APPENDIX D

SEISMIC RETROFIT REPORT

App-A: EQRB Seismic Deficiency Plans
App-B: EQRB Seismic Design Criteria
App-C: EQRB Geotechnical Report
App-D: EQRB Seismic Site Utilities
App-E: EQRB Concept Retrofit Plans
App-F: EQRB Seismic Retrofit Cost Estimate

Seismic Retrofit Report

Multnomah County | Earthquake Ready
Burnside Bridge Project

Contract# 41180 (MC#4400002566)
HDR Project #10040689

Portland, Oregon

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Earthquake Ready Burnside Bridge Project - Seismic Retrofit Report

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Acronyms

AASHTO	American Association of State Highway and Transportation Officials
ARS	acceleration response spectra
BDDM	Bridge Design and Drafting Manual
BES	Bureau of Environmental Services
C/D	Capacity-to-demand
CEI / CA	Construction Engineering and Inspection / Contract Administration
CFR	Code of Federal Regulations
CIP	Capital Improvement Plan
City	City of Portland
Criteria	Seismic Design Criteria
CSO	Combined Sewer Overflow
CSZ	Cascadia Subduction Zone
County	Multnomah County, Oregon
CV	Constructed value
EIS	Environmental Impact Statement
EQRB	Earthquake Ready Burnside Project
FHWA	Federal Highway Administration
FLAC	Fast Lagrangian Analysis of Continua
GDM	Geotechnical Design Manual
HMI	human-machine interface
I-5, I-84	Interstate 5, Interstate-84, etc
LRFD	Load and Resistance Factor Design
NAVD	North American Vertical Datum
NBI	National Bridge Inventory
NEPA	National Environmental Policy Act
ODOT	Oregon Department of Transportation
PDC	Portland Development Commission
PE	Preliminary Engineering
PLC	programmable logic controller
Project	Earthquake Ready Burnside Bridge Project
PS&E	Plans, Specifications, and Estimate
PTFE	polytetrafluoroethylene
RCDG	reinforced concrete deck girder
SRC	Seismic Review Committee
TCE	temporary construction easement

TP&DT	Temporary Protection and Direction of Traffic
TriMet	Tri-County Metropolitan Transportation District of Oregon
UPRR	Union Pacific Railroad
UPS	uninterruptible power supply

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Seismic Retrofit Report Executive Summary

Objectives

The purpose of the Project's Seismic Retrofit Study includes the following:

- Identify the seismic vulnerabilities of the existing bridge.
- Develop feasible concepts for seismic retrofit alternatives.
- Perform a conceptual level seismic retrofit analysis of the existing Burnside Bridge. Develop a list of feasible of Seismic Retrofit and Hybrid alternative strategies, including any necessary rehabilitation measures, to withstand major seismic events as defined in the Project's Seismic Design Criteria.
- Develop feasibility-level project costs, risks, and impacts for feasible Seismic Retrofit and Hybrid alternatives.

Seismic Vulnerabilities

The major seismic vulnerabilities that were identified include the following:

- Liquefiable soil under both west and east approach structures.
- Insufficient footing sizes and pile depths to resist design-level seismic loadings.
- Unreinforced or under-reinforced spread footings, pile caps, pier columns, and walls. The reinforcing details do not conform to current seismic design requirements.
- Under-reinforced stringers, floor beams, and girder column connections. The reinforcing details do not conform to current seismic design requirements.
- Reinforced concrete girders and steel trusses lack proper longitudinal and transverse restrainers or seating lengths.
- Fixed steel trusses and bascule leaves do not have sufficient lateral load transfer capacities.
- Rocker-type bearings are seismically vulnerable and will become unstable.
- Insufficient strength of support frames and anchors for bascule leaves and counterweights.
- Bascule span center lock has insufficient lateral load transfer capacity.
- Bascule span machinery will be damaged and become inoperable during a design-level seismic event.
- Portions of adjacent building structures are seismically vulnerable and may damage the bridge.
- Highway ramp structures under the east approach spans are seismically vulnerable and may damage the bent columns.

Seismic Analysis and Performance Requirements

The project seismic design criteria identified two levels of seismic design performance requirements because of the lifeline designation of the Burnside Bridge:

- Full operation under a Cascadian Subduction Zone seismic event
- Limited operation under a 1000-year return period seismic event

This two-level performance requirement poses significant challenges to the Burnside Bridge, especially on the bascule span. To meet these performance requirements, the bridge must have adequate strength and ductility, and must be capable of tolerating seismic displacements.

For a bascule bridge such as the Burnside Bridge, the existing bascule leaf machinery driving system can tolerate only very small displacements. At a conceptual design level, the analysis model is not capable of precisely predicting such small relative movements within a fraction of an inch. However, conceptual-level analysis has provided predictable ranges and the trends, which are used in this conceptual design.

Concepts for Seismic Retrofit, Widening, and Hybrid Solutions

Chapters 8 and 9 describe the bridge seismic retrofit and widening alternatives. Several alternatives that include seismic retrofit of the existing structures only, or seismic retrofit combined with bridge widening, were studied.

Replacing Spans 20 to 25 of the East Approach Bridge with a longer span structure was determined to be more cost effective and feasible than retrofitting the existing piers and foundations. This is because of the anticipated need for shutting down I-5 for an extended period during construction, as well as required construction access near the UPRR tracks.

Four of these alternatives [4a.1 (Pure Retrofit Alternative); 4b.1, 4c.1, and 4d.1 (Hybrid Retrofit/replacement Alternatives)] are for seismic retrofit of the existing bridge without a widening. The other four alternatives [4a.2 (Pure Retrofit Alternative), 4b.2, 4c.2, and 4d.2 (Hybrid Retrofit/replacement Alternatives)] are for the bridge seismic retrofit combined with a widening for a constant width of 110 feet.

The primary features of four representative alternatives, including the pros and cons, are summarized in the table below. The four were selected as the most reasonable and feasible of the seismic retrofit options because they span a reasonable and feasible range from the of bridge replacement as part of the seismic retrofit effort. They also include widened and unwidened options as a basis of comparison against full replacement bridge alternatives.

Table ES-1. Summary of Representative Alternatives for Seismic Retrofit, Widening, and Partial Bridge Replacement

Alternative	Description	Primary Advantages	Primary Disadvantages	Project Cost ($million)
1 Seismic Retrofit	Pure seismic retrofit of all bridge members (0% bridge replacement)	N/A	Not feasible because it requires the prolonged removal of multiple I-5 mainline and ramp bridges to construct the retrofit.	N/A – Not Feasible
4a.1 Hybrid (Replace / Retrofit) Seismic Retrofit	Includes replacing all highway spans (20 to 23) (10% bridge replacement)	Shortest of all Hybrid alternatives	Versus Alt 4b.1, not reasonable because it requires a very high-cost, high-risk railroad shoofly within the UPRR. It may not be permittable.	N/A – Not Reasonable versus Alt 4b.1
4b.1a Hybrid (Replace / Retrofit) Seismic Retrofit	Includes replacing all highway and RR spans (20 to 24) (13% bridge replacement)	Lowest estimated cost among the four alternatives. By replacing Spans 20 to 24, the constructability challenges at Bents 21, 22, 23, and 24 are minimized because of the close proximity to the highway structures and UPRR tracks.	A narrow retrofitted bridge (86 feet wide)	$ 688
4b.2a Hybrid Retrofit with Widening	Includes replacing all highway and RR spans (20 to 24) (18% bridge replacement)	In addition to above for Alternative 4b.1a, this will be widened to a constant 110-foot bridge.	Highest estimated cost amongst alternatives 4b.1 through 4c.1	$ 844
4c.2a Hybrid (Replace / Retrofit) with Widening	Includes replacing East Approach spans (20 to 27) (30% bridge replacement)	In addition to above for Alternative 4b.2a, by replacing additional spans (from Span 20 to 27), the retrofitted and widened bridge will have more spans with new structures and foundations without a cost increase due to the anticipated cost savings from a simplified construction sequence.	Constructability challenges due to the proximity of existing buildings along the East Approach ROW limits	$1.01B

Table ES-1. Summary of Representative Alternatives for Seismic Retrofit, Widening, and Partial Bridge Replacement

Alternative	Description	Primary Advantages	Primary Disadvantages	Project Cost ($million)
4d.2 Hybrid (Replace / Retrofit) with Widening	Includes replacing Main and East Approach spans (14 to 27) (67% bridge replacement)	In addition to above for Alternative 4c.2a, by replacing the existing structures from Span 14 to Span 27, over the anticipated soil liquefaction area, this retrofitted and widened structure has the most resiliency after a design-level earthquake.	Leaves short portions of the existing approach bridges, requiring additional costs to maintain; Could be the highest cost of the four Hybrid alternatives investigated in detail.	N/A – Not Reasonable versus Alt 4c.2a

The major work items for the seismic retrofit of the existing bridge, and for a combined retrofit and widening of the bridge, are identified below. The spans within the East Approach Bridge that require a replacement due to the infeasibility of a retrofit are also noted below.

Table ES-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit ,Widen, or Replace
Structural			
West Approach Bridge: Bent 1 (West Abut)	Abut. Strengthening		Same
West Approach Bridge: Bents 2-16	Girder Restrainers and Strengthening		Same
	Floor Beam Strengthening		Same
	Column Jacketing		Same
	Footing Enlargement		Same
West Approach Bridge: Bents 17-19	Girder Restrainers and Strengthening	+	Add Girders
	Floor Beam Strengthening	+	Widen
	Column Jacketing	+	Add New Columns
	Footing Enlargement	+	Add Grade Beam
	8-foot Dia Drilled Shafts	+	Deeper Shafts
Main River Bridge: Pier 1	Relocation of Force Mains		Same
	6-foot Dia Shafts	+	Add Shafts
	Pile Cap Enlargement	+	Wider Cap
	Harbor Wall Reconstruction		Same

Table ES-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit ,Widen, or Replace
Main River Bridge: West Truss Span	Pier Column Strengthening	+	Wider Column
	Bearing Replacement	+	Add Bearings
	Lateral Load Member Strengthening	+	Add Two Trusses
	Connection Retrofit		Same
Main River Bridge: Pier 2	10-foot Dia Drilled Shafts	+	Add Shafts
	Pile Cap Enlargement	+	Wider Cap
	Adding Corner Columns	+	Larger Corner Columns
	Pier Wall and House Strengthening	+	Replace Existing Pier Walls
	Support Pedestal Strengthening	+	Wider Pedestal
	Pit Deck Bearing Retrofit	+	Add Bearings
	Trunnion Frame Strengthening	+	Add 2nd Pair Frames
	Trunnion Frame Anchorage Strengthening	+	More Anchorage
	Counterweight Frame Strengthening	+	Widen Counterweight
	Install Lateral Restrainers		Same
	Live Load Shoe Retrofit	+	Add Two Live Load Shoes
Main River Bridge: Pier 3	10-foot Dia Drilled Shafts	+	Add Shafts
	Pile Cap Enlargement	+	Wider Cap
	Adding Corner Columns	+	Larger Corner Columns
	Pier Wall & House Strengthening	+	Replace Existing Pier Walls
	Support Pedestal Strengthening	+	Wider Pedestal
	Pit Deck Bearing Retrofit	+	Add Bearings
	Trunnion Frame Strengthening	+	Add 2nd Pair Frames
	Trunnion Frame Anchorage Strengthening	+	More Anchorage
	Counterweight Frame Strengthening	+	Widen Counterweight
	Install Lateral Restrainers	+	Same
	Live Load Shoe Retrofit	+	Add Two Live Load Shoes
Main River Bridge: Bascule Leaves	Lateral Load Member Strengthening	+	Add Two Trusses
	Connection Retrofit		Same
	Center Lock Retrofit		Replace Center Lock

Table ES-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit ,Widen, or Replace
Main River Bridge: East Truss Span	Lateral Load Member Strengthening	+	Add Two Trusses
	Connection Retrofit		Same
Main River Bridge: Pier 4	6-foot Dia Shafts	+	Add Shafts
	Micropiles	+	Add Micropiles
	Pile Cap Enlargement	+	Wider Cap
	Pier Column Strengthening	+	Wider Column
	Bearing Replacement	+	Add Bearings
East Approach Bridge: Spans 20-24	Replace with three New Spans	+	Replace w Wider Spans
East Approach Bridge: Bents 25-28	10-foot Dia Drilled Shafts	+	Deeper Shafts
	Pile Cap and Grade Bean Extension	+	Larger Caps
	Partial Infill Wall		Same
	Column Strengthening	+	Add Columns
	Floor Beam Strengthening	+	Widen
	Bearing Replacement	+	Add Bearings
	Steel Girder Strengthening	+	Add Girders
East Approach Bridge: Bents 29-34	Girder Restrainers and Strengthening		Same
	Floor Beam Strengthening		Same
	Column Jacketing		Same
	Footing Enlargement		Same
East Approach Bridge: Bent 35 (East Abut)	Abut. Strengthening		Same
Mechanical and Electrical			
Main River Bridge: Bascule Span at Piers 2 and 3	Operating Machinery Replacement	+	Add Machinery for Widen
	Rehabilitation of Trunnions and links	+	Add Trunnions
	Span Balance Work		Same
	Replace incoming electrical service		Same
	Center span lock power feed		Same
	Replace motors and drives	+	Add Motors and Drives
	Relocate and update PLCs		Same
	Replace navigation lighting		Same
	Replace traffic warning gates		Same
	Relocating electrical equipment		Same

Table ES-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit ,Widen, or Replace
Geotechnical Mitigation			
Under West Approach Bridge	Ground Improvement for liquefaction mitigation (see Appendix E for locations and mitigation types)		
Under East Approach Bridge	Ground Improvement for liquefaction mitigation (see Appendix E for locations and mitigation types)		

Note: Utilities, traffic control, etc., are not listed.

Table ES-2 is not a comprehensive list of the work elements. For example, potential utility relocations, maintenance of traffic during construction, site preparation, construction access and staging areas are not included in the list.

Constructability, Risks, and Impacts

In addition to the Burnside Bridge being located in a highly congested downtown area, there are a number of major constructability challenges, associated risks and impacts, such as:

- Buildings attached to or in close proximity on either side of the bridge approach spans.
- MAX lines, Waterfront Park Trail, and roadways under the West Approach Bridge.
- Major utility lines and a pump station under the West Approach Bridge. This includes a City of Portland Combines Sewer Overflow (COS) line.
- A seawall that needs to be temporarily relocated for access to the Pier 1 foundation.
- One of the bascule span leaves should remain operable during construction.
- In-water construction activities at the bascule piers need to keep navigational channel open.
- Interstate 5 (I-5) ramps over the Pier 4 foundation blocks construction access.
- I-5 main lines and a ramp to I-84 under East Approach Bridge spans need to remain open during construction.
- Union Pacific Railroad (UPRR) main lines under East Approach Bridge spans.
- Major utility lines under the East Approach Bridge. This includes a City of Portland Combines Sewer Overflow (COS) line.

Seismic Retrofit Only vs. Retrofit with Bridge Widening

Two baseline retrofit alternatives were studied and compared:

- Widening the middle section of the bridge from 86 feet to 110 feet would provide the bridge a constant width of 110 feet, which is better functionality for traffic flow. The cost increase in foundations is minimal as both the retrofit only and the widening alternatives require enlarged foundations.

- Widened bents and piers would strengthen the existing portions of the structure, thus enhancing the structure's seismic resilience.

1 Introduction

1.1 Project Purpose

Multnomah County is undertaking a feasibility study to evaluate and recommend seismically resilient alternatives for the Burnside Bridge river crossing. The following summarizes the project background, the problem being addressed, and the project's intent.

1.1.1 Background

Burnside Street, which extends from Washington County to Gresham and crosses the Willamette River via the Burnside Bridge, has been designated as a "lifeline" transportation route, meaning it will be expected to enable emergency response, evacuation, and recovery after a major disaster.

The Burnside Bridge carries approximately 40,000 vehicles and over 2,000 bikes and pedestrians per day. Built in 1926, the Burnside Bridge is an aging structure requiring increasingly more frequent and significant repairs and maintenance.

1.1.2 The Problem

Geologically, Oregon is located in the Cascadia Subduction Zone (CSZ), making it subject to some of the world's most powerful, recurring earthquakes. The last major quake in Oregon occurred 317 years ago, a timespan that exceeds 75% of the intervals between the major quakes to hit Oregon over the last 10,000 years. There is a significant risk that the next event will occur soon. Such an earthquake will cause major ground shaking, settling and landslides, and is expected to result in thousands of deaths and widespread damage to buildings, utilities, and transportation facilities.

The next major earthquake is expected to cause moderate to significant damage to the aging downtown bridges, including the existing Burnside Bridge, rendering them unusable immediately following the earthquake. In their current condition, all of the downtown bridges and/or approaches will fail to provide communities and the region with timely and critical emergency response, evacuation, and recovery functions.

1.1.3 Existing Burnside Bridge and Lifeline Route

Burnside Street was designated as a "Primary East-West Emergency Transportation Route" in a 1996 report to Metro's Regional Emergency Management Group. This group was formed by intergovernmental agreement among the region's cities, counties, Metro and Red Cross to improve disaster preparedness, response, recovery and mitigation plans and programs. (Source: Regional Emergency Transportation Routes, Portland Metropolitan Region. Metro Regional Emergency Transportation Routes Task Force. 1996)

The Burnside Street lifeline route is approximately 18.7 miles in length and extends from Highway 26 in Washington County to Gresham, crossing the Willamette River via the Burnside Bridge.

Other agency plans have also identified Burnside Street as an important lifeline route. For example, the City of Portland's Citywide Evacuation Plan addresses evacuation needs for general disasters. The Plan identifies Burnside Street as the primary east-west evacuation route in downtown Portland west of the river. On the east side, I-84 is the Evacuation Plan's designated primary east-west evacuation route; east of the river Burnside Street is designated a secondary route due to less consistent capacity. (Source: Portland Citywide Evacuation Plan (draft) City of Portland Bureau of Emergency Management. 2014). However, while I-84 has greater capacity, it would likely be impassable following a major earthquake due to the collapse of multiple overpasses (18 overpasses cross I-84 between the river and I-205). Burnside Street has no overpasses or bridges through this segment, which is a significant advantage for a lifeline transportation route following a major earthquake.

The statewide Oregon Resilience Plan does not make specific recommendations for seismic resilience of locally owned roads or bridges. The plan's specific roadway and bridge recommendations focus on state-owned facilities. However, the statewide plan does acknowledge and emphasize the importance of creating seismically resilient local bridges and roads, particularly to support lifeline functions in urban areas. Relevant statements in the Oregon Resilience Plan include:

- "Enhance the proposed (state) Highway Lifeline Maps by considering the use of highway segments owned by cities and counties to provide access to critical facilities. Prioritize local routes to provide access to population centers and critical facilities from the identified (state) Tier-1 routes." (Transportation Chapter, page 54)
- "When developing projects for seismic retrofit of (state) highway facilities, consider whether a local agency roadway may offer a more cost effective alternative for all or part of a lifeline route." (Transportation Chapter, page 54)
- Recommendation for "Seismically upgrading lifeline transportation routes into and out of major business centers statewide by 2030" (Executive Summary).

Burnside Bridge traffic counts are from 2014. The Burnside Bridge currently has five general traffic lanes, two bike lanes and sidewalks. (Source: Multnomah County)

1.1.4 Project Intent

This project will address the regional need for a seismically resilient Burnside Street lifeline crossing of the Willamette River that will remain fully operational and accessible for vehicles and other modes immediately following a major CSZ earthquake. It will enable:

- Emergency medical, fire and life safety response
- Evacuation of survivors to safe locations
- Reunification of families and households

- Post-disaster restoration of services, and
- Regional recovery.

The project would help to implement specific and general recommendations for seismic resilience outlined in relevant local, regional and state plans and policies.

The project would be compatible with existing major infrastructure.

The project would provide long-term, low-maintenance, multi-modal transportation functions over the Burnside Street Willamette River crossing consistent with Multnomah County's values.

1.1.5 Seismic Retrofit Report Intent

The purpose of the Project's Seismic Retrofit Study includes the following:

- Identify the seismic vulnerabilities of the existing bridge.
- Develop feasible concepts for seismic retrofit alternatives.
- Perform a conceptual level seismic retrofit analysis of the existing Burnside Bridge. Develop a list of feasible of Seismic Retrofit and Hybrid alternative strategies, including any necessary rehabilitation measures, to withstand major seismic events as defined in the Project's Seismic Design Criteria.
- Develop feasibility-level project costs, risks, and impacts for feasible Seismic Retrofit and Hybrid alternatives.

1.2 General Bridge Description

Originally constructed in 1926, the Burnside Bridge crosses the Willamette River, multiple City of Portland streets, parking lots, parks, TriMet Max lines, and other facilities along Burnside Street (Figure 1-1). This bridge carries five lanes of vehicle traffic (3 eastbound lanes and 2 westbound lanes), and bike lanes and sidewalks in each direction. The total length of the bridge is approximately 2,307 feet, and consists of three separate bridges (see Figure 1-2 and Figure 1-3) (Multnomah County 1924):

- West Approach Bridge (Br. No. 00511A) spans – 602 feet
- Main River Bridge (Br. No. 00511) spans – 856 feet
- East Approach Bridge (Br. No. 00511B) spans – 849 feet

This bridge is also a historically significant structure and is on the National Register of Historic Places.

Figure 1-1. Burnside Bridge Main River Span Bridge over the Willamette River – Portland, Oregon

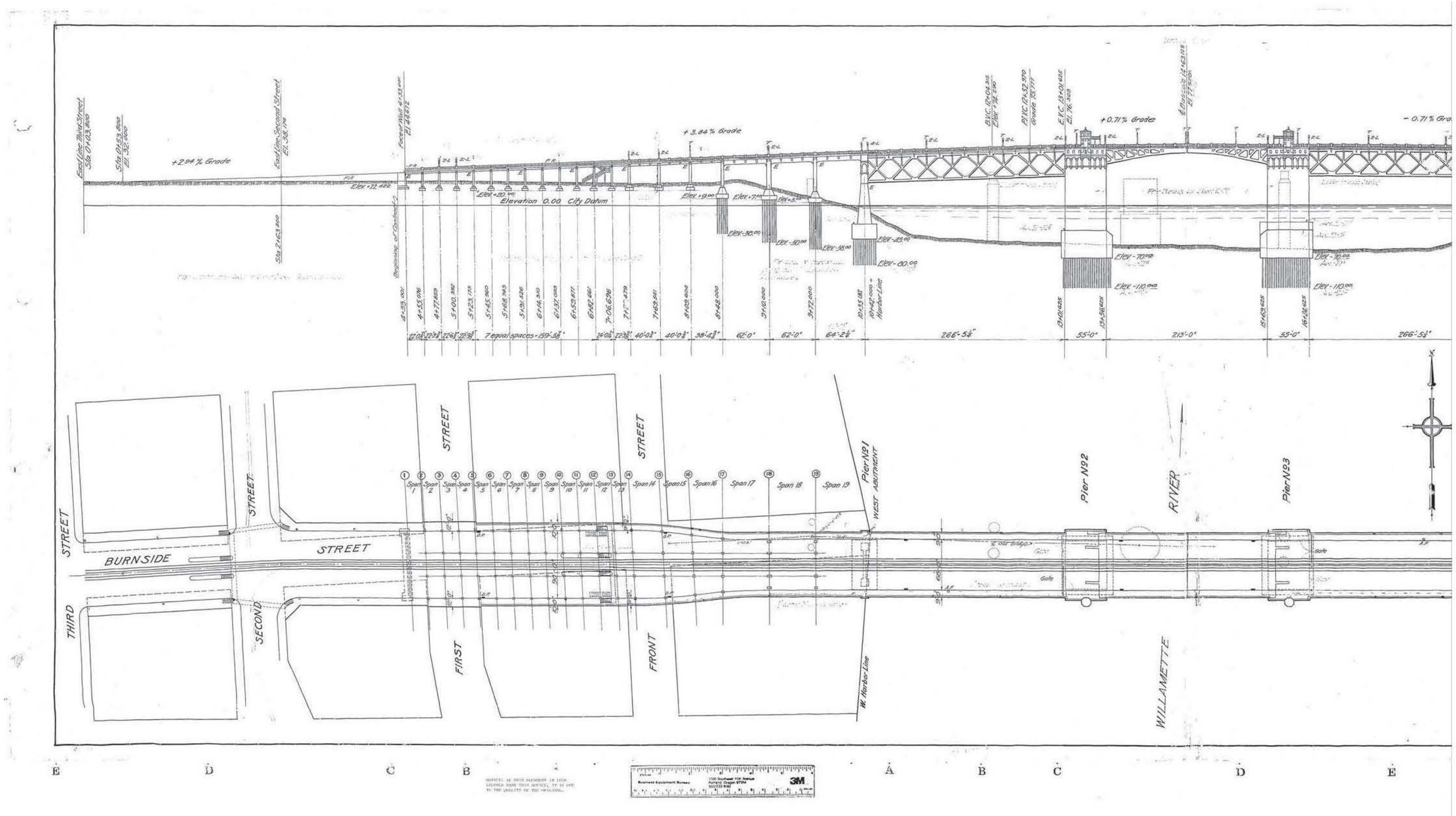


Figure 1-2. Burnside Original As-Built Plans, 1924, West Approach Bridge and a Portion of the Main River Span Bridge

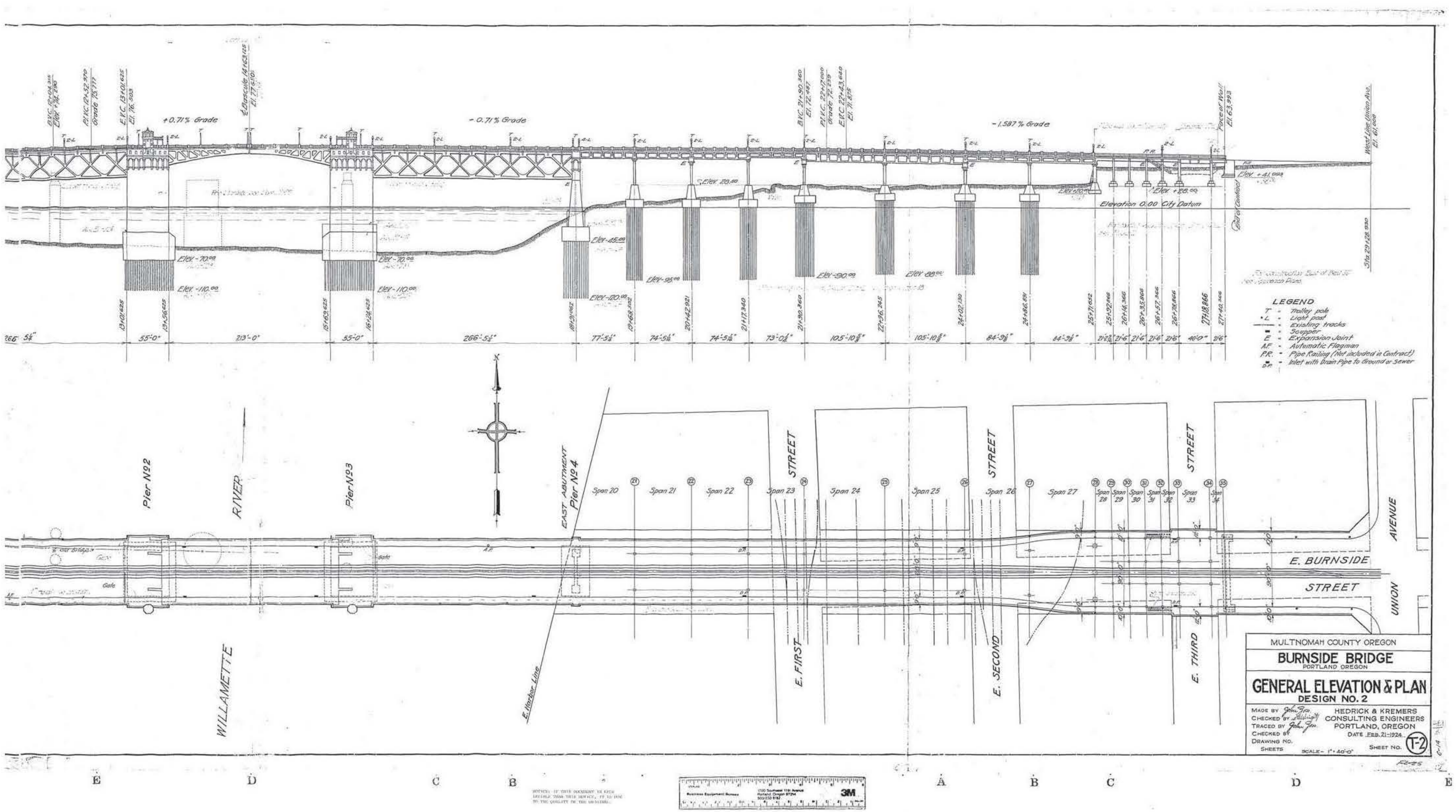


Figure 1-3. Burnside Original As-Built Plans, 1924, a Portion of the Main River Span Bridge and the East Approach Bridge

1.2.1 West Approach Bridge Spans

The Burnside Bridge's West Approach Bridge is 602 feet long and consists of 19 spans, referred to as Span 1 through Span 19. It crosses over City of Portland streets, the TriMet Max line, and the Vera Katz Waterfront Park.

The abutments and piers that support these spans are referred to as *bents* in the as-built plans. Therefore, 19 bents support the superstructure of the west approach spans. The east ends of the Span 19 girders are supported by Pier 1, which also supports the steel truss fixed over the water.

The existing deck width of the structure is 110 feet from Bent 1 to Bent 14, then gradually narrows down to 86 feet at Bent 18, and then remains 86 feet wide up to Pier 1.

The superstructures of Spans 1 to 16 consist of reinforced concrete floor beams with multiple stringers and a concrete deck. The superstructures of Spans 17 to 19 consist of reinforced concrete deck girders.

The west abutment (Bent 1) is a gravity type wall abutment. For Spans 1 to 16, the floor beams are supported by concrete columns on spread footings. For Spans 17 to 19, the deck girders are supported by concrete columns on timber pile supported footings with enlarged bases and pile caps.

1.2.2 Main River Bridge Spans

The Burnside Bridge's Main River Bridge is 856 feet long and consists of two steel truss fixed spans over the Willamette River, and a steel double-leaf bascule span that crosses over the river's main navigation channel. It also crosses over the Eastside Esplanade.

The piers that support these three river spans are referred as Pier 1 to Pier 4 from west to east in the as-built plans.

Each of the two fixed river spans is 268 feet long and consists of constant depth steel deck truss with a reinforced concrete deck. The spans are supported on one end (Pier 1 or 4) by lightly reinforced columns connected to timber pile footings with unreinforced pile caps, and they are supported on the bascule piers on the other end (Pier 2 or 3).

The Burnside Bridge's main river span crossing the navigation channel is a 252-foot-long (trunnion-to-trunnion) double-leaf steel deck truss bascule span. Reinforced concrete decks on the variable-depth bascule leaves are supported on concrete bascule piers, which also house the counterweight and bascule machinery. Each bascule pier includes 35- to 44-foot unreinforced concrete walls from the pit floor to the top of the pile cap. The piers are supported on unreinforced pile caps founded on timber piles.

1.2.3 East Approach Bridge Spans

The Burnside Bridge's East Approach Bridge is 849 feet long and consists of 15 spans referred to as Span 20 through Span 34. It crosses over multiple City of Portland streets, parking lots, and the Burnside Skatepark.

As for the west approach, the abutments and piers that support these east approach spans are referred to as bents in the as-built plans. Therefore, a total of 15 bents support the superstructure of the east approach spans. The west end of Span 20 is supported by Pier 4, which also supports the steel truss fixed span over the water.

The existing deck width of the east side of the structure is a 86 feet from Pier 4 to Bent 26, then gradually widens to approximately 110 feet by Bent 28, and then remains 110 feet wide to Bent 35 (east abutment).

The superstructures of Spans 20 to 27 consist of two concrete-encased steel plate girders with integral concrete-encased floor beams and a concrete deck. These spans are supported by steel piers also encased with concrete. The superstructures of Spans 28 to 34 consist of multiple reinforced concrete deck girders, supported by concrete columns and bent caps.

For Spans 20 to 27, the steel structure spans are supported on two concrete-encased steel columns on timber pile foundations with enlarged bases and pile caps. For Spans 28 to 34, the concrete superstructure spans are supported on four concrete columns on spread footings at each bent. The east abutment (Bent 35) is a gravity type wall abutment.

2 The Need for Bridge Seismic Retrofit

Geologically, Oregon is located in the Cascadia Subduction Zone (CSZ), making it subject to some of the world's most powerful, recurring earthquakes (Oregon Department of Transportation [ODOT] 2014). There is a significant risk that the next event will occur relatively soon. Such an earthquake will cause major ground shaking, settling, and landslides, and it is expected to result in major and widespread damage to buildings, utilities, and transportation facilities (Oregon Seismic Safety Policy Advisory Commission 2014), leaving the City of Portland divided and isolating members of the community.

In response to this future seismic risk, Multnomah County recently completed its 20-year Willamette Bridges Capital Improvement Plan (CIP) (David Evans and Associations 2014). This is a thorough and comprehensive study of the County's six bridges, especially the four downtown structures, which provides a high-level assessment of the condition of these critical transportation infrastructures and a list of required improvements to ensure they continue to be safe and reliable.

The CIP identified that Burnside Bridge seismic resiliency is a top priority for Multnomah County in the next 20 years.

2.1 Burnside Street Lifeline Designation

Burnside Street was designated as a "Primary East-West Emergency Transportation Route" in a 1996 report to Metro's Regional Emergency Management Group. This group was formed by intergovernmental agreement among the region's cities, counties, Metro, and The American National Red Cross to improve disaster preparedness, response, recovery, and mitigation plans and programs (Metro 1996).

The Burnside Street lifeline route is approximately 18.7 miles long and extends from US 26 in Washington County to Gresham, crossing the Willamette River via the Burnside Bridge.

Other agency plans have also identified Burnside Street as an important lifeline route. For example, the City of Portland's Citywide Evacuation Plan addresses evacuation needs for general disasters. The plan identifies Burnside Street as the primary east-west evacuation route in downtown Portland west of the river. On the east side, Interstate 84 (I-84) is the Evacuation Plan's designated primary east-west evacuation route; east of the river Burnside Street is designated a secondary route due to less consistent capacity. (City of Portland 2014). However, while I-84 has greater capacity, it would likely be impassable following a major earthquake due to the collapse of multiple overpasses (18 overpasses cross I-84 between the river and I-205). Burnside Street has no overpasses or bridges above this segment, which is a significant advantage for a lifeline transportation route following a major earthquake.

The Burnside Bridge is a key link for Burnside Street—one of the longest and busiest streets in the Portland area. The five-lane Burnside Bridge is a direct connection between downtown Portland, Beaverton to the west, and Gresham to the east. In 2014, about 40,000 vehicles and more than 2,000 pedestrians and bicyclists used the bridge each day (PBOT 2013).

Through the development of the CIP, it was determined that the Burnside Bridge is a top priority for the County due to its designation as the only Priority 1 lifeline route across the Willamette River in downtown Portland.

2.2 Major Transportation Facilities and Critical Infrastructure

The seismic resiliency of the Burnside Bridge is impacted by the adjacent major transportation facilities and buildings. Therefore, improving the seismic resiliency of the Burnside Bridge will improve the seismic resiliency of the adjacent structures. For example:

- The Tri-County Metropolitan Transportation District of Oregon (TriMet) light rail lines run under the West Approach Bridge (Spans 3 and 4, Second St) of the bridge, and just east of the bridge (Martin Luther King Blvd and Grand Ave).
- Interstate 5 (I-5) south and northbound main lines and the ramps to and from I-84 run under the Burnside Bridge (Spans 20, 21, and 22).
- Union Pacific Railroad (UPRR) lines run under the east approach (span 23).
- The West Approach Bridge (spans 5 to 13) and East Approach Bridge (Spans 28 to 32) are all in close proximity to adjacent buildings.
- The City of Portland Combined Sewer Overflow (CSO) large pipes cross under the West Approach Bridge (span 17) and East Approach Bridge (Span 27).
- Naito Parkway runs under the West Approach Bridge (spans 14 and 15).

2.3 Existing Bridge Seismic Deficiency

The Burnside Bridge is the only County bridge over the Willamette River planned to receive a Phase 2 seismic retrofit to maintain operability following a Magnitude 8+ Cascadia Subduction Zone (CSZ) earthquake. However, as currently built, the bridge is not expected to withstand this major seismic event, nor a significant seismic event from a nearby crustal zone fault.

To improve the seismic resiliency of the existing bridge, the first task under this feasibility study was to identify the seismic deficiencies of the existing bridge. Conceptual plans that show the identified major seismic deficiencies are in Appendix A.

Following this, a series of analyses were conducted to determine the member deficiencies, and recommended retrofit measures to remedy them. Conceptual plans that show the identified major seismic retrofit measures are in Appendix E.

Because of this lifeline designation, the bridge’s performance level for the 1000-year event was set as Limited Operation, and the performance level for the CSZ event was set as Full Operation.

3 Design Criteria and Considerations

Due to the unique nature of the Burnside Bridge, including its age, long bascule spans, and proximity to the Cascadia Subduction Zone, a project-specific seismic design criteria was developed. It was intended for use with the Feasibility phase only, and was vetted with a committee of Oregon industry experts. This criteria is entitled the Earthquake Ready Burnside Bridge – Seismic Design Criteria, and can be found in Appendix B.

3.1 Applicable Design Specifications and Guidelines

The seismic retrofit and widening designs of the Burnside Bridge will conform primarily to the following major design codes (in order of precedence):

1. Earthquake Ready Burnside Bridge – Seismic Design Criteria (Criteria)
2. ODOT Bridge Design and Drafting Manual (BDDM), October 2016 version (ODOT 2016)
3. ODOT Geotechnical Design Manual (GDM), November 2015 version (ODOT 2015)
4. AASHTO LRFD Movable Highway Bridge Design Specifications, 2nd Edition, with 2008, 2010, 2011, 2012, 2014, and 2015 Interim Revisions (AASHTO Movable) (AASHTO 2008)
5. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, with 2012, 2014, and 2015 Interim Revisions (Guide Spec) (AASHTO 2012)
6. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 7th Edition, with 2015 and 2016 Interim Revisions (AASHTO LRFD) (AASHTO 2015)

3.2 Project-Specific Requirements

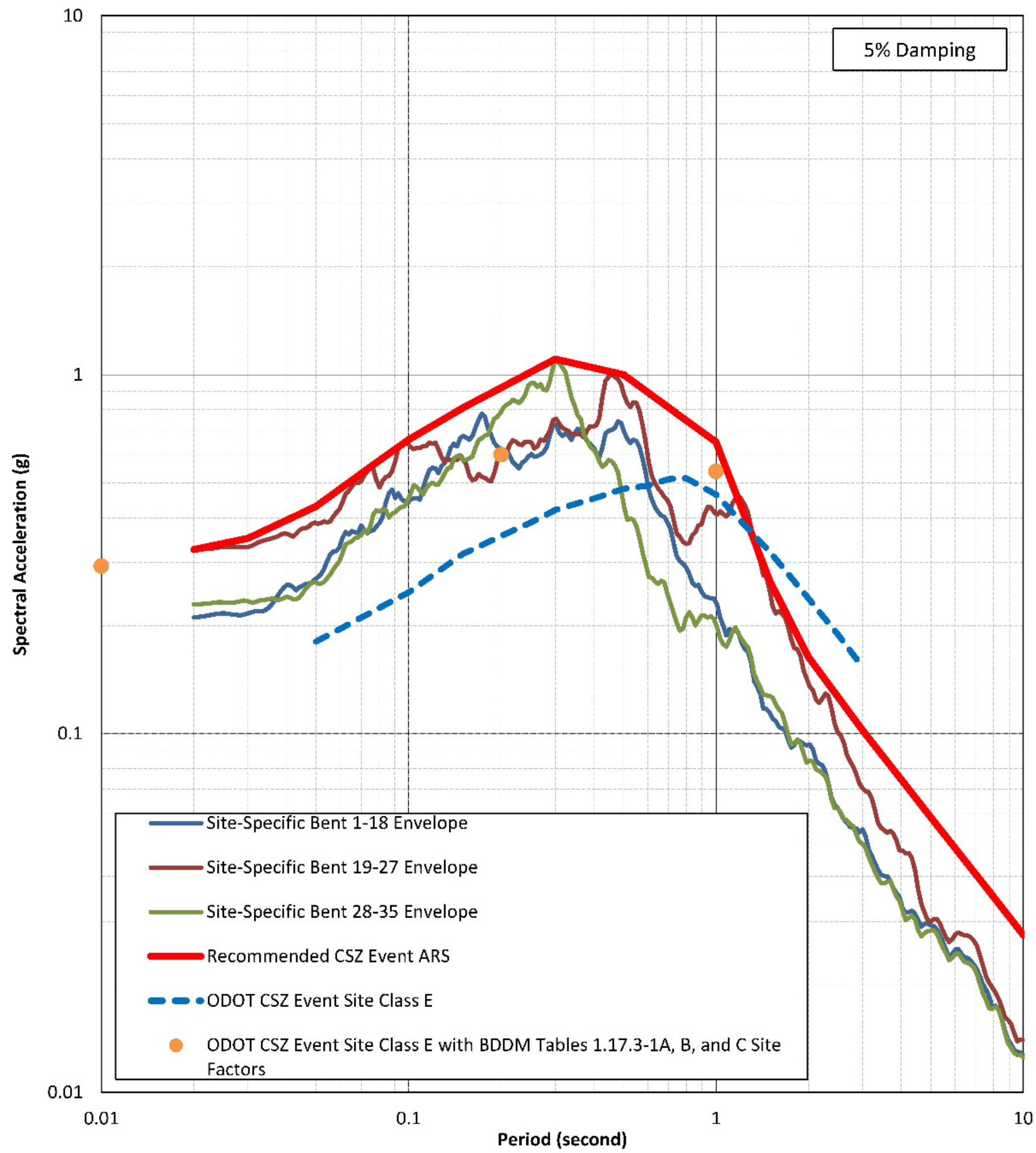
All the above Section 3.1-listed AASHTO design criteria are applicable to this project but for different design aspects. For example, the ductility design requirements described in the Guide Spec (AASHTO 2012) may not apply to the box-shaped bascule pier walls, but will apply to the approach substructures.

3.2.1 Seismic Design Criteria

After studies and Seismic Review Committee (SRC) meetings and discussions, two levels of performance requirements were adopted for this project (Appendix B):

- Full Operation for a CSZ seismic event.
- Limited Operation for a 1000-year return period seismic event.

Site-specific acceleration response spectra (ARS) curves for these two levels of design seismic events were developed by geotechnical engineers for this project (Figure 3-1 and Figure 3-2).



NOTES

1. The ODOT CSZ Event Site Class E spectral values were estimated for Vs30 of 150 m/sec by extrapolating the spectral values from Vs30 of 270 and 200 m/sec.
2. Recommended Spectrum at long periods were obtained by taking two-thirds of ODOT CSZ Event Site Class E linearly extrapolated (in log scale) above 3.0 sec.
3. g = standard gravitational acceleration; CSZ = Cascadia subduction zone; ARS = acceleration response spectrum; ODOT = Oregon department of transportation; Vs30 = time-averaged mean shear wave velocity for 30 meters subsurface; m/sec = meters per second; BDDM = bridge design and drafting manual

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

CSZ EVENT ACCELERATION RESPONSE SPECTRA FOR "FULL OPERATION" PERFORMANCE LEVEL

August 2017 24-1-04065-005

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5

Figure 3-1. ARS – CSZ Event

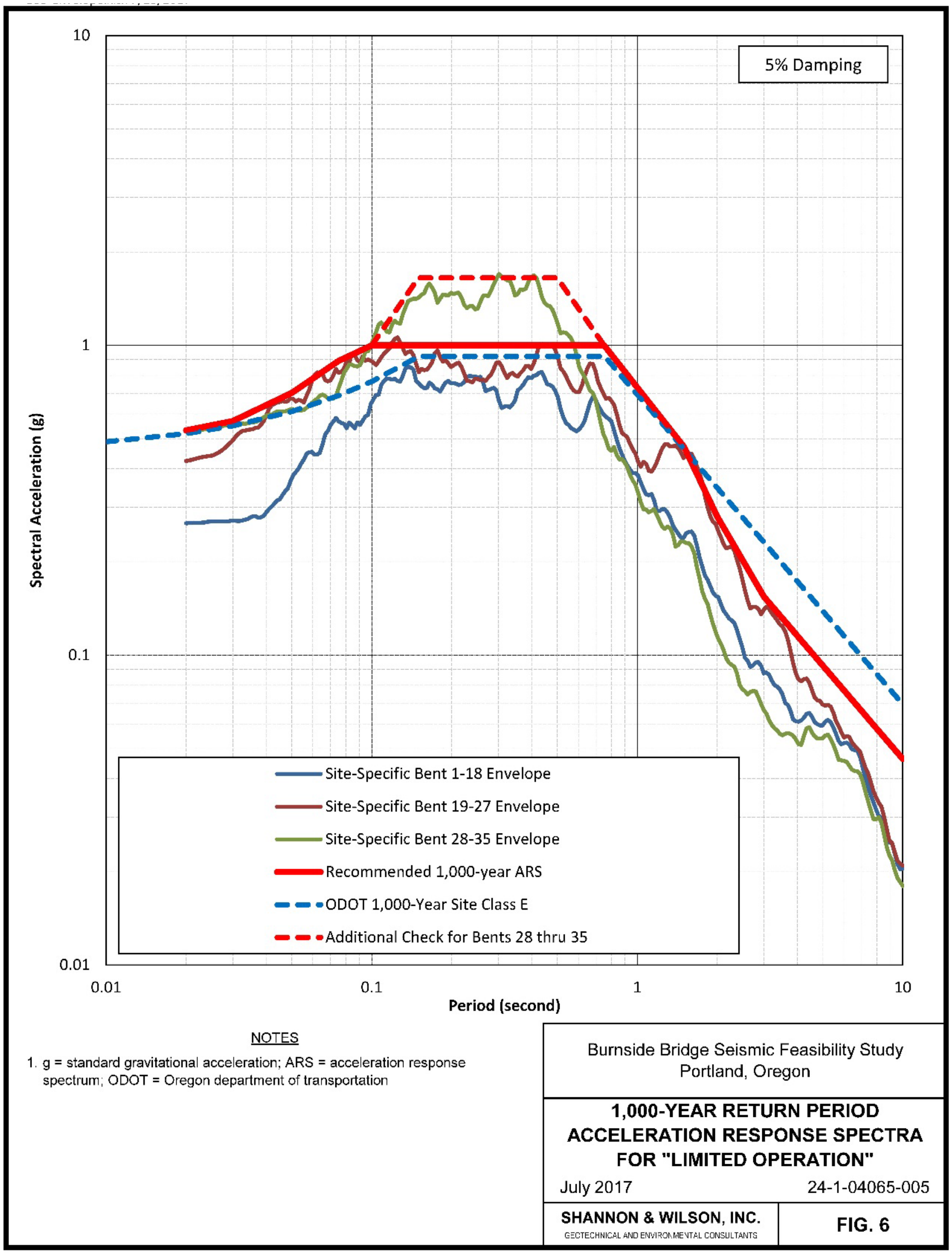


Figure 3-2. ARS – 1000-year Return Period

Because the site soil properties vary, geotechnical engineers developed three separate enveloped ARS curves for each seismic event category:

1. Site-specific Bents 1-18 Envelope
2. Site-specific Bents 19-27 Envelope
3. Site-specific Bents 28-35 Envelope

For this conceptual-level analysis, these three ARS curves were further combined and enveloped into a single recommended ARS curve for each design-level seismic event. Further adjustment was made to the recommended ARS curve to simplify the conceptual analysis for the 1000-year return period seismic event. The higher site-specific Bents 28-35 ARS curve was excluded from the recommended 1000-year return period ARS curve, based on the consideration that this site has no liquefaction concern. More details of the development of these ARS curves can be found in Burnside Bridge Geotechnical Report (Appendix C).

The recommended ARS curves for both levels of seismic events have peak accelerations at approximate 1.0g:

- The ARS curve for the CSZ seismic event has a peak acceleration of approximately 1.0g, between periods of approximately 0.25 to 0.5 second.
- The ARS curve for the 1000-year return period seismic event has a peak acceleration of approximately 1.0g, between periods of approximately 0.1 to 0.7 second.

3.2.2 Operational Performance Requirements

The performance requirements under the two design events (full CSZ rupture and 1000-year event) are described in more detail in Earthquake Ready Burnside Bridge – Seismic Design Criteria (Appendix B). These performance requirements meet or exceed the seismic design requirements in the *ODOT Bridge Design and Drafting Manual* (ODOT 2016), Section 1.17.

This two-level performance requirement poses significant challenges to the Burnside Bridge, especially on the bascule span. To meet these performance requirements, the bridge must have adequate strength and ductility, and must be capable of tolerating seismic displacements.

For a bascule bridge such as the Burnside Bridge, the existing bascule leaf machinery driving system can tolerate only very small displacements. At a conceptual design level, the analysis model is not capable of precisely predicting such small relative movements within a fraction of an inch. However, conceptual-level analysis has provided predictable ranges and the trends, which are used in this conceptual design.

3.2.3 Seismic Hazard and Ground Motions

Project-specific seismic hazard and ground motions are defined in the Burnside Bridge Geotechnical Report, Appendix C.

3.3 Navigation Clearances and Opening

3.3.1 Navigation Clearance

The Willamette River navigation channel clearance requirements under the Burnside Bridge bascule span are shown in Figure 3-3. The proposed seismic retrofit and widening strategies have phased construction that would reduce the channel width near one of the bascule piers during the construction and would restore the channel width to the existing width after the construction.

3.3.2 Bascule Span Open and Close

General requirements for movable bridge opening and closing and specific requirements for the Burnside Bridge, according to the Code of Federal Regulations (CFR), are listed below:

CFR 117.33, **Closure of draw for natural disasters or civil disorder**. Drawbridges need not open for the passage of vessels during periods of natural disasters or civil disorders declared by the appropriate authorities unless otherwise provided for in Subpart B or directed to do so by the District Commander.

CFR 117.36, **Closure of drawbridge for emergency repair**, (c) Repair work under this section must be performed with all due speed to return the drawbridge to operation as soon as possible.

CFR 117.897 **Willamette River**, (C) (3) (iii), Burnside Bridge, mile 12.4, from 8 a.m. to 5 p.m. Monday through Friday, one hour's notice shall be given for draw openings. At all other times, 2 hours' notice is required.

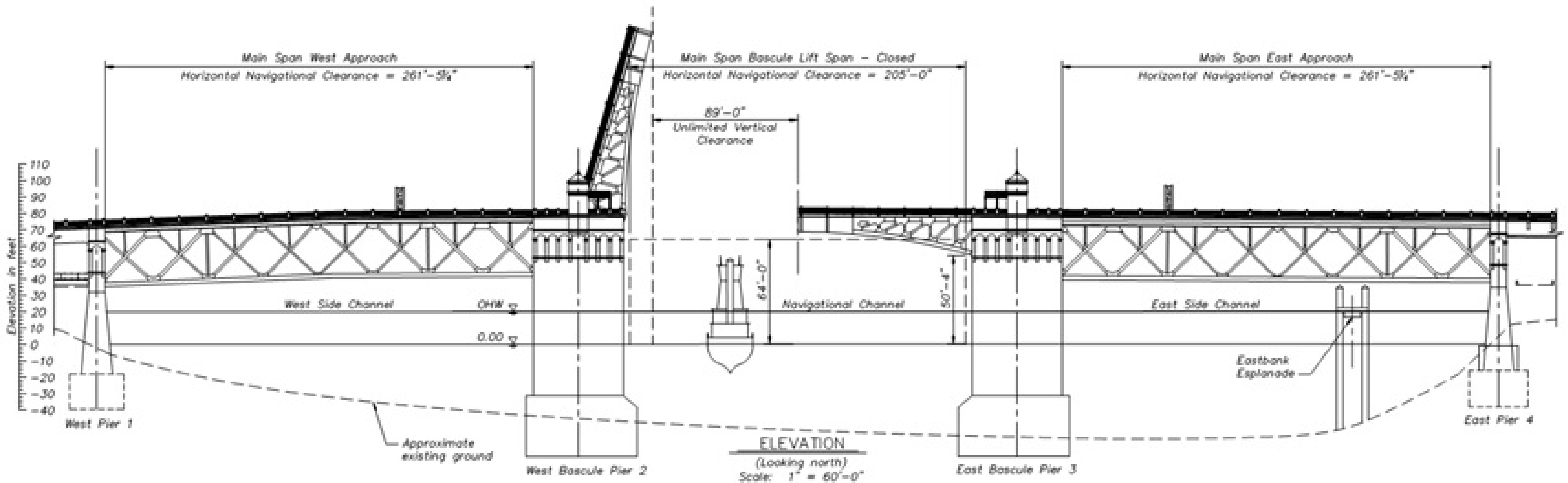


Figure 3-3. Navigation Channel Clearance

4 Existing Site Conditions

4.1 Geotechnical Condition

In order to define the bridge's geologic condition, Shannon & Wilson drilled three geotechnical borings at the project site, 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 the mudline. Boring locations, details of drilling, sampling procedures, and logs of the materials encountered in the explorations are presented in Appendix B of the Geotechnical Investigation Report (Appendix C). All borings included in-situ geophysical testing (OYO Suspension Logging), which is also discussed and presented in Appendix B of the Geotechnical Investigation Report (Appendix C).

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 and logs of the relevant historical borings are presented in Appendix B of the Geotechnical Investigation Report (Appendix C).

The materials encountered in the field explorations and in the historical borings were grouped into ten geotechnical units described in Appendix B of the Geotechnical Investigation Report (Appendix C) and are summarized as follows.

- Fill
- Fine-Grained Alluvium
- Sand/Silt Alluvium
- Sand Alluvium
- Gravel Alluvium
- Catastrophic Flood Deposits – Fine-Grained Facies
- Catastrophic Flood Deposits – Channel Facies
- Upper Troutdale Formation
- Lower Troutdale Formation
- Sandy River Mudstone

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.

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, where thicknesses are up to 110 feet. 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.

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 are about 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 and silty sand.

An approximately 25- to 50-foot-thick layer of sand alluvium is interpreted to be present 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. The sand alluvium consists of loose to medium dense, occasionally dense to very dense, sand to gravelly sand with varying amounts of silt.

A layer of gravel alluvium, ranging from about 10 to 40 feet thick, is interpreted to be underlying the sand alluvium below the Willamette River, and underlying other alluvial deposits on the adjacent banks. The gravel alluvium consists of medium dense to very dense gravel with varying amounts of sand and fines. Portions of the unit contain cobbles and possible boulders.

Catastrophic flood deposits – fine-grained facies sediments were encountered on the east side of the Burnside Bridge. The unit was encountered directly underneath the fill and extended to depths of 13 to 15 feet below the ground surface. In the vicinity of the Burnside Bridge, encountered portions of the unit were reported to consist of stiff to very stiff, brown silt.

An approximately 20-foot-thick layer of catastrophic flood deposits – channel facies sediments was encountered below the catastrophic flood deposits – fine-grained facies on the east side of the Burnside Bridge. In the vicinity of the Burnside Bridge, encountered portions of the unit were reported to consist of dense to very dense interbedded sand and gravel deposits with varying amounts of fines. Lesser layers of stiff sandy silt were also reported in the unit. Portions of the unit contain cobbles and possible boulders.

The Troutdale Formation appears to underlie the entire project site, beneath the overlying alluvial and fill units. An Upper and Lower Troutdale Formation were identified based on interpretation of the existing information. 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. Some cementation was reported in portions of the unit.

The 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 about 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, which appears to be over 100 feet thick. The unit typically consists of very dense gravel with varying amounts of sand and fines. Zones of cementation are noted throughout the unit, and cobbles may be present in some areas.

Sandy River mudstone was interpreted to be encountered below the Lower Troutdale Formation along the western side of the project. The existing borings suggest 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.

Logs of historical borings on the west side of the Willamette River report groundwater elevations that range from approximately 6 to 10 feet (NAVD 88). The log of a historical 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 borings performed for the Portland Development Commission (PDC) on the east side of the river indicate a groundwater elevation of 25 feet. One of the PDC borings encountered a layer of perched water at an elevation of approximately 50 feet. 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 drilling fluids in the boreholes.

Over the course of a year, water levels in the Willamette River typically fluctuate between elevations of approximately 6 and 20 feet. This is comparable to the groundwater elevations reported in the historical on-land borings, with the exception of the perched groundwater reported in the PDC boring. 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. Therefore, a groundwater elevation of 20 feet was assumed for the geotechnical analyses.

4.1.1 Liquefaction

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, excess pore pressure development, and lateral movement were evaluated directly using a nonlinear effective stress site response analysis. The computer program FLAC (Fast Lagrangian Analysis of Continua) was used to perform the site response 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 FLAC analysis.

The ground subsurface soil profile can be seen in Figure 4-1 through Figure 4-4. Liquefaction and associated effects are anticipated at Bents 1 through 27 and Piers 1 through 4. No liquefaction effects are anticipated at Bents 28 through 35. Appendix B of the Geotechnical Investigation Report (Appendix C) presents detailed results of the FLAC analyses and liquefaction evaluation.

Information on how liquefaction will affect the seismic resistance of the existing foundations is provided in Section 0. Conceptual options to mitigate liquefaction effects are presented in Sections 7.2 and 7.3.

4.1.2 Liquefaction-Induced Settlement

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 to 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-induced settlement is determined based on the maximum shear strain from the FLAC analysis. Estimated liquefaction-induced ground settlement at the existing Bent 1 through 17 spread footing foundations ranges from 1 to 4 inches. Estimated liquefaction-induced settlement at the existing Bent 18 through 27 and Pier 1 through 4 pile group foundations ranges from 1 inch to more than 4 feet. Appendix B of the Geotechnical Investigation Report (Appendix C) presents detailed results of the liquefaction-induced settlement evaluation.

The effects of liquefaction and associated settlement on the existing foundations are presented in Section 0.

4.1.3 Liquefaction-Induced Lateral Spreading and Flow Failure

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.

The FLAC analyses indicate that liquefaction-induced permanent ground deformation will occur at the west and east riverbanks to varying displacements and elevations for the ground-motion levels considered. For the 1,000-year ground-motion level, ground surface movements up to 14 feet are calculated for the west riverbank, and flow failure with displacements in excess of approximately 60 feet is anticipated at the east riverbank. For the CSZ event ground-motion level, ground surface movements up to 3 and 23 feet are anticipated at the west and east riverbanks, respectively. Appendix B of the Geotechnical Investigation Report (Appendix C) presents detailed results of the lateral spreading and flow failure evaluation.

The effects of permanent ground displacement on the existing foundations are presented in Section 7.1. Conceptual options to mitigate permanent ground displacement are presented in Section 7.3.

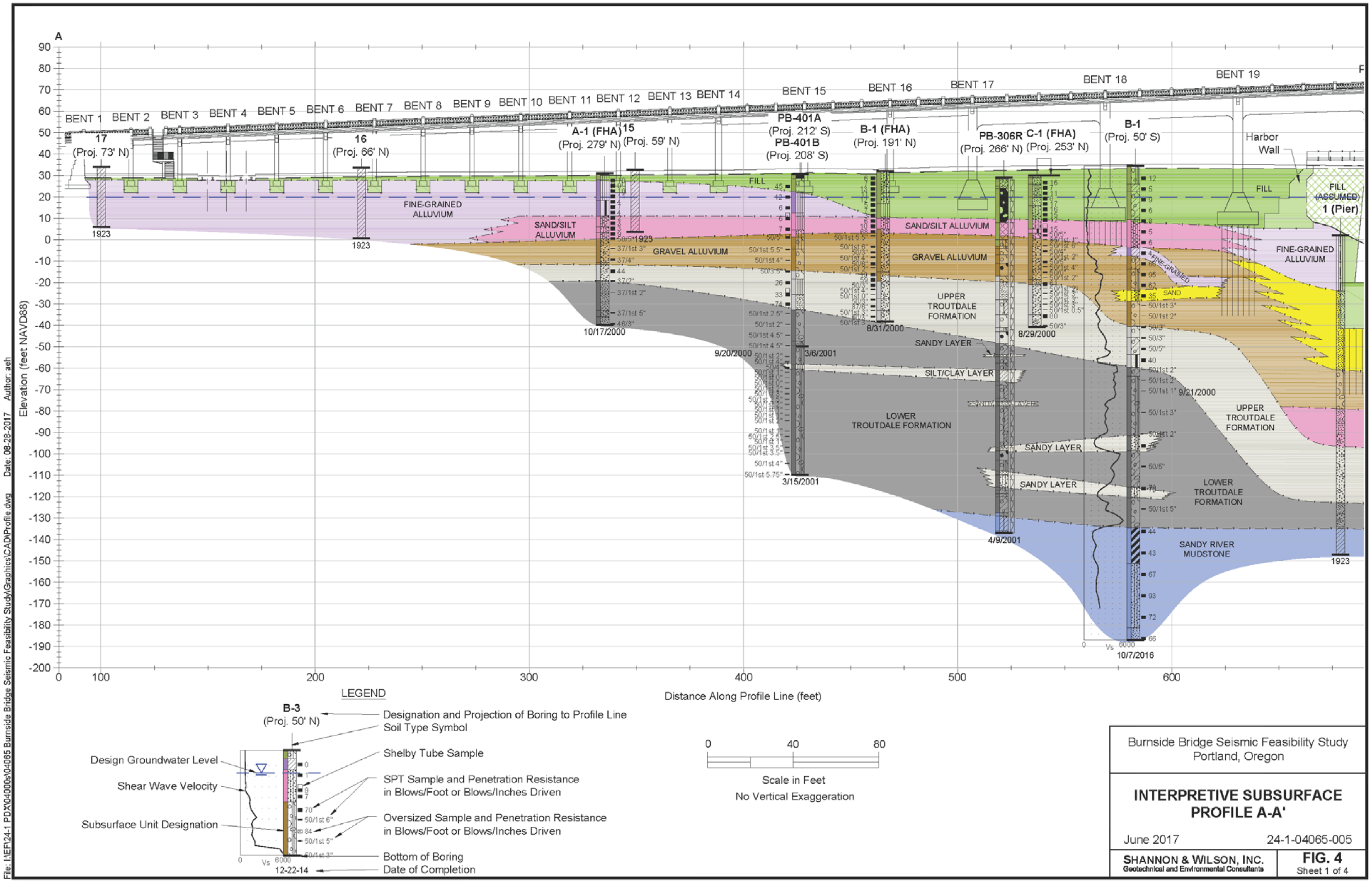


Figure 4-1. Soil Profile Bent 1 to Bent 19

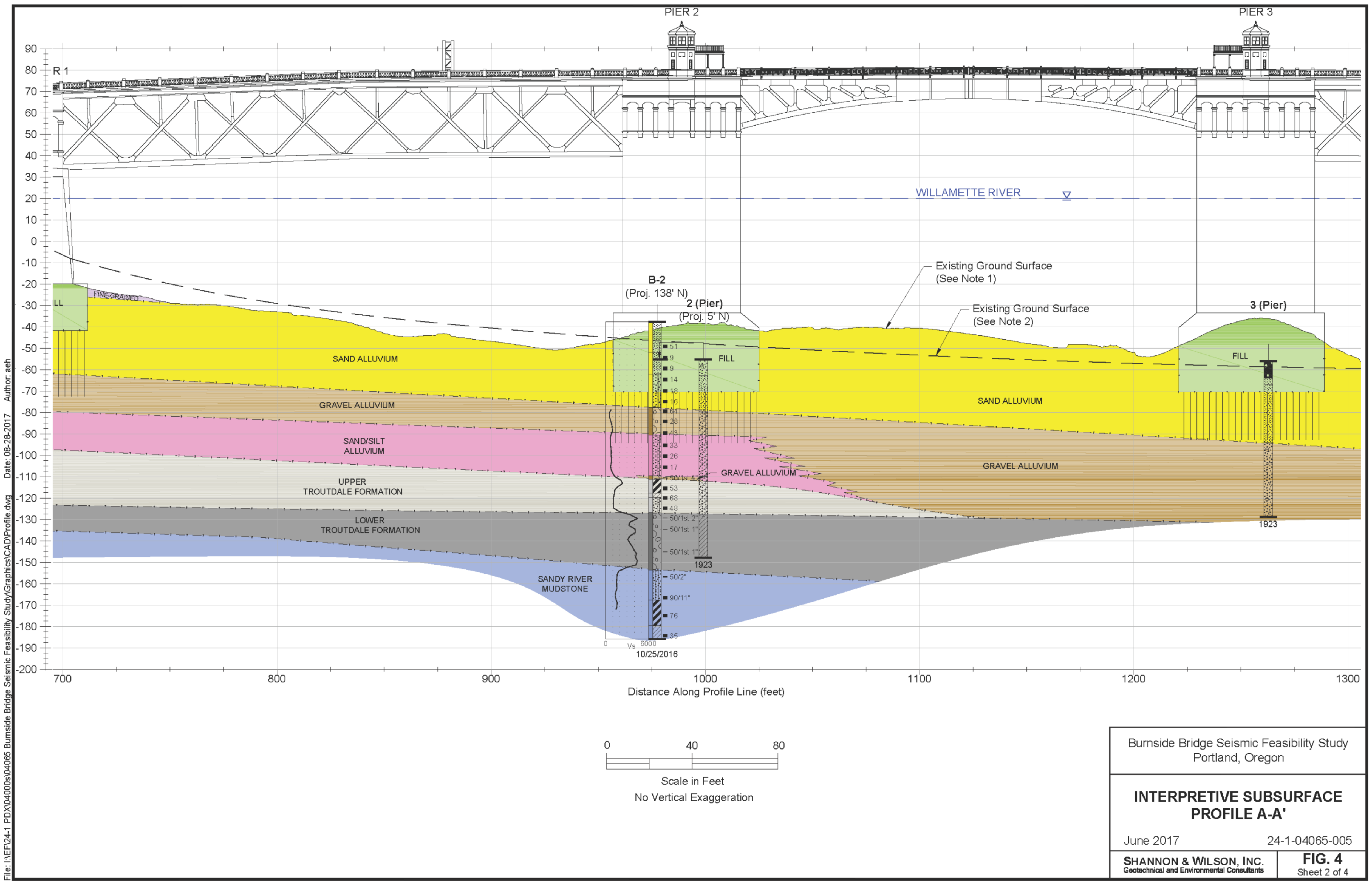


Figure 4-2. Soil Profile Pier 1 to Pier 3

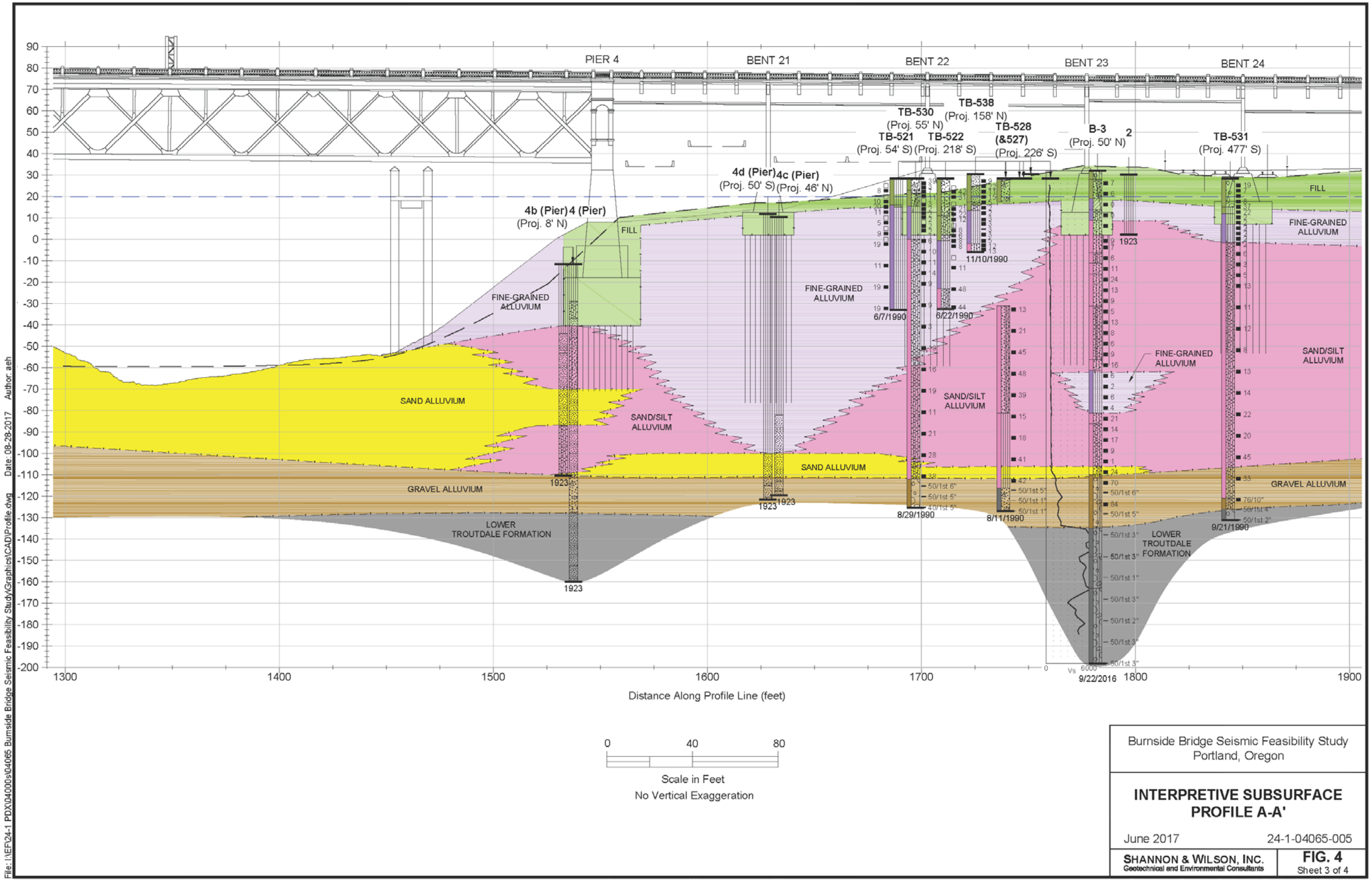
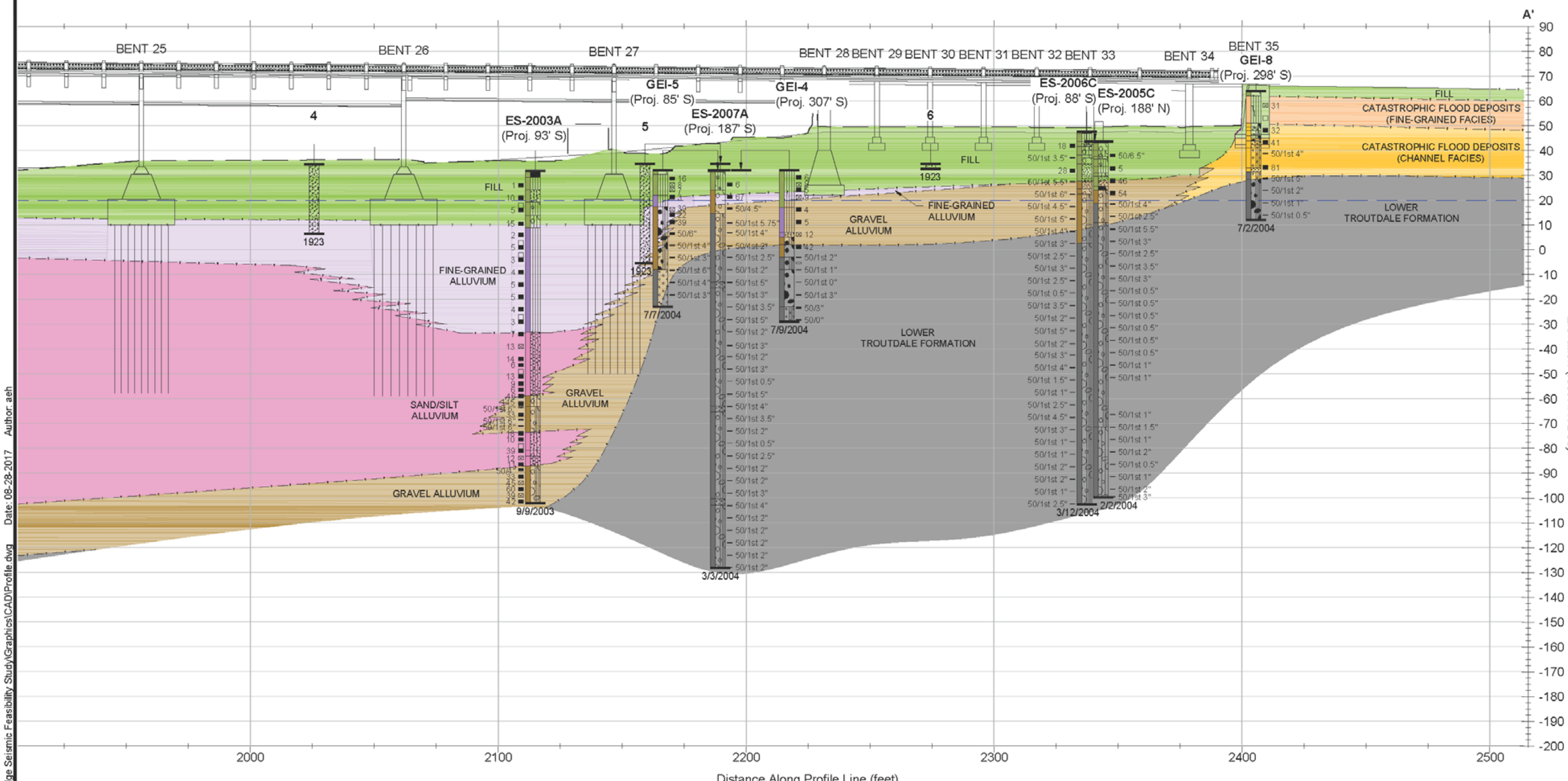


Figure 4-3. Soil Profile from East of Pier 3 to Bent 24



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.

0 40 80
Scale in Feet
No Vertical Exaggeration

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

INTERPRETIVE SUBSURFACE PROFILE A-A'

June 2017 24-1-04065-005

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 4
Sheet 4 of 4

Figure 4-4. Soil Profile from East of Bent 24 to East of Bent 35

4.2 Utilities

The utilities found underground and on the Burnside Bridge structure are generally described below. For details, see the utility drawings in Appendix D.

4.2.1 West Side

The west side utilities include multiple pipes under the streets and in the areas between the streets. The underground pipes accommodate telecommunication, natural gas, electricity, water, sewer, and foul air in structures constructed from clay, ductile iron, PVC, and conduit. Typical pipe sizes range from 1 inch in diameter up to 60 inches for the City of Portland sewer CSP. Of particular note is the 168-inch City of Portland CSP line located between Bents 17 and 18. Utilities related to the Ankeny Pump Station are noted in Section 9.9.3.2. The west approach bridge structure carries various conduits and utilities for the TriMet MAX line including the train overhead catenary lines attached to Bent 3.

4.2.2 Bascule Spans

There is a 6-inch CenturyLink conduit that crosses the Willamette River on the Burnside Bridge. It runs above water on the west approach until it reaches Pier 2. From Pier 2 to Pier 3 the line is submarine. The conduit is attached to Pier 3 where it comes out of the water to continue east on the east approach.

4.2.3 East Approach

East side underground structures accommodate the similar utilities as are present on the west side, in pipes made of the same types of materials. Of note are a 264-inch City of Portland sewer RCP passing under Bents 28 to 30, a 28-inch City of Portland brick sewer pipe, and a 30-inch City of Portland brick sewer pipe. Conduits are attached to the bridge structure at various locations for electrical, street lights, and fiber optic. There are also three communication vaults and an electrical transformer on the east approach structure.

4.3 Water Way Navigation Channel

The vessel navigation channel of the Willamette River is under the bridge's bascule span. Along the centerline of the bridge, the face-to-face distance between the navigational channel side pier walls is 213 feet (See Figure 3-3).

4.4 Adjacent Facilities

4.4.1 Building Adjacent to West Approach Spans

The location and proximity of adjacent buildings can be seen in Figure 4-5. On the north side of the west approach spans and retaining walls, the University of Oregon occupies a building that is immediately adjacent to the north side of the west approach spans between SW Naito Parkway and SW 1st Avenue. In addition, the University of Oregon occupies a classroom space built under Span 1 to the west of SW 1st Avenue. The Portland Rescue Mission occupies a building immediately adjacent to Span 1 and approach retaining walls, and the Central City Concern occupies a building immediately adjacent to the approach retaining walls.

Figure 4-5. Private Building Locations

On the south side of the west approach spans and retaining walls the Portland Saturday Market occupies a building immediately adjacent to Span 1 and the approach retaining walls and also uses space under Span 1 to store materials. The Salvation Army also occupies a building immediately adjacent to the approach retaining walls on the south side.

For the buildings immediately adjacent to the retaining walls, in many cases the buildings are built integrally with the retaining walls. For the buildings immediately adjacent to the bridge spans, an approximately one-inch-wide joint filled with expansion joint material is all that separates the two structures.

4.4.2 Water Facility at Pier 1

The Ankeny Pump Station, owned and operated by the City of Portland's Bureau of Environmental Services (BES), is located along the seawall immediately south of the Burnside Bridge. This wastewater and stormwater station serves downtown and southwest Portland. Originally constructed in 1929, the building is listed on the historic register as a significant structure. Improvements or alterations to the building and surrounding site architecture are severely restricted and subject to stringent land use and zoning review.

When initially constructed in 1929, there was an electrical building immediately adjacent to the south side of Pier 1. This building has since been removed, with the motor control centers relocated inside the Pump Station. In its place, there are several above-grade transformers and switch gear. Electrical power to the Pump Station is routed through underground ducts from a PGE vault located between Bent 18 and Bent 19. Design drawings from the electrical remodel show the power supply ducts running west to east over the top of the below-grade pile cap for Bent 19.

On the north side of the bridge, within Waterfront Park adjacent to Bent 19, BES has two below-grade odor-control vaults. The 19-foot by 19-foot vault contains mechanical equipment and the 25-foot by 26-foot vault contains media for air treatment. Foul air from the Ankeny wet well and Ankeny shaft is piped to the vaults in a 24-inch underground duct that is between Bent 19 and the seawall.

The seawall is recessed into Waterfront Park on the west side of Pier 1 (Figure 4-11). Two sewer force mains running north from the Ankeny Pump Station (one 30-inch and one 42-inch) are attached to the exposed side of the seawall adjacent to Pier 1. The force mains are stacked above each other and follow the seawall recess, turning on the north side of Pier 1, then following the seawall to the north before crossing under the river to the east.

4.4.3 Highway Ramps under East Approach Spans

I-5 and associated ramps pass under Spans 20 to 22 and can be seen in Figure 4-6. The interstate and ramps are all bridges that were built after the Burnside Bridge with foundations on either side of the existing 86-foot Burnside Bridge width at this location. The structures are within inches of the existing bridge bents including the I-5 southbound bridge and its on-ramp from I-84 to both sides of Bent 21, the I-5 northbound bridge to the west side of Bent 22, and the I-5 northbound off-ramp to I-84 to the west side of Bent 23.

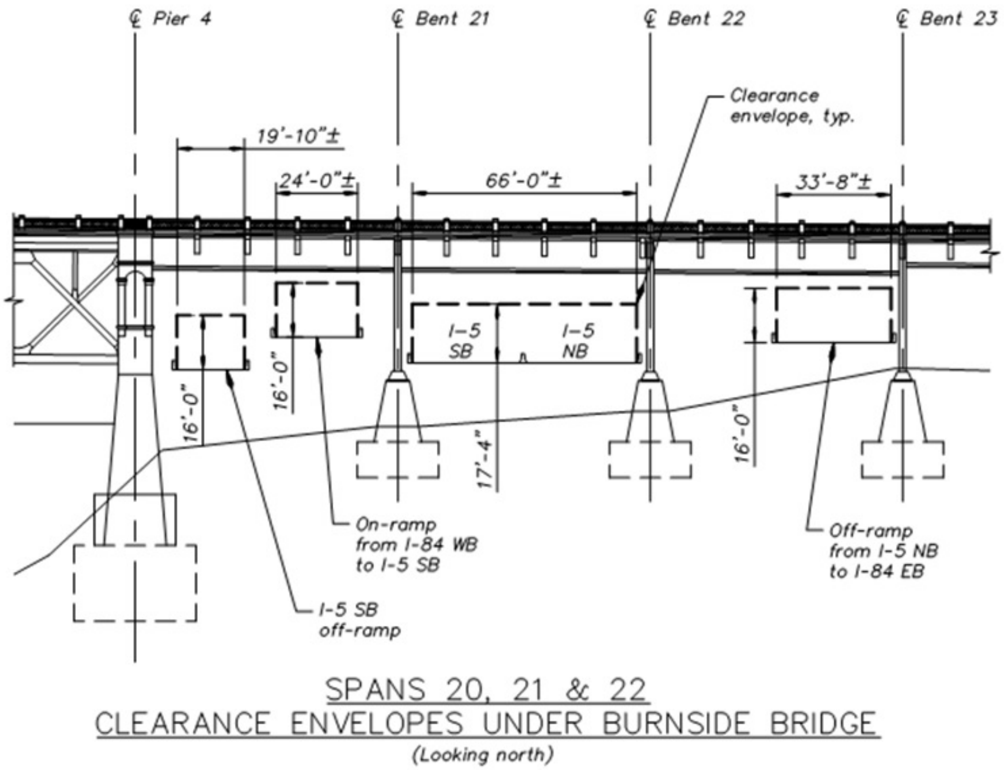


Figure 4-6. ODOT Highway Clearances

4.4.4 Railway Lines under East Approach Spans

Union Pacific Railroad (UPRR) main lines and a railroad spur line pass under Spans 23 and 24 and can be seen in Figure 4-7. The main lines pass to the west side of Bent 24, while the railroad spur line, which does not appear to be in use any longer, passes to the east side Bent 24.

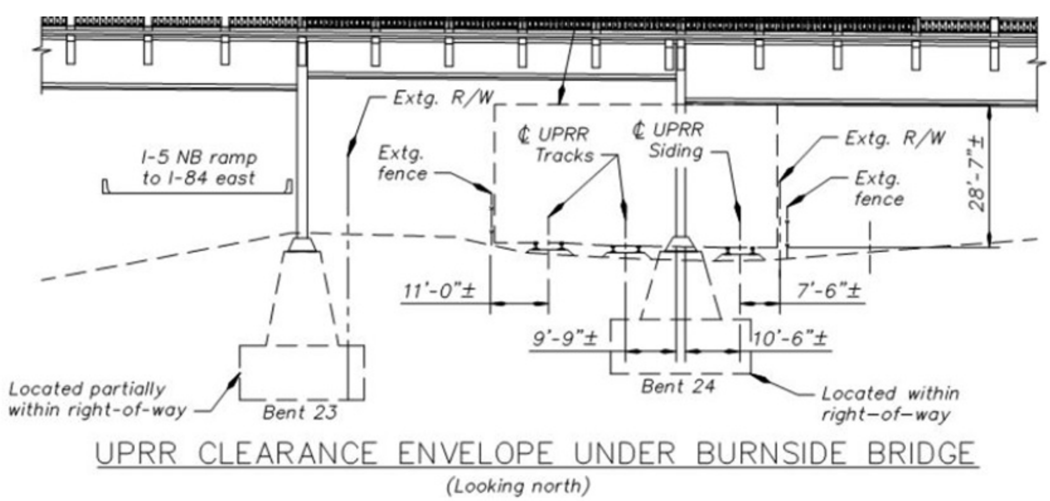


Figure 4-7. Union Pacific Railroad Clearances

4.4.5 TriMet Light Rail under West Approach Spans

The TriMet Red and Blue light rail lines pass under the west approach in Spans 3 and 4, and the Skidmore Fountain station is located under the bridge. The overhead catenary system used to electrify the lines is currently supported from the bridge structure. TriMet light rail clearances are shown in Figure 4-8 and Figure 4-9.

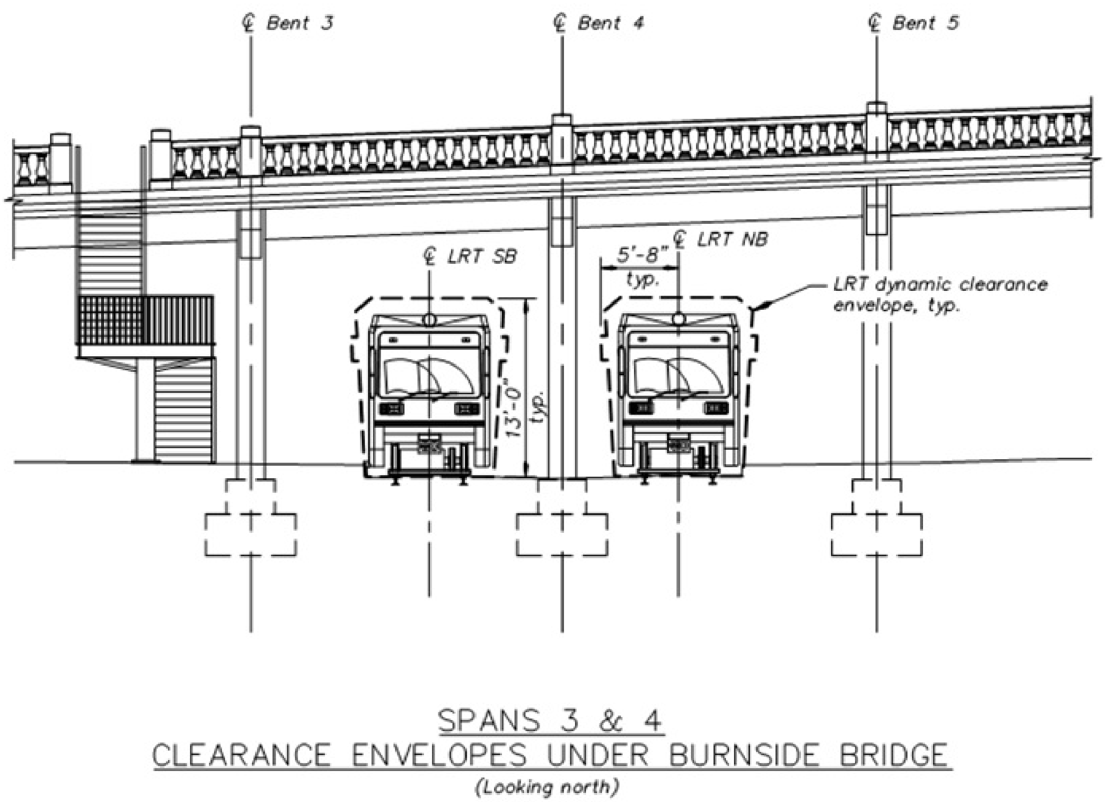


Figure 4-8. TriMet Light Rail Clearances, Spans 3 and 4

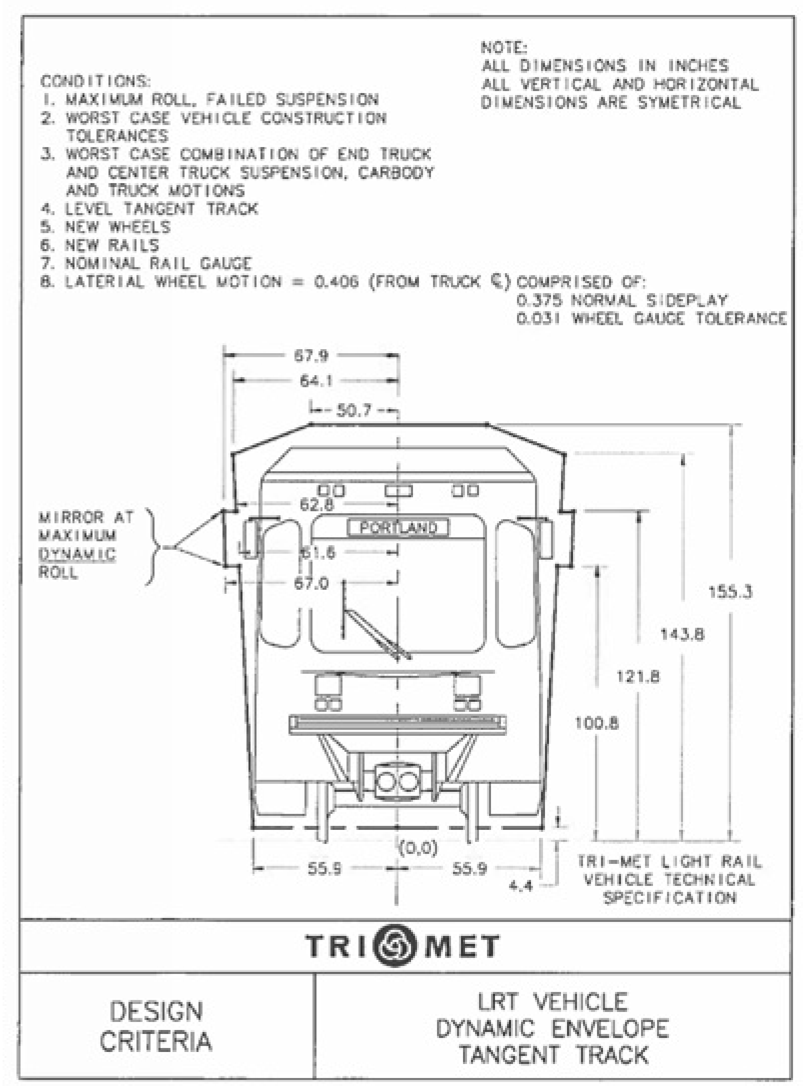


Figure 4-9. TriMet Light Rail Vehicle Dynamic Envelope Tangent Track

4.4.6 City of Portland Facilities

Naito Parkway passes under the west approach in Spans 14 and 15, and the Waterfront Park trail passes under Span 19. Waterfront Park, which houses many community events, extends under the west approach Spans 17 through 19. 2nd Avenue passes under the east approach in Span 26, and 3rd Avenue passes under Span 33. City of Portland facility clearances are shown in Figure 4-10, Figure 4-11, Figure 4-12, and Figure 4-13.

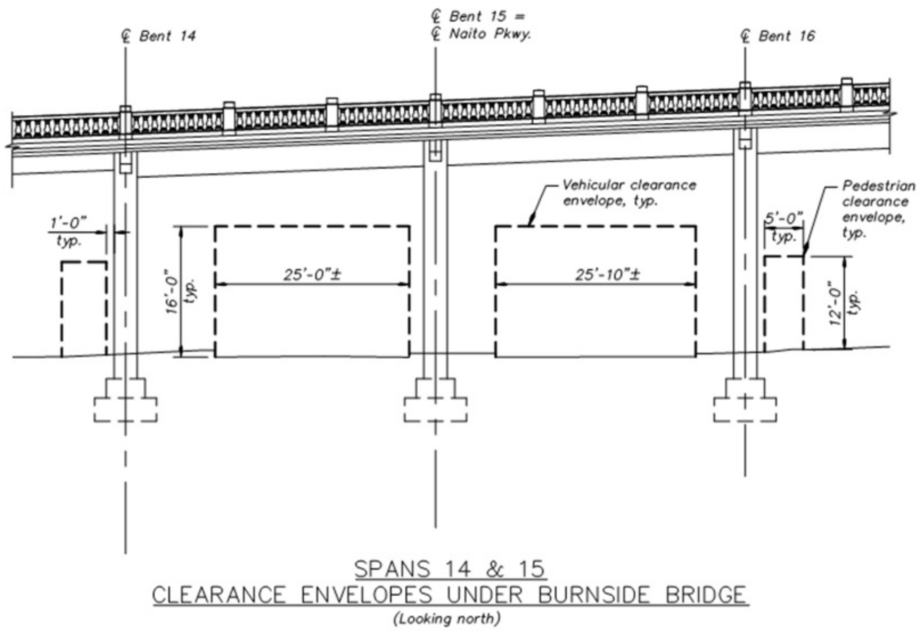


Figure 4-10. Clearance Envelopes under the Burnside Bridge – Spans 14 and 15

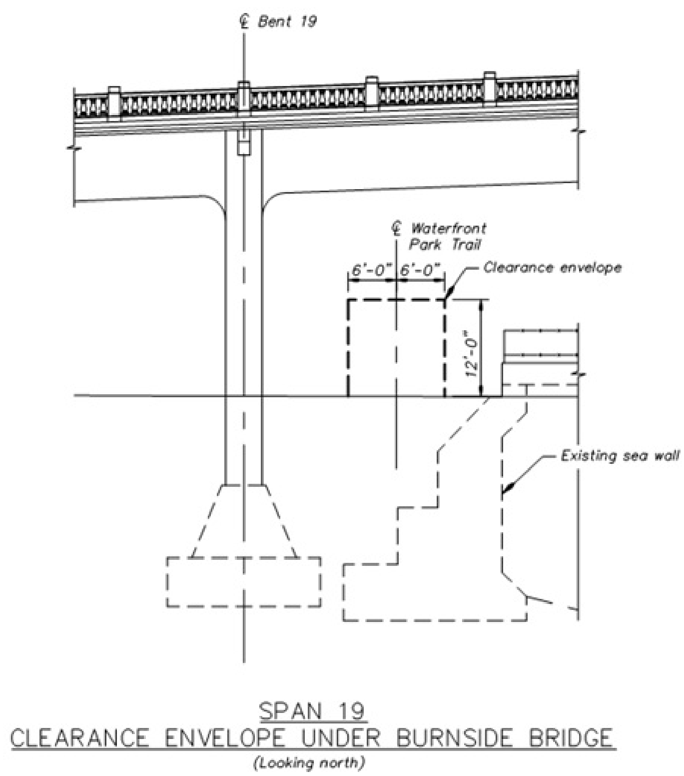
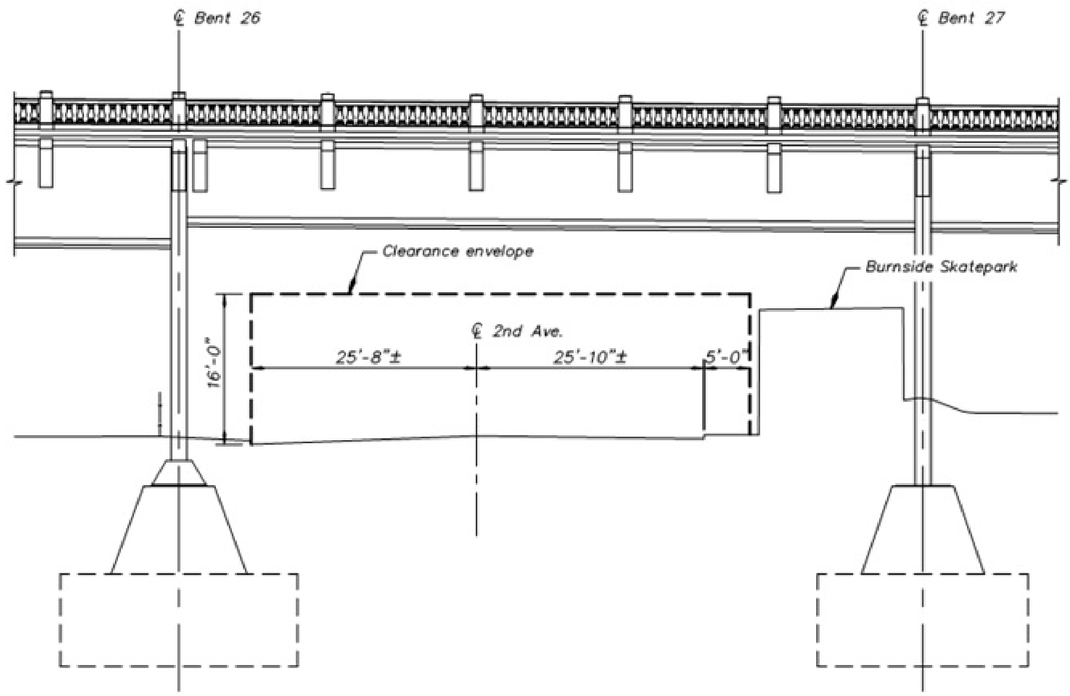
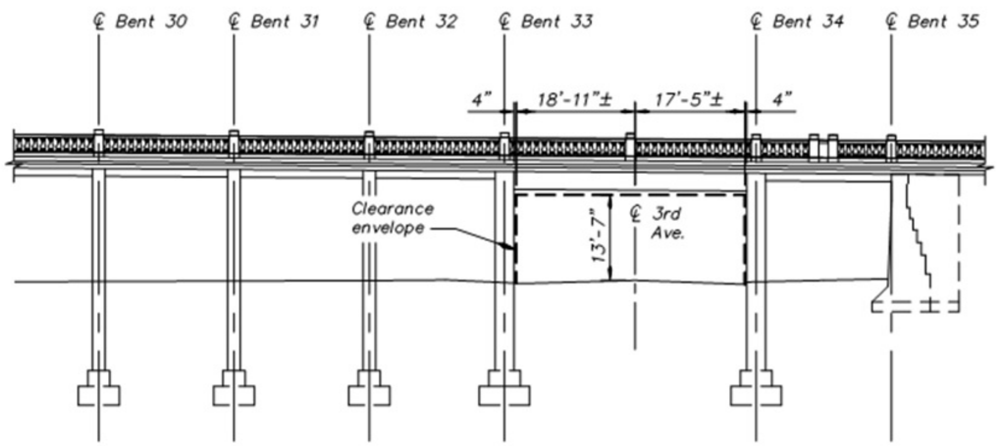


Figure 4-11. Clearance Envelope under the Burnside Bridge – Span 19



SPAN 26
CLEARANCE ENVELOPE UNDER BURNSIDE BRIDGE
(Looking north)

Figure 4-12. Clearance Envelope under the Burnside Bridge – Span 26



SPANS 30, 31, 32 & 34
CLEARANCE ENVELOPES UNDER BURNSIDE BRIDGE
(Looking north)

Figure 4-13. Clearance Envelopes under the Burnside Bridge – Spans 30, 31, 32, and 34

5 Structural Analysis Methodology

5.1 Design and Evaluation Methodologies

For this conceptual-level study, the design and evaluation of the existing structure seismic vulnerability, seismic retrofit needs, and seismic retrofit and widening concepts were primarily conducted using the following methods:

- Review various as-built plans and previous rehabilitation/retrofit plans
- Review previous bridge rehabilitation and retrofit study memos and reports
- Identify seismic vulnerabilities based on engineering judgments and analysis results
- Perform conceptual-level analysis that include hand calculations, spread sheets and dynamic model analysis to support the identification of seismic vulnerabilities
- Develop bridge seismic retrofit and widening concepts according to design code requirements
- Perform dynamic model analysis to support the development of seismic retrofit schemes

The bridge structure was analyzed using the finite element software SAP2000 and the multimodal spectral method to determine force and displacement demands on the critical elements of the structure. Capacities for the critical elements were developed in accordance with the Earthquake Ready Burnside Bridge – Seismic Design Criteria (Appendix B) and compared to the analysis demands. Capacity to demand (C/D) ratios were developed where data are available. A C/D ratio less than one indicates a structural deficiency.

Due to the potential for liquefaction throughout the structure, and poor seismic detailing of the foundation elements, the structure's seismic behavior was modelled considering an assumed liquefaction mitigation and structural foundation retrofit. To do this, conceptual foundation retrofits were developed, and post-retrofit foundation stiffnesses from the geotechnical engineer, were applied to the analysis models.

The structures were analyzed first for the lower level event to determine structural deficiencies in the critical elements. If critical elements did not exhibit deficiencies in the lower level event, the 1000-year event was analyzed for those specific elements to determine if other deficiencies exist.

Conceptual seismic retrofits were developed and sized based on engineering judgment and experience. Where appropriate, conceptual retrofits were incorporated into analysis models to confirm the retrofit could be reasonably expected to resolve the deficiencies in final design.

5.2 Analysis Models

Dynamic analysis is based on site-specific seismic ARS curves developed by the geotechnical engineer for this project. This ARS-based analysis is cost effective and sufficient for this conceptual-level study. (See Figures 3-1 and 3-2).

Three separate models were developed (west approach, east approach, and main river span). Separate analyses were performed on each model using springs as boundary conditions to approximate the response of the adjacent portion of the structure.

5.2.1 Concrete Approach Spans

For the West and East Approach Bridge models, simplified three-dimensional "spine" models were developed. For the Main River Span Bridge, a three-dimensional model was developed. The following model descriptions and assumptions were incorporated:

5.2.1.1 West Approach Bridge

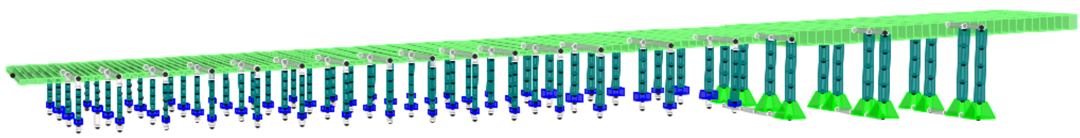


Figure 5-1. West Approach Bridge Analysis Model

- Expansion joints at Bents 1, 5, 8, 11, 14, 16, 17, 18, and 19 were modeled as closed with superstructure moment releases at the following eight locations:
  - Span 4 at Bent 5
  - Span 7 at Bent 8
  - Span 10 at Bent 11
  - Span 13 at Bent 14
  - Span 15 at Bent 16
  - Span 17 at Bent 17
  - Span 18 at Bent 18
  - Span 19 at Bent 19
- Analysis does not include Design Live Load (HL-93) vehicle effects.
- All demands are elastic. Demands may be limited by overstrength of the columns.
- For the RCDG spans, average section properties were developed to be representative of the average girder spacing and deck width for that span. The cantilevered sidewalk and fascia beam were treated as dead weight only and did not contribute to the superstructure section properties.
- Column heights were defined from bottom of end floor beam to top of pile cap or spread footings. As-built column heights were used at all locations.

- For concrete columns, gross section properties were used to determine CSZ event demands, and cracked section properties were used to determine 1000-year event demands. Consideration was given to using cracked sections for the CSZ event in an attempt to improve the C/D ratios for that event, but since C/D ratios for column displacement in the 1000-year event were also low, indicating that column retrofit was required, it was deemed not necessary to analyze the CSZ event with cracked column sections.
- Post-retrofit and footing rocking analyses were not conducted, aside from assuming that all foundations would require retrofit.
- Existing retrofit devices were included in capacity calculations.

5.2.1.2 East Approach Bridge

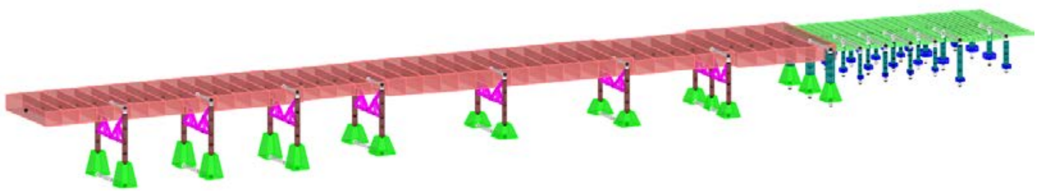


Figure 5-2. East Approach Bridge Analysis Model

- Moments were released at the top of all columns at Bent 28 to capture the behavior of the pinned bars at Bent 28.
- Expansion joints at Bents 22, 24, 26, 31, 33, and 34 were modeled as closed with superstructure moment releases at the following six locations:
  - Span 21 at Bent 22
  - Span 23 at Bent 24
  - Span 25 at Bent 26
  - Span 30 at Bent 31
  - Span 32 at Bent 33
  - Span 34 at Bent 34
- Analysis did not include Design Live Load (HL-93) vehicle affects.
- Roadway slabs were modeled with a 0.5-inch sacrificial wearing surface, assigned to the roadway as additional mass and dead load.
- All demands were elastic. Demands may be limited by overstrength of the columns.
- For the plate girder spans, average section properties were developed to be representative of the plate sizes, girder spacing, and deck width for that span. The deck was considered to be composite. The cantilevered sidewalk and fascia beam were treated as dead weight only and did not contribute to the superstructure section properties.

- For the reinforced concrete deck girder spans, average section properties were developed to be representative of the average girder spacing and deck width for that span. The cantilevered sidewalk and fascia beam were treated as dead weight only and did not contribute to the superstructure section properties.
- Column heights were defined from bottom of end floor beam to top of pile cap or spread footings. As-built column heights were used at all locations.
- The concrete encasement of Spans 20 to 27 and Bents 21 to 27 was treated as dead load only and did not contribute to the section properties of the steel elements.
- For concrete columns, gross section properties were used to determine CSZ event demands and cracked section properties were used to determine 1000-year event demands. Consideration was given to using cracked sections for the CSZ event in an attempt to improve the C/D ratios for that event, but since C/D ratios for column displacement in the 1000-year event were also low, indicating that column retrofit was required, it was deemed not necessary to analyze the CSZ event with cracked column sections.
- Post-retrofit and footing rocking analyses were not conducted, aside from assuming that all foundations would require retrofit.
- Existing retrofit devices were included in capacity calculations.

5.2.1.3 West and East Approach Bridge Widening Models

- The superstructure properties of Spans 14 to 19 and 20 to 27 were modified to account for additional dead load and stiffness based on the average superstructure widening in each span.
- New elements and those requiring structural modification to support the bridge widening were not evaluated for seismic adequacy. It is assumed that the modifications will be designed to address any seismic-related deficiencies.

5.2.2 Main River Span Bridge

- A continuous three-dimensional SAP (version 18.1) analysis model was created for the fixed spans, bascule spans, and Piers 1 to 4, (Figure 5-3).

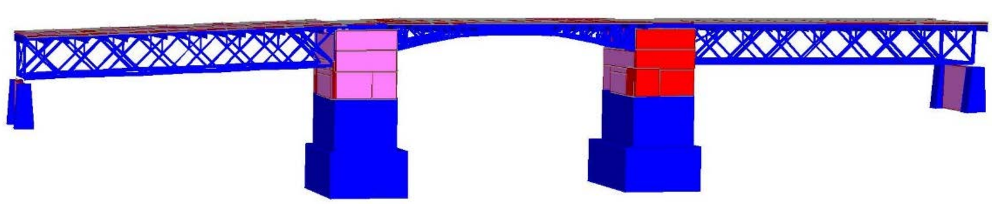


Figure 5-3. Main River Span Analysis Model

- All steel elements were modeled as two force members with moments released at both ends, for both axes, except where noted.
- Two force members were modeled with the gross area noted in the plans dated December 22, 1923 (Bascule Spans) (Multnomah County 1923), and February 5, 1924 (Fixed Spans) (Multnomah County 1924). Where gross area was not available, it was calculated using dimensions from shop drawings.
- Flexural members (brackets, floor beams, stringers, beginning bascule members, end bascule members, trunnion posts, trunnion struts, and live load supports) were first modeled in AutoCAD to determine relevant section properties. These members were then defined in SAP as "generic members" with the section properties input manually.
- All slabs, sidewalks, and walls were input as "thin shell" elements with the appropriate as-built thicknesses, at the structural center of gravity of the element.
- Roadway slabs were modeled with a 0.5-inch sacrificial wearing surface, assigned to the roadway as additional mass and dead load.
- Columns were modeled as continuous frame elements.
- Slabs and sidewalks were modeled without structural stiffness in the "dead load" model.
- Slabs in the composite models, walls, and columns were modeled with 50 percent structural stiffness to approximate cracked conditions.
- Structural systems were connected using rigid links (slab to truss, truss to piers, etc).
- Relatively stiff support elements (buttresses, bearings, and pedestals) were modeled as links, with weights and masses assigned to node points.
- Piers 2 and 3 used body constraints at the bottoms of walls and tops of columns to simulate relative stiffness of the structures.
- The footings of Piers 1 and 4 were modeled with links from the bottoms of columns to tops of piles.
- Non-structural elements (control towers, rails, sidewalk and roadway stringers, etc.) were added to the model and assigned to the nearest structural elements as loads and masses.
- The seismic event was modeled using a response-spectrum analysis based on the CSZ event provided to Parametrix by Shannon & Wilson (Appendix C).
- Details for the pit deck stringer longitudinal supports were not available. End restraints were modeled as expansion joints where connected to the fixed span and fully supported where connected to the bascule span.

5.2.2.1 Fixed Spans

- Fixed spans were connected to piers with "roller" type links on the shore side (Piers 1 and 4), and standard links on the river side (Piers 2 and 3).

- A self-load increase of 45.8 percent was determined using the 15 kip per linear foot truss weight estimate on page T30 of the February 1924 design drawings (Multnomah County 1924). These loads were verified by comparing the load effects calculated in the "original configuration" SAP model with the loads originally shown on page T30.
- Sway bracing on the shore side was noted in the inspection reports, but was not detailed in the design documents. Bracing was assumed to be similar to adjacent bracing and was included in the model.
- Roadway slab height and thickness were adjusted based on ODOT rehabilitation drawings dated September 2001 (Multnomah County 2001).

Bascule Spans

- A self-load increase of 44.5 percent was calculated using "lifting load" estimates from the shop drawings. These loads were verified by comparing the load effects calculated in the "original configuration" SAP model with the loads originally shown on page S2 of the December 1923 design drawings (Multnomah County 1923).
- Bascule span lock was modeled by releasing all axial and moment forces where bascule end chords met. This simulated a mechanism which only transmits shear forces.
- Bascule chords were connected to the trunnions using custom links which do not transmit moment, allowing the bascule to freely rotate about the nodes.
- As-built Node 14 was connected to the live load shoe using a link which only transmits vertical loads. It was not possible to release this connection in tension (upward movement of the bascule) with the response-spectrum analysis.
- At all stages of the model, the weight of the counterweight was determined by adjusting the load until the dead load moment about the trunnion was as near zero as possible.
- Bascule spans do not have adequate lateral support to resist movement, all truss members within Piers 2 and 3 were fully modeled to account for their lateral stiffness.
- As-built chord members 5-3 and 3-1 are built up sections and were modeled as flexural members.
- Roadway slab height and thickness were adjusted based on ODOT rehabilitation drawings dated September 2005 (Multnomah County 2005).
- The trunnion post-seismic restraints noted in page 33 of the ODOT rehabilitation drawings dated September 2005 (Multnomah County 2005) were included as fully defined frame members.

For a bascule bridge such as the Burnside Bridge, the limits on allowable displacement can be in a fractions of an inch for the bascule leaf machinery driving system. At a conceptual design level, the analysis model is not capable of precisely predicting the small relative movement within a fraction of an inch. However, conceptual-level analysis has provided predictable ranges and trends, which are used in this conceptual design

5.3 Boundary Conditions

The west approach and east approach models used spring constants, provided by the geotechnical engineer, for vertical and horizontal displacements at the bottoms of all columns. After a sensitivity study of foundation springs, rotations in all directions of the column bases were set as “fixed.” Where the approach structures meet the Main River Spans (Piers 1 and 4), models used springs for displacement and rotations along and about the horizontal axes to approximate the stiffness of the main spans. Vertical displacements and rotations about the vertical axis were fixed. At the abutments, models used springs to approximate the stiffness of the soil behind the abutment.

All the main span and bascule piers (Piers 1, 2, 3, and 4) were modeled as fixed at the tops of piles for dead load models. For seismic models, rotation was assumed to be relatively fixed, while lateral movement was restrained by springs determined by matching maximum seismic displacements with the load-displacement graphs provided by Shannon & Wilson. Three iterations were used for each lateral spring. Vertical springs were determined by matching dead load reactions with load-displacement graphs.

Design drawings from September 2001 (Multnomah County 2001) show seismic restraints tying the fixed spans to the approach spans. Support springs were assigned to the top chords of the fixed spans to model these connections.

6 Existing Structure and Seismic Vulnerabilities and Deficiencies

6.1 West Approach Bridge

The West Approach Bridge (Spans 1 to 19) consists of reinforced concrete deck girder (RCDG) spans in two main configurations. Spans 1 to 13 consist of three- and four-span continuous units with constant-width RCDG spans framing into end floor beams at each bent. End floor beams are supported by four reinforced concrete columns on reinforced concrete spread footings. Expansion floor beams are present at Bents 5, 8, and 11. Spans 14 to 19 consist of one- and two-span units with variable-width RCDG spans having intermediate floor beams and main supporting girders framing into end floor beams at each bent. Expansion floor beams are present at Bents 14, 16, 17, 18, and 19. Bent floor beams are supported by four columns. Bents 14 to 17 are supported by spread footings while Bents 18 and 19 are supported by reinforced concrete caps on timber piles. A Phase I seismic retrofit was completed in 2001, which provided restrainers at the expansion bents throughout the West Approach Bridge spans.

The following seismic vulnerabilities were identified within the West Approach Bridge:

- Seismic Restrainer – Insufficient strength. The existing restrainers at expansion bents are assumed to be inadequate due to increased seismic loading demand requirements that have been developed in the years since the original Phase I seismic retrofit. Modification of the restrainers will also be required due to floor beam strengthening that is described below; therefore, analysis of the demands and capacities of the existing restrainers was not required (Figure 6-1).

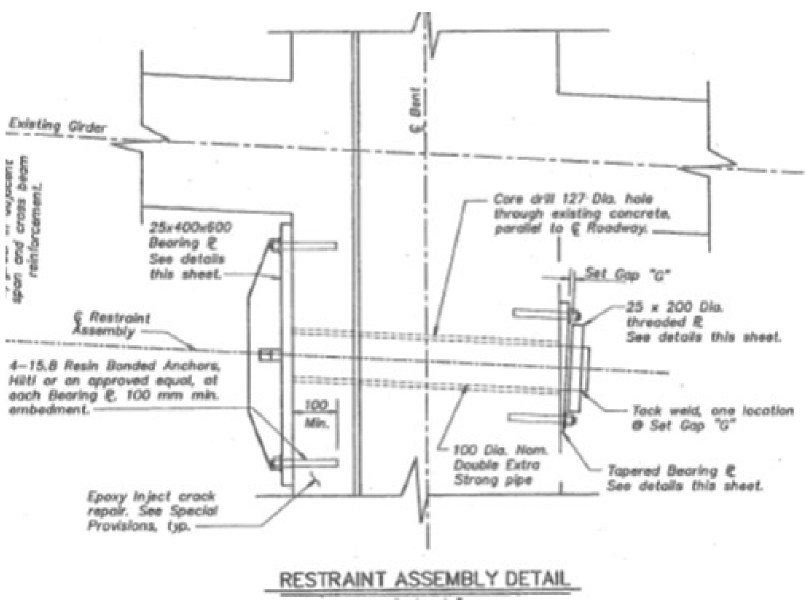


Figure 6-1. Seismic Restrainer Vulnerability

- Superstructure Flexural or Shear Strength – Insufficient strength. The girder's positive moment reinforcement is spliced at the column connection, limiting moment capacity due to inadequate development length. Analysis of the existing structure for the CSZ event shows the C/D ratios for the fixed side of the expansion bents in the west approach are less than 0.75 while C/D ratios for the fixed bents are as low as 0.84 (Figure 6-2).

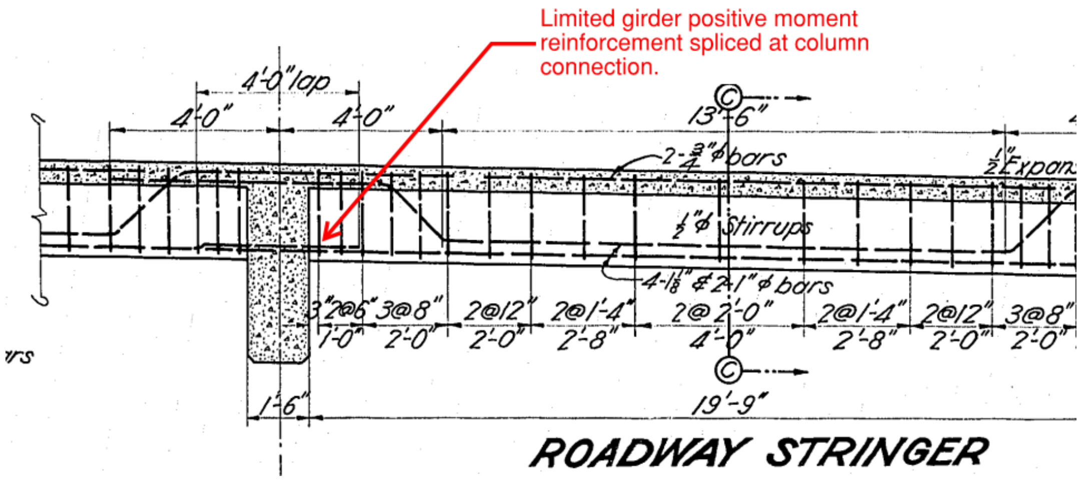


Figure 6-2. Superstructure Vulnerability

- End Floor Beam Flexural or Shear Strength – Poor seismic detailing. The floor beam positive moment reinforcement is spliced and/or has limited embedment at column connection, limiting its moment capacity due to inadequate development length. At isolated locations, inadequate negative moment capacity at midspan that is not sufficient to maintain elastic behavior for a design level seismic event. Analysis of the existing structure for the CSZ event shows the C/D ratios for positive moment at the columns are less than 0.75 (Figure 6-3).

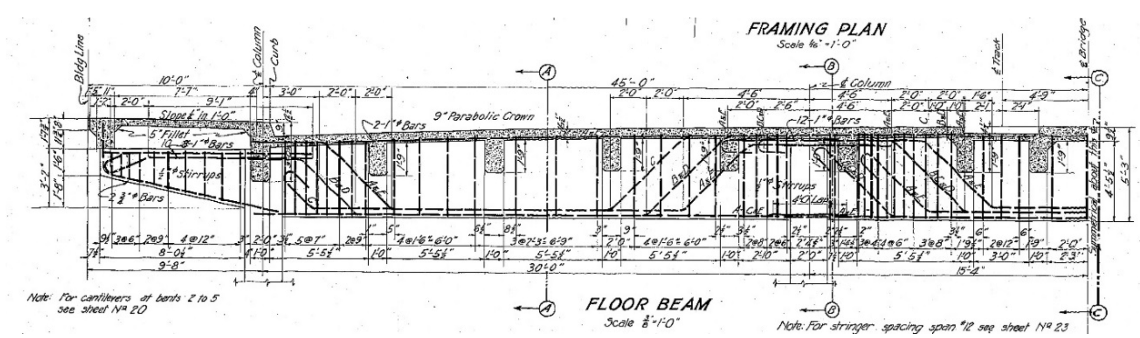


Figure 6-3. Floor Beam Vulnerability

- Column Flexural or Shear Strength – Insufficient strength. There is very little longitudinal column reinforcing extending into footing, compounded by inadequate development length. There is poor confinement and a lack of seismic hooks, with ties and hoops at 1-foot 3-inch spacing. C/D ratios for column flexure and shear for the CSZ event are less than 0.75 in some bents and C/D ratios for column displacement at the 1000-year event are less than 0.75 for all bents (Figure 6-4).

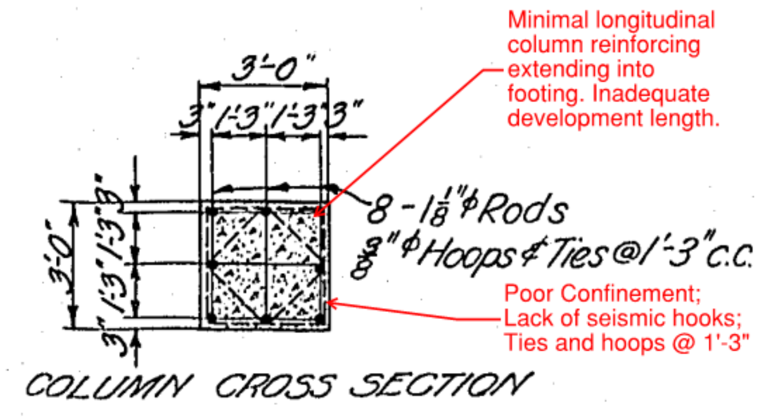


Figure 6-4. Column Vulnerability

- Footing Size and Strength – Unreinforced Footings. The small footing size is inadequate to resist overturning and to limit settlement from liquefaction to a desirable level. Further, the unreinforced footing section has no top mat of reinforcement. The poor connection detail as column reinforcement does not extend into the footing with adequate embedment. Due to the lack of reinforcing in the footing, and liquefaction effects, a foundation retrofit is required because a reasonable load path for seismic forces in these elements does not exist. (Figure 6-5).

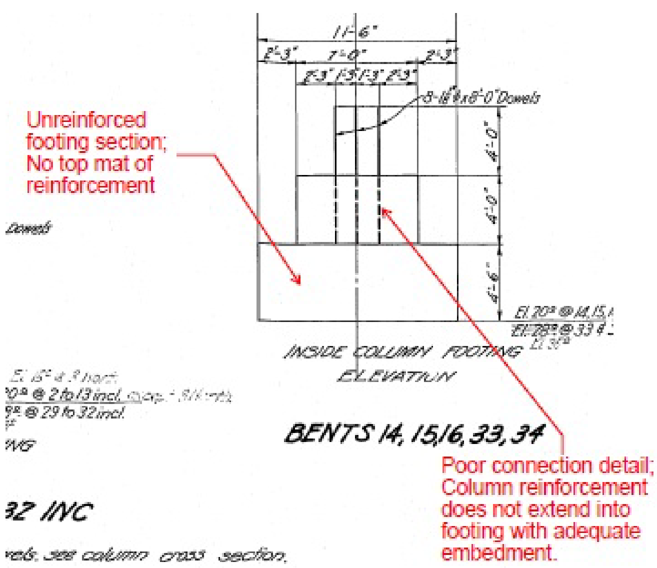


Figure 6-5. Spread Footing Vulnerability

- Timber Pile Lateral Strength and Uplift Capacity – Insufficient pile strength. The pile group capacity is inadequate to limit settlement from liquefaction, resist uplift and downdrag forces, and resist displacements and forces from lateral spreading. The unreinforced footing section has no top mat of reinforcement. It has a poor connection detail as the column reinforcement does not extend into pile cap with adequate embedment. Embedment of piles into pile caps is inadequate to resist seismic uplift forces. Due to the amount of liquefaction and lateral spread at Bents 17 - 19, it was determined that foundation retrofit is required because a reasonable load path for seismic forces in these elements does not exist. (Figure 6-6).

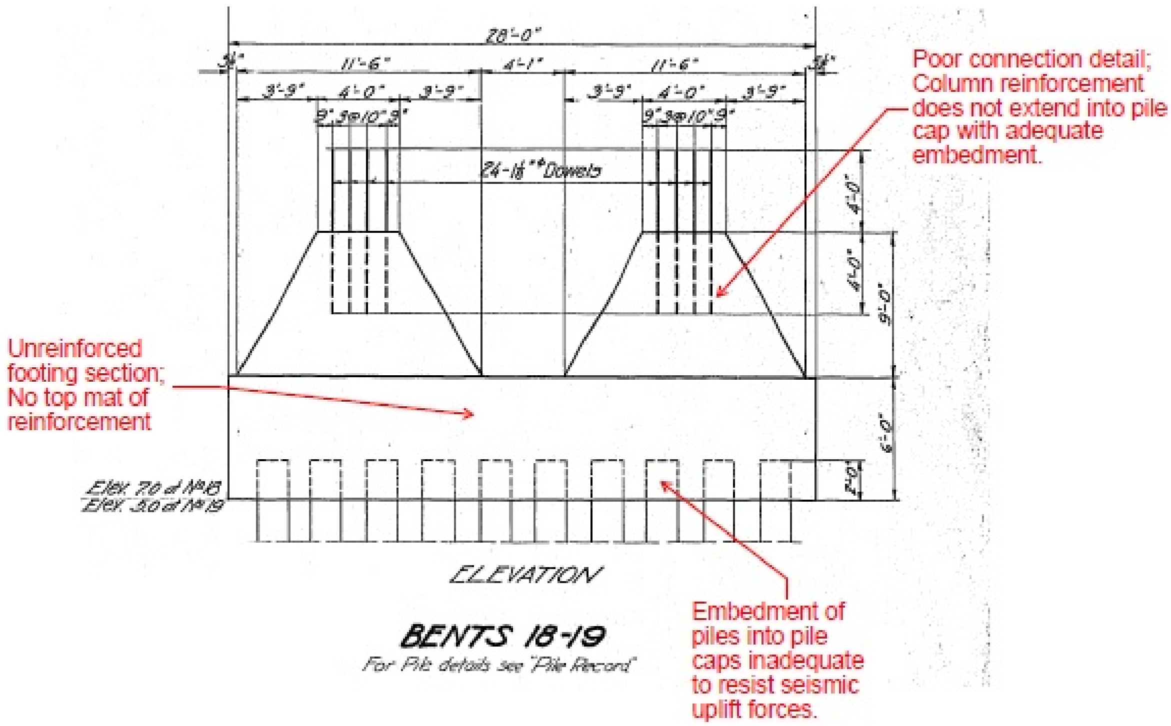


Figure 6-6. Pile Foundation Vulnerability

- Tall Abutment Retaining Wall Footing Size – Insufficient strength. The abutment's narrow footing size is inadequate to resist overturning and limit effects of vertical and differential settlement from liquefaction to a desirable level. Additionally, the abutment wall is unreinforced. Due to the lack of reinforcing in the abutment and the liquefaction effects, it was determined that a retrofit is required because a reasonable load path for seismic forces in these elements does not exist. (Figure 6-7).

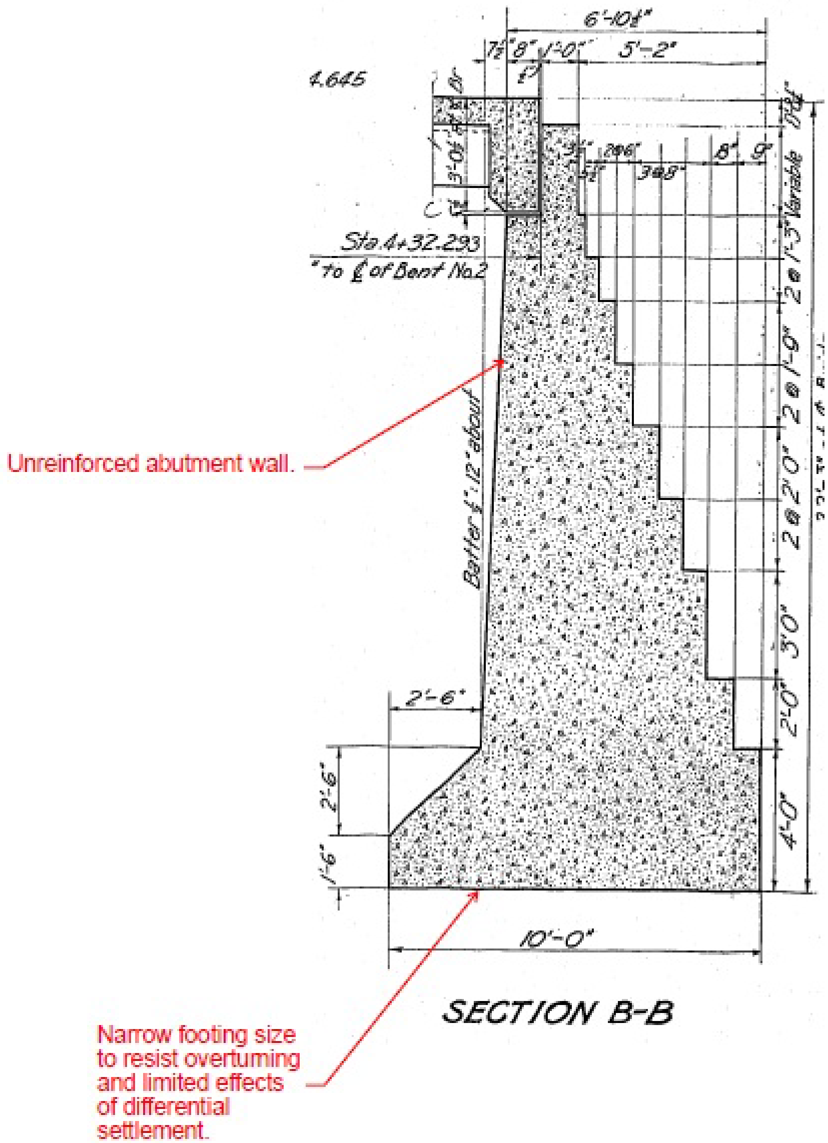


Figure 6-7. Abutment Vulnerabilities

- Liquefiable Soils and Lateral Spreading – Geotechnical hazards. Bents 1 to 17 are on spread footings with limited bearing capacity to resist overturning and liquefaction-induced settlement. Bents 18 and 19 are on timber piles with limited capacity to resist liquefaction-induced settlement and lateral forces and displacements due to lateral spreading. Additional detail regarding the liquefaction and lateral spread analysis can be found in Chapter 7.

6.2 East Approach Bridge

The East Approach Bridge (Spans 20 to 34) consists of two main span configurations: (1) concrete-encased steel plate girder spans (Spans 20 to 27) and (2) RCDG spans (Spans 28 to 34).

6.2.1 East Approach Spans 20 to 27

Spans 20 to 27 are two-span continuous units of deep steel plate girders encased in concrete. The two, or three in some spans, plate girders support the concrete-encased steel floor beams, the reinforced concrete stringers, and the concrete deck. The end floor beams are supported on concrete-encased steel bents with diagonal cross bracing. Bents 21 to 27 are supported by reinforced concrete caps on timber piles.

The following seismic vulnerabilities were identified within the East Approach Bridge steel girder spans (Spans 20 to 27):

- Seismic Restrainer Strength – The existing restrainers at expansion bents are assumed to be inadequate due to increased seismic loading demand requirements that have been developed in the years since the original Phase I seismic retrofit. Modification of the restrainers will also be required due to the floor beam strengthening that is described below; therefore, analysis of the demands and capacities of the existing restrainers was not required.
- Superstructure Flexural or Shear Strength – The strength of the superstructure is potentially a concern after the substructure is strengthened. Where girders are continuous, the riveted column moment connection was not originally designed to resist additional moment from seismic loading. The reduced flange section at the fixed end impacts the moment capacity of the girder/floor beam connection. Analysis of the existing structure for the CSZ event resulted in a minimum C/D ratio of 1.07, but as discussed above, C/D ratios are expected to be below 1.0 following the foundation substructure retrofit and considering overstrength demands (Figure 6-8).

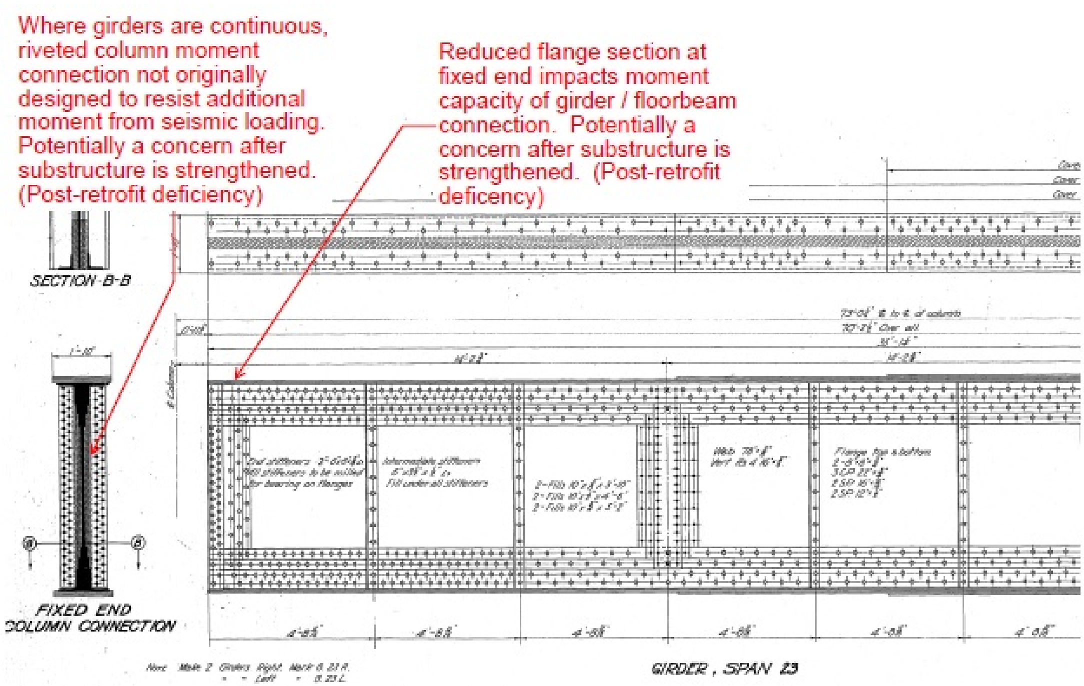


Figure 6-8. Superstructure to Column Connection Vulnerability

- Steel Rocker Bearings – Rocker bearings are not stable for larger displacements and are likely to tip over. Also, the rocker was not designed to restrict transverse movement. Longitudinal restrainers installed in the early 2000s tied the superstructure together near deck level and do not restrict transverse movement of the superstructure. Retrofit of the steel rocker bearings was determined to be required based on the lack of a reasonable load path for seismic loading; therefore, a detailed analysis of the demands and capacities of the rocker bearings was not required (Figure 6-9).

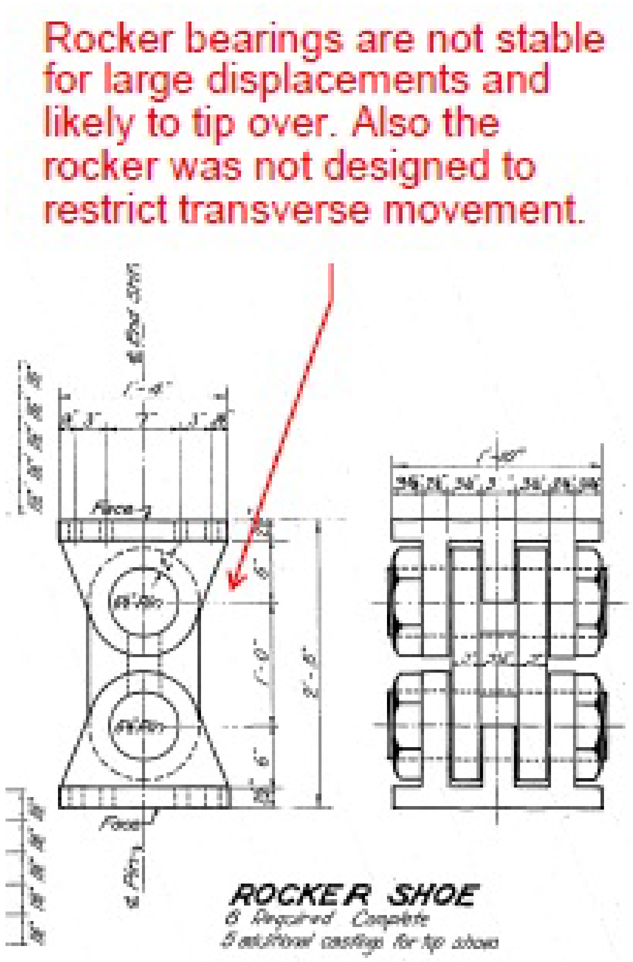


Figure 6-9. Rocker Bearing Vulnerability

- End Floor Beam Flexural or Shear Strength – Although initially acceptable, the superstructure strength is becomes deficient after the substructure columns is strengthened. The riveted connection was not originally designed to resist additional moment from seismic loading, and this is exacerbated by needing to be capacity protected against the column. There is a relatively long cantilever supporting a portion of the roadway. Analysis of the existing structure for the CSZ event resulted in C/D ratios for all end floor beams of less than 0.75 for positive moment at the columns (Figure 6-10).

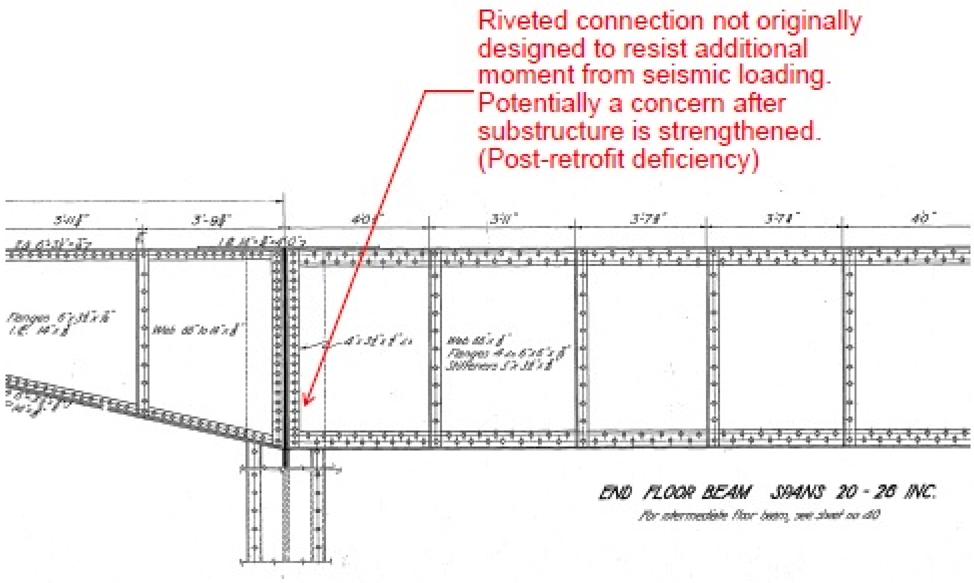


Figure 6-10. Steel Floor Beam Connection Vulnerability

- Column Flexural or Shear Strength – Columns are poorly anchored to footing pedestals. Anchors do not extend into the pile caps. Column orientations do not consider seismic-induced transverse movement. Limited weak axis flexural strength in-plane of bent. Analysis of the existing structure for the CSZ event resulted in C/D ratios for all steel columns of less than 0.75 when evaluating axial-flexure interaction in the columns and tension in the anchor bolts (Figure 6-11).

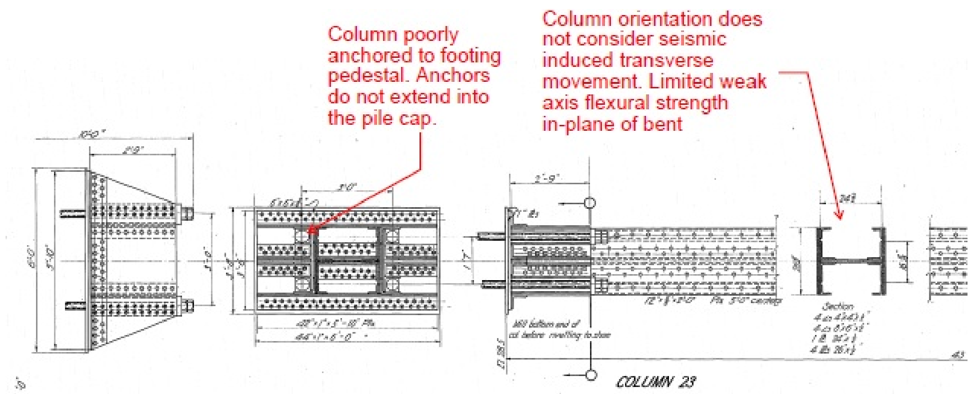


Figure 6-11. Steel Column Vulnerability

- Column Sway Bracing Strength – Column sway bracing was likely designed for limited wind loading. The rivet connections were not designed to resist cyclic seismic-induced moments. The sway bracing horizontal is located at approximately mid-height of the column and stiffens the bent. Analysis of the existing structure for the CSZ event resulted in C/D ratios less than 0.75 for the steel column bracing (Figure 6-12).

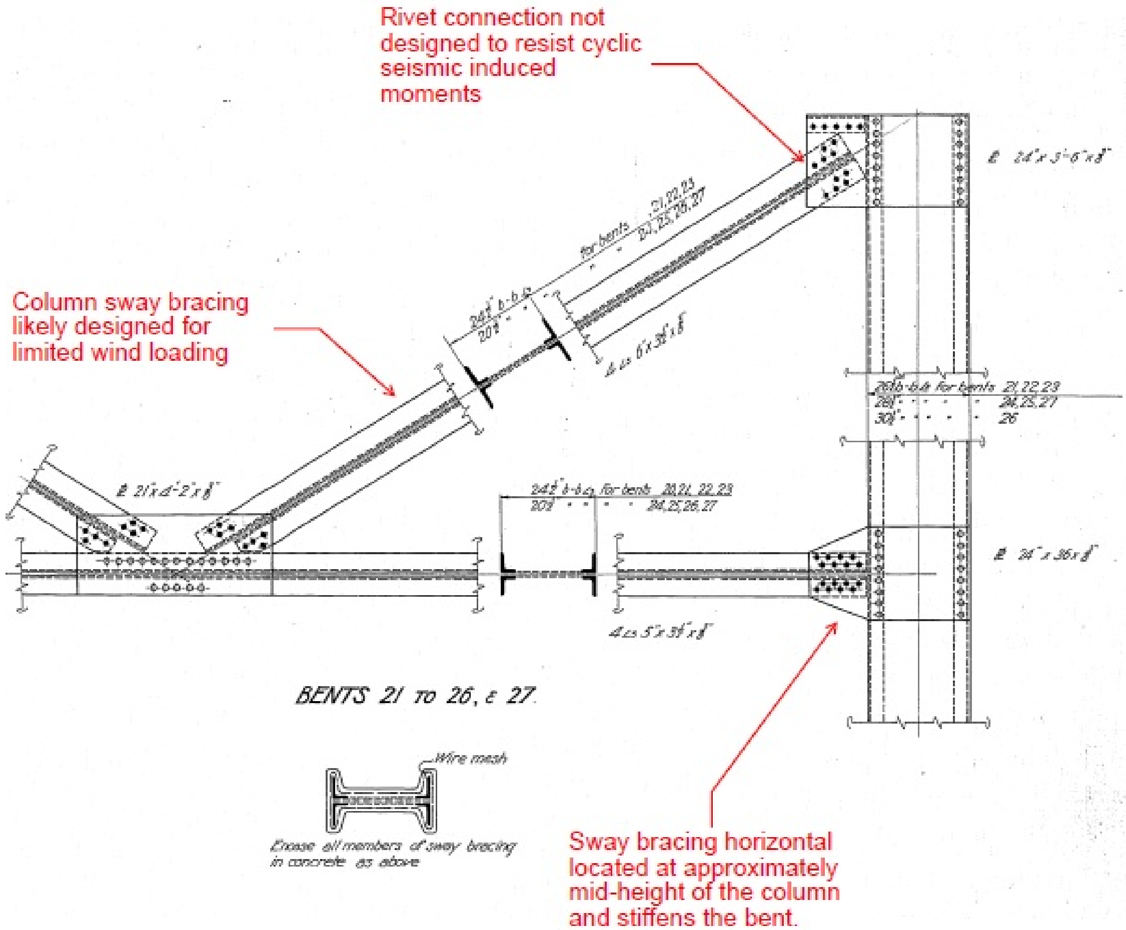


Figure 6-12. Column Bracing Vulnerability

- Timber Pile Lateral Strength and Uplift Capacity – Pile group capacity is inadequate to limit settlement from liquefaction to a desirable level, resist uplift and downdrag forces, and to resist displacements and forces from lateral spreading. Unreinforced footing section with no top mat of reinforcement. Poor connection detail as column anchorage does not extend into pile cap with adequate embedment. Embedment of piles into pile caps is inadequate to resist seismic uplift forces. Due to the amount of liquefaction and lateral spread at Bents 21 through 27, it was determined that foundation retrofit is required because a reasonable load path for seismic forces in these elements could not be found, and structural analysis of the demands and capacities of the timber piles and associated pile caps was not required (Figure 6-6).
- Liquefiable Soils and Lateral Spreading – Bents 21 to 27 are on timber piles with limited capacity to resist liquefaction-induced settlement and lateral forces and displacements due to lateral spreading. Additional detail regarding the liquefaction and lateral spread analysis can be found in Chapter 7.
- Damage from Adjacent Structure – Bents 21 to 23 are adjacent to various highway structures (on-ramp from I-84 westbound to I-5 southbound, I-5 main line north and southbound, and I-5 northbound to I-84 eastbound), which could impact each other

during a seismic event resulting in catastrophic damage to the columns of Bents 21 to 23 (Figure 6-13).

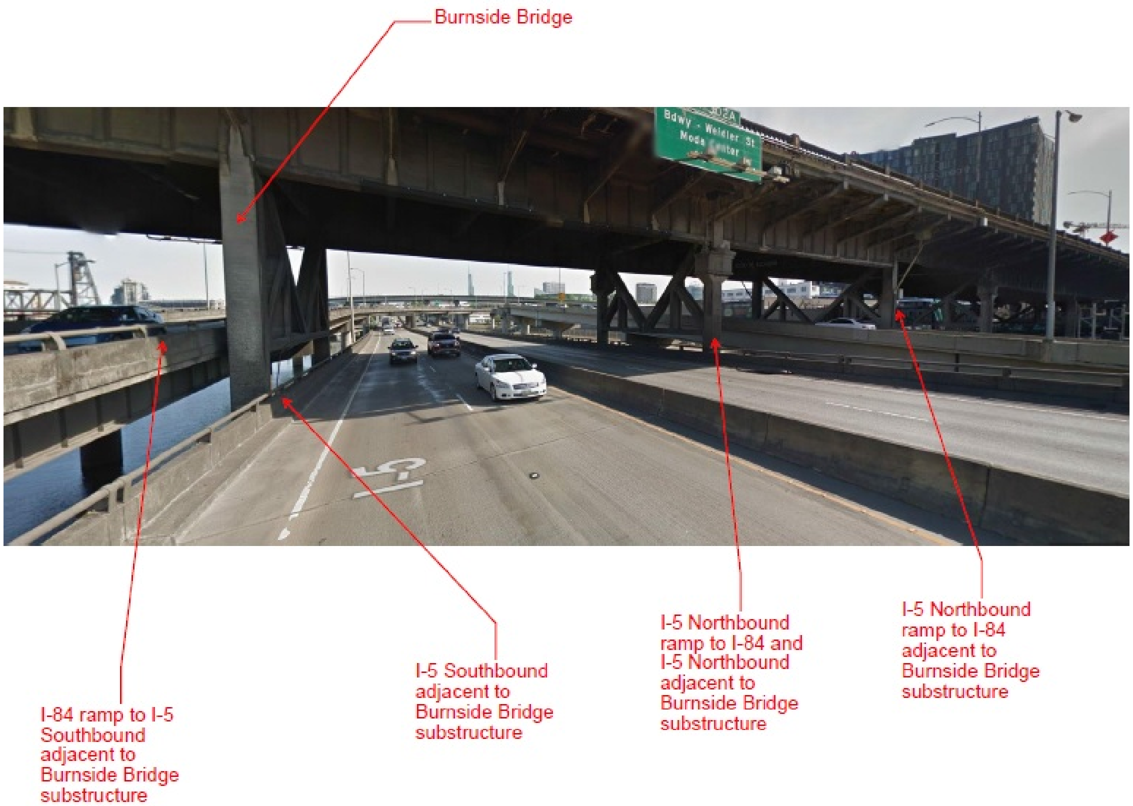


Figure 6-13. Vulnerability to Impact from Adjacent Structures

6.2.2 East Approach Spans 28 to 34

Spans 28 to 34 are RCDG spans that match the description of Spans 1 to 13 in the West Approach Bridge (provided earlier) and the following seismic vulnerabilities were identified with these spans:

- Superstructure Flexural or Shear Strength – Poor seismic detail. Girder positive moment reinforcement spliced at column connection thus limiting moment capacity due to inadequate development length. Analysis of the existing structure for the CSZ event shows the C/D ratios for superstructure flexure at the bents to be less than 0.75 (Figure 6-2).
- End Floor Beam Flexural or Shear Strength – Poor seismic detailing. Floor beam positive moment reinforcement spliced or has limited embedment at column connection thus limiting moment capacity due to inadequate development length. At isolated locations, inadequate negative moment capacity at midspan to maintain elastic behavior for CSZ event. Analysis of the existing structure for the CSZ event shows the C/D ratios for positive moment at the columns are less than 0.75 (Figure 6-3).

- Column Flexural or Shear Strength, Poor Confinement Detailing – Minimal longitudinal column reinforcing extending into footing with inadequate development length. Poor confinement and lack of seismic hooks, with ties and hoops at 1-foot 3-inch spacing. C/D ratios for column displacement for the 1000-year event are less than 0.75 (Figure 6-4).
- Footing Size and Strength, Unreinforced Footings – Bents 28 to 35 on small spread footings with limited bearing capacity to resist overturning. Unreinforced footing section with no top mat of reinforcement. Poor connection detail as column reinforcement does not extend into the footing with adequate embedment. Due to the lack of reinforcing in the footing, it was determined that foundation retrofit is required because a reasonable load path for seismic forces in these elements could not be found, and structural analysis of the demands and capacities of the footings was not required (Figure 6-5).
- Tall Abutment Retaining Wall Footing Size – Narrow footing size to resist overturning. Fixed end connection between superstructure and abutment imparts seismic loads on unreinforced abutment wall. Due to the lack of reinforcing in the abutment it was determined that retrofit is required because a reasonable load path for seismic forces in these elements could not be found and structural analysis of the demands and capacities of the abutment was not required. (Figure 6-6).

6.3 Steel Truss Fixed River Spans

Two steel truss fixed river spans connect the east and west approach structures to the main river bascule span over the water. The west truss span connects the west approach structure at Pier 1 to the bascule span structure at Pier 2. The east truss span connects the east approach structure at Pier 4 to bascule span structure at Pier 3. Steel truss fixed-span elevation and section views are shown in Figure 6-14.

Seismic vulnerabilities of these two spans under the CSZ and 1000-year events were identified during the study, based on conceptual analysis, review of as-built plans and previous study documents.

The analysis was conducted step by step to identify the required retrofit. Table 6-1 below indicates that there are members that have C/D ratios less than 1.0, under a CSZ event, therefore those members need to be strengthened.

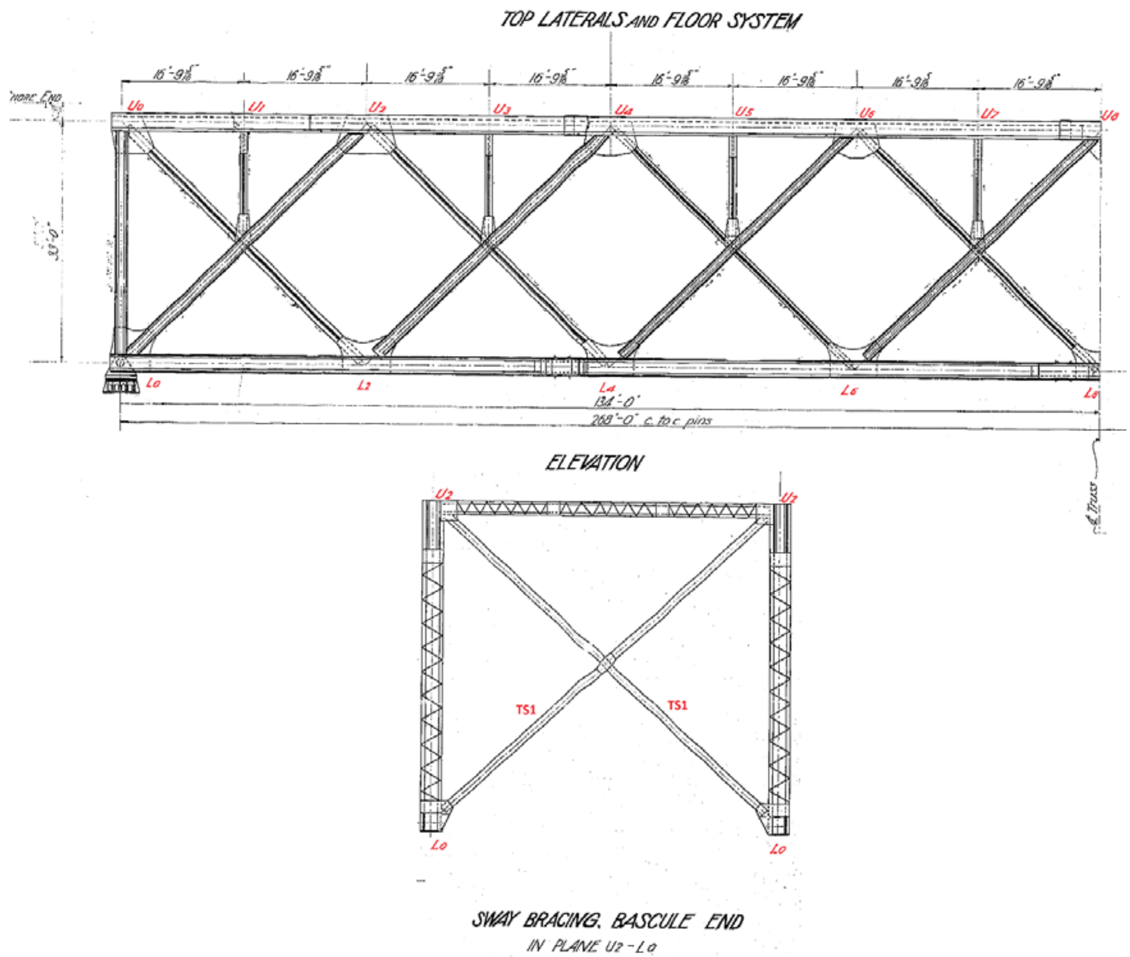


Figure 6-14. Steel Truss Span Elevation and Sway Bracing

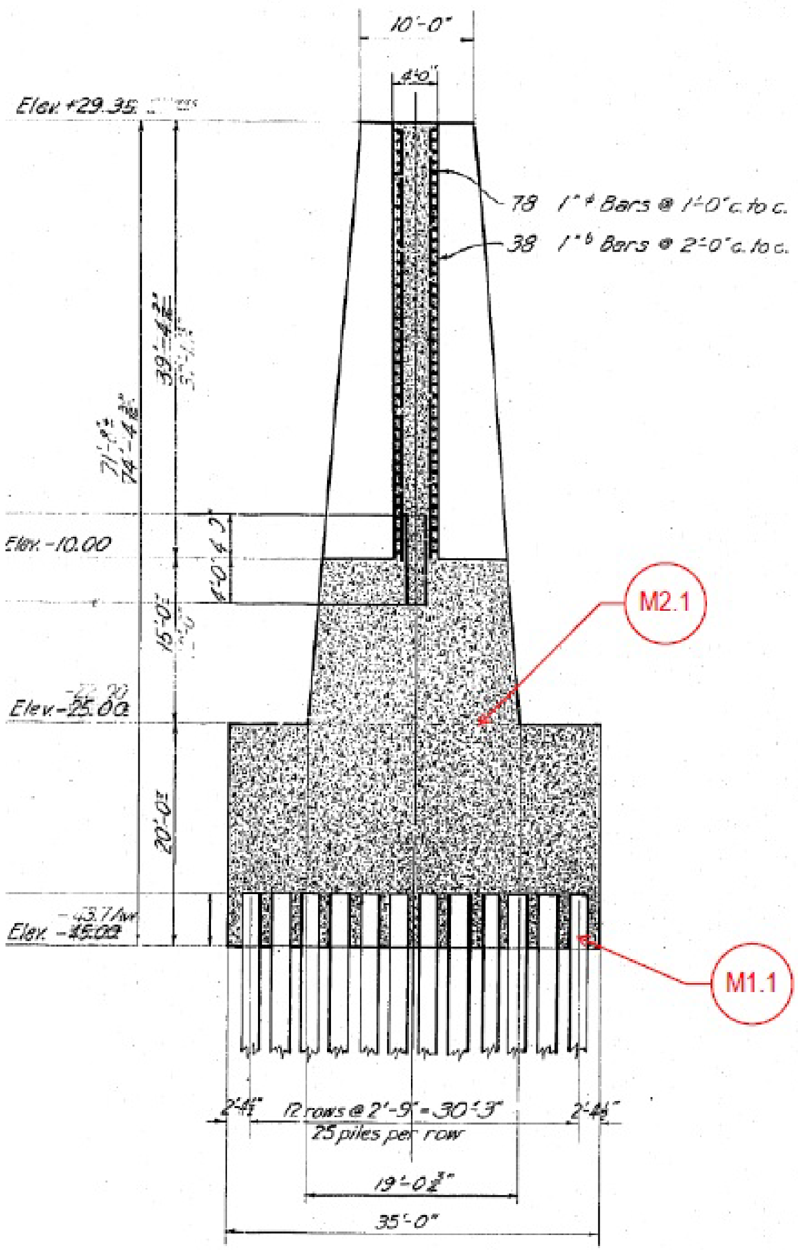
Table 6-1. C/D Ratio Summary Existing Fixed Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio
U8U9	Top Chord at Midspan	Compression	2.10
L8L9	Bottom Chord at Midspan	Tension Yield	2.16
L8L9	Bottom Chord at Midspan	Tension Fracture	3.33
U8L9	Diagonal near Midspan	Compression	1.61
TS1	Out of Plane Bracing near Bascule	Compression	0.19
TS1	Out of Plane Bracing near Bascule	Tension Yield	1.12
TS1	Out of Plane Bracing near Bascule	Tension Fracture	1.71
U16L16	End Post near Bascule	Compression	3.30
—	Fixed End Support Anchor Bolts	Shear	0.09
—	Bottom Lateral Bracing	Compression	0.44

6.3.1 Pier Foundations

6.3.1.1 Pier 1 and Pier 4 Foundations

Existing Pier 1 and Pier 4 foundations consist of unreinforced pile caps and groups of timber piles (see Figure 6-15). These foundations were neither designed nor constructed according to current seismic design requirements and detailing practices. During a design-level earthquake, these foundations can have multiple failure modes.



SECTION THROUGH CENTER

Figure 6-15. Piers 1 and 4 Walls and Foundations

Timber Pile Failure

Geotechnical analysis (Appendix C) indicated that liquefaction-induced settlement of the liquefiable layer and overlaying soil will result in the following:

- Downdrag loads on the existing timber piles at Pier 1 that bear in the Gravel Alluvium below the liquefiable layer, resulting in pile overstressing. Additionally, due to the minimal pile embedment below the liquefiable layer, lateral stability of the pile foundation is also a concern.
- Settlement of the pile cap, downdrag loads on the piles, and reduction in axial pile resistance at Piers 1 and 4. The bottom of pile cap at Pier 4 has predicted 24 inches of liquefaction-induced settlement during the CSZ event.

Pile Cap Failure

Concrete pile caps at Pier 1 and Pier 4 are unreinforced. This limits flexural capacity to the cracking strength of the concrete. Shear capacity is also limited to the concrete shear capacity Vc. Because the unreinforced concrete has low capacities in both flexure and shear (C/D 0.90), the pile caps are predicted to have fracture and shear failures during the design-level earthquakes.

Foundation Collapse

Liquefaction-induced ground displacement at the west and east river banks during and/or after an earthquake will apply pressure on the Pier 1 and Pier 4 foundations, pushing the pier foundations toward the river. Since the existing timber piles have low lateral resistance capacities, the soil lateral movement, if not mitigated, will result in collapse of the existing pier foundations. This concern is even more critical for Pier 4 because the liquefiable soil layer is much deeper.

6.3.1.2 Pier 2 and Pier 3 Foundations

Since the fixed truss spans share the pier supports with the bascule span at Pier 2 and Pier 3, the seismic vulnerabilities of these pier foundations are described under the Bascule Span Section below.

6.3.2 Pier Column and Wall

6.3.2.1 Pier 1 and Pier 4

Each consists of two reinforced concrete columns (Figure 6-16). The lower portions of the columns are connected by a concrete shear wall. The columns and wall are supported on an unreinforced pile cap on a group of timber piles.

Figure 6-16. Pier 4

By examining the as-built plans, the unreinforced or under-reinforced concrete pier column/wall is vulnerable under seismic loads and lateral movement. The pier columns are not properly reinforced to conform to seismic design requirements, and the reinforcing is not detailed for ductile behavior as required per current seismic design standards. The column/wall capacity is limited to the concrete cracking strength and will crack and lose vertical load support capacity during a design-level seismic event. Major deficiencies include:

- Lack of lateral confinement reinforcing in the columns. During an earthquake, the concrete is predicted to crack and fail, and the vertical main reinforcement is predicted to buckle, thus causing the pier columns to lose vertical support capacities.
- Lack of sufficient rebar embedment length, lapping splice length and seismic hook details. During an earthquake, the reinforcement won't be able to develop full strength capacity, will un-bond, and lose the load-carrying capacity. Unreinforced plain concrete is used in the lower portion of the columns and the walls. This unreinforced concrete will crack and fall apart during an earthquake (C/D ratio of 0.42), causing the piers to collapse.
- Because of the above-identified deficiencies, Pier 1 and Pier 4 are anticipated to fail in both flexure and shear during a design-level earthquake even; therefore seismic retrofit is required.

6.3.2.2 Pier 2 and Pier 3

Pier 2 and Pier 3 are described in the Bascule Span Section 6.4.

6.3.3 Truss Supports and Pier Connections

6.3.3.1 Expansion Bearings at Pier 1 and Pier 4

The expansion ends of the steel truss spans are supported on Pier 1 and Pier 4 which also support the concrete approach spans from land side. The support bearings under the steel trusses are rocker type steel bearings (Figure 6-17). During late 2001, the bridge went through a Phase I seismic retrofit (Multnomah County 2001) that included:

- Installing seismic restrainers connecting the top chords of the steel trusses to the concrete approach spans.
- Retrofitting the rocker type bearings by inserting bearing wedges (Figure 6-18 and Figure 6-19).

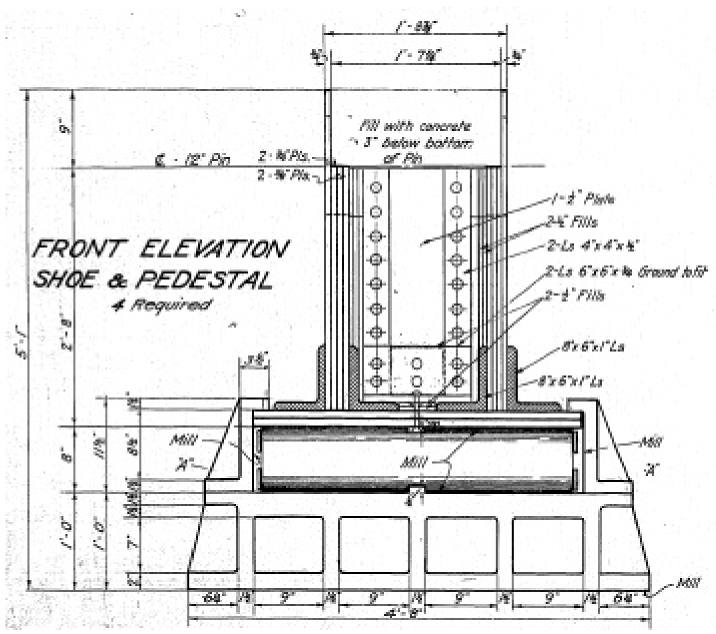


Figure 6-17. Expansion Rocker Bearing

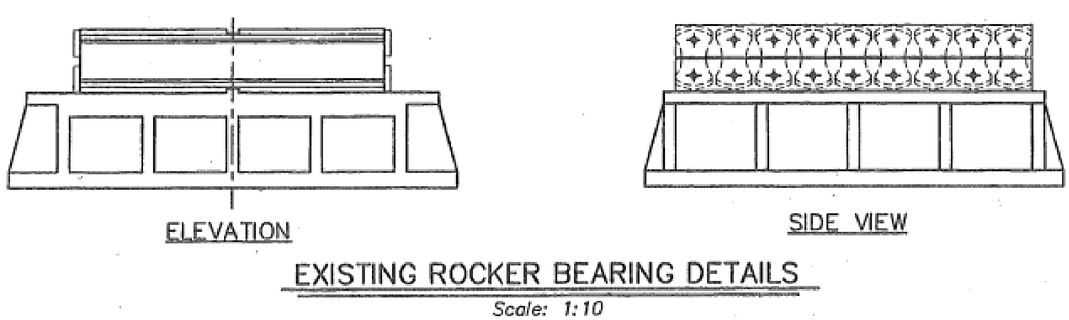


Figure 6-18. Existing Rocker Bearing Details

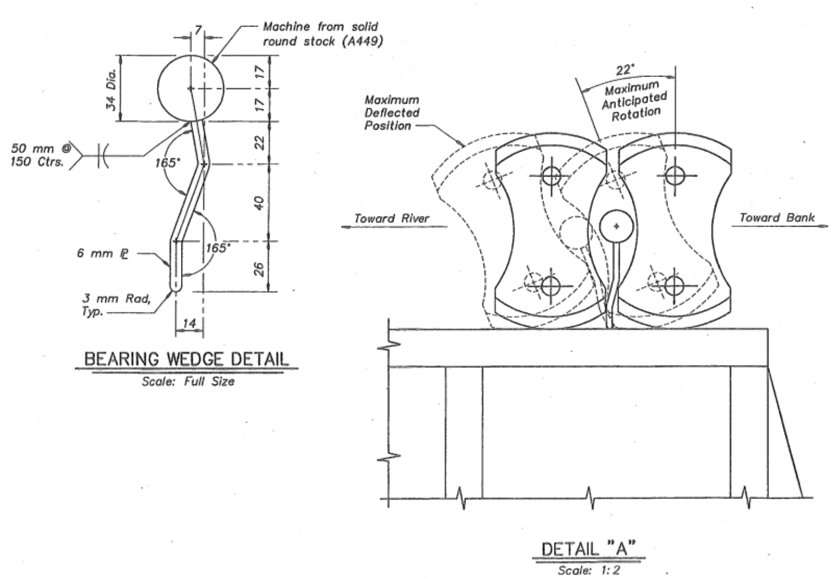


Figure 6-19. Rocker Retrofit in 2001

Expansion Rocker Bearing Failure

The expansion rocker type bearings at Piers 1 and 4 are predicted to fail during a CSZ or 1000-year earthquake. Because the piers under the bearings are massive, rigid concrete structures that have very low or almost no displacement capacity to accommodate seismic movements, the longitudinal seismic movements are anticipated to be accommodated at the bearing level. The predicted longitudinal movement at the bearing level is 6 inches. However, the existing retrofits using wedges at these bearings were designed for maximum movement of less than 4 inches, which is not sufficient to accommodate the seismic movement; the rocker will fall over due to excessive seismic movement. Currently, the displacement required for seismic motion is 23 degrees which exceeds the 22-degree maximum rotation of the retrofitted bearings.

6.3.3.2 Fixed Bearings at Pier 2 and Pier 3

The steel truss spans are supported on fixed bearings (Figure 6-20) at bascule Pier 2 and Pier 3.

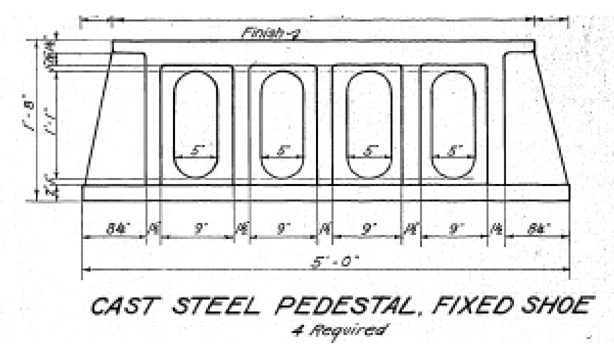


Figure 6-20. Fixed Bearing Shoe

Fixed Bearing Anchor Bolt Shear Failure and Concrete Cracking

Existing anchor bolts of the fixed bearings are insufficient to resist seismic-induced horizontal forces, and the pier concrete wall below the bearing are not reinforced to resist seismic loads, the concrete surrounding the anchor bolts could crack causing the anchor bolts to lose lateral resistance. The C/D ratio for shear of the anchor bolts is only 0.09. Note: For all anchor bolt calculations, grade A36 bolts were assumed due to lack of information provided in the plans.

Short Seating Lengths

On the fixed support ends, the seating lengths do not conform to the current *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (Guide Spec) requirements for seismic design. The seating length provided is 33 inches (Figure 6-21), which is less than the 36 inches required. The truss girders can slip off from the support at the pier tops due to the bearing anchor bolts being sheared off and unconfined concrete cracking, as indicated in the previous section.

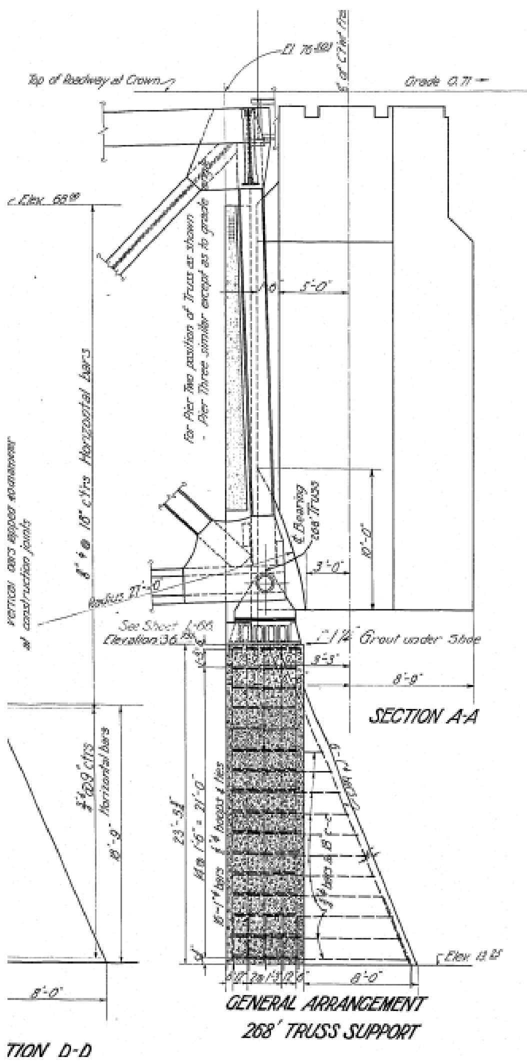


Figure 6-21. Truss Span Support at Piers 2 and 3

6.3.3.3 Lack of Effective Transverse Restrainers

The pier columns or walls at Piers 1, 2, 3, and 4 are not properly reinforced with seismic reinforcing details, therefore, these columns and walls cannot function effectively as transverse seismic restrainers during a design-level earthquake. The C/D ratio is only 0.44, therefore the truss support will pull free from the existing pier structure. Reinforcement of Piers 1 and 4 can be seen in Figure 6-15. Reinforcement of the wall under the truss supports at Piers 2 and 3 can be seen in Figure 6-32.

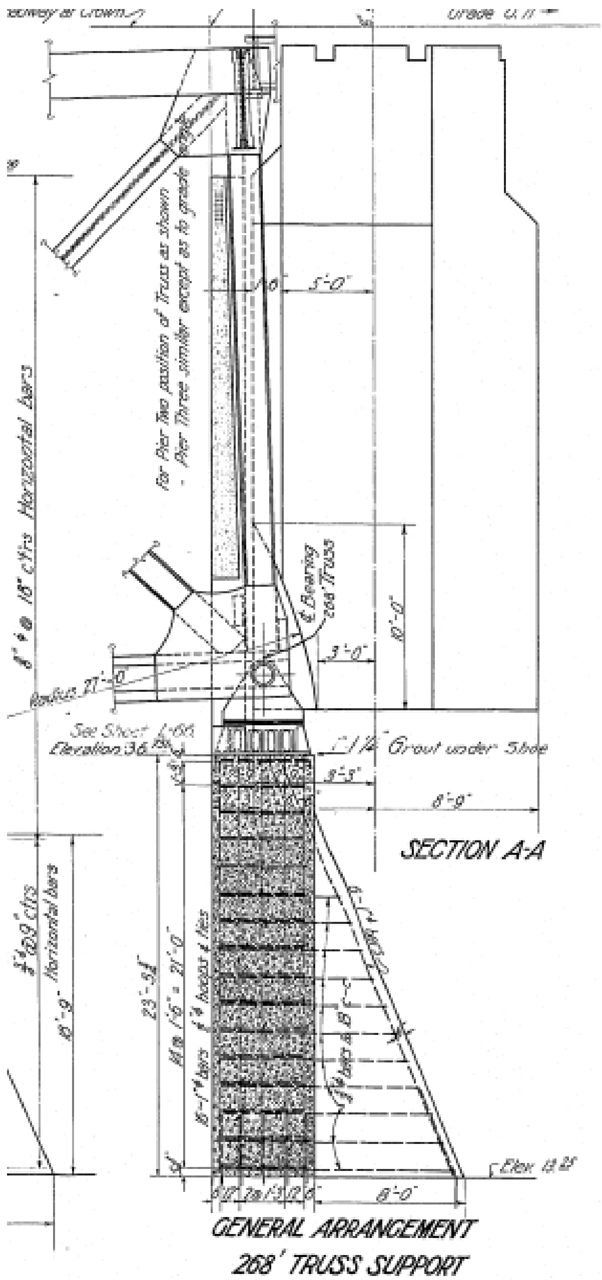


Figure 6-22. Wall Reinforcement under the Bearings at Piers 2 and 3

6.3.4 Steel Truss Superstructure

Each of the truss spans is a 268-foot-long constant depth steel deck truss (see Figure 6-23). A reinforced-concrete bridge deck is supported by steel stringers and floor beams that are connected to the main steel trusses. An analysis of each truss member was conducted to determine the seismic deficiencies. The following sections generally describe the analysis results.

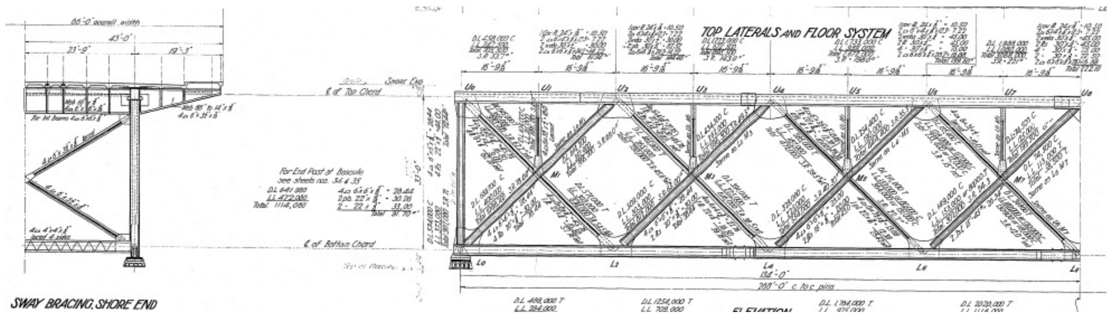


Figure 6-23. Fixed Steel Truss Span

6.3.4.1 Weak Lateral Load Paths

The existing steel truss lacks a proper lateral load transfer path that is capable of transferring the horizontal seismic-induced forces from the deck down to the support at the bearings (Figure 6-24).

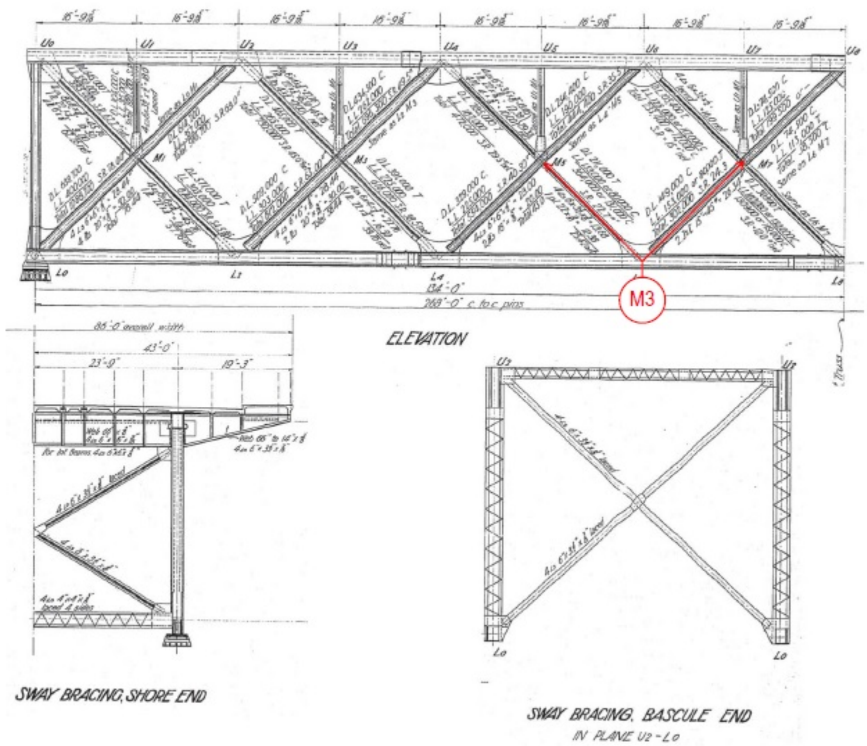


Figure 6-24. Truss Span Bracings

Sway bracing is absent from four of the eight bays in the fixed spans. Sway bracing is required in each bay to prevent collapse of the deck and top chord caused by lateral movement. Due to the limited bracing, the bracing that currently is in place would become overloaded during a seismic event. The out of plane bracing near the bascule end of the fixed span has a C/D ratio of 0.19. The bridge deck to steel floor beam connection shear capacity is insufficient based on the As Built plans, Shop Drawings, and Rehabilitation plans. There are no connectors between the deck and floor beams of the fixed span so the existing deck will not act compositely under seismic loading.

6.3.4.2 Insufficient Bottom Lateral Bracings

The existing bottom lateral bracings were not originally designed for seismic loading. Specifically, those near the end of the span are under-capacity to transfer seismic loads, with a C/D ratio of 0.68.

6.4 Bascule River Span

The Burnside Bridge main river span crossing the Willamette River navigation channel is a 252-foot-long (trunnion-to-trunnion) double-leaf steel deck truss bascule span. According to the original as-built plans (Multnomah County 1923) and (Multnomah County 1924), some of the major dimensions are described below. The elevations referred to below are from the as-built plan datum.

Along the centerline of the bridge, the face-to-face distance between the navigational channel side pier walls is 213 feet. Each pier is 55 feet long measured from outside faces of the pier wall.

The overall bridge deck width is 89 feet that includes 68 feet for five vehicle traffic lanes and two bicycle lanes, and also includes a 9-foot raised pedestrian sidewalk on each side.

Reinforced concrete decks are on top of the variable-depth bascule leaves. Each of the two bascule leaves, including the counterweight, (Figure 6-25), is supported via trunnion support steel frames on concrete pedestals inside the bascule pier. The centerlines of the bascule trunnions are at elevation of 68.5, and the supporting concrete pedestals are at an elevation of 36.5.

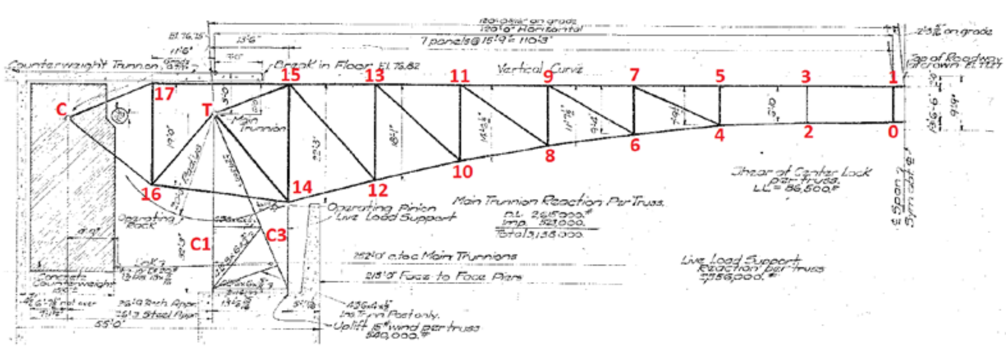


Figure 6-25. Bascule Leaf and Counterweight

Each bascule pier houses the trunnion support frames, counterweight, and bascule machinery.

The upper part of the pier is enclosed by reinforced concrete pier walls from under the bridge deck down to elevation -33. A pit floor inside the bascule pier is at elevation 13.25. The as-built plans show no reinforcement in the concrete pier walls from the pit floor to the top of the pile cap.

The pier walls are connected with straight dowels to the unreinforced pile caps founded on timber piles. The bottom of the pile caps are at elevation -70.

Seismic vulnerabilities of the bascule span under the CSZ and 1000-year seismic events were identified during the study, based on conceptual analysis and review of as-built plans and previous study documents.

The analysis was conducted step by step to identify the required retrofit, Table 6-2 indicates that there are members that have C/D ratios less than 1.0 under a CSZ event (**red** text); therefore, at a minimum, these members need to be strengthened.

Table 6-2. C/D Ratio Summary for Existing Bascule Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio or Interaction
16-C	Counterweight Compression Support	Comp.-Moment Interaction*	0.54
16-C	Counterweight Compression Support	Shear	0.30
Counterweight Link	Counterweight Link	Compression	0.11
Counterweight Link	Counterweight Link	Tension Yield	0.75
Counterweight Link	Counterweight Link	Tension Fracture	0.82
T-C1	Trunnion Post	Compression	3.38
T-C3	Trunnion Diagonal Post	Compression	0.74
—	Trunnion Support Anchor Bolts	Combined Tension and Shear	0.27
14-T	Trunnion Brace	Comp.-Moment Interaction*	2.22
14-T	Trunnion Brace	Shear	0.56
14-T	Trunnion Brace	Ten.-Moment Interaction*	0.89
15-T	Trunnion Link	Comp.-Moment Interaction*	1.52
15-T	Trunnion Link	Shear	0.32
15-T	Trunnion Link	Ten.-Moment Interaction*	1.10
14-15	Bottom Chord Bascule Truss	Compression	2.29
12-15	Diagonal Bascule Truss	Tension Yield	2.16
12-15	Diagonal Bascule Truss	Tension Net Fracture	2.90
13-15	Top Chord Bascule Truss	Tension Yield	2.25
13-15	Top Chord Bascule Truss	Tension Net Fracture	3.19

* For Interaction failure, value shown represents 1/(Interaction Result)

6.4.1 Pier Foundations

Existing bascule Pier 2 and Pier 3 foundations consist of unreinforced pile caps and groups of timber piles (see Figure 6-26). These foundations were neither designed nor constructed according to current seismic design requirements and detailing practices. During a design-level earthquake, these foundations will fail.

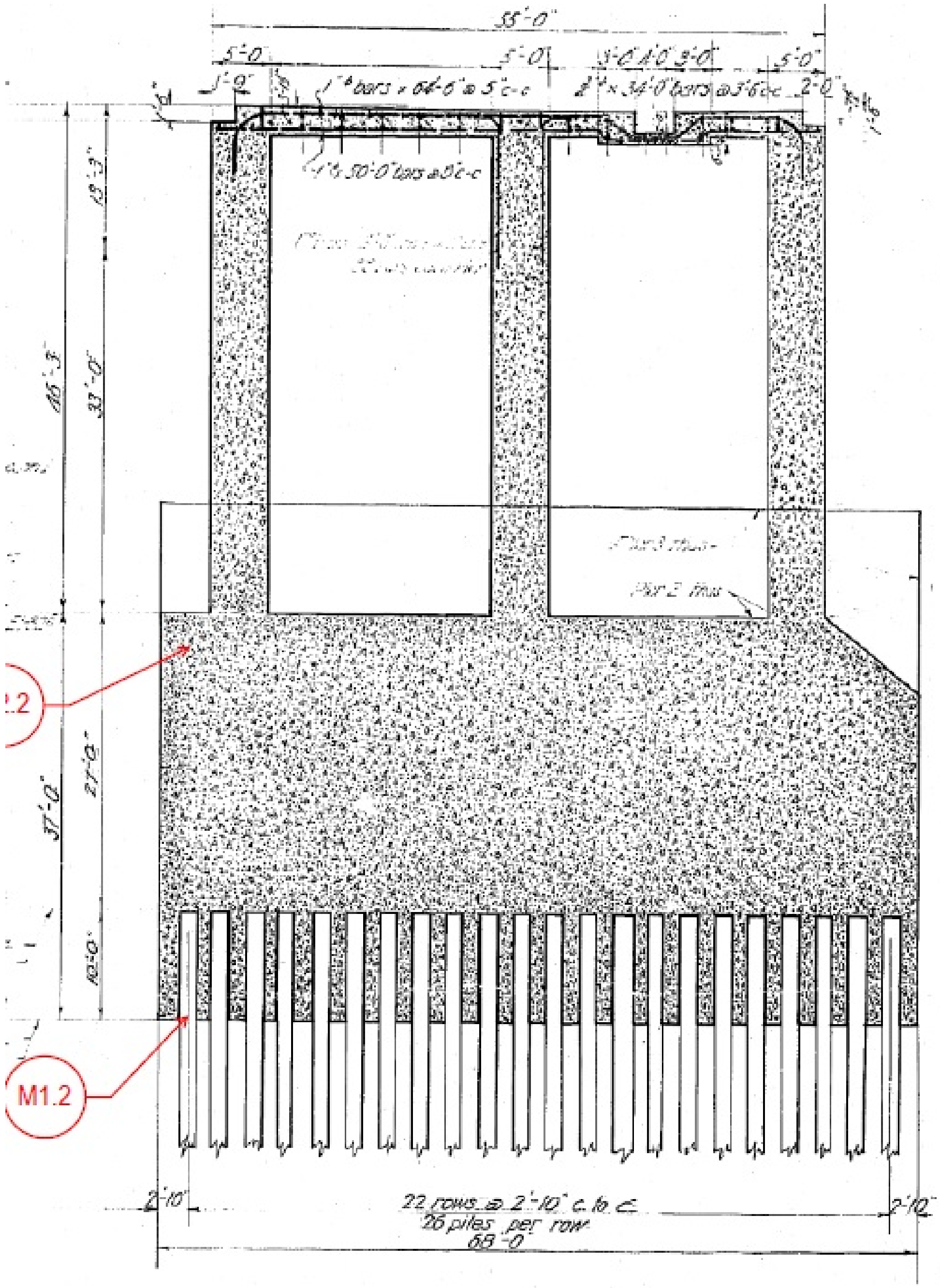


Figure 6-26. Bascule Pier Foundation

6.4.1.1 Timber Pile Failure

Geotechnical analysis (Appendix C) indicated that liquefaction-induced settlement of the liquefiable soil layer will result in the following:

- Downdrag loads on the existing timber piles resulting in pile overstressing.
- Settlement of the pile cap, reduction or loss of vertical pile resistance, and concern of lateral stability of the pile foundation.

6.4.1.2 Pile Cap Failure

Concrete pile caps at Piers 2 and 3 are unreinforced. This limits flexural capacity to the cracking strength of the concrete. Shear capacity is also limited to the concrete shear capacity, Vc. Since the unreinforced concrete has insufficient capacities in both flexure and shear, the pile caps have a C/D ratio of 0.20 for shear.

6.4.2 Pier Walls

Pier walls were designed for non-seismic lateral loads such as wind loads, and gravity loads only. They are not reinforced and detailed to resist seismic forces (Figure 6-27).

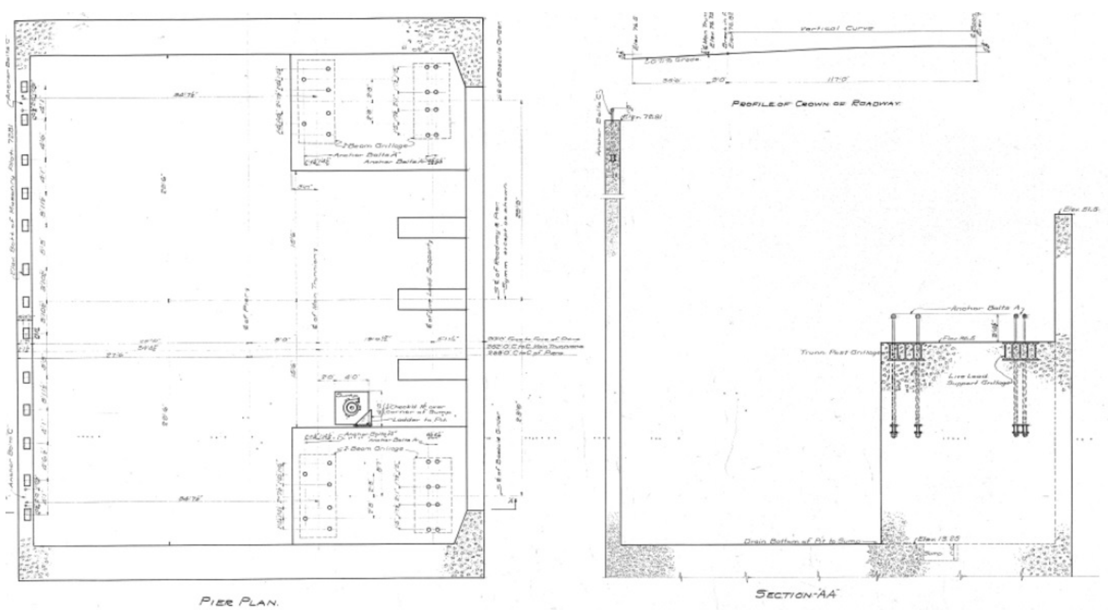


Figure 6-27. Bascule Pier Walls

In examining the as-built plans (Multnomah County 1924), the lower part of the piers below the pit floor are not reinforced (Figure 6-28). The unreinforced and under-reinforced concrete pier wall is vulnerable under seismic loads and lateral movement. The pier wall reinforcing is not detailed as required per current seismic design standards. Major deficiencies include:

- Lack of lateral confinement reinforcing in the walls (). The pier back wall under the bearing of the steel trusses has lateral confinement reinforcing at 1-foot 6-inch vertical spacing, while current seismic design requires confinement reinforcing spacing of less than 6 inches. During a design-level earthquake, the vertical main reinforcement rebar will buckle due to lack of confinement, and the concrete will crack and fall apart, thus causing the pier walls lose vertical support capacities. The C/D ratio for this is 0.81.

- No dimensions in the as-built plans for rebar embedment length, lapping splice length, and seismic hook details. Bridges built in 1920s typically do not meet current seismic design requirements for the embedment length and slice length, etc. During an earthquake, the reinforcement will likely pull out and lose the load-carrying capacity.
- Unreinforced plain concrete was used in the lower portion of the piers and the walls. This unreinforced concrete will crack and fall apart during an earthquake, causing the piers to collapse. This concrete has a C/D ratio of 0.20 for this failure mode.
- Because of the above-identified deficiencies, bascule Pier 2 and Pier 3 will fail in both flexure and shear during a design-level earthquake event; therefore, seismic retrofit is required.

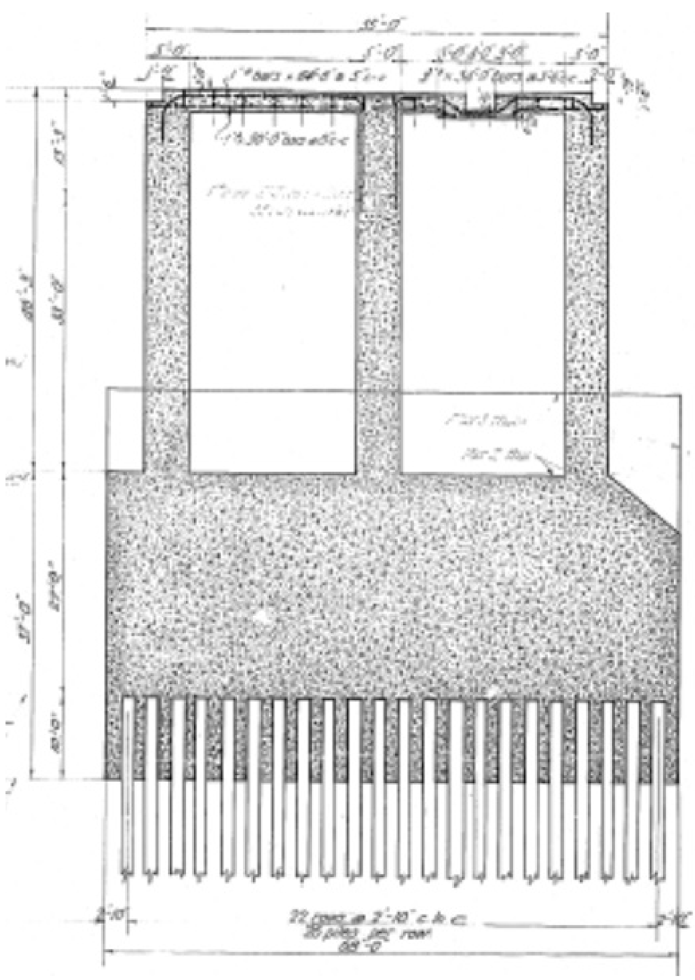


Figure 6-28. Bascule Pier Lower Walls Unreinforced

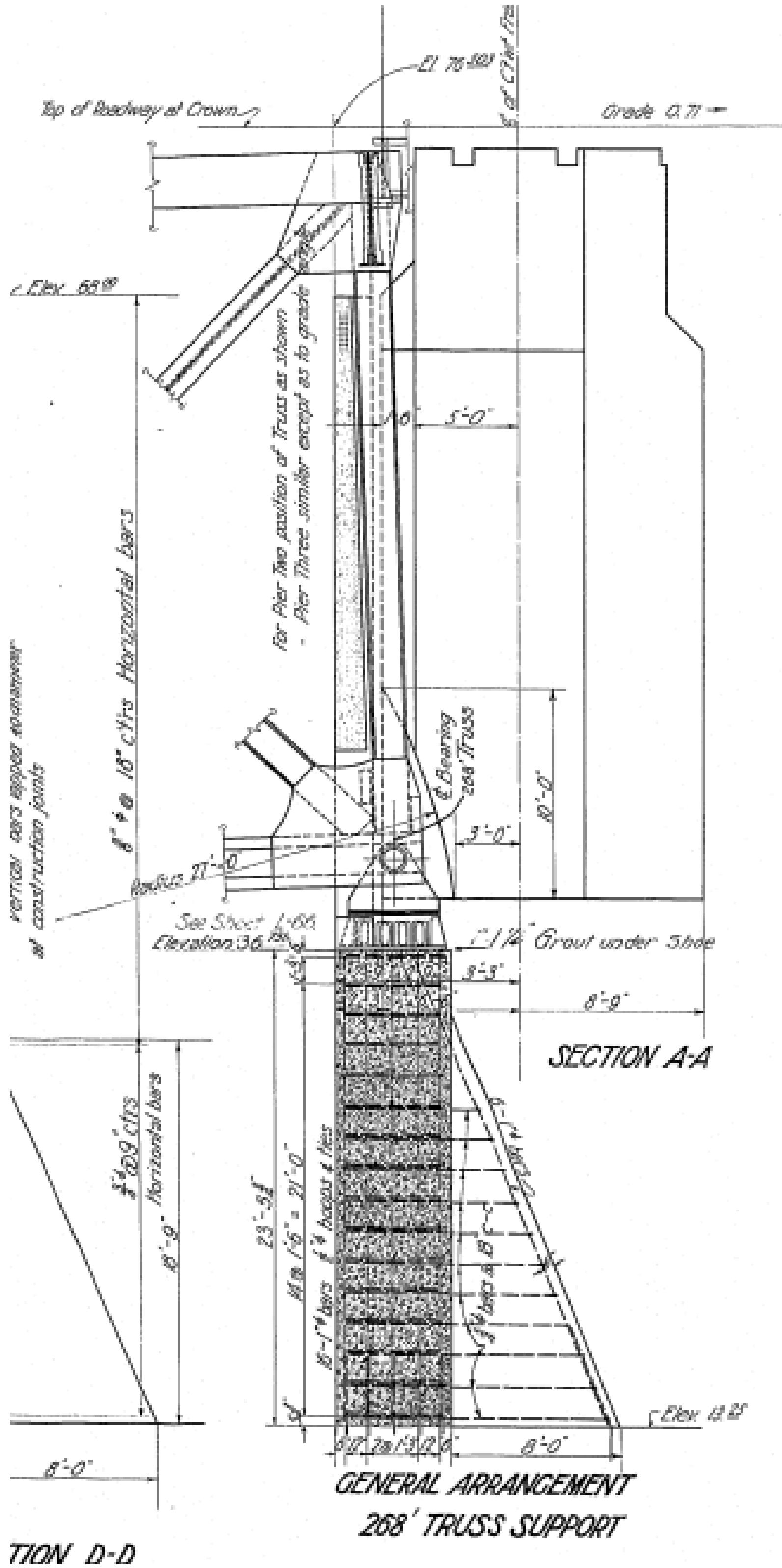


Figure 6-29. Bascule Pier Reinforcing

6.4.3 Trunnion Supports

The trunnion support frames were designed primarily for supporting the vertical loads of the bascule leaves and counterweights (see Figure 6-30).

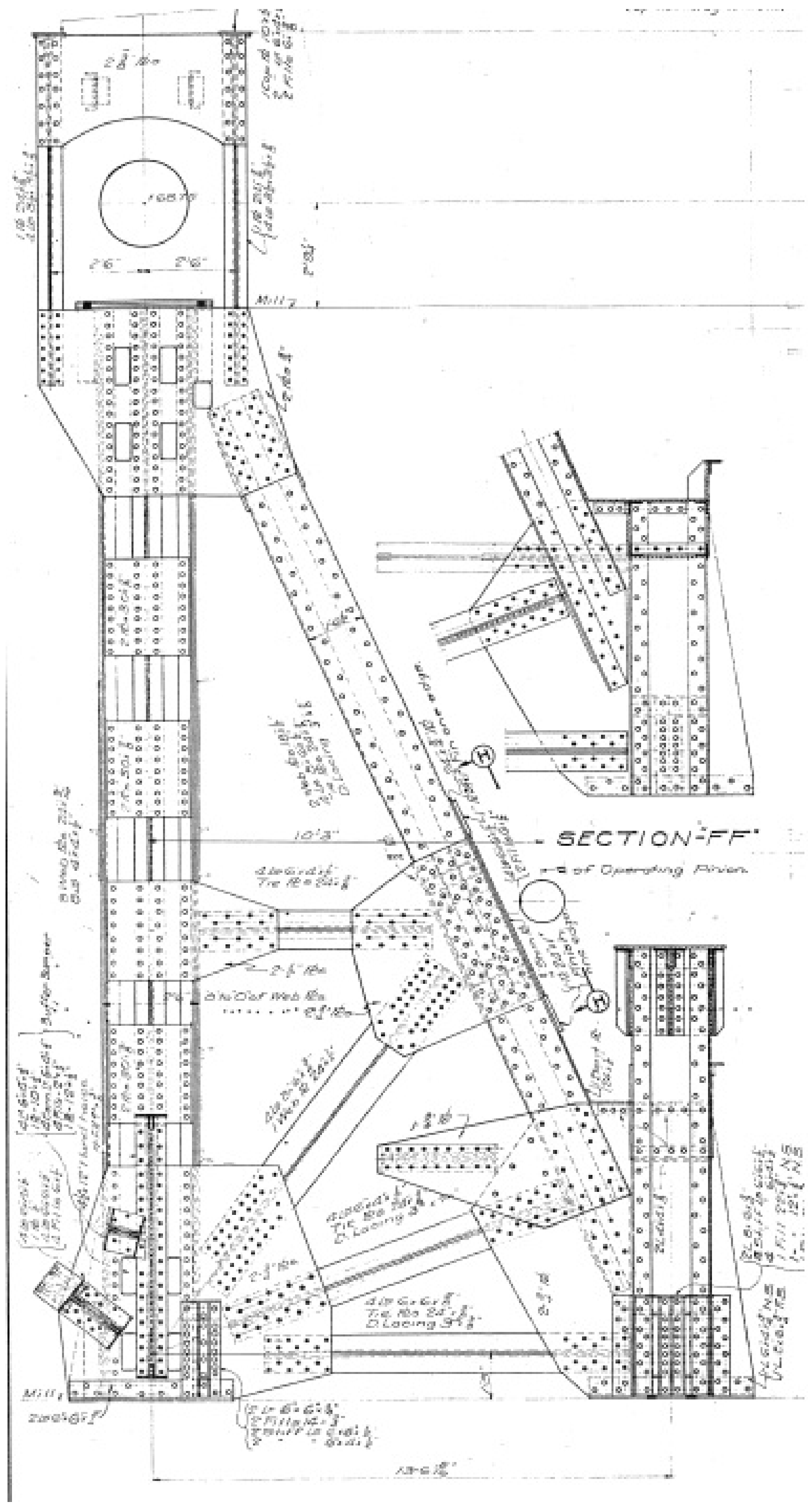


Figure 6-30. Trunnion Tower Support Frame

Lateral restrainers were installed during the Main Span rehabilitation in 2005 (Multnomah County 2005). These restrainers were installed to connect the trunnion tower support frames to the side walls of the bascule piers. Since the pier walls are not reinforced for seismic loads, these restrainers won't be effective during a design-level seismic event.

6.4.3.1 Trunnion Support Frame Failure

The trunnion support frames are heavily loaded because all the loads from the bascule span leaf, including the counterweight, are transferred through the trunnion support frame to the piers. Without effective lateral restrainers or support, under the design-level seismic motion and lateral forces, these trunnion support frames will fail in buckling (C/D ratio = 0.74), see Table 6-2.

6.4.3.2 Anchor Failure

Existing anchor bolts under the trunnion tower support frames are insufficient to resist seismic and longitudinal forces. These anchor bolts will fail as a result of bolt shearing or concrete cracks. For combined tension and shear, the C/D ratio is only 0.27 for the anchor bolts. Note: For all anchor bolt calculations, grade A36 bolts were assumed due to lack of information provided in the plans.

6.4.4 Counterweight Supports

No lateral supports restrain the counterweight. This exposes the counterweight support frames to buckling (see Figure 6-31). Counterweight support member 16-C has a C/D ratio for shear of 0.30 and a value of 0.54 for the inverse of the compression-moment interaction (see Table 6-2). In addition, unrestrained lateral movement of the counterweight can impact the reinforced concrete walls supporting the sidewalks.

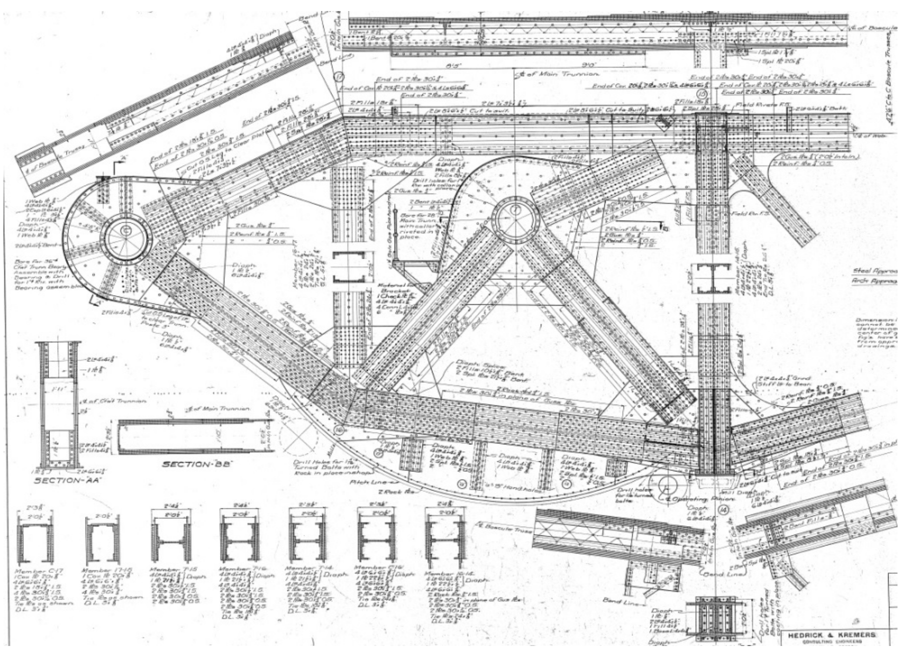


Figure 6-31. Counterweight Support Frame

6.4.4.1 Counterweight Link

The existing counterweight link is exposed to large forces as it is the only member resisting the swinging of the counterweight in the longitudinal direction of the bridge. Currently, the counterweight is over loaded in all checks made. The C/D ratios are 0.11 for Compression, 0.75 for Tension Yield, and 0.82 for Tension Fracture, Table 6-2. Failure of this member would cause unrestrained longitudinal motion of the counterweight which could impact the wall of the pier supporting the fixed span.

6.4.5 Superstructure Connection to Trunnion

Bracing frame T-14 (Figure 6-32) transmits lateral loads from the entire bascule span to the trunnion tower support frame. These members were not originally designed for transmitting the seismic-motion-induced lateral forces, thus are vulnerable to buckling and yielding during a design-level seismic event. For tension, the C/D ratio for T-14 was 0.56. For the inverse of the tension-moment interaction, the resulting value was 0.89. In addition to T-14, member T-15 has C/D ratio values of less than 1.0 where the shear C/D ratio was 0.32, (see Table 6-2). Member T-15 can be seen in Figure 6-31 as the top right member in the connection to the trunnion.

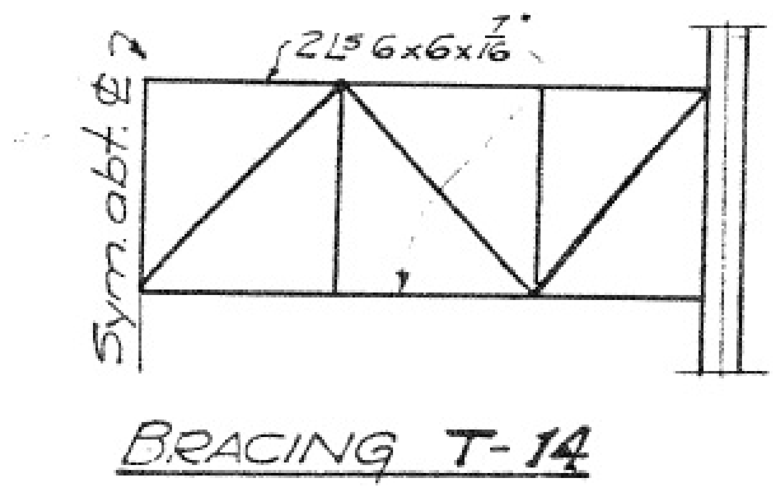


Figure 6-32. Bracing for Trunnion support Frame

6.4.6 Live Load Support Connections

The live load support shoes were designed to resist unbalanced vertical loads and live loads (see Figure 6-33). Because it is a simple bearing plate, the live load support cannot resist lateral or upward (tension) loads present in a seismic event. During a design-level seismic event, the bascule leaves will move horizontally and rotate around approximately the intersection of the centerline of the trunnion and the centerline of the bridge. Without the live load shoes' help in resisting vertical and horizontal rotational movement of the bascule leaf, the bascule trunnion support frames (member T-C3) will be exposed to large forces causing buckling or tension failures of the trunnion support frames.

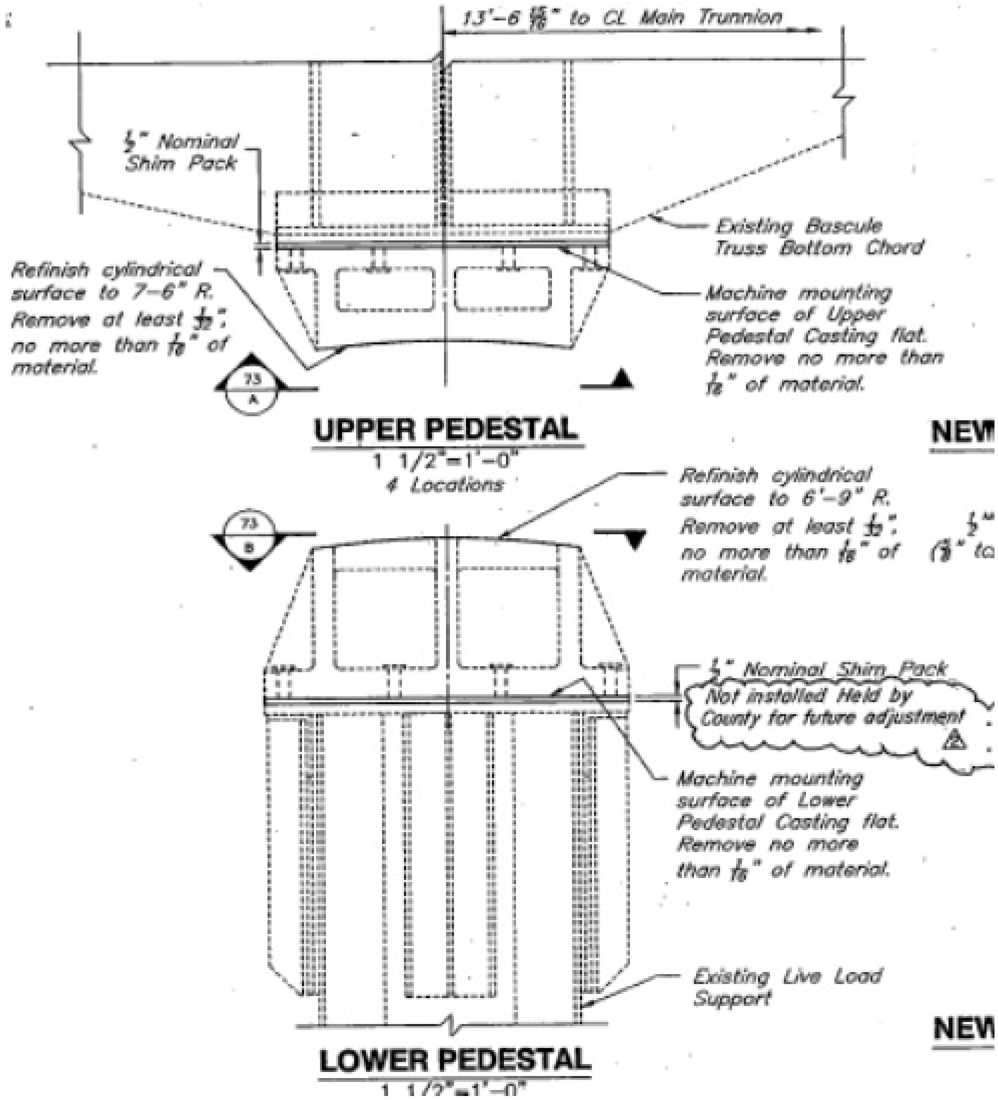


Figure 6-33. Live Load Shoe

6.4.7 Center Lock Shear

A typical bascule leaf center span lock, such as the one on the Burnside Bridge, is not designed to transmit forces caused by the relative transverse displacement of the two bascule spans and can be severely damaged in a significant seismic event (see Figure 6-34). When the center lock is damaged during a seismic event, bascule leaves without the center lock can sway in different directions, causing large horizontal forces on the trunnion support frame.

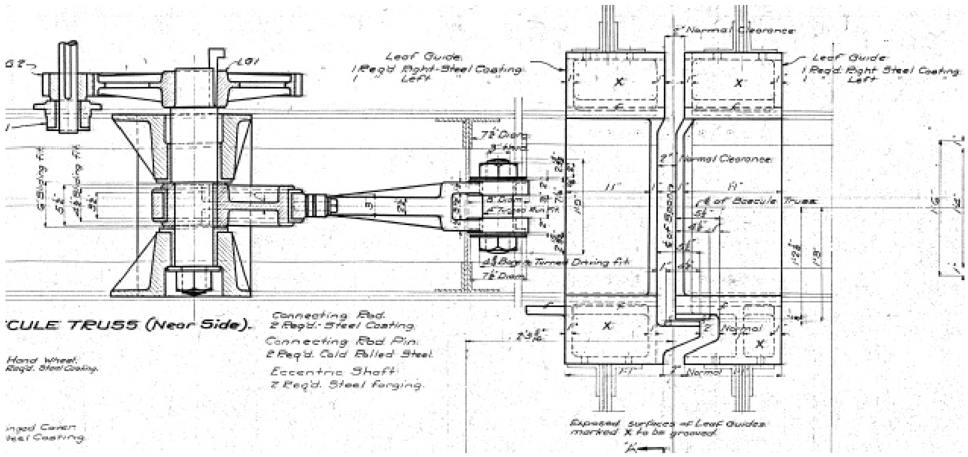


Figure 6-34. Bascule Span Center Lock

6.5 Mechanical and Electrical Equipment

6.5.1 Mechanical Equipment

Each leaf has span drive machinery systems that are identical and symmetric about the centerline of the channel. The span drive system consists of two 75 HP, 540 RPM motors that both drive a central differential gear. The two output shafts from this central differential drive are a series of three open gear reductions. The output from each final reduction drives a pinion that mates with a rack mounted on each of the two main bascule girders. The system has a motor brake on the back of each of the two motors and two machinery brakes on the opposing input to the differential gear opposite the main drive motors.

The west leaf also has center span lock machinery that consists of a single 15 HP motor that drives an enclosed worm gear reducer. Cross shafts connect to the output of the reducer and drive a single set of open gearing located just outside of the truss top chord at each side of the leaf. The open gearing ultimately drives a linkage attached to a set of external jaws. When the span is closed, these jaws engage a receiver on the east leaf to make the shear connection between the two leaves.

Additionally, the bridge has main and counterweight trunnions on each leaf that support the dead load of the entire leaf and the counterweight, respectively. These are both bronze-bushed plain bearings with forged steel shafts. The main trunnions and the east counterweight trunnions are original to the bridge and the west counterweight trunnions were replaced during a recent rehabilitation due to high friction during operation.

Seismic deficiencies are described further in Section 9.4.14.

6.5.2 Electrical Equipment

Piers 2 and 3 each have an incoming service to provide power to each movable span leaf and other equipment. The incoming service is distributed to transformers and panelboards for lighting and receptacles and to motor control centers to operate equipment ancillary to bridge operation, including the traffic warning gates and center span lock.

Span operation is facilitated by two span operation motors for each movable span leaf. The motor speed is controlled by drives that are connected to the motors. Other aspects of controlling the bridge during operation are provided by the bridge operation control system. The basis of the bridge operation control system is a programmable logic controller (PLC) system, which interlocks different stages of operation to prevent unsafe operation of the bridge. The PLC system also provides commands to and receives feedback from bridge operation equipment regarding its status. A human-machine interface (HMI) touchscreen allows the bridge operator to choose which equipment to operate during bridge operation sequences.

Supplemental equipment in the bridge operation control system includes uninterruptible power supplies (UPS), Ethernet network switches, and associated power, control, and communications cables and conduit. The bridge control system includes a PLC, an HMI, and the supplemental components within both Pier 2 and Pier 3.

Seismic deficiencies are described further in Section 9.4.14.

6.6 Structures on the Bridge

Other structures attached on the bridge, such as the overhead sign structure and light poles, could collapse onto the bridge during design-level seismic events and present risks to the public and serviceability challenges. Further analyses are required to determine the extent of these vulnerabilities.

6.7 Approach Retaining Walls

Approach retaining walls at both the west and east end of the bridge consist of a mix of reinforced semi-gravity cantilever walls and counterfort walls. As discussed previously, in many cases these retaining walls are integral with the adjacent buildings (see Figure 6-35).

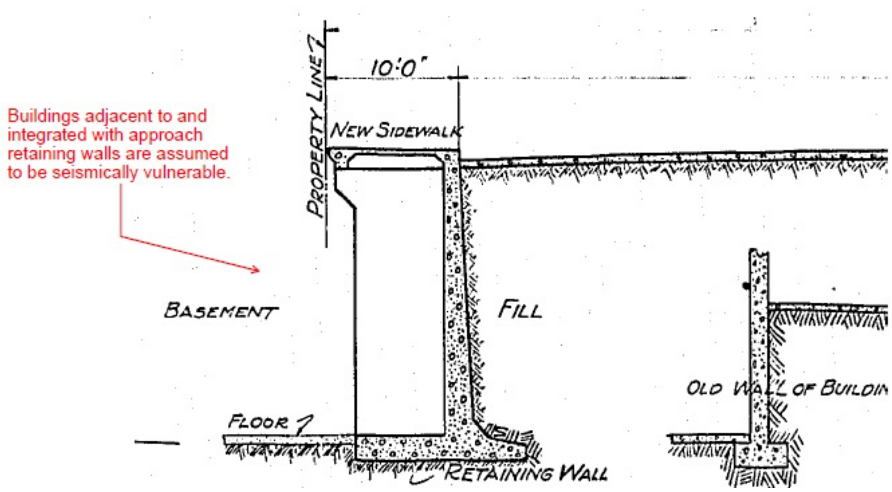


Figure 6-35. Retaining Walls at Approach

Vulnerabilities identified in these approach retaining walls include poor seismic detailing with lap splices in high moment regions, and buildings adjacent to and integrated with approach retaining walls (see Figure 6-36).

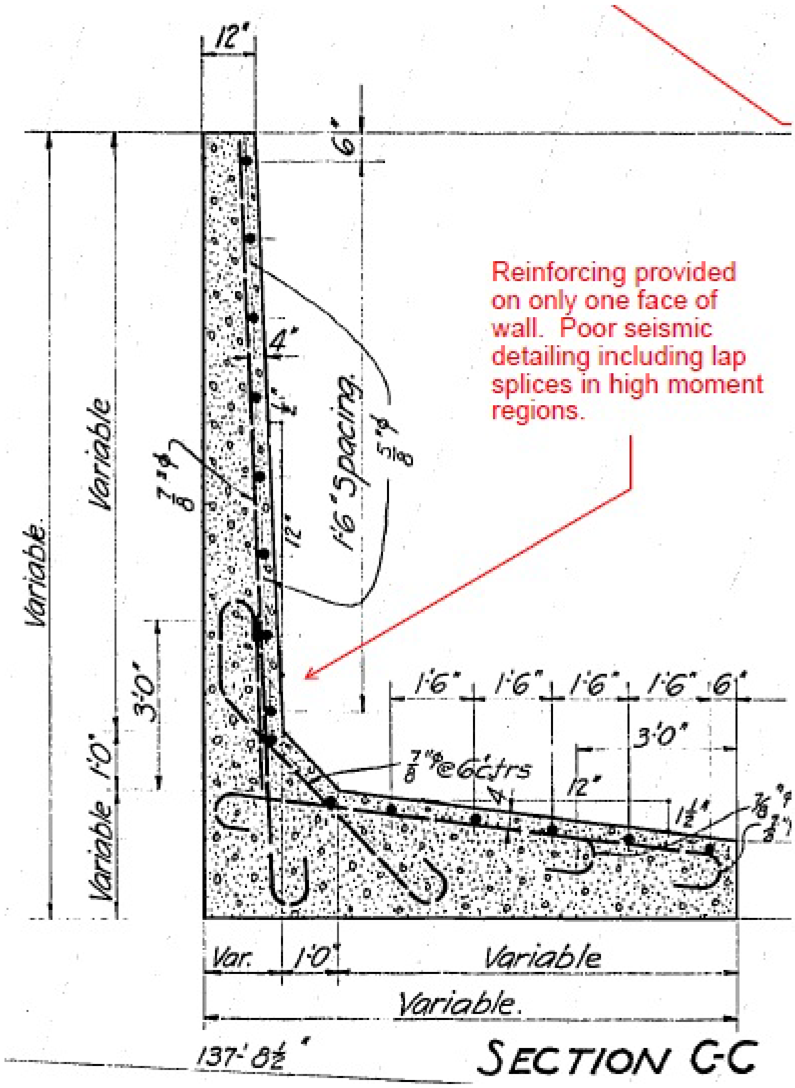


Figure 6-36. Retaining Wall Reinforcement

7 Geotechnical Hazard Mitigation Approach

Site-specific ground response analyses were performed to develop design ground motions and establish geotechnical hazard levels. The ground response analyses included the following steps:

1. Develop base ground motions. Base ground motions are the bedrock ground motions; a deterministic CSZ event corresponding to full rupture of the subduction zone interface and a probabilistic ground motion corresponding to a 1,000 year return period.
   a. Develop base motion time history target spectra.
   b. Develop earthquake time histories that closely match the target spectra.
2. Develop a soil model of the site for one-dimensional dynamic wave propagation (site response) analyses.
3. Propagate the base ground motion time histories through the soil model and calculate the response spectra at the existing bent/pier locations in the soil model.

Site response analyses estimate seismic shaking at the ground surface of a soil model based on earthquake time histories applied to the base of the model. Site response analyses develop pore pressure and liquefaction levels and extends, translating into calculated hazard levels.

7.1 Potential Seismic Related Geotechnical Hazard

Site-specific ground response analyses were performed using FLAC to develop site-specific design ground motions. Site response analyses estimate seismic shaking at the ground surface of a soil model based on earthquake time histories applied to the base of the model. The analyses were performed for the Full Operation Performance Level: CSZ event and Limited Operation Performance Level: 1,000-year ground motion levels. Six spectrum-compatible, scaled ground-motion time histories (three for each of the two ground-motion levels) were developed and input into the base of the FLAC soil model. To develop the site-specific design ground surface ARS for each seismic performance level, a hazard-consistent geometric mean of the response spectra estimated from the three time histories was calculated. Depending on the characteristics of the soil deposit and its response to the base ground motion time histories, the ground surface response spectra were combined in three groups: Bents 1 through 18, Bents 19 through 27 (including Piers 1 through 4), and Bents 28 through 35. The envelope of the site-specific ground surface response spectra at each bent group are plotted on Figures 5 and 6 in Appendix B of the Burnside Bridge Geotechnical Report (Appendix C) for Full Operation and Limited Operation Performance Levels, respectively.

Figures 5 and 6 in Appendix B of the Burnside Bridge Geotechnical Report (Appendix C) also show the code-based (ODOT) ARS calculated for each seismic performance level. When compared to the site-specific ground surface ARS, the ODOT CSZ event ARS is significantly lower, particularly for shorter periods. The lower ODOT CSZ event ARS is the result of lower site response terms for Site Class E in the current subduction ground motion prediction equations that are used by the ODOT web-based application compared to ODOT BDDM code-based site factors F_{pga}, F_a, and F_v (i.e., ODOT BDDM Tables 1.17.3-1A, B, and C). The ODOT site factors are consistent with the current study on the amplification factors observed for crustal ground motion dataset. For comparison, the adjusted Site Class E ODOT CSZ event ARS was calculated by obtaining the Site Class B/C boundary ODOT CSZ event ARS from the ODOT web-based application and applying ODOT BDDM Site Class E site factors (i.e., Tables 1.17.3-1A, B, and C). These adjusted Site Class E CSZ event spectral values are plotted in Figure 5 at periods of 0, 0.2, and 1.0 second. As observed from this figure, the adjusted Site Class E CSZ event ARS is increased for short periods and is consistent with the site-specific ground surface ARS.

Shannon & Wilson developed the recommended smoothed design ARS at the site from the three site-specific ground surface spectra for the three bent groups. AASHTO does not permit seismic design using spectral values less than two-thirds of the code-based design spectrum. Where the two-thirds spectrum for Site Class E is greater than the site-specific ground surface spectrum, AASHTO requires the site-specific spectrum to follow two-thirds of the corresponding Site Class E spectrum. At the Burnside Bridge, the anticipated mean surface response is less than two-thirds of the Site Class E code-based response spectrum at spectral periods beyond approximately 3.0 seconds. Therefore, our recommended site-specific ground surface ARS for the bridge site follows the two-thirds code-based ARS for spectral periods longer than 3.0 seconds and follows the anticipated mean surface response for the periods shorter than 3.0 seconds. For the Full Operation Performance Level, the recommended design ARS was selected to envelope the three bent groups site-specific ground surface ARS. For the Limited Operation Performance Level, the soil response for Bents 28 through 35 are significantly higher than the other bents at periods between 0.1 and 0.75 seconds and therefore, our recommended design ARS was principally created to envelope the three bent groups site-specific ground surface ARS, with an additional check for Bents 28 through 35 using an elevated design spectrum for periods between 0.1 and 0.75 second. The recommended design ARS are plotted on Figures 5 and 6 in Appendix B of the Burnside Bridge Geotechnical Report (Appendix C) for Full Operation and Limited Operation Performance Levels, respectively, and are also provided in Table 7-1.

Table 7-1. Recommended Seismic Design Spectral Accelerations

Period (seconds)	Full Operation Performance Level (CSZ Event)	Limited Operation Performance Level (1,000-Year Return Period)	
	Bents 1 through 35	Bents 1 through 27	Bents 28 through 35
0.02	0.326	0.532	0.532
0.03	0.350	0.569	0.569
0.05	0.430	0.703	0.703
0.075	0.553	0.891	0.891
0.1	0.660	1.000	1.000

Table 7-1. Recommended Seismic Design Spectral Accelerations

Period (seconds)	Full Operation Performance Level (CSZ Event)	Limited Operation Performance Level (1,000-Year Return Period)	
	Bents 1 through 35	Bents 1 through 27	Bents 28 through 35
0.15	0.809	1.000	1.650
0.2	0.921	1.000	1.650
0.3	1.106	1.000	1.650
0.5	1.000	1.000	1.650
0.75	0.777	1.000	1.000
1	0.650	0.731	0.731
1.5	0.265	0.470	0.470
2	0.163	0.280	0.280
3	0.102	0.154	0.154
5	0.059	0.093	0.093
7.5	0.0376	0.062	0.062
10	0.0275	0.0463	0.0463

Based on 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.

Based on 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.

7.2 Soil Improvement

The existing spread footings (except Bent 17) will be enlarged, and the spread footings at Bent 17 and all existing pile group foundations will be retrofitted with drilled shafts. Seismic mitigation will be required to mitigate liquefaction-induced settlement and potential 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. Seismic mitigation for permanent ground displacement (lateral spreading and flow failure) is presented in Section 7.3.

Conceptual seismic mitigation alternatives at Bents 1 through 16 include supporting the enlarged footings on micropiles or ground improvement. 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 appropriate mitigation methods 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. Based on the site conditions and limited overhead clearance to work under the existing bridge, ground improvement using jet grouting is the preferred seismic mitigation alternative at Bents 1 through 16. 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.

Ground improvement at Bents 1 through 16 is proposed to 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. A cellular soil-cement ground improvement width of about 25 feet, length of about 120 feet, and height of about 25 feet is assumed at each bent location, not including the area under the existing spread footings. The conceptual ground improvement design is shown in Appendix K of the Burnside Bridge Geotechnical Report (Appendix C), on Sheet No. 27, Ground Improvements – Under Spread Footings.

7.3 Lateral Spread and Flow Failure Hazard Mitigation

Seismic mitigation will be required at the west riverbank to mitigate the potential permanent ground displacement hazard. Based on the site conditions and limited overhead clearance, ground improvement using jet grouting is the preferred seismic mitigation alternative at the west riverbank. Ground improvement at the west riverbank is proposed to be performed underneath the existing seawall between Bent 19 and Pier 1 with low-overhead jet grouting equipment to form a soil-cement ground improvement zone. Removal of the existing seawall will be performed under the bridge and extend 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 river side of the seawall excavation would be provided by a cofferdam constructed in front of Pier 1. The existing seawall is supported on vertical and battered timber piles. The existing timber piles would remain in place and be encapsulated within the cellular soil-cement panels and struts. A cellular soil-cement ground improvement width of about 40 feet, length of about 100 feet, and height of about 60 feet is assumed at the west riverbank. The conceptual ground improvement design at the west riverbank is shown in Appendix K of the Burnside Bridge Geotechnical Report (Appendix C) on Sheet No. 28, Ground Improvements – Lateral Spread Mitigation.

Seismic mitigation will be required to mitigate the potential permanent ground displacement hazard at the east riverbank. Based on the site conditions and limited overhead clearance, ground improvement using jet grouting is the preferred seismic mitigation alternative at the east riverbank. Ground improvement at the east riverbank is proposed to be performed using low-overhead jet grouting equipment to form two cellular soil-cement ground improvement zones: a primary zone between Pier 4 and the Eastbank Esplanade and a secondary zone between Bent 23 and the UPRR tracks. 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. A cellular soil-cement ground improvement width of about 100 feet, length of about 230 feet, and height of about 100 feet is assumed in front of Pier 4. A cellular soil-cement ground improvement width of about 50 feet, length of about 200 feet, and height of about 120 feet is assumed between Bent 23 and the UPRR. The conceptual ground improvement design at the east riverbank is shown in Appendix K of the Burnside Bridge Geotechnical Report (Appendix C), on Sheet No. 28, Ground Improvements – Lateral Spread Mitigation.

8 Seismic Retrofit and Widening – Concrete Approach Spans

8.1 Seismic Retrofit and Widening Alternatives

Through this study, a number of seismic retrofit and widening alternatives were developed. Alternatives for west and east approach span structure seismic retrofit and widening are listed in Table 8-1.

Table 8-1. Hybrid Seismic Retrofit and Widening Alternatives

Alternative		Description
4a.1	Seismic Retrofit	Replace Highway Spans (20 to 23)
4a.2	Retrofit + Widening	Replace Highway Spans (20 to 23)
4b.1a (Baseline)	**Seismic Retrofit**	**Replace All Highway and RR Spans (20 to 24)**
4b.2a (Baseline)	**Retrofit + Widening**	**Replace All Highway and RR Spans (20 to 24)**
4c.1	Seismic Retrofit	Replace East Approach Spans (20 to 27)
4c.2	Retrofit + Widening	Replace East Approach Spans (20 to 27)
4d.1	Seismic Retrofit	Replace River + East Approach Spans (14 to 27)
4d.2	Retrofit + Widening	Replace River + East Approach Spans (14 to 27)

Among above listed alternatives, Alternative 4b.1 and Alternative 4b.2 are baseline alternatives that are described in detail below. Other alternatives are also briefly described.

8.2 Seismic Retrofit Strategy

As discussed in Section 3.2.1, the project performance requirements for the design events are Full Operation after a CSZ event and Limited Operation after a 1000-year event.

For the west and east approach spans, conventional Phase II seismic retrofit strategies can still apply, although the higher than normal performance requirements mean higher construction cost and longer construction time.

8.3 Construction Sequence

Except where entire span replacements are proposed, such as east approach Spans 20 to 24, the general retrofit approach allows for phased construction, only partially removing the bridge deck during each phase of the construction.

8.4 Alternative 4b.1 Seismic Retrofit without Widening

8.4.1 West Approach Seismic Retrofit

8.4.1.1 Bridge Deck and Girders

At all expansion bents, replacement of the existing seismic restrainers is proposed, as well as using post-tensioning to strengthen the existing positive moment stringer connections to the fixed bents (see Figure 8-1).

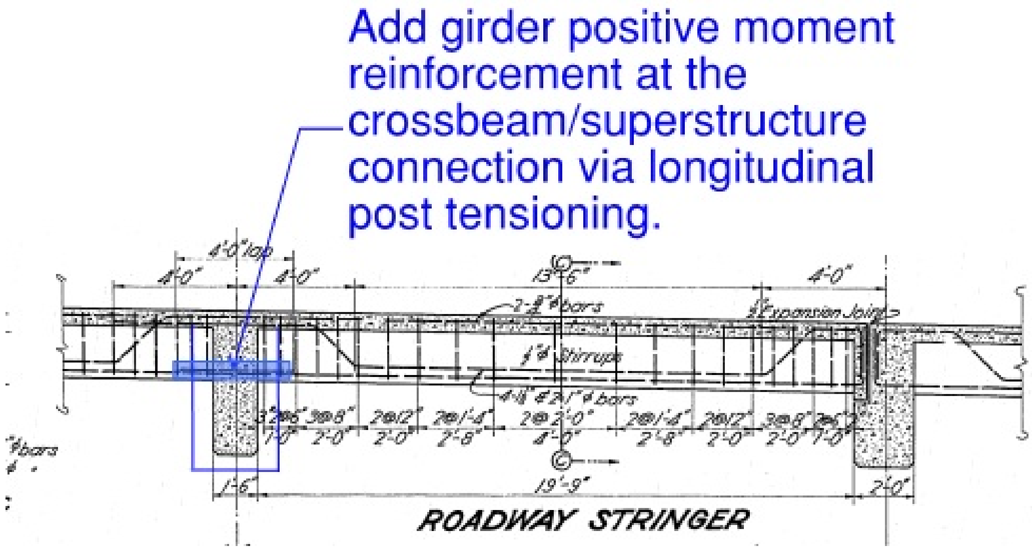


Figure 8-1. Girder Strengthening

8.4.1.2 Bent 1 (Abutment)

At Bent 1, a reinforced concrete thickening of the bent wall is proposed by drilling and doweling reinforcement into the existing wall. Increasing the footing width with a reinforced concrete section is also proposed (see Figure 8-2).

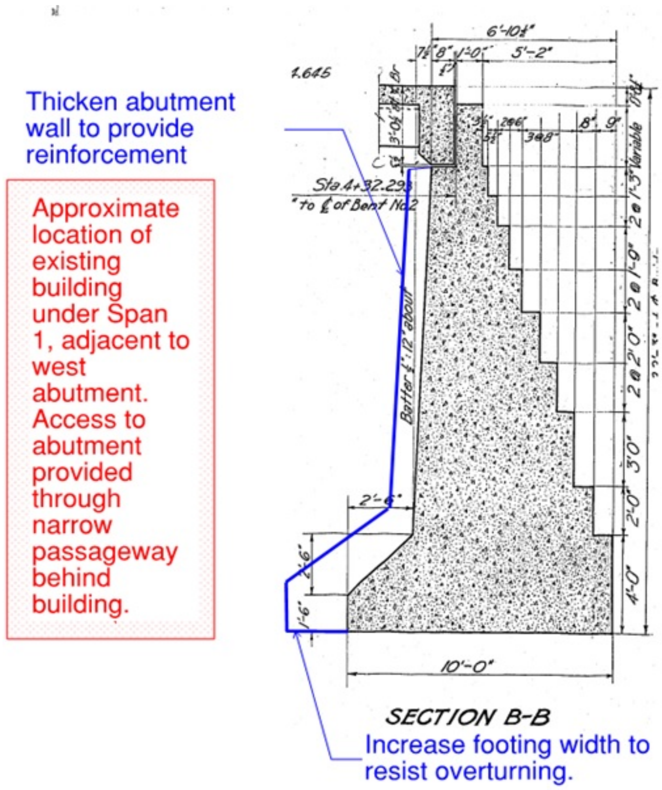


Figure 8-2. Abutment Retrofits

8.4.1.3 Floor Beams and Columns

At Bents 2 through 19, end floor beam strengthening is proposed by enlarging the concrete section and adding post-tensioning. Applying steel column casing with reinforced concrete in the annulus of the casing is also proposed. The new longitudinal column reinforcement would be anchored into the floor beam enlargement and enlarged spread footings or grade beams (see Figure 8-3).

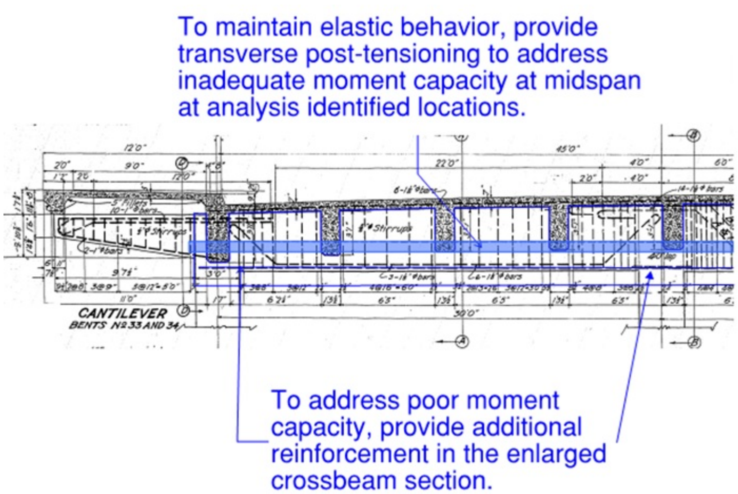


Figure 8-3. Floor Beam Strengthening

8.4.1.4 Spread Footings

At Bents 2 through 16, enlargement of the spread footings in plan with reinforced concrete section is proposed by drilling and doweling reinforcement into the side of the footing. In addition, it is proposed that the footing be thickened to allow for a top mat of reinforcement and for anchorage of the new column reinforcement (see Figure 8-4).

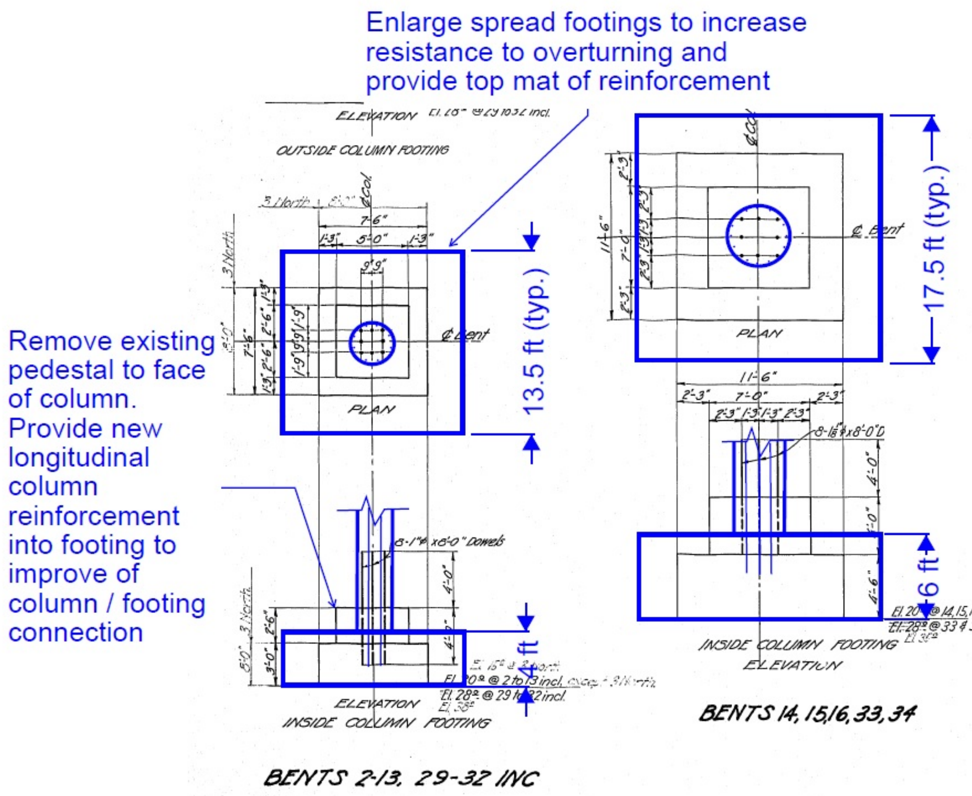


Figure 8-4. Spread Footing Enlargement

8.4.1.5 Pile Foundations

At Bents 17 to 19, a large post-tensioned grade beam is proposed that would carry the loads from the existing columns to new large-diameter drilled shafts constructed on each side of the existing bridge. The new shafts would be constructed outside of the existing bridge deck extents and extend through the liquefiable soil to suitable material for carrying the vertical loads (see Figure 8-5).

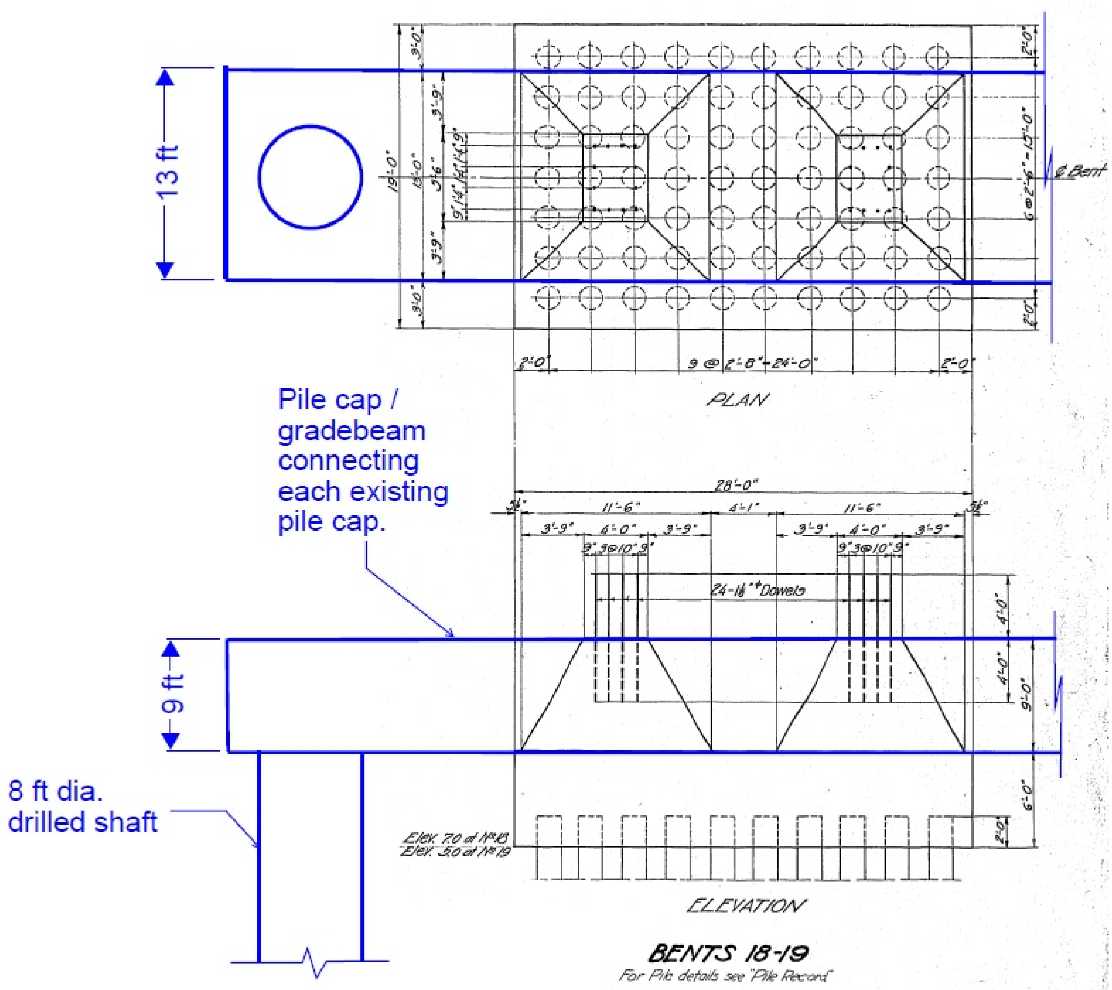


Figure 8-5. Pile Foundation Retrofit

8.4.1.6 Geotechnical Hazard Mitigation

At Bents 2 through 16, cellular soil-cement ground improvement is proposed at each bent. A zone of cellular soil-cement ground improvement is also proposed between Bent 19 and Pier 1. Geotechnical hazard mitigation is described further in Chapter 7.

