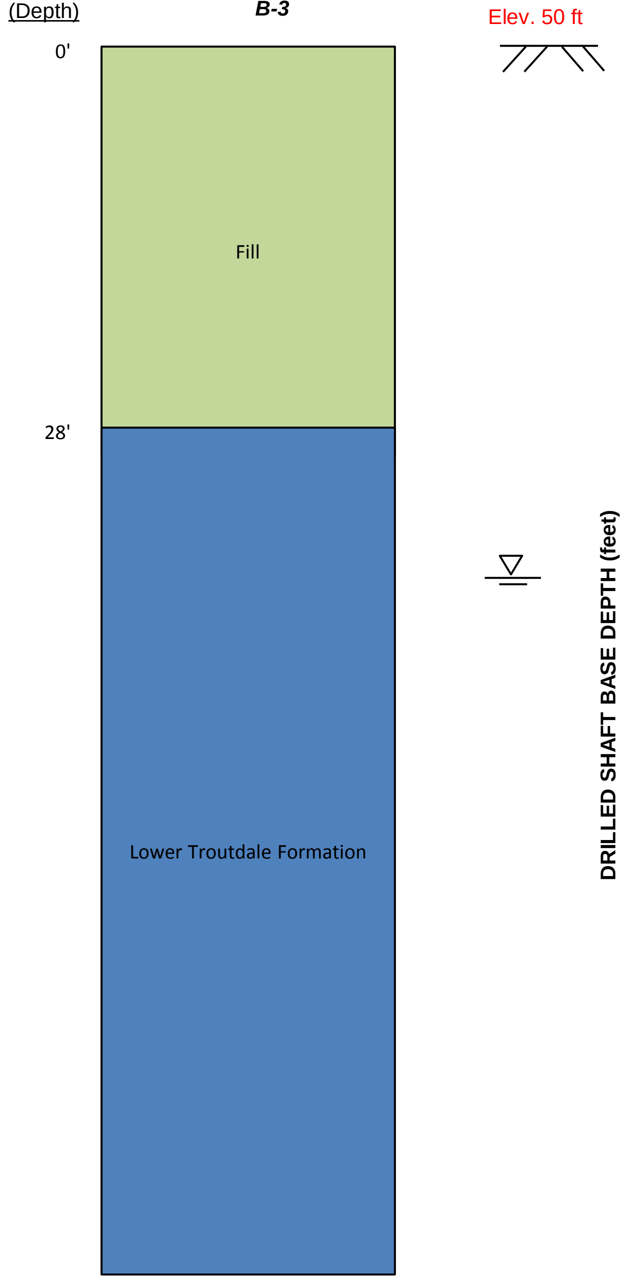
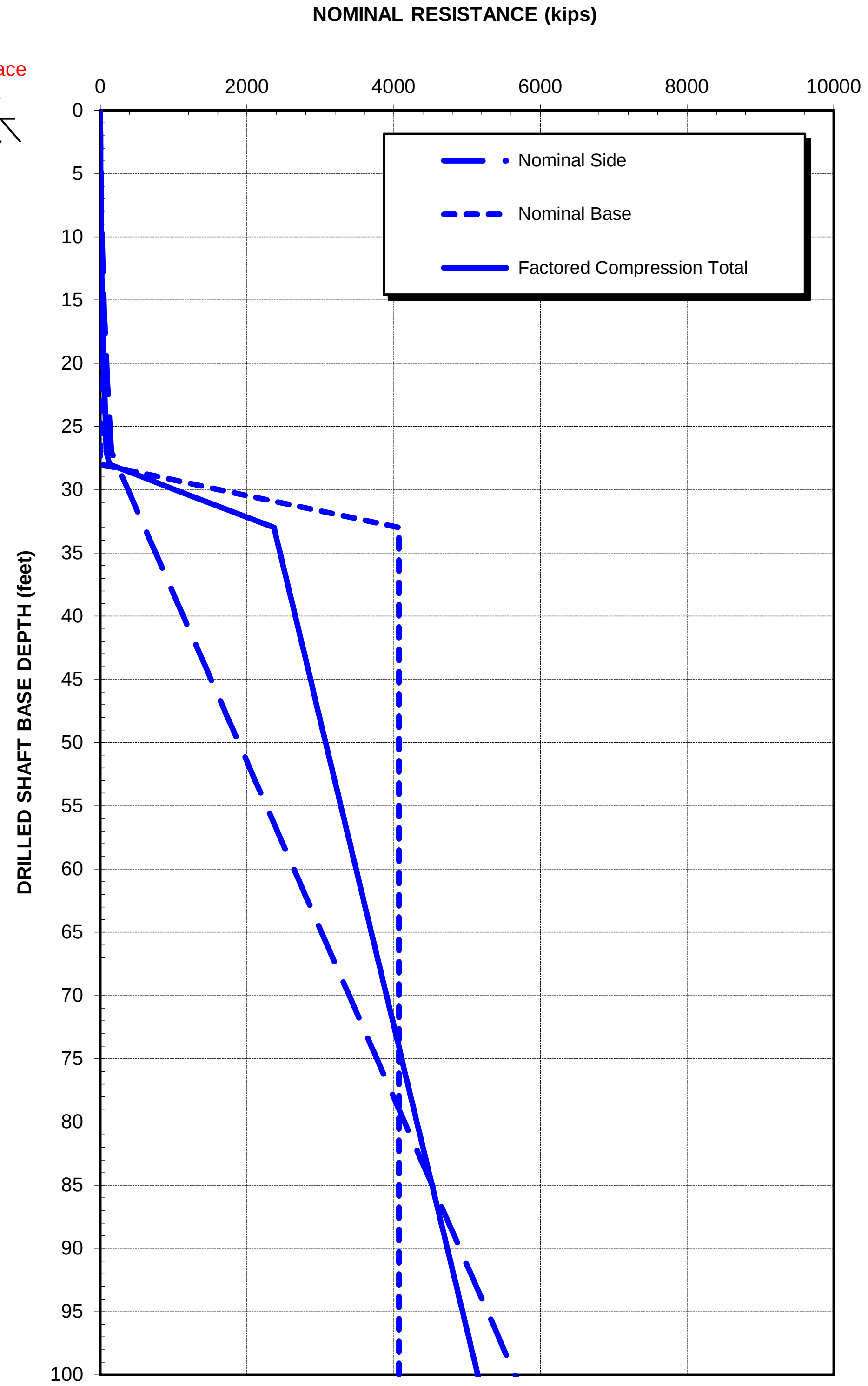
FIG. H10

ASSUMED SUBSURFACE PROFILE

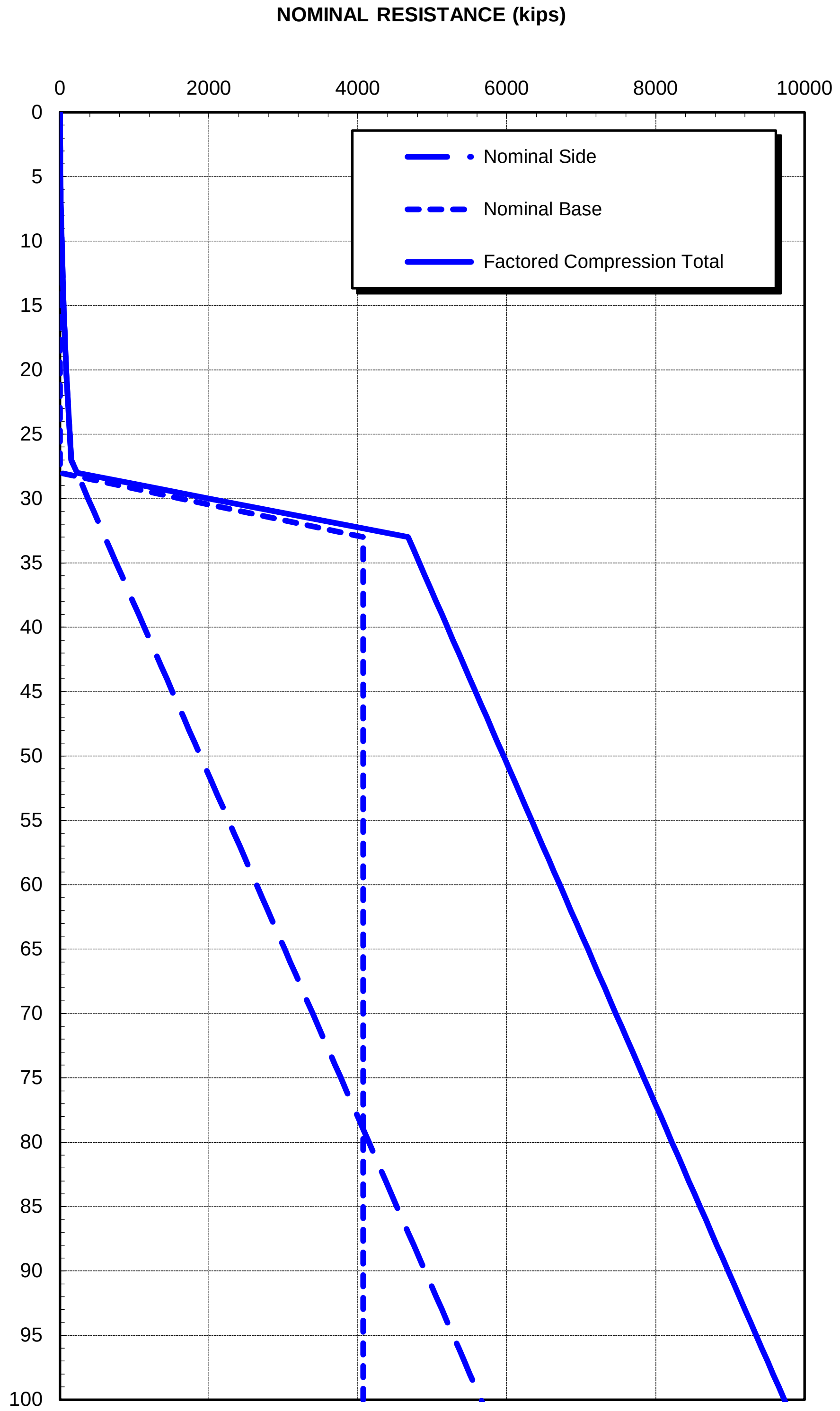
Based on Nearby Explorations:
B-3



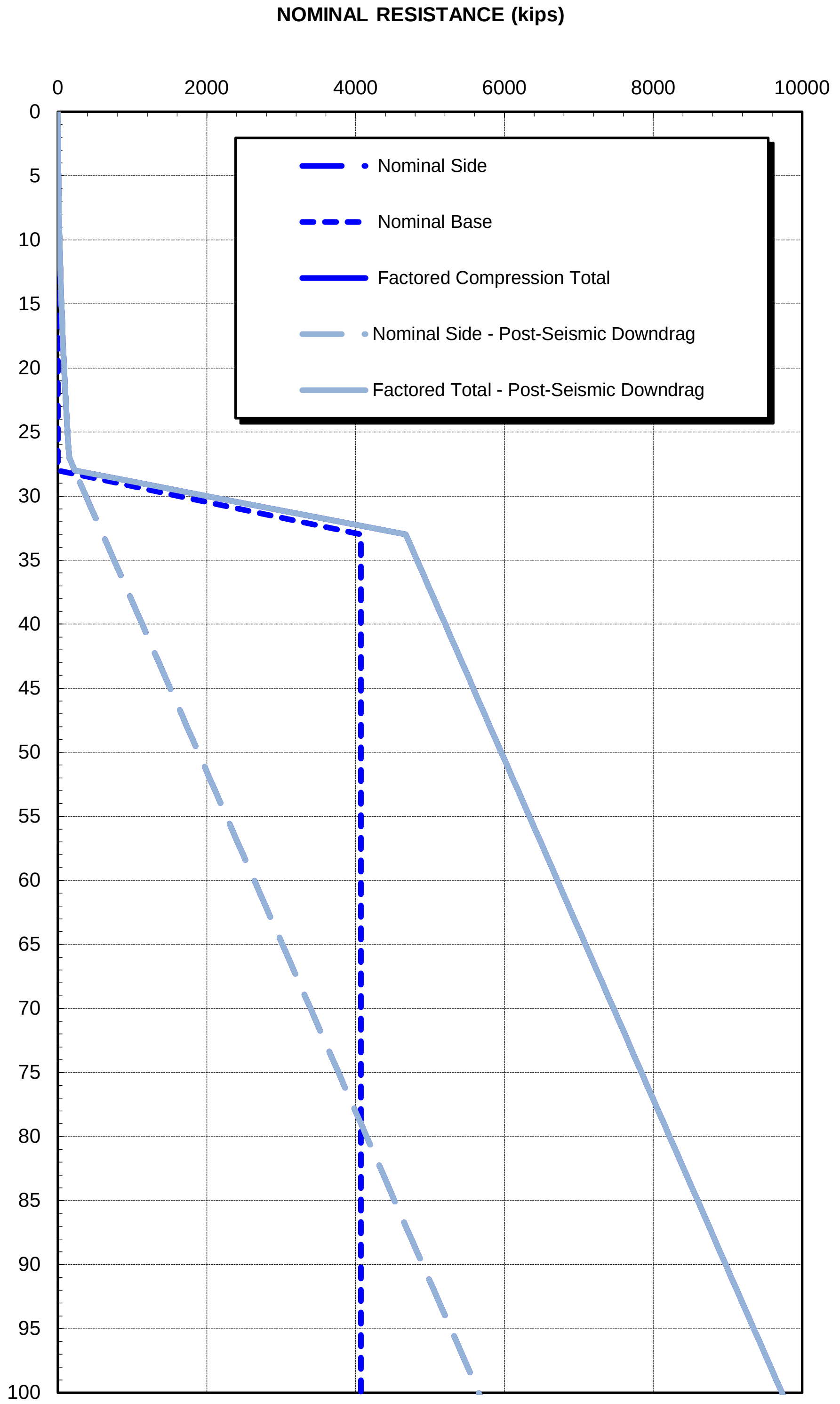
STRENGTH LIMIT



EXTREME EVENT: NON-LIQUEFIED STRENGTH



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 14/S14
3-FOOT DIAMETER DRILLED SHAFT

August 2019
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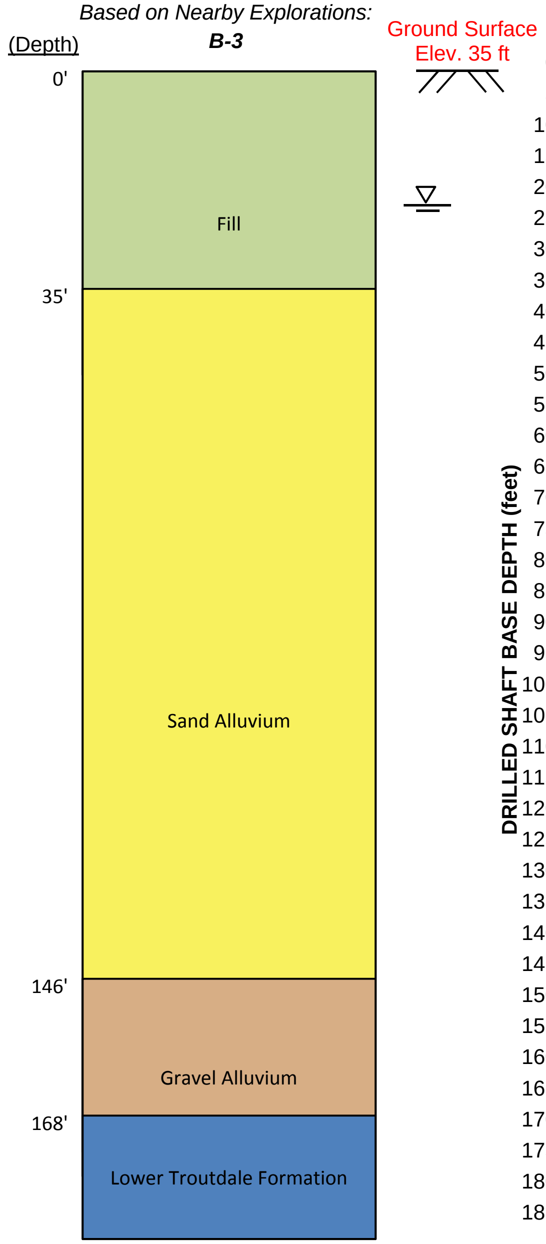
FIG. H11