8.4.2 East Approach Seismic Retrofit

8.4.2.1 Spans 20 to 24 Bridge Replacement

Due to the constructability challenges associated with the I-5 structures beneath Spans 20 to 22 and UPRR beneath Spans 23 and 24, it is proposed that these spans be replaced with a three-span steel plate girder structure on modern reinforced concrete bents supported by large-diameter drilled shafts that extend through the liquefiable material to suitable material for carry vertical loads (Figure 8-6).

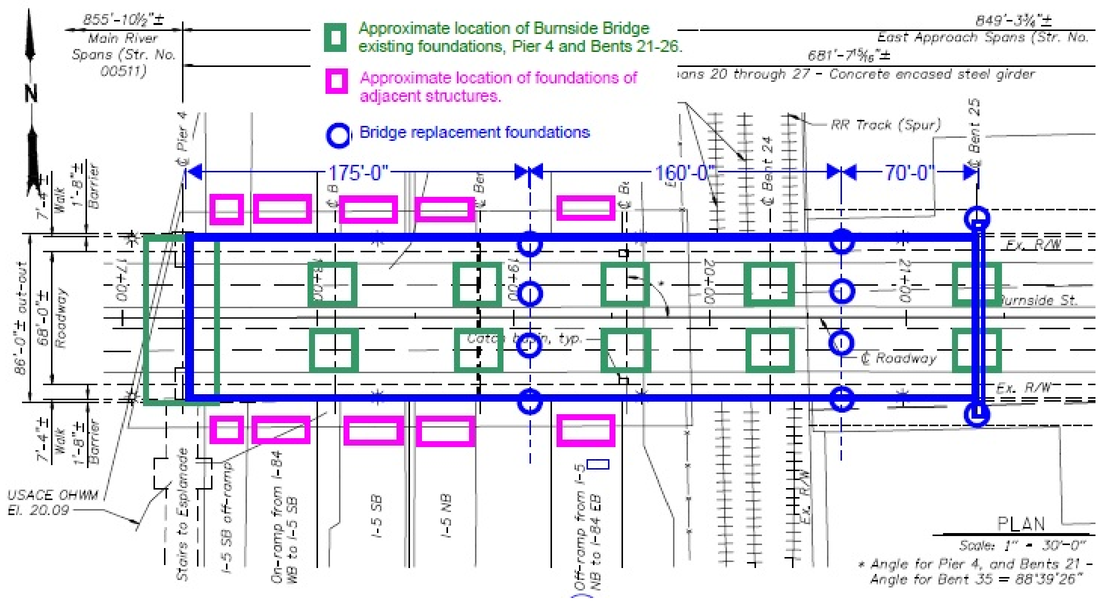


Figure 8-6. Spans 20 to 24 Replacement

8.4.2.2 Bridge Deck and Girders

At all remaining expansion bents, replacement of the existing seismic restrainers is proposed. In addition, strengthening of the existing positive moment stringer connection to fixed bents is proposed using post-tensioning. Remaining rocker bearings supporting the concrete-encased steel girders would need to be replaced (Figure 8-7).

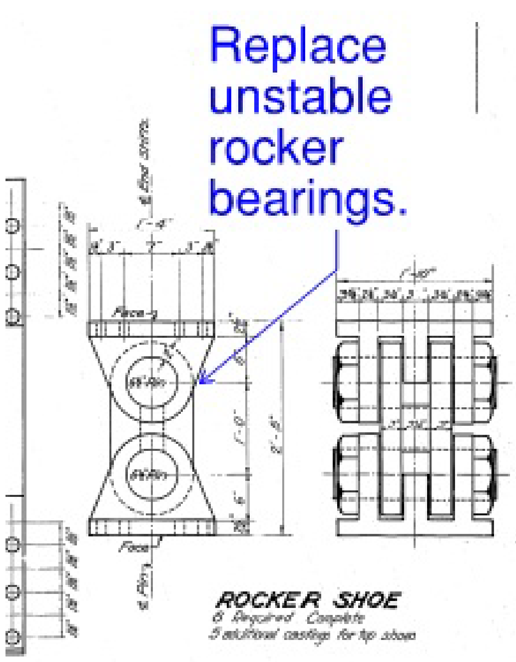


Figure 8-7. Rocker Bearing Replacement

8.4.2.3 Floor Beams and Columns

At Bents 25 to 28, strengthening the concrete-encased steel end floor beams is proposed where the main girders tie into the end floor beams as well as at the column-to-floor beam connection (Figure 8-8). Strengthening of the concrete-encased steel columns and cross bracing is proposed along with the addition of a partial height-reinforced concrete infill wall to strengthen the bents (Figure 8-9).

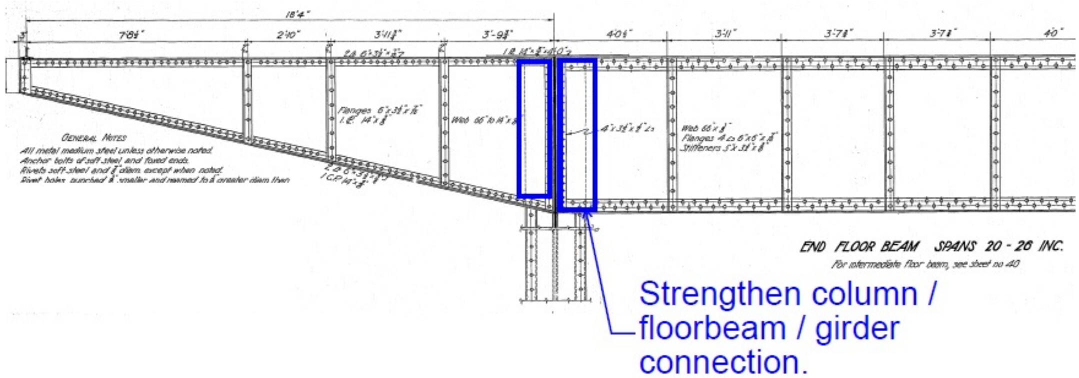


Figure 8-8. Column to Floor Beam Strengthening

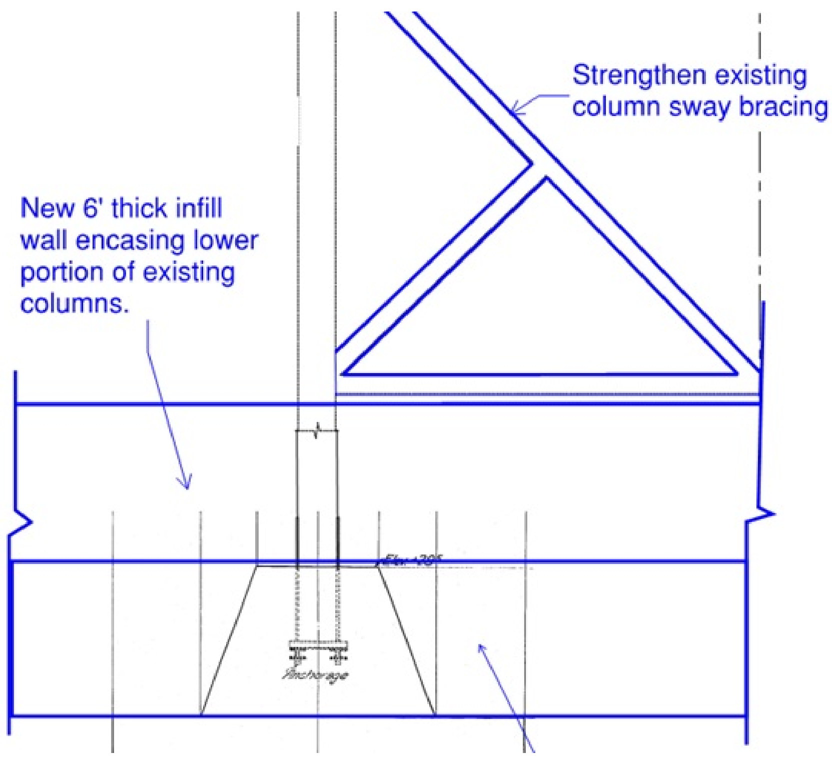


Figure 8-9. Infill Wall

At Bents 29 through 34, end floor beam strengthening is proposed by enlarging the concrete section and adding post-tensioning (Figure 8-3). Applying steel column casing with additional reinforced concrete in the annulus of the casing is also proposed. The new longitudinal column reinforcement would be anchored into the floor beam enlargement and enlarged spread footings (Figure 8-4).

Longitudinal bracing between Bents 25 and 26 is proposed to provide the additional longitudinal stiffness needed (for the bridge) that cannot be addressed by the adjacent bent and foundation retrofits (Figure 8-10).

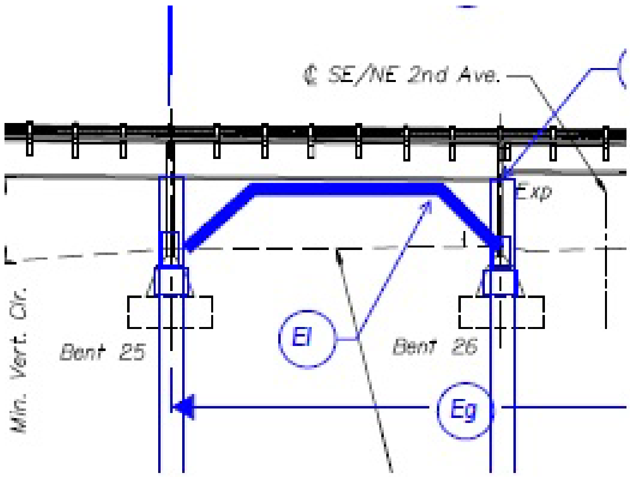


Figure 8-10. Longitudinal Bracing

8.4.2.4 Bent 35 (Abutment)

At Bent 35, a reinforced concrete thickening of the bent wall is proposed by drilling and doweling reinforcement into the existing wall. Increasing the footing width with a reinforced concrete section is also proposed (Figure 8-2).

8.4.2.5 Spread Footings

At Bents 28 to 34, enlargement of the spread footings in plan with reinforced concrete section is proposed by drilling and doweling reinforcement into the side of the footing. In addition, it is proposed that the footing be thickened to allow for a top mat of reinforcement and to allow for anchorage of the new column reinforcement (Figure 8-4).

8.4.2.6 Pile Foundations

At Bents 25 to 27, a large post-tensioned grade beam is proposed that would carry the loads from the existing columns to new large-diameter drilled shafts constructed on each side of the existing bridge. The new shafts would be constructed outside of the existing bridge deck extents and extend through the liquefiable soil to suitable material for carrying the vertical loads (Figure 8-5).

8.4.2.7 Geotechnical Hazard Mitigation

A zone of cellular soil-cement ground improvement is proposed between Bent 23 and 24. Geotechnical hazard mitigation is described further in Chapter 7.

8.4.3 Constructability

The seismic retrofits described above are able to be constructed using a variety of methods and staging including detouring of traffic, staged construction, and rapid reconstruction. Impacts to adjacent facilities and cost will be dependent on the method and staging of construction.

8.5 Alternative 4b.2 Seismic Retrofit and Widening

8.5.1 West Approach Seismic Retrofit and Widening

8.5.1.1 Seismic Retrofit

The proposed seismic retrofit work items for Alternative 4b.2 are the same as Alternative 4b.1, though the amount of strengthening required in some instances may be reduced slightly due the additional load path provided by the widening structure.

8.5.1.2 Widening

Widening of the west approach would be needed in Spans 14 to 19 to achieve an 110-foot structure width throughout the bridge length. In order to widen, additional concrete deck and girders are proposed along with a lengthened floor beam. New columns would be added at Bents 17 to 19 which would be supported by the grade beam and drilled shafts that are also required for the seismic retrofit (Figure 8-11).

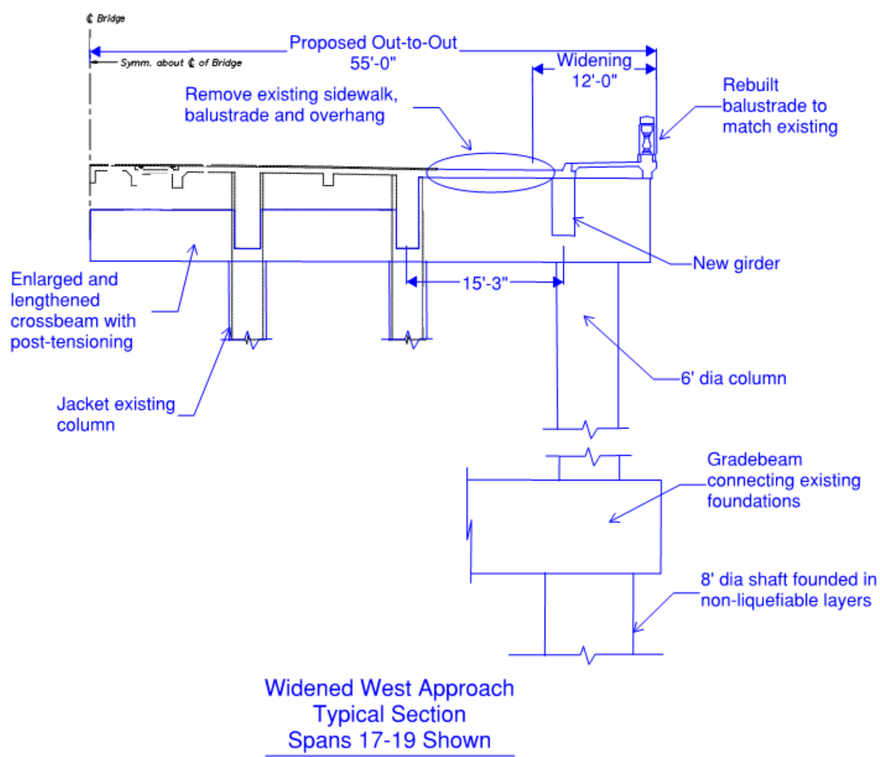


Figure 8-11. West Approach Widening

8.5.2 East Approach Seismic Retrofit and Widening

8.5.2.1 Seismic Retrofit

The proposed seismic retrofit work items for Alternative 4b.2 are the same as for Alternative 4b.1, though the amount of strengthening required in some instances may be reduced slightly due the additional load path provided by the widening structure.

8.5.2.2 Widening

Widening of the east approach would be needed in Spans 21 to 27 to achieve a 110-foot structure width throughout the bridge length. For Spans 20 to 24, which are being replaced in this alternative, the new structure width would be 110 feet. In order to widen the remaining Spans 25 to 27, additional steel girders and concrete deck are proposed along with a lengthened floor beam. New columns would be added at Bents 25 to 26, which would be supported by the grade beam and drilled shafts that are also required for the seismic retrofit (Figure 8-12).

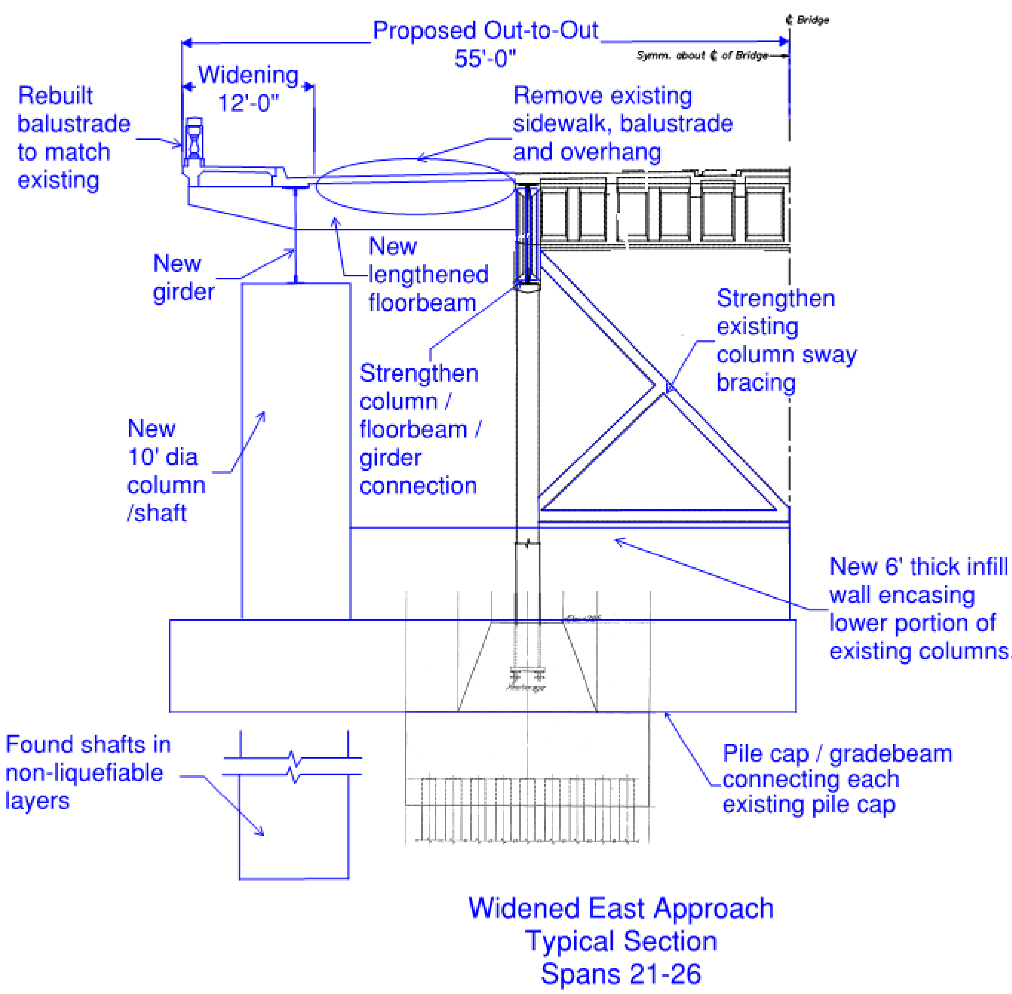


Figure 8-12. East Approach Widening

8.6 Additional Seismic Retrofit and Widening Alternatives

Additional seismic retrofit and widening alternatives considered and the changes from the baseline alternatives described in Sections 8.4 and 8.5 are described in Table 8-2 and Table 8-3.

Table 8-2. Hybrid Seismic Retrofit Alternatives Comparison

Alternative		Comparison to Baseline Seismic Retrofit Alternative
4a.1	Replace Highway Spans (20 to 23)	West Approach – No change East Approach – Seismic retrofit of Spans 23 and 24 required
4b.1	Replace All Highway and RR Spans (20 to 24)	Baseline (See Section 8.4)
4c.1	Replace East Approach Spans (20 to 27)	West Approach – No change East Approach – Seismic retrofit of Spans 25 to 27 addressed by replacement
4d.1	Replace River and East Approach Spans (14 to 27)	West Approach – Seismic retrofit of Spans 14 to 19 addressed by replacement East Approach – Seismic retrofit of Spans 25 to 27 addressed by replacement

Table 8-3. Hybrid Seismic Retrofit (with Widening) Alternatives Comparison

Alternatives		Comparison to Baseline Hybrid Seismic Retrofit + Widening Alternative
4a.2	Replace Highway Spans (20 to 23)	West Approach – No change East Approach – Seismic retrofit and widening of Spans 23 and 24 required
4b.2	Replace All Highway and RR Spans (20 to 24)	Baseline (See Section 8.5)
4c.2	Replace East Approach Spans (20 to 27)	West Approach – No change East Approach – Seismic retrofit and widening of Spans 25 to 27 addressed by replacement
4d.2	Replace River and East Approach Spans (14 to 27)	West Approach – Seismic Retrofit and widening of Spans 14 to 19 addressed by replacement East Approach – Seismic retrofit and widening of Spans 25 to 27 addressed by replacement

9 Seismic Retrofit and Widening –Main River Spans

9.1 Alternatives for Main River Span Structures

The main river span structure includes two fixed-span steel truss spans and the bascule span over the navigational channel. There are two alternatives for main span structure seismic retrofit and widening:

- Seismic Retrofit without Widening
- Seismic Retrofit with Widening

9.2 Steel Truss Fixed-Span Seismic Retrofit

The analysis was conducted to verify the proposed retrofit schemes and effectiveness. Table 9-1, below indicates that after the foundation retrofit and lateral restrainer are installed, the member C/D ratios under a CSZ earthquake event. The numbers in the table indicate that foundation enlargement and lateral restrainer installation can improve members' C/D ratios; however, some members are expecting more seismic demand forces because of the reduced displacements to achieve operational performance requirements. Therefore, the C/D ratio actually reduced, indicating more retrofits are needed on these members. The seismic retrofit analysis is a step-by-step process.

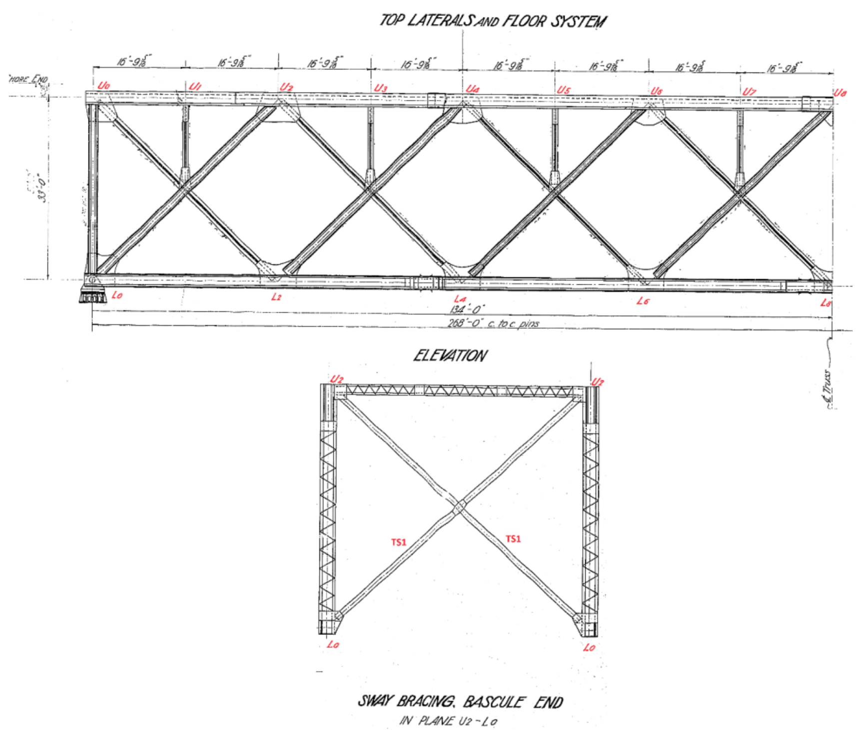


Figure 9-1. Top Laterals and Floor System; Sway Bracing

Table 9-1. C/D Ratio Summary after Foundation Retrofitted of Fixed Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio
U8U9	Top Chord at Midspan	Compression	1.92
L8L9	Bottom Chord at Midspan	Tension Yield	1.82
L8L9	Bottom Chord at Midspan	Tension Fracture	2.80
U8L9	Diagonal Near Midspan	Compression	1.80
TS1	Out of Plane Bracing Near Bascule	Compression	0.15
TS1	Out of Plane Bracing Near Bascule	Tension Yield	0.85
TS1	Out of Plane Bracing Near Bascule	Tension Fracture	1.30
U16L16	End Post Near Bascule	Compression	2.84
—	Fixed End Support Anchor Bolts	Shear	0.34

9.2.1 Pier Foundation Enlargements

9.2.1.1 Piers 1 and 4 Foundations

Because of the multiple potential failures identified in Chapter 6, these pier foundations should be retrofitted.

It would be unpractical to replace the existing unreinforced pile caps and to drill additional piles under the existing structures without removing the bridge superstructures.

One of the primary retrofit objectives is to minimize traffic impact using this bridge. Therefore the proposed retrofits described below would minimize the traffic impacts.

Drilled Shafts around the Existing Pile Caps

New drilled shafts are proposed around the existing pile caps (Figure 9-2) for the following purposes:

- Increase the foundation vertical capacities during a design-level seismic event.
- Increase the lateral load-carrying and ductility capacities.
- Mitigate the foundation settlement risks.

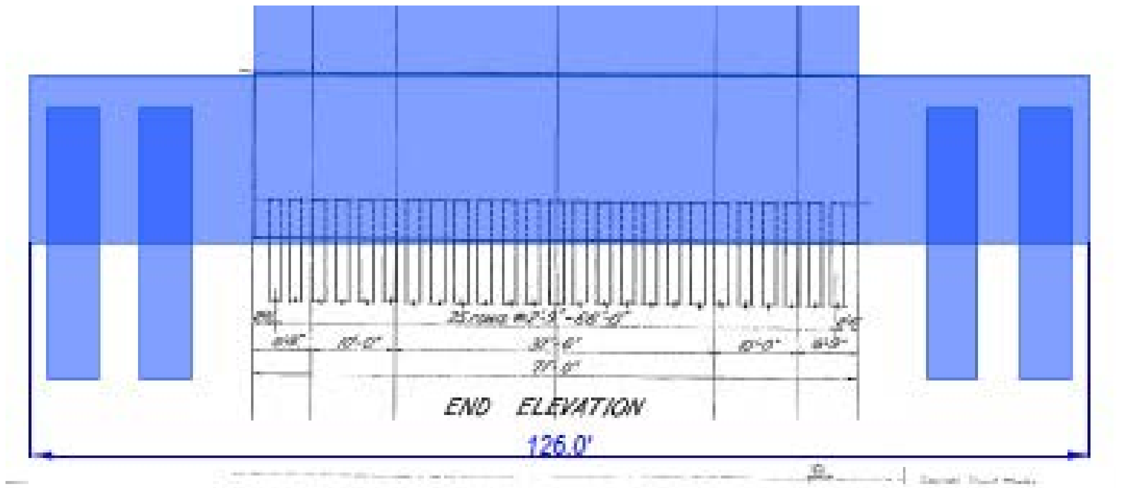


Figure 9-2. Pier 1 and Pier 4 Foundation Enlargement

Pile Cap Enlargement (Extension)

The existing concrete pile caps will be enlarged or extended to cover the new drilled shafts around the existing foundations at Piers 1 and 4. The enlarged pile caps will be connected to the new drilled shafts and also connected to the existing pile caps by using dowel bars and post-tensioning.

9.2.1.2 Piers 2 and Pier 3 Foundations

Foundation retrofits and widening for Pier 2 and Pier 3 are described in the Bascule Span section.

9.2.1.3 Constructability

Vertical Clearances

Some of the drilled shafts will need to be constructed under the existing bridge superstructures, which is feasible; however, this would increase construction costs and extend the construction time. Potential construction methods include the following:

- Construct the drilled shafts in phases by partially closing the traffic on the bridge and removing part of the bridge deck. The shaft steel casings and reinforcing cages could be dropped from the removed deck spaces.
- Construct under the bridge deck where minimum vertical clearance is allowed. The shaft rebar cage would have to be spliced, leading to longer construction time.

Site Restraints and Construction Access

At Pier 1, there are major underground utility lines, a pump station, and a seawall adjacent to the pier foundation. The foundation retrofit and widening would need to be coordinated with potential utility line relocations, reconstruction of the seawall, and avoid impacting the pump station (Figure 9-3). These costs have been included in the estimate.

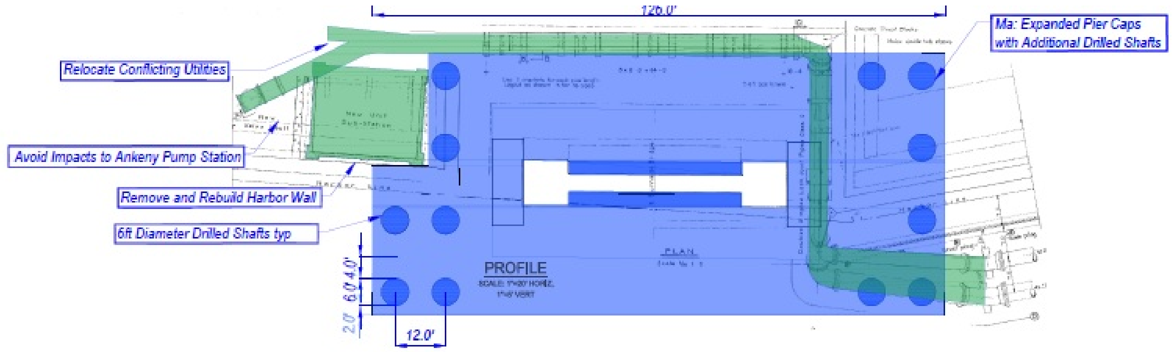


Figure 9-3. Site Restraints at Pier 1

At Pier 4, an I-5 southbound off ramp is immediately adjacent to the Pier 4 columns and above the east portion of the existing foundation, which makes construction access extremely difficult. It is unlikely that large construction equipment could access the east side of the foundation without at least partially removing the elevated I-5 ramp structure.

Since it is required to not have prolonged closures to the I-5 off-ramp bridge during construction, a potential solution is to remove a section of the Eastbank Esplanade to provide construction access. The equipment could therefore be shipped in on a barge. A temporary construction trestle is expected to connect the barge to the land. The extended footing on the east side of Pier 4 could be constructed as follows:

- Micropiles instead of drilled shafts – The micropiles could be constructed under low vertical clearance (Figure 9-4).

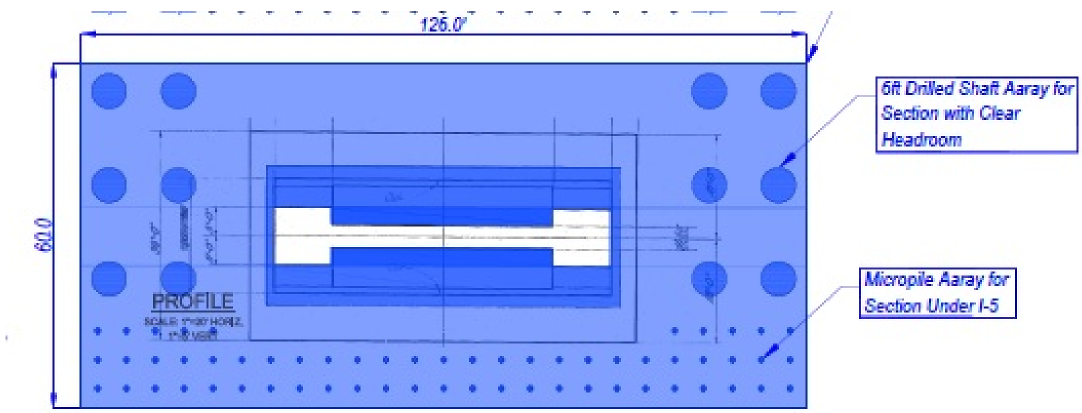


Figure 9-4. Enlarged Pier 4 Pile Layout (Used as the basis of the estimate)

If a refined analysis shows that displacement compatibility between the large shafts and the micropiles cannot be achieved, an alternative solution at Pier 4 is to construct a new Pier 4 on the west side of the existing Pier 4, and demolish the upper portion of the existing Pier 4. The west end of the retrofitted or widened east approach span would be extended and supported by the new Pier 4 (Figure 9-5). It is anticipated that this alternative would increase cost due to the need to revise the truss members and reconstruct the Pier 4 entirely.

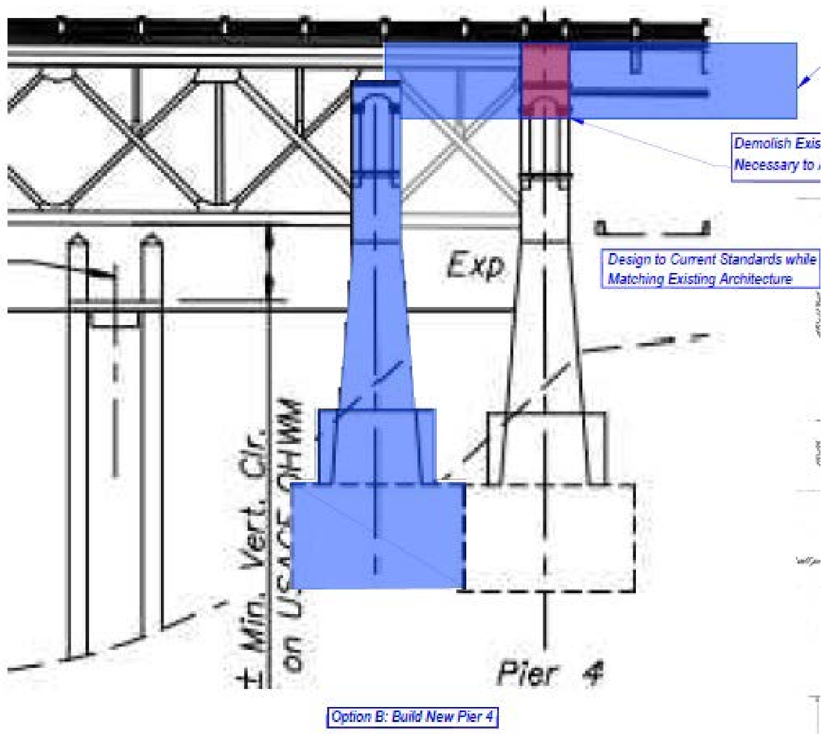


Figure 9-5. Pier 4 Retrofit Alternative (for future consideration)

9.2.2 Piers

9.2.2.1 Pier 1 and Pier 4 Strengthening

Pier 1 and Pier 4 will be strengthened to conform to the seismic design requirements.

The pier columns will be enlarged. The enlarged portions of the columns would have adequate reinforcement to meet the seismic force demand and to provide needed ductility (Figure 9-6).

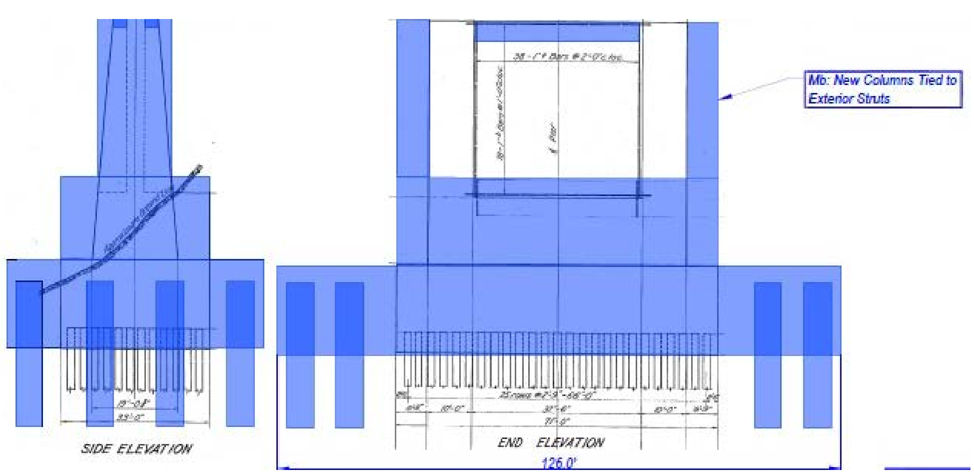


Figure 9-6. Piers 1 and 4 Strengthening

9.2.2.2 Pier 2 and Pier 3

Pier 2 and Pier 3 are described in the Bascule Span section.

9.2.3 Truss Supports and Pier Connections

9.2.3.1 Replace Rocker Type Expansion Bearings

The rocker type expansion bearings at Pier 1 and Pier 4 are recommended to be replaced with low profile type bearings, such as a spherical bearings with a polytetrafluoroethylene (PTFE) sliding surface or fabric bearings.

9.2.3.2 Retrofit Fixed Bearings

The anchor bolts under the fixed bearings at Pier 2 and Pier 3 should be replaced to meet the shear strength requirement for seismic loads. The pier concrete under the bearings should be widened with added reinforcing to prevent concrete splits and cracks. These retrofits will increase the C/D ratio to equal or greater than 1.0 from the existing C/D ratio for shear at 0.34.

9.2.3.3 Seating Length Extension

On the fixed support ends of these steel trusses at Piers 2 and 3, the seating lengths at the pier supports should be extended to conform to the current AASHTO requirements and to prevent the truss girders from falling from the pier tops.

9.2.3.4 Retrofit Transverse Restrainers

As part of the pier column/wall retrofit, the pier columns or walls at Piers 1, 2, 3, and 4 should be strengthened and reinforced with seismic reinforcing details. These retrofitted pier columns or walls would provide effective lateral resistance to the truss supports.

9.2.4 Strengthening the Steel Trusses

9.2.4.1 Add and Strengthen Lateral Bracings

The steel trusses were not designed for seismic loads and movements, therefore adding or strengthening the lateral load-carrying members is required. The members that need to be added or strengthened include diagonal sway bracings and connected vertical members (Figure 9-7), as well as bottom lateral bracings. At minimum, the bracing members at the two bays near each span support should be added or the existing members strengthened. After the foundation and substructure are retrofitted, the bridge will have less movement range during a design-level earthquake, to meet the operational requirements of the bascule span. However, the more stiffed structure will reduce some superstructure members C/D ratio. For example, the C/D ratio for the existing bracing near Pier 2 decreased to 0.15 for compression and 0.85 for tension yield indicating that the forces in these members increase due to the retrofits, therefore these member shall be strengthened.

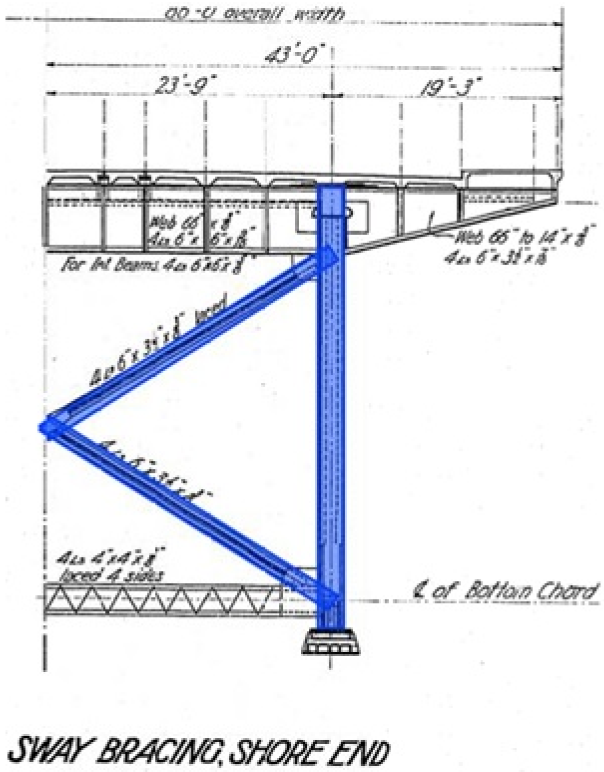


Figure 9-7. Sway Bracing Strengthening Required

9.2.4.2 Strengthen Deck to Floor beam Connections

The bridge deck to steel floor beam connection shear capacity will be strengthened with added shear studs, to enable the deck as part of the lateral load transferring system.

9.3 Steel Truss Fixed-Span Widening

9.3.1 Pier Foundation Widening

For the seismic retrofit combined with widening alternative, the same concept of foundation enlargement as described in the previous retrofit concept section shall apply. New shafts will be drilled around the existing pier caps to the vertical and lateral load carrying capacities. These new shafts are to be connected with the existing pile caps with the enlarged pile cap. The number of new shafts required shall be according to the dynamic model analysis that includes the mass of the widened structure. The size of the pile cap should also the widened accordingly (Figure 9-8).

9.3.2 Pier Widening

For the seismic retrofit combined with widening alternative, the widened structure can be constructed on top of the enlarged pile cap. The widened pier columns would not only support the widened superstructure, but also strengthen the existing pier columns by integrating the new widened portion with the existing pier, (Figure 9-8).

9.3.3 Adding New Steel Trusses

The proposed bridge deck widening is 12 feet on each side. Analysis indicated that strengthening the existing steel truss won't be able to support the added weight of the total 24 feet of widening. This 24-foot widening requires adding a steel truss panel under each of the widened decks. Therefore, the widened truss span would be 110 feet wide (Figure 9-8), with four steel truss panel under the widened deck.

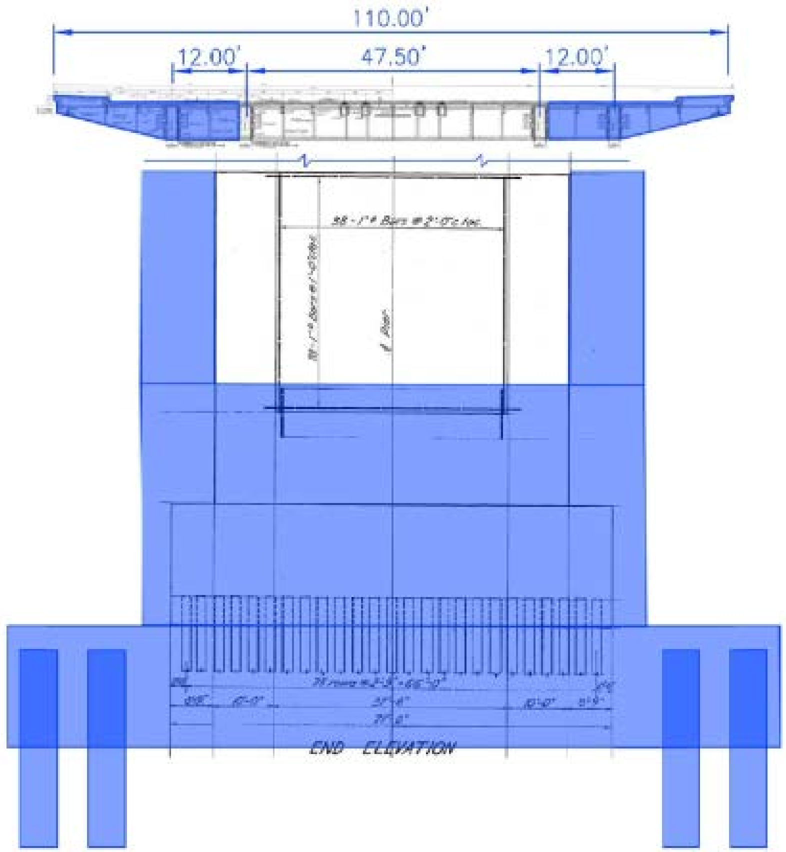


Figure 9-8. Piers 1 and 4 Widening

9.4 Bascule Span Seismic Retrofit

9.4.1 Retrofit Strategies

The bascule piers are massive in size, rigid because of the box shape, and fragile because they are under- or unreinforced. The weight of the entire superstructure, including the bascule leaf, deck, and counterweight, is supported on a set of trunnions, through the trunnion tower support frames down to the concrete pedestals.

Because a bascule bridge structure consists of many rigid elements and links, and due to a lack of ductility, a seismic retrofit strategy using base isolation technology has been discussed in previous project reports and is evaluated in more detail in Section 9.4.5.

In order to meet the operational performance requirements described in Chapter 3and in the EQRB Seismic Design Criteria (Appendix B), the bridge seismic retrofit will not only improve the C/D ratios to meet the strength requirements, but also will also limit the displacements. This requires the analysis to be conducted step by step to identify the required retrofit. For example, eliminating the displacement range to meet the operational performance requirements can result in increased force demands on members and reduce the member C/D ratios.

Unlike a conventional fixed-span bridge that relies on structural ductility to allow movement during a seismic event thus reducing the seismic demand forces, a bascule bridge has very strict displacement restrictions. In order for the bascule span be operational after a design-level earthquake, the bascule leaf mechanical drive gears and pinions have to be functional. Although the exact displacement upper limits are difficult to quantify at this conceptual study, it has been estimated that the displacement limits between the gear racks and the pinions are:

- Transverse: 0.25 inch
- Longitudinal: < 1/100 inch

These displacement limits, together with other factors such as the rigid bascule piers and no ductility capacity at the anchors of the trunnion support frames, make the seismic retrofit of a bascule bridge span more challenging than retrofitting a conventional fixed-span bridge.

Figure 9-9 and Table 9-2 below indicate that after the foundation retrofit and lateral restrainer are installed, the displacements at several key locations are significantly reduced.

Table 9-2. Displacements of Key Points Before and After Foundation Retrofit and Installation of Restrainers

			Post-Retrofit		Existing (As-Built) Condition	
Model Node #	Node Location	Direction	Displacement (in)	Relative Displacement to Top of Pedestal (in)	Displacement (in)	Relative Displacement to Top of Pedestal (in)
20611	Operating Pinion connection To Trunnion Support	X	5.9	0.1	5.8	0.1
		Y	2.8	0.3	2.0	0.4
		Z	0.2	0.0	0.2	0.0
20545	Top of Trunnion Support at Trunnion Pin	X	6.0	0.2	6.4	0.7
		Y	3.2	0.7	2.7	1.1
		Z	0.2	0.0	0.2	0.1
20451	Counterweight Mass	X	6.0	0.2	6.1	0.4
		Y	2.7	0.2	8.7	7.1
		Z	0.2	0.1	0.3	0.1

Table 9-2. Displacements of Key Points Before and After Foundation Retrofit and Installation of Restrainers

			Post-Retrofit		Existing (As-Built) Condition	
Model Node #	Node Location	Direction	Displacement (in)	Relative Displacement to Top of Pedestal (in)	Displacement (in)	Relative Displacement to Top of Pedestal (in)
20716	Live Load Shoe	X	5.9	0.1	5.8	0.1
		Y	2.8	0.2	1.7	0.0
		Z	0.1	0.0	0.1	0.0
20914	Bascule Tip	X	6.1	0.3	6.4	0.6
		Y	4.9	2.4	11.3	9.7
		Z	0.5	0.4	0.6	0.4
20486	Bottom of Pile Cap	X	5.6	-0.3	5.6	-0.2
		Y	2.2	-0.3	1.5	-0.1
		Z	0.2	0.0	0.1	0.0
20503	Top of Pedestal	X	5.9	-	5.7	-
		Y	2.5	-	1.6	-
		Z	0.2	-	0.1	-

Table 9-3 below indicates that after the foundation retrofit and lateral restrainer are installed, member C/D ratios are improved. The table also identifies those members that have C/D ratios less than 1.0, therefore those members need to be strengthened in addition to foundation retrofit and installation of lateral restrainers.

Table 9-3. C/D Ratio Summary After Foundation Retrofitted at Bascule Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio or Interaction
16-C	Counterweight Compression Support	Comp.-Moment Interaction*	0.33
16-C	Counterweight Compression Support	Shear	0.16
Counterweight Link	Counterweight Link	Compression	0.23
Counterweight Link	Counterweight Link	Tension Yield	1.53
Counterweight Link	Counterweight Link	Tension Fracture	1.68
T11	Trunnion Post	Compression	4.54
-	Trunnion Support Anchor Bolts	Combined Tension and Shear	1.55
14-T	Trunnion Brace	Comp.-Moment Interaction*	1.04
14-T	Trunnion Brace	Shear	0.26

Table 9-3. C/D Ratio Summary After Foundation Retrofitted at Bascule Span

Drawing Member #	Member Location	Failure Mode	C/D Ratio or Interaction
14-T	Trunnion Brace	Ten.-Moment Interaction*	0.61
15-T	Trunnion Link	Comp.-Moment Interaction*	0.69
15-T	Trunnion Link	Shear	0.20
15-T	Trunnion Link	Ten.-Moment Interaction*	0.60
14-15	Bottom Chord Bascule Truss	Compression	1.95
12-15	Diagonal Bascule Truss	Tension Yield	2.09
12-15	Diagonal Bascule Truss	Tension Net Fracture	2.80
13-15	Top Chord Bascule Truss	Tension Yield	1.77
13-15	Top Chord Bascule Truss	Tension Net Fracture	2.50
* For Interaction failure, value shown represents 1/(Interaction Result)			

The member designation in the table, such as 16-C, denotes this member connects node 16 to node C, see Figure 9-9.

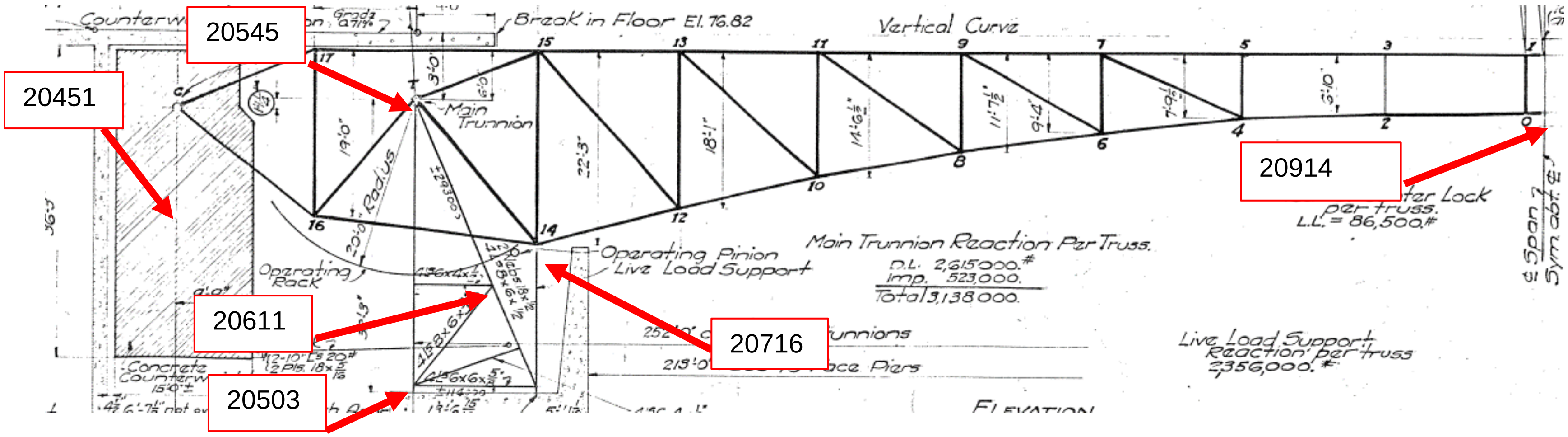


Figure 9-9. Location of Nodes Where Displacement was Checked

9.4.2 Pier Foundation Enlargements

9.4.2.1 Bascule Pier Foundation Enlargements

Because of the multiple potential failures identified in Chapter 6, the bascule pier foundations must be retrofitted.

It is very costly to replace the existing unreinforced pile caps wholesale and place additional piles under the existing structures while minimizing the impacts to the bridge superstructure. Therefore, instead of replacing existing foundations, these existing foundations will be enlarged and strengthened.

One of the primary objectives of the retrofit is to minimize the impacts to bridge traffic and the water traffic below the bridge. Therefore, the proposed retrofits described below would minimize the traffic impacts.

Drilled Shafts around the Existing Pile Caps

New drilled shafts are proposed around the existing pile caps (Figure 9-10) for the following purposes:

- Increase the vertical load-carrying capacities of the foundations during a design-level seismic event.
- Increase the lateral load-carrying and ductility capacities.
- Mitigate the foundation settlement risks.

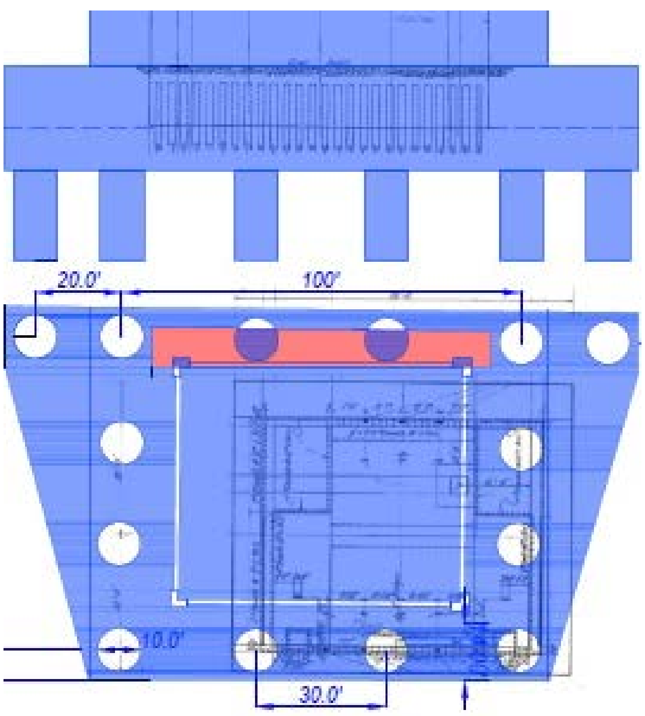


Figure 9-10. Bascule Pier 2 and 3 Enlargement

Pile Cap Enlargement (Extension)

The existing concrete pile caps would be enlarged or extended to cover the new drilled shafts around the existing foundations at Piers 2 and 3. The enlarged pile caps would be connected to the new drilled shafts, and also connected with the existing pile caps by using dowel bars and post-tensioning.

9.4.2.2 Maintain Navigation Channel Clearance

Because Pier 2 and Pier 3 are adjacent to the river navigational channel, the foundation pile cap enlargement on the navigation channel side is restricted.

In order to maintain the existing navigation channel clearance, the proposed pile cap extension on the navigation channel side is limited by rebuilding part of the existing pile cap. The extended pile cap on the navigation channel side is lowered to below the mud line, so the enlarged pile cap is staggered (Figure 9-11).

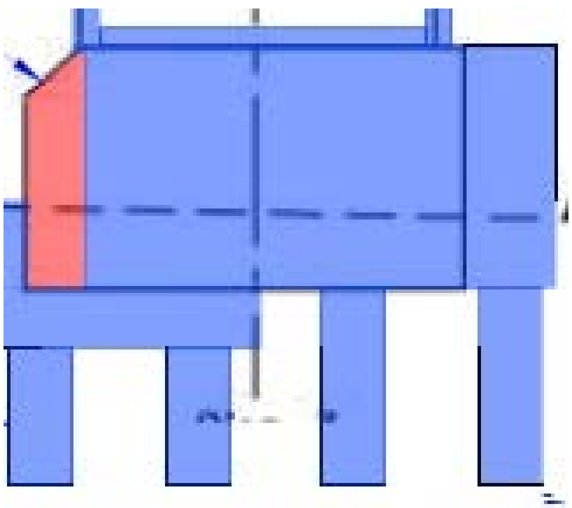


Figure 9-11. Piers 2 and 3 Pile Cap Staggered Enlargement

9.4.2.3 Constructability

Vertical Clearances

Some of the drilled shafts would need to be constructed under the existing bridge superstructures, which is feasible; however, this would increase construction cost and extend construction time. Potential construction methods include the following:

- Construct the drilled shafts in phases by partially closing the traffic on the bridge and removing part of the bridge deck. The shaft steel casings and reinforcing cages could be dropped from the removed deck spaces.
- Construct under the bridge deck where minimum vertical clearance is allowed. The shaft casing and rebar cage would have to be spliced, leading to longer construction time.

Site Restraints and Construction Access

Temporary construction trestles are expected for the in-water foundation construction.

The in-water construction activities may be further restrained by other regulations, such as fish windows, restrictions on pile driving, vessel navigation below the bridge, etc.

9.4.3 Pier Wall

The pier walls at bascule Pier 2 and Pier 3 should be strengthened to conform to the seismic design requirements.

The proposed strengthening is described below.

9.4.3.1 New Columns

Construct four columns, one at each corner of the piers, to act as lateral load-carrying members that transfer the seismic-induced lateral loads from the bridge deck and trunnion support structures to the foundations (Figure 9-12). These corner pier columns are integral with the existing pier walls, and would have an adequate amount of reinforcement to meet the seismic force demand and to provide needed ductility. The bottom of the columns would have dowel bars that are embedded in the pile caps with sufficient embedment length to resist potential uplifting forces during a design-level seismic event.

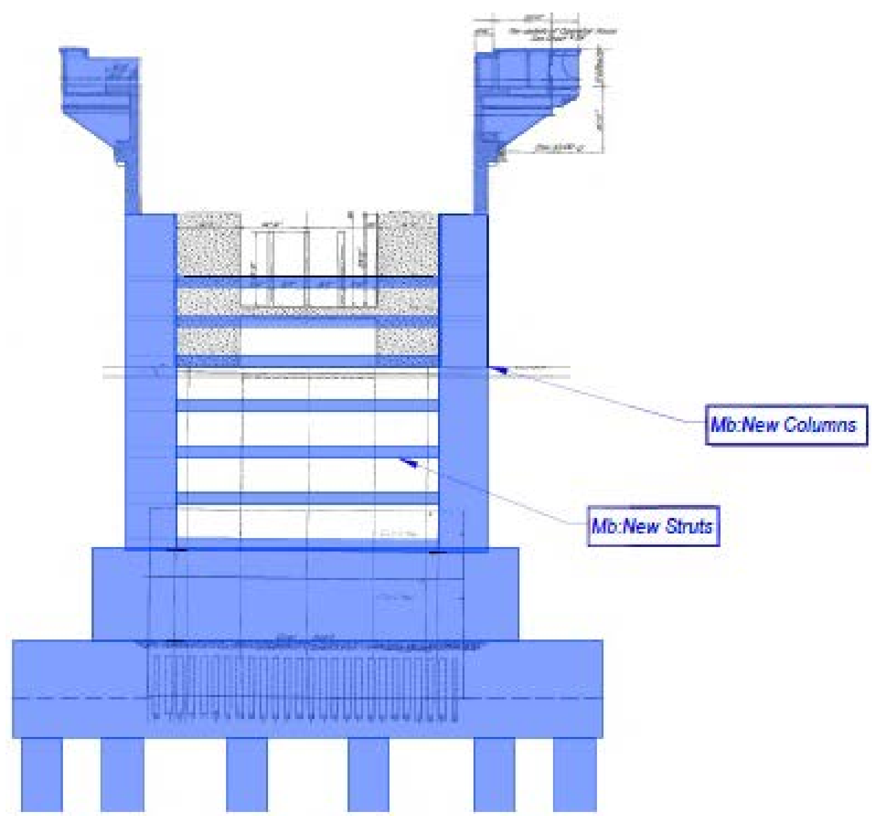


Figure 9-12. Bascule Pier Retrofit with Columns and Horizontal Struts

9.4.3.2 Horizontal Struts

Construct horizontal struts that connect these corner columns, providing confinement and strengthening the pier walls (Figure 9-13).

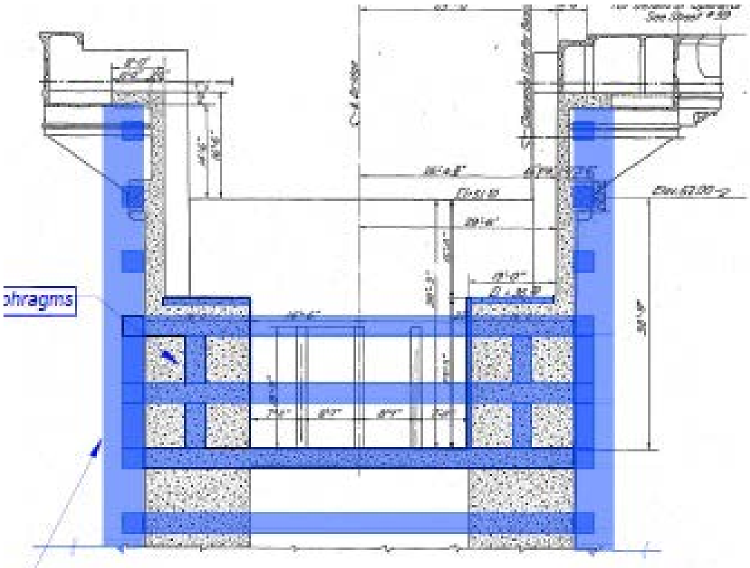


Figure 9-13. Bascule Pier Retrofit with Horizontal Struts

9.4.3.3 Confinements around the Concrete Pedestals and Supports

Provide confinement around the concrete pedestals under the trunnion support frames and fixed truss span supports to prevent concrete from cracking, because the trunnion support frames and the steel fixed truss will exert a huge amount of vertical and horizontal force onto these pedestals and supports during a design-level seismic event (Figure 9-14).

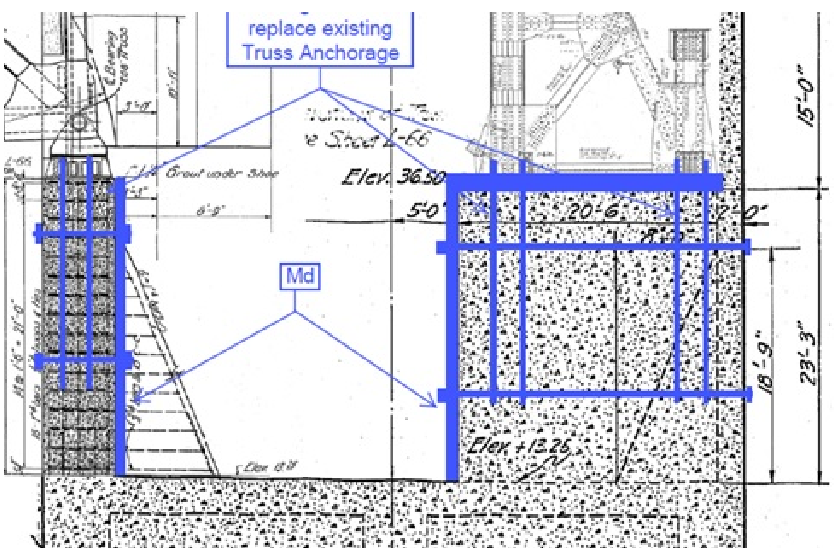


Figure 9-14. Confinement to Concrete

9.4.3.4 Pit Deck Girder Connections

Strengthen the connections between the pier pit deck girders and the pier back walls, as well as the connections between the deck girders and the trunnion support frames (Figure 9-15).

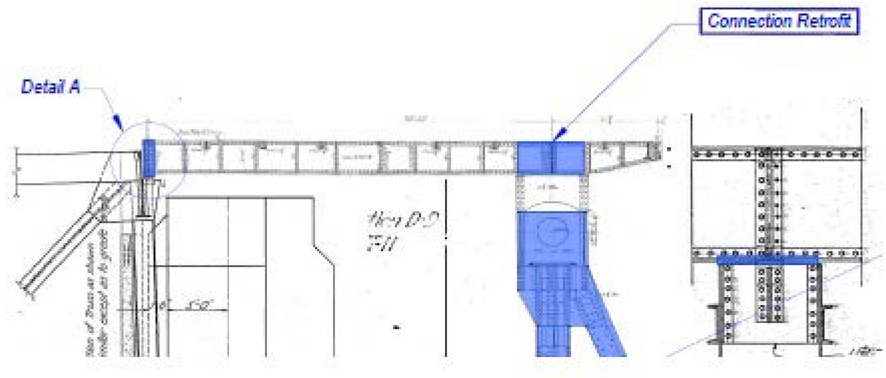


Figure 9-15. Connection Strengthening at Pit Deck Girder Supports

This strengthening will not only prevent the pit deck from falling into the bascule pier pit, but also will provide a horizontal load path that transfers the horizontal load from the top of the trunnion support frames to the pier back wall and corner columns, and further transfer down to the foundation level.

9.4.4 Bascule Leaf Trunnion Supports

9.4.4.1 Trunnion Tower Support Frames

After the foundation retrofit and installation with lateral restrainers to the pier walls, the trunnion tower support frames should be strengthened (Table 9-2) to prevent buckling and to provide adequate supports to the bascule leaves and the counterweights during a design-level seismic event (Figure 9-16).

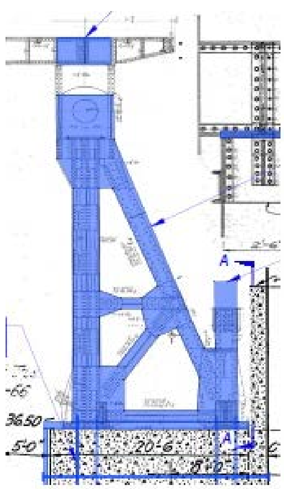


Figure 9-16. Trunnion Tower Support Frame Needs Strengthening

9.4.4.2 Lateral Restrainers

The lateral restrainers installed in 2005 should be replaced as part of the pier wall strengthening. The replaced restrainers should be connected to the retrofitted walls with strut reinforcements that are capable of transferring the lateral load down to the foundations via the corner columns. These retrofitted restrainers will provide longitudinal and transverse restraint to the trunnion support frames to prevent them from buckling or tipping over (Figure 9-17).

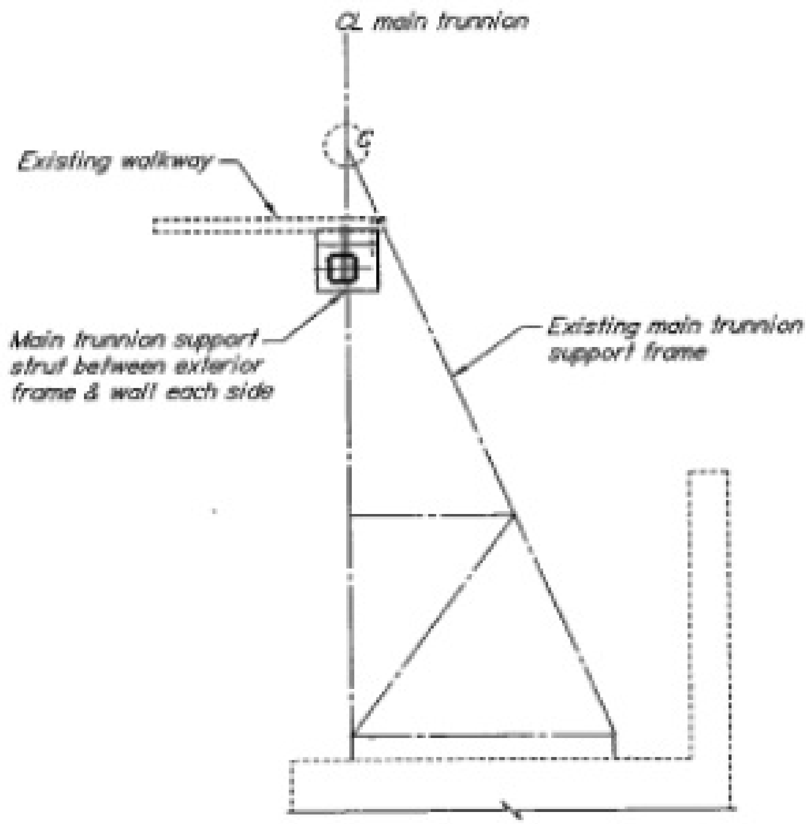


Figure 9-17. Install New Lateral Restrainers

9.4.4.3 Anchor Bolts

Existing anchor bolts should be replaced with larger anchor bolts, and anchor bolts should be added to resist the design-level seismic forces. The embedment depth into the strengthened concrete pedestal below should also be deeper than with the existing condition (Figure 9-18). Comparing the C/D ratios in Table 6-2 and Table 9-2, the installation of lateral restraints reduces the load on the anchor bolts and the C/D ratio for combined tension and shear is improved; however, replacing existing anchor bolts is still recommended at this important location to not solely rely on one retrofit measure. In addition, replacing bolts does not resolve the concrete breakout risk which can fail during a seismic event.

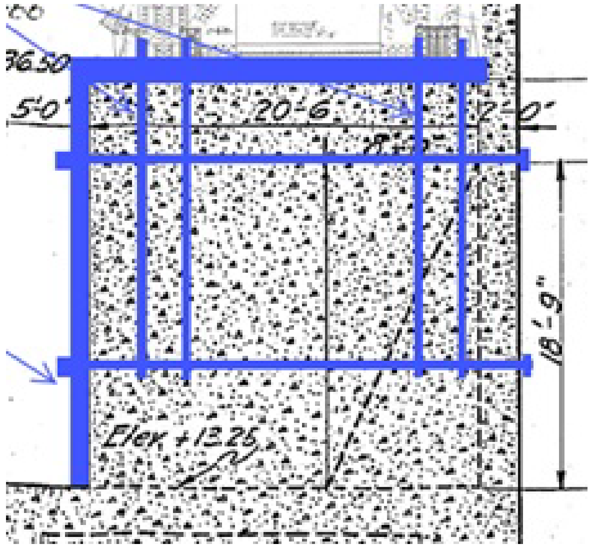


Figure 9-18. Replace Existing Anchor Bolts

9.4.4.4 Trunnion Frame Connections to Pit Deck

The previously described connection retrofits between pit deck girders and the trunnion support frame will provide horizontal supports at the top of the support frame.

9.4.5 Feasibility and Application of Base Isolation

9.4.5.1 Feasibility

A principle of base isolation is minimizing and dissipating the ground movement and energy input to the structure. To achieve this requires two conditions:

- Space to move
- Isolation from the base while retaining a stable structure

Unfortunately, the existing bascule span structure lacks both of these conditions.

- Although the bascule pier looks massive, the space inside the pier available for seismic movement is very limited.
- During a design-level seismic event, the base anchor bolts under the trunnion support frames will resist a significant amount of shear forces and the uplift forces. Should the trunnion support frame be isolated at the base anchor bolt location, the entire bascule span superstructure will become unstable.

In addition, to achieve the Full Operation performance requirement after a CSZ event, the span-driving machinery system has to be functional. This requires the entire machinery system to also be isolated from the bascule pier and attached to the isolated trunnion support frames so that the driving machinery can move together with the bascule leaf. Due to the limited space inside the bascule pier, isolating the machinery system together with the trunnion support system would lead to redesigning, rearranging, and replacing the entire machinery system.

Furthermore, to isolate the bascule superstructure and still keep the span stable, the center span lock needs to be retrofitted to resist the seismic loads. Additional support locations in addition to the supports under the trunnion support frames would need to be provided; such locations may be at the counterweight or retrofitted live load shoes, for example.

9.4.5.2 Base Isolation Evaluation

After evaluating the benefits and concerns, our proposed seismic retrofit approach regarding the application of base isolations are described below.

Seismic Retrofit-Only Alternative

Base isolation will not be applied to the bascule structure due to the abovementioned concerns. Seismic Retrofit and Widening Alternative

It is more feasible to apply base isolation for the Seismic Retrofit and Widening Alternative by incorporating base isolation technology into the widening designs.

Applying base isolation to a bascule bridge to modify its seismic behavior requires redesigning and replacing additional structural elements or other components; for example, the entire electrical and machinery system would be replaced and rearranged, and additional isolated supporting locations would be needed. Therefore, more detailed analysis would need to be performed to confirm the feasibility.

The retrofit strategies described in this report do not incorporate base isolation technology. From a cost point of view, our conceptual engineering judgment is that the construction costs, with base isolation or without, are of approximately the same magnitude.

9.4.6 Counterweight Supports

9.4.6.1 Counterweight Support Frames

The counterweight support frames should be strengthened to prevent buckling and to provide adequate support to the counterweight during a design-level seismic event (Figure 9-19). The need for the strengthening is actually increased, as shown in Table 9-2 for member C-16. C-16 sees an increase in loading and reduction in C/D ratio after retrofits to the piers that limit the displacements for meeting the operational requirements. The C/D ratio for shear decreases to 0.16, and the inverse of the compression-moment interaction decreases to 0.33.

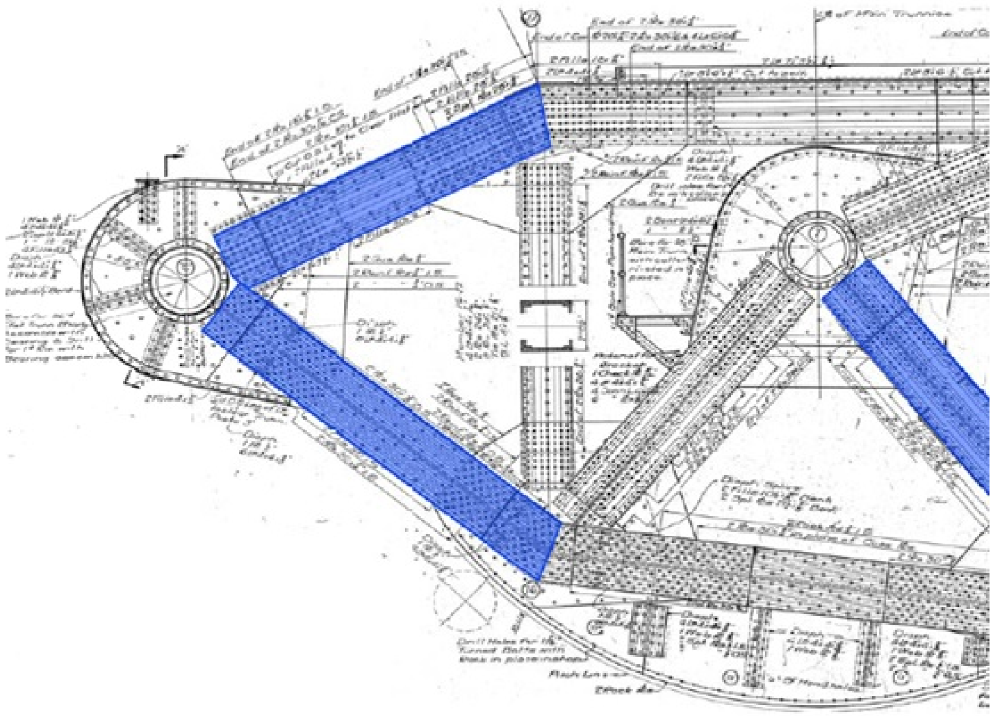


Figure 9-19. Counterweight Support Frame

9.4.6.2 Lateral Restrainers

Lateral restrainers should be installed on counterweight frames and the pier walls to prevent the counterweight from unrestrained sway. Two sets of restrainers are required: one at a position when the bascule span is closed, and the other at a position when the bascule span is fully open (Figure 9-20).

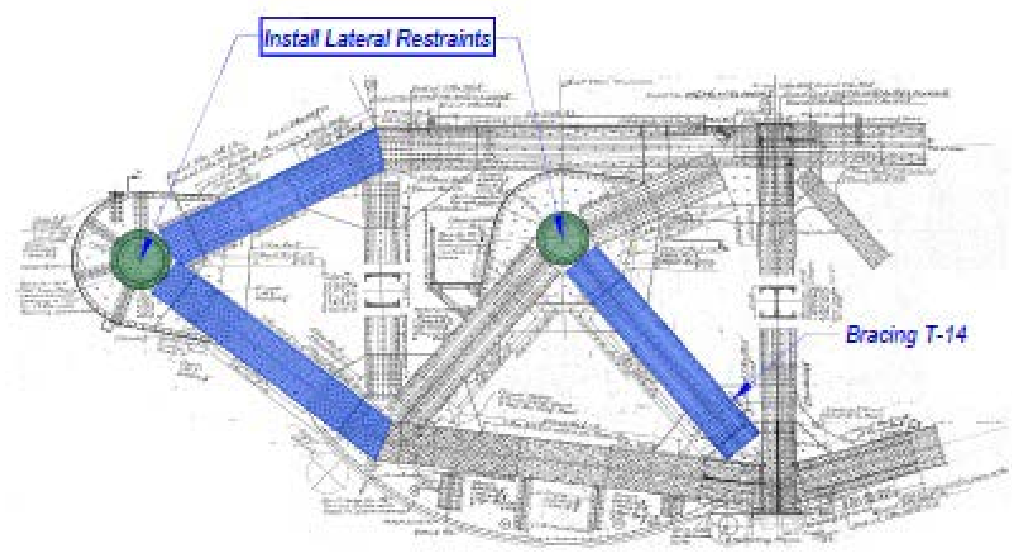


Figure 9-20. Seismic Lateral Restrainer Locations

9.4.6.3 Counterweight Link

The counterweight link resisting motion of the counterweight along the longitudinal direction of the bridge will be strengthened or replaced. With the additional retrofits, the counterweight link increases its C/D ratios but still has a C/D ratio less than 1.0 for compression, which was 0.23.

9.4.7 Superstructure Connection to Trunnion

Since the bracing frame T-14 (Figure 9-21) transmits lateral loads from the entire bascule span to the trunnion support frame, these members will strengthened to prevent them from buckling and yielding. The forces on T-14 increase after the retrofits to the pier that limit displacements for meeting the operational requirements. The C/D ratio for shear in T-14 decreases to 0.26, and the inverse of the tension-moment interaction decreases to 0.61. In addition to T-14, member T-15 has increased forces after the retrofit. The inverse of the compression-moment interaction decreases to 0.69, the shear C/D ratio reduces to 0.20, and the inverse of the tension-moment interaction decreases to 0.60. The members of T-14 and T-15 shall be reinforced to bring the C/D ratio to above 1.

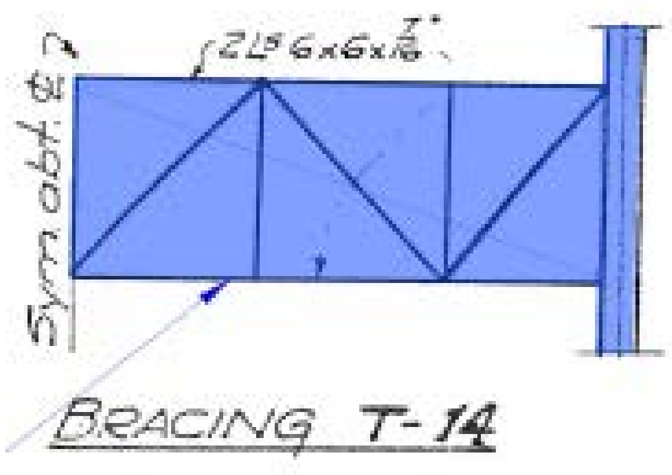


Figure 9-21. Lateral Bracing at Trunnion Support Frame

9.4.8 Live Load Support Connections

The live load support shoes should be retrofitted to provide lateral restraint to the bascule leaves. By provide three lateral restraining points to the bascule leaf—at the trunnion, at the counterweight, and at the live load shoes (Figure 9-22)—it can more effectively reduce the horizontal sway of the bascule leaf.

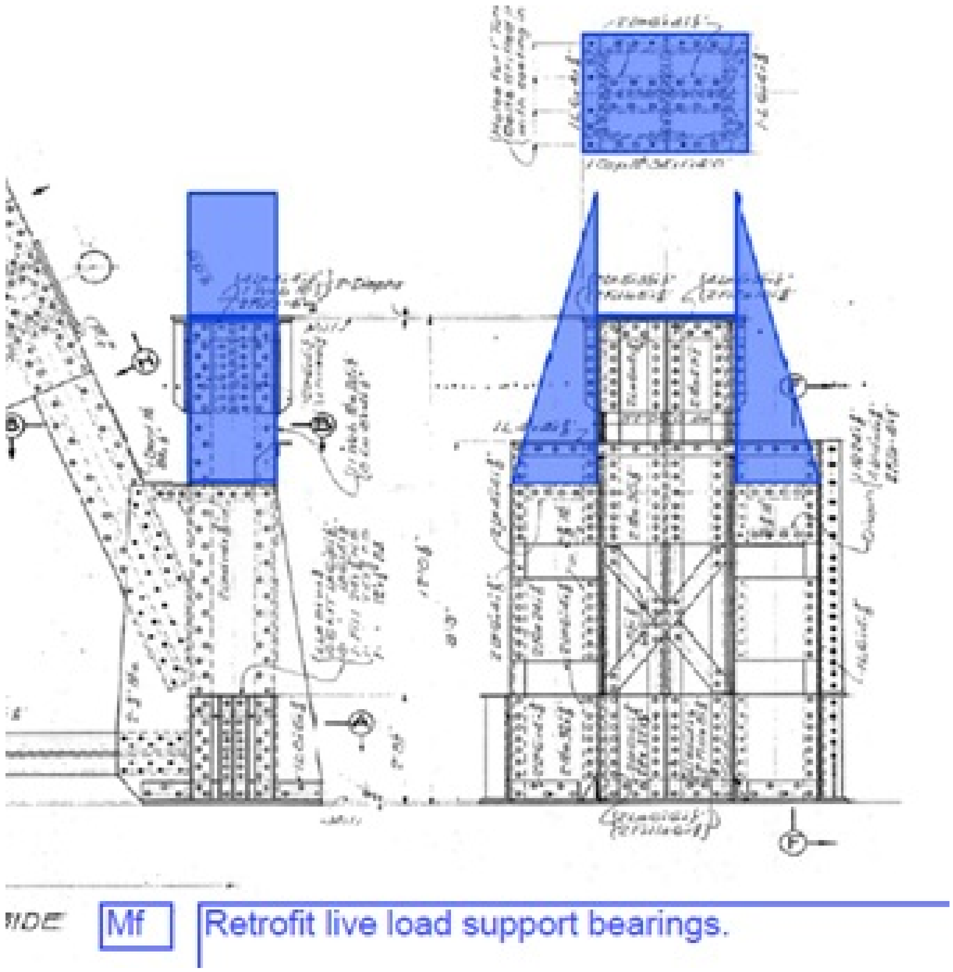


Figure 9-22. Retrofit of Live Load Shoes

9.4.9 Pit Deck Supports

As part of the overall pier wall retrofit, increased seating length on top of the pier walls under the pit deck stringers will be provided to prevent unseating of the pit deck over the tops of Piers 2 and 3 (Figure 9-15).

9.4.10 Strengthening the Bascule Leaves

9.4.10.1 Add and Strengthen Lateral Bracings

The bascule leaf trusses were not designed for seismic loads and movements, therefore adding or strengthening the lateral load-carrying members are required. The members that should be added or strengthened include diagonal sway bracings and connected vertical members, as well as bottom lateral bracings (Figure 9-23). The bracing members at truss member connections 13, 14, and 15 have the most need to be strengthened.

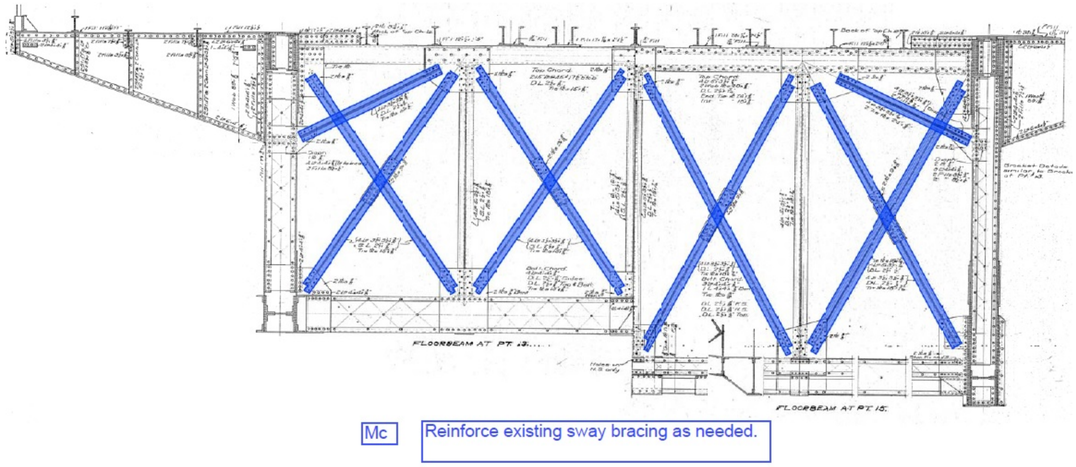


Figure 9-23. Strengthening of Bascule Leaf Lateral Bracings

9.4.10.2 Strengthen Deck to Floor Beam Connections

Shear capacity of the deck to steel floor beam connection should also be verified and strengthened as needed.

9.4.11 Center Lock Shear

The existing center lock should be replaced with a new type that can provide restraint to the relative transverse displacement at the tips of the two bascule leaves or, alternatively, a separate lateral restrainer should be installed to prevent the relative tip movements during a design-level seismic event.

9.4.12 Structures on the Bridge

Other structures attached on the bridge, such as the overhead sign structure and light poles, should be checked and strengthened to prevent them from collapse onto the bridge during a design-level seismic event. It was assumed that these modifications, if required, are absorbed within the cost estimate Contingency.

9.4.13 Geotechnical Hazard Mitigation

With the proposed bascule pier foundation enlargements and the added drilled shafts, limited soil improvement is proposed according to the Burnside Bridge Geotechnical Report (Appendix C); this would minimize the environmental impact to the river.

9.4.14 Mechanical and electrical Equipment Replacement

9.4.14.1 Mechanical Equipment Replacement

The mechanical rehabilitation includes a full replacement of the entire movable span operating machinery up to, but not including, the racks mounted on the bascule girders. The same basic machinery layout would be maintained, but all open gearing sets would be replaced with enclosed gearing. The system would maintain two drive motors with motor brakes. These would drive a single differential gearbox. The differential gearbox would drive two output shafts with machinery brakes mounted along their lengths. These cross shafts would each drive a second gearbox at the north and south ends of the span. This gearbox would be coupled to the final new rack pinion at each of the two existing racks. The operating machinery arrangement for the widened span would be of similar arrangement but with a wider machinery layout with new rack pinions which drive off of new racks mounted to the additional bascule girders of the widened portion of the leaves.

The center span lock machinery would be of the same arrangement as the existing span lock machinery, with the exception that in the widened alternative, the locks would be located in the toe of the new, additional bascule girders.

9.4.14.2 Electrical Equipment Replacement

The electrical rehabilitation includes components that would be replaced due to the increased weight of the movable span leaves, which would result in higher amperage-rated equipment being installed. Incoming power distribution and span operation motor and drive system infrastructure would be replaced, which includes the manual transfer switches, generator receptacles, disconnect switches, circuit breakers, span operation motors, and span operation motor drives. The increased amperage capacity of this equipment would necessitate larger wiring and conduits to be installed with these pieces of equipment.

The bridge operation control system would also be replaced. The bridge operation control system consists of a PLC system, HMI touchscreen, a UPS, an Ethernet network switch, and associated power, control, and communications cables and conduit.

The power and control feeds to the center span lock equipment would also be replaced to support the center span lock equipment replacement.

Additional equipment would be replaced or relocated within Piers 2 and 3 based on structural impacts to equipment areas and widening efforts. These items include the navigation lights and traffic warning gates for marine and vehicular traffic on the exterior of the piers, as well as power distribution equipment in the form of panelboards, MCC, and transformers within the piers. All of these items would require replacement of the associated wiring and conduits.

9.4.14.3 Emergency Winch System

In addition to the above mechanical and electrical work, an additional emergency span operation system would be installed. This system would include an industrial winch which connects to a reinforced point on the counterweight support truss. Powered by a generator, as described below, the winch would pull the counterweight truss down and open the span. During rebalancing during the mechanical rehabilitation, the span would be balanced such that the weight of the leaf is able to overcome friction resistance in the trunnion and span machinery allowing the span to close in a controlled manner as the winch is unspooled.

To support the emergency winch system, a manual transfer switch and generator receptacle would be installed on each bascule pier to provide emergency power after a seismic event. The seismic event may damage the incoming power feed electrical infrastructure along the fixed spans. Locating the backup power equipment near the bascule span would reduce the potential of such damage from preventing operation of the bascule spans. Additional modifications to support the emergency winch include installing an independent circuit breaker and MCC, in the vicinity of the winch, for quick connection after the seismic event. The routing of the conduit from the generator plug and the location of the circuit breaker and MCC could be optimized in design to reduced seismic vulnerability. All controls to operate the winch would be provided in the winch package, so no additional control equipment would be required.

9.5 Bascule Span Widening

In general, widening the bridge will provide additional load paths and strengthen the existing structure, which are advantages to the bridge seismic retrofit. Some of the widening-related costs could be offset by reduced retrofit cost on the existing structures, for example, pier foundation enlargement is required, whether it is for the retrofit only or for the retrofit and widening alternative.

9.5.1 Pier Foundation Enlargements

For the seismic retrofit combined with widening alternative, the same concept of foundation enlargement as described in previous retrofit concept section applies. New shafts should be drilled around the existing pier caps to the vertical and lateral load-carrying capacities. An enlarged pile cap integrates the new shafts with the existing pile caps. The number of new shafts required is determined by the dynamic model analysis that includes the mass of the widened structure. The size of the pile cap shall also the widened accordingly.

9.5.2 Pier Widening

For the seismic retrofit combined with widening alternative, the widened structure can be constructed on top of the enlarged pile cap. The widened pier walls designed according to current seismic design code requirements would replace the existing pier walls. A widened section through the bascule pier is shown in Figure 9-24.

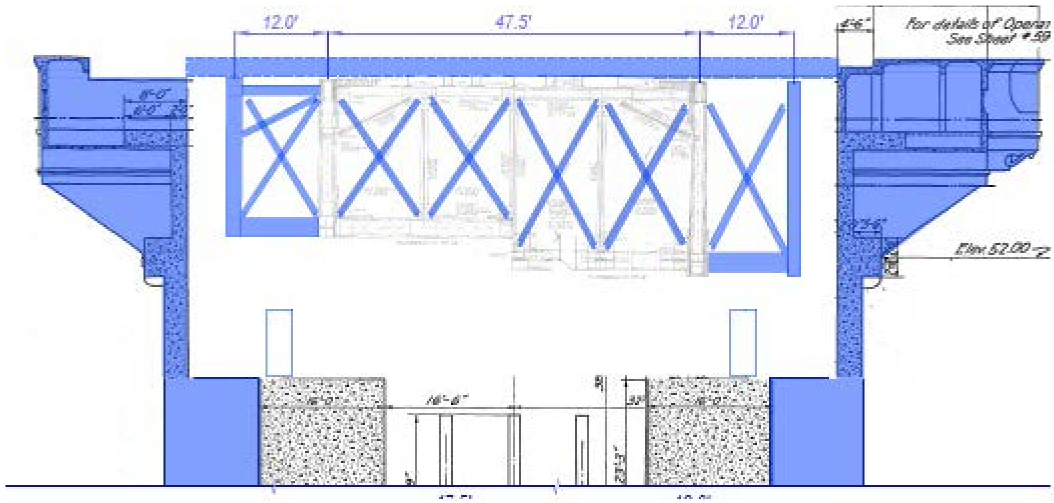


Figure 9-24. A Widened Bascule Pier Section

The widened pier could have more space for constructing the corner columns, which further improves the lateral load resistance capability of the pier.

The concrete pedestal under the trunnion support frame would also be widened, either with reinforced concrete or with steel frames, because of the need for supporting an additional pair of drive gears and pinions (see Figure 9-25).

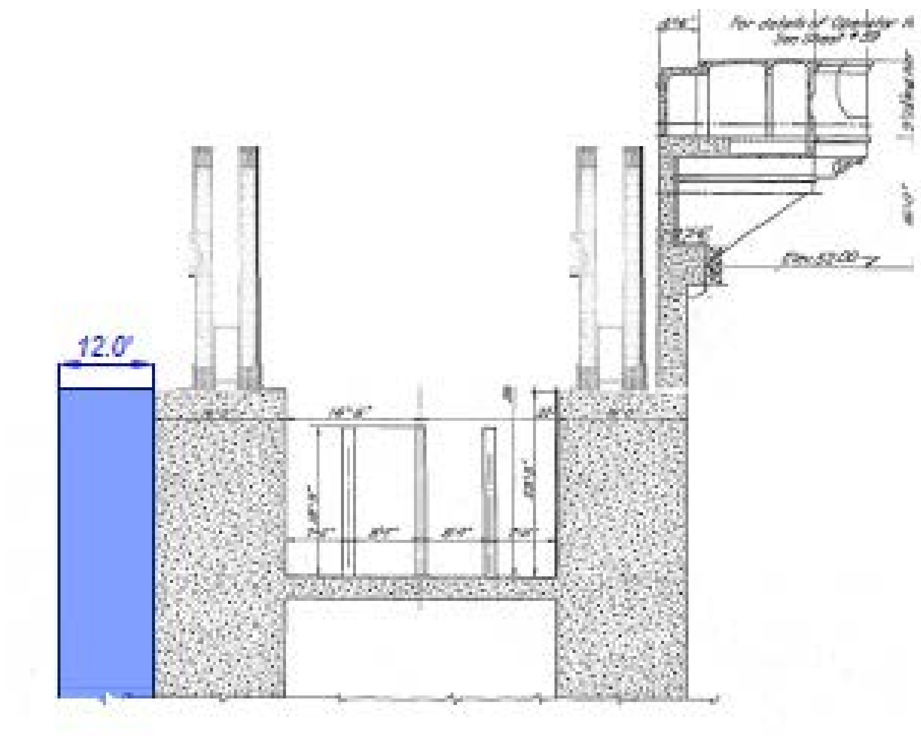


Figure 9-25. Widened Trunnion Frame Support

9.5.3 Second Pair of Bascule Leaf Trunnion Supports

A second pair of gears and pinions is needed for driving the two additional bascule leaf trusses that are outside the existing leaf truss. This second pair of leaf trusses supports the widened portion of the bascule span deck (Figure 9-26).

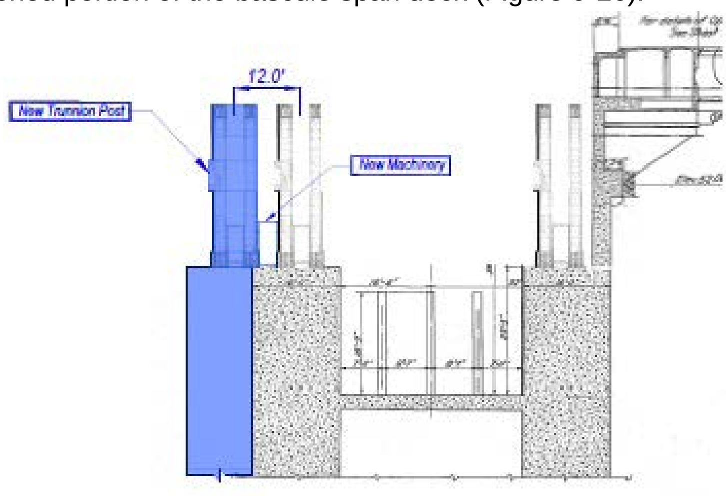


Figure 9-26. Second Pair of Trunnion Support Frames

9.5.4 Counterweight Support Widening

For the seismic retrofit and widening alternative, to balance the additional weight of the widened bascule leaf and the deck, the counterweight would also be widened to add more balancing weight (Figure 9-27), then consequently, the support frame member would also be further strengthened.

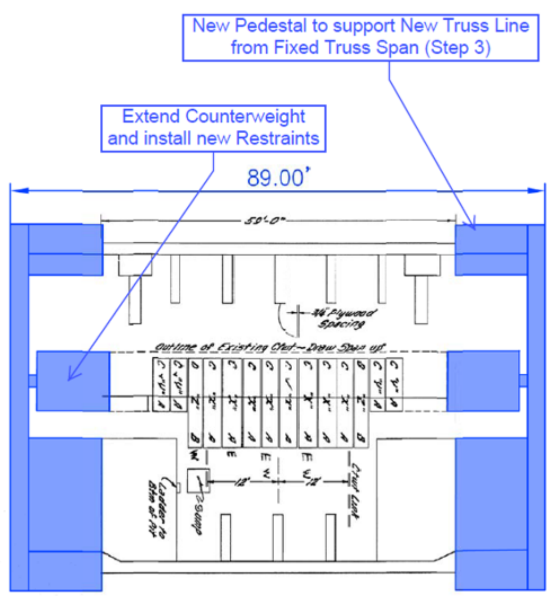


Figure 9-27. Counterweight Widening

9.5.5 Adding Second Pair of Bascule Leaf Trusses

The proposed bridge deck widening is 12 feet on each side. Analysis indicated that strengthening the existing steel bascule truss won't be able to support the added weight of the total 24 feet of widening. This 24-foot widening requires adding a steel truss panel under each of the widened decks (Figure 9-28). Therefore, the widened bascule span would be 110 feet wide, with four steel truss panels for each leaf.

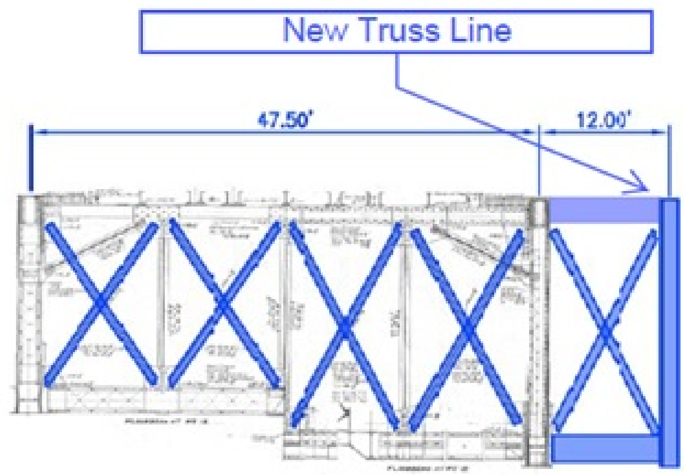


Figure 9-28. Adding Trusses for Bascule Leaf

9.6 Construction Sequence

One of the major considerations when evaluating bridge seismic retrofit versus replacement is the impact on the traffic above the bridge deck and below the bridge spans, and/or on marine traffic.

One of the potential advantages of bridge seismic retrofit, compared to replacement, is that the retrofit alternative may permit keeping travel lanes partially open on the bridge during construction. Our proposed seismic retrofit strategies consider maximizing this potential benefit.

The proposed construction sequence for both the retrofit-only alternative and the retrofit and widening alternative is the same:

- Work on one bascule leaf at a time, so the other leaf is operable and can open for vessel traffic
- When working on one leaf, only remove half of the deck while the other half of the deck remains open for traffic.

A suggested construction sequence is marked in Figure 9-29 as an example of keeping traffic open, both on the bridge deck and below the bridge deck for marine traffic. This phased construction sequence would increase construction time and costs.

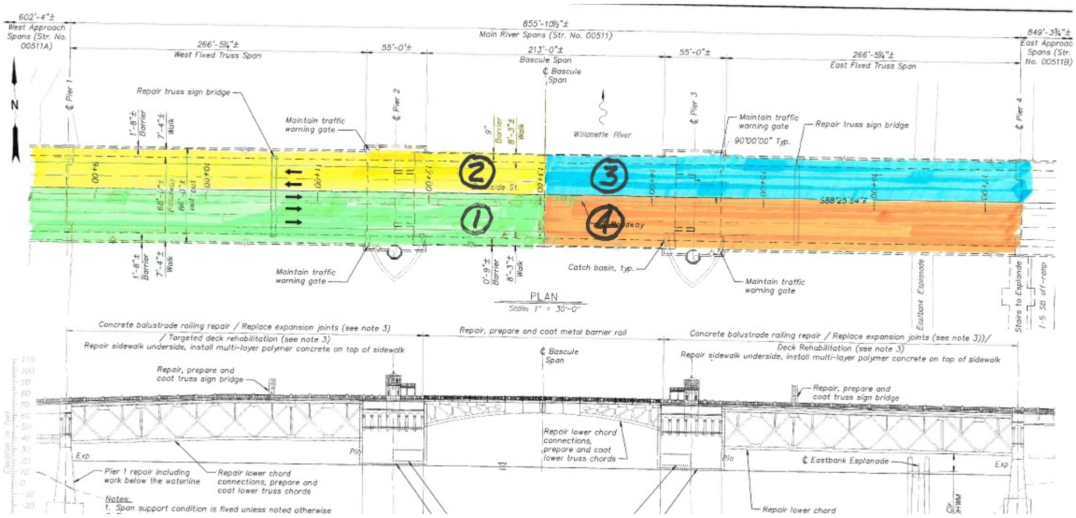


Figure 9-29. Construction Staging for Keeping Traffic Open

9.7 Elimination of Legal and Permit Load Rating Deficiencies

Currently, there are elements of the Burnside Bridge that have load rating factors less than 1.0 for legal, special haul, and permit vehicles. These elements include many RCDG bent crossbeams, concrete girders and stringers, concrete encased girders, and steel stringers in the fixed truss spans. It is assumed that as part of any future seismic retrofit or widening project these deficient elements would be strengthened to achieve a rating factor greater than 1.0 for the legal, special haul, and permit vehicles. For any element requiring work for both seismic retrofit and elimination of load posting, costs have been captured as part of the seismic retrofit work items (for example, the RCDG crossbeams).

Strengthening the structure to achieve a rating factor greater than 1.0 for all legal, special haul, and permit vehicles will allow heavy equipment for debris management to be transported over the bridge on multiple axle truck and low-boy trailers in the event of a catastrophic seismic event as needed.

At many locations throughout the structure, the rating factor for the longitudinal reinforcement check in concrete stringers is less than 1.0. Based on guidance in the ODOT LRFR Manual Sec. 2.4.6, strengthening of elements to eliminate this deficiency is not strictly required unless the deterioration of the bridge warrants strengthening or if the coincident shear rating factor is less than 1.0. For this structure, at no location is the coincident shear rating factor less than 1.0. For the purposes of cost estimating, it was assumed that if the condition rating was expected to deteriorate to a National Bridge Inventory (NBI) rating of 3 within the next 20 years, strengthening would be required. Based on the current NBI rating of the superstructure of 5, deterioration to a rating level of 3 is not expected. Thus, strengthening to eliminate the longitudinal reinforcement check deficiency is not needed.

9.8 Pier Fender Replacements

The existing bascule pier fender systems would be removed during the construction of the drilled shafts and the enlarged pile caps. Therefore, the fenders would be replaced for both retrofit only or retrofit and widening alternatives.

The new fender system consists of large-diameter drilled shafts extended to the water surface. Each bascule pier is protected from vessel collision by two large shafts on the upstream and downstream sides. There would be a total of eight drilled shafts to protect the two bascule piers.

9.9 Impacts

9.9.1 Aesthetics

Bridge aesthetics usually get input from the public and stakeholders, thus the bridge visual appearance shall be further investigated.

From practical engineering and cost points of view:

- Seismic retrofit without widening would be more cost effective if the retrofitted structure maintains similar appearance to the existing structure.
- Seismic retrofit and widening could provide more options for different visual appearances.

9.9.2 Maintenance of Bridge and River Traffic

The proposed retrofit and widening strategies would keep the Burnside Bridge partially open for traffic during construction, except for short-duration closures during the entire span replacement, such as east approach Span 20 to 24 replacement.

Work on the bascule leaves would be on one leaf at a time, so the other leaf could be operated to open and close for marine traffic if needed.

9.9.3 Parks Facilities

9.9.3.1 Waterfront Park Seawall

The widening of Pier 1 on the west bank of the river would require the partial removal of the adjacent seawall. The multi-use trail that runs through Waterfront Park between Bent 19 and Pier 1 would need to be temporarily closed or rerouted during construction. Seismic mitigation for lateral spreading would require ground improvements between Bent 19 and Pier 1, also impacting the seawall and trail.

9.9.3.2 Ankeny Pump Station

Retrofit construction at Bent 18, Bent 19, and Pier 1 would impact operation of the Ankeny Street Pump Station. The PGE vault between Bent 18 and Bent 19 would need to be relocated to accommodate widening of the pile caps. The duct bank providing power to the pump station from the PGE vault would also need to be relocated to accommodate Bent 19 pile cap widening. The above grade power transformers switchgear at the north end of the pump station may also need to be relocated to accommodate north-south widening of Pier 1.

Widening of Pier 1 would require partial or complete removal of the seawall, impacting the 30-inch and 42-inch sewer force mains that are attached to the side of the seawall. A temporary bypass or permanent relocation would be required to keep the pump station in service during construction. Additionally, the 24-inch foul air supply to the odor control vaults on the north side of the bridge may need to be relocated, depending on the extent of the impacts to the seawall.

9.9.4 Eastbank Esplanade

Retrofit construction at Pier 4 on the east side of the river would either consist of modifications to the existing pier or construction of a new pier immediately to the west of the existing. Under either scenario, a partial cofferdam and potential work platform would be required. Due to the limited bank space and close proximity of I-5 on the east side of Pier 4, construction equipment and material access would be primarily from the river. In order to gain access, the Eastbank Esplanade multiuse trail would need to be closed and floating portions of the trail temporarily removed to facilitate access.

9.9.5 Business Operations and Facilities

Proposed seismic retrofit and widening alternatives have taken into consideration minimizing the impacts on adjacent or nearby business operations and facilities, such as UPRR.

Despite the efforts, some impacts may inevitably still exist, for example, to some adjacent buildings or to those attached to the Burnside Bridge. Activities associated with seismic retrofit of the bridge approaches would have to be coordinated with those building owners or tenants.

9.9.6 Right-of-way

Other than for temporary construction staging and access, the seismic retrofit would require no additional right-of-way.

For the seismic retrofit and widening alternative, since the widening is to match the 110-foot deck width on the west and east approaches, additional right-of-way requirements would be minimal.

10 Summary of Retrofit and Widening

10.1 Design Alternatives

Chapter 8 and 9 describe eight of the bridge seismic retrofit and widening alternatives. Four of these alternatives are for seismic retrofit of the existing bridge only, they are Alternatives 4a.1, 4b.1, 4c.1, and 4d.1. The other four alternatives are for the bridge seismic retrofit combined with widening, they are Alternatives 4a.2, 4b.2, 4c.2, and 4d.2.

The primary features of four representative alternatives, including the pros and cons, are summarized in the table below. The costs listed in the table are estimated construction costs only that do not include such as right-of-way, engineering, etc.

Table 10-1. Summary of Representative Alternatives for Seismic Retrofit and Widening

Alternative	Description	Primary Advantages	Primary Disadvantages	Project cost ($million)
1 Seismic Retrofit	Pure seismic retrofit of all bridge members (0% bridge replacement)	N/A	Not feasible because it requires the prolonged removal of multiple I-5 mainline and ramp bridges to construct the retrofit.	N/A – Not Feasible
4a.1 Hybrid (Replace / Retrofit) Seismic Retrofit	Includes replacing all highway spans (20 to 23) (10% bridge replacement)	Shortest of all Hybrid alternatives	Versus Alt 4b.1, not reasonable because it requires a very high-cost, high-risk railroad shoofly within the UPRR. It may not be permittable.	N/A – Not Reasonable versus Alt 4b.1
4b.1a Hybrid (Replace / Retrofit) Seismic Retrofit	Includes replacing all highway and RR spans (20 to 24) (13% bridge replacement)	Lowest estimated cost among the four alternatives. By replacing Spans 20 to 24, the constructability challenges at Bents 21, 22, 23, and 24 are minimized because of the close proximity to the highway structures and UPRR tracks.	A narrow retrofitted bridge (86 feet wide)	$ 688
4b.2a Hybrid Retrofit with Widening	Includes replacing all highway and RR spans (20 to 24) (18% bridge replacement)	In addition to above for Alternative 4b.1a, this will be widened to a constant 110-foot bridge.	Highest estimated cost amongst alternatives 4b.1 through 4c.1	$ 844

Table 10-1. Summary of Representative Alternatives for Seismic Retrofit and Widening

Alternative	Description	Primary Advantages	Primary Disadvantages	Project cost ($million)
4c.2a Hybrid (Replace / Retrofit) with Widening	Includes replacing East Approach spans (20 to 27) (30% bridge replacement)	In addition to above for Alternative 4b.2, by replacing additional spans (from Span 20 to 27), the retrofitted and widened bridge will have more spans with new structures and foundations without a cost increase due to the anticipated cost savings from a simplified construction sequence.	Constructability challenges due to the proximity of existing buildings along the East Approach ROW limits	$1.01B
4d.2 Hybrid (Replace / Retrofit) with Widening	Includes replacing Main and East Approach spans (14 to 27) (67% bridge replacement)	In addition to above for Alternative 4c.2a, by replacing the existing structures from Span 14 to Span 27, over the anticipated soil liquefaction area, this retrofitted and widened structure has the most resiliency after a design-level earthquake.	Leaves short portions of the existing approach bridges, requiring additional costs to maintain; Could be the highest cost of the four Hybrid alternatives investigated in detail.	N/A – Not Reasonable versus Alt 4c.2a

10.2 Major Work Items

Seismic retrofits and widening related major structural, mechanical, and electrical work elements are listed below.

Table 10-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit and Widening
Structural			
Bent 1	Abut. Strengthening		Same
Bents 2-16	Girder Restrainers and Strengthening		Same
	Floor Beam Strengthening		Same
	Column Jacketing		Same
	Footing Enlargement		Same
Bents 17-19	Girder Restrainers and Strengthening	+	Add Girders
	Floor Beam Strengthening	+	Widen
	Column Jacketing	+	Add New Columns

Table 10-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit and Widening
	Footing Enlargement	+	Add Grade Beam
	8-foot Dia Drilled Shafts	+	Deeper Shafts
Pier 1	Relocation of Force Mains		Same
	6-foot Dia Shafts	+	Add Shafts
	Pile Cap Enlargement	+	Wider Cap
	Harbor Wall Reconstruction		Same
	Pier Column Strengthening	+	Wider Column
	Bearing Replacement	+	Add Bearings
West Truss Span	Lateral Load Member Strengthening	+	Add Two Trusses
	Connection Retrofit		Same
Pier 2	10-foot Dia Drilled Shafts	+	Add Shafts
	Pile Cap Enlargement	+	Wider Cap
	Adding Corner Columns	+	Larger Corner Columns
	Pier Wall and House Strengthening	+	Replace Existing Pier Walls
	Support Pedestal Strengthening	+	Wider Pedestal
	Pit Deck Bearing Retrofit	+	Add Bearings
	Trunnion Frame Strengthening	+	Add 2nd Pair Frames
	Trunnion Frame Anchorage Strengthening	+	More Anchorage
	Counterweight Frame Strengthening	+	Widen Counterweight
	Install Lateral Restrainers		Same
	Live Load Shoe Retrofit	+	Add Two Live Load Shoes
Pier 3	10-foot Dia Drilled Shafts	+	Add Shafts
	Pile Cap Enlargement	+	Wider Cap
	Adding Corner Columns	+	Larger Corner Columns
	Pier Wall & House Strengthening	+	Replace Existing Pier Walls
	Support Pedestal Strengthening	+	Wider Pedestal
	Pit Deck Bearing Retrofit	+	Add Bearings
	Trunnion Frame Strengthening	+	Add 2nd Pair Frames
	Trunnion Frame Anchorage Strengthening	+	More Anchorage
	Counterweight Frame Strengthening	+	Widen Counterweight
	Install Lateral Restrainers	+	Same
	Live Load Shoe Retrofit	+	Add Two Live Load Shoes
Bascule Leaves	Lateral Load Member Strengthening	+	Add Two Trusses

Table 10-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit and Widening
	Connection Retrofit		Same
	Center Lock Retrofit		Replace Center Lock
East Truss Span	Lateral Load Member Strengthening	+	Add Two Trusses
	Connection Retrofit		Same
Pier 4	6-foot Dia Shafts	+	Add Shafts
	Micropiles	+	Add Micropiles
	Pile Cap Enlargement	+	Wider Cap
	Pier Column Strengthening	+	Wider Column
	Bearing Replacement	+	Add Bearings
Spans 20-24	Replace with three New Spans	+	Replace w Wider Spans
Bents 25-28	10-foot Dia Drilled Shafts	+	Deeper Shafts
	Pile Cap and Grade Bean Extension	+	Larger Caps
	Partial Infill Wall		Same
	Column Strengthening	+	Add Columns
	Floor Beam Strengthening	+	Widen
	Bearing Replacement	+	Add Bearings
	Steel Girder Strengthening	+	Add Girders
Bents 29-34	Girder Restrainers and Strengthening		Same
	Floor Beam Strengthening		Same
	Column Jacketing		Same
	Footing Enlargement		Same
Bent 35	Abut. Strengthening		Same
Mechanical and Electrical			
Bascule Span	Operating Machinery Replacement	+	Add Machinery for Widen
	Rehabilitation of Trunnions and links	+	Add Trunnions
	Span Balance Work		Same
	Replace incoming electrical service		Same
	Center span lock power feed		Same
	Replace motors and drives	+	Add Motors and Drives
	Relocate and update PLCs		Same
	Replace navigation lighting		Same
	Replace traffic warning gates		Same
	Relocating electrical equipment		Same

Table 10-2. Summary of Major Work Elements for Retrofit and Widening

Location	Retrofit		Retrofit and Widening
Geotechnical Mitigation			
Under West Approach	Ground Improvement for liquefaction mitigation		
Under East Approach	Ground Improvement for liquefaction mitigation		

Note: Utilities, traffic control, etc., are not listed.

The above table is not to be used as a list of the complete work elements. For example, potential utility relocations, maintenance of traffic during constructions, site preparations, construction access and staging areas, etc. are not included in the list.

10.3 Seismic Retrofit vs. Widening

Based on our study, combining the Phase 2 seismic retrofit with a bridge widening provides the following benefits:

- Function-wise, widening the middle section of the bridge from 86 feet to 110 feet would give this bridge a constant width of 110 feet wide that is better for the traffic flow.
- The cost increase in foundations is minimal since both the retrofit only and with the widening alternatives require enlarged foundations.
- Widened bents and piers would strengthen the existing portions of the structure, thus enhancing the structure's seismic resilience.

10.4 Constructability

because the Burnside Bridge is located in a highly congested downtown area, there are several major constructability challenges, such as:

- Buildings attached to or in close proximity on either side of the bridge approach spans
- MAX lines, Waterfront Park Trail, and roadways under the west approach
- Major utility lines and a pump station under the west approach
- A seawall that needs to be temporarily relocated for access to the Pier 1 foundation
- One of the bascule span leaves should remain operable during the construction
- In-water construction activities at the bascule piers need to keep navigational channel open
- I-5 ramps over the Pier 4 foundation blocks construction access

- I-5 main lines and a ramp to I-84 under east approach spans need to remain open during construction
- Union Pacific Railroad (UPRR) main lines under east approach spans

Because of the anticipated challenges for shutting down the I-5 for extended period, and the for construction access near the UPRR tracks, it has been identified that replacing the span 20 to 24 of the east approach with a longer span structure could be more cost effective than retrofitting the existing piers and foundations.

10.5 Concept Retrofit and Widening Plans

Proposed concepts for Burnside Bridge Seismic Retrofits and Widening are attached in Appendix E.

11 Conceptual Cost Estimates for all Alternatives

11.1 Cost Estimate Methodologies

This section provides the approach, assumptions, and process used to generate and assemble the various costs that constitute the project costs for the alternatives of the project. They are appropriate for a feasibility-level of design, and were developed in conjunction with and based on input from Multnomah County Bridge Division. Detailed total project cost estimates can be found in Appendix F.

11.2 Cost Estimate Methodologies

The cost estimate methodology is based on four key estimating sources:

1. The Programmatic Cost Memorandum developed for the Willamette River Bridge CIP (Bridge CIP) Project, in May 2014.
2. Average historical unit bid prices for similar work elements from relevant ODOT bridge cost data, WSDOT bridge cost data, or similar projects constructed in the northwest (for estimated work items for which there is a suitable data source to draw from).
3. Average historical unit bid prices for similar work elements from relevant projects constructed outside of the northwest (for unique items such as movable bridge components) for which there are little cost data to draw from.
4. When pricing from similar projects or work elements was not available or incomplete, engineering judgment was used to develop the costs.

The project cost estimates consider the complexity, nature of the work, and the difficulties with working within a dense urban environment with large amounts of anticipated public accommodation during construction.

11.3 Cost Categories

Cost estimates were compiled based on a combination of four categories:

1. Construction cost
2. Right-of-way cost
3. Engineering and project delivery cost
4. Inflation cost

When added together, they form the total project cost for the alternative.

11.3.1 Raw Construction Cost

11.3.1.1 Construction Cost Methodology

Construction costs are the total of all work items necessary to construct the project. They are based on 2017 construction dollars and do not include any magnification factors for inflation or engineering and project delivery (i.e., PE and CEI / CA).

Key assumptions or methods used to calculate the Constructed Value (CV) are briefly described below:

- Because the project is in an early conceptual stage, only major quantities of work were calculated.
- Unit costs were modified to account for difficulty of work and/or access, such as the site restrictions due to I-5 structures and UPRR tracks.
- Unit costs were magnified to account for difficulty of some unique work elements in this project, such as retrofitting the bascule span while maintaining traffic flow.
- Construction access and temporary work such as work bridges were calculated separately from the widening construction cost estimates to avoid double-counting those items when combining widening construction costs with seismic retrofit costs. The need and cost for construction access and temporary works was determined for each combination of widening and seismic retrofit considered and added as a line item in the project cost estimate.
- Unit cost were magnified to account for identified regulatory and permitting work requirements or constraints, such as in-water work and maintaining the navigation channel during construction.
- Unit costs considered the complexity, nature of the work, and the difficulties with working within a dense urban environment with large amounts of anticipated public accommodation during construction.

Raw construction costs were developed and organized into construction item work for which quantities could be estimated, unit costs assigned, and then multiplied together to yield a raw construction cost for that item of work. All raw construction cost items were then summed together to form the cumulative CV per component.

11.3.1.2 Construction Cost Components

The construction cost (or CV) components consist of the following:

- **Preparation** – Work associated with mobilizing and preparing the site for construction, includes the following subcomponents:
    - o Mobilization – An assumed administrative cost for the construction contractor to mobilize to the site. It was assigned a value of 10 percent of the construction cost of the project (exclusive of site preparation and contingencies) based on common industry practice for a project of this magnitude.

- Temporary Erosion and Sediment Control – Cost to implement best management practices to control stormwater debris flows. It was assigned a value of 0.5 percent of the raw construction cost of the project (excluding site preparation and contingencies) based on the project's proximity to the Willamette River.
- Temporary Protection and Direction of Traffic (TP&DT) – Cost to place temporary traffic features, such as temporary barriers and signage, to control traffic during construction on the bridge and adjacent roadways. This does not, however, include temporary work facilities such as staging/shoring, temporary roadways/bicycle/pedestrian facilities, nor temporary bridges required during construction. It was assigned a value of 4 percent of the raw construction cost of the project (excluding site preparation and contingencies) based on common industry practice for a project of this magnitude.
- Removal of Structures and Obstructions – Cost to remove any conflicting structural elements to facilitate the building of the alternative, such as the adjacent seawall near Pier 1. It also includes the removal of any existing Burnside Bridge elements that are being replaced. The cost was directly calculated based on the alternative under consideration.

- **Civil / Roadwork** – Work associated with constructing the bridge's approach roadways, roadways beneath the bridge, or facilities adjacent to the bridge. It includes the following subcomponents:
  - Roadway Surface/Earthwork – Cost to reconstruct portions of the roadway affected by the structural retrofit work. It includes impacts to Burnside Street, streets crossing underneath the bridge, and any adjacent parking lots. The cost was directly calculated based on the alternative under consideration.
  - Traffic Signals/Illumination – Cost to modify and/or replace existing traffic signals and illumination not mounted to the bridge. It was assigned a value of 1 percent of the raw structural construction costs of the project (excluding site preparation and contingencies).
  - Stormwater, Drainage, and Planting – Cost to modify and/or replace existing stormwater facilities, drainage features, and landscaping, including new facilities required by additional impervious areas. It does not include hardscape features such as bike paths or other amenities which are included in site restoration. It was assigned a value of 2 percent of the raw structural construction costs of the project (excluding site preparation and contingencies).
  - Site Restoration – Cost to re-establish the site to its permanent condition (finish grades, irrigation, landscaping, etc.) after construction of the other infrastructure elements. It was assigned a value of 2 percent of the raw construction cost of the project (excluding site preparation and contingencies).
  - Retaining Walls – Cost to modify, replace, or construct new retaining walls necessary for the project, some of which are located under Burnside Street on the west approach. It does not include the reconstruction of the Burnside Skatepark. The cost was directly calculated based on the alternative under consideration.

  - Utilities – Cost to modify, replace, or construct new utility facilities that are the County's responsibility to maintain (i.e., for which the utility has "prior rights") as part of an existing inter-governmental agreement or permit. The cost was directly calculated based on the alternative under consideration. Based on information provided by the County, it is assumed that the County and City have prior rights designation over all utility agencies impacted. As such, utility costs were assumed to be low, and most costs associated with utility relocations were assumed to be financed by the utility owner. In the event that County-owned or City-owned water, sewer, electrical, or other similar facilities are impacted or needed as part of the project, costs for their reconstruction were assigned a value of 2 percent of the raw structural construction costs (excluding site preparation and contingencies).

- **Bridge Structure Retrofit** – Work associated with constructing the retrofit measures excluding the mechanical and electrical work and geotechnical hazard mitigation costs, which are calculated as separate components. Bridge widening costs were developed based on a build-up of structural elements required. Bridge replacement costs, when applicable, were developed on a per-square-foot basis. Aesthetic costs were included in the estimate by increasing the structural unit costs to which they apply. In general, this consists of adhering to the existing architectural forms for the piers. The bridge structure retrofit component includes the following subcomponents:
  - West River (00511A) – Cost to construct the retrofit measures from Abutment 1 to Pier 19 for the alternative being considered. (Note: Pier 1 costs are included in the Main River Span subcomponent). For each bent/pier or span location, retrofit measures were developed and construction quantities were calculated. Unit costs for the retrofit measures were then developed and applied. This includes quantities for new members associated with widened or replacement spans, columns, and foundations.
  - Main River Spans (00511) – Cost to construct the retrofit measures from Pier 1 to Pier 19 for the alternative being considered. For each pier or span location, retrofit measures were developed and construction quantities were calculated. Unit costs for the retrofit measures were then developed and applied. This includes quantities for new members associated with widened or replacement spans, columns, and foundations.
  - East River (00511B) – Cost to construct the retrofit measures from Pier 4 to Abutment 35 for the alternative being considered (Note: Pier 4 costs are included in the Main River Span subcomponent). For each bent/pier or span location, retrofit measures were developed and construction quantities were calculated. Unit costs for the retrofit measures were then developed and applied. This includes quantities for new members associated with widened or replacement spans, columns, and foundations.

- **Movable Span Mechanical and Electrical** – Work associated with constructing the retrofit measures excluding the mechanical and electrical work and geotechnical hazard mitigation costs, which are calculated as separate components. It includes the following subcomponents:

- Mechanical System – Cost to reconstruct or strengthen the mechanical systems impacted by the retrofit throughout the Main River spans. It includes the following components: operating machinery replacement; rehabilitation of trunnions, counterweight trunnions, and links; additional trunnions and counterweight trunnions for widening; and span balance work. For each item, measures were developed and construction quantities were calculated. Unit cost measures were then developed and applied. For widening options, additional elements were assumed when applicable.
- Electrical System – Cost to reconstruct or strengthen the electrical systems impacted by the retrofit throughout the Main River spans. It includes the following components: replace incoming electrical service from east and west; replace center span lock power feed; replace motors and drives; relocate and update PLCs (programming, start-up and commissioning); replace navigation lighting (pier and span); replace traffic warning gates; and relocate electrical equipment (MCCs, panelboards, and networking equipment). For each item, measures were developed and construction quantities were calculated. Unit costs measures were then developed and applied. For widening options, additional elements were assumed when applicable.
- Emergency Backup System – Cost to construct a hand-operated movable bridge system to enable the bridge to have movable operations within 2 weeks of the CSZ earthquake.

- **Geotechnical Hazard Mitigation –** Work associated with constructing the liquefaction and lateral spreading measures for the west and east approaches. For the Main River Spans, no mitigation measures are included because the retrofitted pile foundations were designed to withstand any liquefaction settlement, and the lateral spreading mitigation was incorporated into the cost estimates for the east and west approaches. It includes the following subcomponents:
  - East Approach – Cost to construct lateral spreading mitigation using two underground cellular-soil cement wedges: a primary one wedge Pier 4 along the riverbank, and a secondary wedge near Bent 23. Two smaller wedges were used because the size and length of a single wedge resulted in a much higher cost. For each wedge, construction quantities were calculated, and unit costs were then developed and applied. Because deep foundations were incorporated as part of the retrofit for areas where liquefactions is anticipated, no geotechnical settlement mitigation was required.
  - West Approach – Cost to construct lateral spreading mitigation using one underground cellular-soil cement wedge near Pier 1 along the riverbank, and localized liquefaction settlement mitigation for Bents 1 through 16 using a smaller form of the cellular-soil cement mixture. For each mitigation type, construction quantities were calculated, and unit costs were then developed and applied.
- **Other Related Items –** Work associated with constructing other features in conjunction with the seismic retrofit work. These features were either identified during the Bridge CIP Project or in response to the Seismic Design Criteria. It includes the following subcomponents:

  - Electrical and Lighting – BUN-MU-01 – Submarine Cable Removal – As identified in the Bridge CIP, this is the cost for underwater demolition of the cables underneath the movable section of the bridge. The Burnside Bridge's portion of the construction cost was extracted from the Bridge CIP and applied.
  - Structural – Elimination of Load Rating Deficiencies – Cost to strengthen the bridge for ODOT permit vehicles which are expected to haul recovery equipment/materials and debris across the bridge after the earthquake. At present, the bridge is structurally load posted to prevent heavy vehicles from using the bridge.
  - Structural – BUN-MU-02 – Scour Remediation – As identified in the Bridge CIP, this is the cost to place additional rock armor at the in-water piers. The Burnside Bridge's portion of the construction cost was extracted from the Bridge CIP and applied.
  - Structural – BUN-MU-03 – Fender Repair – As identified in the Bridge CIP, this is the cost to construct a new fender system appropriate for the site and consistent with the retrofitted bridge pier widths. For estimating purposes, construction quantities were calculated, and unit costs were then developed and applied.
- **Uncertainties Contingency Factor** – A construction cost magnification factor was applied to account for material quantity and cost uncertainties; changes because of the limited level of design conducted to date; unknown environmental or permitting mitigation that might be required; and other incidental project costs. For bridge feasibility studies, a typical contingency factor ranges from 25 to 45 percent. Because quantities were developed and many costs have been established as identified line-items, a contingency factor of 30 percent was implemented. The contingency value was applied to the sum of all construction cost items identified above for each alternative.

11.3.2 Right-of-way

Right-of-way costs are costs borne by the project because of impacts to properties, or property rights, owned by others. Because the bridge hybrid options do not extend beyond the existing bridge width, and because the County owns the land beneath the bridge, no permanent acquisition costs were assumed. Temporary construction easement (TCE) costs, however, have been incorporated into the estimate because adjacent parcels will likely be impacted during the construction of the seismic retrofit and widening. Land in some parcels will be required for construction access and staging, and other parcels will have existing accesses limited by construction activities.

Sixteen parcels owned by private property owners were identified as having potential TCE impacts. Based on an assumed duration of construction impacts, an estimate of damages and relocation benefits was developed for each parcel. Acquisition costs including personnel, legal, and contingencies were also estimated and are included in the overall right-of-way cost.

11.3.3 Engineering and Project Delivery

Engineering and project delivery are administrative costs that include a combination of the following four categories:

1. National Environmental Protection Act (NEPA) Phase – Costs include the necessary effort to develop an approved environmental clearance for the project. Due to the project impacts, it was assumed that an Environmental Impact Statement (EIS) through the Federal Highway Administration would be required for the project. Additionally, it was assumed that the NEPA phase cost for all Hybrid alternatives would be the same as if the bridge was replaced, or $17M. This value was determined during the Bridge CIP project.
2. Preliminary Engineering (PE) Phase – Costs include the necessary effort to develop preliminary and final Plans, Specifications, and Estimate (PS&E) for bidding the project. It was assumed that the project would be designed and constructed using the conventional design-bid-build delivery model. For bridge feasibility studies, a typical PE factor is commonly applied to the construction cost and ranges from 6 to 25 percent. For this project, because of its scale, a factor of 15 percent was used.
3. Construction Engineering & Inspection / Construction Administration (CEI / CA) Phase – Costs include all project costs for overseeing the construction phase, including construction administration, engineering support, responding to contractor inquiries, construction inspection, and coordinating with the public. CEI / CA costs were established as a percentage of construction costs inclusive of utilities, mobilization, temporary traffic control, contingency, and escalation. For bridge feasibility studies, a typical CEI / CA factor is commonly applied to the construction cost and ranges from 6 to 20 percent. For this Project, because of its scale and complexity, a factor of 20 percent was used.
4. County Administration – Cost represents the cost for the County to oversee and administer the Project. This cost was assumed as part of the PE Phase cost.

11.3.4 Inflation

The future cost inflation factor used was based on a WSDOT-projected inflation factor from Connecting Washington Bid Environment presented to the Joint Transportation Committee on July 20, 2017 (WSDOT 2017), and compared with recent ODOT escalation forecasts. Based on these sources, a 3.0 percent per year inflationary rate was used to escalate design and construction costs from 2017 dollars to the mid-point of construction, assumed to be in 2027. A 5.0 percent per year inflationary rate was used to escalate right-of-way costs over a 6-year period.

12 References

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2014 Oregon Highways Seismic Plus Report, Bridge and Geo-Environmental Sections, Technical Services Branch. October 2014

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2013 Portland Bicycle Count Report 2013-2014

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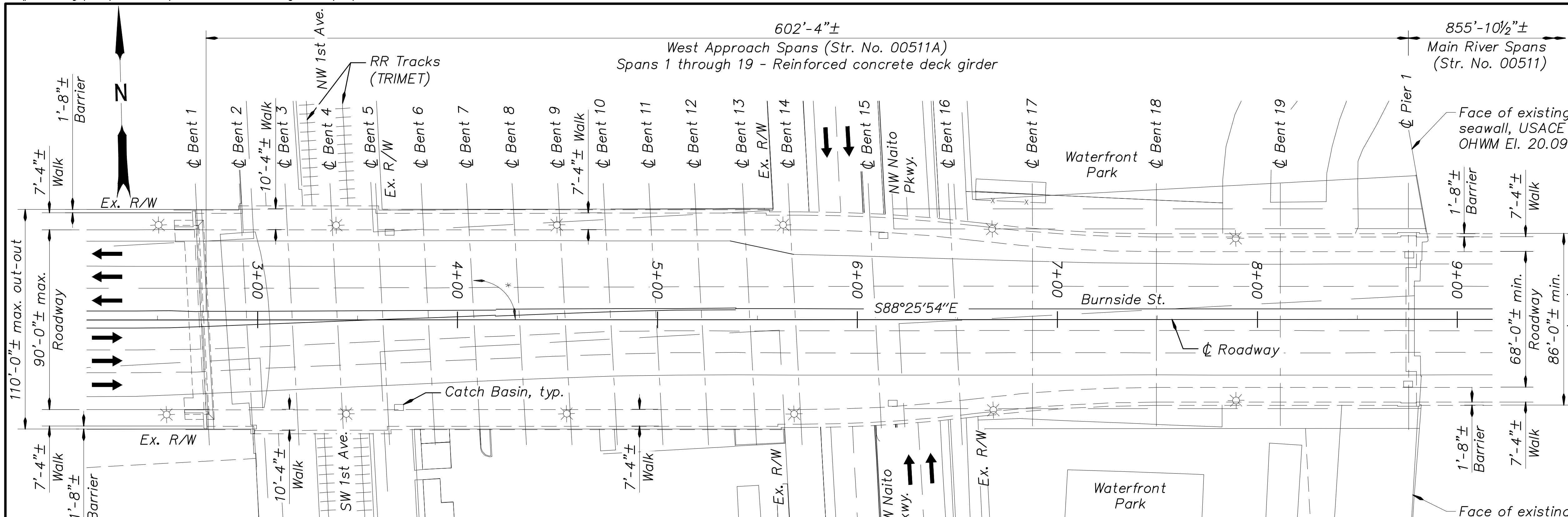
2017 *Connecting Washington Bid Environment* presented to the Joint Transportation Committee on July 20, 2017.

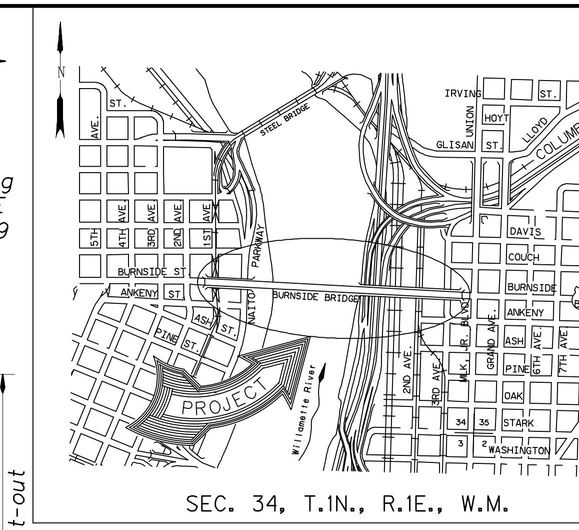
Appendix A. Burnside Bridge Seismic Deficiency Plans



PLAN

Scale: 1" = 30'-0"

∗ Angle for Bents 1 -16 = 93°05'30",
Angle for Bents 17 - 19, and Pier 1 = 90°00'00"

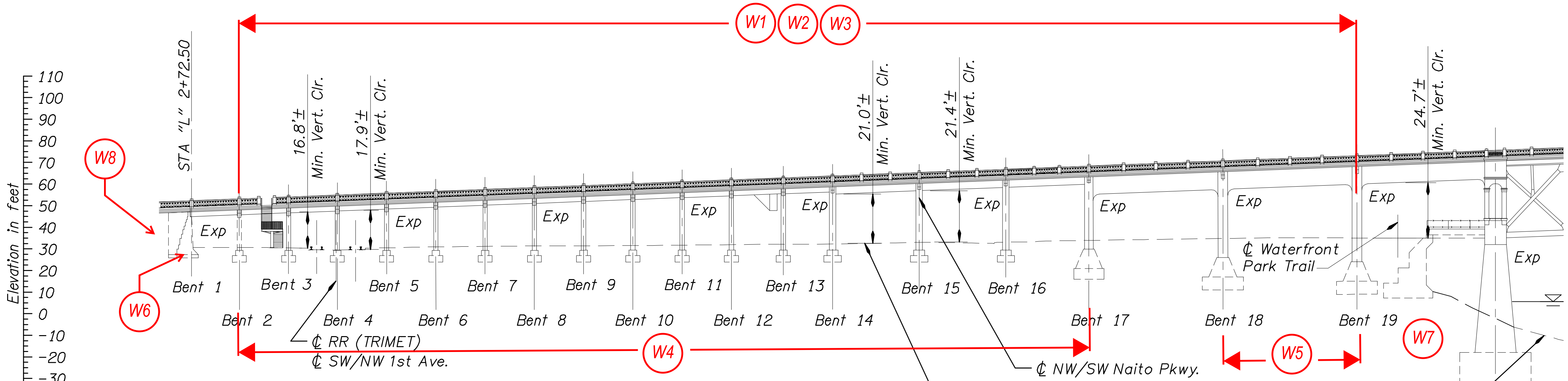


LOCATION MAP

No Scale

POTENTIAL SEISMIC DEFICIENCY

- W1 Superstructure flexural or shear strength
- W2 End floorbeam flexural or shear strength
- W3 Column flexural or shear strength, poor confinement detailing
- W4 Footing size and strength, unreinforced footings
- W5 Timber pile lateral strength and uplift capacity
- W6 Tall abutment retaining wall footing size
- W7 Liquefiable soils and lateral spreading
- W8 Approach retaining wall footing sizes and interaction with adjacent buildings



ELEVATION

Scale: 1" = 30'-0"

00511, 00511A, 00511B

Notes:

1. Span support con... otherwise.
2. Elevations shown are based on North American Vertical Datum, (NAVD) 1988.

SCALE WARNING

If scale bar doesn't measure one inch then drawing is not to scale

△	DATE	REVISION	BY

ACCOMPANIED BY DWGS. ....

DRAFTER: ....
DESIGNER: ....
CHECKER: ....
REVIEWER: ....

EXPIRES: ....

Connecting Commerce and Community
MULTNOMAH COUNTY BRIDGES

HDR

STRUCTURE NO. See above

DATE

CALC. BOOK

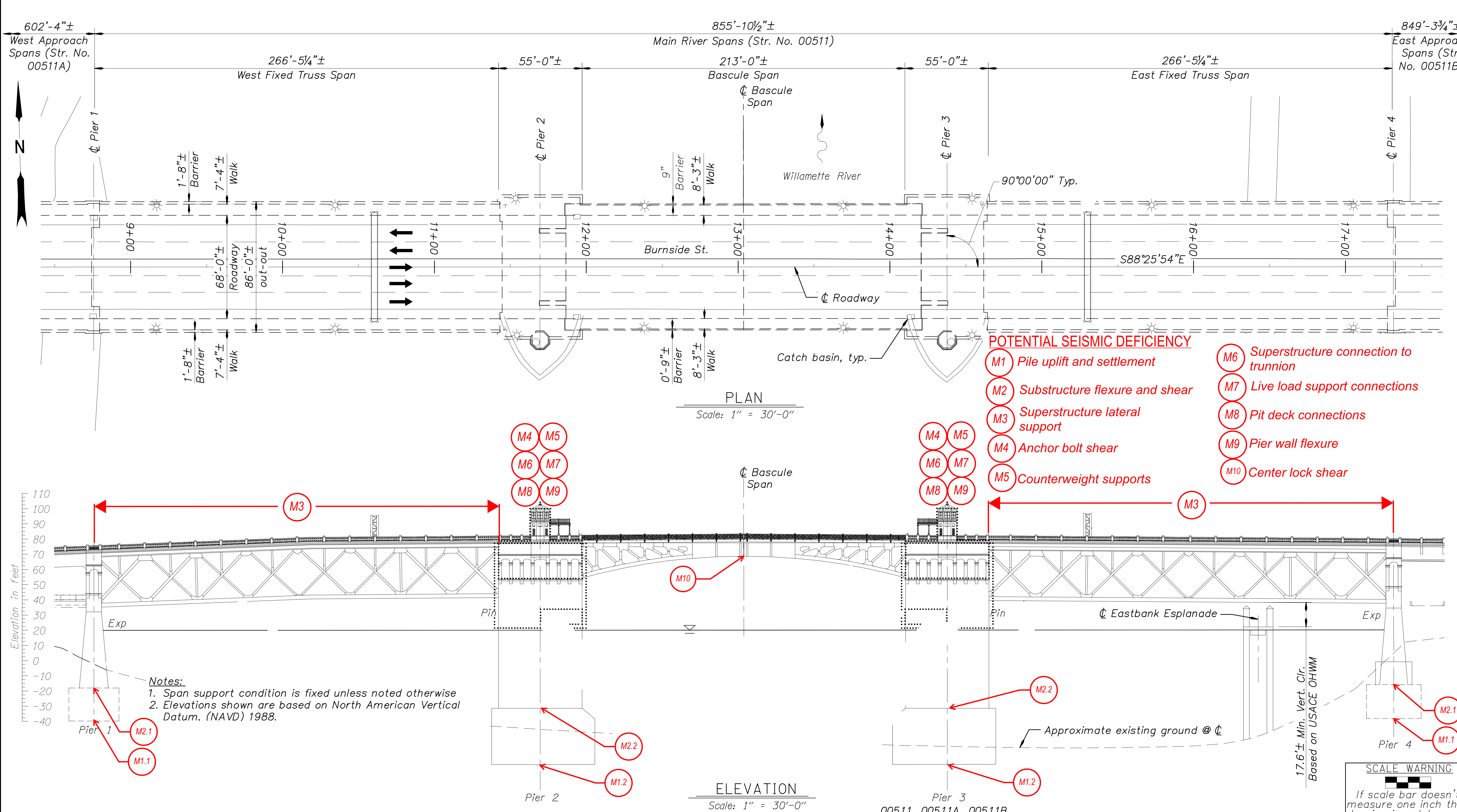
BURNSIDE ST: WILLAMETTE RIVER BRIDGE
WEST APPROACH
POTENTIAL SEISMIC DEFICIENCY

PLAN AND ELEVATION - 1

SHEET .... OF ....

DRAWING NO. ....

C:\pwworking\sea\d1676779\HDR00511_02_PE2.dwg 9/23/2016 11:49 AM



DATE	REVISION	BY

ACCOMPANIED BY DWGS.

DRAFTER:
DESIGNER:
CHECKER:
REVIEWER:
EXPIRES:

Connecting Commerce and Community
MULTNOMAH COUNTY BRIDGES
HDR

STRUCTURE NO. See above
DATE
CALC. BOOK

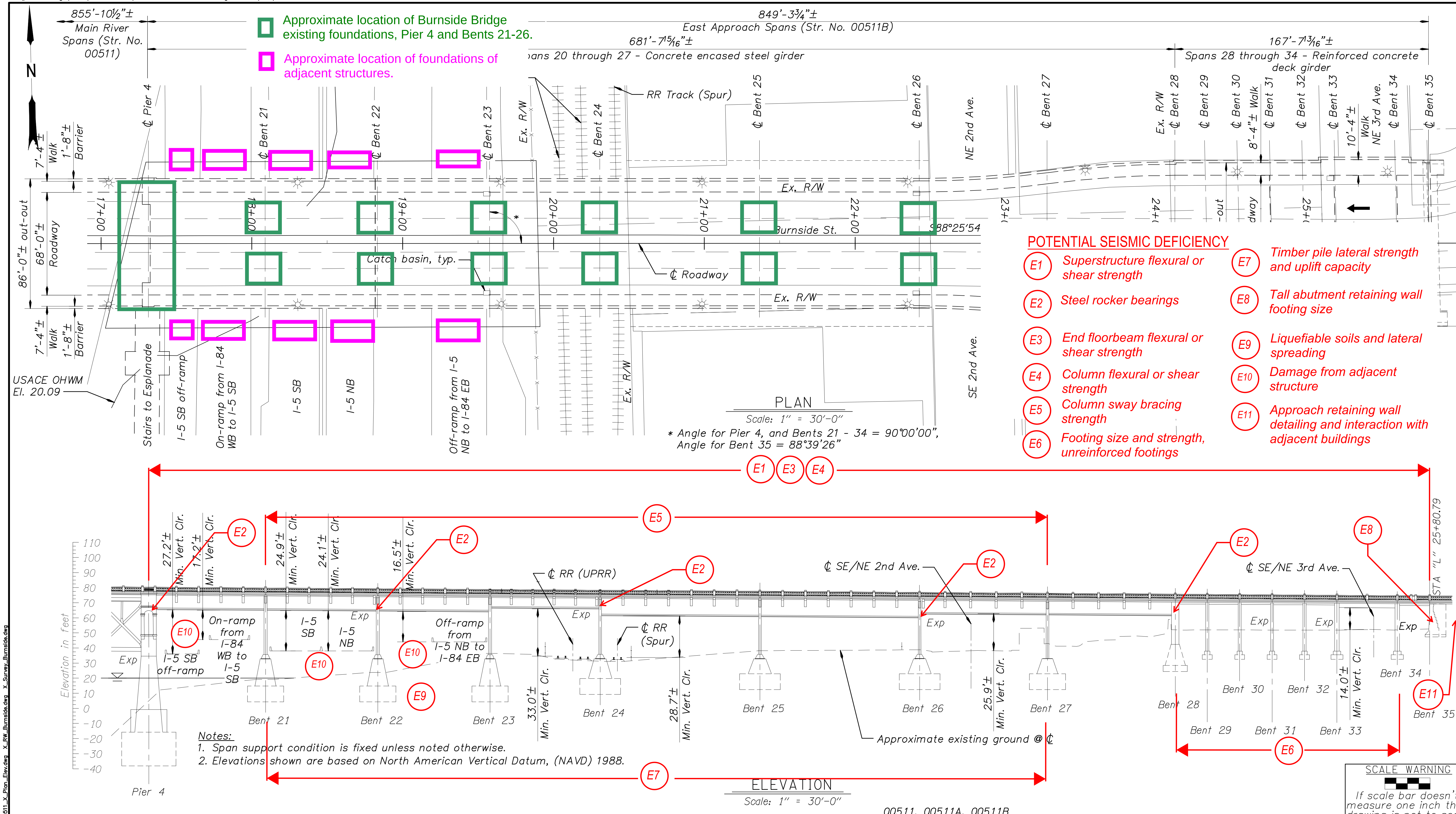
BURNSIDE ST: WILLAMETTE RIVER BRIDGE

PLAN AND ELEVATION - 2

SHEET .... OF ....
DRAWING NO.

Xrefs: Burnside_Bridge_Border.dwg BR00511_X_Plan_Elev.dwg X_Survey_Burnside.dwg

C:\pwworking\sea\d1676779\HDR00511_03_PE3.dwg 9/15/2016 9:56 AM



DATE	REVISION	BY

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DESIGNER:
CHECKER:
REVIEWER:
EXPIRES:

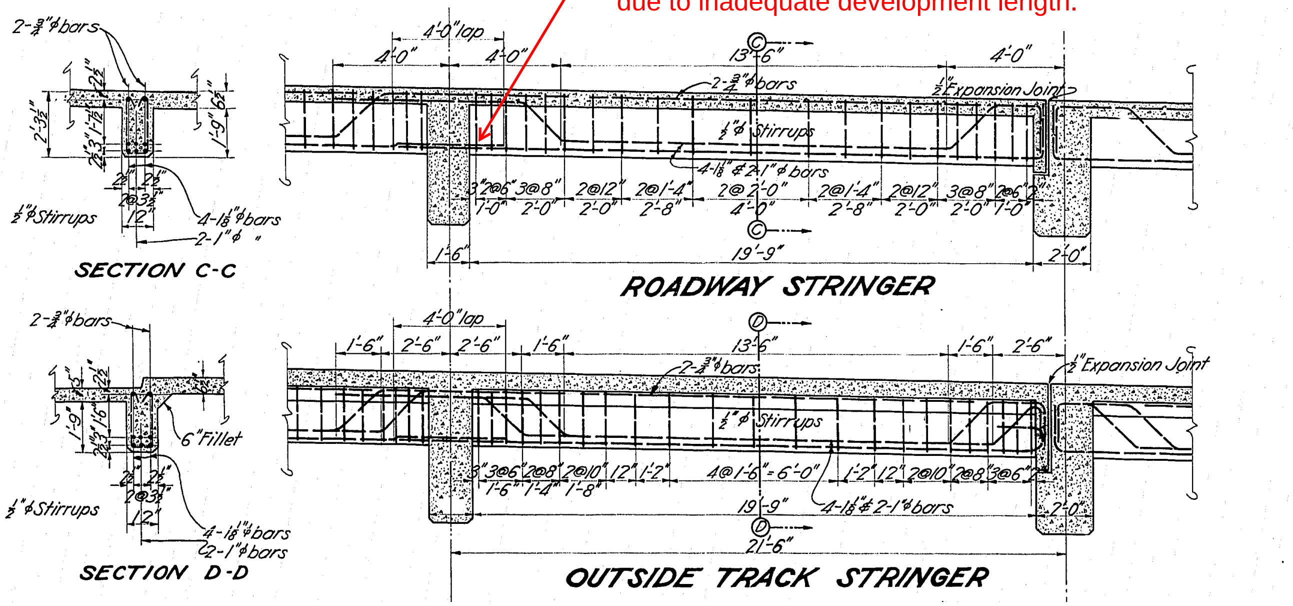
Connecting Commerce and Community
MULTNOMAH COUNTY BRIDGES
HDR

STRUCTURE NO. See above
DATE
CALC. BOOK

BURNSIDE ST: WILLAMETTE RIVER BRIDGE
EAST APPROACH
POTENTIAL SEISMIC DEFICIENCY
PLAN AND ELEVATION - 3

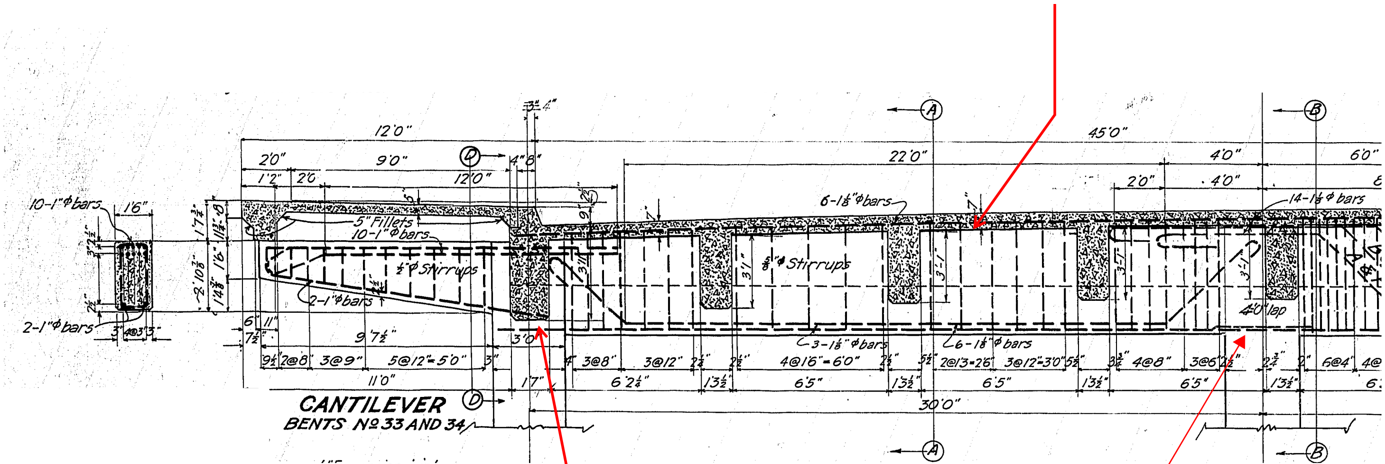
SHEET ... OF ...
DRAWING NO.

Poor seismic detail. Girder positive moment reinforcement spliced at column connection thus limiting moment capacity due to inadequate development length.



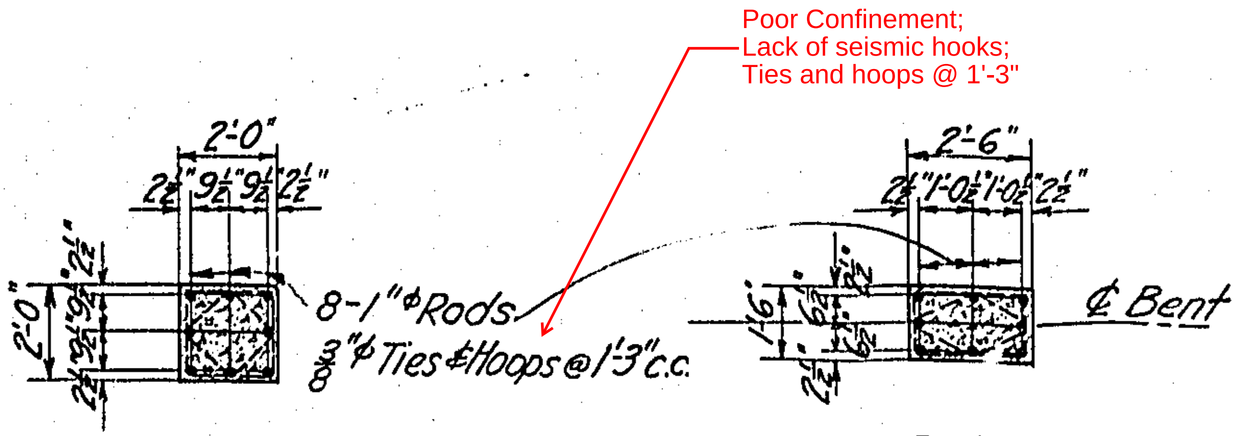
W1

At isolated locations, inadequate negative moment capacity at midspan to maintain elastic behavior for CSZ event.



Poor seismic detail. Floorbeam positive moment reinforcement spliced or has limited embedment at column connection thus limiting moment capacity due to inadequate development length.

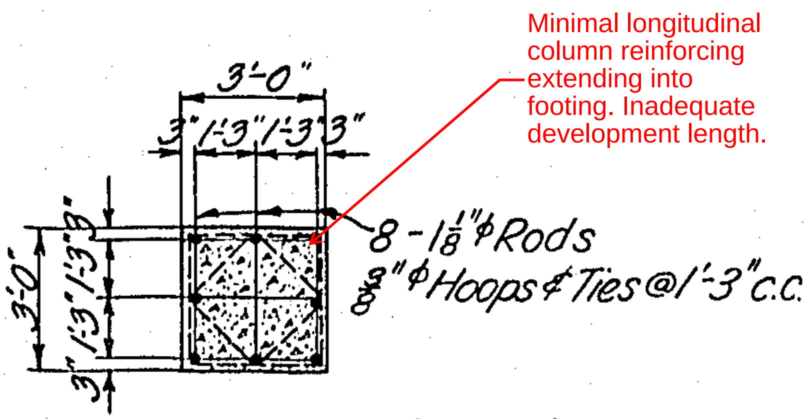
W2



COLUMN CROSS SECTIONS

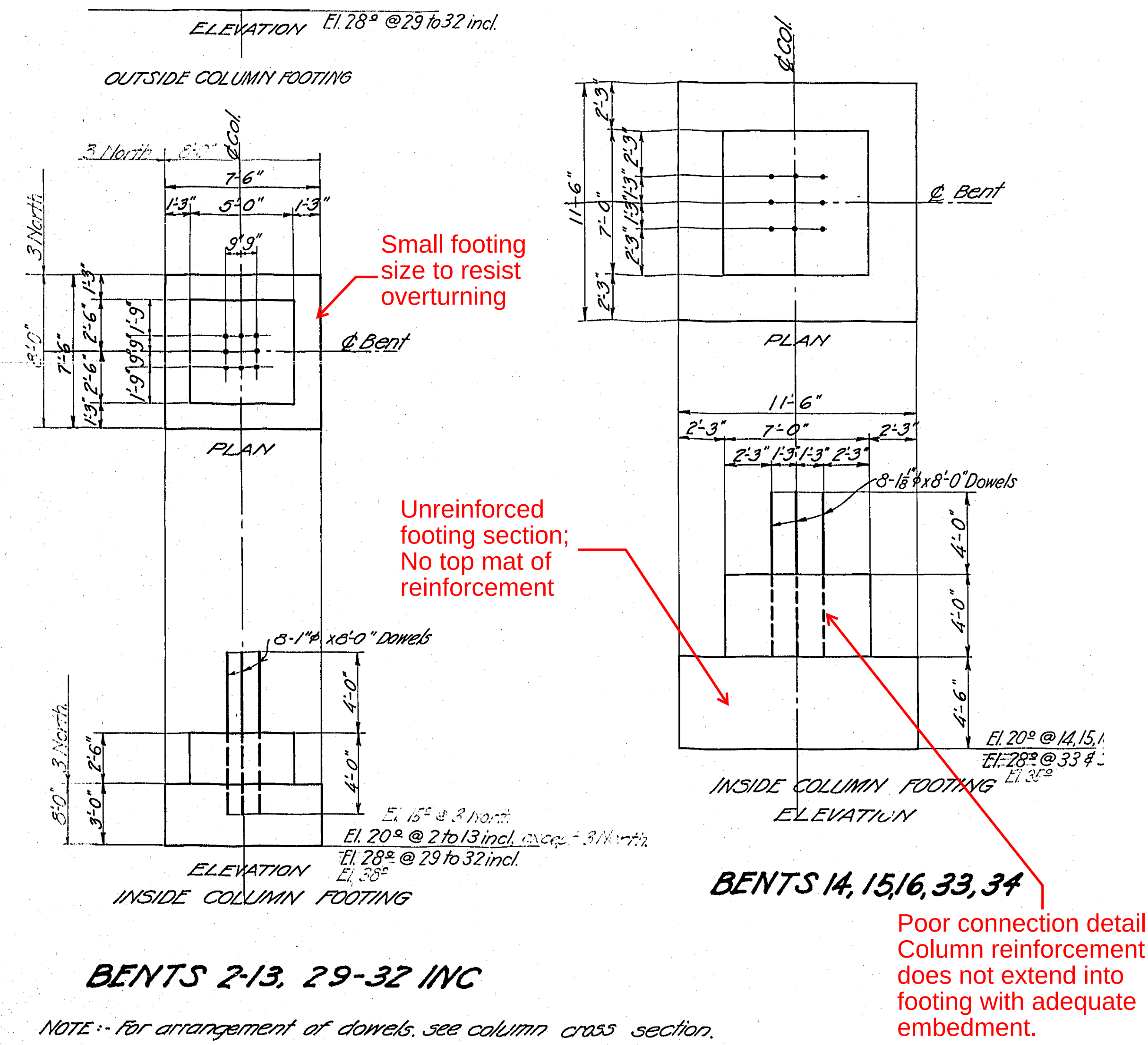
BENTS 2, 5 TO 13, & 29-32 INC

BENTS 3-4 ONLY



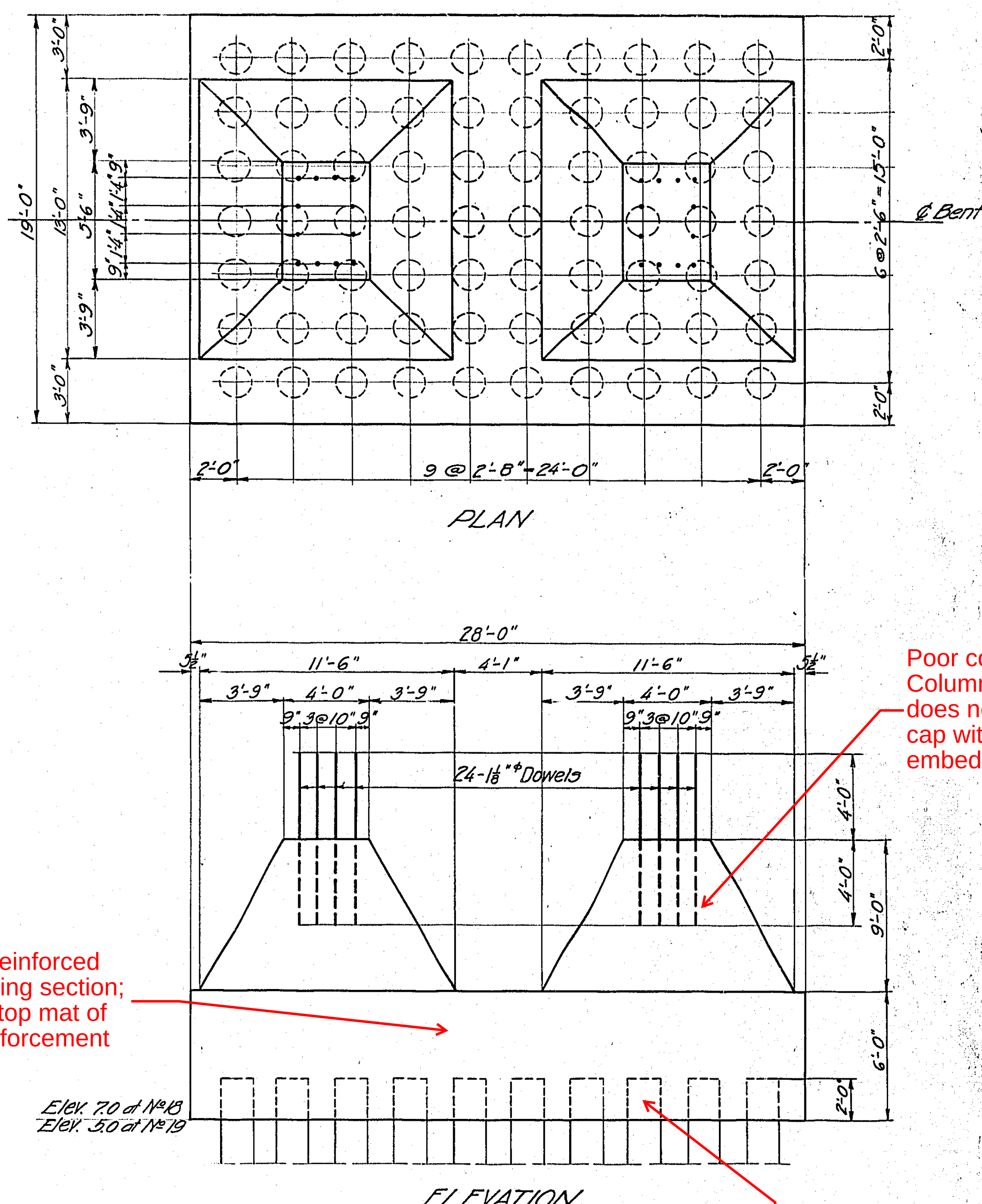
COLUMN CROSS SECTION

W3



Soil Structure Interaction:
Spread footing inadequate to limit settlement from liquefaction to a desirable level.

W4



BENTS 18-19

For Pile details see "Pile Record"

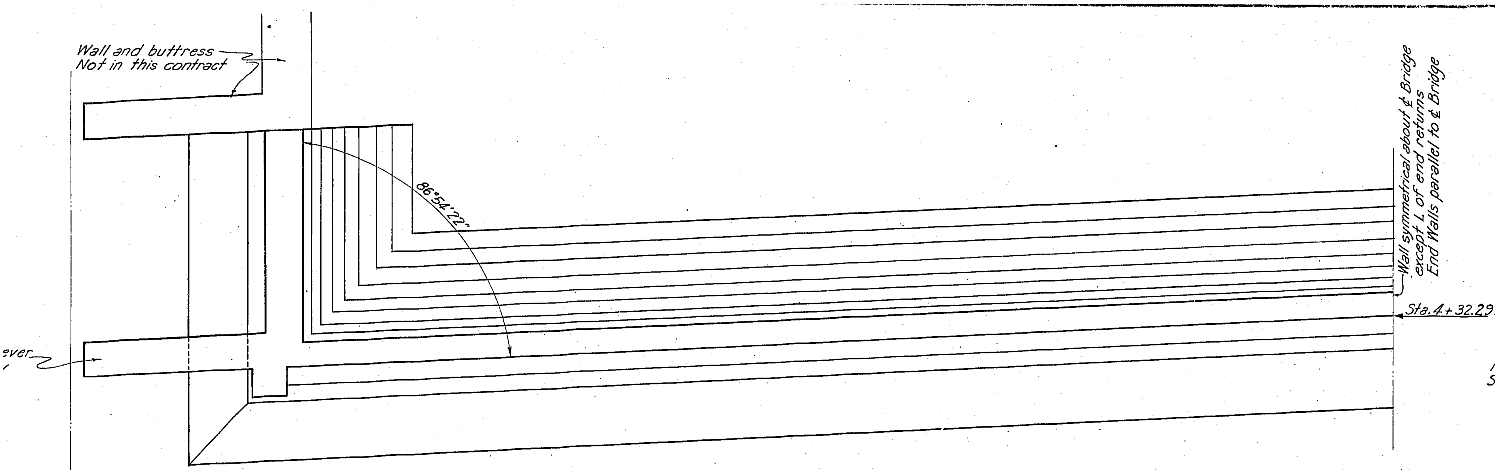
Poor connection detail; Column reinforcement does not extend into pile cap with adequate embedment.

Unreinforced footing section; No top mat of reinforcement

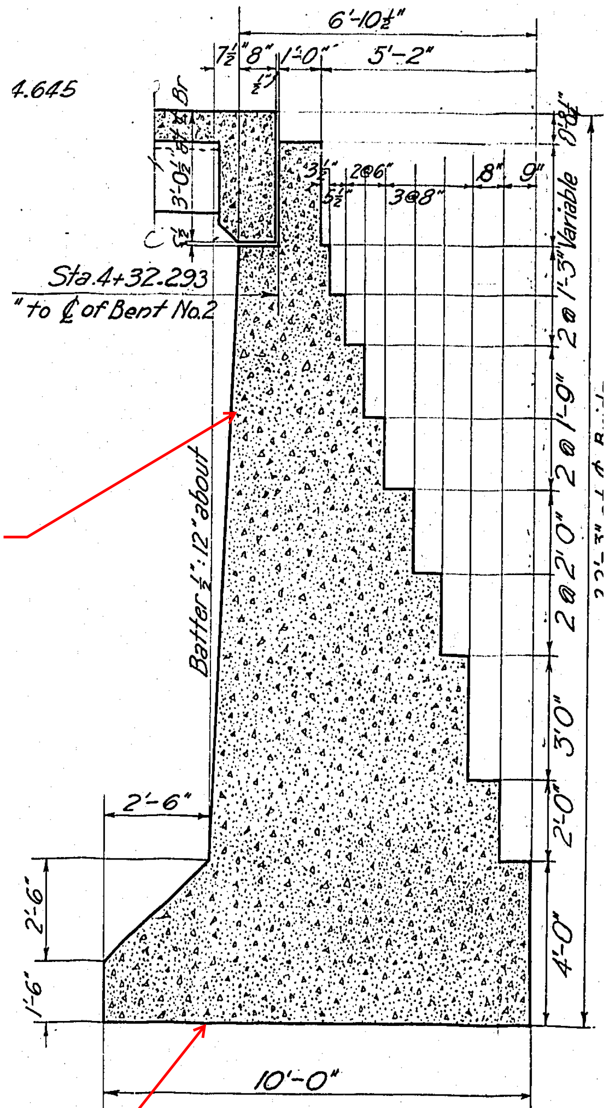
Embedment of piles into pile caps inadequate to resist seismic uplift forces.

<u>Soil Structure Interaction:</u>
Pile group capacity inadequate to limit settlement from liquefaction to a desirable level, resist uplift and downdrag forces, and to resist displacements and forces from lateral spreading.

W5



PLAN



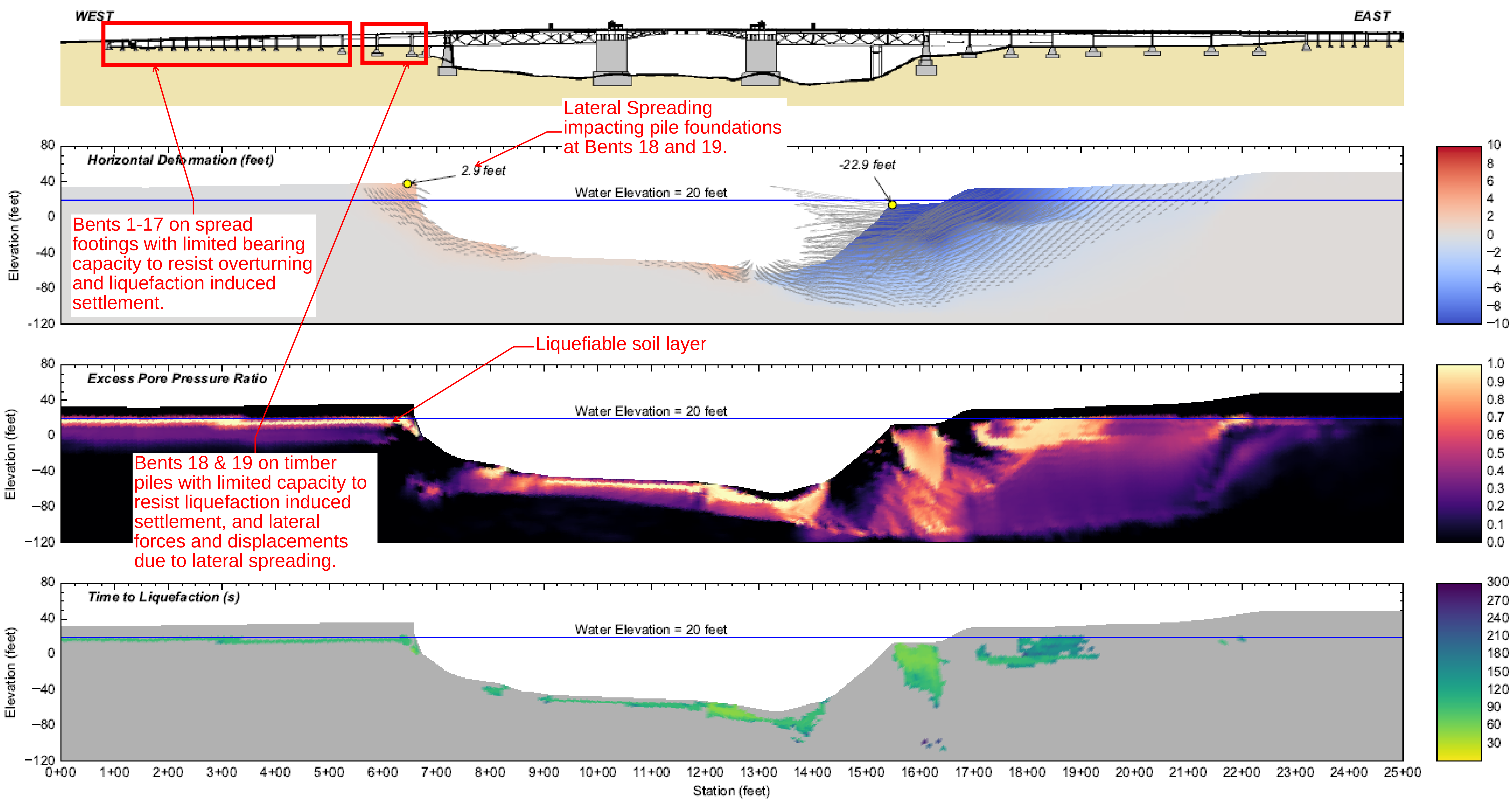
SECTION B-B

Unreinforced abutment wall.

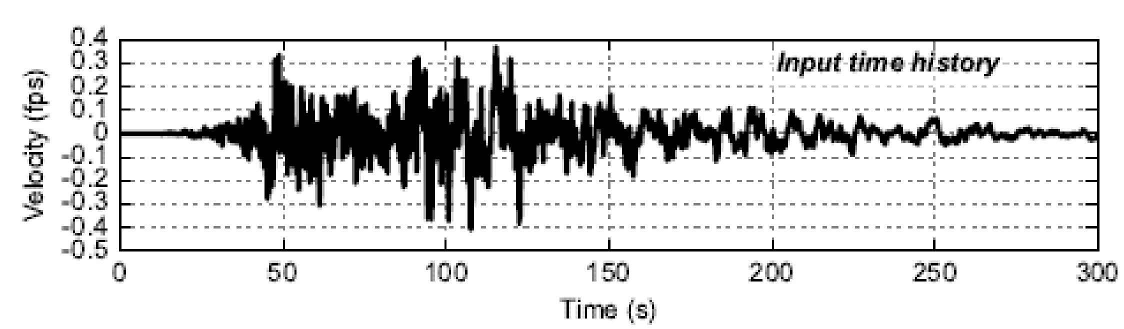
Soil Structure Interaction:
Spread footing inadequate to limit vertical and differential settlement from liquefaction to a desirable level.

Narrow footing size to resist overturning and limited effects of differential settlement.

W6



Notes:
1. The input ground motion was recorded during the Tohoku earthquake at the AKT station (east-west component). We scaled the time history to approximate a Cascadia Subduction Zone event.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
fps = feet per second



Burnside Bridge
Portland, Oregon

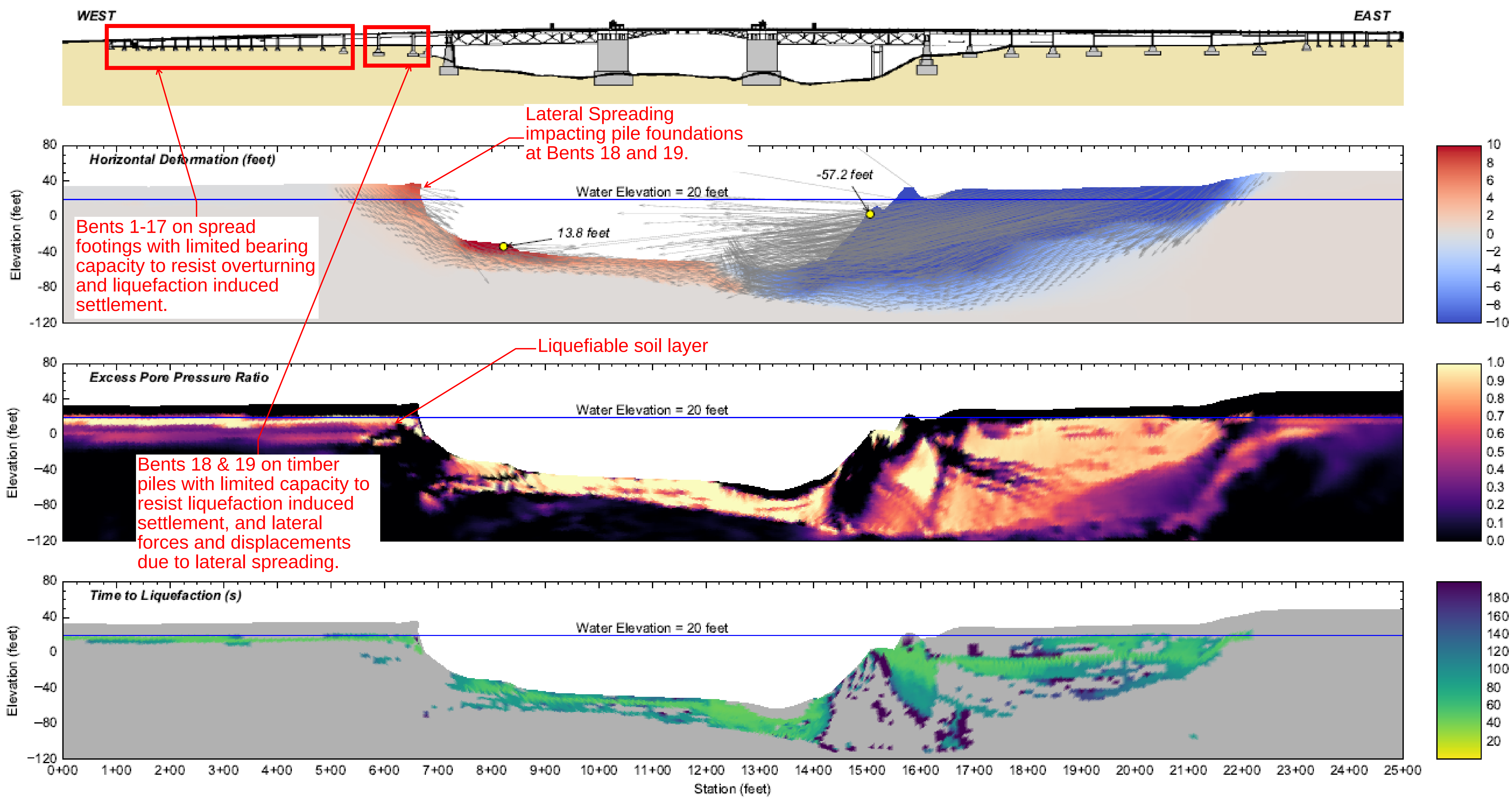
W7
CSZ

MODEL RESPONSE
TIME HISTORY T-CSZ

January 2017 24-1-04065-003

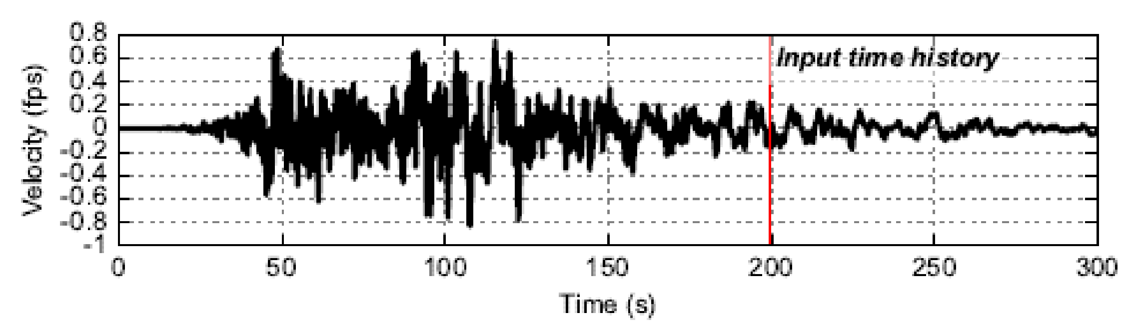
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6



Notes:

1. The input ground motion was recorded during the Tohoku earthquake at the AKT station (east-west component). We scaled the time history to approximate a 1,000 year return period. The model failed due to bad geometry at 199.2 seconds.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
Portland, Oregon

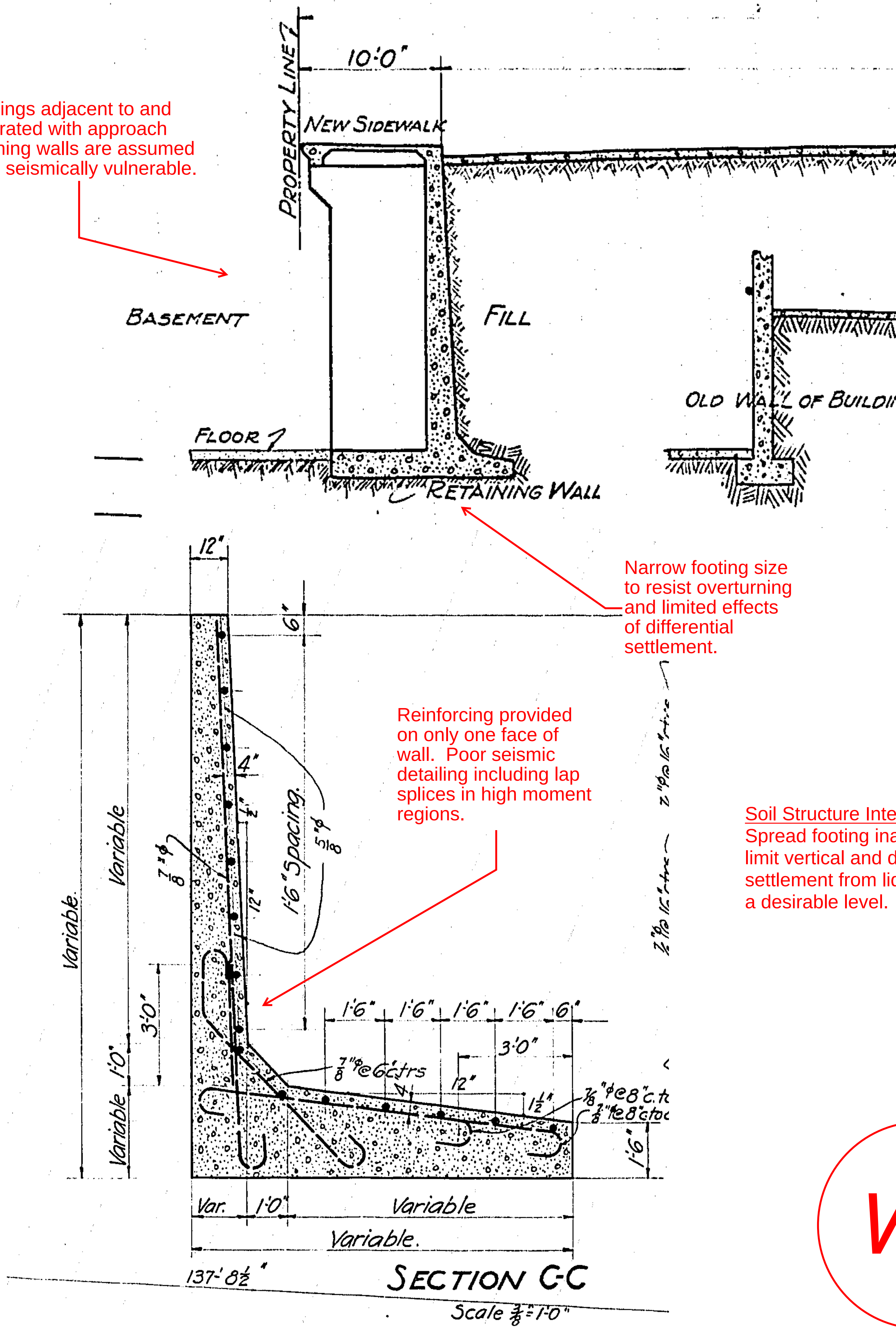
W7
1000-yr

MODEL RESPONSE
TIME HISTORY T-1000

January 2017 | 24-1-04065-003

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2



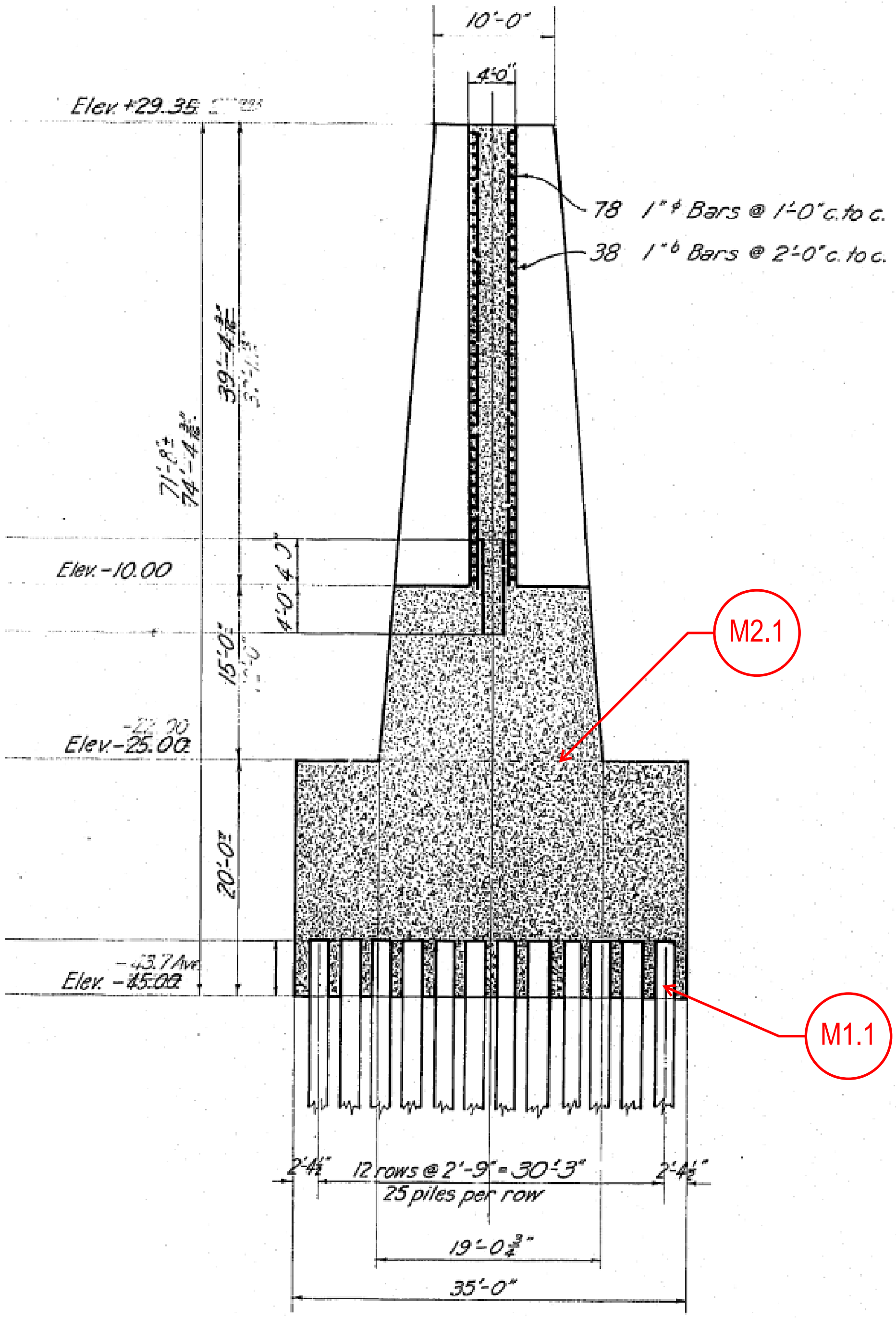
Buildings adjacent to and integrated with approach retaining walls are assumed to be seismically vulnerable.

Narrow footing size to resist overturning and limited effects of differential settlement.

Reinforcing provided on only one face of wall. Poor seismic detailing including lap splices in high moment regions.

<u>Soil Structure Interaction:</u>
Spread footing inadequate to limit vertical and differential settlement from liquefaction to a desirable level.

W8



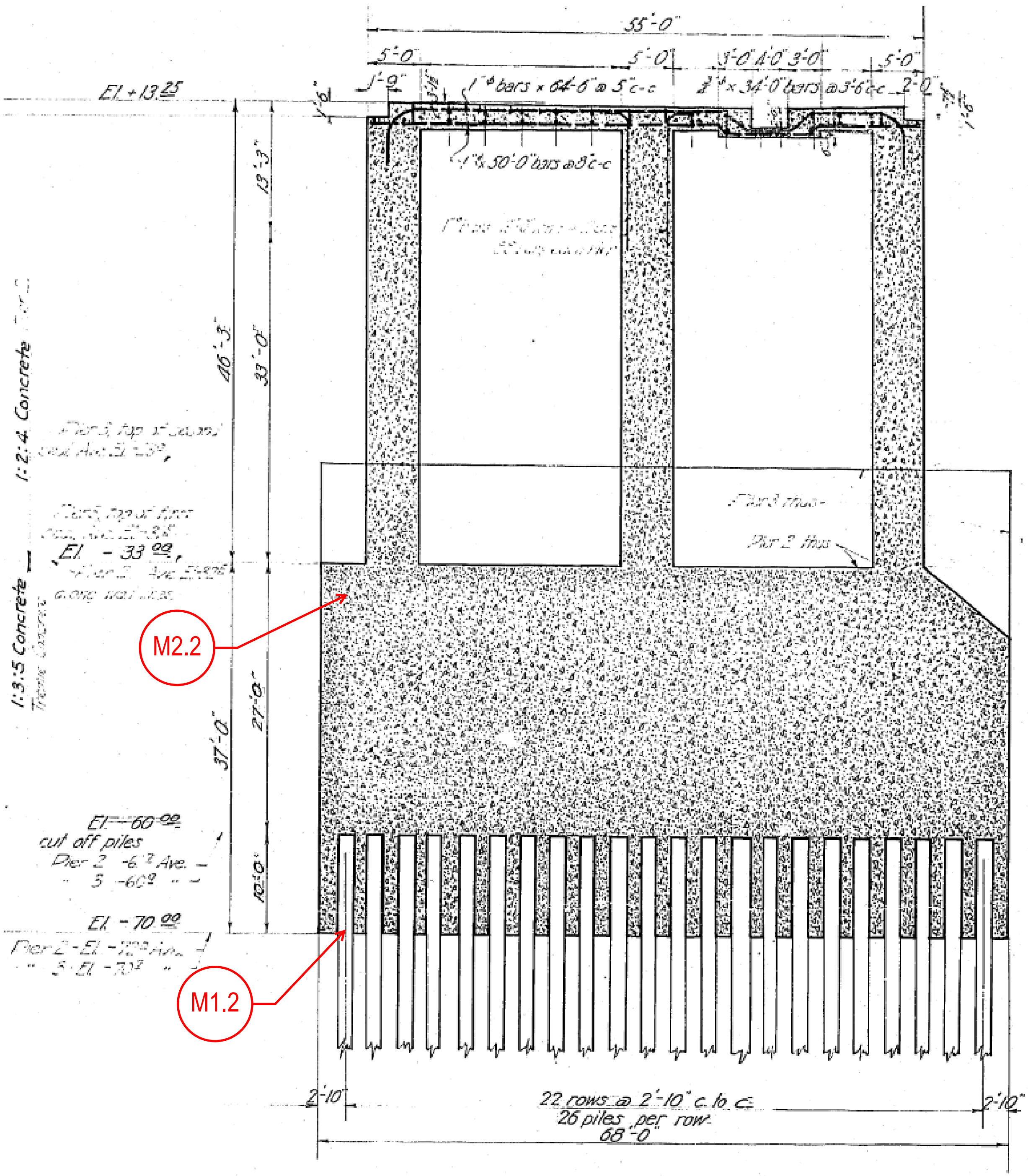
SECTION THROUGH CENTER

M1.1

The tapered timber piles at piers 1 and 4 are vulnerable to uplift due to overturning. Geotechnical analysis suggests piers may be subject to significant settlement and lateral spread during Cascadia level seismic events

M2.1

Concrete pile caps are unreinforced. This limits flexural capacity to the cracking strength of the concrete. Shear capacity is limited to Vc.

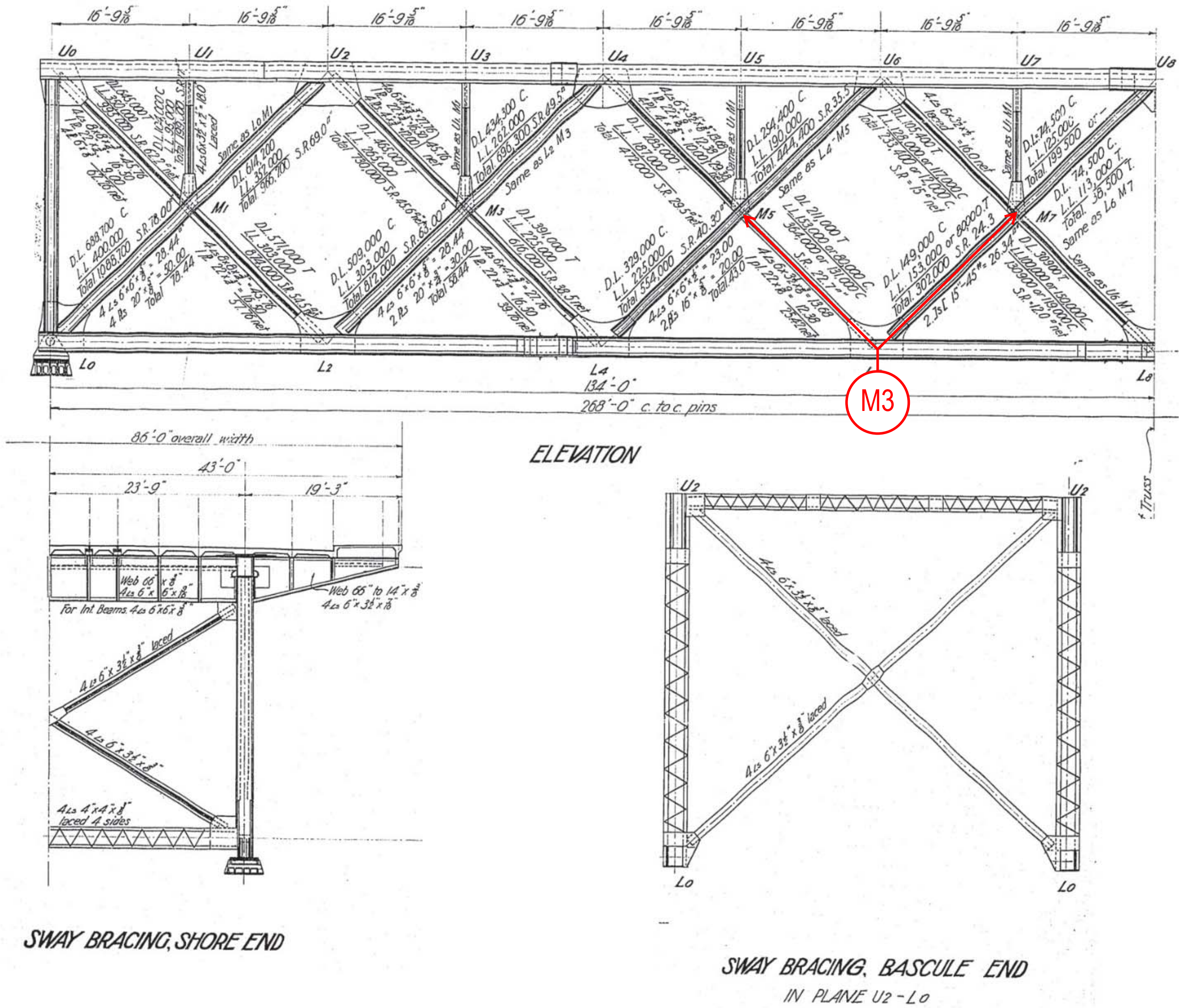


M1.2

The tapered timber piles at piers 2 and 3 are vulnerable to uplift due to overturning. Geotechnical analysis suggests piers may be subject to significant settlement and lateral spread during Cascadia level seismic events

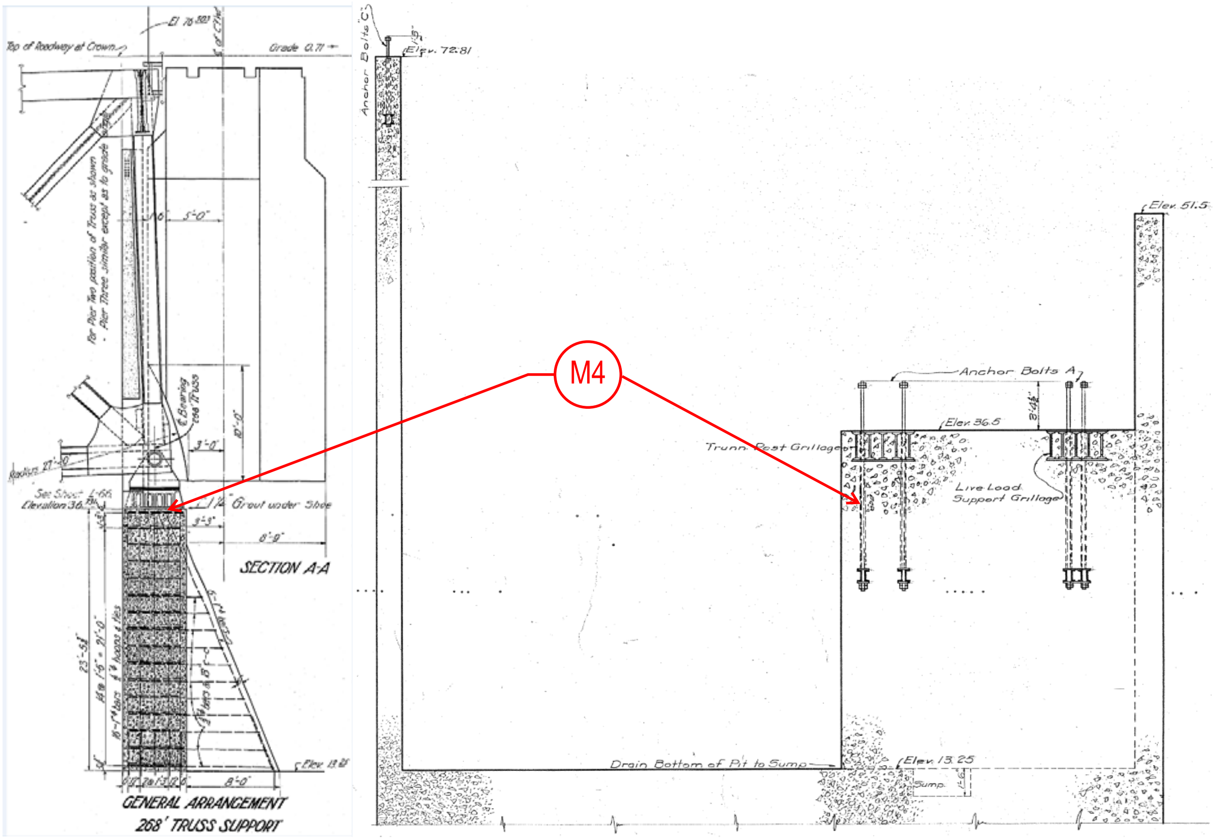
M2.2

Concrete pile caps are unreinforced. This limits flexural capacity to the cracking strength of the concrete. Shear capacity is limited to Vc.



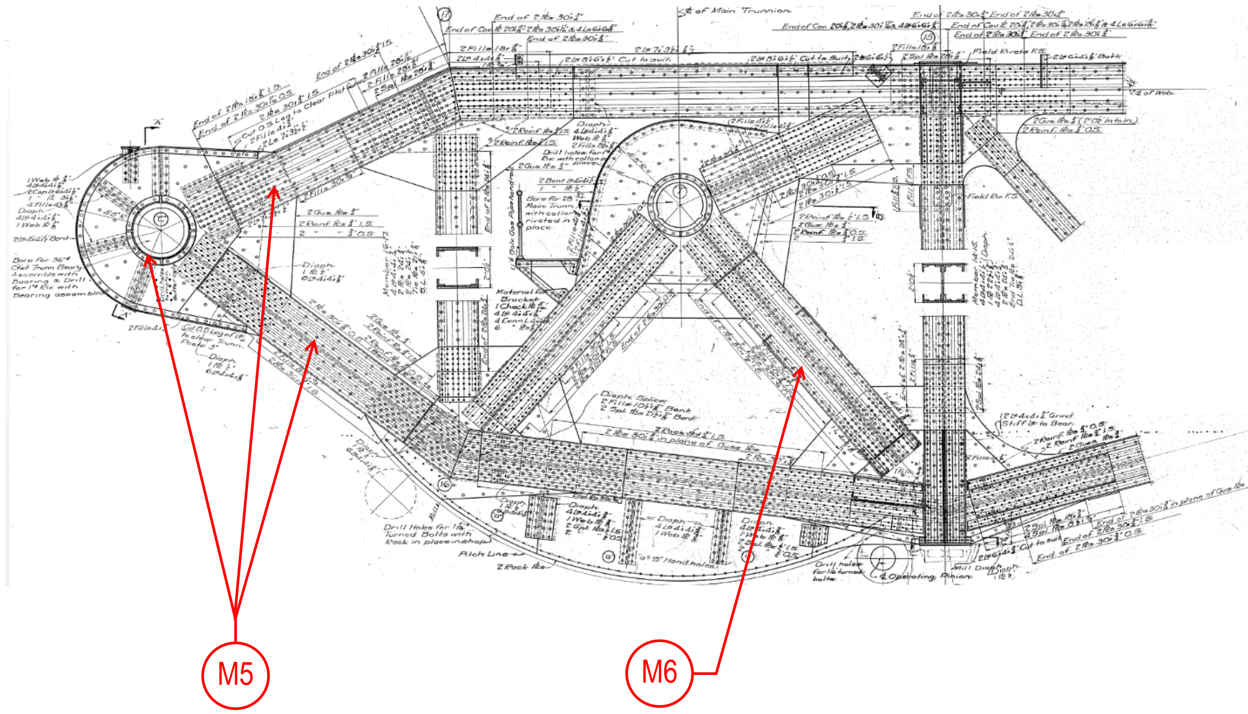
M3

Sway bracing is absent from 4 of the 8 bays in the fixed spans. Sway bracing is required in each bay to prevent collapse of the deck and top chord caused by lateral movement.



M4

Existing anchor bolts are insufficient to resist seismic forces.

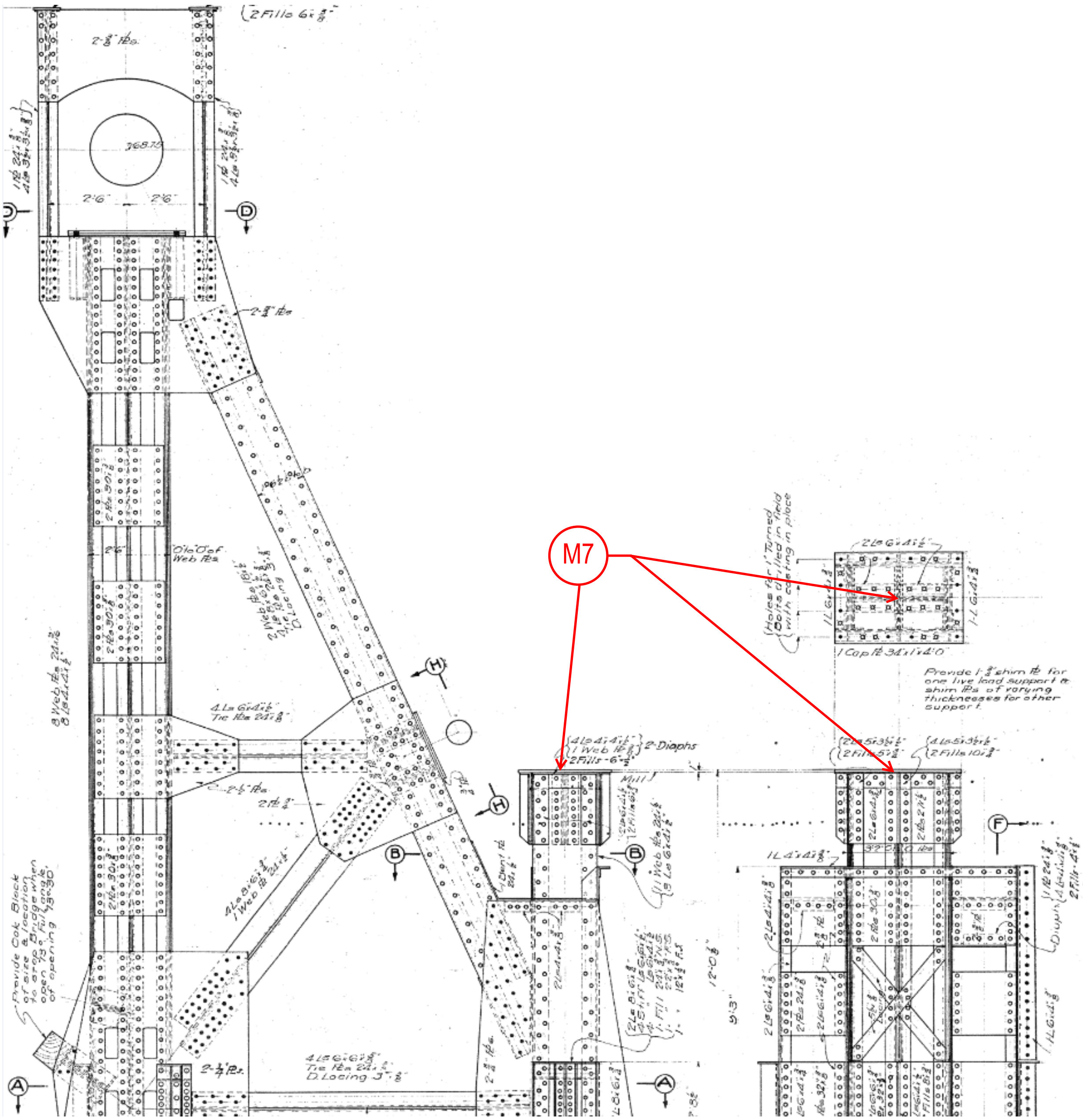


M5

No lateral supports restrain the counterweight. This exposes the counterweight supports to buckling.
In addition, unrestrained lateral movement of the counterweight can impact the reinforced concrete walls supporting the sidewalks. See Note M9

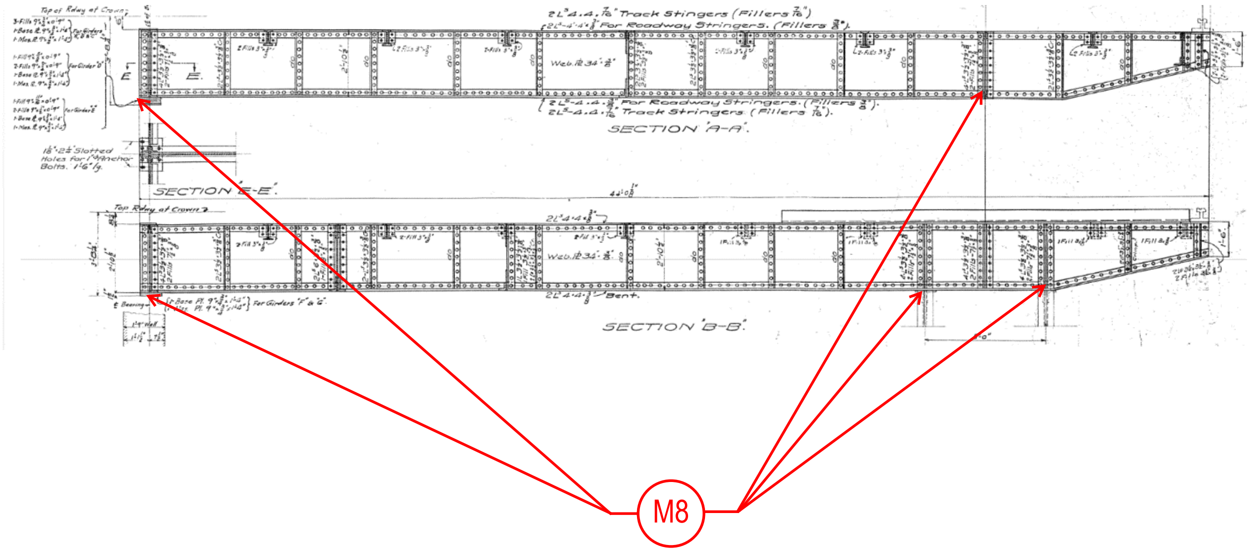
M6

Four members, one for each truss line, transmit lateral loads from the entire bascule span to the substructure. These members are vulnerable to buckling and yielding.



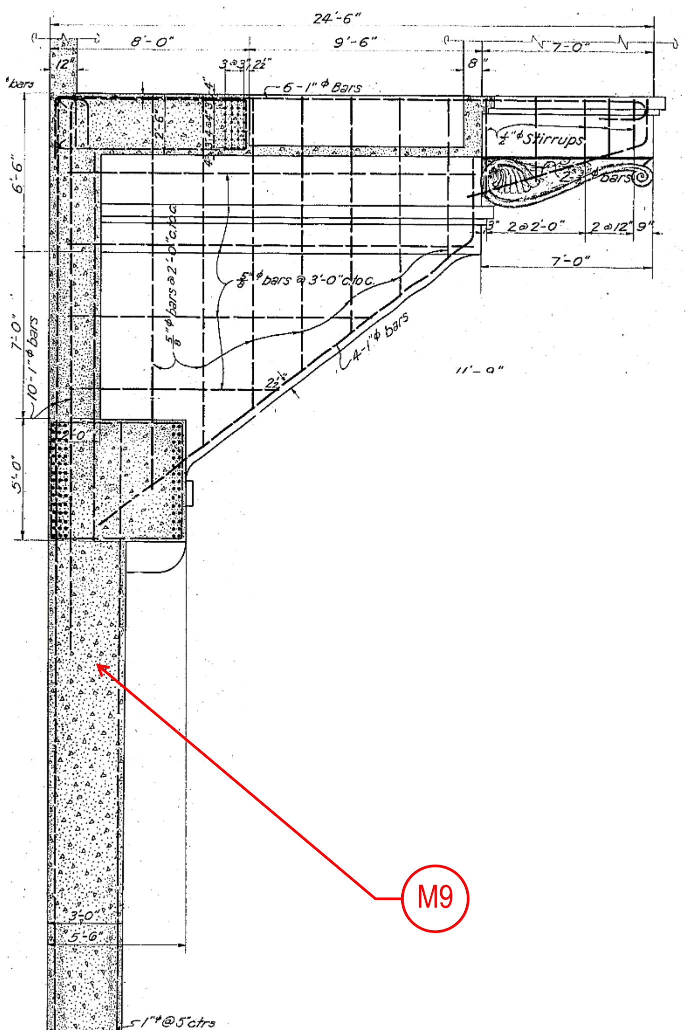
M7

The live load support is intended to resist any vertical loads in addition to the dead loads supported by the Trunnion.
Because it is a simple bearing plate, the live load support cannot resist any upward (tension) loads present in a seismic event. This enables the bascule spans to rotate about the trunnion restrained only by the center lock (See M10). This motion may cause battering forces which cannot be quantified in this analysis model.

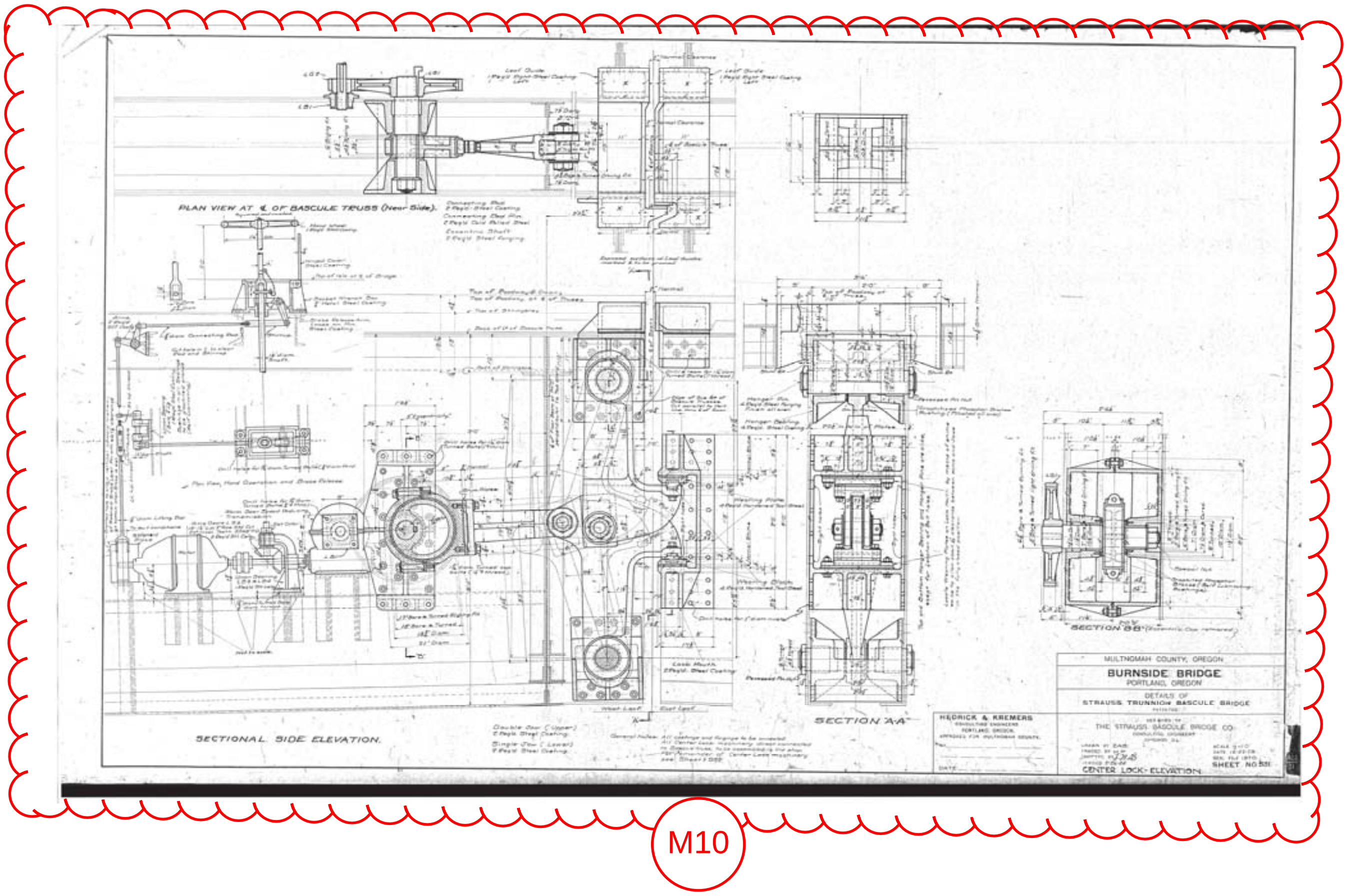


M8

The design plans show pit deck stringers are supported only by bearing supports, and no anchored supports. This configuration allows for unseating of the deck sections directly above Piers 2 and 3.



M9 Pier walls were designed for wind loads and dead loads only, and not detailed to resist seismic forces.

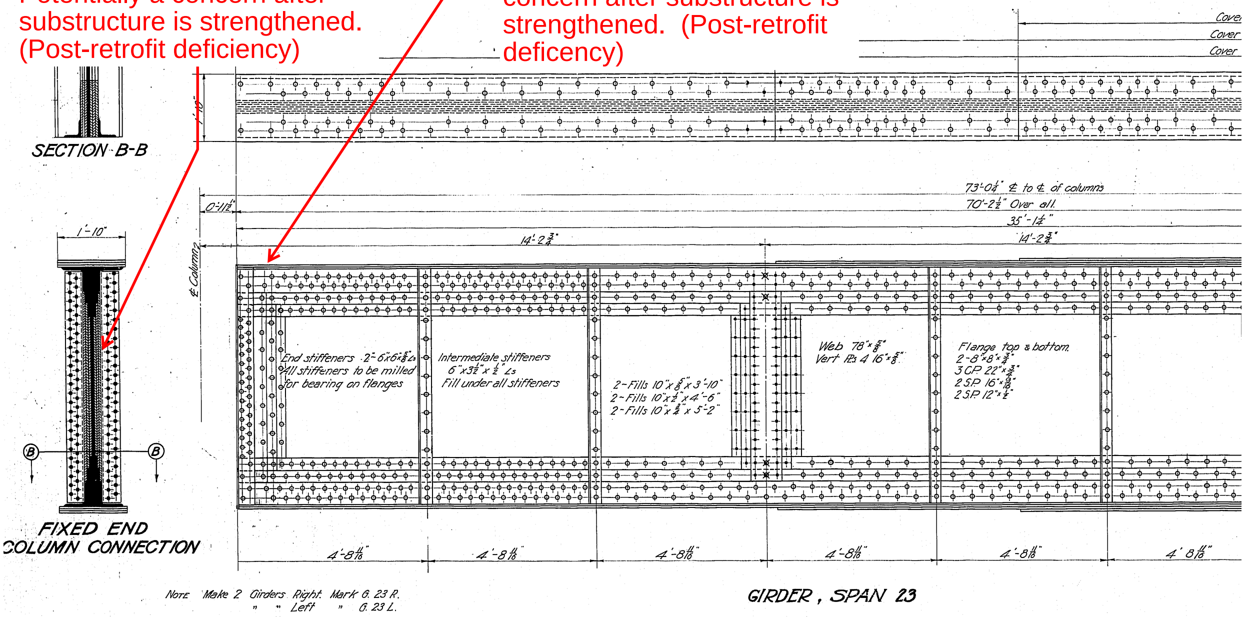


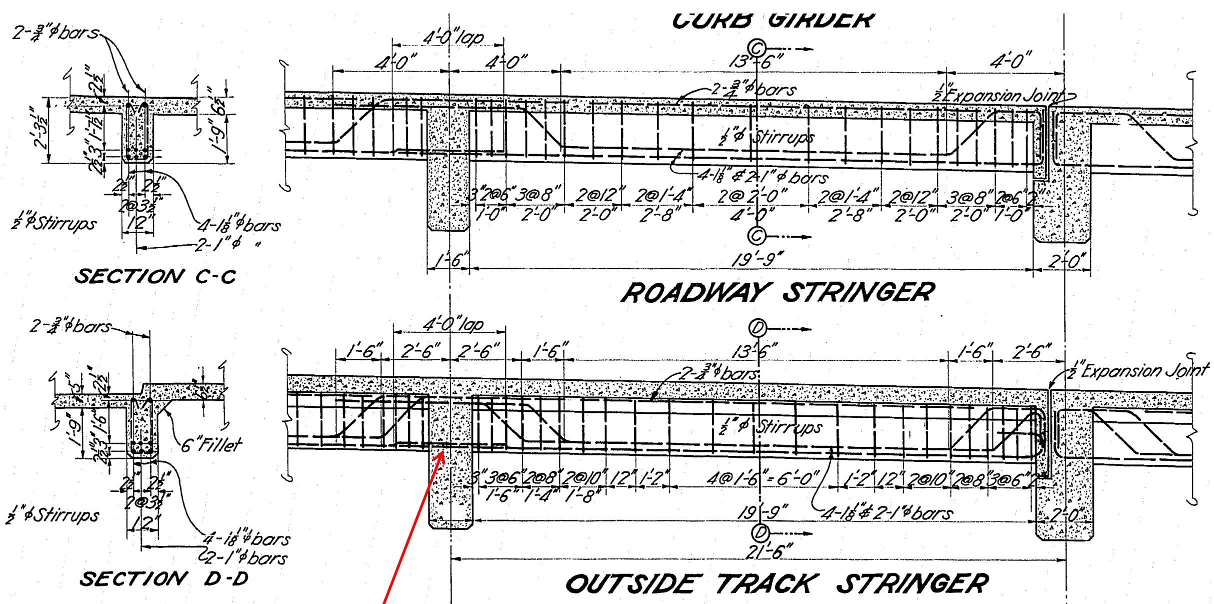
M10

The center lock was not designed to transmit forces caused by the relative displacement of the two bascule spans, and may be severely damaged in a significant seismic event.

Where girders are continuous, riveted column moment connection not originally designed to resist additional moment from seismic loading. Potentially a concern after substructure is strengthened. (Post-retrofit deficiency)

Reduced flange section at fixed end impacts moment capacity of girder / floorbeam connection. Potentially a concern after substructure is strengthened. (Post-retrofit deficency)

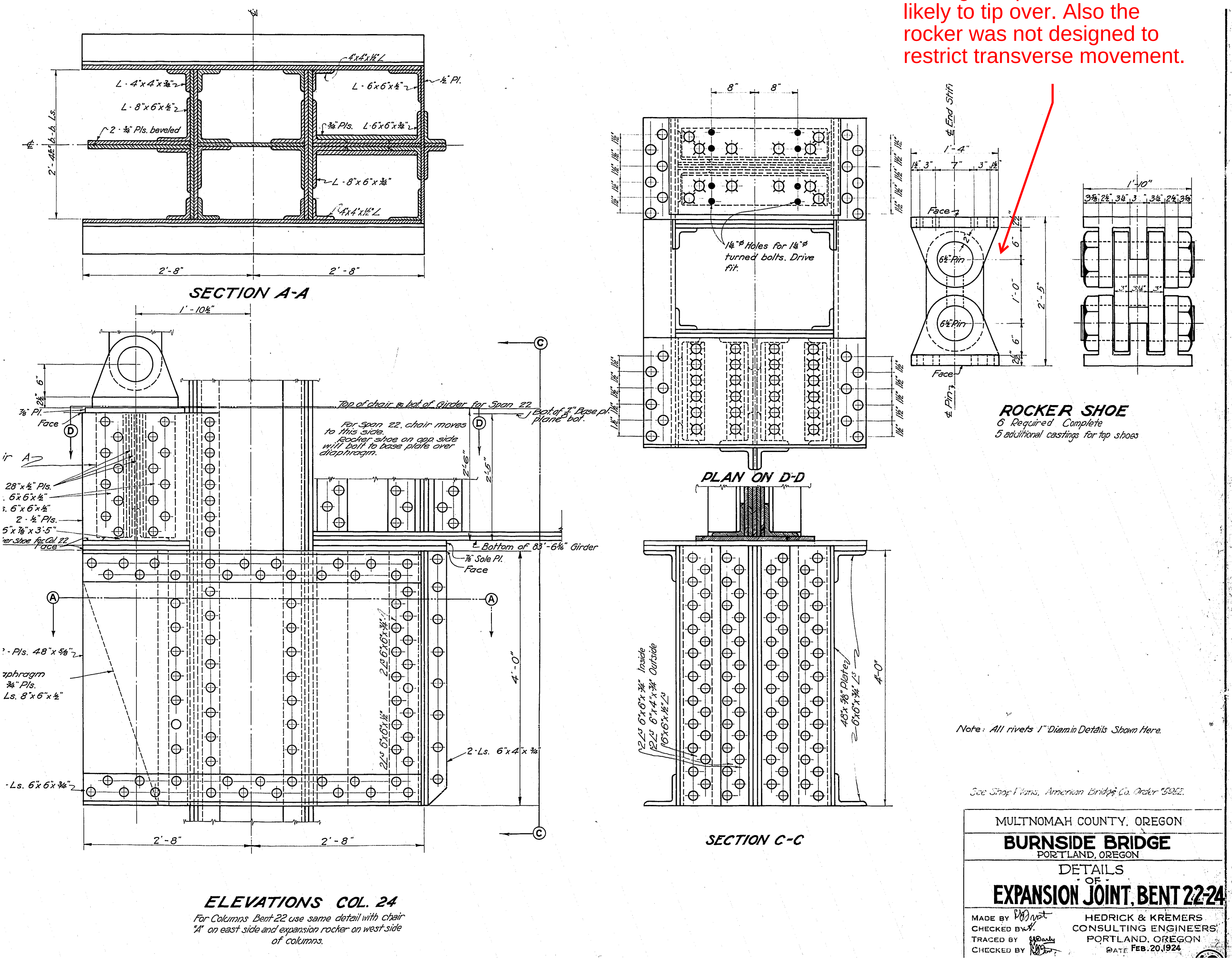




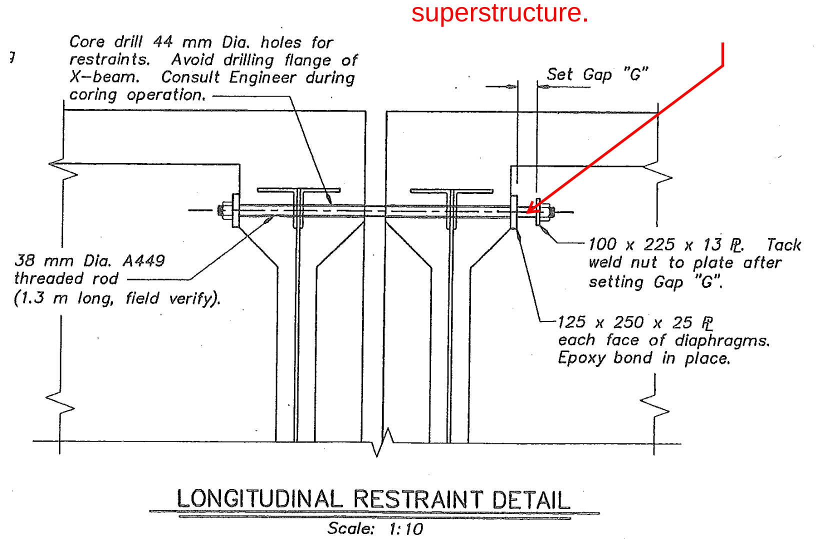
Poor seismic detail. Girder positive moment reinforcement spliced at column connection thus limiting moment capacity due to inadequate development length.

E1

Rocker bearings are not stable for large displacements and likely to tip over. Also the rocker was not designed to restrict transverse movement.



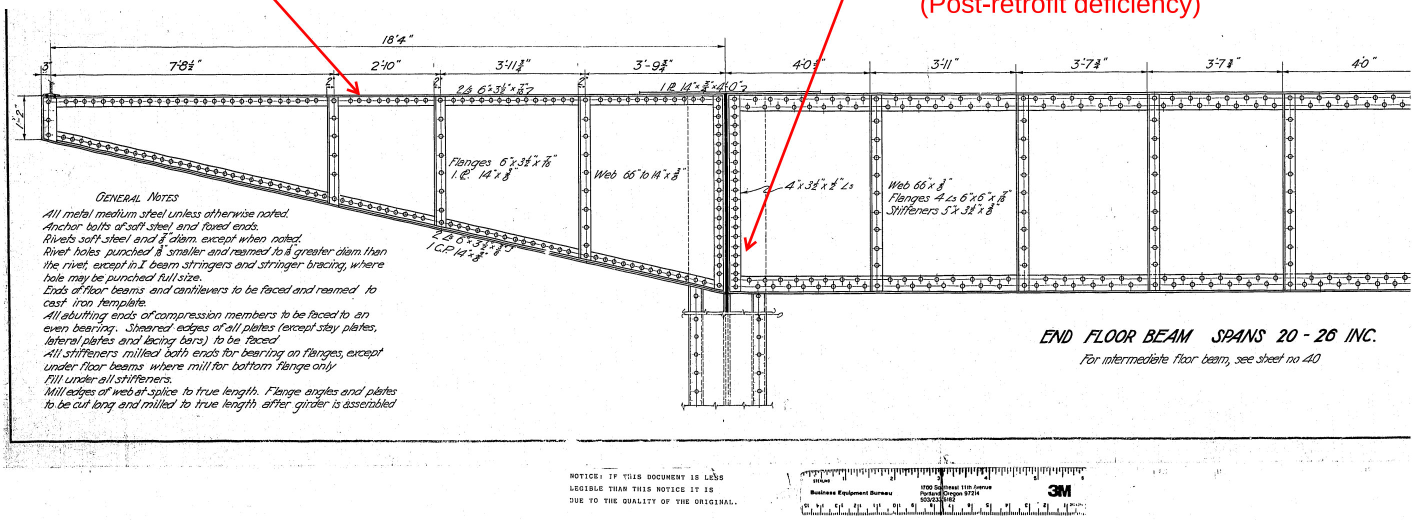
Longitudinal restrainers installed in the early 2000s tied the superstructure together near deck level and do not restrict transverse movement of the superstructure.



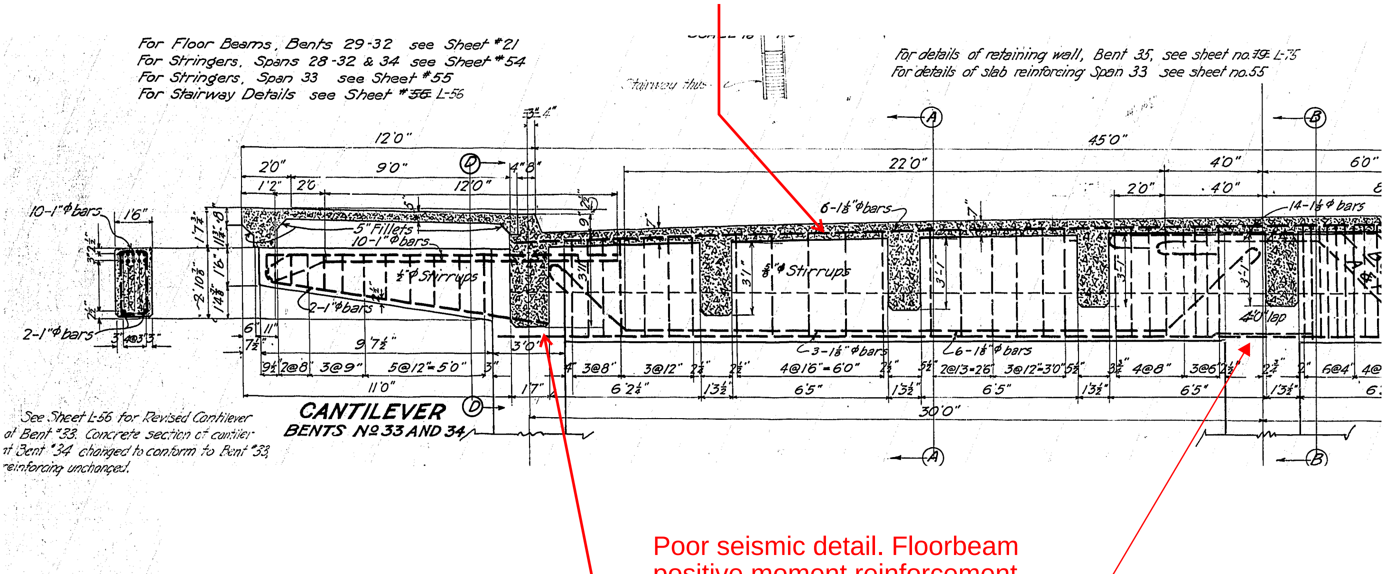
E2

Relatively long cantilever supporting sidewalk and a portion of the roadway.

Riveted connection not originally designed to resist additional moment from seismic loading. Potentially a concern after substructure is strengthened. (Post-retrofit deficiency)



At isolated locations, inadequate negative moment capacity at midspan to remain maintain elastic behavior for CSZ event.

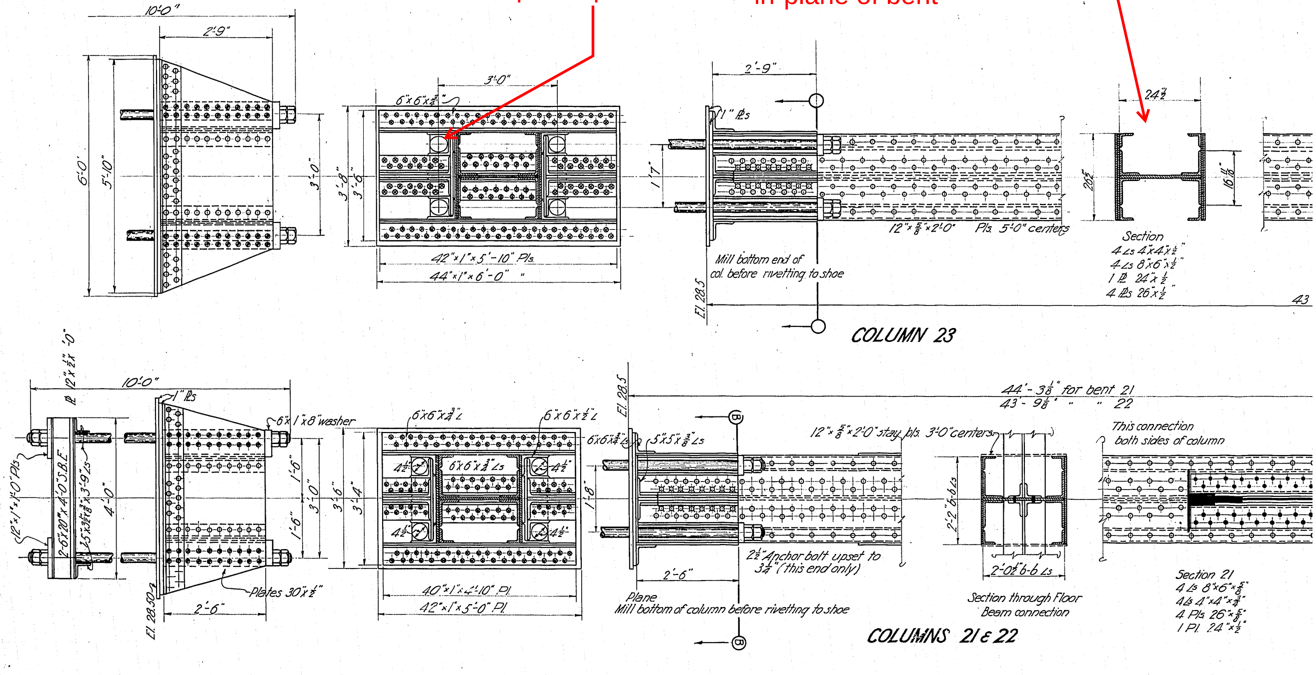


Poor seismic detail. Floorbeam positive moment reinforcement spliced or has limited embedment at column connection thus limiting moment capacity due to inadequate development length.

E3

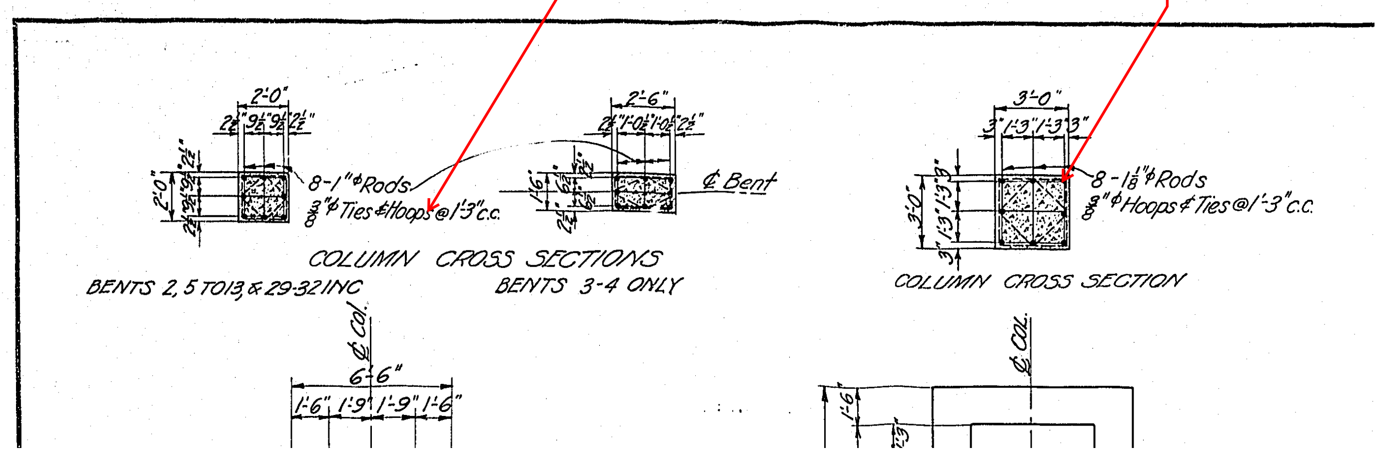
Column poorly anchored to footing pedestal. Anchors do not extend into the pile cap.

Column orientation does not consider seismic induced transverse movement. Limited weak axis flexural strength in-plane of bent

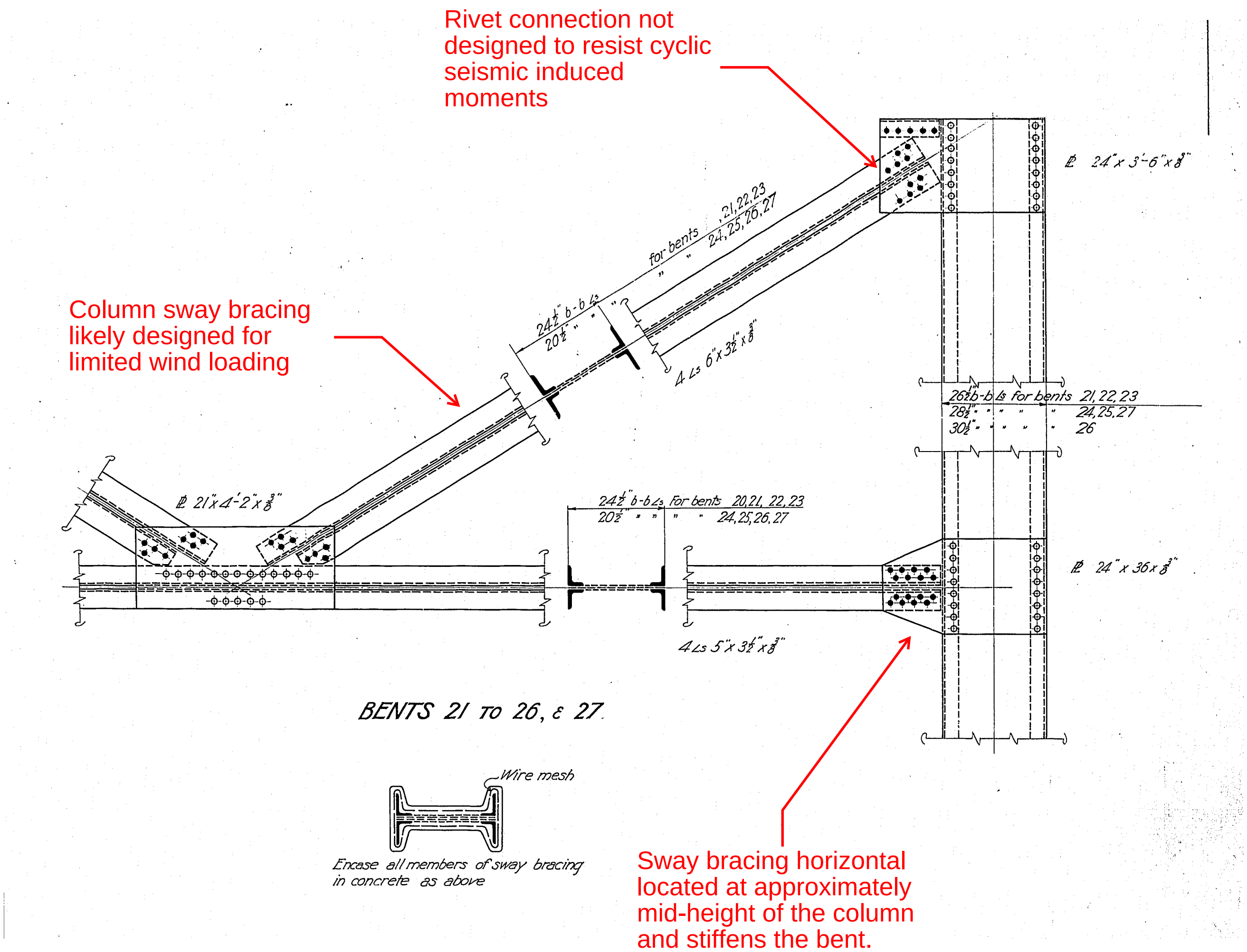


Poor confinement;
Lack of seismic hooks;
Ties and hoops @ 1'-3"

Minimal longitudinal column reinforcing extending into footing. Inadequate development length.

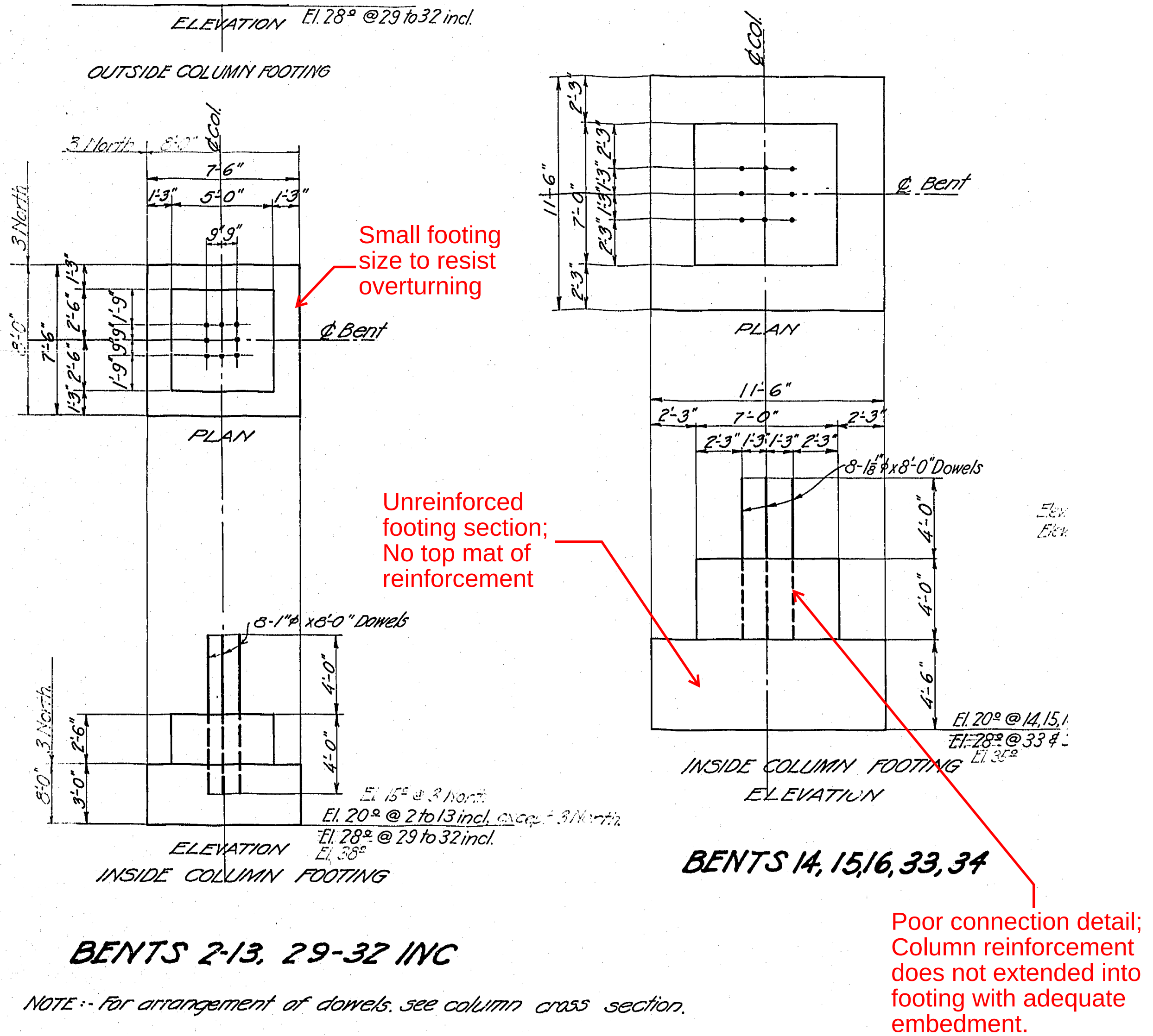


E4



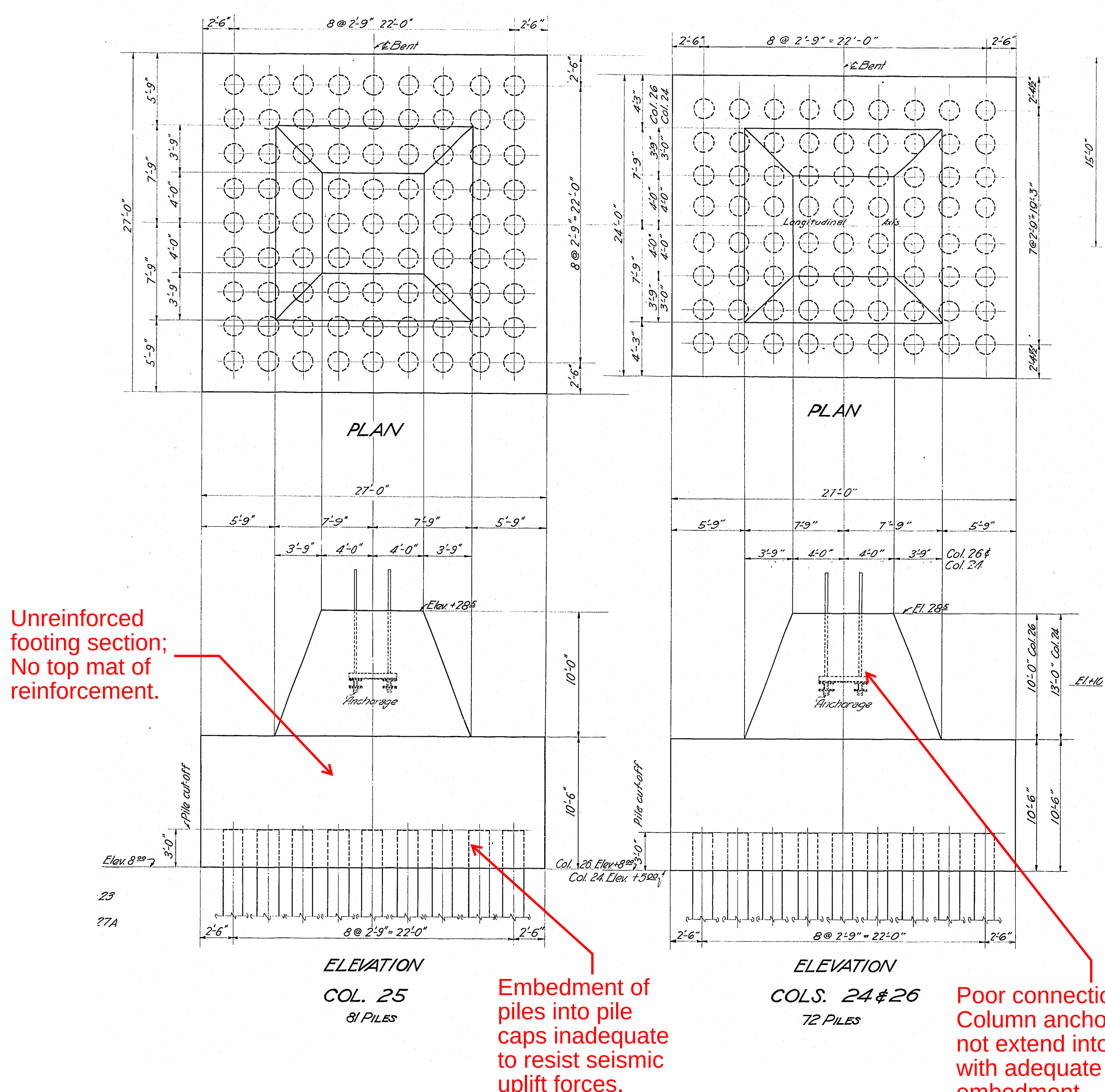
Rivet connection not designed to resist cyclic seismic induced moments
Column sway bracing likely designed for limited wind loading
Sway bracing horizontal located at approximately mid-height of the column and stiffens the bent.
BENTS 21 TO 26, & 27.
Wire mesh
Encase all members of sway bracing in concrete as above

E5



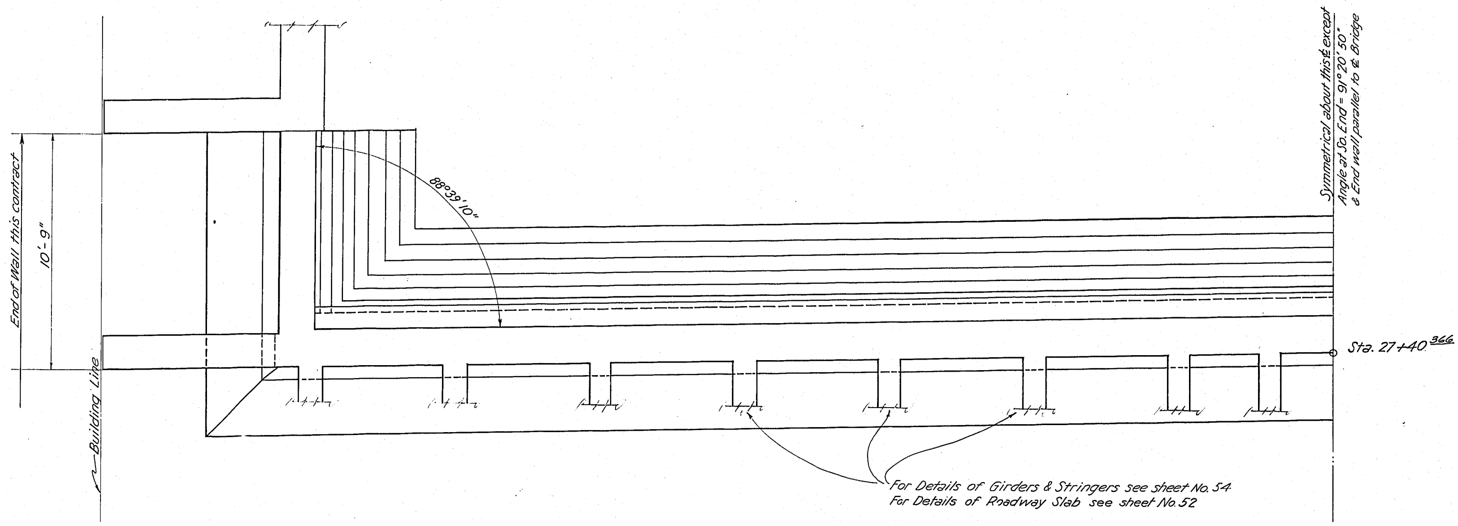
Soil Structure Interaction:
Liquefaction induced settlement and lateral spreading not anticipated at Bents 28-34.

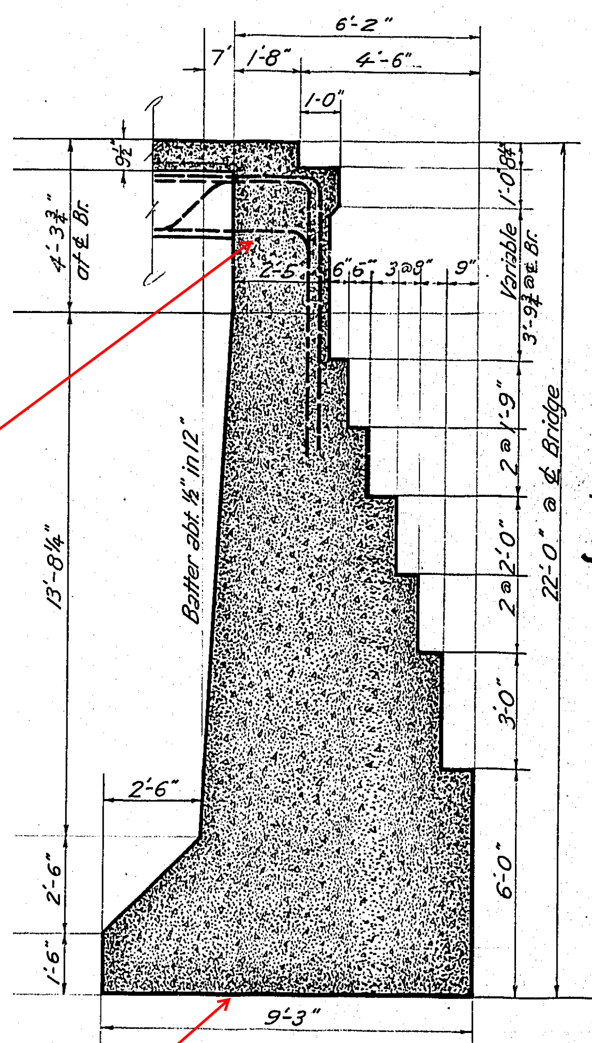
E6



<u>Soil Structure Interaction:</u>
Pile group capacity inadequate to limit settlement from liquefaction to a desirable level, resist uplift and downdrag forces, and to resist displacements and forces from lateral spreading.

E7





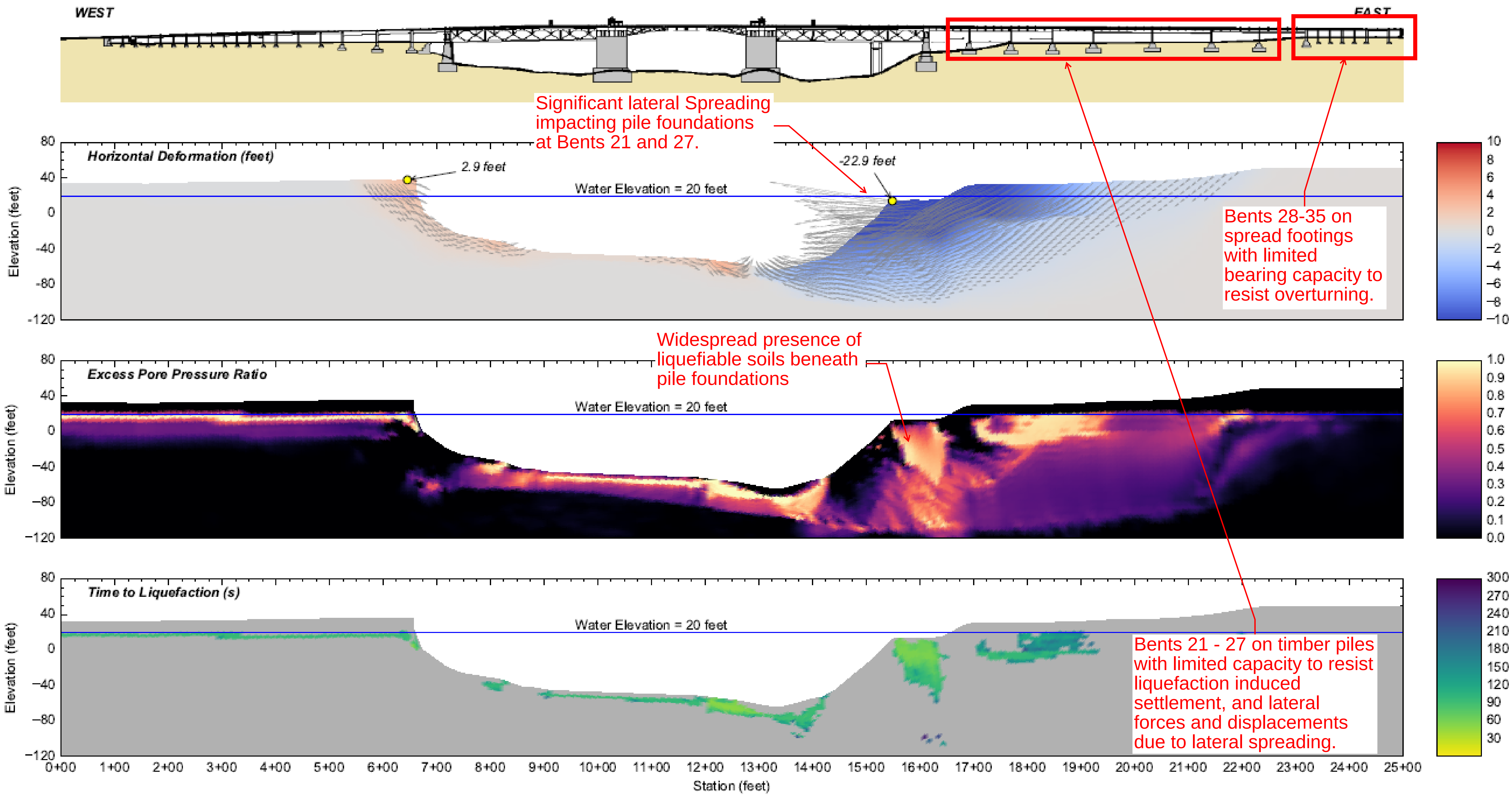
SECTION B-B

Fixed end connection between superstructure and abutment imparts seismic loads on unreinforced abutment wall.

Soil Structure Interaction:
Liquefaction induced settlement and lateral spreading not anticipated at Bent 35.

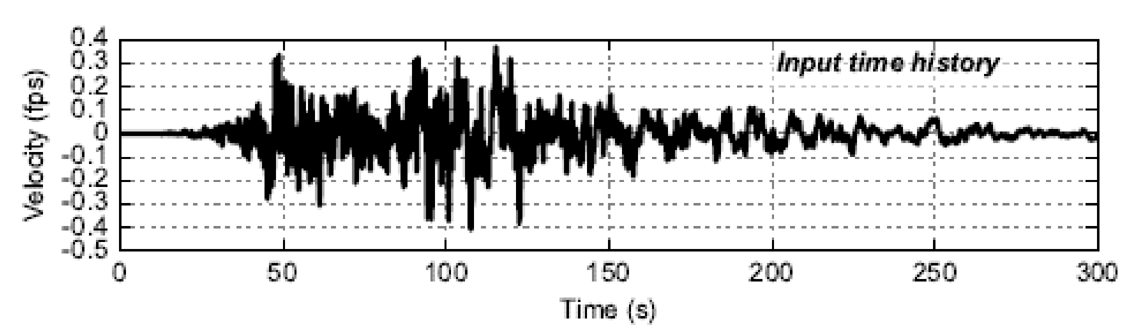
Narrow footing size to resist overturning.

E8



Notes:

1. The input ground motion was recorded during the Tohoku earthquake at the AKT station (east-west component). We scaled the time history to approximate a Cascadia Subduction Zone event.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
Portland, Oregon

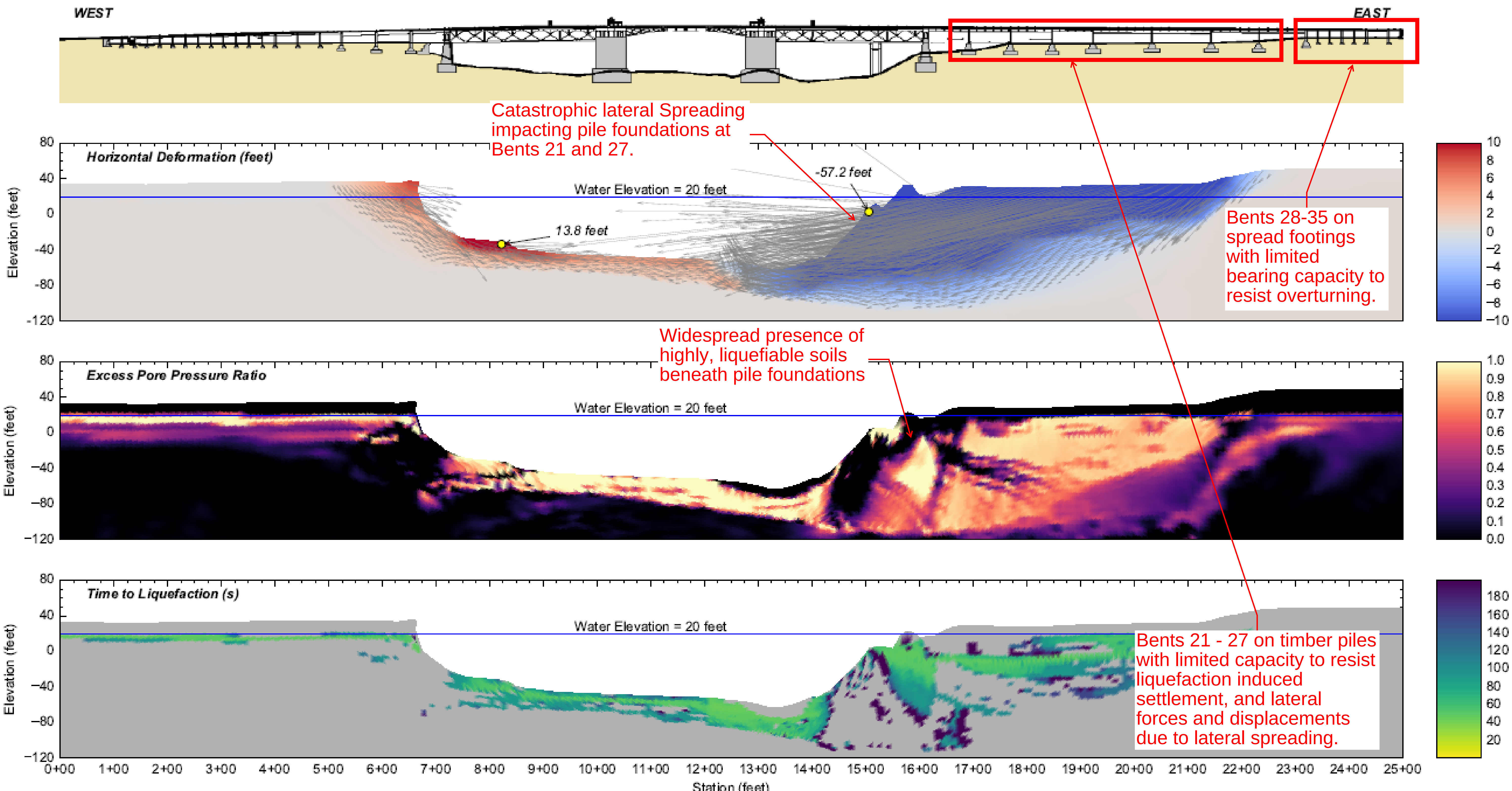
E9
CSZ

MODEL RESPONSE TIME HISTORY T-CSZ

January 2017 | 24-1-04065-003

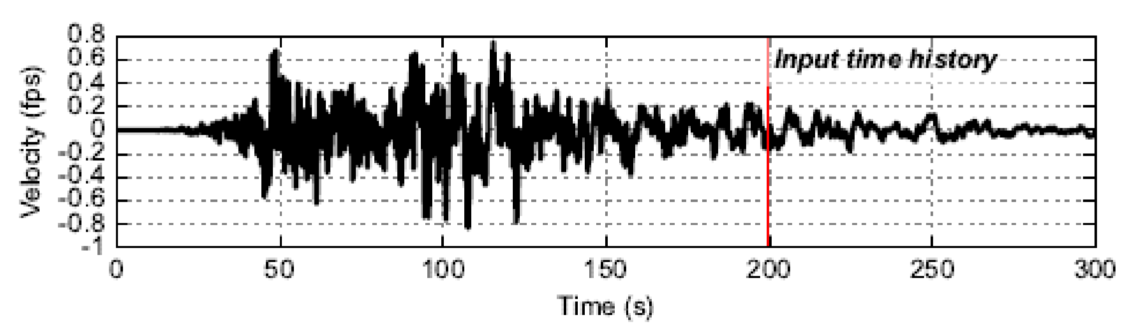
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6



Notes:

1. The input ground motion was recorded during the Tohoku earthquake at the AKT station (east-west component). We scaled the time history to approximate a 1,000 year return period. The model failed due to bad geometry at 199.2 seconds.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
Portland, Oregon

E9
1000-yr

MODEL RESPONSE TIME HISTORY T-1000

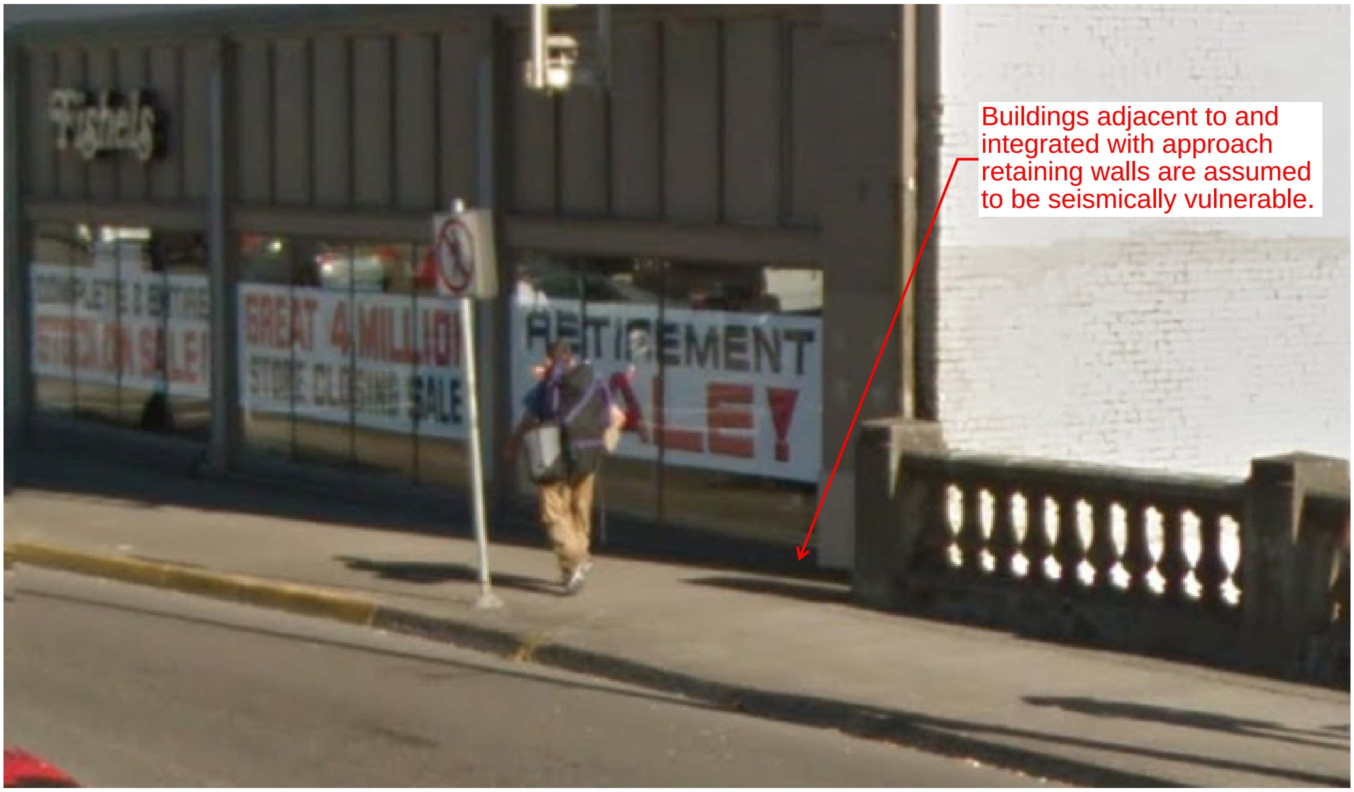
January 2017 24-1-04065-003

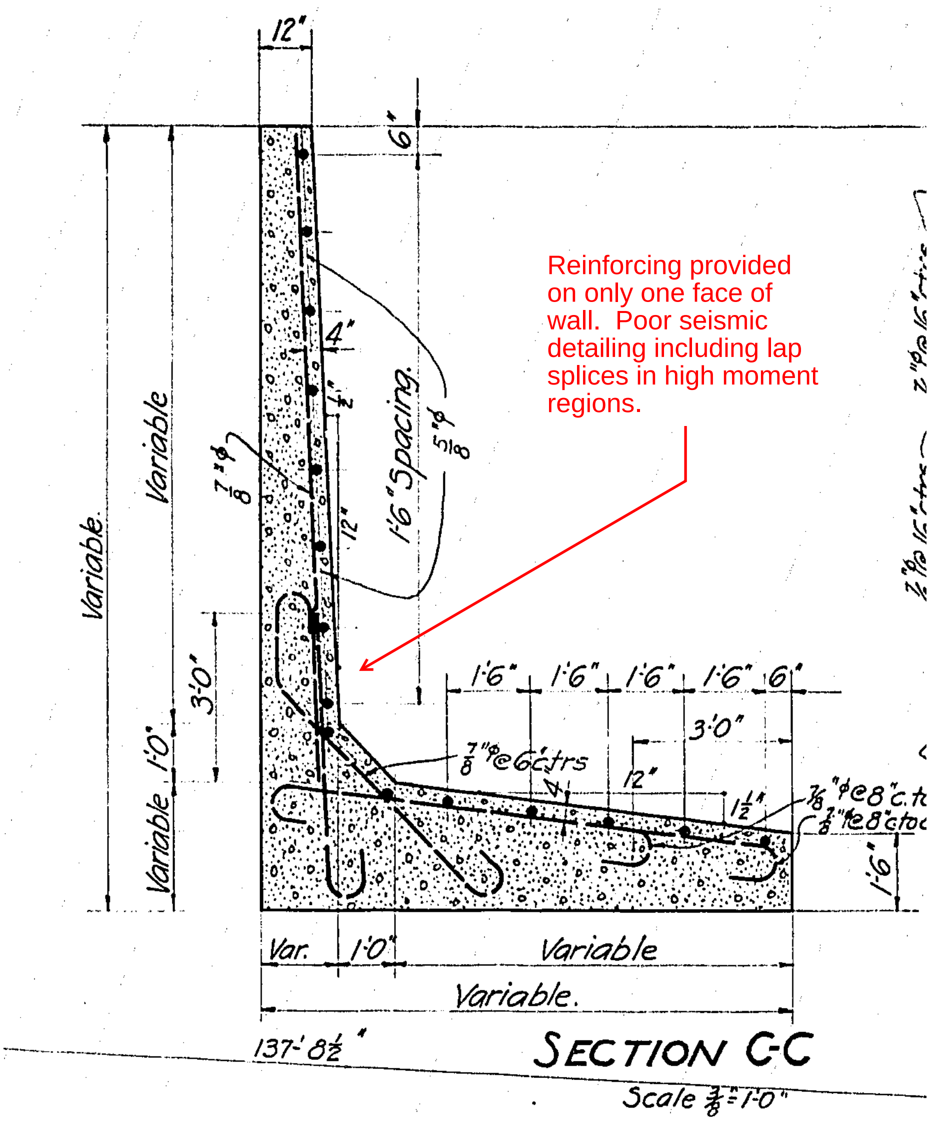
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 2


Burnside Bridge
I-84 ramp to I-5 Southbound adjacent to Burnside Bridge substructure
I-5 Southbound adjacent to Burnside Bridge substructure
I-5 Northbound ramp to I-84 and I-5 Northbound adjacent to Burnside Bridge substructure
I-5 Northbound ramp to I-84 adjacent to Burnside Bridge substructure
Bdwy - Weidler St
I-5

E10





<u>Soil Structure Interaction:</u>
Liquefaction induced settlement and lateral spreading not anticipated at East Approach retaining walls.

E11

Appendix B. EQRB Seismic Design Criteria

Earthquake Ready Burnside Bridge

Seismic Design Criteria

Final Version 2.0
August 23, 2018

TABLE OF CONTENTS

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1.0 INTRODUCTION

Portland is overdue for an unprecedented and catastrophic earthquake that would collapse Multnomah County's historic downtown bridges; leaving the City of Portland divided and isolating members of the community. In response to this, and consistent as designated in the BCC-approved 2015 "Willamette Bridges Capital Improvement Plan" (CIP), making the Burnside Bridge seismically resilient is a top priority for Multnomah County in the next 20 years. The Burnside corridor is one of the City's designated lifeline routes over the Willamette River. Making a wise investment in our lifeline bridge now will help ensure that the region can respond to the earthquake emergency and support the rebuilding of the community. . Multnomah County (County) has taken on the responsibility to seek ways to improve the bridge in order to meet the region's needs for a seismically resilient roadway network connecting to other state lifeline routes. The "Burnside Bridge Earthquake Readiness" study will explore options to create a resilient lifeline crossing that will be a source of pride for our community for generations. One of the primary objectives of the study is to determine the level of seismic vulnerability of the existing bridge, identify the appropriate mitigation measures for these vulnerabilities, and quantify the cost and impacts for constructing these retrofit measures. This action will provide a baseline answer needed to help ensure the long-term safety and viability of the Portland-metro region.

Constructed in 1924-1926, the Burnside Bridge consists of several different types of structural elements and span arrangements. As its most notable feature, this includes a bascule movable span over the navigation channel of the Willamette River, which has a unique set of seismic retrofit design challenges. This existing bridge does not conform to the current seismic design requirements; therefore a seismic retrofit of the existing bridge is anticipated to be necessary. In order to satisfy the long term goal for this bridge designation as the Priority 1 life line across the Willamette River, this study will determine the feasibility of seismically retrofitting the structure to withstand the Cascadia Subduction Zone Earthquake (CSZE) and other designated seismic events.

Developed for Multnomah County's Bridge Division, this "Burnside Bridge Seismic Design Criteria" (Criteria) specifies the minimum seismic design requirements that are necessary to meet the performance goals defined within this Criteria.

The Engineer must exercise judgment in the application of these Criteria. Situations may arise that warrant detailed attention beyond what is provided in the Criteria, including referring to the other design publications for seismic design criteria not explicitly addressed by the Criteria.

2.0 APPLICABLE DESIGN SPECIFICATIONS

The design codes, specifications and guidelines listed below are applicable to this project. These documents are arranged in order of precedence. The provisions in the below codes and standards may be reconsidered if applicable.

1. This “Burnside Bridge Seismic Design Criteria” (*Criteria*)
2. AASHTO LRFD Movable Highway Bridge Design Specifications, 2nd Edition, with 2008, 2010, 2011, 2012, 2014, and 2015 Interim Revisions. (*AASHTO Movable*)
3. AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, with 2012, 2014, and 2015 Interim Revisions. (*Guide Spec*)
4. AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 7th Edition, with 2015 and 2016 Interim Revisions. (*AASHTO LRFD*)
5. ODOT Bridge Design and Drafting Manual (*BDDM*), April 2015 version
6. ODOT Geotechnical Design manual, (*GDM*), November, 2015 version
7. FHWA-HRT-06-032 Seismic Retrofitting Manual for Highway Structures: Part 1- Bridges (*FHWA*)
8. AASHTO Guide Specifications for Design of Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements, 1st Edition
9. AASHTO LRFD Bridge Construction Specifications, 3rd Edition, with 2010, 2011, 2012, 2014, 2015, and 2016 Interim Revisions
10. AASHTO LRFD Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Edition

If the provisions in any of the above-listed codes and standards conflict, the order of precedence is according to their rank. Accordingly, document 1 governs over document 2, document 2 governs over document 3, and so on.

3.0 DESIGN EARTHQUAKES AND PERFORMANCE REQUIREMENTS

3.1 Performance Level Definitions

Multnomah County’s 20 year “Willamette Bridges Capital Improvement Plan” (CIP) defines the following performance objectives for retrofit of existing bridges over Willamette River.

<table>
<tr><td rowspan="4">↓ Decreasing Functionality</td><th>Performance Level</th><th>Abbr.</th><th>Definition</th></tr>
<tr><td>Do Nothing</td><td>DN</td><td>No retrofit measures undertaken</td></tr>
<tr><td>“Full Operation”
(Full Functionality)</td><td>FO</td><td>Essentially elastic for all primary structural components; moveable span remain operable for open and close; Only minimal, superficial repairs and maintenance activities will be required post-earthquake. All traffic modes are able to use the bridge, including river navigation, immediately after the earthquake.</td></tr>
<tr><td>“Limited Operation”
(Limited Functionality)</td><td>LO</td><td>Limited inelastic behavior to substructure components; the bridge allows for emergency vehicles (after inspection and removal of debris). Movable components may not be operable without repairs. Damage is repairable but may impact traffic. Limited permanent deformation may occur.</td></tr>
</table>

3.2 Bridge Operational Category, Performance Requirements, and Ground Motions

Burnside Bridge is designated as the only Priority 1 lifeline route across the Willamette River in downtown Portland. Correspondingly, the bridge classification is “critical” according to AASHTO Movable, Section 3.3, and its Operational Classifications are defined in the table below. Additionally, the design for seismic retrofit of the existing bridge is based on two ground seismic motion levels, as listed in the table below (per BDDM 1.17 and a site-specific analysis).

Category	Full Operation Design Earthquake” (FODE)	Limited Operation Design Earthquake” (LODE)
Designated Performance Level (PL)	Full Operation (FO)	Limited Operation (LO)
Design Level Earthquake (Note 1)	Full rupture of **Cascadian Subduction Zone Earthquake (CSZE)** (Deterministic EQ)	7% probability of exceedance in 75 years (**1,000-year return period**) (Probabilistic EQ)
ODOT Peak Ground Acceleration (PGA) (Note 2)	0.13g	0.27g
Site Specific Acceleration Response Spectra (ARS)	See Appendix A	See Appendix A

Note 1: The FODE and LODE level ground motions shall be characterized by Acceleration Response Spectra (ARS) that correspond to the site subsurface conditions and include near-fault effects as appropriate.

Note 2: For structural applications, seismic demand is represented using an elastic 5% damped response spectrum.

3.2.1 Full Operation (FO) Performance Requirement for the Full Operation Design Earthquake (FODE):

For Full Operation Design Earthquake with full rupture of Cascadian Subduction Zone Earthquake (CSZE), the Performance Level is Full Operation, (FO). The FO performance level requires that after the design level earthquake:

- The bridge remains elastic for all main structural components.
- Moveable spanis able to be operated
- Bridge can be open to all traffic modes on the bridge deck immediately.

A winch system may be required to operate the bascule span leaves while the bascule span operation system is being repaired immediately after the earthquake. Spare parts for critical machinery and electrical components shall be prefabricated and stored on-site or near the bridge for emergency repair, it is expected that the time window for necessary repairs is up to 2 weeks.

Except for the bridge operating machinery, only non-structural repairs are expected. This may include limited concrete cover spalling on structural elements, small cracks on structural elements, or more significant cracks in non-structural concrete elements.

Quantitative definition for permissible displacements that allow the structure to meet the Full Operation performance requirements at key locations shall be studied in the NEPA/Preliminary engineering phase. During the conceptual engineering study, the following permissible displacements are targeted:

- Relative vertical displacements between the bascule leaf cantilever tips: 3 inches
- Differential settlement between the roadway approaches and the bridge will be limited to approximately 0.5 inches.

3.2.2 Limited Operation (LO) Performance Requirement for the Limited Operation Design Earthquake (LODE):

After designing a seismic retrofit for the Full Operation Design Earthquake, check the retrofitted bridge for a Limited Operation Design Earthquake with an earthquake event of a 1000-year return period. For this check, the Performance Level is Limited Operation, (LO). The LO performance requirements exceed the “no-collapse” criteria requirements in applicable design specifications listed in Section 2 in addition to this document, for the 1000-year return period. To meet this performance criteria, this bridge should be repairable with a possible restriction to traffic flow (FHWA 1.4.1, PL2). Moveable span may not operable without inspection or repairs to its components. LO criteria shall be defined as follows:

- The bridge allows for emergency vehicles to pass over the bridge (after inspection and removal of debris).
- For foundation and column members, limit permanent deformation of 50% of their individual displacement capacity.
- For capacity protected structural members, no inelastic deformations are allowed. This includes superstructure, bent cap, crossbeam, trunnion tower, and counterweight supporting members.
- Except for movable operations, the time window for any necessary repairs is up to 2 weeks before opening the bridge to all traffic.
- For movable operations, the time window for any necessary repairs is up to 2 months before opening the bridge to ship navigation traffic.

Damage sustained by the structural shall be minimal, some of the possible damages are described below.

- *Minimal damage* may include minor inelastic response and narrow flexural and shear cracking in concrete. Permanent deformations are not apparent and repairs can be made under non-emergency conditions (with the possible exception of superstructure expansion joints which may need removal and temporary replacement). (*FHWA 1.4.1*)
- Certain elements may be permitted to fuse, provided it can be shown that such an occurrence will not reduce the vertical load-carrying capacity of the bridge or lead to superstructure unseating, and that the fusing of these elements will not preclude the structure from meeting the Limited Operation performance requirements.
- Limited differential settlement between the bridge and approach roadways and roadway fill may be allowed.

Quantitative definition for permissible displacements that allow the structure to meet the Limited Operation performance requirements at key locations shall be studied in the NEPA/Preliminary engineering phase. During the conceptual engineering study, the following permissible displacements are targeted:

- Relative vertical displacements at the bascule leaf cantilever tips: 6 inches

3.3 Design Response Spectrum

For this bridge, site specific seismic design acceleration response spectra (ARS) were developed by Shannon & Wilson, project geotechnical engineer for this project, according to design specifications listed in Section 2, and the tools, source information and procedures specified in those specifications. See Appendix A “Project Site Specific Seismic Hazard Maps”.

The complex geotechnical profile changes significantly along this bridge. The site specific ARS developed by Shannon & Wilson are divided into three zones: Zone 1 from bent 1 to bent 18, Zone 2 from bent 19 to bent 27, and Zone 3 from bent 28 to bent 35. An envelope generalized ARS curve was developed for each of the three zones, for both CSZ spectra and 1000-year spectra, as shown in Appendix A.

To simplify for conceptual engineering analysis, the generalized ARS curves of the three zones were further enveloped to become a single generalized envelope for the entire bridge from bent 1 to bent 35. Each of the single generalized spectra of the CSZ spectra and 1000-year spectra, was further compared to ODOT CSZ and 1000-year site Class E spectra correspondingly. Finally for each of the CSZ and 1000-years seismic events, a recommended site specific design response spectra was produced based on the single generalized spectra, and adjusted for the consideration of the correspondence ODOT seismic ARS curve.

Since the entire bridge structures are connected, and are analyzed in an integrated model for the conceptual engineering analysis, a slightly higher ARS curve between bent 28 to bent 35 under the 1000-year spectra, was removed from the recommended site specific design response spectra, for a better approximation of the real condition.

3.4 Geologic Hazard Considerations

Liquefaction and lateral spreading potential of foundation soils will be evaluated and determined by the project geotechnical engineer. If the foundation soils are predicted to liquefy, the effects of liquefaction on design and performance evaluation should be according to ODOT Bridge Design and Drafting Manual (BDDM) Section 1.10.5 and 1.17.4.

Because of the more stringent performance requirements for this bridge than the required performance level stated in BDDM, the acceptable lateral deformations of the approach fills described in BDDM Section 1.17.4, Note 1 and Note 2 are modified, targeted maximum acceptable lateral deformations of the foundation soil are:

- Full Operation Design Earthquake (FODE): 6 inches
- Limited Operation Design Earthquake (LODE): 12 inches

3.5 Bridge Seismic Retrofit Categories

Four seismic retrofit categories for the Burnside Bridge will be considered. They are:

- Phase 1 Retrofits,
- Phase 2 Retrofits,
- Localized Seismic Load Path Retrofits, and
- System Behavior Modification Retrofits.

For the purpose of achieving the Seismic Performance Levels for the Burnside Bridge, all of these retrofit categories shall be assessed.

3.5.1 Phase 1 Retrofits

Phase 1 retrofits are intended to keep the superstructure from becoming disconnected from the substructure and collapsing. Phase 1 retrofits strengthen the first part of the primary load path, the connection of the superstructure to the substructure. These retrofits by themselves do not provide a complete seismic load path transfer mechanism but instead provide a relatively inexpensive solution to one common seismic failure mode. Typical Phase 1 retrofits for Burnside Bridge may include:

- Replacement of failure prone bearings
- Extension of bearing seats
- Lateral and longitudinal restraint using cable restrainers or shear blocks
- Strengthening of girder bracing and diaphragms at the bearings
- Live load shoe or anchor strut modification to provide longitudinal restraint

The Phase 1 Seismic Retrofit is **not** expected to achieve either the Full Operability or Limited Operability performance levels for any of the anticipated design level earthquake events.

3.5.2 Phase 2 Retrofits

Phase 2 retrofits typically involve substructure (columns, footings and foundations) ductility enhancement and strengthening that is intended to provide a load path for anticipated seismic forces with adequate ductility from the superstructure to the ground. These retrofits typically have significant costs; but together with Phase 1 retrofits, provide a complete seismic retrofit solution for typical girder type bridges, such as the east and west approach spans of the Burnside Bridge. Typical Phase 2 retrofits for Burnside Bridge may include:

- Encasement of columns using steel, concrete, or FRP to provide increased ductility and lateral strength
- Encasement and/or post-tensioning of bent caps and footings to provide adequate joint strength
- Footing enlargement and/or additional pile installation

Any additional or deferred Phase 1 Retrofit work would also be included. For regular bridges, the objective of Phase 1 + Phase 2 Retrofits is a retrofitted bridge with seismic performance near a new bridge at the site.

3.5.3 Localized Seismic Load Path Retrofits

The truss spans and bascule span of the Burnside Bridge require component retrofits that fall outside of the typical Phase 1 and Phase 2 retrofit types. These are typically component retrofits unique to the specific structure. They are designed to provide either a strengthened load path or to limit displacements of some bridge elements. The types of retrofits in this category may include:

- Strengthening of truss lateral bracing including counterweight braces

- Strengthening of trunnion supports
- Introduction of strengthening shear walls

3.5.4 Seismic Behavior Modification Retrofits

Each of the prior retrofit types targeted specific components of the bridge. This last retrofit type is intended to change the seismic response (behavior) of the bridge by modifying its structural system. These may include both base isolation or friction damped bracing systems that modify the bridge's dynamic behavior in a seismic event. This is accomplished by not only changing the capacity of specific components, but the load and displacement demands that the bridge system undergoes during a seismic event.

A base isolation system works by providing a displacement isolation layer coupled with an energy absorbing damping device between components of the bridge. The intent of these measures is to limit the amount of seismic energy that is transferred from the ground into the structure. For Burnside Bridge, this may involve isolating the superstructure from the piers, or accommodating the superstructure relative movements to the substructures, therefore to reduce the force demands in the superstructure and substructure, while increasing the relative displacements between the superstructure and substructure. A modified gap or joint will also be necessary to accommodate this increased movement.

As a consequence of base isolation, due to the higher displacement incurred, there are usually additional bridge components that require retrofit. These include deck joints and other movement limiting elements. This is a particular challenge for the bridge moveable span because large displacements, which are essential for absorbing energy within the base isolation systems, can negatively impact the operating machinery. As such, the system modification devices must be designed in a manner that can achieve the bridge's operability performance goal following a seismic event.

3.6 Seismic Retrofit Service Life Expectation

Bridge seismic retrofit alternatives shall be designed for a **100 year** Design Life.

3.7 Future Maintenance Considerations

Routine bridge maintenance activities are anticipated throughout the bridge's anticipated Design Life. Bridge retrofit alternatives shall minimize impacts to routine maintenance activities. If impacts cannot be avoided, new maintenance accommodations shall be included with the retrofit.

3.8 Future Improvement Considerations

For the seismic retrofit only or the retrofit + widening options, no additional dead load allowance to account for future improvements to the bridge, such as future wearing surface etc.

3.9 Navigational Channel Clearances

The bridge retrofit alternatives shall maintain the minimum clearance requirements of the Willamette River navigational channel, as shown in Appendix C. The clearance requirements shown in the appendix are for concept development only. The clearance requirements are subject to review and approval by the agencies that have jurisdictions in these spaces.

3.10 TriMet Light Rail Clearances

The bridge retrofit alternatives shall maintain the minimum horizontal and vertical clearance requirements over the TriMet Light Rail tracks at SW 1st Ave, as shown in Appendix D. The clearance requirements shown in the appendix are for concept development only. The clearance requirements are subject to review and approval by the agencies that have jurisdictions in these spaces.

3.11 ODOT Facility Clearances

The bridge retrofit alternatives shall maintain the minimum horizontal and vertical clearance requirements over the ODOT facilities of I-5 and I-84, as shown in Appendix E. The clearance requirements shown in the appendix are for concept development only. The clearance requirements are subject to review and approval by the agencies that have jurisdictions in these spaces.

3.12 City of Portland Facility Clearances

The bridge retrofit alternatives shall maintain the minimum horizontal and vertical clearance requirements over the City of Portland facilities at Naito Parkway, SE 2nd Ave, SE 3rd Ave, and Waterfront Park (Ankeny Pump Station and seawall), as shown in Appendix F. The clearance requirements shown in the appendix are for concept development only. The clearance requirements are subject to review and approval by the agencies that have jurisdictions in these spaces.

3.13 Union Pacific Railroad (UPRR) Clearances

The bridge retrofit alternatives shall maintain the minimum horizontal and vertical clearance requirements within the UPRR ROW, as shown in Appendix G. The clearance requirements shown in the appendix are for concept development only. The clearance requirements are subject to review and approval by the agencies that have jurisdictions in these spaces.

3.14 Private Building Impacts

The bridge retrofit alternatives shall attempt to minimize any conflicts with private buildings at locations shown in Appendix H. The clearance requirements shown in the appendix are for concept development only. The clearance requirements are subject to review and approval by the agencies that have jurisdictions in these spaces, and the agreement with the building owners.

4.0 LOAD AND LOAD COMBINATIONS

4.1 Load Factors and Combination

New and retrofitted bridge components shall be designed for the applicable load combinations in accordance with the requirements of AASHTO LRFD.

The load effect shall be obtained by:

$$\text{Load Effect} = \sum \eta\, \gamma_i\, Q_i$$

Where:

Q_i = force effect

η = a load modifier relating to ductility, redundancy, and operational importance

γ_i = load factor corresponding to Q_i

The load modifiers shall be according to AASHTO LRFD, Section 1.3.3, 1.3.4, and 1.3.5

The load factors shall be according to AASHTO LRFD, Section 3.4

4.2 Dead Loads

Dead load shall include the weight of all components of the structure, railing, appurtenances and utilities attached thereto, earth cover, wearing surface, future overlays, and planned widening (if applicable).

- Load factors for all permanent loads γ_p shall be according to AASHTO LRFD, Tables 3.4.1-1, 3.4.1-2, and 3.4.1-3, for Extreme Event I load combination (seismic analysis) $\gamma_p = 1.0$

4.3 Live Loads

Live loads on the structure shall be included in the analysis according to AASHTO LRFD,

- Load factor for live load in Extreme Event I load combination for the FODE shall be $\gamma_{EQ} = 0.25$
- Load factor for live load in Extreme Event I load combination for the LODE shall be $\gamma_{EQ} = 0.10$

The weight or equivalent mass due to live loads on the structure shall NOT be included in the seismic analysis.

4.4 Earthquake Load EQ for Bascule Span

4.4.1 Fixed Spans

The earthquake load – ground motions and response spectra shall be considered for the FODE and LODE ground motions.

- Load factor for earthquake loads EQ, shall be $\gamma_{EQ} = 1.0$

4.4.2 Bascule Span in Closed Position

The same earthquake load as for the fixed span applies. In addition, for Extreme Event I load combination (AASHTO LRFD, Table 3.4.1-1), a combined vertical seismic acceleration and horizontal seismic acceleration analysis is required for both LODE and FODE ground motions (AASHTO Movable, 3.4.1).

4.4.3 Bascule Span in Open Position

The same earthquake load as for the fixed span applies. In addition, for Extreme Event I load combination (AASHTO LRFD, Table 3.4.1-1), a combined vertical seismic acceleration and horizontal seismic acceleration analysis is required for both LODE and FODE ground motions (AASHTO Movable, 3.4.1).

- For LODE ground motion, load factor for EQ $\gamma_{EQ} = 0.5$
- For FODE ground motion, load factor for EQ $\gamma_{\gamma_{EQ}} = 1.0$

4.5 Earthquake Load in Two or Three Orthogonal Directions

When combining the responses of two or three orthogonal directions, the design value of any quantity of interest (displacement, bending moment, shear or axial force) should be obtained either by the 'square root of the sum of the squares' (SRSS) rule or the 100-30 percent combination rule.

The SRSS rule is the most appropriate one for combining the contribution of orthogonal and uncorrelated ground motion components into a single design force. The method is especially recommended if the vertical components of the ground motion are being used in combination with the horizontal components (Button et al., 1999). However, the familiar 100-30 percent combination rule is also suitable.

These two alternative combination rules are summarized as follows.

Method 1, 100-30 percent Combination Rule: The design value is obtained from the largest value given by the following three load cases.

- Load Case 1 (LC1): 100 percent of the absolute value of the response resulting from the analysis in one orthogonal direction (transverse) is added to 30 percent of the responses resulting from analyses in the other two orthogonal directions (longitudinal and vertical):
- Load Case 2 (LC2): 100 percent of the absolute value of the response resulting from an analysis in one of the other orthogonal directions (longitudinal) added to 30 percent of the responses resulting from analyses in the other two orthogonal directions (transverse and vertical):
- Load Case 3 (LC3): 100 percent of the absolute value of the response resulting from an analysis in one of the other orthogonal directions (vertical) added to 30 percent of the response values resulting from analyses in the other two orthogonal directions (transverse and longitudinal):

The response to be used in design is given by the largest value from these three load cases,

i.e. $M_x = \text{maximum of } [\, M_x^{LC1}, M_x^{LC2}, M_x^{LC3}]$

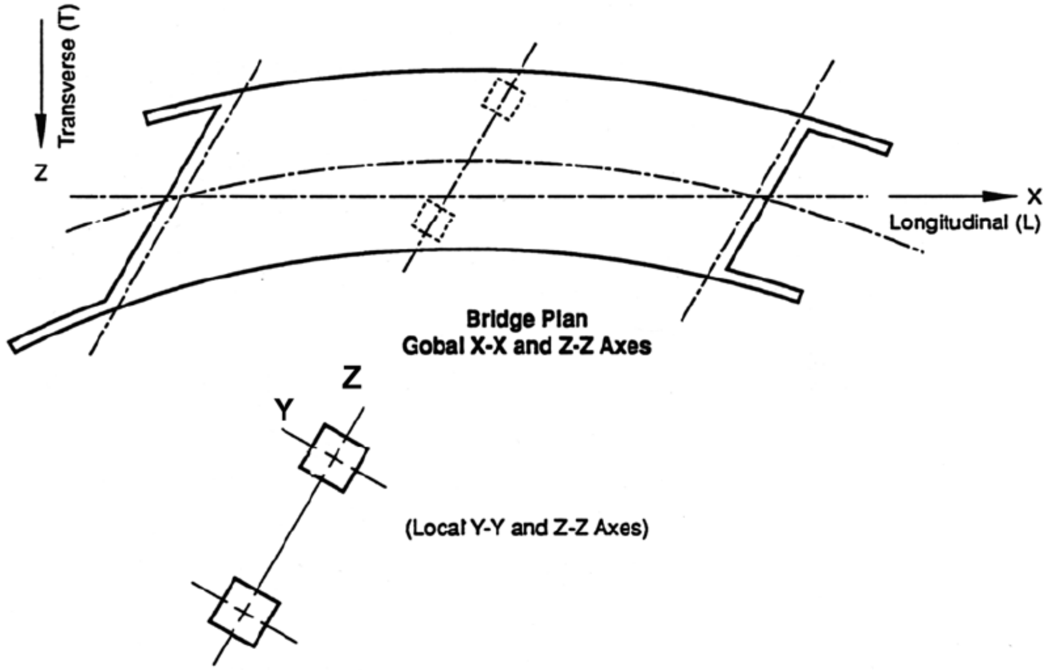
Method 2, SRSS Combination Rule: The design value is the SRSS combination of the response quantity from each of the orthogonal directions:

$$M_x = \sqrt{(M_x^T)^2 + (M_x^L)^2 + (M_x^V)^2}$$

Where M_x^T , M_x^L , M_x^V are the x-components of moment calculated from a transverse, longitudinal and vertical analysis, respectively.

4.6 Earthquake Load Combination – Fixed Spans

A combined vertical/horizontal load analysis is NOT required for the fixed spans. Earthquake effects shall be determined from a horizontal acceleration response spectrum applied by either of the methods described in section 4.5 above. If the seismic loading is applied in only two orthogonal directions (longitudinal and transverse), the above methods 1 or 2 still apply, but Mx^V is set equal to zero, and Load Case 3 need not be calculated, LC3 = 0.



4.7 Earthquake Load Combination – Bascule Span

A combined vertical/horizontal load analysis is required for the bascule span for the FODE and LODE, ground motions. Earthquake effects shall be determined from a horizontal and vertical acceleration response spectrum applied by either of the methods described in section 4.5 above.

4.8 Bascule Span Operating System

Design according to AASHTO Movable, Section 3.4.3.

5.0 STRUCTURAL MATERIALS – EXISTING / NEW

5.1 Existing Materials

5.1.1 Concrete

For normal weight Portland cement concrete, the properties are calculated using the equations below.

Modulus of Elasticity, $E_c = 33 \times w^{1.5} \times \sqrt{f'_{ce}}\ (psi)$

Where w = unit weight of concrete in lb/ft3. For w = 145 lb/ft3

Shear Modulus, $G_c = \frac{E_c}{2\times(1+v_c)}$

Poisson's Ratio, $v_c = 0.2$

Expected concrete strength shall be taken as the most probable long-term concrete strength based on regional experience to account for typical conservative batching practice and strength gain with age. In the absence of regional test data, the analytical expected concrete compressive strength will be calculated by multiplying the original specified 28-day compression strength by a factor of 1.3. The bridges of interest are sufficiently old to justify consideration for strength gain through aging. (Guide Spec 8.4.4)

Expected concrete compressive strength, $f'_{ce} \geq 1.3 f'_c$

Where $f'_c = 2.5$ ksi for the original concrete structural elements as stated on the as-built plans,

or $f'_c = 3.5$ ksi for concrete from the 2001 Phase 1 seismic retrofit project,

or $f'_c = 4.35$ ksi for deck concrete and $f'_c = 3.6$ ksi for all other concrete from the 2005 Main Span rehabilitation project,

or $f'_c = 4.0$ ksi for concrete from the 2017 maintenance and painting project.

For additional concrete modeling properties, such as limits on unconfined concrete compression strain and ultimate compressive strain for confined concrete using Mander's model, see Section 6.3.2.

5.1.2 Reinforcing Steel

Existing reinforcing steel placed during the original construction shall use a yield stress f_y = 33 ksi, as stated on the original as-built plans. Reinforcing placed in subsequent rehabilitation projects shall use a yield stress of f_y = 60 ksi.

Prestressing steel rods used as restrainers in the 2001 Phase 1 seismic retrofit project consist of A449 steel.

5.1.3 Structural Steel

Structural steel properties according to as-built plans.

5.2 New Materials

5.2.1 Concrete

New concrete shall have a minimum specified 28-day compressive strength f´c of 4 ksi. The analytical expected 28-day strength f'_{ce} shall be taken as 1.3 x f´c. Other concrete material properties should be calculated using the equations specified in Section 5.1.1 above

5.2.2 Reinforcing Steel

New retrofit reinforcing steel properties shall be as provided in the following table (Guide Spec 8.4.2):

Property	**Notation**	**Bar Size**	**ASTM A706**	**ASTM A615 Grade 60**	
Modulus of elasticity (ksi)	E_s	No. 3 – No. 18	29,000	29,000	
Specified minimum yield stress (ksi)	f_y	No. 3 – No. 18	60	60	

Expected yield stress (ksi)	f_{ye}	No. 3 – No. 18	68	68	
Specified tensile strength (ksi)	f_u	No. 3 – No. 18	80	90	
Expected tensile strength (ksi)	f_{ue}	No. 3 – No. 18	95	95	
Nominal yield strain	ε_y	No. 3 – No. 18	0.0021	0.0021	
Expected yield strain	ε_{ye}	No. 3 – No. 18	0.0023	0.0023	
Onset of strain hardening	ε_{sh}	No. 3 – No. 8	0.0150	0.0150	
		No. 9	0.0125	0.0125	
		No. 10 & No. 11	0.0115	0.0115	
		No. 14	00075	0.0075	
		No. 18	0.0050	0.0050	
Reduced ultimate tensile strain	$\varepsilon_{su}{}^R$	No.4 – No. 10	0.090	0.060	
		No. 11 – No. 18	0.060	0.040	
Ultimate tensile strain	ε_{su}	No.4 – No. 10	0.120	0.090	
		No. 11 – No. 18	0.090	0.060	

5.2.3 Prestressing Steel

New retrofit prestressing steel shall conform to 270 ksi low relaxation strand.

Prestressing steel can be modeled with an idealized nonlinear stress strain model. The figure below is an idealized stress-strain model for 7-wire low-relaxation prestressing strand. The curves in the figure below can be approximated by the equations below. *(Guide Spec 8.4.3)*

Essentially elastic prestress steel strain $\varepsilon_{ps,EE} = \begin{cases} 0.0075\ for\ f_u = 250\ ksi \\ 0.0086\ for\ f_u = 270\ ksi \end{cases}$

Reduced ultimate prestress steel strain $\varepsilon^R_{ps,u}$ = 0.03

<u>250 ksi Strand</u>

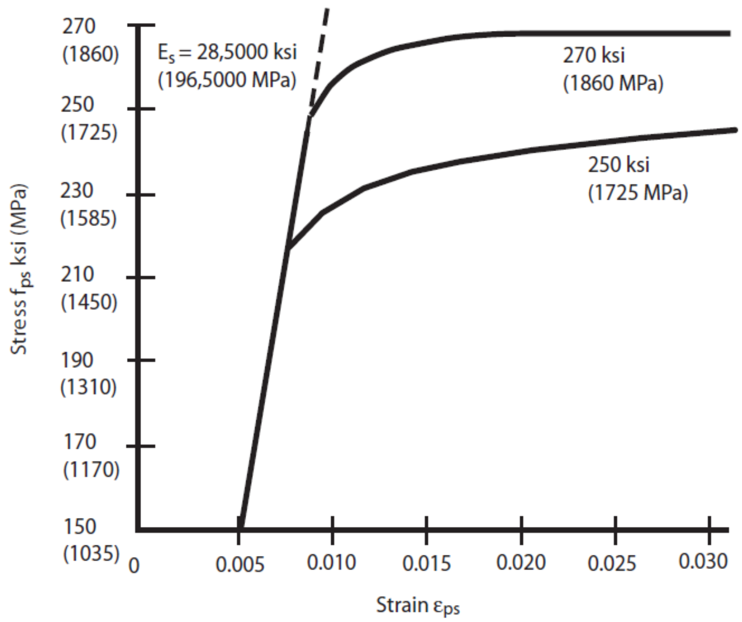
$$\varepsilon_{ps} \leq 0.0076: f_{ps} = 28{,}500 \times \varepsilon_{ps} \ \ (ksi)$$

$$\varepsilon_{ps} \geq 0.0076: f_{ps} = 250 - \frac{0.25}{\varepsilon_{ps}} \ \ (ksi)$$

<u>270 ksi Strand</u>

$$\varepsilon_{ps} \leq 0.0086: f_{ps} = 28{,}500 \times \varepsilon_{ps} \ \ (ksi)$$

$$\varepsilon_{ps} \geq 0.0086: f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \ \ (ksi)$$



5.2.4 Structural Steel

For new structural steel retrofit elements that are expected to remain elastic under the FODE and LODE, events, the following properties shall be used:

- Structural steel conforming to ASTM A709, Grade 50 or 50W shall be evaluated based upon a nominal yield strength Fy of 50 ksi and a nominal tensile strength Fu of 65 ksi.
- Structural steel conforming to ASTM A709, Grade36 shall be evaluated based upon a nominal yield strength Fy of 36 ksi and a nominal tensile strength Fu of 58 ksi.
- Structural HSS shapes shall conform to ASTM A500, Grade B and shall be evaluated based upon a nominal yield strength Fy of 46 ksi for shaped tubes and 42 ksi for round tubes, and a nominal tensile strength Fu of 58 ksi, regardless of cross-sectional shape.

For new structural steel retrofit elements that are permitted to behave in a ductile manner under the LODE event, expected yield strengths shall be used to determine connection and other capacity-protected member force demand. Expected yield strengths shall be calculated by factoring the nominal yield strengths denoted above in accordance with Guide Spec 7.3.

6.0 DETERMINATION OF DEMANDS & CAPACITIES

6.1 Analysis Objective

The objective of seismic analysis is to assess the force and deformation demands and capacities on the structural system and its individual components. Linear elastic dynamic analysis through Response Spectrum Analysis is an analytical tool for estimating the force and displacement demands for the bridge. Inelastic static pushover analysis is an analytical tool to establishing the displacement capacities for the bridges where applicable.

6.2 Demands

6.2.1 Demand Analysis Method

Estimate the earthquake demands by using the elastic Response Spectrum Analysis (RSA) method, as described in Section 6.4.

Method for Estimate Earthquake Demands		
Demands	**Full Operation Design Earthquake (FODE)**	**Limited Operation Design Earthquake (LODE)**
Displacement	RSA	RSA
Force & Moment	RSA	RSA

6.2.2 Demand in Capacity Protected Elements

When evaluating the existing structure, use the lesser of elastic demands or overstrength demands. When designing capacity protected retrofit elements, use the overstrength demands. For determination of force demands in capacity protected elements, expected material properties are to be used. Overstrength factor is required as specified in Section 6.3.5.

However, the column forces reported by the RSA are founded on the assumption that all components of the model remain elastic and the RSA models cannot properly account for nonlinear behavior and column plastic hinging. Therefore the column moments reported from the RSA are often artificially high. Instead, where reinforcement detail condition allows, the column maximum plastic moments obtained from the moment-curvature analysis shall be used to calculate force and moment demands for capacity-protected members, such as superstructure, crossbeams, and foundations. See Section 6.3.3 for moment-curvature analysis.

Alternatively, the demand forces and moments in the capacity-protected members may be determined by reading the final forces and moments from a pushover analysis in which the columns have hinged. See Section 6.5 for Pushover analysis.

6.3 Capacities

The capacities of the bridge globally and locally are generally independent of the ground motion. (One exception to this is that column flexural strength is dependent on the axial load in the column, which varies with the lateral loads induced by ground motions.) The section below lists some common seismic capacity evaluations. For capacity to demand acceptance criteria and other performance acceptance criteria see Section 7.

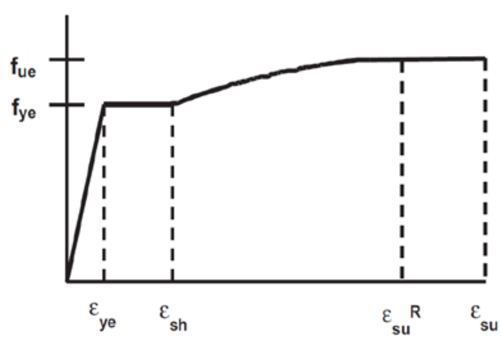
6.3.1 Expected Versus Nominal Material Properties

The capacity of concrete components to resist all seismic demands shall be based on the most probable (expected) material properties to provide a more realistic estimate for design strength. An expected concrete compressive strength, f'_{ce} , recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. The yield stress f_y for ASTM A706 steel can range between 60 ksi and 78 ksi. An expected reinforcement yield stress, f_{ye} , is a "characteristic" strength and better represents the actual strength than the specified minimum of 60 ksi. The possibility that the yield stress may be less than f_{ye} in ductile components will result in a reduced ratio of actual plastic moment strength to design strength, thus conservatively impacting capacity-protected components. Expected material properties shall only be used to assess capacity for earthquake loads.

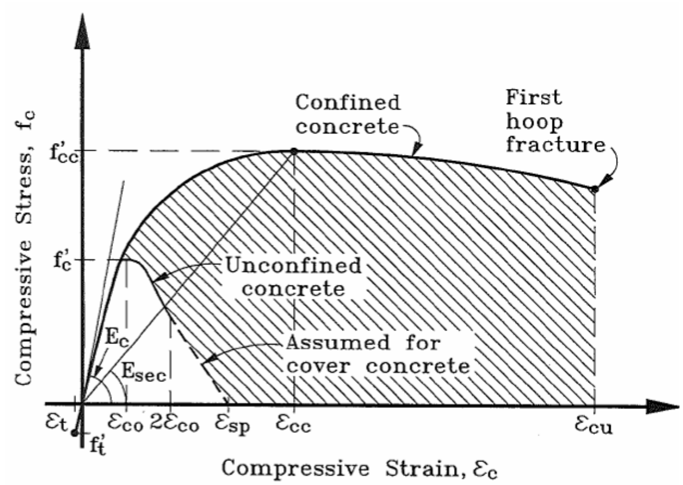
6.3.2 Nonlinear Concrete Models for Ductile Concrete Members

Reinforcing steel shall be modeled with a stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain.

The yield point should be defined by the expected yield stress of the steel, f_{ye}. The length of the yield plateau shall be a function of the steel strength and bar size. The strain-hardening curve can be modeled as a parabola or other non-linear relationship and should terminate at the ultimate tensile strain, ε_{su}. The ultimate strain should be set at the point where the stress begins to drop with increased strain as the bar approaches fracture. It is common practice to reduce the allowable ultimate strain by up to thirty-three percent to decrease the probability of fracture of the reinforcement. The commonly used steel model is shown in below.



A stress-strain model for confined and unconfined concrete shall be used in the analysis to determine the local capacity of ductile concrete members. The initial ascending curve may be represented by the same equation for both the confined and unconfined model since the confining steel has no effect in this range of strains. As the curve approaches the compressive strength of the unconfined concrete, the unconfined stress begins to fall to an unconfined strain level before rapidly degrading to zero at the ultimate compressive strain of unconfined concrete (spalling strain), ε_{sp}. Typically $\varepsilon_{sp} \approx 0.005$. The confined concrete model should continue to ascend until the confined compressive strength f'_{cc} is reached. This segment should be followed by a descending curve that is dependent on the parameters of the confining steel. The ultimate strain for confined concrete, ε_{cu}, should be the point where strain energy equilibrium is reached between the concrete and the confinement steel. A commonly used model is Mander's stress-strain model for confined concrete, shown in the figures below.



For modeling purposes, the unconfined concrete compressive strain at the maximum compressive stress shall be taken as $\varepsilon_{co} = 0.002$ (Guide Spec 8.4.4). The ultimate unconfined compression strain based on spalling shall be taken as $\varepsilon_{sp} = 0.005$ (Guide Spec 8.4.4).

The concrete compressive strain at maximum compressive stress of confined concrete, ε_{cc}, and the ultimate compressive strain for confined concrete, ε_{cu}, should be computed using Mander's model. (Guide Spec 8.4.4)

$$f_c(x) = \frac{(f'_{cc})(x)(r)}{r - 1 + x^r}$$

$$f'_{cc} = f'_c \left[-1.254 + 2.254\sqrt{1 + \frac{7.94 f'_l}{f'_c}} - 2\frac{f'_l}{f'_c} \right]$$

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left[1 + 5\left(\frac{f'_{cc}}{f'_c} - 1\right)\right] = (0.002)\left[1 + 5\left(\frac{f'_{cc}}{f'_c} - 1\right)\right]$$

$$\mathrm{r} = \frac{\mathrm{E_c}}{\mathrm{E_c} - \mathrm{E_{sec}}}$$

$$\mathrm{E_c} = 60{,}000\sqrt{f'_c}\ \text{(psi)}$$

$$E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}}$$

$$\varepsilon_{cu} = 0.004 + \left[\frac{1.4 \times \rho_s \times f_{yh} \times \varepsilon_{su}}{f'_{cc}}\right]$$

Where:

$f_c(x)$ = function for predicting concrete stress at strain condition x

f'_{cc} = confined concrete compressive strength

x = ratio of concrete compressive strain at a given state to concrete compressive strain at maximum compressive stress

ε_c = concrete compressive strain at a given compressive stress

ε_{cc} = confined concrete compressive strain at maximum compressive stress

ε_{c0} = unconfined concrete compressive strain at maximum compressive stress

r = term representing the difference between the concrete modulus of elasticity and the secant modulus of elasticity for confined concrete

f'_c = nominal concrete strength (expected concrete strength, f'_{ce}, is substituted for this term for evaluation of seismic performance)

f'_l = effective lateral confining stress (defined in the discussion that follows)

E_c = modulus of elasticity for concrete; Mander's formulation shown; for this project, however, *AASHTO LRFD* Equation 5.4.2.4-1 will be used, except that f'_{ce} (defined in Section 4), shall be substituted for f'_c

E_{sec} = secant modulus of elasticity for confined concrete

ρ_s = transverse (confinement) reinforcing area ratio

f_{yh} = yield strength of transverse reinforcing; expected yield strength, f_{ye} (defined in Section 4) is substituted for this term for evaluation of seismic performance

ε_{su} = ultimate tensile strain of transverse reinforcing; reduced ultimate tensile strain ε_{su}^R (defined in Section 5.2) is substituted for this term for evaluation of seismic performance

When $f'_l = 0$, the value of f'_{cc} will be equal to f'_c and the equations above produce results that are appropriate for unconfined concrete.

For circular sections, the effective lateral confining stress, f'_l, is related to the average confining stress by the following expressions:

$$f'_l = K_e f_l$$

$$f_l = \frac{2 \times f_{yh} \times A_{sp}}{s \times D'}$$

Where:

A_{sp} = the cross-sectional area of typical transverse confinement reinforcing bar

s = the spacing of the transverse confinement reinforcing bars

D' = the diameter of the confined core, measured at the hoop or spiral centerline

For rectangular sections, with different transverse reinforcement area ratios, ρ_x and ρ_y, in the principal directions, different confining stresses are developed in accordance with the following relationships:

$$f'_{lx} = K_e \times \rho_x \times f_{yh}$$

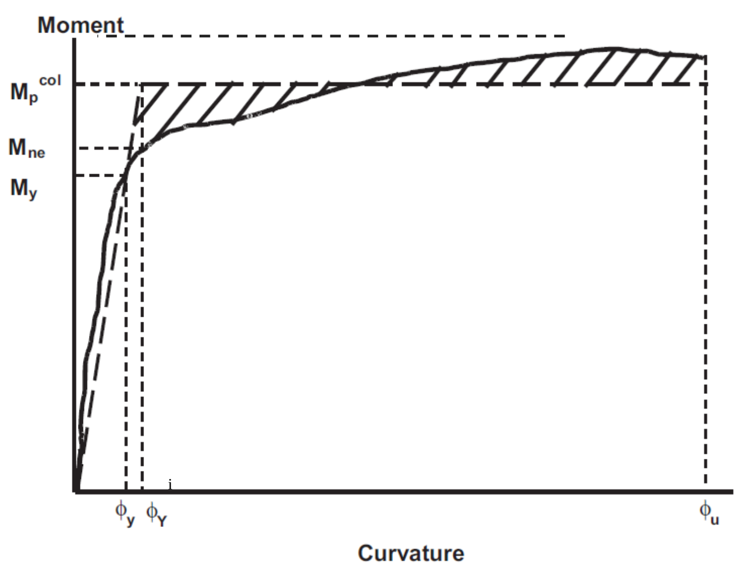
$$f'_{ly} = K_e \times \rho_y \times f_{yh}$$

In the equations above, K_e is a confinement effectiveness coefficient. The typical values of K_e are 0.95 for circular sections, 0.75 for rectangular sections, and 0.6 for rectangular wall sections.

6.3.3 Moment Curvature (M-ϕ) Analysis

The plastic moment capacity of all ductile concrete members shall be calculated by moment-curvature analysis on the basis of the expected material properties. Moment curvature analysis derives the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces. The moment-curvature (M-ϕ) analysis shall include the axial forces due to dead load together with axial forces due to overturning. (Guide Spec 8.5)

The M-ϕ curves shall be idealized with an elastic-perfectly-plastic response to estimate the plastic moment capacity of a member's cross-section. The elastic portion of the idealized curve shall pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity, Mp, shall be obtained by equating the area between the actual and the idealized M-ϕ curve beyond the first reinforcing bar yield point, as shown below. (Guide Spec 8.5)



The ultimate curvature, ϕ_u, is determined as the smaller of:

- The ultimate compressive strain, ε_{cu}, of the confined concrete divided by the distance from the plastic neutral axis to the extreme fiber of the confined concrete core, or
- The reduced ultimate tensile strain, ε_{su}^{R}, of the reinforcing steel divided by the distance from the plastic natural axis to the extreme tension fiber of the longitudinal column reinforcement *(Guide Spec 8.5).*

6.3.4 Seismic Shear for Ductile Concrete Members

The Seismic Shear capacity analysis for seismic retrofit design shall follow AASHTO Guide Spec. This methodology is also consistent with other publications such as Priestly et al., ATC 32, MCEER/ATC 49, and Caltrans SDC.

Explicit Shear Capacity for Ductile Concrete Members (Guide Spec 8.6)

For the capacity determination see Section 8.2.1 for columns and Section 8.2.2 for pier walls.

The following methodology from FHWA shall be used for the evaluation of existing bridges.

Brittle Shear, Semi-ductile Shear, and Flexure-limited Rotation (FHWA 7.8.2.7)

If the shear strength of the member is less than the shear demand (based on flexural strength) the plastic rotation will be limited. Two limiting cases are: (a) brittle shear, and (b) semi-ductile shear. These cases are based on the shear strength relative to the flexural strength. (FHWA 7.8.2.7)

When the initial shear strength of the member is less than the overstrength shear demand V_o, the member is considered to be 'shear-critical' and will fail in a brittle manner with no plastic rotation capacity. See equation below and Section 6.3.6 for definition of overstrength shear demand. This type of failure is considered unacceptable unless the initial shear capacity of the member is sufficient to resist the seismic loads elastically, or there is an alternative load path is identified to assure structural stability. Otherwise, the member must be retrofitted to increase its shear capacity.

$$V_0 = \frac{M_o}{L}$$

When the plastic shear demand lies between the initial shear capacity and the final shear capacity of the member, the rotational capacity of the member is limited. If the limited rotational capacity of the member yields a displacement C/D ratio that is less than one, it may be possible to retrofit the member such that the final shear capacity of the member exceeds the plastic shear demand. Provided the flexure-controlled rotational capacity is greater than the demand, the C/D ratio of the retrofitted member would then exceed 1.0.

When the plastic shear demand is less than the final shear capacity of the member's rotational capacity is flexure-controlled.

Based upon conversations with a representative of FHWA, there are errors in FHWA Equations 7-49 and 7-51. This conclusion is supported by inconsistencies between the equations and the supporting narrative in FHWA 7.8.2.7(b) and 7.8.2.8(b). The following corrected forms will be utilized, as applicable:

$$\phi_p = \left[2 - 5\left(\frac{V_m - V_i}{V_i - V_f}\right)\right]\phi_y \qquad \text{(corrected form of } \textit{FHWA} \text{ Equation 7-49)}$$

$$\phi_p = \left[2 - 4\left(\frac{V_{jh} - V_{ji}}{V_{ji} - V_{jf}}\right)\right]\phi_y$$ (corrected form of *FHWA* Equation 7-51)

The equations above will produce results that are consistent with the narrative in FHWA 7.8.2.7(b) and 7.8.2.8(b), given the following definition of curvature ductility, μ_ϕ:

$$\mu_\phi = \frac{\phi_y + \phi_p}{\phi_y}$$

Buckling of longitudinal bars as addressed in FHWA 7.8.2.3 should also be considered.

6.3.5 Capacity-Protected Concrete Members

Capacity-protected concrete flexural components such as footings, pile shafts, crossbeams, joints and superstructure shall be designed to remain elastic when the column reaches its overstrength moment demand. See below for determination of overstrength factor.

The expected nominal moment capacity, Mne, for capacity-protected concrete components determined by either M-ϕ or strength design, is the minimum requirement for essentially elastic behavior. Ductile behavior (hinging) is not permitted in capacity-protected members. Due to cost consideration a factor of safety is not required (i.e. Resistance factor $\phi = 1.0$ for flexure). Expected material properties shall only be used to assess flexural component capacity for resisting earthquake loads. The material properties used for assessing all other load cases shall comply with the ODOT BDDM.

<u>Expected Nominal Moment Capacity</u>

The expected nominal moment capacity, Mne, is defined as the flexural strength of a reinforced concrete section when the extreme compression fiber of the section reaches a strain of 0.003 or the reinforcing steel strain reaches the reduced ultimate tensile strain, $\varepsilon_{su}{}^{R}$.

<u>Overstrength Factor</u>

The overstrength factor shall be based on one of the following methodology for either assessment of an existing bridge component, or retrofit of an existing bridge component and design of a new element.

<u>Methodology 1 - For Non-Capacity-Protected Bridge Components</u>

For assessment of existing bridge components, use the *maximum* moment M_{max}^{col} taken from the moment-curvature analysis based on expected material properties (Section 6.3.3), using the overstrength factor as 1.0. This is theoretically the maximum moment demand that a capacity-protected element will experience at the maximum allowable curvature, ϕ_u.

$$M_o^{col} = 1.0 \times M_{max}^{col}$$

<u>Methodology 2 – For Capacity-Protected Bridge Components</u>

The standard practice is to use the *expected* material properties to determine idealized plastic moment M_p^{col} (obtained during the moment-curvature analysis in Section 6.3.3) as the moment demand applied by the ductile column. When this method is used to determine the force demand on a capacity-protected member, an overstrength factor of 1.2 shall be used.

$$M_o^{col} = 1.2 \times M_p^{col}$$ *(Guide Spec 4.11.2 and 8.5)*

6.3.6 Superstructure/Crossbeam

The nominal capacity of the superstructure longitudinally and of the crossbeam transversely must be sufficient to ensure that columns have the ability to become fully plastic prior to the superstructure or crossbeam reaching its expected nominal strength M_{ne}, for seismic assessment. Longitudinally, the superstructure capacity shall be greater than the demand distributed to the superstructure on each side of the column by the largest combination of dead load moment, secondary prestress moment, and column earthquake moment. Crossbeams shall meet similar requirements.

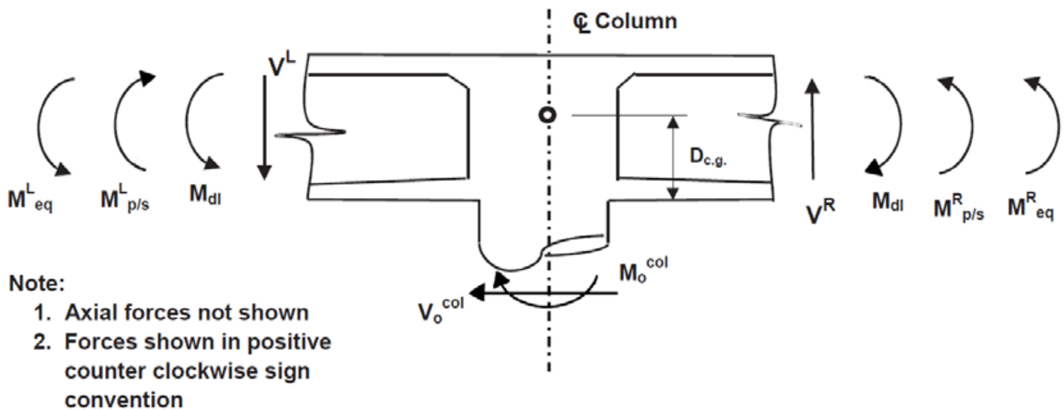
For span containing a hinge, the resisting moment on the hinge span side of the column shall not exceed the moment of the cantilever self-weight coupled with the reaction on the hinge times the distant to the hinge (the strength of the superstructure shall not be effective).

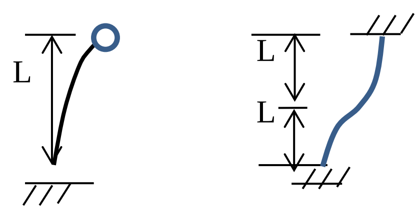
Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire frame. The distribution factors shall be based on cracked sectional properties of the superstructure crossbeam. The column earthquake moment represents the amount of moment induced by an earthquake, when coupled with the existing column dead load moment and column secondary prestress moment or the column's overstrength capacity, whichever is smaller. Subsequently, the column earthquake moment is distributed to the adjacent superstructure spans.

$$M_{ne}^{\sup(L)} \geq \Sigma\, M_{dl}^{L} + M_{p/s}^{L} + M_{eq}^{L} \qquad M_{ne}^{\sup(R)} \geq \Sigma\, M_{dl}^{R} + M_{p/s}^{R} + M_{eq}^{R}$$

$$M_{o}^{col} = M_{dl}^{col} + M_{p/s}^{col} + M_{eq}^{col} \qquad V_0 = \frac{M_o}{L}$$

$$M_{eq}^{R} + M_{eq}^{L} + M_{eq}^{col} + \left(V_o^{col} \times D_{c\,g}\right) = 0$$





Where:

$M_{ne}^{\sup R,L}$ = Expected nominal moment capacity of the adjacent left or right superstructure span.

M_{dl} = Dead load plus added dead load moment (unfactored).

$M_{p/s}$ = Secondary effective prestress moment (after losses have occurred).

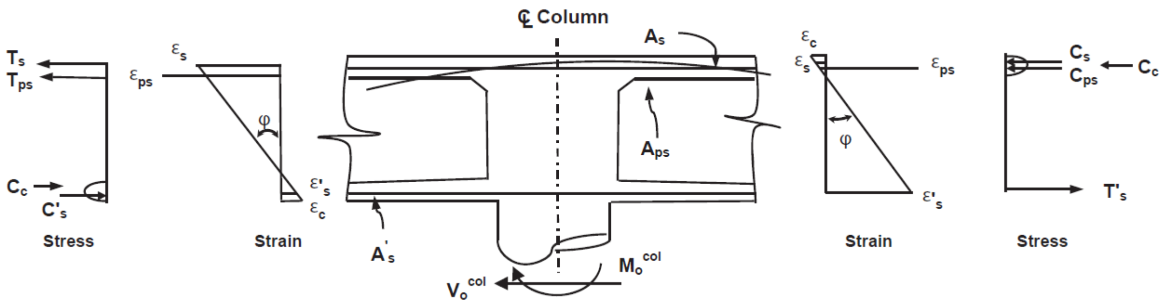
M_{eq}^{col} = The column earthquake moment when coupled with the existing column dead load moment and column secondary prestress moment, *or* the column's overstrength capacity, whichever is smaller.

$M_{eq}^{R,L}$ = The portion of M_{eq}^{col} and $V_o^{col} \times D_{cg}$ (moment induced by the overstrength shear) distributed to the left or right adjacent superstructure span.

L = Member length from point of maximum moment to point of contra-flexure.

6.3.6.1 Longitudinal Superstructure Capacity

Reinforcement included in the deck, A_s and/or soffit A'_s contributes to the moment capacity of the superstructure, see the following figure. The effective width of the superstructure increases and the moment demand decreases with distance from the crossbeam.



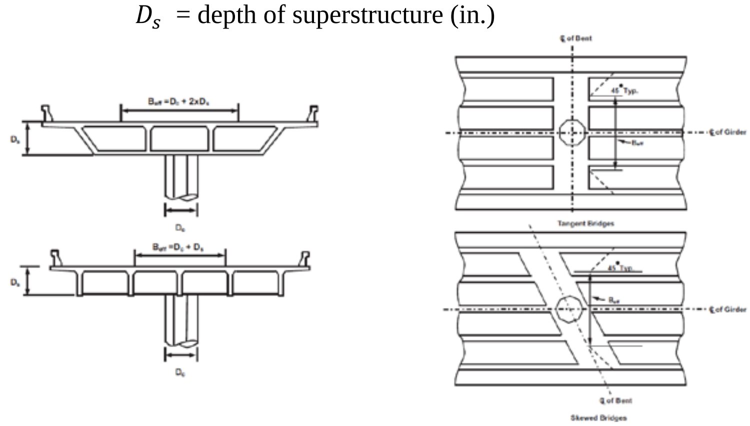
The superstructure shall be designed as a capacity- protected member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the entire width of the superstructure. The column overstrength moment M_o in addition to the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the superstructure shall be distributed to the spans framing into the bent on the basis of their stiffness distribution factors. This moment demand shall be considered within the effective width of the superstructure. The effective width of superstructure resisting longitudinal seismic moments B_{eff} shall be determined by the equations below. (Guide Spec 8.10)

For box girders and solid superstructure: $B_{eff} = D_c + 2D_s$

For open soffit, girder-deck superstructures: $B_{eff} = D_c + D_s$

Where, D_c = diameter of column (in.)

D_s = depth of superstructure (in.)



(Guide Spec Figure C8.10-1)

6.3.6.2 Crossbeam Capacity

Crossbeam reinforcement required for overstrength must be developed beyond the column cap joint. Crossbeams are considered integral if they terminate at the outside of the exterior girder and respond monolithically with the girder system during dynamic excitation. The crossbeam shall be designed as an essentially elastic member. Any moment demand caused by dead load or secondary prestress effects shall be distributed to the effective width of the crossbeam B_{eff} as shown in figure below.

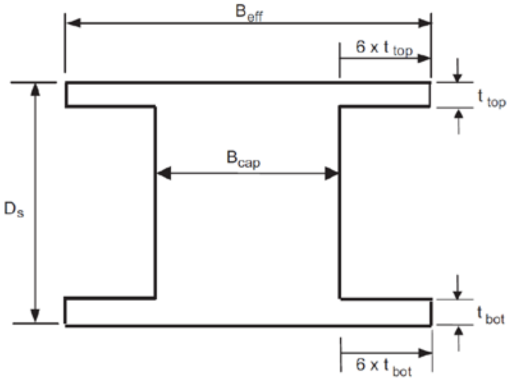
The column overstrength moment M_o and the moment induced due to the eccentricity between the plastic hinge location and the center of gravity of the crossbeam shall be distributed on the basis of the effective stiffness characteristics of the frame. The moment shall be considered within the effective width of the crossbeam. The effective width, B_{eff} shall be determined by the equation below. (Guide Spec 8.11)

$$B_{eff} = B_{cap} + 12t$$

Where,

t = thickness of the top or bottom slab (in.)

B_{cap} = thickness of the crossbeam/bent cap (in.)



(Guide Spec Figure 8.11-1)

6.3.7 Footing or Drilled Shaft Capacity

The foundation must have sufficient strength to ensure the column has moved well beyond its elastic capacity prior to the foundation reaching its expected nominal capacity. Refer to Section 7.2.4 for additional information on foundation performance.

6.3.8 Substructure/K-Braced Columns

K-Bracing to the existing bridge columns shall be analyzed for potential buckling under seismic loads. Seismic retrofit design shall include mitigations for preventing buckling, and adding retrofit details that allow ductile energy-dissipation.

6.4 Response Spectrum Analysis (RSA)

Response Spectrum Analysis shall be used for Global model analysis to determine mode shapes, structure periods and estimated seismic force and displacement demands. The Response Spectrum Analysis is also known as the linear elastic multimode spectral analysis (FHWA 5.2.2, AASHTO LRFD 4.7.4.3.3), dynamic response spectrum analysis, or elastic dynamic analysis (Guide Spec 5.4.3)

The project specific Response Spectrum and the Site Class Definitions shall be provided by the geotechnical engineers for the project.

Table 3.4.2.3-1, 3.4.2.3-2 in AASHTO Guide Spec shall be replaced by Table 1.17.3-1A, 1.17.3-1B, 1.17.3-1C in BDDM.

Acceleration Response Spectrum (ARS) curves with 5% damping shall be used.

Modal responses shall be combined using the complete quadratic combination (CQC) method. (AASHTO LRFD 4.7.4.3.3, Guide Spec 5.4.3, and Guide Spec C4.4)

6.4.1 Model Orientation

The Engineer is responsible for selecting the orientation of the two orthogonal axes that will represent longitudinal and transverse directions of seismic motion for the RSA. In general, the selection will be made from one of the following:

1. Orientation described in Section 4.5 above.
2. For a given frame, the longitudinal axis shall be oriented along a line connecting the centerline of bridge at the first bent in the frame and the centerline of bridge at the last bent in the frame. The transverse axis shall be perpendicular to the longitudinal axis.
3. For skewed structures, the orientation of the longitudinal and transverse motion may be rotated to be parallel to weak and strong axes, respectively, of the intermediate supports.

6.4.2 Modeling Requirements

The RSA model(s) shall contain sufficient detail to assess the anticipated behavior of the structure in a seismic event. Accordingly, the model(s) shall contain a sufficient number of degrees of freedom, nodes, and number of modes to capture at least 90% mass participation in the longitudinal and transverse directions (Guide Spec 5.4.3). The number of modes included in the analysis should be at least three times the number of spans in the model (AASHTO LRFD 4.7.4.3.3). For most bridges, an RSA model that is assigned four segments per column and ten segments per span for superstructure is sufficient to meet this criterion.

6.4.3 I_{eff} for Ductile members

The RSA based on design spectral accelerations will likely produce stress in some elements that exceed their elastic limit. The presence of such stresses indicates nonlinear behavior. The Engineer should recognize that forces generated by linear elastic analysis could vary considerably from the actual force demands on the structure.

For the FODE analysis, column sections shall initially be modeled using gross section properties, as the structure is expected to behave essentially elastically under the FODE response spectrum analysis. The column flexural and torsional stiffness properties may be reduced down to no less than 50% of the gross section properties (to reflect some cracking) if deemed appropriate by the Engineer. (AASHTO LRFD C4.7.1.3)

For the LODE analyse, column sections shall be modeled using equivalent cracked section properties, as the structure is expected to behave inelastically during those analyses.. In plastic hinge zones, cracked section properties shall be determined through a moment curvature analysis. For purposes of calculating effective section properties in plastic hinge zones, the unfactored axial gravity loads are used. (Guide Spec 5.6.2).

Equivalent cracked, or effective, section properties can be estimated by the slope of the moment-curvature curve between the origin and the point designating the first reinforcing bar yield, as defined by the equation below: (FHWA 7.3.2.1, Guide Spec 5.6.2)

$$E_c \times I_{eff} = \frac{M_y}{\phi_y}$$

Where:

I_{eff} = the effective cracked flexural stiffness to be used for modeling ductile elements (in4)

E_c = the modulus of elasticity of concrete (ksi)

M_y = the moment capacity of the section at first yield of the reinforcing steel (kip-in)

ϕ_y = the yield curvature corresponding to the yield of the first tension reinforcement in a ductile component, including the effects of the unfactored axial dead load (1/in)

Between plastic hinge zones, 50% of the column flexural and torsional stiffness properties shall be used. (FHWA 7.3.2.1)

6.4.4 I_{eff} for Superstructures

For the FODE analysis, superstructure sections shall be modeled using gross section properties, as the structure is expected to behave essentially elastically under the FODE response spectrum analysis.

For the LODE analyses, Ieff in box girder superstructures is dependent on the extent of cracking and the effect of the cracking on the element's stiffness. Ieff for reinforced concrete box girder sections may be estimated between 0.5 Ig – 0.75 Ig, if deemed appropriate by the Engineer. The lower bound represents lightly reinforced sections and the upper bound represents heavily reinforced sections. (FHWA 7.3.2.1, Guide Spec 5.6.3)

For prestressed concrete members, the location of the prestressing steel's centroid and the direction of bending have a significant impact on how cracking affects the stiffness. Multi-modal elastic analysis is incapable of capturing the variations in stiffness caused by moment reversal. Therefore, no stiffness reduction is recommended for prestressed concrete box girder sections. (FHWA 7.3.2.1, Guide Spec 5.6.3,)

Reductions to Ig similar to those specified for box girders can be used for other superstructure types and cap beams. A more refined estimate of Ieff based on M-ϕ analysis may be warranted for lightly reinforced girders and precast elements. (Guide Spec 5.6.4)

6.4.5 Effective Torsional Moment of Inertia

A reduction of the torsional moment of inertia is not required for bridge superstructures that meet the Standard Bridge requirements in Section 1.1 and do not have a high degree of in-plane curvature. (Guide Spec 5.6.5)

Since the torsional stiffness of concrete members can be greatly reduced after the onset of cracking, the torsional moment of inertia for columns may be reduced by the equation below. (Guide Spec 5.6.5)

$$J_{eff} = 0.2 \times J_g$$

Where:

J_{eff} = effective torsional (polar) moment of inertia of reinforced concrete (in^4)

J_g = gross torsional (polar) moment of inertia of reinforced concrete (in^4)

6.4.6 Boundary Conditions

Boundary conditions shall be included in the model to represent the behavior of the structure supports and interconnection of member elements. Where a component or boundary condition may behave in a nonlinear manner, an iterative solution is required as prescribed below.

6.4.7 Abutments

6.4.7.1 Longitudinal Abutments Response

The backfill passive pressure force resisting movement at the abutment varies nonlinearly with longitudinal abutment displacement and is dependent upon the material properties of the backfill. Abutment spring stiffness is estimated through abutment longitudinal response analysis using a bilinear approximation of the force-deformation relationship. The bilinear demand model shall include an effective abutment stiffness that accounts for expansion gaps, and incorporates a realistic value for the embankment fill response. The geotechnical professional shall be responsible to provide recommendation for the initial stiffness Ki. In case the geotechnical recommendation is not available, based on passive earth pressure tests and the force deflection results from large-scale abutment testing, the initial stiffness Ki may be estimated between 10kip/in/ft to 50 kip/in/ft for soils ranging from loose sand to dense sand. A reasonable starting point for the initial stiffness may be taken at 50kip/in/ft.

The initial stiffness shall be adjusted proportional to the backwall/diaphragm height.

$$K_{abut} = K_i \times w \times \left(\frac{h}{5.5\ ft}\right)$$

For seat-type abutments, the effective abutment wall stiffness shall account for the expansion hinge gaps as shown in the figures below. Based on a bilinear idealization of the force-deformation relationship, the passive pressure force resisting the movement at the abutment (P_{bw} or P_{dia}) is calculated with the following equation. The maximum passive pressure of 5.0 ksf is based on the ultimate static force developed in the full scale abutment testing. The height proportionality factor, $\frac{h}{5.5\ ft}$ is based on the height of the tested abutment walls.

$$P_{bw} or\ P_{dia} = A_e \times 5.0\ ksf\ \times \left(\frac{h_{bw}\ or\ h_{dia}}{5.5}\right)$$

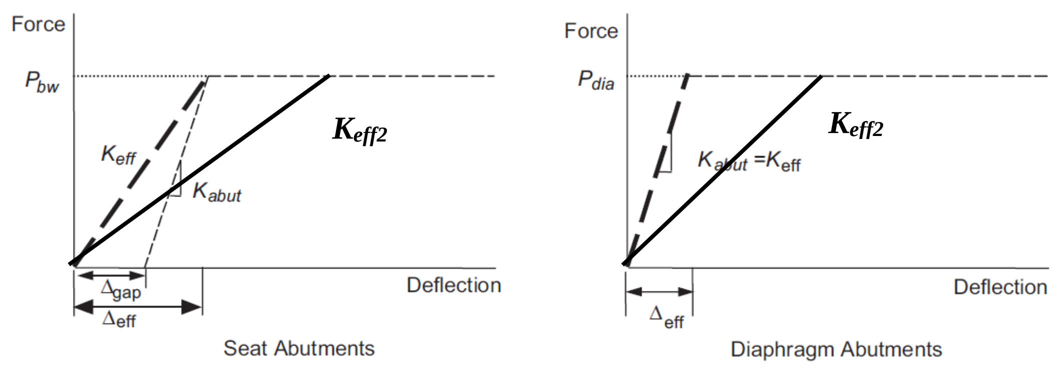
Where, $A_e = h_{bw} \times w_{bw}$ for Seat Abutment, or

$A_e = h_{dia} \times w_{dia}$ for Diaphragm Abutment

$h_{dia} = h^{*}_{dia}$ Effective height if the diaphragm is not design for full soil pressure as shown in figure below

$h_{dia} = h^{**}_{dia}$ Effective height if the diaphragm is design for full soil pressure as shown in figure below

$w_{bw}, w_{dia}, w_{abut} =$ Effective abutment width



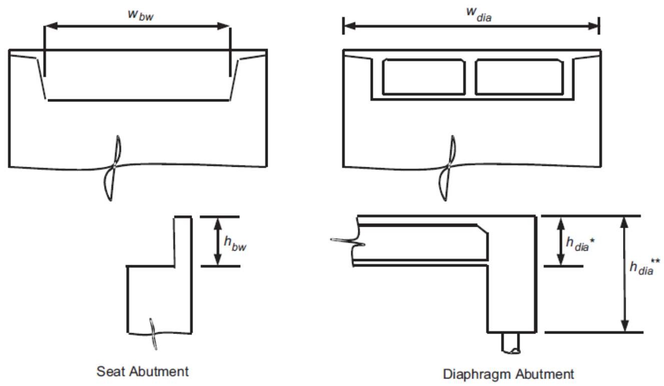
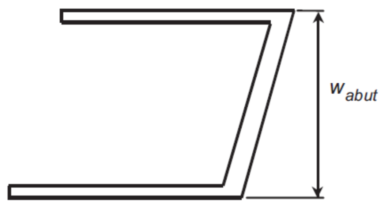


Figure 7.14B Effective Abutment Area



The longitudinal abutment spring magnitude shall be iterated for force convergence, if computed abutment forces exceed the soil capacity. The stiffness should be softened iteratively (K_{eff1} to K_{eff2}) until the abutment displacement are consistent (within 30 percent) with the assumed stiffness (Guide Spec 5.2.3.3.2). The suggested spring iteration procedure is as following:

Step 1. The longitudinal abutment springs shall be modeled with two separate springs, one at each end of the bridge. Each with the stiffness magnitude equal to $\frac{K_{eff1}}{2}$, where the K_{eff1} is the initial spring stiffness described previously in the equation above.

Step 2. Run the Response Spectrum Analysis (RSA), and check the abutment longitudinal reaction, RX, against the abutment passive pressure resisting capacity, Pbw or Pdia, as describe previously in the equation above.

Step 3. If the abutment longitudinal reaction (RX) is smaller than the abutment capacity (Pbw or Pdia), then iteration is not required. The abutment stiffness magnitude can be used as modeled. Each spring will have a longitudinal spring stiffness of K_{eff1}.

Step 4. However, if the abutment longitudinal reaction (RX) is greater than the abutment capacity (Pbw or Pdia), then iteration is required. Reduce the longitudinal abutment springs stiffness magnitude and re-run the Response Spectrum Analysis.

Step 5. Re-check the new abutment longitudinal reaction (RX) against the abutment resisting capacity, to see if the reaction demand is similar in magnitude as the resisting capacity (within approximately 10%).

Step 6. Iterate the spring stiffness either up or down until the abutment longitudinal reaction (RX) and the abutment resisting capacity reaches convergence.

For bridges with unusual geometry or differing connectivity at each abutment, it may be necessary to produce multiple RSA models, each with an appropriate full-stiffness spring at only one abutment, in order to capture directionally-dependent differences in behavior.

Longitudinal springs shall be orientated perpendicular to the abutment backwall.

6.4.7.2 Transverse Abutment Response

Abutments are designed to resist transverse service load and moderate levels of ground motion elastically. Linear elastic analysis cannot capture the inelastic response of the shear keys, wingwalls, or piles that may occur during higher level ground motion. The transverse capacity of an abutment foundation should be considered effective for the design seismic hazards and should include force-deflection characteristics and stiffness for each element that contributes to the transverse resistance. The geotechnical professional shall be responsible to provide abutment foundation springs. The wingwall passive soil stiffness may be estimated and iterated similar the method used for estimating and iterating for abutment stiffness.

6.4.8 Bents and Piers

The RSA model(s) shall include foundation springs at the intermediate bents (unless not required by the BDDM). The Engineer shall coordinate with Geotechnical Engineer to calibrate foundation springs. Foundation spring coefficients for spread footings shall be based upon the estimated soil properties and the footing geometry. Foundation spring coefficients for deep foundations, such as piles and drilled shafts, shall be based on the maximum shear and moment from the applied longitudinal or transverse seismic loading.

The combined load case (1.0L and 0.3T) shall be assumed for the design of structural members only, and not applied when determining foundation response. For the simple case of a regular bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat inexact for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

6.4.9 Tension and Compression Models

Global dynamic analyses are required to capture the assumed nonlinear response of a bridge because it possesses different characteristics in tension versus compression. When hinges or other superstructure structural discontinuities are present in a multi-span bridge, both compression and tension models are necessary to capture the maximum seismic force and displacement effects. (Guide Spec 5.1.2).

A compression model is a continuous model in which the hinges are considered closed/deactivated and restrained. The superstructure elements are locked longitudinally to capture structural response modes where the joints close up, and the abutments are mobilized.

A tension model frees a number of degrees of freedom at the joint location(s) to produce greater relative displacement at hinge/support locations. This is modeled to capture the effects of an open hinge or restrainers.

For the tension model analyses of the Burnside Bridge, the seismic restrainers installed during previous seismic retrofits will not be modelled.

6.4.10 Equal Displacement Rule

The equal displacement rule is a common approximation used for the analysis of bridges that states that the peak displacement amplitude for a structure responding inelastically is equal to the peak displacement amplitude calculated for the same structure responding elastically. The equal displacement rule is not theoretically based; instead, it is an observation made from experimental and analytical studies.

6.5 Pushover Analysis

The pushover method is also known as the Nonlinear Static Procedure. A nonlinear inelastic static pushover analysis, with considerations for geometric nonlinearity (second-order effects, P-Δ effects), should be used for the determination of the seismic displacement capacity, Δ_C, for LODE and ground motion. (FHWA 5.6)

The Pushover analysis may be a stand-alone Local analysis of a bent or the Pushover analysis may be performed on a Global model of an entire bridge.

A pushover analysis without geometric nonlinearity (P-Δ effects) is acceptable if the column exhibits sufficient levels of base shear and provided the equation below is satisfied:

$$P_{dl} \times \Delta_r \leq 0.20 \times M_p^{col} \qquad \textit{(Guide Spec 4.11.5)}$$

Where:

P_{dl} = the dead load on top of the pushover column

Δ_r = the relative lateral offset between the point of contra-flexure and the base of the plastic hinge

M_p^{col} = the plastic moment strength of the column

The column plastic moment capacity, M_p^{col}, shall be obtained using the idealized plastic moment capacity determination process through the moment-curvature process described in Section 6.3.3. The moment-curvature results shall utilize the moment-rotation, and "equivalent cracked" moment of inertia properties of column members within the plastic hinge zone, and gross section properties outside the hinge zone, and for crossbeams for pushover analysis.

The pushover analysis model may consist of an individual local pier/bent model to determine transverse displacement capacities of individual bents. A longitudinal model may also be required to determine displacement capacities of columns in the longitudinal direction of the bridge. Moment-rotation information shall be incorporated to capture the moment-curvature behavior of the ductile members. The component demands due to dead load shall be applied at the initial stage of the pushover model. The pushover analysis shall include sufficient finite step-increments to capture formation of the first plastic hinge, and shall proceed until the first hinge reaches its ultimate capacity, which will define the displacement capacity.

6.5.1 Simplified Analysis

For simple piers and bents, a hand calculation can be performed to verify the pushover analysis local displacement capacity result (Guide Spec C5.4.3). The following equations illustrate the definition of moment-curvature properties and the relationship used to calculate global displacement capacity:

Cantilever column with fixed base:

$$\Delta_c = \Delta_Y^{col} + \Delta_p$$

$$\Delta_Y^{col} = \frac{L^2}{3} \times \phi_Y$$

$$\Delta_p = \theta_p \times \left(L - \frac{L_p}{2}\right)$$

$$\theta_p = L_p \times \phi_p$$

$$\phi_p = \phi_u - \phi_y$$

Framed column (fix-fix condition)

$$\Delta_{c1} = \Delta_{Y1}^{col} + \Delta_{p1} \qquad \Delta_{c2} = \Delta_{Y2}^{col} + \Delta_{p2}$$

$$\Delta_{Y1}^{col} = \frac{(L_1)^2}{3} \times \phi_{Y1} \qquad \Delta_{Y2}^{col} = \frac{(L_2)^2}{3} \times \phi_{Y2}$$

$$\Delta_{p1} = \theta_{p1} \times \left(L_1 - \frac{L_{p1}}{2}\right) \qquad \Delta_{p2} = \theta_{p2} \times \left(L_2 - \frac{L_{p2}}{2}\right)$$

$$\theta_{p1} = L_{p1} \times \phi_{p1} \qquad \theta_{p2} = L_{p2} \times \phi_{p2}$$

$$\phi_{p1} = \phi_{u1} - \phi_{y1} \qquad \phi_{p2} = \phi_{u2} - \phi_{y2}$$

Where:

$L =$ distance from the point of maximum moment to the point of contra-flexure

$L_p =$ equivalent analytical plastic hinge length as defined below

$\Delta_p =$ idealized plastic displacement capacity due to rotation of the plastic hinge

$\Delta_y^{col} =$ idealized yield displacement of the column at the formation of the plastic hinge

$\phi_y =$ idealized yield curvature defined by an elastic-perfectly-plastic representation of the cross section's M-ϕ curve

$\phi_p =$ idealized plastic curvature capacity (assumed constant over L_p)

$\phi_u =$ curvature capacity at the Failure Limit State, defined as the concrete strain reaching ε_{cu} or the confinement reinforcing steel reaching the reduced ultimate strain ε_{su}^R

$\theta_p =$ plastic rotation capacity

The analytical plastic hinge length, L_p, is taken as the equivalent length of column over which the plastic curvature is assumed constant for estimating plastic rotation. (Guide Spec 4.11.6)

For columns & Pile Shafts:

$L_p = 0.08L + 0.15 f_{ye} d_{bl}$, where:

$L =$ distance from the point of maximum moment to the point of contra-flexure (in)

$f_{ye} =$ expected yield strength of longitudinal column reinforcing steel bars (ksi)

$d_{bl} =$ nominal diameter of longitudinal column reinforcing steel bar (in)

For non-cased Pile extensions:

$L_p = 0.1H' + D^* \leq 1.5D^*$, where:

$D^* =$ diameter of circular shafts or cross-section dimension in direction under consideration for oblong shafts (in)

$H' =$ length of shaft from the ground surface to point of contraflexure above ground (in)

For horizontally isolated flared columns:

$L_p = G + 0.3 f_{ye} d_{bl}$, where:

$G =$ The gap between the isolated flare and the soffit of the crossbeam (in)

$f_{ye} =$ expected yield strength of longitudinal column reinforcing steel bars (ksi)

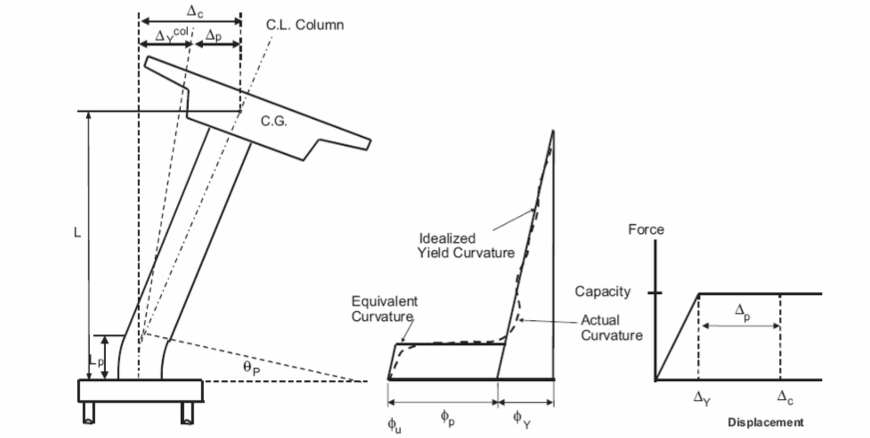
d_{bl} = nominal diameter of longitudinal column reinforcing steel bar (in)

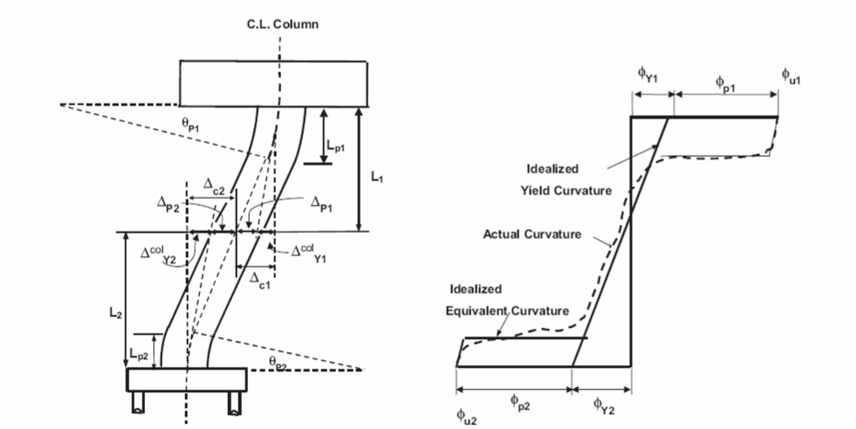
For concrete filled pipe pile extensions:

$L_p = 0.1H' + 1.25D \quad \leq 2D$, where:

D = diameter of concrete filled pipe (in)

H' = length of shaft from the ground surface to point of contraflexure above ground (in)





6.5.2 Stand-Alone Local Analysis

Stand-alone analysis quantifies the strength and ductility capacity of an individual frame, bent, or column. Stand-alone analysis may be performed in both the transverse and longitudinal directions.

The two-dimensional plane frame Pushover Analysis of a bent or frame can further be simplified to a column model (fixed-fixed or fixed-pinned) if it does not cause a significant loss in accuracy in estimating the displacement demands or the displacement capacities. The effect of overturning on the column axial load and associated member capacities must be considered in the simplified model.

7.0 PERFORMANCE ACCEPTANCE CRITERIA

7.1 Full Operation Design Earthquake (FODE) Ground Motion Acceptance Criteria

The performance level for the FODE ground motion is FO – Full Operation, as stipulated in Section 3. Under the FODE event, the bridge should be repairable without restriction on traffic flow. (FHWA 1.4.1)

Minimal damage may include minor inelastic response and narrow flexural or shear cracks in concrete. Permanent deformations are not apparent and repairs can be made under non-emergency conditions with possible exception of superstructure expansion joints which may need removal and temporary replacement. (FHWA 1.4)

> Note the differentiation between <u>Limited Operation</u> (LO) performance level and <u>Fully Operation</u> (FO) performance level. The fully operational criteria require that any damage sustained is negligible and traffic service is available for <u>all</u> vehicles. Except for joint seals, damage is minor that it can be repaired <u>without</u> interruption to traffic *(FHWA 1.4.1)*.

<u>Example of acceptable level of damage:</u>

- ✓ Damage to bearing at the local level that results in a fractions of inches of vertical displacement while maintaining vertical stability.
- ✓ Bearing is damaged and requires replacement after the seismic event.
- ✓ Bearing replacement requires the bridge superstructure to be temporary supported and the bearing repaired after the seismic event.
- ✓ Dowels in pin connections that fuse without resulting in a reduction in vertical load-carrying capacity of the bridge or superstructure unseating, and the loss of which will not preclude the structure from meeting the LODE performance requirements.

<u>Example of unacceptable level of damage:</u>

- ✗ Damages that require extensive time for repairing the bridge before it can be opened for emergency vehicles.

7.1.1 FODE Force Criteria

Bridge component Capacity-to-Demand (C/D) ratios shall be evaluated for all relevant failure modes, including but not limited to: Girders, In-Span Hinges, Bearings, Expansion Joints, Crossbeams, Outriggers, Columns/Piers, Footing/Pile Caps, Column-to-Crossbeam Connections, Column-to-Footing/Pile Cap Connections, Piles, and Pile Connections.

When evaluating the existing structure, use the overstrength demands for capacity protected elements.

Results indicating that $C/D \geq 1.0$ are considered acceptable.

Lateral loads should not fracture any abutment back wall, pier cap, bearing connection, or pile connections that could prohibit traffic flow following an FODE event.

Force and moment reactions in rectangular or oblong columns shall only be evaluated about each principal axis of the column individually, without consideration for biaxial effects (FHWA 7.4.2).

7.1.2 FODE Displacement Criteria

Local displacement capacities, such as at hinges and bearing seats, shall be calculated. Both global and local displacement demands shall comply with the "Full Operation" level of performance following an FODE seismic event.

Abutment or pier bearing displacements should be minimal. Any permanent bearings displacements due to FODE ground motions should be sufficiently small that they will not impede vehicle traffic after the event.

The abutment or pier bearing displacement capacity should be 6" more than the abutment or pier displacement demand.

$$\Delta_c \geq \Delta_d + 6"$$

Where, Δ_c= relative local displacement capacity

Δ_d= relative local displacement demand

Seat width requirements defined in FHWA 5.2.1 and displacement limitations at abutments defined in FHWA Appendix D.6 need not be met.

7.1.3 FODE Stress/Strain Criteria

To achieve the seismic performance objectives, the demands in the various structural components shall be limited to the values listed below.

For concrete elements, the acceptance criteria are defined following:

- Concrete Strain Limit (general)
  - Allowable compression strain for concrete, $\varepsilon_c = 0.003$
- Concrete Strain Limit for Locations with Lap Splices in Tension
  - Allowable compression strain for concrete, $\varepsilon_c = 0.002$ *(Priestley 7.4.5)*
- Reinforcing Steel Strain Limit
  - Allowable tensile strain ε_s = minimum of 0.01 and ε_{sh}, whichever is smaller

7.1.4 FODE Foundation Behavior

The geotechnical capacity of the foundation shall be established based upon the nominal, or ultimate, strength of the soil. Strengths shall be determined by geotechnical analysis or recommendation by the Geotechnical Engineer. Nominal strengths shall take into consideration liquefaction, other earthquake-induced soil strength reduction, existing scour, or other deleterious subsurface effects that may be present or are likely to occur under seismic loading conditions.

Force and moment reactions for evaluation of spread footings shall only be applied about each principal axis of the footing individually, without consideration for off-axis resultants (Guide Spec 6.3.4).

7.1.5 FODE Shallow Foundation (Spread Footing)

7.1.5.1 Bearing

$$q_R = \phi_b \times q_n \qquad \text{(\textit{AASHTO LRFD} 10.6.3.1.1-1)}$$

Where:

q_R = factored bearing resistance

ϕ_b = bearing resistance factor = 1.0

q_n = nominal bearing resistance

7.1.5.2 Sliding

$$R_R = \phi \times R_n = \phi_\tau \times R_\tau + \phi_{ep} \times R_{ep} \qquad \text{(\textit{AASHTO LRFD} 10.6.4.3-1)}$$

Where:

R_R = factored sliding resistance

R_n = nominal sliding resistance

ϕ_τ = resistance factor for shear resistance between soil and foundation = 1.0

R_τ = nominal sliding resistance between soil and foundation

ϕ_{ep} = resistance factor for passive resistance = 1.0

R_{ep} = nominal passive resistance of soil

7.1.5.3 Overturning

In general, the resultant of the reaction forces shall be within the middle two-thirds of the footing (AASHTO LRFD 11.6.5.1). If this condition cannot be achieved, limited unloading of the footing may be allowed so long as the ultimate bearing capacity is not exceeded. Footings experiencing reduced bearing across the footing surface shall be modeled with a bi-linear stress curve, where the maximum stress plateau shall equal the foundation soil bearing resistance. The force resultant shall remain within the footing.

7.1.6 Deep Foundations

7.1.6.1 Pile Axial Resistance

$$R_R = \phi \times R_n \qquad \text{(\textit{AASHTO LRFD} 10.7.3.8.6a-1)}$$

Where:

R_R = factored nominal axial resistance

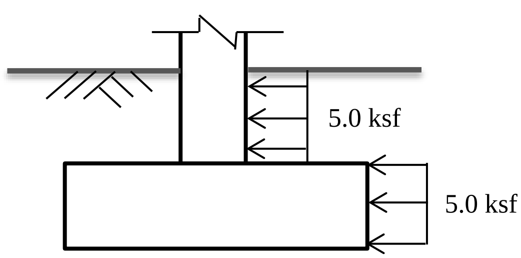
ϕ = axial resistance factor = 1.0

R_n = nominal axial resistance

7.1.6.2 Pile & Footing Lateral Resistance

Pile lateral resistance or capacity shall be determined by a lateral analysis using GROUP. LPILE, or other similar pile analysis software

Footing passive pressure of 5.0 ksf could be utilized (adjusted for depth of soil according to Section 6.4.7.1) for the initial analysis until geotechnical report is available. Similarly the column passive pressure of 5.0 ksf may also be utilized for initial analysis as shown in figure below.



7.1.7 Pile Structural Behavior

7.1.7.1 Steel Piles

Combined Axial Compression and Flexure

$$\frac{P_u}{P_r} + \frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \leq 1.0 \qquad \textit{(AASHTO LRFD 6.9.4.2.1-6)}$$

Where:

P_u = axial compressive load

P_r = factored compressive resistance

M_{ux} = factored flexural moment about the strong axis

M_{uy} = factored flexural moment about the weak axis

M_{rx} = factored flexural resistance about the strong axis = $F_{ye} \times S$

M_{ry} = factored flexural resistance about the weak axis = $F_{ye} \times Z_y$

Z_y = section properties represented by yielding of 50% of flange area.

7.1.7.2 Precast Concrete Piles

Due to historically poor pile reinforcing details in typical ODOT precast prestressed concrete piles, pile structural capacity shall be the nominal capacity of the pile. Pile flexural capacity shall be determined in accordance with strain limits defined in Section 7.1.3.

7.1.7.3 Timber Piles

Reference design values (properties) shall be in accordance with *AASHTO LRFD* 8.4.1.4

Properties shall be subject to the Format Conversion Factor of *AASHTO LRFD* 8.4.4.2, such that pile strength is determined using the Load and Resistance Factor Design methodology.

Properties shall be modified using Adjustment Factors of *AASHTO LRFD* 8.4.4.1.

Combined Axial Compression and Flexure:

$$\left(\frac{P_u}{P_r}\right)^2 + \frac{M_u}{M_r\left(1 - \frac{P_u}{F_{cE} \times A_g}\right)} \leq 1.0 \qquad (\textit{AASHTO LRFD}\ 8.10.2\text{-}1)$$

Where:

P_u = factored compression load

P_r = factored axial compressive resistance

M_u = factored flexural moment

M_r = factored flexural resistance

F_{cE} = Euler buckling stress

A_g = gross cross-sectional area

7.2 Limited Operation Design Earthquake (LODE) Ground Motion Acceptance Criteria

The performance level for LODE ground motion is LO – Limited Operation, as stipulated in Section 3. This criterion is to ensure that during the 1,000-year return probabilistic ground shaking considered feasible for the site, the bridge will enable emergency service and heavy haul vehicles to cross the bridge. Access for first responders and escape for downtown populations are the primary concerns and is the focus of the overall retrofit design philosophy.

In addition to above, the performance objective as described in Section 3 shall be met.

Note: LO performance level is required for the 1,000-year return period Design Earthquake.

Example of acceptable level of damage (In addition to the damage described in Section 7.1):

- ✓ Pier column cracks that require repairs
- ✓ Reduced traffic lanes to limit the total live load on the bridge before the repairs are completed.
- ✓ Posted speed limit to reduce the impact loads on the bridge before the repairs are completed.
- ✓ Misalignment of the bascule leafs that restrict the bascule operation before repairs are completed.

Example of unacceptable level of damage:

- ✗ A vertical displacement large enough (more than 3inches) to prevent emergency vehicles from crossing the bridge.
- ✗ Superstructure element falling off of the abutment seat, hinge seat, or crossbeam.

7.2.1 LODE Force Criteria

Bridge component Capacity-to-Demand (C/D) ratios shall be evaluated for all relevant failure modes, including but not limited to: Girders, In-Span Hinges, Bearings, Expansion Joints, Crossbeams, Outriggers, Columns/Piers, Footing/Pile Caps, Column-to-Crossbeam Connections, Column-to-Footing/Pile Cap Connections, Piles, and Pile Connections.

When evaluating the existing structure, use the overstrength demands for capacity protected elements.

Results indicating that $C/D \geq 1.0$ are considered acceptable using limited ductility displacement capacities.

Lateral loads should not fracture any abutment back wall, pier cap, bearing connection, or pile connections that requires extensive repair and prohibit traffic flow following an LODE event.

7.2.2 LODE Displacement Criteria

Abutment or pier bearing displacements should be minimal. Any permanent bearings displacements due to LODE ground motions should be sufficiently small that they will not require extensive repair thus impede emergency vehicle traffic after the event.

8.0 RETROFIT DESIGN

8.1 General

This section pertains to the design of modification to existing members to be retrofitted and new structure elements added to an existing structure as part of a retrofit strategy. It contains design requirements including commonly used in seismic design and detailing practice. For all other typical non-seismic design requirements not specified in this Criteria, the AASHTO LRFD Bridge Design Specifications shall be utilized.

The retrofit strategy shall mitigate all unacceptable deficiencies identified in the FODE and LODE analyses. If an identified deficiency cannot be feasibly mitigated, a Design Deviation request shall be submitted to the Multnomah County Bridge Division.

For determining force demands, design forces shall be the forces resulting from the overstrength plastic hinging moment capacity (Guide Spec 8.3.3).

For calculating capacities for retrofit design, expected material properties shall be used to determine section stiffness, overstrength capacities, and displacement capacities (Guide Spec 8.4).

For all new elements and retrofitted existing elements, calculations involving capacity of ductile, non-ductile, and capacity-protected members, the resistance factor ϕ shall be taken as 0.90 for shear and 1.0 for flexure. (FHWA, LRFD, Guide Spec). (Note: For existing bridge components, the shear resistance factor ϕ of 1.0 shall be used.)

For new elements that are added to the structure to provide acceptable performance of the foundation under the FODE event, such as drilled shafts at supports, members connecting these elements to the bridge need not be designed as capacity protected elements, provided the failure of these elements does not lead to the global instability of the bridge in the LODE event.

8.2 Ductile Member Requirement

8.2.1 Seismic Shear Design for Ductile Concrete Columns

The seismic shear demand shall be based on the overstrength shear Vo associated with the overstrength moment Mo based on the expected material properties. (Guide Spec 8.6.1)

$$\phi V_n \geq V_o$$

$$V_n = V_c + V_s$$

Where, V_n = nominal shear capacity of member

V_o = overstrength shear as defined in Section 6.3.5

ϕ = resistance factor as defined in Section 8.1

<u>Concrete shear capacity:</u> The concrete shear capacity, V_c, shall be taken as: (*Guide Spec 8.6.2)*

$$V_c = v_c A_e$$

$$A_e = 0.8\, A_g$$

If P_u is compressive: $v_c = 0.032\alpha' \left(1 + \frac{P_u}{2A_g}\right)\sqrt{f_c'} \leq min \begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_c'} \end{cases}$

If P_u is tension:

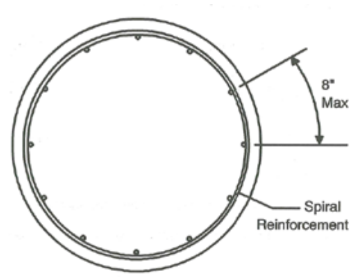
$v_c = 0$

For circular columns with spiral or hoop reinforcing:

$$0.3 \leq \alpha' = \frac{f_s}{0.15} + 3.67 - \mu_D \leq 3$$

$$f_s = \rho_s f_{yh} \leq 0.35$$

$$\rho_s = \frac{4A_{sp}}{sD'}$$



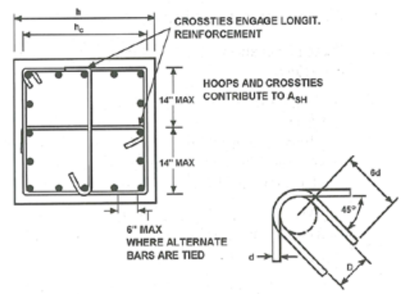
(Guide Spec Figure C8.6.3-1)

For rectangular columns with ties:

$$0.3 \leq \alpha' = \frac{f_w}{0.15} + 3.67 - \mu_D \leq 3$$

$$f_w = 2\rho_w f_{yh} \leq 0.35$$

$$\rho_w = \frac{A_v}{bs}$$



(Guide Spec Figure C8.6.3-2)

Where:

A_g = gross area of member cross-section (in.^{2})

P_u = ultimate compressive force acting on section (kips)

A_{sp} = area of spiral or hoop reinforcing bar (in.^{2})

s = pitch of spiral or spacing of hoops or ties (in.)

D' = diameter of spiral or hoop for circular column (in.)

A_v = total cross-sectional area of shear reinforcing bars in the direction of loading (in.^{2})

b = width of rectangular column (in.)

f'_c = nominal concrete compressive strength (ksi)

f_{yh} = nominal yield stress of transverse reinforcing (ksi)

μ_D = maximum local displacement ductility ratio of member as defined below:

$\mu_D = 3$ (SDC C)

$\mu_D = 1 + \frac{\Delta_p}{\Delta_Y}$ (SDC D)

Δ_p = plastic displacement demand (in.)

Δ_Y = Idealized yield displacement corresponding to the idealized yield curvature.

Shear Reinforcement Capacity: For members that are reinforced with circular hoops, spirals, or interlocking hoops or spirals, the nominal reinforcement strength V_s shall be taken as:

$$V_s = \frac{\pi}{2}\left(\frac{nA_{sp}f_{yh}D'}{s}\right)$$

Where:

n = number of individual interlocking spiral or hoop core sections

A_{sp} = area of spiral or hoop reinforcement bar (in.^{2})

f_{yh} = yield stress of spiral or hoop reinforcement (ksi)

D' = core diameter of column measured from center of spiral or hoop (in.)

s = pitch of spiral or spacing of hoop reinforcement (in.)

For members that are reinforced with rectangular ties or stirrups, including pier walls in the weak direction, the nominal shear reinforcement strength V_s shall be taken as:

$$V_{s=}\frac{A_v f_{yh} d}{s}$$

Where:

A_v =cross sectional area of shear reinforcement in the direction of loading (in.^{2})

d =effective depth of section in direction of loading measured from the compression face of the member to the center of gravity of the tension reinforcement (in.)

f_{yh} =yield stress of tie reinforcement (ksi)

s =spacing of tie reinforcement

8.2.2 Seismic Shear Design for Pier Walls

Pier Wall Shear Capacity in the Weak Direction (*Guide Spec 8.6.8)*

The seismic shear demand V_u shall not be greater than the lesser of: The overstrength capacity of the superstructure to substructure connection, the overstrength capacity of the foundation, or the force demands determined by elastic analysis.

$$\phi V_n \geq V_u$$

$$V_n = V_c + V_s$$

Where, $\phi =$ resistance factor as defined in Section 8.1

The shear capacity for pier walls in the weak direction walls in the weak direction shall be determined according to the column shear reinforcement capacity in Section 8.2.1.

Pier Wall Shear Capacity in the Strong Direction *(Guide Spec 8.6.9)*

The factored nominal shear capacity of pier walls in the strong direction, ϕV_n, shall be greater than the maximum shear demand, $V_{EQ} = V_u$, obtained from the Response Spectrum Analysis.

$$\phi V_n \geq V_u$$

in which:

$$V_n = (0.13\sqrt{f_c'} + \rho_h f_{yh})\, bd \leq 0.25\sqrt{f_c'}\, A_e$$

$$\rho_h = \frac{A_v}{bs}$$

Where:

A_v = cross-sectional area of shear reinforcement in the direction of loading (in.^{2})

d = depth of section in direction of loading (in.)

b = width of section (in.)

f_{yh} = yield stress of tie reinforcement (ksi)

f_c' = compressive strength of concrete (ksi)

s = spacing of horizontal tie reinforcement (in.)

A_e = effective area of the cross-section for shear resistance as defined by below (in.^{2})

$$A_e = 0.8\,A_g$$

A_g = gross area of member cross-section (in.^{2})

Pier Wall Minimum Reinforcement *(Guide Spec 8.6.10)*

The horizontal reinforcement ratio, ρ_h, shall not be less than 0.0025. The vertical reinforcement ration, ρ_v, shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 18 in.

The reinforcement required for shear shall be continuous and shall be distributed uniformly. Horizontal and vertical layers of reinforcement shall be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered.

8.2.3 Maximum & Minimum Longitudinal Reinforcement

The Maximum Longitudinal Reinforcement is intended to apply to the full section of column. The smaller amount of longitudinal reinforcement, the greater the ductility of the column—better seismic performance. In addition, the maximum percentage is to avoid congestion, extensive shrinkage cracking, and to allow anchorage of the longitudinal steel. (*Guide Spec 8.8.1*)

$$A_l \leq 0.04A_g$$

The Minimum Longitudinal Reinforcement is a lower limit to avoid the effects of time-dependent deformation and, avoid large difference between flexural cracking and yield moments. (*Guide Spec 8.8.2, C8.8.2*)

$A_l \geq 0.007A_g$ for columns (SDC C)

$A_l \geq 0.01A_g$ for columns (SDC D)

$A_l \geq 0.0025A_g$ for pier walls (SDC C)

$A_l \geq 0.005A_g$ for pier walls (SDC D)

8.2.4 Splicing of Longitudinal Reinforcing

For columns subject to ductility demands, splicing of longitudinal column reinforcement shall be outside the plastic hinging region. The plastic hinging region is defined as following: (*Guide Spec 8.8.3, 4.11.7*)

The plastic hinging region shall be taken as the larger of:

- 1.5 times the gross cross-sectional dimension in the direction of bending
- The region of the column where the moment demand exceeds 75 percent of the maximum plastic moment
- The analytical plastic hinge length, as defined in Section 6.5.1.

The *no splice* region shall be clearly identified on the design plans.

Reinforcing steel splices in ductile component outside of the *no splice* region shall be capable of developing the expected tensile strength of the bars. *(Guide Spec 8.8.3)*

Weld Splices - Welded splices for longitudinal bars shall be full-penetration butt welds.

Mechanical Splice - Mechanical splices for longitudinal bars shall be capable of transferring a tension force corresponding to a bar stress of at least $1.3f_y$.

8.2.5 Minimum Lateral Strength for Ductile Members

The minimum lateral flexural capacity of each column shall be taken as the following: *(Guide Spec 8.7.1 modified)*

$$M_{ne} \geq 0.1 P_{max} L$$

Where,

M_{ne} = nominal moment capacity of the column based on expected material properties as defined in Section 6.3.5 (kip-ft).

P_{max} = greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kips).

L = member length from point of maximum moment to point of contra-flexure (ft).

8.2.6 Minimum Development Length of Reinforcing Steel

Column longitudinal reinforcement shall be extended into footings and crossbeams as close as practically possible to the opposite face of the footing or crossbeam. The anchorage length for longitudinal column bars developed into the crossbeam or footing for seismic loads shall satisfy the equation below: *(Guide Spec 8.8.4)*

$$l_{ac} \geq \frac{0.79 d_{bl} f_{ye}}{\sqrt{f_c'}}$$

where:

l_{ac} = anchored length of longitudinal reinforcing bars into the cap beam or footing (in.)

d_{bl} = diameter of the longitudinal column bar (in.)

f_{ye} = expected yield stress of the longitudinal reinforcement (ksi)

f_c' = nominal compressive strength of concrete (ksi)

The anchorage length shall not be reduced by means of adding hooks or mechanical anchorage devices. If hooks are provided, the tails should be pointed outward, away from the joint core.

8.2.7 Anchorage of Bundled Bars in Ductile Components

The anchorage length of individual column bars within a bundle anchored into a crossbeam shall be increased by 20 percent for a two-bar bundle and 50 percent for a three-bar bundle. Four-bar bundles shall not be permitted in ductile elements. *(Guide Spec 8.8.5)*

8.2.8 Maximum Bar Diameter

To ensure adequate bond to concrete, the nominal diameter of longitudinal reinforcement, d_{bl}, in columns shall satisfy the equation below: *(Guide Spec 8.8.6)*

$$d_{bl} = \frac{0.79\sqrt{f_c'}(L - 0.5D_c)}{f_{ye}}$$

where:

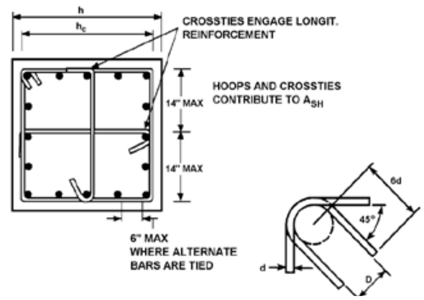
L = length of the column from the point of contraflexure to the point of maximum moment based on capacity design principles (in.)

D_c= diameter or depth of the column in direction of loading (in.)

f'_c= nominal compressive strength of concrete (ksi)

f_{ye}= the expected yield strength (ksi)

Where longitudinal bars in columns are bundled, the requirement of adequate bond (in equation above) shall be checked for the effective bar diameter, assumed as $1.2d_{bl}$ for two-bar bundles, and $1.5d_{bl}$ for three-bar bundles.

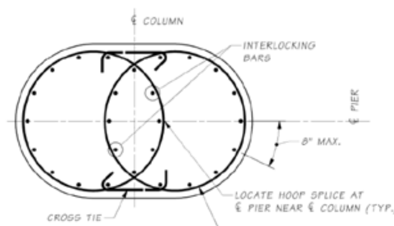


(Guide Spec Figure C8.6.3-2)

8.2.9 Requirements for Lateral Reinforcement

All longitudinal bars in compression members shall be enclosed by lateral reinforcement. *(Guide Spec 8.8.9)*

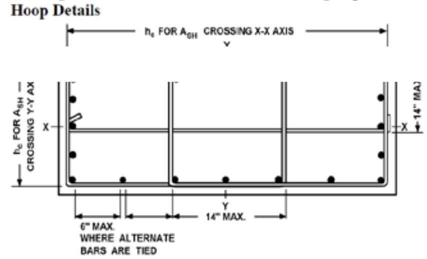
Lateral reinforcement inside the plastic hinge region shall be either butt-welded hoops or spirals. Combination of hoops and spiral shall not be permitted except in the footing or the crossbeam. Hoops may be placed around the column cage (i.e., extended longitudinal reinforcing steel) in lieu of continuous spiral reinforcement in the cap and footing. At spiral or hoop-to-spiral discontinuities, the spiral shall terminate with one extra turn plus a tail equal to the cage diameter. *(Guide Spec 8.8.7)*



(Guide Spec Figure C8.6.3-3)

Figure C8.6.3-3—Column Interlocking Spiral and Hoop Details

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties may be used provided that each end of the cross-tie engages a peripheral longitudinal reinforcing bar. All cross-ties shall have seismic hooks.



(Guide Spec Figure C8.6.3-4)

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- The bar shall be a continuous bar having a hook of not less than 135°, with an extension of not less than six diameters but not less than 3.0 in. at one end and a hook of not less than 90° with an extension of not less than six diameters at the other end.
- The hooks shall engage peripheral longitudinal bars.
- The 90° hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-to-end.

Transverse reinforcement meeting the following requirements shall be considered to be a hoop:

- The bar shall be closed tie or continuously wound tie.
- A closed tie may be made up of several reinforcing elements with 135° hooks having a six-diameter but not less than a 3.0-in. extension at each end.
- A continuously wound tie shall have at each end a 135° hook with a six-diameter but not less than a 3.0-in. extension that engages the longitudinal reinforcement.

The minimum size of lateral reinforcing bars shall be:

- #4 bars for #9 or smaller longitudinal bars,
- #5 bars for #10 or larger longitudinal bars, and

- #5 bars for bundled longitudinal bars.

The maximum spacing for lateral reinforcement in the plastic hinge regions as defined in Section 6.5.1 shall not exceed the smallest of:

- One-fifth of the least dimension of the cross-section for columns and one-half of the least cross-section dimension of piers,
- Six times the nominal diameter of the longitudinal reinforcement,
- 6 in. for single hoop or spiral reinforcement,
- 8 in. for bundled hoop reinforcement.

8.2.10 Development Length for Column Bars Extended in to Shafts

Column longitudinal reinforcement should be extended into enlarged shafts in accordance with the provisions in *BDDM.*

8.2.11 Lateral Reinforcement Requirements for Columns Supported on Shafts

At least 50 percent of the confinement reinforcement required at the base of the column shall extend over the entire embedded length of the column cage. *(Guide Spec 8.8.11)*

Column shear key shall be designed for the axial and shear forces associated with the column's overstrength moment capacity M_o including the effects of overturning. The key reinforcement shall be located as close to the center of the column as possible to minimize developing a force couple within the key reinforcement.

Steel Pipe sections may be used in lieu of reinforcing steel to relieve congesting and reduce the moment generated in the key.

Moment generated by the key reinforcing steel should be considered in applying capacity design principles. *(Guide Spec 8.15)*

8.3 Common Retrofit Measures

Common retrofit elements include columns, footing, abutments and superstructures. The table below lists the common retrofit measure for each of the elements, followed by several examples with schematics.

Columns
Steel Casing
Fiber Wrap
Column Enlargement
In-Fill walls

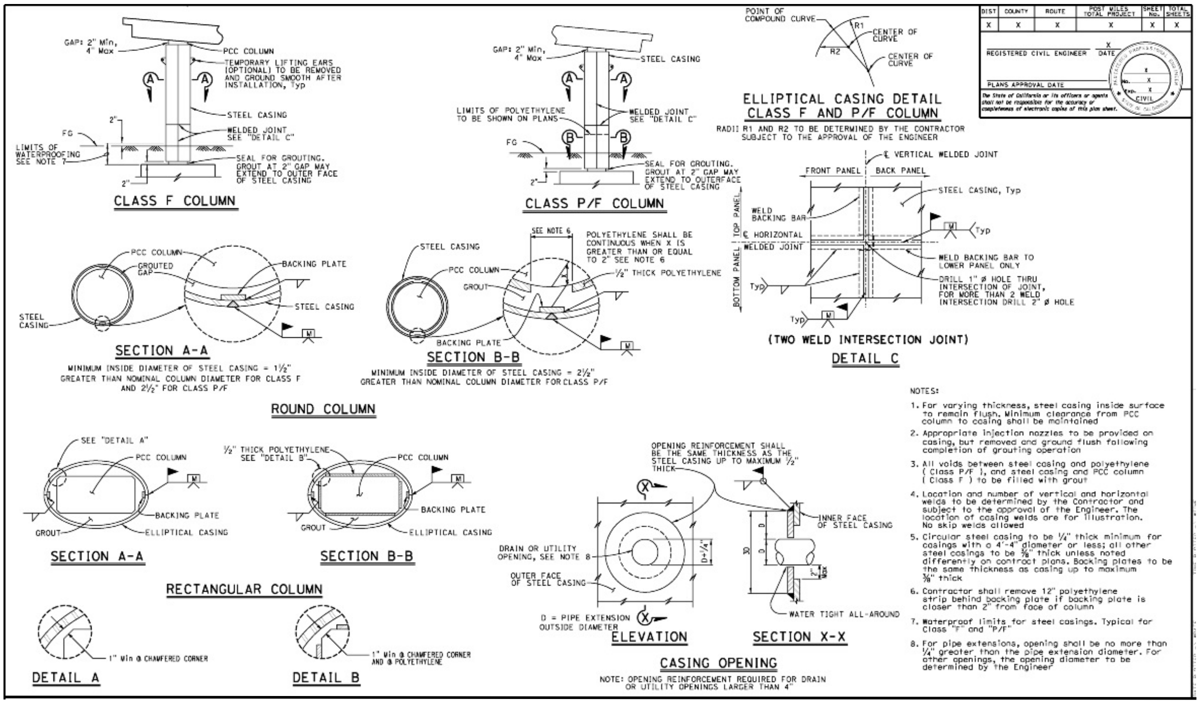
Abutments
Shear keys
Drilled Shafts
Seat Extenders
Footing Enlargement
Diaphragm Strengthening

Footings
Additional Piles
Drilled Shafts
Footing Enlargement
Footing Strengthening

Superstructure
Diaphragm Strengthening
Shear keys
Cable Restrainers
Crossbeam Widening
Crossbeam Strengthening

8.3.1 Steel Column Casing

The most common column retrofit is to encase the column with a steel jacket to increase the confinement and to improve the flexural ductility and shear capacities of the columns. However, when retrofitting for shear only, it is not necessary to maintain a circular or elliptical shape. Flat plates may be used when required due to limited horizontal clearance.



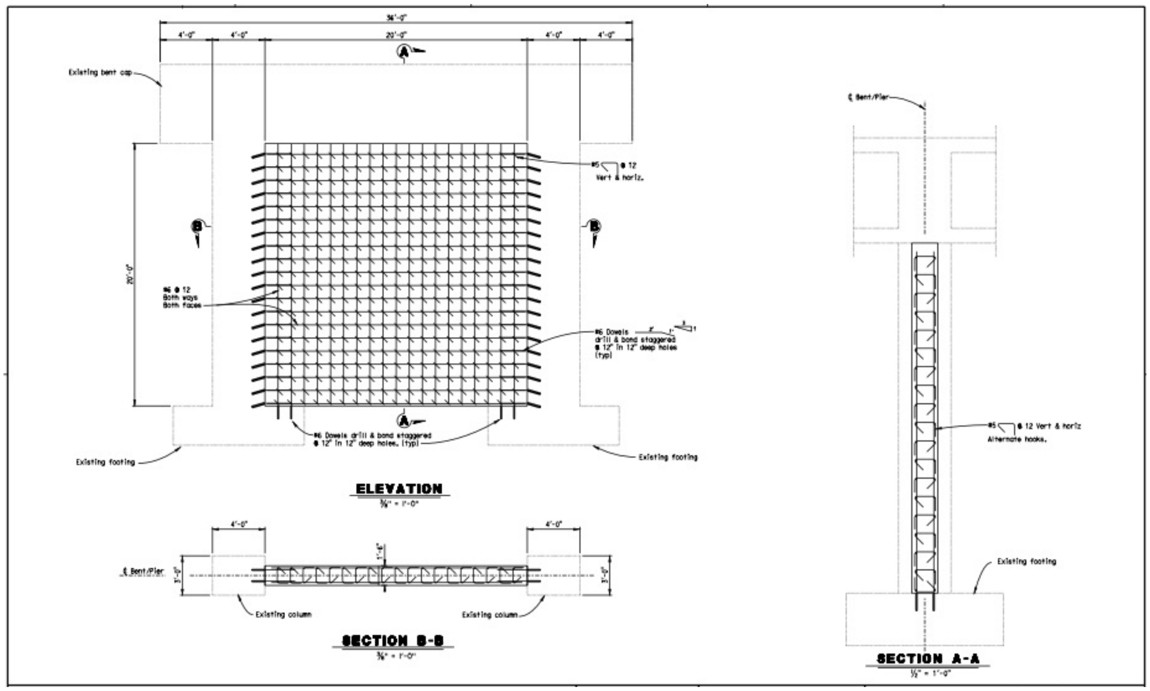
8.3.2 Composite Column Casings

Occasionally, space or clearance considerations do not allow steel column casings to be used for retrofit. In some of these cases, Fiber Reinforced Polymer (FRP) composite jackets may be used instead.

8.3.3 In-Fill Walls

In multi-column bents, the in-fill wall is an inexpensive and effective retrofit for addressing transverse vulnerabilities both in the columns and in the crossbeam. Research has shown that in-fill walls performed best when the concrete is placed directly against the soffit of the crossbeam. Doweling into the soffit of the crossbeam does not provide any additional capacity and thus is not recommended.

Figure below shows an example detail of bridge with infill wall retrofit.



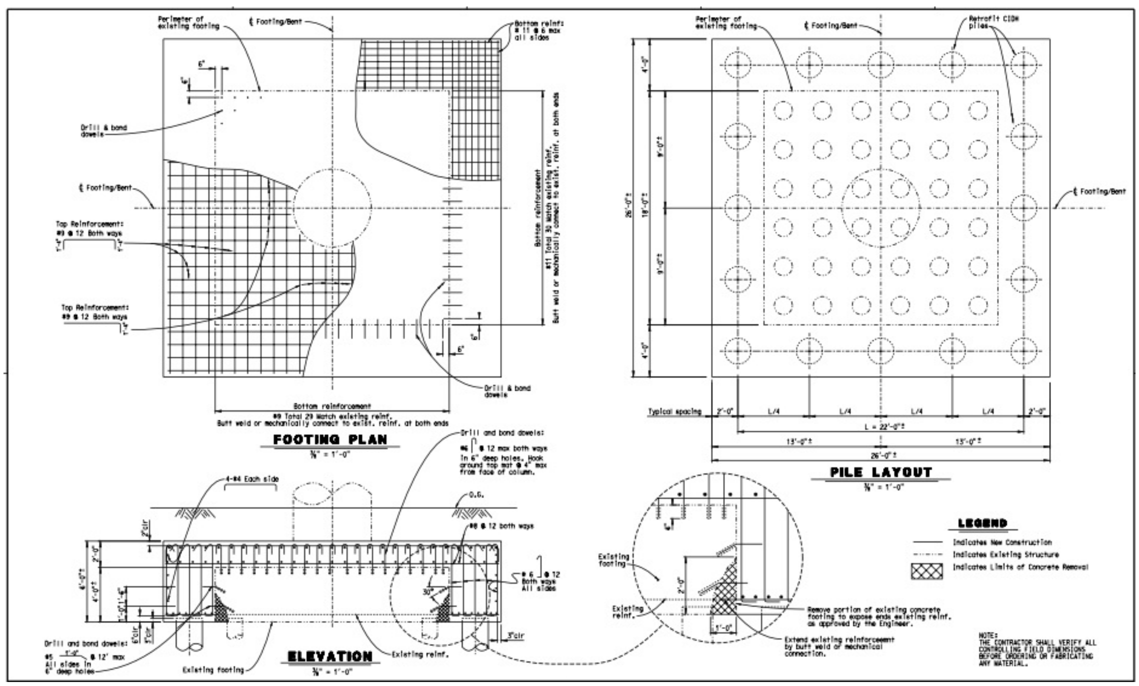
8.3.4 Footing Retrofit

When column casings are used for columns that are fixed to their footing, it is assumed that the footing (including pile caps) should resist the column's plastic moment. The following vulnerabilities may exist in the footings:

- No top mat of reinforcing steel.
- Inadequate tension ties connecting the pile and the footing.
- Inadequate pile axial capacity for the column's plastic moment.
- Insufficient shear strength in the piles to resist the column's plastic shear.

Typically, footings are strengthened by the addition of a top mat of reinforcing steel and additional piles.

Figure below shows an example detail of footing retrofit.



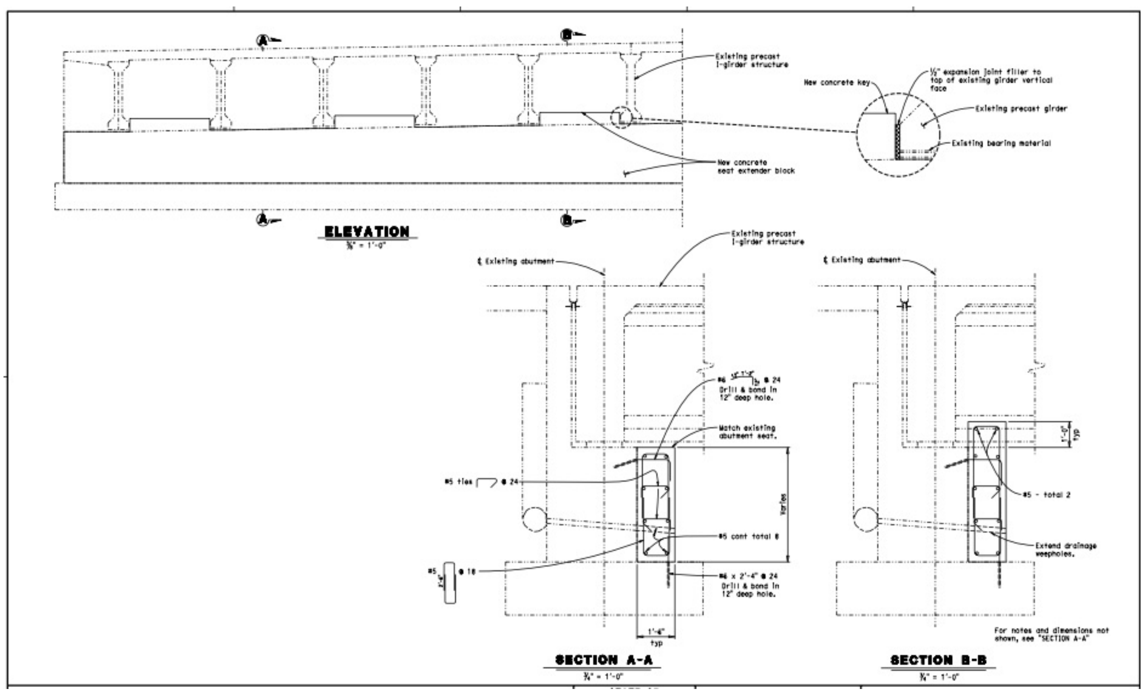
8.3.5 Abutment Strengthening

On short bridges, mobilizing the soil behind the abutments may be sufficient to reduce displacement demands below the structure's displacement capacity. This may be accomplished by strengthening the abutment diaphragm, or in the case of seat type abutments, connecting the superstructure end diaphragm to the seat. In some cases a large gap exists between the end diaphragm and the backwall in the As-built condition. In these cases the soil behind the backwall may be mobilized by eliminating the gap with concrete or timber blocking. The Engineer is cautioned to leave a gap that still allows for service load and temperature movements of the structure.

8.3.6 Catcher Blocks

Abutment bearings frequently fail during seismic events. However, such localized failure is not generally catastrophic unless the drop exceeds six inches. Seat catchers are an effective and inexpensive method of limiting superstructure drop and providing additional seat length as well. Catchers may also be used on crossbeams for simply supported structures.

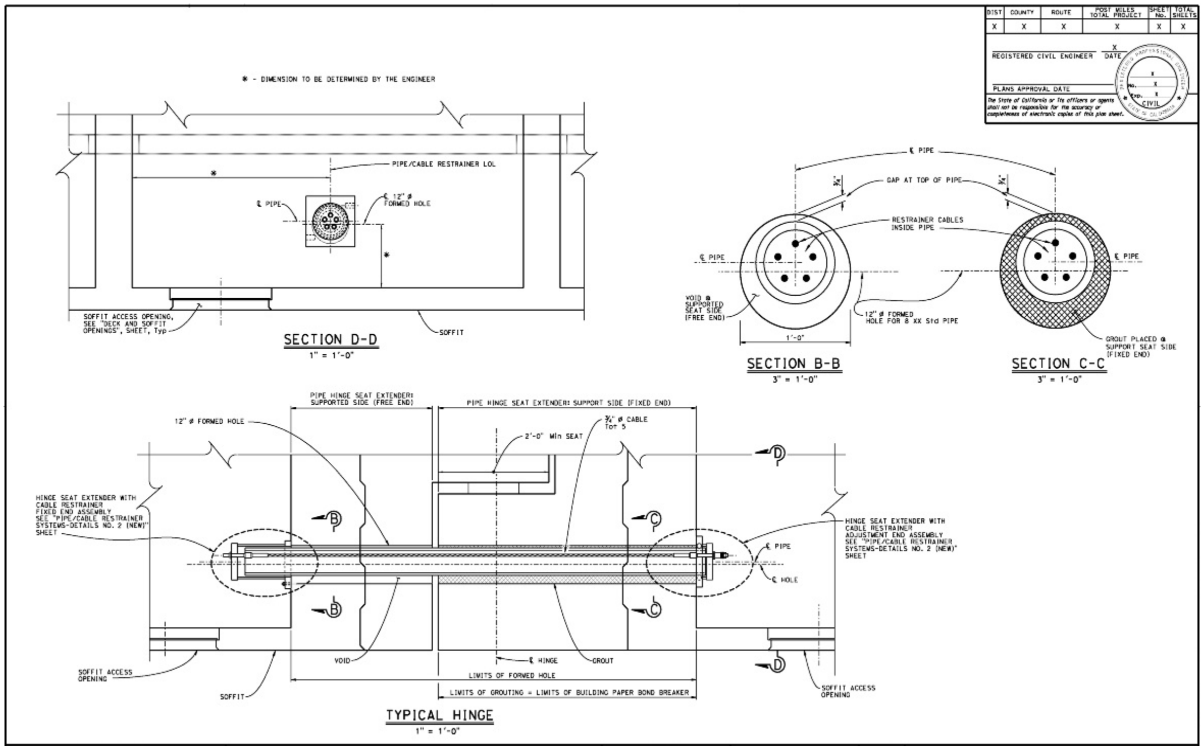
Figure below shows an example detail of abutment extender retrofit.



8.3.7 Cable Restrainer

On longer structures with expansion hinges, tying the frames together to limit differential displacement with cable restrainers may be an inexpensive and effective retrofit method in some circumstances. Cable restrainers may also be effective in preventing unseating of simply supported bridge spans.

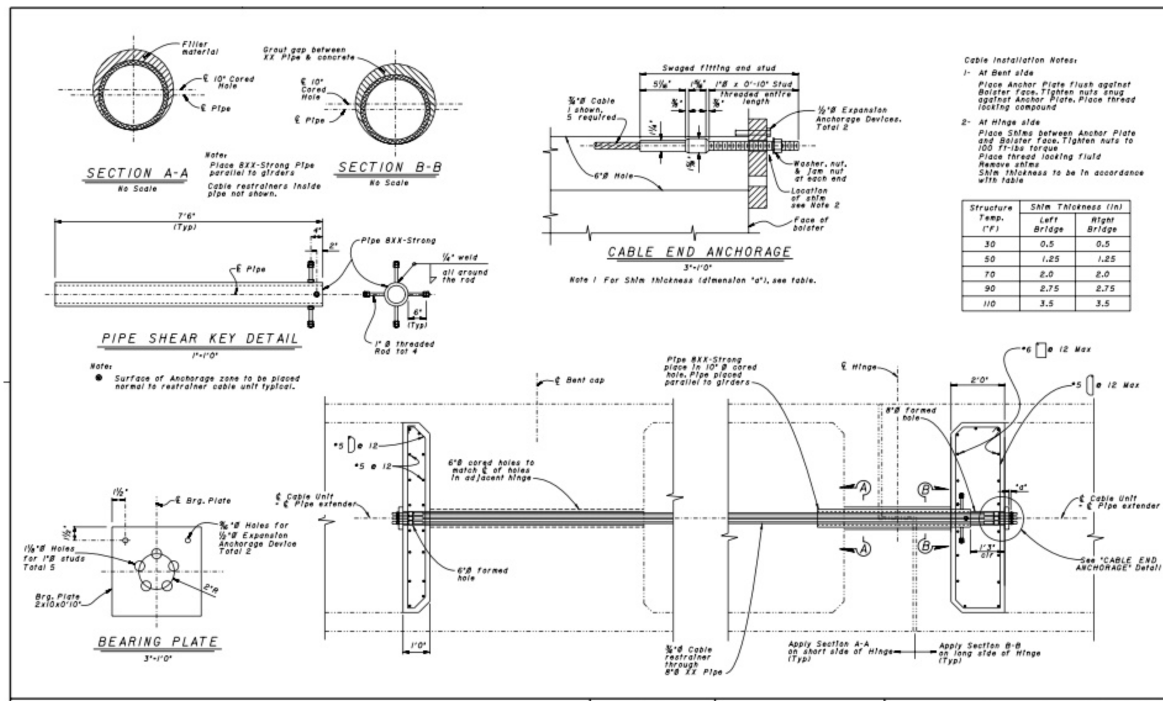
Figure below shows an example detail of cable restrainer detail.



8.3.8 Pipe Seat Extenders

Pipe seat extenders are effective in preventing collapse of a hinge span; however, the bridge may not be serviceable when the hinge opens sufficiently to engage the extenders. Therefore when pipe seat extenders are used for retrofit, consideration should be given to placing cable restrainers through the pipe and anchoring them to the adjacent crossbeam. This should limit the differential movement in the hinge during moderate events and reduce damage to the bearing pads and expansion joints.

The typical detail for a pipe seat extender makes use of Pipe 8" XX-strong. It is common practice to use an allowable force of 100 kips per pipe. However, when space or other considerations limit the number of pipes that can be placed, a higher design capacity (not to exceed 180 kips per pipe) may be used if verified through analysis. The Engineer should also consider that on skewed bridges the pipe seat extenders may be subjected to transverse forces as the superstructure tends to rotate. Pipe seat extenders should be installed so that movement of the bridge under service conditions is not restricted (typically the extenders should be placed parallel to the girders). In addition, the Engineer should evaluate the capacity of the supporting hinge diaphragm. Figure below shows an example detail of pipe seat extender detail.



8.4 Seismic Response Modification Devices

A response modification device changes the structural period and reduces the seismic force induced to the structure. Refer to the AASHTO *Guide Specifications for Seismic Isolation Design* - Third Edition • 2010

8.4.1 Base Isolation – Friction Pendulum / Spherical Sliding Bearing

Friction Pendulum bearings use the characteristics of a pendulum to lengthen the natural period of the isolated structure to avoid the strongest earthquake forces. During an earthquake, the supported structure moves with small pendulum motions. Since earthquake induced displacements occur primarily in the bearings, lateral loads and shaking movements transmitted to the structure are greatly reduced.

8.4.2 Reinforced Elastomeric Bearing

The reinforced elastomeric bearings, so called laminated bearings, are built of different layers, i.e. a layer of synthetic chloroprene rubber or natural rubber and a steel reinforcing sheet follows the next. These laminated material layers are merged by vulcanizing to a single pad as the rubber bonds to the steel reinforcing plates. This bearing provides vertical support while allowing some flexibility and even sliding in translation during a seismic event.

8.4.3 Lead Core Rubber Bearing

Laminated Elastomeric Bearings with one or more lead cylinder / plug in the center are known as lead core rubber bearings. These bearing also allow flexibility, but instead of sliding during an event, deformation of the lead plug facilitates very effective damping during extreme movements of these bearings.

9.0 NOTATIONS, ACRONYMS, & TERMINOLOGY

A_e = effective shear area (in2) (Section 8.2.1, Section 8.2.2)

A_g = gross cross section area (in2) (section 8.2.1, Section 8.2.2)

ARS = 5% damped elastic Acceleration Response Spectrum, expressed in terms of g (Section 6.4)

A_{jh} = the effective horizontal area of a moment resisting joint (Section 8.3.1.2)

A_{jh}^{ftg} = the effective horizontal area for a moment resisting footing joint (Section 8.3.2)

A_{jv} = the effective vertical area for a moment resisting joint (Section 8.3.1.2)

A_l = minimum longitudinal reinforcement (Section 8.2.3)

A_s = area of supplemental non-prestressed tension reinforcement (Section 6.3.6.1)

A'_s = area of supplemental compression reinforcement (Section 6.3.6.1)

A_{sp} = the cross-sectional area of typical transverse confinement reinforcing bar (Section 6.3.2, Section 8.2.1)

A_s^{jh} = area of horizontal joint shear reinforcement required at moment resisting joints (Section 8.3.1.6)

A_s^{jhc} = the total area of horizontal ties placed at the end of the crossbeam in Case 1 knee joints (Section 8.3.1.6)

A_s^{jv} = area of vertical joint shear reinforcement required at moment resisting joints (Section 8.3.1.6)

A_s^{j-bar} = area of vertical j-bar reinforcement required at moment resisting joints with a skew angle >20° (Section 8.3.1.6)

A_s^{sf} = area of crossbeam side face steel required at moment resisting joints (Section 8.3.1.6)

ASL = Anticipated Service Life (Section 2.1)

ASTM = American Society for Testing Materials (Section 5.2.4)

A_v = area of shear reinforcement perpendicular to flexural tension reinforcement (Section 8.2.1, Section 8.2.2)

b = width of rectangular column (Section 8.2.1, Section 8.2.2)

B_c = the other cross-sectional dimension of a column (Section 8.3.2)

B_{cap} = crossbeam width (Section 6.3.6.2, Section 8.3.1.2)

D_{cj} = column width or diameter parallel to the direction of bending (in.) (Section 8.3.2)

B_{eff} = effective width of the superstructure for resisting longitudinal seismic moments (Section 6.3.6.1, Section 6.3.6.2)

B_{eff}^{fig} = effective width of the footing for calculating average normal stress in the horizontal direction within a footing moment resisting joint (Section 8.3.2)

BDDM = ODOT *Bridge Design and Drafting Manual*

Bent Cap = Crossbeam

CIDH = cast-in-drilled-hole pile (Section 1.2)

CISS = cast-in-steel-shell pile (Section 1.2)

CQC = Complete Quadratic Combination – a statistical rule for combining modal responses from an earthquake load applied in a single direction to obtain the maximum response due to this earthquake load. (Section 5.4)

Capacity-Protected Element = member designed to stronger than an adjoining ductile member so that is remains elastic under seismic loading

Criteria = *Seismic Retrofit Criteria* (Chapter 1)

Crossbeam = Crossbeam means cap beam or bent cap in the *Guide Spec* and common practice. Crossbeam is the terminology used by ODOT.

D_c = column cross sectional dimension in the direction of interest (Section 6.3.6.1, Section 8.2.8, Section 8.3.1.2)

D_{ftg} = depth of footing (Section 8.3.2)

D_s = depth of superstructure at the crossbeam (Section 6.3.6.1, Section 6.3.6.2, Section 8.3.1.2)

D' = cross-sectional dimension of confined concrete core measured between the centerline of the peripheral hoop or spiral (Section 6.3.2, Section 8.2.1)

*D** = diameter for circular shafts or the least cross section dimension for oblong shafts (Section 6.5.1)

d = effective depth of section in direction of loading measured from the compression face of the member to the center of gravity of the tension reinforcement (in.) (Section 8.2.1, Section 8.2.2)

d_{bl} = nominal bar diameter of longitudinal column reinforcement (Section 6.5.1, Section 8.2.6, Section 8.2.8)

Ductile Elements = Ductile elements (or members) are parts of the structure that are expected to absorb energy, undergo significant inelastic deformations while maintaining their strength and stability.

E_c = modulus of elasticity of concrete (psi) (Section 5.1.1, Section 6.4.3)

E_s = modulus of elasticity of structural steel (psi); Modulus of elasticity of reinforcing steel (psi) (Section 5.1.2)

E_{sec} = secant modulus of elasticity for confined concrete (Section 6.3.2)

EDA = elastic Dynamic Analysis (Section 3.2.1)

F_a = site coefficient for 0.2-sec period spectral acceleration (Section 6.4.10)

F_v = site coefficient for 1.0-sec period spectral acceleration (Section 6.4.10)

FHWA = Federal Highway Administration *Seismic Retrofitting Manual for Highway Structures*. FHWA-HRT-06-032.

F_y = nominal yield strength of steel (Section 5.2.4, Section 7.1.3, Section 7.2.3)

F_{ye} = expected yield strength of steel (Section 5.2.4)

F_u = nominal tensile strength of steel (Section 5.2.4)

F_{ue} = expected tensile strength of steel (Section 5.2.4)

$f_c(x)$ = function for predicting concrete stress at strain condition x in Mander's model (Section 6.3.2)

f_c' = specified compressive strength of concrete at 28-day (Section 5.1.1, Section 8.2.1, Section 8.2.2, Section 8.2.6)

f_{cc}' = confined concrete compressive strength (Section 6.3.2)

f_{ce}' = expected compressive strength of concrete (Section 5.1.1)

f_l' = effective lateral confining stress (Section 6.3.2)

FODE = Full Operation Design Earthquake seismic event (Section 3.1)

Frame = A length of continuous superstructure between expansion joints (Section 1.2)

f_{sp} = tensile stress for low relaxation prestress strand (ksi) (Section 5.1.3)

f_u = specified minimum tensile strength for reinforcing steel (ksi) (Section 5.1.2)

f_{ue} = expected minimum tensile strength for reinforcing steel (ksi) (Section 5.1.2)

f_y = specified nominal yield stress for reinforcing steel (ksi) (Section 5.1.2)

f_{ye} = expected yield stress for reinforcing steel (ksi) (Section 5.1.2, Section 6.5.1, Section 8.2.6)

f_{yh} = nominal yield stress of transverse column reinforcement (hoops/spirals) (Section 6.3.2, Section 8.2.1 Section 8.2.1, Section 8.2.2)

G = gap between an isolated flare and the soffit of the crossbeam (Section 6.5.1)

G_c = shear modulus (modulus of rigidity) for concrete (ksi) (Section 5.1.1)

GDM = ODOT *Geotechnical Design Manual*

Global = referred as Global system or Global analysis. (Section 1.2)

g = acceleration due to gravity, 32.22 sec/ft (9.812sec/m)

h = height of abutment backwall (Section 6.4.7.1)

h_{dia} = height of diaphragm abutment height (Section 6.4.7.1)

h_{bw} = height of seat abutment backwall (Section 6.4.7.1)

H' = length of pile shaft/column from ground surface to the point of zero moment above ground (Section 6.5.1)

I_{eff} = effective moment of inertia for computing member stiffness (Section 6.4.3, Section 6.6.1)

I_g = moment of inertia about centroidal axis of the gross section of the member (Section 6.4.4, Section 6.6.1)

J_{eff} = effective polar moment of inertia for computing member stiffness (Section 6.4.5)

J_g = gross polar moment of inertia about centroidal axis of the gross section of the member (Section 6.4.5)

K_{abut} = abutment stiffness (Section 6.4.7.1)

K_e = effective confinement effectiveness coefficient (Section 6.3.2)

K_{eff} = effective abutment backwall stiffness ft/in/kip (Section 6.4.7.1)

K_i = Initial abutment backwall stiffness (Section 6.4.7.1)

K_1, K_2 = Iterated abutment backwall stiffness (Section 6.4.7.1)

L = member length from the point of maximum moment to the point of contra-flexure (in) (Section 6.3.6, Section 6.5.1, Section 8.2.5, Section 8.2.8)

LOE = Life Safety Design Earthquake seismic event (Section 3.1)

L_p = equivalent analytical plastic hinge length (in) (Section 6.5.1)

Local = referred as Local system or Local analysis. (Section 1.2)

LODE = Limited Operation Design Earthquake seismic event (Section 3.1)

LRFD = AASHTO *LRFD Bridge Design Specifications*

l_{ac} = length of column reinforcement embedded into crossbeam (Section 8.2.6, Section 8.3.1.2)

M_{dl} = moment attributed to dead load (Section 6.3.6)

M_{eq}^{col} = column moment when coupled with any existing M_{dl} & $M_{p/s}$ will equal the column's overstrength moment capacity, M_o^{col} (Section 6.3.6)

$M_{eq}^{R,L}$ = portion of M_{eq}^{col} distributed to the left or right adjacent superstructure spans (Section 6.3.6)

M_{max}^{col} = maximum moment obtained from moment-curvature analysis (Section 6.3.3, Section 6.3.5)

M_n = Nominal moment capacity based on the nominal concrete and steel strengths when the concrete strain reaches 0.003. (Section 6.3.3)

M_{ne} = nominal moment capacity based on the expected material properties and a concrete strain, $\varepsilon_c = 0.003$ (Section 6.3.3, Section 6.3.5, Section 8.2.5)

$M_{ne}^{\sup R,L}$ = expected nominal moment capacity of the right and left superstructure spans utilizing expected material properties (Section 6.3.6)

M_o^{col}, M_o = column overstrength moment (Section 6.3.5, Section 6.3.6, Section 6.3.6.1, Section 8.3.1.2)

M_p^{col} = idealized plastic moment capacity of a column calculated by M-ϕ analysis (kip-ft) (Section 6.3.3, Section 6.3.5, Section 6.5)

$M_{p/s}$ = moment attributed to secondary prestress effects (Section 6.3.6)

M_y = moment capacity of a ductile component corresponding to the first reinforcing bar yielding (Section 6.3.3, Section 6.4.3)

M-ϕ = moment curvature analysis (Section 6.3.3)

n = number of individual interlocking spiral or hoop core sections (Section 8.2.1)

ODOT = Oregon Department of Transportation (Chapter 1)

P_c, P_{col} = column axial force including the effects of overturning (Section 8.3.2)

P_{dl} = axial load attributed to dead load (Section 5.5)

P_{EQ}, P_u, P_c = axial demand due to seismic and dead load effects (Section 7.2.1, Section 8.3.1.2)

P_{max} = greater of the dead load per column or force associated with the tributary seismic mass collected at the bent (kips). (Section 8.2.5)

P_n = nominal axial capacity (Section 7.2.1)

PL = Performance Level (Section 3.3)

P-y = geometric nonlinearity (P-y effects) (Section 6.5)

p_c = nominal principal compression stress in a joint (Section 8.3.1.2, Section 8.3.2)

p_t = nominal principal tension stress in a joint (Section 8.3.1.2, Section 8.3.2)

Q_i = force effect (Section 4.1)

R_d = Short-period elastic analysis adjustment factor (Section 6.4.10)

RSA = Response Spectrum Analysis (Section 6.4)

S_{DS} = design earthquake response spectral acceleration coefficient at 0.2-sec period (Section 6.4.10)

S_{D1} = design earthquake response spectral acceleration coefficient at 1.0-sec period (Section 6.4.10)

Standard Bridge = a standard bridge classified in Section 1.1

S_s = 0.2-sec period spectral acceleration coefficient on Class B rock (Section 6.4.10)

S_1 = 1.0-sec period spectral acceleration coefficient on Class B rock (Section 6.4.10)

s = spacing of shear/transverse reinforcement measured along the longitudinal axis of the structural member (in) (Section 6.3.2, Section 8.2.1, Section 8.2.2)

Shaft = The term ODOT referred to as shaft is an oversize pile shaft. Also known as type II piles in other practice.

T = natural period of vibration, in seconds $T = 2\pi\sqrt{m/k}$ (Section 6.4.10)

T^* = the characteristic ground motion period corresponding to the peak energy input spectrum (sec) (Section 6.4.10)

T_s = short structural period; period at the end of constant design spectral acceleration plateau (Section 6.4.10)

T_c = total tensile force in column longitudinal reinforcement associated with M_o^{col} (Section 8.3.1.2, Section 8.3.2)

$T_{(i)}^{pile}$ = axial tension demand on a pile (Section 8.3.2)

T_{jv} = net tension force in moment resisting footing joints (Section 8.3.1.2, , Section 8.3.2)

t = top or bottom slab thickness (Section 6.3.6.2)

V_c = concrete shear capacity (Section 8.2.1, Section 8.2.2)

V_n = nominal shear strength (Section 8.2.1, Section 8.2.2)

V_o = overstrength shear associated with the overstrength moment Mo (Section 6.3.6, Section 8.2.1)

V_s = nominal strength provided by the steel shear reinforcement capacity (Section 8.2.1, Section 8.2.2)

v_{jv} = nominal vertical shear stress in a moment resisting joint (psi) (Section 8.3.1.2, Section 8.3.2)

v_c = Poisson's Ration (Section 4.1.1); Permissible shear stress carried by concrete (psi) (Section 8.2.1)

$w_{abut}, w_{bw}, w_{dia}$ = abutment width for seat abutment and diaphragm abutments (Section 6.4.7.1)

γ = unit weight of concrete (Section 5.1.1)

γ_i = load factors (Section 4.1)

γ_p = load factor for permanent loads (Section 4.1)

ε_c = concrete compressive strain (Section 7.1.3, Section 7.2.3)

ε_{cc} = concrete compressive strain at maximum compressive stress of confined concrete (Section 6.3.2)

ε_{co} = concrete compressive strain at maximum compressive stress of unconfined concrete (Section 6.3.2)

ε_s = steel strain (Section 7.1.3)

ε_{sp} = ultimate compressive strain (spalling strain) of unconfined concrete (Section 6.3.2)

ε_{cu} = ultimate compression strain for confined concrete (Section 6.3.2, Section 6.3.3, Section 7.2.3)

ε_{sp} = tensile strain for 7-wire low relaxation prestress strand (Section 5.1.4)

$\varepsilon_{ps,EE}$ = tensile strain in prestress steel at the essentially elastic limit state (Section 5.1.3)

$\varepsilon_{ps,u}^R$ = reduced ultimate tensile strain in prestress steel (Section 5.1.3)

ε_{sh} = tensile strain at the onset of strain hardening for reinforcing steel (ksi) (Section 5.1.2, Section 7.1.3)

ε_{su} = ultimate tensile strain for reinforcing steel (ksi) (Section 5.1.2)

ε_{su}^R = reduced ultimate tensile strain for reinforcing steel (ksi) (Section 5.1.2, Section 6.3.3, Section 7.2.3)

ε_y = nominal yield tensile strain for reinforcing steel (ksi) (Section 5.1.2)

ε_{ye} = expected yield tensile strain for reinforcing steel (ksi) (Section 5.1.2)

Δ_C = global displacement capacity (Section 7.2.2)

Δ_c = local member displacement capacity (Section 6.5.1, Section 7.1.2, Section 7.2.2)

Δ_{EQ}, Δ_D = seismic displacement demand (Section 7.2.2)

Δ_d = local member displacement demand (Section 7.1.2, Section 7.2.2)

Δ_p = local member plastic displacement capacity (in) (Section 6.5.1, Section 8.2.1)

Δ_{pd} = plastic displacement demand (in.) (Section 8.2.1)

Δ_r = relative lateral offset between the point of contra-flexure and the base of the plastic hinge (Section 6.5)

Δ_Y^{col} = idealized yield displacement of the column (Section 6.5.1)

Δ_Y = idealized yield displacement of the subsystem at the formation of the plastic hinge (in) (Section 6.5.1, Section 8.2.1)

θ_p = plastic rotation capacity (radians) (Section 6.5.1)

η = load modifier (Section 4.1)

ρ_h = horizontal reinforcement ratio, Section 8.2.2)

ρ_s = ratio of volume of spiral or hoop reinforcement to the core volume confined by the spiral or hoop reinforcement (measured out-to-out) (Section 6.3.2, Section 8.2.1, Section 8.3.1.5, Section 8.3.1.6)

ρ_x = transverse reinforcement area ratio in the principal x-directions (Section 6.3.2)

ρ_y = transverse reinforcement area ratio in the principal y-directions (Section 6.3.2)

$\emptyset$ = resistance factor (Section 6.3.3, Section 8.2.1)

$\emptyset_p$ = idealized plastic curvature (1/in) (Section 6.3.3, Section 6.5.1)

$\emptyset_u$ = ultimate curvature capacity (Section 6.3.3)

$\emptyset_y$ = yield curvature corresponding to the yield of the first tension reinforcement in a ductile component (Section 6.3.3, Section 6.4.3, Section 6.5.1)

$\emptyset_Y$ = idealized yield curvature (Section 6.3.3)

μ_c = local displacement ductility capacity (Section 8.2.1)

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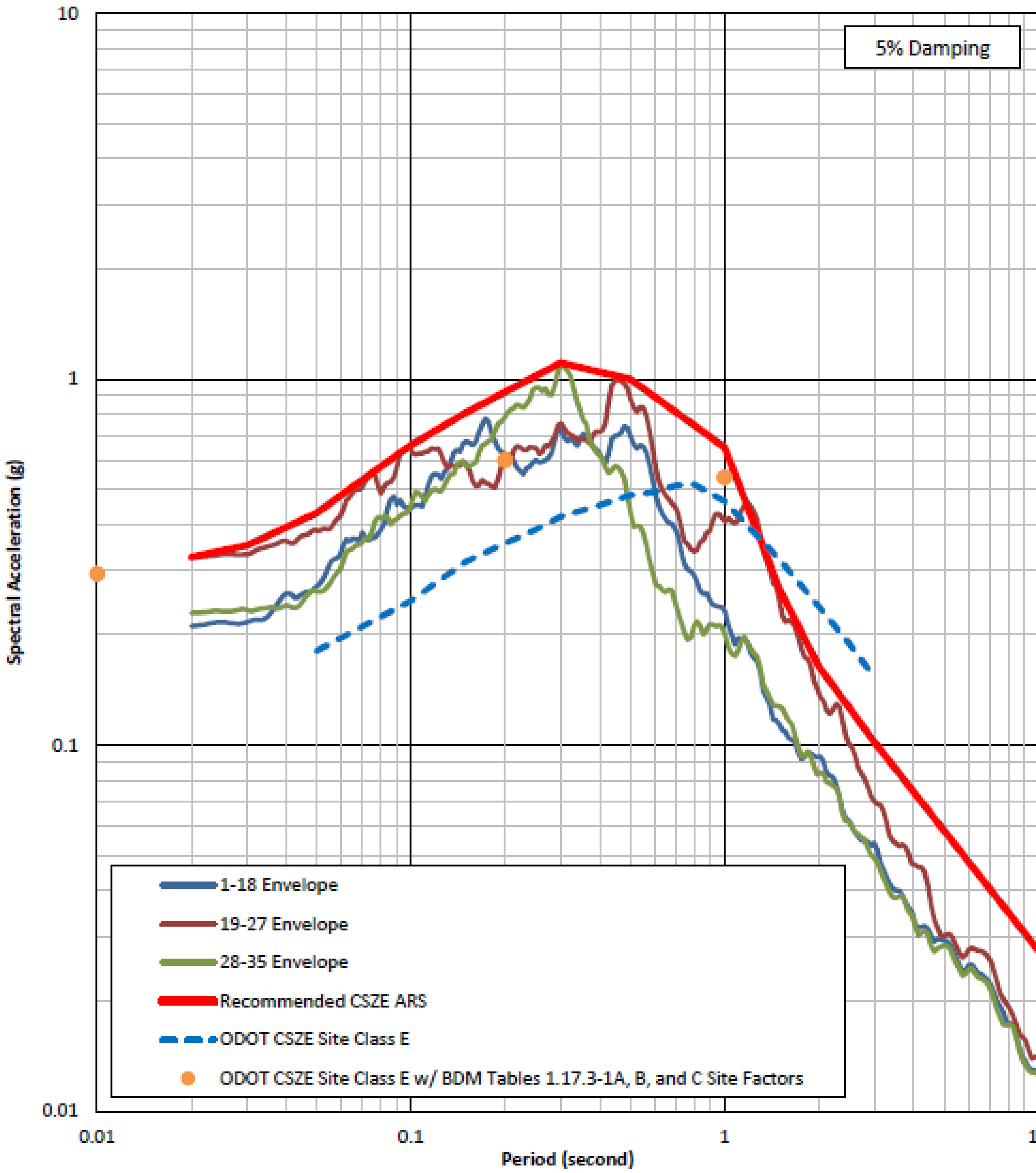
Appendix A
Project Site Specific Seismic Hazard Maps

BSB-Envelope.xlsx 5/22/2017

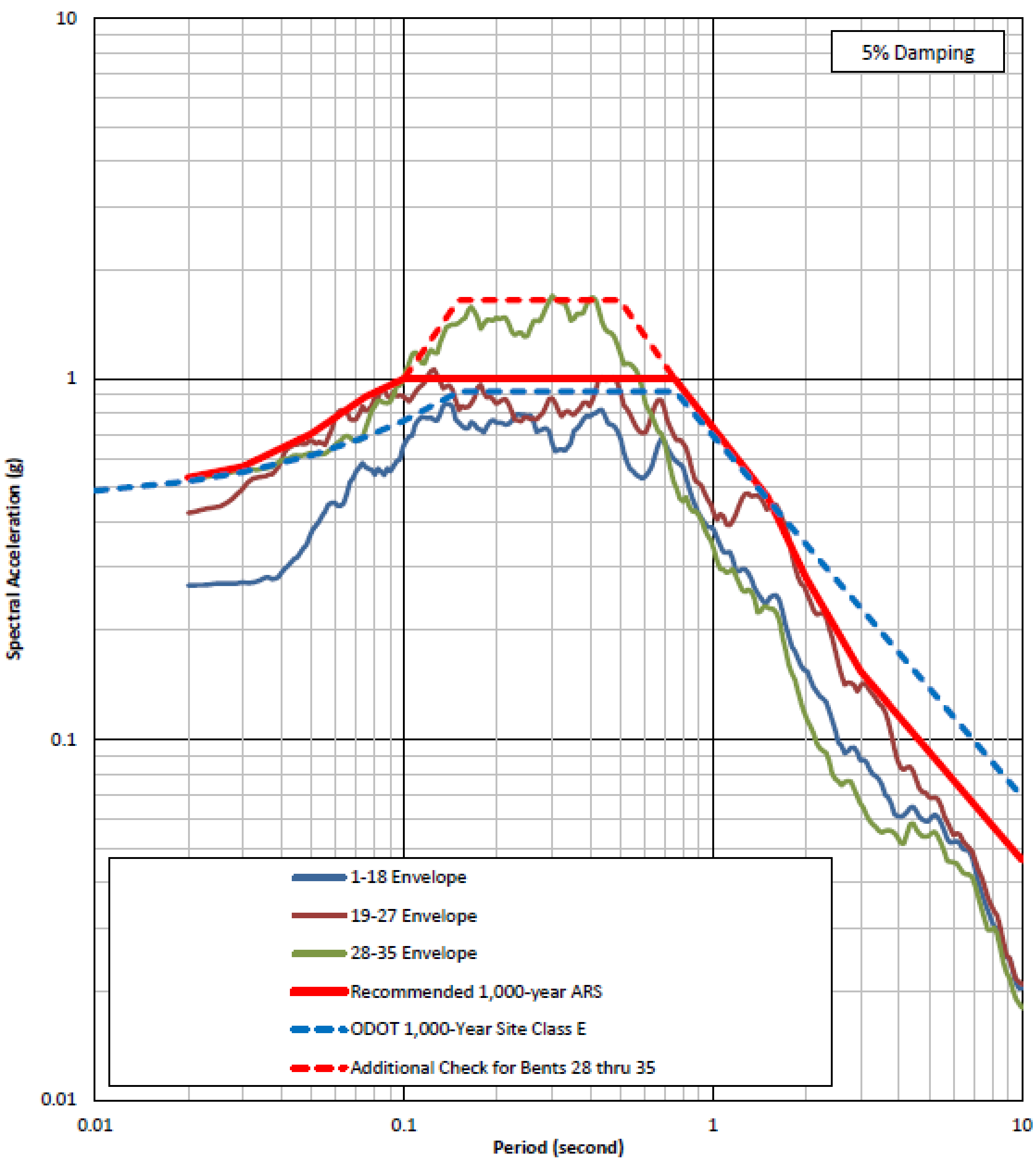


NOTES

1. The ODOT CSZE Site Class E spectral values were estimated for Vs30 of 150 m/sec by extrapolating the spectral values from Vs30 of 270 and 200 m/sec.
2. Recommended Spectrum at long periods were obtained by taking two-thirds of ODOT CSZE Site Class E linearly extrapolated (in log scale) above 3.0 sec.
3. g = standard gravitational acceleration; CSZE = Cascadia subduction zone earthquake; ARS = acceleration response spectrum; ODOT = Oregon department of transportation; Vs30 = time-averaged mean shear wave velocity for 30 meters subsurface; m/sec = meters per second; BDM = bridge design and drafting manual

Burnside Bridge Seismic Feasibility Study Portland, Oregon	
CSZ SPECTRA	
May 2017	24-1-04065-004
SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS	FIG. 4

BSB-Envelope.xlsx 5/22/2017



NOTES

1. g = standard gravitational acceleration; ARS = acceleration response spectrum; ODOT = Oregon department of transportation

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

1,000-YEAR SPECTRA

May 2017 | 24-1-04065-004

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FIG. 5

Appendix B
Probabilistic Seismic Hazard Graphics

Oregon Department of Transportation - **Bridge Section**

Project Name:	Burnside Bridge	*Key Number:*	
Highway:	Burnside Street	*Milepost:*	
Structure:		*Structure No.*	
County:	Multnomah		
Designer:	Drahota		

Tuesday, December 13, 2016

Project Type

Existing Bridge - Seismic Retrofit

EQ Return Period

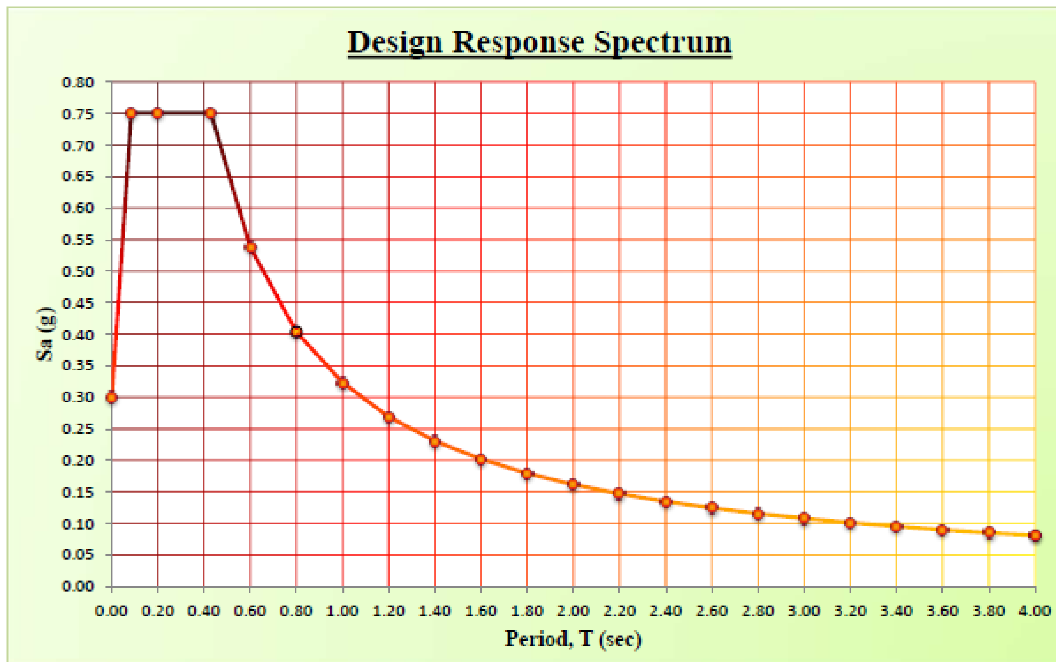
1000 Years ~ (Life Safety Criteria)

Latitude (Deg)	Longitude (Deg)
45.5231	-122.6675
(42°00' to 46°18')	(-116°27' to -124°34')

2014 USGS Seismic Hazard Data

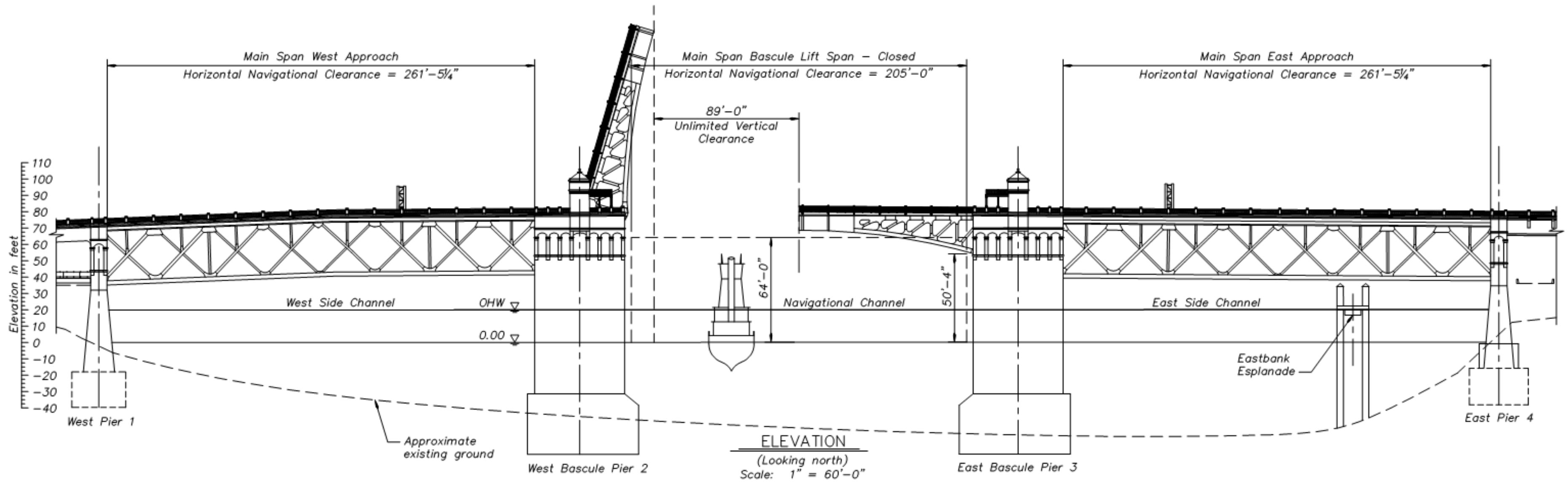
Site Class	PGA (g)	S_s (g)	S_1 (g)	F_{pga}	F_a	F_v	A_s (F_{pga}*PGA)	S_{DS} (F_a*S_s)	S_{D1} (F_v*S_1)	SDC
C	0.2713	0.5950	0.2153	1.2000	1.2620	1.5000	0.3256	0.7509	0.3230	III

Period, T (sec)	S_a (g)
0.00	0.3004
0.09	0.7509
0.20	0.7509
0.43	0.7509
0.60	0.5383
0.80	0.4037
1.00	0.3230
1.20	0.2691
1.40	0.2307
1.60	0.2019
1.80	0.1794
2.00	0.1615
2.20	0.1468
2.40	0.1346
2.60	0.1242
2.80	0.1153
3.00	0.1077
3.20	0.1009
3.40	0.0950
3.60	0.0897
3.80	0.0850
4.00	0.0807
4.00	0.0807

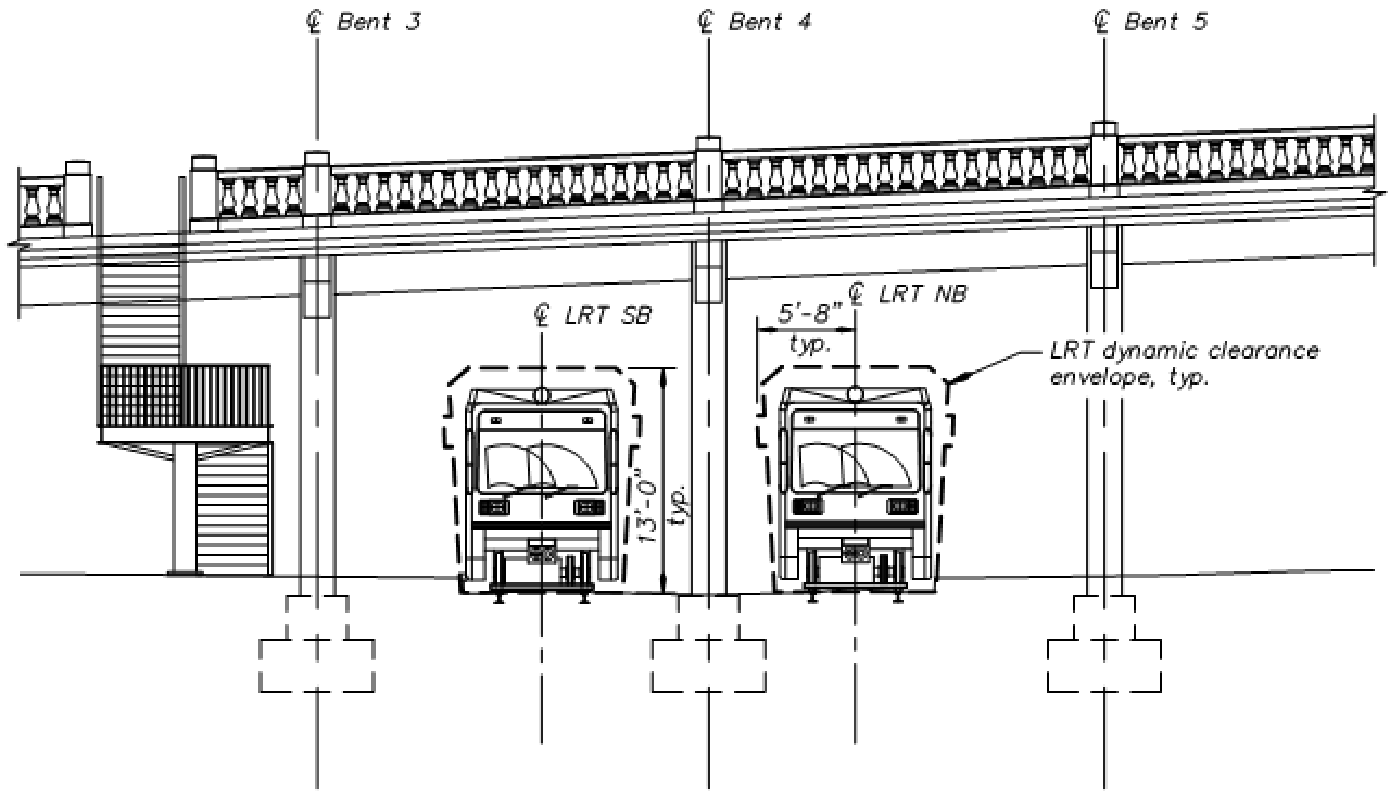


ODOT_ARS_V2014.16

Appendix C
River Navigational Clearances

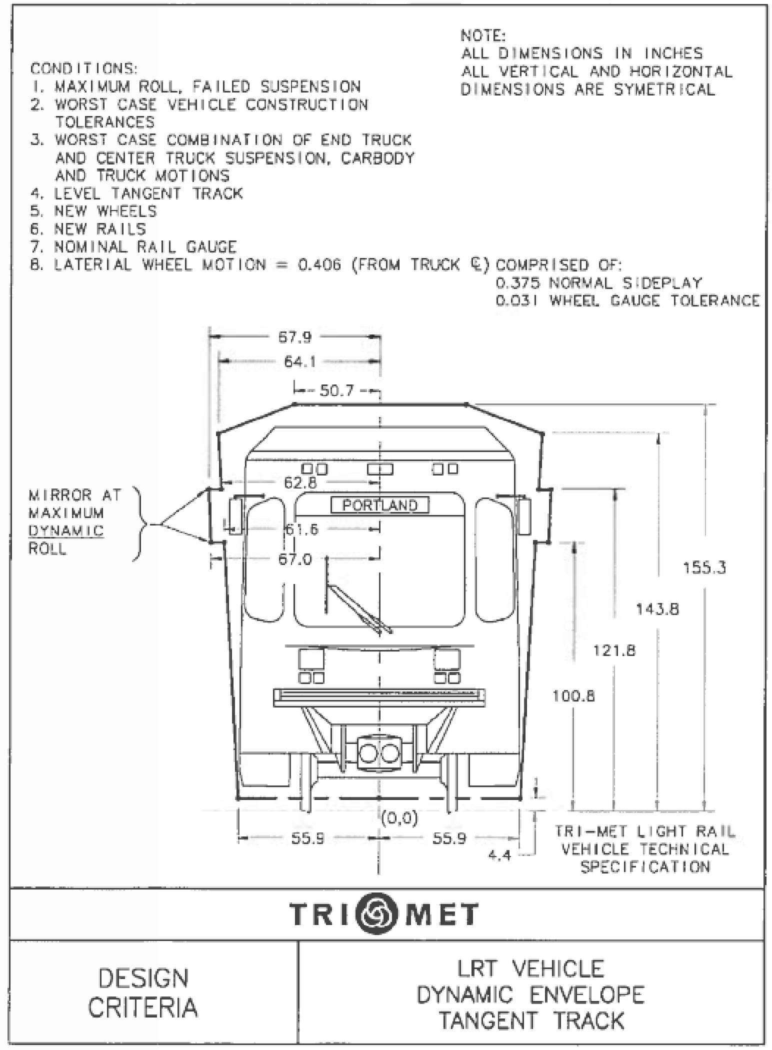

Main Span West Approach
Horizontal Navigational Clearance = 261'-5¼"
Main Span Bascule Lift Span – Closed
Horizontal Navigational Clearance = 205'-0"
Main Span East Approach
Horizontal Navigational Clearance = 261'-5¼"
89'-0"
Unlimited Vertical Clearance
Elevation in feet
110
100
90
80
70
60
50
40
30
20
10
0
-10
-20
-30
-40
West Side Channel
OHW
0.00
64'-0"
Navigational Channel
50'-4"
East Side Channel
Eastbank Esplanade
West Pier 1
Approximate existing ground
West Bascule Pier 2
ELEVATION
(Looking north)
Scale: 1" = 60'-0"
East Bascule Pier 3
East Pier 4

Appendix D
TriMet Light Rail Clearances

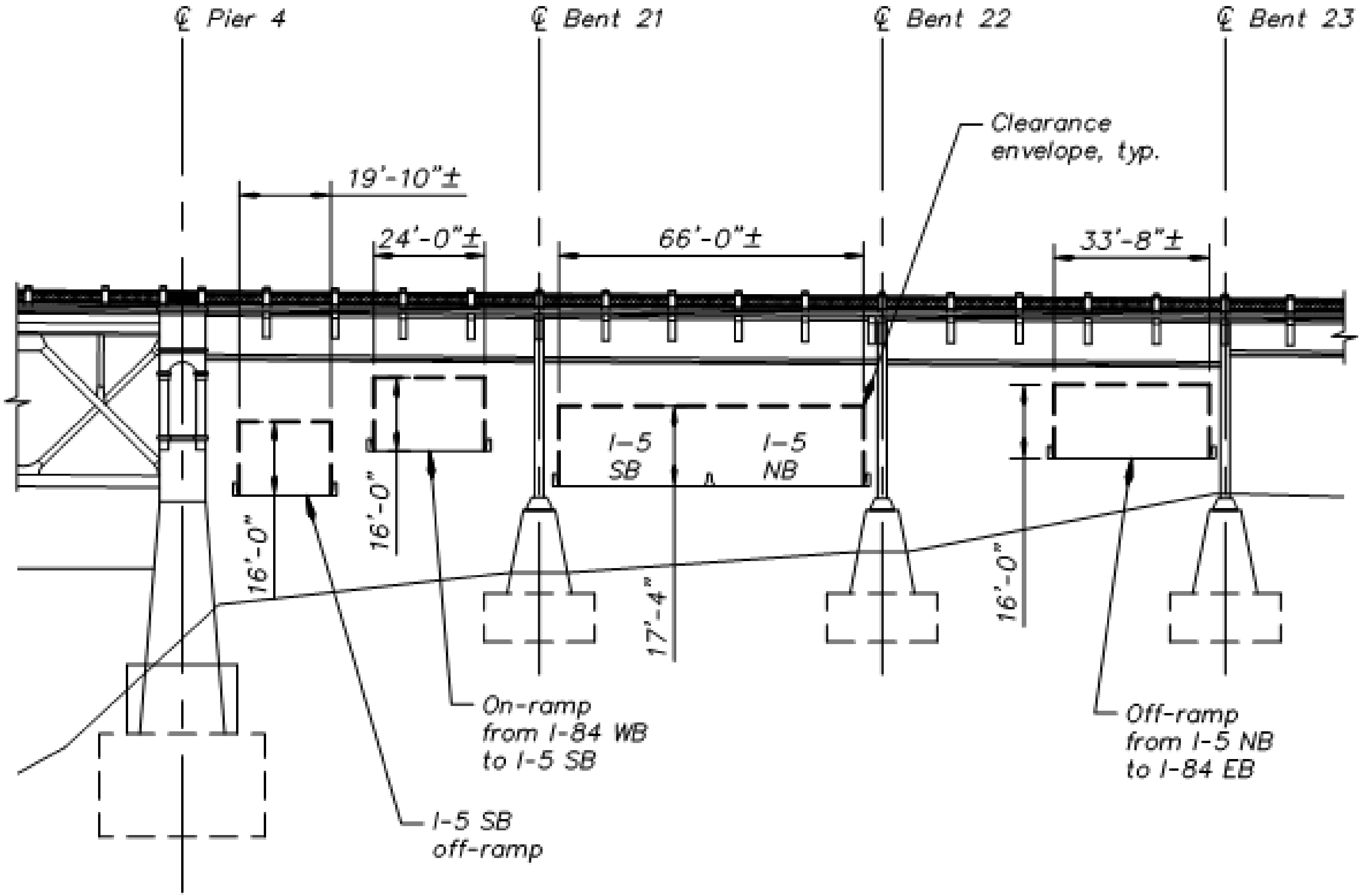


SPANS 3 & 4
CLEARANCE ENVELOPES UNDER BURNSIDE BRIDGE
(Looking north)

FIGURE 8.3.E.2 LRT VEHICLE DYNAMIC ENVELOPE TANGENT TRACK

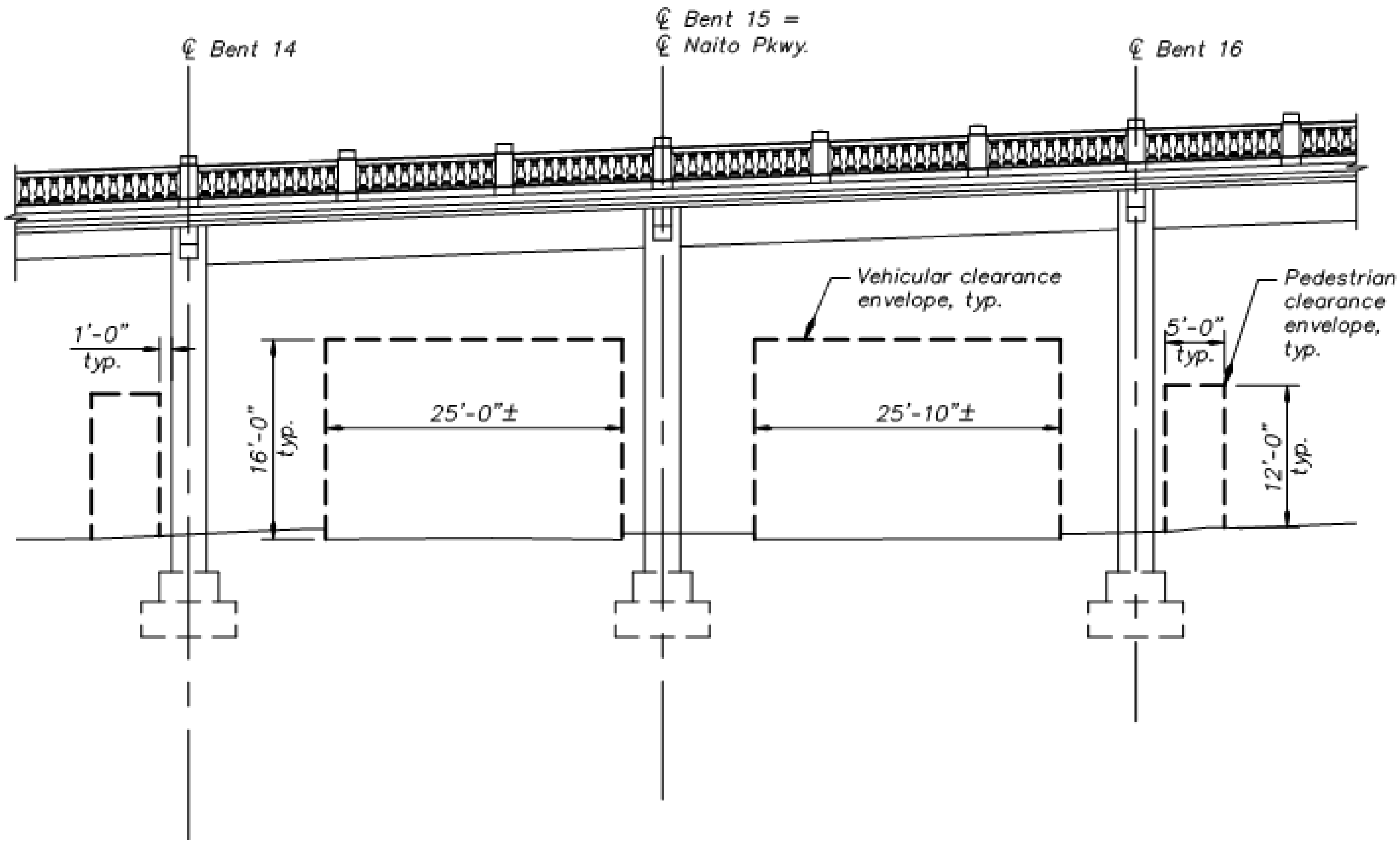


Appendix E
ODOT Facility Clearances

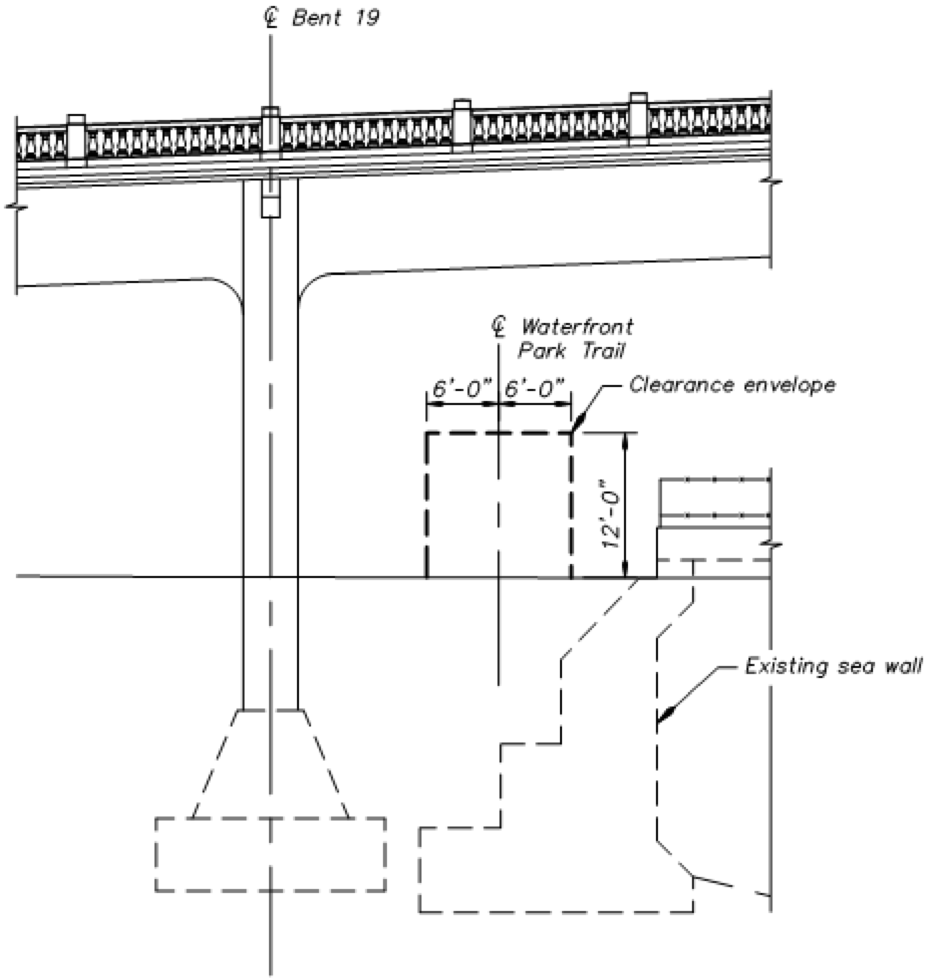


SPANS 20, 21 & 22
CLEARANCE ENVELOPES UNDER BURNSIDE BRIDGE
(Looking north)

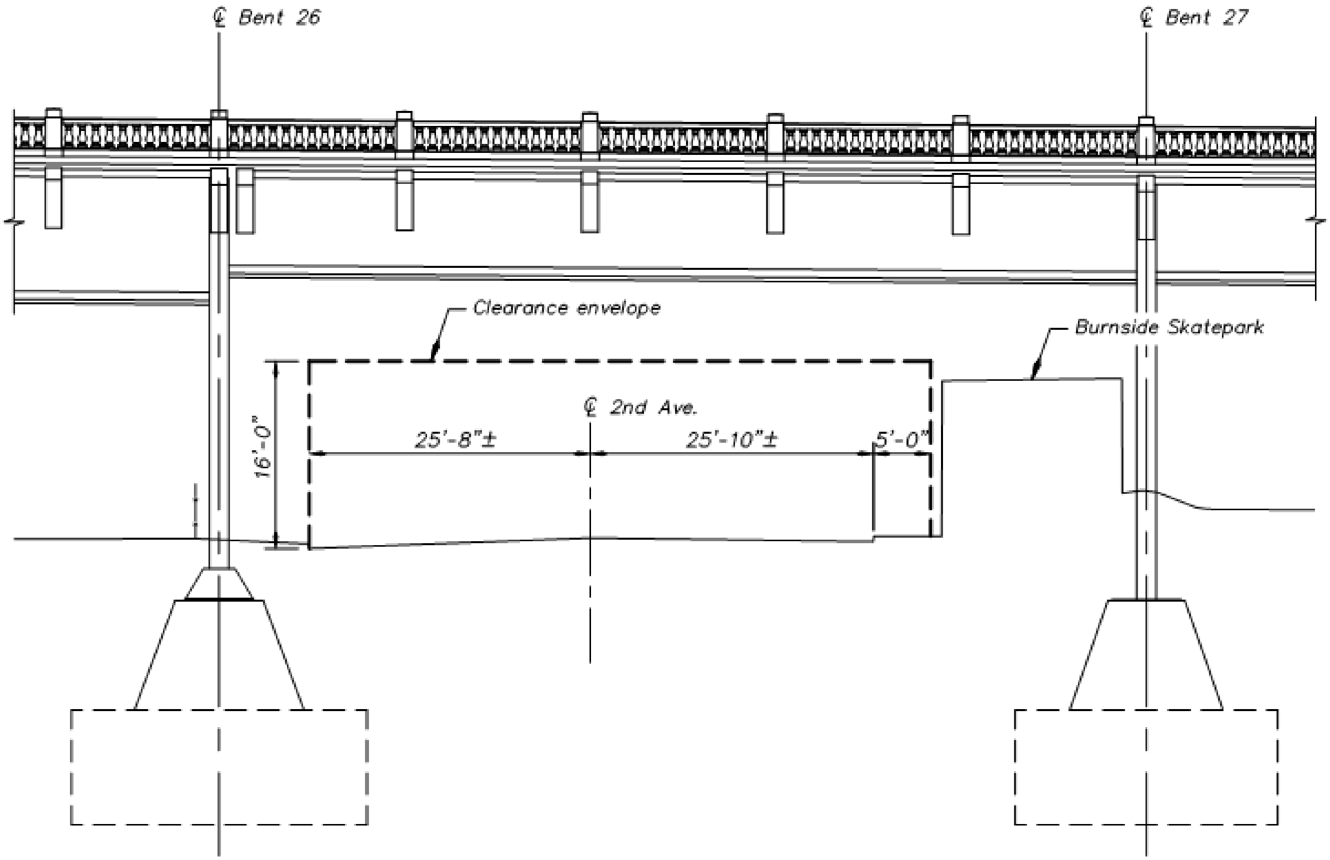
Appendix F
City of Portland Facility Clearances



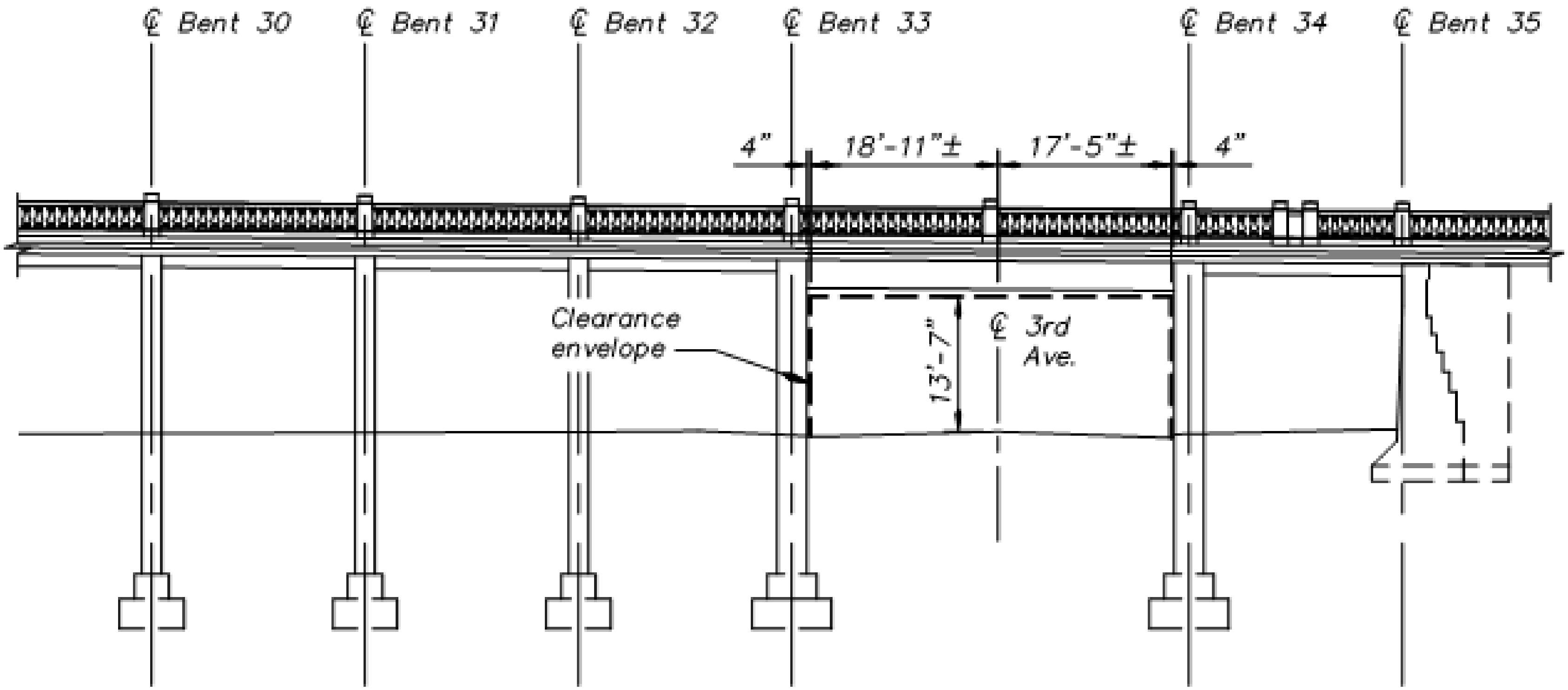
SPANS 14 & 15
CLEARANCE ENVELOPES UNDER BURNSIDE BRIDGE
(Looking north)



SPAN 19
CLEARANCE ENVELOPE UNDER BURNSIDE BRIDGE
(Looking north)

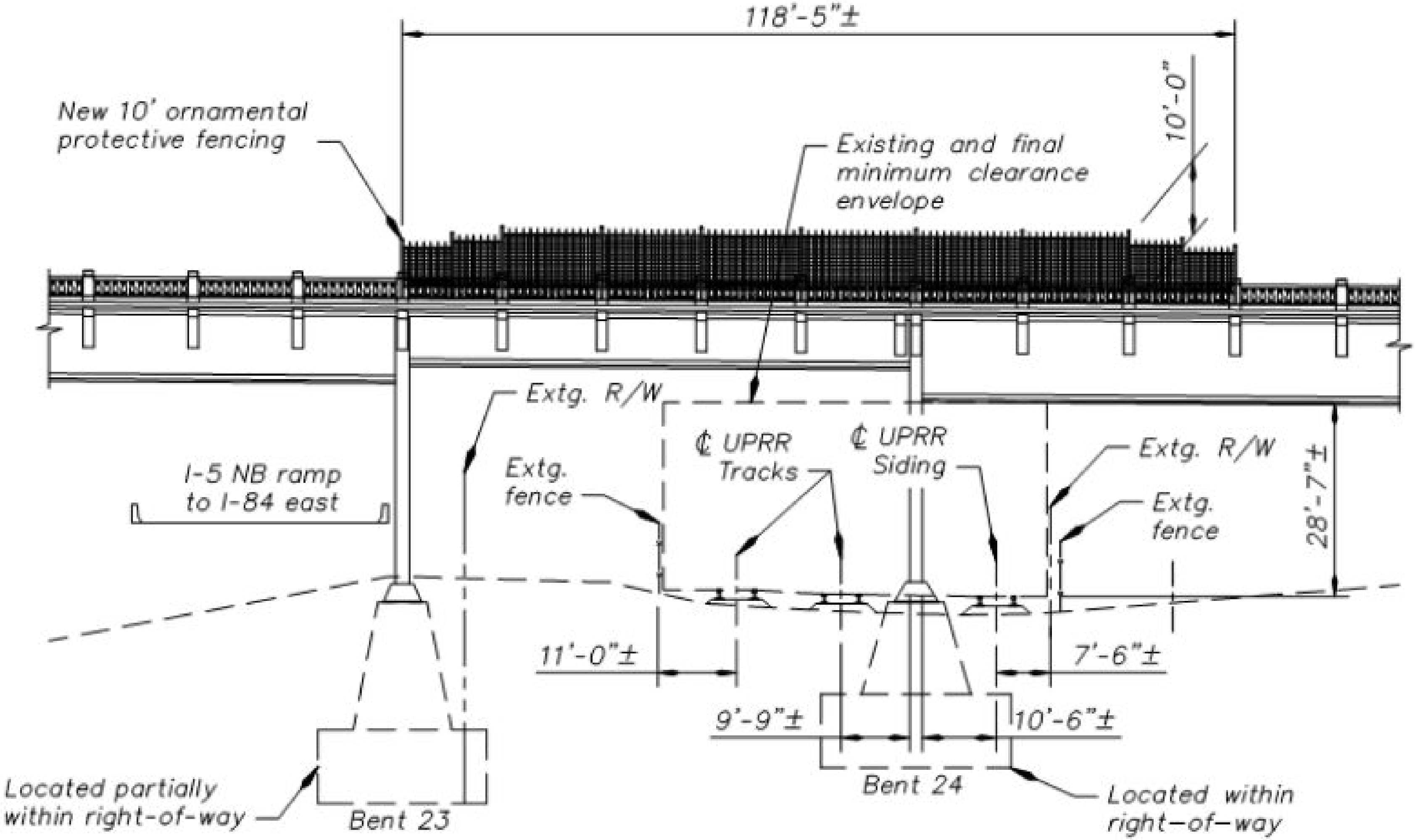


SPAN 26
CLEARANCE ENVELOPE UNDER BURNSIDE BRIDGE
(Looking north)



SPANS 30, 31, 32 & 34
CLEARANCE ENVELOPES UNDER BURNSIDE BRIDGE
(Looking north)

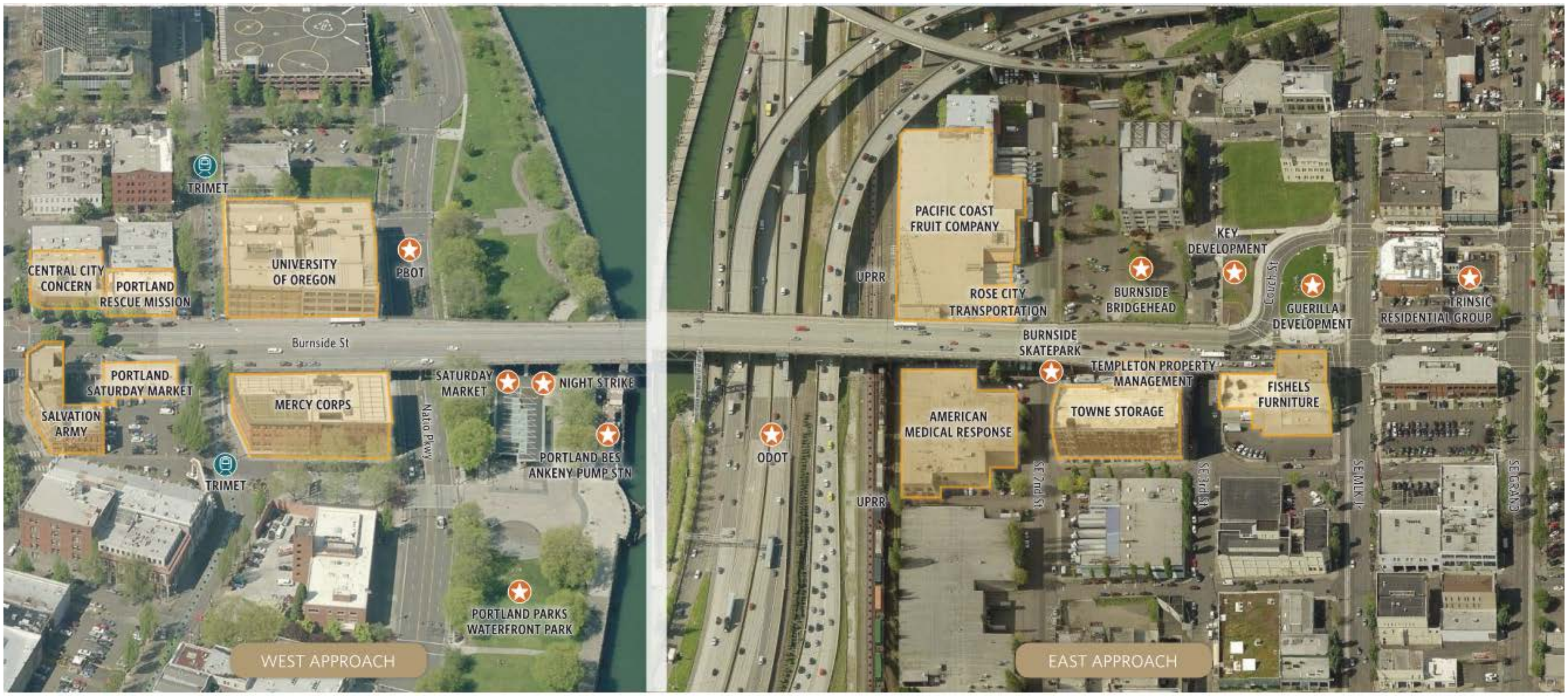
Appendix G
Union Pacific Railroad (UPRR) Clearances



UPRR CLEARANCE ENVELOPE UNDER BURNSIDE BRIDGE

(Looking north)

Appendix H
Private Building Locations



FOR MORE INFORMATION, VISIT THE PROJECT WEBSITE AT **BURNSIDEBRIDGE.ORG**

Multnomah County

Oregon Department of Transportation

HDR

Appendix C. Burnside Bridge Geotechnical Report

Geotechnical Report
Burnside Bridge Seismic Feasibility Study
Portland, Oregon

September 13, 2017

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Geotechnical Report
Burnside Bridge Seismic Feasibility Study
Portland, Oregon

September 13, 2017

SHANNON & WILSON, INC.

GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Excellence. Innovation. Service. Value
Since 1954

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24-1-04065-005

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PORTLAND, OREGON

1.0 INTRODUCTION

1.1 Project Overview

This report presents the results of our geotechnical research, field explorations, laboratory testing, analyses, and design recommendations for the Burnside Bridge Seismic Feasibility Study in Portland, Oregon. The Multnomah County Burnside Bridge Seismic Feasibility Study 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 Seismic Feasibility Study, the County and their consulting team, led by HDR Engineering, Inc. (HDR), will identify potential bridge seismic retrofit, rehabilitation, and/or replacement alternatives. Shannon & Wilson, Inc., 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)
- ODOT Bridge Design and Drafting Manual (BDDM) – October 2016
- ODOT Geotechnical Design Manual (GDM) – December 2016

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

The recommendations in this report 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 encountered in the three borings we 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 HDR Task Order #7, dated September 8, 2016, and our Master Subconsultant Agreement with HDR, dated October 20, 2014. The completed geotechnical design services for the project consisted of the following tasks:

- Review available existing information and visit the site to observe existing site conditions, geologic hazards, site access for the field explorations, and mark proposed exploration locations;
- Develop a field exploration work plan and obtain necessary permits and permissions to perform the field explorations, including right-of-entry and in-water work permits;
- Explore the subsurface conditions with three geotechnical borings and collect soil samples, including in situ geophysical testing (OYO Suspension Logging) in each boring to obtain shear wave velocity measurements;
- Conduct laboratory testing on selected soil samples to characterize soils and develop soil properties for evaluation;
- Provide two bedrock ground motions: a deterministic Cascadia Subduction Zone (CSZ) earthquake event corresponding to full rupture of the subduction zone interface and a probabilistic ground motion corresponding to a 1,000-year return period;
- Perform site-specific ground response analyses using an effective stress nonlinear analysis (FLAC numerical modeling computer program);
- Provide smoothed horizontal and vertical seismic design spectra at existing bent/pier locations for the two ground motion return periods based on the site response analyses;
- Evaluate the site-specific seismic hazards, including liquefaction potential, lateral spreading, and other seismic-related hazards;
- Provide foundation modeling parameters (springs) for structural seismic response analysis of the existing bridge foundations;
- Analyze the effects of liquefaction on the existing bridge, including lateral spreading, settlement, pile downdrag forces, and slope instability;

- Provide conceptual-level alternatives for liquefaction mitigation consistent with recommended bridge retrofit strategies;
- Provide foundation modeling parameters (springs) for structural seismic response analysis of drilled shafts as part of a seismic retrofit alternative;
- Prepare a Geotechnical Report summarizing our research, field explorations, laboratory testing, and geotechnical design recommendations.

2.0 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 (Vera Katz Waterfront Park Seawall) with a level fill surface at about elevation 35 feet behind the wall. 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 Seismic Feasibility Study is to identify potential bridge seismic retrofit, rehabilitation, and replacement alternatives for the existing Burnside Bridge. At the conclusion of the Feasibility Study, it is anticipated that one seismic retrofit alternative, one rehabilitation alternative, and three bridge replacement alternatives will be advanced in the potential National Environmental Policy Act (NEPA) study. We understand that bridge replacement options will include high-elevation fixed bridges, low-elevation movable bridges, and a tunnel. Exact bridge types are yet to be determined.

The project scope of services specifies two earthquake ground motion performance levels for evaluation and retrofit of the bridge: a "Full Operation" Performance Level for CSZ event ground motions and a "Limited Operation" Performance Level for probabilistic 1,000-year return period ground motions.

3.0 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 Table 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 Table 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.

TABLE 1: AS-CONSTRUCTED FOUNDATION SUMMARY FOR 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.

TABLE 2: AS-CONSTRUCTED FOUNDATION SUMMARY FOR DRIVEN PILES

Location	Number of Piles	[1]Pile Cap Dimensions (W x L x H) (ft)	[2]Pile Type and Section	[3]Approximate Bottom Pile Cap Elevation (ft)	[3]Approximate Pile Tip Elevation (ft)	Approximate Pile Length (ft)	[4]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:
[1] W = Pile cap dimension in longitudinal direction (perpendicular to bent/pier centerline), L = Pile cap dimension in transverse direction (parallel to bent/pier centerline)
[2] Pile type and section are not shown in the plans, therefore pile type and section is assumed.
[3] Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set.
[4] Bearing material is interpreted from the information in the plan set, existing borings, and current borings.

4.0 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 fore-arc 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 fore-arc blocks (e.g., Pezzopane and Weldon, 1993; Wells and others, 1998; Wells and Simpson, 2001). As a result, the southern portion of the fore arc, 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 fore arc 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 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 (M_w) 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 M_w 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 M_w 6.7 Olympia earthquake, the 1965 M_w 6.7 earthquake between Tacoma and Seattle, and the 2001 M_w 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 M_w 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 M_w 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 M_w 6.5 to 7.0. Other examples include the 1993 M_w 5.6 Scotts Mill earthquake and the 1993 M_w 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 Table 3. 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).

TABLE 3: 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[a]	Slip Rate Category[b]	Time Since Last Deformation[c]
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:

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

5.0 FIELD EXPLORATIONS

5.1 Existing 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, Existing 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 Geotechnical Explorations

Shannon & Wilson 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.

6.0 LABORATORY TESTING

The samples we obtained during our field explorations were transported to our laboratory for further visual examination. We then selected representative samples for a suite of laboratory tests. 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.

7.0 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. 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. Therefore, we assumed a groundwater elevation of 20 feet in our analysis.

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.

8.0 SEISMIC GROUND MOTIONS AND HAZARD EVALUATIONS

Seismic design for the Burnside Bridge Seismic Feasibility Study is performed following guidelines presented in the ODOT GDM (ODOT, 2016), ODOT BDDM (ODOT, 2016b), AASHTO LRFD Bridge Design Specifications (AASHTO, 2014), and the Burnside Bridge Earthquake Readiness Seismic Design Criteria. In accordance with the project Seismic Design Criteria, the full-rupture CSZ event and 1,000-year ground motion levels are considered for the seismic design. The Performance Level of the CSZ event will be “Full Operation.” For the 1,000-year event, the bridge will be designed for a “Limited Operation” Performance Level.

Site-specific ground response analyses were performed to develop site-specific design ground motions. The ground response analyses included the following steps:

1. Develop base ground motions. Base ground motions are the bedrock ground motions; a deterministic CSZ event corresponding to full rupture of the subduction zone interface and a probabilistic ground motion corresponding to a 1,000-year return period.
    a. Develop base motion time history target spectra.
    b. Develop earthquake time histories that closely match the target spectra.
2. Develop a soil model of the site for one-dimensional dynamic wave propagation (site response) analyses.
3. Propagate base ground motion time histories through the soil model and calculate the response spectra at the existing bent/pier locations in the soil model.

Site response analyses estimate seismic shaking at the ground surface of a soil model based on earthquake time histories applied to the base of the model. Site response analyses can be performed using either equivalent-linear, total-stress analyses methods or nonlinear, effective-stress analyses. Equivalent-linear, total-stress analyses estimate the ground surface response without explicitly modeling pore pressure generation and liquefaction. Nonlinear, effective-stress analyses explicitly model pore pressure generation and liquefaction on the surface response.

Shannon & Wilson performed nonlinear effective stress analyses using the numerical modelling suite FLAC (Fast Lagrangian Analysis of Continua, Itasca, 2016) to perform the analyses. The nonlinear effective stress analyses consider pore pressure development and provide estimates of soil shear strength reductions and permanent ground deformations due to strong shaking. The analyses were performed for the "Full Operation" Performance Level: CSZ event and "Limited Operation" Performance Level: 1,000-year ground motion levels.

We created a free-field model of the soil profile along the Burnside Bridge; our model did not include the bridge structural elements. We then 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 to apply dynamic (i.e. earthquake) loads to the base of the model. 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 Youd et al (2001).

8.1 Base Ground Motions

With the model prepared for dynamic loading, we applied each of the scaled earthquake time histories. We developed suites of six spectrum-compatible ground motion time histories (three for each of the two ground motion levels) for Site Class B/C boundary soil conditions that correspond to the soil conditions at the base of the soil model.

The following time histories were selected and scaled for both "Full Operation" and "Limited Operation" Performance Levels:

- M_w 8.8 Maule 2010, Station: Cerro Santa Lucia, UCS Sta STL, Component: 360°, Source-to-site distance: 77 km (designated M-CSZ and M-1000 for "Full Operation" and "Limited Operation" Performance Levels, respectively)

- M_w 9.0 Tohoku 2011, Station: AKT023, Tsubakidai, Component: EW, Source-to-site distance: 105 km (designated T-CSZ and T-1000 for “Full Operation” and “Limited Operation” Performance Levels, respectively)

The following time history was selected and scaled for the “Full Operation” Performance Level:

- M_w 8.8 Maule 2010, Station: Punta de Chungos, UCS Sta VICH, Component: 360°, Source-to-site distance: 178 km (designated MV-CSZ)

The following time history was selected and scaled for the “Limited Operation” Performance Level:

- M_w 6.9 Loma Prieta 1989, Station: San Jose - Santa Teresa Hills, Component: 225°, Source-to-site distance: 15 km (designated LP-1000)

8.2 Ground Surface Response Spectra

To develop the design response spectra for each seismic Performance Level, we calculated hazard-consistent geometric mean of the response spectra estimated from the three spectrum-compatible time histories. Depending on the characteristics of the soil deposit and its response to the base ground motion time histories at the base of the soil model, the ground surface response spectra were combined in three groups: Bents 1 through 18, Bents 19 through 27 (including Piers 1 through 4), and Bents 28 through 35. The envelope of the site-specific response spectra at each bent group was calculated and plotted in Figures 5 and 6 for “Full Operation” and “Limited Operation” Performance Levels, respectively.

Figure 5 shows that the code-based (ODOT) CSZ event response spectrum is significantly lower than our calculated ground surface response spectra, particularly for shorter periods. We believe the lower ODOT CSZ event response spectrum is the result of lower site response terms for Site Class E in the current subduction ground motion prediction equations that are used by the ODOT web-based application compared to ODOT BDDM code-based site factors F_{pga}, F_a, and F_v (i.e., ODOT BDDM Tables 1.17.3-1A, B, and C). The ODOT site factors are consistent with the current study on the amplification factors observed for crustal ground motion dataset. For comparison, we calculated the adjusted Site Class E ODOT CSZ event response spectrum by obtaining the Site Class B/C boundary ODOT CSZ event response spectrum from the ODOT web-based application (ODOT, 2017) and applying ODOT BDDM Site Class E site factors (i.e., Tables 1.17.3-1A, B, and C). These adjusted Site Class E CSZ event spectral values were plotted in Figure 5 at periods of 0, 0.2, and 1.0 second. As observed from this figure, the adjusted Site Class E CSZ event response spectrum is increased for short periods and is consistent with calculated ground surface response spectra.

8.3 Recommended Seismic Design Ground Motions

Shannon & Wilson developed the recommended smoothed design response spectrum at the bridge location from the three site-specific ground surface spectra for the three bent groups. AASHTO (2014) does not permit seismic design using spectral values less than two-thirds of the code-based design spectrum. Where the two-thirds spectrum for Site Class E is greater than the site-specific ground surface spectrum, AASHTO requires the site-specific spectrum to follow two-thirds of the corresponding Site Class E spectrum. At the Burnside Bridge, the anticipated mean surface response is less than two-thirds of the Site Class E code-based response spectrum at spectral periods beyond approximately 3.0 seconds. Therefore, our recommended ground surface site-specific spectrum for the bridge site follows the two-thirds code-based spectrum for spectral periods longer than 3.0 seconds and follows the anticipated mean surface response for the periods shorter than 3.0 seconds. The recommended design spectra are plotted in Figures 5 and 6 for "Full Operation" and "Limited Operation" seismic Performance Levels, respectively. As observed from Figure 5, for the "Full Operation" Performance Level, the recommended design spectrum was selected to envelope the three bent groups site-specific ground surface response spectrum. For the "Limited Operation" Performance Level (Figure 6), the soil response for Bents 28 through 35 are significantly higher than the other bents at periods between 0.1 and 0.75 seconds and therefore, our recommended design spectrum was principally created to envelope the three bent groups site-specific ground surface response spectrum, with an additional check for Bents 28 through 35 using an elevated design spectrum for periods between 0.1 and 0.75 second.

Table 4 provides the recommended site-specific ground surface design response spectra for "Full Operation" and "Limited Operation" seismic Performance Levels.

TABLE 4: RECOMMENDED SEISMIC DESIGN SPECTRAL ACCELERATIONS

Period (seconds)	"Full Operation" Performance Level (CSZ Event)	"Limited Operation" Performance Level (1,000-Year Return Period)	
	Bents 1 through 35	Bents 1 through 27	Bents 28 through 35
0.02	0.326	0.532	0.532
0.03	0.350	0.569	0.569
0.05	0.430	0.703	0.703
0.075	0.553	0.891	0.891
0.1	0.660	1.000	1.000
0.15	0.809	1.000	1.650
0.2	0.921	1.000	1.650
0.3	1.106	1.000	1.650
0.5	1.000	1.000	1.650
0.75	0.777	1.000	1.000
1	0.650	0.731	0.731
1.5	0.265	0.470	0.470
2	0.163	0.280	0.280
3	0.102	0.154	0.154
5	0.059	0.093	0.093
7.5	0.0376	0.062	0.062
10	0.0275	0.0463	0.0463

8.4 Seismic Hazards 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 ½ 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 the Burnside Bridge Seismic Feasibility Study were utilized to evaluate liquefaction and its associated impacts. A brief summary of the analyses and results is presented in the following sections.

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

Appendix E, Figures E1 through E6, presents contour plots of the excess pore pressure ratio based on the nonlinear effective stress model 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 $(N_1)_{60}$ or $(N_1)_{60\text{-cs}}$ 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. For excess pore pressure ratios between 0 and 0.9, reduced shear strengths are determined as a function of the excess pore pressure accumulation.

Figures E7 through E44 present the excess pore pressure ratio versus depth for each input ground motion at each existing bent/pier location for the 1,000-year return period ground motion level. The plots also present the maximum shear strain, relative horizontal displacement, and vertical displacement versus depth for each existing bent/pier location. Figures E45 through E82 present the same information for the CSZ event ground motion level.

Please see the "Existing Foundation Resistance and Stiffness" section (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.

8.4.2 Liquefaction-Induced Lateral Spreading and Flow Failure

Figures E1 through E6, Appendix E, present contour plots of estimated permanent ground displacement 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 riverbanks to varying displacements and elevations for the ground motion levels considered. For the 1,000-year ground motion level, ground surface movements up to 14 feet are calculated for the west riverbank. Flow failure with displacements in excess of approximately 60 feet is anticipated at the east riverbank. For the CSZ event ground motion level, ground surface movements up to 3 and 23 feet are anticipated at the west and east riverbanks, respectively.

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.

8.4.3 Liquefaction-Induced Settlement

We estimated liquefaction-induced settlement using the average of the maximum shear strains from the three input ground motions for each ground motion level, determined in the nonlinear effective stress models. 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.

Table 5 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.

TABLE 5: ESTIMATED LIQUEFACTION-INDUCED SETTLEMENT AT EXISTING SPREAD FOOTING FOUNDATIONS

Location	Liquefaction-Induced Settlement at Bottom of Footing (in)	
	CSZ Event	1,000-Year Return Period
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
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

Table 6 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.

TABLE 6: 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

9.0 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 Table 5 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 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 condition for the preferred mitigation and retrofit alternative are presented in Section 11.

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 Table 7. 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 BDDM or the FHWA Seismic Retrofitting Manual for Highway Structures. Table 7 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 Table 7, 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 BDDM 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 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 static and seismic frictional sliding coefficient for the existing footings; the results are presented in Table 7 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 Table 7 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 BDDM Section 1.1.4.2 allows the use of a wall height-adjusted pressure value of 5 ksf for calculating seismic translational 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 (EFP_{pE}) 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.

TABLE 7: 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., tan δ	[e]Bearing Material Initial Shear Modulus, (ksi)	Poisson's Ratio	[f]Lateral Earth Coefficients						[f,g,m]Lateral Earth Pressures (psf)					
											K_o	K_a	K_p	[h]ΔK_{oE}	[h]ΔK_{aE}	[h]K_{pE}	[i]EFP_o	[i]EFP_a	[i]EFP_p	[j,k]ΔEFP_{oE}	[j,k]ΔEFP_{aE}	[k,l]EFP_{pE}
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]
Abutments	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]
Footings	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]
Footings	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]
Footings	Bent 28	22 (27)	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]
Footings	Bents 29 through 32	40 (10)	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]
Footings	Bents 33 and 34	37 (12)	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]
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]
Pile Caps	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]

TABLE 7: RECOMMENDED UNFACTORED STATIC AND SEISMIC SOIL PARAMETERS FOR EXISTING SPREAD FOOTINGS AND PILE CAPS (CON'T)

	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	[f]Lateral Earth Coefficients						[f,g,m]Lateral Earth Pressures (psf)					
											K_o	K_a	K_p	[h]ΔK_{oE}	[h]ΔK_{aE}	[h]K_{pE}	[i]EFP_o	[i]EFP_a	[i]EFP_p	[j,k]ΔEFP_{oE}	[j,k]ΔEFP_{aE}	[k,l]EFP_{pE}
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]
Pile Caps	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]
Pile Caps	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]
Pile Caps	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]
Pile Caps	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 mean 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 BDDM 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 BDDM 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 Table 6 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 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 condition for the preferred mitigation and retrofit alternative are presented in Section 11.

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 Table 8. 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.

TABLE 8: 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.

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. We understand that HDR will use the load-displacement relationships to develop the stiffness matrix at the bottom of the pile cap. 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 and Proposed Drilled Shaft 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 Table 7.

10.0 CONCEPTUAL SEISMIC MITIGATION DESIGN

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 mitigation and retrofit may be required at the following existing bridge foundations:

- 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 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 HDR, 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. In addition, we understand the existing bridge between Bents 17 and 27 may be widened. The following sections present our conceptual-level design recommendations for seismic mitigation consistent with the proposed bridge retrofit and widening strategies.

10.1 West Approach (Bents 1-19)

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. 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. 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 assumed that ground improvement at Bents 1 through 16 would 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 underneath the existing seawall between Bent 19 and 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 about 25 feet and length of about 120 feet at each bent location (Bents 1 through 16), not including the area under the existing spread footings. We assumed a cellular soil-cement ground improvement width of about 40 feet and length of about 100 feet at the west riverbank. The estimated extents of cellular soil-cement ground improvement at the west riverbank are shown on Figure 7, Conceptual Ground Improvement Extents for Lateral Spread Mitigation. The total cellular soil-cement ground improvement volume at the west approach is approximately 50,000 cubic yards. Based on our similar project experiences, a soil-grout column replacement ratio (A_s) of 60 percent is assumed to calculate the treated volume for cellular soil-cement ground improvement using jet grouting. The typical unit cost for jet grout is about $400 per cubic yard on a replacement ratio basis. The construction mobilization and demobilization is about $1,200,000 (10 percent of the ground improvement construction cost). Therefore, the total cost estimate for the above conceptual ground improvement configuration at the west approach is about $13,200,000. This conceptual cost estimate does not include removal of the seawall or temporary shoring.

10.2 Main Span (Piers 1-4)

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 12 6-foot diameter drilled shafts, and Piers 2 and 3 will be supported by 14 10-foot diameter drilled shafts. Retrofit options for Pier 4 include enlarging the existing pile cap and supporting it with 12 6-foot diameter drilled shafts and a micropile array under the adjacent I-5 approach ramp (Option A), or constructing a new pier to the west which would be supported by 12 6-foot diameter drilled shafts (Option B). 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 and 10.3, respectively.

10.3 East Approach (Bents 21-35)

We understand the existing spread footings at Bents 28 through 35 will be enlarged, and the existing pile group foundations at Bents 21 through 27 may 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. We also understand that retrofitting Bents 21 through 24 with a drilled shaft superbent may not be feasible due to conflicts with the existing I-5 freeway. Therefore, existing Bents 21 through 24 may be removed altogether and replaced with a three-span structure between Pier 4 and Bent 25. Each superbent will consist of two 10-foot diameter drilled shafts adjacent to the pile caps, connected by a grade beam or infill wall that is also tied into the existing pile caps.

Seismic mitigation will be required to mitigate permanent ground displacement of the east riverbank. 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 would be performed using low-overhead jet grouting equipment to form two cellular soil-cement ground improvement zones: a primary zone between Pier 4 and the Eastbank Esplanade and a secondary zone between Bent 23 and the UPRR tracks. 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.

To develop conceptual-level cost estimate information, we estimated the lateral and vertical extents of potential cellular soil-cement ground improvement at the east riverbank. For the purpose of the conceptual level cost estimate, we used liquefiable layer thicknesses of 100 feet in front of Pier 4 and 120 feet between Bent 23 and the UPRR. We assumed a width of about 100 feet and length of about 230 feet for the primary cellular soil-cement ground improvement zone in front of Pier 4. A width of about 50 feet and length of about 200 feet was assumed for the secondary cellular soil-cement ground improvement zone between Bent 23 and the UPRR. The estimated extents of cellular soil-cement ground improvement at the east riverbank are shown on Figure 7, Conceptual Ground Improvement Extents for Lateral Spread Mitigation. The total cellular soil-cement ground improvement volume at the east riverbank is approximately 130,000 cubic yards. Based on our similar project experiences, a soil-grout column replacement ratio (A_s) of 60 to 65 percent is assumed to calculate the treated volume for cellular soil-cement ground improvement using jet grouting. The typical unit cost for jet grout is about $400 per cubic yard on a replacement ratio basis. The construction mobilization and demobilization is about $3,200,000 (10 percent of the ground improvement construction cost). Therefore, the total cost estimate for the above conceptual ground improvement configuration at the east riverbank is about $35,200,000. This conceptual cost estimate does not include the temporary sheet pile cofferdam for grout containment.

11.0 FOUNDATION RESISTANCE AND STIFFNESS FOR PREFERRED RETROFIT AND SEISMIC MITIGATION ALTERNATIVES

We developed foundation modeling parameters for post-seismic (liquefied) soil conditions for the preferred retrofit and seismic mitigation alternatives presented in Section 10. The post-seismic foundation modeling parameters consider soil strength reductions due to partial or full liquefaction (i.e. excess pore water pressure) as determined from our FLAC analysis. For this feasibility study, we did not perform a separate FLAC analysis that incorporates the proposed cellular soil-cement ground improvement.

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 seismic mitigation is anticipated at Bents 28 through 35. Table 9 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.

TABLE 9: SUMMARY OF SPREAD FOOTING FOUNDATIONS FOR PREFERRED RETROFIT AND SEISMIC MITIGATION ALTERNATIVE

Location	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	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:
[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, current borings, and the preferred seismic mitigation alternative.

11.1.1 Bearing Resistance

We estimated the nominal post-seismic 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 Table 10. The bearing resistances reported in the table are nominal geotechnical resistances and should be reduced by a resistance factor of 0.9 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 BDDM or the FHWA Seismic Retrofitting Manual for Highway Structures. Table 10 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 Table 10, 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 BDDM 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 frictional sliding coefficient for the retrofitted footings; the results are presented in Table 10 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 condition are also presented in Table 10 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.