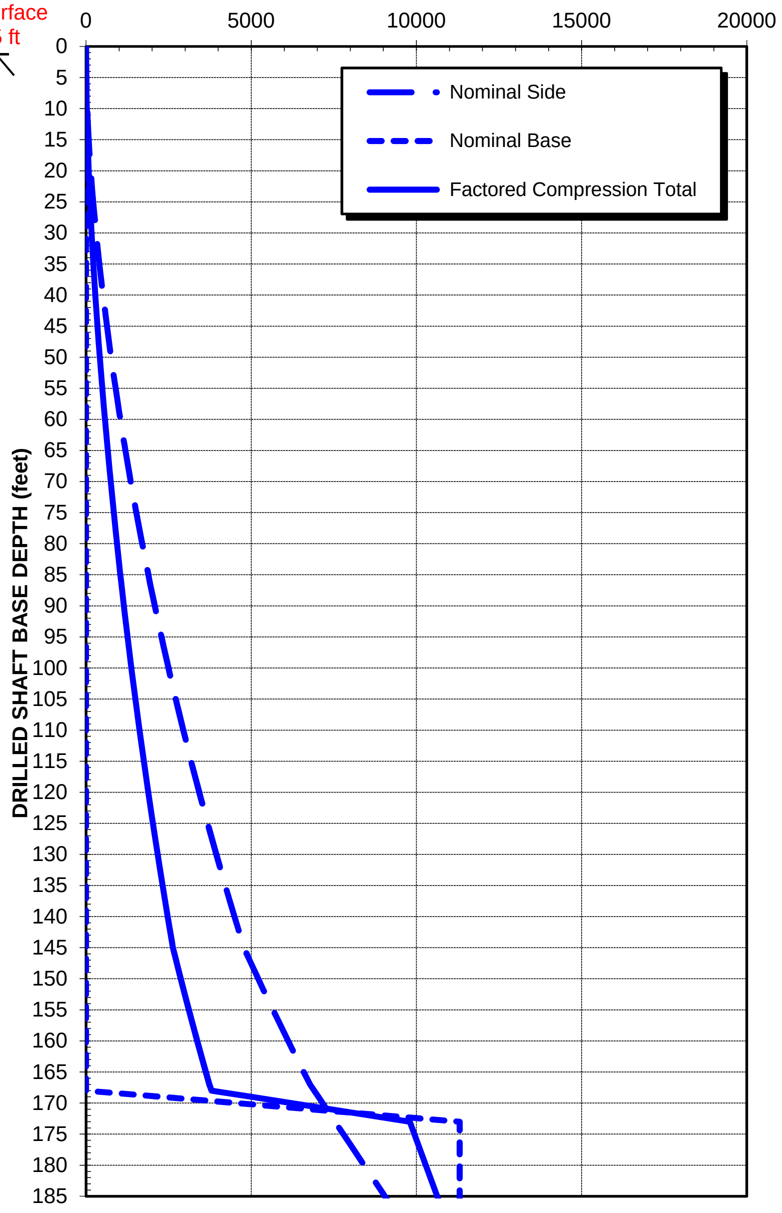
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:



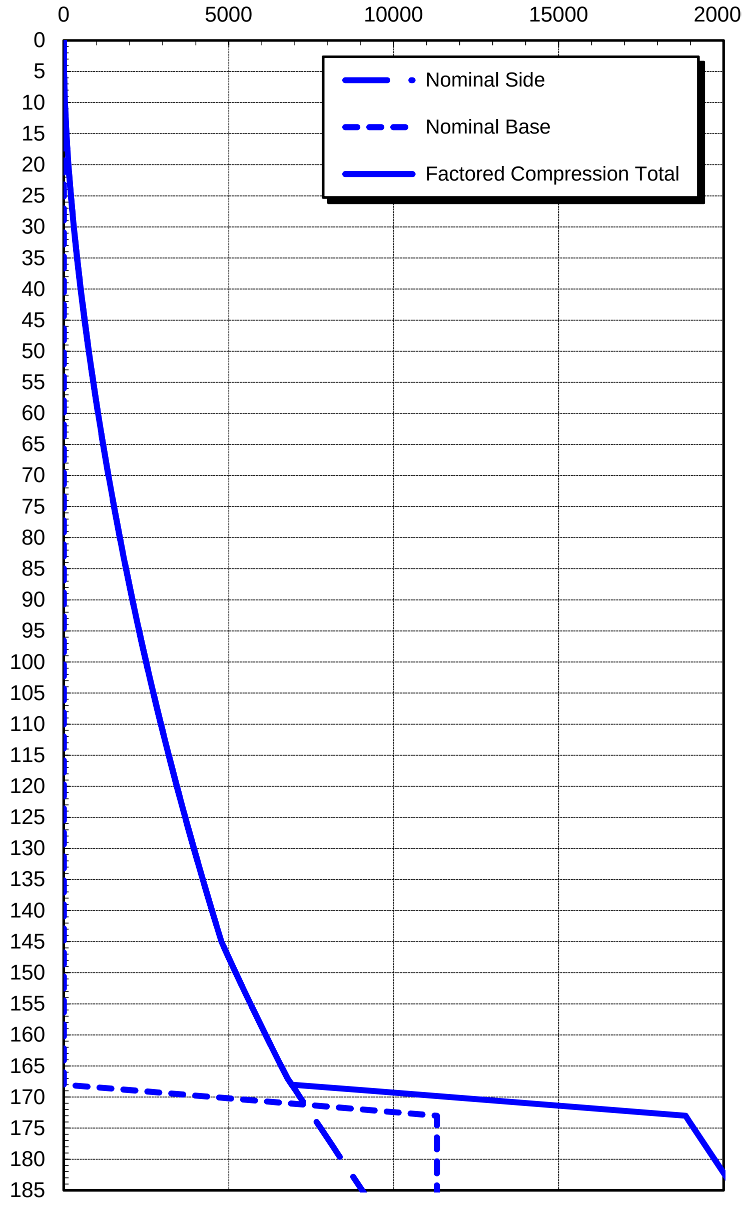
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



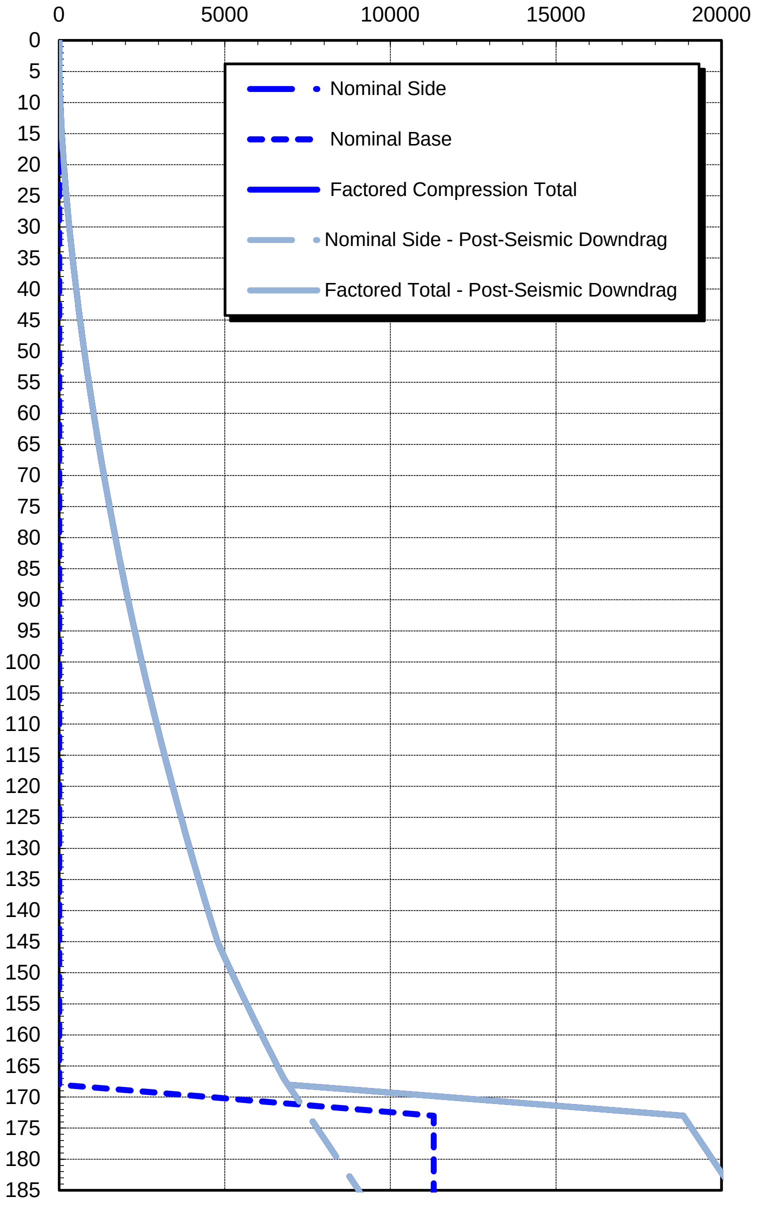
EXTREME EVENT: NON-LIQUEFIED STRENGTH

NOMINAL RESISTANCE (kips)



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOMINAL RESISTANCE (kips)



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT N10
10-FOOT DIAMETER DRILLED SHAFT

August 2019
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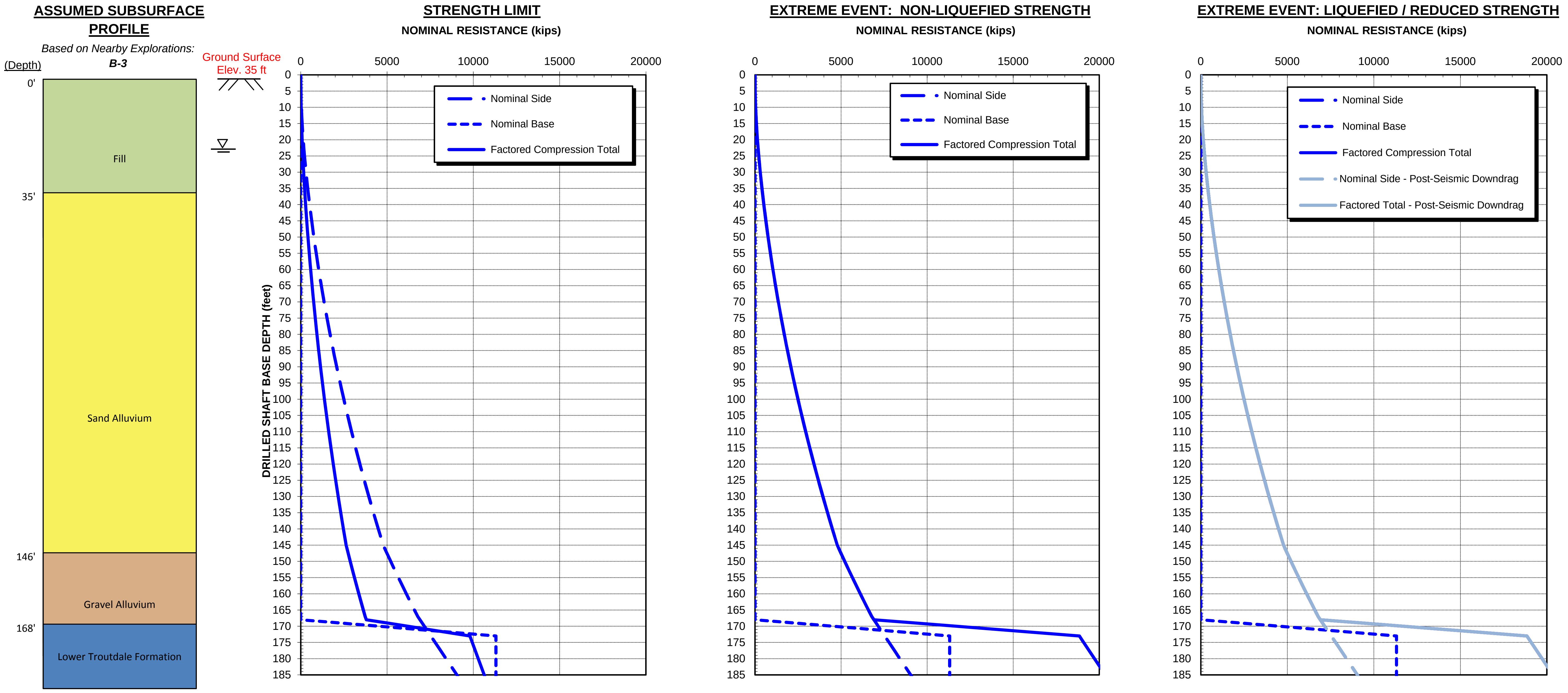
FIG. H12



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT N11
10-FOOT DIAMETER DRILLED SHAFT

August 2019 102636-001

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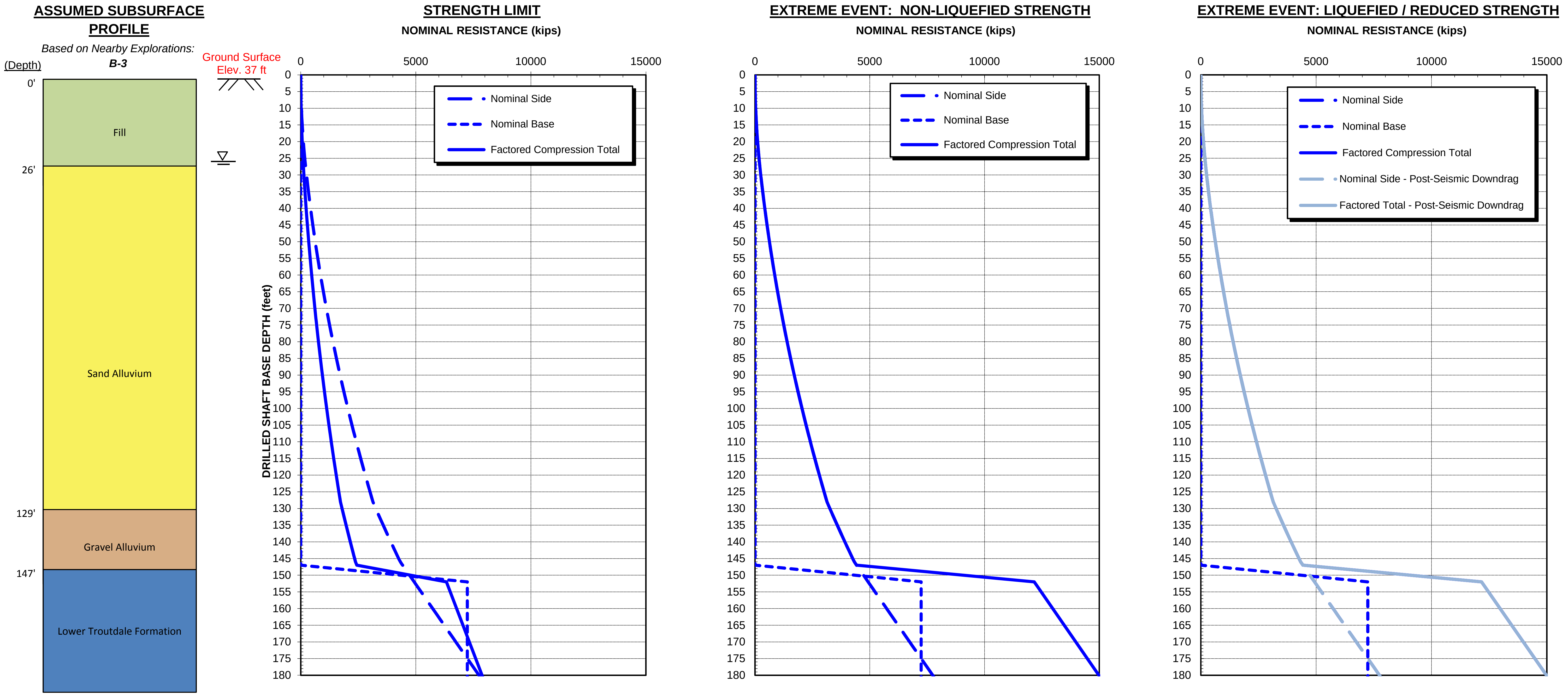
FIG. H13