TABLE 10: RECOMMENDED UNFACTORED POST-SEISMIC 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)	Fine-Grained Alluvium	110	29	---	4	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
	Bents 29 through 32	40 (10)	Fill	110	29	---	4	0.44	7	0.35	0.52	0.35	2.88	57D	39D	317D
	Bents 33 and 34	37 (12)	Fill	110	29	---	4	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	4	---	---[c]	---[d]	---	---	0.3	0.3	---[g]	14D	14D	---[g]
	Piers 2 and 3	-70 (16)	Sand Alluvium	125	10	---	---[c]	---[d]	---	---	0.3	0.3	---[g]	19D	19D	---[g]
	Pier 4 [k]	-40.3 (48)	Fine-Grained Alluvium	110	4	---	---[c]	---[d]	---	---	0.3	0.3	---[g]	14D	14D	---[g]
	Bents 21 and 22	12.5 (3.5)	Fill	110	4	---	---[c]	---[d]	---	---	0.3	0.3	---[g]	14D	14D	---[g]
	Bents 23 and 24	12.5 – 17.5 (11.5)	Fill	110	29	---	---[c]	---[d]	---	---	0.52	0.35	2.88	57D	39D	317D
	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 mean 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 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 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. 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.

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 riverbanks. We understand Bents 17 through 19 and 21 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 12 drilled shafts, and Piers 2 and 3 will be supported by 14 drilled shafts. Retrofit options for Pier 4 include enlarging the existing pile cap and supporting it with 12 drilled shafts and a micropile array under the adjacent I-5 approach ramp (Option A), or constructing a new pier to the west which would be supported by 12 drilled shafts (Option B). For the purposes of this feasibility study, we only evaluated Pier 4 supported by 12 drilled shafts in its existing location. Table 11 provides a summary of the proposed number of shafts, shaft diameter, and pile cap/grade beam dimensions at each bent/pier location based on the preferred retrofit alternative.

11.2.1 Single Shaft Axial and Uplift Resistance

We estimated the nominal axial and uplift resistance of individual shafts in general accordance with the AASHTO LRFD Bridge Design Specifications. We developed engineering parameters for the shaft resistance evaluation based on our characterization of subsurface materials, subsurface explorations, our interpretation of the available subsurface information, and FLAC analysis. Our assumed shaft length for the axial and uplift resistance evaluation is based on the estimated shaft length to establish fixity as determined from our shaft group evaluation (see Section 11.2.2). For preliminary evaluation purposes, we assumed a single value for the resistance of all shafts at each shaft group. The results of the single shaft axial and uplift resistance evaluation for the post-seismic condition are shown on Table 11. The single shaft resistances reported in the table are nominal geotechnical resistances and should be reduced by resistance factors of 1.0 and 0.8 for axial and uplift resistance, respectively, for the extreme event limit state.

The drilled shafts will experience post-seismic downdrag loads due to the liquefaction-induced settlement. We have estimated the unfactored single shaft post-seismic downdrag loads and provided them in Table 11. A load factor of 1.0 is recommended to be applied to this post-seismic downdrag load.

TABLE 11: SUMMARY OF DRILLED SHAFT GROUP FOUNDATIONS AND AXIAL RESISTANCE FOR PREFERRED RETROFIT AND SEISMIC MITIGATION ALTERNATIVE

Location	Number of Shafts	[1]Pile Cap/Grade Beam Dimensions (W x L x H) (ft)	Shaft Diameter (ft)	[2]Approximate Bottom Pile Cap/Grade Beam Elevation (ft)	Assumed Shaft Tip Elevation (ft)	Assumed Shaft Length (ft)	[3,4]Nominal Single Shaft Post-Seismic Axial Resistance (kips)	[3,4]Nominal Single Shaft Post-Seismic Uplift Resistance (kips)	[3,5]Unfactored Single Shaft Post-Seismic Downdrag Load (kips)
Bent 17	2	14' x 110' x 9'	8	17	-55	72	7,500	4,900	15
Bent 18	2	19' x 110' x 9'	8	15	-65	80	8,500	5,900	65
Bent 19	2	19' x 110' x 9'	8	13	-100	113	8,700	6,300	400
Pier 1	12	57' x 95' x 21.7'	6	-41.6	-125	83	3,200	2,100	80
Pier 2	14	86' x 172' x 37'	10	-70	-140	70	6,400	3,700	95
Pier 3	14	86' x 172' x 37'	10	-68.6	-140	71	7,200	4,500	75
Pier 4 [6]	12	60' x 92' x 21.5'	6	-40.3	-145	105	3,300	2,300	380
Bent 21	2	6' x 96' x 18'	10	12.5	-145	157	9,600	5,700	880
Bent 22	2	6' x 96' x 18'	10	12.5	-150	162	10,200	6,100	1,660
Bent 23	2	6' x 96' x 18'	10	12.5	-150	162	10,400	6,300	1,890
Bent 24	2	13' x 110' x 13'	10	17.5	-140	157	10,100	6,000	2,470
Bent 25	2	13' x 110' x 10'	10	20.5	-135	155	10,300	6,200	1,990
Bent 26	2	13' x 110' x 10'	10	20.5	-125	145	10,500	6,500	790
Bent 27	2	13' x 110' x 10'	10	20.5	-95	115	8,000	4,300	35

Notes:
[1] W = Pile cap dimension in longitudinal direction (perpendicular to bent/pier centerline), L = Pile cap dimension in transverse direction (parallel to bent/pier centerline)
[2] Elevations have been converted from City of Portland Datum to NAVD88 by adding 2.1 feet to the elevations shown on the plan set.
[3] Post-seismic single shaft resistance and downdrag load include soil strength reductions due to partial or full liquefaction (i.e. excess pore water pressure).
[4] Resistance factors of 1.0 and 0.8 should be applied to the nominal post-seismic single shaft axial and uplift resistance, respectively, per AASHTO 10.5.5.3.3.
[5] A load factor of 1.0 should be applied to the unfactored single shaft post-seismic downdrag load, per ODOT GDM 8.9.1.
[6] Retrofit Option A includes a micropile array under adjacent I-5 approach ramp in addition to 12 drilled shafts. Retrofit Option B is to construct a new pier founded on 12 drilled shafts.

11.2.2 Drilled Shaft Group Evaluation

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

We evaluated the pile cap response of the drilled shaft group foundations to axial loading and lateral loading in the longitudinal and transverse orientations, for the post-seismic condition. We completed the analysis using the computer program GROUP v2016, (Ensoft, 2016). We modeled the shaft group axial and lateral efficiency and overall stiffness of the piers considering shaft geometry and lateral and axial shaft 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 for the post-seismic condition are discussed in Section 11.3. In addition, we ignored the existing pile caps and pile group foundations in our evaluation. The assumed shaft lengths are based on the estimated shaft lengths to establish fixity. 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 drilled shaft group for the post-seismic condition. We understand that HDR will use the load-displacement relationships to develop the stiffness matrix at the bottom of the pile cap. It was assumed the pile cap is rigid and that the shaft head connection to the pile cap is fixed. The results of the evaluation are shown in Appendix F, Load-Displacement Curves for Existing Pile and Proposed Drilled Shaft Groups.

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. For our evaluation, we assumed the liquefied soil does not provide any passive resistance. The earth pressure parameters we developed for the post-seismic condition for the retrofitted abutment walls and pile caps are presented in Table 10.

12.0 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.

This report was prepared for the exclusive use of HDR Engineering, Inc., and Multnomah County for use in the Burnside Bridge Seismic Feasibility Study. 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 in Appendix G, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

SHANNON & WILSON, INC.

Exp. 12/31/2018

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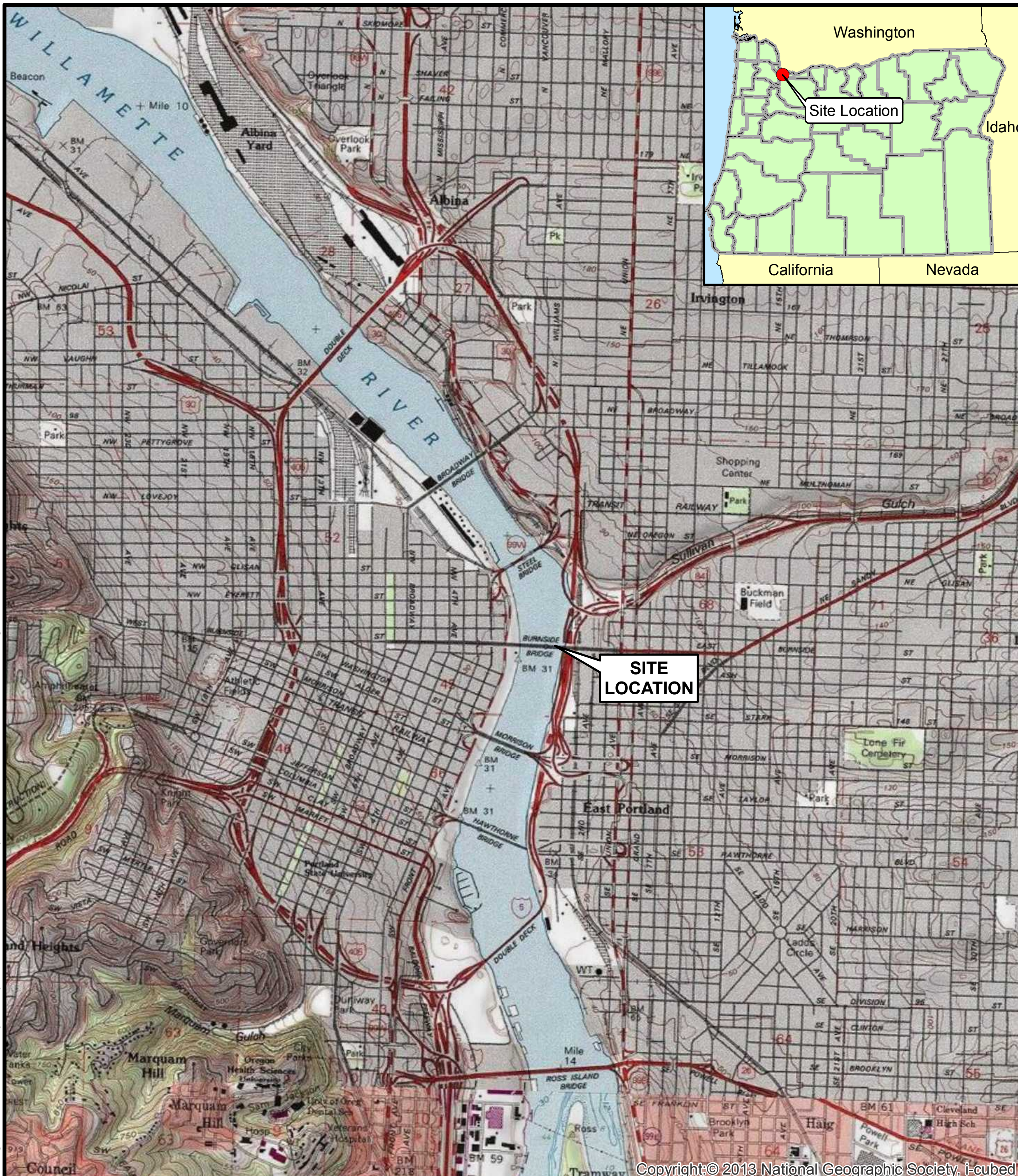

0 0.25 0.5 1

Scale in Miles

N

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

VICINITY MAP

April 2017 24-1-04065-005

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

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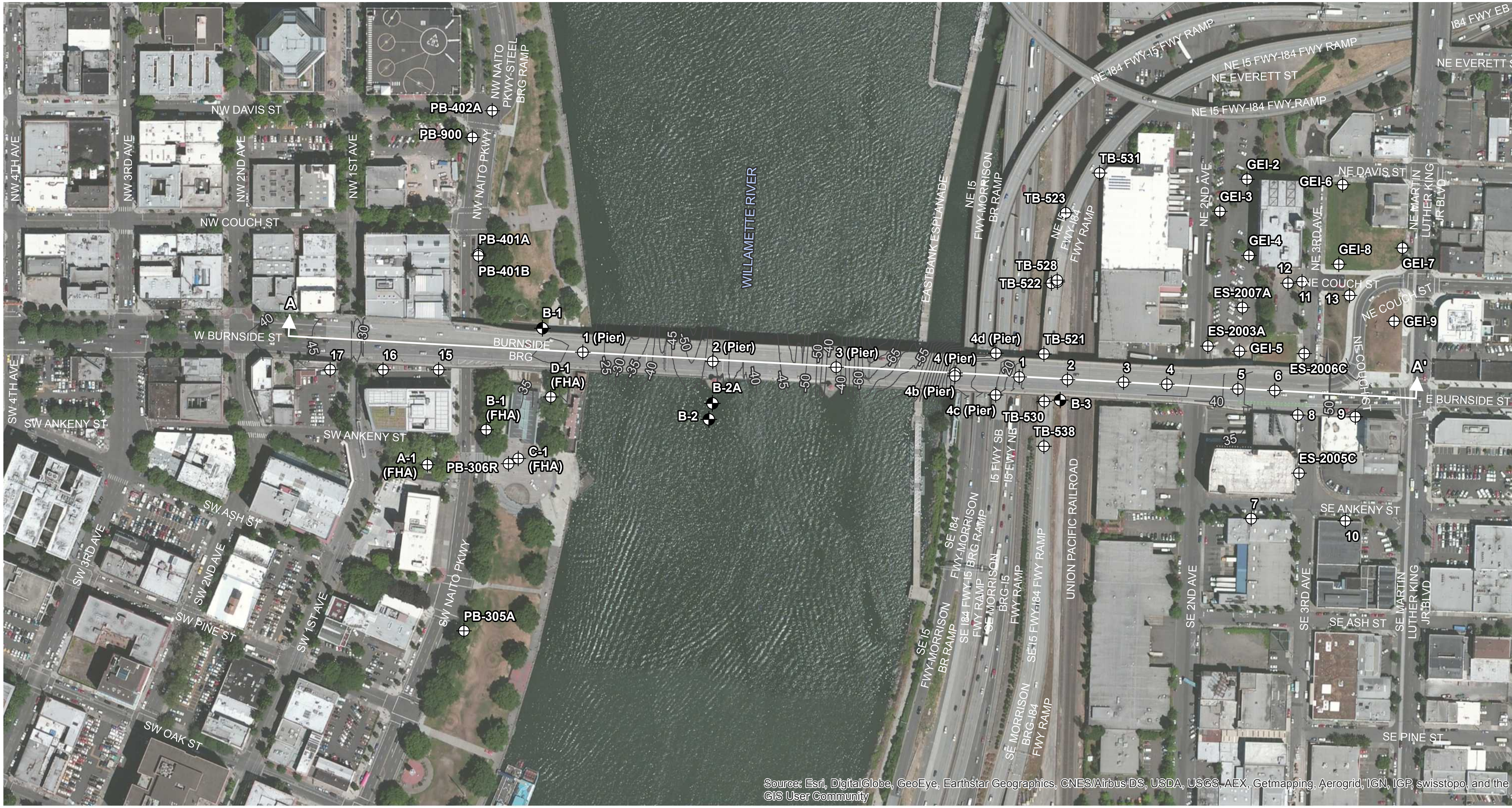

LEGEND

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 Seismic Feasibility Study
Portland, Oregon

SITE AND EXPLORATION PLAN

June 2017 | 24-1-04065-005

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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

LEGEND

Qaf - Artificial Fill	QTs - Troutdale Formation
Qal - Alluvium	Tfg - Basalt of Ginko
Qfch - Catastrophic Flood Deposits, Channel Facies	Tgsb - Columbia River Basalt Group, Sentinel Bluffs Member
Qff - Catastrophic Flood Deposits, Fine-grained Facies	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 Seismic Feasibility Study
Portland, Oregon

PUBLISHED GEOLOGIC MAPPING

April 2017 24-1-04065-005

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FIG. 3

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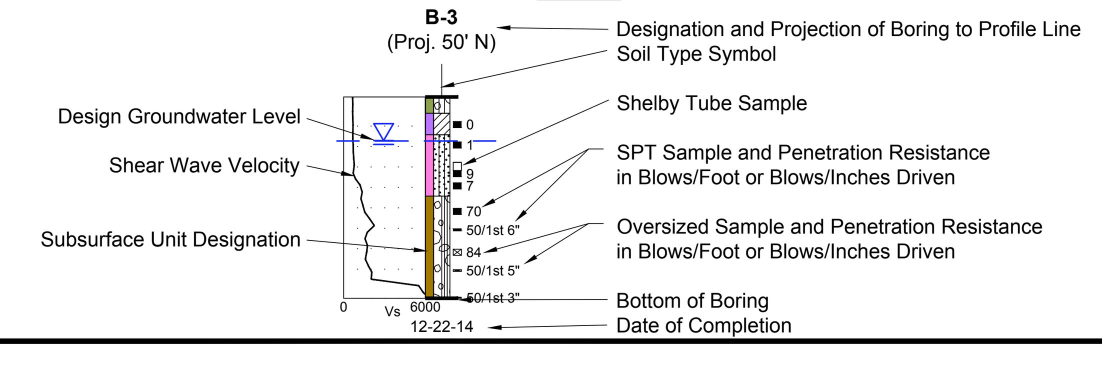
LEGEND



Scale in Feet
0 40 80
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 Seismic Feasibility Study
Portland, Oregon

INTERPRETIVE SUBSURFACE PROFILE A-A'

September 2017 24-1-04065-005

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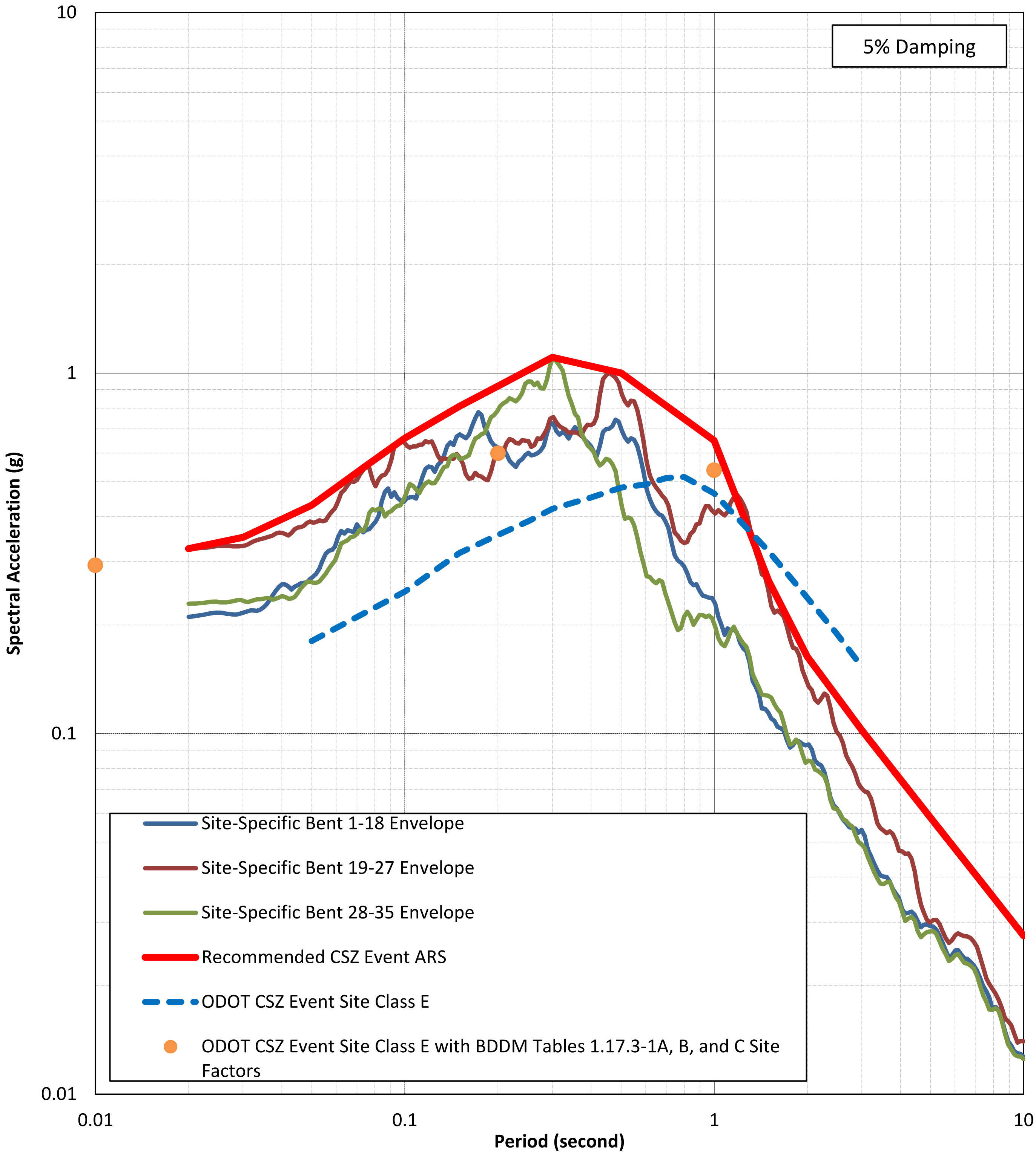
FIG. 4



NOTES

1. The ODOT CSZ Event Site Class E spectral values were estimated for Vs30 of 150 m/sec by extrapolating the spectral values from Vs30 of 270 and 200 m/sec.
2. Recommended Spectrum at long periods were obtained by taking two-thirds of ODOT CSZ Event Site Class E linearly extrapolated (in log scale) above 3.0 sec.
3. g = standard gravitational acceleration; CSZ = Cascadia subduction zone; ARS = acceleration response spectrum; ODOT = Oregon department of transportation; Vs30 = time-averaged mean shear wave velocity for 30 meters subsurface; m/sec = meters per second; BDDM = bridge design and drafting manual

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

CSZ EVENT ACCELERATION RESPONSE SPECTRA FOR "FULL OPERATION" PERFORMANCE LEVEL

August 2017 | 24-1-04065-005

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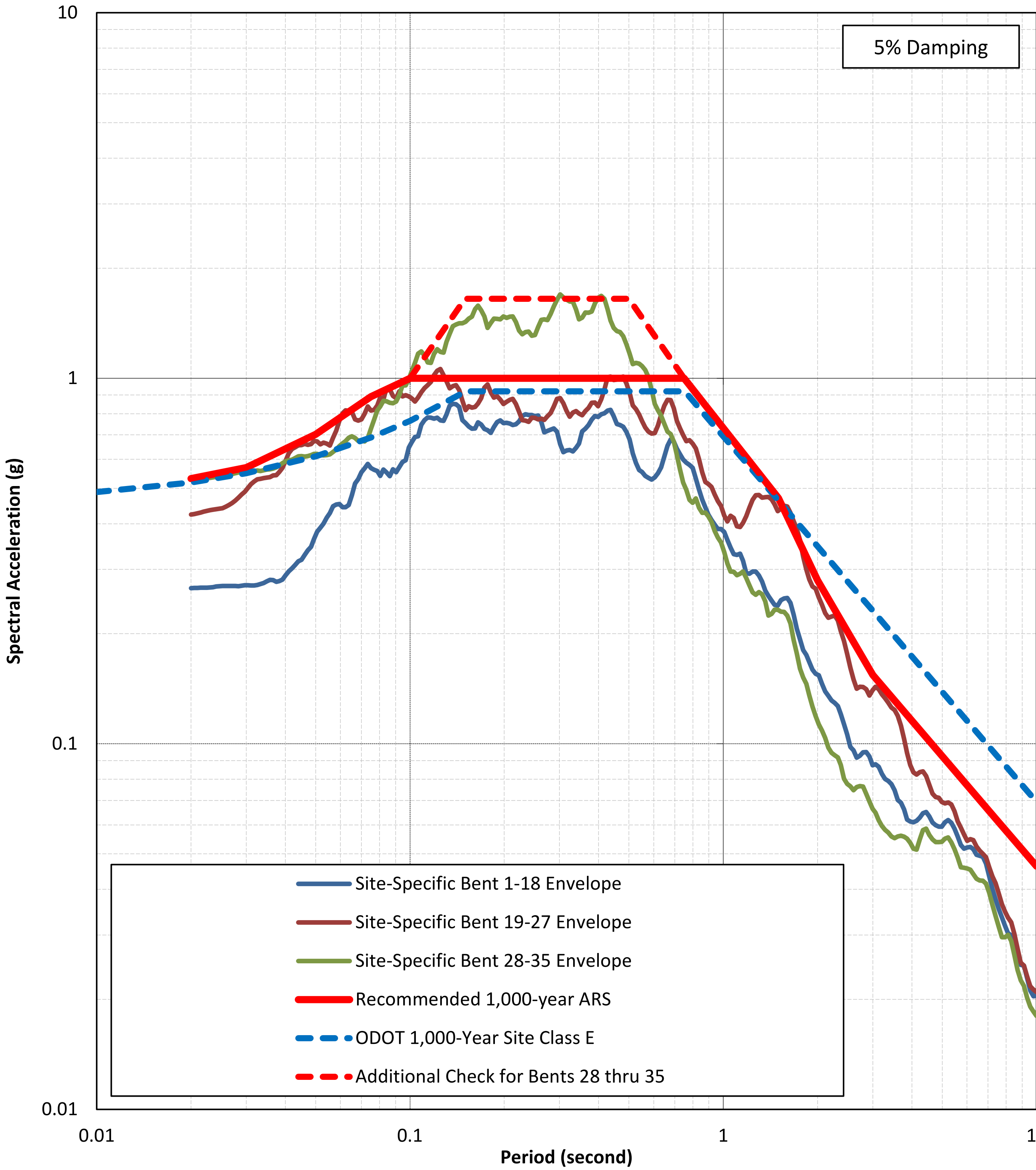
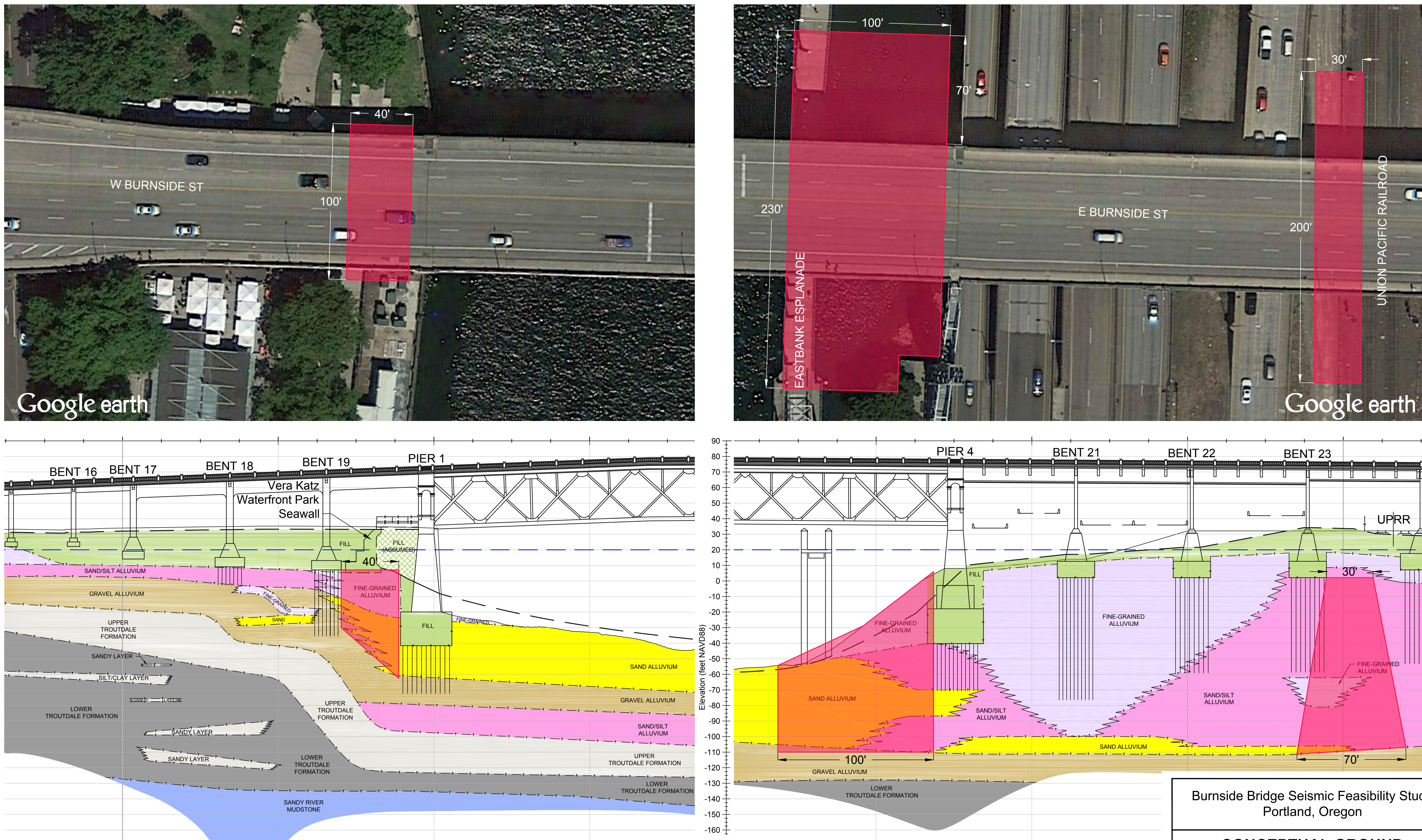
FIG. 5



NOTES

1. g = standard gravitational acceleration; ARS = acceleration response spectrum; ODOT = Oregon department of transportation

Burnside Bridge Seismic Feasibility Study Portland, Oregon	
1,000-YEAR RETURN PERIOD ACCELERATION RESPONSE SPECTRA FOR "LIMITED OPERATION"	
July 2017	24-1-04065-005
SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS	**FIG. 6**



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 Seismic Feasibility Study
Portland, Oregon

CONCEPTUAL GROUND IMPROVEMENT EXTENTS FOR LATERAL SPREAD MITIGATION

September 2017 | 24-1-04065-005

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FIG. 7

File: I:\EF\24-1 PDX\04000s\04065 Burnside Bridge Seismic Feasibility Study\Graphics\CAD\GroundImprovementConcept.dwg Date: 09-13-2017 Author: aeh

APPENDIX A

EXISTING INFORMATION

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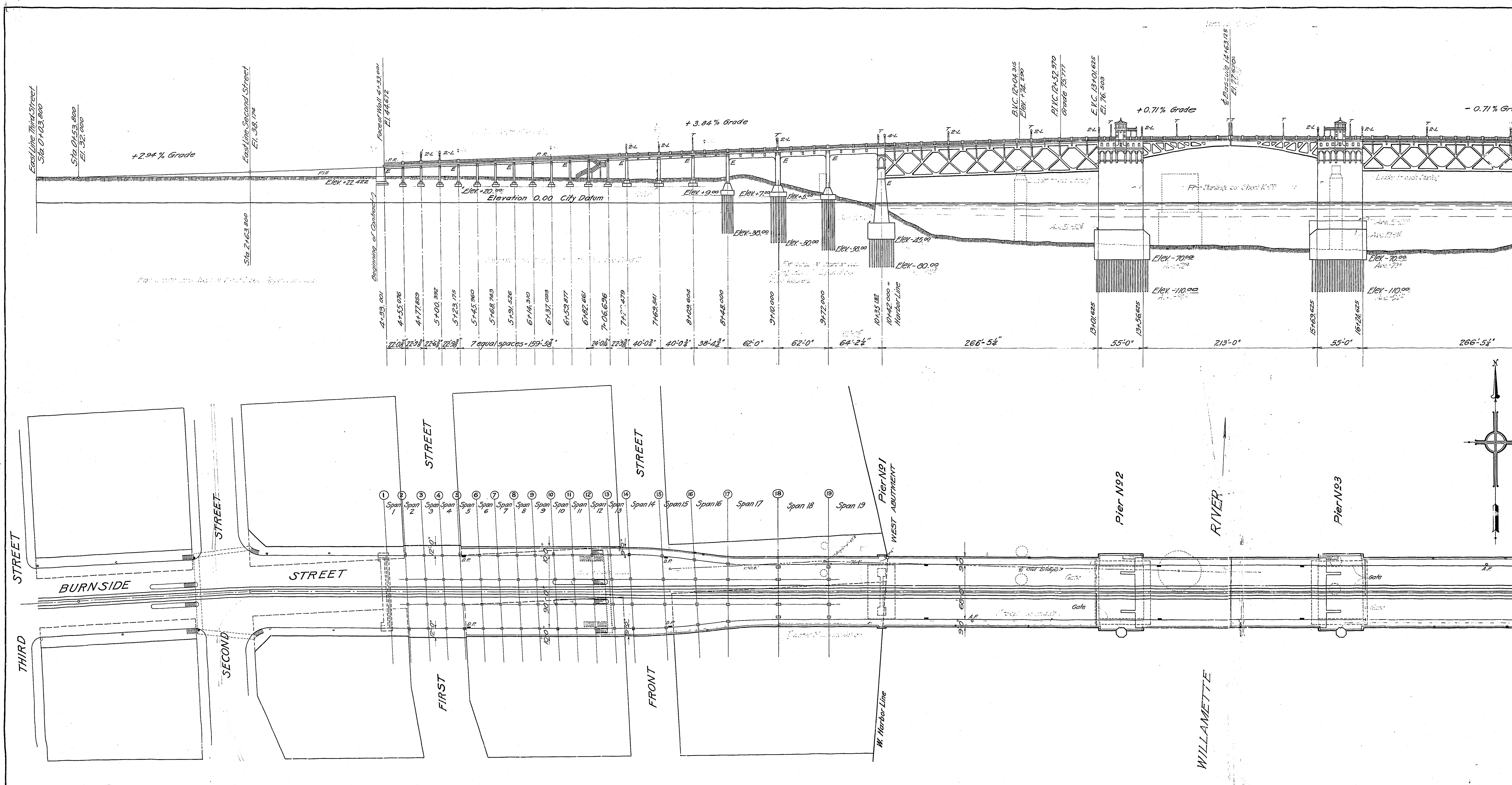
(continued)

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



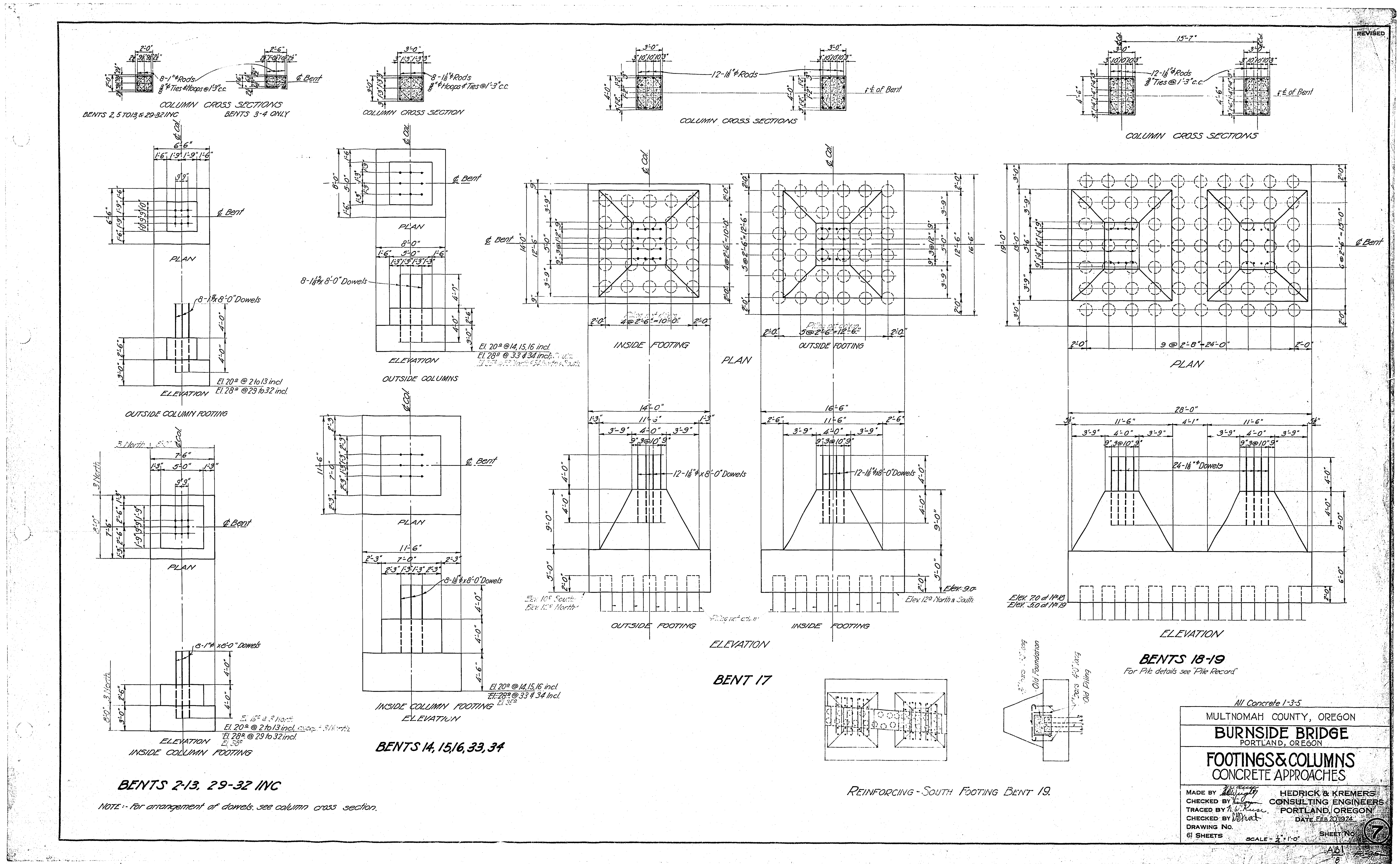
E D C B A A B C D E

NOTICE: IF THIS DOCUMENT IS LESS LEGIBLE THAN THIS NOTICE, IT IS DUE TO THE QUALITY OF THE ORIGINAL.

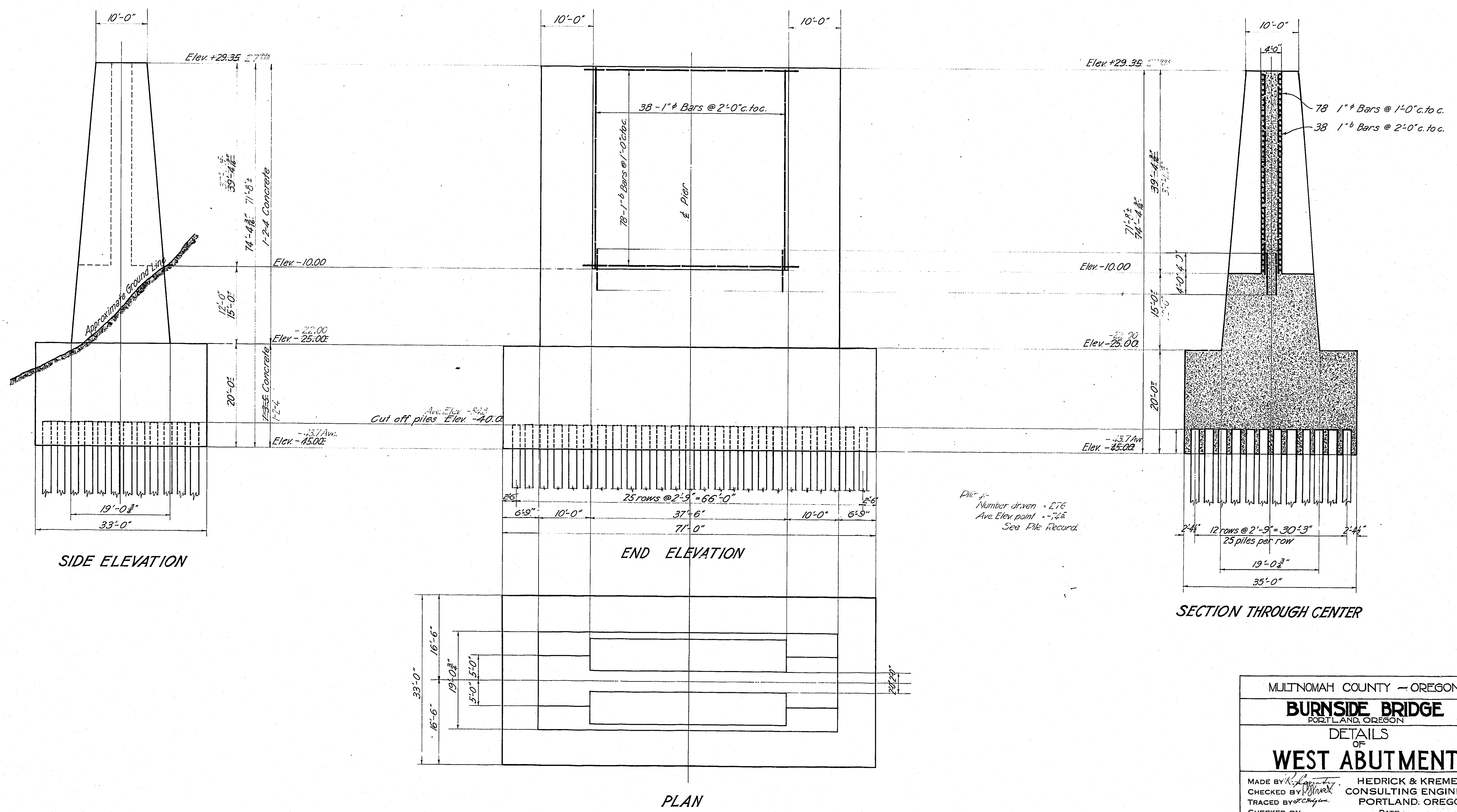
Business Equipment Bureau 1700 Southeast 11th Avenue Portland, Oregon 97214 503/232 8182 3M


+0.71% Grade
- 0.71% Grade
-1.587% Grade
Elevation 0.00 City Datum
Elev. -70.00
Elev. -110.00
Elev. -120.00
Elev. -95.00
Elev. -90.00
Elev. -88.00
Elev. 28.50
Elev. +28.00
Elev. +41.993
Face of Wall El. 63.993
West Line Union Ave. El. 61.000
End of Contract
Sta. 29+28.930
55'-0"
213'-0"
266'-5½"
77'-5¼"
74'-5⅛"
73'-0¼"
105'-10⅝"
84'-9⅜"
40'-0"
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
Pier No 2
Pier No 3
EAST ABUTMENT Pier No 4
RIVER
WILLAMETTE
E. Harbor Line
Span 20
Span 21
Span 22
Span 23
Span 24
Span 25
Span 26
Span 27
Span 28
Span 29
Span 30
Span 31
Span 32
Span 33
Span 34
E. FIRST STREET
E. SECOND STREET
E. THIRD STREET
E. BURNSIDE STREET
UNION AVENUE
MULTNOMAH COUNTY OREGON
BURNSIDE BRIDGE
PORTLAND OREGON
GENERAL ELEVATION & PLAN
DESIGN NO. 2
MADE BY
CHECKED BY
TRACED BY
CHECKED BY
DRAWING NO.
SHEETS
HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB. 21-1924
SCALE - 1" = 40'-0"
SHEET NO. T-2

E D C B A B C D E



NOTICE: IF THIS DOCUMENT IS LESS LEGIBLE THAN THIS NOTICE IT IS DUE TO THE QUALITY OF THE ORIGINAL.



MULTNOMAH COUNTY — OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

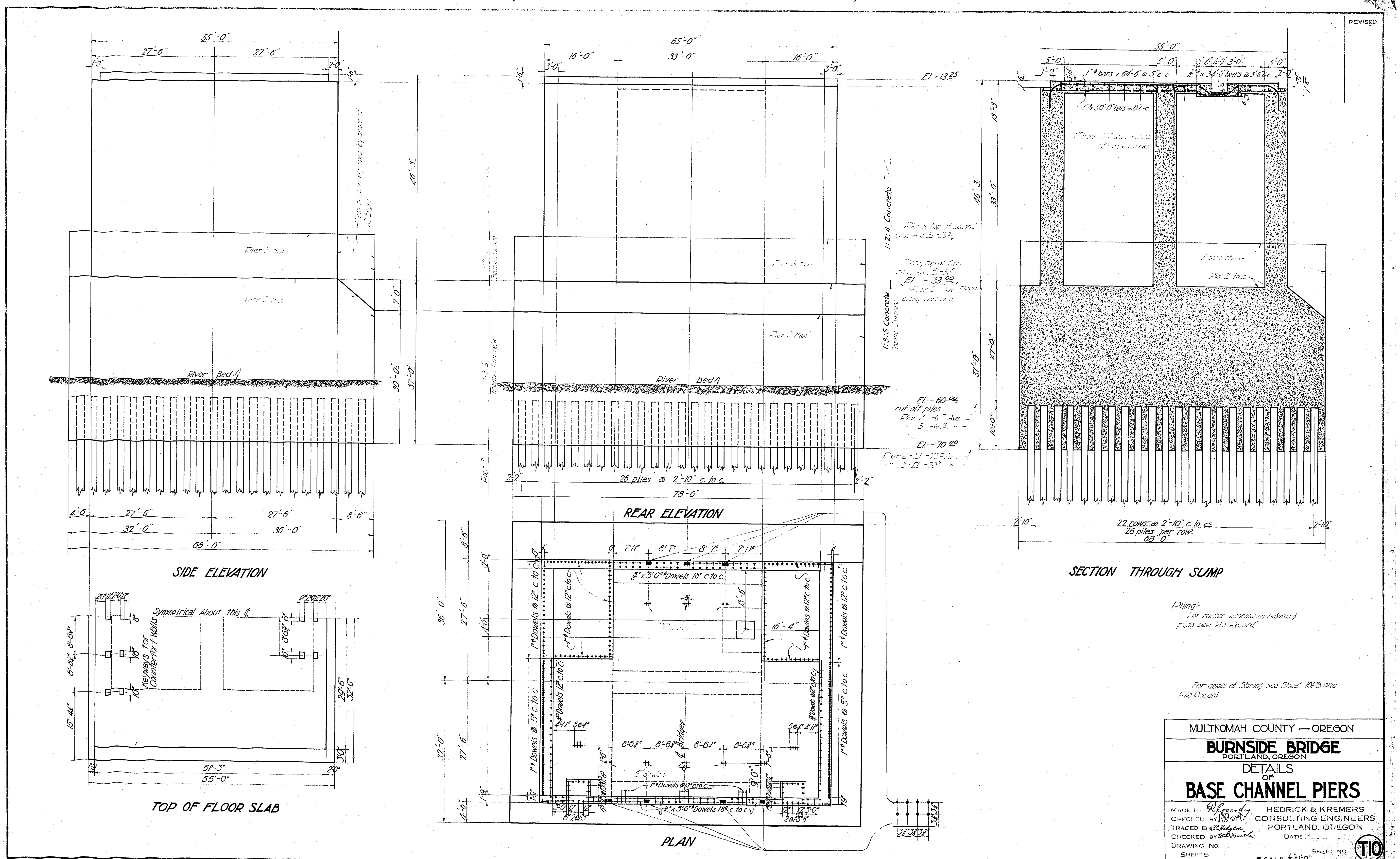
DETAILS OF

WEST ABUTMENT

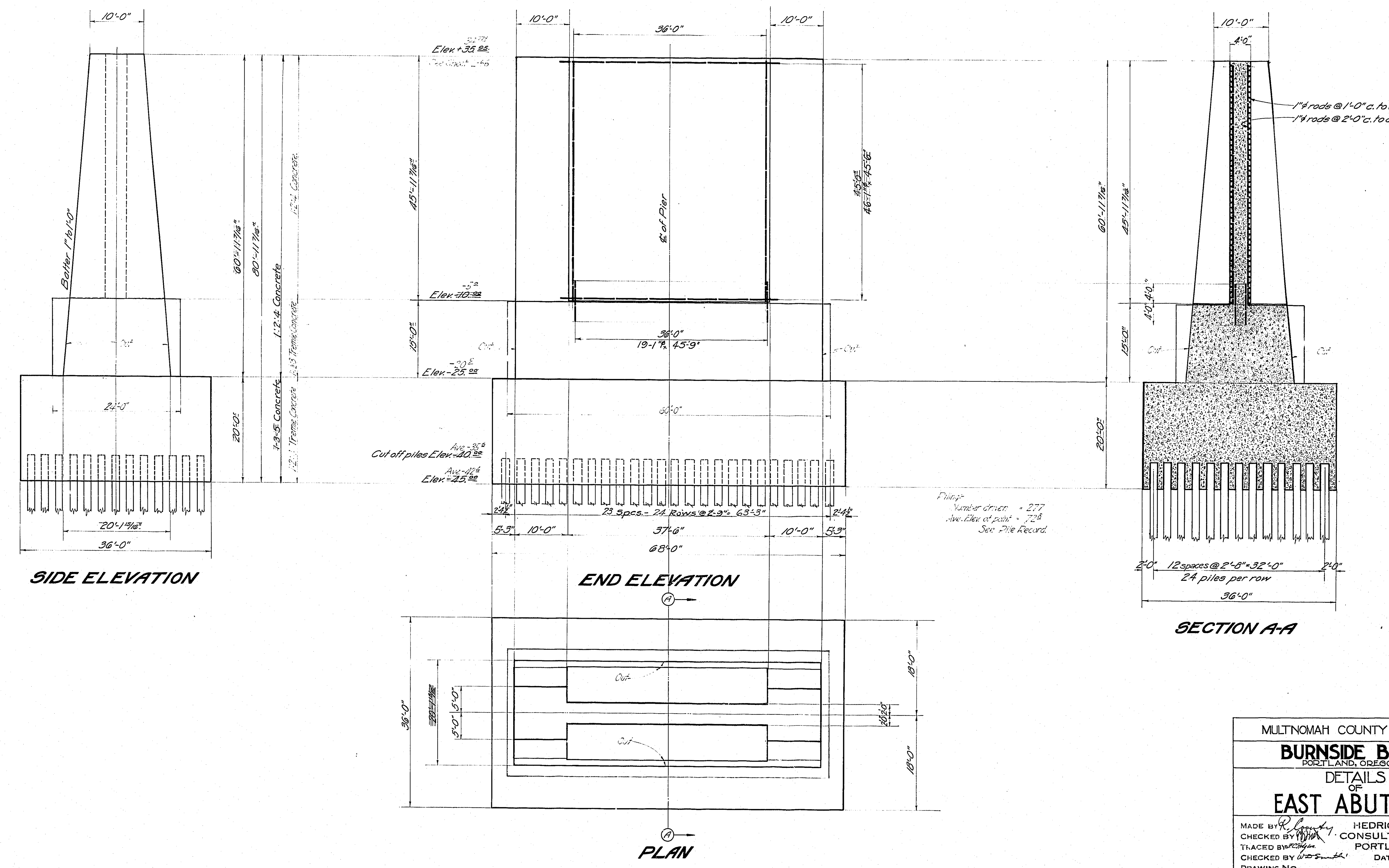
MADE BY
CHECKED BY
TRACED BY
CHECKED BY
DRAWING NO.
SHEETS

HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE

SHEET NO. T 8



NOTICE: IF THIS DOCUMENT IS LESS LEGIBLE THAN THIS NOTICE IT IS DUE TO THE QUALITY OF THE ORIGINAL.



REVISED

MULTNOMAH COUNTY – OREGON

BURNSIDE BRIDGE
PORTLAND, OREGON

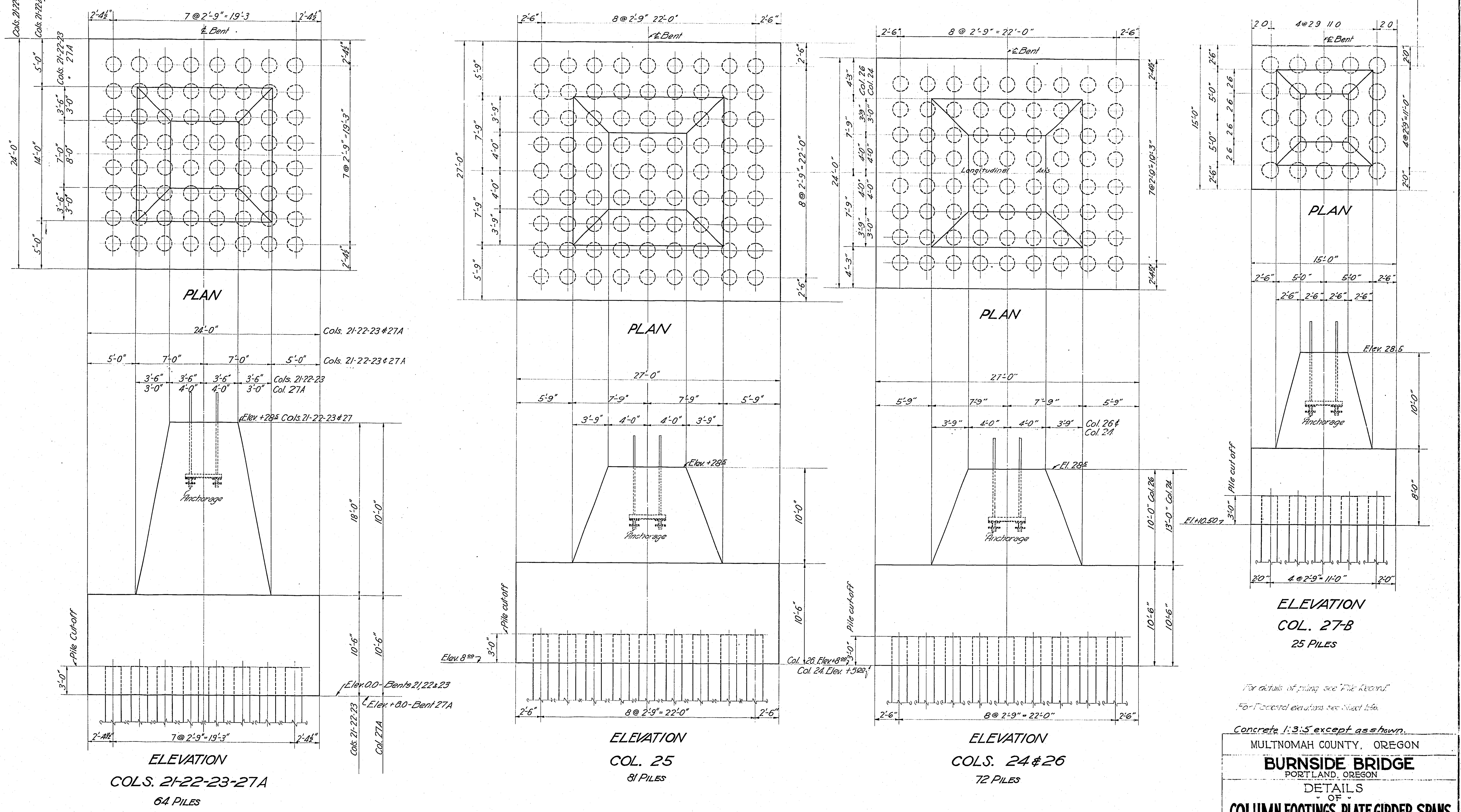
DETAILS OF

EAST ABUTMENT

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

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

SHEET NO. T16



For details of piling see "Pile Record."

For Footing elevations see Sheet 156.

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 A.B. Wright
CHECKED BY
TRACED BY H.L. Moran
CHECKED BY
DRAWING NO.
SHEETS

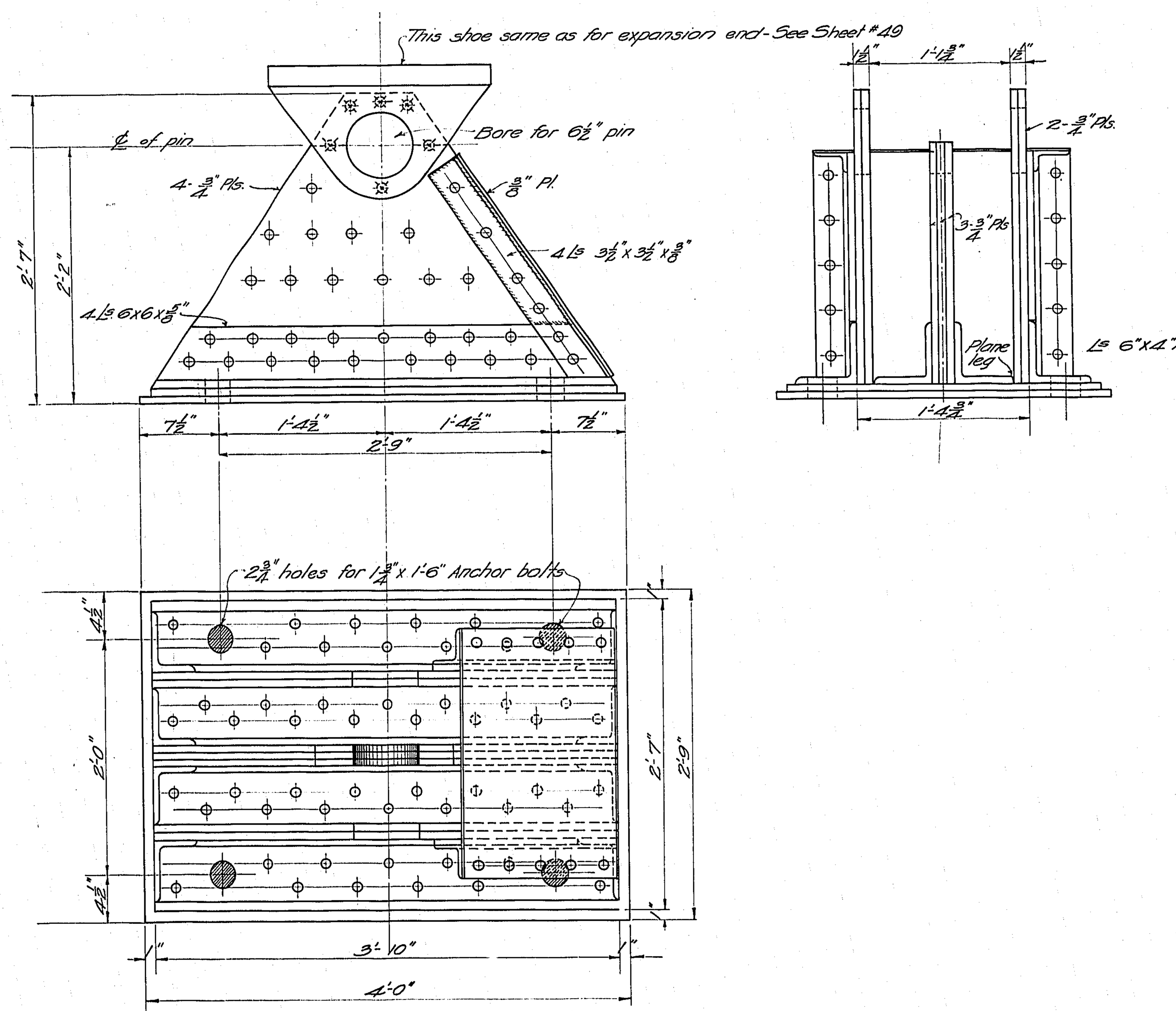
HEDRICK & KREMERS
CONSULTING ENGINEERS
PORTLAND, OREGON
DATE FEB. 18, 1924

Scale 1/4" = 1'-0"

SHEET NO. 18

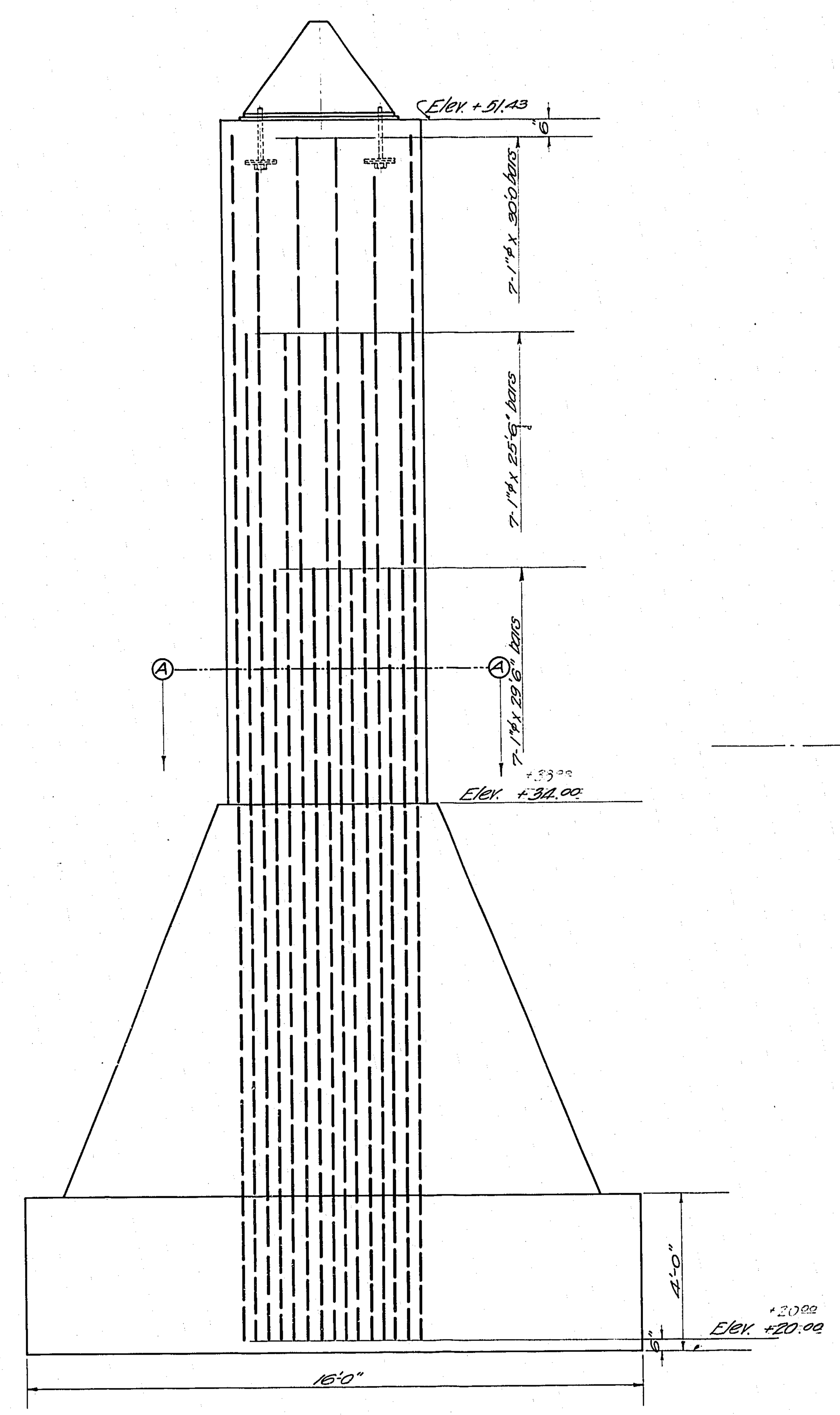
REVISED

REVISED

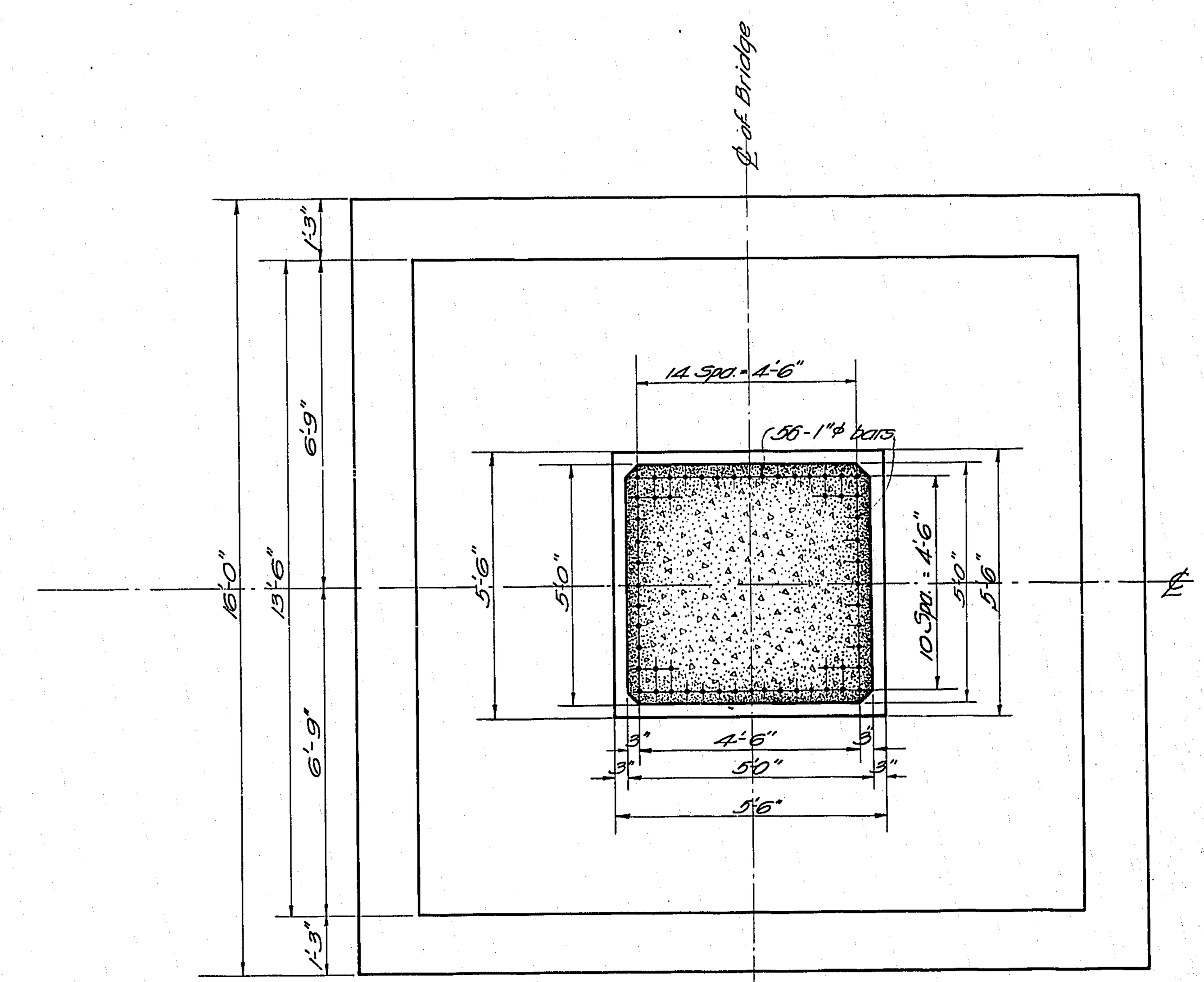


See Drawings Pacific American Bridge Co.

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

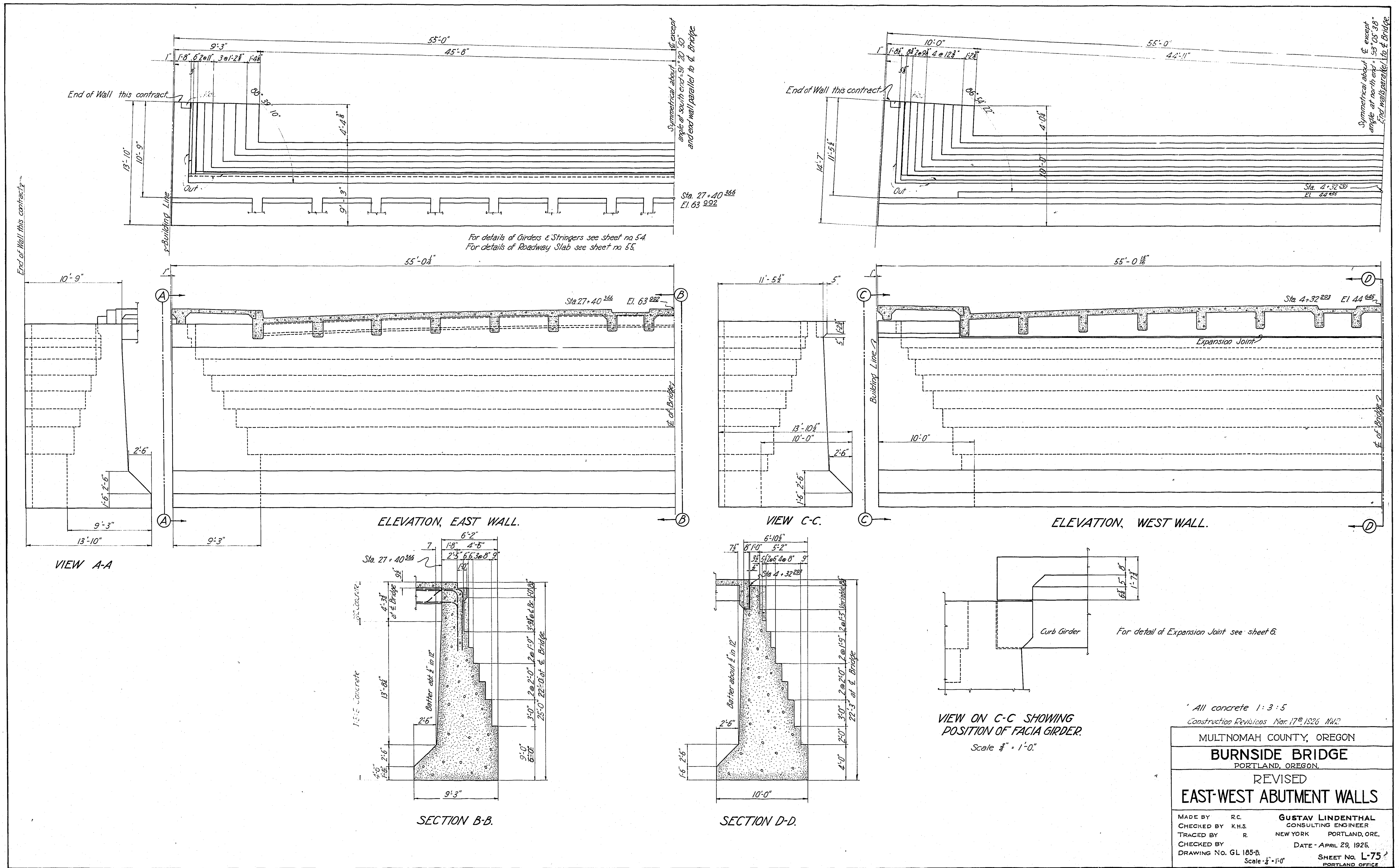
DETAILS
OF
BENT NO. 28

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

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

SHEET NO. 48

SCALE AS SHOWN

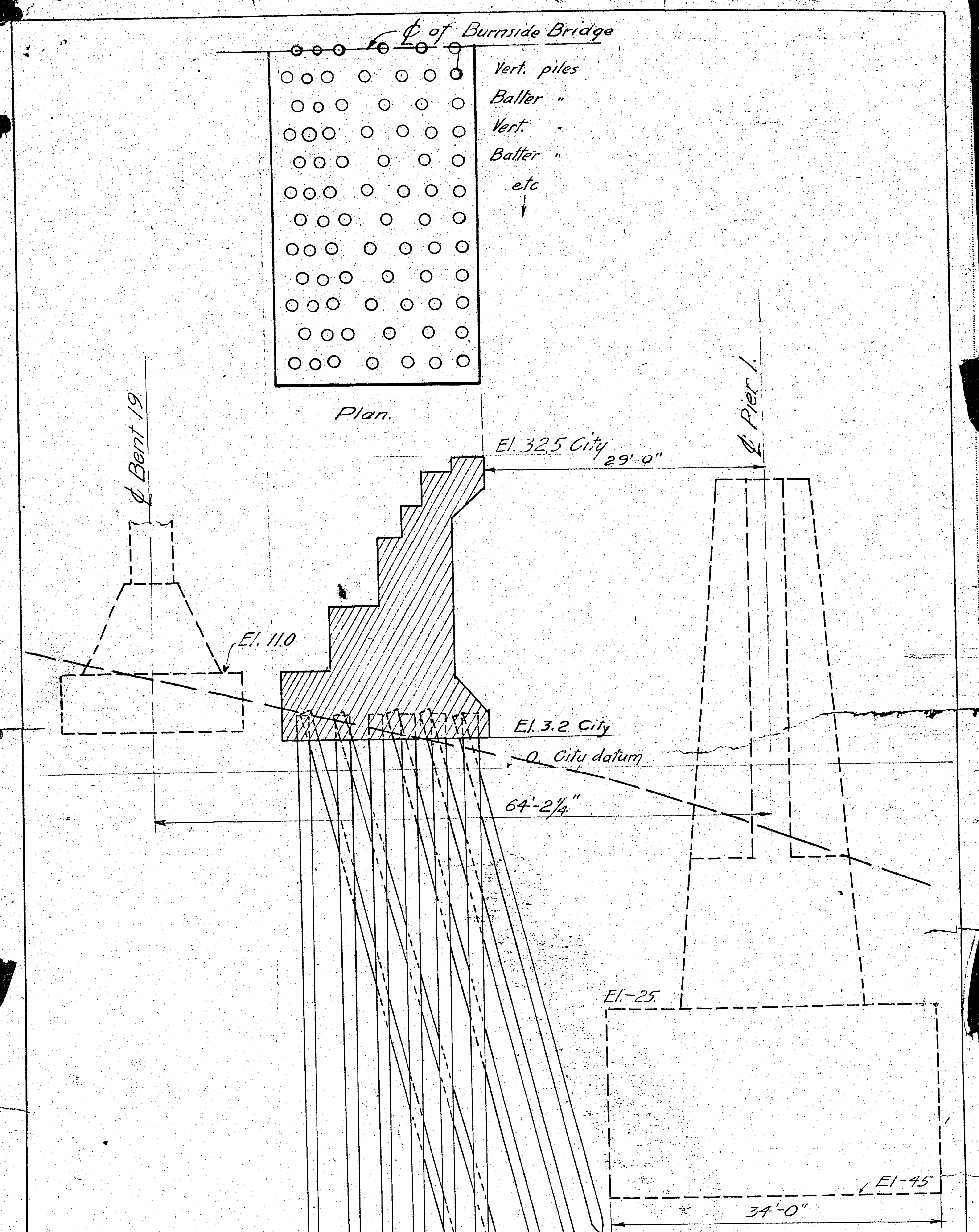


AB1
83

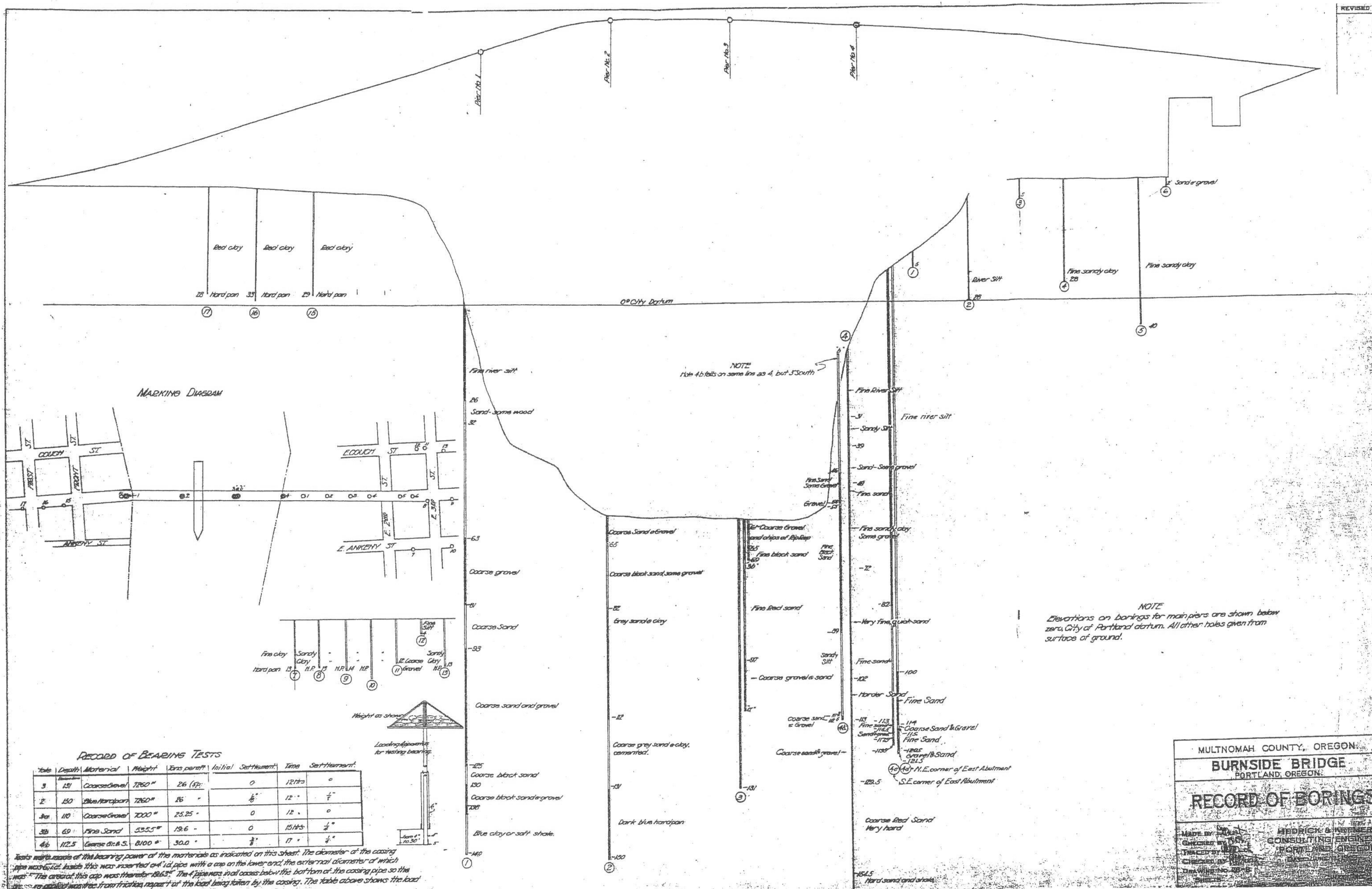
NOTICE: IF THIS DOCUMENT IS LESS LEGIBLE THAN THIS NOTICE IT IS DUE TO THE QUALITY OF THE ORIGINAL.

BURNSIDE BRIDGE — FOUNDATION PILING SUMMARY

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

MULTNOMAH COUNTY, OREGON.

BURNSIDE BRIDGE

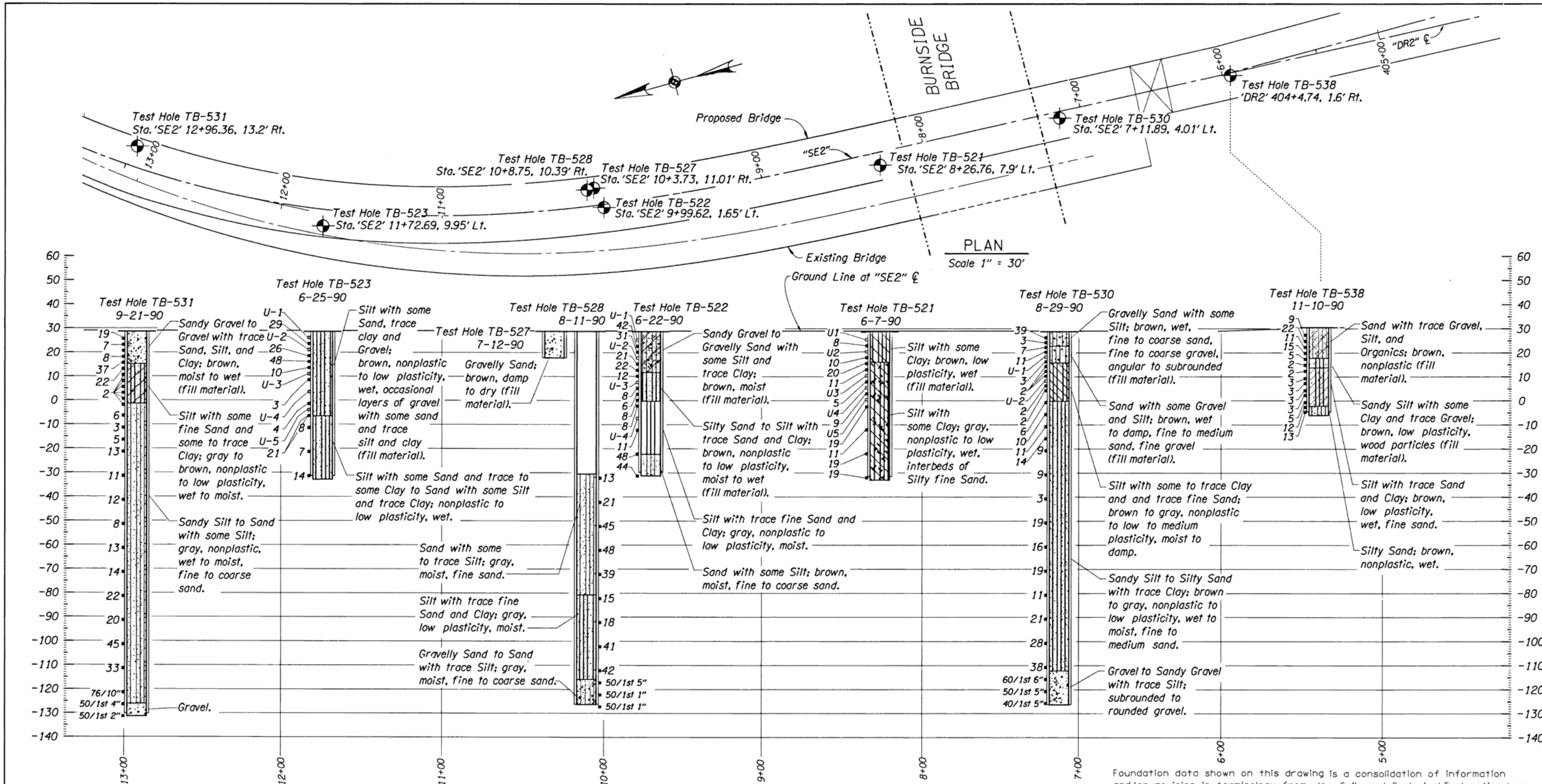
PORTLAND, OREGON.

RECORD OF BORINGS

MADE BY [illegible]
CHECKED BY [illegible]
TRACED BY [illegible]
CHECKED BY [illegible]
DRAWING NO. 28-B
SHEETS [illegible]

HEDRICK & [illegible]
CONSULTING ENGINEERS
PORTLAND, OREGON

NOTICE: IF THIS DOCUMENT IS LESS LEGIBLE THAN THIS NOTICE IT IS

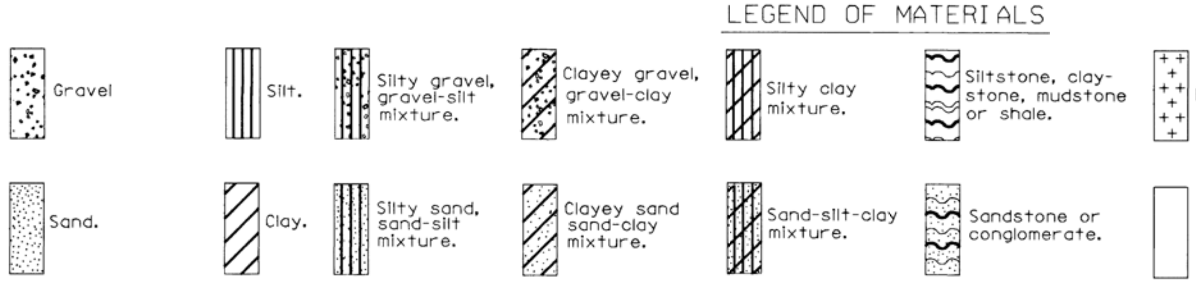


Foundation data shown on this drawing is a consolidation of information and/or revision in terminology from the Soils and Geological Exploration Logs. The Soils and Geological Exploration Logs used in compiling this drawing are available upon request.

FOUNDATION DATA

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

LEGEND OF MATERIALS



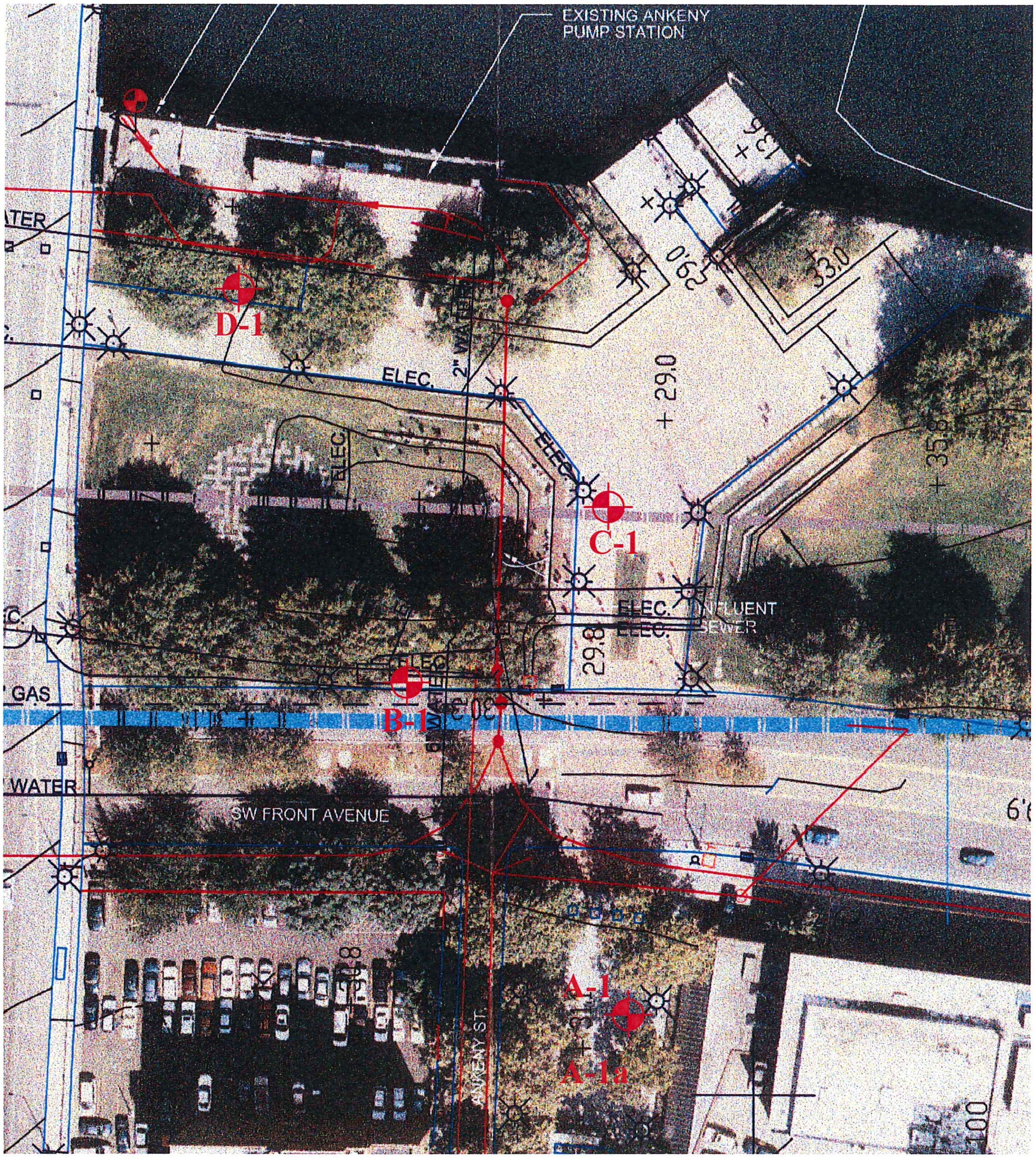
24 Standard Penetration Test. N value.

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

BRIDGE ENGINEER Thomas D. Lulay PE NO. 8491	OREGON DEPARTMENT OF TRANSPORTATION BRIDGE DESIGN SECTION
DESIGNED J. Six PE NO. 13717 / K. Williams	BANFIELD ACCESS RAMP
DRAWN R. W. Burns	
CHECKED / REVIEWED Marvin Goft	FOUNDATION DATA
11-96 As Constructed PGA (DATE / REVISION / BY)	DATE MAR. 1991 CALC.BOOK Fnd. File SHEET 3 OF 66
	BRIDGE NO. 8588D-A DRAWING NO. 47977

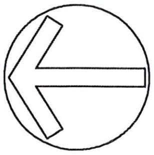
REGISTERED PROFESSIONAL ENGINEER 13717 OREGON JULY 16, 1987 JAN LIEN SIX

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



Scale 1 Inch = 60 Feet

30 0 30 60



NORTH

Ankeny Pump Station
Portland, Oregon

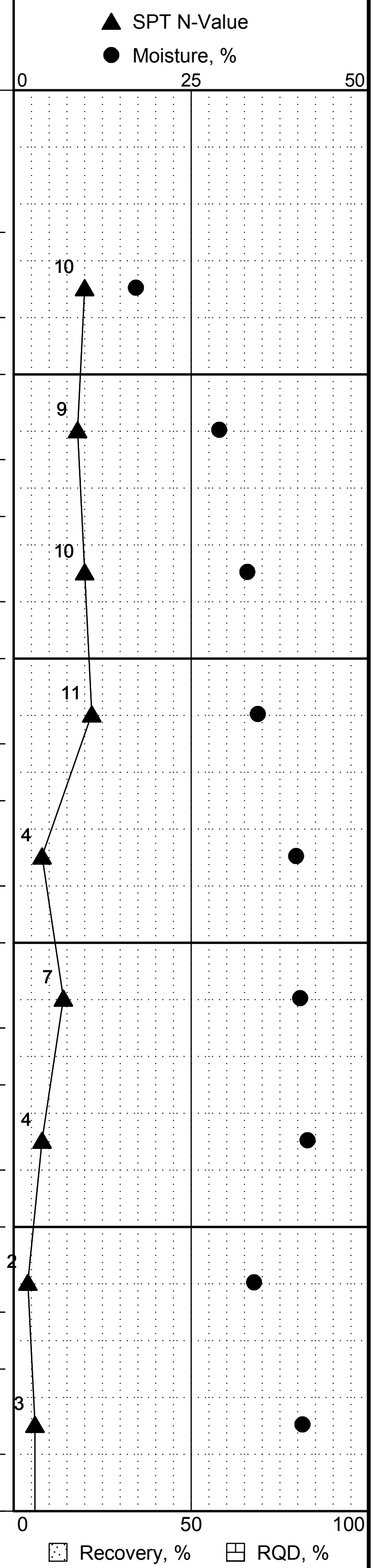
PLAN OF EXPLORATIONS

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	Depth In Feet	Samples
		30.9 / 0.1	**Artificial Granite PAVER** (Pavement)	0	
		30.5 / 0.5	**CONCRETE** (Pavement)		
			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

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 1 of 3

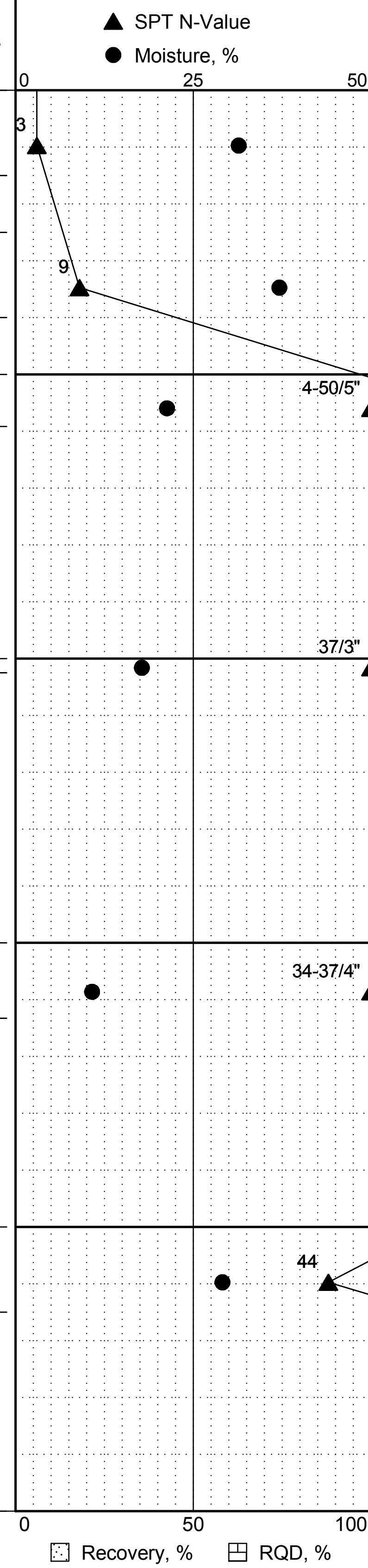
December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

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

Ground Water	Remarks	Elev. Depth Feet	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

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Impervious Seal (Bentonite)
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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

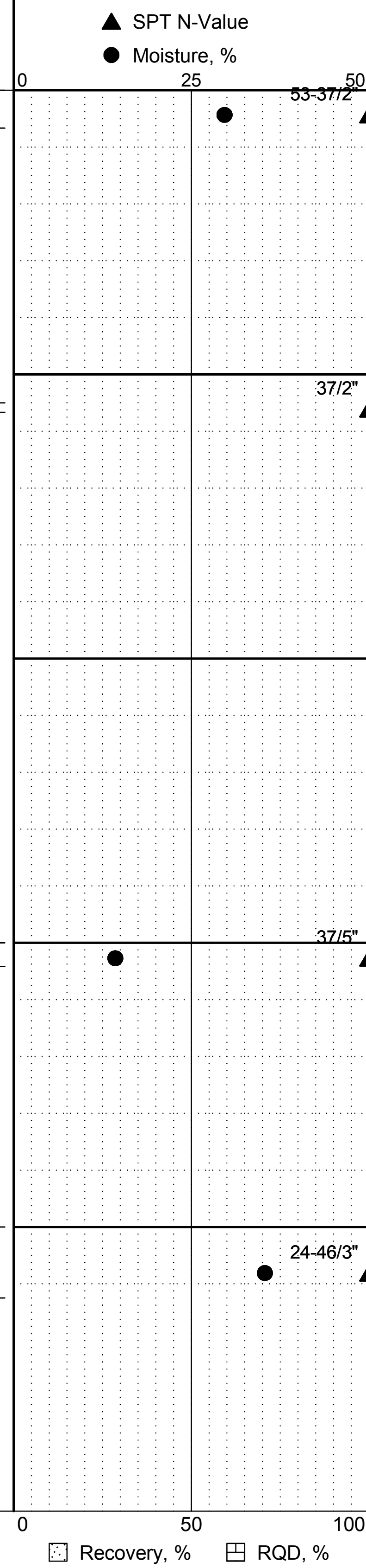
December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

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

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	Depth In Feet	Samples	SPT N-Value / Moisture, %
		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	53-37/2"
					55	S-17	37/2"
					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	37/5"
		-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	24-46/3"
		-40.2 71.2	Bottom of Boring, Completed 10/17/00				



LEGEND

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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

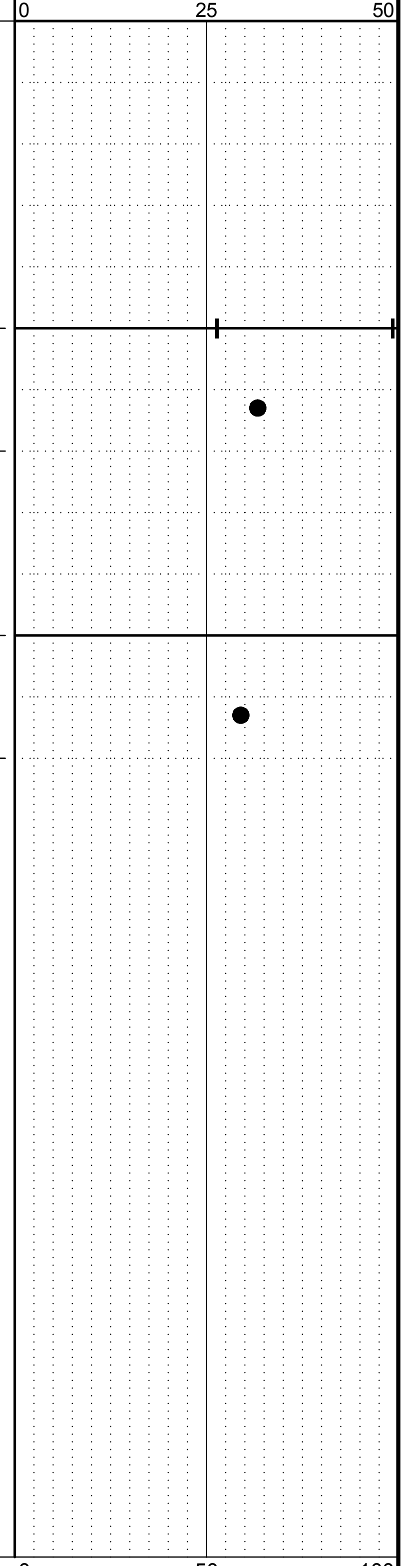
December 2000 *F-3112.01*

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

▲ SPT N-Value
● Moisture, %



Recovery, % RQD, %

LEGEND

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Impervious Seal (Bentonite)
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Perforated Zone

ATTERBERG LIMITS
Liquid Limit
Natural Water Content
Plastic Limit

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Ankeny Pump Station
Portland, Oregon

LOG OF BORING A-1a

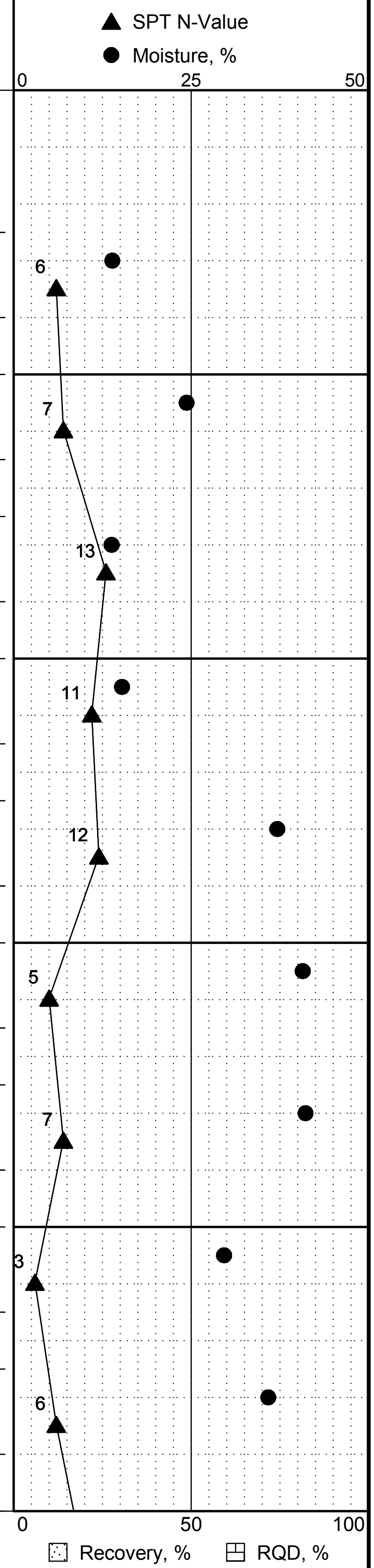
December 2000 *F-3112.01*

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



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NOTE:
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- Impervious Seal (Bentonite)
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- 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

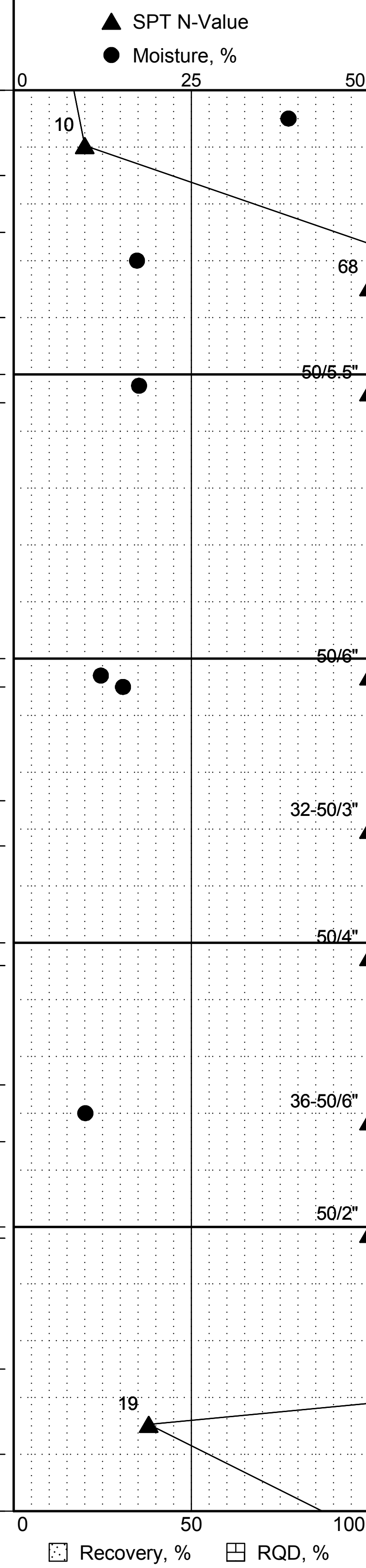
December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

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

Ground Water	Remarks	Elev. Depth Feet	CLASSIFICATION OF MATERIAL	Log	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



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NOTE:
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Ankeny Pump Station
Portland, Oregon

LOG OF BORING B-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	▲ SPT N-Value ● Moisture, % (0 – 25 – 50)
						S-19	59
		-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))				
						S-20	60-50/3"
					55	S-21	50/4"
						S-22*	50/0"
		-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	S-23	31-50/3"
						S-24	14-37-50/0"
		-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	S-25	50/3"
						S-26	50-50/3"
		-38.3 70.3	Bottom of Boring, Completed 8/31/00		70	S-27	50/3"

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

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

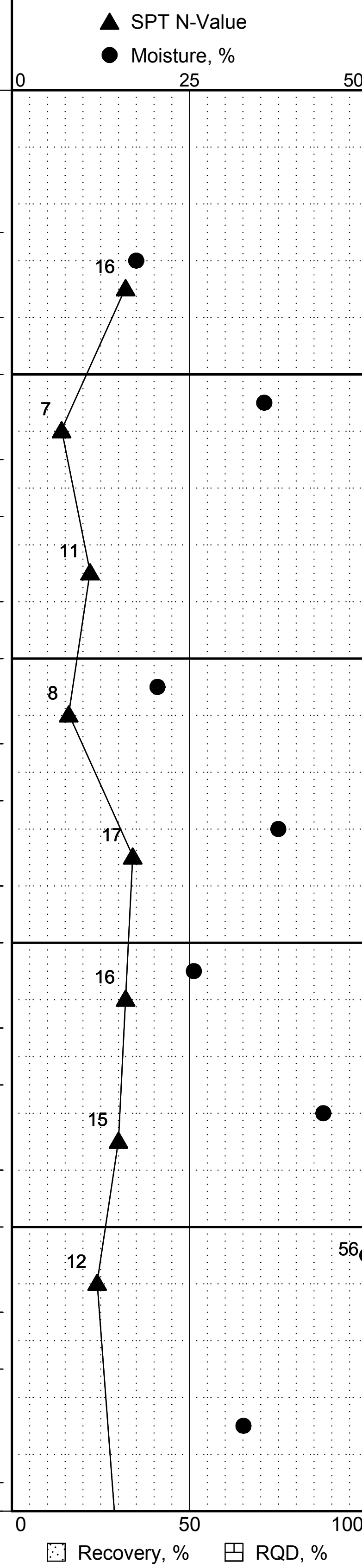
December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

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

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
▼	-200 = 33% 8/28/2000 -200 = 35% -200 = 16%	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 20	S-6 S-7 S-8 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

- 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 C-1

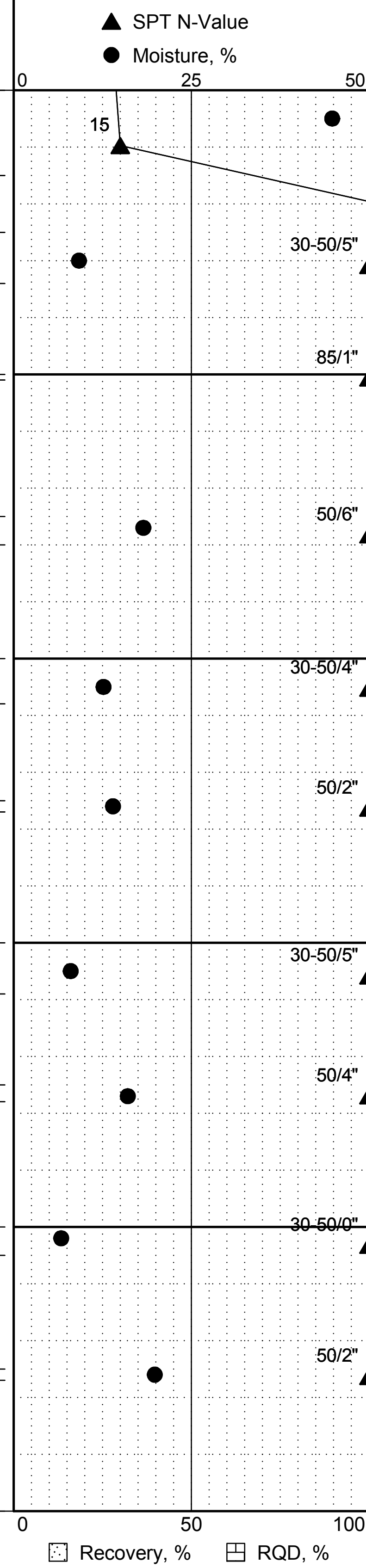
December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

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

Ground Water	Remarks	Elev. Depth Feet	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

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	▲ SPT N-Value ● Moisture, % (0 – 25 – 50)
						S-20*	61
	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-21*	50/2"
					55	S-22	50/3"
						S-23*	50/3"
	Wet density = 170.0 pcf				60	S-24*	50/3"
						S-25*	50/.5"
	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	80
					70	S-27	18-50/3"
		-40.8 70.8	Bottom of Boring, Completed 8/29/00				

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

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 C-1

Page 3 of 3

December 2000 *F-3112.01*

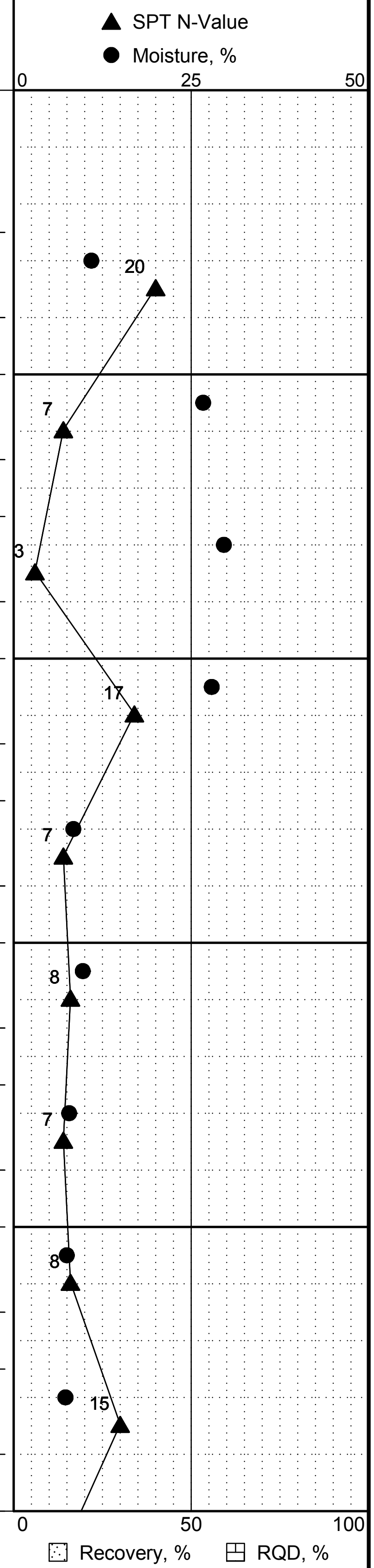
SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

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
			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
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- = 3.25" O.D. Split Spoon Sample
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- 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

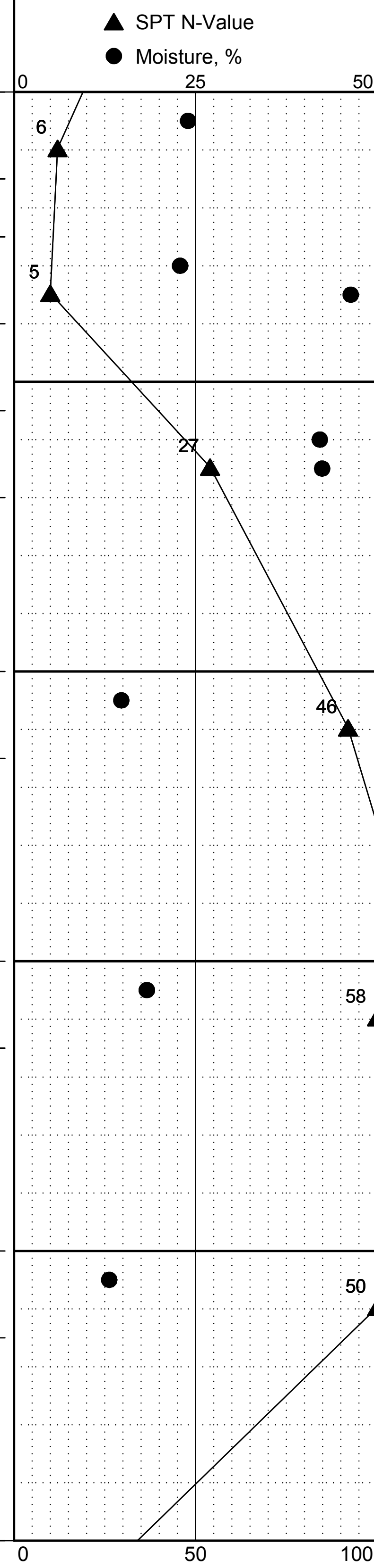
LOG OF BORING D-1

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
▼	-200 = 3% 9/21/2000		Wet, some medium grained sand.			S-10
	-200 = 13%		Grading slightly silty with interstratified Clayey SILT.			S-11
	-200 = 29%	0.2 31.8	3-inch layer of Clayey SILT at base of fill. (Recent Alluvium)		30	S-12
			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)			
	-200 = 9%				35	S-13
	-200 = 67%		**Cobbly Sandy GRAVEL** continued.		40	S-14
	-200 = 5%				45	S-15
		-17.5 49.5	**SILT**, slightly clayey, stiff, light brown, moist, low			



LEGEND

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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

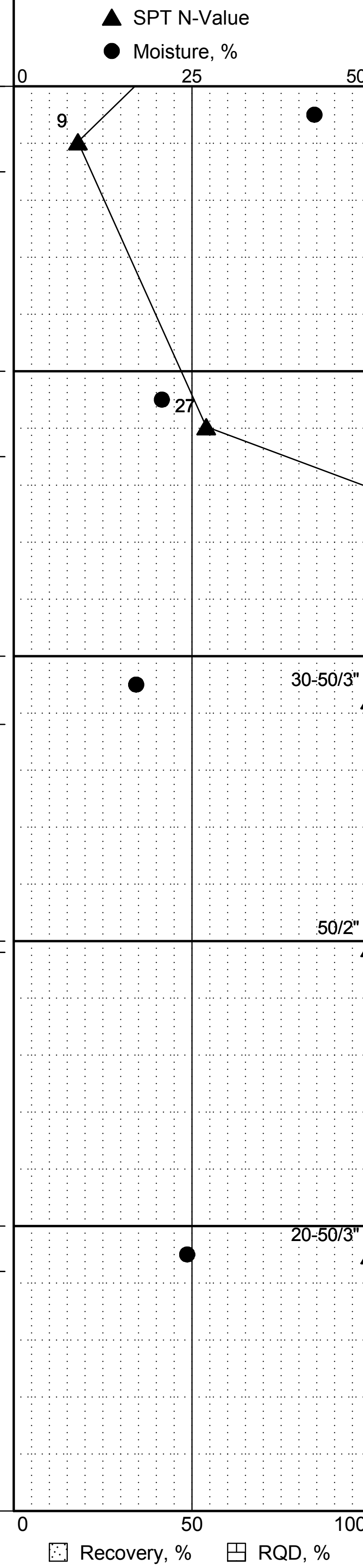
LOG OF BORING D-1

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
	-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	S-19*
					70	S-20



LEGEND

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- = 3.0" O.D. Thin-Walled Sample
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- = 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

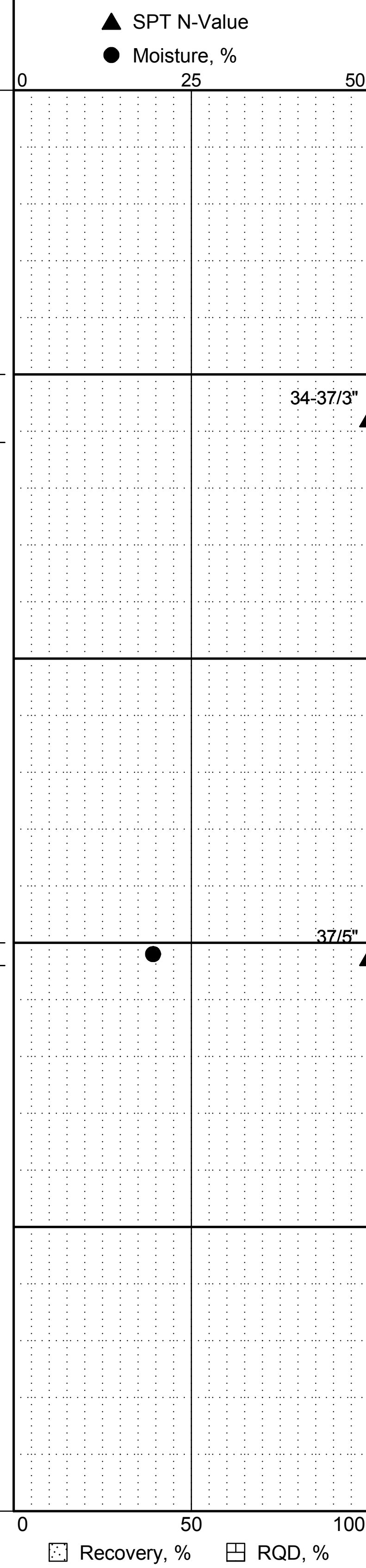
December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG.

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

Ground Water	Remarks	Elev. Depth Feet	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

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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 D-1

Page 4 of 5

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	▲ 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

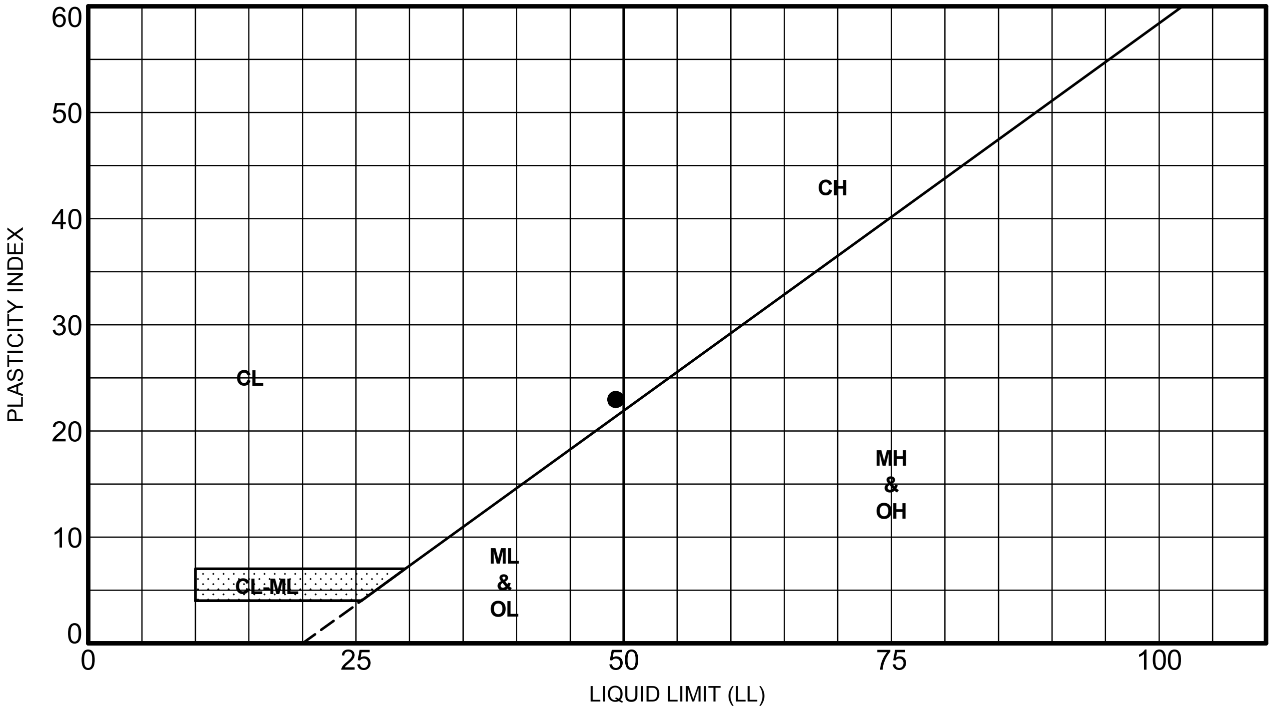
Page 5 of 5

December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

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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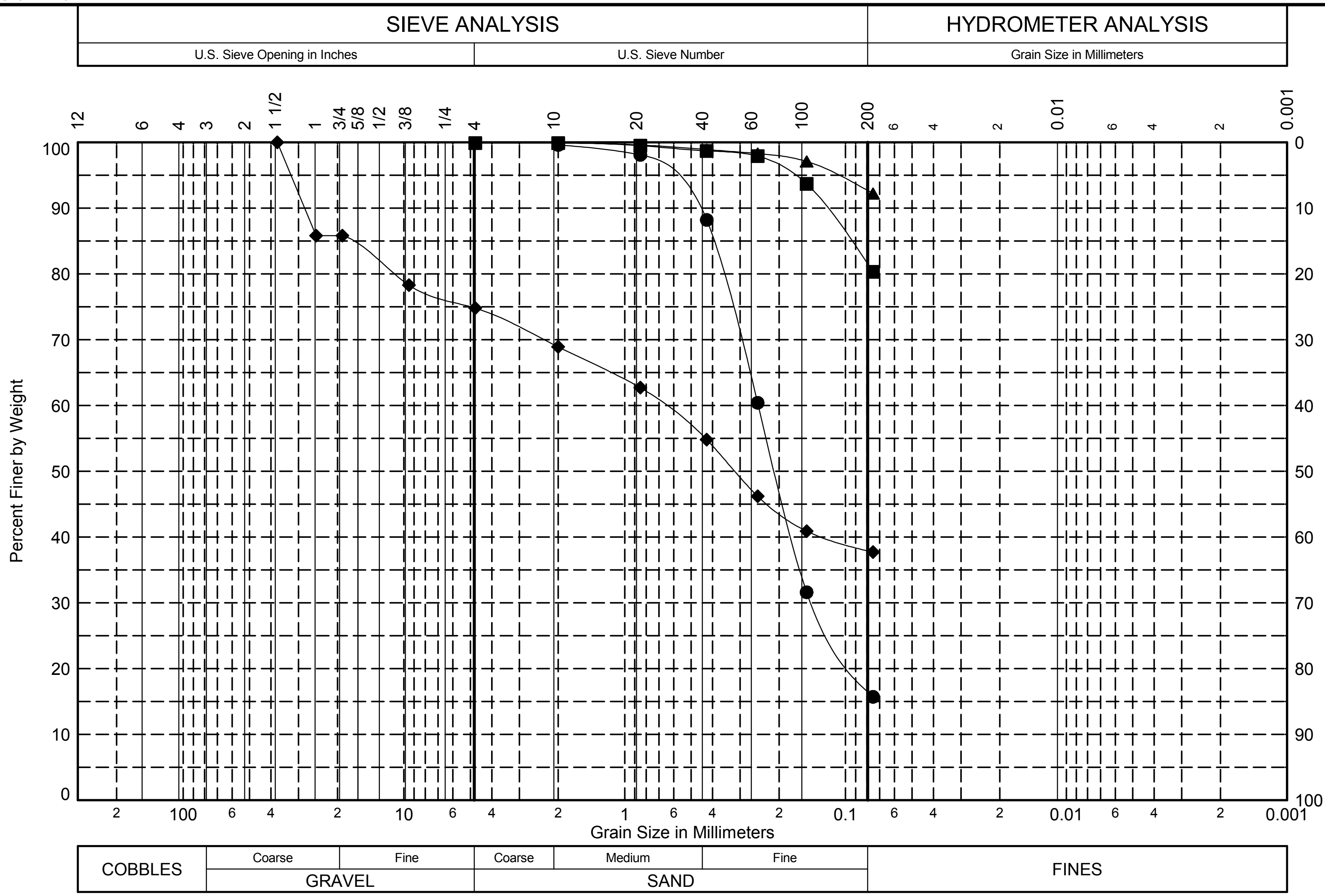
Portland, Oregon

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)				

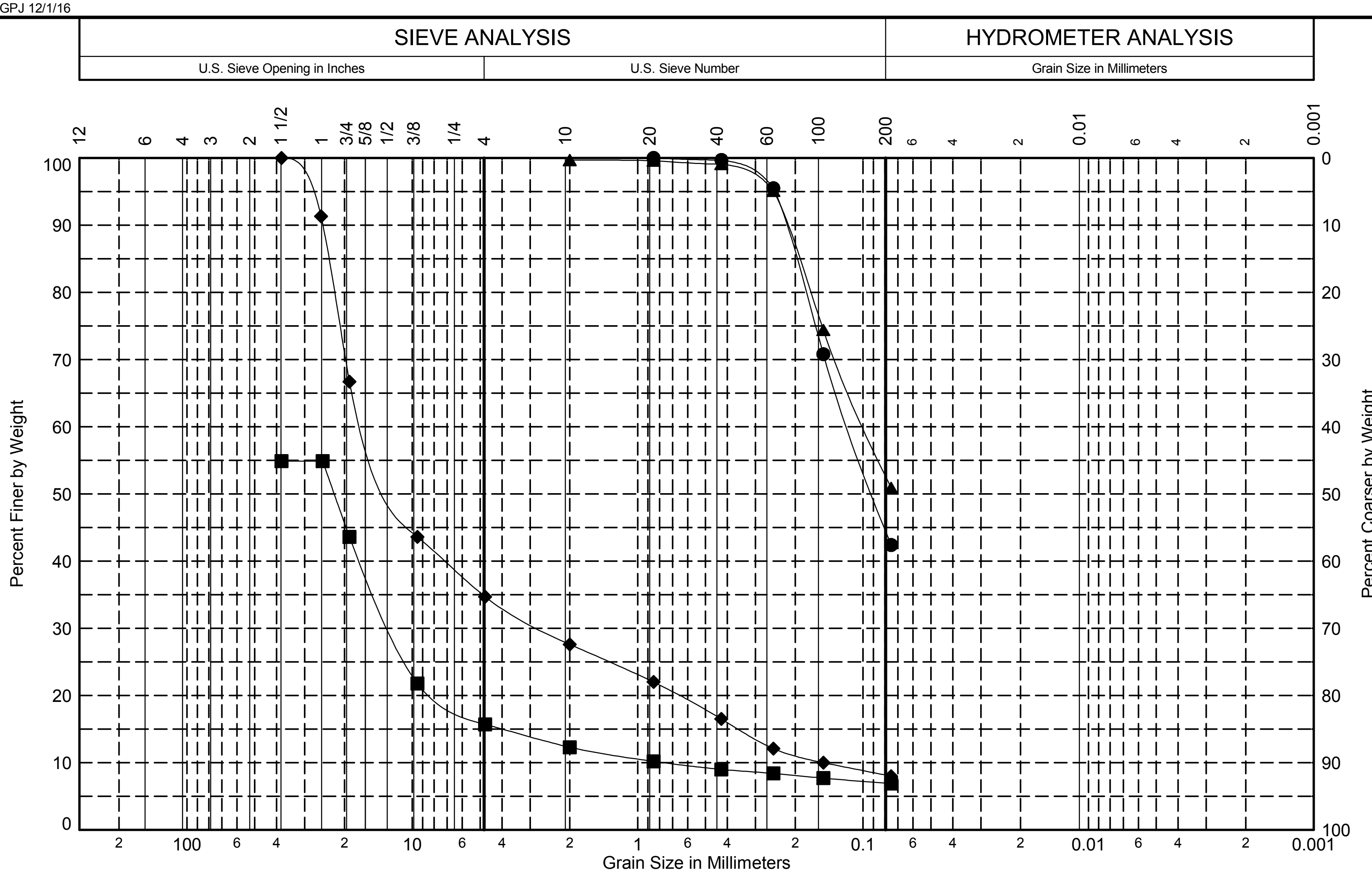
Ankeny Pump Station
Portland, Oregon

GRAIN SIZE DISTRIBUTION

December 2000 *F-3112.01*

SHANNON & WILSON, INC.
Geotechnical Consultants
Portland, Oregon

FIG. 1



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	

	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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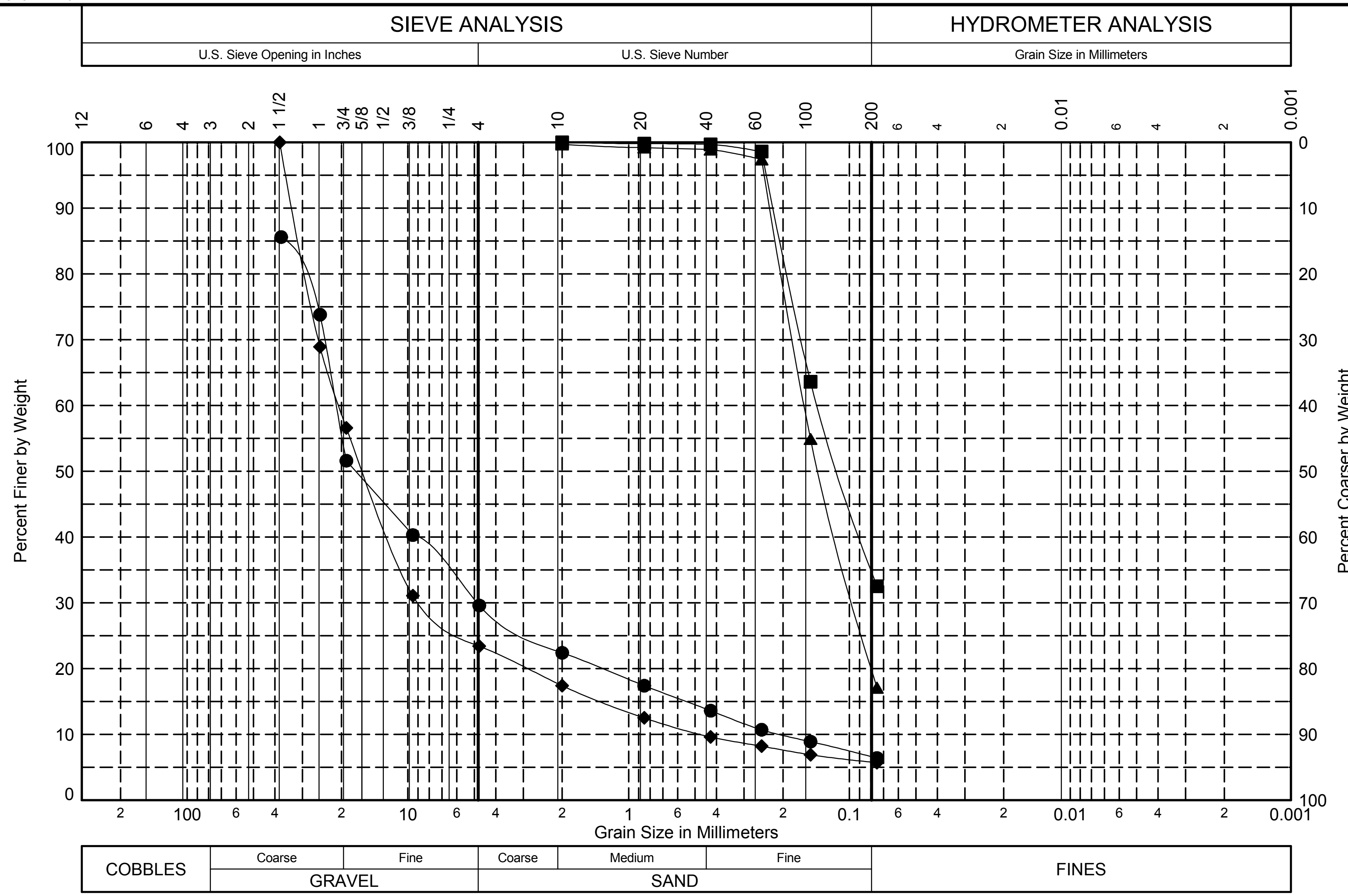
Geotechnical Consultants
Portland, Oregon

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
Portland, Oregon

GRAIN SIZE DISTRIBUTION

December 2000 *F-3112.01*

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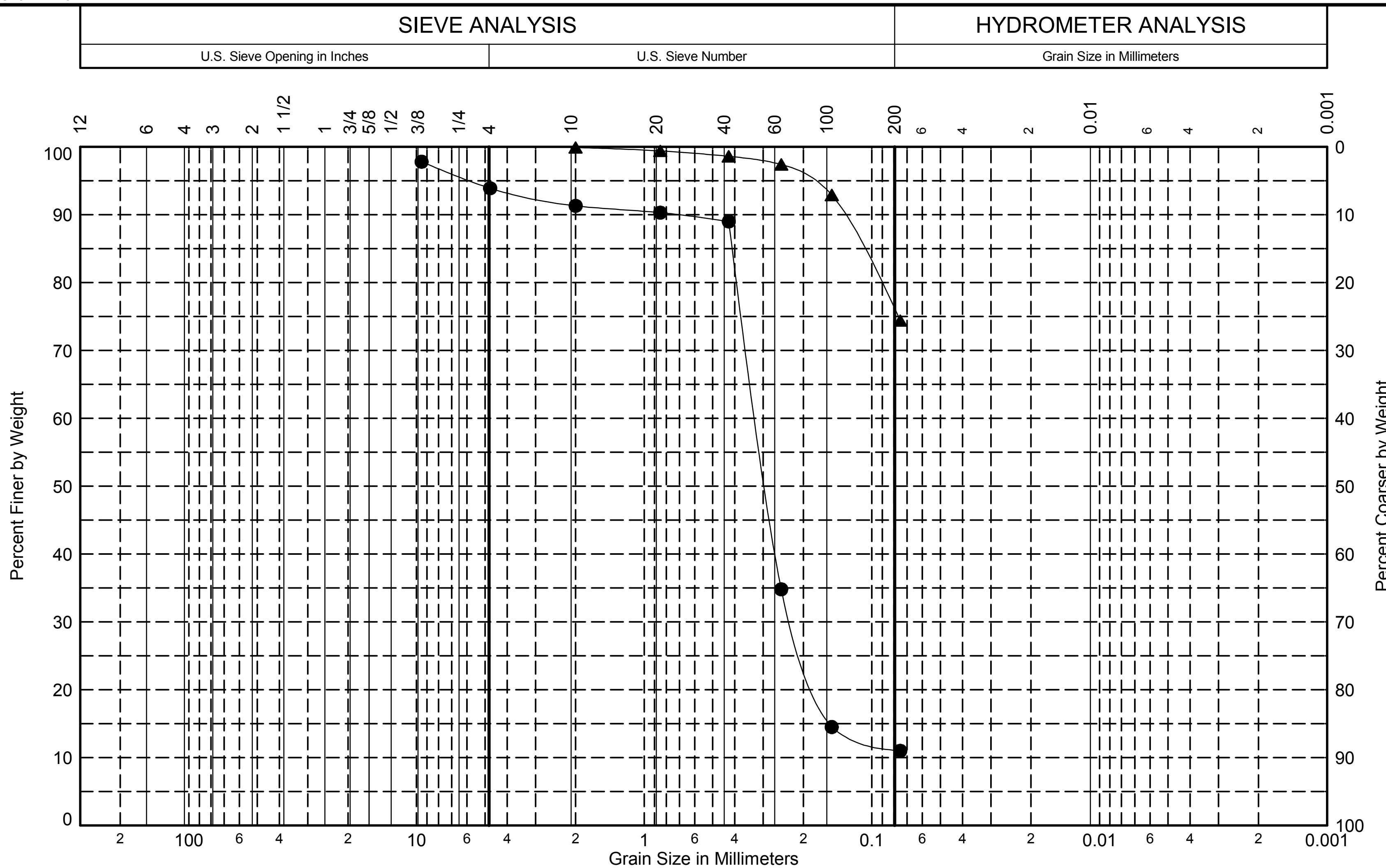
Geotechnical Consultants
Portland, Oregon

FIG. 3



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	

	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

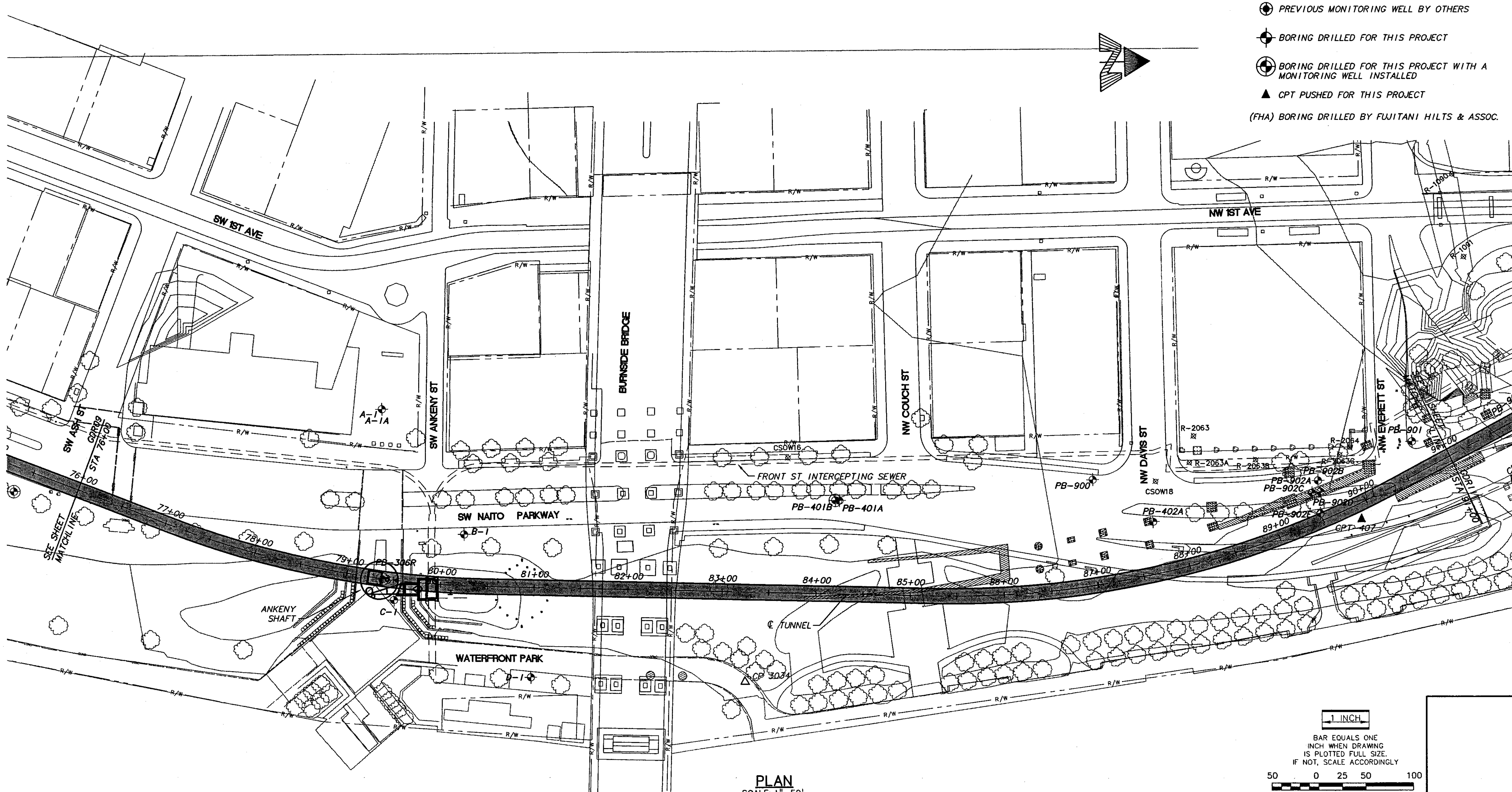
NOTES:

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.



1 INCH

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

50 0 25 50 100

SCALE FEET

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

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube

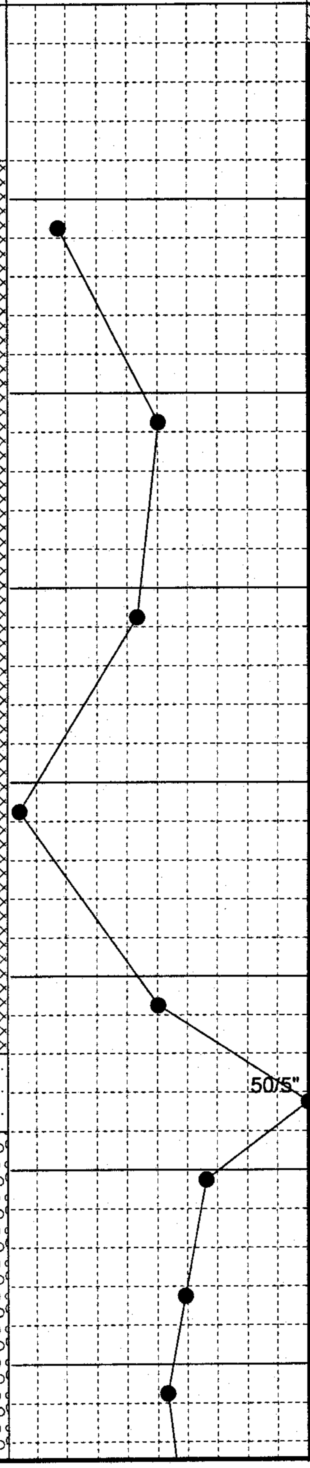
Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	SOIL DESCRIPTION
	0				SILT (ML); low plasticity, brown and grey, moist to wet, (Topsoil).
	4				FILL consisting of layered bricks, gravel, and wood fragments in a silt matrix; medium dense to very dense.
25	5	Standard Penetration Test	1	4, 5, 12	
20	10	Standard Penetration Test	2	23, 31, 19	
15	15	Standard Penetration Test	3	15, 23, 20	
	17				Drilling mud loss (200 gallons) at 17 feet.
10	20	Standard Penetration Test	4	5, 3, 1	Fill becomes very loose at 20 feet.
	25				Fill becomes very dense at 25 feet.
5	25	Standard Penetration Test	5	10, 27, 23	
	27	Ring	6	2, 21, 50	SILTY SAND (SM); few gravel, trace wood fragments, non-plastic silt, grey-brown, wet, very dense, (Sand/silt alluvium).
	29				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.
0	30	Ring	7	34, 35, 31	
	32.5	Ring	8	18, 29, 30	
	34.5				Drilling mud loss encountered at 34.5 feet.
-5	35	Ring	9	22, 28, 25	

PERCENT FINES: 20 40 60 80

PLASTIC — M.C. — LIQUID: 20 40 60 80

LITHOLOGY

UNCORRECTED BLOW COUNTS (last 12")



WELL

PB PARSONS BRINCKERHOFF 100 YEARS

BORING LOG PB-305A

Foundation Engineering, Inc.

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

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube

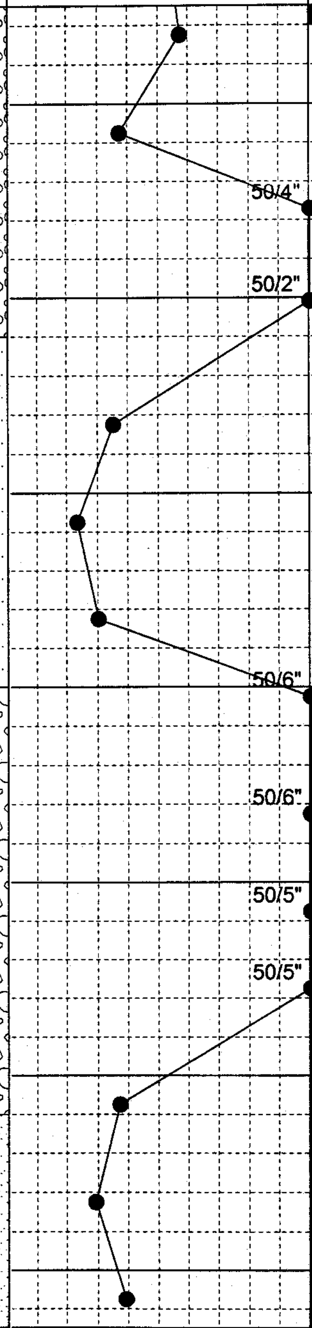
Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	SOIL DESCRIPTION
		10	40 33 24	POORLY GRADED GRAVEL WITH SAND (GP); fine to coarse grained gravel, subangular to rounded gravel, grey, wet, very dense, (Gravel alluvium), clasts consist of basalt and minor quartzite.
-10	40	11	20 20 17	
		12	50	
-15	45	13	50	
		14	12 17 18	SILTY SAND (SM); fine to medium grained, low plasticity silt, grey-brown, wet, medium dense to dense, (Sand/silt alluvium).
-20	50	15	12 11 12	
		16	13 17 23	
-25	55	17	50	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).
		18	21 29 50	
-30	60	19	25 37 50	
		20	50	
-35	65	21	18 18 19	POORLY GRADED SAND WITH SILT (SP-SM); fine to medium grained, low plasticity silt, grey-brown, wet, dense, (Troutdale Formation).
		22	11 13 16	
-40	70	23	17 16 23	
				Bottom of Boring at 71.5 feet.

△ PERCENT FINES △ 20 40 60 80

PLASTIC M.C. LIQUID 20 40 60 80

LITHOLOGY

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



WELL

PARSONS BRINCKERHOFF

BORING LOG PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT: West Side CSO Project		INITIAL GWL@ 23.0 ft (4/10/01)	
CITY: Portland, Oregon	SHEET 1 OF 8	EQUIPMENT: RotoSonic Drill	
PROJECT NO. 027-003	STATION NO. 79+38 (2' R)	DRILLING METHOD: Rotosonic - 6" OD Core Barrel	
DATE DRILLED 4/9/01 to 4/11/01	SURFACE ELEV. 28.95 ft	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
	0									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									
15	15								3	
10	20								4	
										TIMBER PILE.

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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

SAMPLE TYPE: G Grab Sample; O 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
▼									5		(Qal Cont'd)	
5											WOOD TIMBER PILE.	
	25			>1000								
				>1000								
0									6		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.	
	30											
											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.	
					450				7		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.	
-5		G	2									
	35											
											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.	
					600				8			
-10		G	3									
	40				1400						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.	
					1850				9			
-15		G	4								No recovery from 44 ft to 46 ft.	

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 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: 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
					1400						**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.	
									10		POORLY GRADED SAND (SP); 100% sand, fine to medium, subangular to rounded, hard, predominantly quartz, some basalt and mica; yellowish brown, wet.	
-20		Grab	5									
	50				0							
					3500							
									11			
-25		Grab	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.	
		Grab	7A			GSD	10	11	12			
-30		Grab	7									
	60											
									13			
-35		Grab	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.	

PARSONS BRINCKERHOFF

BORING LOG
PB-306R

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PROJECT: West Side CSO Project		INITIAL GWL@: 23.0 ft (4/10/01)	
CITY: Portland, Oregon	SHEET 4 OF 8	EQUIPMENT: RotoSonic Drill	
PROJECT NO.: 027-003	STATION NO.: 79+38 (2' R)	DRILLING METHOD: Rotosonic - 6" OD Core Barrel	
DATE DRILLED: 4/9/01 to 4/11/01	SURFACE ELEV.: 28.95 ft	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.
									15	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.
-45	75	G	9							
					>5000					
									16	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.
-50		G	10			GSD	10	11		
	80				>5000					
					>5000					
			TUNNEL CROWN							
									17	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.
-55		G	11							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.
	85				>5000					
					>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.
									18	
-60		G	12							

PARSONS BRINCKERHOFF 100 YEARS

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	SOIL DESCRIPTION
									23	(Tt Cont'd)
-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					
-95		G	19						25	
	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					
									27	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.
-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
					>5000						(Tt Cont'd)	
					3900						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.	
									28		Becomes dark gray poorly graded gravel with sand, silt and cobbles (GP-GM).	
-110		G	22			GSD	31	67			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.	
	140				3350							
					2600							
		G	23						29			
-115											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.	
	145				>5000						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'.	
					>5000						CONGLOMERATE (GC); with mottled green, dark gray matrix of weakly indurated silty claystone; gravel, fine to coarse, subrounded to rounded.	
									30			
-120		G	24		>5000	GSD	7	31			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".	
	150											
					>5000							
									31			
-125		G	25									
	155				>5000							
											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.	
											(Tsr Cont'd)	
-130		G	26			AL, FC	25	93			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												

PB PARSONS BRINCKERHOFF 100 YEARS

BORING LOG
PB-401A

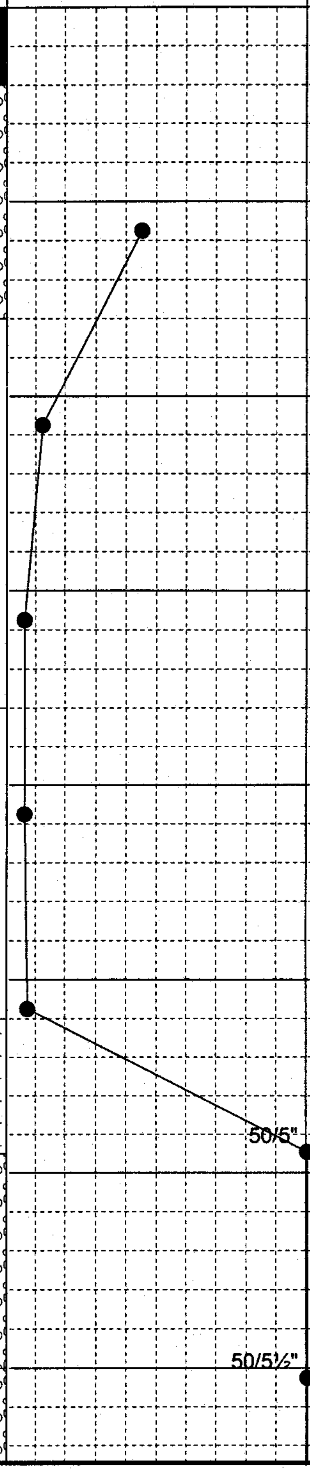
Foundation Engineering, Inc.

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

Column headings: 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

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	SOIL DESCRIPTION
30				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).
25	5	1	24 28 17	
				SILT (ML); trace fine sand, low plasticity, brown, moist, stiff, (Sand/Silt Alluvium).
20	10	2	4 5 7	
15	15	3	2 3 3	Soil becomes medium stiff and increases in sand content below 15 feet.
		4		SILT (ML); low plasticity, brown, moist, medium stiff, (Sand/Silt Alluvium).
10	20	5	3 3 3	
5	25	6	1 2 5	
				SILTY SAND (SM); fine sand, non plastic to low plasticity, brown, wet, medium dense, (Sand/Silt Alluvium)
		7		
		8	30 50	SILTY GRAVEL WITH SAND (GM); some sand, fine to coarse subrounded gravel, low plasticity fines, brown, wet, very dense, (Gravel Alluvium).
0	30			
-5	35	9	50	POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse subrounded gravel, grey, wet, very dense, (Gravel Alluvium).



PARSONS BRINCKERHOFF

BORING LOG PB-401A

Foundation Engineering, Inc.

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

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube

Columns: 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

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"
-10	40	10	50
-15	45	11	41 50
-20	50	12	7 12 14
		13	
-25	55	14	6 12 21
-30	60	15	40 37 37
-35	65	16	50
-40	70	17	50

SOIL DESCRIPTION

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

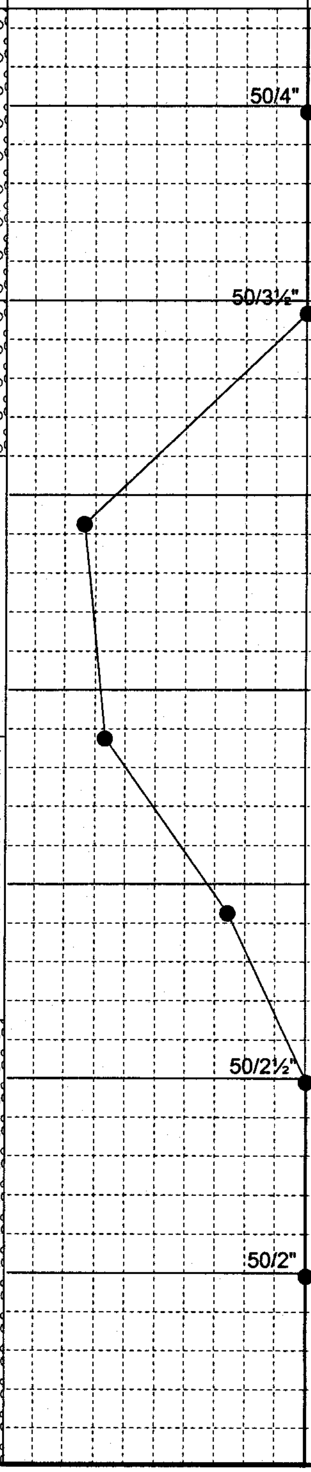
POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse rounded gravel, grey-orange, wet, very dense, (Troutdale Formation).

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).



PARSONS BRINCKERHOFF

BORING LOG
PB-401A

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
SHEET 3 OF 3
PROJECT NO. 2002013
STATION NO. 84+19 (99L)
DATE DRILLED 3/6/01 LOGGED BY AR
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		SPT	18	50		Lense of sand with gravel at 75 feet.		50/4½"	
	80	SPT	19	50		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).		50/4½"	
-50						Bottom of boring at 80.33 feet.			
-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.			
	5								
25									
	10								
20									
	15								
15									
	20								
10									
	25								
5									
	30								
0									
	35								
-5									

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 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
	40					Drilled 0 to 85 feet without sampling. Material description between 0 and 80 feet is show on PB-401A.			
-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

Columns: 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

Soil Description

Drilled 0 to 85 feet without sampling. Material description between 0 and 80 feet is show on PB-401A.

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

85 ft: TUNNEL CROWN

~89 ft: SILT (ML) some clay, low to medium plasticity, grey, wet, hard, (Troutdale Formation).

~91 ft: POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse rounded gravel, grey-brown, wet, very dense, weakly cemented, (Troutdale Formation).

Falling head permeability test conducted at 97.5 to 99.5 ft. Hydraulic conductivity is approx. 5E-06 cm/sec

~101 ft: TUNNEL INVERT

Becomes dark green to grey below 102 feet.

Falling head permeability test conducted at 110 to 112 ft. Hydraulic conductivity is approx. 3E-06 cm/sec

Samples

Depth (ft)	Sample No	Field Blows /6"	Uncorrected Blow Counts (last 12")
85	1	50	50/2"
	2	50	50/4"
90	3	17, 50	50/4"
	4	50	50/1"
95	5	50	50/0"
	6	50	50/0"
100	7	50	50/2"
	8	50	50/3"
105	9	50	50/2½"
	10	50	50/2"
110	11	50	50/1"

Elevation scale: -45, -50, -55, -60, -65, -70, -75, -80
Depth scale: 80, 85, 90, 95, 100, 105, 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 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

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
			12	50		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).		50/1"	
-85	115		13	50				50/2"	
			14	50				50/2½"	
-90	120		15	50				50/1"	
			16	50				50/2½"	
-95	125		17	50				50/1"	
			18	50				50/3½"	
-100	130		19	50		Trace clay encountered below 130 feet.		50/3½"	
-105	135		20	50				50/4"	
-110	140		21	50				50/5¾"	
						Bottom of boring at 140.5 feet.			
-115	145								

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

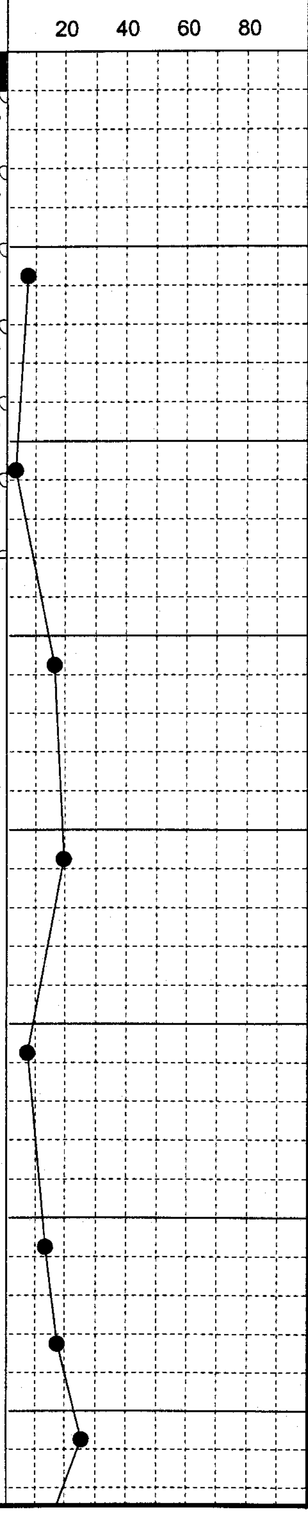
Elevation (ft)	DEPTH (ft)	SAMPLE TYPE	SAMPLE NO	FIELD BLOWS /6"	SOIL DESCRIPTION
31.9	0				ASPHALTIC CONCRETE AND PORTLAND CEMENT CONCRETE (Road surface).
					GRAVELLY SILT WITH SAND (ML); fine to coarse grained sand, angular gravel, low plasticity, grey-brown, moist, medium stiff, (Fill).
	5	SPT	1	6, 4, 3	
	9				Scattered wood fragments encountered at 9 feet.
	10	SPT	2	0, 1, 2	Fill becomes soft with few gravel below 10 feet.
	13				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).
	15	SPT	3	9, 8, 8	
	20	SPT	4	13, 9, 10	
	25	SPT	5	3, 4, 3	
	30	SPT	6	2, 4, 9	
	32.5	SPT	7	11, 8, 9	
	35	Ring	8*	16, 14, 11	* 3 inch sampler/140 lb hammer

Elevation scale: 30, 25, 20, 15, 10, 5, 0, -5

PERCENT FINES (20, 40, 60, 80); PLASTIC — M.C. — LIQUID (20, 40, 60, 80)

LITHOLOGY

UNCORRECTED BLOW COUNTS (last 12")



WELL

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

Column headings: 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

Depth scale: 40, 45, 50, 55, 60, 65, 70. Elevation scale: -10, -15, -20, -25, -30, -35, -40.

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

SOIL DESCRIPTION

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 / PLASTIC – M.C. – LIQUID (20 40 60 80)	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") (20 40 60 80)	WELL
-45		Standard Penetration Test	24	3 3 4					
	80	Standard Penetration Test	25	3 4 4					
-50						Bottom of Boring at 81.5 feet.			
	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.			

PARSONS BRINCKERHOFF

BORING LOG
PB-900

Foundation Engineering, Inc.

PROJECT West Side CSO Project
CITY Portland, Oregon
SHEET 2 OF 3
PROJECT NO. 2002013
STATION NO. 87+12 (111 L)
DATE DRILLED 9/20/00 LOGGED BY AR
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.

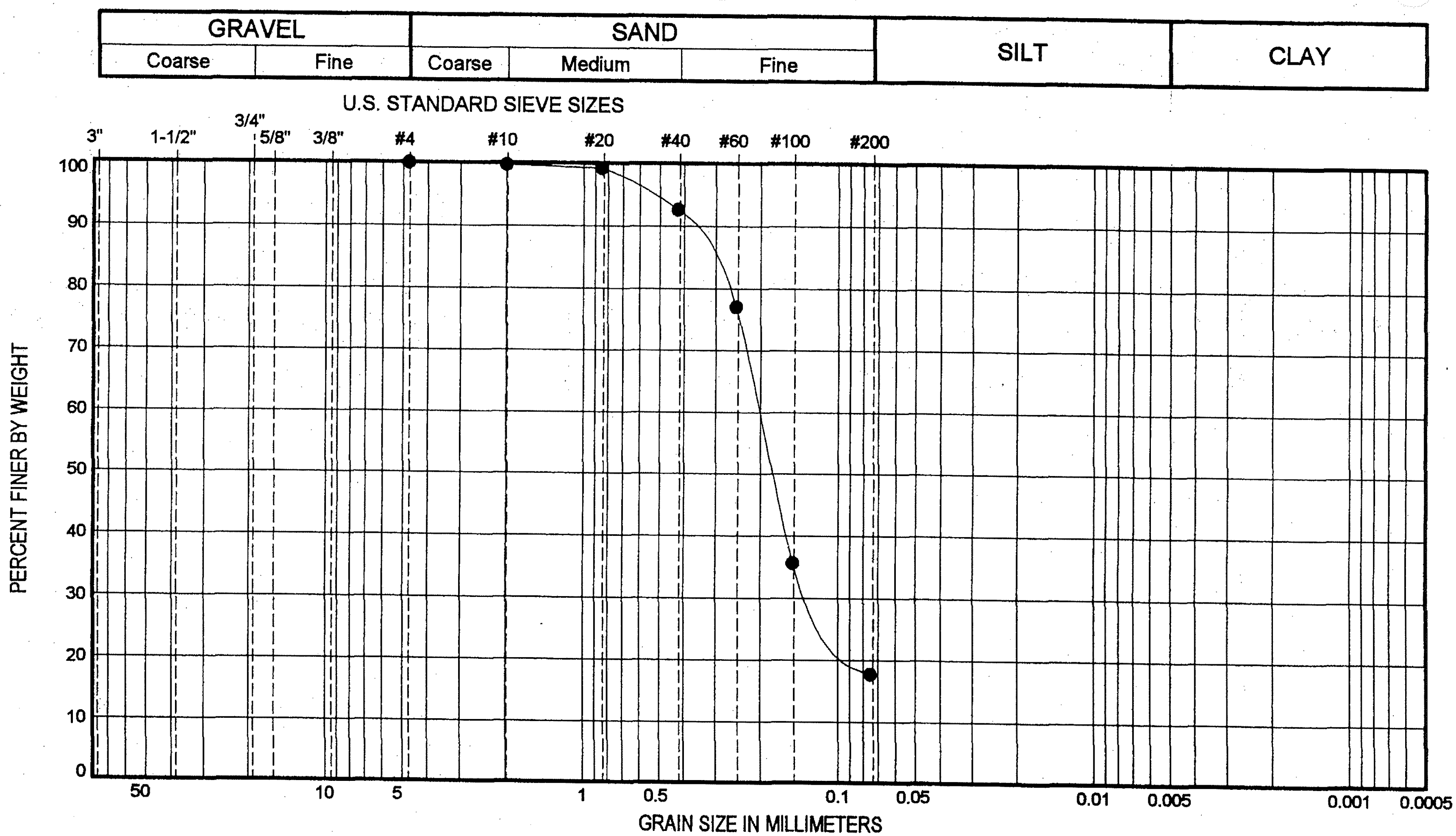
PROJECT West Side CSO Project
CITY Portland, Oregon
PROJECT NO. 2002013
DATE DRILLED 9/20/00 LOGGED BY AR

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

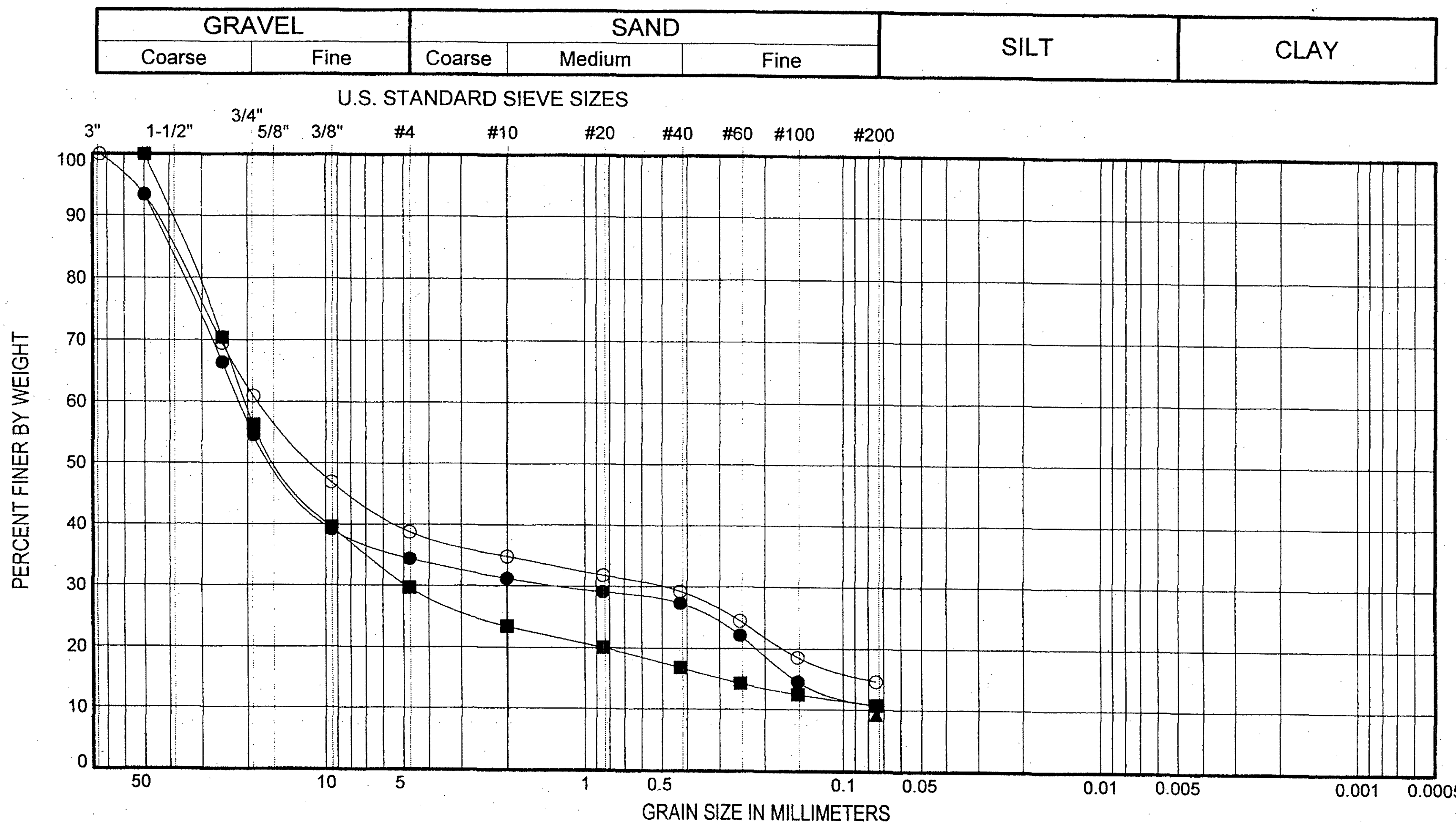
West Side CSO Project

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.

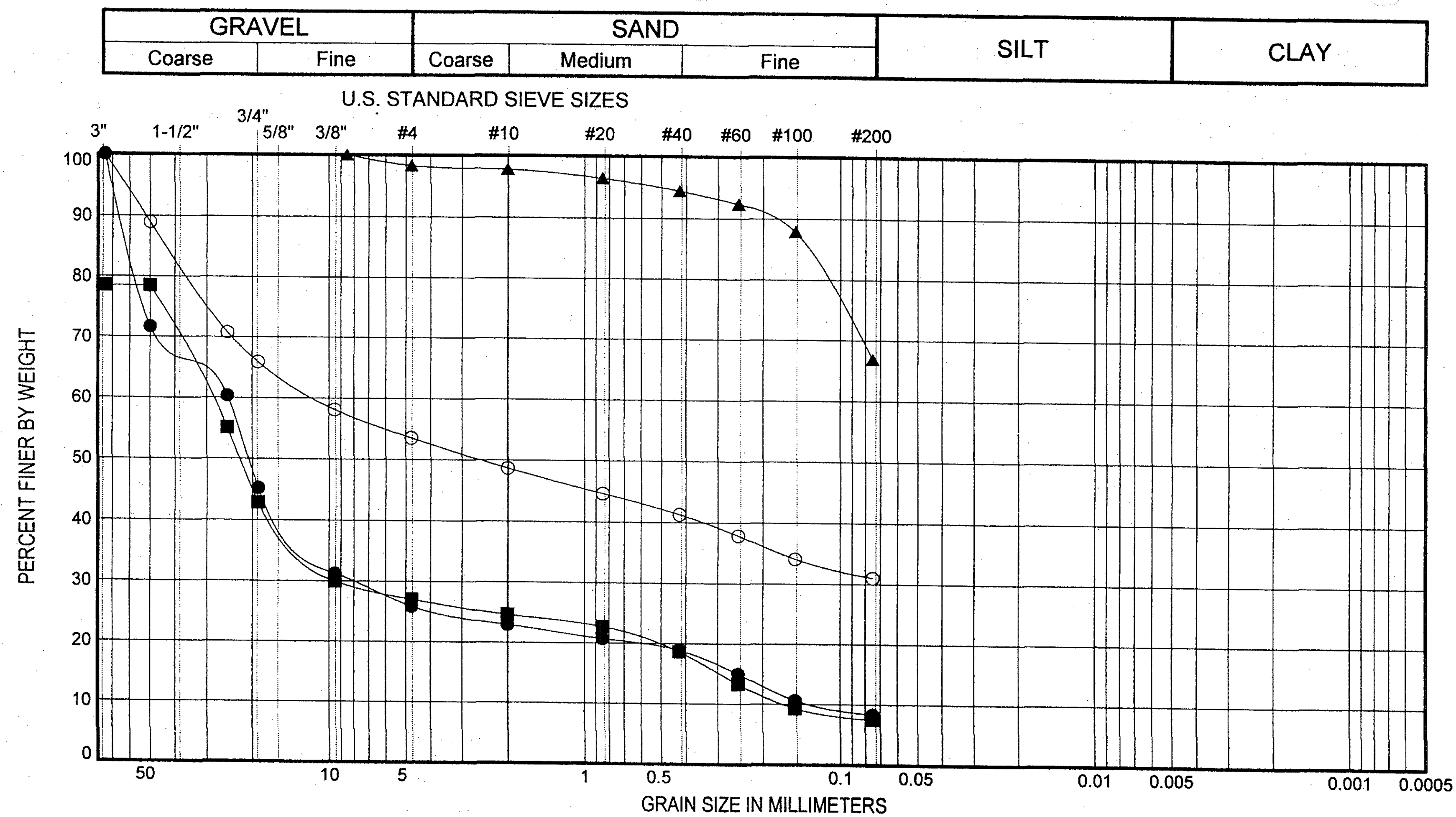
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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.

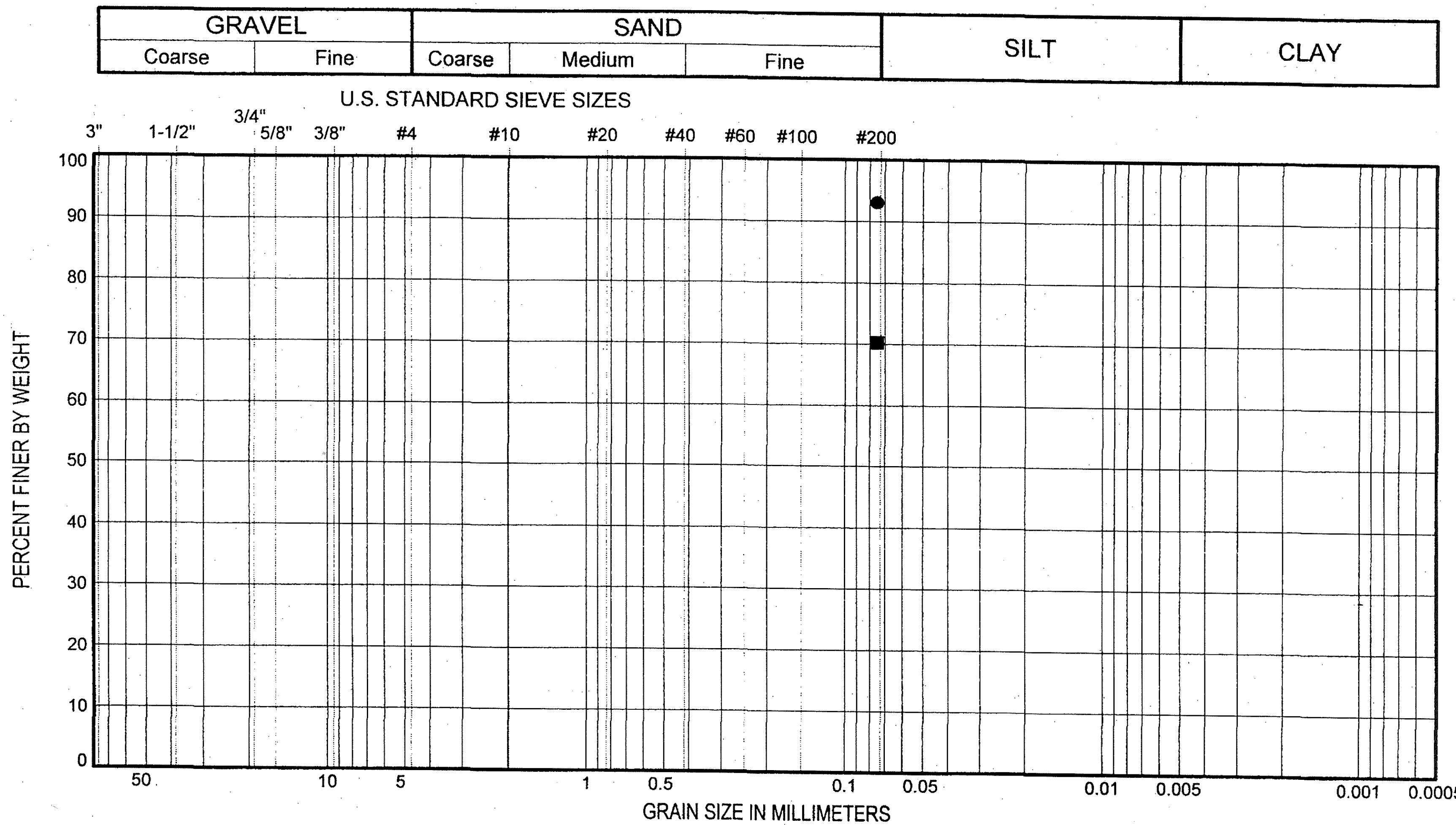
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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.

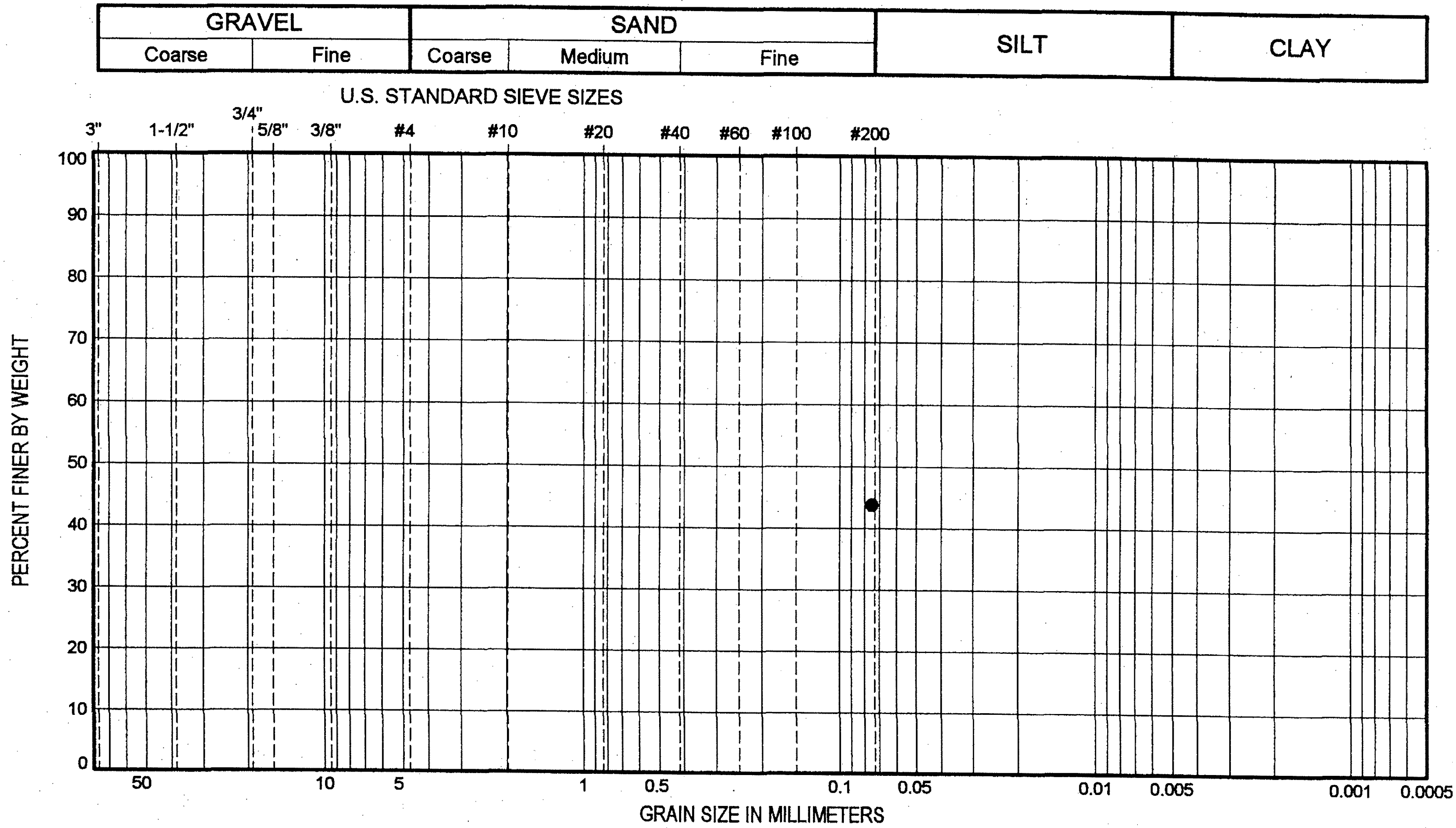
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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

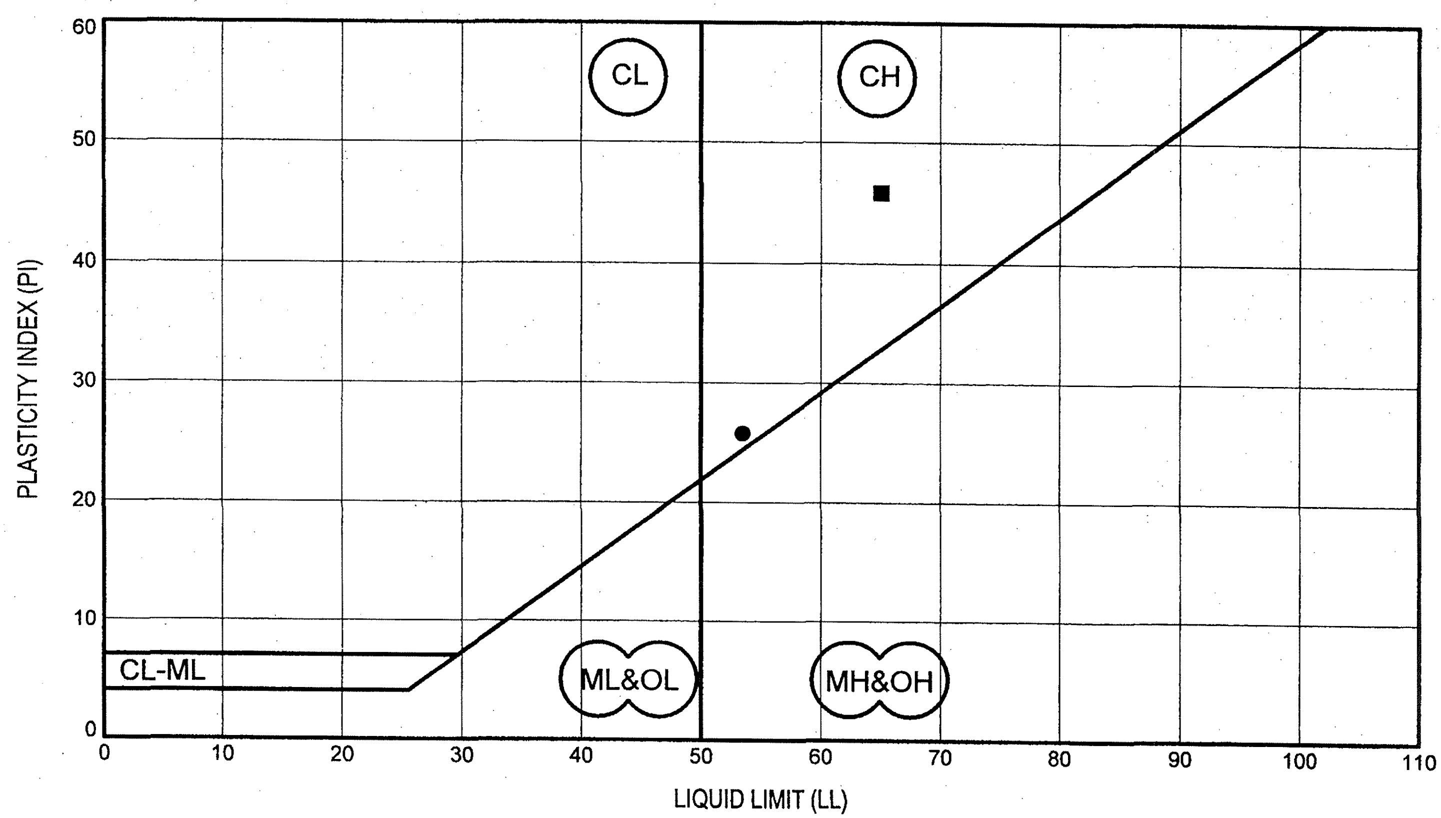
West Side CSO Project

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.

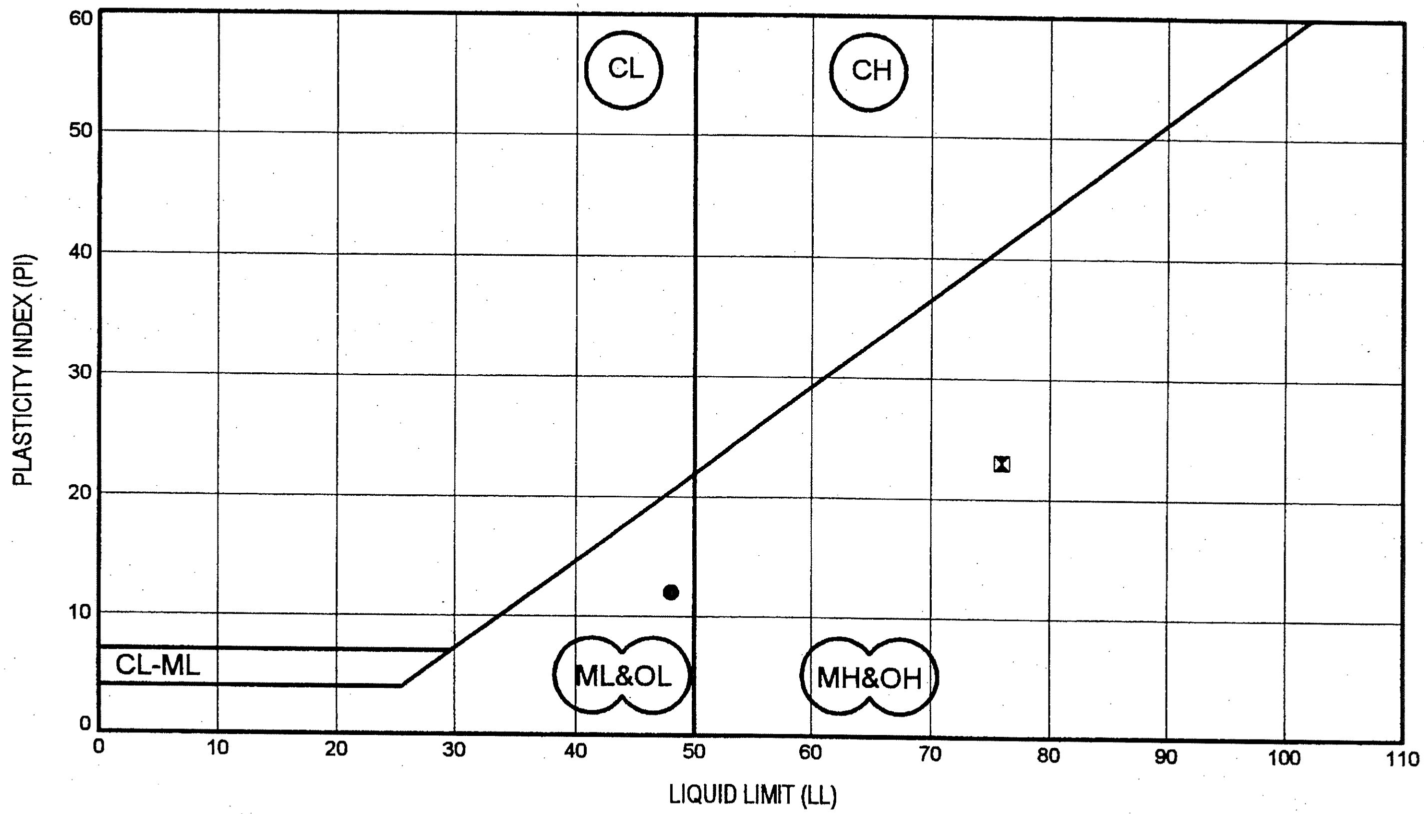
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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

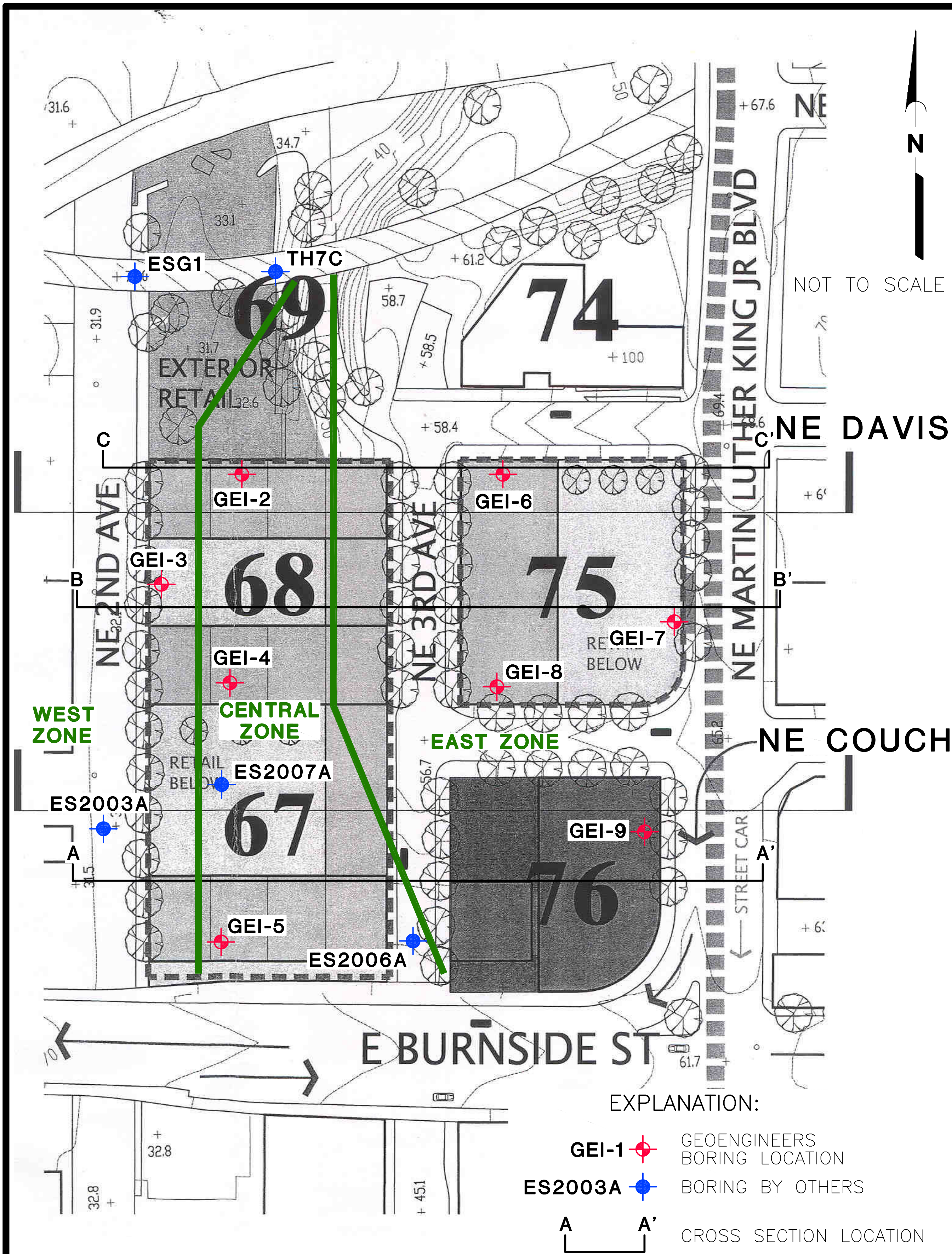
Specific Conductance (µSiemens/cm)	Temp (C)	pH	Eh (mV)	Turbidity Data from wells only (NTUs)
104.3	9.5	5.97	-505	0.67
1,814	26.2	7.83	71.4	8
492	16.8	6.71	-260	0.8
654.7	13.2	6.48	28	
353	14.7	6.7	-177	8
1,122	21.2	7.83	-258	---
995	21.6	6.07	-176.8	---
330.6	17.2	7.52	-306.2	2.81
1,622	15.4	6.57	-215.4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

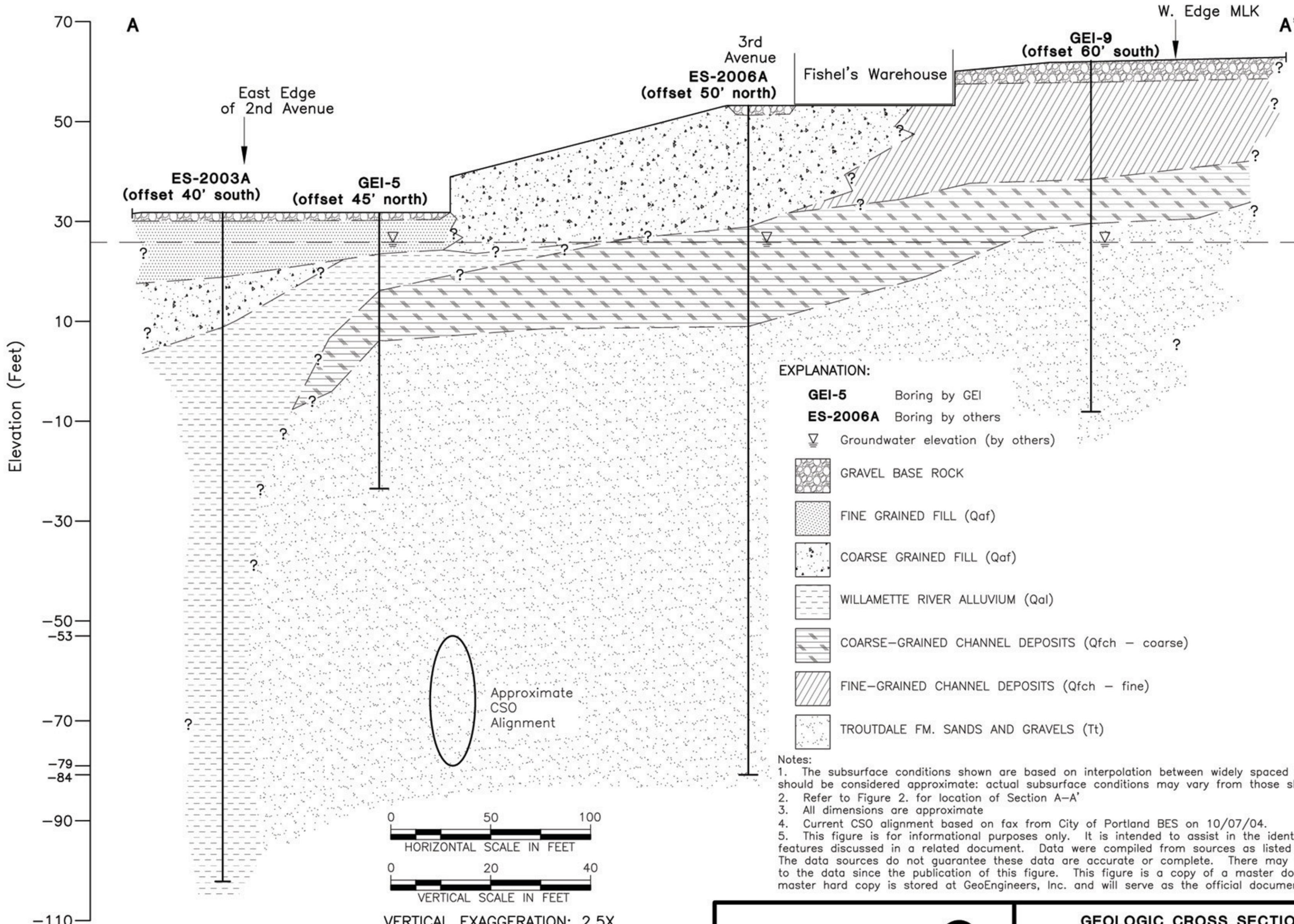


GEOENGINEERS

SITE PLAN

FIGURE 2

Portland\P:\1\1792018\00\Cad\GTFigure2.dwg TWB:MWJ 11/05/04



EXPLANATION:

GEI-5 Boring by GEI

ES-2006A Boring by others

Groundwater elevation (by others)

GRAVEL BASE ROCK

FINE GRAINED FILL (Qaf)

COARSE GRAINED FILL (Qaf)

WILLAMETTE RIVER ALLUVIUM (Qal)

COARSE–GRAINED CHANNEL DEPOSITS (Qfch – coarse)

FINE–GRAINED CHANNEL DEPOSITS (Qfch – fine)

TROUTDALE FM. SANDS AND GRAVELS (Tt)

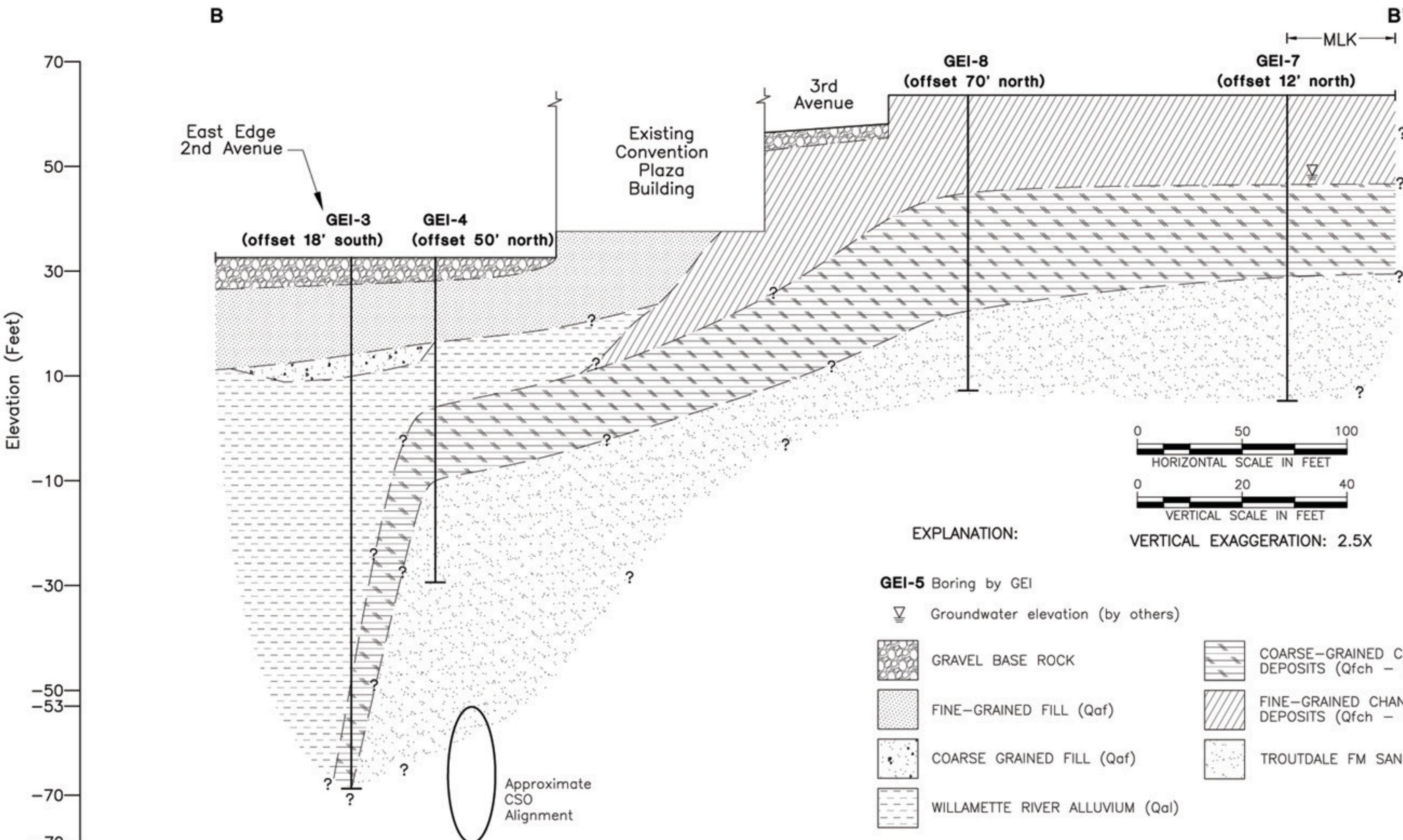
Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate: actual subsurface conditions may vary from those shown.
2. Refer to Figure 2. for location of Section A–A'
3. All dimensions are approximate
4. Current CSO alignment 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

Portland\P:\1\17920018\00\Cad\CrossAA.dwg TWB:MWJ 10/27/04



EXPLANATION:

GEI-5 Boring by GEI

Groundwater elevation (by others)

GRAVEL BASE ROCK

FINE-GRAINED FILL (Qaf)

COARSE GRAINED FILL (Qaf)

WILLAMETTE RIVER ALLUVIUM (Qal)

COARSE-GRAINED CHANNEL DEPOSITS (Qfch – coarse)

FINE-GRAINED CHANNEL DEPOSITS (Qfch – fine)

TROUTDALE FM SANDS AND GRAVELS (Tt)

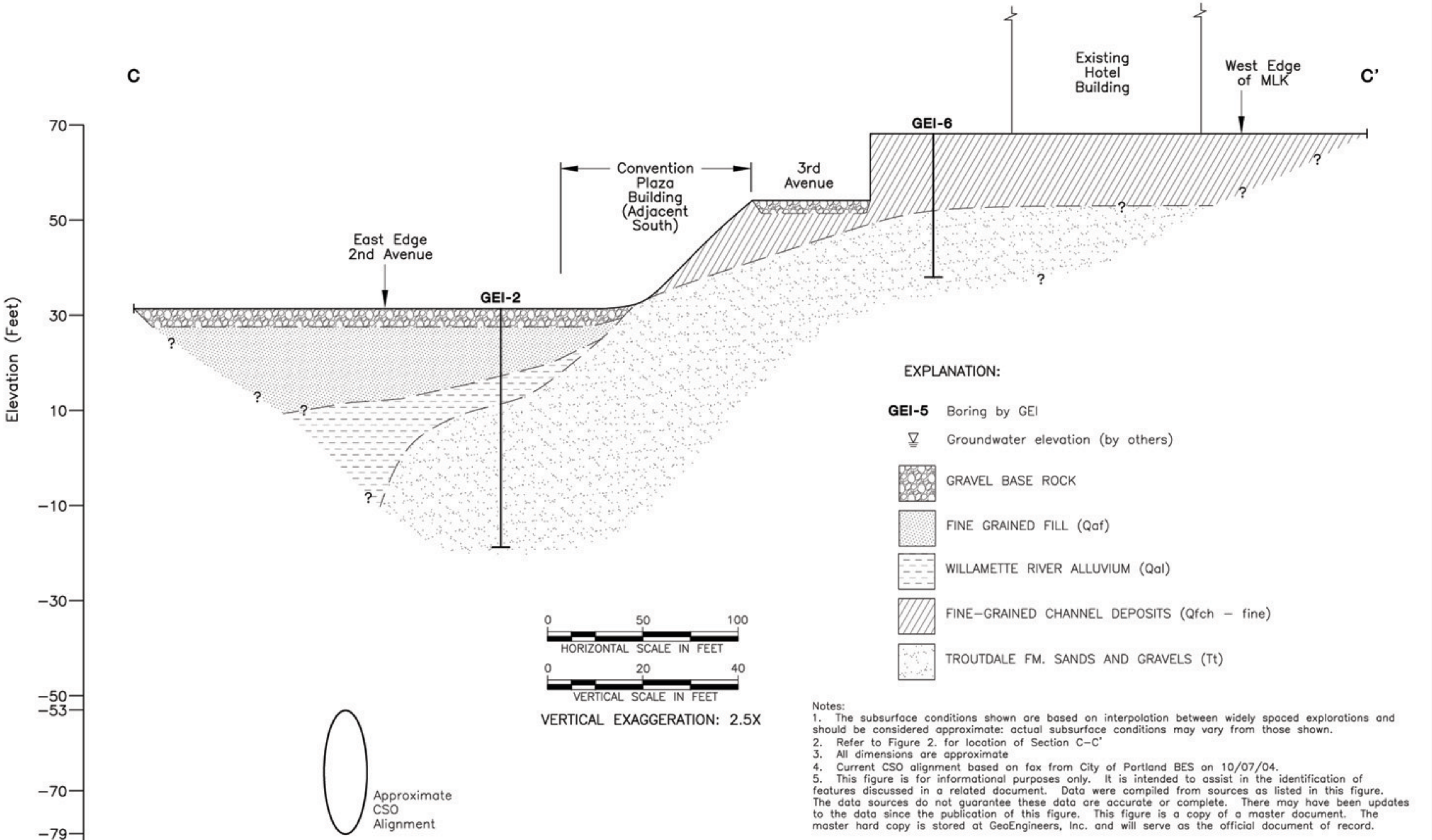
Notes:
1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate: actual subsurface conditions may vary from those shown.
2. Refer to Figure 2. for location of Section B–B'
3. All dimensions are approximate
4. Current CSO alignment 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/ft³	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

V6_GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV6_1.GDT 10/26/04

Depth feet	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
25		6	50/6"	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	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								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		1							(0922)
									Grades to silty fine to coarse gravel			
5		6	9		2			ML	Gray silt with fine gravel (medium stiff to stiff, moist) (fill)	33		(0955)
		5	7		3							
10		0	8		4				Becomes soft			(1005)
15		0			5					30		
								GM	Gray silty fine gravel with sand and wood/organics (medium dense, moist, mottling) (fill)			
		18	65		6							No recovery; buried log 17.5-21'
20		8	67		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

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							

V6_GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV6_1.GDT 10/26/04

LOG OF BORING GEI-3 (continued)

GEOENGINEERS

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

Figure: A- 3

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
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	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

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: Recovered (in)	SAMPLES: Blows/foot	SAMPLES: 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

V6 GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV8 1.GDT 10/28/04

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	Recovered (in)	Blows/foot	Sample Number	Water Level	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			

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	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
								Sandstone.			
60		0	50/0"	16							(1510)

Boring terminated at 61 feet
Groundwater not determined

Depth scale: 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/ft^3	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	Recovered (in)	Blows/foot	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							(1240)
							Becomes very dense			
										(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)			(1015)
							Clasts are 80% basalt, 20% felsic			
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

V6 GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV6_1.GDT 10/28/04

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: Recovered (in)	Blows/foot	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 (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

V6 GTBORING P:\1\1792018\00\FINAL\179201800.GPJ GEIV6_1.GDT 11/4/04

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	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			58		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"		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	Recovered (in)	Blows/foot	Sample Number	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	1			23	84	
10	24		2					DS
15	15	67	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	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: 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		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

V8_GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV8 1.GDT 10/28/04

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
											End drilling at 1520

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

60
65
70
75
80
85

V6_GTBORING P:\1\1792018\00\FINALS\179201800.GPJ GEIV6_1.GDT 10/28/04

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	SAMPLES: Recovered (in)	SAMPLES: Blows/foot	SAMPLES: 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							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
15		16	32	3			ML	Brown sandy silt (stiff, moist)	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

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		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

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: Recovered (in)	Blows/foot	Sample Number	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 (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	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	83/11"		5					16		(1550)
30		12	85		6				Becomes very dense	17		End drilling 6/28/04 at 1635. Start drilling 6/29/04 at 0845
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)

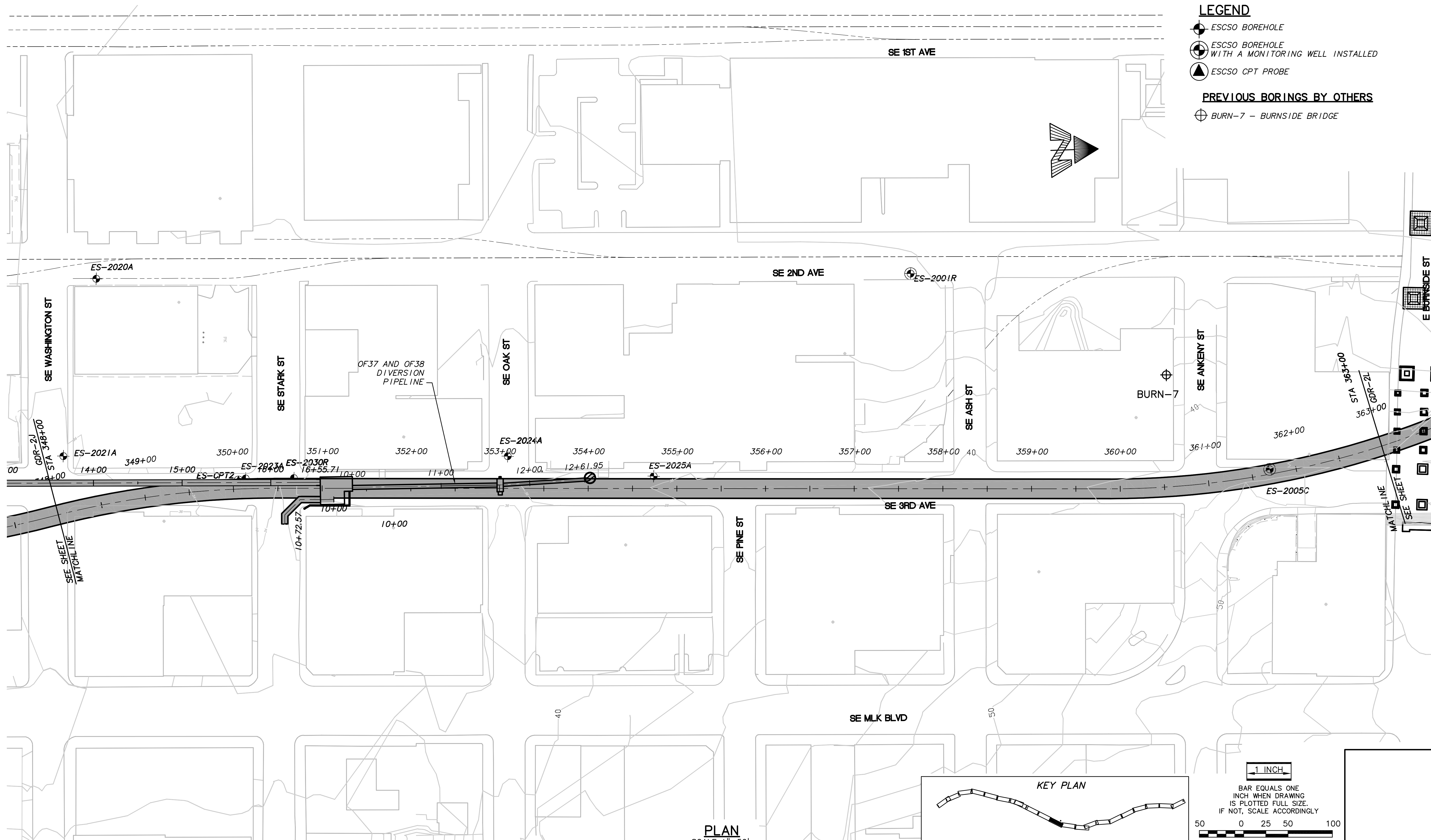
Depth feet	Interval	Recovered (In)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	Dry Unit Weight, lbs/ft^3	OTHER TESTS AND NOTES
60		0									
			50/2"	12							(1220) Grab sample
							GM				
65		0.5	50/1"	13				Brown silty gravel 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



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REVISION

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PROJECT COMPLETED
MAP CORRECTED BY — CHECKED BY
FINAL MAP DATA
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DRAWN BY	TAS	PROGRAM MGR	PTG
CHECKED BY	MLF	CONST. MGR.	GC
DESIGN MGR.	RFC	PROJECT MGR.	TRC

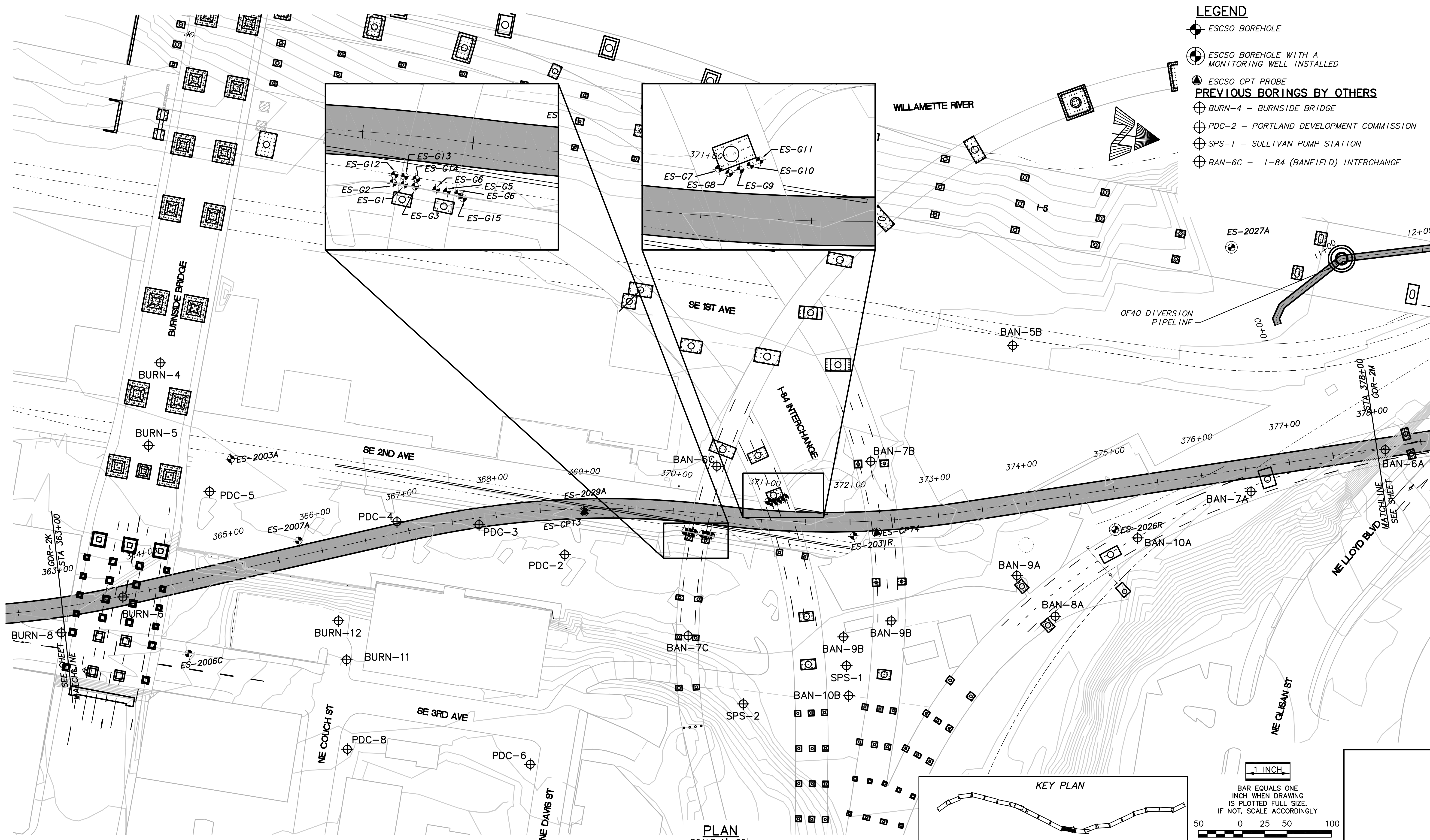
CITY OF PORTLAND
ENVIRONMENTAL SERVICES
WILLAMETTE RIVER CSO CONTROL PROJECT

PB 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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	07/15/05	GDR	
		REVISION	

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CONSTRUCTED BY
PROJECT COMPLETED
MAP CORRECTED BY ______ CHECKED BY ______
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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

PB 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 —

PARSONS BRINCKERHOFF

BORING LOG
ES-2003A

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design | SHEET 1 OF 4 | INITIAL GWL@ Not Available

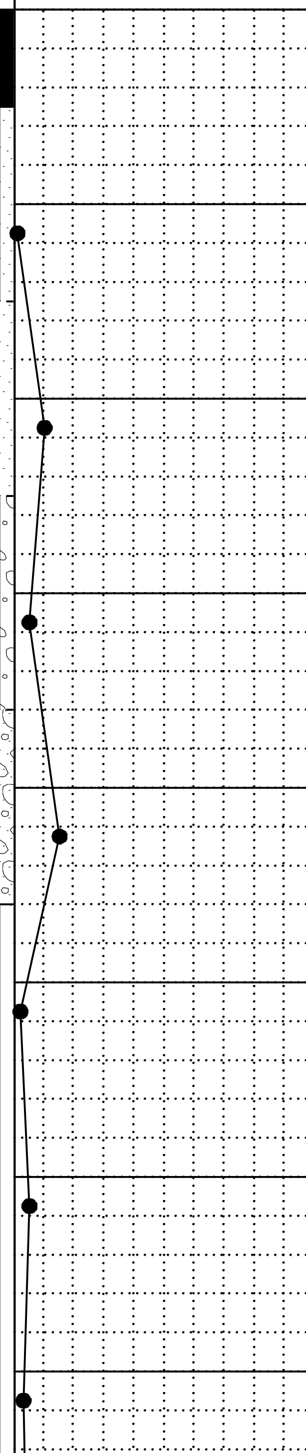
CITY Portland, Oregon | SURF. EL. 31.8 ft STA. OFST. | EQUIPMENT CME 75

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 drop

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube | Grab Sample

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	SOIL DESCRIPTION
31.8	0			**Pavement** ASPHALTIC CONCRETE OVER BASE ROCK.
	2.5			**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.
	5	1	1 0 1	
	7.5			SILTY SAND (SM); trace to some gravel, fine to medium sand, fine gravel, rounded, low plasticity, gray, moist, loose.
	10	2	3 5 5	
	12.5			GRAVELLY SILT (ML); fine gravel, rounded, low plasticity, gray, moist, soft.
	13	3		
	15	4	3 3 2	
	18			SILTY GRAVEL WITH SAND (GM); fine to coarse gravel, subrounded to angular, low plasticity silt, gray, moist to wet, medium dense.
	20	5	12 6 9	±230 gallons of mud loss between 20 and 23 feet.
	23			**Sand/Silt Alluvium (Qal)** SILT (ML); low to medium plasticity, gray mottled brown, moist to wet, soft.
	25	6	1 1 1	
	28	7		
	30	8	2 2 3	
	33	9		
	35	10	1 2 1	



PROJECT East Side CSO - Final Design
SHEET 2 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 drop

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
						(Qal Cont'd) SILT (ML); low to medium plasticity, gray mottled brown, moist to wet, soft.
	40	Standard Penetration Test	11	2 2 2		
-10						
	45	Standard Penetration Test	12	3 2 3		
-15						
	50	Standard Penetration Test	13	10 2 3		
-20						
	55	Standard Penetration Test	14	6 2 2		
-25						
		Shelby Tube	15			
	60	Standard Penetration Test	16	1 0 3		
-30						
	65	Standard Penetration Test	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	Ring	18	7 6 7		
-40						

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

LITHOLOGY

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

WELL

PARSONS BRINCKERHOFF

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

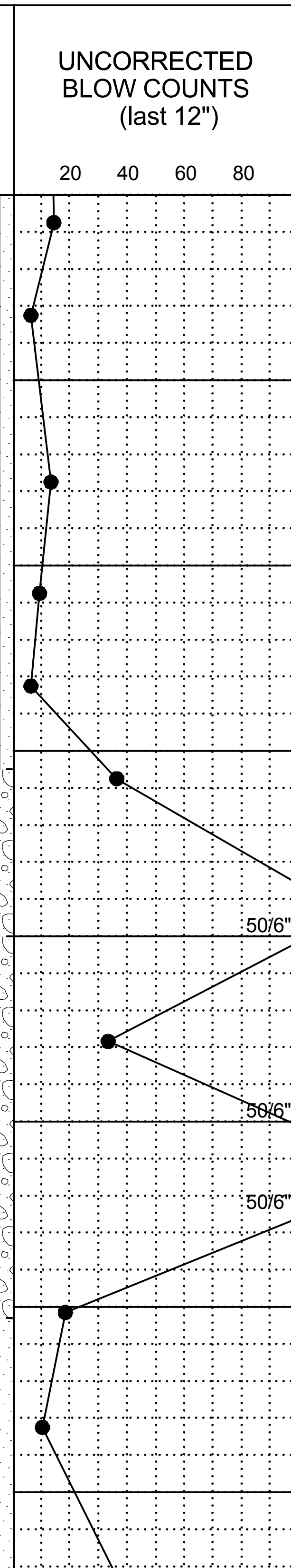
DATE DRILLED 9/9/03 LOGGED BY AR | LOCATION NE 2nd & Burnside | HAMMER SYS. 140 + 300 lb. hammer, 30 inch dro

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	SOIL DESCRIPTION
		19	3 7 7	AL MC	**(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.
-45					
		20	2 3 3		
	80	21		MC	Becomes SILT (ML); low plasticity, gray, wet, stiff.
-50					
		22	3 7 6		
	85	23	2 5 4		
-55					
		24	3 2 4	MC AL FC	
	90	25	4 13 23		SILTY GRAVEL WITH SAND (GM); some sand, fine to coarse gravel, rounded, low plasticity silt, gray to brown, wet, dense to very dense.
-60					
		26	30 65 50		
	95	27	50/6"		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.
-65					
		28	22 18 15		
	100	29	50/6"		
-70					
		30	50/6"		
	105	31A 31B	20 8 10	MC FC	SILTY SAND (SM) to SANDY SILT (ML); fine sand, nonplastic to low plasticity, brown, wet, stiff.
-75					
		32	5 5 5	MC	
	110	33		MC FC	±20 gallons of mud loss between 113 and 115 feet.
-80			11	GSD	

PERCENT FINES 20 40 60 80

PLASTIC M.C. LIQUID 20 40 60 80



LITHOLOGY

WELL

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 drop

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube | Grab Sample

Depth (ft)	Sample No	Field Blows /6"	Lab Tests	Soil Description
	34	10, 29		SILTY GRAVEL WITH SAND (GM); fine sand, fine to coarse gravel, subrounded to rounded, nonplastic to low plasticity, brown, wet, dense.
115	35	4, 6, 6	MC, FC	SANDY SILT (ML); fine sand, nonplastic, brown, wet, stiff.
	36	5, 5, 6	MC, FC	
120	37	31, 50/4"	GSD	**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.
	38	17, 14, 19		
125	39	17, 19, 26		
	40	25, 27, 33	GSD	
130	41	16, 18, 21		
	42	14, 25, 17		
134				Bottom of boring at 134 ft.

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

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

50/4"

LITHOLOGY | WELL

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

PARSONS BRINCKERHOFF

BORING LOG ES-2005C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design SHEET 1 OF 4 INITIAL GWL@ 28.8 ft ()

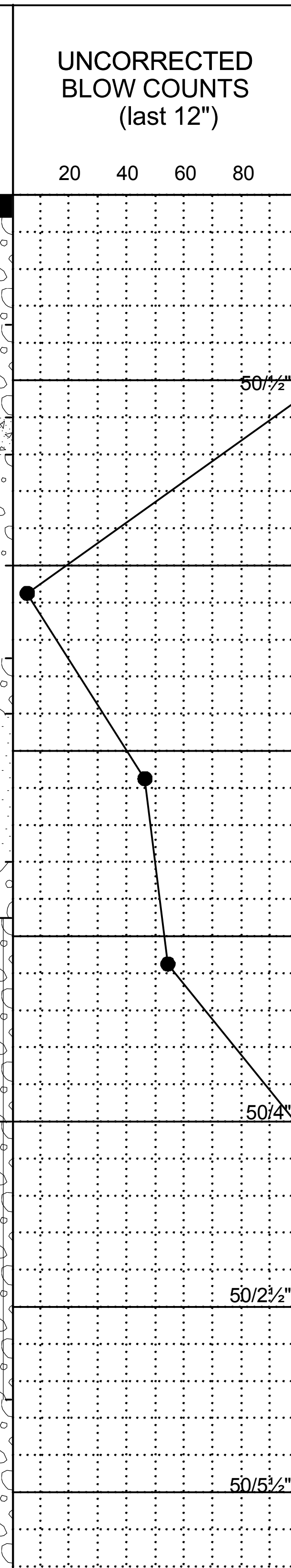
CITY Portland, Oregon SURF. EL. 43.6 ft STA. OFST. EQUIPMENT CME/B-59

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 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	SOIL DESCRIPTION
	0					**Pavement** ASPHALTIC CONCRETE OVER CONCRETE.
						Artificial Fill (Qaf) POORLY GRADED GRAVEL (GP); loose.
40						SILTY GRAVEL (GM) to GRAVELLY SILT (ML); fine to coarse gravel, angular, low plasticity, gray to brown, moist, loose gravel, soft silt.
	5	Standard Penetration Test	1	3 0 50		
						CONCRETE RUBBLE
						GRAVELLY SILT (ML)
35						
	10	Standard Penetration Test	2	3 2 3		SILT (ML); low plasticity, light brown, moist, medium stiff.
						SILTY GRAVEL (GM)
30						POORLY GRADED SAND (SP); trace to few gravel, trace silt, fine sand, fine to coarse gravel, angular, light brown, moist, dense.
	15	Standard Penetration Test	3	17 19 27		
25						BOULDER at 18.0 to 19.5 feet.
	20	Standard Penetration Test	4	28 26 28		**Gravel Alluvium (Qfc)** POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity, gray brown, wet, very dense.
20						
	25	Standard Penetration Test	5	50		**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.
15						
	30	Ring	6	50		
10						POORLY GRADED GRAVEL WITH SAND (GP); trace silt, fine to coarse gravel, subrounded, nonplastic, gray gravel, light brown sand, wet, very dense.
	35	Standard Penetration Test	7	50		



LITHOLOGY | WELL

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

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

PARSONS BRINCKERHOFF

BORING LOG
ES-2005C

Foundation Engineering, Inc.

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 drop

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")
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	Shelby Tube	8	50/3"			50/3"
0							
	45	Shelby Tube	9	50/3"			50/2½"
-5							
	50	Shelby Tube	10	50/3"		With interbedded layers of POORLY GRADED GRAVEL WITH SAND (GP)	50/3½"
-10							
	55	Ring	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							

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

PARSONS BRINCKERHOFF

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 drop

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 / PLASTIC M.C. LIQUID	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12")	WELL
			15	50/3"			**(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.		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"	

PARSONS BRINCKERHOFF 100 YEARS

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 drop

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")
-70					
	115	23	50/3"	FC MC	50/1½"
-75					
	120	24	50/3"		
-80					
	125	25	50/3"	FC MC	50/2"
-85					
	130	26	50/3"		50/½"
-90					
	135	27	50/3"		50/1"
-95					
	140	28	50/3"		50/2"
		29	50/3"		50/3"
-100					
	145				
-105					

PERCENT FINES: 20 40 60 80
PLASTIC M.C. LIQUID: 20 40 60 80
UNCORRECTED BLOW COUNTS: 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.
Becomes SILTY GRAVEL (GM).

Bottom of boring at 143 ft.

LITHOLOGY | WELL

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 dro

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube | Grab Sample

Columns: 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

DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	UNCORRECTED BLOW COUNTS
5	1	2 4 14	FC MC	
10	2	50	MC	50/3½"
15	3	12 12 16	MC	
20	4	50		50/5½"
25	5	50	MC	50/6"
30	6	50		50/4½"
35	7	50		50/5"

Elevation (ft): 45, 40, 35, 30, 25, 20, 15

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 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	SOIL DESCRIPTION	UNCORRECTED BLOW COUNTS (last 12")
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 100 YEARS

BORING LOG
ES-2006C

Foundation Engineering, Inc.

PROJECT East Side CSO - Final Design SHEET 3 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 drop

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")
		15	50		**(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.	50/2"
-30						
	80	16	50	GSD		50/5"
-35						
	85	17	50			50/2"
-40						
	90	18	50			50/3"
-45						
	95	19	50	GSD	Sand lens at 95.4 to 96.5 feet.	50/4"
					Sand lens at 97 to 97.5 feet.	
-50						
	100	20	50			50/1½
-55						
	105	21	50	GSD	Interbedded sand lenses from 105 to 110 feet.	50/1"
-60						
	110	22	50			50/2½"
					±7-inch diameter cobble at 112 feet.	

PARSONS BRINCKERHOFF

BORING LOG ES-2006C

Foundation Engineering, Inc.

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 drop

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")
-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 drop

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 drop

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")
5	1	1 3 3	GSD MC	
10	2	19 29 38	GSD	
15	3	22 50	GSD	50/4½"
20	4	50/5¾"		50/5¾"
25	5	50/4"		50/4"
30	6	50/2"		50/2"
35	7	50/2½"		50/2½"

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

△ PERCENT FINES △: 20 40 60 80

PLASTIC M.C. LIQUID: 20 40 60 80

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

LITHOLOGY | WELL

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 drop

SAMPLE TYPE: Ring (3.25" OD) | Standard Penetration Test (2" OD) | Shelby Tube | Grab Sample

Columns: 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

Elevation (ft)	DEPTH (ft)	SAMPLE NO	FIELD BLOWS /6"	LAB TESTS	SOIL DESCRIPTION	UNCORRECTED BLOW COUNTS
					(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"		Sand lens at 56 to 57 feet.	50/3½"
-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 drop

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")
		15	50/2"		**(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.	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	POORLY GRADED SAND WITH SILT (SP-SM); trace gravel, olive gray, wet, very dense.	50/4"
-65					POORLY GRADED GRAVEL WITH SILT AND SAND (GP-GM); fine to coarse gravel, subrounded, low plasticity silt, olive gray, wet, very dense, some quartzite.	
	100	20	50/3½"			50/3½"
-70						
	105	21	50/2"			50/2"
-75						
	110	22	50/½"			50/½"
-80						

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 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	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						

ESCSO MUD 2005 2032011.GPJ 2042034-REV1.GPJ 6/6/05

PARSONS BRINCKERHOFF

BORING LOG
ES-2007A

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 drop

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 / PLASTIC M.C. LIQUID (20 40 60 80)	SOIL DESCRIPTION	LITHOLOGY	UNCORRECTED BLOW COUNTS (last 12") (20 40 60 80)	WELL
		SPT	30	50/2"			**(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.		50/2"	
-120										
	155	SPT	31	50/2"					50/2"	
-125										
	160	SPT	32	50/2"					50/2"	
							Bottom of boring at 160.2 ft.			
-130										
	165									
-135										
	170									
-140										
	175									
-145										
	180									
-150										
	185									
-155										

APPENDIX B

DRILLING EXPLORATIONS

TABLE OF CONTENTS

FIGURES

APPENDIX B

DRILLING EXPLORATIONS

B.1 GENERAL

Shannon & Wilson, Inc., explored subsurface conditions at the project site 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 MATERIAL DESCRIPTIONS

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.

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

			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

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.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	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										

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
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)					
						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)				
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)					
						63.25 - 75.00 GRAVEL with cobbles; GP; Gray to dark gray; Wet; Very Dense; Fine to coarse, subangular to subrounded gravel; (Gravel 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)			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										

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
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										

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						**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						**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										
170						**169.00 - 185.75 Silty CLAY to CLAY**				

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										

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										

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SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		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

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.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	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)					
15						**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)**				
	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											

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										

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
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

		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

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.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	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)**				
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)	**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)**				
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)					
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)	**38.25 - 43.25 Sandy SILT; ML; Gray; Low plasticity; Wet; Medium Stiff; 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)	**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)**				
50						**48.25 - 63.25 Silty SAND; SM; Gray to gray-brown; Nonplastic to low**				

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										

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										

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: 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	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										

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										

APPENDIX C

IN SITU GEOPHYSICAL TESTS

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ATTACHMENTS

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

APPENDIX C

IN SITU GEOPHYSICAL TESTS

C.1 GENERAL

The field exploration program 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
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APPENDICES

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

11/28/2016

Emily Feldman   Date
Senior Staff Geophysicist
GEO*Vision* Geophysical Services

Reviewed and approved by

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

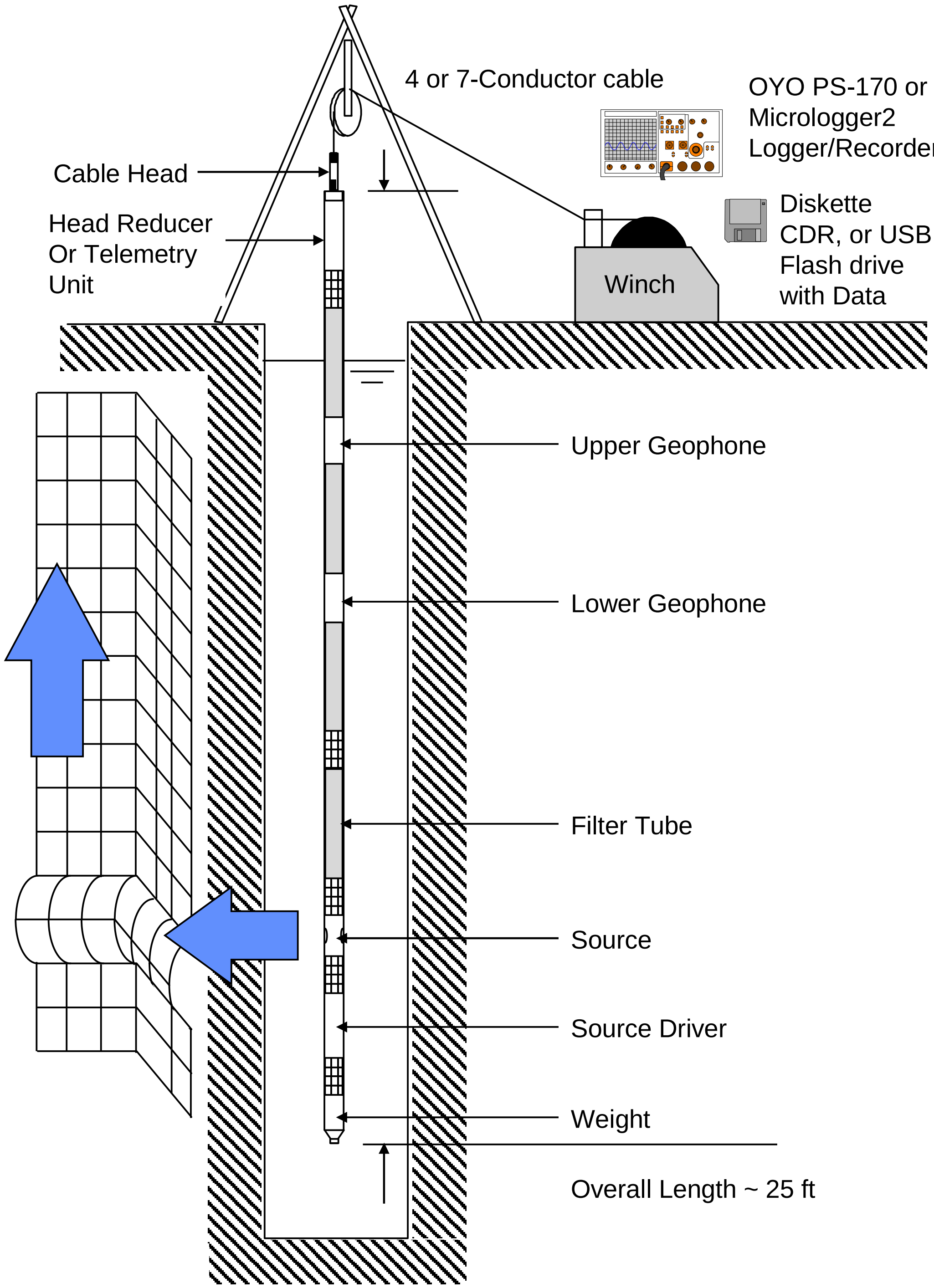


Figure 1: Concept illustration of P-S logging system

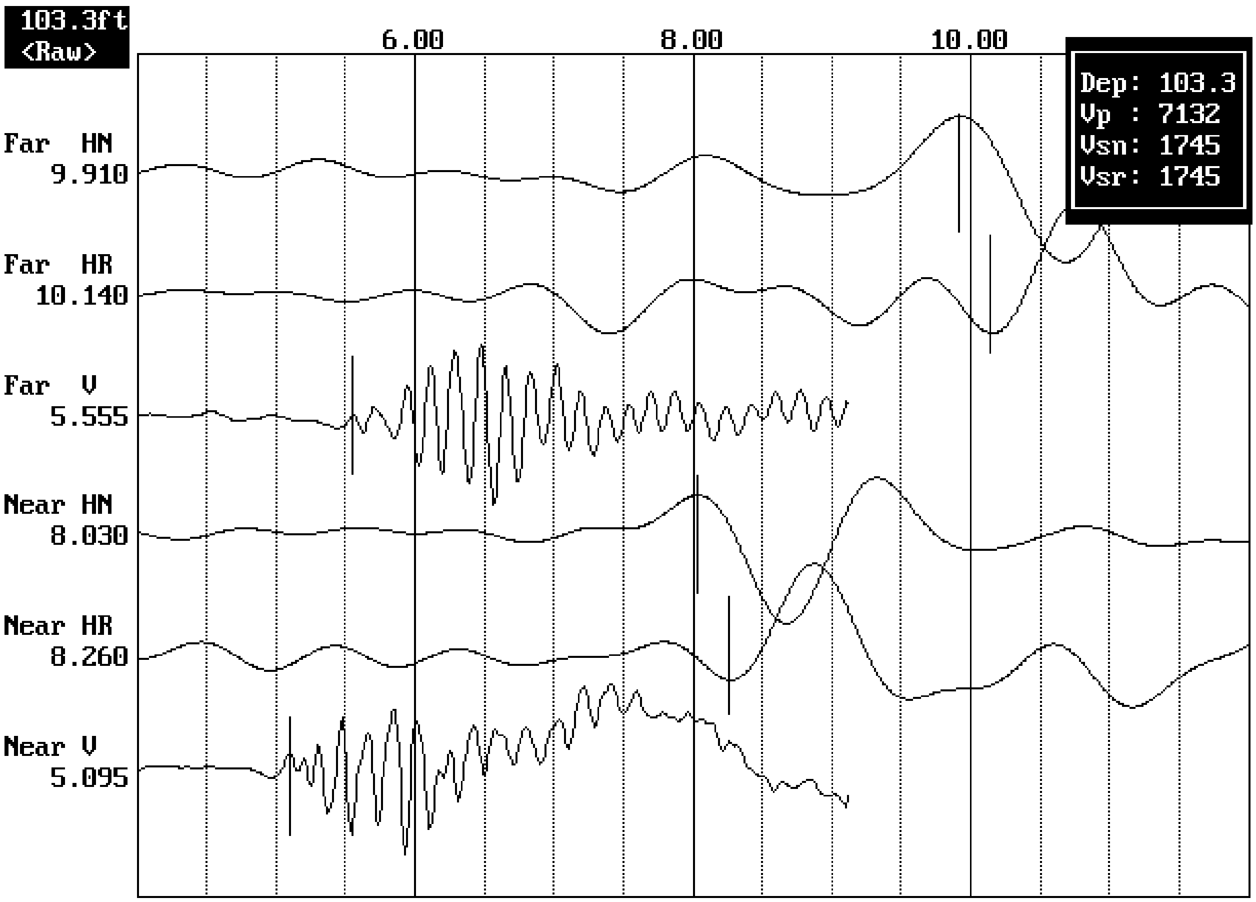


Figure 2: Example of filtered (1400 Hz lowpass) suspension record

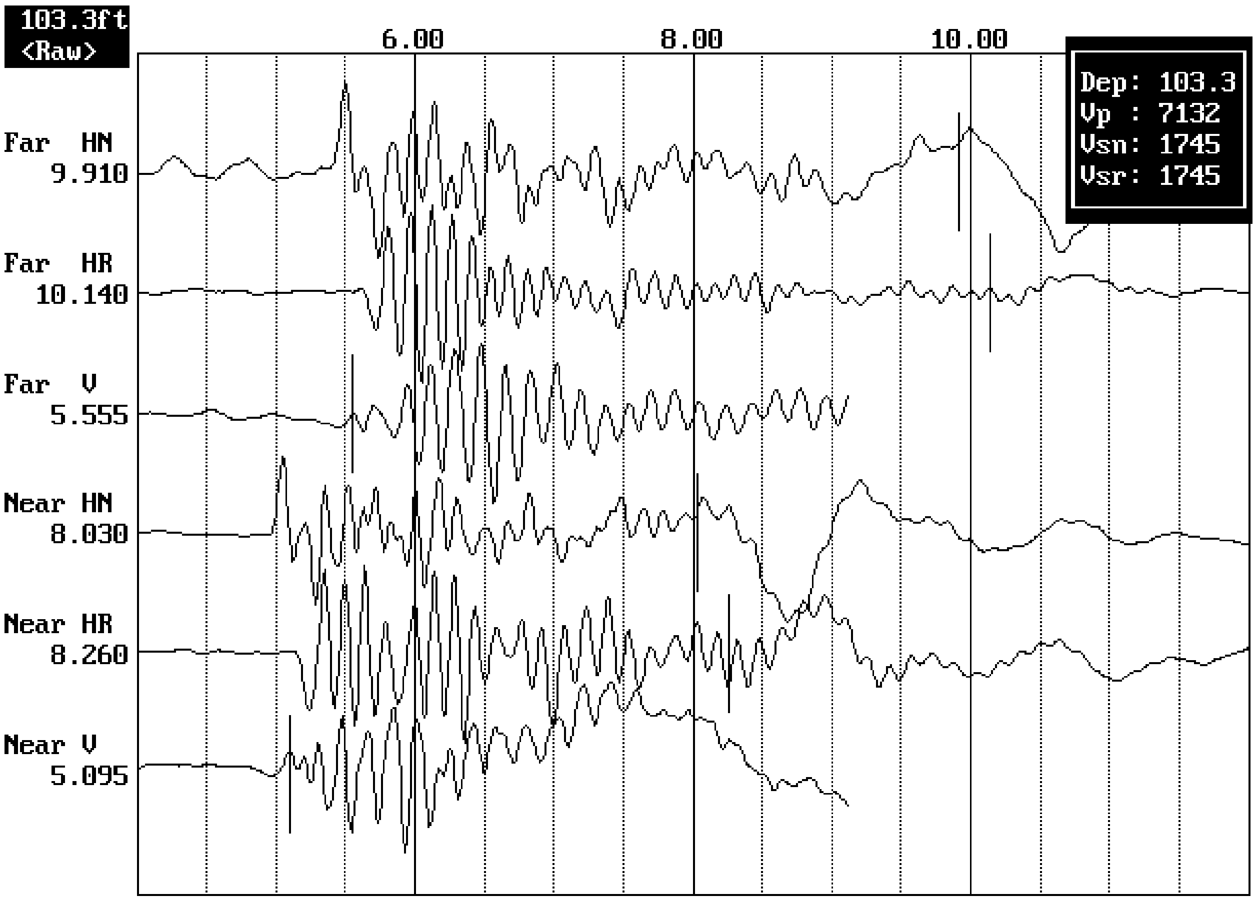


Figure 3. Example of unfiltered suspension record

BURNSIDE BRIDGE BOREHOLE B-1
Receiver to Receiver V_s and V_p Analysis

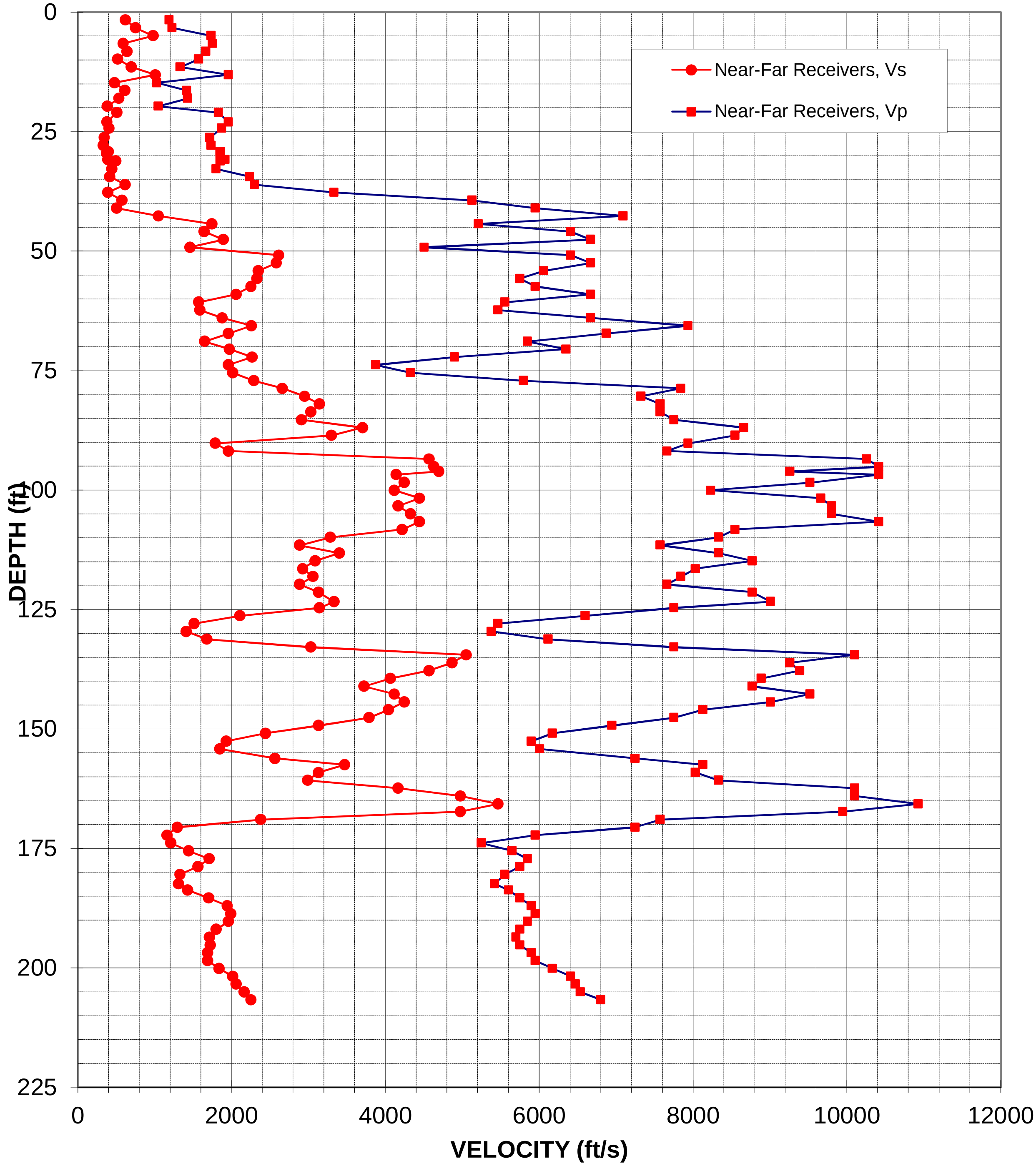


Figure 4: Borehole B-1, Suspension R1-R2 P- and S_H-wave velocities

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				Metric Units			
Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity V_s	Velocity V_p	Poisson's Ratio
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
1.6	620	1190	0.31	0.5	190	360	0.31
3.3	750	1230	0.20	1.0	230	370	0.20
4.9	980	1740	0.27	1.5	300	530	0.27
6.6	590	1750	0.44	2.0	180	530	0.44
8.2	640	1670	0.41	2.5	200	510	0.41
9.8	520	1570	0.44	3.0	160	480	0.44
11.5	700	1330	0.31	3.5	210	410	0.31
13.1	1010	1960	0.32	4.0	310	600	0.32
14.8	480	1030	0.36	4.5	150	310	0.36
16.4	610	1420	0.39	5.0	190	430	0.39
18.0	540	1430	0.42	5.5	160	440	0.42
19.7	380	1050	0.42	6.0	120	320	0.42
21.0	510	1830	0.46	6.4	160	560	0.46
23.0	380	1960	0.48	7.0	120	600	0.48
24.3	410	1870	0.48	7.4	120	570	0.48
26.3	340	1720	0.48	8.0	110	520	0.48
27.9	330	1740	0.48	8.5	100	530	0.48
29.2	400	1850	0.48	8.9	120	560	0.48
29.5	380	1850	0.48	9.0	110	560	0.48
30.8	390	1920	0.48	9.4	120	580	0.48
31.2	500	1850	0.46	9.5	150	560	0.46
32.8	440	1800	0.47	10.0	140	550	0.47
34.5	420	2240	0.48	10.5	130	680	0.48
36.1	620	2300	0.46	11.0	190	700	0.46
37.7	390	3330	0.49	11.5	120	1020	0.49
39.4	580	5130	0.49	12.0	180	1560	0.49
41.0	510	5950	0.50	12.5	150	1810	0.50
42.7	1050	7090	0.49	13.0	320	2160	0.49
44.3	1750	5210	0.44	13.5	530	1590	0.44
45.9	1650	6410	0.46	14.0	500	1950	0.46
47.6	1890	6670	0.46	14.5	580	2030	0.46
49.2	1460	4500	0.44	15.0	450	1370	0.44
50.9	2610	6410	0.40	15.5	800	1950	0.40
52.5	2580	6670	0.41	16.0	790	2030	0.41
54.1	2350	6060	0.41	16.5	720	1850	0.41
55.8	2330	5750	0.40	17.0	710	1750	0.40
57.4	2250	5950	0.42	17.5	690	1810	0.42
59.1	2060	6670	0.45	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
60.7	1570	5560	0.46	18.5	480	1690	0.46
62.3	1590	5460	0.45	19.0	480	1670	0.45
64.0	1880	6670	0.46	19.5	570	2030	0.46
65.6	2260	7940	0.46	20.0	690	2420	0.46
67.3	1960	6870	0.46	20.5	600	2090	0.46
68.9	1650	5850	0.46	21.0	500	1780	0.46
70.5	1970	6350	0.45	21.5	600	1940	0.45
72.2	2270	4900	0.36	22.0	690	1490	0.36
73.8	1960	3880	0.33	22.5	600	1180	0.33
75.5	2010	4330	0.36	23.0	610	1320	0.36
77.1	2290	5800	0.41	23.5	700	1770	0.41
78.7	2660	7840	0.43	24.0	810	2390	0.43
80.4	2950	7330	0.40	24.5	900	2230	0.40
82.0	3140	7580	0.40	25.0	960	2310	0.40
83.7	3030	7580	0.40	25.5	920	2310	0.40
85.3	2910	7750	0.42	26.0	890	2360	0.42
86.9	3700	8660	0.39	26.5	1130	2640	0.39
88.6	3300	8550	0.41	27.0	1010	2610	0.41
90.2	1790	7940	0.47	27.5	540	2420	0.47
91.9	1960	7660	0.46	28.0	600	2340	0.46
93.5	4570	10260	0.38	28.5	1390	3130	0.38
95.1	4630	10420	0.38	29.0	1410	3180	0.38
96.1	4690	9260	0.33	29.3	1430	2820	0.33
96.8	4140	10420	0.41	29.5	1260	3180	0.41
98.4	4250	9520	0.38	30.0	1290	2900	0.38
100.1	4120	8230	0.33	30.5	1250	2510	0.33
101.7	4440	9660	0.37	31.0	1350	2940	0.37
103.4	4170	9800	0.39	31.5	1270	2990	0.39
105.0	4330	9800	0.38	32.0	1320	2990	0.38
106.6	4440	10420	0.39	32.5	1350	3180	0.39
108.3	4220	8550	0.34	33.0	1290	2610	0.34
109.9	3280	8330	0.41	33.5	1000	2540	0.41
111.6	2890	7580	0.42	34.0	880	2310	0.42
113.2	3400	8330	0.40	34.5	1040	2540	0.40
114.8	3090	8770	0.43	35.0	940	2670	0.43
116.5	2920	8030	0.42	35.5	890	2450	0.42
118.1	3060	7840	0.41	36.0	930	2390	0.41
119.8	2890	7660	0.42	36.5	880	2340	0.42
121.4	3130	8770	0.43	37.0	950	2670	0.43
123.4	3330	9010	0.42	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
124.7	3140	7750	0.40	38.0	960	2360	0.40
126.3	2110	6600	0.44	38.5	640	2010	0.44
128.0	1520	5460	0.46	39.0	460	1670	0.46
129.6	1410	5380	0.46	39.5	430	1640	0.46
131.2	1680	6120	0.46	40.0	510	1860	0.46
132.9	3030	7750	0.41	40.5	920	2360	0.41
134.5	5050	10100	0.33	41.0	1540	3080	0.33
136.2	4870	9260	0.31	41.5	1480	2820	0.31
137.8	4570	9390	0.35	42.0	1390	2860	0.35
139.4	4070	8890	0.37	42.5	1240	2710	0.37
141.1	3720	8770	0.39	43.0	1140	2670	0.39
142.7	4120	9520	0.39	43.5	1250	2900	0.39
144.4	4250	9010	0.36	44.0	1290	2750	0.36
146.0	4040	8130	0.34	44.5	1230	2480	0.34
147.6	3790	7750	0.34	45.0	1150	2360	0.34
149.3	3130	6940	0.37	45.5	950	2120	0.37
150.9	2440	6170	0.41	46.0	740	1880	0.41
152.6	1930	5900	0.44	46.5	590	1800	0.44
154.2	1850	6010	0.45	47.0	560	1830	0.45
156.2	2560	7250	0.43	47.6	780	2210	0.43
157.5	3470	8130	0.39	48.0	1060	2480	0.39
159.1	3130	8030	0.41	48.5	950	2450	0.41
160.8	2990	8330	0.43	49.0	910	2540	0.43
162.4	4170	10100	0.40	49.5	1270	3080	0.40
164.0	4980	10100	0.34	50.0	1520	3080	0.34
165.7	5460	10930	0.33	50.5	1670	3330	0.33
167.3	4980	9950	0.33	51.0	1520	3030	0.33
169.0	2380	7580	0.45	51.5	730	2310	0.45
170.6	1290	7250	0.48	52.0	390	2210	0.48
172.2	1160	5950	0.48	52.5	350	1810	0.48
173.9	1210	5250	0.47	53.0	370	1600	0.47
175.5	1440	5650	0.47	53.5	440	1720	0.47
177.2	1710	5850	0.45	54.0	520	1780	0.45
178.8	1560	5750	0.46	54.5	480	1750	0.46
180.5	1330	5560	0.47	55.0	410	1690	0.47
182.4	1310	5420	0.47	55.6	400	1650	0.47
183.7	1430	5600	0.47	56.0	440	1710	0.47
185.4	1710	5750	0.45	56.5	520	1750	0.45
187.0	1940	5900	0.44	57.0	590	1800	0.44
188.7	1990	5950	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
190.3	1960	5850	0.44	58.0	600	1780	0.44
191.9	1800	5750	0.45	58.5	550	1750	0.45
193.6	1710	5700	0.45	59.0	520	1740	0.45
195.2	1720	5750	0.45	59.5	530	1750	0.45
196.9	1690	5900	0.46	60.0	510	1800	0.46
198.5	1690	5950	0.46	60.5	510	1810	0.46
200.1	1840	6170	0.45	61.0	560	1880	0.45
201.8	2010	6410	0.45	61.5	610	1950	0.45
203.4	2060	6470	0.44	62.0	630	1970	0.44
205.1	2160	6540	0.44	62.5	660	1990	0.44
206.7	2250	6800	0.44	63.0	690	2070	0.44

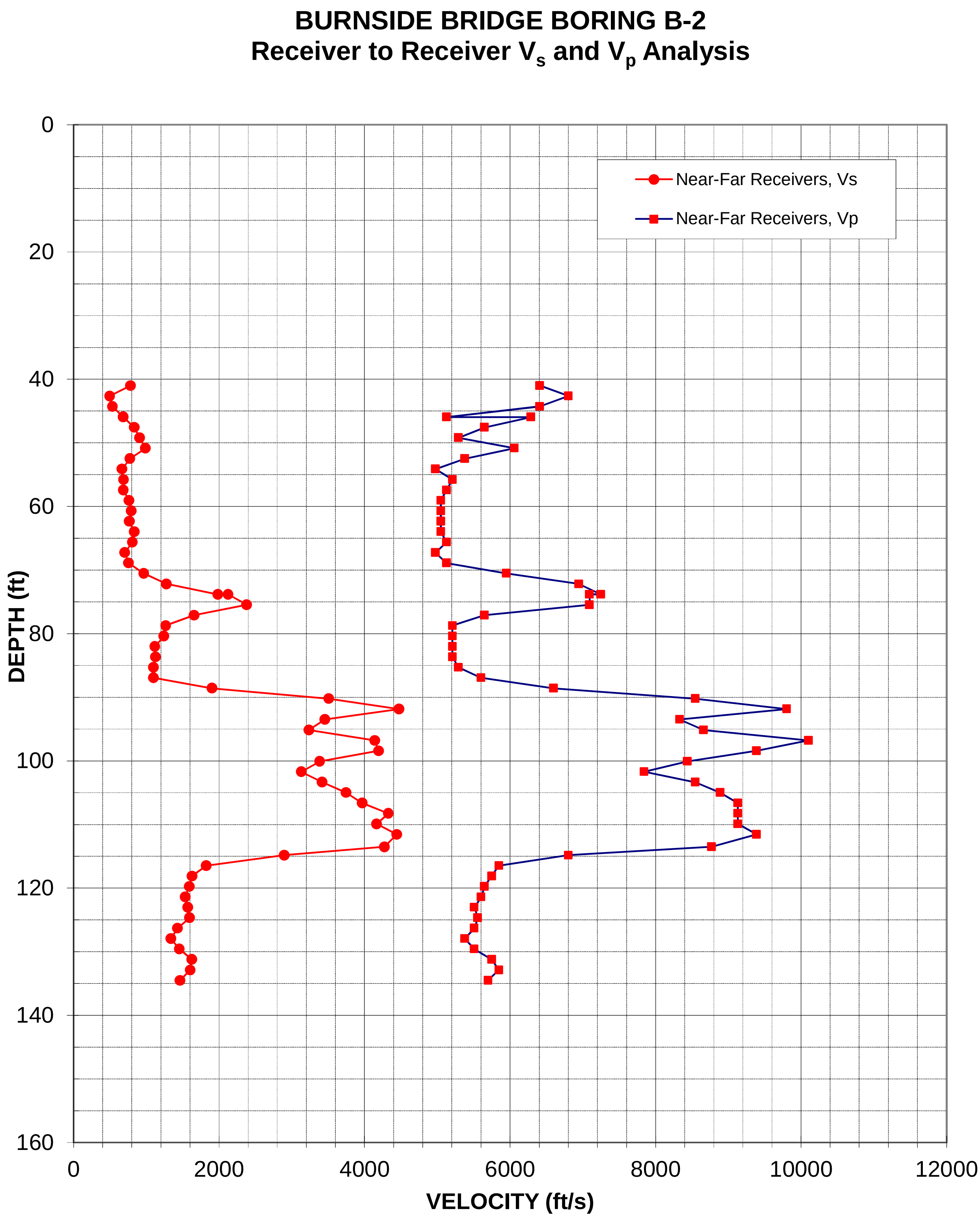


Figure 5: Borehole B-2, Suspension R1-R2 P- and SH-wave velocities

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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
41.0	780	6410	0.49	12.5	240	1950	0.49
42.7	500	6800	0.50	13.0	150	2070	0.50
44.3	540	6410	0.50	13.5	160	1950	0.50
45.9	680	5130	0.49	14.0	210	1560	0.49
45.9	680	6290	0.49	14.0	210	1920	0.49
47.6	840	5650	0.49	14.5	250	1720	0.49
49.2	910	5290	0.48	15.0	280	1610	0.48
50.9	990	6060	0.49	15.5	300	1850	0.49
52.5	780	5380	0.49	16.0	240	1640	0.49
54.1	670	4980	0.49	16.5	200	1520	0.49
55.8	690	5210	0.49	17.0	210	1590	0.49
57.4	680	5130	0.49	17.5	210	1560	0.49
59.1	760	5050	0.49	18.0	230	1540	0.49
60.7	790	5050	0.49	18.5	240	1540	0.49
62.3	770	5050	0.49	19.0	230	1540	0.49
62.3	770	5050	0.49	19.0	230	1540	0.49
64.0	830	5050	0.49	19.5	250	1540	0.49
65.6	810	5130	0.49	20.0	250	1560	0.49
67.3	700	4980	0.49	20.5	210	1520	0.49
68.9	760	5130	0.49	21.0	230	1560	0.49
70.5	970	5950	0.49	21.5	290	1810	0.49
72.2	1280	6940	0.48	22.0	390	2120	0.48
73.8	1980	7250	0.46	22.5	600	2210	0.46
73.8	2120	7090	0.45	22.5	650	2160	0.45
75.5	2380	7090	0.44	23.0	730	2160	0.44
77.1	1660	5650	0.45	23.5	510	1720	0.45
78.7	1270	5210	0.47	24.0	390	1590	0.47
80.4	1240	5210	0.47	24.5	380	1590	0.47
82.0	1120	5210	0.48	25.0	340	1590	0.48
83.7	1130	5210	0.48	25.5	340	1590	0.48
85.3	1100	5290	0.48	26.0	340	1610	0.48
86.9	1100	5600	0.48	26.5	330	1710	0.48
88.6	1900	6600	0.45	27.0	580	2010	0.45
90.2	3510	8550	0.40	27.5	1070	2610	0.40
91.9	4470	9800	0.37	28.0	1360	2990	0.37
93.5	3450	8330	0.40	28.5	1050	2540	0.40
95.1	3240	8660	0.42	29.0	990	2640	0.42
96.8	4140	10100	0.40	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
98.4	4190	9390	0.38	30.0	1280	2860	0.38
100.1	3380	8440	0.40	30.5	1030	2570	0.40
101.7	3130	7840	0.41	31.0	950	2390	0.41
103.4	3420	8550	0.40	31.5	1040	2610	0.40
105.0	3750	8890	0.39	32.0	1140	2710	0.39
106.6	3970	9130	0.38	32.5	1210	2780	0.38
108.3	4330	9130	0.36	33.0	1320	2780	0.36
109.9	4170	9130	0.37	33.5	1270	2780	0.37
111.6	4440	9390	0.36	34.0	1350	2860	0.36
113.5	4270	8770	0.34	34.6	1300	2670	0.34
114.8	2900	6800	0.39	35.0	880	2070	0.39
116.5	1830	5850	0.45	35.5	560	1780	0.45
118.1	1630	5750	0.46	36.0	500	1750	0.46
119.8	1590	5650	0.46	36.5	480	1720	0.46
121.4	1540	5600	0.46	37.0	470	1710	0.46
123.0	1570	5510	0.46	37.5	480	1680	0.46
124.7	1590	5560	0.46	38.0	490	1690	0.46
126.3	1430	5510	0.46	38.5	440	1680	0.46
128.0	1340	5380	0.47	39.0	410	1640	0.47
129.6	1460	5510	0.46	39.5	440	1680	0.46
131.2	1630	5750	0.46	40.0	500	1750	0.46
132.9	1600	5850	0.46	40.5	490	1780	0.46
134.5	1470	5700	0.46	41.0	450	1740	0.46

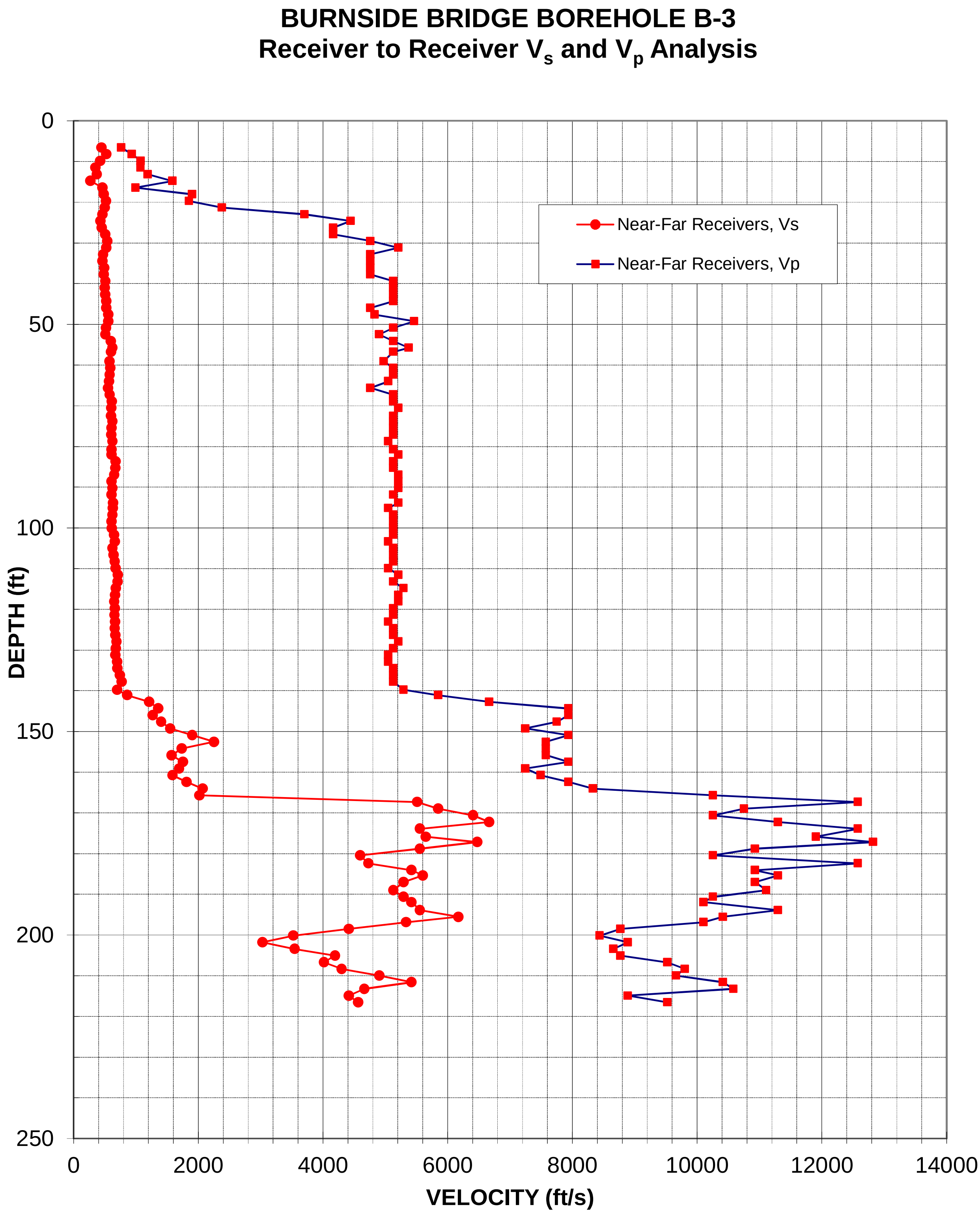


Figure 6: Borehole B-3, Suspension R1-R2 P- and SH-wave velocities

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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
6.6	450	760	0.23	2.0	140	230	0.23
8.2	520	940	0.27	2.5	160	290	0.27
9.8	430	1080	0.41	3.0	130	330	0.41
11.5	350	1080	0.44	3.5	110	330	0.44
13.1	370	1190	0.45	4.0	110	360	0.45
14.8	270	1590	0.49	4.5	80	480	0.49
16.4	460	1000	0.36	5.0	140	300	0.36
18.0	480	1900	0.47	5.5	150	580	0.47
19.7	520	1850	0.46	6.0	160	560	0.46
21.3	500	2380	0.48	6.5	150	730	0.48
23.0	460	3700	0.49	7.0	140	1130	0.49
24.6	430	4440	0.50	7.5	130	1350	0.50
26.3	450	4170	0.49	8.0	140	1270	0.49
27.9	510	4170	0.49	8.5	150	1270	0.49
29.5	540	4760	0.49	9.0	160	1450	0.49
31.2	530	5210	0.49	9.5	160	1590	0.49
32.8	470	4760	0.49	10.0	140	1450	0.49
34.5	460	4760	0.50	10.5	140	1450	0.50
36.1	490	4760	0.49	11.0	150	1450	0.49
37.7	490	4760	0.49	11.5	150	1450	0.49
39.4	510	5130	0.49	12.0	160	1560	0.49
41.0	500	5130	0.50	12.5	150	1560	0.50
42.7	510	5130	0.50	13.0	160	1560	0.50
44.3	530	5130	0.49	13.5	160	1560	0.49
45.9	520	4760	0.49	14.0	160	1450	0.49
47.6	560	4830	0.49	14.5	170	1470	0.49
49.2	560	5460	0.49	15.0	170	1670	0.49
50.9	520	5130	0.49	15.5	160	1560	0.49
52.5	510	4900	0.49	16.0	160	1490	0.49
54.1	600	5130	0.49	16.5	180	1560	0.49
55.8	630	5380	0.49	17.0	190	1640	0.49
56.8	600	5130	0.49	17.3	180	1560	0.49
59.1	580	4980	0.49	18.0	180	1520	0.49
60.7	590	5130	0.49	18.5	180	1560	0.49
62.3	580	5130	0.49	19.0	180	1560	0.49
64.0	570	5050	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
65.6	560	4760	0.49	20.0	170	1450	0.49
67.3	580	5130	0.49	20.5	180	1560	0.49
68.9	610	5130	0.49	21.0	190	1560	0.49
70.5	610	5210	0.49	21.5	180	1590	0.49
72.5	600	5130	0.49	22.1	180	1560	0.49
73.8	620	5130	0.49	22.5	190	1560	0.49
75.5	610	5130	0.49	23.0	190	1560	0.49
77.1	610	5130	0.49	23.5	180	1560	0.49
78.7	620	5050	0.49	24.0	190	1540	0.49
80.7	610	5130	0.49	24.6	190	1560	0.49
82.0	610	5210	0.49	25.0	190	1590	0.49
83.7	680	5130	0.49	25.5	210	1560	0.49
85.3	670	5130	0.49	26.0	210	1560	0.49
86.9	650	5210	0.49	26.5	200	1590	0.49
88.6	610	5210	0.49	27.0	190	1590	0.49
90.2	630	5210	0.49	27.5	190	1590	0.49
91.9	610	5130	0.49	28.0	190	1560	0.49
93.8	630	5210	0.49	28.6	190	1590	0.49
95.1	630	5050	0.49	29.0	190	1540	0.49
96.8	620	5130	0.49	29.5	190	1560	0.49
98.4	610	5130	0.49	30.0	190	1560	0.49
100.1	610	5130	0.49	30.5	190	1560	0.49
101.7	650	5130	0.49	31.0	200	1560	0.49
103.4	660	5050	0.49	31.5	200	1540	0.49
105.0	630	5130	0.49	32.0	190	1560	0.49
106.6	640	5130	0.49	32.5	200	1560	0.49
108.3	660	5130	0.49	33.0	200	1560	0.49
109.9	680	5050	0.49	33.5	210	1540	0.49
111.6	710	5210	0.49	34.0	220	1590	0.49
113.2	710	5130	0.49	34.5	220	1560	0.49
114.8	680	5290	0.49	35.0	210	1610	0.49
116.5	670	5210	0.49	35.5	200	1590	0.49
118.1	650	5210	0.49	36.0	200	1590	0.49
119.8	660	5130	0.49	36.5	200	1560	0.49
121.4	660	5130	0.49	37.0	200	1560	0.49
123.0	670	5050	0.49	37.5	200	1540	0.49
124.7	660	5130	0.49	38.0	200	1560	0.49
126.3	670	5130	0.49	38.5	210	1560	0.49
128.0	690	5210	0.49	39.0	210	1590	0.49
129.6	680	5130	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
131.2	670	5050	0.49	40.0	210	1540	0.49
132.9	700	5050	0.49	40.5	210	1540	0.49
134.5	710	5130	0.49	41.0	220	1560	0.49
136.2	740	5130	0.49	41.5	230	1560	0.49
137.8	780	5130	0.49	42.0	240	1560	0.49
139.8	700	5290	0.49	42.6	210	1610	0.49
141.1	860	5850	0.49	43.0	260	1780	0.49
142.7	1210	6670	0.48	43.5	370	2030	0.48
144.4	1360	7940	0.48	44.0	410	2420	0.48
146.0	1270	7940	0.49	44.5	390	2420	0.49
147.6	1410	7750	0.48	45.0	430	2360	0.48
149.3	1550	7250	0.48	45.5	470	2210	0.48
150.9	1900	7940	0.47	46.0	580	2420	0.47
152.6	2250	7580	0.45	46.5	690	2310	0.45
154.2	1740	7580	0.47	47.0	530	2310	0.47
155.8	1570	7580	0.48	47.5	480	2310	0.48
157.5	1750	7940	0.47	48.0	530	2420	0.47
159.1	1690	7250	0.47	48.5	520	2210	0.47
160.8	1590	7490	0.48	49.0	480	2280	0.48
162.4	1810	7940	0.47	49.5	550	2420	0.47
164.0	2070	8330	0.47	50.0	630	2540	0.47
165.7	2020	10260	0.48	50.5	620	3130	0.48
167.3	5510	12580	0.38	51.0	1680	3830	0.38
169.0	5850	10750	0.29	51.5	1780	3280	0.29
170.6	6410	10260	0.18	52.0	1950	3130	0.18
172.2	6670	11300	0.23	52.5	2030	3440	0.23
173.9	5560	12580	0.38	53.0	1690	3830	0.38
175.9	5650	11900	0.35	53.6	1720	3630	0.35
177.2	6470	12820	0.33	54.0	1970	3910	0.33
178.8	5560	10930	0.33	54.5	1690	3330	0.33
180.5	4600	10260	0.37	55.0	1400	3130	0.37
182.4	4730	12580	0.42	55.6	1440	3830	0.42
184.1	5420	10930	0.34	56.1	1650	3330	0.34
185.4	5600	11300	0.34	56.5	1710	3440	0.34
187.0	5290	10930	0.35	57.0	1610	3330	0.35
189.0	5130	11110	0.36	57.6	1560	3390	0.36
190.6	5290	10260	0.32	58.1	1610	3130	0.32
191.9	5420	10100	0.30	58.5	1650	3080	0.30
193.9	5560	11300	0.34	59.1	1690	3440	0.34
195.5	6170	10420	0.23	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				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V_s	V_p			V_s	V_p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
196.9	5330	10100	0.31	60.0	1630	3080	0.31
198.5	4420	8770	0.33	60.5	1350	2670	0.33
200.1	3530	8440	0.39	61.0	1080	2570	0.39
201.8	3030	8890	0.43	61.5	920	2710	0.43
203.4	3550	8660	0.40	62.0	1080	2640	0.40
205.1	4190	8770	0.35	62.5	1280	2670	0.35
206.7	4020	9520	0.39	63.0	1220	2900	0.39
208.3	4300	9800	0.38	63.5	1310	2990	0.38
210.0	4900	9660	0.33	64.0	1490	2940	0.33
211.6	5420	10420	0.31	64.5	1650	3180	0.31
213.3	4660	10580	0.38	65.0	1420	3230	0.38
214.9	4420	8890	0.34	65.5	1350	2710	0.34
216.5	4570	9520	0.35	66.0	1390	2900	0.35

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

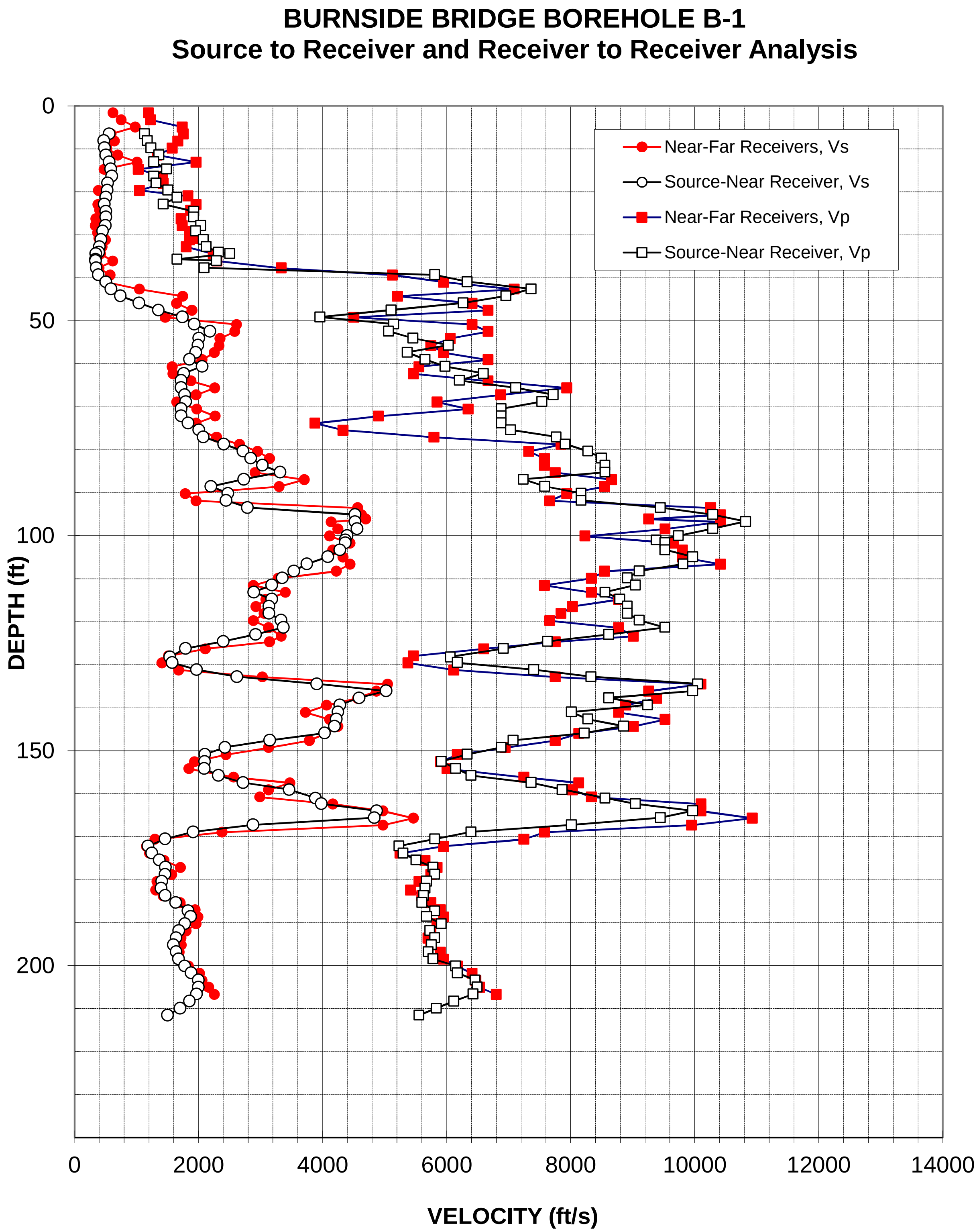


Figure A-1: Borehole B-1, Suspension S-R1 P- and S_H-wave velocities

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				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)	
6.5	560	1130	0.34	2.0	170	340	0.34
8.1	470	1170	0.40	2.5	140	360	0.40
9.8	480	1230	0.41	3.0	150	380	0.41
11.4	500	1360	0.42	3.5	150	410	0.42
13.0	560	1280	0.38	4.0	170	390	0.38
14.7	580	1490	0.41	4.5	180	450	0.41
16.3	600	1280	0.36	5.0	180	390	0.36
18.0	540	1320	0.40	5.5	160	400	0.40
19.6	530	1500	0.43	6.0	160	460	0.43
21.2	500	1650	0.45	6.5	150	500	0.45
22.9	480	1430	0.44	7.0	150	440	0.44
24.5	500	1920	0.46	7.5	150	590	0.46
25.8	510	1920	0.46	7.9	150	580	0.46
27.8	500	2040	0.47	8.5	150	620	0.47
29.1	450	1950	0.47	8.9	140	600	0.47
31.1	430	2080	0.48	9.5	130	630	0.48
32.7	400	2120	0.48	10.0	120	650	0.48
34.0	390	2320	0.49	10.4	120	710	0.49
34.4	340	2500	0.49	10.5	100	760	0.49
35.7	340	1650	0.48	10.9	100	500	0.48
36.0	340	2290	0.49	11.0	100	700	0.49
37.6	350	2090	0.49	11.5	110	640	0.49
39.3	380	5810	0.50	12.0	120	1770	0.50
40.9	500	6330	0.50	12.5	150	1930	0.50
42.6	590	7360	0.50	13.0	180	2240	0.50
44.2	740	6960	0.49	13.5	230	2120	0.49
45.8	1040	6270	0.49	14.0	320	1910	0.49
47.5	1350	5100	0.46	14.5	410	1560	0.46
49.1	1740	3960	0.38	15.0	530	1210	0.38
50.8	1930	5150	0.42	15.5	590	1570	0.42
52.4	2180	5060	0.39	16.0	670	1540	0.39
54.0	2000	5460	0.42	16.5	610	1660	0.42
55.7	1990	6030	0.44	17.0	610	1840	0.44
57.3	1950	5360	0.42	17.5	600	1640	0.42
59.0	1850	5650	0.44	18.0	560	1720	0.44
60.6	2060	5970	0.43	18.5	630	1820	0.43
62.2	1760	6590	0.46	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				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity V_s	Velocity V_p	Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity V_s	Velocity V_p	Poisson's Ratio
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
63.9	1720	6210	0.46	19.5	520	1890	0.46
65.5	1720	7110	0.47	20.0	520	2170	0.47
67.2	1770	7720	0.47	20.5	540	2350	0.47
68.8	1790	7540	0.47	21.0	550	2300	0.47
70.5	1720	6880	0.47	21.5	520	2100	0.47
72.1	1720	6880	0.47	22.0	520	2100	0.47
73.7	1830	6880	0.46	22.5	560	2100	0.46
75.4	2000	7030	0.46	23.0	610	2140	0.46
77.0	2080	7770	0.46	23.5	630	2370	0.46
78.7	2410	7910	0.45	24.0	730	2410	0.45
80.3	2720	8270	0.44	24.5	830	2520	0.44
81.9	2840	8500	0.44	25.0	870	2590	0.44
83.6	3030	8550	0.43	25.5	920	2610	0.43
85.2	3310	8550	0.41	26.0	1010	2610	0.41
86.9	2730	7230	0.42	26.5	830	2210	0.42
88.5	2200	7580	0.45	27.0	670	2310	0.45
90.1	2470	8170	0.45	27.5	750	2490	0.45
91.8	2440	8170	0.45	28.0	740	2490	0.45
93.4	2790	9450	0.45	28.5	850	2880	0.45
95.1	4520	10290	0.38	29.0	1380	3140	0.38
96.7	4520	10820	0.39	29.5	1380	3300	0.39
98.3	4550	10290	0.38	30.0	1390	3140	0.38
100.0	4400	9740	0.37	30.5	1340	2970	0.37
101.0	4370	9380	0.36	30.8	1330	2860	0.36
101.6	4370	9520	0.37	31.0	1330	2900	0.37
103.3	4280	9520	0.37	31.5	1300	2900	0.37
104.9	4080	9970	0.40	32.0	1240	3040	0.40
106.5	3750	9810	0.41	32.5	1140	2990	0.41
108.2	3540	9110	0.41	33.0	1080	2780	0.41
109.8	3350	8920	0.42	33.5	1020	2720	0.42
111.5	3180	9040	0.43	34.0	970	2760	0.43
113.1	2890	8550	0.44	34.5	880	2610	0.44
114.7	3180	8790	0.42	35.0	970	2680	0.42
116.4	3130	8920	0.43	35.5	960	2720	0.43
118.0	3130	8920	0.43	36.0	960	2720	0.43
119.7	3330	9110	0.42	36.5	1020	2780	0.42
121.3	3370	9520	0.43	37.0	1030	2900	0.43
122.9	2920	8610	0.44	37.5	890	2630	0.44
124.6	2400	7630	0.45	38.0	730	2320	0.45
126.2	1790	6920	0.46	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				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)	
128.2	1540	6060	0.47	39.1	470	1850	0.47
129.5	1570	6180	0.47	39.5	480	1880	0.47
131.1	1970	7400	0.46	40.0	600	2260	0.46
132.8	2620	8330	0.45	40.5	800	2540	0.45
134.4	3910	10050	0.41	41.0	1190	3060	0.41
136.1	5020	9970	0.33	41.5	1530	3040	0.33
137.7	4590	8610	0.30	42.0	1400	2630	0.30
139.3	4280	9240	0.36	42.5	1300	2820	0.36
141.0	4250	8010	0.30	43.0	1290	2440	0.30
142.6	4220	8270	0.32	43.5	1290	2520	0.32
144.3	4190	8850	0.36	44.0	1280	2700	0.36
145.9	4030	8220	0.34	44.5	1230	2510	0.34
147.6	3150	7070	0.38	45.0	960	2160	0.38
149.2	2430	6880	0.43	45.5	740	2100	0.43
150.8	2100	6330	0.44	46.0	640	1930	0.44
152.5	2100	5920	0.43	46.5	640	1800	0.43
154.1	2090	6150	0.43	47.0	640	1870	0.43
155.8	2320	6390	0.42	47.5	710	1950	0.42
157.4	2720	7360	0.42	48.0	830	2240	0.42
159.0	3460	7860	0.38	48.5	1050	2400	0.38
161.0	3880	8550	0.37	49.1	1180	2610	0.37
162.3	3980	9040	0.38	49.5	1210	2760	0.38
164.0	4870	9970	0.34	50.0	1480	3040	0.34
165.6	4830	9450	0.32	50.5	1470	2880	0.32
167.2	2880	8010	0.43	51.0	880	2440	0.43
168.9	1910	6390	0.45	51.5	580	1950	0.45
170.5	1460	5810	0.47	52.0	440	1770	0.47
172.2	1190	5230	0.47	52.5	360	1590	0.47
173.8	1250	5300	0.47	53.0	380	1610	0.47
175.4	1370	5500	0.47	53.5	420	1680	0.47
177.1	1470	5780	0.47	54.0	450	1760	0.47
178.7	1460	5810	0.47	54.5	440	1770	0.47
180.4	1410	5680	0.47	55.0	430	1730	0.47
182.0	1390	5650	0.47	55.5	420	1720	0.47
183.6	1460	5630	0.46	56.0	450	1720	0.46
185.3	1630	5600	0.45	56.5	500	1710	0.45
187.2	1830	5810	0.44	57.1	560	1770	0.44
188.6	1870	5680	0.44	57.5	570	1730	0.44
190.2	1780	5920	0.45	58.0	540	1800	0.45
191.8	1680	5730	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				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)	
193.5	1640	5810	0.46	59.0	500	1770	0.46
195.1	1590	5750	0.46	59.5	490	1750	0.46
196.8	1640	5700	0.46	60.0	500	1740	0.46
198.4	1680	5780	0.45	60.5	510	1760	0.45
200.0	1770	6150	0.45	61.0	540	1870	0.45
201.7	1880	6180	0.45	61.5	570	1880	0.45
203.3	2000	6460	0.45	62.0	610	1970	0.45
205.0	2000	6490	0.45	62.5	610	1980	0.45
206.6	1970	6430	0.45	63.0	600	1960	0.45
208.2	1850	6120	0.45	63.5	560	1860	0.45
209.9	1700	5830	0.45	64.0	520	1780	0.45
211.5	1500	5550	0.46	64.5	460	1690	0.46

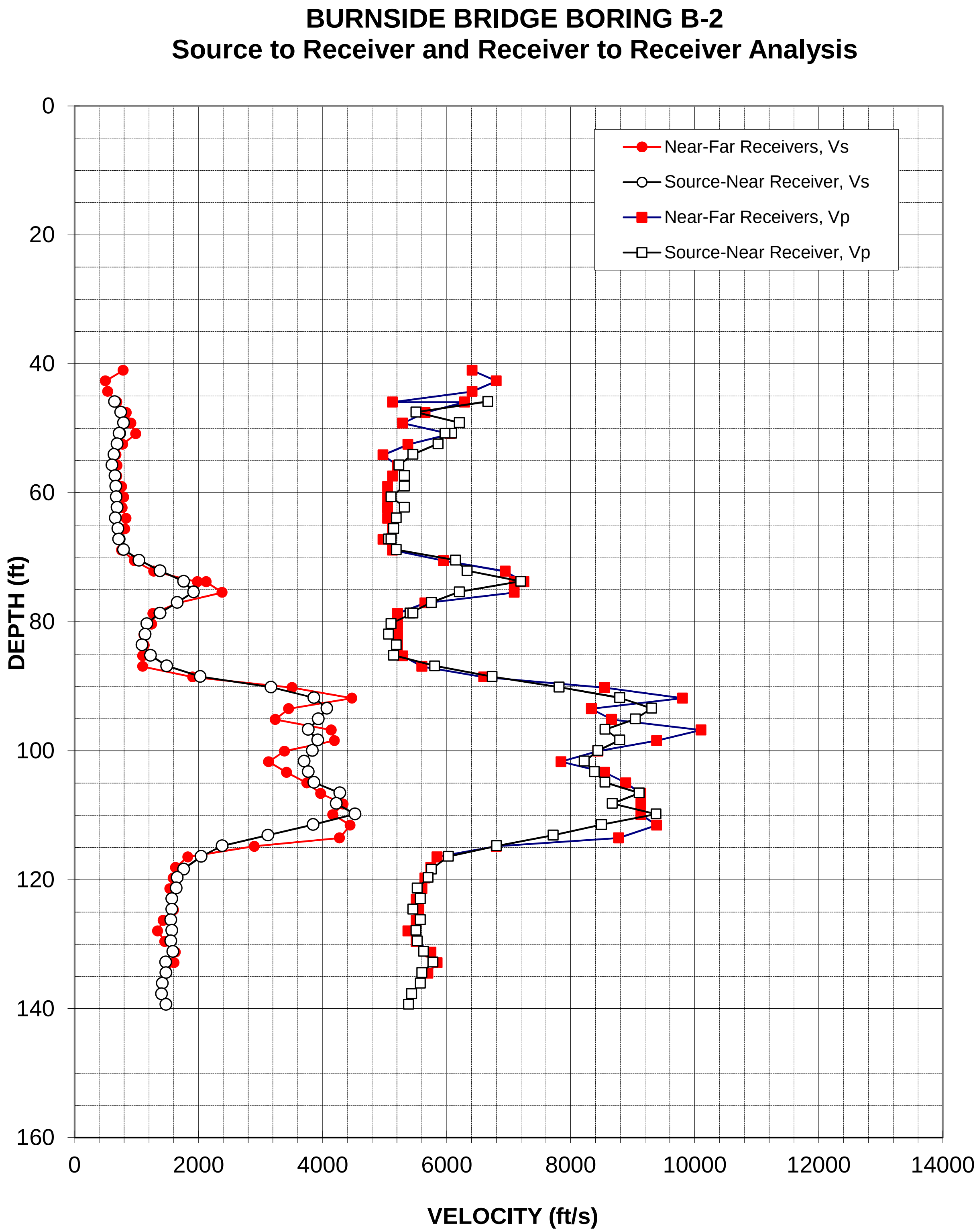


Figure A-2: Borehole B-2, Suspension S-R1 P- and S_H-wave velocities

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				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)	
45.8	650	6660	0.50	14.0	200	2030	0.50
47.5	740	5500	0.49	14.5	230	1680	0.49
49.1	790	6210	0.49	15.0	240	1890	0.49
50.8	730	6090	0.49	15.5	220	1860	0.49
50.8	720	5970	0.49	15.5	220	1820	0.49
52.4	690	5860	0.49	16.0	210	1790	0.49
54.0	630	5460	0.49	16.5	190	1660	0.49
55.7	610	5230	0.49	17.0	180	1590	0.49
57.3	650	5320	0.49	17.5	200	1620	0.49
59.0	660	5320	0.49	18.0	200	1620	0.49
60.6	670	5100	0.49	18.5	210	1560	0.49
62.2	690	5320	0.49	19.0	210	1620	0.49
63.9	660	5190	0.49	19.5	200	1580	0.49
65.5	700	5150	0.49	20.0	210	1570	0.49
67.2	720	5060	0.49	20.5	220	1540	0.49
67.2	710	5100	0.49	20.5	220	1560	0.49
68.8	790	5190	0.49	21.0	240	1580	0.49
70.5	1040	6150	0.49	21.5	320	1870	0.49
72.1	1380	6330	0.48	22.0	420	1930	0.48
73.7	1760	7190	0.47	22.5	540	2190	0.47
75.4	1920	6210	0.45	23.0	580	1890	0.45
77.0	1660	5750	0.45	23.5	510	1750	0.45
78.7	1380	5410	0.47	24.0	420	1650	0.47
78.7	1380	5460	0.47	24.0	420	1660	0.47
80.3	1170	5100	0.47	24.5	360	1560	0.47
81.9	1140	5060	0.47	25.0	350	1540	0.47
83.6	1090	5190	0.48	25.5	330	1580	0.48
85.2	1220	5150	0.47	26.0	370	1570	0.47
86.9	1490	5810	0.46	26.5	450	1770	0.46
88.5	2030	6730	0.45	27.0	620	2050	0.45
90.1	3170	7810	0.40	27.5	960	2380	0.40
91.8	3860	8790	0.38	28.0	1180	2680	0.38
93.4	4070	9310	0.38	28.5	1240	2840	0.38
95.1	3930	9040	0.38	29.0	1200	2760	0.38
96.7	3770	8550	0.38	29.5	1150	2610	0.38
98.3	3920	8790	0.38	30.0	1190	2680	0.38
100.0	3840	8440	0.37	30.5	1170	2570	0.37
101.6	3700	8220	0.37	31.0	1130	2510	0.37
103.3	3770	8380	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			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V_s	V_p	
(ft)	(ft/s)	(ft/s)	
104.9	3860	8550	0.37
106.5	4280	9110	0.36
108.2	4220	8670	0.34
109.8	4520	9380	0.35
111.5	3850	8500	0.37
113.1	3120	7720	0.40
114.7	2380	6810	0.43
116.4	2040	6030	0.44
118.4	1760	5750	0.45
119.7	1660	5700	0.45
121.3	1640	5530	0.45
122.9	1570	5580	0.46
124.6	1570	5460	0.45
126.2	1550	5580	0.46
127.9	1570	5500	0.46
129.5	1550	5530	0.46
131.1	1590	5630	0.46
132.8	1470	5780	0.47
134.4	1470	5600	0.46
136.1	1410	5580	0.47
137.7	1400	5430	0.46
139.3	1480	5390	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)	
32.0	1180	2610	0.37
32.5	1300	2780	0.36
33.0	1290	2640	0.34
33.5	1380	2860	0.35
34.0	1170	2590	0.37
34.5	950	2350	0.40
35.0	730	2070	0.43
35.5	620	1840	0.44
36.1	540	1750	0.45
36.5	510	1740	0.45
37.0	500	1690	0.45
37.5	480	1700	0.46
38.0	480	1660	0.45
38.5	470	1700	0.46
39.0	480	1680	0.46
39.5	470	1690	0.46
40.0	480	1720	0.46
40.5	450	1760	0.47
41.0	450	1710	0.46
41.5	430	1700	0.47
42.0	430	1660	0.46
42.5	450	1640	0.46

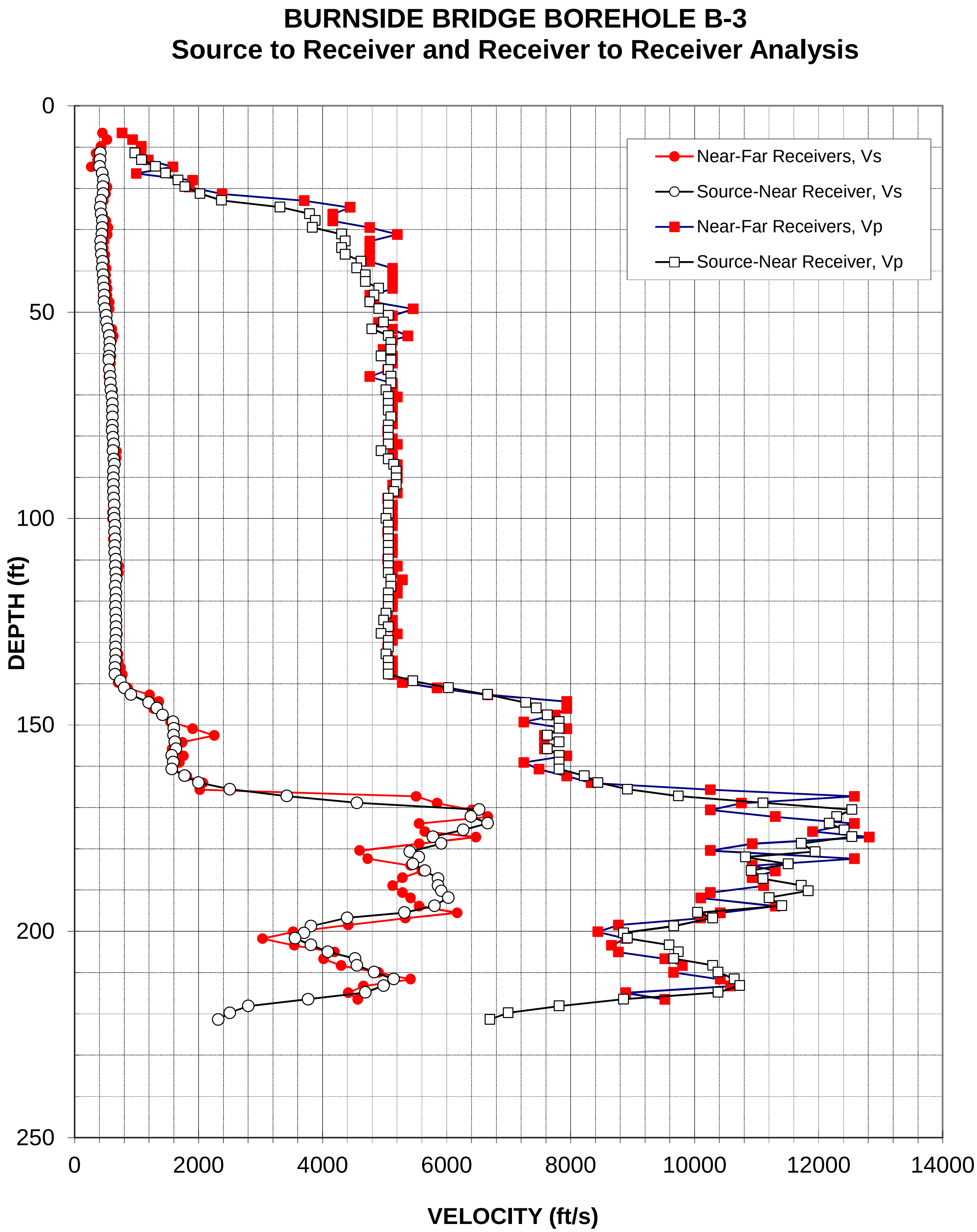


Figure A-3: Borehole B-3, Suspension S-R1 P- and S_H-wave velocities

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				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity V_s	Velocity V_p	Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity V_s	Velocity V_p	Poisson's Ratio
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
11.4	420	980	0.39	3.5	130	300	0.39
13.0	410	1080	0.42	4.0	130	330	0.42
14.7	410	1310	0.45	4.5	120	400	0.45
16.3	440	1470	0.45	5.0	140	450	0.45
18.0	470	1670	0.46	5.5	140	510	0.46
19.6	460	1780	0.46	6.0	140	540	0.46
21.2	460	2020	0.47	6.5	140	620	0.47
22.9	430	2370	0.48	7.0	130	720	0.48
24.5	420	3310	0.49	7.5	130	1010	0.49
26.2	430	3790	0.49	8.0	130	1160	0.49
27.8	450	3880	0.49	8.5	140	1180	0.49
29.4	450	3840	0.49	9.0	140	1170	0.49
31.1	440	4310	0.49	9.5	130	1310	0.49
32.7	420	4370	0.50	10.0	130	1330	0.50
34.4	420	4310	0.50	10.5	130	1310	0.50
36.0	440	4370	0.49	11.0	130	1330	0.49
37.6	450	4620	0.50	11.5	140	1410	0.50
39.3	440	4550	0.50	12.0	140	1390	0.50
40.9	460	4690	0.50	12.5	140	1430	0.50
42.6	460	4690	0.50	13.0	140	1430	0.50
44.2	470	4910	0.50	13.5	140	1500	0.50
45.8	480	4830	0.50	14.0	150	1470	0.50
47.5	470	4760	0.49	14.5	140	1450	0.49
49.1	490	4910	0.49	15.0	150	1500	0.49
50.8	510	5060	0.49	15.5	160	1540	0.49
52.4	520	4980	0.49	16.0	160	1520	0.49
54.0	530	4800	0.49	16.5	160	1460	0.49
55.7	560	5060	0.49	17.0	170	1540	0.49
57.3	570	5100	0.49	17.5	170	1560	0.49
59.0	570	5100	0.49	18.0	170	1560	0.49
60.6	560	4950	0.49	18.5	170	1510	0.49
61.6	550	5100	0.49	18.8	170	1560	0.49
63.9	560	5060	0.49	19.5	170	1540	0.49
65.5	580	5100	0.49	20.0	180	1560	0.49
67.2	580	5100	0.49	20.5	180	1560	0.49
68.8	590	5020	0.49	21.0	180	1530	0.49
70.5	600	5060	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				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)	
72.1	610	5060	0.49	22.0	190	1540	0.49
73.7	610	5060	0.49	22.5	190	1540	0.49
75.4	610	5100	0.49	23.0	190	1560	0.49
77.3	610	5060	0.49	23.6	190	1540	0.49
78.7	610	5060	0.49	24.0	180	1540	0.49
80.3	620	5060	0.49	24.5	190	1540	0.49
81.9	630	5060	0.49	25.0	190	1540	0.49
83.6	620	4950	0.49	25.5	190	1510	0.49
85.5	630	5060	0.49	26.1	190	1540	0.49
86.9	640	5150	0.49	26.5	200	1570	0.49
88.5	630	5190	0.49	27.0	190	1580	0.49
90.1	630	5190	0.49	27.5	190	1580	0.49
91.8	630	5190	0.49	28.0	190	1580	0.49
93.4	630	5150	0.49	28.5	190	1570	0.49
95.1	630	5060	0.49	29.0	190	1540	0.49
96.7	640	5060	0.49	29.5	190	1540	0.49
98.7	640	5060	0.49	30.1	190	1540	0.49
100.0	640	5020	0.49	30.5	190	1530	0.49
101.6	650	5060	0.49	31.0	200	1540	0.49
103.3	640	5060	0.49	31.5	200	1540	0.49
104.9	660	5060	0.49	32.0	200	1540	0.49
106.5	650	5060	0.49	32.5	200	1540	0.49
108.2	650	5060	0.49	33.0	200	1540	0.49
109.8	670	5060	0.49	33.5	200	1540	0.49
111.5	660	5060	0.49	34.0	200	1540	0.49
113.1	670	5060	0.49	34.5	200	1540	0.49
114.7	670	5100	0.49	35.0	210	1560	0.49
116.4	660	5100	0.49	35.5	200	1560	0.49
118.0	670	5060	0.49	36.0	200	1540	0.49
119.7	670	5060	0.49	36.5	200	1540	0.49
121.3	660	5060	0.49	37.0	200	1540	0.49
122.9	670	5020	0.49	37.5	200	1530	0.49
124.6	670	4980	0.49	38.0	200	1520	0.49
126.2	670	5060	0.49	38.5	200	1540	0.49
127.9	670	4950	0.49	39.0	200	1510	0.49
129.5	670	5060	0.49	39.5	200	1540	0.49
131.1	660	5060	0.49	40.0	200	1540	0.49
132.8	670	5020	0.49	40.5	200	1530	0.49
134.4	660	5060	0.49	41.0	200	1540	0.49
136.1	650	5060	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				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)	
137.7	660	5060	0.49	42.0	200	1540	0.49
139.3	740	5460	0.49	42.5	230	1660	0.49
141.0	800	6030	0.49	43.0	250	1840	0.49
142.6	910	6660	0.49	43.5	280	2030	0.49
144.6	1190	7280	0.49	44.1	360	2220	0.49
145.9	1330	7450	0.48	44.5	400	2270	0.48
147.6	1420	7630	0.48	45.0	430	2320	0.48
149.2	1590	7810	0.48	45.5	480	2380	0.48
150.8	1600	7810	0.48	46.0	490	2380	0.48
152.5	1590	7630	0.48	46.5	490	2320	0.48
154.1	1620	7810	0.48	47.0	490	2380	0.48
155.8	1640	7630	0.48	47.5	500	2320	0.48
157.4	1570	7810	0.48	48.0	480	2380	0.48
159.0	1590	7810	0.48	48.5	480	2380	0.48
160.7	1570	7810	0.48	49.0	480	2380	0.48
162.3	1770	8220	0.48	49.5	540	2510	0.48
164.0	2000	8440	0.47	50.0	610	2570	0.47
165.6	2500	8920	0.46	50.5	760	2720	0.46
167.2	3420	9740	0.43	51.0	1040	2970	0.43
168.9	4550	11110	0.40	51.5	1390	3380	0.40
170.5	6530	12530	0.31	52.0	1990	3820	0.31
172.2	6390	12290	0.31	52.5	1950	3750	0.31
173.8	6660	12170	0.29	53.0	2030	3710	0.29
175.4	6270	12410	0.33	53.5	1910	3780	0.33
177.1	5780	12530	0.36	54.0	1760	3820	0.36
178.7	5920	11720	0.33	54.5	1800	3570	0.33
180.7	5410	11940	0.37	55.1	1650	3640	0.37
182.0	5550	10820	0.32	55.5	1690	3300	0.32
183.6	5460	11510	0.35	56.0	1660	3510	0.35
185.3	5650	10910	0.32	56.5	1720	3330	0.32
187.2	5860	11110	0.31	57.1	1790	3380	0.31
188.9	5860	11720	0.33	57.6	1790	3570	0.33
190.2	5920	11830	0.33	58.0	1800	3610	0.33
191.8	6030	11200	0.30	58.5	1840	3410	0.30
193.8	5810	11410	0.33	59.1	1770	3480	0.33
195.5	5320	10050	0.31	59.6	1620	3060	0.31
196.8	4400	10290	0.39	60.0	1340	3140	0.39
198.7	3810	9660	0.41	60.6	1160	2950	0.41
200.4	3700	8850	0.39	61.1	1130	2700	0.39
201.7	3560	8920	0.41	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				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)	
203.3	3810	9590	0.41	62.0	1160	2920	0.41
205.0	4080	9740	0.39	62.5	1240	2970	0.39
206.6	4520	9660	0.36	63.0	1380	2950	0.36
208.2	4550	10290	0.38	63.5	1390	3140	0.38
209.9	4830	10380	0.36	64.0	1470	3160	0.36
211.5	5150	10640	0.35	64.5	1570	3240	0.35
213.2	4980	10730	0.36	65.0	1520	3270	0.36
214.8	4690	10380	0.37	65.5	1430	3160	0.37
216.4	3770	8850	0.39	66.0	1150	2700	0.39
218.1	2800	7810	0.43	66.5	850	2380	0.43
219.7	2500	6990	0.43	67.0	760	2130	0.43
221.4	2320	6700	0.43	67.5	710	2040	0.43

APPENDIX B

BOREHOLE GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

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**

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.

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 logo]

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.:	CF-29 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: [signature] Name 7/14/16 Date Tyler McKeen Signature

Witnessed by: Emily Feldman Name 7/14/16 Date [signature] Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012

APPENDIX D

LABORATORY TEST RESULTS

TABLE OF CONTENTS

FIGURES

ATTACHMENTS

APPENDIX D

LABORATORY TEST RESULTS

D.1 GENERAL

The soil samples obtained during the 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.

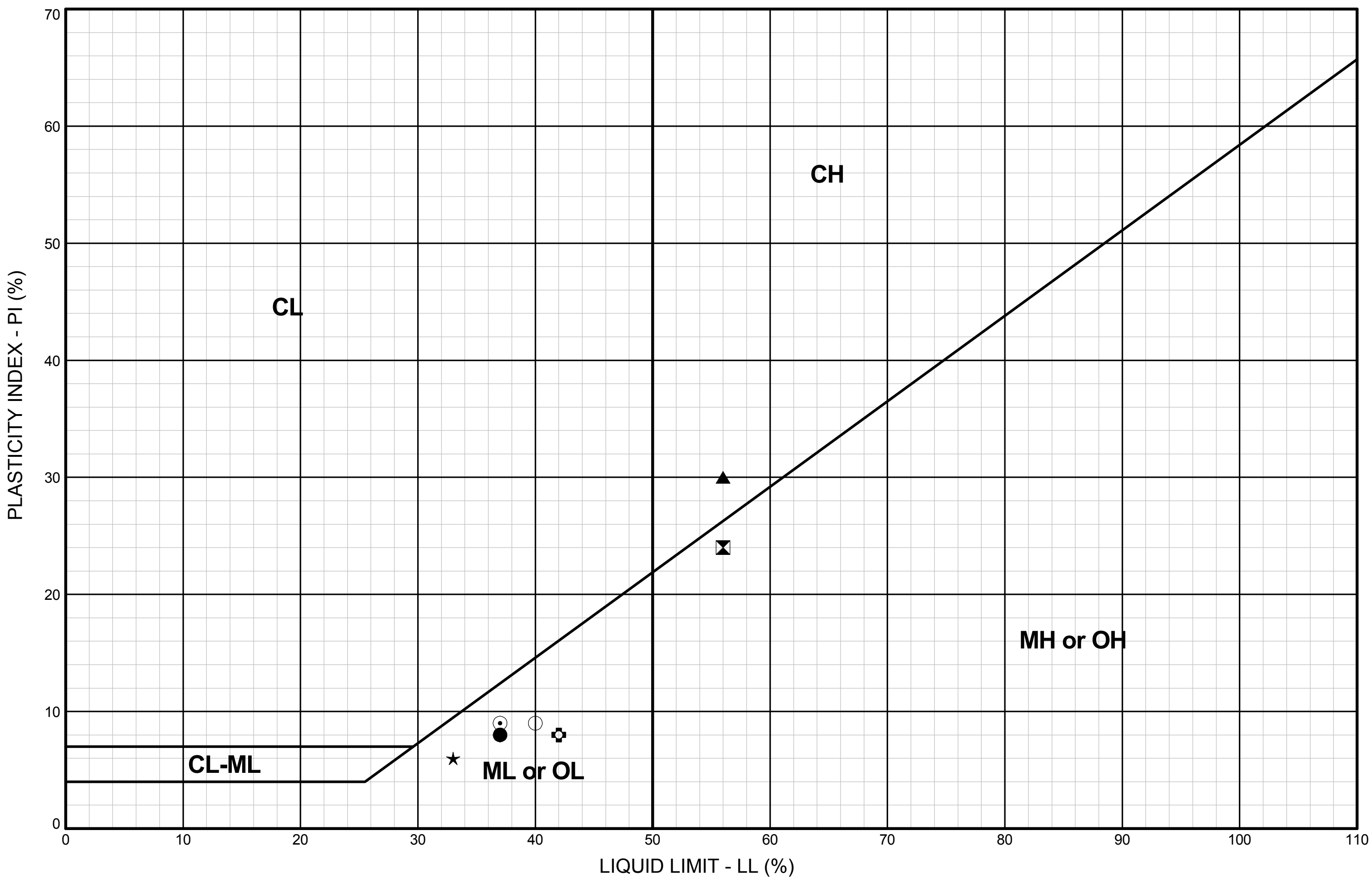
D.2 SOIL TESTING

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.



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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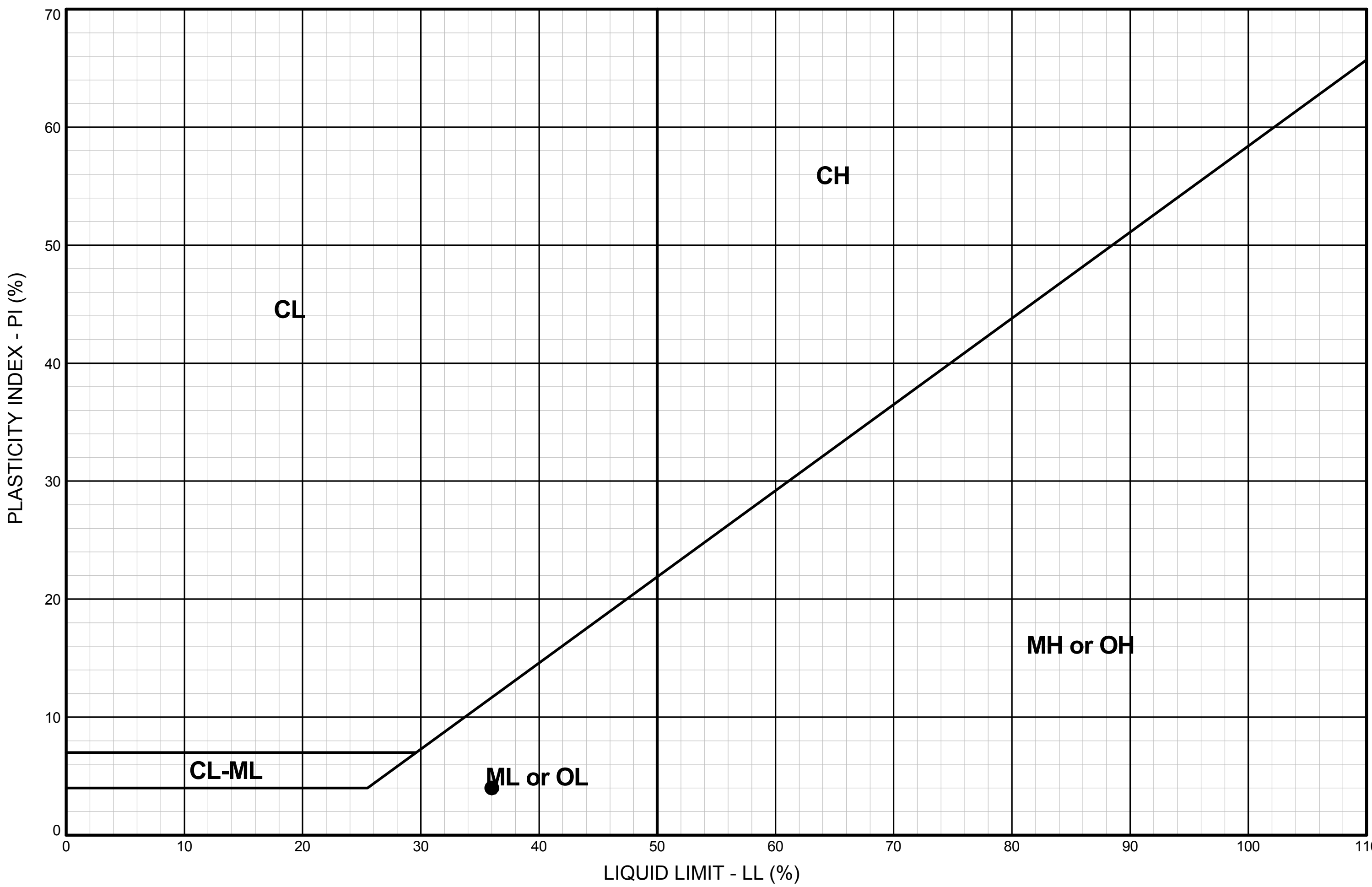
FIG. D1
Sheet 1 of 2

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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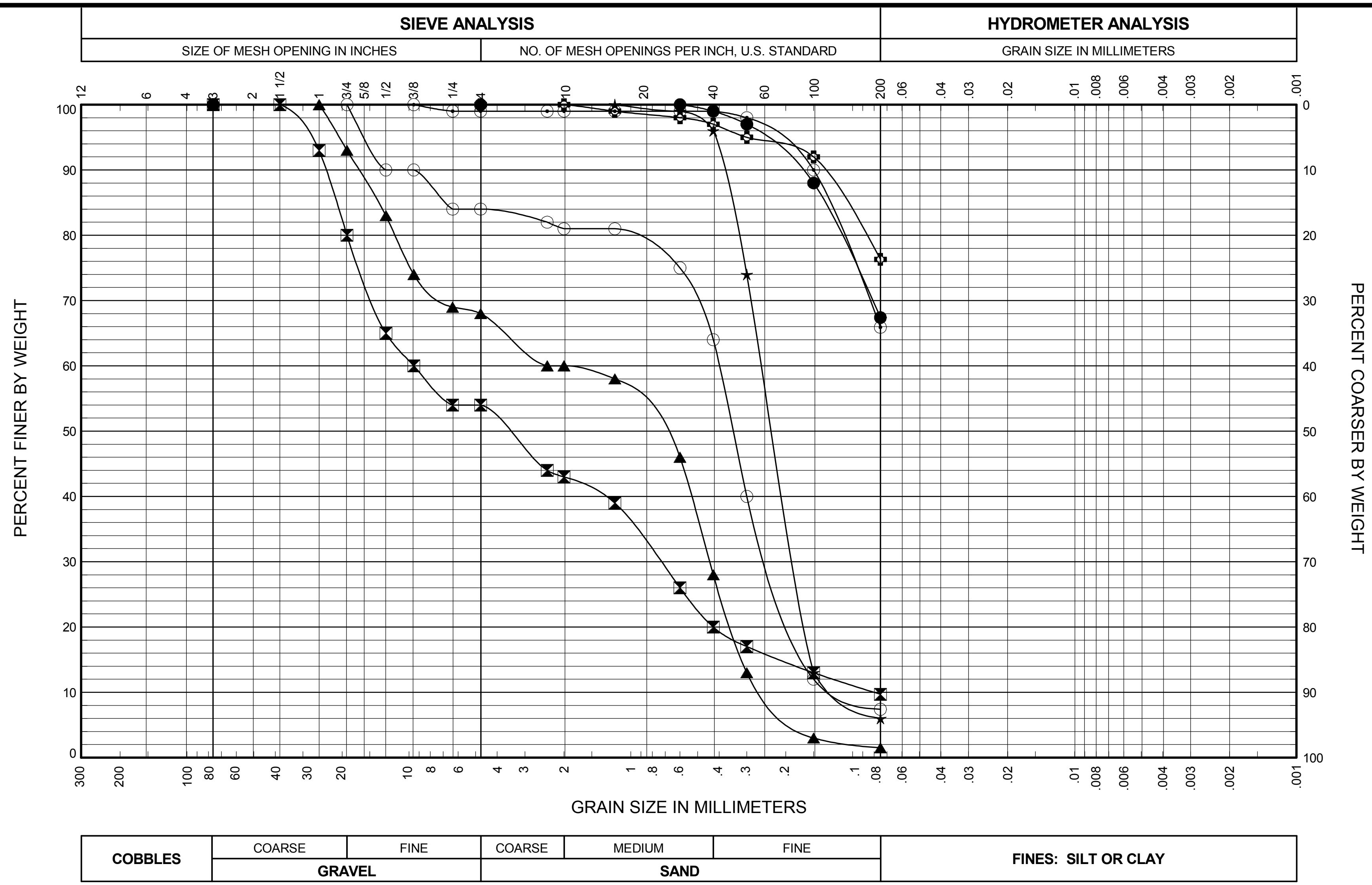
FIG. D1
Sheet 2 of 2

ATT MAIN 24-1-04065.GPJ SHAN WIL.GDT 1/20/17

FIG. D1



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.

GSA_MAIN 24-1-04065.GPJ SHAN_WIL.GDT 1/20/17

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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Portland, Oregon

GRAIN SIZE DISTRIBUTION

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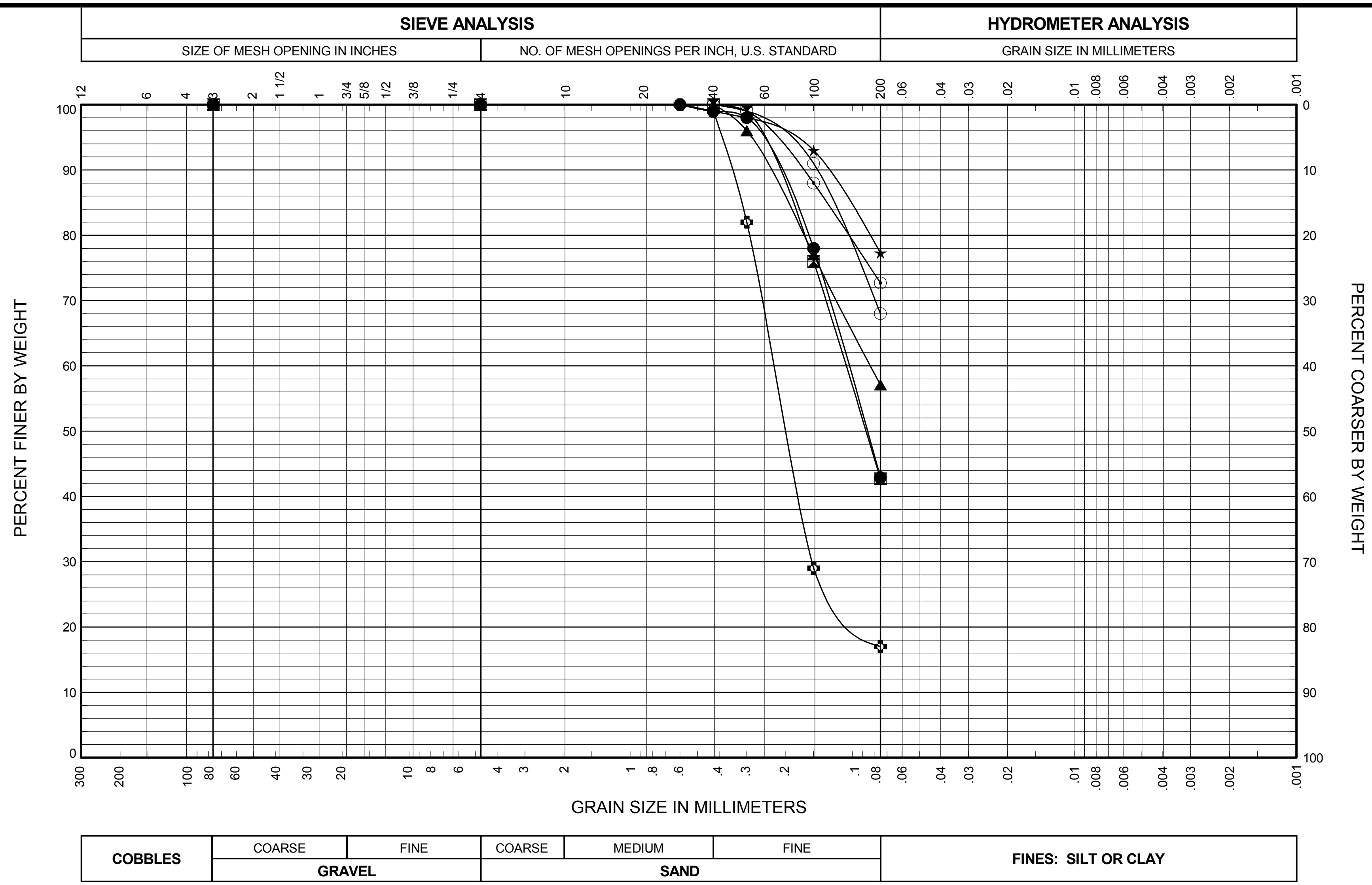
FIG. D2
Sheet 1 of 2

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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Portland, Oregon

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FIG. D2
Sheet 2 of 2

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:** **Lab No.:**	11/28/16 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

 BKA

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

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

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:** **Lab No.:**	11/28/16 16-304
Project:	Laboratory Testing – 24-1-04065-001	**Project No.:**	2966.1.1

Laboratory Testing

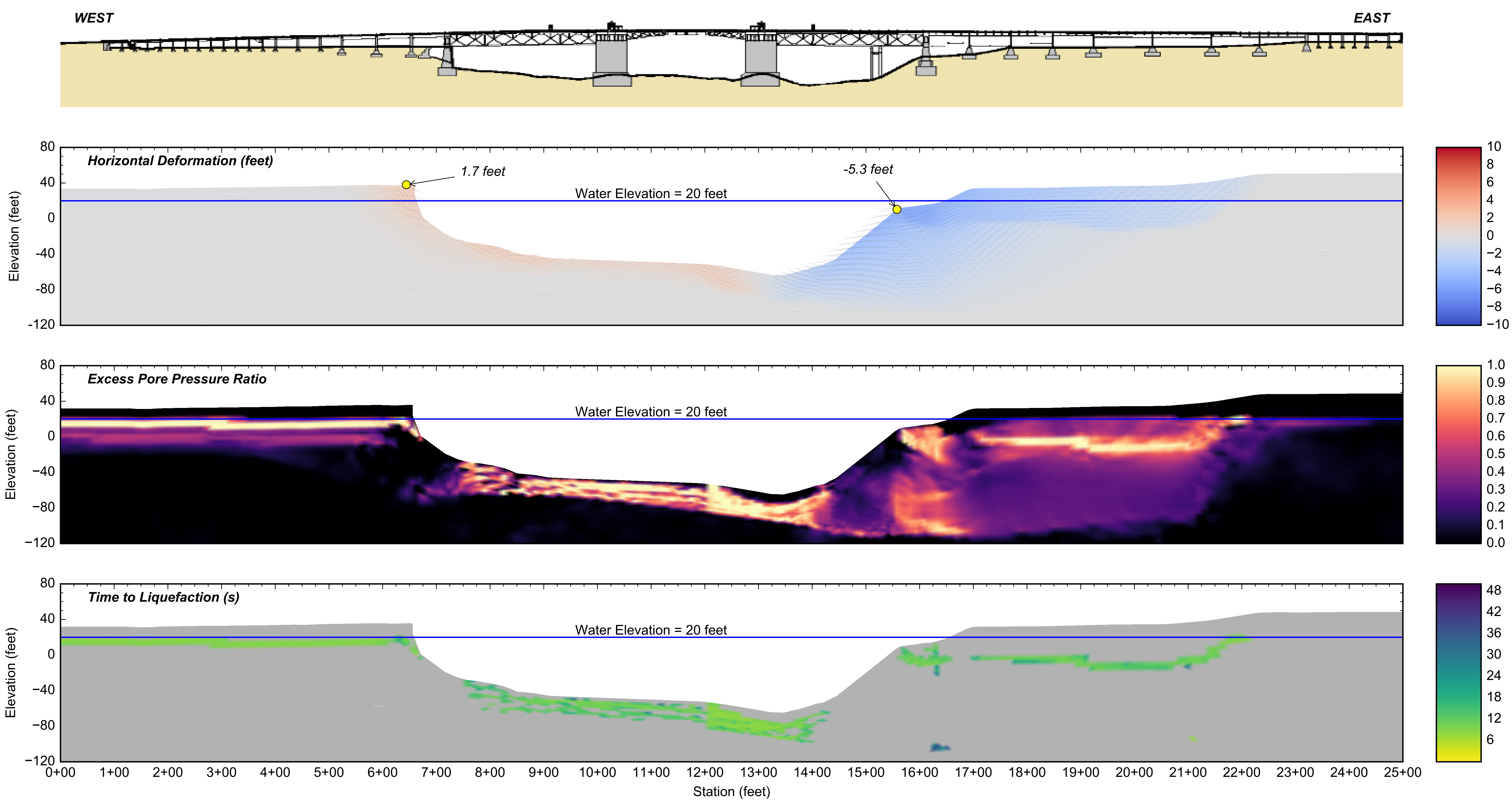
Sieve Analysis of Aggregate (ASTM C117/C136)			
Sieve Size	**B-3 N-6 @ 32 – 33.5 ft. Percent Passing**	**B-3 N-14 @ 70 – 71.5 ft. Percent Passing**	**B-3 N-17 @ 85 – 86.5 ft. Percent Passing**
#30	--	--	100
#40	100	100	99
#50	99	96	98
#100	76	77	93
#200	42.7	57.1	77.3

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

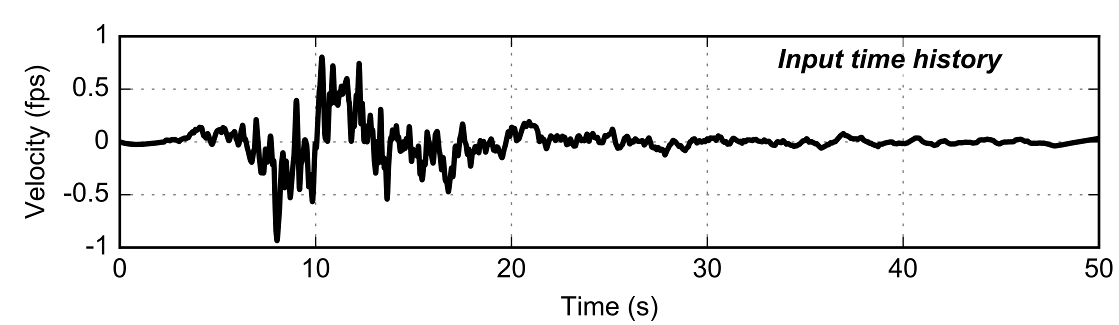
APPENDIX E

FLAC ANALYSIS RESULTS

FIGURES



Notes:
1. The input ground motion was recorded during the Loma Prieta earthquake at the SJTE station (225 degree component). We scaled the time history to approximate a 1,000 year return period.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

MODEL RESPONSE TIME HISTORY LP-1000

July 2017 | 24-1-04065-005

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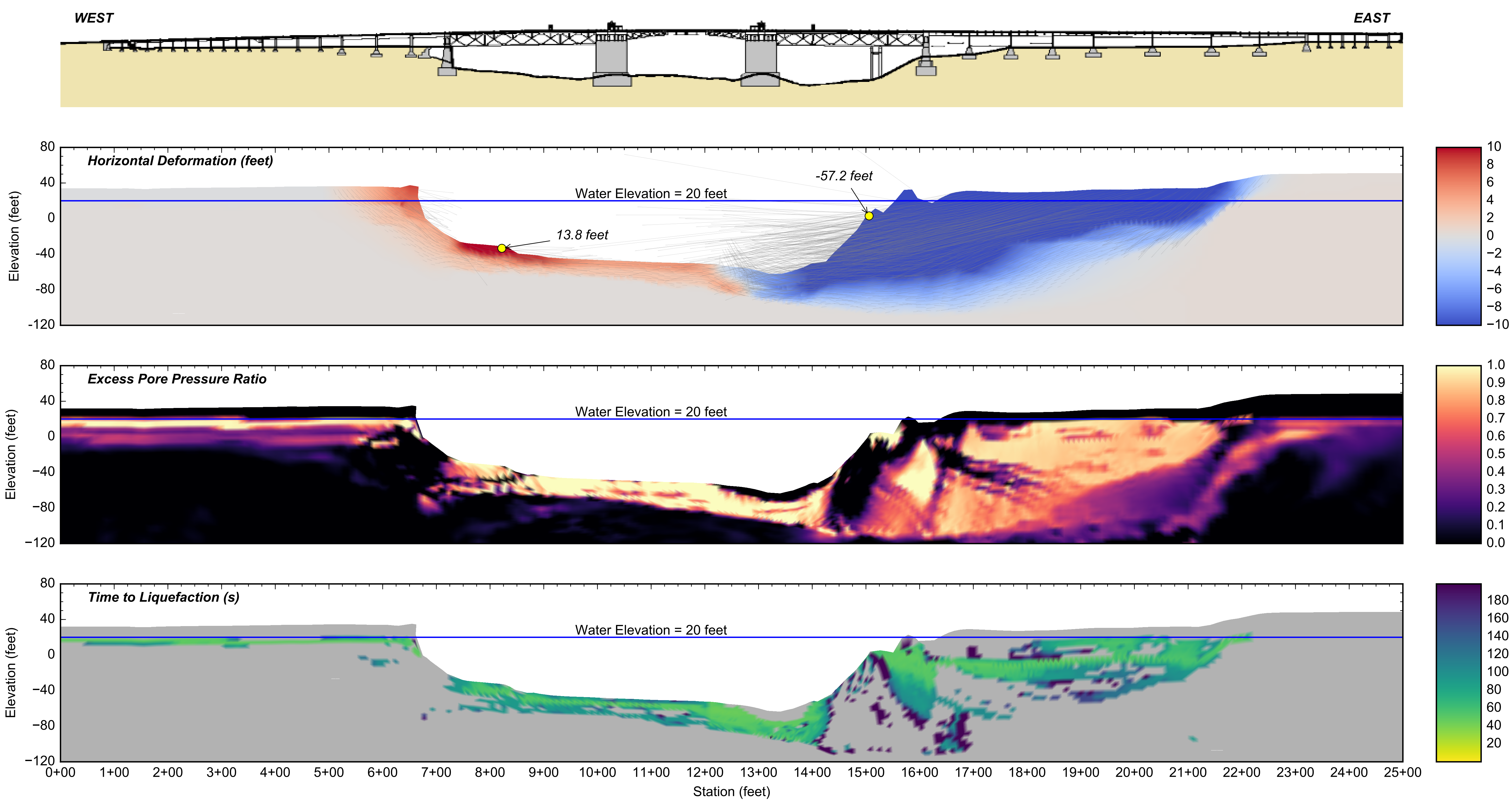
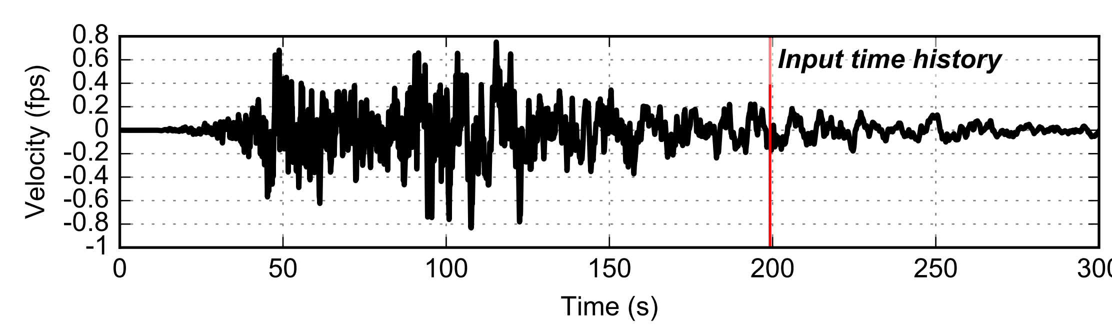
FIG. E-1



Notes:
1. The input ground motion was recorded during the Tohoku earthquake at the AKT station (east-west component). We scaled the time history to approximate a 1,000 year return period. The model failed due to bad geometry at 199.2 seconds.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



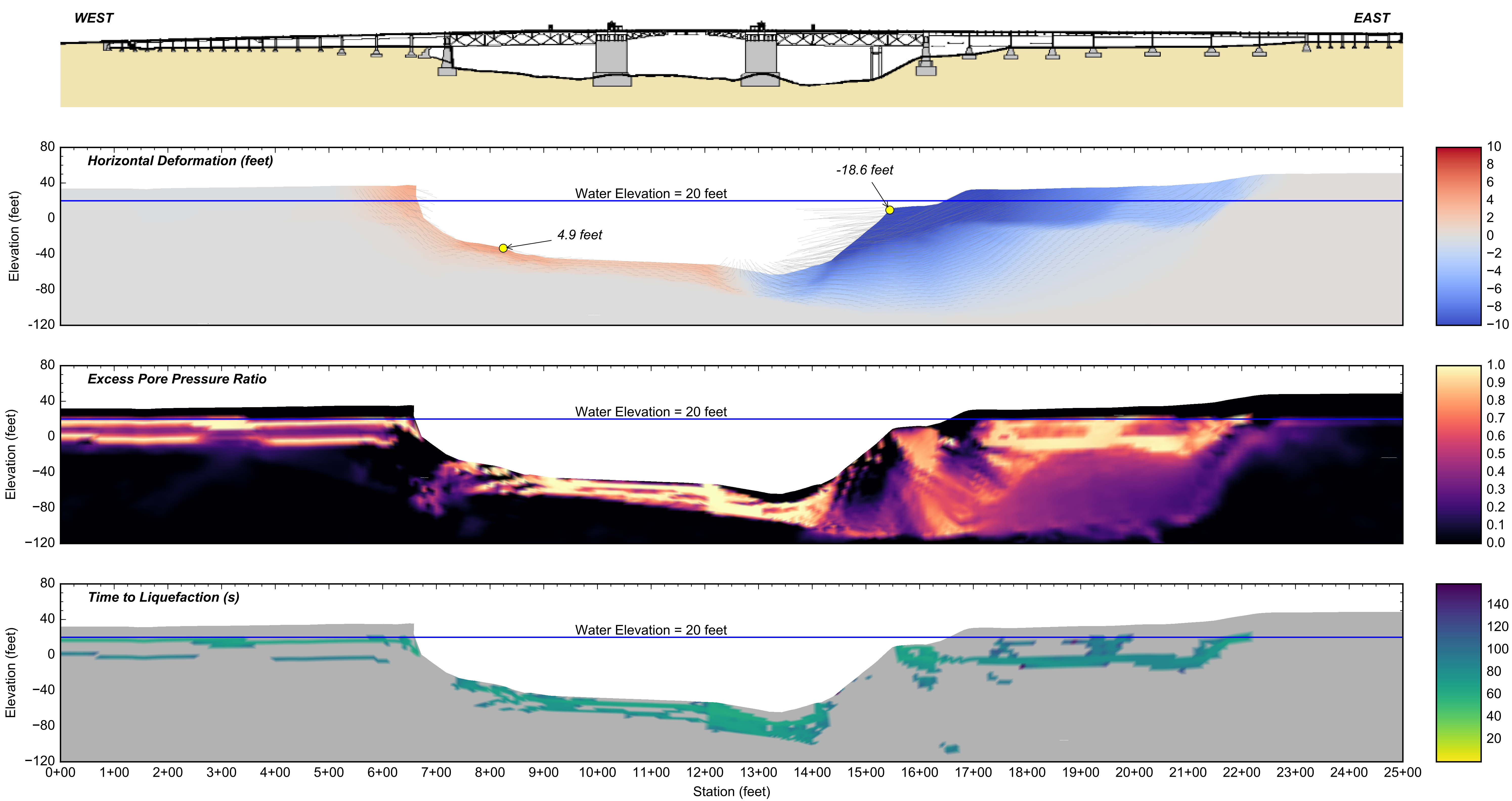
Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

MODEL RESPONSE
TIME HISTORY T-1000

July 2017 24-1-04065-005

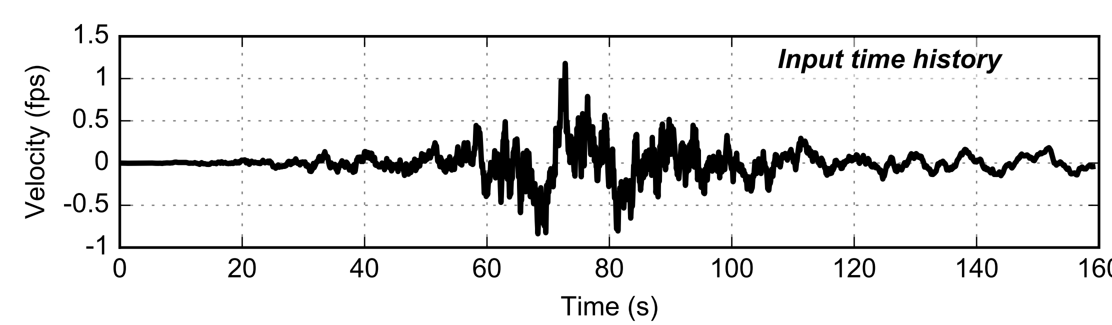
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E-2



Notes:

1. The input ground motion was recorded during the Maule earthquake at the VICH station (360 degree component). We scaled the time history to approximate a 1,000 year return period.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

MODEL RESPONSE
TIME HISTORY M-1000

July 2017 | 24-1-04065-005

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FIG. E-3

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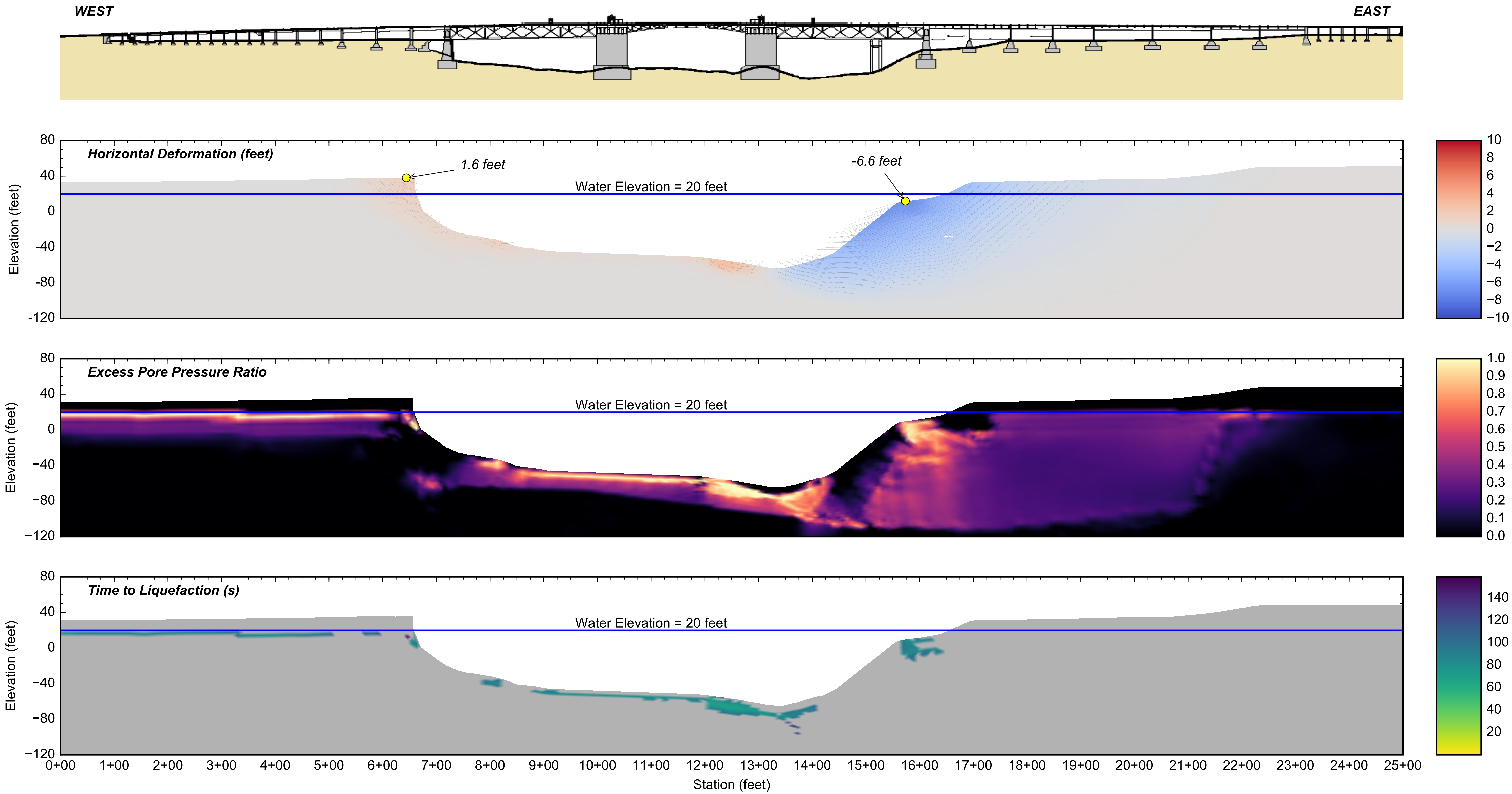
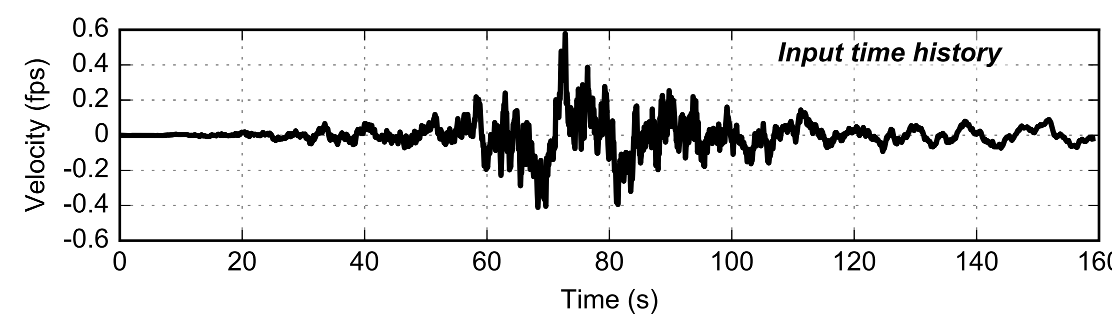
C:\Users\inb\FLAC\Burnside\v5



Notes:
1. The input ground motion was recorded during the Maule earthquake at the STL station (360 degree component). We scaled the time history to approximate a Cascadia Subduction Zone event.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

MODEL RESPONSE
TIME HISTORY M-CSZ

July 2017 | 24-1-04065-005

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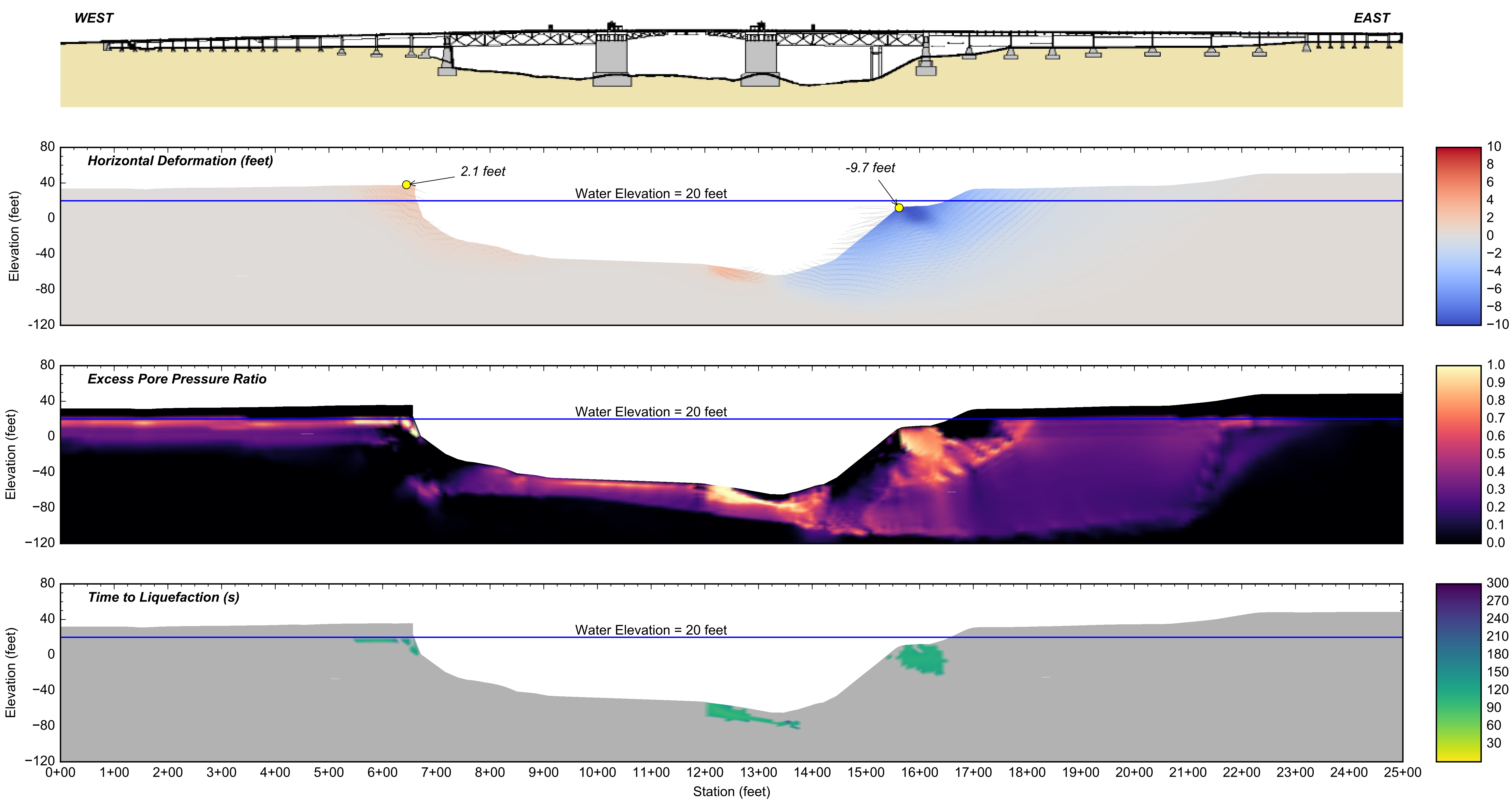
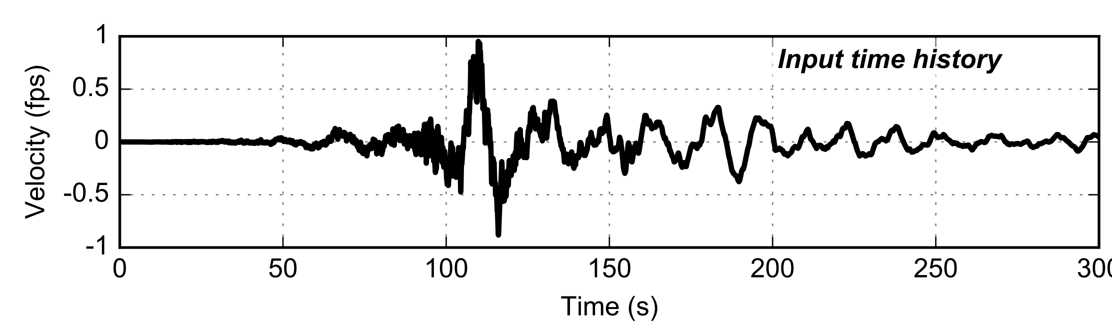
FIG. E-4



Notes:
1. The input ground motion was recorded during the Maule earthquake at the VICH station (360 degree component). We scaled the time history to approximate a Cascadia Subduction Zone event.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



Burnside Bridge
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Portland, Oregon

MODEL RESPONSE
TIME HISTORY MV-CSZ

July 2017 | 24-1-04065-005

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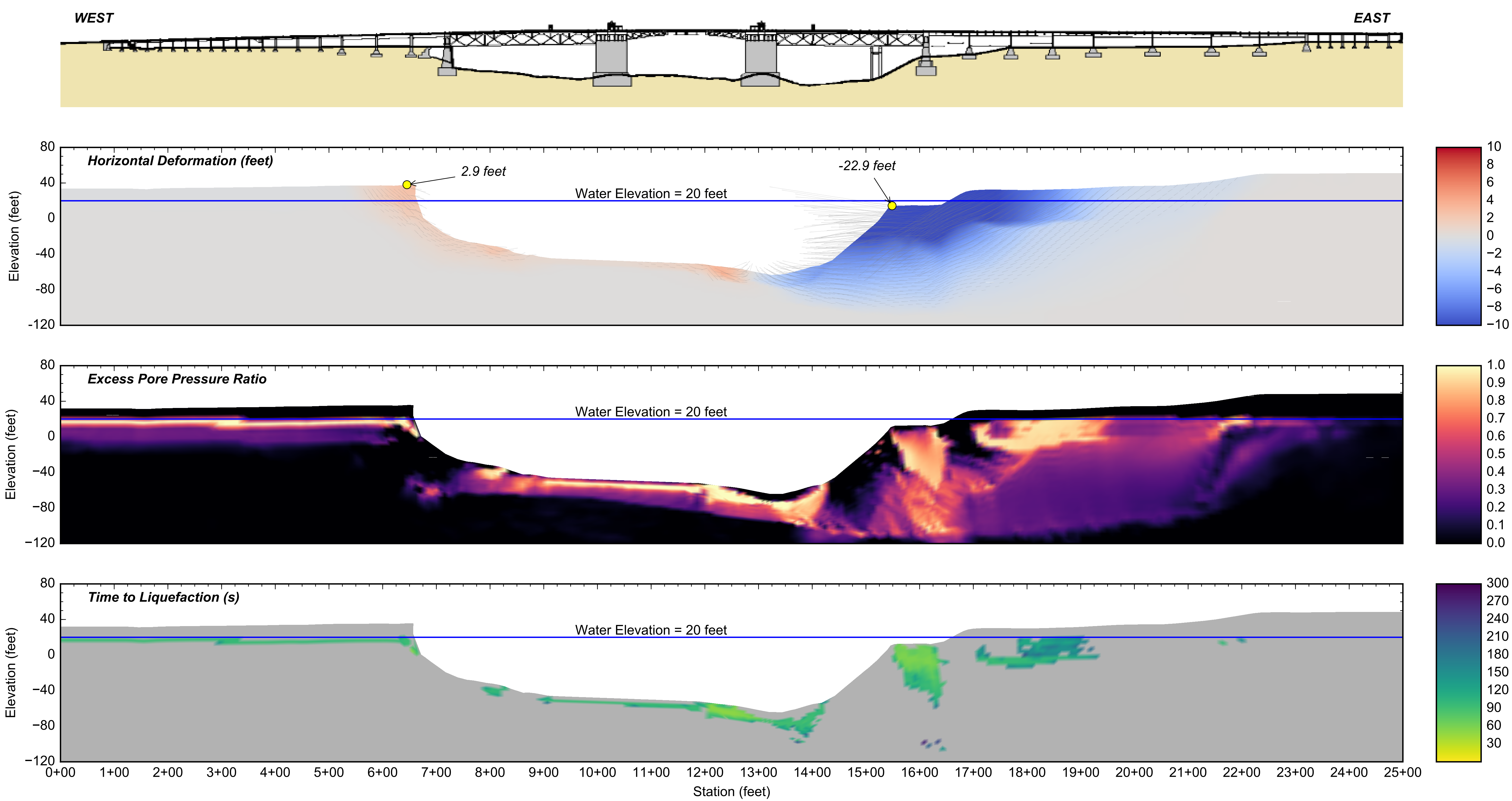
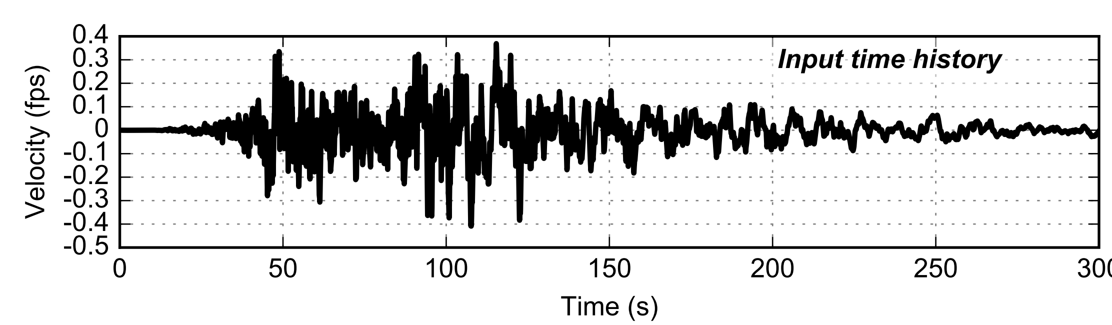
FIG. E-5



Notes:
1. The input ground motion was recorded during the Tohoku earthquake at the AKT station (east-west component). We scaled the time history to approximate a Cascadia Subduction Zone event.
2. Time to liquefaction is the time in the ground motion time history where the excess pore pressure ratio exceeds 0.9.
3. s = seconds
   fps = feet per second



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MODEL RESPONSE
TIME HISTORY T-CSZ

July 2017 24-1-04065-005

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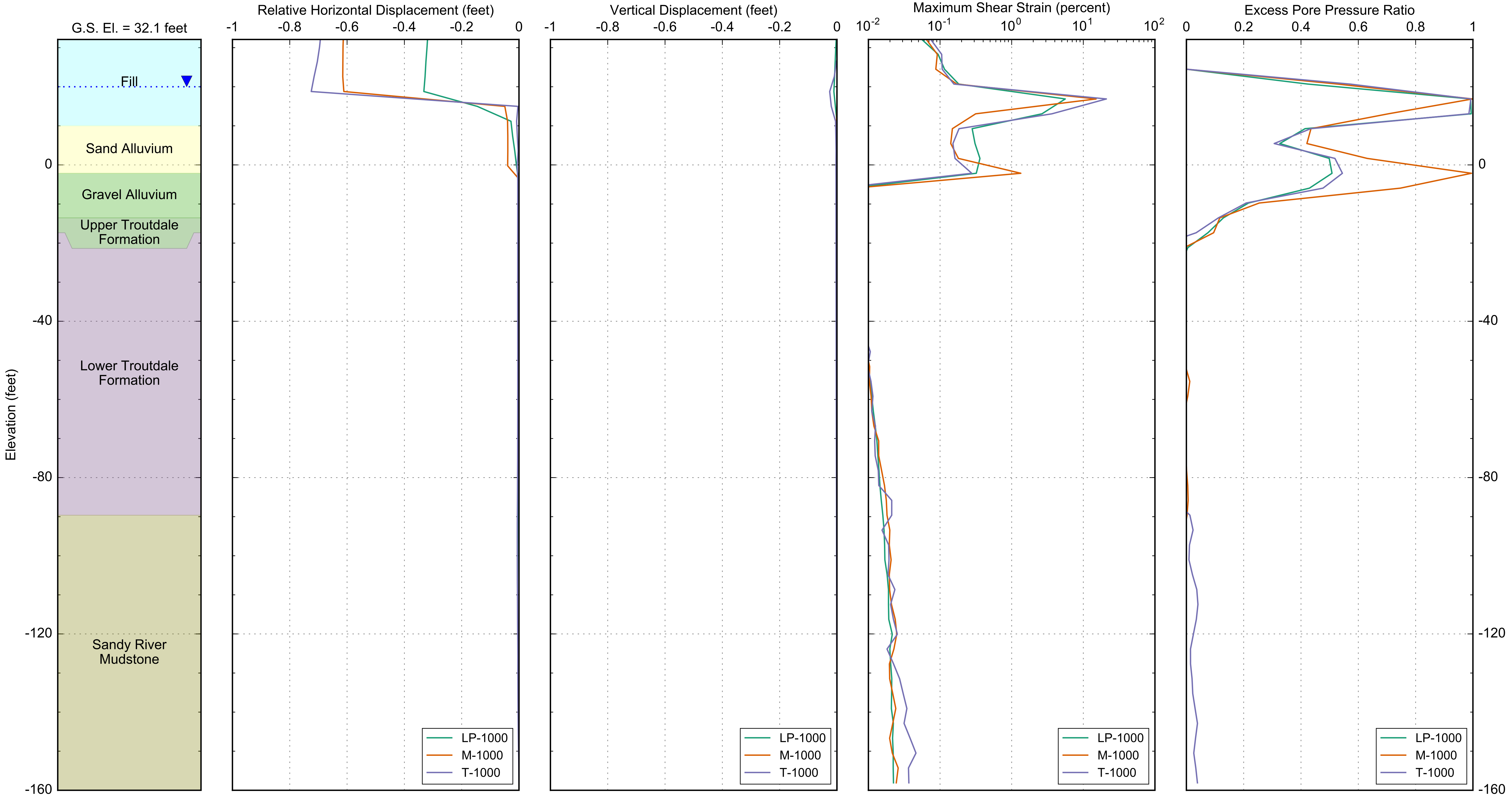
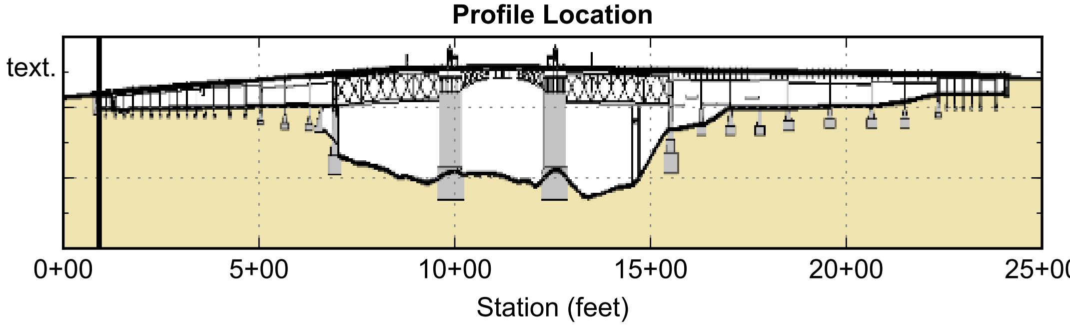
FIG. E-6



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 1

July 2017 24-1-04065-005

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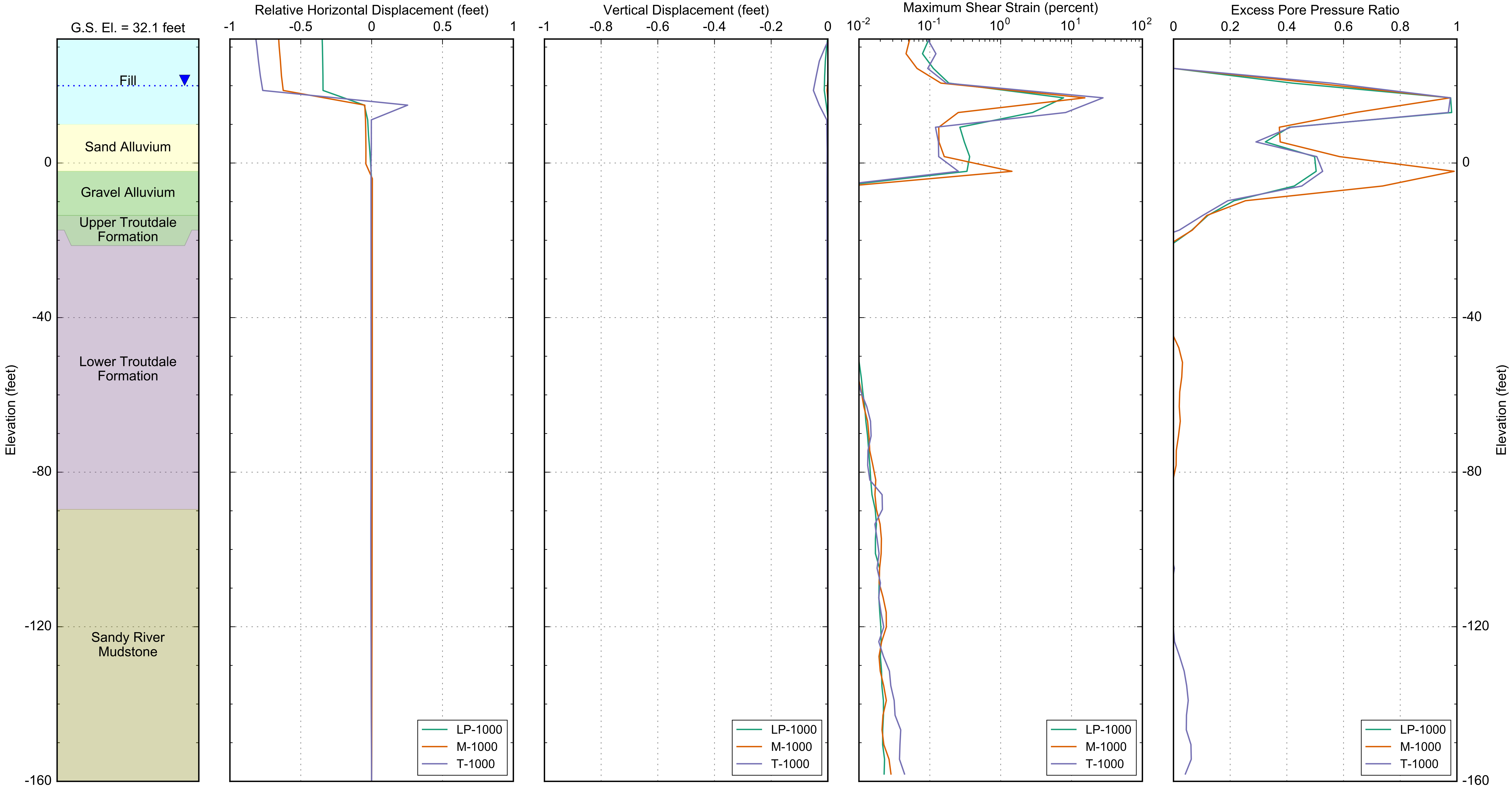
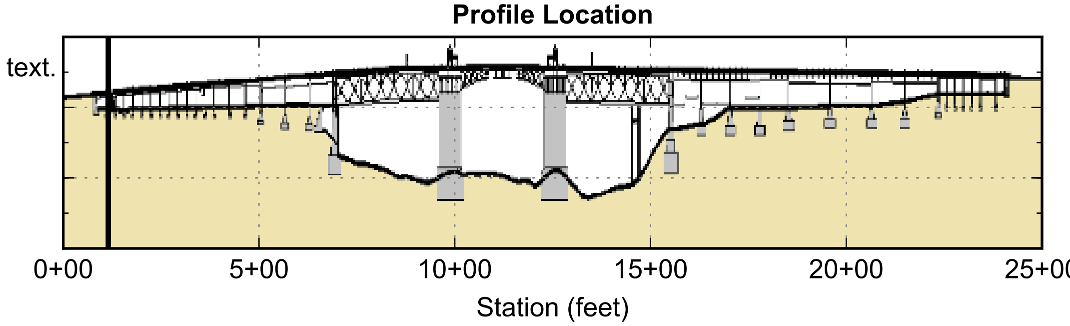
FIG. E-7



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



Burnside Bridge
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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 2

July 2017 | 24-1-04065-005

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FIG. E-8

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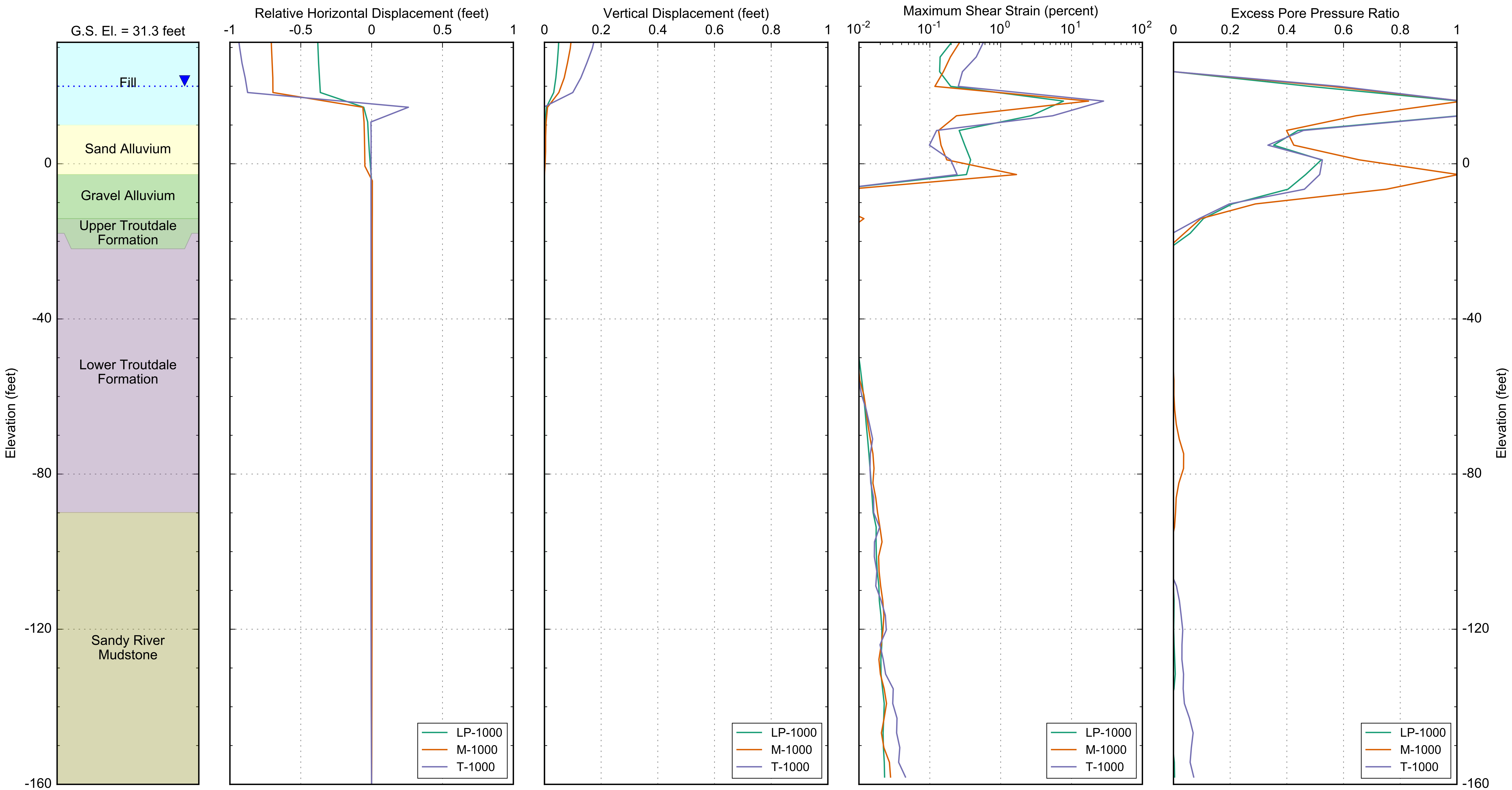
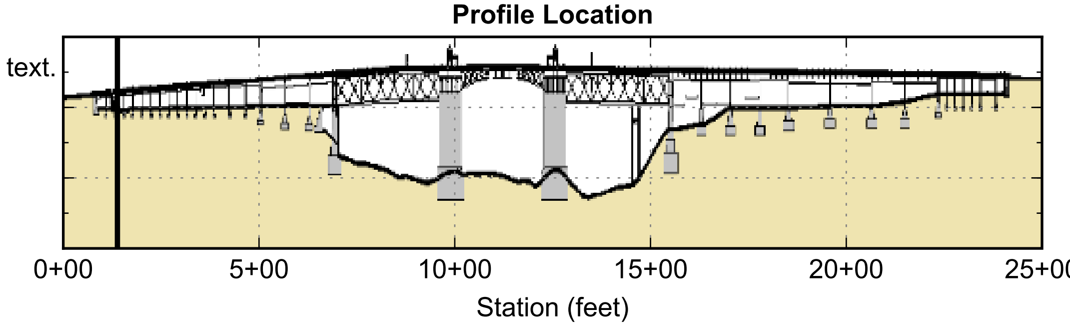


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
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Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 3

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FIG. E-9

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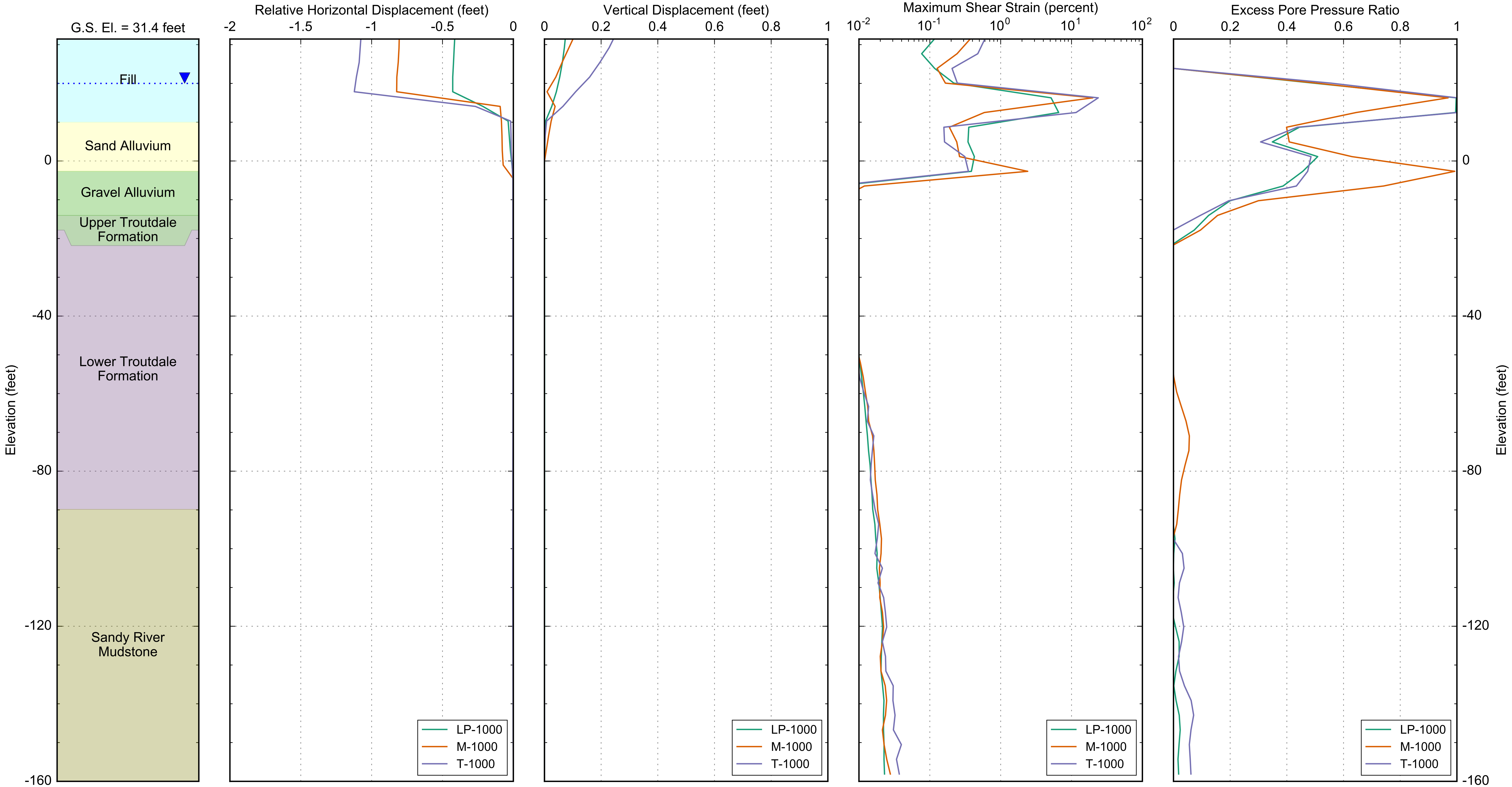
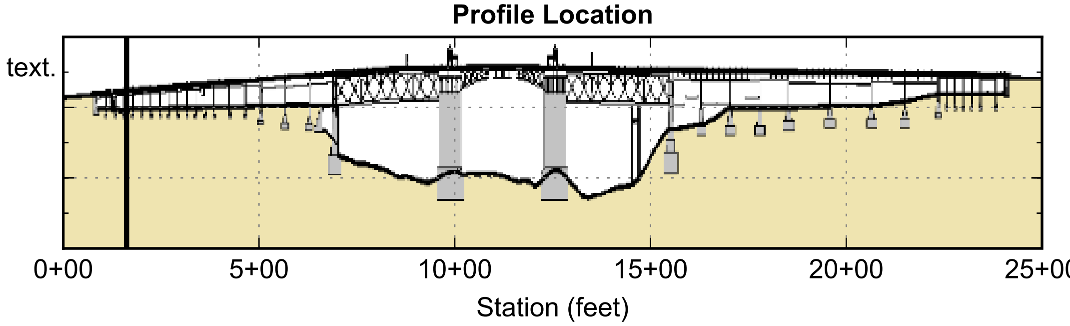


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



Burnside Bridge
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Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 4

July 2017 | 24-1-04065-005

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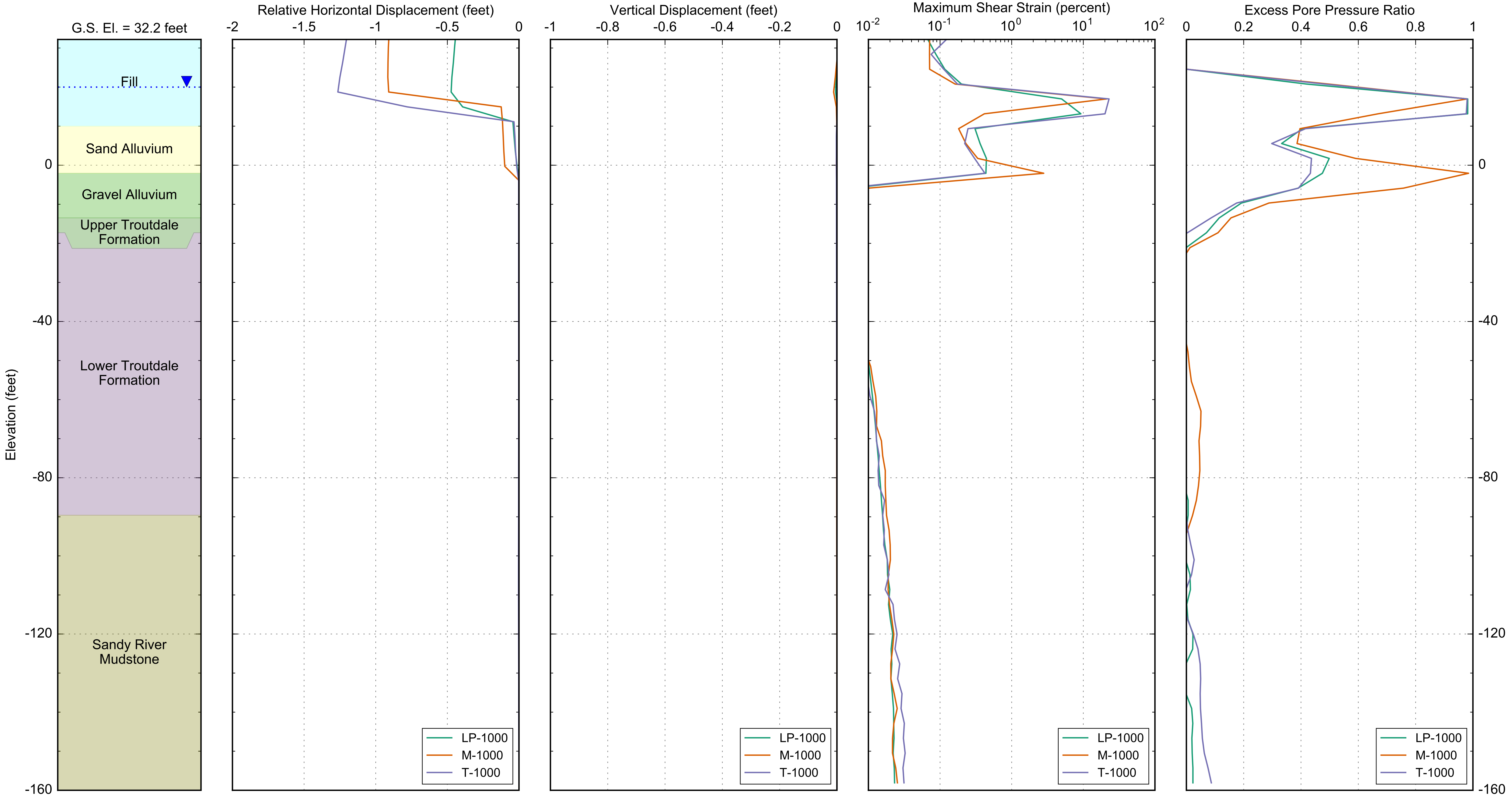
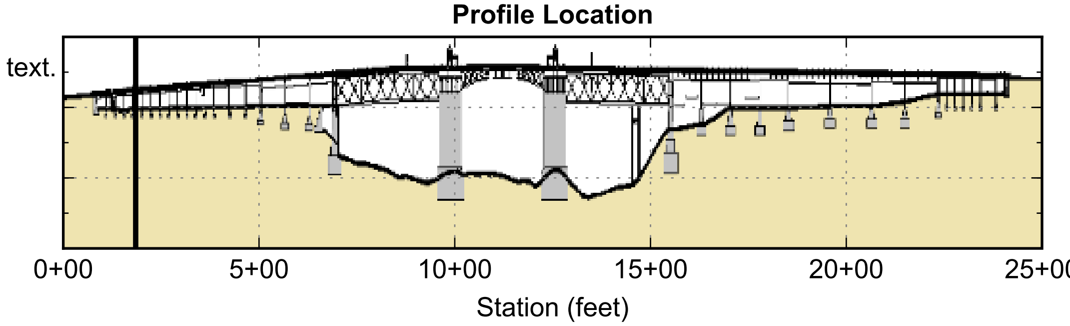
FIG. E-10



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
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Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 5

July 2017 24-1-04065-005

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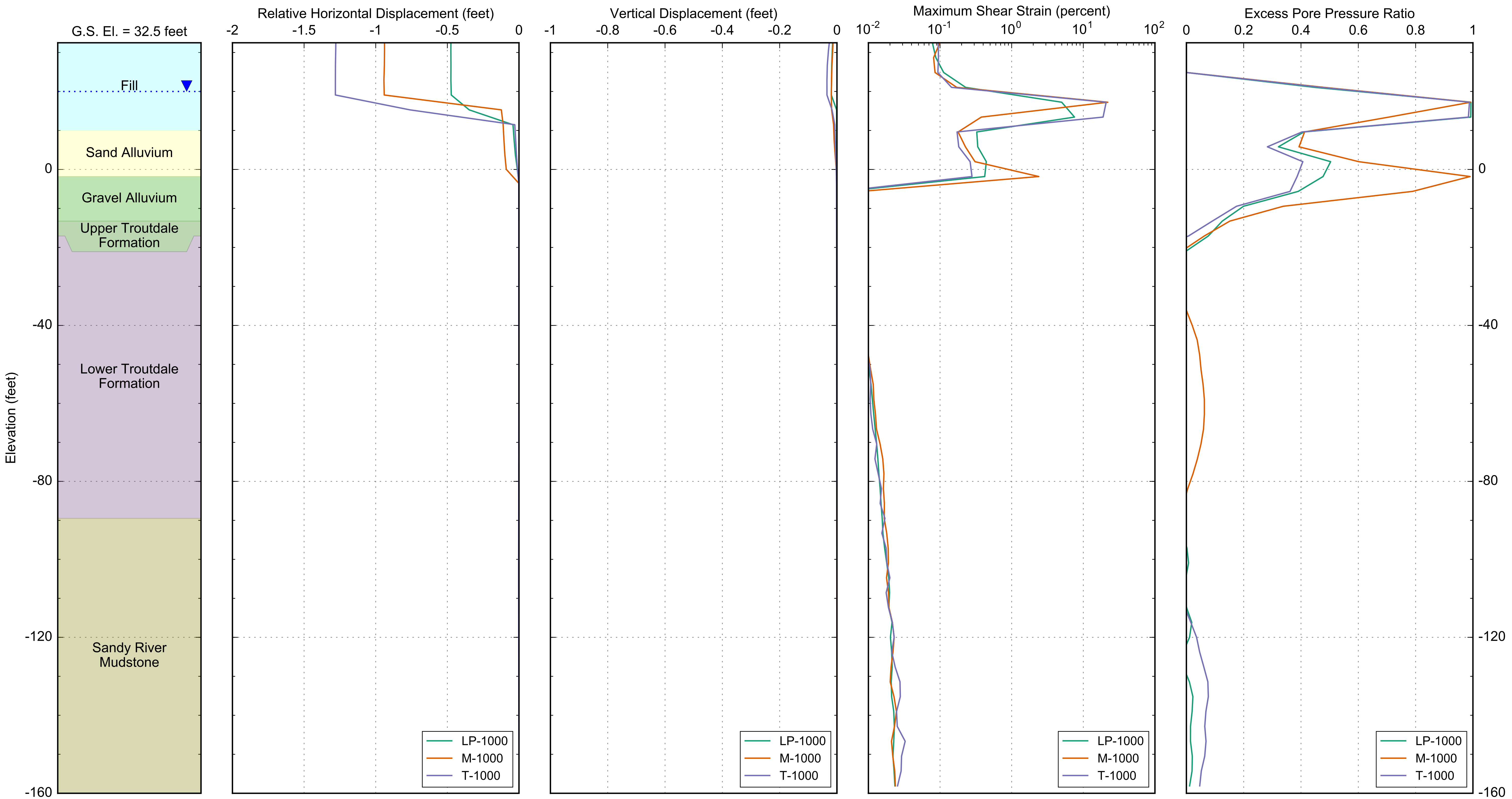
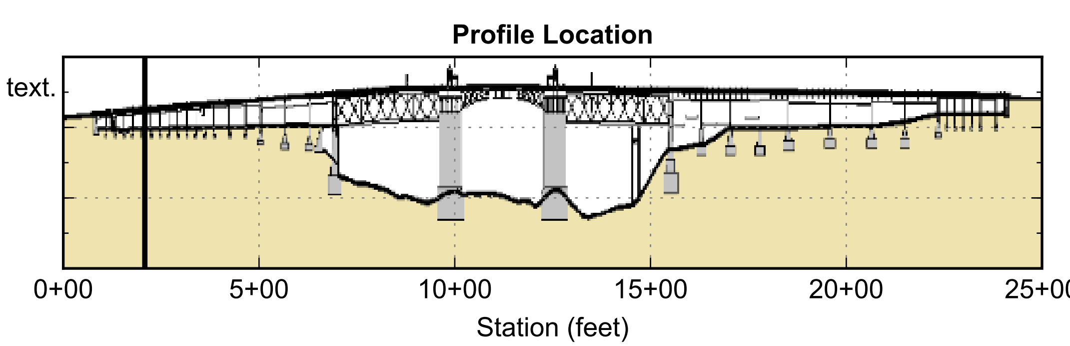
FIG. E-11



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



Burnside Bridge
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Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 6

July 2017 24-1-04065-005

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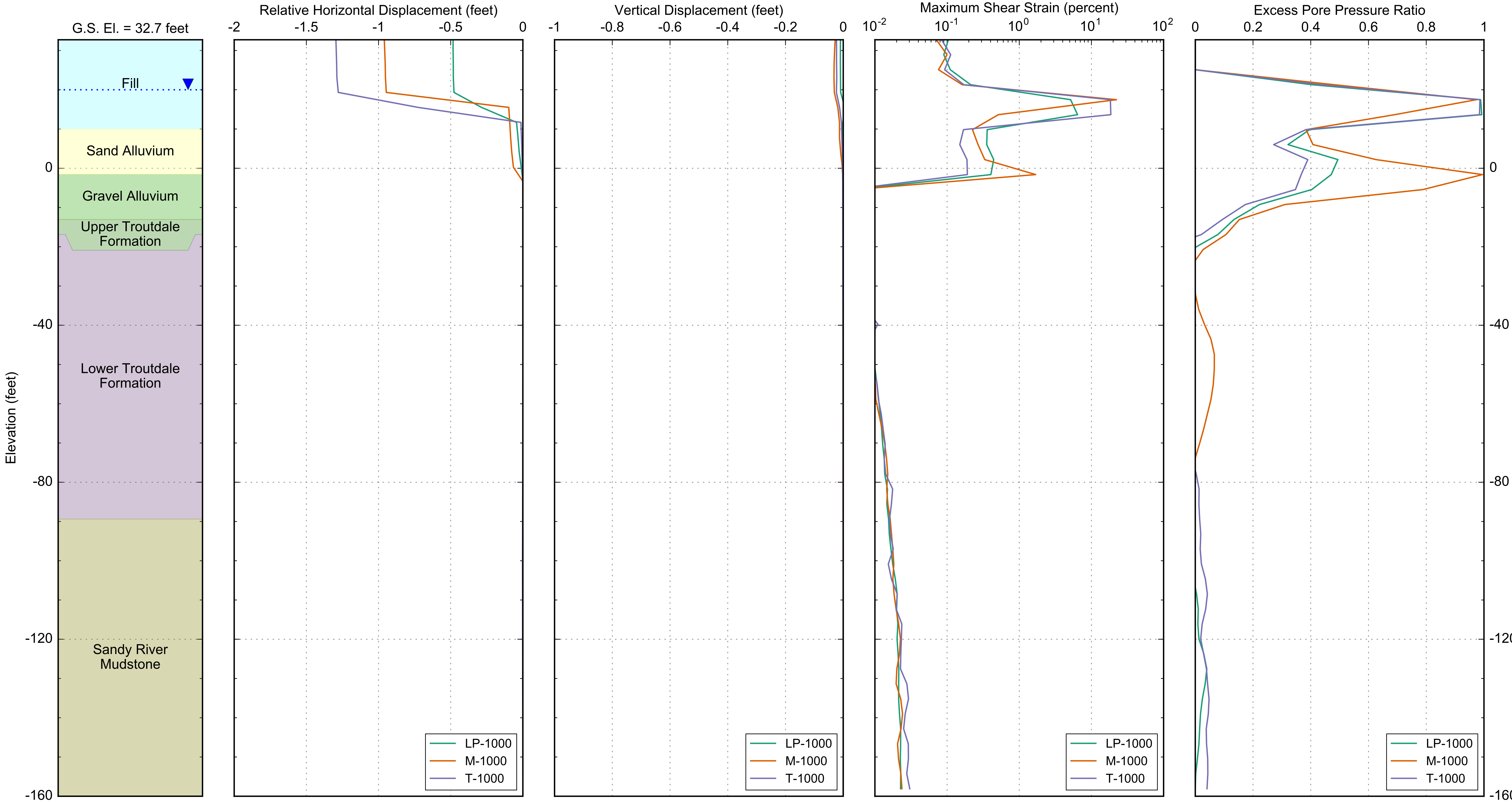
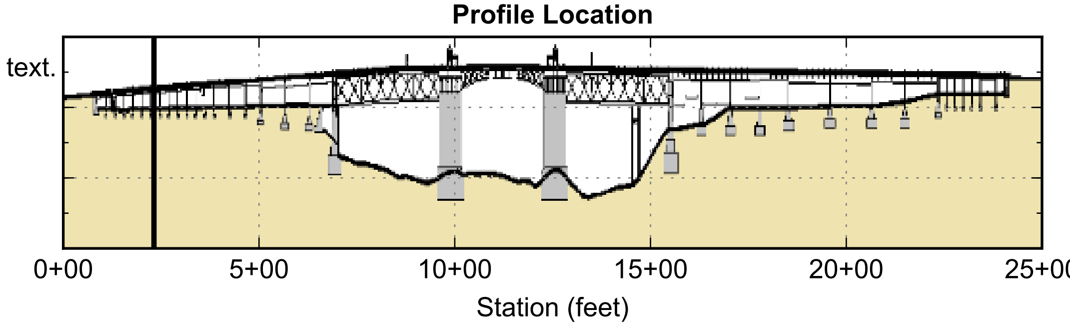
FIG. E-12



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 7

July 2017 | 24-1-04065-005

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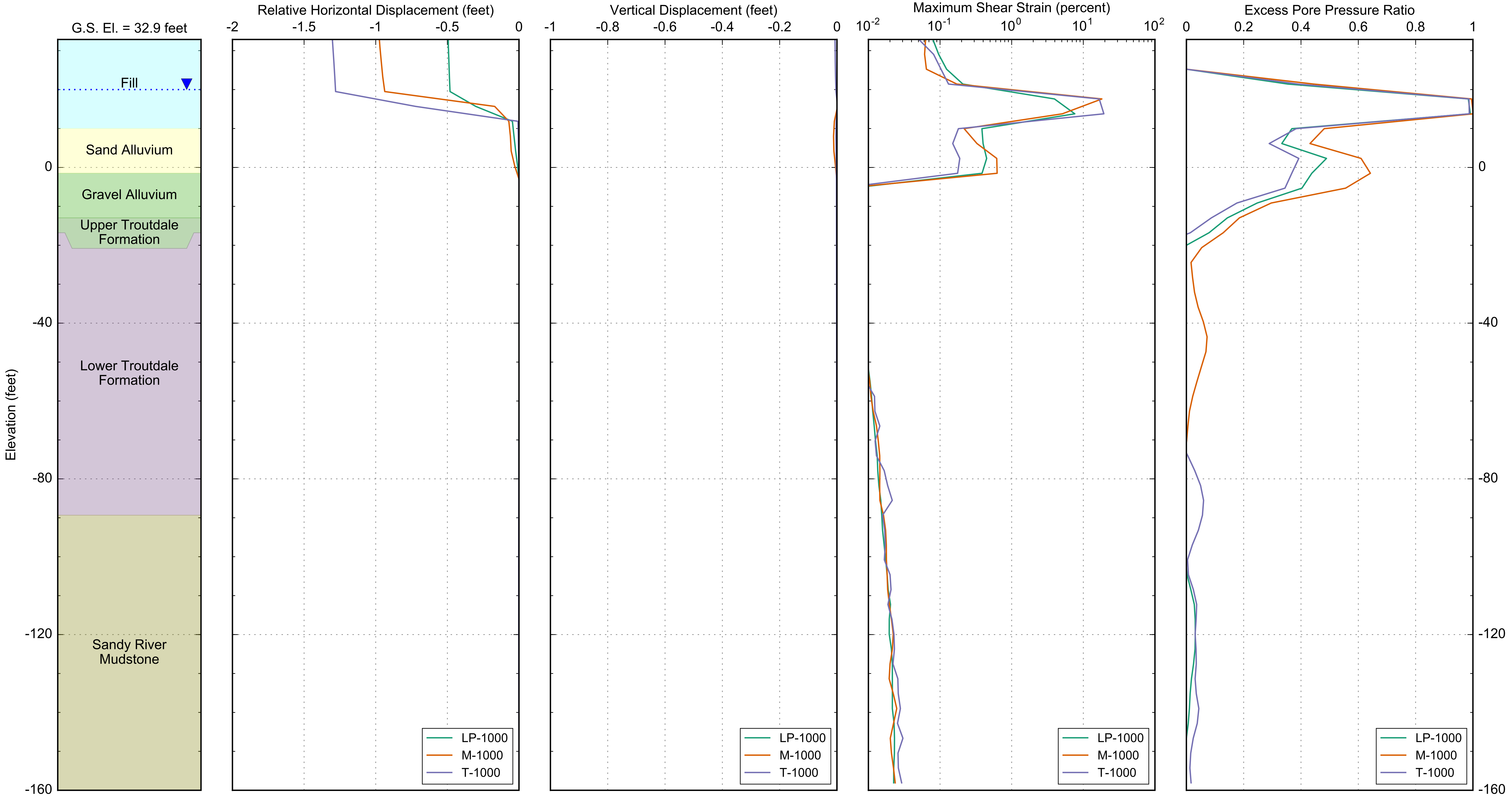
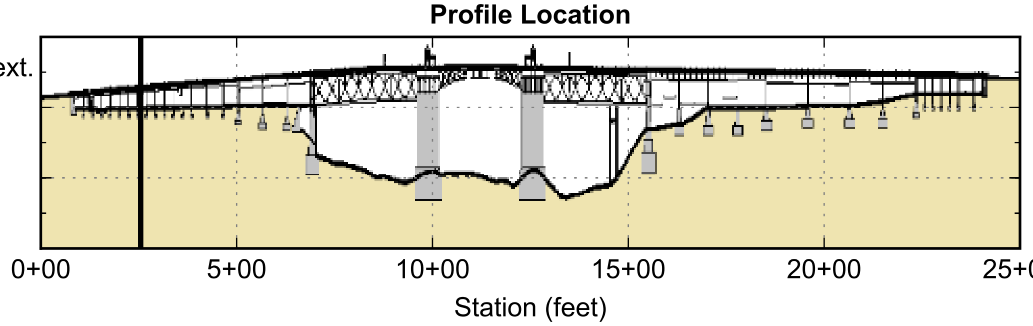
FIG. E-13



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 8

July 2017 24-1-04065-005

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FIG. E-14

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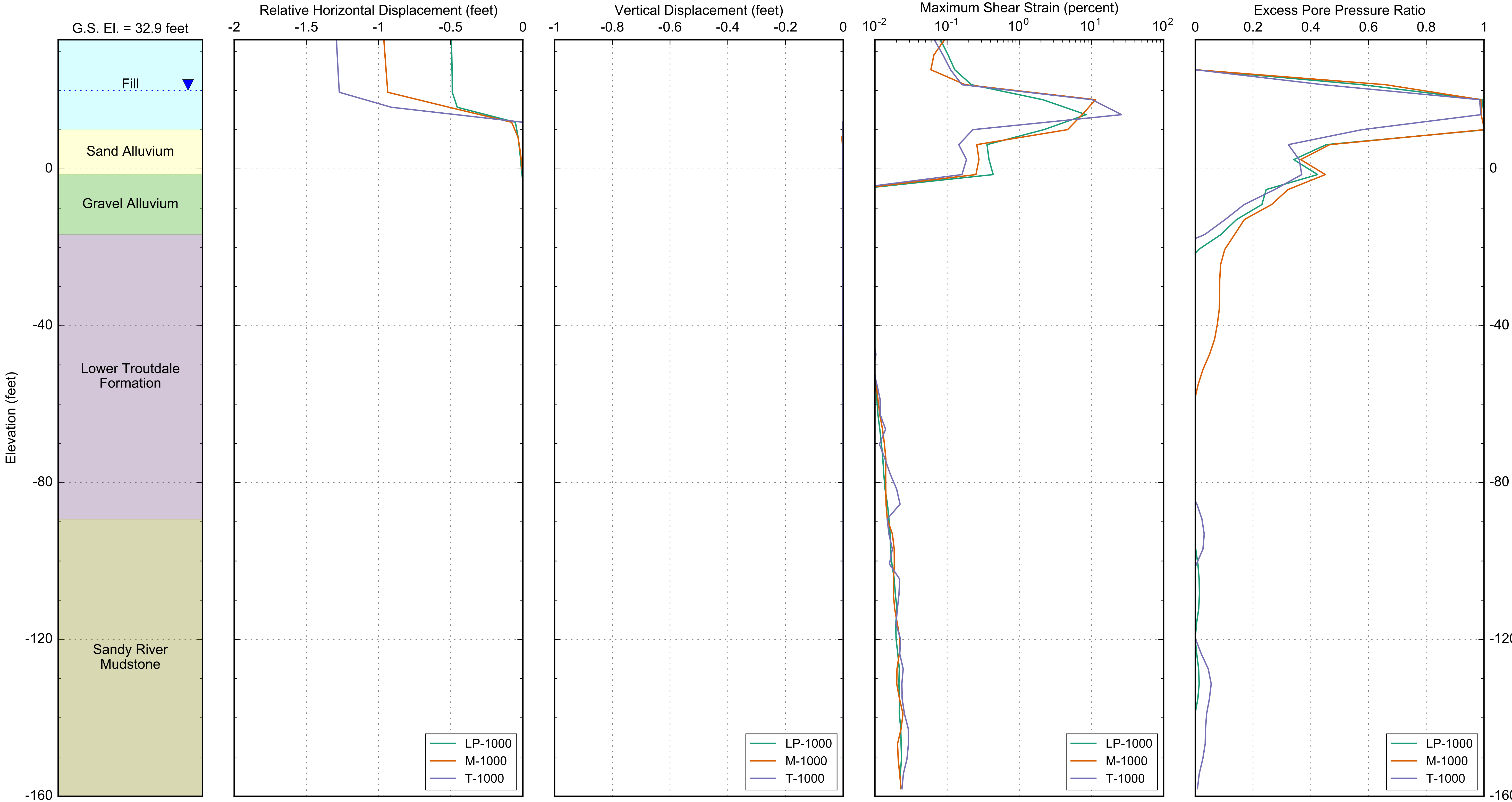
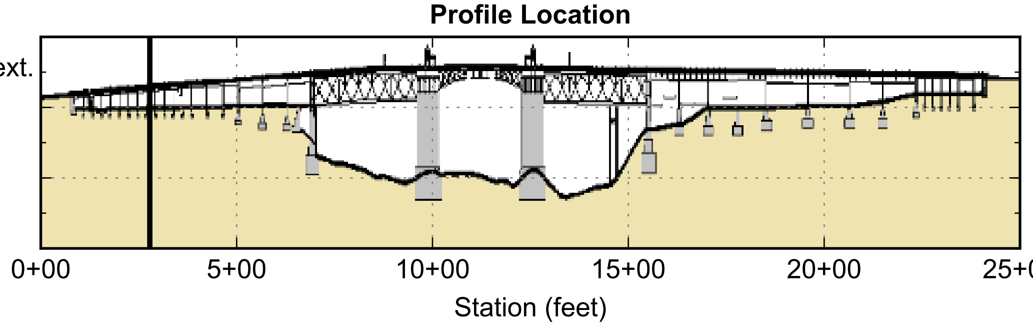


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 9

July 2017 | 24-1-04065-005

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FIG. E-15

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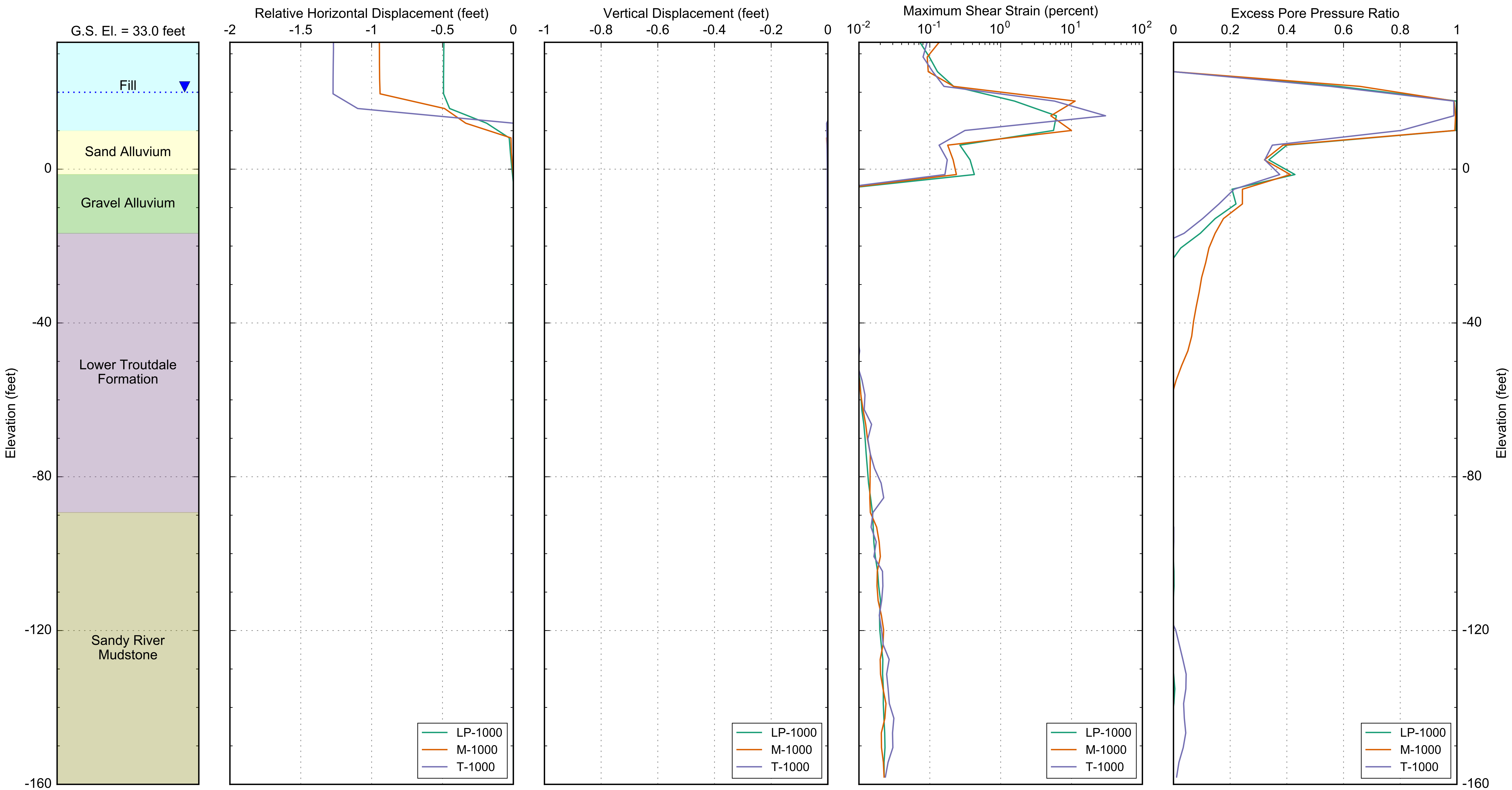
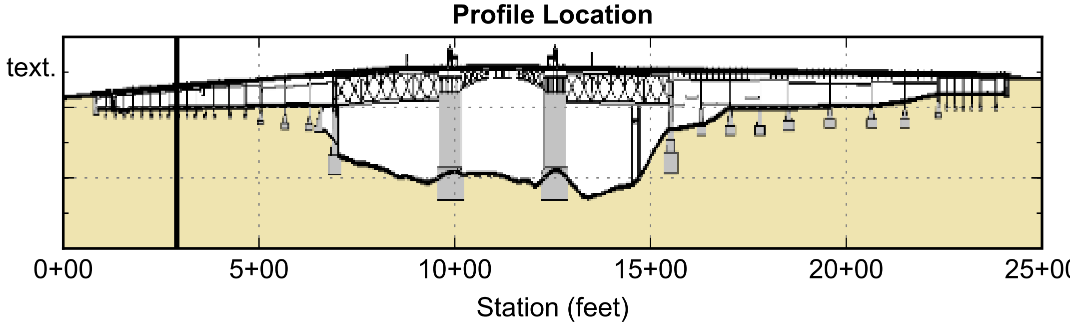


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 10

July 2017 | 24-1-04065-005

SHANNON & WILSON, INC.
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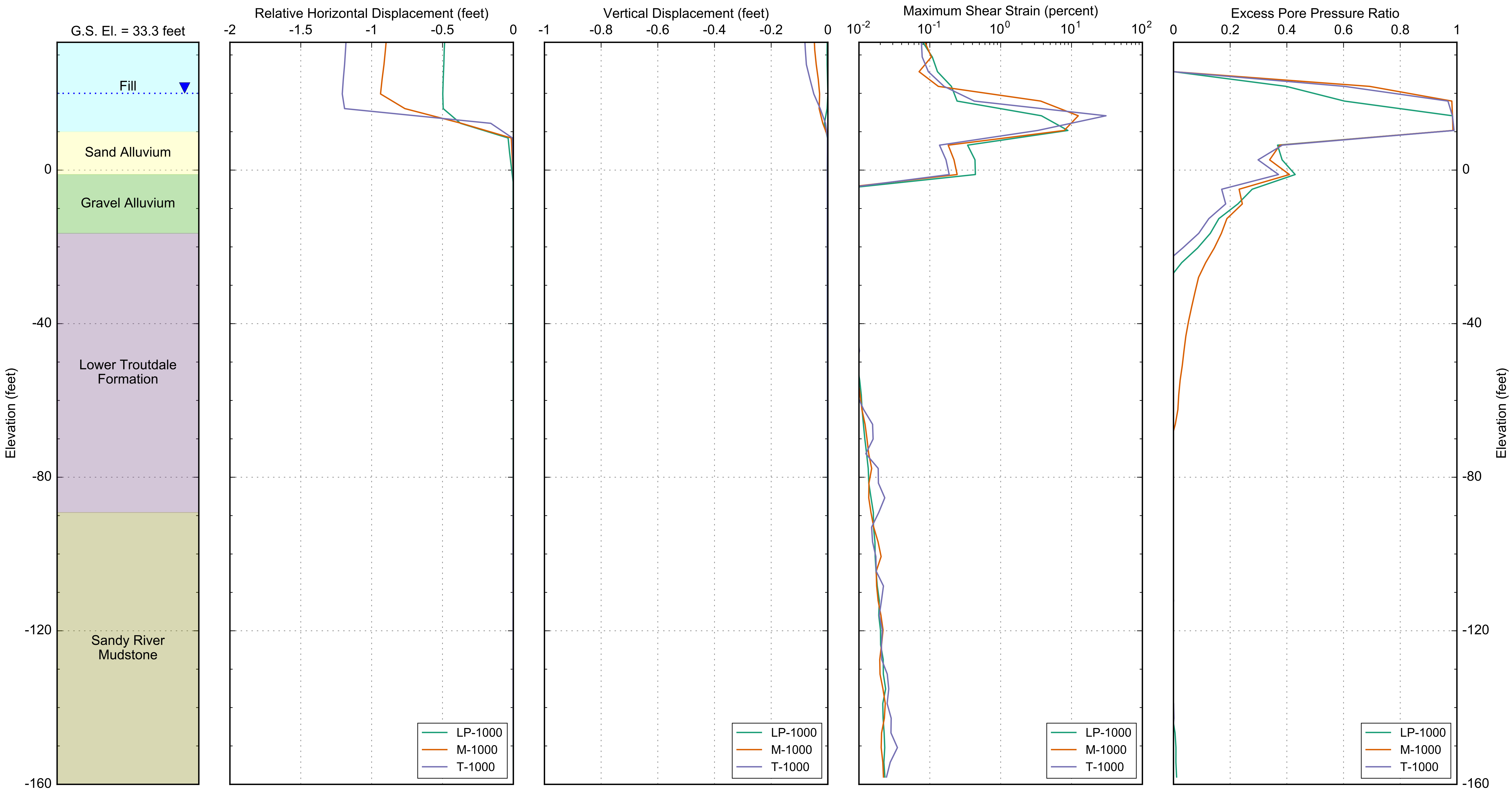
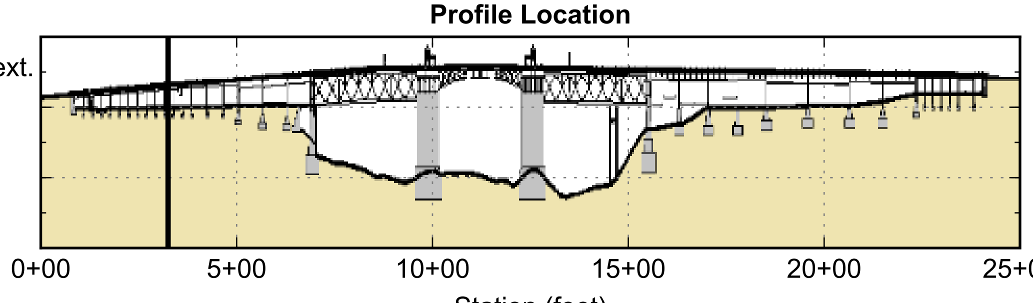
FIG. E-16



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 11

July 2017 24-1-04065-005

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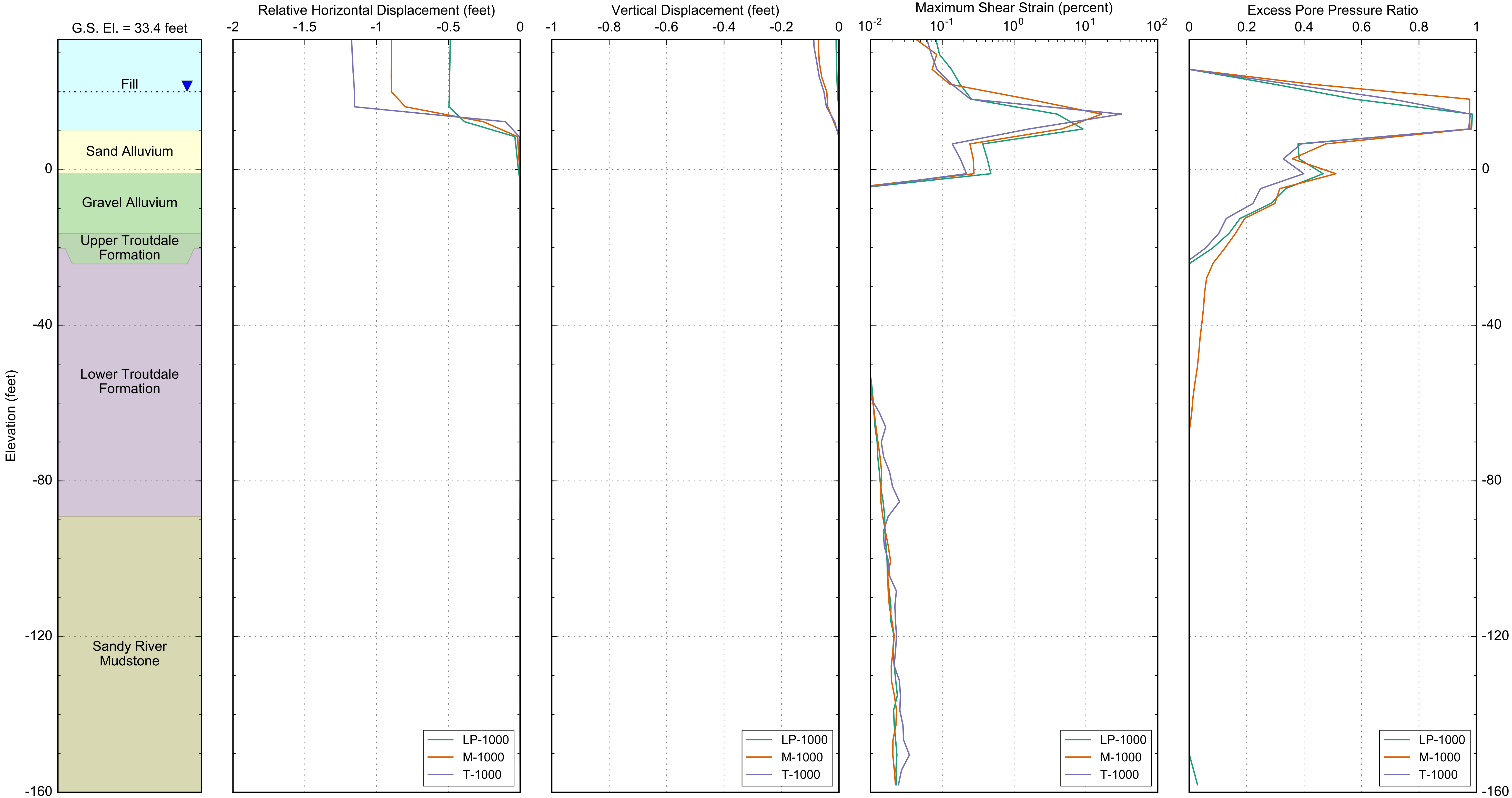
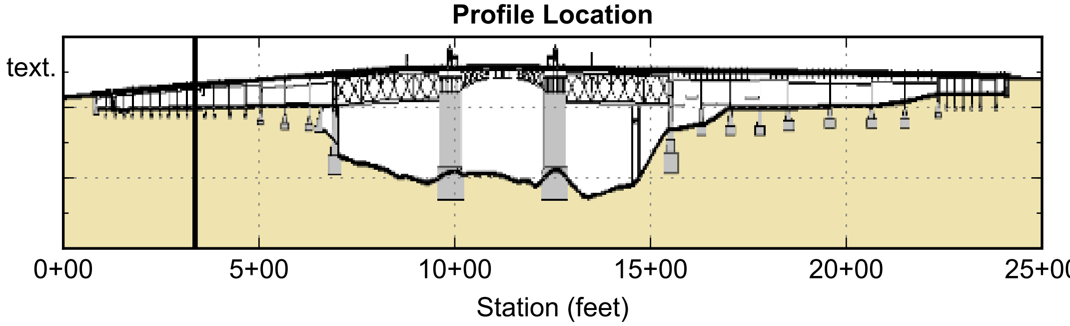
FIG. E-17



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 12

July 2017 | 24-1-04065-005

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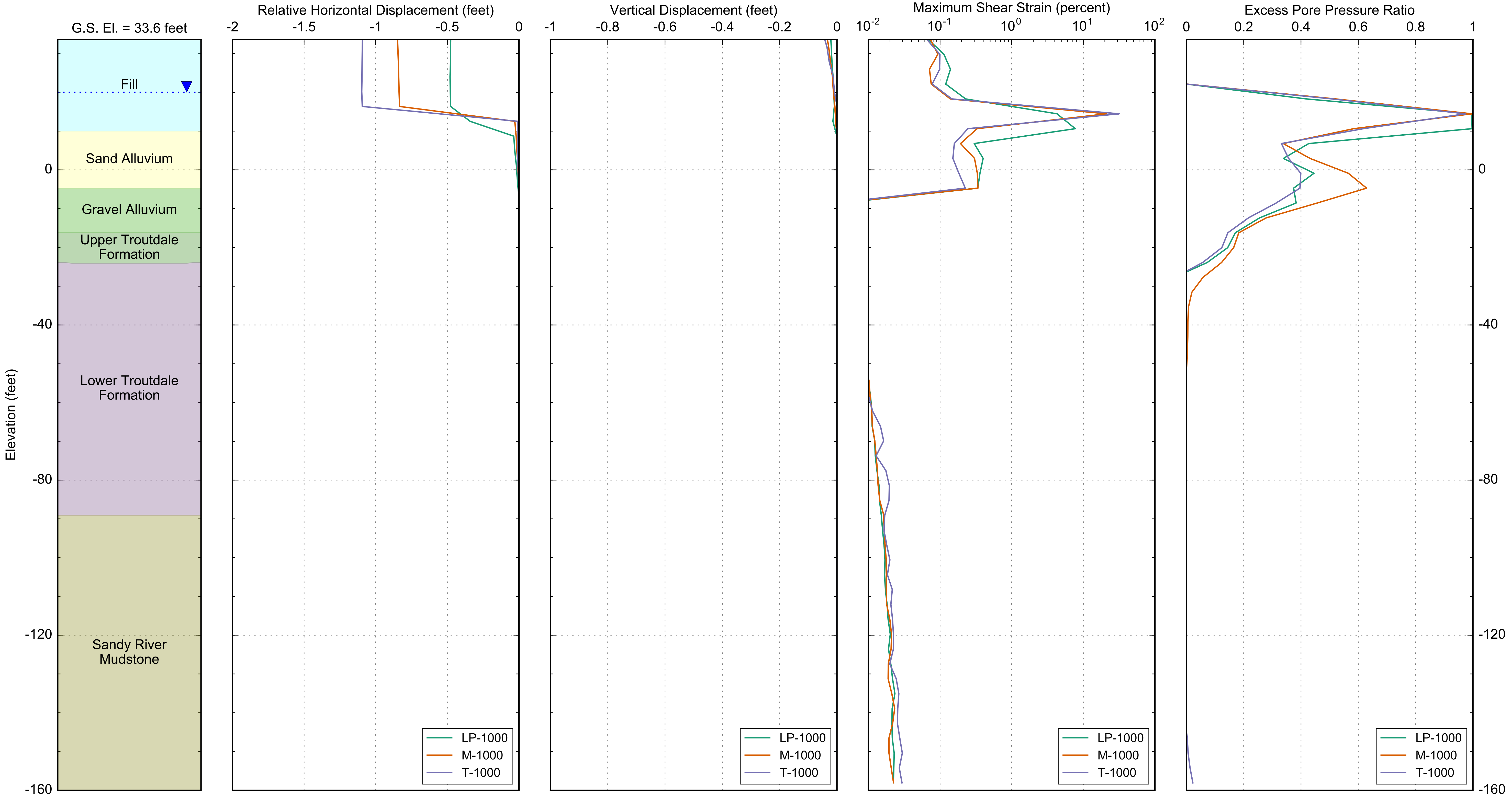
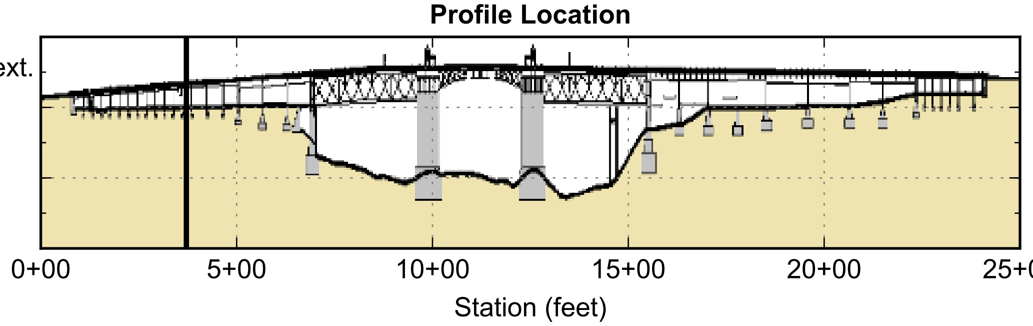
FIG. E-18



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 13

July 2017 24-1-04065-005

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FIG. E-19

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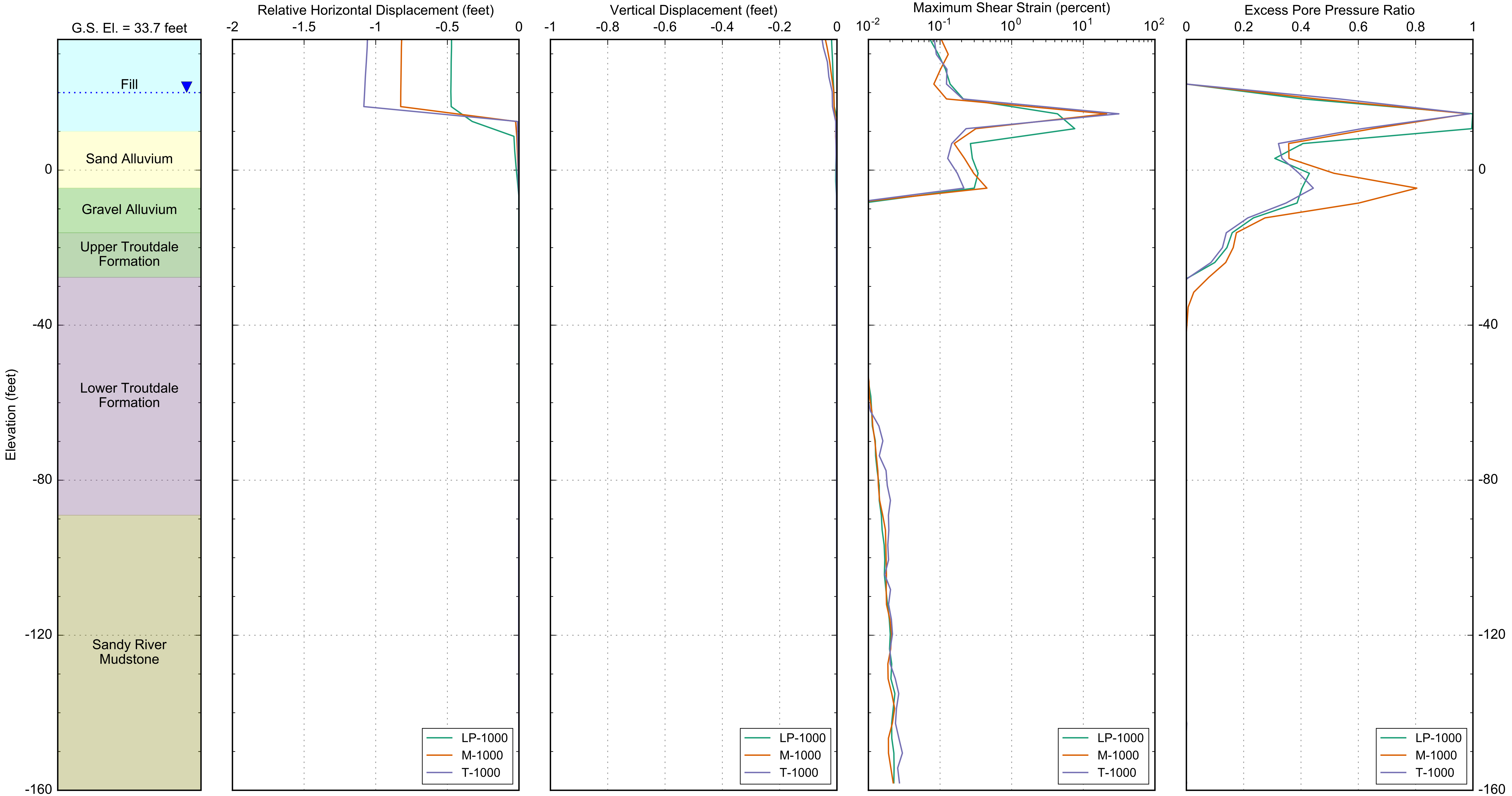
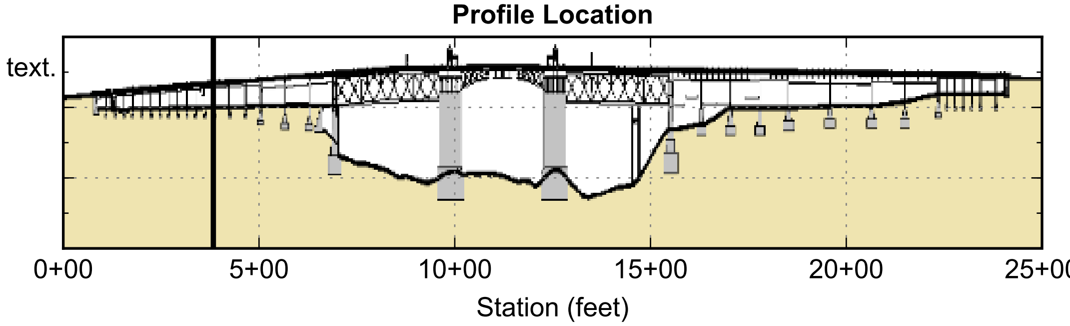


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 14

July 2017 24-1-04065-005

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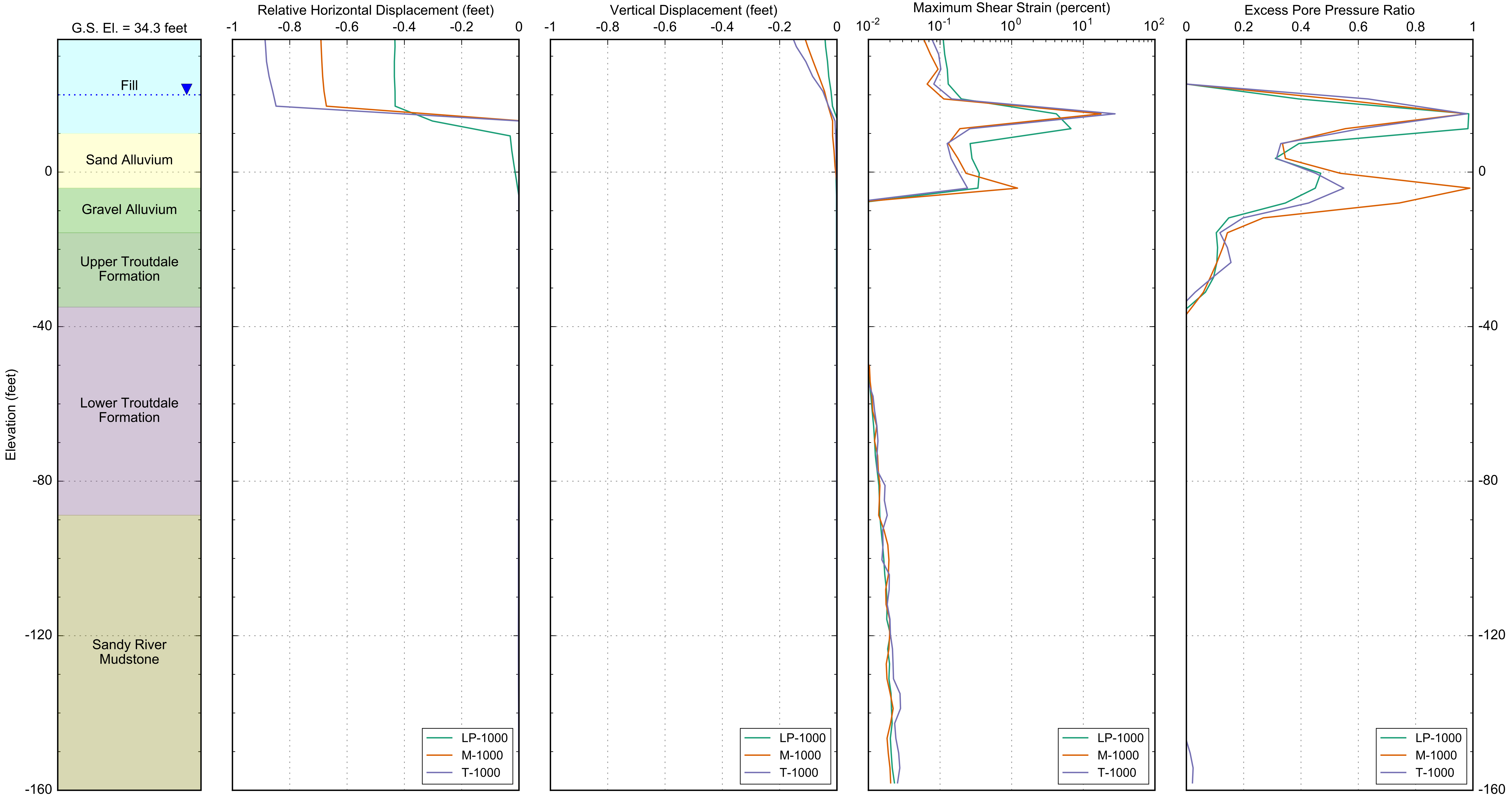
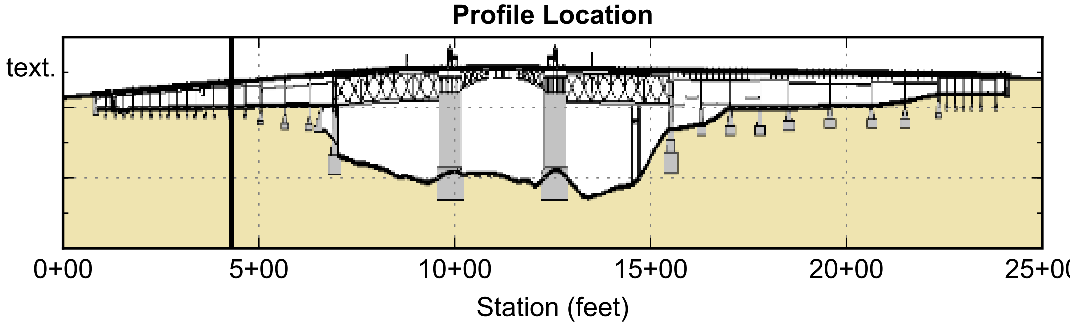
FIG. E-20



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 15

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FIG. E-21

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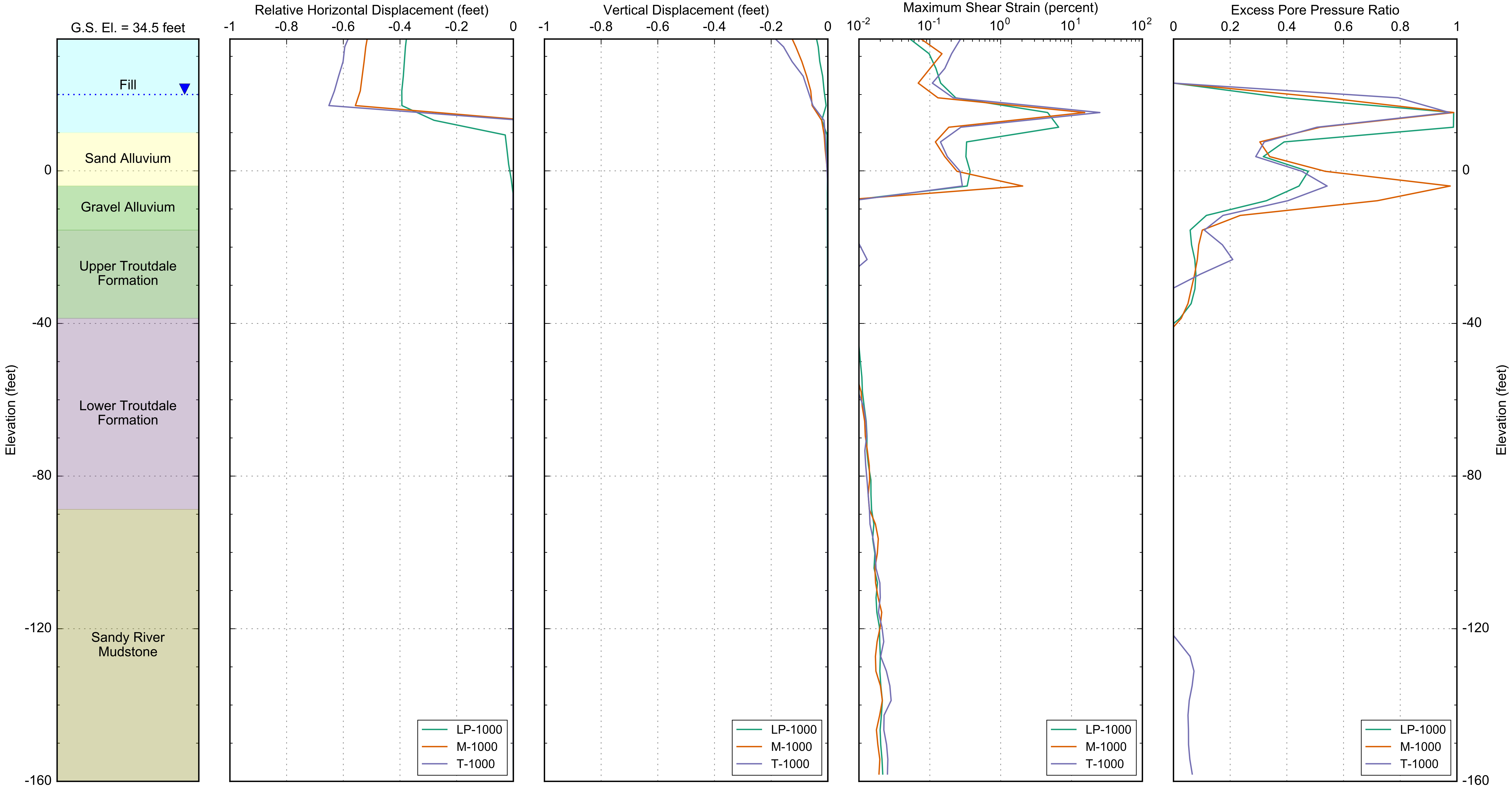
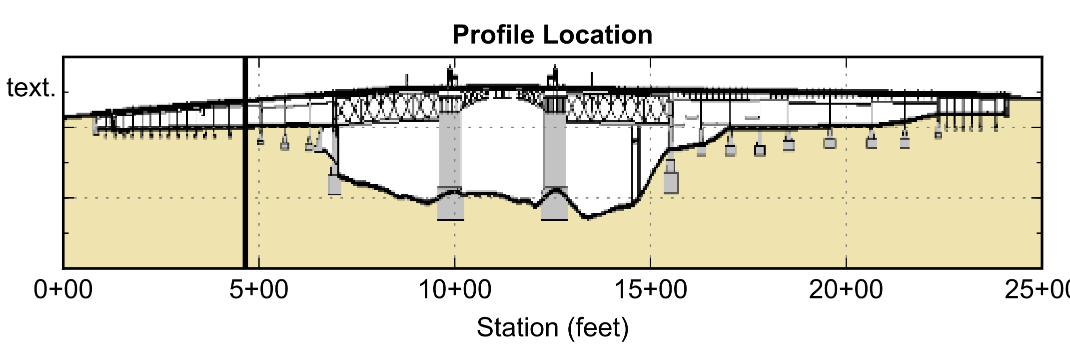


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 16

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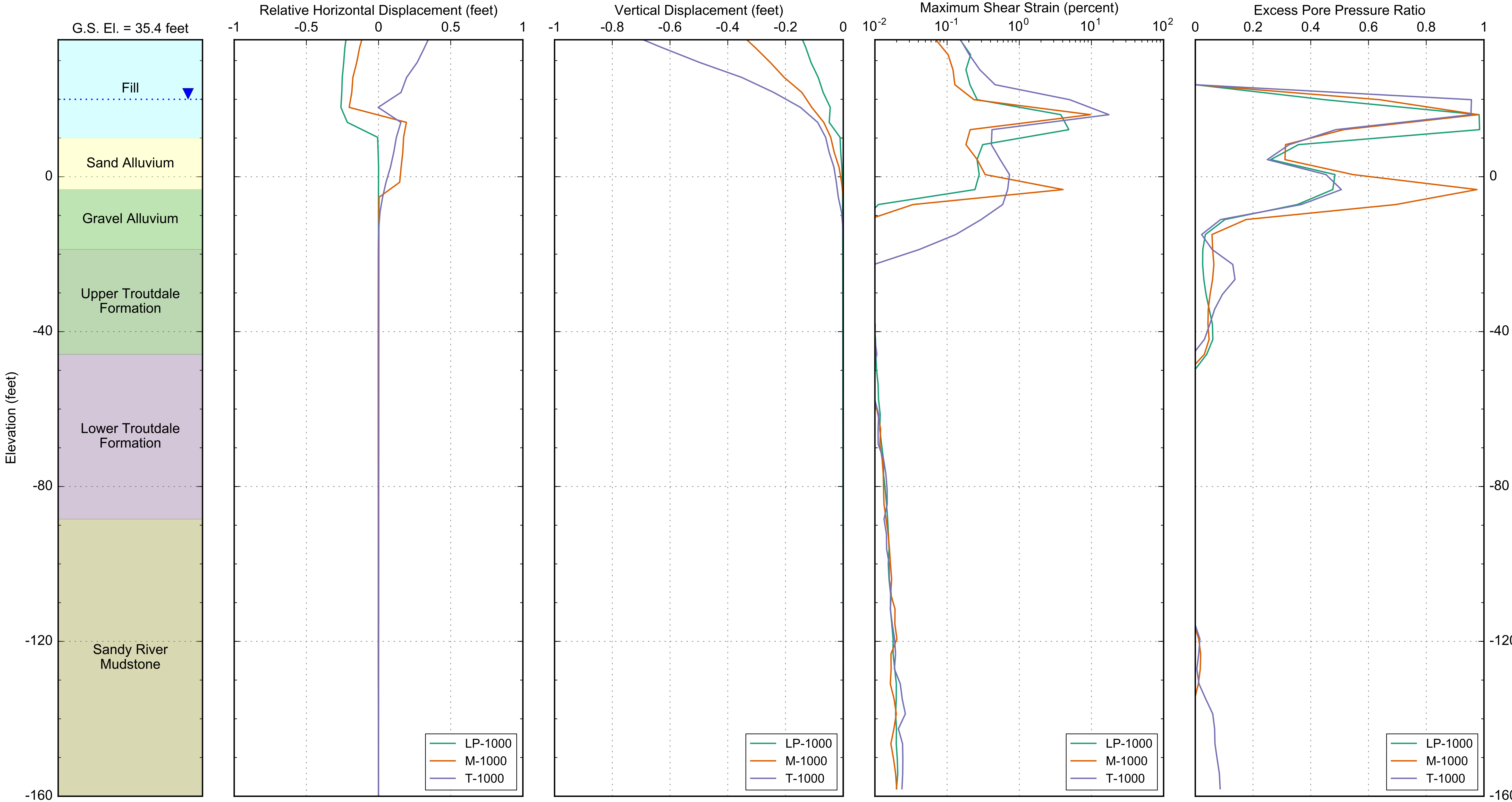
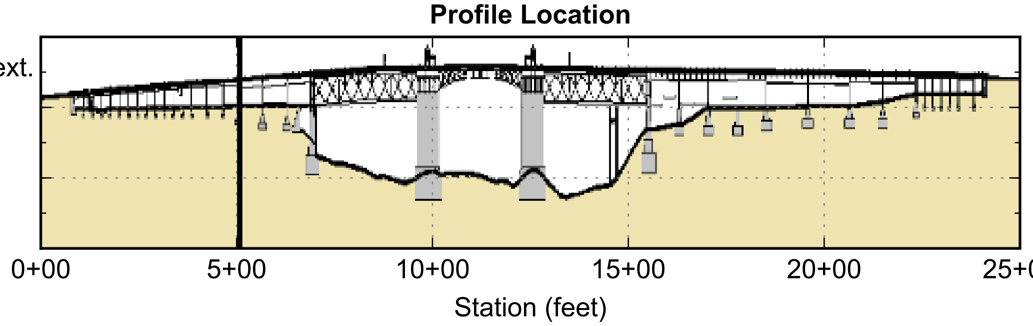
FIG. E-22



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 17

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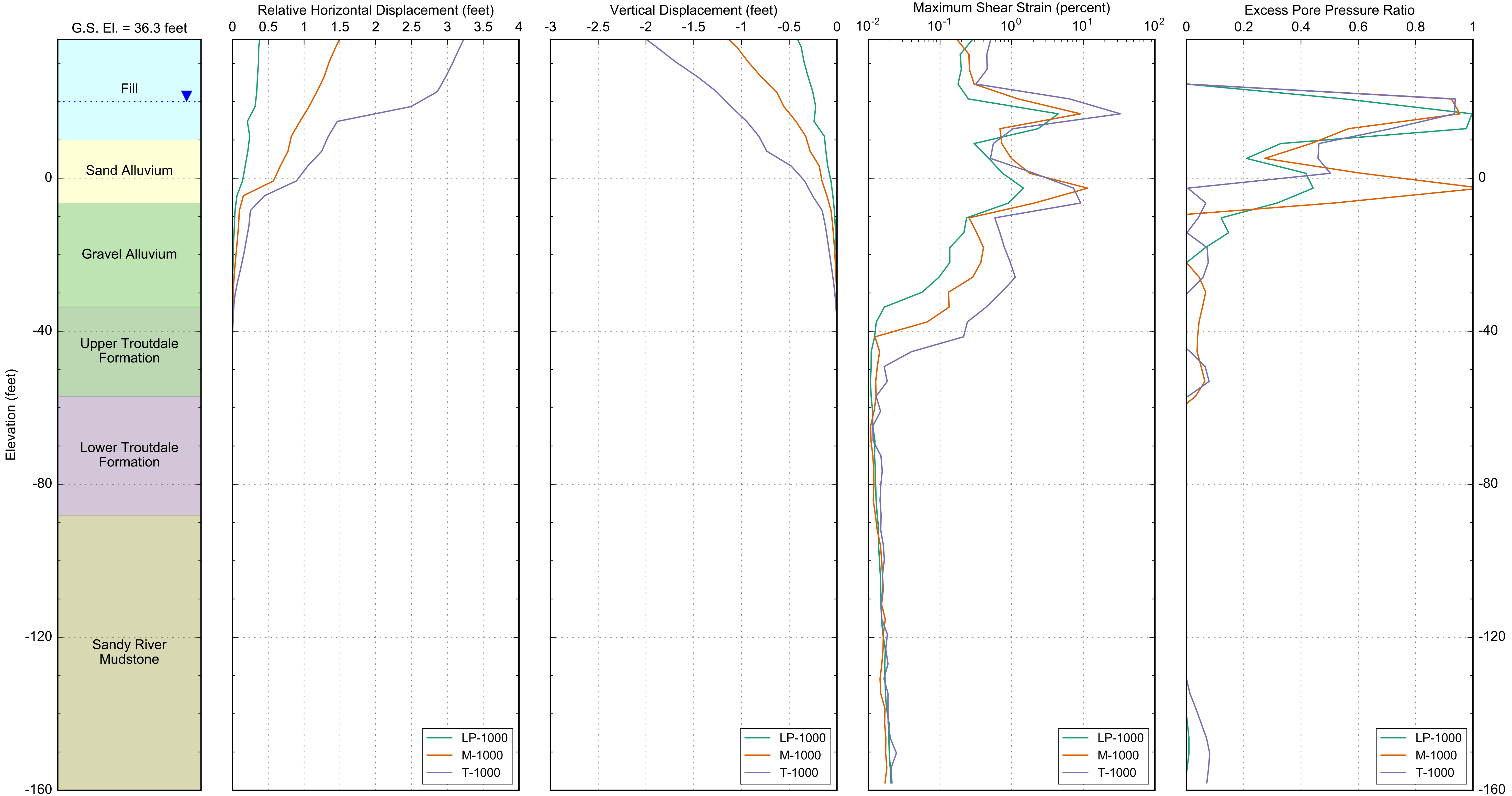
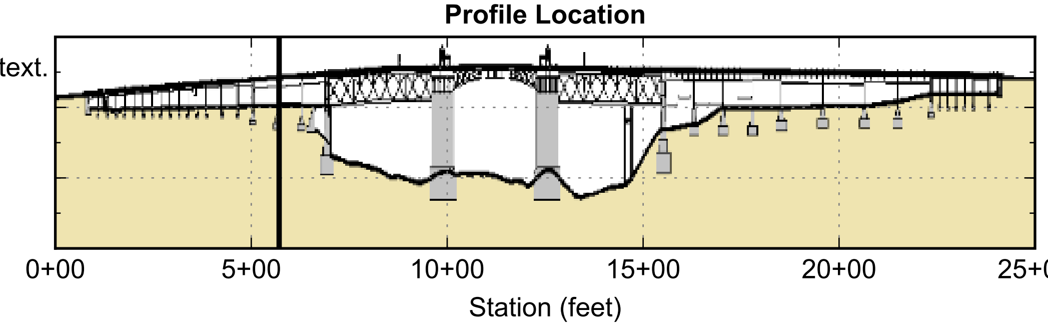
FIG. E-23



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 18

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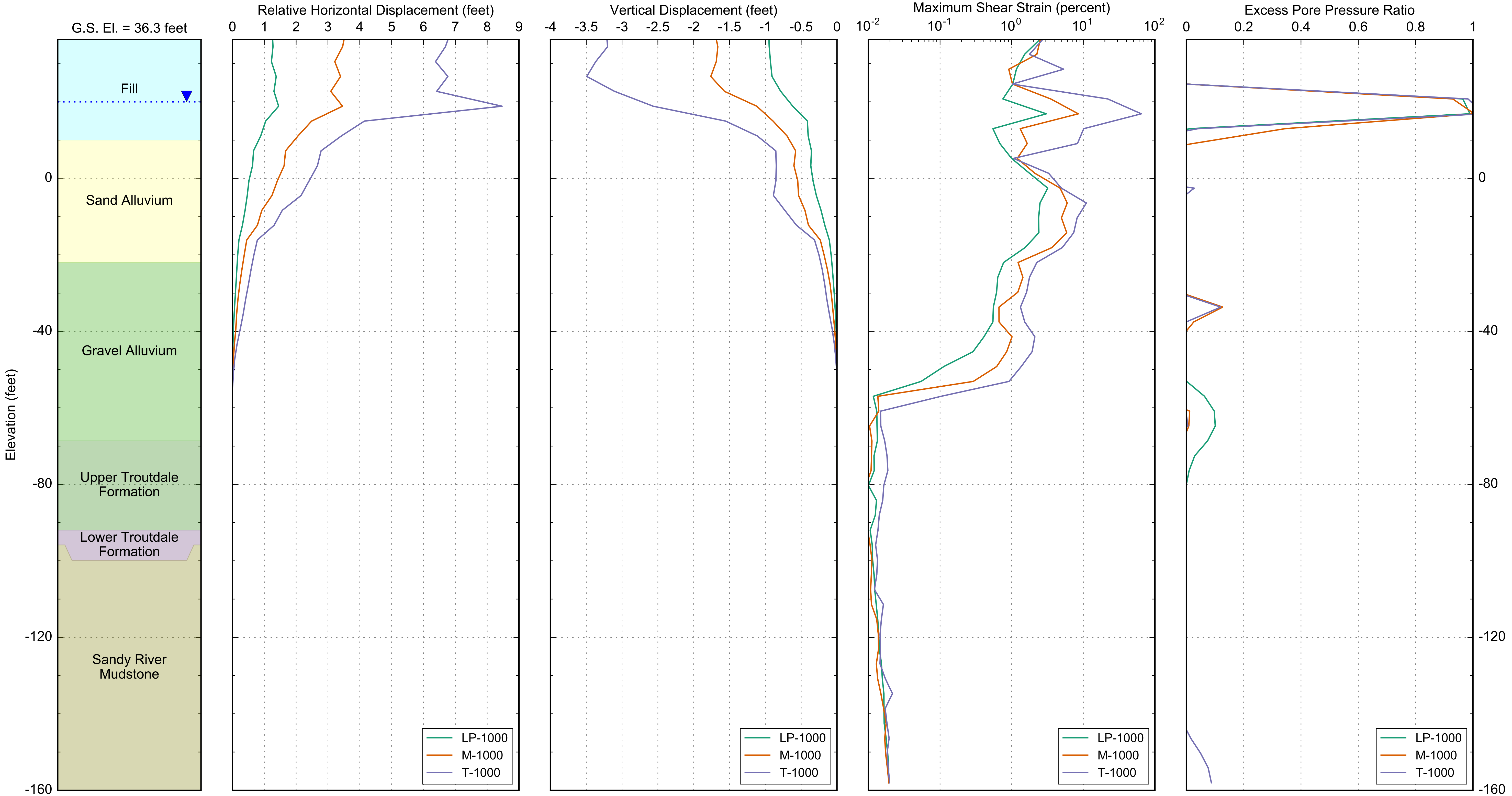
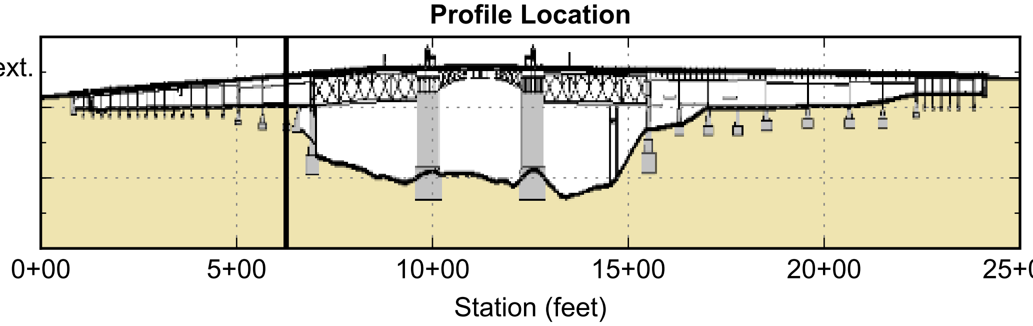
FIG. E-24



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 19

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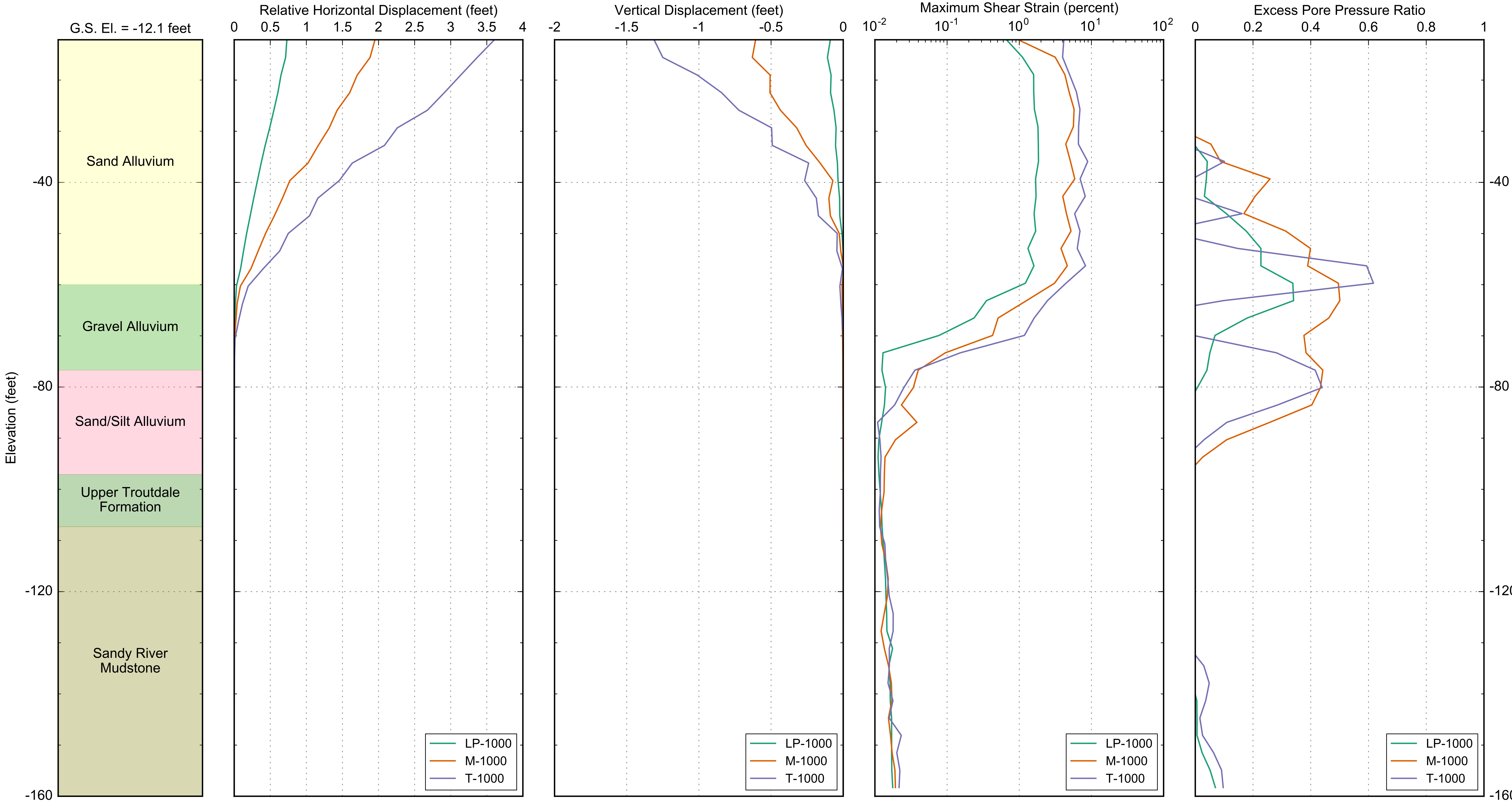
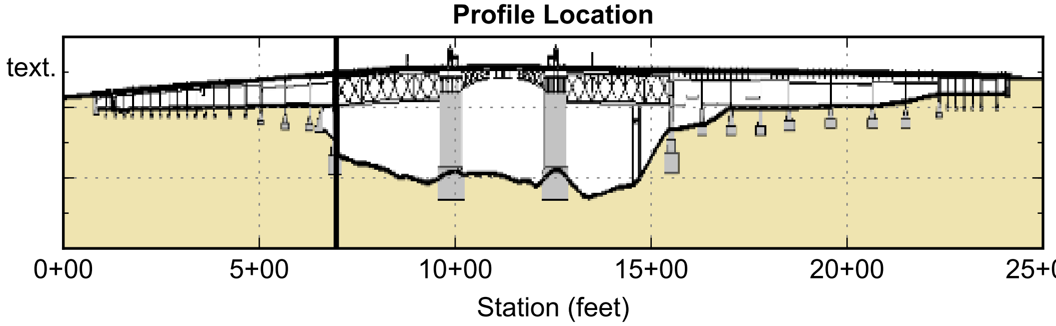
FIG. E-25



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
PIER 1

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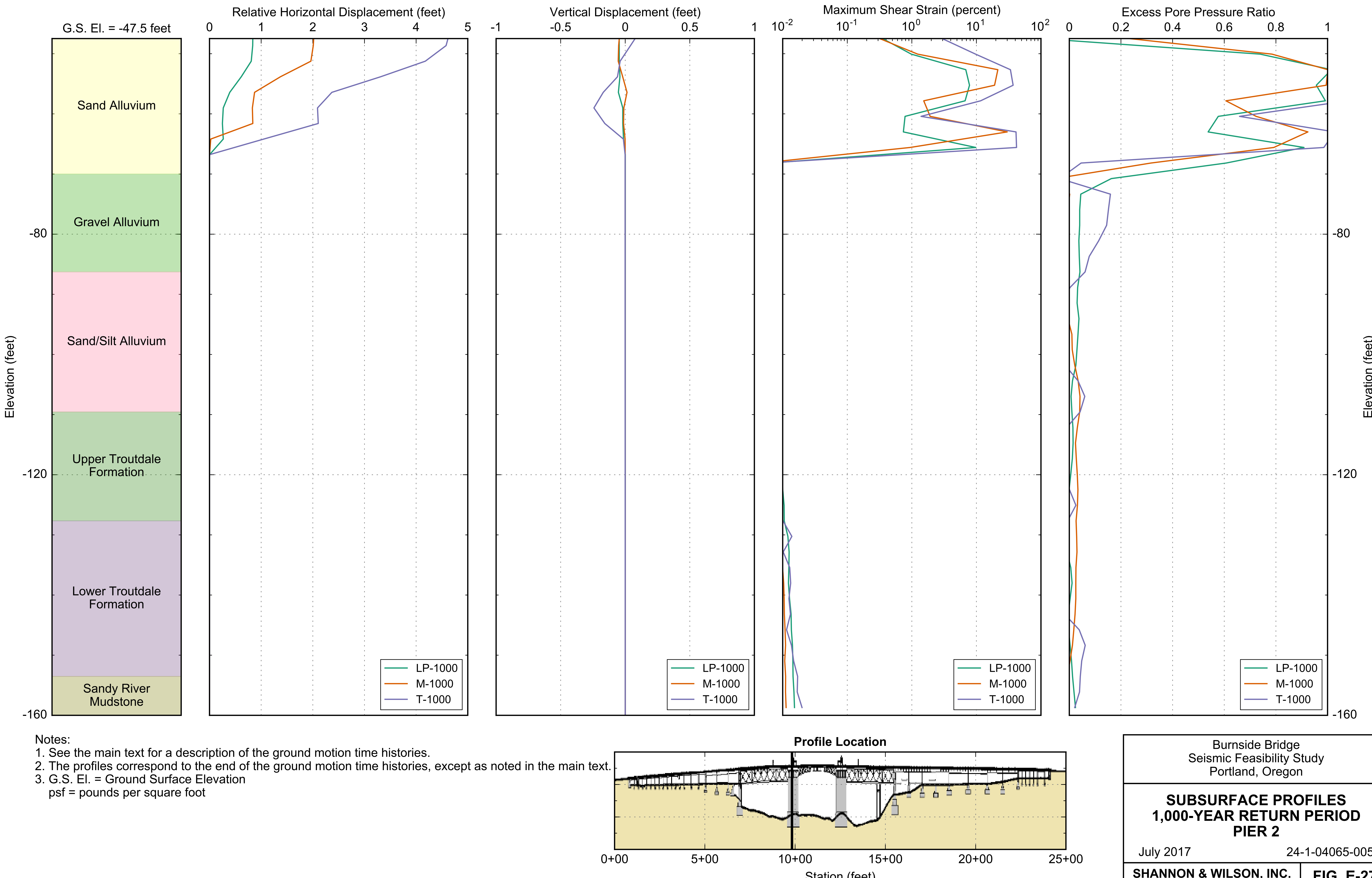
FIG. E-26



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Burnside Bridge
Seismic Feasibility Study
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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
PIER 2

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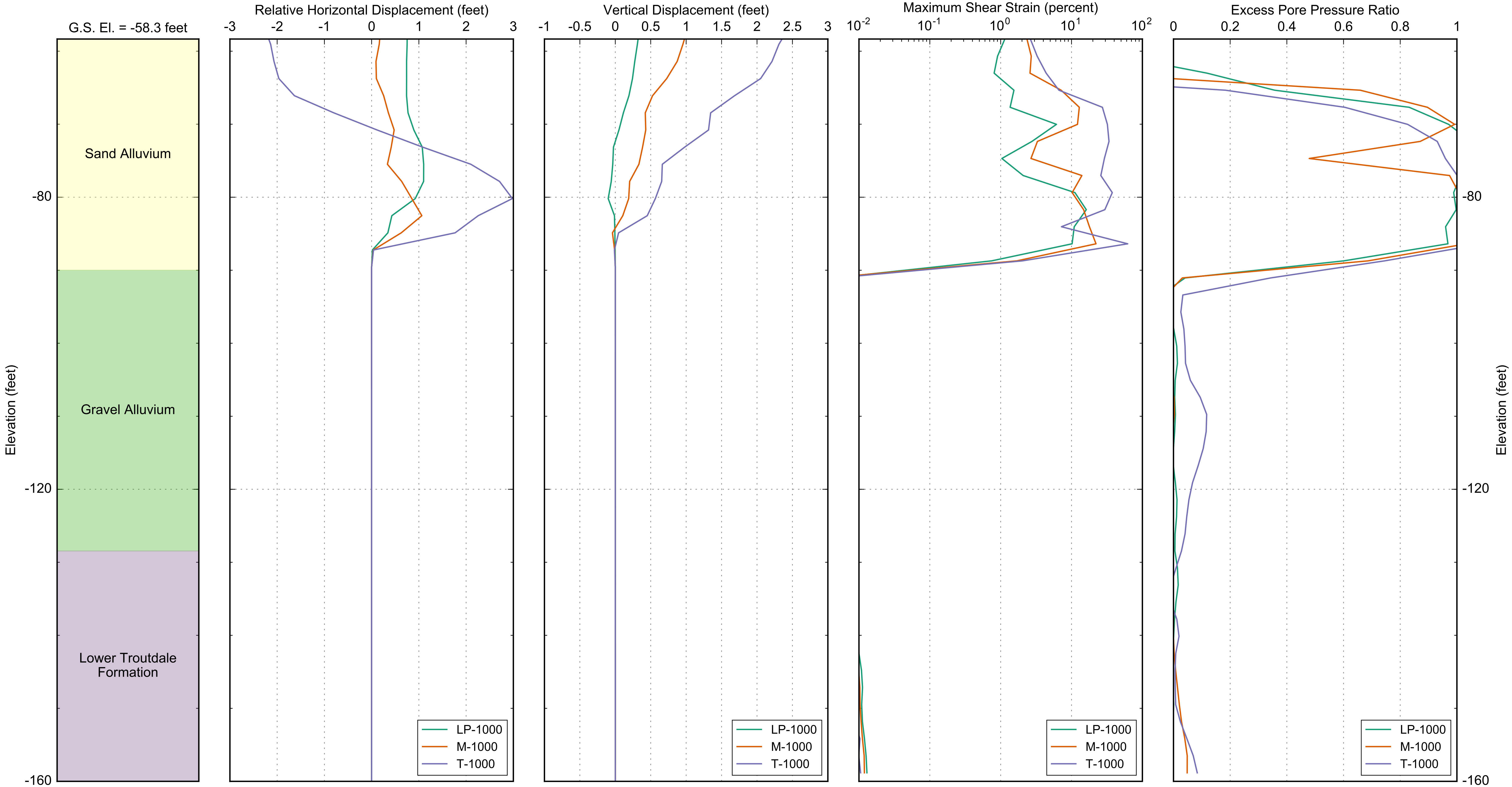
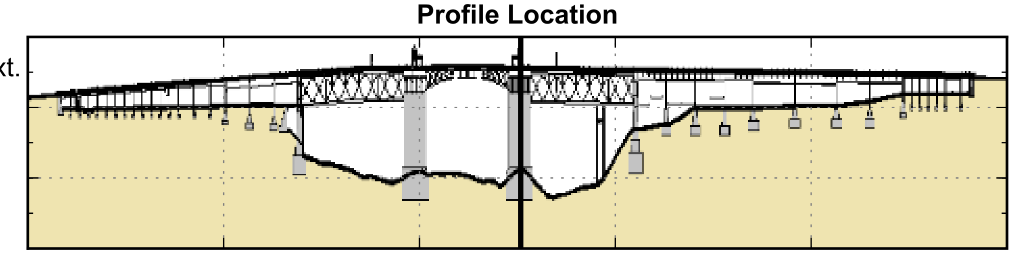
FIG. E-27



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
PIER 3

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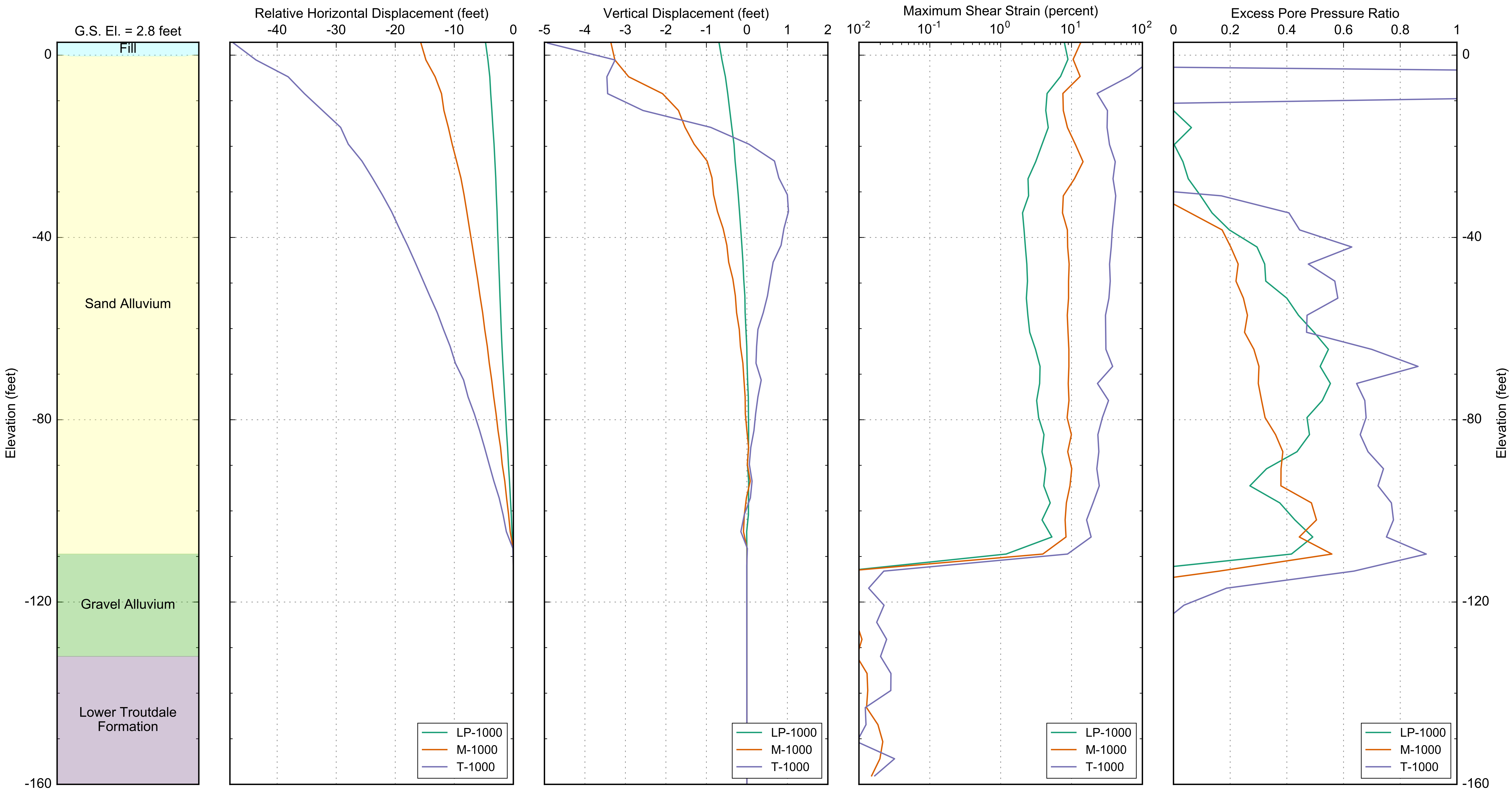
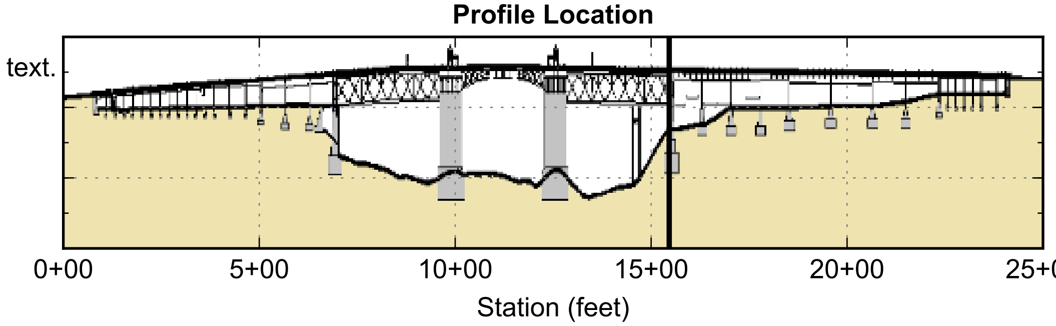
FIG. E-28



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
PIER 4

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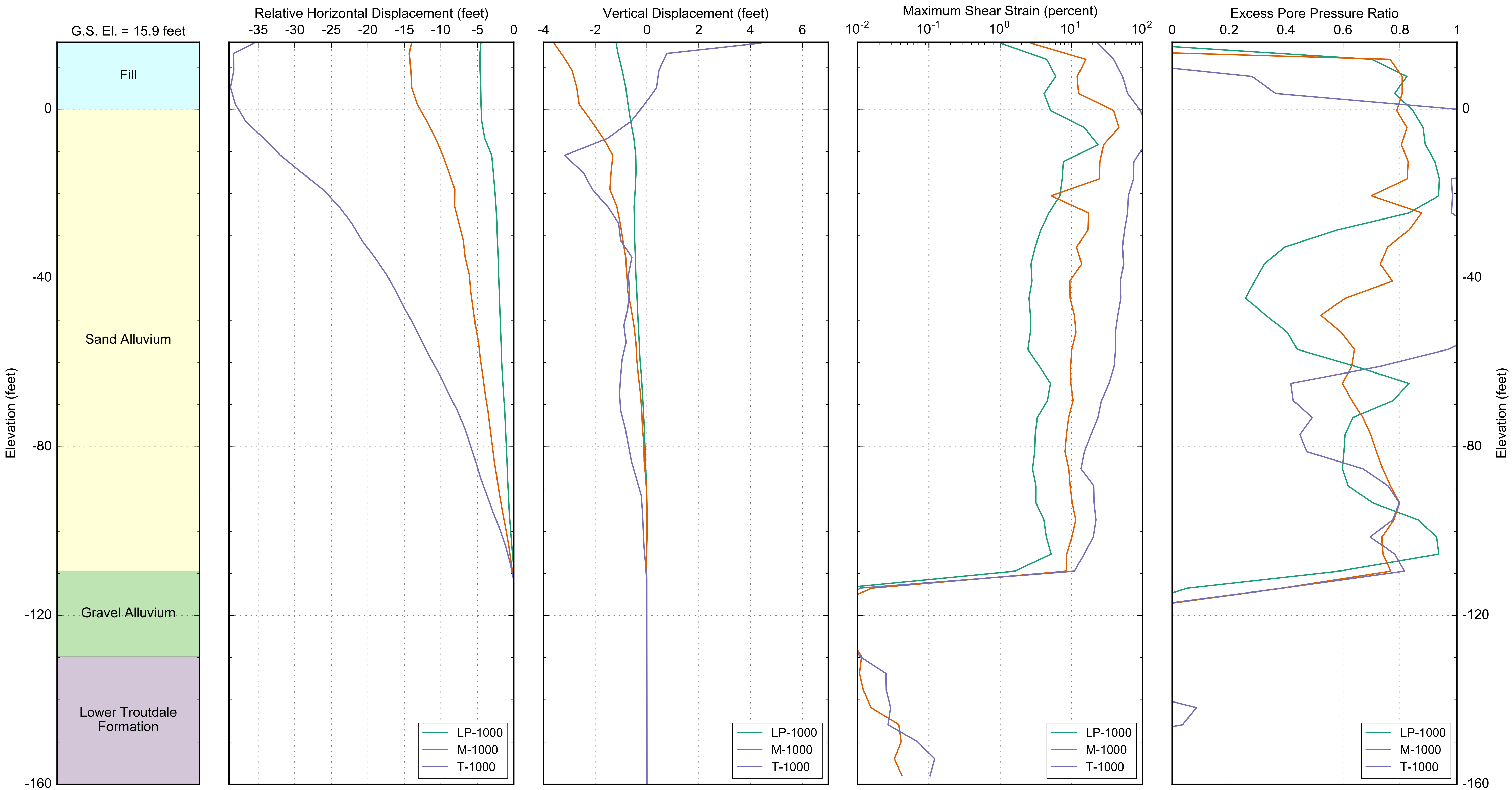
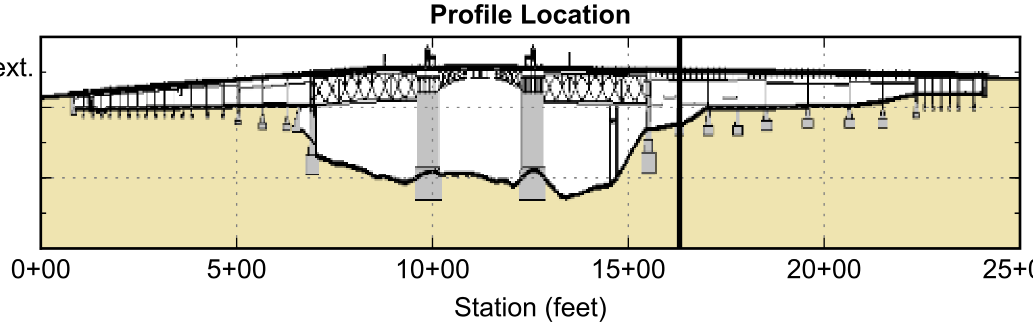
FIG. E-29



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 21

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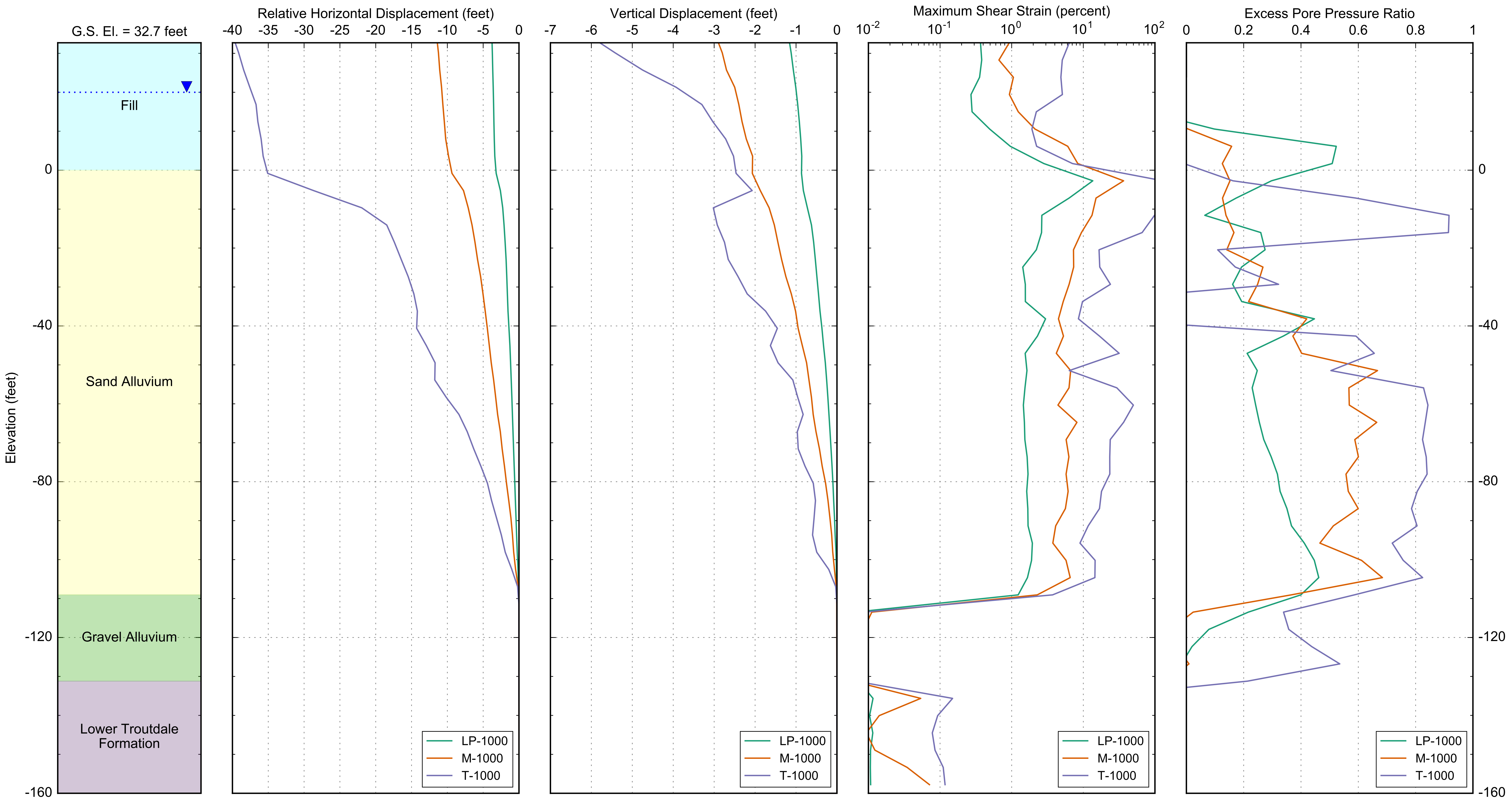
FIG. E-30



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
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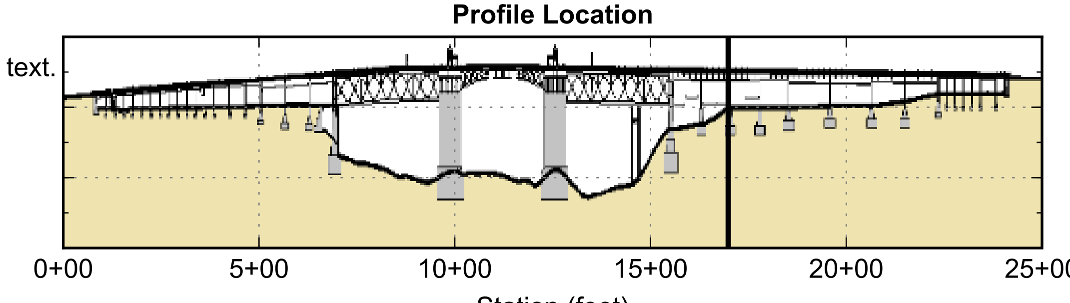
Profile Location



Burnside Bridge
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Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 22

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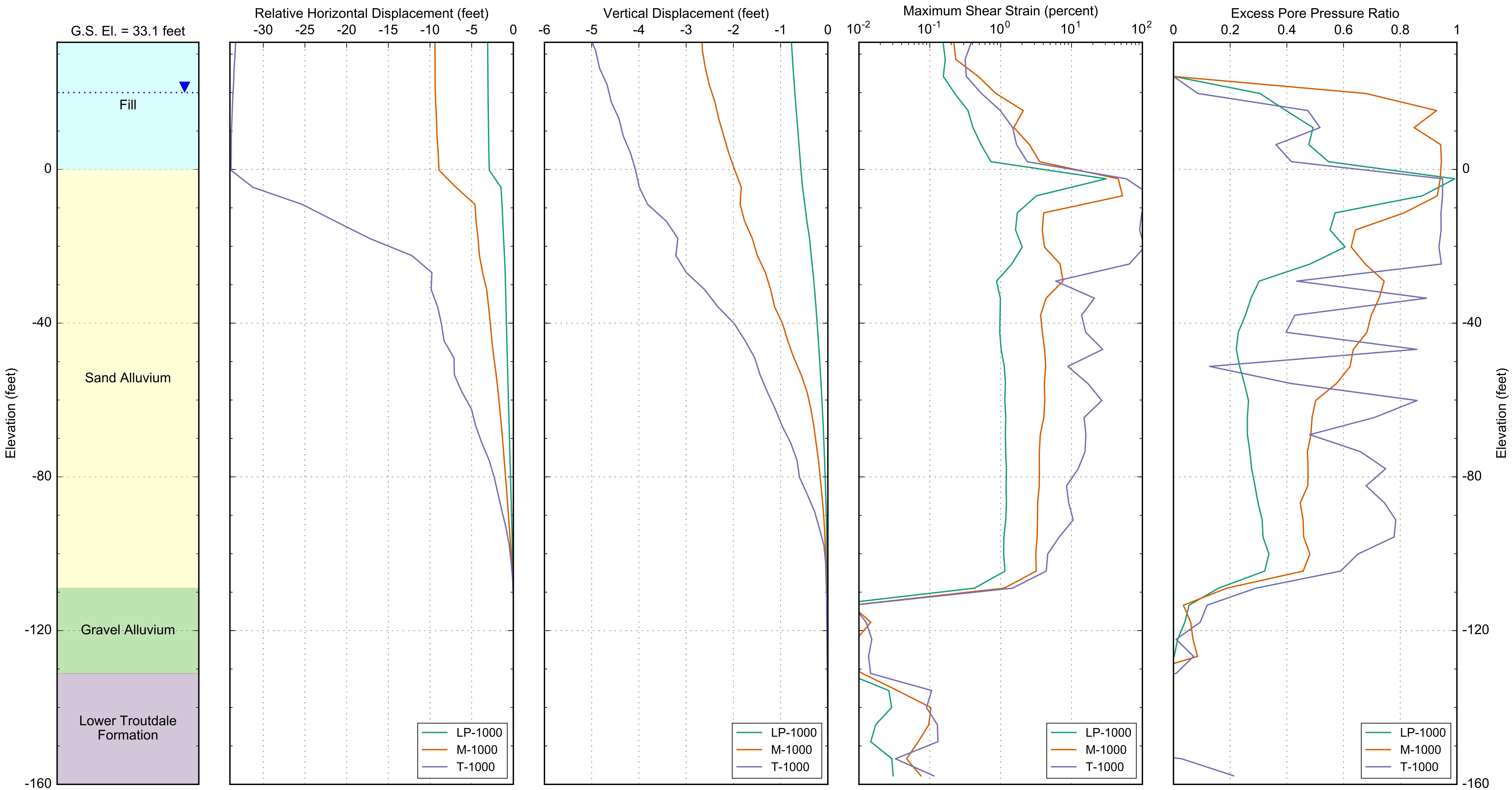
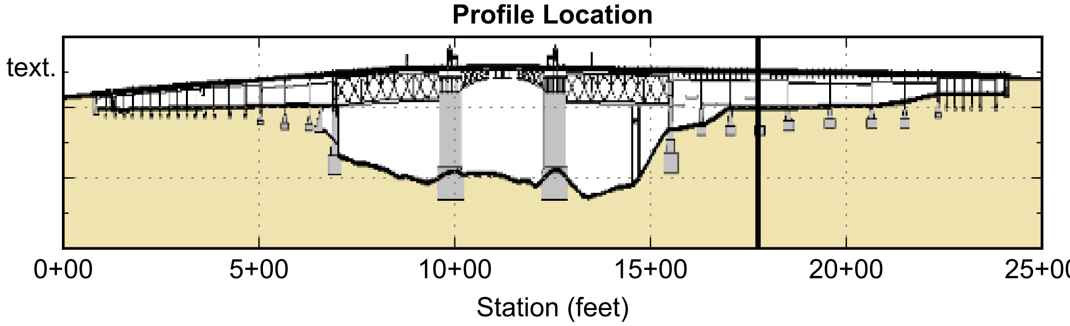
FIG. E-31



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



Burnside Bridge
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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 23

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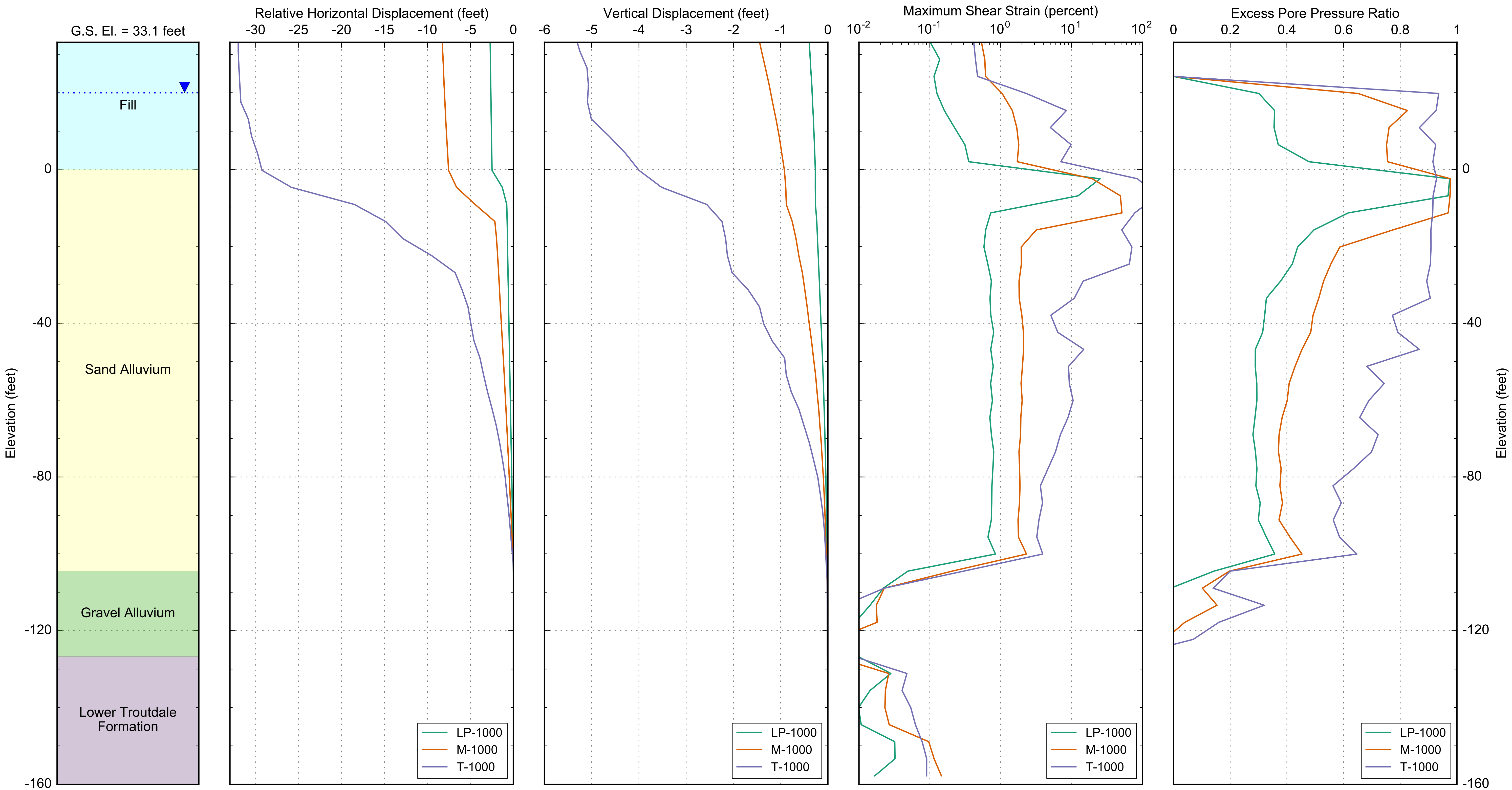
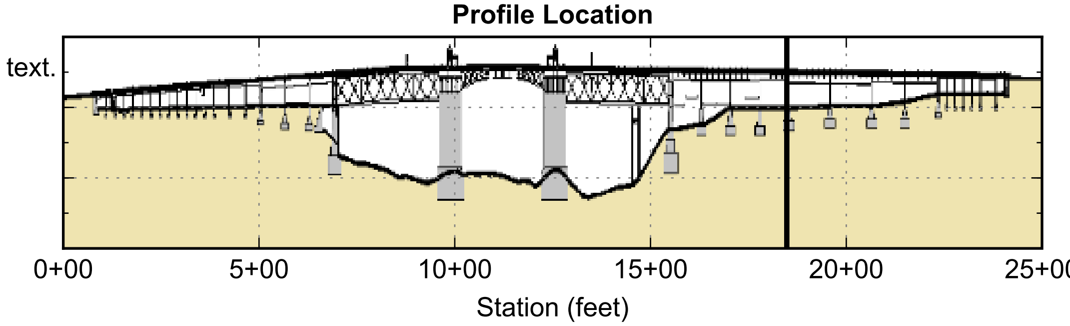
FIG. E-32