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT N12
8-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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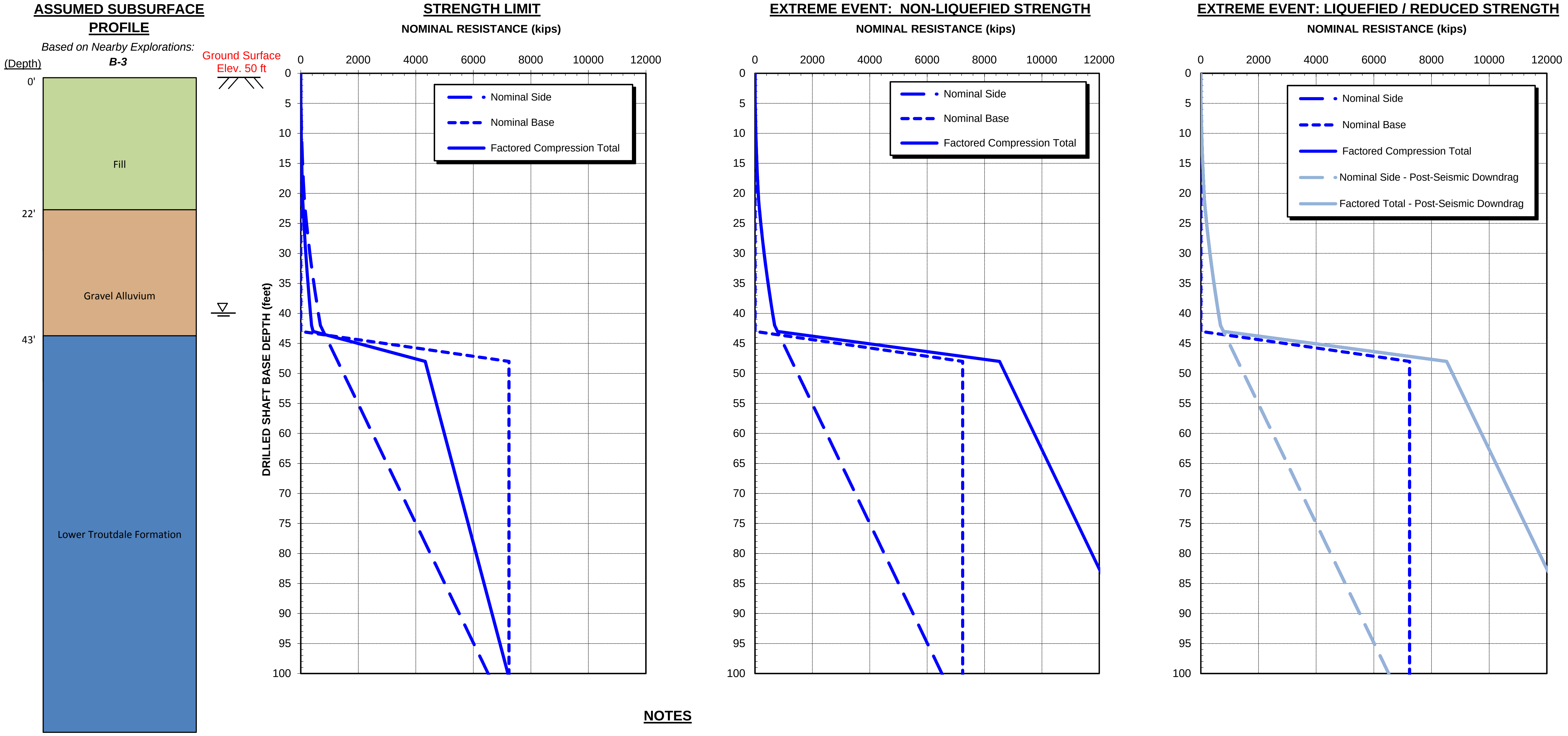
FIG. H14



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT N13
8-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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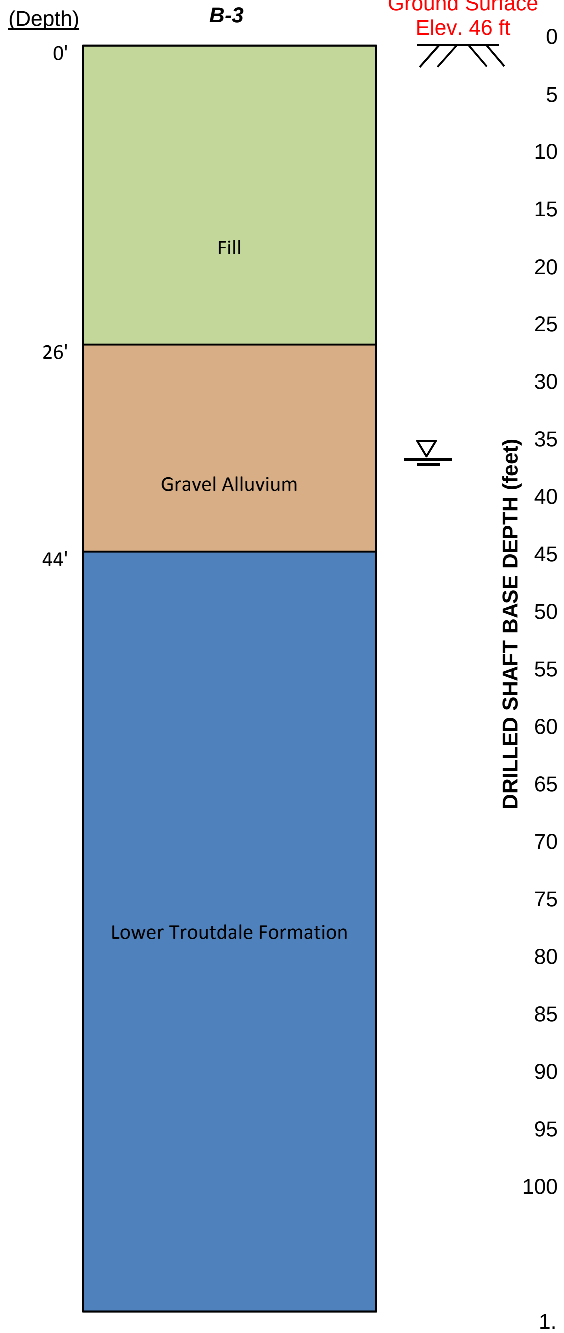
FIG. H15

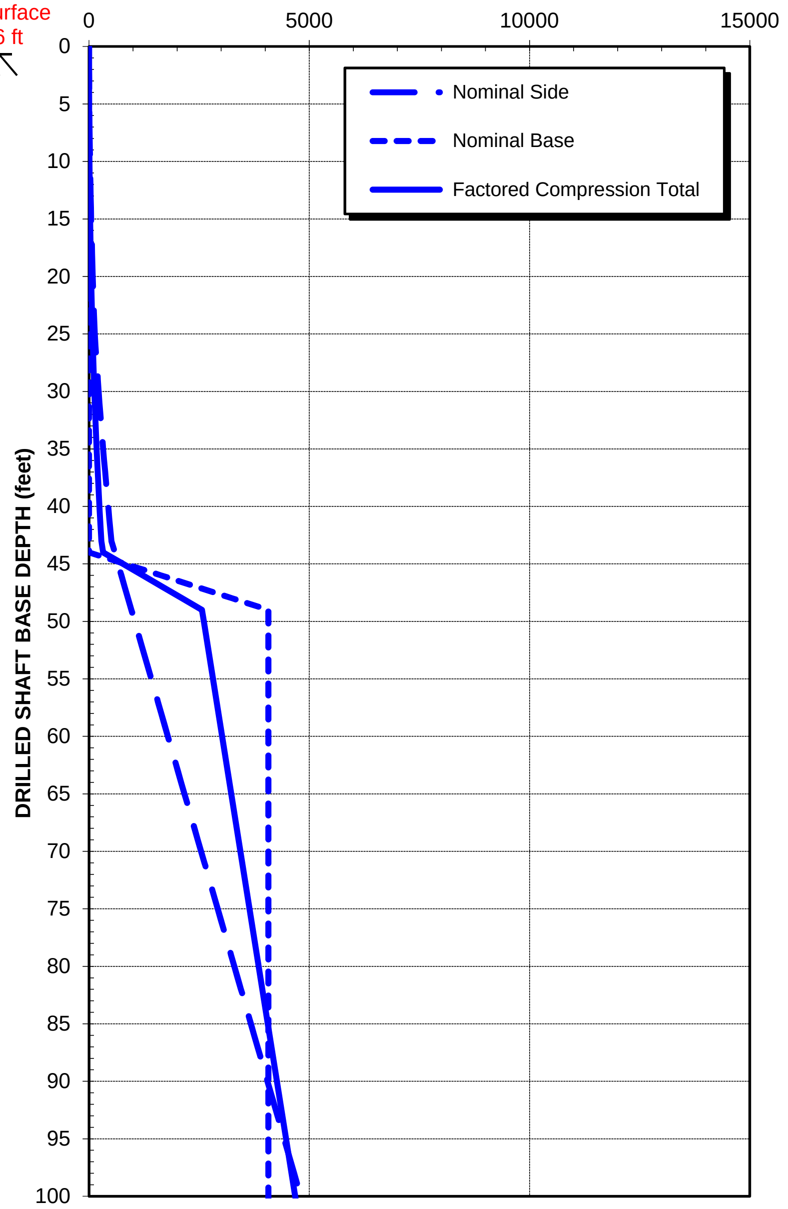
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-3



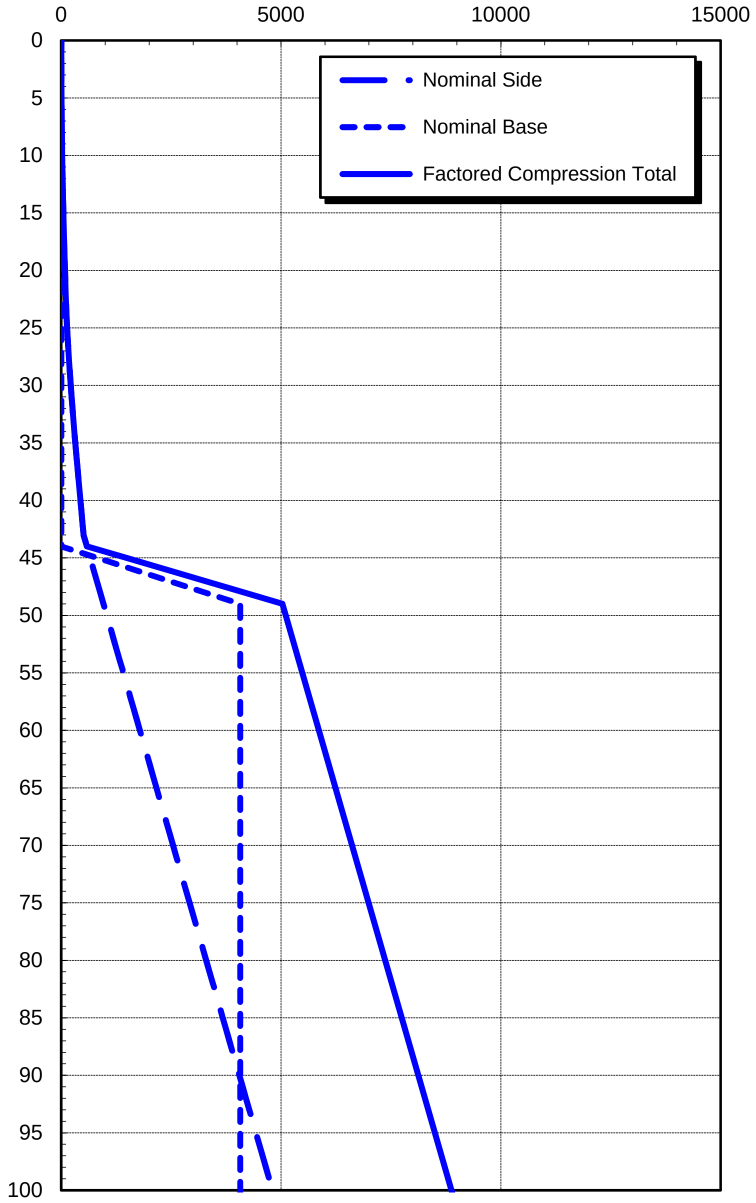
STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



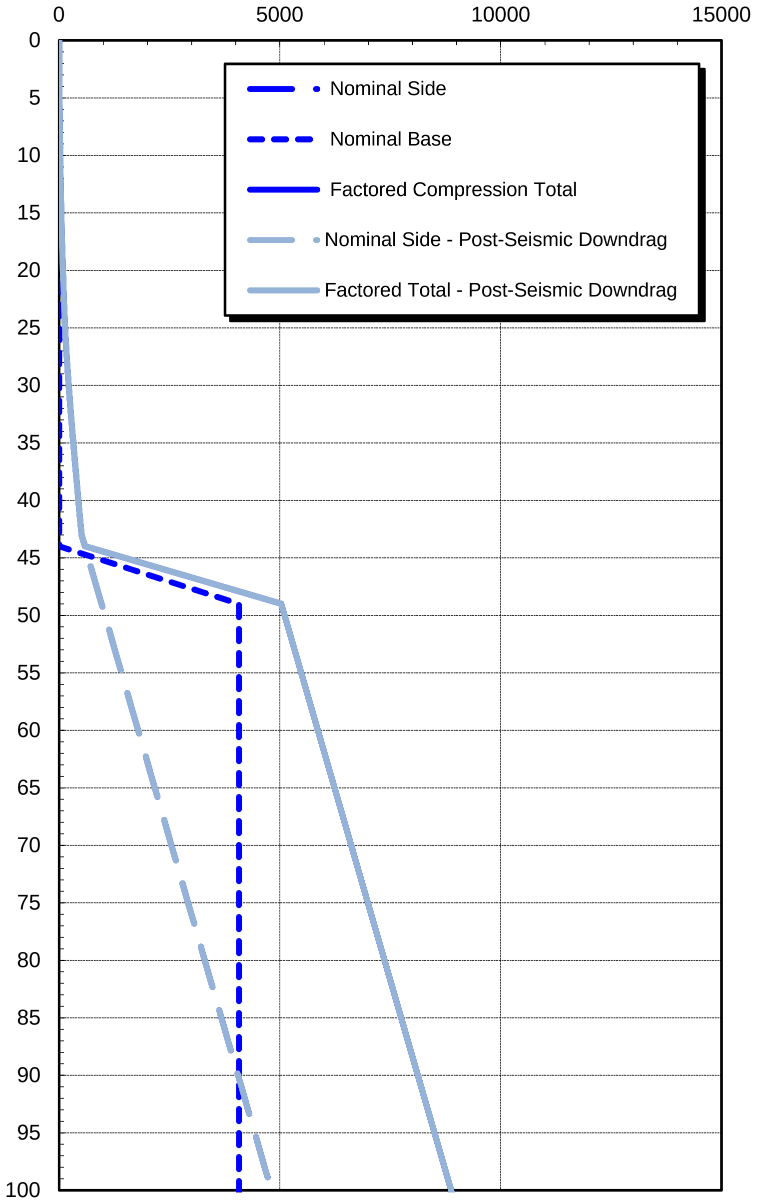
EXTREME EVENT: NON-LIQUEFIED STRENGTH

NOMINAL RESISTANCE (kips)



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOMINAL RESISTANCE (kips)



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT N14
6-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