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 24

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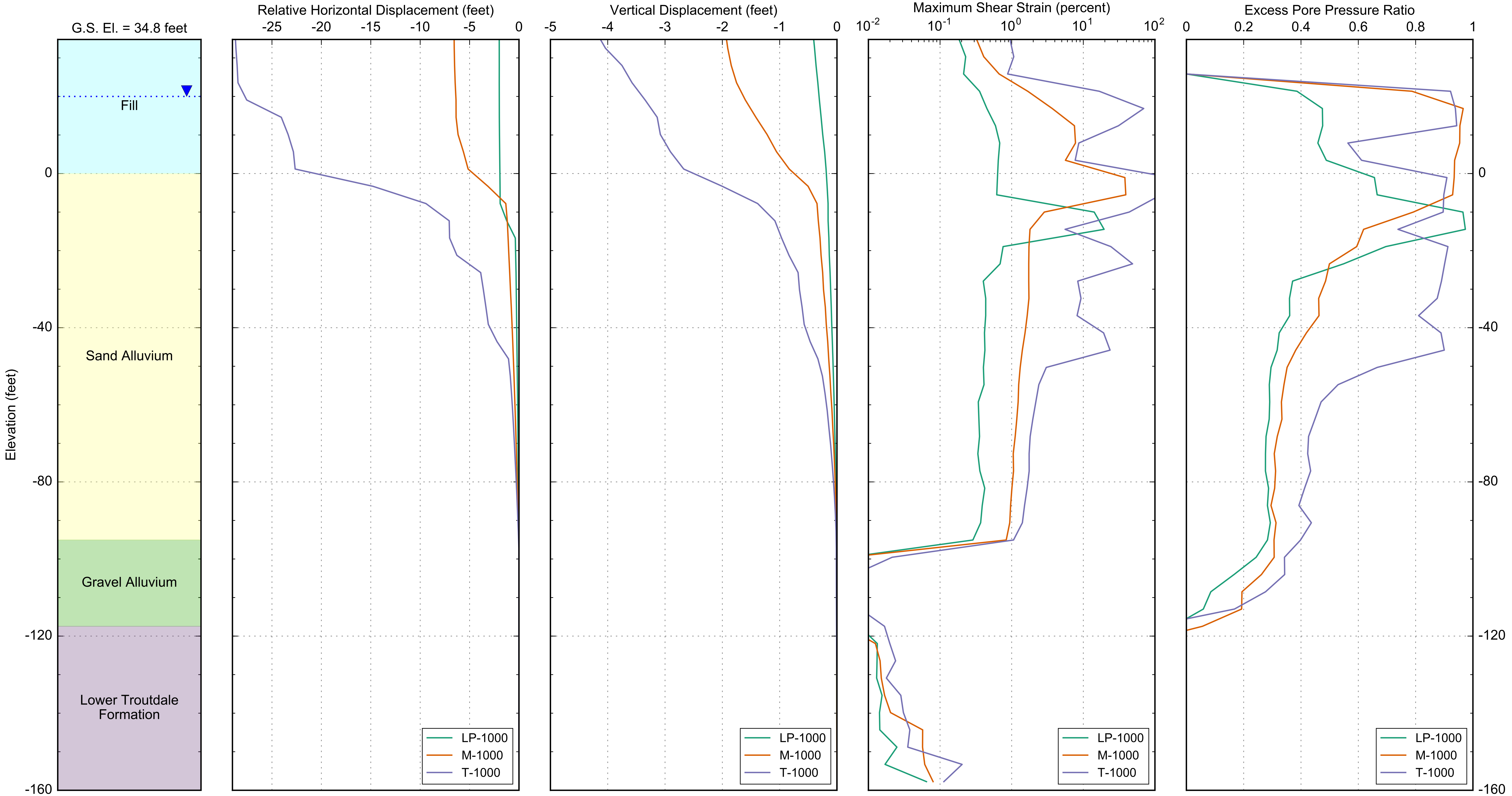
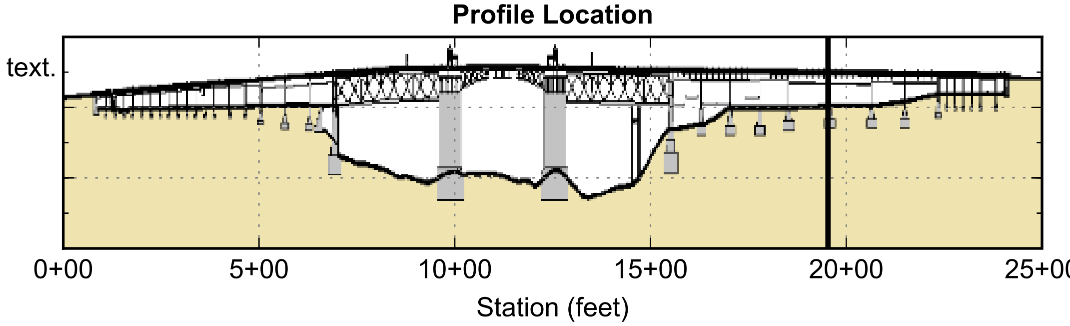
FIG. E-33



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 25

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FIG. E-34

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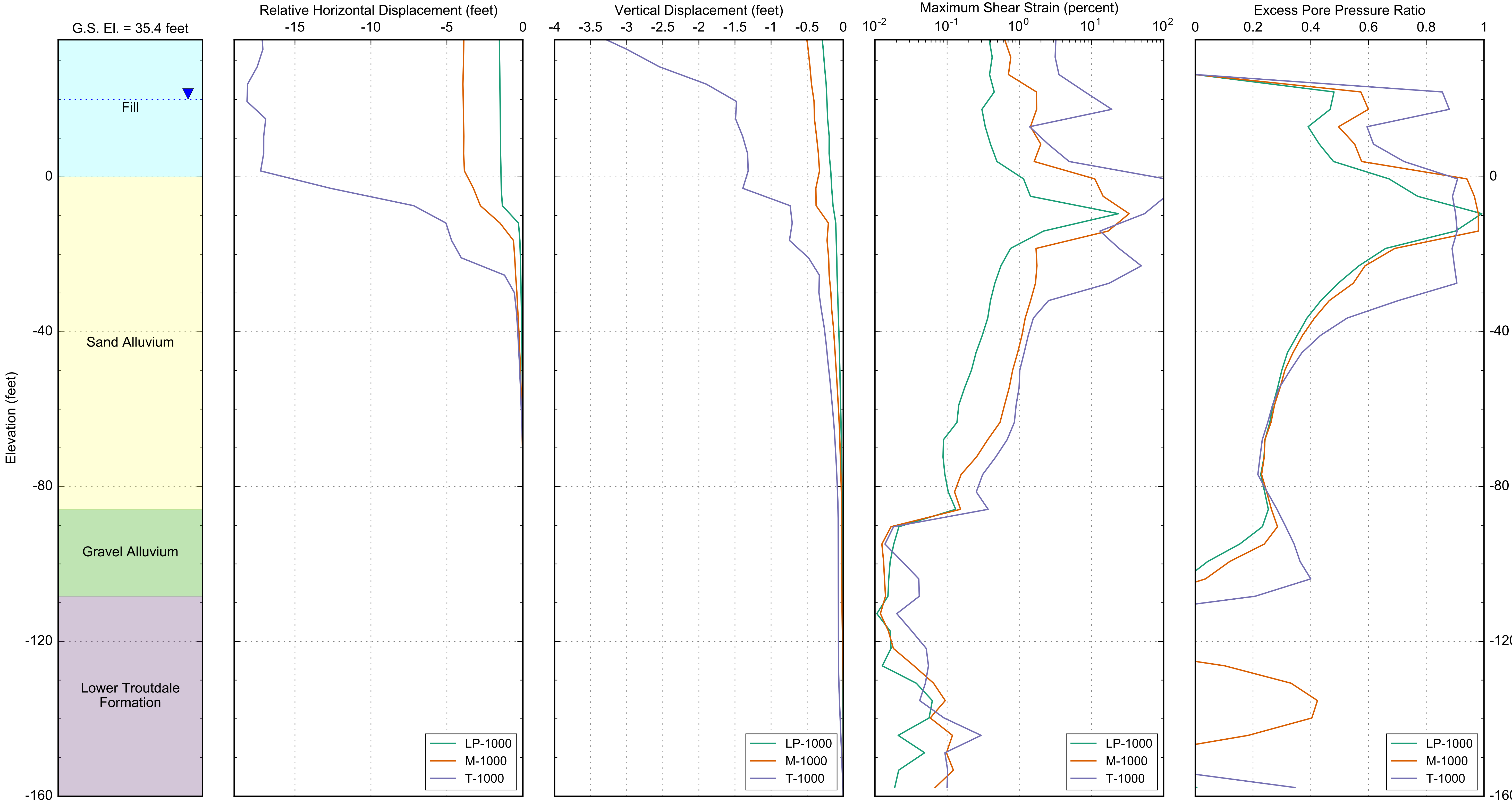
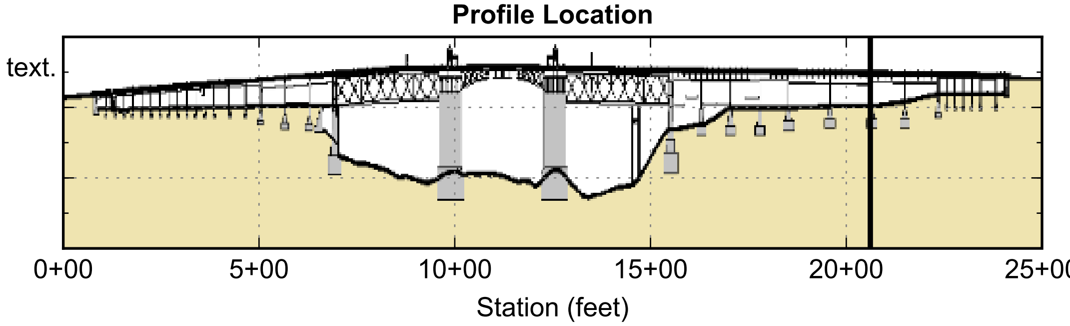


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 26

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FIG. E-35

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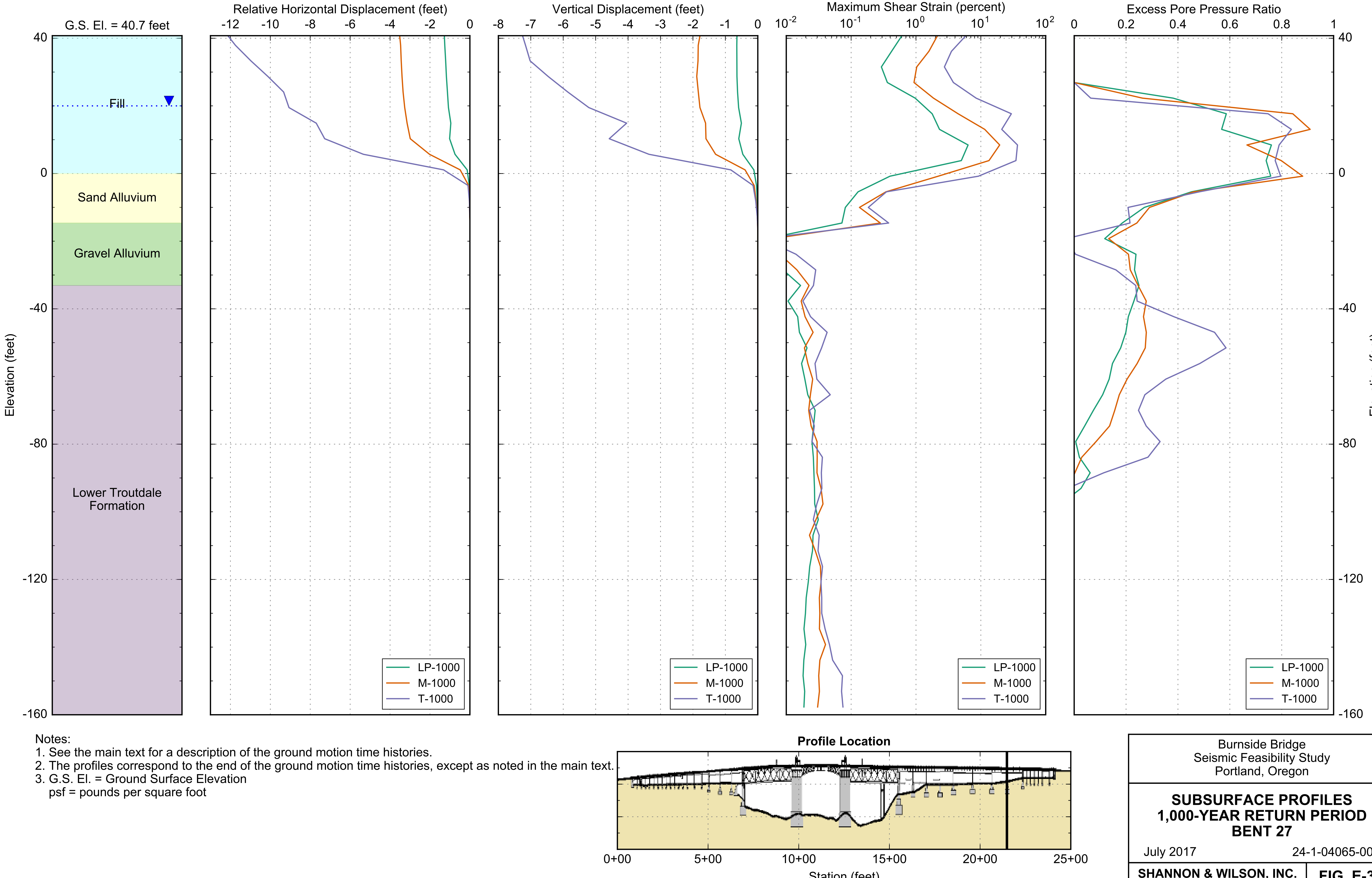

Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 27

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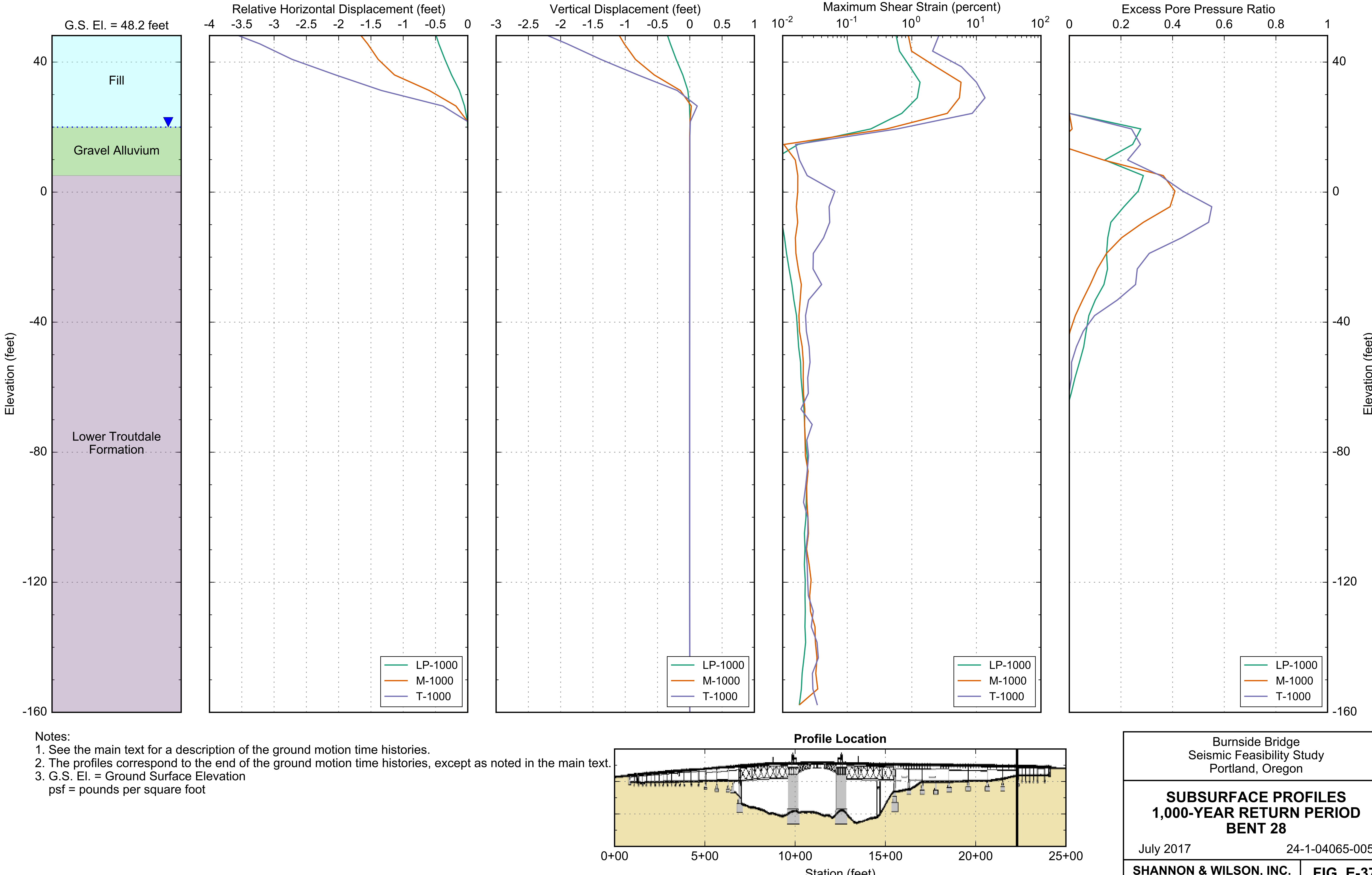
FIG. E-36



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 28

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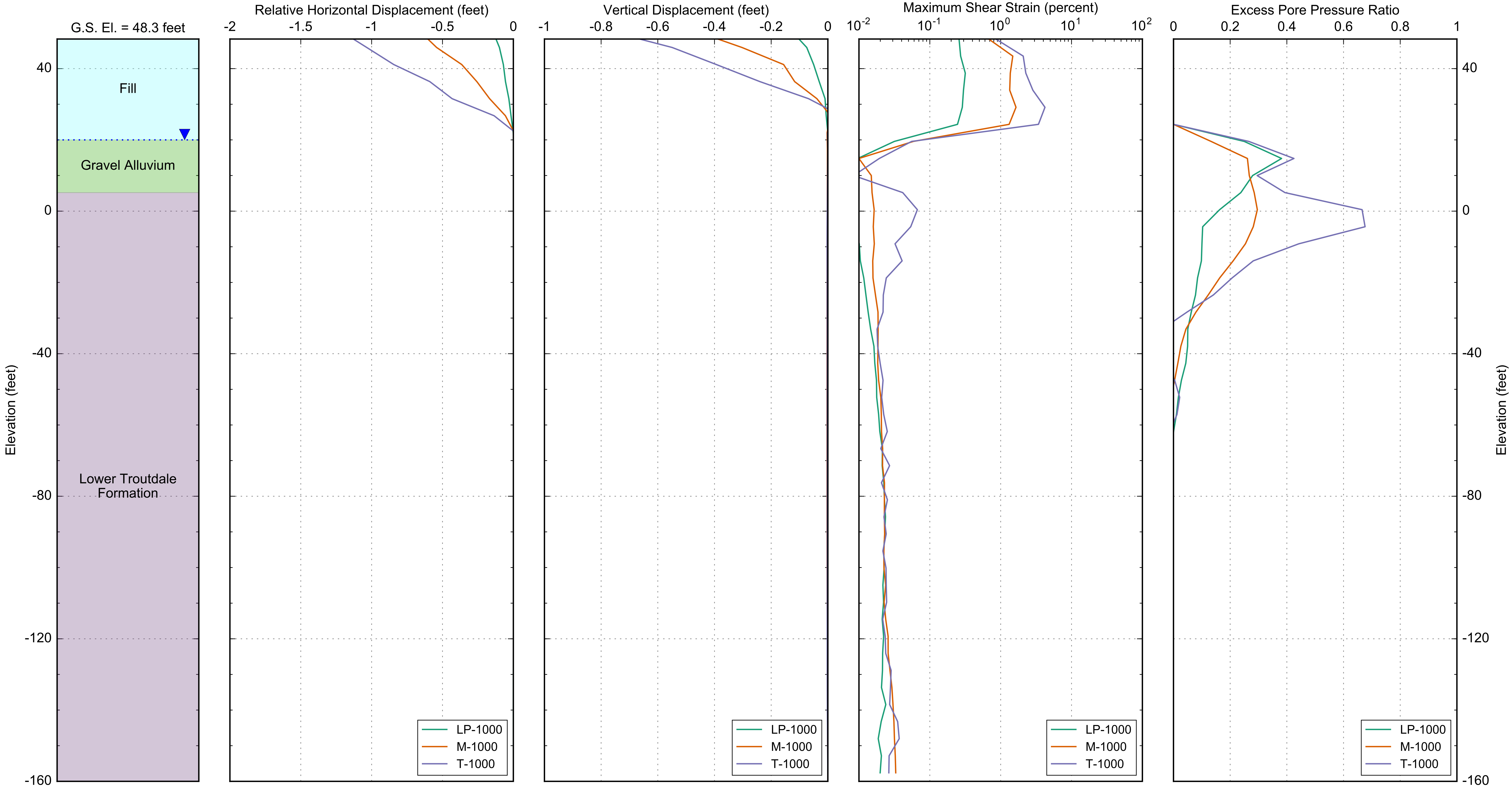
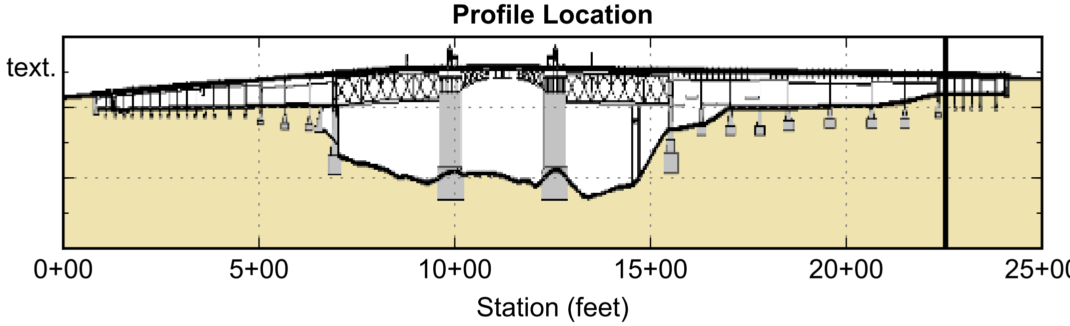
FIG. E-37



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



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Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 29

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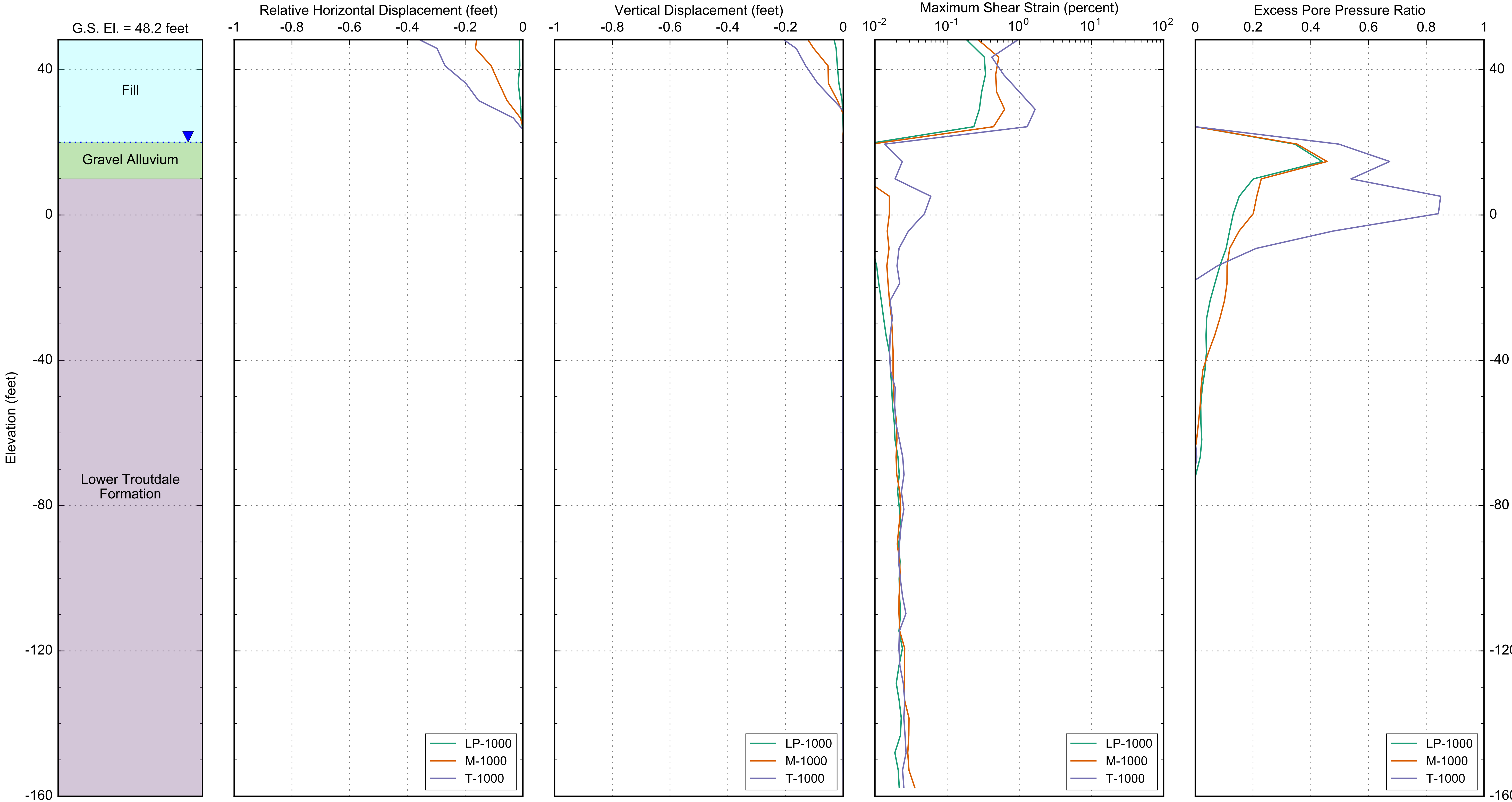
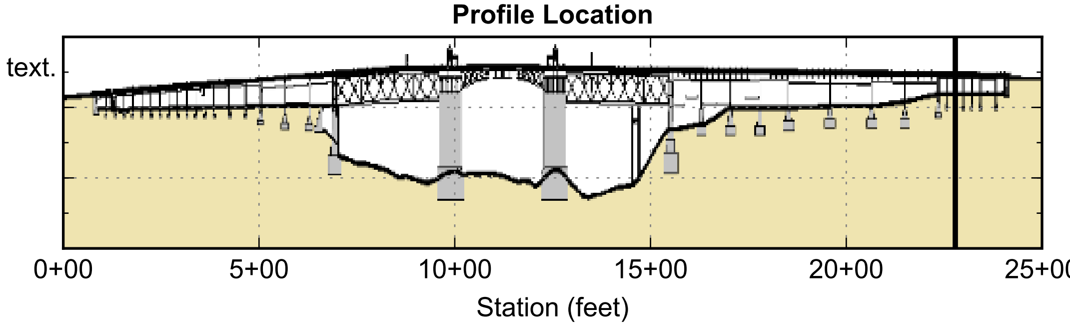
FIG. E-38



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 30

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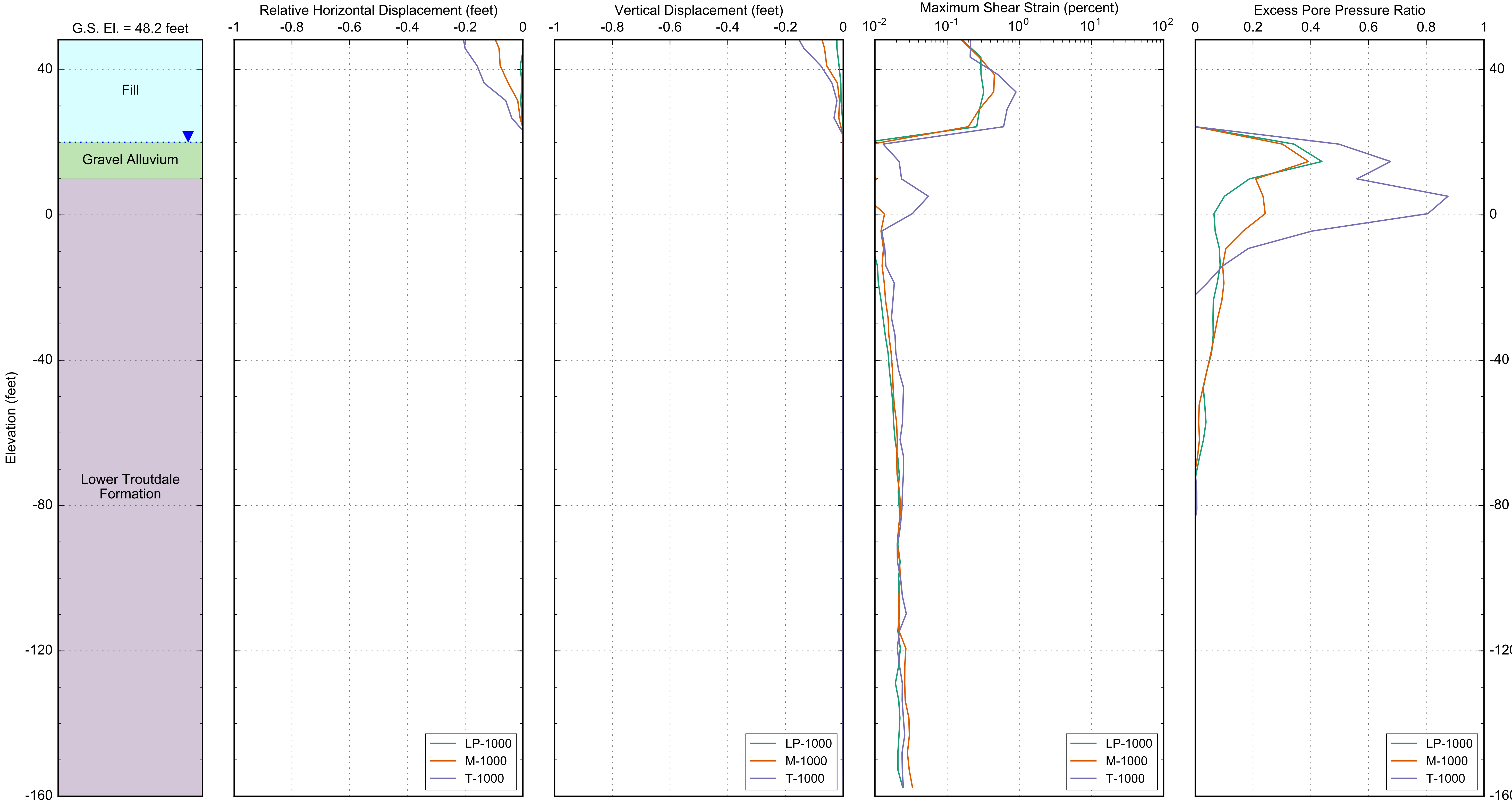
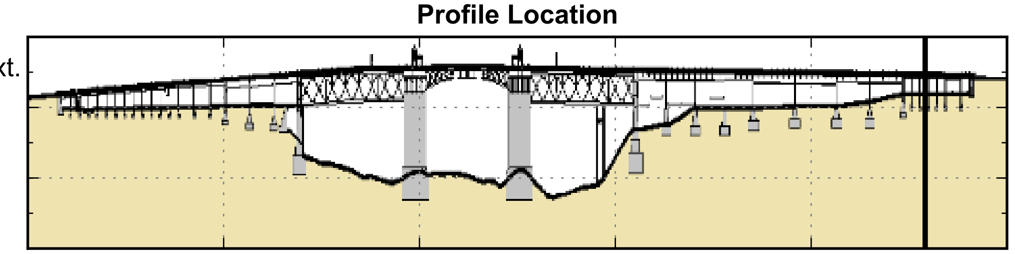
FIG. E-39



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 31

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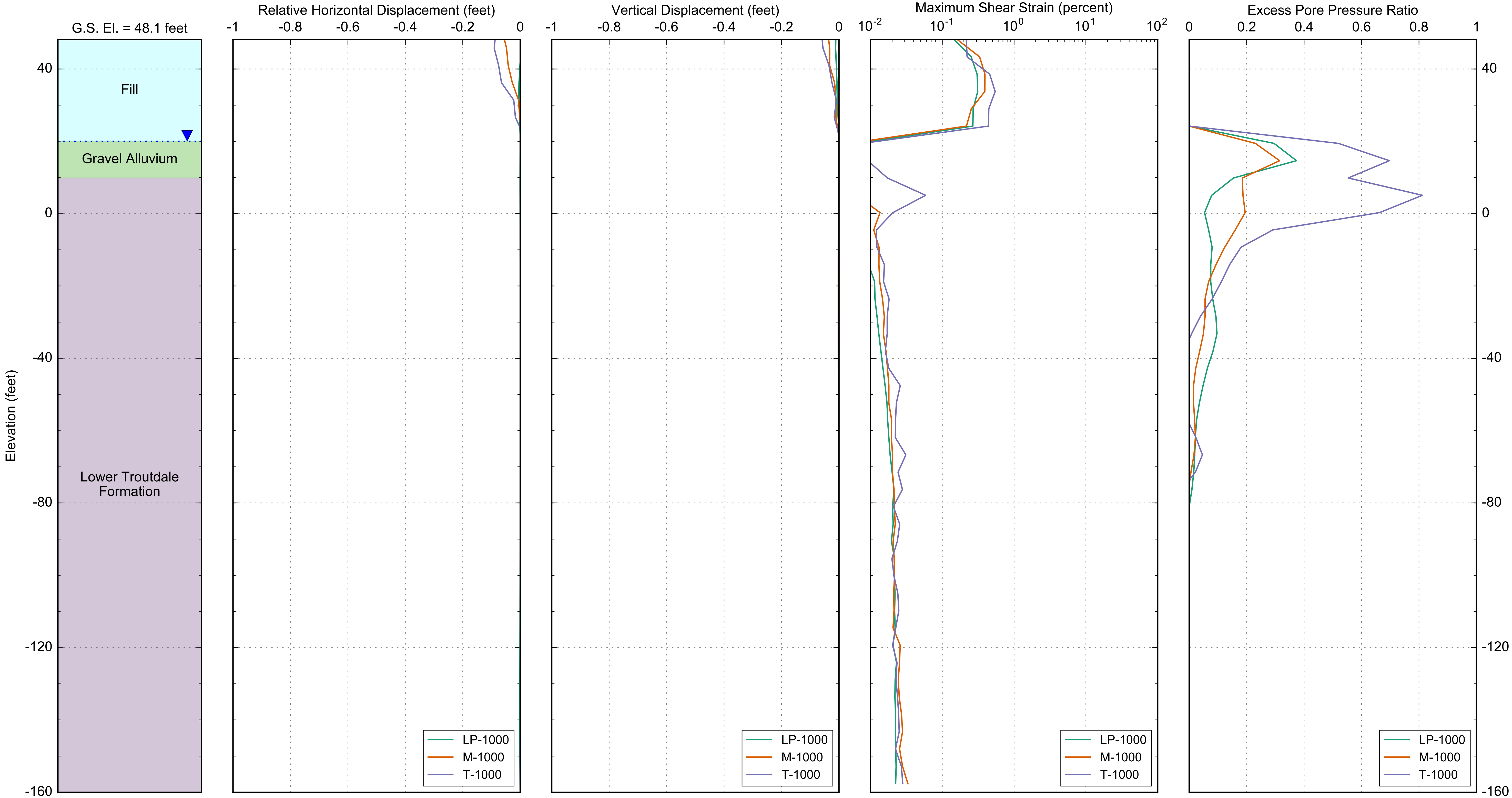
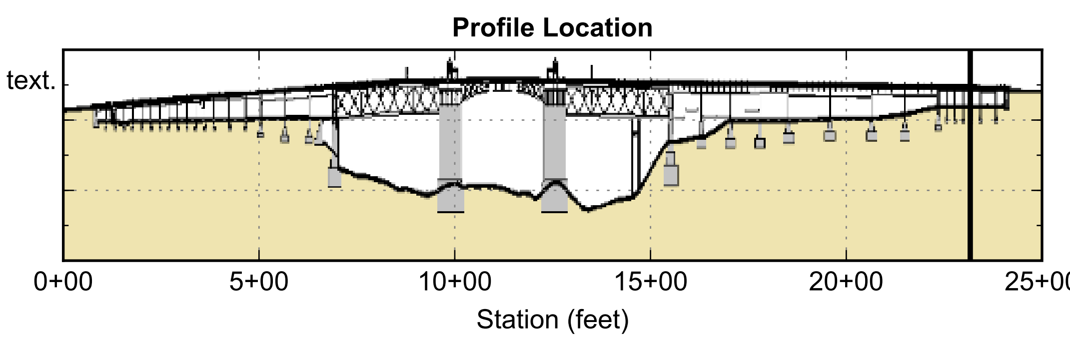
FIG. E-40



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 32

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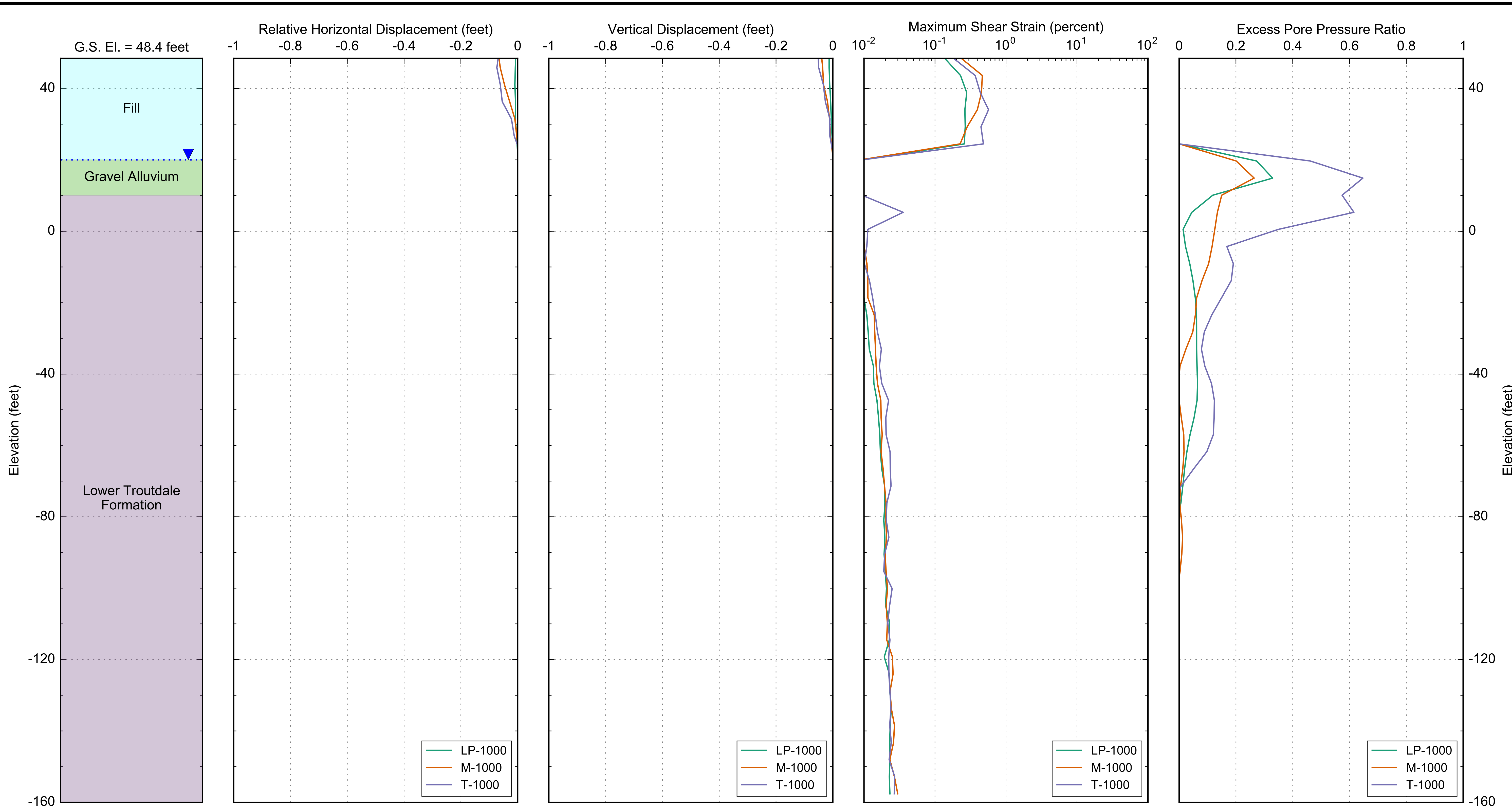
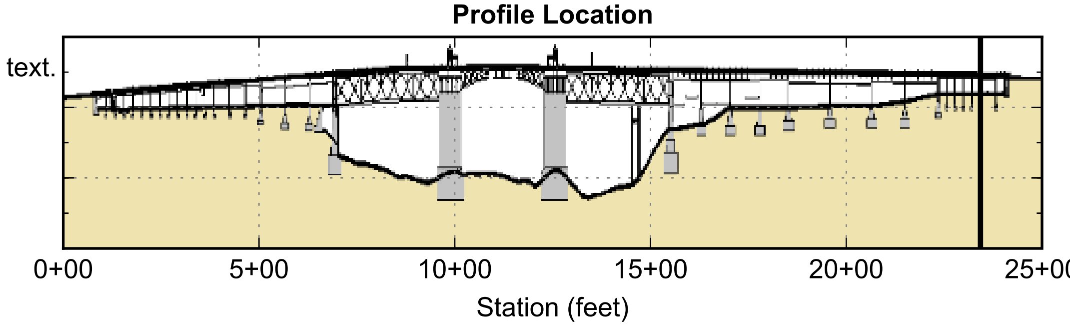
FIG. E-41



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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Portland, Oregon

SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 33

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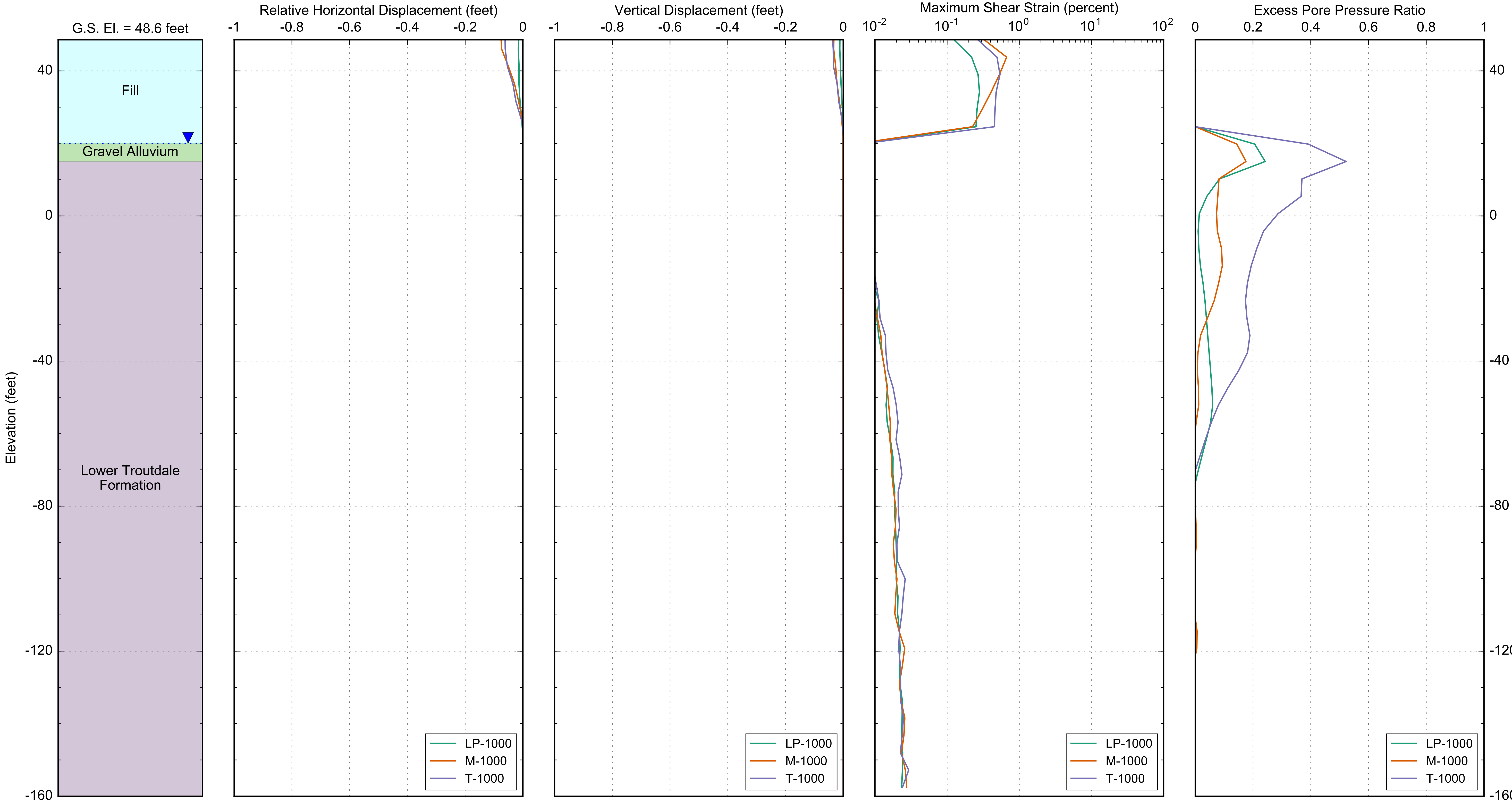
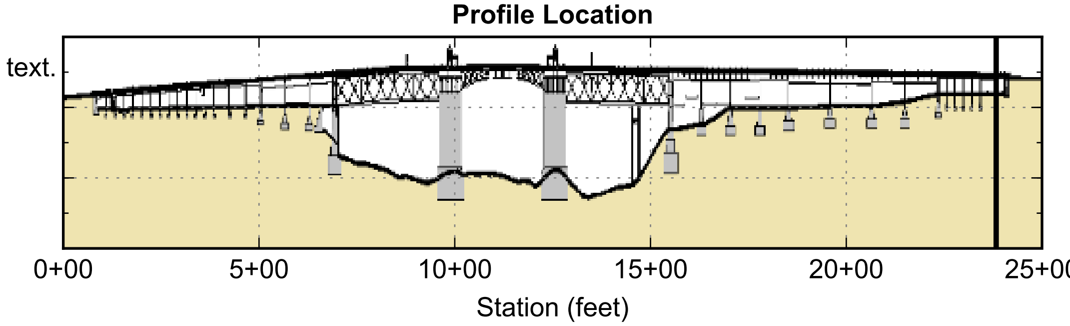
FIG. E-42



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 34

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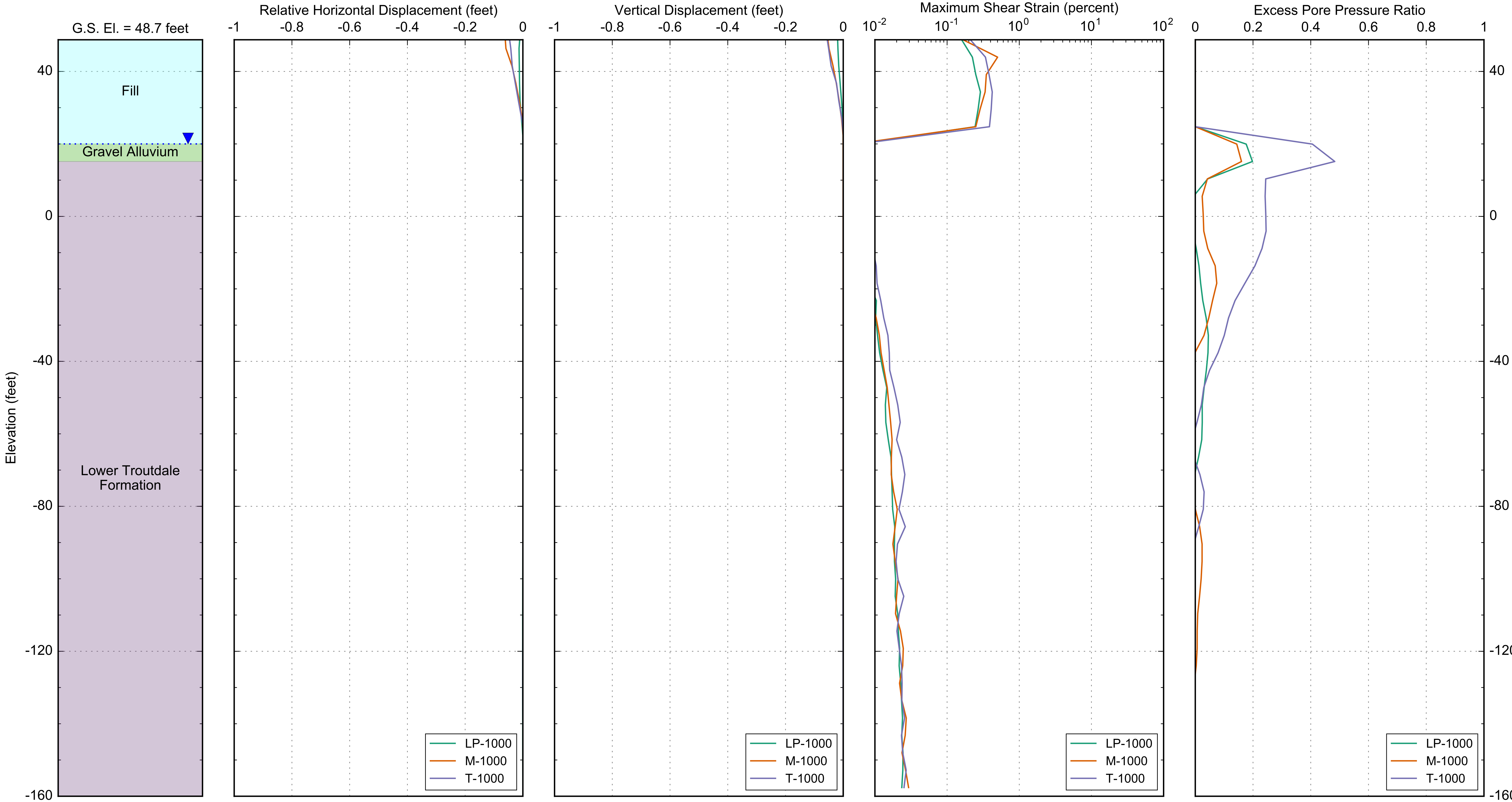
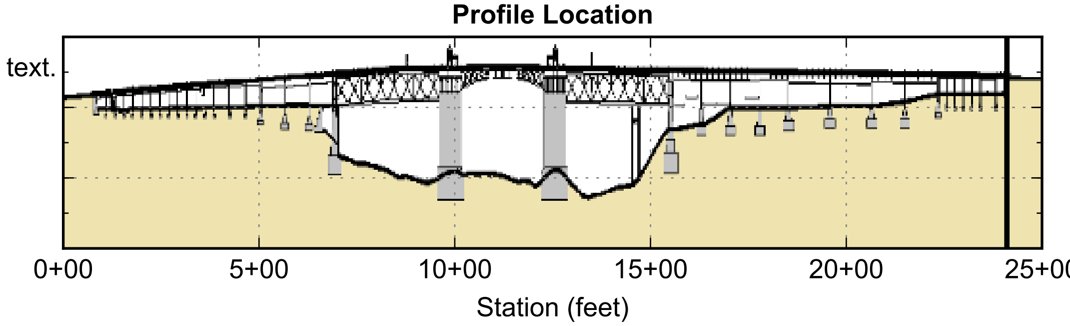
FIG. E-43



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



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SUBSURFACE PROFILES
1,000-YEAR RETURN PERIOD
BENT 35

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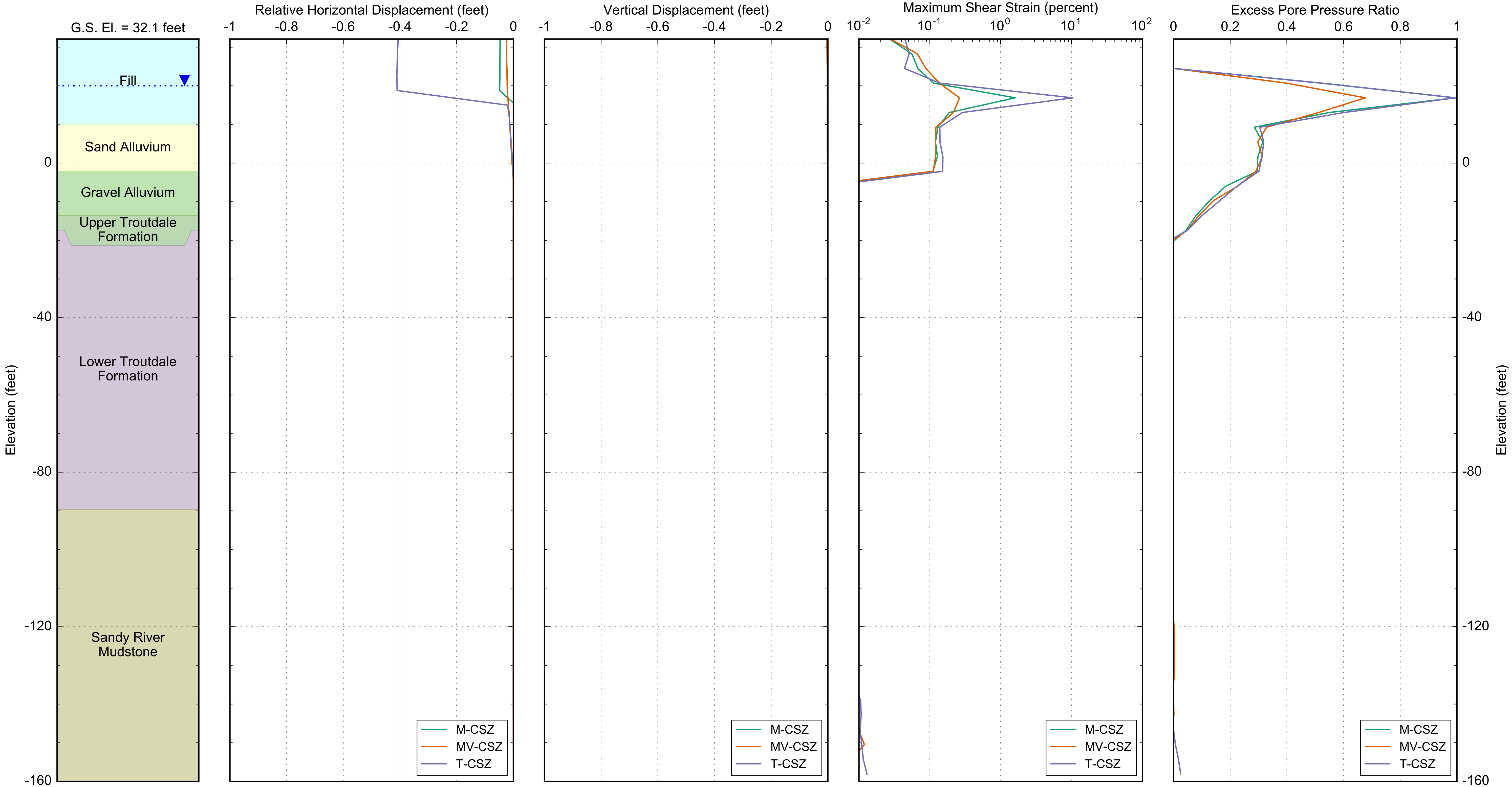
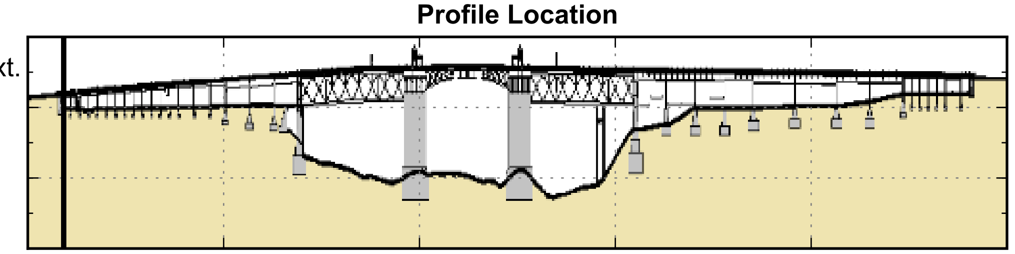
FIG. E-44



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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Seismic Feasibility Study
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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 1

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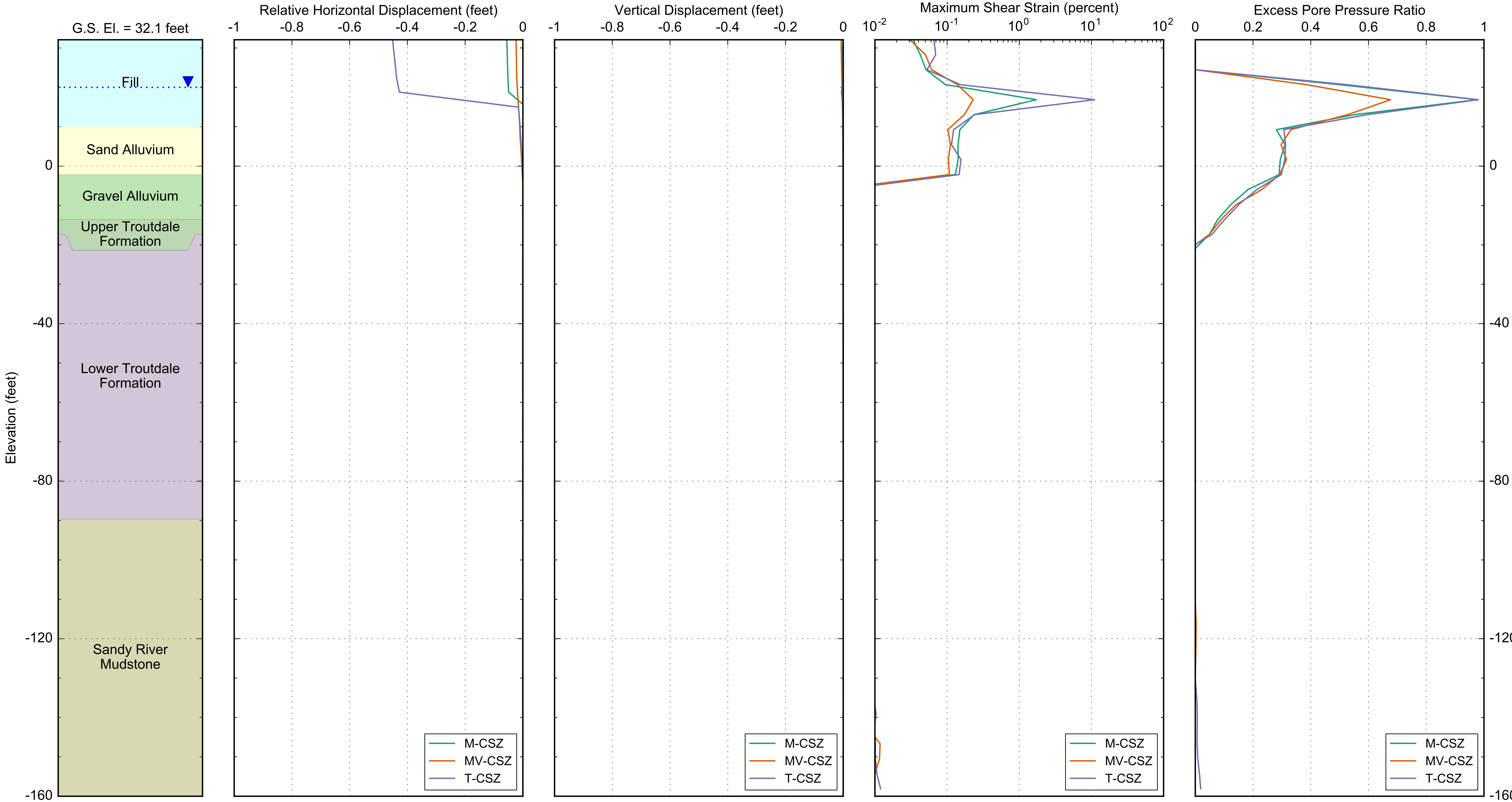
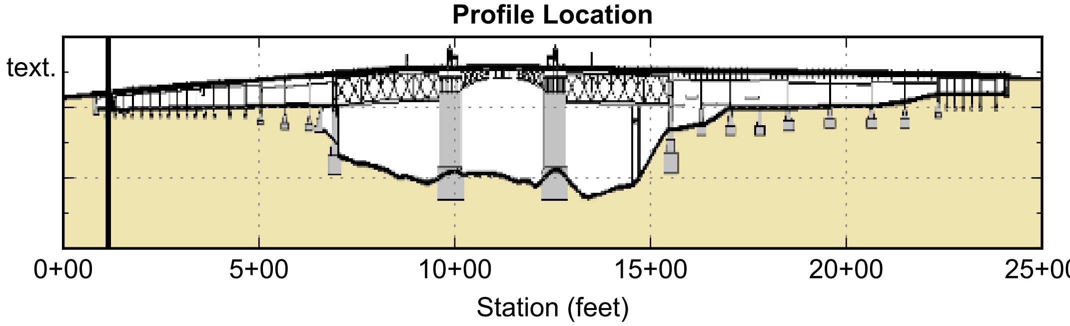
FIG. E-45



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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Seismic Feasibility Study
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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 2

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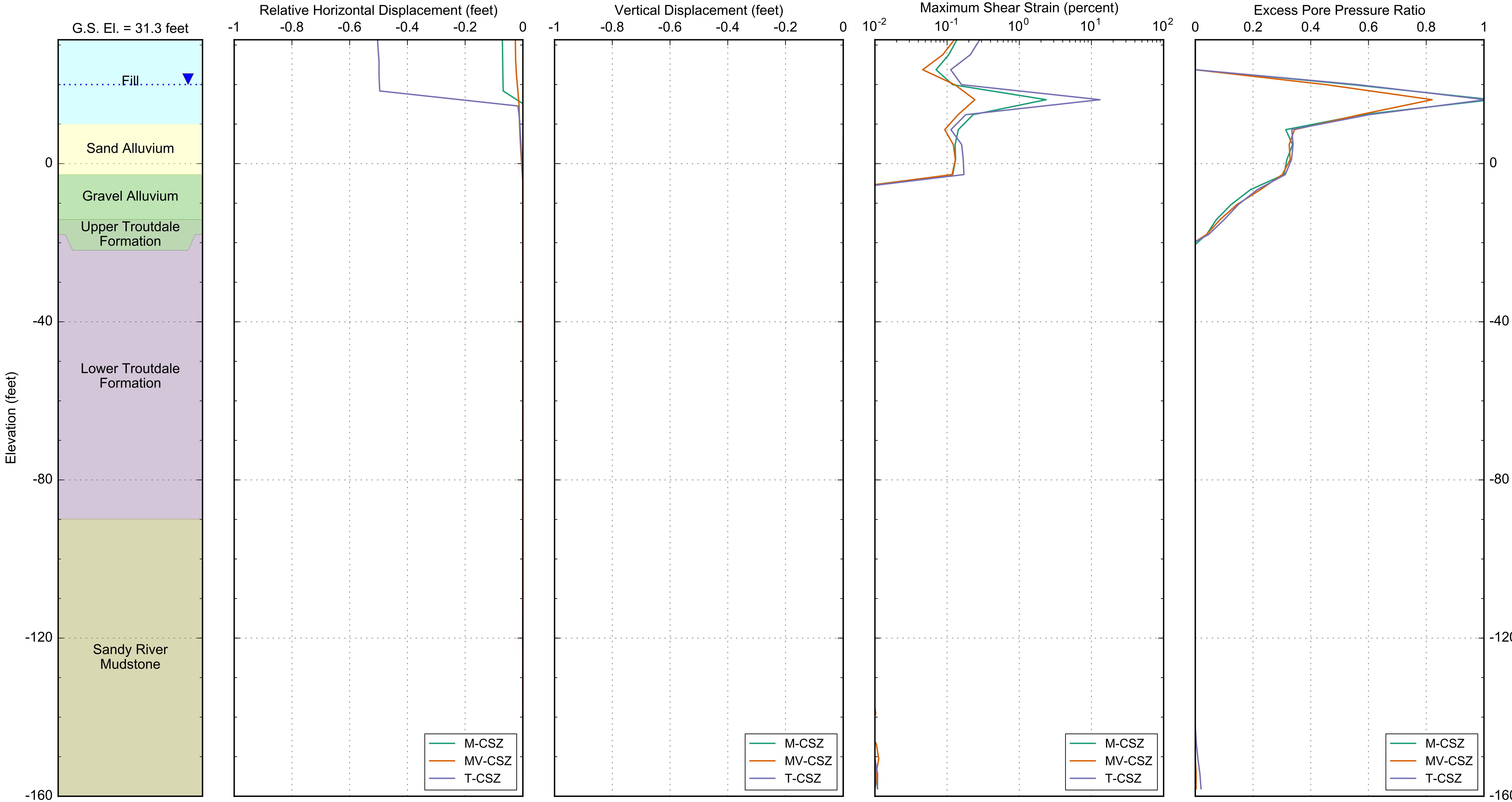
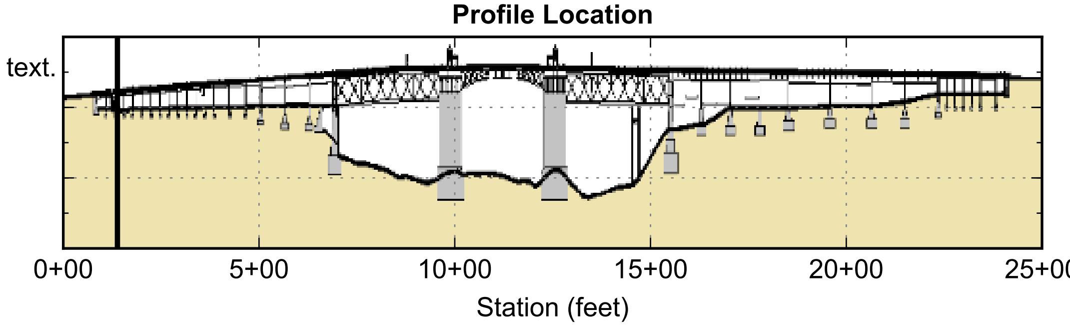
FIG. E-46



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 3

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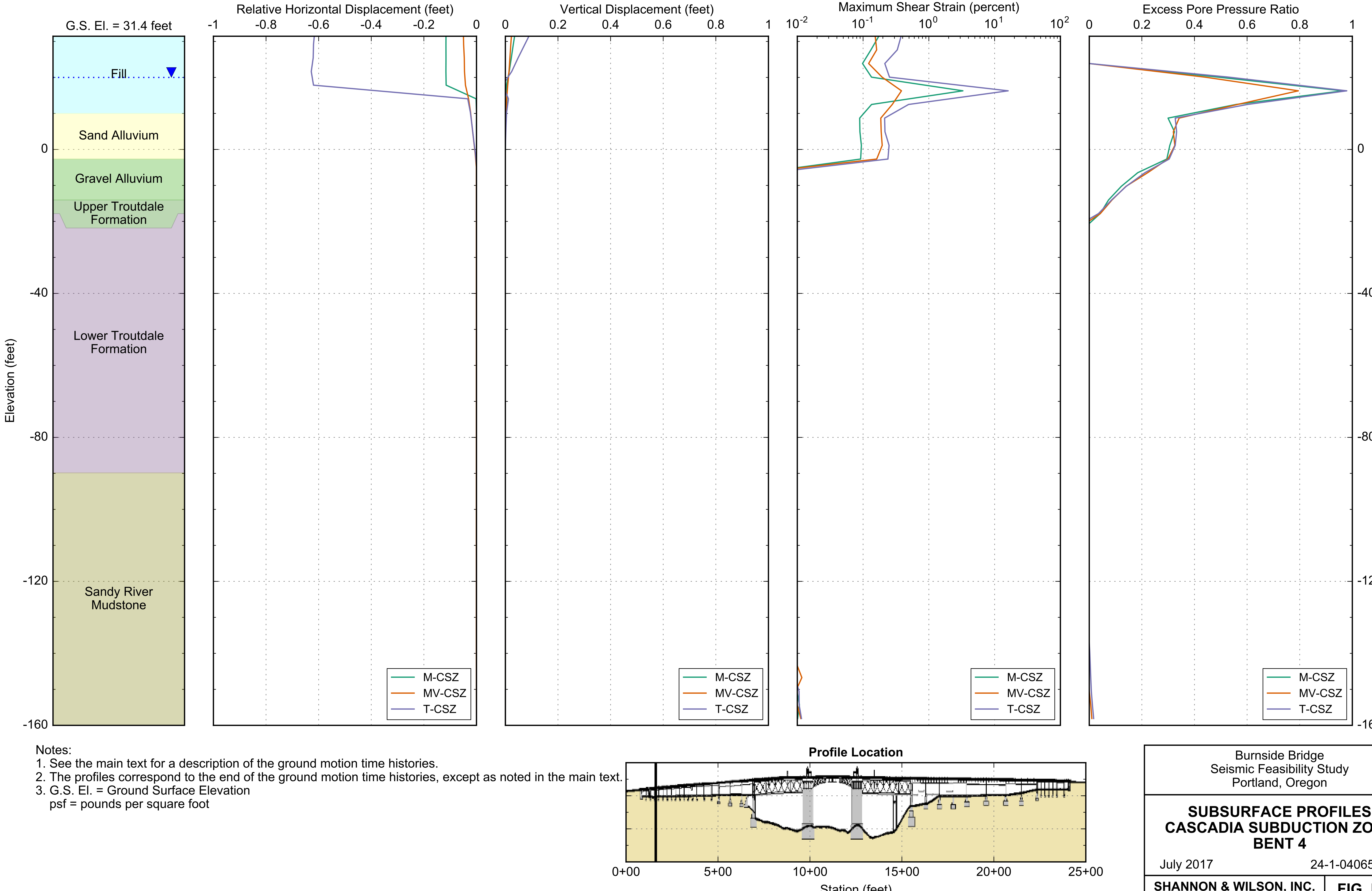
FIG. E-47



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 4

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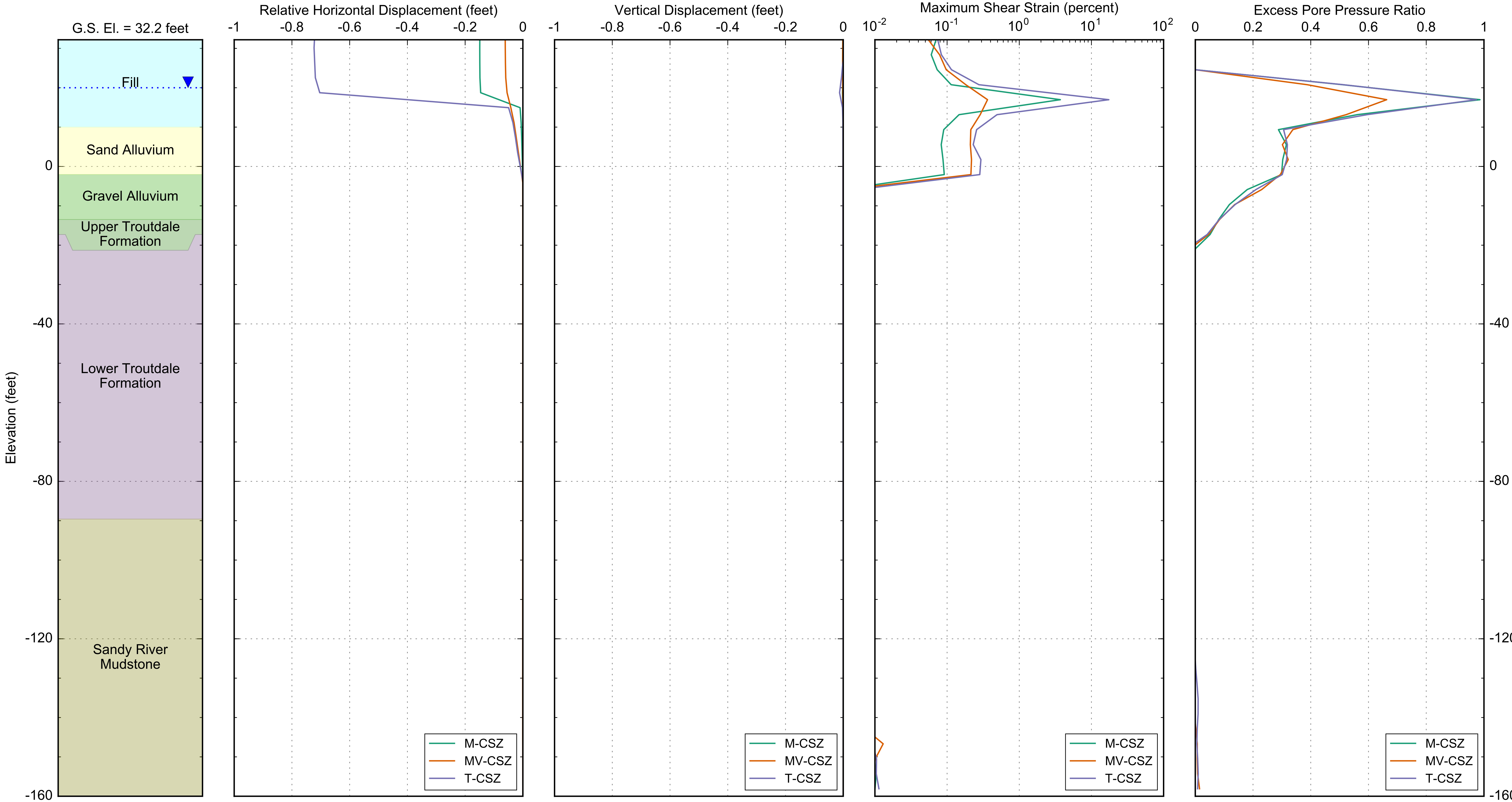
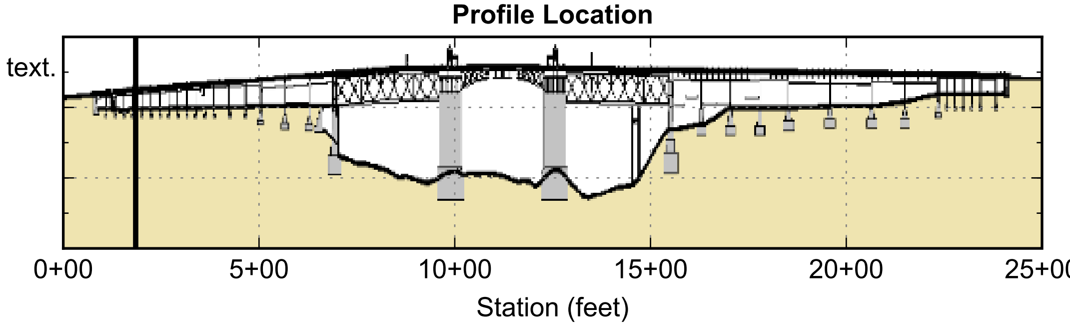
FIG. E-48



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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**SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 5**

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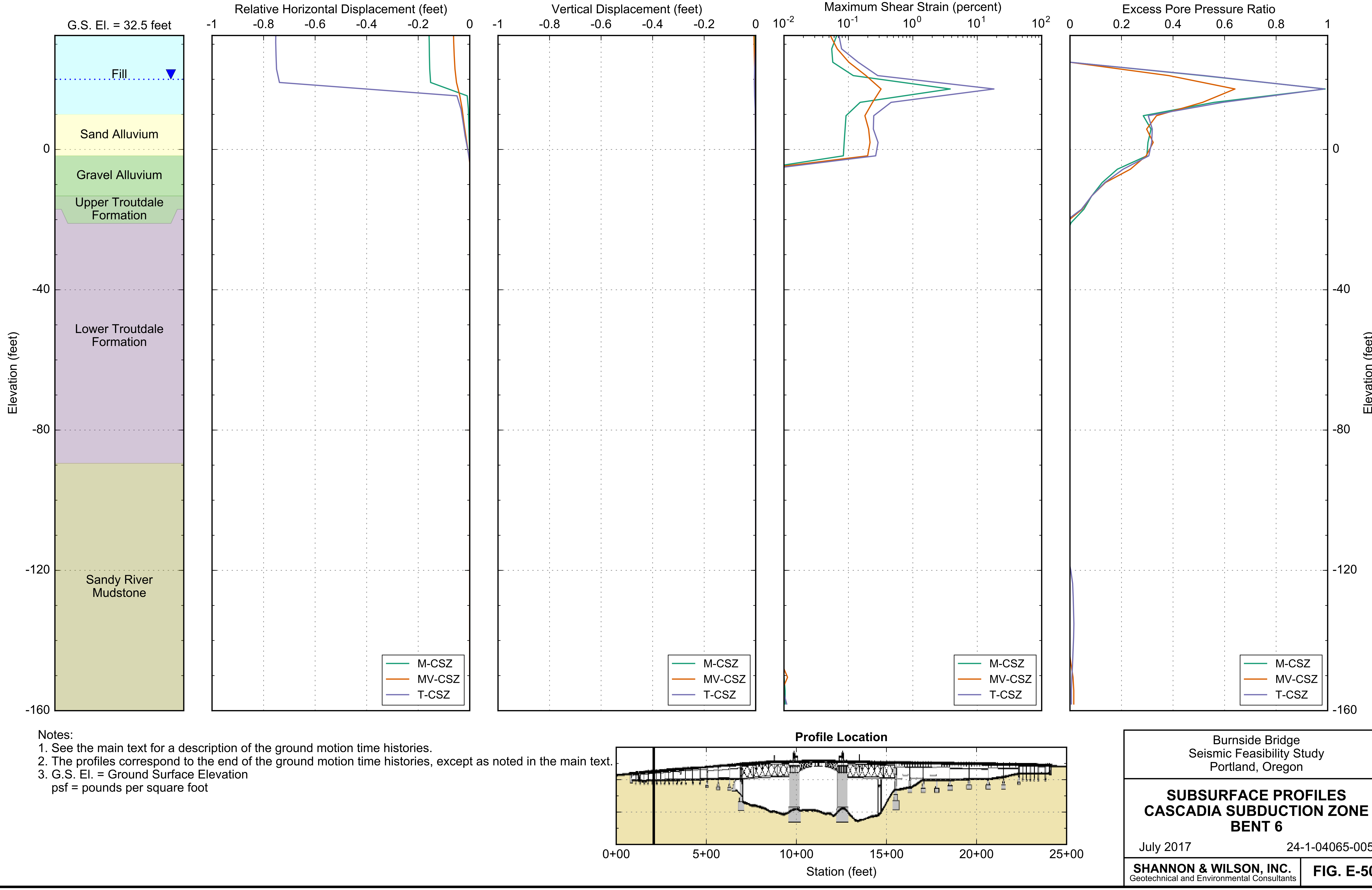
FIG. E-49



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 6

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FIG. E-50

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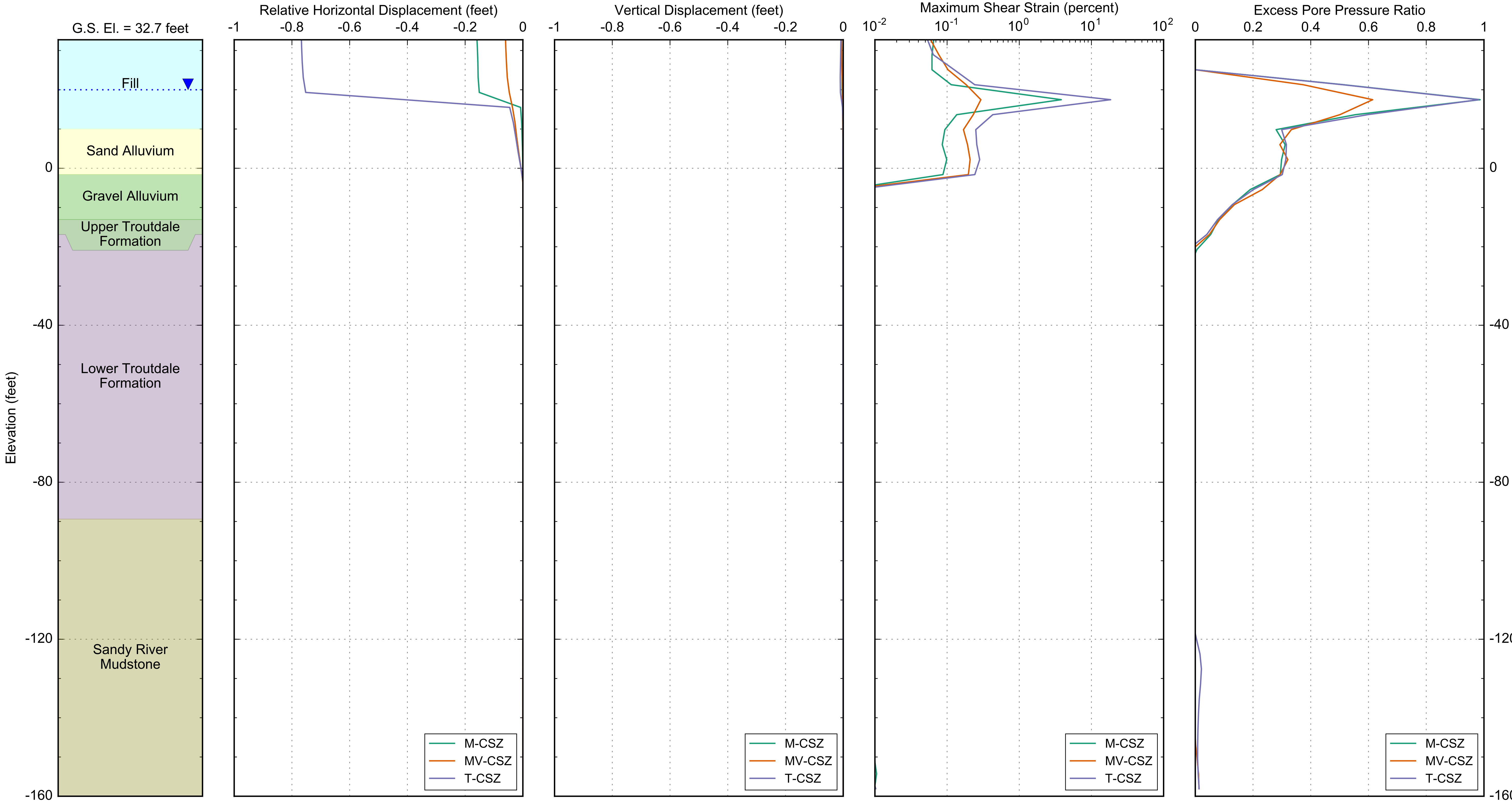
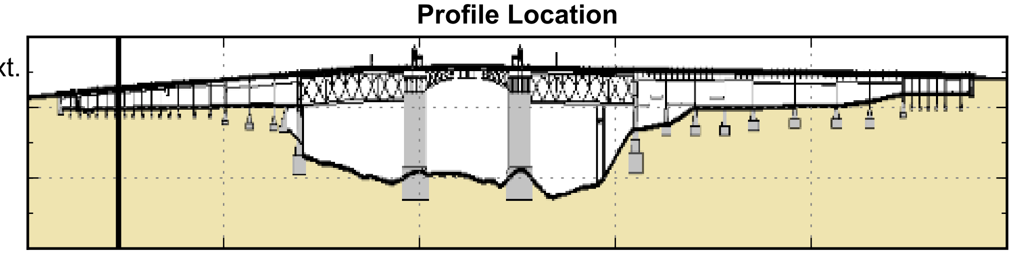


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 7

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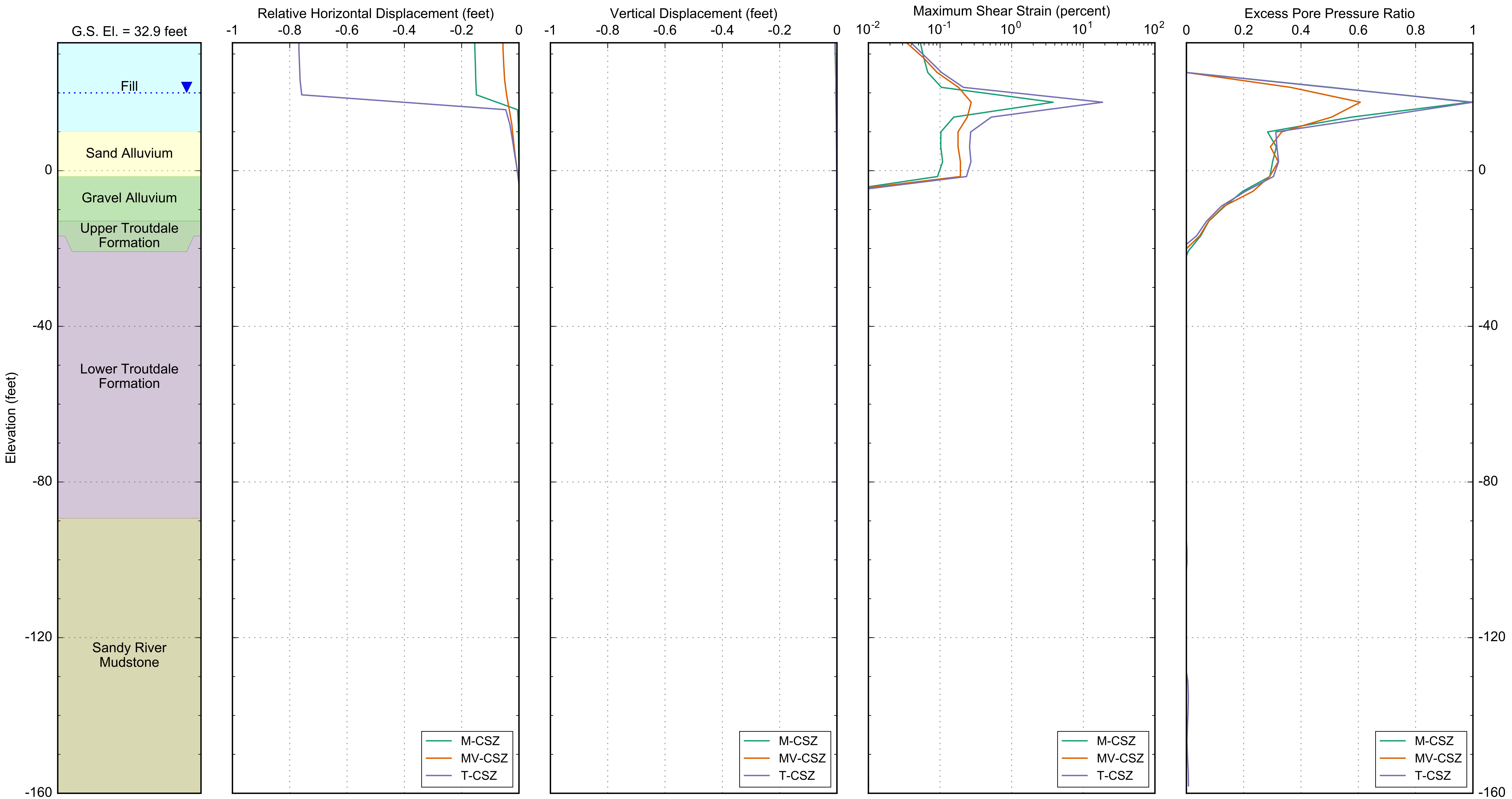
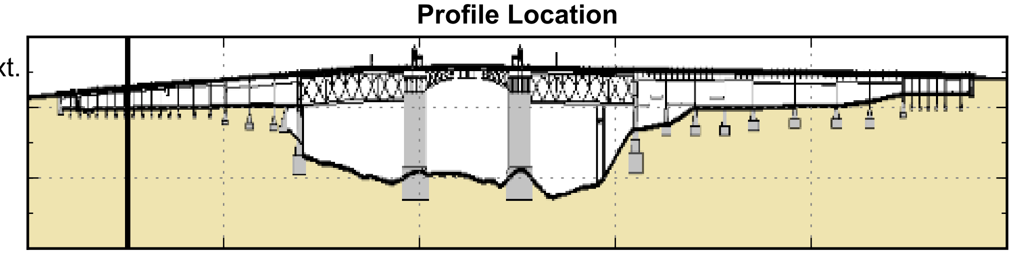
FIG. E-51



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 8

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FIG. E-52

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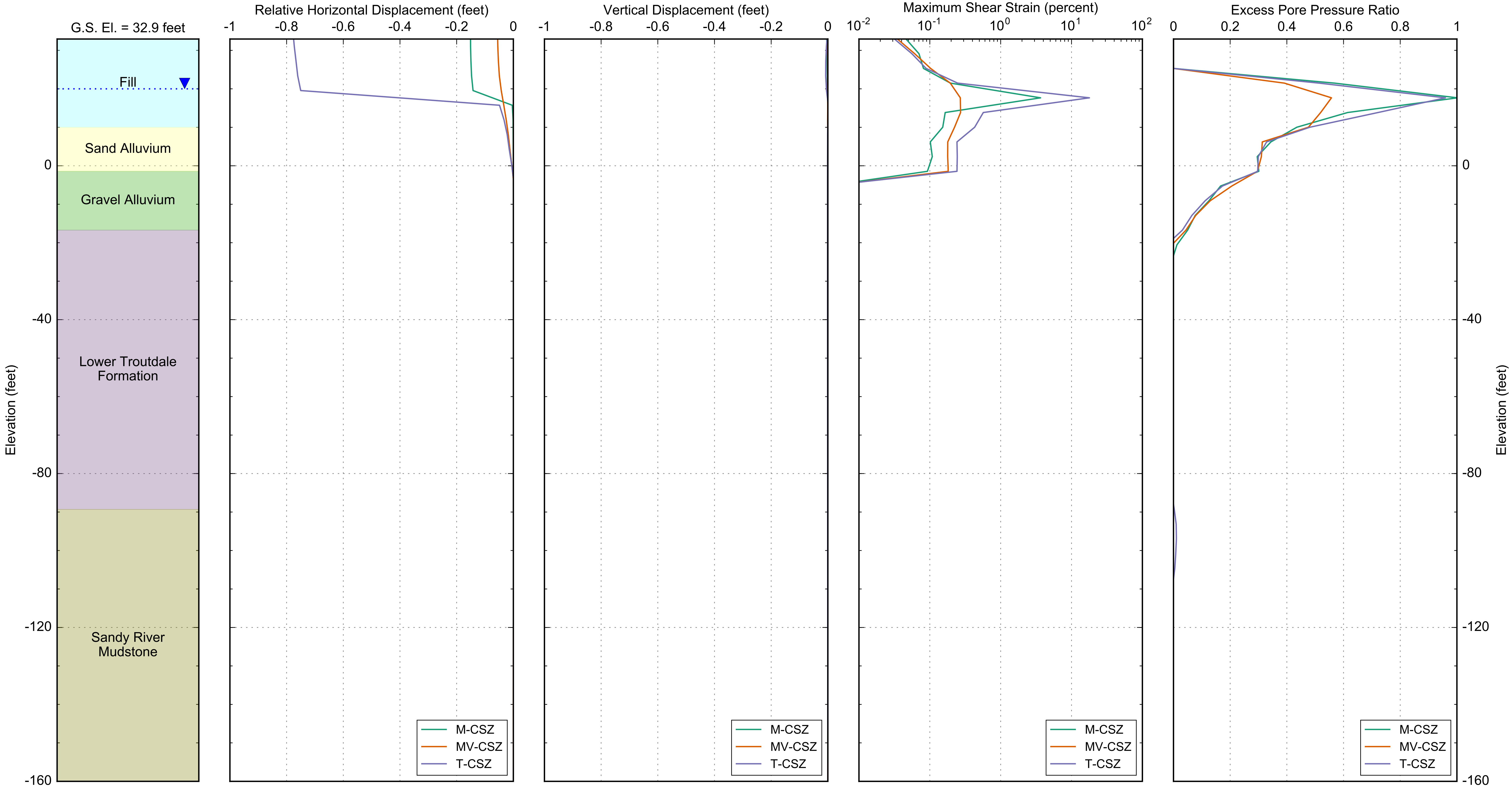
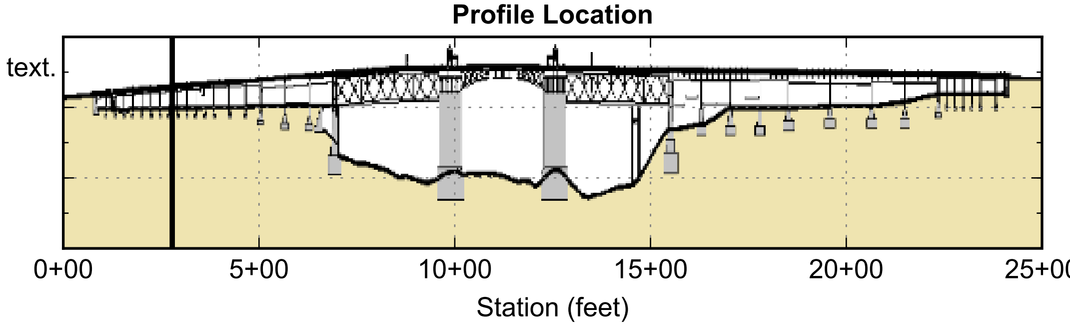


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 9

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FIG. E-53

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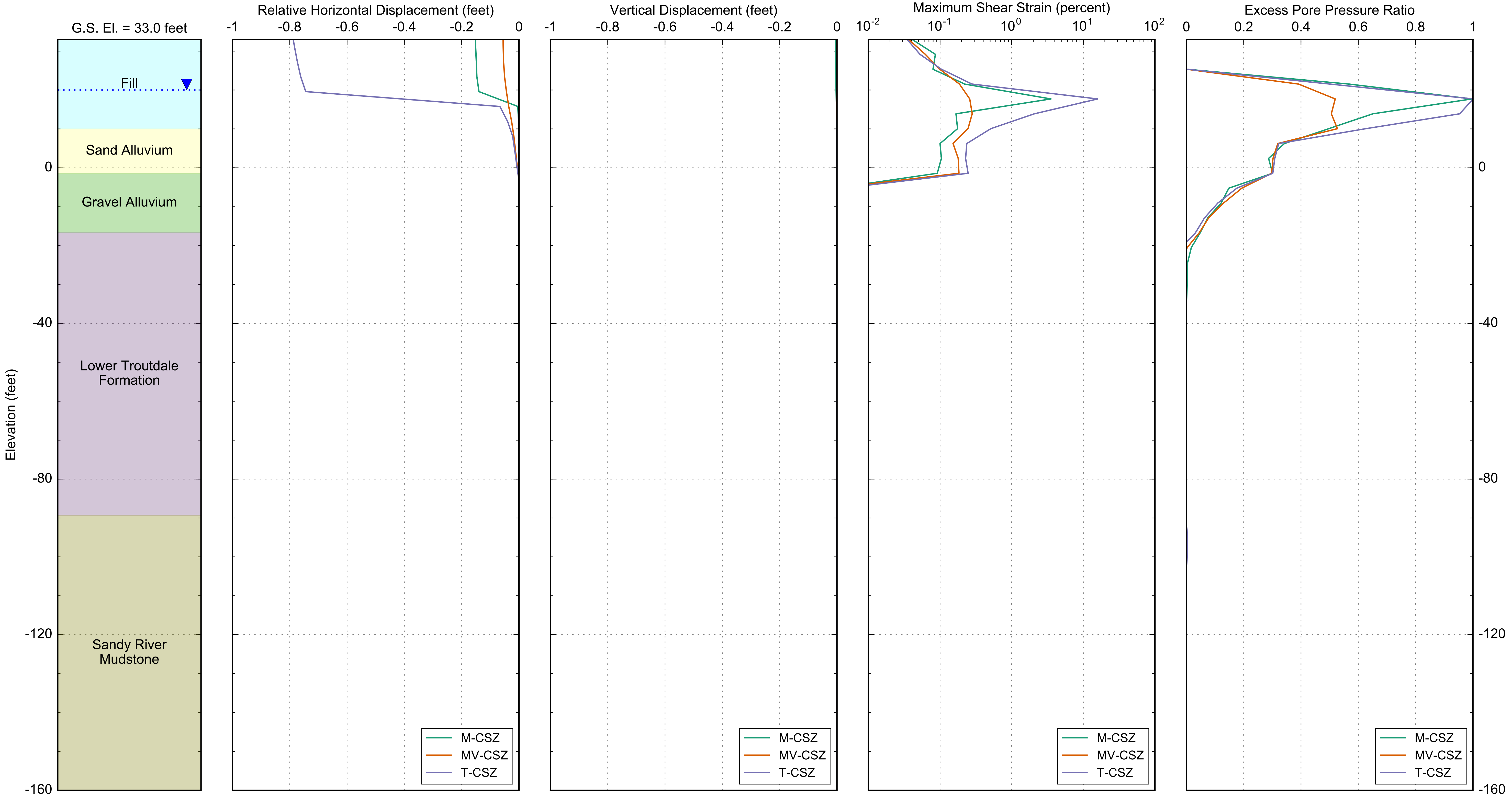
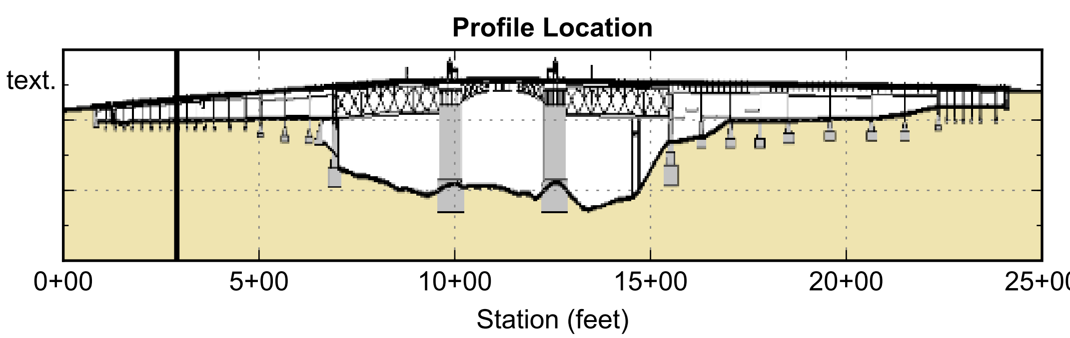


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 10

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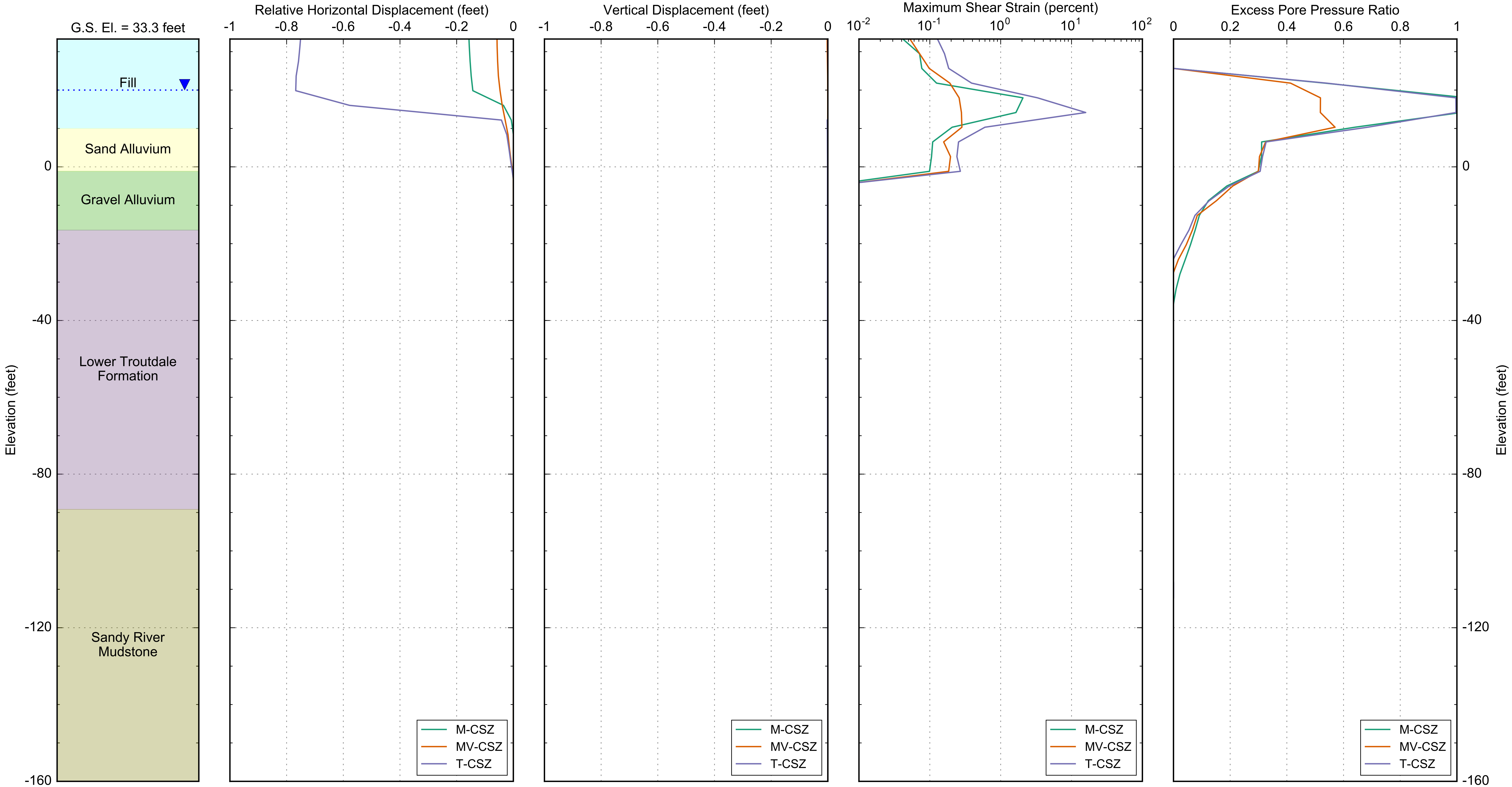
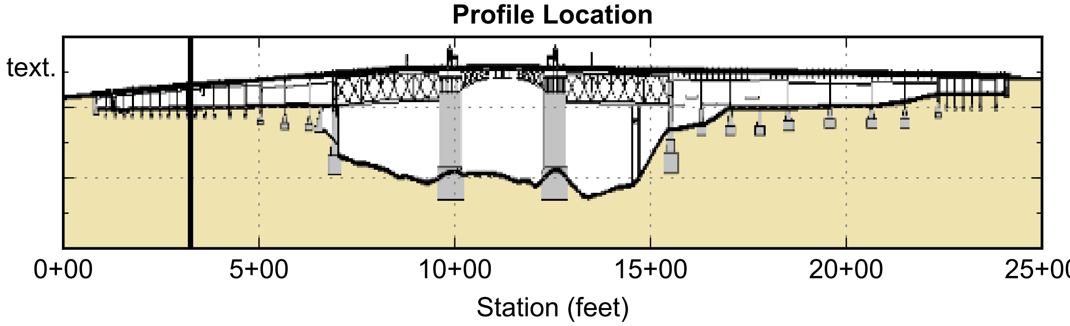
FIG. E-54



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 11

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FIG. E-55

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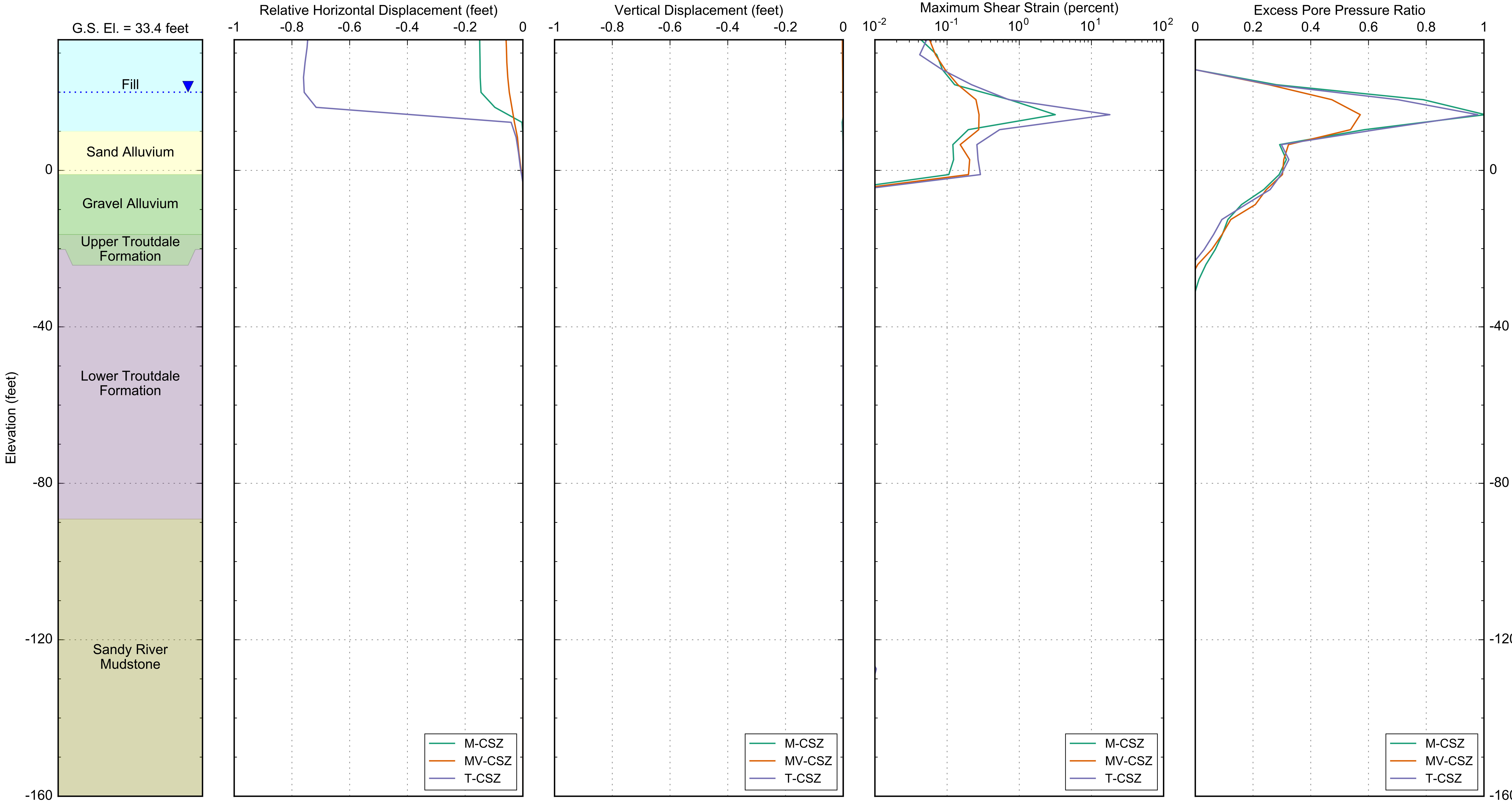
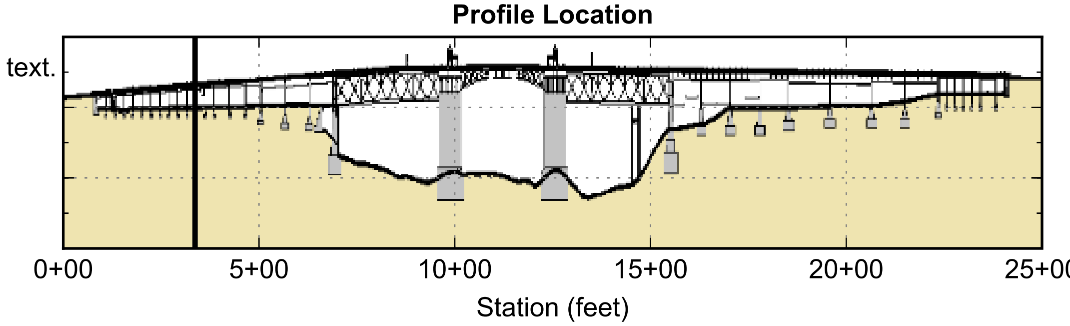


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 12

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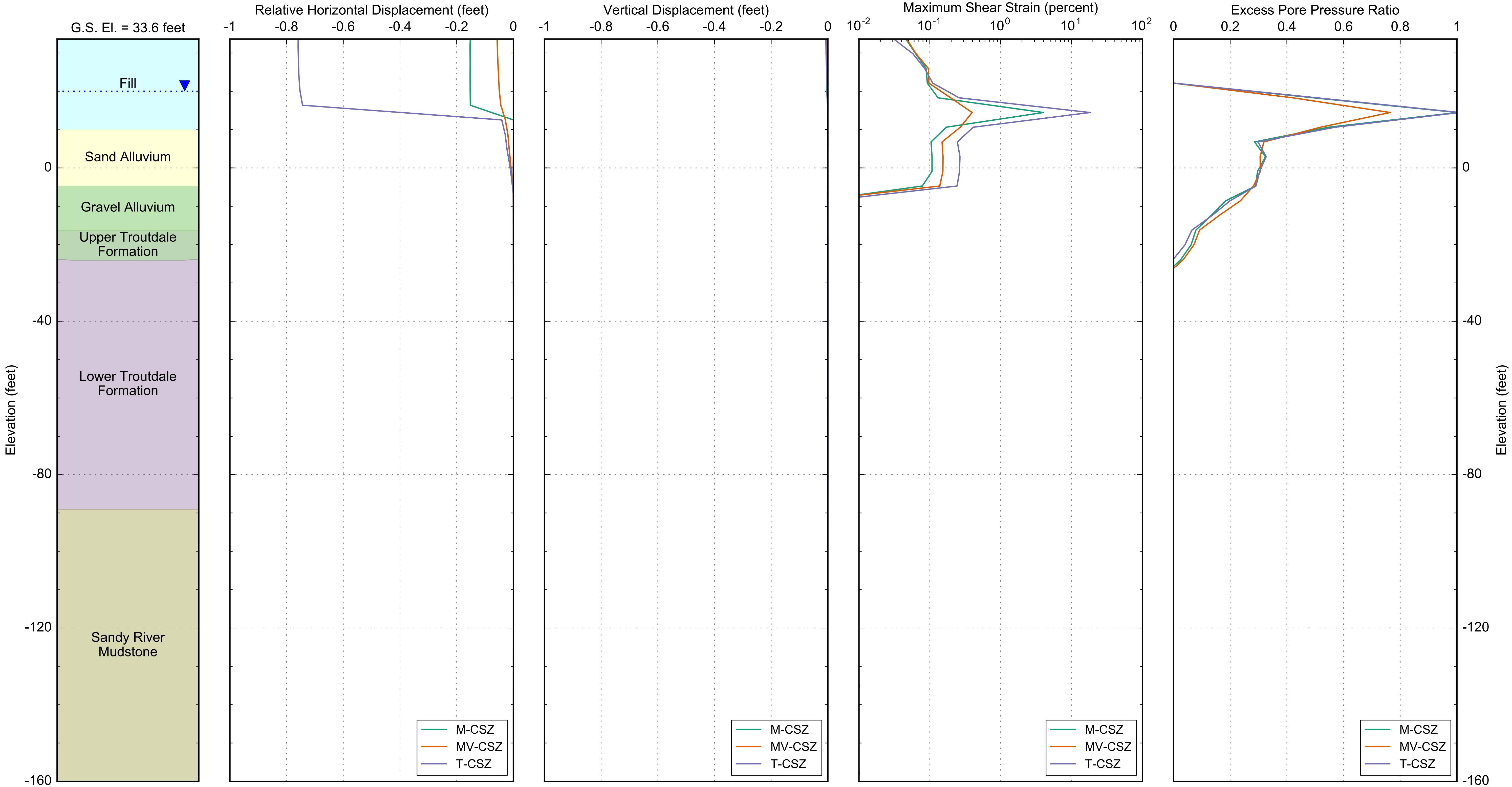
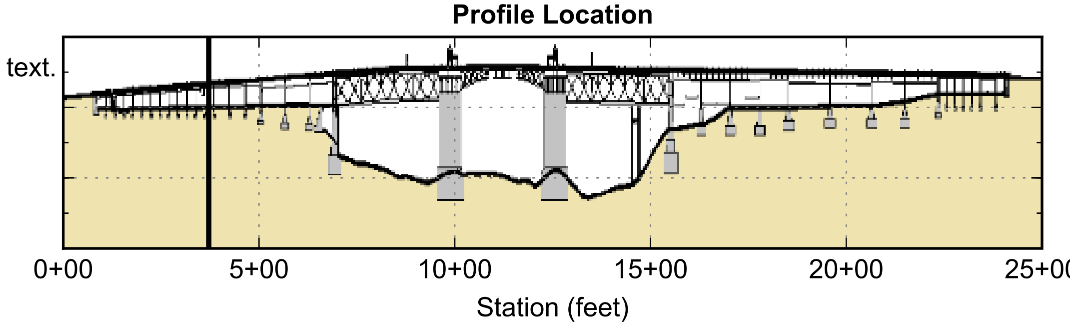
FIG. E-56



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 13

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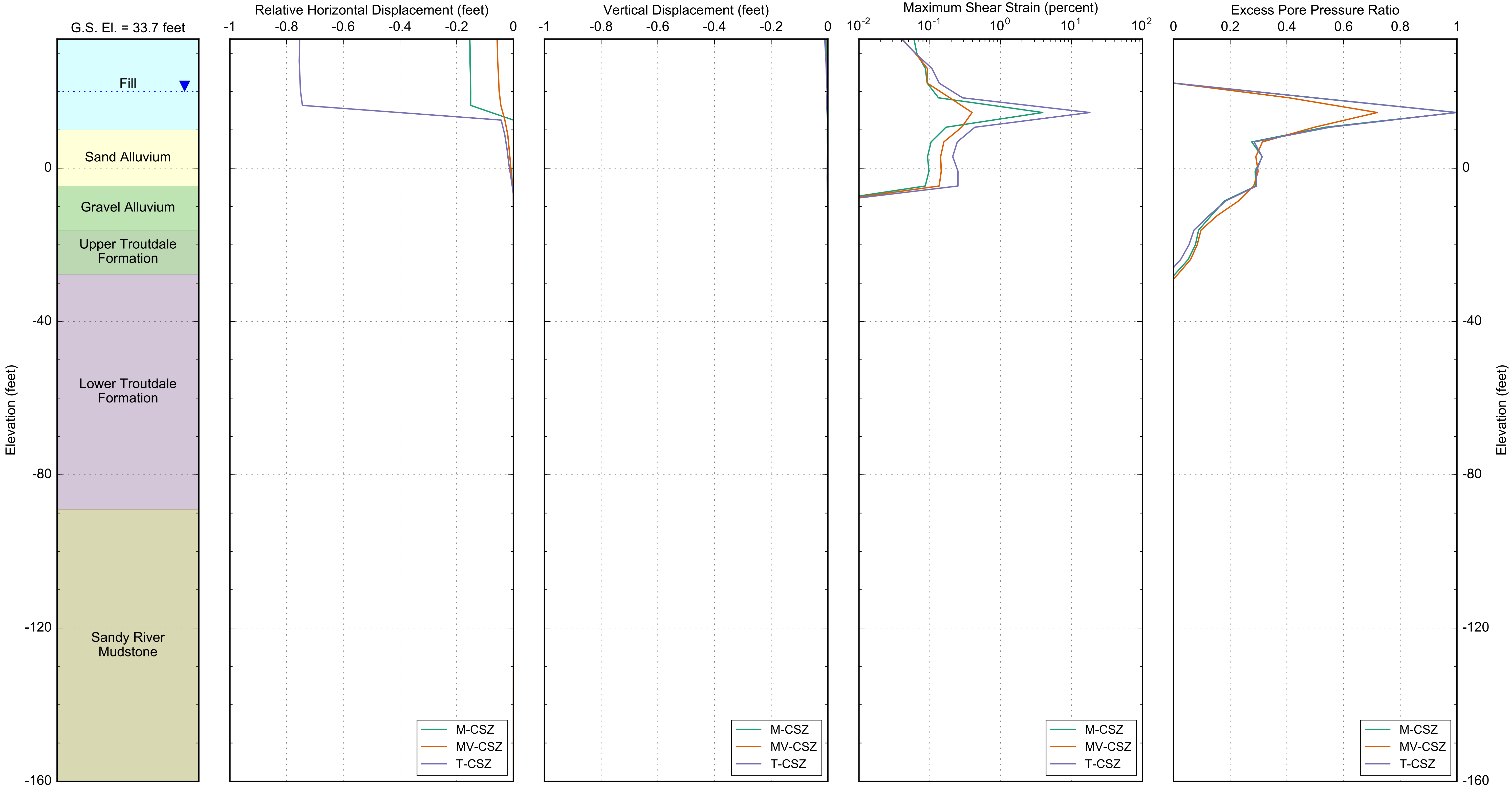
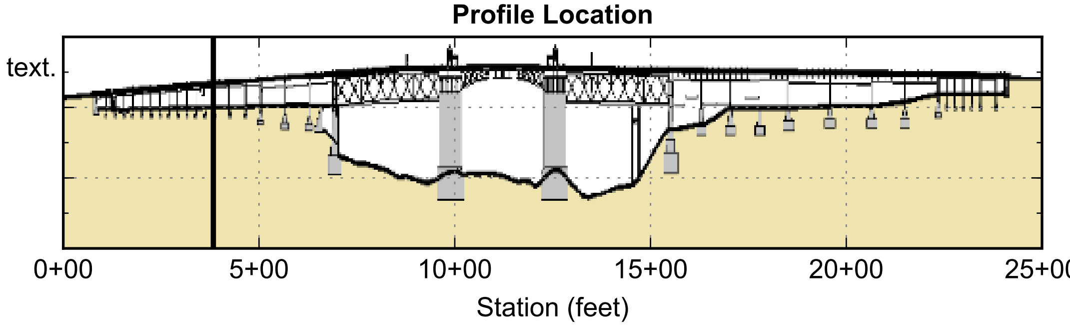
FIG. E-57



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 14

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FIG. E-58

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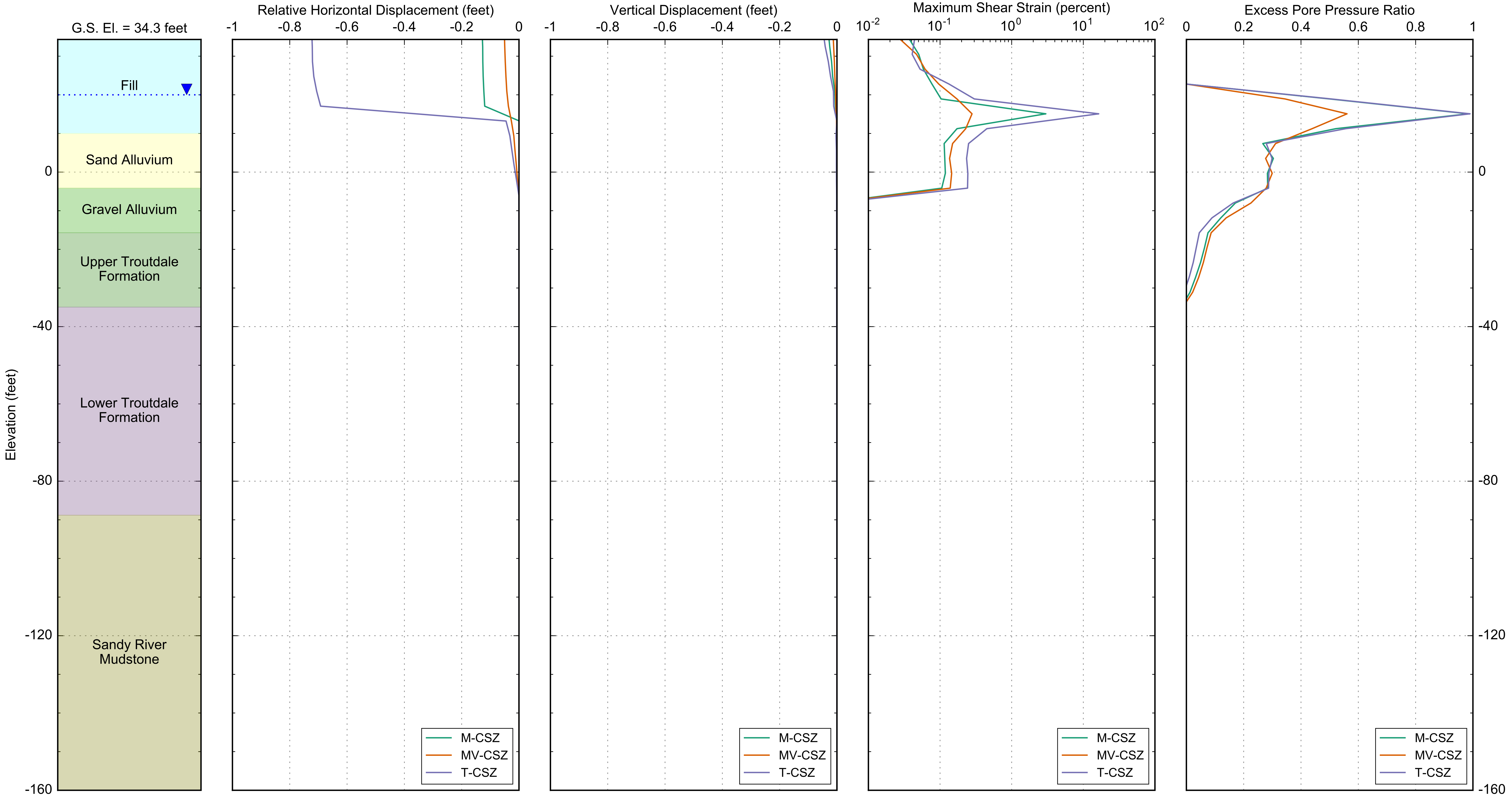
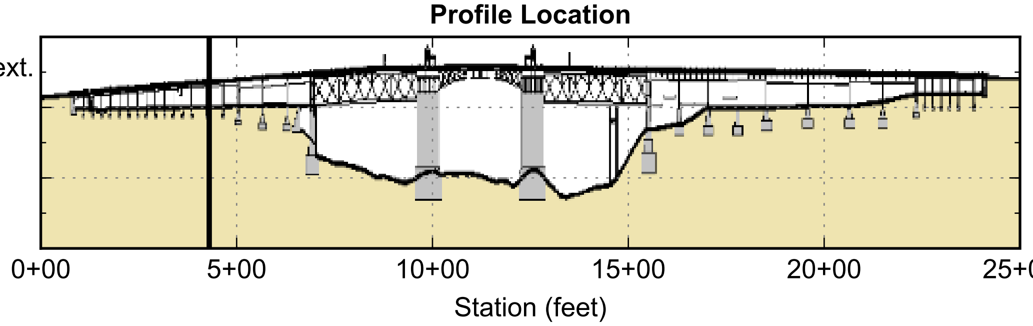


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 15

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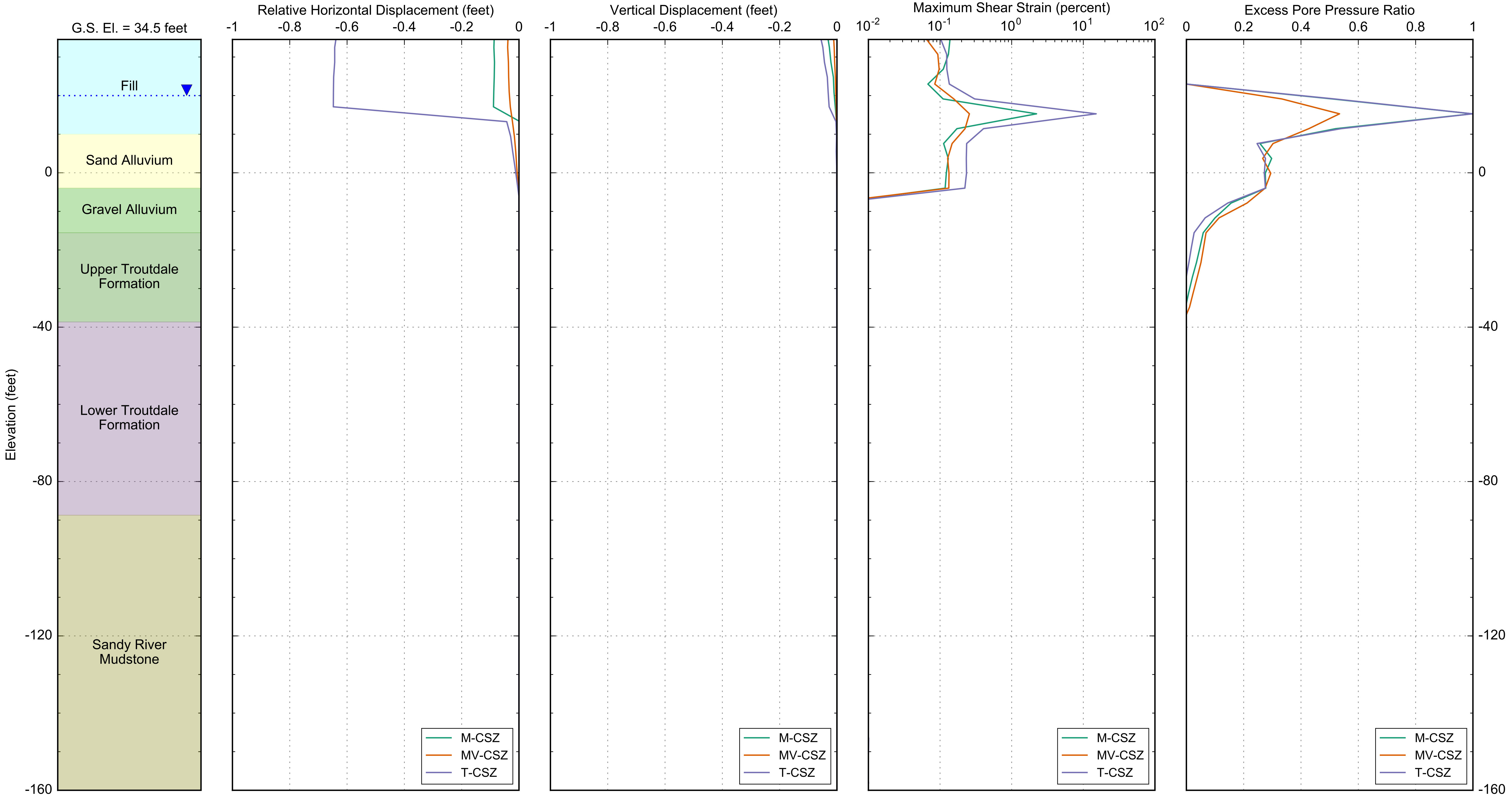
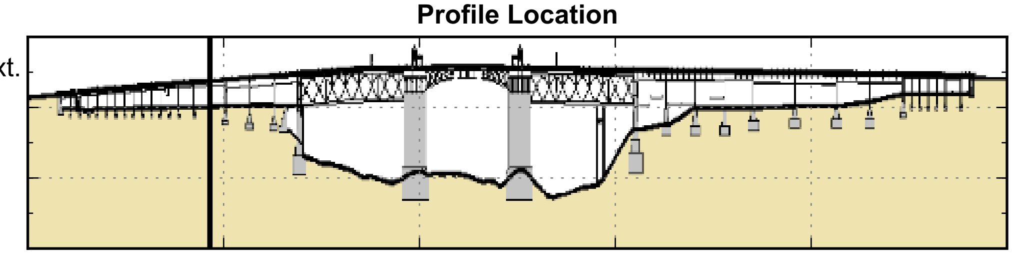
FIG. E-59



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 16

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FIG. E-60

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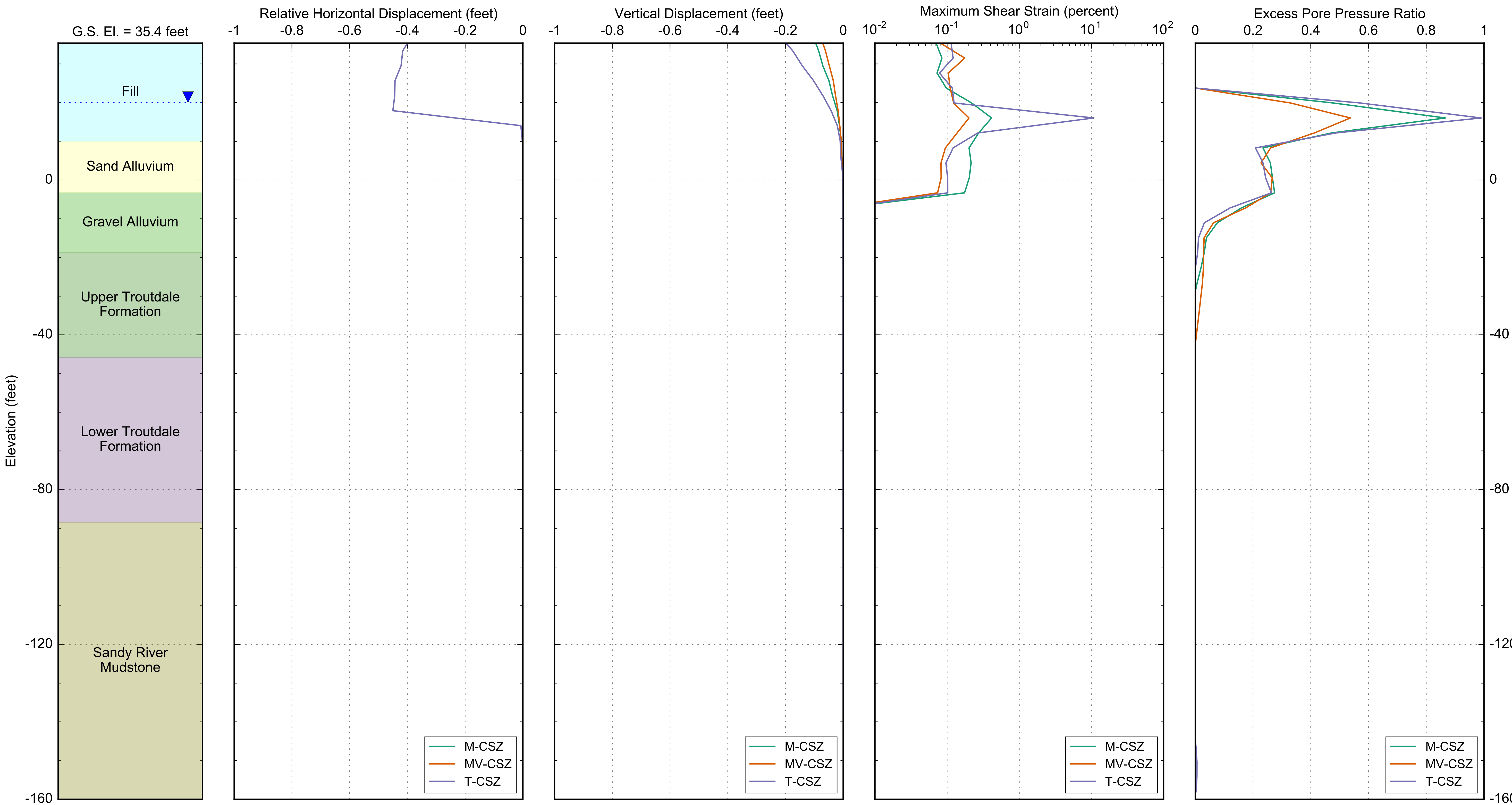
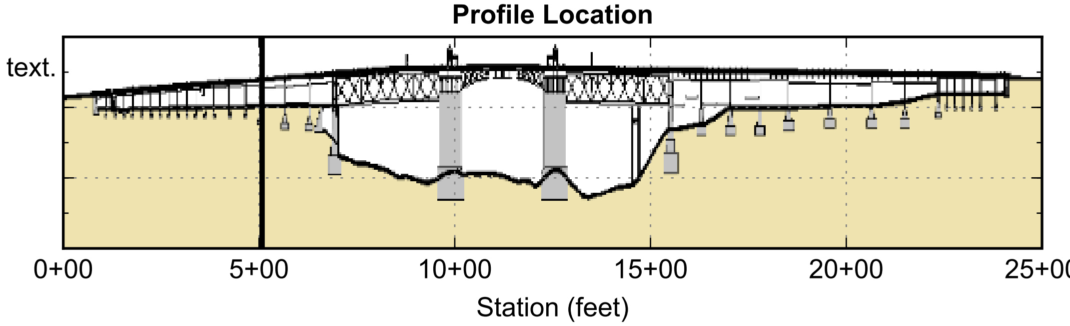


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 17

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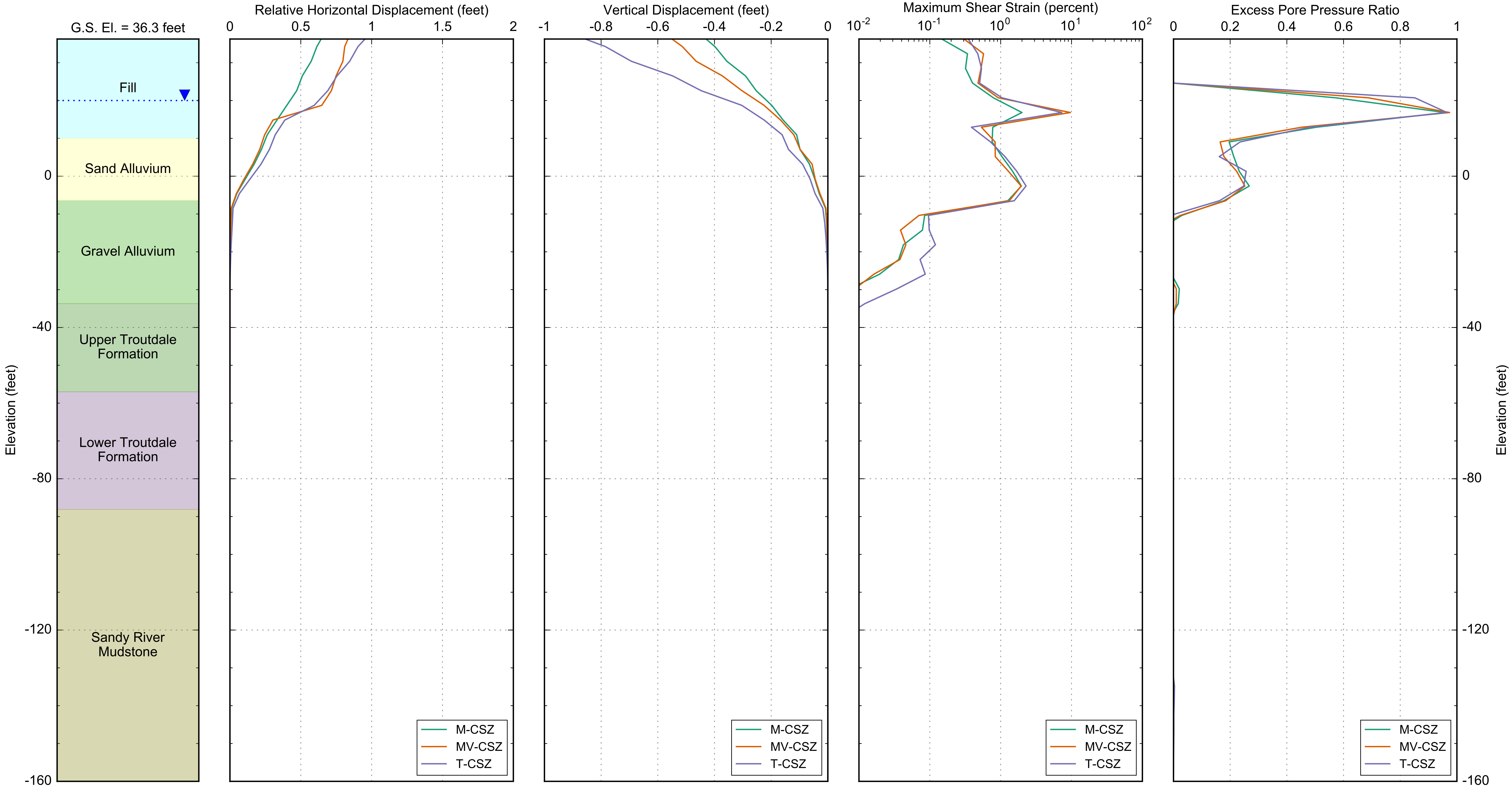
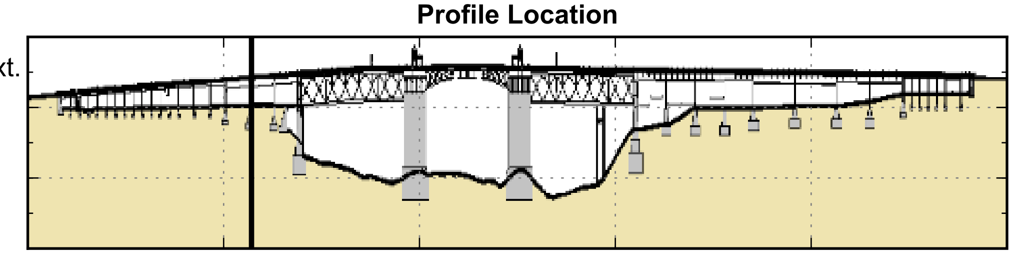
FIG. E-61



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 18

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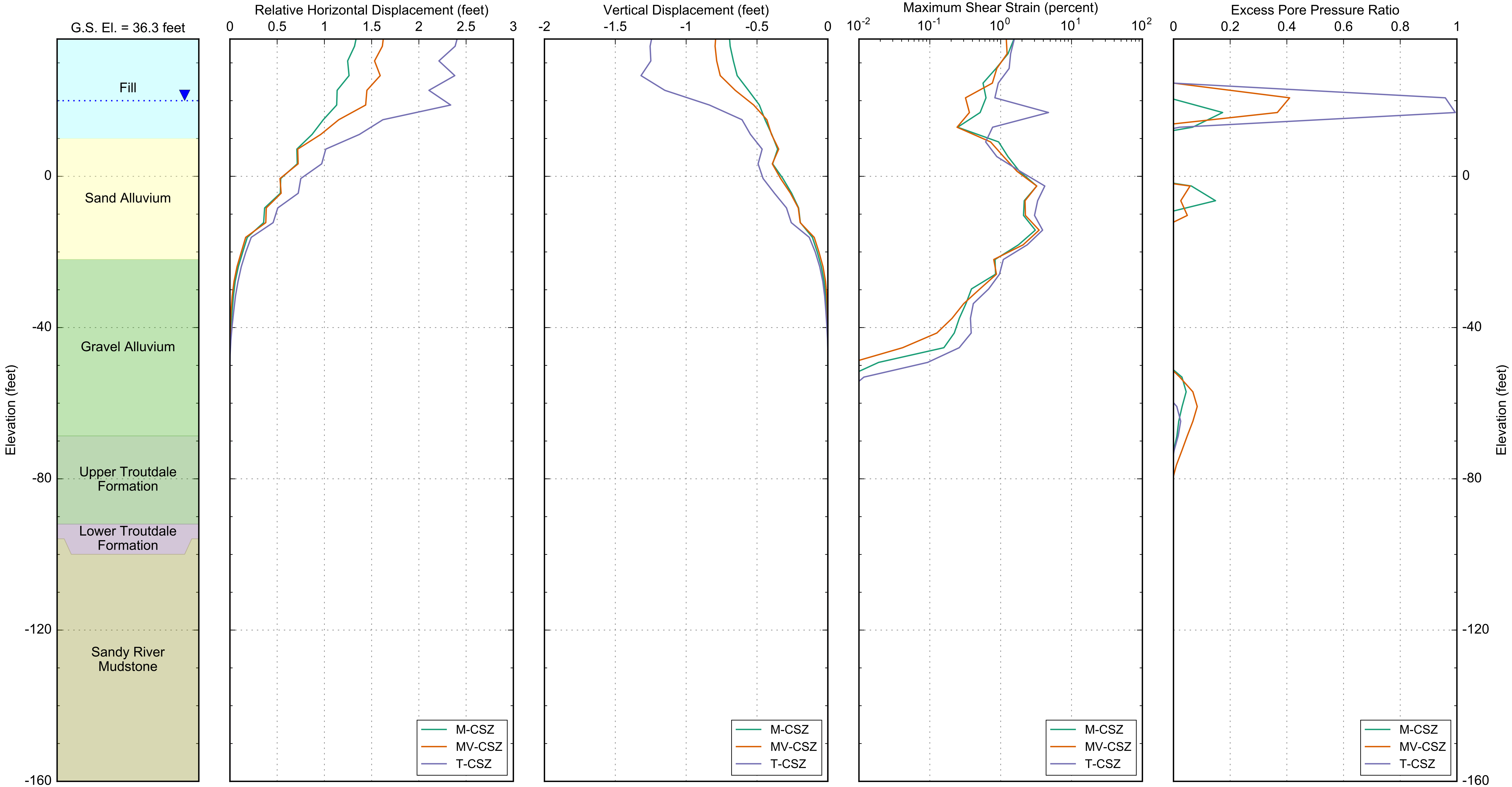
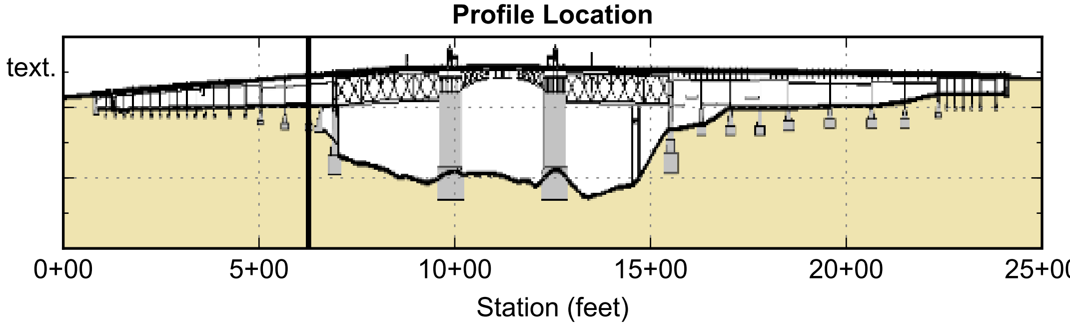
FIG. E-62



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 19

July 2017 24-1-04065-005

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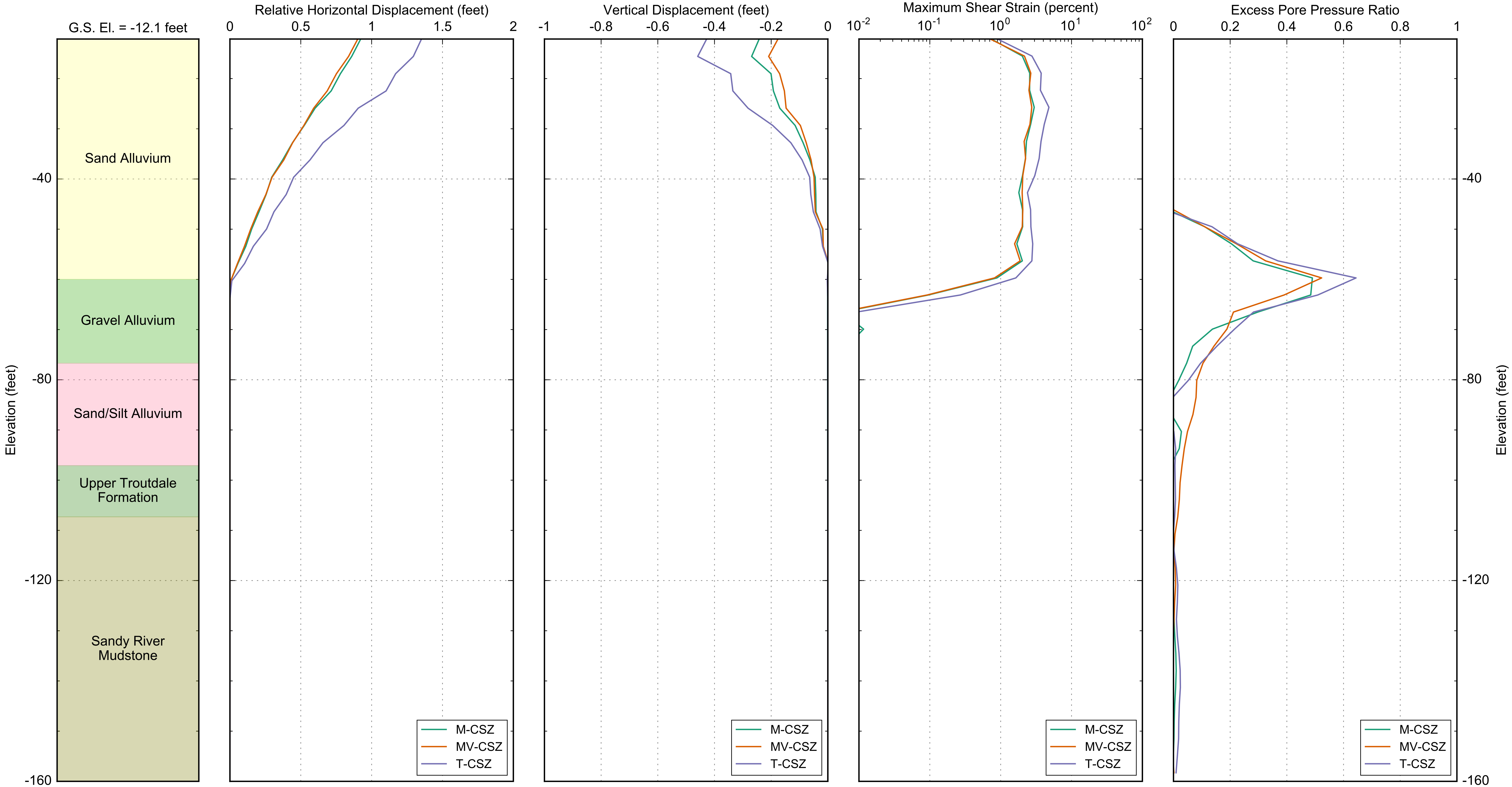
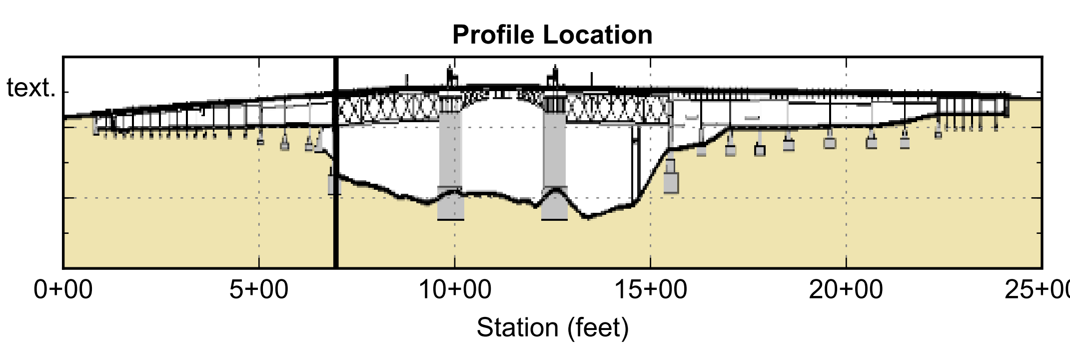
FIG. E-63





Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
PIER 1

July 2017 24-1-04065-005

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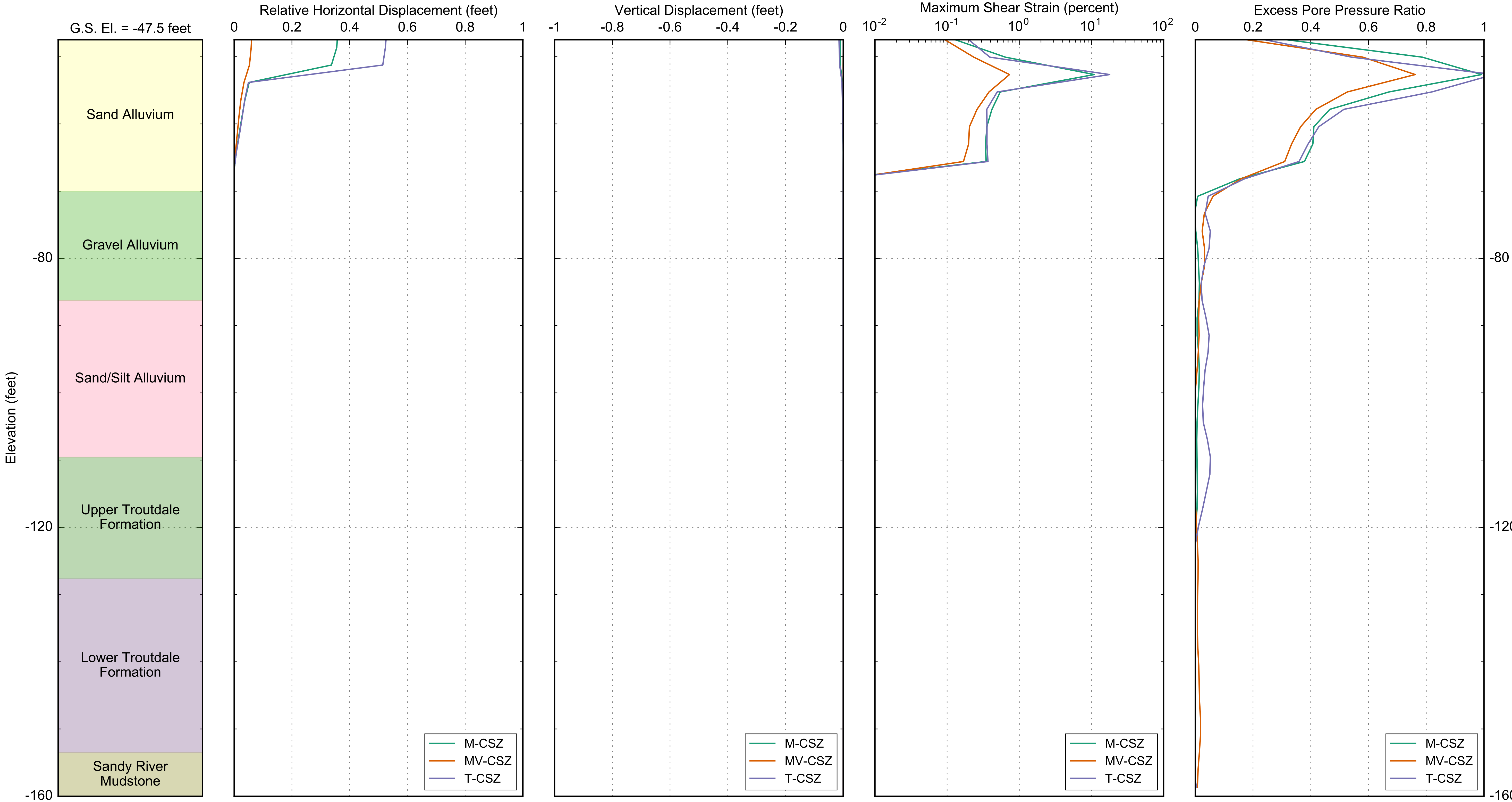
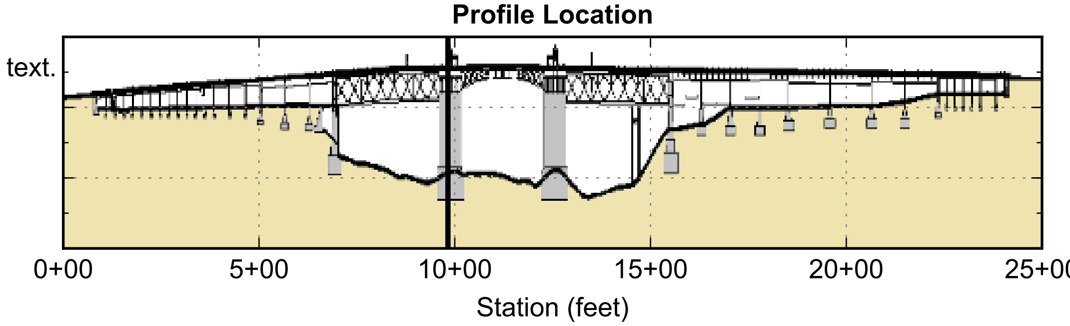
FIG. E-64



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
PIER 2

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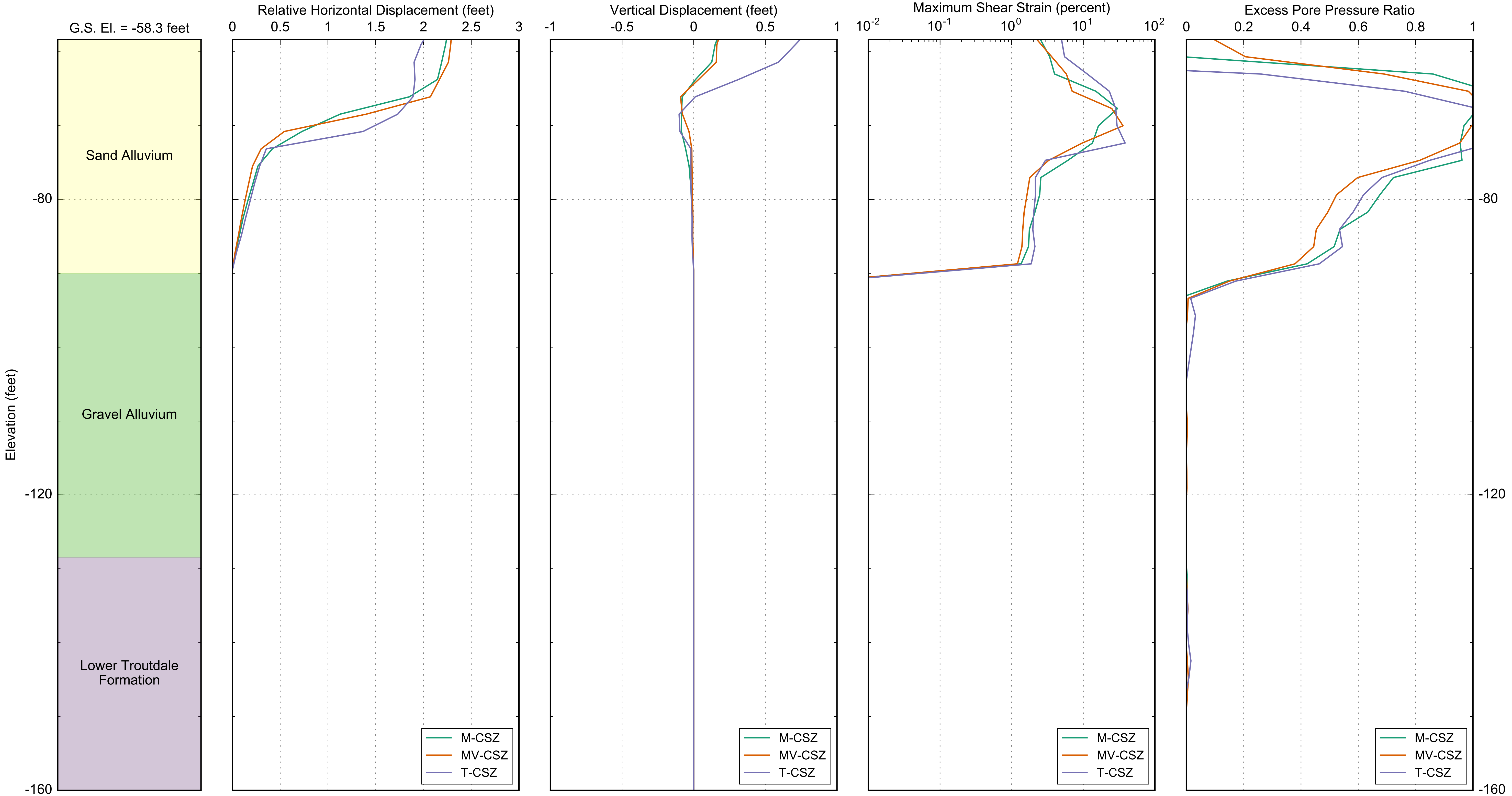
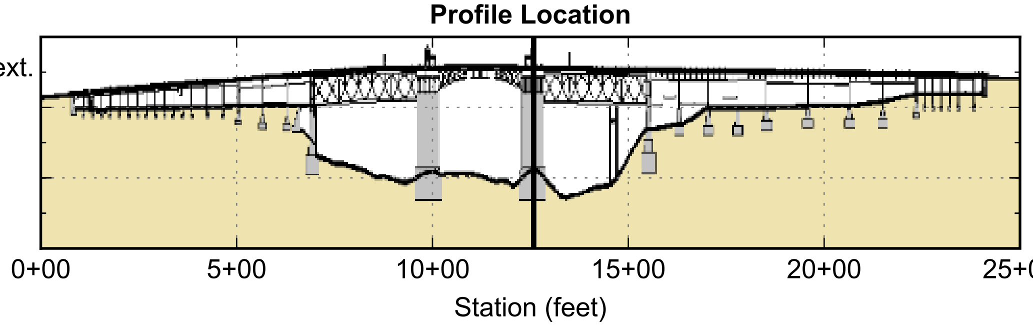
FIG. E-65



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
PIER 3

July 2017 24-1-04065-005

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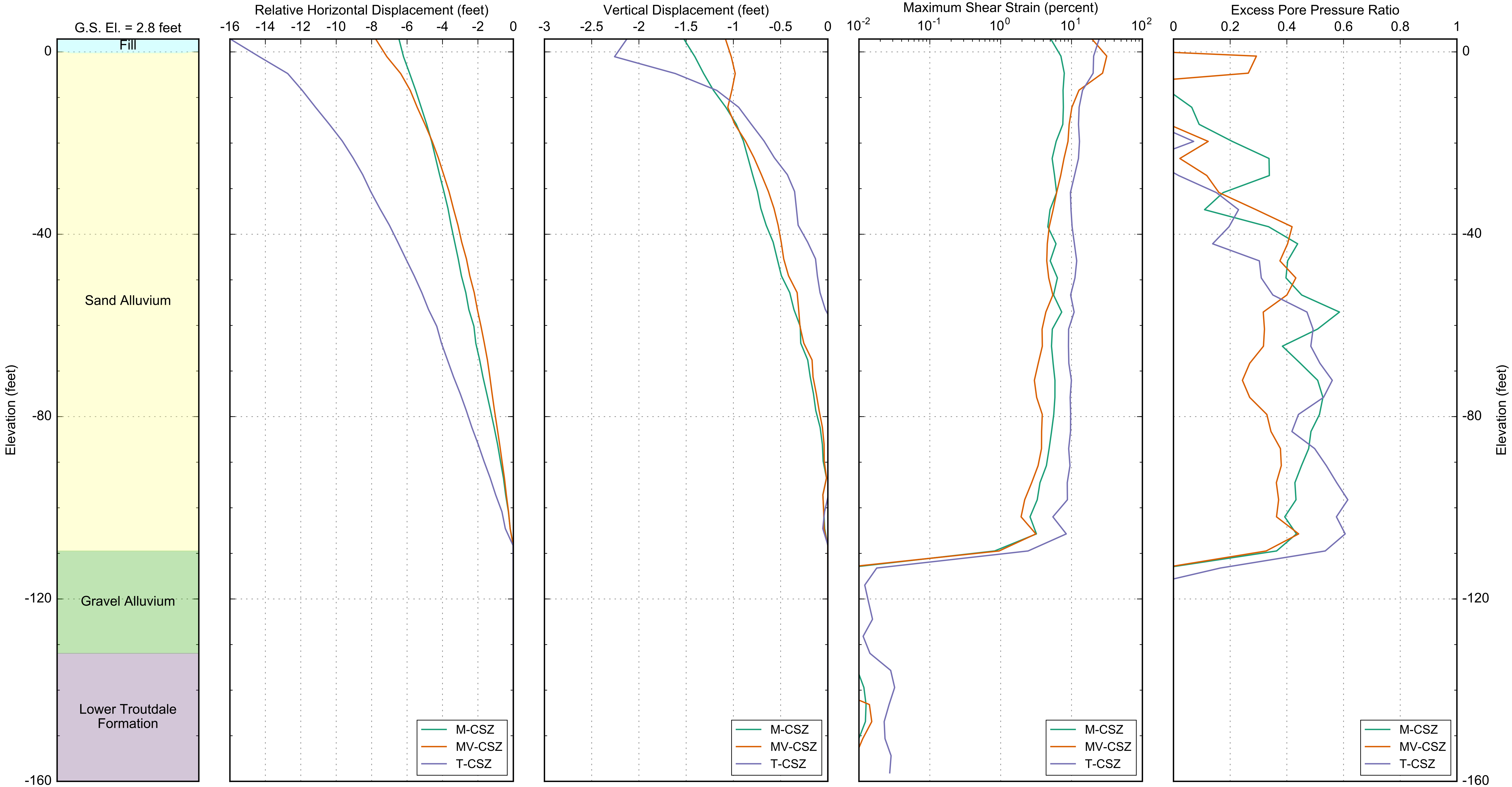
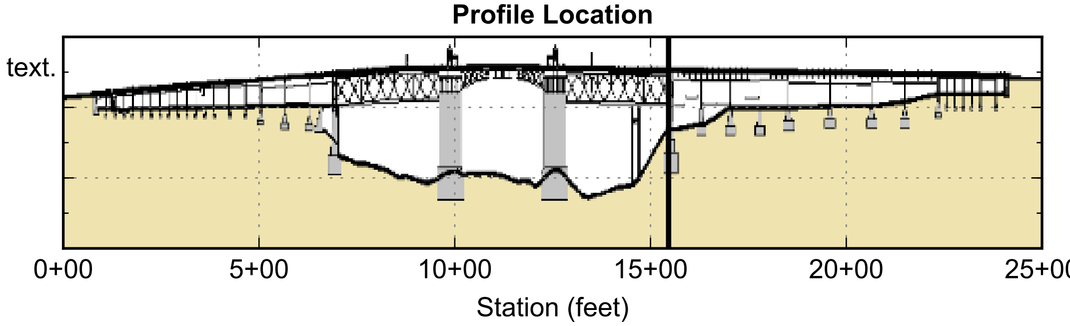
FIG. E-66



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
Portland, Oregon

SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
PIER 4

July 2017 24-1-04065-005

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FIG. E-67

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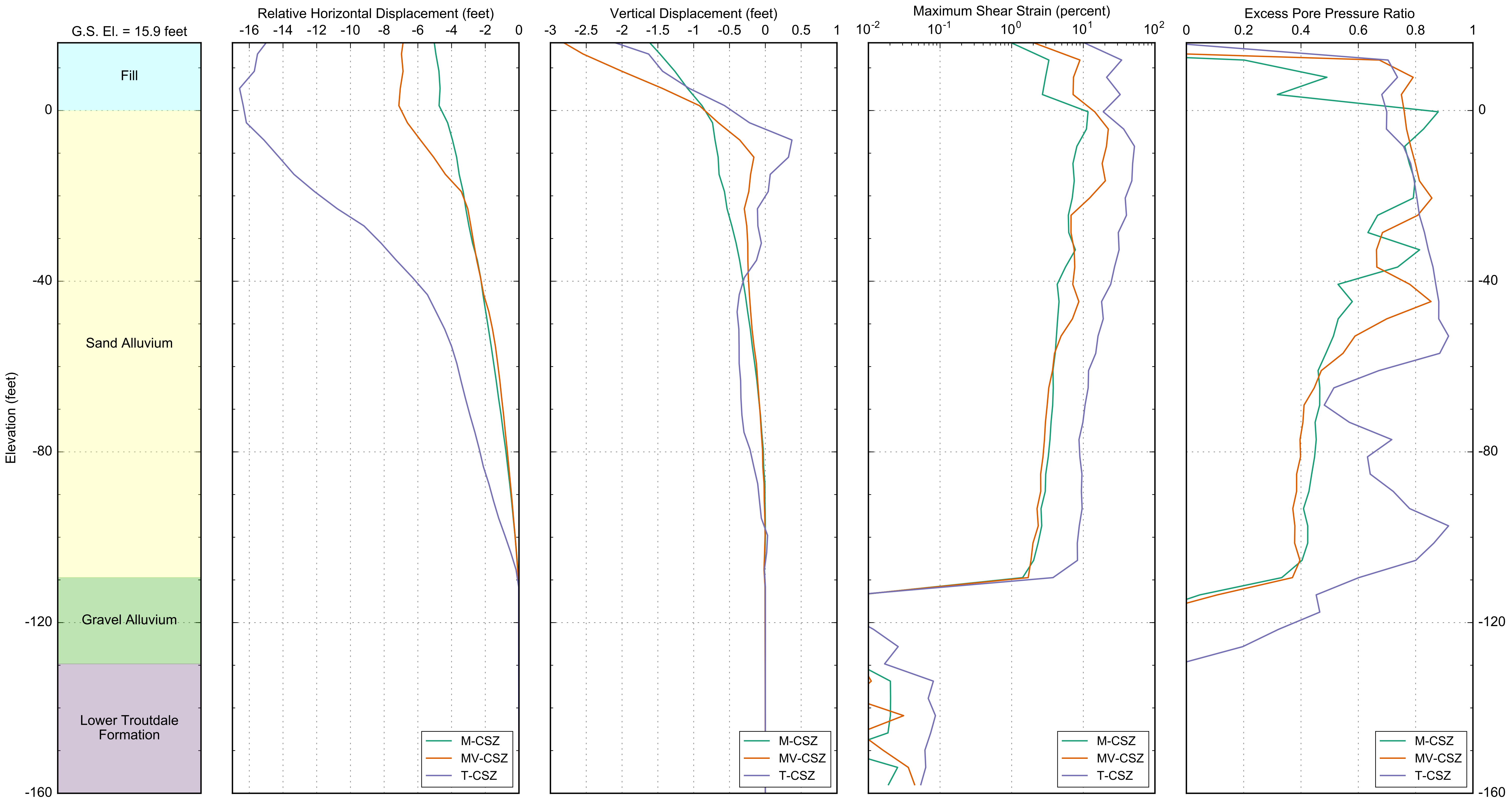
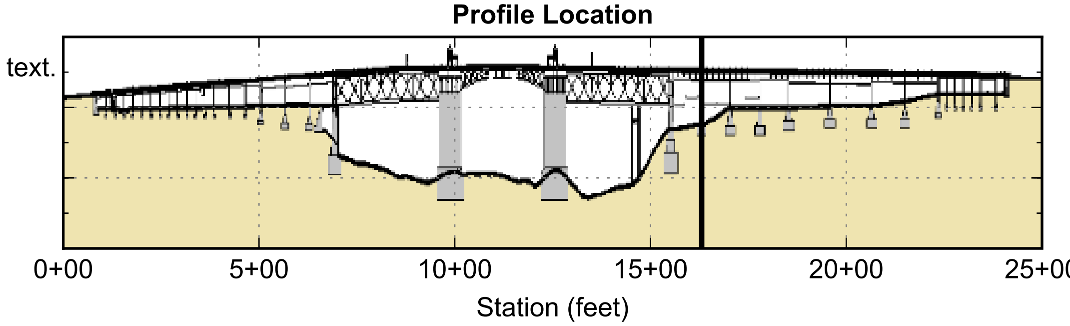


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



Burnside Bridge
Seismic Feasibility Study
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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 21

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FIG. E-68

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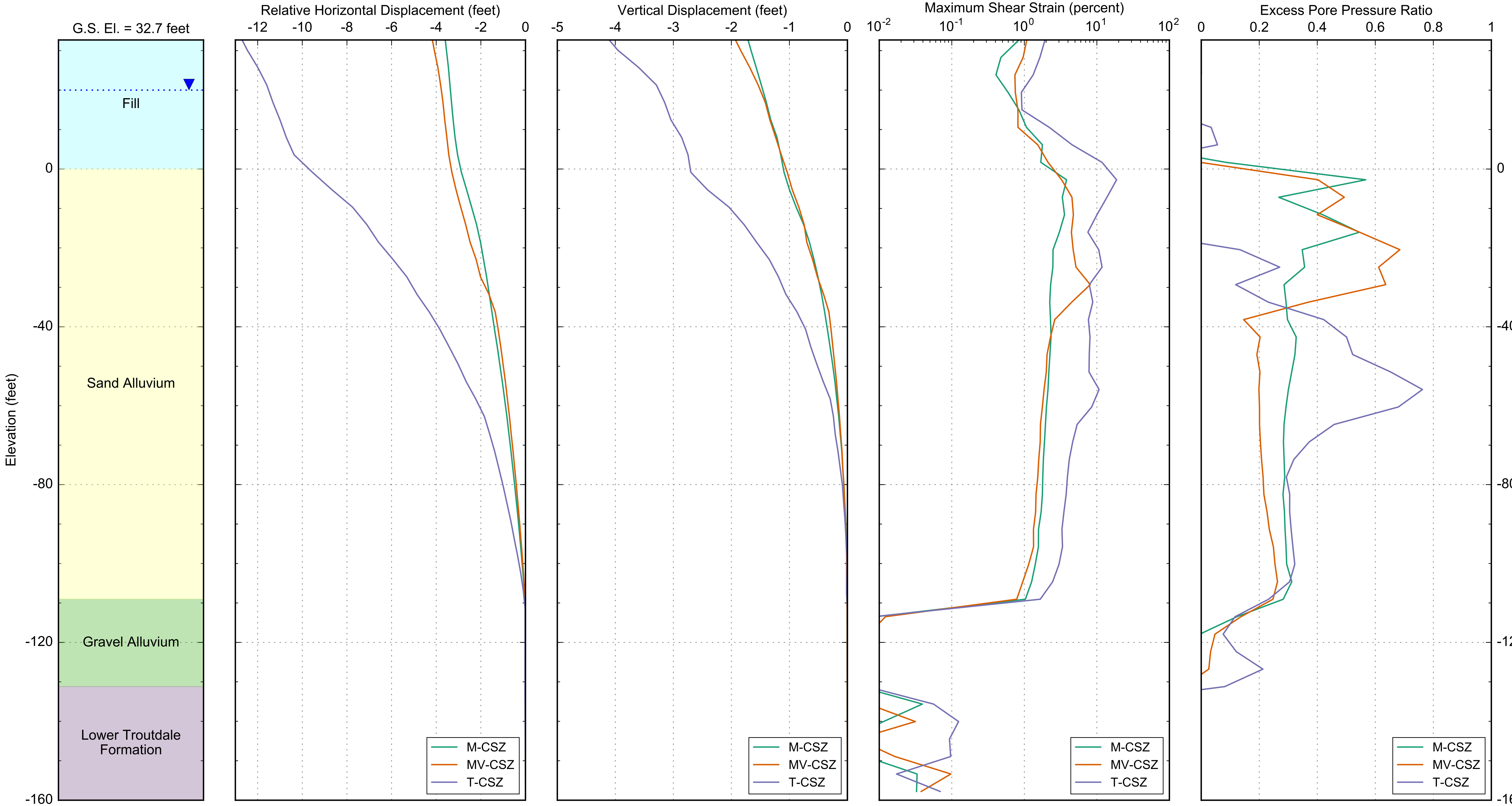
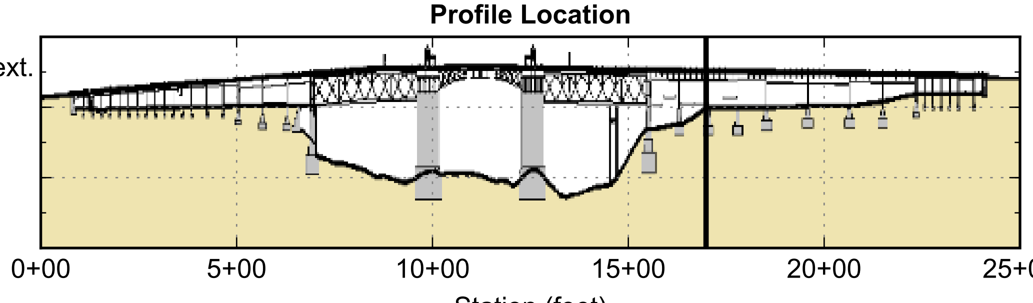


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 22

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FIG. E-69

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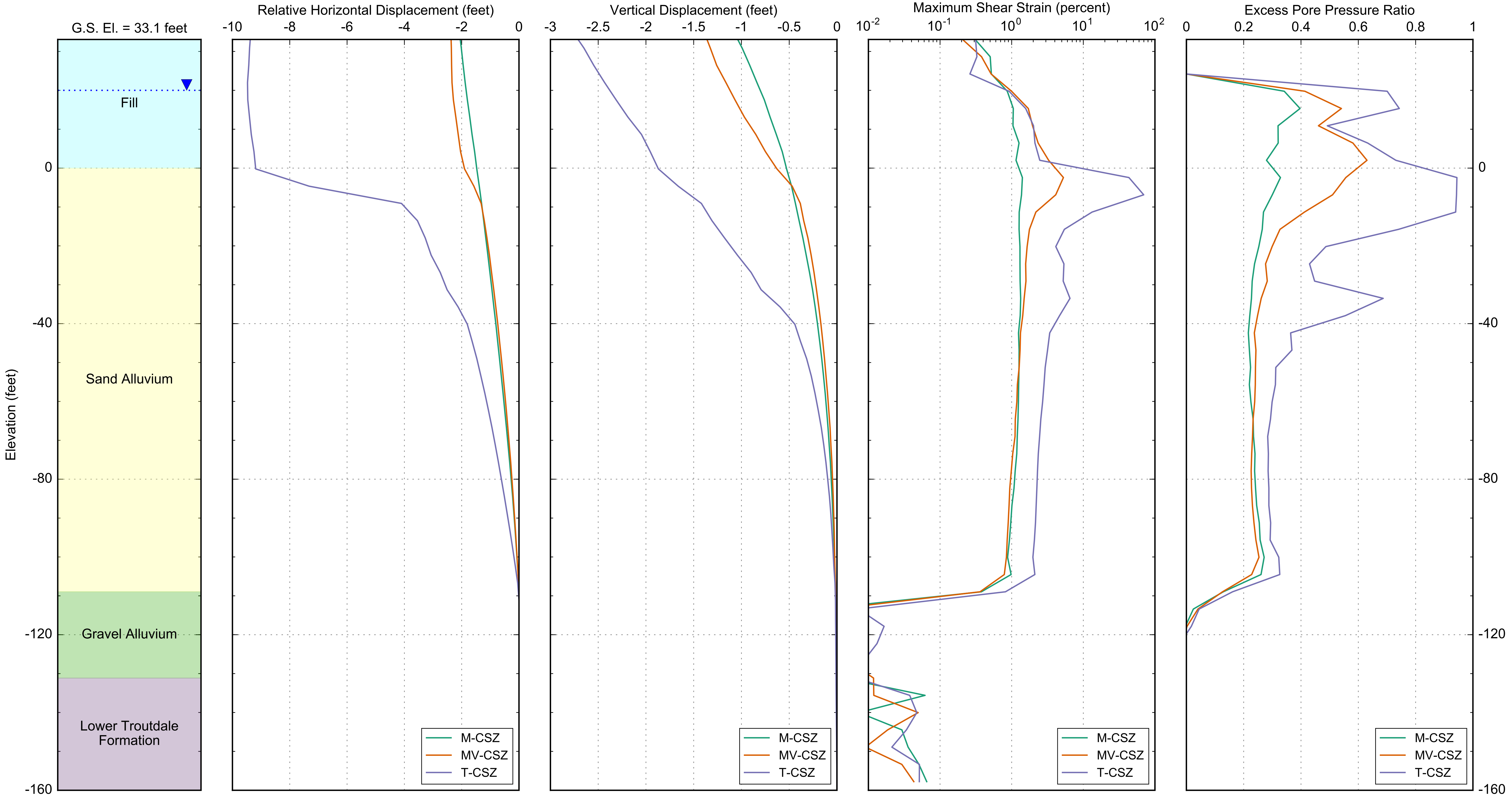
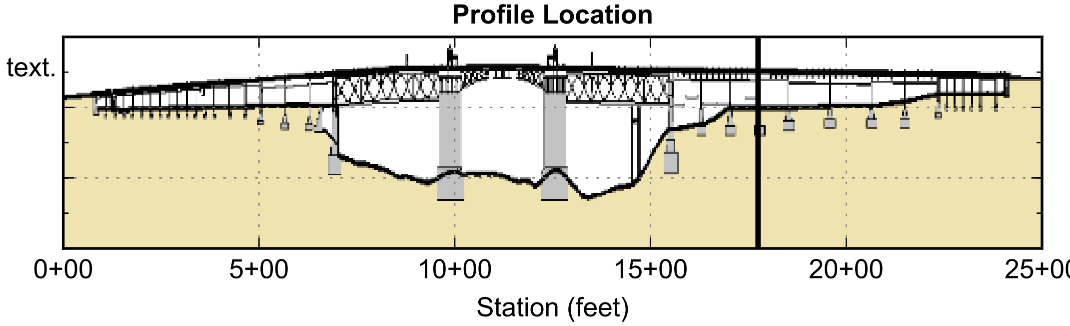


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



Burnside Bridge
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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 23

July 2017 24-1-04065-005

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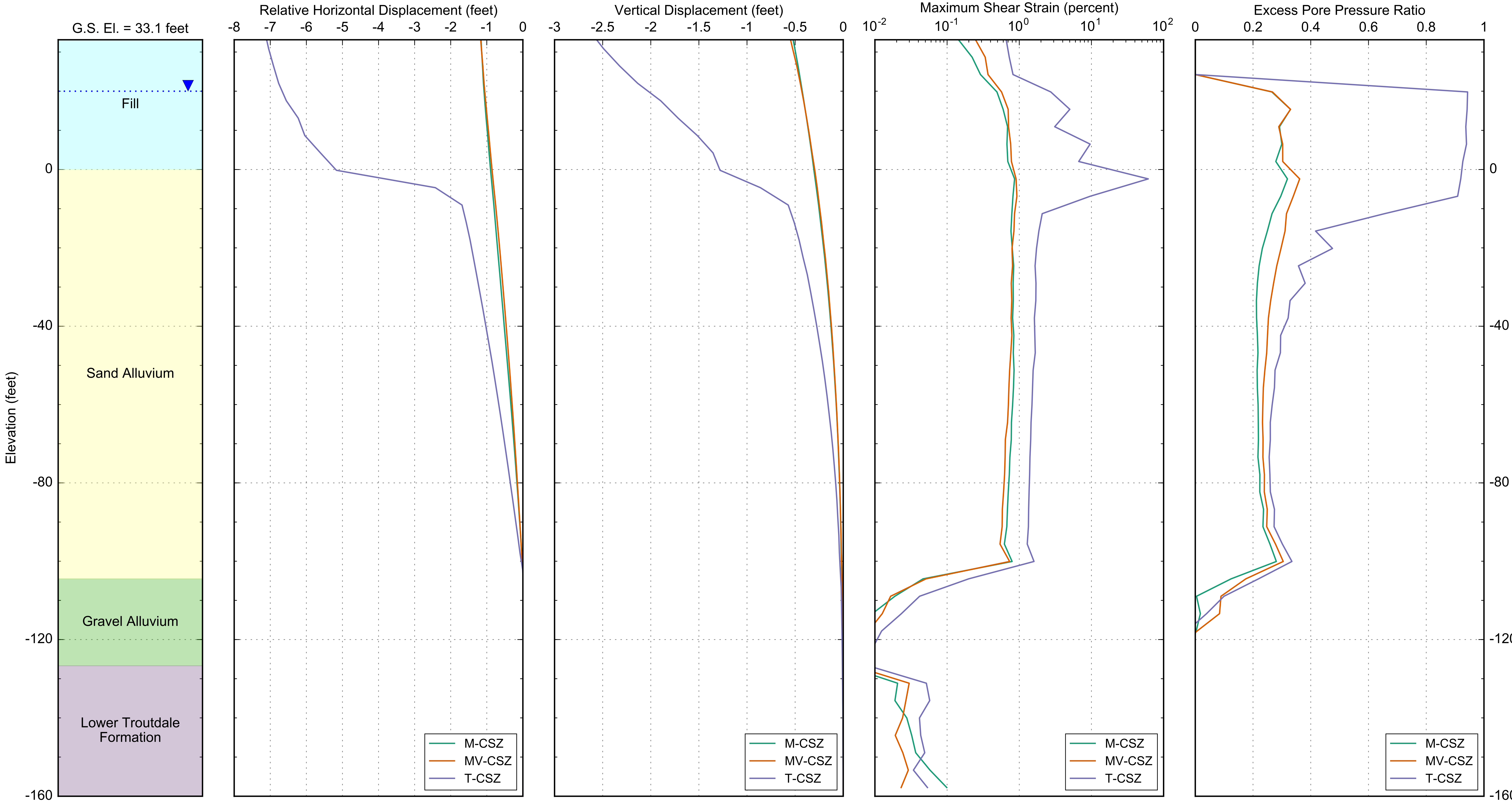
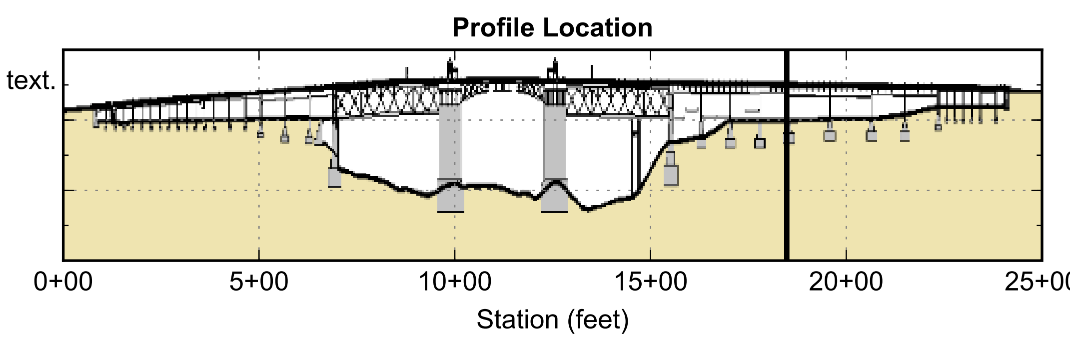
FIG. E-70



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 24

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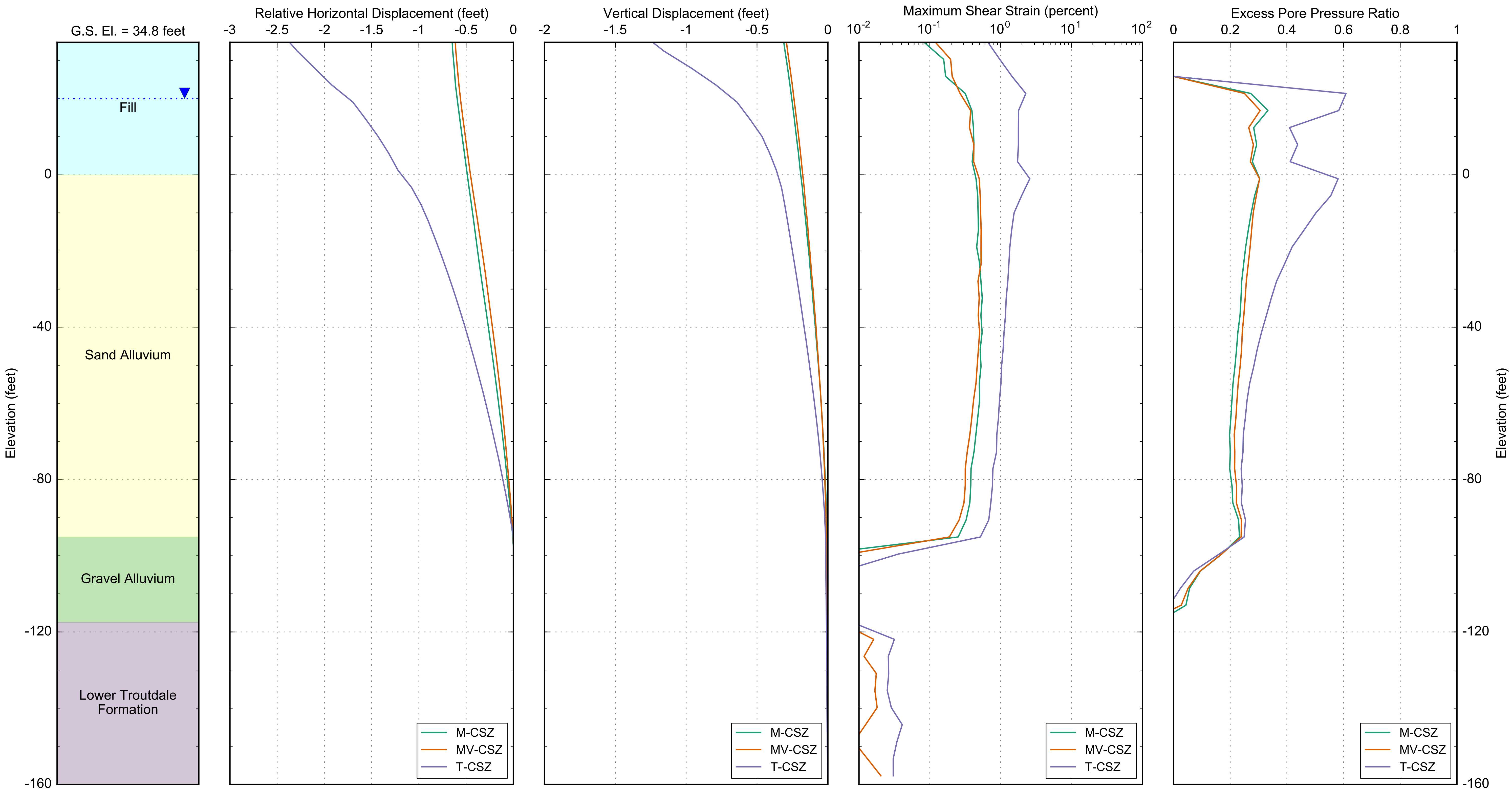
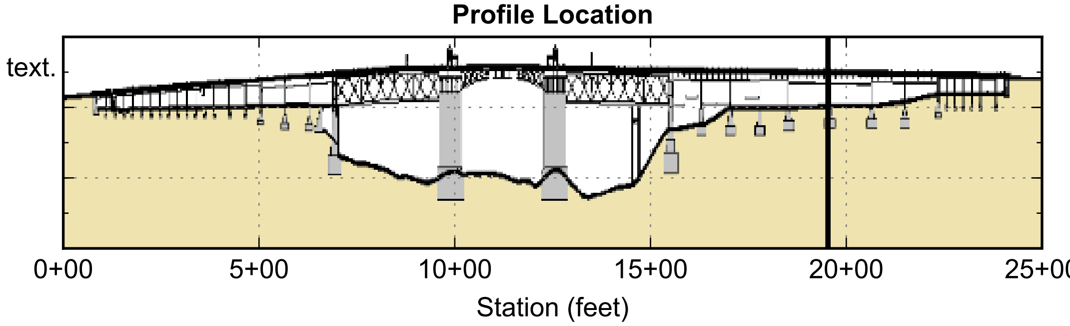
FIG. E-71



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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**SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 25**

July 2017 24-1-04065-005

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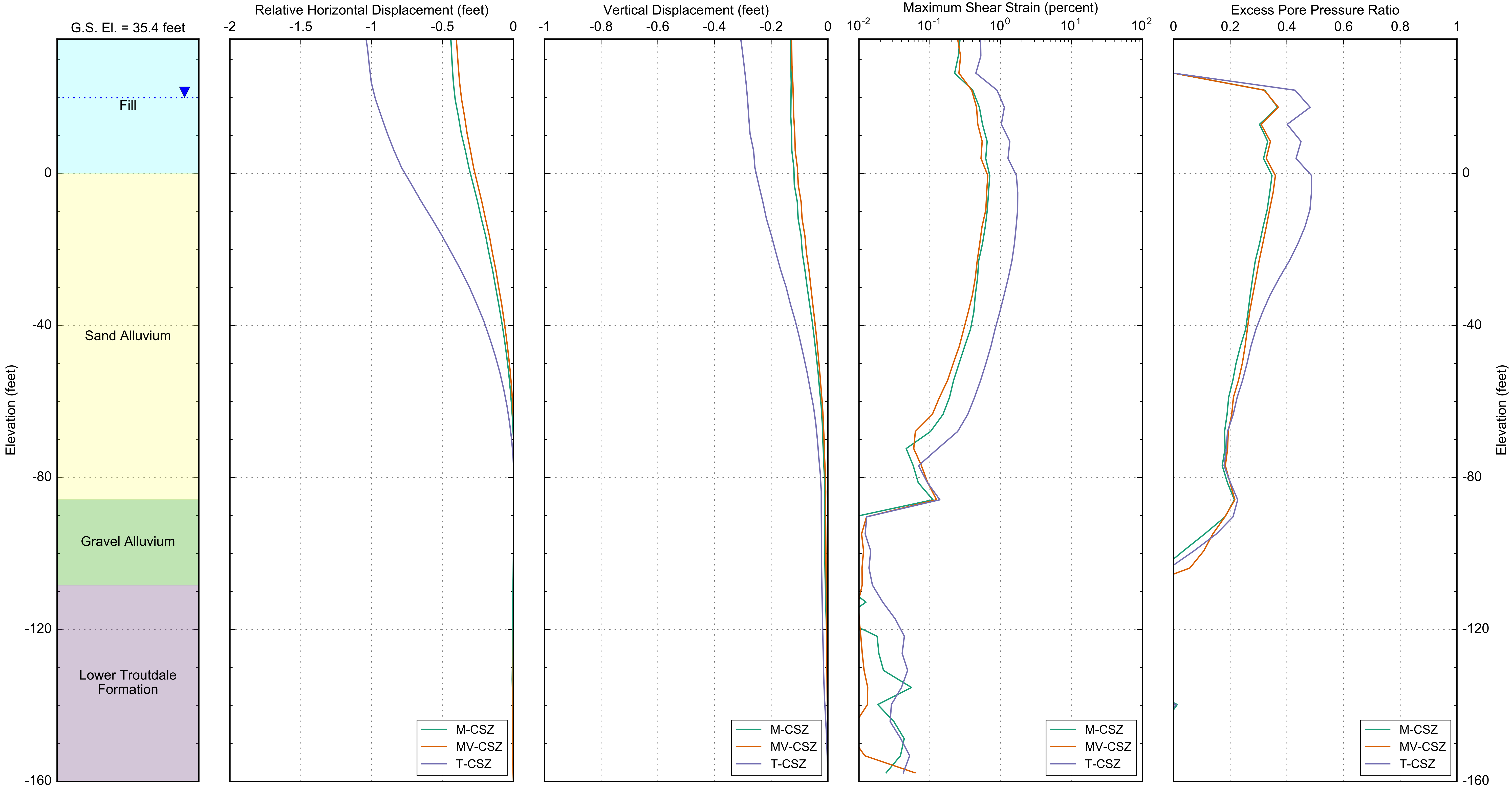
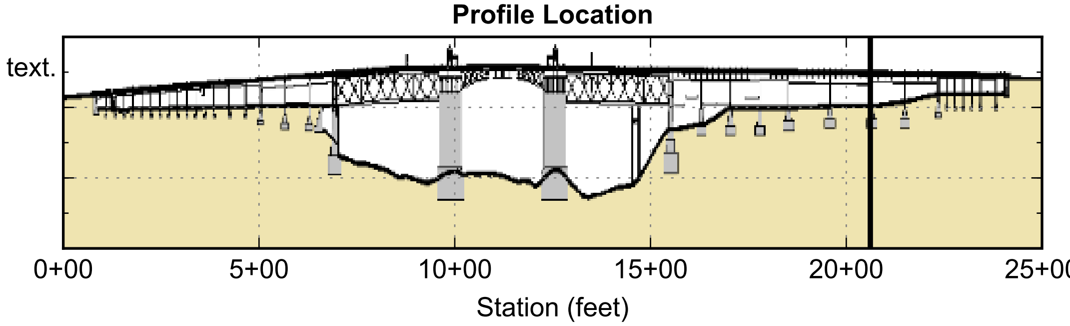
FIG. E-72



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
   psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 26

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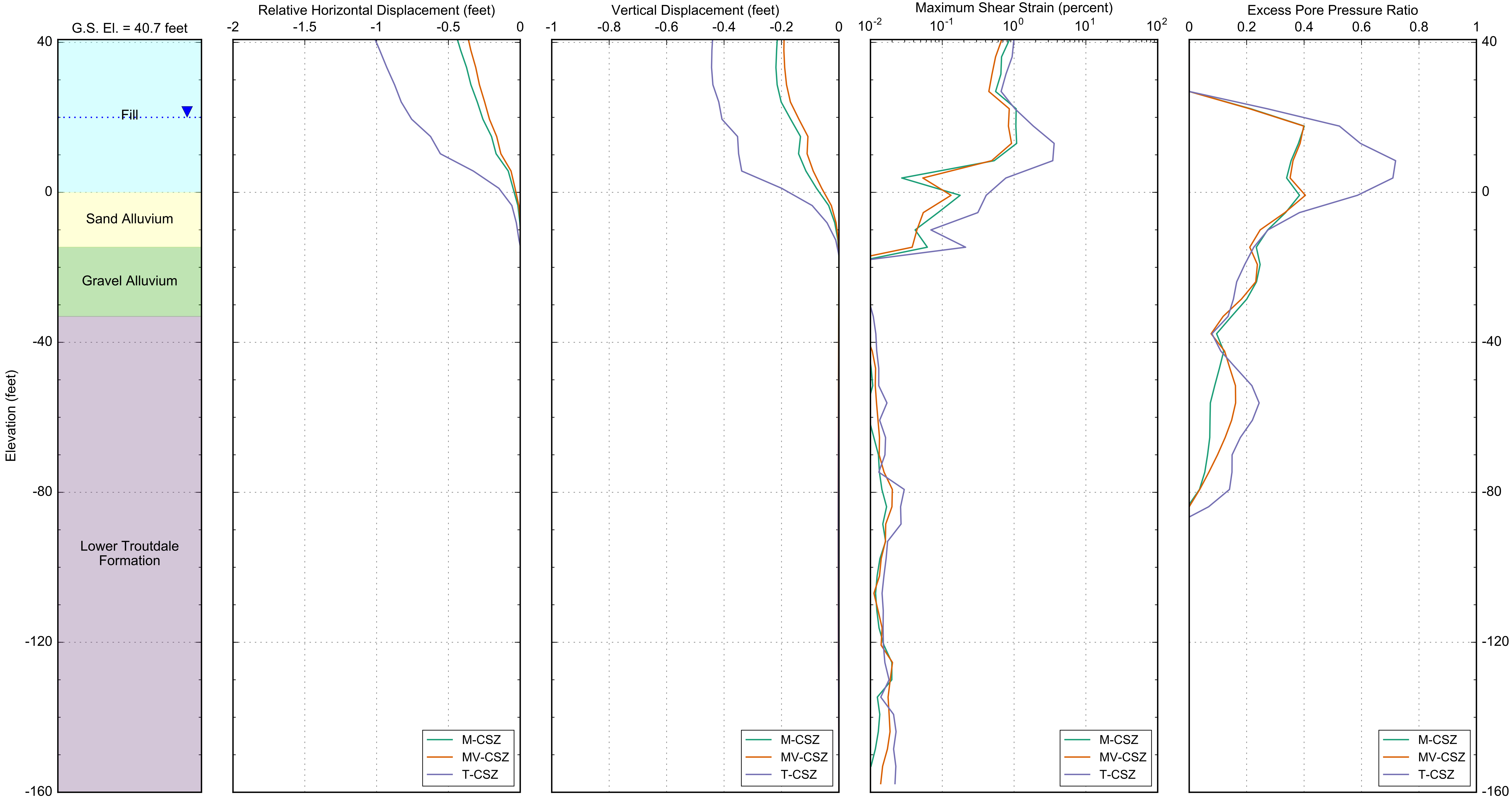
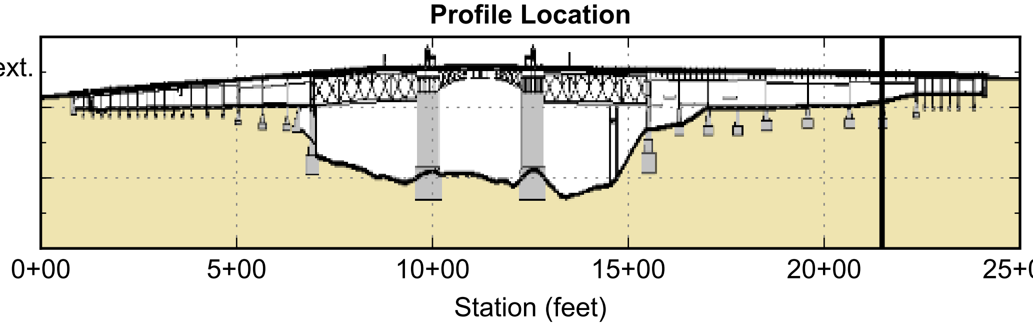
FIG. E-73



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 27

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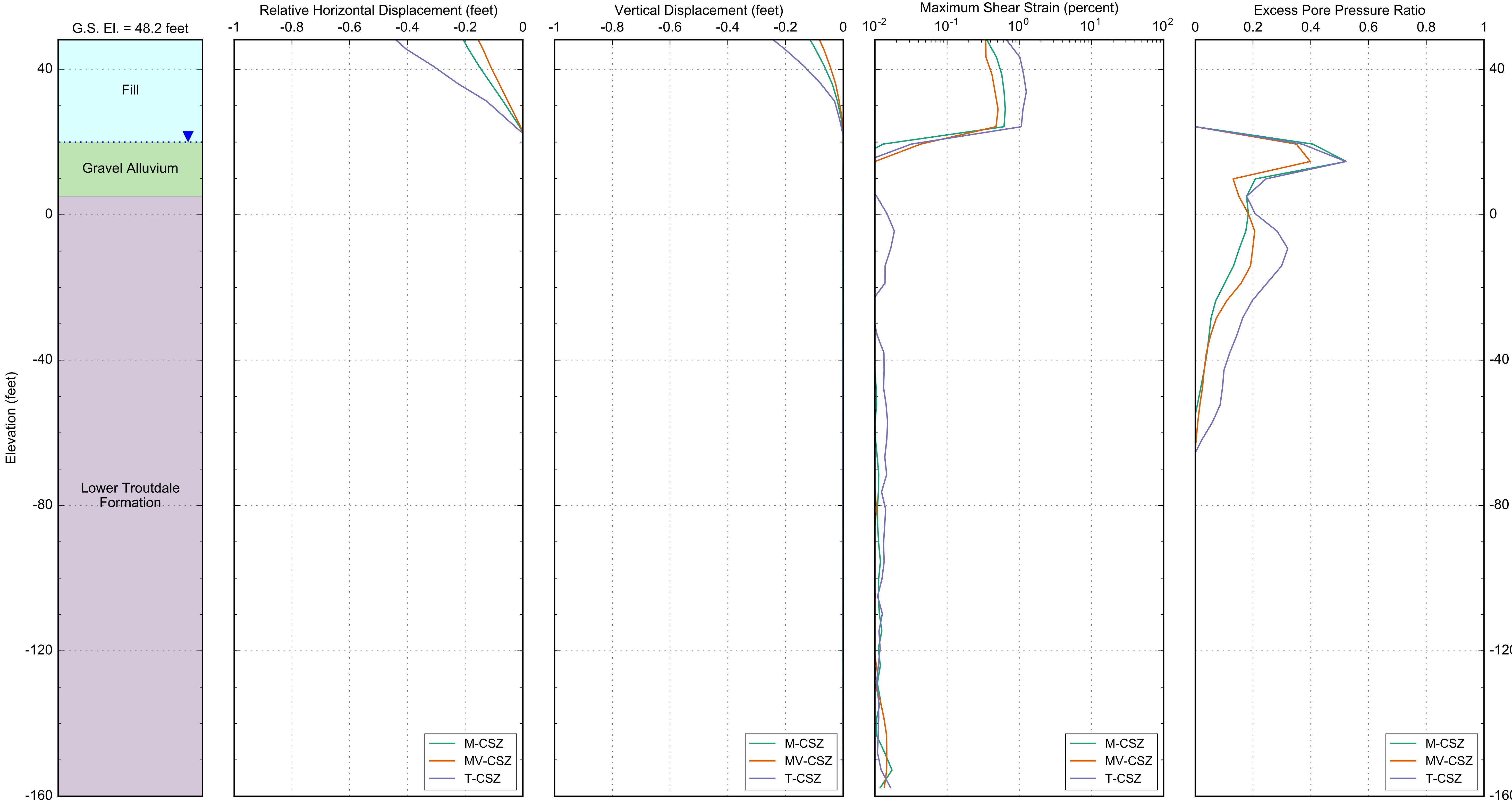
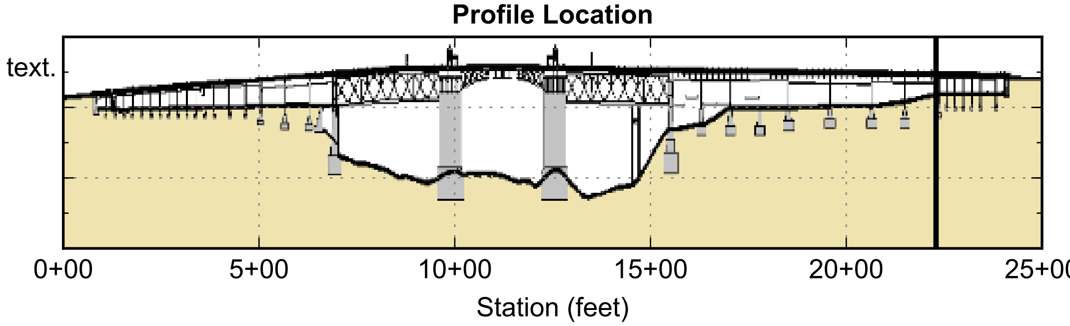
FIG. E-74



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 28

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FIG. E-75

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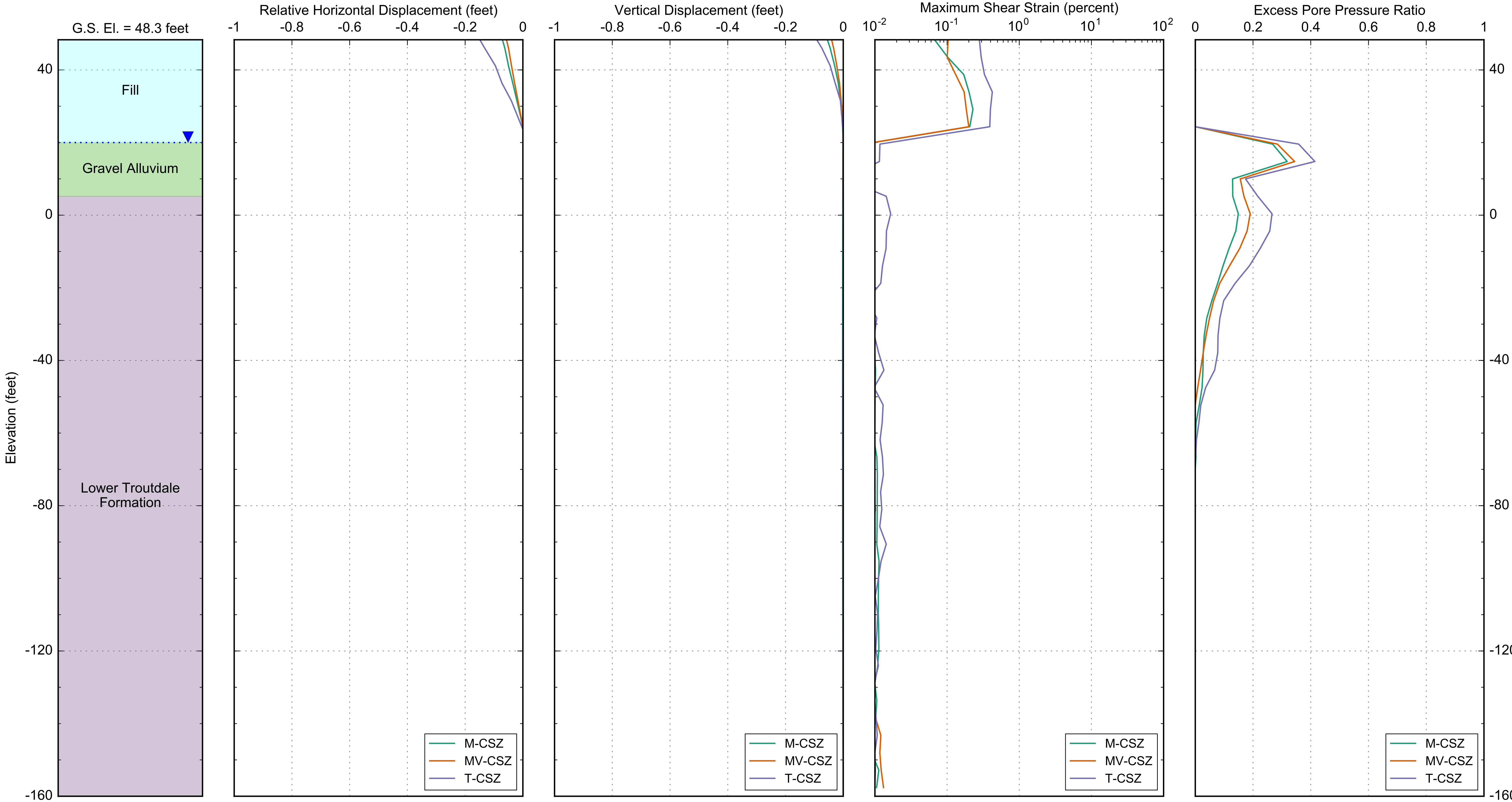
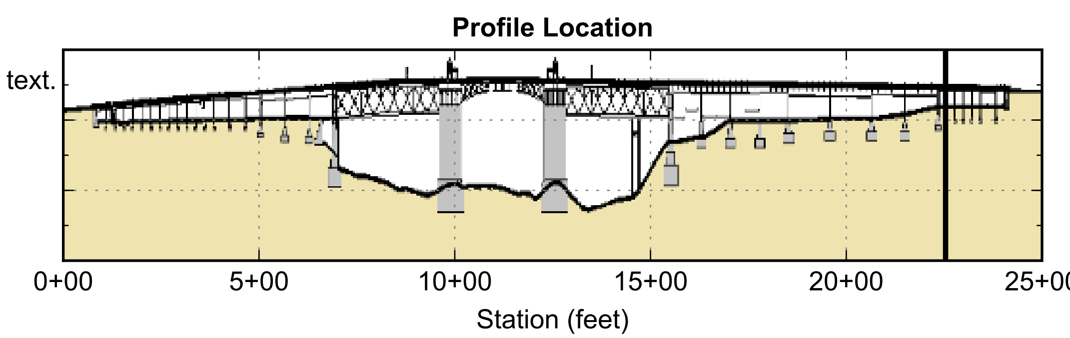


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 29

July 2017 24-1-04065-005

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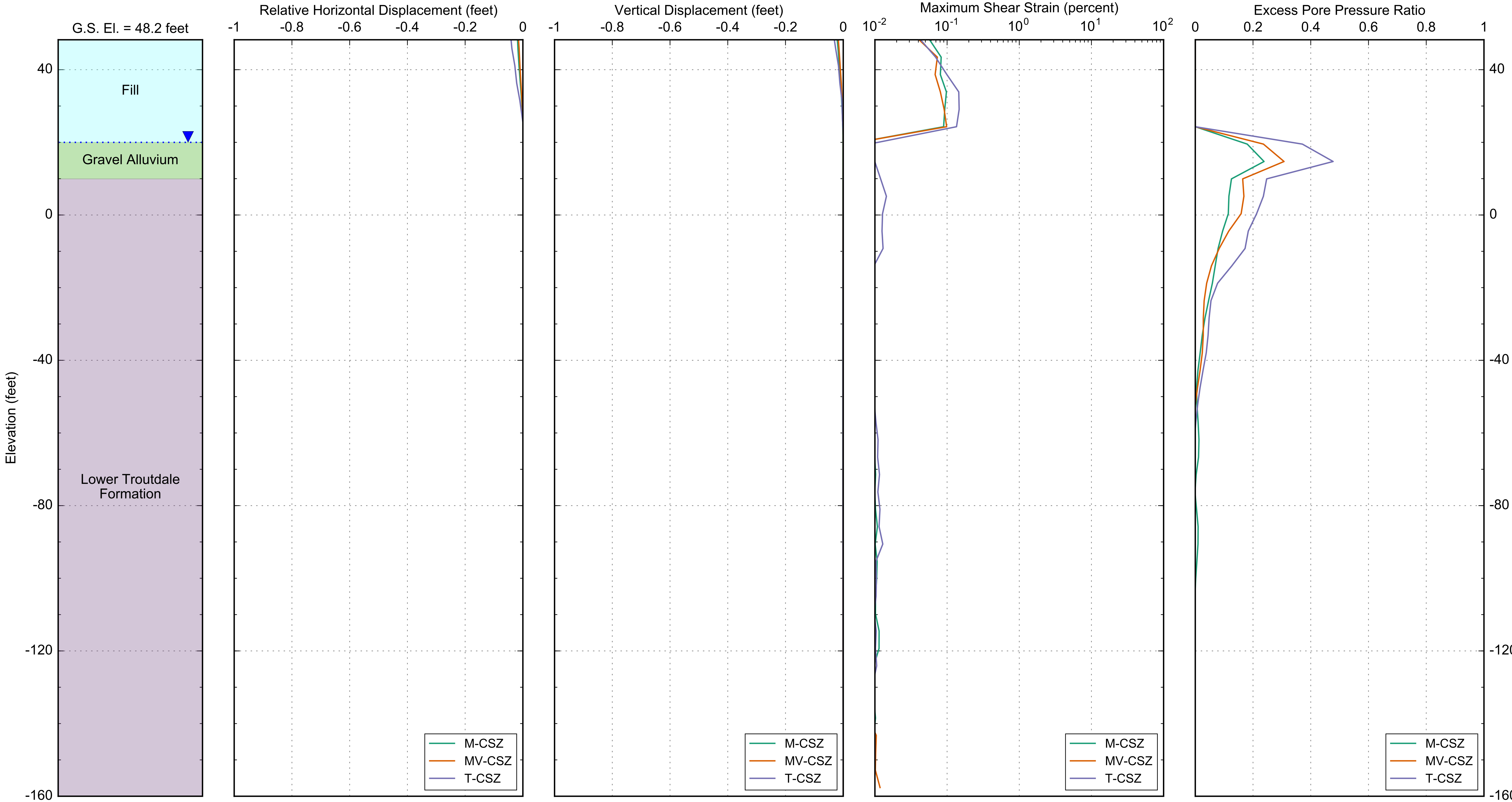
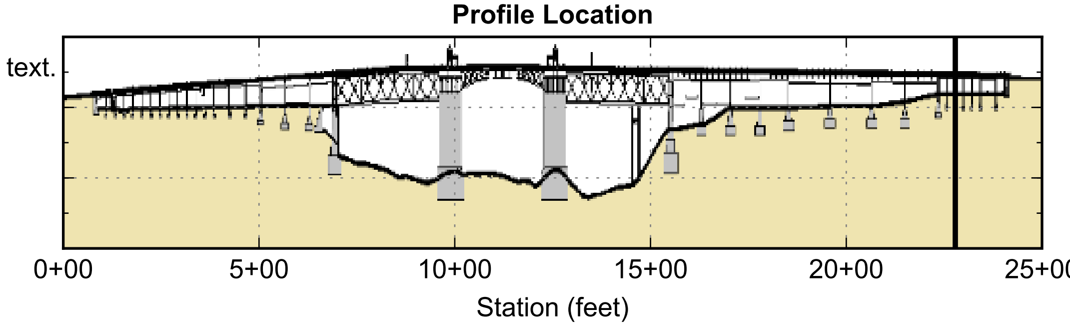
FIG. E-76



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 30

July 2017 24-1-04065-005

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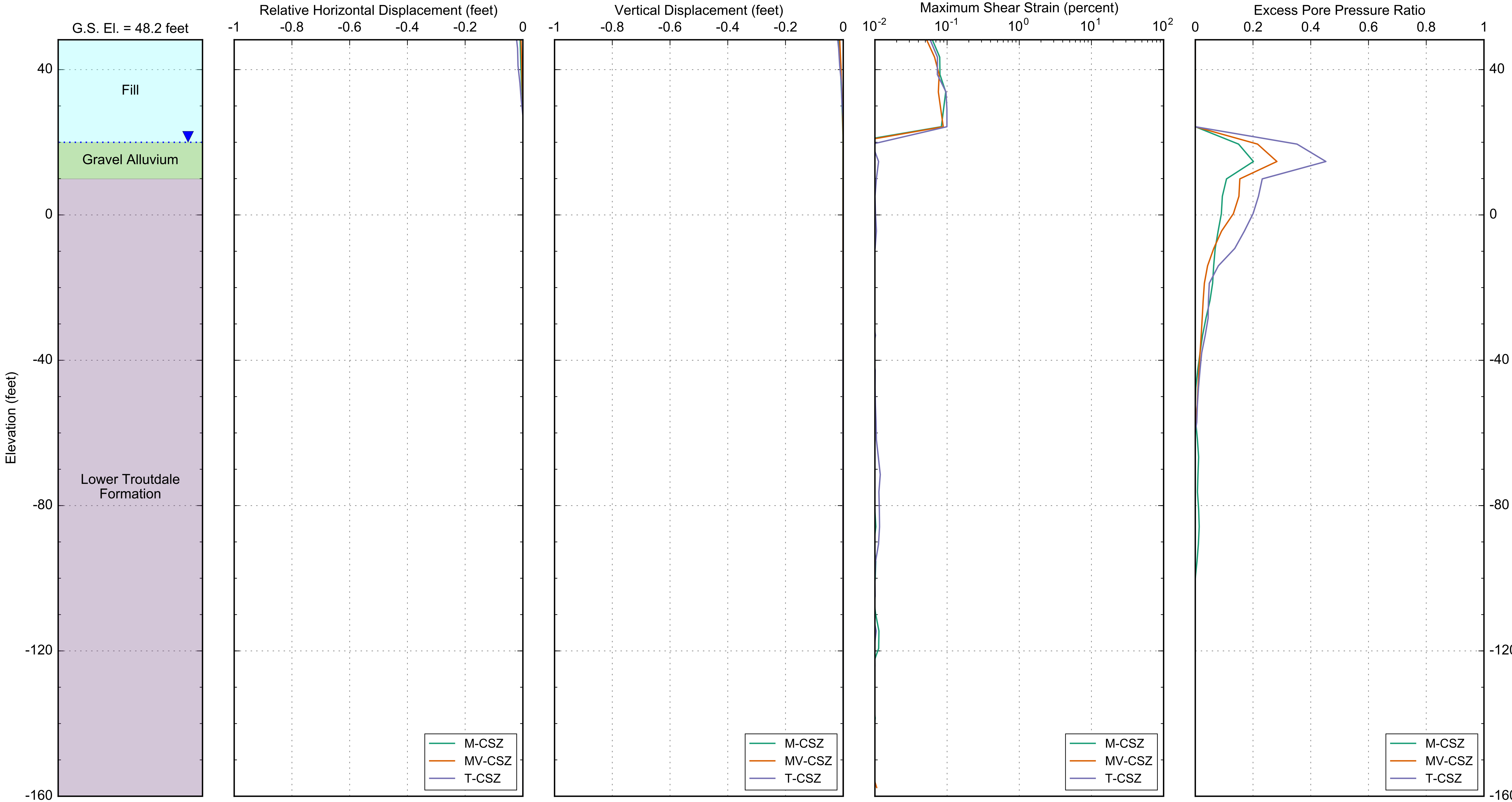
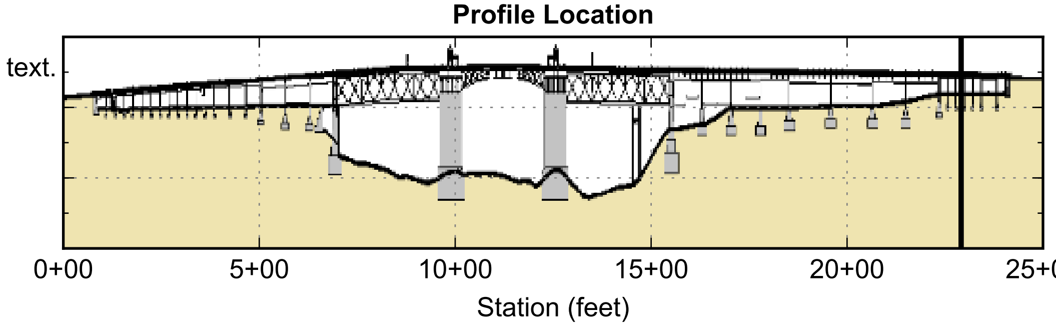
FIG. E-77



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES CASCADIA SUBDUCTION ZONE BENT 31

July 2017 24-1-04065-005

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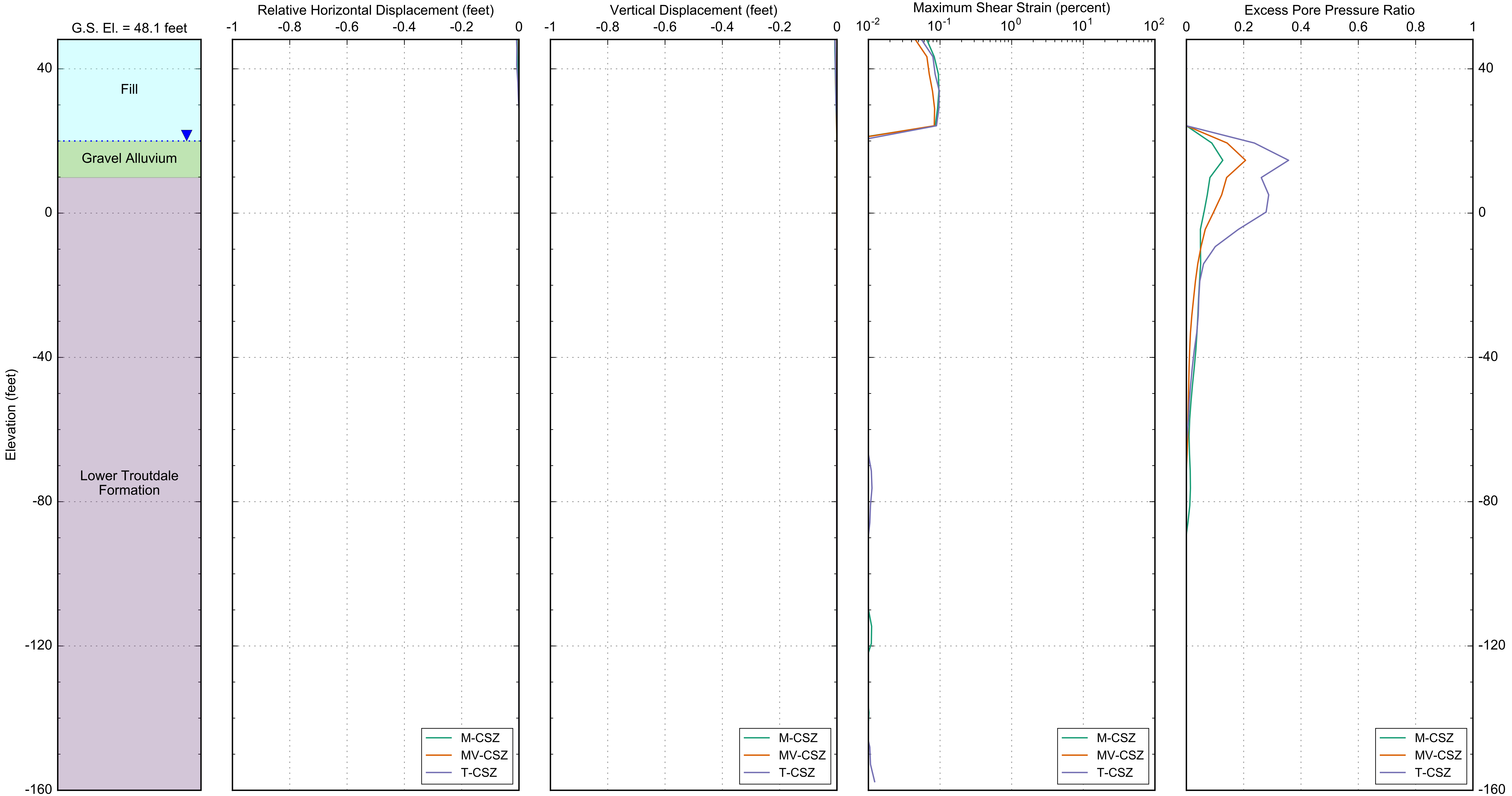
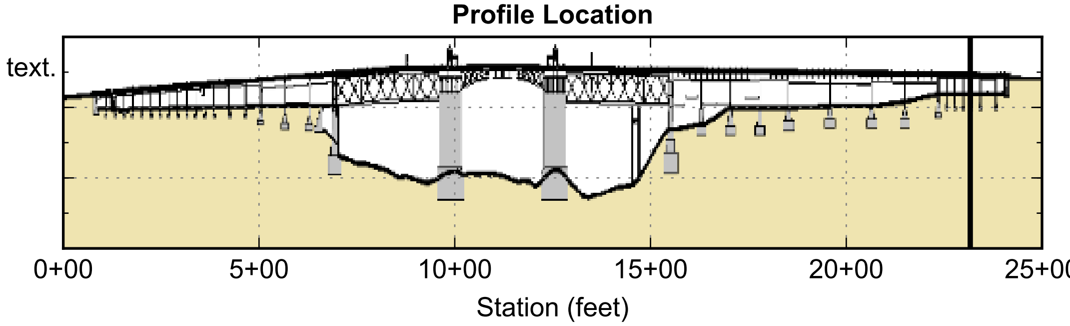
FIG. E-78



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 32

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FIG. E-79

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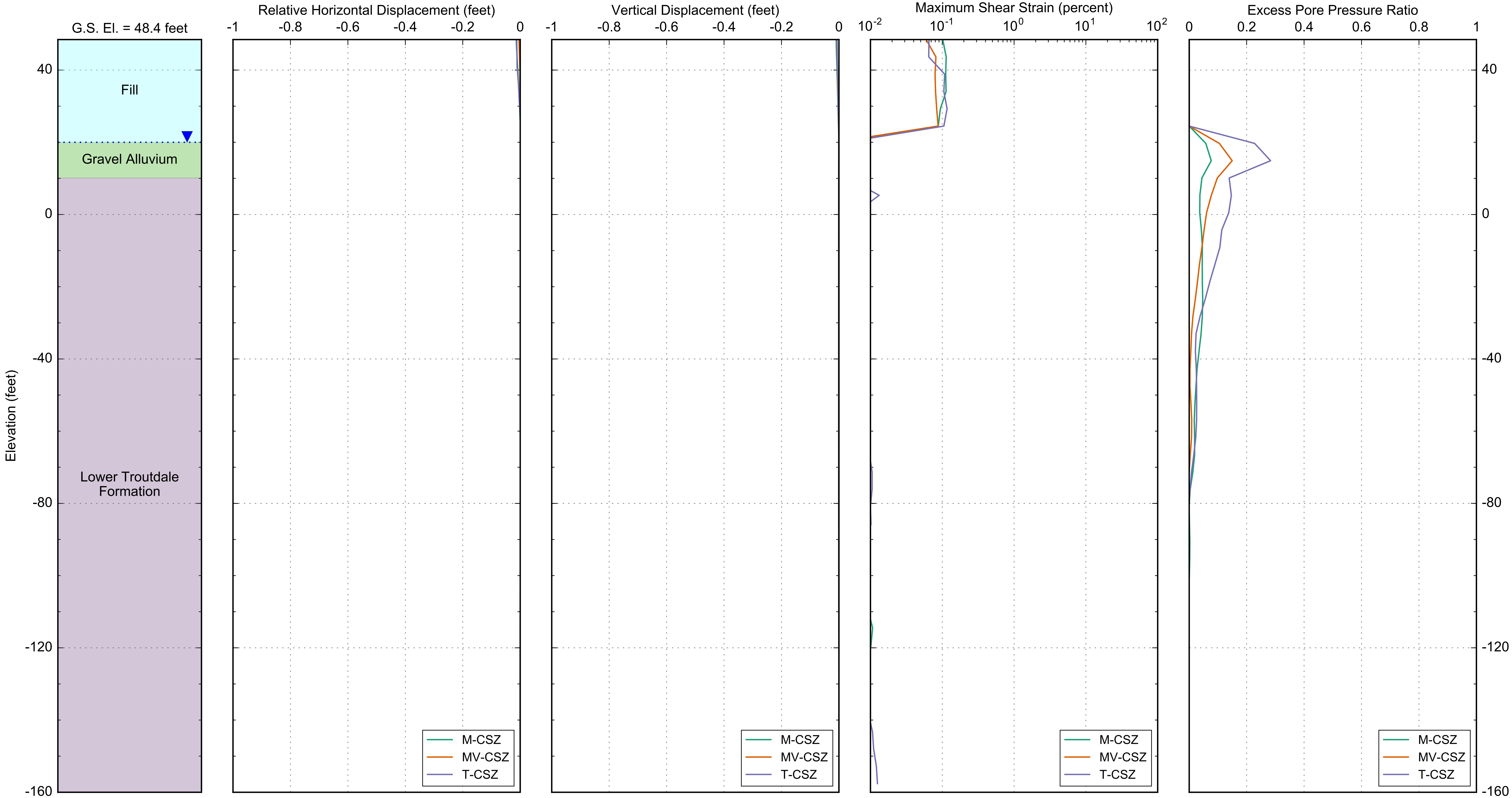
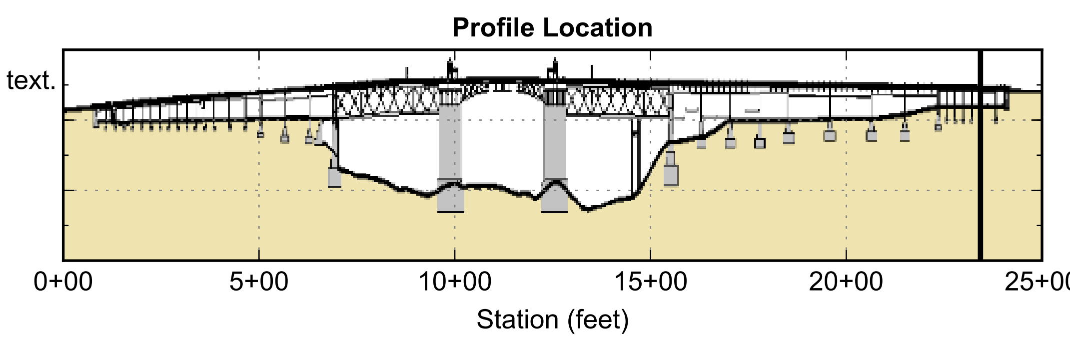


Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 33

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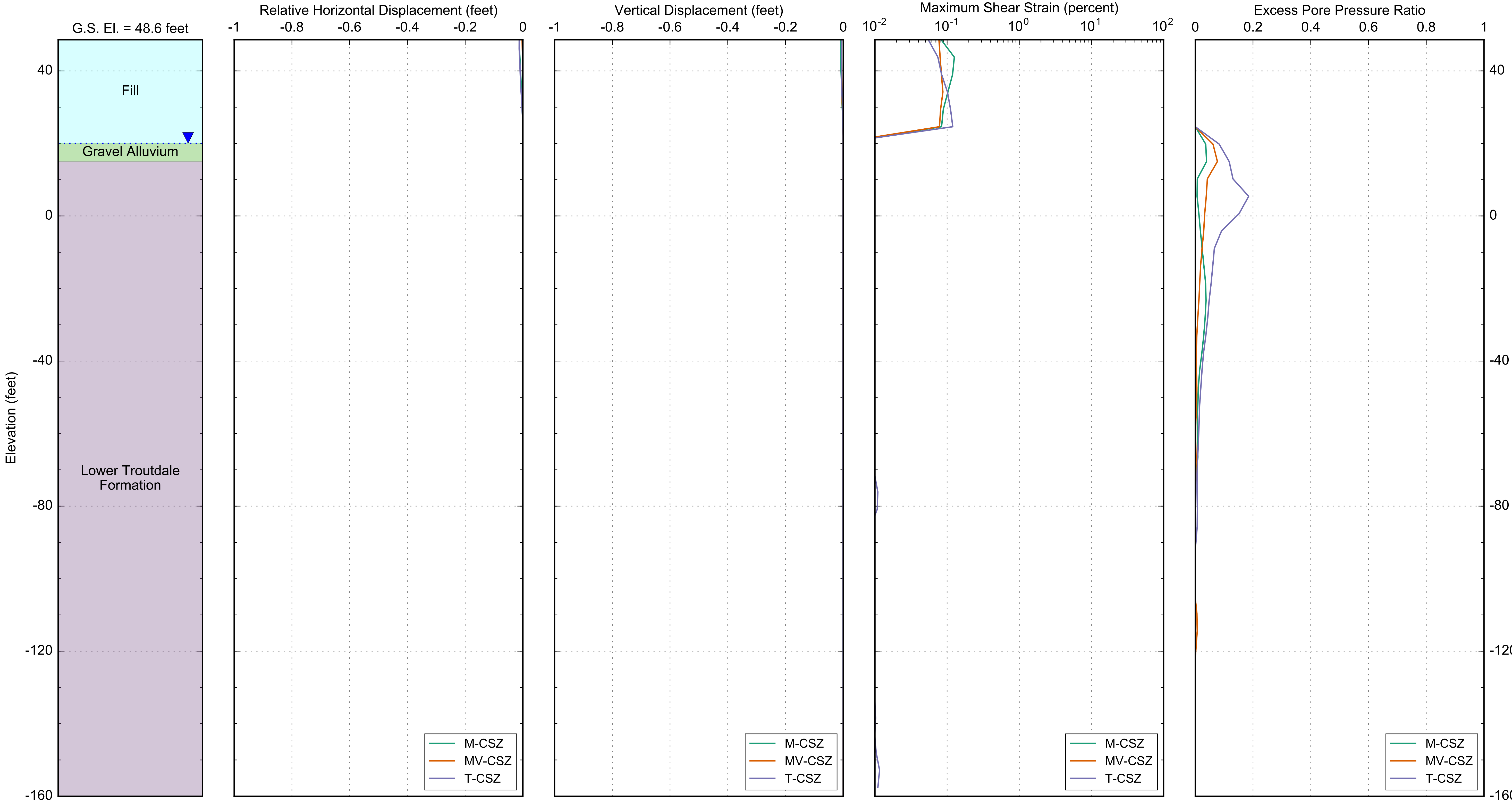
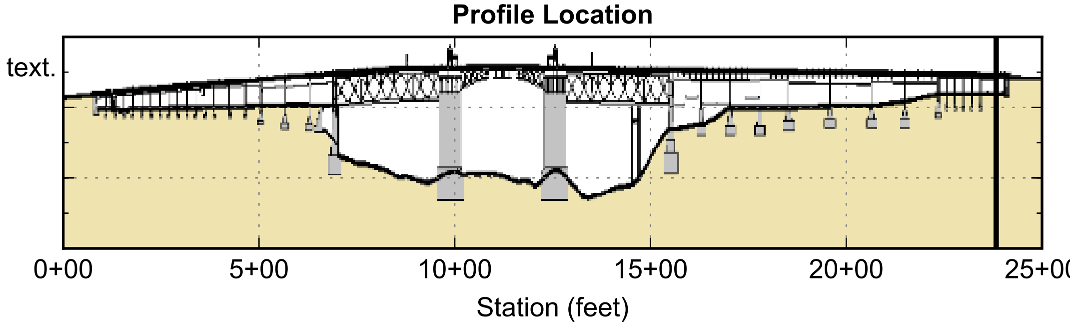
FIG. E-80



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot

Profile Location



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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 34

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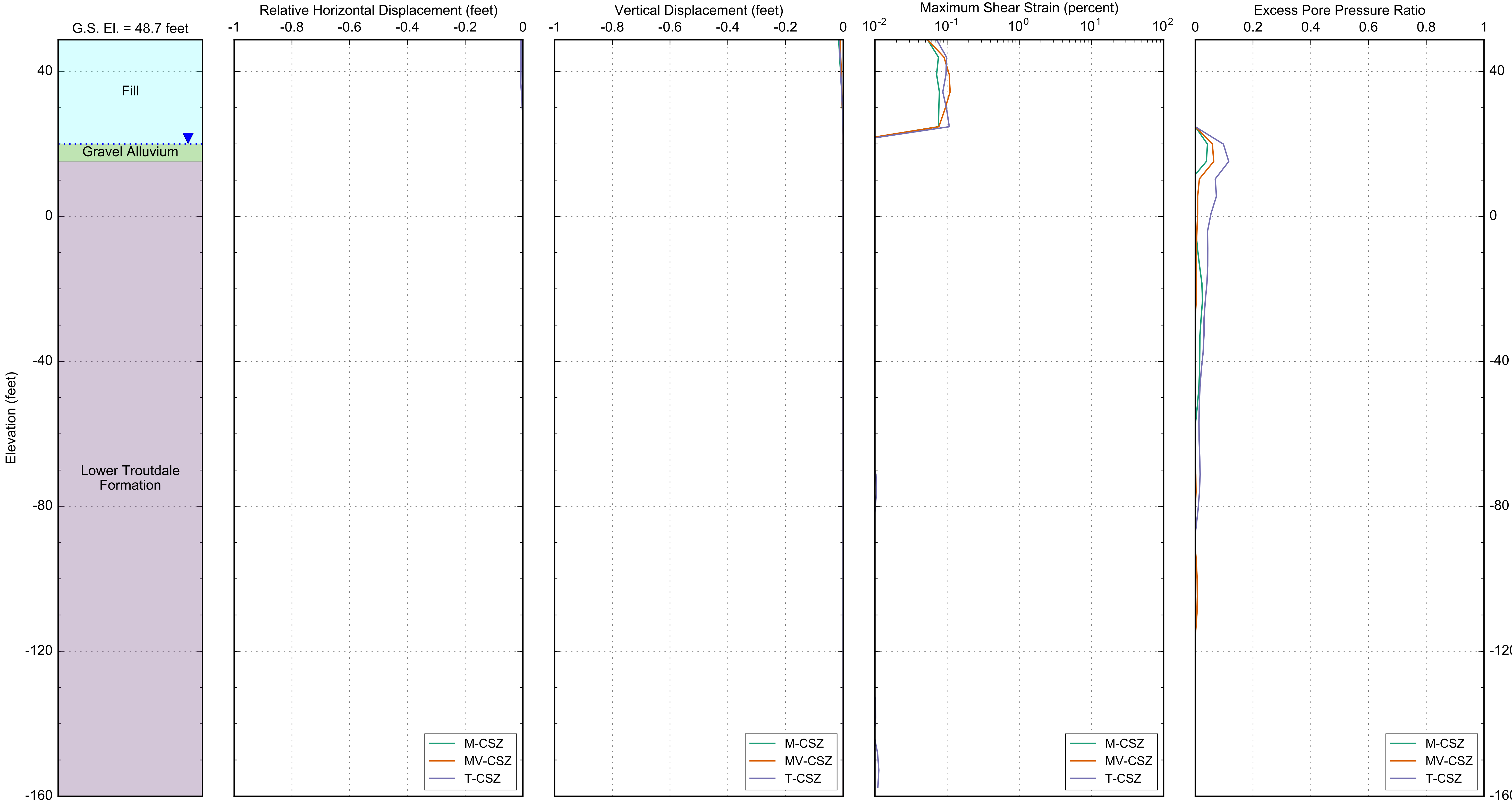
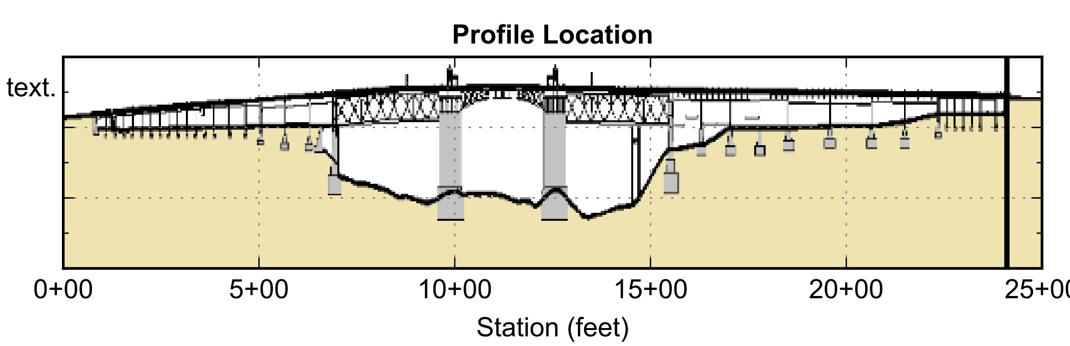
FIG. E-81



Notes:
1. See the main text for a description of the ground motion time histories.
2. The profiles correspond to the end of the ground motion time histories, except as noted in the main text.
3. G.S. El. = Ground Surface Elevation
psf = pounds per square foot



Burnside Bridge
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SUBSURFACE PROFILES
CASCADIA SUBDUCTION ZONE
BENT 35

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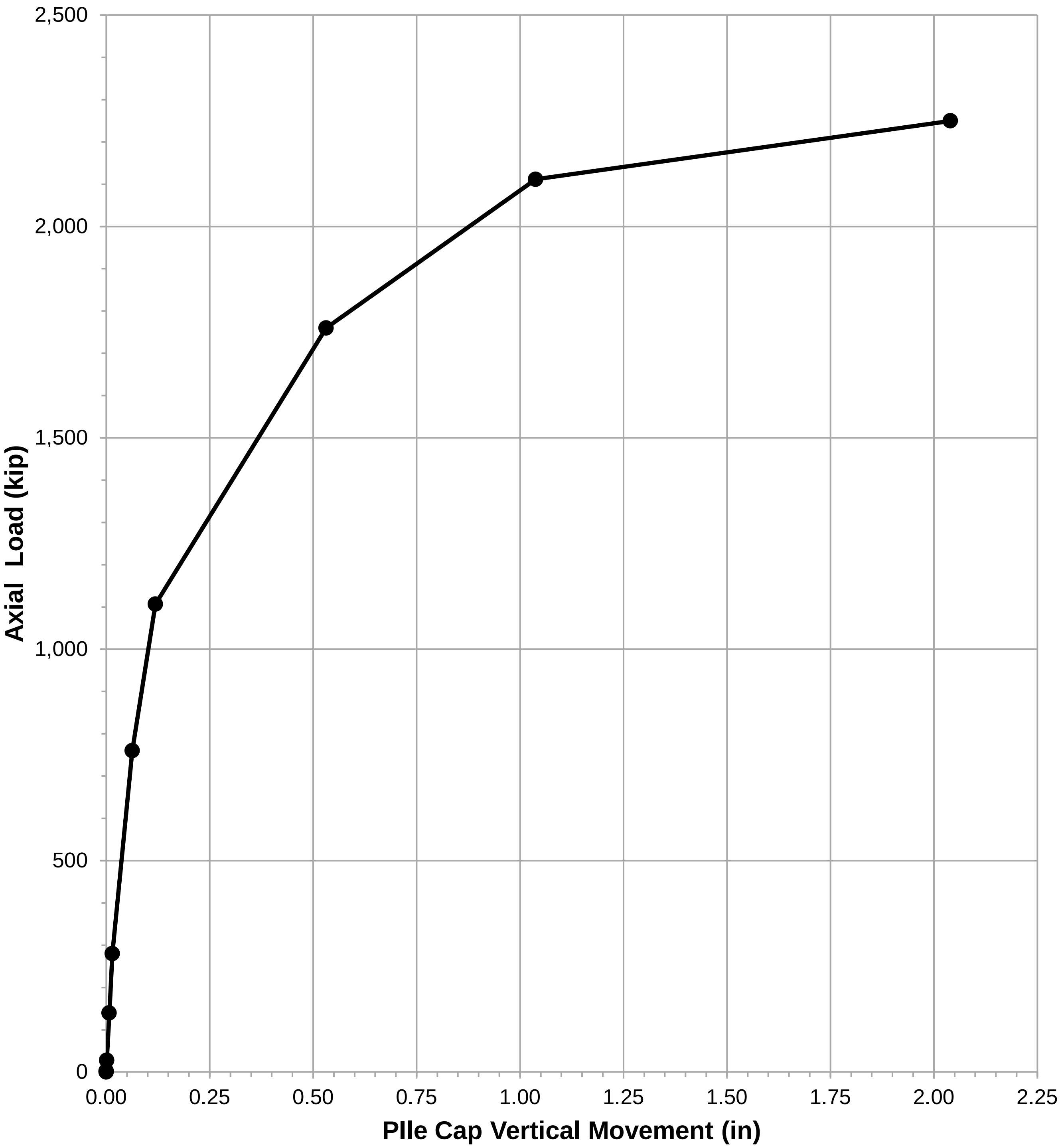
FIG. E-82

APPENDIX F

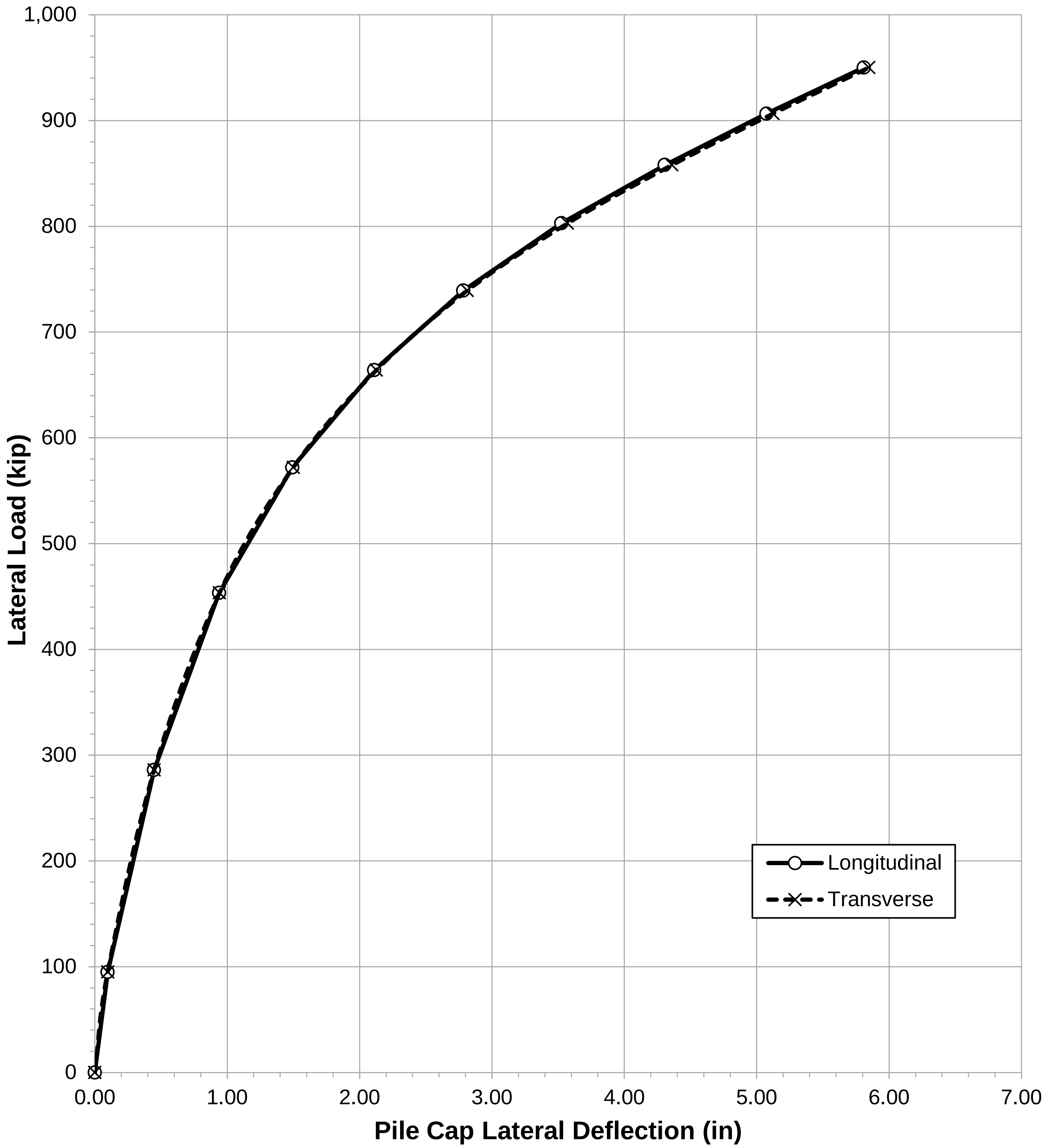
LOAD-DISPLACEMENT CURVES FOR EXISTING PILE AND PROPOSED DRILLED SHAFT GROUPS

FIGURES



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 18 May 2017 24-1-04065-005
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **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.

Burnside Bridge Seismic Feasibility Study
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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 18

May 2017 24-1-04065-005

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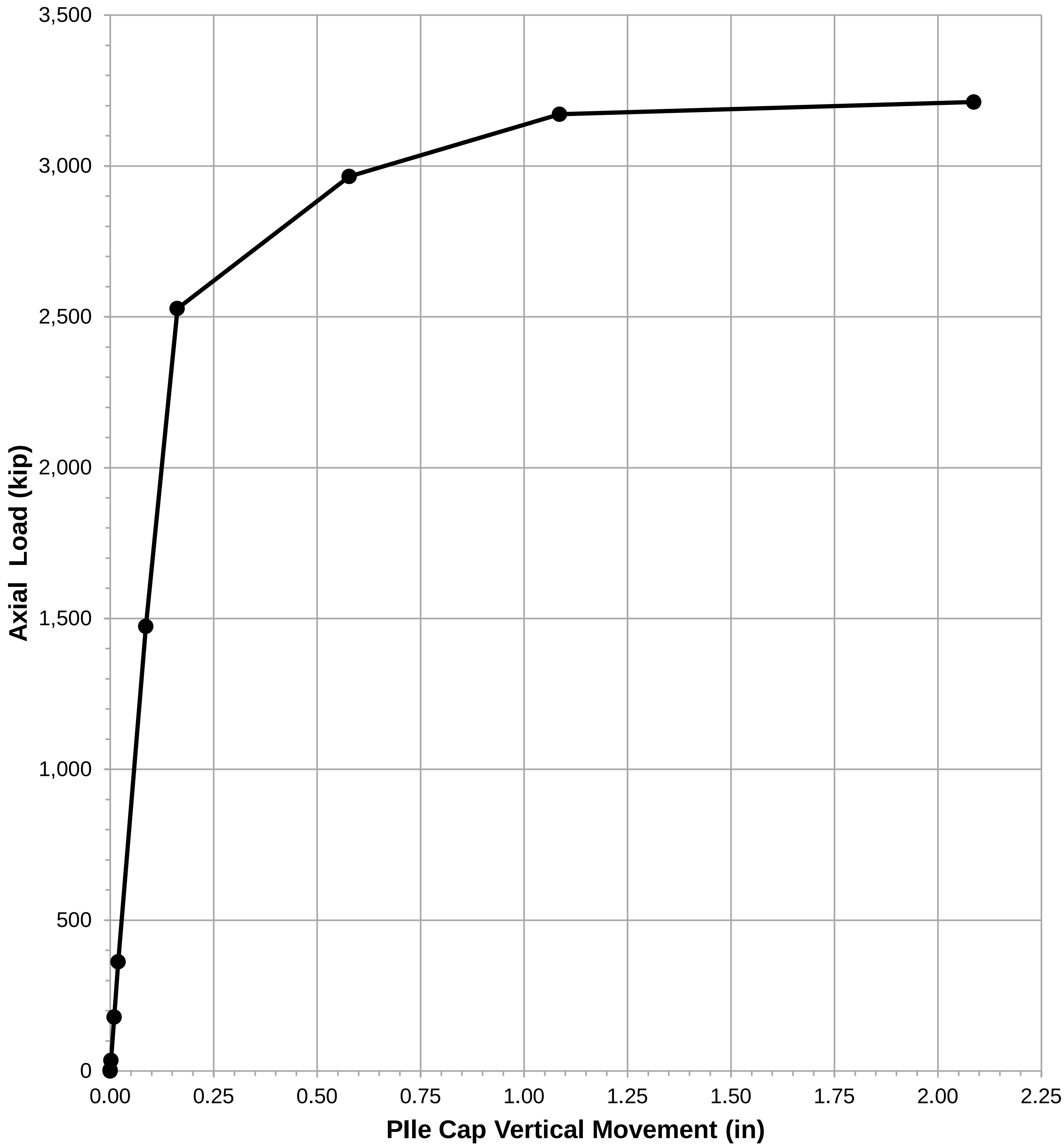
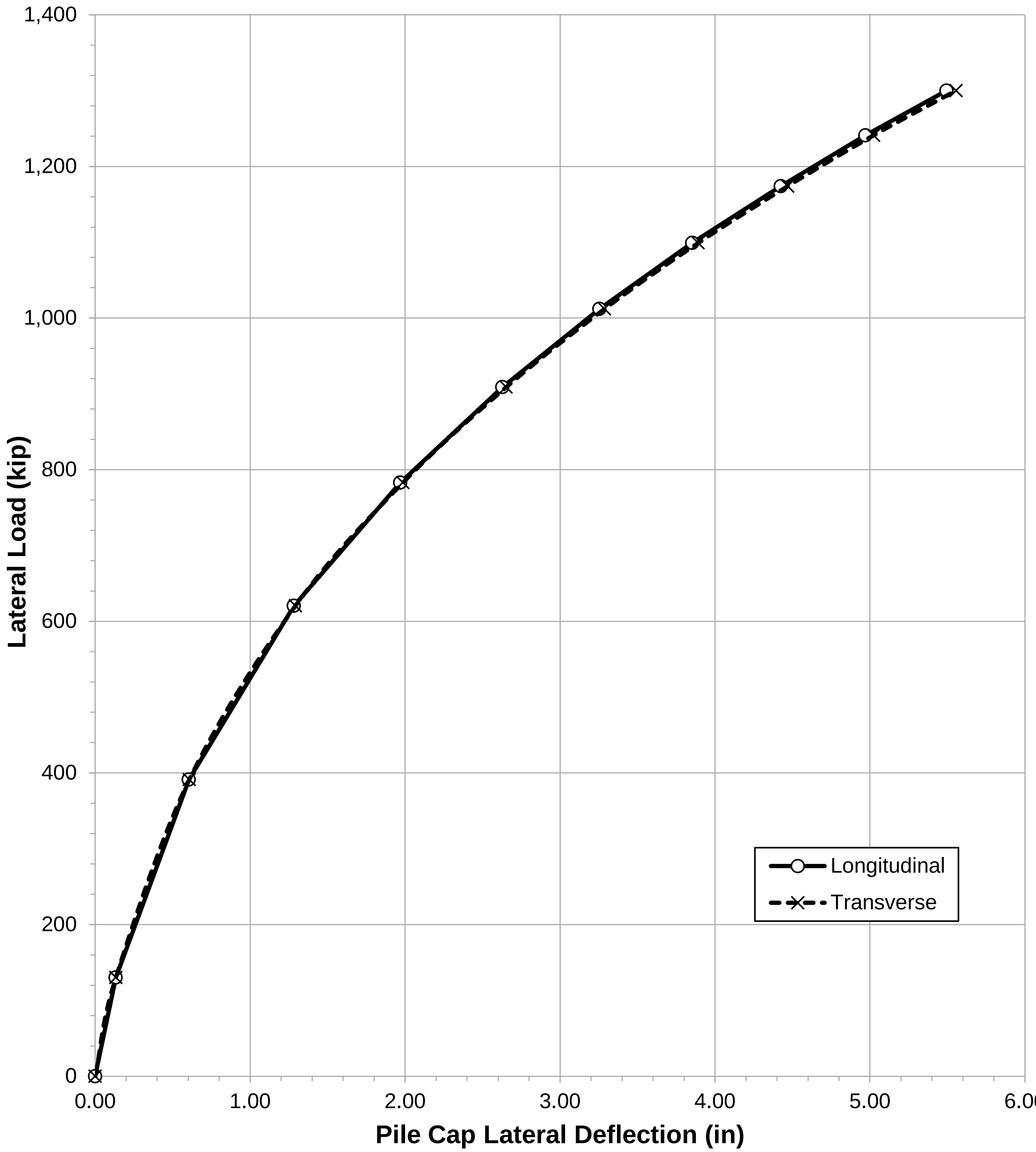
FIG. F2



NOTES

1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study Portland, Oregon	
PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 19	
May 2017	24-1-04065-005
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**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
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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 19

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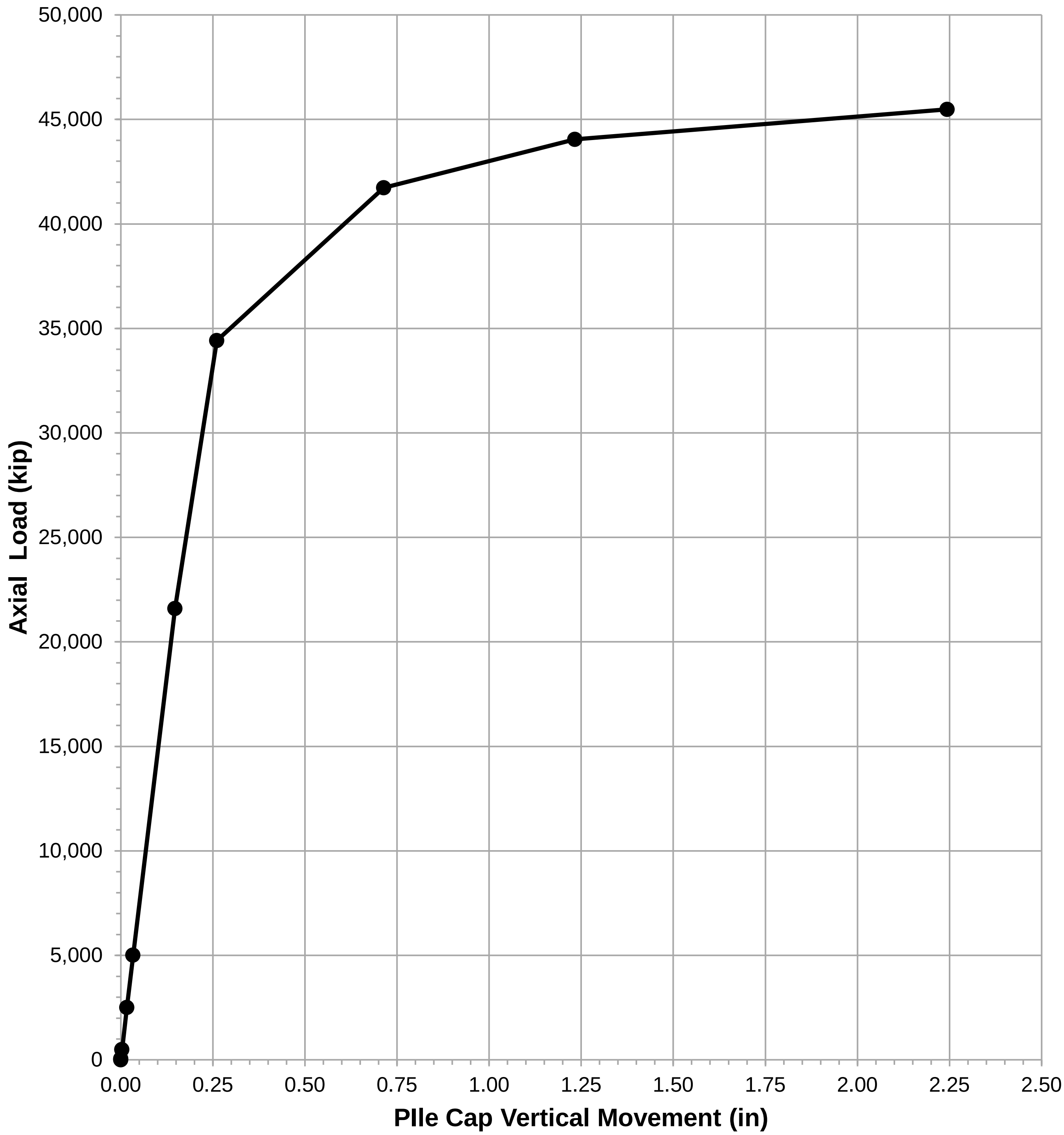
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F4



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 1

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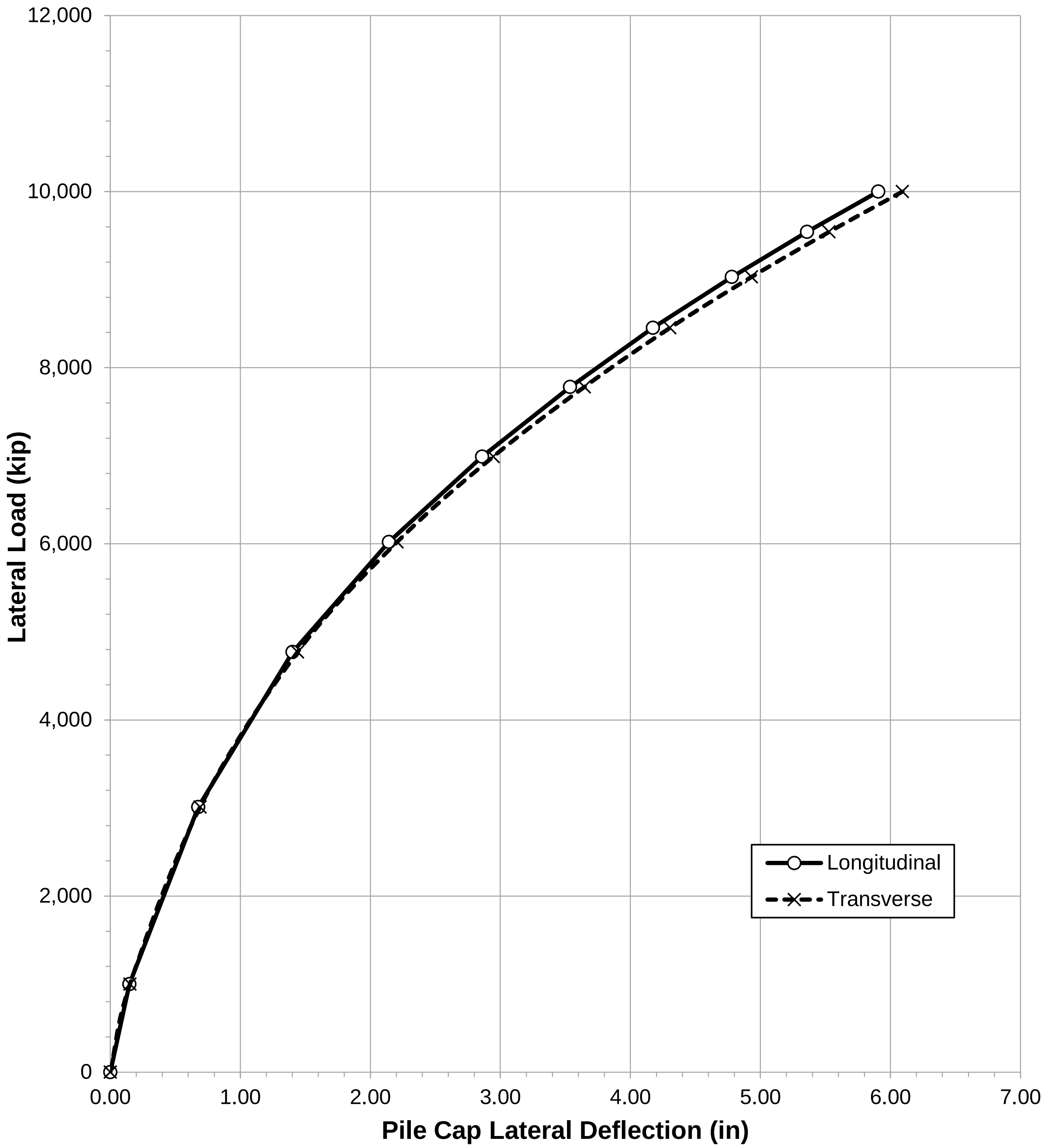
FIG. F5



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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 1

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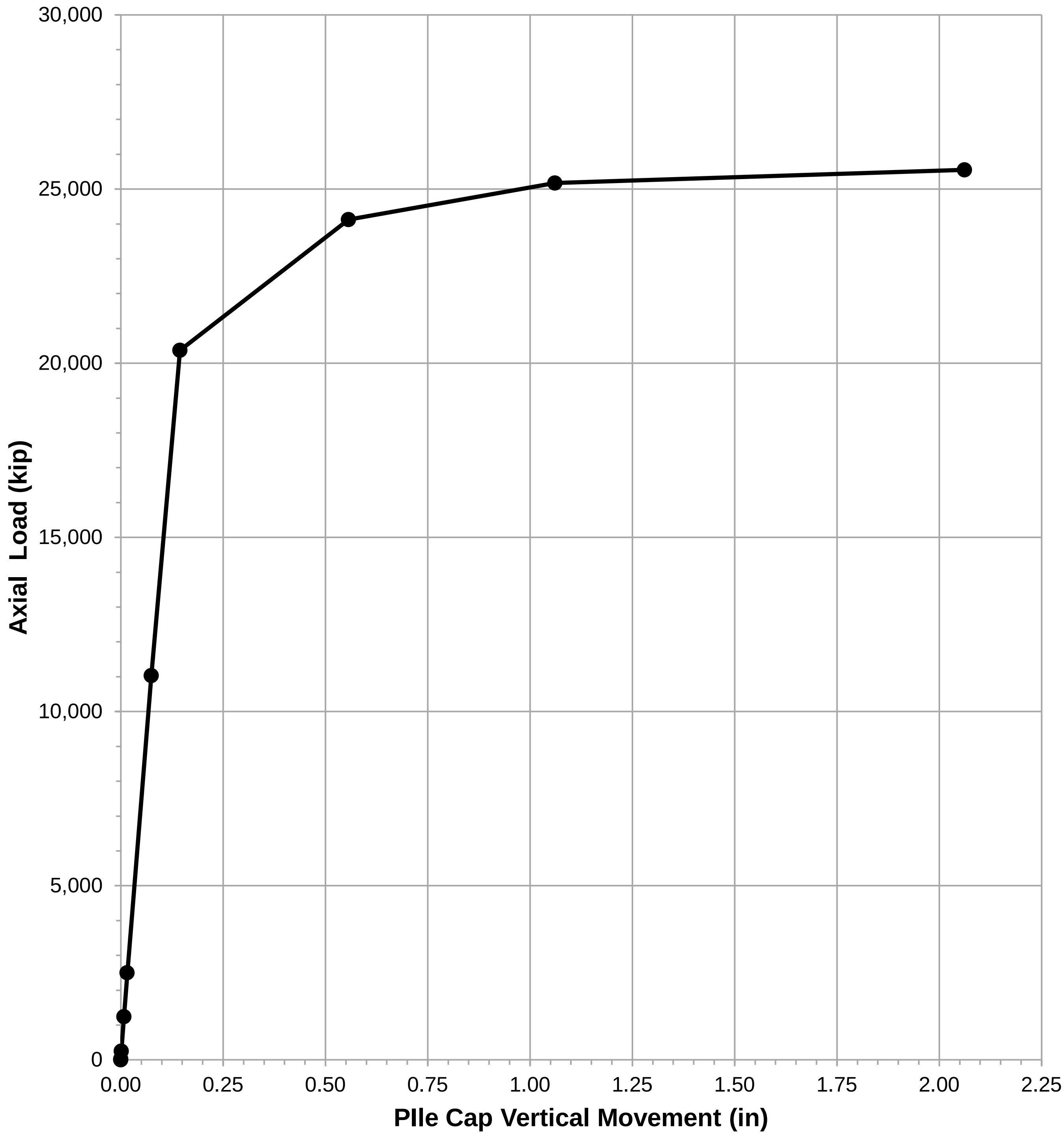
FIG. F6



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 2	
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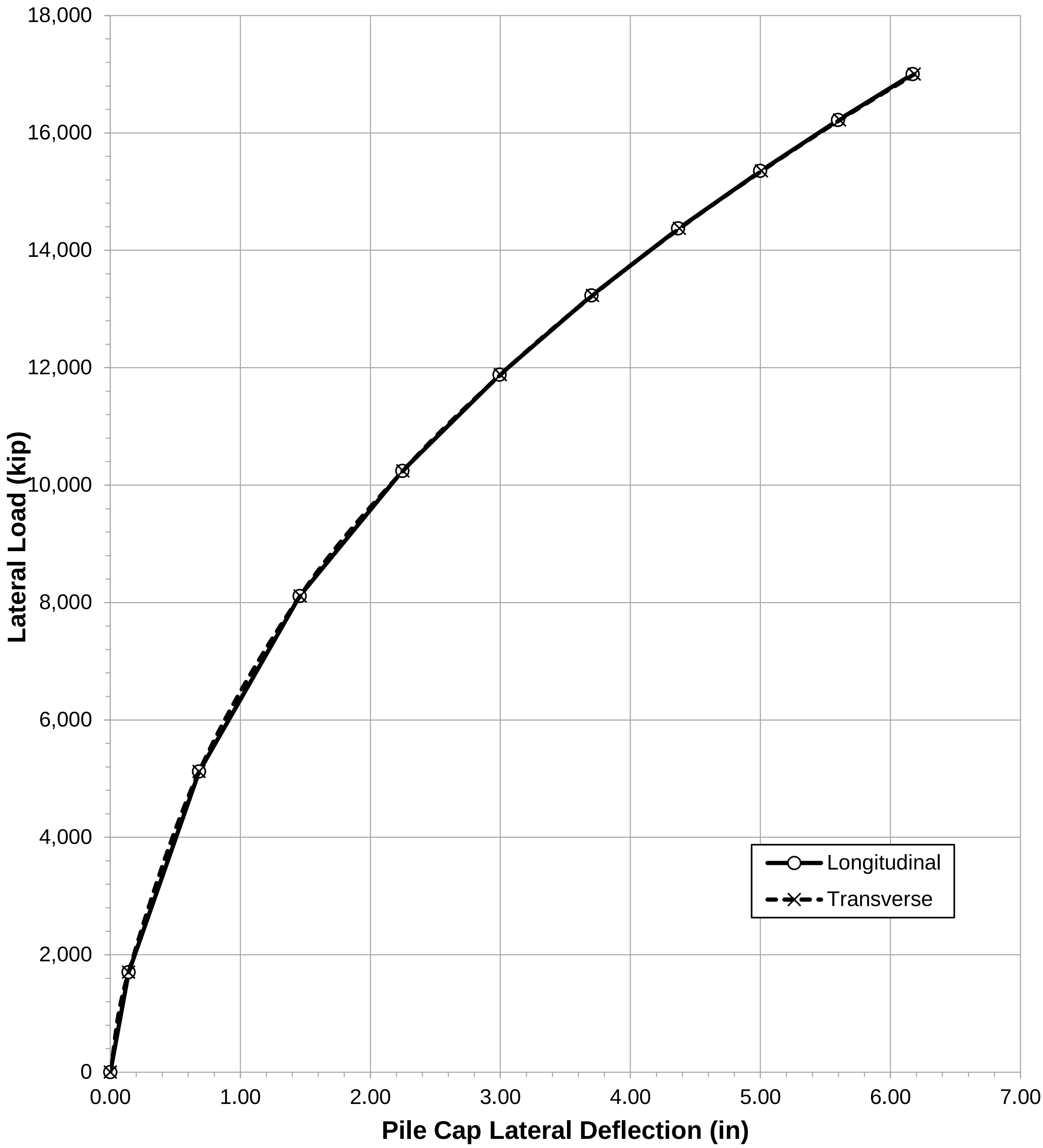
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. F7**



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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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 2

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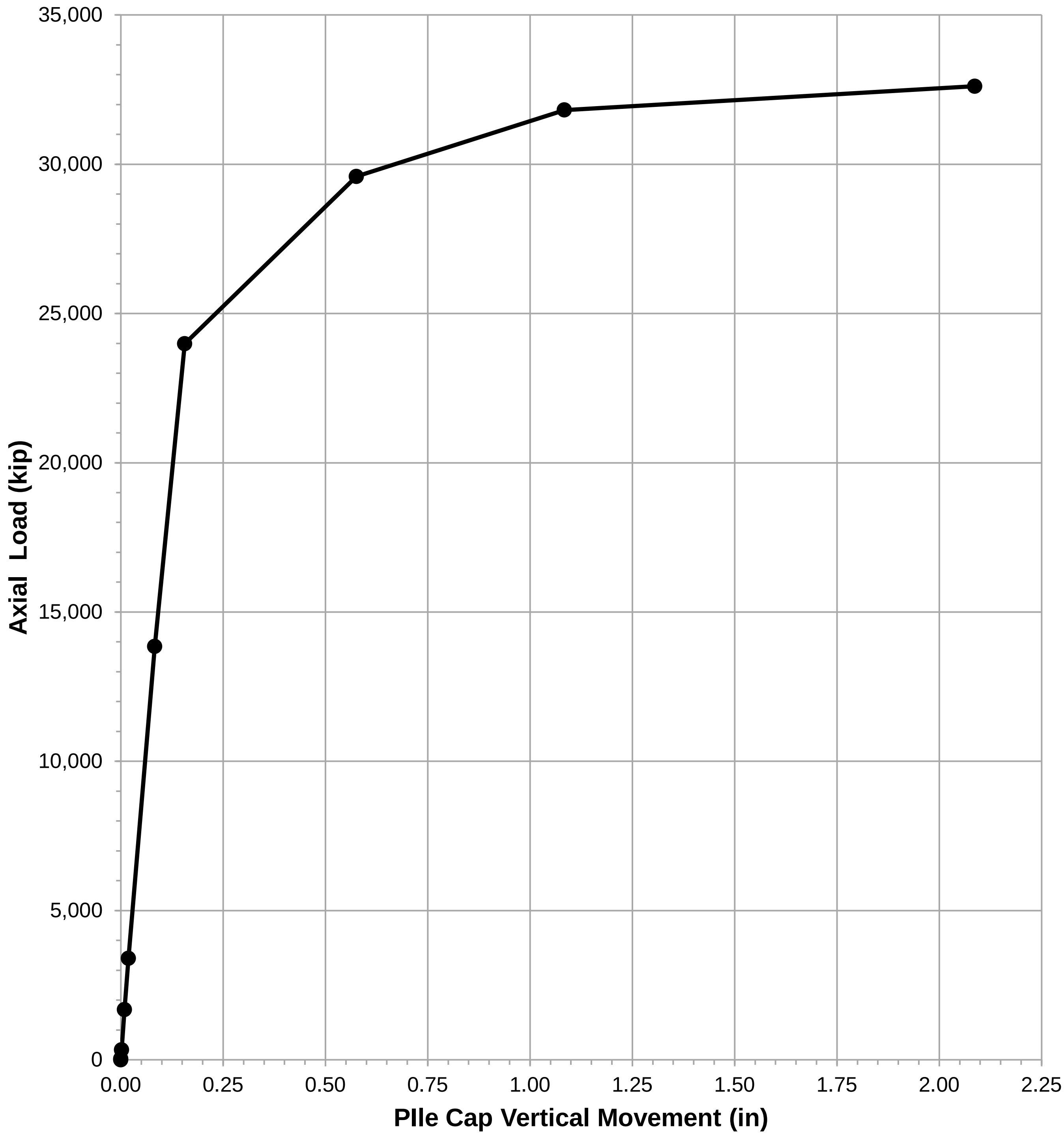
FIG. F8



NOTES
1. See Table 2 for details of each pile group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 3

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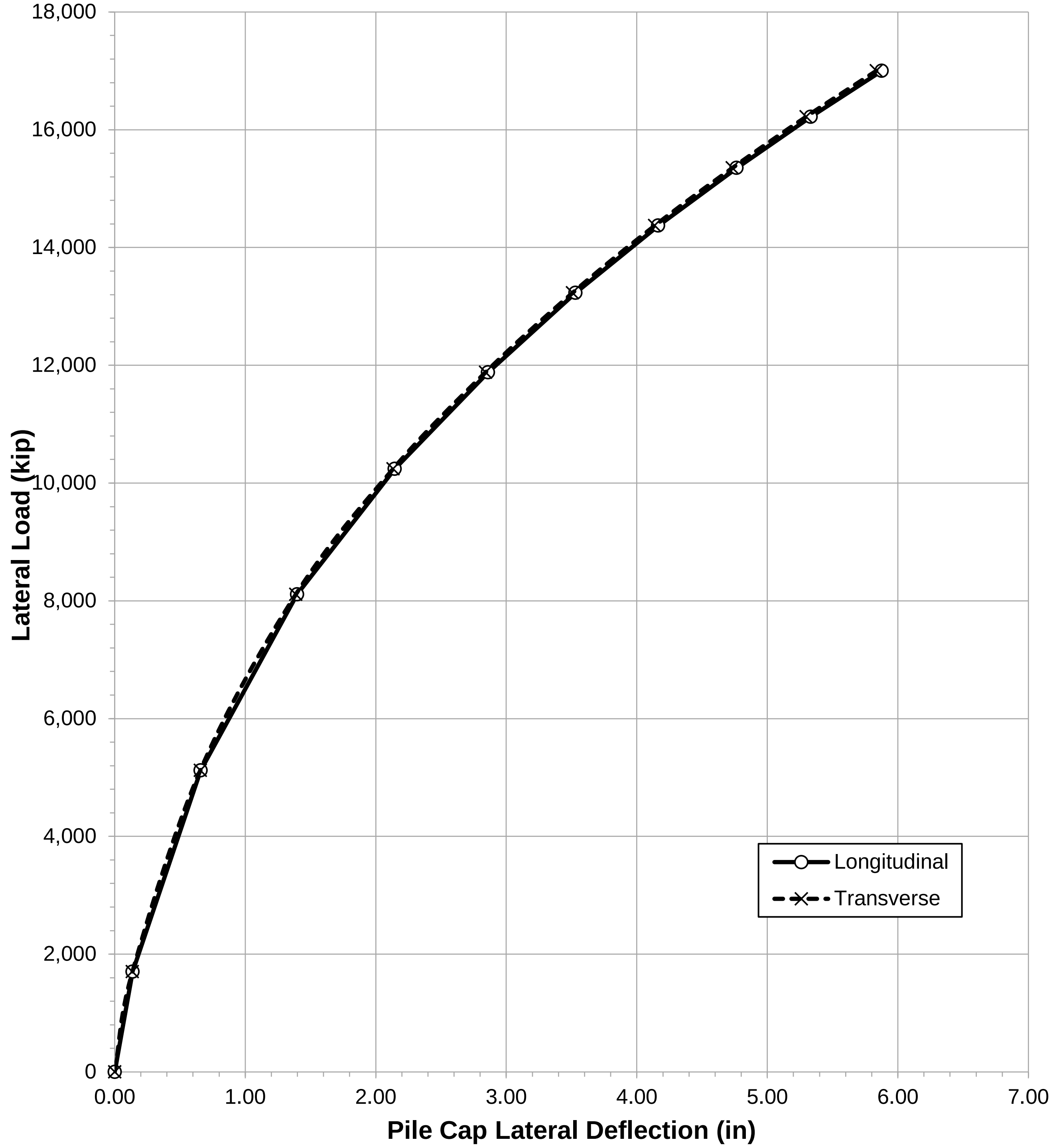
FIG. F9



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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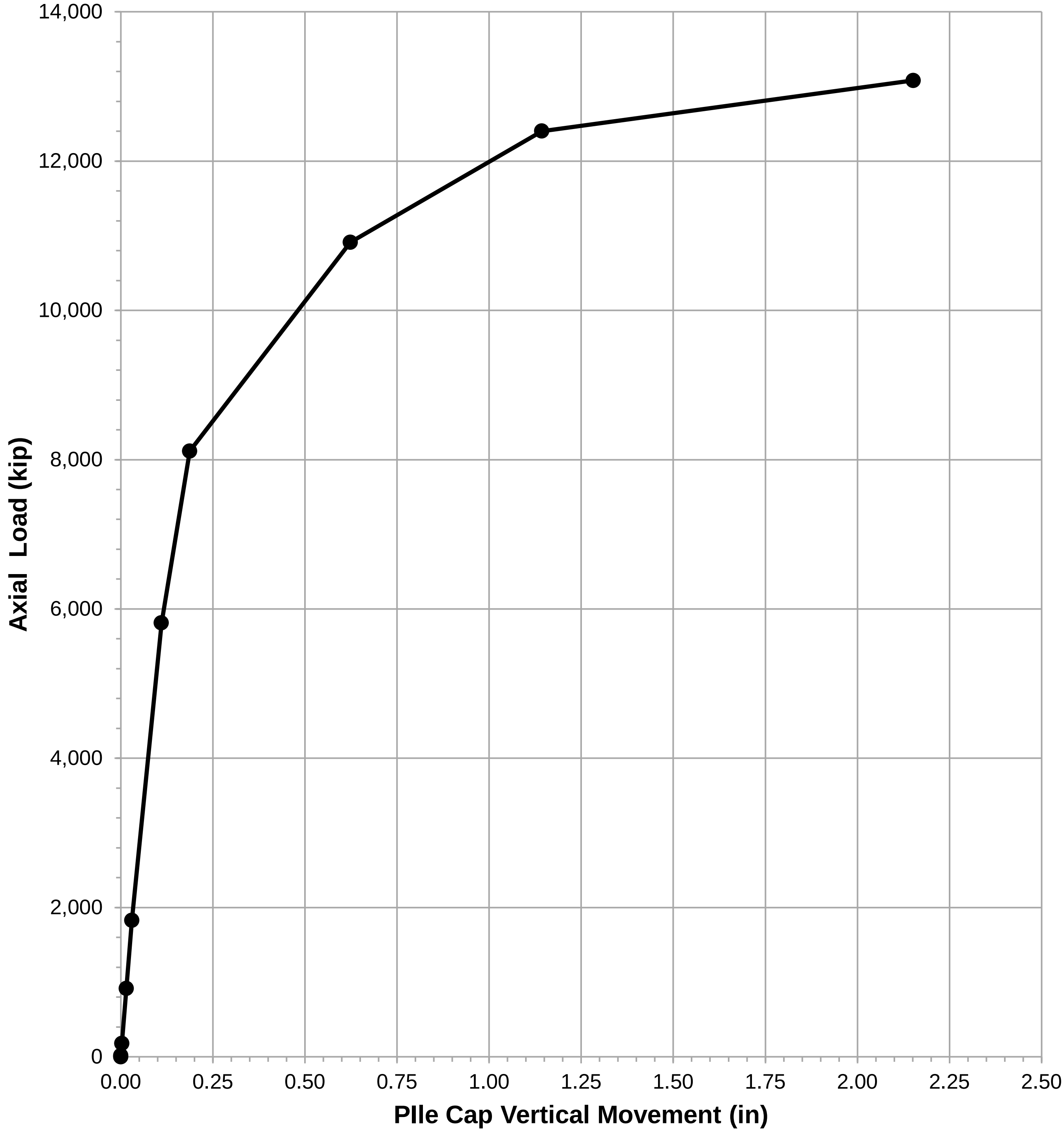
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. F10**



NOTES

1. See Table 2 for details of each pile group evaluated.

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PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 4

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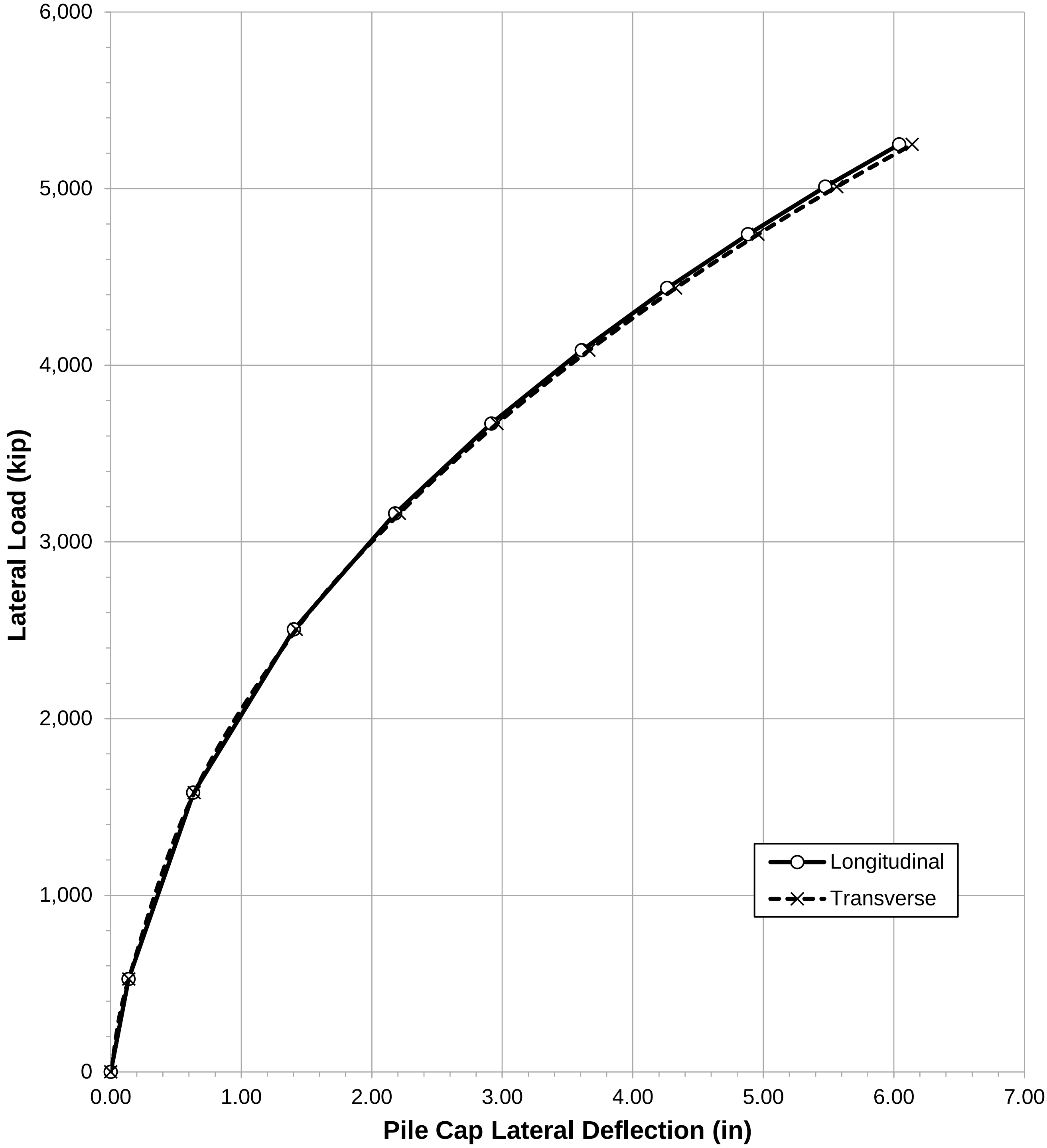
FIG. F11



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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 4

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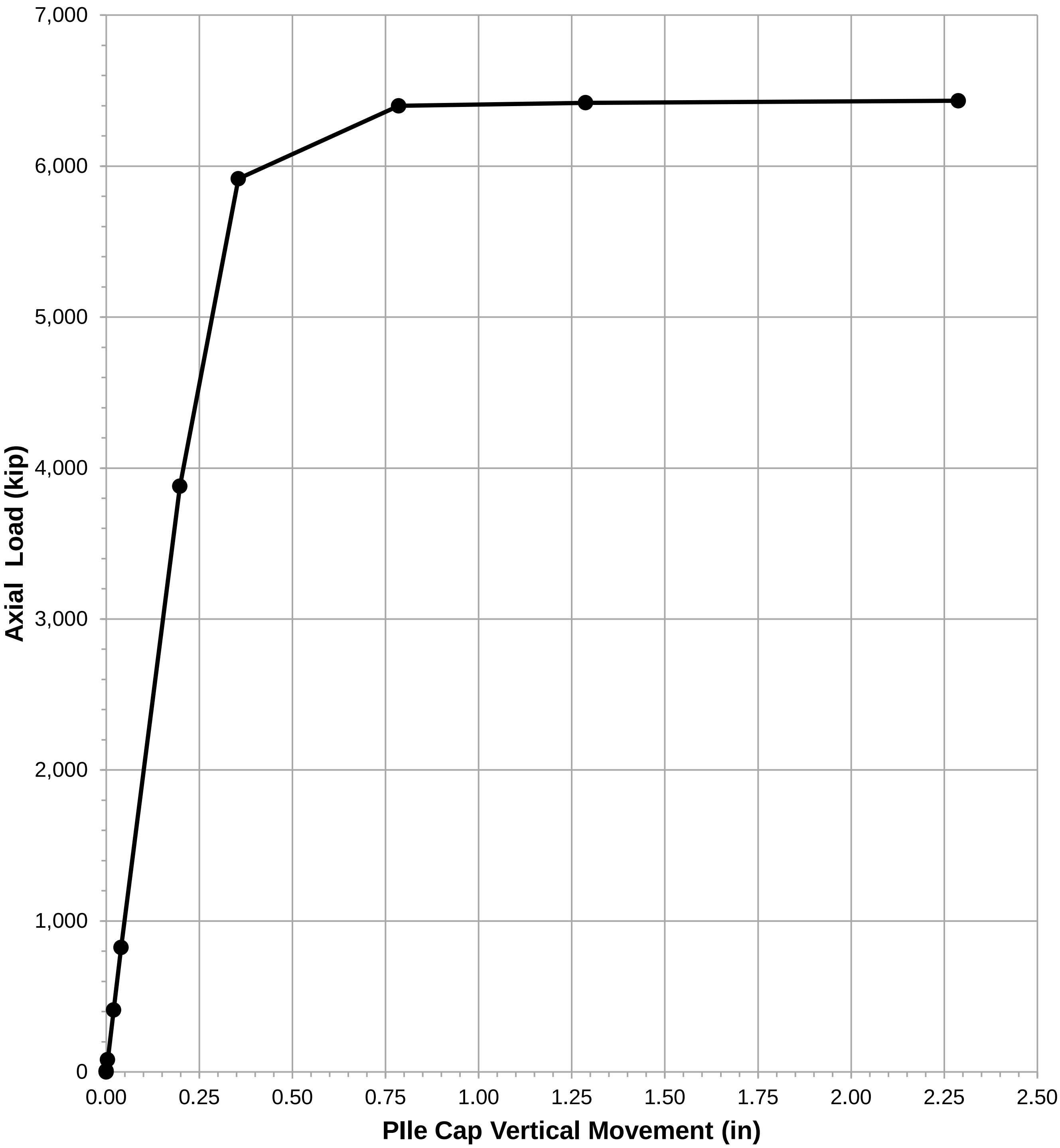
FIG. F12



NOTES

1. See Table 2 for details of each pile group evaluated.

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PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 21

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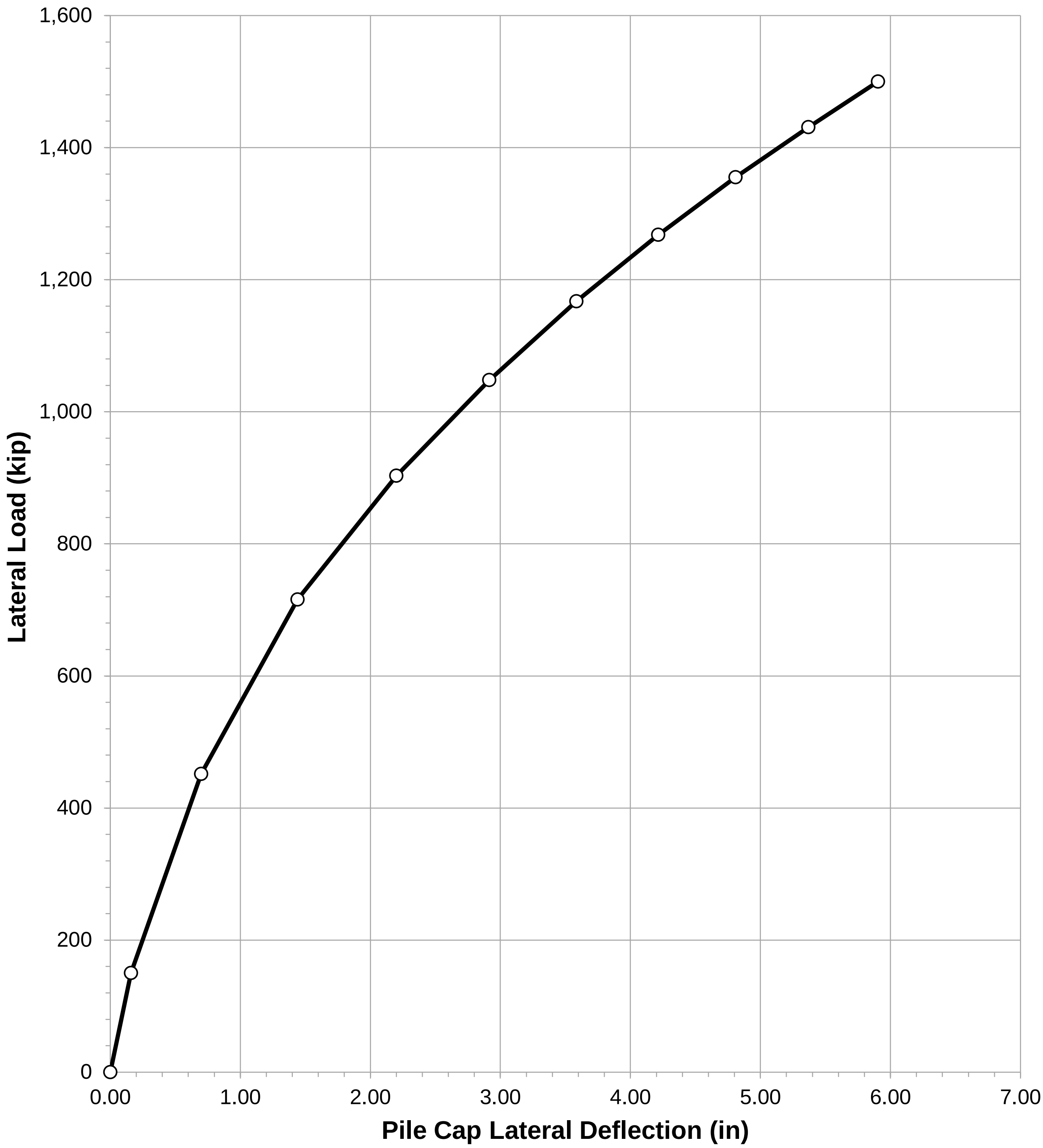
FIG. F13



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.

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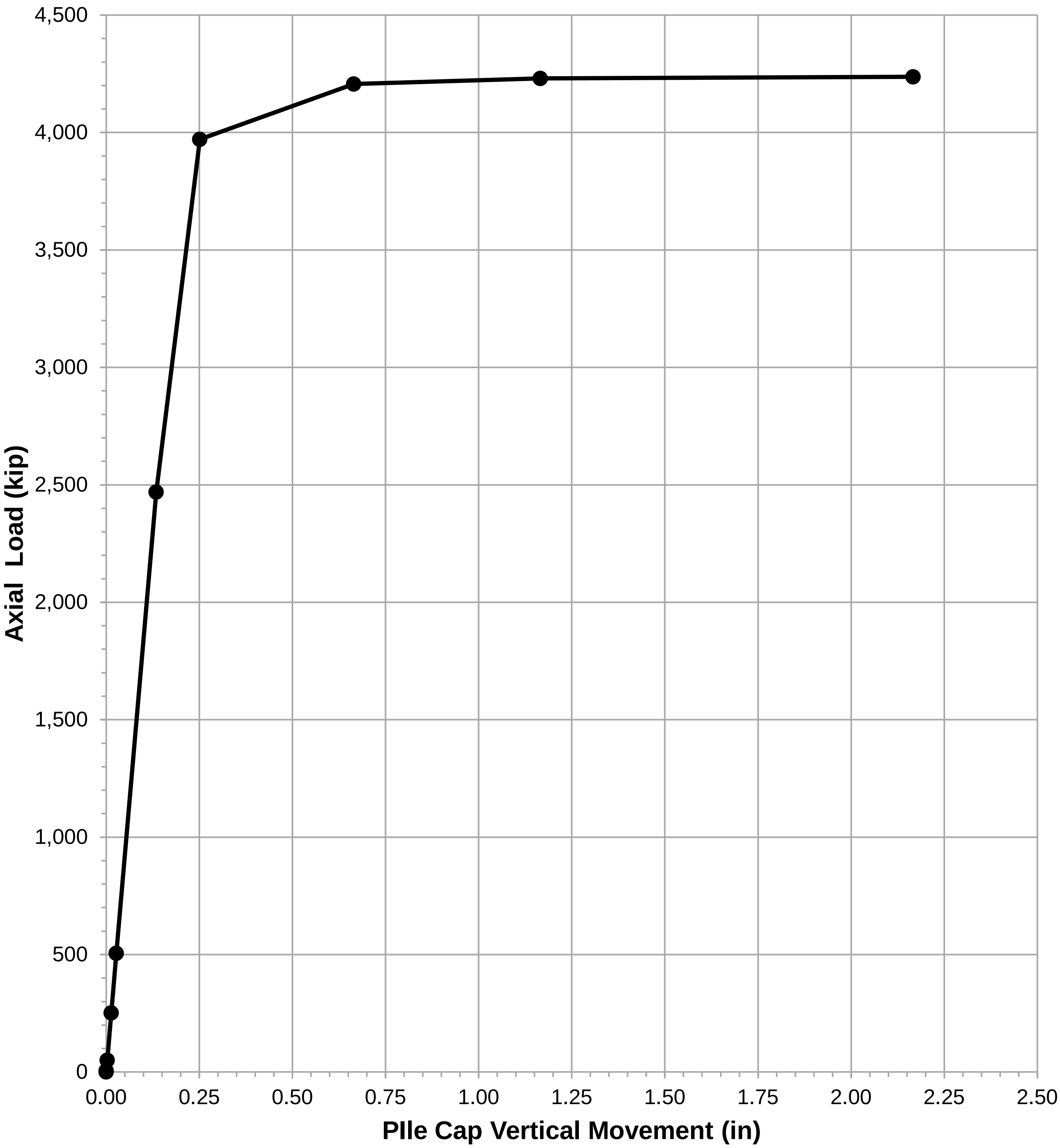
PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 21

May 2017 24-1-04065-005

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F14



NOTES
1. See Table 2 for details of each pile group evaluated.

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PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 22

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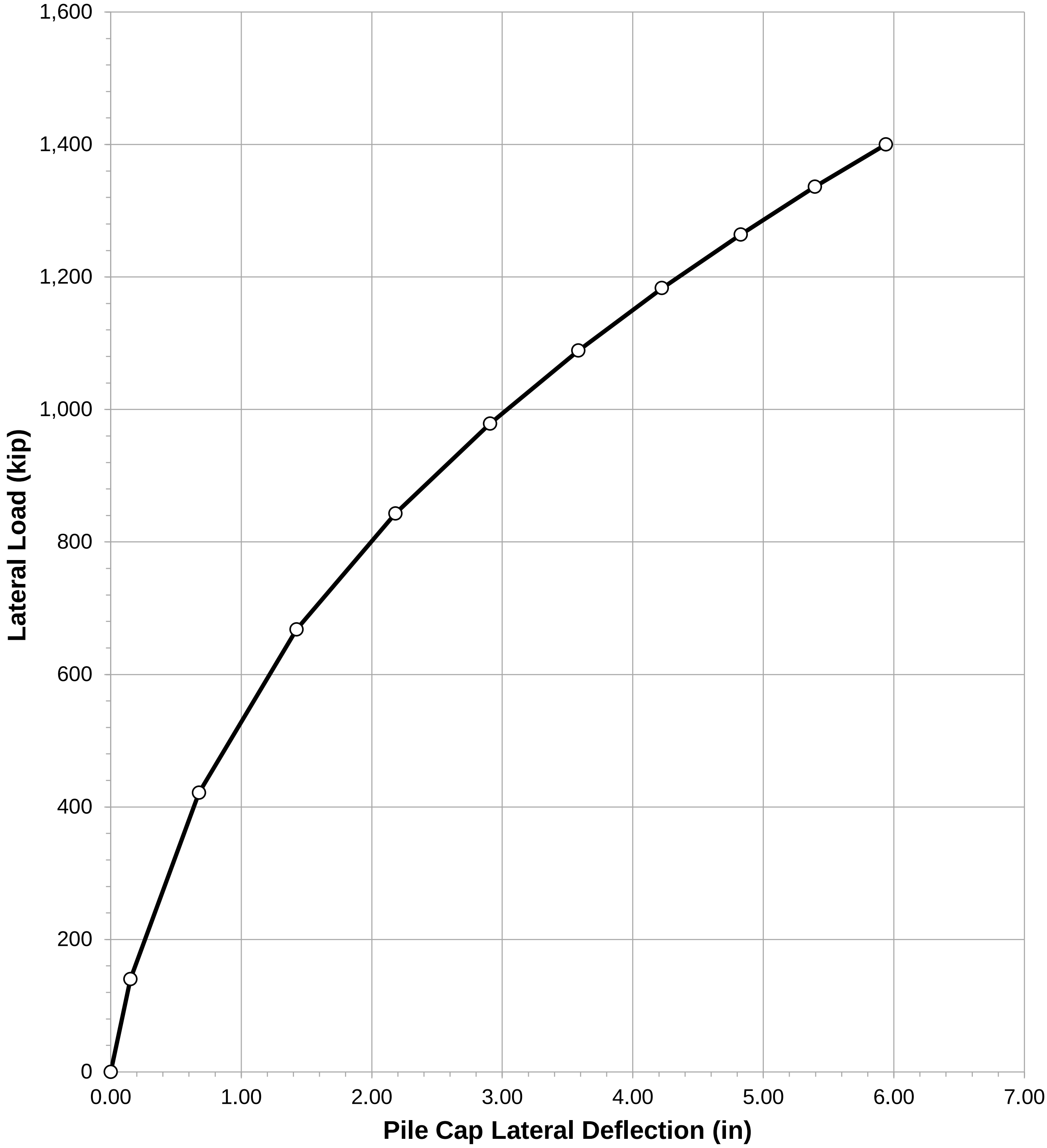
Geotechnical and Environmental Consultants

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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 22

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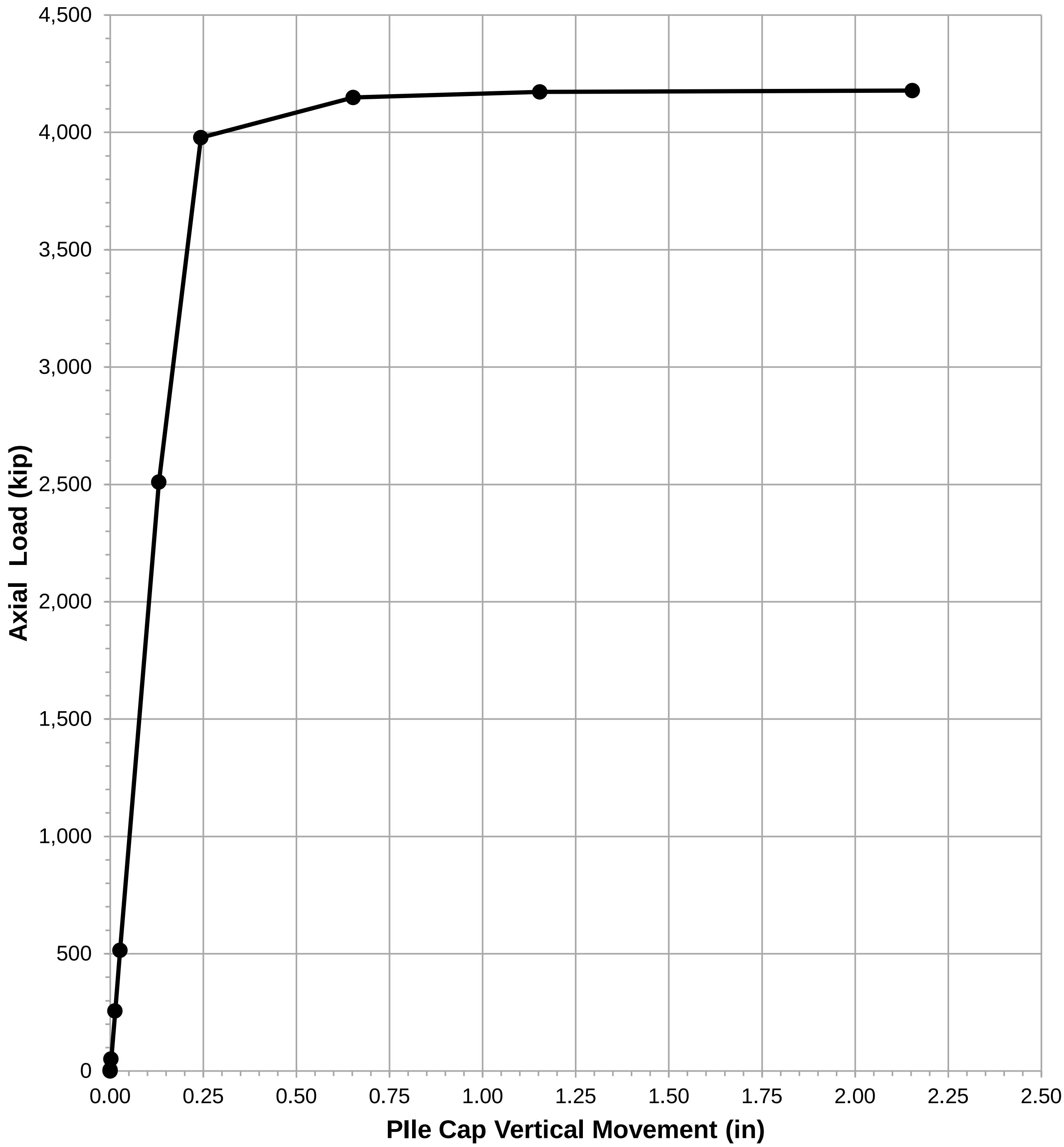
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F16



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 23

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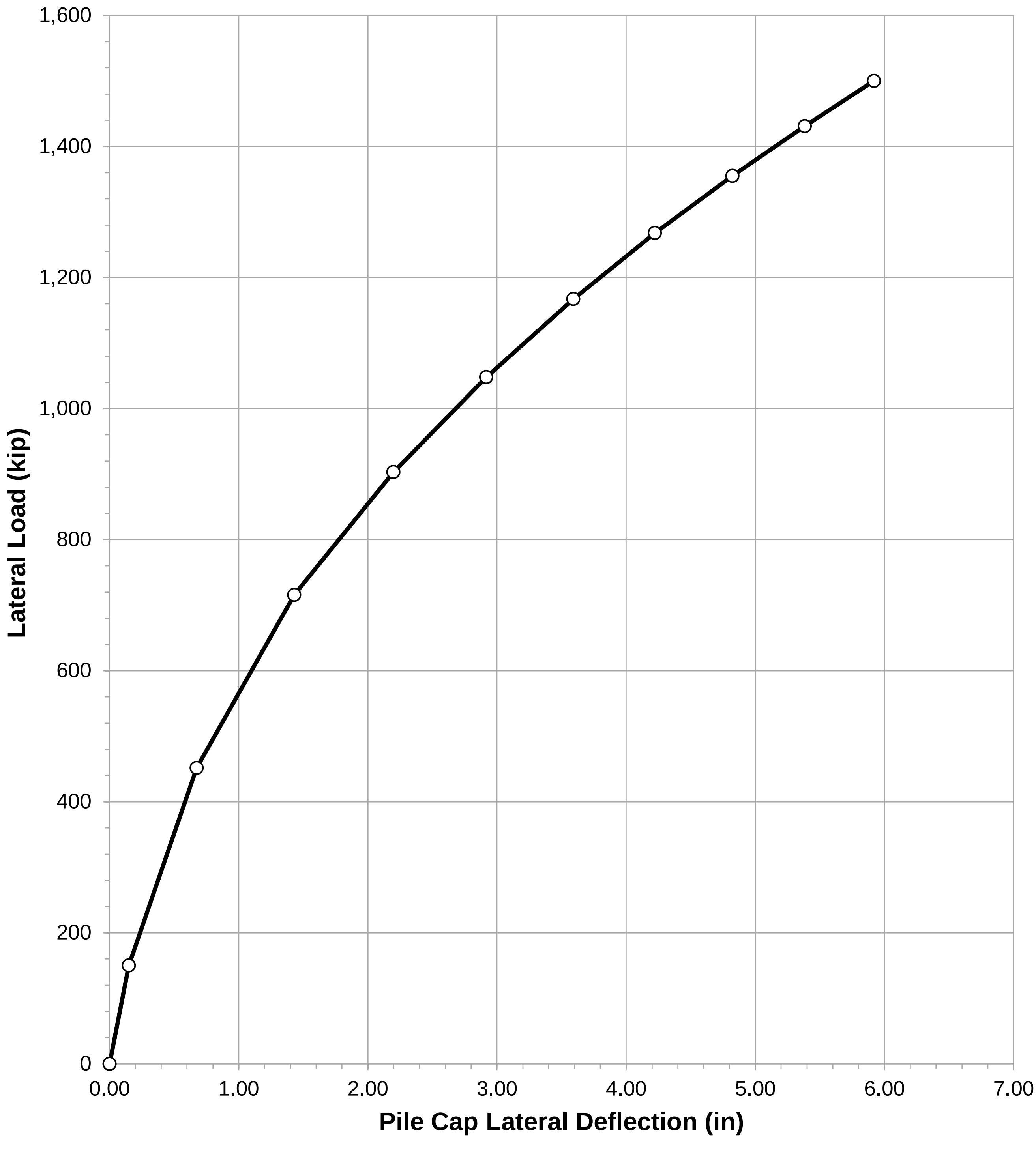
FIG. F17



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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 23

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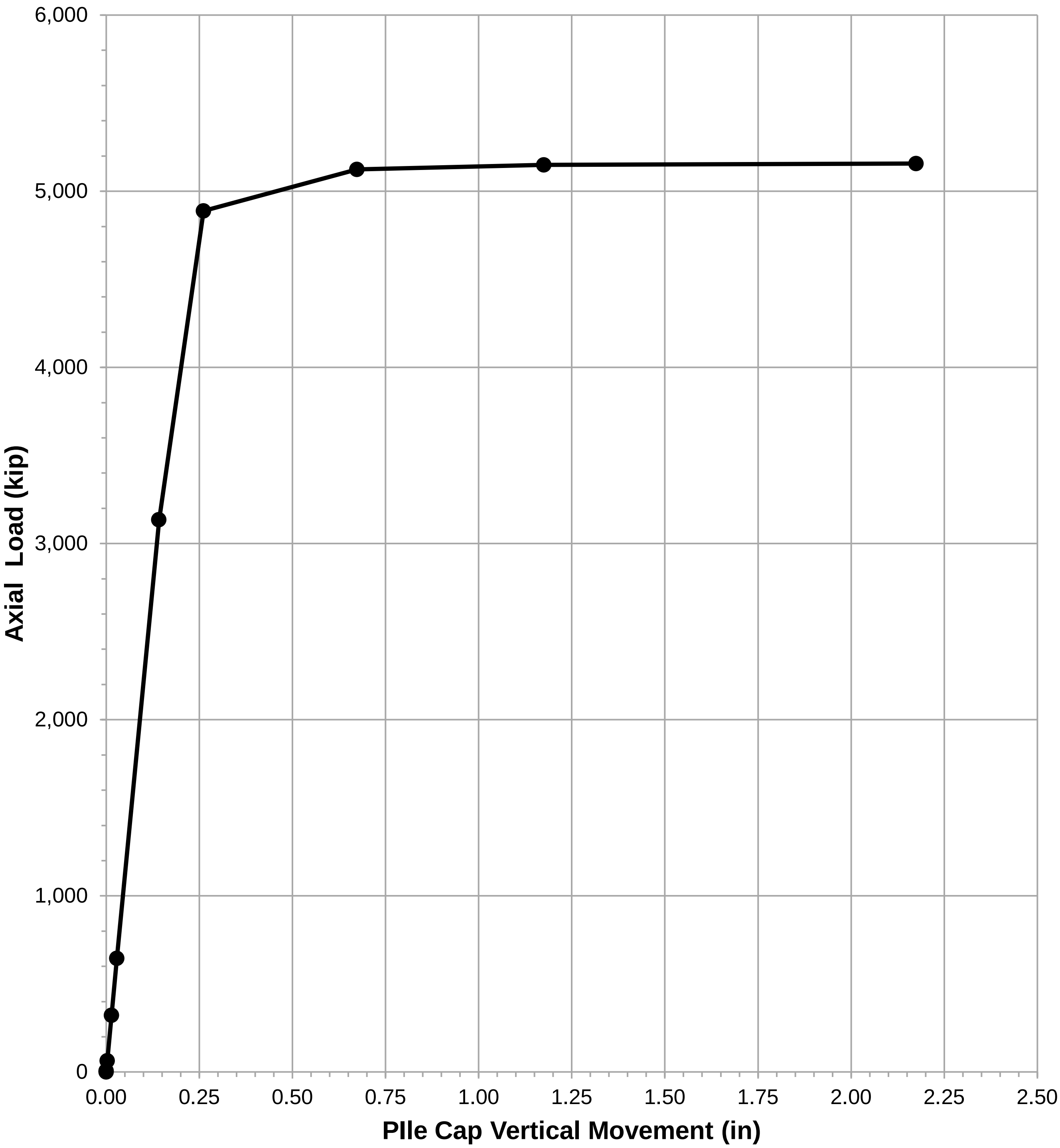
Geotechnical and Environmental Consultants

FIG. F18



NOTES
1. See Table 2 for details of each pile group evaluated.

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PILE GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 24

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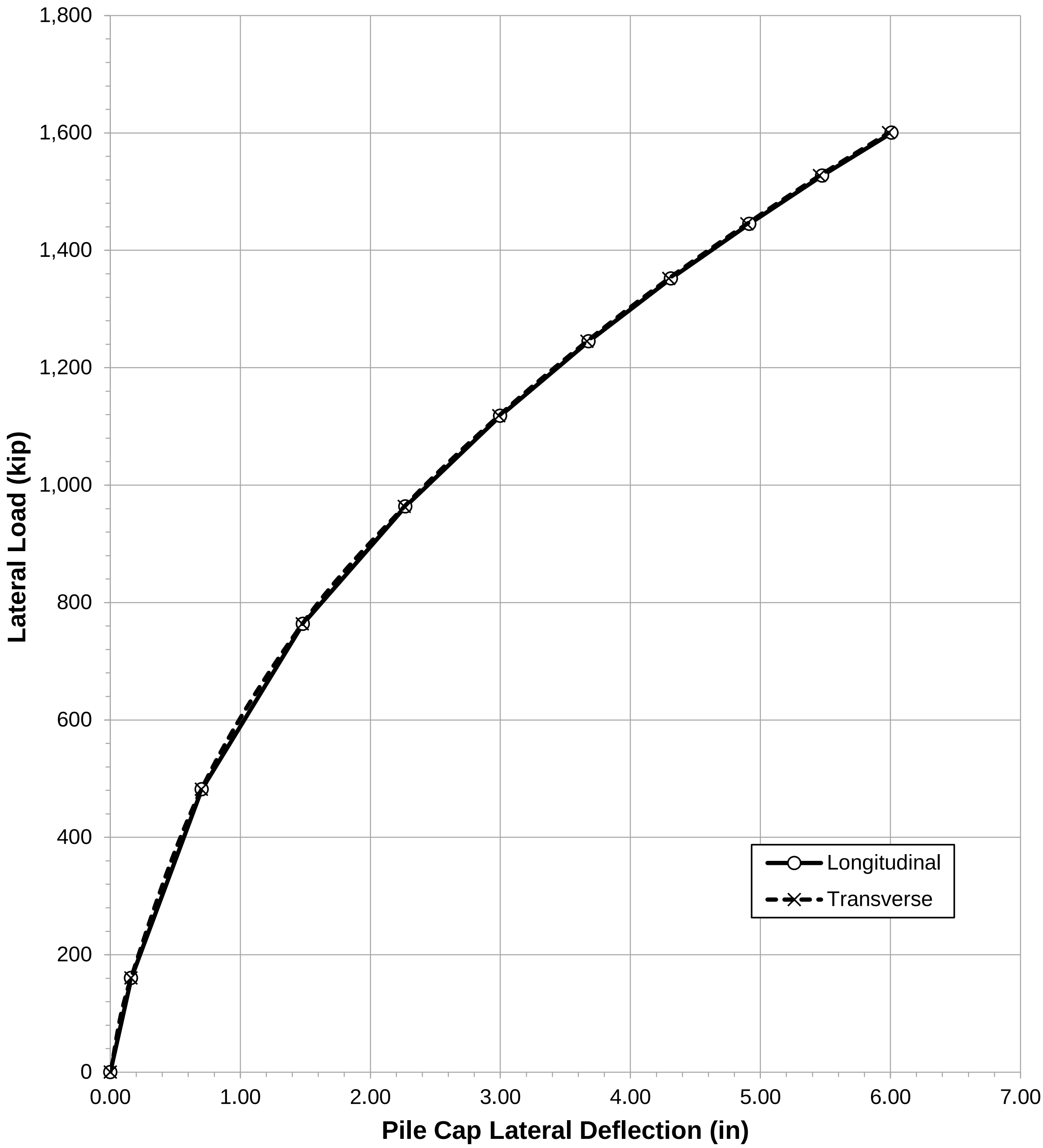
Geotechnical and Environmental Consultants

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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 24

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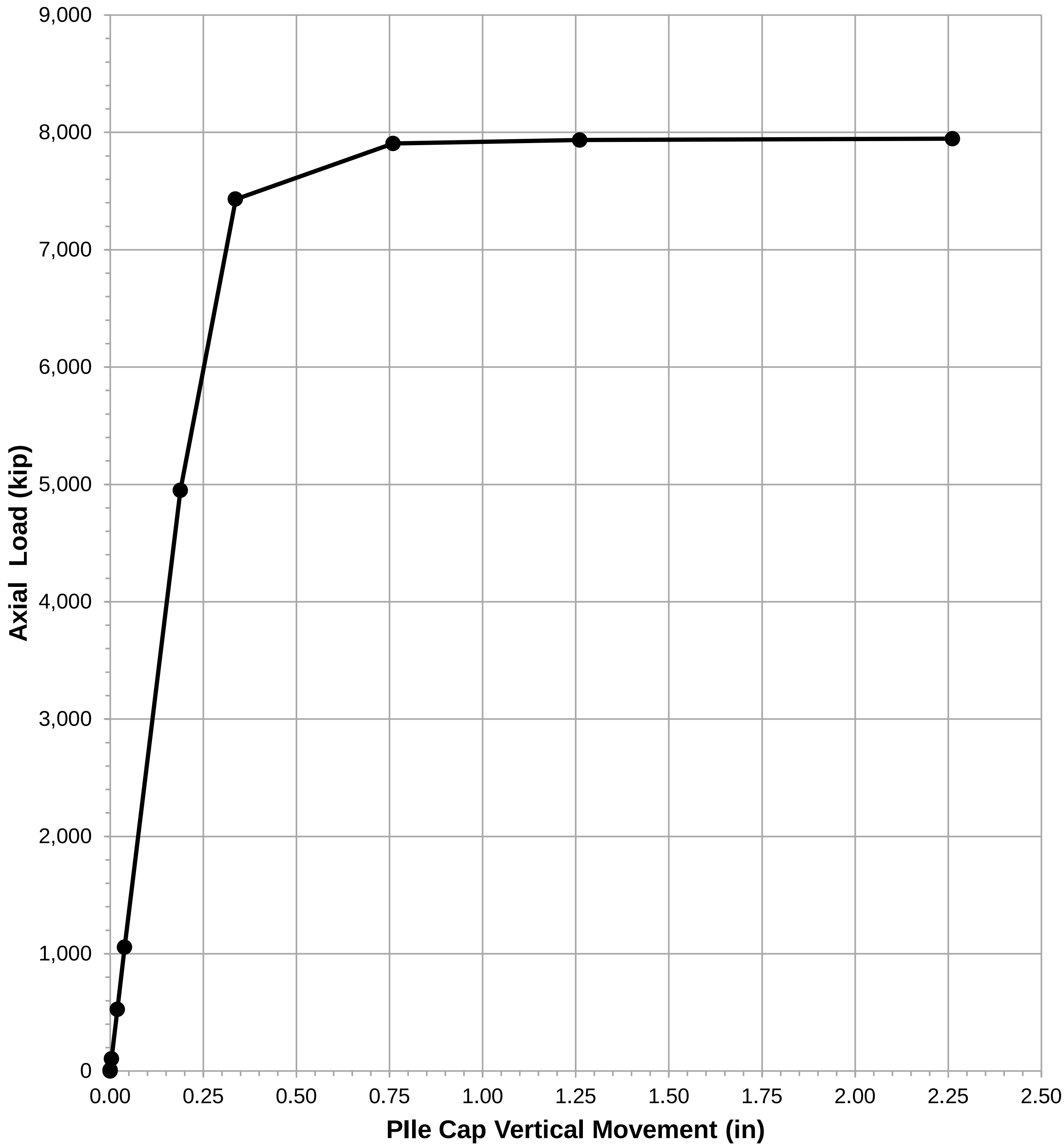
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F20



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 25

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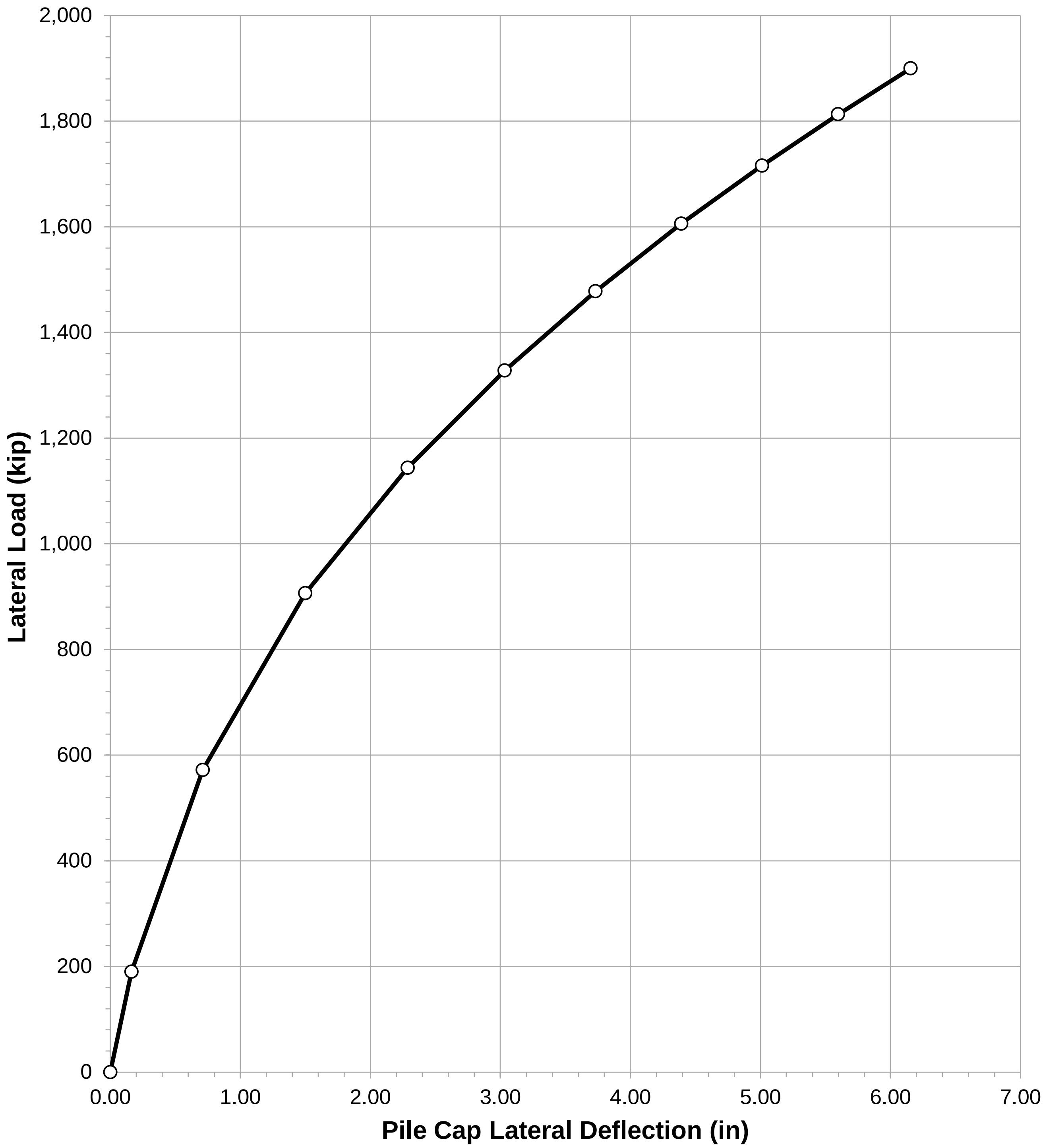
Geotechnical and Environmental Consultants

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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 25

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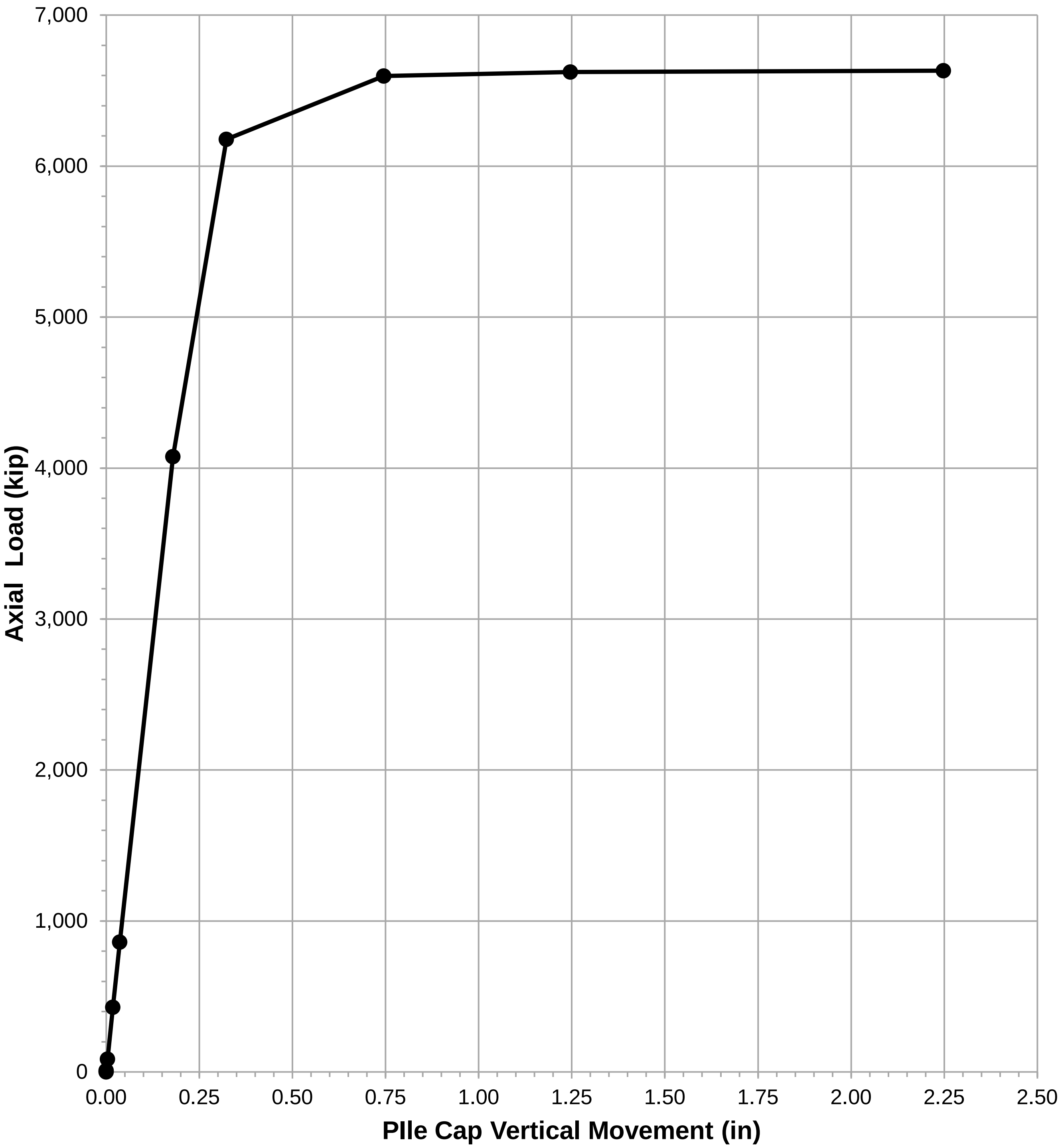
Geotechnical and Environmental Consultants

FIG. F22



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 26

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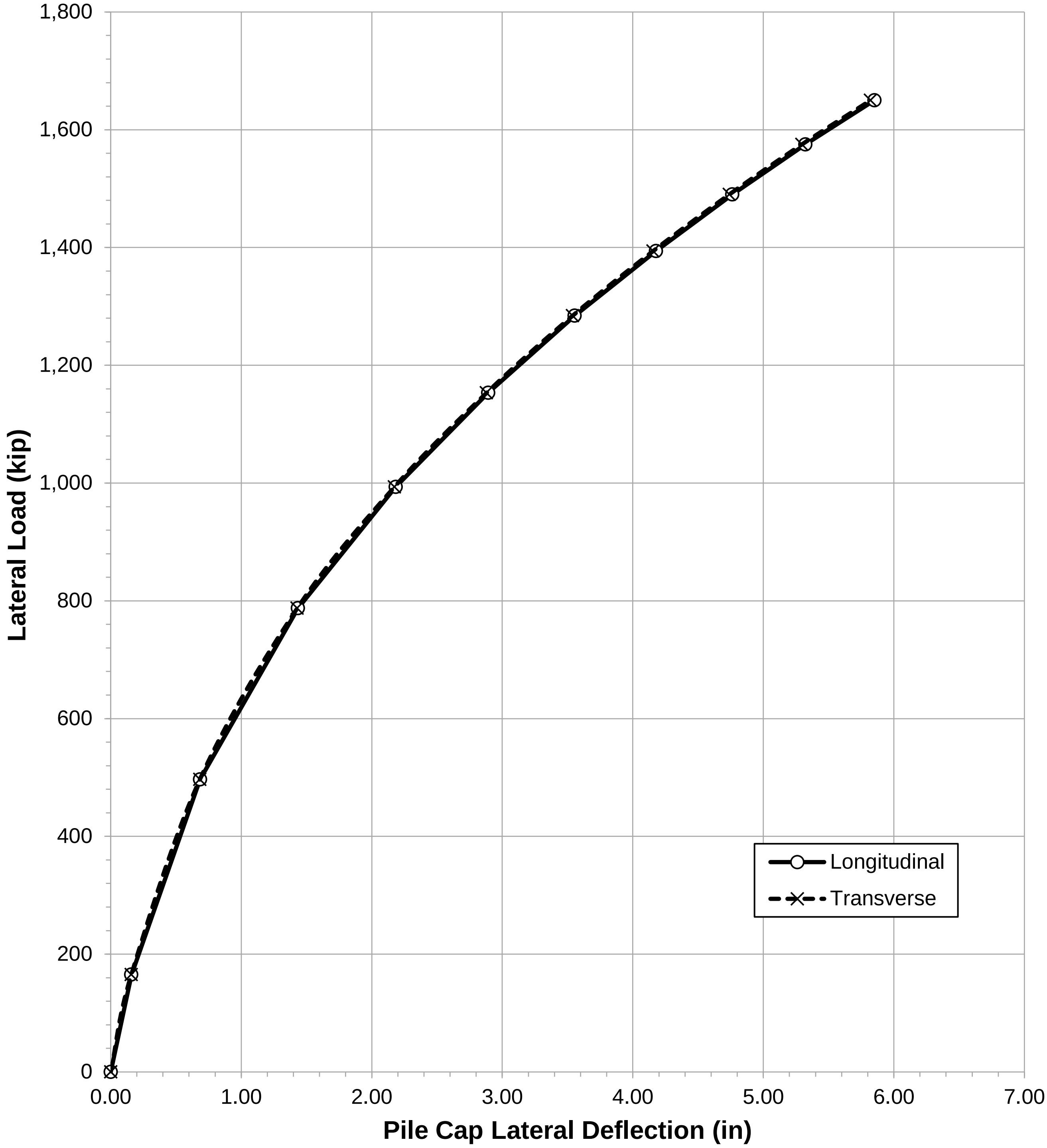
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

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.

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PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 26

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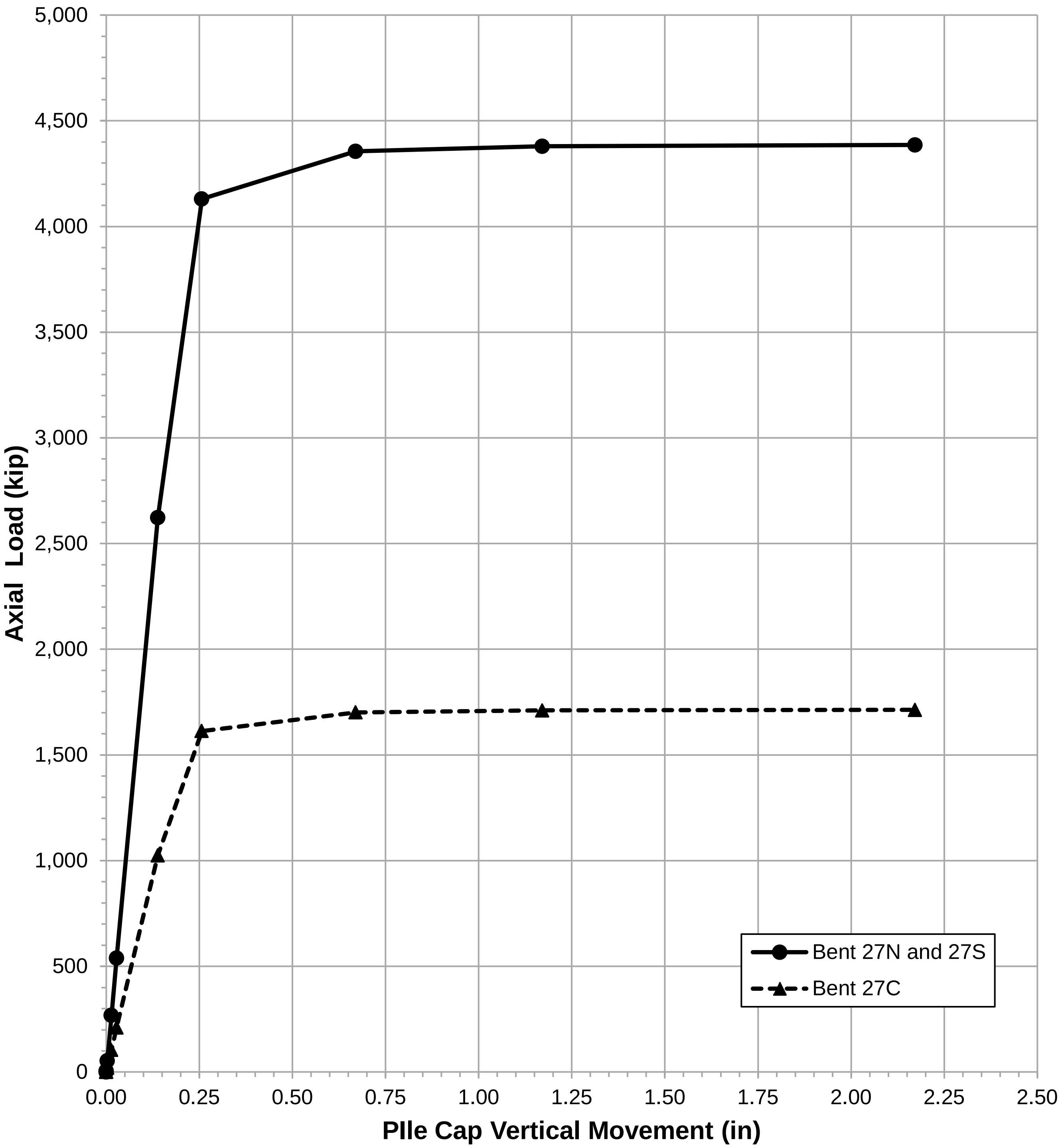
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F24



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 27

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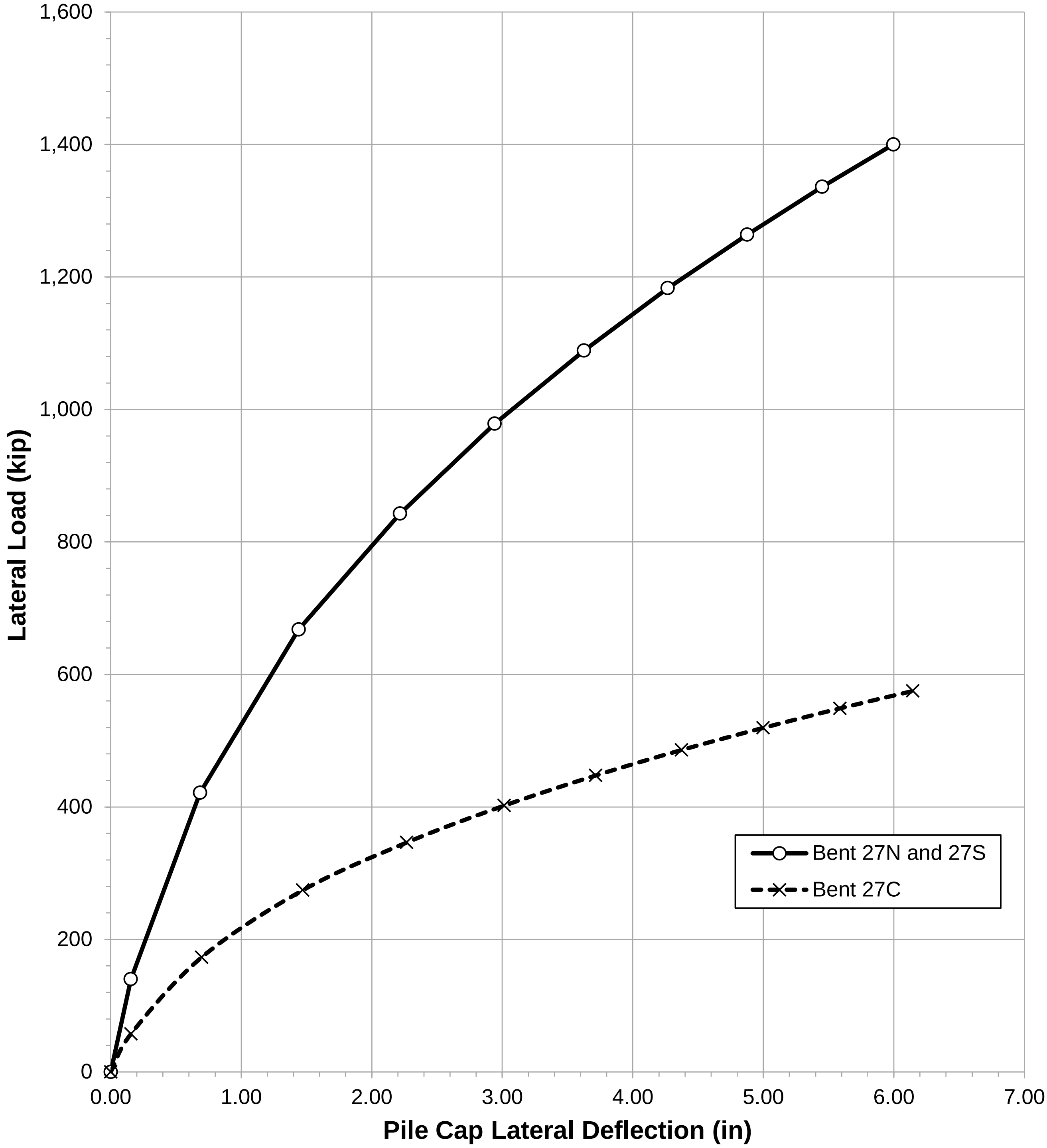
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

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.

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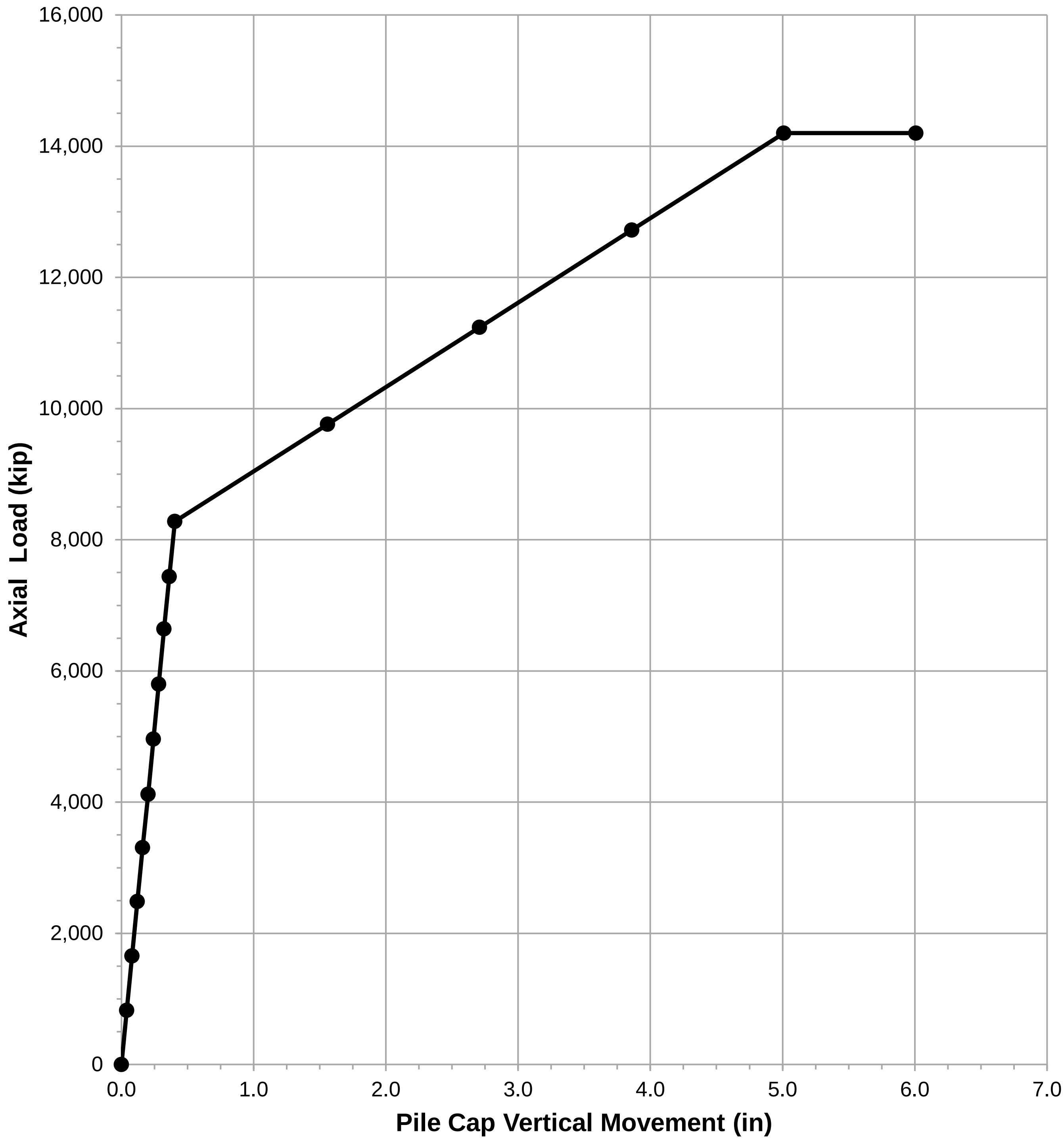
PILE GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 27 May 2017 24-1-04065-005	
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. F26**



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

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Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 17

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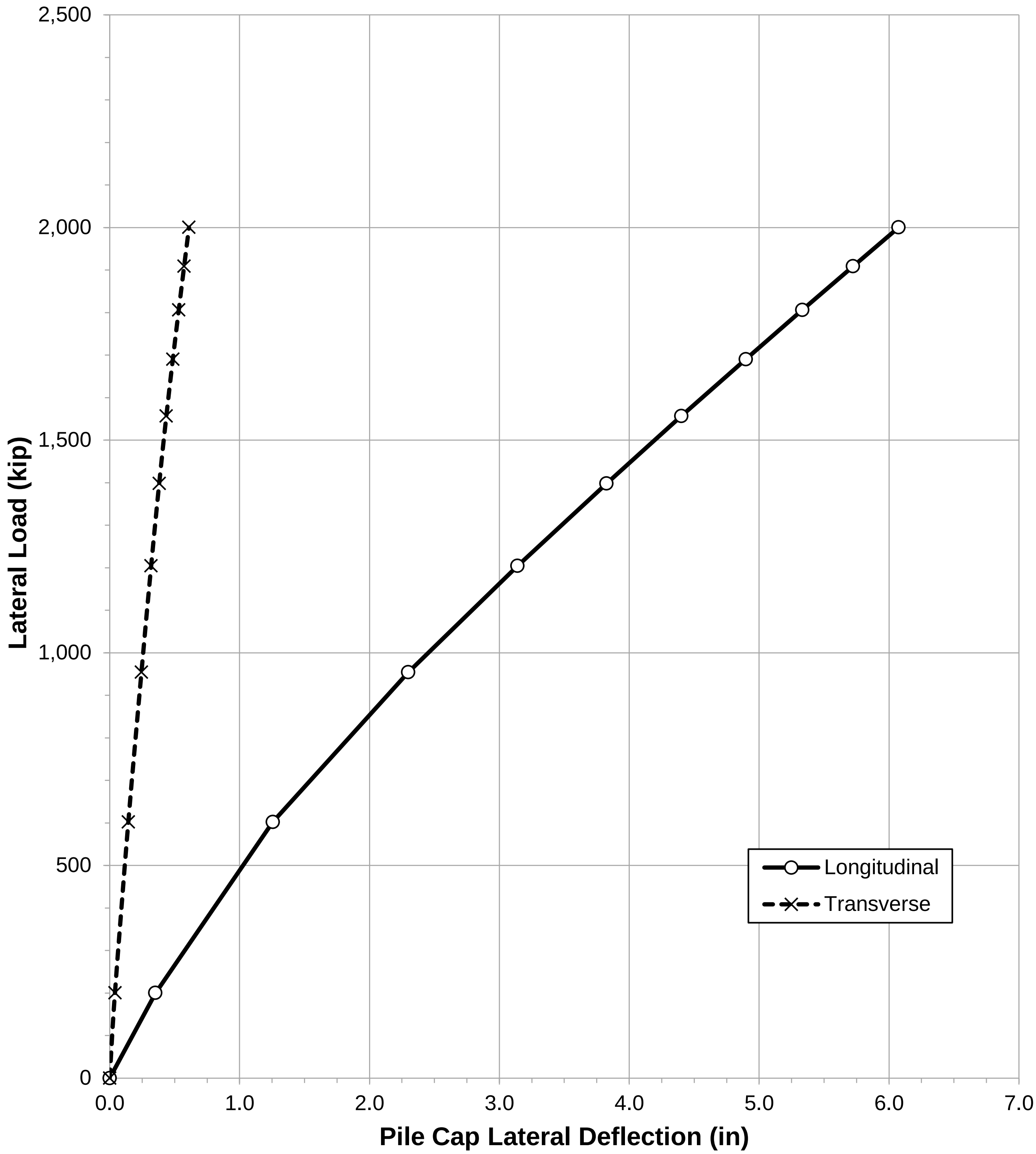
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F27



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 17

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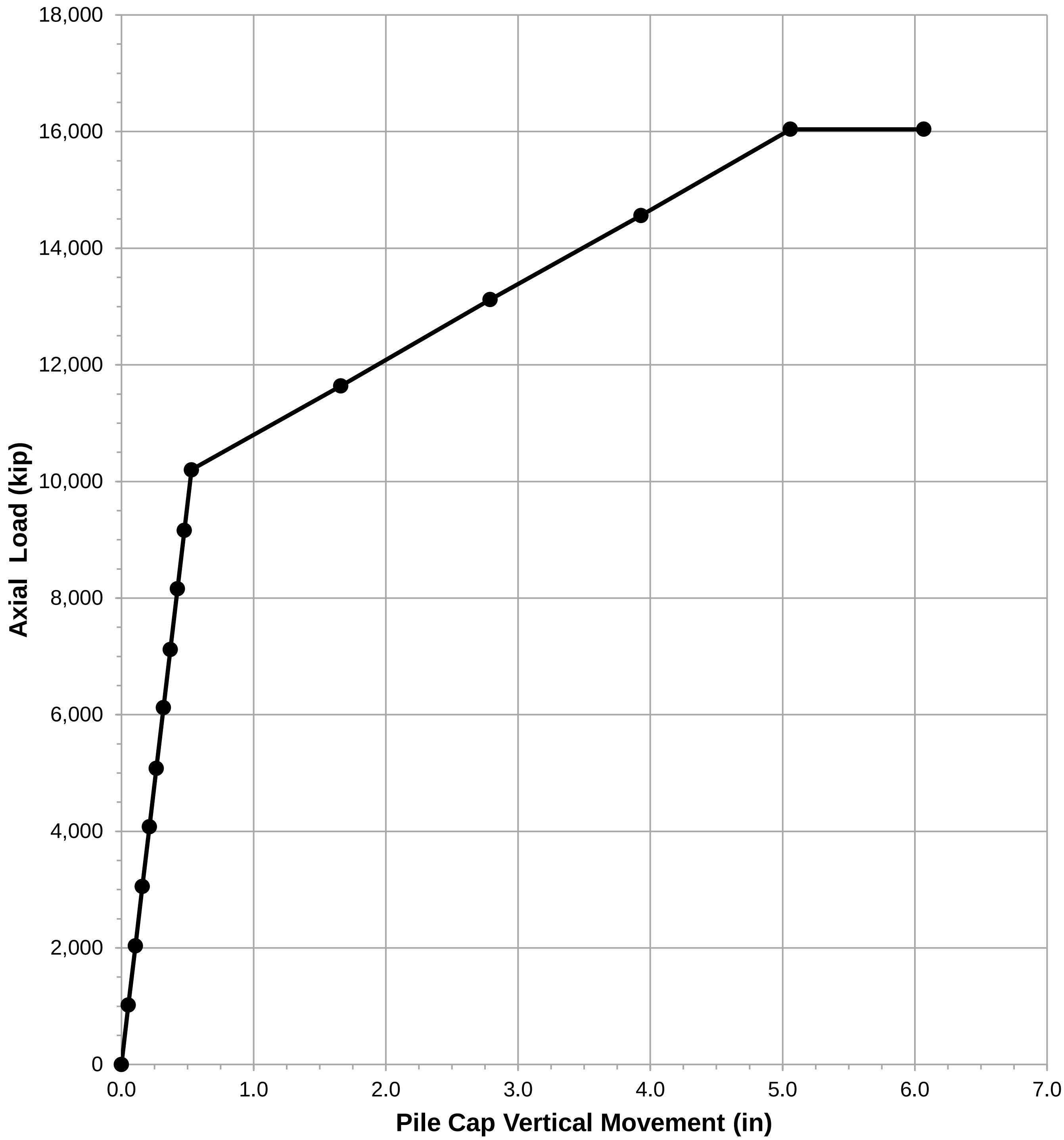
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F28



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

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DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 18

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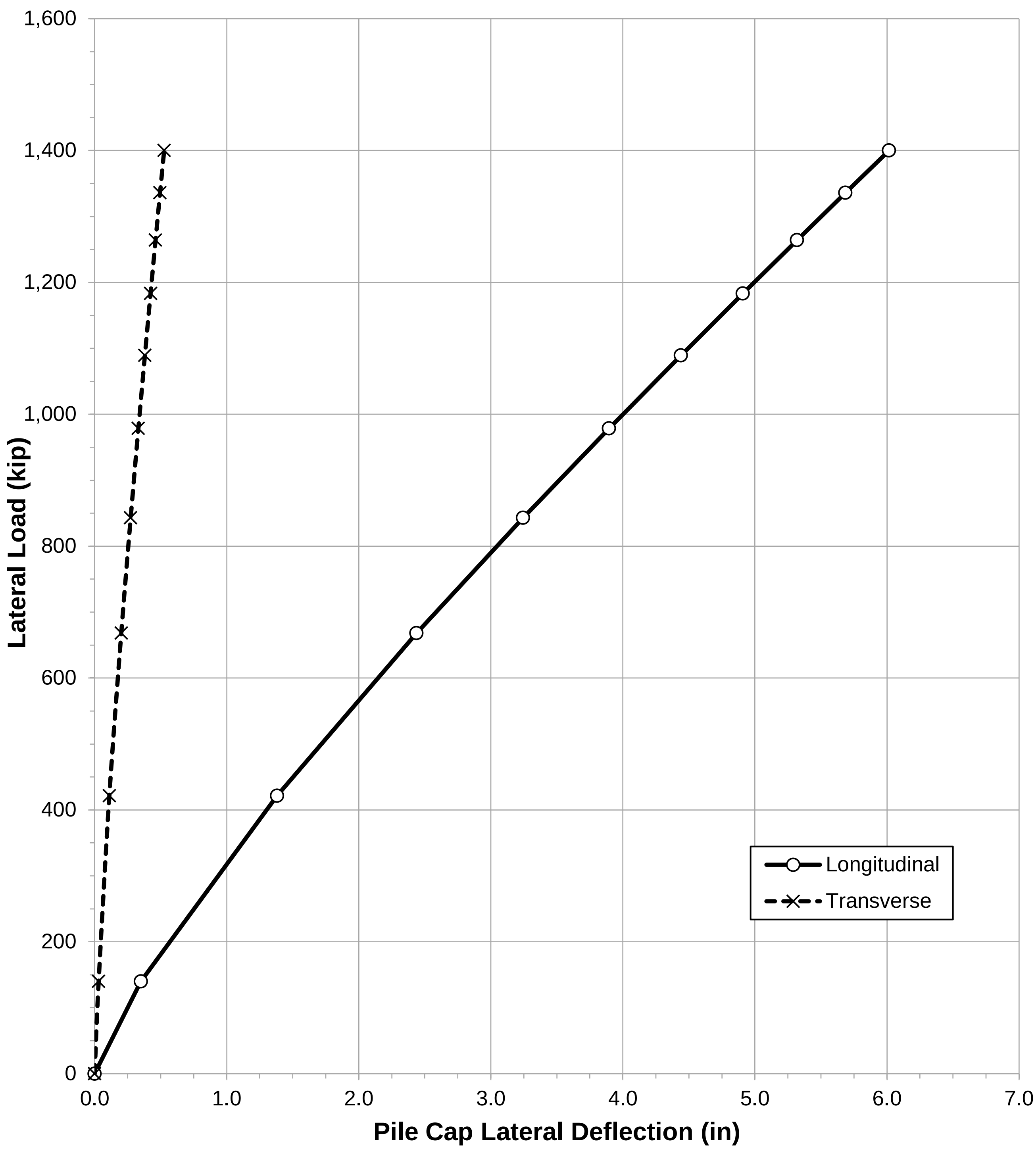
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F29



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 18

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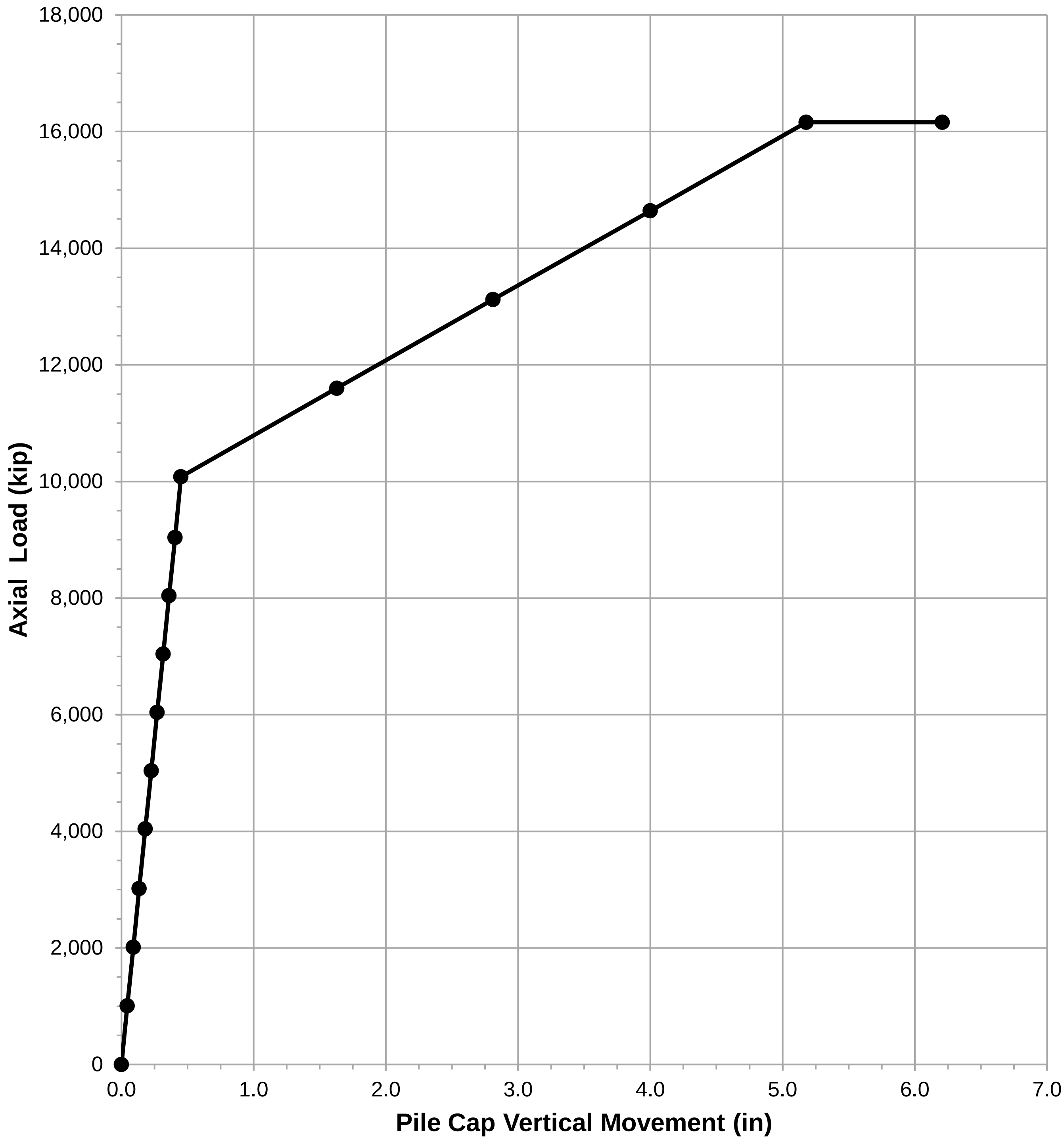
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F30



NOTES
1. See Table 11 for details of each drilled shaft group evaluated.

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Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 19

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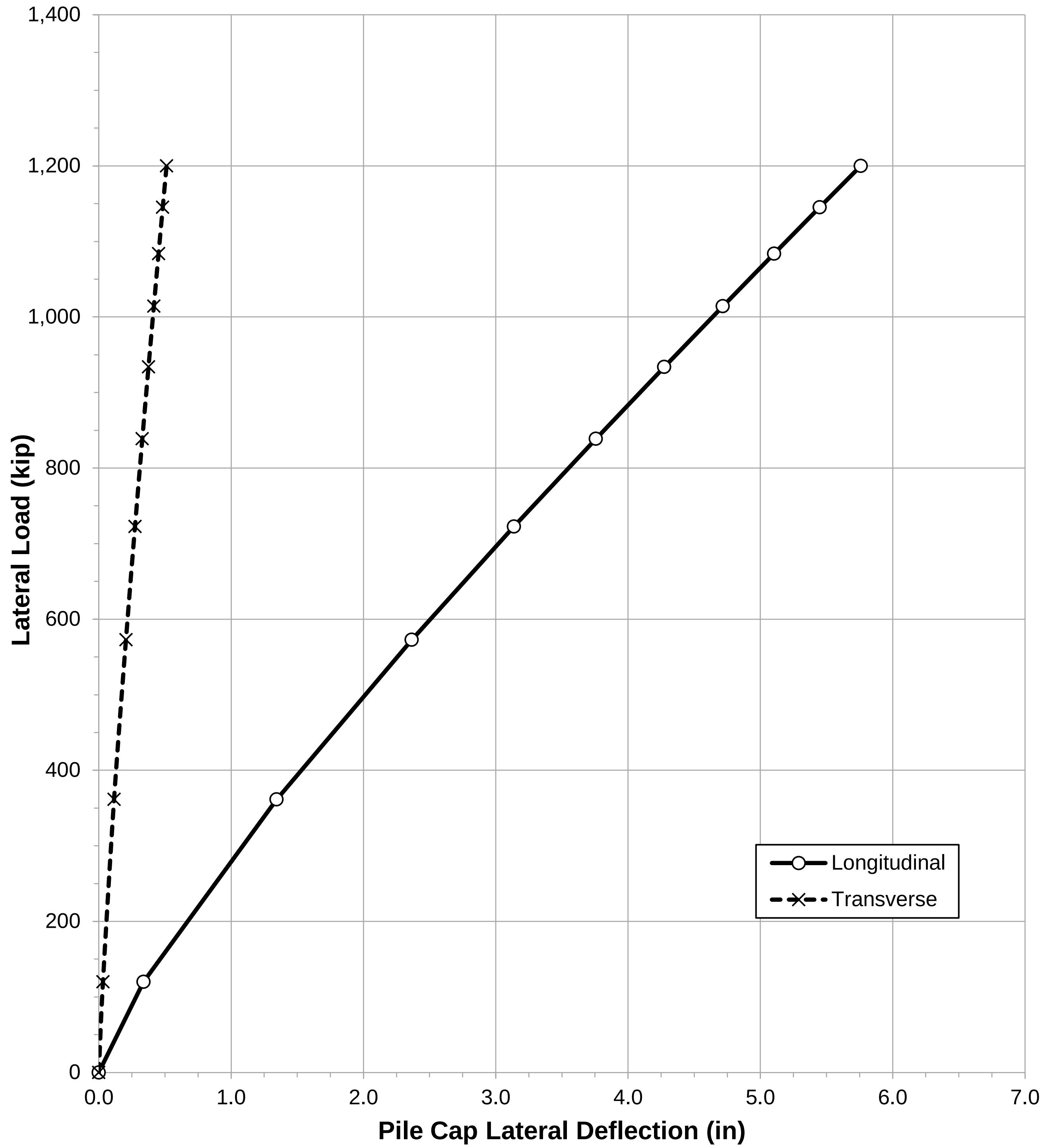
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F31



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 19

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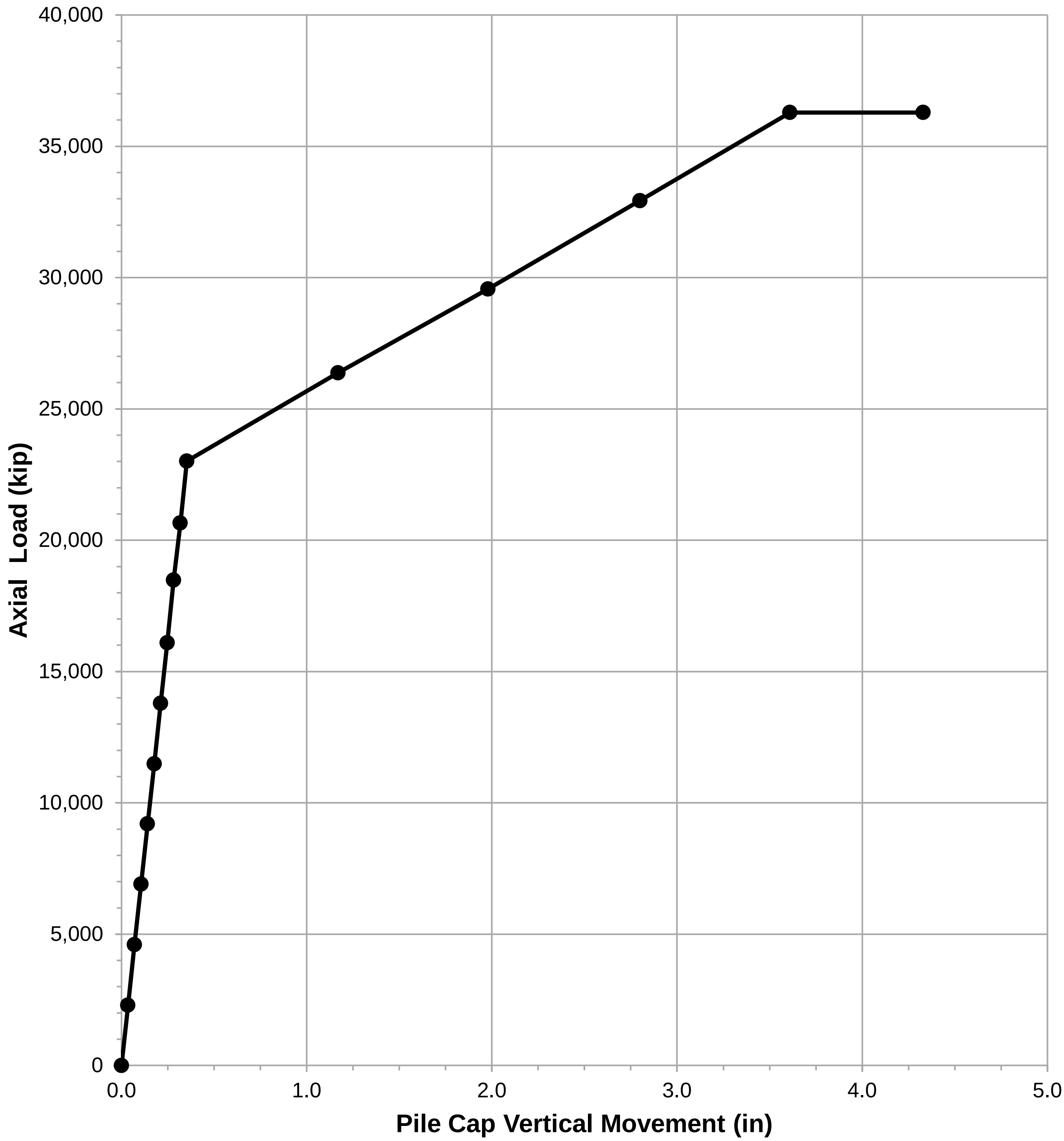
FIG. F32



NOTES
1. See Table 11 for details of each drilled shaft group evaluated.

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DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 1

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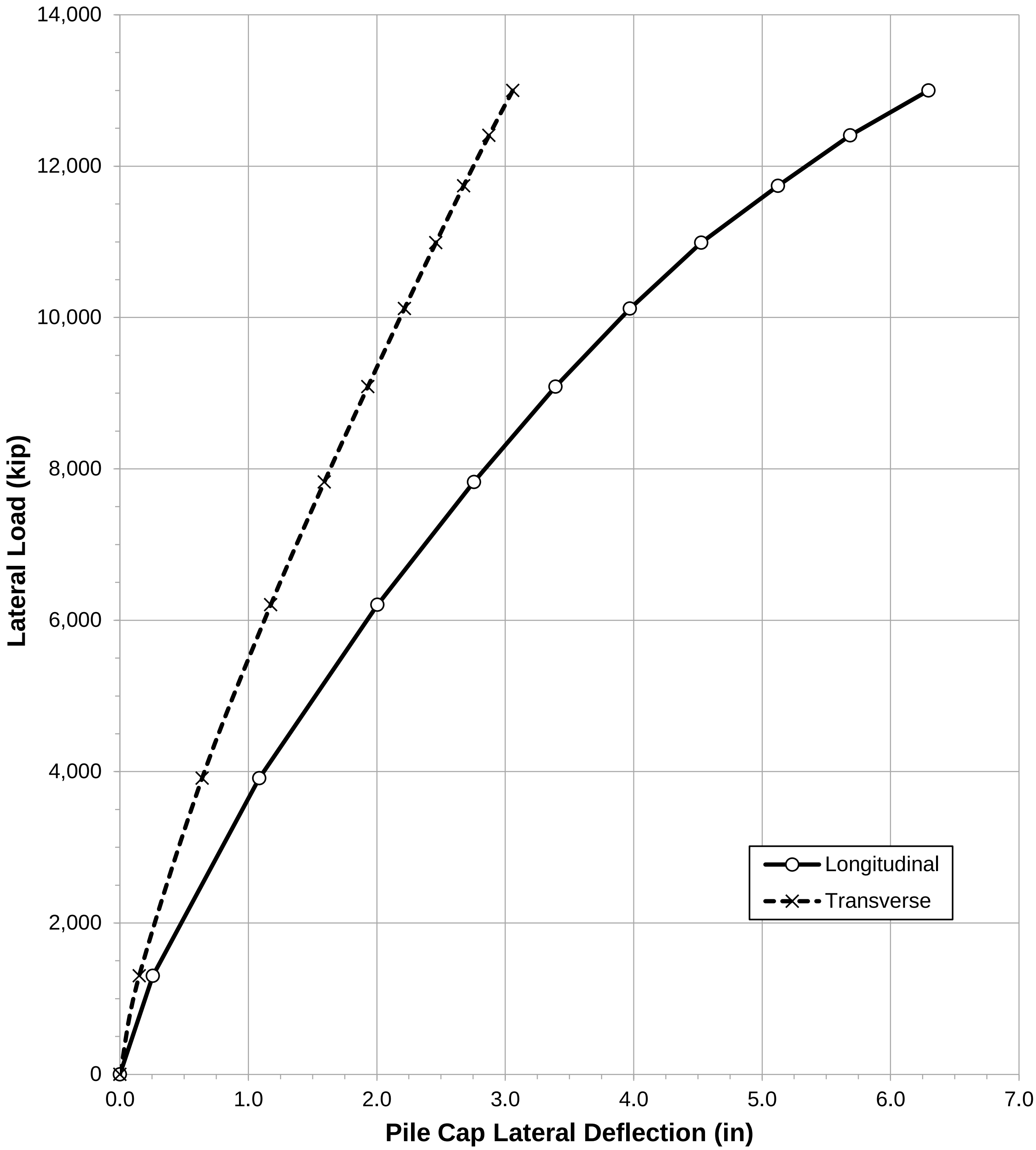
FIG. F33



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 1

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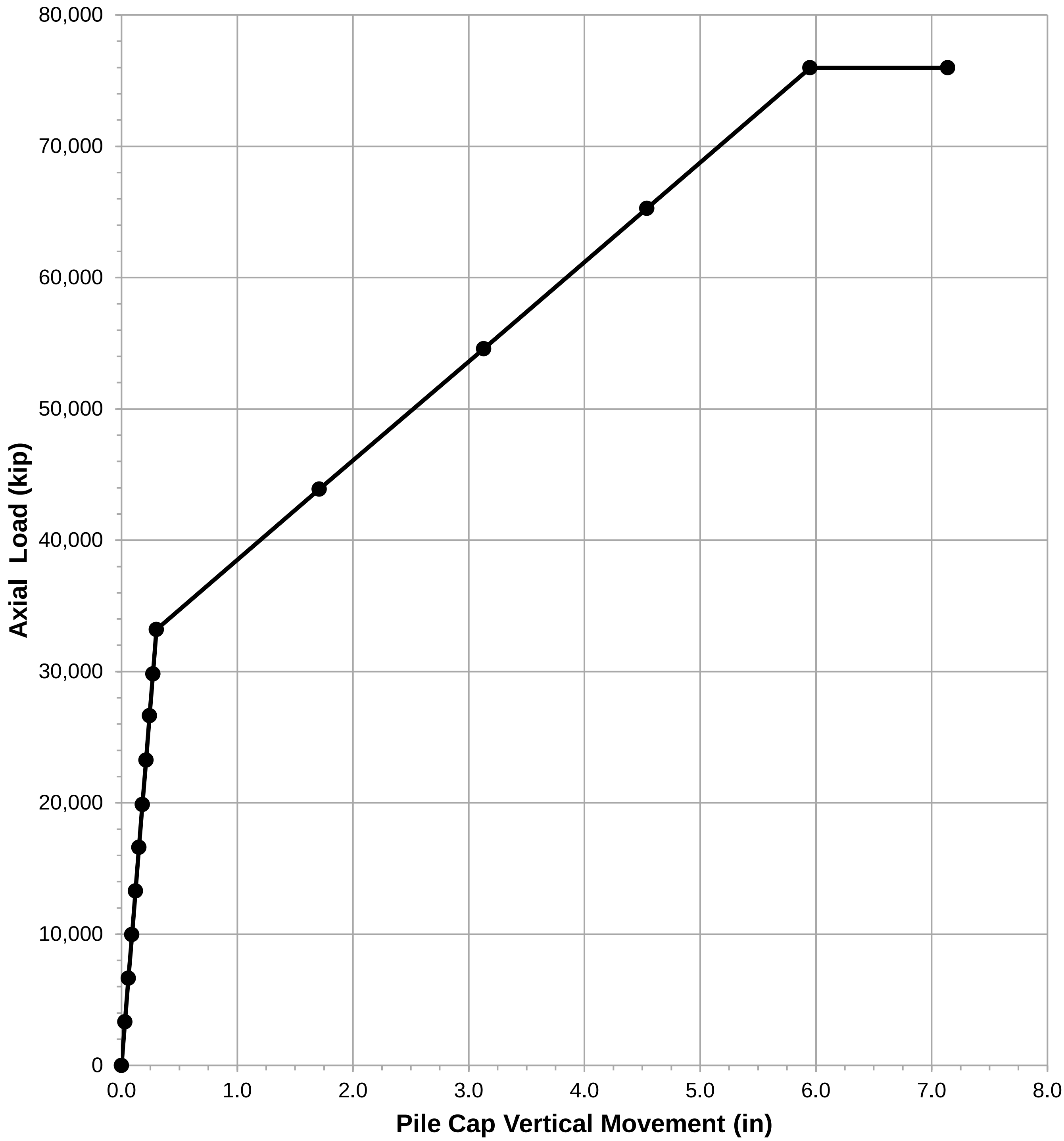
FIG. F34



<u>NOTES</u>

1. See Table 11 for details of each drilled shaft group evaluated.

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DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 2

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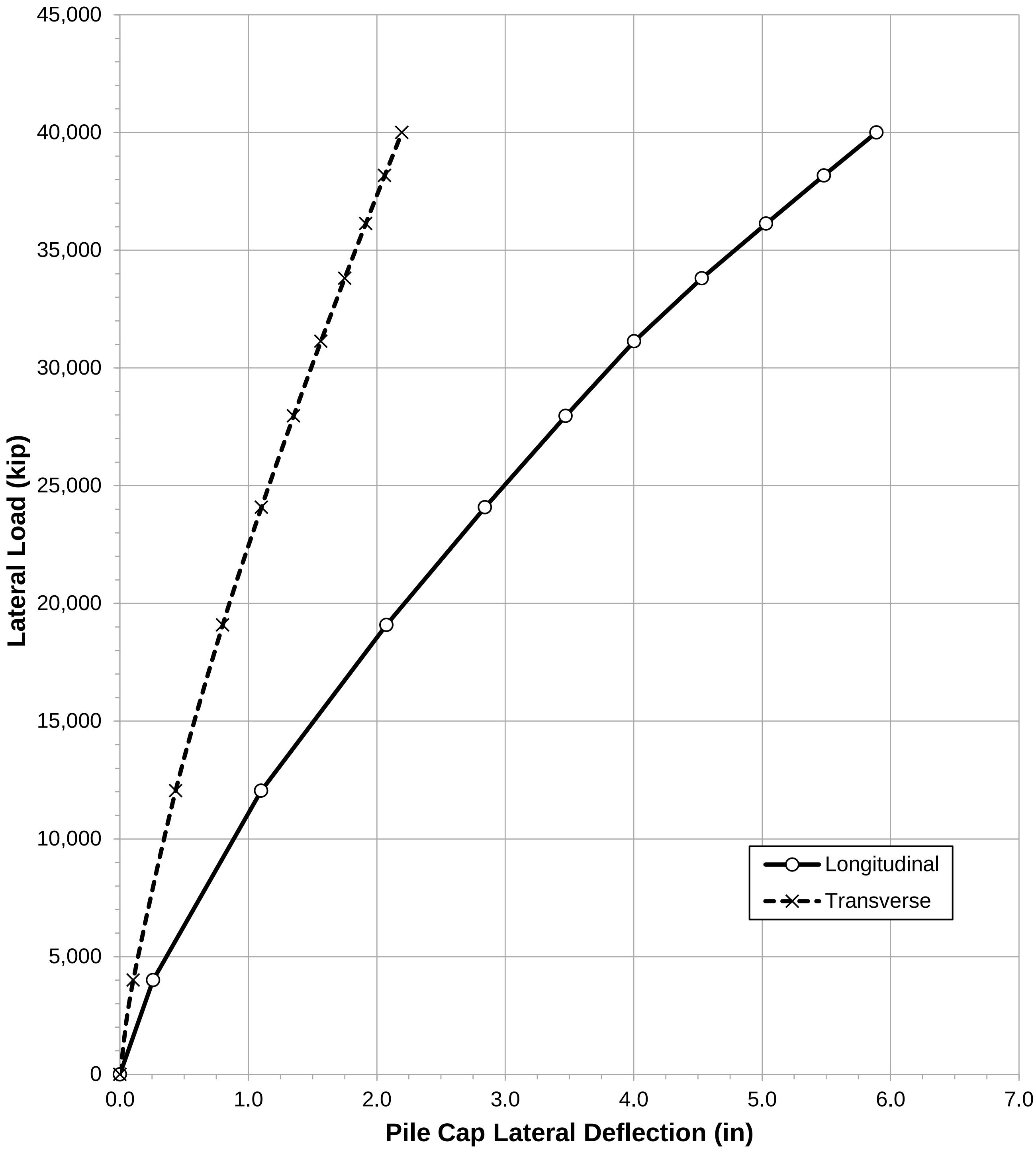
Geotechnical and Environmental Consultants

FIG. F35



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 2

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SHANNON & WILSON, INC.
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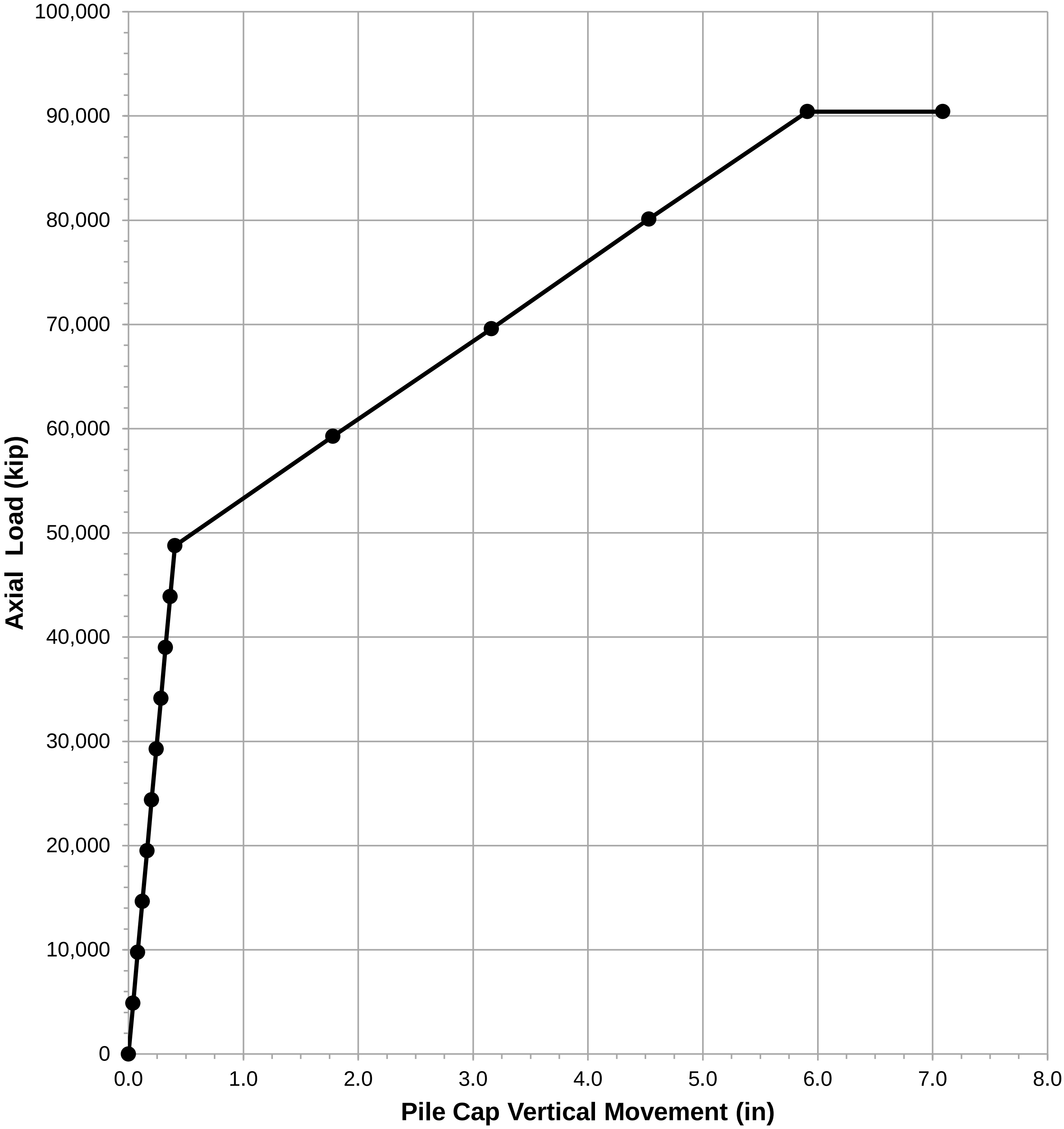
FIG. F36



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

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DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 3

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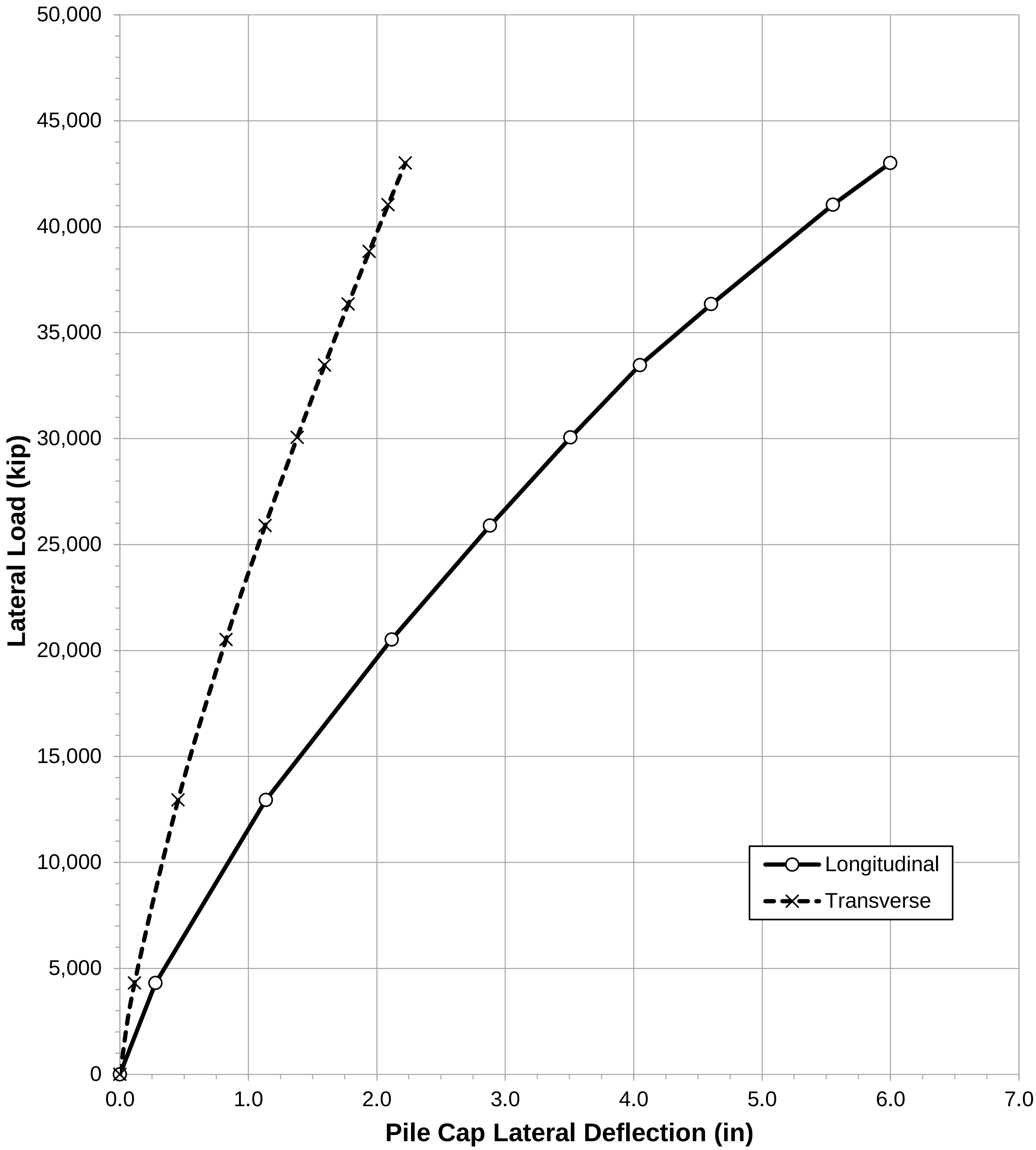
FIG. F37



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 3

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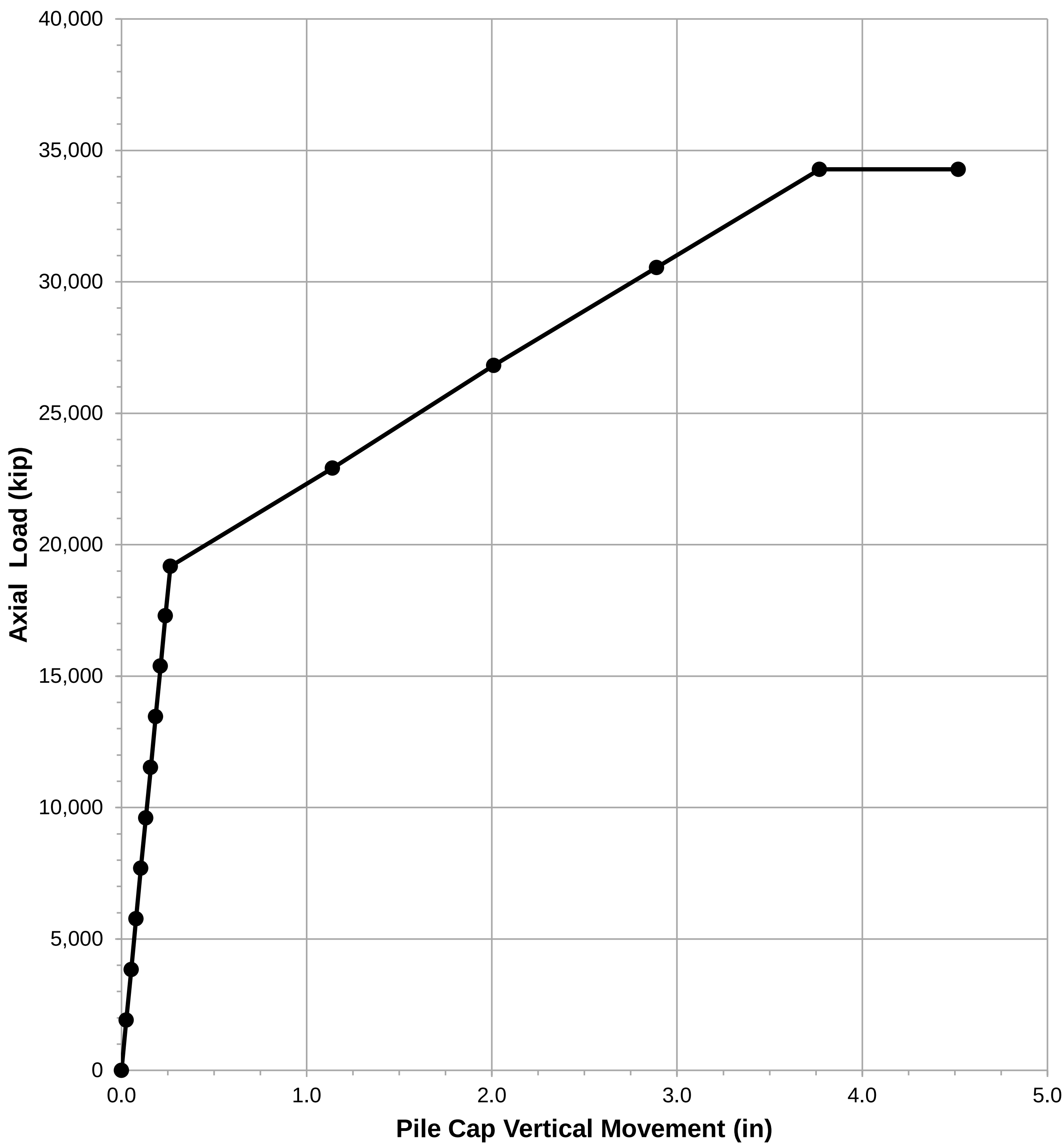
FIG. F38



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

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DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 4

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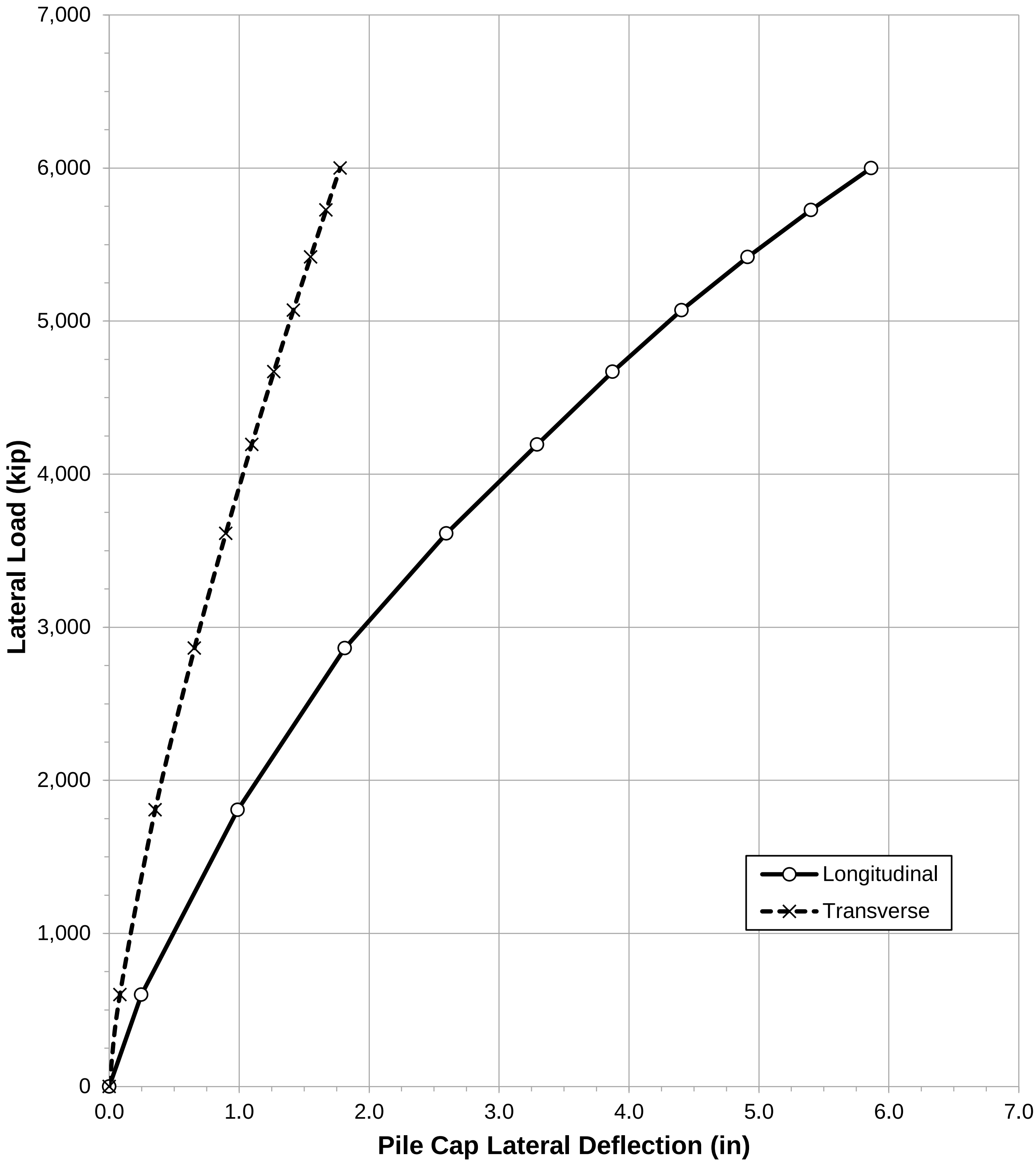
FIG. F39



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

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DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT PIER 4

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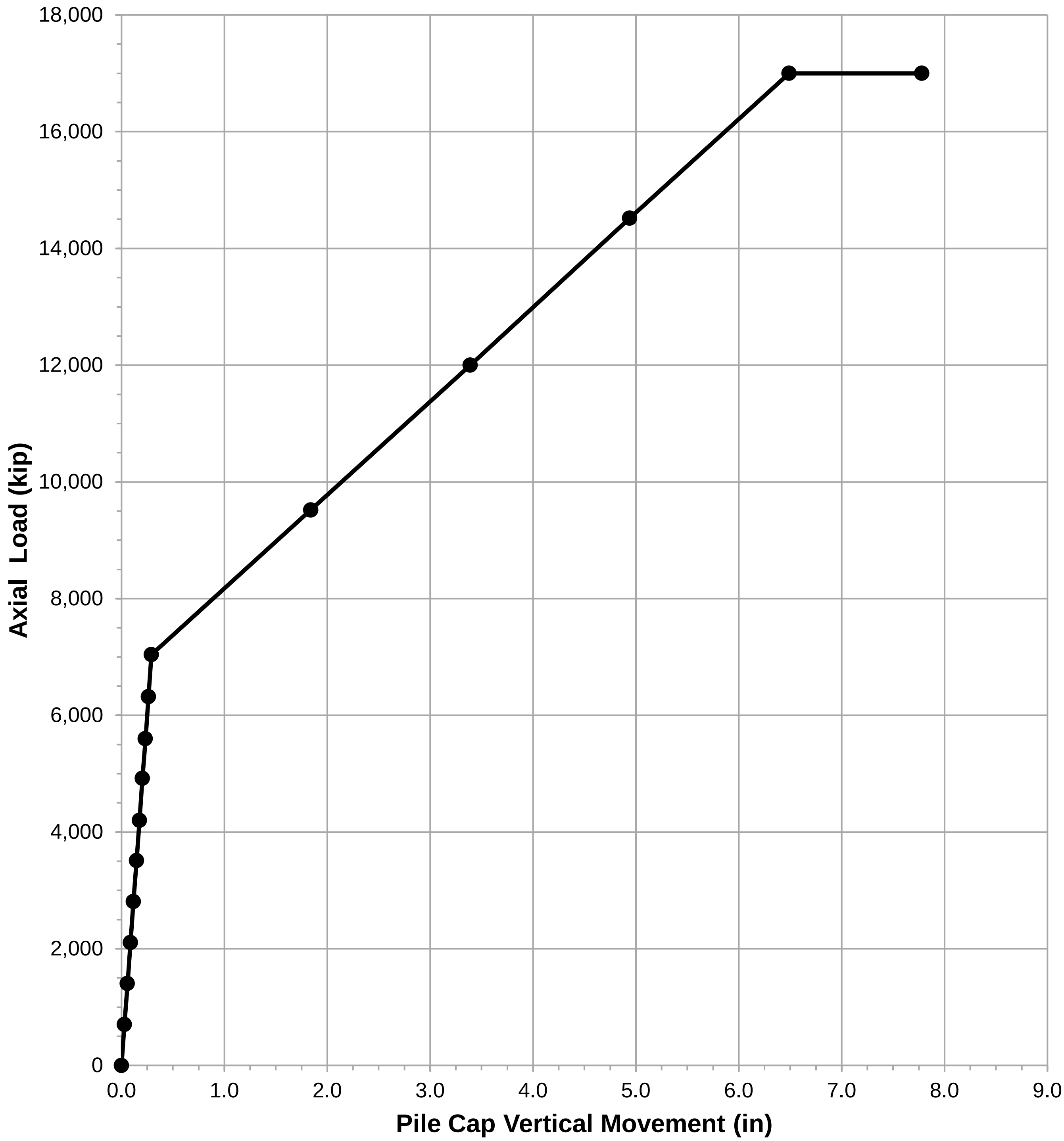
FIG. F40



<u>NOTES</u>

1. See Table 11 for details of each drilled shaft group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 21

August 2017 24-1-04065-005

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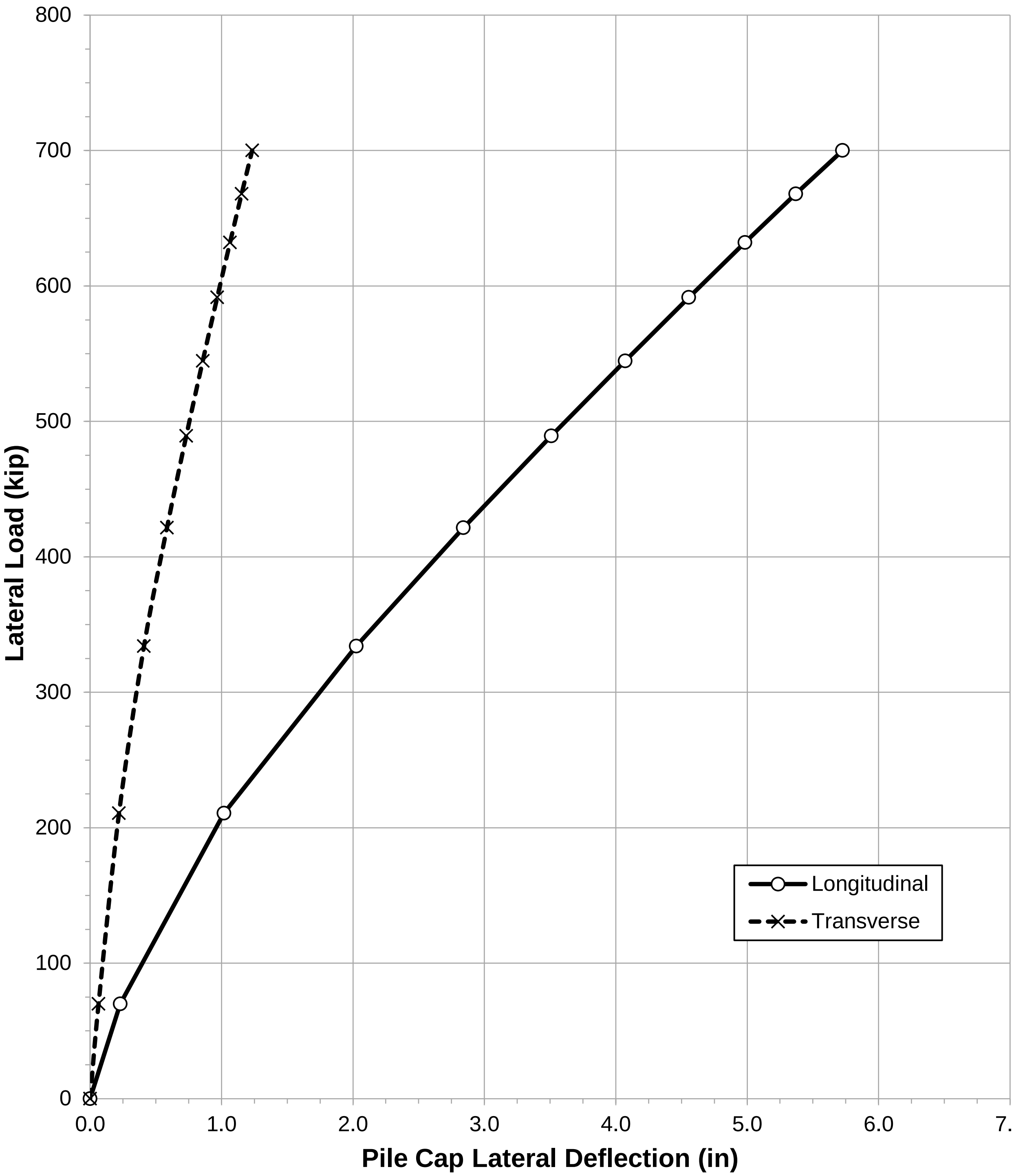
FIG. F41



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 21

August 2017 | 24-1-04065-005

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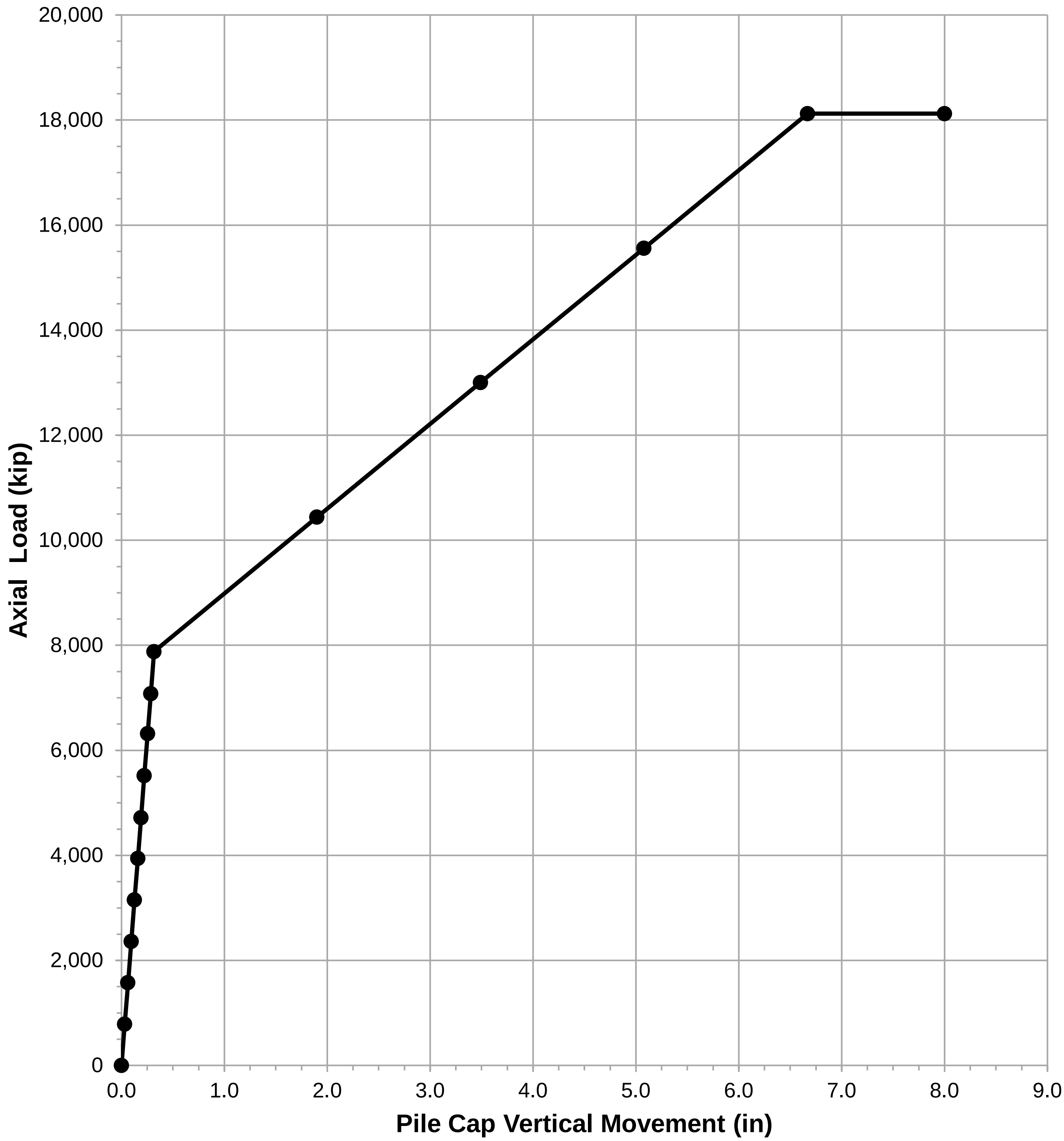
Geotechnical and Environmental Consultants

FIG. F42



NOTES
1. See Table 11 for details of each drilled shaft group evaluated.

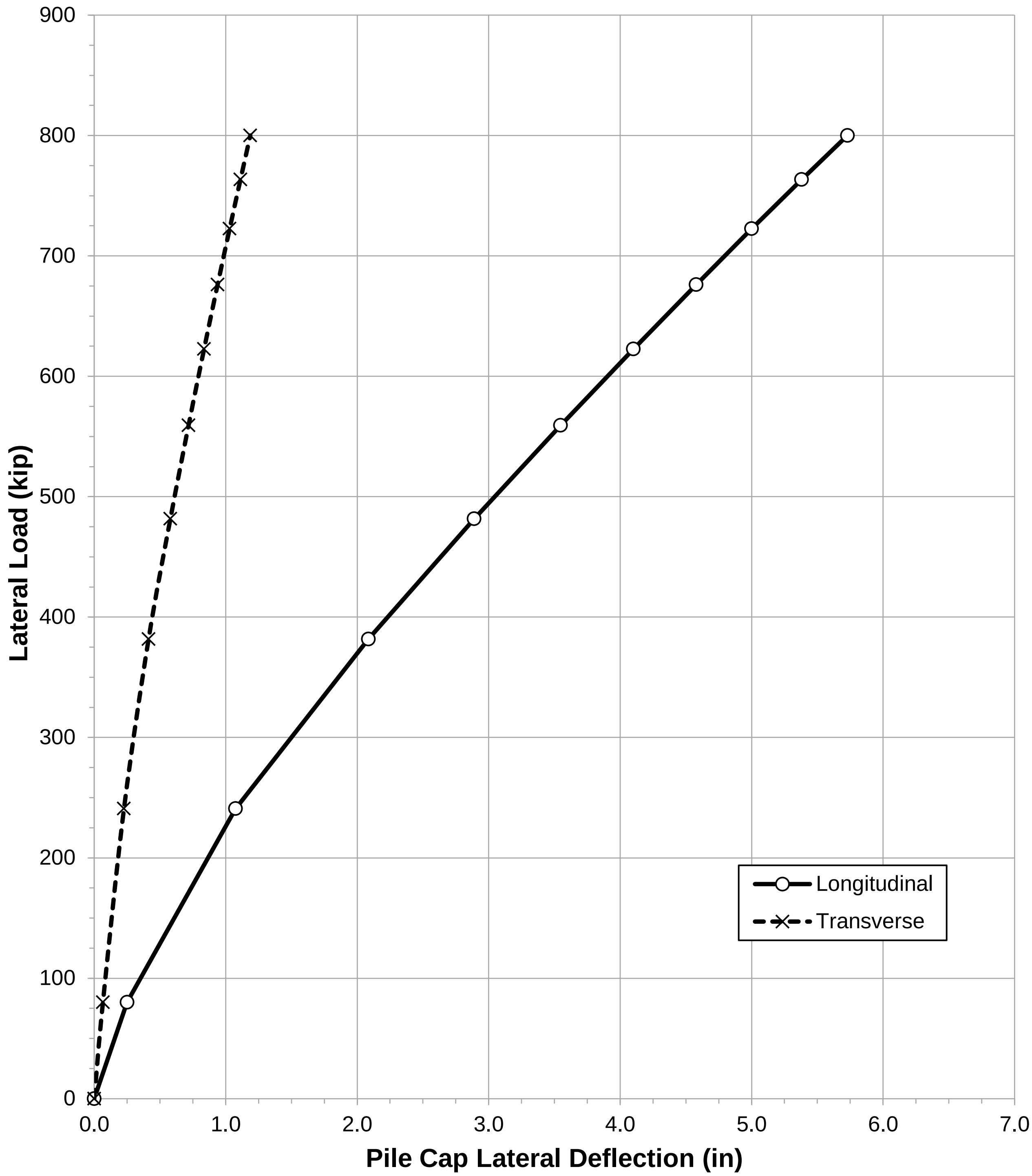
Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 22

August 2017 | 24-1-04065-005

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F43



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

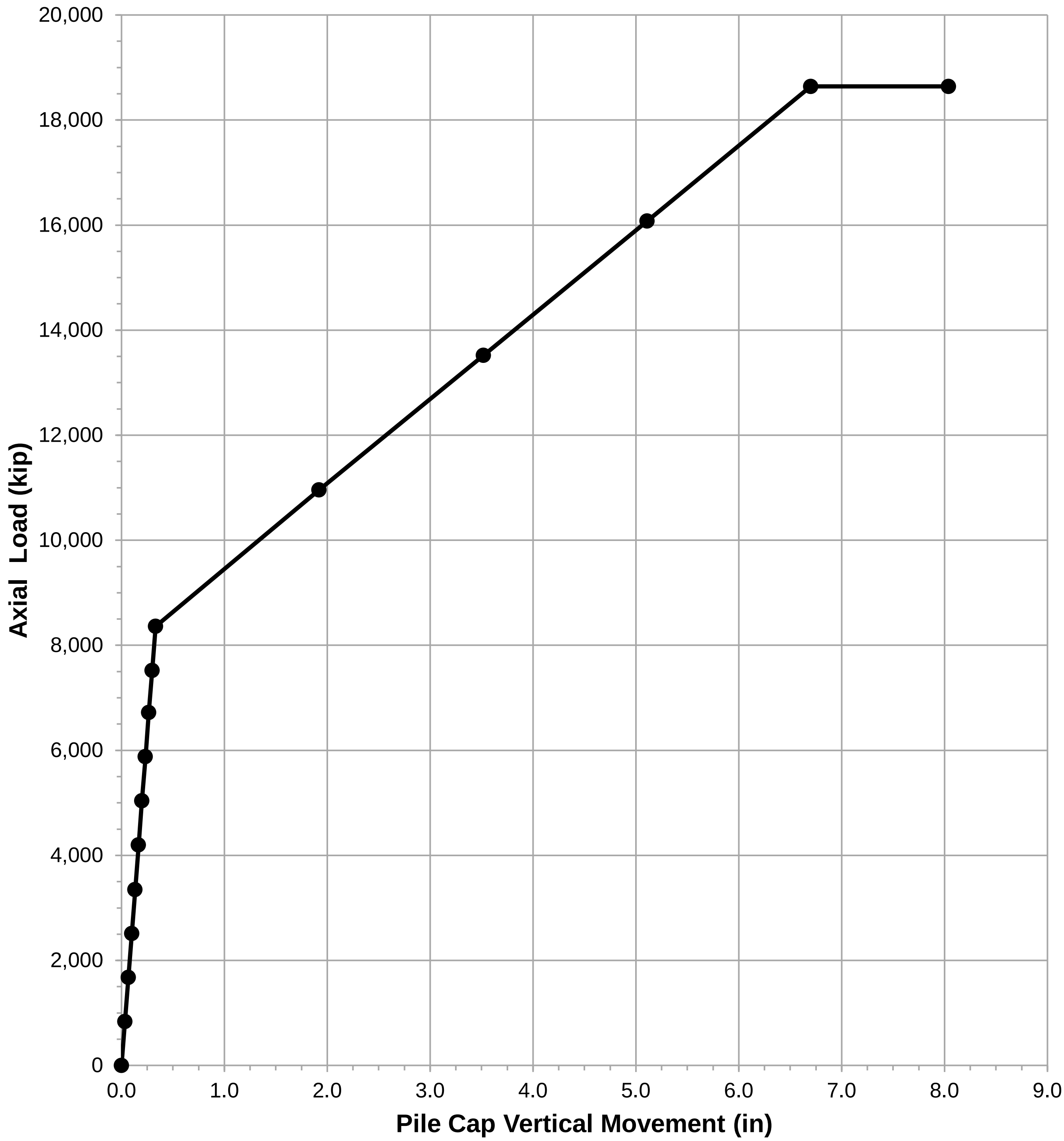
Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 22

August 2017 24-1-04065-005

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F44



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 23

August 2017 | 24-1-04065-005

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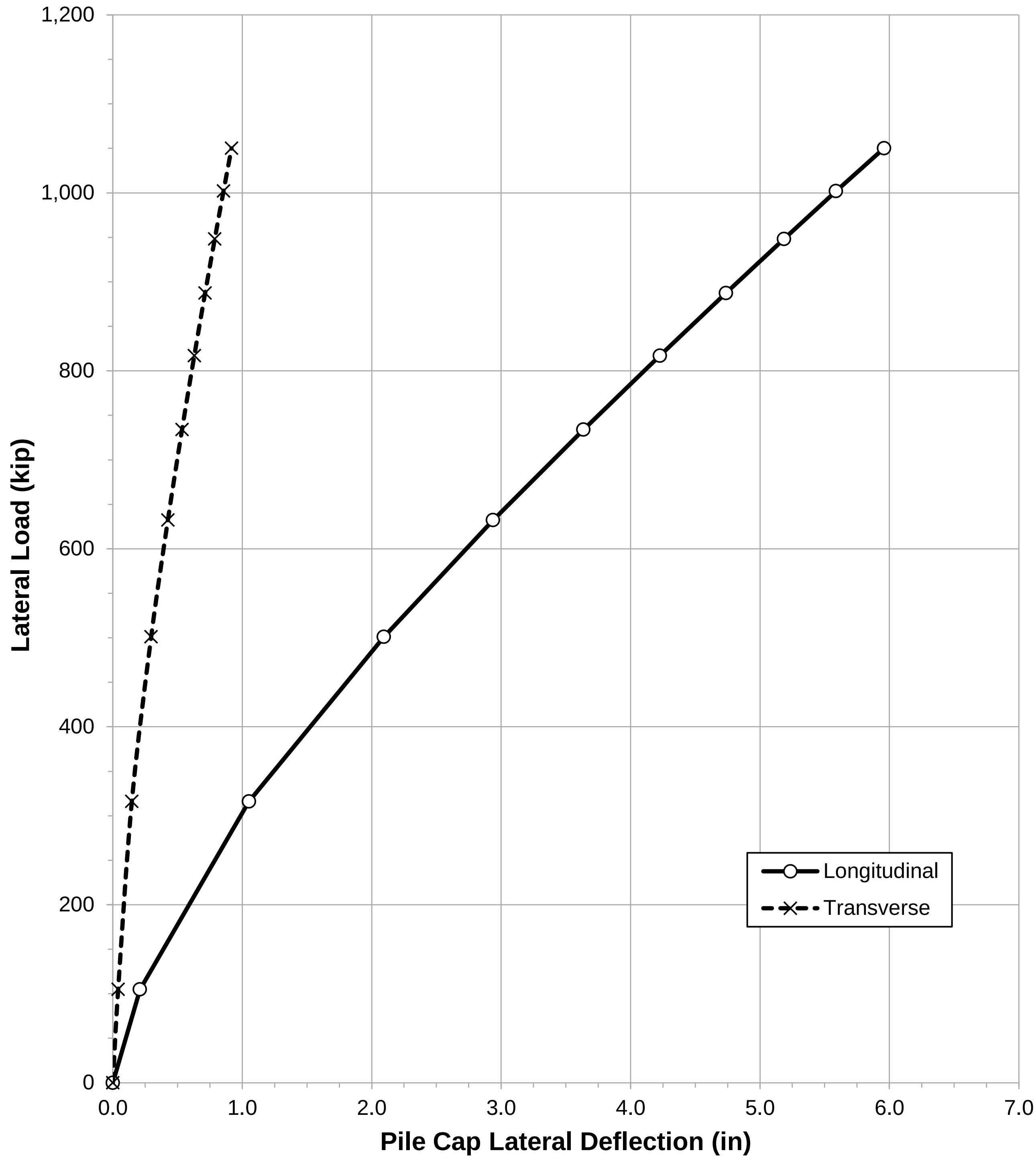
FIG. F45



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 23

August 2017 | 24-1-04065-005

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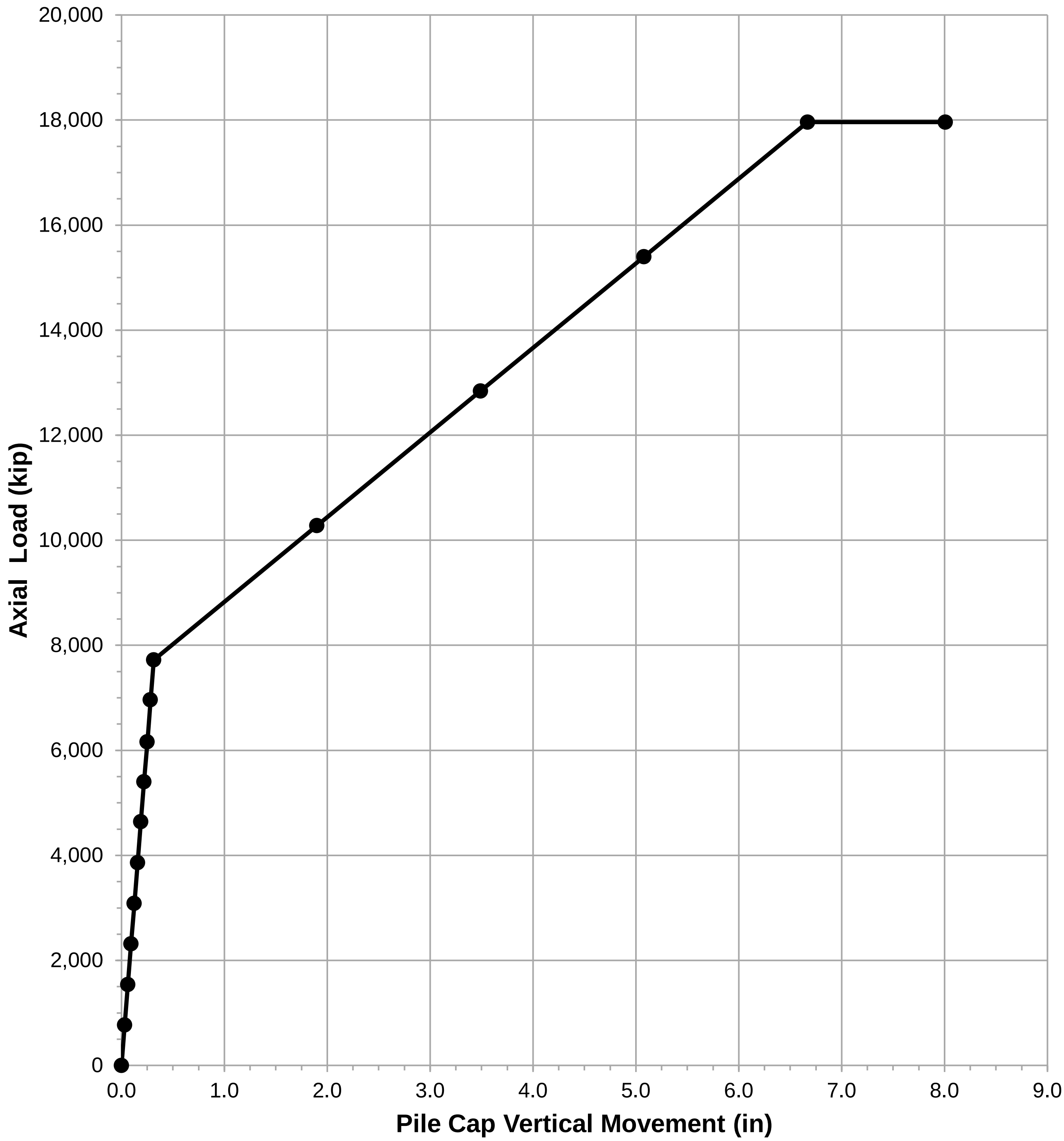
FIG. F46



NOTES
1. See Table 11 for details of each drilled shaft group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 24

August 2017 24-1-04065-005

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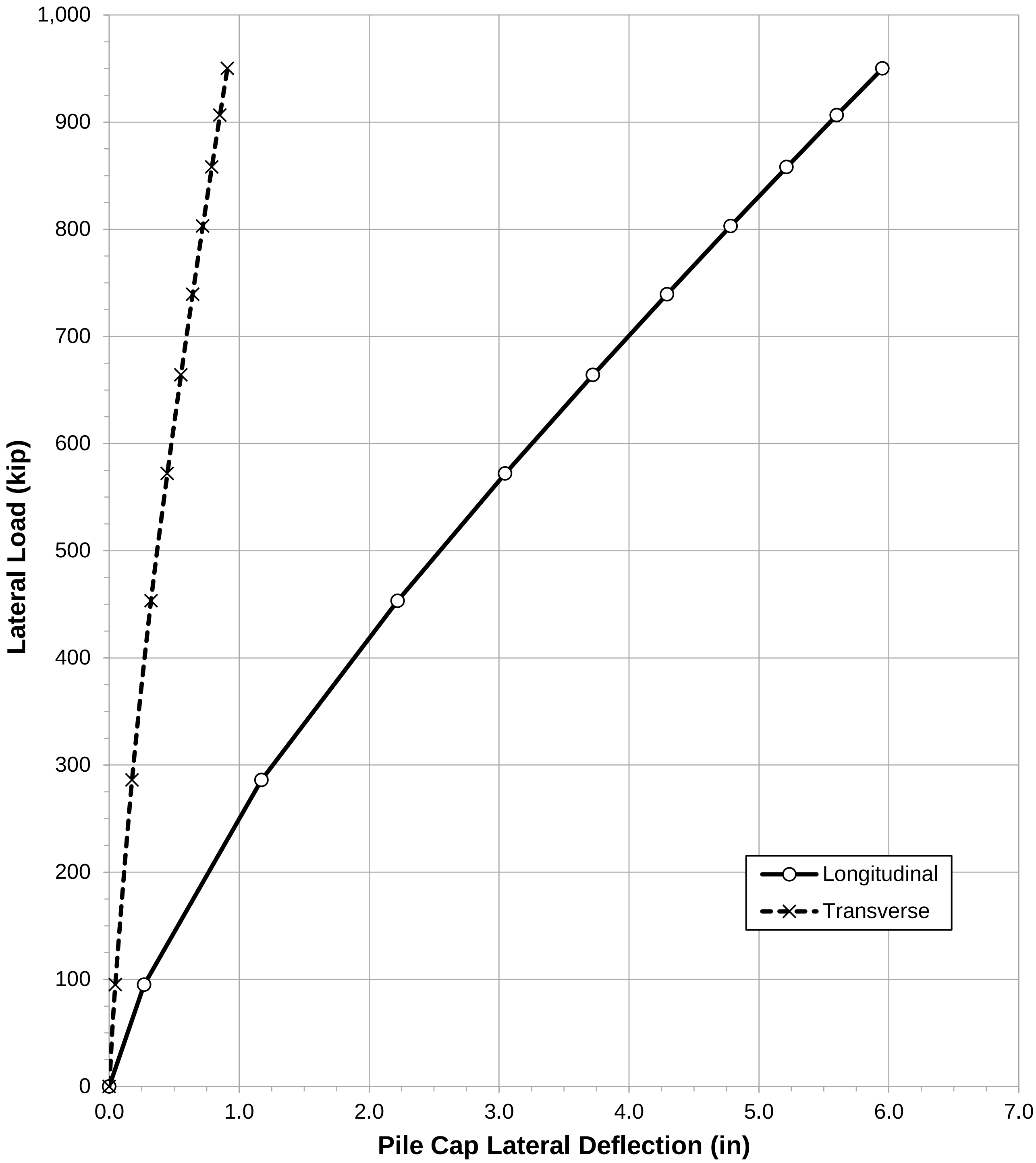
FIG. F47



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 24

August 2017 | 24-1-04065-005

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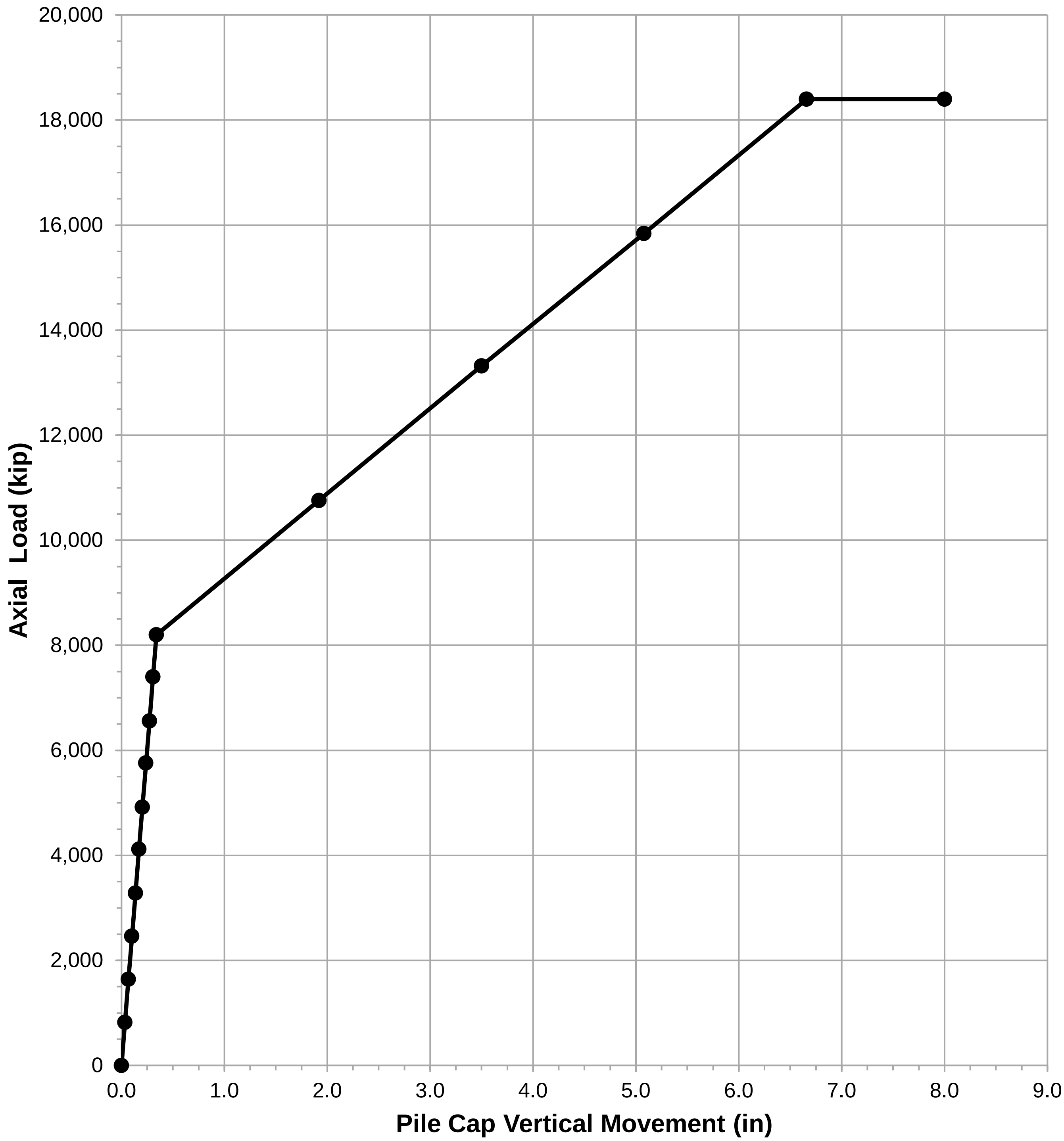
FIG. F48



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 25

August 2017 | 24-1-04065-005

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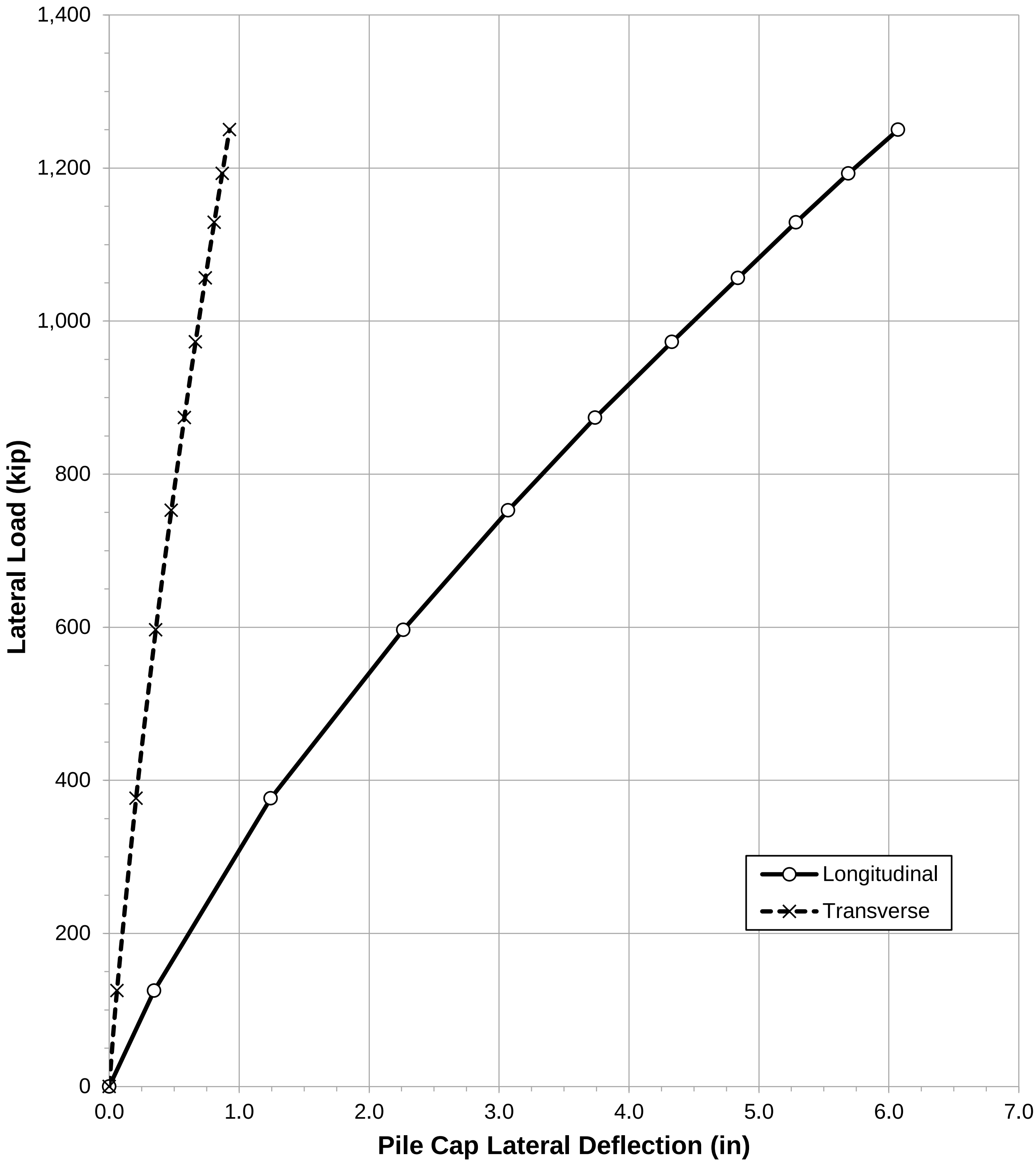
FIG. F49



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

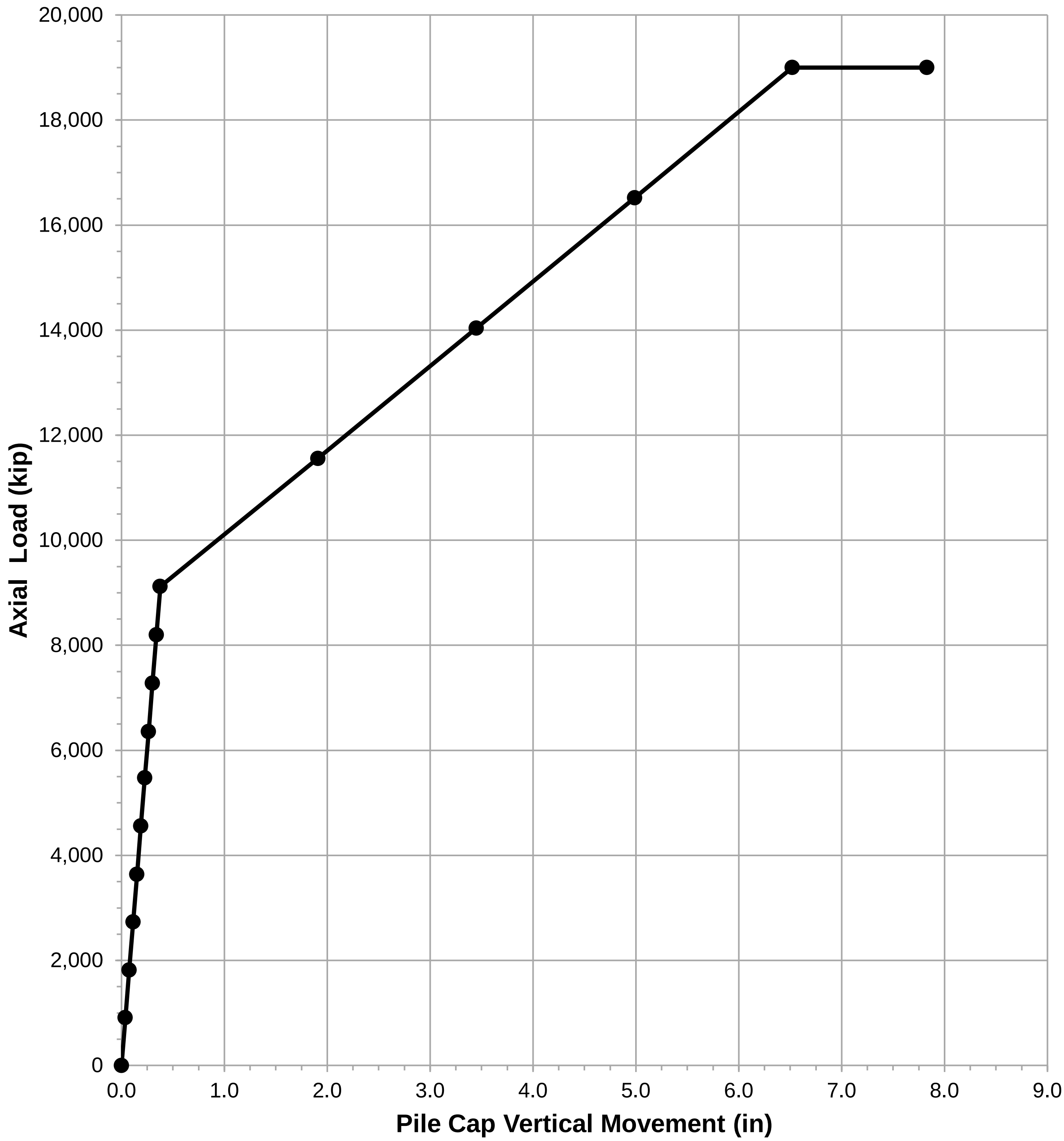
Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 25

August 2017 | 24-1-04065-005

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F50



NOTES
1. See Table 11 for details of each drilled shaft group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 26

August 2017 24-1-04065-005

SHANNON & WILSON, INC.
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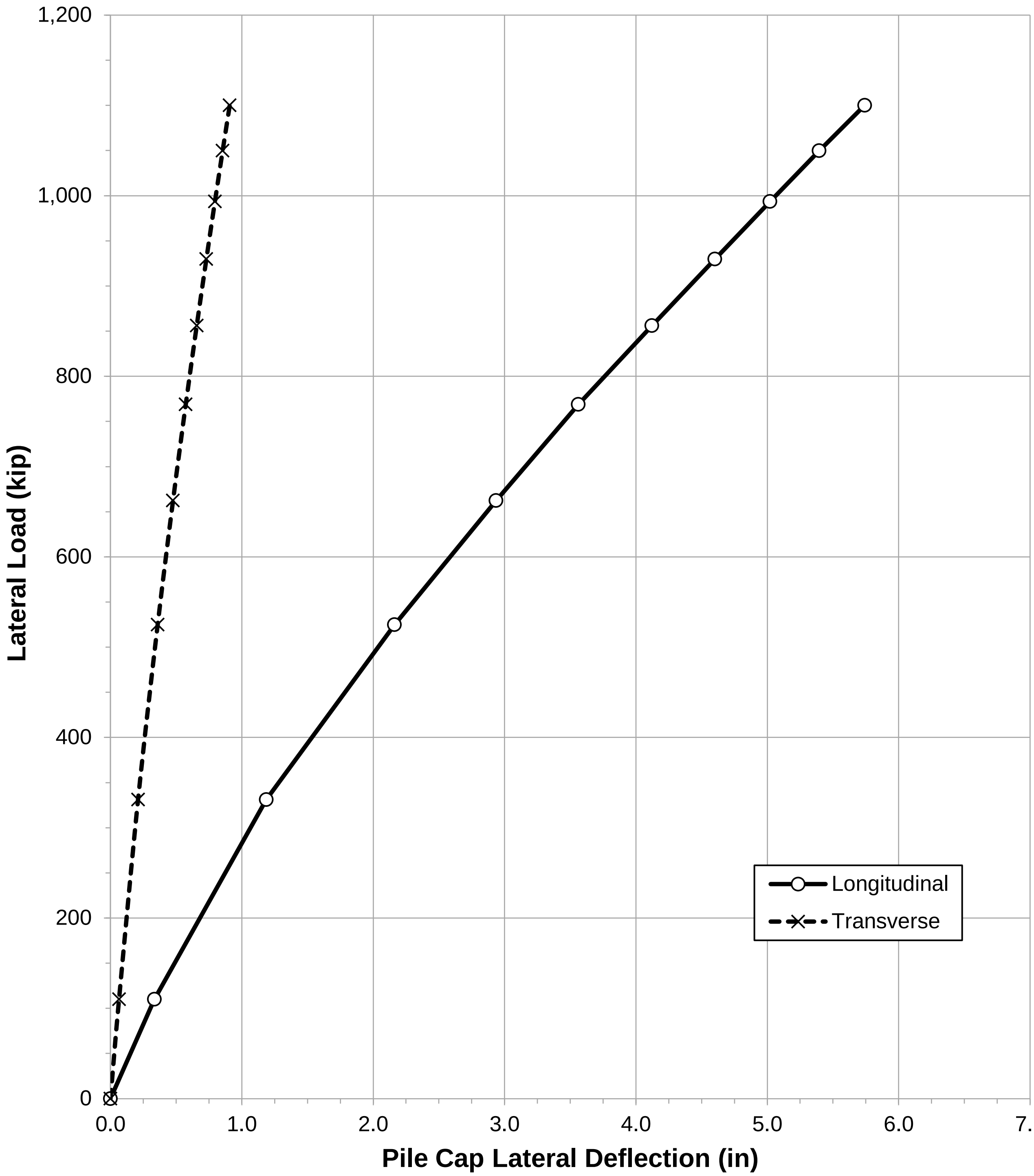
FIG. F51



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 26

August 2017 | 24-1-04065-005

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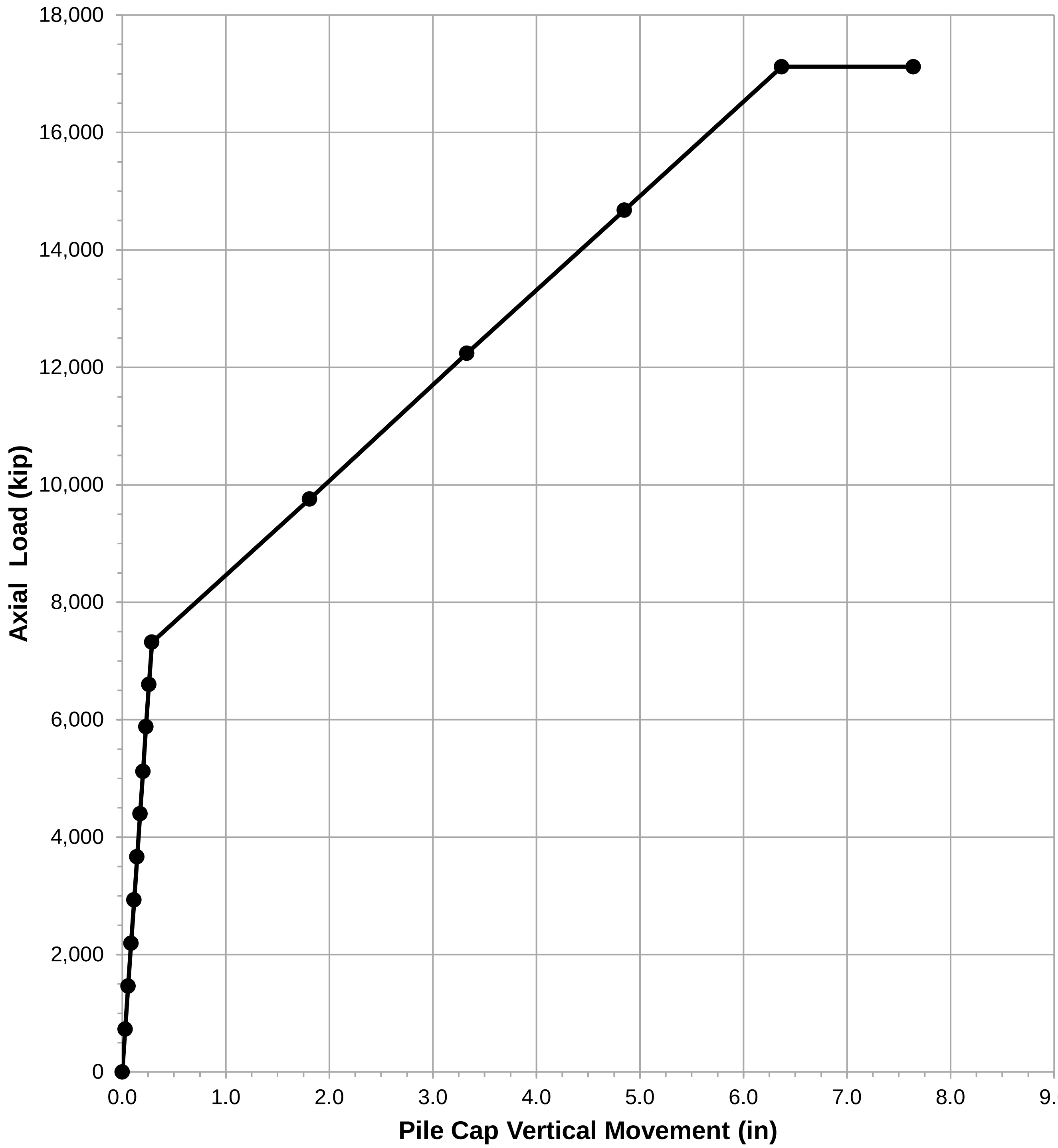
FIG. F52



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.

Burnside Bridge Seismic Feasibility Study
Portland, Oregon

DRILLED SHAFT GROUP AXIAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 27

August 2017 | 24-1-04065-005

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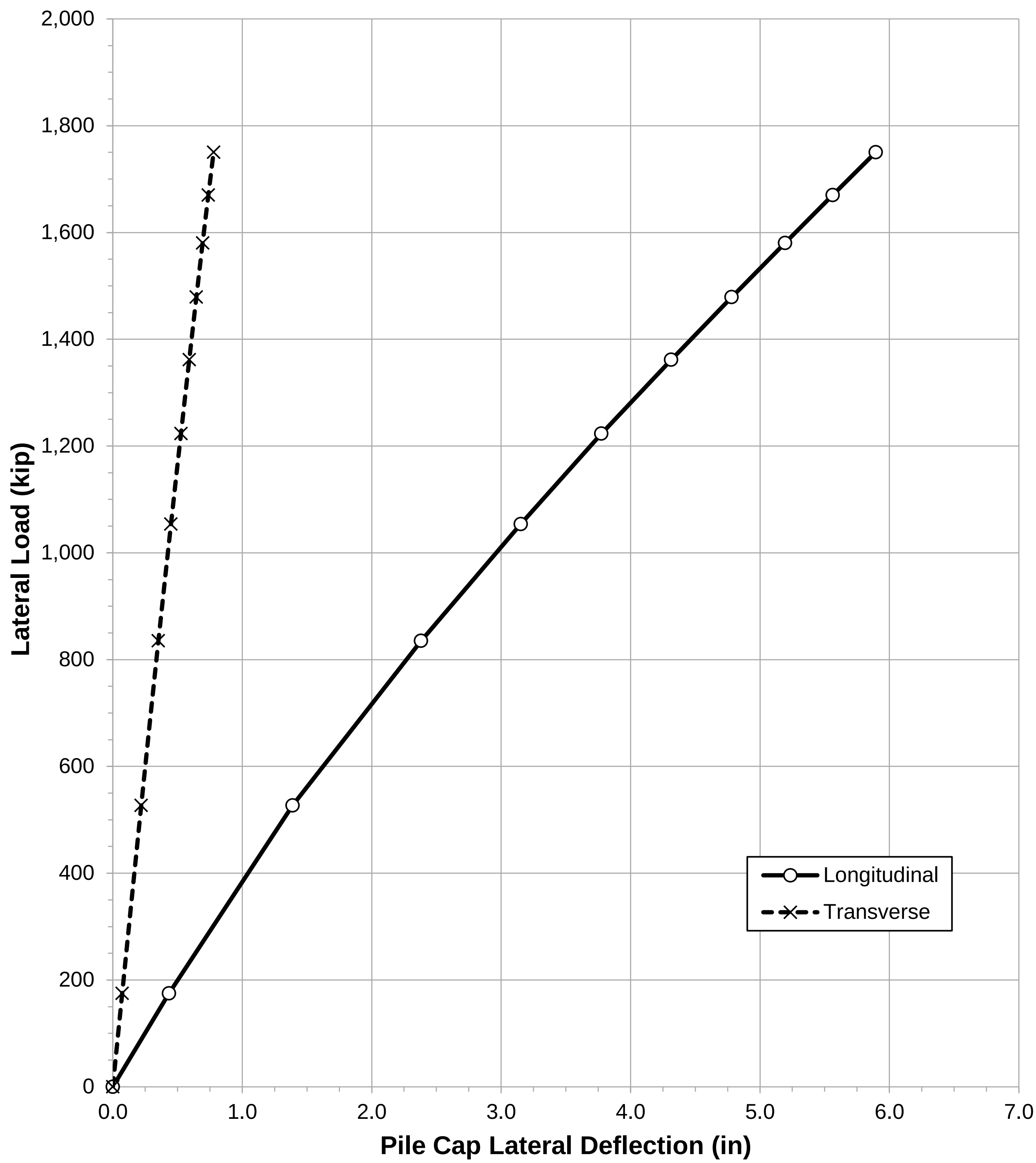
FIG. F53



NOTES

1. See Table 11 for details of each drilled shaft group evaluated.
2. Analysis assumes a fixed shaft head condition and passive resistance from pile cap is not included. See Table 10 for recommended passive resistance on pile caps.

Burnside Bridge Seismic Feasibility Study Portland, Oregon	
DRILLED SHAFT GROUP LATERAL LOAD-DISPLACEMENT RELATIONSHIP AT BENT 27	
August 2017	24-1-04065-005
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. F54**

APPENDIX G

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

Date: September 2017
To: Steve Drahota, PE
HDR

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 which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

Final boring logs developed by the consultant are based 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.

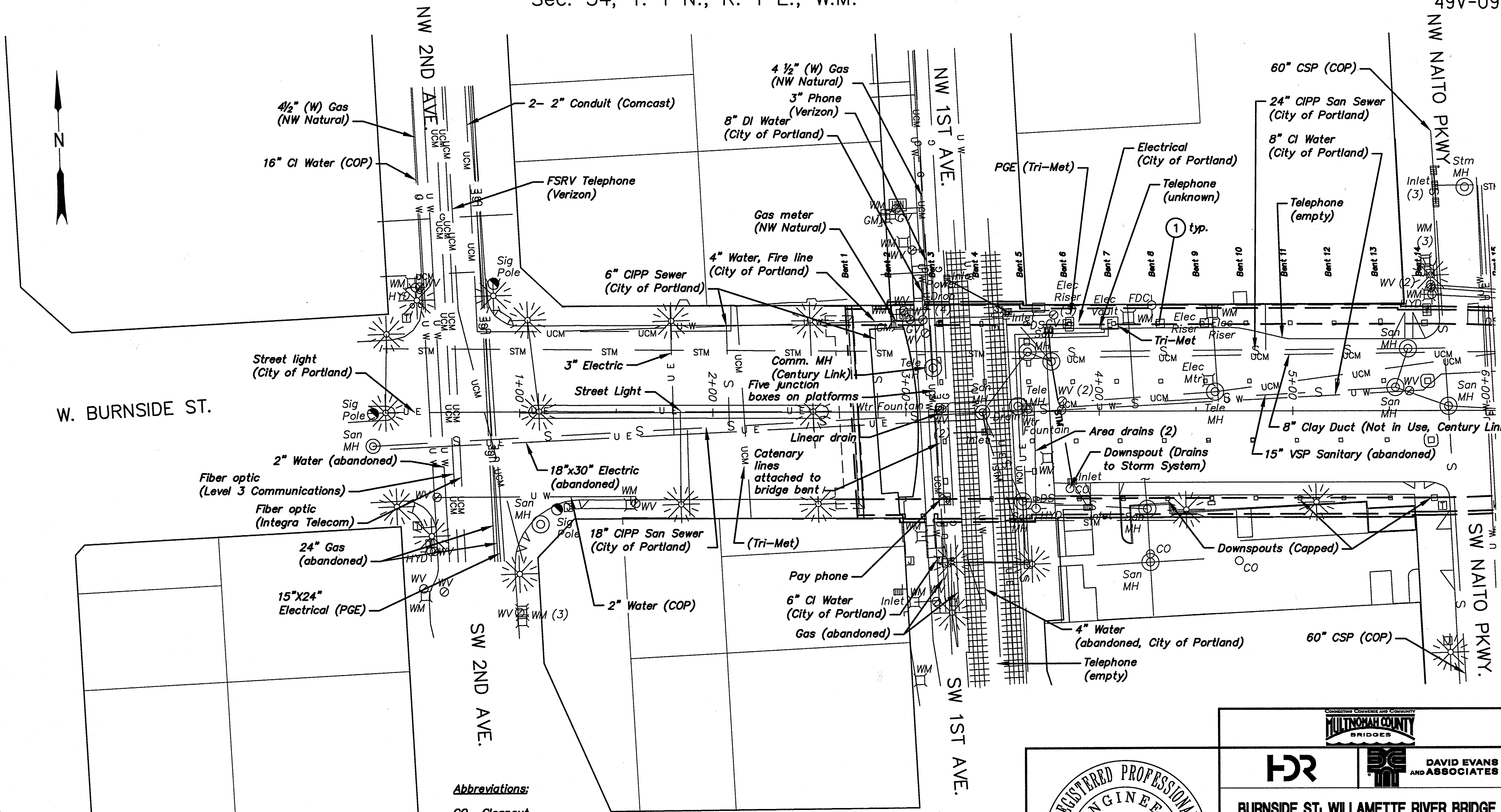
To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors 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

Appendix D. EQRB Seismic Site Utilities



Abbreviations:

CO Cleanout
COP City of Portland
DS Down Spout
GM Gas Meter
GV Gas Vault
HYD Fire Hydrant
WM Water Meter
WV Water Vault

Utility Construction Notes:

1 Bent 1 thru Pier 1 – Remove and reinstall conduit attached to columns for spall repair.

REGISTERED PROFESSIONAL ENGINEER
74896PE
OREGON
JULY 13, 2004
BRENDAN E. LEBLANC
EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES
HDR
DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST. WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
Designed By – Brent Carney
Drafted By – Heather Gonsior

UTILITIES
GROUND

SHEET NO. 3A

C:\pwworking\sea\d1684534\18383_UT_Ground.dwg hgonsior Oct 31, 2016 2:47pm

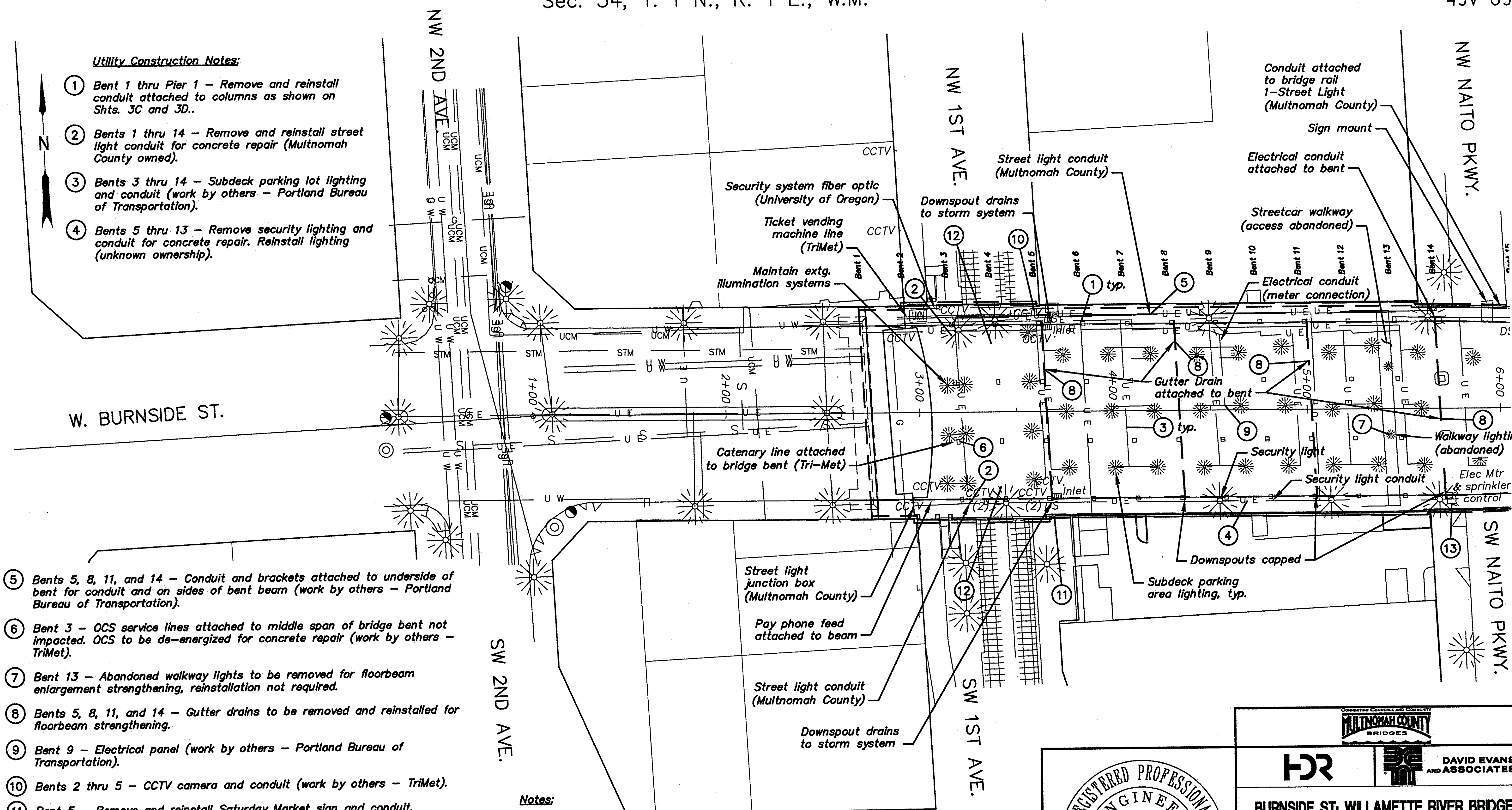
Utility Construction Notes:

(1) Bent 1 thru Pier 1 – Remove and reinstall conduit attached to columns as shown on Shts. 3C and 3D..

(2) Bents 1 thru 14 – Remove and reinstall street light conduit for concrete repair (Multnomah County owned).

(3) Bents 3 thru 14 – Subdeck parking lot lighting and conduit (work by others – Portland Bureau of Transportation).

(4) Bents 5 thru 13 – Remove security lighting and conduit for concrete repair. Reinstall lighting (unknown ownership).



(5) Bents 5, 8, 11, and 14 – Conduit and brackets attached to underside of bent for conduit and on sides of bent beam (work by others – Portland Bureau of Transportation).

(6) Bent 3 – OCS service lines attached to middle span of bridge bent not impacted. OCS to be de-energized for concrete repair (work by others – TriMet).

(7) Bent 13 – Abandoned walkway lights to be removed for floorbeam enlargement strengthening, reinstallation not required.

(8) Bents 5, 8, 11, and 14 – Gutter drains to be removed and reinstalled for floorbeam strengthening.

(9) Bent 9 – Electrical panel (work by others – Portland Bureau of Transportation).

(10) Bents 2 thru 5 – CCTV camera and conduit (work by others – TriMet).

(11) Bent 5 – Remove and reinstall Saturday Market sign and conduit.

(12) Bent 4 – Bridge mounted safety frames for overhead contact system (work by others – TriMet).

(13) Remove and reinstall electric meter and sprinkler controller for concrete repair.

Notes:

1. See Sht. 3A for surface utility notes.
2. All utilities not identified in the Utility Construction Notes shall be protected in place.
3. All conduit attached to the bridge to be rigid galvanized steel.
4. Notify TriMet, PBOT, PP&R per subsection 00150.50(f) of the Special Provisions.
5. All removal and reinstallation work will be limited to what is necessary to accomplish the bridge repair and strengthening work.

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DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST: WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
Designed By – Brent Carney
Drafted By – Heather Gonsior

UTILITIES
BRIDGE

SHEET NO. 3B

Location	Floorbeam Strengthening Locations	Floorbeam Attachments: Description	Floorbeam Attachments: Treatment	Column Attachments: Description	Column Attachments: Treatment
Bent 1	None	UO building systems including gas	Protect in place	UO building systems including gas	Protect in place
Bent 2	None	UO building roof and fascia	Protect in place	Piping and meter on Column 1	Protect in place
		Conduit on east face	Protect in place	N/A	N/A
Bent 3	None	Lighting conduits on west face entire length	Protect in place	TriMet electrical drop from OCS on Column 3	Protect in place
		Lighting conduits on east face entire length	Protect in place	Pay phone conduits on Column 4	Remove and reinstall for concrete repair
		Lighting conduit crossing under floor beam in one location, Span 2	Protect in place	4– conduits on Column 1	Portland Bureau of Transportation will remove during construction
		2– TriMet CCTV conduits on west face entire length	TriMet will perform work	N/A	N/A
Bent 4	None	TriMet OCS	Protect in place	N/A	N/A
		Lighting conduit crossing under floorbeam at Span 2	Portland Bureau of Transportation will perform work	N/A	N/A
Bent 5	Spans 2 thru 4	Saturday Market sign	Remove and reinstall for concrete repair and floorbeam strengthening	Storm drain pipe from catch basins at Columns 1 and 4	Protect in place
		Saturday Market sign conduits on east face of Span 2	Remove and reinstall for concrete repair and floorbeam strengthening	2– TriMet conduits on Column 1	Protect conduits in place. Remove and reinstall support bracket.
		Lighting conduits on west face entire length	Portland Bureau of Transportation will perform work	Downspout from gutter on Column 1	Protect in place
		Lighting conduit crossing under floorbeam in one location	Portland Bureau of Transportation will perform work	N/A	N/A
		Lighting conduit on east face of Span 2	Portland Bureau of Transportation will perform work	N/A	N/A
		TriMet CCTV conduit on west face, entire length	TriMet will perform work	N/A	N/A
		2– TriMet CCTV conduit crossings	TriMet will perform work	N/A	N/A
		Seismic retrofit	Protect in place	N/A	N/A
		Gutter on east face	Remove and reinstall for concrete repair and floorbeam strengthening	N/A	N/A
Bent 6	Spans 2 and 4	Lighting conduit crossing under floorbeam in one location, Span 2	Portland Bureau of Transportation will perform work	Conduit and electrical cabinets on Column 1	Protect in place
Bent 7	None	Lighting conduit on west face, entire length	Portland Bureau of Transportation will perform work	Conduit and electrical cabinet on Column 1	Portland Bureau of Transportation will perform work
		Lighting conduit on bottom face and east face at two locations in Span 2, and one location each in Spans 3 and 4	Portland Bureau of Transportation will perform work	N/A	N/A
Bent 8	None	Lighting conduit crossing under floorbeam in one location, Span 2	Portland Bureau of Transportation will perform work	Downspout from gutter on Column 4	Protect in place
		Seismic retrofit	Protect in place	N/A	N/A
		Gutter on east face	Remove and reinstall for concrete repair	N/A	N/A

Note:
Column numbering convention is north to south; Column 1 is the northern column. Span 1 is the northern overhang; Span 2 is between Columns 1 and 2.

REGISTERED PROFESSIONAL ENGINEER
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JULY 13, 2004
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MULTNOMAH COUNTY BRIDGES
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DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST: WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
Designed By – Brent Carney
Drafted By – Heather Gonsior

UTILITIES
BRIDGE

SHEET NO. 3C

Location	Floorbeam Strengthening Locations	Floorbeam Attachments		Column Attachments	
		Description	Treatment	Description	Treatment
Bent 9	Spans 2, 3, and 4	Lighting conduit on west face, entire length	Portland Bureau of Transportation will perform work	Large conduits on Columns 1 and 2	Portland Bureau of Transportation will perform work
		Conduit on bottom face and east face at two locations in Span 2, and one location each in Spans 3 and 4	Portland Bureau of Transportation will perform work	Cabinets on Column 1	Protect in place
		Large conduit on east face in Span 2 and large electrical box at Column 2	Portland Bureau of Transportation will perform work	Electrical meter at base of Column 2	Protect in place
Bent 10	Spans 2, 3, and 4	Lighting conduit crossing under floorbeam in one location, Span 2	Portland Bureau of Transportation will perform work	N/A	N/A
		Small conduit (light sensor?) on east face, entire length	Portland Bureau of Transportation will perform work	N/A	N/A
Bent 11	Spans 2 and 4	Lighting conduit on west face, entire length	Portland Bureau of Transportation will perform work	Downspout from gutter on Column 4	Protect in place
		Lighting conduit crossing under floorbeam in one location, Span 2	Portland Bureau of Transportation will perform work	N/A	N/A
		Seismic retrofit	Protect in place	N/A	N/A
		Gutter on east face	Remove and reinstall for concrete repair and floorbeam strengthening	N/A	N/A
Bent 12	None	Lighting conduit on west face, entire length	Portland Bureau of Transportation will perform work	N/A	N/A
		Lighting conduit crossing under floorbeam in two locations in Span 2, and one location each in Spans 3 and 4	Portland Bureau of Transportation will perform work	N/A	N/A
Bent 13	Spans 2, 3, and 4	Lighting conduit on west face, bottom, and east face of Span 2	Portland Bureau of Transportation will perform work	Small conduit on Columns 1 and 4	Portland Bureau of Transportation will perform work
		Lighting conduit on east face, entire length	Remove and reinstall for concrete repair and floorbeam strengthening	Downspout from walkway on Column 2	Protect in place
Bent 14	Spans 2, 3, and 4	Gutter on west face	Remove and reinstall for concrete repair and floorbeam strengthening	Downspout from gutter on Column 4	Remove and reinstall for concrete repair
		Conduit on west face, bottom, and east face of Span 4	Portland Bureau of Transportation will perform work	Electrical meter at base of Column 4	Remove and reinstall for concrete repair
		Small conduit along east face of Span 4	Remove and reinstall for concrete repair and floorbeam strengthening	Sprinkler control at base of Column 4	Remove and reinstall for concrete repair
		Seismic retrofit	Protect in place	N/A	N/A

Notes:

1. Column numbering convention is north to south; Column 1 is the northern column. Span 1 is the northern overhang; Span 2 is between Columns 1 and 2.

2. The lighting fixture between Bent 11 and Bent 12 will be removed during construction by the Portland Bureau of Transportation.

REGISTERED PROFESSIONAL ENGINEER
74896PE
OREGON
JULY 13, 2004
BRENDAN E. LEBLANC
EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES

HDR

DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST. WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
Designed By – Brent Carney
Drafted By – Heather Gonsior

UTILITIES
BRIDGE

SHEET NO. 3D

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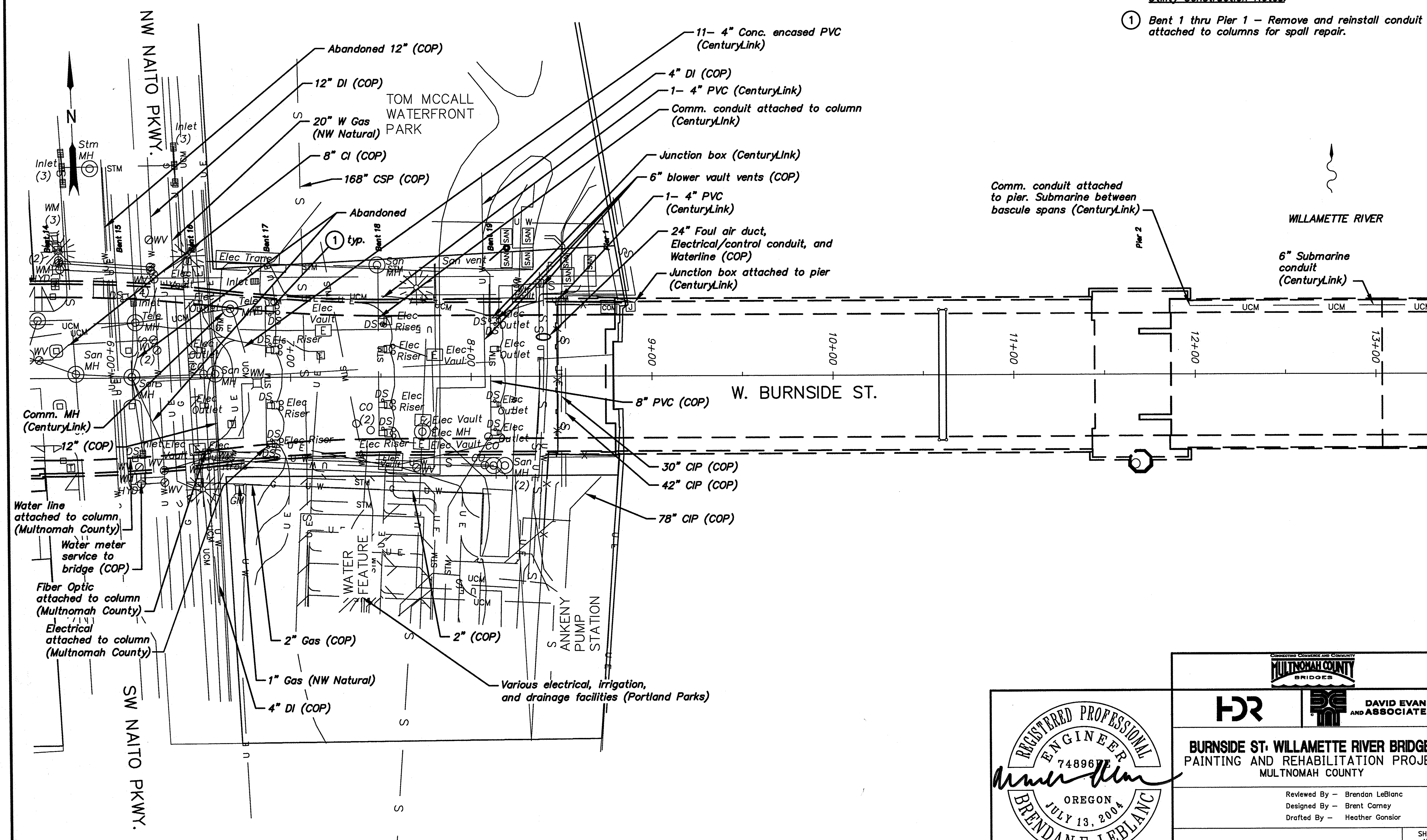
Sec. 34, T. 1 N., R. 1 E., W.M.

49V-098

Utility Construction Notes:

(1) *Bent 1 thru Pier 1 — Remove and reinstall conduit attached to columns for spall repair.*



REGISTERED PROFESSIONAL ENGINEER 74896PE OREGON JULY 13, 2004 BRENDAN E. LEBLANC

EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES

HDR

DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST: WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
Designed By – Brent Carney
Drafted By – Heather Gonsior

UTILITIES
GROUND

SHEET NO. 4A

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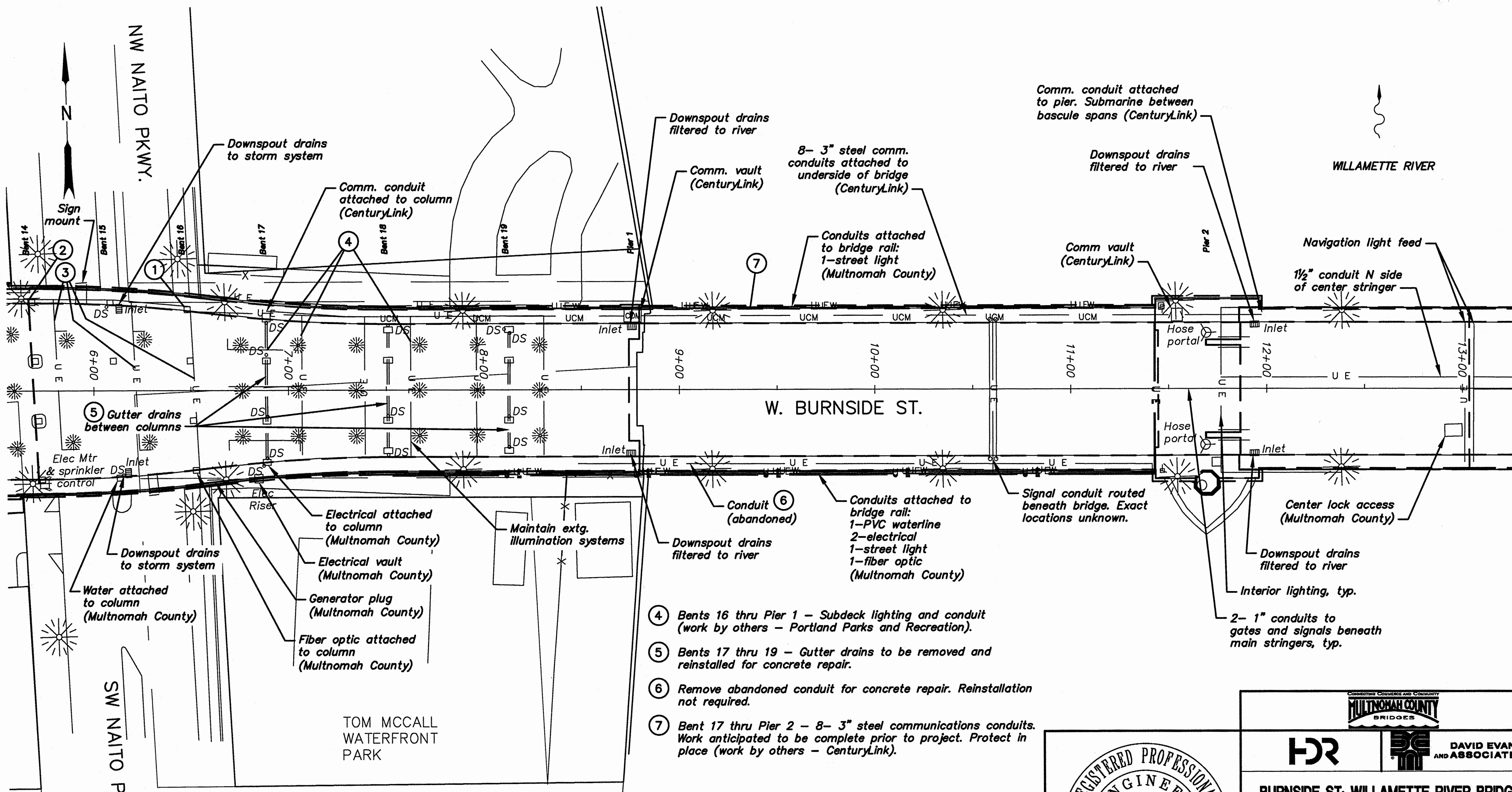

(4) Bents 16 thru Pier 1 – Subdeck lighting and conduit (work by others – Portland Parks and Recreation).

(5) Bents 17 thru 19 – Gutter drains to be removed and reinstalled for concrete repair.

(6) Remove abandoned conduit for concrete repair. Reinstallation not required.

(7) Bent 17 thru Pier 2 – 8– 3" steel communications conduits. Work anticipated to be complete prior to project. Protect in place (work by others – CenturyLink).

Utility Construction Notes:

(1) Bent 1 thru Pier 1 – Remove and reinstall conduit attached to columns as shown on Shts. 3C and 3D.

(2) Bents 1 thru 14 – Remove and reinstall street light conduit for concrete repair (Multnomah County owned).

(3) Bents 14 thru 16 – Subdeck roadway lighting and conduit (work by others – Portland Bureau of Transportation).

REGISTERED PROFESSIONAL ENGINEER
74896PE
OREGON
JULY 13, 2004
BRENDAN E. LEBLANC
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MULTNOMAH COUNTY BRIDGES

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DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST: WILLAMETTE RIVER BRIDGE
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MULTNOMAH COUNTY

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UTILITIES
BRIDGE

SHEET NO. 4B

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Location	Floorbeam Strengthening Locations	Floorbeam Attachments		Column Attachments	
		Description	Treatment	Description	Treatment
Bent 15	None	N/A	N/A	Storm drain pipe from catch basins on Columns 1 and 4	Protect in place
		N/A	N/A	Waterline on Column 4	Protect in place
		N/A	N/A	Abandoned pipe, north side of Column 4	Remove
Bent 16	None	Seismic retrofit	Protect in place	Conduit on Column 1	Portland Bureau of Transportation will perform work
		Lighting conduit on east face, entire length	Portland Bureau of Transportation will perform work	Electrical receptacles at base of all columns	Protect in place
		N/A	N/A	Fiber optic conduit on south side of Column 4	Protect in place
Bent 17	None	Seismic retrofit	Protect in place	Downspout from gutter on Columns 1, 3, and 4	Protect in place
		Gutters on bottom	Remove and reinstall for concrete repair	Electrical cabinets at base of all columns	Protect in place
		Lighting conduit crossing under floorbeam at Span 4	Protect in place	Lighting conduit on Column 1	Portland Parks & Recreation will perform work
		N/A	N/A	8– CenturyLink conduits inside Column 1 emerging just below floorbeam	Protect in place
		N/A	N/A	Small conduit stubbed near floorbeam on Column 1	Protect in place
		N/A	N/A	3– electrical conduits on south side of Column 4	Protect in place
Span 17–1	None	Conduit and light on bottom of each Spans 2, 3, and 4	Protect in place	N/A	N/A
Span 17–2	None	N/A	N/A	N/A	N/A
Span 17–3	None	Conduit and light on bottom of each Spans 2, 3, and 4	Protect in place	N/A	N/A
Bent 18	None	Seismic retrofit	Protect in place	Downspout from gutter on Columns 1, 3, and 4	Protect in place
		Gutters on bottom	Remove and reinstall for concrete repair	Electrical cabinets at base of Columns 2, 3, and 4	Protect in place
		Conduit on bottom face at one location each in Spans 2, 3, and 4	Protect in place	N/A	N/A
Span 18–1	None	Conduit and light on bottom of each Spans 2, 3, and 4	Protect in place	N/A	N/A
Span 18–2	None	N/A	N/A	N/A	N/A
Span 18–3	None	Conduit and light on bottom of each Spans 2, 3, and 4	Protect in place	N/A	N/A
Bent 19	None	Seismic retrofit	Protect in place	Downspout from gutter on Columns 1, 3, and 4	Protect in place
		Gutters on bottom	Remove and reinstall for concrete repair	Electrical receptacles at base of all columns	Protect in place
		N/A	N/A	Odor control vents at east and west sides of Column 1	Protect in place
Span 19–1	None	Conduit and light on bottom of each Spans 2, 3, and 4	Protect in place	N/A	N/A
Span 19–2	None	N/A	N/A	N/A	N/A
Span 19–3	None	Conduit and light on bottom of each Spans 2, 3, and 4	Protect in place	N/A	N/A

Location	Description of utility under sidewalk	Treatment
Bent 17 to Pier 2	8– CenturyLink conduits, north side, resting on floorbeams	CenturyLink will remove and reinstall on hangers beneath floorbeams. Work anticipated to be complete prior to project. Protect in place.

Note:
Column numbering convention is north to south; Column 1 is the northern column. Span 1 is the northern overhang; Span 2 is between Columns 1 and 2.

REGISTERED PROFESSIONAL ENGINEER
74896PE
OREGON
JULY 13, 2004
BRENDAN E. LEBLANC
EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES

HDR | DAVID EVANS AND ASSOCIATES INC.

BURNSIDE ST. WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
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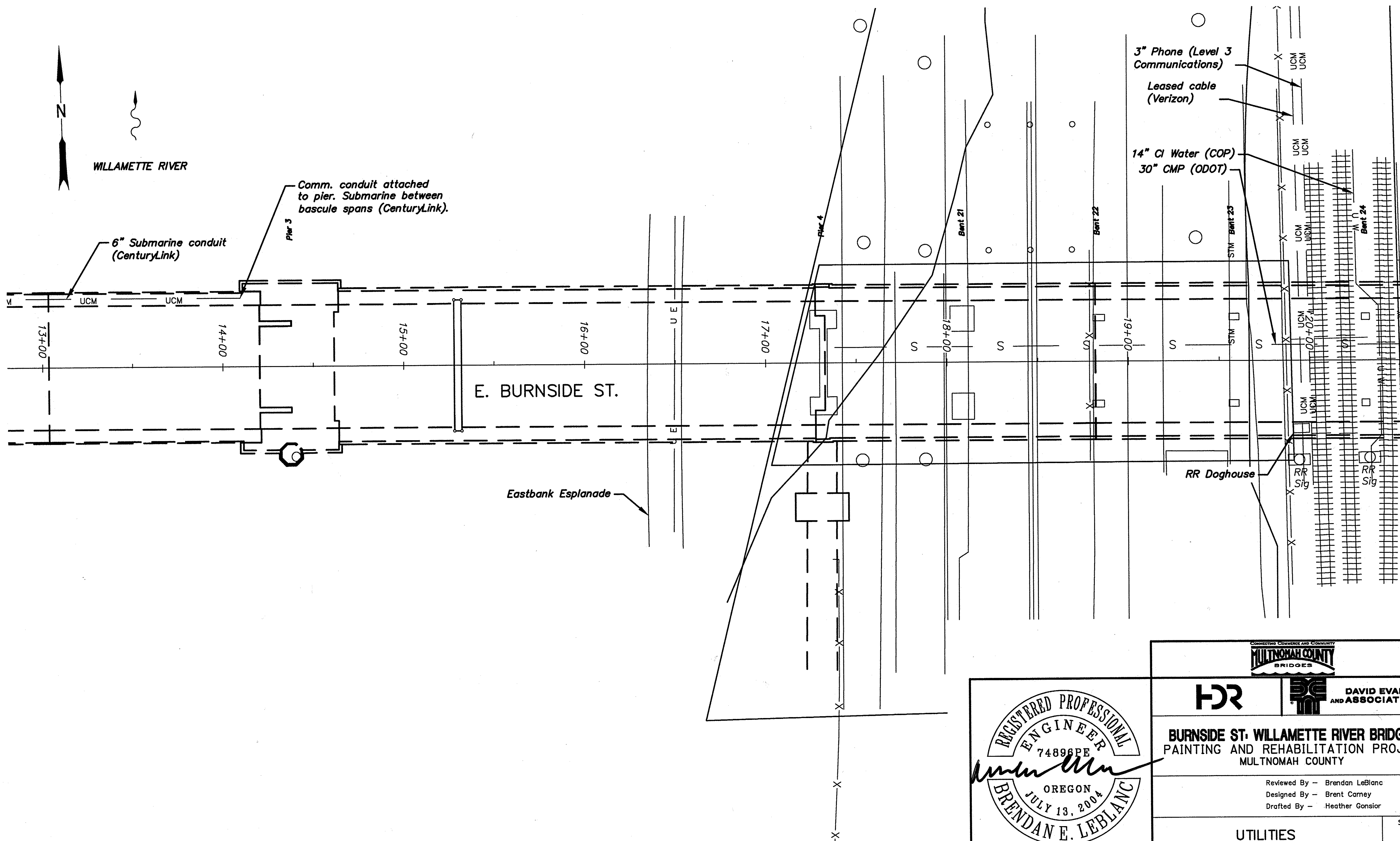
Drafted By – Heather Gonsior

UTILITIES
BRIDGE

SHEET NO. 4C



MULTNOMAH COUNTY BRIDGES

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REGISTERED PROFESSIONAL ENGINEER
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OREGON
JULY 13, 2004
BRENDAN E. LEBLANC

EXPIRES: DEC. 31, 2016

BURNSIDE ST: WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

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Drafted By – Heather Gonsior

UTILITIES
GROUND

SHEET NO.
5A

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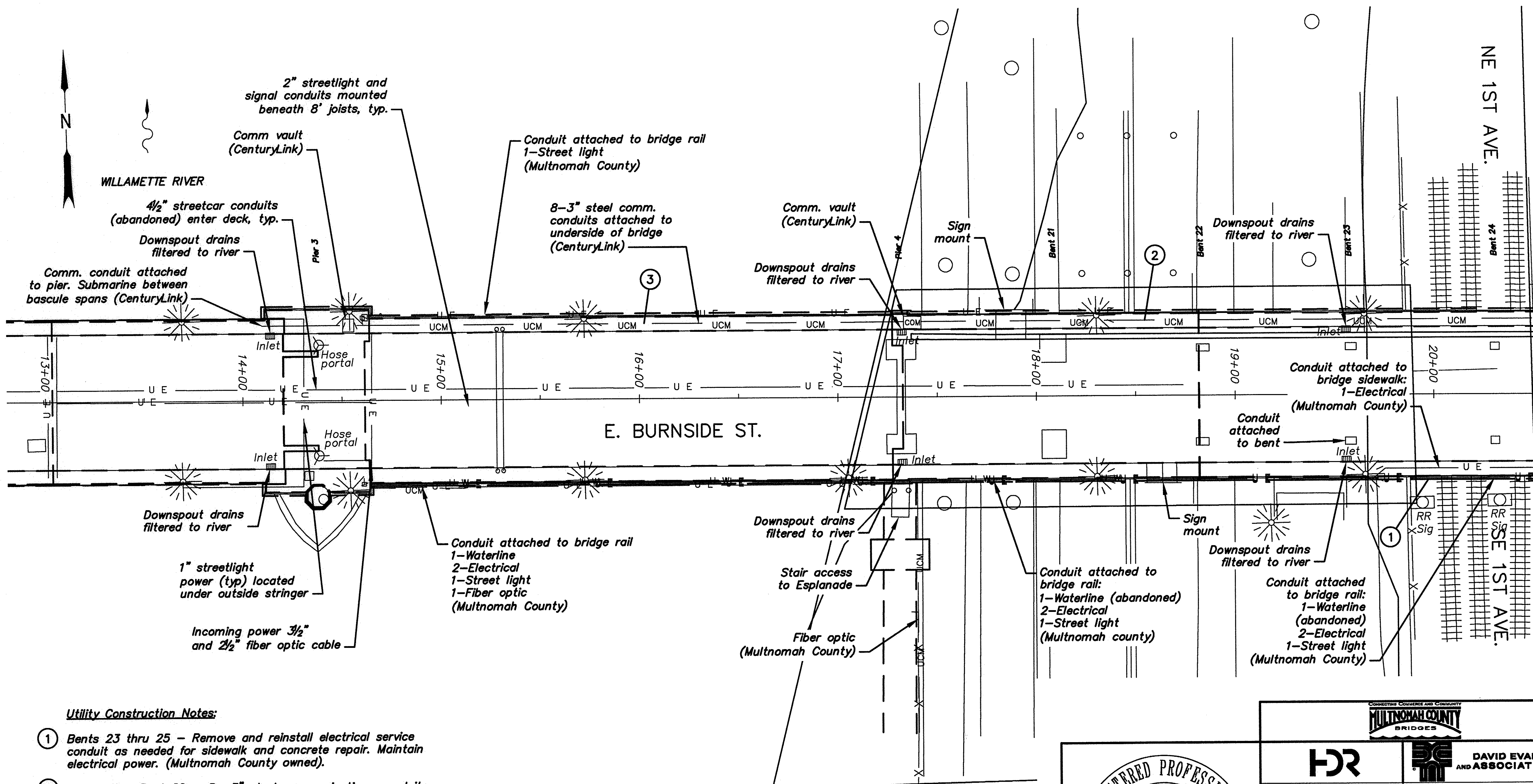
Sec. 34, T. 1 N., R. 1 E., W.M.

49V-098



Utility Construction Notes:

1. Bents 23 thru 25 – Remove and reinstall electrical service conduit as needed for sidewalk and concrete repair. Maintain electrical power. (Multnomah County owned).
2. Pier 4 thru Bent 29 – 8– 3" steel communications conduits attached to bridge bents with sufficient clearance, protect in place (CenturyLink owned).
3. Piers 3 & 4 – 8– 3" steel communications conduits (work by others – CenturyLink).

REGISTERED PROFESSIONAL ENGINEER 74896PE OREGON JULY 13, 2004 BRENDAN E. LEBLANC

EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES

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BURNSIDE ST. WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

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UTILITIES BRIDGE

SHEET NO. 5B

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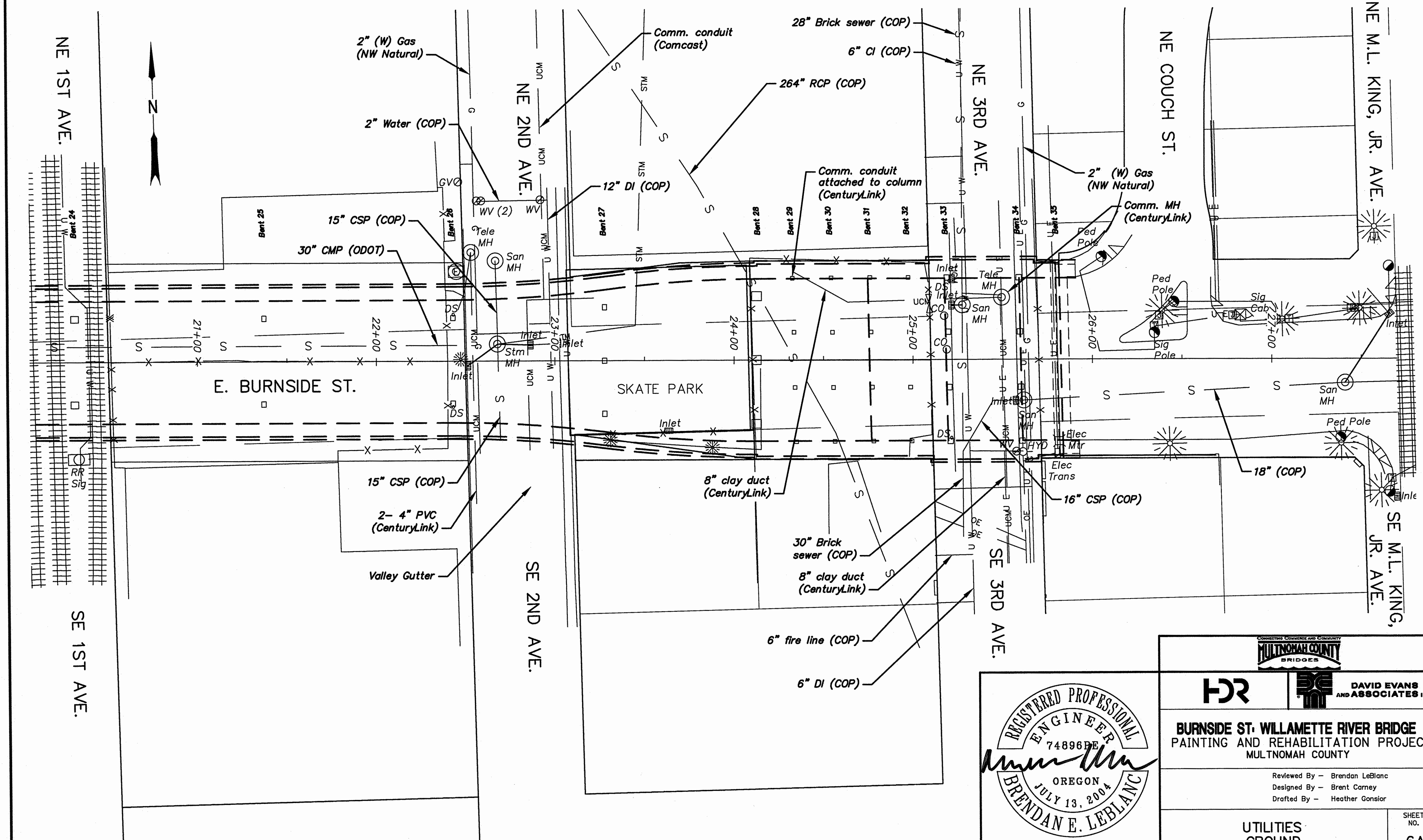
Sec. 34, T. 1 N., R. 1 E., W.M.