SHANNON & WILSON, INC.
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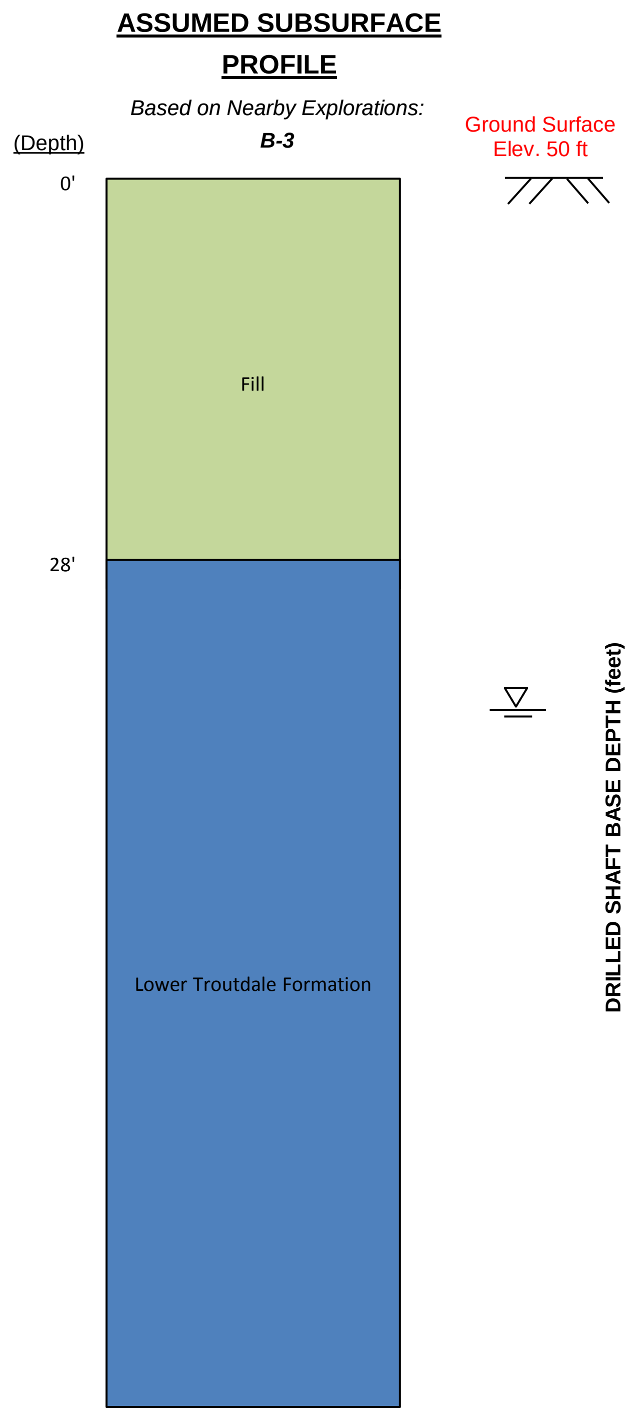
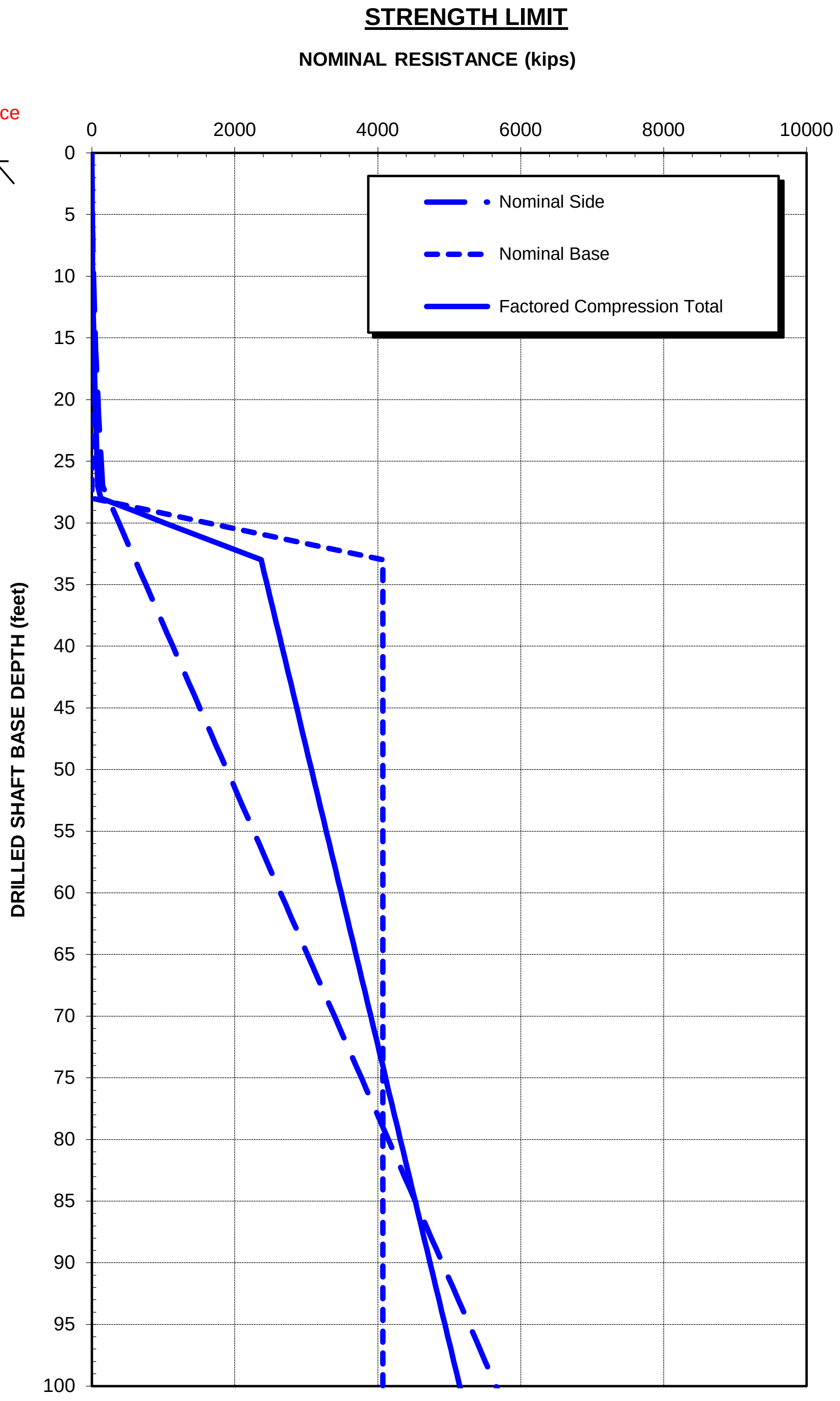
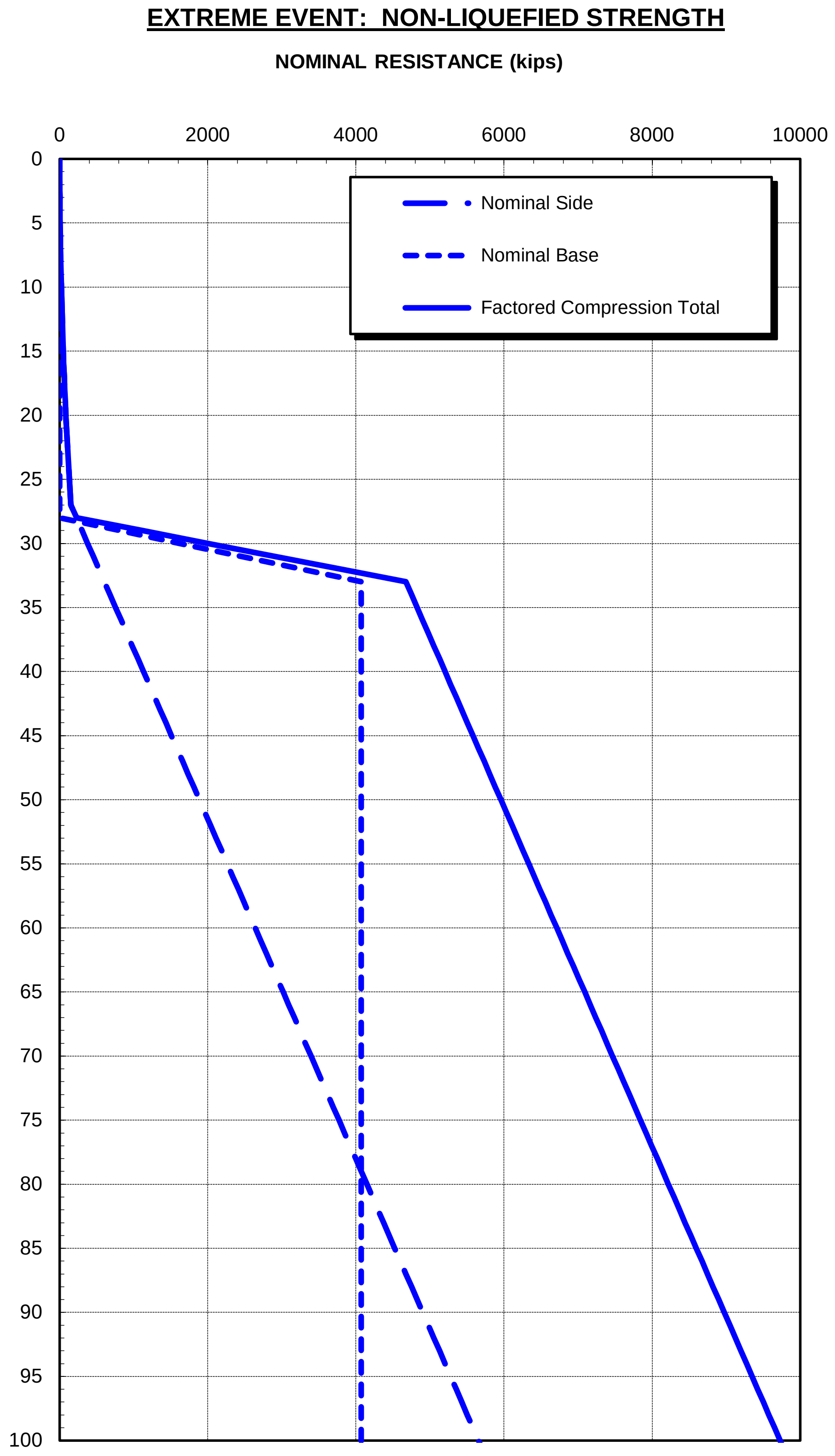
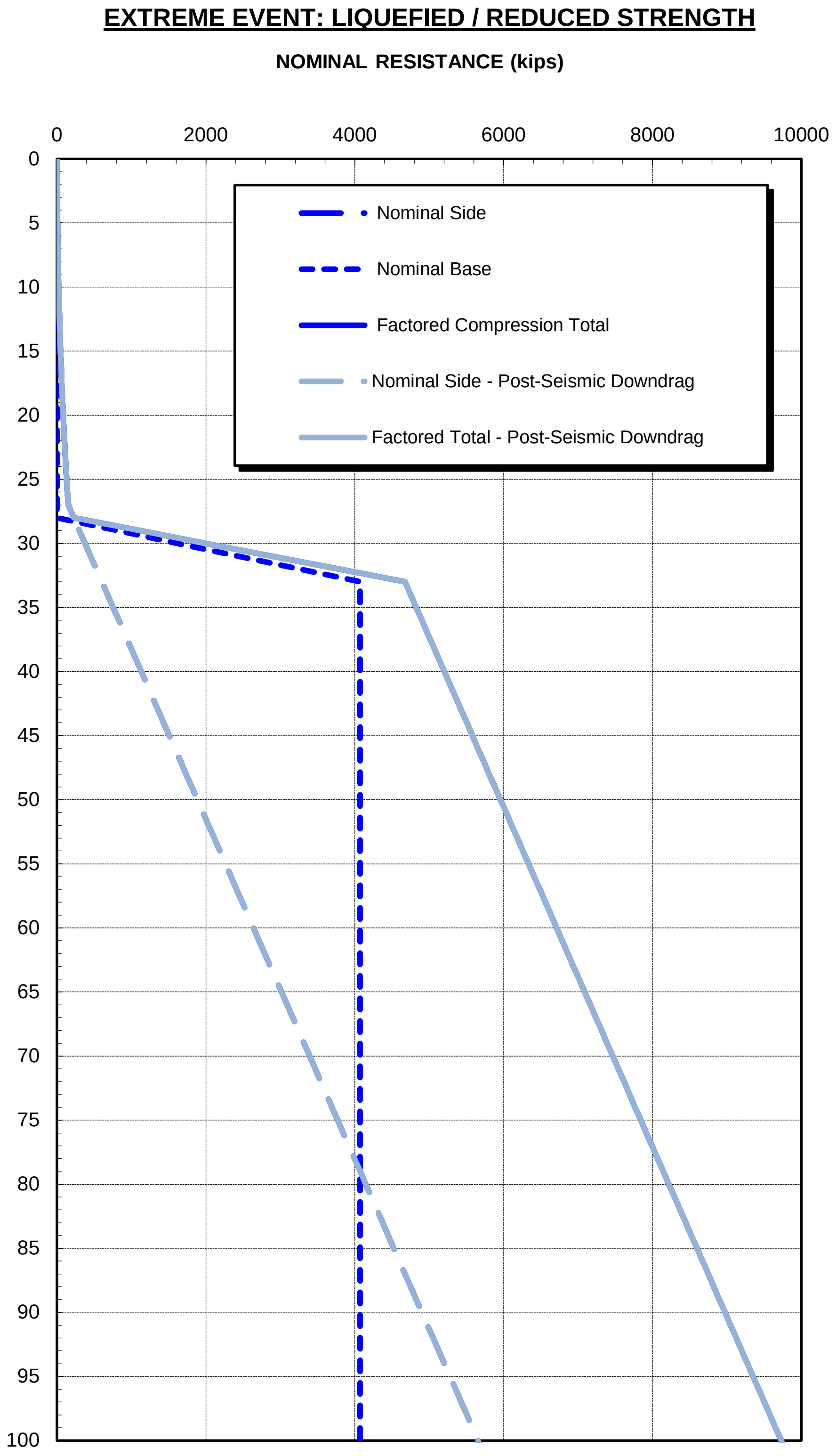
FIG. H16









NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT N15
3-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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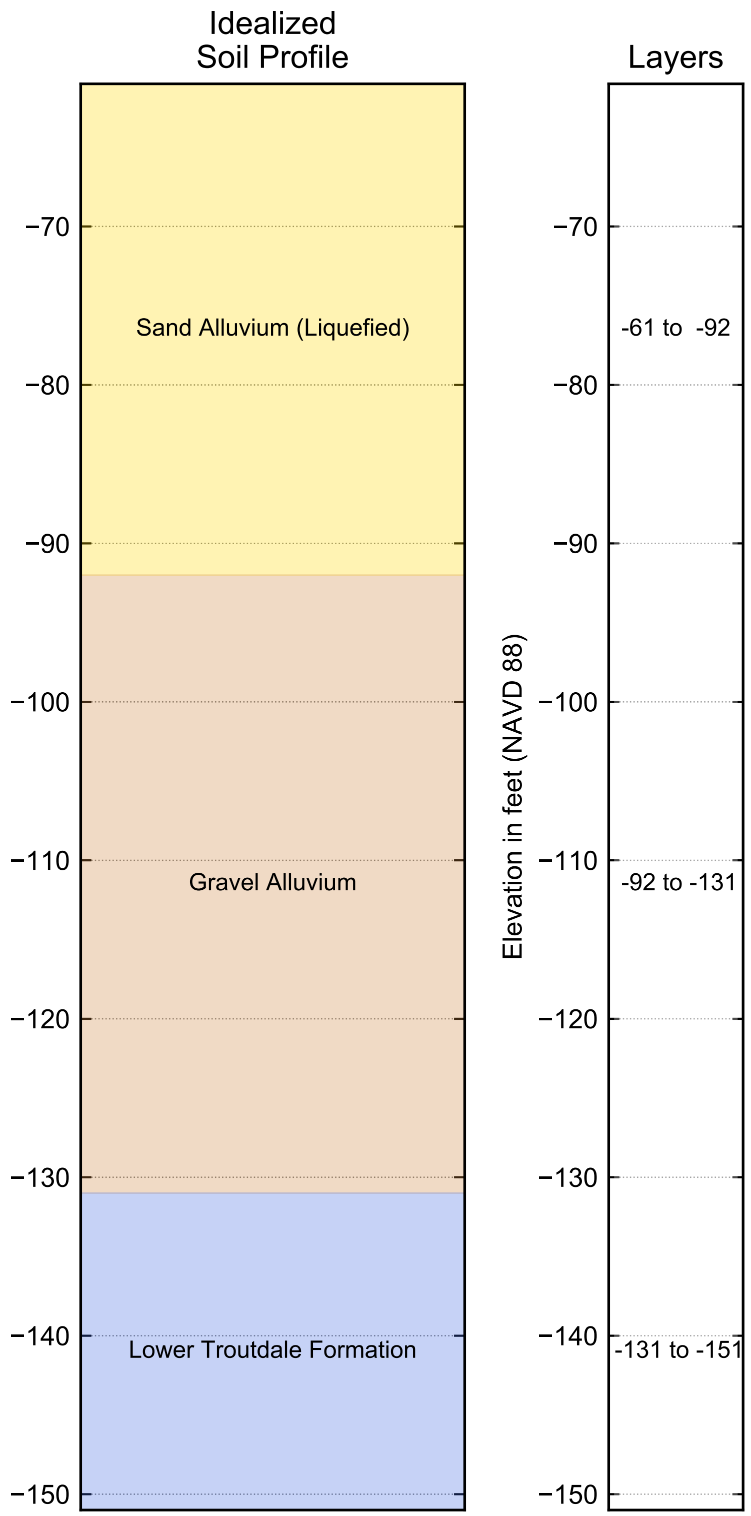
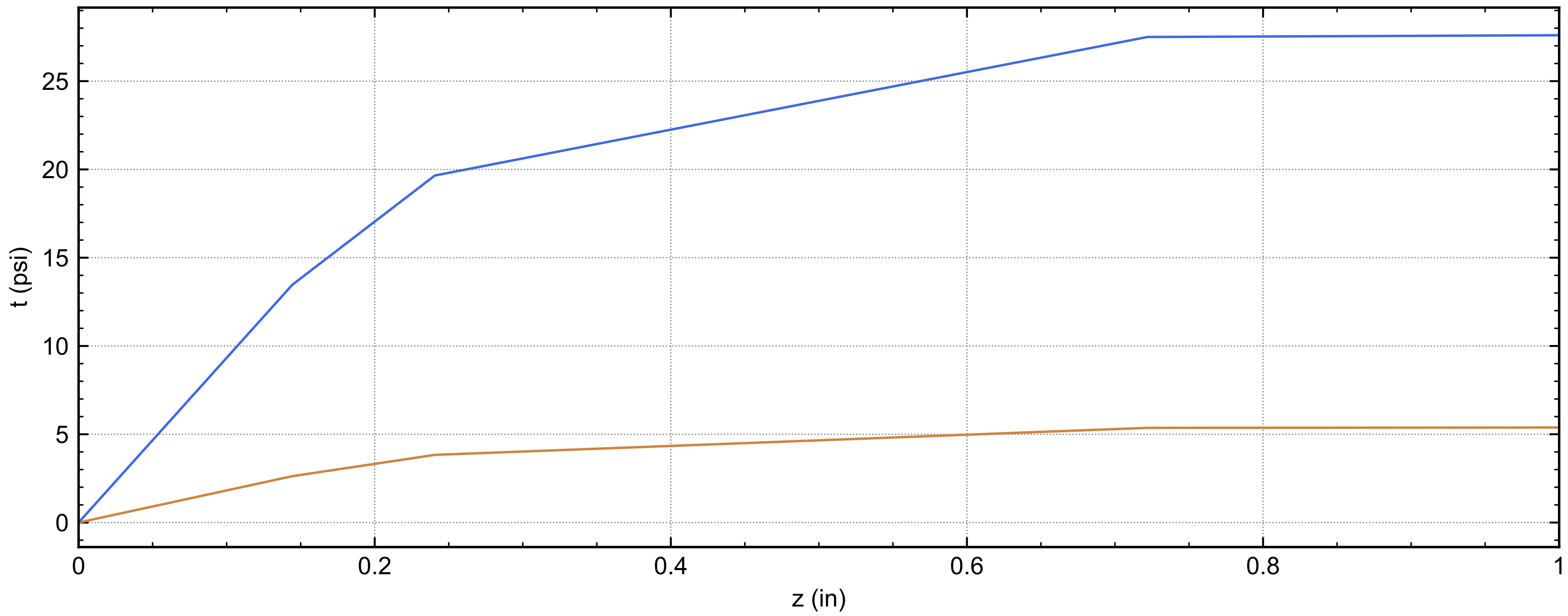
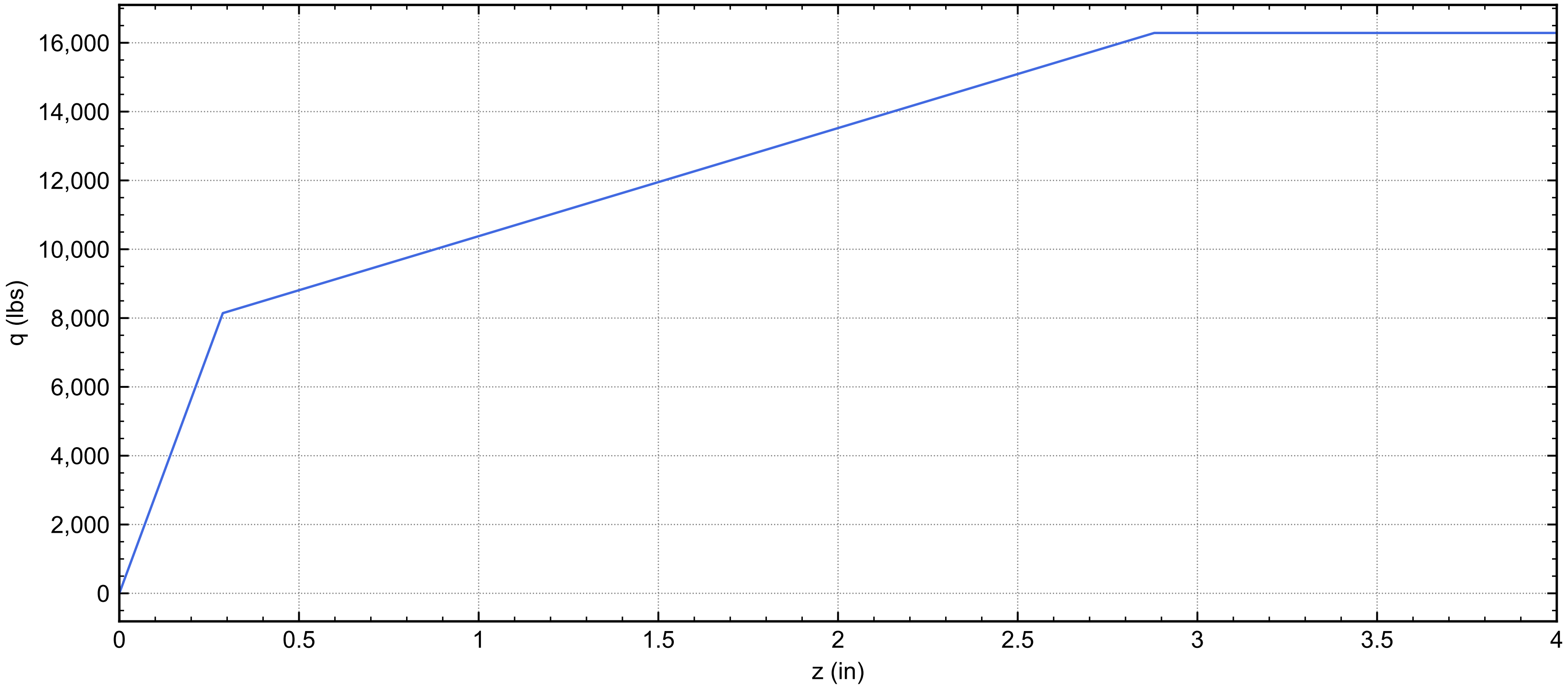
FIG. H17







Notes:
1. Foundation type: 12-ft-diameter Drilled Shaft
2. Assigned end bearing at Node -131.0 to -151.0: 16 kips
3. Number of layers = 3
    Number of t-z files: 2
    Number of q-z files: 1
4. The provided q-z spring is based on nominal resistance values. Spring values should be reduced according to AASHTO (2017) Table 10.8.3.6.3-1 based on shaft center-to-center spacing.

Burnside Bridge
Environmental Impact Study
Portland, Oregon

DRILLED SHAFT SPRINGS
EXTREME EVENT: LIQUEFIED CASE
BENTS 7 AND 8

August 2019 102636-001

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FIG. H18

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I:\EF\PDX\102000s\102636 Burnside Bridge\Analysis\Drilled Shaft Parameters\For Report\Replacement\Bents 8 - 9

Table H1 - L-Pile Parameters for Replacement Bents 1-3 Profile

Elevation			Recommended p-y Curve	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	10	Sand Alluvium	Sand (Reese)	120	32	40	32	40
10	-2	Sand Alluvium (Liquefiable)	Sand (Reese)	58	32	30	4	4
-2	-12	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-12	-20	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-20	-120	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H2 - Lateral Soil Displacement Profile at Replacement Bents 1-3

Depth (feet)	Displacement (inch)
0	0

Table H3 - L-Pile Parameters for Replacement Bent 4 Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	10	Sand Alluvium	Sand (Reese)	120	32	40	32	40
10	0	Sand Alluvium (Liquefiable)	Sand (Reese)	58	32	30	4	4
0	-11	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-11	-20	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-20	-50	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H4 - Lateral Soil Displacement Profile at Replacement Bent 4

Depth (feet)	Displacement (inch)
0	0

Table H5 - L-Pile Parameters for Replacement Bent **5** Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	10	Fill	Sand (Reese)	120	32	25	32	25
10	3	Sand Alluvium	Sand (Reese)	58	32	30	4	4
3	-17	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-17	-40	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-40	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H6 - Lateral Soil Displacement Profile at Replacement Bent **5**

Depth (feet)	Displacement (inch)
0	0

Table H7 - L-Pile Parameters for Replacement Bent **6** Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
35	10	Fill	Sand (Reese)	120	32	25	32	25
10	-23	Sand Alluvium	Sand (Reese)	58	32	30	32	30
-23	-70	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-70	-100	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-100	-115	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H8 - Lateral Soil Displacement Profile at Replacement Bent **6**

Depth (feet)	Displacement (inch)
0	0

Table H9 - L-Pile Parameters for Replacement Bents **7** and **8**

Elevation		Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To				Phi (deg)	k (pci)	Phi (deg)	k (pci)
-61	-92	Sand Alluvium	Sand (Reese)	53	30	38	2	2
-92	-131	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-131	-160	Lower Troutdale Formation	Sand (Reese)	78	44	125	44	125

NOTES:

1 P-y springs generated in FB-Multipier should be reduced according to AASHTO (2017) Table 10.7.2.4-1 based on shaft center-to-center spacing where applicable.

Table H10 - L-Pile Parameters for Replacement Bent 9 Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
-18	-108	Sand Alluvium	Sand (Reese)	53	30	38	30	38
-108	-128	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-128	-161	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225
-161	-180	Sandy River Mudstone	Sand (Reese)	78	44	125	44	125

Table H11 - Lateral Soil Displacement Profile at Replacement Bent 9

Depth (feet)	Displacement (inch)
0	0

Table H12 - L-Pile Parameters for Replacement Bent 10/**S10/N10** Profile

Elevation			Recommended p-y	Unit Weight, γ'	Static Case		Post-Seismic Case	
From	To	Soil Type	Curve Type	(pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	15	Fill	Sand (Reese)	120	32	25	32	25
15	10	Sand Alluvium	Sand (Reese)	115	30	40	30	40
10	-111	Sand Alluvium; below groundwater table	Sand (Reese)	53	30	30	30	30
-111	-133	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-133	-150	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H13 - Lateral Soil Displacement Profile at Replacement Bent 10/**S10/N10**

Depth (feet)	Displacement (inch)
0	0

Table H14 - L-Pile Parameters for Replacement Bent 11/**S11/N11** Profile

Elevation		Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To				Phi (deg)	k (pci)	Phi (deg)	k (pci)
32	13	Fill	Sand (Reese)	120	32	25	32	25
13	10	Sand Alluvium	Sand (Reese)	53	30	40	30	40
10	-102	Sand Alluvium; below groundwater table	Sand (Reese)	53	30	30	30	30
-102	-123	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-123	-145	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H15 - Lateral Soil Displacement Profile at Replacement Bent 11/**S11/N11**

Depth (feet)	Displacement (inch)
0	0

Table H16 - L-Pile Parameters for Replacement Bent **12/S12/N12** Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
37	10	Fill	Sand (Reese)	120	32	25	32	25
10	-92	Sand Alluvium	Sand (Reese)	53	30	30	30	30
-92	-110	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-110	-145	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H17 - Lateral Soil Displacement Profile at Replacement Bent **12/S12/N12**

Depth (feet)	Displacement (inch)
0	0

Table H18 - L-Pile Parameters for Replacement Bent 13/S13/N13/N14 Profile

Elevation			Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To	Soil Type			Phi (deg)	k (pci)	Phi (deg)	k (pci)
50	28	Fill	Sand (Reese)	120	32	25	32	25
28	10	Gravel Alluvium	Sand (Reese)	125	41	225	41	225
10	7	Gravel Alluvium; below groundwater table	Sand (Reese)	125	41	125	41	125
7	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H19 - Lateral Soil Displacement Profile at Replacement Bent 13/S13/N13/N14

Depth (feet)	Displacement (inch)
0	0

Table H20 - L-Pile Parameters for Replacement Bent 14/S14/N15 Profile

Elevation		Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To				Phi (deg)	k (pci)	Phi (deg)	k (pci)
50	22	Fill	Sand (Reese)	120	32	25	32	25
22	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table H21 - Lateral Soil Displacement Profile at Replacement Bent 14/S14/N15

Depth (feet)	Displacement (inch)
0	0

APPENDIX I: DRILLED SHAFT PARAMETERS FOR LONG-SPAN ALTERNATIVE

Appendix I

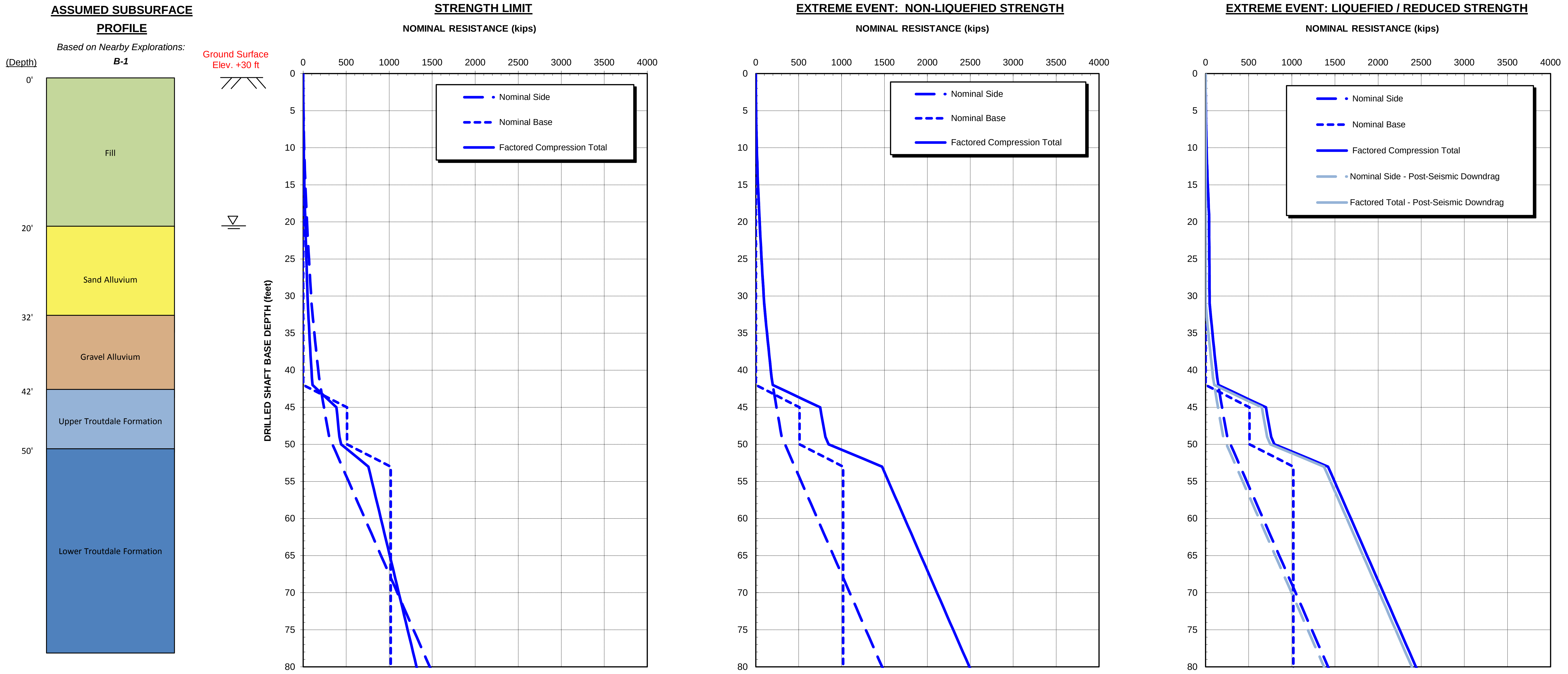
Drilled Shaft Parameters for Long-span Alternative

Figures

Figures I1 through I7: Axial Resistance Curves for Bents 1 through 5 and Bents 8 through 10

Figure I8: Summary of Soil Springs for Bents 6 and 7

Tables I1 through I13: LPILE Parameters for Bents 1 through 10



ASSUMED SUBSURFACE PROFILE

STRENGTH LIMIT

EXTREME EVENT: NON-LIQUEFIED STRENGTH

EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 70 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 1
3-FOOT DIAMETER DRILLED SHAFT

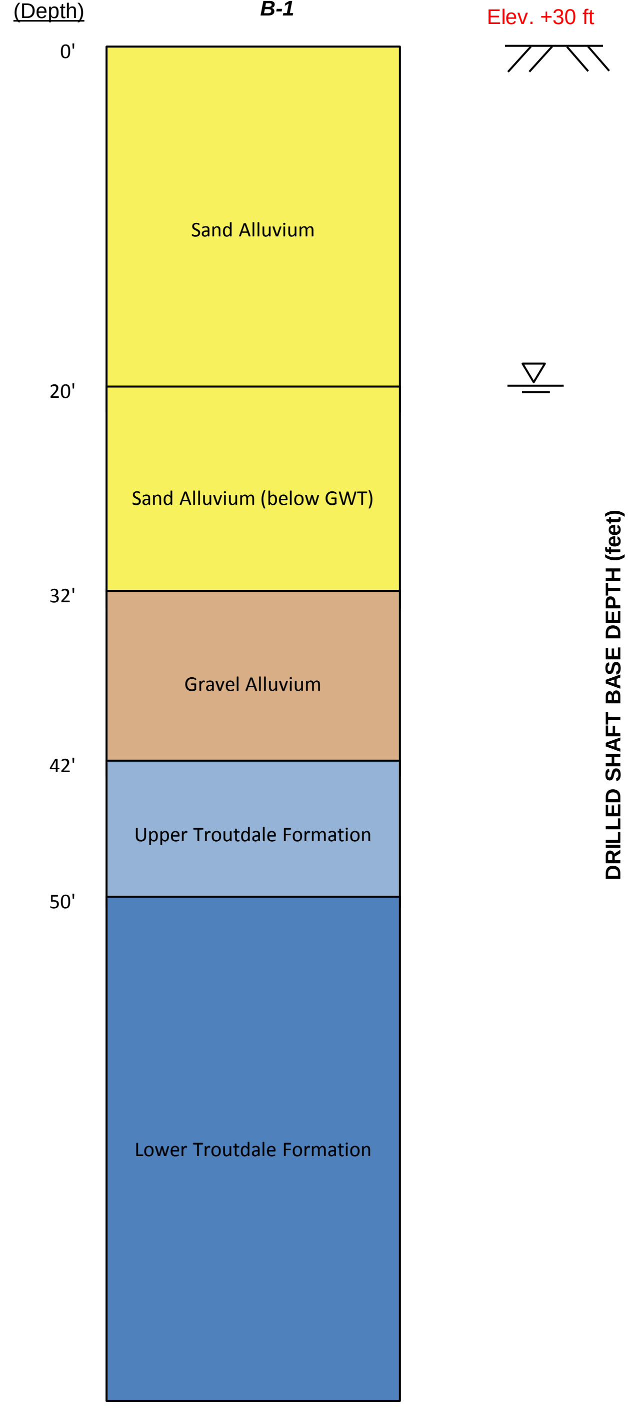
August 2019 102636-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