49V-098



REGISTERED PROFESSIONAL ENGINEER
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OREGON
JULY 13, 2004
BRENDAN E. LEBLANC
EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES

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PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

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UTILITIES GROUND

SHEET NO. 6A

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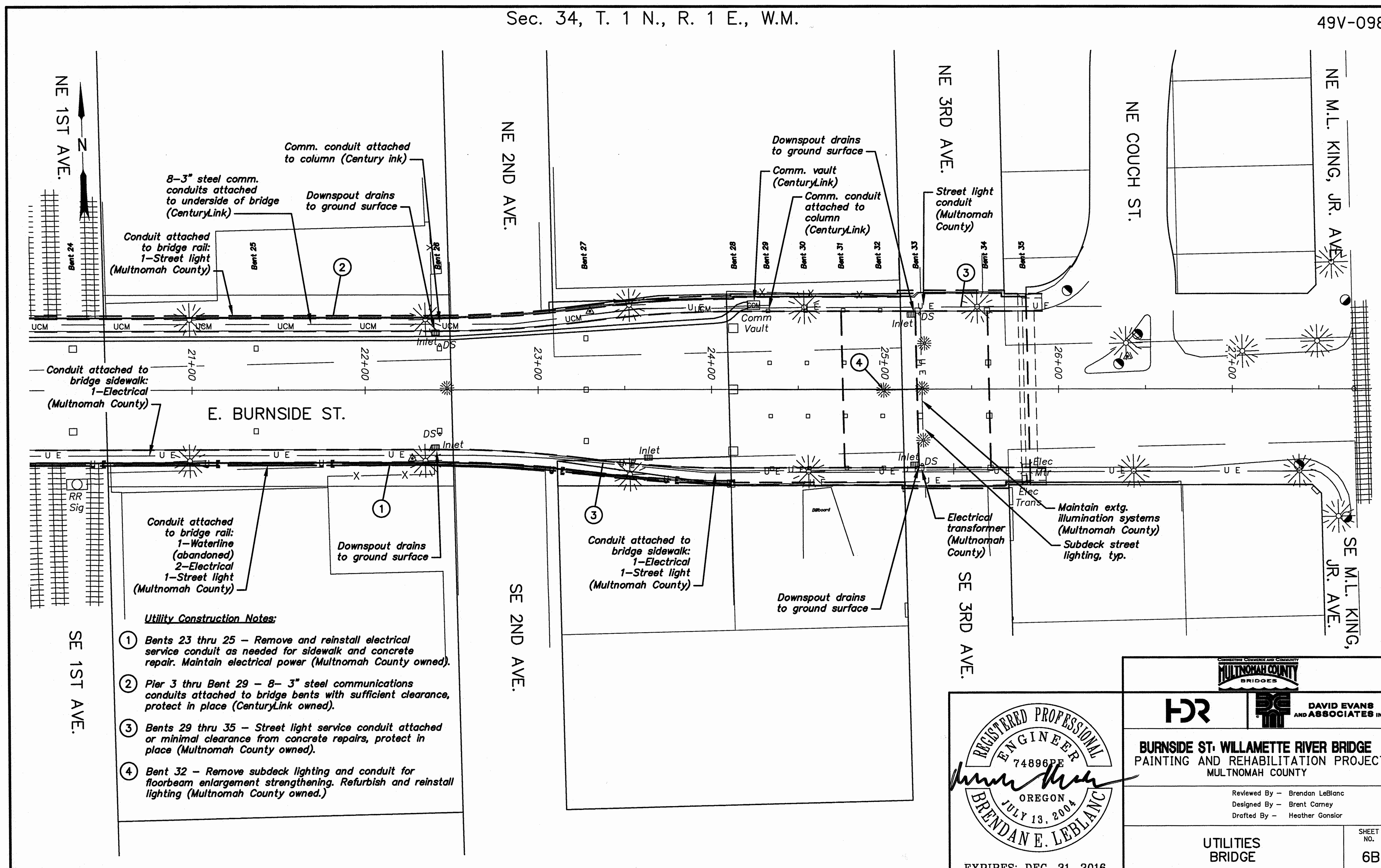

Utility Construction Notes:

1. Bents 23 thru 25 – Remove and reinstall electrical service conduit as needed for sidewalk and concrete repair. Maintain electrical power (Multnomah County owned).
2. Pier 3 thru Bent 29 – 8– 3" steel communications conduits attached to bridge bents with sufficient clearance, protect in place (CenturyLink owned).
3. Bents 29 thru 35 – Street light service conduit attached or minimal clearance from concrete repairs, protect in place (Multnomah County owned).
4. Bent 32 – Remove subdeck lighting and conduit for floorbeam enlargement strengthening. Refurbish and reinstall lighting (Multnomah County owned.)

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MULTNOMAH COUNTY

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UTILITIES
BRIDGE

SHEET NO. 6B

Location	Floorbeam Strengthening Locations	Floorbeam Attachments		Column Attachments	
		Description	Treatment	Description	Treatment
Bent 21	Span 2	None	N/A	None	N/A
Bent 22	Span 2	None	N/A	None	N/A
Bent 23	Span 2	None	N/A	Storm drain pipe from catch basins at Columns 1 and 2	Remove and reinstall for concrete repair
Bent 24	Span 2	None	N/A	None	N/A
Bent 25	Span 2	None	N/A	None	N/A
Bent 26	Span 2	None	N/A	Storm drain pipe from catch basins at Columns 1 and 2	Remove and reinstall for concrete repair
Bent 27	Spans 2 and 3	None	N/A	None	N/A
Bent 28	Spans 2 and 3	None	N/A	None	N/A
Bent 29	Spans 2, 3, and 4	None	N/A	8– CenturyLink conduits inside Column 1 emerging just below floorbeam	Protect in place
Bent 30	Spans 2, 3, and 4	None	N/A	None	N/A
Bent 31	Spans 2, 3, and 4	Seismic retrofit	Protect in place	None	N/A
Bent 32	Spans 2, 3, and 4	Parking lot light and conduit on floorbeam in Span 2	Remove and reinstall for concrete repair and floorbeam strengthening	None	N/A
Bent 33	Spans 2, 3, and 4	Seismic retrofit	Protect in place	Storm drain pipe from catch basins at Columns 1 and 4	Remove and reinstall for concrete repair
		Lighting conduit on east face, entire length	Remove and reinstall for concrete repair and floorbeam strengthening	N/A	N/A
		Lighting conduit crossing under floorbeam, Span 2	Remove and reinstall for concrete repair and floorbeam strengthening	N/A	N/A
		Street light on each floorbeam span	Remove and reinstall for concrete repair and floorbeam strengthening	N/A	N/A
Bent 34	Spans 2, 3, and 4	Seismic retrofit	Protect in place	None	N/A
		Lighting conduit crossing under floorbeam, Span 2	Remove and reinstall for concrete repair and floorbeam strengthening	N/A	N/A
Bent 35	Spans 2, 3, and 4	Electrical conduit on south end of wall	Protect in place	Electrical cabinet on south end of wall	Protect in place
		N/A	N/A	Lighting electrical meter on south end of wall	Protect in place

Location	Description of utility under sidewalk	Treatment
Pier 3 to Pier 4	8– CenturyLink conduits, north side, resting on floorbeams	CenturyLink will remove and reinstall on hangers beneath the floorbeams. Work anticipated to be complete prior to project. Protect in place.
Pier 4 to Bent 29	8– CenturyLink conduits, north side, attached by hangers	Protect in place
Bent 23 to Bent 25	Electrical conduit, south side	Remove and reinstall for sidewalk repairs
Bent 29 to Bent 35	Lighting conduit, both sides	Protect in place

Note:
Column numbering convention is north to south; Column 1 is the northern column. Span 1 is the northern overhang; Span 2 is between Columns 1 and 2.

REGISTERED PROFESSIONAL ENGINEER
74896PE
OREGON
JULY 13, 2004
BRENDAN E. LEBLANC
EXPIRES: DEC. 31, 2016

MULTNOMAH COUNTY BRIDGES

HDR | DAVID EVANS AND ASSOCIATES INC.

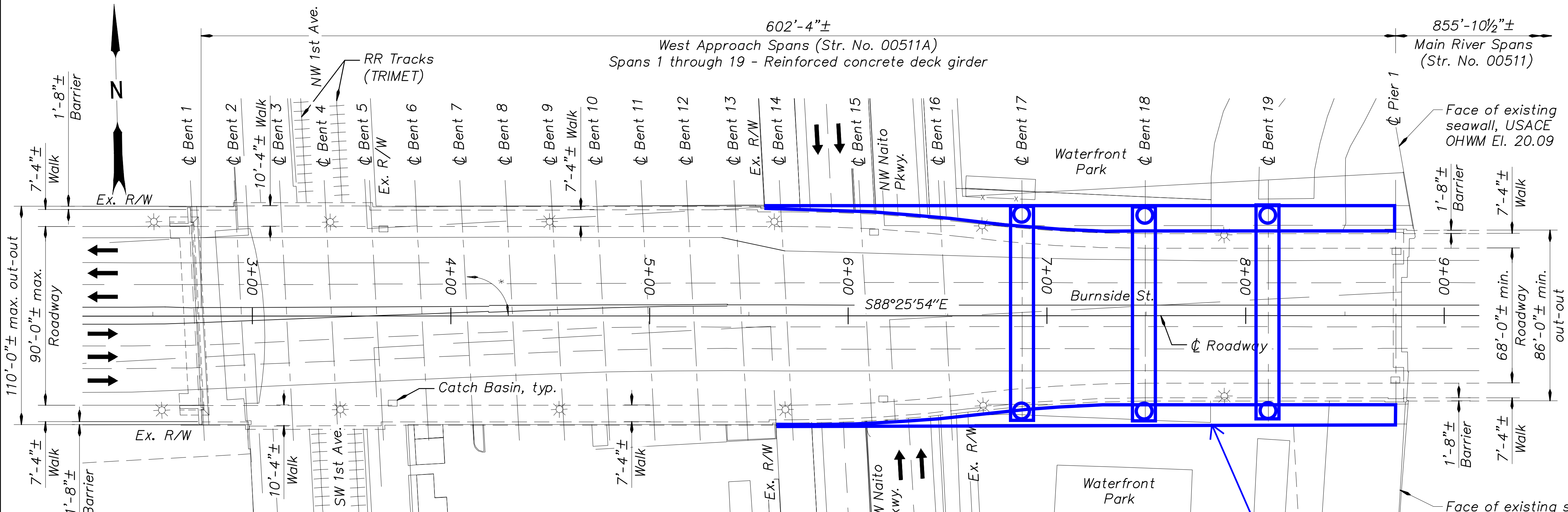
BURNSIDE ST. WILLAMETTE RIVER BRIDGE
PAINTING AND REHABILITATION PROJECT
MULTNOMAH COUNTY

Reviewed By – Brendan LeBlanc
Designed By – Brent Carney
Drafted By – Heather Gonsior

UTILITIES
BRIDGE

SHEET NO. 6C

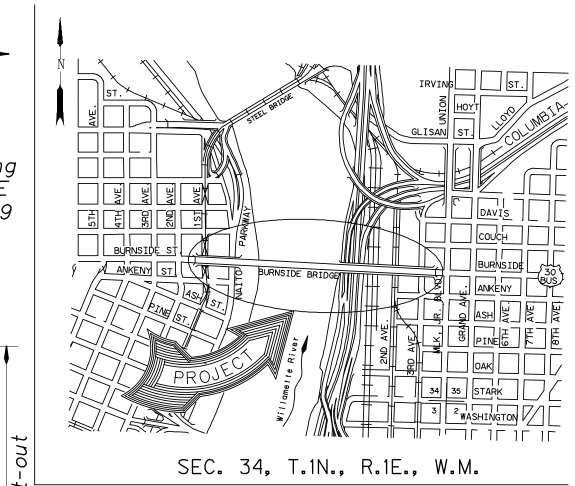
Appendix E. EQRB Conceptual Retrofit Plans



PLAN

Scale: 1" = 30'-0"

∗ Angle for Bents 1 -16 = 93°05'30",
Angle for Bents 17 - 19, and Pier 1 = 90°00'00"



LOCATION MAP

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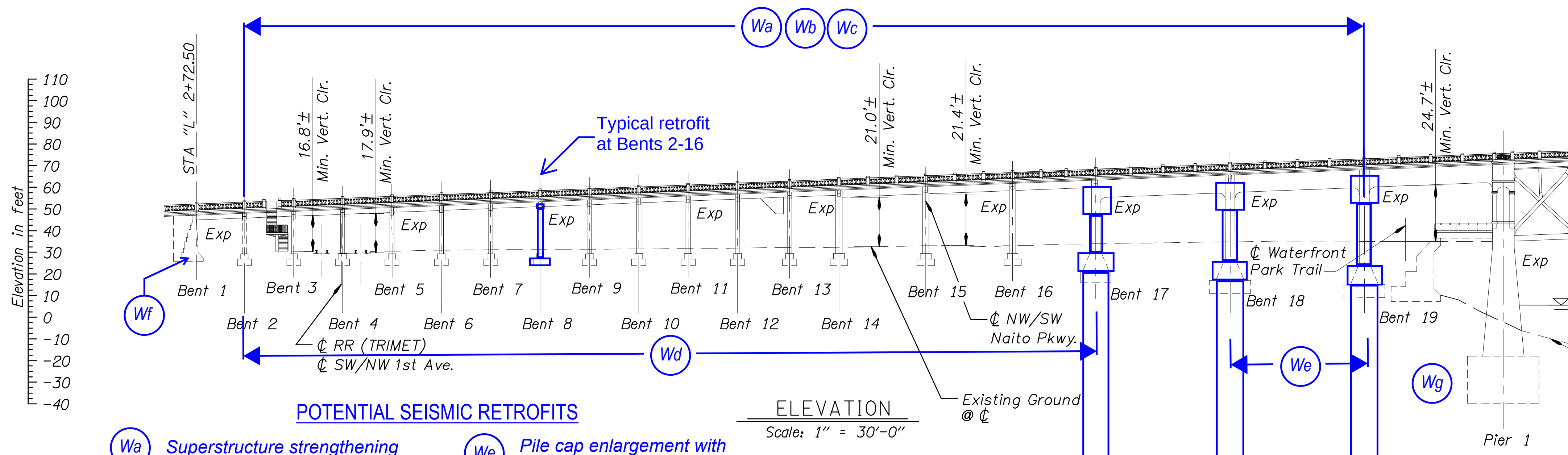

ELEVATION

Scale: 1" = 30'-0"

Notes:

1. Span support condition is fixed unless noted otherwise.
2. Elevations shown are based on North American Vertical Datum, (NAVD) 1988.

POTENTIAL SEISMIC RETROFITS

- Wa Superstructure strengthening
- Wb End floorbeam strengthening
- Wc Column enhancement
- Wd Footing enlargement with or without micro piles or drilled shafts
- We Pile cap enlargement with drilled shafts / superbent widening
- Wf Abutment enhancement to limit differential settlement
- Wg Soil improvements to supplement retrofit measures Wd and We.

Alt. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT AND WIDENING

PLAN & ELEVATION - WEST APPROACH

SCALE: AS SHOWN

DATE: AUGUST 2017

MULTNOMAH COUNTY

DEPARTMENT OF COMMUNITY SERVICES
LAND USE AND TRANSPORTATION PROGRAM
1620 S.E. 190th AVE.
PORTLAND, ORE. 97233-5999

BRIAN S. VINCENT P.E. COUNTY ENGINEER

DESIGNED: DRAWN: CHECKED:

REVISIONS

DATE:

SHEET NO. 1

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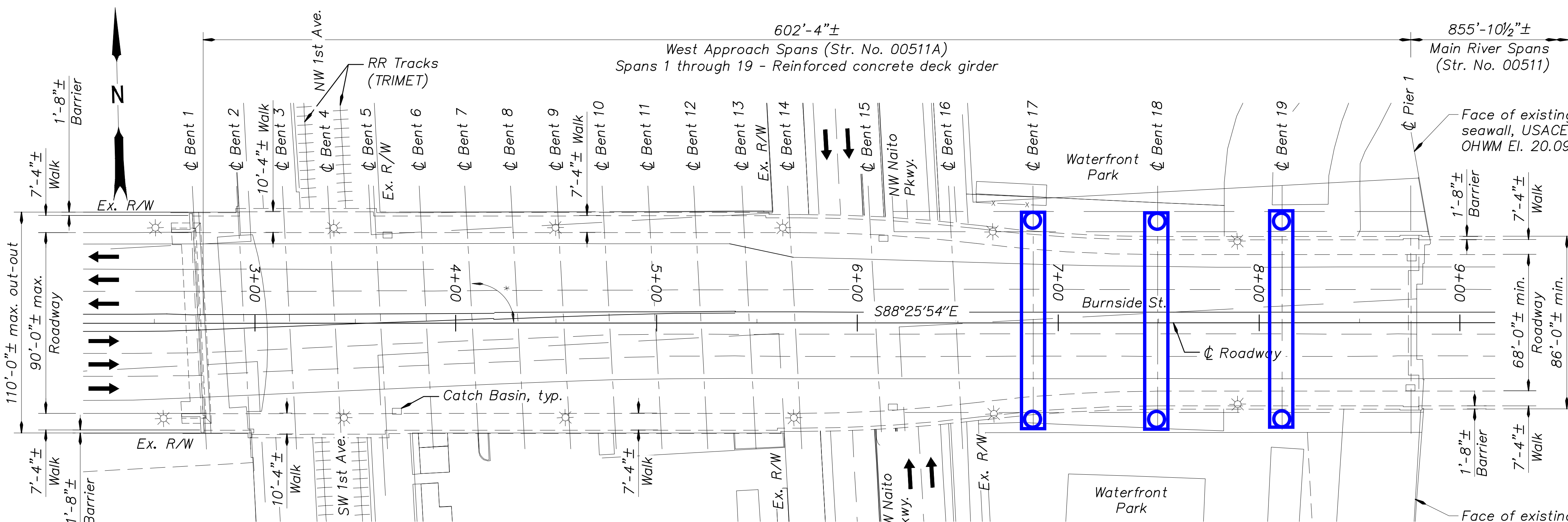

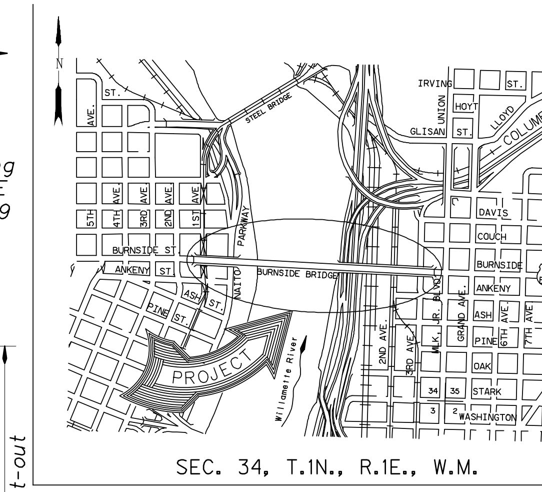

LOCATION MAP

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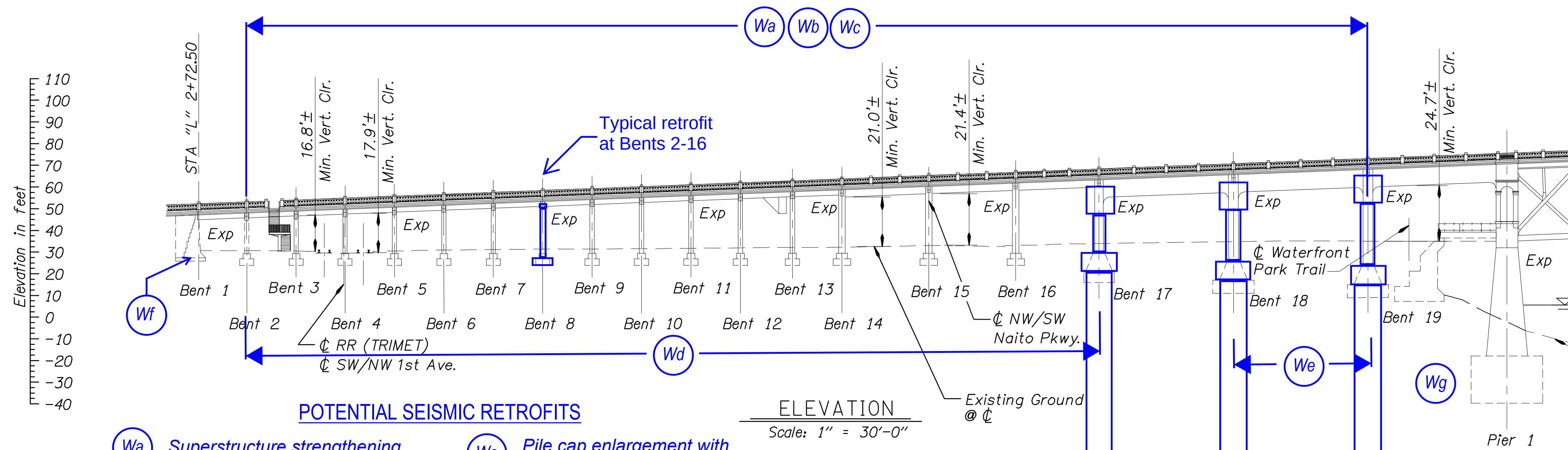

ELEVATION

Scale: 1" = 30'-0"

Notes:
1. Span support condition is fixed unless noted otherwise.
2. Elevations shown are based on North American Vertical Datum, (NAVD) 1988.

POTENTIAL SEISMIC RETROFITS

- Wa Superstructure strengthening
- Wb End floorbeam strengthening
- Wc Column enhancement
- Wd Footing enlargement with or without micro piles or drilled shafts
- We Pile cap enlargement with drilled shafts / superbent widening
- Wf Abutment enhancement to limit differential settlement
- Wg Soil improvements to supplement retrofit measures Wd and We.

Alt. 4b.1a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT - NO WIDENING

PLAN & ELEVATION - WEST APPROACH

SCALE: AS SHOWN

DATE: AUGUST 2017

MULTNOMAH COUNTY

DEPARTMENT OF COMMUNITY SERVICES
LAND USE AND TRANSPORTATION PROGRAM
1620 S.E. 190th AVE.
PORTLAND, ORE. 97233-5999

BRIAN S. VINCENT P.E.
COUNTY ENGINEER

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SHEET NO.
2

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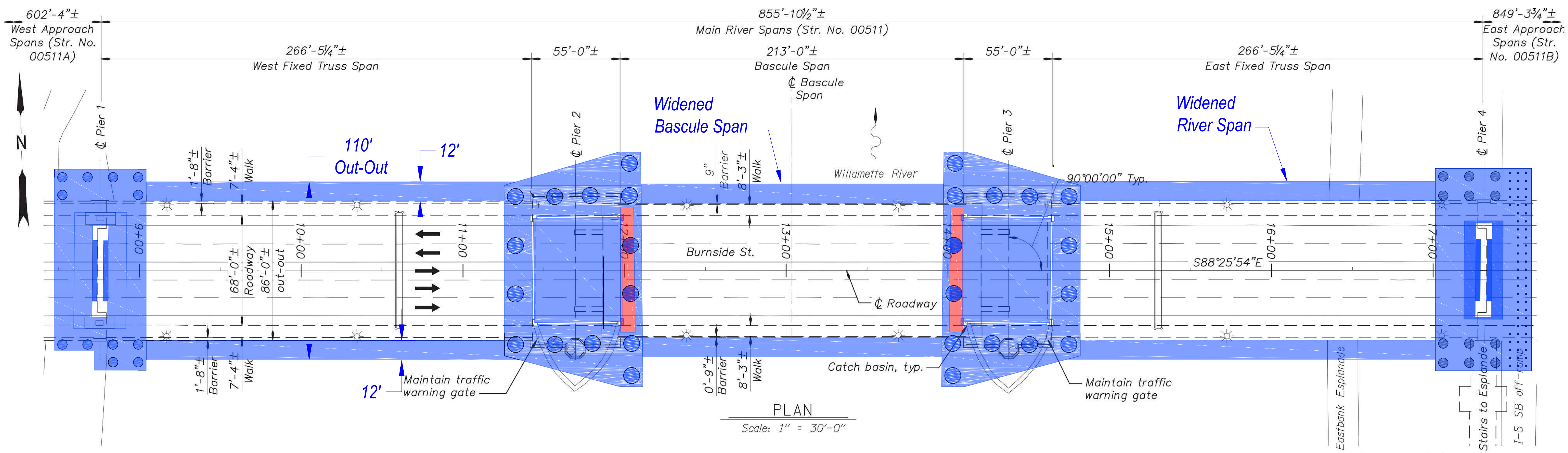

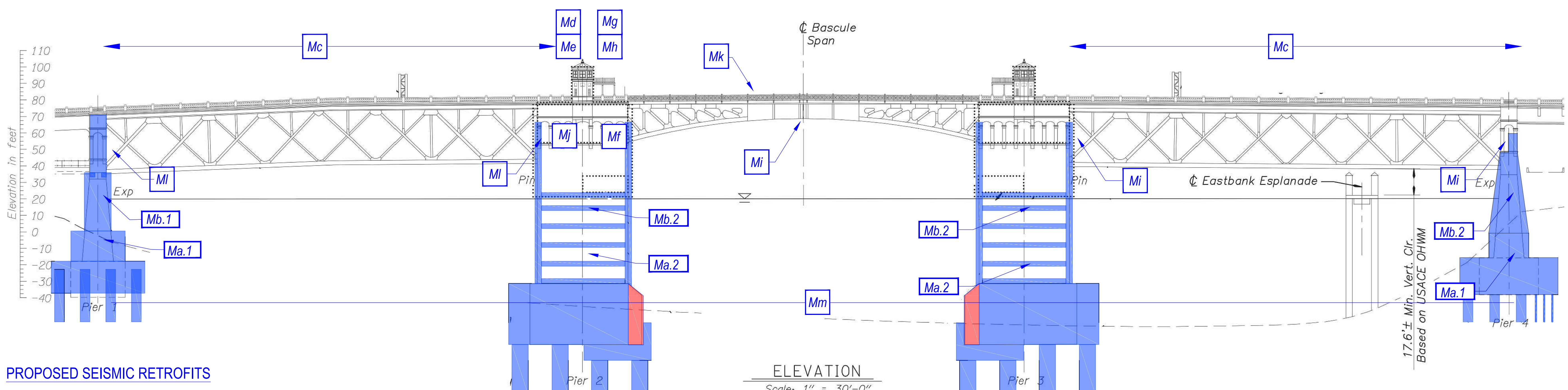

PROPOSED SEISMIC RETROFITS

Ma	Expand Caps with additional Drilled Shafts
Mb	Retrofit Piers
Mc	Install Additional Sway Bracing
Md	Reinforce Truss Supports at Piers 2 and 3
Me	Strengthen Steel Members and add Lateral Bracing
Mf	Retrofit Live Load Support Bearings
Mg	Retrofit Deck Stringer Bearings
Mh	Install Counterweight Restraints
Mi	Redesign and Replace Center Lock
Mj	Retrofit Bascule Trunnion Support
Mk	Replace Bascule Deck with Lightweight Deck
Ml	Retrofit Fixed Span Bearings
Mm	Soil Improvements

4b.2a
EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT + WIDENING
PLAN & ELEVATION -RIVER SPANS

SCALE: AS SHOWN
DATE: AUGUST 2017

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
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COUNTY ENGINEER

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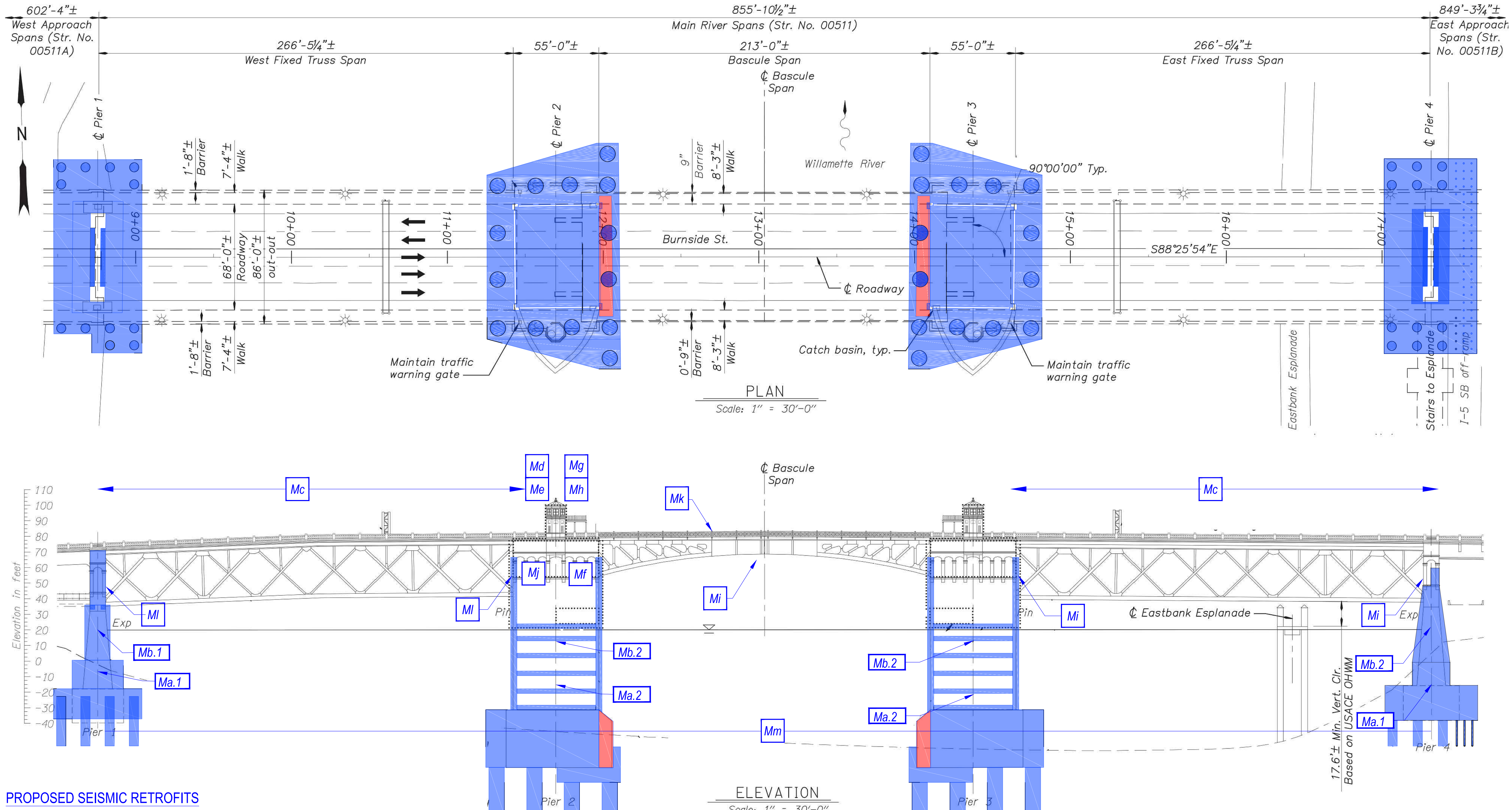

PROPOSED SEISMIC RETROFITS

Ma	Expand Caps with additional Drilled Shafts
Mb	Retrofit Piers
Mc	Install Additional Sway Bracing
Md	Reinforce Truss Supports at Piers 2 and 3
Me	Strengthen Steel Members and add Lateral Bracing
Mf	Retrofit Live Load Support Bearings
Mg	Retrofit Deck Stringer Bearings
Mh	Install Counterweight Restraints
Mi	Redesign and Replace Center Lock
Mj	Retrofit Bascule Trunnion Support
Mk	Replace Bascule Deck with Lightweight Deck
Ml	Retrofit Fixed Span Bearings
Mm	Soil Improvements

4b.1a
EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT - NO WIDENING
PLAN AND ELEVATION- RIVER SPANS

SCALE: AS SHOWN
DATE: AUGUST 2017

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
LAND USE AND TRANSPORTATION PROGRAM
1620 S.E. 190th AVE.
PORTLAND, ORE. 97233-5999
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DATE:

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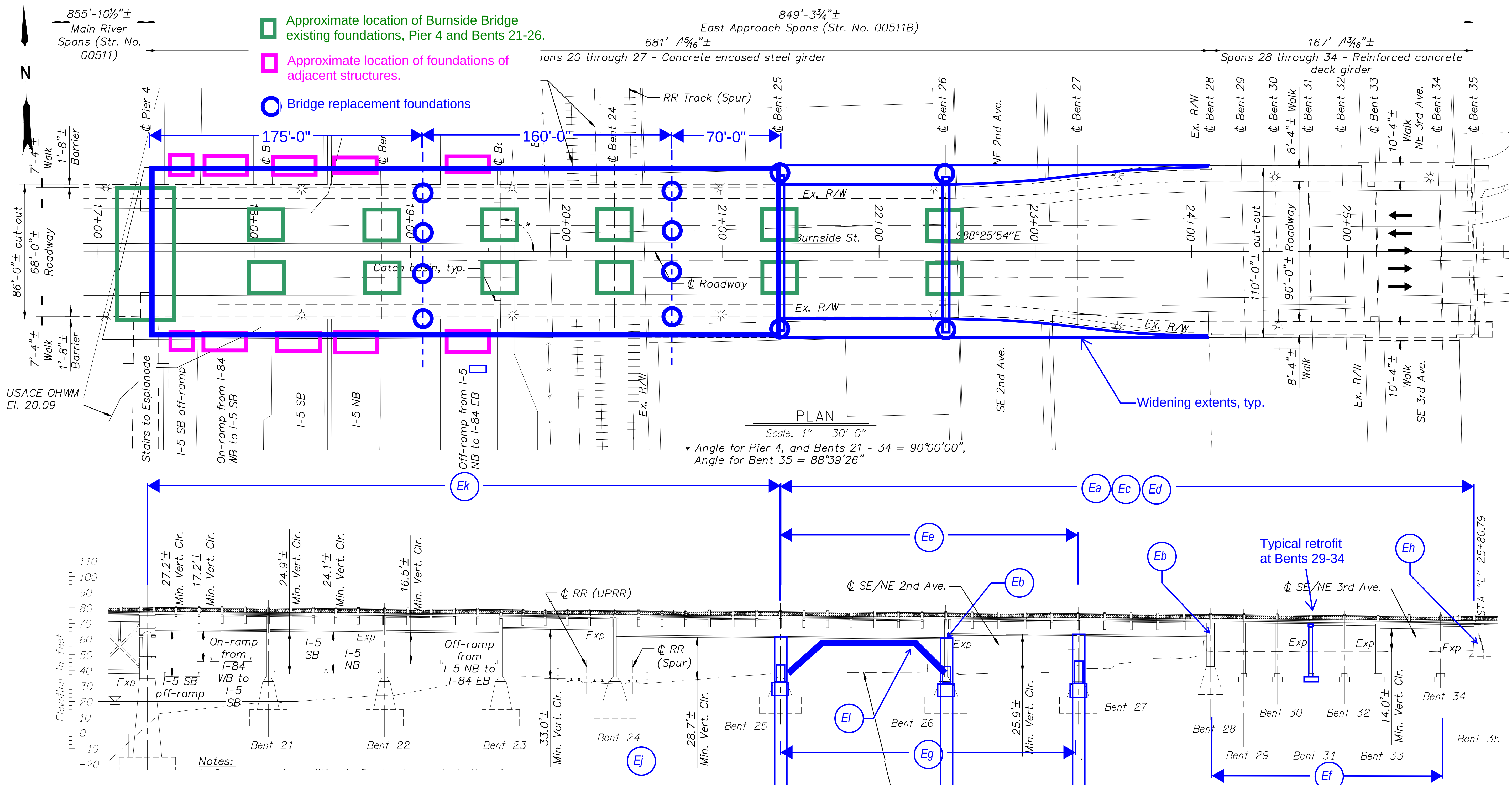
4



PROPOSED SEISMIC RETROFITS

- Ea — Superstructure strengthening
- Eb — Bearing replacement
- Ec — End floorbeam strengthening
- Ed — Column enhancement
- Ee — Column sway bracing enhancement and infill wall
- Ef — Footing enlargement with or without micro piles or drilled shafts
- Eg — Pile cap enlargement with drilled shafts / superbent widening
- Eh — Abutment enhancement to limit differential settlement. Allow connect to superstructure to fuse.
- Ej — Soil improvements to supplement retrofit measures Ef and Eg.
- Ek — Replace Spans 20-24 and Bents 21-24 with a 3-span structure
- El — Longitudinal stiffening between bents

Alt. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
PLAN & ELEVATION - EAST APPROACH

SCALE: AS SHOWN
DATE: AUGUST 2017

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
LAND USE AND TRANSPORTATION PROGRAM
1620 S.E. 190th AVE.
PORTLAND, ORE. 97233-5999

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DESIGNED: DRAWN: CHECKED:

REVISIONS
DATE:

SHEET NO. 5

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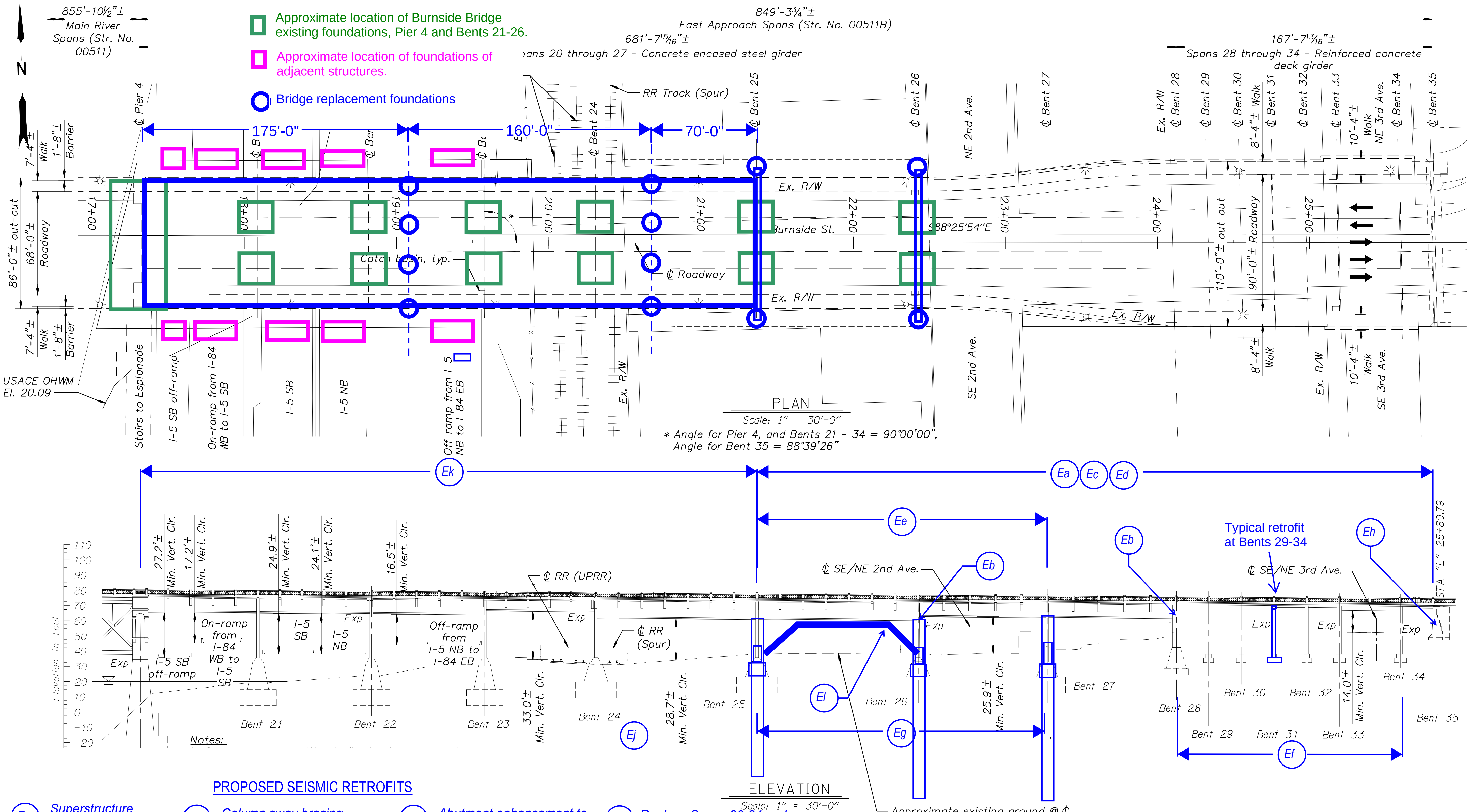

PROPOSED SEISMIC RETROFITS

- **Ea** Superstructure strengthening
- **Eb** Bearing replacement
- **Ec** End floorbeam strengthening
- **Ed** Column enhancement
- **Ee** Column sway bracing enhancement and infill wall
- **Ef** Footing enlargement with or without micro piles or drilled shafts
- **Eg** Pile cap enlargement with drilled shafts / superbent widening
- **Eh** Abutment enhancement to limit differential settlement. Allow connect to superstructure to fuse.
- **Ej** Soil improvements to supplement retrofit measures Ef and Eg.
- **Ek** Replace Spans 20-24 and Bents 21-24 with a 3-span structure
- **El** Longitudinal stiffening between bents

Alt. 4b.1a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT - NO WIDENING
PLAN & ELEVATION - EAST APPROACH

SCALE: AS SHOWN

DATE: AUGUST 2017

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
LAND USE AND TRANSPORTATION PROGRAM
1620 S.E. 190th AVE.
PORTLAND, ORE. 97233-5999

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COUNTY ENGINEER

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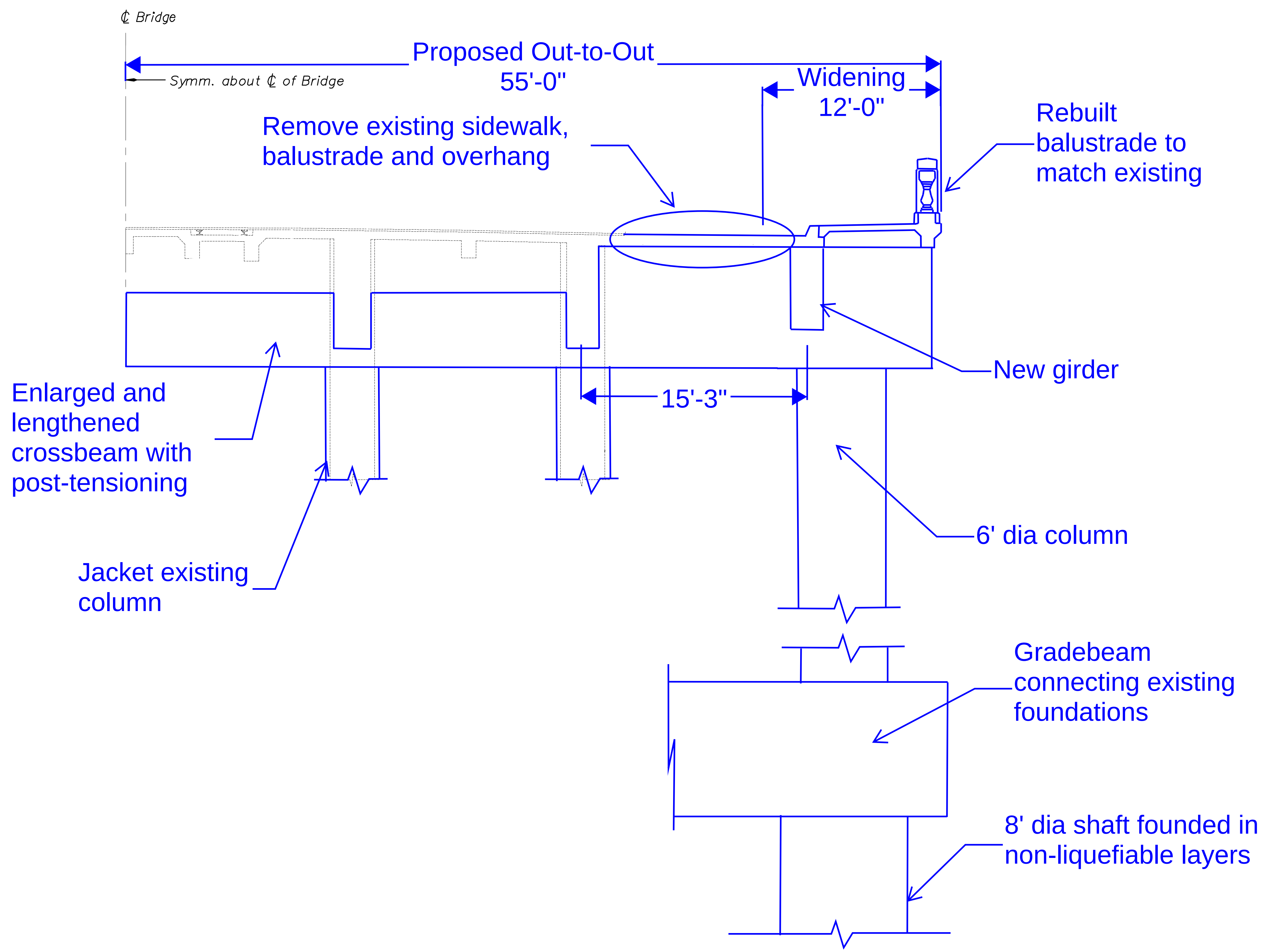

Widened West Approach Typical Section Spans 17-19 Shown

Alt. 4b.2a
EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
TYPICAL SECTION - WEST APPROACH

DATE: AUGUST 2017
SCALE: AS SHOWN

MULTNOMAH COUNTY
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7

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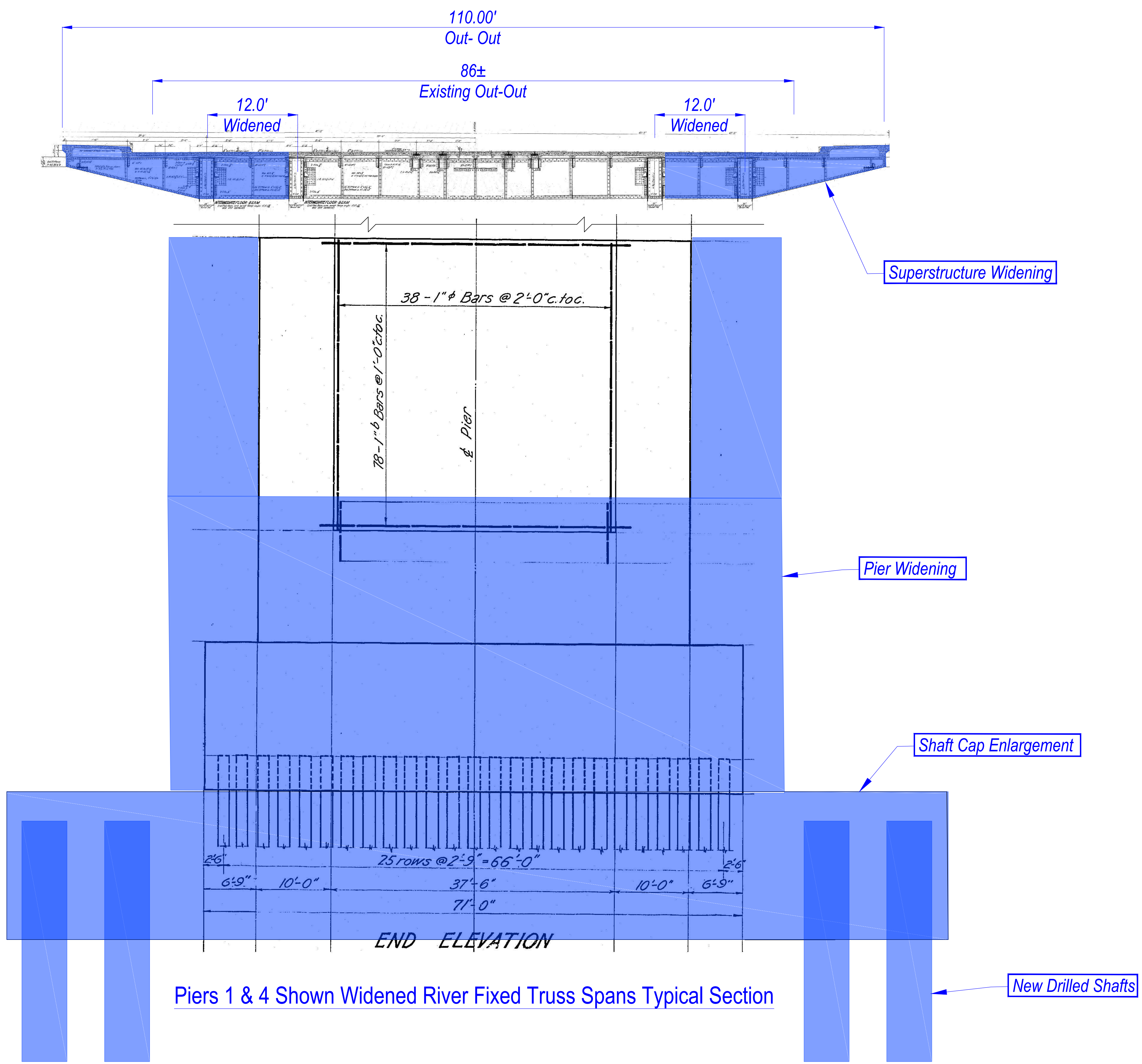

Piers 1 & 4 Shown Widened River Fixed Truss Spans Typical Section

ALT. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

TYPICAL SECTION -RIVER FIXED SPANS

SCALE: AS SHOWN

DATE: AUGUST 2017

MULTNOMAH COUNTY

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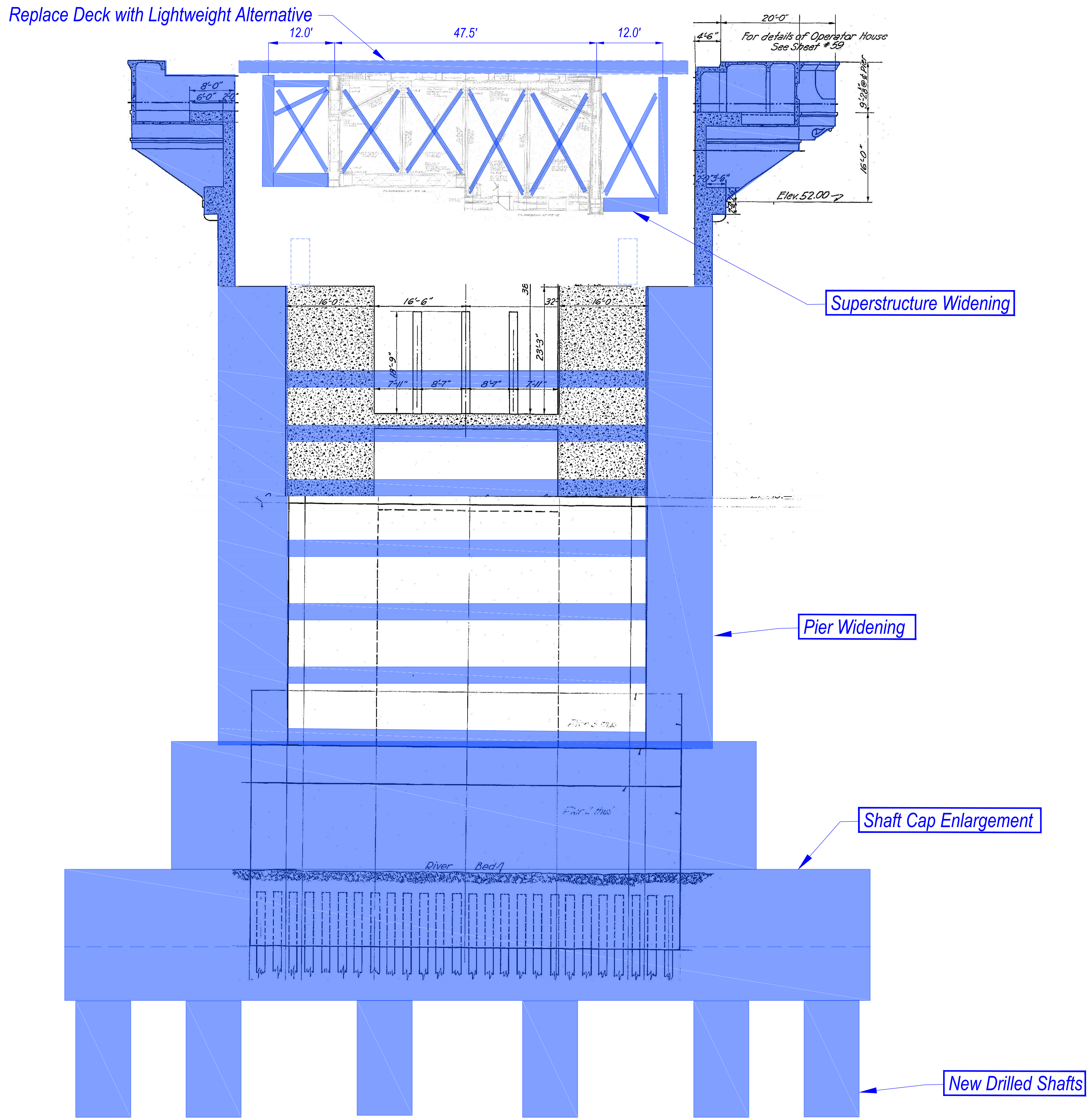

Widened Bascule Span Typical Section, Pier 2&3 Shown

ALT 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

TYPICAL SECTION- RIVER BASCULE SPANS

SCALE: AS SHOWN

DATE: AUGUST 2017

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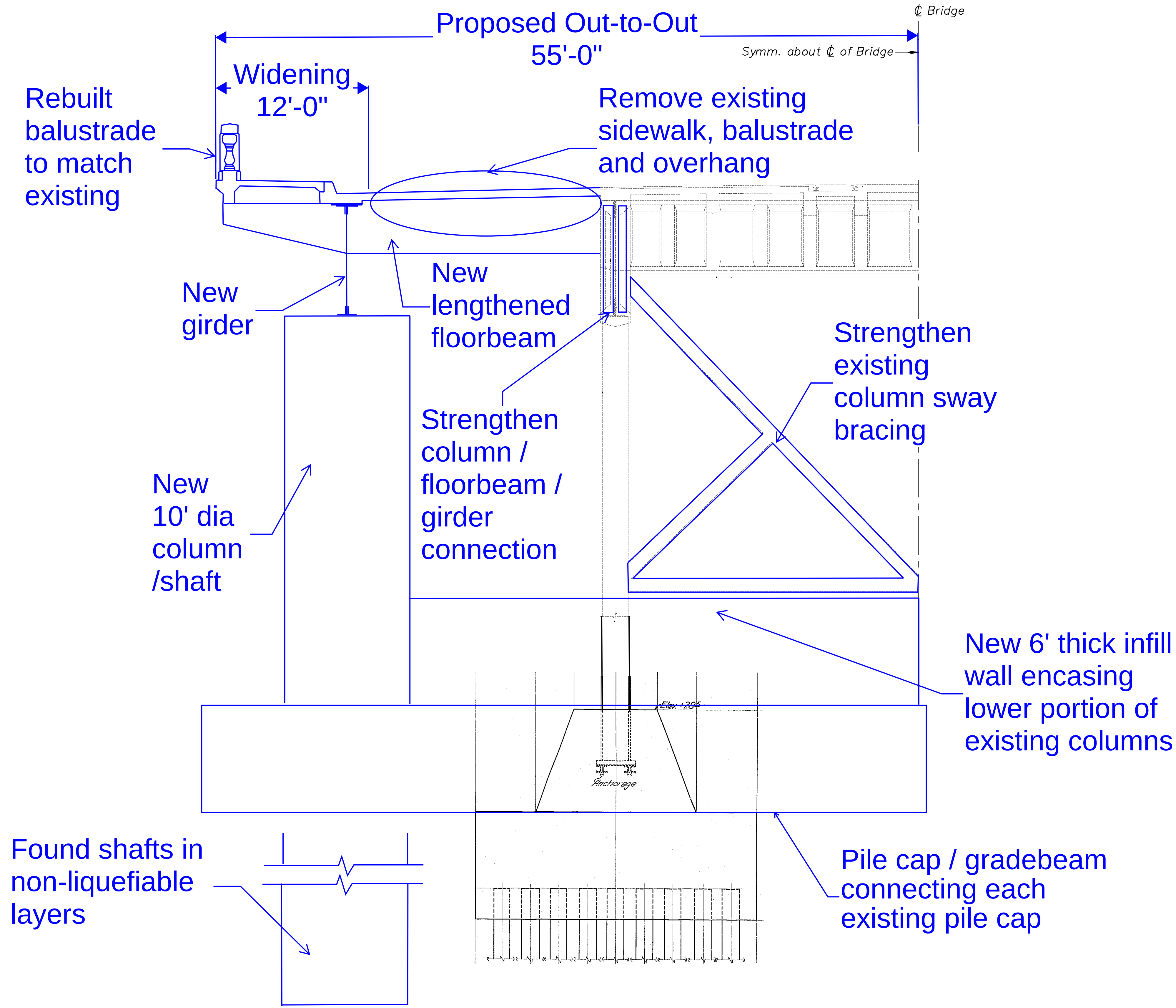

Widened East Approach Typical Section Spans 21-26

Alt. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
TYPICAL SECTION - EAST APPROACH

DATE: AUGUST 2017
SCALE: AS SHOWN

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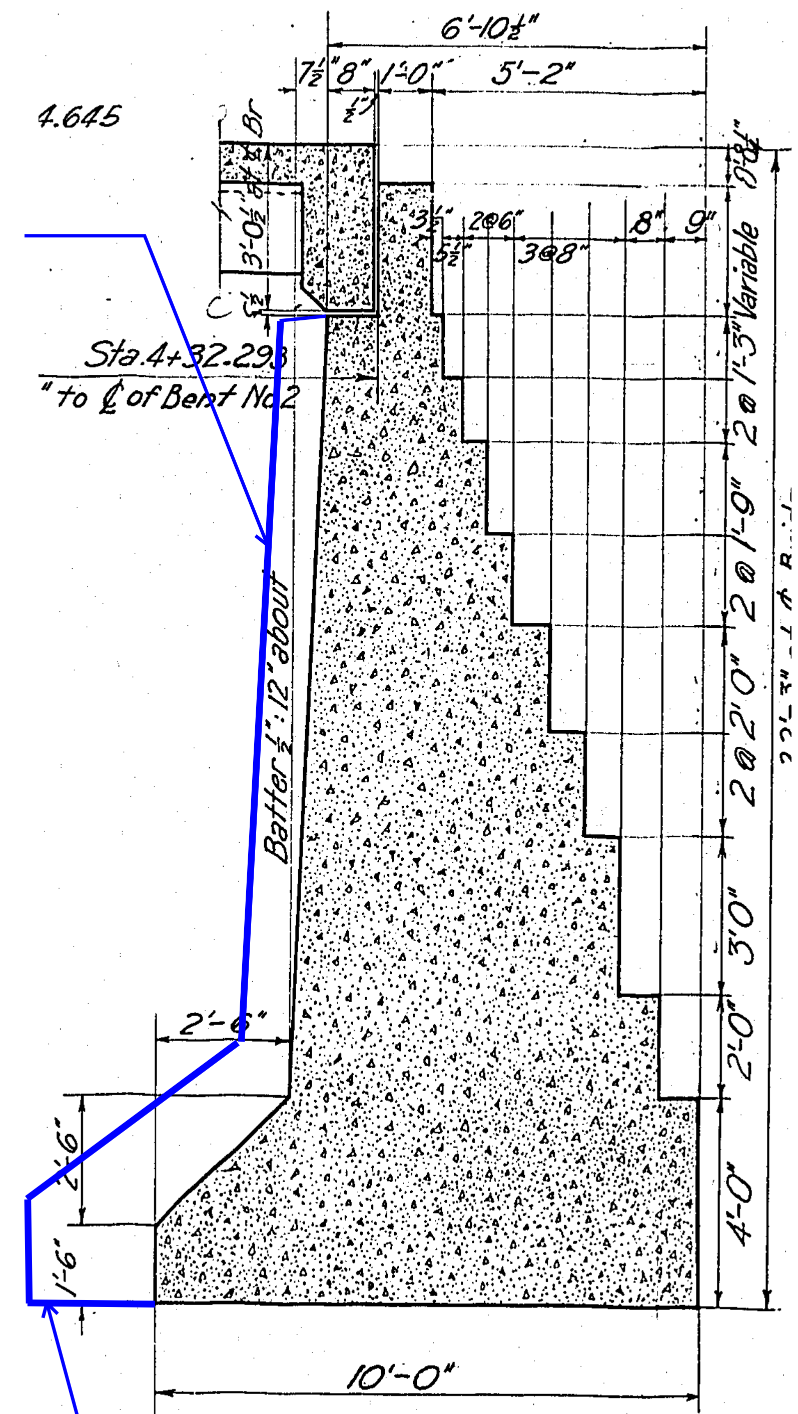
Thicken abutment wall to provide reinforcement



SECTION B-B

Increase footing width to resist overturning.

Alt. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
ABUTMENTS

DATE: AUGUST 2017 SCALE: AS SHOWN

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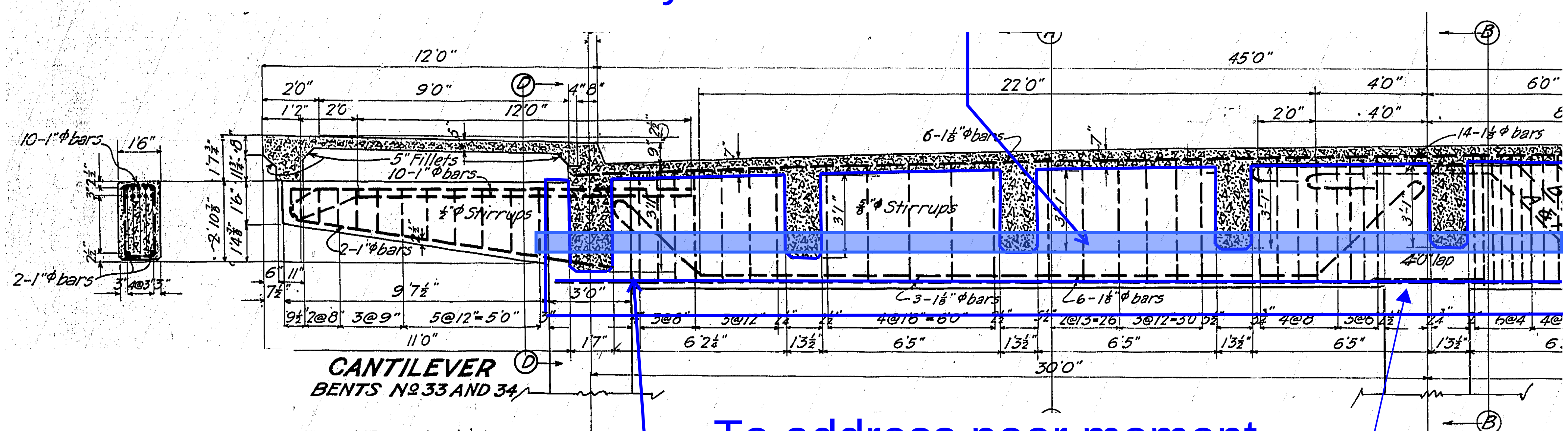
To maintain elastic behavior, provide transverse post-tensioning to address inadequate moment capacity at midspan at analysis identified locations.



To address poor moment capacity, provide additional reinforcement in the enlarged crossbeam section.

Typical Section
Spans 1-13, 28-34 Shown

Alt. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT AND WIDENING

CONCRETE FLOOR BEAMS

DATE: AUGUST 2017

SCALE: AS SHOWN

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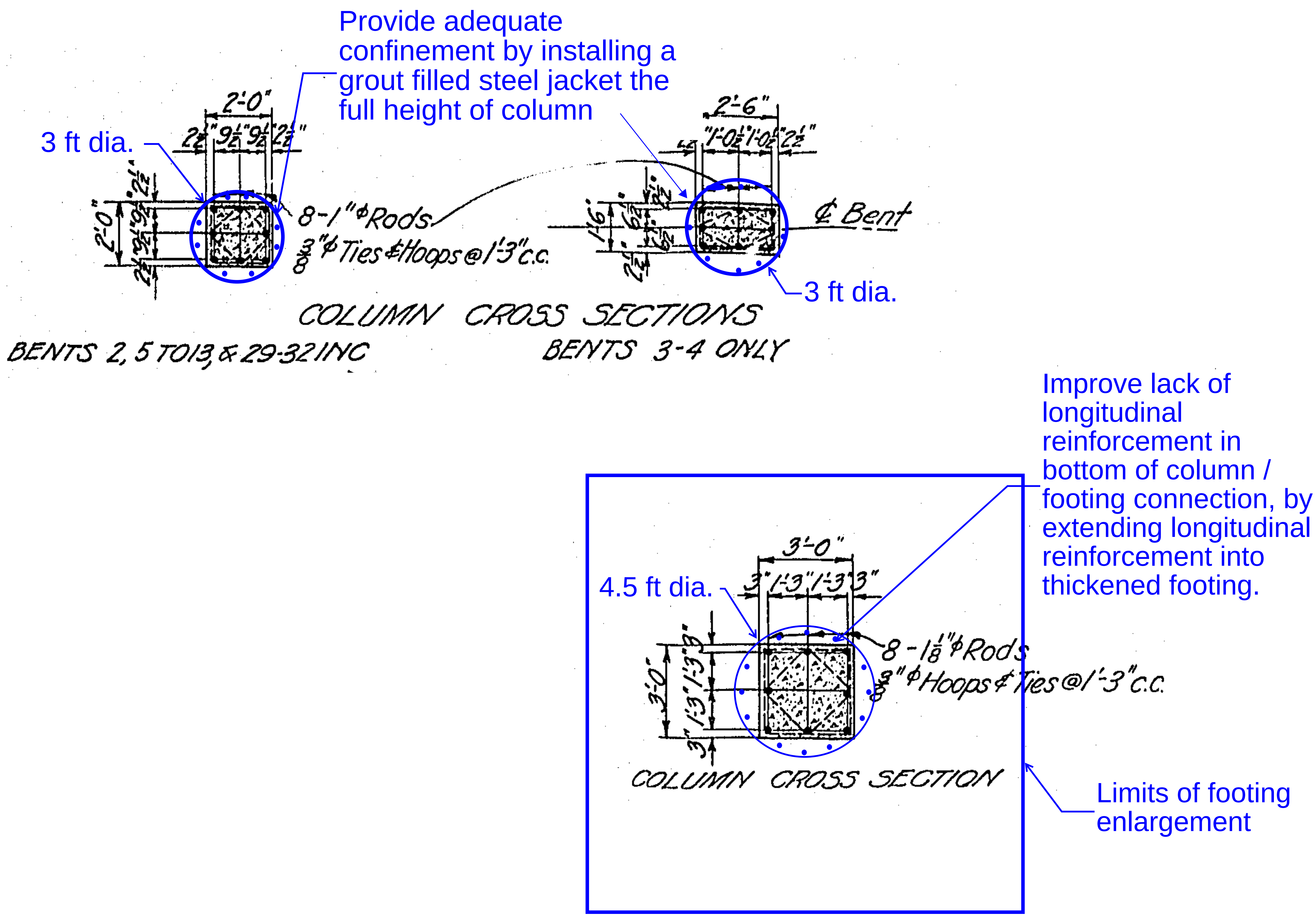
12



Alt. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
CONCRETE COLUMNS

DATE: AUGUST 2017
SCALE: AS SHOWN

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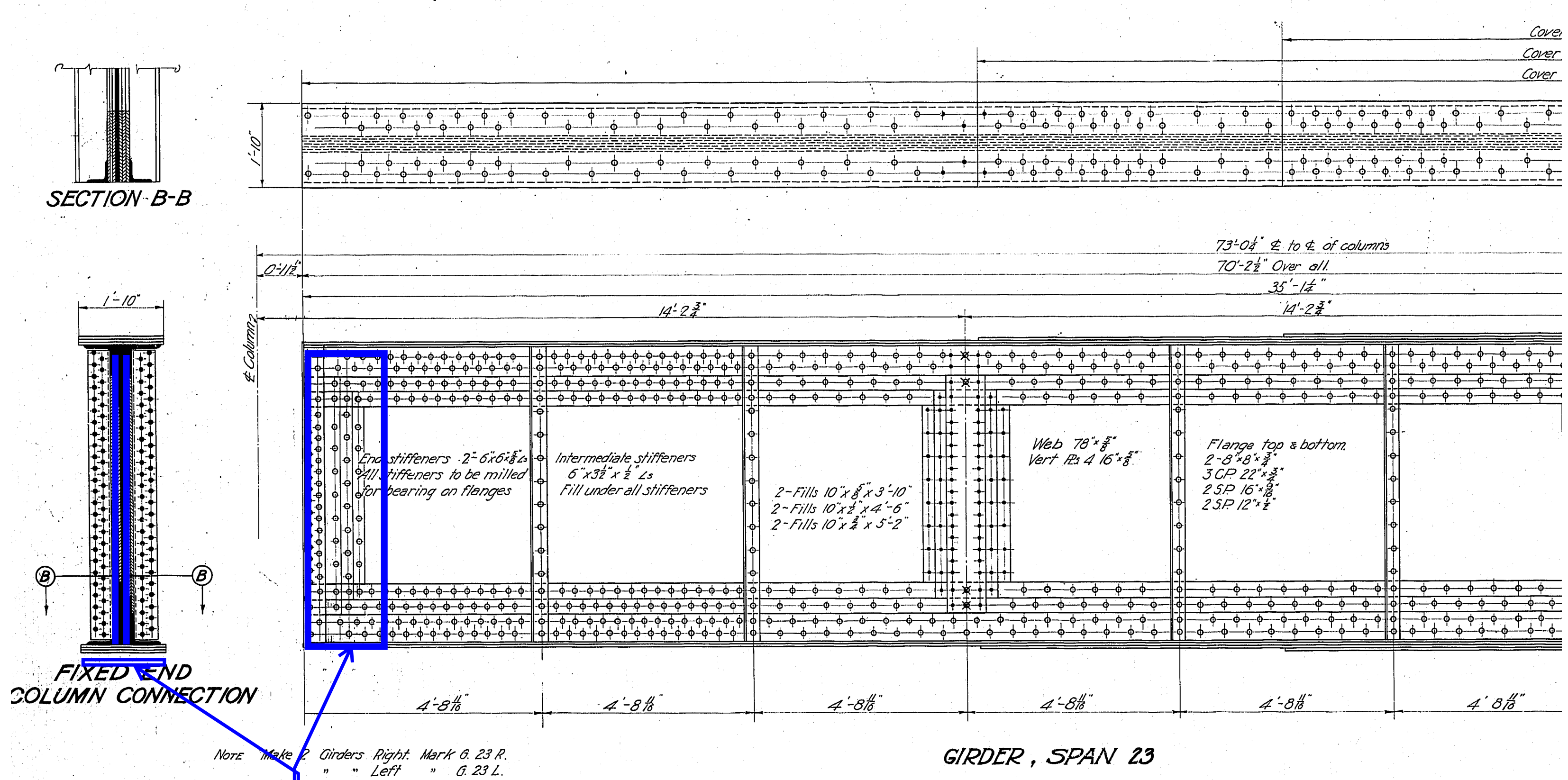
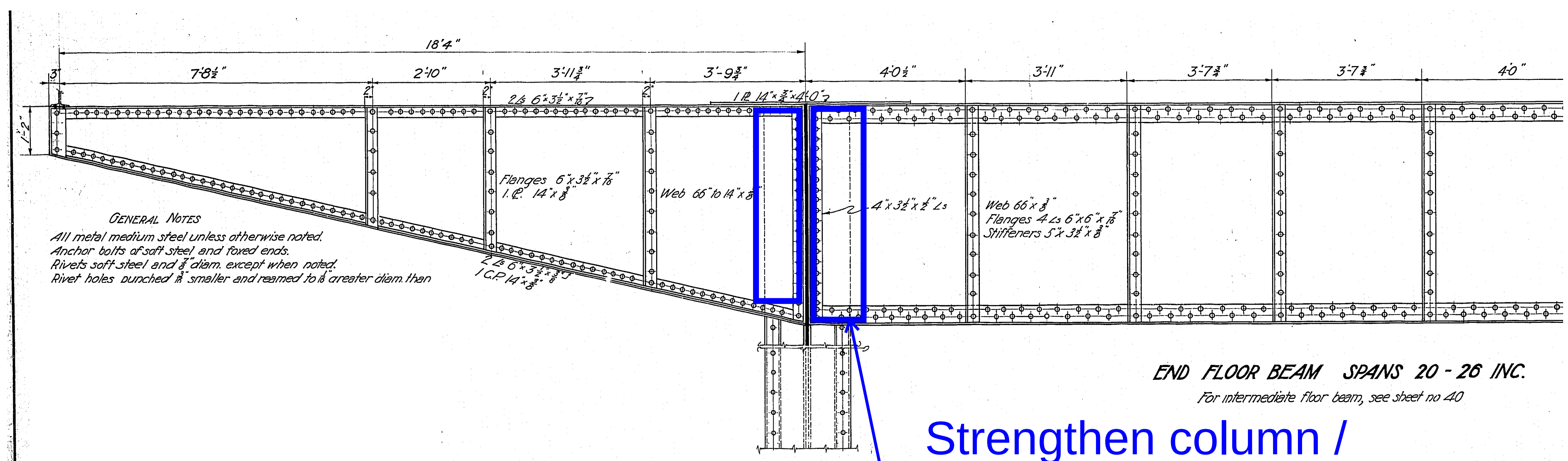


Strengthen column / floorbeam / girder connection.



Strengthen column / floorbeam / girder connection.

Alt. 4b.1a & 2a
EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
STEEL FLOOR BEAMS

DATE: AUGUST 2017
SCALE: AS SHOWN

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14

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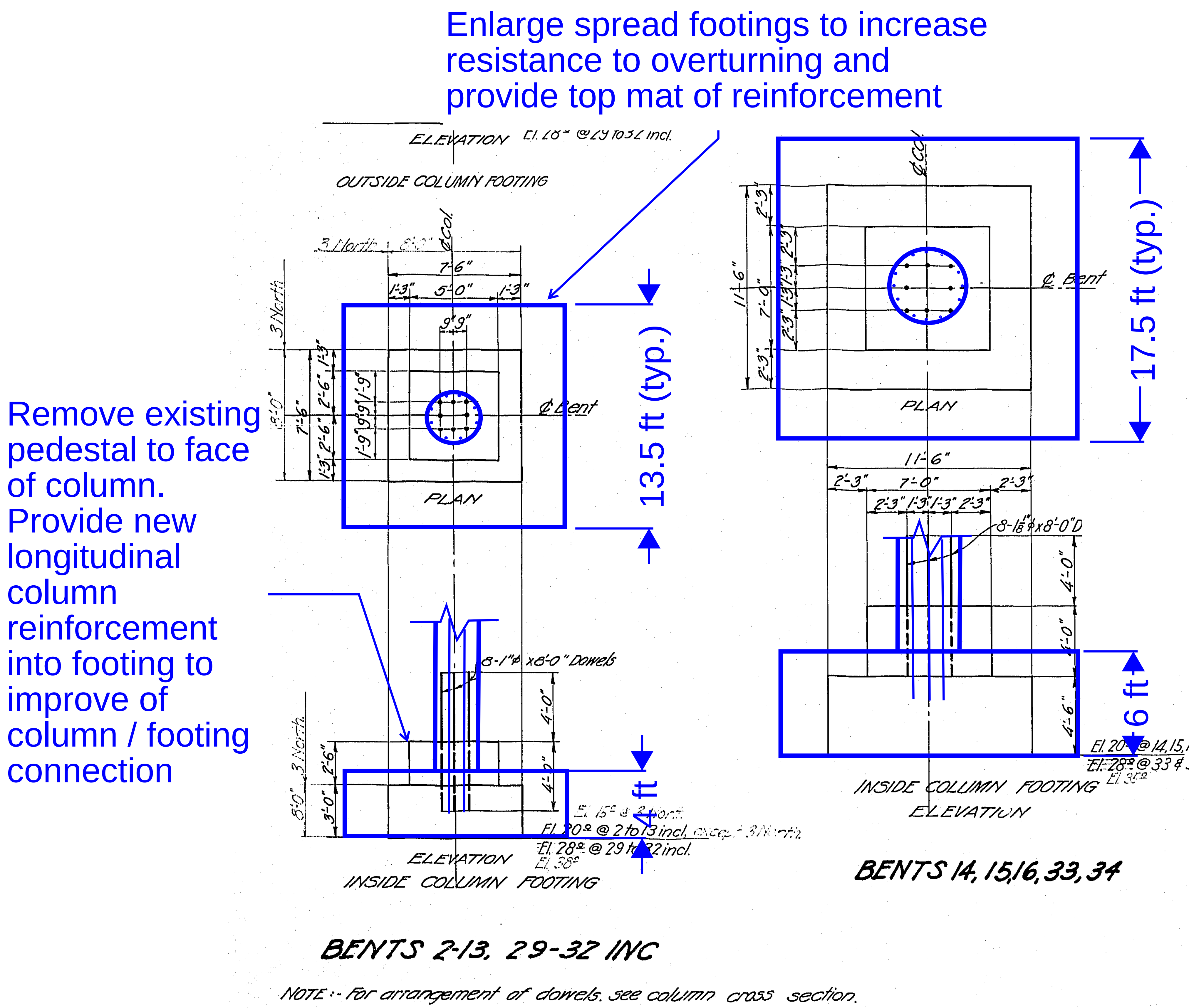

Alt. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
SPREAD FOOTING FOUNDATIONS

SCALE: AS SHOWN
DATE: AUGUST 2017

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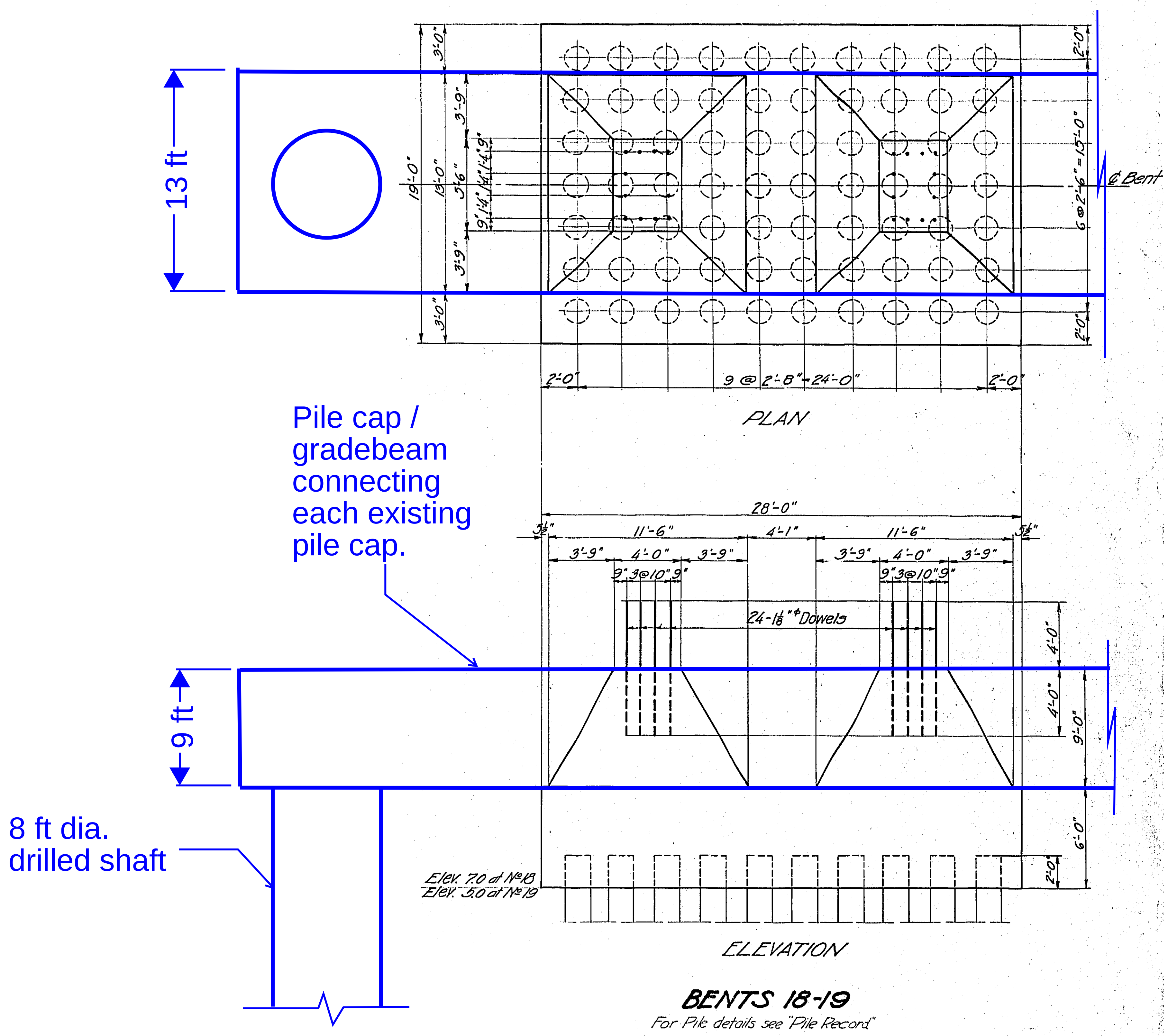

Alt. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT AND WIDENING
TIMBER PILE FOUNDATIONS

SCALE: AS SHOWN

DATE: AUGUST 2017

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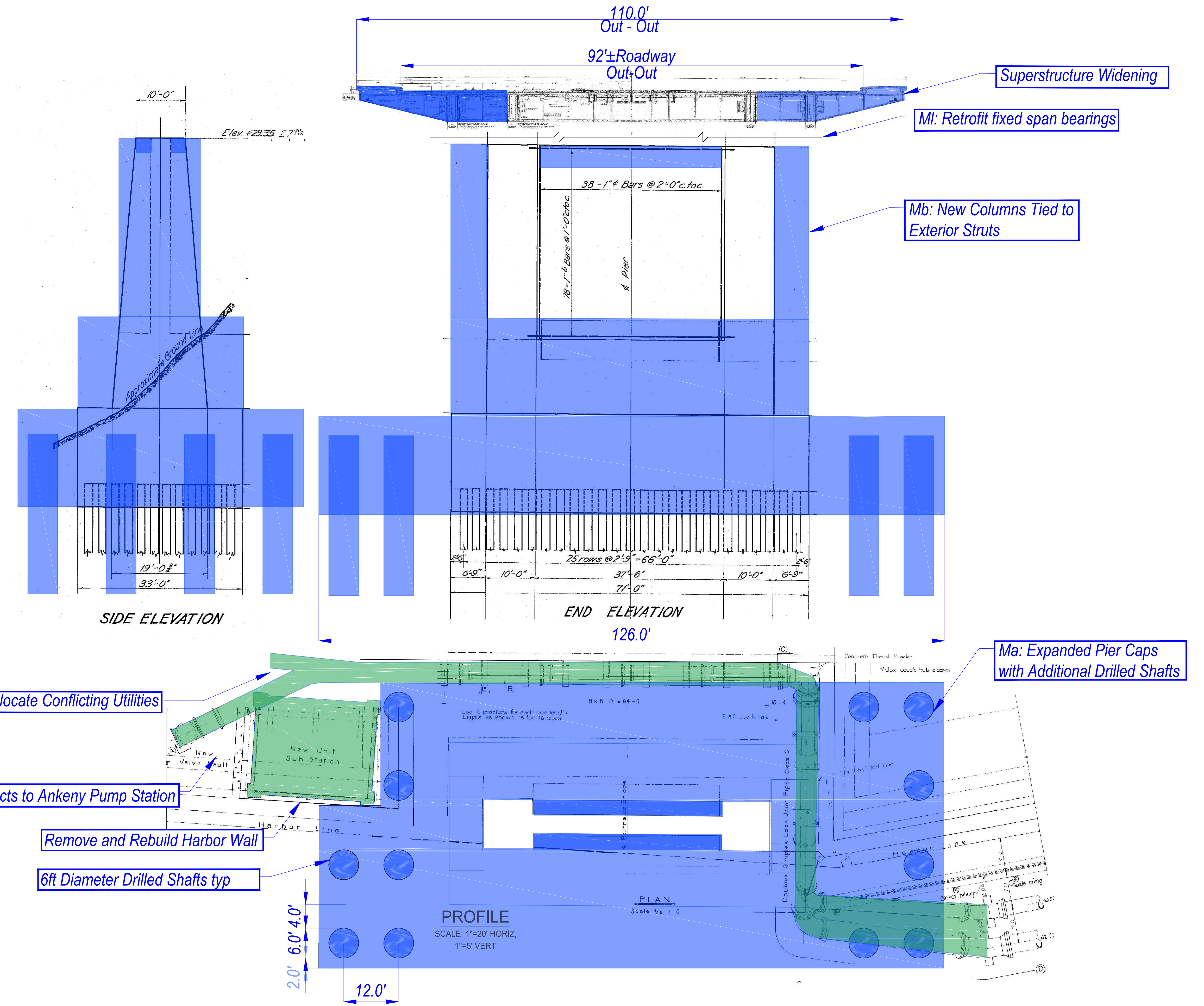

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ALT. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

RIVER FIXED SPAN @ PIER I

SCALE: AS SHOWN

DATE: AUGUST 2017

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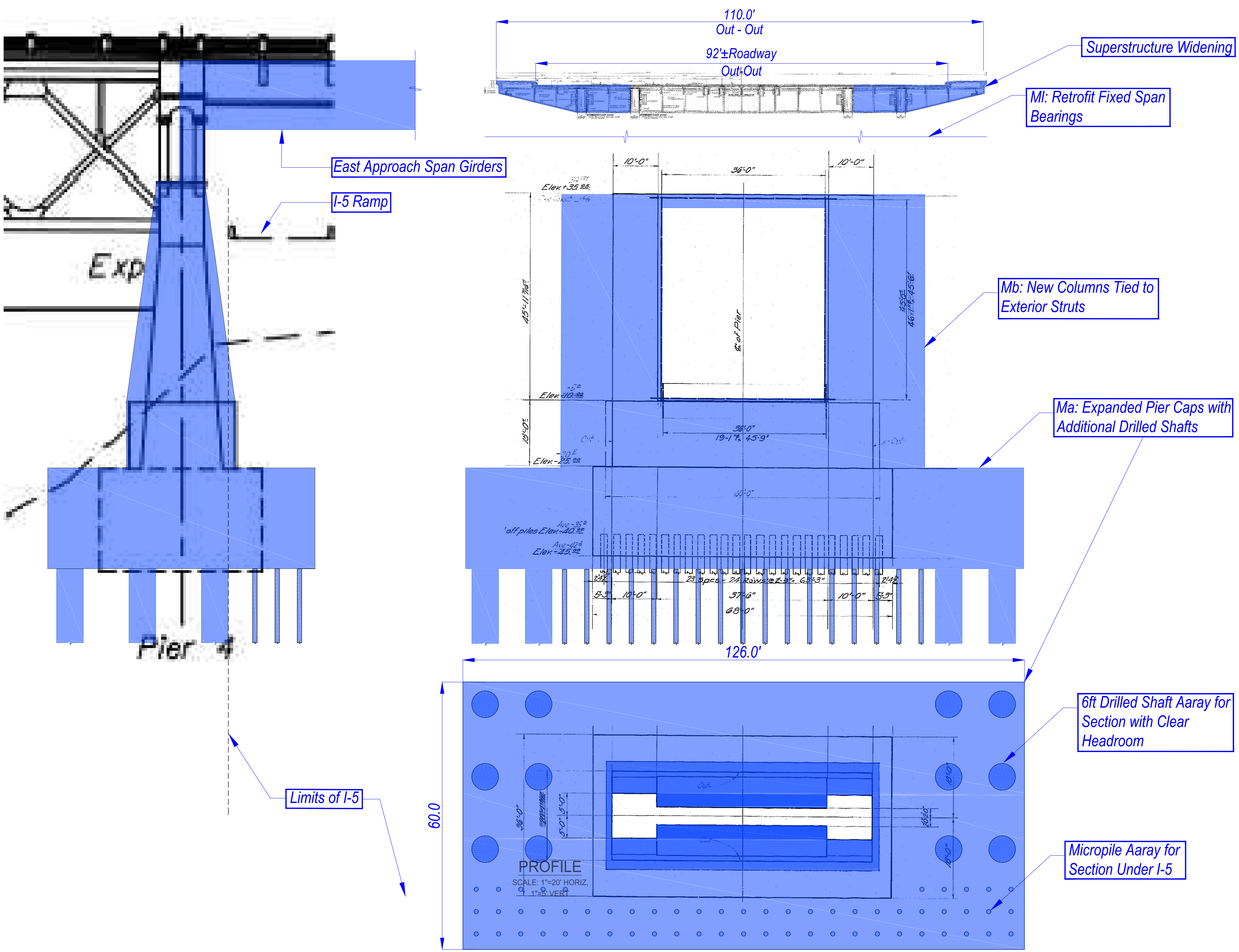
17



ALT. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

RIVER FIXED SPAN @ PIER IV-OPT A

SCALE: AS SHOWN

DATE: AUGUST 2017

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18

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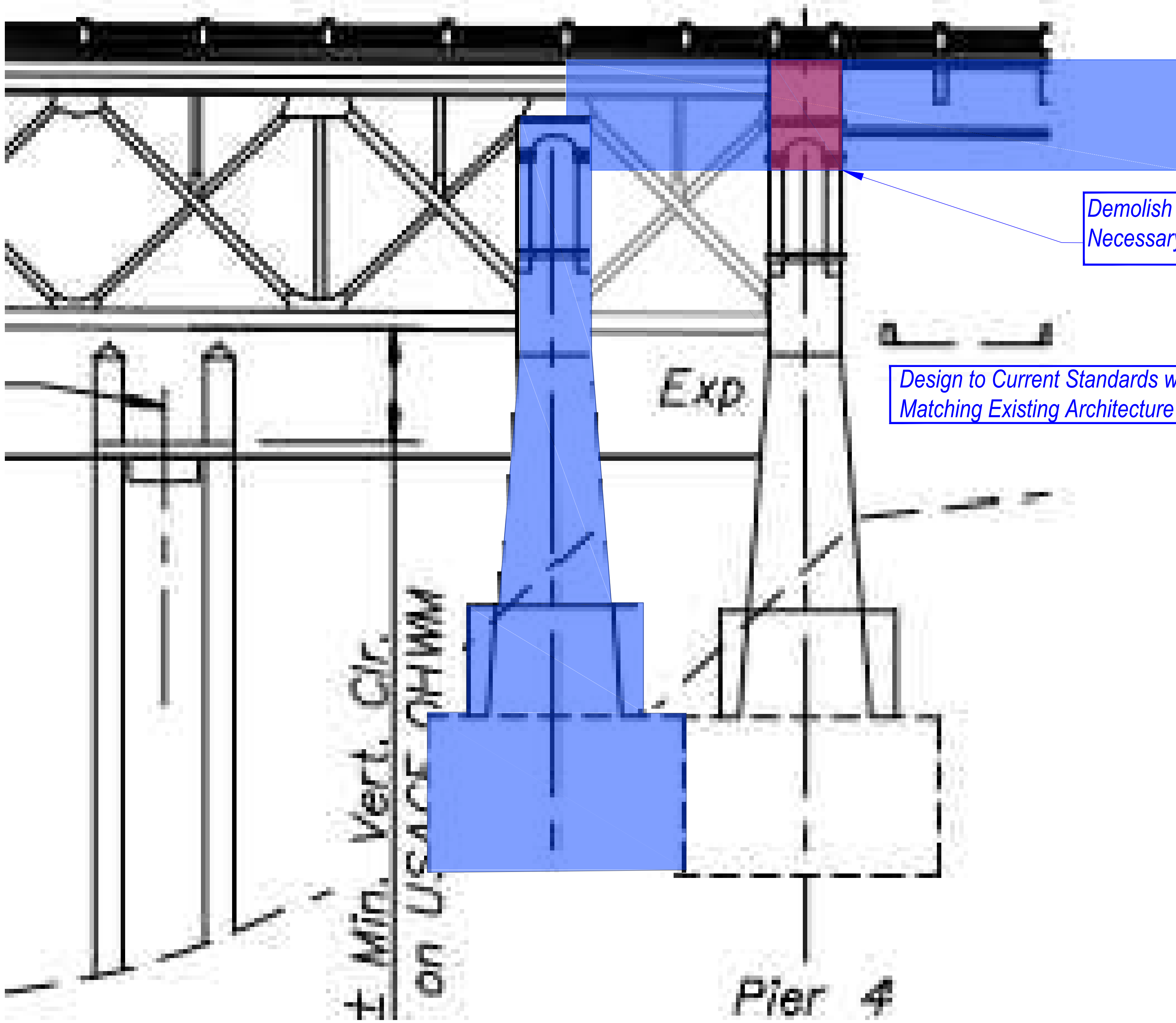
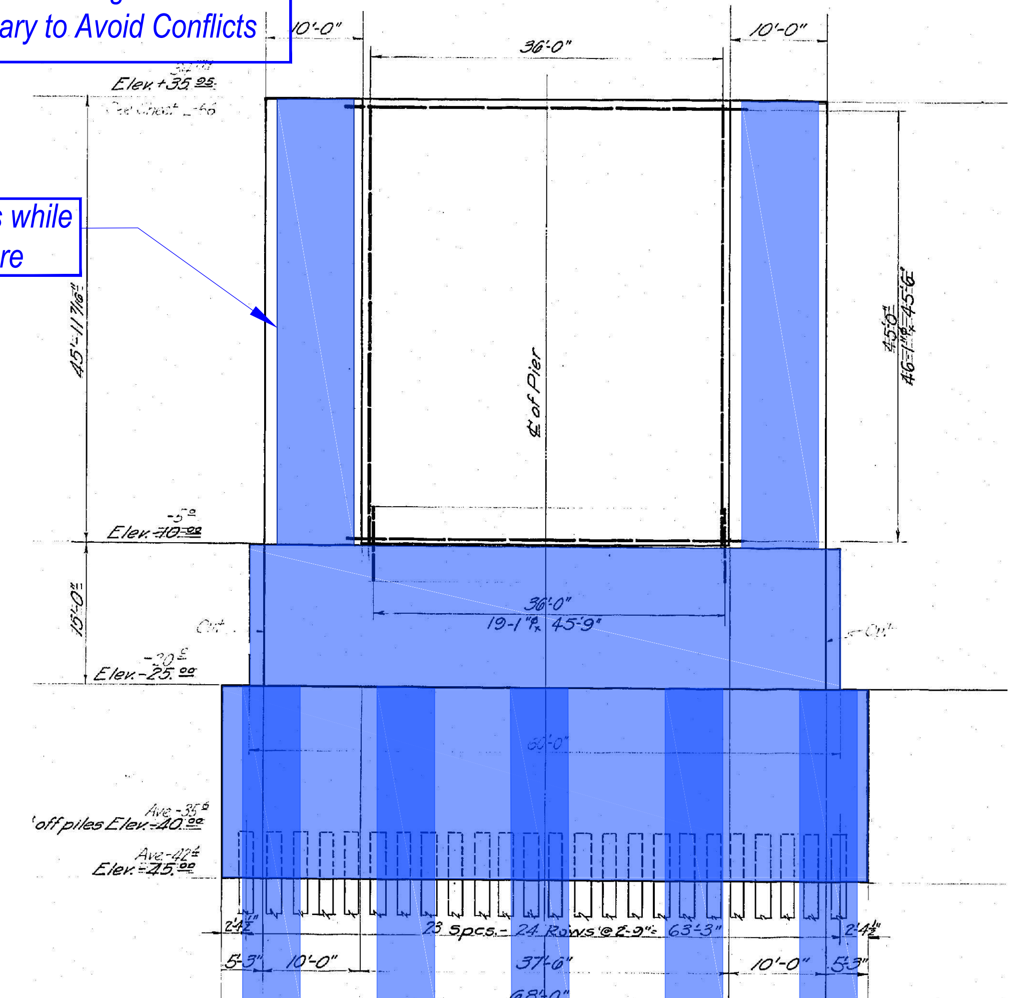


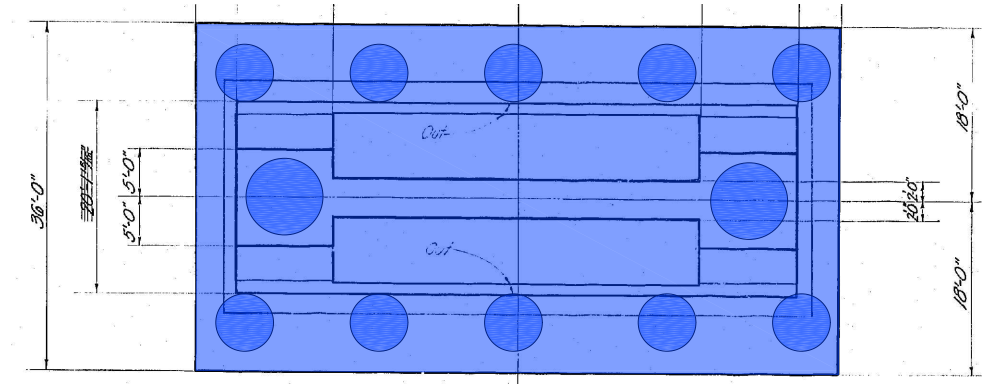
Extend Approach Bents and Shorten Truss Span as Appropriate

Demolish Existing Pier 4 where Necessary to Avoid Conflicts

Design to Current Standards while Matching Existing Architecture

Option B: Build New Pier 4





ALT. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT + WIDENING
RIVER FIXED SPAN @ PIER IV-OPT B

SCALE: AS SHOWN
DATE: AUGUST 2017

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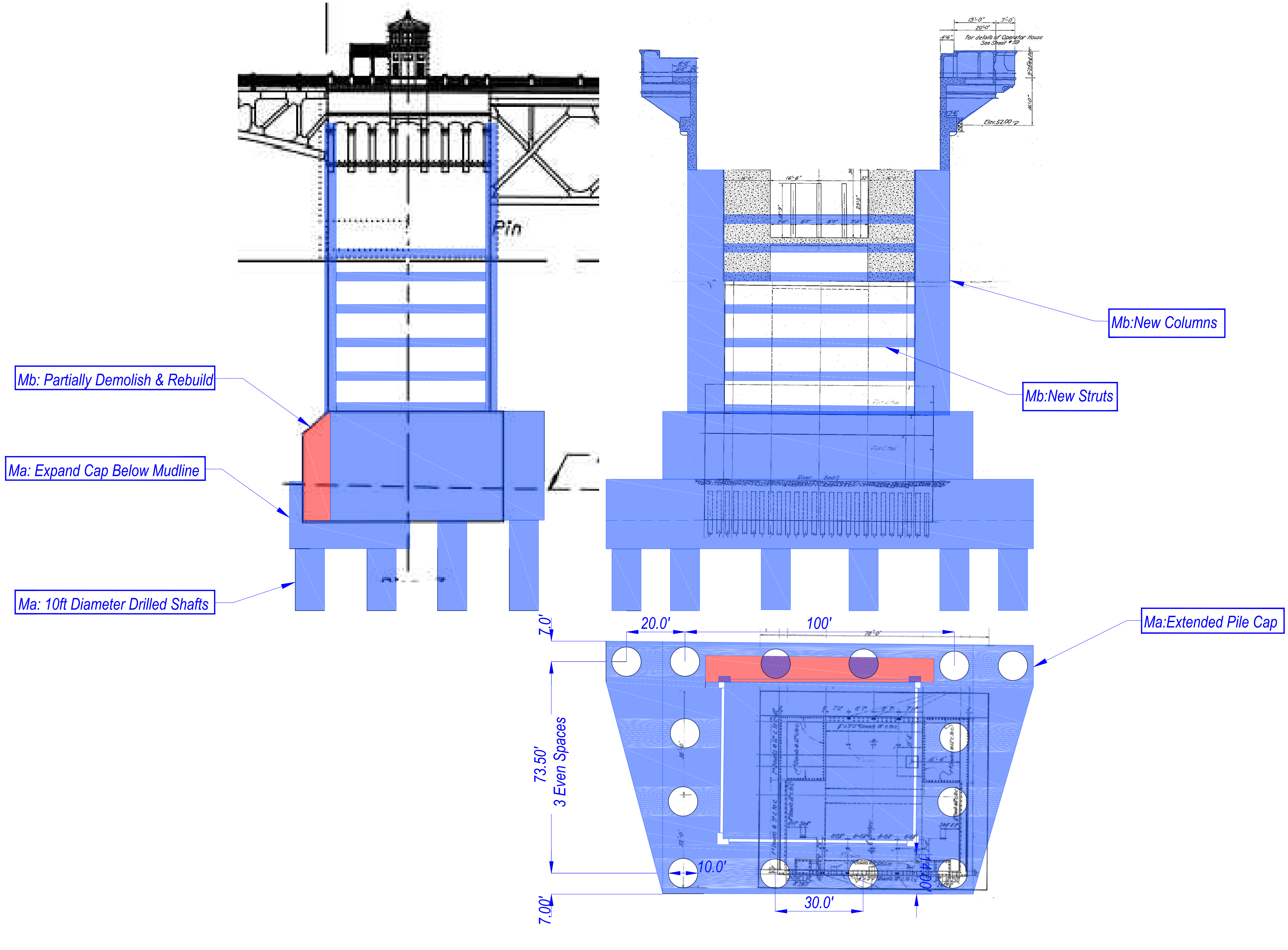

ALT. 4b.1a

EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT - NO WIDENING
RIVER SPAN PIER II & III -1

SCALE: AS SHOWN

DATE: AUGUST 2017

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20

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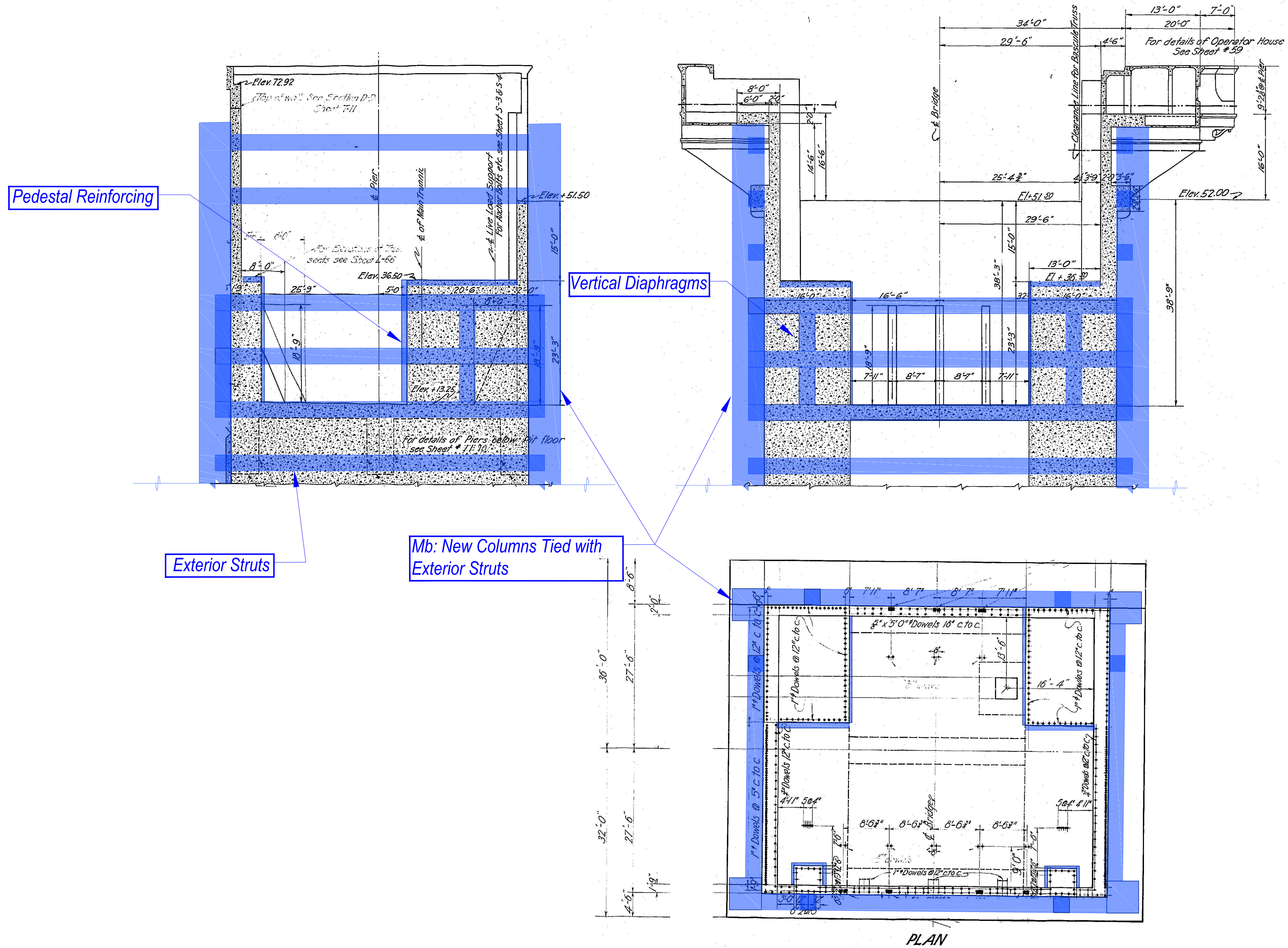

ALT. 4b.1a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT - NO WIDENING

RIVER SPAN PIER II & III -2

DATE: AUGUST 2017

SCALE: AS SHOWN

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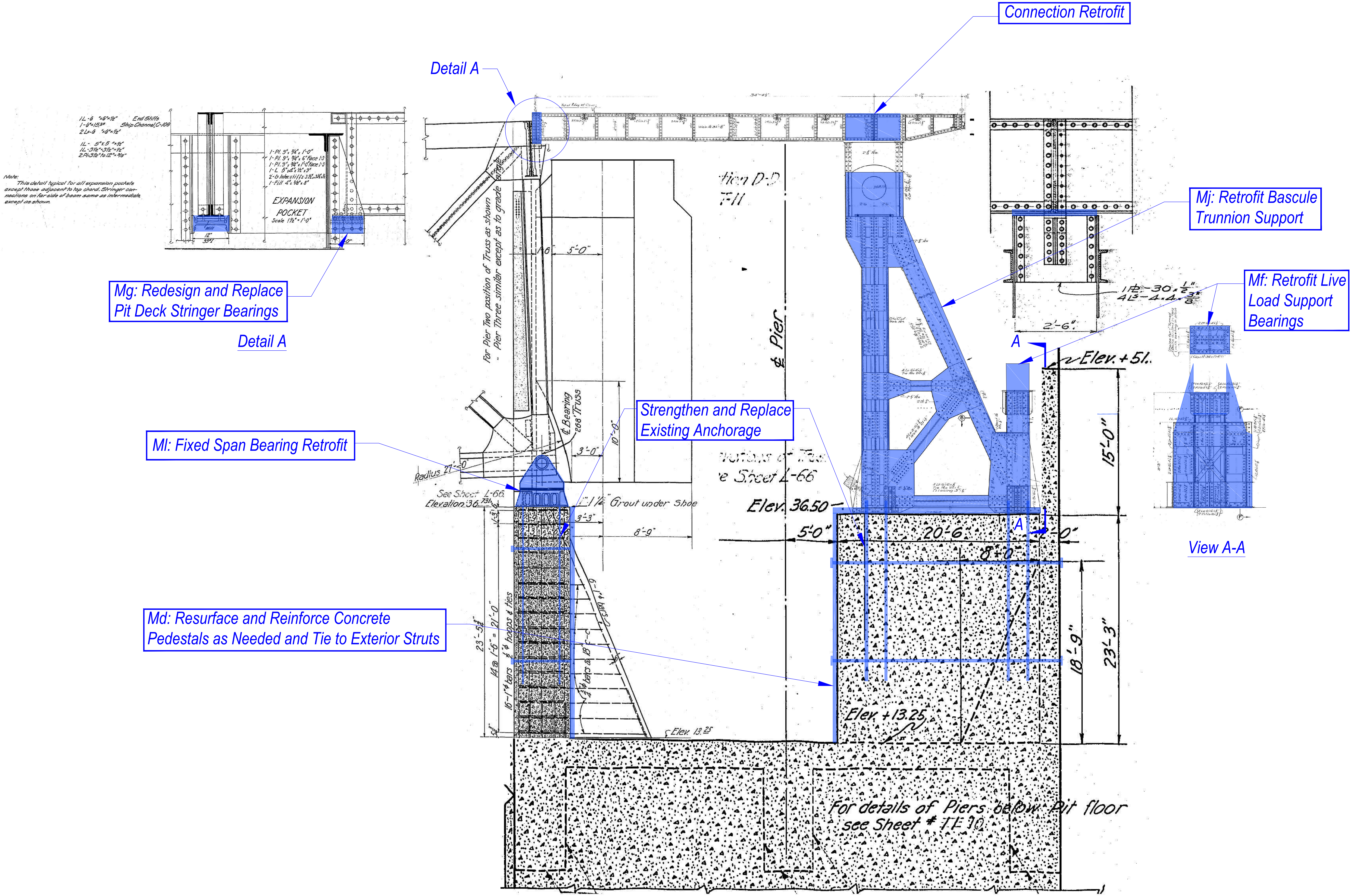
21



ALT. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

TRUNNION SUPPORT & LIVE LOAD SHOES

SCALE: AS SHOWN

DATE: AUGUST 2017

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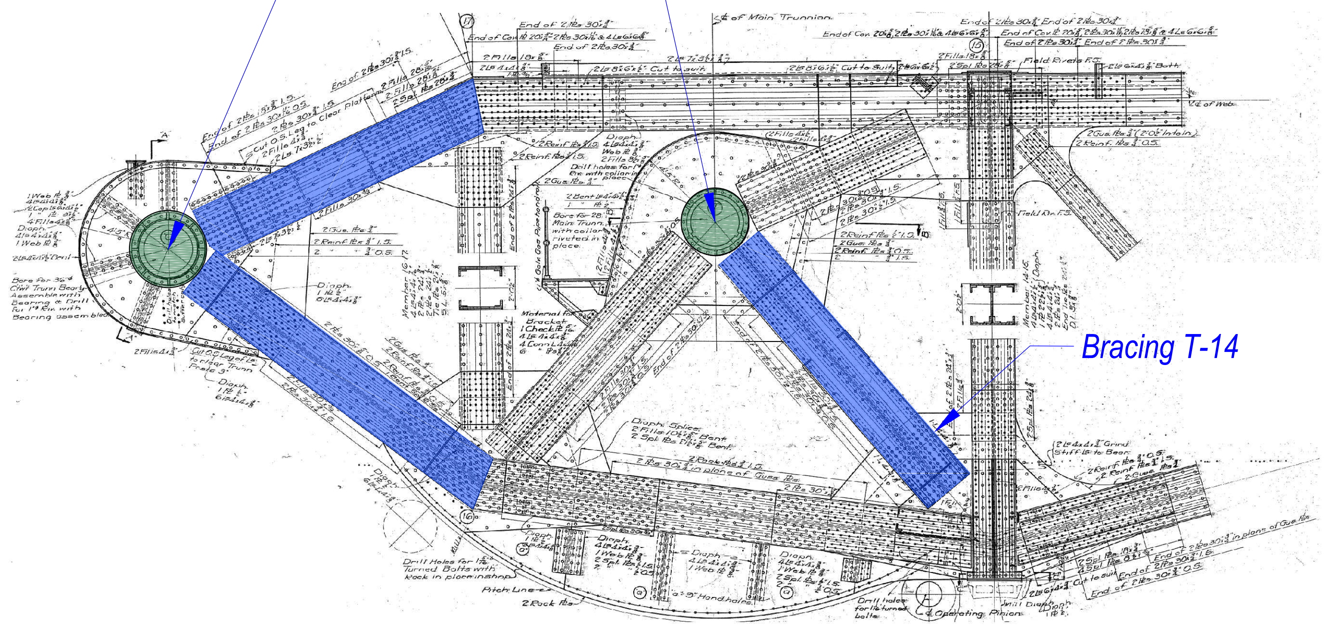
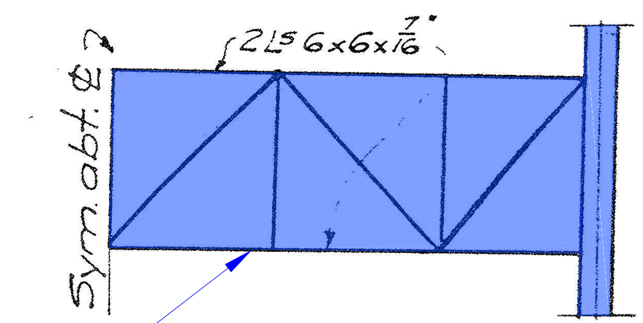
22

Install Lateral Restraints



Bracing T-14



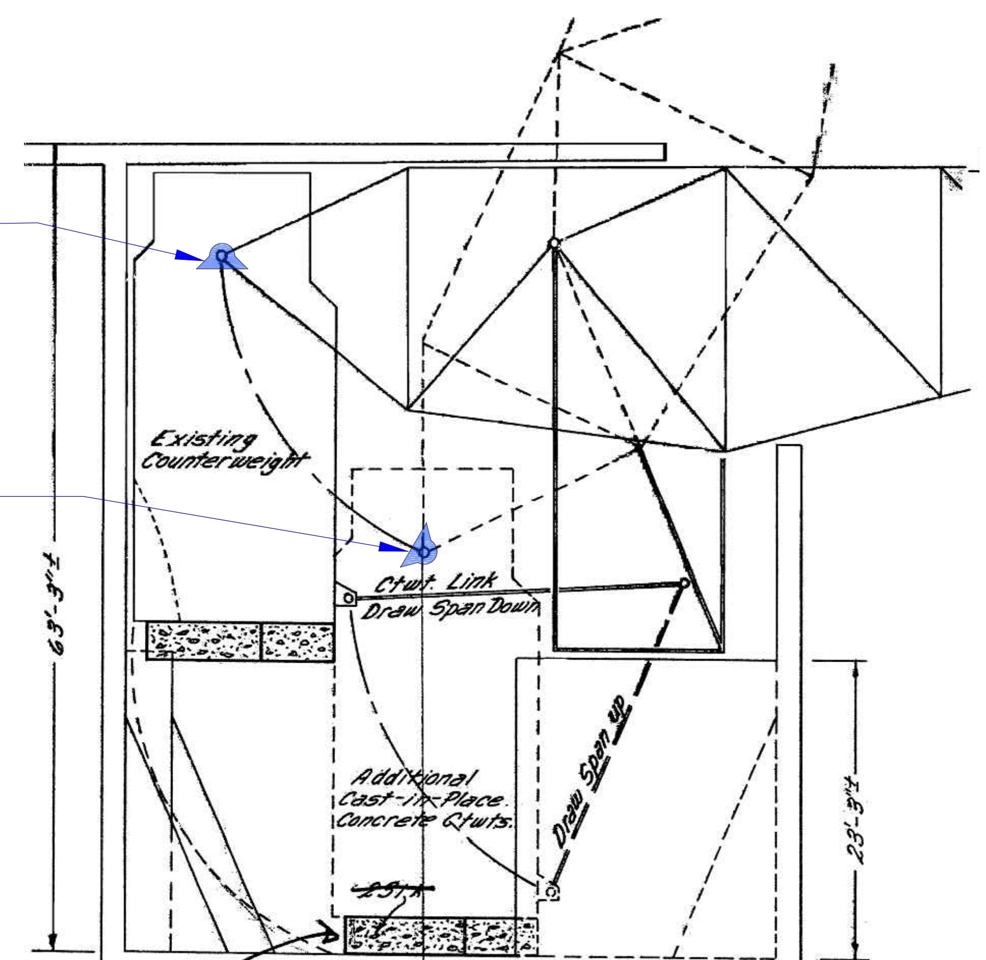
BRACING T-14

Me: Strengthen Steel Members and Lateral Bracing

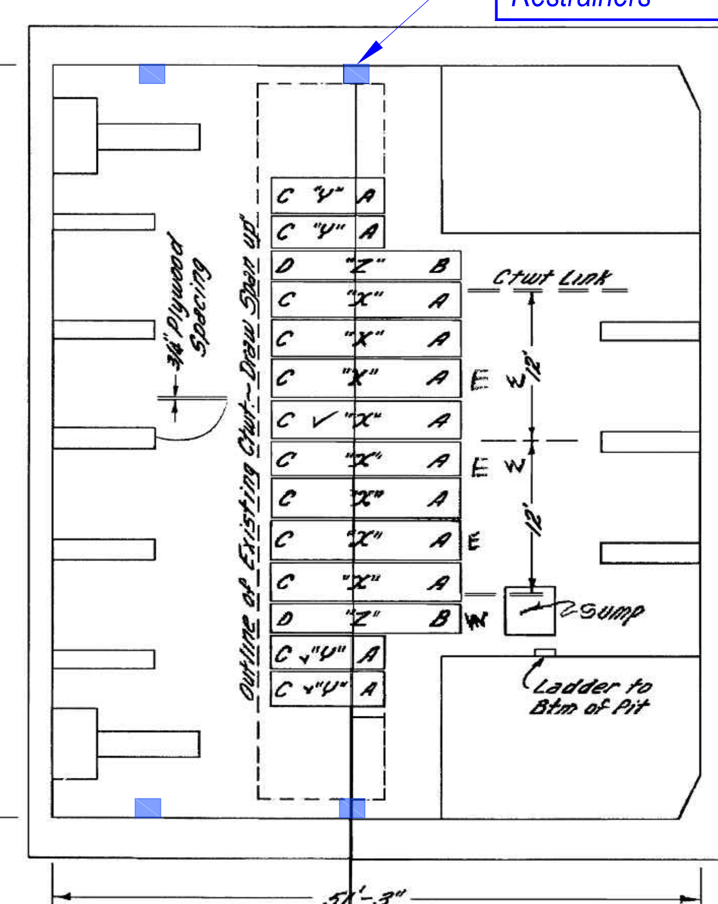
Mi: Install 4 Counterweight Restrainers

Closed Position

OpenPosition



ELEVATION ~ WEST SPAN
(EAST SPAN ~ SIMILAR)



PLAN ~ WEST SPAN

ALT.4b.1a & 2a
EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT + WIDENING
BASCULE SPAN- COUNTERWEIGHT & T.S.

SCALE: AS SHOWN
DATE: AUGUST 2017

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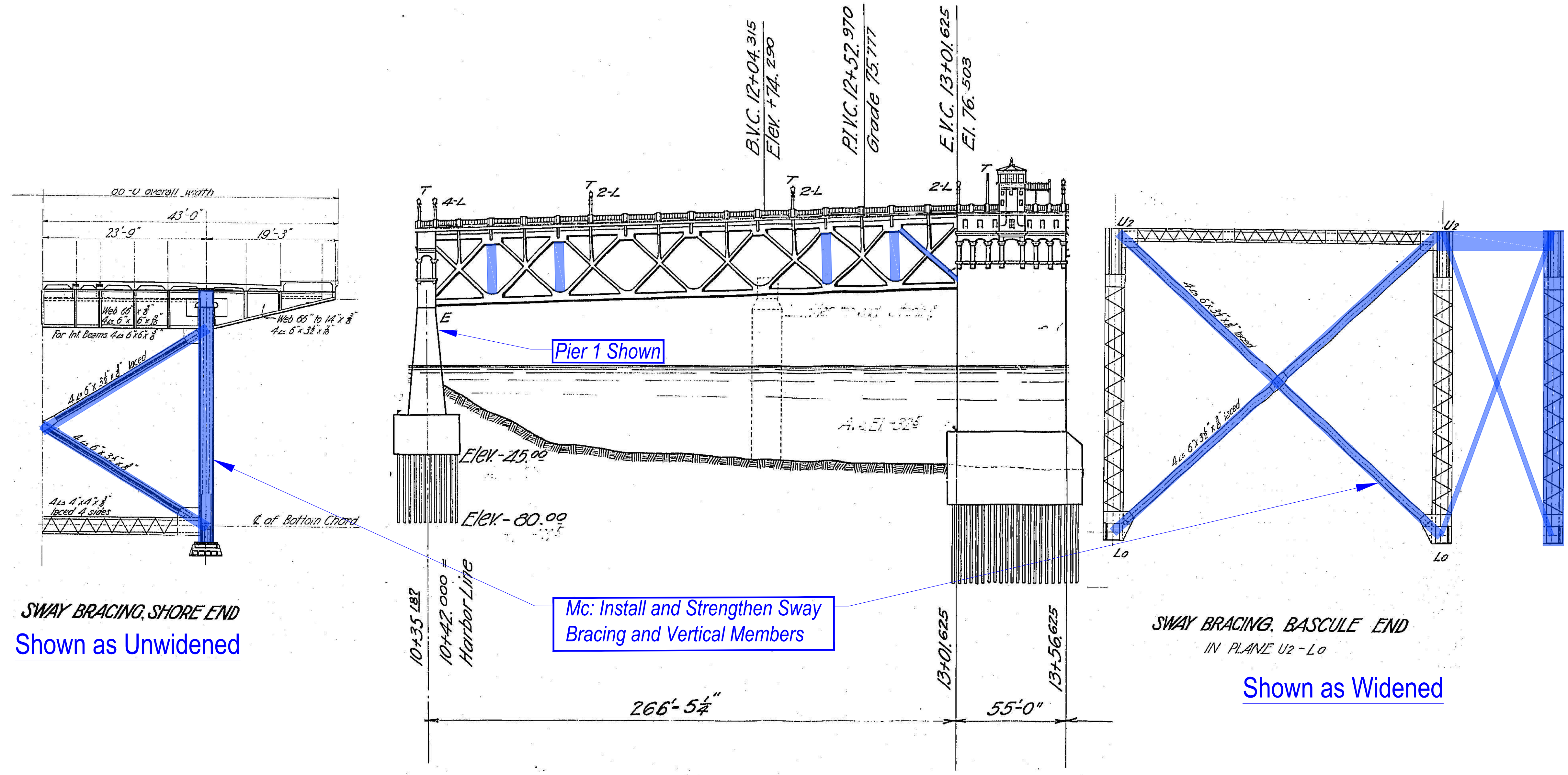

ALT. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

RIVER FIXED STEEL TRUSS SPAN

SCALE: AS SHOWN

DATE: AUGUST 2017

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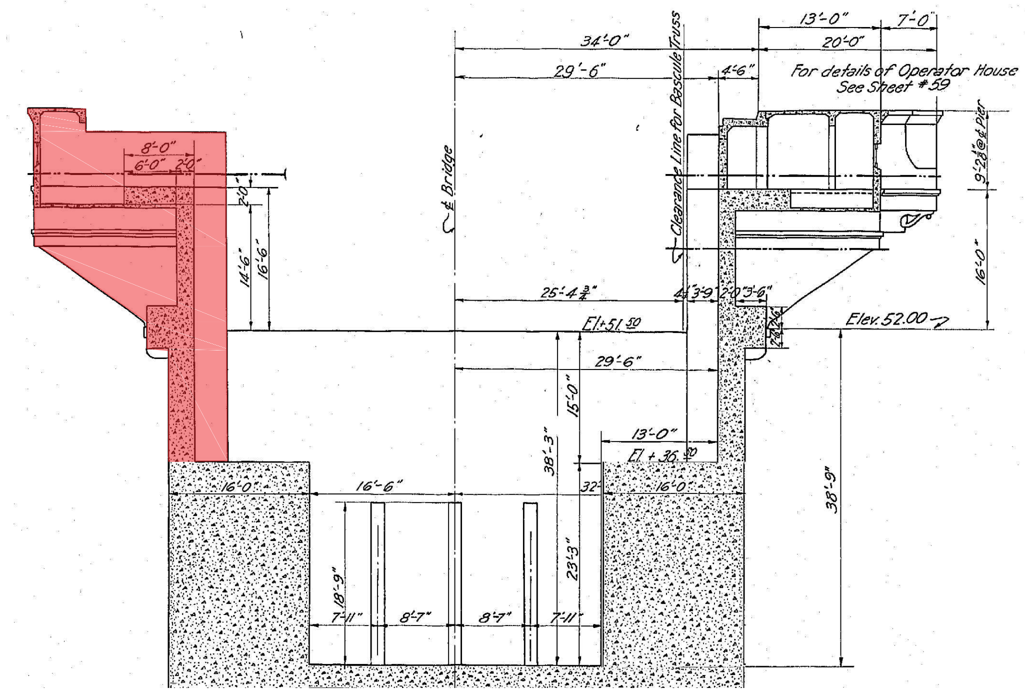
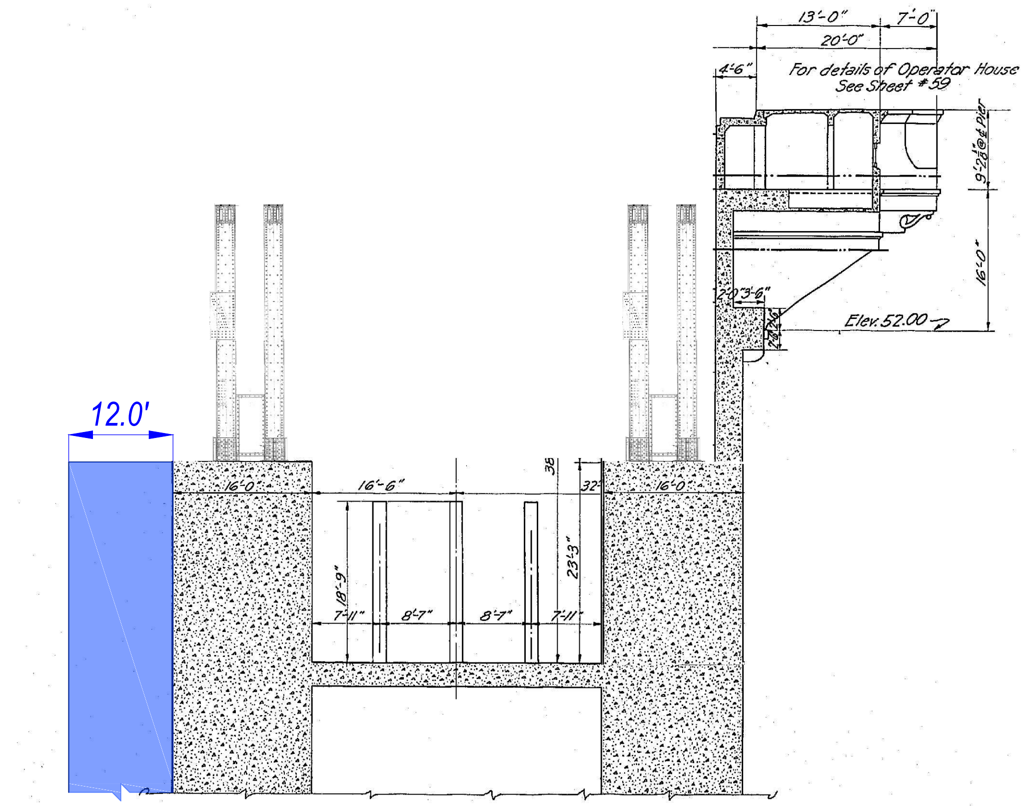
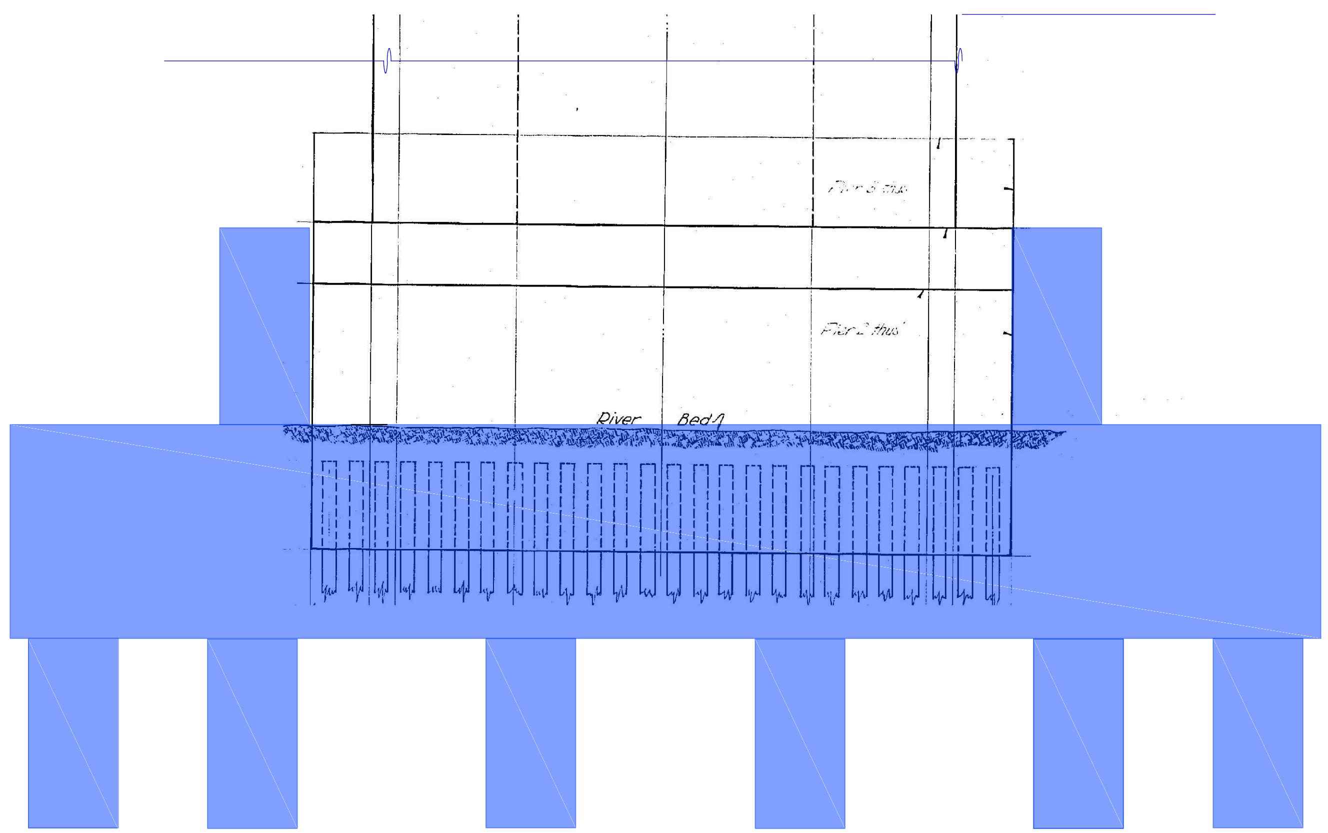


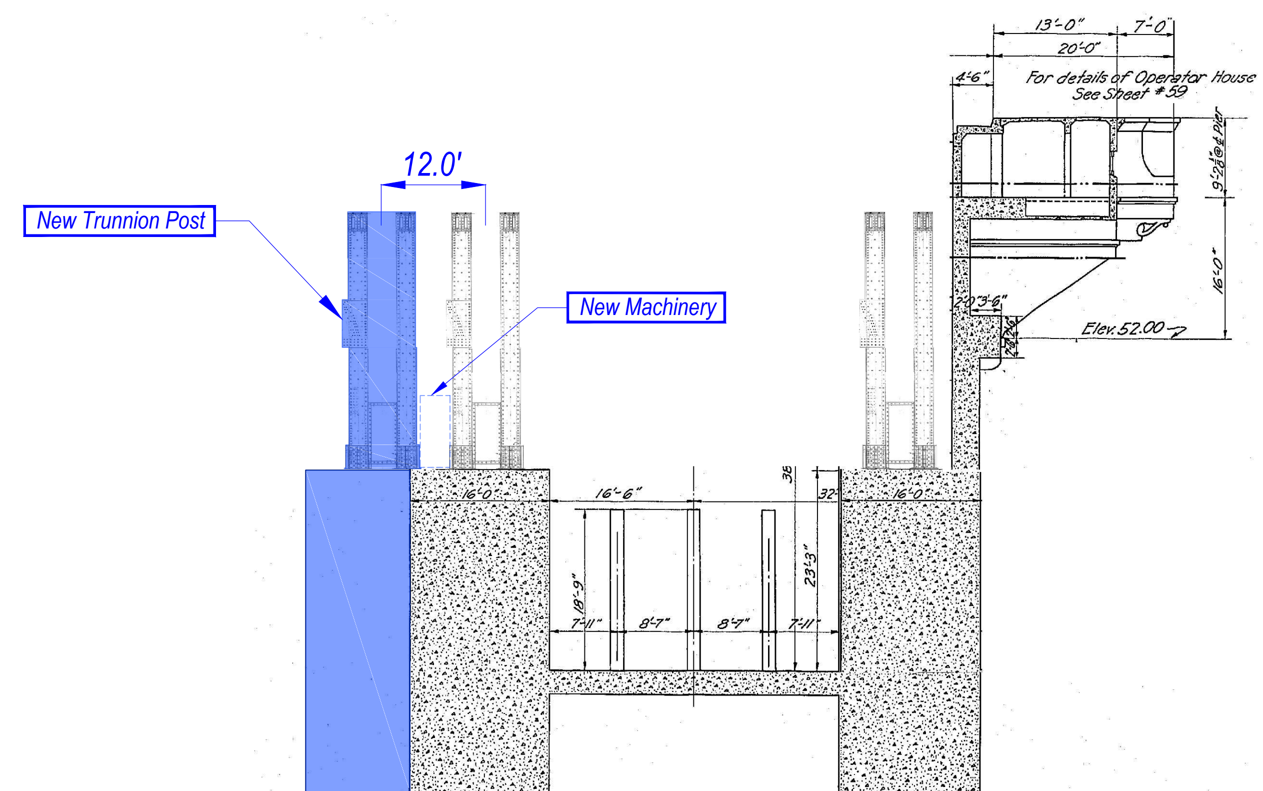
2 Remove North Walls and Sidewalk



3 Widen Northern Wall to Pedestal Level



1 Widen Foundation with Drilled Shafts



4 Install New Machinery and Trunnion Post

ALT. 4b. 2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

BASCULE SPAN CONST SEQUENCE-1

SCALE: AS SHOWN

DATE: AUGUST 2017

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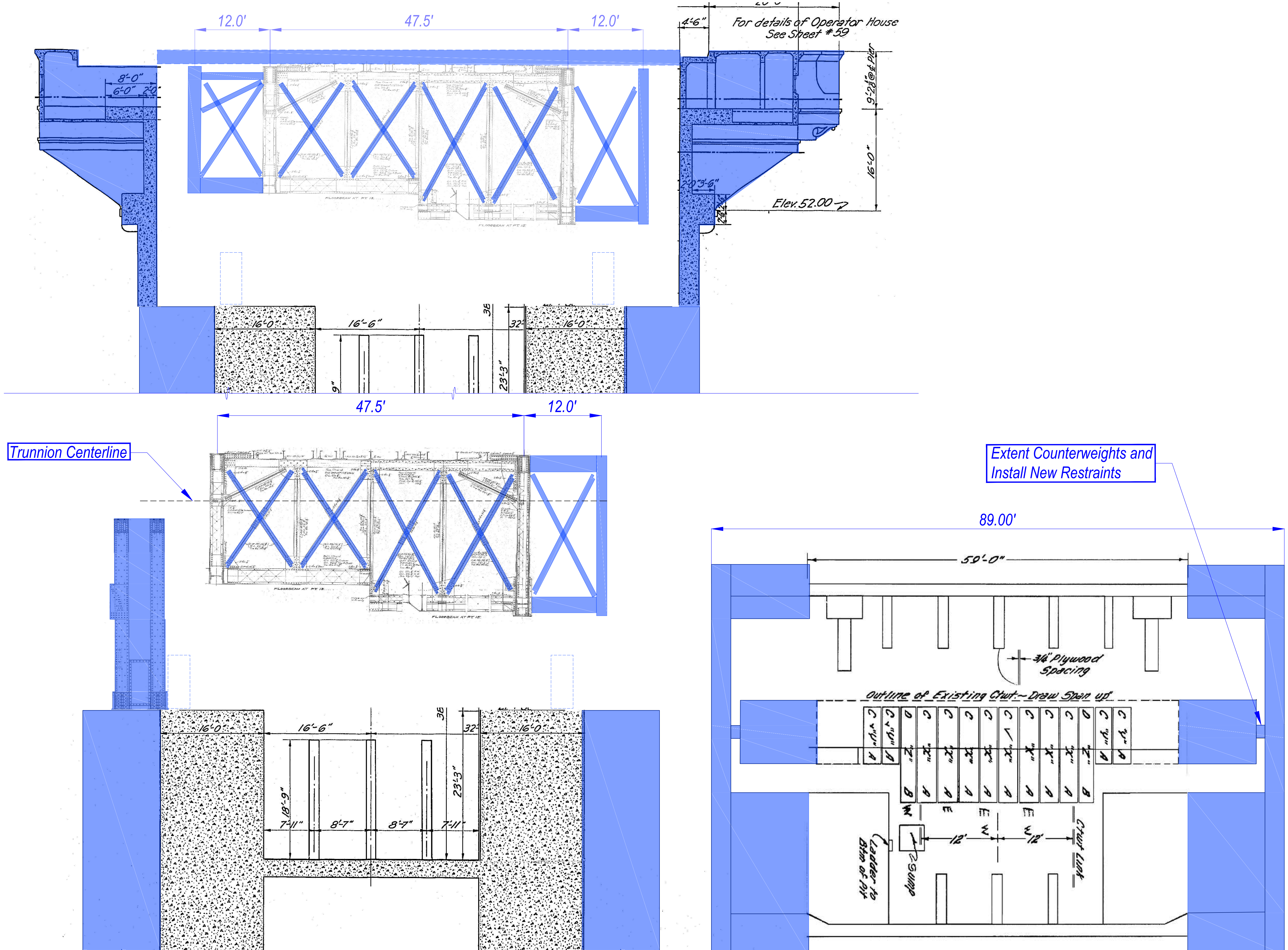

5 Install New Truss Line North Side, Extend Counterweight.

6 - 10 Repeat 1 - 5 for South Side

ALT. 4b.2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

BASCULE SPAN CONST SEQUENCE-2

DATE: AUGUST 2017

SCALE: AS SHOWN

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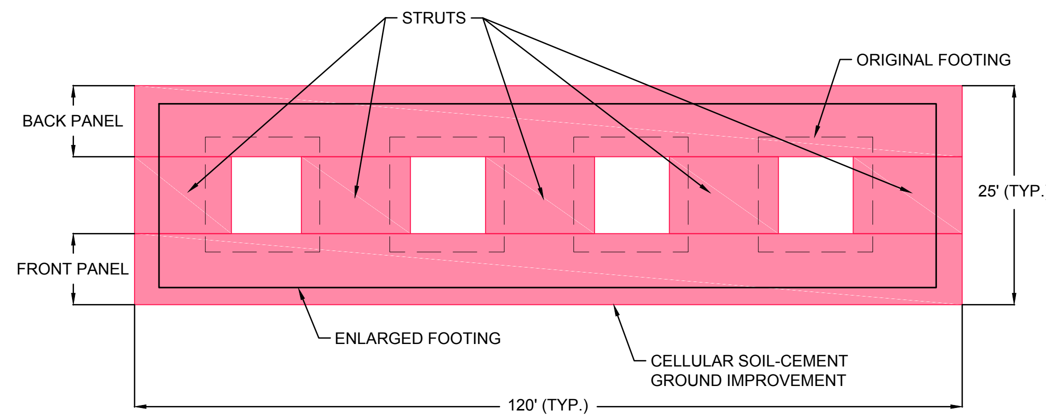
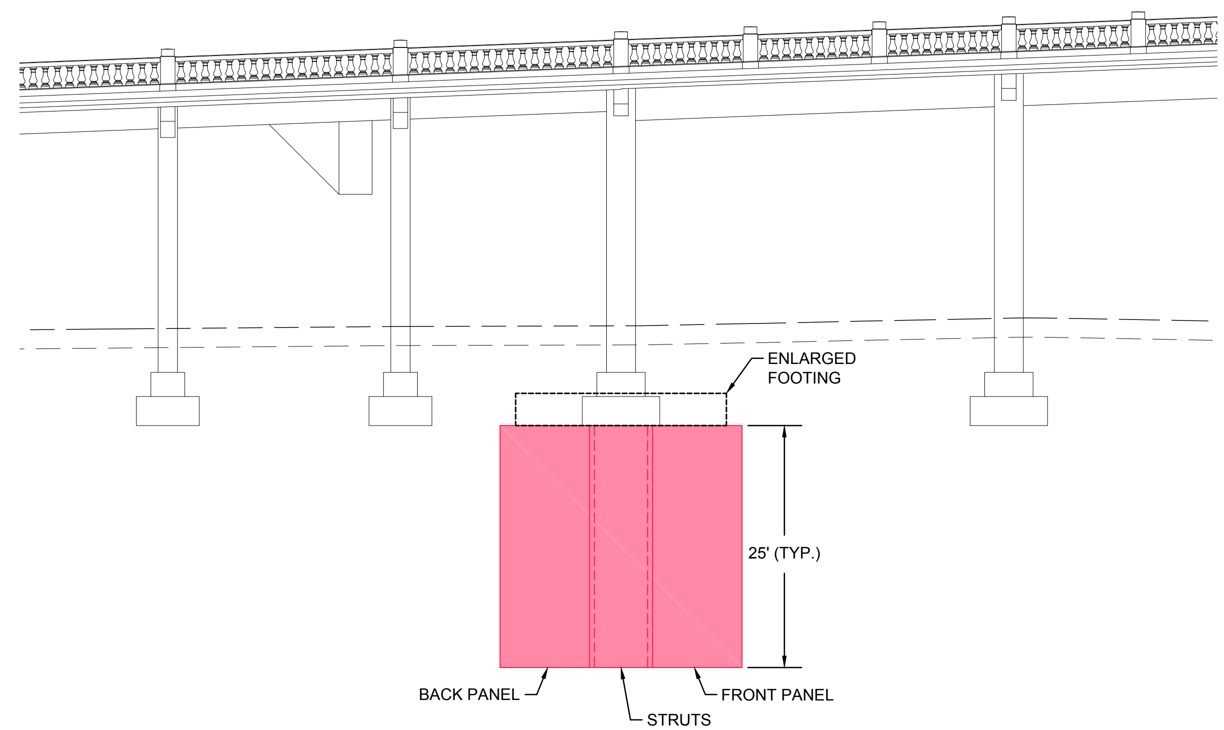


PLAN (TYPICAL AT BENTS 2-16)

SCALE: NOT TO SCALE



PROFILE (TYPICAL AT BENTS 2-16)

SCALE: NOT TO SCALE

LEGEND

APPROXIMATE CELLULAR SOIL-CEMENT GROUND IMPROVEMENT ZONES

ALT. 4b.1a & 2a

EARTHQUAKE READY BURNSIDE BRIDGE

SEISMIC RETROFIT + WIDENING

GROUND IMPROVEMENTS - UNDER SPREAD FOOTINGS

SCALE: NOT TO SCALE

DATE: AUGUST 2017

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CHECKED: RPP

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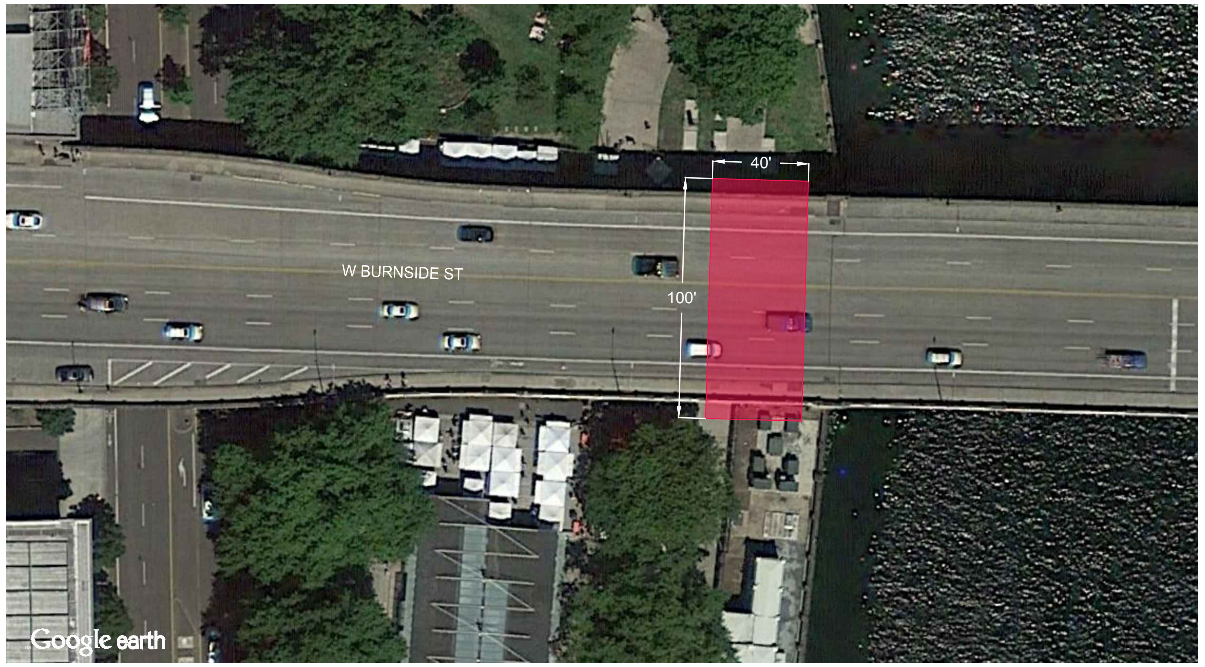
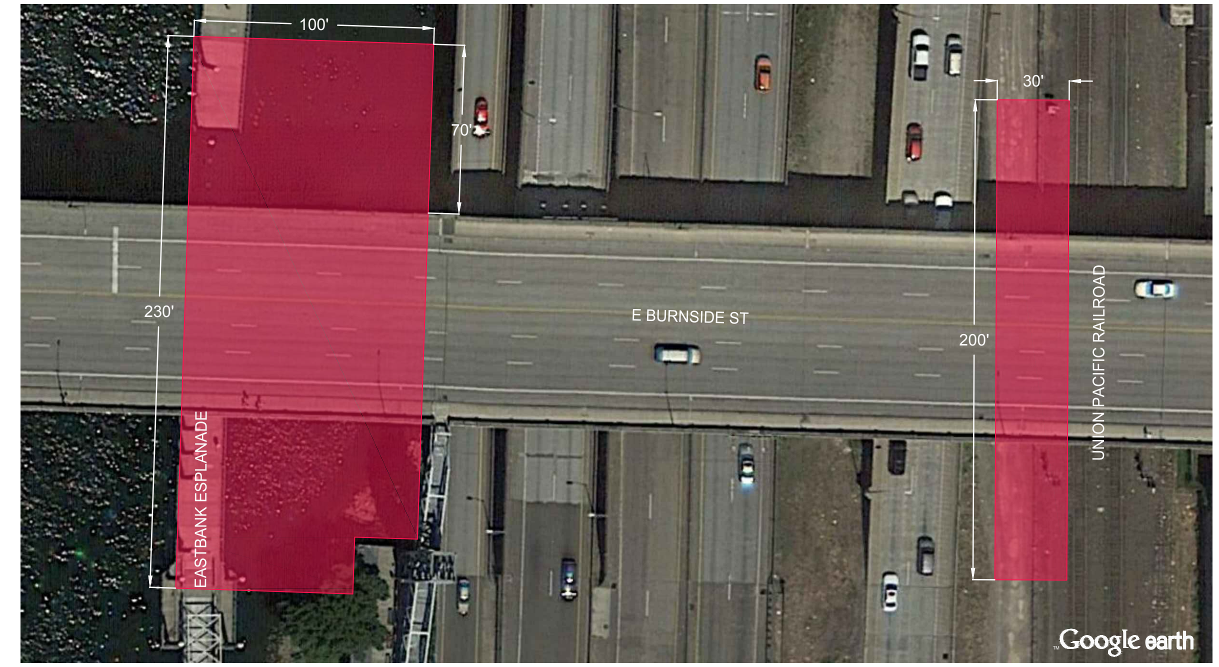
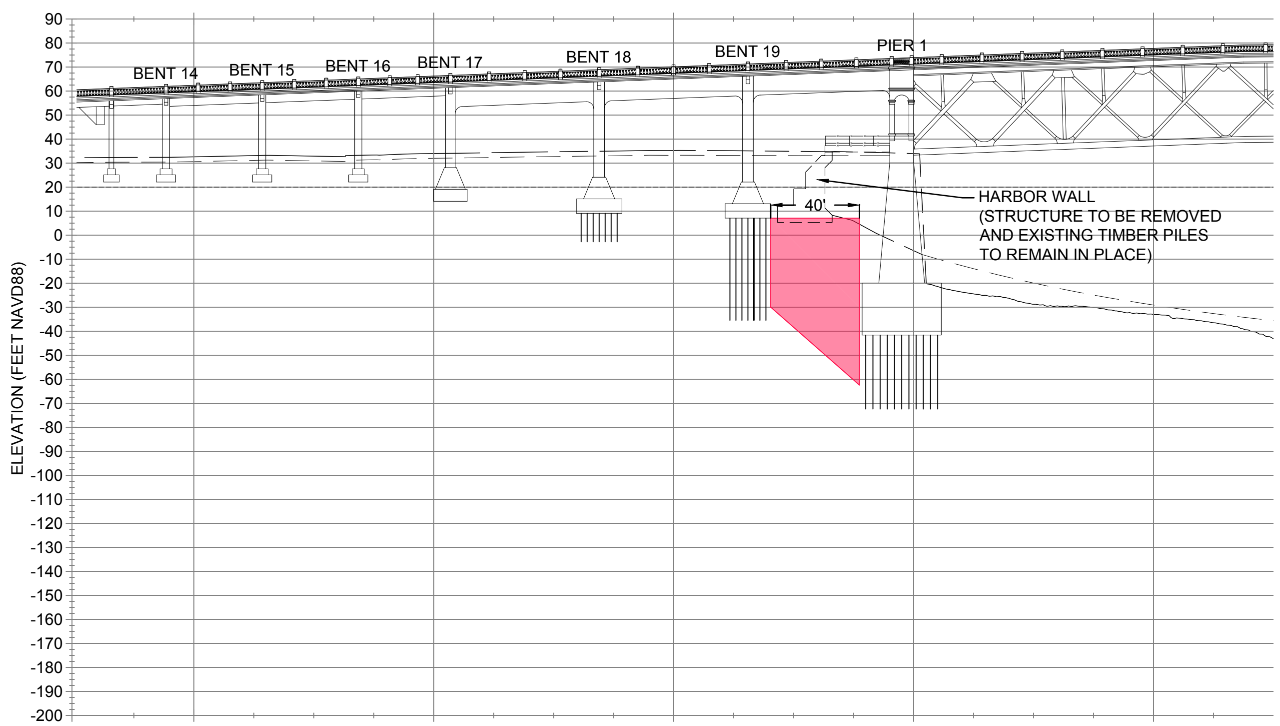
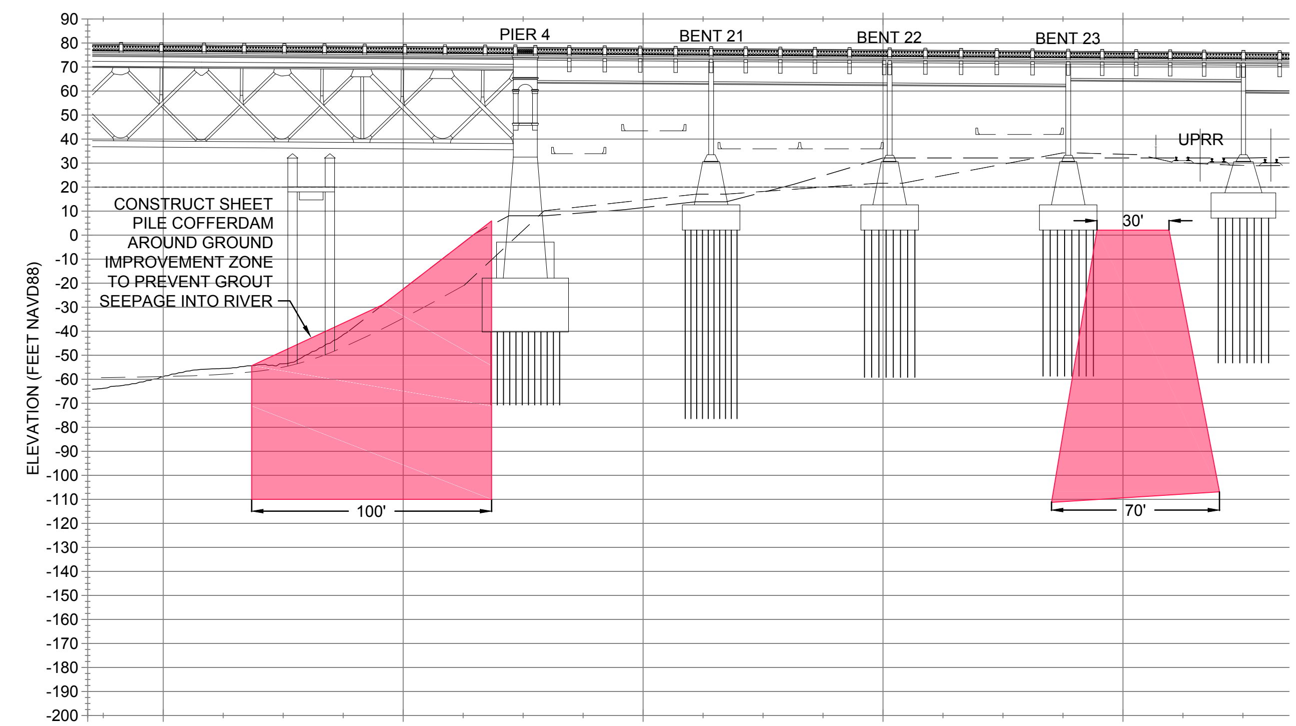




PLAN

SCALE: 1"=40'





PROFILE

SCALE: 1"=40'

LEGEND

APPROXIMATE CELLULAR SOIL-CEMENT GROUND IMPROVEMENT ZONES

GENERAL NOTES:

1. MAP ADAPTED FROM AERIAL IMAGERY PROVIDED BY GOOGLE EARTH PRO, REPRODUCED BY PERMISSION GRANTED BY GOOGLE EARTH™ MAPPING SERVICE.

40 0 40
SCALE IN FEET

ALT. 4b.1a & 2a
EARTHQUAKE READY BURNSIDE BRIDGE
SEISMIC RETROFIT + WIDENING
GROUND IMPROVEMENTS - LATERAL SPREADING MITIGATION

SCALE: AS SHOWN
DATE: AUGUST 2017

MULTNOMAH COUNTY
DEPARTMENT OF COMMUNITY SERVICES
LAND USE AND TRANSPORTATION PROGRAM
1620 S.E. 190th AVE.
PORTLAND, ORE. 97233-5999

BRIAN S. VINCENT P.E. COUNTY ENGINEER

DESIGNED: ECP
DRAWN: AEH
CHECKED: RPP

REVISIONS
DATE:

SHEET NO.
28

I:\EF\24-1 PDX\04000s\04065 Burnside Bridge Seismic Feasibility Study\Graphics\CAD\MultCo 22x34 Conceptual Ground Improvement.dwg Jul 21, 2017 - 1:19pm

Appendix F. EQRB Seismic Retrofit Cost Estimate

BURNSIDE BRIDGE FEASIBILITY-LEVEL COST ESTIMATE WORKSHEET (1)

Rev 7/30

Hybrid Alt 4b.1a (Seismic Retrofit + Partial Replacement) - Replace Spans 20-24

BRIDGE NUMBER	00511, 00511A, 00511B	COUNTY	Multnomah
BRIDGE NAME: Burnside Bridge	STATE HIGHWAY NUMBER: N/A	ROUTE NUMBER: N/A	
Mile Post: Sta 2+72.50 - 25+80.79	SCOPE: Seismic Retrofit & Structural Rehab All Spans Except Replace Spans 20-24 (Unwidened)	REFERENCE NAME/PHONE: Steve Drahota / (503) 423-3712	Date Entered: 8/18/2017; Date Checked: 8/20/2018

No.	ITEM		UNIT	QUANTITY	UNIT COST	SUB TOTAL	Section Totals	Notes
	Site Preparation						$ 36,269,900	
	Mobilization	10.0%	LS	1	$ 23,289,600	$ 23,289,600		
	Temp Erosion & Sediment Control	0.5%	LS	1	$ 1,164,500	$ 1,164,500		
	Temp. Protection and Direction of Traffic	4.0%	LS	1	$ 9,315,800	$ 9,315,800		
	Removal of structure and Obstruction		LS	1	$ 2,500,000	$ 2,500,000		
	Civil/Roadwork						$ 18,021,240	
	Roadway Surface / Earthwork		SY	10,000	$ 100	$ 1,000,000		
	Traffic Signals / Illumination	1.0%	LS	1	$ 2,148,748	$ 2,148,748		
	Site Restoration	2.0%	LS	1	$ 4,297,500	$ 4,297,500		
	Stormwater, Drainage, and Planting	2.00%	LS	1	$ 4,297,496	$ 4,297,496		
	Retaining Walls		SF	22,000	$ 90	$ 1,980,000		
	Utilities	2.00%	LS	1	$ 4,297,496	$ 4,297,496		
	Bridge Structure Retrofit						$ 125,268,051	
	West Approach (00511A)		LS	1	$ 9,838,632	$ 9,838,632		
	Main River Span (00511)		LS	1	$ 92,869,495	$ 92,869,495		
	East Approach (00511B)		LS	1	$ 22,559,924	$ 22,559,924		
	Movable Span Mechanical & Electrical						$ 12,741,000	
	Mechanical System		LS	1	$ 8,031,000	$ 8,031,000		
	Electrical System		LS	1	$ 4,210,000	$ 4,210,000		
	Emergency Backup system		LS	1	$ 500,000	$ 500,000		
	Geotechnical Hazard Mitigation						$ 48,330,578	
	East Approach		LS	1	$ 35,200,000	$ 35,200,000		
	West Approach		LS	1	$ 13,130,578	$ 13,130,578		
	Other Related Items						$ 28,535,166	
	20-Year Bridge Maintenance Needs:							(3)
	Accessibility - BUN-MU-04: Bicycle and Ped Impr. (Feasibility)		LS	0	$ 360,639	$ -		
	Accessibility - BUN-MU-05: Bicycle and Ped Impr. (Const - P1)		LS	0	$ 4,079,927	$ -		
	Accessibility - BUN-MU-06: Bicycle and Ped Impr. (Const - P2)		LS	0	$ 4,080,883	$ -		
	Elect. And Lighting - BUN-MU-01: Submarine Cable Removal		LS	1	$ 1,517,492	$ 1,517,492		(2)
	Structural - BUN-MU-02: Scour Remediation		LS	1	$ 5,575,674	$ 5,575,674		(2)
	Structural - BUN-MU-03: Fender Repair		LS	1	$ 2,592,000	$ 2,592,000		
	Structural - Elimination of Load Rating Deficiencies		LS	1	$ 1,850,000	$ 1,850,000		(4)
	Willamette River Mitigation		LS	1	$ 2,000,000	$ 2,000,000		(4)
	Contractor Access Premium		LS	1	$ 15,000,000	$ 15,000,000		(4)
	Construction Total with Contingency						$ 349,915,735	
	Subtotal (Site Preparation through Other Related Items)					$ 269,165,935		
	Contingencies		30%			$ 80,749,800		
	Right of Way						$ 25,000,000	
	Engineering & Project Delivery						$ 139,470,507	
	NEPA Phase					$ 17,000,000.00		
	PE (Incl. Design, PI, ROW Acquisition)	(PE)	15%			$ 52,487,360.20		
	County Admin. (Oversight, Permits, etc)					$ -		
	Construction Engineering (CEI)	(CEI)	20%			$ 69,983,146.94		
	Total Project Cost before Inflation (2017 $)						$ 514,386,242	
	Const + Design Cumulative Inflation (annual rate)	3.0%	years	10	34.4%	$ 168,307,900		(5)
	ROW Cumulative Inflation (annual rate)	5.0%	years	6	19.4%	$ 4,851,300		(5)
	Total Project Programmatic Cost (2027 $)						$ 687,545,442	

Notes:

(1) This cost sheet only includes roadway improvement costs associated with the scope of work.

(2) From 20-year WR Bridge CIP (Burnside's Allocation from the combined cost with other bridges)

(3) M&E repairs and work over RR lines not included in the 2015 Bridge Maintenance Project have been absorbed by other items

(4) Structural repairs to eliminate the load rating deficiencies established as part of the 2015 Bridge Maintenance Project

(5) Assumes construction mid-point in 2027

WEST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Bent 1		1	LS		$ 229,200.00
Wf	Abutment enhancement	1	EA		$ 229,200.00
	Excavation	82	CY	$ 50.00	$ 4,100.00
	Backfill	17	CY	$ 100.00	$ 1,700.00
	Concrete	126	CY	$ 500.00	$ 63,000.00
	Reinforcement	25200	LB	$ 2.00	$ 50,400.00
	Micropiles	22	EA	$ 5,000.00	$ 110,000.00
Bent 2		1	LS		$ 291,014.00
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 77,000.00
	Steel jacket	15400	LB	$ 5.00	$ 77,000.00
Wd	Footing Enlargement	1	LS		$ 144,500.00
	Excavation	178	CUYD	$ 50.00	$ 8,900.00
	Backfill	100	CUYD	$ 100.00	$ 10,000.00
	Shoring	160	SQYD	$ 200.00	$ 32,000.00
	Concrete	78	CUYD	$ 500.00	$ 39,000.00
	Reinforcement	15600	LB	$ 2.00	$ 31,200.00
	Post-tensioning	3900	LB	$ 6.00	$ 23,400.00
Bent 3-13		11	EA	$ 291,014.00	$ 3,201,154.00
Bent 14		1	LS		$ 459,714.00
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 115,450.00
	Steel jacket	23090	LB	$ 5.00	$ 115,450.00
Wd	Footing Enlargement	1	LS		$ 274,750.00
	Excavation	413	CUYD	$ 50.00	$ 20,650.00
	Backfill	277	CUYD	$ 100.00	$ 27,700.00
	Shoring	316	SQYD	$ 200.00	$ 63,200.00
	Concrete	136	CUYD	$ 500.00	$ 68,000.00
	Reinforcement	27200	LB	$ 2.00	$ 54,400.00
	Post-tensioning	6800	LB	$ 6.00	$ 40,800.00
Bent 15-16		2	EA	$ 459,714.00	$ 919,428.00
Bent 17		1	LS		$ 1,579,374.00

WEST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 141,110.00
	Steel jacket	28222	LB	$ 5.00	$ 141,110.00
We	Foundation Retrofit	1	LS		$ 1,368,750.00
	Excavation	587	CUYD	$ 50.00	$ 29,350.00
	Backfill	74	CUYD	$ 100.00	$ 7,400.00
	Shoring	252	SQYD	$ 200.00	$ 50,400.00
	Concrete	513	CUYD	$ 500.00	$ 256,500.00
	Reinforcement	102600	LB	$ 2.00	$ 205,200.00
	Post-tensioning	25650	LB	$ 6.00	$ 153,900.00
	8' Dia. Drilled Shafts	180	FT	$ 3,700.00	$ 666,000.00
Bent 18-19		2	EA	$ 1,579,374.00	$ 3,158,748.00

Total	**$ 9,838,632.00**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing b

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Pier 1	1	LS		$ 13,165,777.43
Ma Footing enlargement & drilled shafts	1	LS		$ 11,704,595.95
Utility Relocation (30" & 42" Force Mains)	1	LS	$ 200,000.00	$ 200,000.00
Shoring, Cribbing, & Cofferdams	1	LS	$ 3,000,000.00	$ 3,000,000.00
Excavation	5544	CUYD	$ 50.00	$ 277,200.00
Backfill	1358	CUYD	$ 100.00	$ 135,800.00
Drilled Shaft Excavation, 72 Inch Diameter	996	FT	$ 1,500.00	$ 1,494,000.00
Drilled Shaft Concrete	1043	CUYD	$ 1,000.00	$ 1,043,008.76
Drilled Shaft Reinforcement	211209	LB	$ 2.50	$ 528,023.19
Pier cap concrete	3943	CUYD	$ 500.00	$ 1,971,500.00
Pier cap reinforcement	798458	LB	$ 2.00	$ 1,596,915.00
Post-tensioning	159692	LB	$ 6.00	$ 958,149.00
Harbor wall reconstruction	1	LB	$ 500,000.00	$ 500,000.00
				$ -
Mb Pier strengthening	1	LS		$ 1,421,181.48
Base concrete	906	CUYD	$ 500.00	$ 453,055.56
Base reinforcement	183488	LB	$ 2.00	$ 366,975.00
Column concrete	332	CUYD	$ 500.00	$ 165,925.93
Column reinforcement	67200	LB	$ 2.00	$ 134,400.00
Post-tensioning	50138	LB	$ 6.00	$ 300,825.00
				$ -
Mf Truss support bearing retrofit	1	LS		$ 40,000.00
Bearing Replacement	2	EA	$ 20,000.00	$ 40,000.00
				$ -
Pier 2	1	LS		$ 31,317,566.57
Ma Footing enlargement & drilled shafts	1	LS		$ 28,070,266.57
Shoring, Cribbing, & Cofferdams	1	LS	$ 4,200,000.00	$ 4,200,000.00
Excavation	8012	CUYD	$ 50.00	$ 400,592.59
Backfill	1541	CUYD	$ 100.00	$ 154,074.07
Drilled Shaft Excavation, 120 Inch Diameter	980	FT	$ 1,750.00	$ 1,715,000.00
Drilled Shaft Concrete	2851	CUYD	$ 2,250.00	$ 6,414,085.00
Drilled Shaft Reinforcement	577268	LB	$ 2.50	$ 1,443,169.13
Pier cap concrete	11972	CUYD	$ 500.00	$ 5,985,777.78
Pier cap reinforcement	2424240	LB	$ 2.00	$ 4,848,480.00
Post-tensioning	484848	LB	$ 6.00	$ 2,909,088.00
				$ -
Mb Pier wall strengthening	1	LS		$ 800,000.00
Structural Steel	160000	LB	$ 5.00	$ 800,000.00
				$ -
Mb Load transfer columns at pier corners	1	LS		$ 500,000.00
Structural Steel	100000	LB	$ 5.00	$ 500,000.00
				$ -
Md Reinforce Truss Supports at Piers 2 & 3	1	LS		$ 867,300.00
Structural Steel	173460	LB	$ 5.00	$ 867,300.00
				$ -
Mf Truss span bearing retrofit	1	LS		$ 50,000.00
Bearing Replacement	2	EA	$ 25,000.00	$ 50,000.00
				$ -
Mg Pit deck stringer bearings and strengthening	1	LS		$ 195,000.00
Bearings	13	EA	$ 10,000.00	$ 130,000.00
Strengthening	13	EA	$ 5,000.00	$ 65,000.00
				$ -
Mh Install counterweight restrainers	1	LS		$ 60,000.00
Counterweight restrainers	4	EA	$ 15,000.00	$ 60,000.00
				$ -
Mj Trunnion support frame and anchorage strengthening	1	LS		$ 775,000.00
Trunnion support frame structural steel	140000	LB	$ 5.00	$ 700,000.00
Trunnion frame anchorage	1	LS	$ 75,000.00	$ 75,000.00
				$ -
Pier 3	1	LS		$ 31,454,313.06

MAIN RIVER SPAN RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Ma	Footing enlargement & drilled shafts	1	LS		$ 28,207,013.06
	Shoring, Cribbing, & Cofferdams	1	LS	$ 4,200,000.00	$ 4,200,000.00
	Excavation	8012	CUYD	$ 50.00	$ 400,592.59
	Backfill	1541	CUYD	$ 100.00	$ 154,074.07
	Drilled Shaft Excavation, 120 Inch Diameter	994	FT	$ 1,750.00	$ 1,739,500.00
	Drilled Shaft Concrete	2891	CUYD	$ 2,250.00	$ 6,505,714.79
	Drilled Shaft Reinforcement	585514	LB	$ 2.50	$ 1,463,785.83
	Pier cap concrete	11972	CUYD	$ 500.00	$ 5,985,777.78
	Pier cap reinforcement	2424240	LB	$ 2.00	$ 4,848,480.00
	Post-tensioning	484848	LB	$ 6.00	$ 2,909,088.00
					$ -
Mb	Pier wall strengthening	1	LS		$ 800,000.00
	Structural Steel	160000	LB	$ 5.00	$ 800,000.00
					$ -
Mb	Load transfer columns at pier corners	1	LS		$ 500,000.00
	Structural Steel	100000	LB	$ 5.00	$ 500,000.00
Md	Reinforce Truss Supports at Piers 2 & 3	1	LS		$ 867,300.00
	Structural Steel	173460	LB	$ 5.00	$ 867,300.00
					$ -
Mf	Truss span bearing retrofit	1	LS		$ 50,000.00
	Bearing Replacement	2	EA	$ 25,000.00	$ 50,000.00
					$ -
Mg	Pit deck stringer bearings and strengthening	1	LS		$ 195,000.00
	Bearings	13	EA	$ 10,000.00	$ 130,000.00
	Strengthening	13	EA	$ 5,000.00	$ 65,000.00
					$ -
Mh	Install counterweight restrainers	1	LS		$ 60,000.00
	Counterweight restrainers	4	EA	$ 15,000.00	$ 60,000.00
					$ -
Mj	Trunnion support frame and anchorage strengthening	1	LS		$ 775,000.00
	Trunnion support frame structural steel	140000	LB	$ 5.00	$ 700,000.00
	Trunnion frame anchorage	1	LS	$ 75,000.00	$ 75,000.00
					$ -
Pier 4		1	LS		$ 13,864,038.35
Ma	Footing enlargement & drilled shafts	1	LS		$ 12,345,711.98
	Shoring, Cribbing, & Cofferdams	1	LS	$ 3,000,000.00	$ 3,000,000.00
	Excavation	5430	CUYD	$ 50.00	$ 271,496.30
	Backfill	1150	CUYD	$ 100.00	$ 114,962.96
	Drilled Shaft Excavation, 72 Inch Diameter	1260	FT	$ 1,500.00	$ 1,890,000.00
	Drilled Shaft Concrete	1319	CUYD	$ 1,000.00	$ 1,319,468.91
	Drilled Shaft Reinforcement	267192	LB	$ 2.50	$ 667,981.14
	Pier cap concrete	4165	CUYD	$ 500.00	$ 2,082,666.67
	Pier cap reinforcement	843480	LB	$ 2.00	$ 1,686,960.00
	Post-tensioning	168696	LB	$ 6.00	$ 1,012,176.00
	Micro-piles	60	EA	$ 5,000.00	$ 300,000.00
					$ -
Mb	Pier strengthening	1	LS		$ 1,478,326.37
	Base concrete	906	CUYD	$ 500.00	$ 453,055.56
	Base reinforcement	183488	LB	$ 2.00	$ 366,975.00
	Column concrete	382	CUYD	$ 500.00	$ 190,814.81
	Column reinforcement	77280	LB	$ 2.00	$ 154,560.00
	Post-tensioning	52154	LB	$ 6.00	$ 312,921.00
					$ -
Mf	Truss support bearing retrofit	1	LS		$ 40,000.00
	Bearing Replacement	2	EA	$ 20,000.00	$ 40,000.00
					$ -
West Fixed Truss Span		1	LS		$ 200,000.00
Mc	Cross frame strengthening	1	LS		$ 100,000.00
	Structural Steel	20000	LB	$ 5.00	$ 100,000.00

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
				$ -
Mc Adding vertical truss members	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
				$ -
Bascule Leaves	2	LS		$ 2,667,800.00
Me Counterweight support frame strengthening	1	LS		$ 600,000.00
Structural Steel	120000	LB	$ 5.00	$ 600,000.00
				$ -
Mf Live load shoe retrofit	1	LS		$ 100,000.00
Structural steel and bearings shoe seats	4	EA	$ 25,000.00	$ 100,000.00
				$ -
Me Trunnion diaphragm strengthening	1	LS		$ 200,000.00
Structural Steel	40000	LB	$ 5.00	$ 200,000.00
				$ -
Me Cross frame strengthening	1	LS		$ 300,000.00
Structural Steel	60000	LB	$ 5.00	$ 300,000.00
				$ -
Mk Lightweight deck panel replacement	1	LS		$ 604,800.00
Deck replacement	12096	SQFT	$ 50.00	$ 604,800.00
				$ -
Mi Center span lock replacement	1	LS		$ 863,000.00
Structural steel	1	LS	$ 100,000.00	$ 100,000.00
Mechanical lock components	1	LS	$ 763,000.00	$ 763,000.00
				$ -
East Fixed Truss Span	1	LS		$ 200,000.00
Mc Cross frame strengthening	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
				$ -
Mc Adding vertical truss members	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
				$ -

Total	**$ 92,869,495.41**

Totals by Retrofit Designation

Expand caps with additional drilled shafts	Ma	$ 80,327,587.55
Retrofit piers 1 & 2	Mb	$ 5,499,507.85
Additional sway bracing in fixed trusses	Mc	$ 400,000.00
Reinforce truss supports at piers 2 & 3	Md	$ 1,734,600.00
Strengthen steel members and add lateral bracing	Me	$ 1,100,000.00
Strengthen/ replace live load support bearings	Mf	$ 280,000.00
strengthen/ replace deck stringer bearings	Mg	$ 390,000.00
Install counterweight restraints	Mh	$ 120,000.00
Strengthen/ replace center lock	Mi	$ 863,000.00
Retrofit bascule trunnion support	Mj	$ 1,550,000.00
Replace bascule deck with lightweight deck	Mk	$ 604,800.00
	Total	**$ 92,869,495.41**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing bridge.

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Bascule machinery & electrical system	1	LS		$ 12,241,000.00
Ml Machinery	1	LS		$ 8,031,000.00
Operating Machinery Replacement	1	EA	$ 3,665,000.00	$ 3,665,000.00
Rehabilitation of Trunnions, Counterweight Trunnions, and links	4	EA	$ 552,000.00	$ 2,208,000.00
Additional Trunnions and Counterweight Trunnions for Widening	4	EA	$ 527,000.00	$ 2,108,000.00
Span Balance Work	1	EA	$ 50,000.00	$ 50,000.00
				$ -
Mm Electrical	1	LS		$ 4,210,000.00
Replace incoming electrical service from east and west	2	EA	$ 500,000.00	$ 1,000,000.00
Center span lock power feed	1	LS	$ 40,000.00	$ 40,000.00
Replace motors and drives	4	EA	$ 350,000.00	$ 1,400,000.00
Relocate and update PLCs (programming, start-up and commissioning)	1	LS	$ 350,000.00	$ 350,000.00
Replace navigation lighting (pier and span)	8	EA	$ 35,000.00	$ 280,000.00
Replace traffic warning gates	4	EA	$ 135,000.00	$ 540,000.00
Relocating electrical equipment (MCCs, panelboards, networking equipment)	1	LS	$ 600,000.00	$ 600,000.00
				$ -

Total	**$ 12,241,000.00**

Totals by Retrofit Designation

Expand caps with additional drilled shafts	Ma	$ -
Retrofit piers 1 & 2	Mb	$ -
Additional sway bracing in fixed trusses	Mc	$ -
Reinforce truss supports at piers 2 & 3	Md	$ -
Strengthen steel members and add lateral bracing	Me	$ -
Strengthen/ replace live load support bearings	Mf	$ -
strengthen/ replace deck stringer bearings	Mg	$ -
Install counterweight restraints	Mh	$ -
Machinery	Ml	$ 8,031,000.00
Electrical	Mm	$ 4,210,000.00
Replace bascule deck with lightweight deck	Mk	$ -
	Total	**$ 12,241,000.00**

EAST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Spans 20-24		1	LS		$ 10,449,000.00
Ek	Replace with 3 Spans	1	EA		$ 10,449,000.00
	New Structure	34830	SF	$ 300.00	$ 10,449,000.00
Bent 25		1	LS		$ 2,533,910.00
Ea	Superstructure strengthening	1	LS		$ 26,630.00
	Additional Steel	3726	LB	$ 5.00	$ 18,630.00
	Concrete Encasement	4	CY	$ 2,000.00	$ 8,000.00
Eb	Bearing Replacement	1	LS		$ 20,000.00
	Bearing Replacement	2	EA	$ 10,000.00	$ 20,000.00
Ec	Floor beam strengthening	1	LS		$ 11,780.00
	Additional Steel	1556	LB	$ 5.00	$ 7,780.00
	Concrete Encasement	2	CY	$ 2,000.00	$ 4,000.00
Ed	Column Enhancement	1	LS		$ 61,000.00
	Additional Steel	9800	LB	$ 5.00	$ 49,000.00
	Concrete Encasement	6	CY	$ 2,000.00	$ 12,000.00
Ee	Partial Infill Wall	1	LS		$ 66,600.00
	Concrete	74	CY	$ 500.00	$ 37,000.00
	Reinforcement	14800	LB	$ 2.00	$ 29,600.00
Eg	Foundation Retrofit	1	LS		$ 2,347,900.00
	Excavation	794	CUYD	$ 50.00	$ 39,700.00
	Backfill	118	CUYD	$ 100.00	$ 11,800.00
	Shoring	361	SQYD	$ 200.00	$ 72,200.00
	Concrete	676	CUYD	$ 500.00	$ 338,000.00
	Reinforcement	135200	LB	$ 2.00	$ 270,400.00
	Post-tensioning	33800	LB	$ 6.00	$ 202,800.00
	10' Dia. Drilled Shafts	314	FT	$ 4,500.00	$ 1,413,000.00
Bent 26-28		3	EA	$ 2,533,910.00	$ 7,601,730.00
Bent 29		1	LS		$ 291,014.00
Ea	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Ec	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Ed	Column Enhancement	1	LS		$ 77,000.00
	Steel jacket	15400	LB	$ 5.00	$ 77,000.00
Ef	Footing Enlargement	1	LS		$ 144,500.00
	Excavation	178	CUYD	$ 50.00	$ 8,900.00
	Backfill	100	CUYD	$ 100.00	$ 10,000.00
	Shoring	160	SQYD	$ 200.00	$ 32,000.00
	Concrete	78	CUYD	$ 500.00	$ 39,000.00
	Reinforcement	15600	LB	$ 2.00	$ 31,200.00

EAST APPROACH RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Post-tensioning	3900	LB	$ 6.00	$ 23,400.00
Bent 30-34	5	EA	$ 291,014.00	$ 1,455,070.00
Bent 35	1	LS		$ 229,200.00
Eh Abutment enhancement	1	EA		$ 229,200.00
Excavation	82	CY	$ 50.00	$ 4,100.00
Backfill	17	CY	$ 100.00	$ 1,700.00
Concrete	126	CY	$ 500.00	$ 63,000.00
Reinforcement	25200	LB	$ 2.00	$ 50,400.00
Micropiles	22	EA	$ 5,000.00	$ 110,000.00

Total	**$ 22,559,924.00**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing b

Ar = 60%
Total Volume = 49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 1	1	LS		$422,222
Cellular Soil-Cement (CSC) G-I	1	EA		$422,222
Existing Footing Area	1100	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	1	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	1759	CY	n/a	
Treated Volume	1056	CY	$ 400.00	$422,222
Bent 2	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 3	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 4	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 5	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 6	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 7	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333

Ar = 60%
Total Volume = 49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 8	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 9	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 10	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 11	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 12	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 13	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 14	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 15	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778
Bent 16	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778
West Riverbank	1	LS		$2,133,333
Cellular Soil-Cement (CSC) G-I	1	EA		$2,133,333
CSC Area	4000	SF	n/a	
Treated Layer Thickness	60	FT	n/a	
Gross Volume	8889	CY	n/a	
Treated Volume	5333	CY	$ 400.00	$2,133,333
Mob/Demob	1	LS	10%	$1,193,689

Total	**$13,130,578**

Ar =	60%	
Total Volume =	49737	cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
East Riverbank (Primary)	1	LS		$20,444,444
Cellular Soil-Cement (CSC) G-I	1	EA		$20,444,444
CSC Area	23000	SF	n/a	
Treated Layer Thickness	100	FT	n/a	
Gross Volume	85185	CY	n/a	
Treated Volume	51111	CY	$ 400.00	$20,444,444
East Riverbank (Secondary)	1	LS		$11,555,556
Cellular Soil-Cement (CSC) G-I	1	EA		$11,555,556
CSC Area	10000	SF	n/a	
Treated Layer Thickness	120	FT	n/a	
Gross Volume	44444	CY	n/a	
Treated Volume	28889	CY	$ 400.00	$11,555,556
Mob/Demob	1	LS	10%	$3,200,000

Total	**$35,200,000**

Ar (Primary) =	60%	
Ar (Secondary) =	65%	
Total Volume =	129630	cy

Structural - Elimination of Structural Load Rating Deficiencies				
Location and Item	Quantity	Unit	Unit Cost	Total
West Approach	1	LS		$ 150,000.00
Span 17-19 girders	1	LS	$ 150,000.00	$ 150,000.00
Main Span	1	LS		$ 1,180,000.00
Strengthen Fixed Span Stringers	1	LS	$ 1,180,000.00	$ 1,180,000.00
East Approach	1	LS		$ 520,000.00
Span 25 girders	1	LS	$ 320,000.00	$ 320,000.00
Span 26-27 stringers	1	LS	$ 200,000.00	$ 200,000.00

Total	**$ 1,850,000.00**

Structural - Pier Fender Repair/Replacement				
Location and Item	Quantity	Unit	Unit Cost	Total
West Approach		LS		$ -
Work Item				$ -
Sub item				$ -
Main Span	1	LS		$ 2,592,000.00
Pier Fender Replacement	1	LS		$ 2,592,000.00
	960	LF	$ 2,700.00	$ 2,592,000.00
				$ -
East Approach		LS		$ -
Work Item				$ -
Sub item				$ -

Total Replace	**$ 2,592,000.00**
Total Repair	**$ 1,296,000.00**

Assume:
1) Repair cost 50% of replacement
2) Replacement fenders are drilled shafts 10 foot diamenter
3) Four shafts at each bascule pier
4) Depth of shaft is 120 feet based on soil profile fromS&W

BURNSIDE BRIDGE FEASIBILITY-LEVEL COST ESTIMATE WORKSHEET (1)

Rev 7/30

Alt 4b.2a (Seismic Retrofit + Partial Replacement + Widen) - Replace Spans 20-24

BRIDGE NUMBER	00511, 00511A, 00511B	COUNTY	Multnomah
BRIDGE NAME: Burnside Bridge	STATE HIGHWAY NUMBER: N/A	ROUTE NUMBER: N/A	
Mile Post: Sta 2+72.50 - 25+80.79	SCOPE: Seismic Retrofit & Structural Rehab & Widen All Spans Except Replace Spans 20-24 (Widened)	REFERENCE NAME/PHONE: Steve Drahota / (503) 423-3712	Date Entered: 3/20/2018; Date Checked: 8/20/2018

No.	ITEM		UNIT	QUANTITY	UNIT COST	SUB TOTAL	Section Totals	Notes
	Site Preparation						$ 43,384,500	
	Mobilization	10.0%	LS	1	$ 28,196,200	$ 28,196,200		
	Temp Erosion & Sediment Control	0.5%	LS	1	$ 1,409,800	$ 1,409,800		
	Temp. Protection and Direction of Traffic	4.0%	LS	1	$ 11,278,500	$ 11,278,500		
	Removal of structure and Obstruction		LS	1	$ 2,500,000	$ 2,500,000		
	Civil/Roadwork						$ 21,231,126	
	Roadway Surface / Earthwork		SY	10,000	$ 100	$ 1,000,000		
	Traffic Signals / Illumination	1.00%	LS	1	$ 2,607,305	$ 2,607,305		
	Site Restoration	2.0%	LS	1	$ 5,214,600	$ 5,214,600		
	Stormwater, Drainage, and Planting	2.00%	LS	1	$ 5,214,610	$ 5,214,610		
	Retaining Walls		SF	22,000	$ 90	$ 1,980,000		
	Utilities	2.00%	LS	1	$ 5,214,610	$ 5,214,610		
	Bridge Structure Retrofit						$ 169,599,773	
	West Approach (00511A)		LS	1	$ 11,451,420	$ 11,451,420		
	Main River Span (00511)		LS	1	##########	$ 130,548,945		
	East Approach (00511B)		LS	1	$ 27,599,409	$ 27,599,409		
	Movable Span Mechanical & Electrical						$ 14,265,000	
	Mechanical System		LS	1	$ 9,455,000	$ 9,455,000		
	Electrical System		LS	1	$ 4,210,000	$ 4,210,000		
	Emergency Backup System		LS	1	$ 600,000	$ 600,000		
	Geotechnical Hazard Mitigation						$ 48,330,578	
	East Approach		LS	1	$ 35,200,000	$ 35,200,000		
	West Approach		LS	1	$ 13,130,578	$ 13,130,578		
	Other Related Items						$ 28,535,166	
	20-Year Bridge Maintenance Needs:							(3)
	Accessibility - BUN-MU-04: Bicycle and Ped Impr. (Feasibility)		LS	0	$ 360,639	$ -		
	Accessibility - BUN-MU-05: Bicycle and Ped Impr. (Const - P1)		LS	0	$ 4,079,927	$ -		
	Accessibility - BUN-MU-06: Bicycle and Ped Impr. (Const - P2)		LS	0	$ 4,080,883	$ -		
	Elect. And Lighting - BUN-MU-01: Submarine Cable Removal		LS	1	$ 1,517,492	$ 1,517,492		(2)
	Structural - BUN-MU-02: Scour Remediation		LS	1	$ 5,575,674	$ 5,575,674		(2)
	Structural - BUN-MU-03: Fender Repair		LS	1	$ 2,592,000	$ 2,592,000		
	Structural - Elimination of Load Rating Deficiencies		LS	1	$ 1,850,000	$ 1,850,000		(4)
	Willamette River Mitigation		LS	1	$ 2,000,000	$ 2,000,000		(4)
	Contractor Access Premium		LS	1	$ 15,000,000	$ 15,000,000		(4)
	Construction Total with Contingency						$ 422,949,942	
	Subtotal (Site Preparation through Other Related Items)					$ 325,346,142		
	Contingencies		30%			$ 97,603,800		
	Right of Way						$ 45,000,000	
	Engineering & Project Delivery						$ 165,032,480	
	NEPA Phase					$ 17,000,000.00		
	PE (Incl. Design, PI, ROW Acquisition)	(PE)	15%			$ 63,442,491.37		
	County Admin. (Oversight, Permits, etc)							
	Construction Engineering (CEI)	(CEI)	20%			$ 84,589,988.49		
	Total Project Cost before Inflation (2017 $)						$ 632,982,422	
	Const + Design Cumulative Inflation (annual rate)	3.0%	years	10	34.4%	$ 202,216,800		(5)
	ROW Cumulative Inflation (annual rate)	5.0%	years	6	19.4%	$ 8,732,400		(5)
	Total Project Programmatic Cost (2027 $)						$ 843,931,622	

Notes:

(1) This cost sheet only includes roadway improvement costs associated with the scope of work.
(2) From 20-year WR Bridge CIP (Burnside's Allocation from the combined cost with other bridges)
(3) M&E repairs and work over RR lines not included in the 2015 Bridge Maintenance Project have been absorbed by other items
(4) Structural repairs to eliminate the load rating deficiencies established as part of the 2015 Bridge Maintenance Project
(5) Assumes construction mid-point in 2027

WEST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Bent 1		1	LS		$ 229,200.00
Wf	Abutment enhancement	1	EA		$ 229,200.00
	Excavation	82	CY	$ 50.00	$ 4,100.00
	Backfill	17	CY	$ 100.00	$ 1,700.00
	Concrete	126	CY	$ 500.00	$ 63,000.00
	Reinforcement	25200	LB	$ 2.00	$ 50,400.00
	Micropiles	22	EA	$ 5,000.00	$ 110,000.00
Bent 2		1	LS		$ 291,014.00
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 77,000.00
	Steel jacket	15400	LB	$ 5.00	$ 77,000.00
Wd	Footing Enlargement	1	LS		$ 144,500.00
	Excavation	178	CUYD	$ 50.00	$ 8,900.00
	Backfill	100	CUYD	$ 100.00	$ 10,000.00
	Shoring	160	SQYD	$ 200.00	$ 32,000.00
	Concrete	78	CUYD	$ 500.00	$ 39,000.00
	Reinforcement	15600	LB	$ 2.00	$ 31,200.00
	Post-tensioning	3900	LB	$ 6.00	$ 23,400.00
Bent 3-13		11	EA	$ 291,014.00	$ 3,201,154.00
Bent 14		1	LS		$ 651,714.00
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 115,450.00
	Steel jacket	23090	LB	$ 5.00	$ 115,450.00
Wd	Footing Enlargement	1	LS		$ 274,750.00
	Excavation	413	CUYD	$ 50.00	$ 20,650.00
	Backfill	277	CUYD	$ 100.00	$ 27,700.00
	Shoring	316	SQYD	$ 200.00	$ 63,200.00
	Concrete	136	CUYD	$ 500.00	$ 68,000.00
	Reinforcement	27200	LB	$ 2.00	$ 54,400.00
	Post-tensioning	6800	LB	$ 6.00	$ 40,800.00
	Widening				$ 192,000.00
	Superstructure	960	SF	$ 200.00	$ 192,000.00

WEST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Bent 15-16		2	EA	$ 651,714.00	$ 1,303,428.00
Bent 17		1	LS		$ 1,924,969.87
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 141,110.00
	Steel jacket	28222	LB	$ 5.00	$ 141,110.00
We	Foundation Retrofit	1	LS		$ 1,368,750.00
	Excavation	587	CUYD	$ 50.00	$ 29,350.00
	Backfill	74	CUYD	$ 100.00	$ 7,400.00
	Shoring	252	SQYD	$ 200.00	$ 50,400.00
	Concrete	513	CUYD	$ 500.00	$ 256,500.00
	Reinforcement	102600	LB	$ 2.00	$ 205,200.00
	Post-tensioning	25650	LB	$ 6.00	$ 153,900.00
	8' Dia. Drilled Shafts	180	FT	$ 3,700.00	$ 666,000.00
	Widening				$ 345,595.87
	Column Concrete	52	CUYD	$ 700.00	$ 36,651.91
	Column Reinforcement	10472	LBS	$ 2.00	$ 20,943.95
	Superstructure	1440	SF	$ 200.00	$ 288,000.00
Bent 18-19		2	EA	$ 1,924,969.87	$ 3,849,939.73

Total	**$ 11,451,419.60**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing b

MAIN RIVER SPAN RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Pier 1		1	LS		$ 13,281,596.32
Ma	Footing enlargement & drilled shafts	1	LS		$ 11,704,595.95
	Utility Relocation (30" & 42" Force Mains)	1	LS	$ 200,000.00	$ 200,000.00
	Shoring, Cribbing, & Cofferdams	1	LS	$ 3,000,000.00	$ 3,000,000.00
	Excavation	5544	CUYD	$ 50.00	$ 277,200.00
	Backfill	1358	CUYD	$ 100.00	$ 135,800.00
	Drilled Shaft Excavation, 72 Inch Diameter	996	FT	$ 1,500.00	$ 1,494,000.00
	Drilled Shaft Concrete	1043	CUYD	$ 1,000.00	$ 1,043,008.76
	Drilled Shaft Reinforcement	211209	LB	$ 2.50	$ 528,023.19
	Pier cap concrete	3943	CUYD	$ 500.00	$ 1,971,500.00
	Pier cap reinforcement	798458	LB	$ 2.00	$ 1,596,915.00
	Post-tensioning	159692	LB	$ 6.00	$ 958,149.00
	Harbor wall reconstruction	1	LB	$ 500,000.00	$ 500,000.00
Mb	Pier strengthening	1	LS		$ 1,421,181.48
	Base concrete	906	CUYD	$ 500.00	$ 453,055.56
	Base reinforcement	183488	LB	$ 2.00	$ 366,975.00
	Column concrete	332	CUYD	$ 500.00	$ 165,925.93
	Column reinforcement	67200	LB	$ 2.00	$ 134,400.00
	Post-tensioning	50138	LB	$ 6.00	$ 300,825.00
Mf	Truss support bearing retrofit	1	LS		$ 80,000.00
	Bearing Replacement & New Bearings	4	EA	$ 20,000.00	$ 80,000.00
Wd	Widening	1	LS		$ 75,818.89
	Column concrete	84	CUYD	$ 500.00	$ 41,888.89
	Column reinforcement	16965	LB	$ 2.00	$ 33,930.00
Pier 2		1	LS		$ 35,429,033.24
Ma	Footing enlargement & drilled shafts	1	LS		$ 28,070,266.57
	Shoring, Cribbing, & Cofferdams	1	LS	$ 4,200,000.00	$ 4,200,000.00
	Excavation	8012	CUYD	$ 50.00	$ 400,592.59
	Backfill	1541	CUYD	$ 100.00	$ 154,074.07
	Drilled Shaft Excavation, 120 Inch Diameter	980	FT	$ 1,750.00	$ 1,715,000.00
	Drilled Shaft Concrete	2851	CUYD	$ 2,250.00	$ 6,414,085.00
	Drilled Shaft Reinforcement	577268	LB	$ 2.50	$ 1,443,169.13
	Pier cap concrete	11972	CUYD	$ 500.00	$ 5,985,777.78
	Pier cap reinforcement	2424240	LB	$ 2.00	$ 4,848,480.00
	Post-tensioning	484848	LB	$ 6.00	$ 2,909,088.00
Mb	Pier wall strengthening	1	LS		$ 400,000.00
	Structural Steel	80000	LB	$ 5.00	$ 400,000.00
Mb	Load transfer columns at pier corners	1	LS		$ 250,000.00
	Structural Steel	50000	LB	$ 5.00	$ 250,000.00
Md	Reinforce Truss Supports at Piers 2 & 3	1	LS		$ 867,300.00
	Structural Steel	173460	LB	$ 5.00	$ 867,300.00
Mf	Truss span bearing retrofit	1	LS		$ 100,000.00
	Bearing Replacement & New Bearings	4	EA	$ 25,000.00	$ 100,000.00
Mg	Pit deck stringer bearings and strengthening	1	LS		$ 195,000.00
	Bearings	13	EA	$ 10,000.00	$ 130,000.00
	Strengthening	13	EA	$ 5,000.00	$ 65,000.00
Mh	Install counterweight restrainers	1	LS		$ 60,000.00
	Counterweight restrainers	4	EA	$ 15,000.00	$ 60,000.00
Mj	Trunnion support frame and anchorage strengthening	1	LS		$ 775,000.00
	Trunnion support frame structural steel	140000	LB	$ 5.00	$ 700,000.00
	Trunnion frame anchorage	1	LS	$ 75,000.00	$ 75,000.00

MAIN RIVER SPAN RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Wd	Widening	1	LS		$ 4,711,466.67
	Concrete - Below Pedestal	2933	CUYD	$ 500.00	$ 1,466,666.67
	Concrete - Above Pedestal	560	CUYD	$ 500.00	$ 280,000.00
	Concrete reinforcement	707400	LB	$ 2.00	$ 1,414,800.00
	Sidewalk, railing, operator house	1	LS	$ 250,000.00	$ 250,000.00
	New trunnion posts	260000	LB	$ 5.00	$ 1,300,000.00
Pier 3		1	LS		$ 35,565,779.72
Ma	Footing enlargement & drilled shafts	1	LS		$ 28,207,013.06
	Shoring, Cribbing, & Cofferdams	1	LS	$ 4,200,000.00	$ 4,200,000.00
	Excavation	8012	CUYD	$ 50.00	$ 400,592.59
	Backfill	1541	CUYD	$ 100.00	$ 154,074.07
	Drilled Shaft Excavation, 120 Inch Diameter	994	FT	$ 1,750.00	$ 1,739,500.00
	Drilled Shaft Concrete	2891	CUYD	$ 2,250.00	$ 6,505,714.79
	Drilled Shaft Reinforcement	585514	LB	$ 2.50	$ 1,463,785.83
	Pier cap concrete	11972	CUYD	$ 500.00	$ 5,985,777.78
	Pier cap reinforcement	2424240	LB	$ 2.00	$ 4,848,480.00
	Post-tensioning	484848	LB	$ 6.00	$ 2,909,088.00
Mb	Pier wall strengthening	1	LS		$ 400,000.00
	Structural Steel	80000	LB	$ 5.00	$ 400,000.00
Mb	Load transfer columns at pier corners	1	LS		$ 250,000.00
	Structural Steel	50000	LB	$ 5.00	$ 250,000.00
Md	Reinforce Truss Supports at Piers 2 & 3	1	LS		$ 867,300.00
	Structural Steel	173460	LB	$ 5.00	$ 867,300.00
Mf	Truss span bearing retrofit	1	LS		$ 100,000.00
	Bearing Replacement & New Bearings	4	EA	$ 25,000.00	$ 100,000.00
Mg	Pit deck stringer bearings and strengthening	1	LS		$ 195,000.00
	Bearings	13	EA	$ 10,000.00	$ 130,000.00
	Strengthening	13	EA	$ 5,000.00	$ 65,000.00
Mh	Install counterweight restrainers	1	LS		$ 60,000.00
	Counterweight restrainers	4	EA	$ 15,000.00	$ 60,000.00
Mj	Trunnion support frame and anchorage strengthening	1	LS		$ 775,000.00
	Trunnion support frame structural steel	140000	LB	$ 5.00	$ 700,000.00
	Trunnion frame anchorage	1	LS	$ 75,000.00	$ 75,000.00
Wd	Widening	1	LS		$ 4,711,466.67
	Concrete - Below Pedestal	2933	CUYD	$ 500.00	$ 1,466,666.67
	Concrete - Above Pedestal	560	CUYD	$ 500.00	$ 280,000.00
	Concrete reinforcement	707400	LB	$ 2.00	$ 1,414,800.00
	Sidewalk, railing, operator house	1	LS	$ 250,000.00	$ 250,000.00
	New trunnion posts	260000	LB	$ 5.00	$ 1,300,000.00
Pier 4		1	LS		$ 13,991,186.50
Ma	Footing enlargement & drilled shafts	1	LS		$ 12,345,711.98
	Shoring, Cribbing, & Cofferdams	1	LS	$ 3,000,000.00	$ 3,000,000.00
	Excavation	5430	CUYD	$ 50.00	$ 271,496.30
	Backfill	1150	CUYD	$ 100.00	$ 114,962.96
	Drilled Shaft Excavation, 72 Inch Diameter	1260	FT	$ 1,500.00	$ 1,890,000.00
	Drilled Shaft Concrete	1319	CUYD	$ 1,000.00	$ 1,319,468.91
	Drilled Shaft Reinforcement	267192	LB	$ 2.50	$ 667,981.14
	Pier cap concrete	4165	CUYD	$ 500.00	$ 2,082,666.67
	Pier cap reinforcement	843480	LB	$ 2.00	$ 1,686,960.00
	Post-tensioning	168696	LB	$ 6.00	$ 1,012,176.00
	Micro-piles	60	EA	$ 5,000.00	$ 300,000.00

MAIN RIVER SPAN RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Mb	Pier strengthening	1	LS		$ 1,478,326.37
	Base concrete	906	CUYD	$ 500.00	$ 453,055.56
	Base reinforcement	183488	LB	$ 2.00	$ 366,975.00
	Column concrete	382	CUYD	$ 500.00	$ 190,814.81
	Column reinforcement	77280	LB	$ 2.00	$ 154,560.00
	Post-tensioning	52154	LB	$ 6.00	$ 312,921.00
Mf	Truss support bearing retrofit	1	LS		$ 80,000.00
	Bearing Replacement & New Bearings	4	EA	$ 20,000.00	$ 80,000.00
Wd	Widening	1	LS		$ 87,148.15
	Column concrete	96	CUYD	$ 500.00	$ 48,148.15
	Column reinforcement	19500	LB	$ 2.00	$ 39,000.00
West Fixed Truss Span		1	LS		$ 9,172,070.93
Mc	Cross frame strengthening	1	LS		$ 100,000.00
	Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Mc	Adding vertical truss members	1	LS		$ 100,000.00
	Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Wd	Widening	1	LS		$ 8,972,070.93
	Structural Steel - new trusses & bracing	1760000	LB	$ 5.00	$ 8,800,000.00
	Concrete - deck and sidewalk	263	CUYD	$ 250.00	$ 65,675.93
	Concrete reinforcement	53198	LB	$ 2.00	$ 106,395.00
Bascule Leaves		2	LS		$ 13,937,207.16
Me	Counterweight support frame strengthening	1	LS		$ 600,000.00
	Structural Steel	120000	LB	$ 5.00	$ 600,000.00
Mf	Live load shoe retrofit	1	LS		$ 100,000.00
	Structural steel and bearings shoe seats	4	EA	$ 25,000.00	$ 100,000.00
Me	Trunnion diaphragm strengthening	1	LS		$ 200,000.00
	Structural Steel	40000	LB	$ 5.00	$ 200,000.00
Me	Cross frame strengthening	1	LS		$ 300,000.00
	Structural Steel	60000	LB	$ 5.00	$ 300,000.00
Mk	Lightweight deck panel replacement	1	LS		$ 604,800.00
	Deck replacement	12096	SQFT	$ 50.00	$ 604,800.00
Mi	Center span lock replacement	1	LS		$ 939,300.00
	Structural steel	1	LS	$ 100,000.00	$ 100,000.00
	Mechanical lock components	1	LS	$ 839,300.00	$ 839,300.00
Wd	Widening	1	LS		$ 11,193,107.16
	Structural Steel - new trusses & bracing	2080000	LB	$ 5.00	$ 10,400,000.00
	Lightweight decking	5112	sqft	$ 50.00	$ 255,600.00
	concrete sidewalk and railing	84	CUYD	$ 300.00	$ 25,244.44
	concrete reinforcement	17040	LB	$ 2.00	$ 34,080.00
	Concrete - counterweight	598	CUYD	$ 800.00	$ 478,182.72
East Fixed Truss Span		1	LS		$ 9,172,070.93
Mc	Cross frame strengthening	1	LS		$ 100,000.00
	Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Mc	Adding vertical truss members	1	LS		$ 100,000.00
	Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Wd	Widening	1	LS		$ 8,972,070.93

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Structural Steel - new trusses & bracing	1760000	LB	$ 5.00	$ 8,800,000.00
Concrete - deck and sidewalk	263	CUYD	$ 250.00	$ 65,675.93
Concrete reinforcement	53198	LB	$ 2.00	$ 106,395.00

Total	**$ 130,548,944.79**

Totals by Retrofit Designation

Expand caps with additional drilled shafts	Ma	$ 80,327,587.55
Retrofit piers 1 & 2	Mb	$ 4,199,507.85
Additional sway bracing in fixed trusses	Mc	$ 400,000.00
Reinforce truss supports at piers 2 & 3	Md	$ 1,734,600.00
Strengthen steel members and add lateral bracing	Me	$ 1,100,000.00
Strengthen/ replace live load support bearings	Mf	$ 460,000.00
strengthen/ replace deck stringer bearings	Mg	$ 390,000.00
Install counterweight restraints	Mh	$ 120,000.00
Strengthen/ replace center lock	Mi	$ 939,300.00
Retrofit bascule trunnion support	Mj	$ 1,550,000.00
Replace bascule deck with lightweight deck	Mk	$ 604,800.00
Work associated with widening	Wd	$ 38,723,149.38
	Total	**$ 130,548,944.79**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing bridge.

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Bascule machinery & electrical system	1	LS		$ 13,665,000.00
Ml Machinery	1	LS		$ 9,455,000.00
Operating Machinery Replacement	1	EA	$ 3,665,000.00	$ 3,665,000.00
Rehabilitation of Trunnions, Counterweight Trunnions, and links	4	EA	$ 552,000.00	$ 2,208,000.00
Additional Trunnions and Counterweight Trunnions for Widening	4	EA	$ 527,000.00	$ 2,108,000.00
Span Balance Work	1	EA	$ 50,000.00	$ 50,000.00
Operating Machinery Replacement for Widening (Additional Cost)	1	EA	$ 1,424,000.00	$ 1,424,000.00
Mm Electrical	1	LS		$ 4,210,000.00
Replace incoming electrical service from east and west	2	EA	$ 500,000.00	$ 1,000,000.00
Center span lock power feed	1	LS	$ 40,000.00	$ 40,000.00
Replace motors and drives	4	EA	$ 350,000.00	$ 1,400,000.00
Relocate and update PLCs (programming, start-up and commissioning)	1	LS	$ 350,000.00	$ 350,000.00
Replace navigation lighting (pier and span)	8	EA	$ 35,000.00	$ 280,000.00
Replace traffic warning gates	4	EA	$ 135,000.00	$ 540,000.00
Relocating electrical equipment (MCCs, panelboards, networking equipment)	1	LS	$ 600,000.00	$ 600,000.00

Total	**$ 13,665,000.00**

Totals by Retrofit Designation

Expand caps with additional drilled shafts	Ma	$ -
Retrofit piers 1 & 2	Mb	$ -
Additional sway bracing in fixed trusses	Mc	$ -
Reinforce truss supports at piers 2 & 3	Md	$ -
Strengthen steel members and add lateral bracing	Me	$ -
Strengthen/ replace live load support bearings	Mf	$ -
strengthen/ replace deck stringer bearings	Mg	$ -
Install counterweight restraints	Mh	$ -
Machinery	Ml	$ 9,455,000.00
Electrical	Mm	$ 4,210,000.00
Replace bascule deck with lightweight deck	Mk	$ -
Work associated with widening	Wd	$ -
	Total	**$ 13,665,000.00**

EAST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Spans 20-24		1	LS		$ 13,365,000.00
Ek	Replace with 3 Spans	1	EA		$ 13,365,000.00
	New Structure	44550	SF	$ 300.00	$ 13,365,000.00
Bent 25		1	LS		$ 3,064,781.18
Ea	Superstructure strengthening	1	LS		$ 26,630.00
	Additional Steel	3726	LB	$ 5.00	$ 18,630.00
	Concrete Encasement	4	CY	$ 2,000.00	$ 8,000.00
Eb	Bearing Replacement	1	LS		$ 20,000.00
	Bearing Replacement	2	EA	$ 10,000.00	$ 20,000.00
Ec	Floor beam strengthening	1	LS		$ 11,780.00
	Additional Steel	1556	LB	$ 5.00	$ 7,780.00
	Concrete Encasement	2	CY	$ 2,000.00	$ 4,000.00
Ed	Column Enhancement	1	LS		$ 61,000.00
	Additional Steel	9800	LB	$ 5.00	$ 49,000.00
	Concrete Encasement	6	CY	$ 2,000.00	$ 12,000.00
Ee	Partial Infill Wall	1	LS		$ 66,600.00
	Concrete	74	CY	$ 500.00	$ 37,000.00
	Reinforcement	14800	LB	$ 2.00	$ 29,600.00
Eg	Foundation Retrofit	1	LS		$ 2,347,900.00
	Excavation	794	CUYD	$ 50.00	$ 39,700.00
	Backfill	118	CUYD	$ 100.00	$ 11,800.00
	Shoring	361	SQYD	$ 200.00	$ 72,200.00
	Concrete	676	CUYD	$ 500.00	$ 338,000.00
	Reinforcement	135200	LB	$ 2.00	$ 270,400.00
	Post-tensioning	33800	LB	$ 6.00	$ 202,800.00
	10' Dia. Drilled Shafts	314	FT	$ 4,500.00	$ 1,413,000.00
	Widening				$ 530,871.18
	Column Concrete	112	CUYD	$ 700.00	$ 78,190.75
	Column Reinforcement	22340	LB	$ 2.00	$ 44,680.43
	Superstructure	2040	SF	$ 200.00	$ 408,000.00
Bent 26-28		3	EA	$ 3,064,781.18	$ 9,194,343.54
Bent 29		1	LS		$ 291,014.00
Ea	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Ec	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Ed	Column Enhancement	1	LS		$ 77,000.00
	Steel jacket	15400	LB	$ 5.00	$ 77,000.00
Ef	Footing Enlargement	1	LS		$ 144,500.00
	Excavation	178	CUYD	$ 50.00	$ 8,900.00

EAST APPROACH RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Backfill	100	CUYD	$ 100.00	$ 10,000.00
Shoring	160	SQYD	$ 200.00	$ 32,000.00
Concrete	78	CUYD	$ 500.00	$ 39,000.00
Reinforcement	15600	LB	$ 2.00	$ 31,200.00
Post-tensioning	3900	LB	$ 6.00	$ 23,400.00
Bent 30-34	5	EA	$ 291,014.00	$ 1,455,070.00
Bent 35	1	LS		$ 229,200.00
Eh Abutment enhancement	1	EA		$ 229,200.00
Excavation	82	CY	$ 50.00	$ 4,100.00
Backfill	17	CY	$ 100.00	$ 1,700.00
Concrete	126	CY	$ 500.00	$ 63,000.00
Reinforcement	25200	LB	$ 2.00	$ 50,400.00
Micropiles	22	EA	$ 5,000.00	$ 110,000.00

Total	**$ 27,599,408.72**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing b

Ar = 60%
Total Volume = 49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 1	1	LS		$422,222
Cellular Soil-Cement (CSC) G-I	1	EA		$422,222
Existing Footing Area	1100	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	1	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	1759	CY	n/a	
Treated Volume	1056	CY	$ 400.00	$422,222
Bent 2	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 3	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 4	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 5	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 6	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 7	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333

Ar = 60%
Total Volume = 49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 8	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 9	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 10	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 11	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 12	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 13	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 14	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 15	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778
Bent 16	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778
West Riverbank	1	LS		$2,133,333
Cellular Soil-Cement (CSC) G-I	1	EA		$2,133,333
CSC Area	4000	SF	n/a	
Treated Layer Thickness	60	FT	n/a	
Gross Volume	8889	CY	n/a	
Treated Volume	5333	CY	$ 400.00	$2,133,333
Mob/Demob	1	LS	10%	$1,193,689

Total	**$13,130,578**

Ar =	60%	
Total Volume =	49737	cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
East Riverbank (Primary)	1	LS		$20,444,444
Cellular Soil-Cement (CSC) G-I	1	EA		$20,444,444
CSC Area	23000	SF	n/a	
Treated Layer Thickness	100	FT	n/a	
Gross Volume	85185	CY	n/a	
Treated Volume	51111	CY	$ 400.00	$20,444,444
East Riverbank (Secondary)	1	LS		$11,555,556
Cellular Soil-Cement (CSC) G-I	1	EA		$11,555,556
CSC Area	10000	SF	n/a	
Treated Layer Thickness	120	FT	n/a	
Gross Volume	44444	CY	n/a	
Treated Volume	28889	CY	$ 400.00	$11,555,556
Mob/Demob	1	LS	10%	$3,200,000

Total	**$35,200,000**

Ar (Primary) =	60%	
Ar (Secondary) =	65%	
Total Volume =	129630	cy

Structural - Elimination of Structural Load Rating Deficiencies				
Location and Item	Quantity	Unit	Unit Cost	Total
West Approach	1	LS		$ 150,000.00
Span 17-19 girders	1	LS	$ 150,000.00	$ 150,000.00
Main Span	1	LS		$ 1,180,000.00
Strengthen Fixed Span Stringers	1	LS	$ 1,180,000.00	$ 1,180,000.00
East Approach	1	LS		$ 520,000.00
Span 25 girders	1	LS	$ 320,000.00	$ 320,000.00
Span 26-27 stringers	1	LS	$ 200,000.00	$ 200,000.00

<--- Needed strengthening included as part of seismic retrofit.

Total	**$ 1,850,000.00**

Structural - Pier Fender Repair/Replacement				
Location and Item	Quantity	Unit	Unit Cost	Total
West Approach		LS		$ -
Work Item				$ -
Sub item				$ -
Main Span	1	LS		$ 2,592,000.00
Pier Fender Replacement	1	LS		$ 2,592,000.00
	960	LF	$ 2,700.00	$ 2,592,000.00
				$ -
East Approach		LS		$ -
Work Item				$ -
Sub item				$ -

Total Replace	**$ 2,592,000.00**
Total Repair	**$ 1,296,000.00**

Assume:

1) Repair cost 50% of replacement
2) Replacement fenders are drilled shafts 10 foot diamenter
3) Four shafts at each bascule pier
4) Depth of shaft is 120 feet based on soil profile fromS&W

BURNSIDE BRIDGE FEASIBILITY-LEVEL COST ESTIMATE WORKSHEET (1)

Rev 7/30 — Alt 4c.2a (Seismic Retrofit + Partial Replacement + Widen) - Replace East Steel Spans

HDR

BRIDGE NUMBER	00511, 00511A, 00511B	COUNTY	Multnomah
BRIDGE NAME: Burnside Bridge	STATE HIGHWAY NUMBER: N/A	ROUTE NUMBER: N/A	
Mile Post: Sta 2+72.50 - 25+80.79	SCOPE: Seismic Retrofit & Structural Rehab & Widen All Spans Except Replace All East Steel Spans (Widened)	REFERENCE NAME/PHONE: Steve Drahota / (503) 423-3712	Date Entered: 8/18/2017; Date Checked: 8/20/2018

No.	ITEM		UNIT	QUANTITY	UNIT COST	SUB TOTAL	Section Totals	Notes
	Site Preparation						$ 43,309,700	
	Mobilization	10.0%	LS	1	$ 27,799,800	$ 27,799,800		
	Temp Erosion & Sediment Control	0.5%	LS	1	$ 1,390,000	$ 1,390,000		
	Temp. Protection and Direction of Traffic	4.0%	LS	1	$ 11,119,900	$ 11,119,900		
	Removal of structure and Obstruction		LS	1	$ 3,000,000	$ 3,000,000		
	Civil/Roadwork						$ 20,971,820	
	Roadway Surface / Earthwork		SY	10,000	$ 100	$ 1,000,000		
	Traffic Signals / Illumination	1.0%	LS	1	$ 2,570,264	$ 2,570,264		
	Site Restoration	2.0%	LS	1	$ 5,140,500	$ 5,140,500		
	Stormwater, Drainage, and Planting	2.00%	LS	1	$ 5,140,528	$ 5,140,528		
	Retaining Walls		SF	22,000	$ 90	$ 1,980,000		
	Utilities	2.00%	LS	1	$ 5,140,528	$ 5,140,528		
	Bridge Structure Retrofit						$ 166,415,648	
	West Approach (00511A)		LS	1	$ 11,451,420	$ 11,451,420		
	Main River Span (00511)		LS	1	###########	$ 130,548,945		
	East Approach (00511B)		LS	1	$ 24,415,284	$ 24,415,284		
	Movable Span Mechanical & Electrical						$ 14,265,000	
	Mechanical System		LS	1	$ 9,455,000	$ 9,455,000		
	Electrical System		LS	1	$ 4,210,000	$ 4,210,000		
	Emergency Backup System		LS	1	$ 600,000	$ 600,000		
	Geotechnical Hazard Mitigation						$ 48,330,578	
	East Approach		LS	1	$ 35,200,000	$ 35,200,000		
	West Approach		LS	1	$ 13,130,578	$ 13,130,578		
	Other Related Items						$ 28,015,166	
	20-Year Bridge Maintenance Needs:							(3)
	Accessibility - BUN-MU-04: Bicycle and Ped Impr. (Feasibility)		LS	0	$ 360,639	$ -		
	Accessibility - BUN-MU-05: Bicycle and Ped Impr. (Const - P1)		LS	0	$ 4,079,927	$ -		
	Accessibility - BUN-MU-06: Bicycle and Ped Impr. (Const - P2)		LS	0	$ 4,080,883	$ -		
	Elect. And Lighting - BUN-MU-01: Submarine Cable Removal		LS	1	$ 1,517,492	$ 1,517,492		(2)
	Structural - BUN-MU-02: Scour Remediation		LS	1	$ 5,575,674	$ 5,575,674		(2)
	Structural - BUN-MU-03: Fender Repair		LS	1	$ 2,592,000	$ 2,592,000		
	Structural - Elimination of Load Rating Deficiencies		LS	1	$ 1,330,000	$ 1,330,000		(4)
	Willamette River Mitigation		LS	1	$ 2,000,000	$ 2,000,000		(4)
	Contractor Access Premium		LS	1	$ 15,000,000	$ 15,000,000		(4)
	Construction Total with Contingency						$ 417,700,312	
	Subtotal (Site Preparation through Other Related Items)					$ 321,307,912		
	Contingencies		30%			$ 96,392,400		
	Right of Way						$ 45,000,000	
	Engineering & Project Delivery						$ 163,195,109	
	NEPA Phase					$ 17,000,000.00		
	PE (Incl. Design, PI, ROW Acquisition)	(PE)	15%			$ 62,655,046.73		
	County Admin. (Oversight, Permits, etc)							
	Construction Engineering (CEI)	(CEI)	20%			$ 83,540,062.30		
	Total Project Cost before Inflation (2017 $)						$ 625,895,421	
	Const + Design Cumulative Inflation (annual rate)	3.0%	years	10	62.9%	$ 365,322,000		(5)
	ROW Cumulative Inflation (annual rate)	5.0%	years	6	34.0%	$ 15,304,300		(5)
	Total Project Programmatic Cost (2027 $)						$ 1,006,521,721	

Notes:

(1) This cost sheet only includes roadway improvement costs associated with the scope of work.

(2) From 20-year WR Bridge CIP (Burnside's Allocation from the combined cost with other bridges)

(3) M&E repairs and work over RR lines not included in the 2015 Bridge Maintenance Project have been absorbed by other items

(4) Structural repairs to eliminate the load rating deficiencies established as part of the 2015 Bridge Maintenance Project

(5) Assumes construction mid-point in 2027

WEST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Bent 1		1	LS		$ 229,200.00
Wf	Abutment enhancement	1	EA		$ 229,200.00
	Excavation	82	CY	$ 50.00	$ 4,100.00
	Backfill	17	CY	$ 100.00	$ 1,700.00
	Concrete	126	CY	$ 500.00	$ 63,000.00
	Reinforcement	25200	LB	$ 2.00	$ 50,400.00
	Micropiles	22	EA	$ 5,000.00	$ 110,000.00
Bent 2		1	LS		$ 291,014.00
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 77,000.00
	Steel jacket	15400	LB	$ 5.00	$ 77,000.00
Wd	Footing Enlargement	1	LS		$ 144,500.00
	Excavation	178	CUYD	$ 50.00	$ 8,900.00
	Backfill	100	CUYD	$ 100.00	$ 10,000.00
	Shoring	160	SQYD	$ 200.00	$ 32,000.00
	Concrete	78	CUYD	$ 500.00	$ 39,000.00
	Reinforcement	15600	LB	$ 2.00	$ 31,200.00
	Post-tensioning	3900	LB	$ 6.00	$ 23,400.00
Bent 3-13		11	EA	$ 291,014.00	$ 3,201,154.00
Bent 14		1	LS		$ 651,714.00
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 115,450.00
	Steel jacket	23090	LB	$ 5.00	$ 115,450.00
Wd	Footing Enlargement	1	LS		$ 274,750.00
	Excavation	413	CUYD	$ 50.00	$ 20,650.00
	Backfill	277	CUYD	$ 100.00	$ 27,700.00
	Shoring	316	SQYD	$ 200.00	$ 63,200.00
	Concrete	136	CUYD	$ 500.00	$ 68,000.00
	Reinforcement	27200	LB	$ 2.00	$ 54,400.00
	Post-tensioning	6800	LB	$ 6.00	$ 40,800.00
	Widening				$ 192,000.00
	Superstructure	960	SF	$ 200.00	$ 192,000.00

WEST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Bent 15-16		2	EA	$ 651,714.00	$ 1,303,428.00
Bent 17		1	LS		$ 1,924,969.87
Wa	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Wb	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Wc	Column Enhancement	1	LS		$ 141,110.00
	Steel jacket	28222	LB	$ 5.00	$ 141,110.00
We	Foundation Retrofit	1	LS		$ 1,368,750.00
	Excavation	587	CUYD	$ 50.00	$ 29,350.00
	Backfill	74	CUYD	$ 100.00	$ 7,400.00
	Shoring	252	SQYD	$ 200.00	$ 50,400.00
	Concrete	513	CUYD	$ 500.00	$ 256,500.00
	Reinforcement	102600	LB	$ 2.00	$ 205,200.00
	Post-tensioning	25650	LB	$ 6.00	$ 153,900.00
	8' Dia. Drilled Shafts	180	FT	$ 3,700.00	$ 666,000.00
	Widening				$ 345,595.87
	Column Concrete	52	CUYD	$ 700.00	$ 36,651.91
	Column Reinforcement	10472	LBS	$ 2.00	$ 20,943.95
	Superstructure	1440	SF	$ 200.00	$ 288,000.00
Bent 18-19		2	EA	$ 1,924,969.87	$ 3,849,939.73

Total	**$ 11,451,419.60**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing b

MAIN RIVER SPAN RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Pier 1		1	LS		$ 13,281,596.32
Ma	Footing enlargement & drilled shafts	1	LS		$ 11,704,595.95
	Utility Relocation (30" & 42" Force Mains)	1	LS	$ 200,000.00	$ 200,000.00
	Shoring, Cribbing, & Cofferdams	1	LS	$ 3,000,000.00	$ 3,000,000.00
	Excavation	5544	CUYD	$ 50.00	$ 277,200.00
	Backfill	1358	CUYD	$ 100.00	$ 135,800.00
	Drilled Shaft Excavation, 72 Inch Diameter	996	FT	$ 1,500.00	$ 1,494,000.00
	Drilled Shaft Concrete	1043	CUYD	$ 1,000.00	$ 1,043,008.76
	Drilled Shaft Reinforcement	211209	LB	$ 2.50	$ 528,023.19
	Pier cap concrete	3943	CUYD	$ 500.00	$ 1,971,500.00
	Pier cap reinforcement	798458	LB	$ 2.00	$ 1,596,915.00
	Post-tensioning	159692	LB	$ 6.00	$ 958,149.00
	Harbor wall reconstruction	1	LB	$ 500,000.00	$ 500,000.00
Mb	Pier strengthening	1	LS		$ 1,421,181.48
	Base concrete	906	CUYD	$ 500.00	$ 453,055.56
	Base reinforcement	183488	LB	$ 2.00	$ 366,975.00
	Column concrete	332	CUYD	$ 500.00	$ 165,925.93
	Column reinforcement	67200	LB	$ 2.00	$ 134,400.00
	Post-tensioning	50138	LB	$ 6.00	$ 300,825.00
Mf	Truss support bearing retrofit	1	LS		$ 80,000.00
	Bearing Replacement & New Bearings	4	EA	$ 20,000.00	$ 80,000.00
Wd	Widening	1	LS		$ 75,818.89
	Column concrete	84	CUYD	$ 500.00	$ 41,888.89
	Column reinforcement	16965	LB	$ 2.00	$ 33,930.00
Pier 2		1	LS		$ 35,429,033.24
Ma	Footing enlargement & drilled shafts	1	LS		$ 28,070,266.57
	Shoring, Cribbing, & Cofferdams	1	LS	$ 4,200,000.00	$ 4,200,000.00
	Excavation	8012	CUYD	$ 50.00	$ 400,592.59
	Backfill	1541	CUYD	$ 100.00	$ 154,074.07
	Drilled Shaft Excavation, 120 Inch Diameter	980	FT	$ 1,750.00	$ 1,715,000.00
	Drilled Shaft Concrete	2851	CUYD	$ 2,250.00	$ 6,414,085.00
	Drilled Shaft Reinforcement	577268	LB	$ 2.50	$ 1,443,169.13
	Pier cap concrete	11972	CUYD	$ 500.00	$ 5,985,777.78
	Pier cap reinforcement	2424240	LB	$ 2.00	$ 4,848,480.00
	Post-tensioning	484848	LB	$ 6.00	$ 2,909,088.00
Mb	Pier wall strengthening	1	LS		$ 400,000.00
	Structural Steel	80000	LB	$ 5.00	$ 400,000.00
Mb	Load transfer columns at pier corners	1	LS		$ 250,000.00
	Structural Steel	50000	LB	$ 5.00	$ 250,000.00
Md	Reinforce Truss Supports at Piers 2 & 3	1	LS		$ 867,300.00
	Structural Steel	173460	LB	$ 5.00	$ 867,300.00
Mf	Truss span bearing retrofit	1	LS		$ 100,000.00
	Bearing Replacement & New Bearings	4	EA	$ 25,000.00	$ 100,000.00
Mg	Pit deck stringer bearings and strengthening	1	LS		$ 195,000.00
	Bearings	13	EA	$ 10,000.00	$ 130,000.00
	Strengthening	13	EA	$ 5,000.00	$ 65,000.00
Mh	Install counterweight restrainers	1	LS		$ 60,000.00
	Counterweight restrainers	4	EA	$ 15,000.00	$ 60,000.00
Mj	Trunnion support frame and anchorage strengthening	1	LS		$ 775,000.00
	Trunnion support frame structural steel	140000	LB	$ 5.00	$ 700,000.00
	Trunnion frame anchorage	1	LS	$ 75,000.00	$ 75,000.00

MAIN RIVER SPAN RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Wd	Widening	1	LS		$ 4,711,466.67
	Concrete - Below Pedestal	2933	CUYD	$ 500.00	$ 1,466,666.67
	Concrete - Above Pedestal	560	CUYD	$ 500.00	$ 280,000.00
	Concrete reinforcement	707400	LB	$ 2.00	$ 1,414,800.00
	Sidewalk, railing, operator house	1	LS	$ 250,000.00	$ 250,000.00
	New trunnion posts	260000	LB	$ 5.00	$ 1,300,000.00
Pier 3		1	LS		$ 35,565,779.72
Ma	Footing enlargement & drilled shafts	1	LS		$ 28,207,013.06
	Shoring, Cribbing, & Cofferdams	1	LS	$ 4,200,000.00	$ 4,200,000.00
	Excavation	8012	CUYD	$ 50.00	$ 400,592.59
	Backfill	1541	CUYD	$ 100.00	$ 154,074.07
	Drilled Shaft Excavation, 120 Inch Diameter	994	FT	$ 1,750.00	$ 1,739,500.00
	Drilled Shaft Concrete	2891	CUYD	$ 2,250.00	$ 6,505,714.79
	Drilled Shaft Reinforcement	585514	LB	$ 2.50	$ 1,463,785.83
	Pier cap concrete	11972	CUYD	$ 500.00	$ 5,985,777.78
	Pier cap reinforcement	2424240	LB	$ 2.00	$ 4,848,480.00
	Post-tensioning	484848	LB	$ 6.00	$ 2,909,088.00
Mb	Pier wall strengthening	1	LS		$ 400,000.00
	Structural Steel	80000	LB	$ 5.00	$ 400,000.00
Mb	Load transfer columns at pier corners	1	LS		$ 250,000.00
	Structural Steel	50000	LB	$ 5.00	$ 250,000.00
Md	Reinforce Truss Supports at Piers 2 & 3	1	LS		$ 867,300.00
	Structural Steel	173460	LB	$ 5.00	$ 867,300.00
Mf	Truss span bearing retrofit	1	LS		$ 100,000.00
	Bearing Replacement & New Bearings	4	EA	$ 25,000.00	$ 100,000.00
Mg	Pit deck stringer bearings and strengthening	1	LS		$ 195,000.00
	Bearings	13	EA	$ 10,000.00	$ 130,000.00
	Strengthening	13	EA	$ 5,000.00	$ 65,000.00
Mh	Install counterweight restrainers	1	LS		$ 60,000.00
	Counterweight restrainers	4	EA	$ 15,000.00	$ 60,000.00
Mj	Trunnion support frame and anchorage strengthening	1	LS		$ 775,000.00
	Trunnion support frame structural steel	140000	LB	$ 5.00	$ 700,000.00
	Trunnion frame anchorage	1	LS	$ 75,000.00	$ 75,000.00
Wd	Widening	1	LS		$ 4,711,466.67
	Concrete - Below Pedestal	2933	CUYD	$ 500.00	$ 1,466,666.67
	Concrete - Above Pedestal	560	CUYD	$ 500.00	$ 280,000.00
	Concrete reinforcement	707400	LB	$ 2.00	$ 1,414,800.00
	Sidewalk, railing, operator house	1	LS	$ 250,000.00	$ 250,000.00
	New trunnion posts	260000	LB	$ 5.00	$ 1,300,000.00
Pier 4		1	LS		$ 13,991,186.50
Ma	Footing enlargement & drilled shafts	1	LS		$ 12,345,711.98
	Shoring, Cribbing, & Cofferdams	1	LS	$ 3,000,000.00	$ 3,000,000.00
	Excavation	5430	CUYD	$ 50.00	$ 271,496.30
	Backfill	1150	CUYD	$ 100.00	$ 114,962.96
	Drilled Shaft Excavation, 72 Inch Diameter	1260	FT	$ 1,500.00	$ 1,890,000.00
	Drilled Shaft Concrete	1319	CUYD	$ 1,000.00	$ 1,319,468.91
	Drilled Shaft Reinforcement	267192	LB	$ 2.50	$ 667,981.14
	Pier cap concrete	4165	CUYD	$ 500.00	$ 2,082,666.67
	Pier cap reinforcement	843480	LB	$ 2.00	$ 1,686,960.00
	Post-tensioning	168696	LB	$ 6.00	$ 1,012,176.00
	Micro-piles	60	EA	$ 5,000.00	$ 300,000.00

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Mb Pier strengthening	1	LS		$ 1,478,326.37
Base concrete	906	CUYD	$ 500.00	$ 453,055.56
Base reinforcement	183488	LB	$ 2.00	$ 366,975.00
Column concrete	382	CUYD	$ 500.00	$ 190,814.81
Column reinforcement	77280	LB	$ 2.00	$ 154,560.00
Post-tensioning	52154	LB	$ 6.00	$ 312,921.00
Mf Truss support bearing retrofit	1	LS		$ 80,000.00
Bearing Replacement & New Bearings	4	EA	$ 20,000.00	$ 80,000.00
Wd Widening	1	LS		$ 87,148.15
Column concrete	96	CUYD	$ 500.00	$ 48,148.15
Column reinforcement	19500	LB	$ 2.00	$ 39,000.00
West Fixed Truss Span	1	LS		$ 9,172,070.93
Mc Cross frame strengthening	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Mc Adding vertical truss members	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Wd Widening	1	LS		$ 8,972,070.93
Structural Steel - new trusses & bracing	1760000	LB	$ 5.00	$ 8,800,000.00
Concrete - deck and sidewalk	263	CUYD	$ 250.00	$ 65,675.93
Concrete reinforcement	53198	LB	$ 2.00	$ 106,395.00
Bascule Leaves	2	LS		$ 13,937,207.16
Me Counterweight support frame strengthening	1	LS		$ 600,000.00
Structural Steel	120000	LB	$ 5.00	$ 600,000.00
Mf Live load shoe retrofit	1	LS		$ 100,000.00
Structural steel and bearings shoe seats	4	EA	$ 25,000.00	$ 100,000.00
Me Trunnion diaphragm strengthening	1	LS		$ 200,000.00
Structural Steel	40000	LB	$ 5.00	$ 200,000.00
Me Cross frame strengthening	1	LS		$ 300,000.00
Structural Steel	60000	LB	$ 5.00	$ 300,000.00
Mk Lightweight deck panel replacement	1	LS		$ 604,800.00
Deck replacement	12096	SQFT	$ 50.00	$ 604,800.00
Mi Center span lock replacement	1	LS		$ 939,300.00
Structural steel	1	LS	$ 100,000.00	$ 100,000.00
Mechanical lock components	1	LS	$ 839,300.00	$ 839,300.00
Wd Widening	1	LS		$ 11,193,107.16
Structural Steel - new trusses & bracing	2080000	LB	$ 5.00	$ 10,400,000.00
Lightweight decking	5112	sqft	$ 50.00	$ 255,600.00
concrete sidewalk and railing	84	CUYD	$ 300.00	$ 25,244.44
concrete reinforcement	17040	LB	$ 2.00	$ 34,080.00
Concrete - counterweight	598	CUYD	$ 800.00	$ 478,182.72
East Fixed Truss Span	1	LS		$ 9,172,070.93
Mc Cross frame strengthening	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Mc Adding vertical truss members	1	LS		$ 100,000.00
Structural Steel	20000	LB	$ 5.00	$ 100,000.00
Wd Widening	1	LS		$ 8,972,070.93

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Structural Steel - new trusses & bracing	1760000	LB	$ 5.00	$ 8,800,000.00
Concrete - deck and sidewalk	263	CUYD	$ 250.00	$ 65,675.93
Concrete reinforcement	53198	LB	$ 2.00	$ 106,395.00

Total	**$ 130,548,944.79**

Totals by Retrofit Designation

Expand caps with additional drilled shafts	Ma	$ 80,327,587.55
Retrofit piers 1 & 2	Mb	$ 4,199,507.85
Additional sway bracing in fixed trusses	Mc	$ 400,000.00
Reinforce truss supports at piers 2 & 3	Md	$ 1,734,600.00
Strengthen steel members and add lateral bracing	Me	$ 1,100,000.00
Strengthen/ replace live load support bearings	Mf	$ 460,000.00
strengthen/ replace deck stringer bearings	Mg	$ 390,000.00
Install counterweight restraints	Mh	$ 120,000.00
Strengthen/ replace center lock	Mi	$ 939,300.00
Retrofit bascule trunnion support	Mj	$ 1,550,000.00
Replace bascule deck with lightweight deck	Mk	$ 604,800.00
Work associated with widening	Wd	$ 38,723,149.38
	Total	**$ 130,548,944.79**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing bridge.

MAIN RIVER SPAN RETROFIT COST BUILDUP				
Location and Item	Quantity	Unit	Unit Cost	Total
Bascule machinery & electrical system	1	LS		$ 13,665,000.00
Ml Machinery	1	LS		$ 9,455,000.00
Operating Machinery Replacement	1	EA	$ 3,665,000.00	$ 3,665,000.00
Rehabilitation of Trunnions, Counterweight Trunnions, and links	4	EA	$ 552,000.00	$ 2,208,000.00
Additional Trunnions and Counterweight Trunnions for Widening	4	EA	$ 527,000.00	$ 2,108,000.00
Span Balance Work	1	EA	$ 50,000.00	$ 50,000.00
Operating Machinery Replacement for Widening (Additional Cost)	1	EA	$ 1,424,000.00	$ 1,424,000.00
Mm Electrical	1	LS		$ 4,210,000.00
Replace incoming electrical service from east and west	2	EA	$ 500,000.00	$ 1,000,000.00
Center span lock power feed	1	LS	$ 40,000.00	$ 40,000.00
Replace motors and drives	4	EA	$ 350,000.00	$ 1,400,000.00
Relocate and update PLCs (programming, start-up and commissioning)	1	LS	$ 350,000.00	$ 350,000.00
Replace navigation lighting (pier and span)	8	EA	$ 35,000.00	$ 280,000.00
Replace traffic warning gates	4	EA	$ 135,000.00	$ 540,000.00
Relocating electrical equipment (MCCs, panelboards, networking equipment)	1	LS	$ 600,000.00	$ 600,000.00

Total	**$ 13,665,000.00**

Totals by Retrofit Designation

Expand caps with additional drilled shafts	Ma	$ -
Retrofit piers 1 & 2	Mb	$ -
Additional sway bracing in fixed trusses	Mc	$ -
Reinforce truss supports at piers 2 & 3	Md	$ -
Strengthen steel members and add lateral bracing	Me	$ -
Strengthen/ replace live load support bearings	Mf	$ -
strengthen/ replace deck stringer bearings	Mg	$ -
Install counterweight restraints	Mh	$ -
Machinery	Ml	$ 9,455,000.00
Electrical	Mm	$ 4,210,000.00
Replace bascule deck with lightweight deck	Mk	$ -
Work associated with widening	Wd	$ -
	Total	**$ 13,665,000.00**

EAST APPROACH RETROFIT COST BUILDUP					
Location and Item		Quantity	Unit	Unit Cost	Total
Spans 20-27		1	LS		$ 22,440,000.00
Ek	Replace with multiple spans	1	EA		$ 22,440,000.00
	New Structure	74800	SF	$ 300.00	$ 22,440,000.00
Bent 29		1	LS		$ 291,014.00
Ea	Superstructure strengthening	1	LS		$ 13,840.00
	Post-tensioning	1730	LB	$ 8.00	$ 13,840.00
Ec	Floor beam strengthening	1	LS		$ 55,674.00
	Concrete	22	CUYD	$ 500.00	$ 11,000.00
	Reinforcement	4400	LB	$ 2.00	$ 8,800.00
	Post-tensioning	5979	LB	$ 6.00	$ 35,874.00
Ed	Column Enhancement	1	LS		$ 77,000.00
	Steel jacket	15400	LB	$ 5.00	$ 77,000.00
Ef	Footing Enlargement	1	LS		$ 144,500.00
	Excavation	178	CUYD	$ 50.00	$ 8,900.00
	Backfill	100	CUYD	$ 100.00	$ 10,000.00
	Shoring	160	SQYD	$ 200.00	$ 32,000.00
	Concrete	78	CUYD	$ 500.00	$ 39,000.00
	Reinforcement	15600	LB	$ 2.00	$ 31,200.00
	Post-tensioning	3900	LB	$ 6.00	$ 23,400.00
Bent 30-34		5	EA	$ 291,014.00	$ 1,455,070.00
Bent 35		1	LS		$ 229,200.00
Eh	Abutment enhancement	1	EA		$ 229,200.00
	Excavation	82	CY	$ 50.00	$ 4,100.00
	Backfill	17	CY	$ 100.00	$ 1,700.00
	Concrete	126	CY	$ 500.00	$ 63,000.00
	Reinforcement	25200	LB	$ 2.00	$ 50,400.00
	Micropiles	22	EA	$ 5,000.00	$ 110,000.00

Total	**$ 24,415,284.00**

Note: Shaft, concrete and reinforcement unit cost is higher due to construction difficulty under existing bridge.

Ar = 60%
Total Volume = 49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 1	1	LS		$422,222
Cellular Soil-Cement (CSC) G-I	1	EA		$422,222
Existing Footing Area	1100	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	1	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	1759	CY	n/a	
Treated Volume	1056	CY	$ 400.00	$422,222
Bent 2	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 3	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 4	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 5	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 6	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 7	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 8	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 9	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 10	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 11	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 12	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 13	1	LS		$629,333
Cellular Soil-Cement (CSC) G-I	1	EA		$629,333
Existing Footing Area	42	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2622	CY	n/a	
Treated Volume	1573	CY	$ 400.00	$629,333
Bent 14	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778

Ar =	60%
Total Volume =	49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
Bent 15	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778
Bent 16	1	LS		$609,778
Cellular Soil-Cement (CSC) G-I	1	EA		$609,778
Existing Footing Area	64	SF	n/a	
CSC Area	3000	SF	n/a	
Number of Footings	4	EA	n/a	
Treated Layer Thickness	25	FT	n/a	
Gross Volume	2541	CY	n/a	
Treated Volume	1524	CY	$ 400.00	$609,778
West Riverbank	1	LS		$2,133,333
Cellular Soil-Cement (CSC) G-I	1	EA		$2,133,333
CSC Area	4000	SF	n/a	
Treated Layer Thickness	60	FT	n/a	
Gross Volume	8889	CY	n/a	
Treated Volume	5333	CY	$ 400.00	$2,133,333
Mob/Demob	1	LS	10%	$1,193,689

Total	**$13,130,578**

Ar = 60%
Total Volume = 49737 cy

Geotechnical Hazard Mitigation				
Location and Item	Quantity	Unit	Unit Cost	Total
East Riverbank (Primary)	1	LS		$20,444,444
Cellular Soil-Cement (CSC) G-I	1	EA		$20,444,444
CSC Area	23000	SF	n/a	
Treated Layer Thickness	100	FT	n/a	
Gross Volume	85185	CY	n/a	
Treated Volume	51111	CY	$ 400.00	$20,444,444
East Riverbank (Secondary)	1	LS		$11,555,556
Cellular Soil-Cement (CSC) G-I	1	EA		$11,555,556
CSC Area	10000	SF	n/a	
Treated Layer Thickness	120	FT	n/a	
Gross Volume	44444	CY	n/a	
Treated Volume	28889	CY	$ 400.00	$11,555,556
Mob/Demob	1	LS	10%	$3,200,000

Total	**$35,200,000**

Ar (Primary) =	60%	
Ar (Secondary) =	65%	
Total Volume =	129630	cy

Structural - Elimination of Structural Load Rating Deficiencies				
Location and Item	Quantity	Unit	Unit Cost	Total
West Approach	1	LS		$ 150,000.00
Span 17-19 girders	1	LS	$ 150,000.00	$ 150,000.00
Main Span	1	LS		$ 1,180,000.00
Strengthen Fixed Span Stringers	1	LS	$ 1,180,000.00	$ 1,180,000.00
East Approach	1	LS		

Total	**$ 1,330,000.00**

Structural - Pier Fender Repair/Replacement				
Location and Item	Quantity	Unit	Unit Cost	Total
West Approach		LS		$ -
Work Item				$ -
Sub item				$ -
Main Span	1	LS		$ 2,592,000.00
Pier Fender Replacement	1	LS		$ 2,592,000.00
	960	LF	$ 2,700.00	$ 2,592,000.00
				$ -
East Approach		LS		$ -
Work Item				$ -
Sub item				$ -

Total Replace	**$ 2,592,000.00**
Total Repair	**$ 1,296,000.00**

Assume:
1) Repair cost 50% of replacement
2) Replacement fenders are drilled shafts 10 foot diamenter
3) Four shafts at each bascule pier
4) Depth of shaft is 120 feet based on soil profile fromS&W