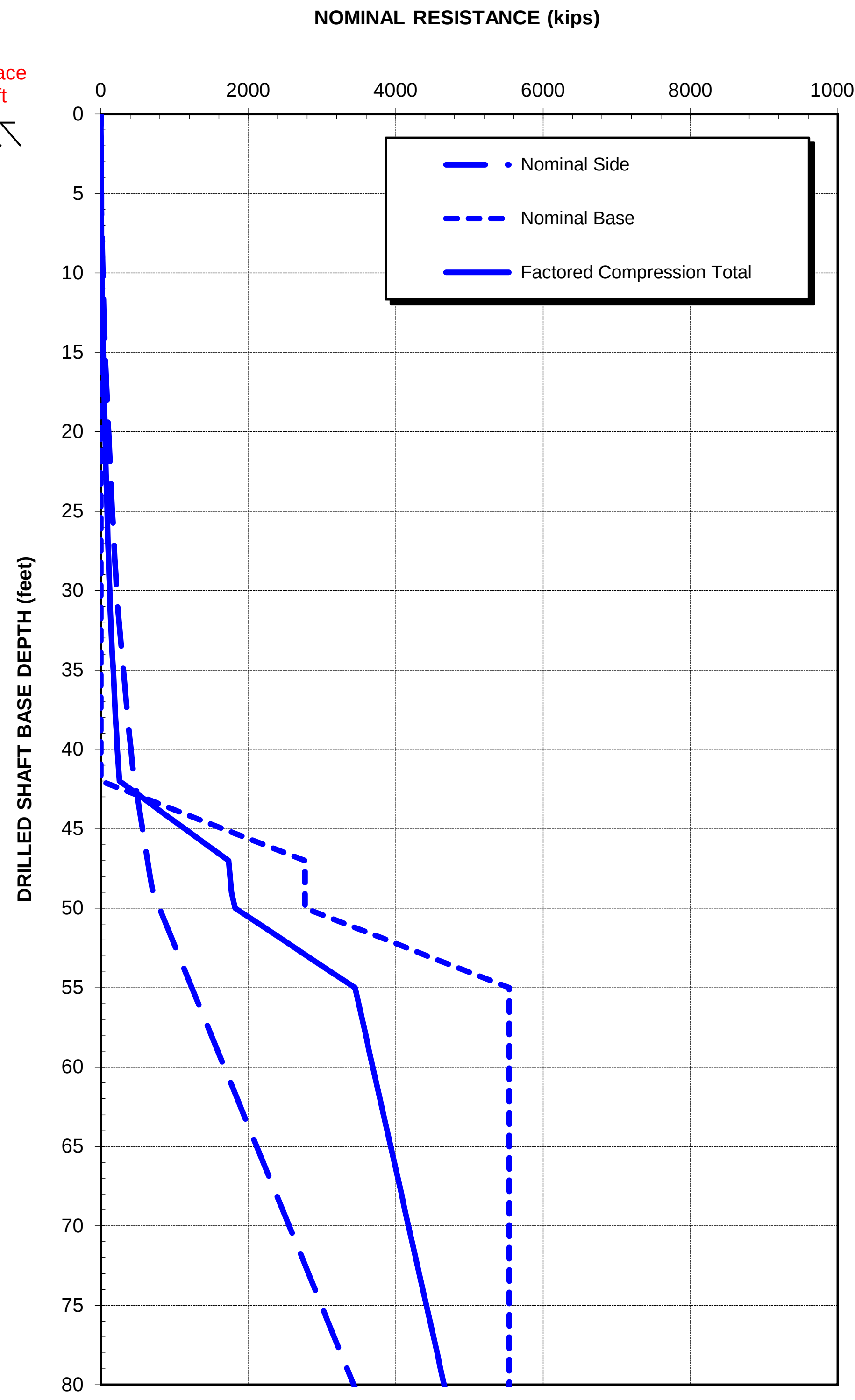
FIG. I1

ASSUMED SUBSURFACE PROFILE

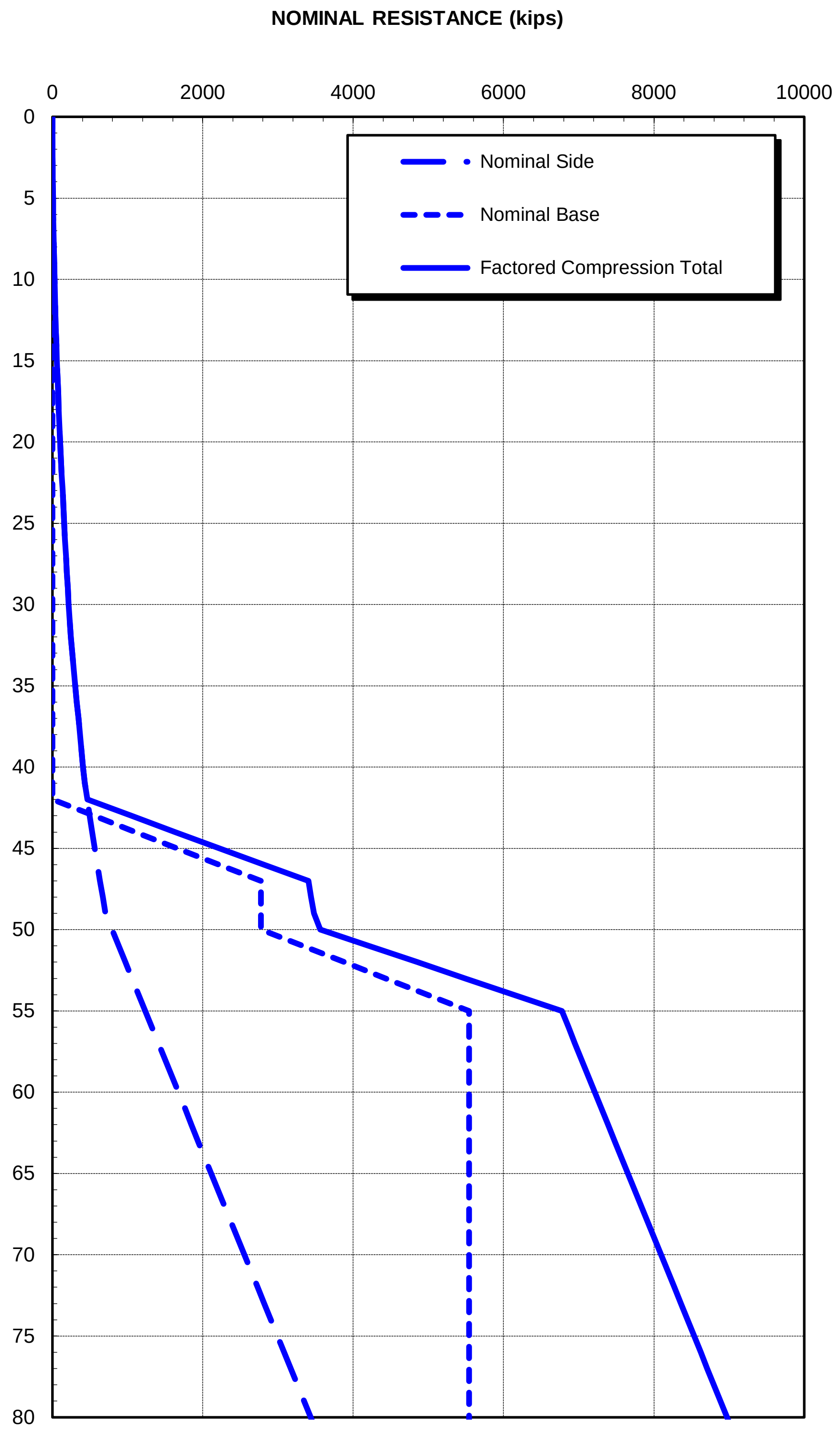
Based on Nearby Explorations:
B-1



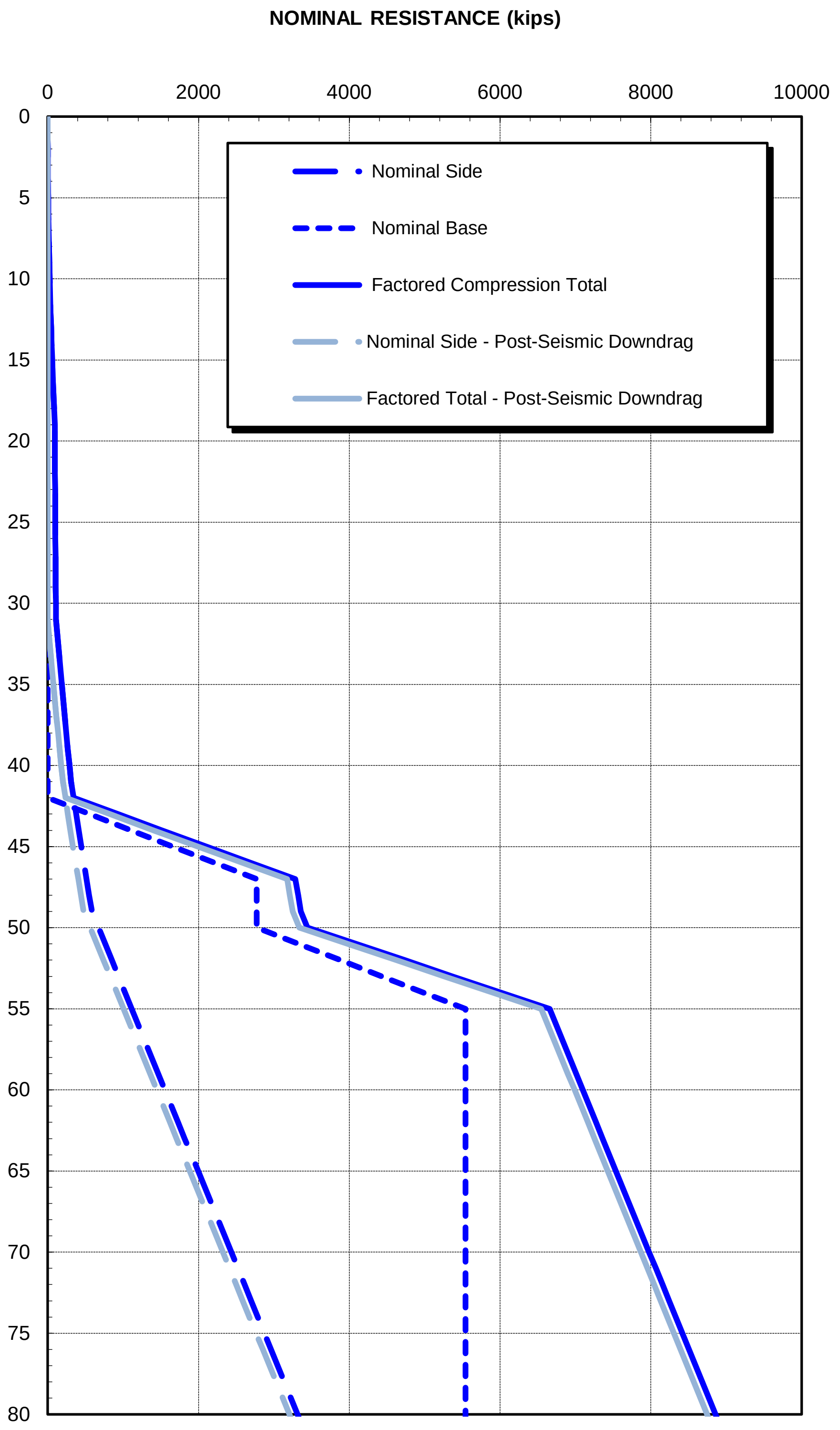
STRENGTH LIMIT



EXTREME EVENT: NON-LIQUEFIED STRENGTH



EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 160 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENTS 2 AND 3
7-FOOT DIAMETER DRILLED SHAFT

August 2019
102636-001

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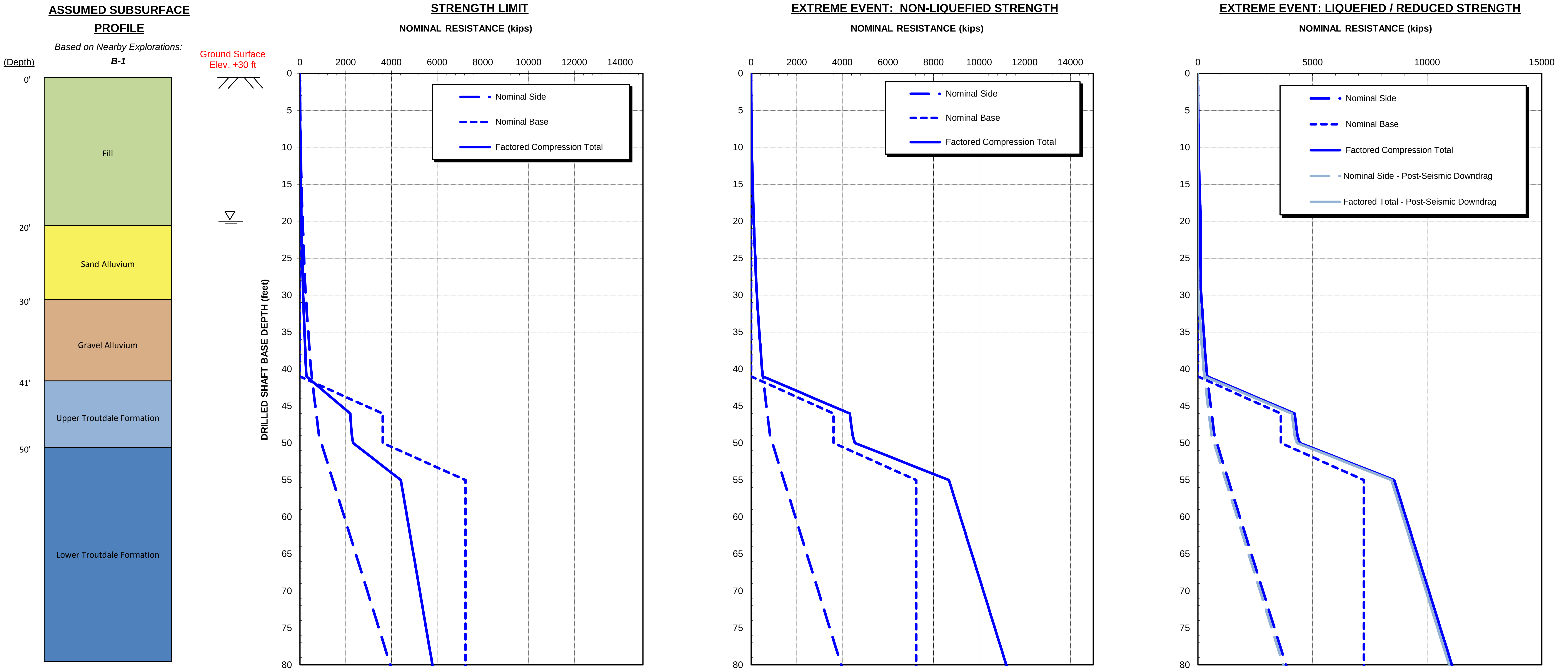
FIG. I2



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 170 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 4
8-FOOT DIAMETER DRILLED SHAFT

August 2019
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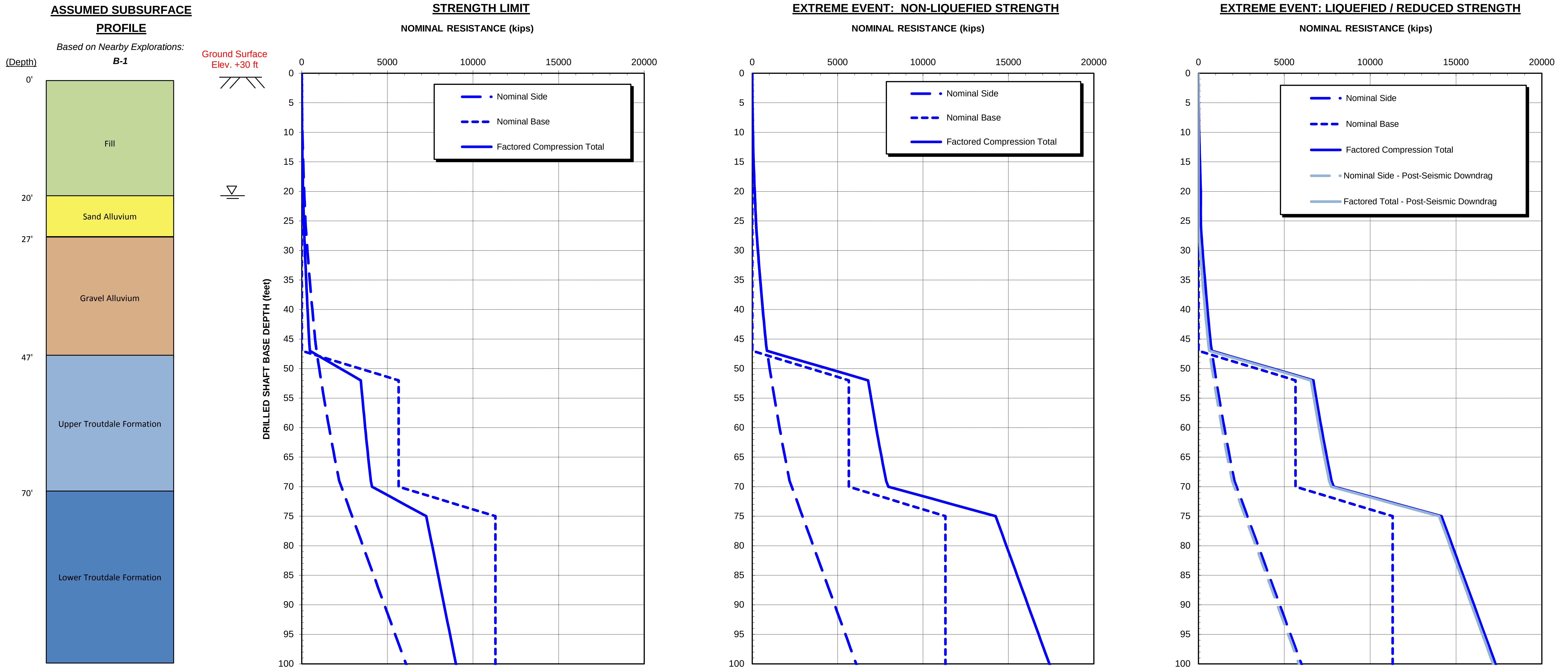
FIG. I3



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 180 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 5
10-FOOT DIAMETER DRILLED SHAFT

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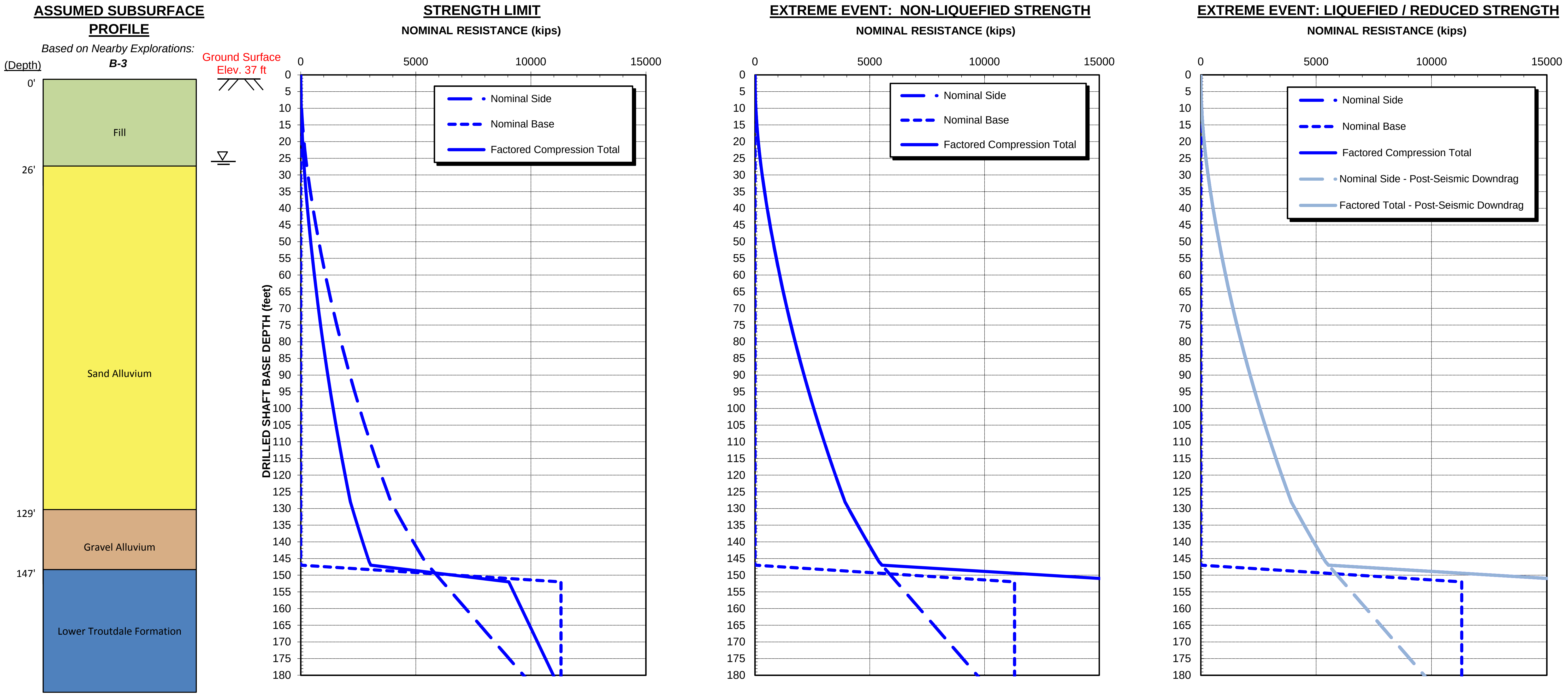
FIG. I4



ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations: ***B-3***

STRENGTH LIMIT

EXTREME EVENT: NON-LIQUEFIED STRENGTH

EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

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ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 12/S12
10-FOOT DIAMETER DRILLED SHAFT

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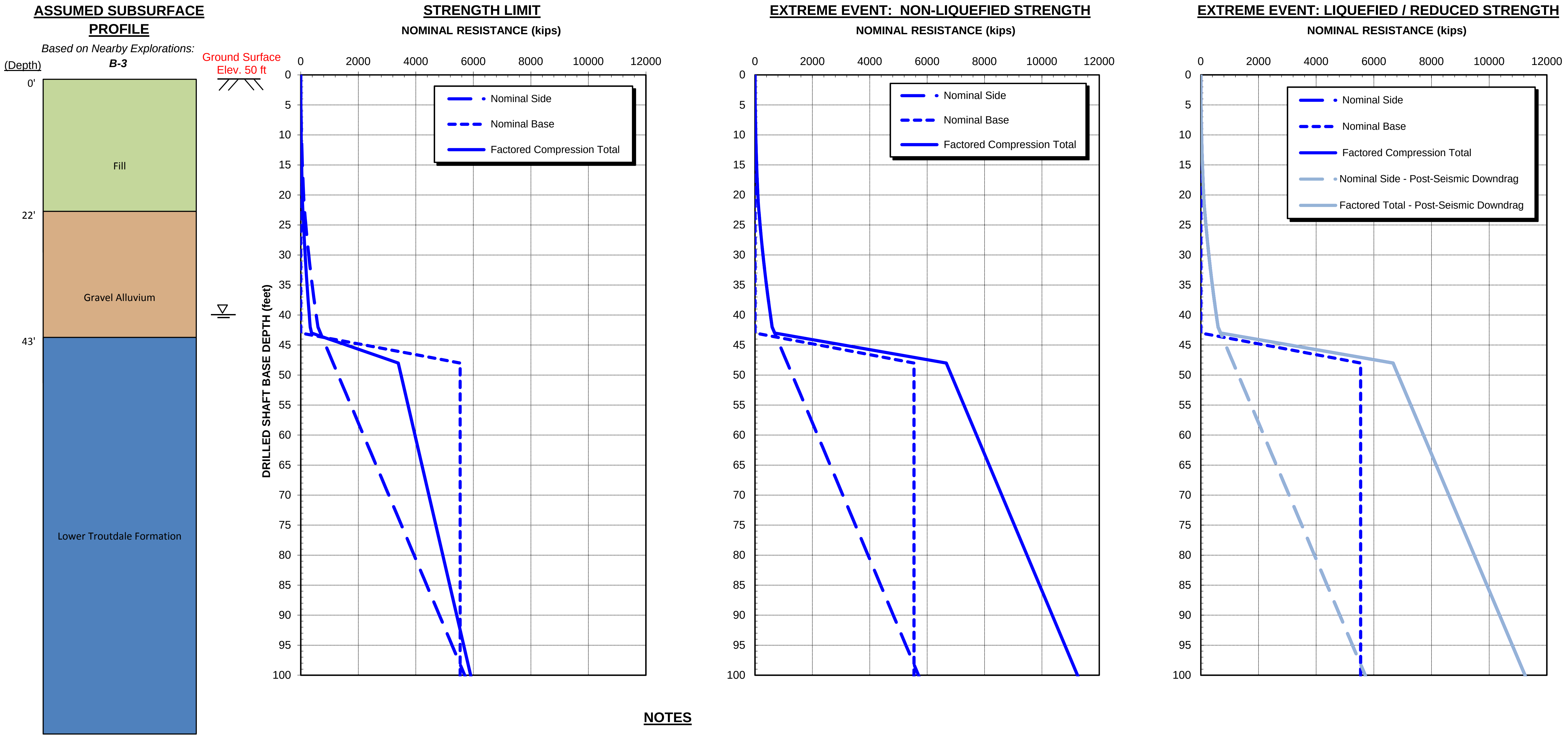
FIG. I5



NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 13/S13
7-FOOT DIAMETER DRILLED SHAFT

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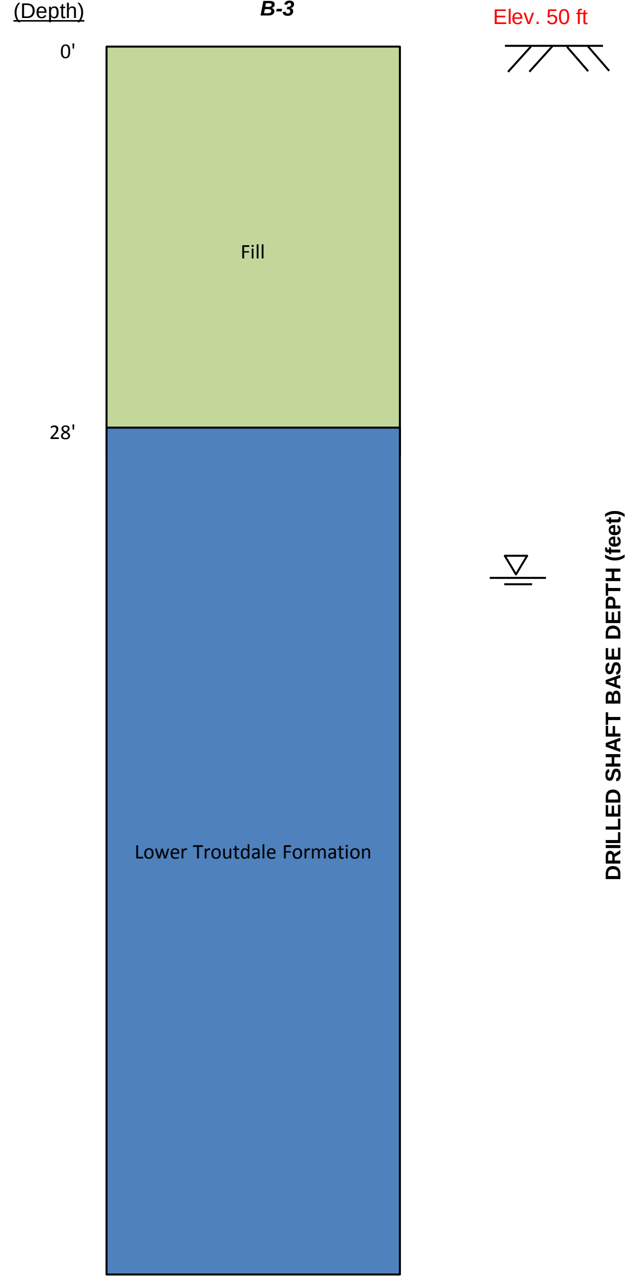
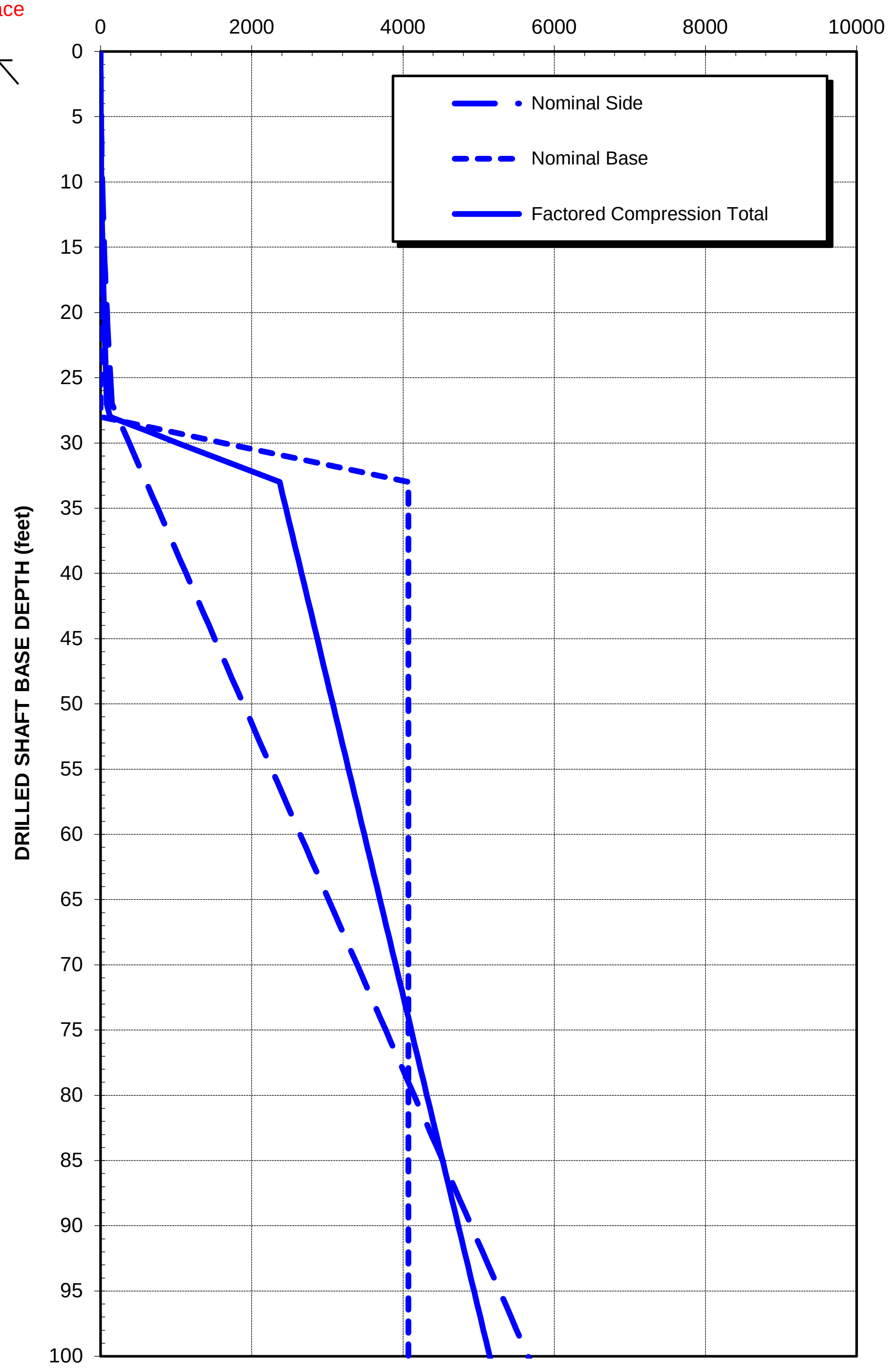
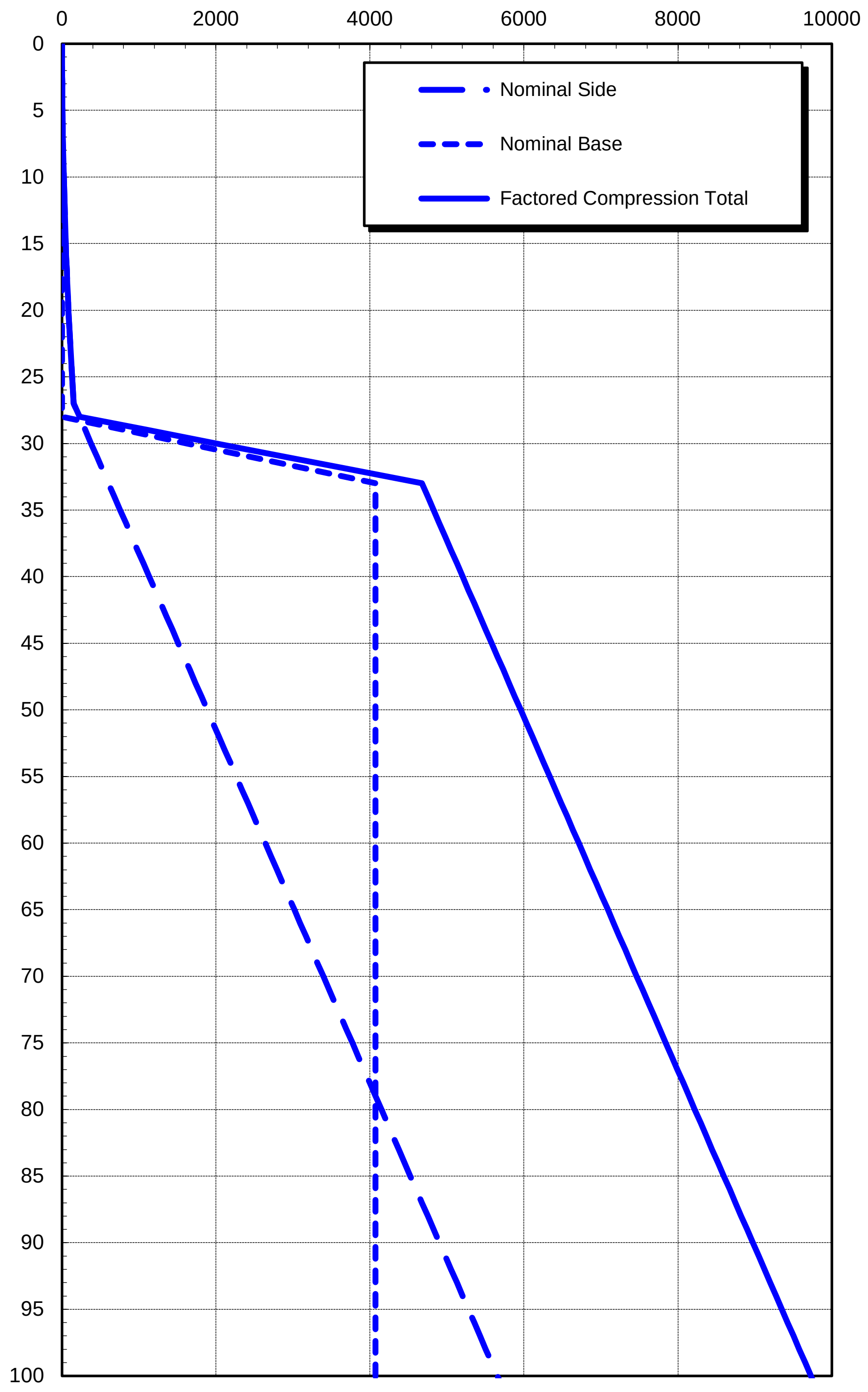
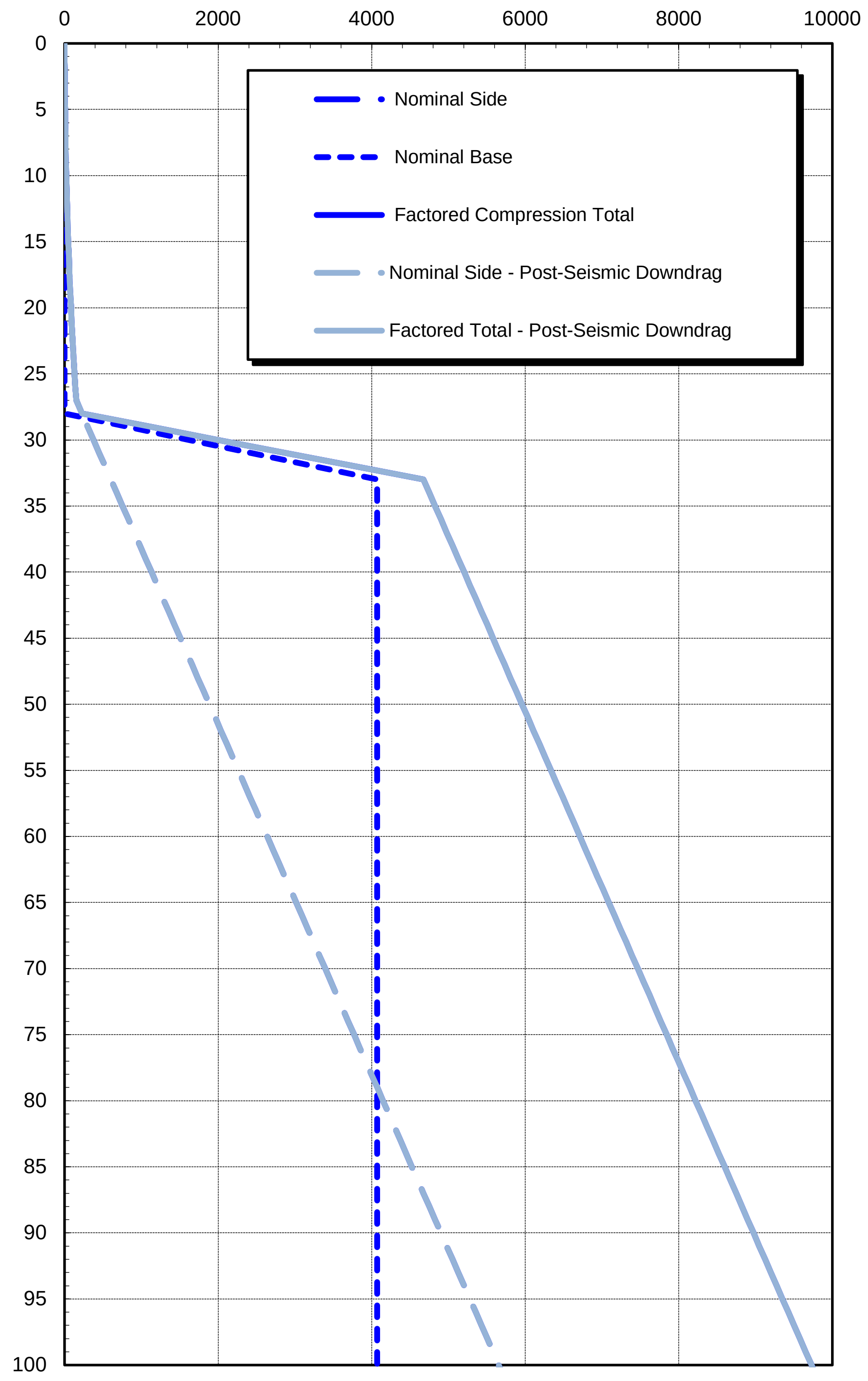
FIG. I6


ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:


STRENGTH LIMIT


EXTREME EVENT: NON-LIQUEFIED STRENGTH


EXTREME EVENT: LIQUEFIED / REDUCED STRENGTH

NOTES

1. The analyses were performed based on guidelines included in AASHTO 2015, the ODOT GDM and local experience. The analyses are based on a single shaft or a group of shafts at a center to center spacing greater than or equal to 4 diameters.
2. Factored total shaft compression resistance shown on plots is determined by adding nominal side and base resistances multiplied by the corresponding resistance factor:

	Strength	Extreme: Non-Liquefied	Extreme: Liquefied/Reduced
Side Resistance - Compression	0.55	1.0	1.0
Side Resistance - Uplift	0.45	0.8	0.8
Base Resistance	0.50	1.0	1.0

3. **The resistance factors should be reduced by 20% for a non-redundant single shaft supporting a bridge pier.**
4. Estimated shaft resistance assumes permanent casing will be installed to the top of Troutdale Formation.
5. Estimated shaft resistance for the Extreme Event: Liquefied/Reduced Strength case is based on estimated residual shear strengths of soils.
6. The estimated post-seismic downdrag force is 0 kips. Post-seismic downdrag forces should be considered with the post-seismic downdrag resistance curves on the Extreme Event: Liquefied/Reduced Strength plot.

Burnside Bridge Environmental Impact Study
Portland, Oregon

**ESTIMATED AXIAL SHAFT RESISTANCE
REPLACEMENT BENT 14/S14
3-FOOT DIAMETER DRILLED SHAFT**

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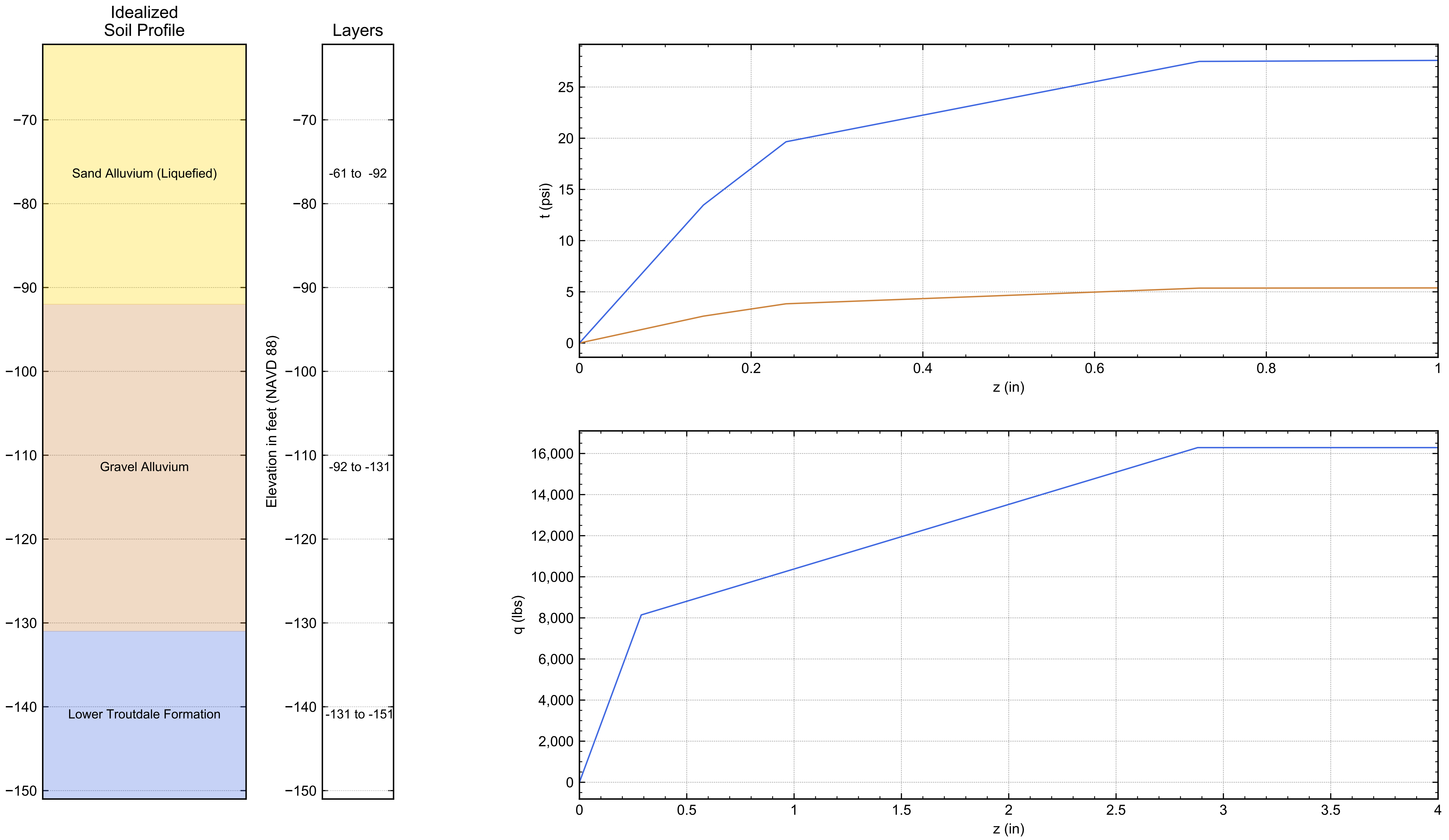
FIG. I7



Notes:
1. Foundation type: 12-ft-diameter Drilled Shaft
2. Assigned end bearing at Node -131.0 to -151.0: 16 kips
3. Number of layers = 3
    Number of t-z files: 2
    Number of q-z files: 1
4. The provided q-z spring is based on nominal resistance values.
   Spring values should be reduced according to AASHTO (2017)
   Table 10.8.3.6.3-1 based on shaft center-to-center spacing.

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**DRILLED SHAFT SPRINGS
EXTREME EVENT: LIQUEFIED CASE
BENTS 6 AND 7**

August 2019 102636-001

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FIG. I8

\\EF\PDX\102000s\102636 Burnside Bridge\Analysis\Drilled Shaft Parameters\For Report\Replacement\Bents 8 - 9

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Table I1 - L-Pile Parameters for Replacement Bents 1-3 Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	10	Sand Alluvium	Sand (Reese)	120	32	40	32	40
10	-2	Sand Alluvium (Liquefiable)	Sand (Reese)	58	32	30	4	4
-2	-12	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-12	-20	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-20	-120	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table I2 - Lateral Soil Displacement Profile at Replacement Bents 1-3

Depth (feet)	Displacement (inch)
0	0

Table I3 - L-Pile Parameters for Replacement Bent 4 Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	10	Sand Alluvium	Sand (Reese)	120	32	40	32	40
10	0	Sand Alluvium (Liquefiable)	Sand (Reese)	58	32	30	4	4
0	-11	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-11	-20	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-20	-50	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table I4 - Lateral Soil Displacement Profile at Replacement Bent 4

Depth (feet)	Displacement (inch)
0	0

Table I5 - L-Pile Parameters for **Replacement** Bent 5 Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
30	10	Fill	Sand (Reese)	120	32	25	32	25
10	3	Sand Alluvium	Sand (Reese)	58	32	30	4	4
3	-17	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-17	-40	Upper Troutdale Formation	Sand (Reese)	68	44	125	44	125
-40	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table I6 - Lateral Soil Displacement Profile at **Replacement** Bent 5

Depth (feet)	Displacement (inch)
0	6
22	4
26	0

Table I7 - L-Pile Parameters for Replacement Bents 6 and 7

Elevation			Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To	Soil Type			Phi (deg)	k (pci)	Phi (deg)	k (pci)
-61	-92	Sand Alluvium	Sand (Reese)	53	30	38	2	2
-92	-131	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-131	-160	Lower Troutdale Formation	Sand (Reese)	78	44	125	44	125

NOTES:

1 P-y springs generated in FB-Multipier should be reduced according to AASHTO (2017) Table 10.7.2.4-1 based on shaft center-to-center spacing where applicable.

Table I8 - L-Pile Parameters for Replacement Bent 8 Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
37	10	Fill	Sand (Reese)	120	32	25	32	25
10	-92	Sand Alluvium	Sand (Reese)	53	30	30	30	30
-92	-110	Gravel Alluvium	Sand (Reese)	63	41	125	41	125
-110	-145	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table I9 - Lateral Soil Displacement Profile at Replacement Bent 8

Depth (feet)	Displacement (inch)
0	0

Table I10 - L-Pile Parameters for Replacement Bent 9 Profile

Elevation					Static Case		Post-Seismic Case	
From	To	Soil Type	Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Phi (deg)	k (pci)	Phi (deg)	k (pci)
50	28	Fill	Sand (Reese)	120	32	25	32	25
28	10	Gravel Alluvium	Sand (Reese)	125	41	225	41	225
10	7	Gravel Alluvium; below groundwater table	Sand (Reese)	125	41	125	41	125
7	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table I11 - Lateral Soil Displacement Profile at Replacement Bent 9

Depth (feet)	Displacement (inch)
0	0

Table I12 - L-Pile Parameters for Replacement Bent 10 Profile

Elevation			Recommended p-y Curve Type	Unit Weight, γ' (pcf)	Static Case		Post-Seismic Case	
From	To	Soil Type			Phi (deg)	k (pci)	Phi (deg)	k (pci)
50	22	Fill	Sand (Reese)	120	32	25	32	25
22	-100	Lower Troutdale Formation	Sand (Reese)	78	44	225	44	225

Table I13 - Lateral Soil Displacement Profile at Replacement Bent 10

Depth (feet)	Displacement (inch)
0	0

IMPORTANT INFORMATION

Important Information

About Your Geotechnical/Environmental 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 that 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 upon 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